NZ TRANSPORT AGENCY
WAKA KOTAHI

Peka Peka to North Ōtaki Expressway
Hydraulic Investigations for Expressway Crossing of Waitohu Stream and Floodplain

Quality Assurance Statement

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NZ Transport Agency

Contents

List of Abbreviations

AEE	Assessment of Environmental Effects
AEP	Annual Exceedance Probability
DS	Downstream
GEV	Generalised Extreme Value (a statistical distribution used to fit flood maxima series data)
GWRC	Greater Wellington Regional Council
KCDC	Kāpiti Coast District Council
LiDAR	Light Detection and Ranging[1]
MfE	Ministry for the Environment
MSL	Mean Sea Level
NIMT	North Island Main Trunk (railway)
NZTA	New Zealand Transport Agency
NZVD	New Zealand Vertical Datum
PE3	Log Pearson 3 (a statistical distribution used to fit flood maxima series data)
PP2O	Peka Peka to North Ōtaki
RMA	Resource Management Act
RoNS	Roads of National Significance
SAR	Scheme Assessment Report
SH1	State Highway 1
TRB	True Right Bank (as viewed looking downstream in direction of river or stream flow)
US	Upstream

[1] This is an airborne laser remote sensing technology used for the acquisition of detailed and accurate topographic survey data.

1. Introduction

1.1 Rationale for Investigations

The flood hazard map incorporated in the Kāpiti Coast District Plan (refer to Appendix A) indicates that the routes of both the State Highway 1 (SH1) and the North Island Main Trunk (NIMT) railway are exposed to existing flood hazards across the Waitohu Stream alluvial fan and floodplain. These flood hazards are caused by floodwaters breaking out of the main channel of the Waitohu Stream along both banks upstream of the SH1 bridge and by shallow overland flow from tributary sub-catchments, principally the Greenwood sub-catchment. The exposure of SH1 to these flood hazards is borne out by incidences of flood inundation across the road in recent storm events.

The route of the proposed Peka Peka to North Ōtaki (PP2O) Expressway lies between SH1 and the NIMT railway line and will therefore be exposed to the same flood hazards (see the topographic relief map in Figure 1-1). In order to achieve a minimum level of service with respect to these flood hazards, the expressway will need to be elevated as a raised embankment with adequate conveyance capacity through the embankment to provide for continuity of existing overland flow paths across the floodplain. However, in a departure from the initial scheme design, the northern approach embankment to the bridge crossing of the Waitohu Stream will now slope down to grade from the bridge to before the Greenwood sub-catchment culvert.

For the initial scheme design, design levels were set for the proposed vertical alignment of the expressway and the sizes of culverts through the expressway embankment were defined with the aid of an adapted version of an existing computational hydraulic model of the extensive Waitohu Stream alluvial fan and floodplain system. It was recognised at the time that the flood flow patterns across the fan and floodplain system would strictly be two-dimensional in nature (i.e. horizontal flow velocities across the nearly flat surface would have two primary components of direction rather than only a single dominant direction) and that a two-dimensional hydraulic modelling approach would provide more certainty to the predictions of flood inundation.

This report presents the results of more detailed computational hydraulic modelling investigations for the Waitohu Stream alluvial fan and floodplain system using a two-dimensional modelling approach. The results of these investigations are also summarised in the overview assessment of hydraulic effects (Webby and Smith, 2013) covering all the major watercourses crossed by the proposed PP2O Expressway.

1.2 Feedback on Initial Investigations from GWRC and KCDC

Greater Wellington Regional Council (GWRC) provided feedback on our initial investigations for the scheme development. They noted apparent differences in flood extent across the alluvial fan and floodplain system compared to the extent mapped previously by themselves and which are incorporated in the Kāpiti Coast District Plan flood hazard map (see a copy of this map in Appendix A). In discussions with GWRC, we attributed these apparent differences in flood extent to differences caused by the extrapolation technique required to extend flood level predictions along a defined longitudinal flow path over a nearly flat surface. We expressed the view that any uncertainties resulting from these differences could best be resolved by adopting a two-dimensional computational hydraulic modelling approach for the existing and proposed situations.

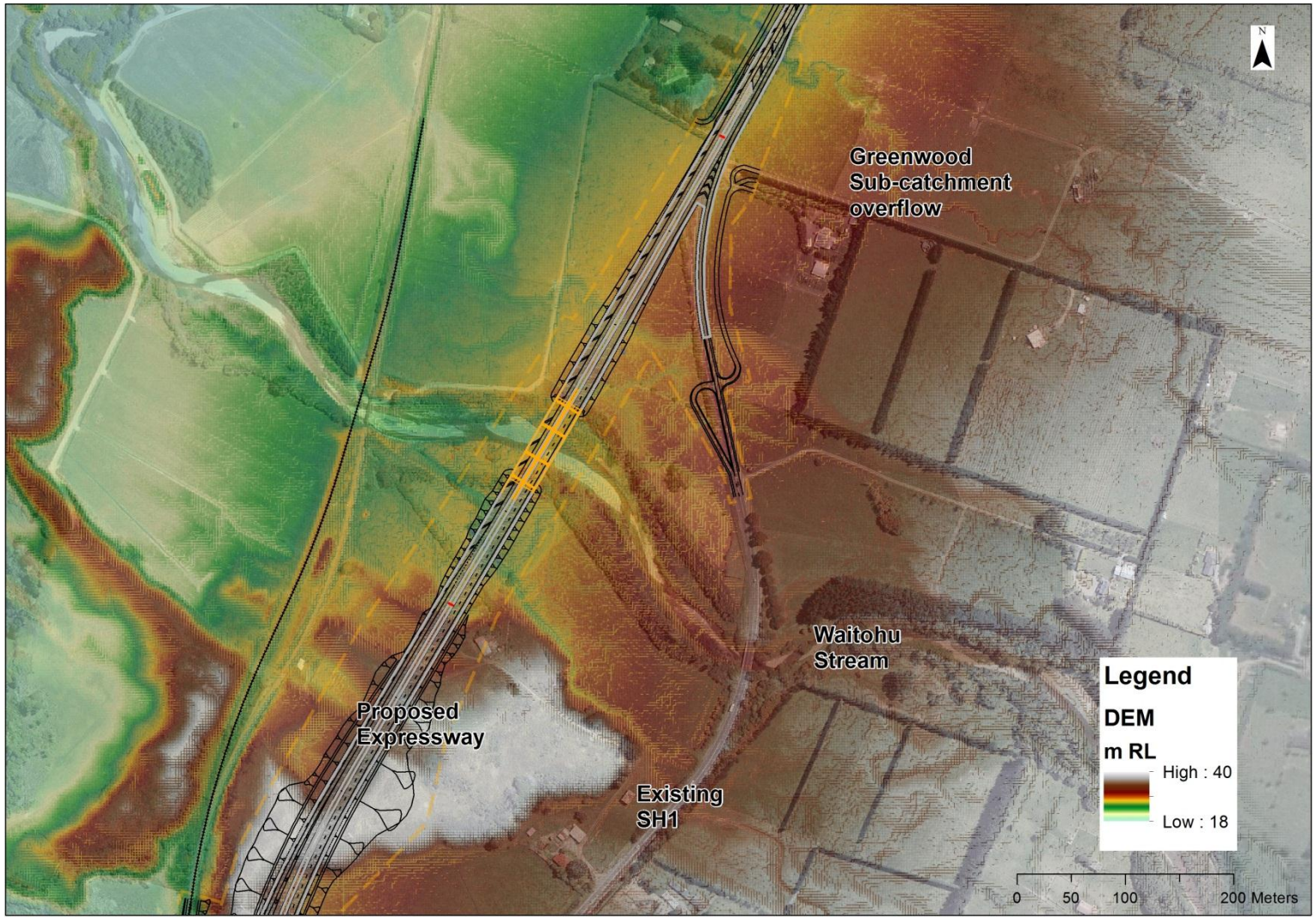

Figure 1-1 Topographic relief map of Waitohu Stream and floodplain area with route of proposed expressway superimposed

From an effects assessment perspective, GWRC made a number of other comments on the preliminary investigations:

- the flood inundation map should show ranges of peak flood depths in addition to the areal extent of inundation;
- a map showing changes in peak flood depth between the existing and proposed situations should also be included to highlight the effects of the proposed expressway on flood levels;
- a map showing predicted flow velocities across the alluvial fan and floodplain system should be included for both the existing and proposed situations; and
- a map showing changes in peak flow velocities between the existing and proposed situations should also be included to highlight the effects of the proposed expressway on flow directions and velocities.

GWRC noted the provision of a large dry culvert to convey runoff from the Greenwood sub-catchment in the initial scheme design. They suggested therefore that consideration should be given to the potential effects of partial blockage of this normally dry culvert. This issue would be less of a concern with an elevated road embankment and a very wide box culvert type structure as envisaged in the initial scheme design but, with the subsequent changes to the scheme design at the northern end (with the expressway sloping down to grade immediately to the north of the Waitohu Stream crossing, the effects of a partial culvert blockage become much more significant. This is because the vertical alignment of the expressway relative to local ground levels constrains the size of culvert that can be used to convey surface runoff from the Greenwood sub-catchment under the road (the existing culverts on SH1 are only twin 1.05m diameter circular culverts).

GWRC were also interested in other effects of the expressway and the Greenwood sub-catchment dry culvert including:

- potential sediment deposition in low velocity ponding areas;
- potential erosion of the floodplain surface due to increased flow velocities; and
- any reduction in the time to drain the floodplain relative to the existing situation.

KCDC deferred to GWRC on the initial investigations undertaken for the expressway crossing of the Ōtaki River and Floodplain as they considered these watercourses to be outside their specific jurisdiction.

1.3 Scope and Methodology for Detailed Investigations

As noted in Section 1.1, the preliminary investigations for the initial scheme design used an adaptation of an existing one-dimensional MIKE11[2] computational hydraulic model previously developed by GWRC (and made available by them) to:

- examine the effects of the proposed PP2O Expressway on existing flood patterns and levels across the alluvial fan and floodplain system for the Waitohu Stream and its tributaries;

[2] MIKE11 is an internationally recognised computational hydraulic modelling software package developed by the Danish Hydraulic Institute (DHI) and designed for simulating flow behaviour in complex river and floodplain systems. It is widely used in New Zealand

- determine appropriate culvert sizes in order to provide continuity for existing secondary flow paths and mitigate any adverse effects of the proposed expressway; and
- set design levels for the new road.

The analyses using the adapted MIKE11 model of the Waitohu Stream and alluvial fan and floodplain system considered the 1% annual exceedance probability (AEP) flood adjusted for the possible effects of future climate change to 2090 as a base case but also evaluated the sensitivity of the initial scheme design to the 0.5% AEP flood adjusted for the possible effects of future climate change.

The purpose of the more detailed investigations then was to replicate the preliminary investigations using an alternative two-dimensional computational hydraulic modelling approach in order to provide more confidence in the assessment of effects of the proposed expressway crossing of the Waitohu Stream alluvial fan and floodplain system. The primary tool for these more detailed investigations was a further adaptation of the existing MIKE11 model of the Waitohu Stream and alluvial fan and floodplain system in which the one-dimensional overland flow paths within the primary area of interest were replaced with a two-dimensional representation of the nearly flat ground surface.

Examination of the original MIKE11 model branch structure and the flood inundation patterns shown on the Kāpiti Coast District Plan flood hazard map (see Appendix A) indicated that a suitable approach would be to incorporate a two-dimensional floodplain surface into the model from just upstream of the Waitohu Valley Road Bridge to about 2.7km downstream of the NIMT railway bridge (Figure 1-2). In this manner the MIKE11 model was transformed into a MIKEFLOOD[3] model. The latter model incorporated two linked components that were run simultaneously in parallel:

- a one-dimensional MIKE11[3] computational hydraulic model component representing the Waitohu Stream and other tributaries and used for simulating flow behaviour within the main stream and tributary channels; and
- a two-dimensional MIKE21[3] computational hydraulic model component representing the Waitohu Stream alluvial fan and floodplain surface between the Waitohu Valley Road Bridge and the Ngatotara Stream[4] and used for simulating flow behaviour across this two-dimensional nearly flat surface.

Appendix B summarises the changes made to the original MIKE11 model provided by GWRC to transform it into the new MIKEFLOOD model over the area described. Three versions of the new MIKEFLOOD model were developed: version one represented the existing situation with the old NIMT railway bridge in place, version two also represented the existing situation but with the new NIMT railway bridge in place and version three represented the proposed situation with the Expressway and the new NIMT railway bridge in place.

[3] MIKEFLOOD is an overarching software shell which incorporates both the MIKE11 one-dimensional and MIKE21 two-dimensional computational hydraulic modelling packages. It enables the hydraulic interaction of linear watercourses linked to two-dimensional water bodies or surfaces to be more accurately simulated where overland flow paths are uncertain. MIKE11 and MIKE21 are internationally recognised computational hydraulic modelling software packages developed by the Danish Hydraulic Institute (DHI) and designed for simulating flow behaviour in complex river and floodplain systems. They are both widely used in New Zealand.

[4] The areal extent of the two-dimensional floodplain surface within the MIKEFLOOD model was deliberately restricted in size for practical reasons to minimise the model simulation run times.

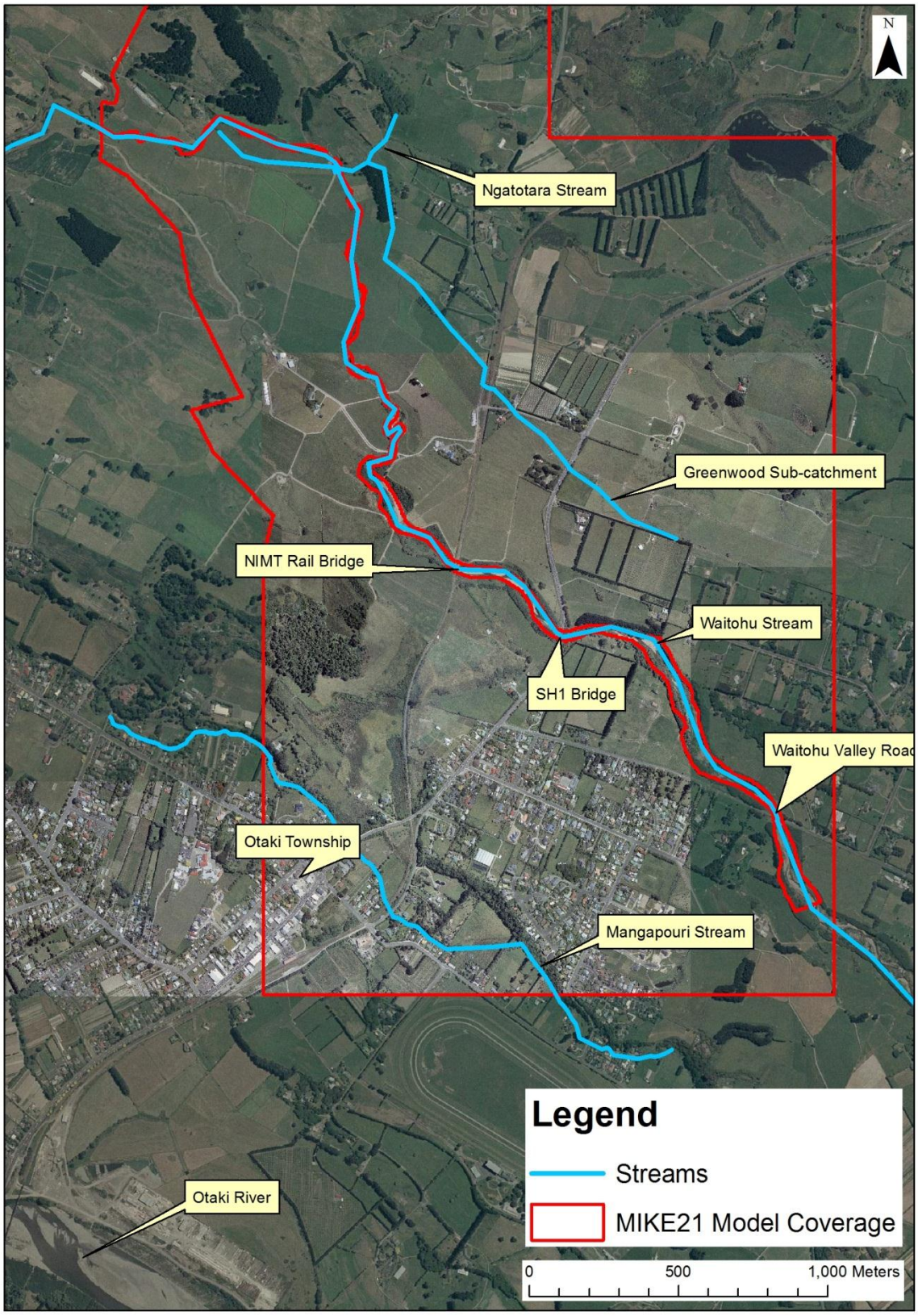

Figure 1-2 Aerial photograph of alluvial fan and floodplain system for Waitohu Stream within area of interest showing key features and locations

The new MIKEFLOOD model was verified by simulating the February 2004 calibration flood event. Appendix C summarises the comparison between the peak flood level predictions for the calibration of the new model and the original MIKE11 model.

The further detailed investigations undertaken using the two-dimensional computational hydraulic modelling approach were focussed on confirming the effects of the proposed expressway on existing flood inundation patterns, levels and flow velocities and on confirming the appropriateness or otherwise of the culvert sizes determined from the preliminary investigations. In addition consideration was given to other potential effects identified by GWRC:

- potential sediment deposition in low velocity ponding areas;
- potential erosion of the floodplain surface due to excessively high flow velocities; and
- any change in the time to drain the floodplain relative to the existing situation.

Appropriate outputs from the detailed hydraulic modelling were produced to enable these points to be specifically addressed in a quantitative manner.

1.4 Proposed Expressway Crossing of Waitohu Stream and Floodplain

The route of the Expressway lies between SH1 to the east and the NIMT railway line to the west across the Waitohu Stream alluvial fan and floodplain system. It therefore passes directly through the extensive overland flow paths that currently along both sides of the Waitohu Stream past the existing SH1 bridge under significant flood conditions. In order to remain flood free up to a specified flood standard, the Expressway will therefore need to be elevated as an embankment. However, on the northern side of the stream crossing the vertical alignment of the proposed expressway will now slope down to grade and transition into the existing horizontal alignment of SH1 before the twin Greenwood sub-catchment culverts. This culvert location is a known flooding hotspot and achieving the desired level of service while maintaining a near at-grade vertical alignment for the new road at the culvert crossing will be problematic. This is because the soffit level of the existing culvert system is not far below existing road level.

To maintain existing overland flow paths across the alluvial fan and floodplain system and to allow overland flows to pass under the Expressway, culverts to convey these flood flows through the approach embankments to the bridge crossing will be provided. A replacement culvert on the Greenwood sub-catchment overland flow path will also be provided.

Williams (2004) has noted that the bed slope of the existing stream channel starts to significantly reduce in slope downstream of the existing SH1 bridge. As a response to this reduction in stream bed slope, he has identified a geomorphologic zone of instability characterised by sediment deposition and potential lateral channel instability between the SH1 bridge and the Wakapua Farm bridge. The proposed Expressway bridge will cross the Waitohu Stream approximately 260m downstream of the existing SH1 bridge within this zone of geomorphologic instability.

The Expressway bridge crossing has therefore been designed to have an approximately 75m total span length so as not to encroach on the 75m wide fairway width defined by GWRC to allow for potential future channel

migration. The effect of the resulting large set back of the bridge abutments from the current active channel location minimises the risk of future abutment attack by high velocity flood flows.

Note that modification of the existing SH1 bridge to increase its waterway capacity and reduce the volume of flood breakout flows upstream of the bridge does not form part of the proposed Scheme.

1.5 Principle of Hydraulic Neutrality

An elevated transport link constructed across a floodplain interferes with the natural drainage function of such a feature. Adequate provision must therefore be made for relief measures within the elevated link to allow the safe passage of floodplain through it or over it.

A fundamental principle which has been applied consistently with respect to the treatment of individual floodplain crossings on the PP2O Expressway Project is that of hydraulic neutrality. What this means is that the impact of flood hazards from the proposed expressway should be no worse than in the current situation. This objective can sometimes be extremely difficult to achieve while still maintaining the required level of service for the expressway. Where it has not been possible to achieve this desired objective, a fall-back position has been adopted whereby flood hazards that have been made worse are kept away from residential properties and instead redirected towards uninhabited rural areas[5].

Application of the principle of hydraulic neutrality in this particular context is demonstrated by the proposed inclusion of dry culverts through the approach embankments to the Expressway bridge over the Waitohu Stream.

1.6 Flood Magnitudes and Climate Change Effects

In this report, flood magnitudes are identified by reference to their annual exceedance probability (AEP). This is a statistical measure of how large a flood is and is generally evaluated from a flood frequency analysis of the annual flood maxima series for a continuous measured flow record from a hydrological gauging station (there is a gauging station on the Waitohu Stream further upstream). For example a 1% (1 in 100) AEP flood is one that would be exceeded on average once every 100 years over a very long period of time (very much longer than 100 years).

The floods of interest with respect to the Waitohu Stream and floodplain system were the 1% and 0.5% AEP floods and also the 5% AEP flood for the Greenwood sub-catchment. The estimates for these floods were adjusted for the effects of possible future climate change to 2090 based on a mid-range estimate for increased average temperature and hence rainfall for the Wellington and Manawatu regions from the MfE (2010) Guidelines. The time frame for consideration of climate change effects reflects the projected design life of the required bridge and culvert structures for the Waitohu Stream and floodplain crossing.

[5] In discussions with GWRC and KCDC, their preferred position is that complete hydraulic neutrality should be achieved if possible.

1.7 Level Datum

Since flood levels in a river or stream near the outlet to the sea are affected by sea levels, Greater Wellington Regional Council (GWRC) consistently uses the Mean Sea Level Wellington (1953) level datum for their flood hazard investigations and flood protection works design. The investigations described in this report have made use of stream cross-section and culvert level data sourced from GWRC which are expressed in terms of this mean sea level datum. To ensure consistency with GWRC publications and information then, these investigations have used the same level datum to evaluate flood levels along the Mangapouri Stream for both the existing situation and for the proposed expressway situation.

Existing ground levels from LiDAR data and construction levels for the Expressway on the other hand are expressed in terms of the NZ Vertical Datum (2009). It has therefore been necessary to translate between the two level data when specifying design flood levels and road design levels at key stream / river crossing locations.

Throughout this report then, flood levels are expressed in terms of Mean Sea Level (MSL) Wellington (1953) datum. To adjust these levels to be in terms of NZ Vertical Datum (2009), 0.44m needs to be subtracted. Conversely, to adjust levels in NZ Vertical Datum (2009) to be in terms of MSL Wellington Datum, 0.44m needs to be added.

2. Flood Hydrology

2.1 Description of Waitohu Stream Catchment

The Waitohu Stream drains a catchment on the steeply sloping western flanks of the Tararua Ranges. The head of the catchment borders the adjacent Ōtaki River Catchment.

The Waitohu Stream Catchment has two distinct components: a steep upland component and a flatter alluvial fan and floodplain component. The upland component ends at about the Ringawhati Road bridge (about 2.6km upstream of the SH1 bridge) where the stream breaks out of a narrow valley onto an alluvial fan. From the Ringawhati Road bridge, the Waitohu Stream follows an approximately straight course between lower river terraces past the SH1 and NIMT railway bridges (Figure 2-1). Below the NIMT railway bridge, the stream runs through low swampy ground behind a coastal dune barrier, being joined by a number of other tributaries including the Ngatotara Stream to the north and the Mangapouri Stream to the south. The Waitohu Stream breaks out through the coastal dune barrier to the sea at the north end of Ōtaki Beach.

The Waitohu Stream has a catchment area of 19.1km^2 at the site of the flow gauging station at the water supply intake (about 4km upstream of the SH1 bridge). This increases to 23.4km^2 at the SH1 bridge and about 53km^2 at the mouth of the stream. The very significant increase in catchment area between the SH1 bridge and the mouth is due to the large number of minor tributaries draining the low lying swampy area below the NIMT railway bridge.

2.2 Flood Estimates from Frequency Analysis of Annual Flood Maxima Series

The Waitohu Stream is gauged at the water supply intake about 4km upstream of the existing SH1 bridge. The gauging station has been open since October 1994 so that a 17 year long flow record is theoretically available for carrying out a flood frequency analysis of the annual flood maxima series. However there are number of problems with the stream flow record (Harkness, 2003):

- The gauging station site is slightly unstable so that the stage / discharge rating curves used to define the stream flow from measured stage values have shifted over time. This creates some degree of uncertainty in the stream flow record.
- There are a number of high stage (i.e. flood flow) gaugings available between 33 and 88m^3/s. In 2003, the highest flood gauging was a back-calculated slope / area estimate of 87.5m^3/s for the 20 February 1996 flood. Since then other flow gaugings between 33 and 75m^3/s have provided more uncertainty in the form and trend of the stage / discharge rating curves over time, and therefore the accuracy of the stream flow record.
- There are a number of gaps in the stream flow record such that the 2002 flood maximum is likely to have been missed.

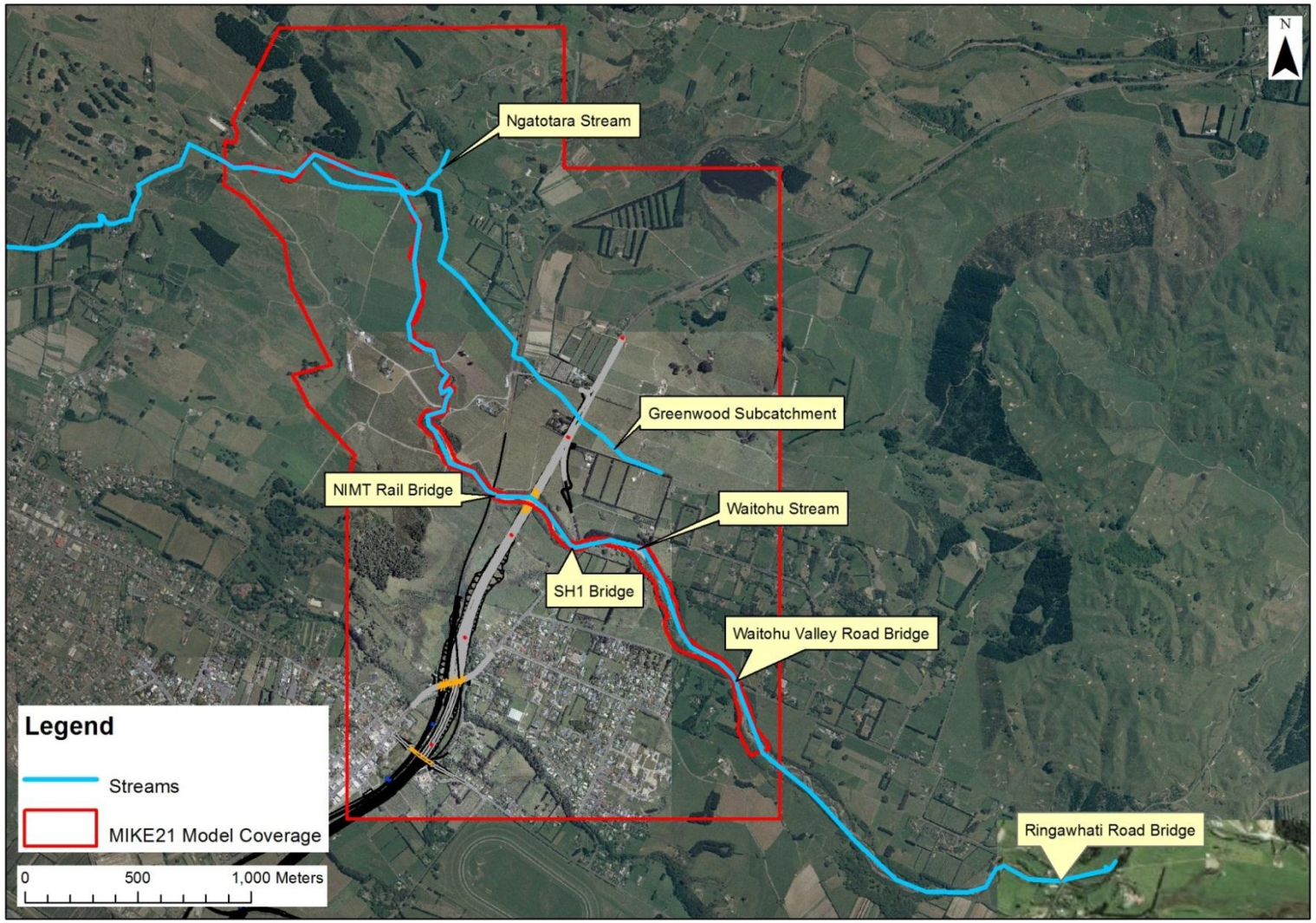

Figure 2-1 Lower part of Waitohu Stream Catchment with key locations marked

Despite the variability in the stage / discharge rating curves caused by channel instability, they would appear to provide a fairly reliable estimate of the actual stream flow record. A flood frequency analysis was therefore carried out on the annual flood maxima series derived from the flow record. Figure 2-2 shows the results of the flood frequency analysis of the 17 year record from 1994-2010. The three frequency distributions fitted to the annual flood maxima series (Gumbel, GEV and Log Pearson 3) all show reasonably good agreement with only a relatively small amount of variation between them when extrapolated to obtain estimates of low frequency floods.

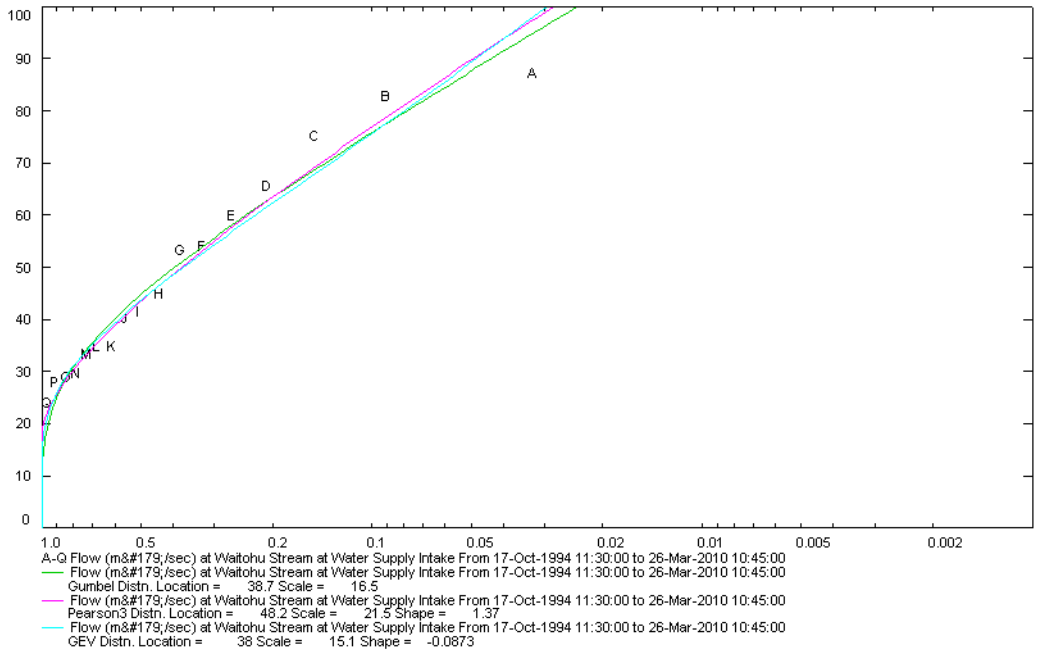

Figure 2-2 Frequency analysis of Waitohu at Water Supply Intake gauging station flow record (1994-2010) (horizontal axis – annual exceedance probability, vertical axis – discharge in m^3/s)

Table 2-1 summarises the flood frequency estimates at the gauging station site for annual exceedance probabilities (AEP) in the range of 43% (1 in 2.33 or the mean annual flood) to 0.1% (1 in 1000). These flood frequency estimates have also been scaled using the catchment scaling approach of McKerchar and Pearson (1989) to obtain corresponding estimates at the site of the existing SH1 bridge, 4km downstream. The 1% AEP flood is estimated to have a magnitude in the range of 139-146m^3/s at the bridge for current climate conditions. Based on the most recent MfE (2010) Guidelines, this estimated range for the 1% AEP flood would increase to 163-171m^3/s to allow for the effects of possible future climate change to 2090 (based on a mid-range increase in average temperature and hence rainfall).

Table 2-1 Flood estimates for Waitohu Stream derived from frequency analysis of Waitohu at Water Supply Intake gauging station flow record ((1994-2010) (based on current climate conditions)

AEP (%)	Flood Estimate (m^3/s)					
	Water Supply Intake (Gumbel)	Water Supply Intake (PE3)	Water Supply Intake (GEV)	Scaled to SH1 bridge (Gumbel)	Scaled to SH1 bridge (PE3)	Scaled to SH1 bridge (GEV)
42.9	48	47	47	57	56	55
20	63	63	62	75	75	73
10	76	77	76	89	91	89
5	88	90	89	103	106	105
2	103	106	108	122	125	128
1	114	118	124	135	139	146
0.5	126	130	140	149	153	165
0.2	141	145	163	167	171	192
0.1	152	157	181	180	185	214

2.3 Frequency Estimates from Rainfall / Runoff Model Predictions

Flood levels and extents in the Waitohu Stream are affected not only by the magnitude of flood flows in the main stem of the stream but also by the flood flow contributions of tributaries (such as the Mangapouri Stream) and surface runoff from other parts of the catchment. In order to assess the extent of the flood hazard in the lower (more developed and populated) part of the catchment, Harkness (2003) constructed a rainfall / runoff model of the catchment and used this to estimate flood magnitudes for the catchment based on rainfall records from the local area[6]. While this helped at the time to get around the problem of only a relatively short stream flow record (10 years duration) for the gauging station with which to undertake an at-site flood frequency analysis (and which would have produced flood frequency estimates of even lower reliability than those in Table 2-1), the rainfall / runoff modelling approach was still constrained by the availability of only short term rainfall records with which to estimate the model inputs. However the rainfall runoff modelling approach did enable consideration of the runoff produced by storms of varying durations from 1 hour to 24 hours, something the flood frequency analysis of annual flood maxima did not allow.

[6] The rainfall / runoff model for the Waitohu Catchment (Harkness, 2003) was calibrated against six separate storm events with peak flows in the range of 39-86m3/s (i.e. up to at least a 10% AEP magnitude based on the flood frequency estimates derived from the actual stream flow record given in Table 2-1). All of the calibration storms were multi-peaked ones reflecting the occurrence of separate heavy burst of rainfall within the over rainfall pattern for each storm event. The recorded runoff patterns were generally reproduced by the model fairly well with peak values predicted within ± 10% of the measured values.

The rainfall / runoff model had several limitations. Firstly, the catchment area over which it was applied is fairly large. Secondly, there is a non-uniform rainfall gradient over the catchment, particularly for the more extreme storm events (SKM, 2008). (this latter limitation was countered to some extent by using separate rainfall inputs for the steeper upland country component of the catchment and the flatter plains component). Thirdly, the rainfall records from which the rainfall frequency inputs to the model were derived were of relatively short duration in 2003 so that there was some uncertainty in the accuracy of the rainfall frequency inputs to the model. Fourthly, the rainfall frequency estimates were extrapolated to establish the rainfall inputs to the low frequency storms of interest to the related flood hazard study (Wallace, 2004).

Without updating the original hydrological analysis of the rainfall records providing input to the rainfall / runoff model and revising the model itself, it is difficult to speculate on how accurate the peak flood discharge predictions of the model are. However, when compared against the flood frequency estimates obtained for the measured flow record, it can be inferred that rainfall / runoff model predictions are conservative.

As noted in Section 2.1, in the lower part of the Waitohu Stream Catchment, there are large areas of low-lying land behind the coastal sand dune barrier which provide flood storage for floodwaters in a significant flood before they exit through the gap in the dune barrier cut by the stream. For these flood storage areas the key parameter influencing peak flood levels is the flood volume rather than the peak discharge. For such areas, Harkness's (2003) analysis found the critical storm duration to generally be 6 hours for most floods. In contrast the key parameter influencing peak flood levels at the sites of the existing SH1 and proposed Expressway bridges is the peak flood discharge. For these locations, the critical storm duration is 2 hours for most floods.

Table 2-2 summarises the flood estimates obtained by Harkness (2003) for 2 hour duration storms from the rainfall / runoff modelling. This includes the flow contributions from the Mangapouri Stream and the surface runoff from the Greenwood and Ngatotara sub-catchments. The latter sub-catchment lies to the north of the Greenwood sub-catchment and the runoff from it enters the main stem of the Waitohu Stream downstream of the existing NIMT railway bridge (Figure 2-1). Similarly the inflow from the Mangapouri Stream enters the Waitohu Stream even further downstream. The streambed slope of the Waitohu Stream past the bridges is hydraulically quite steep so that the backwater effect from these tributary and sub-catchment inflows does not extend far enough upstream to affect flood levels at these bridge sites.

Comparison of Harkness's (2003) rainfall / runoff model sourced flood estimates at the gauging station site in Table 2-2 with the flood estimates from the frequency analysis of the 17 year long gauging station flow record scaled to the SH1 bridge site in Table 2-1 indicates that the former are much more conservative than the latter. For example, for the 1% AEP flood, the Harkness (2003) flood estimate of 181m^3/s for the gauging station site is 46-59% larger than the flood estimate from the frequency analysis of the 1994-2010 gauging station flow record scaled to the bridge site (135-146m^3/s), depending on the frequency distribution assumed as the best fit for the annual flood maxima series.

Table 2-2 Flood estimates for Waitohu Stream derived from rainfall / runoff model predictions by Harkness (2003) for 2 hour duration storms

AEP (%)	Flood Estimate (m^3/s)			
	Gauging Station	Greenwood sub-catchment	Mangapouri Stream	Ngatotara sub-catchment
50	49	2.8	7.3	3.1
20	78	4.6	12.5	5.0
10	96	6.0	16.4	6.4
5	123	7.7	21.1	8.3
2	159	9.2	26.0	10.9
1	181	10.6	29.0	12.4
0.5	202	12.0	33.0	14.0

2.4 Flood Estimates Used as Inputs to Hydraulic Model of Waitohu Stream

As noted previously, the limited length of the Waitohu Stream gauging station flow record gives rise to some uncertainty in the flood estimates predicted by the frequency analysis of the record. Harkness's (2003) rainfall / runoff model also used design rainfall inputs derived from short term rainfall records and the calibration of the model against a number of storm events for which the peak discharge has subsequently been revised in the stream flow record (based on adjustments to the stage / discharge rating curves) also gives rise to some uncertainty in the flood estimates based on this approach. The uncertainties with both approaches mean that the flood estimates from neither approach can be preferred although the rainfall / runoff model derived flood frequency estimates are likely to be fairly conservative.

Wallace (2004) used the runoff hydrograph predictions of Harkness's (2003) rainfall / runoff model as an input to a one-dimensional computational hydraulic model of the stream and floodplain system in order to assess the flood hazard throughout the lower part of the Waitohu Catchment (i.e. along the coastal plain between the foothills and the sea). To reflect the likely rainfall gradient across the whole catchment due to orographic effects, Wallace (2004) assumed for the 1% and 0.5% AEP 2 hour duration rainstorm events adjusted for possible future climate change effects to 2090 that the runoff from these storms in the upper hill country part of the catchment would coincide with the runoff from a 5% AEP rainstorm of the same duration over the lower coastal plain part of the catchment.

In order to assess the effect of the proposed Expressway on the existing flood hazard and to derive design flood levels for the Expressway crossing of the Waitohu Stream and floodplain, an adaptation of the one-dimensional computational hydraulic model of the stream and floodplain system developed by GWRC (Wallace, 2004) was utilised as discussed subsequently in Sections 4 and 5. For reasons of consistency then, the flood estimates obtained from the rainfall / runoff model approach by Harkness (2003) were adopted for the investigations of the effect of the proposed Expressway crossing of the Waitohu Stream and floodplain in this report. However it is noted that these flood estimates are likely to be conservative.

The MfE (2010) Guidelines for estimating the effects of possible future climate change on flood flows suggest a mid-range estimate for increased average rainfall of +17% to 2090 for the Wellington and Manawatu regions. However GWRC have recently revised their flood hazard assessment for the Waitohu Catchment to account for the effects of possible future climate change to 2090 assuming increased flood magnitudes of 20% for the main Waitohu Stream at the gauging station site and 16% for the Greenwood and Ngatotara sub-catchments with the original estimated inflows from the Mangapouri Stream being retained (P Wallace, pers. comm.)[7]. These adjusted flood discharge estimates are summarised in Table 2-3 below.

Table 2-3 Flood estimates based on 2 hour duration rainstorm for Waitohu Stream approximately adjusted for the effects of possible future climate change to 2090

[7] The peer reviewer (MacMurray, 2012) has pointed out that the catchment response to increased rainfall due to climate change effects could be non-linear as opposed to the assumed linear response. This implies that the linearly adjusted flood estimates in Table 2-3 will be on the conservative side.

AEP (%)	Flood Estimate (m^3/s) (current climate conditions)				Flood Estimate (m^3/s) (adjusted for climate change to 2090)			
	Gauging Station	Greenwood sub-catchment	Ngatotara sub-catchment	Mangapouri sub-catchment	Gauging Station	Greenwood sub-catchment	Ngatotara sub-catchment	Mangapouri sub-catchment [2]
1	181	7.7[1]	8.3[1]	21.1[1]	217	8.9[1]	9.6[1]	21.1[1]
0.5	202	7.7[1]	8.3[1]	21.1[1]	242	8.9[1]	9.6[1]	21.1[1]
1	181	10.6	11.3	29.0	217	12.3	13.1	29.0

[1] 5% AEP flood
[2] No climate change adjustments made to flood flows for this sub-catchment (P Wallace, pers. comm.)

The first two flood scenarios were the ones used by GWRC in their flood hazard assessment for the Waitohu Stream (Wallace, 2004). For consistency, the same flood scenarios were used in this investigation. The third scenario was used to determine the appropriate size for the Greenwood sub-catchment culvert under the Expressway and to check whether or not the design freeboard criterion could be satisfied.

3. Treatment Philosophy for Expressway Crossing of Floodplain

The 1% AEP flood adjusted for possible future climate change effects to 2090 was adopted as the Serviceability Limit State flood for the proposed bridge crossing of the Waitohu Stream as per the guidelines of the NZ Transport Agency's *Bridge Manual* (Transit NZ, 2003). The Bridge Manual requires that the "total waterway" be designed to pass the Serviceability Limit State flood. In this context we have taken the "total waterway" to mean the combined stream and floodplain flow.

In terms of a design freeboard standard for the bridge crossing, this is influenced by two factors: the potential for sediment aggradation along the stream bed past the bridge and the potential for snagging of flood transported woody debris by the either the bridge deck or the bridge piers.

The upper catchment is heavily forested and the stream channel across the alluvial fan and floodplain has willow trees for bank protection in places (including between the existing SH1 bridge and the proposed Expressway bridge) so there are two readily available sources for woody debris. However the stream is crossed by a number of existing bridges upstream (the waterworks access bridge, the Ringawhati Road bridge, the Waitohu Valley Road bridge and the SH1 bridge). The first three bridges do not have particularly large waterway areas and are predicted to be overtopped and outflanked by the 1% AEP flood (Wallace, 2004). They are also likely to act as a trap for flood transported woody debris. The same applies to the existing SH1 bridge.

However the presence of these upstream bridges acting as debris traps for woody material and affording a degree of protection to the Expressway bridge needs to be balanced against the design life for the new structure. It is conceivable that over the design life of the Expressway bridge that all of the upstream bridges could eventually be replaced. In the case of the local road bridges and particularly the waterworks access bridge, these would not necessarily be replaced by new structures with a significantly larger waterway area so that any replacement structure could still potentially act as a woody debris trap in extreme floods.

If the existing SH1 bridge was replaced in the future, then there might be a case for designing it for a higher level of service than it currently has. Any replacement structure would still suffer adversely from being located immediately downstream of a sharp bend in the stream channel although, if it was designed for a higher level of service, then it would be less prone to snagging woody debris under flood conditions. The effect of replacing the existing SH1 bridge with a new one having a much larger waterway area is considered further in Section 5.4.

The NZ Transport Agency's *Bridge Manual* (Transit NZ, 2003) requires a minimum freeboard of 0.6m above the Serviceability Limit State design flood level in normal circumstances and 1.2m "*where the possibility that large trees may be carried down the waterway exists*". Therefore, from a woody debris perspective, a design freeboard allowance of between 0.6m and 1.2m from the design flood level to the underside of the Expressway bridge would be a minimum design standard in this context based on the guidelines of the *Bridge Manual*

Williams (2004) has identified the reach between the existing SH1 bridge and the Wakapua Farm bridge downstream of the NIMT railway bridge as zone of sediment deposition due to a sharp reduction in the streambed slope. GWRC monitor stream bed levels by means of periodic cross-section surveys of the stream

channel and have in the past actively managed the deposition of gravel bed material in the vicinity of the NIMT railway bridge which appears to be the primary sediment aggradation hotspot.

Gardner (2009) has analysed mean bed levels of the Waitohu Stream based on cross-section surveys in 1992, 2003 and 2009. He has confirmed increases in mean bed level in the reach between the existing SH1 bridge and the NIMT railway bridge of 0.1-0.3m and up to 1m at one cross-section 720 downstream of the railway bridge from 1992 to 2003 and much lesser increases in mean bed level including slight decreases at some cross-sections from 2003 to 2009. Gardner (2009) notes that gravel extraction since 2004 in this aggrading reach appears to have been effective in causing the reduced aggradational response over the 2003 to 2009 period.

It is clear then that the design freeboard allowance for the proposed Expressway bridge needs to reflect both the potential for increased head losses due to the snagging of woody debris material on the piers and the ongoing occurrence of sediment aggradation in the 1.7km long reach below the existing SH1 bridge. Based on the evidence available regarding sources of woody debris material and the magnitude of historic streambed aggradation, it would appear reasonable to adopt a design freeboard allowance for the Expressway bridge of 1.2m[8].

The same Serviceability Limit State flood adopted for the proposed bridge crossing of the Waitohu Stream is also appropriate as the design standard for the large dry culverts through the left and right approach embankments and for the Greenwood sub-catchment culvert. Normally these culverts would require a minimum design freeboard allowance of 0.5m as per the guidelines of the NZ Transport Agency's *Bridge Manual* (Transit NZ, 2003). However, in the case of the left and right approach embankment culverts, there is potential for upstream flood levels to be partially influenced by bed aggradation in the mainstream channel. It would therefore be appropriate to adopt a higher design freeboard allowance of 0.8m for these culverts and for the approach embankments generally. The *Bridge Manual's* standard guideline for a design freeboard allowance of 0.5m would however be suitable for the Greenwood sub-catchment culvert and the road embankment either side of it.

In the case of the large dry culverts through the bridge approach embankments, it is very easy to satisfy these design standards as the approach embankments are elevated fairly high above the floodplain and provide ample freeboard. In the case of the Greenwood sub-catchment culvert, they will be more difficult to satisfy. This is because the existing culvert location is a known flooding hotspot with the culvert system being under capacity and the existing road being overtopped from time to time. The available depth in which to form any new culvert structure is very constrained as the proposed road level is no more than about 2m above the invert level of the dry channel leading to the culvert. Structural considerations will require a minimum depth of fill over any culvert.

The design philosophy originally adopted for the northern end of the Expressway was to accept the existing flood risk at the Greenwood sub-catchment culvert in the interim until the Ōtaki to Levin Expressway is

[8] In view of the history of stream bed aggradation in the reach below the existing SH1 bridge where the proposed Expressway bridge will cross the Waitohu Stream, streambed levels will need to be actively monitored in consultation with GWRC. Any future aggradation within the immediate vicinity of the Expressway bridge may well require ongoing intervention by NZTA to maintain current streambed levels.

constructed in the future. The existing flood risk would then be resolved. This philosophy has not changed even though the Ōtaki to Levin Expressway as part of the Wellington Northern Corridor RoNS Project has been shelved.

The vertical alignment for the northern end of the Expressway has now been defined to transition into the existing vertical alignment of SH1 past Taylors Road. This is a fixed constraint therefore for this investigation. The limited discharge capacity of the existing culvert system has been addressed in order to future-proof the northern end of the Expressway. This means that an appropriate culvert size has been determined so that only the vertical alignment of the Expressway needs to be raised in the future in order to achieve the required level of service with respect to freeboard.

4. Hydraulic Performance in Current Situation

4.1 Outline of Existing Situation

Figure 4-1 shows a detailed aerial photograph of the area of floodplain crossed by the existing SH1 and the NIMT railway line and also to be crossed by the proposed Expressway.

Culverts under SH1, Taylors Road and the NIMT railway line provide continuity for the existing secondary flow path across the alluvial fan for the Greenwood sub-catchment. There is no culvert under SH1 on another existing secondary flow path along the north (right) bank of the Waitohu Stream (draining what Coles and Bird (2012) refer to as the Coopers sub-catchment) but there is a culvert on this flow path under the NIMT railway line. Similarly there is no culvert under SH1 on an existing secondary flow path along the south (left) bank of the stream. Where no culvert is provided, surface runoff will simply flow over the road as weir flow.

Where culverts are provided, the design capacity of them could well be less than the magnitude of the overland flow resulting from the 1% AEP flood adjusted for possible future climate change effects in combination with tributary inflows. Tables 4-1 and 4-2 summarise the dimensions and levels for the culverts on the secondary flow paths for the Greenwood and Coopers sub-catchments respectively.

Table 4-1 Dimensions and levels for existing culverts on Greenwood sub-catchment overland flow path

Location	Type	Size (m)	Invert Level (m MSL Wellington)		Length (m)	Slope (%)	Road Level (m MSL Wellington)
			u/s	d/s			
SH1 [1]	Circular (2x)	1.05m dia 0.9m dia.	23.11 23.15	22.96 23.21	20.9 18.1	0.72% -0.33%	25.1
Taylors Rd No. 1	Circular	1.5m dia.	18.34	18.11	18.4	1.50%	20.3
NIMT railway line	Box	1.15m x 1.25m [2]	16.20	16.09	14.8	0.74%	19.1
Taylors Rd No. 2		1.15m x 1.25m [2]	16.09	16.09	12.6	0%	18.4

[1] The two culverts at this location follow different alignments and have a minimal depth of fill over them.

[2] Dimensions given as width x height.

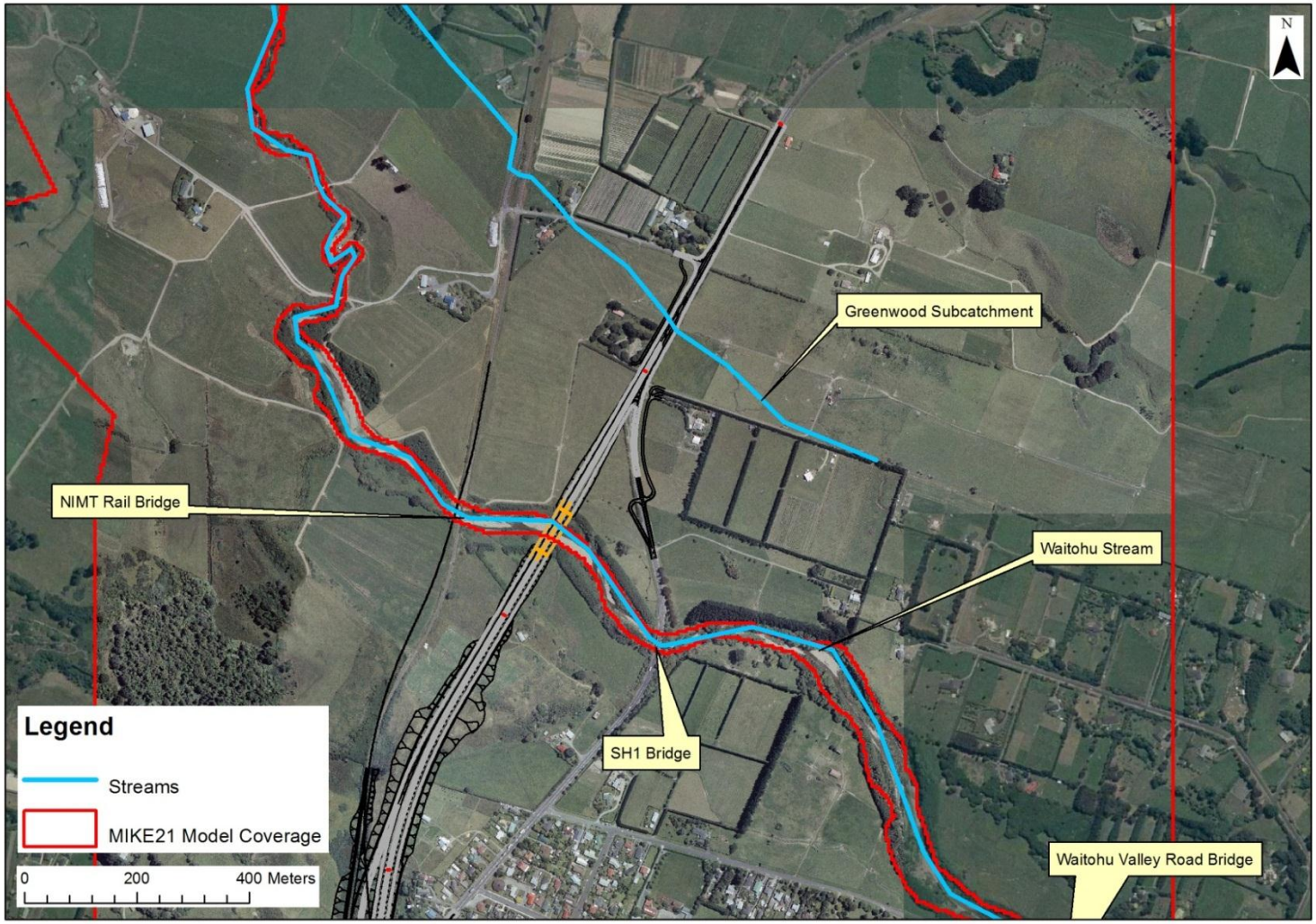

Figure 4-1 Aerial photograph of Waitohu Stream alluvial fan and floodplain crossed by existing SH1 and NIMT railway routes and proposed PP2O Expressway route

Table 4-2 Dimensions and levels for existing culverts on Cooper's sub-catchment overland flow path

Location	Type	Size (m)	Invert Level (m MSL Wellington)		Length (m)	Slope (%)	Rail Level (m MSL Wellington)
			u/s	d/s			
NIMT railway line	Box	0.92m x 0.97 m [1]	20.28	20.31	9.5	-0.32%	21.3

[1] Dimensions given as width x height.

The existing SH1 bridge over the Waitohu Stream (see Figure 4-2) is a fairly low three span structure with a total span length of 27.43m (centre span of 10.97m and two outer spans of 8.23m each). The bridge lies on a slight curve requiring the bridge deck to be superelevated. Superelevation of the bridge deck means that the soffit is 0.41m higher on the upstream side compared to the downstream side (the upstream and downstream soffit levels are 30.75m and 30.34m respectively, MSL Wellington datum).

The existing SH1 bridge crossing is located immediately downstream of a sharp, nearly ninety degree, bend. This causes the long rectangular-shaped piers to be skewed at a fairly large angle to the general direction of stream flow. The modest total span length and the low height of the structure above stream bed level mean that the bridge waterway area is constrained in size relative to the magnitude of the flood estimates for the Waitohu Stream (Table 2-2). These aspects of bridge location, bridge geometry and waterway capacity are contributing factors to the potential for flood breakout along both stream banks upstream of the bridge, particularly on the right bank.

The other significant factor contributing to the potential for flood breakout along both stream banks is the low height of the banks as seen in the photograph on the front cover of this report (the photograph shows a view of the stream looking upstream of the bridge from the right bank).

The NIMT railway bridge over the Waitohu Stream (downstream of the SH1 bridge crossing – Figure 4-1) is a relatively new four span structure as seen in Figure 4-3. The total span length of this structure is 40.3m made up of four approximately equal spans varying between 10.06m and 10.11m. The estimated soffit levels are about 24.41m and 24.12m (MSL Wellington datum) at the south (left bank) and north (right bank) abutments respectively. The longer span length of this bridge gives it a greater waterway capacity than the existing SH1 bridge. It is noted that the waterway width at the new rail bridge (≈ 40m) is larger than the active channel width of the Waitohu Stream upstream and downstream (~ 25m).

4.2 Flood Inundation Predictions for Existing Situation for 1% AEP Flood

Figure 4-4 shows the extent of flood inundation across the floodplain predicted by the MIKEFLOOD model for the 1% AEP flood adjusted for the effects of possible future climate change to 2090 (this is the flood generated by the 2 hour duration 1% AEP rainstorm adjusted for possible climate change effects). The extent of inundation also shows ranges of peak flood depths.

Figure 4-2 Photograph looking downstream at existing State Highway bridge over Waitohu Stream

Figure 4-3 Photograph looking upstream at existing NIMT railway bridge over Waitohu Stream

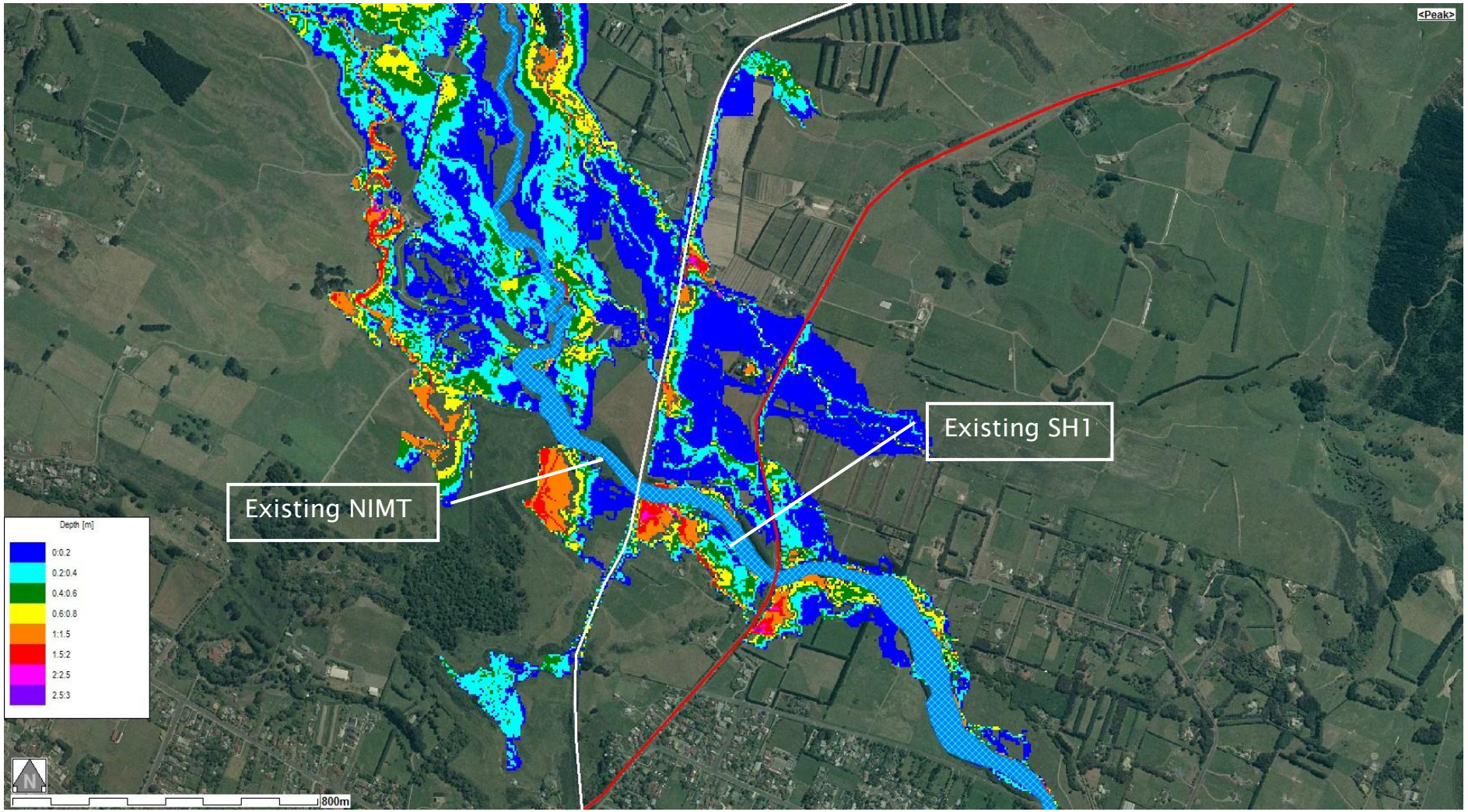

Figure 4-4 Predicted flood depths across Waitohu Stream alluvial fan and floodplain in existing situation for 1% AEP flood (based on 2 hour duration 1% AEP rainstorm) adjusted for possible future climate change effects to 2090

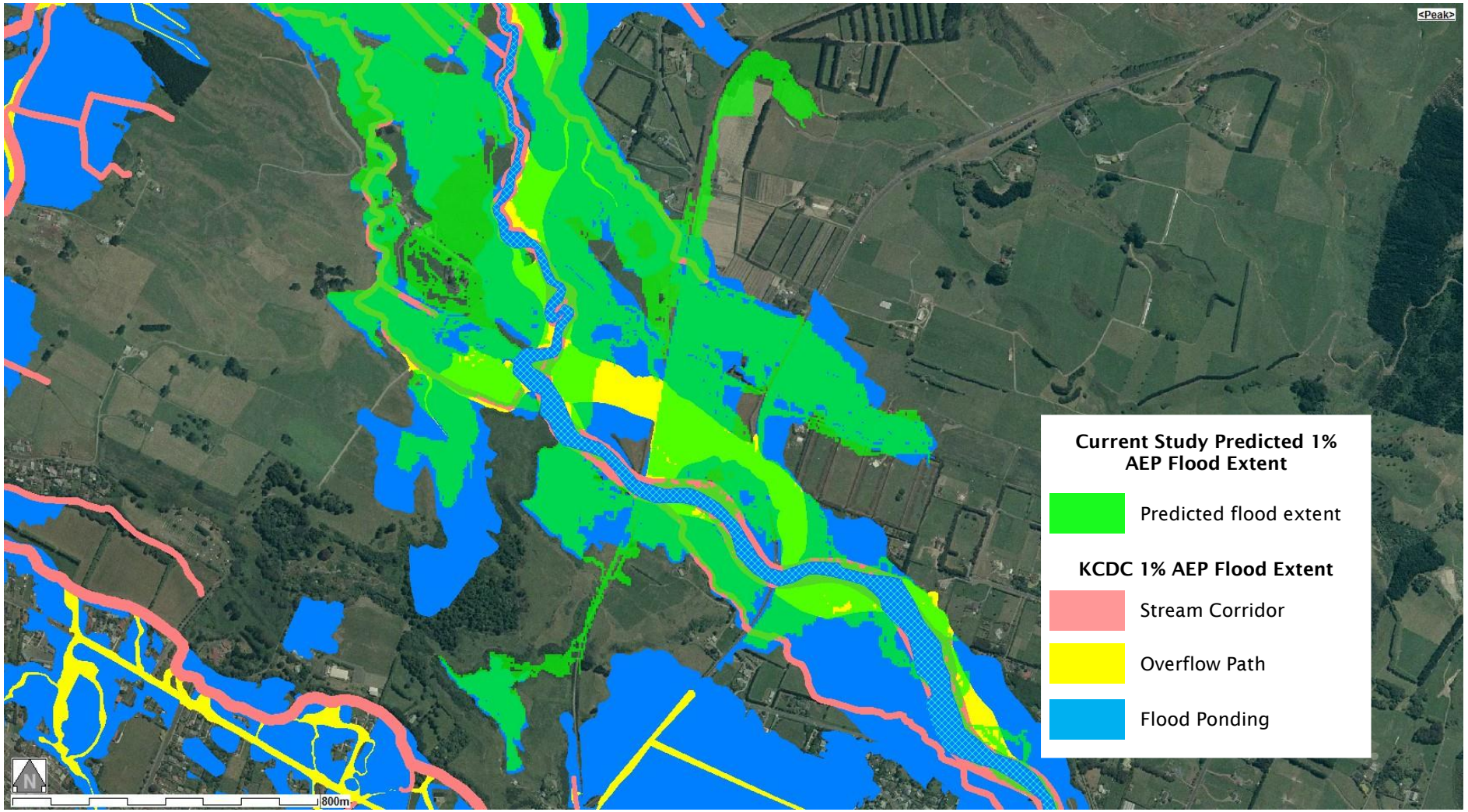

Figure 4-5 Predicted flood extent (green shading) across Waitohu Stream floodplain in existing situation for 1% AEP flood adjusted for possible future climate change effects compared to flood extent on Kāpiti Coast District Plan flood hazard map (pink, yellow and blue shading).

The predicted flood extent from Figure 4-4 has been overlaid on the relevant flood hazard map from the Kāpiti Coast District Plan in Figure 4-5. The blue, yellow and pink under-layer is from the District Plan flood hazard map with the pink area representing the stream corridor, yellow areas the main overland flow paths and blue areas flood ponding zones. The overlying green area represents the predicted flood extent from the MIKEFLOOD model for the 1% AEP flood adjusted for possible climate change effects.

It is not known whether the flood hazard extent from the District Plan in the area of interest is based on the estimated flood extent for the flood resulting from a 6 hour duration 1% AEP rainstorm (which is critical for the low-lying land behind the coastal dune barrier along the coast) or the flood resulting from a 2 hour duration 1% AEP rainstorm (which is more critical for the floodplain area crossed by SH1 and the NIMT railway line) or a worst case combination for both floods. Irrespective of the origin of the flood extents on the Kāpiti Coast District Plan flood hazard maps, the agreement between the two flood inundation patterns is fairly good.

However there are some points of difference between the two flood hazard extents. Firstly, the extent of flood inundation predicted by the MIKEFLOOD model for the existing situation is not as large as that shown on the Kāpiti Coast District Plan flood hazard map does. As noted above, it is not known what flood magnitude the latter hazard map is based on. However those areas where the District Plan flood hazard map indicates more widespread inundation than that shown in Figure 4-4 are predicted by the MIKEFLOOD model to be inundated by the 2 hour duration 0.5% AEP rainstorm adjusted for possible climate change effects to 2090 as seen in Figure 4-7.

The second point of difference between the two flood hazard extents is along the upstream side of the NIMT railway line to the north of the Greenwood sub-catchment overflow path. Figure 4-4 shows floodwaters ponding upstream of the NIMT railway culvert on this overflow path spreading north whereas the Kāpiti Coast District Plan flood hazard map does not show any flood inundation spreading north of the railway culvert. The inundation in this area of difference in Figure 4-4 is a conservative prediction as it assumes that the railway embankment is impervious whereas in reality the ballast would allow some leakage of floodwaters through it.

The third point of difference between the two flood hazard extents is between the Greenwood sub-catchment overflow path and the main stream channel on the downstream side of the NIMT railway line. This is probably due to a couple of factors. Firstly, the NIMT railway bridge and its approaches have been raised in the order of 1.2m since the Kāpiti Coast District Plan flood hazard map was prepared. Secondly, flood inundation prediction across a flat floodplain by a two-dimensional computational hydraulic model is generally more accurate than inferring the flood inundation extent across the same floodplain from flood levels predicted by a one-dimensional computational hydraulic model.

The flood inundation pattern in Figure 4-4 predicted by the MIKEFLOOD model confirms that the predominant overland flow paths follow the course of natural dry channels across the alluvial fan and floodplain surface. It also indicates that the restricted waterway area of the existing SH1 bridge as well as the location of the bridge immediately downstream of a sharp bend act as a throttle on flood flows and contribute to flood breakout along both banks upstream of the bridge, particularly along the right bank.. The low bank heights on either side of the active channel upstream of the bridge as seen in the cover photograph of this report are another contributing factor to this behaviour.

The effect of these features is to cause extensive inundation of SH1 across the floodplain by the 1% AEP flood adjusted for possible future climate change effects. SH1 is also predicted to be inundated at the Greenwood sub-catchment culvert location although this is due to a sub-catchment flood resulting from a 2 hour duration 5% AEP rainstorm (adjusted for possible future climate change effects) in combination with the flood resulting from the 1% AEP rainstorm (also adjusted for possible future climate change effects) impacting on the main Waitohu Catchment. Note that, except along the narrow vee-shaped dry channel forming the main drainage path for the Greenwood sub-catchment, the peak flow depths across the inundated area of this sub-catchment are very shallow (< 0.2m). This reflects the very flat nature of the sub-catchment. The flood inundation over SH1 is also very wide and shallow.

Peak flow depths across the floodplain as seen in Figure 4-4 are generally predicted to be fairly shallow in most areas, typically less than 0.4m. Along the natural dry channels within the predominant overland flow paths, flow depths are deeper. There are other isolated pockets including in front of the elevated railway track and SH1 where flow depths are also predicted to be deeper (as much as 2m).

Complementing Figure 4-4, Figure 4-6a shows peak flow velocities across the floodplain predicted by the MIKEFLOOD model for the 1% AEP flood adjusted for the effects of possible future climate change to 2090. Peak flow velocities across the floodplain are quite high in places (up to 1-1.5m/s and occasionally higher) along the predominant left and right bank flood plain flow paths past the SH1 and NIMT railway line. In the Greenwood sub-catchment, the peak flow velocities are typically less than 0.4m/s implying that the shallow floodwaters are not surprisingly fairly slow moving.

Figure 4-6b shows a zoomed-in part of Figure 4-6a around the existing SH1 and NIMT railway crossings of the Waitohu Stream and floodplain with velocity vectors superimposed to indicate the directions of overland flow paths. This highlights the predominant flow paths on both the left and right bank floodplains between the SH1 and NIMT railway bridges.

A twin culvert system (one 0.9m diameter culvert and one 1.05m diameter culvert – see Table 4-1) allows surface runoff from the Greenwood sub-catchment to pass under the existing SH1. Figure 4-7 shows flood discharge hydrographs for this system for the 1% AEP flood adjusted for possible future climate change effects to 2090. The two culverts cross under SH1 on diverging courses. The southern culvert is also slightly smaller than the northern one. These aspects give rise to differing hydraulic capacities with the southern culvert passing a peak discharge of only 1.75m^3/s compared to 2.4m^3/s for the larger northern culvert. The total peak discharge passed by the two culverts of 4.15m^3/s is only marginally more than the peak discharge over the road of 3.4m^3/s (see the road overflow discharge hydrograph in Figure 4-7 also).

The other significant feature of the road overflow discharge hydrograph in Figure 4-7 is that it has two peaks. The first peak corresponds to the surface runoff from the Greenwood sub-catchment itself (due to a 5% AEP rainstorm over the sub-catchment adjusted for possible future climate change effects) while the second peak corresponds to floodplain overflow resulting from the 1% AEP flood adjusted for possible future climate change effects breaking out of the main stream channel along the right bank upstream of the existing SH1 bridge.

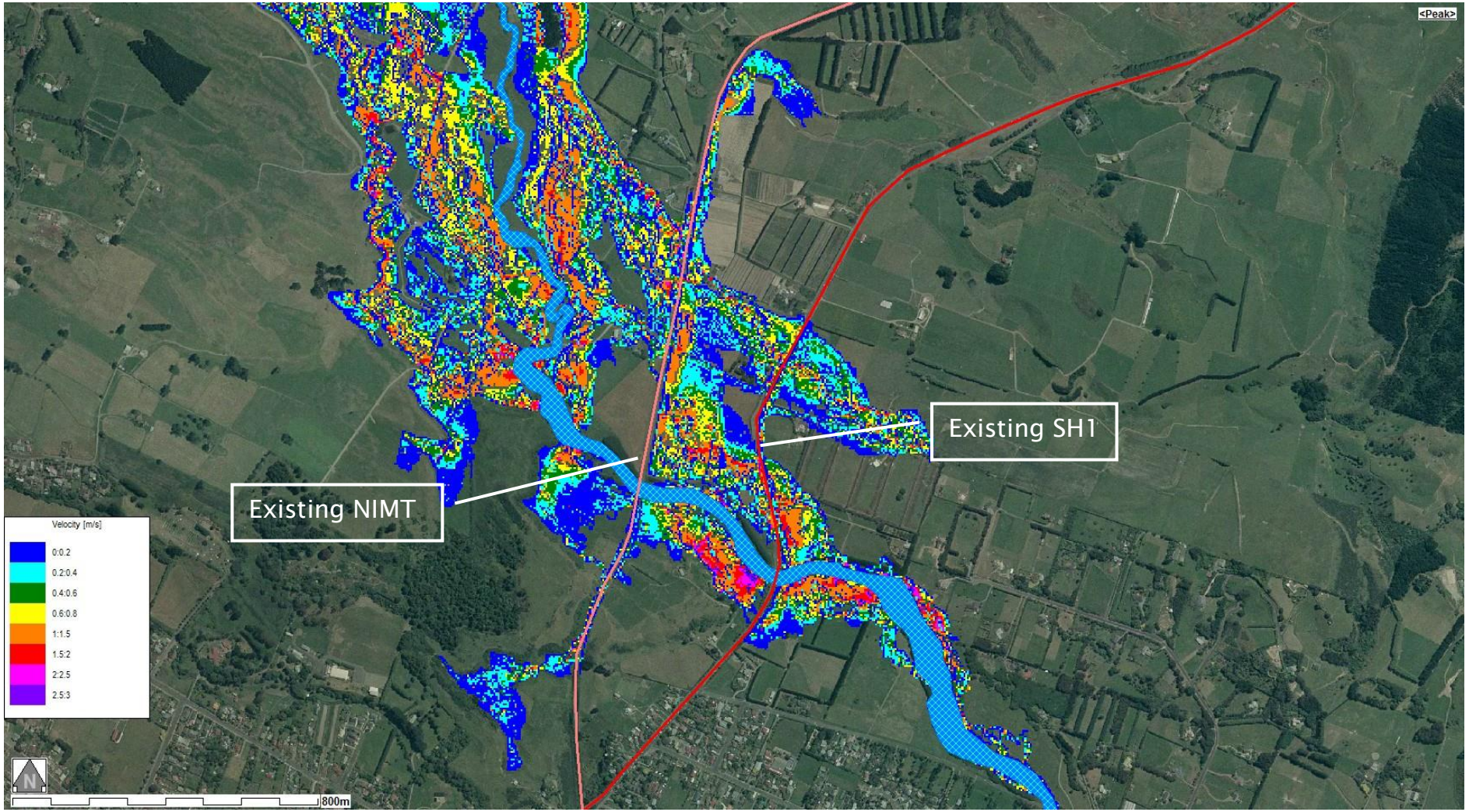

Figure 4-6a Predicted flow velocities across Waitohu Stream alluvial fan and floodplain in existing situation for 1% AEP flood (based on 2 hour duration 1% AEP rainstorm) adjusted for possible future climate change effects to 2090

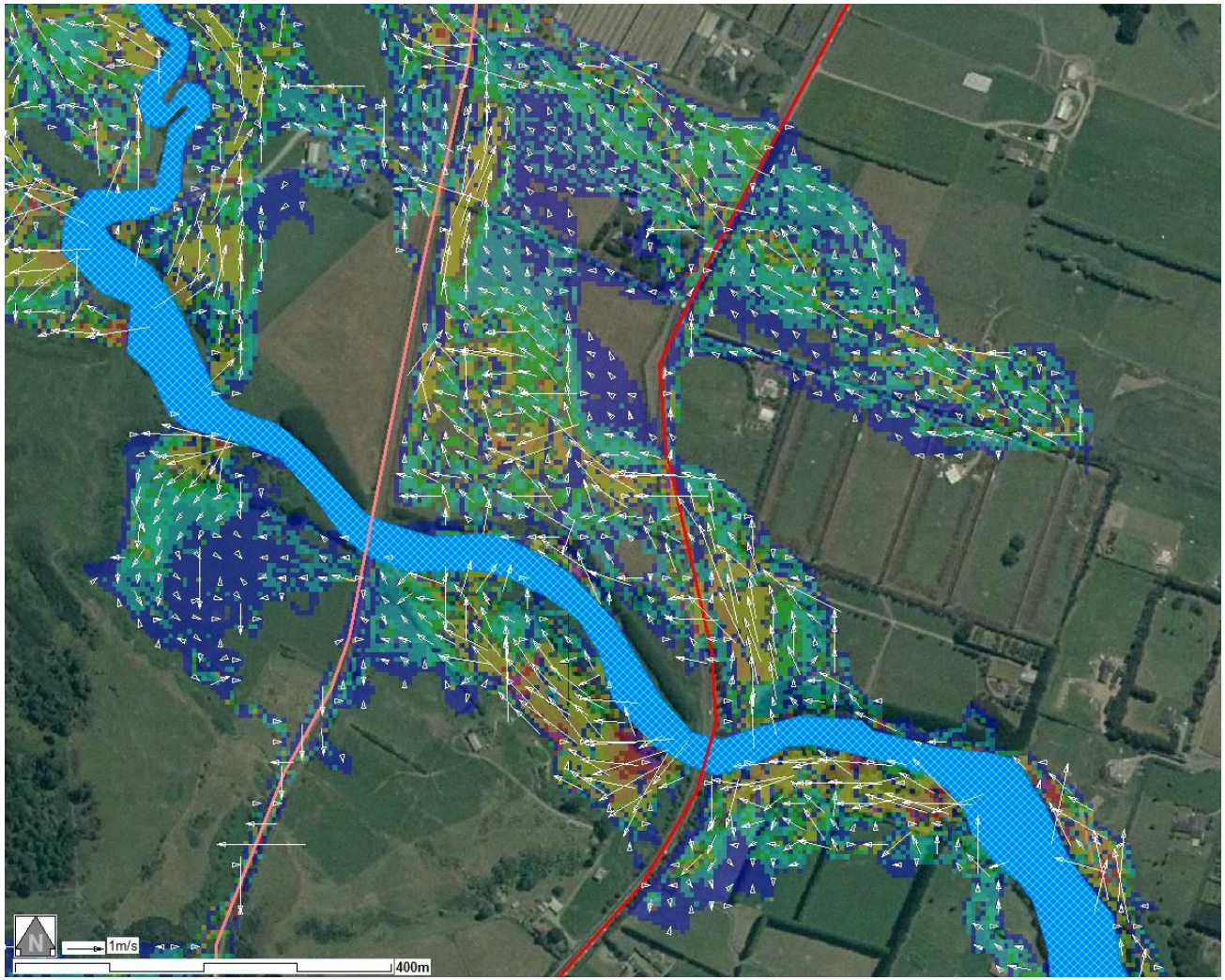

Figure 4-6b Blown-up section of Figure 4-6a showing directions of overland flow paths past the existing SH1 and NIMT railway crossings of the Waitohu Stream and floodplain

The fact that SH1 overflows at this culvert system location in only a 5% AEP flood (adjusted for possible future climate change effects) means that the existing highway is flood-prone and does not meet the required design flood standard identified in Section 3. This observation is consistent with anecdotal evidence of the highway flooding at this location periodically.

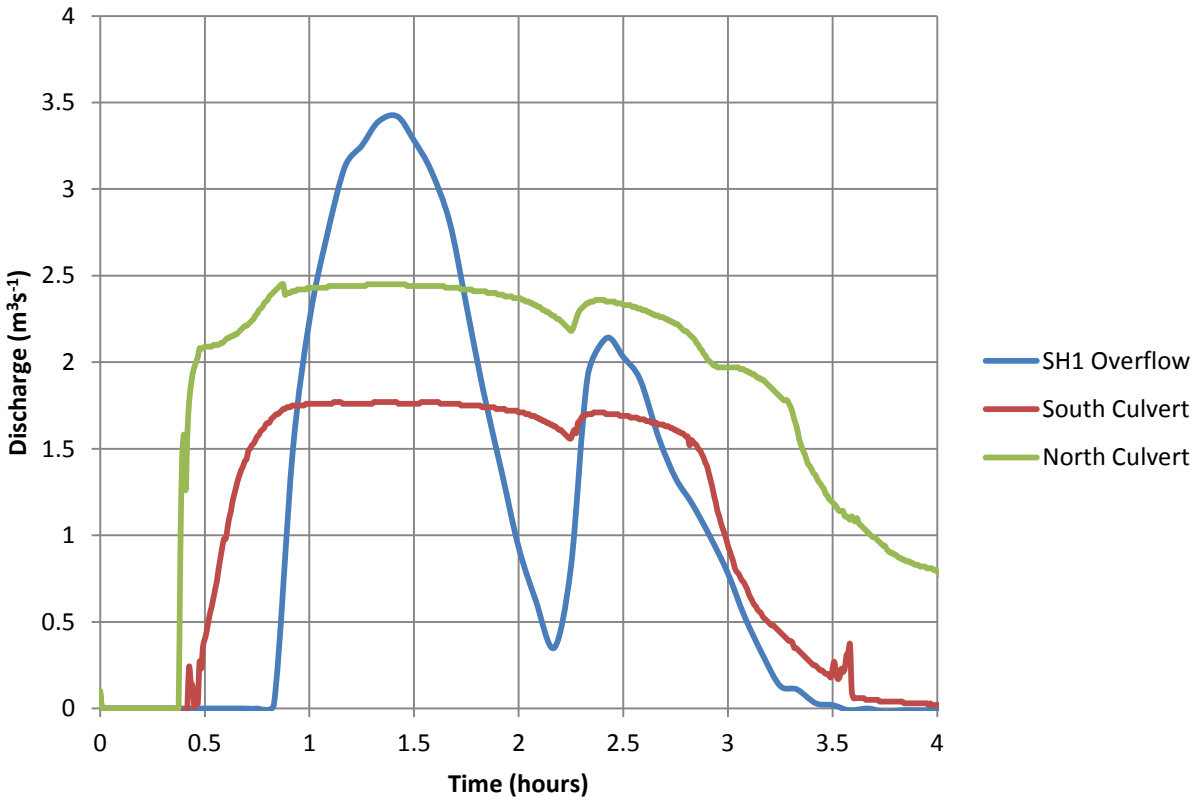

Figure 4-7 Flood hydrographs for Greenwood sub-catchment twin culverts and overflow path on SH1 for existing situation for 1% AEP flood adjusted for possible future climate change effects to 2090

4.3 Sensitivity Tests for Existing Situation

As a sensitivity test, the MIKEFLOOD model of the existing situation was also used to simulate the flood pattern across the floodplain resulting from the 2 hour duration 0.5% AEP rainstorm adjusted for the effects of possible future climate change to 2090. Figure 4-8 shows this flood pattern and the range of peak flood depths. Similarly Figure 4-9 shows the distribution and range of peak flow velocities.

The flood inundation extent and pattern for the 0.5% AEP flood adjusted for possible climate change effects is very similar to that for the 1% AEP flood in Figure 4-4 with only marginally increased flow depths evident from some of the deeper pockets (indicated by the slightly larger extent of the maximum range of flow depths).

The extent of flood breakout across the alluvial fan 1km upstream of the existing SH1 bridge is slightly greater.

Peak flow velocity magnitudes in Figure 4-9 are very similar to those for the 1% AEP flood adjusted for possible future climate change effects in Figure 4-6. The predominant flow paths on both the left and right bank floodplains are again evident from the lines of higher flow velocities.

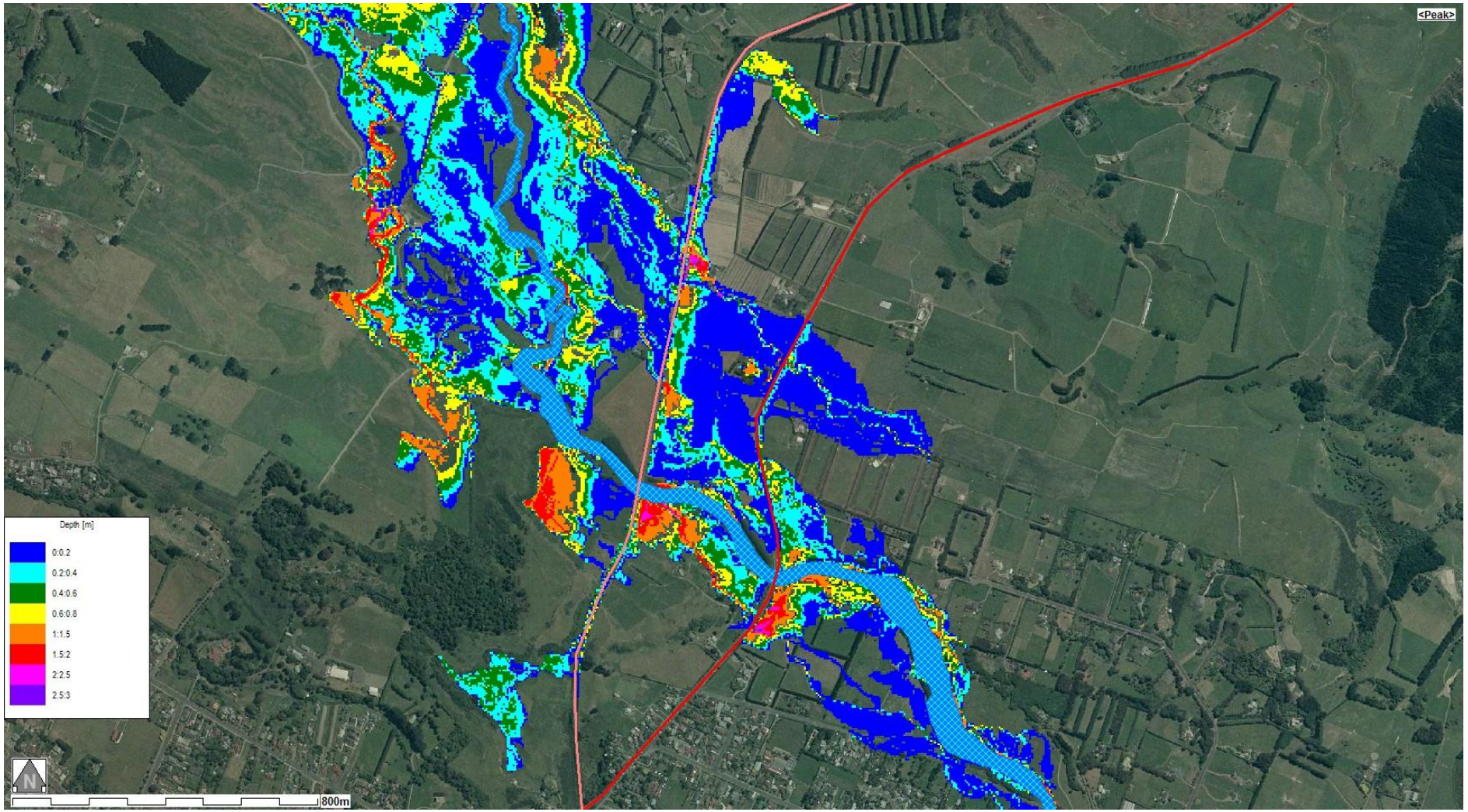

Figure 4-8 Predicted flood depths across Waitohu Stream alluvial fan and floodplain in existing situation for 0.5% AEP flood (based on 2 hour duration 0.5% AEP rainstorm) adjusted for possible future climate change effects to 2090

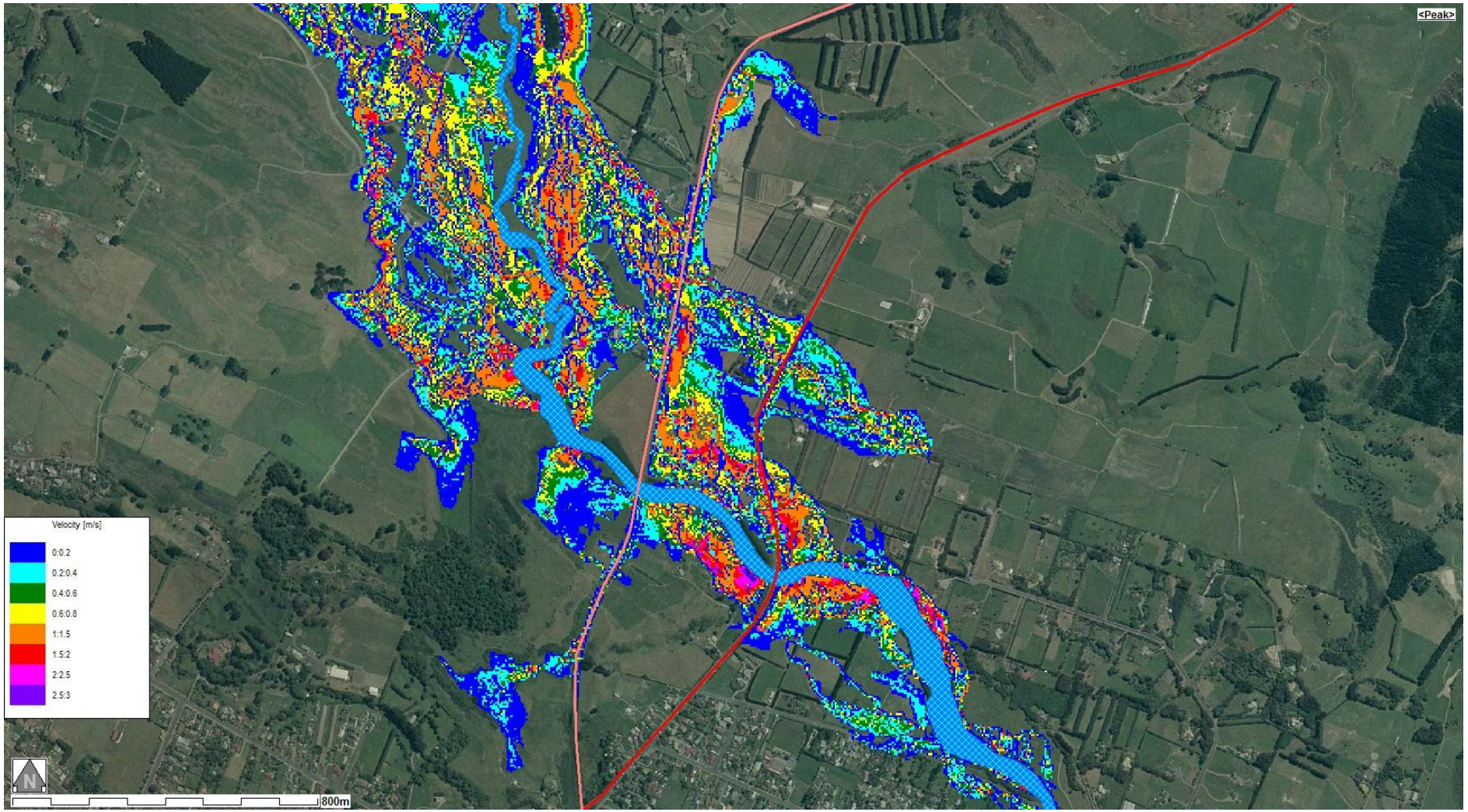

Figure 4-9 Predicted flow velocities across Waitohu Stream alluvial fan and floodplain in existing situation for 0.5% AEP flood (based on 2 hour duration 0.5% AEP rainstorm) adjusted for possible future climate change effects to 2090

5. Hydraulic Performance in Proposed Expressway Situation

5.1 Outline of Proposed Situation

The Expressway bridge will cross the Waitohu Stream approximately 260m downstream and to the west of the existing SH1 bridge crossing. As noted in Section 1.4, the Expressway crossing of the stream is located within a geomorphologic zone of instability characterised by sediment deposition and potential lateral channel instability (Williams, 2004). The bridge crossing has therefore been designed to have an approximately 75m total span length so as not to encroach on the 75m wide fairway width defined by GWRC to allow for potential future channel migration. The effect of the resulting large set back of the bridge abutments from the current active channel location minimises the risk of future abutment attack by high velocity flood flows.

The alignment of the proposed expressway passes directly through the extensive overland flow paths across the alluvial fan and floodplain system on both sides of the Waitohu Stream seen in Figures 4-4 and 4-7. In order to achieve the required level of service outlined in Section 3 for this watercourse crossing, the Expressway will need to be elevated on an embankment. However, on the northern side of the stream crossing the vertical alignment of the proposed expressway will now slope down to grade and transition into the existing horizontal alignment of SH1 before the twin Greenwood sub-catchment culverts. This culvert location is a known flooding hotspot and achieving the desired level of service while maintaining a near at grade vertical alignment for the new road is problematic.

To provide for continuity of existing overland flow paths across the alluvial fan and floodplain surface and to allow overland flows to pass under the Expressway, culverts to convey these flood flows will be provided in addition to a replacement culvert on the Greenwood sub-catchment overland flow path. Table 5-1 summarises the recommended culvert types, dimensions and levels. The location of these culverts is illustrated in Figure 5-1. Note that modification of the existing SH1 bridge to increase its waterway capacity and reduce the volume of flood breakout flows upstream of the bridge does not form part of the proposed PP2O Scheme.

Table 5-1 Culvert types, dimensions and levels for proposed expressway

Location	Type	Size (m)	Invert Level (m MSL Wellington)		Length (m)	Slope (%)	Road Level (m MSL Wellington)
			u/s	d/s			
Southern approach embankment	box	8m x 2.5m [1]	24.5	24	50	1.0	27.7
Northern approach embankment – Coopers overland flow path	box	10m x 1.5m [1]	25.5	25	45	1.1	28.3
Greenwood sub-catchment overland flow path	box	4m x 1.5m	23.05	22.65	40	1.0	25.1

[1] Dimensions given as width x depth.

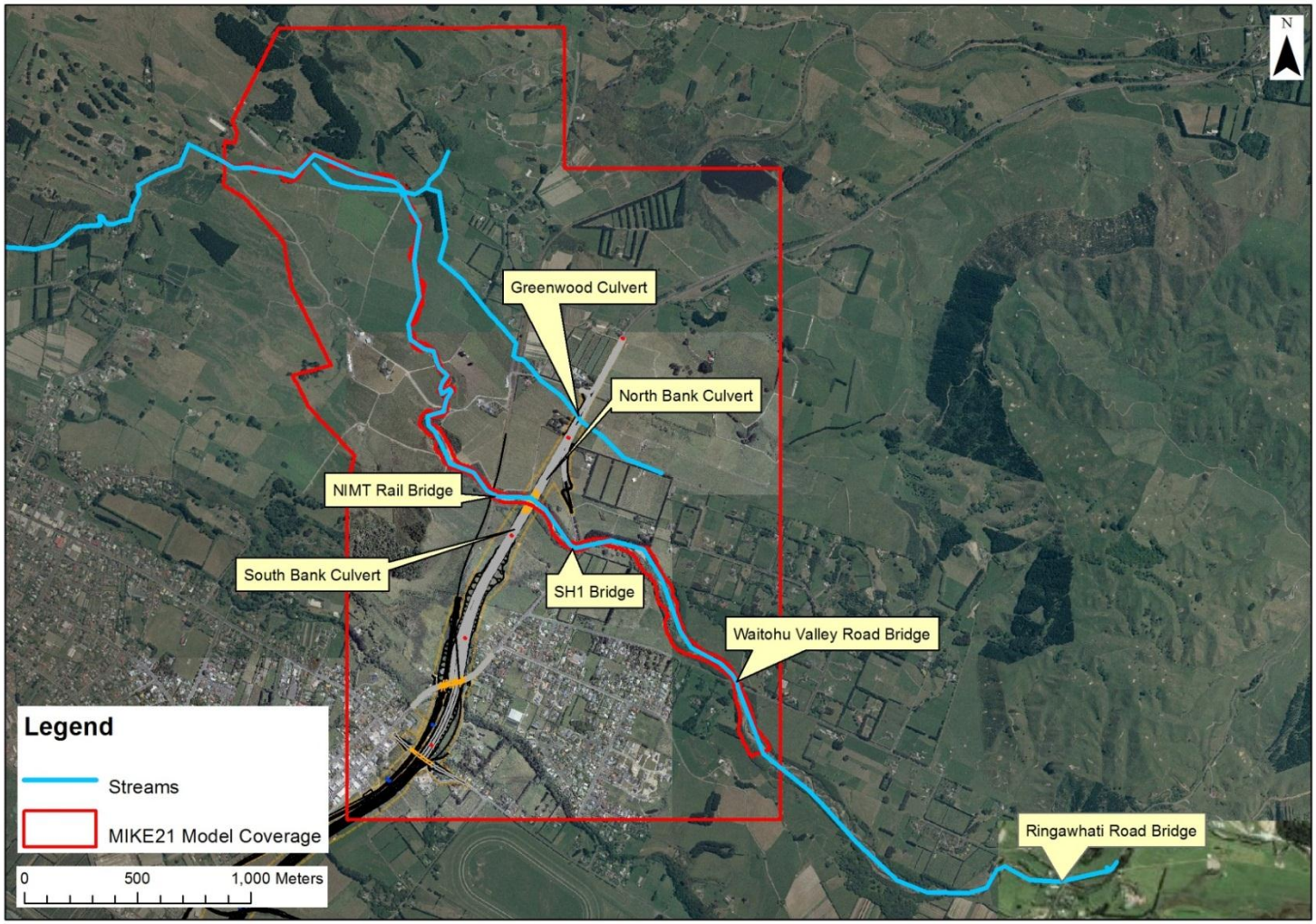

Figure 5-1 Proposed treatment measures to mitigate effects of PP2O Expressway on overland flows across Waitohu Stream alluvial fan and floodplain system resulting from an extreme flood

The design flood magnitude of 12.3m^3/s for the Greenwood sub-catchment culvert under the Expressway (1% AEP flood adjusted for possible future climate change effects to 2090 in Table 2-3) is such that it requires a much larger capacity culvert system than exists currently. The low height of the road relative to natural dry channel levels at the Greenwood sub-catchment overland flow path crossing constrains the maximum culvert size able to be used. The recommended 4m wide by 1.5m culvert in Table 5-1 has very similar invert levels to the existing culverts yet would still have sufficient depth of fill over the top to satisfy structural requirements. The culvert will be inlet controlled and will require an energy dissipation facility at the downstream end.

5.2 Effects of Expressway on Floodplain Inundation for 1% AEP Flood

Figure 5-2 shows a flood inundation map indicating the predicted flood extent across the Waitohu Stream alluvial fan and floodplain system in the proposed situation for the 1% AEP flood adjusted for possible future climate change effects to 2090. The ranges of peak flood depths are shaded in different colours across the flood inundation map. The predicted flood inundation pattern is very similar to that shown in Figure 4-4 for the existing situation.

Figure 5-3 shows the changes in peak flood depth between the existing and proposed situations for the 1% AEP flood adjusted for possible climate change effects. Peak flood depths are generally increased upstream of the Expressway crossing of the stream and floodplain due to the partial damming effect of the bridge approach embankments across the floodplain.

Table 5-2 summarises peak flood levels at common locations upstream of the Expressway for both the existing and proposed situations. Across the floodplain, the Expressway causes flood levels upstream of the 8m wide by 2.5m wide dry culvert on the left bank floodplain overflow path to increase by 0.84m and flood levels upstream of the 10m wide by 1.5m high dry culvert on the right bank floodplain overflow path (referred to as Coopers Culvert by Coles and Bird (2012)) to increase by 0.48m. Figure 5-2 and 5-3 indicate that the extent of these increased flood levels is limited to a distance of only about 100m upstream of the Expressway stream crossing approach embankments. The affected areas are both used for pastoral purposes and are uninhabited[9]. The increased inundation would also only be of limited duration (about 2.5 hours for the left bank overflow path and about 1 hour for the right bank overflow path for the 1% AEP flood adjusted for possible future climate change effects) as well as fairly rare.

In the main stream channel, the Expressway will cause flood levels to increase by 0.16m (Table 5-2) excluding the effect of the bridge piers. The two piers will be rectangular-shaped with a tapered nose and tail (measuring up to 3.1m long and 1.75m wide) and will induce additional head losses. Table 5-3 summarises the estimated pier head losses for the 1% AEP flood adjusted for possible future climate change effects using two standard approaches – Yarnell's method and the rational method (Montes, 1998). The estimated values in Table 5-3 indicate that pier head losses will be in the range of 0.09-0.16m for this flood (this assumes that the existing SH1 bridge remains unmodified[10] and limits the flow past it in a significant flood).

[10] This assumption is based on the brief for the PP2O Project which excludes any remedial treatment of the existing SH1 bridge.

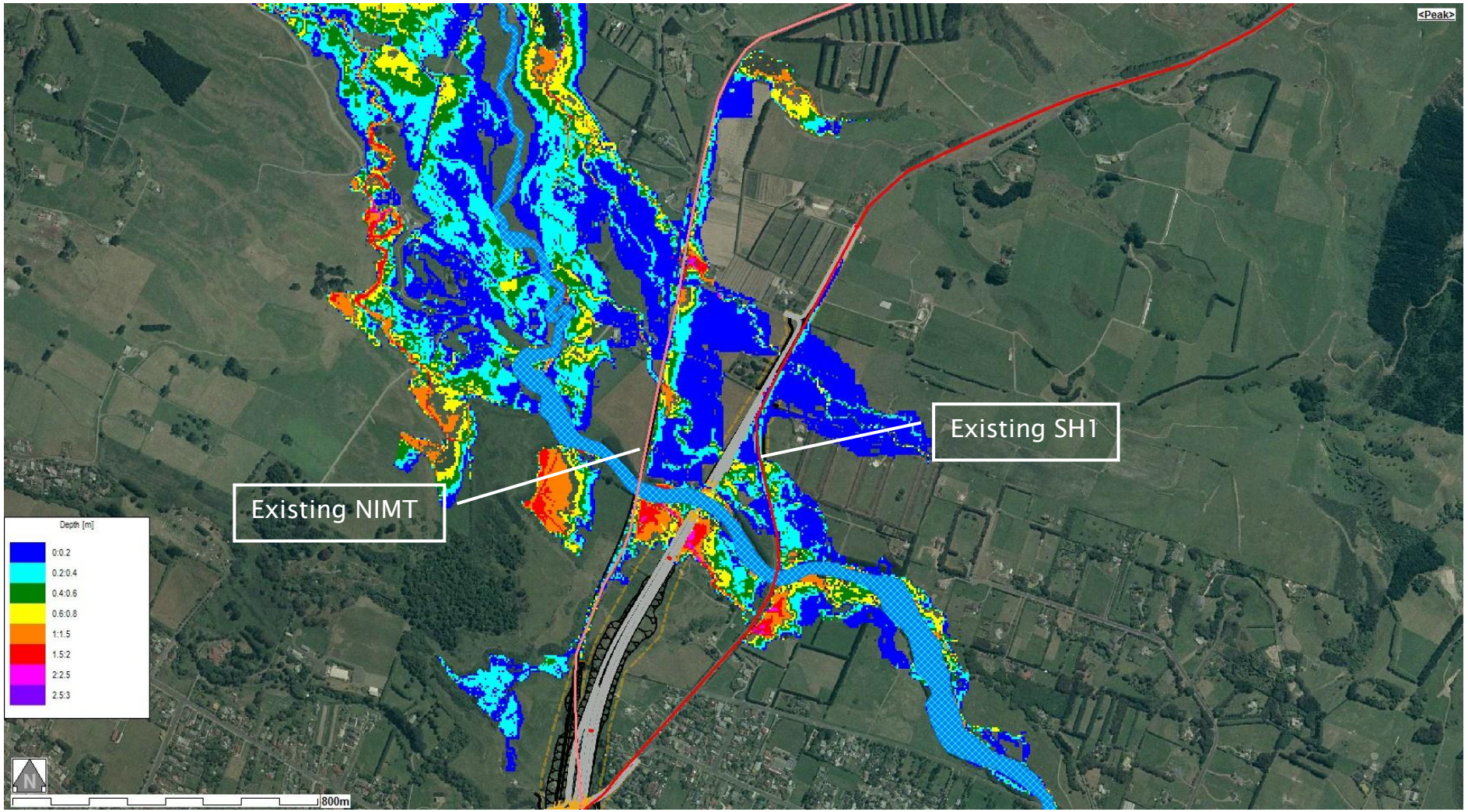

Figure 5-2 Predicted peak flood depths across the Waitohu Stream alluvial fan and floodplain system in proposed situation for 1% AEP flood adjusted for possible future climate change effects to 2090

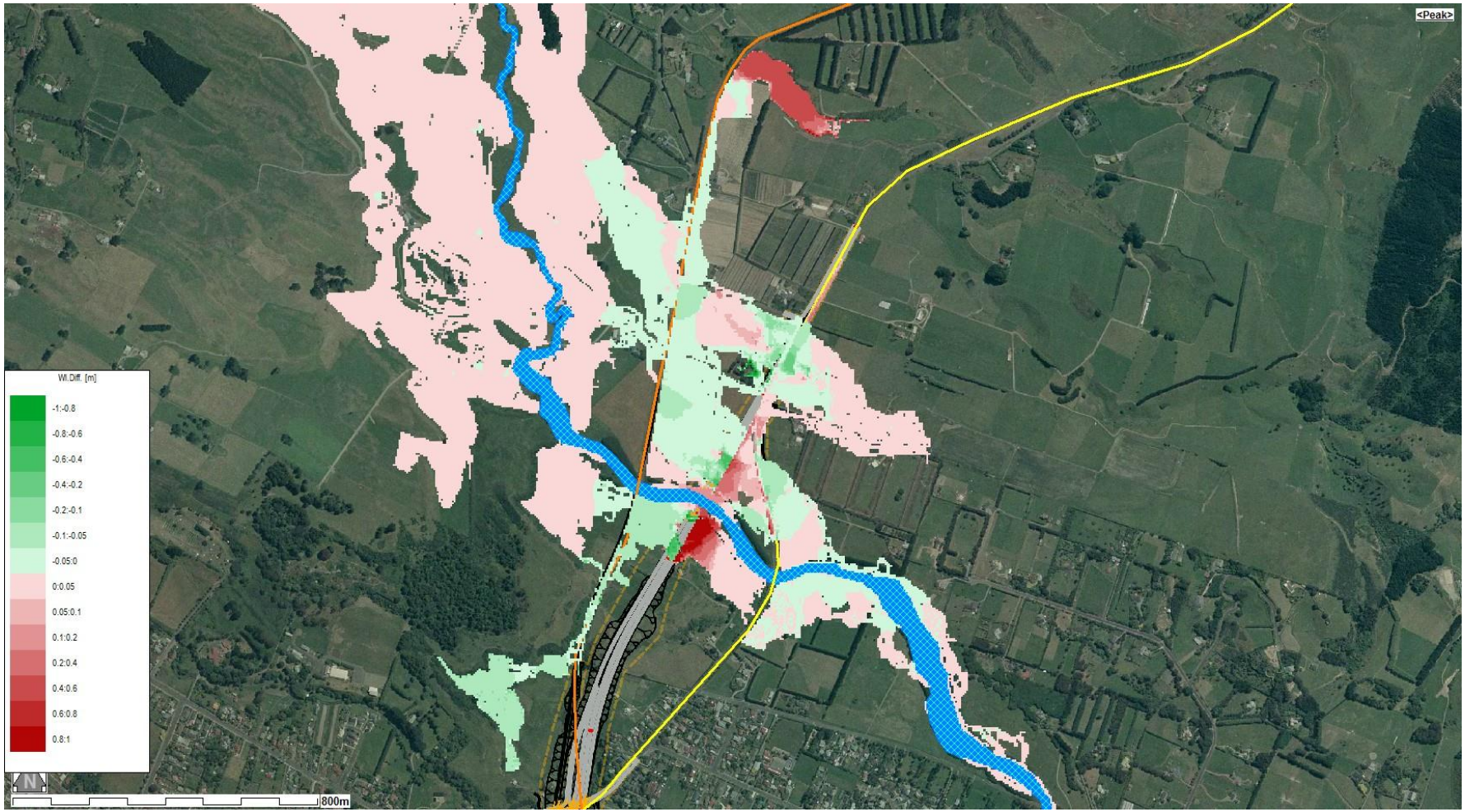

Figure 5-3 Changes in predicted peak flood depths across the Waitohu Stream alluvial fan and floodplain system between proposed and existing situations for 1% AEP flood adjusted for possible future climate change effects to 2090 (pink and red shading indicates areas of increased flow depths, green shading indicates areas of decreased flow depths)

Overall then the Expressway will cause flood levels to increase by up to 0.32m upstream of the bridge crossing in the main stream channel.

Table 5-2 Predicted peak flood levels for existing and proposed situations for flood induced by 2 hour duration design 1% AEP rainfall adjusted for effects of possible future climate change to 2090

Location	Peak Flood Level (m MSL Wellington (1953) datum)		Difference (m)
	Existing Situation	Proposed Situation	
u/s of existing SH1 Bridge	30.62	30.62	0
u/s of dry culvert on southern approach embankment to proposed expressway bridge	25.64	26.48	0.84
u/s of proposed expressway bridge	25.54	25.70	0.16
u/s of dry culvert on northern approach embankment to proposed expressway bridge	25.86	26.34	0.48
u/s of Greenwood sub-catchment culvert	25.15	24.29	-0.86
House 1 – Lot 1 DP 59942	no flood inundation	no flood inundation	-
House 2 – Pt Lot 2 DP59942	no flood inundation	no flood inundation	-
House 3 – Pt Pukehou SL7	23.28	23.19	-0.05

Table 5-3 Estimated pier head losses for Expressway bridge crossing of Waitohu Stream for 1% AEP flood adjusted for effects of possible future climate change to 2090

Flood	Peak Discharge at Bridge (m^3/s)	Peak Flood Level at Bridge (m)	Pier Head Loss (m)	
			Yarnell's Method	Rational Method
1% AEP flood adjusted for climate change to 2090	110 (main channel only)	25.70	0.161	0.069

Except for the localised areas of increased peak flood depths immediately upstream of the Expressway (i.e. within about 100m) seen in Figure 5-3 for the 1% AEP flood adjusted for possible future climate change effects, the rest of the floodplain upstream of these localised areas shows an apparent mix of increased (light pink shading) and decreased (light green shading) peak flood depths. However these correspond to peak flood depth differences of ±0.05m which is within the predictive accuracy of the MIKEFLOOD model across the floodplain. The effect of the Expressway would not be expected to extend this far upstream in any case due to the steepness of the slope of the alluvial fan and floodplain. Therefore it can be reasonably concluded that the Expressway will have no influence on peak flood levels (and hence depths) upstream beyond the localised areas in front of the Expressway embankment. This conclusion also applies to the flood inundation area for the Greenwood sub-catchment upstream of the Expressway culvert (which also has light pink shading in Figure 5-3).

Between the Expressway and the NIMT railway line, Figure 5-3 shows a mix of green and pink shading. In those areas where the shading is either light green of light pink (indicating peak flood depth differences of within ±0.05m) it is again reasonable to say that there is no real difference between the proposed and existing situations as the differences are within the predictive accuracy of the MIKEFLOOD model. However there is one area on the south bank of the Waitohu Stream along the NIMT railway line where the reduction in peak flood

depths is slightly greater (up to 0.1m) which may be more credible. This is probably caused by the sheltering effect the Expressway embankment and the channelling of the bulk of the flood flows in this case through the 75m bridge waterway rather than across the left and right bank floodplains.

Downstream of the NIMT railway line, Figure 5-3 also shows a mix of light green and light pink shading indicating peak flood depth differences are within ±0.05m. As have been argued before, it is reasonable to infer that there is no real difference between the proposed and existing situations over this area of the floodplain as the differences are within the predictive accuracy of the MIKEFLOOD model.

Figure 5-4 shows a predicted backwater profile along the Waitohu Stream for the proposed situation compared to that for the existing situation for the 1% AEP flood adjusted for possible future climate change effects. This indicates that the extent of the backwater effect of the Expressway rapidly tails off and extends no more than about 120m upstream of the bridge crossing.

Upstream of the new Greenwood sub-catchment culvert, Table 5-2 indicates that the peak flood level will be reduced by 0.86m for the 5% AEP flood (adjusted for possible climate change effects). This reduction in peak flood level can also be seen in Figure 5-5 which shows several backwater profiles along the dry channel in the Greenwood sub-catchment upstream of the Expressway culvert in the proposed situation compared to that for the existing situation for the 5% AEP flood (adjusted for possible future climate change effects).

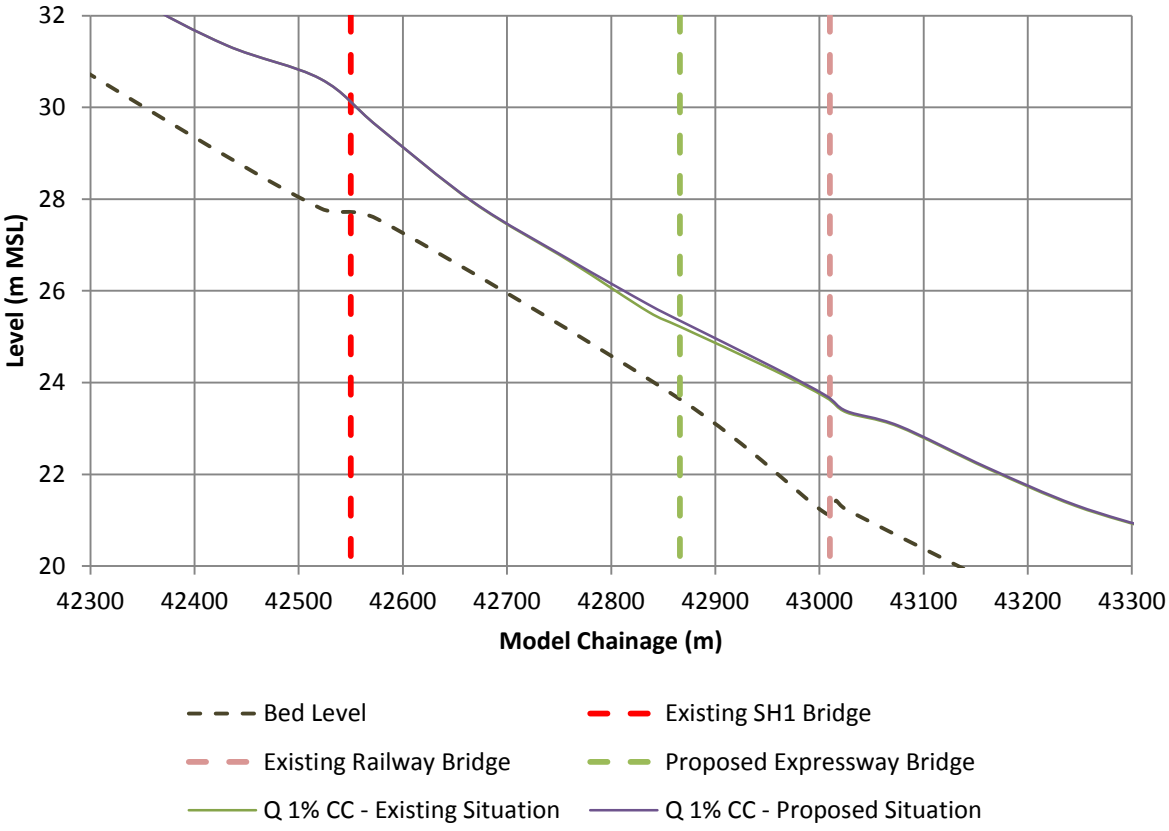

Figure 5-4 Comparison of predicted backwater profiles along Waitohu Stream past Expressway bridge crossing for existing and proposed situations for 1% AEP flood adjusted for the effects of possible future climate change to 2090

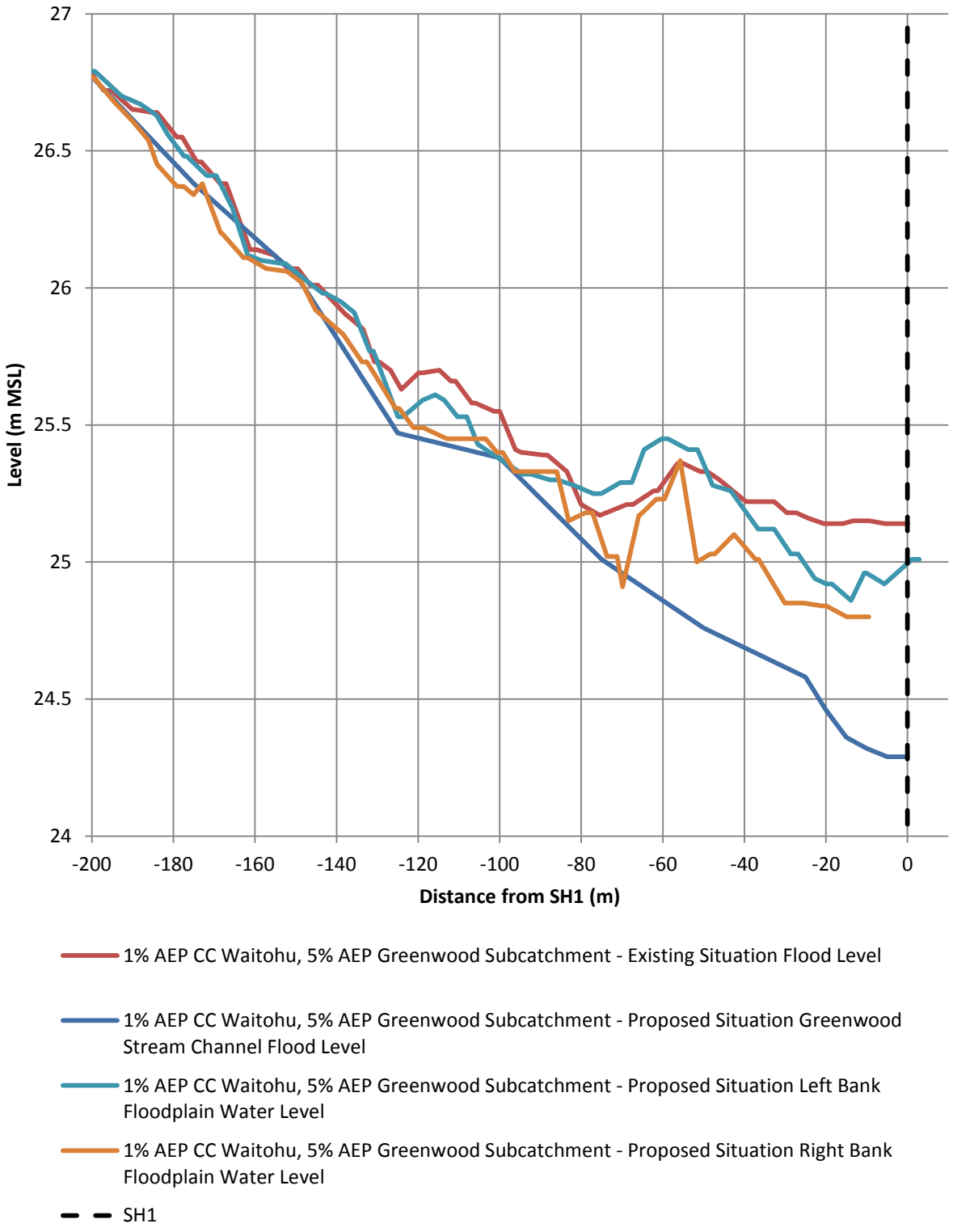

Figure 5-5a Comparison of predicted backwater profiles along dry channel of Greenwood sub-catchment upstream of Expressway culvert for existing and proposed situations for 5% AEP flood adjusted for the effects of possible future climate change to 2090

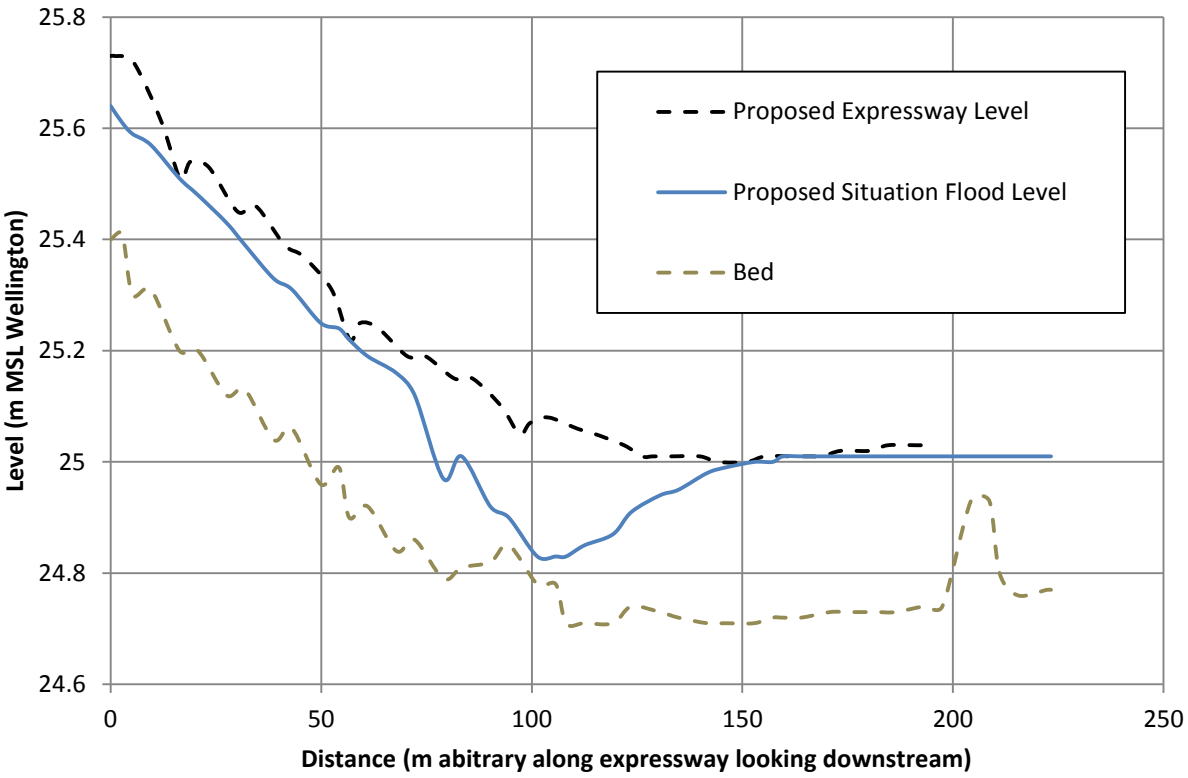

Figure 5-5b Long-section peak water surface profile upstream of Expressway at Greenwood sub-catchment culvert for 5% AEP flood in sub-catchment and 1% AEP flood in Waitohu Stream (both floods adjusted for possible future climate change effects to 2090)

The dry channel in the Greenwood sub-catchment is a highly meandering vee-shaped one. It is difficult for the coarse 5m x 5m grid of the digital terrain model (DTM) for the MIKE21 component of the MIKEFLOOD model to accurately reflect the geometry of this channel. In the proposed situation version of the MIKEFLOOD model therefore, the DTM was linked to a 200m long vee-shaped one-dimensional channel within the MIKE11 component of the model to more accurately represent the actual geometry of the dry channel leading to the 4m x 1.5m dry box culvert under the Expressway. The backwater profile for this one-dimensional channel is therefore much smoother than the corresponding backwater profiles along either bank extracted from the MIKE21 model predictions (the reason for the waviness of these latter profiles is due to the difficulty of tracing a line along the left and right banks exactly parallel to the dry channel forming the main drainage path).

The predicted backwater profile for the existing situation (from the MIKE21 component) is very similar to those along the left and right banks of the dry channel for the proposed situation. This is not surprising as these reflect the shallow overland flow moving slowly across the flat floodplain towards the dry channel. Due to the limited discharge capacity of the twin culvert system in the existing situation, there is unlikely to be the drawdown effect of the dry channel towards the 4m x 1.5m box culvert in the proposed situation. This drawdown effect would extend nor more than about 70m upstream of the culvert in the proposed situation.

To provide a more complete picture of what would happen in this area either side of the Greenwood sub-catchment culvert in the proposed situation, Figure 5-5b shows a long section profile of peak flood levels upstream of the Expressway for a 5% AEP flood in the Greenwood sub-catchment (adjusted for possible future climate change effects) and a 1% AEP flood in the Waitohu Stream (also adjusted for possible future climate change effects). This is a composite profile as the component on the left side reflects the peak flood levels resulting from overland flow out of the Waitohu Stream spreading into the Greenwood sub-catchment while the component on the right side, and at the lowest drawdown point, reflects the peak flood levels from the 5% AEP flood (adjusted for possible future climate change effects) in the Greenwood sub-catchment.

The peak flood levels from the overland flow breaking out of the Waitohu Stream are predicted to be within about 0.1m of the crest level of the Expressway (on the left side of the water surface profile in Figure 5.6b). Allowing for the cross-fall on the road, this means peak flood levels for the 1% AEP flood (adjusted for possible future climate change effects) would be approximately coincident with the shoulder of the road within about 100m south of the Greenwood sub-catchment culvert. Meanwhile peak flood levels on the north side of the Greenwood sub-catchment culvert resulting from the 5% AEP flood (adjusted for possible future climate change effects) in the Greenwood sub-catchment would be just starting to spill over the road.

Two houses are located close to the predicted area of flood inundation upstream of the Greenwood sub-catchment culvert. However Table 5-2 indicates that these houses would not be inundated in either the existing or proposed situations.

There is one additional house on the downstream side of the Expressway along the Greenwood sub-catchment overland flow path. Table 5-2 indicates that there would be a reduction in peak flood depth at this location for the 5% AEP flood adjusted for possible future climate change effects in the Greenwood sub-catchment. However as has been argued previously, this is within the predictive accuracy of the MIKEFLOOD model so that it can only be concluded that the Expressway will not make the existing flood hazard affecting this house any worse.

Figure 5-6a shows predicted peak flow velocities and flow vectors across the Waitohu Stream alluvial fan and floodplain system in the proposed situation for the 1% AEP flood adjusted for possible future climate change effects to 2090 in a similar fashion to the peak flood depths in Figure 5-2. Figure 5-6b shows a zoomed-in part of Figure 5-6a around the existing SH1 and NIMT railway crossings of the Waitohu Stream and floodplain with velocity vectors superimposed to indicate the directions of overland flow paths (similar to Figure 4-6b for the existing situation). Meanwhile Figure 5-7 shows changes in peak flow velocity between the existing and proposed situations.

Figures 5-6a and 5-7 indicate that the Expressway generally causes a reduction in flood flow velocities along the north bank floodplain overflow path relative to the existing situation between SH1 and the NIMT railway line. This is due to the ponding effect of the upstream side of the road embankment and the sheltering effect on the downstream side. The pink areas indicating apparent increases in peak flow velocity, principally downstream of the NIMT railway line are almost certainly an artefact of model inaccuracy and do not reflect real differences. The floodplain area downstream of the railway line is well beyond the influence of the Expressway.

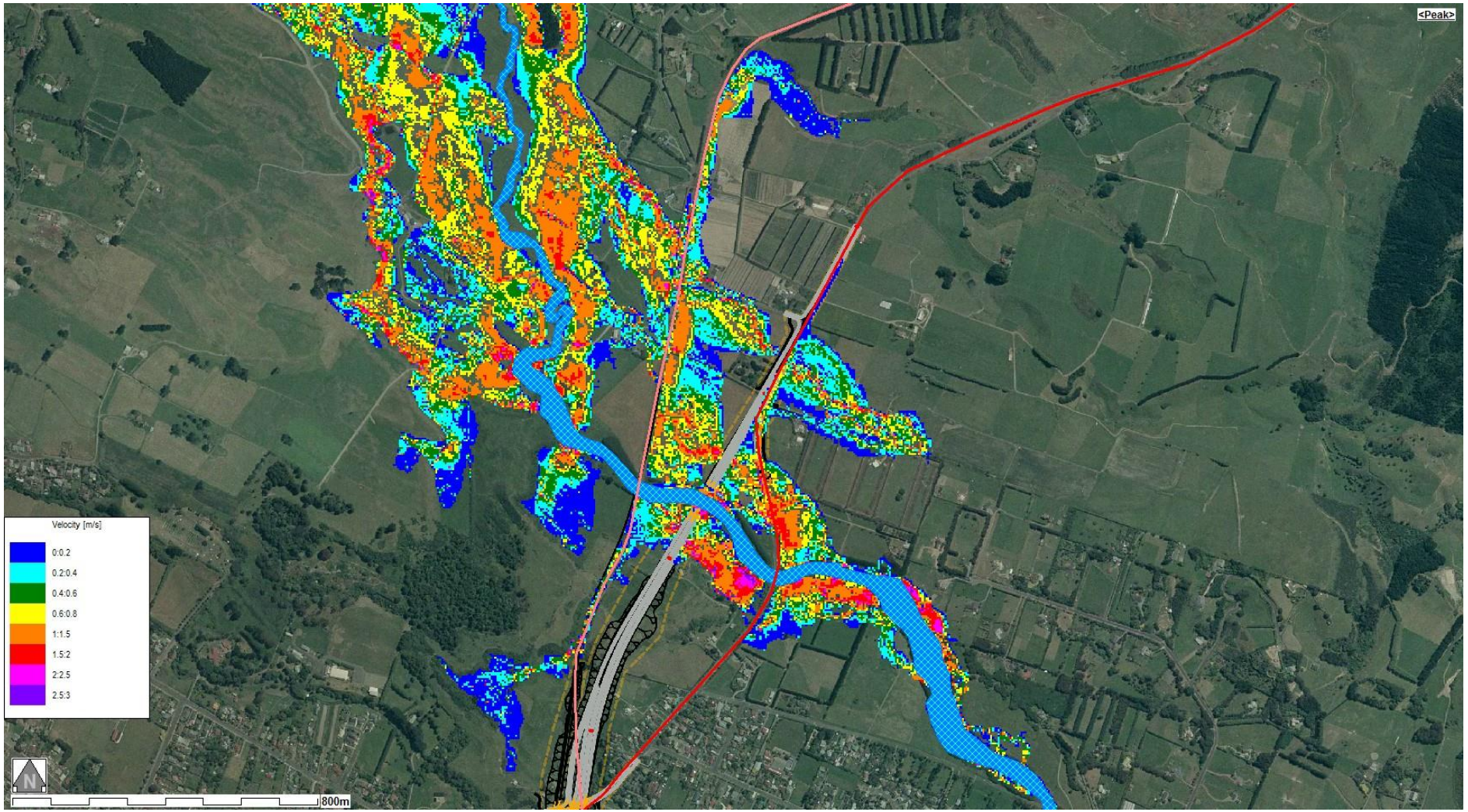

Figure 5-6a Predicted peak flow velocities across the Waitohu Stream alluvial fan and floodplain system in proposed situation for 1% AEP flood adjusted for possible future climate change effects to 2090

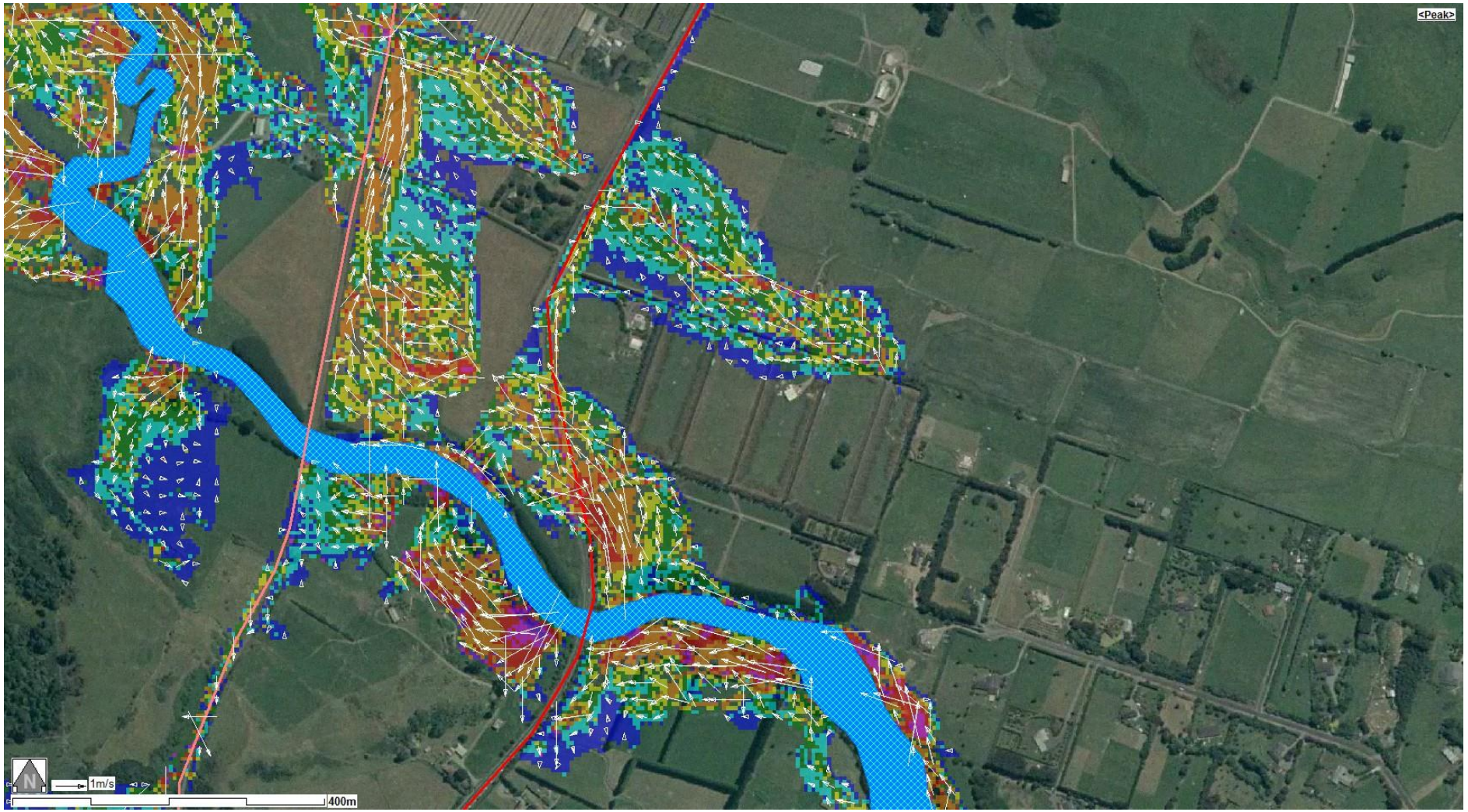

Figure 5-6b Blown-up section of Figure 5-6a showing directions of overland flow paths past the old SH1, new Expressway and NIMT railway crossings of the Waitohu Stream and floodplain

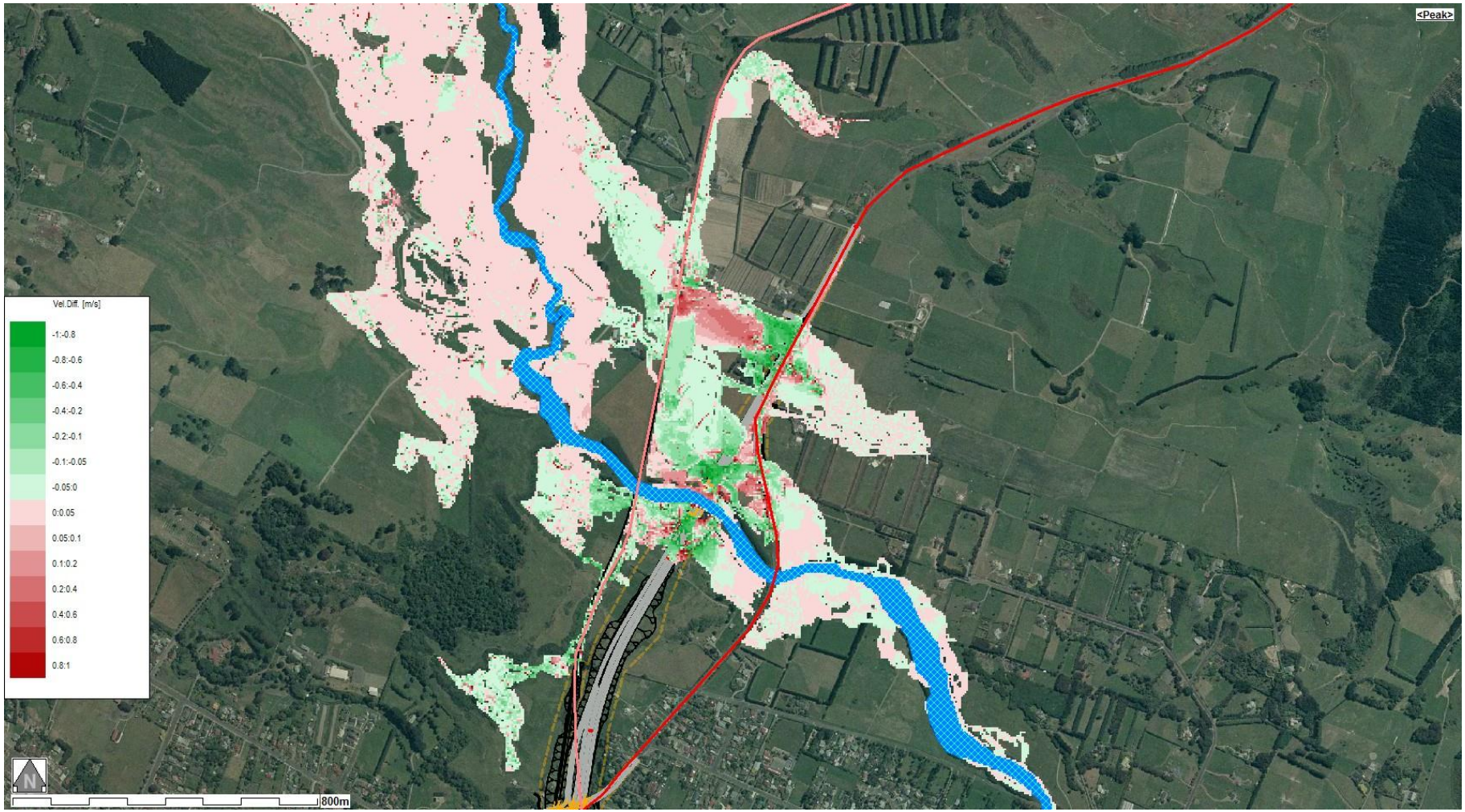

Figure 5-7 Changes in predicted peak flow velocities across the Waitohu Stream alluvial fan and floodplain system between proposed and existing situations for 1% AEP flood adjusted for possible future climate change effects to 2090 (pink and red shading indicates areas of increased flow velocities, green shading indicates areas of decreased flow velocities)

There are two primary areas in Figure 5-7 where increases in peak flow velocity are more significant: in the centre of the triangular area bounded by SH1, the main stream channel and the Expressway, and along the Greenwood sub-catchment overland flow branch. Inspection of Figure 5-6a shows that the peak flow velocities within these areas of increased velocities are generally less than 1m/s, with velocities of up to 1-1.5m/s along the narrow course of the primary drainage channel for the Greenwood sub-catchment. Short duration peak flow velocities of this magnitude do not generally pose an erosion threat to grassed floodplain surfaces (Hewlett *et al*, 1985).

Figure 5-8 compares flood discharge hydrographs for the proposed and existing situations for the left bank floodplain overflow path while Figure 5-9 compares flood discharge hydrographs for the proposed and existing situations for the right bank floodplain overflow path[11]. The Expressway embankment with its 75m wide bridge waterway will force a redistribution of flood flows across the floodplain so that the proposed and existing flood discharge hydrographs in these figures may not be directly comparable. However what they do demonstrate is the large flow volume conveyed by these left and right bank overflow paths in both situations.

Figure 5-10 shows a flood discharge hydrograph for the Greenwood sub-catchment culvert similar to Figure 4-7. With the 4m wide by 1.5m high box culvert in the proposed situation, the volume of flow passed increases to a peak of 7.8m^3/s (compared to 4.15m^3/s for the existing twin culvert system) without the road being overtopped.

As in the existing situation, the double-peaked nature of the culvert discharge hydrograph at this location is due to the Greenwood sub-catchment runoff contribution (first peak) arriving before the contribution from breakout flows along the right bank of the Waitohu Stream upstream of the existing SH1 bridge.

[11] The flood hydrographs for the proposed situation in Figures 5-8 and 5-9 have been deliberately truncated as they simply merge over the top of the corresponding hydrographs for the existing situation.

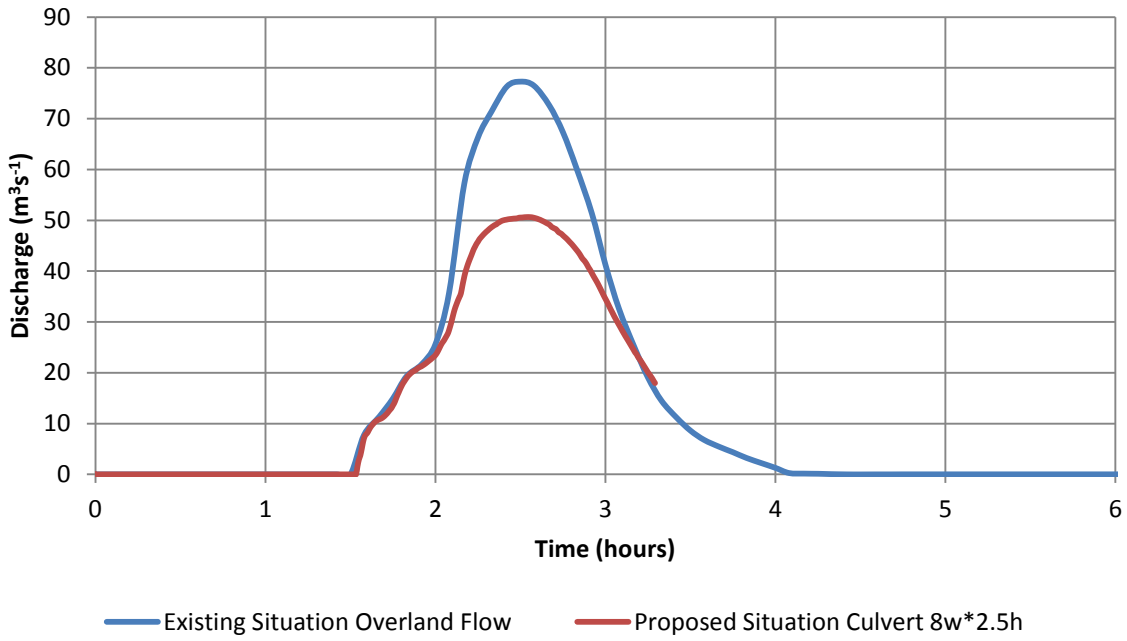

Figure 5-8 Comparison of flood hydrographs for left (south) bank overland flow path downstream of SH1 bridge for proposed and existing situations for 1% AEP flood adjusted for possible future climate change effects to 2090

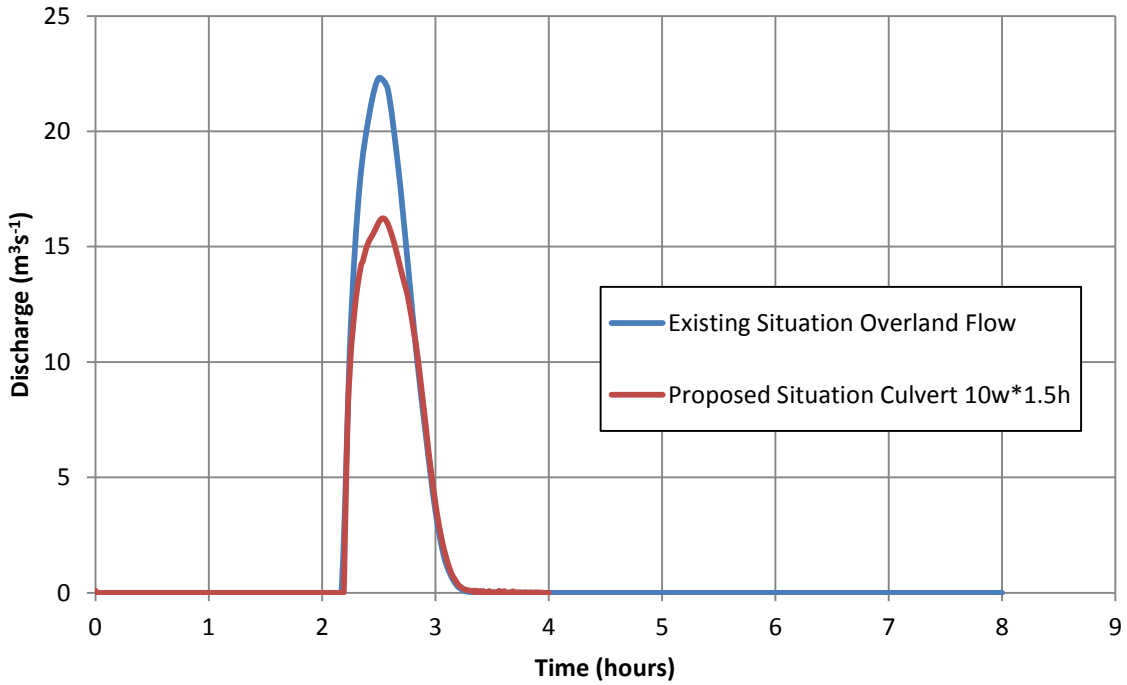

Figure 5-9 Comparison of flood hydrographs for right (north) bank overland flow path downstream of SH1 bridge for proposed and existing situations for 1% AEP flood adjusted for possible future climate change effects to 2090

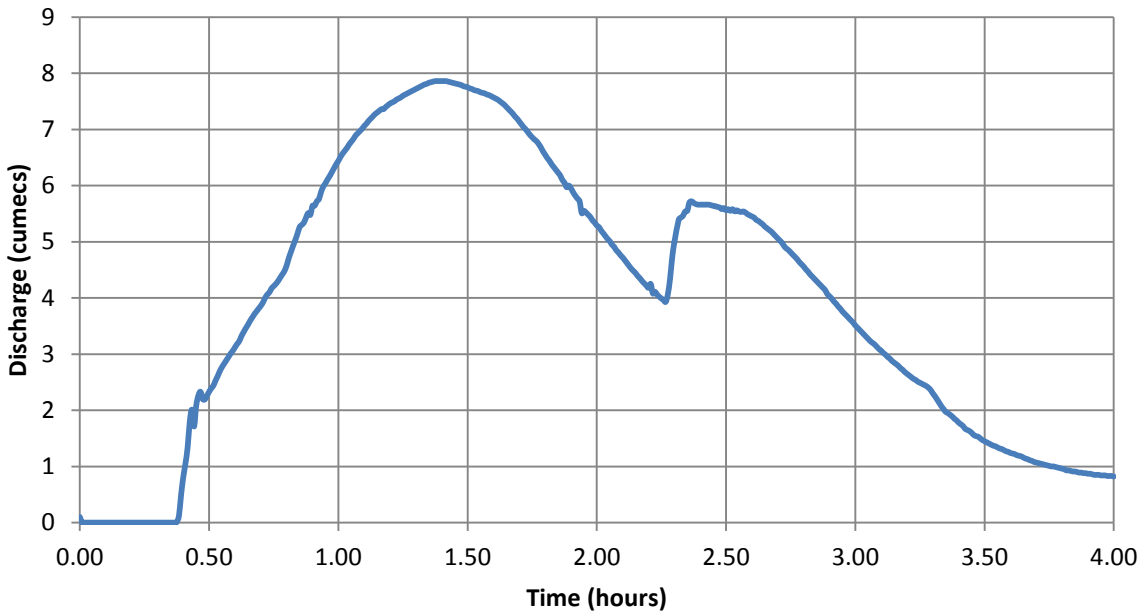

Figure 5-10 Flood hydrograph for Greenwood sub-catchment overland flow path for proposed situation for 5% AEP flood adjusted for possible future climate change effects to 2090

5.3 Sensitivity of Predicted Peak Flood Levels to Flood Magnitude

A couple of sensitivity tests were carried out to check the sensitivity of peak flood levels to flood magnitude. These are outlined as follows:

- A 0.5% AEP flood in the Waitohu Stream and a 5% AEP flood in the Greenwood sub-catchment both generated by a 2 hour duration rainstorm as per Wallace (2004) (with adjustments for the effects of possible future climate change to 2090)
- A 1% AEP flood generated by a 2 hour duration rainstorm in both the Waitohu Stream and the Greenwood sub-catchment (with adjustments for the effects of possible future climate change to 2090)

The first sensitivity test checks the sensitivity of the Expressway crossing design to flood magnitude and keeps the runoff contribution from the Greenwood sub-catchment unchanged from the base case (this is reasonable as the Greenwood sub-catchment runoff contribution only affects flood levels for the corresponding overland flow path). It is also consistent with the assumption of the 2004 flood hazard assessment carried out for the Waitohu Stream and floodplain by GWRC (Wallace, 2004).

The second sensitivity test checks the sensitivity of the Greenwood culvert design to a larger flood which corresponds to the recommended design flood standard in Section 3. From Figure 5-10, the surface runoff contribution from the Greenwood sub-catchment dominates over the contribution from cross-country breakout flows from the Waitohu Stream which lag the former. Therefore this second sensitivity test is a reasonable one for checking the culvert sizing and determining the level of service for the Expressway.

No sensitivity test for the Greenwood culvert design has been carried out to test for the effects of coincidence of the peak of the cross-country breakout flows from the Waitohu Stream with the peak of the surface contribution from Greenwood sub-catchment. This is a physically unrealistic scenario as it is based on the premise that the larger steep upland component of the Waitohu Catchment responds to an extreme rainfall event before the much smaller Greenwood sub-catchment.

Table 5-4 compares predicted peak flood levels for both sensitivity tests compared to the base case of the 1% AEP flood adjusted for the effects of possible future climate change to 2090 in the Waitohu Stream and the 5% AEP flood also adjusted for possible climate change effects in the Greenwood sub-catchment.

Table 5-4 indicates that increasing the total inflow past the expressway by 25m^3/s (i.e. from 217m^3/s to 242m^3/s – Table 2-3) increases peak flood levels upstream of Expressway embankment across the Waitohu Stream alluvial fan and floodplain as follows:

- by 0.16m upstream of the 8m x 2.5m dry culvert on the south (left) bank;
- by 0.08m on the main stream channel (excluding the effect of bridge pier head losses); and
- by 0.19m upstream of the 10m x 1.5m dry culvert on the north (right) bank.

There is no change in peak flood levels at the Greenwood sub-catchment culvert as this is determined exclusively by the volume of sub-catchment runoff rather than breakout flows from the main stream channel.

As noted in Section 5.2, the effect of the bridge piers is to increase flood levels slightly in the main stream channel. For the 0.5% AEP flood adjusted for the effects of possible future climate change, the bridge pier head losses are estimated to be in the range of 0.08-0.18m (giving a total estimated peak flood level of 25.86-25.96m MSL Wellington datum) compared to a range of 0.07-0.16m (giving a total estimated peak flood level of 25.77-25.86m MSL Wellington datum) for the 1% AEP flood adjusted for possible climate change effects. The relative increase in main stream channel peak flood level including the effect of bridge pier head losses due to the increase in flood magnitude could therefore be about 0.10m.

Table 5-4 Predicted peak flood levels for proposed Expressway crossing of Waitohu Stream and floodplain for different flood magnitudes adjusted for effects of possible future climate change to 2090

Location	Peak Flood Level (m MSL Wellington (1953) datum)		
	1% AEP Flood _Waitohu Stream 5% AEP flood Greenwood sub-catchment	1% AEP Flood _Waitohu Stream 1% AEP flood Greenwood sub-catchment	0.5% AEP Flood _Waitohu Stream 5% AEP flood Greenwood sub-catchment
u/s of existing SH1 Bridge	30.62	30.62	30.78
u/s of dry culvert on southern approach embankment to proposed expressway bridge	26.48	26.48	26.55
u/s of proposed expressway bridge	25.70	25.70	25.78
u/s of dry culvert on northern approach embankment to proposed expressway bridge	26.34	26.37	26.53
u/s of Greenwood sub-catchment culvert	24.29	24.70	24.35
House 1 – Lot 1 DP 59942	no flood inundation	no flood inundation	-
House 2 – Pt Lot 2 DP59942	no flood inundation	no flood inundation	-
House 3 – Pt Pukehou SL7	23.19	23.23	23.24

Table 5-5 Estimated pier head losses for Expressway bridge crossing of Waitohu Stream for 0.5% AEP flood adjusted for effects of possible future climate change to 2090

Flood	Peak Discharge at Bridge (m^3/s)	Peak Flood Level at Bridge (m)	Pier Head Loss (m)	
			Yarnell's Method	Rational Method
0.5% AEP flood adjusted for climate change to 2090	120 (main channel only)	25.78	0.180	0.076

Figure 5-11 shows the change in peak flood depth relative to the existing situation across the Waitohu Stream alluvial fan and floodplain system for the 0.5% AEP flood adjusted for the effects of possible future climate change. This shows a very similar pattern to the changes in peak flood depth for the 1% AEP flood adjusted for the effects of possible future climate change in Figure 5-3. The same conclusions arrived at with respect to the effects of the Expressway for the 1% AEP flood are therefore also valid for the 0.5% AEP flood.

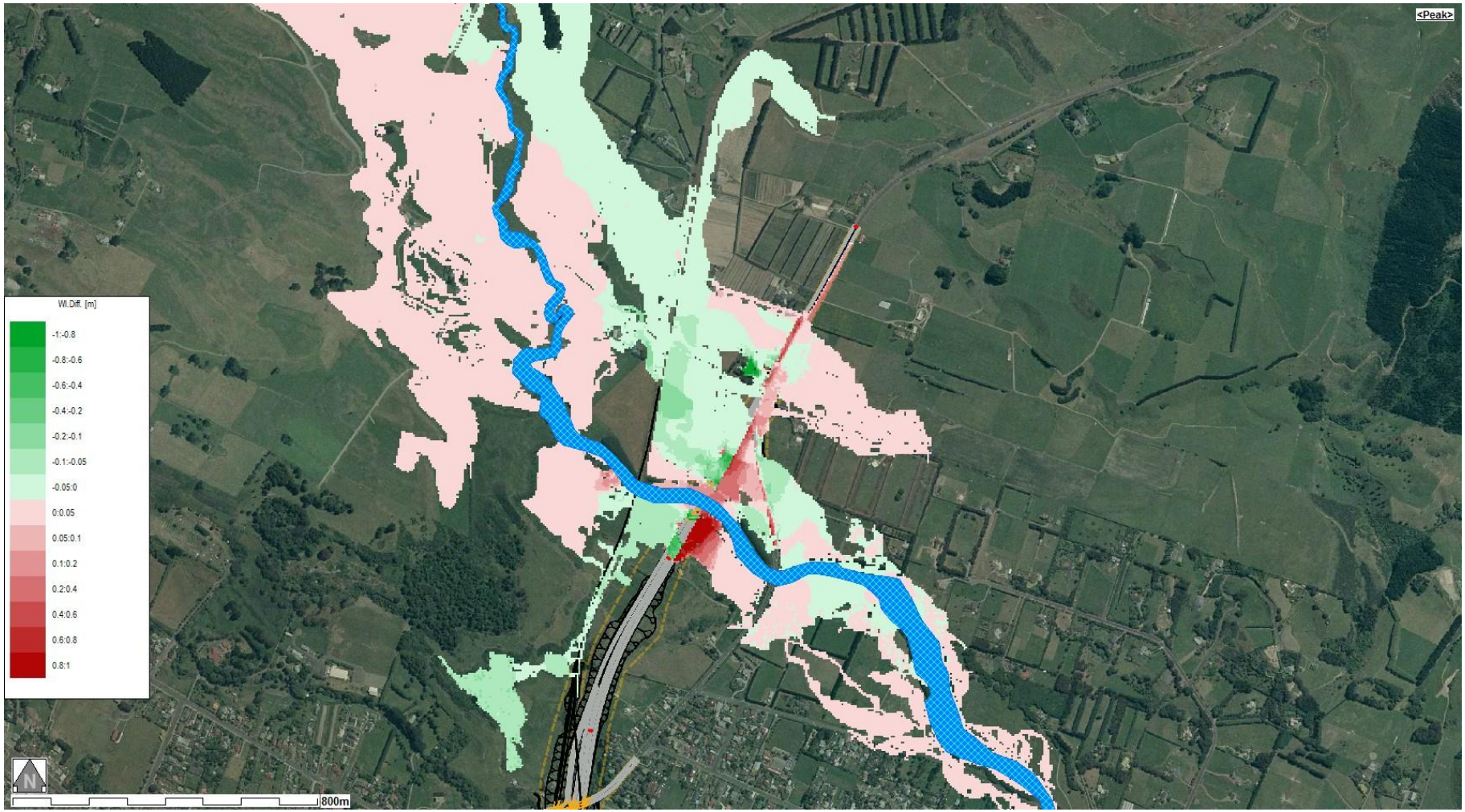

Figure 5-11 Changes in predicted peak flood depths across the Waitohu Stream alluvial fan and floodplain system between proposed and existing situations for 0.5% AEP flood adjusted for possible future climate change effects to 2090 (pink shading indicates areas of increased flow depths, green shading indicates areas of decreased flow depths)

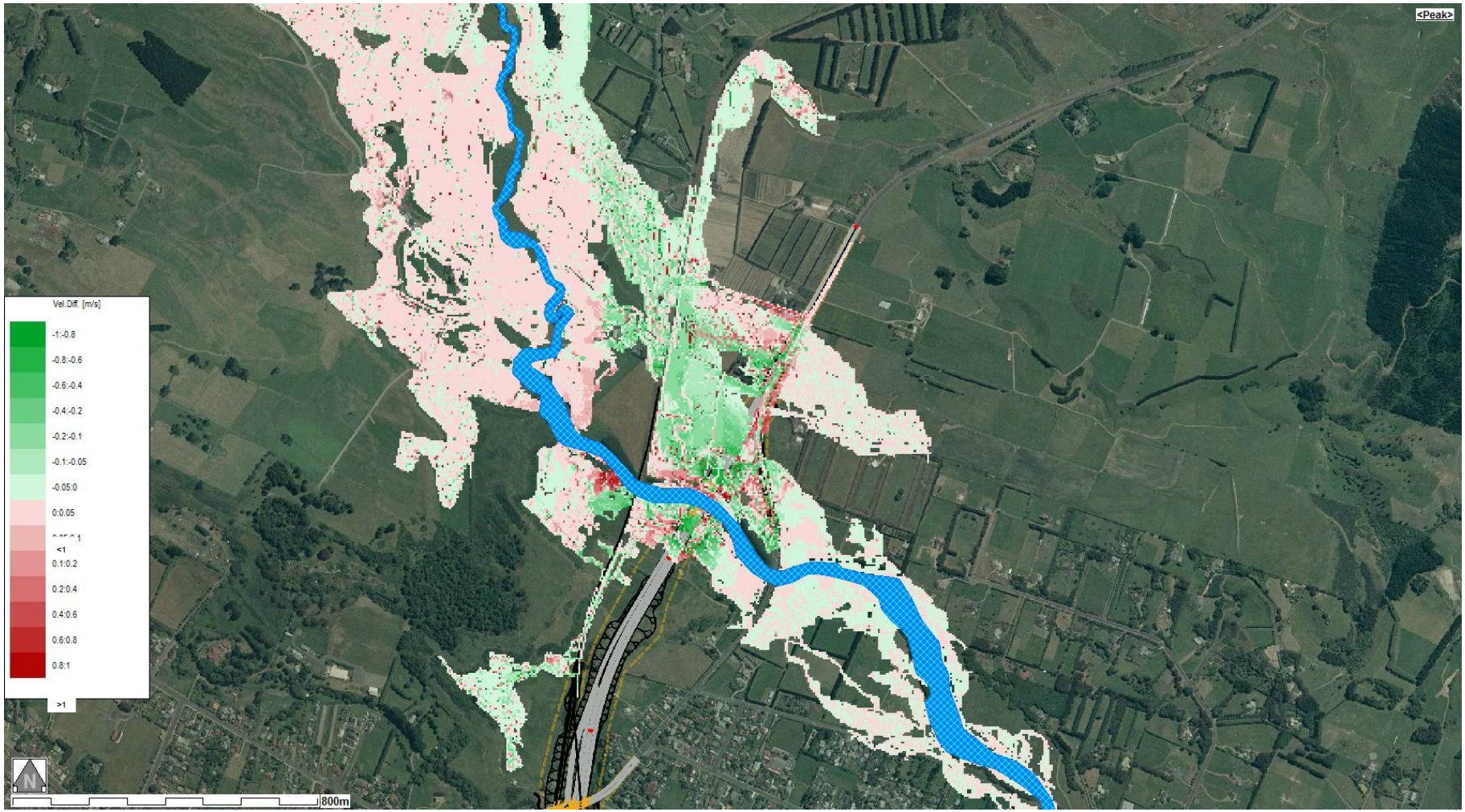

Figure 5-12 Changes in predicted peak flow velocities across the Waitohu Stream alluvial fan and floodplain system between proposed and existing situations for 0.5% AEP flood adjusted for possible future climate change effects to 2090 (pink shading indicates areas of increased flow velocities, green shading indicates areas of decreased flow velocities)

These conclusions are that:

- the effects of the Expressway in the form of increased peak flood depths principally occur over a distance of less than 100m upstream of where the approach embankments to the bridge crossing of the Waitohu Stream and floodplain are elevated above the floodplain;
- there may be a minor reduction in peak flood levels in the lee of both approach embankments to the Expressway bridge crossing as far downstream as the NIMT railway line; but
- elsewhere across the floodplain, any apparent differences between the proposed and existing situations are within the predictive accuracy of the model (± 0.05m) so that the Expressway has no effect on flood inundation patterns and levels.

Figure 5-12 shows the change in peak flow velocity relative to the existing situation across the Waitohu Stream alluvial fan and floodplain for the 0.5% AEP flood adjusted for the effects of possible future climate change. This shows a similar pattern to the changes in peak flood depth for the 1% AEP flood adjusted for the effects of possible future climate change in Figure 5-7. In the 0.5% AEP flood there appears to be a more marked reduction in peak flow velocities in the lee of the northern (right bank) approach embankment to the Expressway bridge crossing. This suggests that the Expressway bridge approach embankments cause a slight redistribution of flow between the main stream channel and the right bank overflow path with a slightly greater volume of flow being forced through the 75m wide waterway of the Expressway bridge crossing.

With respect to the second sensitivity test, Figure 5-13 shows a flood inundation map for a 1% AEP flood in the Greenwood sub-catchment in combination with a 1% AEP flood in the Waitohu Stream (both floods adjusted for possible future climate change effects). The flood inundation pattern at the Greenwood sub-catchment culvert is the primary aspect of interest. Figure 5-13 indicates that the effect of the Greenwood sub-catchment runoff increasing from a peak value of 8.9m^3/s to 12.3m^3/s (Table 2-3) is to cause the Expressway to be overtopped over about a 70m width on the north side of the culvert. Because of the width of the overflow and the small magnitude of the flow volume, the depth of overtopping would be very shallow.

Figure 5-14 shows a long-section peak flood level profile along the upstream side of the Expressway for the second sensitivity test with a 1% AEP flood in the Greenwood sub-catchment in combination with a 1% AEP flood in the Waitohu Stream (both floods adjusted for possible future climate change effects). This complements Figure 5-13 in demonstrating the overtopping of SH1 to the north of the Greenwood sub-catchment culvert by a 1% AEP flood in that sub-catchment.

Table 5-4 indicates that a 1% AEP flood adjusted for the effects of possible future climate change to 2090 would cause the peak flood level upstream of the new Greenwood sub-catchment culvert to increase by 0.04m relative to the peak level for the 5% AEP flood adjusted for possible future climate change effects.

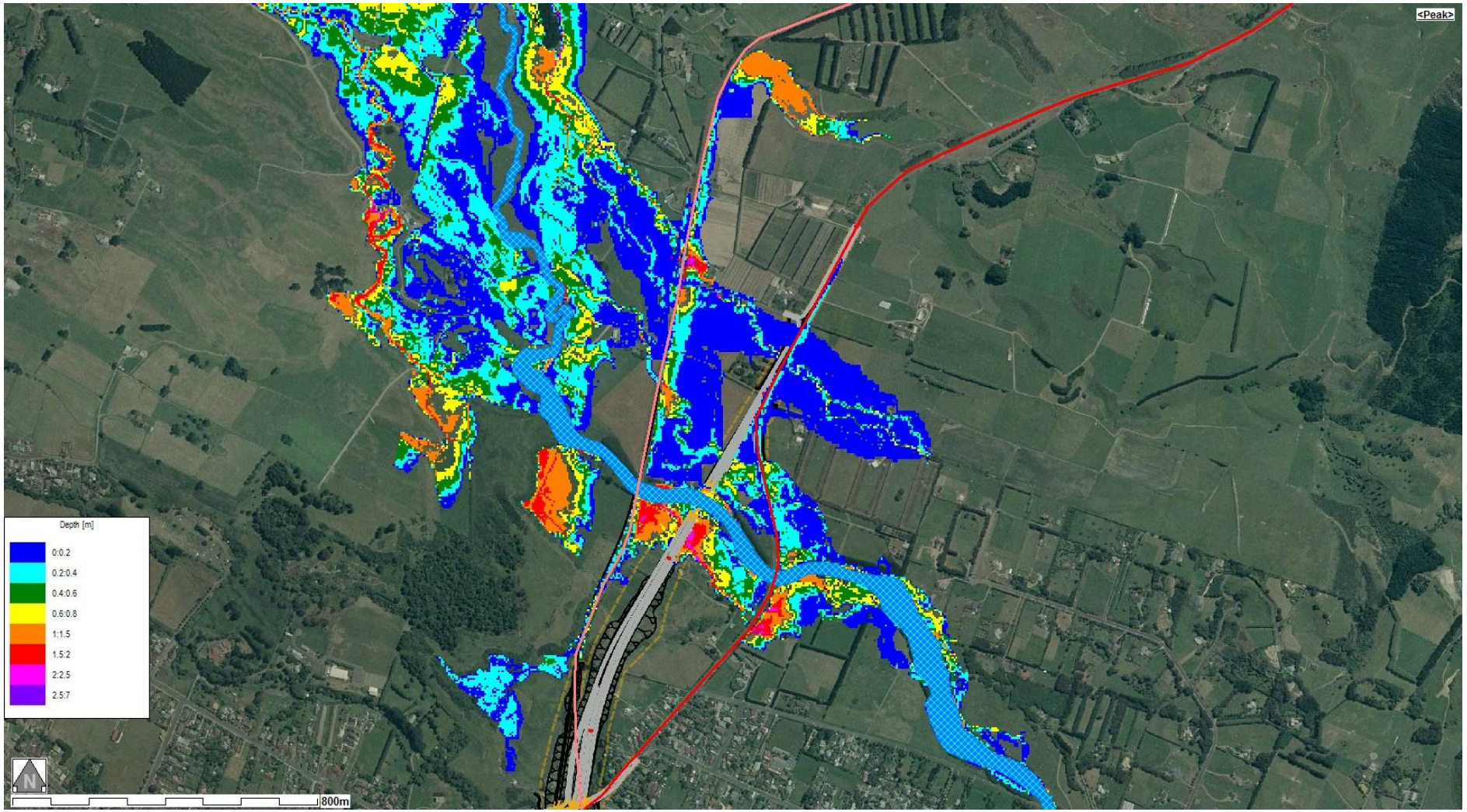

Figure 5-13 Predicted peak flood depths across the Waitohu Stream alluvial fan and floodplain system in proposed situation for 1% AEP flood adjusted for possible future climate change effects to 2090 in both Greenwood sub-catchment and Waitohu Stream

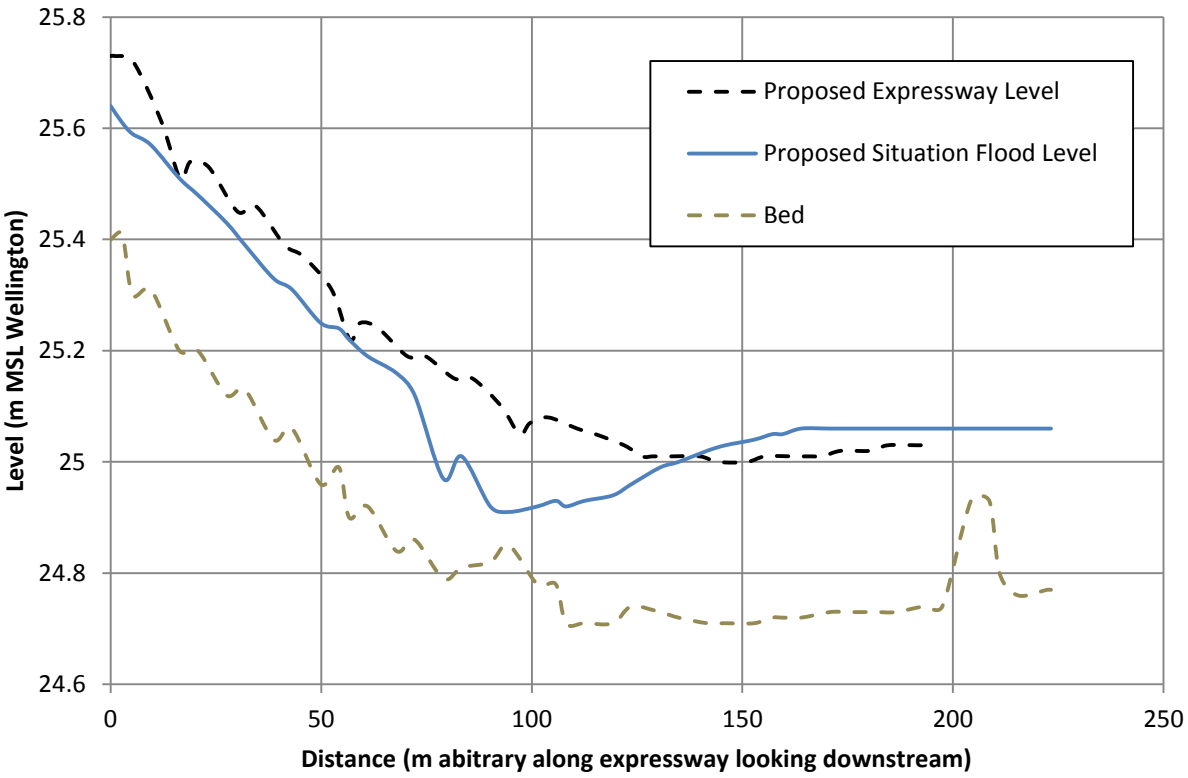

Figure 5-14 Long-section peak water surface profile upstream of Expressway at Greenwood sub-catchment culvert for 1% AEP flood in sub-catchment and 1% AEP flood in Waitohu Stream (both floods adjusted for possible future climate change effects to 2090)

5.4 Sensitivity of Predicted Flood Levels to Replacement of SH1 Bridge

In view of the longevity of the life of the proposed Expressway bridge structure, it is conceivable that the existing SH1 bridge could reach the end of its serviceable life and be replaced. The form and dimensions of any new bridge are unknown but it is reasonable to expect that the design flood and freeboard standards for the replacement structure would be carefully reviewed. The constrained waterway capacity of the existing SH1 bridge appears to contribute to the breakout of floodwaters along both banks upstream of the bridge so that a conservative approach to assessing the effects of bridge replacement would be to assume that the bridge was completely removed. A further sensitivity test was therefore carried out based on this assumption.

Figure 5-15 shows the predicted flood inundation pattern for the 1% AEP flood adjusted for possible future climate change effects to 2090 in both the main Waitohu Catchment and the Greenwood sub-catchment with the existing SH1 bridge removed. Table 5-6 gives the predicted peak flood levels upstream of the bridge and culvert locations for this scenario compared to the corresponding peak flood levels for the same flood case but with the existing SH1 bridge in place. Table 5-7 gives the predicted pier head loss values for this flood scenario.

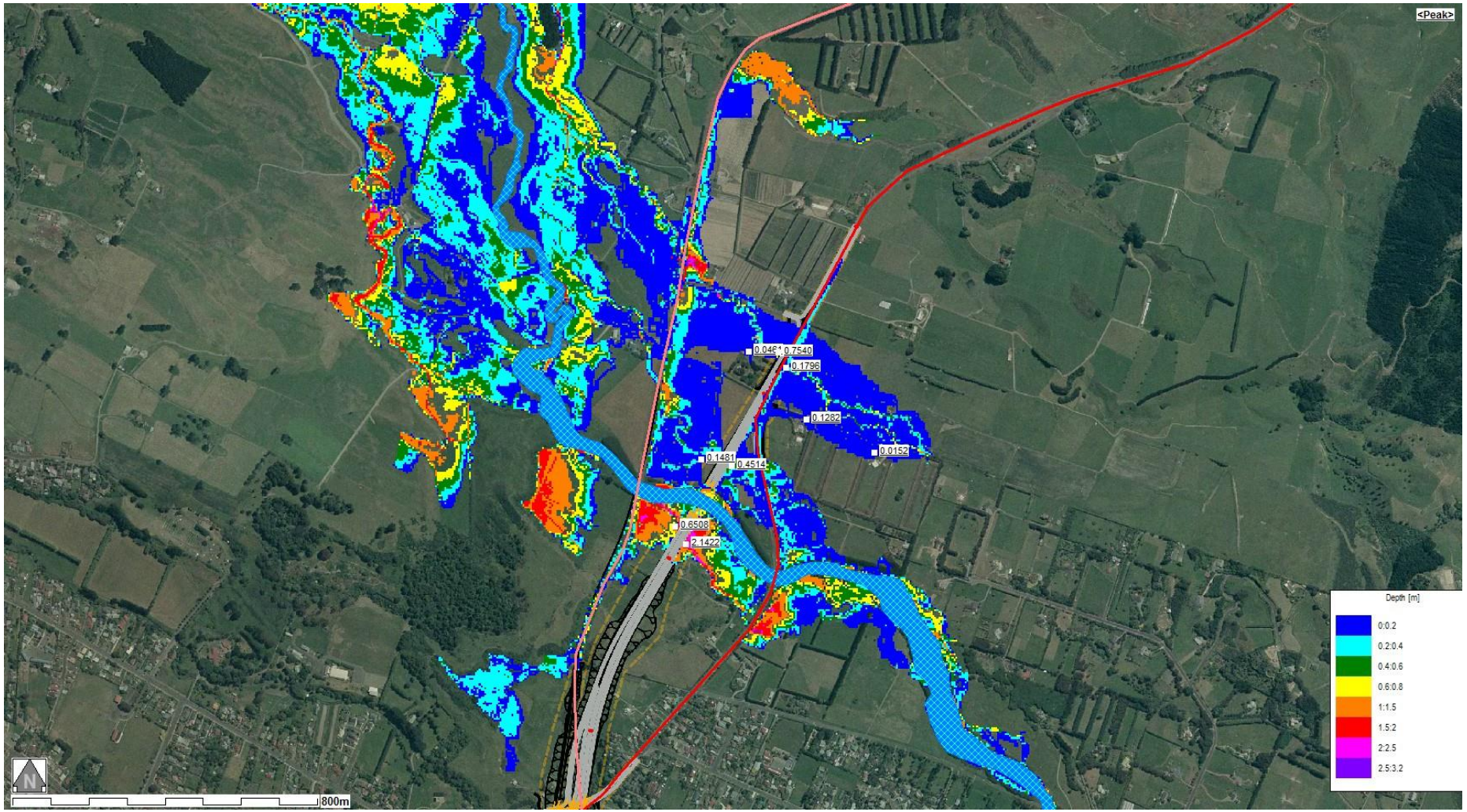

Figure 5-15 Predicted peak flood depths across the Waitohu Stream alluvial fan and floodplain system in proposed situation for 1% AEP flood adjusted for possible future climate change effects to 2090 in both Greenwood sub-catchment and Waitohu Stream with existing SH1 bridge removed

Table 5-6 Predicted peak flood levels for proposed Expressway crossing of Waitohu Stream and floodplain with and without existing upstream SH1 bridge in place (1% AEP flood adjusted for possible climate change effects to 2090 in both Waitohu Stream and Greenwood sub-catchment)

Location	Peak Flood Level (m MSL Wellington (1953) datum)	
	1% AEP Flood _Waitohu Stream 1% AEP flood Greenwood sub-catchment with existing SH1 bridge in place	1% AEP Flood _Waitohu Stream 1% AEP flood Greenwood sub-catchment with existing SH1 bridge removed
u/s of existing SH1 Bridge	30.62	30.38
u/s of dry culvert on southern approach embankment to proposed expressway bridge	26.48	26.48
u/s of proposed expressway bridge	25.70	25.73
u/s of dry culvert on northern approach embankment to proposed expressway bridge	26.37	26.00
u/s of Greenwood sub-catchment culvert	24.70	24.70

Table 5-7 Estimated pier head losses for Expressway bridge crossing of Waitohu Stream with and without existing upstream SH1 bridge in place (1% AEP flood adjusted for possible climate change effects to 2090 in both Waitohu Stream and Greenwood sub-catchment)

Flood	Peak Discharge at Bridge (m^3/s)	Peak Flood Level at Bridge (m)	Pier Head Loss (m)	
			Yarnell's Method	Rational Method
With existing SH1 bridge in place	110 (main channel only)	25.70	0.161	0.069
with existing SH1 bridge removed	116 (main channel only)	25.73	0.183	0.077

The pattern of flood inundation indicated by Figure 5-15 is very similar to that shown in Figure 5-2 for the same flood case but with the existing SH1 bridge in place. There are slight differences in peak flow depths on the right bank floodplain upstream of the Expressway which are verified by the reduced peak flood level upstream of the northern (right bank) approach embankment to the Expressway bridge (26.00m compared to 26.37m MSL Wellington datum for base flood case with the SH1 bridge in place). There also appear to be very slight differences in peak flood depth on the left bank floodplain upstream of the Expressway although the peak flood level upstream of the southern (left bank) approach embankment culvert is unchanged.

Table 5-6 indicates that the main stream channel flow volume is increased marginally from 110 to 116m^3/s and that bridge pier head losses resulting from this flow would be in the range of 0.08-0.18m. The pier head losses imply a total peak flood level upstream of the Expressway bridge of 25.81-25.91m (MSL Wellington datum) which compares with an estimated range of 25.77-25.86m for the same flood case with the SH1 bridge

in place. This implies an estimated increase in water level of about 0.05m if the upstream SH1 bridge is removed.

In summary then, removing (or replacing) the upstream SH1 bridge does not have any effect on the overall flood inundation pattern. It does appear to allow a greater flow volume to be conveyed by the main stream channel and the left bank floodplain downstream of the SH1 route. However the resulting increase in peak flood level at the Expressway bridge is only in the order of about 0.05m. Right bank floodplain levels upstream of the Expressway are reduced while left bank flood levels upstream of the Expressway may be very slightly higher.

5.5 Consideration of Other Effects

5.5.1 Partial Culvert Blockage

In order for a partial blockage of a culvert in a watercourse by flood-transported woody debris to occur, the following criteria need to be satisfied:

- there has to be an abundant supply of woody debris material available to be transported;
- the flow depths need to be deep enough to transport the available woody debris material; and
- the culvert structure needs to incorporate a number of narrow cells or pipes that would make it prone to blockage.

In this particular context, the first of these criteria would not appear to be relevant while the last two criteria would not appear to be satisfied with respect to the dry culverts through the Expressway embankment. The two culverts are distant from the main stream channel. While the Waitohu Stream is capable of transporting large woody debris material under flood conditions, either flushed out of its upper catchment or sourced from eroded willow trees lining the stream banks, the breakout flow depths across the floodplain are not particularly deep and would be insufficient to transport large tree branches from the main stream channel to the dry culverts to form a partial blockage. The floodplain itself is mainly used for pastoral purposes with trees forming shelterbelts and hedges but again the breakout flow depths would be insufficient to transport any wind-thrown woody material from this alternative source to also block the culverts. The proposed dry culverts through the Expressway embankment are wide and would need to be formed as two or three cell structures which would not be prone to blockage by other detritus conveyed by floodwaters flowing overland.

The potential for partial blockage by flood-transported woody debris and other detritus of the two dry culverts through the proposed expressway embankment is therefore considered to be negligible. If partial blockage of either of these culverts did by some remote chance ever occur, the effects due to increased peak flood levels would be very localised as indicated by the change in peak flood depths between the proposed and existing situations in Figures 5-3 and 5-10.

In contrast the existing Greenwood sub-catchment culvert system consisting of one 0.9m diameter culvert and one 1.05m diameter culvert would be much more prone to blockage by small flood-transported woody debris and other detritus. As noted previously, this culvert system is under capacity relative to the magnitude of the sub-catchment flood estimates so that it requires upgrading to a much larger size in order to meet the

required design flood standard. Being larger, the proposed 4m wide x 1.5m high replacement box culvert would be less prone to a partial blockage under flood conditions. The culvert would probably be formed from one 4.0m by 1.5m box cell unit.

5.5.2 Sediment Deposition on Floodplain

The overall extent of flood inundation across the alluvial fan and floodplain system predicted for the proposed situation is broadly similar to that for the existing situation.

Breakout of sediment-laden floodwaters onto the floodplain upstream of the existing SH1 bridge is influenced by the restricted waterway capacity of the bridge and the low bank levels upstream. As the existing SH1 bridge and stream channel geometry will be unchanged by the Expressway, the potential for sediment deposition on the floodplain upstream of the existing SH1will remain much the same as at present. Even if the present SH1 bridge is replaced in the future by a new structure, existing floodplain flow patterns will be largely unchanged as discussed in Section 5.4 so that the potential for sediment deposition on the floodplain will continue to be much the same.

Upstream of the Expressway, the road embankment across the floodplain will cause a slight redistribution of floodplain flows with slightly reduced floodplain flow volumes and slightly increased main stream channel flow volumes. Consequently it is expected that the potential for sediment deposition on the floodplain would be slightly reduced.

In summary then, the proposed situation will not be significantly different from the existing situation with respect to the sedimentation risk.

5.5.3 Erosion of Floodplain Surfaces Due to High Flow Velocities

Figures 5-7 and 5-12 demonstrate that the Expressway with the large dry culverts through the approach embankments to the bridge crossing of the Waitohu Stream providing continuity for existing overland flow paths does not in general increase peak flow velocities across the floodplain under flood conditions.

The magnitude and duration of the absolute peak flow velocities shown in Figures 5-6a and b for the design flood are within the range that grassed surfaces can tolerate (Hewlett *et al*, 1985). Any surface erosion that did occur in a major flood event, particularly around culvert entrances and exits, would be localised and easily repairable.

Therefore erosion of the floodplain surface by high flow velocities is not an issue with the Expressway.

5.5.4 Drainage Times for Ponding Areas

Figures 5-8 and 5-9 indicate that floodplain drainage times upstream of the Expressway embankment will be unaffected compared to the existing situation. Furthermore, comparison of Figure 5-10 with Figure 4-7 indicates that drainage times upstream of the Greenwood sub-catchment culvert under the Expressway will also be unchanged from the existing situation.

Comparison of other flood hydrographs for the existing and proposed situations for additional locations upstream of SH1 on the south side of the Waitohu Stream, upstream of the Expressway embankment on both sides of the Waitohu Stream and upstream of the SH1 Greenwood Branch culvert confirm this.

Therefore extended floodplain drainage times will also not be an issue for the Expressway.

5.6 Design Recommendations for Bridge Soffit Levels and Minimum Road Embankment Shoulder Levels

Based on the flood level predictions for the Serviceability Limit State Flood given in Sections 5.2 to 5.4 and the design freeboard requirements outlined in Section 3, revised design levels have been established for the soffit level of the proposed Expressway bridge and the shoulder levels of the bridge approach embankments at each of the culvert locations. These are summarised in Table 5-8 below.

Table 5-6 Revised design levels for proposed Expressway bridge and bridge approach embankments at culvert locations

Location	Design Flood Level (m MSL Wellington)	Design Level for Structure (m MSL Wellington)
dry culvert on southern approach embankment to proposed expressway bridge	26.48	27.28 (road shoulder level)
proposed expressway bridge	25.73	26.93 (soffit level)
dry culvert on northern approach embankment to proposed expressway bridge	26.37	27.17 (road shoulder level)
Greenwood sub-catchment culvert	24.70	25.20 (road shoulder level)

Comparison of revised the design levels for the road embankment shoulders at the southern and northern approach embankment culvert locations with road levels in Table 5-1 indicate that the design freeboard requirement at these culvert locations is easily met. However, as demonstrated by Figures 5-5b and 5-13, the current vertical profile of the road past the Greenwood sub-catchment culvert does not satisfy the design freeboard requirement of the NZ Transport Agency's *Bridge Manual* (Transit NZ, 2003) due to the shallow overland flow from the main channel of the Waitohu Stream that flow parallel with the road embankment to the culvert. As noted previously in Section 3, the vertical alignment of the road is constrained past the Greenwood culvert by the need to transition into the existing vertical alignment of SH1 past Taylors Road. Within this constraint, the vertical alignment of this section of the road needs further revision if the design freeboard requirement of the *Bridge Manual* is to be satisfied.

6. Bed Level Management in Waitohu Stream

GWRC has responsibility under the Resource Management Act for managing the flood hazard posed by the Waitohu Stream. The flood hazard potential of the stream is strongly influenced by stream bed levels, requiring occasional gravel extraction in aggrading reaches to maintain existing bed levels and to prevent existing flood hazards from being exacerbated.

Consequently Resource Consent No. WGN070242, held by GWRC's Flood Protection Group, incorporates a recommendation that streambed levels are surveyed at least every 5 years to provide evidence to support the management of gravel extraction from the stream (Gardner, 2009). In the past cross-section surveys have been carried out in 1992, 2003 and 2009 and, on the basis of the first two surveys, gravel extraction has been undertaken in the aggrading reach between the existing SH1 bridge and a point about 700 downstream of the NIMT railway bridge.

Based on his analysis of the 2009 stream cross-section data, Gardner (2009) recommended that:

- gravel extraction be continued in the 1.7km long aggrading reach between the existing SH1 and cross-section 170 located 720m downstream of the NIMT railway bridge; and
- a full stream cross-section survey and data analysis is carried out every 5 years or after any flood greater than a 1 in 20 AEP flood.

On the basis of this evidence there are reasonable grounds for assuming that GWRC will continue to actively monitor and manage bed levels in the Waitohu Stream in the future. However, GWRC's Flood Protection Group have advised that, while they hold the Resource Consents which allow gravel extraction to take place, they are not resourced to undertake this themselves (S Westlake, pers. comm.). Gravel extraction in the aggradational reach downstream of the NIMT railway bridge is currently undertaken by others on demand through an arrangement with the local landowner as this reach is on private land. This gravel extraction could cease if there is no demand for the gravel material.

The Expressway bridge crossing of the Waitohu Stream lies within the wider aggradational reach of primary concern to GWRC below the existing SH1 bridge. As owner of the bridge, the NZ Transport Agency will therefore have a direct interest in the future monitoring and management of streambed levels in the immediate vicinity of the bridge to ensure that freeboard at the bridge is maintained. This could potentially lead to carrying out gravel extraction in the reach between the existing SH1 and NIMT railway bridges (by arrangement with GWRC's Flood Protection Group using their Resource Consent)s in order to maintain design bed levels.

7. Response to Comments by GWRC

Table 6-1 summarises the key comments made by GWRC in their review of the preliminary investigations for the initial scheme design of the proposed expressway carried out to set design levels for the new road at each waterway crossing, and our responses to these comments. Our responses are based on the content of this report.

Table 6-1 Key comments from GWRC and our response to them

Comment	Our Response
Differences in predicted flood extents compared to Kāpiti Coast District Plan flood hazard maps are explained	A comparison between the Kāpiti Coast District Plan flood hazard map and the flood extent predicted by the MIKEFLOOD model is shown in Figure 4-5. The inundation extents are very similar. The two areas of significant difference are satisfactorily explained.
Flood inundation maps should show ranges of predicted peak flood depths. Peak flow velocity maps should show ranges of flow velocities.	These styles of flood inundation maps and peak flow velocity maps have been used throughout this report (Figures 4-4, 4-6a, 4-6b, 4-8, 4-9, 5-2, 5-6a and 5-6b).
Peak flood depth and flow velocity difference maps should be produced.	Difference maps of this type have also been produced (Figures 5-3, 5-7, 5-10 and 5-11).
Consideration should be given to the potential effects of partial blockage of culverts by flood-transported woody debris.	This matter is addressed in Section 5-5 of this report.

Comment	Our Response
Consideration should be given to other potential effects: - sediment deposition in flood ponding areas - Erosion of floodplain surfaces by high velocity flows - changes in the drainage times of ponding areas	These matters are addressed in Section 5-5 of this report.

8. Conclusions

Floodwaters naturally break out of the Waitohu Stream upstream of the existing SH1 bridge due to a number of factors including the limited waterway capacity of the bridge, the bridge location immediately downstream of a very sharp bend and the low height of the stream banks. Flood breakout gives rise to extensive floodplain inundation along both banks including inundation of the highway.

The approach embankments to the 75m long bridge crossing of the Waitohu Stream and floodplain will act as a barrier to these overland flow paths. An 8m wide by 2.5m high box culvert on the south (left) bank and a 10m wide by 1.5m high box culvert on the right (north) bank are recommended to provide continuity for the overland flow paths. The large size of the culverts is necessary because of the large flow volumes that are conveyed by the left and right bank floodplains.

The bridge crossing of the Waitohu Stream is located near the beginning of a zone of geomorphic instability (Williams, 2004) caused by a change in streambed slope. The 75m bridge span provides ample fairway width for future potential lateral migration of the active stream channel and sediment deposition induced by this geomorphologic instability. The active stream channel is monitored and managed by GWRC in response to natural river processes including channel migration and sediment aggradation.

The effects of the Expressway with the 75m long bridge crossing of the Waitohu Stream and floodplain and the large culverts incorporated in the bridge approach embankments are no more than minor:

- the extent of flood inundation upstream and downstream is very similar to that for the existing situation;
- flood levels across the flood plain and in the main stream channel immediately upstream of the Expressway are increased relative to those in the existing situation but the increased levels extend no more than 100m upstream due to the steepness of the floodplain slope;
- the duration of flood inundation across the floodplain is not exacerbated; and
- flow velocities across the floodplain are not made any worse with the large culverts in the bridge approach embankments providing continuity for existing overland flow paths.

If the upstream SH1 bridge was ever replaced, the flood inundation pattern across the floodplain would be largely unchanged with increases in peak flood level in the main stream channel upstream of the Expressway bridge likely to be about 0.05m

The Greenwood sub-catchment is a known flooding hotspot for SH1 with the present culvert system considerably undersized for the sub-catchment runoff. Floodwaters breaking out of the Waitohu Stream along the right bank upstream of the SH1 bridge and flowing overland also spread into the Greenwood sub-catchment although this flow contribution lags and is secondary to the direct surface runoff contribution.

Right bank breakout flows from the Waitohu Stream upstream of the Expressway in the proposed situation also spread into the Greenwood sub-catchment along the upstream side of the Expressway embankment. However because the embankment is only slightly elevated above the floodplain, peak flood levels for the 1% AEP flood

adjusted for possible future climate change effects will be approximately coincident with the shoulder of the road.

SH1 is presently overtopped at the location of the Greenwood sub-catchment culvert by floods smaller than a 5% AEP flood adjusted for the effects of possible future climate change. There is minimal flood storage volume upstream of the culvert system so that there is negligible attenuation of peak flows past the culverts.

Because the vertical alignment of the Expressway is required to transition into the existing vertical alignment of SH1 immediately to the north of the Greenwood sub-catchment culvert, the recommended 4m wide by 1.5m high box culvert under the Expressway only just eliminates the occurrence of road overtopping for the 5% AEP flood adjusted for the effects of possible future climate change and satisfies the design freeboard standard. It does not make downstream flood inundation any worse than it would be in the existing situation. The 1% AEP flood adjusted for the effects of possible future climate change in the Greenwood sub-catchment would overtop the Expressway over about a 70m width at a very shallow depth to the north of the Greenwood sub-catchment culvert.

The recommended size for the Greenwood sub-catchment culvert would enable the flood risk at this location to be reduced in the future by requiring only the road level to be raised.

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