Technical Report 22

Assessment of Hydrology and Stormwater Effects

NZ TRANSPORT AGENCY
WAKA KOTAHI

MacKays to Peka Peka Expressway

Revision History

Revision Nº	Prepared By	Description	Date
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on behalf of	Beca Infrastructure Ltd		

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1 Executive summary

This report has been prepared for the purpose of statutory approval applications for the construction of the proposed MacKays to Peka Peka Expressway. It addresses stormwater in the broader sense, including providing a background to:

- surface water hydrology of the Project area;
- watercourse (drain, stream and river) crossings;
- Proposed Expressway stormwater management (both quantity and quality); and
- management of flood risk to both the proposed Expressway and adjacent land.

The report has been prepared in close liaison with other Project components, including proposed Expressway geometry, bridge designs, landscape and planting. Further, it takes into account other factors relevant to any design affecting the surface water environment including, in particular, groundwater and stream ecology.

The Project area primarily crosses a mix of low-lying peat land and sand dunes. The general fall of the land is from the hills in the east, across the coastal plains and the proposed Expressway, through to the coast in the west. There are a number of low-gradient watercourses that cross the proposed Expressway Alignment of which many have moderate to high ecological value. There is also one major river crossing, the Waikanae River, although a further four smaller watercourses will also have bridge crossings rather than culverts (two of these watercourses involve multiple crossings). The catchments of the larger watercourses extend up onto the eastern hills, while the watercourse catchments are confined to the coastal plains. There are many areas of natural or modified wetlands, some in peat areas, and some in low points in the sand dunes of the coastal plains. The principal catchment systems are shown on drawings CV-SW-010 and 011, Technical Report Appendices, Report 22, Volume 5.

The proposed Expressway Alignment crosses a number of areas that are flood prone, with significant areas that currently provide flood storage within the footprint of the proposed Designation. Of particular note are areas around the:

- Wharemauku Stream and its tributaries;
- Waikanae River and flood plain areas to the south;
- Kakariki Stream; and
- Paetawa Drain in the north.

Further, there is an identified flood overflow path north from the Waikanae River to the Waimeha Stream which would flow in the event of a Waikanae River stopbank failure or overtopping. These areas are sensitive to any change in waterway capacity, flood storage volume or barriers to secondary flow, and so influence the proposed Expressway design and mitigation, as described later in this report.

The stormwater design and assessment of effects has been undertaken in the context of a number of guidelines and statutory instruments, with the principal ones listed below. The principal guidelines and statutory instruments are discussed briefly in this Executive Summary, and discussed in detail later in this report.

At a high level, the MacKays to Peka Peka Project team[1] and KCDC have agreed to Guiding Objectives, which include broad principles to address potential hydrological and hydraulic effects of the proposed Expressway. From a stormwater design perspective, the principal guideline is the NZTA Stormwater Treatment Standard for State highway Infrastructure, 2010. From a hydraulic and capacity perspective, bridge waterways are designed in accordance with NZTA's Bridge Manual, 2003. Other documents influencing design approach and assumptions include KCDC's Kāpiti Stormwater Management Strategy, 2005, and the Ministry for the Environment's Climate Change Effects and Impacts Assessment - A Guidance Manual for Local Government in New Zealand, 2008.

The stormwater design and effects assessment requires the use of hydrological and hydraulic models. KCDC have calibrated models for the majority of the catchments. GWRC has a calibrated model of the Waikanae River and flood plain. Rather than duplicating these models we have reached agreement for the Project team to use them, and to superimpose the proposed Expressway onto the existing environment as represented in these models. This is the best method available to test the efficacy of the proposed Expressway mitigation. The models were run for the Project team by KCDC/GWRC's incumbent modelling consultants, with the details of the proposed Expressway and the modelling scenarios supplied by the Project team. Further modelling was undertaken by the Project team of the stormwater works within the proposed Expressway itself, and the results of this modelling were incorporated into KCDC's/GWRC's models of the wider environment. Two areas are not covered by the models. These areas are at the southern end around Poplar Avenue and at the northern end around Hadfields / Te Kowhai Stream, these were assessed using manual methods.

KCDC and GWRC are responsible for managing the flood risk in these watercourses so design has been undertaken in discussion with their technical staff (GWRC for the Waikanae River only).

The design approach has the following broad principles:

- Aim for hydraulic neutrality, taking into account both increased runoff from the proposed Expressway footprint, and loss of flood plain storage under the footprint in some areas;
- Treat stormwater runoff to remove entrained contaminants, in accordance with industry standard guidelines and "best practicable option" design standards;
- Attenuate peak runoff flow rates from the proposed Expressway targeting 80% of pre-Expressway rates;
- Use deep, flat-gradient roadside swales as the principal means of conveyance, water quality treatment and peak flow rate attenuation. Where swales are in low-lying peat areas, use wetland planting in the base. Where in sand, use grassed swales;
- Where swales are not able to provide for the treatment and/or attenuation required, use wetlands and/or storage areas to achieve the required performance before discharge;
- As far as practicable discharge to the nearest watercourse in order to retain current drainage routes as close as possible;
- Where the proposed Expressway fills in existing floodplain storage, provide equivalent offset storage volume to mitigate the effects of increased flood levels. This typically

[1] This Technical Report refers to the Project team as carrying out works on behalf of and as contracted by the NZTA. The NZTA is the requiring authority and the consent holder.

involves excavation of an adjacent area to provide live storage within the flood level range;

- Design watercourse crossings to pass the 1%AEP (100 year return period) storm flow without adversely affecting adjacent properties;
- Design cross-culverts to allow for fish passage where this is appropriate to the watercourse ecology; and
- Use the hydraulic models to confirm the above measures address the potential effects of the proposed Expressway development and operation.

The measures outlined above have been incorporated into the design with the aim of first avoiding effects and if this is not possible then mitigate the effect.

The design including mitigation is presented on the drawings in Appendix 22.A, Technical Report Appendices, Report 22, Volume 5.

Operation and maintenance will be undertaken in accordance with the principles set out in NZTA's Stormwater Treatment Standards for State highway Infrastructure. The highway will become part of NZTA's existing regional maintenance contract, which already sets out inspection and monitoring and maintenance requirements. Because some of the features of the proposed Expressway design differ from what is currently in that contract, specific requirements for these will need to be added to the contract. The principles relating to operation and maintenance are also set out in this report.

The hydraulic models demonstrate that the effects of the proposed Expressway on flood levels can be avoided or mitigated by the works proposed. The treatment design has been able to be achieved in accordance with the NZTA's Stormwater Treatment Standard for State Highway Infrastructure. The details of the system performance are contained in the body of the report, and in the modelling reports attached as appendices.

List of Abbreviations

5yr7d	5 year average recurrence interval 7 day duration low flow
0.04% AEP	1 in 2500yr storm
1% AEP	1 in 100 year storm
10% AEP	1 in 10 year storm
50% AEP	1 in 2 year storm
AEP	Annual Exceedence Probability
GWRC	Greater Wellington Regional Council
GWRC CM	Greater Wellington Regional Council – Catchment Management
ha	Hectare
KCDC	Kāpiti Coast District Council
km	Kilometre
L/s	Litres per second
m	Metre
m^3	Cubic metre
m^3/s	Cubic metres per second – flow rate
MALF	Mean Annual Low Flow – the mean of all annual lowest flows
MfE	Ministry for the Environment
M2PP	MacKays to Peka Peka
mm	Millimetre
NIMT	North Island Main Trunk railway
NIWA	National Institute of Water and Atmospheric Research
NZTA	NZ Transport Agency
Q95	95% percentile flow – equalled or exceeded 95% of the time
QE Park	Queen Elizabeth Park
RL (or r.l.)	Reduced Level in m
REC	River Edge Consultancy Ltd
SH1	Existing State Highway 1
SKM	Sinclair Knight Mertz Ltd
WRENZ	Water Resources Explorer New Zealand
WWTP	Waste Water Treatment Plant

2 Introduction

The scope of this report is to detail the issues, design, effects and mitigation measures relating to stormwater from the proposed MacKays to Peka Peka Expressway (the Project). For the full Project description (construction and operation) refer to Part D, Chapters 7 and 8, Volume 2.

This report details the following stormwater topics:

- Hydrology (surface water only) including climate change;
- Drainage;
- Attenuation (peak flow control);
- Treatment (quality);
- Watercourse crossings (culvert and bridges);
- Watercourse diversions; and
- Flooding and floodplain issues.

This report should be read in association with the following stormwater related Technical Reports (Volume 3) and Management Plans (Volume 4):

- Technical Report 4, Construction Methodology Report, – for construction stormwater, particularly the main construction yard at Otaihanga Landfill;
- Technical Report 5, Urban Design and Landscape Framework – for bridge form and landscaping;
- Technical Report 7, Assessment of Landscape and Visual Effects – for bridge form, swales, wetland, watercourse and storage area planting;
- Technical Report 21, Assessment of Groundwater Effects – for groundwater interactions of swales, wetlands and offset storage areas;
- Technical Report 24, Baseline Water and Sediment Quality Investigation – for existing receiving watercourse water quality;
- Technical Report 25, Contaminant Load Assessment – Expressway runoff water quality effects;
- Technical Report 26, Ecological Impact Assessment – receiving watercourse ecological effects;
- Technical Report 27, Terrestrial Vegetation and Habitats (Including Wetlands) – Ecological
  description and evaluation of existing wetlands;
- Technical Report 30, Freshwater Habitat Description and Values – Ecological description and evaluation of existing receiving watercourse habitats;
- Technical Report 35, Assessment of Ground Settlement Effects; and
- Construction Environmental Management Plan (CEMP, Volume 4) – construction related effects;
- Appendix H of the CEMP, Erosion and Sediment Control – for construction stormwater management;
- Appendix I of the CEMP, Groundwater (Level) Management Plan – for monitoring of groundwater levels;
- Appendix J of the CEMP, Settlement Effects Management Plan – for monitoring and management of settlement; and
- Appendix M of the CEMP, Ecological Management Plan – for the ecological aspects of designing stream diversions.
- Appendix T of the CEMP, Landscape Management Plan – for landscaping of stormwater management areas during construction.

Section 7 of the Design Philosophy Statement[2] also outlines in detail the design standards, processes and criteria used for the stormwater design presented in this report.

NZTA's approach to stormwater management is set out in its Stormwater Treatment Standard for State highway Infrastructure, which provides the context and framework for stormwater management on this Project. The overall approach of the Standard is:

> *"To provide best practice for both stormwater quantity and quality control that, in the absence of local requirements or where local requirements are limited, NZTA will undertake to demonstrate environmental responsibility"* [3]

Kāpiti Coast District Council (KCDC) has formal stormwater and flood management strategies and preferred approaches and as the Project is within their jurisdiction, the design took this into account when selecting options and features.

The following are identified as the high level stormwater design constraints or influences:

- Neither increasing flooding to land upstream or downstream of the proposed Expressway, nor increasing peak flow discharges leaving the Designation corridor;
- Various flood levels, routes, storage areas and floodplains as shown on KCDC flood maps,
- Existing wetlands;
- Spatial constraints for locating new wetlands/ponds;
- Groundwater levels for wetland designs; and
- Bridge piers/abutments to be clear of any primary watercourse channel.

The key assumptions used in the design of the stormwater drainage systems and for hydrological management are:

- KCDC/Greater Wellington Regional Council (GWRC) catchment/river models provide the water level, flow and velocity information for the majority of the watercourses crossed;
- Stormwater treatment is required and will be designed in accordance with NZTA's Stormwater Treatment Standard for State highway Infrastructure;
- The effects of climate change will be incorporated for "mid estimate" to 2090, in accordance with Ministry for the Environment (MfE), KCDC & GWRC parameters;
- Seek to minimise stormwater effects on the environment by using a low impact design approaches;
- Attenuation of peak flows will be to NZTA's Stormwater Treatment Standard for State Highway Infrastructure targeting 80% of pre-Expressway peak flow rates (where pre-Expressway means the flow rate that would run off of from a catchment prior to the proposed Expressway being constructed i.e. what currently occurs);

[2] Refer to Part A, Chapter 2, Volume 2.

[3] Stormwater Treatment Standard for State Highway Infrastructure, 2010, NZTA

- Overall, the design will meet KCDC's "hydraulic neutrality"[4] criterion whilst keeping open the opportunity of achieving this outcome outside the footprint of the proposed Expressway (for example, integrating proposed Expressway stormwater management with that associated with KCDC's town centre development works); and
- As agreed with KCDC, simulations of future catchment development scenarios are not required due to KCDC's requirement that new developments be "hydraulically neutral", meaning that future peak flows arriving at the proposed Expressway from areas upstream will be no greater than current.

3 Existing environment

3.1 Project Area - Overview

The proposed Expressway crosses the low-lying coastal plains and dune areas of the western Kāpiti District. The characteristics of the land are described below, insofar as they are relevant to hydrology, stormwater and flood risk management.

The majority of the land crossed is modified farm land except in the urban areas of Raumati, Paraparaumu and Waikanae. The land is characterised by a mix of low peat flats and sand dune formations. The inter-dunal areas are generally low lying, in which wetlands have formed, often with no formal drainage connection.

There are many wet areas and wetlands along the route, many of which have been heavily modified by farm or urban development. The proposed Expressway also passes through the headwaters of the regionally significant Te Harakeke/Kawakahia Wetland complex located near the coast between Waikanae Beach and Peka Peka Beach settlements.

The main watercourse systems are shown on drawings CV-SW-10 and 11 Technical Report Appendices, Report 22, Volume 5 and are described in detail in GWRC/NIWA/KCDC's stormwater reports (listed in the reference section). A brief summary of the main catchments is included in Sections 3.2 to 3.5 of this report.

The Waikanae River is the largest watercourse that will be crossed by the proposed Expressway. The river is managed by the Catchment Management division of the GWRC which has a flood protection scheme for the Waikanae River and actively manages the river corridor in accordance with this.

There are also a number of watercourses that will be crossed by the proposed Expressway and as with the wetlands, all of them have been heavily modified by farm or urban development.

The areas of low lying flat land within the Project area are generally subject to significant flooding during heavy rainfall events and these areas also generally coincide with peat flats.

The Project commences at chainage 1900m just south of Poplar Avenue. Earlier in the design it started at chainage 0m, with widening of the Highway through the Raumati Straight area further to the south. This is no longer included.

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[4] Hydraulic Neutrality has been agreed with KCDC to mean not discharging at greater rates than the existing peak flows nor to cause a significant increase in flood levels by either filling in floodplain storage or in the sizing of culverts.

3.1.1 Topography

The topography along the route has a strong influence on hydrology, flood risks and on aspects of stormwater management. In simplified terms, the route passes through coastal floodplains and sand dunes. The sand dunes dominate the topography, appearing both in areas where they are the principal land form, but also as local isolated mounds within the floodplain areas. The coastal floodplains are typically low-lying peat areas, and are particularly dominant at the southern and northern ends of the Project.

3.1.2 Rainfall

The rainfall patterns of the District are strongly influenced by the prevailing westerly winds along with the presence of the coastal hills and, further east, the Tararua Ranges. Mean rainfall across the Project area is of the order of 1100 to 1200mm per year[5], with a tendency for greater rainfall depths in winter than in summer.

There is a strong increasing rainfall gradient east into the hills, with the upper portions of the Waikanae River catchment having mean annual rainfall of nearly 3000mm[6].

Rain storm intensity is slightly greater towards the south, apparently reflecting the closer proximity of the coastal hills in the Paekakariki area, which influence rainfall patterns as far north as Paraparaumu. Isohyetal maps have been prepared for KCDC by Sinclair Knight Mertz (SKM)[7] to cover the Kāpiti Coast area. To illustrate the rainfall intensity, distributions for the 1%, 10%, 50% Annual Exceedance Probability (AEP) 24 hour design rainfall have been included in the drawing set as drawings CV-SW-050, 051 and 052 Technical Report Appendices, Report 22, Volume 5.

3.1.3 River and Stream Form

The stream form in the Project area varies depending on the nature of the catchments. There are three main catchment types:

i. <u>Large hill catchments:</u>
   The dominant flow comes from the coastal hills, with varying amounts contributed by low-land catchments. The most significant of these is the Waikanae River but the Wharemauku Stream also falls into this category;
ii. <u>Low-land catchments:</u>
   Many of these receive some runoff from the western hill faces, but do not have substantial hill catchments. Low-land runoff is a significant component of these stream flows e.g. the Mazengarb Drain.
iii. <u>Spring-fed streams:</u>
   These tend to be a subset of the low-land streams e.g. the Waimeha Stream. Although there are also spring flow components to some hill watercourses e.g. the Wharemauku Stream.

For the Project, the type of stream and its hydrological characteristics depend also on the point at which the proposed Expressway will cross it. Thus, for example, in the north the proposed Expressway crosses Te Kowhai/Hadfield's Stream at the point where a relatively small but steep hill catchment reaches the coastal plain. Therefore the stream at this point carries large flows and significant sediment and debris. Further downstream, the lowland part of the catchment is much larger and flatter so the meanders across the floodplain. Thus the flows slow down, floods spread out across the floodplain and the larger sediments drop out of suspension, depositing on the floodplain.

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[5] Figures from NIWA's Water Resource Explorer (WRENZ), as of September 2011.

[6] Figure from NIWA's Water Resource Explorer (WRENZ), as of September 2011.

[7] Update of Kāpiti Coast Hydrometric Analysis, SKM for KCDC, Draft A, 14 April 2008.

3.1.4 Hydrology

Hydrology is of interest to the Project both in terms of flood flows (for bridge and culvert design) and in terms of the day-to-day flows and low flows (for ecological understanding).

The low-flows in Table 1 have been estimated using a variety of methods, depending on data availability. Where there is a flow record, that data has been used. Where the watercourse characteristics are available on NIWA's WRENZ model, these have been used. For other streams, characteristics have been estimated based on catchment area, hydrological character and generic flow characteristics derived from other nearby streams. Refer the memorandum in Appendix 22.C for more detailed information on the derivation of the low-flows.

Table 1: Low- Flow Hydrology

M2PP Crossing Reference	Watercourse	Mean Flow (L/s)	Q95[8] Flow (L/s)	MALF[9] (L/s)	5yr Low Flow (7d)[10] (L/s)
11.1	Wharemauku Stream	158	18	15	8
14	Mazengarb Drain	51	8	-	-
NA[11]	Muaupoko Stream	145	22	-	-
23	Waikanae River	4766	760	960	735
25	Waimeha Stream	169	112	-	-
26	Ngarara Creek	28	4	-	-
29	Kakariki Stream	126	19	-	-
36	Paetawa Drain	66	10	-	-
40	Hadfield / Te Kowhai Stream	17	3	-	-

In addition to the principal watercourses listed above, there are many other minor watercourses and drains that will be crossed by the proposed Expressway (refer Technical Report Appendices, Report 22, Volume 5 for the stormwater drawings and also the associated Culvert Schedule in Appendix 22.B of this report). Apart from flood flows for culvert design, the hydrological characteristics for these have not been calculated.

3.1.5 Ecological and Freshwater Habitat Descriptions and Values

For further descriptions on the ecological and habitat values of the existing watercourses and wetlands within the Project refer Technical Report 26, Volume 3 (Ecological Impact Assessment) and also particularly Technical Report 30, Volume 3 (Freshwater Habitat Description and Values) for watercourses and Technical Report 27, Volume 3 (Terrestrial Vegetation and Habitats [Including Wetlands]) for the wetlands.

3.1.6 Flood Risk

Much of the urban or peri-urban area has been modelled by KCDC because they are at significant risk of flooding during severe storms. The resulting flood maps have been prepared by KCDC for their own uses. However, these maps provide a robust means of assessing areas where the proposed Expressway interacts with watercourses and flooding. It provides the basis for not only assessing the flood risk to the proposed Expressway but also assessing the effects of the proposed Expressway on those flood risks.

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[8] Flow that is exceeded 95% of the time.

[9] Mean Annual Low Flow – available only where the watercourse has recorded flow data.

[10] Five year low flow seven day duration – available only where the watercourse has recorded flow date.

[11] Muaupoko Stream will not be crossed by the proposed Expressway.

Both natural and artificial flood storage and peak flow attenuation play a significant role in protecting downstream areas from flooding which is highly relevant in the context of a coastal floodplain. It is therefore important that, in assessing the effects of the proposed Expressway, consideration is given to both the additional runoff that comes from the paved carriageway and the loss of floodplain storage through embankment fill.

The 1% AEP (i.e. 1 in 100 year average recurrence interval) flood extents are shown on drawings CV-SW-022 through 031, Technical Report Appendices, Report 22, Volume 5. The extent of flooding is shaded blue. This information has been generated from KCDC and GWRC's models and takes into account the effects of climate change as discussed in Section 3.1.7.

Table 2 below shows the design flows for the major watercourses listed in Table 1. The flows of all the watercourse crossings (mostly drains) are have been determined for culvert sizing. The full schedule of culverts is included in Appendix 22.B. The 50% AEP flows were determined for use in the Waikanae River waterway investigations and are generally not needed for the other watercourses hence they have not been determined.

Table 2: Flood Flow Hydrology

M2PP Crossing Reference	Watercourse	50% AEP m^3/s	10% AEP m^3/s	1% AEP m^3/s
11.1	Wharemauku Stream[12]	-	22.1	32.6
14	Mazengarb Drain[12]	-	5.5	8.2
NA	Muaupoko Stream[12]	12	17	24
23	Waikanae River[13]	158	-	488
25	Waimeha Stream[12]	-	5.1	9.8
26	Ngarara Creek[12]	-	2.1	3.5
29	Kakariki Stream[12]	-	16.4	22
36	Paetawa Drain[12]	-	12.2	23.8
40	Hadfield / Te Kowhai Stream[14]	-	3.9	6.9

[12] Data Source: KCDC model.

[13] Data Source: GWRC model.

[14] Data Source: Project team calculation.

3.1.7 Climate Change

The Project is affected by climate change in two principal respects:

i. changes in rainfall intensity; and
ii. sea level rise.

Our assessment for these effects is based on the relevant MfE guidelines[15]. Climate change can be represented by mean annual temperature increase. There are many global models available internationally, with differing assumptions and consequent conclusions as to what temperature rise might be. These models have been downscaled to predict temperature change and consequences for other climate parameters for different parts of New Zealand. For the Kāpiti Coast area, the mean of the modelled values for temperature rise out to 2090 is 2.1º, although the range is 0.6 to 5.2º. This increase has been adopted for the Project, and is in accordance with the recommendations of MfE, GWRC and KCDC.

It is anticipated that rainfall in the Kāpiti Coast District will increase, with that increase varying according to the season. Storminess will also increase, leading to even greater increases in rainfall intensity in extreme storm events. Thus the mean annual rainfall on the Kāpiti Coast is expected to increase by about 3%, with seasonal increases varying from slightly drier summers (-1%) to wetter winters (+9%). Storm intensity is predicted to increase by up 16.8% for more extreme events of shorter durations, although the increase will be less in more frequent, longer duration events.

It is important to keep the effects of climate change in context, and to recognise that while climate change is predicted to result in a shift in rainfall characteristics, rainfall itself is very variable and is also driven by cyclical patterns over years and decades – patterns such as the Interdecadal Pacific Oscillation (IPO). These patterns lead to even greater variation from year to year than the overall shift from climate change, and are very much part of the normal, existing environment.

Currently guidelines for sea level rise would require provision for a 0.5m rise out to 2090. However, there is ongoing research into the appropriate increase to allow for, and figures vary widely. Recently, it has become evident that there is justification for adopting a higher sea level rise of 0.8m. KCDC has adopted this value, and for consistency with local practice this has been adopted by the proposed Expressway Project as well. The Project design is relatively insensitive to the assumed sea level rise, as it is generally upstream of the zone of influence of sea level, except marginally at the points where it crosses the Waikanae River and Waimeha Streams.

[15] Climate Change Effects and Impacts Assessment. A Guidance Manual for Local Government in New Zealand. 2nd Edition, Ministry for the Environment, May 2008

3.2 Sector 1 - Raumati South

This Sector covers from south of Poplar Avenue to Raumati Road i.e. chainage 1900 to 4500. The key stormwater features in this Sector include:

- Queen Elizabeth (QE) Park Drain – also known as Whareroa Tributary;
- Drain 7 south; and
- Flood storage area along Drain 7.

3.2.1 QE Park Drain

The existing SH1 crosses the Raumati peat flats alongside the North Island Main Trunk (NIMT) railway line. The railway has been built on top of an embankment and is set well above SH1. This acts to cut off the catchments east of SH1 and control the extent of flooding to SH1 and the downstream peat flats of QE Park.

South of Poplar Avenue, SH1 meets the toe of the western escarpment of the Tararua Ranges where it then runs along the base of this escarpment, between railway and QE Park.

The main watercourse in the south of Sector 1 is the QE Park Drain (photos 1 and 2). It drains part of the hillside catchments east of SH1, the railway, SH1 itself, northern QE Park and a small partially urbanised catchment north of Poplar Avenue.

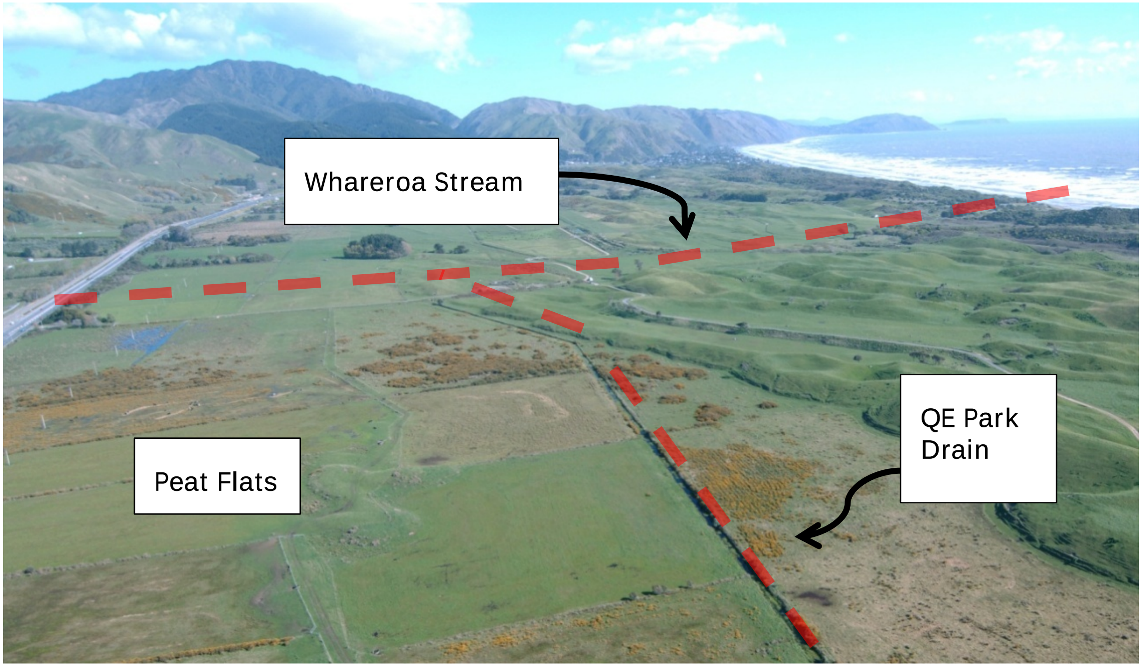


Photo 1: QE Park looking south towards MacKays Crossing with SH1 on the far left of the photo.

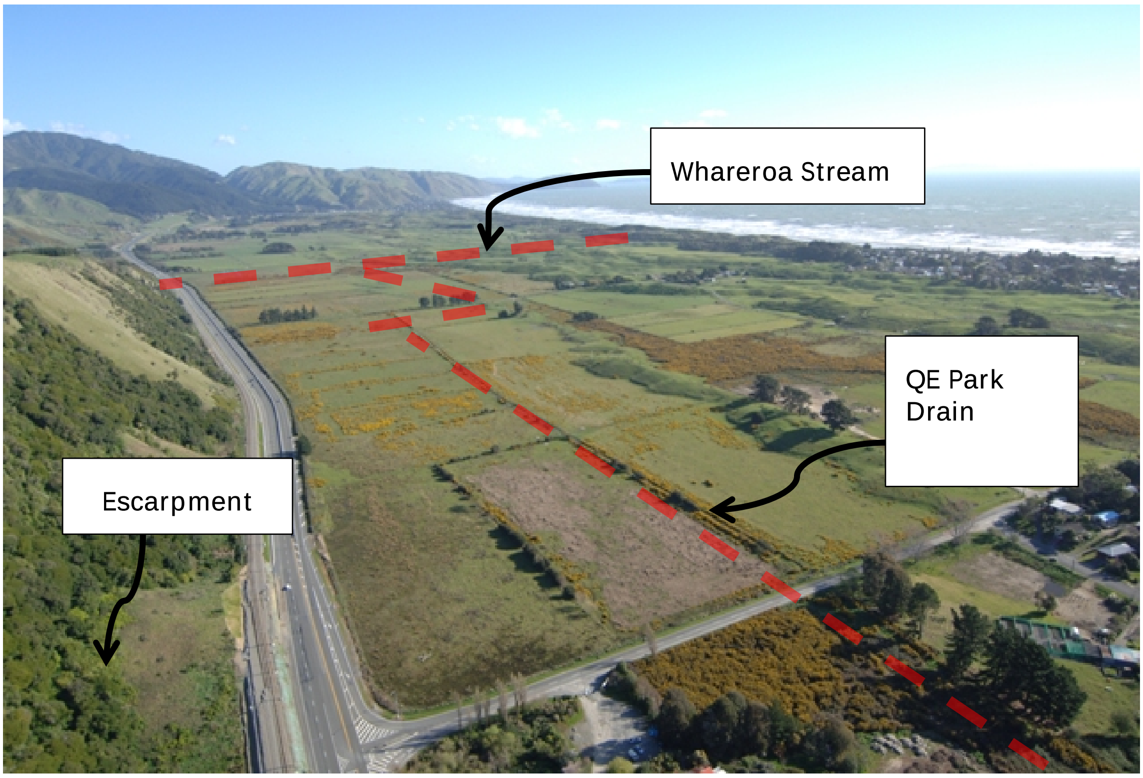


Photo 2: North end of QE Park looking south. Poplar Avenue is in the foreground, escarpment on the left of shot.

As part of the recent KiwiRail railway upgrade (NIMT double tracking project), the drainage on SH1 south of Poplar Avenue was significantly upgraded with the construction of new sump and pipe drainage. While the southbound lanes of SH1 are drained by these, the northbound lanes remained unchanged with runoff direct to an adjacent table drain (i.e. an open channel drain providing pavement subgrade drainage as well as stormwater drainage) or direct to ground.

The SH1 drainage discharges into culverts under SH1 that in turn discharge either to overgrown open channel drains or, in some instances, directly to ground. The table drain noted above also receives some of these culverts discharges conveying to connecting shallow open channel drains that head west across QE Park. Many of these drains do not have a formal outlet, with only some reaching the main QE Park Drain that in turn discharges into the Whareroa Stream.

The drains through QE Park are in varying condition but are generally over grown with vegetation and appear to be poorly maintained with eroding banks accessed by stock.

3.2.2 Poplar Avenue to Leinster Avenue

The land bounded by SH1, Poplar Avenue and Leinster Avenue forms a small catchment that drains into the upper end of the QE Park Drain by a 450mm diameter culvert under Poplar Avenue (photo 3). This area also receives some stormwater from the escarpment east of SH1. Again, the railway embankment cuts off the escarpment catchments and culverts under the railway restrict the flow enough that small informal wetlands have formed on the upstream side of the railway.

Photo 3: QE Park Drain looking downstream from the Poplar Avenue culvert.

3.2.3 Drain 7 South

North of Leinster Avenue, SH1 is drained by a sump and pipe network northwards to an open channel drain at No. 226 SH1. This open channel drain in turn runs west into KCDC's Drain 7.

The land west of SH1 is significantly lower (typically 2m lower) than SH1, but it still drains to the same drainage network noted above. In one location, No. 272 SH1, part of the land is low enough that a small stormwater pump is needed to drain runoff. This pump was installed and is maintained by NZTA.

Drain 7 is the main watercourse in the north of Sector 1 and it serves the area west of Leinster Avenue running north to Raumati Road passing under it in a culvert. The undeveloped land surrounding the upper reaches of Drain 7 (Photo 4) is low lying and is zoned as a flood storage area in KCDC's District Plan. For further details on the ecology of these wetlands refer to Technical Reports 26, 27 and 30, Volume 3.

Drain 7 has limited capacity leading to significant flooding issues at several locations along its length all the way down to its confluence with the Wharemauku Stream near Paraparaumu Airport.

The land north-west of the flood storage area through to north of Raumati Road is formed from sand dunes and, aside from a small marshy depression at the Raumati Road, is relatively free of flooding issues.

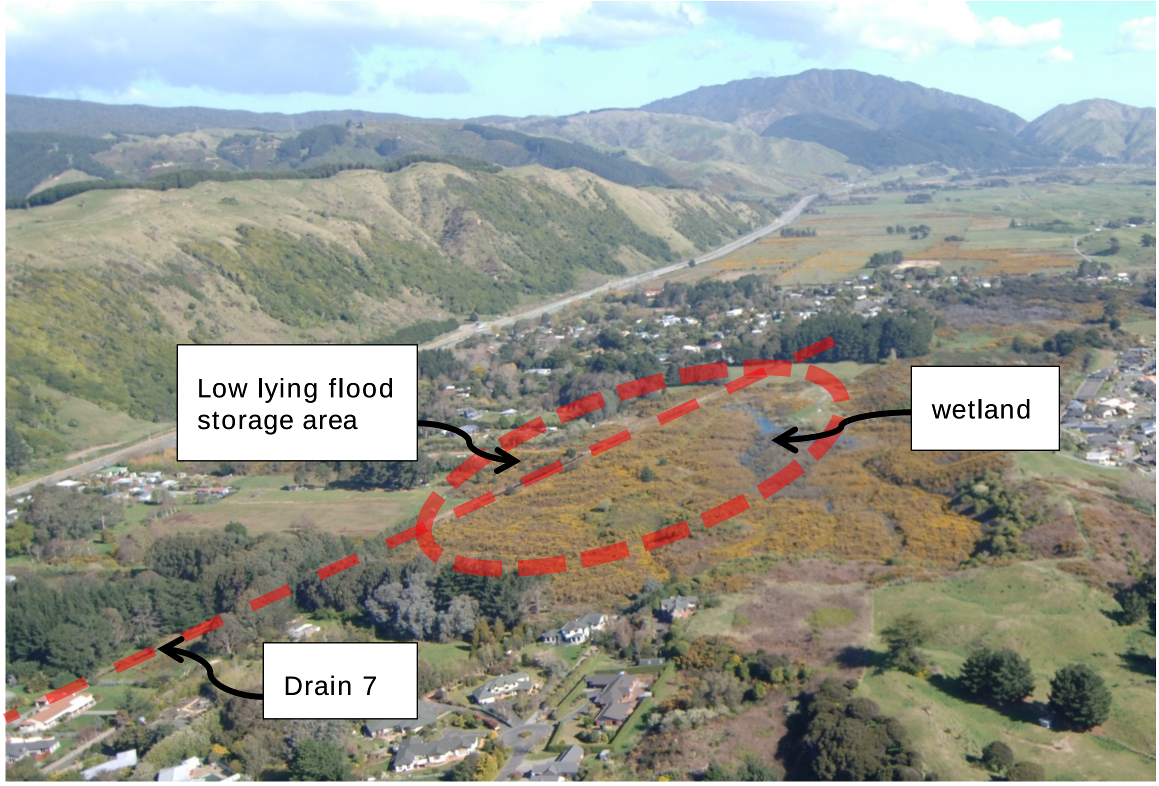


Photo 4: Drain 7 south and associated existing flood storage area and existing wetland.

3.3 Sector 2 – Raumati / Paraparaumu

This Sector covers from Raumati Road to the Waste Water Treatment Plant (WWTP) that is just north of Mazengarb Road i.e. chainage 4500 to 8300m. The key stormwater features in this Sector include:

- Drain 7 north;
- Wharemauku Stream;
- Wharemauku flood storage area;
- Drain 5 and Kāpiti Road; and
- Mazengarb Drain.

3.3.1 Drain 7 North

Drain 7 continues to drain north from Sector 1, through residential areas (in a mix of pipe and open channel sections) to Rata Road where it is culverted under the road (refer Photos 5, 6 and 7). From there it runs in an open channel through pastoral land at the foot of a sand dune formation. It then is culverted under Kiwi Road and drains into the Wharemauku Stream at the south end of the airport.

Photo 5 and 6: Drain 7 north, looking upstream of Rata Road (left) and to the culvert under Rata Road (right).

Photo 7: Drain 7 north, looking downstream of Rata Road.

The residential properties around Rata Road and Kiwi Road are known to have poor drainage and high groundwater that is controlled by the surrounding peat flats of the Drain 7/Wharemauku Stream floodplain.

3.3.2 Wharemauku Stream and Flood Storage Area

The Wharemauku Stream catchment has a mixture of residential and commercial properties sited on the coastal plain, with an upper catchment of rural farm land and forestry blocks. Most of Paraparaumu Township drains to the Wharemauku Stream.

The upper catchment comprises steep hill lands with shallow clayey soils, in contrast to the flat coastal peat and dune areas of the lower catchment. The groundwater is relatively high throughout the peat areas of this catchment.

The area of the Wharemauku catchment upstream of the proposed Expressway is approximately 1,000ha. The Wharemauku Stream is the main watercourse through Paraparaumu town centre and is fed by many tributary watercourses that generally all come together in the area between the town centre and the Airport. It then outfalls to the sea at Raumati beach, west of the Airport.

The tributaries of Wharemauku Stream that interact with the proposed Expressway are:

- Drain 7 – draining down from catchments discussed in Sections 3.2.3 and 3.3.1. Drain 7 joins the Wharemauku at the Airport; and
- Drain 5 – drains north-west parts of the catchment from Mazengarb Road to Kāpiti Road, joining the Wharemauku Stream upstream of Kiwi Road. Drain 5 is discussed further in Section 3.3.3.

The Wharemauku Stream is a highly modified watercourse with generally a uniform trapezoidal section running with few natural meanders or areas of riparian cover. It has been engineered and maintained to optimise flood conveyance (Photo 8).

Photo 8: Wharemauku Stream (looking upstream from the proposed Expressway crossing) with walking track along the top of its stopbank.

Just upstream of the Airport, the stream passes through a narrow gap between the sand dunes at the end of Kiwi Road (refer photos 9 and 10). This gap functions as a flood corridor and controls flood flows in extreme storms so that flood water spills out into paddocks upstream of Kiwi Road. This large open area is zoned as KCDC flood storage and is integral to KCDC's flood management for the Wharemauku catchment.

Photo 9: Wharemauku floodway between urbanised sand dune formations to the north and south.

The Wharemauku flood storage area is also an integral part of KCDC's future town centre planning. KCDC has advised that any town centre development plan for this area will take into account its effects on peak flows and flood storage. KCDC is currently working through the planning for the future town centre.

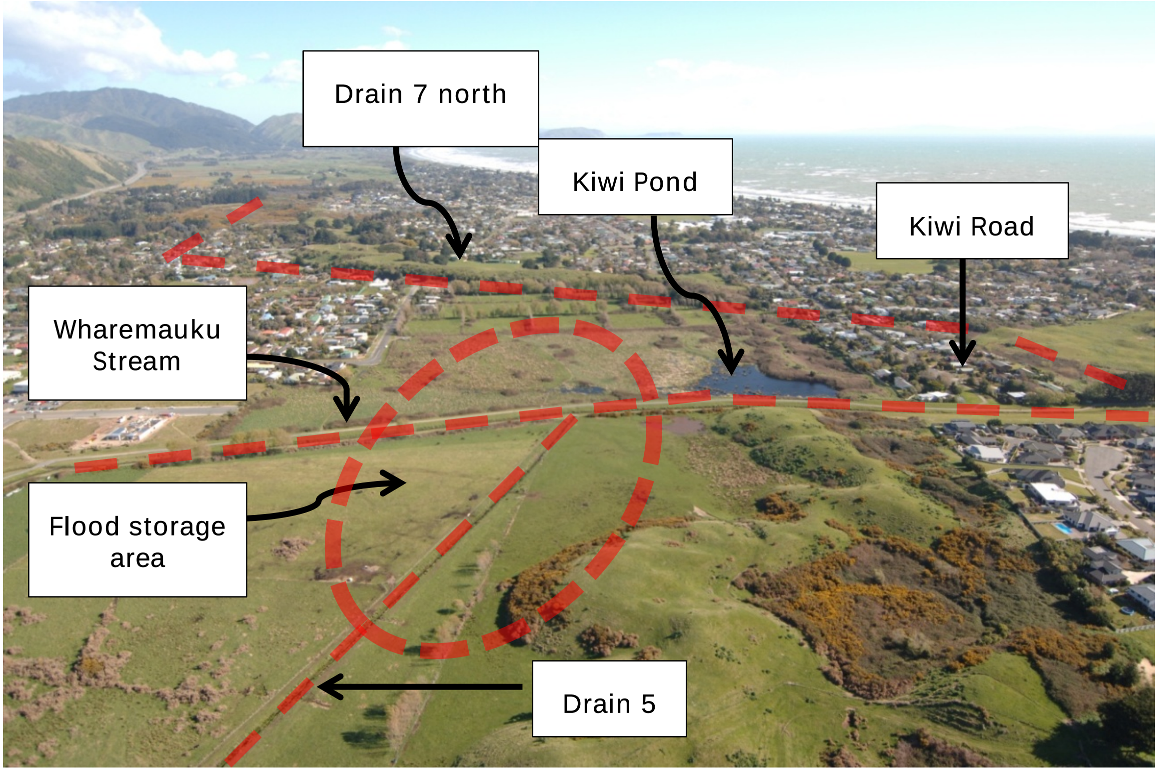


Photo 10: Wharemauku Stream flood storage area. The floodway corridor can be seen to the right of Kiwi Pond.

KCDC has progressively stopbanked the Wharemauku Stream since the 1970's[16] and part of this stopbank cuts off a low point in the topography east of Kiwi Road. As no drain was installed through the stopbank, water naturally collected behind it and over time formed what is now known locally as the Kiwi Pond (Photos 10 and 11). This pond was drained farmland prior to the stopbank being constructed.

KCDC advised they were planning to install a new drain through the stopbank to drain down and control the water level within the pond.

The properties on the eastern side of Kiwi Road are located in an area of high groundwater and in winter time their drainage is poor as it relies on soakage.

[16] Wharemauku Stream Stormwater Runoff and Floodplain Assessment, Connell Wagner, June 2006.

Photo 11: Kiwi Pond with overgrown stopbank in the foreground.

3.3.3 Drain 5

North of the Wharemauku Stream, the route crosses a line of sand dunes before returning to more low lying scrub land.

The residential area between Kāpiti Road and Mazengarb Road is drained by KCDC's sump and pipe network that discharges into the top of Drain 5 at Kāpiti Road. Drain 5 is a trapezoidal open channel farm drain with no appreciable meander and minimal native riparian cover. It joins the Wharemauku Stream east of Kiwi Road (Photo 12 and 13).

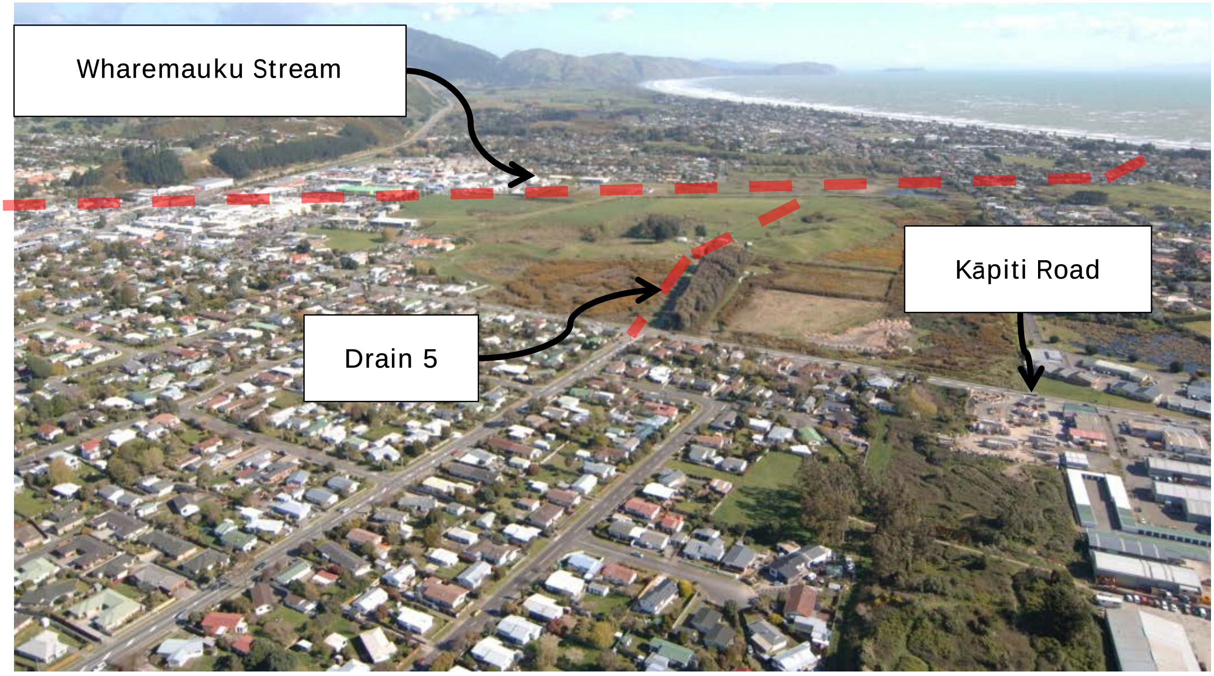


Photo 12: Northern Wharemauku catchment, Drain 5 and Kāpiti Road.

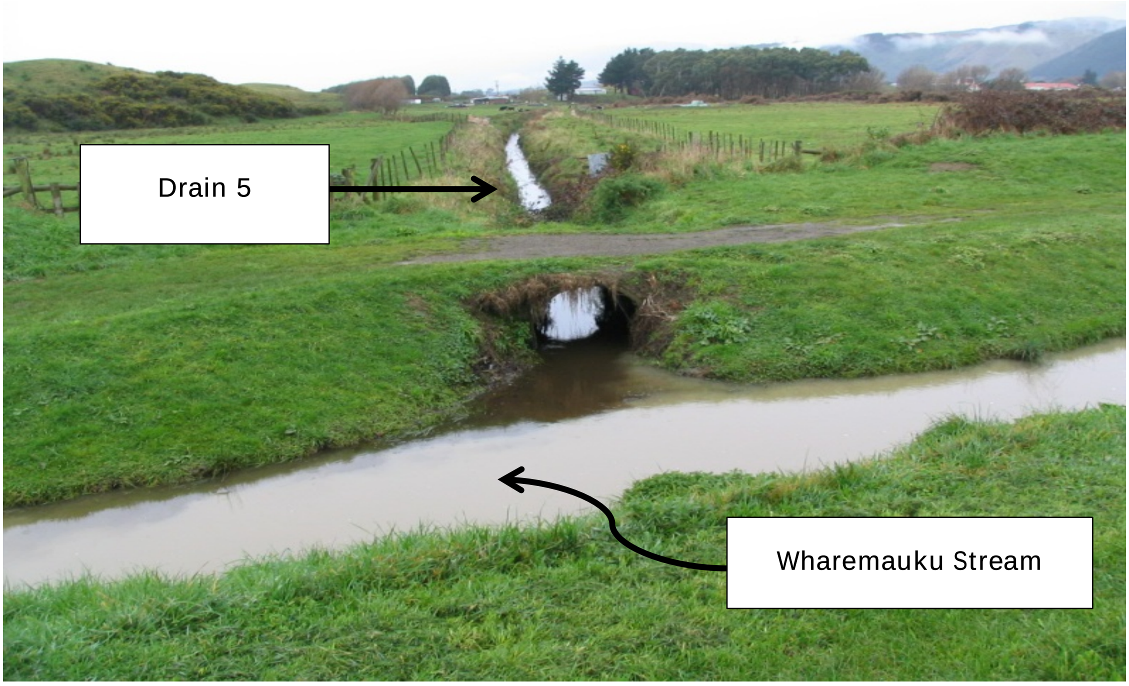


Photo 13: Drain 5- Wharemauku Stream confluence.

3.3.4 Mazengarb Drain

The Mazengarb catchment is similar to the Wharemauku in land use, but on a smaller scale. The wider Mazengarb catchment includes the Waikanae WWTP and the Otaihanga Landfill (now closed). The area of the Mazengarb catchment upstream of the proposed Expressway is 425ha.

The Mazengarb Drain is located just north of Mazengarb Road and it drains residential catchments, including recent subdivisions with their own stormwater attenuation ponds. The watercourse cuts through a line of sand dunes at the point where the proposed Expressway will cross it (refer photo 14).

Both downstream and upstream the Drain passes through relatively low lying residential properties. Further downstream it joins with the WWTP Drain (refer Section 2.4.1), before ultimately discharging into the lower reaches of the Waikanae River.

The Mazengarb Drain also receives flow from a small area on the South side of Mazengarb Road that is the site of the proposed Meadows New Life Trust development. The plans for which include their own stormwater wetlands/ponds.

Flood flow in the drain is intentionally constricted by KCDC at the Fytfield Place culvert (approximately 200m upstream of the proposed Expressway crossing) as part of KCDC's flood management of this catchment.

Photos 14, 15 and 16: Aerial view of the Mazengarb Drain area looking south (top). Mazengarb Drain downstream (bottom left) and upstream (bottom right) of the proposed Expressway crossing. Note the house with a relatively low floor level compared to the drain.

3.4 Sector 3 – Otaihanga / Waikanae

This Sector covers from the WWTP to north of the Waimeha Stream i.e. chainage 8300 to 12400m. The key stormwater features in this Sector include:

- WWTP Drain;
- Landfill Drain and wetlands;
- Otaihanga Drain;
- Muaupoko Stream;
- Waikanae River and floodplain;
- Te Moana floodway;
- Wetlands north of the Waikanae River; and
- Waimeha Stream.

3.4.1 Waste Water Treatment Plant Drain

The WWTP Drain (refer photographs 17, 18 and 19) serves a relatively small catchment upstream of the proposed Expressway and its main source of flow is the discharge from the WWTP. A site visit in February 2011 noted that there was no baseflow in the drain upstream of the plant outlet i.e. on that day the plant was source of all of the flow in the drain.

Downstream of the proposed Expressway, the drain passes through the southern end of a dune and wetland complex that borders the Otaihanga Landfill. It then flows west through low lying rural properties before joining the Mazengarb Drain approximately 330m downstream of where the proposed Expressway will cross the Mazengarb Drain.

Photos 17 and 18: WWTP Drain showing the WWTP inflow (top) and the WWTP drain looking downstream from the WWTP's western fence line (bottom).

3.4.2 Landfill Drain and Wetlands

The Otaihanga Landfill is drained by an open channel drain (photo 19) that runs through a one of a series of wetlands that have formed between the north-south aligned bands of sand dunes. The drain passes through private rural lifestyle properties west of the proposed Expressway and ultimately drains into the Waikanae River.

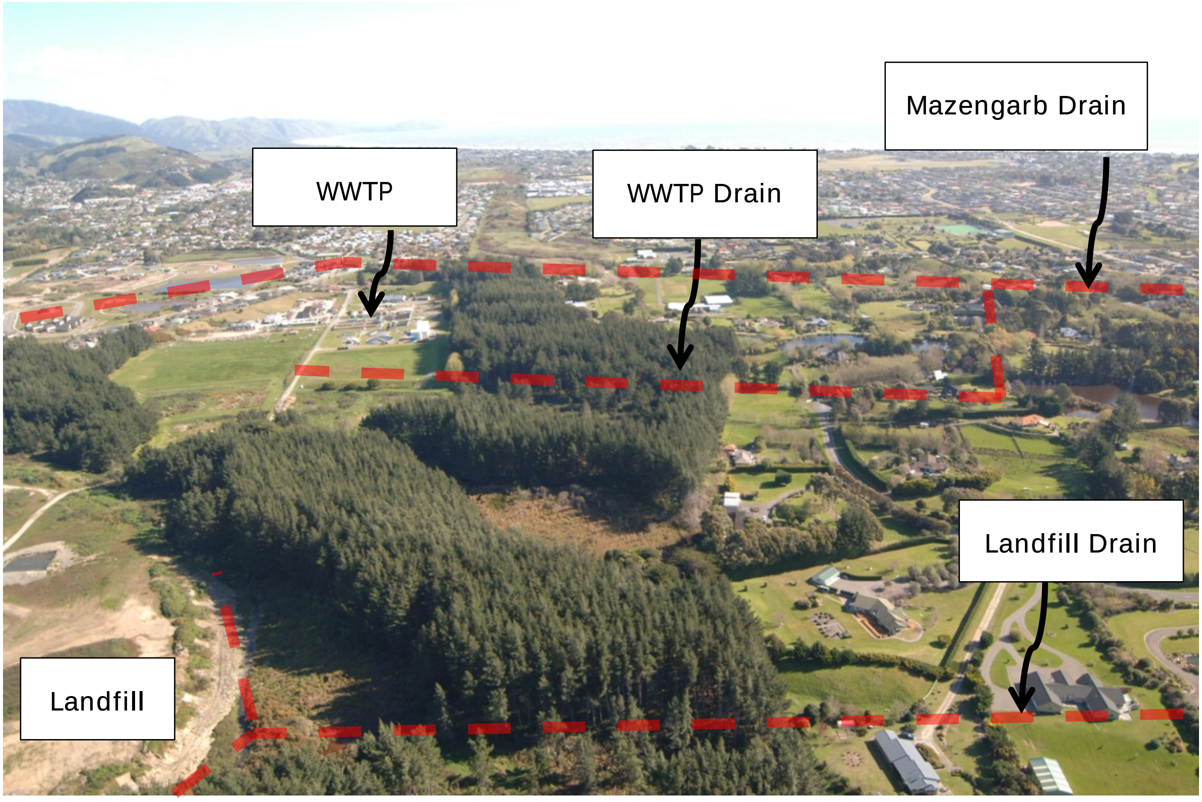
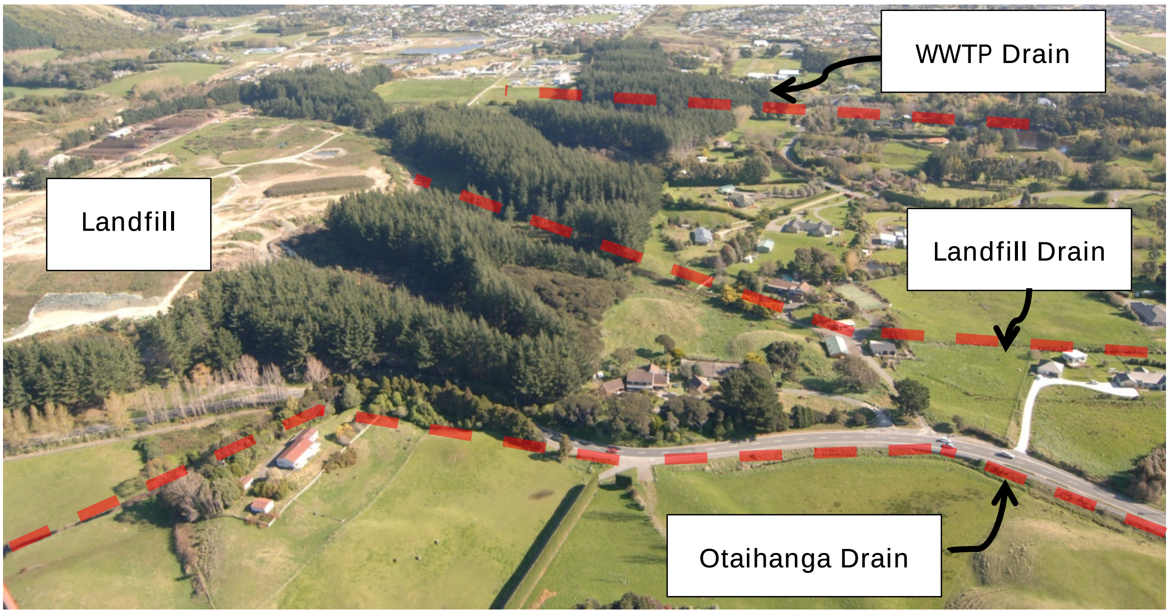


Photo 19: WWTP and Landfill Drains looking south - sand dunes are planted in pine trees and wetlands sit in the inter- dunal low areas.

3.4.3 Otaihanga Drain

North of Otaihanga Road there is a relatively small open channel farm drain that serves a small rural catchment. It passes through a KCDC culvert at the same location as Otaihanga Road passes through a gap in the sand dunes (refer photos 20 and 21). The drain ultimately outfalls into the Waikanae River, approximately 1.4km downstream of where the proposed Expressway will cross the Waikanae River.



Photos 20 and 21: Otaihanga Drain and Landfill Drain looking south (top) and the Otaihanga Drain upstream of KCDC's Otaihanga Road culvert (bottom).

3.4.4 Muaupoko Stream

The Muaupoko Stream meanders through farm land east of SH1 joining the Waikanae River at the point where the proposed Expressway will cross it (photo 22). It has a relatively large catchment (approximately 764ha) that stretches well into the hills east of SH1. The stream links several important ecological areas its route.

The stream is also part of the wider Waikanae River floodplain and frequently overtops its banks and floods the adjacent low lying farm land. The end of the stream forms an important part of a local community planting/restoration project along the true left bank of the Waikanae River. Photos 23 and 24 show views upstream and downstream of where the Muaupoko Stream is culverted under the existing Waikanae River walking track.

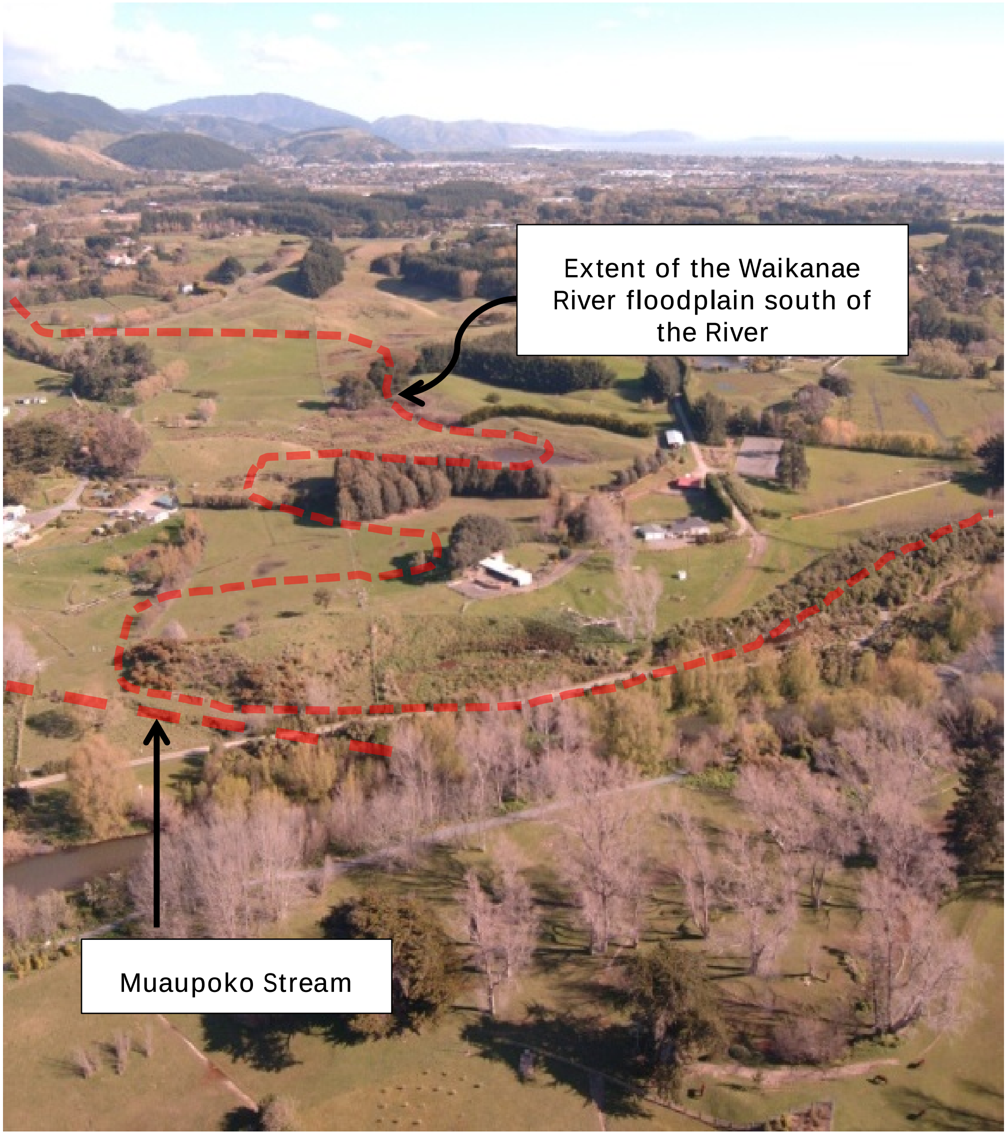


Photo 22: Muaupoko Stream and Waikanae River aerial view across the southern floodplain at the location of the proposed Expressway Waikanae River bridge.

Photos 23 and 24: Muaupoko Stream downstream (top) and upstream (bottom) taken from the walking track - native riparian planting can be seen downstream of the track contrasting with the rural features upstream.

3.4.5 Waikanae River and Floodplain

The Waikanae River is the largest watercourse along the proposed Expressway route (photos 22 and 25). It is a large gravel and sand bed river with a relatively mobile and (mostly) single-channel bed form. The river is actively managed by GWRC for flood and erosion protection purposes, and gravel is extracted from the bed for construction purposes. The area of the combined Waikanae and Muaupoko catchment upstream of the proposed Expressway is approximately 13,000ha of which most is located in the steep hills east of the existing SH1. The Kāpiti Coast water supply draws from the Waikanae River, upstream of SH1.

The large steep upper catchment is mostly covered in native bush but also has significant areas of pasture and forestry. The lower reaches of the river pass through farm land and alongside the southern edge of Waikanae township. With such a large catchment comes large flood flows during extreme storms. These floods rise and fall relatively rapidly due to the combination of a steep upper catchment and a flat lower catchment. The Waikanae River has a very large floodplain both upstream and downstream of the proposed Expressway. However, the proposed Expressway crosses just downstream of a relatively narrow (150m wide) point on the floodplain formed by localised promontories in the topography on both left and right banks of the river.

Photo 25: Waikanae River downstream of the proposed Expressway crossing.

GWRC is responsible for managing the River, and it is progressively constructing flood and river management works, including:

- river channel realignments;
- riverbank armouring (photo 26);
- willow and native riparian planting;
- grade control weirs;
- groynes (photo 26);
- stopbanks (photo 27); and
- contributing to the cost of raising flood prone houses[17].

These works are being carried out under a formal river management plan[18] and an associated environmental plan[19], both of which are currently under review by GWRC.

As part of its flood management, GWRC has an existing preferred river channel alignment that is 35m wide with an additional 20m wide vegetated buffer zone on each side. The existing banks are often well within this design alignment as is the case at the proposed Expressway crossing where the channel is only approximately 16m wide (refer photo 28). GWRC's policy is to generally maintain the main river channel so that it is kept within their preferred corridor, i.e. if the river breaks out from the design alignment GWRC will act to repair the River back into the corridor.

While the land on the south side of the Waikanae River is mainly rural, the north has residential development of Waikanae township encroaching well onto the floodplain, including across an historical overflow path towards the Waimeha Stream. GWRC has responded by constructing a line of stopbank protection works. This has been carried out in a staged manner over the past several decades. The last significant works was completed in 1997 in the reach immediately upstream of the proposed Expressway bridge. This involved constructing the Greenaway Road stopbank, construction of several groynes and a significant realignment of the river channel to remove an abrupt bend in the river. The old channel was then converted into an oxbow wetland that is now regionally significant for ecological reasons.

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[17] Waikanae Floodplain Management Plan 10 Year Review, GWRC, May 2010.

[18] Waikanae Floodplain Management Plan, GWRC, 1997.

[19] Waikanae River Environmental Strategy, GWRC and KCDC, March 1999.

Photo 26: A GWRC construction groyne in the Waikanae River upstream of the proposed Expressway crossing. Rock armour to the bank can be seen in the background.

Construction of the stopbank has required KCDC to construct a stormwater pump station to serve the Kauri/Puriri Road residential area. This area is significantly lower than the 1% AEP flood level (3.5 to 4m RL ground level compared to a flood level in the river of approximately 5m RL). When the Waikanae River is in flood, the Kauri/Puriri Road area cannot drain under gravity. The pump station was installed to lift local stormwater over the stopbank and into the river.

As noted above, the gradient of the river flattens out as it crosses the coastal lowlands. This causes the river to deposit sediment in the main channel and, during floods, finer sediment on the floodplain. GWRC has resource consent to annually remove 10,000m^3 of river gravel; however, recently this has not been permitted in the normally wet areas of the river channel.

Photo 27: GWRC's Puriri /Greenaway Road stopbank upstream of the proposed Expressway crossing. KCDC's pump station outlet can be seen in the centre of the photo.

Photo 28: Waikanae River at the approximate location of the proposed Expressway crossing.

The river discharges at the coast through a small estuary formed behind a coastal sand bar that develops from the southwards littoral drift of sand along the coast. This forces the river to bend sharply to the south prior to outfalling to the sea. At times of high flood, the Waikanae River cuts through the sand bar taking a direct route the sea. Generally, GWRC excavates the sand bar out every 5 years to control the migration of the mouth and realign the outfall to this more favourable condition, termed “short mouth”. The longer route around the sand bar is called the “long mouth”, both of which have been modelled by GWRC.

GWRC’s modelling has shown that flooding from the river near the proposed crossing is not significantly affected by high tides.

3.4.6 Te Moana Floodway

Before the stopbanks were constructed, the Waikanae River overflowed to the north during floods. This overflow came from several locations along the river but the topography of the land to the north guided the flows towards the Waimeha Stream and Te Moana Road. The KCDC District Plan includes a protected floodway from the Waikanae River to Te Moana Road termed “residual overflow” as this is still the route floodwater would take if the stopbank were to be overtopped or breached (refer Figure 1).

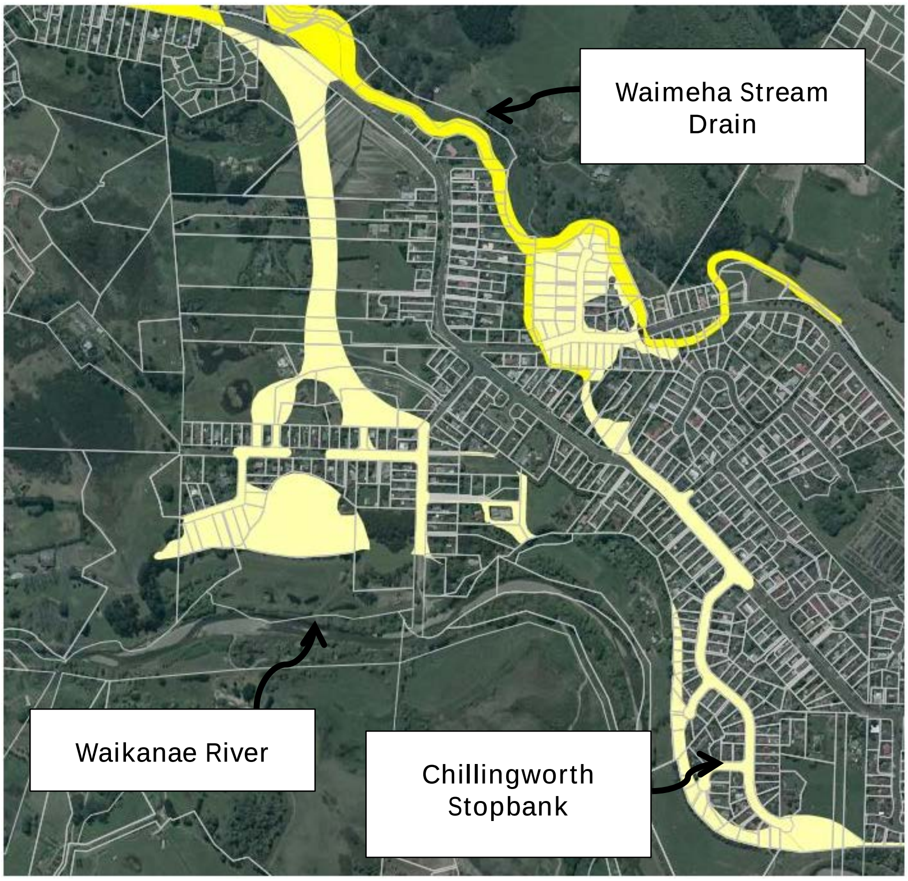

Figure 1: Overflow routes as per the KCDC District Plan (screen capture of KCDC’s online GIS). The overflow route is shown in yellow running north from the river across Te Moana Road.

Recent investigations with GWRC, indicate that the critical location for a stopbank breach is at the Chillingworth stopbank (refer Figure 1). A breach in the stopbank at this location would flow north through residential areas arriving at Te Moana Road near to the location of the proposed Expressway crossing.

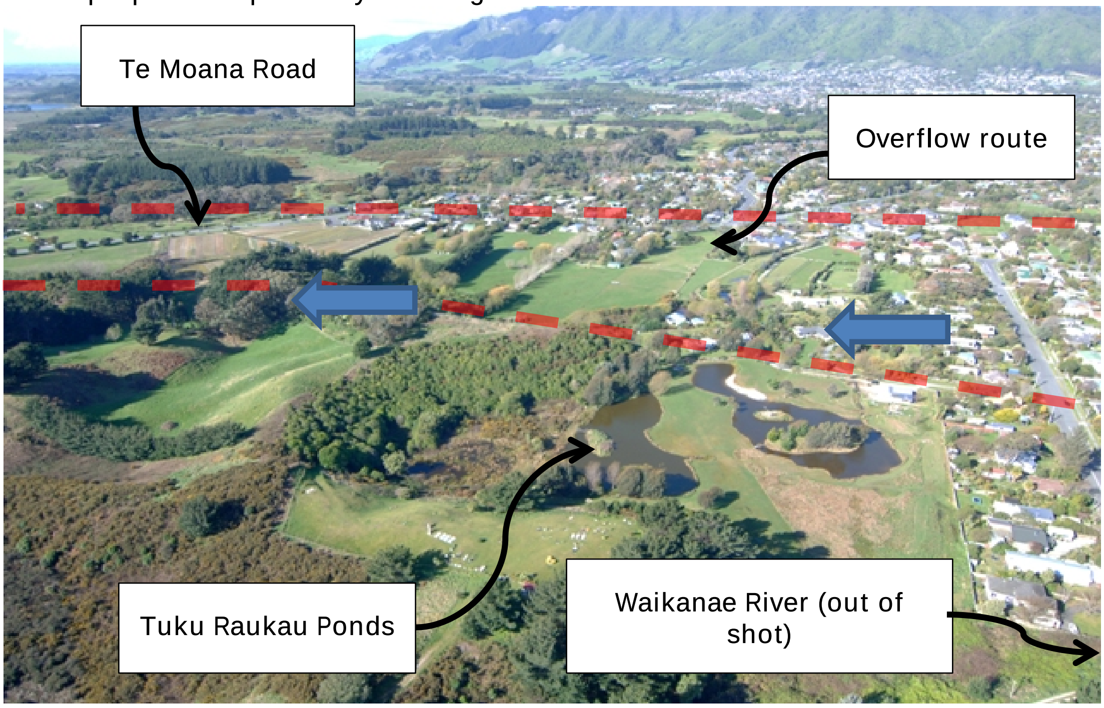

Photo 29: Approximate route of Waikanae River overflow.

3.4.7 Wetlands North of the Waikanae River

There are several significant wetlands just north of the Waikanae River that form a wider complex of wetlands located both upstream and downstream of the proposed Expressway river crossing. Some of these have formed in old river oxbows and depressions on the floodplain. For further details on these wetlands refer to Technical Reports 26 and 27, Volume 3.

To the immediate west of the proposed Expressway is the large El Rancho wetland that has no formal surface drainage outlet and is isolated in a pocket within the sand dunes (refer Figure 8c, within Technical Report 26, Volume 3).

The proposed Expressway will cross below and immediately east of the Takamore Urupa crossing the edges of two modified wetlands known as the Tuku Raukau ponds (refer photos 29, 30 and 31). These ponds are residuals of larger historical wetlands that have been highly modified by landowners so that now only small remnants remain, particularly in a pocket of land below the Urupa. At one time these wetlands would have been part of a far larger wetland complex that covered much of the Kāpiti Coast before being drained and developed (refer Technical Report 27, Volume 3).

Also, the main north-south bulk gas main separates each of the Tuku Raukau ponds as it runs across the Waikanae floodplain.

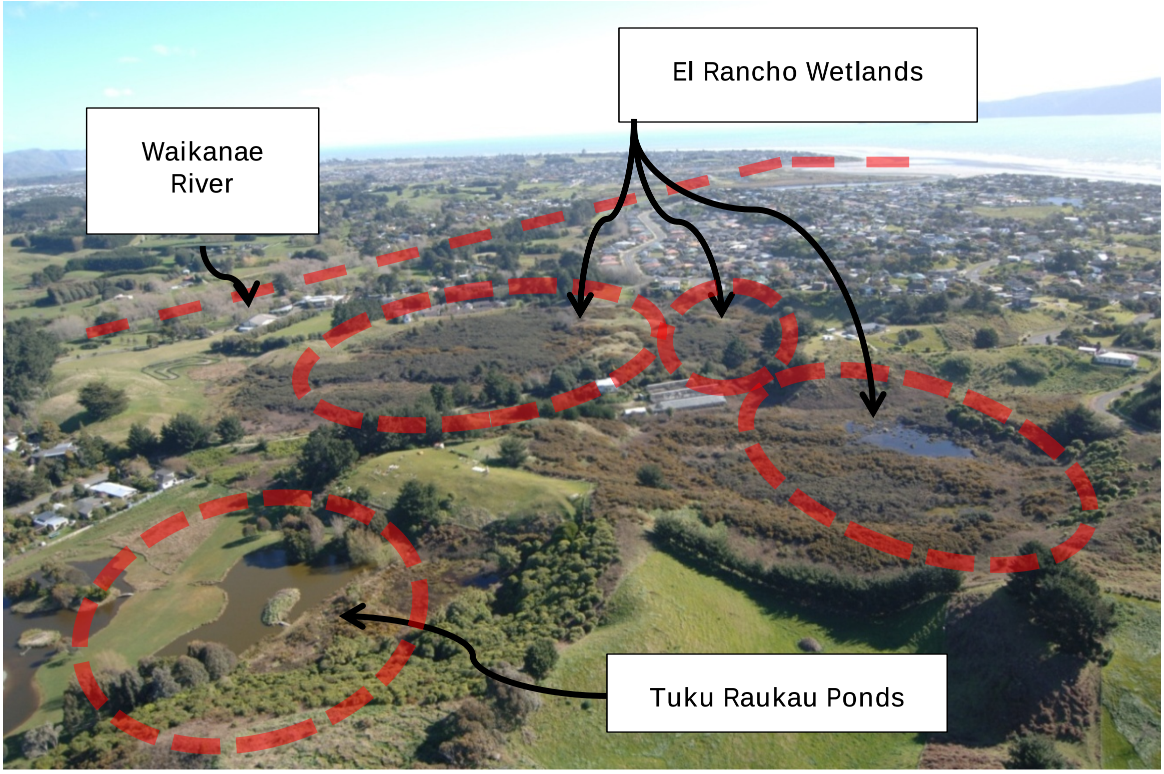

Photo 30: Wetlands along the north bank of the Waikanae River.

Photo 31: Tuku Raukau ponds looking north with the remnant wetland in the background across the other side of the pond.

3.4.8 Waimeha Stream

The Waimeha Stream (photos 32 to 36) has a catchment area of 219ha upstream of the proposed Expressway and is fed by a mixture of natural springs and discharge from part of Waikanae's town drainage network. From its source in the Waikanae Domain the stream follows a gentle gradient to the sea[20] separated from Te Harakeke/Kawakahia wetland by an area of sand dunes.

European settlement and subsequent land development has significantly altered the original course of the Waimeha Stream. Figure 2[21] shows that in 1873 the Waimeha Stream was not separate from the Waikanae River and it was later land development that has made it a separate stream with its own outfall to the sea.

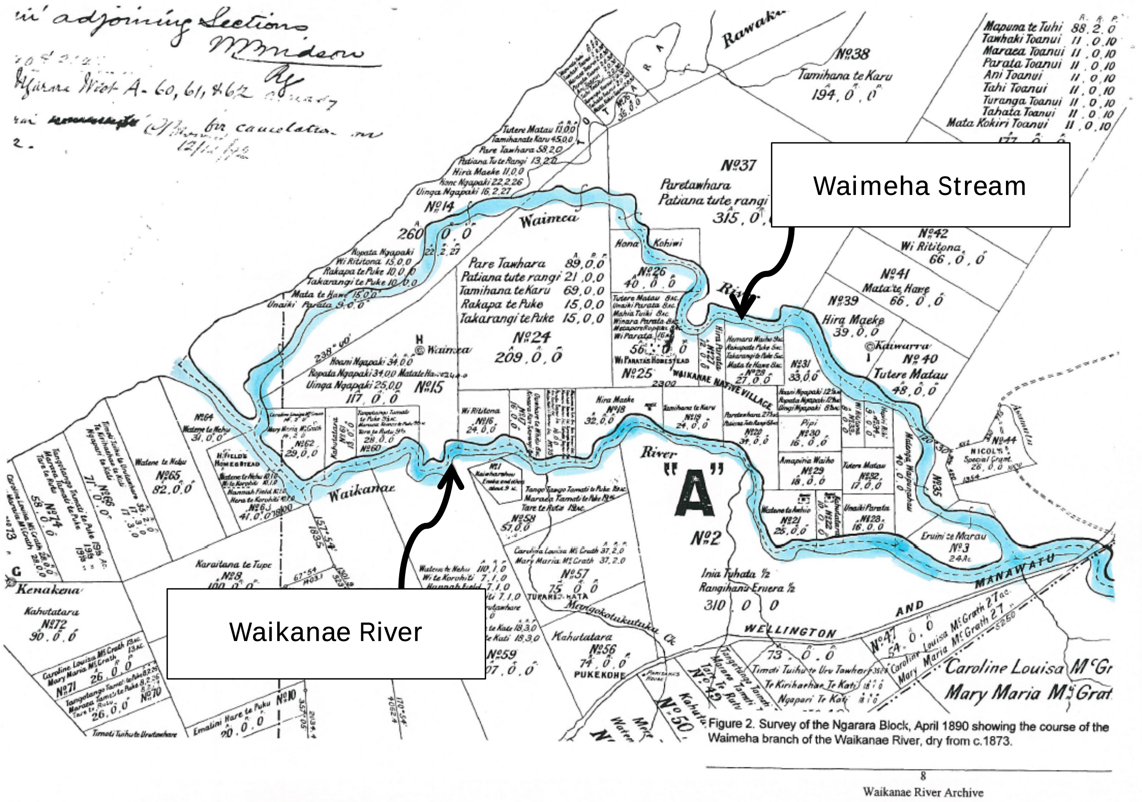

Figure 2: Waikanae River alignment in 1873 showing the Waimeha Stream as an arm of the river and not a separate watercourse that it is today.

Further downstream the Waimeha Stream joins with the Ngarara Stream prior to outfalling to the sea. The Waimeha is very low lying and tidally affected as far up as the proposed Expressway crossing. There is also a constraint on flow capacity at the Field Way Road Bridge just upstream of the sea outfall.

A small catchment south of Te Moana Road also drains into the Waimeha Stream via an open channel drain (photo 33) and a culvert under Te Moana Road. Part of this catchment is currently used for market gardening.

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[20] Waikanae Flood Hazard Mapping, Volume 1, SKM, March 2010.

[21] Waikanae River Archive, Rivers Department, Wellington Regional Council, 1991.

A feature of the stream is its clear spring water, yet only part of it has good native riparian cover. The proposed Expressway will cross the stream where is passes through paddocks and is generally overgrown with weeds. It is maintained with an excavator as required.

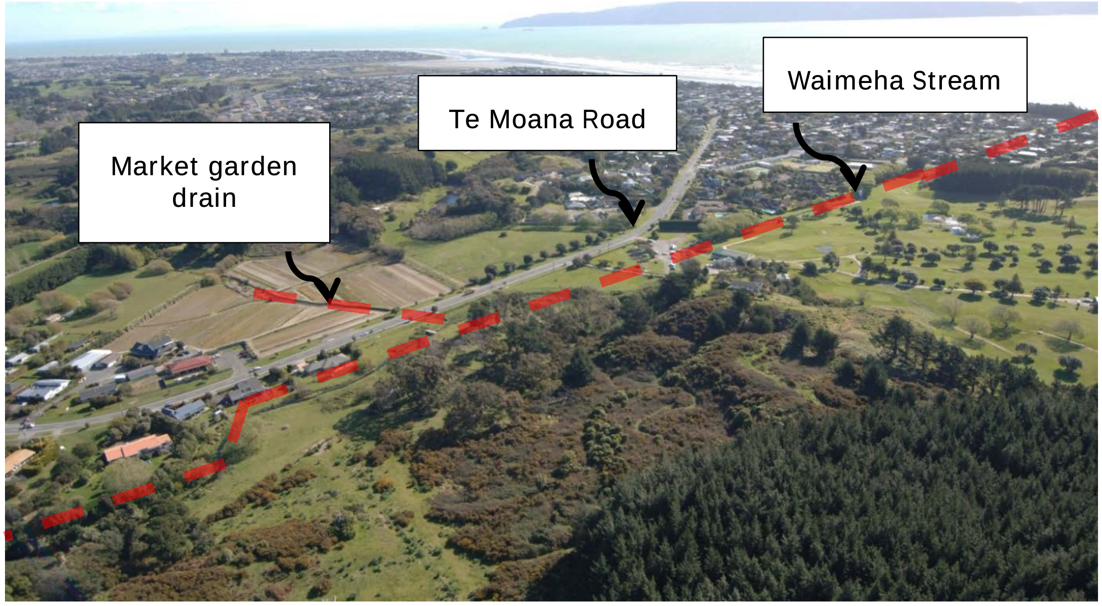


Photo 32: Aerial looking south across the Waimeha Stream.

Photo 33: Highly modified drain in the market garden area looking south from Te Moana Road.

Photo 34, 35 and 36: Upper reach of the Waimeha Stream with native riparian planting (top). Looking upstream (middle) and downstream (bottom) of the proposed Expressway crossing.

3.4.9 Isolated Catchment at Ch12100m

North of the Waimeha Stream is a small catchment (between chainages 12100m and 12300m) that has no formal surface drainage outlet. It is an isolated basin in the sand dunes and any stormwater that collects in this area soaks into the ground over time.

3.5 Sector 4 – Waikanae North

This Sector covers from north of the Waimeha Stream to Peka Peka Road (chainage 12,400 to 18,050m). The key stormwater features in this Sector include:

- Ngarara Stream and Te Harakeke/Kawakahia wetland;
- Ngarara Creek;
- Kakariki Stream and floodplain;
- Smithfield Drain;
- Paetawa Drain and floodplain; and
- Hadfields / Te Kowhai Stream.

3.5.1 Ngarara Stream and Te Harakeke/Kawakahia Wetland

The land between the Waimeha Stream and Smithfield Road is characterised by high dunes and hollows that presents folded and uneven topography. Many of the inter-dunal hollows have no clear drainage path leaving runoff to slowly soak away into the ground and contribute to forming wetlands.

North of Smithfield Road the returns to peat flat flood plains all the way to Peka Peak. The majority of this area is currently farmed thus most of watercourses have been highly modified and channelised as part of making the land suitable for farming.

This sector contains Te Harakeke/Kawakahia wetland (photo 37), the largest most ecologically significant wetland in the region. Refer Technical Reports 26 and 27, Volume 3 for more detail and descriptions of the ecology of this wetland. All of the watercourses between the Waimeha Stream and Peka Peka Road drain into Te Harakeke/Kawakahia wetland. The largest of these are the Ngarara Creek, Kakariki Stream and the Ngarara Stream (of which the Paetawa Drain the main tributary).

Ngarara Stream continues through Te Harakeke/Kawakahia and joins with the Waimeha Stream prior to its outfall to the sea. It has many smaller tributaries that cross the proposed Expressway Alignment. The larger ones are summarised in the following sections.

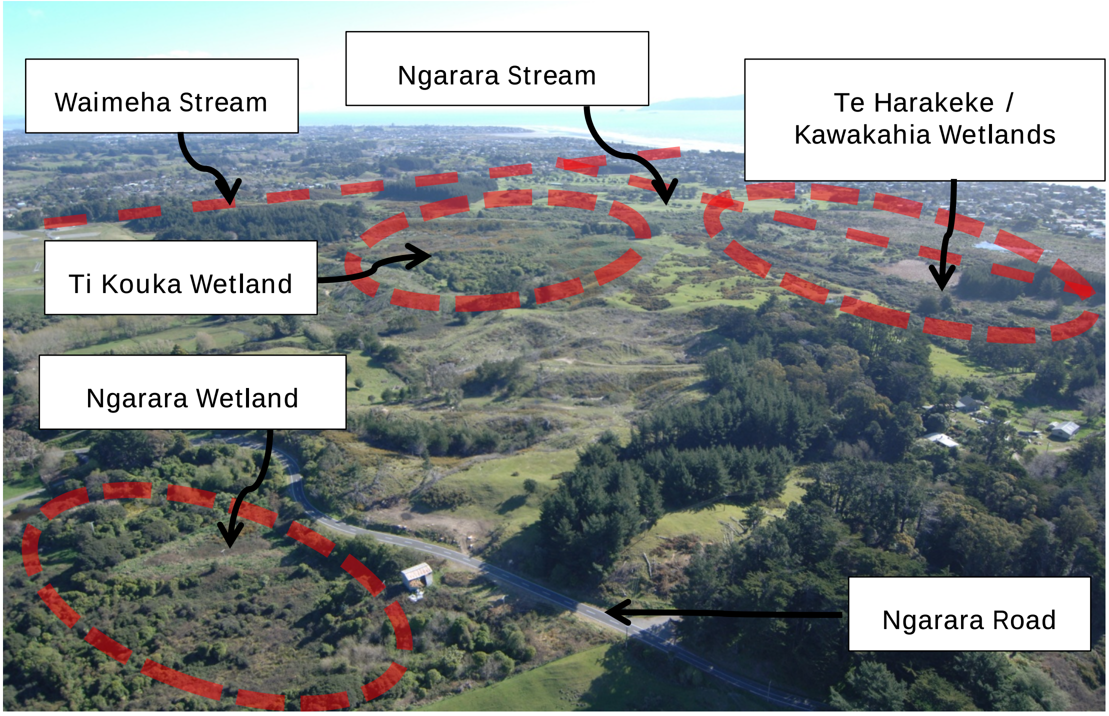


Photo 37: Te Harakeke/Kawakahia wetland complex.

3.5.2 Ngarara Creek

Ngarara Creek (photo 38) is a small watercourse tributary of the Ngarara Stream that drains part of Waikanae township and its lower reaches pass through rural pastoral land. The proposed Expressway will cross the creek where it runs through some forestry blocks prior to discharging into Te Harakeke/Kawakahia wetland.

Photo 39: Ngarara Creek at the proposed Expressway crossing location, looking upstream.

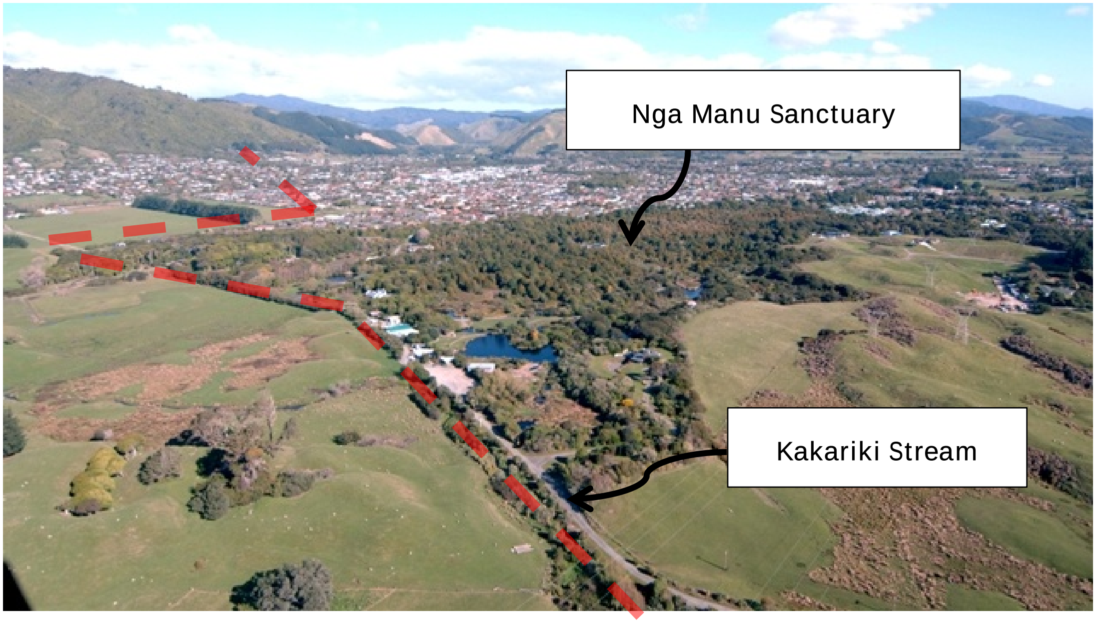
3.5.3 Kakariki Stream and Floodplain

The Kakariki Stream runs from the steep slopes of the hills west of SH1, through Waikanae township (in a section of piped drain) before passing Nga Manu Bird Sanctuary (photo 40) and flowing into Te Harakeke/Kawakahia wetland. The catchment above the proposed Expressway crossing is 618ha in area with native bush covering the upper catchment and a mix of urban and rural land in the lower catchment.

At the location of the proposed Expressway, the stream is deeply incised with partially restored native riparian vegetation that is now mostly overgrown with weeds (photos 41 and 42). The planting was carried by Nga Manu Sanctuary staff and local community interest groups.

The Kakariki Stream forms an important part of an ecological corridor being developed by the community to enhance the habitat linkages from Kāpiti Island to Te Harakeke and Nga Manu to the Tararua Ranges beyond.

The pastoral land between Ngarara Road and Nga Manu on the true right bank of the Kakariki Stream is low lying and prone to flooding (refer photo 42). Nga Manu's access is also regularly cut off by flooding from the Kakariki Stream.[22]



Photos 40: Nga Manu Bird Sanctuary and Kakariki Stream.

[22] As advised by Nga Manu Bird Sanctuary Trust.

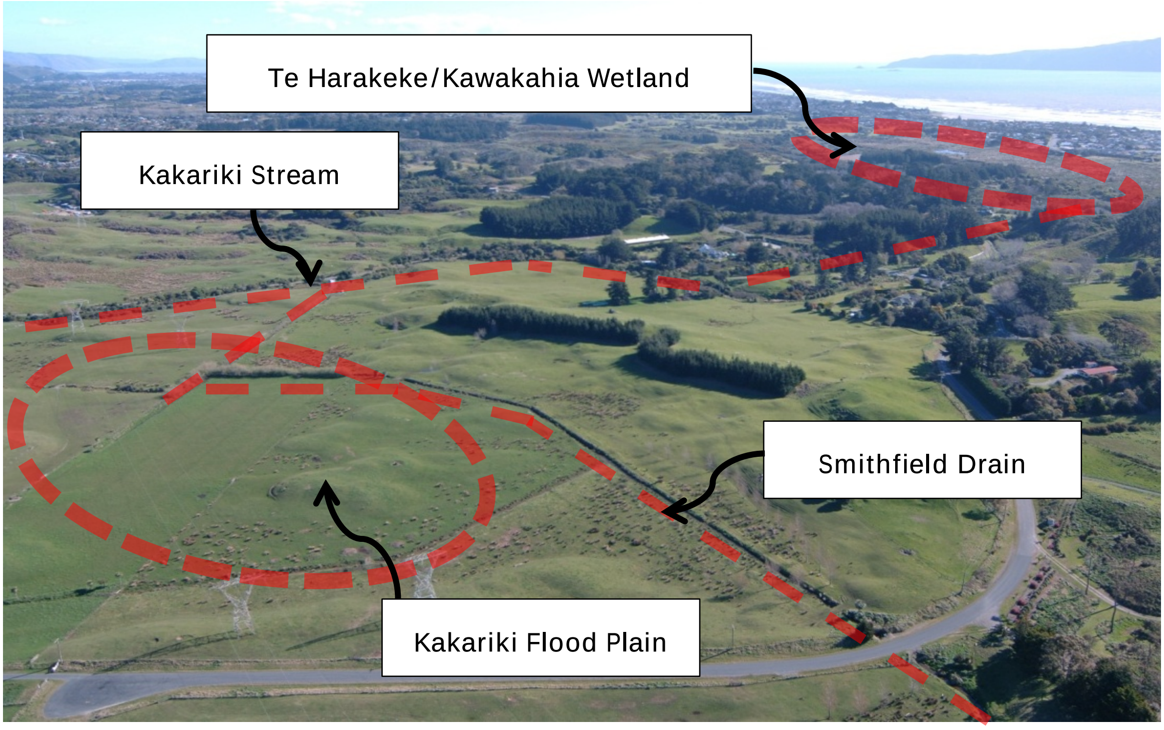


Photo 41: Kakariki Stream, Smithfield Drain and associated floodplain.

Photo 42: Kakariki Stream looking downstream from the access bridge off of the Nga Manu Sanctuary access road. Note the overgrown riparian planting.

Photo 43: Kakariki Stream looking upstream from an access bridge off of the Nga Manu Sanctuary access road.

3.5.4 Smithfield Drain

The Smithfield Drain is a significant tributary of the Kakariki Stream and has been significantly modified by farming activities. The lower end to the drain is maintained by KCDC. It drains a small rural pastoral catchment and is generally overgrown with grasses and weeds (refer photo 44, 45 and 46). The drain generally runs parallel to the proposed Expressway, with a substantial length that lies directly under the proposed Expressway footprint.

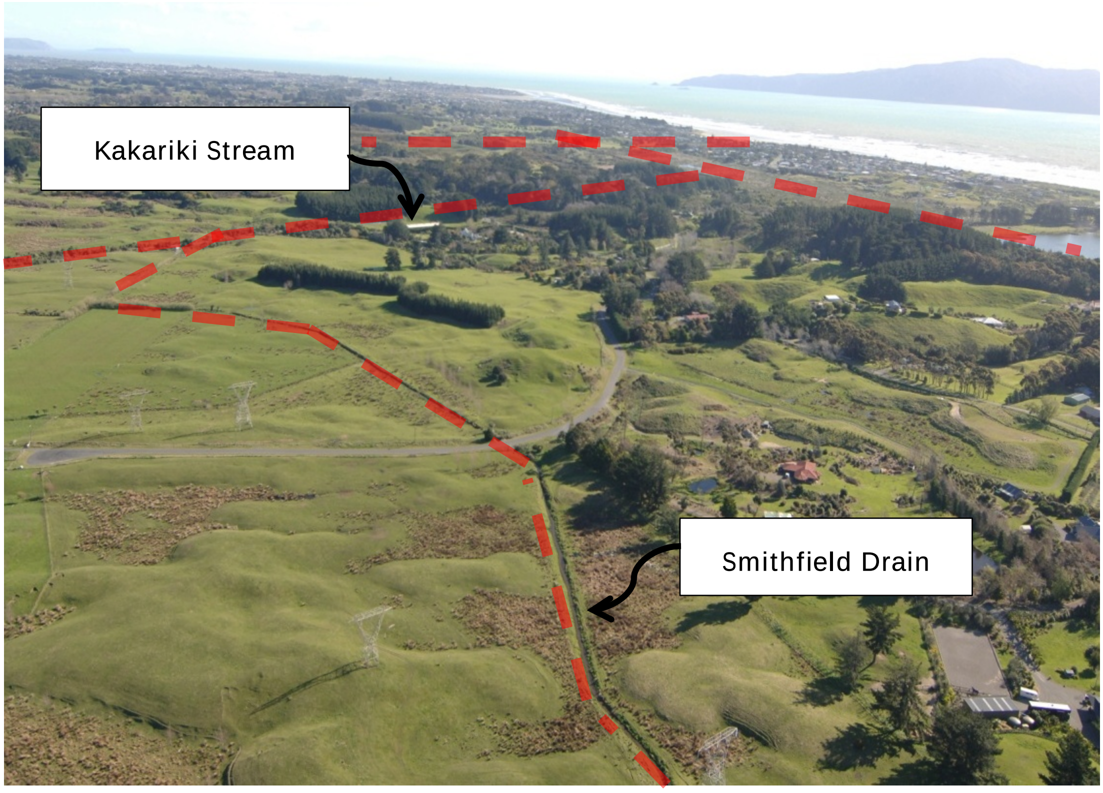


Photo 44: Smithfield Drain. The section that will be directly beneath the proposed Expressway is in the foreground to each side of the road.

Photos 45 and 46: Smithfield Drain at Smithfield Road looking upstream (top) and the confluence of the Smithfield Drain and Kakariki Stream (bottom).

3.5.5 Paetawa Drain and Floodplain

The Paetawa Drain is a large tributary of the Ngarara Stream and it is the main drain for the peat flats south of Peka Peka Road (photos 47, 48 and 49). Its catchment above the proposed Expressway is 408ha in area (including various smaller tributaries). The catchment includes large areas of pasture and also steep hillside areas east of SH1.

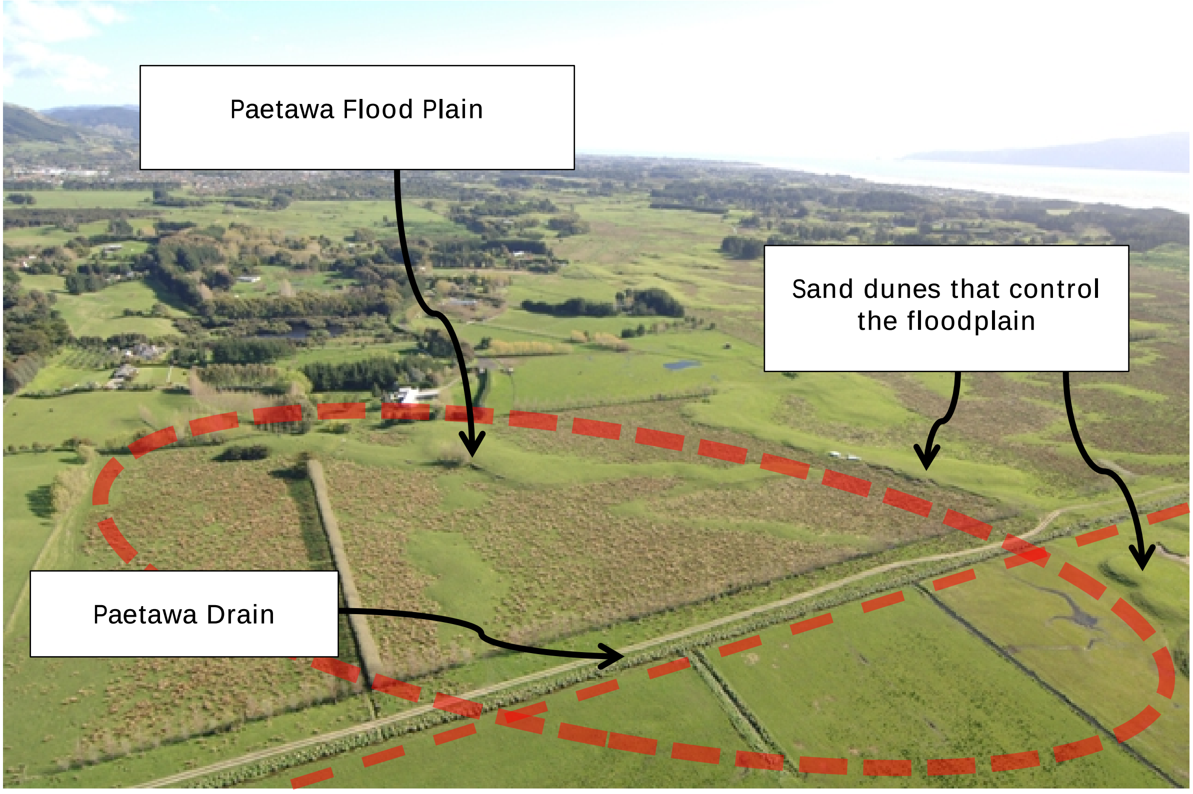


Photo 47, 48 and 49: Paetawa Drain and low lying floodplain (top) and typical sections of the drain near the proposed Expressway Alignment (bottom).

The Paetawa Drain has many farm drain tributaries (photo 50) of its own which have been highly modified and straightened by past farming practices. Downstream the existing SH1 many of these tributaries consolidate before reaching the proposed Expressway.

Much of the low lying land on either side of the Paetawa Drain becomes inundated during floods and stores large volumes of ponded floodwater. While not zoned as flood storage in the District Plan, this function serves to limit flooding of land further downstream in the catchment. The Paetawa Drain passes through a gap in a local dune formation that acts as a flood control, holding back floodwater and creating the floodplain (photo 47).

Photo 50: Typical Paetawa Drain tributary.

3.5.6 Hadfield / Te Kowhai Stream

The Hadfield / Te Kowhai Stream (photos 51, 52 and 53) serves a moderately sized steep hillside catchment east of SH1. The catchment is approximately 90ha in area upstream of the proposed Expressway.

The culverts that pass the stream under the railway and SH1 are reported by NZTA to be under capacity and subject to significant gravel deposition in their inverts.

The stream runs west through rural/pastoral land to outfall at the coast to the near Peka Peka beach settlement. It does not feed into Te Harakeke/Kawakahia wetland.

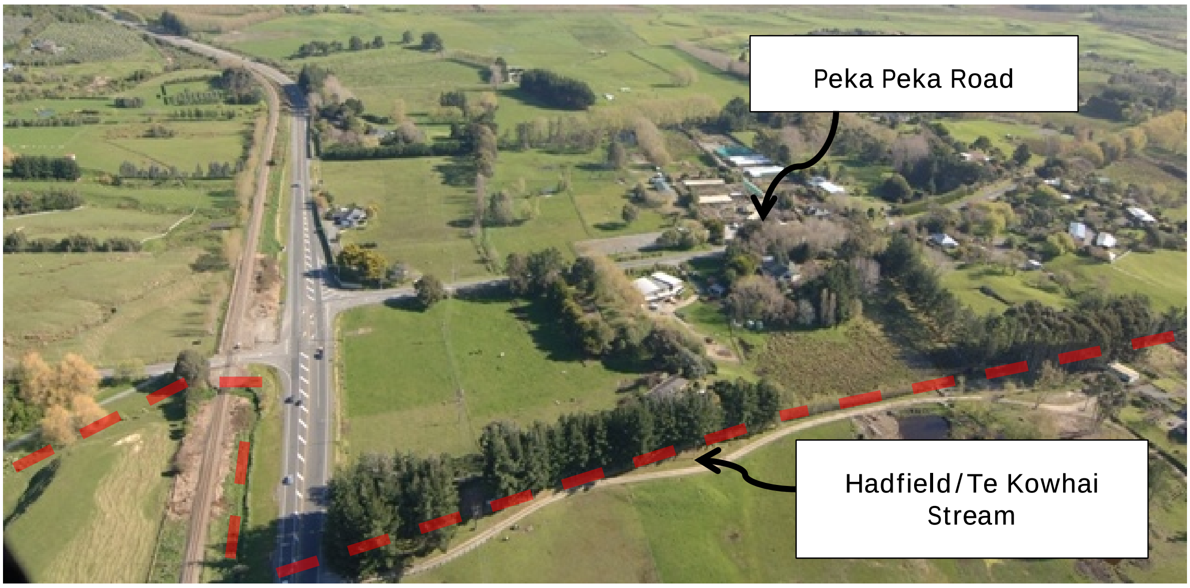


Photo 51: Aerial of Hadfield/Te Kowhai Stream crossing SH1.

Photos 52 and 53: Hadfield/Te Kowhai Stream downstream, showing the SH1 culvert outlet (left) and a typical section of the stream (right).

4 Project description

4.1 Project Overview

For the overall Project description refer to Part D, Chapters 7 and 8, Volume 2.

A general description of the stormwater, flood risk management and bridge/culvert waterway aspects of the proposed Expressway design is provided in the following section. These elements are relevant for the entire length of the proposed Expressway. Sections 3.3 to 3.6 provide Sector specific descriptions of the proposed design.

Refer to the Drainage Layout drawings CV-SW-100 through 132, Technical Report Appendices, Report 22, Volume 5 and the schedule of watercourse crossings, wetlands, flood storage areas and watercourse diversions included in Appendices 22.A and 22.B.

4.2 Design Approach

This section summarises the design approach used for the proposed Expressway. It is based on the application of a set of key principles and standards that have been determined in agreement with KCDC. These are in turn based on various industry best practice standards and guidelines used in hydrological analysis, stormwater design and effects mitigation.

For management of stormwater during construction refer Technical Report 4, Volume 3 and Appendix H of the CEMP, Volume 4.

4.2.1 Standards and Guidelines

The key design standards and guidelines are:

- Stormwater Treatment Standard for State Highway Infrastructure, 2010, NZTA;
- Bridge Manual, 2003, Transit NZ;
- Alliance/KCDC Guiding Objectives, 2010[23];
- Kāpiti Stormwater Management Strategy, KCDC; and
- Austroads Guidelines for the Collection and Discharge of Stormwater from Road Infrastructure, 1994, ARRB.

4.2.2 Key Principles

The key principles from these documents include the following items (there are further more detailed criteria set out in the Project's Design Philosophy Statement[24]):

- Attenuate peak flows from the proposed Expressway to avoid increasing flooding to adjacent land i.e. part of achieving hydraulic neutrality;
- Treat stormwater from the proposed Expressway to best practicable option (BPO) standard for contaminant removal before discharge to existing drainage systems;
- Provide offset to lost floodplain storage taken up by the proposed Expressway in order to avoid flooding effects on adjacent land (an aspect of hydraulic neutrality);

[23] Refer to Part A, Chapter 2, Volume 2.

[24] Refer to Technical Report 1, Volume 3.

- The stormwater treatment, attenuation and offset storage areas are integral with the operation of the proposed Expressway and as such these areas will be within the final proposed Expressway Designation;
- Aim to keep the permanent water level inside of wetlands within the existing groundwater seasonal range to avoid effecting areas beyond the wetland as a result of lowering/raising groundwater levels;
- Keep the proposed Expressway carriageway 0.5m above the 1% AEP flood level;
- Bridges to pass a 1% AEP design flood with appropriate freeboard in accordance with the Bridge Manual and GWRC requirements , with a sensitivity check for performance in 1.5 times the 1% AEP flow;
- Culverts to pass a 10% AEP flow with the head water not being above the pipe soffit (note, in some low-lying areas with very flat gradients this criterion may be relaxed to suit site conditions);
- Culverts to pass a 1% AEP flow with heading up limited to no more than 2m depth above the pipe soffit and at least 0.5m below road level (whichever level is lower)[25];
- Culverts to accommodate fish passage by either setting the culvert below the existing watercourse bed level and/or placing gravel or equivalent bed forms through the invert of the culvert to create a low flow channel; and
- Allowance for the effects of climate change out to 2090, to accommodate 16% increase in rainfall intensity, and 0.8m sea level rise.

4.2.3 Methodology

Stormwater assessment and design for the proposed Expressway falls into three broad components:

i. Hydrology – rainfall, catchments and runoff;
ii. Hydraulics – flow, velocity, water levels and pipe sizes; and
iii. Water Quality – treatment of runoff.

The design process starts with the hydrology which involves investigating and assessing catchments, their characteristics, determining rainfall intensities and calculating subsequent runoff quantities (flow rate and volume).

Once the hydrology of a catchment has been determined and the design flows understood, then the hydraulic designs can be investigated. This includes determining flood levels and water depths, sizing of structures like bridges and culverts and investigating how these behave under various scenarios.

Water quality is a function of catchment land use and the sources of contaminants within each catchment. Road, residential, commercial, industrial and rural catchments each tend to have characteristic water quality issues. For example, road catchments tend to contain more hydrocarbon and heavy metal contaminants than rural catchments that typically have higher nutrient loadings. Refer to Technical Reports 24 and 25, Volume 3 for further details on the water quality of the existing watercourses, the highway runoff, and the net effects after treatment.

[25] It is noted that this is a standard NZTA criterion that is unlikely to be triggered on this site as it is overridden by the need to avoid increases in flood levels. It would only govern where a culvert was set very low relative to existing flood levels.

The key methodologies applied to each facet of the design are:

i. *Where available, use existing KCDC and GWRC hydrological and hydraulic models to:*

- determine design flows;
- determine pre and post Expressway flood levels (in both floodplains and watercourses);
- confirm culvert/bridge waterway sizing;
- determine the effects of proposed Expressway discharges and the efficacy of proposed Expressway peak flow attenuation;
- determine the effects partially filling in floodplains and size any subsequent offset storage; and
- confirm the adequacy of proposed mitigation measures.

Both KCDC and GWRC have prepared and maintain hydrological and hydraulic stormwater models for their own management purposes. GWRC manages the Waikanae River, while KCDC manages the other watercourses along the proposed Expressway route.

The key design storms that have been modelled (including climate change as set out later) are:

- 10% AEP;
- 1% AEP;
- 1.5 x 1% AEP; and
- 0.04% AEP (Waikanae River only).

The 1.5x1% AEP storm is KCDC's standard method for testing overdesign events, while the 0.04% AEP storm is used in the structural design of bridges in accordance with the Bridge Manual. The modelling discussed in more detail in the following sections, examines the implications of overland flow path blockage effects up to the 1%AEP storm. While this has not specifically tested culvert blockage, an equivalent method has been used to examine the sensitivity during overdesign events i.e. instead of reducing culvert capacity as would occur during a blockage, the flows have been increased to 1.5x1%AEP with the results experienced by the land upstream being essentially the same. As blockage is considered to be an overdesign event, the effects are investigated such that they are understood but they are not mitigated for.

Both of KCDC's and GWRC's incumbent modelling consultants, SKM and River Edge Consulting (REC), have been engaged by the Project team to modify these models to include the proposed Expressway and then test the stormwater designs. This means that the majority of the stormwater catchments and associated hydrology, including climate change, have already been well investigated and the models calibrated by KCDC and/or GWRC. This is considered to be the most accurate and efficient method of investigating the effects of the proposed Expressway on stormwater and flood risk for areas both upstream and downstream of the proposed Expressway.

SKM are responsible for models that cover the Wharemauku Stream / Drain 7 / Waimeha Stream / Paetawa Drain catchments and REC for the Mazengarb and Waikanae River catchments. SKM's and REC's modelling reports are included in Appendix 22.E, 22.F and 22.G and the results of these reports are summarised and discussed further in Sections 3.3 to 3.6 of this report. In some cases the modelling has been completed and the design has then needed to include additional works as a response to the modelling. Therefore, while the flood levels reported in this Report match those of the modelling reports, the flood levels shown on the drawings include the effects of these additional mitigation works. Hence are some differences between the drawings and the reports. Also the modelling reports generally look at water levels at culverts whereas the levels quoted in this report is intended to examine effects more widely so both upstream and downstream levels have been reported. Therefore it is not that the levels are different but the locations are different.

While KCDC and GWRC have primarily used these models to set building floor levels and quantify flood risk, the Project team's use for the models has been slightly different in that the Project team is primarily interested in understanding the effect of the proposed Expressway on existing flood levels, determining the extent of any consequences and developing mitigation as required. In this respect, it is the relative difference between the pre-Expressway (existing) and post-Expressway (after) that the Project team needs to understand rather than absolute flood levels. This is why modelled water surface levels rather than with freeboard added (as per KCDC and GWRC practice) is the appropriate method. Appropriate freeboard is then added on a site-specific basis to meet NZTA's standard at culverts and bridges.

The areas around Poplar Avenue and north of Peka Peka Road are not covered by existing KCDC models. The hydrology and hydraulic designs have been carried out by applying KCDC's standard design methods. The catchment hydrology has been compared against similar adjacent catchments that are covered by a model.

The models determine the appropriate culvert or bridge watercourse sizing. However, prior to modelling a crossing an initial determination was made as to whether a crossing should be a pipe culvert, box culvert or a bridge. Obviously bridges are more expensive than culverts and this is particularly when compared to pipe culverts. So only the main watercourses (both in terms of size and ecological value) warrant this consideration. Selection of crossing type depended on:

- the size of the water course;
- the sensitivity of the flood levels in the area;
- floodplain issues;
- ecological significance of the watercourse;
- existing drainage (i.e. is there a culvert there already);
- existing topography; and
- economics of the structural types.

ii. *All design storms will include mid-range climate change effects estimates out to 2090.*

This is in accordance with the recommendations of the MfE guidelines.[26] In

[26] Climate Change Effects and Impacts Assessment. A Guidance Manual for Local Government in New Zealand. 2nd Edition. Ministry for the Environment, May 2008.

addition, sea level rise has been set in accordance with KCDC guidelines, rather than the slightly lower value recommended by MfE.

These parameters are to mid-estimate range of:

- 16% increase in rainfall intensity; and
- 0.8m rise in sea level.

Long-term groundwater rise from climate change is not expected to affect the functionality of swales and wetlands as these will have positive gravity outlets that set and maintain water levels during dry and wet periods, including taking account of any increased groundwater levels.

The MfE guidelines also recommend that more detailed investigations be carried out if significant climate change impacts are indicated. KCDC commissioned NIWA to carry out a specific climate change study for the Kāpiti District in 2005[27] which they subsequently updated in 2007[28]. KCDC then aligned their design rainfall information in 2008[29]. These design rainfalls and the associated climate change assumptions are an integral component of KCDC's stormwater models that are used in the design of the proposed Expressway. The design rainfalls isohyets are shown on Drawings CV-SW-050 through 052, Technical Report Appendices, Report 22, Volume 5.

iii. *Mitigate for increased runoff from the proposed Expressway by providing attenuation – i.e. part of being hydraulically neutral.*

"Hydraulic neutrality" is an often misunderstood term and its most theoretical interpretation involves not changing peak flow rates, total volume discharged and the timing of that discharge. This is nearly impossible to achieve in practice so KCDC applies a pragmatic interpretation for developments in the Kāpiti District that recognised the volume discharged does increase, and therefore the downstream system must be able to handle this. The Council's interpretation covers development not discharging at greater than the existing peak flows or not to cause a significant increase in flood levels i.e. whether by in filling of floodplain storage, increased peak discharge or in the sizing of culverts/bridges. In order to avoid confusion, we have deliberately agreed with KCDC to adopt their interpretation given the proposed Expressway crosses land under KCDC's jurisdiction.

Hydraulic neutrality is generally provided by swales, wetlands or flood storage areas to suit local topography. In some areas, a mix of all three elements is needed to achieve the required attenuation, but swales are preferred over wetlands due to simplicity and ease of maintenance. During later design stages the swales will be optimised further with a view to reducing the number of wetlands and their footprint, while still achieving the required performance.

[27] Kāpiti Coast Ground Water and Ponding, NIWA, 2005.

[28] Updated Climate Change Scenarios for Kāpiti Coast, NIWA, Nov 2007.

[29] Update of Kāpiti Coast Hydrometric Analysis, SKM, April 2008.

The proposed Expressway catchments were modelled using InfoWorks CS software. The models were set up in accordance with NZTA's standard for attenuation for the 50% AEP, 10% AEP and 1% AEP events. The 10% and 1% AEP discharge hydrographs from the swales and wetlands were then used as an input to SKM/REC's models to test the effects on flooding in receiving watercourses. Refer to the report *Expressway Stormwater Attenuation Modelling*, Beca, 2011 included in Appendix 22.D for further detail and results of the proposed Expressway runoff modelling.

Where flooding is a known issue in downstream areas then attenuation was set so that discharges were 80% of pre-Expressway peak flows in accordance with NZTA's standard. It has been agreed with KCDC that this, along with offset storage, would achieve their hydraulic neutrality requirements. The reduction to 80% is also a widely recognised stormwater management technique to offset the effects of increases in volume.

The attenuation modelling carried out to date has focused on achieving the 80% target during the 1% AEP storm, because if attenuation can be achieved for this event then achieving it for the 10% and 50% AEP storms is just a mix of fine tuning the swale outlet design and/or minor modifications to the swale cross-section which will be carried out during later stages of the Project design. We are confident that there is adequate flexibility available in the swales and basins to achieve this within the proposed Designation.

KCDC's hydraulic neutrality requirement also addresses the need to test scenarios concerning future catchment development for what is commonly known as "cumulative effects" i.e. effects of the proposed Expressway coupled with effects of future development within the wider catchment. This was explicitly discussed with KCDC, from whom guidance was sought as to the assumptions that should be made about such development. KCDC advised that their policy is to assume that all future developments will achieve KCDC's definition of hydraulic neutrality. Since KCDC's models have also been used for design, it was not considered appropriate to adopt an approach that was at variance to the principles underlying their models and their management of the catchments. It is also noted that the target 80% attenuation level is a widely used practice for addressing potential cumulative effects of increased runoff volume from ongoing wider catchment development.

iv. Mitigate for the proposed Expressway partially filling in existing floodplain storage – i.e. the other part of being hydraulically neutral.

As the proposed Expressway passes through several low lying floodplain areas, it will part fill in existing flood storage volume, resulting in slightly increased flood levels on adjacent land. In the majority of areas the simplest way to mitigate this is to provide additional flood storage to offset that taken up. This can be achieved by one of more of:

- removing areas of higher ground, such as sand dunes and allowing these areas to flood;
- lowering the existing ground surface in areas that currently flood;
- restricting drainage outlets so that floodwater backs up more in an attenuation device;
- where the affected area is small and localised, designate it as flood storage area for the proposed Expressway; and
- oversizing the proposed Expressway treatment and attenuation wetlands.

These options all have their own limitations and effects, which need to be taken into account. Thus site-specific solutions are needed to determine the most appropriate approach, and to identify the footprint and functionality of these storage areas. For example:

- natural groundwater level influences the lower level limit of the storage area;
- groundwater level influences the form of storage area i.e. creating wetlands is better suited to areas of high groundwater;
- surrounding infrastructure e.g. proximity of railway, buildings etc that could be affected by changes in groundwater level (buoyancy/settlement/flooding);
- property boundaries;
- flood levels influence the top water level, which along the groundwater level (which sets the lower level) determines the area needed for the required storage volume;
- surrounding topography influences cost-effectiveness; and
- land use affects appropriateness of using any particular site.

The topography and land use of the adjacent land influences what is appropriate for mitigation, so for example in urban developed areas full offset mitigation is usually required. However, in some areas that are currently rural or already wetlands then a relatively minor increase in occasional flooding may not be considered significant and so mitigation solutions may vary for these areas.

v. *Expressway stormwater will be treated prior to discharge.*

All proposed Expressway runoff will be treated prior to discharge. In addition, where local roads are within the Designation and their drainage systems can be readily joined to the proposed Expressway system (particularly at the interchanges) then they will also receive treatment.

Treatment will be to the Best Practicable Option (BPO) as specified in NZTA's Stormwater Treatment Standard for State Highway Infrastructure. NZTA's Standard reflects internationally accepted best practice for road stormwater treatment. The term BPO is used to refer to the sizing and performance of devices selected from a range of options that could all be regarded as potentially appropriate. Device selection has been governed by topography, drainage form, minimising land requirements and KCDC's requirements for treatment devices to be "natural" rather than proprietary devices. Given these factors, an explicit comparison of options became somewhat inconsequential and so has not been carried out.

As the topography is generally low lying, it is often difficult to use piped drainage to convey flows to an end of pipe treatment device, be it a wetland or a proprietary device. To address this, swales are being used to treat, attenuate and convey stormwater all in one.

In areas of peat or high groundwater, the swales will act and look more like long narrow wetlands than traditional grass swales. They will be relatively deep when compared to traditional swales (i.e. in the order of 1m deep) and will have wetland tolerant plants in the bed. They are intentionally set as flat as possible in order to minimise flow velocity and attenuate flows. This adds as to their wetland appearance. However, in areas of sand and/or relatively low groundwater, then it is more appropriate to use the more traditional grassed swales than to plant them with wetland vegetation.

Planted swales include the functionality of conventional swales but with a flatter grade to reduce velocity and increase retention time and treatment, plus they have the benefits that arise from flow through wetland vegetation. They are specifically identified as a suitable design approach in NZTA's Standard[30] which provides design guidelines. They are also identified as an approach in Auckland Council's (was Auckland Regional Council) TP 10 but in that context there are no distinct design guidelines provided. Wetland swales like this have been consented for NZTA's Tauranga Eastern Link project in the Bay of Plenty as meeting the requirements for both treatment and attenuation.

Wetlands will also be used where either the topography better suits them or where the swales cannot provide all the required attenuation and treatment

For both swales and wetlands the objective is to design them to operate as close as possible to current groundwater levels (to avoid adverse effects on existing wetlands, and risk of settlement beyond the Project boundaries), this will mean that there will not be any more than a minor change in discharge of groundwater coming from the areas of peat and so little change from existing.

Generally, the majority of road stormwater contaminants are flushed off roads in a pulse in the first stages of a rainstorm (subject to various factors including event size and the length of the inter-event period during which the contaminants build up). This is initial runoff is called the "first flush" or the "water quality storm".

The design method in NZTA's Standard determines what is called a Water Quality Volume (WQV) for ponds, but uses an area ratio (wetland area to be 2% of the catchment area) for wetlands and a water quality peak flow rate for swales. The water quality volume is the volume that is needed in order to treat the "first flush" of stormwater runoff. This is defined by the NZTA Standard as being the volume that is generated from the 90th percentile storm: i.e. a storm that 90% of all storms are less than on an annual basis. NZTA has produced nationwide 90th percentile rainfall maps and the 90th percentile Kāpiti is 23mm.

In order to treat the first flush effectively, it is necessary to prevent the runoff from the water quality storm from discharging immediately into the receiving watercourse. In accordance with NZTA's Standard, for swales a water residence time (how long the water flows through the swale) of 9 minutes has been applied to provide approximately an 80% removal of total suspended solids.

Flows from the proposed Expressway near the downstream end of a swale cannot meet the 9 minute residence time requirement. On this issue the Standard notes that "*the normal approach is to accept that the average flow through the swale does take 9 minutes. There will be areas in the upper part of the swale that will exceed the required residence time so the average is considered appropriate in light of the benefits that swales provide.*" [31]

The residence time is achieved with the long lengths of swale (over 100m) and the low gradients (<2%) in the majority of the swales. These two factors have also meant that flow velocity within the swales are such that they are lower than the 0.8m/s in a water quality storm (to promote deposition) and less than 1.5m/s in a 10% AEP event (to avoid erosion and re-suspension) as required by the Standard.

[30] Section 8.5.6.3, Stormwater Treatment Standard for State Highway Infrastructure, 2010, NZTA.

[31] Section 8.5.1.1, Stormwater Treatment Standard for State Highway Infrastructure, 2010, NZTA

NZTA's Standard notes that, for wetlands to be feasible in the long term, they either need catchments greater than 4ha in area or be set low enough for existing groundwater levels to maintain permanent water level within the wetland. Most of the selected wetland locations along the proposed Expressway are in naturally low lying land that has relatively high groundwater, making this requirement easier to meet, as most of the proposed wetland catchments are less than 4ha.

The Standard also recommends using a bathymetric wetland layout with areas of varying depths up to 1m to promote proper wetland treatment functions and establish viable habitats.

The depth distributions provided in the Standards:

- 60% of the total wetland area 0-0.5m deep (below permanent water level);
- 40% of the total wetland area 0.5m to 1.0m deep (below permanent water level); and
- Sediment forebay a maximum of 2.0m deep (below permanent water level) and 15% of the WQV;

Associated with the above requirements is the wetland planting guide adopted that 60% of a wetland will be planted and 40% open water. Refer Drawing CV-SW-212, Technical Report Appendices, Report 22, Volume 5 for a typical wetland arrangement.

The above will assist in preventing nuisance stagnation, algal blooms and the odour issues that are more common with open pond systems. The depth ranges also provide for effective habitat establishment for animals that feed on mosquitoes so minimising the potential for nuisance mosquitoes.

On-going landscaping and maintenance is very important to the proper establishment and on-going performance of wetlands. Refer Section 3.7 for further detail.

Table 3 lists the treatment attributes of swales and wetlands as detailed in NZTA's Standard.

Table 3: Treatment device ability to address water quality for various contaminants. [32]

Mechanism	Ability to Address Contaminants			
	Sediment	Metals (lead, Zinc, Copper)	Total Petroleum Hydrocarbons	Nutrients
Swales	High	Lead - High Zinc - Moderate Copper - Moderate	Moderate	Moderate to low
Wetland Swales[33]	High	High	Moderate	Moderate
Wetland	High	High for all	High	High

[32] Taken from Table 5-7, Stormwater Treatment Standard for State Highway Infrastructure, NZTA, 2010.

[33] The ability of wetland swales are not specifically detailed in the Standard and their rating would be fit between that of wetlands and swales.

The net effects of the treatment design are reported in more detail in Technical Report 25, Volume 3 and discussed further in Section 4.4 of this report.

Natural treatment mechanisms have been used in preference to proprietary end-of-pipe systems as wetlands and swales can also provide attenuation whereas proprietary treatment systems generally cannot. This means that attenuation ponds would still be needed. KCDC's guidelines[34] also require *"stormwater treatment systems based on created natural systems (e.g. wetlands, lakes and detention ponds) able to function as entire ecosystems".*

Where the receiving watercourse is considered to be particularly sensitive, then an additional level of treatment has been provided by locating wetlands at the end of a run of swales prior to discharge into that watercourse.

vi. *Culverts will be designed to "fish friendly" guidelines.*

In general, open channel drains have been used where practical; however, where the proposed Expressway crosses a watercourse and culverts are used, the design will allow for fish passage as appropriate using principles outlined in GWRC's "fish friendly" design guidance pamphlet.[35]

Almost all of the watercourses that the proposed Expressway will cross are relatively flat. The culverts are therefore also nearly flat. This factor alone means the culverts need to be quite large to accommodate flood flows. Large, flat culverts make it easier to accommodate appropriate fish passage, as the inverts can be set below stream bed level resulting in fully flood culvert inverts with low velocity.

The larger pipe culverts will be designed to reflect what GWRC terms as a "low slope" culvert. Generally, this involves:

- minimising the culvert length;
- keeping the culvert as wide as the average natural watercourse bed;
- aligning the culvert with the natural channel (where practical, refer below for further commentary);
- keeping sufficient water in the invert of the culvert by setting the culvert invert lower than the watercourse invert (the design uses an inset of 0.2 x the pipe diameter);
- allowing bed material to settle into the culvert overtime by setting the culvert lower than the stream invert; and
- protecting the inlets and outlets with scour and erosion protection either through rip rap rock and planting or other similar methods that incorporate riparian planting.

The box culverts will be designed to reflect what GWRC terms as a "natural stream bed" culvert. Generally, this involves:

- minimising the culvert length;
- placing gravels, stones, rocks into the floor of the culvert to continue a low flow channel similar to open channel drains. The design mixes finer materials in

[34] Subdivision and Development Principles and Requirements, KCDC, 2005.

[35] Fish Friendly Culverts and Rock Ramps in Small Streams, GWRC, 2003.

with the gravel to better represent each type of stream bed and also so that water flows on top of the gravel rather than through it;

- Sizing gravels to stay in place under flood flow conditions. Given the near flat gradient and large size of the culverts, flow velocity in the culverts is relatively low;
- keeping the culverts as wide as the average natural watercourse bed;
- aligning the culverts with the natural channel (where practical, refer below);
- setting the base of the box culverts lower than the watercourse invert to achieve a smooth transition into and out of the culvert;
- allowing bed material to settle into the culvert over time by setting the culvert lower than the stream invert; and
- protecting the inlets and outlets with scour and erosion protection either through rip rap rock and planting or other similar methods that incorporate riparian planting.

Further details for fish passage design are also outlined in Technical Report 26, Volume 3 (Ecological Impact Assessment).

It is noted that to minimise the length of culvert crossings, it is not always practical to keep the culvert on the same alignment as the overall watercourse. However, gentle transitions into the culvert will help mitigate for this modification.

The drawings generally show the longest culvert route for a crossing as this will have the most adverse effect on flood levels. However, the final design may employ shorter culverts, which will be a slight improvement in this respect. For some culverts that would most obviously benefit from this, an alternative alignment has been shown on the drawings. This allows the culvert length to be minimised and makes it easier to be construct them offline from the watercourse rather than within the bed, thereby helping to avoid the environmental effects that these works would otherwise have.

It is noted that culverts listed in Appendix 22.B and shown on the drawings are not sequentially numbered. This is a result of changes during the design so that several culverts have been added in, moved or removed which has resulted in a non-sequential numeric reference.

vii. Culvert alignment and structural form to reduce the extent of culverts and disturbance of watercourses.

As noted above, there are some culverts where alternative routes will be considered during later design stages. The alignments shown at present reflect the worst case with respect to effects on flood levels and culvert sizing. Shorter culverts on a slightly different alignment are expected to have effects that are slightly more favourable than the longer culverts.

These alternative alignments for the culverts where this would most clearly provide a benefit have been shown on the drawings in order to identify the extent of the flexibility required to select an appropriate culvert alignment and thereby optimise these structures in terms of their performance and environmental effects. As such, it is important to have flexibility during future detailed design stages to revise the angle (or skew) that culverts cross the proposed Expressway.

Similarly, the structural form of the large box culverts is yet to be finalised for all of the culverts. The culvert sections shown on the drawings are typical, and flexibility for later design changes and construction methodology input is needed to optimise their performance. For example, concrete box culverts constructed online of a watercourse would need to have the stream temporarily diverted during construction. However, if a sheet pile walled culvert were determined to be cost effective then this may result in less disturbance of the watercourse during construction. Alternatively, the culverts could be positioned so that they are constructed off line with the existing watercourse maintained until such a time as the culvert is ready to have flow diverted into it.

The form and alignment of the crossings are expected to be confirmed in the detailed design stages of the Project which is expected to be carried out after the consents have been granted.

viii. *Where watercourses and open channel drains will need to be diverted a "natural" stream channel cross section will be used wherever practicable.*

Wherever practicable, new open channels or diverted watercourses will have a slight meander to them and their banks will be planted with riparian vegetation. The will be formed with a main channel for everyday low flow and with flood berms of varying slope for higher flood flows. They will be reinstated with a substrate to match existing and where appropriate fish refugia will also be included and will look more like natural watercourses in appearance than straight engineered drains or farm ditches.

However, the drains will need to fit within the specific spatial constraints of each site that will affect the cross-section of each drain e.g. proximity of the NIMT railway, property boundaries, roads etc. They will also be designed as to accommodate maintenance requirements.

Drawing CV-SW-231, Technical Report Appendices, Report 22, Volume 5 shows a typical arrangement of a similar watercourse located near Smithfield Road. The other open channel drains in the Project will be similar to this detail but most on a much smaller scale. Further details of design considerations for new or diverted watercourses are also outlined in Technical Report 26, Volume 3 and Appendix M of the CEMP, Volume 4 (which includes guidance for designing stream diversions from an ecological perspective).

A schedule of the locations of all the diverted watercourses in the Project is included in Appendix 22.B. It is noted that this schedule may change if the alignments of the culverts and hence watercourses change during later design stages.

4.3 Sector 1- Raumati South

Key stormwater design features in this Sector include:

- Swales;
- Existing culverts south of Poplar Avenue;
- Drain 7 south culvert (culvert 10);
- Poplar interchange;
- Wetland 0A; and
- Offset storage areas 0B and 0C.

Refer Drainage Layout drawings CV-SW-104 through 107, Technical Report Appendices, Report 22, Volume 5 and the schedule of watercourse crossings included in Appendix 22.B.

4.3.1 Watercourse Crossings

i. Existing Raumati Straight Culverts

The existing culverts under SH1 along Raumati Straight between chainage 0 and 1900m are not being affected by the proposed Expressway as the carriageway widening works now commence from chainage 1900m. As such there are no works proposed for these culverts. Hence the culvert referencing on the drawings commences at culvert 6 not 1).

Culverts 6 and 7.1 will be extended to discharge beyond the widened road pavement. To accommodate the increased culvert length, the new sections are slightly larger in diameter than the existing upstream sections. However, further detailed design may determine that matching pipe diameters will be sufficient and still not increase flood levels on the proposed Expressway. These culverts currently discharge directly to the ground instead of a downstream watercourse. This arrangement is not proposed to be changed.

ii. Poplar Interchange Culverts (culverts 7.5 through 9.3)

The existing culverts that currently run under SH1 would either need to be extended a long way under the proposed Expressway embankment or the various drains would need to be diverted to a new single crossing to accommodate the new Poplar Avenue interchange.

The preference is to divert the drains to a single culvert rather than extending them. That is, consolidating the drains into a single crossing avoids difficulties in providing maintenance access, avoids clashes with the new swales and also avoids having to design structures suitable for the high structural loadings resulting from proposed Expressway embankment.

Therefore, many of the existing culverts in this area will be consolidated into one crossing that better uses the open space provided by the proposed new Expressway bridge. However, this consolidation cannot be achieved for all of the drains in this area due to limits on the longitudinal gradient and associated pipe cover. Therefore culvert 7.1 remains as an extension rather than a diversion.

Culvert 9.3 will discharge into a new open channel drain that will be constructed to link the interchange drainage to the existing QE Park drain.

The open channel drains that will achieve the diversion/consolidation will have the form of a natural channel section (as much as spatial constraints will allow). These will have appropriate riparian planting but not hinder the principal purpose of conveying flood flow, or the ability to maintain them. It is considered that open channels will be preferred over pipes from a cost and an environmental perspective.

These open channel drains will be formed in a fashion as detailed to Section 3.2.3 viii of this report and relate closely to the principles noted in the Stream Diversion Guidelines included in Appendix M of the CEMP, Volume 4 (Ecological Management Plan).

iii. Poplar Avenue Culvert (culvert 8)

With the widening and realignment of Poplar Avenue the existing 450mm diameter culvert under Poplar Avenue will be replaced with a new culvert rather than extending the existing culvert.

A KCDC study[36] identified that the existing 450mm diameter culvert under Poplar Avenue was undersized and recommended it be replaced with a 900mm diameter culvert. The diverted road is also wider than the existing road resulting in additional length of culvert. Therefore, a 1050mm diameter culvert is required instead of a 900mm one.

This would have the effect of lowering the 1% AEP flood level north of Poplar Avenue from 6.8m R.L to about 6.3m RL i.e. a decrease of 500mm. This matter is addressed further in section 4.3.2.iv.

iv. Leinster Avenue Drain

As the proposed Expressway passes north of Leinster Avenue, it will cross a small catchment that drains back toward SH1 cutting off the drainage path for this catchment. A new drain is therefore needed to mitigate this effect.

The available corridor for this drain is very restricted due to the need to fit in a cycleway/walkway, a property access way, a noise bund and a drain all between the proposed Expressway and surrounding residential properties. This means that the majority of this drain needs to be piped. Given the low lying land in this area, the gradient of the drain will be flat resulting in a large diameter pipe. However, the extent of pipework will be kept to a minimum with the northern 120m of this new being less restricted and so it will be formed as an open channel (similar to those described in 4.2.3.viii). This new drain will joins Drain 7 just upstream of culvert 10.

This drain is expected to deliver an improvement to the drainage for several properties at the eastern end of Leinster Avenue as there is little existing KCDC municipal drainage in this area.

v. Drain 7 South Culvert (culvert 10)

The proposed Expressway encounters the first significant watercourse at Drain 7 which will be crossed with a 1500mm diameter culvert sized to minimise the headloss across it.

The effect of the culvert and the nearby offset storage and wetland areas will be to reduce upstream 1% AEP flood levels from 6.75m to 6.66m RL, that is a reduction of 90mm. Downstream levels are also reduced from 6.71m to 6.66m RL, or by 50mm. It is noted that the levels reported in the modelling report for this culvert are taken at the culvert whereas the levels stated above are taken from the model up and downstream of the culvert hence the difference if directly compared.

In the overdesign scenario the modelling indicates that flooding upstream of the culvert would decrease from the current level of 6.98 to 6.91m RL as a result of the works associated with the proposed Expressway.

It is noted that the effect of the culvert alone on flood levels has not been separately identified from the net effect of the offset storage and wetland areas.

This culvert will have “low slope” culvert features (as per section 4.2.3 vi.) to accommodate fish passage.

[36] Poplar Avenue Flood Hazard Plan, SKM, 22 June 2005.

vi. Diversion of a Drain 7 Tributary

The end of an existing open channel tributary of Drain 7 will be filled in by the proposed Expressway right at its confluence with Drain 7. This will be diverted into a new open channel drain of similar form to existing to join back into Drain 7 downstream of culvert 10. The design of this diversion will be in accordance with Section 4.2.3.viii and as noted in the Stream Diversion Guidelines included in Appendix M of the CEMP, Volume 4 (Ecological Management Plan).

4.3.2 Stormwater Management

i. Swales South of Poplar Avenue

The land south of Poplar Avenue is low lying, making drainage to “end of pipe” treatment devices very difficult to achieve. These were initially considered but discounted for that reason. The use of very flat swales (some with no fall along them at all) provides for attenuation, treatment and conveyance all in one.

As the land in this area is predominantly peat with high groundwater these swales will function very much like long narrow wetlands: i.e. they will almost be wetland swales. The only difference is they need not necessarily have any standing water in them. This is because the treatment provided by the swales comes from “through flow” as opposed to extended detention. That is, the treatment aspects are provided by slow flow through the vegetation allowing sediments to settle out as opposed to holding back the flow for very long periods of time.

As noted in section 4.2.3.v, swales in high groundwater and/or peat areas will be planted with wetland species and wet tolerant species higher up the sides of the swales where inundation will only be periodic with rainfall.

The swales around the Poplar Avenue interchange will provide treatment and attenuation of stormwater where there currently is none. Therefore, the overall effects on stormwater along this part of the route are considered to be beneficial.

These swales provide attenuation varying from between 45% to 79% of pre-Expressway flows.

The ground conditions mean these swales will be wetland planted. The swales will also run along the base of the Poplar Interchange embankment, allowing runoff to flow off the road, down the planted/grassed batter and into the swale.

Due to the spatial constraints north of Leinster Avenue, the runoff from the proposed Expressway needs to be collected and piped under the proposed Expressway to a single treatment swale on the western side of the proposed Expressway.

One swale is adequate to attenuate the runoff from both northbound and southbound lanes of the proposed Expressway, with the modelling showing attenuation in a 1% AEP storm to 62% of pre-Expressway flows bettering the 80% target.

Again, there are no existing attenuation or treatment devices for stormwater in this area, so the overall effects are considered to be beneficial in terms of improving local flooding problems and water quality.

ii. Wetland 0A

Wetland 0A	
Total Catchment Area	*5.4ha*
Wetland Area	*4950m²*
Water Quality Volume	*560m³*
1% AEP Storage Volume	*4960m³*

The existing low lying land where the proposed Expressway crosses Drain 7 South already contains an important natural wetland. This feature indicates that in this area, providing treatment with a wetland would be more suitable than swales.

All proposed Expressway stormwater will be collected and drained into the newly constructed wetland 0A which will be located on the opposite side of the proposed Expressway from the existing wetland specifically to separate the wetlands so the natural one does not receive runoff from the road.

Wetland 0A will provide water quality treatment, attenuation and contribute to offset storage related to offset storage areas 0B and 0C. It will attenuate the 1% AEP flow to 46% of Pre-Expressway flows. The combined effect on flood levels of wetland 0A and areas 0B and 0C is detailed in item iii. below.

To the north of this wetland all the way up to Raumati Road, the proposed Expressway will cross sand dunes so that drier grassed swales are a more suitable method of draining the proposed Expressway. However, on the eastern side of the proposed Expressway tight spatial constraints mean that drainage back to wetland 0A will provided by a sumps and pipe network.

iii. Offset Flood Storage Areas 0B & 0C

Offset flood storage needs to be provided in order to mitigate the effect of the proposed Expressway filling in part of the existing storage associated with this part of Drain 7. The volume currently modelled to compensate for this loss is 16,500m³. This volume has been arrived at by testing various simulations within KCDC's stormwater model and is to be provided from a mix of:

- enlarging the floodable footprint of the floodplain next to existing natural wetland i.e. area 0B;
- lowering the existing ground in area 0C so that this area becomes available for flood storage; and
- increasing the size of wetland 0A more than is required for just treatment.

Flood offset area 0B involves excavating out an old unapproved clean fill that was used by a contractor to dispose of construction rubbish. This area will be reinstated as a low lying wetland area planted with native species. It will be formed with localised depressions and low mounds to present a more natural surface and as such the new ground level noted on the drawings is an average ground level. This area will be formed to tie into and enhance the existing adjacent natural wetland. It will not receive proposed Expressway runoff.

Flood offset area 0C involves the localised lowering and shaping of the ground surface, while remaining above the water table so it does not need wetland planting. This area will then be reinstated with mass native planting (refer to Technical Report 7, Volume 3). A low flood containment bund (in the order of 0.5-1m high) will prevent flood water extending north onto private property.

The combined result of the offset storage areas 0B and 0C and wetland 0A on the 1% AEP flood levels is an overall lowering of the peak flood levels as shown in table 4.

Table 4: 1% AEP flood levels including mitigation measures.

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Upstream of Culvert 10	6.74	6.67	0.07	Lower
Downstream of Culvert 10	6.73	6.66	0.07	Lower

Subsequent to the completion of the stormwater modelling the arrangement of the wetland and storage areas has been amended from that detailed in the modelling report (Appendix 22.E). The total offset volume provided remains the same but it has been arranged differently across the areas involved. Particularly, this involved keeping the treatment functions to a central "core" wetland and providing the bulk of the flood offset storage in the areas of re-graded land. These areas could possibly be reinstated with grass, but given the high water table, reinstating the area with water tolerant native species may be more appropriate.

This has resulted in wetland 0A and area 0B being supplemented with offset area 0C in order to achieve the same overall flood storage volume.

iv. Offset Storage at Poplar Avenue

In the vicinity of Poplar Avenue, the proposed Expressway footprint will occupy flood plain storage. This area drains north to the Whareroa Stream, with extensive low-lying farmed land on the western parts of QE Park. There is also flood prone land in the area enclosed by Leinster Avenue, Poplar Avenue and Main Road, which includes some buildings at risk of flooding.

There is no hydraulic model available for this area, but we have been supplied with information by KCDC[37] that shows the existing 1% AEP flood level north of Poplar Avenue is assessed at 6.8m. South of Poplar Avenue the flood level has been estimated at approximately is 6.2m. KCDC also identified that an increased culvert capacity at Poplar Avenue would reduce the risk of flooding in the area to the north.

To fully model and map flood risk through this area would require analysis and survey data extending well beyond the extents of the proposed Expressway Project. In the absence of such extensive modelling, we have carried out a volumetric assessment of the flood storage in these two areas, based on the flood level data supplied above, and the local topographic data we have for the areas near the proposed Expressway.

The volumetric analysis shows that upgrading the culvert at Poplar Avenue would significantly reduce the flood level to the north, but would slightly increase the flood level in QE Park to the south. On top of this effect, the filling of the edge of the

[37] Poplar Avenue Flood Hazard Plan, SKM, June 2005.

flood plain would result in a further slight increase in flood levels both north and south. The net effect would be that in the urban area to the north, flood level would reduce by about 500mm to 6.3m, while to the south on QE Park it would increase by up to 100mm, to 6.3m. Because this assessment is only volumetric, and does not account for increased flow out of the area to downstream, it is conservative, and the increase on the QE Park land is likely to be lower than stated. It is also related to a severe storm event, so a very infrequent occurrence.

Overall, this is considered a positive outcome, given the improvement to urban land, and the relatively minor increase in the rural land. The only practical means of fully offsetting this would be to designate additional urban properties to the north, near Leinster Avenue, to provide for offset storage.

4.4 Sector 2 – Raumati / Paraparaumu

Key stormwater design features in this Sector include:

- Drain 7 north culvert (culvert 11);
- Wharemauku stream bridge;
- Mazengarb culvert (culvert 14)
- Swales;
- Offset storage areas 2 & 3A;
- Wetland 3;
- Wetland 4 and Kāpiti interchange; and
- Wetland 5.

Refer Drainage Layout drawings CV-SW-108 through 114, Technical Report Appendices, Report 22, Volume 5 and the schedule of watercourse crossings in Appendix 22.B.

4.4.1 Watercourse Crossings

i. Drain 7 North Culvert (culvert 11)

The proposed Expressway crosses Drain 7 for the second time just downstream of Rata Road. At this location, the drain now has a catchment area of 151ha which is more than three times the size of the catchment upstream of culvert 10. Accordingly, this culvert needs to be significantly larger to accommodate the increased flows.

Hydraulic modelling has shown that the culvert needs to be a 5x3m box culvert to keep the effect on upstream flood levels to a minimum.

The effect of the culvert coupled with the flood storage provided areas 0A, 0B, 0C, 2 and 3A, is that the upstream 1% AEP flood level in the drain will not change from existing i.e. 4.82m RL. Downstream flood levels will also be maintained at existing levels at 4.75m RL. It is noted that the levels reported in the modelling report for this culvert are taken at the culvert whereas the levels stated above are taken from the model up and downstream of the culvert hence the difference if directly compared.

In the overdesign scenario the modelling indicates that flooding upstream of the culvert would increase by a minor amount (i.e. 10mm) from current the level of 5.24 to 5.25m RL.

This culvert will have “natural stream bed” culvert features (as per section 4.2.3 vi.) to accommodate fish passage.

An alternative alignment for culvert 11 will be to locate it on a small branch drain north of the existing Drain 7 channel and divert Drain 7 into it. This will allow the culvert to be constructed off line and also reduce the length of the culvert as the skew would be lessened. There would be a short length of new open channel required, which would be designed accordance with the with Section 4.2.3.viii and as noted in the Stream Diversion Guidelines included in Appendix M of the CEMP, Volume 4 (Ecological Management Plan). The final location of culvert 11 will be determined during later design stages. The effects of the shorter culvert are considered to be no more than the longer culvert route.

ii. Wharemauku Stream Bridge

The proposed Expressway will cross the Wharemauku Stream on a bridge that must also be long enough to allow the future extension of a local road beneath it (by others). Therefore, the waterway beneath the bridge does not set the span nor the vertical clearance requirements for the bridge, except insofar as flood levels influence the level of the proposed local road.

Refer structural drawings ST-BR-250, Structural – Bridges, Volume 5, for details of this bridge.

The bridge crossing has been conservatively modelled, with the local road set above the 1% AEP flood level. However, KCDC has advised that the occasional flooding of the road would be acceptable. If, during detailed design, the level of the local road was set lower, then the effects on the Wharemauku Stream and its associated floodway would be minor. This is because lowering the road would slightly increase the capacity of the floodway beneath the proposed Expressway as in extreme floods there will be greater waterway area available with flow being allowed to spread out over top the local road.

While the bridge piers have been positioned clear of the main channel, they are sited in the floodway. However, the effect of the crossing on flood levels upstream of the proposed Expressway will be minor and would be offset by the effects of storage areas 2 and 3A discussed in Section 4.4.2 iii.

The effect of the bridge and associated offset storage areas would reduce the upstream 1% AEP flood levels in the Wharemauku Stream channel from 5.17m to 5.07m, that is a reduction of 100mm. Downstream levels are maintained at existing levels of 4.75m RL. It is noted that the levels reported in the modelling report for this culvert are taken at the culvert whereas the levels stated above are taken from the model up and downstream of the culvert hence the difference if directly compared.

Underneath the proposed Expressway bridge, the stream channel will be lined with rock rip rap. As the stream is relatively narrow, lining the banks with rip rap will require lining the full width of the stream from one bank top to the other. The rip rap is needed to protect the channel from scour as vegetation cover will not be reliable under the bridge deck as both light and rainfall will be limited.

iii. Mazengarb Culvert (culvert 14)

The next watercourse crossing north of the Wharemauku Stream is the Mazengarb Drain. Due to the upstream catchment being 379ha in area, the culvert needs to be a 5x3m box culvert.

Modelling shows that the 1% AEP flood levels are increased upstream of the culvert from 6.99m to 7.05m RL i.e. by 60mm. Downstream of the culvert levels are also increased from 6.93m to 7.02m RL i.e. 90mm. KCDC have advised that flooding is particularly sensitive in this area and that such an increase needs to be mitigated. This is even though the increase is confined to a relatively small length of the drain i.e. 20m upstream and downstream of the culvert.

In order to mitigate this effect the design of wetland 5 has been revised to include significant additional offset storage as detailed under Section 4.4.2 v.

Initial investigations have found that much of this culvert could be constructed off line from the drain, thus minimising construction environmental effects. The culvert may also benefit from incorporating long radius swept bends (in plan) in order to better align the inlet/outlet with the drain alignment upstream and downstream. These details will be finalised during later detailed design stages of the Project.

In the overdesign scenario the modelling indicates that flooding upstream of the culvert would increase from current level of 7.23 to 7.31m RL as a result of the works associated with the proposed Expressway. However, the final effect is expected to be less than this as it does not take into account the effect of the increased storage in wetland 5 discussed in section 4.4.2 v.

This culvert will have “natural stream bed” culvert features (as per section 4.2.3 vi) to accommodate fish passage.

4.4.2 Stormwater Management

i. Swales

The swales north of Raumati Road drain northwards and discharge into Drain 7 on the downstream side of culvert 11. As this area is mainly sand dunes, the swales will be grassed. These will attenuate the 1% AEP flows to 65% of pre-Expressway peak flows.

North of Drain 7, the swales will run at the base of the proposed Expressway embankment as the road rises up for the Wharemauku Stream bridge. Runoff will flow down the embankment slopes and into the swales positioned at the toe of the embankment. The land in this area is low lying peat flats and so these swales will be wetland planted.

The eastern swale runs alongside the margin of offset storage area 2 and, while both of these features will look very similar in appearance; they will be separated from each other so that runoff treatment only occurs in the swale. This swale will also attenuate the 1% AEP flows to 46% of pre-Expressway peak flows.

The western swale will perform a similar function however it will also drain into wetland 3 prior to discharging to the Wharemauku Stream. This is because this swale cannot fully provide the required attenuation and so must be supplemented with a wetland. Further refinement of this specific swale in later design stages may reduce or even remove the need for this wetland, either through increased swale attenuation capacity, or by overspill to the adjacent offset storage area. Refer item ii for comment on the performance of wetland 3.

A small section of the proposed Expressway to the north of the Wharemauku Stream will have stormwater piped down into a swale at the foot of the road embankment. This swale would then outlet into the Wharemauku Stream. Due to its relatively short length it will be twice the width of a standard swale. Similar to the swales on the southern side of the Wharemauku Stream, it will be wetland planted. This swale attenuates the 1% AEP peak flows to 62% of pre-Expressway flows.

All of the above swales contribute to the lowering of flood levels in Drain 7 and the Wharemauku Stream along with the offset storage areas. The results of this are shown in Table 5.

ii. Wetland 3

As noted above, wetland 3 is needed to supplement the attenuation performance of the swale and is not required for water quality treatment although it will assist in this respect. As such, the water quality volume does not need to be separately identified.

Wetland 3 attenuates the 1% AEP peak flow to 70% of pre-Expressway flows. The effect of this attenuation, along the effects offset flood storage areas 2 and 3A, are detailed in item iii below.

Wetland 3	
Total Catchment Area	*5.9ha*
Wetland Area	*1800m²*
Water Quality Volume	*NA*
1% AEP Storage Volume	*2,600m³*

This wetland will be located in the same position as the existing Kiwi Pond. The permanent water level in this wetland will be lower than that currently in Kiwi Pond in order to assist in providing the required volume. This will be maintained by a controlled outlet.

Kiwi Pond formed when the Wharemauku stopbank was constructed without a piped outlet through it, so that the low lying land behind it became a pond. KCDC have advised that it was their intention to construct an outlet drain to control the water level in the pond. The residences along Kiwi Road suffer problems associated with high groundwater and resulting poor drainage. KCDC further noted that whenever they clean out the bed of the Wharemauku Stream, the water level in Kiwi Pond drops significantly, even to the extent that in one instance the pond dried out.

Lowering the water level in Kiwi Pond when it is converted into wetland 3 may have a positive effect for local residences as it may locally reduce the current high water table. Kiwi Pond is not considered to be an ecological site of high significance, as outlined in Technical Report 26, Volume 3.

Culvert 11.2 will drain a small isolated catchment into wetland 3 to manage runoff from this area. Similar to the above it will also assist in controlling high ground water in adjacent properties.

iii. Offset Storage Areas 2 and 3A

The proposed Expressway passes through the Wharemauku floodplain and so it takes up volume currently available to store floodwater. This needs to be offset in order to mitigate the resulting increases in flood levels.

The required offset volume is approximately 76,000m^3 and has been determined by testing in KCDC's stormwater model. The available level range for achieving this volume is restricted by the relatively high groundwater levels, which affects the footprint required. It is expected that, with further design iterations, the storage areas can be optimised further and the required volume and footprint reduced while maintaining the requirement of not increasing flood levels. As such the areas shown on the drawings and modelled are conservative.

The effect of the storage areas on the flood levels in Drain 7 and the Wharemauku Stream is to maintain the existing 1% AEP flood level in both Drain 7 and downstream of the Wharemauku bridge. However, upstream of the bridge there will be a reduction of 100mm as a result (refer table 5).

The new volume has been distributed across two available open space areas on each side of the proposed Expressway. Given the very large volume required, these are the only two areas readily available where this can be practically achieved without significant additional private property purchase.

Area 2, east of the proposed Expressway, can hold 38,000m^3 of additional storage and area 3A, to the west, further 38,000m^3. The only way to achieve these volumes in this flat land is to lower the ground level. Doing this to area 3 will require the groundwater level to be lowered by approximately 600mm so that this new storage area is not filled up with groundwater inflow. This could risk settlement occurring within surrounding areas (refer to Technical Reports 21 and 35, Volume 3 for further assessment and details on this issue).

The high groundwater also means that area 3A would be ideally suited to be reinstated as a wetland. Discussions around this issue have been held with KCDC to understand the proposed future town centre expansion and their likely stormwater offset requirements. It was agreed that it would be best if the offset areas were coordinated and took the form of a coherent wetland complex that worked together from an aesthetic and community amenity standpoint. However, the storage demands from proposed Expressway and town centre would otherwise function independently i.e. neither areas needs to be oversized to accommodate the effects of the other.

Earlier alternatives had considered just lowering the ground in area 2 by around 1m and providing most, if not all, of the required storage on one side of the proposed Expressway. However, this would lead to an increased risk of property settlement with the greater draw down of groundwater and so this option was discarded.

Another alternative to providing the offset storage is to upgrade the Wharemauku Stream all the way to its outfall to the sea. This option has not been considered further as it would require significant additional property purchase, with significant and costly works in a highly developed area. Further, KCDC already has a programme for similar works in order to resolve existing flooding problems elsewhere in the catchment.

Unlike area 3A, area 2 has some areas of higher ground that can be excavated to provide the volume and thereby lessen the extent of overall ground lowering that is needed.

In order for flood flows to more efficiently use area 2, a flow balancing culvert (culvert 11.3) is needed beneath the proposed Expressway linking the two areas together.

The modelling report in Appendix 22.E considers wetland 3 and storage area 3A as one combined area. The separation is due to the area around Kiwi Pond being better suited to a traditional wetland.

Table 5: 1% AEP flood levels including mitigation measures.

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Upstream of the Expressway (Drain 7 North)	4.82	4.82	0.00	No change
Downstream of the Expressway (Drain 7 North)	4.75	4.75	0.00	No change
Upstream of the Expressway (Wharemauku Stream)	5.17	5.07	0.10	Lower
Downstream of the Expressway (Wharemauku Stream)	4.75	4.75	0.00	No change
Offset storage area 2	4.81	4.74	0.07	Lower
Offset storage area 3A	4.74	4.74	0.00	No change

iv. Wetland 4 and Kāpiti Interchange

Wetland 4 provides attenuation and treatment for the 1.3km of proposed Expressway from just north of the Wharemauku Stream bridge to halfway to Mazengarb Road.

Wetland 4 uses the principle of a core central wetland for treatment with a surrounding larger area for flood water to spill into for attenuation. This is common to most of the wetlands that also provide attenuation or storage functions.

Wetland 4	
Total Catchment Area	***15ha***
Wetland Area[38]	***2214m²***
Water Quality Volume	***1560m³***
1% AEP Storage Volume	***6250m³***

[38] The area is less than the guidelines of 2% of total catchment area as large parts of the catchment are being served by swales.

Such a large catchment results in a large wetland and associated attenuation volume.

This is further exacerbated by not being able to make use of swales at and north of the Kāpiti Road interchange due to very tight spatial constraints. Earlier drainage options had swales north of the interchange; however, with the ramps, noise mitigation and cycleway/walkway added this solution would have required additional private properties to be purchased and so was considered undesirable. This same spatial constraint has also lead to the interchange being drained to wetland 4 by a standard drainage system of sumps, manholes and pipes.

Earlier design iterations for this wetland found that during the 10% AEP storm the additional volume discharged was causing increased flooding in Drain 5 immediately downstream of the wetland outlet. This is because the proposed Expressway drainage provides a more efficient drainage route than existing flow paths (currently over paddocks and via KCDC piped drains). In order to address this effect, the wetland has been oversized to compensate.

The proposed Expressway will result in approximately 350m of Kāpiti Road being drained into this wetland providing both treatment and peak flow attenuation that currently does not occur. This will therefore have a beneficial effect on existing water quality and flooding from stormwater that runs off Kāpiti Road.

Wetland 4 attenuates the 1% AEP peak flow to 78% of pre-Expressway levels. The effect of this wetland is detailed in Table 6. This wetland would also contribute to the overall lowering of flows in the Wharemauku Stream into which it ultimately drains.

Table 6: 1% AEP flood levels including mitigation measures.

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Upper Drain 5	6.35	6.32	0.03	Lower

v. Wetland 5

Wetland 5 provides attenuation and treatment for the northern half of the proposed Expressway between Kāpiti Road and Mazengarb Road, a length of some 900m.

Wetland 5 has a core treatment wetland with an associated wider flood storage area.

Unlike Wetland 4, there is less of a spatial constraint and swales can be used

Wetland 5	
Total Catchment Area	*9ha*
Wetland Area[39]	*900m²*
Water Quality Volume	*180m³*
1% AEP Storage Volume	*6234m³*

[39] The area is less than the guidelines of 2% of total catchment area as large parts of the catchment are being served by swales.

to drain most of the carriageway. However, south of Mazengarb Road there is a long section of noise barrier proposed along the edge of the proposed Expressway. In this area sump and pipe drainage will be used.

Wetland 5 attenuates the 1% AEP peak flow to 15% of pre-Expressway flows. The effect of the discharge from this wetland in combination with culvert 14 is detailed in Table 7.

Table 7: 1% AEP flood levels including mitigation measures.

Location	Pre-Expressway (m r.l)	Post Expressway (m r.l)	Difference (m)	Effect
Downstream of Culvert 14	6.93	7.02	0.09	Increase
Upstream of Culvert 14	6.99	7.05	0.06	Increase

The increase in flood levels is a result of the more efficient drainage path provided by the new drainage into the Mazengarb Drain downstream of the proposed Expressway. This allows stormwater to more quickly enter the drain affecting an increase in flood level. In order to mitigate this effect the storage volume in wetland 5 has been increased by 240% from that model in table 7. This is expected to reduce the discharge from wetland 5 to such an extent as to no longer cause an increase in the flood levels. The final size of wetland 5 will be confirmed during later design stages of the Project.

Also, an alternative, the Project team is reviewing whether Wetland 5 can be combined into a joint wetland located on land owned by Meadows New Life Trust. They are planning to develop their land with an auditorium, commercial and residential areas. This development also requires its own stormwater treatment/attenuation pond or wetland. This alternative would require the agreement of the landowner and initial discussions have found the owner receptive to this option if joint benefit can be achieved. The alternative location is shown on drawing CV-SW-114, Technical Report Appendices, Report 22, Volume 5. If this does not eventuate, then wetland 5 would be used as described above.

4.5 Sector 3 – Otaihanga / Waikanae

The key stormwater design features in this Sector include:

- WWTP drain culvert (culvert 15);
- Wetland 6;
- Offset storage area 6A;
- Landfill drain culvert (culvert 17);
- Otaihanga project yard;
- Otaihanga drain;
- Wetland 8;
- Muaupoko stream;
- Waikanae river and floodplain
- Waikanae floodplain culverts (culverts 21, 22 and 22.1);
- Wetland 9
- Flood offset storage area 9A;

- Te Moana floodway and proposed Expressway bridge;
- Waimeha stream bridges; and
- Wetland 10;
- Isolated catchment at chainage 12100m.

Refer Drainage Layout drawings CV-SW-115 through 120, Technical Report Appendices, Report 22, Volume 5 and the schedule of watercourse crossings in Appendix 22.B.

4.5.1 Watercourse Crossings

i. WWTP Drain Culvert (culvert 15)

Modelling has shown that a 1500mm diameter culvert provides sufficient capacity to convey the drain flow under the proposed Expressway.

The effect of the swale discharges and filled in floodplain storage increases the 1% AEP flood levels upstream from 6.67m to 6.71m RL i.e. an increase of 40mm. Downstream levels are also increased from 6.58m to 6.62m RL, also 40mm.

In order to fully mitigate this effect wetland 6 incorporates additional offset storage as detailed under Section 4.5.2 ii.

In the overdesign scenario the modelling indicates that flooding upstream of the culvert would increase from the current level of 6.86 to 6.99m RL. However, this does not take into account the effects of the offset storage area 6 that was increased after the model was run as noted above.

As this Drain is mainly fed by the WWTP outflow, the Project ecologist has advised that fish passage is not considered to be a significant issue for this culvert. However, fish passage is expected to be accommodated anyway due to the very flat nature of the drain and culvert.

The size of the culvert has been further tested in consultation with KCDC for sensitivity against KCDC's expected future upgrade of the WWTP. It was found that the culvert is appropriately sized to accommodate KCDC's expected increases.

ii. Landfill Drain Culvert (culvert 17)

Modelling has shown that a new Expressway culvert will remove an existing constriction in the Landfill Drain. This will improve the drainage sufficiently that it would increase the 1% AEP flood levels downstream of the proposed Expressway by 30mm, from 8.11 to 8.14m RL. In order to mitigate this effect the inlet will be designed to restrict the flow, causing flood water back up and be stored above ground upstream of the proposed Expressway but within the Designation.

This control will take the form of a slotted weir structure that will allow low flow passage relatively unhindered for fish passage but will also control larger flows to allow floodwater to back up into offset storage area 6A (refer 4.5.2iii).

This culvert will have "low slope" culvert features (as per section 4.2.3 vi) to accommodate fish passage except at the inlet as noted above.

There is space around culvert 17 to allow the culvert to be constructed offline. This would decrease the skew on the culvert and shorten it but would require a short length of watercourse diversion to direct the flow into and then away from the culvert. There would be no effect of this change on flood levels because they are set by the inlet control (slotted weir) the culvert itself. A slightly shorter culvert would mean less of the existing watercourse would be disturbed (assuming an off line construction methodology).

iii. Otaihanga Drain

It is not expected that the existing 600mm diameter piped section of the Otaihanga Drain can be retained during construction as it passes through an area where ground improvements are needed for the Otaihanga Road bridge (including the approach embankments).

It will be replaced with a new piped drain that will tie into the existing drain on both sides of the proposed Expressway. As such, the effects on the Otaihanga Drain are not expected to be significant as this is just a pipe replacement and does not involve introducing a new pipe to the drain.

iv. Waikanae River Floodplain Culverts (culverts 21, 22, 22.1 and 22.2)

The proposed Expressway will cross the Waikanae River's southern floodplain and in doing so will cut off a small section of the floodplain west of the proposed Expressway. In order to minimise the total volume affected, three culverts (21, 22, and 22.1) have been designed to pass flood water back into these areas that would otherwise have been cut off. These culverts act as flood balancing culverts as well as low flow drainage connections to the Muaupoko Stream.

Modelling shows that even with these culverts, flooding in these western areas will be reduced, not maintained at existing levels. However, the total storage lost as a result of this and from the proposed Expressway embankment fill will be mitigated by the effects of the river channel widening associated with the bridge (refer Section 3.5.1v for detailed discussion of the bridge). Overall, the flood level on the southern part of the river floodplain east of the proposed Expressway will be lower than currently occurs i.e. 5.35m to 5.27m RL, a reduction of 80mm.

In one of these cut off areas, a new stretch of open channel drain will be constructed to allow the upstream "clean water" to bypass wetland 8 so that the extra flow does not affect its efficiency. The channel will run to a new culvert under the existing access way and outfall into the Muaupoko Stream at approximate chainage 10,260m. This would replace the existing perched 300mm diameter farm culvert currently in this location.

Together with the new culverts the channel will allow the land to the west of the proposed Expressway to be much better drained. Currently there is little formal drainage out of this area so when it does flood the floodwater must either soak away or evaporate over a long period of time.

This new drainage connection to the Muaupoko Stream will restore fish passage into the existing wetlands further west of the proposed Expressway where it has currently been severed by farming practices.

The open channel will be planted with riparian vegetation and will more closely resemble a natural stream than an engineered drain. This drain does not carry high flood flows, but is needed to provide a positive gravity outlet for wetland 8, assisting in the establishment of the riparian planting and natural character.

The modelling also suggests that culvert 21 and 22 could be consolidated into a single larger culvert, with some minor additional channelling works needed on the western side of the proposed Expressway. Initial results show the culvert would need to be in the order of 1500mm in diameter. This alternative will be investigated further during later design stage and will become part of the works if it is proved to be beneficial. Changing to a single culvert will not involve any additional works in existing permanent watercourses.

The KCDC standard overdesign scenarios have not been applied to these culverts as they fall under the Waikanae River set of overdesign scenarios. The effects of overdesign flows on upstream areas of these culverts would be relatively minor due to the small, steep sided upstream catchments and when compared to the very much larger and flatter Waikanae floodplain.

v. Waikanae River Bridge and Floodplain

The Waikanae River is the largest watercourse that the proposed Expressway will cross. The river has a large floodplain and, during 1% AEP storms, it floods significant areas of adjacent land.

Refer structural Drawings ST-BR-450 and 451, Structural – Bridges, Volume 5, for details of this bridge and civil drawings CV-SW-391 through 394, Technical Report Appendices, Report 22, Volume 5 for details of the associated waterway and scour protection works discussed below.

The location of the bridge has been determined primarily by factors other than those relating to the river and its flood plain, although the span is driven by waterway requirements. Influences on the location are:

- road geometrics;
- property requirements;
- the presence of the large trunk gas mains under the river upstream of the proposed bridge site; and
- iwi and wāhi tapu related considerations.

The bridge is located just downstream of the crossing that was proposed for the Western Link Road and in the same location as the Sandhills Highway alignment that preceded this. The position takes advantage of the local topography as the floodplain is at its narrowest of any point downstream of the existing SH1 bridge. This allows the overall length of the bridge to be 182m with the waterway width beneath abutments being approximately 180m (subject to final detailed abutment design).

GWRC's Catchment Management department (GWRC CM) has built and maintains a hydraulic model of the Waikanae River to assist in their management of the river and to determine flooding extents (informing the setting of building floor levels by KCDC etc).

The design process for the new bridge involved modelling it in GWRC's model. This included several iterations during the refinement of the design in order to achieve mitigation of the effects of the bridge. This modelling was carried out GWRC's current incumbent consultant, REC and the modelling report is included in Appendix 22.F.

The model is a combined 1D (main river channel) and 2D (flood plain) model, thereby giving a good representation both of the river channel and bridge hydraulics, but also the interaction of flow between the river and the floodplain, and the extent of flooding on the floodway (discussed further in Section 4.5.2 vi).

In the process of amending the model to meet the Project team's requirements, REC updated the following items that have much improved the accuracy of the model:

- Updated for 2010 GWRC surveyed cross sections improving the accuracy of the 1D channel model to reflect most recent recorded river channel morphology;
- Updated the floodplain for recent 2010 LiDAR survey that has been validated with photogrammetry and field survey and as such is considered more accurate than the previous floodplain ground surface in the model; and
- Additional successful validation was carried out against the 2008 (0.8% AEP) and 2005 (10% AEP) storms using actual GWRC flow records of these storms and compared the resulting model levels with photo and anecdotal flood information provided by Waikanae On One (Community Group) and the Commodore of the Otaihanga Boat Club. The model predicted flood levels in the 2008 storm to within 90mm of that determined from site observations at Greenaway Road and outside 266 Te Moana Road – both of which are areas upstream of the proposed Expressway on the floodplain that would be affected by any increase in flood levels. The model is therefore considered to be an accurate representation of the river.

During early discussions with GWRC CM, they advised the assumptions set out below should be used to prepare the initial bridge design. As the design has progressed, there has been on-going discussion with GWRC to seek as much agreement as possible on the design approach and details.

- GWRC CM maintains a 35m wide design alignment for the main channel (the red lines on Drawing CV-SW-391, Technical Report Appendices, Report 22, Volume 5) with 20m wide buffer zones on each side of this. GWRC require the piers to be outside the 35m corridor. Note that in some locations the existing river channel is narrower and does not align with the 35m wide corridor preferred by GWRC, particularly downstream of the proposed crossing;
- The 1% AEP storm including climate change and effects of debris loading should be the design case for setting the bridge height (with freeboard);
- A freeboard from the 1% AEP flood level to the underside of the bridge of 2.2m was recommended by GWRC CM. This was used for the Western Link Road bridge and has also been adopted for the proposed Expressway bridge. It is noted that this is significantly greater than the maximum 1.2m required under NZTA's Bridge Manual;
- Effects on upstream flooding must not be significant and they would consider an increase of 100mm to be significant;
- Minimise the number of piers in the floodplain;
- Single, round, column piers are preferable but if multiple columns are needed then they should align with the flood flow; and
- Overdesign events and effects on the wider floodplain need to be investigated.

However, GWRCC CM did note that it was the Project team's responsibility to determine an appropriate bridge design and to prove that the effects are acceptable.

The initial span arrangement tested in the model took into account a span limited by the use of 1500mm deep concrete Super T beam for the bridge deck. This was the preference of the structural designers and the contractor while allowing the 35m river corridor to be cleared in one span of the bridge. Spans longer than this would require either changing the bridge form to a structural steel bridge or mean using 1800mm deep Super T beams, for which the pre-casting moulds are not available in New Zealand. Either change would involve a significant increase in the construction cost and result in a much “heavier” appearing structure affecting the visual aesthetics of the structure.

The initial design also took into account the design prepared for the Western Link Road bridge. However, the design parameters for an Expressway standard bridge are significantly greater than those of the local Western Link Road bridge (even though both were/are to be 4 lanes wide)[40]. The key differences are:

1) The seismic design standard is a 1 in 1000 year return period earthquake for a local road but a 1 in 2500 year earthquake for a highway (refer NZTA Bridge Manual). Further, the Kāpiti/Wellington region also has a seismic hazard factor of 0.4 which is twice (or more) as high as any other region in New Zealand where a four lane Expressway bridge is likely to be needed (i.e. Auckland has a factor less than 0.13, Tauranga 0.2 and Christchurch 0.2). This coupled with the return period has a significant impact on designing a practical bridge with single columns at the piers.

2) The roading dimensions (shoulders/medians/cycleway/walkway widths etc) for the proposed Expressway are greater than for a local road due in part to the greater design speed. This results in the proposed Expressway bridge being approximately 5m wider than the Western Link Road bridge which makes it more difficult to achieve the design performance under seismic loading.

The combination of the above would have required a change to a structural steel superstructure bridge with much larger pile caps and many more piles at each pier or allowing piers to be positioned within the main river channel river. The later was considered to be undesirable from an environmental and flooding standpoint and the former would be significantly more costly.

Bridge aesthetics are also important in this location due to the high public usage of the river corridor. A longer span bridge generally equates to a heavier looking structure with bigger piers and this was not desirable for aesthetic reasons.

These factors mean that using a single column pier for this bridge is not practicable. The initial design was for four columns per pier. However, modelling has shown the effects on flood levels to be greater than 100mm and so unacceptable.

The design was refined to modify the span arrangement to remove one pier, change from spill through abutments to near vertical abutments, slightly increase the pier column size, change to twin column piers and remove the pier skew i.e. no longer skewed to the bridge deck but slightly out of alignment with the flood flow. Removing the skew is desirable in order to simplify construction. Given the roading alignment, the skew was only minor (7 degrees) and removing it allowed for a “worst case” to be tested in the model.

An alternative flood mitigation design was also tested, which involved the removal of a large part of an existing spur that projects into the floodplain just upstream of the bridge. The aim of this was to offset the effects of the bridge by providing

[40] The Western Link Road Waikanae Bridge was to be capable of accommodating future four laning.

greater capacity through the existing floodplain constriction. In the end, incorporating this option into the design has proved not to be necessary, and there were some collateral effects of this option (e.g. on archaeological and ecological sites) that were better avoided.

To achieve the current design span arrangement, GWRC CM's 35m design corridor needs to be slightly re-aligned. This is in order for the bridge span arrangement to fit into the existing topography while keeping the number of piers to a minimum.

The proposed amendments to the corridor are shown as the blue lines on Drawing CV-SW-391, Technical Report Appendices, Report 22, Volume 5. Feedback from GWRC CM and confirmed by a peer reviewer is that this amendment is acceptable. Refer below for further comments on the peer review of the design.

If, over time, the river moves its alignment (which is possible) then the piers may become located in the main channel. In order to prevent this from occurring, and to protect the structure and river banks from scour, the channel position needs to be fixed under the bridge. This is achieved by placing rock rip rap along the banks to armour and protect them.

As the current river channel near the proposed bridge is narrower than the 35m wide corridor, there needs to be a transition from the rip rap back to the existing channel alignment. This has been designed in consultation with GWRC CM and follows the river corridor alignment on the true right (north) bank downstream to where a small watercourse outlets into the river (some 160m downstream of the bridge). This downstream transition was proposed by GWRC CM and supported by the peer review carried out on the proposed design and has been adopted into the design.

The new river alignment was adopted after the majority of the modelling had been completed using the previous transition (which was a shorter and straighter downstream transition flaring at 1 in 5 back to the existing river bank) which was based on traditionally accepted river engineering design principles. However, GWRC CM noted that this layout would not be acceptable to them and that a different transition layout would better fit with the river design corridor and with their river management practices. The peer reviewer agreed with GWRC.

It was acknowledged that this change will increase the footprint of the works, require more excavation of the river bank, disturb more river bank riparian planting, require more environmental offset, require additional private land purchase and cost more. However, it was agreed that the amended transition would give a better overall outcome for the river long term, and so was adopted.

The updated transition was not expected to increase the flood levels from those currently modelled so only the 50% AEP and 1% AEP scenarios were re-run to test the sensitivity of this assumption. The results have shown a slight improvement of lowering flood levels compared to the early results. The results for the other scenarios are therefore considered to still be valid, if slightly conservative.

This river bank transition would be protected from scour with planting willows inter-spaced with native plants in accordance with the GWRC/KCDC Environmental Strategy[41] for the river.

REC's modelling of the current bridge design and channel works shows the 1% AEP flood levels are reduced from:

- 5.03m to 4.95m RL i.e. lowered by 80mm at the bridge,
- 5.26m to 5.18m RL i.e. lowered by 80mm at a point approximately 100m upstream of the bridge; and
- 4.95m to 4.86m RL i.e. lowered by 90mm at a point approximately 100m downstream of the bridge.

These effects are attributed to the channel widening under and downstream of the bridge.

GWRC CM requested modelling of a range of scenarios for different return periods (all with climate change) and other potential situations in order to better understand the effects on the river system and test the sensitivity of the design. These are detailed in Table 8.

Table 8: Model simulations with flood level at the bridge. The design case is highlighted blue. All include climate change.

Scenario	AEP	Description	Flood level (m RL)
1	1%	Existing river (no bridge)	5.04
2	1%	Existing channel with bridge and debris	5.05
3a	1%	Old channel design, new bridge & debris	5.03
3b	1%	New channel design, new bridge & debris	4.95
4	1%	Old channel design, bridge, no debris	5.03
5	1%	Old channel design, debris and cut back spur	5.03
4a	50%	New channel design, bridge, no debris	3.98
4b	50%	New channel design, bridge, no debris	3.78
5	5%	Bridge, existing river banks	4.74
6	5%	Bridge, no debris	4.73
7	1%	Increased floodplain level (500mm)	5.13
8	1%	50% climate change instead of 16%	5.31
9	0.04%	For bridge structural design	5.49
10	0.04%	Debris, berm aggradation & raised stopbanks	5.58

Scenario 1 is the existing base case.

Scenario 3b is the design case of the channel design and bank transition agreed with GWRC (and peer reviewed), climate change and debris.

Scenario 7 was run to understand the implications of long term floodplain aggradation.

Scenario 8 was to understand the effects of greater climate change than the MfE guidelines.

[41] Waikanae River Floodplain Management Plan, 1997, and the Environmental Strategy, 1999.

Scenario 9 was run to inform the bridge structural design in accordance with the Bridge Manual.

Scenario 10 was run to understand the effects of any future raising of the stop bank. This was modelled as if a future stop bank was able to fully contain the flow.

The lowest point of GWRC's stopbank immediately upstream of the proposed bridge is at 6.08m RL. This means that there generally remains 500mm of freeboard to the top of the GWRC stopbank and so there will be no effect on the performance of the stop bank and associated flood risk.

At GWRC CM's request the Project team engaged Gary Williams (GWRC's independent river design consultant) to carry out a peer review of the Waikanae River corridor amendments, river works and effects of the bridge. The Project team has adopted all of the peer review recommendations reported by the reviewer and included in Appendix 22.H. A summary of the comments on the design are:

- Amendments to the river corridor are acceptable;
- Bridge span arrangement is acceptable;
- Remove the rock rip rap to the true right bank if the existing channel were retained as GWRC CM would like that bank to be more flexible;
- The bank armour detail should not have a low level toe but rather be thicker towards the bottom;
- Consider extending the right bank rip-rap further downstream than the left bank rip-rap;
- The ends of the rip rap should be "turned into" the banks;
- The rock size should be increased to D_{50} = 400mm;
- Angular rock is acceptable. It provides better protection than rounded rock;
- The north bank downstream transition needs to extend back to the design corridor and also extend a further 20m downstream to better tie in as it will otherwise have an adverse effect on how GWRC CM manage the river.

vi. Te Moana Floodway and proposed Expressway Bridge

As described in Section 3.4.6, the proposed Expressway crosses an existing flood overflow route south of the proposed Te Moana Road interchange.

The overflow route is designated in KCDC's District Plan and runs from the true right bank of the Waikanae River to and across Te Moana Road (alongside the Waimeha Stream). It remains an important flood risk management feature for the area even though GWRC have constructed stopbank works that protect the floodway for storms up to and beyond the 1% AEP storm. It has been retained by GWRC and KCDC to address the risk of stopbank failure and to provide a robust protection in the event of storms in excess of the design standard. Modelling of the Waikanae River shows that the flood level during the 0.04% AEP storm (scenario 10 of table 8) along the Puriri/Greenaway stopbank varies from 5.7 to 5.9m RL against the stopbank levels that vary from 6.08m RL at its southernmost end to 7.0m RL, 600m upstream at Greenaway Road. However, this is a result of the relief provided by overflows spilling out of the river corridor upstream of Greenaway Road.

Given the stopbanks currently provide protection to the 1% AEP standard plus freeboard, an overflow or stopbank breach scenario is considered very much as an overdesign event. However, the proposed Expressway could affect this overflow path and increase flooding risks to private properties, including residential properties, between the Waikanae River and Te Moana Road during such an event.

As the proposed Expressway needs to cross a bridge over the Waimeha Stream and Te Moana Road there will to be an embankment across the floodway for the bridge approaches. It is therefore not practicable to allow such rare events to overtop the proposed Expressway. Instead, the proposed Expressway design includes works to continue this floodway under the proposed Expressway and mitigate these potential effects. The solution combines the Waimeha Stream, Te Moana Road and the floodway under a multi-span bridge.

Refer Structural drawings ST-BR-550 and 551, Structural – Bridges, Volume 5, for details of this bridge.

An alternative considered was to provide large culverts under the proposed Expressway to maintain the overflow path in its current location. Such culverts would need to be large, but of low height. Such an approach is considered inappropriate because of cost, maintenance and more particularly because of the risk of blockage by debris.

If the Te Moana Road/ proposed Expressway arrangement were to be reversed, and Te Moana Road cross over the proposed Expressway, then either a long, low height, land bridge or a set of multi-cell culverts would be needed for the flow to pass under the proposed Expressway (with the risk of debris blockage) or the proposed Expressway would need to be set slightly below existing ground to allow the flow to cross over the top of it. Neither of these options was considered to be desirable and would require significant additional works or a relaxation of the proposed Expressway design standards with respect to flood clearance.

The floodway design includes the following:

- Re-grading local ground levels to divert the floodway north alongside the proposed Expressway towards the Te Moana interchange bridge, under the proposed Expressway parallel to Te Moana Road, then back to its existing alignment downstream of the proposed Expressway;
- A low entrainment stopbank to guide the flow within the diverted floodway and protect the properties along the southern side of Te Moana Road;
- Setting the southern ramps low enough so that the flows can pass over the ramps;
- Extending the proposed Expressway bridge to provide sufficient capacity for flow under the proposed Expressway while minimising flow depths; and
- Using a vertical southern abutment to maximise the flow capacity whilst minimising the length of the bridge.

The above was designed as a 3D ground surface model in MX (earthworks design software) and incorporated into KCDC's Waimeha stormwater model for testing.

As part of modelling the Waikanae River, REC reviewed GWRC's 1997 stopbank work into failure scenarios that suggested a breach flow of $80m^3/s$. The 1997 modelling occurred before the GWRC flood protection works were carried out in this area. The river morphology has also been changed by GWRC around this time (for full details of REC's investigation refer Appendix 22.D of REC's report included in Appendix 22.F).

It was agreed with GWRC CM that a breach in a 1% AEP storm is appropriate to use as a design standard. A breach in a 0.04% AEP storm was also run to understand the effects of an over-design event but not to mitigate to this level.

Initially, REC advised that a breach in the Chillingworth stopbank (refer figure 1 for location) would be the critical location that generates the greatest peak outflow during a 1% AEP storm. This was assessed at approximately $26m^3/s$. The breach hydrograph was output from GWRC's model and input into KCDC's Waimeha model for testing. The breach flow was also set up to coincide with a 1% AEP storm in the Waimeha Stream catchment. Subsequent review of the breach modelling determined that the breach flow during a 1% AEP storm would be nearer to $17m^3/s$, and that the $26m^3/s$ modelled by KCDC was closer to a breach in a 0.5% AEP. However, the higher flow rate was retained, adding some conservatism into the results. The modelling has also shown that there is very little attenuation of the breach flow down the Waimeha to the proposed Expressway bridge. This is because much of the natural floodplain storage would already be filled by runoff from the Waimeha catchment itself before a breach occurs. Therefore, the model does not significantly attenuate the breach flow.

The model runs show that the only increases in 1% AEP flood levels occurred within diverted floodway and under the interchange itself. The modelling also showed that a breach during a 1% AEP storm would not cross Te Moana Road at the interchange (cutting off through access) in what would be a significant regional emergency. However, the flow would still cross Te Moana Road further to the west as currently occurs. The proposed Expressway does not change this.

During discussions with GWRC CM, a question was raised as to whether the hydrographs adequately represented long duration storms, and it was suggested that a longer duration storm would be appropriate.

The design hydrograph used was based on an average of historical storms with the peak duration increased, somewhat artificially, by 25% to add further conservatism. Consideration of local design rainfall relationships shows that for a sustained storm of 24 to 72 hours in duration, that the hydrograph would still have some similarity to the historical storms. It would also have much lower shoulders (or sub-peaks) that would likely not be sufficiently high enough to result in additional breach flow, or to increase the peak flow in the breach.

Therefore, the modelled hydrographs are considered appropriate for representing the peak flow that might discharge though a breach in the event of a much longer duration storm event.

The estimated probability of a breach occurring at Chillingworth in a 1% AEP event is about 5%, meaning that the scenario modelled is very conservative and well above the standard that would normally be modelled for flood protection and assessment of effects for any roading project.

Modelling of the floodway shows that during a 1% AEP storm with a breach that the flood levels would remain unchanged upstream of the floodway i.e. at 5.51m and increased downstream of the floodway from 3.94m to 3.95m i.e. by only 10mm which is considered a minor effect.

In reviewing the effects of an overdesign event (i.e. an overflow of the Waikanae River stop banks as distinct from a stop bank breach), it is noted that the 0.04%AEP storm was only run in the GWRC's Waikanae River model but not in KCDC's Waimeha model. This is because KCDC's overdesign storm requirement is a 1.5x1%AEP storm, which is slightly larger than 0.04%AEP event. The GWRC model is not set up to easily determine the peak combined overflow that would occur during a 0.04%AEP storm as it spills from several areas along the upper true right bank. However, REC found that in this area overflows on the north bank only occur between Greenaway Rd and the Chillingworth stopbank and estimated a combined overflow of only approximately $3m^3/s$ in 0.04%AEP storm. Given the small quantity of overflow and that the capacity of the proposed Te Moana floodway is $32m^3/s$ in order to deal with a much larger flow from a stopbank breach, then there is not expected to be any significant increase in the flooding effects as a result of the proposed Expressway in such an event.

vii. Waimeha Stream Ramp Bridges

As the Waimeha Stream is spring fed and is has high environmental value, it was considered too difficult to achieve a satisfactory design other than by using bridges to cross the stream. In addition to the proposed Expressway bridge described above, there are two low-level bridges for the two proposed Expressway ramps joining to Te Moana Road.

Refer to Structural drawings ST-BR-600 and 650, Structural – Bridges, Volume 5 for details of these bridges.

The length of the bridges will be sufficient to span KCDC's river corridor as detailed in the District Plan. Each bridge consists of two spans each 15m long. One span will be across the main channel of the stream and the other over the stream berm.

The bridge height has been set with 600mm freeboard above the 1% AEP flood level in accordance with NZTA's Bridge Manual. The catchment is not expected to generate sufficient debris to require a 1.2m freeboard. This freeboard has been agreed as being appropriate with GWRC and KCDC.

The Waimeha Stream is highly skewed to the eastern ramp bridge and it is expected that some minor bank realignment and shaping will be needed to keep the bridge piers out of the main stream channel. Straightening the skew was considered and determined to be impractical on the advice from the Project team's Geotechnical Engineers and Ecologist who noted the presence of springs in this area would make any successful realignment technically challenging and the environmental effects would be significant.

The land between the stream and Te Moana Road will be lowered to create a floodable berm between the two ramp bridges in order to assist in conveying flood flows and to provide some minor additional flood storage. This area will also be reinstated with riparian planting along both banks of the stream, returning a significant length to a more natural state.

The modelling of the Waimeha Stream bridges and associated floodplain lowering shows the proposed works will reduce upstream 1% AEP flood levels from 3.19m to 3.16m, that is a reduction of 30mm. Downstream of the Waimeha Bridges the levels will also be reduced from 3.15m to 3.06m, or by 90mm, as a result of the attenuation in the swales and wetland 10. It is noted that the levels reported in the modelling report for this culvert are taken at the culvert whereas the levels stated above are taken from the model up and downstream of the culvert hence the difference if directly compared.

In the overdesign scenario the modelling indicates that flooding upstream of the culvert would decrease from the current level of 3.37 to 3.28m R.L as a result of the works associated with the proposed Expressway.

Under the shadow of the bridges sunlight and rainfall will be much reduced so that the stream bank vegetation cannot be relied upon to provide bank scour control. Therefore, the banks will be lined with rock rip rap. Given the relatively narrow width of the stream this will mean that the rip rap will be continued across the full width of the stream.

viii. Te Moana Interchange Culverts (culverts 24, 24.1, 24.2, 24.3)

The location of Te Moana interchange will require the filling in of several open channel drains in the area south of Te Moana Road in the area currently used for market gardening.

The west branch of the existing drain that runs through the market garden area will be diverted to a new culvert under Te Moana Road (culvert 24.1) in order to keep “clean” up catchment stormwater from mixing with proposed Expressway stormwater or effecting the efficiency of the swales or wetland 10.

Again, this drain will look more like a natural watercourse in appearance as opposed to straight engineered drain or farm ditch. Drawing CV-SW-231, Technical Report Appendices, Report 22, Volume 5 shows a typical arrangement of a similar watercourse located elsewhere on the Project. This drain would be to a smaller scale due to its smaller catchment.

Culvert 24.1 will have “low slope” culvert features (as per section 4.2.3 vi) to accommodate fish passage.

The effects on an overdesign event for these culverts are considered to be minor when compared to the flows that result from the Waikanae River stopbank breach scenario that the floodway, culverts, interchange and associated bridges can accommodate. Therefore, these relatively minor overdesign scenarios have not been considered.

The new drainage arrangement will maintain 1% AEP storm flood levels unchanged at 3.54m upstream of the proposed Expressway (not the Waimeha Stream which is discussed above).

4.5.2 Stormwater Management

i. Swales

The swales along each side of the proposed Expressway from Mazengarb Road to Otaihanga Road, approximately 1km of proposed Expressway, will drain to the WWTP Drain. These will be mostly wetland planted swales due the relatively high groundwater evidenced by the presence of several existing wetlands in the area. Refer wetland 6 for details of the attenuation performance for these swales.

The swales will also be perched above existing ground level in order to drain to wetland 6. This approach was taken for the following reasons:

- the swales are unable to completely provide the required attenuation (although this is subject to further design refinement with the view to minimising the size of wetland 6);
- there is a need to avoid discharging untreated stormwater into the above mentioned natural wetlands; and
- to minimise the amount of stormwater that infiltrates into the ground around the Otaihanga Landfill as there are concerns about the potential presence of existing

contaminated material/groundwater derived from the Landfill that could be further mobilised by new stormwater discharges.

North of Otaihanga Road, there is sufficient space for swales to drain both sides of the proposed Expressway to wetland 8. As these swales will run through sand dunes they will be grassed swales.

North of the Waikanae River bridge a swale on the eastern side of the proposed Expressway will provide treatment and attenuation of flows from the bridge to chainage 11100m. However, as the swale cannot fully achieve the required attenuation it is supplemented by offset storage area 9A prior to discharge to an existing watercourse on the Waikanae River floodplain.

At the Te Moana Interchange, the floodway and roading geometry provides significant areas of open space so that swales can be worked in around the various sections of main Alignment, ramps and local road making up the interchange. Similar to the other interchanges, runoff will be allowed to flow down the embankment batters into the swales. In instances where this is not practicable for example, where traffic barriers cut off the surface flow path, runoff will be collected in sumps and piped down the slopes into the swales. For a description of the attenuation performance of the swales on the south side of Te Moana interchange refer wetland 10.

ii. Wetland 6

Wetland 6	
Total Catchment Area	*9.4ha*
Wetland Area	*6022m²*
Water Quality Volume	*NA*
1% AEP Storage Volume	*4854m³*

Wetland 6 supplements the swales to provide attenuation proposed Expressway runoff from between Mazengarb Road and Otaihanga Road.

Wetland 6 has also been sized to provide offset storage for the floodplain storage that will be filled by the proposed Expressway embankment. This results in a significantly larger area than is needed for attenuation of proposed Expressway flows. The wetland will be bunded to allow the attenuation part of the wetland to fill up to a higher level than in the offset storage part.

Wetland 6 attenuates the 1% AEP peak flow to 70% of pre-Expressway rates.

Wetland 6 has been located on the Southern side of the WWTP drain as the area to the north has been identified as an area needed for ecological mitigation works (refer to Technical Report 26, Volume 3. It has also been located over a small sand dune in order to minimise excavation into low lying land that could contain contaminated materials and/or groundwater from the adjacent landfill. A wetland in this location will fit in well with the planned ecological mitigation wetland on the north side of the WWTP drain.

iii. Offset Storage Area 6A

As noted in section 4.5.1.ii, modelling has shown that a new culvert under the proposed Expressway will remove an existing constriction in the Landfill Drain. While this would improve the drainage upstream of this area, it would also mean that a greater volume of stormwater is allowed to move downstream during floods with the effect of increasing flooding further downstream.

In order to mitigate this effect, the culvert inlet will be designed to restrict the flow that can pass through it. This will cause water to back up into the low lying wetland area that sits between the proposed Expressway and the landfill, creating above ground offset storage, i.e. this volume is stored above the existing wetland area and does not involve widespread disturbance and then reinstatement of the ground surface in order to achieve this mitigation. REC modelling has determined that 1790m^3 is required to mitigate the increased downstream flooding. This equates to an increase in flood levels from 8.25m to 8.37m, i.e. by 120mm. As this existing flooded area is contained by the topography, this increase does not significantly increase the footprint of the flooding.

The restriction to the culvert will take the form of a weir across the inlet of the culvert with either a vertical slot or an orifice through it. The constriction provided by the slot/orifice will cause flood water to back up in order to store it but it will also maintain low flow drainage through the culvert and provides for fish passage. In case of blockage or over design storms then the stormwater can overflow the weir into the culvert.

Table 9: 1% AEP flood levels including mitigation measures.

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Offset Storage Area 6A	8.25	8.37	0.120	Increase
Downstream of Culvert 17	8.11	8.11	0.00	No Change

An alternative considered was to provide the offset storage on the western side of the proposed Expressway. This would involve constricting the drain west of the proposed Expressway, which would require construction of a cut off bund along the western property boundary. This would dam the wetland between the bund and the proposed Expressway backing up floodwater behind it. However, the volume required if the attenuation were provided via this option would need to be greater, at 2150m3. It would require much greater construction disturbance to the existing wetland and drain and would also result in the being control further away from the proposed Expressway, hindering maintenance access. For these reasons this option was not used and the minor increase in water level of the preferred option is considered to result in lesser overall effect on the environment.

iv. Otaihanga Project Yard

The main construction yard (offices, workshops, pre-casting stockpile and carpark) is proposed to be formed in part of the Otaihanga Landfill. The stormwater management for this area (including details of a temporary treatment and attenuation wetland specifically for the yard) is detailed is Technical Report 4, Volume 3 (Construction Methodology Report).

v. Muaupoko Stream

The Waikanae River bridge is located above the confluence of the Muaupoko Stream with the Waikanae River. The extent of the rock rip rap associated with protecting the river bank and piers at the bridge means that the last 30m of the stream needs to be diverted slightly. Refer Drawing CV-SW-392 and 394, Technical Report Appendices, Report 22, Volume 5 for further details.

The diversion starts at the outlet of the existing culvert under the existing access track along the south bank of the Waikanae River, and follows the curve of the rip rap armouring. As shown on Drawing CV-SW-394, Technical Report Appendices, Report 22, Volume 5 the rip rap will be continued below the stream invert with reinstatement of the stream substrate to facilitate the re-establishment of the natural stream bed and banks. The banks of the new diversion will be riparian planted with natives to reflect the cultural, ecological and community significance of this part of the stream.

The diversion of the Muaupoko Stream is not expected to affect the flood levels in the stream or the Waikanae River.

vi. Wetland 8

Wetland 8 provides attenuation and treatment for the proposed Expressway from Otaihanga Road to the north side of the Waikanae River bridge, some 1.6km in length.

The wetland will be supplemented by the attenuation provided in the swales draining to Wetland 8.

The wetland will consist of a core treatment wetland with a wider flood attenuation area.

Wetland 8	
Total Catchment Area	*13.8ha*
Wetland Area	*1625m²*
Water Quality Volume[42]	*560m³*
1% AEP Storage Volume	*1850m³*

Due to the fall provided by the proposed Expressway's southern approach to the Waikanae River bridge it is more appropriate to drain the bridge approach with sumps and pipes. This also allows the runoff from the bridge to be discharged into a piped system and conveyed to the wetland for treatment.

The outlet from this wetland will be linked into a new open channel drain that will run from the outlet of culvert 22 to the Muaupoko Stream.

Wetland 8 attenuates the 1% AEP peak flow to 77% of pre-Expressway rates. While this meets the target 80% of pre-Expressway rates, in practice the effects of this attenuation on the Waikanae River floodplain are insignificant due to the much greater relative size of the Waikanae River flood flows.

[42] The area is less than the guidelines of 2% of total catchment area as large parts of the catchment are being served by swales.

Initially the wetland was located on the western side of the proposed Expressway. However, this would have resulted in a more complicated arrangement for maintaining a “clean” water bypass around the wetland while also keeping the stored floodwater from spilling out onto neighbouring private property. Therefore the option of locating the wetland east of the proposed Expressway has been adopted.

vii. Wetland 9

Wetland 9	
Total Catchment Area	*7.6ha*
Wetland Area	*1520m²*
Water Quality Volume	*593m³*
1% AEP Storage Volume	*6206m³*

Wetland 9 provides attenuation and treatment for the proposed Expressway from just north of the Waikanae River bridge to chainage 11,300m, some 400m of proposed Expressway.

This wetland is located on the site of the existing highly modified Tuku Raukau ponds and will make use of the ponds as a base for developing the wetland.

Swales have not been used in this area due to the spatial restrictions involved with this section of the proposed Expressway. The Alignment through this area has sought to minimise the extent of the private property take and the effects on the Takamore Wāhi Tapu precinct whilst balancing other factors such as stormwater treatment. Together, these factors result in wetland 9 needing to provide all of the attenuation and treatment without being supplemented by swales.

The area around this wetland is very low lying and as such the groundwater is very close to the surface with poor drainage and significant flooding risks. The proposed Expressway passes through this area and takes up some of the floodplain storage. To mitigate the effect of this on flood levels, wetland 9 is designed to provide significantly greater attenuation of peak flows being discharged into KCDC’s drainage than the 80% target used elsewhere. This method of mitigation does not provide offset by way of additional floodplain storage, but rather acts on the flows being discharged to achieve the same overall outcome downstream.

In order to maximise the attenuation achieved, the small upstream catchment to the west of the proposed Expressway will be drained directly into the wetland as opposed to bypassing it. However, within the wetland itself, there will be separate areas for treatment and flood storage. The treatment component of the wetland has a much smaller footprint than the attenuation component. This allows the proposed Expressway runoff to be discharged to an area of the wetland that is separate from the “clean water” inflows noted above.

Wetland 9 attenuates the 1% AEP peak flow to only 9% of pre-Expressway rates. The effects of this attenuation are localised to the floodplain behind the Waikanae River stopbank. The effects on flooding in the River itself are insignificant due to the much greater relative size of the Waikanae River flood flows.

Due to the high groundwater level the attenuation component of the wetland needs to be achieved by constructing an enclosing bund around the wetland to enable flood storage be contained above existing ground. This bund is expected to have a height of around 1.5 to 2m with the final height subject further detailed design; however, it will be tied in with the much higher noise bund along the proposed Expressway.

It was recognised during the design development of wetland 9 that the footprint was very large and it would be desirable to minimise this as much as possible. This was due to the potential for the wetland to increase groundwater levels around it. It is therefore expected that this wetland will be lined with peat or clay to minimise these effects. Also a section of the proposed Expressway (north of Waikanae River) will be diverted to drain back to wetland 9A.

The small pipe drain from the existing ponds is not suitable to be retained as the outlet for the new wetland and so it will be blocked off and a new 300mm diameter outlet drain constructed to connect the wetland into KCDC's downstream drainage network, which is the same size.

Even when wetland 9's proposed Expressway catchment is minimised and the attenuation is greatly increased, KCDC's model still shows a residual effect of a KCDC drainage manhole in Puriri Road overflowing. This is a result of a combination of local catchment flooding (already present in this area) and the discharge from wetland 9 into this drain. As the attenuation achievable in wetland 9 is already maximised an alternative solution is needed to mitigate this effect. SKM advise that providing a further 263m^3 of offset storage outside of the wetland will remedy this effect. This volume will be provided by lowering the ground level of a small area adjacent the wetland.

Table 10: 1% AEP flood levels including mitigation measures.

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Area around Wetland 9 (not in wetland 9)	4.0	4.0	0.0	No change

An earlier alternative solution involving a complete replacement of the current 2x250L/s pump station with a much larger 1,000L/s pump station solution was tested in KCDC's model and found to match the overall effects provided of the attenuation in wetland 9 but this option would prove to be costly and involve significant on-going pump running costs and so was discounted.

viii. Offset Storage Area 9A

Offset storage area 9A provides attenuation for a section of the proposed Expressway north of the Waikanae River bridge. It supplements a swale that drains into it from the north. This area was designed in order to reduce the overall size of wetland 9 further to the north.

The outlet from this wetland will discharge into an existing watercourse to the north of the Waikanae River bridge.

Area 9A attenuates the 1% AEP peak flow to 55% of pre-Expressway levels. As with wetlands 8 and 9, the effects of this attenuation on the Waikanae River floodplain are insignificant due to the much greater relative size of the Waikanae River flood flows.

The attenuation will be provided by the upstream swale and above ground storage in area 9A. This will be achieved by constructing an earth bund across the end of the basin formed by the proposed Expressway embankment and the existing topography.

While area 9A will provide attenuation of flows being discharged into the Waikanae River (from 0.244m^3/s down to 0.133m^3/s), it will have very little real benefit to flooding on the river floodplain. This is because the 1%AEP flow in river is so much larger i.e. 480m^3/s than what discharges from area 9A. This difference is so big that if the flow were not attenuated then the flood level on the Waikanae would not measurably increase as a result. It may even act to marginally increase the flood level on the Waikanae floodplain by holding back the discharge to coincide more with when the peak flow rate occurs in the Waikanae River (which takes much longer to reach its peak level than the smaller area 9A catchment).

The final attenuation requirements in this area, and therefore the need for area 9A, will be reviewed during later design stages of the Project.

ix. Wetland 10

Wetland 10 supplements the attenuation provided by the swales on the south side of Te Moana interchange.

The outlet from this wetland will discharge into a new watercourse that in turn is culverted under Te Moana Road to the Waimeha Stream.

Wetland 10 attenuates the 1% AEP peak flow to 79% of pre-Expressway levels. This will be beneficial to local flooding in the area.

Wetland 10	
Total Catchment Area	*8.3ha*
Wetland Area	*1071m^2*
Water Quality Volume	*NA*
1% AEP Storage Volume	*750m^3*

Runoff from approximately 300m of Te Moana Road will now be served by swales and wetland 10 providing attenuation and treatment where there currently is none. This will give an overall benefit from the existing situation.

x. Isolated Catchment at Chainage 12,100m (culvert 25.3)

North of the Waimeha Stream, the proposed Expressway crosses an undeveloped rural catchment that has no drainage outfall i.e. it is an enclosed “basin” surrounded by sand dunes on all sides.

This area currently floods to a level of 4.1m RL in a 1% AEP storm. Once the proposed Expressway embankment has filled in part of this basin this level will increase to 4.26m, i.e. by 120mm.

Table 11: 1% AEP flood levels excluding mitigation measures still to be agreed with land owner.

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Upstream of Culvert 25.3	4.10	4.26	0.120	Increase
Downstream of Culvert 25.3	4.10	4.26	0.120	Increase

Culvert 25.3 allows the flows on both sides of the proposed Expressway to balance flood levels out. As the basin has relatively steep sides, the increase in flooded area is not large. In this instance, the effect of the proposed Expressway is considered to be minor.

Mitigation with 5,000m^3 of offset storage could be provided but will require taking significantly more private land and more importantly, disturbing a large area of land. This may have effects that are worse than allowing flooding to a slightly higher level on land that is already subject to flooding. NZTA is currently negotiating a wider property agreement with this owner (this property is one of the largest single block of private land that the proposed Expressway will cross) and the flood effects are part of that process. The proposed Expressway Designation that is part of these negotiations has sufficient area to provide for the offset volume if an agreement is not reached over this issue with the property owner.

4.6 Sector 4 – Waikanae North

The key stormwater features in this Sector include:

- Swales;
- Offset storage areas 10C and 10D;
- Ngarara Creek culvert (culvert 26);
- Wetlands 10A & B;
- Wetlands 11 A & B;
- Wetland 12;
- Kakariki Stream bridges;
- Offset storage area 11;
- Smithfield Drain diversion;
- Culverts between Kakariki Stream and Paetawa Drain;
- Paetawa Drain bridge and floodplain;
- Culvert 38;
- Paetawa Drain offset storage area;
- Peka Peka interchange; and
- Hadfield / Te Kowhai Stream culvert (culverts 40 through 40.3)

Refer Drainage Layout drawings CV-SW-120 through 132, Technical Report Appendices, Report 22, Volume 5 and the schedule of watercourse crossings included in Appendix 22.B.

4.6.1 Waterway Crossings

i. Ngarara Creek Culvert (culvert 26)

This stream discharges to the Te Harakeke/Kawakahia wetland. Modelling has confirmed that the Ngarara Creek culvert needs to be a 3x2m box culvert in order to keep the effects on flood levels to a minimum.

The effect of the culvert is to slightly increase 1% AEP flood levels from 6.10m to 6.14m i.e. by 40mm this increase is considered to be minor as the existing flood area is contained by relatively steep hill slopes such that the increase in level does not correspond to a significant increase in flooded area. Downstream levels are unchanged from existing at 6.12m. It is noted that the levels reported in the modelling report for this culvert are taken at the culvert whereas the levels stated above are taken from the model up and downstream of the culvert hence the difference if directly compared.

In the overdesign scenario the modelling indicates that flooding upstream of the culvert would unchanged from current the level of 6.37m R.L as a result of the works associated with the proposed Expressway.

This culvert will have “natural stream bed” culvert features (as per section 4.2.3 vi.) to accommodate fish passage.

ii. Kakariki Stream Bridges

The proposed Expressway crosses the Kakariki Stream at a location where the stream cuts though a line of north-south running sand dunes. The existing topography in this location lends itself to the proposed Expressway crossing being made by a bridge.

Refer Structural drawings ST-BR-900, Structural – Bridges, Volume 5 for details of this bridge.

The stream is also seen by many as part of an important ecological corridor that links Te Harakeke/Kawakahia wetland to the Nga Manu Bird Sanctuary (within a wider corridor from Kāpiti Island to the Tararua Ranges). The ecological importance of this stream further promotes a bridge crossing over that of a culvert.

A culvert was initially tested in this location but it soon became apparent that a culvert would need to be very wide (more than 10m) in order to minimise the effect on upstream flooding.

The Kakariki Stream Bridge is skewed in alignment as it crosses the stream but it is crossed in a single span so there are no piers in the main channel or floodway. The bridge has an 18m span with vertical abutments set at the top of the Stream banks and above the 1% AEP flood level.

In order to minimise the bridge span the Kakariki Stream will be realigned and straightened where it passes under the proposed Expressway. This will involve approximately 60m of new rip rap lined stream channel (of matching cross section) with 10m and 25m long planted transitions both upstream and downstream respectively. Refer drawing CV-SW-232, Technical Report Appendices, Report 22, Volume 5.

The effect of the bridges and associated offset storage area 11 is to reduce upstream 1% AEP flood levels from 6.41m to 6.36m, that is a reduction of 50mm. Downstream levels are also reduced from 6.23m to 6.18m, i.e. by 50mm.

Upstream of the proposed Expressway, there will also be a new local road bridge. Refer to Structural Drawing ST-BR-950, Structural – Bridges, Volume 5 for details of this bridge.

There is an existing farm access bridge in same location as the proposed local road bridge that acts to control the flow and flood levels upstream of it. This bridge will be removed to allow the construction of the new local road bridge. The new bridge will be sized to be clear of the 1% AEP flood level and extents in order to accommodate any possible future stormwater upgrades carried out KCDC (i.e. seeking to reduce flood risk further upstream). However, at this stage the Project is focused on resolving effects of the proposed Expressway so the channel beneath the bridge will be sized such that the constriction is maintained and in the event of future upgrades the channel could then be widened under the bridge by removing the constriction. Removing this constriction now might otherwise result in increased flood levels downstream of the proposed Expressway which would then require mitigation.

However, even though the constriction is being maintained, the benefits provided by offset storage area 11 do result in improvements in flood levels to areas upstream of this bridge. The 1% AEP flood level is reduced from 7.69m to 7.64m, i.e. by 50mm.

As with the other bridges, light and rainfall will be restricted under these so that riparian vegetation cannot be relied upon to provide bank scour control. Therefore, the banks will be lined with rock rip rap under the footprint of the bridge. Given the relatively narrow width of the stream this will mean that the rip rap will be continued across the full width of the stream.

iii. Culverts Between Kakariki Stream and Paetawa Drain (culverts 30.4 through 35.1)

The proposed Expressway crosses several smaller farm drains between the Kakariki Stream and the Paetawa Drain to the north. These watercourses will all be culverted under the proposed Expressway, with pipes ranging from 1050mm to 1800mm diameter.

All but culvert 35.1 will have “low slope” culvert features (as per section 4.2.3 vi.) to accommodate fish passage. Culvert 35.1 is not on a watercourse as its purpose is to maintain a flood flow path during the 1% AEP storm and will otherwise remain dry.

iv. Paetawa Drain Bridge

As with the Kakariki Stream culverting the Paetawa Drain was found to require such a large culvert that it became more practical to construct a bridge.

Refer Structural Drawing ST-BR-960, Structural – Bridges, Volume 5 for details of this bridge.

The drain carries flood flow from a very large catchment and eventually discharges to Te Harakeke/Kawakahia wetland.

The Paetawa drain is slightly skewed as it crosses the road and the bridge has a 10m wide single span which avoids having any piers in the channel or floodway. The bridge will provide the capacity for the 1% AEP flows.

The effect of the bridge and associated offset storage areas 13 & 13A is to reduce upstream 1% AEP flood levels from 9.45m to 9.41m, that is a reduction of 40mm. Downstream levels are maintained at 9.13m. Much further downstream the flood levels are improved as a result of the works, in areas by up to approximately 130mm. It is noted that the levels reported in the modelling report for this culvert are taken at the culvert whereas the levels stated above are taken from the model up and downstream of the culvert hence the difference if directly compared.

In the overdesign scenario the modelling indicates that flooding upstream of the culvert would decrease from current the level of 9.5m to 9.4m R.L as a result of the works associated with the proposed Expressway.

As with the other bridges, light and rainfall will be restricted under the bridge so that riparian vegetation cannot be relied upon to provide bank scour control.

Therefore, the banks will be lined with rock rip rap under the footprint of the bridge. Given the relatively narrow width of the drain this will mean that the rip rap will be continued across the full width of the drain.

v. Culvert 38

Within the Peka Peka interchange area the proposed Expressway will cross a tributary to the Paetawa Drain. A 3x2m box culvert has been modelled and the effects of this culvert are listed in Section 4.6.2vii detailing the effect of offset storage area 13A.

To accommodate fish passage, culvert 38 will have “low slope” culvert features (as per section 4.2.3 vi.) except that the inlet control structure will require the inlet to be narrower than the approach channel. However, flow will be maintained though the inlet without a vertical step in order to provide for fish passage.

vi. Peka Peka Interchange Local Road Culverts (culverts 38.1 through 38.4 and 39)

The local roading/existing SH1 changes associated with the Peka Peka interchange affects several watercourses.

Culvert 38.1 is a 3x2m box culvert that conveys a tributary of the Paetawa Drain that will be filled in by the proposed roundabout. This north branch of the Paetawa Drain crosses the existing SH1 in two 900mm diameter culverts approximately 80m apart. With the realignment of the road associated with the roundabout the existing SH1 is shifted further to the west away from the NIMT railway. This creates enough space so that these drains can be consolidated into a single culvert with a new section of open channel drain. The southern branch was chosen to remain, as this would require the shortest length of culvert thereby having the least effect.

Culvert 38.2 will be an extension of an existing culvert that drains a small road catchment and does not receive any flow from east of the railway.

Culvert 38.3 drains the area of land enclosed by the proposed Expressway and the new southern local road crossing connecting to Peka Peka Road. It drains into the new section of open channel drain west of the new connector road.

Culverts 38.4 and 39 drain a large hillside catchment that will be culverted under the railway into a new watercourse. These culverts are also supplemented by the existing 750mm diameter culvert that will remain.

Several existing farm drains need to be diverted to avoid having to culvert them under the proposed Expressway. These drains still ultimately discharge into the Paetawa Drain, although the route to the drain has been amended to assist with providing offset flood storage 13A. These drains will have a slight meander to them and their banks planted with riparian vegetation. They will look more like natural watercourses in appearance than straight engineered drains or farm ditches. Drawing CV-SW-231, Technical Report Appendices, Report 22, Volume 5 shows a typical arrangement of a similar watercourse located elsewhere on the Project. The drains around Peka Peka interchange will be similar to this detail but on a smaller scale.

To accommodate fish passage, culvert 38.1 will have “natural stream bed” culvert features (as per section 4.2.3vi of this report and the guideline in Appendix M of the CEMP, Volume 4), and culverts 38.3, 38.4, and 39 will have “natural stream bed” culvert features.

vii. Hadfield / Te Kowhai Stream Culverts (culverts 40 to 40.3)

North of Peka Peka Road the proposed Expressway will cross the Hadfield/Te Kowhai Steam, the last significant watercourse in the Project. Initial designs considered a single 3x3m box culvert crossing on the existing stream alignment. However, this would require the culvert to be 130m long to clear all of the converging on/off ramps of the Peka Peka interchange. This was considered to be undesirable from a maintenance, environmental (“closing in” a large length of stream) and cost perspective.

An alternative alignment was therefore sought that was straighter, running from the existing rail bridge directly beneath both ramps and the proposed Expressway. This allowed significant stretches of open channel to be incorporated into the design as well as reducing the overall length of culvert down to approximately 100m.

Removing the existing 90 degree bend upstream of the proposed Expressway will have a favourable effect on gravel transport in the stream bed as it runs through the culvert, reducing the likelihood of deposition within the culvert.

Between culvert 40.2 and 40.3, there is a 150m long stretch of new stream channel that connects the stream back to the existing drain downstream of the new local connector road.

These culverts will have “natural stream bed” culvert features (as per section 4.2.3 vi.) to accommodate fish passage.

4.6.2 Stormwater Management

i. Swales

North of Te Moana interchange the proposed Expressway swales drain from chainage 12,750m back to the Waimeha Stream. These do not currently meet the target 80% attenuation of peak flows. The 1m deep swales currently attenuate to 98% of the pre-Expressway flows. Deeper or wider swales with check dams are expected to result in attenuation meeting the 80% target. Refining the swale design for this area will be carried out during detailed design.

Between chainage 12,750m and 13,550m, swales drain into wetlands 10 A and B for extra water quality polishing before discharging into Ngarara Creek. For attenuation performance, refer wetlands 10A and 10B.

From Ngarara Road the swales will drain down to Kakariki Stream and discharge into wetland 11A, again for extra water quality polishing prior to discharge into the stream. For attenuation performance, refer to wetland 11A.

From just north of Waimeha Stream through to Kakariki Stream the proposed Expressway will pass through broken sand dune country so the swales will be predominantly grassed rather than wetland planted.

North of Kakariki Stream the landform reverts back to low lying peat flats with intermittent sand dunes, similar to the Raumati straight at the southern end of the proposed Expressway. The swales north of Kakariki Stream will therefore generally be wetland planted.

The swales from Kakariki Stream to chainage 15,000m generally drain back to wetland 12. For the attenuation performance, refer wetland 12. The exception to this is the swale that outlets into the diverted Smithfield Drain at chainage 14,630m.

This swale will attenuate the 1% AEP peak flow to 57% of pre-Expressway rates. This will have an overall benefit to existing flooding in the area.

Between of chainages 15,000m and 16,800 the swales will discharge at each of the watercourses encountered along the route. These swales attenuate the 1% AEP peak flow from between 6% to 79% of pre-Expressway rates. One swale attenuates to 81% but further refinement during detailed design will bring the attenuation under the 80% target. This will have an overall beneficial effect on existing flooding in the area.

The swales around the Peka Peka interchange provide attenuation and treatment to between 8% and 79% of pre-Expressway peak flows.

ii. Wetlands 10A & B

Wetlands 10A and B will supplement the treatment provided by the swales that discharge into Ngarara Creek. These wetlands have been designed to "polish" the stormwater prior to its discharge into the watercourse, given that is close to Te Harakeke/Kawakahia Wetland.

Wetland 10A	
Total Catchment Area	*5.1ha*
Wetland Area	*450m^2*
Water Quality Volume	*235m^3*
1% AEP Storage Volume	*NA*

Wetland 10B	
Total Catchment Area	*2.2ha*
Wetland Area	*1020m^2*
Water Quality Volume	*534m^3*
1% AEP Storage Volume	*NA*

Wetlands have been used for this extra water quality treatment as they provide treatment via a different mechanism to swales, giving a better overall treatment outcome. Adding in more swales or fine tuning the swale design, would not provide the same level of water quality benefits as would occur when treatment devices of differing functionality are used in series. Wetland 10A and the swales that drain into it attenuate the 1% AEP peak flows to 79% of pre-Expressway levels. This will have a beneficial effect on local flooding in the area.

Wetland 10B and its associated upstream swales attenuate the 1% AEP peak flows to 74% of pre-Expressway rates. This too will have a beneficial effect on local flooding in the area.

iii. Offset Storage Areas 10C & D

North of Waimeha Stream the proposed Expressway passes through land made up of sand dunes with low lying inter-dune areas. In two locations (chainage 12,950m and 13,400m) the low lying areas filled in by the proposed Expressway embankment cause a loss of flood storage resulting in an increase of the 1% AEP flood levels.

The 1% AEP flood level in area 10C is increased from 7.54m to 7.84m, i.e. by 300mm. The increase in area 10D is from 7.82m to 8.02m, ie by 200mm.

Providing offset storage in these areas will address these increases by providing the same volume that has been filled in. This will mitigate the increase and maintain the flood levels at existing.

Modelling has determined that at offset area 10C needs to be 1500m^3 in volume and area 10D 500m^3. The existing ground level in both of these areas is relatively variable in height and also higher than the groundwater level. This allows the volume to be created by excavating down into the ground in areas contained within the proposed Expressway Designation. These areas would then be reinstated with mass native planting.

iv. Offset Storage Area 11

Similar to the situation where the proposed Expressway crosses the Wharemauku floodplain, the proposed Expressway also passes through a floodplain associated with the Kakariki Stream. The volume the proposed Expressway needs to provide to offset these effects is approximately 25,000m^3 and has been determined by testing in KCDC's stormwater model. The distribution of this volume is restricted by the relatively high groundwater levels, again similar to Wharemauku. It is expected with further design iterations that the storage areas can be optimised further and the required volume may be reduced.

The land identified in the drawings to provide the offset flood storage is land that is currently inundated by the 1% AEP flood. Given the very large volume required there are few, if any, practical alternatives available to readily provide the volume required.

The volume will be achieved by lowering the ground level within this area and allowing the land to flood to a greater depth than it currently does. Doing this will bring the ground level down much closer to the groundwater table and make it difficult to return parts of it back to pasture. Similar to offset storage areas 0C, 2 and 3A the finished ground will be formed to produce shallow depressions and mounds to and be planted as wetland areas.

This area also includes the following related elements (refer below sections):

- The Kakariki Stream and riparian planting;
- Wetlands 11A, 11B and 12; and
- The existing and diverted Smithfield drains and riparian planting.

The interrelated layout of these features and high groundwater makes this area ideally suited to being reinstated as a wetland complex. This approach will be consistent with its environmental context, given the proximity of both Te Harakeke/Kawakahia wetland and the Nga Manu Bird Sanctuary. Initial discussions along these lines have been held with KCDC and the Nga Manu Bird Sanctuary Trust.

The effects of this mitigation are detailed in Table 12 below.

Table 12: 1% AEP flood levels including mitigation measures.

Location	Pre-Expressway (m RL)	Post-Expressway (m RL)	Diff (m)	Effect
Expressway bridge downstream	6.23	6.18	0.05	Lower
Expressway bridge upstream	6.41	6.36	0.05	Lower
Offset storage area 11	6.62	6.58	0.04	Lower
Local road bridge upstream	7.69	7.64	0.05	Lower

Parts of storage area 11 include low sand dunes that it may be beneficial to borrow for fill into the proposed Expressway. If this is carried out then these areas (shown shaded yellow on Drawing CV-SW-125, Technical Report Appendices, Report 22, Volume 5) could be backfilled with excess peat from the Project to maintain and reinstate the natural dune/rolling landscape form in this area.

v. Wetlands 11 A, B and 12

Wetland 11A has been included for the same reason as Wetlands 10A and B, that is, for treatment. The attenuation requirements are provided in the wider offset storage area 11. Again this is due to the proximity of Te Harakeke / Kawakahia Wetland.

Wetland 11A also provides treatment for runoff from part of the new Smithfield Road connection.

Similarly, wetland 11B provides and treatment for the northern part of the new Smithfield Road.

Wetland 12 provides attenuation and treatment for the proposed Expressway from Kakariki Stream to chainage 15,000m.

Wetland 12 will attenuate the 1% AEP peak flows to 82% of pre-Expressway rates almost achieving the 80% target. The 80% level will be achieved with further refinement during detailed design.

Wetland 11A	
Total Catchment Area	*0.8ha*
Wetland Area	*200m²*
Water Quality Volume	*113m³*
1% AEP Storage Volume	*NA*

Wetland 11B	
Total Catchment Area	*0.9ha*
Wetland Area	*142m²*
Water Quality Volume	*250m³*
1% AEP Storage Volume	*NA*

Wetland 12	
Total Catchment Area	*3.9ha*
Wetland Area	*786m²*
Water Quality Volume	*496m³*
1% AEP Storage Volume	*782m³*

vi. Smithfield Drain Diversion

The proposed Expressway Alignment passes over the top of approximately 510m of an existing farm drain that runs parallel with the proposed Expressway.

In order to keep the proposed Expressway runoff separate from the upstream "clean water" flows the drain needs to be diverted. The diversion provides the opportunity to restore the drain to a more natural stream-like character. This will include a 5m wide zone of retired pasture, extensive riparian planting and a meandering alignment that appears more natural than an engineered drain. This also fits in well with the philosophy behind offset storage area 11 and the natural/ecological enhancements that will result. Drawing CV-SW-231, Technical Report Appendices, Report 22, Volume 5 shows the typical diversion details.

The detailed design of the stream diversion will be carried out in accordance to the principles discussed in Section 4.2.3 viii and the guidelines provided in Appendix M of the CEMP, Volume 4 (Ecological Management Plan).

vii. Paetawa Drain Offset Storage Area 13 & 13A

Similar to the Wharemauku and Kakariki floodplains (and offset storage areas), the proposed Expressway will pass through a floodplain associated with the Paetawa Drain. The flood storage volume the proposed Expressway will take up needs to be offset in order to mitigate the effects on flooding.

The volume required to offset these effects is approximately 36,000m^3 and has been determined by testing in KCDC's stormwater model. The distribution of this volume is again restricted by the relatively high groundwater levels and the proximity of the NIMT railway (because if groundwater is drained down there is a risk of settlement occurring to the railway). Again, similar to the other three large offset areas 2, 3A and 11, it is expected with further design iterations that the storage areas can be optimised further and the required volume and/or footprint may be reduced.

Subsequent to the modelling being completed, NZTA advised that it would be preferable to contain the storage on one property that will be purchased as part of the Project. This is why the offset area 13 is shown in a different location on the drawings than the modelling report. The volume provided is the same and the benefits are expected to be similar to that modelled.

The layout of the Peka Peka interchange has resulted in a large area of land being effectively cut off by the new local roads and the proposed Expressway embankments. This above ground "basin" of land makes for an ideal location to create flood storage. However, it is very close to the railway meaning the storage must be provided without lowering the groundwater i.e. by not lowering the existing ground surface.

The embankments formed by the new roads present the possibility of controlling the extent of ponding upstream of culvert 38 and provide a ready-made area for ponding of flood water. This area can be used to store flood water above existing ground if upstream flows can be diverted to it, and the outlet constrained so that the water ponds. This is storage area 13A.

The restriction described above will be achieved by constructing a weir across the inlet of culvert 38 with either a vertical slot or an orifice through it. The constriction provided by either of these will cause flood water to back up but also allows low flow drainage through the structure to be maintained. The weir also allows for the possibility of blockage or over-design storms as the stormwater can overflow the weir and drain through the culvert.

Due to the flood storage being above ground the existing pastoral land need not be changed.

However, not all of the required storage volume can be provided in this one location. Approximately 6,000m^3 of the total needs to be found elsewhere, and this is the purpose of storage area 13. In order to have minimal impact on groundwater and to also allow this to be reinstated in pasture, the volume will be achieved by lowering the ground only by an average of 140mm in height.

Further design refinement is expected to reduce the requirement for two separate storage locations with the goal of providing all of the required storage in Area 13A. The significant increase in storage area 13A will be fully contained within the proposed Expressway footprint.

The effects of this mitigation are detailed in Table 13 below.

Table 13: 1% AEP flood levels including mitigation measures.

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Downstream of the Expressway in the Paetawa Drain	9.13	9.13	0.00	No change
Upstream of the Expressway in the Paetawa Drain	9.45	9.41	0.04	Lower
Offset Storage Area 13	8.90	8.92	0.02	Increase
Offset Storage Area 13A	8.41	10.50	2.09	Increase

The increased flood levels in the offset storage areas are a result of the function of storing floodwater. This increase does not flood land other than that of the storage area.

viii. Offset Storage North of Peka Peka Road

There is a flood prone rural area north of Peka Peka Road, associated with Hadfields Drain. As far as we are aware, there is no estimate available of flood levels in this area. KCDC have not carried out flood modelling in this area, and any modelling to define flood levels would need to cover a very extensive area well beyond the proposed Expressway to be definitive.

We have undertaken a volumetric analysis of the flood plain in this immediate area to assess the relative effects of the proposed Expressway footprint and loss of flood plain on flood levels. This shows that there could potentially be an increase of flood level over the lowest-lying areas of up to 180mm in a 1% AEP storm. Our assessment of contour data and likely flood level ranges suggests there is a possibility that the risk of flooding of dwellings could slightly increase, further downstream. The topographic data in that area, remote from the proposed Expressway corridor, is inadequate for a conclusive assessment, and the alternative is to address the risk by avoiding it.

There are two means by which this could be addressed. The first is to increase the Designation to the west to provide an area for additional flood storage to be created. The second, and preferred solution, would be to provide an offset storage area similar to area 13A south of Peka Peka Road, where local catchment flows are attenuated as an offset to the lost storage. This would require the further diversion of Hadfield / Te Kowhai Stream into the area bounded by the proposed Expressway, the new Peka Peka roundabout and the new local road west of the proposed Expressway. This would allow for an above ground storage area to be created minimising land disturbance. Details of this area will be confirmed during later design stages of the Project.

4.7 Operation and Maintenance

On completion, the proposed Expressway will become part of the State highway system. It will be managed as part of NZTA's maintenance Contract 497N which currently is in the form of a single hybrid performance contract involving both contractor and engineering responsibilities. This section of State highway comes under the Wellington Regional Office of NZTA.

Operational and maintenance requirements will in due course need to be incorporated into the maintenance contract. Maintenance activities for stormwater devices such as swales and wetlands are well established. They are set out in some detail in NZTA's stormwater treatment standard, and specifications have also been developed for other projects, particularly the Auckland Motorway Alliance.

Prior to handover, it will be important that the vegetation associated with the stormwater works is fully established. For wetlands, this will require at least two years of intensive maintenance.

Principal features of the operation and maintenance of the stormwater systems are:

- A regular programme of inspection and reporting for all devices, including swales, wetlands, pipe systems and culverts, to confirm they are fully functional, and identify any maintenance required;
- In wetland swales and treatment wetlands, intensive maintenance for the establishment period;
- Regular mowing of grass swales, to maintain the grass typically in the 50mm to 150mm height range;
- As a general rule grassed flood storage areas will be leased for grazing, where not planted with native vegetation and where suitable on a site-by-site basis; and
- When sediment and contaminant build-up in wetlands or swales is such that it reduces the effective capacity beyond that required by the design, the accumulated sediment will need to be excavated, and the topsoil and vegetation re-established.

Works in the Waikanae River outside the main proposed Expressway corridor are expected to be handed over to GWRC for ongoing maintenance, once vegetation is well established. The Project will take responsibility for maintenance of the protection works up to handover, and will continue to maintain the works associated directly with protection of the bridge structure.

5 Effects assessment and conclusions

5.1 Hydrological Effects

5.1.1 Summary of Land Type and Topography

The proposed Expressway passes through two main land types with distinctly different characteristics. The first is the low lying flat lands containing peat and is associated with characteristically high groundwater. The other is the sand dune formations which are relatively free draining in comparison to the peat.

The location of the peat and sand dunes is a result of historic geomorphological processes such as coastal dune migration and the wind. There are three large peat flat areas along the proposed Expressway: QE Park, around Wharemauku Stream and either side of the Paetawa Drain.

The proposed Expressway crosses many watercourses both large and small. It also crosses several floodplains, the largest of these is the Waikanae River Floodplain but the Wharemauku Stream, Kakariki Stream and Paetawa Drain floodplains are also sizable.

The above factors influence both the potential effects, and the measures taken to avoid, remedy or mitigate those effects.

5.1.2 Groundwater Effects

The effects of the swales, wetlands and offset storage areas on groundwater has being investigated, modelled, assessed. For details of this work refer Technical Report 21, Volume 3 (Assessment of Groundwater Effects). This report found that assessed changes in groundwater level or flows to wetlands were expected to be negligible.

An objective of the swale and wetland designs is to have them operating as close as possible to current groundwater levels. This is particularly the case with the wetlands and the swales located in areas of peat. This avoids adverse effects on local groundwater and by extension on existing wetlands adjacent the proposed Expressway and risk of settlement beyond the Project boundaries.

For the same reason, there will not be any more than a minor change in the discharge of groundwater from the peat areas into receiving watercourses i.e. only minor change from what currently occurs.

Where the operational ranges of the wetlands are outside the range of existing groundwater levels then they will be lined to minimise the effect on surrounding groundwater. This is required in wetlands 0A, 9, 10A and 10B. This also serves to avoid or minimise any potential hydrological effects on groundwater.

Offset storage areas 2, 3A and wetland 3 will require the lowering of local groundwater by up to 600mm. The effect of this on the Wharemauku Stream is not considered to be significant. Another effect is the potential for settlement outside of the Project boundaries.

The geotechnical investigations have also concluded that the groundwater drawdown from construction of the proposed Expressway and stormwater devices is expected to result in a negligible reduction in the volume of groundwater discharging to surface water bodies and/or potentially a negligible increase in the amount of water in surface water bodies that is lost through their beds to the groundwater system.

All of the above issues are discussed in more detail in Technical Report 21, Volume 3 (Assessment of Groundwater Effects) and Technical Report 35, Volume 3 (Assessment of Ground Settlement Effects).

5.1.3 Low Flow Effects

The low or normal flows have been determined to assist in understanding and characterising existing catchments and watercourses and also for input to the ecological investigations carried out for the Project.

These flows were sourced from NIWA's WRENZ model, where available; otherwise the Project team has estimated them from available information. The results are presented in Appendix 22.C.

The proposed Expressway stormwater system is not expected to significantly change the low flows in watercourses, because these are primarily determined either by upstream catchment characteristics, or by local shallow groundwater inflows / outflows. Because the design seeks to minimise the changes in local groundwater level, these flows will exhibit only minor change. For further information on assessment of effects of low flows and related groundwater effects refer Technical Report 21, Volume 3 (Assessment of Groundwater Effects).

5.1.4 Climate Change Effects

KCDC / GWRC's models include climate change increases in rainfall intensity (+16%) and sea level rise (+0.8m). These allowances have been developed by KCDC from a specific risk assessment carried out by NIWA in 2005 and updated in 2007. These are in accordance with MfE guidelines and represent the most up to date and officially accepted understanding of climate change in the District.

The modelling of watercourse crossings has shown that the Project is relatively insensitive to sea level rise as it is generally upstream of the tidal zone influence (except marginally at the Waikanae River and Waimeha Stream).

Increased rainfall leads to increased stormwater runoff which has been accounted for in the sizing of culverts and bridges.

If this increase in rainfall affects groundwater levels, then it is not expected to affect the proposed treatment wetlands and swales. This is because these have a gravity drainage outlet that maintains the permanent water level. Therefore, if groundwater were to regionally rise then the water levels in the wetlands and swales would not rise as the outlet would drain down any increase, maintaining the original design water level.

While Kāpiti District is very flood prone, the effects of the proposed Expressway on flood levels are relatively minor and insensitive to variations in climate change assumptions. The main watercourse crossings have been checked to 1.5 x 1%AEP flows, which is KCDC's standard approach to overdesign in general.

5.2 Mitigation of Flood Effects

5.2.1 Use of Stormwater Models

The stormwater design for the proposed Expressway has used KCDC's and GWRC's stormwater models to determine the effects on surrounding catchments and to identify the peak flows that need to be passed beneath the proposed Expressway in either culverts or bridges.

These models reflect the most up to date understanding of the hydrology and hydraulic systems of the Kāpiti District.

The 1%, 10%, 1.5x1% AEP storms were used for testing the crossings, understanding effects and determining mitigation. Other events (including the 0.04% and 50% AEP storms) have been used for other design inputs such as bridge structural design.

5.2.2 Peak Flow Effects

The proposed Expressway will change the existing ground surface from previous grass/bush/scrub cover to impervious pavement. Although natural peat deposits are often already saturated and can be almost as impervious as the pavement will be, the stormwater will run off much faster from pavement than it does peat.

Both NZTA and KCDC design standards require attenuation of peak flows prior to discharge in order to mitigate downstream effects on flooding and watercourse erosion. KCDC use the term "hydraulic neutrality" meaning areas outside the proposed Expressway should not experience any additional effects on flooding risks i.e. from increase peak flow discharges or loss of floodplain storage.

The adoption of hydraulic neutrality for the design means that future catchment development scenarios, such as further infill development in the catchments, do not need to be tested as each development will have to achieve its own hydraulic neutrality.

The design provides attenuation of peak flows from the proposed Expressway through the use of swales and/or wetlands. These have been modelled to target restricting peak flow discharges to no more than 80% of pre–Expressway peak discharges for the 1%, 10% and 50% AEP storms.

The complex undulating sand dune topography and the linear nature of the proposed Expressway has required the modelling to focus on attenuation to areas that drain to an existing watercourse instead of at every local low point and gully in every sand dune along the way. This approach allows simplification without losing the effects on flooding associated with each watercourse which, in terms of assessing the overall effects, is the most important principle to identify and understand.

The swales and wetlands provide attenuation to varying degrees, ranging from 6% to 82% for the 1% AEP flows. This generally achieves the targeted 80%, in some cases achieving results significantly better than this. Where results in some few areas do not achieve the 80% target, the final design will be optimised to achieve this target during later design stages of the Project. The swales do not currently achieve the 80% target for the other storms but this is a reflection of how the outlet orifice has been modelled. Further refinement of the outlet design and minor modifications to the swale section will see the swales meet the target attenuation for all events.

Therefore, the proposed Expressway swales and wetlands will fully mitigate the effects of increased peak flows on flood levels.

5.2.3 Volume Effects

As a result of the change in surface permeability and new, formalised drainage systems associated with the proposed Expressway, the total volume of stormwater discharged into a watercourse will increase. Potential effects of increased volume are increases in flooding and erosion of receiving watercourses.

Attenuation to 80% of pre-Expressway peak flow discharges is recognised in national and international practise as the most practical method of mitigating resulting effects. In many cases the attenuation achieved for the proposed Expressway improves on this 80%, in some cases by a significant margin, more than offsetting the potential effects of increased volume of discharge. The only alternative method that could be considered for mitigating volume would be to discharge runoff to ground. This is not possible in the low-lying peat areas, and is not appropriate in the sand areas due to high water table in many cases.

5.2.4 Floodplain Storage Effects

As the proposed Expressway crosses floodplains, the embankment on which it will be constructed will fill in some of the volume currently available for flood storage. Without mitigation this would have the effect of raising flood levels in adjacent areas. In the majority of cases this effect has been fully mitigated to the extent that flood levels are generally marginally lower as a result of the proposed Expressway.

Partially filling in of floodplains has been mitigated by providing offset storage (i.e. lowering local ground levels) to hold more flood water than it currently does and or by over-attenuating flows in the wetlands and swales to such an extent that it has the same effect on flood levels as would be achieved by providing offset storage.

This has resulted in several offset storage areas with the largest located at:

- Drain 7 South;
- Wharemauku Stream;
- Kakariki Stream; and
- Paetawa Drain/Peka Peka interchange.

The effect of the mitigation is an overall improvement of flood levels during the 1% and the 10% AEP storms when compared to existing flood levels. However, one area remains at chainage 12,100m where the mitigation is yet to be resolved as it relies on the outcome on-going property negotiations. In this area the Designation has been set such that there is room to provide mitigation if required.

5.3 Watercourse Crossings

5.3.1 Overview

The Project contains a mix of bridges for the larger watercourses, box culverts for the medium sized watercourses and pipe culverts for the smallest. Selection of crossing type depended on:

- the size of the water course;
- the sensitivity of the flood levels to bridge/culvert size;
- floodplain issues;
- existing topography;
- existing drainage (i.e. is there a culvert already there);
- ecological significance; and
- economics of the structural types.

It is noted that of the above factors, cost was not always the decider, nor necessarily was the decision based on drainage considerations, for example:

- Te Moana Interchange ramps – uses bridges to cross Waimeha Stream primarily due to existing ecological value (or potential value). Culverts could be less costly and would have provided a simpler engineering solution[43].
- Kakariki Stream Expressway – in this location initially a large box culvert was planned. However, the topography better suited a bridge (spanning across a gap in a line of high sand dunes) and also it was more appropriate given the significant ecological value (or potential value) involved with this particular stream corridor. Again this is a more costly solution.
- Paetawa Drain – initially tested as a box culvert. It would have needed to be so large that it was more structurally suited to being a bridge, despite the additional cost this incurs.

Bridges and culverts have been tested in KCDC/GWRC's models and all achieve NZTA's standard 1% AEP capacity. Overdesign events, such as larger than 1%AEP storms or culvert blockage have been investigated by applying KCDC's standard 1.5x1%AEP storm flows (with the exception of the Waikanae River where a variety of specific overdesign scenarios have been considered). The effects of these events have been investigated for understanding purposes and not for mitigation as they are events beyond the required level of design. These effects range from small decreases in the level and extent of flooding during overdesign events up to small increases depending on the sites considered. In general, the increases are less than 100mm and do not cause significant changes in the existing pattern of flooding.

5.3.2 Flooding Effects at Bridges

Several large bridges are needed along the route to span watercourses and their floodplains. None of these bridges have piers within the main channel of any watercourse.

Wharemauku

The Wharemauku Stream and floodway does not set the bridge size as the clearance required to accommodate a future local road and this dominates this design. As such, this structure does not adversely affect flood risk on the Wharemauku Stream.

Waikanae

The Waikanae River bridge needs to be 180m long to minimise its effect on flooding. The combined effects of piers in the floodway plus enhancement works to the river channel will result in flood levels decreasing. The location of the bridge has been set by design factors other than river engineering (although the location and form are nevertheless appropriate from the point of view of river hydraulics and management). As a result, the last 30m of the Muaupoko Stream needs to be diverted.

Given the widening of the river channel at the bridge, significant works will be needed to the river channel in order to transition it back to existing banks downstream of the bridge. The effects of this are mainly ecological and aesthetic rather than flood related and so mitigation of these aspects is not detailed here, but is covered in the respective reports on ecology and landscape.

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[43] In this instance the bridges and their associated freeboard clearance creates a roading issue that requires the ramp/Te Moana Road intersections to be raised up above existing levels so that the bridge can achieve freeboard. A culvert has different freeboard requirements that would have significantly reduced this effect.

The Waikanae River waterway design has been peer reviewed by GWRC's consultant river designer who has noted that the crossing position, length, freeboard and span arrangement are acceptable. The reviewer's comments on rock rip rap and the downstream transition arrangement have been adopted into the design. The future maintenance responsibility for those areas of the river not directly related to the bridge structure and protecting it will remain with GWRC, which will then manage them as part of their overall responsibility for the river.

Waimeha / Te Moana

The Te Moana bridge is sized both to clear the road and stream, and also to provide for an over-design event of the Waikanae River stopbank breaching. Without this structure the proposed Expressway would present a barrier to passage of the breach flows and could result in significant additional localised flooding. The proposed bridge and associated floodway mitigates this effect and still allows Te Moana Road to remain flood free at the proposed interchange.

The spring-fed Waimeha Stream will be crossed by three bridges, one for the proposed Expressway and one for each for the north-facing on/off ramps. Bridges were chosen for the ramps due to the potential difficulty in realigning the Stream into a culvert and the effects on flood levels and effect on the ecological values of the stream that culverts would cause.

Kakariki

The Kakariki Stream is an important ecological corridor and, if a culvert were to be used instead of a bridge, it would significantly affect these and also it would need to be so large as to be a bridge in all but name.

Paetawa

The Paetawa Drain is to be bridged primarily due to the very large peak flow that needs to pass under the proposed Expressway. A culvert would again need to be so large that it would, in effect, be a bridge.

Scour risk

Under all bridges the existing vegetation can no longer be relied on to protect against river bank and berm scour. This is because the bridge deck will block direct sunshine and rainfall limiting the viability of riparian planting. In order to mitigate this, rock rip rap will be placed to areas in the footprint of the deck (both floodplain and waterway). There is the possibility to inter-plant shade tolerant species into the riprap to soften its appearance.

Hydraulic effects summary

The effects of the bridges on the 1% AEP design flood levels detailed in Section 3 and is summarised below in table 14.

Table 14: 1% AEP flood levels upstream and downstream of bridges (including mitigation measures).

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Wharemauku Stream Bridge (downstream)	4.75	4.75	0.00	No change
Wharemauku Stream Bridge (upstream)	5.17	5.07	0.10	Lower
Waikanae River Bridge (downstream)	4.95	4.86	0.09	Lower
Waikanae River Bridge (upstream)	5.26	5.18	0.08	Lower
Waimeha Stream Ramp Bridges (downstream)	3.15	3.06	0.09	Lower
Waimeha Stream Ramp Bridges (upstream)	3.19	3.16	0.03	Lower
Kakariki Stream Bridge (downstream)	6.23	6.18	0.05	Lower
Kakariki Stream Bridge (upstream)	6.41	6.36	0.05	Lower
Kakariki Stream Local Road Bridge (downstream)	6.41	6.36	0.05	Lower
Kakariki Stream Local Road Bridge (upstream)	7.69	7.64	0.05	Lower
Paetawa Drain Bridge (downstream)	9.13	9.13	0.00	No change
Paetawa Drain Bridge (upstream)	9.45	9.41	0.04	Lower

In summary, the hydraulic effects of the bridges are all adequately addressed so that adverse effects are avoided.

For details of the ecological effects and mitigation relating to the bridges refer to Technical Report 26, Volume 3 (Ecological Impact Assessment).

5.3.3 Flooding Effects at Culverts

The culverts have been sized and tested in KCDC's models. As for any structure in a watercourse, it is inevitable that there will be some effect on water levels, however small. However, the culverts have been sized to be large enough as to have minimum effect on flooding. Often the effect is hidden in an overall reduction in flood levels provided by the attenuation and offset storage.

Culverts have generally been aligned with existing watercourses. However, in some cases this results in longer culverts that enclose more of the watercourse and as such could be considered to have greater effects on that watercourse. Alternatively, reduced skew (shorter) alignments will be investigated during later design stages and may result in lesser adverse effects on the watercourse.

All culverts have fish friendly design features. The larger box culverts have "natural stream bed" culvert features and the smaller pipe culverts have "low slope" culvert features. Due to the naturally flat topography, all culverts have very little fall across them resulting in relatively low flow velocity and the ability to maintain water in them in times of low flow.

The inlets and outlets will be protected from scour and erosion through rock rip rap armour and planting or using similar means of protection.

The effects of the culverts on the 1% AEP design flood levels is detailed in Section 3 and is summarised below in Table 15.

Table 15: 1% AEP flood levels upstream and downstream of main watercourse culverts (including mitigation measures). Culverts on smaller watercourses are not included.

Location	Pre-Expressway (m RL)	Post Expressway (m RL)	Difference (m)	Effect
Drain 7 South Culvert 10 (upstream)	6.74	6.67	0.07	Lower
Drain 7 South Culvert 10 (downstream)	6.73	6.66	0.07	Lower
Drain 7 North Culvert 11 (upstream)	4.82	4.82	0.00	No change
Drain 7 North Culvert 11 (downstream)	4.75	4.75	0.00	No change
Mazengarb Drain Culvert 14 (upstream)	6.99	6.99	0.00	No Change
Mazengarb Drain Culvert 14 (downstream)	6.93	6.93	0.00	No Change
WWTP Drain Culvert 15 (upstream)	6.67	6.67	0.00	No Change
WWTP Drain Culvert 15 (downstream)	6.58	6.58	0.00	No Change
Landfill Drain Culvert 17 (upstream in the offset storage area)	8.25	8.37	0.120	Increase
Landfill Drain Culvert 17 (downstream)	8.11	8.11	0.00	No Change
Ngarara Creek Culvert 26 (upstream)	6.10	6.14	0.04	Increase
Ngarara Creek Culvert 26 (downstream)	6.12	6.12	0.00	No Change

For details of the ecological effects and mitigation relating to the “culverting in” of watercourses refer to Technical Report 26, Volume 3 (Ecological Impact Assessment).

In summary, for all but two culverts, there is either no increases or a reduction in flood level. In the case of landfill Drain, the increase is confined to within the Designation. In the case of Ngarara Creek the increase is locally upstream of the culvert, and does not adversely affect other property. In all cases provision will be made for fish passage.

5.4 Water Quality Effects

In overview, the proposed Expressway will provide overall water quality benefits. That is because the current SH1 does not have any stormwater treatment, whereas all of the stormwater from proposed Expressway will be treated prior to discharge. While there will now be two roads, the traffic volumes and the congestion on existing SH1 will be reduced, reducing the contaminant loading from that source.

The treatment provided by the swales and wetlands is to BPO standard where the term BPO refers to the sizing and performance of devices selected from a range of options that could be regarded as potentially appropriate. While the selection of devices was not carried out by explicit comparison, selection was governed by topography, drainage form, existing KCDC guidelines and available land or minimising additional land take requirements.

The analysis and results from the contaminant load modelling are presented in Technical Report 25, Volume 3 (Contaminant Load Assessment).

The need to mitigate the effects on water quality has led the design to make extensive use of swales (both wetland–planted and grassed) and treatment wetlands. These have been designed in accordance with NZTA's Stormwater Standard, which reflects current international best practice. The proposed Expressway will also use KCDC standard siphon sumps that trap gross litter more efficiently than do standard barrel sumps.

In summary, the proposed Expressway runoff with be managed to address the risk of discharge of contaminants, in accordance with currently industry standard practice.

5.5 Conclusion

In conclusion, the effects of the proposed Expressway on stormwater and surface hydrology will be mitigated by the following:

i. Increased peak flow discharge – mitigated by attenuation in swales and wetlands to no more than 80% of pre-Expressway peak flows, in some instances bettering this target by large margins.

ii. Filling of existing floodplain storage – mitigated by the creation of offset storage areas and attenuation of peak flows in wetlands and swales.

iii. Increased flood levels – mitigated by the attenuation in the swales and wetlands provision of, offset storage areas and design of low head culverts.

iv. Increased scour and erosion of watercourses - mitigated by providing attenuation of flows in swales and wetlands and rip rap protected culverts and outlets and at bridges.

v. Adverse water quality effects – mitigated by the use of swales and wetlands to treat stormwater prior to discharge. New open channel drains are also designed to resemble natural streams with riparian vegetation to provide shade and cover.

vi. Effects on fish passage – mitigated by the inclusion of fish friendly features in the design and designing new open channels drains to resemble natural streams with natural stream beds, riparian planting and refuges.

vii. The related effects on groundwater, freshwater ecology and water quality are detailed in the respective groundwater, ecology and contaminant loading assessment reports.

Overall, our conclusion is that the potential effects of the proposed Expressway on flood risk are able to be addressed in a satisfactory manner, and the use of best practice stormwater treatment will address potential water quality effects.

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Appendix 22. A

Drawings (included separately in Volume 5)

Refer to Drawings CV-SW-010 – 394, Technical Report

Appendices, Report 22, Volume 5

Doc N°	Document Title	Revision					
M2PP - AEE - DWG - CV - SW - 010	Main Watercourses Sheet 1 of 2	C					
M2PP - AEE - DWG - CV - SW - 011	Main Watercourses Sheet 2 of 2	C					
M2PP - AEE - DWG - CV - SW - 022	1% AEP Flooding Extents Sheet 1 of 10	C					
M2PP - AEE - DWG - CV - SW - 023	1% AEP Flooding Extents Sheet 2 of 10	C					
M2PP - AEE - DWG - CV - SW - 024	1% AEP Flooding Extents Sheet 3 of 10	C					
M2PP - AEE - DWG - CV - SW - 025	1% AEP Flooding Extents Sheet 4 of 10	C					
M2PP - AEE - DWG - CV - SW - 026	1% AEP Flooding Extents Sheet 5 of 10	C					
M2PP - AEE - DWG - CV - SW - 027	1% AEP Flooding Extents Sheet 6 of 10	C					
M2PP - AEE - DWG - CV - SW - 028	1% AEP Flooding Extents Sheet 7 of 10	C					
M2PP - AEE - DWG - CV - SW - 029	1% AEP Flooding Extents Sheet 8 of 10	C					
M2PP - AEE - DWG - CV - SW - 030	1% AEP Flooding Extents Sheet 9 of 10	C					
M2PP - AEE - DWG - CV - SW - 031	1% AEP Flooding Extents Sheet 10 of 10	C					
M2PP - AEE - DWG - CV - SW - 050	50% AEP 24 Hours Event Rainfall Isohyet	C					
M2PP - AEE - DWG - CV - SW - 051	10% AEP 24 Hours Event Rainfall Isohyet	C					
M2PP - AEE - DWG - CV - SW - 052	1% AEP 24 Hours Event Rainfall Isohyet	C					
M2PP - AEE - DWG - CV - SW - 100	Drainage Layout Sheet Index	C					
M2PP - AEE - DWG - CV - SW - 104	Drainage Layout 4 of 37	C					
M2PP - AEE - DWG - CV - SW - 105	Drainage Layout 5 of 37	C					
M2PP - AEE - DWG - CV - SW - 106	Drainage Layout 6 of 37	C					
M2PP - AEE - DWG - CV - SW - 107	Drainage Layout 7 of 37	C					
M2PP - AEE - DWG - CV - SW - 108	Drainage Layout 8 of 37	C					
M2PP - AEE - DWG - CV - SW - 109	Drainage Layout 9 of 37	C					
M2PP - AEE - DWG - CV - SW - 110	Drainage Layout 10 of 37	C					
M2PP - AEE - DWG - CV - SW - 113	Drainage Layout 13 of 37	C					
M2PP - AEE - DWG - CV - SW - 114	Drainage Layout 14 of 37	C					
M2PP - AEE - DWG - CV - SW - 115	Drainage Layout 15 of 37	C					
M2PP - AEE - DWG - CV - SW - 116	Drainage Layout 16 of 37	C					
M2PP - AEE - DWG - CV - SW - 117	Drainage Layout 17 of 37	C					
M2PP - AEE - DWG - CV - SW - 118	Drainage Layout 18 of 37	C					
M2PP - AEE - DWG - CV - SW - 119	Drainage Layout 19 of 37	C					
M2PP - AEE - DWG - CV - SW - 120	Drainage Layout 20 of 37	C					
M2PP - AEE - DWG - CV - SW - 122	Drainage Layout 22 of 37	C					
M2PP - AEE - DWG - CV - SW - 123	Drainage Layout 23 of 37	C					
M2PP - AEE - DWG - CV - SW - 124	Drainage Layout 24 of 37	C					
M2PP - AEE - DWG - CV - SW - 125	Drainage Layout 25 of 37	C					
M2PP - AEE - DWG - CV - SW - 126	Drainage Layout 26 of 37	C					
M2PP - AEE - DWG - CV - SW - 127	Drainage Layout 27 of 37	C					
M2PP - AEE - DWG - CV - SW - 128	Drainage Layout 28 of 37	C					
M2PP - AEE - DWG - CV - SW - 129	Drainage Layout 29 of 37	C					
M2PP - AEE - DWG - CV - SW - 130	Drainage Layout 30 of 37	C					
M2PP - AEE - DWG - CV - SW - 131	Drainage Layout 31 of 37	C					
M2PP - AEE - DWG - CV - SW - 132	Drainage Layout 32 of 37	C					
M2PP - AEE - DWG - CV - SW - 201	Swales Typical Cross-Sections	B					
M2PP - AEE - DWG - CV - SW - 203	Swale Outlet Detail	B					
M2PP - AEE - DWG - CV - SW - 212	Wetland Typical Longsection	B					
M2PP - AEE - DWG - CV - SW - 231	Smithfield Stream Realignment	A					
M2PP - AEE - DWG - CV - SW - 232	Kakariki Stream Diversion	A					
M2PP - AEE - DWG - CV - SW - 303	Typical Culvert Arrangement	C					
M2PP - AEE - DWG - CV - SW - 304	Culvert Long Section	B					
M2PP - AEE - DWG - CV - SW - 391	Waikanae Bridge and River Alignment	D					
M2PP - AEE - DWG - CV - SW - 392	Waikanae River Bridge Riprap Plan	D					
M2PP - AEE - DWG - CV - SW - 393	Waikanae River Bridge - Bridge and River Section	D					
M2PP - AEE - DWG - CV - SW - 394	Waikanae River Bridge - Bridge and River Details	C					

Appendix 22.B

Culverts and Watercourse Diversion Schedule

Appendix B - M2PP Culvert Schedule (including Bridges)

Major watercourse crossings

Ref	Chainage	Name	Catchment Area (ha)	Dia / Dim	Length	Headwall Type	Headwall Apron Length (us+ds)	US Riprap Length (2.5xdia)	DS Riprap Length (4xdia)	Total Length of Watercourse Culverted / Ripraped	Note
								2.5	4		
6	2000		4.91	600	20	pc	2		2.4	24.4	remove 10m of existing pipe
7.1	2220		5.7	600	50	pc	2		2.4	na	
7.2	not used										
7.3	not used										
7.4	2675		na	1050	5	pc	4	2.6	4.2	15.8	
7.5	2605		23.33	1200	60	pc	4	3.0		67.0	
8	2600		12.48	1050	40	pc	4	2.6	4.2	50.8	Poplar Ave
9	not used										
9.1	not used										
9.2	not used										
9.3	not used										
9.4	not used										
10	3685	Drain 7 Upper	44.41	1500	60	ww only		3.75	6	69.75	Drain 7
11	4930	Drain 7 Lower	151.32	3x2	100	ww only		7.5	12	119.5	Drain 7
11.1		Wharemauku Stream	1007.76			bridge				32	Wharemauku Bridge. Ihakara Rd at 5.5m.
11.2	5400		0.7	600	30			1.5	2.4	na	
11.3	5200	Storage Area 2 / 3 connector	na	1800	70			4.5	7.2	na	flood balancing culvert
12.2	not used										
13	7465		na	750	30			1.9	3.0	na	
14	8040	Mazengarb Drain	378.83	5x3	111	ww only		12.5	20	143.5	Mazengarb Drain
15	8500	WWTP Drain	17.04	1500	60	pc	4	3.75	6	73.75	WWTP Drain
16	8725		3.73	1200	60	ww only		3.0	4.8	67.8	
17	8930	Landfill Drain	15.22	1200	75	ww only		3	4.8	82.8	Landfill Drain.
18	not used										
18.1	9270	Otaihanga Drain	10.38	1050	10	pc	4	2.6	4.2	20.8	
21	10290			1800	50	pc	4	4.5	7.2	na	
22	10290		4.4	750	50	pc	4	1.9	3.0		
22.1	10465		1.34	750	65	pc	4	1.9	3.0	na	
22.2	10290		4.4	1050	10	pc	2	2.6	4.2	18.8	
23		Waikanae River	13005.22			bridge				83	Waikanae River bridge
23.3	11110		1.06	1050	50		2	2.6		54.6	
23.4	11125		1.89	1050	60		2	2.6		64.6	
24	11800		14	1050	15	pc	4	2.6	4.2	na	cleanwater diversion
24.1	11820		31	1050	15	pc	2	2.6	4.2	na	outlet
24.2	11780		12	1050	15	pc	4	2.6	4.2	na	
24.3	11700		12	1050	15			2.6	4.2	21.8	
24.4	11650		2.2	300	10			0.8	1.2	na	
25		Waimeha Stream ramp	218.8			bridge				32	Waimeha Stream Ramp Bridge
25.1		Waimeha Stream / floodway				bridge				15	
25.2		Waimeha Stream ramp	218.8			bridge				15	
25.3		flow balancing culvert	9.46	600	70			1.5	2.4	na	
26	13200	Ngarara Creek	164.19	3x2	70	ww only		7.5	12	89.5	Ngarara Creek
27	13400	flow balancing culvert	2.21	600	65	pc	4	1.5	2.4	72.9	
28	not used										
29		Kakariki Stream 1	575.86			bridge				60	Kakariki Stream bridge
30	14060		6.33	1050	30	ww only	4	2.6	4.2	40.8	
30.1	14270		1.8	600	25	ww only	4	1.5	2.4	32.9	Smithfield Rd culverts 2No
30.2	14375		1.04	600	25	ww only	4	1.5	2.4	32.9	
30.3	14480		14.93	1050	25	ww only	4	2.6	4.2	35.8	
30.4	14340		5.5	1200	90	ww only	4	3.0	4.8	101.8	
30.5		Kakariki Stream 2	617.95			bridge				25	Kakariki Stream bridge No2
31	15100		1.66	1050	60	ww only		2.6	4.2	66.8	
32	Not used										
33	15650		5.15	1050	65	ww only	4	2.6	4.2	75.8	
34	15780		16.95	1500	50	ww only	4	3.8	6.0	63.8	
35	15910		39.78	1500	48	ww only	4	3.8	6.0	61.8	
35.1		flow balancing culvert	na	1800	50			4.5	7.2	na	
36		Paetawa Drain	271.22			bridge				30	Paetawa drain - bridge
37	Not used										
38	16805		83.81	3x2	65	ww only		7.5	12.0	84.5	
38.1	16710		82.52	3x2	30			7.5	12.0	49.5	Not used
38.2	16840		na	525	20	ww only		1.3	2.1	23.4	
38.3	17140		2.1	1050	30			2.6	4.2	36.8	Not used
38.4	17165		53.74	1800	25			4.5	7.2	36.7	Not used
39	17170		25.02	1500	25	ww only		3.8	6.0	34.8	
40	17465	Hadfield Drain /Te Kowhai Stream	104.06	3x2	20	ww only		7.5	12	39.5	Hadfield Drain - Remove existing culvert under existing SH1 2 x 900 dia.
40.1	17465	Hadfield Drain /Te Kowhai Stream	104.06	3x2	40	ww only		7.5	12	59.5	
40.2	17465	Hadfield Drain /Te Kowhai Stream	104.06	3x2	20	ww only		7.5	12	39.5	Hadfield Drain - Remove existing culvert under existing SH1 2 x 900 dia.
40.3	17630	Hadfield Drain /Te Kowhai Stream	104.06	3x2	20	ww only		7.5	12	39.5	Hadfield Drain - Allow undercut 2m deep x 20m x 20m

pc = standard precast
ww only = wingwall only
us = upstream
ds = downstream

Appendix B - Diversion of Existing Watercourses

Note:

1 Does not include the length of watercourse replaced with a culvert nor new watercourses where ones did not exist prior.
2 Length of watercourse diversion where culvert skew is reduced would be additional.
3 Does not include relatively minor diversion lengths associated with constructing culverts off line.

Drawing		Water Course Reference	Associated culvert	Approximate Length of diversion (m)	Reason
CV-SW-	104				
CV-SW-	105				
CV-SW-	106	Tributary of Drain 7	10	50	Existing drain filled in by embankment
CV-SW-	107				
CV-SW-	108	Tributary of Drain 7	11	20	Existing drain filled in by embankment
CV-SW-	109				
CV-SW-	110				
CV-SW-	111				
CV-SW-	112				
CV-SW-	113				
CV-SW-	114				
CV-SW-	115				
CV-SW-	116				
CV-SW-	117				
CV-SW-	118	Muaupoko Stream	-	30	
CV-SW-	119				
CV-SW-	120				
CV-SW-	121				
CV-SW-	122				
CV-SW-	123				
CV-SW-	124				
CV-SW-	125	Kakariki Stream	bridge	110	
CV-SW-	126	Smithfield Drain	-	600	
CV-SW-	127				
CV-SW-	128				
CV-SW-	128	Paetawa Drain	bridge	70	Existing drain filled in by embankment
CV-SW-	129	Tributary of Paetawa Drain	38.3	220	
CV-SW-	129	Tributary of Paetawa Drain	38, 38.4, 39	200	Drain filled in by emabankment and needed to facilitate offset area 13A
CV-SW-	130	Hadfeild / Te Kowhai Stream	40, 40.1, 40.2, 40.3	170	Culverts diverted. 470m if diverted further south as part of off set storage
CV-SW-	130	Tributary of Paetawa Drain	38, 38.4, 39	280	Drain filled in by emabankment and needed to facilitate offset area 13A
CV-SW-	132	Tributary of Paetawa Drain	38.1	90	Drain filled in by local road changes

Appendix 22.C

Low Flow Memorandum

Memorandum

From:	Mike Law	**Date:**	8 September 2011
Doc Ref:	M2PP-AEE-FLN-CV-SW-058	**Our Ref:**	3320901
Subject:	M2PP Low Flow Hydrology		

1 Low and normal flow statistics

This memo briefly describes the derivation of low flow statistics for the layer watercourse crossings along the M2PP route.

The low flow statistics have been based on recorded data, flow gaugings and estimates of mean flow from NIWA's WRENZ[1] website.

Low flow statistics were requested for the ten largest crossings, though these included some watercourses that are too small to be included on WRENZ or where no data has been collected. In these cases, flow statistic estimates have been derived based on neighbouring gauged catchments. Catchment areas for all of the watercourses have been taken from a M2PP worksheet[2]. Where necessary, WRENZ mean flow estimates have been adjusted to account for differences between M2PP and WRENZ catchment areas.

The following statistics have been calculated for the ten crossings and are presented in Table 1, though some statistics have only been calculated for the larger crossings due the requirement for time-series data.

- Mean Q Mean Flow
- Q50 Median Flow
- Q95 Flow exceeded 95% of the time
- MALF Mean annual low flow (*larger crossings only)*
- 5yLQ(1d) 5 year low flow of 1 day duration (*larger crossings only)*
- 5yLQ(7d) 5 year low flow of 7 days duration (*larger crossings only)*

There are about 40 watercourse crossings along the M2PP route, the majority of which are too small to be gauged and for which no data is available. Based on the statistics generated for the ten larger crossings, flow statistics for the smaller crossings can be estimated[3] from the catchment area using the following approximations:

- MeanQ = 20*Area
- Q95 = 0.15*MeanQ

Where MeanQ and Q95 are L/s and Area is km^2

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[1] http://wrenz.niwa.co.nz/webmodel/

[2] \\BECA.NET\PROJECTS\332\3320901\Design\Drainage and Hydraulics\Culverts and ponds schedule 29.07.11.xls

[3] A review of the limited number of gaugings from low-lying catchments indicates that the approximate relationship of mean flow to area is not significantly different from those catchments with a hill component.

Memorandum

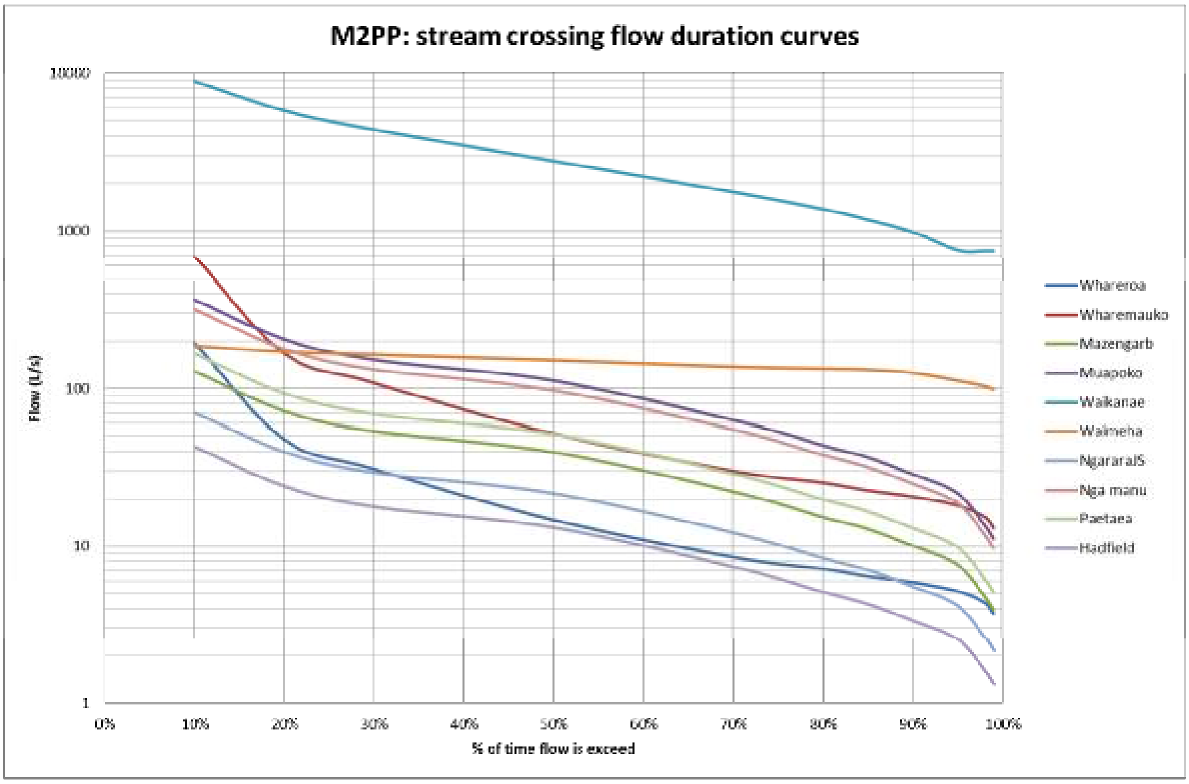
Table 1 – M2PP low flow statistics

M2PP Crossing Ref	*4*	*11.1*	*14*	*-*	*23*	*25*	*26*	*30.5*	*36*	*40*
Area (ha)	222	1007	378	750	13,010	219	164	6.18	271	90
Area (km2)	2.22	10.07	3.78	7.5	130.1	2.19	1.64	6.18	2.71	0.9
Flow statistic	**Whareroa**	**Wharemauku**	**Mazengarb**	**Muaupoko**[4]	**Waikanae**	**Waimeha**	**Ngarara Creek**	**Kakariki**	**Paetawa**	**Hadfield / Te Kowhai**
Mean flow	*45*	*158*	*51*	*145*	*4766*	*169*	*28*	*126*	*66*	*17*
Median flow (Q50)	15	52	40	112	2790	151	22	98	51	13
Q95	5	18	8	22	760	112	4	19	10	3
MALF		15			960					
5yLQ(1d)		-			807					
5yLQ(7d)		8			734					
% of time flow exceeded	**Flow Duration curves**									
10%	195	684	129	366	8890	187	71	318	167	43
20%	48	167	72	206	5800	171	40	179	94	24
30%	31	109	54	152	4390	164	29	132	69	18
50%	15	52	40	112	2790	151	22	98	51	13
70%	9	30	22	63	1770	138	12	55	29	7
80%	7	25	15	44	1380	134	8	38	20	5
85%	6	23	13	37	1180	132	7	32	17	4
90%	6	21	10	29	990	126	6	25	13	3
95%	5	18	8	22	760	112	4	19	10	3
98%	4	15	5	14	750	104	3	12	6	2
99%	4	13	4	11	750	99	2	10	5	1

[4] The main channel of the Muaupoko Stream is not crossed by the M2PP route. The stream discharges into the Waikanae at the M2PP crossing, and the outlet is likely to need to be diverted.

Memorandum

Figure 1 Flow duration curves



The approximation that mean flow is 20L/s per km^2 of catchment is the mid-point in the range observed in the ten larger catchments, excluding the Waikanae River and the Waimeha Stream. South of the Waikanae, the range was 15-20L/s per km^2, while north of the Waikanae it was 17-25L/s per km^2. The mean flow of the Waikanae is about 37L/s per km^2, reflecting the higher rainfall on the Tararua Ranges behind the Kapiti Coast. The mean flow of the Waimeha is 77L/s per km^2, as a result of the stream being fed by spring flow from the adjacent Waikanae.

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based on analysis of the 1975-2003 flow record, which includes the severe droughts of 1978 and 2003.

Table 2 Waikanae low flow statistics[6]

Return Period (years)	Average annual probability (%)	Flow (L/s)	
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The figures presented in Table 2 have been reviewed, as part of this study, to include the daily mean flows updated to 2010, and found to be still valid.

Daily flow data for the Wharemauku Stream and Mazengarb Drain for the last ten years are available from GWRC, with flow and stage plots presented on the GWRC website. However, flow statistics are not presented on the website.

The Wharemauku Stream is monitored at Coastlands where the stream crosses SH1 at the foot of the hills before the stream crosses the coastal plain. The catchment area at Coastlands is 7.3km^2 according to WRENZ. The M2PP route crosses Wharemauku Stream further downstream where the catchment area is 10.1km^2, with the additional catchment being generally low-lying land covered with residential development. The annual minimum flows for the Wharemauku for 2000 to 2010 were used to calculate the mean annual low flow (MALF) of 15 L/s and to estimate a 1 in 5-year low flow (1-day duration) of 8L/s.

The flow recorder on Mazengarb Drain is less than a kilometre downstream of the M2PP crossing. Flow and level data is available from 1995 to present. The recorded data is supported by 128 gaugings. From this data it was possible to derive a flow duration curve for the Scaife Drive site. The recorded mean flow at Scaife Drive is 153L/s. However, the discharge from Paraparaumu Wastewater Treatment Plant enters the drain between the M2PP crossing and the flow recorder.

The flow data shows a daily fluctuation in flows of 20-30L/s due to diurnal variations in the flow from the wastewater treatment works, which enters Mazengarb Drain between the M2PP crossing and Scaife Drive. The average flow from the treatment plant is estimated as about 100L/s according to a study into shallow groundwater interaction along the Kapiti Coast[7]. However, annual minimum flows of less than 40L/s at the recorder indicate that discharges from the treatment plant are much lower during dry periods.

The influence of discharges from the treatment works meant that it was unrealistic to use the gauged flow record to calculate the MALF and 5-year low flows for the M2PP crossing site.

---

[6] OPUS 2004

[7] ***Investigating the sustainable use of shallow groundwater on the Kapiti Coast***. February 2005. http://www.gw.govt.nz/assets/council-publications/Investigating%20the%20sustainable%20use%20of%20shallow%20groundwater%20on%20the%20Kapiti%20Coast.PDF

Memorandum

Similarly for the other water courses crossed by the M2PP route there was insufficient information to calculate the MALF and 5-year low flows. Should such statistics be required, then estimates could be made based on the relationship between Q95, MALF and 5-year low flow for the Wharemauku Stream. However, the hydrological characteristics of the smaller lowland streams may not be well represented by the Wharemauku, which is predominantly a hill catchment at the recorder site. It should also be noted that low flows are very small on some of the watercourses crossed by the M2PP and they may expected to be dry for part of the year.

Mike Law

Memorandum

From:	Mike Law	**Date:**	8 September 2011
Doc Ref:	M2PP-AEE-FLN-CV-SW-058	**Our Ref:**	3320901
Subject:	M2PP Low Flow Hydrology		

1 Low and normal flow statistics

This memo briefly describes the derivation of low flow statistics for the layer watercourse crossings along the M2PP route.

The low flow statistics have been based on recorded data, flow gaugings and estimates of mean flow from NIWA's WRENZ[1] website.

Low flow statistics were requested for the ten largest crossings, though these included some watercourses that are too small to be included on WRENZ or where no data has been collected. In these cases, flow statistic estimates have been derived based on neighbouring gauged catchments. Catchment areas for all of the watercourses have been taken from a M2PP worksheet[2]. Where necessary, WRENZ mean flow estimates have been adjusted to account for differences between M2PP and WRENZ catchment areas.

The following statistics have been calculated for the ten crossings and are presented in Table 1, though some statistics have only been calculated for the larger crossings due the requirement for time-series data.

- Mean Q Mean Flow
- Q50 Median Flow
- Q95 Flow exceeded 95% of the time
- MALF Mean annual low flow (*larger crossings only)*
- 5yLQ(1d) 5 year low flow of 1 day duration (*larger crossings only)*
- 5yLQ(7d) 5 year low flow of 7 days duration (*larger crossings only)*

There are about 40 watercourse crossings along the M2PP route, the majority of which are too small to be gauged and for which no data is available. Based on the statistics generated for the ten larger crossings, flow statistics for the smaller crossings can be estimated[3] from the catchment area using the following approximations:

- MeanQ = 20*Area
- Q95 = 0.15*MeanQ

Where MeanQ and Q95 are L/s and Area is km^2

[1] http://wrenz.niwa.co.nz/webmodel/

[2] \\BECA.NET\PROJECTS\332\3320901\Design\Drainage and Hydraulics\Culverts and ponds schedule 29.07.11.xls

[3] A review of the limited number of gaugings from low-lying catchments indicates that the approximate relationship of mean flow to area is not significantly different from those catchments with a hill component.

Memorandum

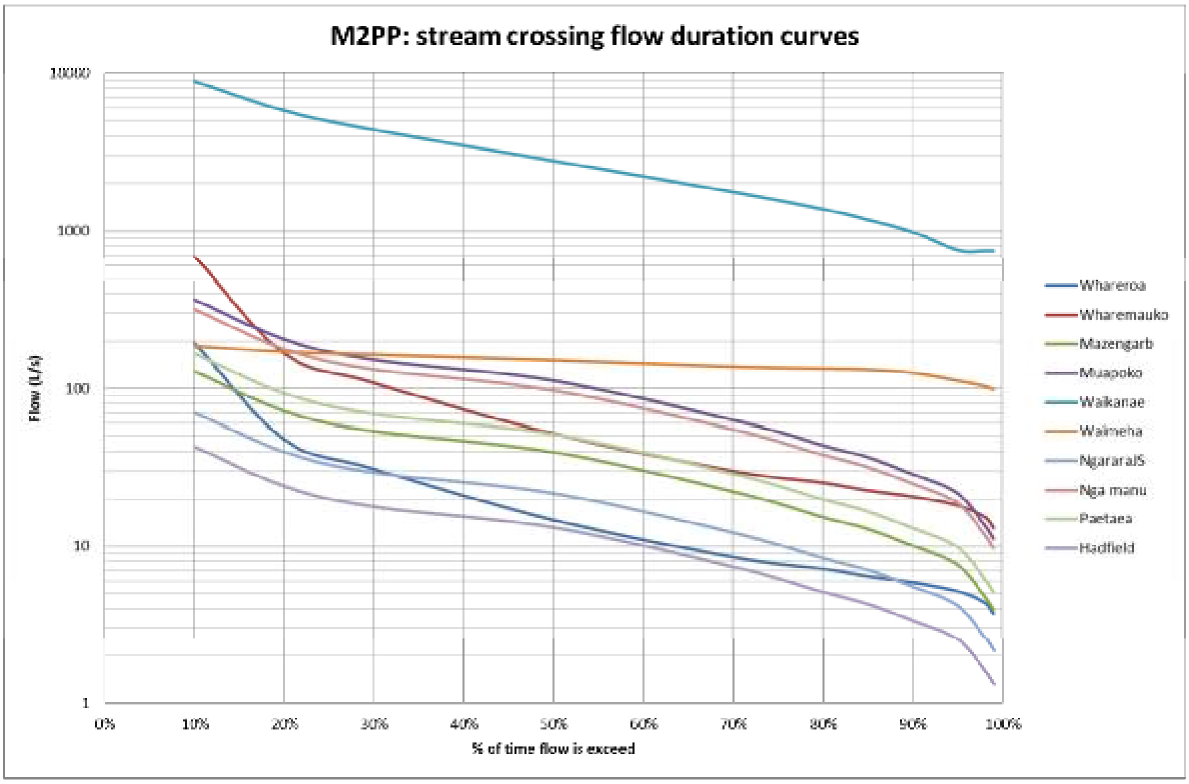
Table 1 – M2PP low flow statistics

M2PP Crossing Ref	***4***	***11.1***	***14***	***-***	***23***	***25***	***26***	***30.5***	***36***	***40***
Area (ha)	222	1007	378	750	13,010	219	164	6.18	271	90
Area (km2)	2.22	10.07	3.78	7.5	130.1	2.19	1.64	6.18	2.71	0.9
Flow statistic	**Whareroa**	**Wharemauku**	**Mazengarb**	**Muaupoko**[4]	**Waikanae**	**Waimeha**	**Ngarara Creek**	**Kakariki**	**Paetawa**	**Hadfield / Te Kowhai**
Mean flow	*45*	*158*	*51*	*145*	*4766*	*169*	*28*	*126*	*66*	*17*
Median flow (Q50)	15	52	40	112	2790	151	22	98	51	13
Q95	5	18	8	22	760	112	4	19	10	3
MALF		15			960					
5yLQ(1d)		-			807					
5yLQ(7d)		8			734					
% of time flow exceeded	**Flow Duration curves**									
10%	195	684	129	366	8890	187	71	318	167	43
20%	48	167	72	206	5800	171	40	179	94	24
30%	31	109	54	152	4390	164	29	132	69	18
50%	15	52	40	112	2790	151	22	98	51	13
70%	9	30	22	63	1770	138	12	55	29	7
80%	7	25	15	44	1380	134	8	38	20	5
85%	6	23	13	37	1180	132	7	32	17	4
90%	6	21	10	29	990	126	6	25	13	3
95%	5	18	8	22	760	112	4	19	10	3
98%	4	15	5	14	750	104	3	12	6	2
99%	4	13	4	11	750	99	2	10	5	1

[4] The main channel of the Muaupoko Stream is not crossed by the M2PP route. The stream discharges into the Waikanae at the M2PP crossing, and the outlet is likely to need to be diverted.

Memorandum

Figure 1 Flow duration curves



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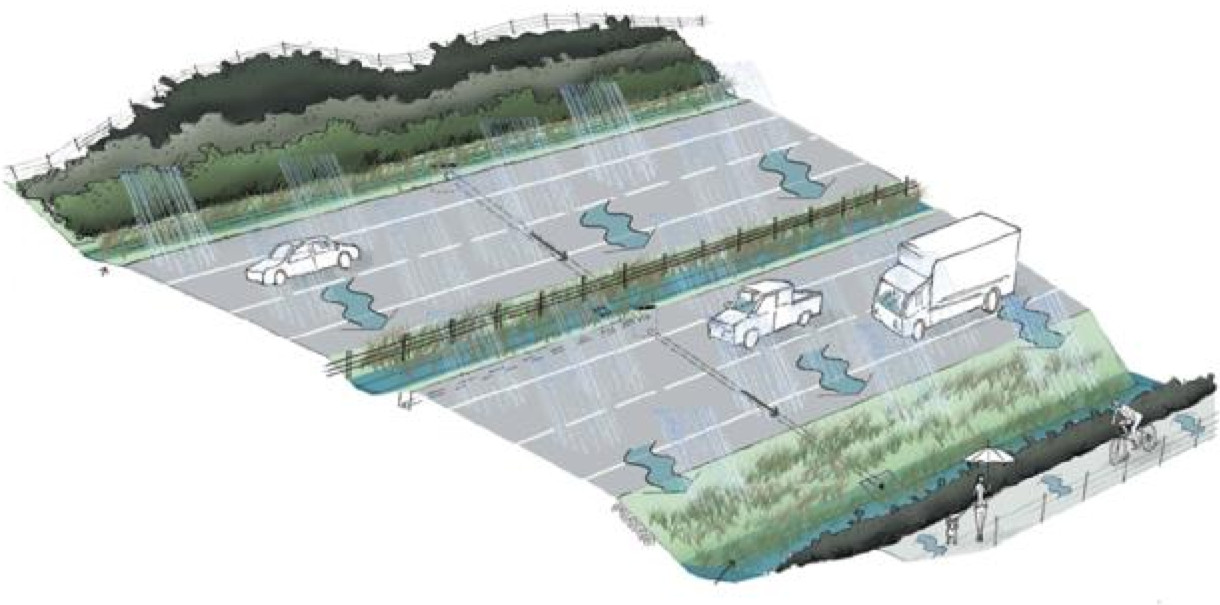
Mike Law

Appendix 22.D

Expressway Stormwater Attenuation Modelling Report



Report

M2PP - SAR - RPT - CV - SW - 179

Expressway Stormwater Attenuation Modelling

By Beca

14 November 2011

NZ TRANSPORT AGENCY WAKA KOTAHI

MacKays to Peka Peka Expressway

Revision History

Revision Nº	Prepared By	Description	Date
A	Elliot Tuck	Draft for review	15.09.2011
B	Elliot Tuck	For AEE	17.09.2011
C	Iain Smith	Updated for AEE	23.09.2011
D	Iain Smith	Updated for VE	14.11.2011

Document Acceptance

Action	Name	Signed	Date
Prepared by	Elliot Tuck		
Reviewed by	Mike Law		
Approved by			
on behalf of	Beca		

Table of Contents

1 Introduction

Modelling of the proposed stormwater system in the MacKays to Peka Peka Expressway has been undertaken using InfoWorks CS hydraulic modelling software package. The main purposes of the modelling at this stage of the project are to:

i. Determine the effectiveness of the proposed stormwater attenuation design for various storm events;
ii. Create data for input to wetland design; and,
iii. Generate output hydrographs for use in catchment flood modelling in KCDC/GWRC models.

Additional modelling will be undertaken at the detailed design stage to refine the swale and wetland designs with the aim of fine tuning the swale and wetland interactions in order to attenuate the 10% and 50%AEP events to meet the target 80% attenuation and also to optimise the wetland footprint whilst not compromising performance.

The Expressway is approximately 16 km long and stormwater will be attenuated and treated though a mixture of grassed swales, planted swales and wetlands that discharge into existing watercourses. Figure 1 shows the approximate location of the wetlands and outlets but for more detail refer to Drawings CV-SW-100 through 132.

This report should be read in conjunction with the M2PP Design Philosophy Statement, Section 7 Stormwater Management, which provides the background on the design philosophy.

2 Summary of the Stormwater Management Strategy

Rainfall from the road surface and adjacent catchments drains into swales, of which there are two general types:

i. Attenuation swales – typically a 1m deep 11m wide (2.5m at the base) swale that is planted with wetland/wet tolerant species in areas of peat and grassed in sand areas. These swales are and flat enough to provide the storage required to attenuate peak flows; and,
ii. Conveyance swales- Similar to the attenuation swales but shallower and smaller. Typically 0.5m deep and 6m wide. These are not needed for attenuation but to convey flows to either or attenuation swales.

The basic criteria to be met for both swales and wetlands are:

i. Where known flooding is an issue downstream, then attenuation will be to 80% of pre-development flows. This is considered to be part of what is what is meant by KCDC’s concept of “hydraulic neutrality” (along with floodplain off-set storage).
ii. Water quality treatment of the first flush volume from the road surface though water residence time and “through flow” in the swale or wetland.

This report considers the peak flow attenuation design effects. The overall water quality effects are discussed separately. Refer the Contaminant Load Modelling Report.

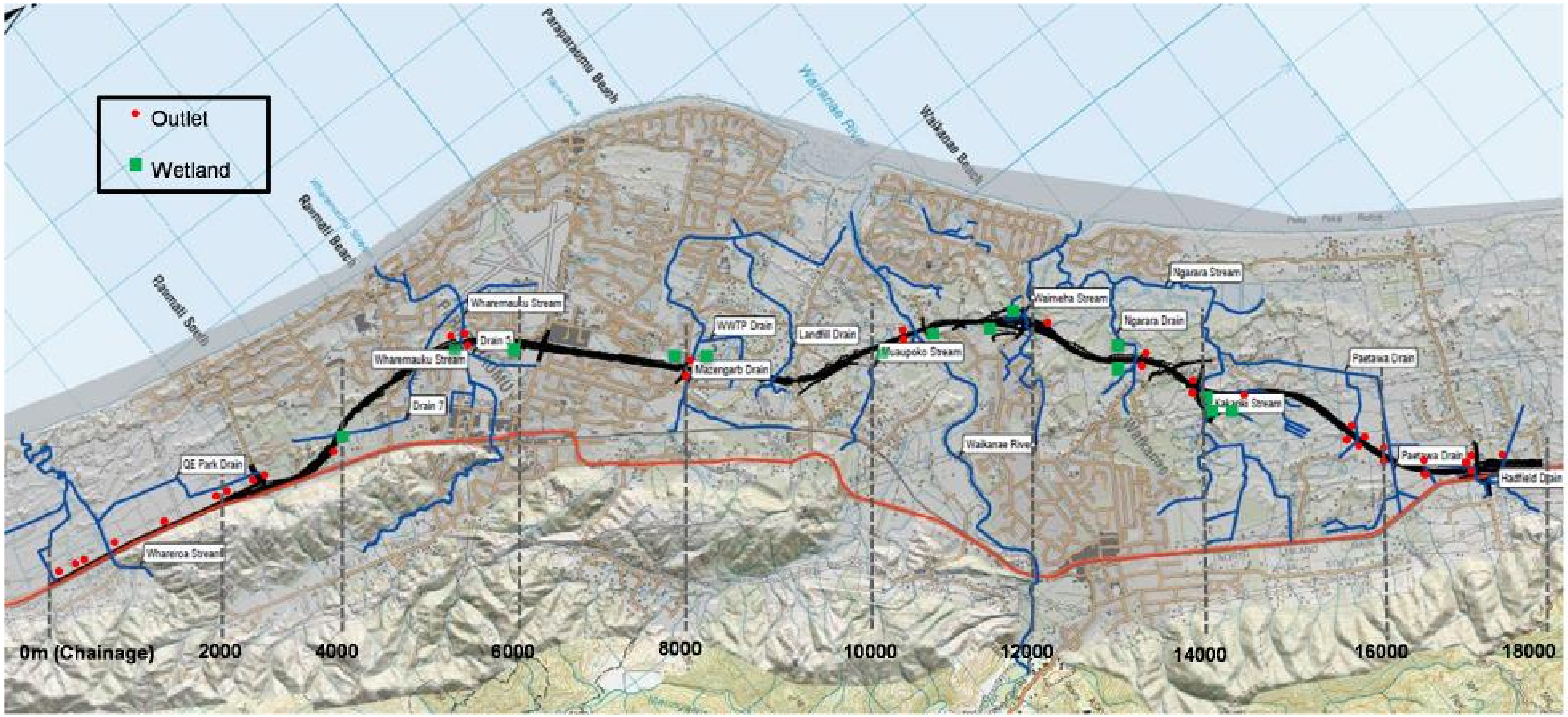


Figure 1: Approximate locations of wetlands and swale outlets (for further detail refer drawings CV-SW-100 through 132). Note the discharge points south of chainage 1,900m are no longer required.

3 Hydrological Modelling

The pre and post-expressway cases have been hydrologically and hydraulically modelled using Infoworks CS.

3.1 Parameters

The parameters used were generally as described in the Design Philosophy Statement section 7. Further detail is provided in the sections below.

3.2 Rainfall Hyetograph

This study has modelled a proposed stormwater management system that has been designed to meet the requirements of NZTA Stormwater Treatment Standard for State Highway Infrastructure and Kāpiti Coast District Council's (KCDC) standards.

A report prepared for KCDC, *Isohyet Based Calculation of Design Peakflows*[1] states that a balanced 24 hour storm should be used when calculating peak flows and runoff. KCDC supplied isohyet contour maps of the Kāpiti region. Comparison of these contours against data from HIRDS[2] showed HIRDS to be more conservative (giving higher rainfall depths) and for that reason the Infoworks modelling has been completed using HIRDS 24 hour rainfall depths. Rainfall depths for a 1% annual exceedance probability (AEP) or 100 year, 10% AEP (10 year) and 50% AEP (2 year) have been modelled.

Table 1 shows the rainfall depths used along the length of the expressway. Four different profiles were used and as we moved north up the coast the rainfall depth decreased.

It was accepted that the factors in Table 0-1 from KCDC's *Isohyet Based Calculation of Design Peak Flows* are correct and were used in the calculation of smaller duration events. The figures in Table 2 have been used to produce the example design storm in Figure 2.

Smaller duration storm intensities have then been grouped together to give a 24 hour nested or balanced storm.

---

[1] Kāpiti Coast District Council, Isohyet Based Calculation of Design Peak Flows, 2005.

[2] HIRDS: High Intensity Rainfall Design System Version 3, NIWA, (http://hirds.niwa.co.nz/)

Table 1: Rainfall depths along the Expressway.

M2PP Chainage (south to north)	Without Climate Change Depth (mm)			With Climate Change Depth (mm)		
	1%AEP	10%AEP	50%AEP	1%AEP	10%AEP	50%AEP
0-3000m	179.3	121.8	89.3	207.9	136.4	96.4
3000-6450m	157.9	107.5	79.0	183.2	120.4	85.3
6450-9250m	149.2	101.9	75.0	173.1	114.1	81.0
9250-18000m	146.8	100.0	73.6	170.3	112	79.5

Table 2: Normalised depth-duration relationship for 24 hour rainfall.

Duration	Normalised Rainfall Depth (I/I24)
5 Mins	0.08
10 Mins	0.11
15 Mins	0.14
30 Mins	0.19
1 Hour	0.26
2 Hour	0.35
3 Hour	0.46
6 Hour	0.60
12 Hour	0.81
24 Hour	1

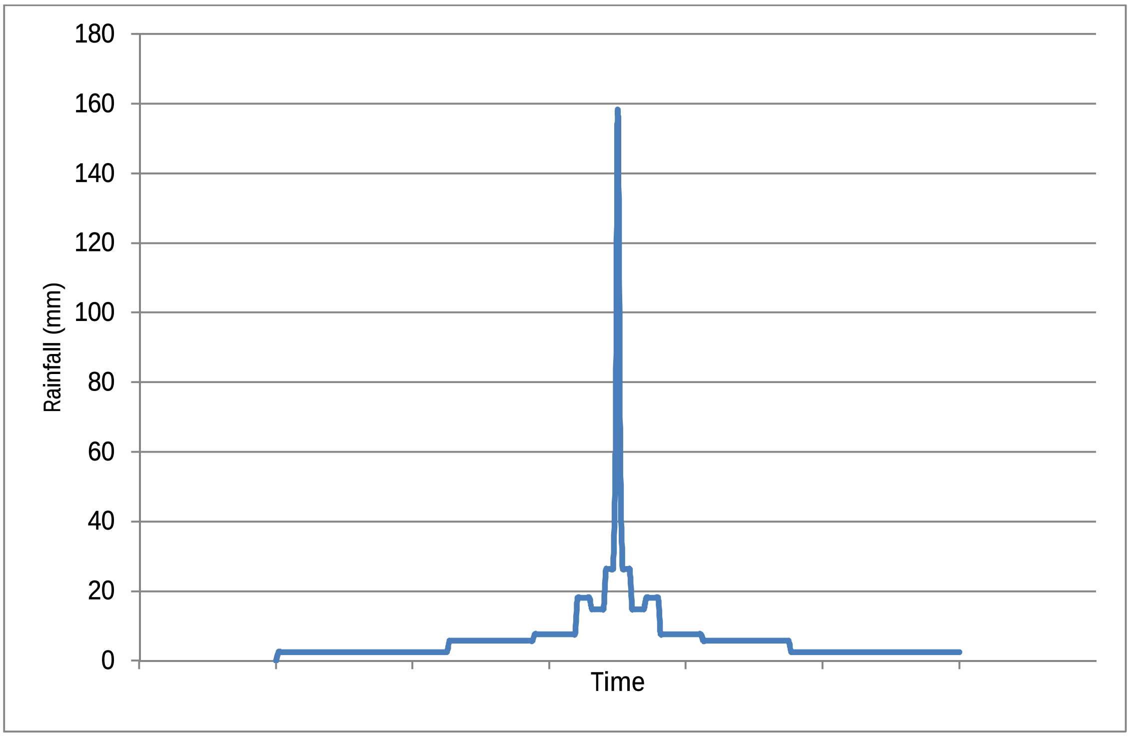


Figure 2: 24 hour nested storm hyetograph

Figure 2 shows a dip in intensity either side of the peak whereas nested storms usually show a smooth profile. The dip is caused by the 2 hour normalised rainfall depth being 0.35 of I_{24}, a 2 hour I/I_{24} value of 0.36 or 0.37 would remove the dips and provide a smoother shape to the nested storm.

However for the sake of consistency with other hydrological models being developed in the Kāpiti Coast District it was decided that this design storm shape be used.

3.3 Expressway Runoff

Runoff from the expressway has been calculated using the SCS method[3] with a time of concentration (Tc) set to 10 minutes and for the post-expressway model and 40 minutes for the pre-expressway model. In areas where existing roading is already in the pre-expressway model a Tc of 10 minutes has been used. The longer time in the pre-development model is to take into account the generally flat terrain the expressway passes though.

Curve numbers in Table 3 have been converted to equivalent volumetric runoff coefficients for use in Infoworks. The coefficients vary with rainfall intensity and therefore different coefficients have been used between the 1%AEP and the other events.

It has been assumed that the post-expressway runoff is generally discharged into the same watercourse as the pre-expressway.

[3] Soil Conservation Service, 1986, Technical Release 55 (TR55)

3.4 Curve Numbers and Land Use

Curve numbers and corresponding runoff coefficients in Table 3 are in accordance with the Design Philosophy Statement and are in agreement with KCDC figures. Estimates of percentage of each catchment's soil types were made using available soil mapping.

3.5 Downstream Boundary Conditions

Tail water levels in the receiving watercourses were compiled from various KCDC and Greater Wellington Regional Council (GWRC) flood levels. The flood levels were plotted on long sections of the expressway and water levels at each outlet noted. Where more than one source of flood level was available the highest (worst case) figure was used.

Table 3 – Curve number and volumetric runoff coefficients.

Soil Type		Curve Number (KCDC)	Equivalent Volumetric Runoff Coefficient	
			1%AEP	+10%AEP year
1.0	**Loose Dune Sands**	45	0.34	0.25
2.0	**Gravel/Silt/Loams**			
	Pasture	69	0.57	0.47
	Urban Gardens	61	0.49	0.38
	Bush	48	0.36	0.27
3.0	**Residential Inland Dune Sands**	61	0.49	0.38
4.0	**Steepland Hill Soils**			
	Pasture	79	0.69	0.59
	Urban	74	0.63	0.53
	Bush	65	0.53	0.42
5.0	**Peat**[1]	89	0.81	0.74

4 Swale Modelling

4.1 Attenuation Swales

The general attenuation swale design is for 1m deep swale with a 2.5m wide base and bank slope of 4H:1V making the top width generally 10.5m wide (in some cases local topography requires deeper swales).

Grassed swales in areas of sand have a Manning's roughness (n) of 0.05 and planted swales in areas of peat have a Manning's n of 0.07 due to the planting. It has been assumed no infiltration loss to ground occurs though the beds or banks of the swales.

In order to obtain more accurate time of travel results swale sub-catchments were broken into 100m long sections that drain into 100m long sections of swale.

4.2 Conveyance Swales

Conveyance swales have been designed as typically 0.5m deep with sides of 1:4 and a base 1.5m wide making an overall top width of 5.5m (again in some cases the topography makes the swale deeper and so wider than this). Conveyance swales have generally been used to convey water directly to a wetland for treatment or where other land constraints apply. The same Manning's n figures from the attenuation swales apply to the conveyance swales also.

4.3 Outlet design

The outlet control structure was modelled as a raised manhole. The height of the manhole has been set to meet the 80% discharge condition in combination with an orifice in the manhole at its base. The orifice controls the flow discharging from the swales and also the residence time. It controls the attenuation performance while providing enough residence time for the treatment of the runoff.

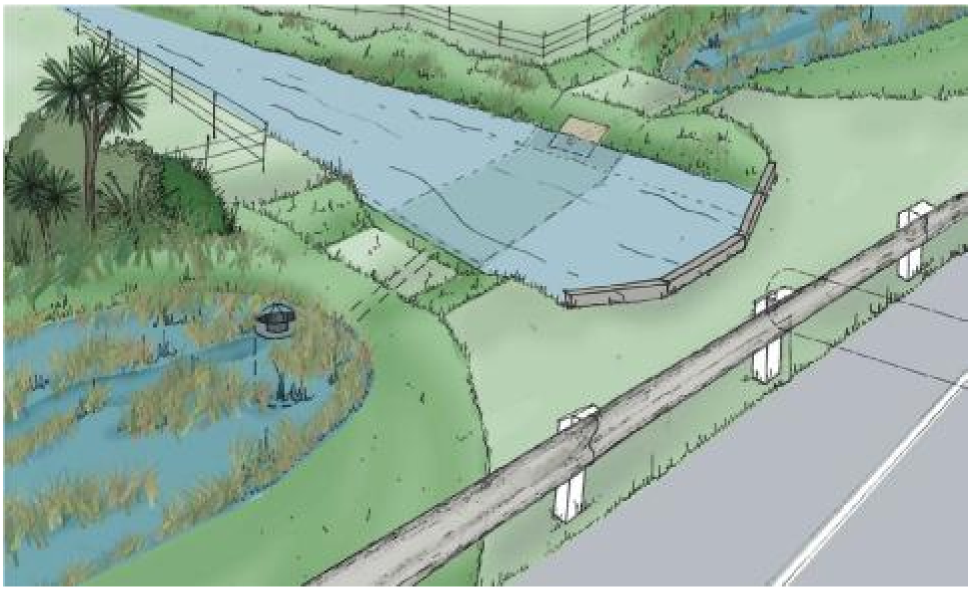


Figure 3: Mock-up of a typical swale outlet.

The manhole has been modelled as a sharp crested weir, with a weir length of the manhole circumference. Infoworks uses the following Kindsvater and Carter equation:

$Q_o = C_d \sqrt{g} B D_u^{3/2}$

Where: Q_o is the free outfall discharge

C_d is the discharge coefficient

g is the acceleration due to gravity

B is the width of the weir

D_u is the upstream depth with respect to the crest

To minimise the risk of blockage a minimum orifice size of 100mm was used. This size has meant that attenuation of the smaller storms cannot always meet the 80% requirement. Detailed design of the outlet structure may result in a design that remedies or improves this issue.

Infoworks uses a series of orifice equations depending on the upstream and downstream conditions as detailed in Table 4. The orifice size has been controlled to allow attenuation but also to limit drain-down times to less than 24hrs to prevent grass dying.

Table 4: Infoworks orifice equations

		Upstream Condition	
		Free Discharge	Surcharged
Downstream Condition	Independent	$Qs = C_d B D_u \sqrt{g D_u}$ Scenario A	$Qs = C_d A_o \sqrt{g D_d}$ Scenario B
	Boundary	$Qs = C_d B D_u \sqrt{g(D_u - D_d)}$ Scenario C	$Qs = C_d A_o \sqrt{g(D_u - D_d)}$ Scenario D

4.4 Check Dams

Some attenuation swales have a check dam to help achieve attenuation and treatment by increasing the residence time. The check dam has been modelled as a 100-200mm orifice at the base level of the swale and a V notch weir. The height of the weir has been set to achieve the desired attenuation while still providing adequate freeboard. Detailed design may result in these being formed from a bund instead of from a V notch.

4.5 Catchpits, Median Swales and Pipes

No catchpits or median swales have been modelled on the expressway as these will be included in the detailed design stage. Some pipes have been modelled where they would have a major influence on the hydrology (peak flow, water depth and time of travel) of the receiving swale. For example, where runoff is being moved from a swale on one side of the road to a wetland on the other side.

Where pipes have been included, it has been assumed that the pipes can accommodate the runoff and do not provide any additional storage or attenuation.

5 Wetland Modelling

Wetlands have been modelled as a large storage devices with no infiltration to ground. This is because they are constantly wetted.

The outlet structure consists of a manhole with two orifices placed between normal water level and the top of the manhole. These orifices have been sized to control the discharge to 80% of pre-expressway flow (or less) and to not overtop the sides of the wetland. An emergency overflow weir has been set at the peak 1% AEP flood level and it is assumed that it drains directly into the receiving watercourse.

The wetlands have a 300mm diameter (minimum) piped outlet from the manhole to the receiving watercourse,

In some instances the attenuation has been oversized to assist with other flood mitigation, such as for off-set storage.

In some situations it has been possible to direct runoff from existing catchments, external to the expressway, to a receiving watercourse bypassing the wetland and its expressway stormwater with associated contaminants. These flows have not been attenuated and do not need to be treated.

6 Attenuation Modelling Results

Model runs were completed for 1%, 10% and 50%AEP storms. The main focus of the modelling was on the 1% EP event. If storage could be found or created to attenuate the 1%AEP peak flows then it would be possible to attenuate the smaller storms, through fine tuning of the outlet design during later stages of the design. Given this, the results for some swales show that post development flows in the smaller storms are not yet sufficiently attenuated. This will be resolved during later detailed design.

An example of an outlet hydrograph output from Infoworks is shown in Figure 4. The hydrograph is of discharge at Outlet 16400E in a 1%AEP storm. It shows that the peak flow of the expressway is approximately 80% of the pre-expressway flow. The bulge on the right is the orifice slowly draining down the volume of water that has been held back in the swale during the storm.

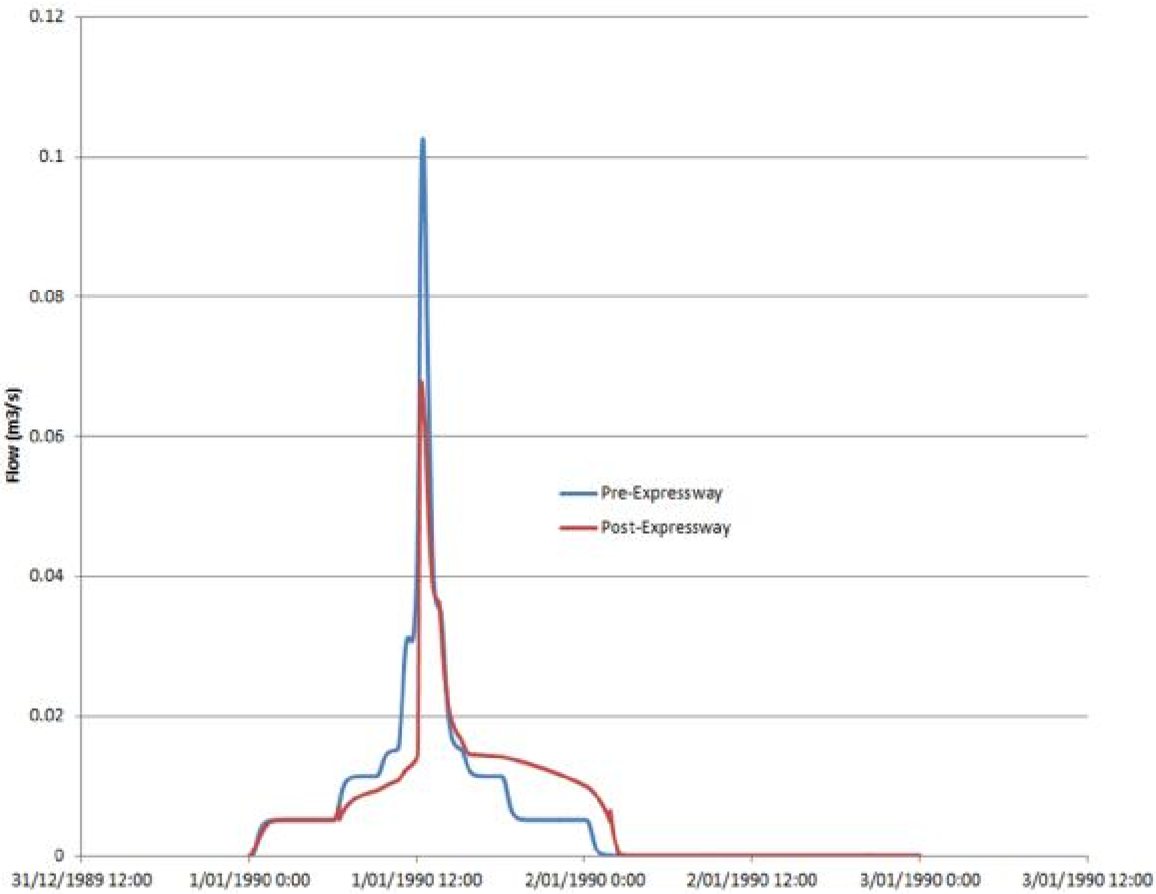


Figure 4: Discharge hydrograph at outlet 16400E.

Tables 5, 6 and 7 show the modelling results for the swales. E (east) and W (west) relates to the side of the Expressway that the swale discharges to.

Table 8 shows the modelling results of the wetlands. The wetlands are numbered from the southern most end of the expressway (0A) to the northern most end (12).

Table 5: 1%AEP storm swale attenuation results

Outlet Chainage	Max depth in swale (m)	Max swale Velocity (m/s)	Total Volume (m3)	Post Exp Peak Discharge (m3/s)	Pre Exp Peak Discharge (m3/s)	% of Pre Exp Peak Discharge (m3/s)
Outlet 1940 W	0.57	0.13	5549	0.266	0.590	45%
Outlet 2200 W	0.57	0.06	2911	0.167	0.214	78%
Outlet 2600 W	0.96	0.70	8452	0.202	0.312	65%
Outlet 2600 Pop Ave Nth	0.95	0.54	3708	0.212	0.270	79%
Outlet 2600 Pop Ave Sth	0.97	0.36	5214	0.126	0.236	53%
Outlet 3700 E	1.03	0.08	6608	0.192	0.311	62%
Outlet 4950 W	0.85	0.23	1080	0.063	0.101	62%
Outlet 4950 E	0.65	1.39	1281	0.055	0.084	65%
Outlet 5400 E	0.45	0.14	945	0.033	0.071	46%
Outlet 5500 E	0.92	0.04	2452	0.112	0.181	62%
Outlet 8020 E	0.83	0.01	287	0.013	0.020	65%
Outlet 8020 W	0.87	0.02	321	0.014	0.023	60%
Outlet 10400 E	0.81	0.07	1875	0.056	0.102	55%
Outlet 11900[4]	1.00	0.48	6658	0.404	0.412	98%
Outlet 13200 W	1.00	0.05	3231	0.173	0.217	80%
Outlet 14000 W	1.08	0.07	2899	0.167	0.212	78%
Outlet 14000 E	0.70	0.05	3178	0.12	0.236	49%
Outlet 14630 E	0.86	0.03	2563	0.07	0.115	57%
Outlet 15650 W (1)	0.52	0.12	2247	0.056	0.189	30%
Outlet 15650 W (2)	0.51	0.03				
Outlet 15650 E	0.63	0.05	3234	0.17	0.209	79%
Outlet 15750 W	0.50	0.02	859	0.016	0.058	28%
Outlet 15750 E	0.39	0.04	607	0.01	0.049	26%
Outlet 15900 W	0.89	0.03	2403	0.14	0.196	73%
Outlet 15900 E	0.69	0.05	1705	0.06	0.152	42%
Outlet 16400 W	0.85	0.04	2631	0.15	0.204	75%
Outlet 16400 E	0.54	0.05	1170	0.07	0.103	66%
Outlet 16800 E	0.96	0.54	10985	0.109	1.870	6%
Outlet 17200 W	0.30	0.22	7846	0.08	1.010	8%
Outlet 17250 W	0.55	0.20	764	0.07	0.134	54%
Outlet 17600 W	0.62	0.24	5460	0.44	0.559	79%

[4] 80% could be achieved with deeper or wider swales with check dams or during detailed design stages.

Table 6: 10%AEP storm swale attenuation results

Outlet Chainage	Max depth in swale	Max Velocity (m/s)	Total Volume (m3)	Post Exp Peak Discharge (m3/s)	Pre Exp Peak Discharge (m3/s)	% of Pre-Exp Peak Discharge (m3/s)
Outlet 1940 W	0.44	0.13	3405	0.210	0.523	40%
Outlet 2200 W	0.47	0.05	1787	0.041	0.138	30%
Outlet 2600 W	0.73	0.73	4978	0.170	0.290	59%
Outlet 2600 Pop Ave Nth	0.84	0.32	2246	0.042	0.189	22%
Outlet 2600 Pop Ave Sth	0.71	0.29	3184	0.109	0.204	53%
Outlet 3700 E	0.84	0.08	3753	0.152	0.257	59%
Outlet 4950 E	0.82	0.19	842	0.020	0.042	48%
Outlet 4950 W	0.70	0.17	709	0.043	0.051	84%
Outlet 5400 E	0.34	0.14	621	0.026	0.041	63%
Outlet 5500 E	0.76	0.02	1480	0.043	0.103	42%
Outlet 8020 E	0.53	0.02	171	0.010	0.011	94%
Outlet 8020 W	0.59	0.03	190	0.011	0.012	88%
Outlet 10400 E	0.63	0.08	1233	0.041	0.058	71%
Outlet 12211 E	0.90	0.39	3497	0.111	0.213	52%
Outlet 13200 W	0.80	0.05	2013	0.112	0.143	78%
Outlet 14000 W	0.79	0.07	1784	0.073	0.140	52%
Outlet 14000 E	0.56	0.05	1951	0.052	0.155	34%
Outlet 14630 E	0.70	0.04	1588	0.017	0.078	22%
Outlet 15650 W (1)	0.26	0.08	1285	0.075	0.124	60%
Outlet 15650 W (2)	0.37	0.14				
Outlet 15650 E	0.55	0.04	2034	0.053	0.137	38%
Outlet 15750 W	0.41	0.04	544	0.013	0.039	34%
Outlet 15750 E	0.20	0.06	381	0.009	0.032	28%
Outlet 15900 W	0.77	0.08	1509	0.080	0.127	63%
Outlet 15900 E	0.53	0.09	1034	0.033	0.100	33%
Outlet 16400 W	0.75	0.12	1679	0.041	0.134	30%
Outlet 16800 E	0.83	0.38	6372	1.23	0.076	6%
Outlet 16400 E	0.47	0.07	704	0.014	0.067	21%
Outlet 17200 W	0.20	0.19	4894	0.056	0.660	8%
Outlet 17250 W	0.49	0.19	496	0.016	0.089	18%
Outlet 17600 W	0.45	0.19	3480	0.217	0.414	52%

Table 7: 50%AEP storm swale attenuation results

Outlet Chainage	Max depth in swale	Max Velocity (m/s)	Total Volume (m3)	Post Exp Peak Discharge (m3/s)	Pre Exp Peak Discharge (m3/s)	% of Pre-Exp Peak Discharge (m3/s)
Outlet 1940 W	0.36	0.101	2407	0.131	0.423	31%
Outlet 2200 W	0.36	0.055	1263	0.026	0.097	27%
Outlet 2600 W	0.63	0.674	3401	0.155	0.272	57%
Outlet 2600 Pop Ave Nth	0.68	0.245	1586	0.017	0.133	13%
Outlet 2600 Pop Ave Sth	0.58	0.298	2250	0.099	0.177	56%
Outlet 3700 E	0.75	0.071	2491	0.127	0.211	60%
Outlet 4950 E	0.66	0.22	596	0.017	0.030	57%
Outlet 4950 W	0.58	0.13	502	0.033	0.036	92%
Outlet 5400 E	0.27	0.12	440	0.022	0.030	73%
Outlet 5500 E	0.65	0.02	1047	0.035	0.073	48%
Outlet 8020 E	0.38	0.01	122	0.009	0.008	106%
Outlet 8020 W	0.44	0.01	135	0.009	0.009	104%
Outlet 10400 E	0.50	0.09	875	0.033	0.041	80%
Outlet 12211 E	0.86	0.325	2736	0.086	0.151	57%
Outlet 13200 W	0.70	0.116	1429	0.039	0.040	98%
Outlet 14000 W	0.62	0.26	1266	0.064	0.066	97%
Outlet 14000 E	0.45	0.053	1384	0.031	0.031	98%
Outlet 14630 E	0.58	0.042	1127	0.016	0.016	100%
Outlet 15650 W (1)	0.24	0.048	912	0.047	0.058	81%
Outlet 15650 W (2)	0.30	0.14				
Outlet 15650 E	0.49	0.043	1443	0.015	0.016	92%
Outlet 15750 W	0.32	0.04	386	0.001	0.012	10%
Outlet 15750 E	0.10	0.046	270	0.006	0.007	86%
Outlet 15900 W	0.72	0.058	1071	0.027	0.031	87%
Outlet 15900 E	0.42	0.028	734	0.029	0.030	97%
Outlet 16400 W	0.60	0.132	1191	0.036	0.036	100%
Outlet 16400 E	0.38	0.09	499	0.013	0.013	97%
Outlet 16800 E	0.54	0.243	4087	0.600	0.875	69%
Outlet 17200 W	0.15	0.15	3452	0.360	0.473	76%
Outlet 17250 W	0.39	0.16	352	0.014	0.063	22%
Outlet 17600 W	0.36	0.1	2470	0.069	0.293	24%

Wetland Reference	1%AEP Volume[5] (m^3)	Pre Exp Max Flow (m^3/s) 1%AEP	Post Exp Max Flow (m^3/s) 1%AEP	Percentage of Pre Exp Flow 1%AEP	Pre Exp Max Flow (m^3/s) 10%AEP	Post Exp Max Flow (m^3/s) 10%AEP	Percentage of Pre Exp Flow 10%AEP	Pre Exp Max Flow (m^3/s) 50%AEP	Post Exp Max Flow (m^3/s) 50%AEP	Percentage of Pre Exp Flow 50%AEP
Wetland 0A	500	0.250	0.114	46%	0.229	0.112	49%	0.213	0.110	52%
Wetland 3	2590	0.544	0.380	70%	0.605	0.307	51%	0.218	0.084	39%
Wetland 4	6242	1.470	1.140	78%	0.851	0.698	68%	0.602	0.385	64%
Wetland 5	2642	0.845	0.128	15%	0.476	0.043	9%	0.338	0.020	6%
Wetland 6[6]	2118	0.967	0.678	70%	0.567	0.495	87%	0.402	0.384	96%
Wetland 8	1848	1.036	0.800	77%	0.568	0.537	95%	0.402	0.384	96%
Offset Storage Area 9a	806	0.244	0.133	55%	0.117	0.068	58%	0.083	0.053	64%
Wetland 9	6206	0.544	0.051	9%	0.297	0.033	11%	0.211	0.022	10%
Wetland 10	750	0.755	0.597	79%	0.442	0.407	92%	0.314	0.287	91%
Wetland 10a[7]	-	0.220	0.180	82%	0.143	0.097	68%	0.04	0.039	98%
Wetland 10b[8]	-	0.490	0.365	74%	0.358	0.286	80%	0.207	0.173	84%
Wetland 11a	140	0.046	0.008	17%	0.03	0.005	17%	0.021	0.003	14%
Wetland 11b	40	0.016	0.007	44%	0.011	0.007	64%	0.008	0.005	63%
Wetland 12	782	0.400	0.326	82%	0.263	0.093	35%	0.186	0.081	44%

Table 8: Wetland attenuation results

[5] Water quality volume is additional to this and has been calculated separately.

[6] Wetland 6 works in combination with swales to achieve required attenuation.

[7] Wetlands 10A and 10B are for additional treatment, not attenuation. Therefore no 1%AEP volume is listed. The swales that drain into them give the attenuation listed in this table.

7 Conclusions

Attenuation has been achieved in most swales and wetlands to 80% of pre-expressway flows along the length of the expressway for the 1% AEP event. The main focus of the modelling was based around this 1% AEP event because if the design has the capacity to contain the volume of water generated in the 1% AEP event then it could contain the 10% AEP and the 50% AEP also. The results show more outlets are not attenuating to the required standard for these smaller events. However, the outlet design can be refined at the detailed design stage to attenuate more effectively. The majority of the swales are attenuating flows to a level considerably less than the 80% requirement.

Where existing known flooding issue have been identified, for example Wetland 9, post expressway flows have intentionally been attenuated down to well below 80%, in this case 10%, to help reduce the stormwater flooding in the area.

Outlet 12211E could not be reduced to 80% of pre-expressway flow in the 1% AEP event. A wetland would have been the ideal choice for this location but area constraints meant this was not possible. One option that will be investigated in later design stages will be to increase the depth of swale in this area. However, this is also a rural area and attenuation to 100% of pre-expressway levels it is considered to be acceptable.

The wetland results summarised in Table 7 show that all wetland outflows are attenuated to approximately 80% of pre expressway flows. The wetlands were sized to accommodate the additional volume required to attenuate the 1% AEP storm

8 References

Stormwater Treatment for State Highway Infrastructure, New Zealand Transport Agency, 2010.

Subdivision and Development Principles and Requirements, Appendix 1, Isohyet Based Calculation of Design Peak Flows, KCDC, 2005.

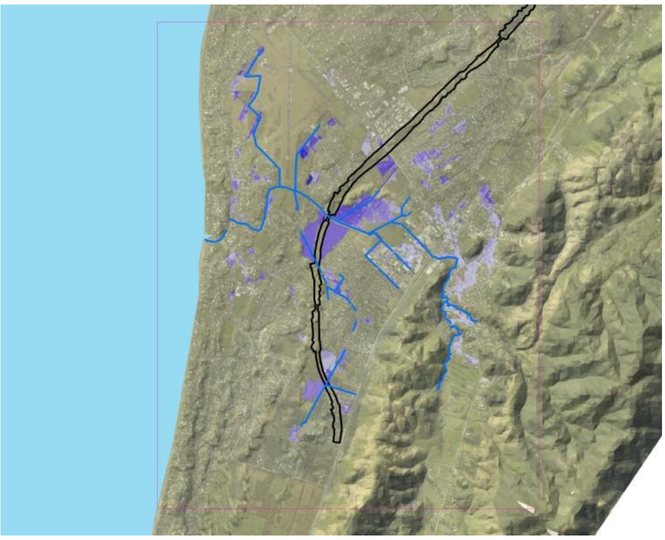
Design guidelines for the calculation of Stormwater Peakflows for the Kāpiti Coast District Council, SKM, 2003.

Update of Kāpiti Coast Hydrometric Analysis: Updated Rainfall Analysis, SKM, 2008.

Appendix 22.E

Drain 7 / Wharemauku Flood Modelling Report

Mackays to Peka Peka Alliance Flood Hazard Mapping



WHAREMAUKU AND DRAIN7 AREA

HYDRAULIC MODELLING AND MAPPING REPORT

M2PP- AEE- RPT-CV –SW- 181

Revision B

20 September 2011

Mackays to Peka Peka Alliance Flood Hazard Mapping

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1. Introduction

The Mackays to Peka Peka Expressway Alliance commissioned SKM to utilise the existing hydrological and hydraulic models of the Wharemauku Catchment, to identify the impacts on flooding associated with the proposed Mackays to Peka Peka Expressway. The existing model was built as part of a comprehensive flood hazard assessment undertaken by SKM and Kapiti Coast District Council (KCDC) in 2009. This model is part of a range of investigations commissioned by KCDC. Other relevant previous studies include:

- Kapiti Town Centre Development: concept design options (Boffa Miskell, Connell Wagner and Traffic Design Group, 2000)
- Wharemauku Stream: Stormwater runoff and floodplain assessment (Connell Wagner, 2001)
- Wharemauku Stream strategic planning – preliminary floodplain assessment (SKM, 2006)
- Wharemauku Stream wider catchment analysis – options report and effects assessment (SKM, 2006)
- Wharemauku Stream Floodplain Management, Volume 1: current status report (SKM, 2009)
- Town centre floodplain management advice: constraints and opportunities (SKM, 2009)
- Long Term Town Centre Flood Storage Options Assessment, SKM July 2010

This report describes the methodology, updates made to the existing model and results to assist the Alliance in undertaking an assessment of hydraulic impacts associated with the expressway.

2. Existing Wharemauku Catchment Model

This section provides a brief overview of the critical information relating to the baseline hydraulic model that has been used as the primary tool in the assessment of hydraulic impacts of the expressway. A comprehensive description of the existing model setup is described in the KCDC Wharemauku Stream Floodplain Management Report Vol 1 (SKM, 2009).

2.1. Hydrology

Hydrological modelling was undertaken using the balanced storm approach that is detailed in the *KCDC Subdivision and Development Principles and Requirements*, KCDC, 2005, with the updated isohyets developed in 2008. The hydrological modelling methodology used has been previously peer reviewed by Beca Infrastructure Ltd (Beca) in the report titled, *Wharemauku Stream Peer Review – Document Review* – September 2008.

The Wharemauku Stream catchment (Figure 1) is approximately 1500 ha in size with a mixed landuse of residential and commercial properties on the plains, and rural farmland and pine forest in the steepland hills east of State Highway 1. The main channel of the Wharemauku Stream is fed from multiple smaller rural drains and branches that feed into it. While the stream appears small in dry weather its proximity to development means that this stream is one of the major flood risks on the Kapiti Coast.

The upper half of the catchment is dominated by steepland hills with high runoff capacity due to shallow clay soils. These steepland areas have also been affected by landslides, and subsequent stream bank erosion as the material from these landslide events have made their way into the stream.

The lower half of the catchment is dominated by a low lying coastal dune/swamp landscape. The development of this land has led initially to drainage of the swamps to allow for rural grazing, and subsequently to residential development. The rural drains still provide the main drainage network into the Wharemauku Stream main channel.

Figure 1shows the extents sub-catchment definition and modelled waterways.

2.2. Hydraulic Model

The existing hydraulic model was developed for KCDC as part of a flood hazard assessment for the area and has previously been peer reviewed by Danish Hydraulic Institute (DHI). The model comprised of a combined 1D and 2D dynamically linked model using the DHI software package

MikeFlood. The lateral linking capability of MikeFlood was used to combine a 1D model of the stream channel constructed in Mike11 and a Mike21 2D model of the floodplain.

This modelling technique allows for the maximising of the strengths of both the 1D and the 2D packages. 1D models are able to accurately simulate in channel process and the impacts of structures while 2D models allow for improved modelling of secondary flow paths and dynamic representation of storage on the floodplain. The Wharemauku catchment hydraulic models were set up to incorporate the piped stormwater network, open channel flow of the Wharemauku Stream and adjacent drains, and surface overland flow of water unconfined to the channel banks or stormwater network. The DHI software programme MOUSE was used to model flow through the piped stormwater network, while an integrated MIKEFlood model combined the channel flow (MIKE11) and surface water on the floodplain (MIKE21).

3. Expressway Modelling Methodology

The model of the Wharemauku catchment was used to undertake the comparison of water levels, extents, flows and velocities in the pre and post construction scenarios to implementation of the expessway. Specifically there are four potential impacts that are being investigated:

- The change in runoff as a result of the change in land use directly associated with the new expressway project.
- Potential changes in flood storage volumes as a result of earthworks on the floodplain
- Alterations to primary flowpaths through new or altered stream crossings
- Alteration to secondary flowpaths through earthworks on the floodplain

3.1. Modelled Scenarios and Boundary Conditions

Three different storm event scenarios were modelled for both the pre and post construction setup. These events were the 10 year and 100 year ARI events and an Extreme event which equates to 1.5 times the flows in a 100 year event. Table 3.1 summarises the modelled scenarios.

Table 1 Waikanae River Boundaries Conditions

Storm Event	Return Period
10% AEP (10 Year ARI incl. CC)	Primary (no nuisance) Event
1% AEP (100 Year ARI incl. CC)	Design Event
1.5x 1% AEP (1.5x 100 Year ARI incl. CC)	Extreme Case

(incl CC = including the predicted midrange impacts of climate change)

All events included an allowance for the predicted midrange impacts of climate change at 2090 in accordance with KCDC standard practice. This allowance includes a 16% increase in rainfall depths and intensities described in the report *Preparing for Climate Change – A guide for Local Government in New Zealand*, MfE, July 2008.

In all scenarios an oscillating 20 year tidal boundary has been used to model the tidal impacts on flooding. This tidal level has been further increased by 0.8m to allow for the predicted impacts of climate change. The Ministry for the Environment report *Coastal Hazards and Climate Change – A Guidance Manual for Local Government in New Zealand* – 2nd Edition, July 2008 includes the recommendation that for long term planning a 0.8m sea level rise should be expected by the year 2090. The peak of the tidal water levels have been synchronised to coincide with the peak flow rates in the open channel.

3.2. Baseline Model

3.2.1. Hydrology

Runoff from the proposed expressway will typically pass through a range of stormwater devices including collection pits, pipes, swales, treatment and storage devices before discharging into the existing drainage networks. The Alliance has investigated, designed and modelled the runoff from the road footprint. They have supplied the runoff hydrographs for the footprint area and their discharge location for both the pre and post construction scenarios. To incorporate these flows into the hydraulic model of the Wharemauku catchment the original sub-catchments and hydrology was reconfigured to accurately reflect the pre and post construction changes and avoid double counting the runoff. Figure 1 shows the updated subcatchment areas and the expressway footprint.

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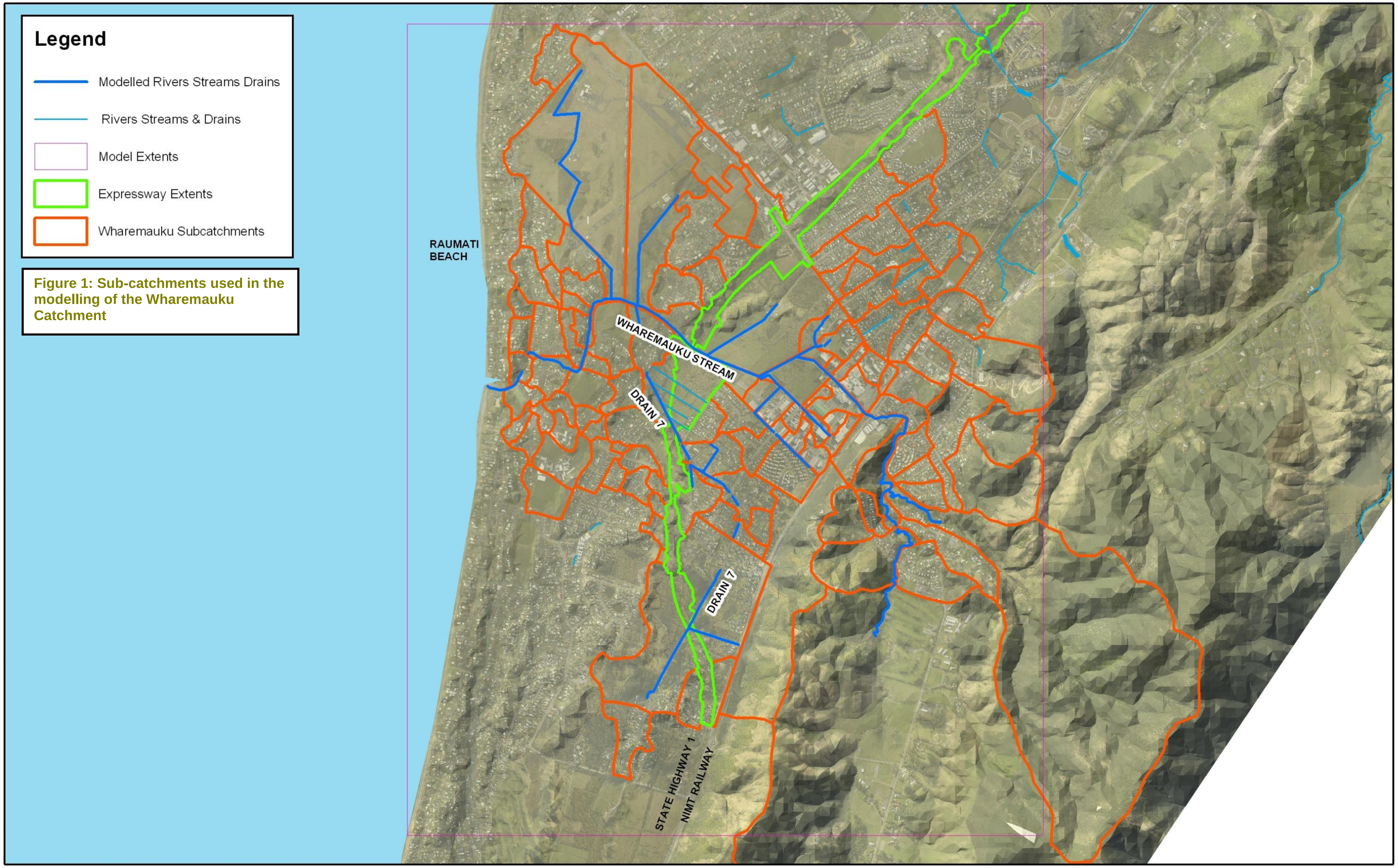

Figure 1: Sub-catchments used in the modelling of the Wharemauku Catchment

PLOT DATE:

FILE NAME:

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CLIENT: SH1 MACKAYS TO PEKA PEKA EXPRESSWAY			
PROJECT: Wharemauku Flood Assessment Modelling			
DRAWN: NC	DATE: 1 AUG 11	REVIEWED PROJECT MANAGER: BF	APPROVED PROJECT DIRECTOR:
DESIGNED:	DESIGN REVIEW:		

TITLE: Wharemauku Catchment Boundaries and Model Extents			
SCALE: 1:22,000	SKM PROJECT No: AE04038	DRAWING No:	AMDT:

A3

3.3. Post Construction Model

To prepare the post construction scenario the following changes were made to the baseline model.

3.3.1. New Crossings

Within the Wharemauku catchment the proposed expressway crosses both the main channel of the Wharemauku Stream as well as the Drain 7 tributary in two locations. The baseline MIKE11 model was updated to reflect three new drain and stream crossing structures as well as some changes to existing channels to incorporate proposed new flood storages and diversions. A model schematic showing the key model updates is shown in Figure 2.

The expressway intersects the modelled channels at three locations, twice across Drain7 and once over the Wharemauku stream. The Wharemauku Stream crossing consists of a 74m (4 spans bridge) structure. As the expressway at this location is on a dune ridge the bridge encompasses the whole bank to bank stream cross-section. Incorporated into the Stream crossing is a cycleway and potential for an Ihakra Street extension, both of which pass under the railway.

The bridge has been incorporated into the hydraulic model using a combination of a MIKE11 bridge structure to allow for the simulation of the hydraulic losses around the piers and altering the MIKE21 bathymetry to incorporate the cycleway and the future Ihakra Street Extension. The cycle way has been set at approximately current top of bank height and the Ihakra Street extension was set at 5.5m above MSL (Wgtn 1953 Datum), that is 0.5m above the peak level predicted in a 10 year event including the predicted midrange impacts of climate change. The stream bed around the structures is designed to include riprap to reduce any erosion risk and this has been reflected in the model by increasing the bed resistance in the relevant locations.

The bridge deck has not been included in the model as this has been designed to be well above the peak water levels in the stream. That is greater than the peak water level that is experienced at this location in the extreme scenario (Q100 x1.5)

The two expressway crossings over Drain7 are represented in the 1D model as culverts. The details of these two structures are presented in Table1.

Table 2 Modelled Drain7 Culvert Structures

Name	U/S Invert	D/S Invert	Shape	Size (m)	Length (m)	Mannings Coefficient	Head Loss Factor Inflow	Head Loss Factor Outflow
Culvert10	5.160	5.090	Circular	1.5	60	0.013	0.5	1
Culvert11	2.975	2.965	Rectangular	3x2	100	0.02	0.5	1
Culvert11.3	3.600	3.500	Circular	1.8	70	0.013	0.5	1

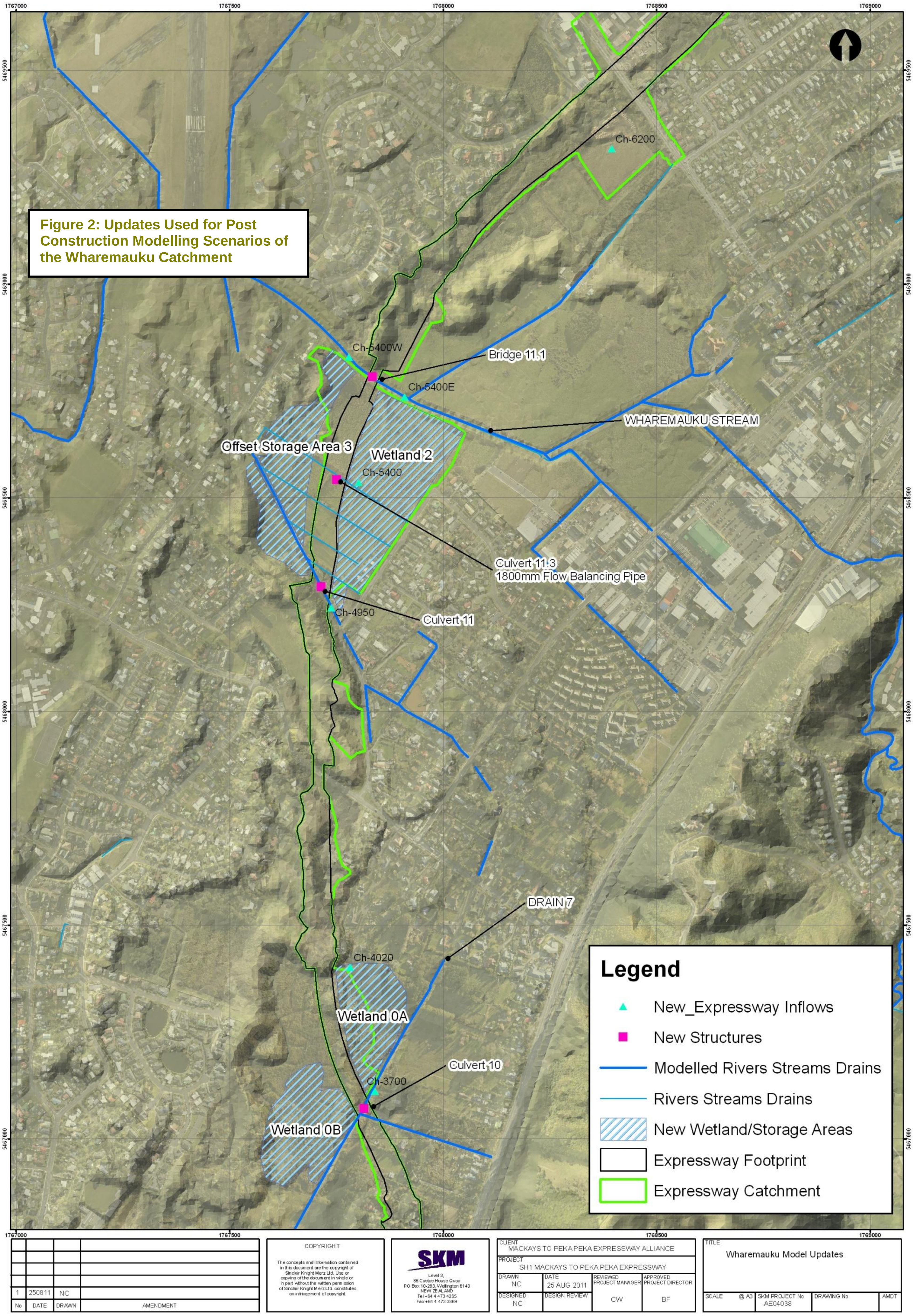

Figure 2: Updates Used for Post Construction Modelling Scenarios of the Wharemauku Catchment

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3.3.2. Earthworks on the Floodplain

The modelled floodplain was updated to reflect the proposed changes to the existing drains and streams. The expressway alignment was also included in the 2D domain as an area of land elevated above the predicted adjacent flood levels. This reflects the design elevation of the expressway which is set above the 1%AEP flood level. Within the model flows on the floodplain can only pass the expressway alignment under bridges or through culverts.

Through a number of modelling iterations the Alliance has developed and designed a range of mitigation measures to address the impacts of the proposed expressway on the flooding. This primarily involved modifications to the topography to recreate flow paths and flood storage that had been altered by the expressway. The modified areas of new flood storage and wetlands are identified in Figure 2 and detailed in Table 2 below. These flood storage areas are provided in addition to any treatment and attenuation provided as part of the expressway stormwater management.

Table 3 Additional Wetland and Flood Storage Compensation

Storage Location	Average Existing Ground Level (m MSL)	Average Modified Ground Level (m MSL)	Storage / Wetland Area (m^2)	Additional Q100 Flood Storage (m^3) above existing
0B	8.6	6.2	32,000	13,000
0A	7.9	6.5	25,000	3,500
3	4.5	3.5	51,500	38,000
2	4.1	3.4	53,500	38,000

4. Modelling Results

Flooding extents, levels, flows and velocities have been extracted for the Q10, Q100 and the Extreme event (Q100 x 1.5). The pre and post construction results have been compared to assist the Alliance in undertaking an assessment of flooding effects. The results are reported in Appendix A as Pre and Post inundation flood maps while an explanation of the hydraulics in specific areas is discussed in further detail in this section.

4.1. Change in Runoff as a Result of Change in Land Use

The Alliance has supplied the pre and post construction hydrographs for the road alignment and the approximate location of discharge into the surrounding drainage network. These discharges are from the expressway stormwater management system and have been attenuated prior to discharge. The nominal target reduction in peak flow to 80% of Pre expressway flow has been achieved in all but a few locations and will be further refined during detailed design.

4.2. New stream crossings

The impacts on the flows and water levels in Drain 7 and the Wharemauku as a result of the expressway have been assessed and the modelled results are presented below in Table 4 and in Figure 3 and Figure4. Overall there has been a reduction in water levels and peak discharges in the vicinity of the new structures. Long section profiles of the relevant stream sections showing pre and post construction peak water levels and discharges are presented in Appendix B. Some variation in peak discharge is noted in the long sections however these are short duration effects associated with the wetland storage areas and have almost no impact to the overall flood risk.

Table 4 Peak Water Levels and Discharge for the Expressway Structures

CULVERT - 10	10%AEP-PRE	10%AEP-POST	1%AEP-PRE	1%AEP-POST	1.5x 1%AEP-PRE	1.5x 1%AEP-POST
WL (m)	6.40	6.38	6.74	6.67	6.98	6.91
Q (m³/s)	0.8	0.9	1.6	0.8	1.9	1.0
CULVERT - 11						
WL (m)	4.16	4.02	4.81	4.82	5.24	5.25
Q (m³/s)	1.0	1.0	1.1	1.1	1.2	1.6
BRIDGE - 11.1						
WL (m)	4.57	4.46	4.99	4.86	5.32	5.35
Q (m³/s)	22.1	21.4	32.6	28.5	35.3	35.0
CULVERT - 11.3						
WL (m)	-	3.96	-	4.87	-	5.36
Q (m³/s)	-	0.5	-	2.9	-	7.0

4.3. Changes in Secondary Flowpaths and Floodplain Storage

Overland flows on the floodplain have been altered at two locations as a result of the new expressway. Earthworks will result in local changes to the flows and loss of floodplain storage around culvert 10 on Drain 7. Wetland 0A and 0B have been constructed to compensate for these impacts. Both Wetland 0A and 0B are designed to operate in a 10%AEP flood event and the true left bank has been modified accordingly as described in Figure 3 below.

The second overland flow area affected is between Wetland 2 and flood storage area 3 as shown in Figure 4. The earthworks in this area are within the town storage area that is a critical component of the flood protection scheme on the Wharemauku Stream. The existing flood storage provided between Wetland 2 and 3 have been extended and are designed to mitigate any impacts as a result of the expressway. Both these storage areas are connected to the Wharemauku and Drain 7 Streams and are designed to operate during a 10%AEP flood event as described in Figure 4. A linking flow balancing culvert has been included between Wetlands 2 and 3.

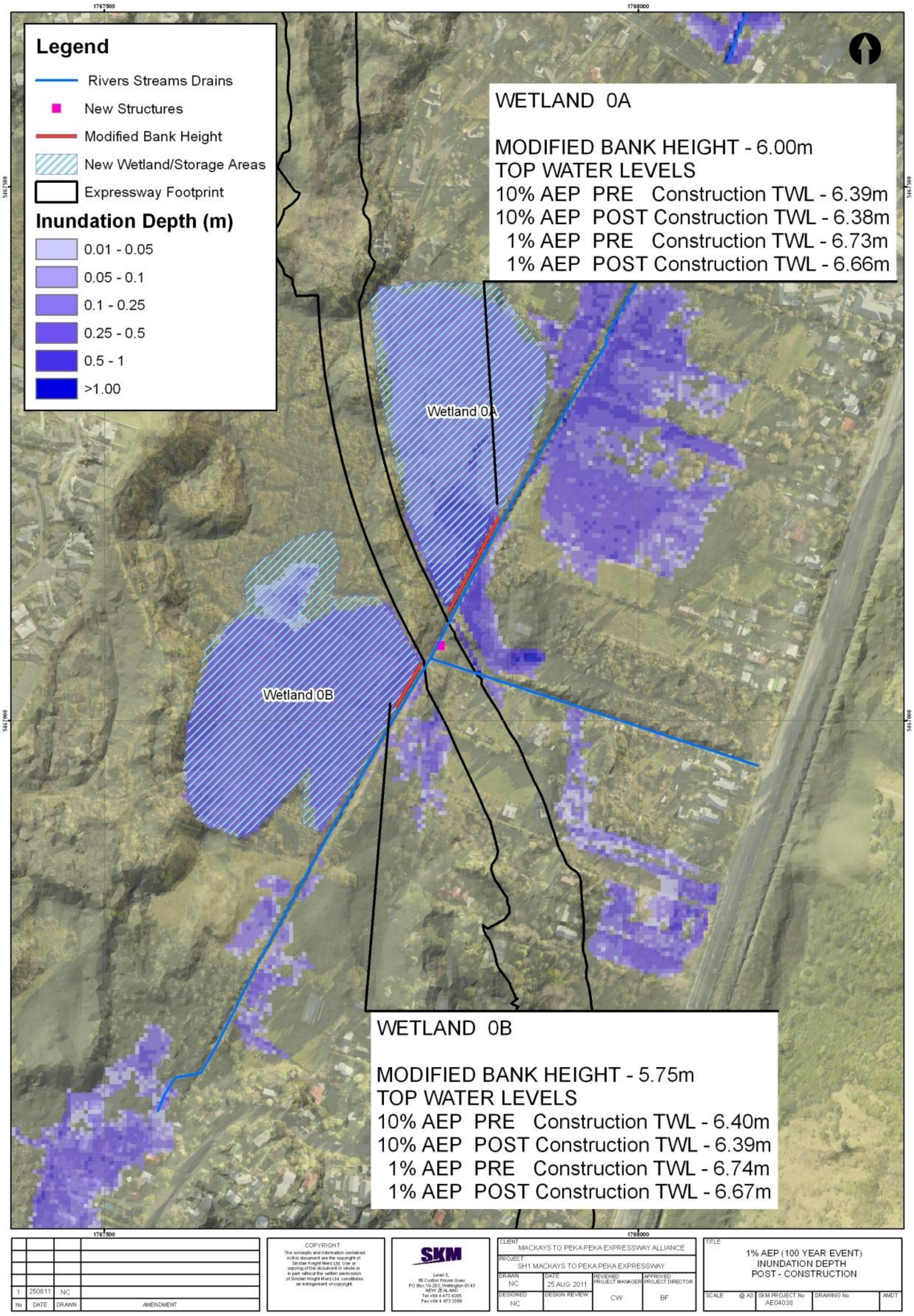


Figure 3 1%AEP Model Results at Culvert 10

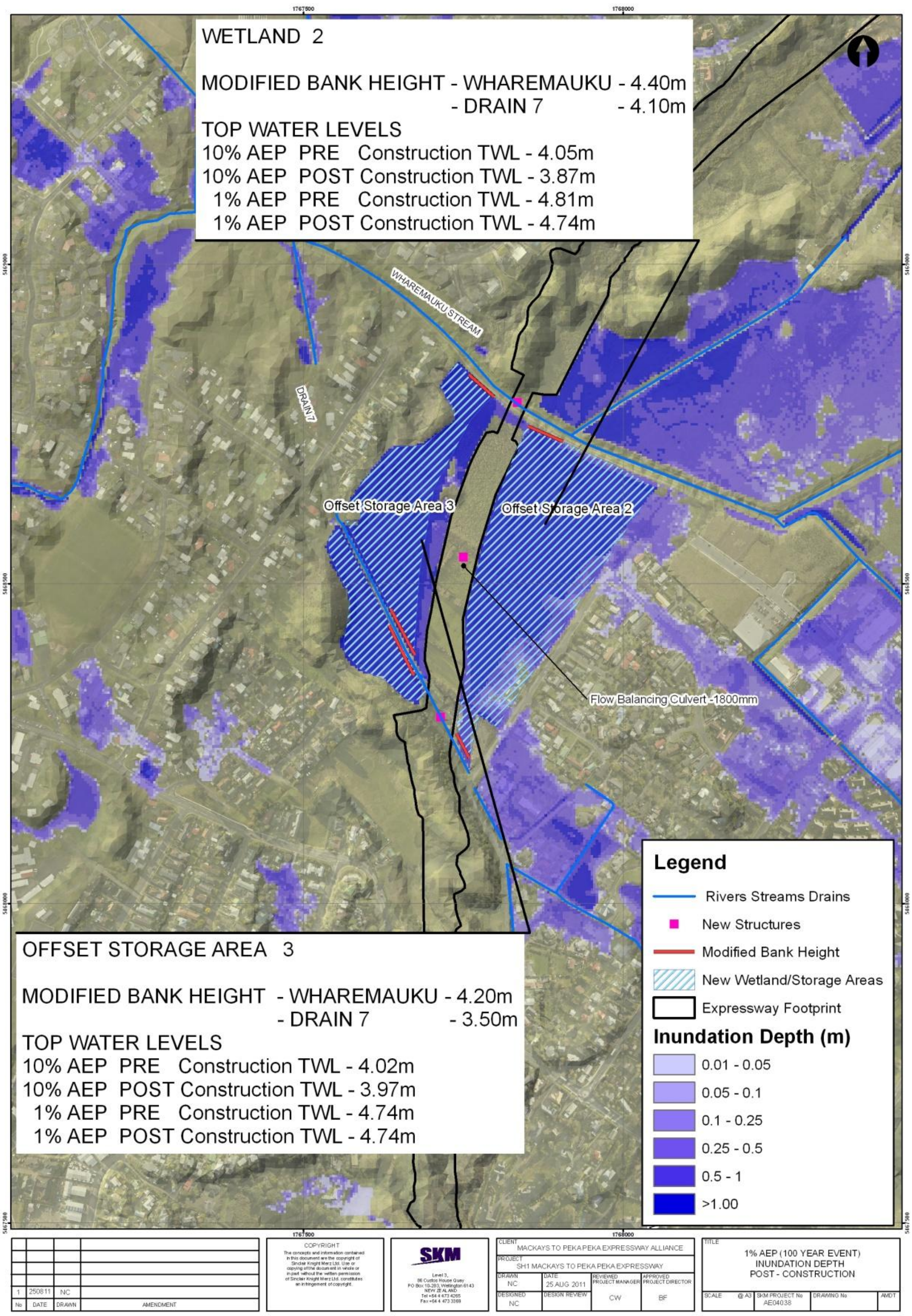
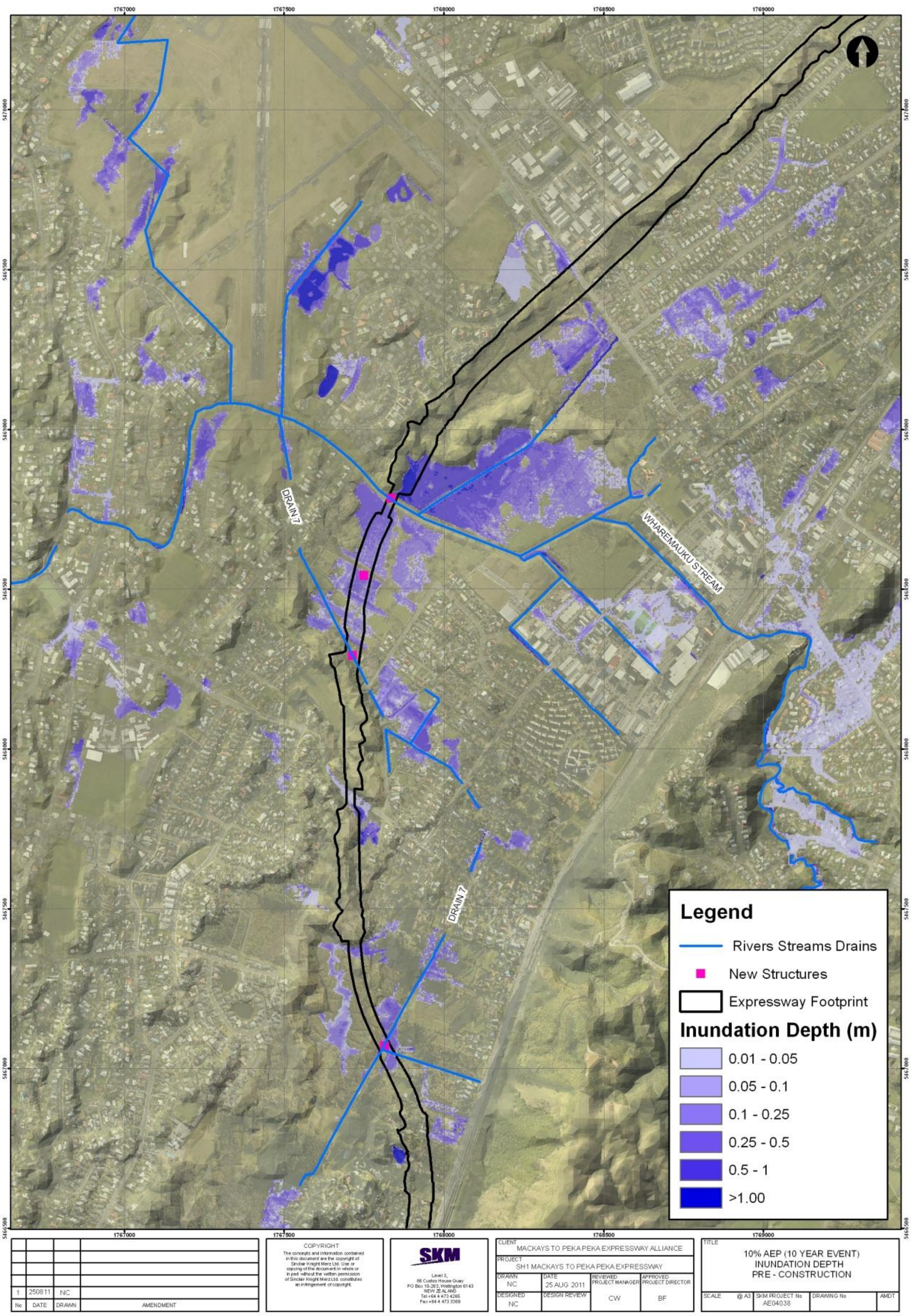


Figure 4 1%AEP Model Results at Culvert 11 and Bridge 11.1

5. Conclusion

The hydraulic modelling of the proposed expressway has proven to be an effective tool in identifying the impacts on flood risk. The key issue that has been identified in the Wharemauku catchment is the loss of storage on the floodplain created by the proposed earthworks. Through a range of mitigation measures including the enlargement of the existing wetland areas, particularly in the town centre storage area, the hydraulic model results suggest that adverse flood risk impacts can be mitigated.

Appendix A – Inundation Maps and Comparisons



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DRAIN 7
WHAREMAUKU STREAM
DRAIN 7
Legend
Rivers Streams Drains
New Structures
Expressway Footprint
Inundation Depth (m)
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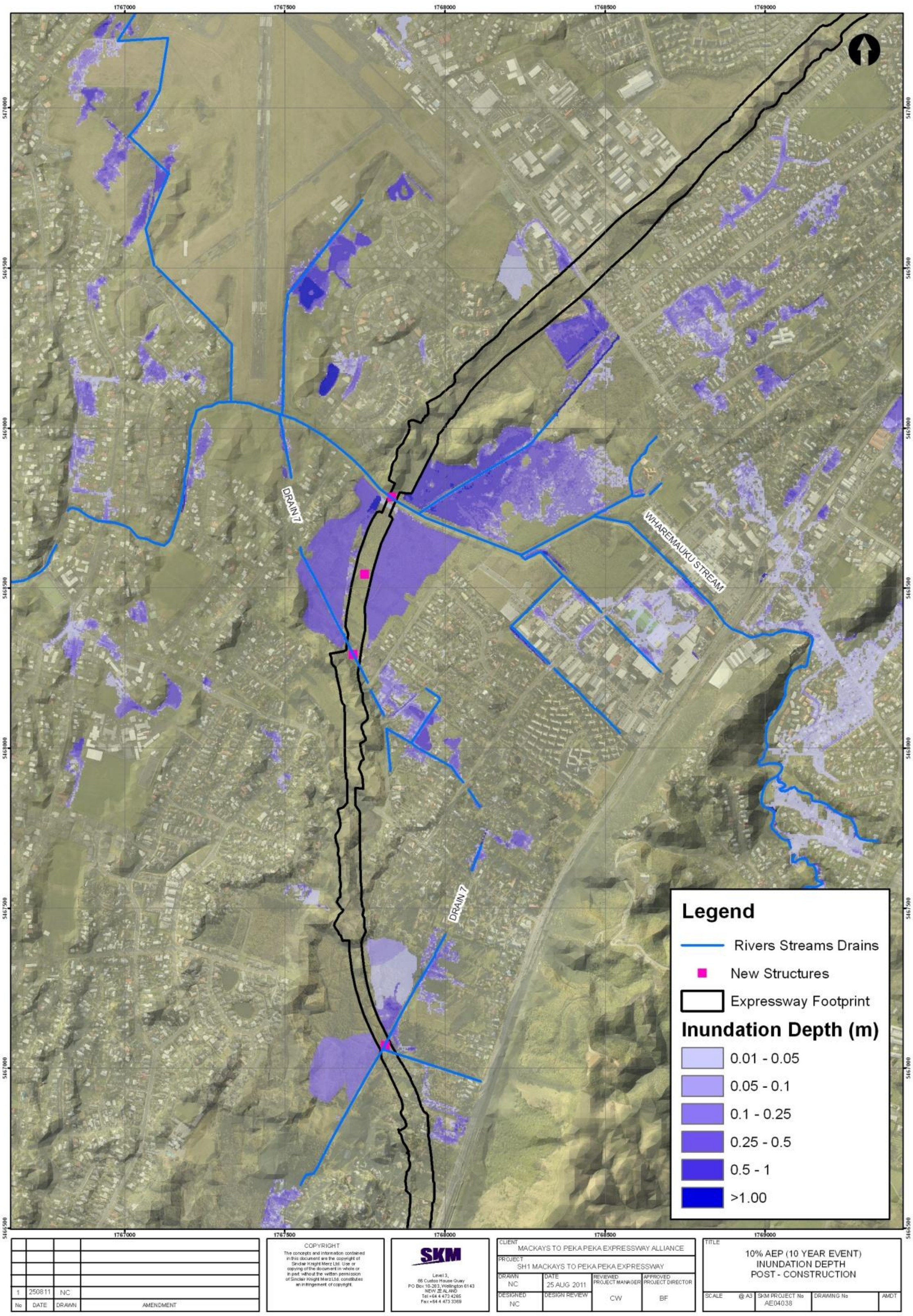
PROJECT SH1 MACKAYS TO PEKA PEKA EXPRESSWAY
DRAWN NC
DATE 25 AUG 2011
REVIEWED PROJECT MANAGER CW
APPROVED PROJECT DIRECTOR BF
DESIGNED NC
DESIGN REVIEW
TITLE
10% AEP (10 YEAR EVENT)
INUNDATION DEPTH
PRE - CONSTRUCTION
SCALE @ A3
SKM PROJECT No AE04038
DRAWING No
AMDT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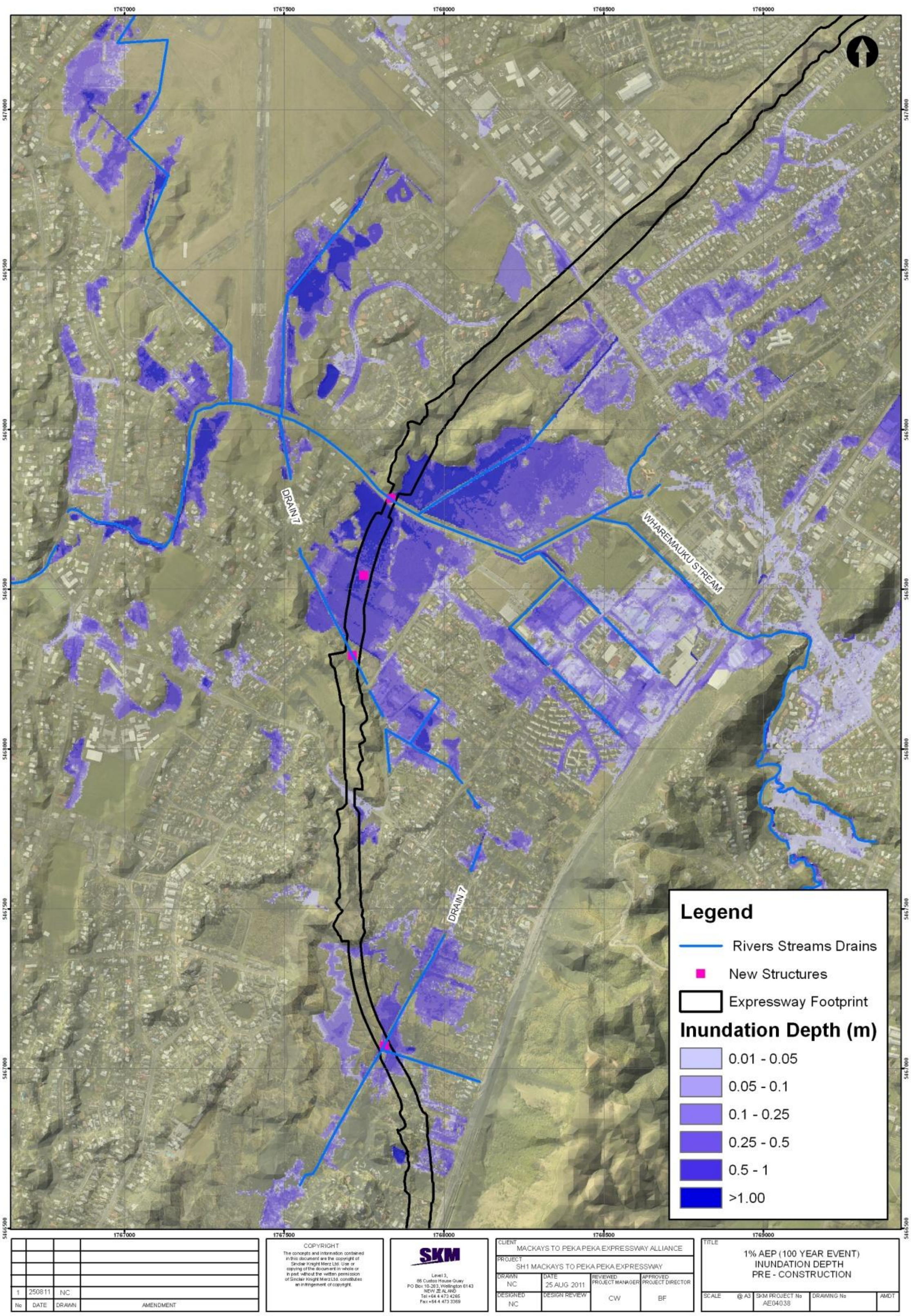
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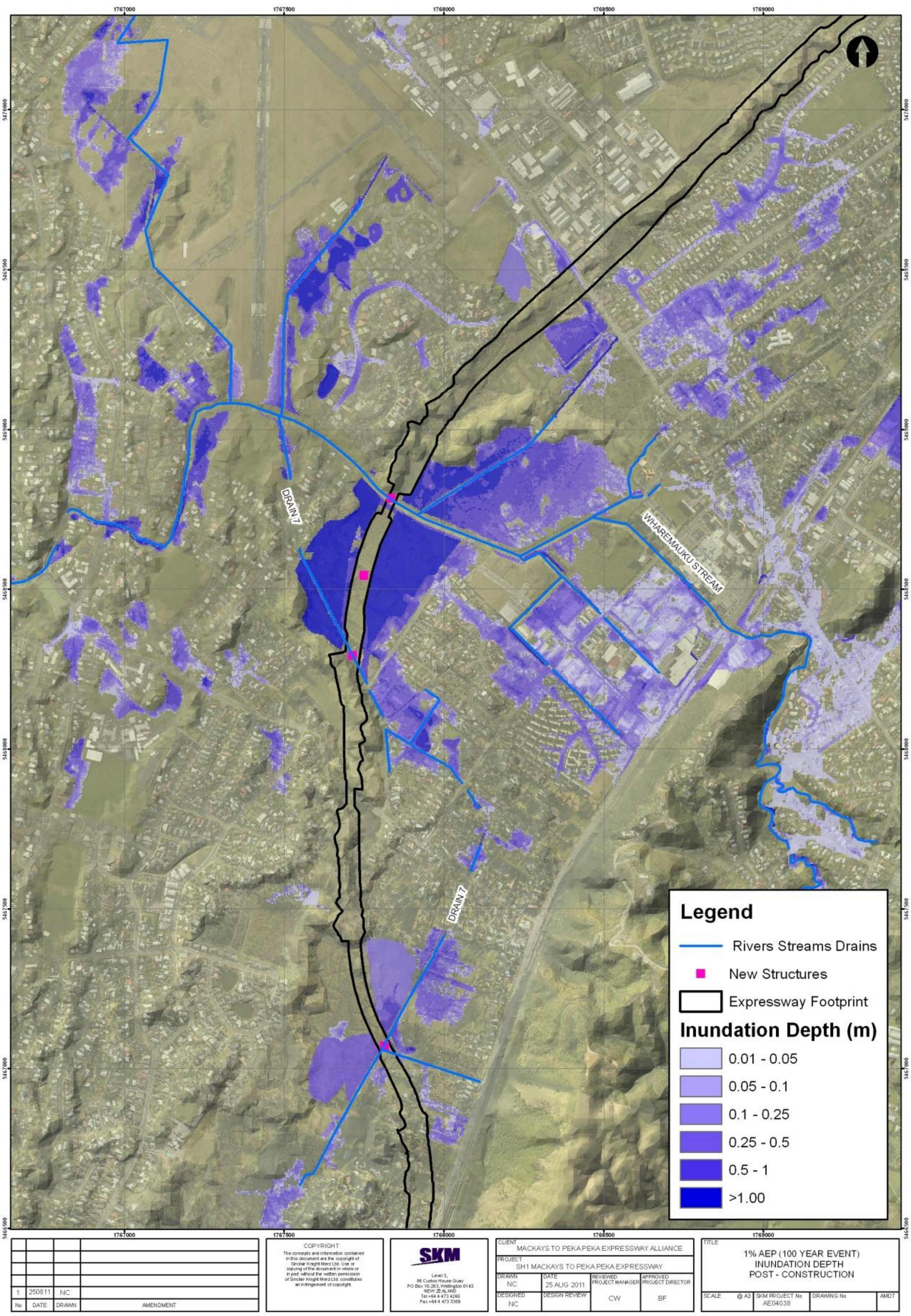
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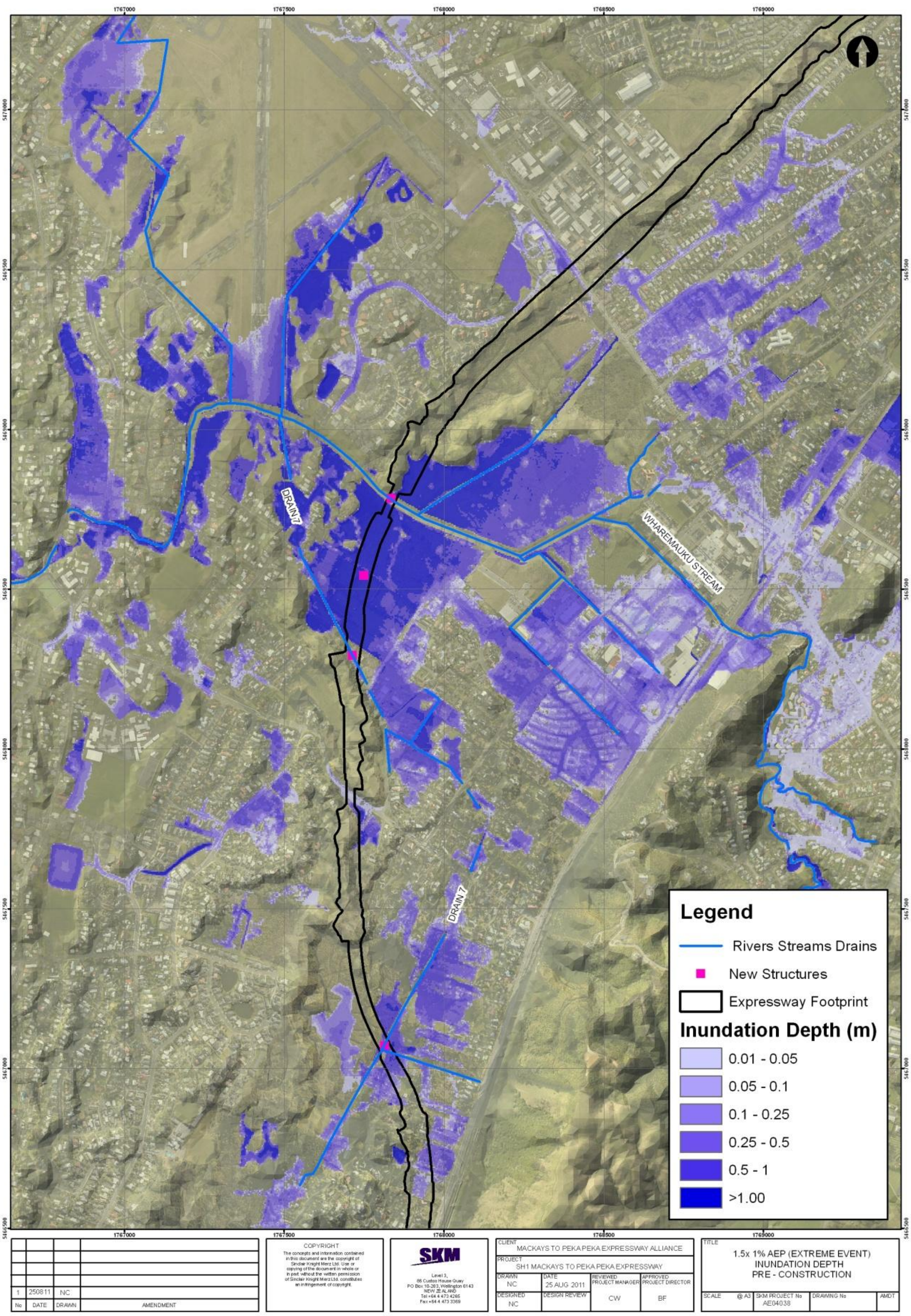
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1.5x 1% AEP (EXTREME EVENT)
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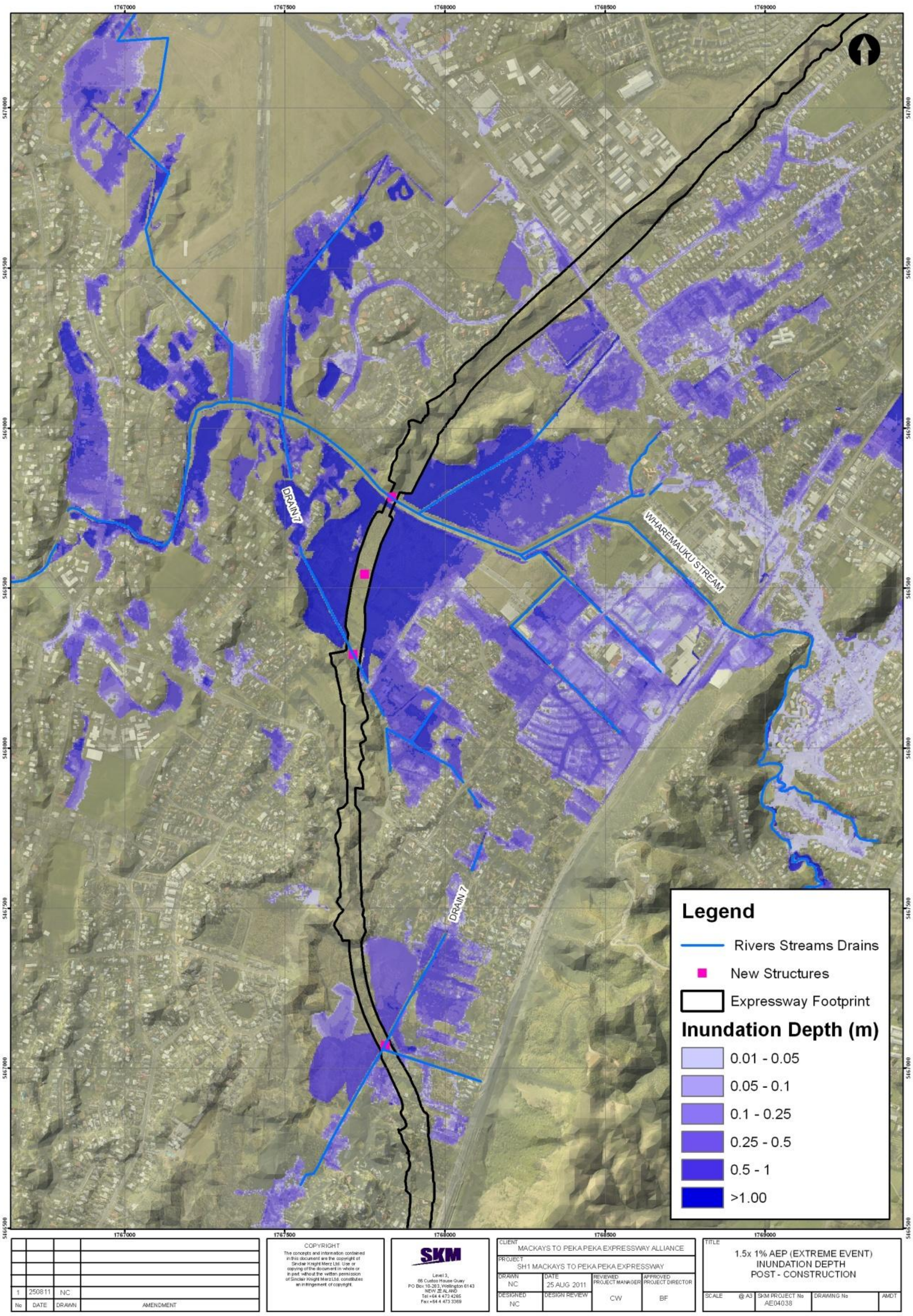
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Appendix B – Long section Profiles

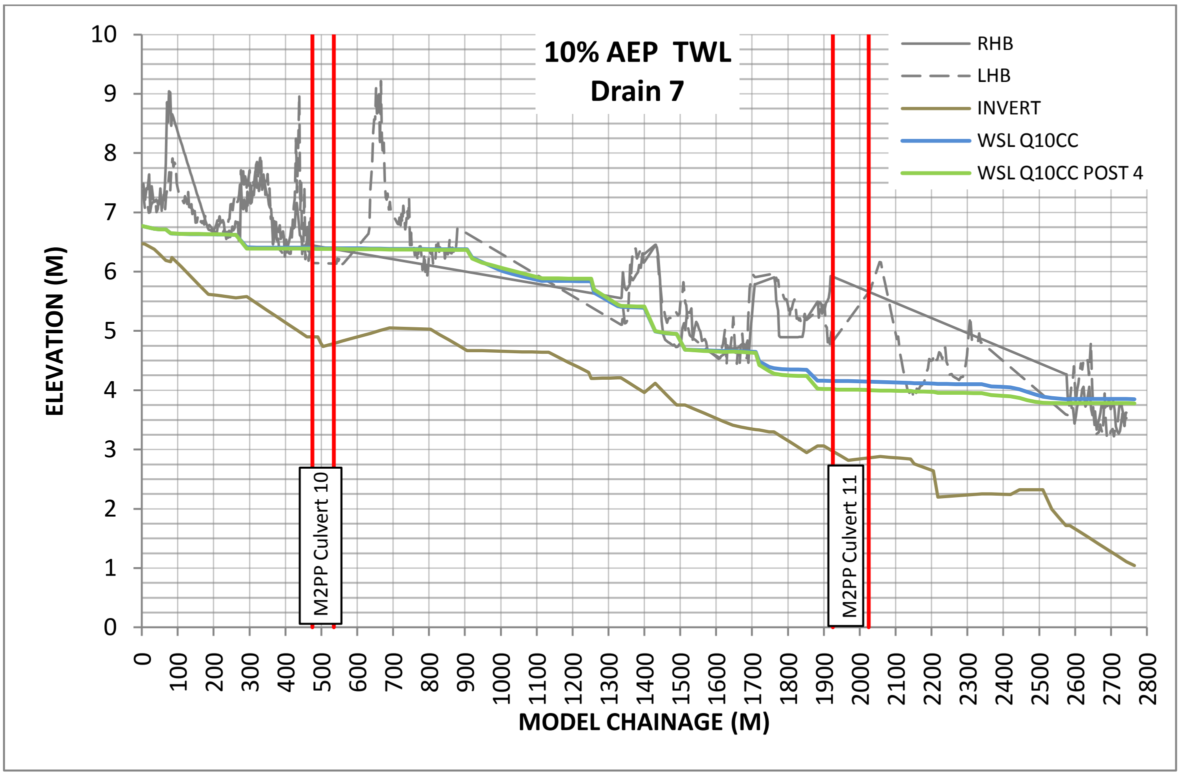


Figure 5 Drain 7- 10% AEP Top Water Level

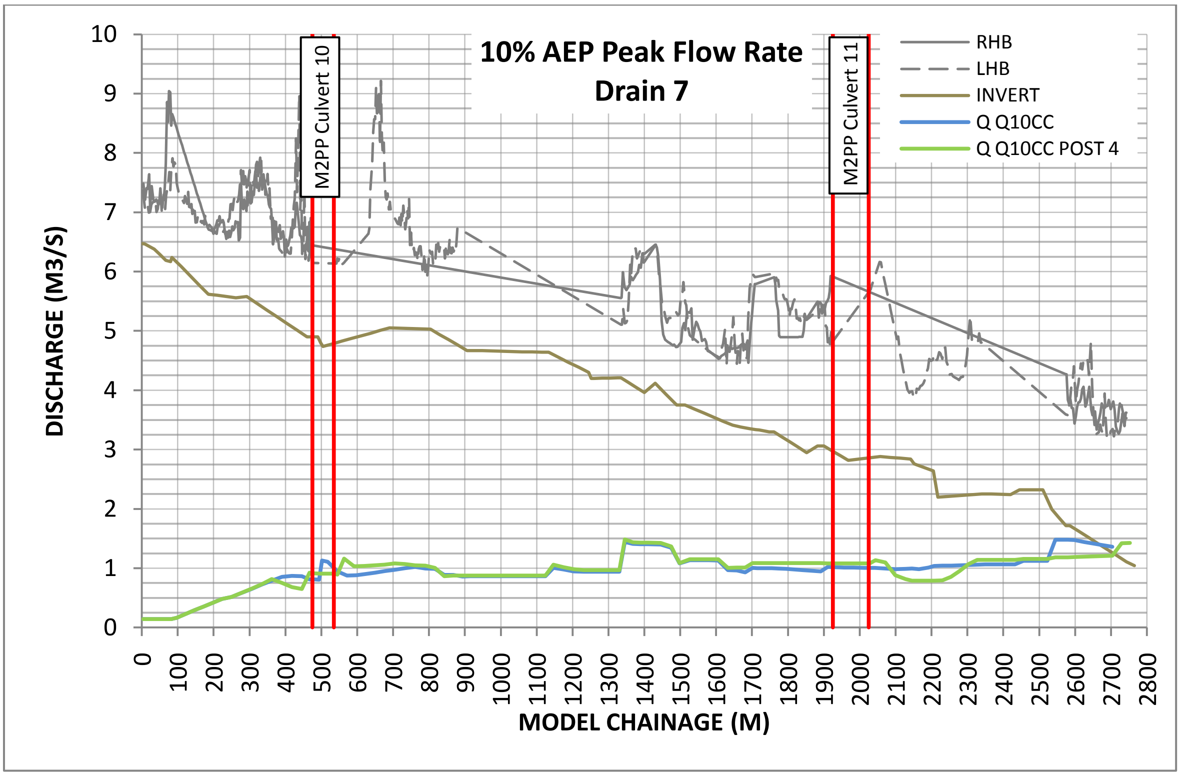


Figure 6 Drain 7- 10% AEP Peak Flow Rate

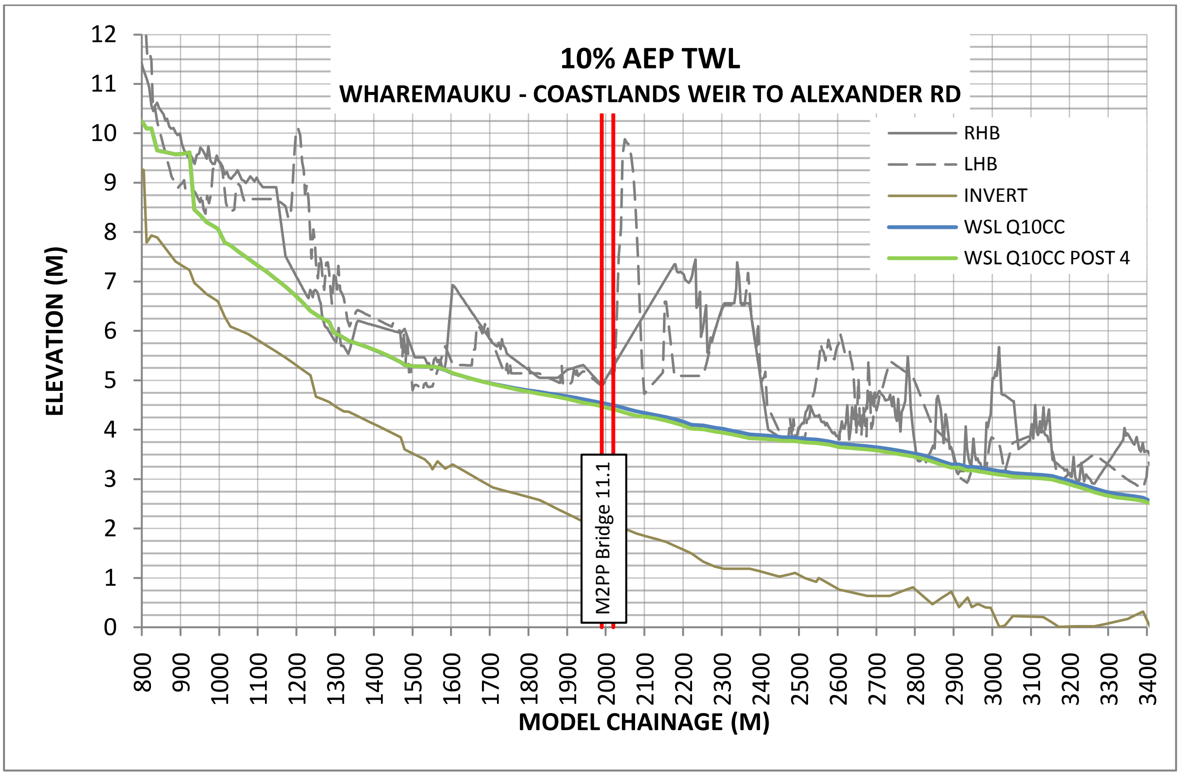


Figure 7 Wharemauku- 10% AEP Top Water Level

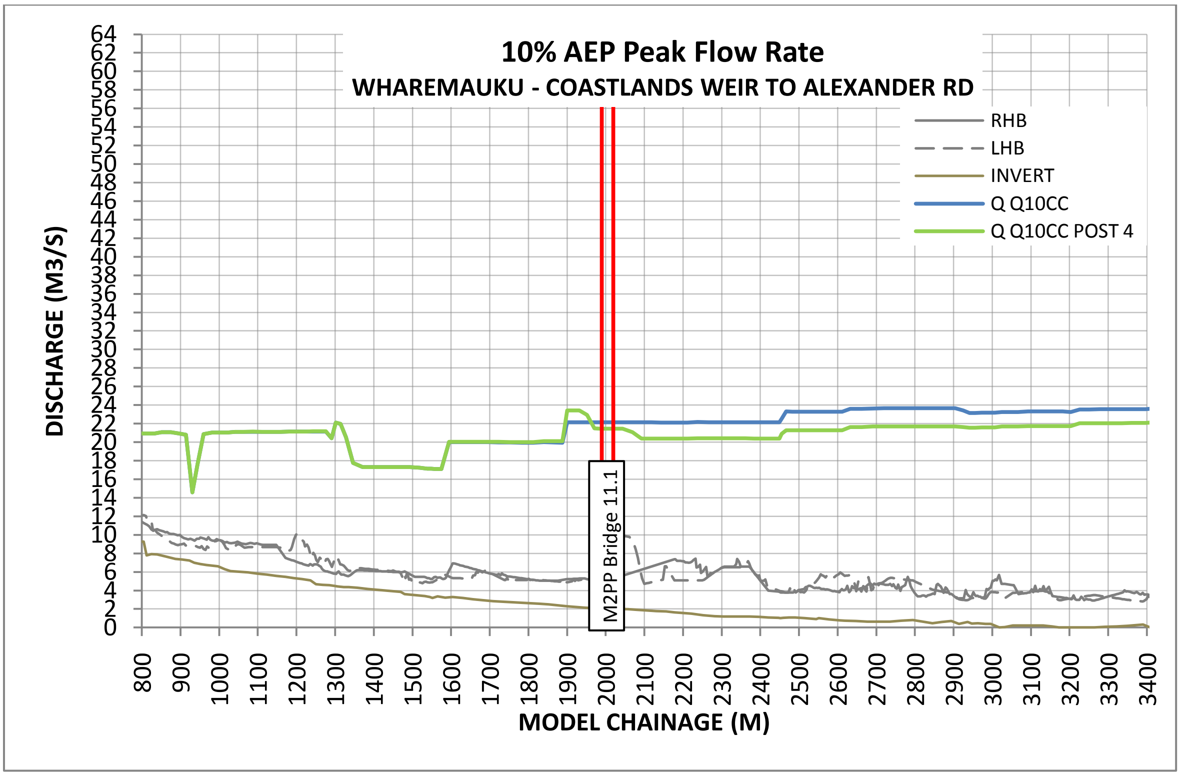


Figure 8 Wharemauku- 10% AEP Peak Flow Rate

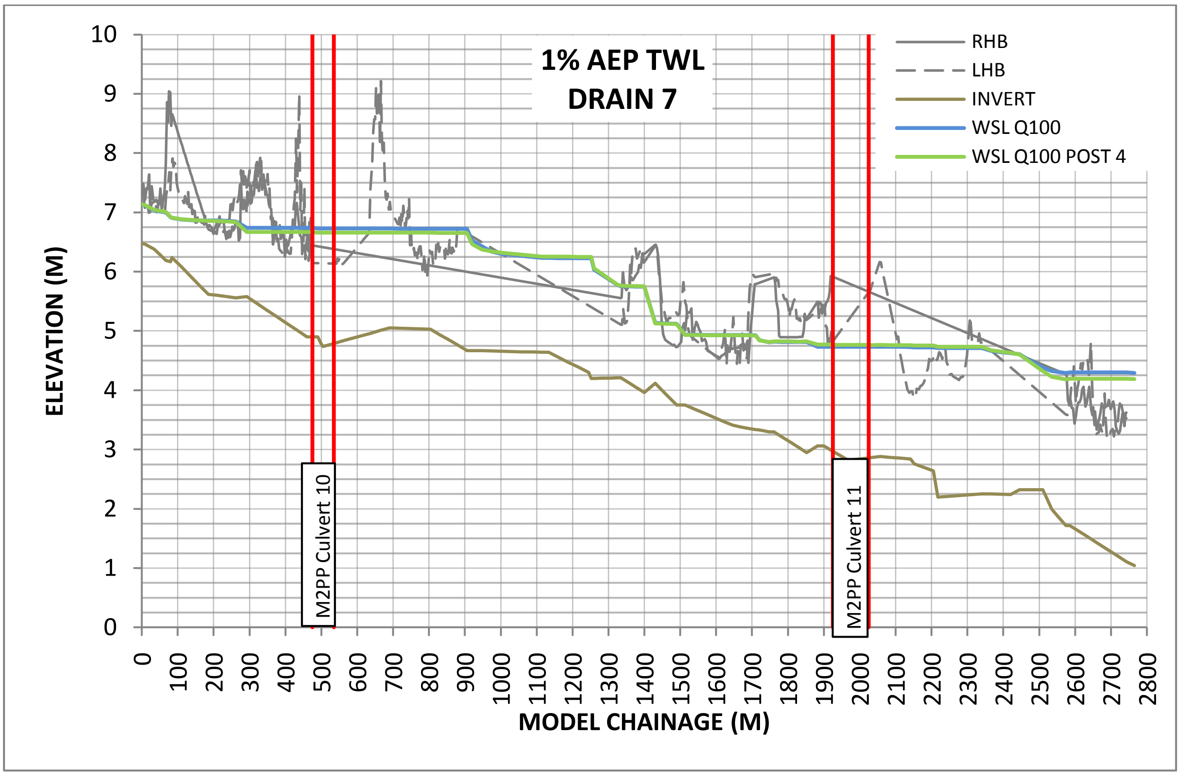


Figure 9 Drain 7- 1% AEP Top Water Level

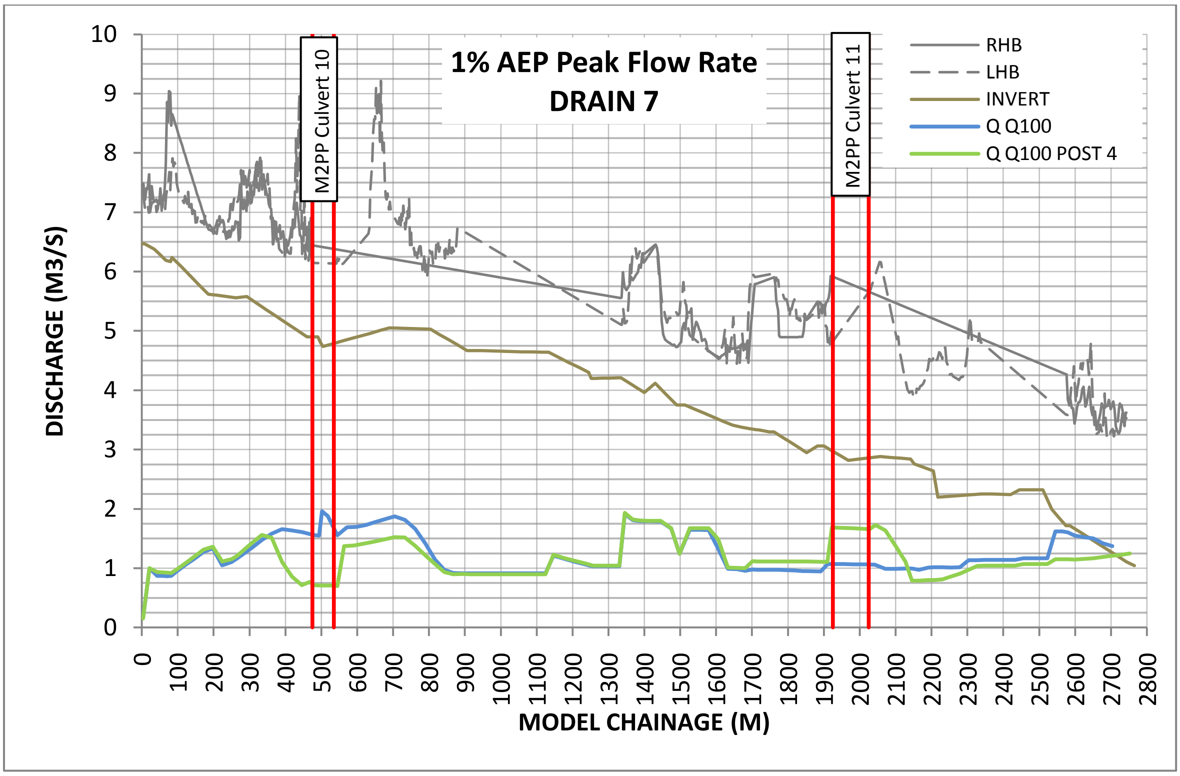


Figure 10 Drain 7- 1% AEP Peak Flow Rate

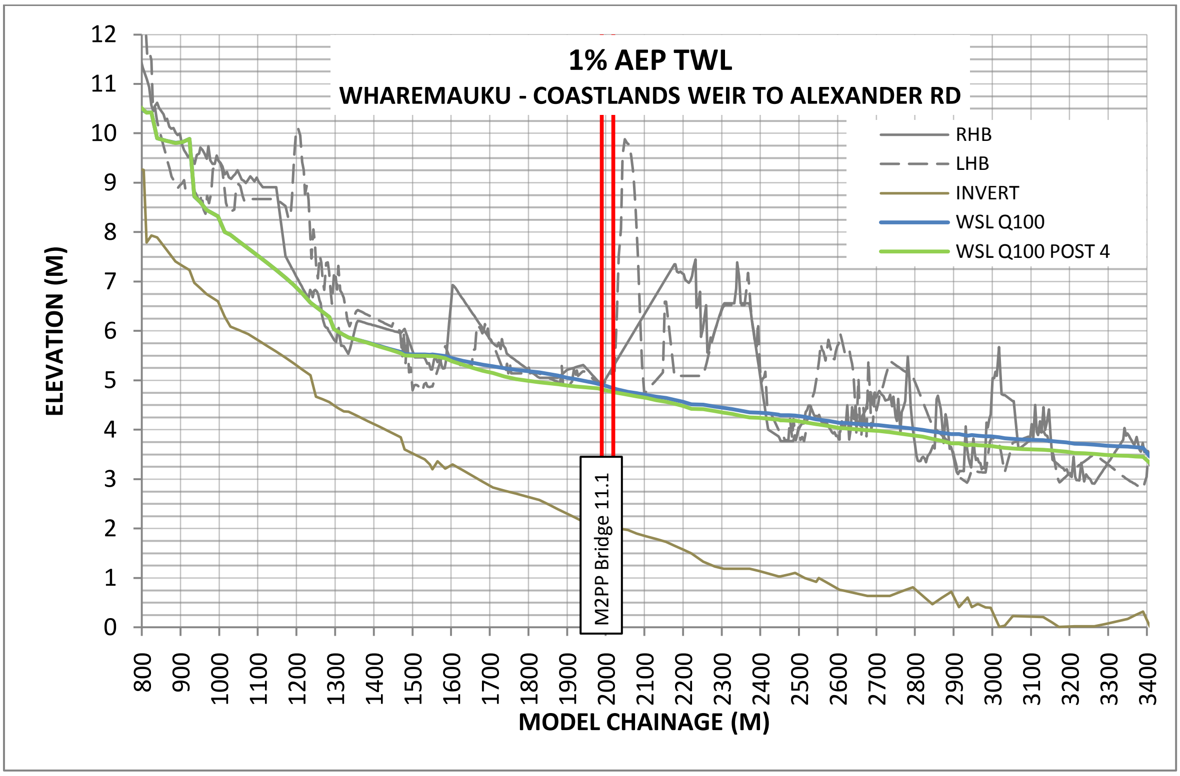


Figure 11 Wharemauku- 1% AEP Top Water Level

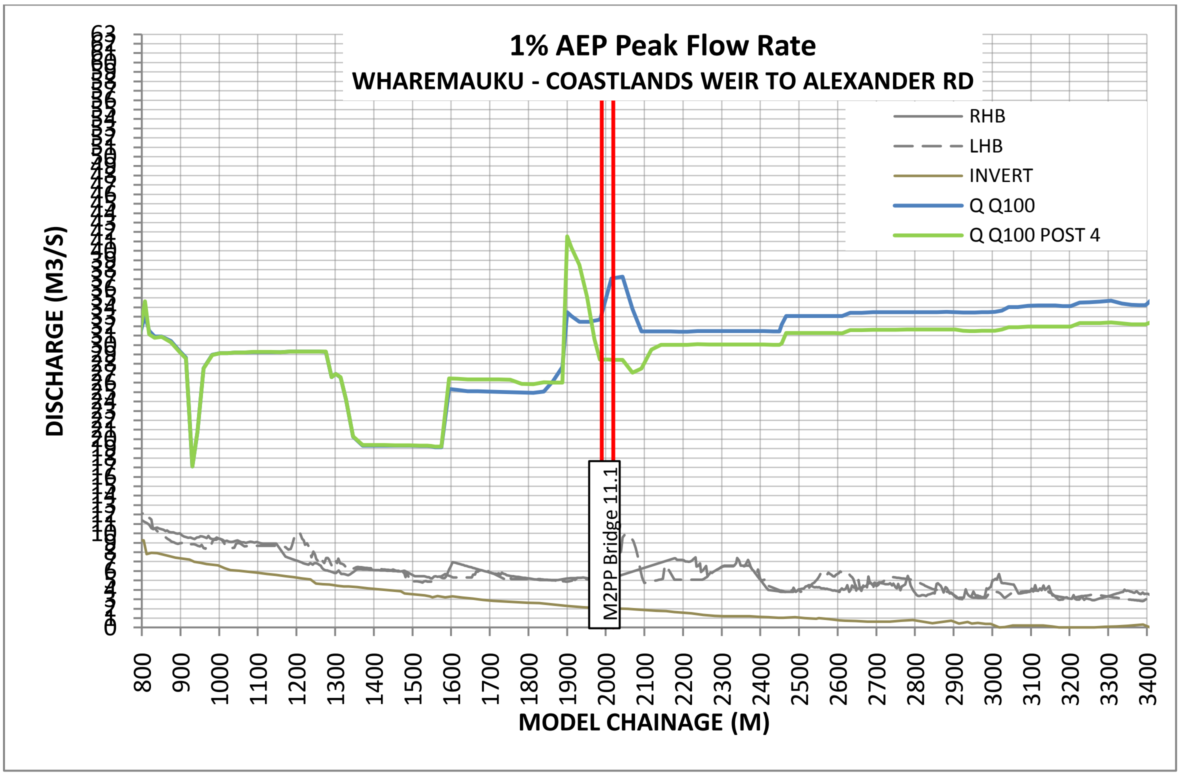


Figure 12 Wharemauku- 1% AEP Peak Flow Rate

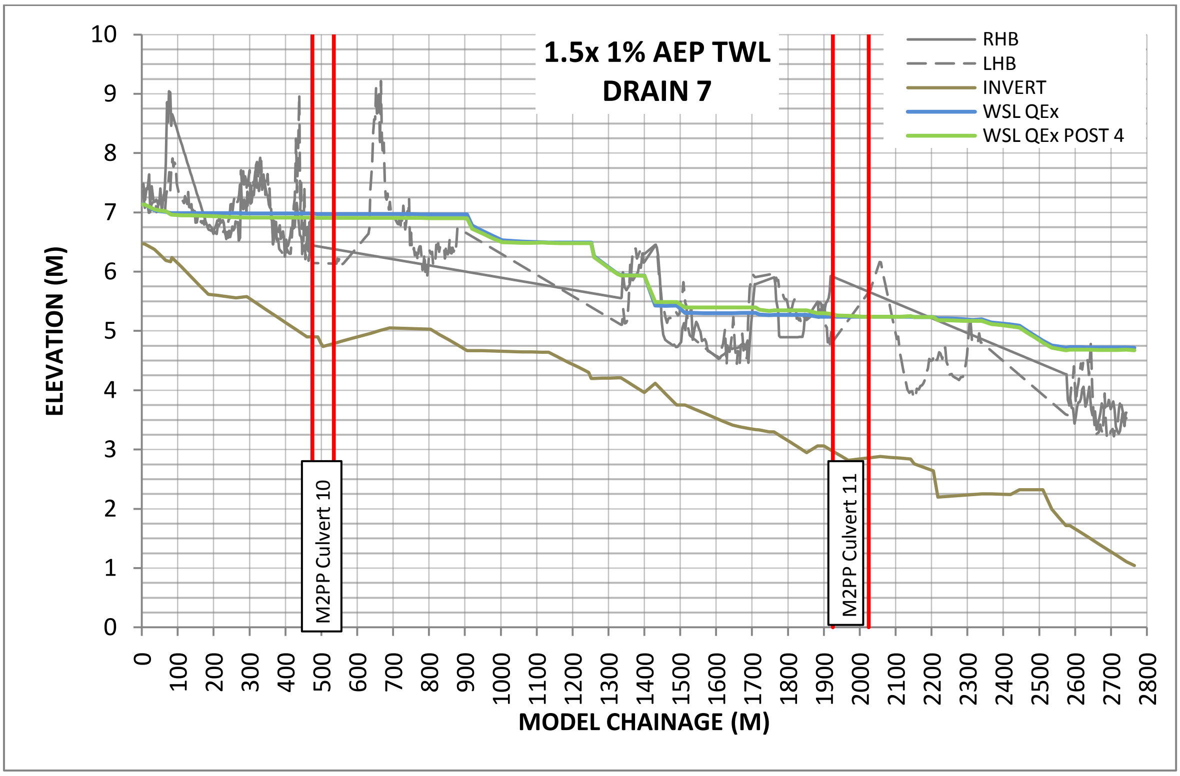


Figure 13 Drain 7- 1.5x 1% AEP Top Water Level

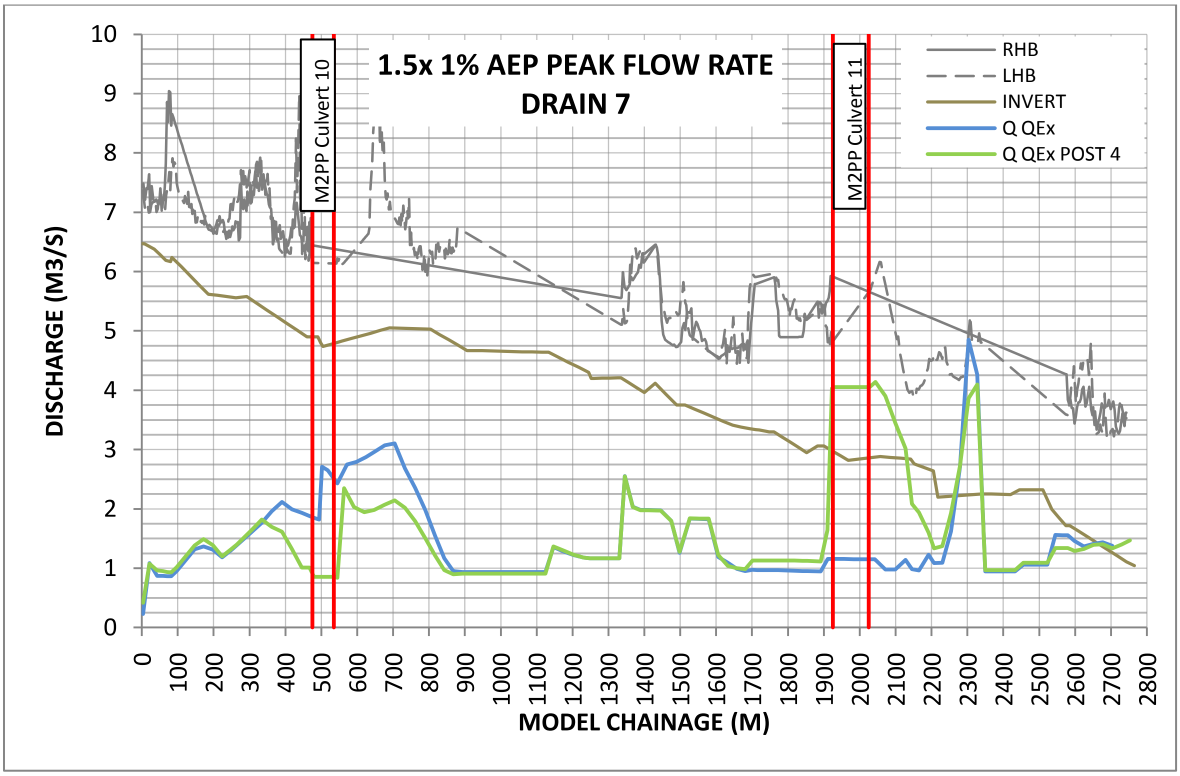


Figure 14 Drain 7- 1.5x 1% AEP Peak Flow Rate

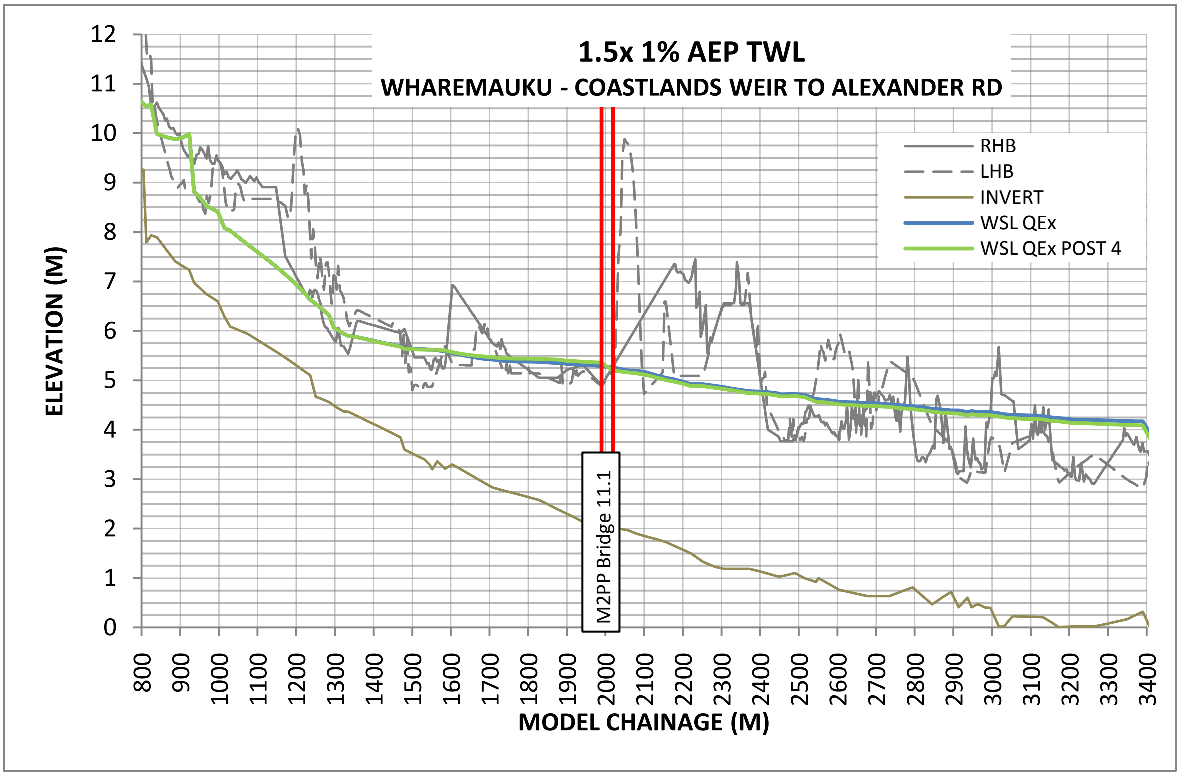


Figure 15 Wharemauku- 1.5x 1% AEP Top Water Level

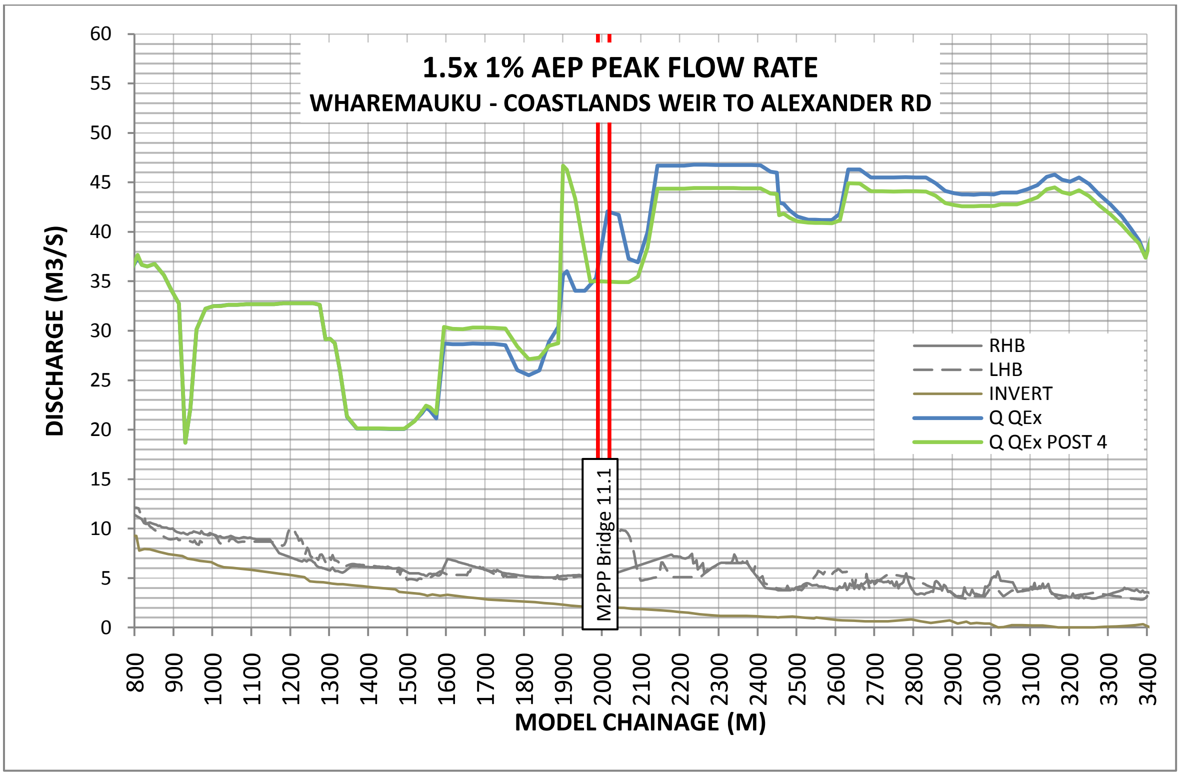


Figure 16 Wharemauku- 1.5x 1% AEP Peak Flow Rate

Appendix 22.F

Mazengarb / Waikanae River Flood Modelling Report

Hydraulic Impacts of M2PP Expressway

(Mazengarb Stream to Waikanae River)

Report prepared for the M2PP Alliance
by

Philip Wallace
River Edge Consulting Limited

November 2011

Contents

1 Introduction

River Edge Consulting Limited was engaged by the Mackays to Peka Peka (M2PP) Expressway Alliance to model the flood impacts of the proposed expressway, between the Mazengarb Drain and the Waikanae River (Figure 1).

The modelling is based on previous modelling undertaken by River Edge Consulting for Greater Wellington Regional Council and Kapiti Coast District Council.

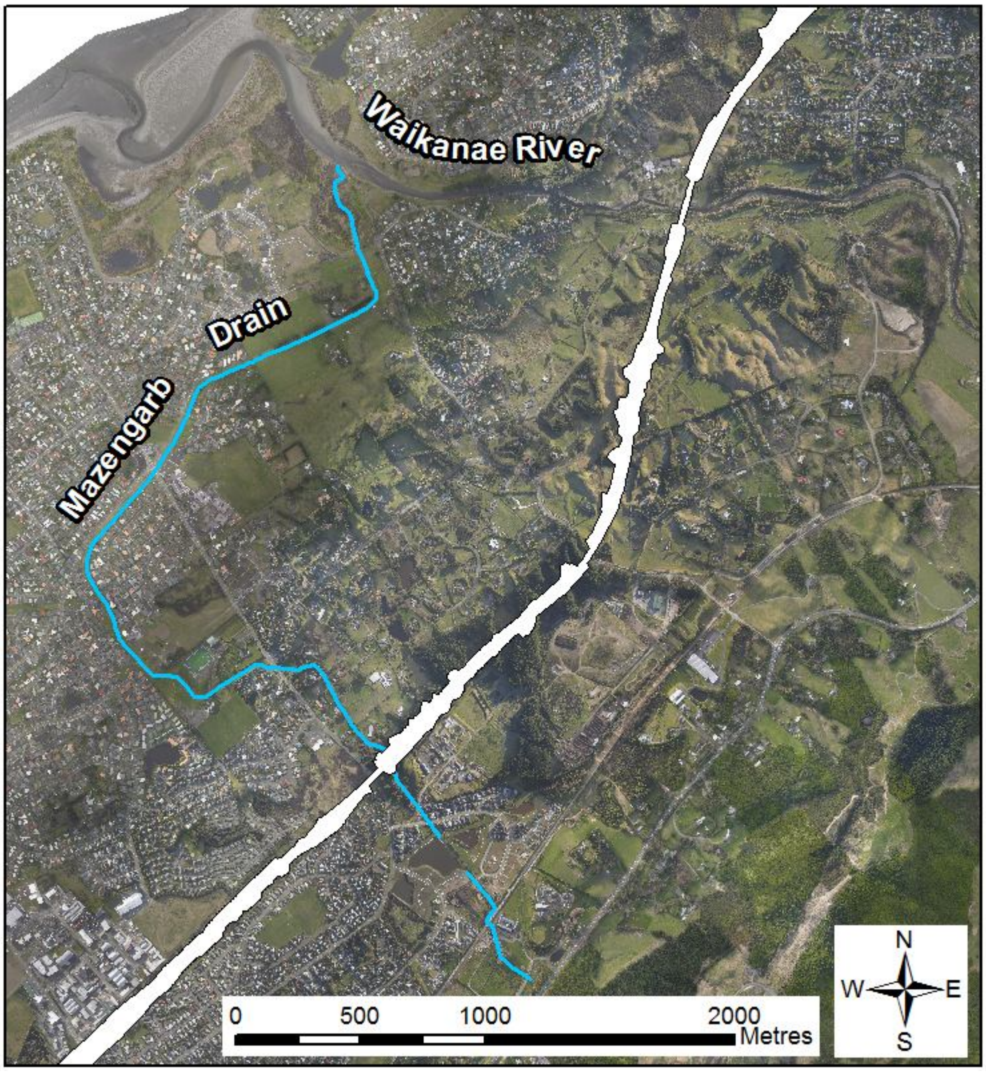


Figure 1 Location map with footprint of proposed expressway

2 Hydraulic Model

The modelling tool used in this exercise is the software program MIKE FLOOD. MIKE FLOOD incorporates MIKE 21 (i.e. 2-D flow equations) and MIKE 11 (1-D flow equations), allowing them to be dynamically linked during a simulation (DHI, 2011).

A model of the area for the existing situation was originally built for Greater Wellington Regional Council, focussing on the Waikanae River and adjacent floodplain. This model was subsequently extended to include the Mazengarb Drain and floodplain, in a study commissioned for Kapiti Coast District Council. Details of the modelling process, data inputs and model findings can be found in reports prepared for the two councils (Wallace (2010) and River Edge Consulting (2011)). The model layout and extent is shown in Figure 2.

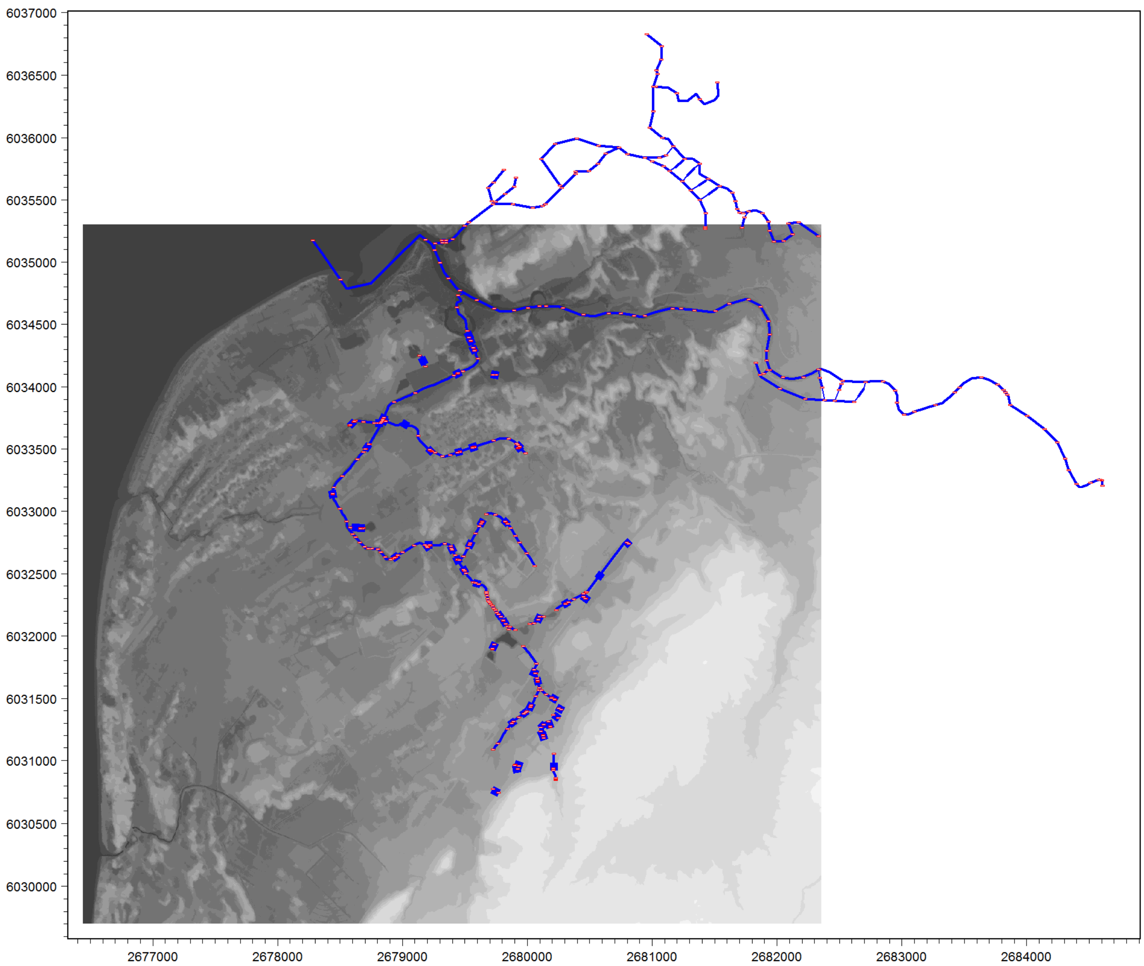


Figure 2 MIKE 21 model area (in grey, showing the topography), with MIKE 11 model network overlaid. (Axis shows NZMG coordinates)

An equivalent model has been constructed for the M2PP expressway proposal. The expressway modelling required a more detailed breakdown of subcatchment hydrology adjacent to the expressway (so as to ensure flows through culverts were adequately modelled). In order to directly compare the expressway results with the existing case, the model hydrology for the existing case has been modified from that reported in REC (2011).

2.1 Model verification

The Waikanae model built previously for Greater Wellington was calibrated to the January 2005 flood event, which had a return period of around 80 years (1.25% AEP). That model was built using Lidar data collected in 2003. Calibration results were reported in Wallace (2010).

The current model uses more recent and more reliable Lidar data, collected in 2010. Although there will have been some changes to the berms between 2005 and 2010, meaning that the 2010 Lidar may not accurately represent the 2005 topography in some locations, overall the later Lidar are considered a better data set. The January 2005 flood event was run with the current model and found to give improved calibration results (Appendix B).

The small flood event of 8 January 2008 in the Waikanae River was also run with the model in order to check the model calibration. This event had an estimated return period of around 10 years (10% AEP). Limited anecdotal flood level information was provided by residents, with which to test the model (Table 1). Modelled sea levels for the event have been based on the forecast tide levels for that time, with a small adjustment to allow for barometric pressure effects.

Location	Estimated flood level	Model prediction
Otaihanga Boat Club	2.3	2.1
River end of Greenaway Rd	6	6.08
Adjacent to 266 Te Moana Rd	7	7.09

Table 1 8 January 2008 flood peak flood levels

Reports also indicated that water flowed across Makora Road near Ruru Road. The model did not predict this, but was only a few centimetres short of doing so.

To test the sensitivity of the model to higher sea levels, given the uncertainty of the amount of storm surge during the flood, a second simulation of the January 2008 flood was made with the sea level increased by 400 mm. Results showed almost no difference in levels upstream of the beach – levels at the Boating Club for example were only about 1 cm higher.

The model results indicate a good agreement with the limited 2008 observations upstream of the proposed expressway. Overall, the model also predicts the 2005 flood levels reasonably well, despite some isolated variations. Further discussion of the calibration results are given in REC (2010).

3 Expressway Features and Model

3.1 Footprint

The footprint of the expressway is shown in Figures 1 and 3. The expressway is modelled as being high enough to prevent any overflow. There are two overbridges in the model area, across Mazengarb Road and Otaihanga Road. In addition, a bridge over the Waikanae River is required and several culverts under the expressway are proposed (Figure 3).

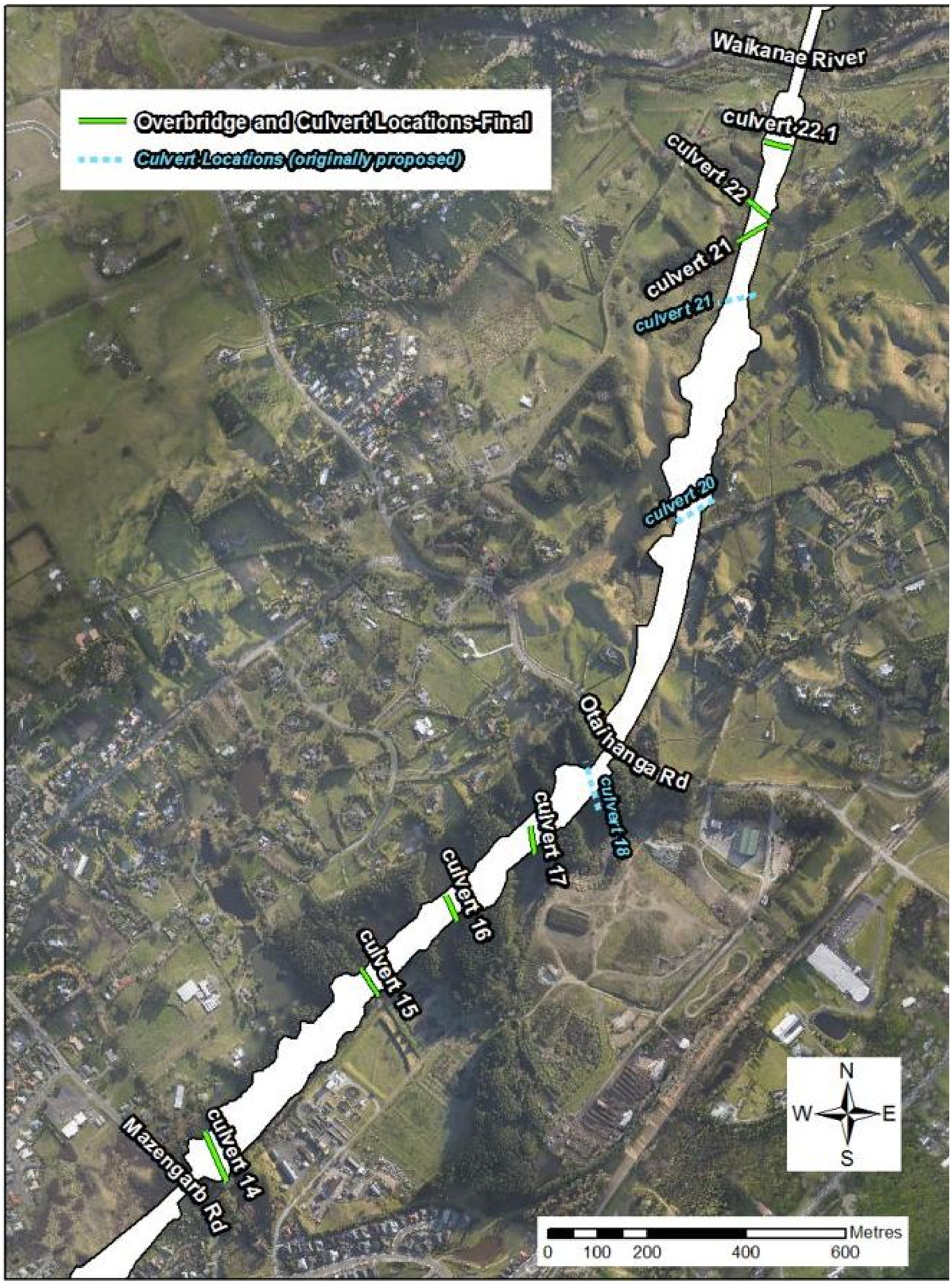


Figure 3 Expressway footprint, and proposed overbridge and culvert locations

3.2 Culverts

The design originally envisaged nine culverts between Mazengarb Road and the Waikanae River (Figure 3). After closer inspection of the topography, no need was seen for culverts 18 and 20 and they were removed from the model. (However it is understood that culvert 18 will remain in the works for ecological connectivity reasons.) Culvert 21 was also moved to discharge into the same drain as culvert 22.

A simulation was made with culvert 22 removed, but results suggested adverse effects. An alternative would be to combine culverts 22 and 21 in to a single culvert and to excavate a channel between their catchments. Although such an alternative has not been modelled, a single culvert of 1500 mm diameter and a 20 m long and 1.5m deep channel may suffice.

Table 2 shows the revised culvert schedule.

Reference	Distance	Diameter / Dimensions	Length	Roughness	Upstream IL	Downstream IL	Note
14	8000	5 m wide x3 m high	111	0.020	4.70	4.47	Mazengarb Drain
15	8500	1500 mm	60	0.013	5.96	5.80	WWTP Drain
16	8700	1200 mm	65	0.013	8.30	8.20	
17	8900	1200 mm	75	0.013	7.75	7.25	Landfill Drain.
18	9100						Not needed
20	9700						Not needed
21	10250	750 mm	55	0.013	3.75	3.70	Revised location
22	10300	750 mm	50	0.013	3.75	3.75	
22.1	10500	750 mm	65	0.013	3.20	3.15	

Table 2 Culvert schedule, Mazengarb Drain to Waikanae River (as modelled)

3.3 Waikanae River bridge

The design dated 22 June calls for four sets of twin piers, diamond shape in section and tapering from 1.5 m wide and 3 m deep at the base to 2.5 m wide x 5 m deep at the top.

As the bridge and piers are slightly skew (7°) to the river channel alignment, the model includes an adjustment to the bridge pier dimensions.

The design also includes a 35 m channel width in the vicinity of the bridge, a modified version of a GWRC design alignment, although this was not adopted until late in the modelling exercise. It is described further in Section 4.4 of this report. Results with this channel are also not presented until Section 4.4; prior to that, the results refer to an initial design assumption of a 35 m width only at the bridge itself and existing (2010) cross-sections elsewhere.

3.3.1 Debris

The bridge soffit is set clear of any potential debris blockage, nominally 2.2 m above early estimates of the 100 year design flood level.

A separate scenario with debris trapped against the piers has been modelled. Two methods of allowing for debris were investigated. The first is based on the Bridge Manual (Transit New Zealand, 2003), with a floating triangular debris raft at each pier. This gave an effective pier ratio (i.e. proportion of channel width blocked by the piers and debris) of 0.12.

The second method is based on work by GWRC (Wallace, 1991). The no-debris pier ratio is increased by 0.1 to allow for debris. The first bridge design supplied gave a pier ratio of 0.04 (no-debris case) + 0.1 = 0.14. This is similar to that obtained by the Bridge Manual approach, and considering the approximations in both estimating the pier ratio and in representing it in a model, the simpler GWRC method was been adopted.

For the refined bridge design supplied (22 June), the no-debris pier ratio is approximately 0.039. For debris simulation, the GWRC method gives a pier ratio of 0.139.

The modelled impact of the bridge piers on water levels, i.e. the difference between the bridge with and without piers has been checked by manual calculations. Results show the piers would increase levels in the 1% AEP event by approximately 4 cm (without debris) and by approximately 8 cm (with debris). Calculations are provided in Appendix C. (Note that these results and conclusions are based on the initial assumed river channel alignment, with a 35 m width at the bridge only and existing river cross-sections elsewhere. Nonetheless the conclusions are expected to be similar with the modified river channel alignment.)

3.4 Spur Removal

Flow is subject to a constriction just upstream of the bridge from a spur on the right (northern) bank. An option to cut back this spur has been modelled as a possible means to offset any adverse effects of bridge/expressway, i.e. to lower flood levels upstream. Results are presented in section 4.3.4.5 below. At this stage, it does not form part of the design however.

4 Scenarios

4.1 Design Flows and Scenarios

Hydrology inputs have been obtained from NIWA (2009) for the Waikanae and Muaupoko catchments, from SKM for the Mazengarb catchments (using a HEC-HMS model and 24 hour rainfall isohyets) and from the Alliance (for expressway catchments).

In the case of the Waikanae hydrographs, NIWA recommended a hydrograph shape based on the median shape of the largest six floods since the recorder at the Waikanae Water Treatment Plant was installed (1982). This shape has been used for the majority of the simulations. The exceptions are for the breach scenario investigations (see Appendix D), where a hydrograph 25% longer than the median (and longer than any of the six recorded flood hydrographs) has been used for some of the simulations.

The 0.04% AEP (1 in 2500 year) Waikanae River peak flow rate (590 m^3/s) has been estimated by extrapolating a Gumbel EV1 flood frequency relationship that fits through the values provided by NIWA for 2% AEP, 1% AEP and 0.5% AEP flows.

A storm event producing design flows is also likely to lead to elevated sea levels. Table 3 summarises the coincidences of river/stream flow and sea level events that were assumed in this study. (Nonetheless, the results around the expressway for each of the Mazengarb Drain and Waikanae River are largely independent of conditions in the other and of the sea level.)

Scenario description		AEP	Current Climate Peak Flow/Level	CC allowance	Modelled Peak Flow/Level
Mazengarb 10yr &	Waikanae	50%	150 m^3/s	0%	150 m^3/s
Waikanae 2yr	Mazengarb catchment	10%	(outputs at numerous subcatchments from HEC-HMS model)	16%	
	Muaupoko	5%	18.5 m^3/s	20%	22.2 m^3/s
	Sea Level	50%	1.85 m	0 mm	1.85 m
Mazengarb 100yr &	Waikanae	5%	300 m^3/s	20%	360 m^3/s
Waikanae 20yr	Mazengarb catchment	1%	(outputs at numerous subcatchments from HEC-HMS model)	16%	
	Muaupoko	5%	18.5 m^3/s	20%	22.2 m^3/s
	Sea Level	5%	2.1 m	800 mm	2.9 m
Mazengarb 1.5 x 100yr	Waikanae	5%	300 m^3/s	20%	360 m^3/s
Waikanae 20yr	Mazengarb catchment	< 1%	1.5 x 1% AEP flows for subcatchments	16%	
	Muaupoko	5%	18.5 m^3/s	20%	22.2 m^3/s
	Sea Level	5%	2.1 m	800 mm	2.9 m
Waikanae 100yr	Waikanae	1%	400 m^3/s	20%	480 m^3/s
	Mazengarb catchment	10%	(outputs at numerous subcatchments from HEC-HMS model)	16%	
	Muaupoko	20%	18.5 m^3/s	20%	22.2 m^3/s
	Sea Level	5%	2.1 m	800 mm	2.9 m
Waikanae Extreme	Waikanae	1%	400 m^3/s	50%	600 m^3/s
	Mazengarb catchment	10%	(outputs at numerous subcatchments from HEC-HMS model)	16%	
	Muaupoko	20%	18.5 m^3/s	50%	27.8 m^3/s
	Sea Level	5%	2.1 m	800 mm	2.9 m
Waikanae 2500yr	Waikanae	0.04%	590 m^3/s	20%	702 m^3/s
	Mazengarb catchment	1%	(outputs at numerous subcatchments from HEC-HMS model)	16%	
	Muaupoko	1%	24 m^3/s	20%	28.8 m^3/s
	Sea Level	5%	2.1 m	800 mm	2.9 m

Table 3 Design flow and sea level assumptions

4.1.2 Climate Change Allowance

The design practice of both GWRC and KCDC is to allow for a 16 % increase in rainfall intensity in 1% AEP storms for climate change, in accordance with the recommendations of the Ministry for the Environment (MfE, 2008). The hydrology for the Mazengarb catchment (using a HEC-HMS rainfall-runoff model) incorporates this adjustment. No rainfall-runoff model has been used for the Waikanae catchment, so the flow increase due to climate change has had to be assumed. Typically however, rainfall-runoff models predict that the flow increase resulting from a rainfall increase is slightly greater than the rainfall increase. NIWA has recommended a 20 % increase in flow for climate change for the Waikanae.

The 16% rainfall increase represents a mid-range estimate of climate change effects, and is based on a 2.1°C temperature increase to 2090. The MfE guidelines also note that the likely upper end of the range of temperature increase by 2090 is 5.2°C, which would increase 100 year rainfall depths by 41.6%. Accordingly, a scenario with the Waikanae River flows increased by 50% increase has also been modelled.

4.2 Waikanae River overflow scenario

A Waikanae River right bank overflow scenario (from either stopbank overtopping or a breach) needs to be modelled to determine the effect of the proposed expressway on such overflows. Of the conceivable scenarios analysed, a breach in the Chillingworth stopbank, about 900 m upstream of the Waikanae bridge site, would be the one generating the greatest overflows reaching the Waimeha/Te Moana Rd crossing (Appendix D).

A recommended overflow hydrograph to model is given in Appendix D. This hydrograph would result from a 60 m long breach occurring in the Chillingworth stopbank during a 0.5% AEP flood event in the Waikanae River (with a 25% longer hydrograph than the standard design). The overflow hydrograph peaks at 23 m^3/s.

4.3 Results

A summary of key discharges, velocities and flood levels is provided in Tables 4 - 6. More detailed tables of channel flood levels are given in Appendix E. Flood maps and graphs showing flood levels, flood depths and the impacts of the proposed M2PP expressway (compared to the existing, pre-expressway, situation) also follow.

Note that the flood maps do not show the flooding predicted in the Waikanae River and Mazengarb and WWTP Drain channels.

(As noted above, these results are for the initially assumed Waikanae River channel alignment , i.e. 35 m width only at the bridge itself and existing (2010) cross-sections elsewhere. Results for the subsequently adopted channel are presented in Section 4.4 below.)

Scenario	No debris	No debris	No debris	No debris	Debris	Debris	Debris	Debris
								Berm & SB raise
Mazengarb & culvert catchments	10% AEP	1% AEP	1.5 x (1% AEP)	10% AEP	10% AEP	10% AEP	1% AEP	1% AEP
Waikanae	50% AEP	20% AEP	20% AEP	1% AEP	1% AEP	1% AEP (50% incr CC)	0.04% AEP	0.04% AEP
Culvert 14	5.54	8.23	11.97	5.79	5.81	5.80	8.23	11.97
Culvert 15	0.51	0.88	1.53	0.59	0.59	0.60	0.88	1.53
Culvert 16	0.00	0.02	0.04	0.00	0.00	0.00	0.02	0.04
Culvert 17	0.36	0.72	1.11	0.36	0.36	0.36	0.72	1.11
Culvert 21	-0.19	-0.23	-0.37	-0.97	-0.97	-1.08	-1.13	-1.13
Culvert 22	-0.34	-0.53	-0.69	-1.09	-1.10	-1.23	-1.30	-1.31
Culvert 22.1	-0.34	-0.34	-0.34	-0.55	-0.56	-0.69	-0.77	-0.80
Waikanae Bridge	158	372	372	487	488	607	700	673

Table 4 Culvert and bridge peak discharges

(Debris/No Debris refer to the proposed Waikanae bridge. Berm raise and SB (stopbank) raise also refer to the Waikanae River.)

Scenario	No debris	No debris	No debris	No debris	Debris	Debris	Debris	Debris
								Berm & SB raise
Mazengarb & culvert catchments	10% AEP	1% AEP	1.5 x (1% AEP)	10% AEP	10% AEP	10% AEP		1% AEP
Waikanae	50% AEP	20% AEP	20% AEP	1% AEP	1% AEP	1% AEP (50% incr CC)	0.04% AEP	0.04% AEP
Culvert 14	0.52	0.68	0.89			0.52		
Culvert 15	1.59	1.54	1.55			1.65		
Culvert 16	0.22	0.66	0.85			0.22		
Culvert 17	1.49	1.83	2.09			1.49		
Culvert 21	-1.03	-1.12	-1.17			-2.64		
Culvert 22	-1.66	-1.92	-2.10			-2.97		
Culvert 22.1	-1.58	-1.65	-1.65			-2.13		
Waikanae Bridge (main channel)	1.32	1.64	1.64	1.73	1.83	1.95	2.12	2.28

Table 5 Peak velocities

Scenario		No debris	No debris	No debris	No debris	Debris	Debris	Debris	Debris	Debris
							Berm raise			Berm & SB raise
Mazengarb & culvert catchments		10% AEP	1% AEP	1.5 x (1% AEP)	10% AEP	10% AEP	10% AEP	10% AEP	1% AEP	1% AEP
Waikanae		50% AEP	20% AEP	20% AEP	1% AEP	1% AEP	1% AEP	1% AEP (50% incr. CC)	0.04% AEP	0.04% AEP
Culvert 14	Upstream	6.73	7.04	7.31	6.73	6.73	6.73	6.73	7.04	7.04
	Downstream	6.72	7.02	7.27	6.72	6.72	6.72	6.72	7.02	7.02
Culvert 15	Upstream	6.54	6.74	6.99	6.54	6.54	6.54	6.54	6.74	6.74
	Downstream	6.31	6.61	6.87	6.31	6.31	6.31	6.32	6.61	6.61
Culvert 16	Upstream	8.32	8.40	8.46	8.32	8.32	8.32	8.32	8.40	8.40
	Downstream	8.10	8.31	8.42	8.10	8.10	8.10	8.10	8.31	8.31
Culvert 17	Upstream	8.21	8.41	8.60	8.21	8.21	8.21	8.21	8.41	8.41
	Downstream	7.74	8.14	8.20	7.74	7.74	7.74	7.74	8.14	8.14
Culvert 21	Upstream	4.51	4.87	5.15	5.07	5.08	5.16	5.28	5.48	5.64
	Downstream	4.51	4.87	5.15	5.39	5.41	5.58	5.74	5.97	6.09
Culvert 22	Upstream	4.51	4.86	5.15	5.07	5.08	5.16	5.28	5.48	5.64
	Downstream	4.51	4.87	5.15	5.39	5.40	5.58	5.74	5.97	6.09
Culvert 22.1	Upstream	4.08	4.98	4.98	5.32	5.33	5.50	5.62	5.84	5.99
	Downstream	4.05	4.98	4.98	5.32	5.33	5.51	5.65	5.88	6.02
Waikanae Bridge Upstream		4.00	4.74	4.74	5.04	5.05	5.16	5.33	5.52	5.61

Table 6 Culvert peak flood levels upstream and downstream (no freeboard)

4.3.1 Mazengarb 10% AEP

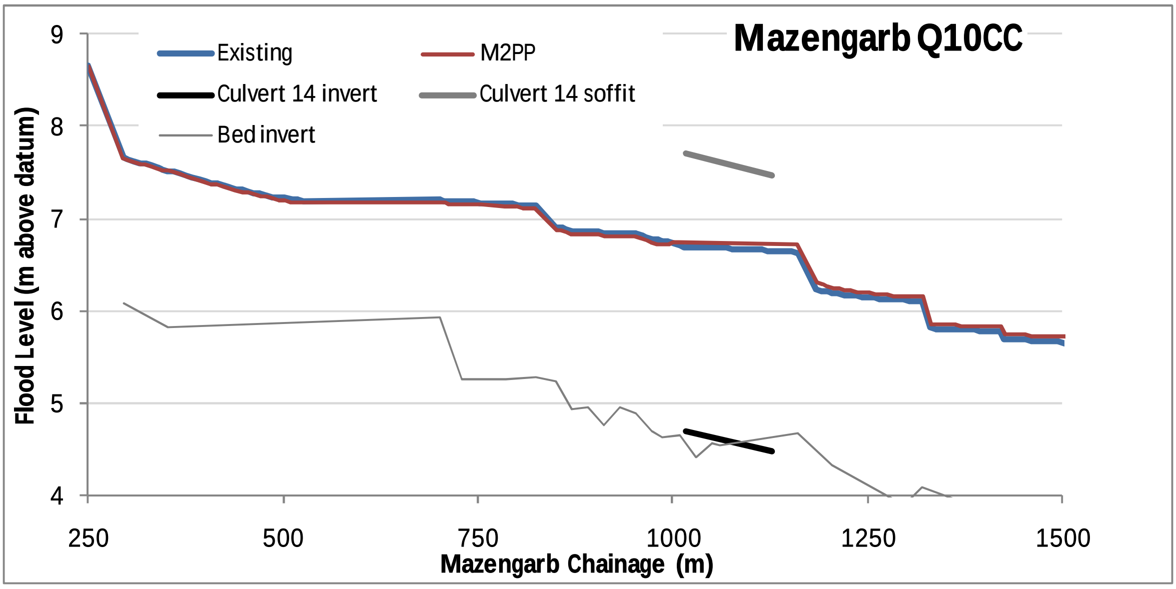


Figure 4 Mazengarb Drain profiles, Mazengarb 10% AEP scenario, M2PP proposal

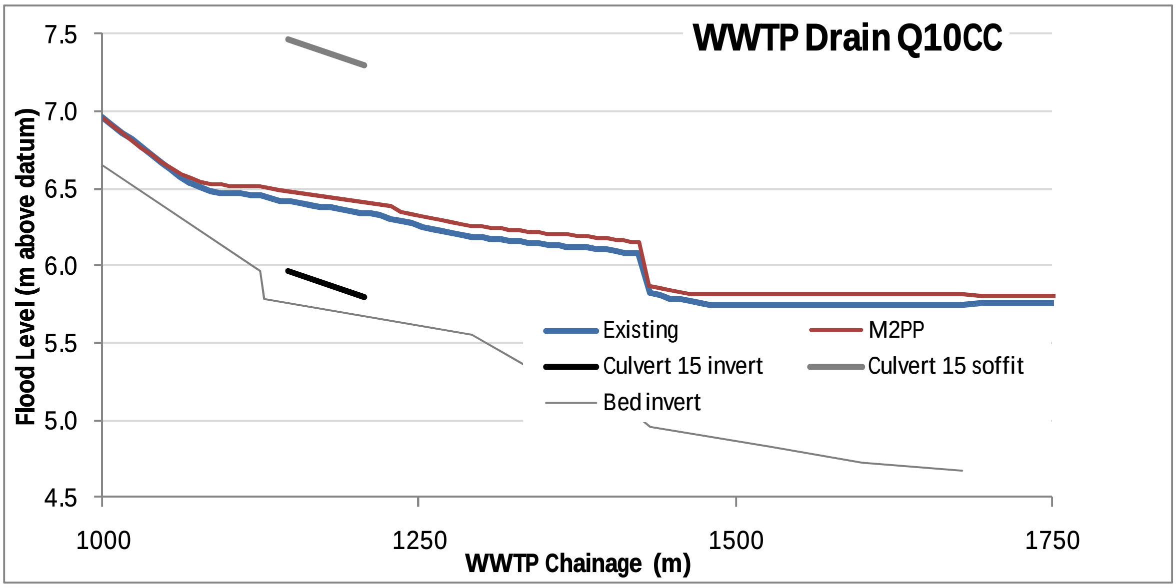
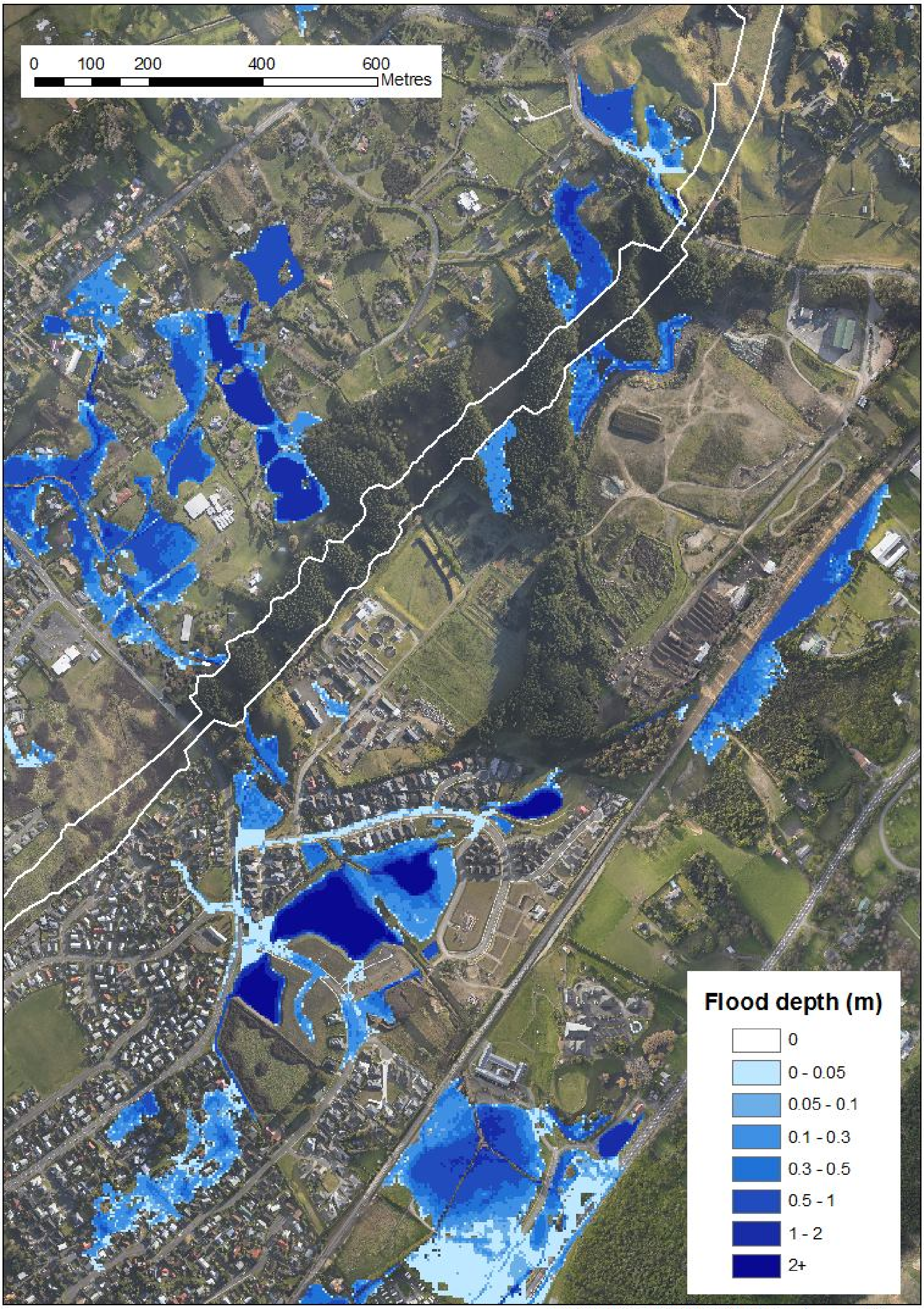


Figure 5 WWTP Drain profiles, Mazengarb 10% AEP scenario, M2PP proposal



Q10CC_MZ_WaiQ2CC_M2PP-Piers1-maxy.dfs2

Figure 6 Peak flood depths, Mazengarb 10% AEP scenario, M2PP proposal

4.3.2 Mazengarb 1% AEP

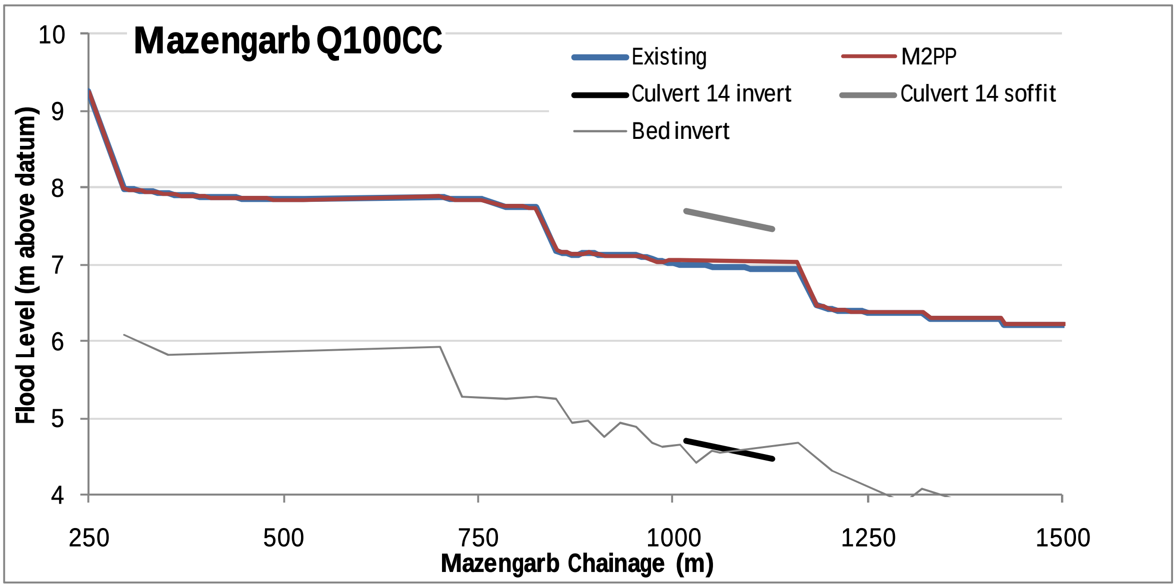


Figure 7 Peak flood depths, Mazengarb Drain, 1% AEP scenario, M2PP proposal

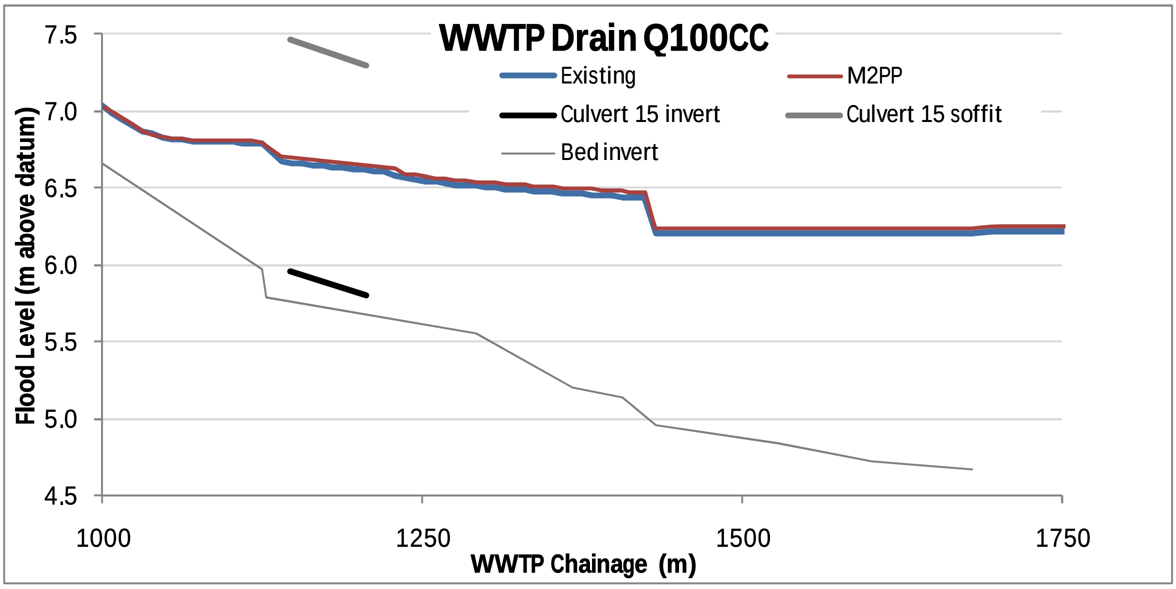
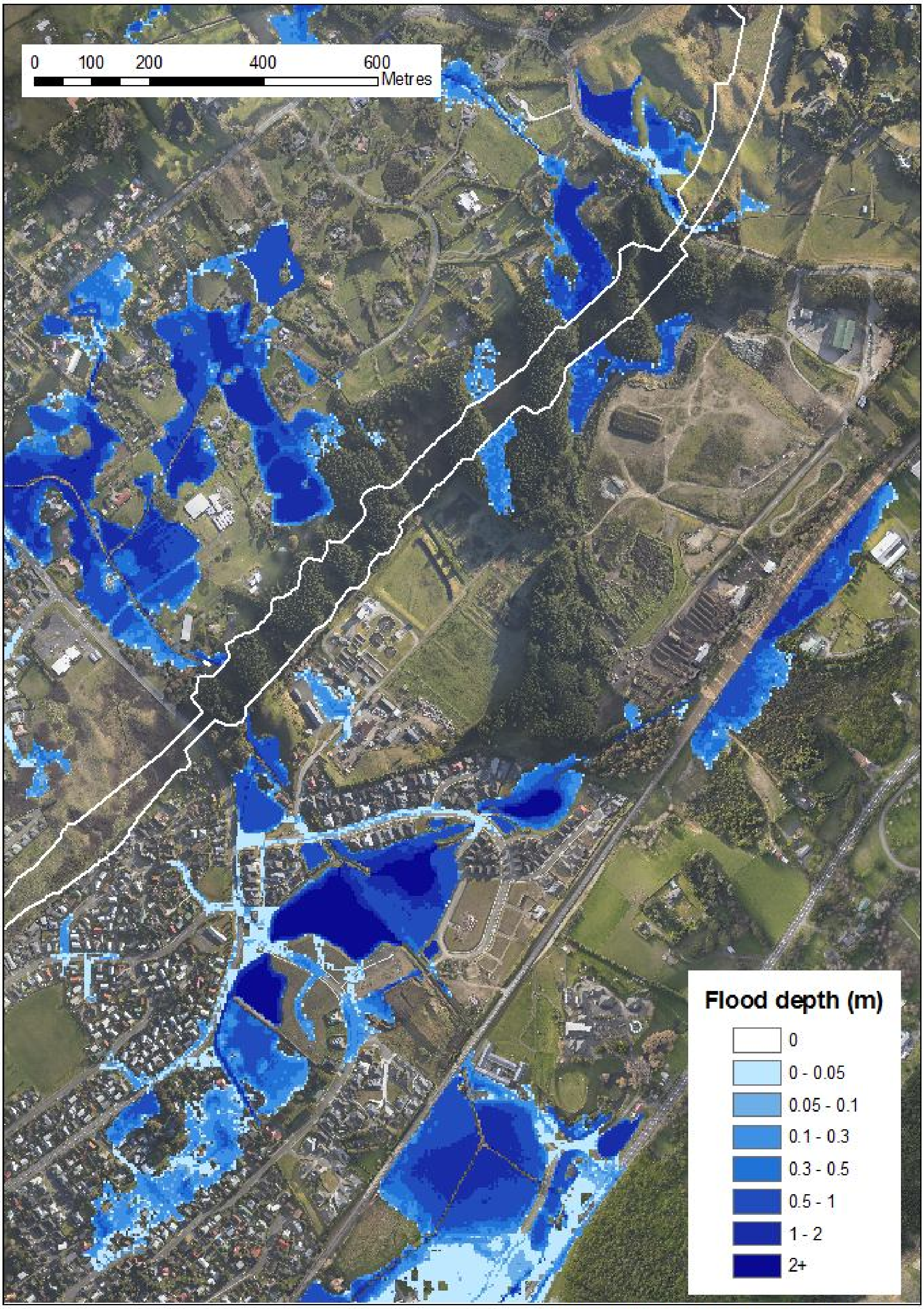


Figure 8 Peak flood depths, WWTP Drain, 1% AEP scenario, M2PP proposal



Q100CC_MZ_WaiQ20CC_M2PP-Piers1-maxy.dfs2

Figure 9 Peak flood depths, Mazengarb 1% AEP scenario, M2PP proposal

4.3.3 Mazengarb 1.5 x 1% AEP

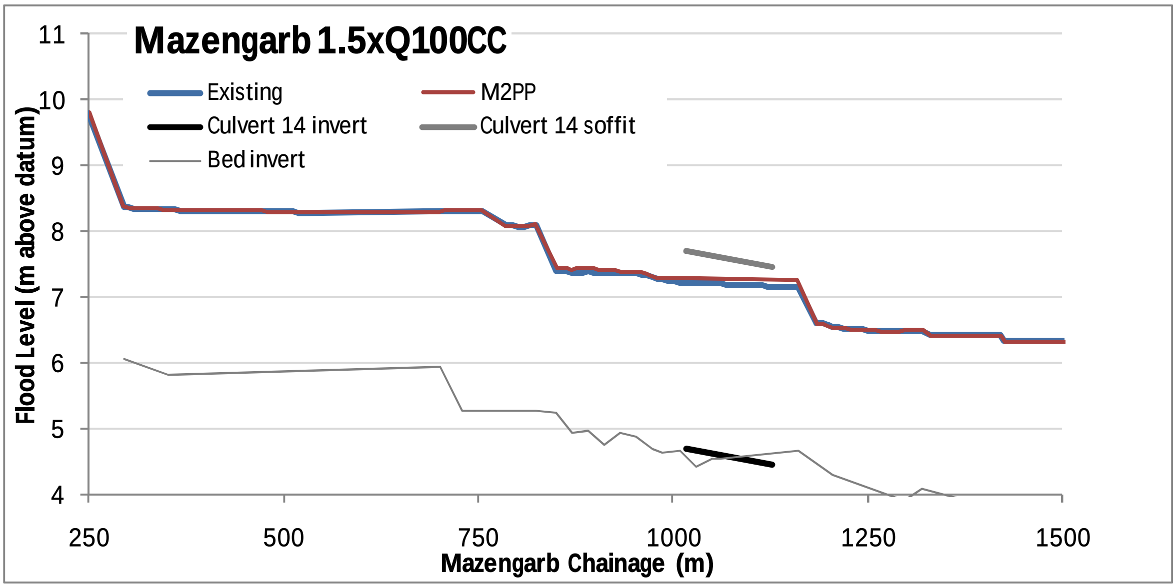


Figure 10 Peak flood depths, Mazengarb Drain, 1.5 x1% AEP scenario, M2PP proposal

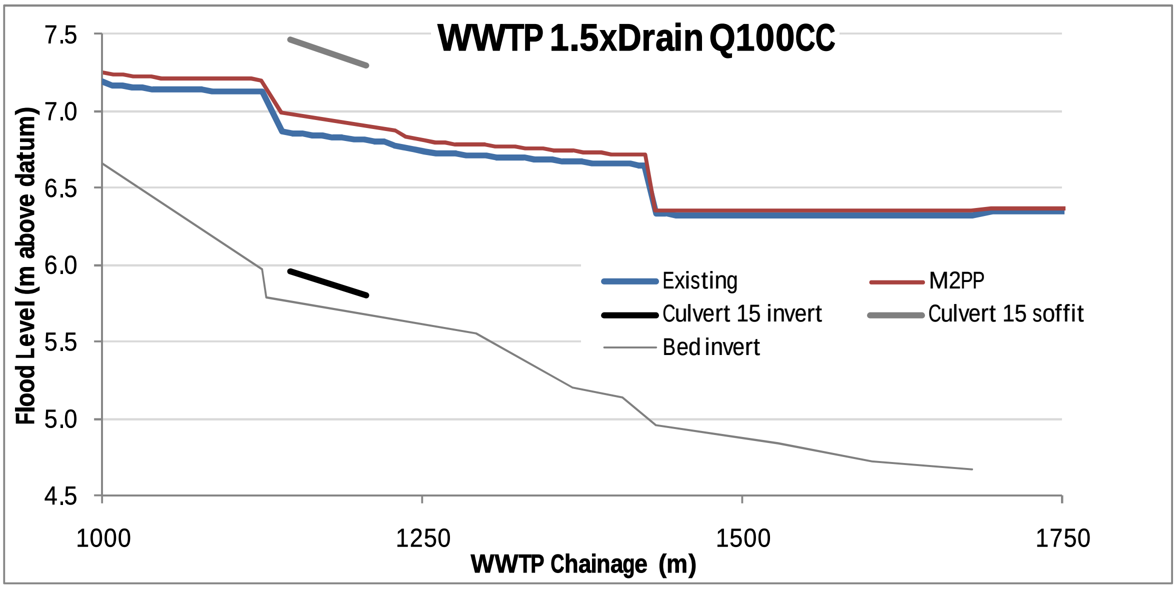
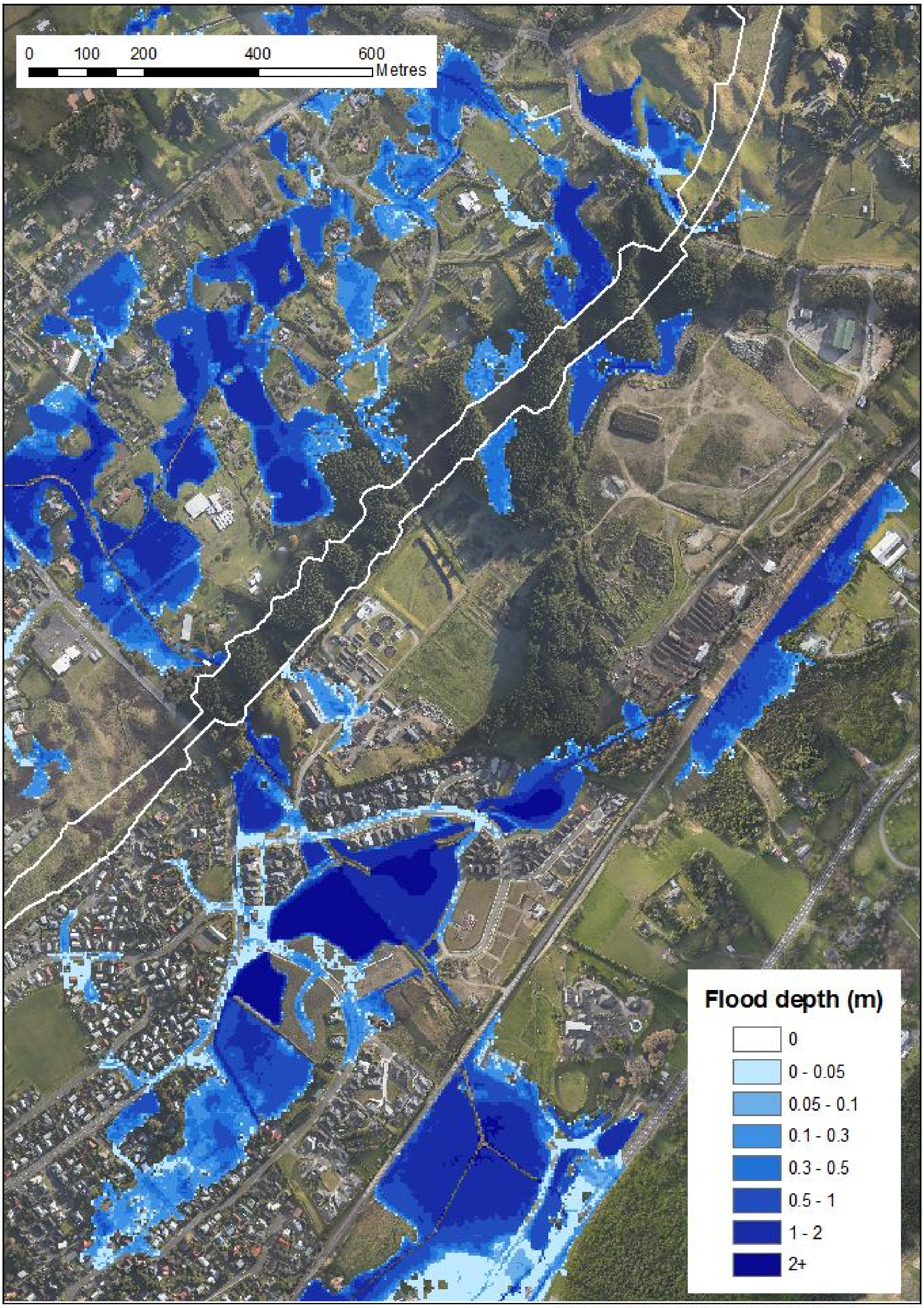


Figure 11 Peak flood depths, WWTP Drain, 1.5x1% AEP scenario, M2PP proposal



1.5xQ100CC_MZ_WaiQ20CC_M2PP-Piers1-maxy.dfs2

Figure 12 Peak flood depths, Mazengarb 1.5 x 1% AEP scenario, M2PP proposal

4.3.4 Waikanae 50% AEP (No debris on bridge)

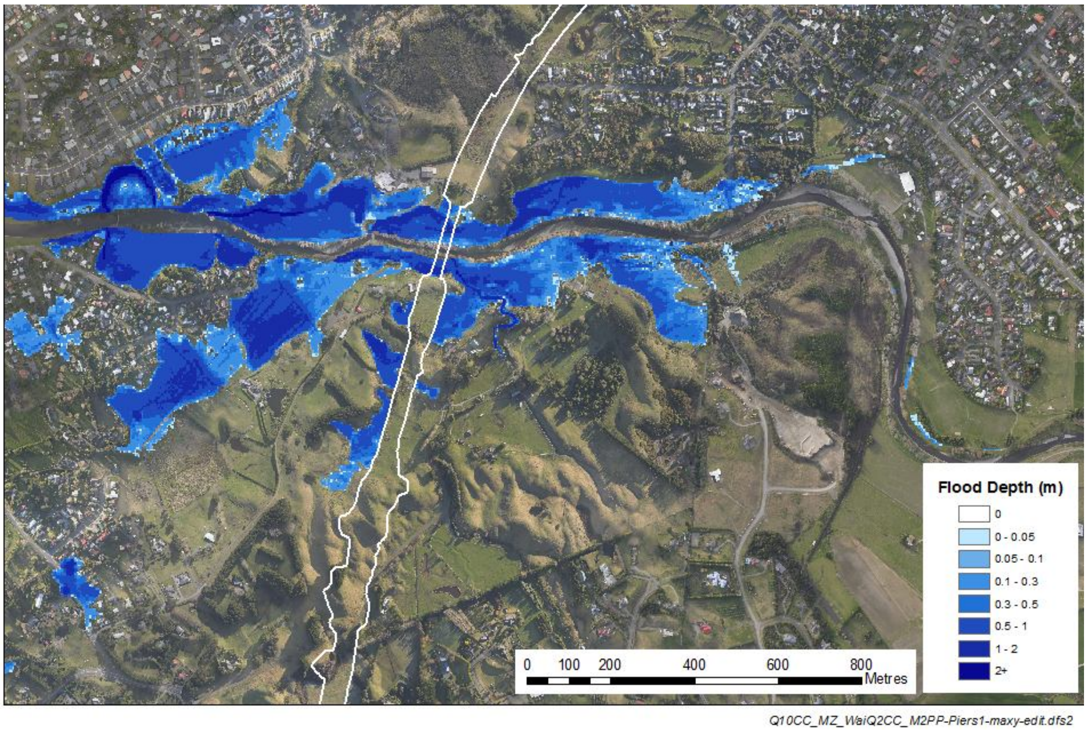


Figure 13 Peak flood depths, Waikanae 50% AEP scenario, M2PP proposal

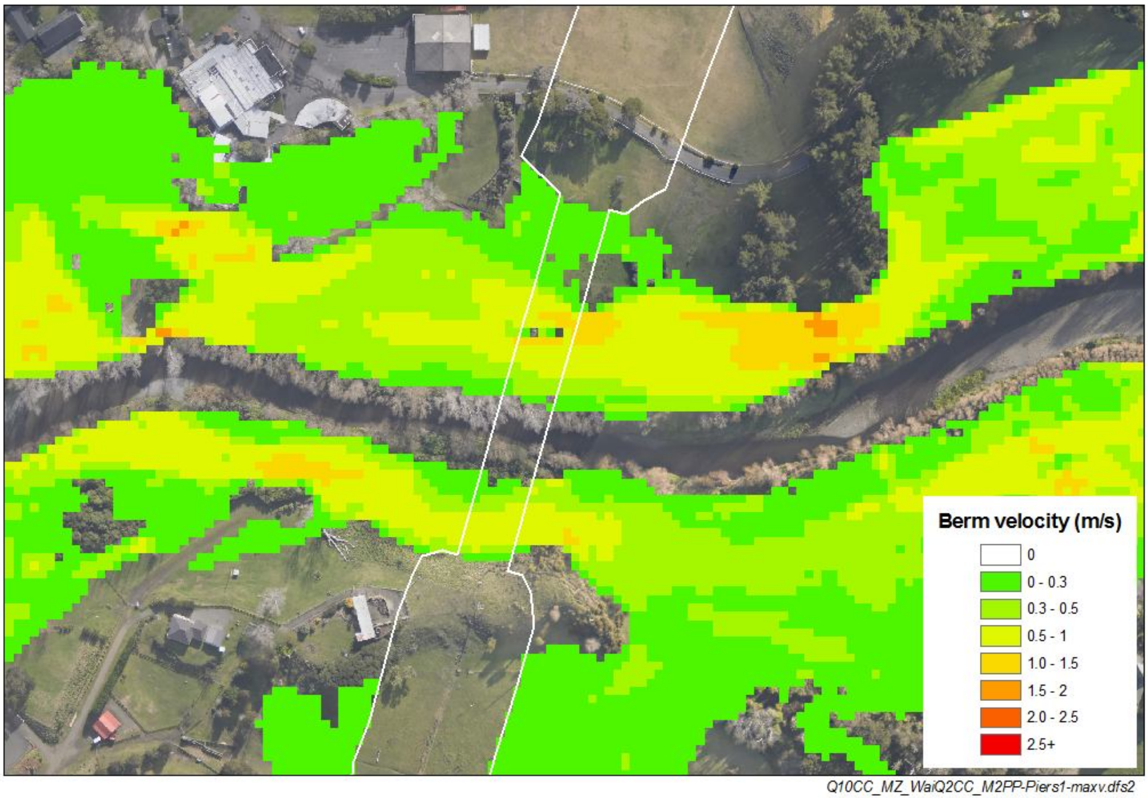


Figure 14 Peak berm velocities, Waikanae 50% AEP scenario, M2PP proposal

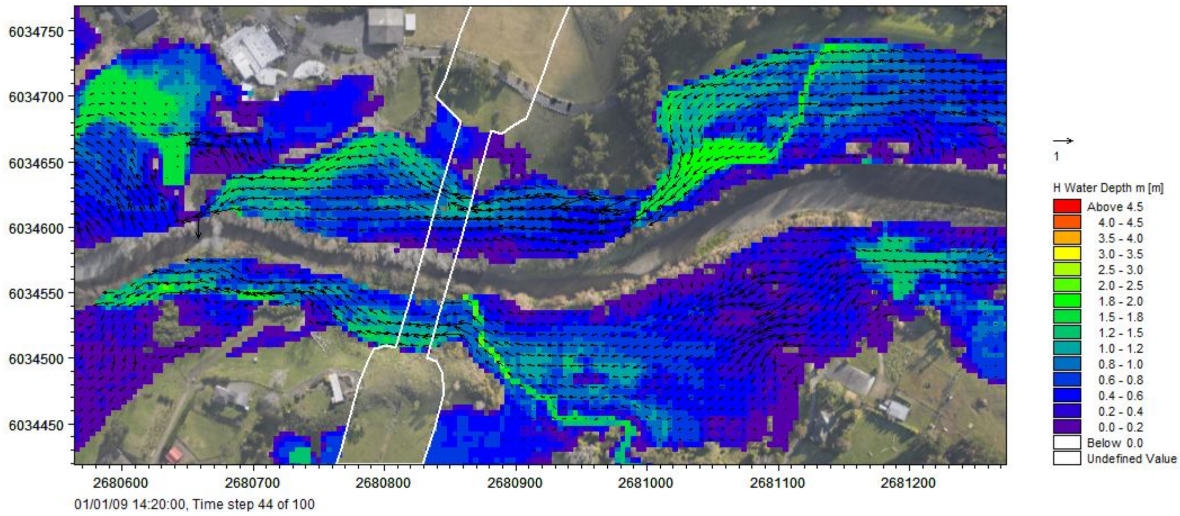


Figure 15 Berm depths and velocity vectors (near peak), Waikanae 50% AEP scenario, M2PP proposal

4.3.5 Waikanae 5% AEP (No debris)

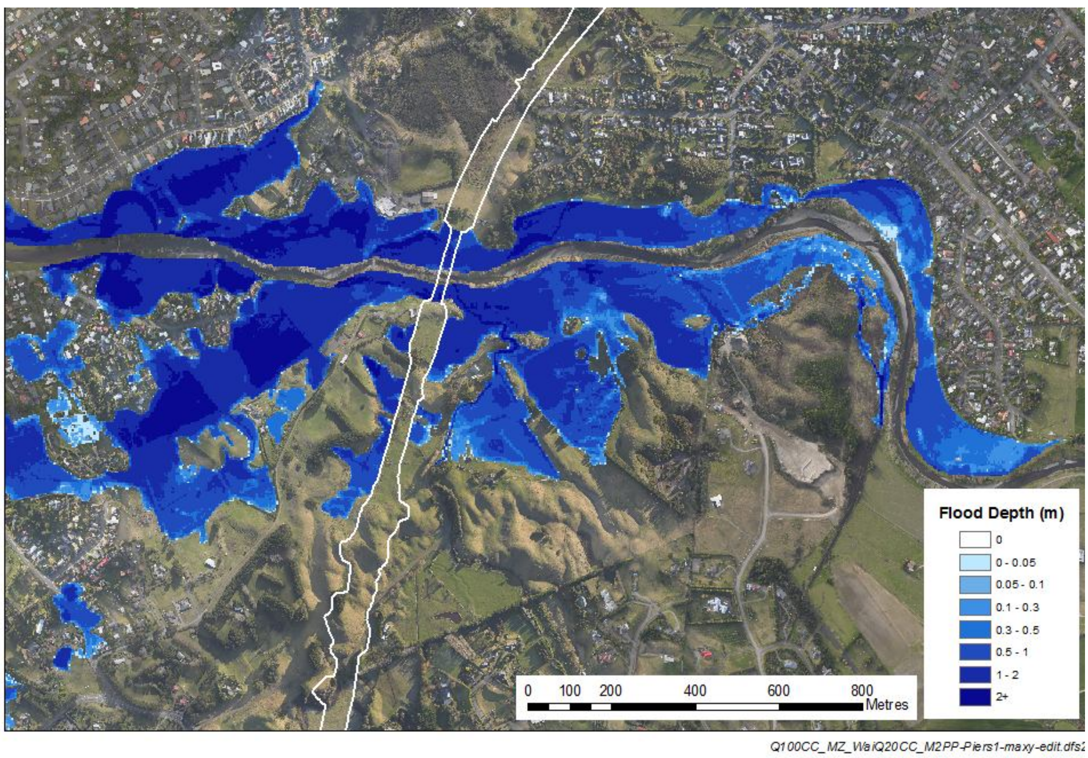


Figure 16 Peak flood depths, Waikanae 5% AEP scenario, M2PP proposal

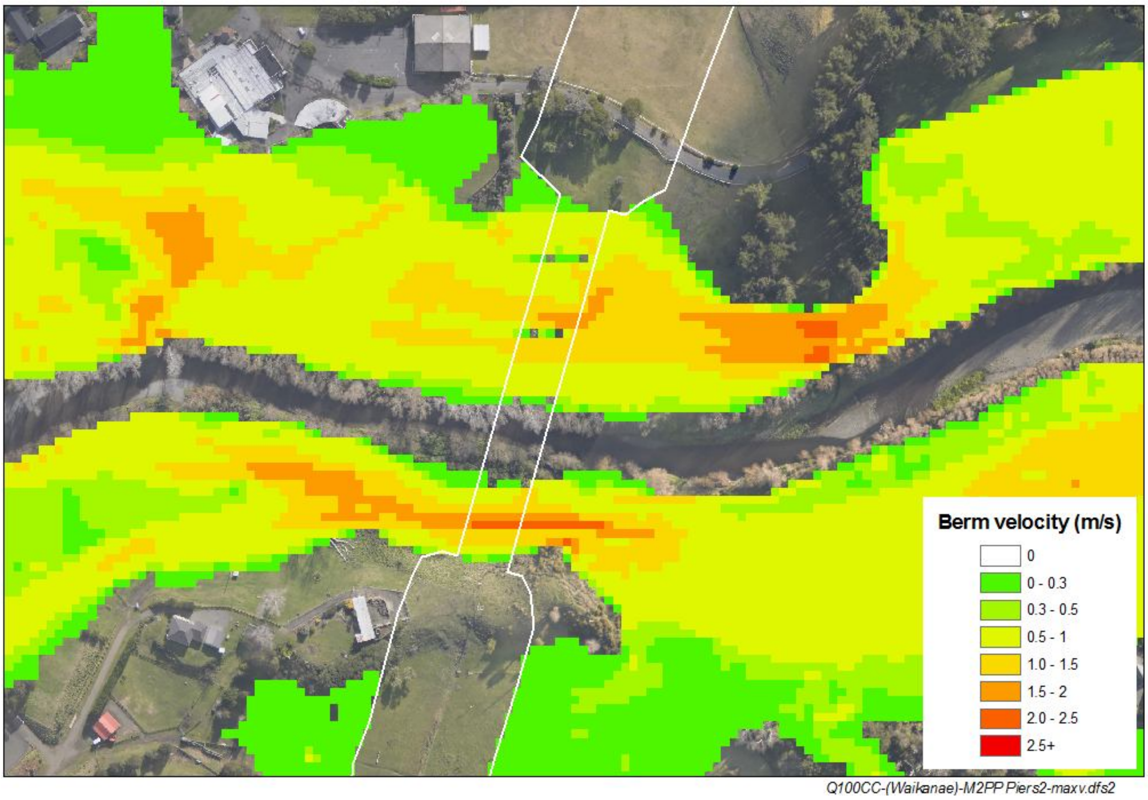


Figure 17 Peak berm velocities, Waikanae 5% AEP scenario, M2PP proposal

4.3.6 Waikanae 1% AEP

4.3.6.1 No debris

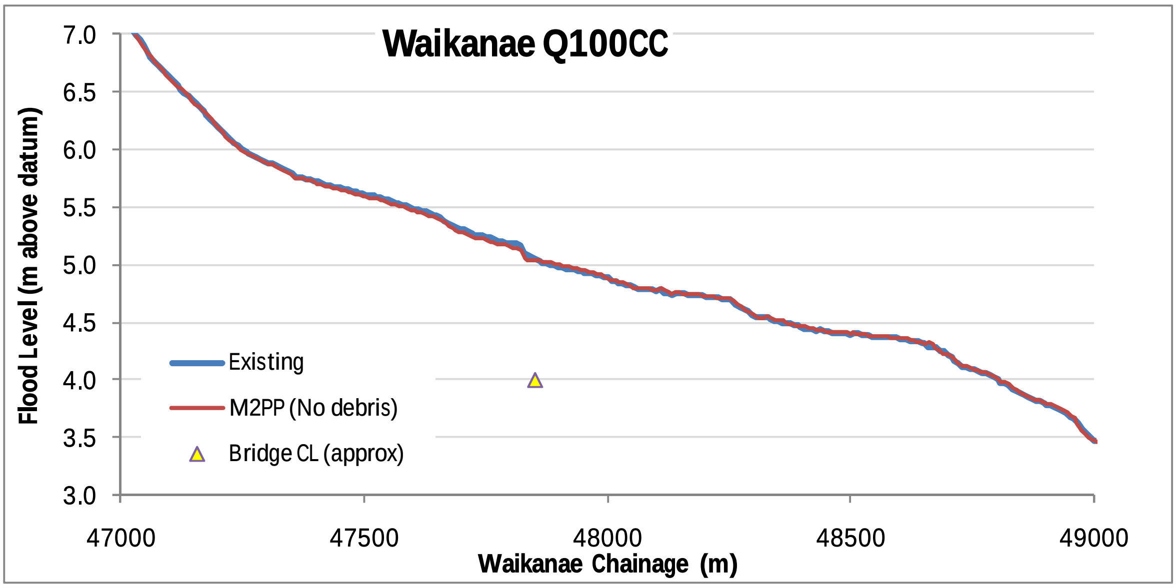


Figure 18 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (no debris on bridge)

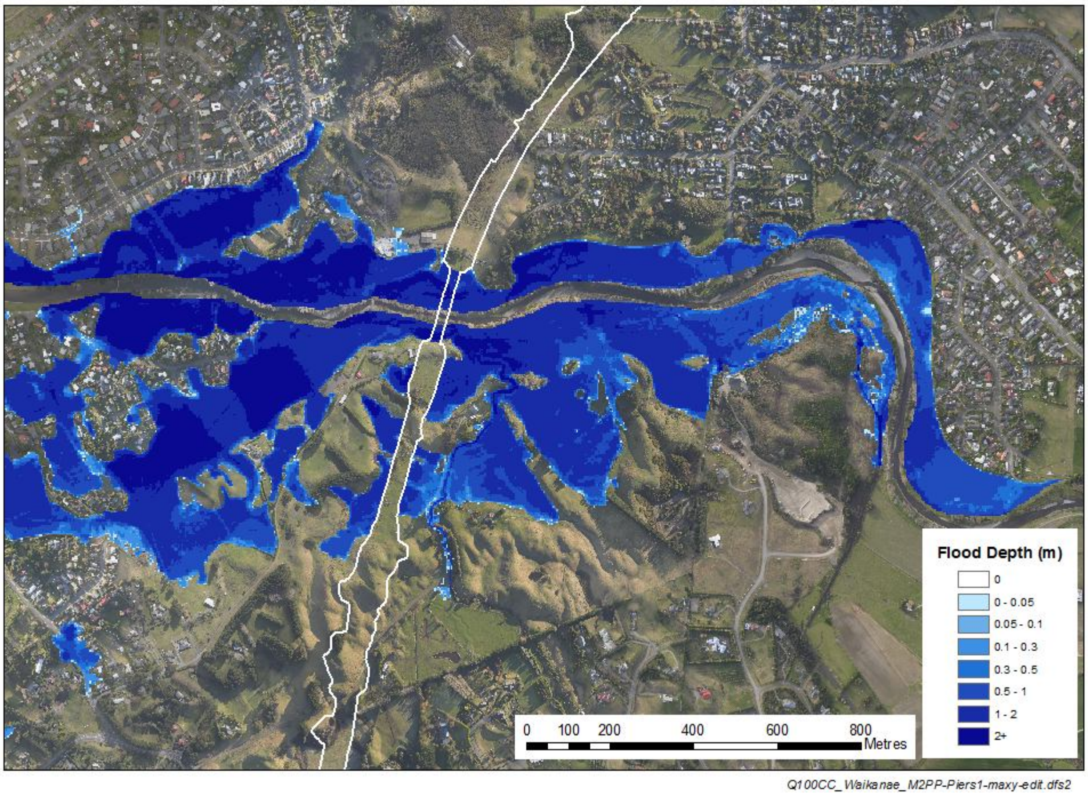


Figure 19 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (no debris on bridge)

4.3.6.2 Debris

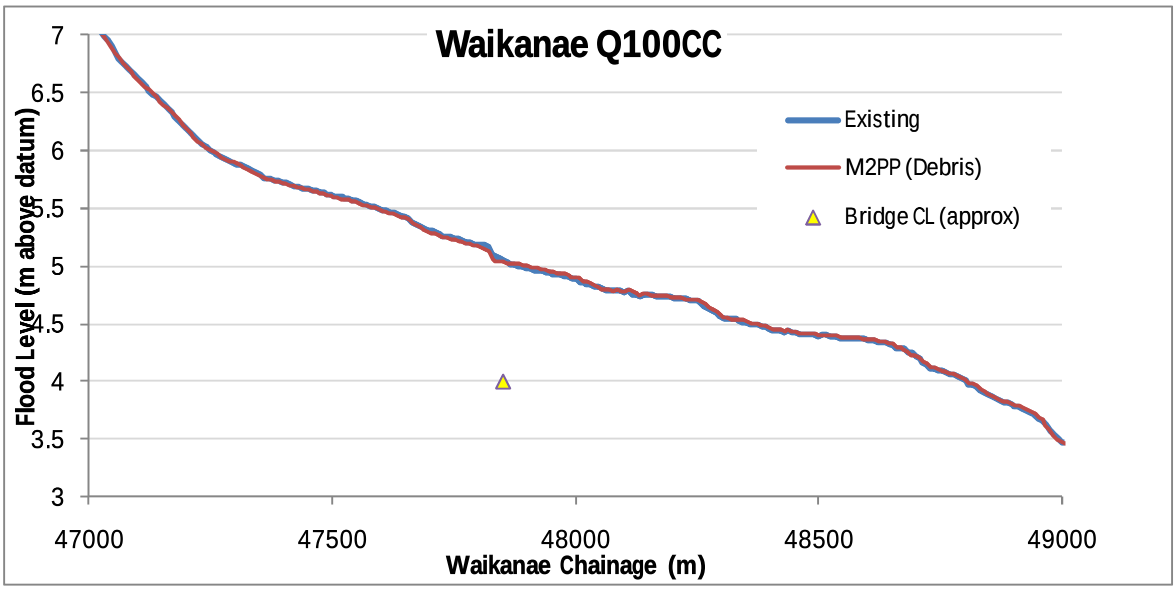


Figure 20 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (debris on bridge)

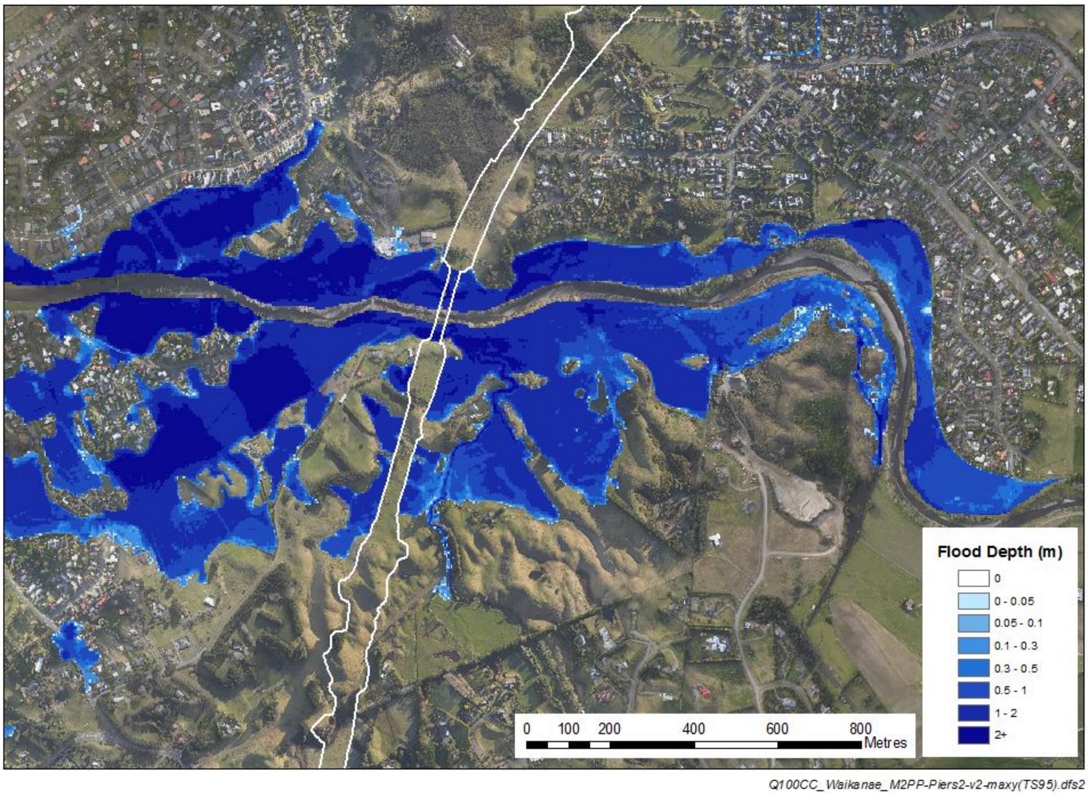


Figure 21 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (debris on bridge)

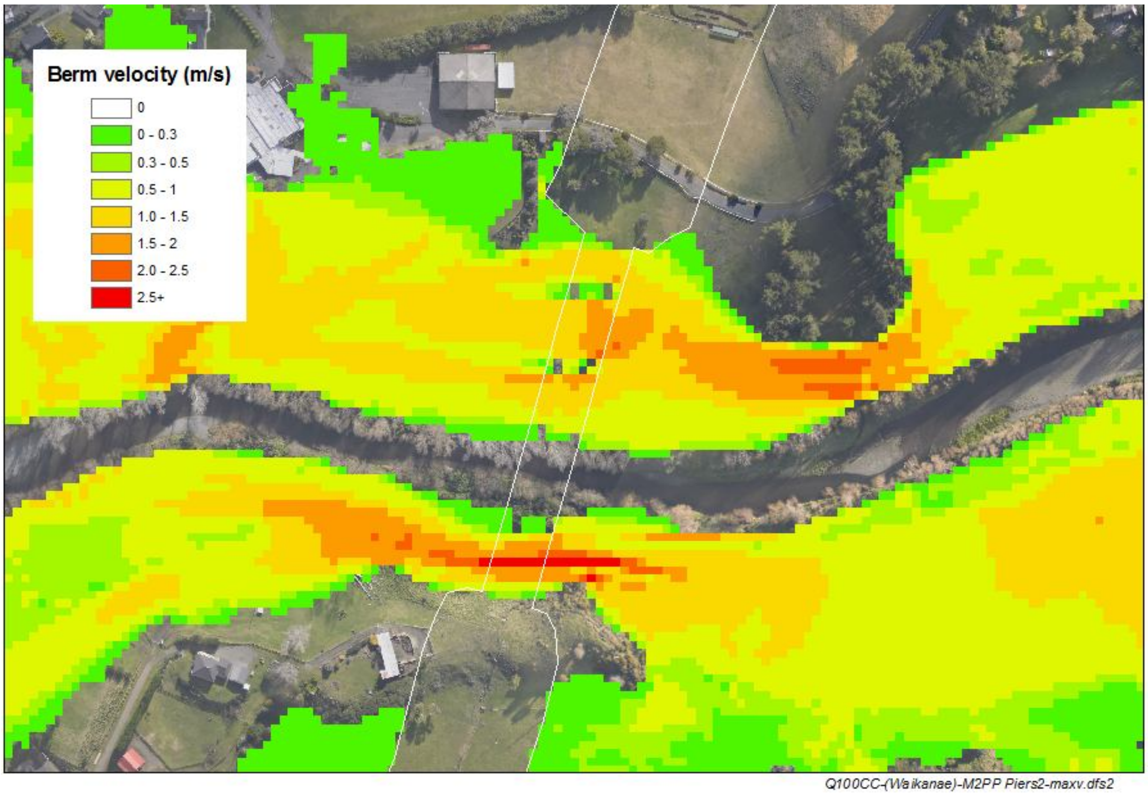


Figure 22 Peak berm velocities, Waikanae 1% AEP scenario, M2PP proposal (debris on bridge)

Note that predicted velocities are only marginally higher if a low tide is assumed.

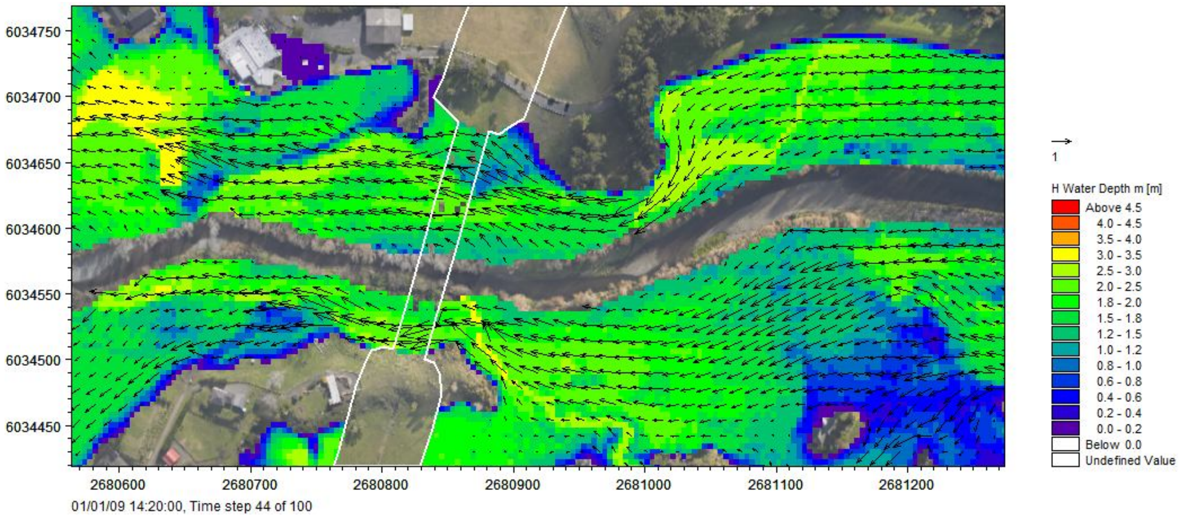


Figure 23 Berm depths and velocity vectors (near peak), Waikanae 1% AEP scenario, M2PP proposal (debris on bridge)

4.3.6.3 Debris, with existing channel

This scenario, with the existing channel under the bridge rather than a design 35 m wide channel, was run to enable comparison with the existing case (no expressway). It is for information purposes only and does not represent the proposed approach.

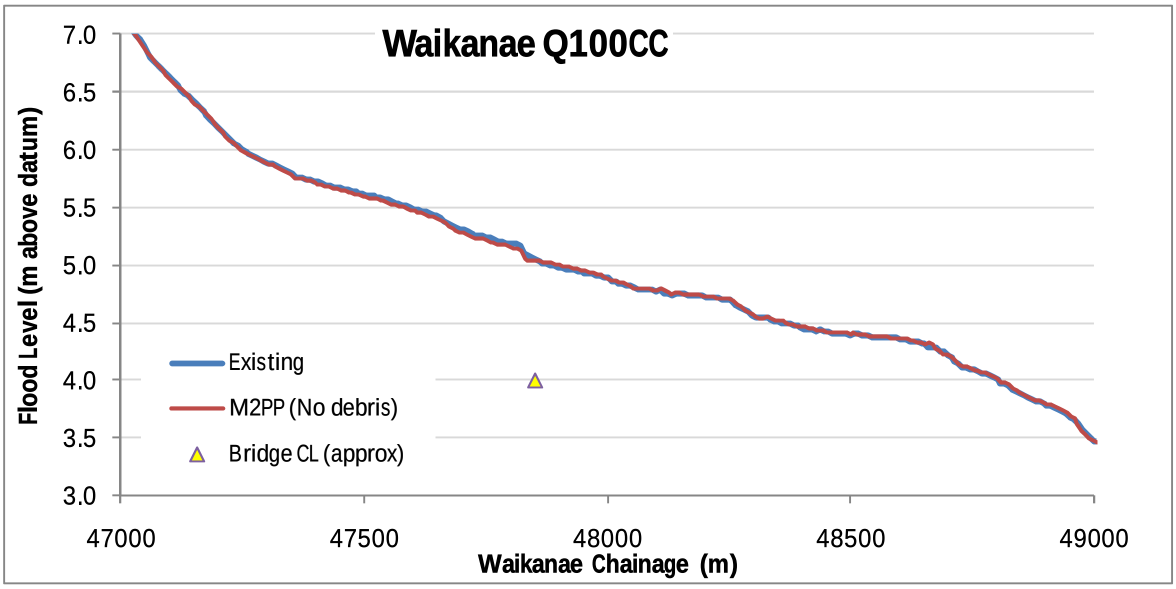


Figure 24 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (debris on bridge, existing channel)

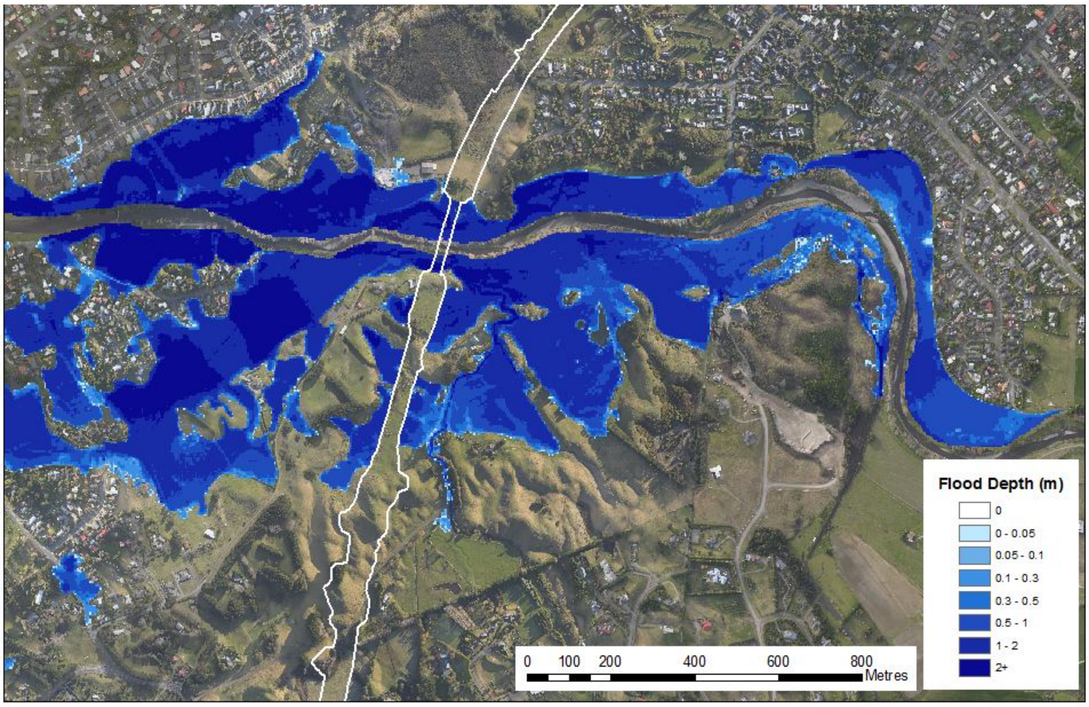


Figure 25 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (debris on bridge, existing channel)

4.3.6.4 Debris, 500mm aggradation on berms

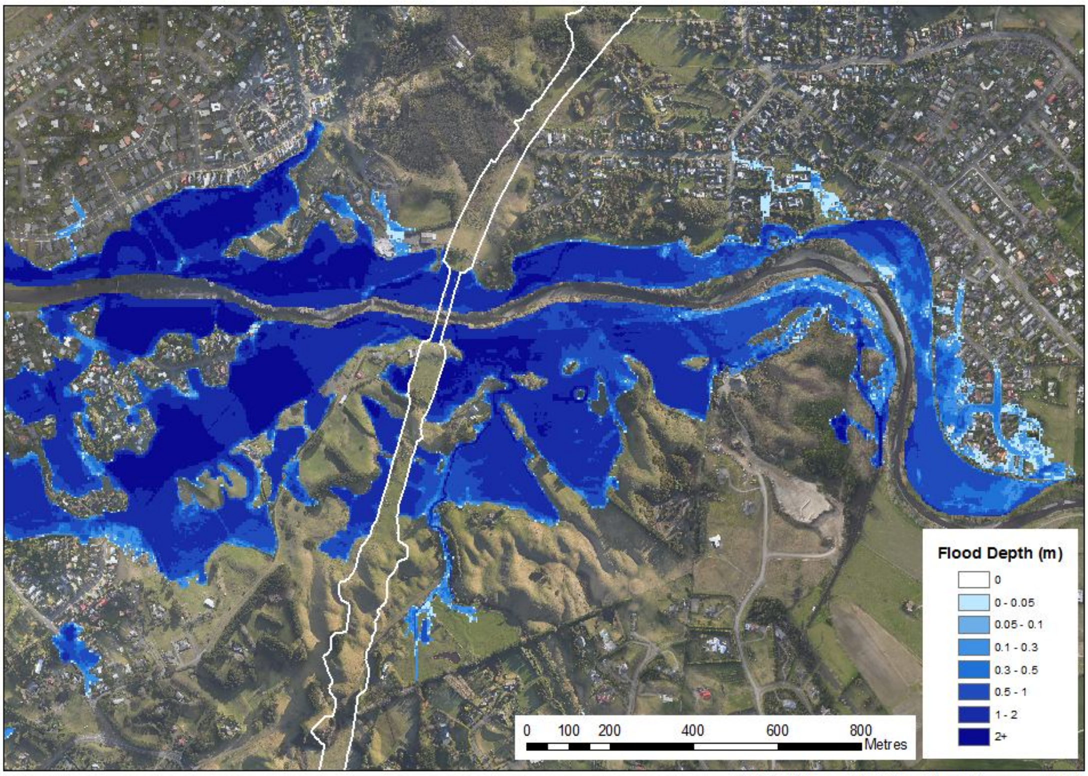


Figure 26 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (debris on bridge, berms with 500 mm aggradation)

4.3.6.5 Debris, Cut back of spur upstream of bridge

Removal of the spur is predicted to lower flood levels in the main channel for a distance of around 600 m upstream of the spur, by up to 70 mm in the 1% AEP scenario (Figure 27).

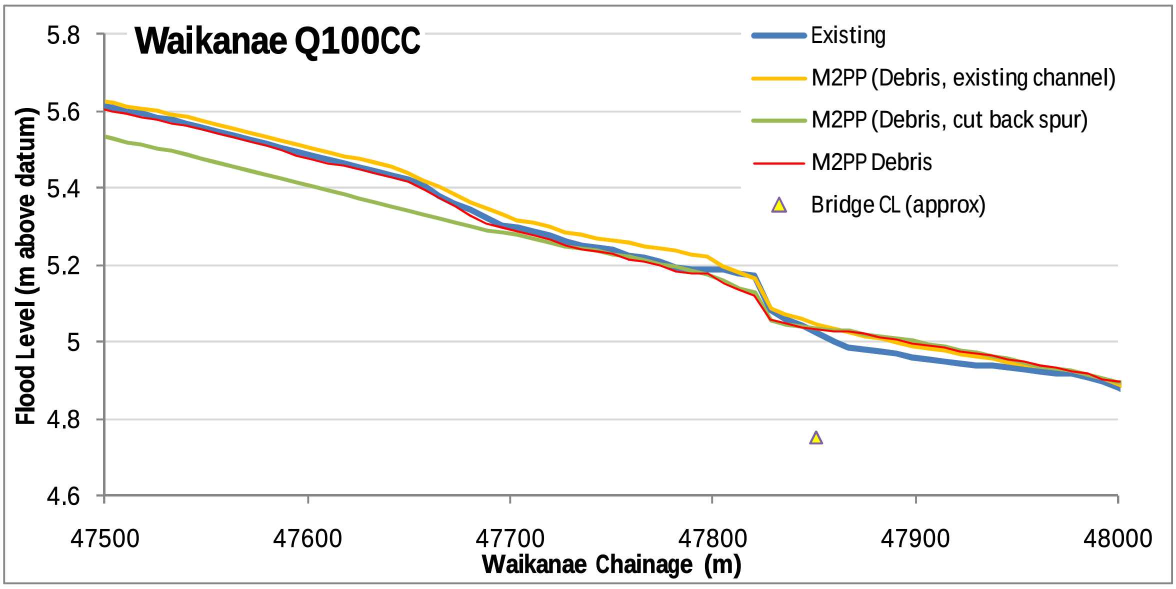


Figure 27 Comparison of flood levels for various cases, Waikanae 1% AEP scenario

4.3.7 Waikanae 1% AEP (50% increase for climate change) (Debris)

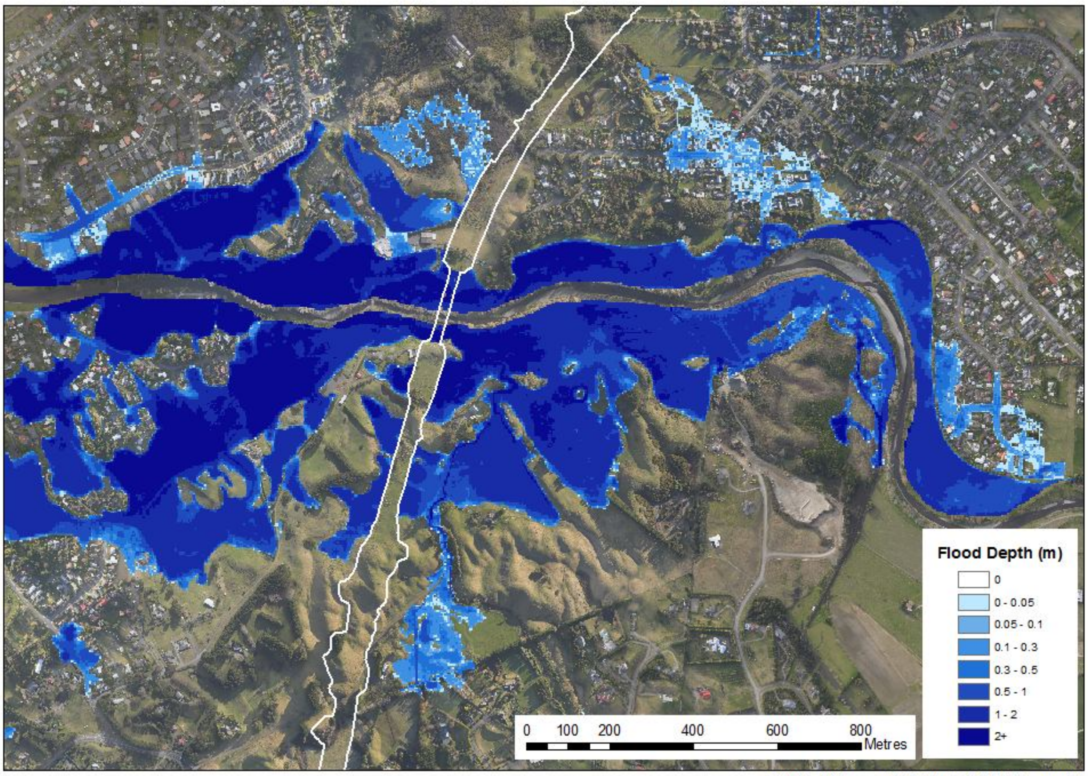


Figure 28 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (50% increase for climate change)

4.3.8 Waikanae 0.04% AEP (Debris)

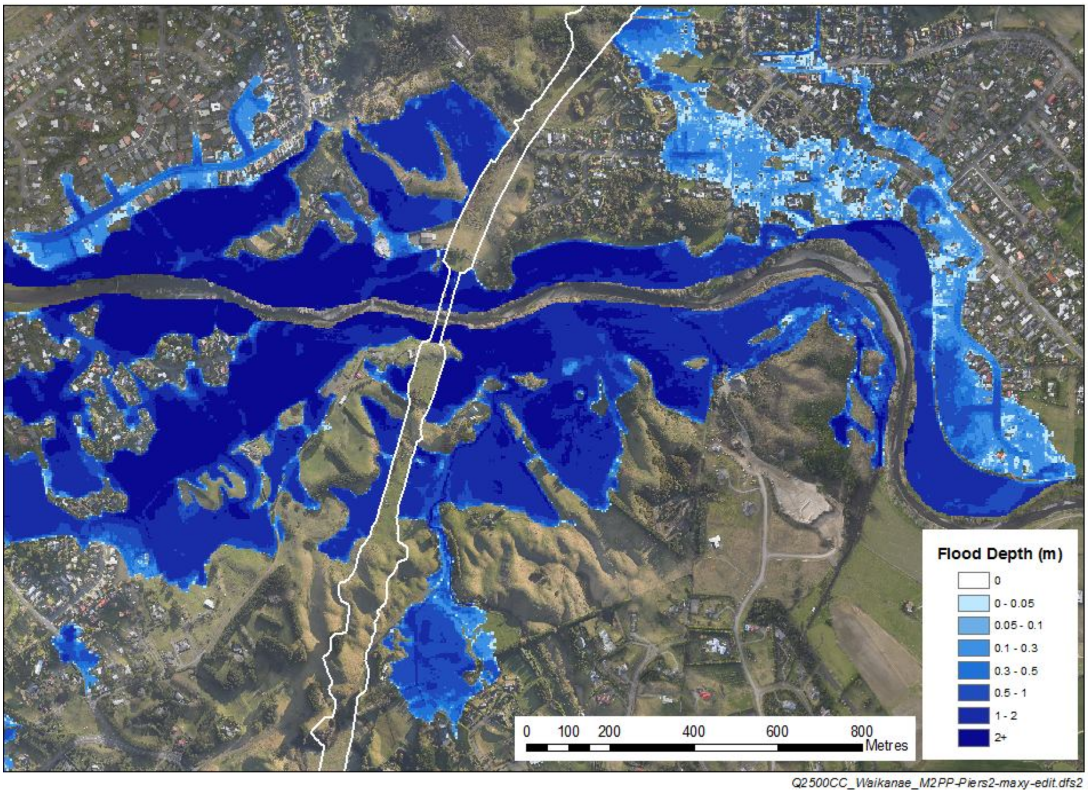
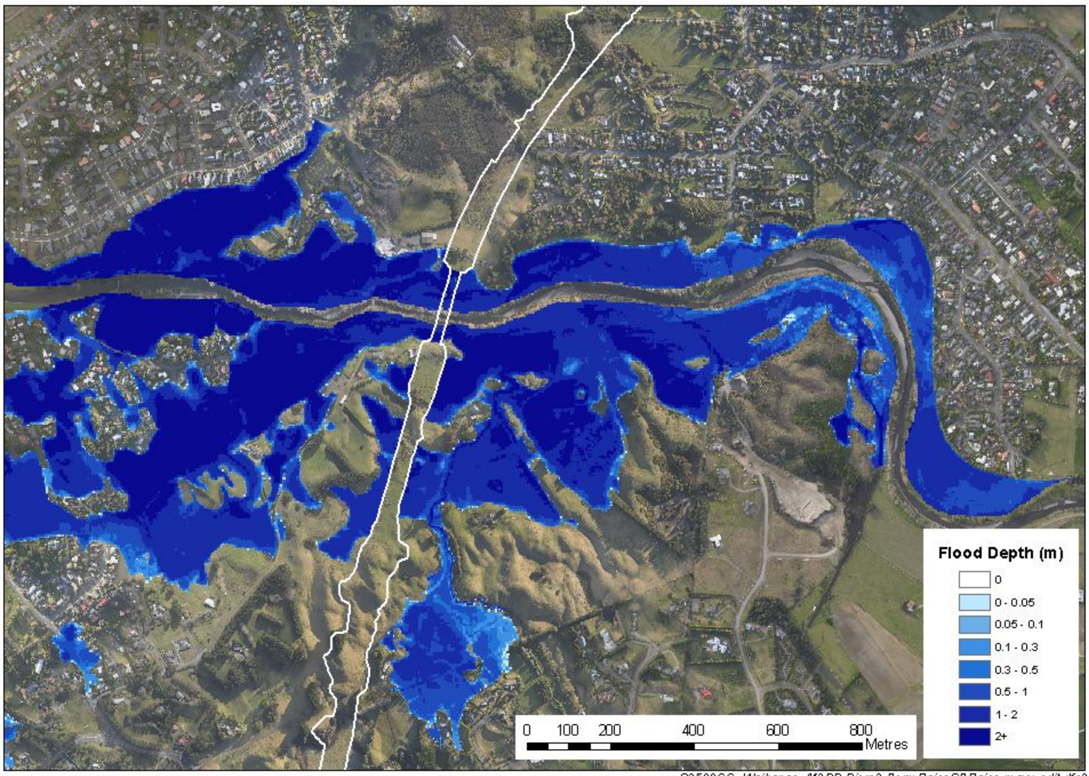


Figure 29 Peak flood depths, Waikanae 0.04% AEP scenario, M2PP proposal

4.3.9 Waikanae 0.04% AEP (Debris, berm and stopbank raised)

This scenario allows for 500 mm aggradation on the berms and assumes stopbanks are raised to prevent overflow in a 0.04% AEP event. New stopbanks or floodwalls to protect residential areas on the right bank downstream of the bridge are assumed.



Q2500CC_Waikanae_M2PP-Piers2-BermRaiseSBRaise-maxy-edit.dfs2

Figure 30 Peak flood depths, Waikanae 0.04% AEP scenario, M2PP proposal, (debris on bridge, berms with 500 mm aggradation, stopbanks raised)

4.4 Modified river alignment

Subsequent to the above modelling, and following a peer review of proposed river works, a revised channel alignment was agreed with GWRC. This alignment calls for a 35 m wide main channel between cross-sections 110 and 155 (Figure 31). (The modelling above was based on a 35 m wide channel only at the bridge, with the existing channel width elsewhere.)

The M2PP proposal was remodelled with this new alignment, for the 50% AEP and 1% AEP scenarios. Tables 7-9 provide summaries peak discharges, velocities and flood levels for these scenarios, with more detailed tables provided in Appendix E. Figures 32-35 show peak flood depths and velocities on the floodplain.

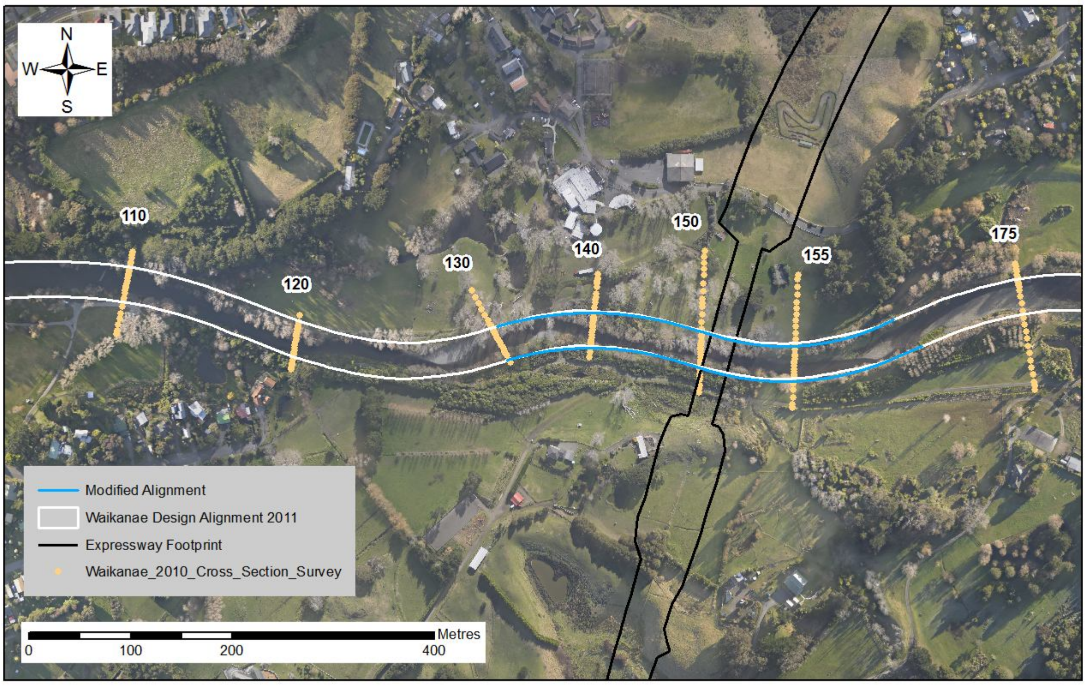


Figure 31 Modified main channel design alignment

Scenario	No debris	Debris
Mazengarb & culvert catchments	10% AEP	10% AEP
Waikanae	50% AEP	1% AEP
Culvert 21	-0.19	-0.93
Culvert 22	-0.34	-1.06
Culvert 22.1	-0.32	-0.42
Waikanae Bridge	158	487

Table 7 Culvert and bridge peak discharges, modified main channel alignment

Scenario	No debris	Debris
Mazengarb & culvert catchments	10% AEP	10% AEP
Waikanae	50% AEP	1% AEP
Culvert 21	-1.03	-2.11
Culvert 22	-1.66	-2.67
Culvert 22.1	-1.59	-1.76
Waikanae Bridge (main channel)	1.52	2.00

Table 8 Peak velocities, modified main channel alignment

Scenario		No debris	Debris
Mazengarb & culvert catchments		10% AEP	10% AEP
Waikanae		50% AEP	1% AEP
Culvert 21	Upstream	4.51	4.99
	Downstream	4.51	5.33
Culvert 22	Upstream	4.51	4.99
	Downstream	4.51	5.33
Culvert 22.1	Upstream	3.86	5.27
	Downstream	3.86	5.27
Waikanae Bridge Upstream		3.80	5.01

Table 9 Culvert peak flood levels upstream and downstream (no freeboard) modified main channel alignment

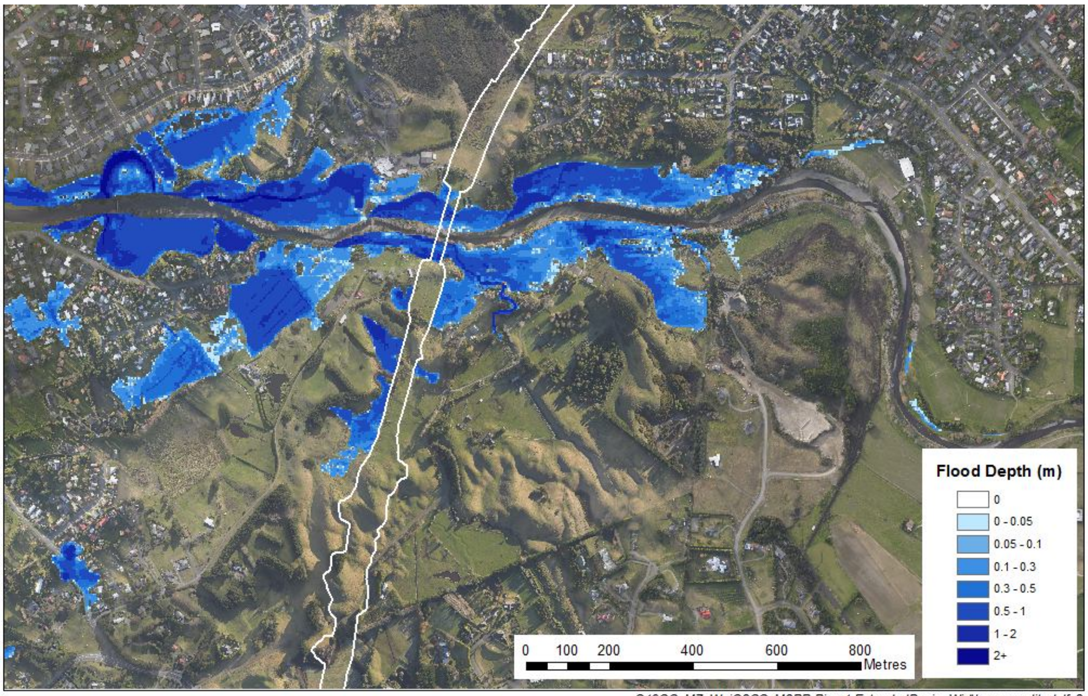


Figure 32 Peak flood depths, Waikanae 50% AEP scenario, M2PP proposal, modified main channel alignment

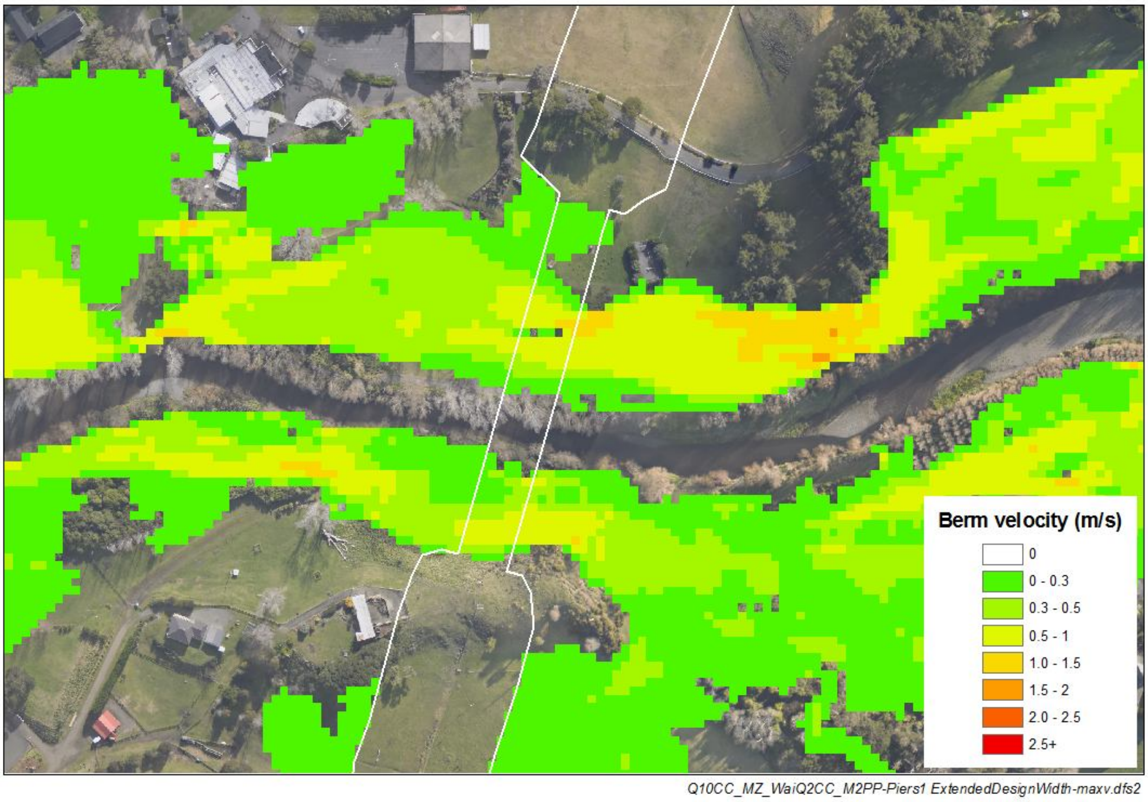


Figure 33 Peak berm velocities, Waikanae 50% AEP scenario, M2PP proposal, modified main channel alignment

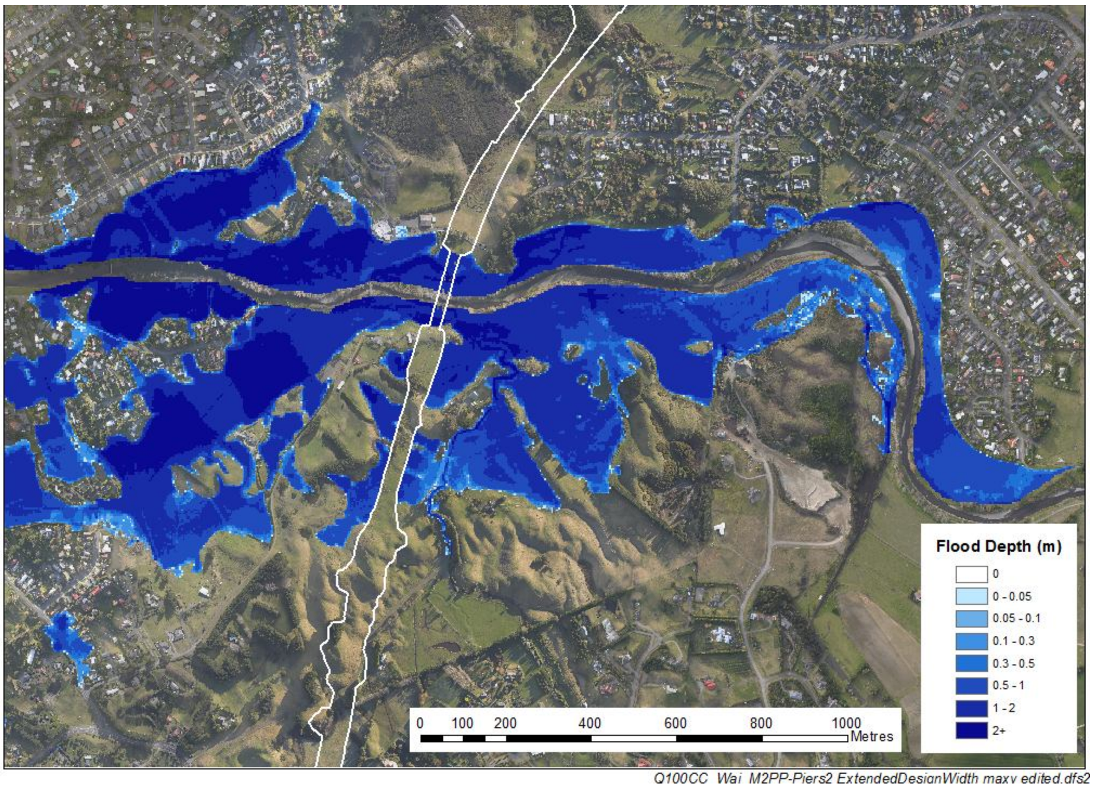


Figure 34 Peak flood depths, Waikanae 1% AEP scenario, M2PP proposal (debris on bridge), modified main channel alignment

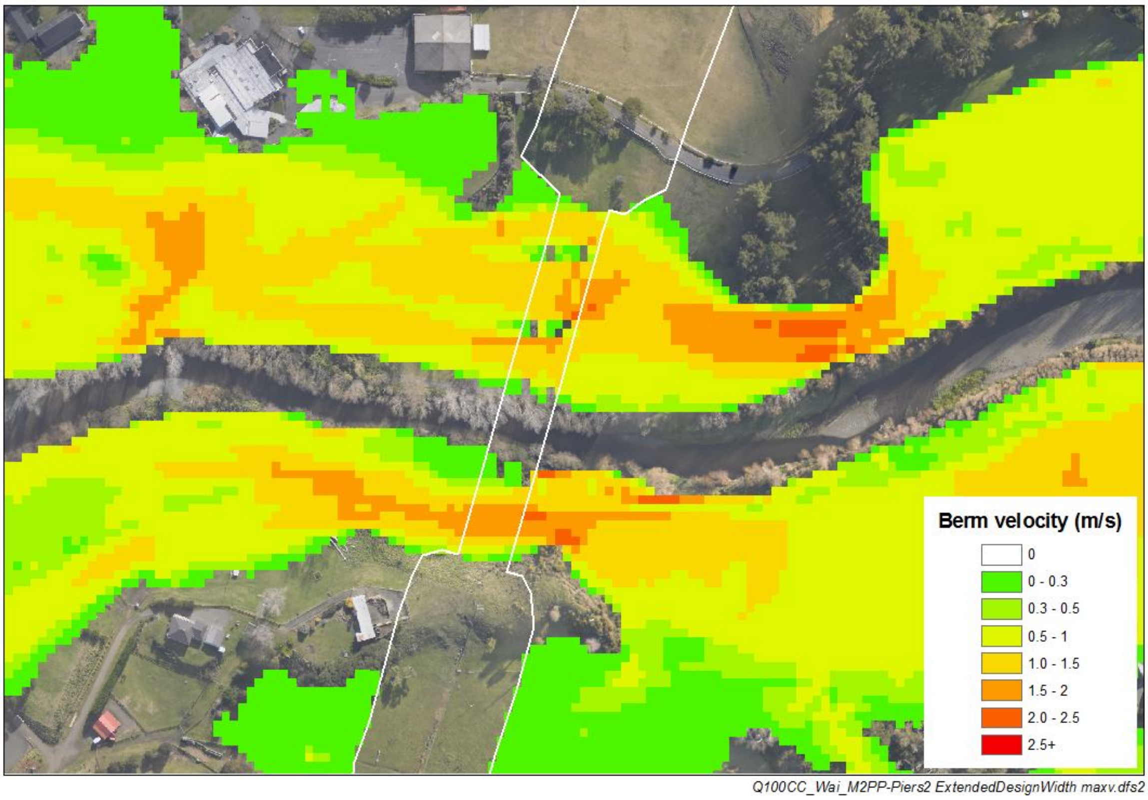


Figure 35 Peak berm velocities, Waikanae 1% AEP scenario, M2PP proposal (debris on bridge), modified main channel alignment

5 Discussion

Several points can be made about the results presented above:

- Culvert 14 (Mazengarb Drain) – Twin 1050 mm diameter culverts in the Mazengarb Drain, about 50 m downstream of the outlet location of culvert 14, constrict flow and raise flood levels in both the existing and proposed cases. Flood levels could be lowered by enlarging the culverts and regrading the channel.
- Culvert 15 (WWTP) – The increase in flood levels predicted in the WWTP Drain (Figure 8) can be accommodated by the drain in the reach around the expressway.
- Culvert 16 – It is doubtful if this culvert is needed for flood conveyance. However it is understood that it is needed for ecological reasons.
- Culvert 17 – This culvert removes a minor obstruction to flow, as well as having less roughness than the natural ground. This has the effect of increasing flow to the area downstream of the culvert, resulting in a slight increase in depths.
- Culvert 18 – This culvert is not needed for flood conveyance. However it is understood that it is needed for ecological reasons.
- Culverts 21, 22, 22.1 – Flow in these culverts is influenced more from backflow from Waikanae River and Muaupoko stream spillover than from their own catchments.
- Culvert inverts levels for all but culverts 14 and 15 are based on lidar data rather than a more accurate ground survey. Flow and level results at the culvert locations need to be reassessed once ground survey done around the proposed locations of these.
- Waikanae Bridge – The proposal shows slight benefits upstream of the bridge (compared to the existing situation), but these are due to the channel widening to 35 m in the vicinity of the bridge. If the existing channel is assumed for the expressway proposal,

the expressway is predicted to increase levels in the river and on the adjacent berms by up to 30 mm. Further up the Muaupoko, the increase is higher but still generally less than 50 mm.

- Removal of the right bank spur upstream of the proposed bridge would lower flood levels, if required, but would involve several other issues (e.g. consenting and cost issues).
- Freeboard has not been included in any of the tables and figures in this report. Within the Waikanae River (main channel and berms), GWRC uses 600 mm freeboard for the 1% AEP event, which would be appropriate for the modelling here also. A freeboard of 300 mm would be appropriate for the Mazengarb Drain and for the remainder of the floodplain.

References

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Transit New Zealand (2003); Bridge Manual, 2nd Edition 2003 (Includes amendments June 2004, September 2004, and July 2005).

Wellington Regional Council (1997); Waikanae River Floodplain Management Plan: Phase Four - Numerical Hydraulic Modelling of Extreme Flows. Report WRC/RI-T-97/28

Appendix A Model Files

Model input and output files can be tracked via the following .couple files.

Description	couple file
January 2005 flood	Jan05-rerun
January 2008 flood	Jan08
January 2008 flood, assumed 400mm storm surge	Jan08-400SS
10% AEP Mazengarb & 50% AEP Waikanae flows, existing situation	Q10CC MZ existing
1% AEP Mazengarb & 5% AEP Waikanae flows, existing situation	Q100CC (MZ) existing
1.5x1% AEP Mazengarb & 5% AEP Waikanae flows, existing situation	1.5xQ100CC (MZ) existing
1% AEP Waikanae & 10% AEP Mazengarb flows, existing situation	Q100CC Waikanae existing
10% AEP Mazengarb & 50% AEP Waikanae flows, M2PP proposal	Q10CC (MZ) M2PP Piers1
1% AEP Mazengarb & 5% AEP Waikanae flows, M2PP proposal	Q100CC (MZ) M2PP Piers1
1.5x1% AEP Mazengarb & 5% AEP Waikanae flows, M2PP proposal	1.5xQ100CC (MZ) M2PP Piers1
1% AEP Waikanae & 10% AEP Mazengarb flows, M2PP proposal, no debris on bridge	Q100CC (Wai) M2PP Piers1
1% AEP Waikanae & 10% AEP Mazengarb flows, M2PP proposal, debris on bridge	Q100CC (Wai) M2PP Piers2
1% AEP Waikanae & 10% AEP Mazengarb flows, M2PP proposal, debris on bridge, Waikanae berms with 500 mm aggradation	Q100CC (Wai) M2PP Piers2-BermRaise
1% AEP Waikanae & 10% AEP Mazengarb flows, M2PP proposal, debris on bridge, existing Waikanae channel	Q100CC (Wai) M2PP Piers2-existchan_at_bridge
1% AEP Waikanae (climate change allowance: 50% increase in flow) & 10% AEP Mazengarb flows, M2PP proposal, debris on bridge	Q100CC (Wai+50%CC) M2PP Piers2
1% AEP Waikanae & 10% AEP Mazengarb flows, low tide, M2PP proposal, debris on bridge	Q100CC (Wai) M2PP Piers2 LowTide
0.04% AEP Waikanae & 1% AEP Mazengarb flows, M2PP proposal, debris on bridge	Q2500CC (Wai) M2PP Piers2
0.04% AEP Waikanae & 1% AEP Mazengarb flows, M2PP proposal, debris on bridge, Waikanae berms with 500 mm aggradation, stopbanks raised	Q2500CC (Wai) M2PP Piers2-BermRaiseSBRaise
1% AEP Waikanae (extended hydrograph) & 10% AEP Mazengarb flows, M2PP proposal, debris on bridge, Assumed breach Chillingworth stopbank	Q100CC (Wai) M2PP Piers2-ChillBreach(longHG)
0.5% AEP Waikanae (extended hydrograph) & 10% AEP Mazengarb flows, M2PP proposal, debris on bridge, Assumed breach Chillingworth stopbank	Q200CC (Wai) M2PP Piers2-ChillBreach(longHG)
0.04% AEP Waikanae (extended hydrograph) & 1% AEP Mazengarb flows, M2PP proposal, debris on bridge, Assumed breach Chillingworth stopbank	Q2500CC (Wai) M2PP Piers2-ChillBreach(longHG)

Appendix B January 2005 – Calibration Simulation

The January 2005 flood event was rerun with the most recent Lidar data set. The following table is an update of Table 1 of REC (2010). Results show an improvement on the previous results: the average difference between recorded and model levels has been improved from -0.071 m to -0.014 m, while the average absolute difference has lowered from 0.212 m to 0.204 m.

Location (approximate)	Recorded level	Model level	Difference	Comment
xs 410 (RB)	20.792	20.517	-0.275	
xs 400 (RB)	19.880	19.433	-0.447	
xs 390 (RB)	21.178	19.149	-2.029	Recorded level in error?
xs 380 (RB)	17.475	18.228	0.753	
xs 345 (RB)	16.172	15.702	-0.470	
xs 340 (RB)	14.868	14.899	0.031	
xs 320 (RB)	13.019	12.981	-0.038	
xs 310 (RB)	12.377	12.152	-0.225	
xs 40 (RB)	1.635	1.784	0.149	
Jim Cooke Park stopbank (xs 260)	10.287	10.033	-0.254	
Near xs 210 (RB)	6.537	6.525	-0.012	Points close by, difference may reflect local afflux or subjectivity of recording
Greenaway Rd	6.672	6.337	-0.335	
Greenaway Rd	6.002	Dry		Localised stormwater?
Near xs 175 (RB)	5.325	5.346	0.021	
Near xs 155 (RB)	5.014	4.745	-0.269	
Footbridge, Otaihanga Domain (LB)	3.76	3.839	0.079	
ROW to 35-39 Otaihanga Rd	4.266	4.345	0.079	
ROW to 35-39 Otaihanga Rd	4.196	4.338	0.142	
ROW to 35-39 Otaihanga Rd	4.251	4.312	0.061	
Otaihanga Rd at substation	3.514	3.129	-0.385	Maybe culvert more blocked than assumed
Mid-way between xs 70 & 80 (RB)	2.677	3.197	0.520	
Toilet block, Otaihanga Domain	3.8	3.848	0.048	
73 Makora Rd (inside house)	4.586	4.387	-0.199	
River side of flood wall	3.792	3.878	0.086	
11 Toroa Rd	4.325	4.388	0.063	
3 Toroa Rd	4.424	4.387	-0.037	
21 Makora Rd (inside house)	3.124	3.458	0.334	
17?? Makoroa Rd (inside garage)	2.559	2.564	0.005	
Driveway to 46 Makora Rd	4.002	Dry		Localised stormwater? Couldn't flood from river
42 Makora Rd (garage)	4.012	Dry		Localised stormwater? Couldn't flood from river
42 Makora Rd (inside house)	3.94	Dry		Localised stormwater? Couldn't flood from river
42 Makora Rd (inside house)	3.487	Dry		Same location as point above. Error?
1 Makora Rd	2.203	2.401	0.198	
Average			-0.014	Highlighted cells only
Average (absolute)			0.204	

Further anecdotal and photographic evidence regarding flood levels has also been obtained. This suggests that the peak level at the Otaihanga Boating Club was 3 m RL (model level 2.8 m RL), and that flood levels reached 3 m in Ruru Road (model level 2.51 m). The accuracy of these observed levels is unknown.

Appendix C Pier Effects

Initial bridge design (supplied 4 May 2011):

Pier ratio, no-debris: Piers 5 of 1.35 m diameter. Channel width = 170 m approx at water level.

=> Pier ratio = 5 x 1.35/170 (approx) = 0.0397.

Debris allowance – 2 alternative approaches:

1. Bridge Manual - (Transit New Zealand, 2003). Intended for structural design of piers, but useful also to give guide to hydraulic impacts of debris. Assume floating debris raft at each pier, triangular raft, raft depth = half water depth (to max 3 m), raft width = half sum of adjacent spans (to max 15 m). (Figure 3.4, Transit New Zealand).

   Initial bridge design, and assuming WL = 5.3 m, A = 381 m^2,
   Debris area = 15 x 1.5/2 (Pier 1) + 15 x 1.3/2 (Pier 2) + (15 x 1.1/2)x3 (Piers 3-5) approx

   = 46 m^2

   => Pier ratio = 46/381 = 0.12.

2. GWRC method – typically add 0.1 to pier ratio to allow for debris (Wallace, 1991).
   => Pier ratio = 0.0397 +0.1 = 0.14 approx.

Results similar – suggest use GWRC as simpler and slightly more conservative in this case.

Subsequent bridge design (supplied 22 June 2011):

Pier width at base 1.5 m , at top 2.5 m, allow for 7° skew. Pier spacing B = 34.7 m

Pier ratio (using simplified pier profile): A of waterway (at 5.7 m) = 491m^2 (no piers), 472 m^2 (piers).

=> Pier ratio = 0.039 (slightly less if have lower flood level e.g. 5.2 m, but have used simplification of pier profile, i.e. approximation)

Bridge without piers (i.e. with abutment) - model output at bridge site:

$Q = 484\ m^3/s$, W = 180 m, H = 5.023 m, A = 368 m^2

=> v = 1.31 m/s, $Fr = v\sqrt{(gA/W)} = 0.293$, y = 5.023 – 0.81 = 4.2 m

Pier effect calculations – Benn et al (2004)

For no-debris case, T = 2 m average (say), c_d = 1.4 (diamond shape)

$0.5\ c_d T/B = 0.04$

From Fig 6.3 Benn et al, λ = 0.01 approx.

Afflux = λy = 4 cm.

For debris case, T = 2 x 0.139/0.039 = 7.1 m

c_d = 1.7 say

$0.5\ c_d T/B = 0.17$

λ = 0.02 approx

Afflux = λy = 8 cm.

Model: Compare models of piers (no debris) and piers (with debris) with model of bridge without piers.

Results – no debris model (Q100CC) shows increase in flood level of around 4 cm immediately upstream of 3 of the 4 pier sets, and 1-2cm more generally (Figures C.1, C.3).

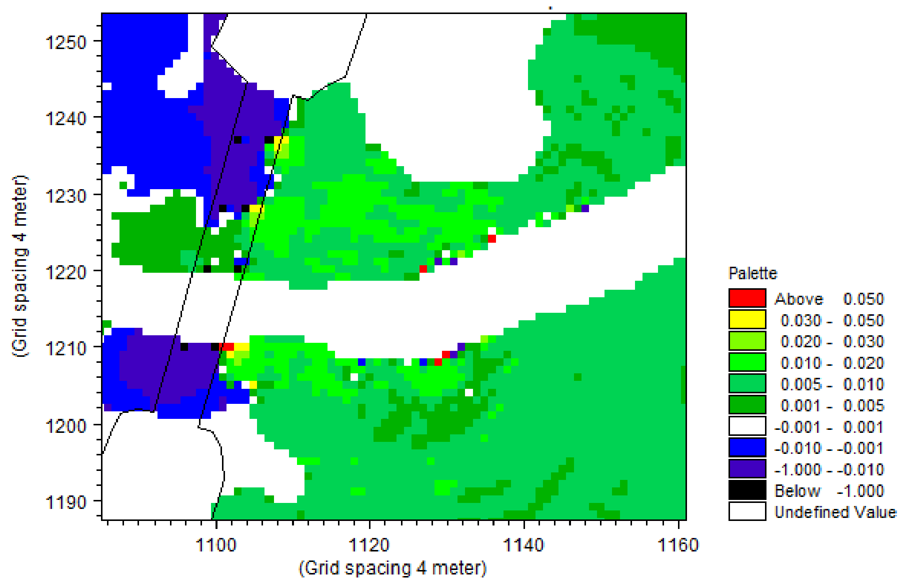


Figure C.1 Effect of piers on peak flood levels, no debris (Q100CC flood)

The debris model shows increase in flood level of around 5-10 cm immediately upstream of 3 of the 4 pier sets, and 2-4 cm more generally (Figures C.2, C.3). For both the no-debris and the debris models, the predictions are considered acceptably close to the afflux calculations based on Benn et al.

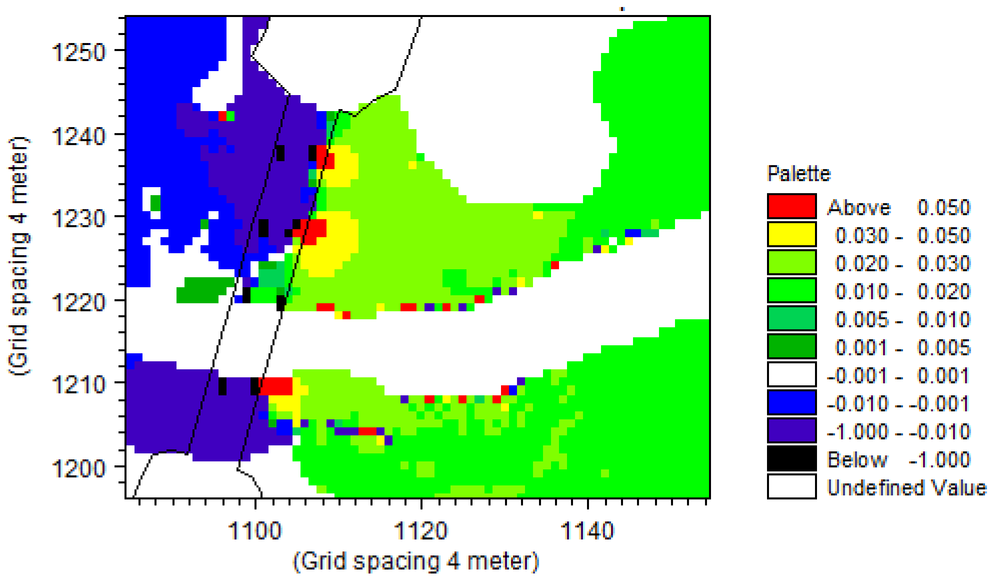


Figure C.2 Effect of piers on peak flood levels, debris on piers (Q100CC flood)

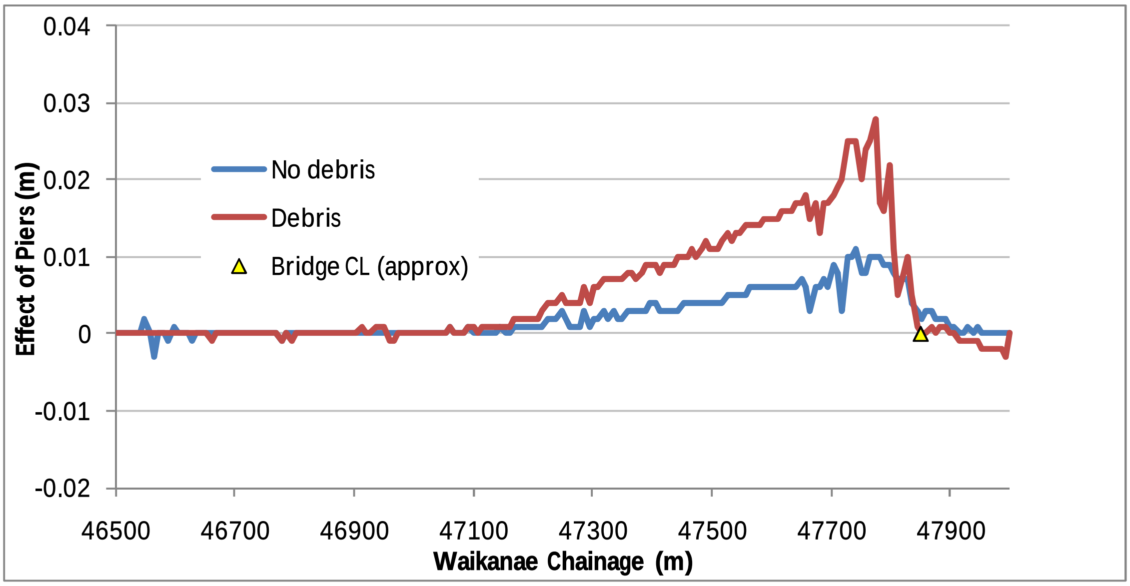


Figure C.3 Effect of piers on flood levels in main channel (Q100CC flood)

Appendix D Waikanae River Breach & Overflow Scenarios

D.1 Introduction

The design of the M2PP expressway north of the Waikanae River requires the consideration of a a scenario where the Waikanae River overflows its stopbanks. The overflow could be from either overtopping or breaching of the stopbanks.

Greater Wellington originally requested an 80 m^3/s flow, based on earlier modelling undertaken by Greater Wellington (GWRC). Further research has revealed that in 1997 GW considered various breach scenarios (locations, dimensions and rates of development), estimated their probability of occurrence and modelled those scenarios[1]. The breaches relevant to the M2PP investigations were at:

- Kauri –Puriri stopbank
- Chillingworth stopbank
- Jim Cooke Memorial Park stopbank (upstream end)

Of these three locations, the Jim Cooke Park site was considered the most likely to breach. GWRC considered that there was a high probability of a breach at that site in a 100 year flood event. However, the probability of breaches at Chillingworth or Kauri-Puriri stopbanks was estimated at only around 5% in a 100 year event. Since that time, the river has moved away from the Jim Cooke Park stopbank and flood protection works including rock groynes have been constructed in the river, and hence the likelihood of a breach there will have diminished.

The 80 m^3/s flow came from the Kauri-Puriri stopbank breach scenario, with a Waikanae River flow of 415 m^3/s (the then estimate of the 100 year flow).

River Edge Consulting has remodelled these breaches with the current model. This includes recent flood plain LiDAR survey, river channel survey, mid-level estimate climate change parameters to 2090 and also with the expressway bridge with debris.

D.2 Kauri-Puriri Breach

Of the original GWRC modelling, a Kauri-Puriri breach gave the greatest outflow, and hence this was examined first.

D.2.1 Q100CC (i.e. 480 m^3/s Waikanae River flow)

Assumptions: breach timing (beginning just before flood peak), dimensions and rate of development all as per the original GW assumptions c1997 (50m wide, down to 3.8m RL). Figure D.1 shows the breach location and the topography in the area.

[1] Wellington Regional Council (1997); Waikanae River Floodplain Management Plan: Phase Four - Numerical Hydraulic Modelling of Extreme Flows. Report WRC/RI-T-97/28

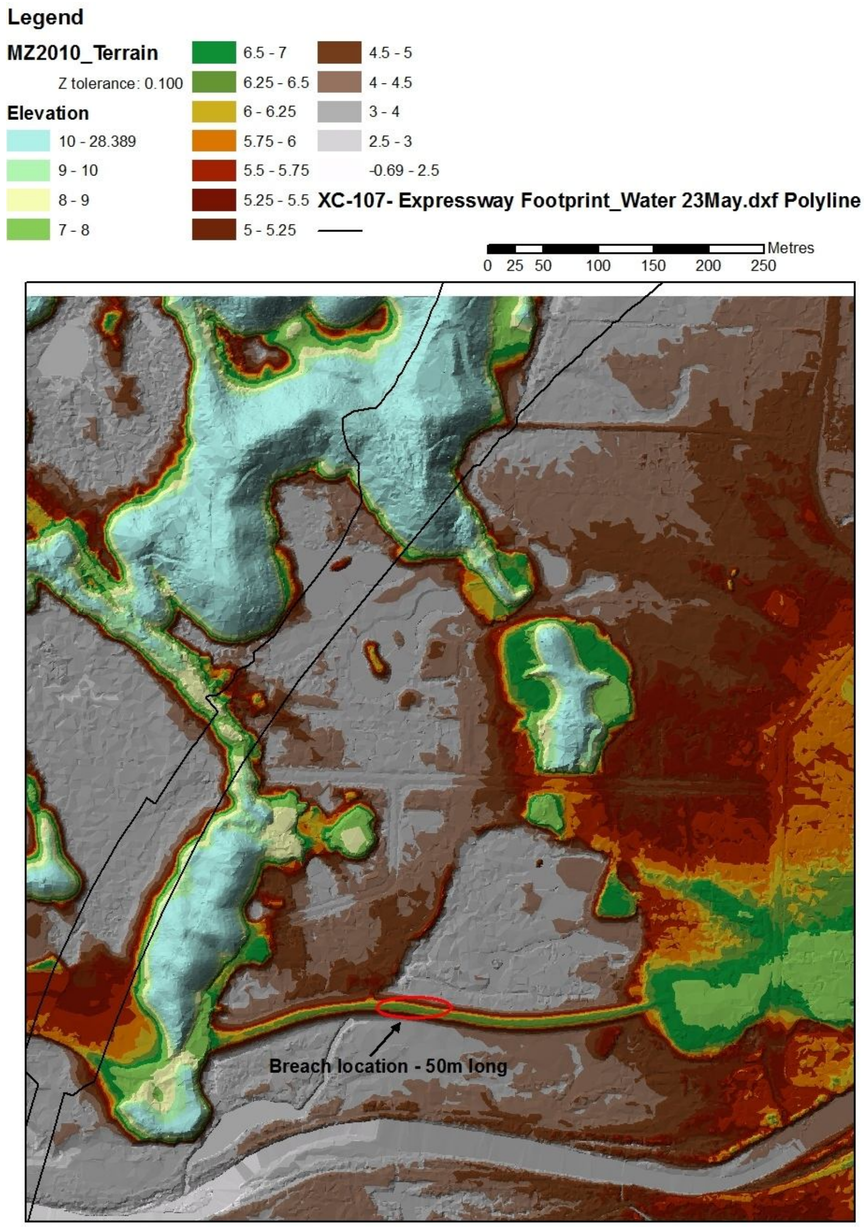


Figure D.1 Kauri Puriri stopbank breach location

Results showed a peak outflow of approximately 60 m^3/s, but this outflow filled up large ponding area with outflow via a narrow gap between sandhills. The outflow was only around 1 m^3/s (Figure D.2).

The model floodplain topography is based on LiDAR survey, including the topography of the gap. As LIDAR data are subject to uncertainty/accuracy of +/-150mm on clear ground, the model was rerun with the bed level in the gap lowered to 4.75 m RL, i.e. up to 350mm lower than in the base model. Results still showed that the outflow would be small (1.6 m^3/s).

Figure D.2 Kauri Puriri stopbank breach peak flood depths and flows (Q100CC)

D.2.2 Q2500CC (i.e. 700 m^3/s Waikanae River flow)

To get an understanding of the attenuation caused by the ponding behind the breach location, an extreme event, 2500 year return period, was run with the same breach assumptions.

The peak outflow along the Kauri-Puriri stopbank, from the breach plus overtopping, was 130 m^3/s. The total outflow from the pond was then 20 m^3/s – i.e. significant attenuation also. The 20 m^3/s was split roughly evenly between the north (KPOut1) and south (KPOut2) locations shown (Figure D.3).

Figures D.3 and D.4 show the flood map and the flows for this scenario.

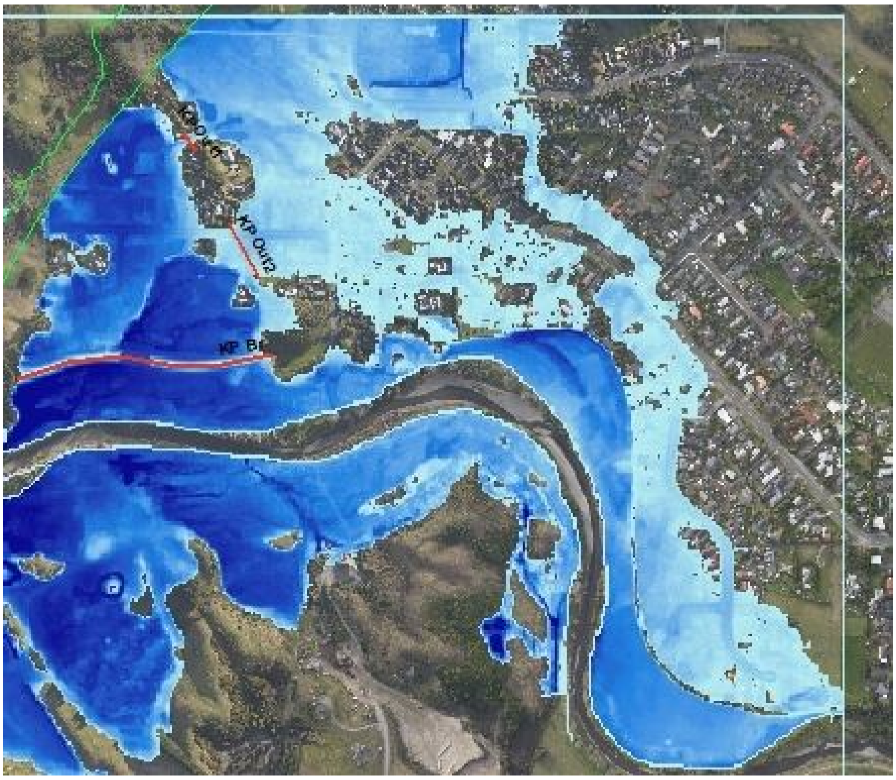


Figure D.3 Kauri Puriri stopbank breach peak flood depths (Q2500CC)

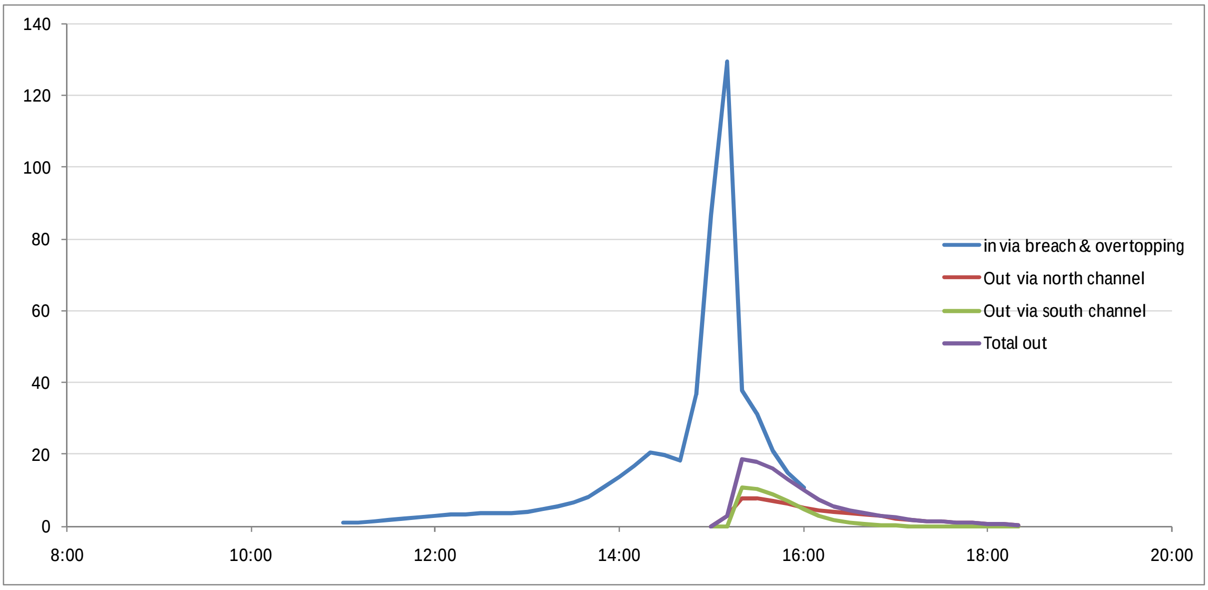


Figure D.4 Kauri Puriri stopbank flows, into and out of ponding area (Q2500CC)

D.3 Jim Cooke Park

The first breach site in Jim Cooke Park examined was that originally modelled by GWRC – i.e. at the upstream end of the stopbank. Again the GWRC breach dimensions (80 m length), rate of development and timing were used. Results showed a breach outflow of 15 m^3/s, attenuating to 1 m^3/s downstream (Figure D.5).

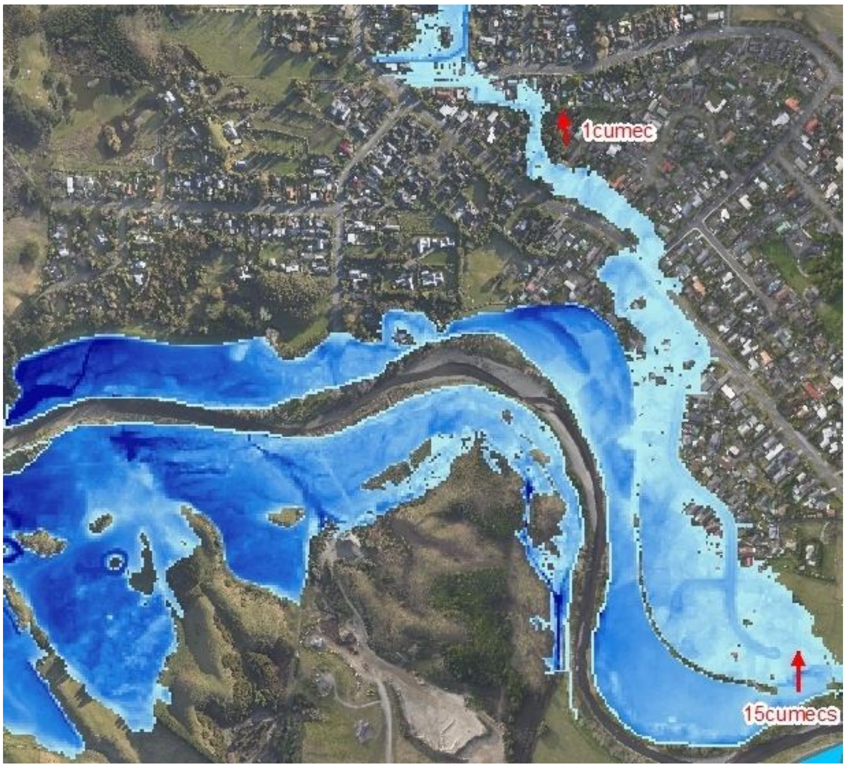


Figure D.5 Jim Cooke Park stopbank breach peak flood depths and flows (Q100CC)

A 120 m long breach was then considered at the same location. Peak outflow was about 22 m^3/s, again attenuating to around 1 m^3/s further downstream. Closer inspection of results showed that some of the outflow made its way back into the river (Figure D.6).

Finally, a longer breach (180 m) was modelled at a site downstream of the original breach site and developing to full size before the flood peak. For this scenario, a 200 year flood was modelled with a 25% longer duration than for the previous scenario. The standard design hydrograph is that proposed by NIWA[2], being based on the median shape of the six largest floods over the last 30 years. There appears to be little difference in the duration of those events, and a 25% increase in duration is a conservative assumption.

Results for this scenario showed outflows did not extend very far, as the floodplain behind the stopbanks is relatively high (Figure D.7).

[2] NIWA (2009); Review of the flood hydrology for the Waikanae and Otaki Rivers.

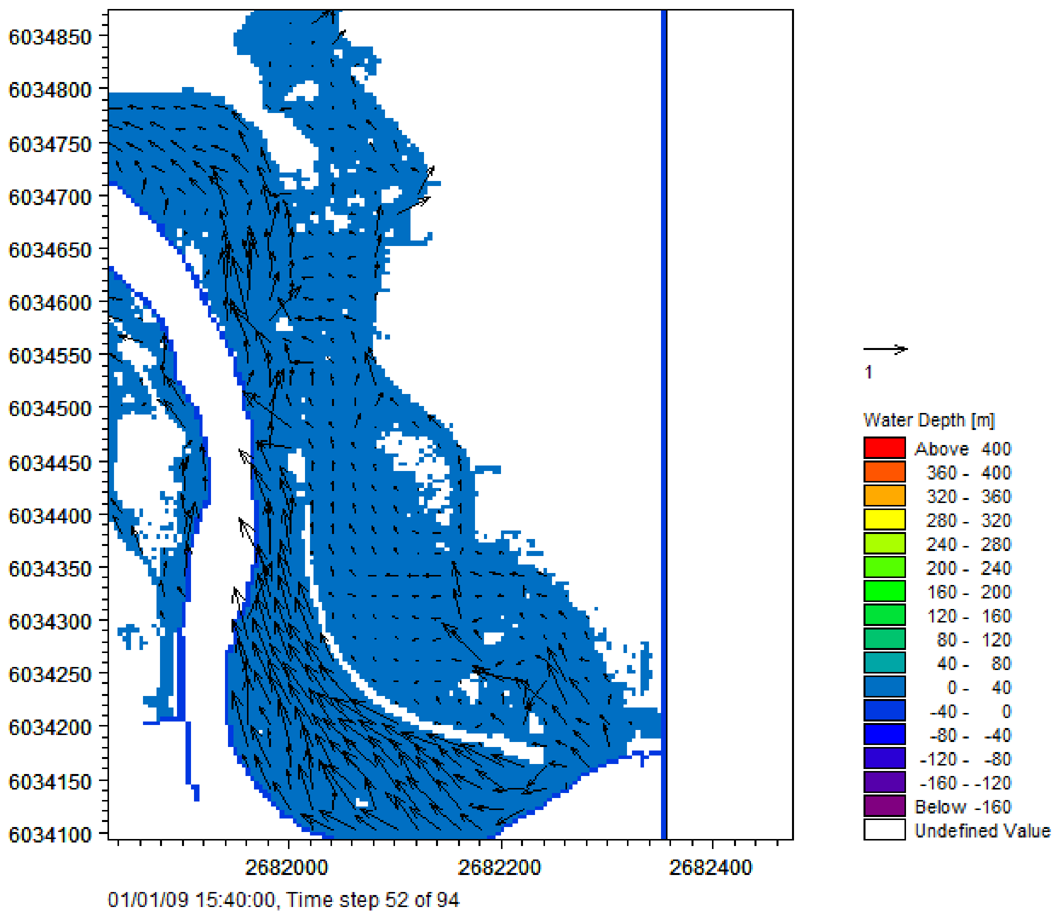


Figure D.6 Jim Cooke Park stopbank breach (longer breach) overflow directions (Q100CC)

Figure D.7 Jim Cooke Park stopbank breach (downstream site) peak flood depths (Q200CC)

D.4 Chillingworth breach

The last breach site examined was at the small Chillingworth stopbank. The original GWRC breach considered a 20 m breach, cutting down to 6.85 m RL. A 60 m long breach was modelled down to 6.5 m RL, and starting before the flood peak arrived. Four flow scenarios were modelled, as described below.

D.4.1 Q100CC (standard design hydrograph for Waikanae River)

Results with the standard design hydrograph gave a breach flow of 17 m^3/s, which attenuated slightly to around 15.5 m^3/s at the outflow location shown in Figure D.8. (Figure D.8 also shows an area of ponding from expressway runoff, unaffected by the breach flow.) Figure D.9 shows the breach hydrograph and the hydrograph at the outflow location.

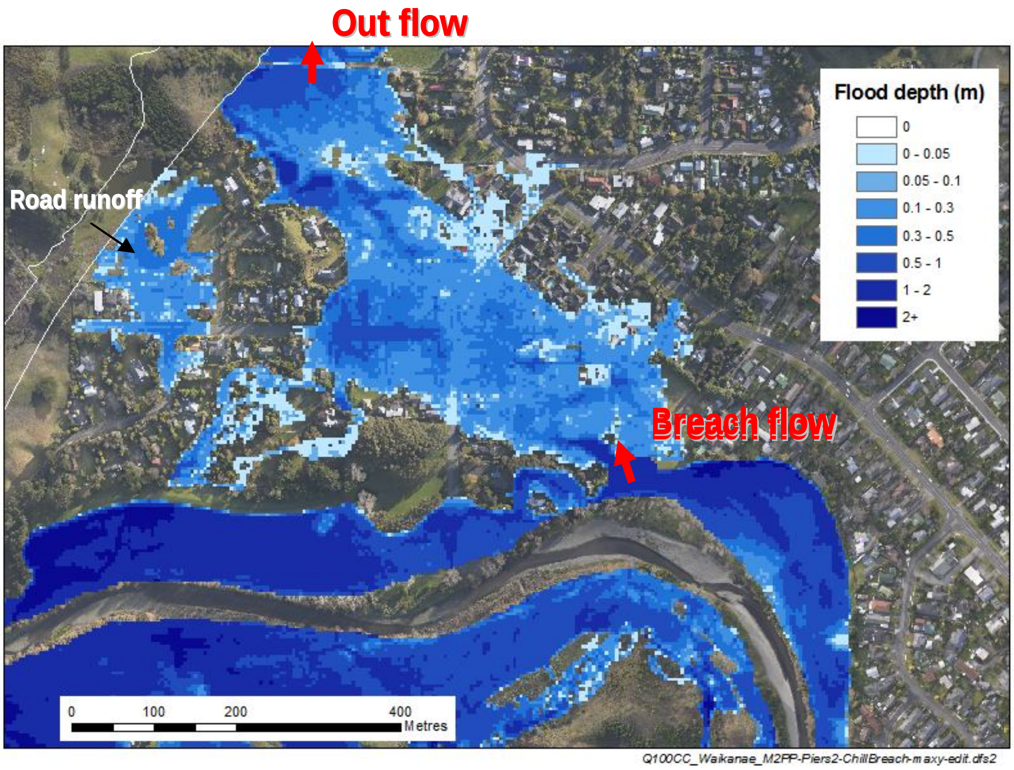


Figure D.8 Breach flow extent, Chillingworth breach, 1% AEP flood (standard hydrograph)

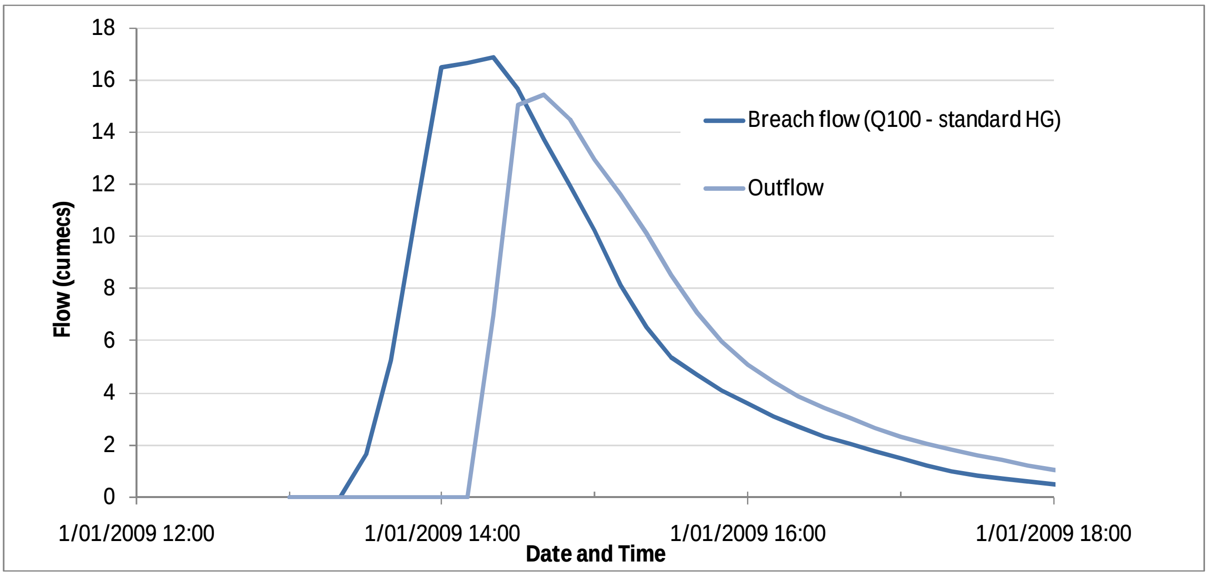


Figure D.9 Chillingworth breach hydrographs, 1% AEP, standard design hydrograph

D.4.2 Q100CC, longer duration

A 1% AEP flow with 25% longer duration was run. Peak breach flows were similar to those of the standard hydrograph, but the duration of the breach flow was longer (Figure D.10)

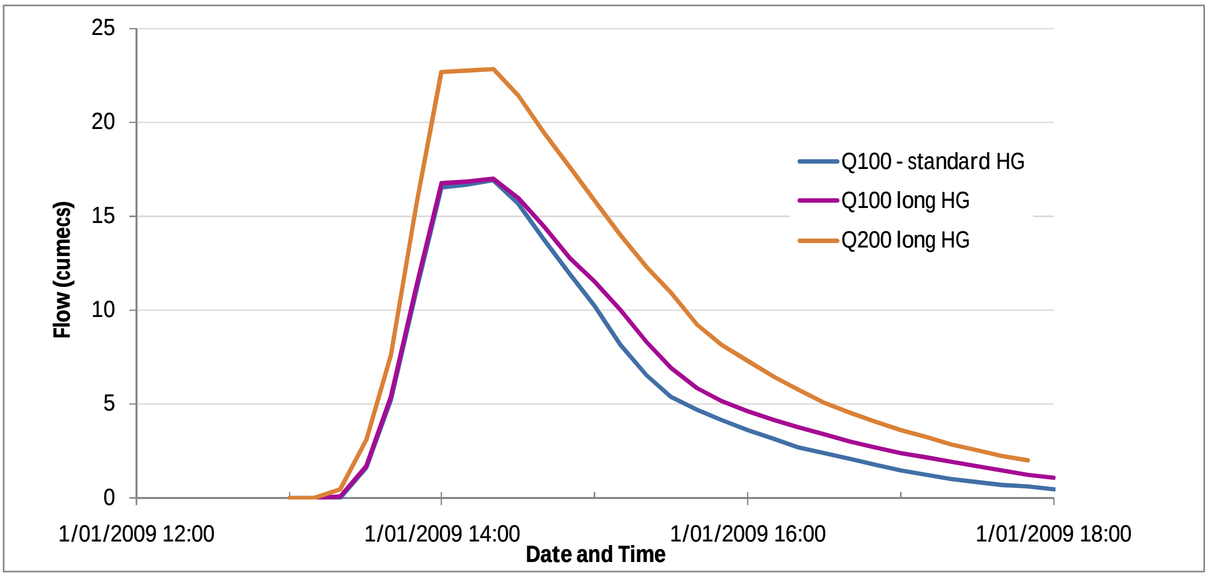


Figure D.10 Chillingworth stopbank breach flows

D.4.3 Q200CC, longer duration

Breach flows in a 0.5% AEP Waikanae River flow, with the extended duration hydrograph shape, were predicted to be around 23 m^3/s (Figure D.10). The downstream outflow (at the location shown in Figure D.8) was around 20 m^3/s (Figure D.11).

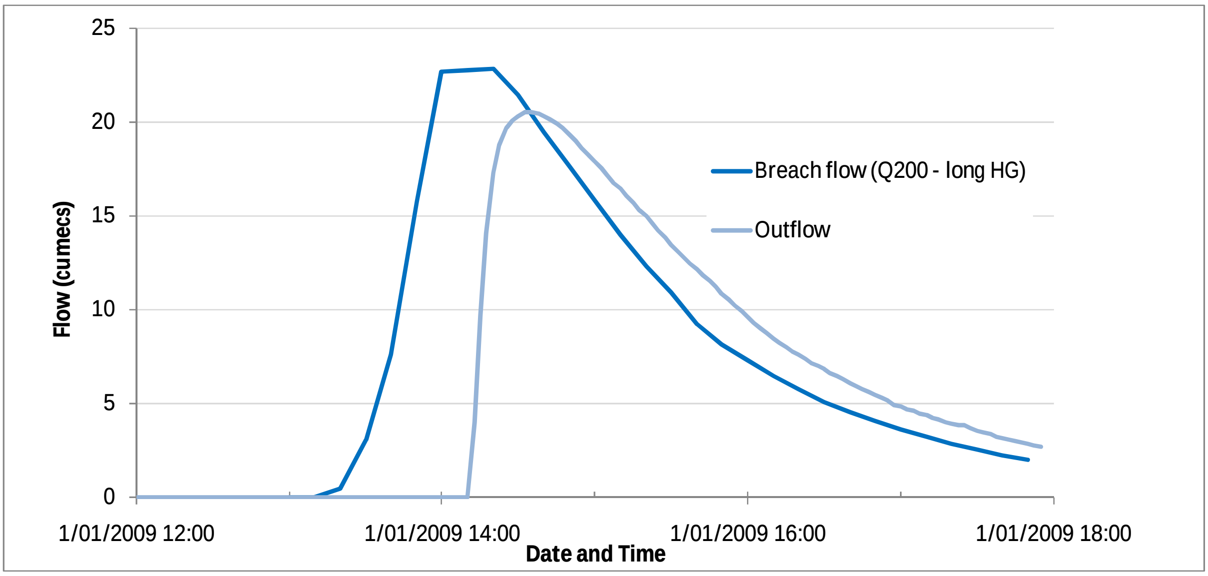


Figure D.11 Chillingworth breach hydrographs, 0.5% AEP, extended hydrograph

D.4.4 Q2500CC , longer duration

A final simulation with this breach was made with the 2500 year flow, with the extended hydrograph. The predicted peak outflows (at locations 6 and 7 shown in Figure D.12) was 40 m^3/s, resulting from breach flow and overtopping flows. Figure D.13 shows floodplain flows at various locations.

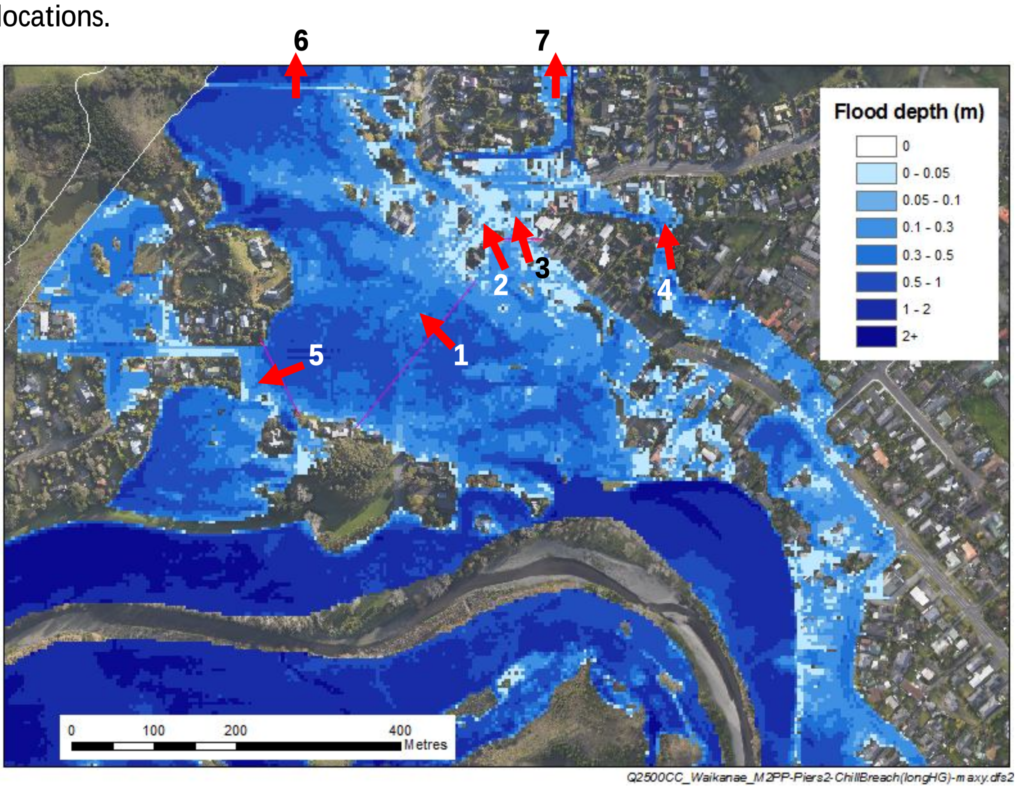


Figure D.12 Breach flow extent, Chillingworth breach, 0.04% AEP flood (extended hydrograph)

Figure D.13 Breach plus overflow hydrographs, Chillingworth breach, 0.04% AEP flood (extended hydrograph)

D.5 Conclusions

Of the breach scenarios modelled, the Chillingworth breach scenario would have the most impact on the expressway design. Previous GWRC assessment suggests that the likelihood of a breach there is fairly unlikely in design events, but nonetheless conceivable.

A recommended overflow hydrograph to model is that resulting from a 200 year flow event plus a 60 m long breach in the Chillingworth stopbank. This hydrograph is in Figures D.10 and D.11, i.e. peaking at around 23 m^3/s. Note that the hydrograph results from an extended version of the design Waikanae River hydrographs (25% longer duration than the standard design hydrographs).

This flow should be applied along the Chillingworth stopbank and routed through the SKM model of the Waimeha floodplain. At this stage the Alliance proposes to test a conservative breach scenario coupled to a 100yr storm in the Waimeha Stream catchment such that the breach and Waimeha peaks coincide. However, it is noted that this assumption would be subject to review if clearer local knowledge becomes available on how these catchments perform.

Appendix E Tabular Summary of Channel Peak Flood Levels

		10% AEP			1% AEP			1.5 x (1% AEP)		
Location	Chainage	Existing	M2PP	Difference	Existing	M2PP	Difference	Existing	M2PP	Difference
	700	7.20	7.172	-0.028	7.885	7.881	-0.004	8.30	8.31	0.007
	730	7.18	7.16	-0.029	7.85	7.84	-0.004	8.32	8.32	0.008
Waterstone Dr	754	7.18	7.15	-0.030	7.84	7.84	-0.004	8.31	8.32	0.009
	785	7.16	7.14	-0.028	7.76	7.76	-0.004	8.09	8.10	0.012
Fytfield Place	823	7.13	7.10	-0.030	7.75	7.74	-0.004	8.09	8.10	0.013
	851	6.91	6.88	-0.026	7.17	7.18	0.007	7.40	7.45	0.052
	851	6.91	6.88	-0.026	7.17	7.18	0.007	7.40	7.45	0.052
	877	6.86	6.83	-0.031	7.13	7.14	0.010	7.38	7.43	0.056
	898	6.86	6.83	-0.032	7.14	7.14	0.007	7.38	7.43	0.051
	912	6.84	6.81	-0.032	7.12	7.12	0.002	7.37	7.41	0.044
	933	6.84	6.81	-0.033	7.12	7.12	0.002	7.37	7.40	0.029
	959	6.82	6.79	-0.036	7.10	7.10	0.000	7.35	7.38	0.030
	987	6.76	6.72	-0.043	7.04	7.03	-0.011	7.27	7.28	0.010
	994	6.75	6.73	-0.015	7.03	7.04	0.019	7.26	7.30	0.042
	1001	6.73	6.73	0.007	7.01	7.05	0.038	7.25	7.31	0.060
	1008	6.70	6.73	0.033	6.99	7.05	0.061	7.23	7.31	0.080
Culvert 14 inlet	1015	6.69			6.99			7.23		
	1022	6.69			6.99			7.23		
Culvert 14 outlet	1152	6.64			6.93			7.16		
	1160	6.63	6.71	0.082	6.93	7.02	0.091	7.16	7.26	0.108
	1185	6.23	6.30	0.069	6.46	6.45	-0.008	6.61	6.59	-0.021
	1205	6.20	6.25	0.050	6.42	6.41	-0.001	6.55	6.55	-0.005
	1251	6.15	6.19	0.045	6.38	6.38	-0.001	6.50	6.50	-0.004
	1274	6.13	6.17	0.044	6.37	6.37	-0.001	6.49	6.49	-0.004
	1321	6.10	6.15	0.043	6.38	6.37	-0.002	6.50	6.50	-0.006
	1331	5.81	5.85	0.041	6.30	6.30	0.000	6.42	6.42	-0.004
WWTP Drain inflow	1421	5.78	5.83	0.050	6.29	6.29	0.000	6.42	6.42	-0.004
	1426	5.68	5.74	0.054	6.22	6.22	0.000	6.34	6.33	-0.004

Table E.1 Mazengarb Drain peak levels

		10% AEP			1% AEP			1.5 x (1% AEP)		
Location	Chainage	Existing	M2PP	Difference	Existing	M2PP	Difference	Existing	M2PP	Difference
	1000	6.96	6.96	0.000	7.04	7.04	0.000	7.19	7.25	0.066
	1031	6.77	6.77	0.000	6.87	6.88	0.004	7.15	7.22	0.074
	1062	6.58	6.59	0.012	6.81	6.82	0.007	7.14	7.21	0.075
	1093	6.48	6.53	0.049	6.80	6.81	0.009	7.13	7.21	0.075
	1109	6.46	6.52	0.055	6.80	6.80	0.009	7.13	7.21	0.075
	1116	6.46	6.52	0.056	6.79	6.80	0.009	7.13	7.21	0.075
	1124	6.46	6.52	0.057	6.79	6.80	0.009	7.13	7.21	0.075
Culvert 14 inlet	1140	6.42	6.49	0.065	6.67	6.71	0.038	6.86	6.99	0.123
	1148	6.41			6.66			6.86		
	1156	6.40			6.65			6.85		
	1164	6.39			6.65			6.85		
	1180	6.38			6.64			6.83		
	1196	6.36			6.62			6.82		
	1204	6.35			6.62			6.81		
	1220	6.33			6.60			6.80		
Culvert 14 outlet	1228	6.31	6.38	0.076	6.58	6.62	0.046	6.77	6.87	0.100
	1292	6.19	6.26	0.069	6.51	6.54	0.032	6.71	6.78	0.069
	1367	6.13	6.20	0.073	6.47	6.50	0.034	6.67	6.74	0.066
	1406	6.09	6.17	0.076	6.44	6.48	0.035	6.66	6.72	0.066
Killalea Pl	1424	6.08	6.15	0.076	6.43	6.47	0.036	6.65	6.72	0.067
	1432	5.82	5.87	0.049	6.21	6.24	0.035	6.33	6.35	0.022

Table E.2 WWTP Drain peak levels

Location	Waikanae Chainage	50% AEP M2PP	5% AEP CC Existing	5% AEP CC M2PP No debris	1% AEP CC Existing	1% AEP CC M2PP No debris	1% AEP CC M2PP Debris	1% AEP CC M2PP Debris, Existing Channel	1% AEP CC M2PP Debris, Cut back Spur	1% AEP CC M2PP Debris, berm raise	1% AEP (50% CC) M2PP Debris	0.04% AEP M2PP Debris	0.04% AEP M2PP Debris, Berm Raised & SB Raised
XS 175	47540	4.587	5.258	5.239	5.571	5.555	5.563	5.586	5.488	5.742	5.864	6.086	6.210
	47547.8	4.57	5.246	5.226	5.560	5.543	5.551	5.575	5.477	5.730	5.854	6.077	6.200
	47555.6	4.553	5.234	5.214	5.549	5.532	5.540	5.565	5.467	5.718	5.844	6.068	6.190
	47594.6	4.471	5.177	5.154	5.496	5.477	5.486	5.513	5.413	5.660	5.793	6.024	6.142
	47602.4	4.455	5.166	5.142	5.485	5.466	5.475	5.503	5.403	5.649	5.783	6.011	6.132
	47610.2	4.439	5.155	5.13	5.475	5.456	5.465	5.493	5.392	5.639	5.773	5.996	6.123
	47618	4.423	5.145	5.12	5.466	5.446	5.455	5.484	5.382	5.629	5.764	5.987	6.114
	47625.8	4.409	5.135	5.11	5.456	5.437	5.446	5.475	5.372	5.619	5.754	5.981	6.104
	47633.6	4.394	5.124	5.099	5.445	5.426	5.435	5.464	5.362	5.608	5.742	5.972	6.094
	47641.4	4.379	5.113	5.087	5.435	5.415	5.423	5.454	5.351	5.596	5.731	5.957	6.084
	47649.2	4.364	5.1	5.074	5.422	5.404	5.413	5.441	5.341	5.583	5.718	5.950	6.074
	47657	4.348	5.086	5.056	5.403	5.386	5.395	5.422	5.331	5.565	5.698	5.926	6.052
	47664.8	4.331	5.063	5.039	5.383	5.362	5.370	5.403	5.321	5.548	5.677	5.907	6.035
	47672.6	4.313	5.044	5.018	5.361	5.338	5.352	5.382	5.311	5.526	5.661	5.890	6.018
	47680.4	4.296	5.022	4.994	5.339	5.320	5.329	5.361	5.300	5.504	5.639	5.862	5.994
	47688.2	4.276	5.004	4.974	5.322	5.298	5.311	5.346	5.290	5.485	5.620	5.847	5.979
	47696	4.255	4.988	4.956	5.307	5.283	5.297	5.330	5.283	5.469	5.603	5.828	5.962
	47703.8	4.239	4.977	4.944	5.298	5.274	5.286	5.317	5.278	5.457	5.587	5.810	5.942
	47711.6	4.225	4.967	4.934	5.286	5.262	5.272	5.311	5.268	5.445	5.578	5.805	5.934
	47719.4	4.211	4.956	4.923	5.274	5.244	5.261	5.302	5.257	5.430	5.567	5.786	5.922
	47727.2	4.198	4.946	4.912	5.262	5.236	5.253	5.286	5.248	5.418	5.557	5.771	5.912
	47735	4.185	4.937	4.903	5.256	5.228	5.244	5.279	5.241	5.409	5.550	5.763	5.907
	47742.8	4.174	4.93	4.896	5.248	5.220	5.233	5.271	5.236	5.401	5.540	5.755	5.892
	47750.6	4.164	4.921	4.886	5.240	5.211	5.225	5.265	5.229	5.392	5.533	5.747	5.884
	47758.4	4.154	4.914	4.879	5.231	5.200	5.214	5.259	5.221	5.381	5.523	5.734	5.873
	47766.2	4.143	4.903	4.87	5.219	5.194	5.206	5.249	5.213	5.370	5.508	5.727	5.855
XS 155	47774	4.132	4.894	4.861	5.206	5.179	5.197	5.240	5.202	5.355	5.495	5.708	5.843
	47781.75	4.121	4.885	4.855	5.194	5.173	5.188	5.235	5.194	5.346	5.485	5.691	5.829
	47789.5	4.111	4.879	4.849	5.190	5.168	5.183	5.227	5.185	5.334	5.476	5.689	5.820
	47797.25	4.1	4.878	4.838	5.192	5.159	5.173	5.221	5.176	5.320	5.475	5.682	5.807
	47805	4.088	4.871	4.828	5.188	5.148	5.155	5.198	5.161	5.297	5.459	5.664	5.781
	47812.75	4.077	4.869	4.82	5.181	5.135	5.136	5.181	5.139	5.272	5.454	5.667	5.751
Muaupoko Inflow	47820.5	4.066	4.869	4.809	5.171	5.118	5.123	5.164	5.126	5.255	5.423	5.633	5.736
	47828.25	4.019	4.79	4.744	5.106	5.051	5.056	5.087	5.057	5.174	5.338	5.541	5.633
	47836	4.004	4.772	4.738	5.075	5.042	5.045	5.073	5.046	5.155	5.332	5.522	5.609
	47843.75	3.99	4.757	4.733	5.058	5.038	5.038	5.059	5.040	5.139	5.315	5.493	5.582
Bridge CL (approx)	47851.5	3.975	4.744	4.728	5.043	5.034	5.033	5.046	5.034	5.131	5.308	5.491	5.578
	47859.25	3.959	4.731	4.724	5.026	5.030	5.029	5.034	5.029	5.125	5.304	5.485	5.571
	47867	3.943	4.719	4.721	5.016	5.027	5.027	5.022	5.027	5.120	5.302	5.484	5.568
	47874.86	3.929	4.71	4.713	5.008	5.020	5.020	5.014	5.021	5.110	5.297	5.479	5.562
	47882.71	3.916	4.701	4.706	5.000	5.013	5.013	5.006	5.015	5.099	5.292	5.477	5.555
	47890.57	3.903	4.691	4.698	4.992	5.006	5.006	4.998	5.008	5.089	5.285	5.471	5.546
	47898.43	3.889	4.682	4.691	4.983	4.998	4.999	4.990	5.001	5.079	5.278	5.464	5.537
	47906.29	3.875	4.672	4.683	4.975	4.992	4.992	4.983	4.993	5.069	5.271	5.457	5.528
	47922	3.848	4.654	4.668	4.959	4.978	4.978	4.968	4.978	5.049	5.258	5.444	5.509
	47929.86	3.835	4.646	4.66	4.952	4.971	4.971	4.961	4.971	5.041	5.251	5.438	5.501
	47945.57	3.814	4.631	4.643	4.938	4.956	4.956	4.947	4.954	5.022	5.237	5.422	5.483
	47953.43	3.805	4.625	4.634	4.932	4.948	4.948	4.941	4.946	5.013	5.230	5.415	5.474
	47961.29	3.797	4.619	4.625	4.926	4.940	4.941	4.935	4.938	5.004	5.223	5.409	5.466
	47969.14	3.789	4.613	4.616	4.921	4.933	4.933	4.929	4.930	4.996	5.216	5.404	5.459
XS 140	47977	3.782	4.607	4.607	4.916	4.925	4.926	4.924	4.923	4.988	5.210	5.398	5.454
	47984.69	3.771	4.598	4.597	4.906	4.915	4.916	4.915	4.913	4.978	5.200	5.389	5.444
	47992.38	3.76	4.589	4.586	4.896	4.904	4.906	4.905	4.903	4.966	5.190	5.379	5.436
	48000.08	3.752	4.579	4.572	4.882	4.890	4.899	4.891	4.896	4.952	5.185	5.368	5.424

Table E.3 Waikanae River peak levels

Location	Waikanae Chainage	50% AEP M2PP		5% AEP M2PP		1% AEP M2PP No Debris		1% AEP M2PP Debris	
		Q	v	Q	v	Q	v	Q	v
XS 175	47540	102.57	1.35	157.08	1.42	185.02		185.20	1.45
	47547.8	102.83	1.35	157.66	1.42	185.43		185.91	1.45
	47555.6	103.23	1.35	158.19	1.43	186.17		186.54	1.46
	47563.4	103.34	1.36	158.52	1.44	186.82		186.94	1.47
	47571.2	103.30	1.36	158.79	1.44	187.23		187.32	1.48
	47579	103.28	1.38	159.05	1.45	187.60		187.69	1.49
	47586.8	103.22	1.39	159.30	1.46	187.96		188.07	1.49
	47594.6	103.22	1.40	159.67	1.46	188.31		188.66	1.50
	47602.4	103.21	1.41	159.96	1.47	188.90		189.17	1.52
	47610.2	103.20	1.43	159.93	1.47	189.44		189.30	1.54
	47618	103.08	1.44	159.55	1.48	189.60		189.10	1.55
	47625.8	102.82	1.45	159.25	1.50	189.39		189.08	1.57
	47633.6	102.57	1.47	159.34	1.51	189.31		189.87	1.59
	47641.4	102.45	1.48	159.66	1.53	189.68		190.69	1.61
	47649.2	102.45	1.49	160.24	1.55	190.29		191.70	1.63
	47657	102.71	1.51	162.18	1.57	191.33		194.59	1.65
	47664.8	103.26	1.52	165.40	1.58	194.71		198.46	1.66
	47672.6	103.91	1.54	168.44	1.60	199.26		202.11	1.67
	47680.4	104.45	1.55	171.44	1.61	202.88		205.94	1.69
	47688.2	105.99	1.56	173.99	1.65	206.27		208.78	1.72
	47696	107.43	1.56	175.00	1.68	209.16		209.55	1.74
	47703.8	107.46	1.57	174.32	1.68	209.89		208.45	1.74
	47711.6	107.12	1.59	173.08	1.67	208.77		207.71	1.73
	47719.4	106.67	1.60	172.05	1.66	208.12		207.28	1.73
	47727.2	106.30	1.61	171.27	1.65	207.98		206.62	1.73
	47735	105.84	1.62	170.01	1.65	207.89		205.20	1.74
	47742.8	105.23	1.63	168.31	1.68	206.83		203.54	1.77
	47750.6	104.68	1.64	166.80	1.70	204.69		202.86	1.80
	47758.4	104.37	1.65	165.54	1.73	203.33		201.47	1.83
	47766.2	104.21	1.66	164.59	1.76	201.92		200.61	1.87
XS 155	47774	104.12	1.67	164.02	1.79	201.14		200.50	1.91
	47781.75	104.08	1.59	163.42	1.71	200.79		200.61	1.83
	47789.5	104.05	1.51	163.21	1.63	200.56		199.51	1.75
	47797.25	104.01	1.44	164.07	1.56	199.56		199.77	1.68
	47812.75	103.95	1.34	166.69	1.50	203.02		206.76	1.66
Muaupoko Inflow	47820.5	103.83	1.31	169.04	1.50	204.89		209.84	1.67
	47836	113.28	1.38	189.68	1.69	230.19		233.67	1.87
	47843.75	113.22	1.35	189.12	1.66	230.00		233.33	1.85
Bridge CL (approx)	47851.5	113.18	1.32	188.62	1.64	228.98		232.82	1.83
	47859.25	113.19	1.29	188.16	1.61	228.04		231.94	1.80
XS 150	47867	113.19	1.26	187.40	1.58	227.18		230.51	1.77
	47874.86	113.19	1.27	186.67	1.59	226.02		229.21	1.77
	47882.71	113.21	1.28	185.88	1.59	224.95		227.92	1.77
	47890.57	113.27	1.30	185.14	1.59	223.86		226.79	1.76
	47898.43	113.39	1.31	184.49	1.60	222.84		225.79	1.76
	47906.29	113.52	1.32	183.79	1.60	221.92		224.70	1.76
	47914.14	113.70	1.34	183.24	1.61	220.94		223.61	1.76
	47922	113.91	1.36	182.79	1.62	219.98		222.51	1.76
	47929.86	114.04	1.37	182.32	1.63	218.99		221.43	1.76
	47969.14	113.70	1.45	181.36	1.67	215.43		217.59	1.77
XS 140	47977	113.47	1.46	181.28	1.68	214.80		216.91	1.77
	47984.69	113.56	1.47	181.26	1.70	214.26		216.51	1.79
	47992.38	113.65	1.48	181.25	1.71	214.02		216.09	1.80
	48000.08	113.45	1.48	181.75	1.72	213.79		215.76	1.82

Table E.4 Waikanae River peak flows and velocities (main channel)

Location	Waikanae Chainage	50% AEP M2PP No debris	1% AEP M2PP Debris
XS 175	47540	4.490	5.499
	47547.8	4.465	5.486
	47555.6	4.438	5.474
	47594.6	4.330	5.416
	47602.4	4.310	5.404
	47610.2	4.291	5.393
	47618	4.272	5.384
	47625.8	4.255	5.373
	47633.6	4.237	5.361
	47641.4	4.220	5.349
	47649.2	4.203	5.336
	47657	4.184	5.316
	47664.8	4.163	5.296
	47672.6	4.141	5.277
	47680.4	4.119	5.259
	47688.2	4.092	5.239
	47696	4.064	5.226
	47703.8	4.045	5.215
	47711.6	4.028	5.206
	47719.4	4.011	5.198
	47727.2	3.995	5.184
	47735	3.980	5.176
	47742.8	3.967	5.170
	47750.6	3.956	5.161
	47758.4	3.945	5.153
	47766.2	3.933	5.142
XS 155	47774	3.922	5.132
	47781.75	3.910	5.118
	47789.5	3.898	5.114
	47797.25	3.887	5.107
	47805	3.876	5.101
	47812.75	3.866	5.099
Muaupoko Inflow	47820.5	3.856	5.102
	47828.25	3.812	5.034
	47836	3.801	5.005
	47843.75	3.790	4.972
Bridge CL (approx)	47851.5	3.779	4.948
	47859.25	3.769	4.939
XS 150	47867	3.759	4.930
	47874.86	3.748	4.923
	47882.71	3.738	4.916
	47890.57	3.728	4.909
	47898.43	3.718	4.903
	47906.29	3.709	4.896
	47922	3.689	4.884
	47929.86	3.678	4.878
	47945.57	3.658	4.868
	47953.43	3.650	4.864
	47961.29	3.642	4.861
	47969.14	3.635	4.857
XS 140	47977	3.627	4.853
	47984.69	3.617	4.844
	47992.38	3.607	4.835
	48000.08	3.597	4.820

Table E.5 Waikanae River peak levels, modified design channel

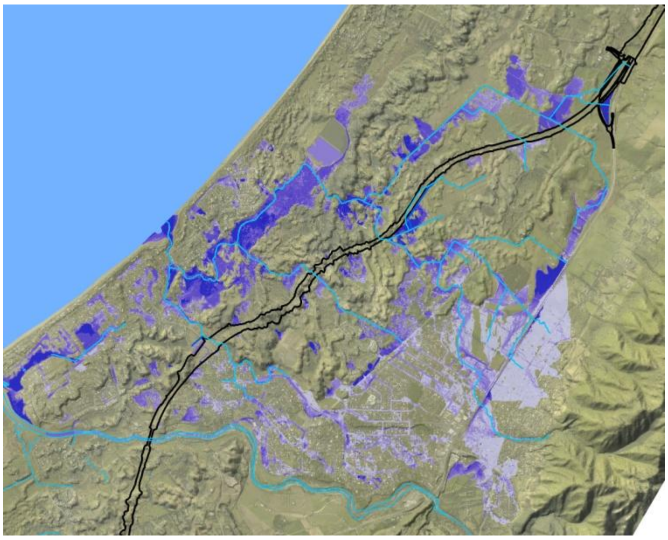
Location	Waikanae Chainage	50% AEP M2PP		1% AEP M2PP Debris	
		Q	v	Q	v
XS 175	47540	106.19	1.52	187.74	1.61
	47547.8	107.15	1.54	188.81	1.62
	47555.6	108.33	1.55	189.80	1.64
	47563.4	108.80	1.56	190.48	1.65
	47571.2	108.88	1.56	191.06	1.66
	47579	109.04	1.57	191.66	1.68
	47586.8	109.08	1.58	192.39	1.68
	47594.6	109.11	1.58	193.62	1.70
	47602.4	109.16	1.58	194.95	1.71
	47610.2	109.21	1.59	195.71	1.72
	47618	109.25	1.59	196.00	1.73
	47625.8	109.24	1.59	196.53	1.73
	47633.6	109.24	1.59	197.90	1.74
	47641.4	109.31	1.59	199.41	1.75
	47649.2	109.53	1.59	200.91	1.75
	47657	110.28	1.58	204.54	1.76
	47664.8	111.52	1.58	209.45	1.76
	47672.6	112.85	1.57	213.50	1.77
	47680.4	114.36	1.57	217.51	1.76
	47688.2	117.00	1.57	220.44	1.74
	47696	119.25	1.60	221.64	1.75
	47703.8	119.70	1.60	221.40	1.76
	47711.6	119.67	1.60	221.14	1.76
	47719.4	119.63	1.59	221.69	1.75
	47727.2	119.63	1.58	222.43	1.75
	47735	119.47	1.57	221.85	1.75
	47742.8	118.97	1.56	220.43	1.74
	47750.6	118.26	1.54	220.24	1.73
	47758.4	117.71	1.52	220.47	1.73
	47766.2	117.40	1.51	220.72	1.72
XS 155	47774	117.22	1.50	221.75	1.71
	47781.75	117.14	1.49	222.72	1.70
	47789.5	117.12	1.48	221.87	1.69
	47797.25	117.11	1.47	220.19	1.68
	47812.75	117.02	1.45	217.00	1.66
Muaupoko Inflow	47820.5	116.91	1.43	213.82	1.64
	47836	126.24	1.54	240.88	1.88
	47843.75	126.22	1.53	249.05	1.96
Bridge CL (approx)	47851.5	126.20	1.52	252.25	2.00
	47859.25	126.14	1.51	251.34	1.99
XS 150	47867	126.10	1.50	250.45	1.99
	47874.86	126.09	1.49	249.82	1.96
	47882.71	126.08	1.48	249.47	1.95
	47890.57	126.07	1.48	249.23	1.93
	47898.43	126.08	1.47	249.11	1.91
	47906.29	126.12	1.46	248.93	1.89
	47914.14	126.25	1.46	248.79	1.87
	47922	126.65	1.45	248.91	1.86
	47929.86	127.29	1.45	248.82	1.84
	47969.14	127.83	1.41	246.69	1.74
XS 140	47977	127.91	1.40	246.38	1.72
	47984.69	128.13	1.41	246.13	1.73
	47992.38	128.38	1.41	245.97	1.75
	48000.08	128.65	1.42	246.03	1.77

Table E.6 Waikanae River peak flows and velocities (main channel, modified design channel)

Appendix 22.G

Waimeha Flood Modelling Report

Mackays to Peka Peka Alliance Flood Hazard Mapping



WAIMEHA AREA

HYDRAULIC MODELLING AND MAPPING REPORT

M2PP- AEE- RPT- CV- SW- 182

Revision B

20 September 2011

Mackays to Peka Peka Alliance Flood Hazard Mapping

HYDRAULIC MODELLING AND MAPPING REPORT

M2PP- AEE- RPT- CV- SW- 182

Revision B

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1. Introduction

The Mackays to Peka Peka Expressway Alliance commissioned SKM to utilise the existing hydrological and hydraulic models of the Waimeha, Kakariki and Ngarara Catchment; to identify the impacts on flooding associated with the proposed Mackays to Peka Peka Expressway. The existing model was built as part of a comprehensive flood hazard assessment undertaken by SKM and Kapiti Coast District Council (KCDC) in 2009.

This report describes the methodology, updates made to the existing model and results to assist the Alliance in undertaking an assessment of hydraulic impacts associated with the expressway.

2. Existing Waimeha Catchment Model

This section provides a brief overview of the critical information relating to the baseline hydraulic model that has been used as the primary tool in the assessment of hydraulic impacts of the expressway. A comprehensive description of the existing model setup is described in the KCDC *Waikane Flood Hazard Mapping Hydraulic Modelling and Mapping Report Vol 1* (SKM, 2010) .

2.1. Hydrology

Hydrological modelling was undertaken using the balanced storm approach that is detailed in the *KCDC Subdivision and Development Principles and Requirements*, KCDC, 2005, with the updated isohyets developed in 2008. The hydrological modelling methodology used has been previously peer reviewed by Beca Infrastructure Ltd (Beca) in the report titled, *Wharemauku Stream Peer Review – Document Review* – September 2008.

The local Waikanae catchment area (Figure 1) covers approximately 2500 ha and is bounded by the Waikanae River to the South and Peka Peka Road to the North. Landuse in the local drainage area is a mixture of built up residential and commercial areas with pasture, horticulture and vegetated areas. The catchment is comprised of several streams which originate on the steep vegetated slopes of Hemi-Matenga Memorial Park entering the residential area to the east of State Highway 1 (SH1).

There are three main streams flowing through the investigation area, the Karkariki, Waimeha and Ngarara Streams. The Karkariki and Waimeha are typical of urban streams in that the channels are well defined and have many bridge and culvert structures along their length. The Karkariki stream has an additional level of complexity in that lengths of the stream are piped while the Waimeha is spring and stormwater fed. The Ngarara Stream has few substantial structures along its length however, the streams location and bed have undergone modification and the channel is not well defined as it passes through the Te Harakeke Wetland.

Within the upper residential areas the streams are routed through the stormwater network before exiting into open drains west of SH1 which confluence into the Ngarara Stream. There is significant fall over this section of the catchment with peak elevations in the upper catchment (eastern end) in the vicinity of 450m above sea level falling to around 30 metres above sea level at SH1. The Ngarara Stream flows through the Te Harakeke Wetland before it joins the Waimeha Stream upstream of Fieldway Bridge. Near the coast the grade flattens as the streams pass through the dune environment.

Figure 1shows the extents, sub-catchment definition and modelled waterways.

2.2. Hydraulic Model

The existing hydraulic model was developed for KCDC as part of a flood hazard assessment for the area and has previously been peer reviewed by Danish Hydraulic Institute (DHI). The model comprised of a combined 1D and 2D hydraulically linked model using the DHI software package MikeFlood.

This modelling technique allows for the maximising of the strengths of both the 1D and the 2D packages. 1D models are able to accurately simulate in channel process and the impacts of structures while 2D models allow for improved modelling of secondary flow paths and dynamic representation of storage on the floodplain. The Waikanae catchment hydraulic models were set up to incorporate the piped stormwater network, open channel flow of the various streams and drains, and surface overland flow of water unconfined to the channel banks or stormwater network. The DHI software programme MOUSE was used to model flow through the piped stormwater network, while an integrated MIKEFlood model combined the channel flow (MIKE11) and surface water on the floodplain (MIKE21).

3. Expressway Modelling Methodology

The model of the Waikanae catchment was used to undertake the comparison of water levels, extents, flows and velocities in the pre and post construction scenarios of the expressway. Specifically there are four potential impacts that are being investigated:

- The change in runoff as a result of the change in land use directly associated with the new expressway project.
- Potential changes in flood storage volumes as a result of earthworks on the floodplain
- Alterations to primary flowpaths through new or altered stream crossings
- Alteration to secondary flowpaths through earthworks on the floodplain

3.1. Modelled Scenarios and Boundary Conditions

Four different storm event scenarios were modelled for both the pre and post construction setup. These events were the 10% AEP (10 year ARI), 1%AEP (100 year ARI), an Extreme event which equates to 1.5 times the flows in a 1%AEP flood and a Waikanae River Stopbank Breach scenario. Table 3.1 summarises the modelled scenarios.

Table 1 Waikanae River Boundaries Conditions

Storm Event	Return Period
10% AEP (10 Year ARI incl. CC)	Primary (no nuisance) Event
1% AEP (100 Year ARI incl. CC)	Design Event
1.5x 1% AEP (1.5x 100 Year ARI incl. CC)	Extreme Case
1% AEP with Waikanae River Stopbank Breach	Breach Scenario

(incl CC = including the predicted impacts of climate change)

All events included an allowance for the predicted midrange impacts of climate change at 2090 in accordance with the KCDC standard practice. This allowance includes a 16% increase in rainfall depths and intensities described in the report *Preparing for Climate Change – A guide for Local Government in New Zealand*, MfE, July 2008.

In all scenarios an oscillating 20 year tidal boundary has been used to model the tidal impacts on flooding. This tidal level has been further increased by 0.8m to allow for the predicted impacts of climate change. The Ministry for the Environment report *Coastal Hazards and Climate Change – A Guidance Manual for Local Government in New Zealand* – 2nd Edition, July 2008 includes the recommendation that for long term planning a 0.8m sea level rise should be expected by the year 2090. The peak of the tidal water levels have been synchronised to coincide with the peak flow rates in the open channel.

3.2. Baseline Model

3.2.1. Hydrology

Runoff from the proposed expressway will typically pass through a range of stormwater devices including collection pits, pipes, swales, treatment and storage devices before discharging into the existing drainage networks. The Alliance has investigated, designed and modelled the runoff from the road footprint. They have supplied the runoff hydrographs for the footprint area and their discharge location for both the pre and post construction scenarios. To incorporate these flows into the hydraulic model of the Waikanae catchment the original sub-catchments and hydrology was reconfigured to accurately reflect the pre and post construction changes and avoid double counting the runoff. Figure 1 shows the updated subcatchment areas and the expressway footprint. Some catchments in the vicinity of the expressway are considered 'Volcano catchments' as a result of their topography, (characterised by a low point surrounded by higher ground from which there is no drainage to a water course) resulting in all rainfall runoff being confined within these catchments.

In addition to local rainfall the model was used to investigate the impacts of the expressway on potential breaches of the Waikanae River that are considered to be a risk in the area around Puriri Road and Jim Cooke Memorial Park. In consultation with Greater Wellington Regional Council the location and magnitude of a stop bank breach has been developed. Our modelling has included a breach located as shown in Figure 2 and peaking at around 23m3/s and was run coincident with a 1% AEP event in the Waimeha Stream. Further information relating to the breach is included in the report Hydraulic Impacts of Makays to Peka Peka Expressway- Mazengarb Stream to Waikanae River, RiverEdge Consulting, September 2011.

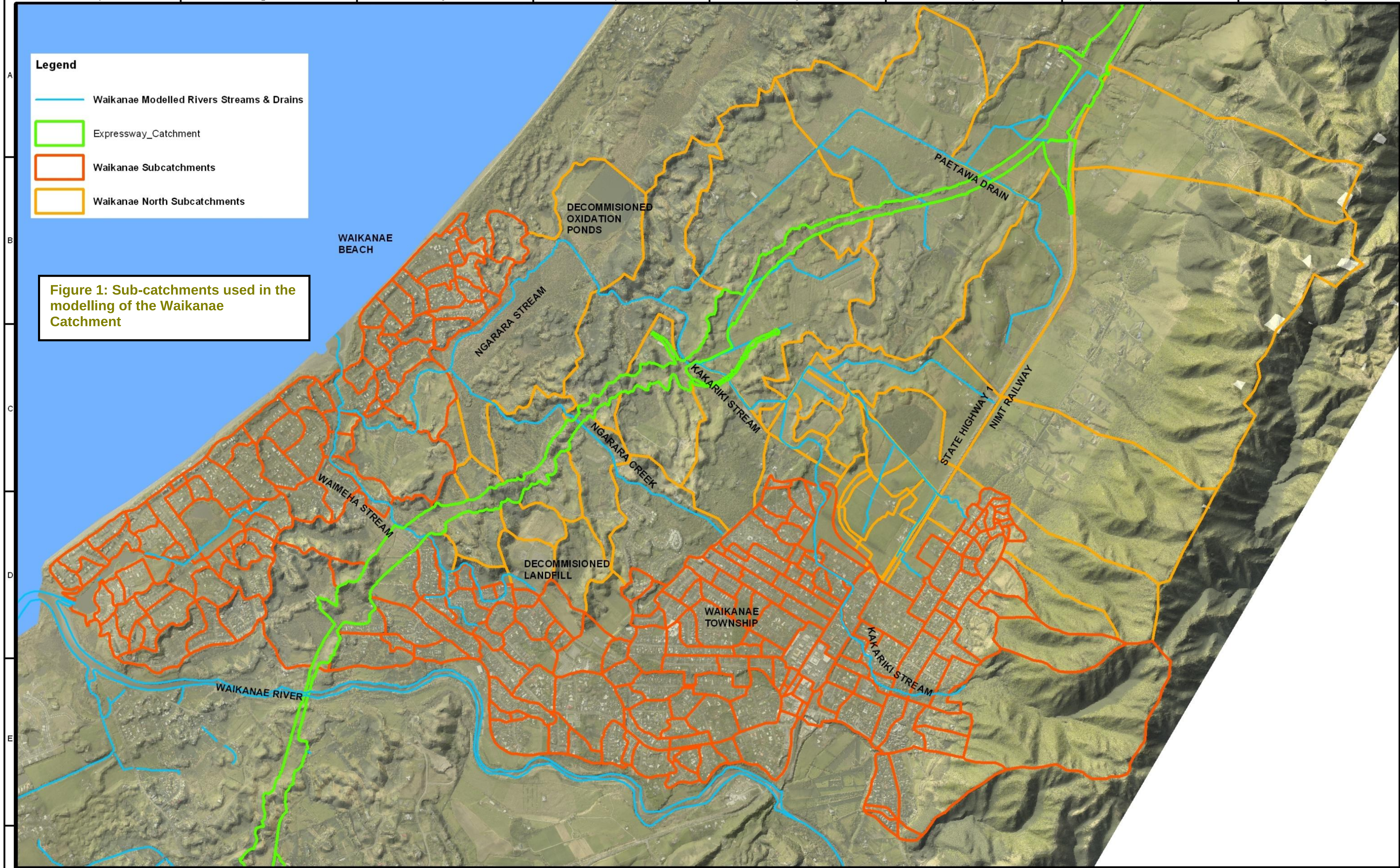

Figure 1: Sub-catchments used in the modelling of the Waikanae Catchment

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DESIGNED	DESIGN REVIEW		

TITLE WAIMEHA UPDATED MODEL CATCHMENTS			
SCALE 1:22,500	SKM PROJECT No AE04038	DRAWING No	AMDT

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3.3. Post Construction Model

To prepare the post construction scenario the following changes were made to the baseline model.

3.3.1. New Crossings

Within the Waikanae catchment the proposed expressway crosses six streams using bridge and culvert structures as well as several additional culvert structures to maintain overland flow paths. The baseline MIKE11 model was updated to reflect the new drain and stream crossing structures as well as some changes to existing channels to incorporate proposed new flood storages and diversions. A model schematic showing the key model updates is shown in Figure 2.

The Waimeha Stream crossing consists of three bridge structures servicing the South bound slip road, Main expressway and North bound slip road. The north and south bound slip road bridges are designed with a 30m span crossing the stream and include 1200mm diameter piers in the centre. The expressway crossing bridge is approximately150m long and is designed to encompass the whole stream cross-section, Te Moana Road and a floodway. The bridge has been incorporated into the hydraulic model using a combination of a MIKE11 bridge structure to allow for the simulation of the hydraulic losses around the piers and altering the MIKE21 bathymetry to incorporate the design floodway. The stream bed in the area around the structures is designed to include riprap to reduce the erosion risk and this has been reflected in the model by increasing the bed resistance at the relevant locations. The bridge deck has not been included in the model as this has been designed to be well above the peak water levels in the extreme scenario (1%AEP x1.5).

The Kakariki Stream expressway bridge crossing consists of one bridge structure designed with a 30m span and a length of 60m. The 1D model has been adjusted to reflect the designed trapezoidal channel geometry and road way on the left bank. The stream bed around the structure has been designed to include riprap to reduce erosion and this has been reflected in the model by increasing the relative bed resistance in the relevant locations. Upstream of the expressway crossing, the local road bridge crossing has not been highlighted as a new structure as the upgraded bridge dimensions match those of the existing bridge.

The Paetawa Stream bridge crossing has been modelled as a 10m wide by 3m deep box structure that includes a low flow channel in the centre. The 1D model has been adjusted to reflect the 70m long structure and low flow channel, as well as increasing the relative bed resistance to represent the riprap scour protection.

The remaining stream expressway crossings are represented in the 1D model as culverts. The details of these structures are presented in Table1.

Table 2 Modelled Waimeha Culvert Structures

Name	U/S Invert	D/S Invert	Shape	Size (m)	Length (m)	Manning's coefficient	Head Loss Factor Inflow	Head Loss Factor Outflow
Culvert 24	2.50	2.40	Circular	1.05	15	0.013	0.5	1
Culvert 24.1	0.78	0.68	Circular	1.05	15	0.013	0.5	1
Culvert 24.2	2.90	2.80	Circular	1.05	15	0.013	0.5	1
Culvert25.3	4.00	3.55	Circular	0.6	70	0.013	0.5	1
Culvert26	4.90	4.71	Rectangular	3x2	70	0.020	0.5	1
Culvert27	7.10	5.80	Circular	0.6	65	0.013	0.5	1
Culvert34	6.20	6.20	Circular	1.5	70	0.013	0.5	1
Culvert 35	6.14	6.07	Circular	1.5	60	0.013	0.5	1
Culvert35.1	7.70	7.65	Circular	1.8	80	0.013	0.5	1
Bridge36	6.99	6.91	Rectangular	10x3	70	0.013	0.5	1
Culvert38	7.58	7.40	Rectangular	3x2	70	0.013	0.5	1
Culvert24.4	4.00	3.95	Circular + flood Valve	0.3	10	0.013	0.5	1

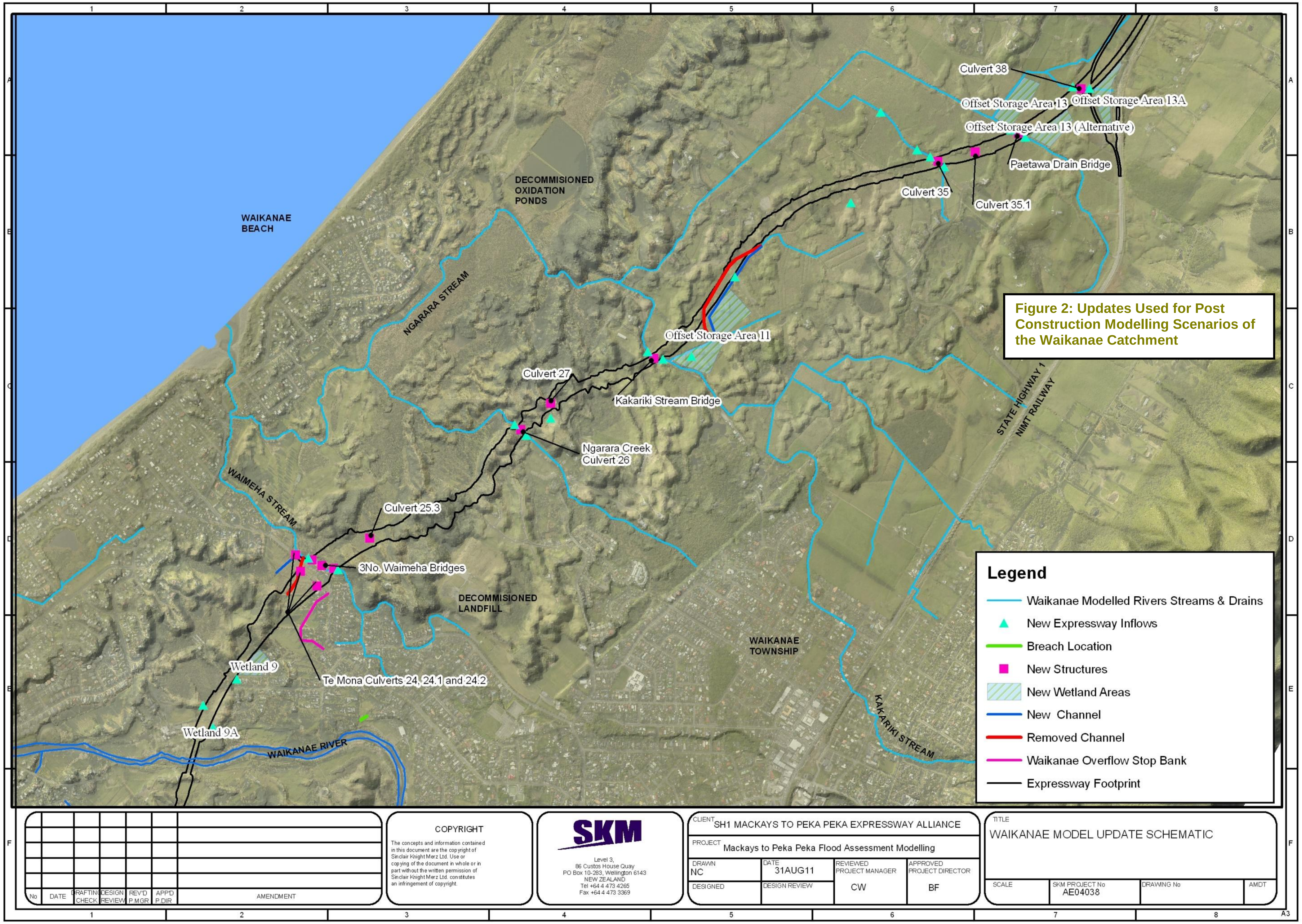
Figure 2: Updates Used for Post Construction Modelling Scenarios of the Waikanae Catchment
WAIKANAE BEACH
DECOMMISIONED OXIDATION PONDS
NGARARA STREAM
WAIMEHA STREAM
DECOMMISIONED LANDFILL
WAIKANAE TOWNSHIP
WAIKANAE RIVER
KAKARIKI STREAM
STATE HIGHWAY 1
NIMT RAILWAY
Culvert 38
Offset Storage Area 13
Offset Storage Area 13A
Offset Storage Area 13 (Alternative)
Paetawa Drain Bridge
Culvert 35
Culvert 35.1
Offset Storage Area 11
Culvert 27
Kakariki Stream Bridge
Ngarara Creek Culvert 26
Culvert 25.3
3No. Waimeha Bridges
Wetland 9
Te Mona Culverts 24, 24.1 and 24.2
Wetland 9A
Legend
Waikanae Modelled Rivers Streams & Drains
New Expressway Inflows
Breach Location
New Structures
New Wetland Areas
New Channel
Removed Channel
Waikanae Overflow Stop Bank
Expressway Footprint
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DESIGNED
DESIGN REVIEW
TITLE WAIKANAE MODEL UPDATE SCHEMATIC
SCALE
SKM PROJECT No AE04038
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3.3.2. Earthworks on the Floodplain

The modelled floodplain was updated to reflect the proposed changes to the existing drains and streams. The expressway alignment was also included in the 2D domain as an area of land elevated above the predicted adjacent flood levels. This reflects the design elevation of the expressway which is set above the 1%AEP flood level. Within the model flows on the floodplain can only pass through the expressway alignment under bridges or through culverts.

Through a number of modelling iterations the Alliance has developed and designed a range of mitigation measures to address the impacts of the proposed expressway on the flooding. This primarily involved modifications to the topography to recreate flow paths and flood storage that had been altered by the expressway. The modified areas of new flood storage and wetlands are identified in Figure 2 and detailed in Table 2 below. These flood storage areas are provided in addition to any treatment and attenuation provided as part of the expressway stormwater management.

Table 3 Additional Wetland and Flood Storage Compensation

Storage Location	Average Existing Ground Level (m MSL)	Average Modified Ground Level (m MSL)	Storage / Wetland Area (m^2)	Additional 1%AEP Flood Storage (m^3)
9	Included as part of the Expressway Stormwater management			
9A	Included as part of the Expressway Stormwater management			
11	6.00	5.49	51,000	25,000
13A	9.17	8.46	18,000	30,000
13	7.77	7.57	36,000	6,000

Wetland 9 and 9A have been sized and designed by the Alliance and integrated into the Expressway storm water management. The subsequent storm hydrograph outflow from these wetlands has been applied to the model and included into the existing modelled storm water network. The net impact of the modified Wetland 9 and associated outflows has been to reduce peak flows entering the stormwater network in Puriri Road and Kauri Road.

4. Modelling Results

Flooding extents, levels, flows and velocities have been extracted for the Q10, Q100 and the Extreme event (Q100 x 1.5). The pre and post construction results have been compared to assist the Alliance in undertaking an assessment of flooding effects. The results are reported in Appendix A as Pre and Post inundation flood maps while an explanation of the hydraulics in specific areas is discussed in further detail in this section.

4.1. Change in Runoff as a Result of Change in Land Use

The Alliance has supplied the pre and post construction hydrographs for the road alignment and the approximate location of discharge into the surrounding drainage network. The locations of the discharges are shown in Figure 2. It should be noted that these discharges are from the expressway stormwater management system and have been attenuated prior to discharge, and that a nominal target reduction in peak flow to 80% of Pre construction flow has been achieved in all but a few locations. These flows will be further refined during detailed design.

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4.2. New Water Course Crossings

The impacts to flows and water levels in Waimeha, Ngarara, Kakariki and Paetawa watercourses as a result of the expressway have been assessed and the modelled results are presented below in Table 4 and Figures 3 to Figure 6.

Table 4 Peak Water Levels and Discharge for the Expressway Structures

WAIMEHA BRIDGE	10%AEP-PRE	10%AEP-POST	1%AEP-PRE	1%AEP-POST	1.5x 1%AEP-PRE	1.5x 1%AEP-POST
WL (m)	3.01	2.94	3.16	3.09	3.37	3.28
Q (m³/s)	5.0	5.1	9.4	9.7	14.6	14.5
NGARARA CREEK CULVERT - 26						
WL (m)	6.09	5.97	6.10	6.14	6.37	6.37
Q (m³/s)	2.4	1.8	2.5	2.2	2.9	2.9
KAKARIKI BRIDGE						
WL (m)	6.05	6.02	6.33	6.30	6.62	6.53
Q (m³/s)	16.4	16.4	19.8	21.9	23.9	27.9
CULVERT 35						
WL (m)	7.24	7.11	7.46	7.34	7.73	7.52
Q (m³/s)	1.2	1.2	1.6	1.8	2.7	2.5
PAETAWA DRAIN						
WL (m)	8.81	8.80	9.21	9.21	9.50	9.40
Q (m³/s)	12.5	12.4	23.8	23.9	35.8	35.9
CULVERT 38*						
WL (m)	8.70	8.24	8.41	8.34	8.75	8.76
Q (m³/s)	0.6	0.7	0.7	0.9	0.8	2.4
CULVERT 24.4						
WL (m)	-	4.07	-	4.13	-	4.16
Q (m³/s)	-	0.01	-	0.02	-	0.025
CULVERT 25.3						
WL (m)	-	4.33	-	4.52	-	4.65
Q (m³/s)	-	0.12	-	0.26	-	0.38
CULVERT 27						
WL (m)	-	7.31	-	7.40	-	7.53
Q (m³/s)	-	0.05	-	0.10	-	0.15
CULVERT 35.1						
WL (m)	-	8.02	-	8.23	-	8.54
Q (m³/s)	-	0.22	-	0.6	-	1.5

- Note values for culvert 38 are taken down stream as upstream of the culvert is proposed as flood offset storage area 13A.

Overall the modelling indicates that there has been no significant impact to water levels and peak discharges in the vicinity of the new structures. Long section profiles of the relevant stream sections showing pre and post construction peak water levels and discharges are presented in Appendix B. Water levels in the stream and flood plain have been compared for Pre and Post construction scenarios with the results indicating that there is little impact to peak flood levels.

4.3. Changes in Secondary Flowpaths and Floodplain Storage

Overland flows on the flood plain have been altered in several locations as a result of the new expressway. Culverts 25.3, 27 and 35.1 have been included to replicate existing overland flow paths and maintain existing flood extents. Figure 3 show the mitigation design for the Waimeha Stream area. Included in this area is Wetland 9. This wetland has been designed as part of the Expressway stormwater management system and includes some additional flood storage and attenuation. The new attenuated outflow from this wetland has been included in the model and results predict a reduction in flood risk from the stormwater network at Puriri and Kauri Roads.

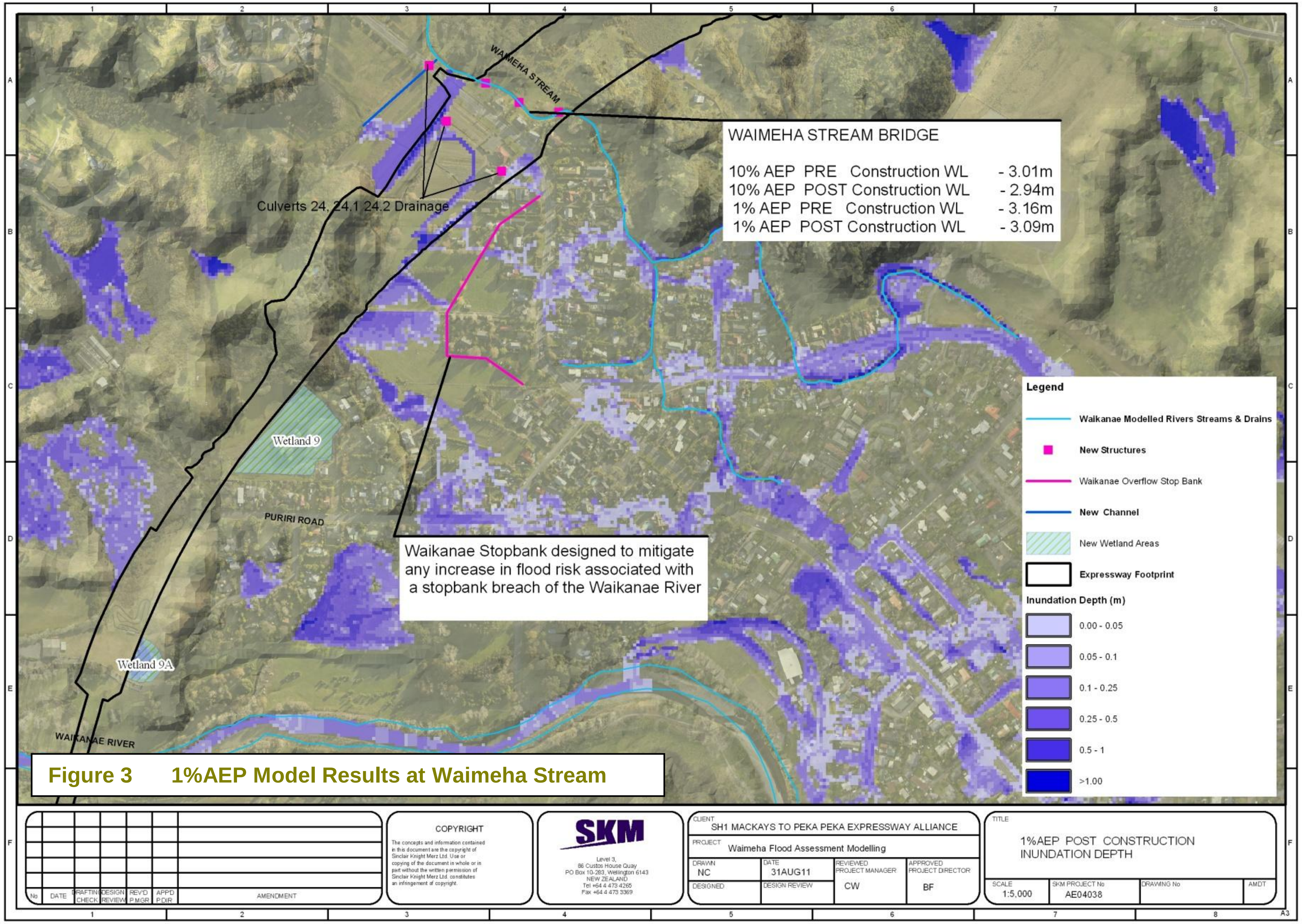

Figure 3 1%AEP Model Results at Waimeha Stream

Other key features to help mitigate potential adverse flooding impacts are the proposed flood bund and the floodway under the Waimeha Stream Bridge, also shown in Figure 3. These have been designed though various model simulations of predicted flows from a breach of the Waikanae River Stopbank. A comparison of the Pre and Post construction scenarios, using a breach flow that peaks at about 23m^3/s, indicates that the expressway will alter the path of this flow over the floodplain, see figures in Appendix A. The combination of the stopbank, the Waimeha Bridge floodway and appropriate design of Te Moana interchange has shown to largely confine the adverse impacts to the area between the proposed stopbank and the expressway, see Figure 4.

Figure 5 shows the flooding in the vicinity of the Ngarara Creek. The model results indicate no significant change to upstream or downstream flood risk as a result of the expressway with no significant impact to peak water levels and flows predicted. Two volcano style catchments that do not drain to any watercourse are bisected by the Expressway. These catchments have been linked via new culverts (25.3 and 27) to maintain any existing flows paths.

Flood impacts around the Kakariki have been modelled and are displayed in Figure 6. The Expressway is not predicted to have any impacts to flows in the Kakariki but does impact the adjacent flood plain and drain. To compensate for this the drain alignment has been modified and some compensation storage has been provided by the creation of offset storage area 11. A comparison of Pre and Post construction peak inundation shows no predicted adverse impacts upstream or downstream of the Expressway in a 1%AEP flood event. Modelled ground elevations in Area 11 have been reduced to provide 25000m^3 of additional storage with bank heights set so the storage functions during a 10%AEP flood event as described in Figure 6.

Flooding and flood mitigation results for the Paetawa Drain are shown in Figure 7. The model results indicate no significant change to upstream or downstream flood risk as a result of the expressway in the 1%AEP flood event. The Expressway bisects a key overland flow path and this has been mitigated though the addition of culvert 35.1. This Culvert has been designed to take the overland flow from the Paetawa Drain and allow it to maintain its original flow path South. The resulting loss of flood plain storage has been compensated for through additional storage and attenuation provided by offset storage areas 13 and 13A. Area 13A has been designed to provide 30,000m^3 of attenuated flood storage through a restricted slot and weir overflow system on the inlet of Culvert 38, as detailed in Figure 7.

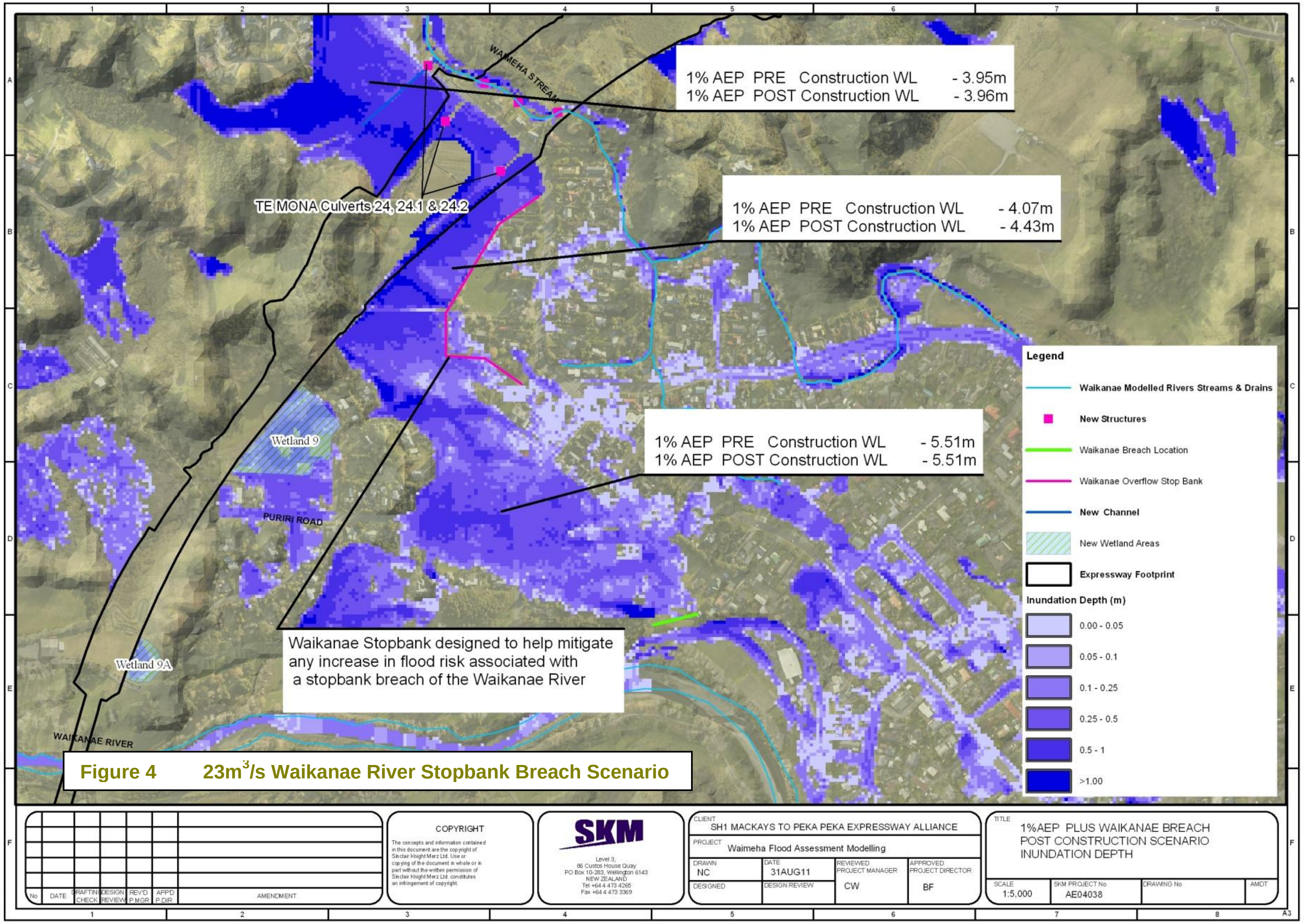

Figure 4 23m³/s Waikanae River Stopbank Breach Scenario

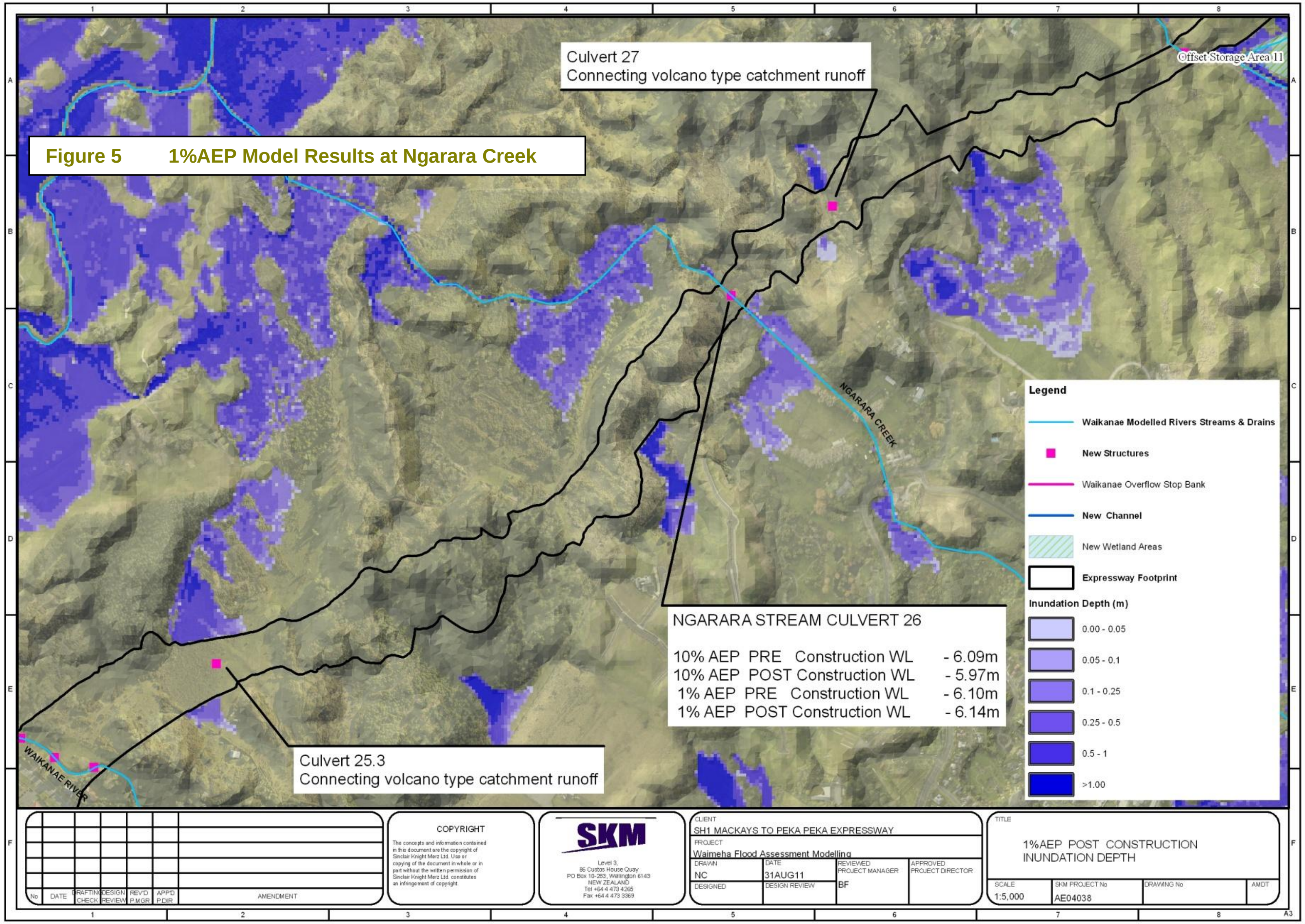
Figure 5 1%AEP Model Results at Ngarara Creek
Culvert 27
Connecting volcano type catchment runoff
Offset Storage Area 11
NGARARA CREEK
NGARARA STREAM CULVERT 26
10% AEP PRE Construction WL - 6.09m
10% AEP POST Construction WL - 5.97m
1% AEP PRE Construction WL - 6.10m
1% AEP POST Construction WL - 6.14m
Culvert 25.3
Connecting volcano type catchment runoff
WAIKANAE RIVER
Legend
Waikanae Modelled Rivers Streams & Drains
New Structures
Waikanae Overflow Stop Bank
New Channel
New Wetland Areas
Expressway Footprint
Inundation Depth (m)
0.00 - 0.05
0.05 - 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 1
>1.00
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DESIGN REVIEW
TITLE
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INUNDATION DEPTH
SCALE
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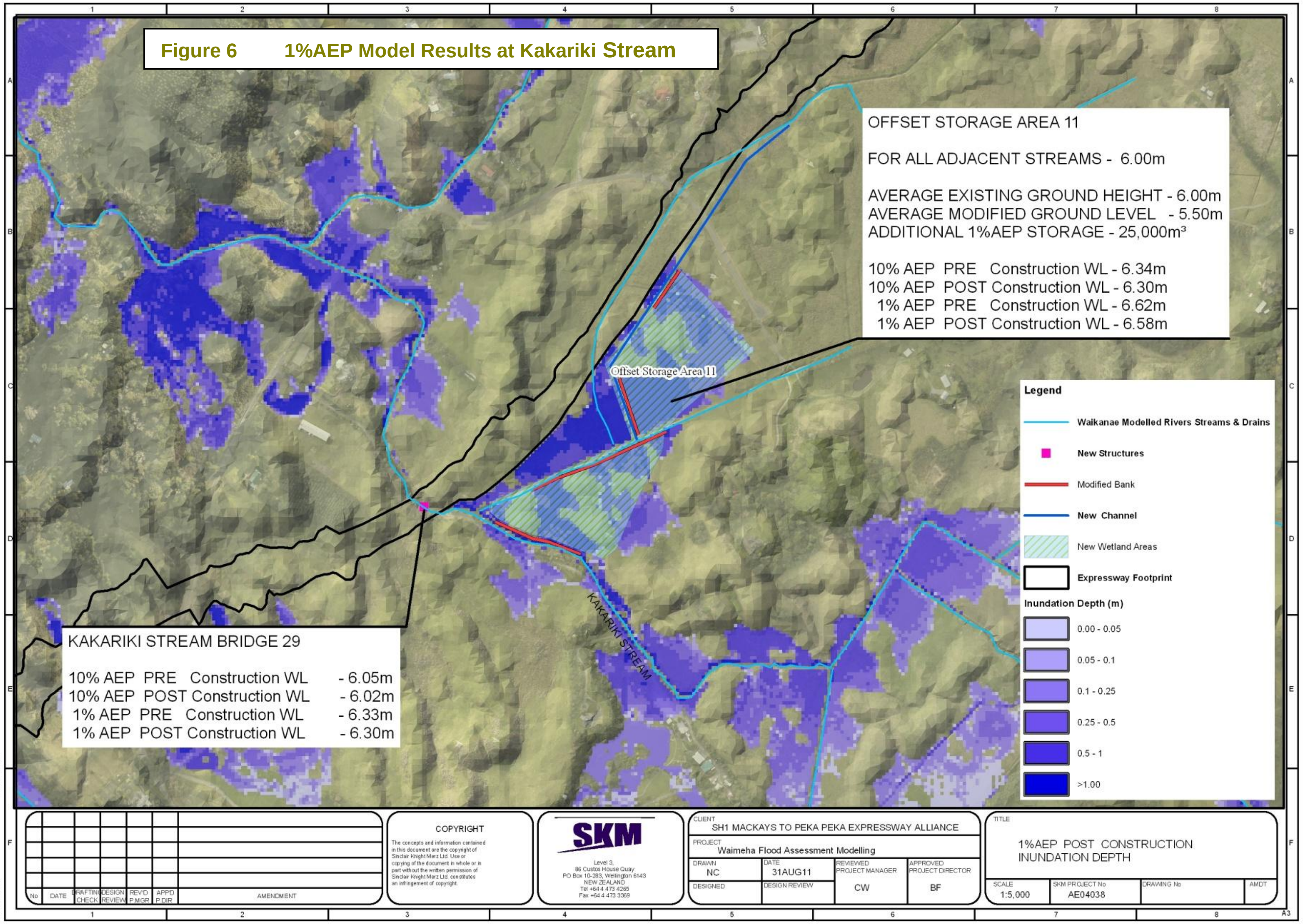
Figure 6 1%AEP Model Results at Kakariki Stream
OFFSET STORAGE AREA 11
FOR ALL ADJACENT STREAMS - 6.00m
AVERAGE EXISTING GROUND HEIGHT - 6.00m
AVERAGE MODIFIED GROUND LEVEL - 5.50m
ADDITIONAL 1%AEP STORAGE - 25,000m³
10% AEP PRE Construction WL - 6.34m
10% AEP POST Construction WL - 6.30m
1% AEP PRE Construction WL - 6.62m
1% AEP POST Construction WL - 6.58m
Offset Storage Area 11
KAKARIKI STREAM
KAKARIKI STREAM BRIDGE 29
10% AEP PRE Construction WL - 6.05m
10% AEP POST Construction WL - 6.02m
1% AEP PRE Construction WL - 6.33m
1% AEP POST Construction WL - 6.30m
Legend
Waikanae Modelled Rivers Streams & Drains
New Structures
Modified Bank
New Channel
New Wetland Areas
Expressway Footprint
Inundation Depth (m)
0.00 - 0.05
0.05 - 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 1
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SCALE 1:5,000
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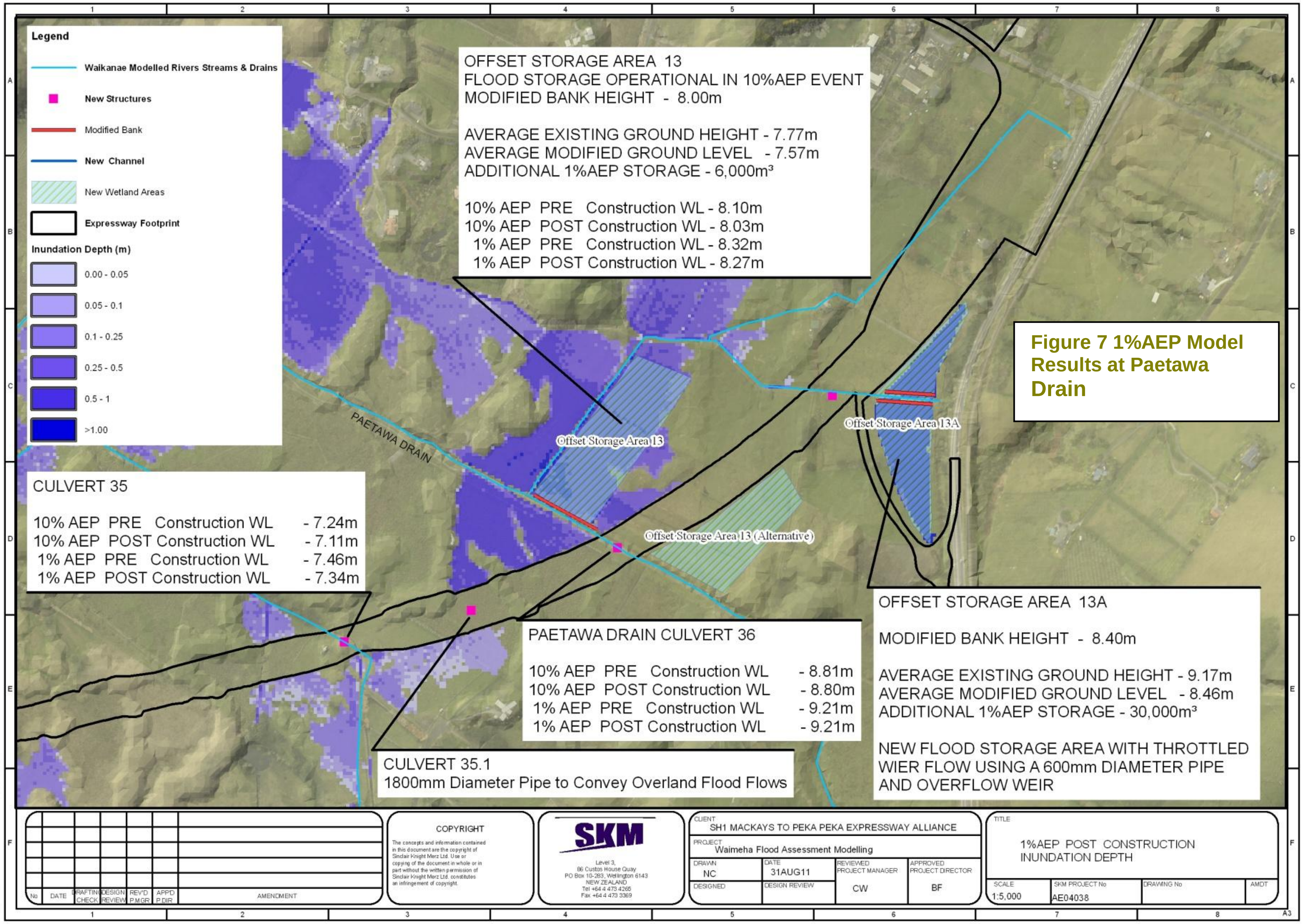
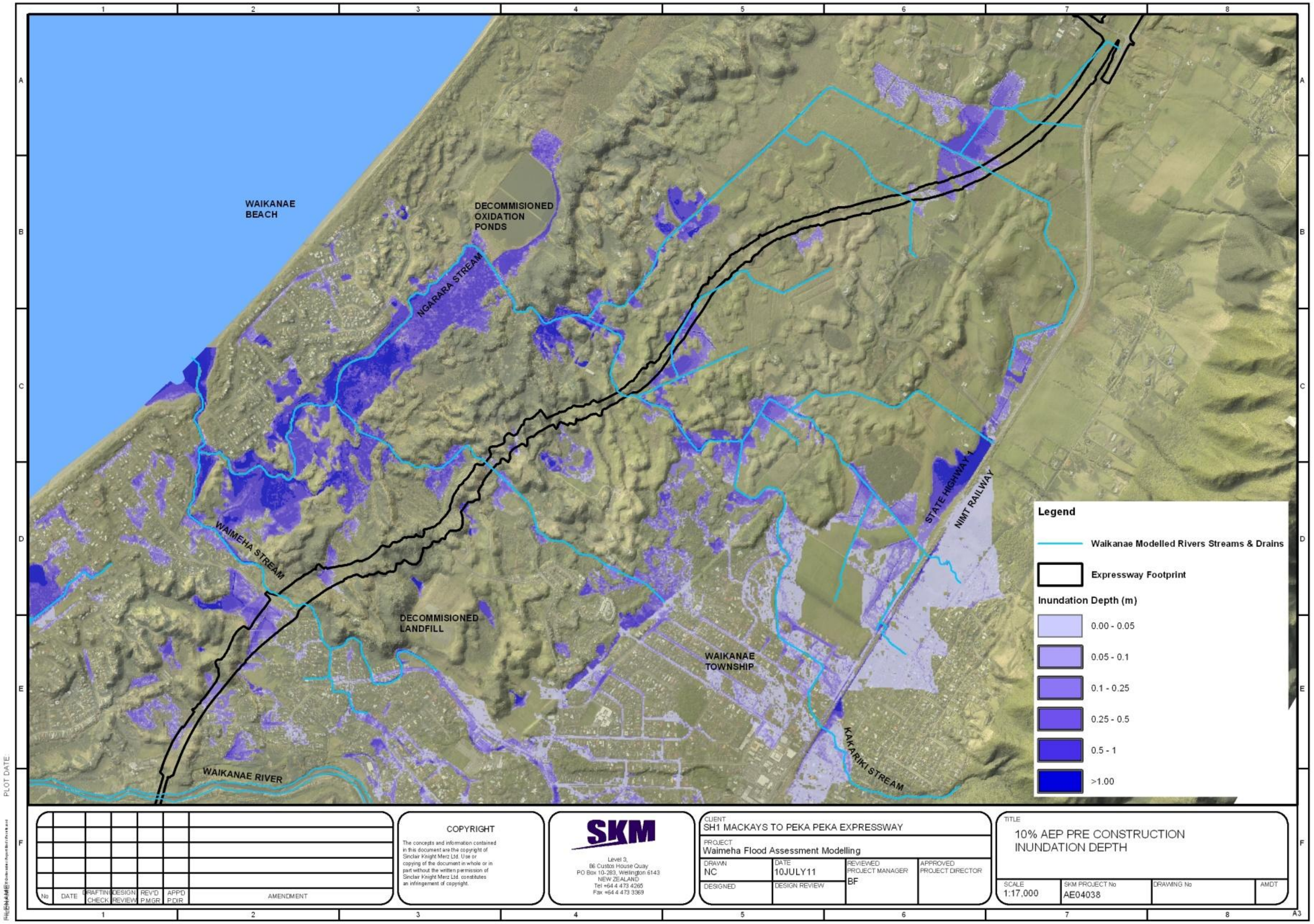


Figure 7 1%AEP Model Results at Paetawa Drain

5. Conclusion

This hydraulic modelling investigation has identified that the proposed expressway will have a complex interaction with the floodplains of the Waimeha and Ngarara Streams. The key issues that have been identified in this area are the alteration of secondary flow paths by the proposed earthworks and the loss of storage on the floodplain. Through a range of mitigation measures the hydraulic model results suggest that the adverse flood risk impacts can be largely mitigated or confined to within the expressway designation. Key mitigation measures include the floodway and stopbank around the Waimeha Stream crossing to guide potential Waikanae River stopbank breaches past the expressway. Another important feature are the wetlands created to provide compensatory storage, particularly around the Paetawa Drain crossing.

Appendix A – Inundation Maps and Comparisons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DECOMMISIONED OXIDATION PONDS
NGARARA STREAM
WAIMEHA STREAM
DECOMMISIONED LANDFILL
WAIKANAE TOWNSHIP
WAIKANAE RIVER
STATE HIGHWAY 1
NIMT RAILWAY
KAKARIKI STREAM
Legend
Waikanae Modelled Rivers Streams & Drains
Expressway Footprint
Inundation Depth (m)
0.00 - 0.05
0.05 - 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 1
>1.00
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DATE 10JULY11
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APPROVED PROJECT DIRECTOR
DESIGNED
DESIGN REVIEW
TITLE
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SCALE 1:17,000
SKM PROJECT No AE04038
DRAWING No
AMDT
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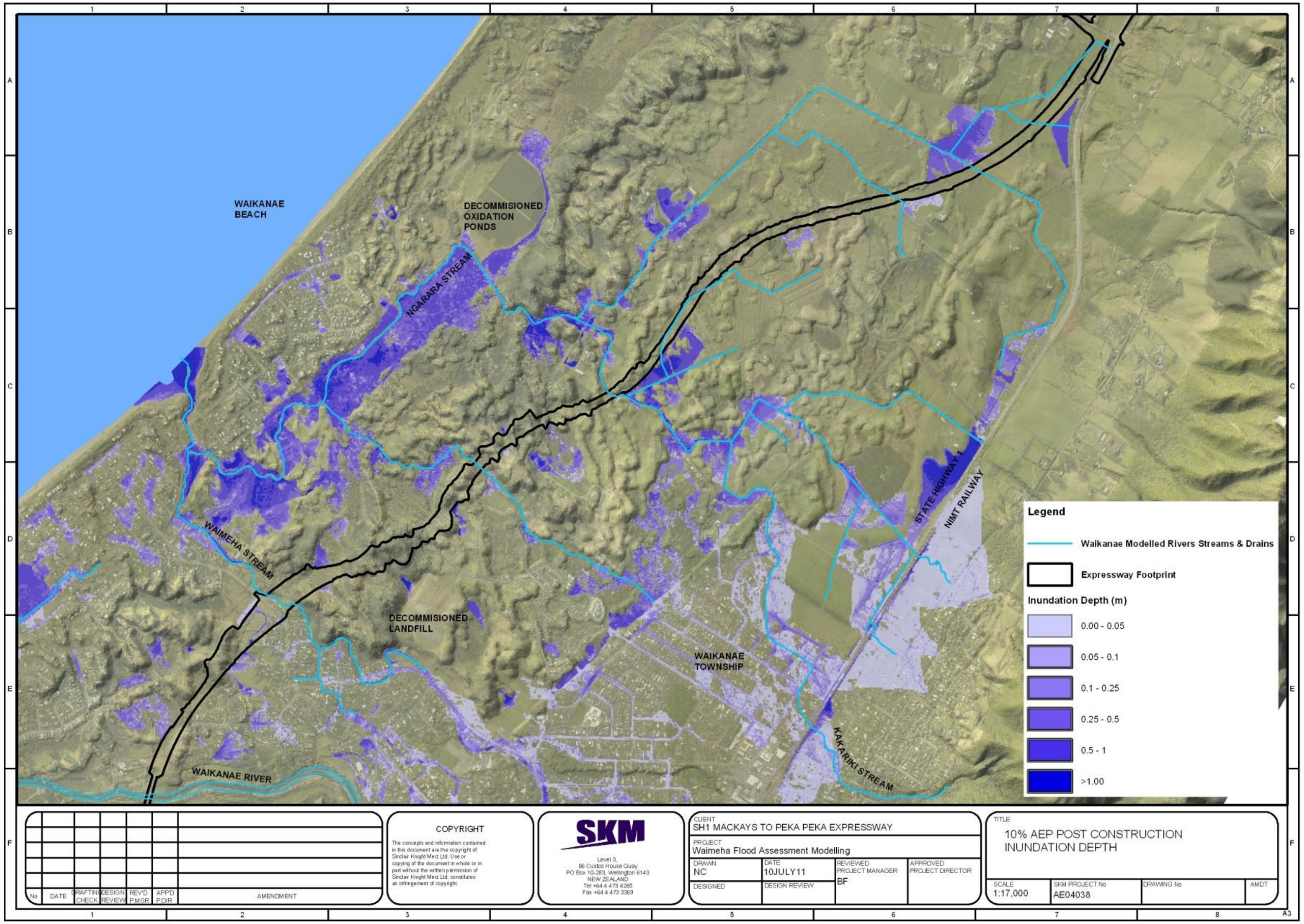
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NGARARA STREAM
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STATE HIGHWAY 1
NIMT RAILWAY
KAKARIKI STREAM
Legend
Waikanae Modelled Rivers Streams & Drains
Expressway Footprint
Inundation Depth (m)
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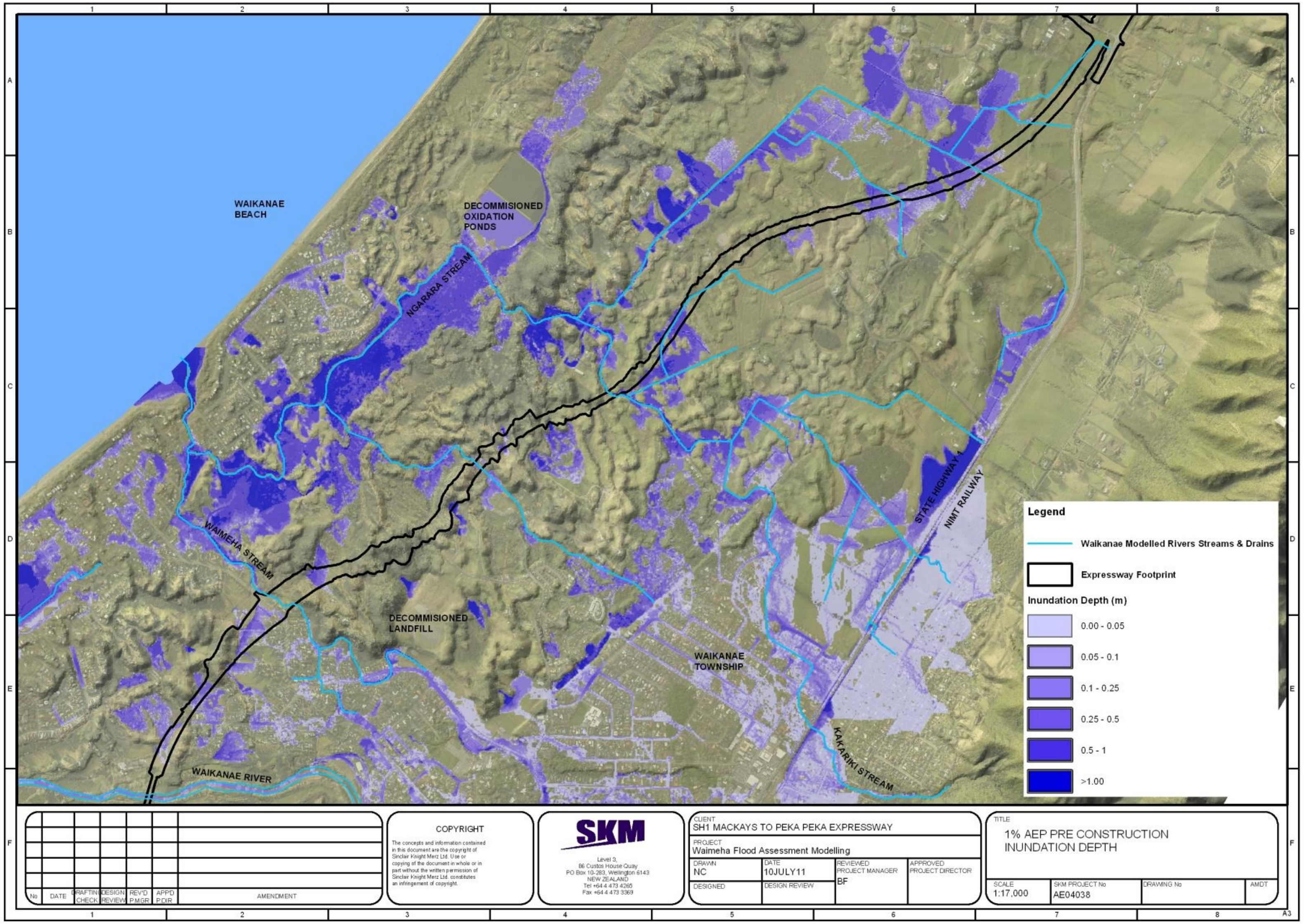
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NIMT RAILWAY
Legend
Waikanae Modelled Rivers Streams & Drains
Expressway Footprint
Inundation Depth (m)
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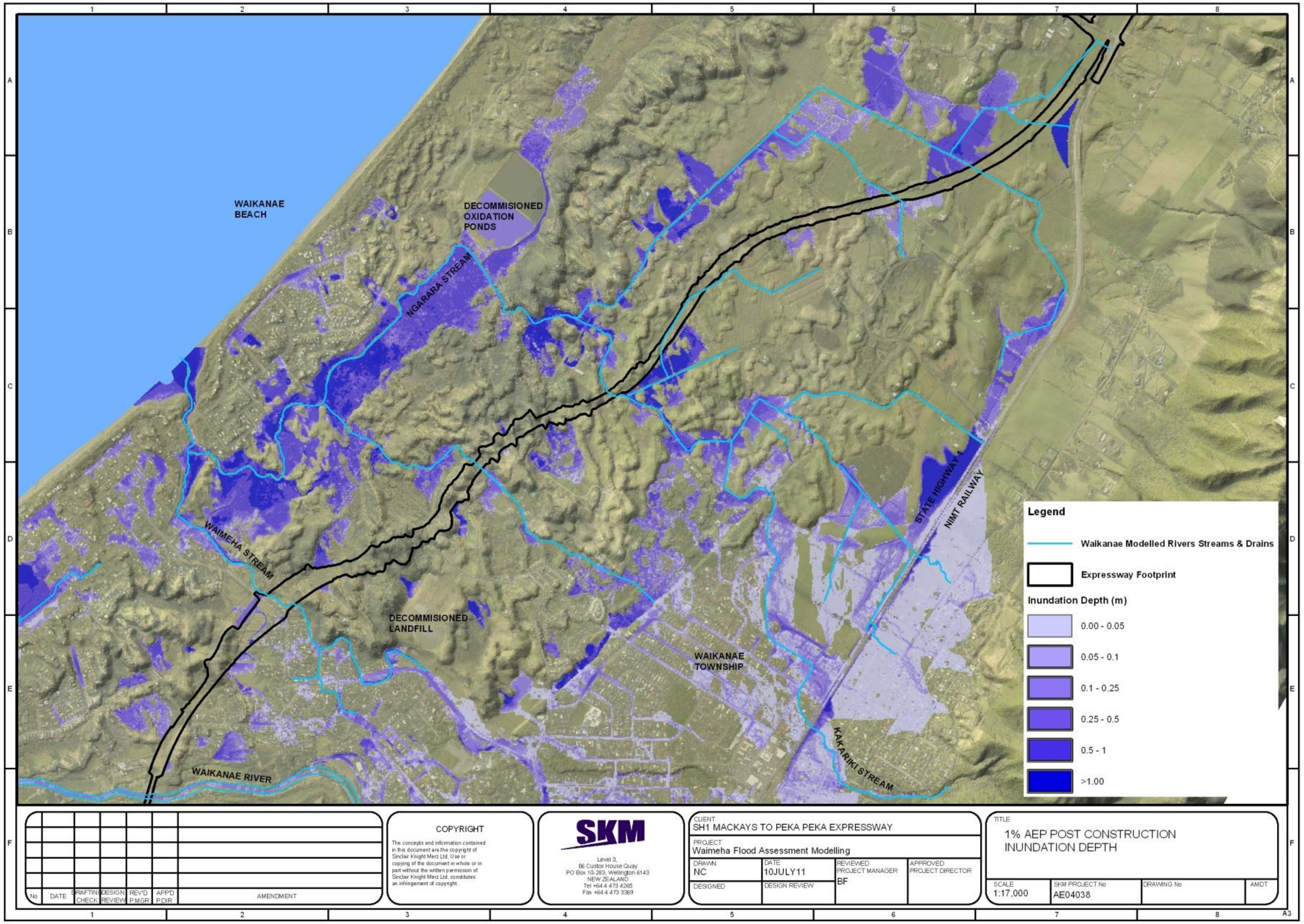
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NIMT RAILWAY
Legend
Waikanae Modelled Rivers Streams & Drains
Expressway Footprint
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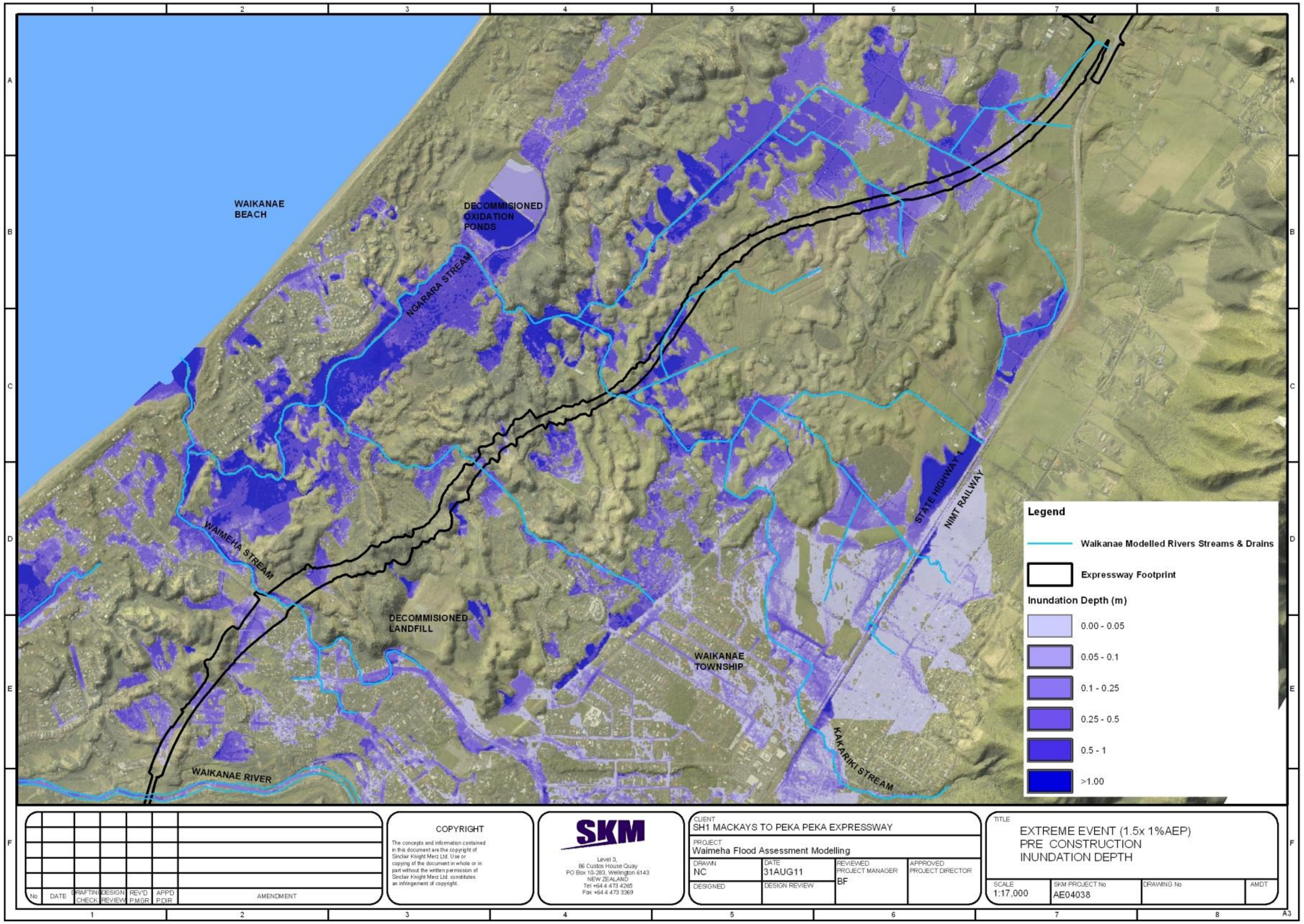

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STATE HIGHWAY 1
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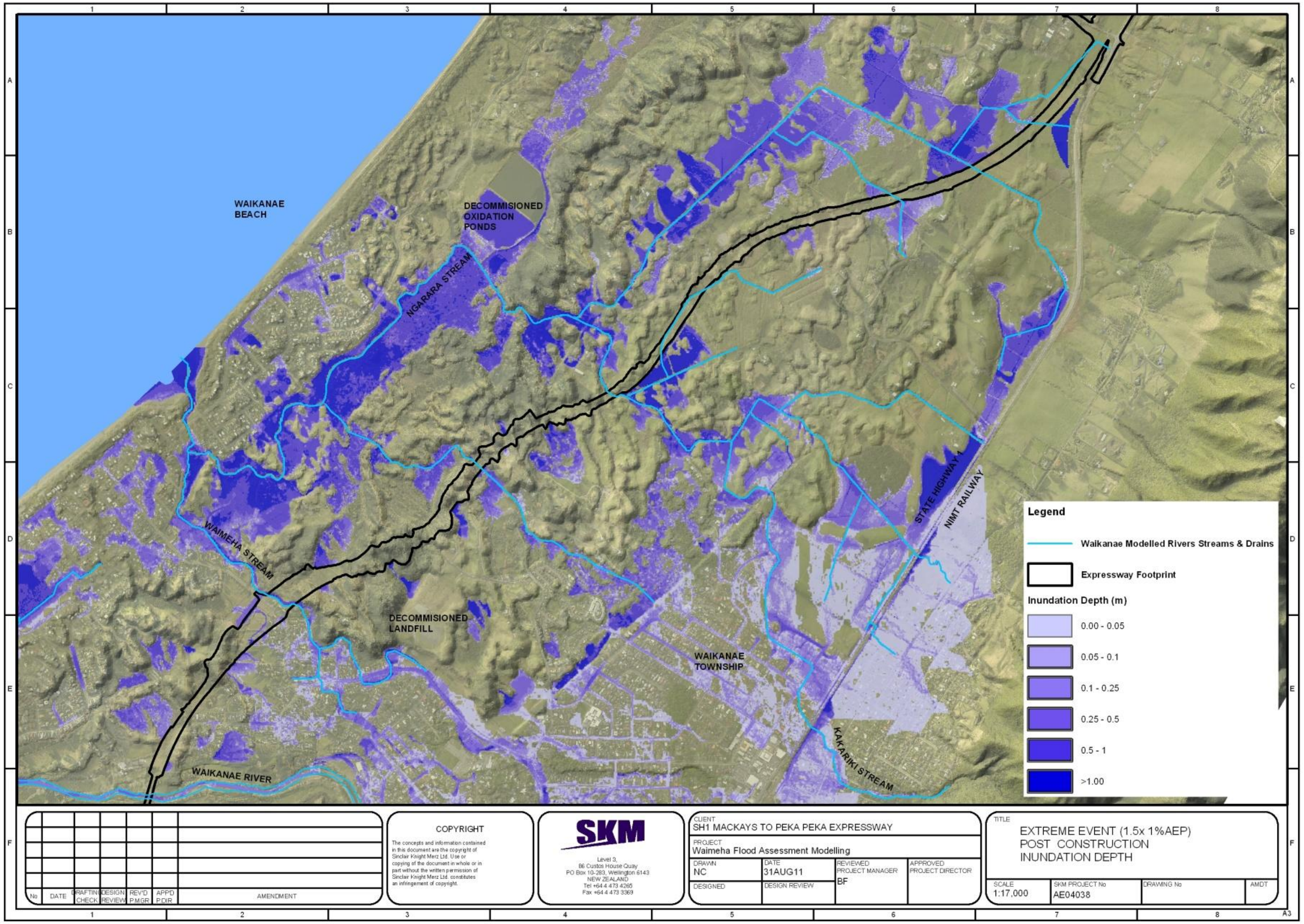
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KAKARIKI STREAM
STATE HIGHWAY 1
NIMT RAILWAY
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Waikanae Modelled Rivers Streams & Drains
Expressway Footprint
Inundation Depth (m)
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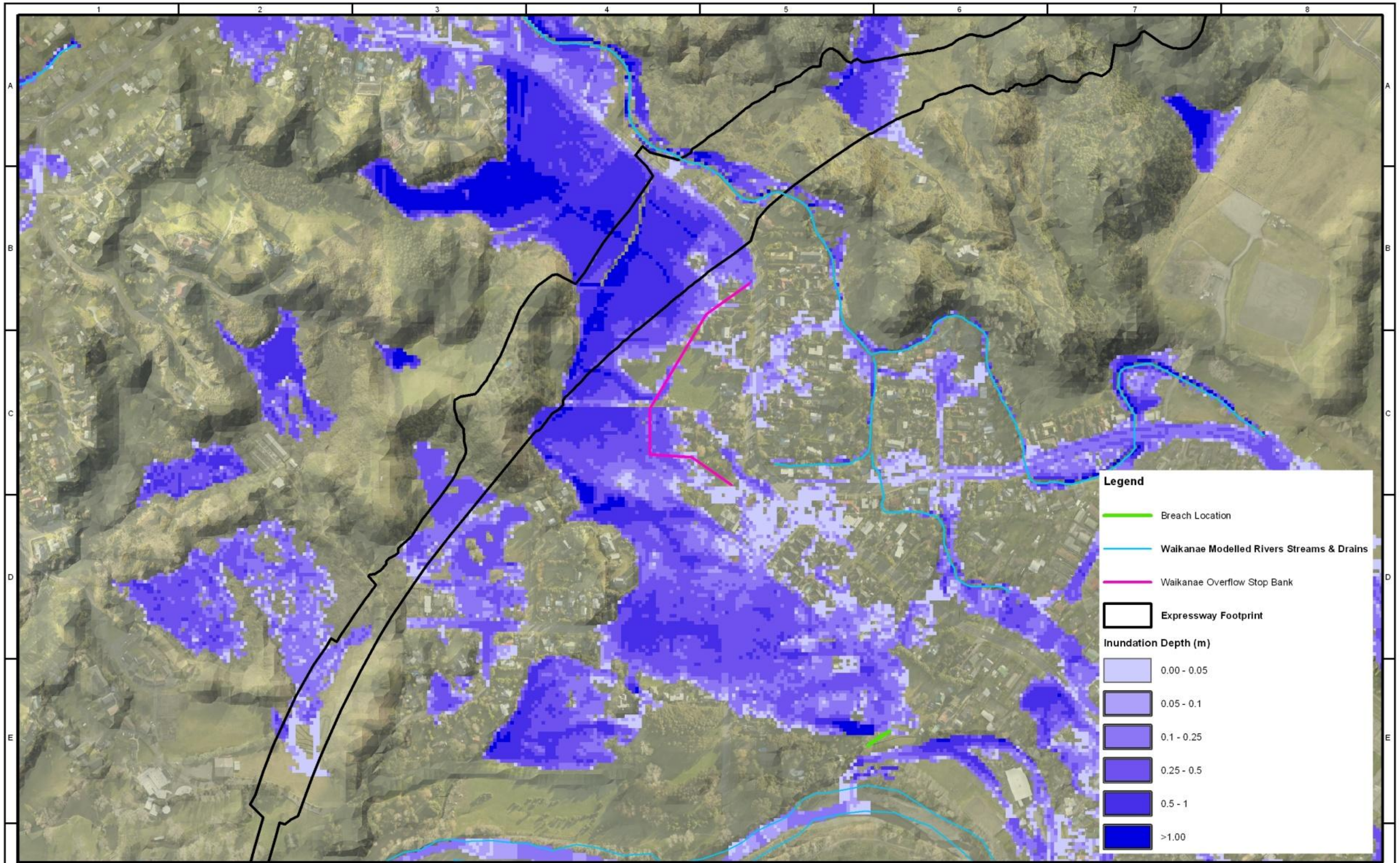

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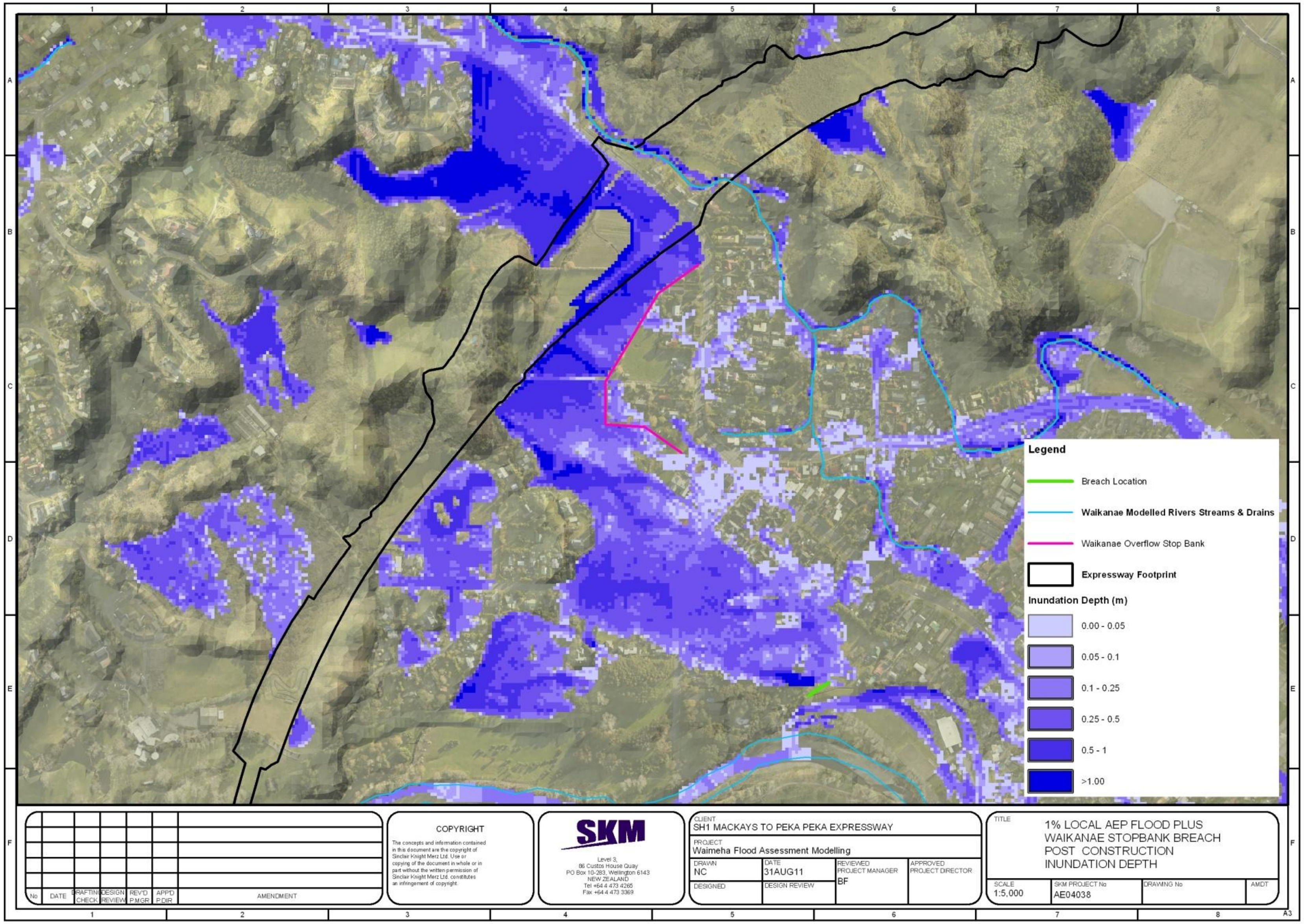

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Appendix B – Long section Profiles

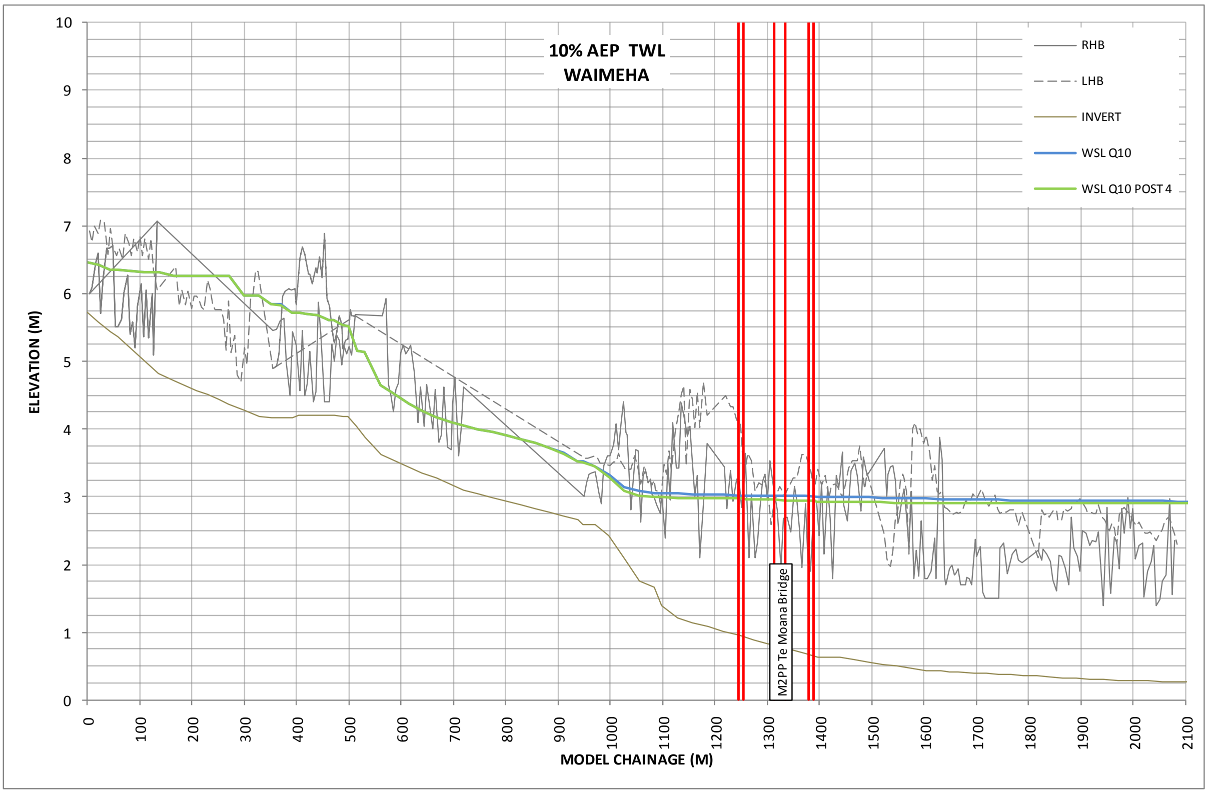

Figure 8 Waimeha- 10%AEP Top Water Level

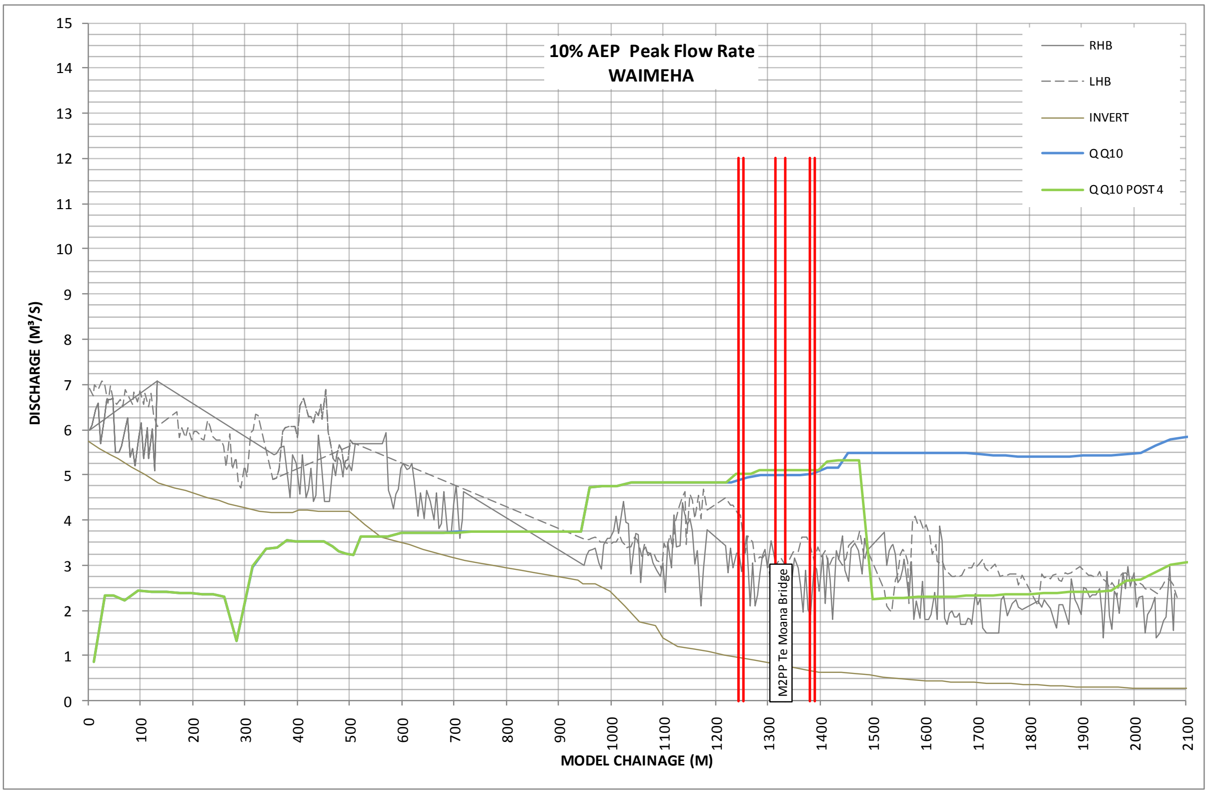

Figure 9 Waimeha - 10%AEP Peak Flow Rate

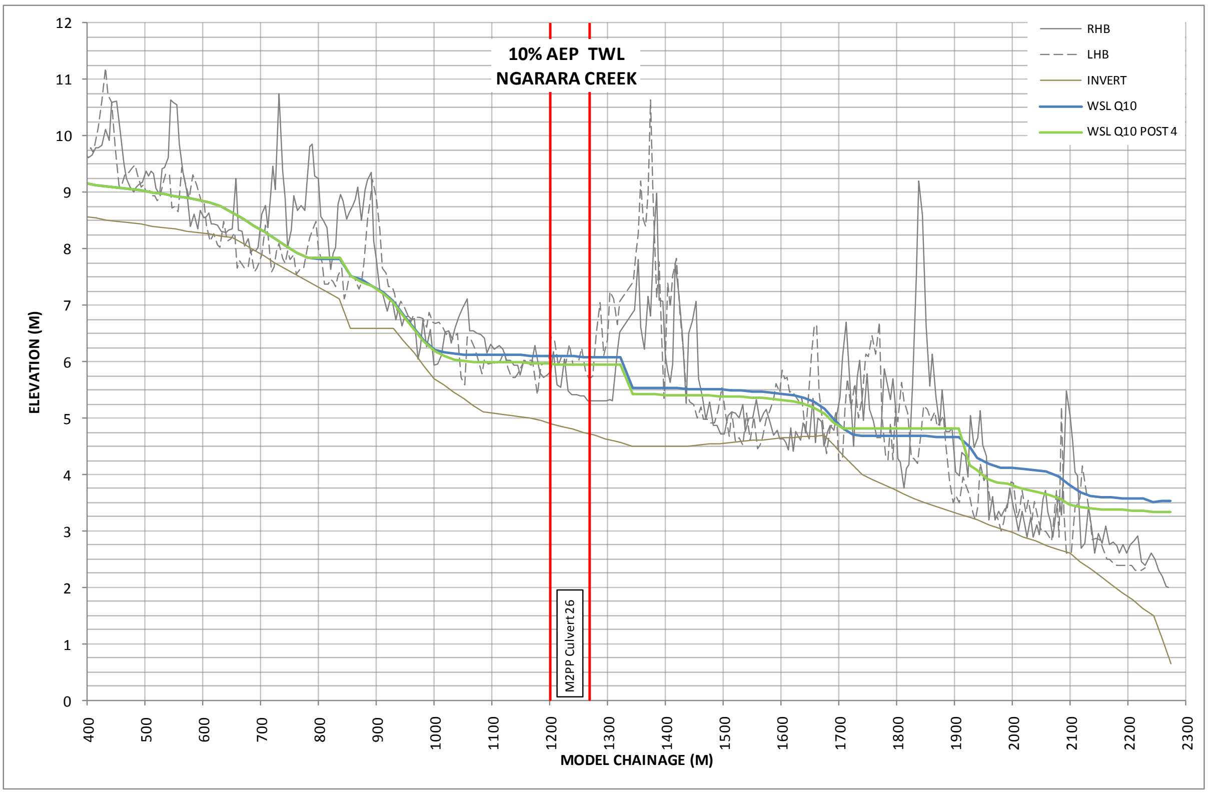

Figure 10 Ngarara- 10%AEP Top Water Level

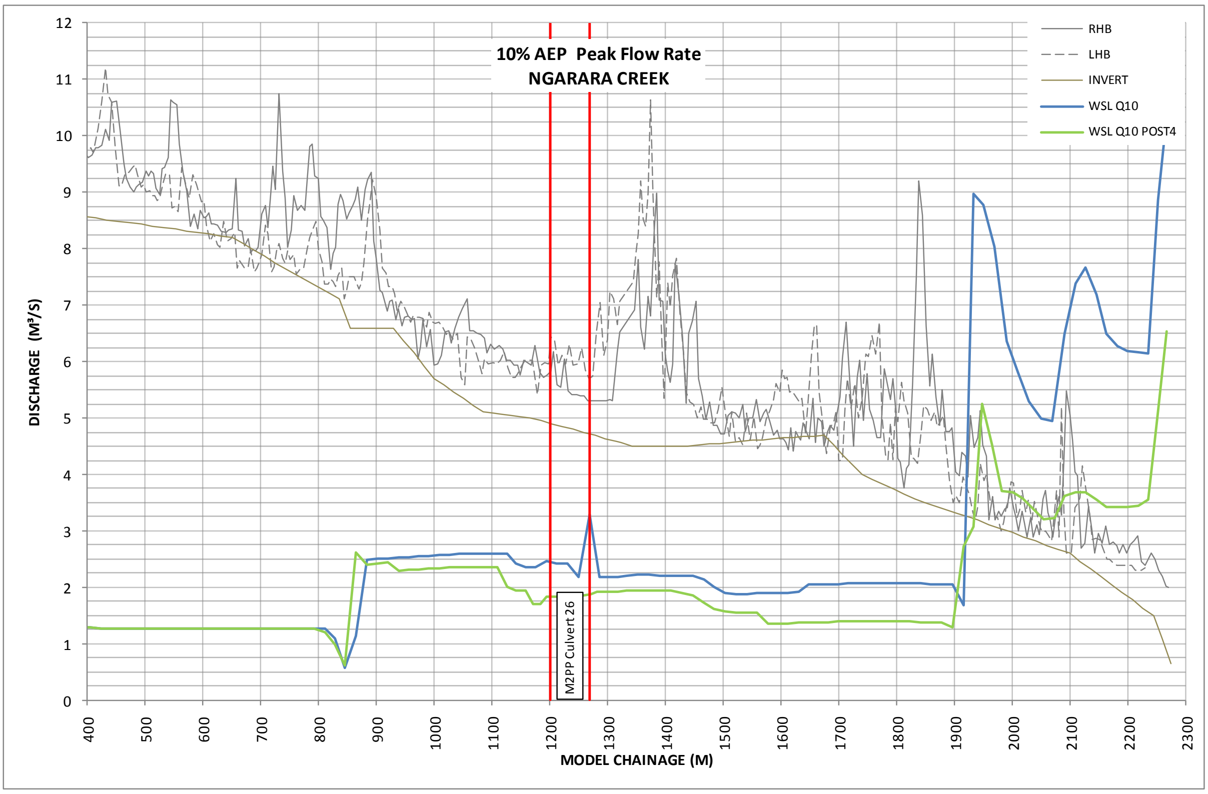

Figure 11 Ngarara- 10%AEP Peak Flow Rate

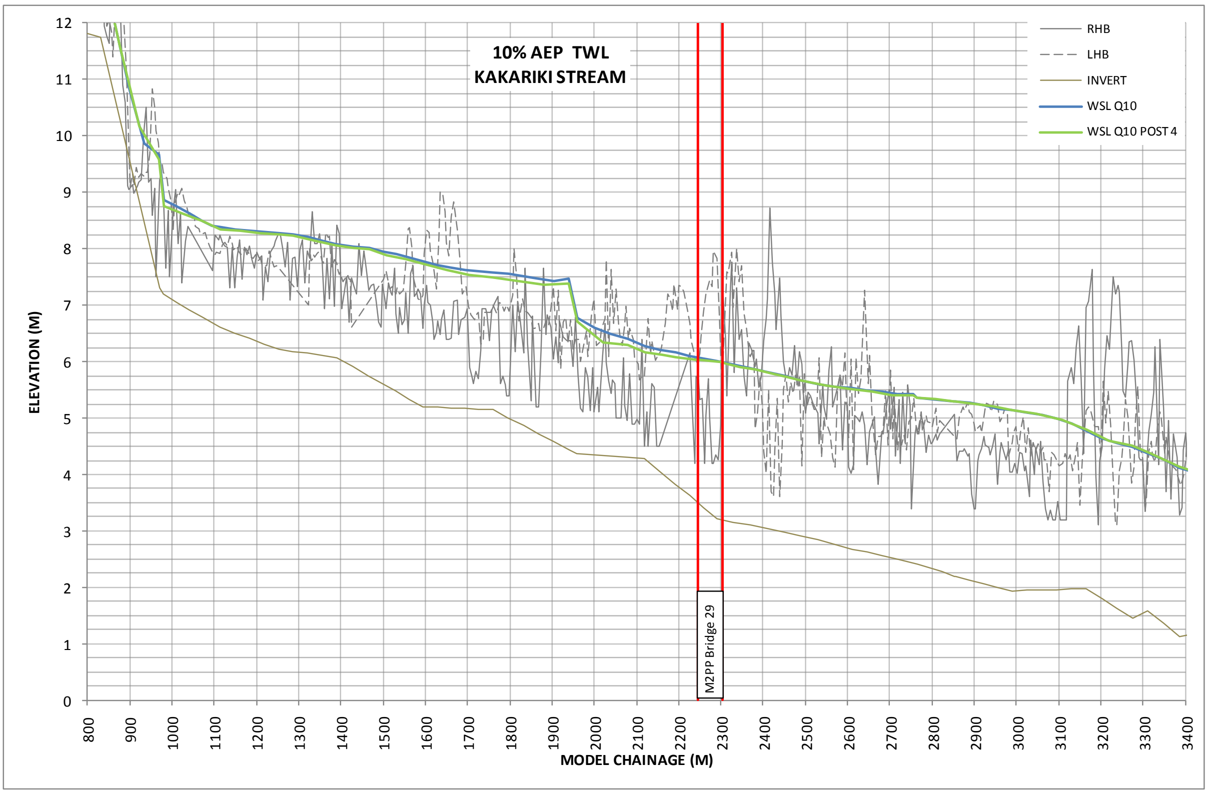

Figure 12 Kakariki- 10%AEP Top Water Level

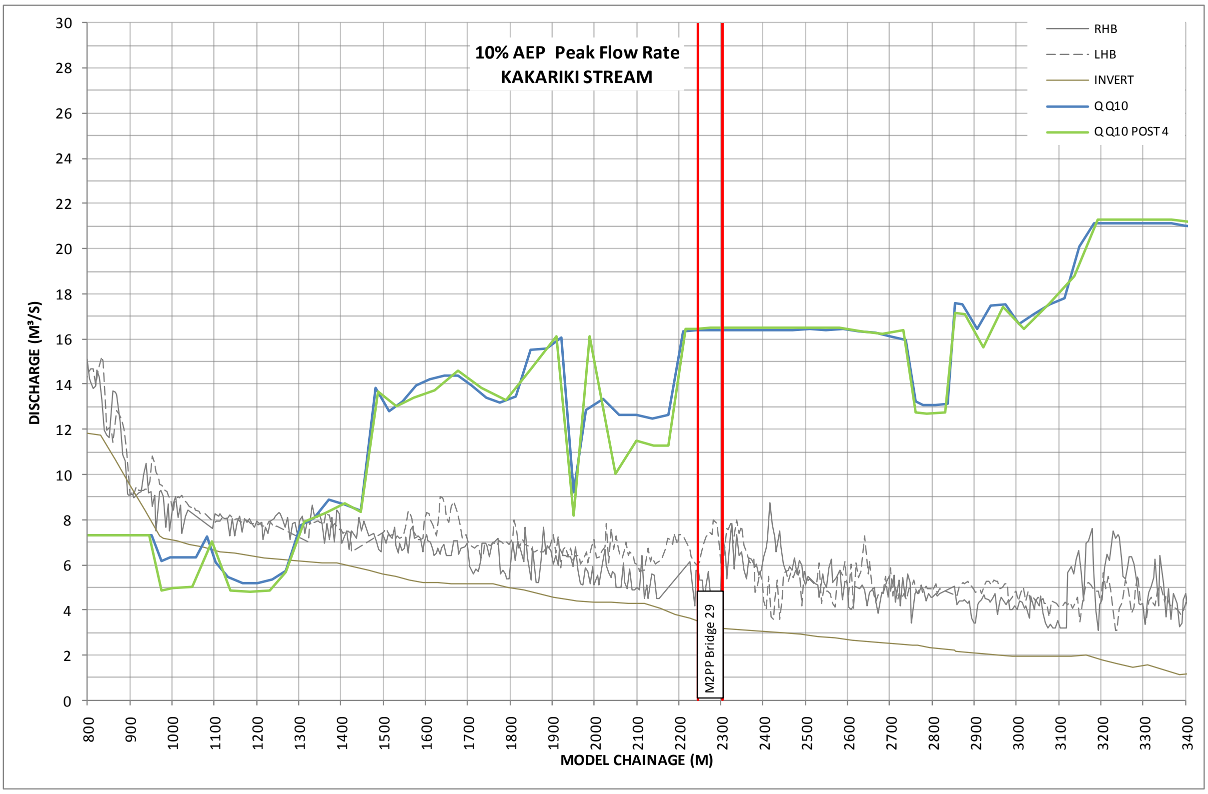

Figure 13 Kakariki- 10%AEP Peak Flow Rate

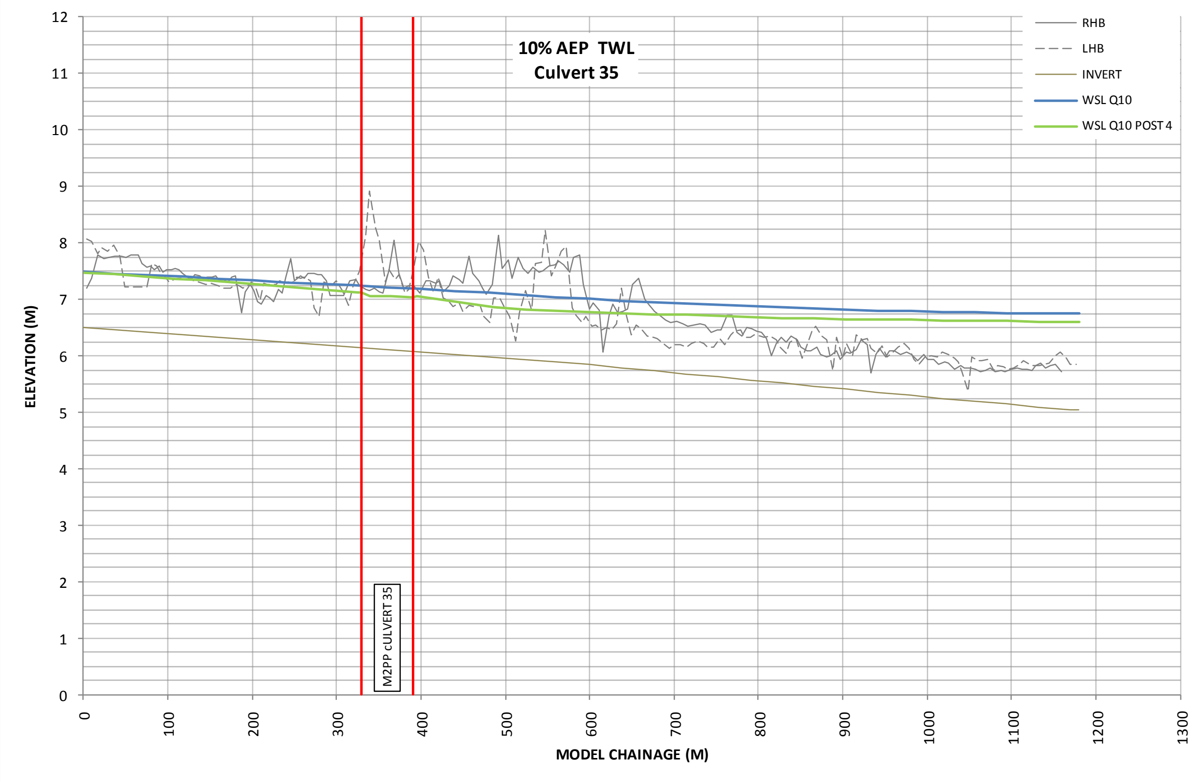

Figure 14 Culvert 35- 10%AEP Top Water Level

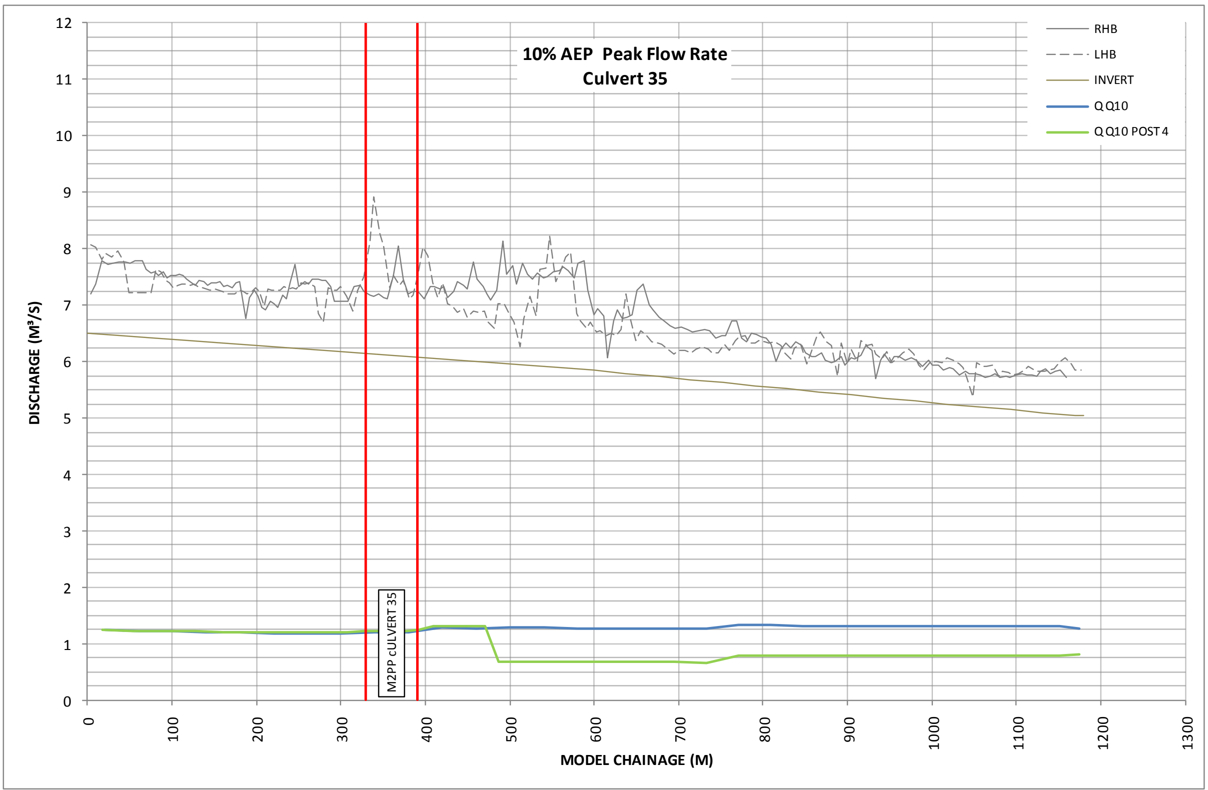

Figure 15 Culvert 35- 10%AEP Peak Flow Rate

Figure 16 Paetawa Drain- 10%AEP Top Water Level

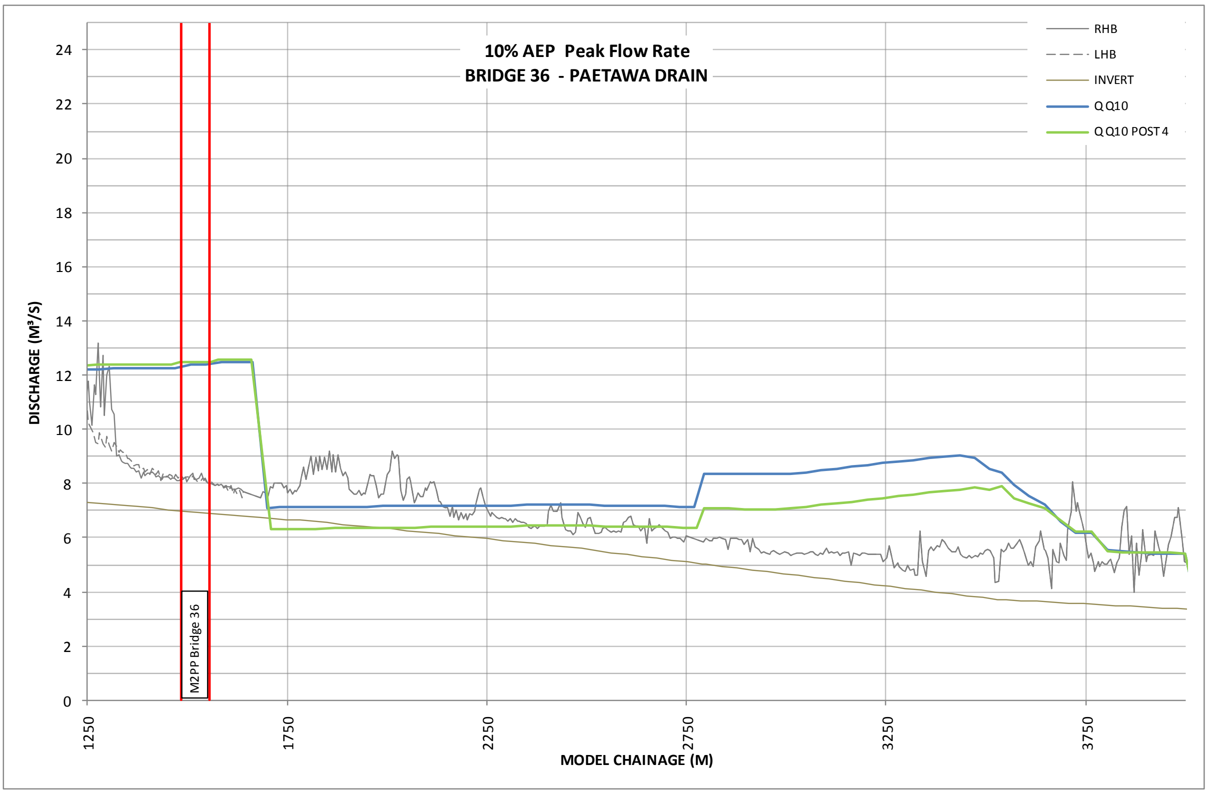

Figure 17 Paetawa Drain – 10%AEP Peak Flow Rate

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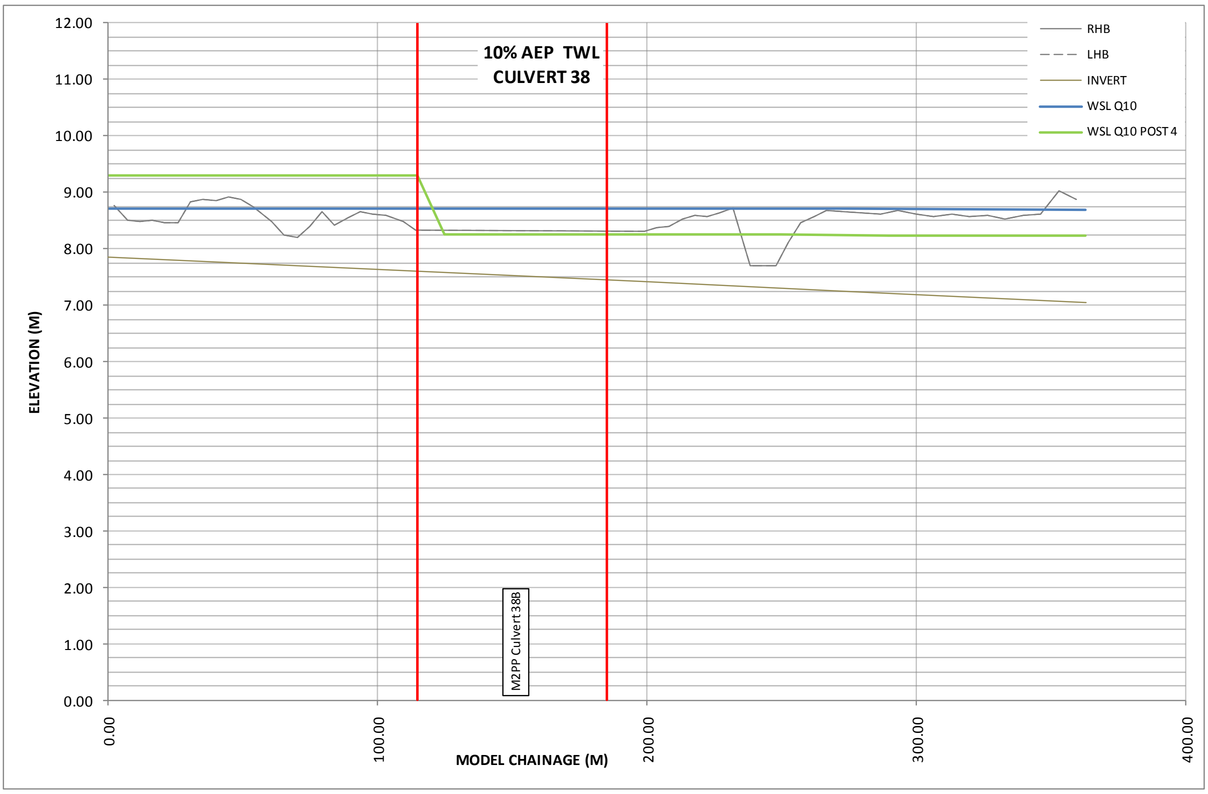

Figure 18 Culvert 38- 10%AEP Top Water Level

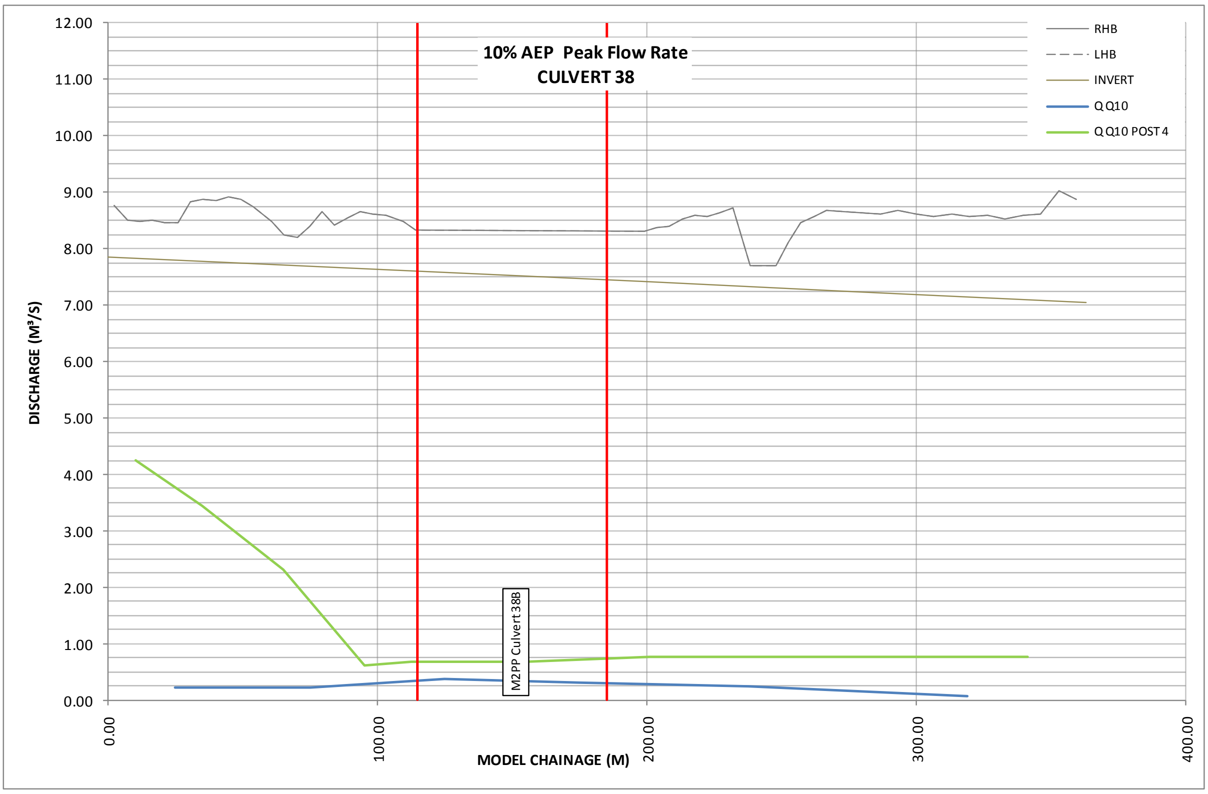

Figure 19 Culvert 38- 10%AEP Peak Flow Rate

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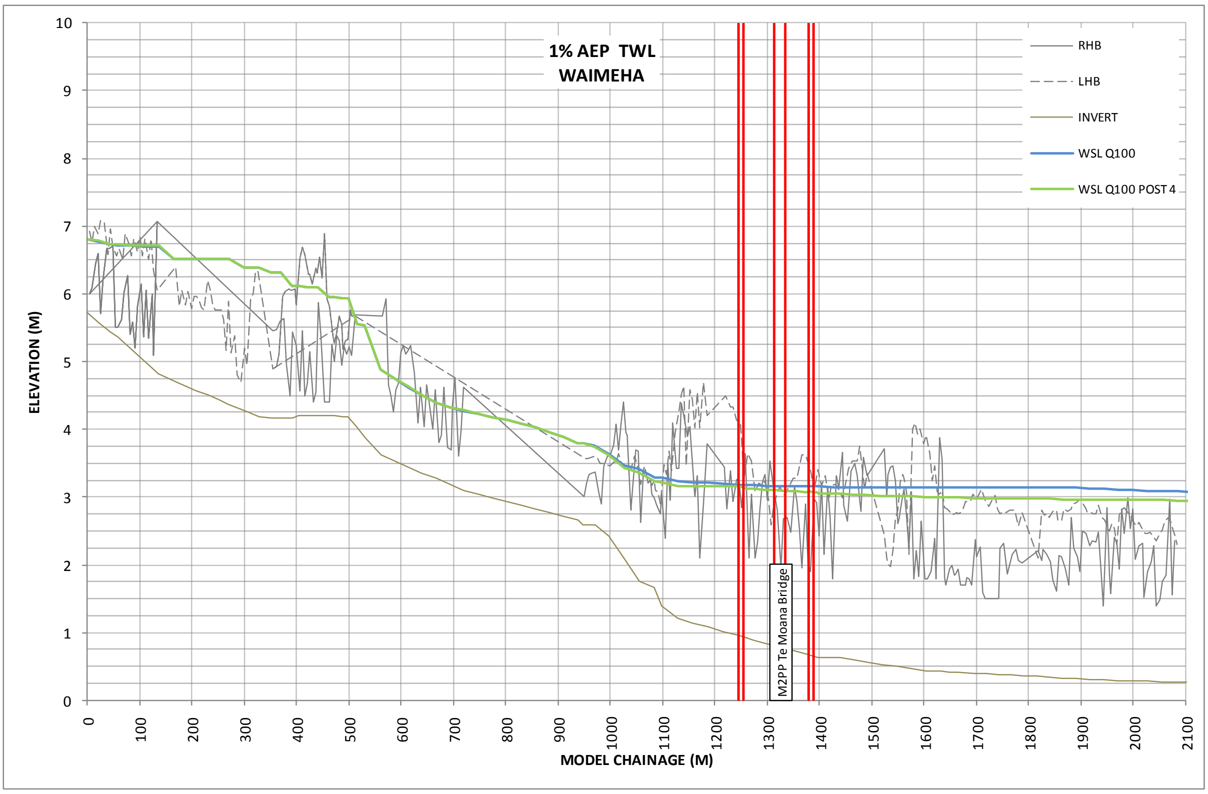

Figure 20 Waimeha- 1%AEP Top Water Level

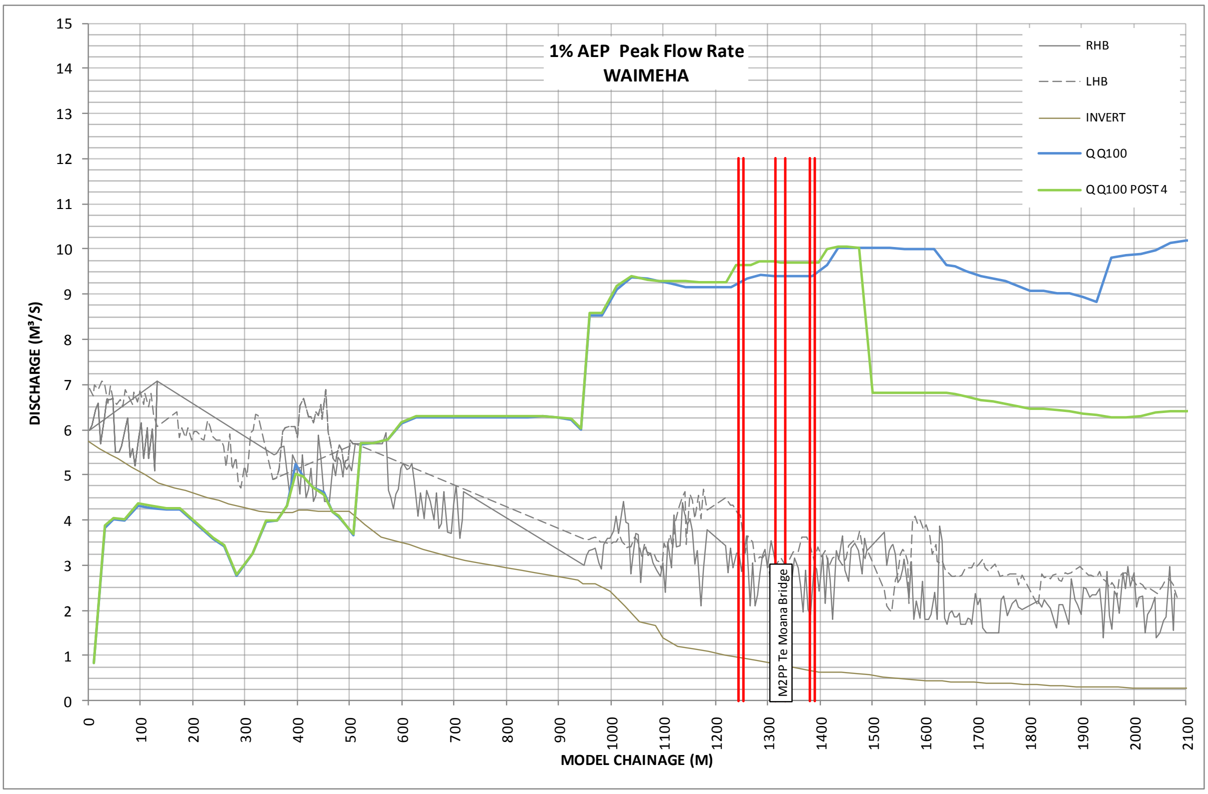

Figure 21 Waimeha - 1%AEP Peak Flow Rate

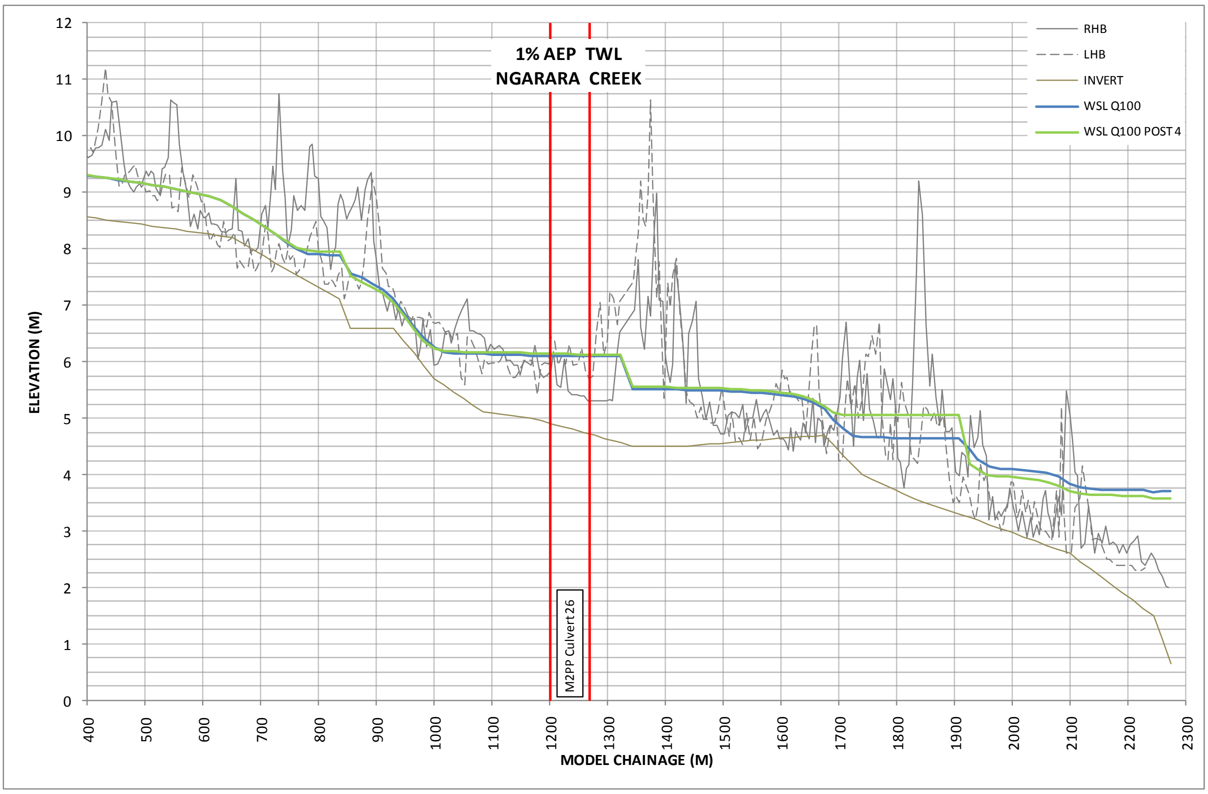

Figure 22 Ngarara Creek- 1%AEP Top Water Level

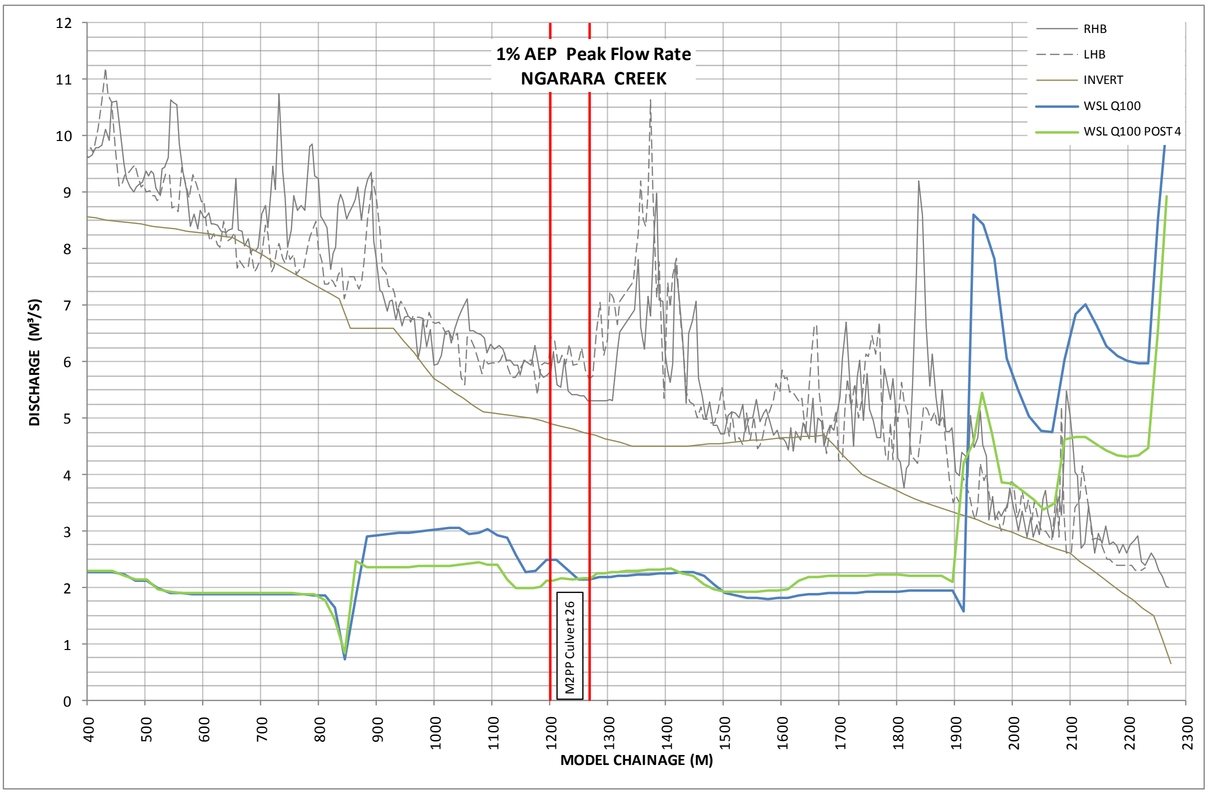

Figure 23 Ngarara Creek- 1%AEP Peak Flow Rate

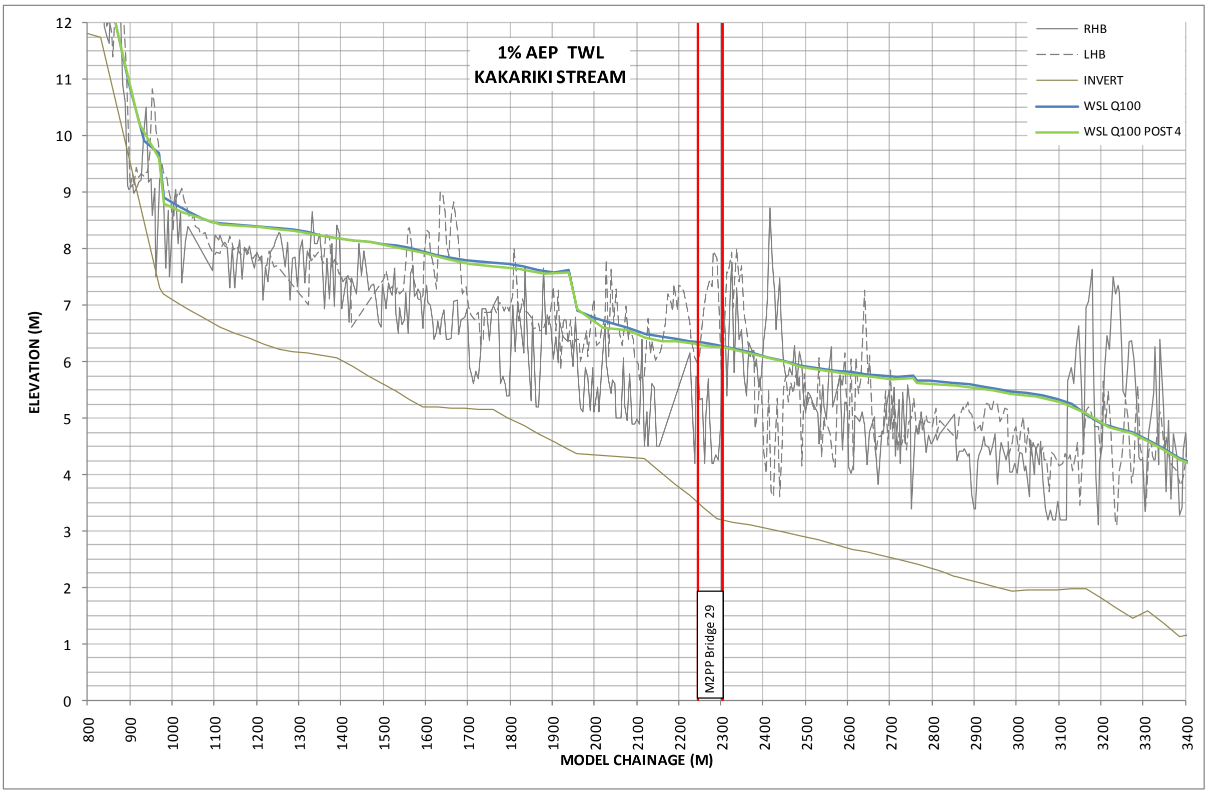

Figure 24 Kakariki- 1%AEP Top Water Level

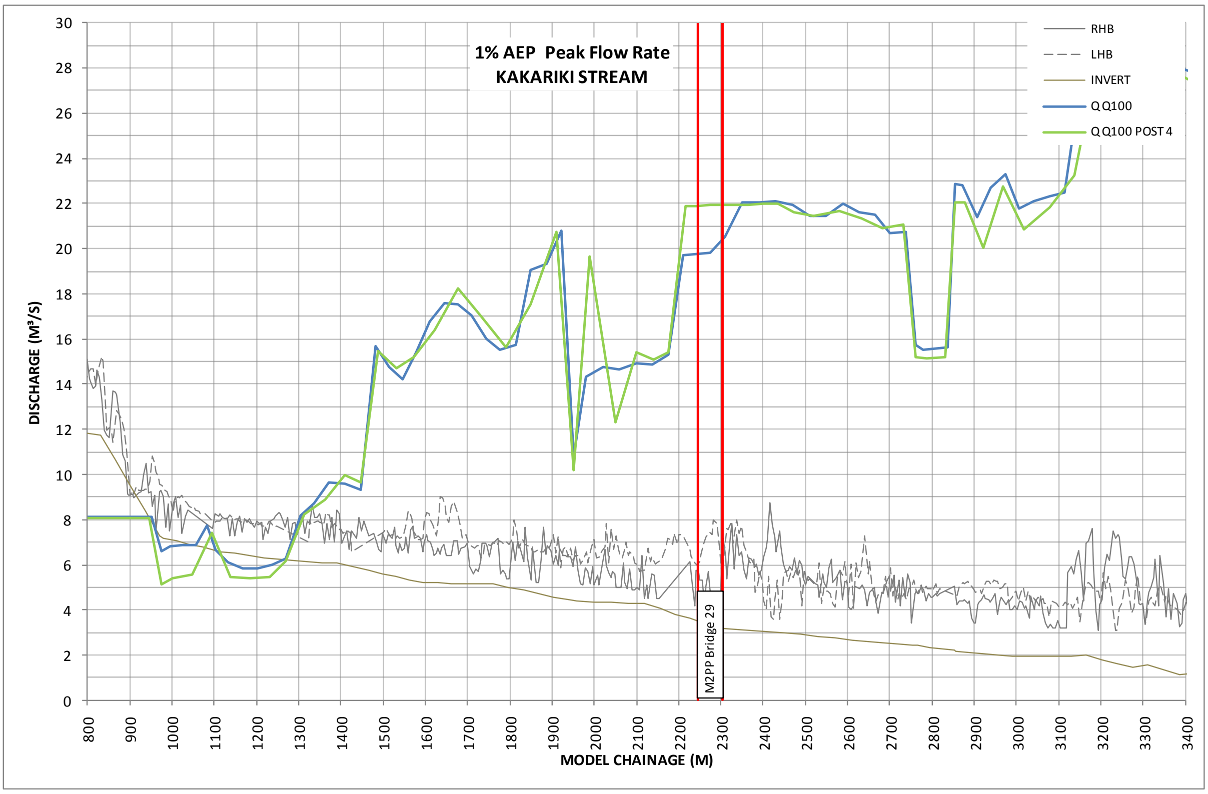

Figure 25 Kakariki- 1%AEP Peak Flow Rate

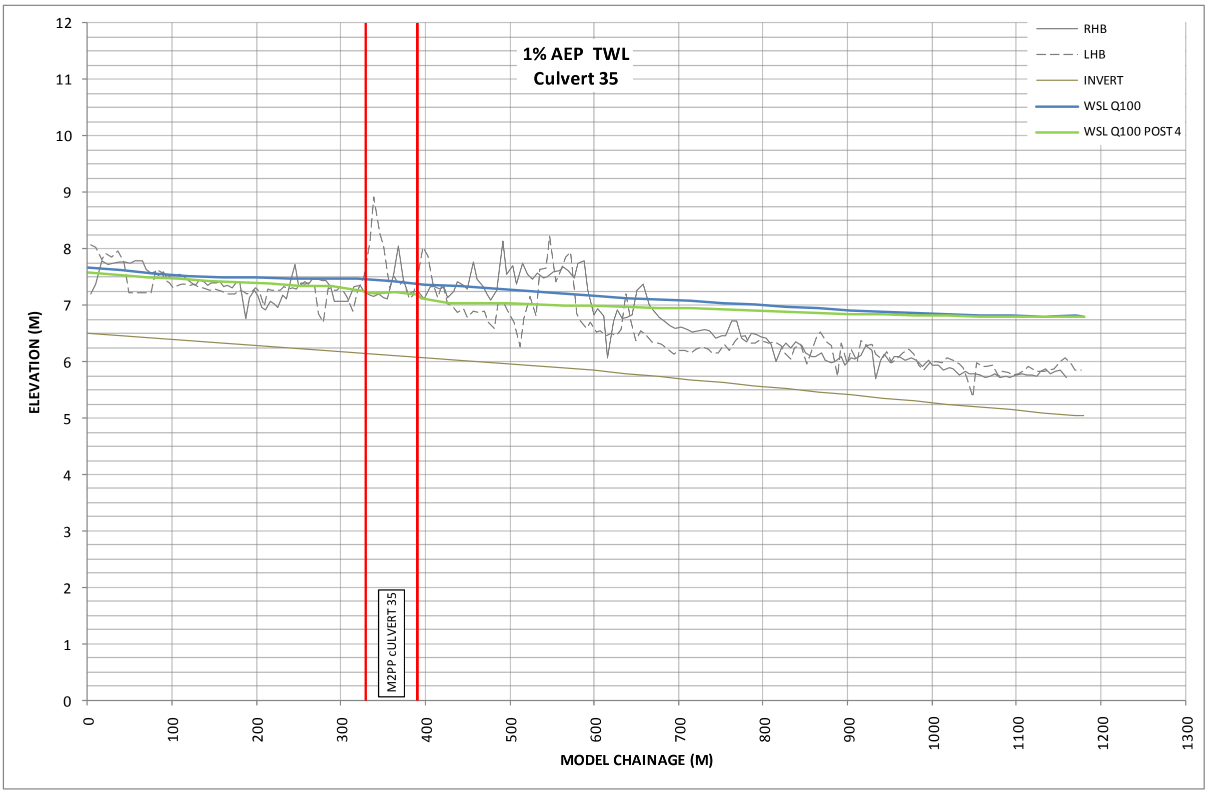

Figure 26 Culvert 35- 1%AEP Top Water Level

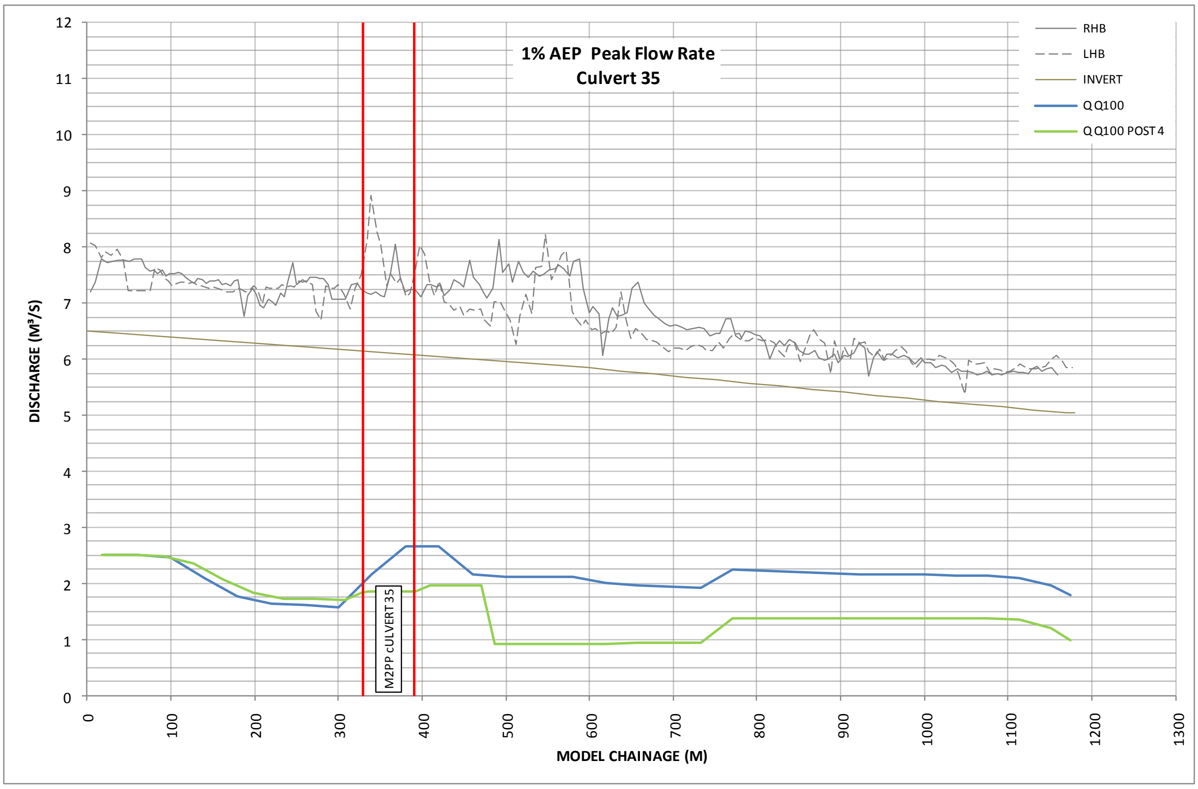

Figure 27 Culvert 35- 1%AEP Peak Flow Rate

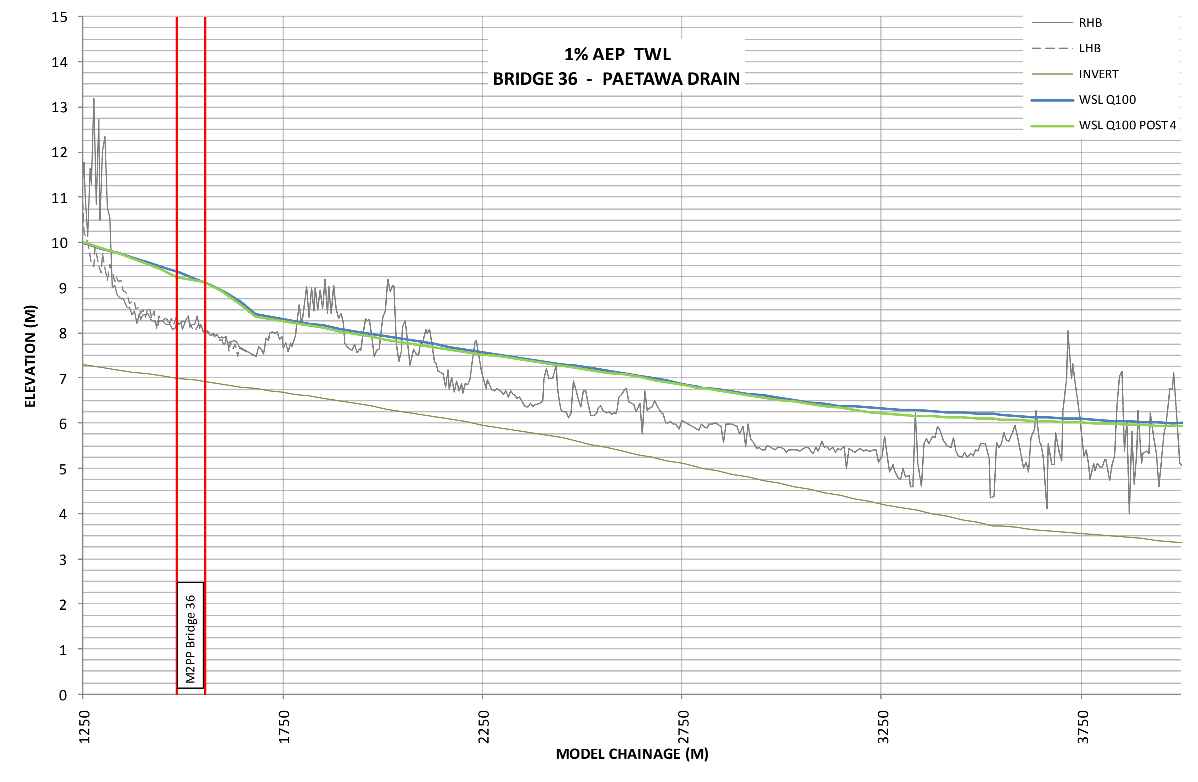

Figure 28 Paetawa Drain- 1%AEP Top Water Level

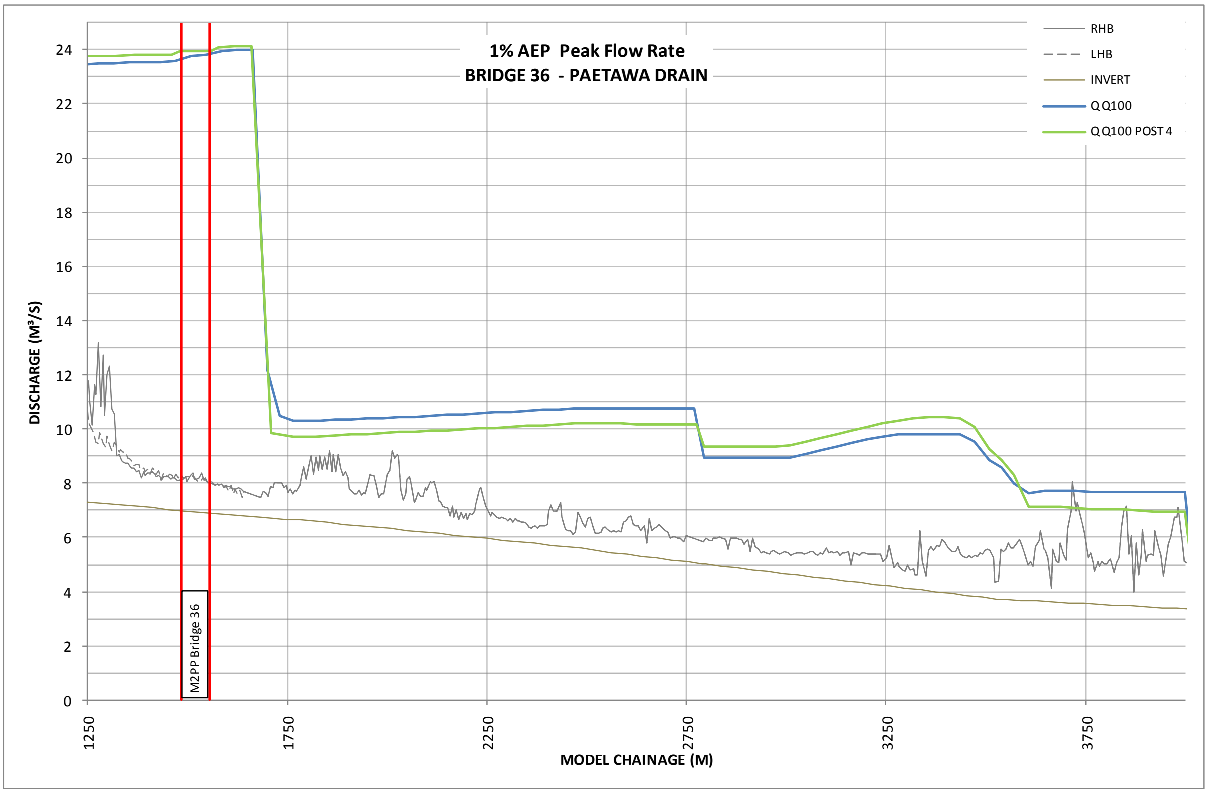

Figure 29 Paetawa Drain – 1%AEP Peak Flow Rate

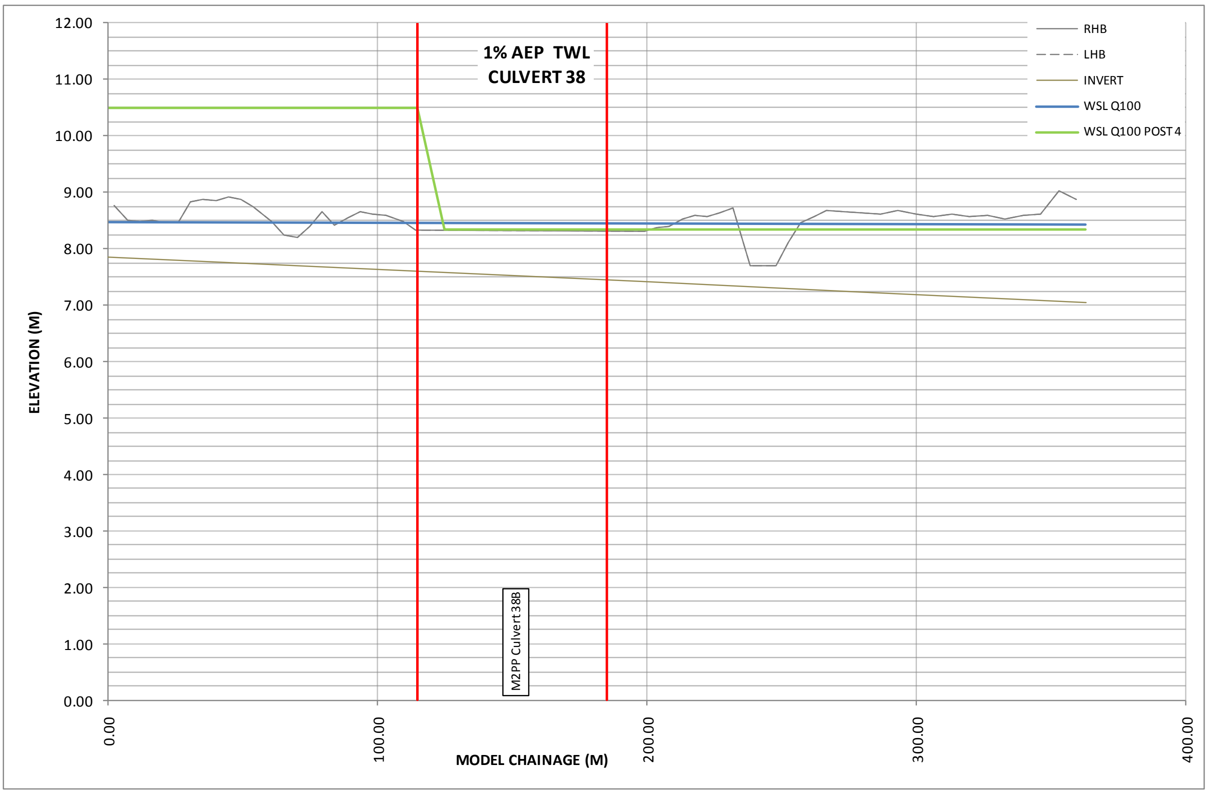

Figure 30 Culvert 38- 1%AEP Top Water Level

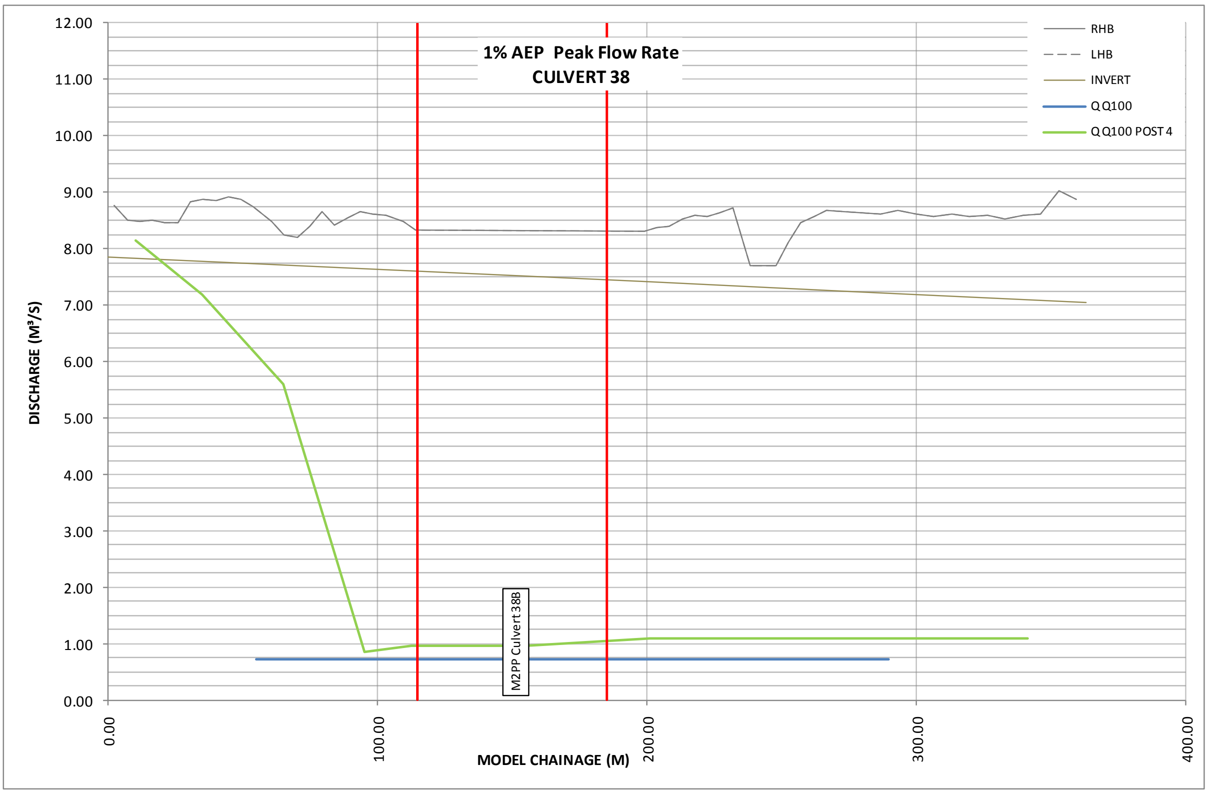

Figure 31 Culvert 38- 1%AEP Peak Flow Rate

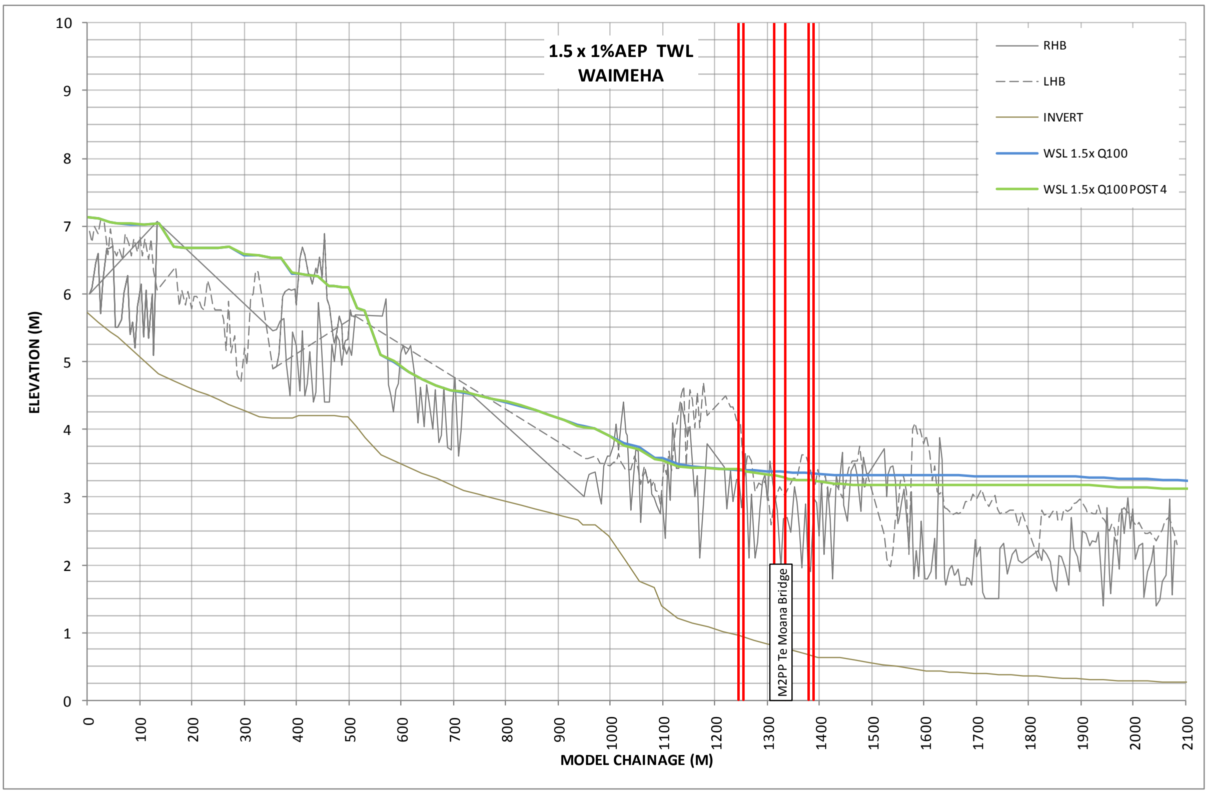

Figure 32 Waimeha- 1.5x 1%AEP Top Water Level

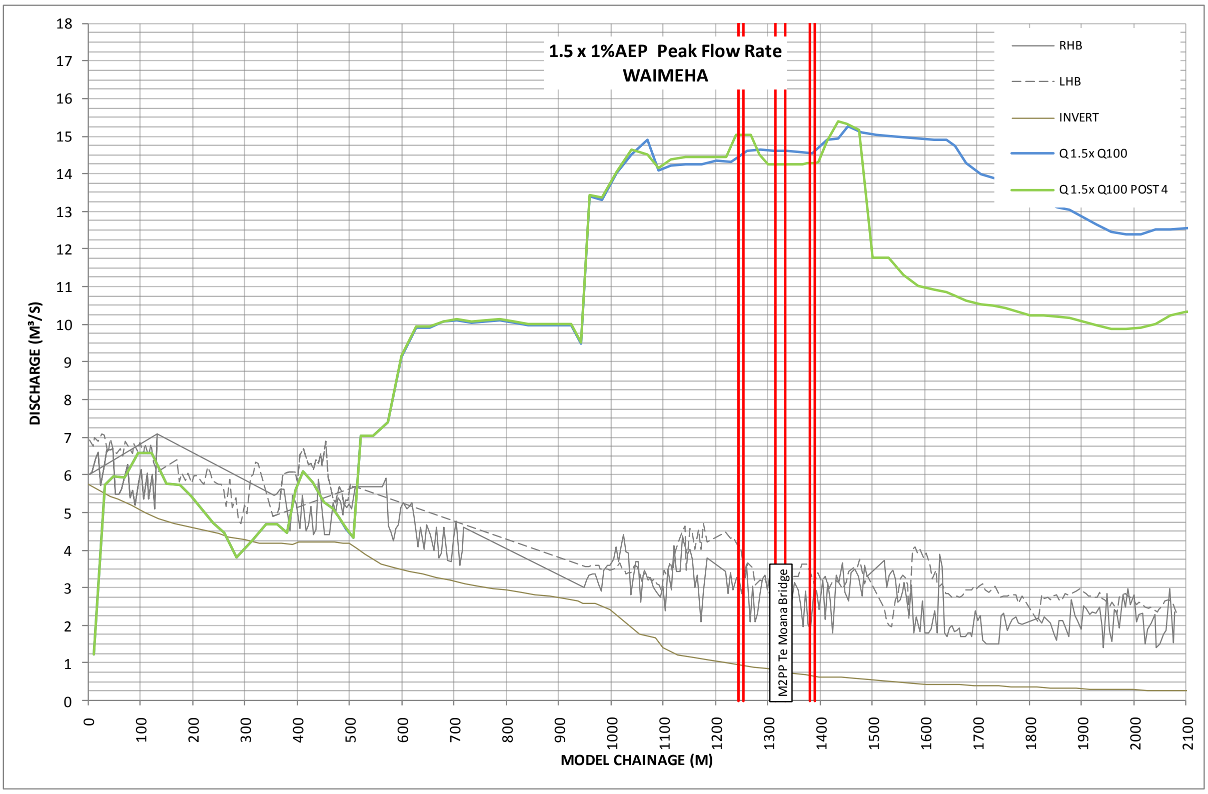

Figure 33 Waimeha - 1.5x 1%AEP Peak Flow Rate

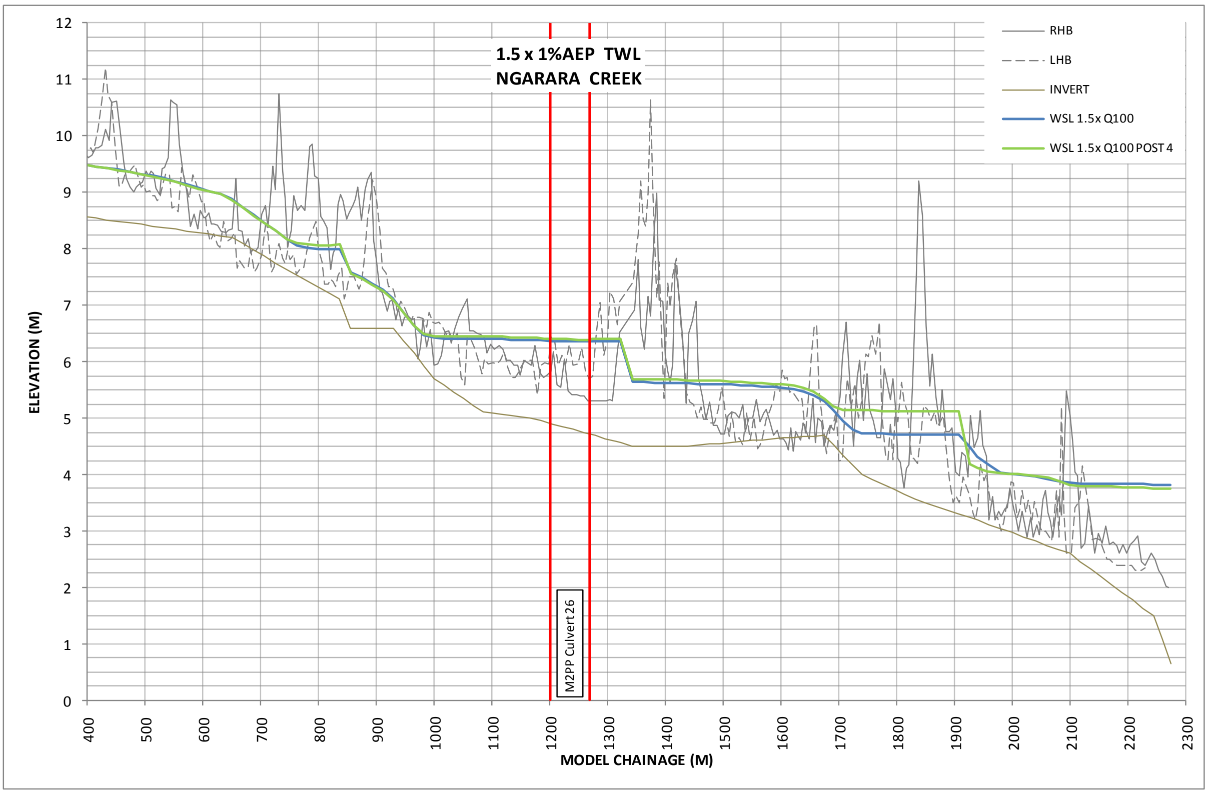

Figure 34 Ngarara- 1.5x 1%AEP Top Water Level

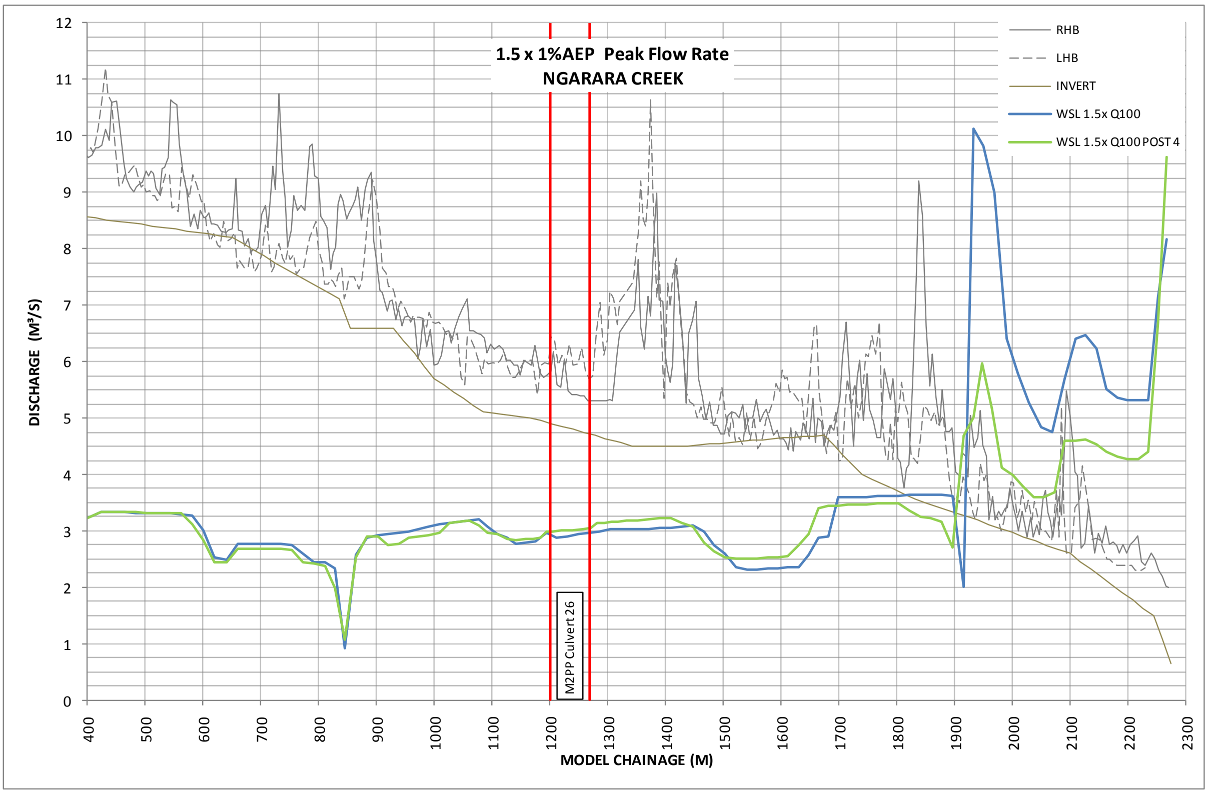

Figure 35 Ngarara- 1.5x 1%AEP Peak Flow Rate

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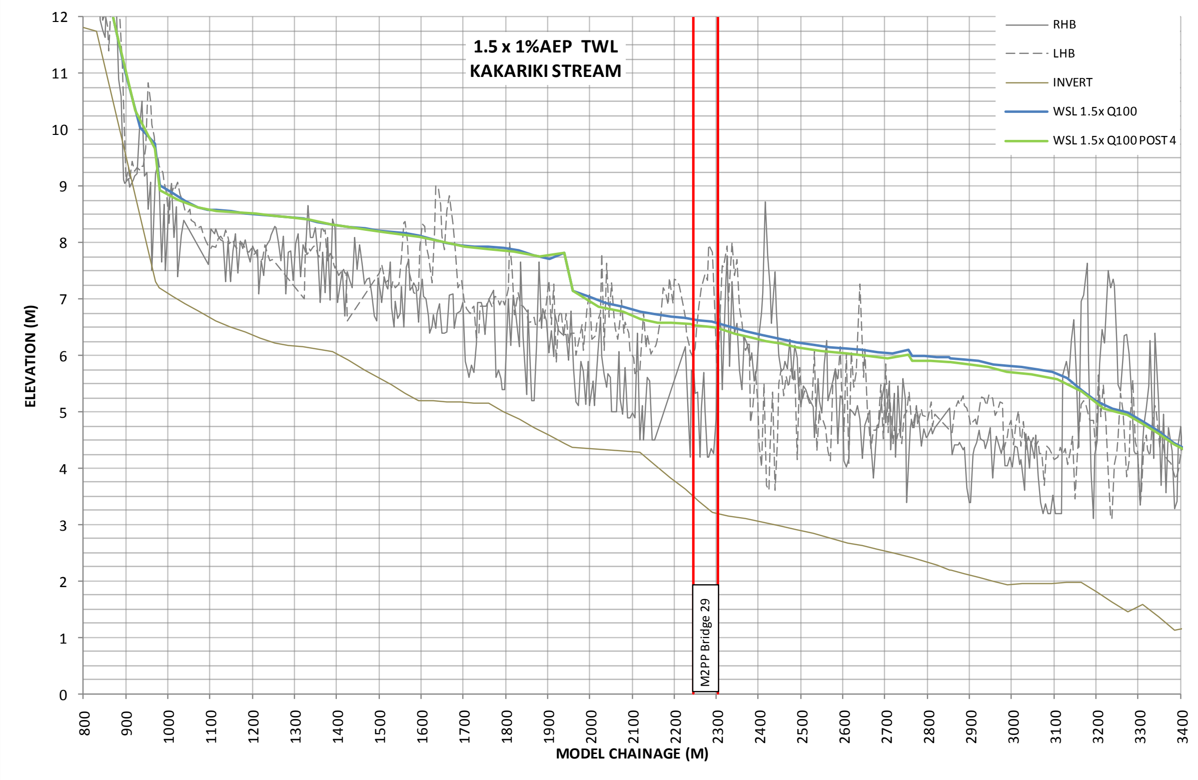

Figure 36 Kakariki- 1.5x 1%AEP Top Water Level

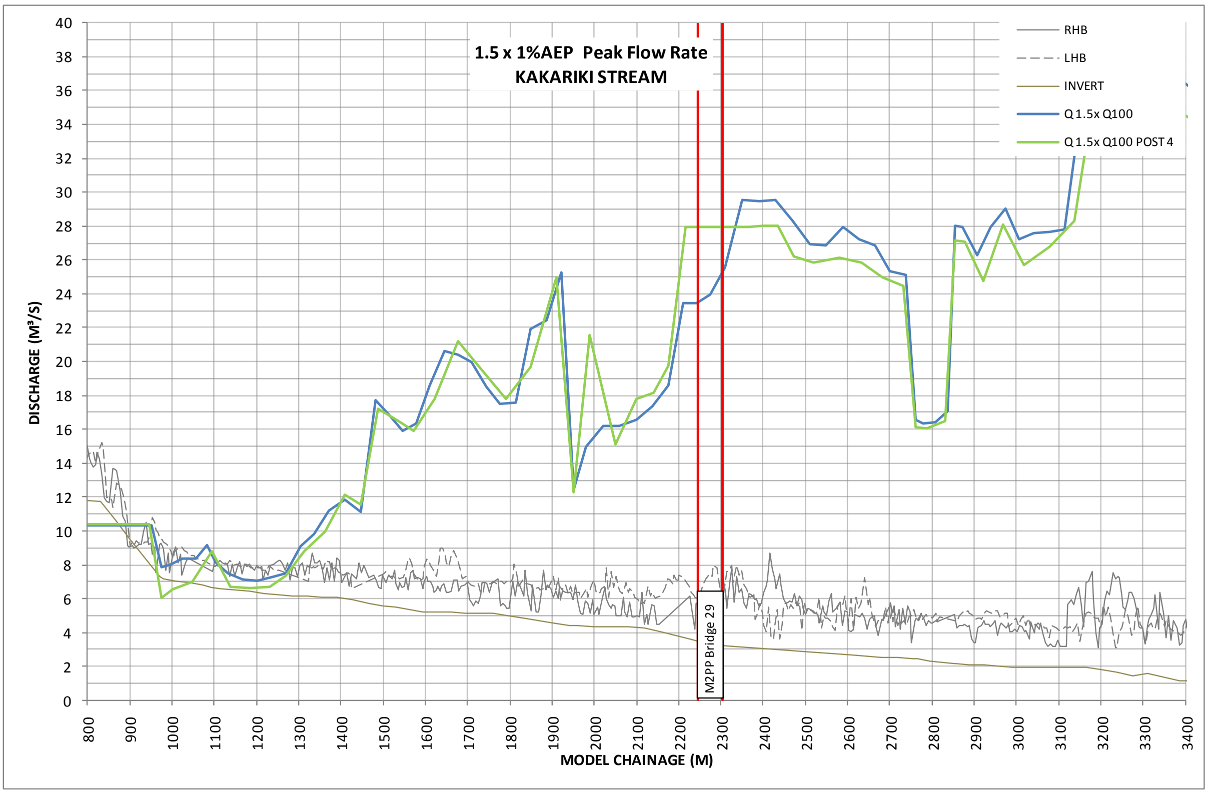

Figure 37 Kakariki- 1.5x 1%AEP Peak Flow Rate

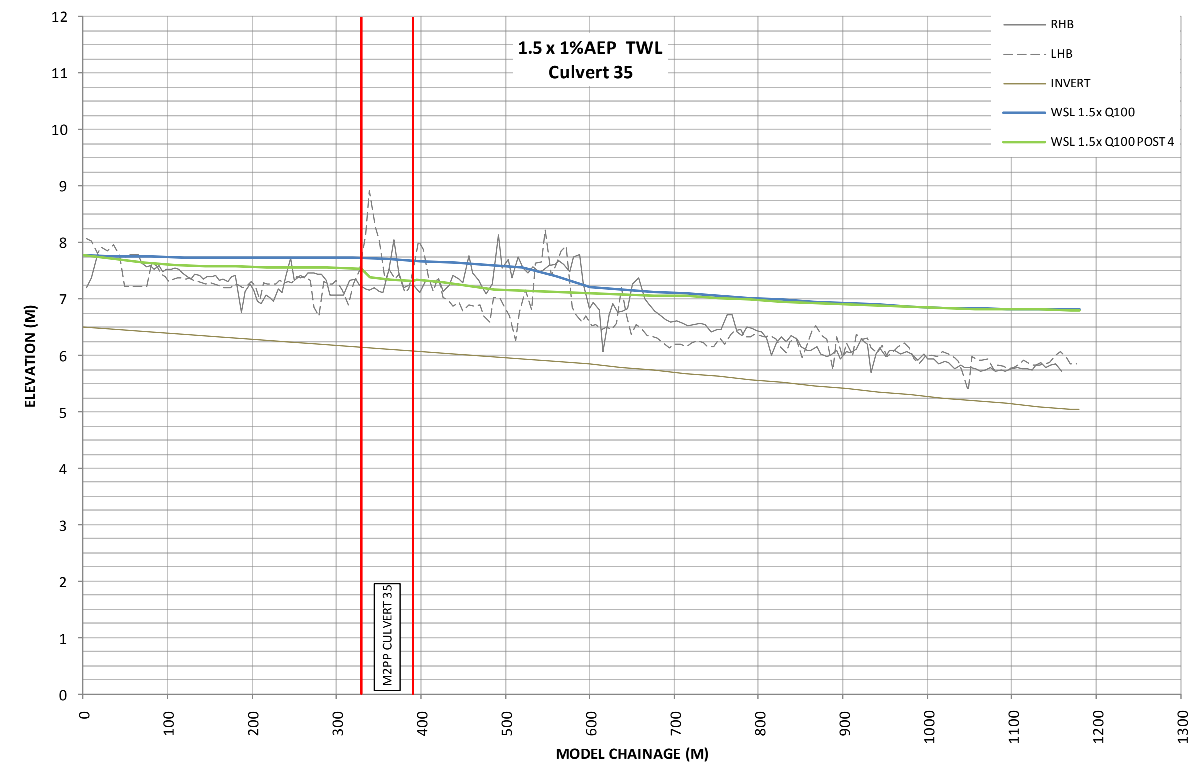

Figure 38 Culvert 35- 1.5x 1%AEP Top Water Level

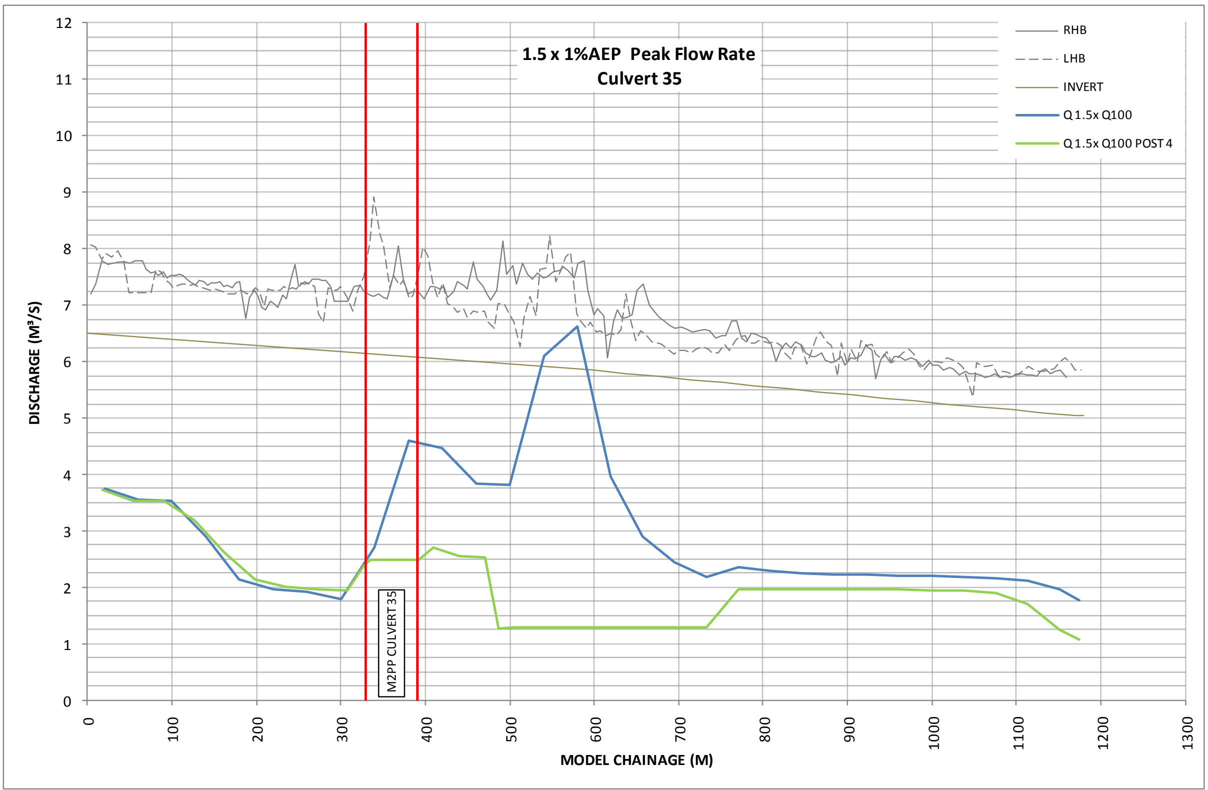

Figure 39 Culvert 35- 1.5x 1%AEP Peak Flow Rate

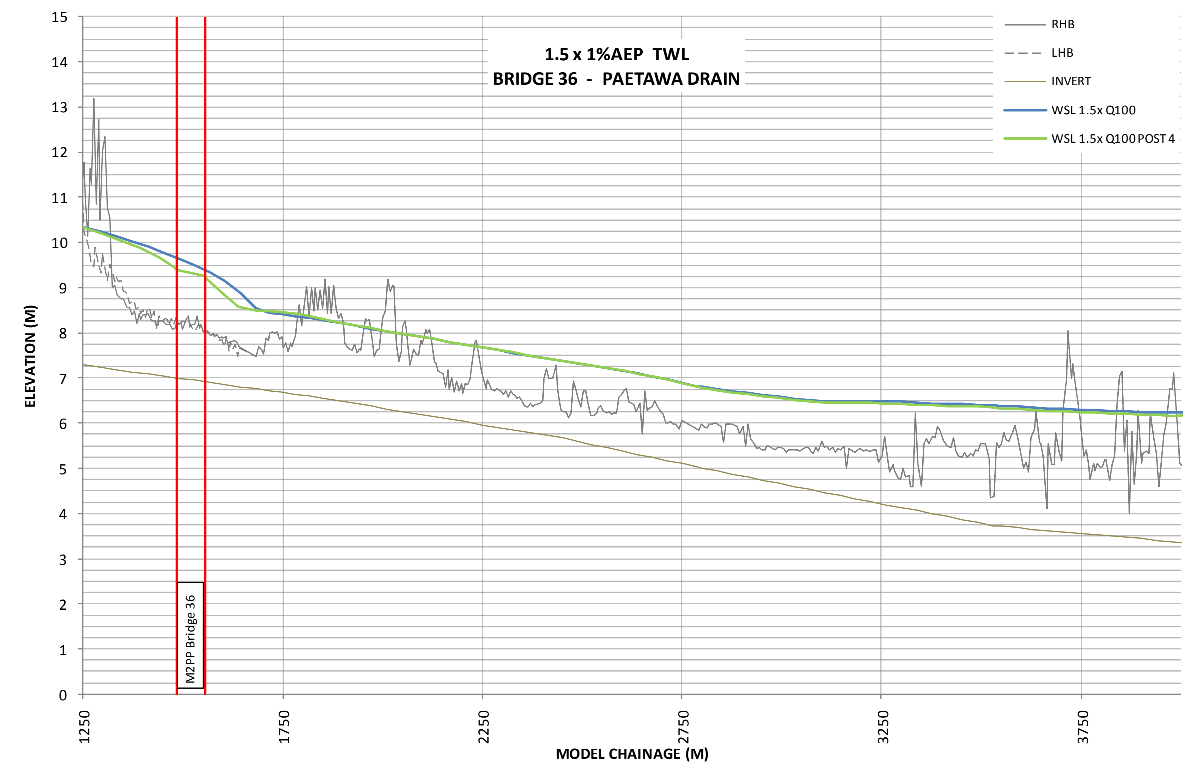

Figure 40 Paetawa Drain- 1.5x 1%AEP Top Water Level

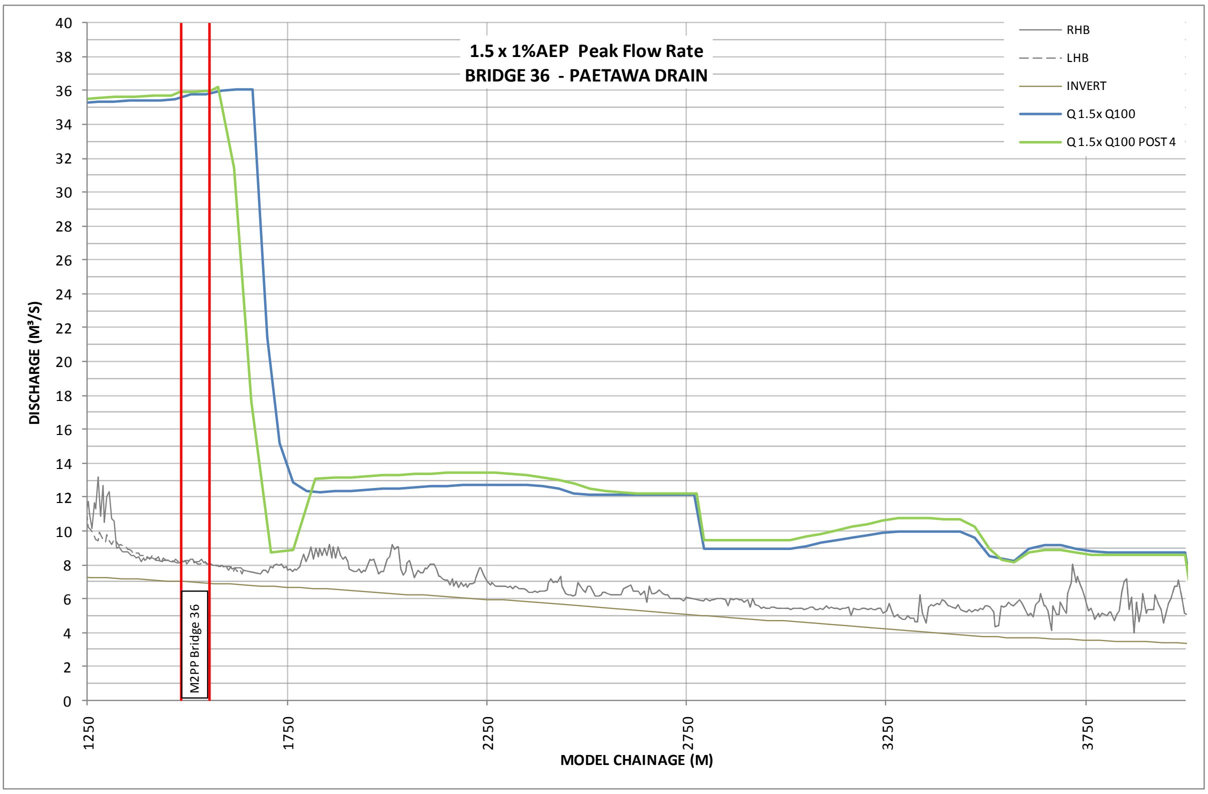

Figure 41 Paetawa Drain – 1.5x 1%AEP Peak Flow Rate

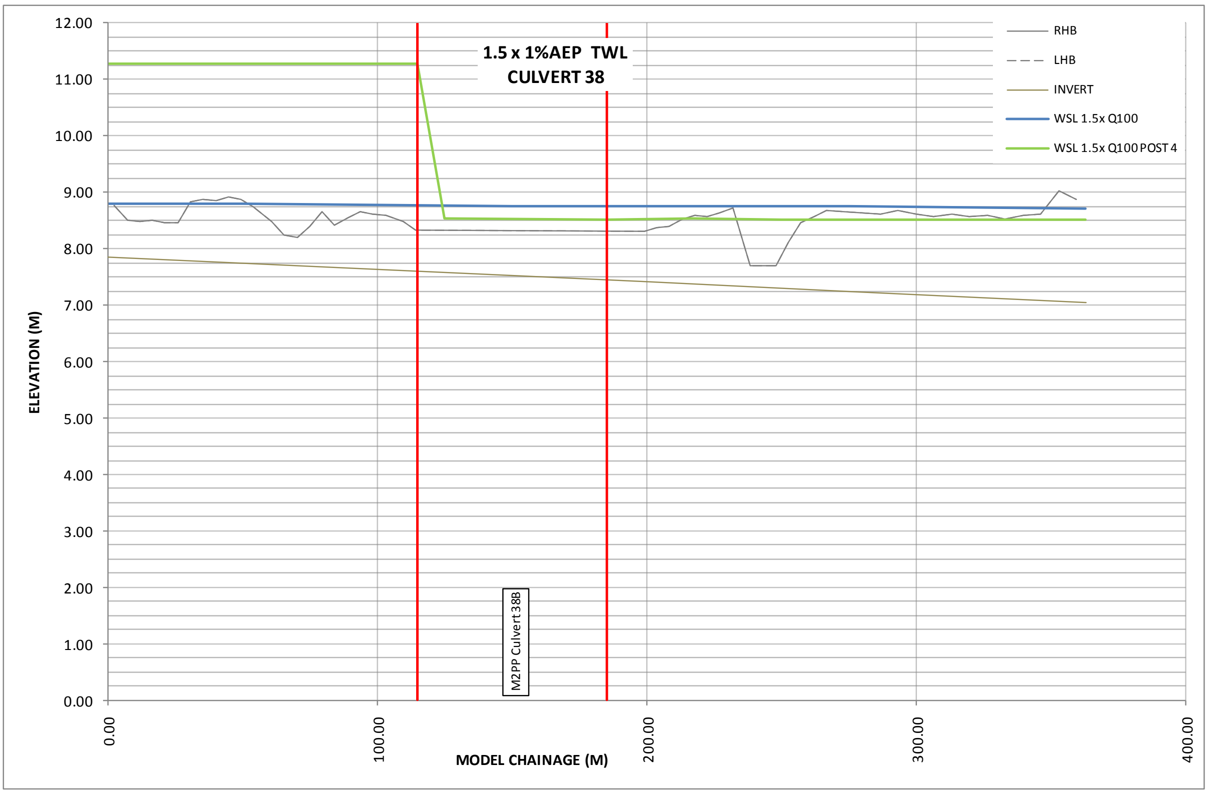

Figure 42 Culvert 38- 1.5x 1%AEP Top Water Level

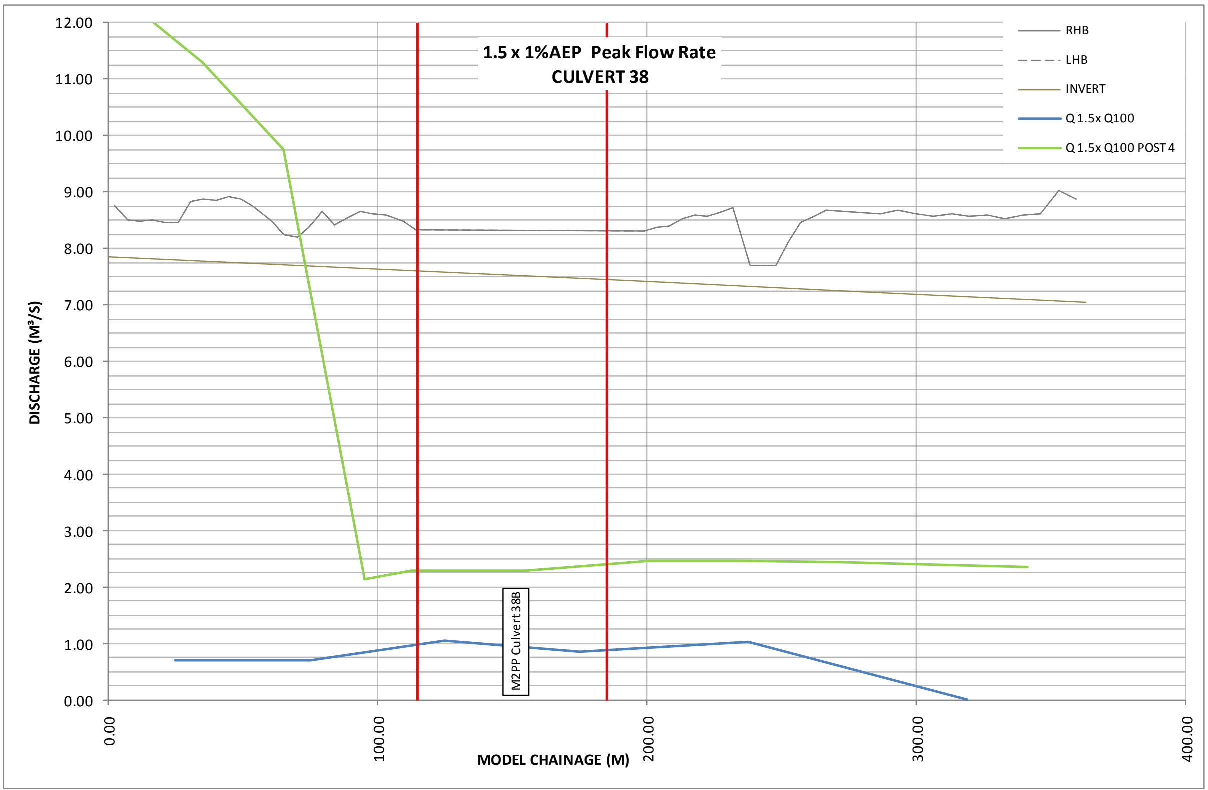

Figure 43 Culvert 38- 1.5x 1%AEP Peak Flow Rate

Appendix 22.H

Waikanae River Peer Review Report

G & E WILLIAMS CONSULTANTS LTD
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MEMORANDUM

Date: 6 December 2011

To: Iain Smith

Of: Beca

Cc: Greater Wellington Regional Council

SUBJECT: WAIKANAE RIVER — M2PP RIVER CROSSING

PAGES 1 + 5

Introduction

As part of the proposed Mackays to Pekapeka [M2PP] expressway a four lane bridge will cross the Waikanae River. A five span bridge is proposed across the river floodplain at the site, from sandhill to sandhill, with 38 m spans. One span is positioned across the main channel, to fit the 35 m design channel, with a slight adjustment of the alignment of this design channel.

A preliminary design of bridge protection and river management works has been undertaken for the bridge site, and proposed works have been determined in concept, for scheme assessment and cost estimation purposes, and for the Assessment of Environmental Effects [AEE]. A review of this design has been commissioned to advise on the adjustment to the 35 m design channel alignment of the Greater Wellington Regional Council [GWRC] and the proposed scour protection works at the bridge. The GWRC has also raised a number of issues concerning the proposed measures, and river management issues arising from these works.

This review is based on my earlier investigations and design work on the Waikanae River, and the information supplied by the M2PP Alliance and GWRC specifically on the bridge site and proposed measures. This includes the concept plans of the proposed measures from the M2PP Alliance, and a report on hydraulic modeling, which includes studies of Waikanae River flood flows around the bridge site. A report on channel changes and the gravel bed material resource of the Waikanae River was provided by GWRC. This included plots of the river cross sections from the river surveys.

An initial scoping and discussion meeting was held on 5 September, with representatives of the M2PP Alliance, GWRC and Kapiti Coast District Council. Iain Smith provided comprehensive minutes of the meeting, which covered the issues and concerns raised. The site was visited on 16 September.

Given the design is at a preliminary stage, this review is more a commentary on what has been proposed, than a peer review. At the meeting, the widening of the main channel along the bridge reach, to the design channel width, was discussed, with hydraulic modeling of this design channel requested.

Following an initial assessment and recommendations, the M2PP Alliance commissioned further hydraulic modeling with a design channel from the bridge to the downstream bend, and revised the design. The review, then, included an assessment of this design, which incorporated all the recommendations of the initial assessment.

Background

The proposed bridge is sited on the lower Waikanae River, below Jim Cooke Park and close to the El Rancho Campground. There is a major change in river grade below Jim Cooke Park, and the reach around the bridge site is a natural aggradation zone for the gravel bed material of the river. The main channel is also relatively narrow and flood waters overflow onto the adjacent berm or floodplain land in relatively small floods.

Over recent years substantial river management measures have been undertaken, from State Highway 1 to Jim Cooke Park, and a design channel has been established to a consistent width and meander form. Some bank protection works and vegetation management has been undertaken downstream of Jim Cooke Park, but the main channel is significantly narrower than the design width, especially at the bridge site.

The hydraulic modeling shows substantial overflows of flood waters in a 2 year return period event, with berm flow velocities of over 1 m/s on the open land beyond the channel (Figure 4.3.4). Generally a 2 year flood flow would be contained within the main channel of a river. In this case, the flattening of the river grade as it flows through the coastal sandhills, the narrowness of the main channel and the willow vegetation along the channel margins restrict channel capacity. This flooding at small flood levels has significant implications for the approach and measures proposed by the M2PP Alliance.

There is a natural levee alongside the main channel of the river, with a drop in level across the floodplain away from the main channel. Local stormwater flows and flood overland flows tend to concentrate on the outer side of the floodplain away from the main channel.

The analysis of the channel cross sections indicates that there has been little aggradation downstream of Jim Cooke Park in recent years, above the amount of gravel bed material extracted. There has, though, been an accumulation of bed material between the 1991 and 2004 surveys. The large flood of January 2005 may have had an influence in scouring out bed material from the lower reaches of the river.

The river channel is at its narrowest along the bridge reach. The main channel is significantly under size upstream of about XS 100, with an especially narrow length around XS 120, where there is a low floodplain area in El Rancho. The channel is also especially narrow at XSs 140 and 150. Upstream of XS 155, the main channel is generally around the design width, and the upstream works have been established around the design width and alignment.

This narrowness of the channel also has significant implications for the approach and measures proposed by the M2PP Alliance. The proposed bridge pier spacing and protection rock works at the design 35 m width gives rise to major transition issues with a narrower channel upstream and especially downstream of the bridge.

Proposals

The bridge piers spanning the main channel are to be sited on either side of the channel, at its design width of 35 m. A rock riprap lining would extend from the pier caps to below channel bed levels, long the bank at the bridge piers. Originally these linings were to extend a short distance downstream of the bridge to slightly further upstream of the bridge. On the left bank the rock lining would wrap around into the bank of the Muaupoko Stream. There would be some readjustment at the stream mouth, and there would be a long transition (of 1:5, sideways to downstream) along the willow lined river bank downstream on the right bank.

The downstream transition was not determined in detail, but the bank would be cut away, as shown on the plans, and willows re-established on the new bank margin. The willows on the existing bank would have to be removed and disposed off. This channel adjustment extended just about to the river bend downstream, stopping at about XS 140.

With the widening of the main channel to its design width down to the bend below XS 140, there would be a more extensive cutting away of the bank, and native species would be interplanted along the landward side of the re-established willows. The rock lining on the right bank has also been shortened, with the ends of the linings warped around into the bank.

The rock works have been outlined in some detail, with crest and founding levels, and a thicker lining below normal water levels. A light rock is proposed, given the low flow velocities and turbulence in flood events, but the medium size has been increased from 300 to 400 mm, with associated increases in the lining thickness. The ground under the bridge, on the berms, would have a rock blanket with a medium size of 300 mm and a thickness of 600 mm, with the edges tucked into the adjacent ground.

There would be a rock lining around the bridge abutment on the left (south) side, but not on the right abutment, which is further away from the main channel, beside the El Rancho access way.

The berm rock blanket is shown following the general contour of the existing berm land. At the meeting, it was suggested it could be placed level at the lower general ground level (below that of the levee at the channel edge). It was also noted that there might be a suitable vegetation cover for the conditions under the bridge, either growing in a rocky cover material or in the existing ground.

The amount of enlargement of the channel at the bridge is indicated on the section plans of the drawings.

Assessment

Design Alignment

The small change in the alignment of the design channel through the bridge site is within tolerances for these design channels, and the general meander form and curvature has been retained. This is both reasonable and acceptable. The skew of the bridge piers to the river alignment is also very small, and the piers are at the edge of the channel (not in it) or across the berm land, where overland flows will generally align with them.

Rock Linings

Rock linings, generally as proposed, are an acceptable protection measure for the bridge piers. They are located at the bank position of the design channel. The rock works as proposed are reasonable measures, but I would make the following comments on these proposals.

The original medium rock size of 300 mm was on the small size. The main channel velocities are relative low along this reach of the river, with its flat grade, channel roughness and berm spill. In my 1992 report I recommended a medium size of 350 mm downstream of Greenaway Road. Even at this size the smaller rocks would be small enough to easily pick up and move. A medium size of 400 mm may be more appropriate for this bridge protection purpose, with a corresponding thicker lining.

The toe extension of the lining, as originally proposed, has been standard practice, based on overseas practices, but has been found to be unduly prone to rock displacement and progressive under scour and rock removal or settlement into the river bed. This has been observed in mobile bed physical model studies and in reality. Instead of the toe extension, a thickening of the lining below low flow water level, from 2 times to 2.5 or 3 times the medium size, is used. This is easier to place and less disruptive of the river bed, and gives rise to a more robust lining better able to cope with under scouring and settlement of the lining.

The crest of the lining is best positioned at the height of the bank, to give even overflow of flood waters with a minimum of deflection and turbulence generation at this edge of the lining. A re-shaping of the bank can be undertaken to give this effect (see my comments below on the berm rock blanket).

The ends of the rock lining should be returned into the bank, and the adjacent bank planted to mitigate erosion at this structural edge. The radius of this return can be taken as the (horizontal) width of the lining at the crest, and the horizontal length of the lining (from crest to foundation) at the foundation. A small rounding out of the lining at the downstream end, as a protrusion into the channel, is very useful at the end of linings to deflect flood currents across to the opposite bank, and mitigate the downstream extension of the scour hole that develops alongside linings. This can be done where one (outer) bank is lined, and there is a natural crossing over of the flow towards the other bank. In this case, both banks are to be lined, and with the full design channel being developed, these linings would be at the straight length where there would be a natural cross-over between meanders. In this case, a simple return of the linings into the bank is sufficient.

The structural lining of both banks of the river fixes the channel in plan position, but with the design channel being developed, with its natural curvature, this position can be maintained and channel movement managed within the design channel. There is still a transition from rock to willow vegetation, and some on going management at these transitions will be necessary. The river bed and bank will, though, be reformed to a natural shape at the lining edges.

It is understood that the piers are deeply founded, and not at risk from river scour or bank erosion. The rock linings are proposed more because of the difficulties of maintaining vegetation and a management buffer zone under the bridge. The linings are at a meander cross over (and not at a bend), and a well formed cross over can develop with the proposed wider (design) channel along the reach. This will mitigate the management issues that arise from the different protection measures at the bridge itself, and allow well integrated management of the reach, as part of the management of the Waikanae River as a whole.

Rock lining just the left bank and the Muaupoko Stream mouth was discussed at the meeting, with possibly some toe rock at the bridge pier cap itself on the right side. This was a possibility if the existing channel was maintained with its existing bank vegetation. The right side pier would then be constructed on the existing berm just beside the walkway, where there are presently some poplars. For this approach, the question would be how to maintain the required edge and buffer vegetation under the bridge.

Berm Measures

There are relatively fast berm velocities in large flood events, especially along the lower lying land near the outer edge of the floodplain. A rock blanket cover of the ground under the bridge is an appropriate protective measure, and a well graded angular rock material would be best. There is again an edge issue with the grassed or treed land upstream and downstream, and the blanket edges should be tucked down into the ground to reduce the potential for edge attack.

There could be an especially problematic edge where there is an intersection of the rock blanket, grassed land and trees of the channel edge buffer. Strong in and out flows would occur over the rock lining and blanket under the bridge and between the trees of the upstream and downstream buffers. A set back of the rock linings (to the design channel width) with a narrower channel upstream and downstream would exacerbate these localised currents and turbulence from the geometric changes. The widening of the channel and wrapping of the rock linings into the bank will mitigate these edge effects, but some well arranged vegetation at this transition (of buffer trees, rock lining, grassed berm and rock covered berm) would be worthwhile.

Some ground shaping under the bridge may be warranted to be able to lay a more uniform blanket cover. The natural levee at the channel edge could be cut down to keep the edge lining lower, and to give a well formed area under the bridge for flood water inflows and outflows. Local runoff could also be directed to the existing drainage way downstream, beside a low terrace (of an old channel). This ground shaping and detailing needs to be considered during the final design stage.

There will be a gabion wall and toe rock at the left abutment, and some detailing of the proposed rock blanket (covering the berm land under the bridge), such as a wrapping up at the edges, would be worth considering. No protection measure is shown at the right abutment. This is beside the proposed El Rancho access realignment, and is in a back area, at a slightly higher level, but some toe protection may be desirable at this spill through abutment. The realigned access road would be subject to flood flows, even in small flood events, and there would be on going maintenance issues from silt deposition and scouring along the road edge.

Channel Width

The bridge spans the full floodplain at the site, and there is very little impact on flood levels and flow conditions from the bridge. The main effect is from the piers themselves and debris held up by the piers in flood events. There is a large margin above the design 100 year flood level to the underside of the bridge superstructure for debris clearance (of over 2 metres).

The principal concerns were around the width of the existing main channel and the effect of any localised widening at the bridge, which would be fixed on both sides by rock works.

The channel widening would be mostly on the right side, where there is a band of willow trees on the channel edge, and then poplars beyond the walkway path. The proposed downstream bank would continue to the river bend below XS 140, where there is an area of willows and weeds beyond the bank edge willows (in an old channel area). The design channel extends out into this area of willows, and eases the bend at this point. Some Otaki River boulders have been placed on the right bank at this bend, and they can be seen in the bank. Realignment to the design channel would then shift out the proposed bank realignment into this area of willows and the upstream poplars at the bridge site.

A widening out to the design channel, down to the bend, would mitigate much of the concerns over the localised widening at the bridge and fixing of the channel position by rock linings (and the pier caps) on both sides.

There is a Telecom cable and two gas pipeline crossings just upstream of the proposed bridge. No information is available on the depth of the Telecom cable under the river, at the channel or on the berms. As-built crossing profiles were provided for the two gas pipelines. This showed that the older (1960s) pipeline is over 2 metres below the river bed and has a long curve out under the berm land. This pipeline would not be affected by any widening of the river channel.

However, the 1980s duplication pipeline has a minimum 1.5 m cover across the berm land, with a relatively sharp drop down to a level crossing under the existing main channel, where the minimum cover is 2 metres. The lower level pipe is only around 25 metres long. Thus a 35 m wide main channel could impact on this pipeline crossing, and bring the river banks close to the rising part of the pipeline.

Conclusions

The span and position of the proposed bridge piers fits in well to the floodplain geometry and the design channel for river management purposes. The slight adjustment to the alignment of the design channel is acceptable.

The proposed rock works are appropriate protection measures for the bridge, and some modification of the detailing has been made, including a somewhat larger rock size and thicker linings. Some reshaping of the berm can be undertaken along with a protection cover, and this should fit in with the local drainage pattern and the height of the channel bank.

The main concerns arising from the proposals were around the transitions from vegetation or grass cover to rock, especially the bank linings. It is agreed that the piers should be placed outside the main channel, with a span over the 35 m design channel. The enlargement of the reach from the bridge to the downstream bend ensures that the linings at the bridge site are at a cross-over area between meanders. This will mitigate the management issues that arise from the fixing of the banks at the bridge and ease the transitions between this rock and the vegetated banks above and below the bridge. A well integrated management of the reach can then be undertaken, as part of the management of the Waikanae River as a whole.

Gary Williams

Water & Soil Engineer