Transmission Gully Project

Assessment of Hydrology and Stormwater Effects

Technical Report 14

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Contents

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Executive Summary

The Transmission Gully Project (the Project) comprises the construction and operation of a 27 km State highway and associated link roads north of Wellington between Linden and MacKays Crossing.

This report assesses issues associated with water quantity. This includes catchment and road runoff, stream flows and velocities and flood risk. It addresses the impact of the road both during construction, and in the long term, defining:

- Appropriate sizing of culverts and bridges for streams (ephemeral and permanent) that cross the road, including fish passage considerations
- Requirements for outlet erosion control along the alignment
- The impact of increased flows on flood risk
- The impact of stream diversions on stream length/grade and related velocities.

The receiving environment is considered to be the streams, floodplains and any infrastructure potentially at risk from flooding within the catchments traversed by the Project. Modelling of potential effects on these receiving environments has been undertaken for both the pre-construction and post-construction scenario. This was used in the design of cross culverts, bridges and stream diversions to test their ability to convey flows and minimise adverse effects, including consideration of rainfall, climate change and future planned development in the catchments.

As part of the assessment process a risk assessment identified six areas where the potential for hydrological or hydraulic effects was more complex. These areas included the Pauatahanui, Horokiri and Te Puka Streams, Duck Creek and the urban stormwater networks at Linden and Waitangirua. These areas were the subject of more detailed modelling and assessment.

The assessment of effects concluded the following:

- Culverts and bridges have been sized to allow for design flows and velocities. They have been designed with outlet erosion control and to allow for fish passage where required. It is considered that culverts can comply with the relevant standards in all situations.
- Significant diversions are to take place in the Pauatahanui, Horokiri and Te Puka streams. Without mitigation these could result in increased downstream velocities and flood risk. The recommended mitigation is to design the morphology of the new channels to replicate as closely as possibly the existing channels, allow sufficient floodplain, and plant stream banks to reduce velocities. With these measures in place it is not considered the diversions will have significant hydraulic effects.
- There is the potential for a change in flood risk through loss of floodplain storage, alteration of secondary flow paths, increased runoff from the change in land use, and increased velocities due to changes in stream shape. Recommendations have been made to minimise each of these potential effects in each area. It is considered that with these recommendations implemented the flooding effects will not be more than minor.
- The Linden and Waitangirua urban stormwater networks do not have sufficient capacity for the increased runoff from the change in land use associated with the road. In Linden it is recommended that flood

storage is provided in the upper catchment. In Waitangirua it is recommended that the stormwater capacity is increased in co-ordination with Porirua City Council's planned upgrades for this area.

1. Introduction

The Transmission Gully Project (the Project) consists of three components:

- The Transmission Gully Main Alignment (the Main Alignment) involves the construction and operation of a State highway formed to expressway standard from Linden to MacKays Crossing. The NZ Transport Agency (NZTA) is responsible for the Main Alignment.
- The Kenepuru Link Road involves the construction and operation of a road connecting the Main Alignment to existing western Porirua road network. The NZTA is responsible for the Kenepuru Link Road.
- The Porirua Link Roads involves the construction and operation of two local roads connecting the Main Alignment to the existing eastern Porirua road network. Porirua City Council (PCC) is responsible for the Porirua Link Roads.

1.1. The Transmission Gully Main Alignment

The Main Alignment will provide an inland State highway between Wellington (Linden) and the Kapiti Coast (MacKays Crossing). Once completed, the Main Alignment will become part of State Highway 1 (SH1). The existing section of SH1 between Linden and MacKays Crossing will likely become a local road.

The Main Alignment is part of the Wellington Northern Corridor (Wellington to Levin) road of national significance (RoNS). The Wellington Northern Corridor is one of the seven RoNS that were announced as part of the Government Policy Statement on Land Transport Funding (GPS) in May 2009. The focus of the RoNS is on improved route security, freight movement and tourism routes.

The Main Alignment will be approximately 27 kilometres in length and will involve land in four districts: Wellington City, Porirua City, Upper Hutt City, and Kapiti Coast District.

The key design features of the Main Alignment are:

- Four lanes (two lanes in each direction with continuous median barrier separation)
- Rigid access control
- Grade separated interchanges
- Minimum horizontal and vertical design speeds of 100 km/h and 110 km/hr respectively
- Maximum gradient of 8%
- Crawler lanes in some steep gradient sections to account for the significant speed differences between heavy and light vehicles.

1.1.1. The Kenepuru Link Road

The Kenepuru Link Road will connect the Main Alignment to western Porirua. The Kenepuru Link Road will provide access from Kenepuru Drive to the Kenepuru Interchange. This road will be a State highway designed to following standards:

- Two lanes (one in each direction)

- Design speeds of 50 km/h
- Maximum gradient of 8%
- Limited side access.

1.1.2. Porirua Link Roads

The Porirua Link Roads will connect the Main Alignment to the eastern Porirua suburbs of Whitby (Whitby Link Road) and Waitangirua (Waitangirua Link Road). The Porirua Link Roads will be local roads designed to the following standards:

- Two lanes (one in each direction)
- Design speeds of 50 km/h
- Maximum gradient of 10%
- Some side access will be permitted.

1.2. Background to the Transmission Gully Project

The concept of an inland, alternative route to bypass the existing SH1 coastal route and communities north of Wellington was first raised in the early 1940s and has been under consideration by various parties ever since.

The key events in the development of the Project are:

- In the early 1940s, there was first talk of an alternative inland route for SH1 north of Wellington
- In 1981, the National Roads Board embarked on an assessment of the Western Corridor (undertaken by the Ministry of Works and Development and the Ministry of Transport) looking at options for an inland route (now known as Transmission Gully) in comparison to an upgrade of the coastal route
- In 1986, the findings of the National Roads Board's Western Corridor Report were released with the report rejecting an inland route and supporting major improvements along the existing coastal route
- In 1987, the Greater Wellington Area Land Use and Transportation Strategic Review (GATS) was jointly funded by the National Roads Board, Wellington Regional Council and the Urban Transport Council. The Western Corridor section was separated out for early consideration. The GATS considered a large number of options including routes through Porirua East/Whitby, Takapu Valley, Belmont deviation through Belmont Regional Park to SH2, as well as upgrades to the coastal route
- In 1989, an environmental impact report (EIR) was produced to compare the impacts of options proposed in GATS including public transport and roading upgrades. The EIR considered both coastal and inland options. The EIR concluded that in addition to public transport upgrades, roading improvements were required to address the growing congestion on SH1. The EIR found the inland route was more environmentally and socially acceptable. The favoured route was an inland alignment from MacKays Crossing to Takapu, continuing through the Takapu Valley with an interchange on SH1 at Tawa
- In 1990, the Parliamentary Commissioner for the Environment (PCE) conducted an audit of the EIR. The PCE agreed in principle with the findings of the EIR with some reservations and recommendations. The audit found that Takapu Valley was not necessarily the best alignment at the southern end and that further investigation of the links to the Hutt Valley and Porirua was required. The PCE's principal recommendations were to finalise and designate the inland route and to consult with the public to reduce uncertainty for both the coastal and inland route communities

- In 1991, the Wellington Regional Council conducted further investigations into possible alignments at the southern end. A number of alignments were examined and the conclusion was for a connection to SH1 at Linden as well as connection to western Porirua via a Kenepuru link. Justification for this was clear benefits to the management of Porirua traffic and relief to SH58 around Pauatahanui Inlet. This would also reduce environmental and social impacts associated with the Takapu Valley option
- In 1996, a preliminary design was produced for the Linden to MacKays Crossing alignment and the NoRs were lodged
- In 1997, the hearing takes place for the NoRs for the Linden to MacKays Crossing alignment
- In 2003, all the appeals on the notices were finally resolved and the designations for the Linden to MacKays Crossing alignment were included in the relevant district plans
- In 2004, an existing local road designation was altered to provide local road access to the Linden to MacKays Crossing alignment from eastern Porirua
- In 2004, the Western Corridor Transportation Study (jointly commissioned by Greater Wellington Regional Council and Transit New Zealand) commenced to provide the basis for an integrated transportation strategy to manage travel demands in the Western Corridor. The resulting Western Corridor Plan (WCP) included consideration of major public transport and roading options and travel demand management (TDM) initiatives. Consultation on the WCP indicated that affected communities did not support the coastal route and expressed a strong preference for Transmission Gully
- In 2006, the WCP was endorsed by the Transit NZ Board and adopted by the Greater Wellington Regional Council and included Transmission Gully in the Regional Land Transport Strategy (2007 to 2016) for construction within 10 years as part of a balanced multi-modal approach to addressing transport needs within the Western Corridor
- In 2008, a draft scheme assessment report (SAR) was undertaken which involved the assessment of numerous options for a Transmission Gully alignment both within and outside the confines of the existing designation. Together with a detailed consultation process, preferred alignment for Transmission Gully was produced
- In 2009, detailed environmental and engineering investigation work commenced for Transmission Gully.
- In May 2009 the GPS is released which included the RoNS programme. The Wellington Northern Corridor is one of the RoNS
- In December 2009, NZTA's Board announces that Transmission Gully is the preferred route to improve access through the southern end of the Western Corridor. The NZTA press release stated; "our task was to choose the route which would deliver the best result for the region and New Zealand [as part of the Roads of National Significance], while also bearing in mind the potential impact on the environment and surrounding communities. In the end it was clear that Transmission Gully was the better choice. It is less expensive, it will provide a safer four-lane route, it's better for local communities and better for the environment, and it will reduce travel times between Kapiti and Wellington."
- In 2010, detailed environmental and engineering investigation work is progressed and the preferred alignment is optimised to accommodate road design, ecological, water quality and other considerations. In March, the NZTA signals its intention to lodge the statutory RMA documentation with the EPA using the new "national consenting process".

1.3. Project Description

1.3.1. Transmission Gully Main Alignment

The Main Alignment is a proposed 27km expressway from Linden in Wellington City to MacKays Crossing on the Kapiti Coast.

The Main Alignment consists of nine sections:

Section Number	Section Name	Station Value (m)	Length (km)
1	MacKays Crossing	00000 – 03500	3.5
2	Wainui Saddle	03500 – 06500	3.0
3	Horokiri Stream	06500 – 09500	3.0
4	Battle Hill	09500 – 12500	3.0
5	Golf Course	12500 – 15500	3.0
6	State Highway 58	15500 – 18500	3.0
7	James Cook	18500 – 21500	3.0
8	Cannons Creek	21500 – 24900	3.4
9	Linden	24900 – 27700	2.8

Section 1: MacKays Crossing

This section is approximately 3.5km long, and extends from the tie-in at the existing MacKays Crossing Interchange on SH1 to the lower part of the Te Puka Stream valley. The Main Alignment will connect to the existing SH1 at approximately 00700m. The first 700m is the existing State Highway 1 alignment which is a grade separated interchange providing access across the North Island Main Trunk rail line (NIMT). Any alteration to the MacKays Crossing Interchange will be minimal.

This section of the Main Alignment will provide for three lanes in the northbound carriageway from 00700m and from 02100m in the southbound carriageway. Southbound traffic will be able to exit the Main Alignment at approximately 01250m. This exit will pass under the Main Alignment at approximately 01800m and will connect to the existing SH1 heading south towards Paekakariki. Traffic heading northbound from Paekakariki will be able to join the Main Alignment from a connection at approximately 01200m.

A subway at 01990m will provide vehicular access across the state highway to three properties. This subway will also provide access across the Main Alignment for pedestrians, cyclists and stock. For the rest of this section heading south, the carriageway will be three lanes in both directions and rises up the Te Puka Stream valley. At approximately 02900m there will be an arrestor bed adjacent to the northbound carriageway for any out of control vehicles heading downhill. The section finishes at 03500m.

Section 2: Wainui Saddle

Section 2 starts at approximately 03500m and will continue climbing for about 2km to the top of the Wainui Saddle at approximately 262m above sea level (at about 05500m). This will be the highest point of the Main

Alignment. Just south of the Wainui Saddle peak at about 05600m there will be a brake check area for both northbound and southbound carriageways. Slightly further south, at approximately 06000m, three lanes in each direction will be reduced to two lanes in each direction. Section 2 finishes at 06500m.

Section 3: Horokiri Stream

This section is approximately 3km long and extends from the southern end of the Wainui Saddle to the northern end of Battle Hill Farm Forest Park. For the entire length of this section, the Main Alignment will run generally parallel to the Horokiri Stream. From 06500m to approximately 08550m the Main Alignment will be to the west of the Horokiri Stream, while from 08550m to 09500m it will be to the east of the stream. As the Main Alignment runs parallel to the stream it will cross a number its minor tributaries which generally run perpendicular to the Horokiri Stream and the Main Alignment.

Over this section, the Main Alignment will cross the Horokiri Stream once with a bridge at 08540m. The section finishes towards to northern boundary of the Battle Hill Farm Forest Park (BHFFP) at approximately 09500m.

Section 4: Battle Hill

This section is approximately 3km long and extends from the northern boundary of the BHFFP to the Pauatahanui Golf Course. Shortly after the Main Alignment enters the BHFFP from the north it crosses over the Horokiri Stream with a bridge at approximately 09720m. Over the remainder of this section heading south the Main Alignment will follow the Horokiri Valley floor which widens from north to south through the BHFFP.

Access across the Main Alignment for park users will be provided by a subway located at approximately 10500m. This will provide a connection between the eastern and western part of the park for pedestrians, cyclists and stock. The Main Alignment will continue south from the BHFFP boundary towards the Pauatahanui Golf Course. At about 11750m it will crosses an unnamed stream with a bridge. Access across the Main Alignment will be available underneath this bridge. The section finishes at 12500m where there will be a subway providing pedestrian and stock access across the Main Alignment.

Section 5: Golf Course

This section is approximately 3km long, and extends from north to south through rural land adjacent to the Pauatahanui Golf Course and Flighty's Road. The Main Alignment will cross a number of small tributaries along this section but there will be no major stream crossings requiring bridges.

Section 6: State Highway 58

This section is approximately 3km long and starts at 15500m. The SH58 / Pauatahanui Interchange will be located at approximately 17500m. At this interchange the Main Alignment will be elevated above a roundabout which will provide access to and from the Main Alignment for traffic travelling in both directions on existing SH58. Immediately south of this interchange, at approximately 17660m, there will be a bridge across the Pauatahanui Stream.

At approximately 18250m the Main Alignment will widen to provide three lanes in each direction. This section finishes at approximately 18500m.

Section 7: James Cook

This section starts just south of the State Highway 58 / Pauatahanui Interchange, at approximately 18500m. Three lanes will be provided for both the northbound and southbound carriageways. The James Cook Interchange will be located at approximately 19500m. This will be a dumbbell interchange with the Main Alignment being elevated above the local road connections. These roads will provide access to the Main Alignment in both directions to and from the Porirua Link Roads. In the vicinity of this interchange, the number of lanes in each direction will be reduced from three to two. This will occur at approximately 18900m in the northbound carriageway and at 19500m in the southbound carriageway. From the James Cook Interchange, the Main Alignment will continue southwards for a further 2km. This section finishes at approximately 21500m.

Section 8: Cannons Creek

This section begins at 21500m and is approximately 3.4 km long. Throughout this section the Main Alignment will run along the eastern side of Duck Creek valley, and across an undulating, weathered greywacke plateau between Duck and Cannons Creeks.

There will be four bridges in this section:

- A 140m long bridge starting at 21555m, crossing a tributary of Duck Creek
- A 150m long bridge starting at 21845m, crossing a tributary of Duck Creek
- A 160m long bridge starting at 22780m, crossing a tributary of Duck Creek
- A 260m long bridge starting at 23550m, crossing Cannons Creek.

These bridges will follow the horizontal alignment of the Main Alignment. This section finishes at 24900m.

Section 9: Linden

This southernmost section is approximately 2.8km long. From the start of the section at approximately 24900m, a third lane will be provided in the northbound carriageway heading uphill.

There will be two bridges:

- A 50m long bridge starting at 25790m, crossing an unnamed stream that flows into the Onepotu arm of the Porirua Harbour
- A 90m long bridge starting at 26010m, crossing an unnamed stream that flows into the Onepotu arm of the Porirua Harbour.

The Kenepuru Interchange will be located at approximately 26700m. This interchange will involve the Main Alignment being elevated above a roundabout which will connect to the Kenepuru Link Road.

South of the Kenepuru Interchange, the Main Alignment will continue downhill to where it will tie into the existing SH1 along the Tawa straight. For traffic joining the Main Alignment in a northbound direction, the carriageway will be elevated and will pass over the existing southbound SH1 carriageway. Traffic continuing to Porirua will be able to do so by taking the left lane exit from the existing SH1.

1.3.2. The Kenepuru Link Road

The Kenepuru Link Road will provide a connection from the Main Alignment to western Porirua. This link road will provide a connection from the Kenepuru Interchange to the existing Kenepuru Drive and will be approximately 600m long. There will be a roundabout at the intersection with Kenepuru Drive. The Kenepuru Link Road will be a State Highway (limited access road) designed to the following standards:

- Two lanes (one in each direction)
- Design speeds of 50 km/h
- Maximum gradient of 8%
- Limited access only.

The Kenepuru Link Road will contain a curved 240m long bridge over the existing SH1 and the NIMT.

1.3.3. Porirua Link Roads

The Porirua Link Roads will connect the Main Alignment to the eastern Porirua suburbs of Whitby and Waitangirua. The Porirua Link Roads will be local roads designed to the following standards:

- Two lanes (one in each direction)
- Design speeds of 50 km/h
- Maximum gradient of 10%
- Some side access will be permitted.

The Waitangirua Link Road will be approximately 2.5km long will run from the James Cook Interchange to the existing intersection of Niagara Street and Warspite Avenue. This will be a signalised intersection. The Waitangirua Link Road will cross five waterways. The most significant of these will be a crossing of Duck Creek requiring a culvert. The Waitangirua Link Road will link into the western side of the James Cook Interchange.

The Whitby Link Road will be 0.9km long and will run from the existing roundabout at the intersection of James Cook Drive and Navigation Drive to the Waitangirua Link Road. The new intersection of the proposed Waitangirua and Whitby link roads will be an unsignalised T-intersection with traffic from the Whitby Link Road giving way to Waitangirua Link Road traffic.

1.4. Development of the Current Design

The scheme assessment report (SAR) was undertaken between 2006 and 2008. The key objective for this phase was to identify the most advantageous route alignment which could then be further refined and used for assessment and consenting.

The SAR is referred to as Phase I and the investigations and assessments (the current phase) are referred to as Phase II. Phase III refers to the consenting of the Project.

Work undertaken on the route since 2006 provided the first real opportunity to conduct on-site, in-depth investigations into the impact of the proposed alignment from an engineering and environmental perspective.

The key aspects that were considered during the SAR phase were:

- Geotechnical constraints
- Physical environmental impacts
- Social impacts
- Cost
- Timeliness
- Network flexibility
- Route performance and safety.

The associated findings from these investigations indicated that the proposed route provides several significant benefits over the existing designated alignment and the coastal route.

The key benefits include:

Improving Route Security

While both the existing coastal route and the Transmission Gully Project route traverse fault lines, the Transmission Gully Project’s proposed design offers greatly improved route security for the existing State Highway 1 and the region's road network over the existing coastal route.

Where the route is vulnerable to damage from major seismic events, engineered earth embankments have been used rather than bridge structures, which will provide greater resilience and allow easier and quicker reinstatement in order to restore road access to the region.

Improving Highway Safety and Function

The alignment will be constructed for open road speed limits (100km/h) and a median barrier will be provided along the entire route. Crawler lanes and an arrester bed as well as ‘run-off areas’ for out of control vehicles) on the steepest sections, along with grade separated interchanges to remove conflicts associated with vehicle turning movements provide additional safety improvements over the coastal route.

Managing Environmental Impacts

Generally, the proposed route provides greater opportunities to manage environmental impacts as compared to the previously designated alignment or the coastal route. The mitigation measures required by conditions on the existing designation (such as the planting of approximately 150,000 native trees and shrubs) will still be able to be utilised in the proposed alignment.

Improving Connections to Local Roads

An eastern Porirua interchange known as the James Cook Interchange will connect to both James Cook Drive in Whitby and Warspite Avenue in Waitangirua, providing improved connections with the wider Porirua area.

The Kenepuru Link Road will also connect the Main Alignment to western Porirua.

2. Purpose and Scope of this Assessment

2.1. Purpose and Scope

This report presents the findings of a hydrology and stormwater control assessment conducted as part of the environmental assessment of the Project. It addresses issues associated with the quantity of water that is expected to run off the land and into the local streams, estuaries and coastal areas. It addresses the impact of the road both during construction, and in the long term, defining:

- Appropriate sizing of culverts and bridges for streams (ephemeral and permanent) that cross the road, including fish passage considerations
- Requirements for outlet erosion control along the alignment
- The impact of increased flows on flood risk
- The impact of stream diversions on stream length/grade and related velocities.

This report also considers the climate change requirements for rainfall runoff. Design flows for sediment control devices (during construction) and permanent stormwater treatment devices is covered in Technical Report 15: *Assessment of Water Quality Effects* (SKM, 2011).

2.2. Environmental Assessment Documents

This report is part of a suite of reports that have been prepared in support of the NoR and resource consent applications for the Project.

The set of application documents comprises:

Volume I	Assessment of Effects on the Environment
Volume II	Resource Management Act 1991 Forms
Volume III	Technical Reports
Volume IV	Plan Set
Volume V	Management Plans

This report forms part of Volume III.

2.3. This Report

This assessment of hydrology and stormwater effects is structured as follows:

Section 1	Introduction
Section 2	Purpose and Scope of this Assessment
Section 3	Existing Environment
Section 4	Methodology

Section 5	Hydrological Modelling and Assessment
Section 6	Hydraulic Modelling and Assessment
Section 7	Summary Assessment of Effects
Section 8	References

Appendix 14.A	Topographic Data for Hydraulic Models
Appendix 14.B	Rainfall Isohyet Report
Appendix 14.C	Flood Frequency Analysis
Appendix 14.D	Calibration Results
Appendix 14.E	Hydraulic Assessment Methodology
Appendix 14.F	Culvert Catchment Assessment
Appendix 14.G	Culvert Catchments
Appendix 14.H	Maps

2.4. Design Standards

The following summarises the design standards for hydrology and stormwater matters under which the Project has been designed and assessed. These standards are based on the following documents, which are considered to represent industry best practice in the New Zealand setting:

- Transit New Zealand's *Bridge Manual* (2003), supported by Austroads (1994)
- NZTA's *Stormwater Treatment Standard for State Highway Infrastructure* (2010)
- Transit New Zealand's F/3 *Specification for Pipe Culvert Construction* (2000)

2.4.1. Bridge and Culvert Design

All cross-culverts have been assessed in accordance with the SAR which required that;

> *Culverts along the route should be sized to convey the critical duration 10% annual exceedances probability rainfall storm event without heading up above the pipe soffit. The road surface level should be at least 500 mm above design stormwater levels for a 1% annual exceedances probability event*. (OPUS, 2008)

These requirements are supplemented by the Transit New Zealand's *Bridge Manual* (2003) sections 2.3.2 (b) and 2.3.4 (a) for larger crossings as follows;

- In the design of a stream crossing, the total waterway shall be designed to pass a 1% annual exceedance probability (AEP) flood without significant damage to the road and waterway structure(s)
- When considering the level of serviceability to traffic the following freeboards shall be used:

Waterway Structure	Situation	Freeboard	
		Measurement Points	Depth (m)
Bridge	Normal circumstances	From the predicted flood stage to the underside of the superstructure	0.6
	Where the possibility that large trees may be carried down the waterway exists		1.2
Culvert	All situations	From the predicted flood stage to the road surface	0.5

A minimum culvert size of 600 mm has been assumed for all catchments. In addition debris control devices and fish passage will be provided where required while maintaining flow.

Transit New Zealand's *Bridge Manual* and relevant material design codes specify levels of adequate durability that culverts, bridge structures, and their component members will comply with for this Project. Culverts and bridges have an expected design life of 100 years.

2.4.2. Outfalls and Erosion Protection

The standards given in NZTA's *Stormwater Treatment Standard for State Highway Infrastructure* (May 2010) for stream erosion control measures are that they are applicable when subject to the following conditions:

- There is a new highway project
- There is a natural stream
- Catchment imperviousness exceeds 3% (although this does not apply to catchments zoned urban)
- There is potential for future development to increase stream channel instability
- There is no tidal influence to the stream where the new highway discharges to it.

Where these criteria apply erosion control measures will be installed to protect receiving environments from erosion.

2.4.3. Fish Passage Design

Fish passage design guidance is provided by the Auckland Regional Council TP131 *Fish Passage Guidelines for the Auckland Region* (2000) and NIWA publications *Fish Passage of Culverts: a Review with Possible Solutions for New Zealand Indigenous Species* (1999), *Successful Fish Passage Past Weirs* (2002), and *Using Ramps for Fish Passage Past Small Barriers* (2003).

In streams where fish passages are required, the culvert pipe size has been upgraded by 300mm. This allows pipes to be countersunk by 300mm to form a continuous wetted perimeter making the culvert passable to native fish species. Where this is not possible an alternative fish passage solution has been used, discussed in more detail in Technical Report 11: *Ecological Impact Assessment* (Boffa Miskell, 2011).

2.4.4. Surface Drainage

The conveyance and collection system will be designed so that no more than 4mm depth of water occurs at the edge of the traffic lanes for a 5% annual exceedance probability (AEP), 10 minutes duration design storm event. This will reduce the risk of hydroplaning / aquaplaning.

Where wetlands are provided these will have extended detention for the first 34.5mm of rainfall, which will reduce potential for stream bank and bed erosion.

Surface drainage piping systems have an expected design life of 50 years.

2.4.5. Stormwater Treatment

The NZTA's *Stormwater Treatment Standard for State Highway Infrastructure* (2009) provides a variety of stormwater treatment techniques for stormwater runoff from State highway infrastructure. The methods used in the Project have been selected from this document.

The first flush in a rainfall event will be treated. This has been calculated to be 27mm of rainfall depth (including an allowance for climate change). See Technical Report 15: *Water Quality Assessment of Effects* for the derivation of this standard.

2.4.6. Assessment of Flooding

The guidance given in NZTA's *Stormwater Treatment Standard for State Highway Infrastructure* (May 2010) for flooding are that the following should be considered when determining where and how flooding should be assessed:

- Are there existing flooding problems downstream?
- Where is a given highway located within a catchment?
- What is the development potential for a given catchment?
- Are there downstream points of control that would mitigate any possible effects that a given project might have on flooding?

This report assesses flooding for the 10% and 1% AEP flooding events, including mid-range climate change. These storm events have been chosen to illustrate compliance with the NZTA requirements for bridges and culverts. During the preparation of the scope of this report it was decided not to assess smaller events (such as the two year event) in line with the NZTA guidance on the basis that the systems are all either:

1) Rural with minimal risk to dwellings in events less than a 10% AEP rainfall event
2) Residential with a floodplain set aside with minimal risk to dwellings in events less than a 10% AEP event
3) Residential where piped networks are designed to have a minimum capacity for a 10% AEP event.

2.4.7. Other Relevant Policy and Guidance

Other policy and guidance documents from local authorities and further abroad have been used to assist in the design and assessment of hydrology and stormwater matters. These include:

- Greater Wellington Regional Council’s *Regional Freshwater Plan* (1999)
- Auckland Regional Council’s TP10 *Design Guideline Manual for Stormwater Treatment Devices* (2003).
- ARRB Austroads *Guidelines for the Collection and Discharge of Stormwater from the Road Infrastructure* (1994)
- Transit New Zealand’s *Environmental Plan: Improving Environmental Sustainability and Public Health in New Zealand*, Version 2 (2008)
- Local Authority Guidance on drainage design.

3. Existing Environment

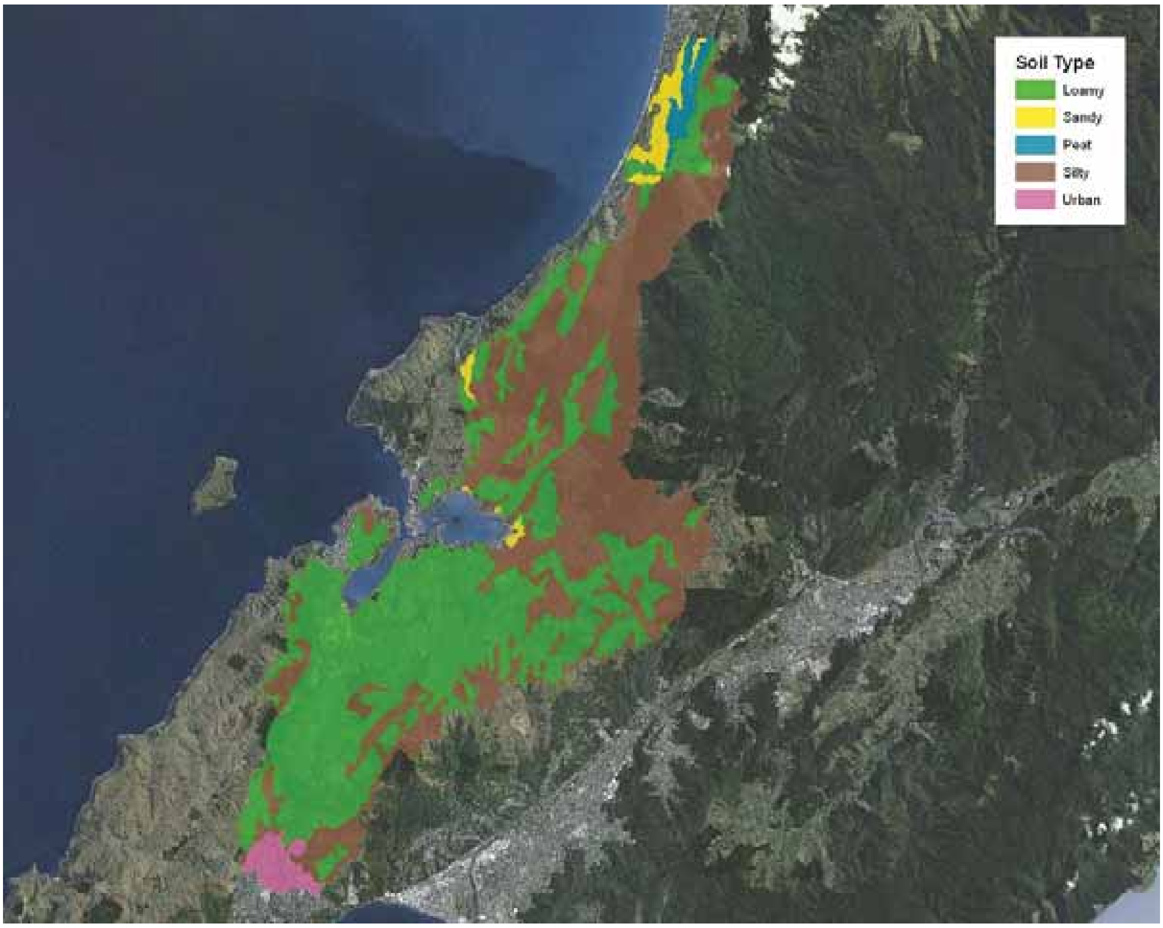
3.1. Catchment Soils and Land Use

The proposed road is largely greenfield and runs through pasture and shrublands as well as some pockets of native and exotic forest (Refer to **Map 1 Catchment Land Cover**, **Appendix 14.H**). The current landuse reflects a heavily modified environment as historically these catchments would have been characterised by podocarp dominated native bush.

General soil types in the Project area are shown in **Figure 14.1**. Soils have also been categorised on the basis on infiltration rates (Refer to **Map 2 Soil Drainage Classes**, **Appendix 14.H**) for the purposes of this assessment.

A	High infiltration rates even when thoroughly wetted. High rate of water transmission (greater than 7.5mm/hr)
B	Moderate infiltration rates when thoroughly wetted These soils have a moderate rate of water transmission (4 – 7.5mm/hr)
C	Low infiltration rates when thoroughly wetted These soils have low rate of water transmission (1.25 – 4mm/hr)
D	Very low infiltration rates when thoroughly wetted. These soils have a very low rate of water transmission (0-1.25mm/hr)

As can be seen from Map 2, soil drainage rates are consistently classed as low, reflecting the steepland soils associated with the proposed road alignment.



- **Figure 14.1 - Soil Types in the Project Area**

3.2. Catchments

Two thirds of the route traverses catchments that converge on Pauatahanui Inlet, including Horokiri Stream, Ration Stream, Pauatahanui Stream and Duck Creek. Most of the rest of the route is within catchments that flow to the southern arm of the Porirua Harbour by way of Kenepuru Stream. Only the northernmost 5 km encompassing the Te Puka catchment does not drain toward Porirua Harbour/Pauatahanui Inlet, but instead joins the Wainui Stream to flow across the narrow coastal plain to the Wainui Stream mouth, north of Paekakariki.

The Greater Wellington Regional Council (GWRC) classifies freshwater bodies in the *Regional Freshwater Plan* (GWRC, 1999) for the Wellington Region according to their values. Some waterbodies have multiple values and the intent is that they are managed to provide for all of these. The Horokiri, Ration, Pauatahanui, Wainui, Whareroa and Duck are all either managed for aquatic ecosystem values, or for nationally threatened indigenous fish species.

Technical Report 9: *Freshwater Habitat and Species: Description and Values* (Boffa Miskell, 2011) provides a great level of detail on the specific values captured within the GWRC freshwater plan.

3.3. Streams

The proposed road crosses eight catchments and will involve earthworks, the construction of a number of large culverts and bridges and significant lengths of both temporary and permanent stream diversions (refer to **Map 3 Stream Catchments**, **Appendix 14.H**).

Many of the waterways crossed by the proposed road are small ephemeral streams. These waterway crossings are technically simple and need to be:

- Appropriately sized
- Provide fish passage as required by the project ecologists
- Consider the management of debris above the inlet
- Carefully consider outlet erosion control.

There are also a number of crossings of larger waterways with the potential for a broader and more significant set of effects. In particular for this assessment, an increase in the quantity of runoff can impact on flood risk, as can the sizing of major culverts and the selection of grade and size for stream diversions.

The scale of the Transmission Gully Project necessitated a targeted approach to the assessment of impacts for streams. A risk assessment process (**Table 14.1**) was undertaken to identify where more detailed investigation of streams would be required. Those identified were tagged for closer scrutiny using hydraulic models for potential impacts due to a combination of a range of factors including the scale of works in the stream bed, major constraints to the floodplain, ecological significance or flood risk potential. The lower level of relevant risks or the smaller scale of works in other watercourses justified a more generalised approach to the assessment of impacts in those watercourses.

- **Table 14.1 Risk Assessment**

Catchment	Risk Assessment Criteria				Risk Assessment Outcome
	Diversions of Major Watercourses	Major Constraints to the Waterway or Floodplain	Significant Downstream Flood Risk Potential	Aquatic Habitat Value (from Technical Report 11)	
Collins Stream	No	No	No	Not assessed	Standard assessment required
Duck Creek	No	Yes	Yes	Mostly high	A more detailed investigation required
Horokiri Stream	Multiple	Yes	Flood prone rural properties, some dwellings at risk	High	A more detailed investigation required
Kenepuru Stream	No	No	Yes – stormwater enters urban network	Low to moderate	The urban stormwater network requires a more detailed investigation, otherwise a standard assessment required

Pauatahanui Stream	Diversion required at the SH58 interchange	Yes	Yes	Some high	A more detailed investigation required
Porirua Stream	No	No	Yes – stormwater enters urban network	Low	The urban stormwater network requires a more detailed investigation, otherwise a standard assessment required
Ration Stream	No	Yes	No	Mostly low	Standard assessment required
Te Puka	Yes	Yes	Residential property in the vicinity of Tilley Road already flood prone	Mostly high	A more detailed investigation required
Whareroa	No	No	No, little development downstream (QEII Park)	Not assessed	Standard assessment required

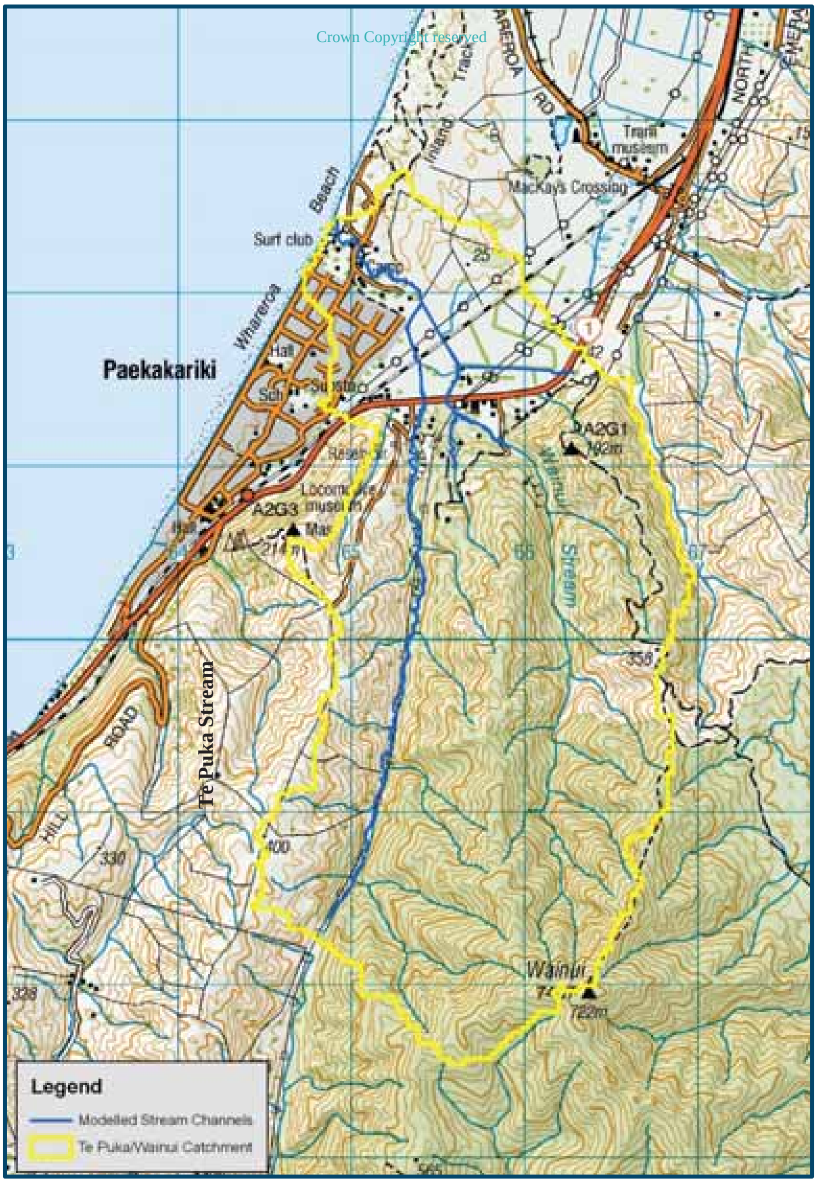
This is consistent with the SAR (OPUS, 2008), which identified stream catchments where there was potential for significant effects depending on the design of the alignment. These stream catchments were Wainui/Te Puka, Horokiri and Duck. In addition we have also identified the Pauatahanui Stream, due to the potential influence of the interchange and site compound on the existing flood risk. Each of these streams is discussed in more detail below.

3.3.1. Wainui / Te Puka

The Te Puka and Wainui streams have a catchment area of some 8 km^2 (830 ha) and drains through a very steep gradient north from the Wainui saddle (around 500m above Mean Sea Level (AMSL)) , as shown in **Figure 14.2**. The streams pass under SH1 and converge into the Lower Wainui Stream north of the Paekakariki residential area and follow a gentle gradient to the sea.

Landuse in the local drainage area is a mixture of residential, commercial and agriculture on the coastal dunes and a mixture on pasture, plantation pine and native bush on the hills.

The topography traversed by the Te Puka and Wainui Streams is typical of the Kapiti region. The steep upper catchment drops down onto an undulating dune environment. This change in grade between the hills and the coastal zone, combined with the restrictions as the stream runs through the dunes, has resulted in historical flooding problems for the developed land surrounding the stream.



■ **Figure 14.2 - Te Puka and Wainui Catchment**

The assessment of the Te Puka and Wainui streams has been targeted to address the following constraints:

- The coastal plain is adjacent to Paekakariki and the stream has previously added to flooding of residential property in the vicinity of Tilley Road
- The stream is subject to rapid aggradation due to erosion in the upper catchments
- There are two major crossings, one of the Te Puka Stream and one of the Wainui Stream
- A significant diversion in the upper Te Puka arm of the stream will need to be assessed in detail to ensure channel form and ecological outcomes are appropriately understood.

3.3.2. Horokiri Stream

The Horokiri stream has a catchment area of some 33 km^2 (3300 ha) and drains from north to south from the Wainui Saddle at around 500 m AMSL down into the head of the Pauatahanui Inlet.

The assessment of the Horokiri Stream has been targeted to address the following constraints:

- While there is not flooding of a residential community in this catchment, there are a number of flood prone rural residential properties. In addition there is some significant filling of the floodplain, particularly through Battle Hill
- There are a number of major crossings on the main stem waterway which need to be assessed in detail
- There are several diversions, particularly in the upper Horokiri, that need to be assessed.

The Horokiri Stream also discharges into the head of the Pauatahanui inlet. Water quality parameters for this waterway are described in Technical Report 15: *Water Quality Assessment of Effects* (SKM, 2011).

3.3.3. Pauatahanui

The Pauatahanui catchment is the largest affected by the project at around 41 km^2 (4170 ha). The majority of the feeder tributaries arise in the south of the catchment from the hills at an altitude of around 430 m AMSL.

The assessment of the Pauatahanui Stream has been targeted to address the following constraints:

- The major interchange and associated facilities (site compound) that will exist in this location following road construction. The interchange and site compound are sited within an existing floodplain and need to be assessed on this basis
- There is a major stream crossing and diversion associated with the interchange that needs to be detailed.

The Pauatahanui Stream also discharges into the head of the Pauatahanui Inlet. Water quality parameters for this waterway are described in Technical Report 15: *Water Quality Assessment of Effects* (SKM, 2011).

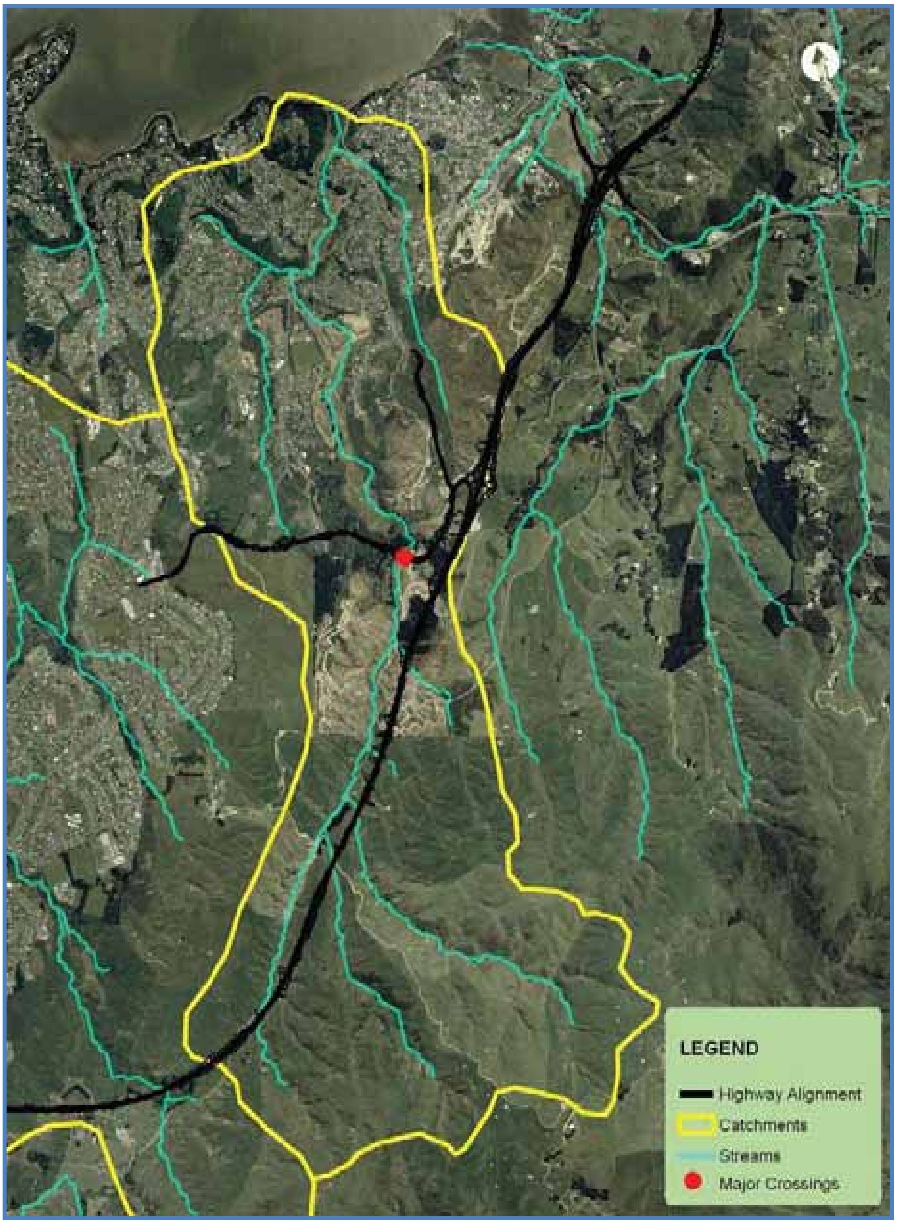
3.3.4. Duck Creek

Duck Creek is a north-facing catchment, collecting runoff from the rural land in the upper catchment before passing through the suburban development of Whitby and discharging to Pauatahanui estuary, as shown in **Figure 14.3**. The creek has a catchment area of some 10 km^2 (1030 ha) and drains west through a very steep gradient from 490 m AMSL to sea level over a distance of around 7.2 km.

Duck Creek runs through an existing residential community below the State highway. Many of the lower lying areas in this community are currently open space but are under significant development pressure. There is also a major crossing of Duck Creek as part of the Waitangirua Link Road which will need to be considered in some detail.

Duck Creek discharges into the head of the Pauatahanui Inlet. Water quality parameters for this waterway are described in Technical Report 15: *Water Quality Assessment of Effects* (SKM, 2011).

While there are some good habitat in the lower reaches of the stream, fish passage in the upper catchment is restricted by both the steep topography and three track crossing culverts which are perched and do not provide a continuous up-stream swimming passage.



- **Figure 14.3 - Duck Creek Catchment**

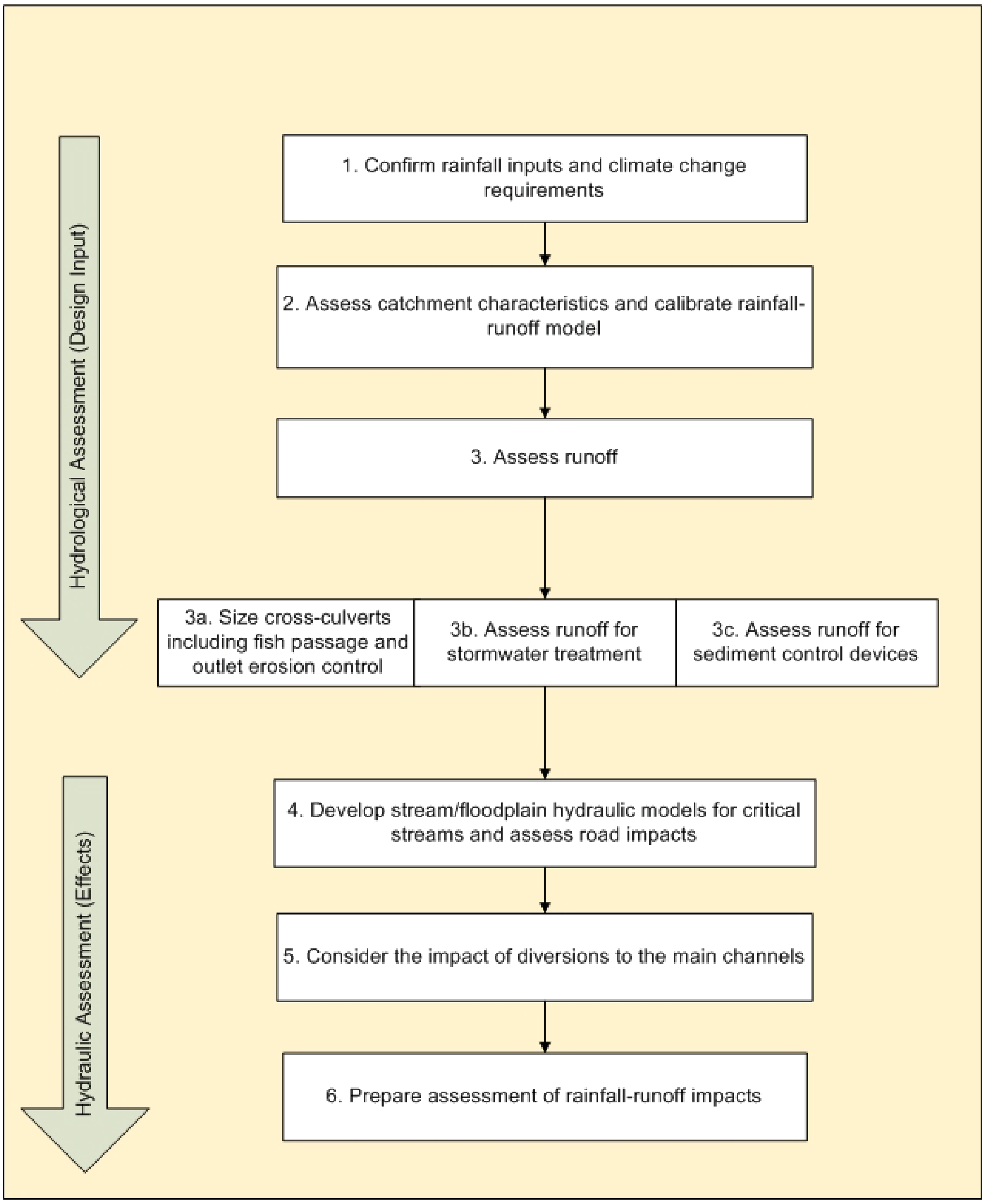
3.4. Urban Stormwater Networks

Stormwater runoff from some sections of the Project will not be discharged to natural waterways but rather directly into the urban stormwater network. This is the case in both the Waitangirua and Linden catchments and as such these have been assessed in greater detail to ensure local effects of increased flows can be mitigated.

There is also a significant portion of the road that will drain into the Kenepuru Stream and lower Porirua Stream via the urban stormwater network, but in these cases storage is available behind the culverts to mitigate increased peaks. As this storage is in a rural setting with no chance of affecting existing dwellings a first principals assessment of this risk is appropriate.

4. Methodology

The assessment of rainfall-runoff effects falls into two broad categories, the hydrological assessment that determines the amount of runoff that will occur in a given storm event (Section 5), and the hydraulic assessment that determines the hydraulic effects of those given events on the environment (Section 6). A summary of key steps undertaken for this assessment is provided in **Figure 14.1.**



- **Figure 14.4 Rainfall-Runoff Assessment Process**

4.1. Assumptions

Climate Change

The predicted mid-range impacts of climate change were added to the 1% AEP rainfall event to help ensure that the assessment of effects would be appropriate for the foreseeable life of the asset being constructed. The impacts of climate change have been predicted by The Ministry for the Environment in the report *Preparing for Climate Change – A Guide for Local Government in New Zealand* (July 2008). The predictions include a 16% increase in heavy rainfall intensities by the year 2090.

It should be noted that while climate change predictions have been used in the past to help set freeboard requirements for bridges and culvert structures, we have incorporated mid range climate change into the culvert sizing and then assumed freeboard as discussed in Section 2.4.1. This makes the culvert sizing conservative and also provides a further allowance (freeboard) to manage the risk of higher climate change scenarios.

Hydrological Scenarios - Waterway Crossing Design

The hydrology used in the design of the cross-culverts was developed using long-term predictions of development in the catchment. Based on the district plan maps for the area, all zones of potential development were predicted to be fully developed in accordance with current land use requirements. The hydrological impacts of the Transmission Gully Project were also incorporated into this predicted catchment.

To demonstrate that the new culverts are in compliance with the design standards contained in the NZTA *Bridge Manual* (2005), these future development catchments have been modelled for both the 10% AEP ($Q10^{cc}$) and 1% AEP ($Q100^{cc}$) rainfall. These design flows also includes the predicted 2090 mid-range impacts of climate change to ensure the culverts perform for their full design life which is typically 80-100 years.

Hydrological Scenarios – Flood Risk

For the purposes of assessing the potential changes in runoff, the following scenarios have been used:

- Pre-construction situation (baseline): Construction of the highway is scheduled to begin in 2016. Pre-construction catchments are therefore based on existing land use with the planned and likely development in the catchment in 2031 without the Transmission Gully Project. These catchments have been modelled for both the 10% and the 1% AEP rainfall event, including the predicted mid-range impacts of climate change
- Post-construction situation: post-construction catchments have used the same 2031 catchment characteristics but also include any catchment changes directly associated with the Transmission Gully Project. These catchments have been modelled for both the 10% and the 1% AEP rainfall event, including the predicted mid-range impacts of climate change.

4.1.1. Data Collection

A range of data was collected as part of this assessment including:

- Catchment land use (Refer to **Map 1 Catchment Land Use**, **Appendix 14.H**)
- Soil information from the New Zealand Land Resources Inventory (NZLRI) (Refer **Map 2 Soil Drainage Classes**, **Appendix 14.H**)

- Stream and catchment boundaries (Refer to **Map 3 Stream Catchments**, **Appendix 14.H**)
- Rainfall data from various stations (Refer **Map 4 Rainfall and Flow Gauging Stations**, **Appendix 14.H**)
- Flow data (Pauatahanui at Gorge, Horokiri at Snodgrass and Poriura at Town Centre) (Refer **Map 4 Rainfall and Flow Gauging Stations**, **14.Appendix H**)
- Test pits which provided soil and gravel information at a greater level of detail
- Topographic data used in hydraulic models (Refer to **Appendix 14.A**)
- Surveyed cross sections of critical waterways.

4.2. Design Rainfall Definition

Meteorological and hydrological analyses were undertaken to provide up-to-date information for input to road drainage and waterway crossing design. In 2008 SKM updated the 24-hour storm isohyet maps for the Kapiti Coast District Council. These plans take all of the available local daily rainfall records, and combine them with regional instantaneous recorders to develop a regional model for design rainfall depths. This is a similar approach as taken to the regionalised model in Auckland (TP108) and has a similar benefit in that it feeds directly into unit hydrograph-based estimates of design peak flows.

For this project, additional rainfall data was collated and analysed to extend these isohyet maps south to Wellington. The process included:

- Collection of daily and sub-daily rainfall data recorded at all the rainfall stations in the study area
- Analysis to determine period and completeness of the rainfall records
- Extraction of daily, and where data is available 24-hour, annual maximum rainfall depth time series for each rainfall record
- Determine suitable factor to adjust daily rainfall maxima to 24-hour maxima
- Regional statistical analysis of annual maximum 24-hour rainfall to determine rainfall depths for return periods of 2, 5, 10, 20, 50 and 100-years for all rainfall stations
- Impact of projected climate change on return period rainfall for 2090 time horizon
- Preparation of isohyet maps for each return period and current and 2090 climate change scenarios.

Details of this analysis are covered in **Appendix 14.B Rainfall Isohyet Report**.

4.3. Catchment Characteristics

Once the rainfall has been defined, there are a range of empirical methods for calculating peak flows for further analysis, all of which require an input of catchment characteristics.

For this project we have chosen to develop a unit hydrograph based approach to the peak rainfall inputs. The main advantages of this approach are that it:

- Connects directly with 24-hour rainfall isohyets model
- Provides peak flow analysis required for culvert sizing and assessment of flood risk
- Provides storm volume that allows for the assessment of storage behind culvert entrances and an assessment of loss of storage in the floodplain

- Can be readily calibrated based on catchment characteristics and has been used widely in New Zealand for this purpose so that good data exists for calibration parameters.

4.3.1. Runoff Volume and Losses

A technical discussion of the unit hydrograph methodology can be found in Hoggan (1997) and is briefly discussed below.

The SCS Runoff Curve Number method relates runoff losses to rainfall based on the empirical curve number equation below (Hoggan, 1997).

$$Q = \frac{(P - I_a)^2}{P - I_a + S}$$

$$S = \frac{1000}{CN} - 10$$

Where, Q= Accumulated runoff (inch)

P= Accumulated rainfall (inch)

I_a= Initial abstraction (inch)

S= Potential maximum retention after runoff begin (inch)

CN= Curve Number, percent of runoff

Hoggan (1997) used an empirical equation I_a=0.2S to estimate the initial abstraction in his above equation. This assumption appears to overestimate I_a in the New Zealand context however. This was noted in the Auckland situation in TP108, and reflected in more recent work undertaken by Watts (2002) in urban catchments in Kapiti and Tauranga.

Recently, as part of preparing for updated catchment plans in Kapiti, local work was undertaken that considers the initial abstraction in Kapiti's rural rivers and streams (SKM, 2008). The results of this work suggest that the I_a for rural catchments is between 4 and 5 mm. These results were similar to work previously undertaken in Auckland and on this basis we have assumed that for catchments with 0% connected impervious area (CIA) the I_a is 5mm and 0mm for catchments which have 100% CIA. A weighted average is applied for all other CIA values.

4.3.2. Catchment Delineation

Catchments and sub-catchments have been delineated along the alignment to provide a logical framework for analysis. These are labelled based on underlying catchment boundaries (e.g. Rations, Horokiri, Te Puka etc.). It has been assumed in the analysis of the cross-culverts that the catchment runoff does not include the road drainage as this will be directed to discharge on the downstream side of the culverts.

4.3.3. Runoff Factors

The empirical unit hydrograph approach we have selected (Clarke) utilises the US Soil Conservation Service (SCS) loss method. The SCS runoff curve number method developed by the SCS relates accumulated rainfall excess (or runoff) to accumulated rainfall with an empirical curve number. The curve number is a function of soil classification and land cover.

4.3.3.1. Soils Classification

The variation in the infiltration rates of different soils is incorporated in the curve number selection through the classification of soils into four hydrologic soil groups: A, B, C and D. These represent soils having high, moderate, low and very low infiltration rates are described in **Table 14.1** (Hoggan, 1997).

- **Table 14.2 - SCS Soil Classification**

Soil Classification	Description
A	Low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep well to excessively drained sands or gravels and have a high rate of water transmission (greater than 7.5 mm/hr)
B	Moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep moderately well drained to well drained soils with moderately fined to moderately coarse textures. These soils have a moderate rate of water transmission (4 – 7.5 mm/hr)
C	Low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine textures. These soils have low rate of water transmission (1.25 – 4 mm/hr)
D	High runoff potential. Very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with high swelling potential, soils with permanent high water table, soils with a clay pan of clay layer near the surface and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-1.25 mm/hr)

To classify the soils within the Project catchments, the New Zealand Land Resource Inventory (NZLRI) was utilised. The NZLRI classifies soil drainage parameters for soils throughout New Zealand on a scale between 1-5 from very poor to well drained. It also classifies the permeability of soils in three classes from slow to rapid.

A combination of the NZLRI drainage class and permeability class was used to classify soils into the SCS soil classification, as shown in **Table 14.1**. These related classifications are shown in **Table 14.2**.

- **Table 14.3 - NZLRI and SCS Soil Classification**

NZLRI Permeability Class	NZLRI Drainage Class	(SCS) Hydrological Drainage Class
Rapid	Well	A
Rapid	Moderately well	A
Rapid	Imperfect	B
Moderate/Rapid	Well	B
Moderate/Rapid	Moderately well	B
Moderate/Rapid	Imperfect	B

NZLRI Permeability Class	NZLRI Drainage Class	(SCS) Hydrological Drainage Class
Rapid	Poor	C
Moderate/Rapid	Poor	C
Moderate	Well	C
Moderate	Moderately well	C
Moderate	Imperfect	C
Rapid	Very poor	D
Moderate/Rapid	Very poor	D
Moderate	Poor	D
Moderate	Very poor	D
Moderate/Slow	Well	D
Moderate/Slow	Moderately well	D
Moderate/Slow	Imperfect	D
Moderate/Slow	Poor	D
Moderate/Slow	Very poor	D
Slow	Well	D
Slow	Moderately well	D
Slow	Imperfect	D
Slow	Poor	D
Slow	Very poor	D

The NZLRI classifies soils based on their physical properties, including their gravel content. The gravel classes from the NZLRI are provided below in **Table 14.3.**

Information taken from test pits taken along the proposed road alignment indicated that in some catchments the gravel content was likely to be much higher than the NZLRI assessment.

To ensure that the correct soil characteristics were represented in the study, the initial classification of soils into the SCS soil groups (**Table 14.2**) was then modified dependent on a combination of existing test-pit and NZLRI data (**Table 14.3**).

- **Table 14.4 - Gravel Content Classification**

Gravel Class	Description	%	SOIL Class (A,B,C,D)
1	Non-gravelly to very slightly gravelly	0-4	Use Table 14.2 to determine

Gravel Class	Description	%	SOIL Class (A,B,C,D)
2	Slightly gravelly	5-14	Use Table 14.2 to determine
3	Moderately gravelly	15-34	B
4	Very gravelly	35-69	A
5	Extremely gravelly	70-100	A

4.3.3.2. Land Cover

For each of the modelled scenarios, a CN number was calculated using a combination of the soil classification and the land cover. The matrix used to describe the catchment conditions is covered in **Table 14.4.**

- **Table 14.5 - CN Number**

Land Cover Group	Hydrological Drainage Class			
	A	B	C	D
Urban (not connected)	39	61	74	80
Parkland/gardens	39	61	74	80
Water	100	100	100	100
Crops	66	77	85	89
Pasture/grassland	49	69	79	84
Bush	30	48	65	73
Plantation forest	43	65	76	82
Wetlands	48	67	77	83
Transport infrastructure	98	98	98	98
Fallow	76	85	90	93
Bare	77	86	91	94

4.3.3.3. Time of Concentration

The time of concentration (Tc) is the time taken for water to travel from the remotest part of the catchment to the head of the drain or culvert in question.

Many of the catchments potentially affected by the Project are large rural catchments where the flow path from the remotest part of the catchment must be defined and the length and average slope calculated for each section of the flow (overland, channel and pipe flow). The minimum time of concentration is 10 minutes in all areas.

Flow paths were identified using ArcGIS software to provide a consistent level of accuracy for the large number of catchments to be analysed. Catchments were defined as polygon shapes and overlaid on a digital elevation model (DEM). The model was created using a 5 m resolution, where sufficient elevation data existed. The DEM allowed for the creation of a flow direction and pathway layer for each catchment polygon. Based on this information the time of concentration was calculated.

Time of concentration is made up of the following components:

Time of concentration = overland flow + open channel flow + kerb and channel flow +pipe flow

Overland Flow

Time of overland flow can be obtained using the empirical formula (NZIE, 1980):

$$t = 100\frac{nL^{0.33}}{S^{0.2}}$$

Where t is the time of concentration in minutes, L is the length of overland flow in metres, S is the slope in percent and n is a value selected for surface roughness (Refer to **Table 14.5**).

- **Table 14.6 - Surface Roughness Values**

Surface	n
Paved	0.015
Bare Soil	0.0275
Poorly Grassed	0.035
Average Grassed	0.045
Densely Grassed	0.06

Open Channel

Time of open channel flow has been obtained using Manning's formula, assuming a trapezoidal channel cross-section.

Kerb and Channel Flow

Time of kerb and channel flow has been obtained using Manning's formula for n 0.018 as an average value.

Pipe Flow

Time of pipe flow can be obtained using Manning's formula.

For larger catchments, alternative methodologies such as the Ramser-Kirpich have been found to be appropriate for Wellington. When comparison is made between the method used and the Ramser-Kirpich time of concentration method, very good agreement is found, with time of concentration from the method used tending to be slightly longer.

4.4. Rainfall-Runoff Calibration

While it is acceptable to develop rainfall runoff models based on known catchment characteristics where little empirical data exists (rainfall and gauged stream flows), it is common practice to undertake a calibration and validation of the model when there is historical data available. In this case there are a number of rainfall stations and three gauged flow recorders (Refer to **Map 4 Rainfall and Flow Gauging Stations**, **Appendix 14.H**) that can be used for the calibration analysis as detailed below.

4.4.1. Analysis of Gauged Flows

The first step of the calibration process is to undertake a frequency analysis on the gauged flow recorders. It should be noted that the quality of gauged records is mixed and based on a large number of factors including the length of record, quality of the site, monitoring programme and the number of larger events for which the site has been rated (a detailed assessment on depth verses flow). In the case of these three gauging sites, the Horokiri stands out as a site with a shorter record and less rating data, particularly in high flows.

4.4.1.1. Data

The Horokiri, Pauatahanui and the Porirua Streams are continuously gauged. The flow records from these streams have been used to verify our rainfall runoff hydrological models against flows measured in real events, and for calibrating stream models against statistical estimates of return period flows.

Flow data, at 15 minute time steps, was obtained from the three gauges and used to prepare annual maximum discharge time series for statistical analysis. The data is summarised in **Table 14.6.**

- **Table 14.7 Summary of Streamflow Gauges**

Streamflow Gauge	Area (km^2)	Start and end date	No of Maxima
Pauatahanui Stream at Gorge	38.27	Jun 1975 – Dec 2009	35
Porirua Stream at Town Centre	40.30	Sep 1965 – Feb 2010	45
Horokiri Stream at Snodgrass	28.69	Feb 2002 – Feb 2010	8

This data is used to assess flood frequency, which provides a peak flow for different return period storm events for each of the catchments. The flood frequency analysis and results are provided in **Appendix 14.C – Flood Frequency Analysis**.

4.4.2. Selection of Calibration Events

The second phase of the calibration process is to select the events against which calibration should be undertaken.

Calibration of the Porirua catchment was undertaken in significant detail in 1997 (Connell Wagner, 1997). The catchment was calibrated against a hydraulic model and took into account upstream catchment constraints including peak attenuation dams in the upper catchment. The information found in the Connell Wagner (1997) report is used as the calibration of this catchment for the Project.

Calibration of the hydrological models for the Project has therefore been carried out for two catchments, the Horokiri and Pauatahanui. We have selected the largest events on record where we have corresponding rainfall records for the analysis.

Data collection has provided a number of mid range events. Although these events may not represent the 'extreme event' calibration we are ideally looking for, they can provide a useful understanding of catchment characteristics for application in the more extreme event analysis. The larger events will be used for the calibration proper to try and reflect the 'extreme event' being assessed for design and flooding. On this basis we would expect there to be some over estimation of the verification events.

Three events will be used for calibration and two events for validation. Events considered at each site are shown in **Table 14.7**.

- **Table 14.8 - Events Selected for Calibration**

	Date	Approx. Peak	Approx. Return Period	Name
Horokiri Stream	05/01/2005	49	7	Calibration A
	15/02/2004	36	3	Calibration B
	03/10/2003	70	20	Calibration C
	17/11/2006	26	2	Validation D
	30/08/2009	24	2	Validation E
Pauatahanui Stream	05/01/2005	67	17	Calibration V
	15/02/2004	66	17	Calibration W
	03/10/2003	65	16	Calibration X
	21/12/1982	66	17	Validation Y
	23/07/2009	47	3	Validation Z

The objectives of the calibration are therefore to:

- Using calibration of three initial hydrographs, identify appropriate catchment parameters for the rainfall-runoff model (calibrating the curve number, initial abstraction, storage ratio and time of concentration parameters)
- Verify these results with a further two hydrographs.

4.4.3. Calibration

The hydrologic model of the catchments has been analysed and modelled using the HEC-HMS software to produce flood hydrographs. The computed values were then compared to observed storm events. The catchment parameters were calibrated to the observed storms by comparing the discharge.

4.4.3.1. Assumption in Rainfall Data

Three rainfall recording stations were used to provide rainfall information for the two catchments. The rainfall recorders "Taupo Stream at Whenua Tapu" and "Whakatiki River at Blue Gum Spur" were used to provide rainfall data for the Horokiri and Pauatahanui catchments. The rainfall recorder "Taupo Stream at Whenua Tapu" has a continuous rainfall record from July 1992 through to present; the "Whakatiki River at Blue Gum Spur" has a continuous rainfall record from October 1981 to present. Continuous raingauge stations provide 'shape' to a rainfall event as the rainfall data is collected at short intervals (often 15mins) throughout the day. (This kind of station can be used to provide the 'shape' of a rainfall event from local raingauge sites that may only provide a record of daily rainfall depths. This allows daily rainfall stations to be adapted for calibration against continuous stream flow information for the same storm event).

How these two rainfall recorders relate geographically to the streams being assessed for peak flow is shown in **Map 4 Rainfall and Flow Gauging Stations**, **Appendix 14.H**. The location of these particular sites was useful, as it would be expected that in dominant rainfall conditions (north to northwest storms) they would provide a good picture of the lower and upper rainfall depths that would be expected in the Horokiri and Pauatahanui Streams. The rainfall distribution recorded by "Taupo Stream at Whenua Tapu" was scaled in all events on the Horokiri catchment by half of the difference in volume between the records of "Taupo Stream at Whenua Tapu" and "Whakatiki River at Blue Gum Spur". The rainfall distribution recorded by "Taupo Stream at Whenua Tapu" was chosen to describe the rainfall distribution across the catchment.

For all events on the Pauatahanui catchment a similar scaling process was applied. A rainfall gauge exists within the Pauatahanui catchment, "Judgeford", which has a daily rainfall record from October 1978 to present. For the relevant events post 1992 the rainfall distribution recorded by "Taupo Stream at Whenua Tapu" was scaled to match the volume recorded at "Judgeford". For the one event in 1982 the rainfall distribution was taken from "Whakatiki at Blue Gum Spur" and scaled to match the volume at "Judgeford".

4.4.3.2. Base Flow Separation

For events that had a significant initial baseflow component, a constant baseflow value was included in the HEC-HMS simulation equal to the lowest value of the initial flow.

4.4.3.3. Calibration Factors

The primary purpose of the calibration is to obtain best fit parameters for the rainfall-runoff model. In the Clarke Unit Hydrograph methodology (Hoggan, 1996) the input parameters to estimate the flood hydrographs are runoff curve number (CN), connected impervious area (CIA), initial abstraction (I_a), storage ratio (R), and time of concentration (Tc). The storage coefficient (ST) is a function of R and Tc.

$$ST = \frac{RTc}{1 - R}$$

In the SCS dimensionless unit hydrograph methodology the CN value accounts for catchment losses. This is most accurately obtained through analysis of the soil and land use types and should only be altered slightly to calibrate the hydrograph with observed events. Likewise the CIA is a factor of the catchment landuse; and the I_a has been based on specific catchment studies, so neither parameter should be varied during calibration.

The overall shape and position of the hydrograph are dependent on the storage coefficient and time of concentration respectively. During the catchment delineation process, time of concentration was estimated based on first principles of overland flow and channelised flow. The Ramser-Kirpich methodology was also trialled and gave very similar results. In the calibration process it was seen that the calculated times of concentration estimated actual times of concentration accurately.

Previous analysis has been completed in the Wellington region based on this methodology. These previous studies include the Porirua report previously referenced (Connell Wagner, 1997), the *Wharemauku Stream Floodplain Management Current Status Report* (SKM, 2009), which is a catchment of similar characteristics just to the north of the study area, and SKM (2008) *Update of Kapiti Coast Hydrometric Analysis* report which assessed typical initial abstraction values for the Kapiti Coast District.

4.4.4. Summary

Details of the technical calibration assessment are covered in **Appendix 14.D – Calibration Results**. Final calibrated catchment characteristics used for the three catchments are summarised in **Table 14.8.**

- **Table 14.9 Calibrated Model Parameters in HEC-HMS**

Catchment Name	Area (km^2)	Initial Abstraction (mm)	SCS Curve No	Connected Impervious Area (CIA)	Tc (hr)	Ratio
Horokiri	28.7	5	80	0	2.0	0.65
Pauatahanui	38.3	5	75	0	1.7	0.65
Porirua (1997)	40.0	4	78	16	2.6	-

Initially, assessed runoff volumes were in the right order of magnitude, hence CN numbers only needed to be adjusted slightly. We raised storage ratios reflecting a higher volume of storage in these catchments than initial estimated values. Times of concentration were found to be calculated accurately as well as initial abstraction.

The calibration of design storm parameters is considered to be good for many of the events and acceptable overall.

4.5. Hydraulic Modelling Methodology

4.5.1. Stream Modelling Methodology

For each of the detailed stream investigations (see Section 3.3 for the selection criteria for detailed assessment), a combined 1D and 2D hydraulic model was constructed using the DHI software package MIKEFlood. The lateral linking capability of MIKEFlood was used to combine the 1D model of the stream channel constructed in MIKE11 and a MIKE21 2D model of the floodplain.

This modelling technique maximises 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.

Modelling of floodplains using a 2D hydraulic model does provide a number of advantages over traditional 1D modelling. These include:

- Dynamic, as opposed to static, modelling of storage

- Greatly improved assessment of overflow paths
- An improved understanding of floodplain velocities.

To quantify the hydraulic impacts of the proposed road, the base models of the existing streams and floodplains were altered to include the scheme design of the Transmission Gully highway. Both the hydraulic and hydrological models were updated to reflect the changes to factors such as topography and land use directly associated with the new highway. Both the pre and post-construction models were run using the 10% AEP rainfall event (Q10) and the 1% AEP rainfall event including the predicted mid range impacts of climate change ($Q100^{cc}$).

Further details for the hydraulic modelling methodology are contained in **Appendix 14.E – Hydraulic Assessment Methodology**.

4.5.2. Urban Stormwater Network Modelling

Localised hydraulic models of the Waitangirua and Linden stormwater networks were constructed to understand the existing flood risk and compare to the potential flood risk if additional runoff from the Project is diverted through the existing stormwater network. The models focused on the main drainage path. Stormwater branches feeding into the main network were removed as these were not considered critical to analyse. Subsequently the flood risk analysis is not representative of how the entire network is likely to flood, but only of the network area of interest.

5. Hydrological Modelling and Assessment

Quantification of road runoff along the proposed alignment was required as an input to the design for the proposed road and the assessment of effects. This assessment was then used to size cross-culverts including fish passage, and assess runoff for stormwater treatment devices and sediment control devices. Details on stormwater treatment and sediment control devices are contained within the Technical Report 15: *Assessment of Water Quality Effects* (SKM, 2011).

5.1. Culvert Catchment Characteristics

Catchment characteristics for each of the culvert sub-catchments have been assessed and are contained in **Appendix 14.F – Culvert Catchment Assessment**.

5.2. Provision of Fish Passage

There are 28 culverts along the alignment that have been identified as requiring fish passage. These are identified in the culvert catchment plans in **Appendix 14.G**. In catchments where the stream bed is not being altered (i.e. the stream is being bridged), it is assumed that fish passage is not affected.

Two methods of providing fish passage have been considered at this design stage depending on culvert and catchment characteristics. The most common design solution for culverts requiring fish passage is to bury the culvert pipe to maintain a constant wetted perimeter. In this situation, culverts have been oversized to allow for burial of the inverts up to 300 mm. Where this has not been appropriate due to ecological or topographical factors, an alternative fish passage solution has been used which is discussed in more detail in Technical Report 11: *Ecological Impact Assessment* (Boffa Miskell, 2011).

Culverts and catchments that require fish passage have been clearly identified in **Appendix 14.F.**

5.3. Preliminary Cross-Culvert Design

Cross-culverts have been designed according to the following design requirements (see the design standards in Section 2.4.1):

- Culverts should be capable of conveying the critical duration 10% annual exceedance probability (AEP) rainfall storm event without head rising above the pipe soffit
- The road surface level should be at least 500 mm above design stormwater levels for a 1% AEP storm event.

Modelled peak flows (10% and 1% AEP events) for the 2090 post-construction timeframe have been used to ensure appropriately sizing of culverts over their full life. Modelling results for cross-culvert peakflows have been attached in **Appendix 14.F.**

Preliminary design details of all cross-culverts along the alignment have been summarised in **Appendix 14.F**. Culverts are labelled according to their upstream catchments, as shown on the culvert catchment plans, **Appendix 14.G**. In culverts handling large flow volumes, alternate designs have been considered by the Technical Report 2: *Design Philosophy Statement: Bridges and Retaining Walls*.

Design details provided include culvert pipe diameter, length, slope and flow rates for both the 1% and 10% AEP storm events. The locations of these culverts are shown on the culvert catchment plans in **Appendix 14.G**.

5.4. Temporary Access Roads

5.4.1. Purpose

There is a requirement for existing access roads into the site to be used in the early construction phase in each section of the road. In some cases new tracks will need to be developed for this purpose. Some of the existing access tracks already have culverts over the existing waterways (such as in the Duck Creek catchment) whereas other areas rely on fording streams (such as in Horokiri). For all those areas where the streams are currently forded, and for any new tracks, we have undertaken an assessment for temporary culverts.

Culverts are sized for a 2-year return period (Q2) flood event. The location of the access tracks and temporary culverts are included in the Construction Access Plans (AC 01-21) in Volume 4.

5.4.2. Hydrology

The proposed access road crosses stream channels at 61 places (see drainage layout plans in **Volume 4: Plan Set, DA 01 – 21**). Some of these crossings are over ephemeral streams, whereas others cross more significant stream channels. The hydrology was calculated assuming concrete pipes or box culverts would be required at all 61 crossings.

Peak flood flows in a 2-year return period event were calculated using the rational method as per Transit's *Bridge Manual* (2003).

Appendix 14.F Table 3 details the input parameters, 2-year peak flows and required culvert size. Climate change scenarios were not considered as the timeframe for construction was assumed to be within current rainfall expectations. Proposed landuse in 2021 was considered.

5.4.3. Culvert Sizing

Culverts have been sized to pass the 2-year peak flow. Depending on the peak flow, concrete pipes and box culverts are required. The length of pipe is assumed to be on average 10 m (4 m road, with 3 m pipe either side to allow for road buildup. We do not expect buildup to be much greater than 500 – 600mm in most cases). The temporary access culverts will be constructed in a similar manner to the permanent culverts with respect to fish passage and erosion and sediment control.

Catchments requiring fish passage are shown in Table 3 in **Appendix 14.F**. This is based on a previous assessment of the Project catchments by Boffa Miskell. Most access road stream crossings are at a similar location to the Main Alignment, so the Boffa Miskell work has been directly applied to these catchments.

In streams where fish passages are required, the pipe size has been upgraded by 300 mm. This allows pipes to be countersunk by 300 mm to form a continuous wetted perimeter making the culvert passable to native fish species.

A minimum culvert size of 600 mm has been assumed for all catchments.

5.4.4. Impact Assessment

All culverts will be constructed on grade with the existing stream channel to minimise point source erosion at the outlet. Where culverts are being upgraded along the existing access road, little erosion or sediment runoff is expected. Where new access roads are constructed, sediment control measures will be applied as detailed in Section 5.5. Sediment runoff from construction of the access roads will be below the expected runoff from construction of the main highway; therefore will be within the design capabilities of the sediment control devices.

Flood flows larger than a 2-year return period event may cause some damage to the temporary culverts. By constructing the culverts using strong reinforced concrete pipes (z-class pipe or similar) with a shallow cover, flood flows will be designed to overflow the road reducing significant damage to the culvert. Following a large event, all culverts should be assessed and repaired prior to use, as necessary. Existing overland flowpaths will convey larger flood flows.It is expected that temporary culverts will be in use for up to two years in most locations.

5.4.5. Culvert Inlet Structures

Culvert inlets are susceptible to blockage or partial blockage from waterborne debris. Avoiding blockage of the culvert entrance is critical to maintaining the flow capacity of the structure, which protects the roadway against inundation.

The design philosophy assumed for cross culverts is that;

1) A debris screen is constructed of a series of solid ‘stakes’ driven into the stream bed. The ‘stakes’ are spaced such that debris large enough to block the culvert barrel is stopped at the screen. The screen is placed at an angle of 120 degrees to the direction of flow so that floating debris is driven to one side of the channel by the force of the flow, allowing the other side to remain clear. This is defined as an appropriate angle for the design of river training groynes (Prezedwojski, Blazejeweski & Plilarczyk,1995) and can be assumed to be appropriate to this application, as the debris screen performs a similar river training function. A debris screen directly on the face of the culvert is not preferred due to its greater potential for blockage.
2) Downstream of the debris screens, it is proposed that stilling basins will be created by excavating the existing stream channel. The creation of a basin will slow the velocity of the water in the channel, allowing some of the suspended debris that passes through the screen to ‘drop out’ of the flow before entering the culvert barrel. The stilling basin is designed for ease of construction and to provide smooth hydraulic transitions. Standard precast wingwall structures at the culvert inlet also perform this function, directing flow into the culvert barrel.
3) In those areas where there is some risk of colluvium debris flows, culverts have been designed to provide ease of maintenance by reducing the length and grade and using drop structures (discussed in Section 5.6.2) to take flows down the embankment face.

Both the debris screen and stilling basin should be periodically cleared of accumulated debris as part of the road maintenance programme. An exception to this philosophy is the culvert D18 on the Duck Stream where a culvert has been proposed instead of a bridge to provide flood storage. In this case a secondary intake structure should be installed at a higher level to provide secondary intake into the main culvert in case of blockage.

An example of the debris screen and stilling basin proposed is provided Volume 5 SSEMP's for Horkiri and Te Puka.

5.4.6. Culvert Outlet Structures

The Project will require the construction of multiple culverts to transport stream flows beneath the roadway. The majority of these culverts will have significant slopes and flow rates resulting in outlet velocities in excess of the existing velocity in the downstream channel. Excessive outlet velocities can result in erosion with associated sediment and the potential to undermine culvert structures and road embankments requiring ongoing maintenance.

To protect downstream channels and reduce water velocity immediately downstream of each culvert, erosion control structures will need to be installed. In most cases these will take the form of either a rip rap stilling basin and apron, or a baffle apron. Both of these options can be equally as effective at controlling culvert outlet velocity. Baffle aprons are a highly engineered structure and generally require a smaller footprint than a rip rap stilling basin with apron. A rip rap stilling basin provides a resting place for migrating fish and has a more natural aesthetic. For these reasons a rip rap stilling basin is the preferred option where fish passage is required.

Indicative sizing has been carried out for one rip rap basin and apron (Duck 17). The size of this culvert is representative of many of the culverts along the length of the Project, as is the slope. Sizing will need to be carried out for each of the culvert outlets along the alignment individually, to ensure outlet velocities are reduced below the allowable velocity.

Indicative sizing has been carried out for two culverts for the baffle apron option (Duck 17 and Ration 7). The baffle apron solution for Duck 17 is a relatively small baffle apron whereas the solution given for Ration 7 represents an indication of the largest size this type of structure would need to be to reduce outlet velocities from any culvert along the alignment.

For a number of the culverts being assessed the 'fall' between the intake and the outlet is considered to be too great for the traditional approach of laying culvert pipe on grade. Technically such culverts could be prone to blockage, would be difficult to maintain and costly to maintain and replace. As part of our assessment we considered a number of solutions to this problem including a range of drop structures to remove the energy from flows prior to discharging. Key issues considered in the assessment of options were;

- Energy dissipation
- Maintenance and replacement costs
- Safety in design
- Ease of construction, in particular with the MSE walls that are in many cases associated with the steeper culverts.

The preferred solution for these drop structures is a stepped cascade structure (following Chanson, 2002) that is constructed over the face of the embankment. While fish passage requirements are being met with secondary fish culverts the cascade structures are also likely to provide some improved alternative for fish passage as an added benefit.

6. Hydraulic Modelling and Assessment

Not all watercourses / networks potentially affected by the Project have been the subject of detailed hydraulic modelling, as discussed in Section 3.3. In total, six watercourses / networks were modelled, as follows:

- Pauatahanui Stream
- Horokiri Stream
- Te Puka/Wainui Stream
- Duck Creek
- Linden stormwater network
- Waitangirua stormwater network

A 1D/2D coupled hydraulic model was constructed of the three streams and used as an efficient tool to undertake the assessment of effects and assist with identifying options to avoid, remedy or mitigate potential impacts. In the case of Duck Creek, the focus has been on sizing the crossing of the creek to help ensure that the post construction peak flows, in high rainfall events, are restricted to pre development levels. The two stormwater networks were modelled to ensure they are sufficiently sized to accept any increases in runoff as a result of the Project, and where not, to assess mitigation options. The detailed methodology for the hydraulic assessment and modelling is contained in **Appendix 14.E – Hydraulic Assessment Methodology**.

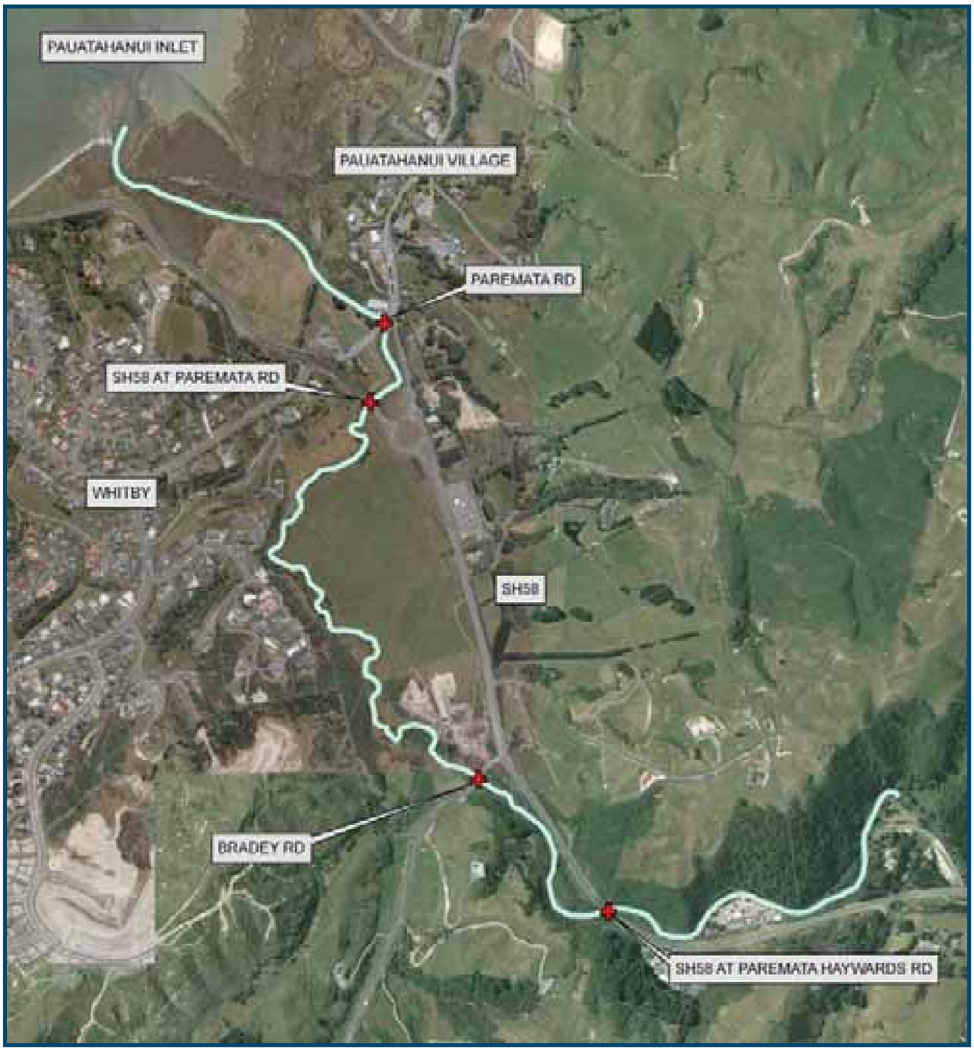
The hydraulic investigation modelled both the pre-construction and post-construction hydraulic impacts of the Project and investigated how these potential impacts can be avoided, remedied, minimised to a shorter duration and/or mitigated. In general the following key impacts were assessed:

- Loss of storage on the floodplain due to earthworks
- Alteration of secondary flowpaths by the road alignment
- Increased runoff associated with the change in land use
- Hydraulic impacts of changes in stream alignment and shape.

The focus of the investigation into each stream / network was tailored to reflect the specific issues of the catchment and the varying impacts of the highway construction at each location.

6.1. Pauatahanui Stream

The reach of stream channel that was modelled is shown in **Figure 14.5**. The hydraulic model starts adjacent to the sawmill on SH58 (Paremata-Haywards Road) and extends 3.4 km downstream to the Pauatahanui Inlet.



▪ **Figure 14.5 The Modelled Reach of the Pauatahanui Stream**

In the upper extent of the model, the channel is located in a narrow steep sided gorge. The stream is constrained as it runs adjacent to SH58 until the topography levels out downstream of the Bradey Road Bridge. Downstream of Bradey Road, the grade of the stream flattens out as it skirts the western perimeter of the floodplain before passing beneath SH58 at Paremata Road and the Paremata Road Bridge adjacent to Pauatahanui Villages, and finally into the Pauatahanui arm of Porirua Harbour.

6.1.1. Pre Construction Situation (Baseline)

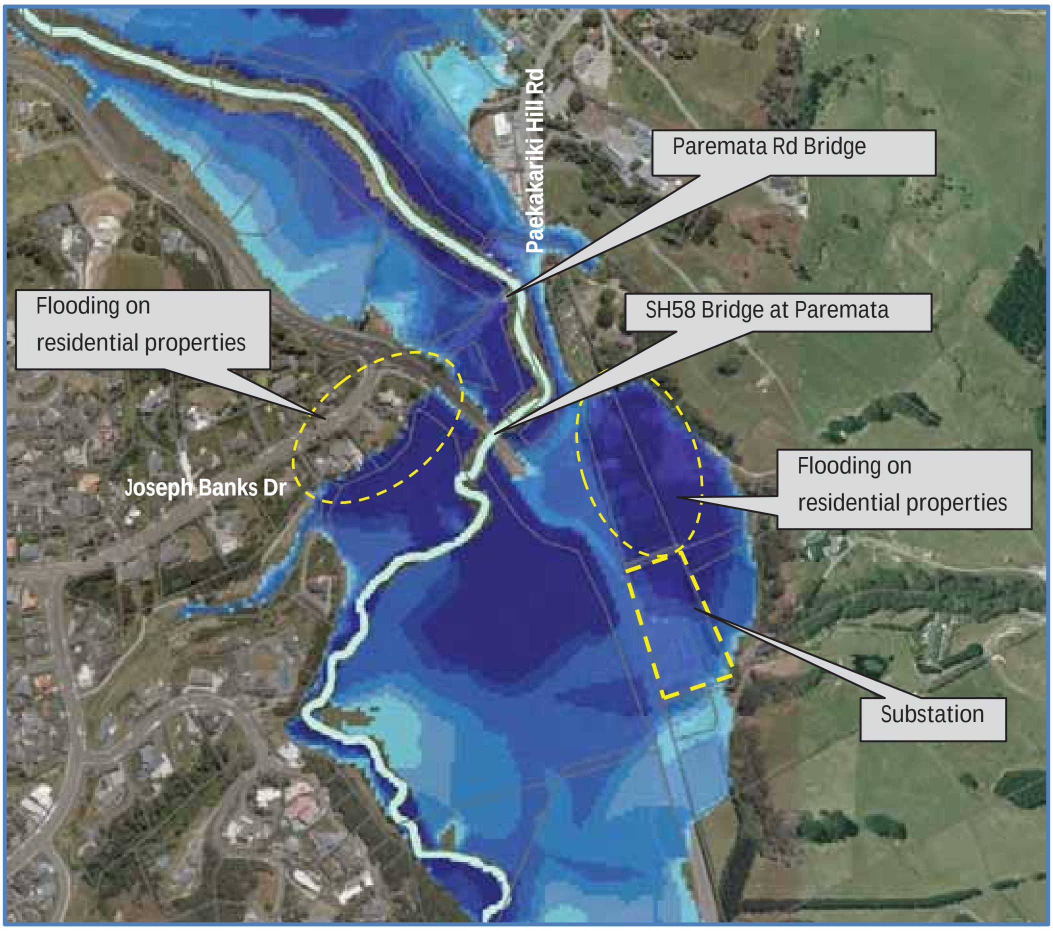
The hydraulic model was used to further understand the Pauatahanui Stream's flooding in the pre-construction catchment in both a 10% (Q10) and 1% AEP rainfall event including the mid-range predicted impacts of climate change ($Q100^{cc}$). The peak inundation depths for these storm events are shown in **Map 5A-5B Pauatahanui, Appendix 14.H**.

Within the gorge reaches upstream of the Bradey Road Bridge, the model results show that the flood flows in a 10% AEP event are largely contained within the steep sided channel. In the 1% AEP event, the stream banks are overtopped in a number of locations, spilling over SH58. The steep valley sides in this area mean that these overflows quickly re-enter the channel.

Between Bradey Road Bridge and Paremata Road Bridge, the topography flattens and widens allowing the stream to spill over a wider floodplain. In the 10% AEP event, the model suggests that the floodwaters cover much of the floodplain, but are shallow and generally less than 200mm. The model predicts that this shallow flooding could cross SH58 in places, which could result in deep ponding on the eastern side of the highway which is below the road level in a localised hollow. There are a number of residential properties and a substation in this low lying area.

In the 1% AEP event, the depths of flooding across the floodplain are increased with much of this area inundated by over 500mm. The model predicts extensive and deep flooding across SH58 and in the localised hollow on the far side.

The hydraulic model indicates that the two existing bridges, near the roundabout that joins SH58 and Paekakariki Road, are constraints to high stream flows and contribute to the upstream flooding (**Figure 14.6**). In a 1% AEP flood event, the model predicts these constraints will increase upstream flooding by up to a metre. In the current situation during high rainfall events, the flooding upstream of these bridges is expected to inundate the lower back yards of four residential properties on Joseph Banks Drive (**Figure 14.6**).



- **Figure 14.6 - Existing Constraints on the Lower Pauatahanui Stream as Seen in a 1% AEP Flood**

6.1.2. Transmission Gully Project

The proposed road alignment was inserted into the model and the hydrological inputs updated to reflect the change in land use associated with the highway construction. The model was rerun for the 10% and 1% AEP rainfall events and compared with the pre-construction scenarios to identify potential hydraulic impacts of the development. The model was then used to evaluate options to avoid, temporally limit and remedy, or mitigate and the impacts.

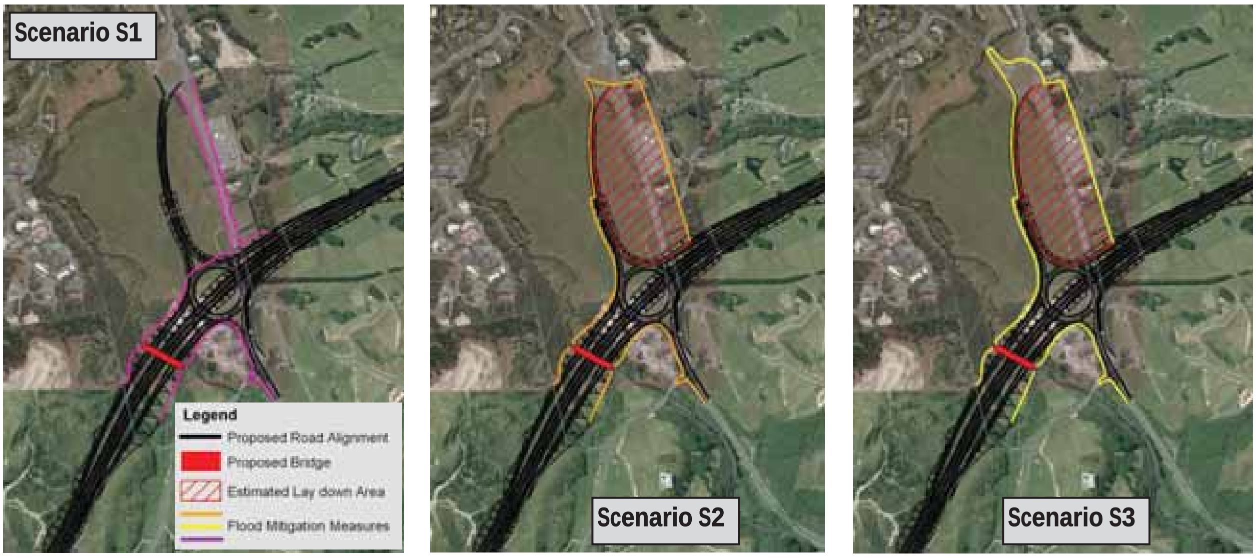
6.1.3. Results Comparison and Options Assessment

The initial model results identified that there are three key areas of effects that would need to be closely assessed:

- The flooding effects associated with the loss of storage on the Pauatahanui floodplain
- The flooding impacts upstream of the new crossing
- The alteration of the stream channel in the vicinity of the new crossing.

6.1.4. Loss of Storage on the Floodplain

The earthworks required to construct the highway and the associated temporary works, such as laydown or construction areas, could result in a loss of available storage on the floodplain. A number of scenarios were tested in the model to identify the impacts of the loss of storage (see **Figure 14.7)**. In all cases the new highway was assumed to be protected to avoid inundation in a 1% AEP event. This was achieved through either elevated earthworks platforms or flood protection measures such as floodwalls or bunds.



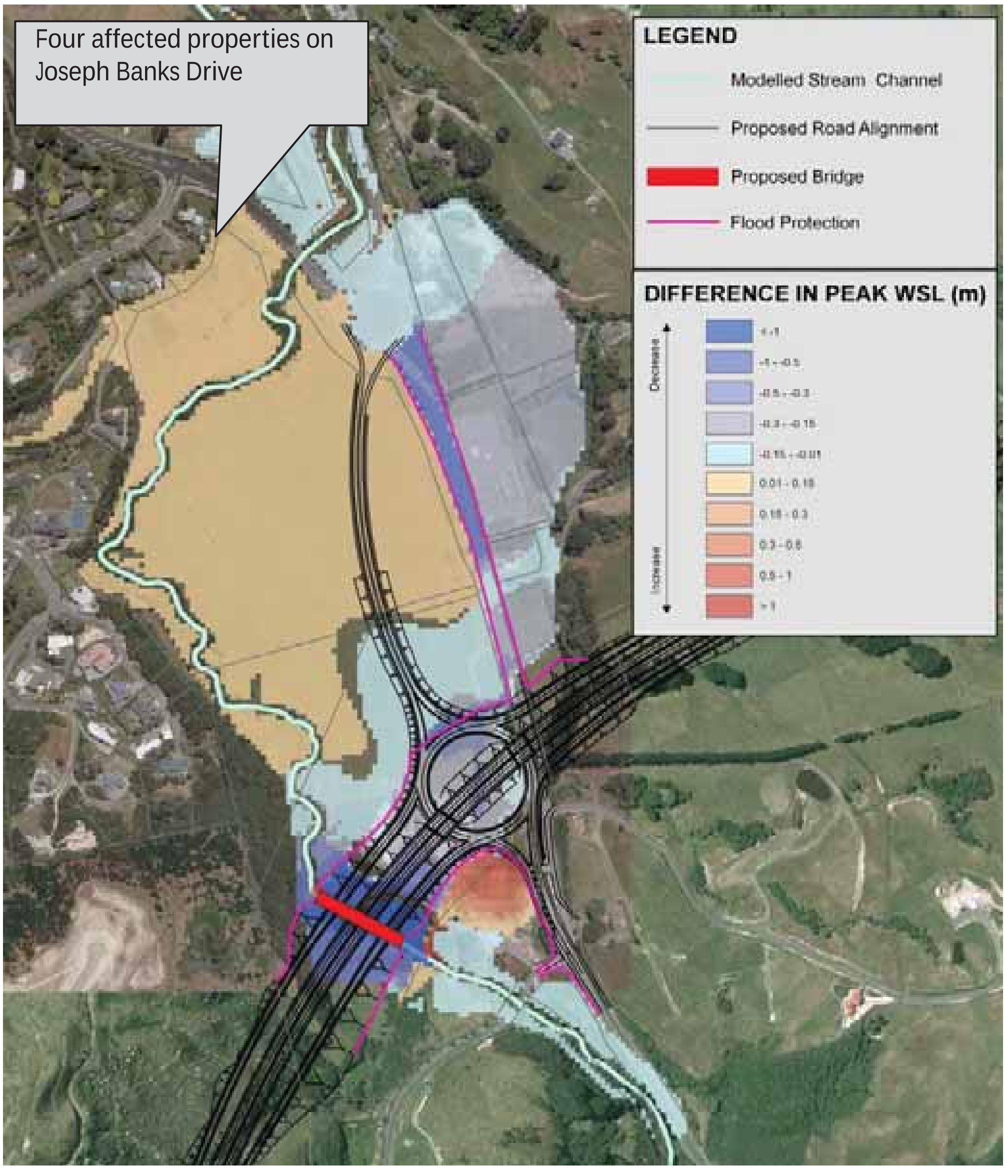
- **Figure 14.7 - Scenarios to Quantify the Impacts of the Loss of Storage on the Floodplain**

Scenario One (S1)

This scenario tested the impacts of the new highway construction, assuming no laydown area and minimum of earthworks for the link road with SH58.

The model identified that the impacts of the loss of storage in the first scenario resulted in approximately a 100mm increase in flood levels surrounding the new highway (**Figure 14.8**). In some places, the model predicts a slight reduction in flood levels due to the protection measures of the new highway. Just upstream of the new bridge, the increase in flood depths was more severe; however, these were localised and relate mostly to the new bridge design rather than loss of storage.

The private properties that could be adversely affected in this scenario are the four residential properties on Joseph Banks Road (Pt Lot 2055 DP 74735, Lot 2056 DP 74735, Lot 2057 DP 74735 and Lot 2726 DP 85792). While the model shows that the four dwellings on these properties are not at risk, there is an increase in flooding in a 1% AEP event of 100mm in the low lying areas of their back yards and over the driveway of Lot 2726 DP 85792.



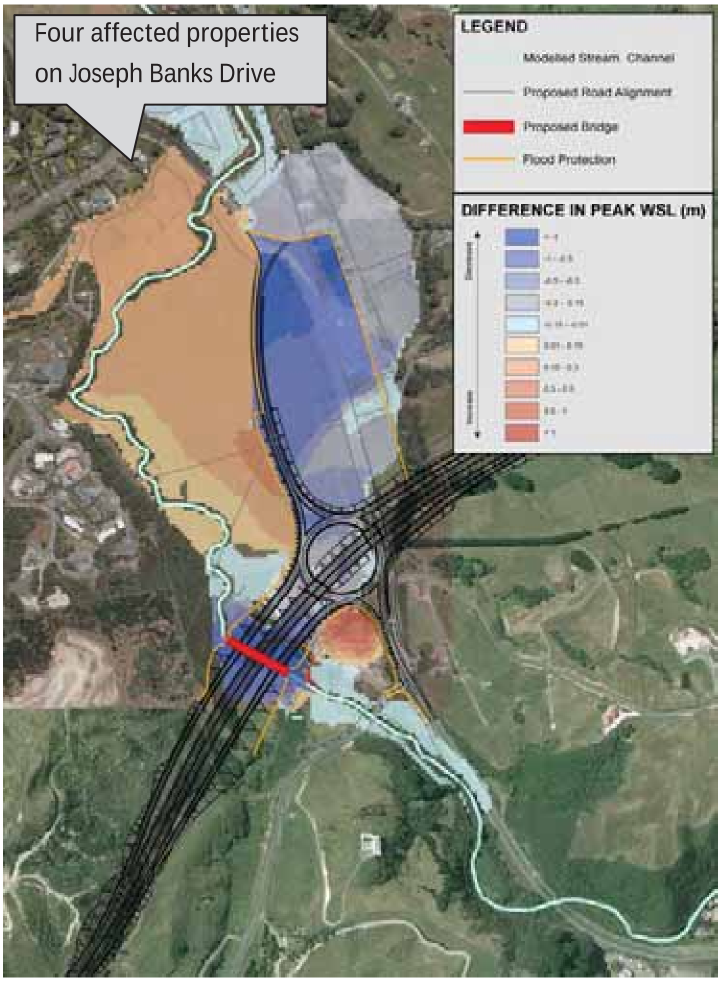
- **Figure 14.8 - Comparison in the 1% AEP Flood Event of the Peak Water Levels in Scenario S1 with the Pre-Construction Scenario to Quantify the Impacts of Loss of Storage on the Floodplain**

Scenario Two (S2)

The advice received on constructability of the highway from Macdonald International indicated that the Pauatahanui floodplain location will be one of the most efficient locations for a laydown and staging area for the construction of both the interchange and the surrounding sections of highway. Scenario S2 models the preferred option from a constructability perspective.

Figure 14.9 shows the comparison between the top water levels in a 1% AEP flood in the existing situation and in the post-construction scenario S2. The new highway and laydown area reduce the available storage on the floodplain, resulting in increases in flooding levels between the interchange and the SH58 roundabout. The model suggests approximately a 200mm increase in flood levels in this area. As previously identified, the private properties adversely affected by this increase are the four adjacent properties on Joseph Banks Road (Pt Lot 2055 DP 74735, Lot 2056 DP 74735, Lot 2057 DP 74735 and Lot 2726 DP 85792).

It is also important to note that the model predicts that the highway construction in this scenario provides a slight reduction in flood risk to properties to the east of SH58.

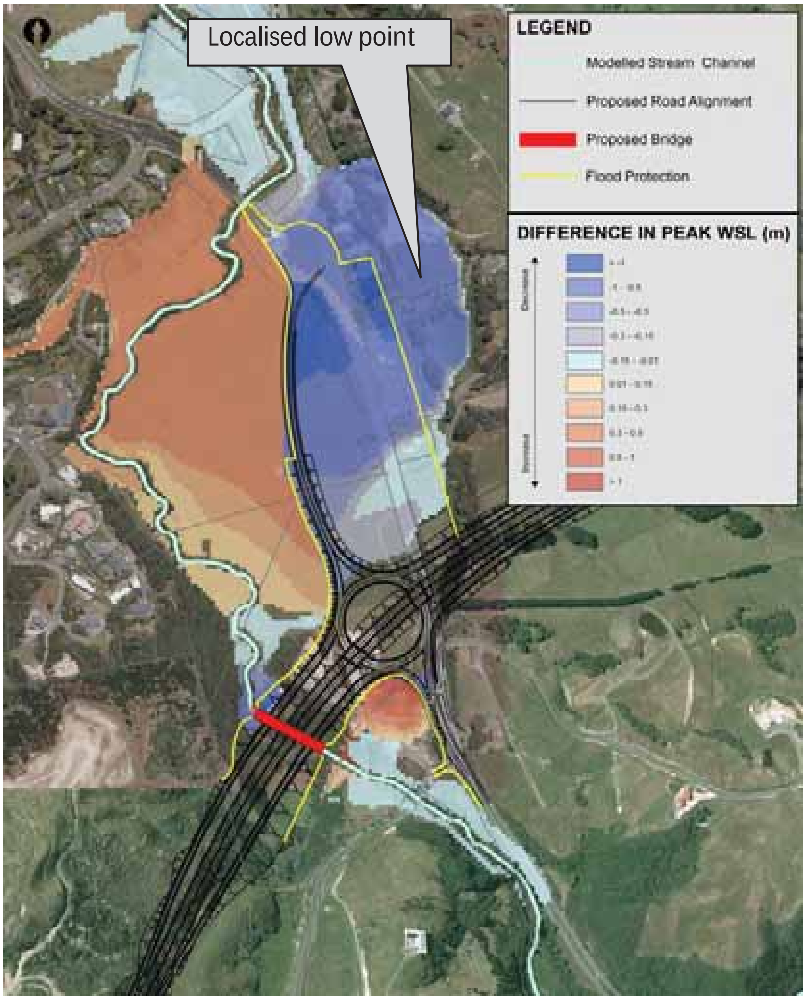


- **Figure 14.9 - Comparison in the 1% AEP Flood Event of the Peak Water Levels in Scenario S2 with the Pre-Construction Scenario to Quantify the Impacts of Loss of Storage on the Floodplain**

Scenario Three (S3)

The model was used to investigate the opportunity to enhance the flood protection to the properties east of SH58 as part of the Transmission Gully project. The constructability preferred option (S2) was modified to protect SH58 from overtopping in a 100-year event. This resulted in all flows in the Pauatahanui Stream passing under the SH58 Bridge at Paremata Road. The results of blocking the existing secondary flowpath over the SH58 roundabout were quantified in the model by comparing the top water levels in the pre and post-construction scenario (see **Figure 14.10**).

The impacts of this scenario, as quantified by the model, are that there was approximately a 400mm increase in flood levels up stream of the SH58 Bridge at Paremata Road. While no dwellings are predicted to be additionally flooded in this scenario, there is an increase to the flood risk on the four adjacent properties on Joseph Banks Road. The model also shows that there is reduced flooding to the east of SH58. In the localised low point, the flooding in a 1% AEP is predicted to be reduced by up to 800mm, providing a much higher level of protection than in the pre-construction situation.



- **Figure 14.10 - Comparison in the 1% AEP Flood Event of the Peak Water Levels in Scenario S3 with the Pre Construction Scenario to Quantify the Impacts of Loss of Storage on the Floodplain**

Loss of Storage on the Floodplain Recommendations

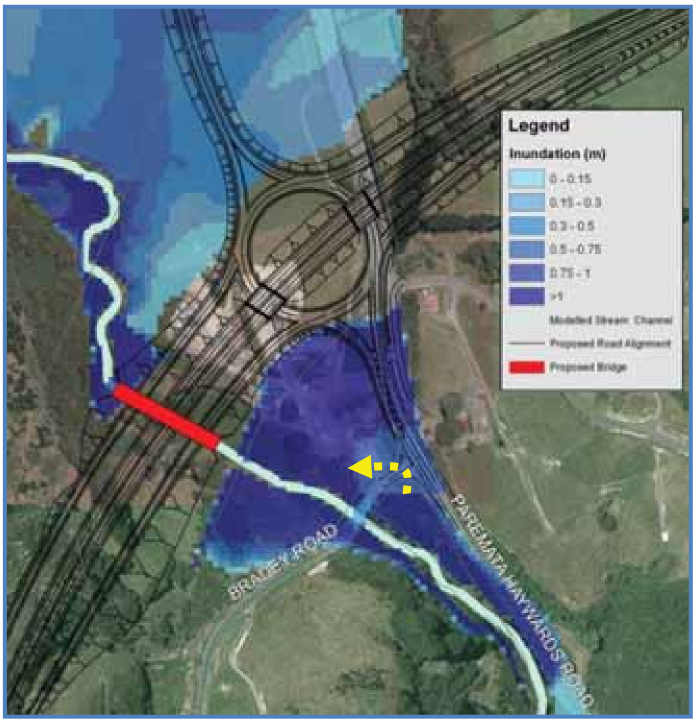
The hydraulic model identified the impacts of the loss of storage on the floodplain by the construction of the Transmission Gully highway and the associated laydown area. Filling on the floodplain can restrict flood flows or reduce the available flood storage resulting in increased flood levels. As the increased flood risk to the low lying areas on the four adjacent private properties on Joseph Banks Drive is only 100-200mm, and the dwellings are not at risk of flooding in a 1% AEP flood, there are a range of options to address this impact. It is possible to mitigate the adverse impacts on these properties by localised protection, such as bunds or floodwalls on the property boundaries. Testing of a bund protection for these properties in the model indicates that this will have no detectable adverse impact on flooding in the area. However while these measures could reduce the adverse flooding impacts, they may not be considered as enhancements by the residents, or necessary.

6.1.5. New Stream Crossing

In the initial design it was proposed to culvert the Pautahanui Stream where it passes under the Transmission Gully highway. The model results quickly identified that the catchment has potential for high flows that would result in a culvert causing unacceptable flooding effects and bed scour. The Pautahanui Stream is also considered to have high ecological value and therefore, limiting the impact on the stream bed is also desirable, providing further reasons for avoiding culverts.

The hydraulic model was used to test a range of bridge options to identify an appropriate waterway crossing to avoid or limit adverse effects. In addition to economical and practical considerations, the key criteria in sizing the crossing were:

- Minimise the upstream flooding effects
- Maintain the existing stream channel shape under the bridge
- Protect the new infrastructure from flooding in a 100-year event.



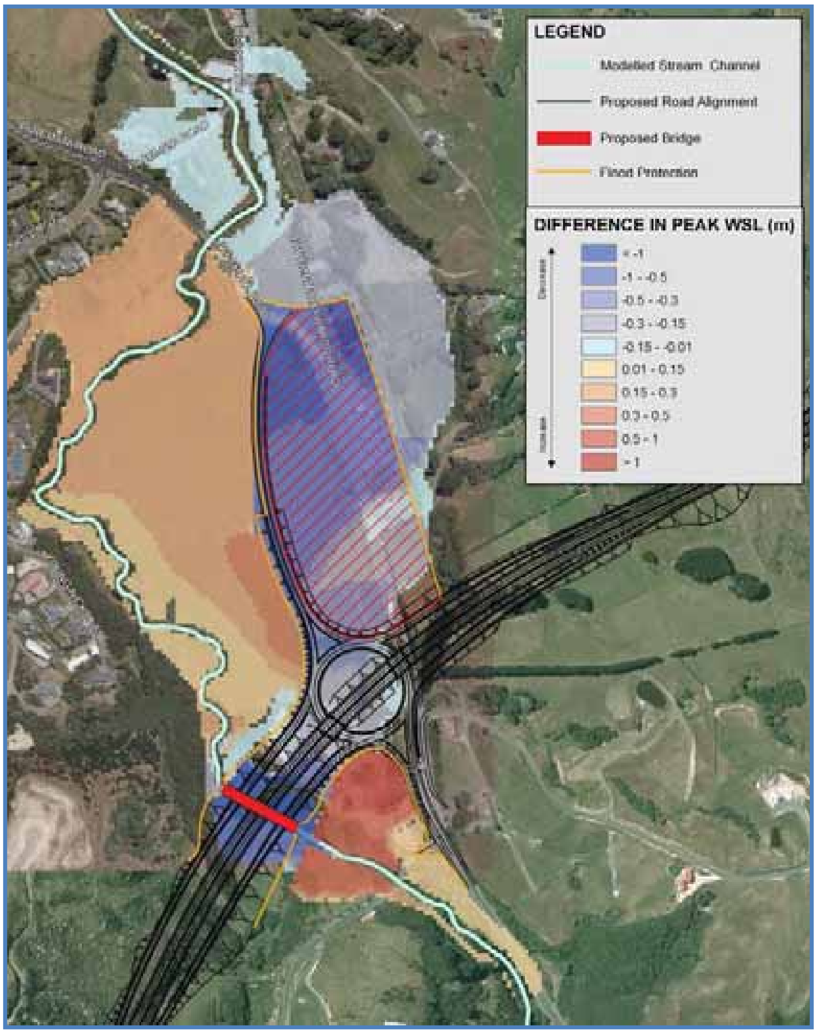
Early options testing with a 20m wide bridge found that this was still a constraint and resulted in increases in upstream flood levels that would over top the Bradey Road Bridge with deep and dangerous flows in a 1% AEP event. Furthermore, the increased water levels would increase the flooding risk to SH58 with potentially deep flooding in excess of a metre. This can be seen in **Figure 14.11**.

- **Figure 14.11 - Potential Increased Risk of Overtopping of Bradey Road as a Result of the Highway Construction, in a 1% AEP Flood Event**

These effects associated with the new highway were considered to be unacceptable from the perspective of road user safety. One option to address this issue was to increase the capacity of the new bridge by excavating a wider stream channel under the bridge rather than maintaining the existing stream shape. It was considered that this change would result in a permanent change to the stream environment and was discounted out of consideration of the ecological significance of the stream.

With the constraints of topography and the maximum allowable road gradients, the bridge was resized to a single span, 28m wide structure. Bridges wider than this are likely to require a double span, which could require piers in the stream bed. Also, additional fill would be required on the floodplain to elevate the on and off-ramps to the highway, reducing further the available storage on the floodplain. Both of these impacts are undesirable.

The modelling revealed that even with a 28m wide bridge, there were still adverse upstream effects. **Figure 14.12** shows that there is approximately an 800mm increase in peak water surface levels upstream of the new bridge. This will still require localised protection of the existing infrastructure to avoid dangerous flooding depths on the roads.



- **Figure 14.12 - Comparison in the 1% AEP Flood Event of the Peak Water Levels in the Post and Pre-Construction Situations with the Inclusion of a New 28m Span Bridge Crossing the Pauatahanui Stream**

To reduce the depth of upstream flooding the decision was made, in consultation with the Transmission Gully Phase 2 Lead Roading and Structural Engineers, to allow the lower level of the interchange to be used as a secondary flowpath in extreme events. While the new 28m span Pauatahanui Stream Bridge will be able to convey all frequent flooding events including the 10-year flood, in larger events the lower level of the interchange (SH58) could be used to convey the secondary overflows reducing the upstream flooding. We have modelled an interchange constructed at a level between 7.0 and 7.5m above MSL and found that the peak water depths over the interchange are predicted to be up to 500mm in a 100-year event, including the predicted impacts of climate change.

Based on this scenario, and using the 1.2m freeboard allowance, as recommended by the Transit *Bridge Manual* for streams with the potential for blockages, the minimum level of the underside of the bridge deck would be at RL8.7m.

The model results for this preferred option in the 10 and 100-year flood events are shown in **Map 6A-6B Pauatahanui Flood Risk, Appendix 14.H**. The comparison of peak flooding levels with and without the proposed highway is shown in **Map 7 – Comparison of peak water levels in Pauatahanui.**

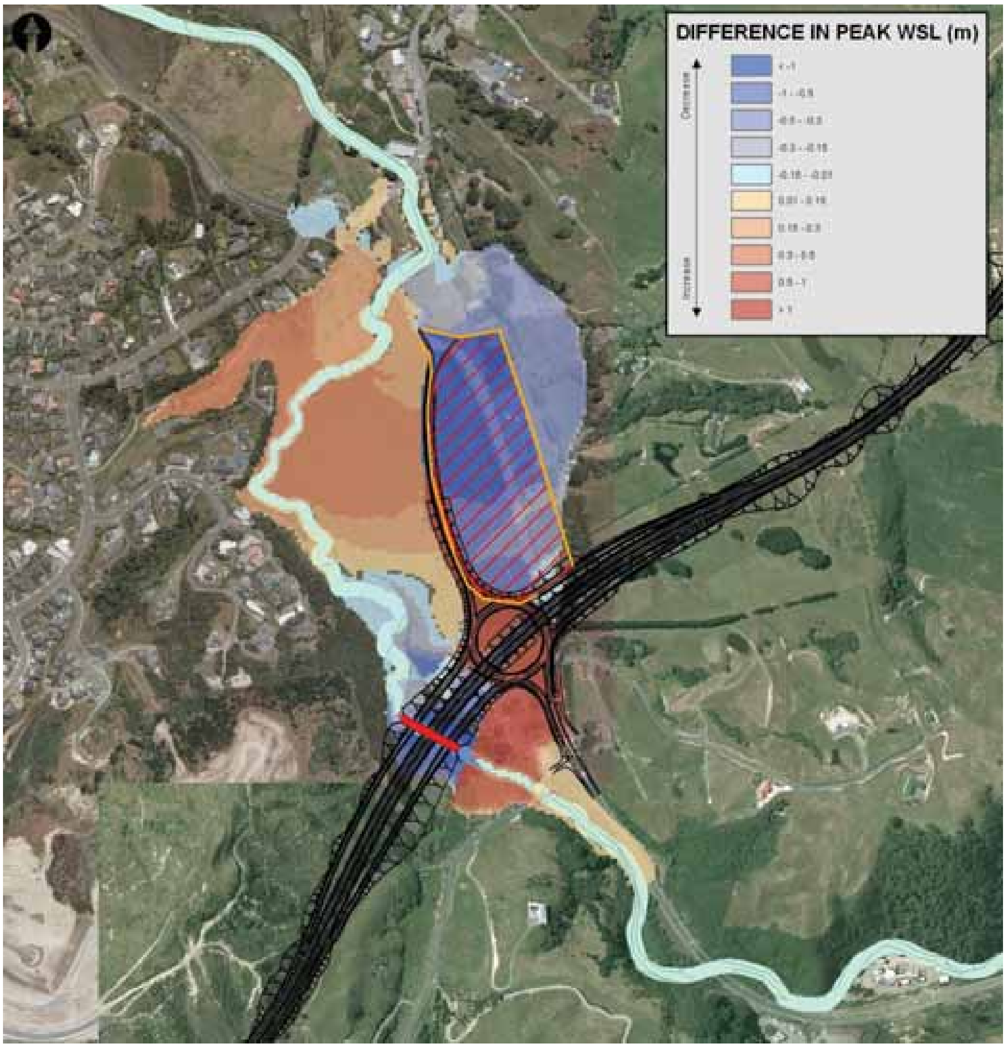
While there is still an increase in flood levels upstream of the new highway crossing, these effects are observed only in extreme flooding and are localised around the bridge inlet. The top of the deck of the Bradey Road Bridge is 8.1m above MSL. With the construction of the preferred option the model predicts that the Bradey Road Bridge will be just overtopped and inundated to approximately 100mm. This is not considered to be a significant increase in risk to road users.

6.1.6. Oversized Events

To further investigate the impacts of the road and the hydraulic model's sensitivity to the hydrological inputs the model was rerun with a greater than design hydrological event. The inflow hydrographs were created by increasing the 1% AEP flood event with the predicted midrange impacts of climate change (100^{cc}) by half again. These inputs were run in the pre construction scenario and in the model of the preferred option which includes the 28m wide bridge and the secondary overflow paths.

The results from the oversized event in the pre construction situation were compared to those of the 100^{cc} year storm event. This analysis revealed that the predicted flooding extents in the oversized event did not increase greatly from the 1% AEP storm event, however there were significant increases in the peak water surface levels. Water surface levels upstream of Bradey Road were predicted to increase between 0.5m and 1m as were water surface levels in the pasture land upstream of the SH58 bridge at Paremata Rd. The lower catchment in the vicinity of Pauatahanui Village was predicted to experience increase in peak water surface levels of between 150mm and 300mm. This comparison provides further confidence in the appropriateness of the 1.2m freeboard recommended for the bridge construction.

The oversized event hydrology was also used to test the impacts on flood risk, in the post construction situation, during a flood event larger than that used for the design of the infrastructure. **Figure 14.13** shows a comparison of peak water surface levels in the oversized storm event pre and post road construction. The model results indicate that there will be localised increase around the proposed interchange. However even in the oversized event the dwellings on Joseph Banks Drive are predicted to be flood free and there is almost no predicted increased flood risk to Pauatahanui Village. It should be noted that in this extreme event there is considerable depth of flooding (1.2m) over the new roundabout, through the designated secondary flow paths. This depth of flooding would be a risk to road users.

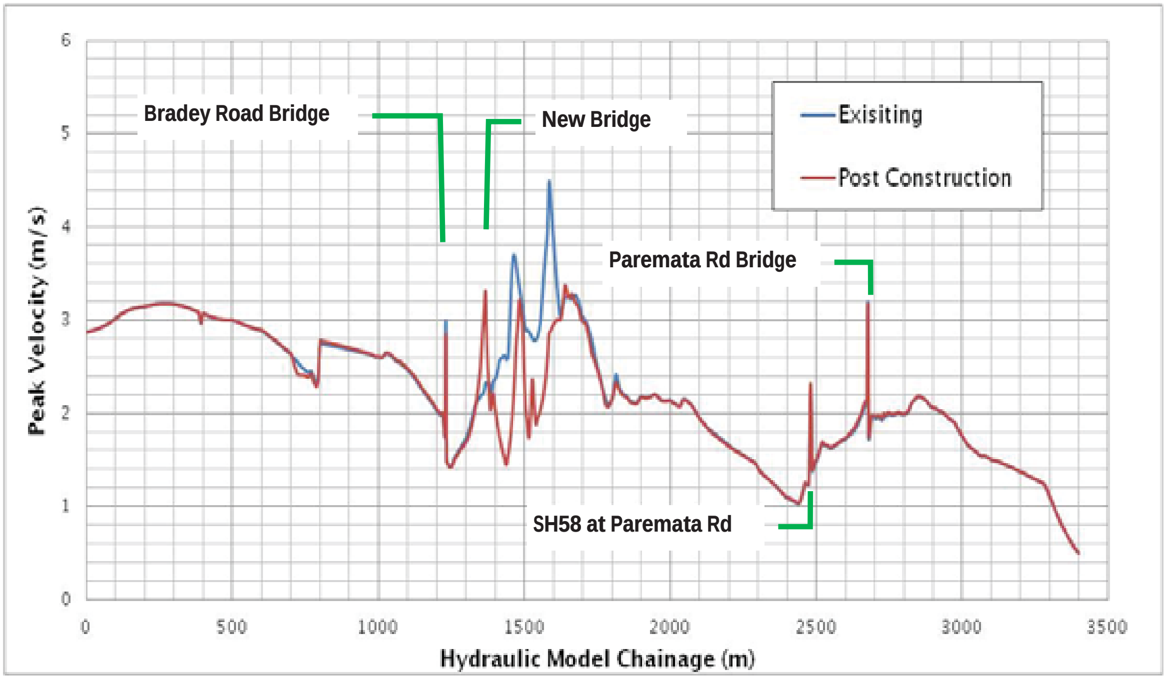


- **Figure 14.13 - Comparison of Peak Water Surface Levels in an Oversized Event Pre and Post Road Construction**

6.1.7. Pauatahanui Stream Velocity Effects

The hydraulic model was used to investigate the impacts of the new highway on the Pauatahanui Stream channel velocities. New structures or constraints in the floodplain can lead to increased velocities, resulting in scour or changing the stream environment. Furthermore, the new highway will require a 100m section of the stream to be realigned to allow for the construction of the new bridge.

A comparison of the pre-construction peak stream velocities was made with the predicted velocities for the preferred new highway option including a laydown area, a 28m span bridge and secondary overflow paths. The comparison for the 10-year flood event is shown in **Figure 14.14**.

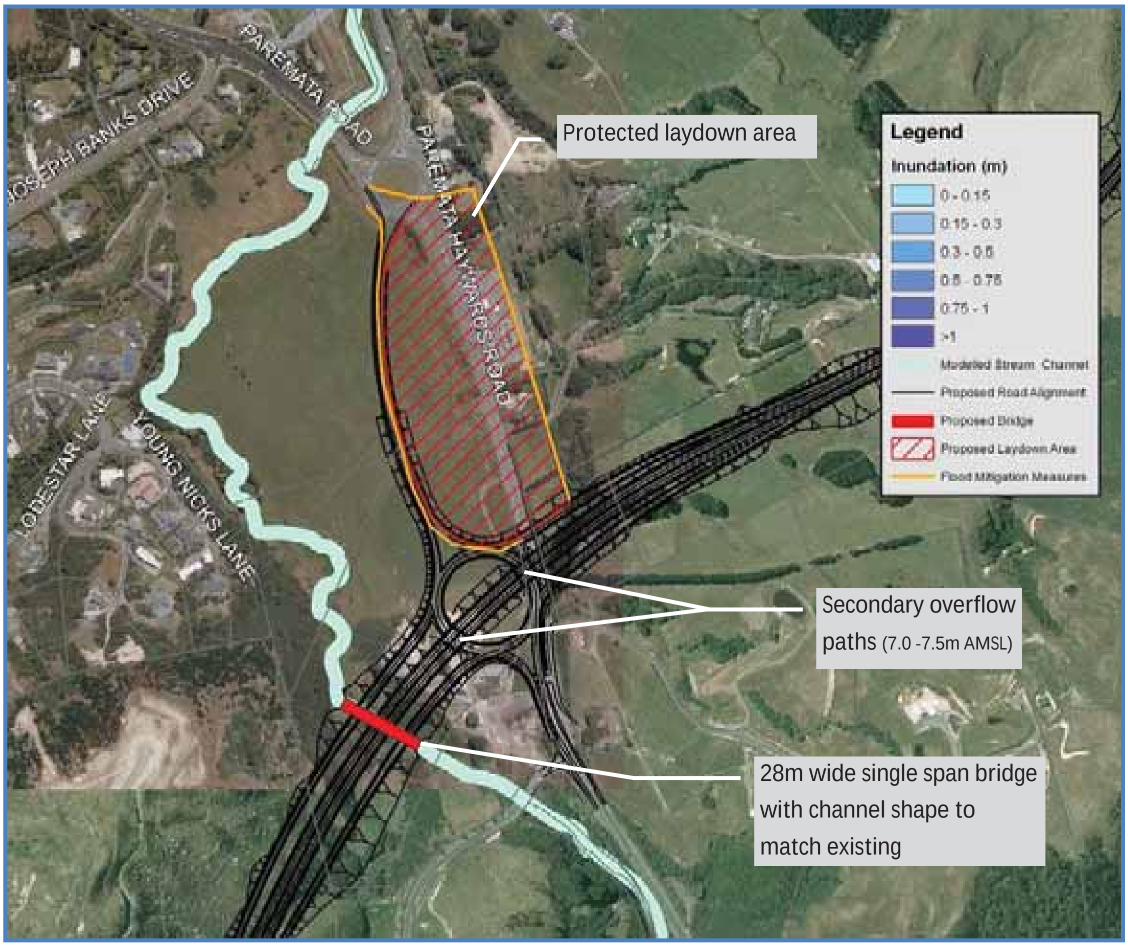


- **Figure 14.14 - Comparison of Peak Velocities in the Pre and Post-Construction Scenarios in a 10% AEP Flood Event in the Pauatahanui Stream**

The results show that the proposed development results in localised changes in the velocity profile. At the intake of the new bridge, there is approximately a 1m/s increase in velocities. However, the reshaping of the channel and the inclusion of the secondary overflows result in reduced velocities for approximately 300m downstream of the bridge. Given the geology of the area and the expected bank vegetation cover, the predicted increased velocities are unlikely to cause significant additional scouring of the banks or bed.

6.1.8. Pauatahanui Stream Preferred Option

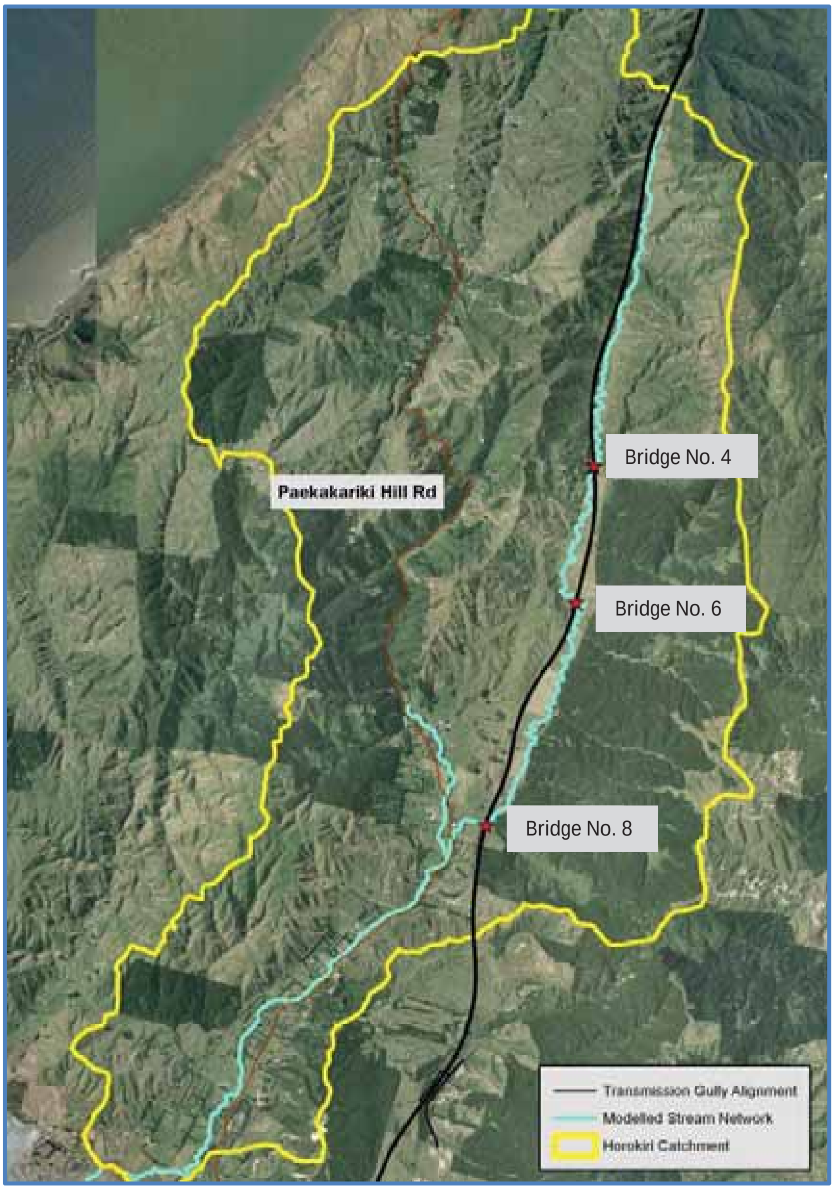
The hydraulic model was used to investigate the impacts of a range of options for the construction of the highway surrounding the Pauatahanui Stream. Based on this analysis, a preferred option was selected (see **Figure 14.15**) as it provides for the practicalities of construction and limits hydraulic effects to localised impacts around the new highway that are likely to be able to be remedied or mitigated.



■ **Figure 14.15 - Components of the Pauatahanui Stream Preferred Option**

6.2. Horokiri Stream

The reach of the Horokiri Stream channel that has been modelled is shown above in **Figure 14.16**. The hydraulic model incorporates c14km of the main stream channel beginning near the Wainui Saddle and extending down to the Pauatahanui Inlet. The hydraulic model also included a short reach of the western tributary of the stream. This tributary drains the western catchment before connecting to the main stream channel south of Paekakariki Hill Road. The majority of the modelled channel network is constrained by the steep sided valley topography typical of the catchment. Only in the reach c1.5km upstream of the Pauatahanui inlet does the topography open out onto a wide floodplain.



▪ **Figure 14.16 - Main Alignment in the Horokiri Catchment**

6.2.1. Pre Construction Situation

The hydraulic model was run using the pre-construction conditions to better understand the existing flood hazards in both the 10% (Q10) and the 1% AEP flood event, including the predicted mid-range impacts of climate change (Q100^{CC}). The modelled peak inundation depths for these flood events are shown on **Map 8A-8F Horokiri Peak Inundation Depths**, **Appendix 14.H**.

The modelling confirmed residents' experience that the Horokiri Stream can carry high sediment loads. This is exacerbated by the erosion-prone underlying geology and the steep and highly modified catchment.

Upstream of Paekakariki Hill Road, overflows from the main stream channel, in both the 10 and 1% AEP flood events, are predicted to be constrained by the steep valley topography. This results in limited flooding extents but localised inundation depths in excess of 1m adjacent to the main stream channel. Due the existing pastoral land use in the upper catchment, the flooding impacts in both 10% and 1% AEP events are predicted to be minimal.

At Paekakariki Hill Road the main channel is constrained by an existing box culvert. This culvert is predicted to have insufficient capacity to convey the 1% AEP stream flows, resulting in the over topping of Paekakariki Hill Road. Overflows from this culvert are also predicted to threaten a building on the true left bank of the stream immediately upstream of the culvert.

In the main tributary branch of the stream draining the north western catchment, some overflows are predicted downstream of the entrance to the BHFFP in the 1%AEP flood event, but these are largely shallow (less than 150mm). Overflows in the tributary branch are not predicted to endanger any residential buildings on Paekakariki Hill Road.

Downstream of the confluence of the main channel and its main tributary, the valley topography begins to open out and flood flows are not as constrained as they are in the upper catchment. Residential properties are also more numerous in the lower catchment, resulting in flooding being predicted to threaten up to 4 buildings in a 10% AEP flood and up to 7 buildings in the 1% AEP flood event, including the allowance for the predicted impacts of climate change. As the stream approaches the Pauatahanui Inlet, overflows are predicted to spread out across the floodplain inundating a wide area upstream of the mouth of the stream. Grays Road is also predicted to overtop.

6.2.2. Results - Transmission Gully Project

The proposed road alignment was inserted into the model and the hydrological inputs updated to reflect the change in land use associated with the new highway. The model was rerun for the 10% and 1% AEP flood events and compared with the pre-construction scenarios to identify potential hydraulic impacts of the development. The model was then used to evaluate options to avoid, temporally limit and remedy, or mitigate the impacts.

The initial model results identified that there are two key areas of effects that would need to be closely assessed:

- Changes in flooding impacts associated with the new bridges on the main channel
- The alteration of the stream channel in the vicinity of the new diversions.

The additional runoff associated with the change in land use was found to increase the peak discharge from the stream by less than 1% and therefore, has almost no observable impact on the flood levels in the model.

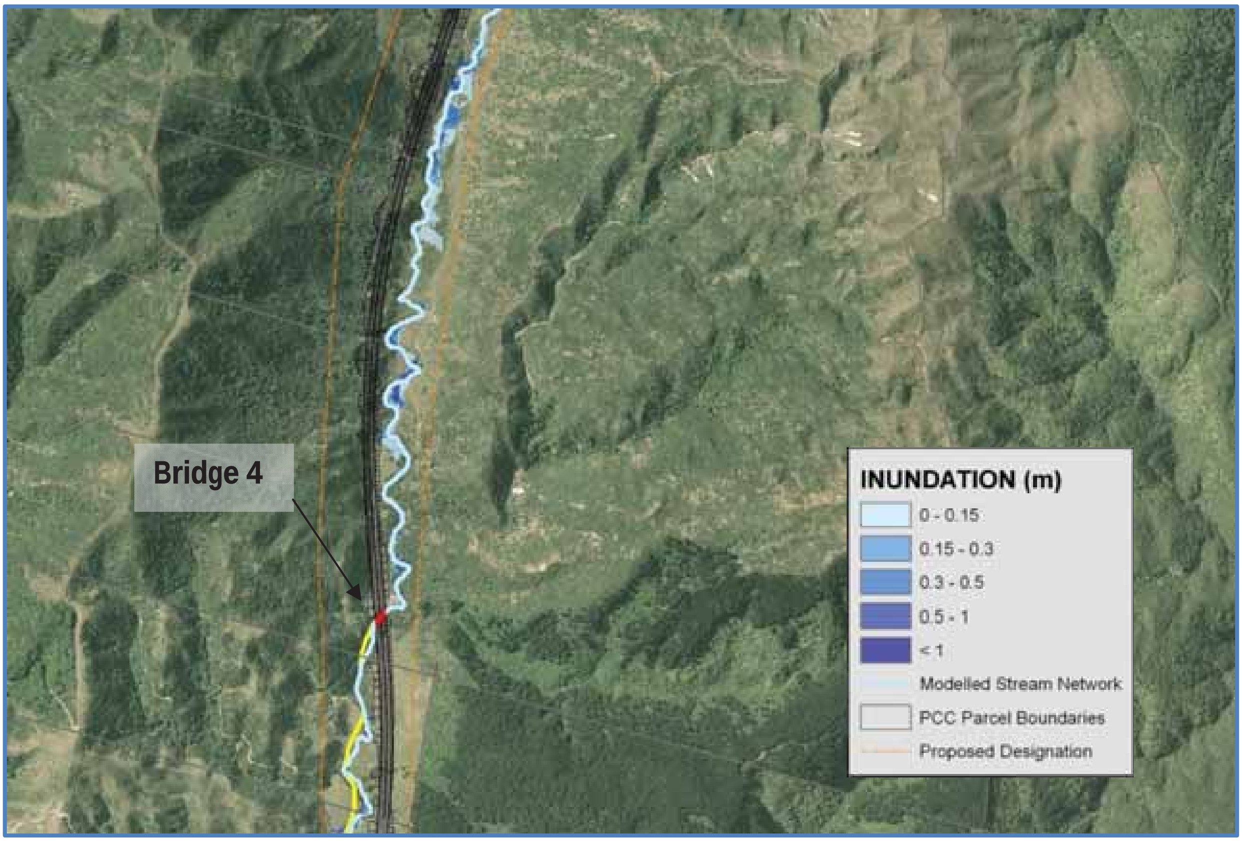
6.2.3. New Stream Crossings

Three bridges are proposed to allow for the crossing of the Main Alignment in the upper reaches of the main Horokiri Stream channel. Each of these structures has the potential to alter stream hydraulics and cause adverse impacts such as increases in flooding levels, additional bed scour, etc. The hydraulic model was used to test a range of bridge dimensions to identify appropriate waterway crossings to avoid or limit adverse effects. In addition to economical and practical considerations the key criteria in sizing the crossings were:

- Minimise the flooding effects
- Maintain the existing stream channel shape under the bridge
- Protect the new infrastructure from flooding in a 1% AEP event.

Bridge No. 4, Scheme Design Chainage 8,450m

Options testing of Bridge No. 4 in the hydraulic model indicated that a minimum 16m wide structure, with a low flow channel matching the existing stream channel dimensions, was sufficient to protect the new highway and minimise the upstream flooding effects. **Figure 14.17** illustrates that the steep topography surrounding Bridge No. 3 results in flooding being largely contained within the stream channel. This means that the effects of the constraint to the stream channel are localised to the channel just upstream of the bridge. Peak water surface levels immediately upstream of the structure were predicted to increase by up to 700mm in the 1% AEP storm with the predicted impacts of climate change. A 600mm freeboard is considered appropriate for sizing this bridge and therefore the minimum level of the underside of the bridge deck will need to be at least 113.6m AMSL.



- **Figure 14.17 - Peak Inundation Levels Surrounding Bridge No. 3 in the Post-Construction Scenario in the 1% AEP Storm Event**

Bridge No. 6, Scheme Design Chainage 9,720m

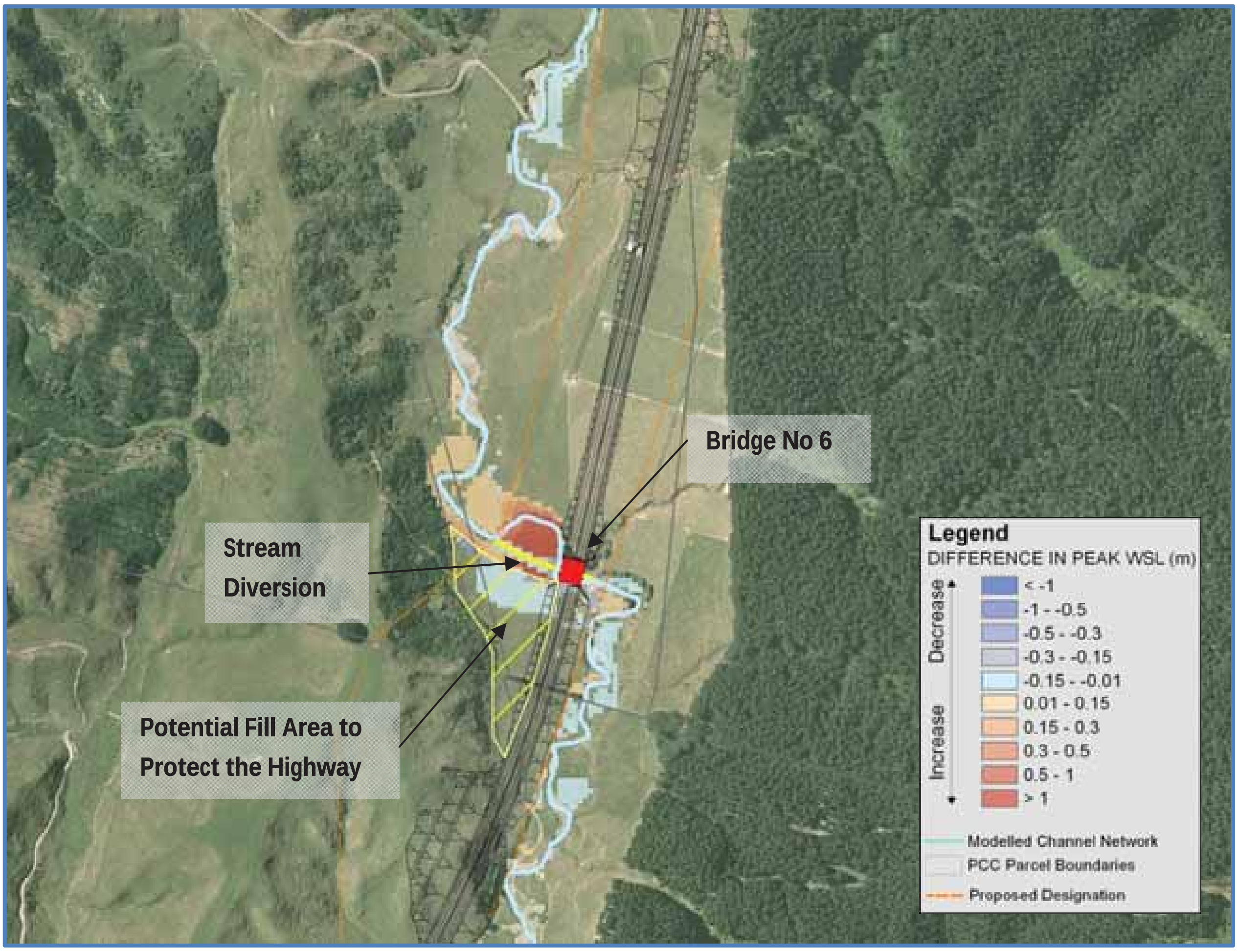
Early options testing indicated that the initially proposed 25m wide structure, with a low flow channel matching the existing stream dimensions, would result in increases in peak water levels upstream of the bridge to an extent that the carriageway would be endangered.

Options to manage the flooding effects included increasing the hydraulic efficiency of flows through the bridge through altering the stream alignment, increasing the bridge capacity through excavating a wider stream channel under the bridge, increasing the dimensions of the bridge or protecting the carriageway through additional earthworks.

Where possible, this project attempts to limit the alteration of the existing stream environment. The excavation of a wider stream channel under Bridge No. 6 would result in a permanent change to the existing stream hydraulics, with potentially much shallower base flows. To avoid these impacts, the options assessment instead focussed on resizing of the bridge, realigning the stream and localised protection of the carriageway. **Figure 14.18** illustrates the outcome of the options assessment.

In consultation with the Transmission Gully Phase 2 Lead Structural Engineer, the bridge width was increased by 3m to a single span of 28m. Testing with the model identified that wider bridges had only limited additional benefit in mitigating the effects of the loss of storage on the floodplain.

100m of the stream channel on the upstream side of the bridge was realigned to allow for a hydraulically smoother approach to the bridge. The existing stream channel shape was maintained through this new alignment. The realignment at this location also helps protect the highway embankment by reducing the velocities adjacent to the fill areas. The hydraulic model also revealed that the flooding upstream of the bridge could pond in the topographic depression formed between the new highway and the western side of the valley. This low point could pond to a depth where the carriageway could be overtopped. This low point could be protected from the stream flooding by stop banks or could be filled as part of the road construction, see **Figure 14.18**.



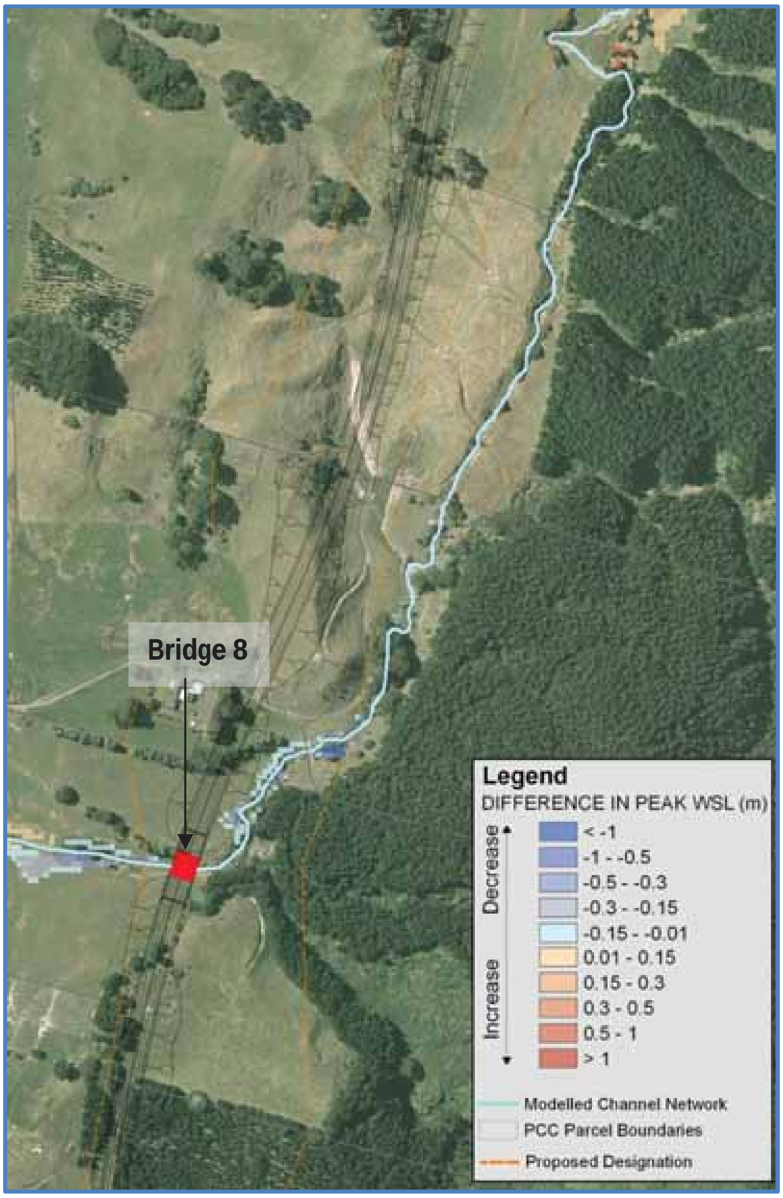
- **Figure 14.18 - Comparison in a 1% AEP Flood Event of the Pre and Post-Construction Scenario Around Bridge No. 5 on the Horokiri Stream**

The hydraulic modelling of the preferred option for Bridge No. 6, as shown in **Figure 14.18**, indicates there are still increases in flooding upstream of the bridge in a 100-year flood event with the predicted impacts of climate change. 50m upstream of the bridge the flooding is predicted to increase by up to a metre. However, this adverse effect quickly reduces until there is no detected increase approximately 300m upstream of the bridge. The recommended stream straightening, filling and bridge sizing described above help protect the new carriageway and maintain the existing hydraulics of the stream during low flows. Based on the modelled top water levels and applying a 600mm freeboard allowance, the minimum level to the underside of the bridge deck would need to be at least 86.4m above MSL.

Bridge No. 8, Scheme Design Chainage 11,750m

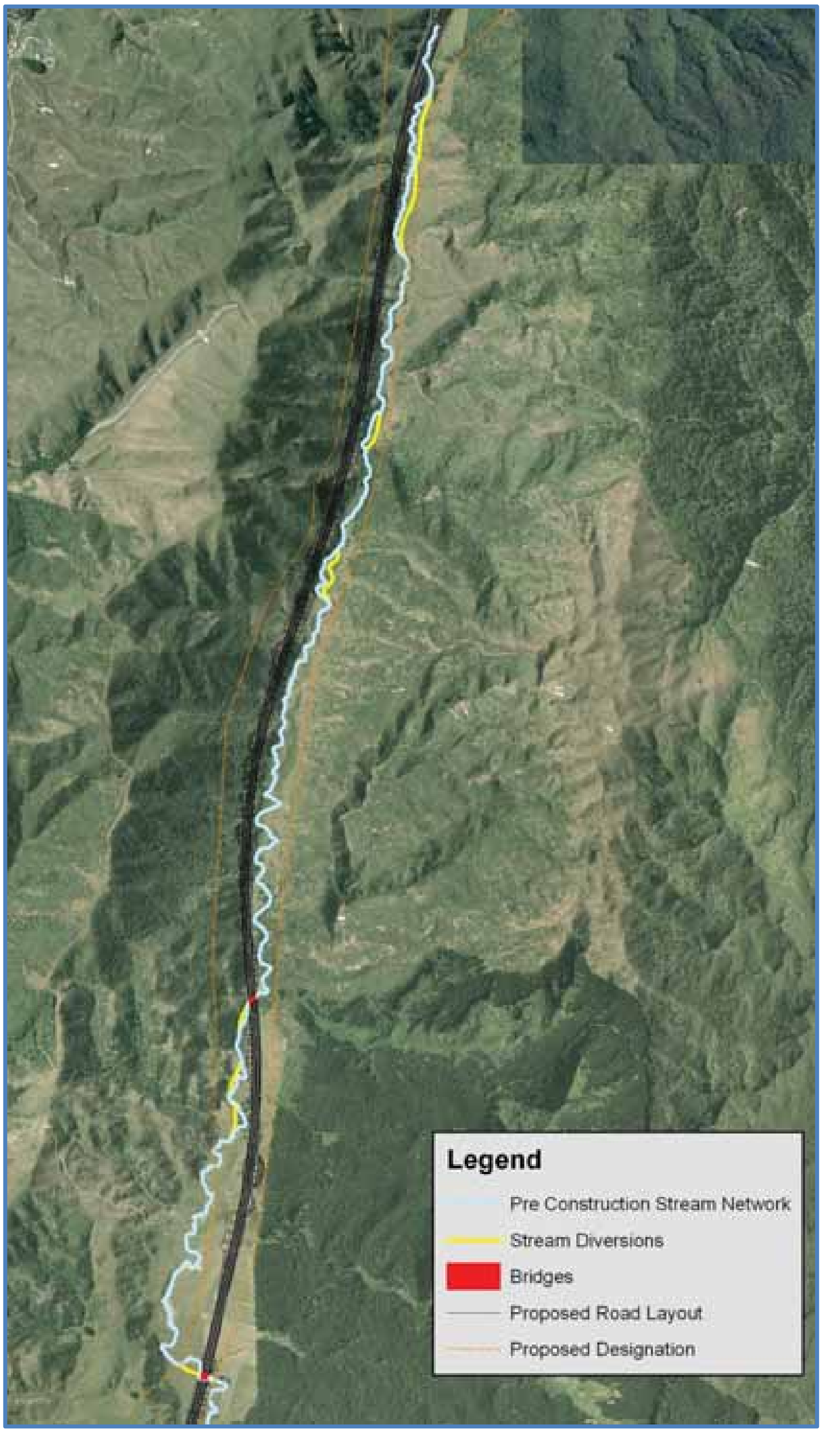
Options testing indicated that a 30m wide structure with a low flow channel matching the existing stream dimensions was sufficient to avoid adverse flooding effects upstream of Bridge No. 8. The change in peak water surface levels upstream of Bridge No. 8 are shown in **Figure 14.19**. Peak water surface levels immediately upstream of the structure were not predicted to adversely affect in either the 10% or 1% AEP events. This is in part due to the restriction of Bridge No. 6 upstream.

The minimum level of the underside of the bridge deck would need to be 45.4m AMSL based on the top water levels and a 600mm freeboard.



- **Figure 14.19 - Impacts of the Construction of Bridge 8 on Peak Water Surface Levels in a 1% AEP Storm Event including the Predicted Impacts of Climate Change**

6.2.4. Stream Diversions and Loss of Storage



▪ **Figure 14.20 - Stream Diversions in the Upper Horokiri Stream**

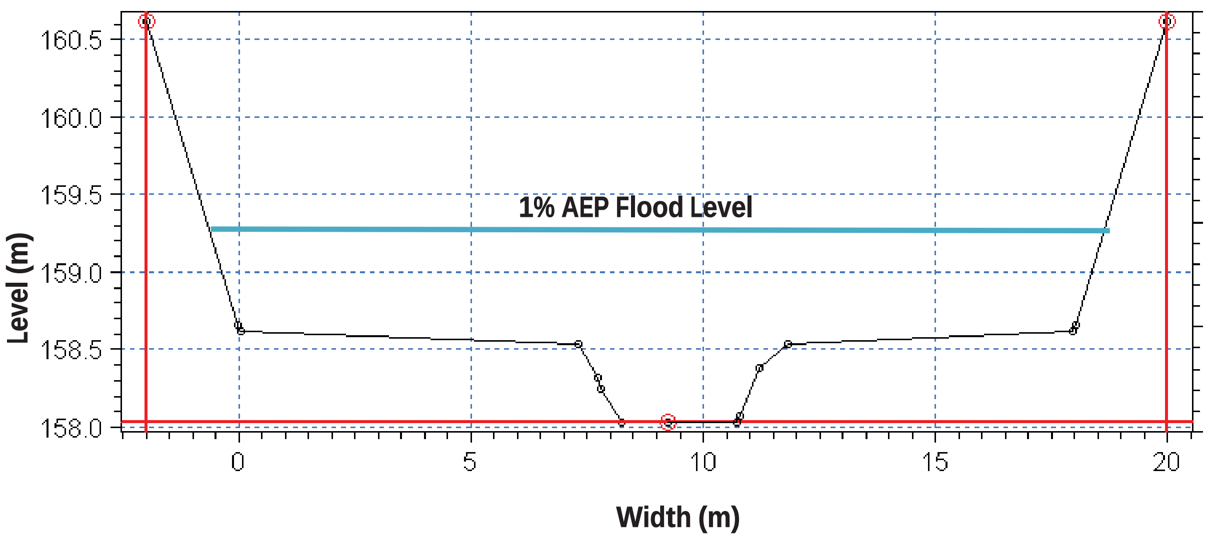
The steep and constrained topography of the upper Horokiri catchment presents few options for the Main Alignment. In a number of locations the earthworks for the highway construction encroach on the Horokiri Stream. In these locations stream diversions are proposed (**Figure 14.20**).

It is anticipated that the diversions would be constructed to minimise the impacts on the stream environment and the channel shape of the new stream would be constructed to provide for the stream ecology. Even so, these diversions could still result in adverse impacts such as:

- Increase in flood levels due to restriction of the channel
- Loss of storage by the straightening of the stream alignment and filling of the floodplain
- Increase stream gradient resulting in elevated velocities
- Reduction in ecological health and/or diversity.

The stream diversions were investigated in the hydraulic model to assist in their design and identify the potential hydraulic effects.

The model was used to size the diversions to avoid significant increases in flooding depths due to restrictions in the channel width. The model results indicated that in general, the diversions should include a low flow channel meandering over a minimum 20m wide flood flow channel. A typical cross section is shown in **Figure 14.21.**

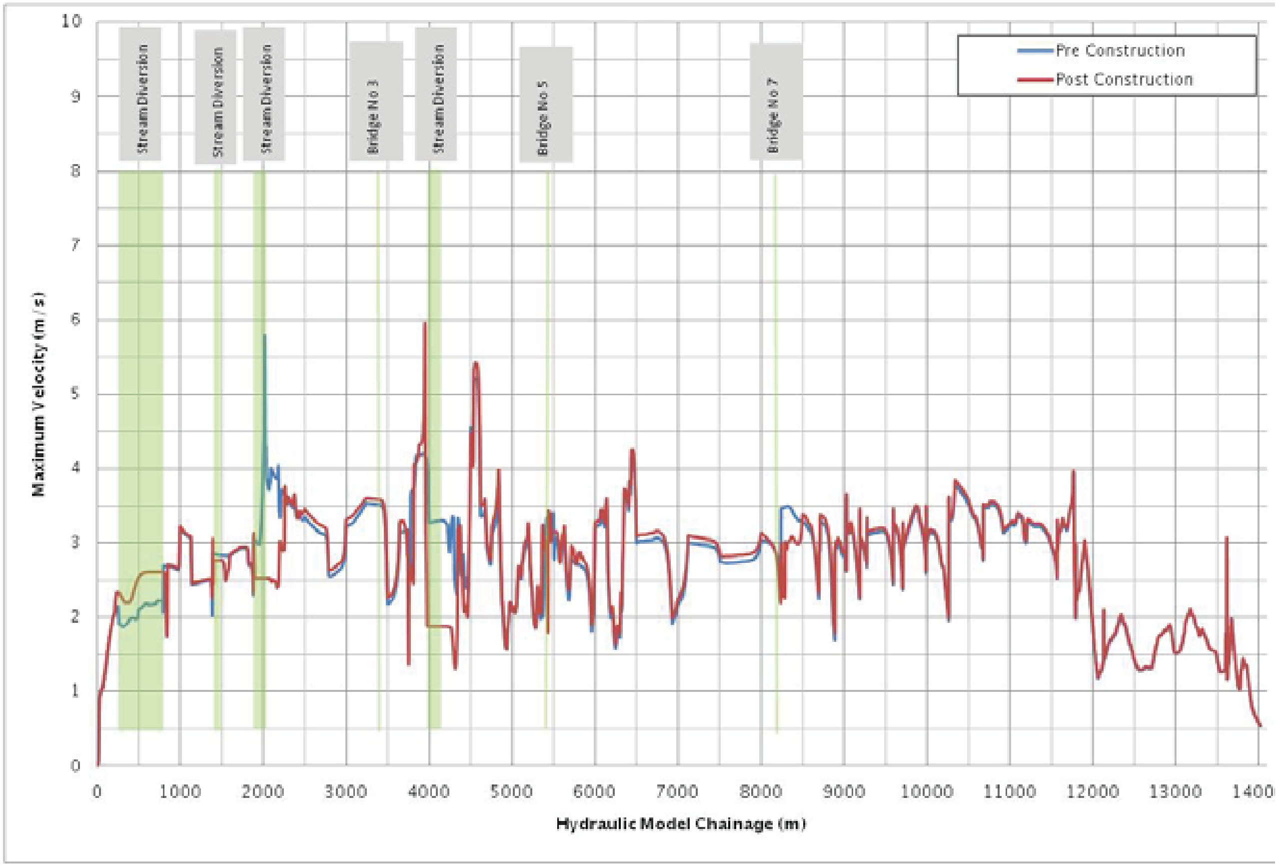


- **Figure 14.21 - Typical Diversion Cross Section**

The modelled roughness of the diversion channel was increased to represent landscaping and vegetation on the banks. Under these conditions, the hydraulic model identified that there was almost no increase (less than 100mm) in flood levels associated with the diversions.

The hydraulic model was also used to investigate the impacts of the Project on the Horokiri Stream channel velocities. New structures, diversions, or the constraining of the floodplain can increase stream velocities resulting in additional scour or changing of the stream environment.

A comparison of the pre and post-construction peak stream velocities was made using the recommendations of diversion design and the bridge widths described in this report (16m, 28m and 30m wide structures at Bridges 3, 5 and 7 respectively). The comparison for the 10% AEP flood event pre and post-construction is shown in **Figure 14.22.**



▪ **Figure 14.22 - 10% AEP Peak Velocities Pre-Construction vs. Post-Construction**

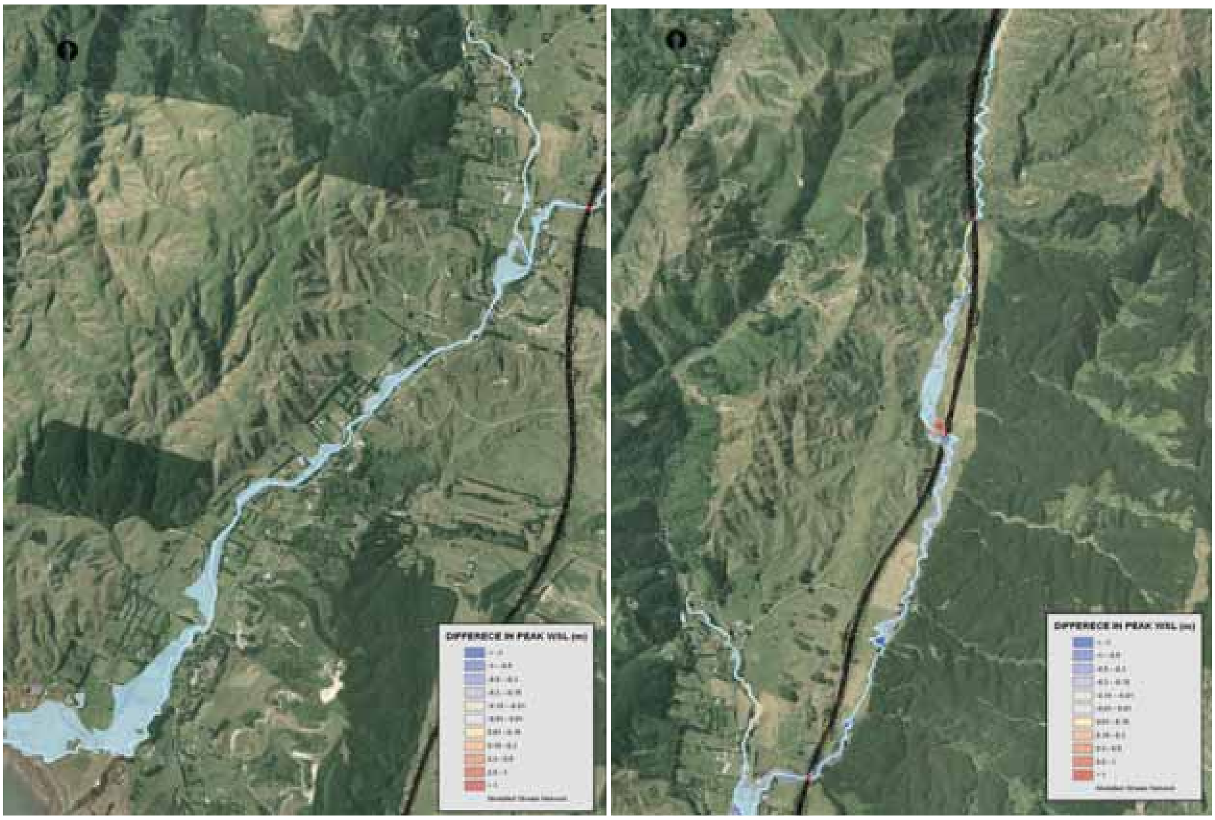
The model results show that in many places, the diversions slightly decrease the peak flood velocities. This confirms the appropriateness of the recommended width of the diversions flood channel, the associated bank planting and the stream diversions design assumptions.

Figure 14.22 also shows that there is very little difference in velocities around the 3 bridges, providing further confidence in the sizing of the bridge widths.

Detailed design of the diversions will need to consider average flow velocities and ensure these allow for current fish passage access to be maintained. The ability to allow for hydraulic jumps (waterfalls) in these sections of the stream will make this relatively straightforward from a hydraulic perspective.

6.2.4.1. Oversized Events

To further understand the risks during events larger than the highway and bridge design event an oversized flood was modelled in the pre and post construction scenarios. A comparison of the results is shown in **Figure 14.20**. Increases in peak water surface levels as a result of the Transmission Gully Project were found to be localised around the new crossings. There was almost no increased risk to the dwellings in the lower catchment. Furthermore even in this extreme event the highway in this catchment did not overtop.



- **Figure 14.23 - Comparison of Peak Water Surface Levels Between an Oversized Event Pre and Post Road Construction**

6.2.5. Horokiri Stream Conclusion

The hydraulic model was used to investigate the impacts of a range of options for the construction of the highway surrounding the Horokiri Stream. Based on the analysis of the model results, a number of recommendations have been documented in this report. These recommendations consider the practicalities of construction and are considered to limit the hydraulic effects to localised impacts around the new highway that are likely to be able to be remedied, mitigated or contained within the designation.

6.3. Te Puka/Wainui

6.3.1. Pre Construction Situation

The hydraulic model was used to further understand flooding from the Te Puka and Wainui Streams in the pre-construction catchment for both a 10% (Q10) and a 1% AEP rainfall event, including the mid-range predicted impacts of climate change ($Q100^{cc}$). The peak inundation depths for these storm events are shown in **Maps 9A-9B Te Puka/Wainui Peak Inundation Depths**, **Appendix 14.H**.

The model indicates that in both the 10% and 1% AEP flood event, the steep sided valley constrains overflows on the Te Puka Stream until immediately upstream of the culvert that passes beneath SH1. This triple box culvert on the Te Puka Stream is predicted to have sufficient capacity to convey the 10% AEP flows but to overtop in the 1% AEP event. The overtopping of this structure is likely to result in shallow inundation depths (up to 150mm) on SH1 and on the pastureland immediately downstream.

The Wainui Stream is also constrained by the twin culverts that pass beneath SH1 and the bridge under the NIMT railway. In both the Q10 and $Q100^{cc}$ storm events, the culverts under the existing highway are predicted to have insufficient capacity to convey the flood flows, resulting in overflows across the State highway. In the $Q100^{cc}$ storm, a portion of these overflows enter the Te Puka Stream immediately upstream of SH1.

Between SH1 and the railway is a storage area for flood waters. Ponding in this location is predicted to be deep; in excess of 1m even in a 10% AEP flood. In the 1% AEP flooding, the model predicts that the railway will be overtopped, flooding the pastureland to the east and endangering the residential properties at the northern end of Tilley Road.

From the confluence of the Wainui and Te Puka Streams to the stream mouth, overflows are predicted to occur over the true right-hand bank of the stream. In the 10% AEP storm, the overflows primarily impact on the camping ground at the northern end of Paekakariki but, with the increased flow of the 1% AEP storm, including the predicted impacts of climate change, the flooding extents are predicted to also impact on the residential properties on Tilley Road.

6.3.2. Results –Transmission Gully Project

The Main Alignment, as supplied in 3D format, was inserted into the model and the hydrological inputs updated to reflect the change in land use associated with the highway construction. The model was rerun for the 10 and 100-year rainfall events and compared with the pre-construction scenarios to identify potential hydraulic impacts of the development. The model was then used to evaluate options to avoid, temporally limit and remedy, or mitigate the impacts.

The initial model results identified that there was very little loss of storage associated with the new highway and the additional runoff was less than 1% of the total catchment runoff. The results indicated that there are two key areas of effects that would need to be closely assessed:

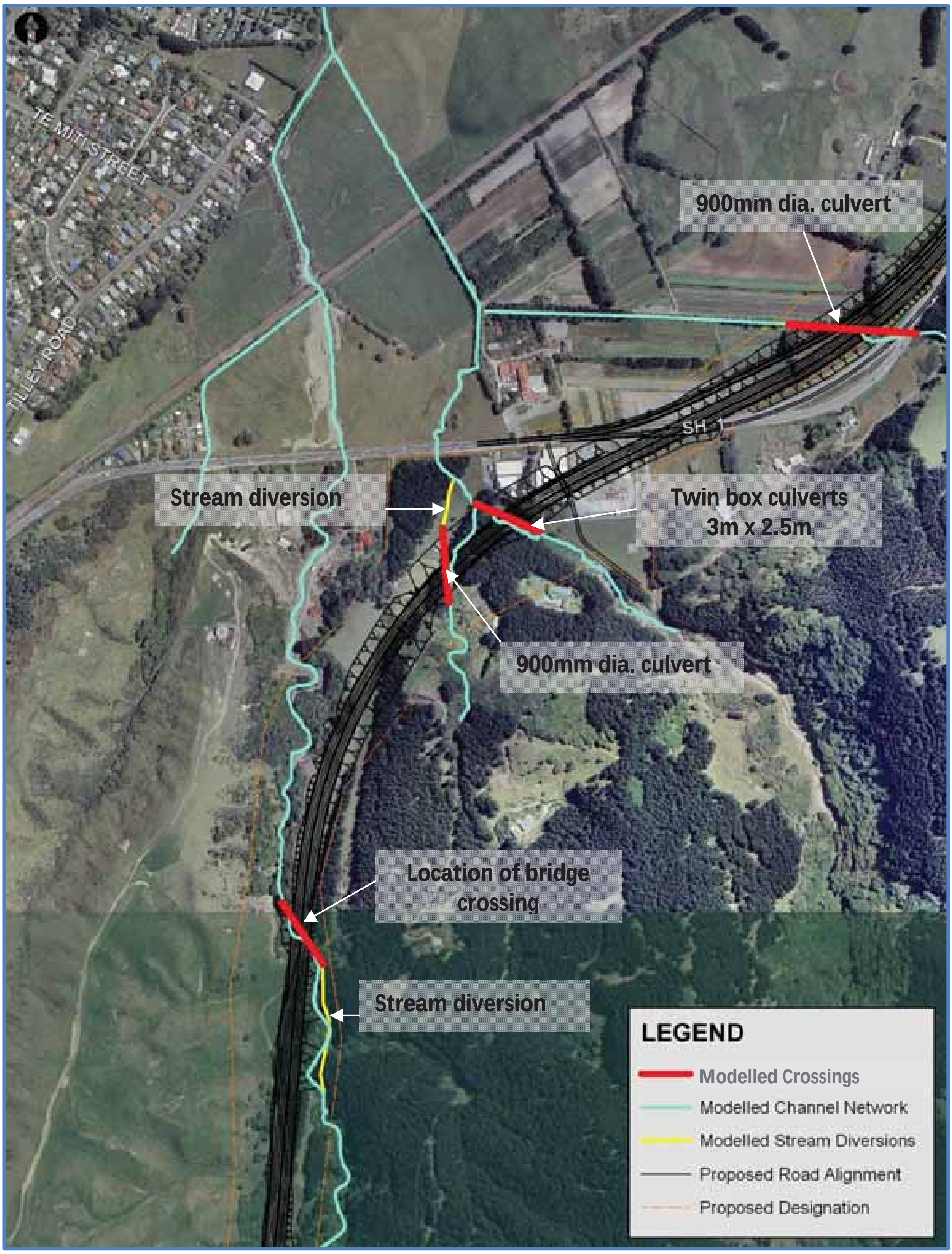
- The flooding impacts upstream and downstream of the proposed culverts on the stream
- The alteration of the stream channel.

6.3.3. New Stream Crossings

A bridge is proposed for the Te Puka Stream at scheme design chainage 2,780m. This bridge will require a diversion of approximately 200m immediately upstream to avoid the proposed earth fill embankments that are to be constructed to support the road. This structure and diversion have the potential to alter the Te Puka Stream hydraulics and cause an increase in flooding and bed scour. In addition to the structure on the Te Puka Stream, the construction of the road will require two culverts and a diversion on the Wainui Stream which also have the potential to alter stream hydraulics.

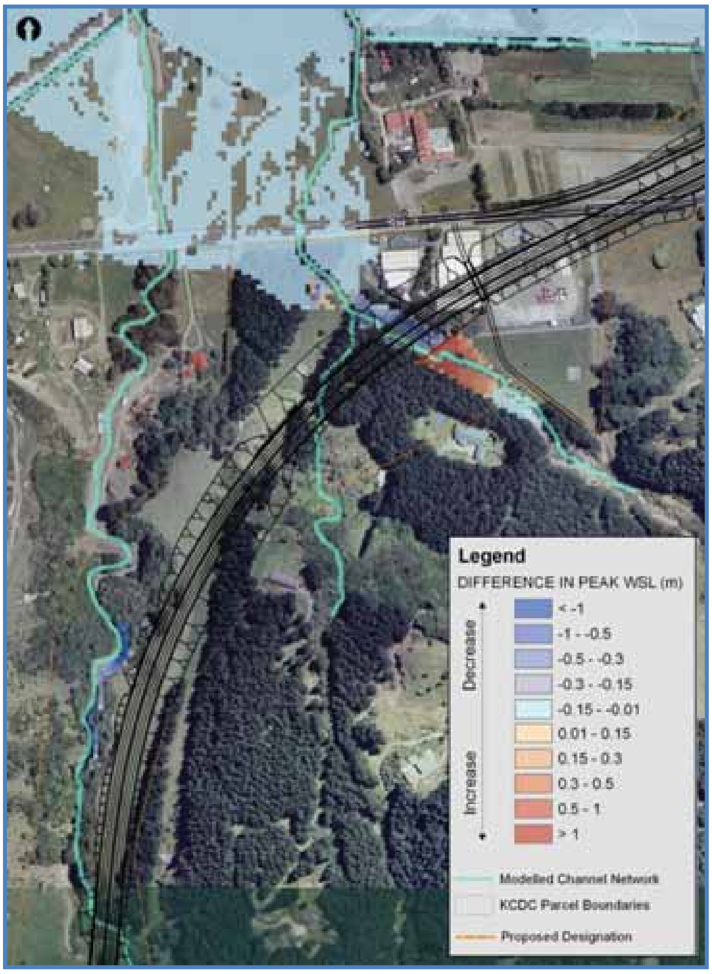
Preliminary sizing of each of the bridge & culverts on the Te Puka and Wainui Streams was undertaken using peak flow discharges for these catchments as a separate component of Workstream 4.The hydraulic model was used to further test the culvert sizes, shown in **Figure 14.24**, to identify potential adverse effects. In addition to economical and practical considerations the key criteria in the assessment were:

- The minimisation of upstream and downstream flooding effects
- Protection of the new highway infrastructure from flooding in a 100-year event.



- **Figure 14.24 - Proposed Structures on the Te Puka & Wainui Streams**

Hydraulic modelling indicates that in general the impacts of the new culverts will not be significant. **Figure 14.26,** on the following page, shows the comparison in peak water surface levels in the 100-year rainfall event between the pre and post-construction scenarios. Adverse effects are predicted to be localised to the area just upstream of the twin box culvert on the Wainui Stream (this is shown in more detail in **Figure 14.25**). The results indicate up to a metre increase in flooding depths. However, the increase in peak flooding level is topographically constrained and within the proposed designation. All three stream crossings in this location are able to convey the 100-year flood flows with little or no heading up of water levels above the soffit level, upstream of the culvert. The peak water levels are well below the carriageway. Detailed design will need to consider protection of the inlets from the potential of blockages or debris flows.

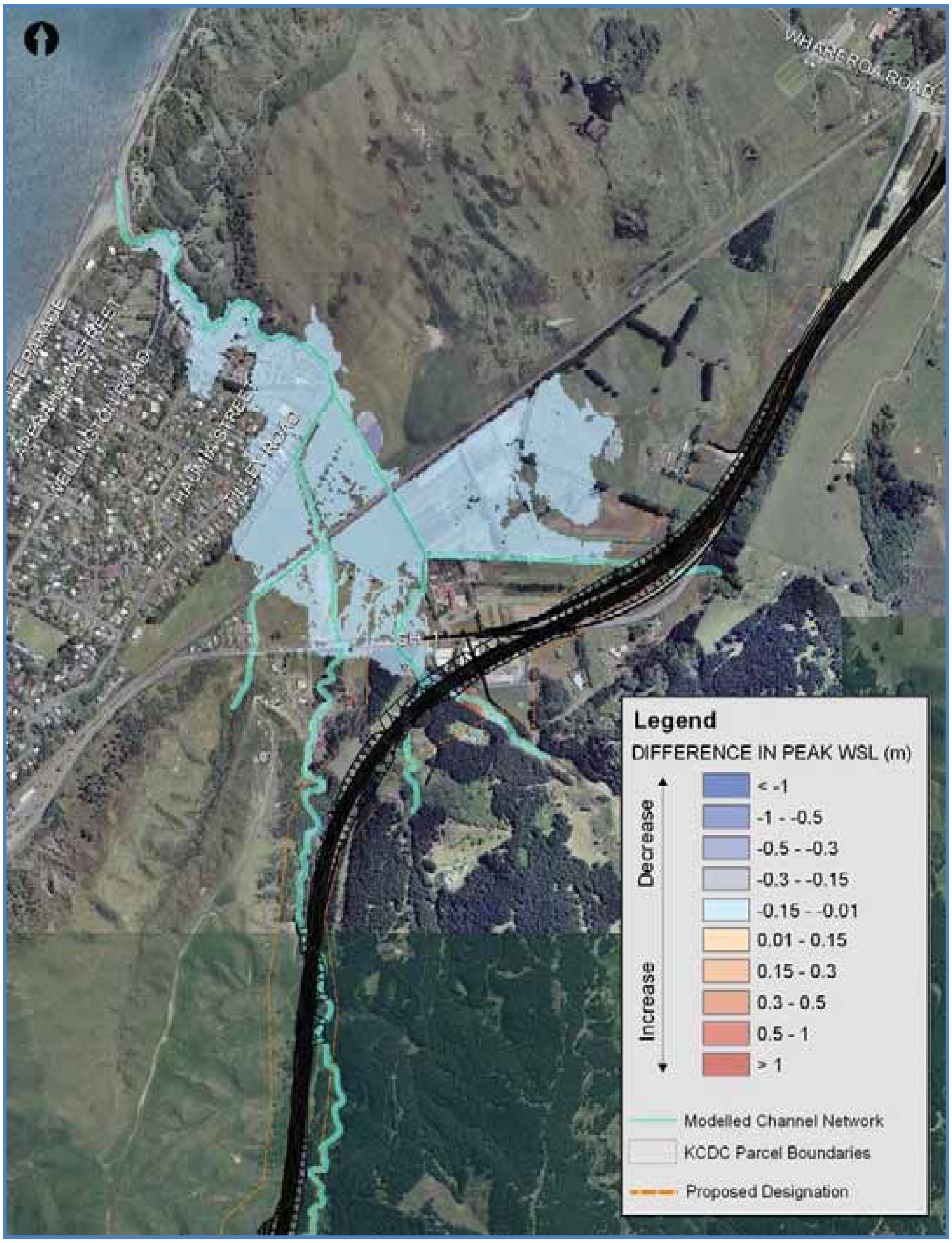


- **Figure 14.25 - Comparison of Peak Water Levels in the 1% AEP Rainfall Event between the Pre and Post-Construction Scenarios**

Testing of a larger culvert structure on the Wainui Stream was undertaken to investigate the potential for reduced adverse impacts. However, even with a doubling of the culvert width, the model revealed that the peak water surface levels would still increase at this location and there were also adverse impacts downstream.

As the culvert entrance at this location will already need protecting to prevent scour of the channel bed and banks, the impacts just upstream of the new highway crossing of the Wainui Stream are considered to be able to be effectively mitigated by localised protection.

Downstream of the alignment, the model indicates that there could be minor reductions in peak water surface levels as the result of the extra storage created by the culverts under the proposed highway.

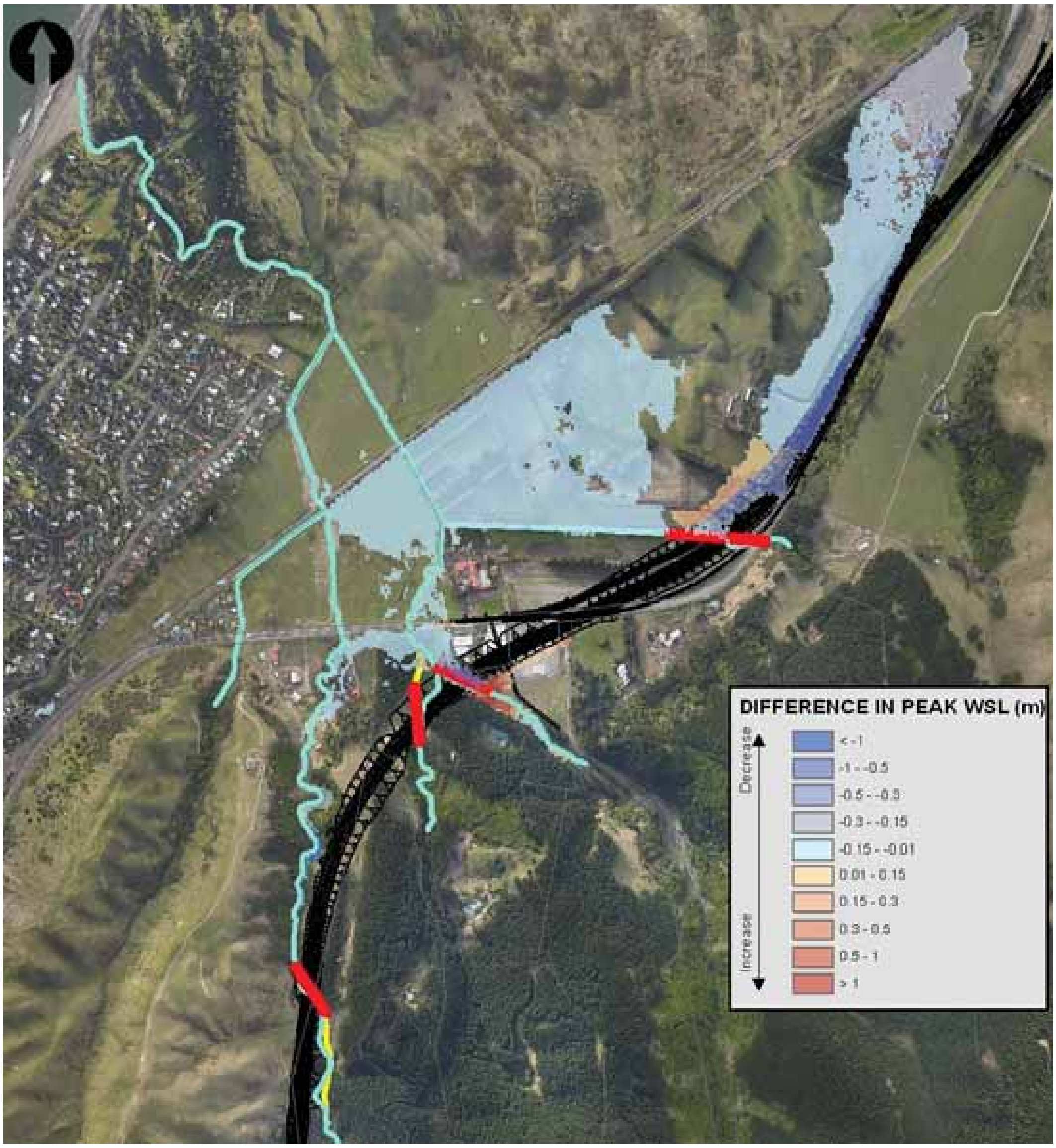


- **Figure 14.26 - Comparison of Peak Water Levels in the 1% AEP Rainfall Event from the Pre and Post-Construction Scenarios**

6.3.3.1. Oversized Events

To further investigate the flood risks of the proposed highway under flooding events larger than the 100 year design the model was rerun with input flows of 1.5x100^{cc} year event. These inputs were run in the pre and post construction scenarios and the results compared (see **Figure 14.27**).

Increases in peak water surface levels were found to be localised and generally upstream of the proposed alignment the residential areas of Paekakariki were found to have almost no additional increase in risk even in this extreme event. Furthermore the highway was not found to overtop.

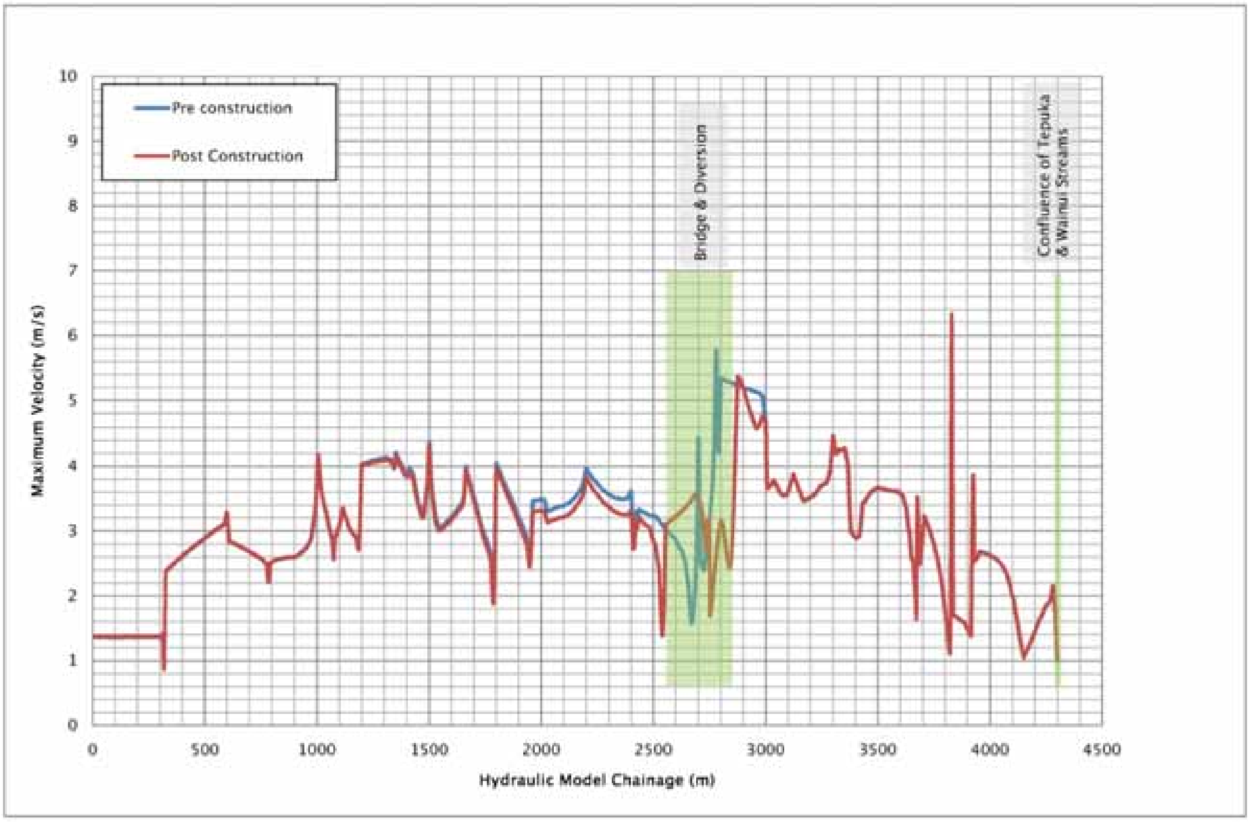


- **Figure 14.27 - Comparison of Peak Water Surface Levels Between an Oversized Event Pre and Post Road Construction**

6.3.4. Stream Velocity Effects

The hydraulic model was used to investigate the impacts of the new highway on the Te Puka and Wainui Stream channel velocities. New structures and diversions can lead to increased velocities, resulting in scour or changes the stream environment.

A comparison of the pre and post-construction peak stream velocities was made for the 10% AEP flood event in the Te Puka Stream, as shown in **Figure 14.28**.

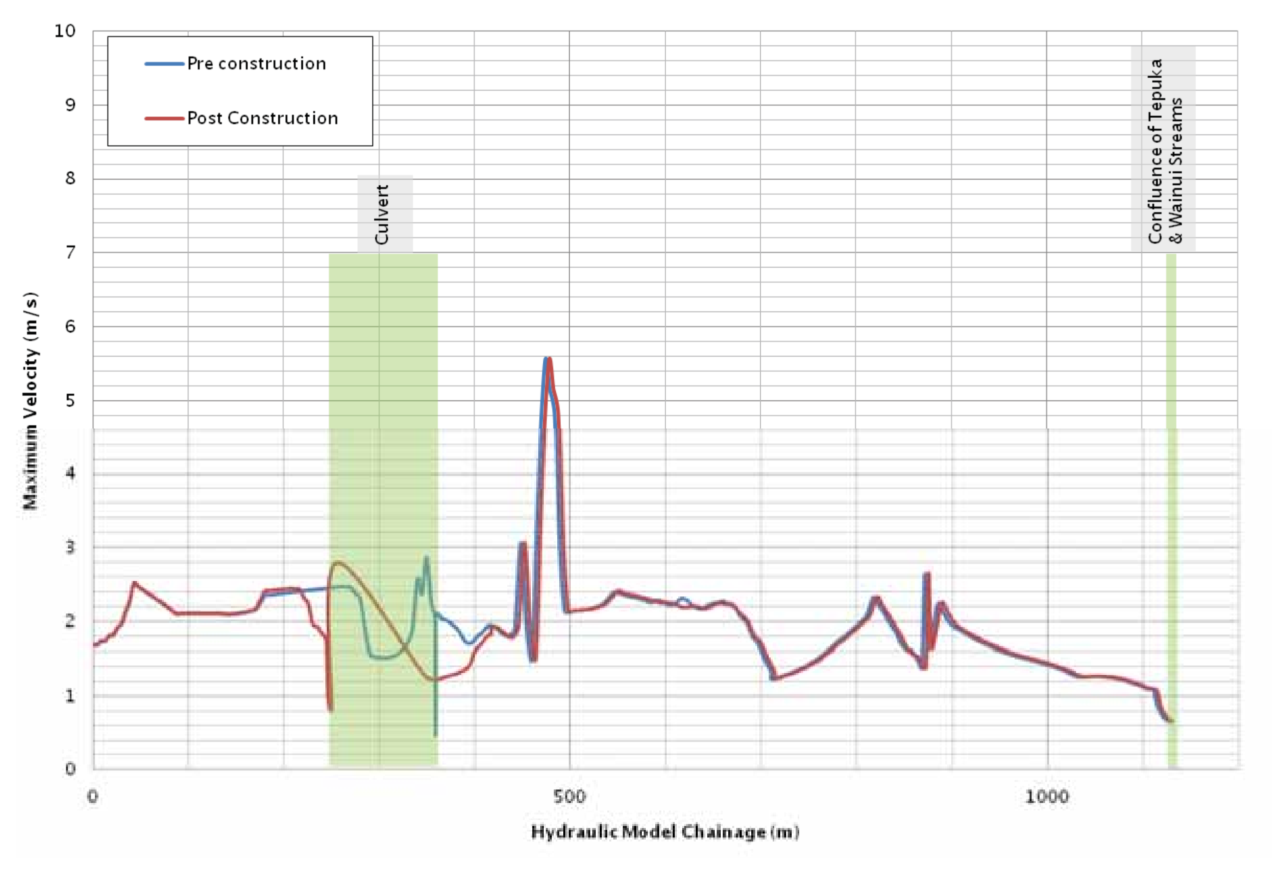


- **Figure 14.28 - Peak Velocities Comparison Between the Pre and Post-Construction Scenario in the Te Puka Stream in a 10% AEP Storm Event**

The results show that the Transmission Gully Project results in localised changes in the velocity profile around the proposed diversion and bridge. A localised increase in velocity of 0.5-2m/s is predicted in the diversion upstream of the bridge as the diversion shortens the stream length and removes the natural meander. This increase is not anticipated to cause additional scour of the stream bed and is comparable to the surrounding stream velocities. It is expected the design of the diversion will include vegetation on the stream banks which will reduce velocities and prevent additional stream scouring.

The model of the pre-construction stream indicates already high velocities at the location of the proposed bridge. This is due to the steep and confined gorge-like environment of the Te Puka Stream in this area. As a result, the model indicates that the proposed bridge is unlikely to significantly increase the velocities above those expected in the pre-construction situation. However, these velocities in the 10-year flood are close to 6m/s and will require careful scour protection at the culvert outlet.

Similarly the comparison for the 10-year flood event pre and post-construction in the main branch of the Wainui Stream is shown in **Figure 14.29.**



- **Figure 14.29 – 10% AEP Peak Velocities Comparison in the Pre and Post-Construction Scenario in the Wainui Stream**

The hydraulic model indicates that the impacts of culvert construction on the Wainui Stream will be limited to 100m both upstream and downstream of the structure. Both upstream and downstream of the culvert the channel will need to be widened to facilitate a smooth flow transition to the proposed twin box culverts. These alterations will increase the stream cross-sectional area from the pre-construction scenario and result in localised decreases in stream velocities of 0.5-1m/s in a 10-year flood event. This minor reduction in velocity is not considered a significant impact and will be managed in the detailed design of the culvert inlet and outlet.

6.3.5. Upstream Te Puka Diversion

In the upper Te Puka there is a major diversion as show in Sheet 2 in Appendix 14.G. This diversion has been assessed in detail in Volume 5 as part of SSEMP 1. As the existing stream will be replaced by a constructed waterway velocities will be considered with the ecology requirements as part of the design brief.

6.3.6. Te Puka / Wainui Conclusion

The hydraulic analysis of the Transmission Gully highway through the Te Puka and Wainui catchments indicates that the effects of the new highway are localised and manageable. The model indicates no increase in flood risk to the properties downstream of the highway.

6.4. Duck Creek

6.4.1. Results –Transmission Gully Project

The alignment of the Transmission Gully highway passes through the upper catchment of Duck Creek, running parallel to the main channel. Proportional to the catchment area, this catchment is expected to have one of the highest percentage changes in land use associated with the new highway (approximately a 2% increase in connected impervious area). Furthermore, both the Waitangirua and Whitby Link Roads that connect the suburbs to the highway are within the catchment. Construction of the Waitangirua Link Road will require a crossing of Duck Creek (culvert BSN 17). This crossing provides the opportunity to mitigate the flooding impacts associated with the additional runoff from the highway. The Waitangirua Link Road is designed to cross Duck Creek at an elevation in excess of 10m above the creek bed. By appropriately sizing the culvert through the embankment, the flow rate during a flood could be restricted and excess water stored behind the embankment.

Preliminary peak flow analysis based on the scoping design of the highway has been undertaken to confirm the merits of this approach. Upstream of the Main Alignment crossing of Duck Creek, the catchment has been hydrologically assessed to predict runoff in a 10% and 1% AEP flood events, including the predicted impacts of climate change. The results are summarised in **Table 14.10.**

Table 14.10 - Duck Creek Catchment Characteristics and Peak Flows

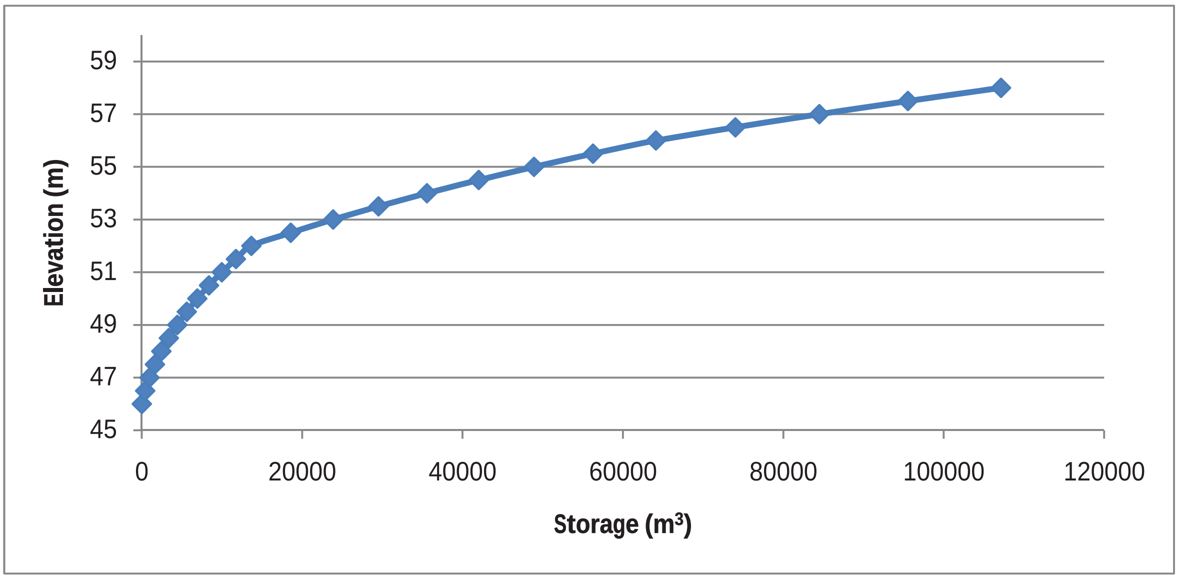
	Upstream Catchment Area (ha)	Connected Impervious Area	Peak Discharge (m^3/s)	
			10% AEP	1 % AEP
Pre construction	605	0	28.8	52.2
Post construction	605	4.3	29.4	52.8

These results indicate that there is only a 2% difference in peak flows between the pre and post-construction situations in a 10% AEP rainfall event, indicating the impacts of the road construction on peak flows are likely to be minor. To demonstrate this would require a hydraulic model to predict impacts on downstream flooding, alternatively the Duck Creek culvert could be sized to utilise the upstream storage to mitigate the impacts on flood flows of the road construction.

The stage storage relationship behind the road alignment has been calculated using the 2m contour data (see **Figure 14.30**). The chart shows that at a depth of 10m above the invert of the culvert inlet there is approximately 70,000m^3 of storage. Based on this data the preliminary culvert sizing was undertaken using the following criteria:

- As per NZTA *Bridge Manual*, the culvert must pass the 10-year peak runoff from the catchment (27.4m^3/s) without the culvert heading up over the soffit
- The culvert must pass the 100-year peak runoff (53.9m^3/s) without ponding/stored water coming within 0.5m of the road level
- The low flow velocities must be controlled to allow for fish passage

- The flows in the 2% and 1% AEP flood event must be restricted to or below pre-development levels.



- **Figure 14.30 - Stage Storage Relationship above the Duck Creek Culvert**

Preliminary calculations show that a 3.0m wide by 4m high box culvert would be sufficient to meet the above criteria. The 2% slope will keep velocities low and facilitate fish passage. Furthermore, the culvert is expected to be constructed slightly lower than the existing bed level to allow a layer of gravel to cover the invert assisting fish passage. The attributes of the culvert are shown in **Table 14.11**.

- **Table 14.11 - Duck Creek Culvert Dimensions**

Culvert Inlet Dimensions	U/S Invert Level (m)	D/S Invert Level (m)	Culvert Length (m)	Culvert Slope	Highway Level	Peak U/S Headwater Level in a 1% AEP Flood	Peak U/S Headwater Level for 10% AEP Flood	Velocity in a 10% AEP Flood (m/s)
3.5x4m	45.0	44.0	55	2%	75	50.8	48.8	2.3

An alternative solution to using the Waitangirua Link Road culvert would be to utilise storage behind the D7 culvert. Of the four culverts upstream of the highway, D7 provides the most storage with approximately 30,000m^3 (**Table 14.12**). Approximately 10,000m^3 is required to reduce peak flow to pre-development levels. The storage potential can be optimised by reducing the size of the culvert to restrict runoff during flood events.

- **Table 14.12 - Duck Catchment Storage Potential**

Culvert	Volume (m^3)	Maximum Storage Depth (m)

D7	30,000	11.2
D8	3,000	5.5
D9	7,000	7.9
D14	4,500	5.7

6.5. Linden Stormwater Network

The catchment area draining through the existing stormwater network in Linden is small at only 0.24km^2. The network collects runoff from the rural area east of SH1, and from the Linden township west of the highway (**Figure 14.31**). The main stormwater network is approximately 600m long, on a reasonable grade, and drains to the Porirua Stream. Several depressions through the Linden township allow water to pond. Given the terrain the stormwater network is relatively deep to form a continuous grade beneath these depressions.

6.5.1. Pre Construction Baseline

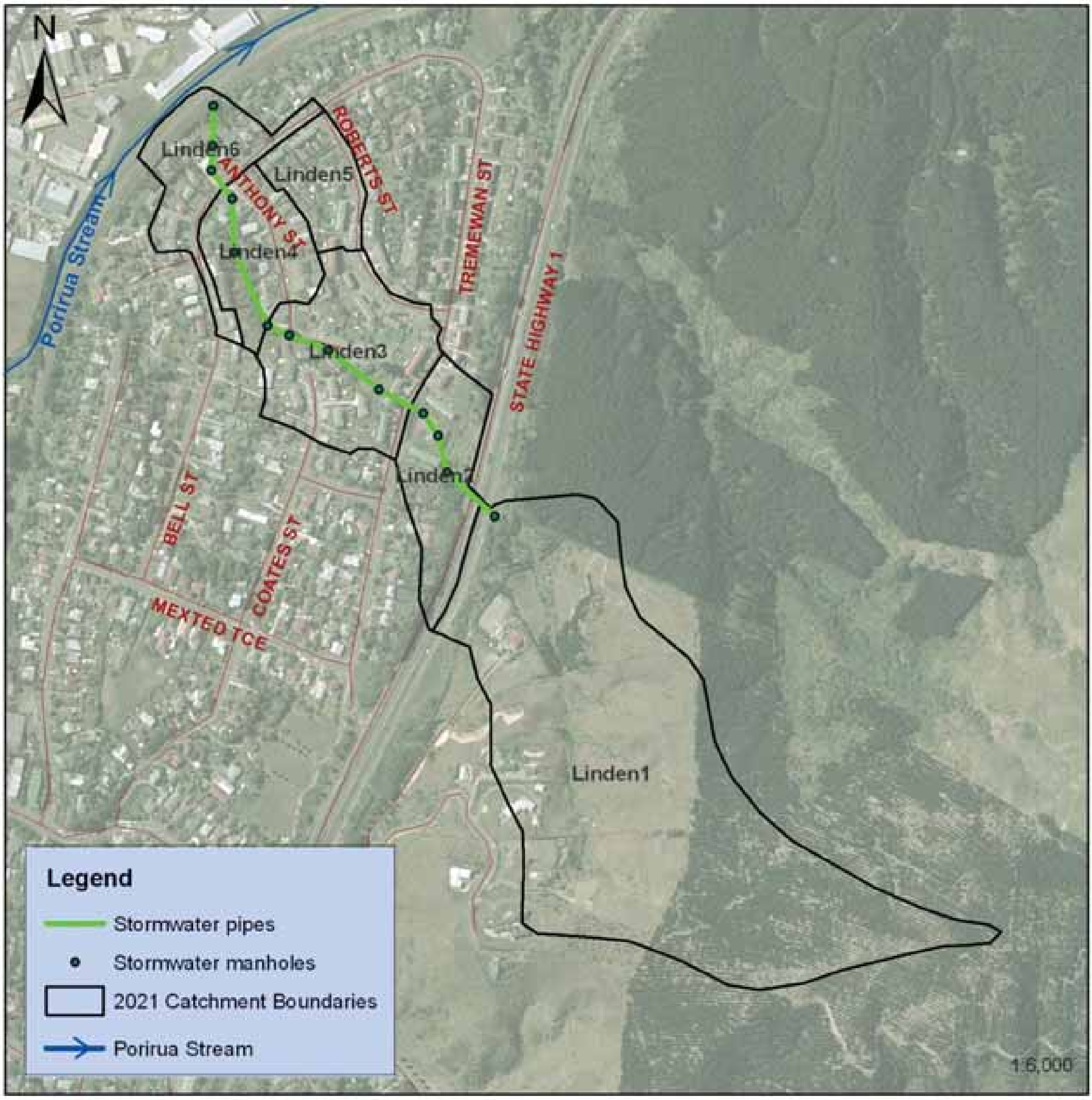
Wellington City Councils stormwater database contains very little information on this section of network. A site survey of manholes on public land was conducted to supplement the information. Where manholes were on private land, or could not be located, data was interpolated providing a best estimate of reality.

Hydrological assessment involved dividing the Linden area into six subcatchments including the natural hill catchment east of SH1, and the residential area surrounding Roberts and Anthony Streets. This is shown in **Figure 14.31**, along with the modelled stormwater network.

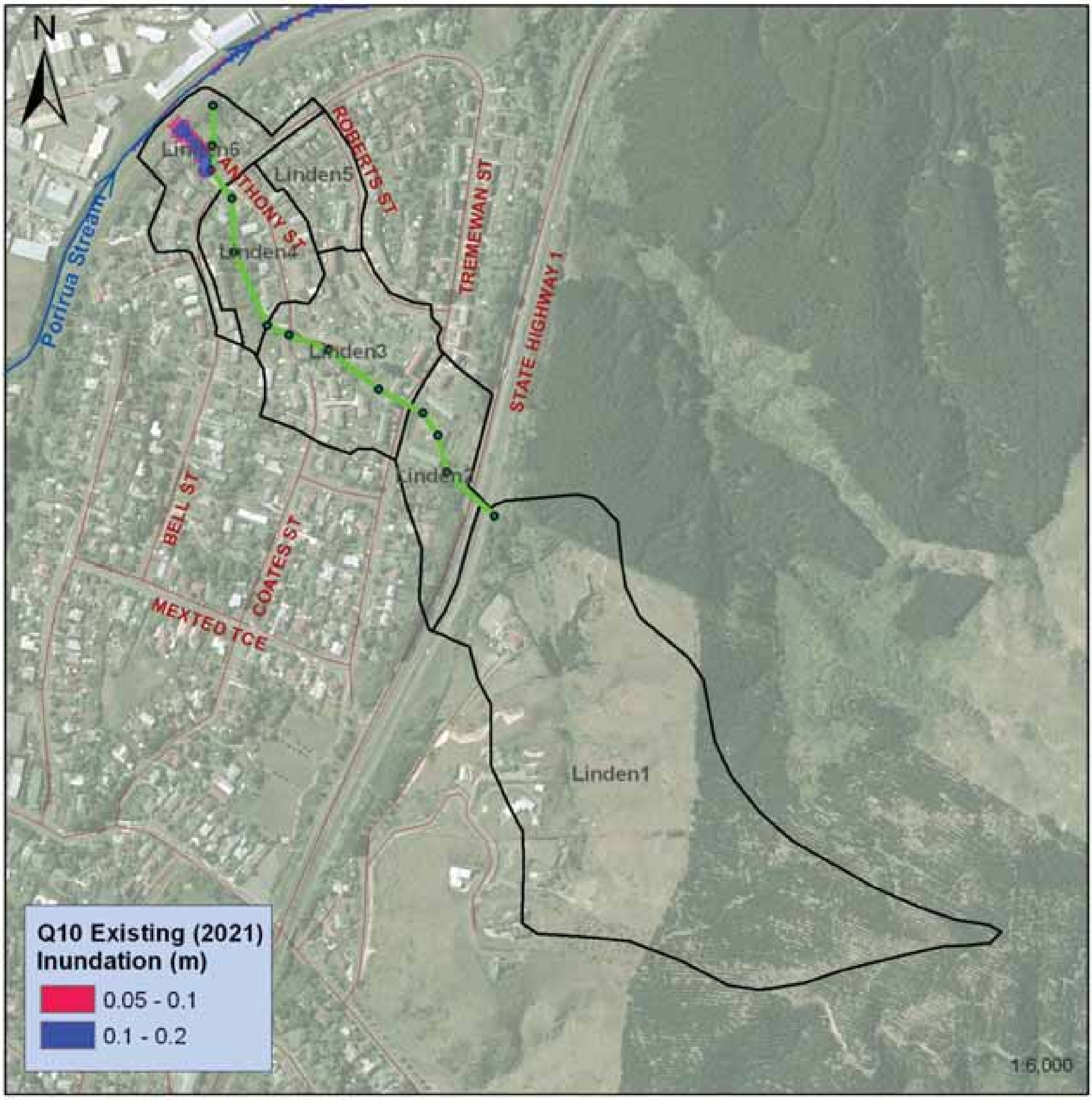
Modelling of the existing network has shown that overflows occur only in the lower system in the 10% AEP storm event, and in the lower system and two basins east of Coates St and west of SH1 in the 1% AEP storm, as shown in **Figure 14.32** and **Figure 14.33**.

6.5.2. Results - Transmission Gully Project

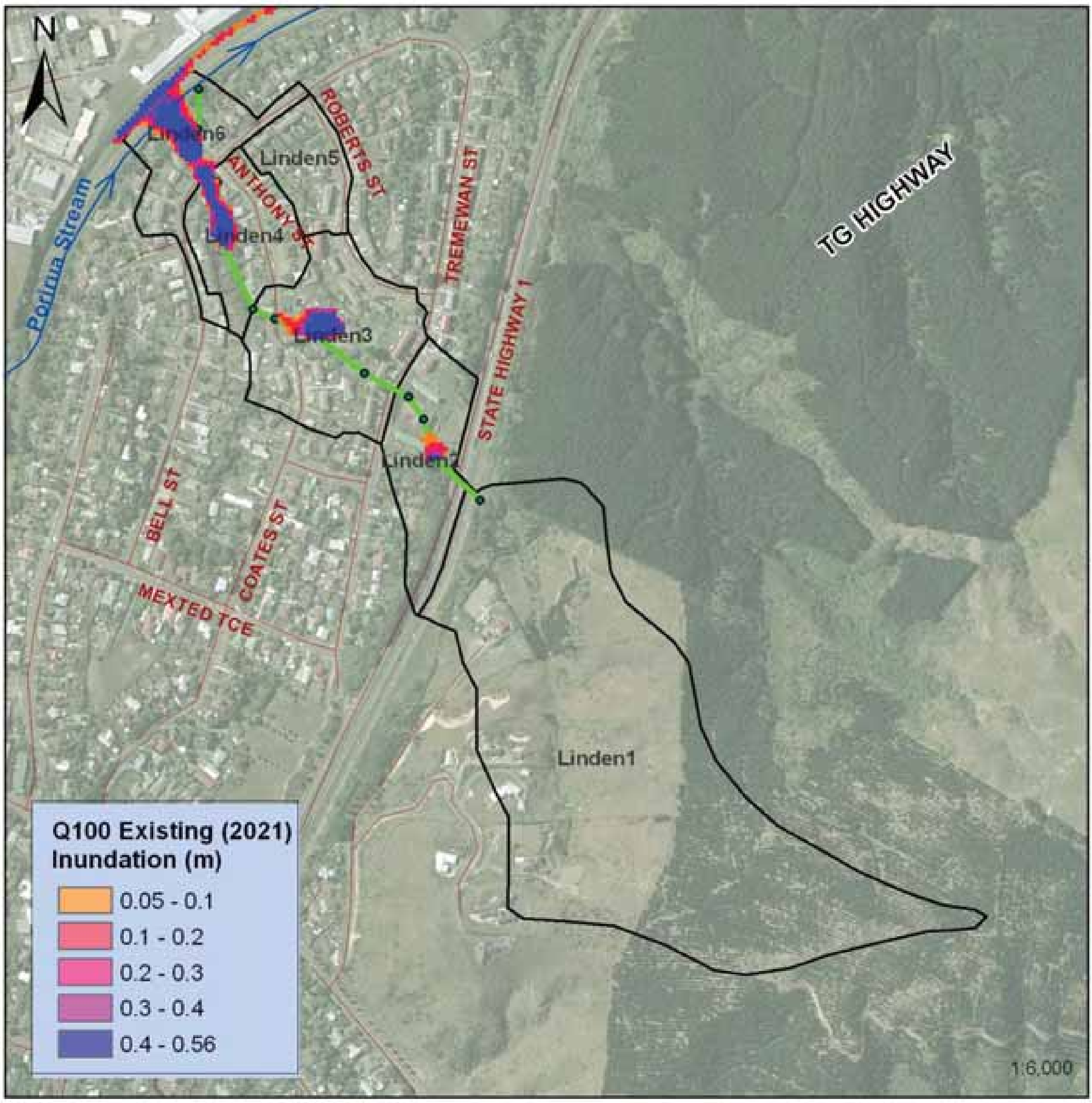
The post Project catchment boundaries and stormwater network are displayed in **Figure 14.34**. Modelling the stormwater overflows in the Mike21 surface elevation model indicated potential areas of flooding. Inundation depths from the 2021 (existing) scenario have been subtracted from the 2031 (post Transmission Gully runoff) scenario to assess the difference. These are displayed in **Figure 14.35** and **Figure 14.36**. Changes in inundation of less than 50mm have not been shown.



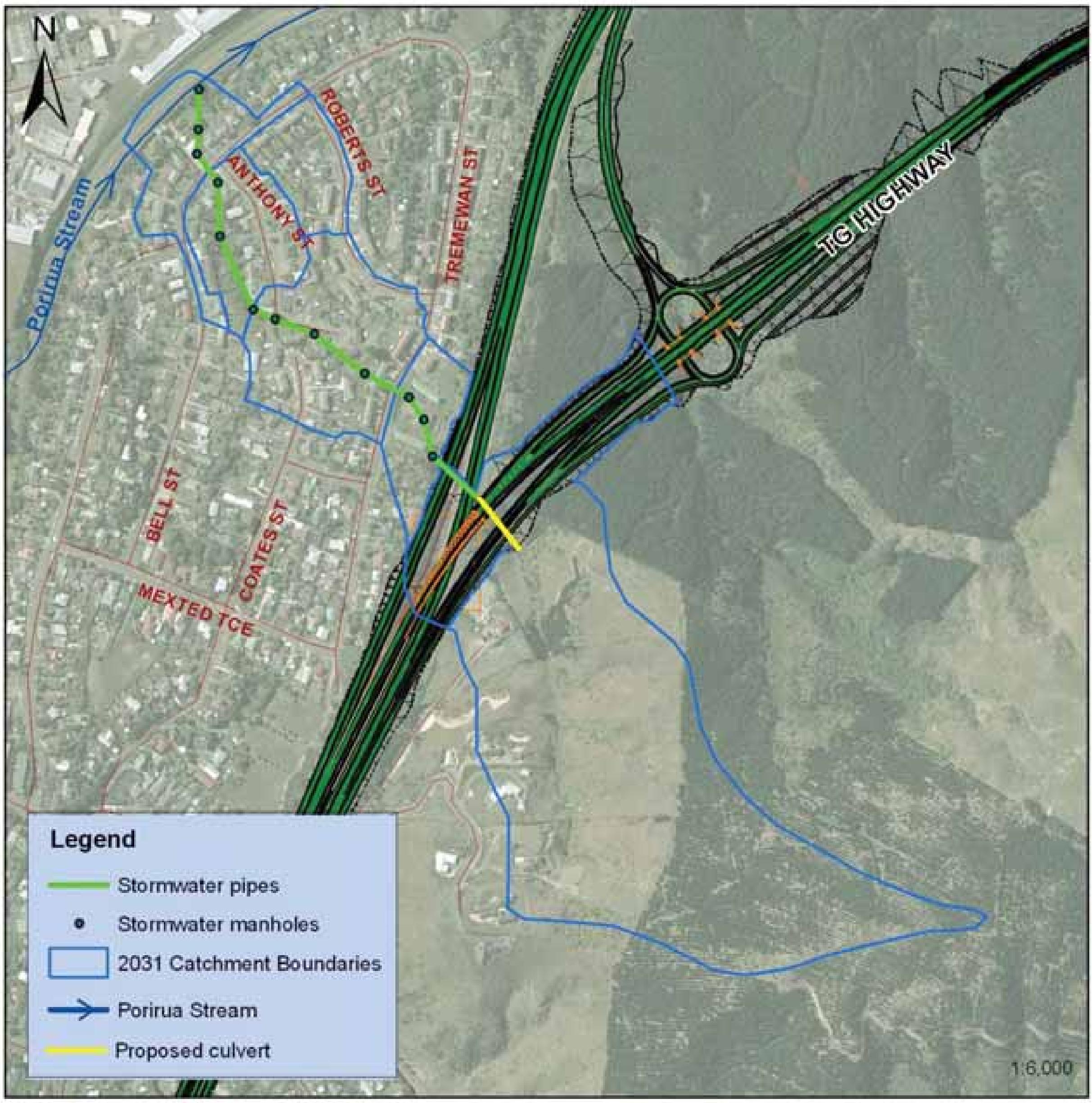
- **Figure 14.31 – Existing Linden Catchment Boundaries and Modelled Stormwater Network.**



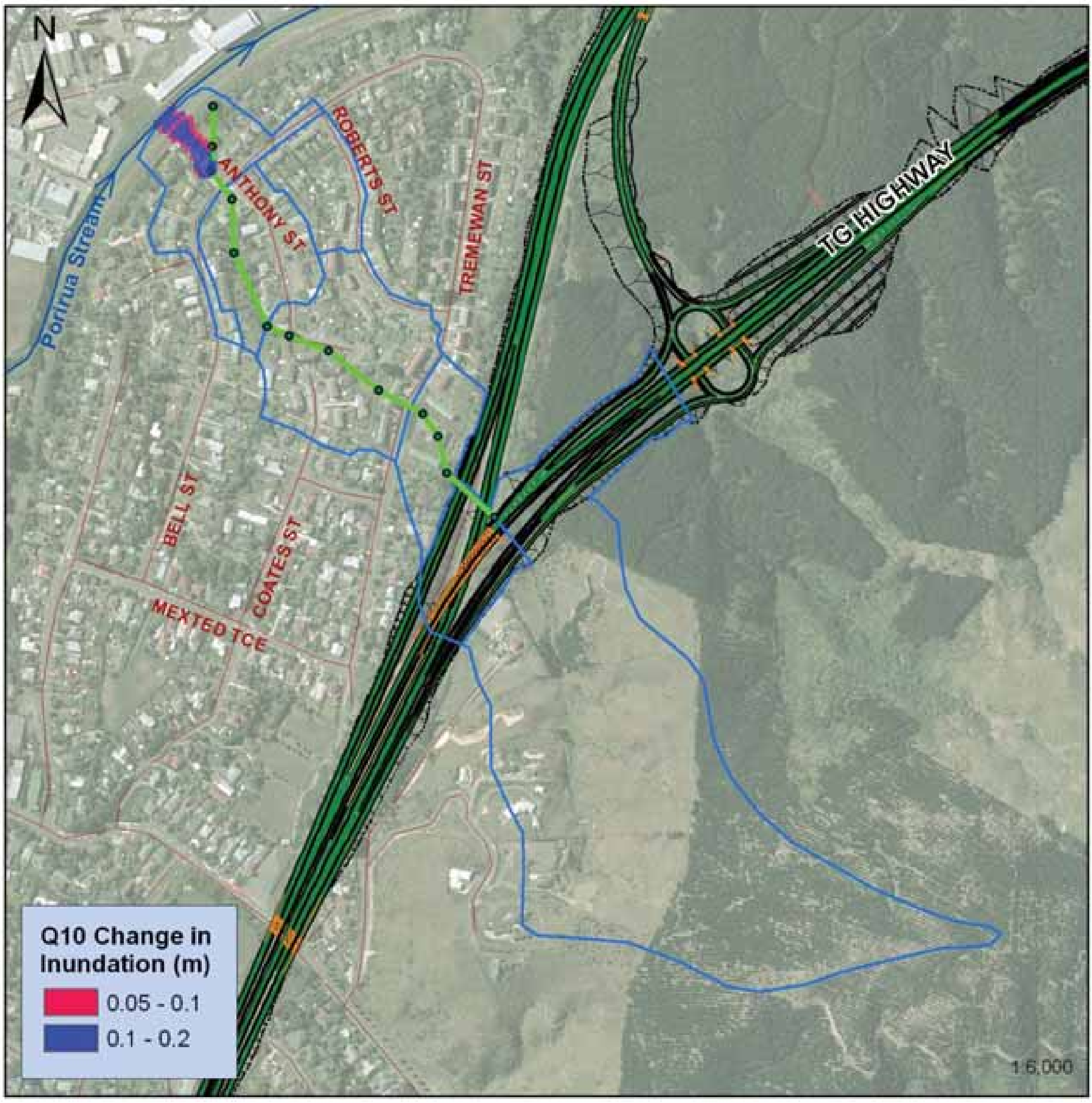
- **Figure 14.32 - Existing Surface Flooding in a 10% AEP flood.**



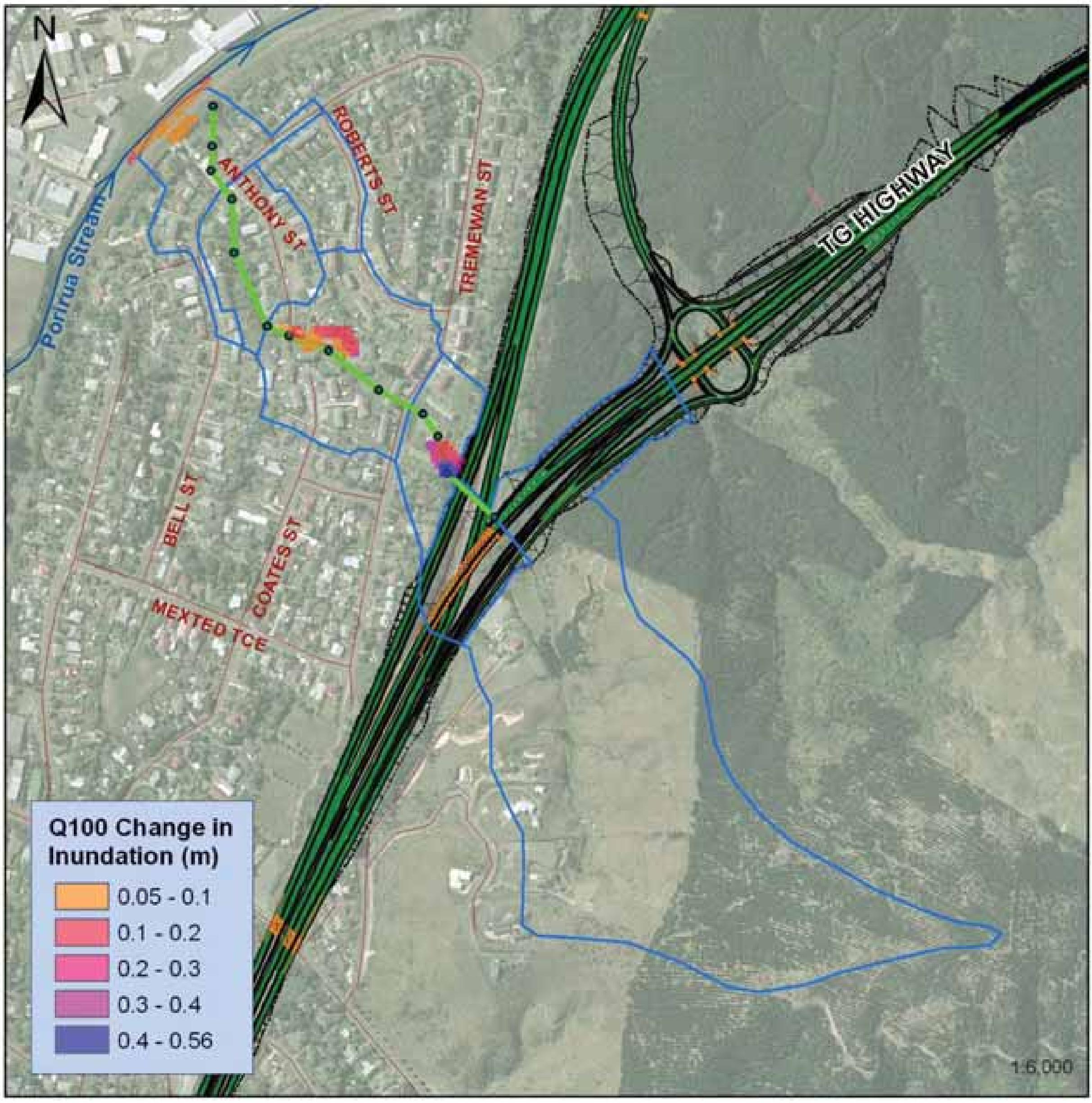
- **Figure 14.33 Existing Surface Flooding in a 1% AEP Flood.**



- **Figure 14.34 - Linden Catchment Boundaries and Modelled Stormwater Network (including a section of the Transmission Gully Project, proposed to drain through the stormwater network).**



- **Figure 14.35 - Change in Inundation in a 10% AEP Flood.**



- **Figure 14.36 - Change in Inundation in a 1% AEP Flood.**

In the 10% AEP scenario, the maximum increase in inundation caused by additional runoff from the Project is 200mm. This occurs in a very limited area downstream of Bell Street, adjacent to the outlet. In the 1% AEP event, the maximum increase in inundation is 550mm. This occurs between the existing SH1 and Tremewan Street.

6.5.3. Mitigation Options

There is potential to store water on the upstream (eastern) side of the road to attenuate the flood peak and reduce the impact of flooding downstream. The upper catchment has the potential to store nearly 18,000m^3 of water. The NZTA guidelines states that culverts should be capable of conveying the 10% AEP without head rising above the pipe soffit, and the road surface level should be at least 500mm above design stormwater levels in a 1% AEP.

To reduce the impact of flooding to the Linden township, several options are considered.

1) Upgrade the existing stormwater network
2) Store peak flows in the upper catchment
3) Divert runoff to secondary overflow paths.

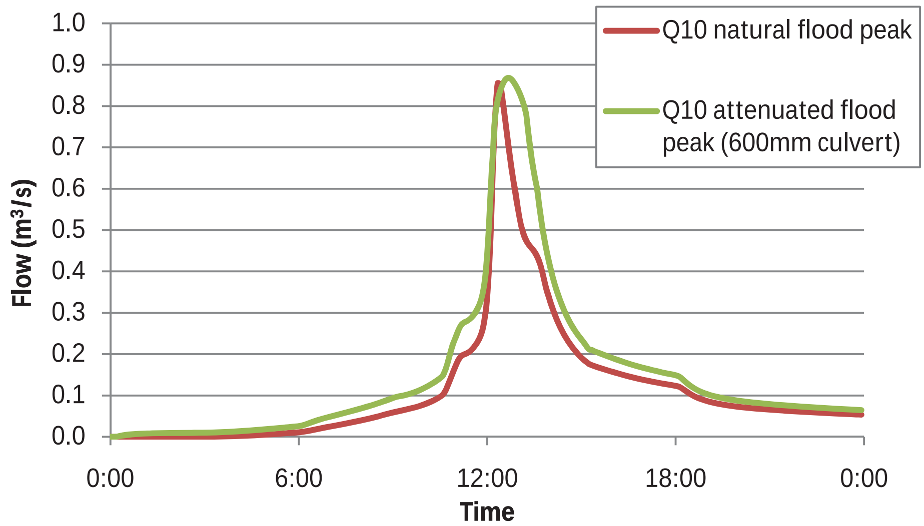
The existing stormwater network from SH1 to the Porirua Stream lies relatively deep (depth to inverts of up to 4.1m) and runs beneath private property and local roads. There is no major bottleneck in the network, with flooding occurring as a result of grade changes reducing the head-pressure. Minimising inundation in a 1% AEP event would require upgrading the majority of the network, which is more complex in a residential situation than 'greenfield' road drainage.

Storage in the upper catchment can be maximised to reduce the downstream flood peak by reducing the diameter of the proposed culvert beneath the Main Alignment. The proposed culvert in this location is a 975mm pipe. By reducing the diameter to 600mm will cause approximately 550m^3 of water to be stored in a 10% AEP event, and raise the water level above the inlet level by 1.4m. In a 1% AEP event, approximately 2600m^3 of water will be stored bringing the water level to 3.0m above the inlet. By attenuating the flood peak NZTA guidelines are met at the 1% AEP with levels remaining at least 500mm below the road surface, however guidelines are not met at the 10% AEP with surcharge at the inlet. This non-conformance is considered acceptable in this circumstance.

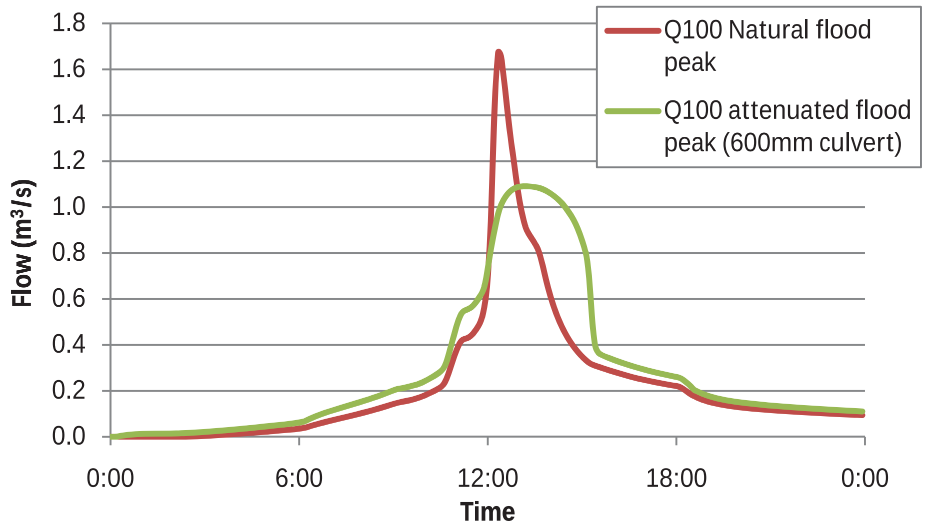
Attenuating flow results in a reduction of the 1% AEP flood peak from 1.67m^3/s to 1.09m^3/s. The 10% AEP increases marginally from 0.85m^3/s to 0.87m^3/s. This information is tabulated in **Table 14.12**. **Figure 14.37** compares the 10% AEP flow discharging from the rural catchment upstream of SH1 to the attenuated flow from restricting the culvert to 600mm. The 1% AEP flow attenuated by restricting the culvert to 600mm is shown in **Figure 14.38**.

- **Table 14.13 Linden 1 (Upper Catchment) Peak Flows and Maximum Storage Volumes.**

AEP	No Road Flood Peak		With Road Flood Peak - no attenuation		With Road Flood Peak - 975mm culvert		With Road Flood Peak utilising storage - 600mm culvert	
	Peak (m^3/s)	Storage (m^3)	Peak (m^3/s)	Storage (m^3)	Peak (m^3/s)	Storage (m^3)	Peak (m^3/s)	Storage (m^3)
10%	0.85	0	1.13	0	1.11	130	0.87	550
1%	1.67	0	2.11	0	2.01	350	1.09	2630



▪ **Figure 14.37 - 10% AEP flood peak from the rural catchment upstream of SH1 compared to the attenuated peak from restricting flow with a 600mm culvert.**



▪ **Figure 14.38 - 1% AEP flood peak from the rural catchment upstream of SH1 compared to the attenuated peak from restricting flow with a 600mm culvert.**

Secondary overflow paths do not appear to be an option given the terrain of the residential area. Between SH1 and Tremewan Street is a gully where water would naturally pond. West of Tremewan Street, the natural overflow path is behind the houses between Anthony Street and Bell Street. This would cause a heightened flood risk to properties. Downstream of Bell Street, overflow paths would pass directly through properties.

6.5.4. Conclusion

The existing stormwater network (as modelled) does not have the capacity in a 1% AEP event to convey the additional runoff from the Project. In a 10% AEP event, there is some capacity, however minimal flooding may occur at the bottom of the network between Bell Street and Porirua Stream. The extent of flooding is a best estimate, based on the assumed depth of the stormwater network. Of the proposed mitigation options, storing water within the upper catchment to attenuate the flood peak is preferred. The benefits will need to be confirmed in the detailed design phase of the project.

6.6. Waitangirua Stormwater Network

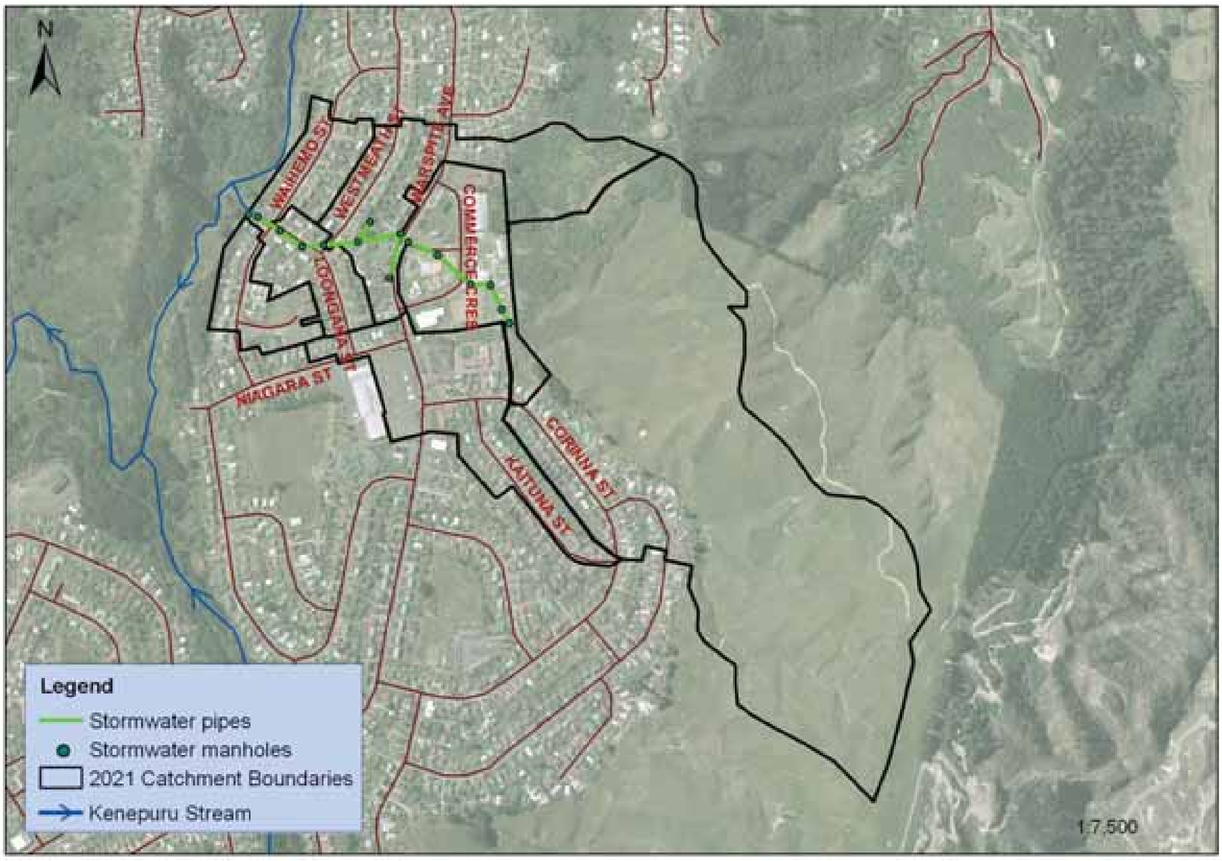
The Waitangirua stormwater network drains runoff from the urban area of Waitangirua and the rural hillside behind the town, and discharges into the Kenepuru Stream (**Figure 14.39**). The catchment area is approximately 0.68km^2. It is proposed that a small section of drainage from the Waitangirua Link Road, from the ridge down towards the urban area be diverted through the existing stormwater network. This is approximately 600m of road.

6.6.1. Pre Construction Baseline

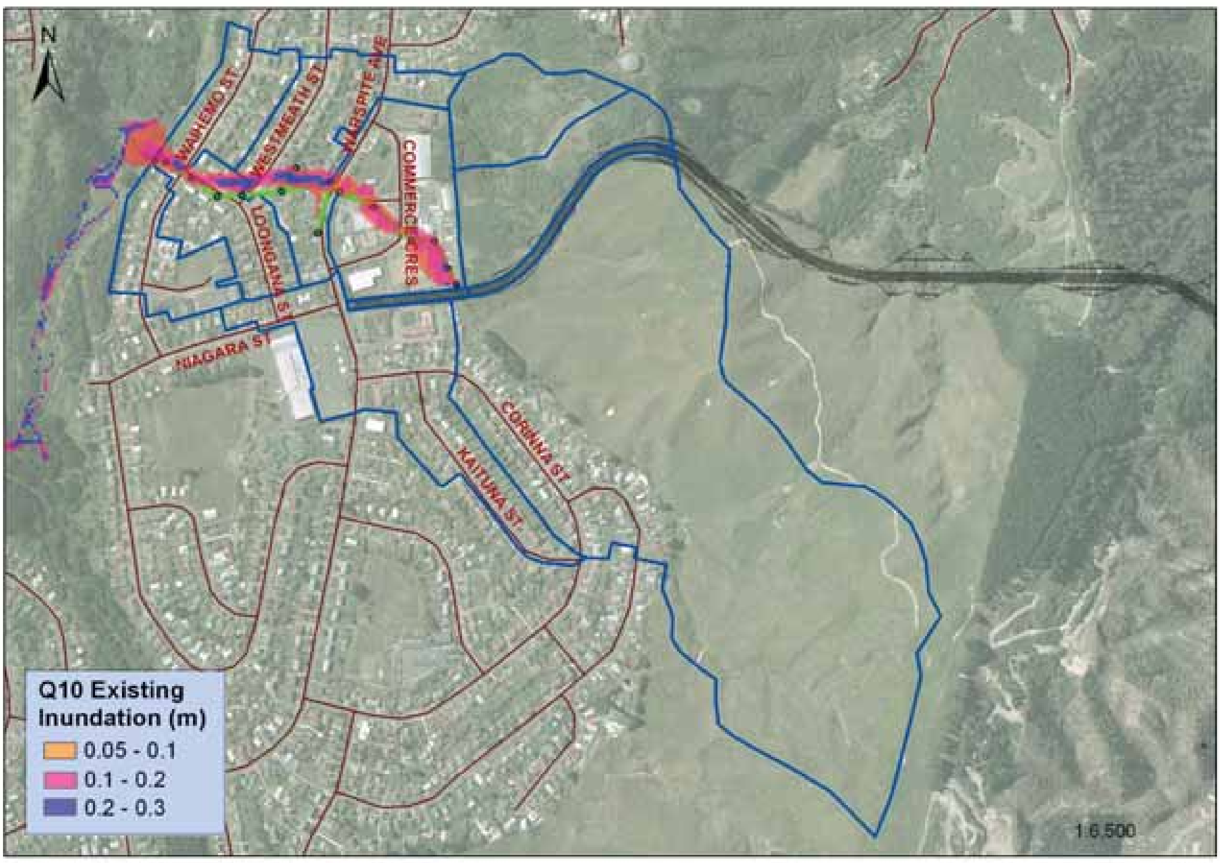
The localised hydraulic model of the Waitangirua stormwater network was constructed to understand the existing flood risk to the Waitangirua township, and compare it to the potential flood risk if runoff from the Link road is diverted through the existing stormwater network.

Hydrological assessment involved dividing the area into six subcatchments. The subcatchments, and modelled stormwater network and shown in **Figure 14.39**.

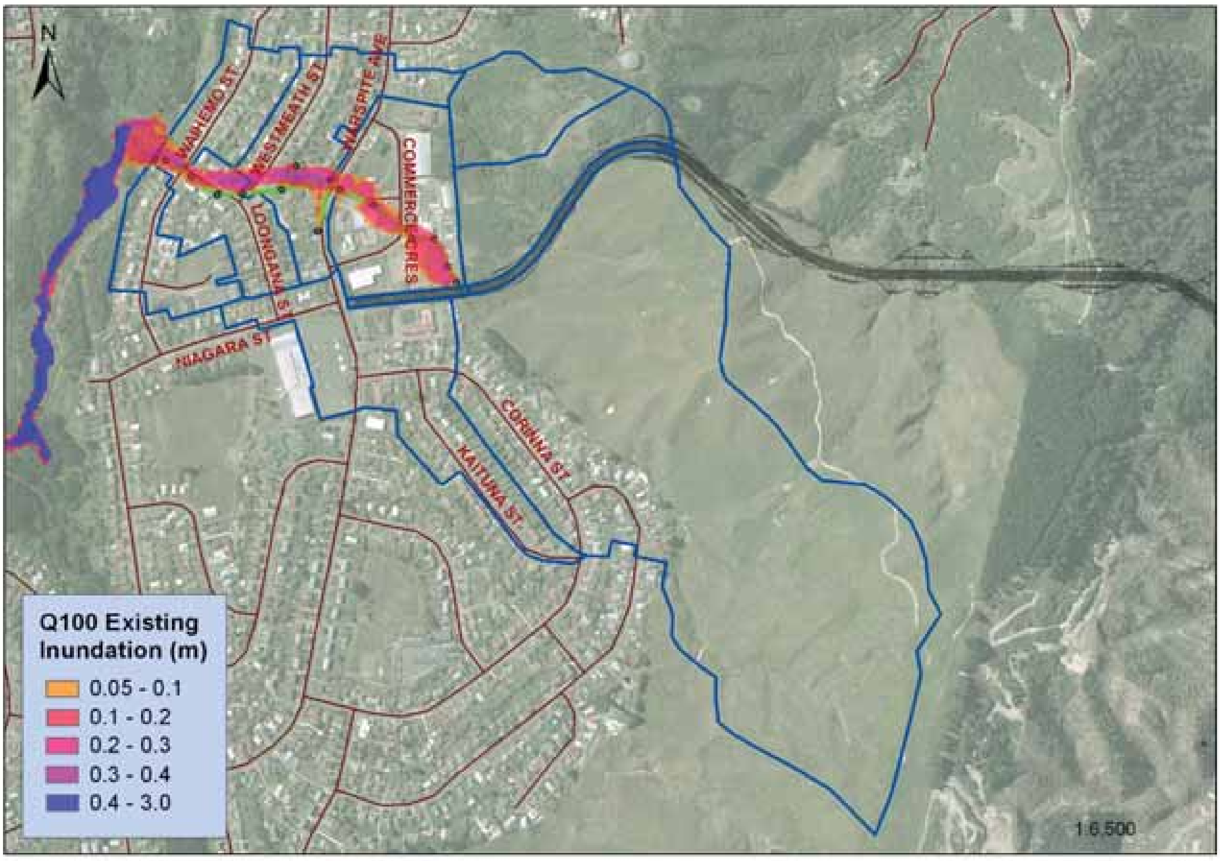
Modelling of the existing network has shown that overflows occur in the 10% AEP storm event as shown in **Figure 14.40**, and in the 1% AEP storm event as shown in **Figure 14.41**.



- **Figure 14.39 - Waitangirua Catchment Boundaries and Modelled Stormwater Network, pre the Transmission Gully Project**



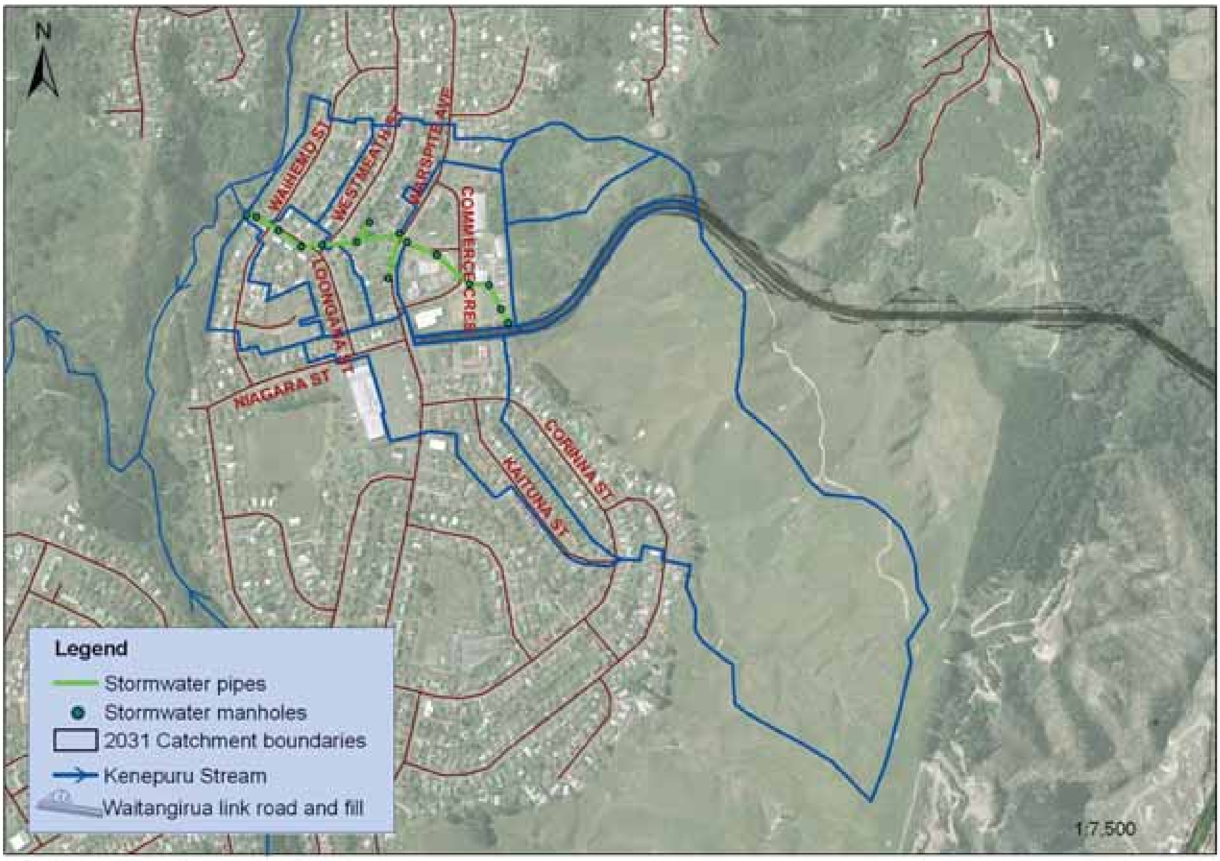
- **Figure 14.40 - Existing Surface Flooding in a 10% AEP flood.**



- **Figure 14.41 Existing Surface Flooding in a 1% AEP flood.**

6.6.2. Proposed Waitangirua Link Road

In the 2031 scenario the Waitangirua Link Road divides the rural catchment. Runoff from a section of the road is routed through the Waitangirua local stormwater network. The post construction catchment boundaries and stormwater network are displayed in **Figure 14.42**.

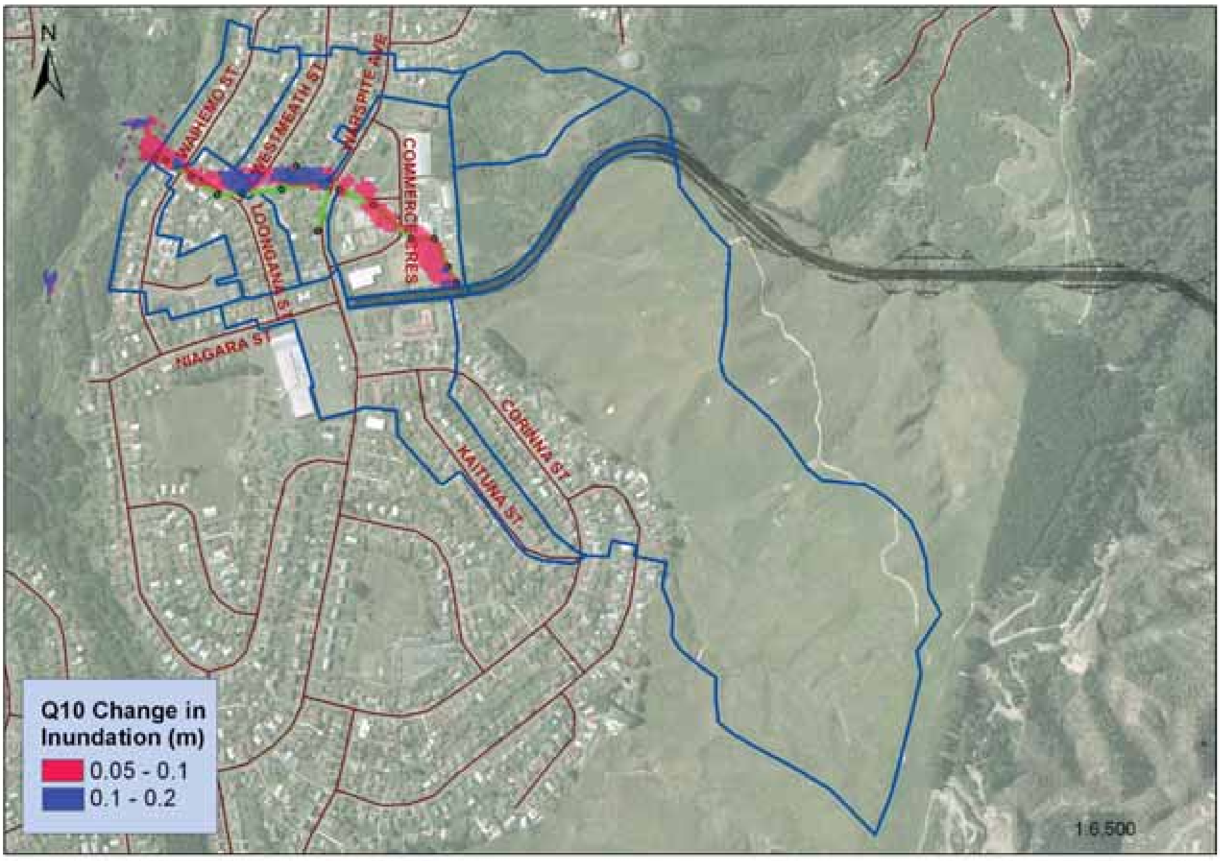


- **Figure 14.42 - Waitangirua catchment boundaries and modelled stormwater network (including a section of the Waitangirua Link Road, proposed to drain through the stormwater network).**

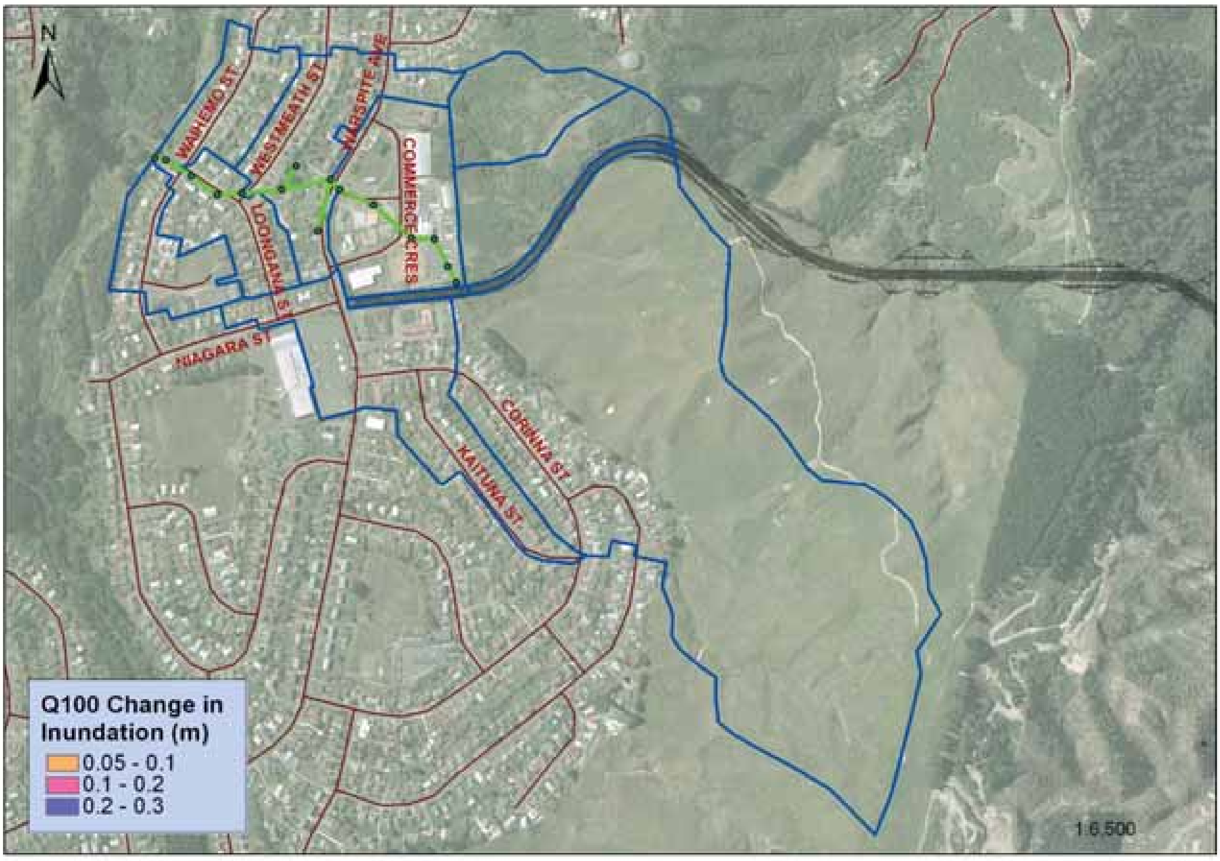
Inundation depths from the 2021 (pre Waitangirua Link Road, 2021 landuse) scenario have been subtracted from the 2031 (post-Link road, 2031 landuse) scenario to assess the difference. These are displayed in **Figure 14.43** and **Figure 14.44**. Changes in inundation of less than 50mm have not been shown.

Results from the stormwater model showed that the existing network is under capacity in both a 10% AEP event and the 1% AEP event. In a 10% AEP event, a peak of approximately 2.4m^3/s passes through the outlet, and a maximum flow of 1.8m^3/s surcharges at the top of the network. When additional runoff from the Link road is incorporated in analyses, flow through the outlet did not change, and surcharges increased slightly peaking at 1.9m^3/s.

In a 1% AEP event, a peak of approximately 2.9m^3/s passes through the outlet, yet a maximum surcharge of 4.4m^3/s occurs at the top of the network. When the additional runoff from the Waitangirua Link Road was incorporated in analyses, flow through the outlet did not change as the network is at capacity, however surcharges increased slightly with a maximum surcharge of 4.6m^3/s occurring at the top of the network.



- **Figure 14.43 - Change in Inundation in a 10% AEP Flood.**



- **Figure 14.44 - Change in Inundation in a 1% AEP Flood.**

Mapping the network surcharges (flooding) showed that in the 10% AEP event flooding occurs over the majority of the modelled network line, with a maximum change in inundation of 200mm. The increased flooding looks to be confined to the existing flood-prone areas. The most extensive flooding occurs in the low-lying residential properties between Warspite Road and Waihemo Street.

In the 1% AEP event the change in inundation is comparatively minimal at less than 50mm. This is because in the 1% AEP significant overflows occur in the existing case so the impact of increased flows is lessened. Once at capacity the network surcharges and results in surface flooding to the low-lying areas. In the 1% AEP the Waitangirua Link Road contributes runoff, however surface flooding is already occurring so the change in inundation is not significant.

6.6.3. Mitigation Options

To reduce the impact of flooding to the Waitangirua township several options have been considered:

1) Upgrade the existing stormwater network
2) Divert runoff to secondary overflow paths or adjacent stream channels.

Given the natural terrain of Waitangirua, secondary overflow paths are not recommended. The natural terrain ponds water in the residential area surrounding Westmeath Street. Overflow paths would potentially cause greater inundation to residential homes in this area, and to houses downstream of any surcharges.

6.6.4. Conclusion

The existing stormwater network beneath Waitangirua is undersized. The network does not have the capacity to convey a 10-year peak flow (including a mid-range climate change scenario). Pipe diameters currently vary from 525mm in the upper reaches to 750mm.

Porirua City Council is responsible for construction of the Waitangirua Link Road between Waitangirua/Whitby and the Main Alignment. When Council construct this road, it would be advisable that they upgrade the stormwater network beneath the urban area. Upgrading the network to convey a 1% AEP flood would eliminate any adverse effects from construction of the Waitangirua Link Road.

The upgrade of this network has been incorporated into the estimated project costs for the link road based on initial sizing.

7. Summary Assessment of Effects

The following provides a summary of the assessment of environmental effects and proposed mitigation from Sections 5 and 6.

7.1. Culvert Sizing

Culverts have all been sized to meet NZTA requirements and have considered design flows, velocities, outlet erosion control, upstream storage, maintenance requirements, and debris control and freeboard. There are no significant issues associated with the sizing of culverts due to the hydraulically steep terrain and standards can be complied with in all situations.

Culvert velocities have been assessed and matched to those culverts that require fish passage from the ecologists' assessment of the freshwater environment. In general, low flow velocities in culverts can be maintained at levels that allow fish passage for the recorded species. Where the drop from the upstream side of the proposed road to the floodplain is substantial and does not easily allow for ongoing fish passage, stepped erosion control structures are proposed that will provide fish passage opportunities. The ecology effects of cross culvert design is covered in the Technical Report 11: *Ecological Impact Assessment.*

7.2. Culvert Outlet Erosion Control

It is proposed to that culvert outlet erosion control is provided at all culvert outlets. Some innovative designs have been considered to manage specific project risks, in particular the cascade structures proposed in 5.4.6. It is considered that if erosion control is formalised as proposed the impacts will be no more than minor.

7.3. Stream Diversions

The Pauatahanui, Horokiri and Te Puka Streams all have major diversions associated with the construction of the new highway. In general, the hydraulic modelling has demonstrated that, to minimise the impacts on the hydraulic character of the streams, the diversions will need to be constructed to meet the following scheme design criteria:

- The existing channel shape and gradient should be duplicated as closely as possible
- Sufficient floodplain must be available to allow for flood flows to be conveyed without significant increases in velocities
- Where the diversions result in changes in length and gradient, the stream banks surrounding the diversions should be planted to help reduce increases in velocities in high flows.

The following summarises the result of each assessment.

Pauatahanui Stream

The assessment of hydraulic effects using the combined 1D-2D model identified that the proposed crossing of the stream will need to be a minimum of a 28m span bridge. A bridge of these dimensions will allow for the realignment of the stream to be constructed in a shape and gradient reflecting the existing channel as well as conveying the flows expected in a heavy rainfall. The interchange beneath the highway will need to allow for a secondary flowpath to convey extreme flood flows.

Comparison of pre and post-construction velocities through this area demonstrated that the hydraulics within the stream channel is similar in both scenarios. Therefore, if the scheme design is adhered to, the diversions can be constructed with no more than minor impacts on stream velocities or form.

Horokiri Stream

There are a number of major diversions that are likely to be required on the Horokiri Stream. The model was used to size the diversions to help avoid significant changes in stream hydraulic characteristics. The model results indicated that in general, the diversions should include a low flow channel meandering over a minimum 20m wide flood flow channel. The banks and floodplain surrounding the diversions should be landscaped and vegetated and the stream crossings sized to convey high flows without significant increases in channel velocity (minimum 16m, 28m and 30m wide structures at Bridges 3, 5 and 7 respectively). If these guidelines are followed, the crossings and diversions can be constructed with no more than minor impacts on stream velocities and stream form.

Te Puka Stream

The modelling of the diversion of approximately 200m of the Te Puka Stream upstream of the new culvert illustrated that in general the impacts were not significant. There is likely to be a localised increase in stream velocity within the diversion but this is not anticipated to increase scour, particularly if combined with riparian planting.

7.4. Flood Risk

A risk assessment determined six streams / networks for hydraulic modelling. The hydraulic models of these streams / networks were used to assess the potential flooding impacts associated with:

- Loss of storage on the floodplain due to earthworks
- Alteration of secondary flowpaths by the proposed road alignment
- Increased runoff associated with the change in land use
- Impacts of changes in stream alignment and shape.

All six streams / networks have also been the subject of SSEMPS. These contain further details of the mitigation and management measures provided at each location to avoid, mitigate or remedy any potential environmental effects.

The following summarises the results of each assessment.

Pauatahanui Stream

A major interchange is proposed on the Pauatahanui Floodplain. A range of options were tested to reduce the hydraulic impacts of the associated filling on the lower floodplain and the constraint caused by the stream crossing. The hydraulic model of the stream was used to size the crossing, locate overflow paths and allow for maintaining of the existing stream channel shape under the bridge.

The modelling indicated that there was likely to be an increase in water levels upstream on the new bridge during extreme flooding events such as the 100-year average recurrence interval flood event. These effects are localised to the area immediately upstream and can be managed to avoid increasing the flood risk to the existing infrastructure.

The model also identified that the low lying sections of the back yards of four properties in Joseph Banks Drive are also likely to experience increases in flood levels as a result of the construction of the new highway. The increase is more than minor but does not endanger any of the existing buildings and there are a range of options to mitigate these impacts. Considering the low consequence of the impact the affected owners should be consulted and agreement sought on the acceptability of the increase in flood risk. Alternatively localised protection works could be undertaken.

If the recommendations made by this report are incorporated into the highway design the adverse hydraulic effects can be largely eliminated and in some locations, such as the existing substation, there is a reduction in flood risk.

Horokiri Stream

The key potential impacts of the new highway in the Horokiri catchment include the changes in flooding levels associated with the new bridges on the main channel and the alteration of the stream channel in the vicinity of the new diversions. Based on the analysis of the model results, a number of recommendations have been documented in this report including the dimensions of the diversions, bridge widths and localised protection measures. These recommendations are considered to limit the hydraulic effects to localised impacts around the new highway that will be able to be remedied, or mitigated within the proposed designation.

Te Puka/Wainui

Limiting or mitigating the hydraulic effects of the new highway in the Te Puka / Wainui catchments will require careful design of new stream crossings and management of high stream velocities. The hydraulic model was used to identify the 'at risk' locations and resulted in recommendations for culvert sizes and scour protection. The modelling demonstrates that the new highway will not adversely impact flooding downstream of the proposed road.

Duck Creek

Both the Transmission Gully Main Alignment and the Waitangirua Link Road cross Duck Creek. The predicted 2% change in peak flows resulting from the construction of the Project is assessed to be minor. Mitigation can be provided by creating storage upstream of the Waitangirua Link Road crossing. Here the peak flows in a 100-year flood event can be restricted to below the pre-construction situation, reducing any potential effects to negligible levels.

Linden

In the Linden catchment stormwater runoff from the Project will enter the urban stormwater system. Modelling indicated that there is not capacity within the existing system in large (10% or 1% AEP) storm events to accept the additional runoff resulting from the Project. Several mitigation options have been considered, with attenuation of peak flows in the upper catchment the preferred option. With this implemented the effect of the Project on downstream flooding is assessed to be less than minor.

Waitangirua

The existing stormwater network beneath Waitangirua is undersized. The network does not have the capacity to convey a 10-year peak flow (including a mid-range climate change scenario) without the road. It is recommended that when the Porirua Link Roads are constructed the stormwater network is upgraded, which will remove any adverse effects from the construction of the Waitangirua Link Road.

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