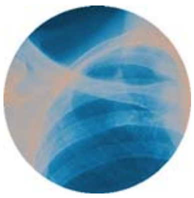
REPORT

STRUCTURAL AND CIVIL ENGINEERS



TRANSMISSION GULLY PROJECT

TECHNICAL REPORT NO.2:

DESIGN PHILOSOPHY BRIDGES

AND RETAINING WALLS

PREPARED FOR

NZ TRANSPORT AGENCY & PORIRUA CITY

COUNCIL

29 JULY 2011

REPORT

Transmission Gully Project
Technical Report No. 2: Design Philosophy Bridges and Retaining Walls

Prepared For:
NZ Transport Agency & Porirua City Council

| | |
|---|---|
| Date: | 29 July 2011 |
| HCG Project No: | 103457 |
| Revision No: | Final |

Prepared By:

Phil Gaby
PROJECT DIRECTOR

Reviewed By:

Rob Presland
PROJECT DIRECTOR

Holmes Consulting Group Limited
Wellington Office

CONTENTS

Page

FIGURES

## GENERAL

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.

## BRIDGES & RETAINING WALLS

The route includes 29 bridges crossing a variety of obstacles including streams, property access ways, State highways and local roads and at the southern end of the route and the North Island Main Trunk (NIMT) railway corridor.

Each of the bridges are described in the Bridge Schedule (see Section 5) and in drawings which are included in the plan set contained in Volume 4.

Major retaining walls and reinforced soil slopes feature along the route. These are located on the roading geometry drawings in Volume 4 plan set.

## SUMMARY OF GUIDING PRINCIPLES FOR DESIGN OF BRIDGES & RETAINING WALLS

A selection of the key 'Guiding Principles' for development of the bridge and retaining wall solutions are listed below:

- Bridge and retaining wall solutions are to be developed in conformance with the Transit New Zealand Bridge Manual (TNZBM) (Transit, 2003).

- Best value bridge solutions, with due consideration for whole of life performance, are recommended ahead cheapest conforming design options.

- The high risk of large earthquakes in the region influences the selection of structural types with robust structural forms with high levels of redundancy recommended for the bridges.

- Bridge substructures with fewer, larger piles ahead of a multiple smaller pile arrangement are recommended. The larger, more robust piles provide additional protection to the structures in the event of potential ground movements induced by earthquakes and by other environmental actions.

- Where ever possible, integral abutments and piers should be adopted for the bridges, eliminating maintenance intensive bearings and joints in addition to providing additional structural redundancy and robustness.

- Appropriate aesthetics are important and due consideration must therefore been given to this in the development of the structural concepts.

- Retaining wall solutions that are known to perform well in earthquakes should only be considered for the Transmission Gully Project.

## BRIDGE TYPES

The guiding principles above have led to the following bridge options to be proposed for the project:

- Spans up to 6m: Fully framed robust reinforced concrete box type structures. Six bridges fall into this structural category. In addition to providing a best value, low maintenance option, seismic performance of these types of bridges is expected to be excellent with little remedial work required after a major earthquake.

- Spans up to 35m: Precast beam and reinforced concrete slab, hollow core and super 'T' bridges. There are 15 no. precast beam and reinforced concrete slab bridges detailed as fully integral structures. These structures have cast in-situ concrete connections between the superstructure components (deck and beams), and substructure (piers and abutments), which provide very good resistance to earthquake forces and potential ground movements. The bridge solutions are cost efficient and low maintenance.

- Steel composite bridges are proposed for a span range of 35m – 60m. There are seven steel composite bridges located through the route. Base isolation with lead rubber bearings is recommended for the longer span steel bridges. This well proven approach can greatly enhance seismic performance of structures. The bridge type is cost efficient and with careful selection of coating systems, joints and bearing, can be relatively low maintenance.

- Bridge 20 (Cannons Creek Bridge), with a main span of 115m is scoped as a post tension concrete box balanced cantilever structure. With fully integral piers, the bridge's performance in a major earthquake is expected to be excellent. This form of construction is economic for spans in excess of 60m and is typically low maintenance.

## RETAINING WALLS

- Mechanically stabilised earth (MSE) walls have been selected for vertical retaining walls and reinforced soil slopes adopted for embankments detailed with 45º slopes. Both solutions are known to perform very well in large earthquakes.

# 1 BACKGROUND

## 1.1 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.

## 1.2 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 110km/hr respectively; and

- Maximum gradient of 8%;

- Crawler lanes in some steep gradient sections to account for the significant speed differences between heavy and light vehicles.

## 1.3 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 10%; and
- Limited side access.

## 1.4 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%; and
- Some side access will be permitted.

## 1.5 PURPOSE AND SCOPE OF THIS REPORT

The purpose of this report is:

- To outline the governing design principles behind the development of the bridges and retaining walls found along the Transmission Gully Main Alignment, the Kenepuru Link Road and the Porirua Link Roads.
- To present selected forms for sites where bridges are required throughout the route.

## 1.6 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 Transmission Gully 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 notices of requirement were lodged.

- In 1997, the hearing takes place for the notices of requirement 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 the Transmission Gully Project.

- In 2006, the WCP was endorsed by the Transit NZ Board and adopted by the Greater Wellington Regional Council and included the Transmission Gully Project 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 Project alignment both within and outside the confines of the existing designation. Together with a detailed consultation process, preferred alignment for Transmission Gully Project was produced.

- In 2009, detailed environmental and engineering investigation work commenced for the Project.

- 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 the Transmission Gully Project 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".

## 1.7 PROJECT DESCRIPTION

### 1.7.1 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.

### 1.7.2 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 designed to the following standards:

- Two lanes (one in each direction);
- Design speeds of 50 km/h;
- Maximum gradient of 10%; and
- Limited access only.

The Kenepuru Link Road will run under existing SH1 and will be bridged over the NIMT.

### 1.7.3 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%; and
- 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.

## 1.8 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; and
- 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.

## 2.1 TRANSMISSION GULLY BRIDGES AND RETAINING WALLS

The Transmission Gully Project scheme includes 30 bridges occupying some 1.7 km of highway.

Each of the bridges are described in the Bridge Schedule (Section 5) and in drawings which are included in the plan set contained in Volume 4.

In addition to bridges, major retaining walls (generally mechanically stabilised earth (MSE)) and reinforced soil embankments (RSE) are also anticipated along the route. These are located in plan on the roading geometry drawings in Volume 4.

## 2.2 INFLUENCES ON DESIGN

Bridge and retaining walls solutions have generally been developed in conformity with the structures design philosophy recorded below. The design philosophy has been influenced by a number of key factors including:

- Design standards
- Cost competitiveness with consideration for whole of life costing
- Regional network security requirements
- Functional requirements
- Geotechnical conditions
- Seismicity
- Environmental and social considerations
- Aesthetics
- Durability and maintenance

With due consideration to the above, a design philosophy with a holistic approach has evolved for Transmission Gully's many structures. This philosophy is recorded below.

### 2.2.1 Design Standards

Preliminary designs have been developed in conformance with the Transit New Zealand Bridge Manual (TNZBM) (Transit, 2003). HN-HO-72 live loading has been assumed in preliminary designs. Other relevant standards include:

- NZS 1170 part 5 – Seismic Loading Standard

- NZS 3101 - Concrete Structures Standard
- NZS 3404 – Steel Structures Standard
- AS 5100 – Bridge Design Code (Australia)

Consideration has also been given to the effects of the recently introduced high productivity motor vehicle (HPMV) weight allowances on the designs. The allowances will permit trucks of around 60 tonnes (current limit 44 tonnes) with heavier axial loads to use parts of the road network. Work carried out to-date however, indicates that the load effects of HPMV vehicles are similar to that of HN loading applied in accordance with the Bridge Manual. Little if any impact on the design of the Project's structures is therefore anticipated with the introduction of these vehicles.

### 2.2.2 Cost Competitiveness

Wherever possible structures have been avoided in preference to embankments due to the disproportionate cost of providing structures when compared to earthworks. Where structures cannot be avoided best value bridge solutions, with due consideration for whole of life performance, are proposed.

Broadly speaking this has resulted in reinforced concrete and/or prestressed concrete bridges being proposed for all bridges with shorter spans (spans up to approximately 35m). These are typically detailed using integral abutments and piers.

Steel composite bridges have become more cost competitive recently, particularly where taller bridge piers and spans between 35m and 60m are required. Steel bridge solutions have therefore been adopted for the longer span arrangements. Steel structures are considerably lighter than concrete equivalents resulting in smaller dead loads and correspondingly lower seismic forces, hence their good economy in the span range noted. More elegant and cost efficient substructures also result when compared with longer span concrete bridges.

The exception is the Cannons Creek Bridge (Br 20). This bridge is a post tensioned concrete box girder balanced cantilever structure with a central span of 115m. The span of the bridge probably exceeds the typical economic range for steel girders hence the adoption of a balanced cantilever form. Balanced cantilever construction typically provides good economy for bridges with spans over 60m. In the proposed bridge concept, each carriageway is carried by its own box girder. This enables efficient use of formwork with the same formwork been used to construct the bridge under one carriageway followed by the structure under the other carriageway.

### 2.2.3 Regional Network Security

A key objective of the Transmission Gully Project is to provide an alternative strategic link for Wellington that improves regional network security.

The most significant threat to route security is large earthquakes. This risk has influenced the selection of structural configurations in a number of ways. In particular, robust structural forms with high levels of redundancy have been adopted for the bridges wherever possible. Features of the various bridge forms used for the Project follows.

Culverts and the smaller span underpass structures are robust, fully framed reinforced concrete box type structures. Seismic performance of these types of bridges are expected to be excellent with little remedial work required after a major earthquake.

The Project's concrete bridges are typically integral structures. These structures have cast in-situ concrete connections between the superstructure components (deck and beams), and substructure (piers and abutments), which provide very good resistance to earthquake forces and potential ground movements.

In most instances, pier and abutment caps are cast fully continuous with pier columns and abutment piles to provide robustness and redundancy under seismic loads. Details generally apply to the substructure of both concrete and steel composite structures.

Base isolation with lead rubber bearings is recommended for the longer span steel bridges. This will well understood approach greatly enhances seismic performance of structures. Steel is very cost competitive in the span range noted.

Bridge substructures have adopted fewer, larger piles ahead of a multiple smaller pile arrangement. The larger, more robust piles provide additional protection to the structures in the event of potential ground movements induced by earthquakes and or by other environmental actions.

Mechanically stabilised earth (MSE) walls have been selected for vertical retaining walls and reinforced soil embankments (RSE) adopted for embankments detailed with 45° slopes. Both solutions are known to perform very well in large earthquakes.

Another risk to route security is flooding. This has been accounted for by sizing bridge water clearances (freeboard) and culverts for flows generated by a 100 year return period event required in the TNZBM (Table 2.1 & Section 2.3 Waterway Design). Due allowance for climate change has been included in waterway calculations.

### 2.2.4 Functional Requirements

Bridge widths are determined by the roadway geometric design. Clearances under the bridges follow the recommendations of Appendix A of the TNZBM. Bridges typically carry 4 to 6 lanes of traffic depending on their location along the route, in addition to shoulders, verges and central reserves.

Underpasses provide access for pedestrians and vehicles under the Main Alignment, often to land that would otherwise be severed by the new road. Where required, underpasses have been sized to accommodate forestry activities.

### 2.2.5 Geotechnical Conditions

Geotechnical conditions are reasonably well understood throughout the route although bridge specific investigations are still to be undertaken. The Geotechnical Report records ground conditions in general terms through the project route. Typically the ground profile consists of a thin soil layer overlaying colluvium and or highly weathered greywacke of varying depth. This is followed by closely jointed greywacke bed rock. Ground conditions in general suit bored piled foundations socketed into the bedrock for the larger bridges (spans in excess of 15m) with shallow spread foundations being appropriate for box culverts and underpasses.

Shallow undercutting of soils and replacement of these with compacted selected material is likely to be required at retaining wall, culvert and underpass sites.

### 2.2.6 Seismicity

Given the route's close proximity to active faults, a site specific seismic hazard study has been undertaken by the Institute of Geological and Nuclear Sciences (IGNS) (see 0 for IGNS's report) to estimate the design spectra for calculating the seismic loadings for bridges, retaining walls and embankments on this project. This is in accordance with the requirements of the TNZBM, draft amendment section titled 'Earthquake Resistance Design', June 2005. This document is yet to be formally adopted by NZTA but has been taken into account and used as the basis to the preliminary design of the Transmission Gully structures.

A major contributor to the seismic hazard on the route is the Ohariu fault and the Ohariu splinter fault located south of Wainui Saddle. The Ohariu fault runs along the Te Puka Stream valley floor and part way along the Horokiri Stream valley floor before veering to the south west, north of Battle Hill.

In addition to the Ohariu fault (recurrence between 1800 – 3450 yrs depending on location) other active earthquake sources in the region pose a significant hazard including the Wellington fault (700 yr recurrence), Wairarapa fault (1000 yr recurrence), Moonshine fault (5150 yr or greater recurrence depending on location), Shepherds Gully fault (3450 yr recurrence) and the Hikurangi Subduction zone (420 yr recurrence). Contributions to the seismic hazard from these faults are included in the derivation of the site specific hazard spectra. Influences of the seismic risk on design of structures are also covered in Section 2.2.3 above.

The spectra derived by IGNS are fairly similar to those contained in NZS1170 with generally no significant increases or reductions in seismic hazard identified. We understand however, that recent research and understanding of local seismology and seismic hazards will likely result in a revision to the NZS1170 hazard factor (Z) for structures in Wellington from 0.4 to 0.5. The net affect is likely to be an increase in seismic demand in Wellington. An increase in seismic demand is likely to result in higher construction costs.

Project seismic event return periods and performance criteria for the structures are in accordance with tables 2.1 and 5.1 of the June 2005 draft amendment. Bridge structures and retaining structures associated with bridges will be designed for a seismic return period of 2500 years at ULS. Serviceability limit state SLS 1 & SLS 2 return periods are generally 100 years and 500 years respectively for all of the Project's bridges and retaining walls.

Retaining walls not associated with bridges will in general be designed to withstand a seismic event with a return period of 1000 years. A displacement based design approach is considered appropriate for the design of all retaining walls (both MSE and RSE walls). Design displacements of around 150mm are considered appropriate in most instances.

A number of bridges and retaining walls are located in very close proximity to major faults along the project and structural forms chosen will help ensure that an appropriate level of functionality, as required by the Bridge Manual, remains in the event of fault rupture.

In general, site specific spectra with 5% damping in accordance with NZS1170 will be used for the seismic design. Damping values appropriate to chosen systems will be calculated as part of the analysis where mechanical damped devices (such as lead rubber bearings in the case of the projects larger steel bridges) are used.

### 2.2.7 Environmental Considerations

Environmental considerations have influenced the selection of bridge solutions in a number of ways as summarized below.

In some instances bridges have been included where culverts would have provided a satisfactory and cheaper engineering solution. This is because culverts would not provide an environmentally acceptable solution in terms of footprint, sediment movement, flow velocity, fish passage and/or aesthetic aspects.

Bridges 17, 18 & 19 are examples of bridges being used in lieu of culverts. These multi span steel composite structures cross over relatively small tributaries of Duck Creek. It was felt that the resultant loss of habitat, if culverts were adopted, would be unacceptable. In particular, encroachment of the embankment fill slopes on the tributaries, valleys and the main channel of Duck Creek would be substantial with a culverting solution. The environmental footprint of the bridges is much smaller, being limited to the width of the carriageway above. Natural waterways remain unaffected by the bridges providing additional environment benefits when compared with the culvert solution. Where culverts are detailed, fish passage requirements have been considered in the preliminary design.

Bridges 21 & 22 cross steep gullies that could have otherwise been filled. The bridges minimise the footprint of the road to the width of the carriageway above. Earth embankments would have a much wider footprint in the steep country found at these locations with fill slopes extending a long way down into the valleys below.

Adoption of a long span balanced cantilever bridge at Cannons Creek Bridge (Bridge 20) minimises construction effects compared to a shorter span steel composite bridge arrangement. The longer span structure has fewer piers thereby limiting the number of construction access tracks that will be required to enable the bridge piers to be built. The final ground level footprint of the structure is also reduced as a result of fewer pier locations.

### 2.2.8 Aesthetics

#### 2.2.8.1 General

Bridge aesthetics are covered in more detail in the Urban Design and Landscape & Visual sections of this study. Appropriate aesthetics are important in roading schemes and due consideration has therefore been given to this in the development of the structural concepts. In particular, clean structural lines and plain or regular pattern concrete finishes are the preferred approach for the bridges and vertical retaining walls. A theme driven approach to concrete finishes is not recommend and should be avoided for cost and aesthetic reasons. It must be noted however that the route is generally a rural highway with few vantage points for road users to view structures except at interchange locations.

#### 2.2.8.2 Substructures

In general, all bridges feature fewer, larger pier members over many smaller elements thereby reducing clutter on the landscape. The engineering advantages in adopting fewer, larger piers has been covered in earlier sections of this report.

Abutment treatments throughout the route typically include reinforced concrete piles and cap beams fronted by MSE walls. Consistent treatment of abutments provides a continuity of approach throughout the Main Alignment.

#### 2.2.8.3 Superstructures

Clean superstructure lines are achieved with care given to detailing. In the case of prestressed concrete beam bridge decks located at interchanges, parapets include a drop down skirt that covers the deck edges and associated construction joints (see Bridges 13, 14, 23, 24, 25 & 28 drawings).

Steel box girder bridge superstructures provide elegant uncluttered solutions for the highly visible interchange at Linden (Bridge 25).

Cannons Creek (Bridge 20) with its varying depth box girders and tall box piers fits well into the surrounding steep landscape with a minimum of clutter.

Edge protection to the bridges is typically either concrete TL4 or TL5 (TL4 shape with elliptical steel top rail) barriers.

### 2.2.9 Durability & Maintenance

#### 2.2.9.1 Integral Bridges

Where ever possible, integral abutments and piers have been adopted for the bridges, eliminating maintenance intensive bearings and joints. Other engineering benefits associated with adoption of integral construction have been noted in previous sections.

#### 2.2.9.2 Bridges with Joints and Bearings

With longer span bridges, where joints and bearings cannot be avoided, adequate provision for inspection and maintenance of bearings and joints has been provided.

#### 2.2.9.3 Retaining Walls

At bridge locations, mechanically stabilized earth retaining walls with galvanised steel straps and concrete facing panels have been provided in the design.

#### 2.2.9.4 Structural Steel

High durability, long lasting coating systems have been assumed in the costing of the structural steel options. Systems that are likely to provide up to 40 years to first maintenance are preferred. This will generally include either thermal metal spray coatings (either zinc or aluminium) further protected by a sealer coat or single coat inorganic zinc silicate coating systems.

The need for special care in the detailing of steel elements to avoid areas of ponding (corrosion traps) and poor accessibility will form part of the design brief for the next phase of the project.

#### 2.2.9.5 Concrete

Concrete elements in bridges and retaining walls will be designed in accordance with the 100 year design life requirements in NZS 3101:2006. Little, if any, maintenance of the concrete elements is anticipated over the life of structures as a result.

## 3.1 SUBSTRUCTURES

### 3.1.1 General

For reasons discussed above, bridges are supported on fewer, larger diameter piles rather than many smaller diameter piles. Wherever possible, integral abutments and piers are also proposed (see earlier sections).

The piers and their foundations will be constructed of reinforced concrete and will be detailed to provide high levels of ductility. Ductility will enable these components to absorb high earthquake induced forces from the superstructure and potential ground movements in the foundation soils, without risk of catastrophic failures that could lead to collapse.

Specific site geotechnical investigations at bridge locations will identify the potential for landslides and ground movements so that appropriate localised mitigation measures can be put in place and the structure designed with additional robustness to counter any unpredictable effects.

### 3.1.2 Types of Pile

Piled foundations will generally be bored piles. Large diameter bored piles are considered most appropriate, given:

- The good capacity that can generally be achieved in greywacke bedrock
- The ability to advance the pile through the widespread shear, crush and fault zones within the bedrock
- The need to socket the piles into shallow bedrock to achieve adequate lateral resistance for earthquake and other lateral loads
- The presence of coarse gravel, and possibly boulders, in the alluvium and colluvium that overlies bedrock, which could retard penetration of driven piles
- A small number of larger piles would be more robust in the steep-sloped terrain, with its landslide potential, particularly during earthquakes.

### 3.1.3 Permanent Casing

Bored piles will penetrate variable alluvium materials at Paekakariki, Battle Hill, SH58, Linden and Kenepuru. Groundwater levels are likely to be high, and it is considered that bored piles should generally be permanently cased with large diameter steel casings in the upper ground strata described above. Permanent casing ensures good quality construction by preventing contamination of the concrete in the piles with soil and water during installation of the piles. The piles should be uncased in bedrock to achieve good socket capacity, except where casings are required to be advanced through fault-disturbed materials within the bedrock.

#### 3.1.4 Allowance for Displacement of Embankment or Landslide Materials

Piled foundations should be sleeved down to bedrock level to allow for displacement of the surrounding landslide materials or fill, where lateral loads may otherwise exceed the pile capacity in the following situations:

Bridge foundations in landslides, where displacement of the landslide could generate excessive lateral pile loads and/or unacceptable displacements. This could be mitigated through the use of an oversize collar or casing or by a structure designed to deflect slope slip debris.

Bridge abutment seatings that are supported on piles placed though backfill soil behind mechanically stabilised earth or other types of abutment walls, should allow for movements in the wall backfill or bridge approach fills in earthquakes. Typically this is achieved by locating oversized concrete pipe sleeves around the piles in the back fill layers.

#### 3.1.5 Pile Lengths

Pile lengths allow for:

- Achieving adequate pile foundation socket and end bearing capacity in bedrock.
- On slopes, extending the piles of foundations for bridge piers sufficient to provide adequate lateral load capacity unaffected by the slope, and to found the pile at a depth below the influence of any slope instability from static, storm or earthquake conditions.
- The presence of crushed or sheared zones in bedrock, and lenses of weaker materials in alluvium and estuarine deposits where these are found.

## 3.2 BRIDGE SUPERSTRUCTURES

### 3.2.1 General

In summary, the following deck forms have been adopted for the project:

- Spans up to 6m - Reinforced concrete box culvert structures.
- Spans up to 25m- Generally hollow core prestressed concrete decks with the exception of BSN 26A which is a prestressed beam and slab bridge.
- Spans ranging from 25m to 35m - Super 'T' prestressed concrete girder decks.
- Spans ranging from 35m to 60m (with low horizontal curvature and low torsion) - Steel 'I' girders with composite reinforced concrete deck slabs.
- Spans ranging from 35m to 60m (with significant curvature and/or high torsion situations) – torsionally strong steel box girders with composite reinforced concrete deck slabs.

- Long Span Bridge (Bridge 20, Cannons Creek) - Post tensioned concrete balanced cantilever deck.

Illustrations and photographs of various deck arrangements adopted for this project are found in Figure 2-1 below.

### 3.2.2 Reinforced Concrete Box Underpasses and Culverts

Short span culverts and underpasses are economic, robust, fully framed reinforced concrete box type structures.

### 3.2.3 Hollow Core Superstructures

Hollow core prestressed concrete deck superstructures are a proven solution with excellent robustness and economy for spans up to a maximum of 25m. Wherever possible, these types of deck have being detailed as fully integral with piers and abutments.

### 3.2.4 Super 'T' Superstructures

For spans between 25m and 35m Super 'T' prestressed concrete girders with cast in situ top slab provide excellent economy and long term performance.

### 3.2.5 Steel Girders with Composite Reinforced Concrete Deck Slab

Seismic demand on foundations is a function of a structure's height as well as its structural mass. Because of this steel composite bridges, which are much lighter than concrete equivalents, are the preferred solution for the taller, longer span bridges on this project (pier heights in excess of 12m and spans between 35m to 60m). In the case of the taller bridges, savings in the cost of the substructures are significant due to reduced seismic demand compared with the much heavier concrete only alternatives. The typical outcome for steel composite over concrete superstructures is fewer piles and smaller pier columns.

### 3.2.6 Cannons Creek Bridge (Bridge 20)

Cannons Creek Bridge with a main span of 115m makes this bridge more suited to post tensioned concrete balanced cantilever construction.

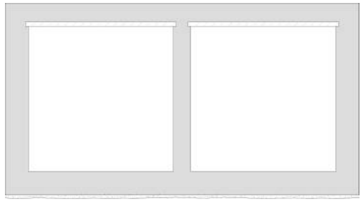
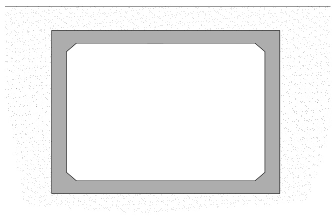
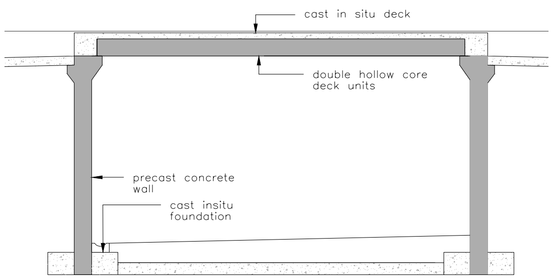
| Type | Notes | Detail | Illustration |
|---|---|---|---|
| Box Culverts | The larger box culverts through the route are sized to enable easy debris clearing by machine and, as a result, generally have considerable reserves of hydraulic capacity. The structures are likely to be constructed from either in-situ or precast concrete, or a combination of both. | <br>Twin Cell Box Culvert |  |
| Underpass Structures<br><br>(Typical clear spans ranging from 6m to 10.6m.) | Underpasses will provide access for vehicles, people and or stock under Transmission Gully in a number of locations along the route. The shorter span underpasses have been detailed as robust cast in-situ boxes. These underpasses could also be assembled from precast or part precast/ part in-situ elements.<br><br>Longer span underpasses are typically detailed with precast concrete walls with reinforced concrete foundations supporting hollow core deck units. The fully framed structural form provides an economic, durable and robust solution for the larger underpasses. Another viable option for the longer span underpasses include hollow core units supported on reinforced soil walls (see photograph opposite). | <br>Reinforced concrete underpass<br><br><br><br>Underpass with Hollow Core Deck on Precast Concrete Walls | <br>Underpass with Hollow Core Deck supported on MSE walls |

Figure 2-1: Bridge Types for Transmission Gully

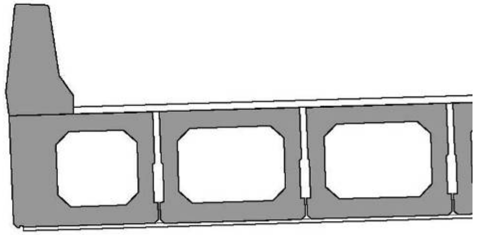
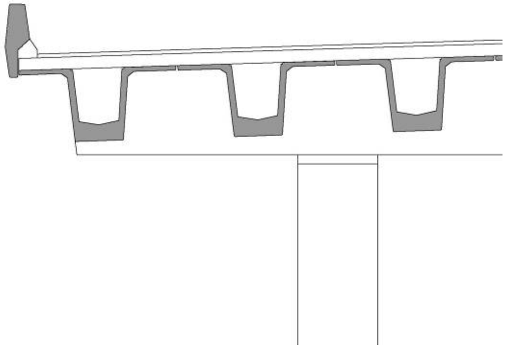
| Type | Notes | Detail | Illustration |
|---|---|---|---|
| Spans Up To 25m:<br><br>Prestressed Concrete Hollow Core Bridges<br><br>(650mm deep units - spans up to 18m; 900mm deep units - spans 18m to 25m.) | This economic, robust, durable and frequently used bridge solution is proposed where spans up to 25m are required. Deck units are typically supported on reinforced concrete cap beams founded on cast in-situ bored piles | <br>Hollow Core Bridge Deck |  |
| Spans Between 25m & 35m:<br><br>Prestressed Concrete Super 'T' Bridges<br><br>(1200mm deep units – spans up to 30m, 1500mm deep units – spans between 30m and 35m.) | Super 'T' bridges have in recent years become increasingly widely used for spans up to 35m. A growing number of contractors/precast manufacturers are able to make Super 'T' beams, ensuring competitive pricing. The beam flanges provide a safe working platform during construction of the deck slab. | <br>Super 'T' Bridge Deck |  |

Figure 2-1 (cont): Bridge Types for Transmission Gully

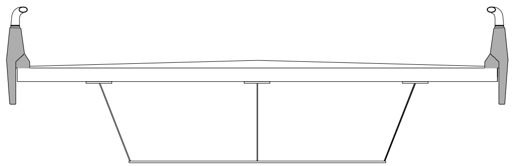
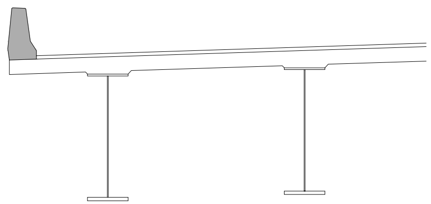
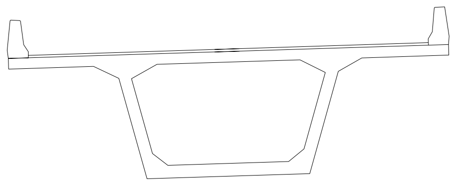
| Type | Notes | Detail | Illustration |
|---|---|---|---|
| Spans from 35m to 60m:<br>Steel Composite Bridges | Steel bridges provide cost effective solutions for longer-span, taller structures, and are suited for a number of crossings, particularly where steep topography and difficult access favours fewer piers and longer spans. This form is also appropriate in a number of locations south of SH58, where environmentally sensitive stream crossings in deeply incised valleys are required. Steel composite structures weigh around 60% of their concrete equivalents, making construction easier in difficult country with significant savings in substructure costs. Modern coating systems, such as hot metal zinc spray and single coat inorganic zinc silicate systems, provide highly-durable protection for extended periods (up to 40 years before first maintenance), resulting in a positive acceptance of steel structures locally. The adjacent cross-sections show two forms of steel bridge configuration. | <br>Steel box girder bridge<br><br>Steel 'I' girder bridge | <br>Steel box girder bridge<br><br>Steel 'I' girder bridge |
| Spans more than 60m:<br>Post-tensioned Box Structures | This form of bridge would suit the crossing of Cannons Creek (Bridge 20) which has a central span of around 115m. | <br>Post tensioned concrete box girder bridges |  |

Figure 2-1 (cont): Bridge Types for Transmission Gully

## 4.1 PHILOSOPHY

Walls will serve a number of purposes, including forming bridge abutment walls and supporting bridge approaches, as well as supporting embankments and cut slopes. They will also be used where the alignment results in the carriageway being partly on original ground.

Walls will be designed to meet the requirements of the TNZBM. A displacement-based design philosophy is proposed for the retaining structures, which is appropriate for areas of high seismicity. In this design approach the outward movements and settlements of walls in earthquakes and are limited to levels that do not cause serious damage to the highway formation or any structure supported by the wall.

## 4.2 TYPES OF WALLS

The two main types of retaining structure that have been adopted for this project are:

- Mechanically stabilised earth (MSE) walls for vertical walls.
- Reinforced soil embankments for embankments with slopes equal to or greater than 45º.

Benefits and disadvantages of these and other wall types are described in Figure 2-2 below. Details of the various wall solutions that could be used are found in Figure 2-3 below.

## 4.3 EARTHQUAKE PERFORMANCE

Only robust wall types with proven good performance in earthquakes will be used for this project.

Both MSE walls and reinforced soil embankments can be designed to undergo earthquake displacement and can be expected to perform well. Roads supported by these walls should remain serviceable after a major earthquake, although some surface cracking may occur, and repairs to facing panels may be required.

## 4.4 RETAINING WALLS SUPPORTING HIGHWAY ON STEEP SLOPES

In some locations, lengths of road will be supported by a retaining wall with a steep natural slope below. Walls would generally be up to 10m high above the slope surface, and up to 15m high in places. This form of wall is vulnerable to failure from instability of the slope below. A MSE wall is appropriate and there are two options for founding the wall:

- Support the outer edge of the wall with reinforced concrete bored piles, socketed into bedrock and taken down to a depth that would not be affected by potential soil failures. The bored piles may need to be anchored or tied back into the slope.

- Excavate into bedrock and found the reinforced soil wall at a level that would provide adequate foundation stability and protection from any instability of the slope below.

- A combination of the two options could be adopted.

Alternatively, a more cost-effective solution would be to replace sections of the wall with a 45° sloping reinforced soil embankment. This solution can be used where there is sufficient space available on the valley floor beneath the slope to accommodate the additional width of the inclined reinforced slope and where the slope does not encroach on stream waterways.

| No | Retaining Wall Type | Advantages | Disadvantages | Application for Transmission Gully |
|---|---|---|---|---|
| 1 | MSE Walls (also known as Reinforced Soil Walls) | Suitable for supporting fill with a vertical or semi-vertical face. Can accommodate a displacement-based design approach for high seismicity. Allow construction using on-site gravel from rock cuttings, and can be built in conjunction with earthworks operations. Able to accommodate some settlement on compressible ground. Cost-effective. | Require space to form reinforced soil block. | Suitable for bridge abutments. Suitable for supporting ramp fills at interchanges. Suitable for walls supporting highways on steep slopes, assuming they can be founded on stable level ground. |
| 2 | Soldier Piled Walls | Can be anchored to provide greater capacity as height increases. Require limited space and suitable to retain existing ground or cuttings. Can be used where ground conditions are poor to some depth. | Not readily suitable for displacement-based design, particularly when anchored. Time-consuming construction; costly. Not suited to retained heights greater than 7m, as will require costly and time-consuming multi-level anchors. | Suitable for ramps cut into existing ground. Possibly useful for supporting highway edges in steep ground. |
| 3 | Contiguous Bored Pile Walls | Can be anchored to provide greater capacity as height increases. Require limited space, and suitable to retain existing ground or cuttings. Can be used where ground conditions are poor to some depth. Can be used to support bridge abutments. | Not readily suitable for displacement-based design, particularly when anchored. Time-consuming construction; costly. Retained heights greater than 7 m will require costly and time-consuming multi-level anchors. | Suitable for supporting bridges in cuts at interchanges. |
| 4 | Soil Nail Walls | Suitable for walls supporting cuts in existing ground. Can accommodate a displacement-based design approach for high seismicity. Suited to excavation-support in steep ground areas. Enable top-down construction as excavation proceeds. | Require some space beyond/ below the cut face to install soil nails. Unsuitable for retaining fill. Slow construction. | Suitable for support of overburden at the top of cuttings. Suitable to support ramps cut into existing ground. |
| 5 | Crib Walls | Commonly used in Wellington Region. | Poor seismic performance, particularly for important applications. Require competent foundations. Unsuited to displacement-based design. | None |
| 6 | Concrete Cantilever Walls | Constructed using common materials. Can be ground-anchored to provide capacity. | Require a large space. Unsuited to displacement-based design approach, as need anchors for significant height. | None |

Figure 2-2: Retaining Wall Types & Applicability

| Type | Notes | Detail |
|---|---|---|
| Mechanically Stabilized Earth Retaining Walls & (Heights in excess of 15m achievable.) | These types of structures are one of the most widely used in New Zealand and overseas, and rely on metal straps or geotextile grid reinforcement embedded in the fill behind the concrete face panels to provide embankment stability. Walls are typically faced with concrete panels or concrete blocks.<br><br>Retaining walls will be required to support the approach ramp fill for interchanges. Reinforced soil walls are an appropriate and cost-effective solution. The walls can use geogrid reinforcement with a modular block facing. There are a variety of systems, but some have poor connections between the reinforcement and wall facing. It will be important to select systems that have a positive facing connection. Polyethylene grids in an alkaline environment (concrete blocks) are vulnerable to hydrolysis, and their use therefore needs to be carefully considered.<br><br>Abutment walls with heights in the range of 5m to 10m will be required. Reinforced soil walls are a cost-effective and visually attractive bridge abutment solution, and have been extensively used in the Wellington region. Reinforced earth walls with cruciform wall panels are predominantly used, e.g. at MacKays Crossing, SH2 Dowse-Petone.<br><br>The TNZBM requires the use of inextensible reinforcement for bridge abutments. Steel strips are appropriate, with selected granular fill within the reinforced earth block to minimise wall deformation and provide a positive connection to the wall face.<br>Normal Wellington practice is to found the abutment seat directly on the reinforced soil block, design bearings to accommodate some displacement, and to allow for adjustment after earthquake events. |  |

Figure 2-3: Retaining Wall Types Proposed for Transmission Gully

| Type | Notes | Detail |
|---|---|---|
| Reinforced Soil Slopes | Steeper embankments are achievable by introducing layers of geogrid between embankment fill layers. Cost savings are significant, and a more natural appearance can be achieved where space permits. Vertical retaining walls can be replaced with this form of construction.<br><br>This form of construction is proposed at the northern end of the route in lieu of vertical wall solutions. |  |
| Contiguous Pile Walls with Sprayed Concrete Jack Arches and Precast Concrete Facing Panels | Particularly suited to top-down construction, contiguous bored pile walls can be used to provide an abutment, as well as for the foundation of a bridge.<br><br>Piles are generally located along the wall at twice their diameter in spacing. The highway is excavated after the piles are constructed, with sprayed concrete applied between the piles. The walls can then be faced with pre-cast panel facings or shotcrete to provide an aesthetic finish.<br><br>Although not currently envisaged for Transmission Gully, this type of wall may feature in some locations as the project evolves through subsequent phases. |  |

Figure 2-3 (cont): Retaining Wall Types Proposed for Transmission Gully

| Type | Notes | Detail |
|---|---|---|
| Soil Nail Walls | These types of walls provide a cost-effective approach for stabilising cut faces in steep country, but their appearance can be less aesthetically-pleasing than other structural forms. A soil nailed wall may be an appropriate solution to support cut faces at interchanges, where space is available. The walls would be constructed top-down, with the soil nails installed as excavation progresses.<br>Some soil nailing is likely to be required through the route to stabilise localised areas of cuttings. |  |
| Soldier Pile Walls | Soldier pile walls can be used to support the cuttings, by forming piles followed by top-down excavation, with rock anchors installed as the excavation proceeds. A shotcrete facing, or possibly a pre-cast panel facing, can be used to provide an aesthetic finish. This type of wall is currently not detailed in the Transmission Gully design. As the design of the project evolves however, it is possible that this type of wall may be adopted in some locations along the route. |  |

Figure 2-3 (cont): Retaining Wall Types Proposed for Transmission Gully

# 5 STRUCTURES SCHEDULE

The figure below describes each bridge in the Project by type and size.

| Bridge Number | Chainage (m) | Obstacle Crossed | Bridge Type | Special features | Number of spans | Length | Width |
|---|---|---|---|---|---|---|---|
| 01 | 01800 | TG crosses local road (old SH1) | Hollow core deck underpass | Integral abutments | 1 | 11.8m | 110.40m |
| 02 | 01990 | TG crosses access road | Hollow core deck underpass | Integral abutments | 1 | 13m | 39.75m |
| 03 | 2730 | TG over Te Puka Stream | Steel 'I' girder bridge | Two separate bridge structures. One under N/B & S/B carriageways | N/B & S/B – 2 spans. | N/B 75.6m S/B 59.6m | N/B 13.5m S/B 13.5m |
| 04 | 08540 | TG crosses Horokiri Stream | Hollow core bridge | Integral abutments | 1 | 27.4m | 21.85m |
| 05 | 09300 | TG crosses access road | Reinforced concrete underpass | | 1 | 6.9m | 27.8m |
| 06 | 09720 | TG crosses Horokiri Stream | Super 'T' bridge | Integral abutments | 1 | 31.6m | 21.80m |
| 07 | 10500 | TG crosses access road | Reinforced concrete underpass | | 1 | 5.8m | 28.20m |
| 08 | 11750 | TG crosses Horokiri Stream | Hollow core bridge | Integral abutments & piers | 3 | 67.2m | 21.85m |

Figure 2-4: Bridge Schedule

| Bridge Number | Chainage (m) | Obstacle Crossed | Bridge Type | Special features | Number of spans | Length | Width |
|---|---|---|---|---|---|---|---|
| 09 | n/a | Access road crosses Horokiri Stream | Hollow core bridge | Integral abutments | 1 | 26m | 5.775m |
| 10 | 12600 | TG crosses access road | Reinforced concrete underpass | | 1 | 6.9m | 34.81m |
| 11 | 12840 | TG crosses access road | Reinforced concrete underpass | | 1 | 6.9m | 24.8m |
| 12 | 13965 | TG crosses access road | Reinforced concrete underpass | | 1 | 6.9m | 32m |
| 13 | 17460 | TG crosses SH58 | Hollow core bridge | Integral abutments | 1 | 22.2m | 21.85m |
| 14 | 17520 | TG crosses SH58 | Hollow core bridge | Integral abutments | 1 | 22.2m | 21.85m |
| 15 | 17690 | TG over Pauatahanui Stream | Super 'T' bridge | Integral abutments | 3 single span decks | 32m | 2 @ 10.5m<br>1 @ 21.80m |
| 16 | 19500 | TG over Porirua Link | Super 'T' bridge | Integral abutments | 1 | 27.6m | 24.3m |
| 17 | 21555 | TG over Duck Creek | Steel 'I' giver bridge | Base isolated bridge deck | 3 | 142m | 21.8m |
| 18 | 21860 | TG over Duck Creek | Steel 'I' giver bridge | Base isolated bridge deck | 4 | 147m | 21.8m |
| 19 | 22780 | TG over Duck Creek | Steel 'I' giver bridge | Base isolated bridge deck | 4 | 162m | 21.8m |

Figure 2-4 (cont): Bridge Schedule

| Bridge Number | Chainage (m) | Obstacle Crossed | Bridge Type | Special features | Number of spans | Length | Width |
|---|---|---|---|---|---|---|---|
| 20 | 23550 | TG over Cannons Creek Gully | Post tensioned concrete box bridge | Balanced cantilever bridge form. | 3 | 263.4m x 2 no. | 2 x 11m |
| 21 | 25795 | TG over stream & gully | Steel 'I' girder bridge | Two separate bridge structures. One under N/B & S/B carriageways | N/B – 3 spans. S/B – 2 spans | N/B 71.4m S/B 53.4m | N/B 13.5m S/B 11m |
| 22 | 26010 | TG over stream & gully | Steel 'I' girder bridge | Two separate bridge structures. One under N/B & S/B carriageways | N/B – 3 spans. S/B – 3 spans | 99.9m N/B & S/B | 13.5m N/B & S/B |
| 23 | 26660 | TG over Kenepuru Link interchange | Hollow core bridge | Integral abutments | 1 | 16m | 21.85m |
| 24 | 26720 | TG over Kenepuru Link interchange | Hollow core bridge | Integral abutments | 1 | 16m | 21.85m |
| 25 | 27015 | TG over local road (old SH1) | Steel box girder bridge | Base isolated bridge deck | 3 | 129m | varies 11m – 16.6m |
| 26 | 27510 | TG over Collins Ave | Hollow core deck underpass | Integral abutments | 1 | 18.6m | 36.25m |

Figure 2-4 (cont): Bridge Schedule

| Bridge Number | Chainage (m) | Obstacle Crossed | Bridge Type | Special features | Number of spans | Length | Width |
|---|---|---|---|---|---|---|---|
| 27 | N/A | Kenepuru Link under existing SH1 | Prestressed beam & slab on precast concrete walls | Day lighting of bridge deck at portals | 1 | 16.7m | 123m |
| 28 | N/A | Kenepuru Link over local NIMT railway | Super 'T' bridge | Integral piers. | 4 | 121.5m | 13m |
| 29 | n/a | Porirua Link (Waitangirua) over Duck Creek | Box culvert | | 1 | 5m | 55.7m |

Figure 2-4 (cont): Bridge Schedule

APPENDIX 2.A Estimation of Earthquake Spectra for Transmission Gully

# Estimation of Earthquake Spectra for Transmission Gully

G.H.McVerry

U. Destegul

GNS Science Consultancy Report 2008/92
May 2008

**CONFIDENTIAL**

This report has been prepared by the Institute of Geological and Nuclear Sciences Limited (GNS Science) exclusively for and under contract to Opus International Consultants Ltd. Unless otherwise agreed in writing, all liability of GNS Science to any other party other than Opus International Consultants Ltd in respect of the report is expressly excluded.

The data presented in this Report are available to GNS Science for other use from December 2007

# CONTENTS



# FIGURES

## TABLES



## APPENDICES

# EXECUTIVE SUMMARY

Site-specific acceleration response spectra for horizontal earthquake motions are presented for eight locations along the Transmission Gully route, for the site class specified by Opus for each location. The results are presented in terms of smoothed spectra for 5% damping. The spectra are presented for return periods of 250 years, 500 years, 1000 years and 2500 years. Comparisons are provided of the smoothed spectra with scenario spectra estimated for rupture of the Wellington-Hutt Valley fault segment, the south and central segments of the Ohariu Fault and the Moonshine Fault. Expressions are also provided for constructing smoothed spectra for the four site classes of NZS1170 for all eight locations.

The hazard estimates vary only moderately along the route for a given site class. Apart from the peak ground acceleration values, for which the range is close to 20% for the 2500-year values, other parameters generally vary by less than 10% across the sites. The 500-year peak ground accelerations for shallow soil site conditions range from 0.47g to 0.54g, and the 2500-year values from 0.78g to 0.94g. The highest hazard estimates occur at the Intermediate Interchange site, with similar values at Cannons Creek and the SH58 Interchange. These sites are the closest to the Wellington Fault, and also to the Moonshine Fault. The hazard estimates are generally higher towards the southern end of the route than towards the north.

The smoothed 2500-year spectra are sufficient to account for 50-percentile motions from all faults, and are at most marginally exceeded by 84-percentile motions from the Ohariu or Wellington Faults, and even then, generally over only short period bands. Motions at the 84-percentile level were not considered for the Moonshine Fault, because of its long estimated average recurrence interval of rupture of over 11,000 years.

The 1000-year spectra are exceeded by the 50-percentile scenario spectra for locations within about 2 km of the Ohariu or Moonshine Faults. The 1000-year spectra are never exceeded by the Wellington Fault 50-percentile motions.

The recommended smoothed hazard spectra are generally similar to those estimated using NZS1170.5:2004. The main differences are from different smoothed spectra shapes recommended in this study, to better match the location-specific spectra rather than to approximately envelop spectral shapes calculated for locations throughout New Zealand.

The NZS1170 Near-Fault Factors are recommended as appropriate for the Transmission Gully locations. These modifications to the hazard spectra for the Transmission Gully sites are modest for periods up to 3s period.

In NZS1170, Near-Fault factors are required for the Wellington Fault, but not the longer recurrence-interval Ohariu, Moonshine and Shepherd's Gully-Pukerua Bay faults. Near-fault effects estimated for scenario ruptures of the Ohariu and Moonshine Faults for the Transmission Gully sites that are close to these faults, and for the maximum near-fault effects at some sites from rupture of the Wellington-Hutt Valley segment, will be considerably larger than the NZS1170 Near-Fault Factors based on distances from the Wellington Fault.

# 1.0 PROJECT OUTLINE AND BRIEF

GNS Science was engaged by Opus International Consultants Ltd to perform site-specific hazard analyses for eight locations along the Transmission Gully route, as listed together with their site classes in Table 1. The route is shown in Figure 1.

The technical specification in the brief was as follows:

> "Seismic hazard analysis will be undertaken for Transmission Gully to develop 5% damped elastic acceleration response spectra for horizontal motions, smoothed appropriately for their use as design spectra. Spectra will be developed for each of the site classes of AS/NZS1170. The spectra will be provided for a range of return periods up to 2500 years and for spectral periods up to 3s. Magnitude-weighting will be incorporated for periods up to 0.5s, consistent with the development of the NZS1170.5 spectra. Near-fault factors appropriate for the location of Transmission Gully with respect to the Ohariu, Moonshine and Wellington Faults will be included. The results will be compared with the corresponding NZS1170.5 spectra, and with spectra estimated for the 50- and 84-percentile motions associated with rupture of the Ohariu and Moonshine Faults."

> "The calculations will be performed using the latest version of GNS's National Seismic Hazard Model (NSHM). The NSHM includes the contributions of major active faults, as well as taking account of smaller magnitude earthquakes used to model the historical earthquake catalogue. The NSHM was used to develop the hazard section of the New Zealand Standard NZS1170.5:2004 for earthquake loads in New Zealand, and for the December 2004 Provisional Amendment to Transit's Bridge Manual."

In addition, a map was requested to indicate the variation of hazard along the route.

**Table 1 Locations and Site Characterisation for Transmission Gully Route**

| Sector | Location | Station | Ground Conditions | Site Class |
|---|---|---|---|---|
| 1 | SH1 Crossing at Perkins | 2,150m | Dense sand and gravel, localised peat to ~25m depth | C (shallow soil) |
| 3 | Te Puka Stream Valley north of saddle | 4,600m | Bedrock overlain by up to 2m of sandy gravel | B (rock) |
| 5 | Battle Hill | 12,000m | Up to 30m + of sandy gravel overlying bedrock? | C (shallow soil) |
| 6 | Golf Course | 14,000m | Up to 8m to 22m of gravelly silt overlying bedrock | C (shallow soil) |
| 7 | SH 58 Interchange | 17,600m | About 25m of sandy gravel overlying bedrock | C (shallow soil) |
| 8 | Intermediate Interchange | 21,600m | ~10m of silty gravel overlying bedrock | C (shallow soil) |
| 9 | Cannon's Creek | 24,000m | ~10m of silty gravel overlying bedrock?? | C (shallow soil) |
| 9 | SH1 Interchange, Linden | | Bedrock overlain by up to 3m of silty gravel | B (rock) |

Note: Information supplied by P. Brabhaharan of Opus International Consultants Ltd

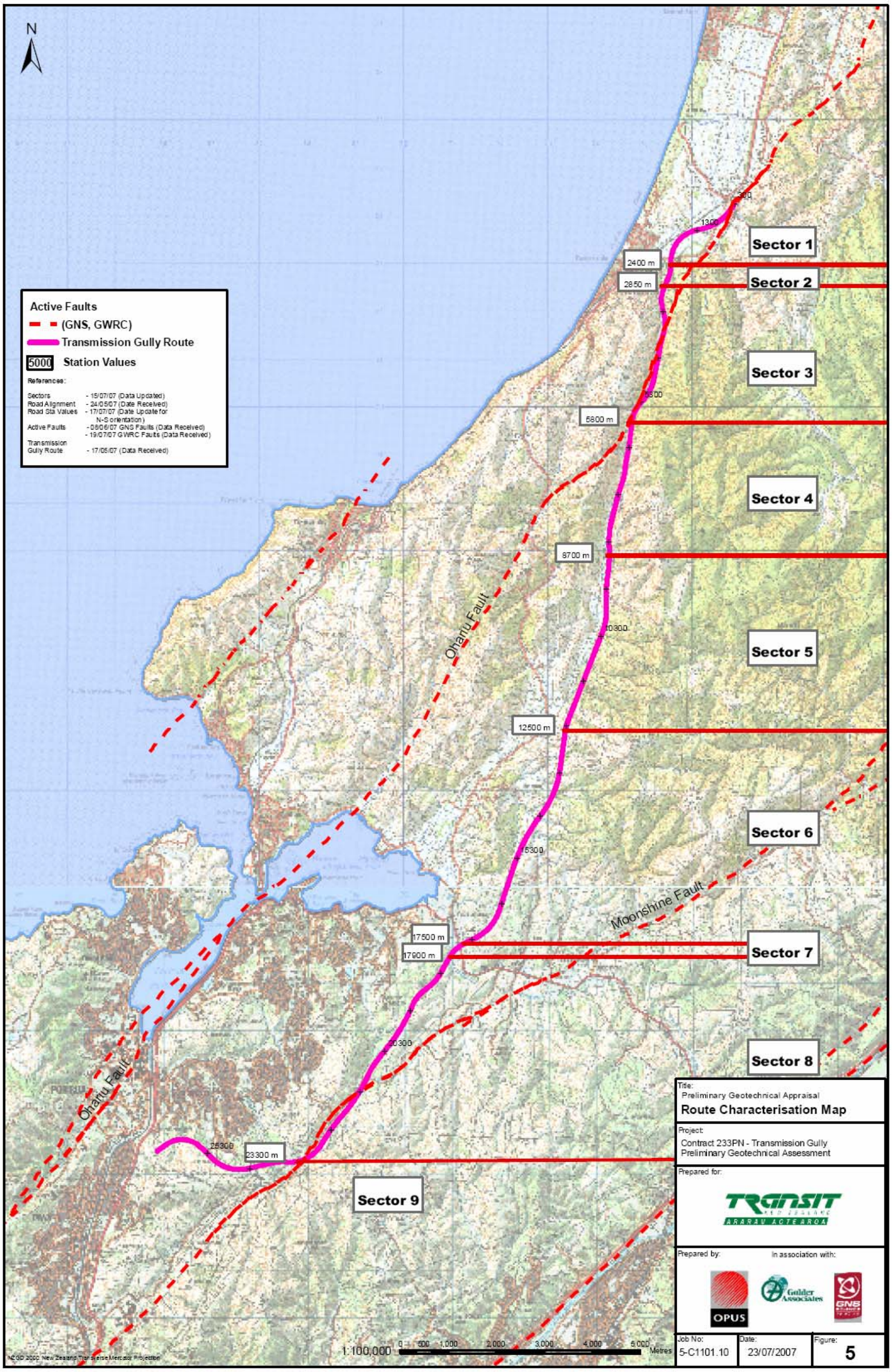



Figure 1 Station distances and Sector numbers along the Transmission Gully route (from P. Brabhaharan of Opus).

## 2.0 SEISMICITY AND FAULT MODELS

The brief for the study called for the calculations to be performed using the latest version of GNS Science's National Seismic Hazard Model (NSHM). The NSHM of Stirling et al. (2000, 2002), referred to in the remainder of this report as the 2000 model, was used to develop the hazard section of the New Zealand Standard NZS1170.5:2004 for earthquake loads in New Zealand, and for the December 2004 Provisional Amendment to Transit's Bridge Manual. The input data and methodology for deriving source parameters have been significantly updated since the 2000 model, both for the grid of point sources with parameters derived from the historical seismicity catalogue since 1840, and for the fault sources with parameters based largely on geological information. The hazard analysis for Transmission Gully used the recently developed 2007 version of the NSHM (Stirling & the Earthquake Hazards Team, 2007). The changes from the 2000 to the 2007 NSHM affected both the background seismicity model, as discussed in Section 2.1, and the modelling of the fault sources, as discussed in Section 2.2.

### 2.1 Modifications to the modelling of the distributed seismicity sources

The input data and methodology for characterizing the distributed seismicity sources have been significantly updated since the Stirling et al. (2000, 2002) model. The same overall approach is used, in that the b-value of the Gutenberg-Richter distribution $logN=a\text{-}bM$ ($N$=number of events $\geq$ magnitude $M$, and $a$ and $b$ are empirical constants of the Gutenberg & Richter relationship) is calculated initially for each seismotectonic region and subsequently smoothed horizontally across grid points, while the smoothed a-value is calculated directly at each grid point. The seismotectonic zones are the same as those used for the 2000 model, but the model now incorporates seismicity data past the 1997 cut-off year for the earlier model up to the end of 2005, and the a-value has been recalculated according to an adaptive kernel method (Stock & Smith, 2002). The adaptive kernel method allows the smoothing parameters for the a-value to vary according to the spatial distribution of seismicity, rather than simply using one set of parameters as in the 2000 model. The final a-value for each grid cell remains a maximum-likelihood estimate based on the various sub-catalogues identified in the New Zealand earthquake catalogue, a sub-catalogue being a space-time subset of the catalogue with a complete record above a specific magnitude threshold.

### 2.2 Recent update of fault sources

The second component of the seismicity model in the NSHM is that for the fault sources. In the main, the fault sources represent earthquakes that are modelled as being produced by faults with geologically-identified surface traces. In the NSHM, the fault sources are represented by planar segments, with a single fault source perhaps represented by several end-to-end planar surfaces to model changes in strike or dip along a fault. Each of these sources is assigned a characteristic magnitude and average recurrence interval, with each fault source modelled as producing earthquakes of only its characteristic magnitude, rather than a distribution of different magnitudes as given by the Gutenberg-Richter magnitude-frequency relation for the distributed seismicity grid points. Some long faults, such as the Alpine and Wellington Faults, are separated into several independent segments, each with its own characteristic magnitude and average recurrence interval.

In the 2000 NSHM, there was a hierarchy of methods used to assign the magnitudes and average recurrence intervals. In the updated NSHM, we use a single method for estimating the likely characteristic magnitude ($M_{max}$) and recurrence interval of $M_{max}$ earthquakes for each fault source. This method utilizes newly developed regression equations of $M_w$ on fault area for New Zealand earthquakes (Villamor et al. 2001; Berryman et al. 2002), with an internationally-based regression for plate boundary strike-slip faults (Hanks & Bakun 2002) used for the Alpine Fault.

### 2.2.1 Faults affecting the Transmission Gully Route

Table 2 lists parameters of fault sources in the Cook Strait-Wellington region that are relevant to the seismic hazard for the Transmission Gully route, indicating changes between the 2000 and 2007 models. The fault sources in the current model are shown in Figure 2.

The principal contributions to the estimated seismic hazard for Transmission Gully come from the Wellington-Hutt Valley segment of the Wellington Fault (ID 1 in Figure 2) at shortest distances of between 6 km and16 km from various locations on the Transmission Gully route, the central (ID5) and south (ID6) segments of the Ohariu Fault at distances between 0 km and 5 km and the Wairarapa 1855-Nicholson source (ID 2) at distances of 21-27 km. The Moonshine Fault at distances between 0.1 km and 11 km could also give rise to very strong motions, but its recurrence interval of over 11,000 years means that it contributes little to the exceedance rates of motions with return periods of up to 2500 years that are considered in the probabilistic studies considered in this report.

The parameters of these faults have changed since the 2000 model (Table 2). The magnitude estimated for the Wellington-Hutt Valley fault segment has increased from 7.3 to 7.6, accompanied by a slight increase in recurrence interval from 600 years to 700 years. The modelled average recurrence interval for rupture of the Ohariu Fault has decreased from 3250 years to 1800 years for the central segment and 2300 years for the south segment. The estimated recurrence interval of rupture of the Wairarapa 1855-Nicholson Fault has reduced from 1500 years to 1000 years. The Moonshine-Otaki Fault of the 2000 model with a very long recurrence interval of 125,000 years for magnitude 7.2 earthquakes has been separated into the Moonshine and Akatarawa-Otaki Faults, with recurrence intervals of about 11,000 years for characteristic magnitude 7.2 earthquakes and about 5000 years for characteristic magnitude 7.4 earthquakes respectively.

Major changes occurred in the fault modelling in Cook Strait area between the 2000 and 2007 models, but these faults are generally sufficiently distant from Transmission Gully that they have little effect on its estimated hazard because of the strong influence of the closer and more active Wellington Fault. Of the modelling of the Cook Strait faults, only the changes to the Wairarapa Fault, and the offshore segment of the Wairau Fault to a minor extent, have any effect on the hazard estimates for Transmission Gully.

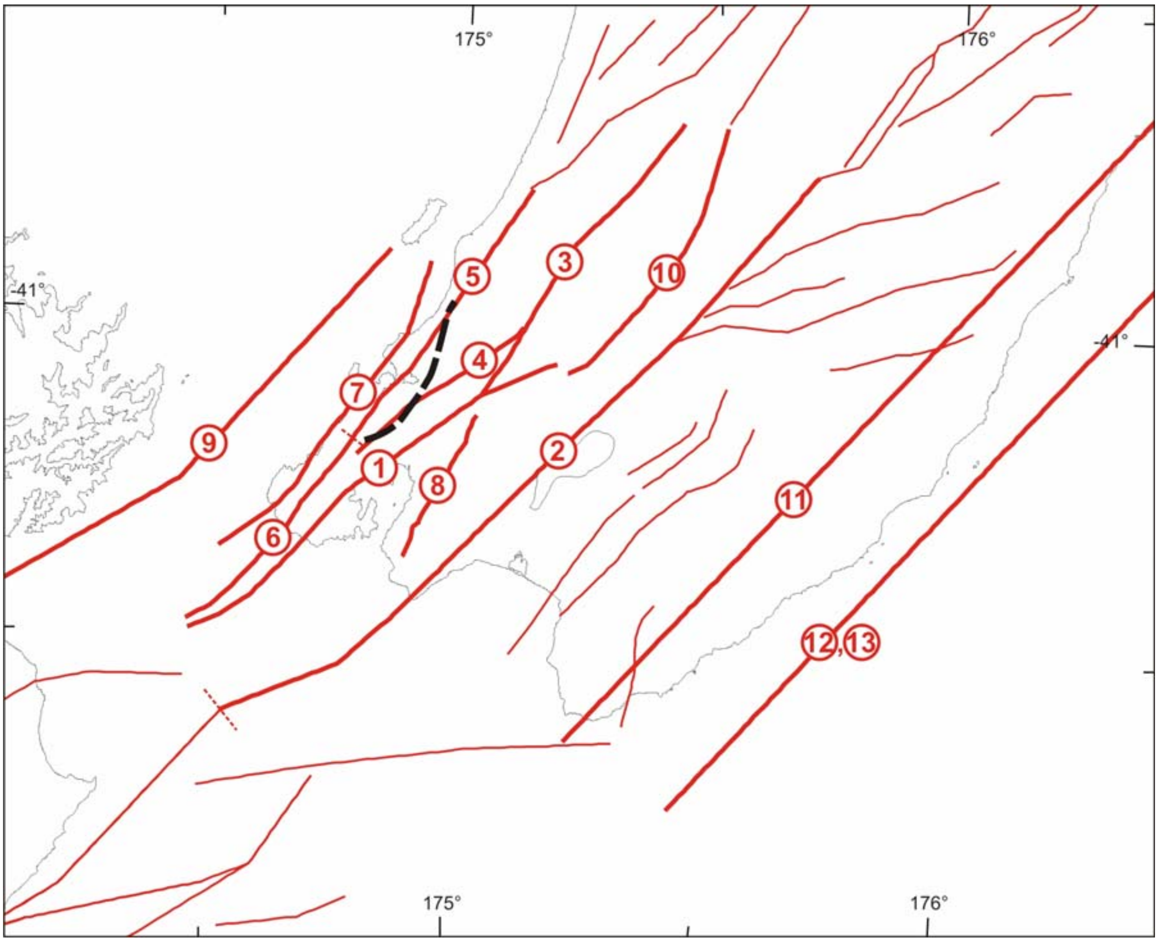



Figure 2 The approximate route of Transmission Gully (bold dashed line) and the faults listed in Table 2. Numbers correspond to the IDs in Table 1. Unnumbered faults make insignificant contributions to the estimated hazard for Transmission Gully.

**Table 2** Fault sources affecting Transmission Gully, comparing the 2000 and 2007 models

| Fault Source | ID | [1]Slip Type | 2000 Model | | 2007 Model | | Range of shortest distances to Transmission Gully sites (km) |
|---|---|---|---|---|---|---|---|
| | | | Magnitude | Recurrence Interval (yrs) | Magnitude | Recurrence Interval (yrs) | |
| Wellington-Hutt Valley | 1 | ss | 7.3 | 600 | 7.6 | 700 | 6-16 |
| Wairarapa 1855-Nicholson | 2 | sr | 8.1 | 1500 | 8.1 | 1000 | 21-27 |
| Akatarawa-Otaki | 3 | rv | Segments Moonshine-Otaki | | 7.4 | 5150 | 10-17 |
| Moonshine | 4 | rv | | | 7.1 | 11,150 | 0.1-11 |
| Moonshine-Otaki | 3& 4 | | 7.2 | 125,000 | Separated into Moonshine and Akatarawa-Otaki segments | | |
| Central Ohariu | 5 | ss | 7.4 | 3250 | 7.2 | 1800 | 0-4.4 |
| Ohariu South | 6 | ss | 7.4 | 3250 | 7.4 | 2300 | 1.5-19 |
| Pukerua-Shepherds Gully | 7 | ss | 7.2 | 3750 | 7.4 | 3450 | 4.9-8 |
| Whitemans | 8 | rv | Combines Moores and Whitemans | | 7.1 | 17,350 | 8-17 |
| Moores | | rv | 6.7 | 20,000 | Combined in new Whitemans | | |
| Whitemans | | rv | 6.4 | 20,000 | | | |
| Wairau Offshore | 9 | ss | 7.3 | 1650 | 7.5 | 1900 | 14-21 |
| Wellington-Tararua East | 10 | ss | Combines Wellington Central and West | | 7.3 | 650 | 20-32 |
| Wellington Central | | ss | 7.2 | 1200 | See Wellington-Tararua East | | |
| Wellington West | | ss | 7.2 | 1200 | | | |
| Subduction interface model unchanged | | | | | | | |
| Model 1 (restricted width) | 11 | if | 7.8 | 450 (0.25 weight) | 7.8 | 450 (0.25 weight) | 26-31 |
| Model 2 | 12 | if | 8.1 | 420 (0.25 wt) | 8.1 | 420 (0.25 wt) | 22-23 |
| Model 3 | 13 | if | 8.4 | 1200 (0.5 wt) | 8.4 | 1200 (0.5 wt) | 22-23 |

Notes:

1 ss=strike-slip
sr=strikeslip/reverse
rv=reverse
if=interface

**Table 3** Distances of sites from the most important faults

| | Fault Distances (km) | | | | | | |
|---|---|---|---|---|---|---|---|
| | Wellington-Hutt Valley | Central Ohariu | Ohariu South | Moonshine | Wairarapa | Pukerua-Shepherds Gully | Subduction Interface |
| SH1 Perkins | 16 | 1.0 | 19 | 11 | 27 | 5 | 23 |
| Te Puka Stream | 14 | 0 | 17 | 10 | 26 | 5 | 23 |
| Battle Hill | 10 | 2.3 | 11 | 4.6 | 23 | 6 | 23 |
| Golf Course | 8 | 2.9 | 9 | 3.3 | 22 | 7 | 23 |
| SH58 Interchange | 7 | 3.6 | 7 | 1.5 | 21 | 8 | 22 |
| Intermediate Interchange | 6 | 4.4 | 4.8 | 0.1 | 21 | 8 | 22 |
| Cannon’s Creek | 7 | 4.0 | 3.9 | 0.3 | 21 | 7 | 22 |
| SH1 Linden Interchange | 6 | 2.8 | 1.5 | 2.2 | 23 | 4.9 | 23 |

## 3.0 HAZARD ESTIMATES

Hazard estimates have been performed for each of the locations, for the site class indicated in Table 1. For the two rock sites, Te Puka Stream and Linden SH1 interchange, estimates have also been performed for shallow soil site conditions, to allow comparison of results for uniform site conditions.

### 3.1 Variation of Hazard Estimates along the Transmission Gully Route

Variation of the hazard along the route has been reported in terms of several earthquake ground-motion parameters: magnitude-weighted peak ground acceleration for shallow soil; the equivalent of the hazard factor Z of NZS1170.5; and the 5% damped response spectral acceleration at 1.5s period, SA(1.5s) for shallow soil. These parameters have been reported for return periods of 500 years, 1000 years and 2500 years.

The NZS1170 hazard factor Z is defined as half of the 500-year acceleration response spectral value at 0.5s period and 5% damping for shallow soil site conditions

$Z = 0.5\ SA_{shallow\_soil\_500\_yrs}(0.5s)$

The results provided in this report also include two parameters denoted as $Z_{1000}$ and $Z_{2500}$. These are defined in a similar manner to Z, but in terms of 1000-year and 2500-year motions. In NZS1170, these values are approximated by the product RZ, where R=1.3 for a return period of 1000 years and R=1.8 for a return period of 2500 years.

The hazard estimates vary only moderately along the route for a given site class. This is demonstrated in Table 4, which gives the maximum and minimum values of several hazard parameters for the eight locations considered along the Transmission Gully route. Apart from the peak ground acceleration values, for which the range is close to 20% for the 2500-year values, other parameters generally vary by about 10% across the sites. The highest hazard estimates occur at the Intermediate Interchange site, with similar values at Cannons Creek and the SH58 Interchange. These sites are the closest to the Wellington Fault, and also to the Moonshine Fault. The hazard estimates are generally higher towards the southern end of the route than towards the north.

**Table 4** Minimum and Maximum Hazard Values along the Transmission Gully Route

| Hazard parameter | Minimum value (g) | Maximum value (g) |
|---|---|---|
| $PGA_{shallow\ soil\ 500yrs}$ (mag. wt) | 0.47 | 0.54 |
| $PGA_{shallow\ soil\ 2500yrs}$ (mag. wt) | 0.78 | 0.94 |
| Z | 0.45 | 0.49 |
| $Z_{1000}$ | 0.57 | 0.63 |
| $Z_{2500}$ | 0.72 | 0.81 |
| $SA(1.5s)_{500yrs}$ | 0.30 | 0.33 |
| $SA(1.5s)_{2500yrs}$ | 0.53 | 0.60 |

These ranges are demonstrated in Figures 3 to 6. Figure 3 shows the variation of the 500-year and 2500-year magnitude-weighted peak ground accelerations along the route, for the site class identified for each site in Table 1. The peak ground acceleration is strongly dependent on the site class, so Figure 4 compares the peak ground accelerations at each location for the same site conditions, Site Class C Shallow Soil. Figure 5 shows the Z, $Z_{1000}$ and $Z_{2500}$ values. Figure 6 is for the SA(1.5s) values. An unusual feature of the attenuation model developed for New Zealand is that the rock and shallow soil values are almost identical at 1.5s period.

Values from four scenario events are also plotted in Figures 3 and 4, namely a magnitude 7.6 earthquake on the Wellington-Hutt Valley fault segment (recurrence interval 700 years), the larger motion from a magnitude 7.2 earthquake on the Ohariu Central fault segment (RI 1800 years) or a magnitude 7.4 earthquake on the Ohariu South fault segment (RI 2300 years), and a magnitude 7.1 earthquake on the Moonshine Fault (RI 11,000 years). Motions for the Wellington and Ohariu Fault events are shown at the 50- and 84-percentile levels. Those for the Moonshine Fault are shown only for the 50-percentile level because of its long recurrence interval. Also shown on this plot are the shallow soil peak ground acceleration of 0.96g that corresponds to the Wellington region hazard factor Z=0.4 and the 2500-year return period factor R=1.8, and the value of 0.93g corresponding to the maximum required RZ factor of 0.7 in NZS1170.5. The 2500-year peak ground accelerations estimated in this study slightly exceed the value corresponding to the RZ limit at one of the eight locations, Cannon’s Creek, and are close to it at the SH58 and Intermediate Interchanges.

Figure 5 indicates that the 2500-year estimates of the 5% damped response spectral accelerations at 0.5s period exceed the value corresponding to the maximum required design limit of RZ=0.7 in NZS1170 along the entire length of the Transmission Gully route.

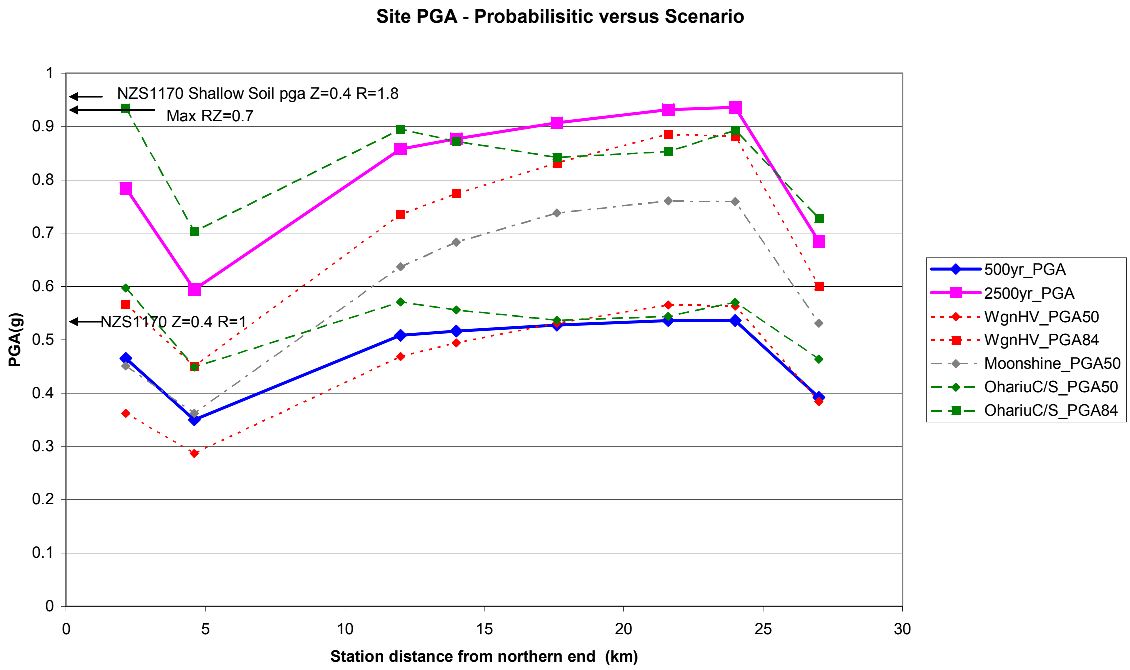



Figure 3 Variation of magnitude-weighted peak ground accelerations along the Transmission Gully Route. The lower values at distances of 4.6 km and 27 km are for the two rock sites, Te Puka Stream and Linden SH1 interchange. Otherwise there is a steady trend of motions increasing towards the south, as distances of the Wellington Fault reduce.

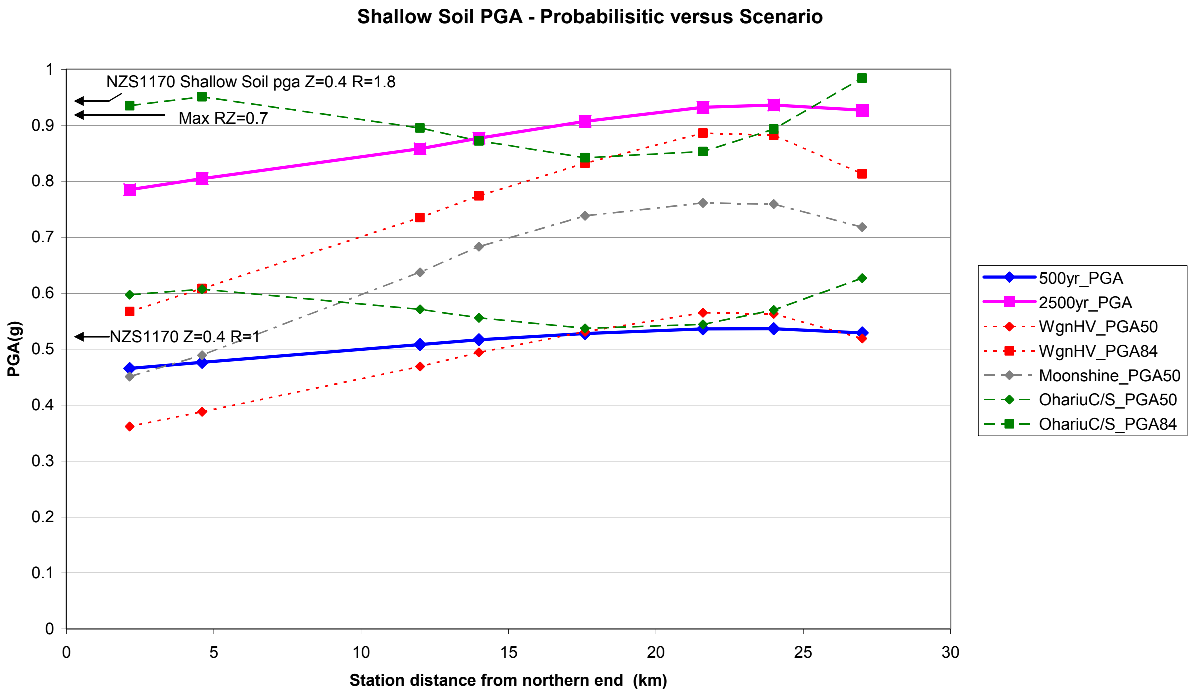



Figure 4 Variation of magnitude-weighted peak ground accelerations along the Transmission Gully Route for Class C Shallow Soil site conditions at all locations. The 500-year and 2500-year pgas generally increase towards the south, as distances of the Wellington Fault. reduce

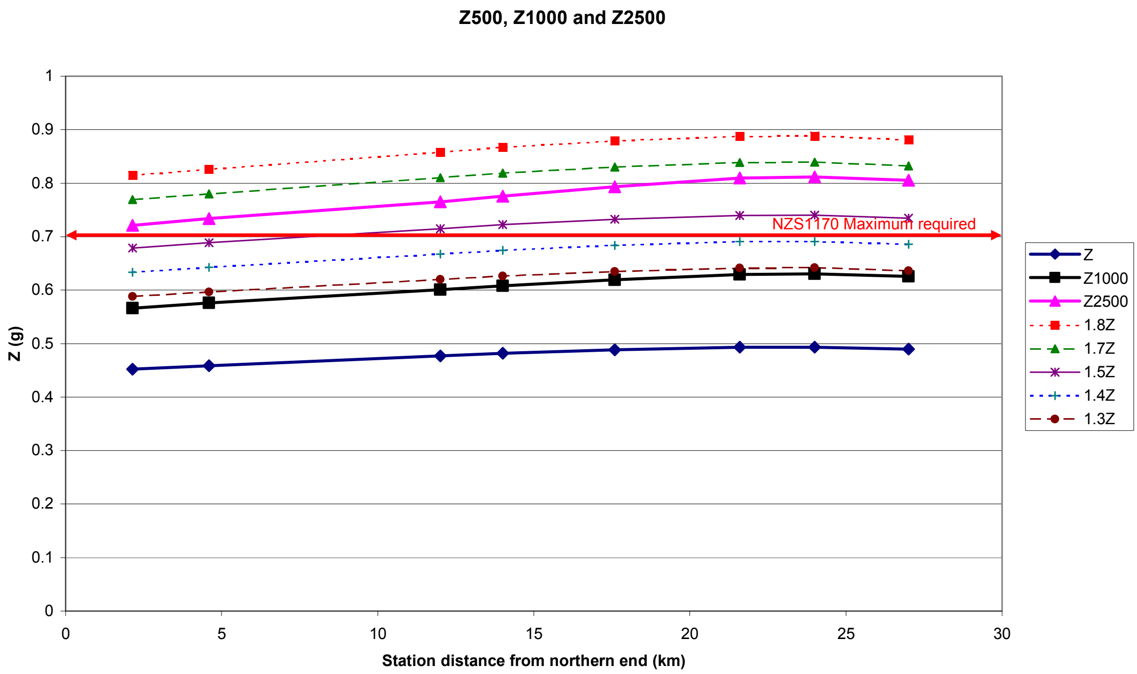



Figure 5 Z, $Z_{1000}$ and $Z_{2500}$ estimated along the Transmission Gully route. $Z_{1000}$ and $Z_{2500}$ are both less than their NZS1170 approximations of 1.3Z and 1.8Z. $Z_{2500}$ also exceeds the maximum ZR requirement of 0.7 in NZS1170. There is a moderate increase in the values from north to south along the route.

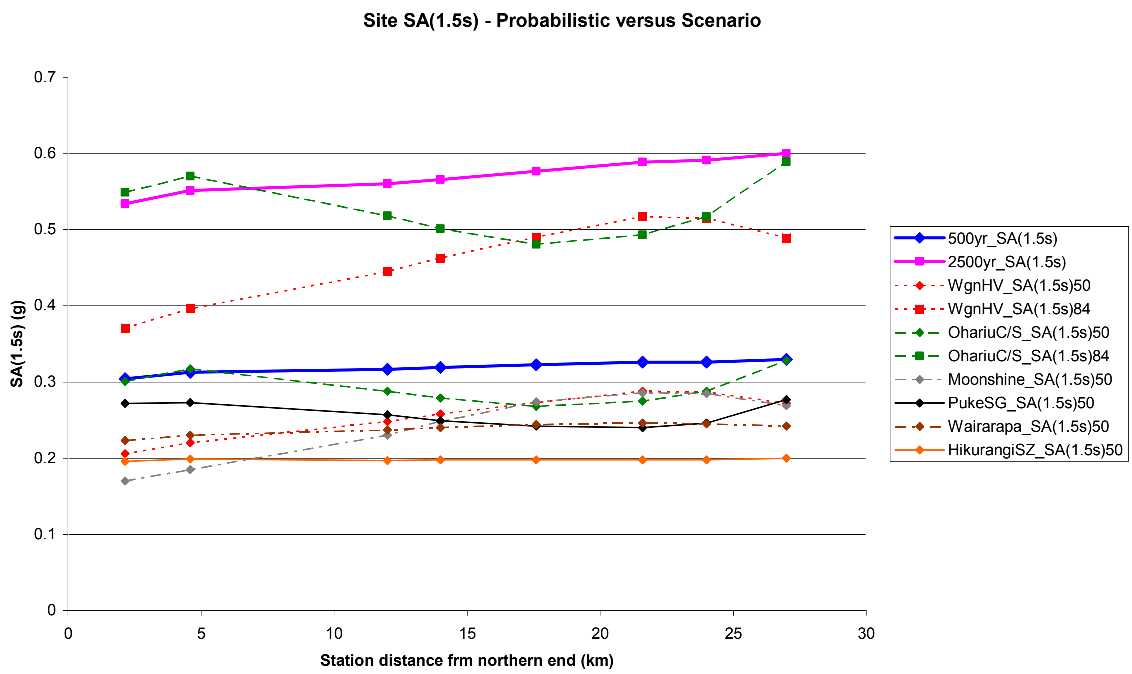



Figure 6 Variation of 1.5s spectral accelerations SA(1.5s) for 5% damping along the Transmission Gully Route. In the New Zealand attenuation model, SA(1.5s) is almost identical for rock and shallow soil sites.

Additional scenarios are included for the SA(1.5s) values in Figure 6. Large magnitudes make relatively stronger contributions at longer spectral periods than for peak ground accelerations, so other faults contribute more to the estimated exceedance rates than for the pga motions. The additional scenarios added in Figure 6 are 50-percentile motions for a magnitude 7.4 earthquake on the Pukerua-Shepherds Gully Fault (RI 3450 years), for a magnitude 8.1 Wairarapa 1855_Nicholson Fault scenario (RI 1000 years) and for a magnitude 8.4 Hikurangi subduction Zone scenario (RI 2400 years).

An observation from Figures 3 and 4 is that the 500-year and 2500-year peak ground accelerations along the route generally lie between the values estimated for the Wellington-Hutt Valley and the stronger of the Ohariu Fault scenarios, at the 50-percentile level from the 500-year motions and at the 84- percentile level for the 2500-year motions. Figure 7 shows that the 500-year and 2500-year pgas are in fact approximated very closely by the average and 1.7 times the average of the Wellington-Hutt Valley and Ohariu Central 50-percentile motions.

The SA(1.5s) hazard estimates also correlate well with the average of the two scenario motions (Figure 8). At 1.5s period, other faults make a relatively stronger contribution to the hazard than for the pga values. A factor of 1.2 is required to raise the average of the 50-percentile motions to the 500-year SA(1.5s) values. Scaling by an additional factor of 1.7 leads to reasonable approximation to the 2500-year SA(1.5s) values, but the fit is not as good as for the other three cases.

The PGA and SA(1.5s) values for the return periods of interest are the parameters used later in this report to construct smoothed hazard spectra, in place of the Hazard Factor Z and Return Period Factors R that are used in NZS1170. The Appendix contains maps of these parameters for return periods of 500 years and 2500 years (Figures A17 to A19).

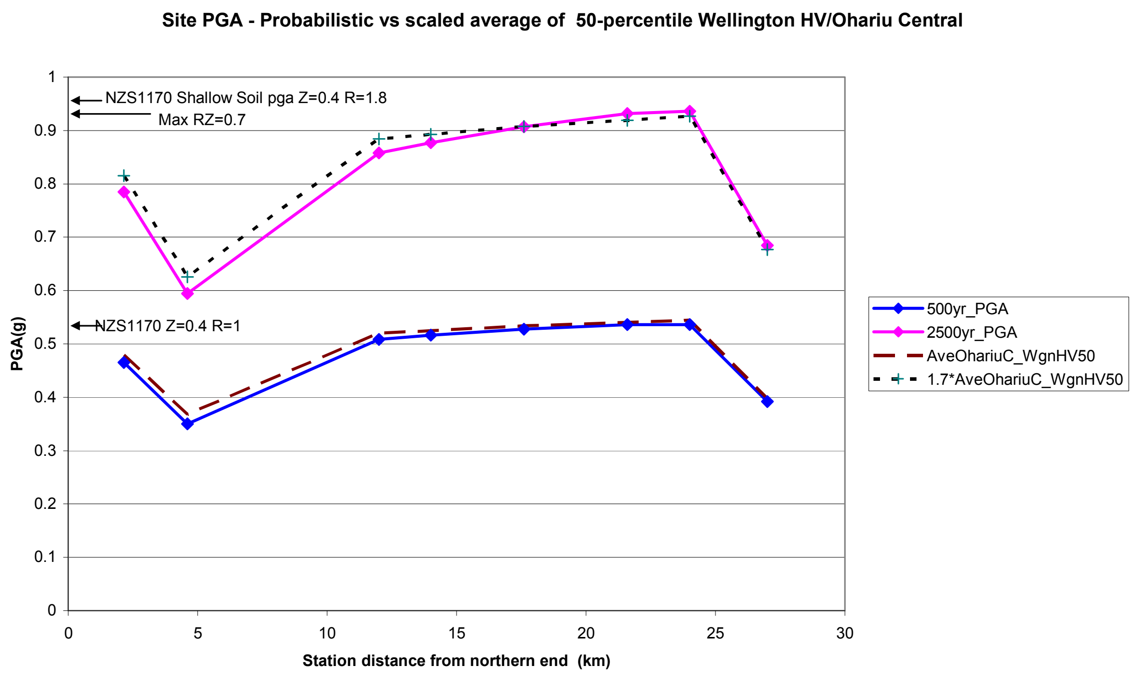



Figure 7 The 500-year and 2500-year peak ground accelerations correlate very well with the average of the 50-percentile motions estimated for the Wellington-Hutt Valley and Ohariu Central fault segments, scaled by 1.0 and 1.7 respectively.

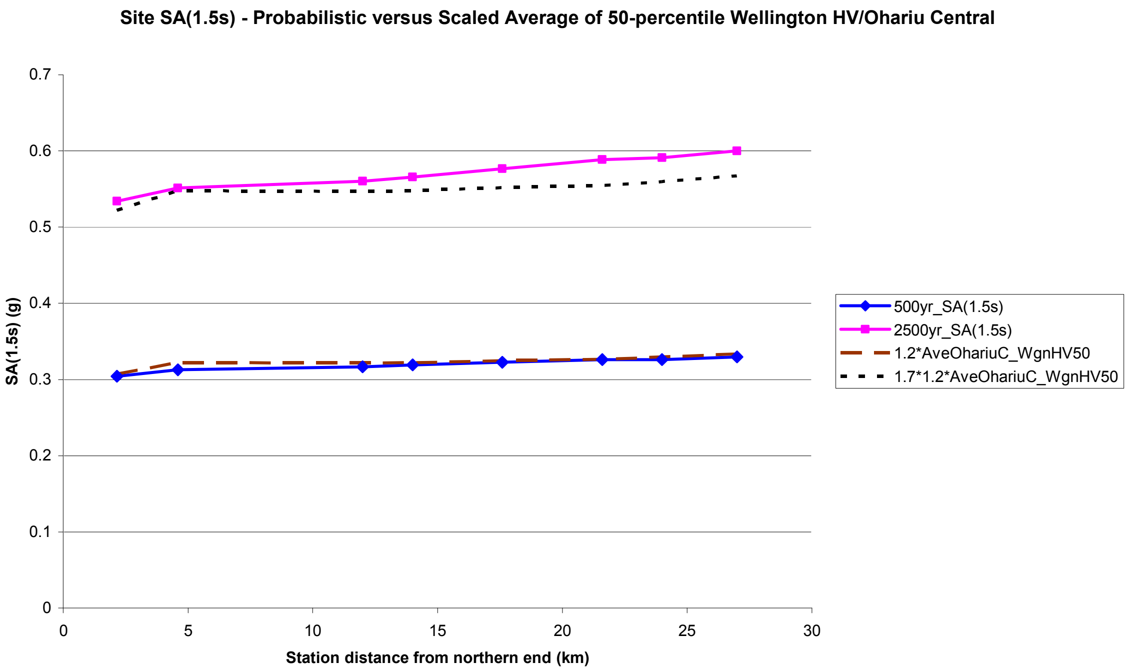



Figure 8 Correlation of the 500-year and 2500-years SA(1.5s) motions with the scaled average of the 50-percentile SA(1.5s) motions estimated for the Wellington-Hutt Valley and Ohariu Central fault segments. A scaling of 1.2 is required to match the 500-year values, and an additional scaling of 1.7 to give a poorer match to the 2500-year motions.

## 3.2 Estimated Spectra

The previous discussion shows that, for a given site class, there is little variation of estimated hazard along the Transmission Gully Route. Also, the spectral shapes are similar for the same site class at different locations. This section presents estimated smoothed spectra for all eight locations.

Close inspection of the log-log plots of the raw acceleration response spectra versus spectral period in Figures A1 to A8 shows localised irregularities, in particular some localised troughs. Also hazard spectra are often more sharply peaked than the spectra of recorded motions. To overcome these features of the estimated hazard spectra, smoothing is applied to the raw spectra to obtain the recommended smoothed spectra, which can be expressed in equation form for spreadsheet applications.

Smoothed design envelopes were developed to largely envelope the raw spectra from the hazard analyses for the recommended design spectra. The construction of these envelopes followed procedures similar to those used in developing code spectra, although different from the specific procedures used for NZS1170.5:2004. Each spectrum comprises a segment rising linearly with period T from the 0s value to period $T_o$, a constant spectral acceleration plateau at the peak of the smoothed spectrum to period $T_C$, and a descending branch in which the spectral acceleration reduces with increasing spectral period T. The smoothing procedure involves defining an appropriate amplitude and period band for the constant acceleration plateau, and approximating the descending branch by segments proportional to $T^{-\gamma}$, where the exponent γ takes values such as 2/3, 3/4, 1 or 2 in various segments. The smoothed spectral shapes used in NZS1170 have a branch proportional to T $^{-0.75}$ in the period range from the corner period $T_C$ to 1.5s, a constant-velocity branch proportional to T $^{-1}$ between 1.5s and 3s, and a constant-displacement branch proportional to T $^{-2}$ at periods beyond 3s. The smoothed spectra recommended in this study have been guided by the code spectra. However, the corner periods $T_o$ and $T_C$ and the exponents of the descending branches have been varied to more appropriately reflect the site-specific study results established from the hazard analyses.

Smoothed Rock spectra are tabulated for the Linden SH1 interchange (Table 5), at the southern end of the route, and Smoothed Shallow Soil spectra for the Intermediate Interchange at station 21,600m (Table 6). These sites have the strongest estimated spectra of the locations with these site classes. The smoothed spectra for a given site class and location are described by equations in terms of the pga and SA(1.5s) values for that site class and location, corner periods $T_s$ and $T_c$ of the plateau of the spectrum, and the return period. The smoothed spectra for the other locations can be constructed from the pga values and SA(1.5s) values listed in Tables 7 (Rock sites) and 8 (Shallow Soil sites). Plots are provided in the Appendix A1 comparing the unsmoothed hazard spectra and the smoothed spectra for the eight locations for return periods of 250 years, 500 years, 1000 years and 2500 years.

It is also desired to produce spectra for Class D Deep or Soft Soil and Class E Very Soft Soil Sites. These can also be defined in terms of the peak ground acceleration and SA(1.5s) values for rock listed in Table 7 or shallow soil listed in Table 8.

**Table 5** Smoothed rock spectra for Linden SH1 Interchange

| Spectral Period T(s) | 5% damped acceleration response spectra SA(T) (g) | | | |
|---|---|---|---|---|
| | Return Period | | | |
| | 250 years | 500 years | 1000 years | 2500 years |
| 0 (pga) | 0.27 | 0.39 | 0.52 | 0.69 |
| 0.1 | 0.77 | 1.09 | 1.69 | 2.31 |
| 0.2 | 0.77 | 1.09 | 1.69 | 2.31 |
| 0.25 | 0.77 | 1.09 | 1.46 | 1.99 |
| 0.5 | 0.49 | 0.69 | 0.92 | 1.25 |
| 0.75 | 0.37 | 0.52 | 0.70 | 0.95 |
| 1.0 | 0.31 | 0.43 | 0.58 | 0.79 |
| 1.5 | 0.23 | 0.33 | 0.44 | 0.60 |
| 2 | 0.17 | 0.25 | 0.33 | 0.45 |
| 3 | 0.12 | 0.16 | 0.22 | 0.30 |

**Table 6** Smoothed shallow soil spectra for the Intermediate Interchange

| Spectral Period T(s) | 5% damped acceleration response spectra SA(T) (g) | | | |
|---|---|---|---|---|
| | Return Period | | | |
| | 250 years | 500 years | 1000 years | 2500 years |
| 0 (pga) | 0.37 | 0.54 | 0.71 | 0.93 |
| 0.1 | 0.78 | 1.40 | 1.85 | 2.52 |
| 0.15 | 0.99 | 1.40 | 1.85 | 2.52 |
| 0.35 | 0.99 | 1.40 | 1.85 | 2.52 |
| 0.5 | 0.69 | 0.98 | 1.30 | 1.77 |
| 0.75 | 0.46 | 0.65 | 0.86 | 1.18 |
| 1.0 | 0.35 | 0.49 | 0.65 | 0.88 |
| 1.5 | 0.23 | 0.33 | 0.43 | 0.59 |
| 2 | 0.17 | 0.24 | 0.32 | 0.44 |
| 3 | 0.12 | 0.16 | 0.22 | 0.29 |

### 3.2.1 Equations for smoothed horizontal spectra

Rock spectra

$SA_{rocksmooth}(0) = PGA_{rock}$ $T=0s$ (1a)

$SA_{rocksmooth}(T) = PGA_{rock} + (SA_{rockmax}-PGA_{rock})(T/0.1)$ $0s<T\leq0.1s$ (1b)

$SA_{rocksmooth}(T) = SA_{max}$ $0.1s\leq T\leq T_c$ (1c)

$SA_{rocksmooth}(T) = SA_{rock}(1.5s)\ (1.5/T)^{0.67}$ $T_c\leq T\leq1.5s$ (1d)

$SA_{rocksmooth}(T) = SA_{rock}(1.5s)\ (1.5/T)$ $1.5s<T<3s$ (1e)

$SA_{rocksmooth}(T) = SA_{rocksmooth}(3s)\ (3/T)^2$ $T>3s$ (1f)

where

$T_c$ = 0.2s, 0.25s or 0.3s (see Table 7)

$SA_{rockmax} = SA_{rock}(1.5s)(1.5/0.2)^{0.67} = 3.86\ SA_{rock}(1.5s)$ for $T_c=0.2s$ (1g)

$= SA_{rock}(1.5s)(1.5/0.25)^{0.67} = 3.30\ SA_{rock}(1.5s)$ for $T_c=0.25s$ (1h)

$= SA_{rock}(1.5s)(1.5/0.3)^{0.67} = 2.92\ SA_{rock}(1.5s)$ for $T_c=0.3s$ (1i)

$PGA_{rock}$ and $SA_{rock}(1.5s)$ are the peak ground acceleration (magnitude-weighted) and 1.5s spectral acceleration estimated for rock for the return period of interest, as listed in Table 7 for the sites classified as Class B Rock in Table 1.

For the locations classified as Class C Shallow Soil in Table 1, spectra for Rock conditions may be constructed using the $T_C$ values for Te Puka (Table 7) using the following approximate relations between the rock and shallow soil parameters that apply for the Transmission Gully locations:

$PGA_{rock} = 0.75\ PGA_{shallow}$ (1j)

$SA_{rock}(1.5s) = SA_{shallow}(1.5s)$ (1k)

Shallow Soil spectra

$SA_{shallowsmooth}(0) = PGA_{shallow}$ T=0s (2a)

$SA_{shallowsmooth}(T) = PGA_{sgallowl}+(SA_{shallowmax}-PGA_{shallow})(T/Ts)$ 0s<T≤$T_s$ (2b)

$SA_{shallowsmooth}(T) = SA_{shallowmax}$ $T_s$ ≤T≤0.35s (2c)

$SA_{shallowsmooth}(T) = 1.5\ SA_{shallow}(1.5s) / T$ 0.35s≤T≤3s (2d)

$SA_{shallowsmooth}(T) = 4.5\ SA_{shallow}(1.5s) / T^2$ T>3s (2e)

where

$SA_{shallowmax} = SA_{shallow}(1.5s)(1.5/0.35) = 4.29\ SA_{shallow}(1.5s)$ (2f)

$T_s$ = 0.15s for a return period of 250 years and 0.1s for return periods of 500, 1000 and 2500 years.

$PGA_{shallow}$ and $SA_{shallow}(1.5s)$ are the peak ground acceleration (magnitude-weighted) and 1.5s spectral acceleration estimated for shallow soil for the return period of interest, as listed in Table 8 for the sites classified as Class C Shallow Soil in Table 1.

For the locations classified as Class B Rock in Table 1, spectra for Shallow Soil conditions may be constructed using the following approximate relations between the rock and shallow soil parameters that apply for the Transmission Gully locations:

$PGA_{shallow} = 1.33\ PGA_{rock}$ (2g)

$SA_{shallow}(1.5s) = SA_{rock}(1.5s)$ (2h)

**Table 7** Parameter values for smoothed rock spectra

| | Return Period | | | |
|---|---|---|---|---|
| | 250 years | 500 years | 1000 years | 2500 years |
| Linden SH1 | | | | |
| $PGA_{rock}$ (g) | 0.27 | 0.39 | 0.52 | 0.69 |
| $SA_{rock}(1.5s)$ (g) | 0.23 | 0.33 | 0.44 | 0.60 |
| Tc (s) | 0.25 | 0.25 | 0.2 | 0.2 |
| Te Puka Valley | | | | |
| $PGA_{rock}$ (g) | 0.25 | 0.35 | 0.45 | 0.59 |
| $SA_{rock}(1.5s)$ (g) | 0.23 | 0.31 | 0.41 | 0.55 |
| Tc (s) | 0.3 | 0.3 | 0.25 | 0.25 |

**Table 8** Parameter values for smoothed shallow soil spectra

| | Return Period | | | |
|---|---|---|---|---|
| | 250 years | 500 years | 1000 years | 2500 years |
| SH1 Perkins | | | | |
| $PGA_{shallow}$ (g) | 0.34 | 0.47 | 0.60 | 0.78 |
| $SA_{shallow}$(1.5s) (g) | 0.22 | 0.30 | 0.40 | 0.53 |
| Battle Hill | | | | |
| $PGA_{shallow}$ (g) | 0.36 | 0.51 | 0.66 | 0.86 |
| $SA_{shallow}$(1.5s) (g) | 0.23 | 0.32 | 0.42 | 0.56 |
| Golf Course | | | | |
| $PGA_{shallow}$ (g) | 0.36 | 0.52 | 0.67 | 0.88 |
| $SA_{shallow}$(1.5s) (g) | 0.23 | 0.32 | 0.42 | 0.57 |
| SH58 Interchange | | | | |
| $PGA_{shallow}$ (g) | 0.37 | 0.53 | 0.69 | 0.91 |
| $SA_{shallow}$(1.5s) (g) | 0.23 | 0.32 | 0.43 | 0.58 |
| Intermediate Interchange | | | | |
| $PGA_{shallow}$ (g) | 0.37 | 0.54 | 0.71 | 0.93 |
| $SA_{shallow}$(1.5s) (g) | 0.23 | 0.32 | 0.43 | 0.59 |
| Cannon's Creek | | | | |
| $PGA_{shallow}$ (g) | 0.37 | 0.54 | 0.71 | 0.94 |
| $SA_{shallow}$(1.5s) (g) | 0.23 | 0.33 | 0.43 | 0.59 |

Deep or Soft Soil Spectra

None of the eight specified locations of Table 1 were associated with NZS1170 Class D Deep or Soft Soil site conditions in the information supplied by Opus. Nevertheless, GNS was requested to provide spectra for this site class. The recommended equations for constructing Class D spectra for the Transmission Gully sites are:

$SA_{deepsmooth}(0) = PGA_{deep}$ T=0s (3a)
$SA_{deepsmooth}(T) = PGA_{deep}+(SA_{deepmax}-PGA_{deep})(T/Ts)$ 0s <T≤ 0.15s (3b)
$SA_{deepsmooth}(T) = SA_{deepmax}$ 0.15s ≤T≤0.75s (3c)
$SA_{deepsmooth}(T) = 1.5\ SA_{deep}(1.5s) / T$ 0.75s≤T≤3s (3d)
$SA_{deepsmooth}(T) = 4.5\ SA_{deep}(1.5s) / T^2$ T>3s (3e)
where
$SA_{deepmax} = SA_{deep}(1.5s)(1.5/0.75) = 2\ SA_{deep}(1.5s)$ (3f)

The two parameters $PGA_{deep}$ and $SA_{deep}$(1.5s) may be found using approximate relationships that apply for the Transmission Gully locations between the values for rock, shallow soil and deep soil. The rock and shallow soil values are listed in Tables 7 and 8

$PGA_{deep} = 0.93\ PGA_{rock} = 0.7\ PGA_{shallow}$ (3g)

$SA_{deep}(1.5s) = 1.9\ SA_{rock}(1.5s) = 1.9\ SA_{shallow}(1.5s)$ (3h)

The Class D spectra have been developed in terms of peak ground accelerations and SA(1.5s) values for rock and shallow soil site conditions, as these parameters were calculated for at least one of these site classes for all eight locations. The smoothed Class D spectra constructed from these parameters are virtually identical to those constructed directly from Class D parameters. This is demonstrated in Figure 9, which shows the unsmoothed Class D spectra estimated directly for the Intermediate Interchange, and their smoothed representations calculated from the shallow soil and deep soil parameters. The two sets of smoothed spectra are almost identical.

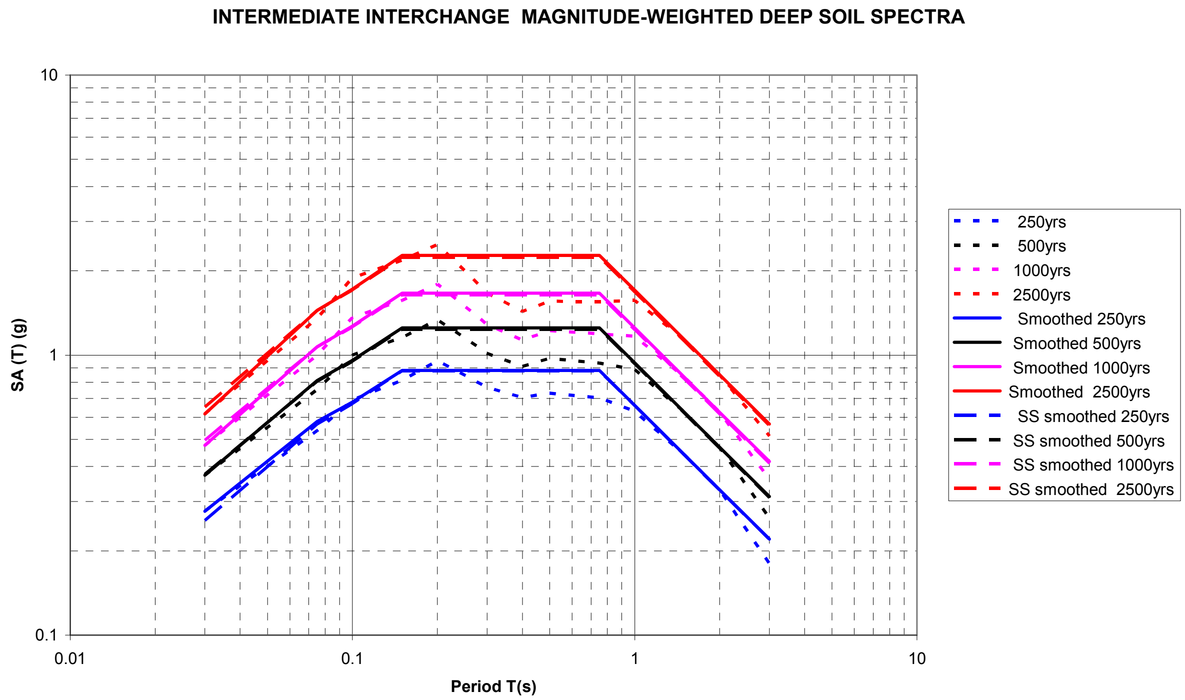



Figure 9 Class D Deep or Soft Soil spectra for the Intermediate Interchange. The unsmoothed estimates are shown as dotted lines, while the smoothed spectra calculated from shallow soil parameters (SS curves shown as dashed lines) are almost indistinguishable from those developed from deep soil parameters (solid curves).

## Very Soft Soil spectra

Response spectrum attenuation models, including the McVerry et al. (2006) model used in the New Zealand NSHM, generally do not cater for site conditions that fall into Class E Very Soft Soil Sites. The approach used in this study is to assign Class E sites a similar spectral shape to that assigned to Class E sites in NZS1170. As in NZS1170, at short spectral periods, the Class E spectrum is taken identical to the corresponding Class D spectrum, with the long-period corner of the plateau taken as 1s. The amplification of the Class E spectrum with respect to the Class D spectrum is only 1.33 in the long-period range (beyond 1s), compared to 1.55 in NZS1170, because the Class D spectra recommended in this study have a longer corner period of 0.75s than the value of 0.56s used in NZS1170. The long-period amplification of Class E with respect to Class C Shallow Soil is 2.53, almost equal to this ratio in NZS1170.

$SA_{verysoft}(0) = PGA_{deep}$ T=0s (4a)
$SA_{verysoft}(T) = PGA_{deep}+(SA_{deepmax}-PGA_{deep})(T/Ts)$ 0s <T≤ 0.15s (4b)
$SA_{verysoft}(T) = SA_{deepmax}$ 0.15s ≤T≤1s (4c)
$SA_{verysoft}(T) = 2\ SA_{deep}(1.5s) / T$ 1s≤T≤3s (4d)
$SA_{verysoft}(T) = 6\ SA_{deep}(1.5s) / T^2$ T>3s (4e)
where
$SA_{deepmax} = 2\ SA_{deep}(1.5s)$ (4f)

The parameters $PGA_{deep}$ and $SA_{deep}(1.5s)$ may be taken as given by equations (3g) and (3h).

Figure 10 compares the recommended smoothed spectra for the four site classes, for the 1000 year spectra at the Intermediate Interchange.

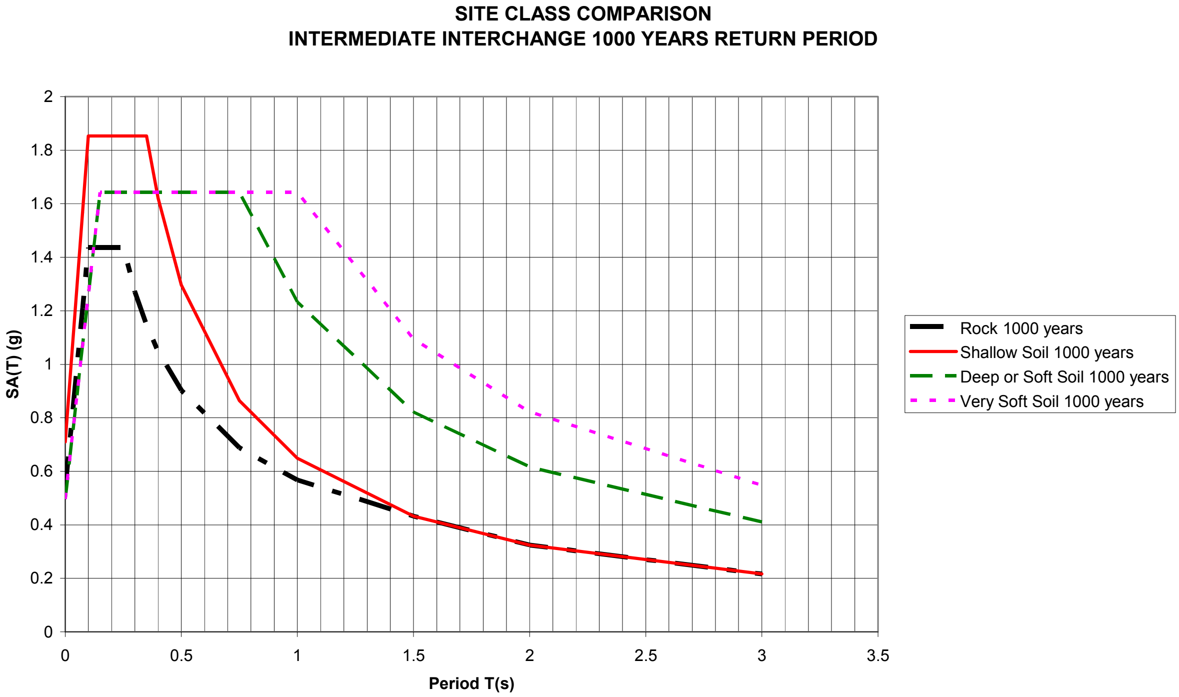



Figure 10 Comparison of the spectra for the four site classes, for the 1000-year spectra at the Intermediate Interchange.

## 3.3 Scenario spectra

The smoothed spectra for return periods of 250 years, 500 years, 1000 years and 2500 years for the eight sites are compared with scenario spectra in Figures A9 to A16 in the Appendix. The scenario spectra considered are the 50- and 84-percentile spectra for the Wellington-Hutt Valley segment of the Wellington Fault, the 50- and 84-percentile spectra for the Ohariu Fault, and the 50-percentile spectra for the Moonshine Fault. The 50-percentile spectra are shown in part (a) of each plot, and the 84-percentile spectra in part (b)

The Wellington-Hutt Valley segment is the main contributor to the estimated hazard at all sites, but generally produces weaker scenario spectra for a given percentile level than the Ohariu Fault because of its greater distance than from the Transmission Gully sites (Table 3). The short average recurrence interval of 700 years for rupture of the Wellington-Hutt

Valley fault segment means that it is appropriate to consider both the 50- and 84-percentile spectra for this earthquake source.

For the Ohariu Fault, the stronger of the scenario spectra for the central or south segment is presented; for all but the Linden SH1 site, the stronger spectrum is produced by the central segment. The average recurrence intervals of rupture of 1800 years for the central segment and 2300 years for the south segment of the Ohariu Fault means that it is less clear-cut than for the Wellington Fault whether the 84-percentile spectra need be considered for these sources.

The Moonshine Fault has a very long estimated recurrence interval of over 11,000 years. In this circumstance, only the 50-percentile spectrum need be considered. Although its magnitude of 7.1 is smaller than the magnitudes for the other faults, it has a reverse mechanism. The attenuation model produces stronger motions for a given magnitude and distance for reverse-mechanism earthquakes than for the strike-slip mechanism associated with the other two faults.

The smoothed 2500-year probabilistic spectra for all sites exceed the 50-percentile scenario spectra for all three sources.

The 1000-year spectra are significantly exceeded by the Moonshine 50-percentile spectra at the SH58 Interchange, the Intermediate Interchange and Cannon’s Creek, all sites within 1.5 km of this fault. The Ohariu Fault 50-percentile spectra marginally exceed the 1000-year spectra at Perkins and Te Puka Stream over a narrow period band at the peak of the spectrum; both these sites are within 1 km of the fault. The 1000-year spectra are never exceeded by the Wellington Fault 50-percentile motions.

The 2500-year spectra exceed the 84-percentile motions for the Ohariu and Wellington Faults at most spectral periods. The Ohariu 84-percentile spectra exceed the 2500-year spectra for all sites at the peak of the spectrum, but for periods beyond the plateau of the smoothed spectrum only for Perkins and Te Puka, and then only marginally. The peak of the Wellington Fault 84-percentile spectrum reaches or slightly exceeds the 2500-year spectrum over a narrow period band at the six sites within 10 km of the fault, but never for periods beyond the plateau of the spectrum.

In conclusion, the smoothed 2500-year spectra are sufficient to account for 50-percentile motions from all faults, and are at most marginally exceeded by 84-percentile motions from the Ohariu or Wellington Faults, and even then, generally over short period bands.

## 3.4 Comparison with NZS1170 spectra

The spectra recommended in Section 3.2 in compared with the NZS1170 spectra for the four site classes in Figures 11 to 14. The comparisons are shown for the highest hazard site, the Intermediate Interchange, and compared with the NZS1170 spectra for Z=0.42, as interpolated from the NZS1170 Hazard Factor map.

**COMPARISON OF SMOOTHED ROCK SPECTRA WITH NZS1170**
**INTERMEDIATE INTERCHANGE**

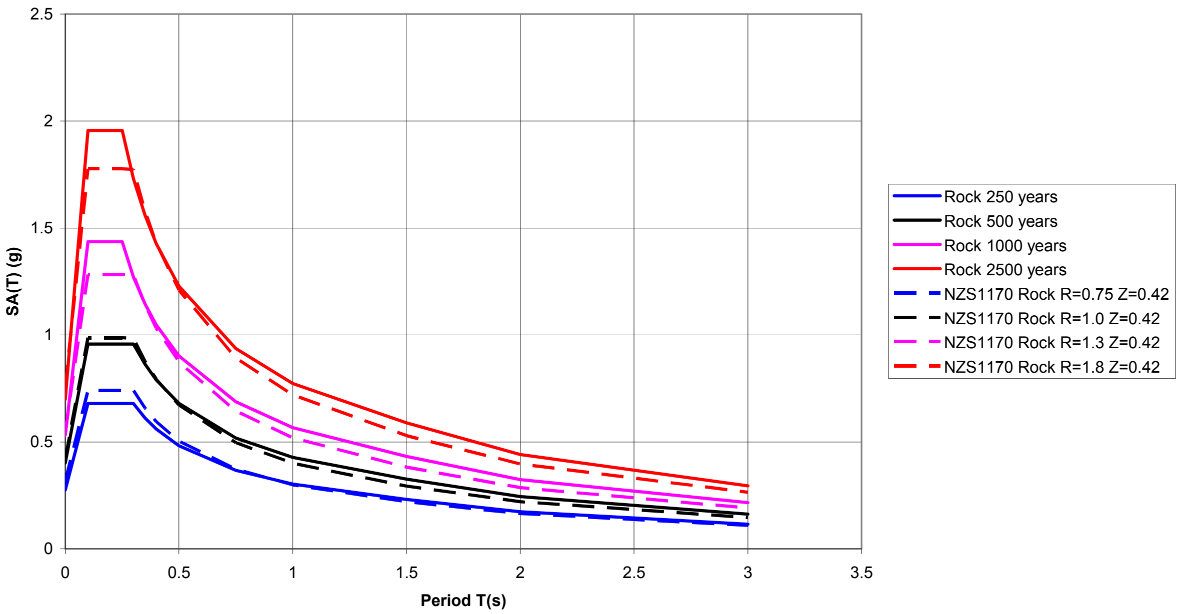



Figure 11 Comparison of recommended spectra (solid curves) for the Intermediate Interchange location with the NZS1170 spectra (Z=0.42) (dashed curves), for NZS1170 Class A/B Rock site conditions.

**COMPARISON OF SMOOTHED SHALLOW SOIL SPECTRA WITH NZS1170**
**INTERMEDIATE INTERCHANGE**

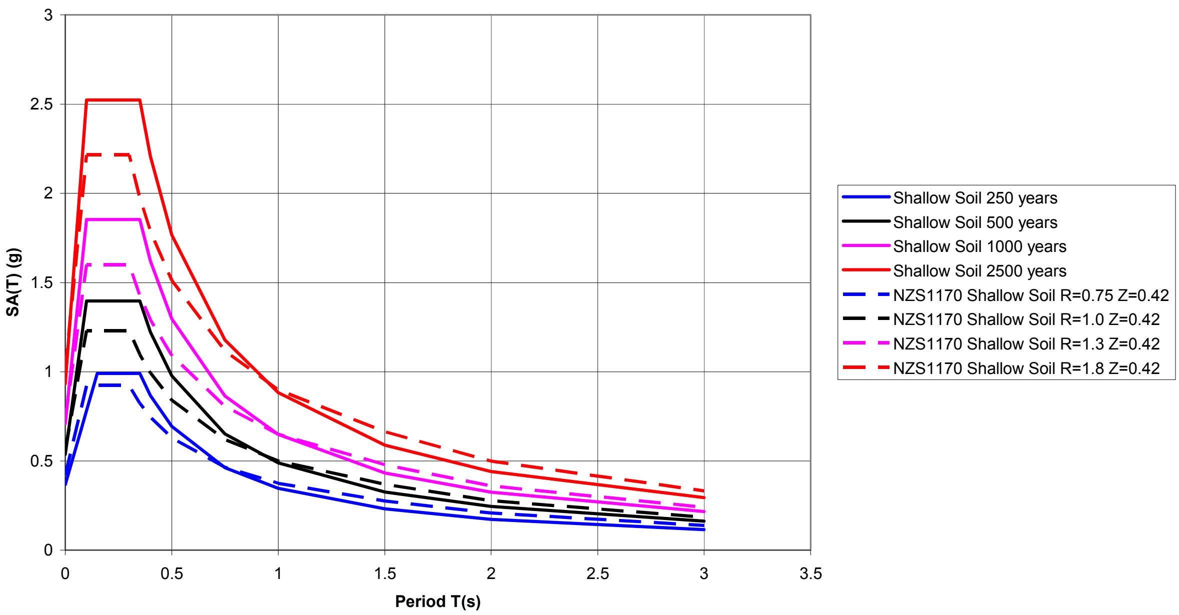



Figure 12 Comparison of recommended spectra for the Intermediate Interchange location with the NZS1170 spectra (Z=0.42), for NZS1170 Class C Shallow Soil site conditions.

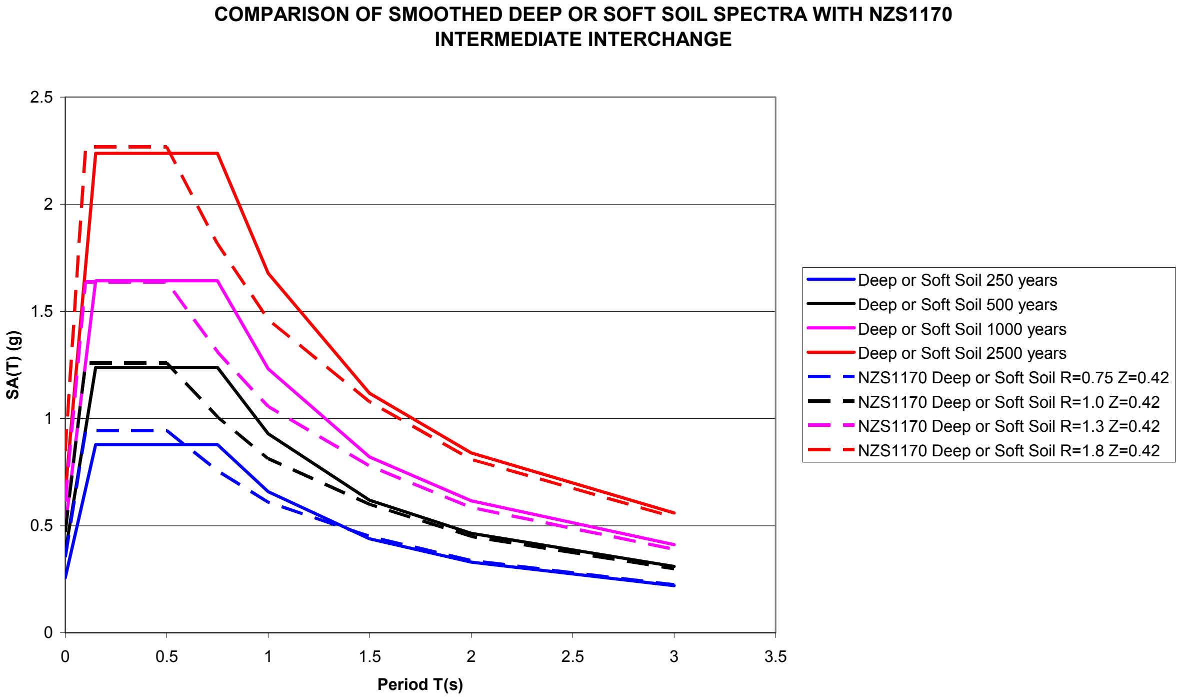



Figure 13 Comparison of recommended spectra for the Intermediate Interchange location with the NZS1170 spectra (Z=0.42), for NZS1170 Class D Deep or Soft Soil site conditions.

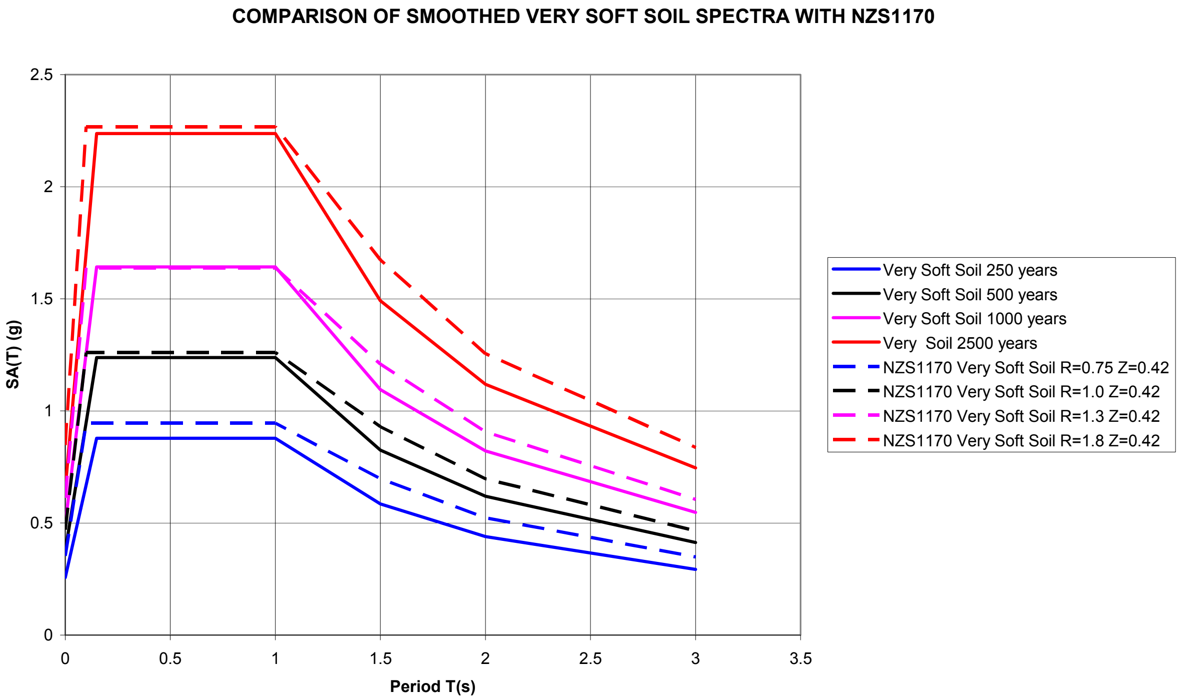



Figure 14 Comparison of recommended spectra for the Intermediate Interchange location with the NZS1170 spectra (Z=0.42), for NZS1170 Class E Very Soft Soil site conditions.

The NZS1170 spectra and the recommended smoothed spectra of this study are generally similar. The differences result mainly from different spectral shapes used in this study. The falloff with period T from the plateau of the spectra beyond the corner period $T_c$ has been taken proportional to $T^{-1}$ in this study for all but the rock site class, for which a branch proportional to $T^{-0.67}$ has been use from $T_c$ to 1.5s. The NZS1170 spectra have a transitional segment proportional to $T^{-0.75}$ from the plateau to 1.5s which has been omitted in the smoothed spectra recommended for Transmission Gully. In the NZS1170 spectra, Return Period Factors of R=0.75, 1, 1.3 and 1.8 are used for return periods of 250 years, 500 years, 1000 years and 2500 years, respectively.

The smoothed spectra shapes recommended in this study differ from the NZS1170 spectral shapes to better match the location-specific spectra estimated for the Transmission Gully sites. The NZS1170 spectra are intended to approximately envelop spectral shapes calculated for locations throughout New Zealand.

### 3.5 Near-Fault Factors

For the region traversed by the Transmission Gully route, NZS1170 requires using the Near-Fault Factor N(T,D) for spectral period T corresponding to the shortest distance D of the site of interest from the Wellington Fault. Basing the near-fault factors on distance from the Wellington Fault seems a reasonable starting point in that estimated hazard rates along the route are influenced most by the Wellington-Hutt Valley segment of the Wellington Fault, as noted in Section 3.1 and demonstrated by the general alignment of the hazard contours parallel with this fault segment, as shown in Figures A17 to A21.

The Wellington Fault is one of the eleven faults for which Near-Fault Factors are required for return periods exceeding 250 years according to Clause 3.1.6 of NZS1170.5:2004. The offshore segment of the Wairau Fault is another of the twelve faults, but because of its longer average recurrence interval of rupture of 1900 years compared to the 700 years of the Wellington-Hutt Valley fault segment, it makes less contribution to the hazard Also, its generally greater distance from the Transmission Gully route leads to lower Near-Fault factors than for the Wellington Fault.

The potential near-fault effects for the Central Ohariu fault segment (and the Ohariu South segment for the SH! Linden Interchange) are also considered in this report, because of its closer proximity to the Transmission Gully route, although the Ohariu Fault is not among the faults for which near-fault factors need be applied according to NZS1170. Its slip rate does not fit the NZS1170 criterion of 5mm/year or greater to be included among the eleven faults (NZS1170.5 Commentary Clause C3.1.6). This criterion was chosen to obtain average recurrence intervals that are short enough for the selected faults to make dominant contributions to the hazard at return periods of up to 2500 years that are usually of interest for earthquake-resistant design.

Near-fault factors have been estimated from the Somerville *et al*. (1997) model for five rupture scenarios of each of the Wellington-Hutt Valley fault segment and the Central Ohariu (or Ohariu South) fault segment. The five rupture scenarios for each fault segment correspond to rupture initiating at the southern end and at locations at fractions of 0.25, 0.5, 0.75 and 1 of the fault length from its southern end.

For the Wellington-Hutt Valley Fault, the various sites considered along the Transmission Gully route are at fractions of between 0.6 and 0.875 of the segment length from the southern end of the Wellington-Hutt Valley fault segment, at closest distances between of 6 km and 16 km from the fault. The near-fault factors for the five rupture scenarios considered for the Wellington-Hutt Valley Fault for the eight sites along the Transmission Gully route are shown in Figures A21 to A28, together with their average $NF_{ave}(T)$. Also shown is the NZS1170 Near-Fault Factor N(T,D) that applies for each site, for its shortest distance D from the Wellington-Hutt Valley Fault. The average factor $NF_{ave}(T)$, the maximum factor $NF_{max}(T)$ corresponding to rupture initiating at the southern end of the fault segment and propagating towards the site for most of the rupture length, and the NZS1170 Near-Fault Factor N(T,D) are listed in Tables A1 to A8 that accompany the Figures.

The largest enhancement from forward-directivity effects of the Wellington-Hutt Valley rupture scenarios is produced by rupture initiating at the southern end of the fault in Cook Strait, furthest from the Transmission Gully route and propagating towards it. Some of the rupture scenarios give calculated near-fault factors of less than 1.0 for the Transmission Gully. In calculating the average factors, the scenarios which produce reduced motions are included, so the average values are considerably less than the values for the case of maximum forward-directivity. In fact, the near-fault factors at the Transmission Gully sites averaged for the five rupture scenarios of the Wellington-Hutt Valley Fault differ only slightly from 1.0 for all periods (Tables A10 to A8 and Figures A22 to A29). In light of these results, a case could be made for using near-fault factors of 1.0 for Transmission Gully. However, it is recommended that some recognition should be given to the potential for motions greatly enhanced by forward-directivity effects resulting from ruptures initiating near the southern end of fault segment with rupture propagation mainly in a north-easterly direction towards Transmission Gully.

Transmission Gully is also susceptible to strong-forward directivity effects for some rupture scenarios of the Ohariu Fault (Tables A1to A8)

The NZS1170 Near-Fault Factor N(T, D) corresponding to the shortest distance D of a site to the Wellington-Hutt Valley fault segment seems a reasonable compromise between the average near-fault factors of close to 1.0 and the maximum values for the rupture scenarios of the Wellington-Hutt Valley and Ohariu Faults. The NZS1170 factor makes a reasonable allowance for near-fault effects, while using the average factor for the five Wellington-Hutt Valley rupture scenario makes essentially no allowance for enhancement of motions from near-fault effects. The scenarios considered demonstrated that substantial enhancement of motions from near-fault effects is quite possible along the Transmission Gully route.

Table 9 lists the NZS1170 values for the shortest distance of 6 km and the longest distance of 16 km of the eight Transmission Gully sites from the Wellington Fault. The factors for other distances from the Wellington Fault can be calculated using the NZS1170 expression for the Near Fault factor, and are listed in Tables A1 to A8 for each of the sites.

The values in Table 9 show that the Near-Fault modifications to the hazard spectra for the Transmission Gully sites are modest for periods up to 3s period.

The NZS1170 Near-Fault Factors affect only longer period components, beyond 1.5s, for return periods of 500 years and longer. Near-fault factors need not be considered for the 250-year spectra.

The appropriate factors to use for the scenario spectra should be the maximum values estimated for the site and fault combination being studied. These are generally larger than the NZS1170 factors.

**Table 9** NZS1170 Near-Fault Factors for shortest and longest distances from the Wellington Fault

| Period | ≤ 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|
| NZS1170 Factor for shortest distance to Wellington Fault N(T, D=6 km) | 1 | 1.093 | 1.28 | 1.47 | 1.56 |
| NZS1170 Factor for longest distance to Wellington Fault N(T, D=16 km) | 1 | 1.027 | 1.08 | 1.13 | 1.16 |

## 4.0 SUMMARY

The main findings of this report are summarised as follows:

- The principal contributions to the hazard spectra estimated for the eight locations along the Transmission Gully route are provided by the Wellington-Hutt Valley segment of the Wellington Fault, which lies at shortest distances ranging from 6 km to 16 km from the eight sites.
- All eight locations lie within 5 km of the Ohariu Fault, and six of the locations are within 5 km of the Moonshine Fault.
- Scenario motions estimated for the Ohariu and Moonshine Faults are stronger at some of the sites than the scenario motions for the Wellington Fault, but they contribute less to the estimated hazard because of their much longer recurrence intervals of fault rupture.
- The hazard estimates vary only moderately along the route for a given site class. The estimated 500-year peak ground accelerations for shallow soil conditions range from 0.47g to 0.54g, and the 2500-year values from 0.78g to 0.94g.
- The smoothed 2500-year spectra are sufficient to account for 50-percentile motions from all faults, and are at most marginally exceeded by 84-percentile motions from the Ohariu or Wellington Faults, generally over only short period bands.
- The 1000-year spectra are exceeded by the 50-percentile scenario spectra for locations within about 2 km of the Ohariu or Moonshine Faults. The 1000-year spectra are never exceeded by the Wellington Fault 50-percentile motions.
- The recommended smoothed hazard spectra are generally similar to those estimated using NZS1170.5:2004. The main differences are from different smoothed spectra shapes recommended in this study.
- The NZS1170 Near-Fault factors for the Transmission Gully locations, based on the shortest distance of a site to the Wellington-Hutt Valley fault segment, seem a reasonable compromise between the near-fault factors averaged across the rupture scenarios of the

fault segment, which are close to 1.0, and the maximum values from the rupture scenarios.

- The NZS1170 Near-Fault Factors for the Transmission Gully locations are modest for periods up to 3s.
- Near-fault effects are likely to be considerably greater than the NZS1170 factors for some ruptures of the Ohariu, Moonshine and Wellington Faults that involve rupture-propagation towards the Transmission Gully sites over most of the rupture length.

## 5.0 ACKNOWLEDGEMENTS

Jim Cousins and Warwick Smith are thanked for performing in-house reviews of this report.

## 6.0 REFERENCES

Berryman, K., Webb, T., Hill, N., Stirling, M., Rhoades, D., Beavan, J., Darby, D., 2002. Seismic loads on dams, Waitaki system. Earthquake Source Characterisation. Main report. GNS client report 2001/129.

Hanks, T.C. and Bakun, W.H. 2002. A bilinear source-scaling model for M-logA observations of continental earthquakes. *Bulletin of the Seismological Society of America* 92, 1841-1846.

McVerry, G.H., Zhao, J.X., Abrahamson, N.A., Somerville, P.G. 2006. New Zealand acceleration response spectrum attenuation relations for crustal and subduction zone earthquakes. *Bulletin of the New Zealand Society of Earthquake Engineering 38(1)*, 1-58.

Standards New Zealand 2004. *Structural Design Actions– Part 5 Earthquake Actions – New Zealand. New Zealand Standard NZS 1170.5:2004*.

Stirling, M.; McVerry, G.; Berryman, K.; McGinty, P.; Villamor, P.; Van Dissen, R.; Dowrick, D.; Cousins, J.; Sutherland, R. 2000: Probabilistic seismic hazard assessment of New Zealand: New active fault data, seismicity data, attenuation relationships and methods. Lower Hutt, Institute of Geological and Nuclear Sciences. 117 p.

Stirling, M.W., McVerry, G.H., Berryman, K.R., 2002. A new seismic hazard model for New Zealand. *Bulletin of the Seismological Society of America* 92/5 *pp 1878-1903.*

Stirling, M.W. & the Earthquake Hazards Team. 2007. Updating the national seismic hazard model for New Zealand. 8th Pacific Conference on Earthquake Engineering, Paper No. 8PCEE/072, Singapore, 5-7 December 2007.

Stock, C. and Smith, E.G.C. 2002. Comparison of seismicity models generated by different kernel estimations. *Bulletin of the Seismological Society of America* 92, 913-922

Villamor, P., Berryman, K., Webb, T., Stirling, M., McGinty, P., Downes, G., Harris, J., Litchfield, N., 2001. Waikato seismic loads - Task 2.1. Revision of seismic source characterisation. GNS client report 2001/59.

# APPENDIX 1 SPECTRA, HAZARD MAPS AND NEAR-FAULT FACTORS

## A1 COMPARISON OF SMOOTHED SPECTRA WITH HAZARD ESTIMATES

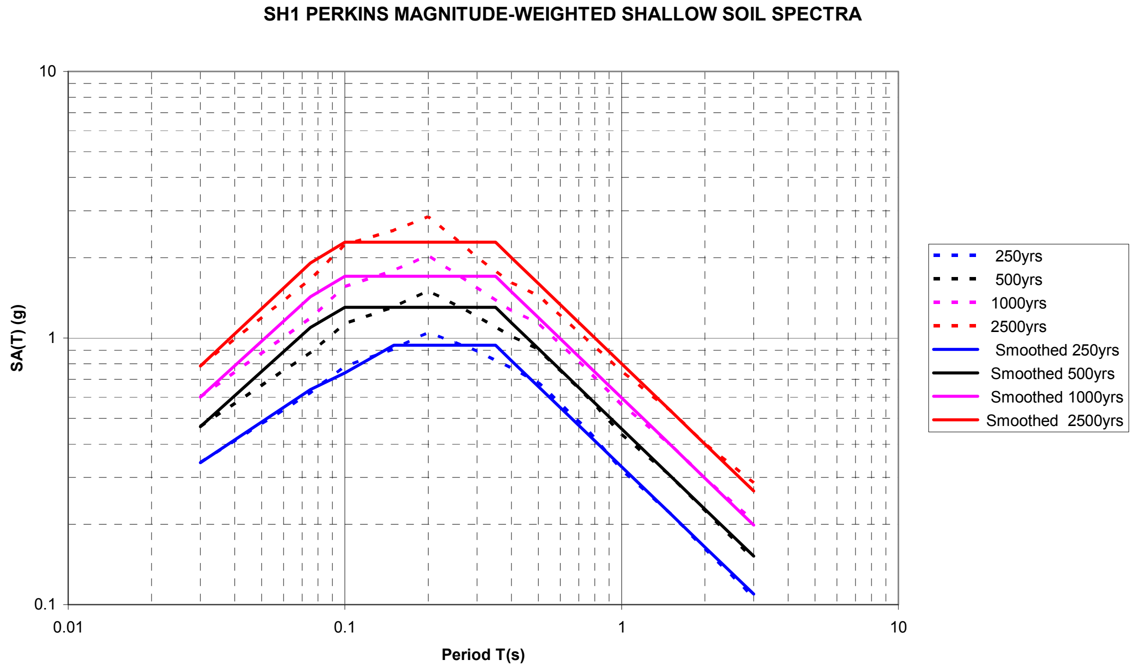



Figure A1 Comparison of hazard estimates and smoothed spectra for shallow soil site conditions at the SH1 crossing at Perkins, at station distance 2150m from the northern end of the route.

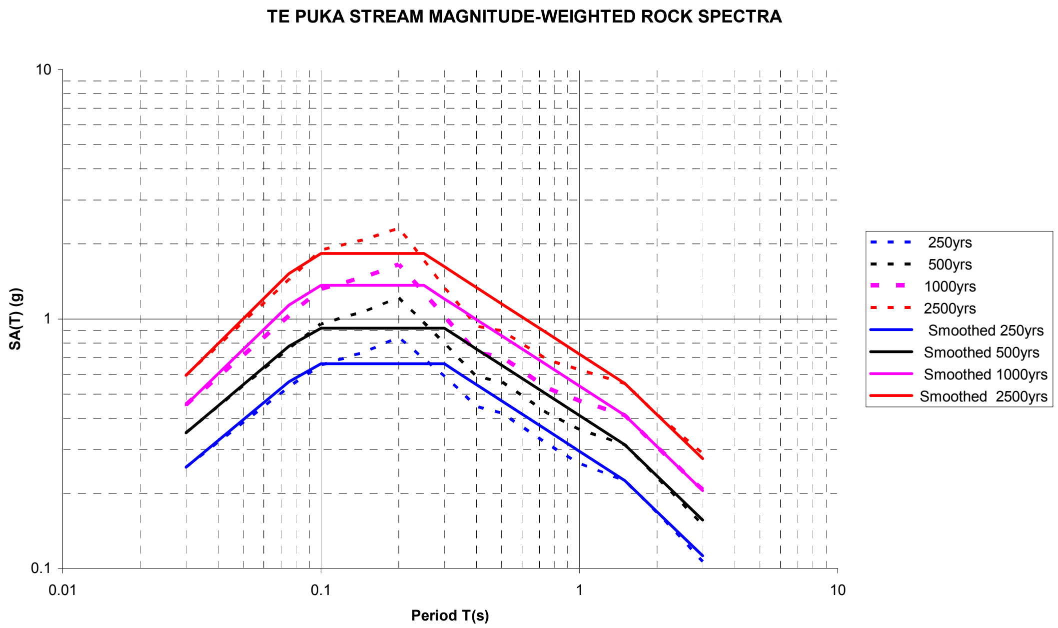



Figure A2 Comparison of hazard estimates and smoothed spectra for rock site conditions at Te Puka Valley, at station distance 4600m from the northern end of the route.

BATTLE HILL MAGNITUDE-WEIGHTED SHALLOW SOIL SPECTRA

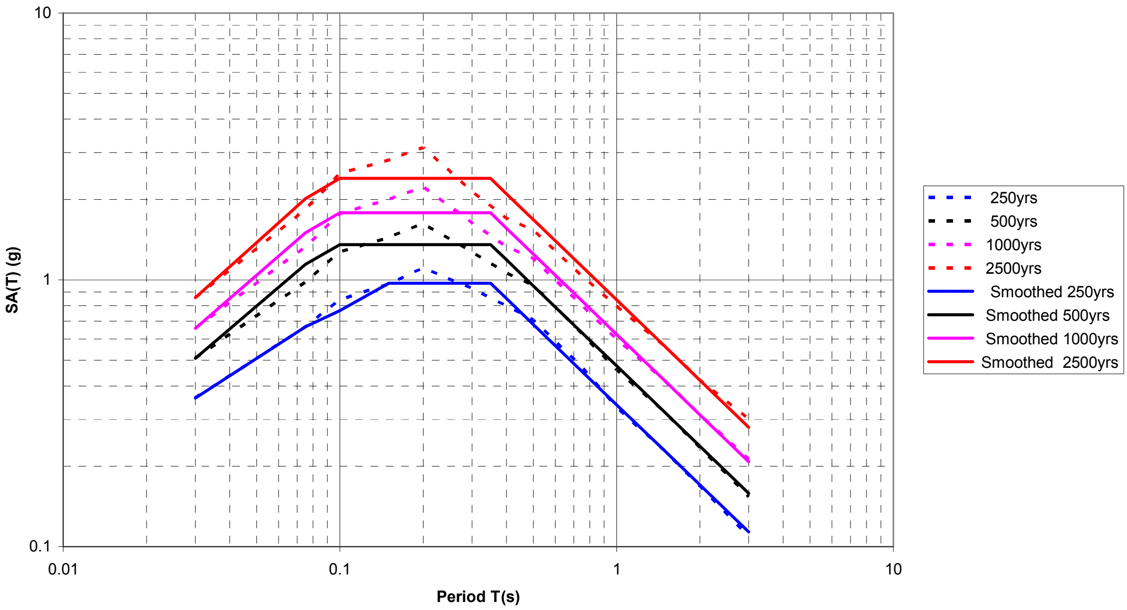



Figure A3 Comparison of hazard estimates and smoothed spectra for shallow soil site conditions at Battle Hill, at station distance 12,000m from the northern end of the route.

GOLF COURSE MAGNITUDE-WEIGHTED SHALLOW SOIL SPECTRA

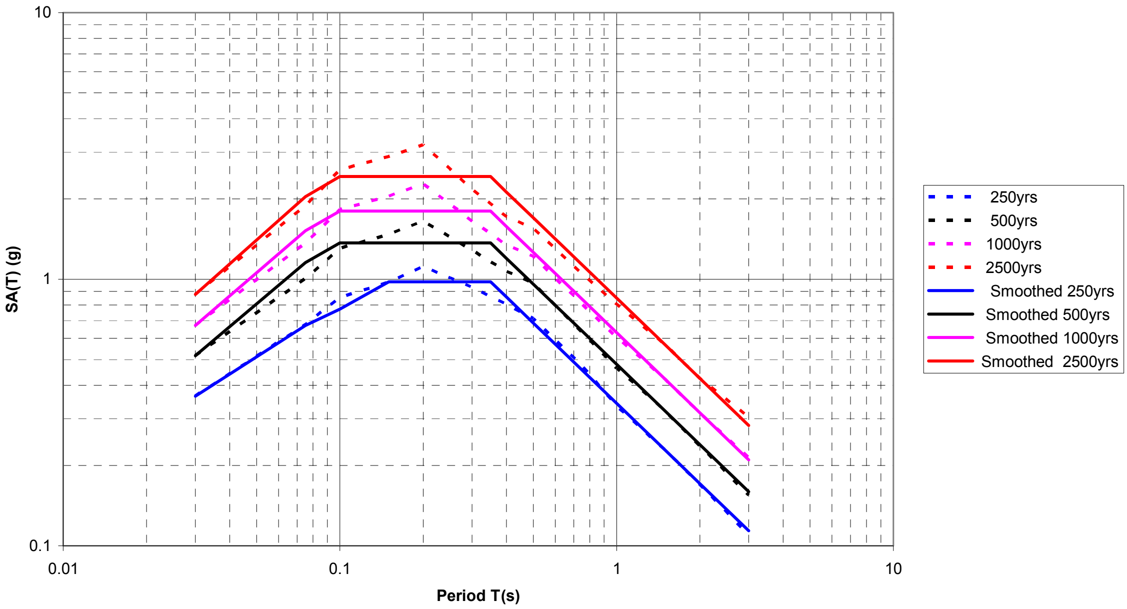



Figure A4 Comparison of hazard estimates and smoothed spectra for shallow soil site conditions at the Golf Course, at station distance 14,000m from the northern end of the route.

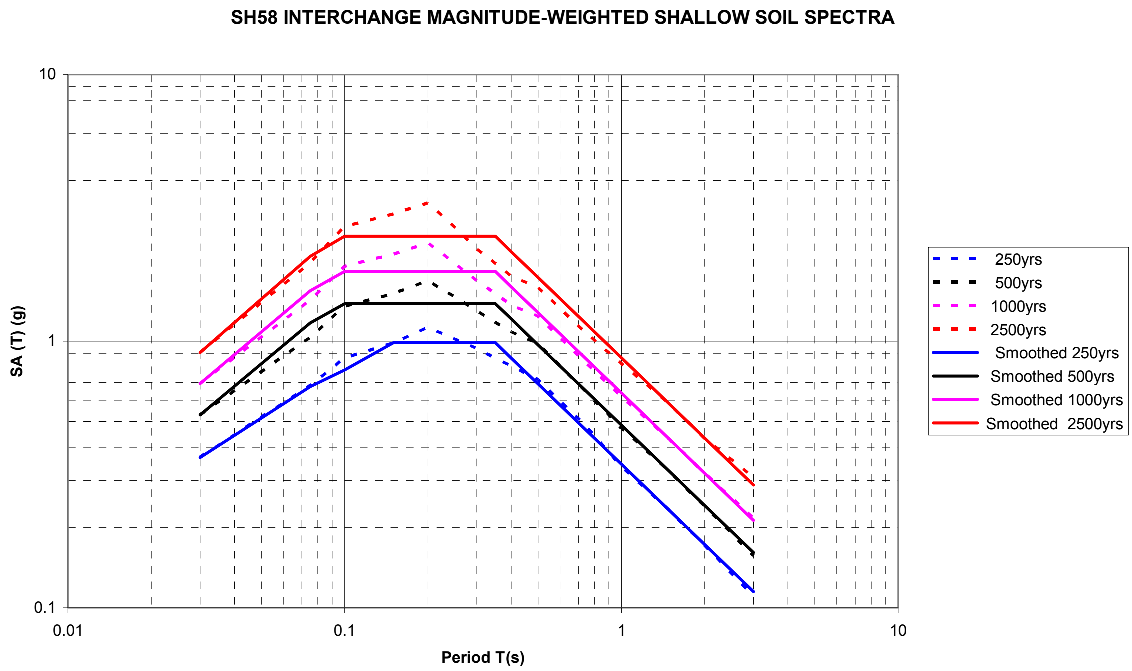



Figure A5 Comparison of hazard estimates and smoothed spectra for shallow soil site conditions at the SH58 Interchange, at station distance 17,600m from the northern end of the route.

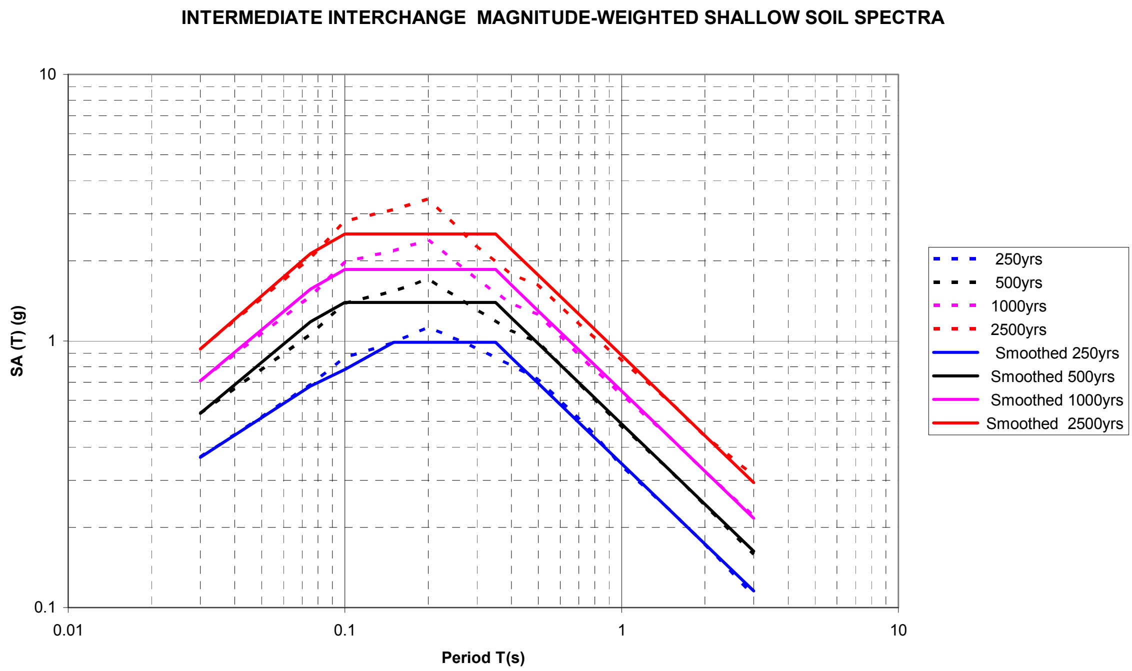



Figure A6 Comparison of hazard estimates and smoothed spectra for shallow soil site conditions at the Intermediate Interchange, at station distance 21,600m from the northern end of the route.

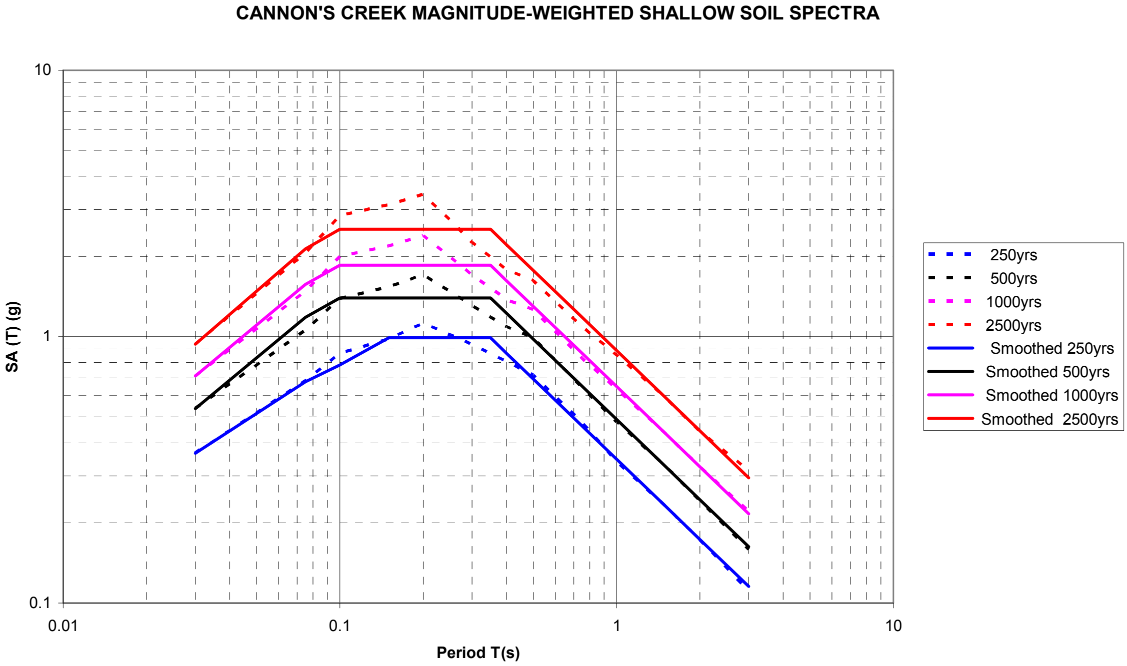



Figure A7 Comparison of hazard estimates and smoothed spectra for shallow soil site conditions at Cannon's Creek, at station distance 24,000m from the northern end of the route.

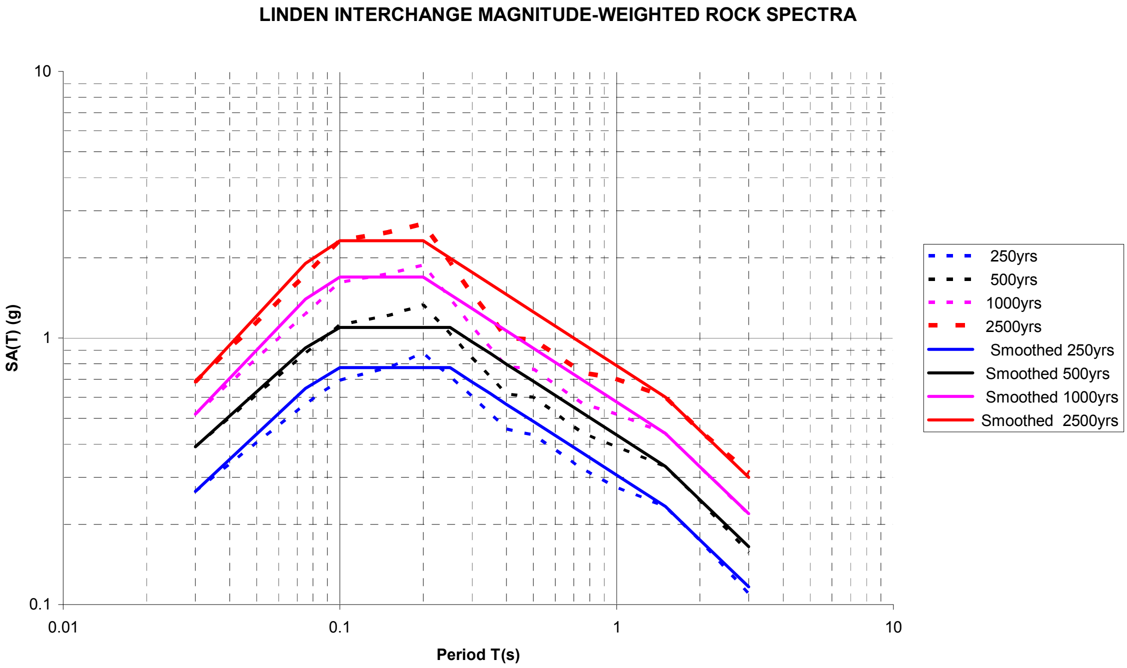
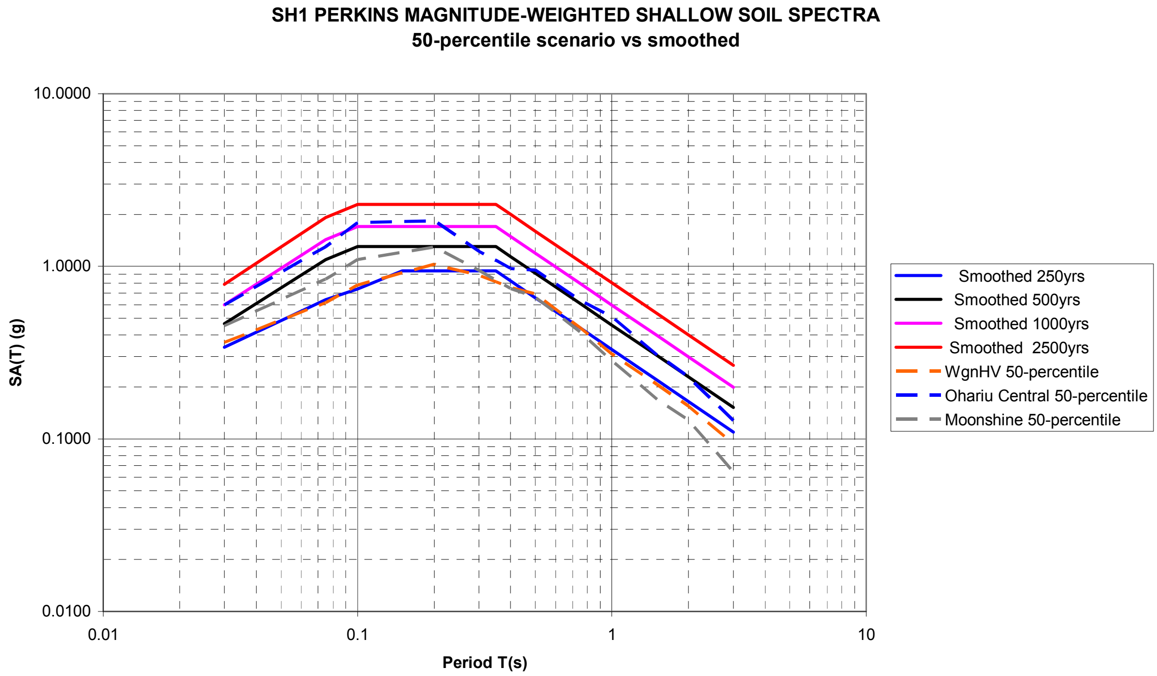
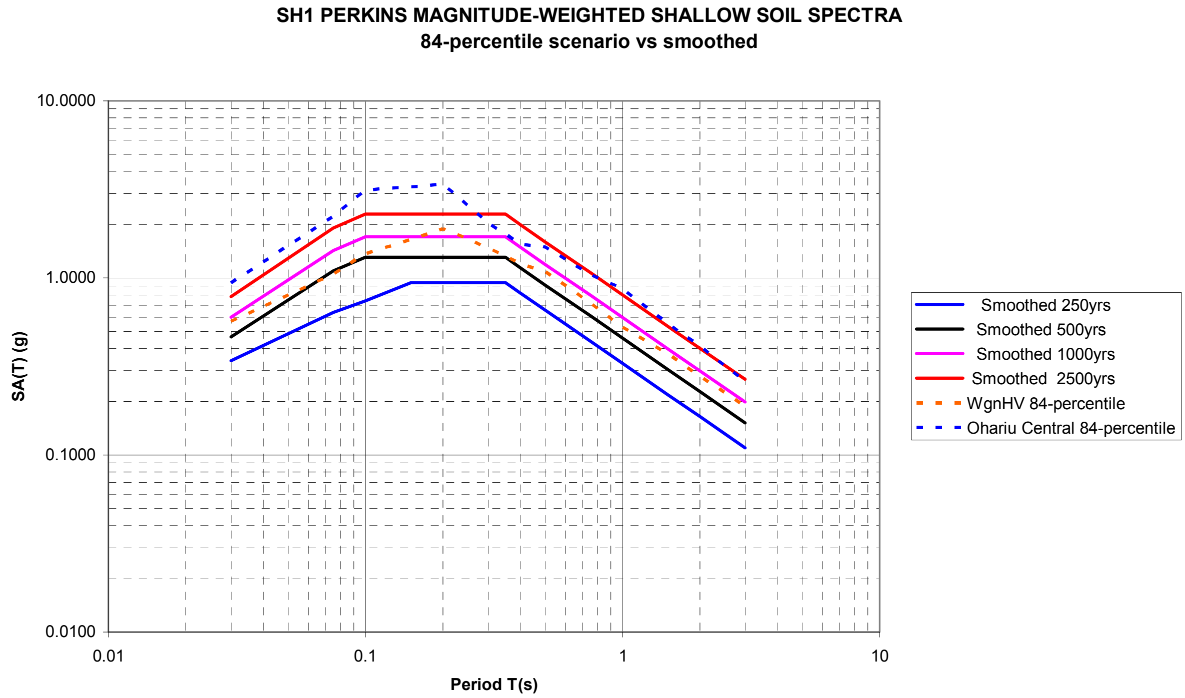



Figure A8 Comparison of hazard estimates and smoothed spectra for rock site conditions at Linden SH1 Interchange, at the southern end of the route.

# A2 COMPARISON OF SMOOTHED SPECTRA AND SCENARIO SPECTRA

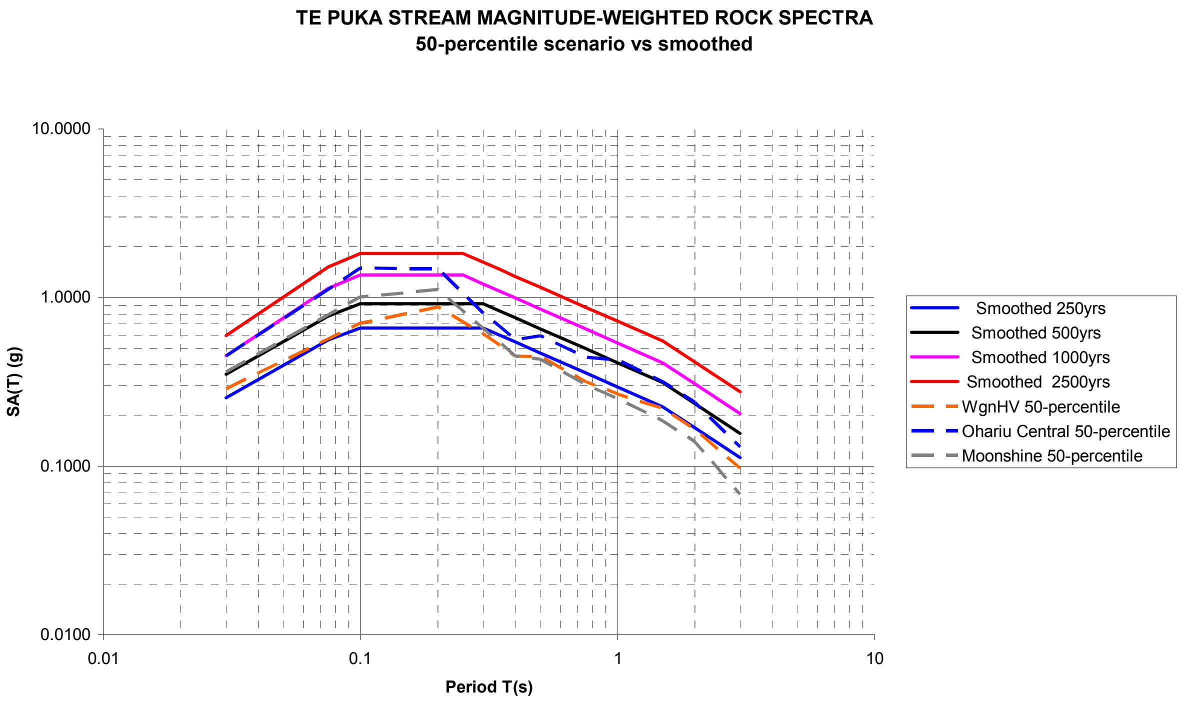



(a)

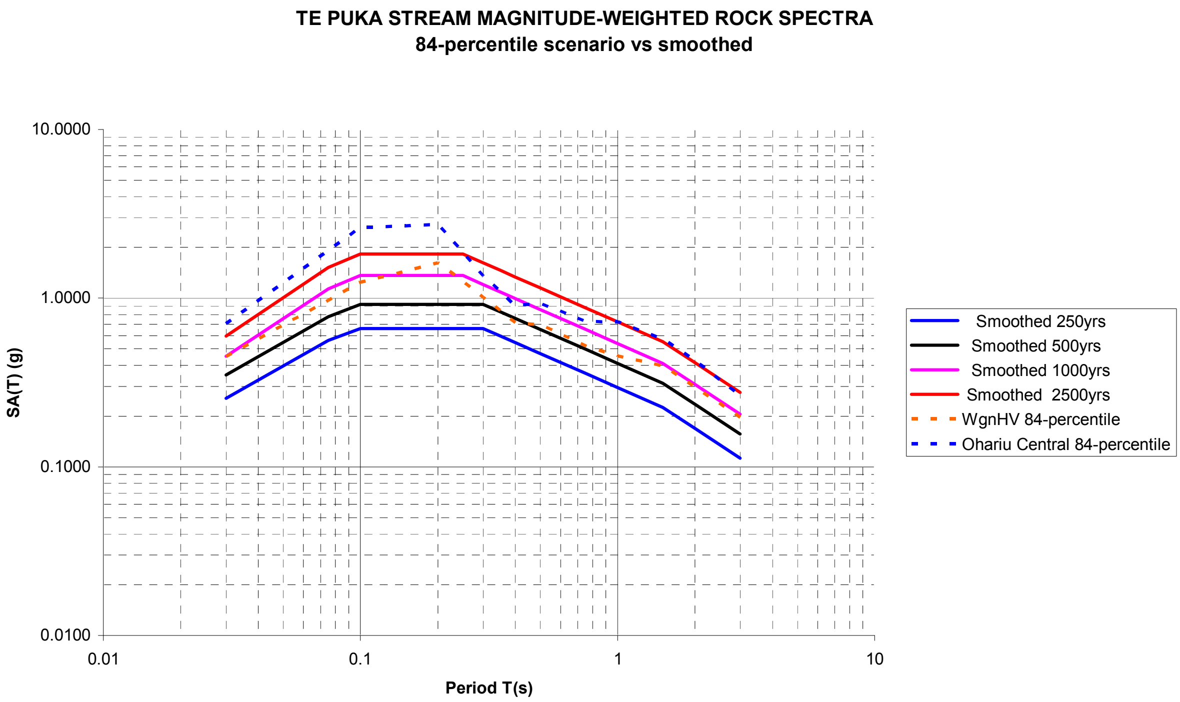



(b)

Figure A9 Comparison of (a) 50-percentile and (b) 84-percentile scenario spectra with smoothed spectra for shallow soil site conditions at the SH1 crossing at Perkins. Scenario spectra are shown for the Wellington-Hutt Valley fault segment, the central segment of the Ohariu Fault, and the Moonshine Fault.

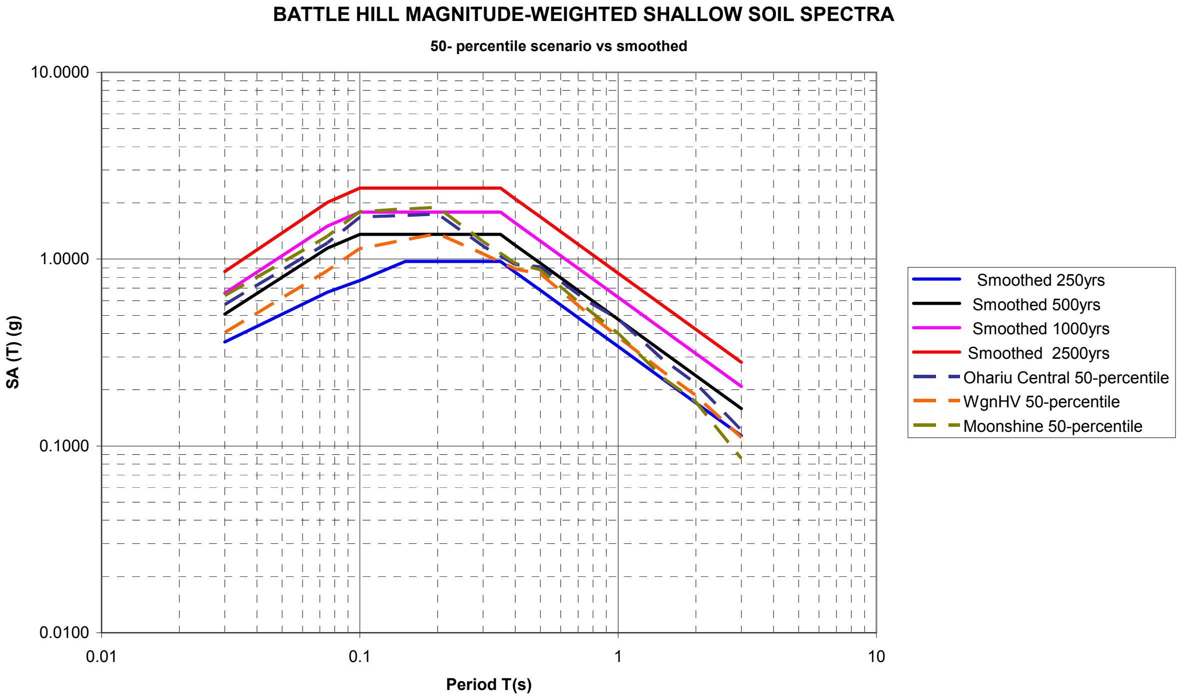



(a)

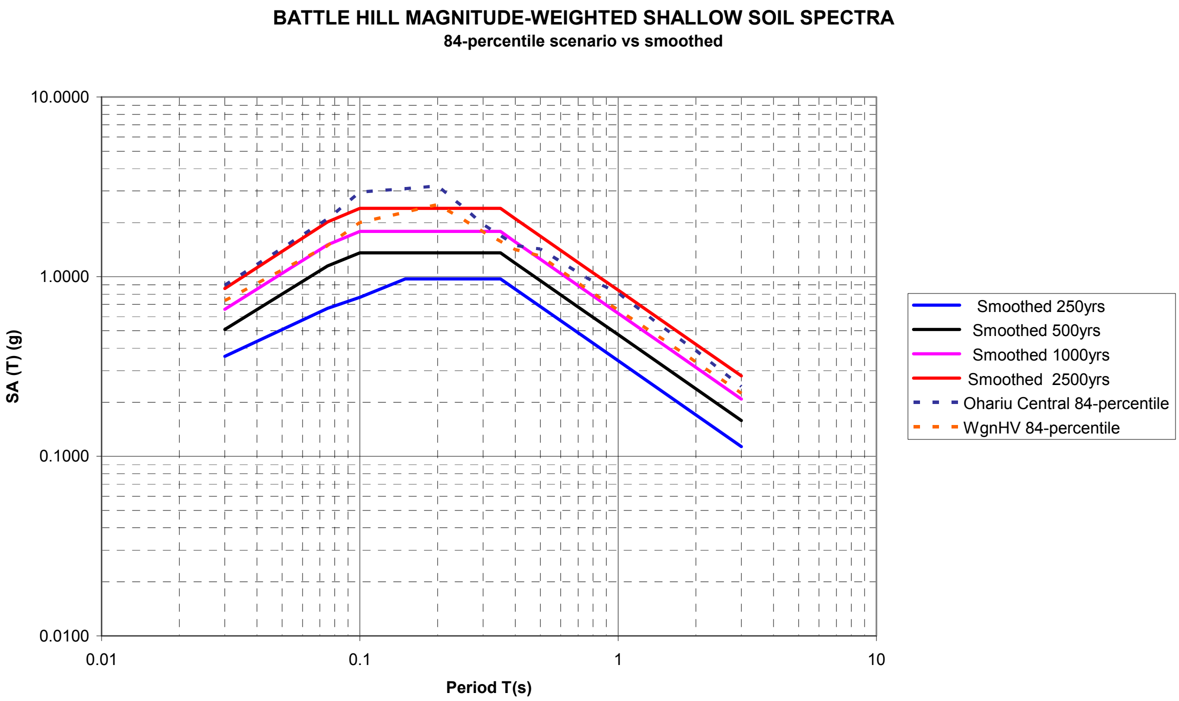



(b)

Figure A10 Comparison of (a) 50-percentile and (b) 84-percentile scenario spectra with smoothed spectra for rock site conditions at Te Puka Stream. Scenario spectra are shown for the Wellington-Hutt Valley fault segment, the central segment of the Ohariu Fault, and the Moonshine Fault.

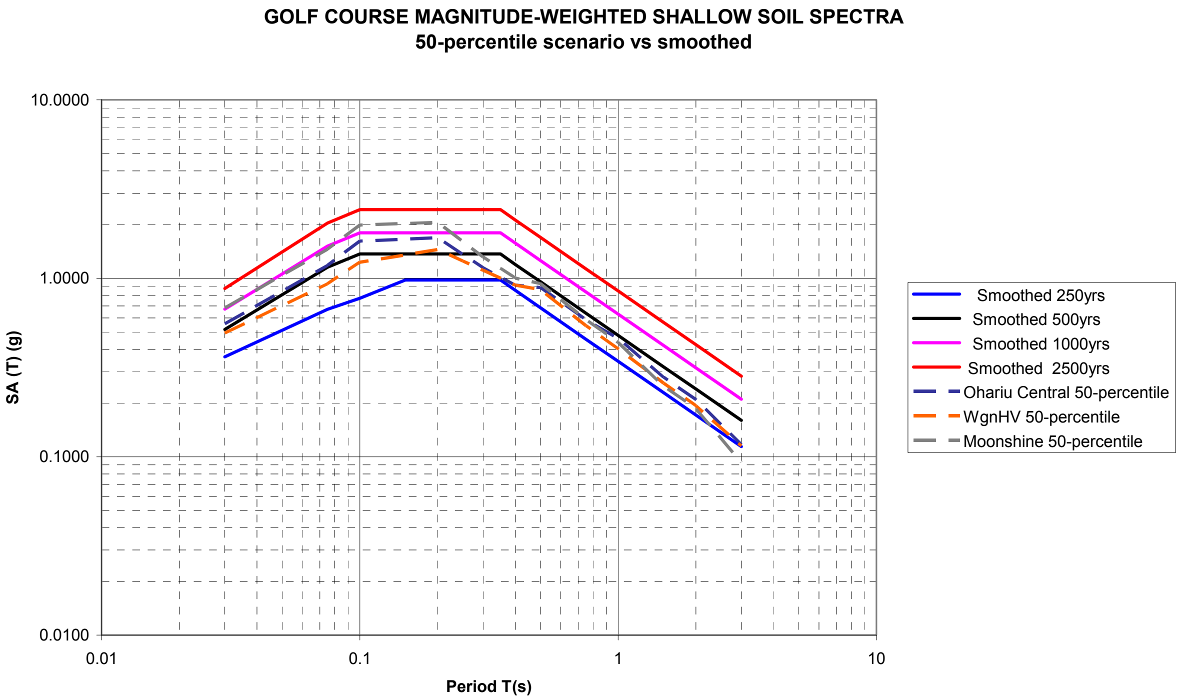



(a)

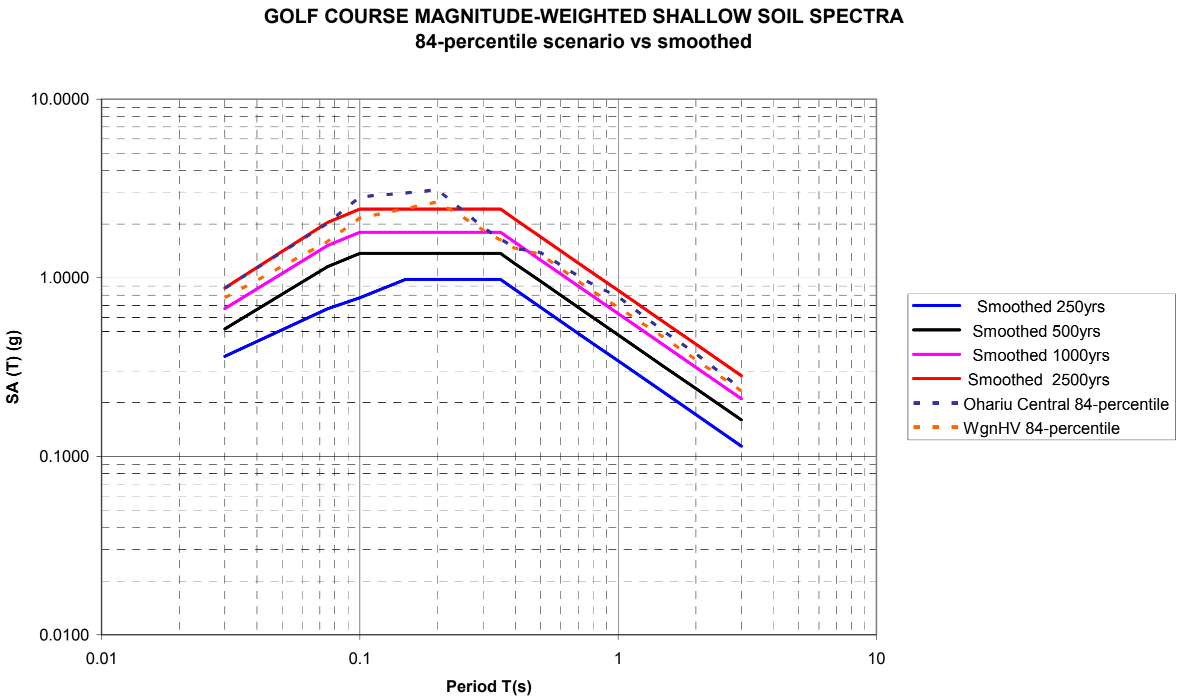



(b)

Figure A11 Comparison of (a) 50-percentile and (b) 84-percentile scenario spectra with smoothed spectra for shallow soil site conditions at Battle Hill. Scenario spectra are shown for the Wellington-Hutt Valley fault segment, the central segment of the Ohariu Fault, and the Moonshine Fault.

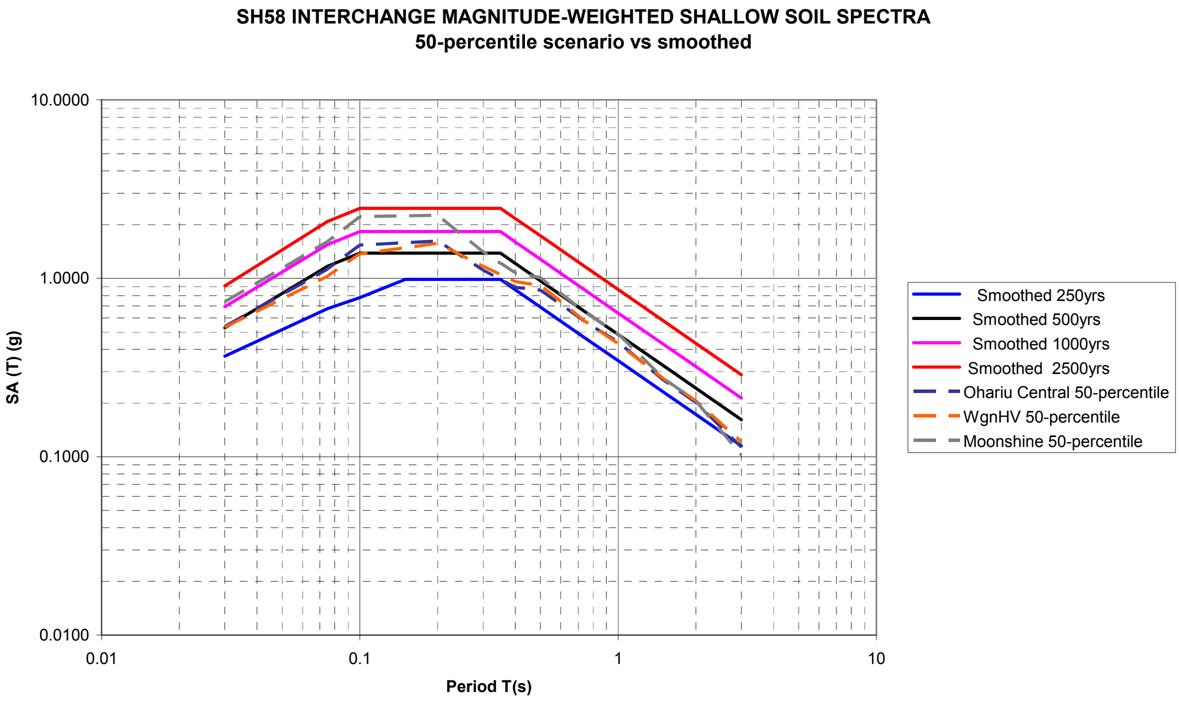



(a)

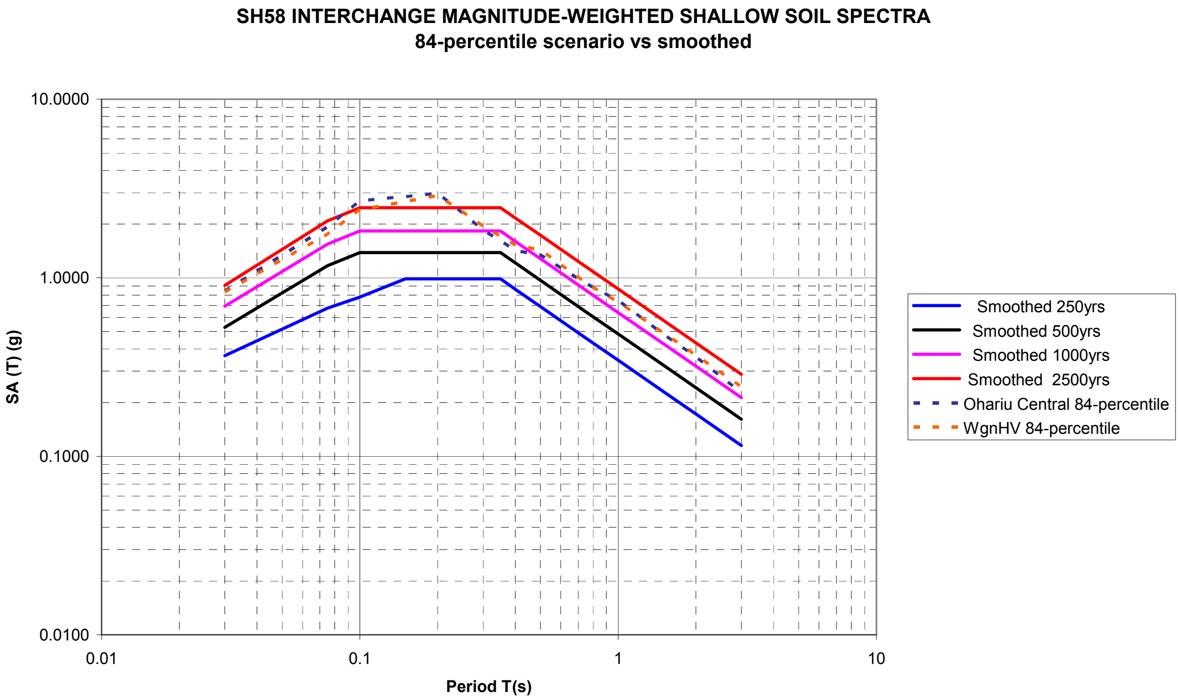



(b)

Figure A12 Comparison of (a) 50-percentile and (b) 84-percentile scenario spectra with smoothed spectra for shallow soil site conditions at the Golf Course. Scenario spectra are shown for the Wellington-Hutt Valley fault segment, the central segment of the Ohariu Fault, and the Moonshine Fault.

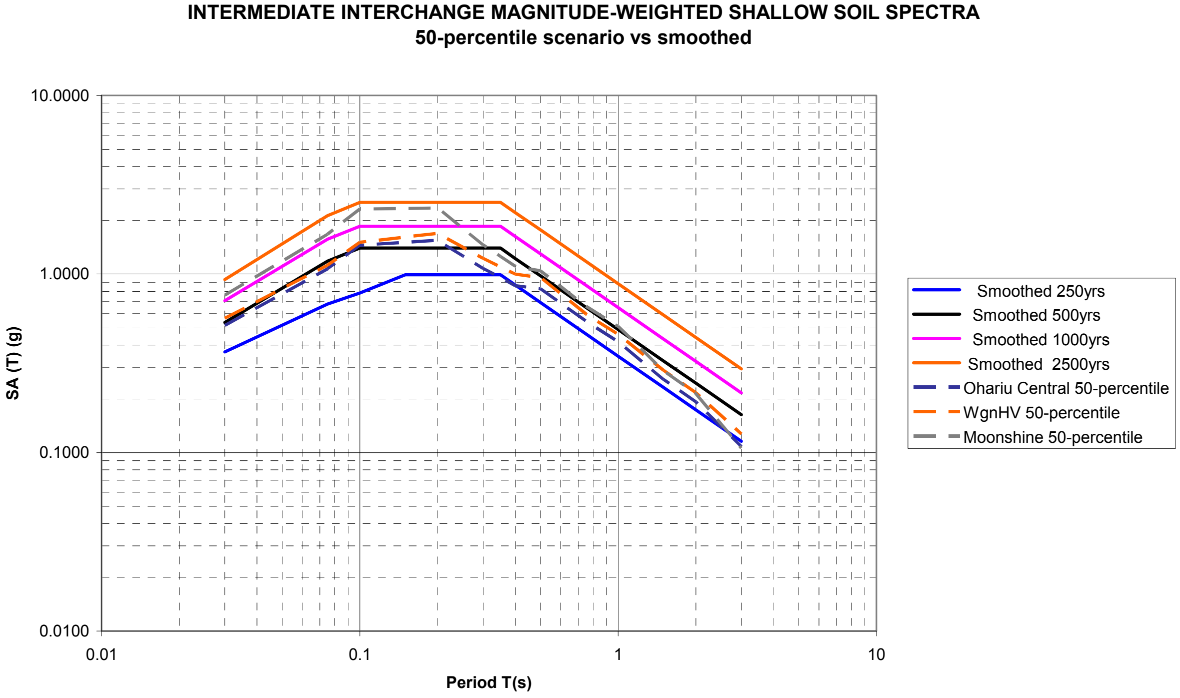



(a)

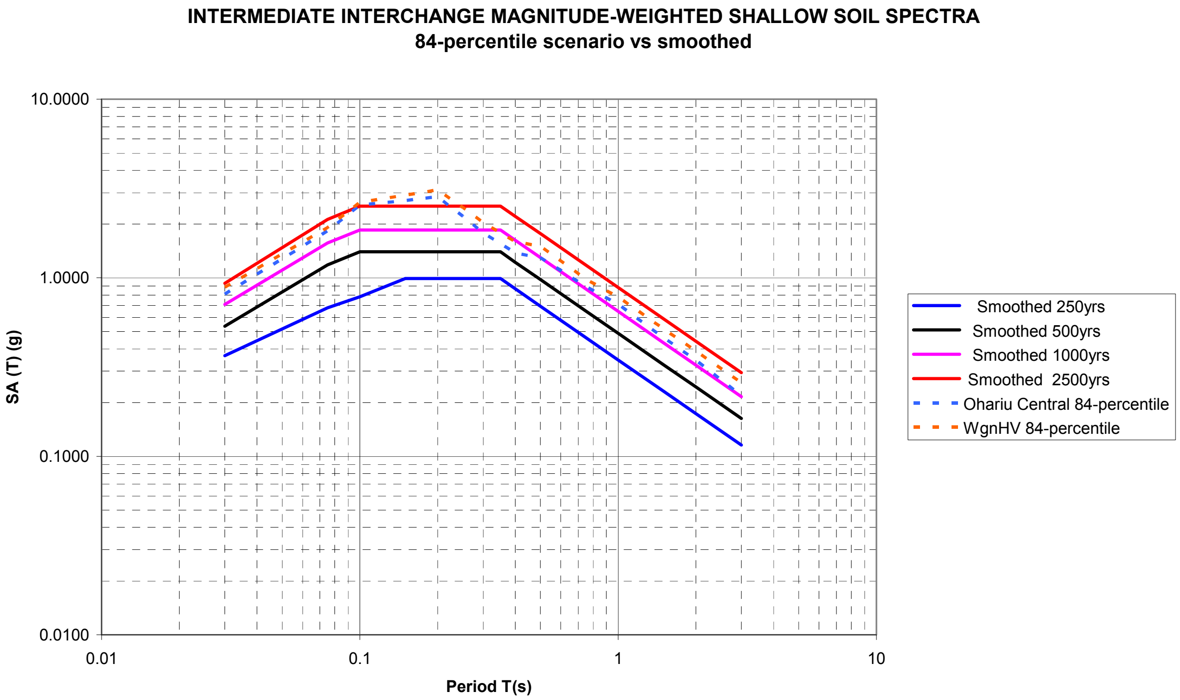



(b)

Figure A13 Comparison of (a) 50-percentile and (b) 84-percentile scenario spectra with smoothed spectra for shallow soil site conditions at SH58Interchange. Scenario spectra are shown for the Wellington-Hutt Valley fault segment, the central segment of the Ohariu Fault, and the Moonshine Fault.

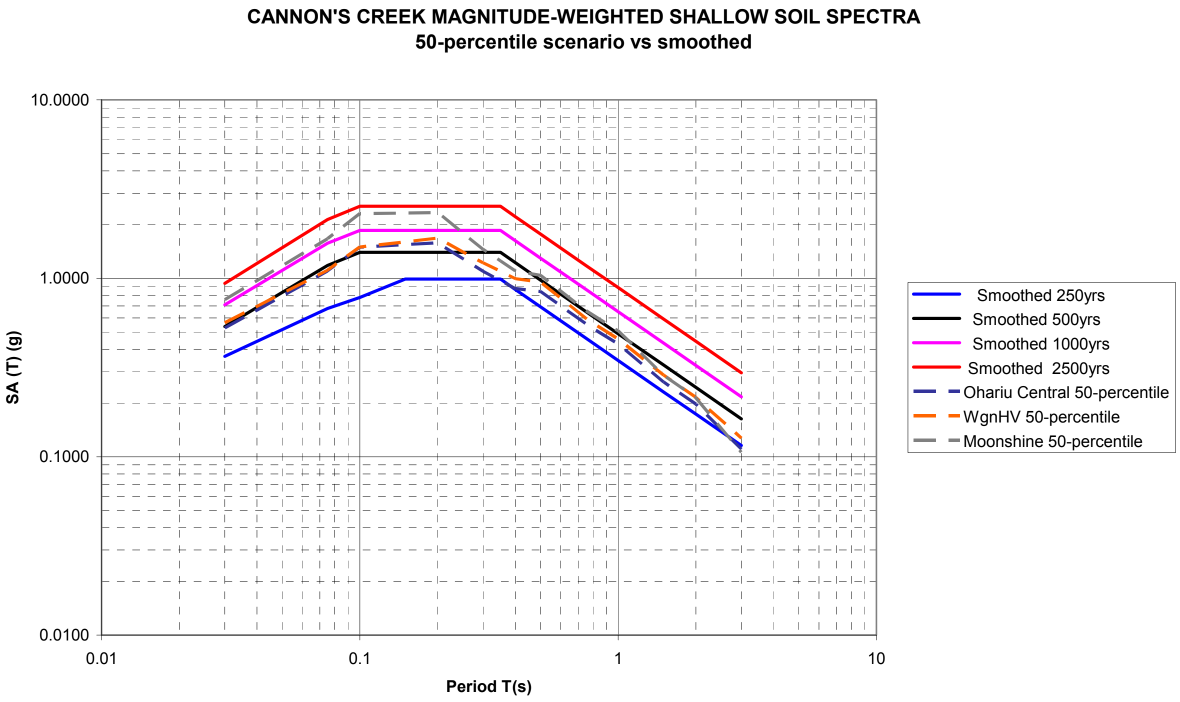



(a)

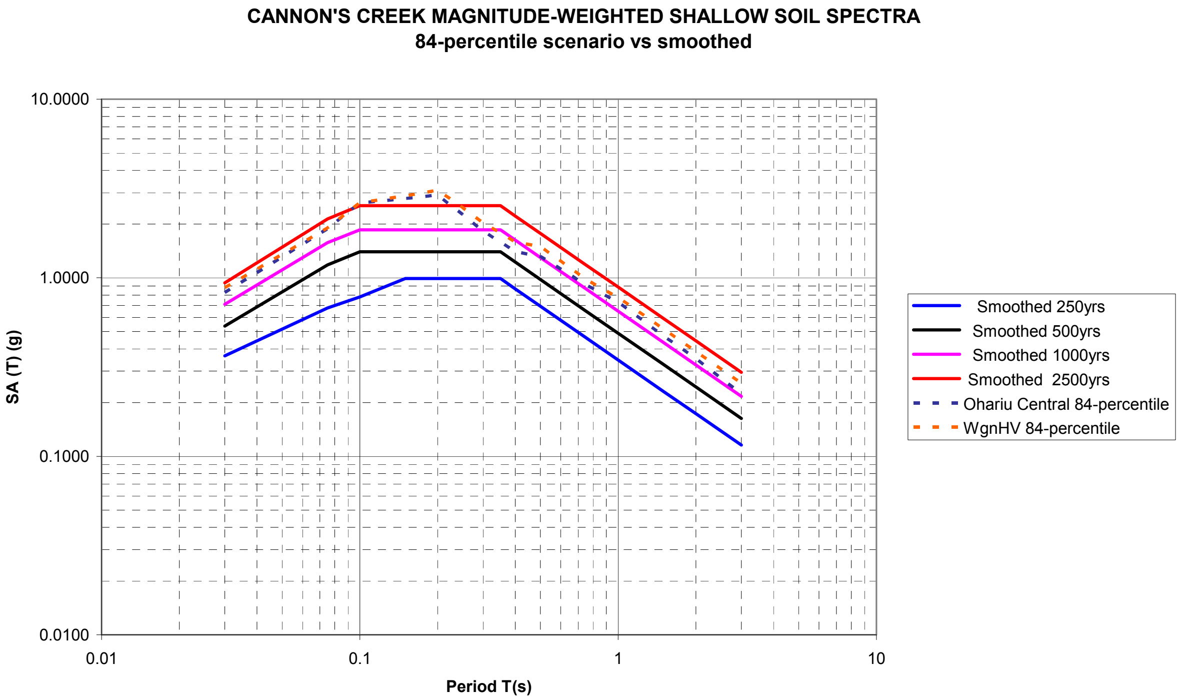



(b)

Figure A14 Comparison of (a) 50-percentile and (b) 84-percentile scenario spectra with smoothed spectra for shallow soil site conditions at the Intermediate Interchange. Scenario spectra are shown for the Wellington-Hutt Valley fault segment, the central segment of the Ohariu Fault, and the Moonshine Fault.

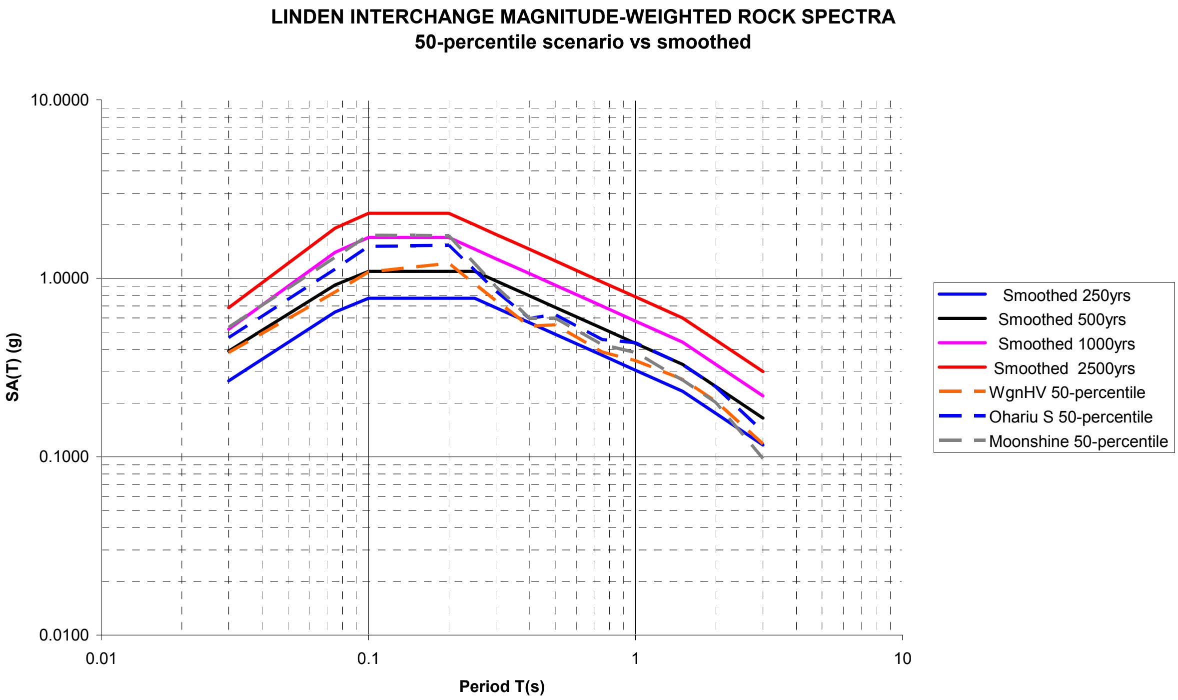



(a)

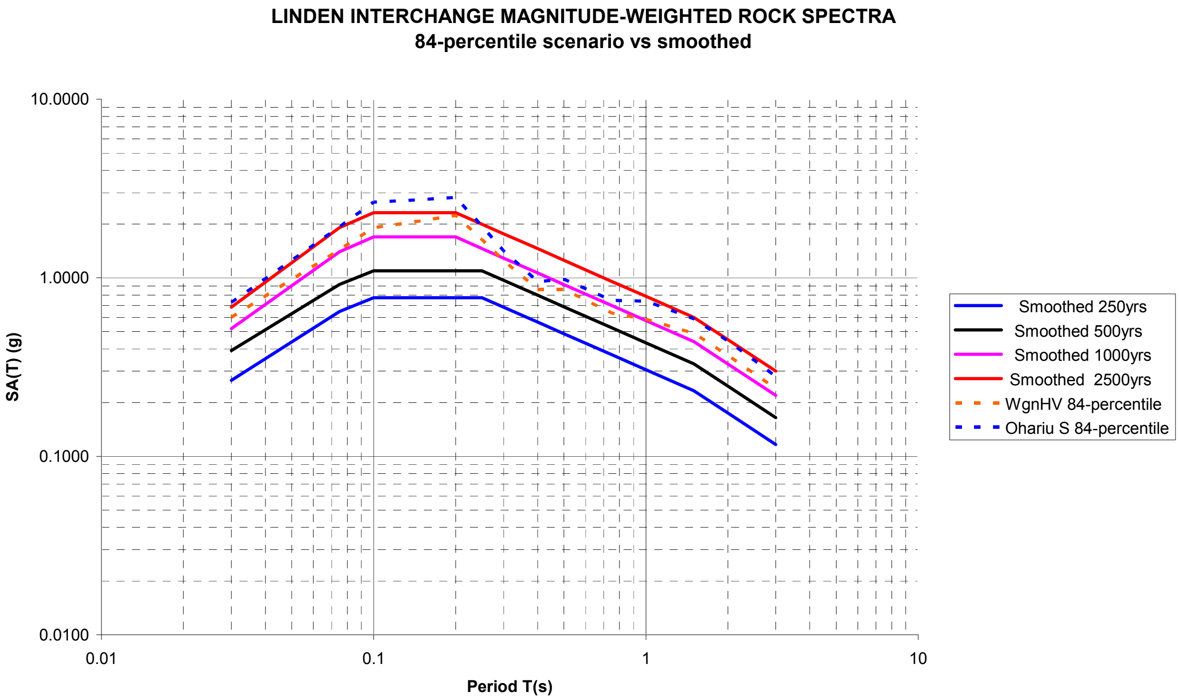



(b)

Figure A15 Comparison of (a) 50-percentile and (b) 84-percentile scenario spectra with smoothed spectra for shallow soil site conditions at Cannon’s Creek. Scenario spectra are shown for the Wellington-Hutt Valley fault segment, the central segment of the Ohariu Fault, and the Moonshine Fault.




(a)




(b)

Figure A16 Comparison of (a) 50-percentile and (b) 84-percentile scenario spectra with smoothed spectra for rock site conditions at Linden SH1 Interchange, at the southern end of the route. Scenario spectra are shown for the Wellington-Hutt Valley fault segment, the south segment of the Ohariu Fault, and the Moonshine Fault.

## A3 HAZARD MAPS FOR THE REGION AROUND TRANSMISSION GULLY

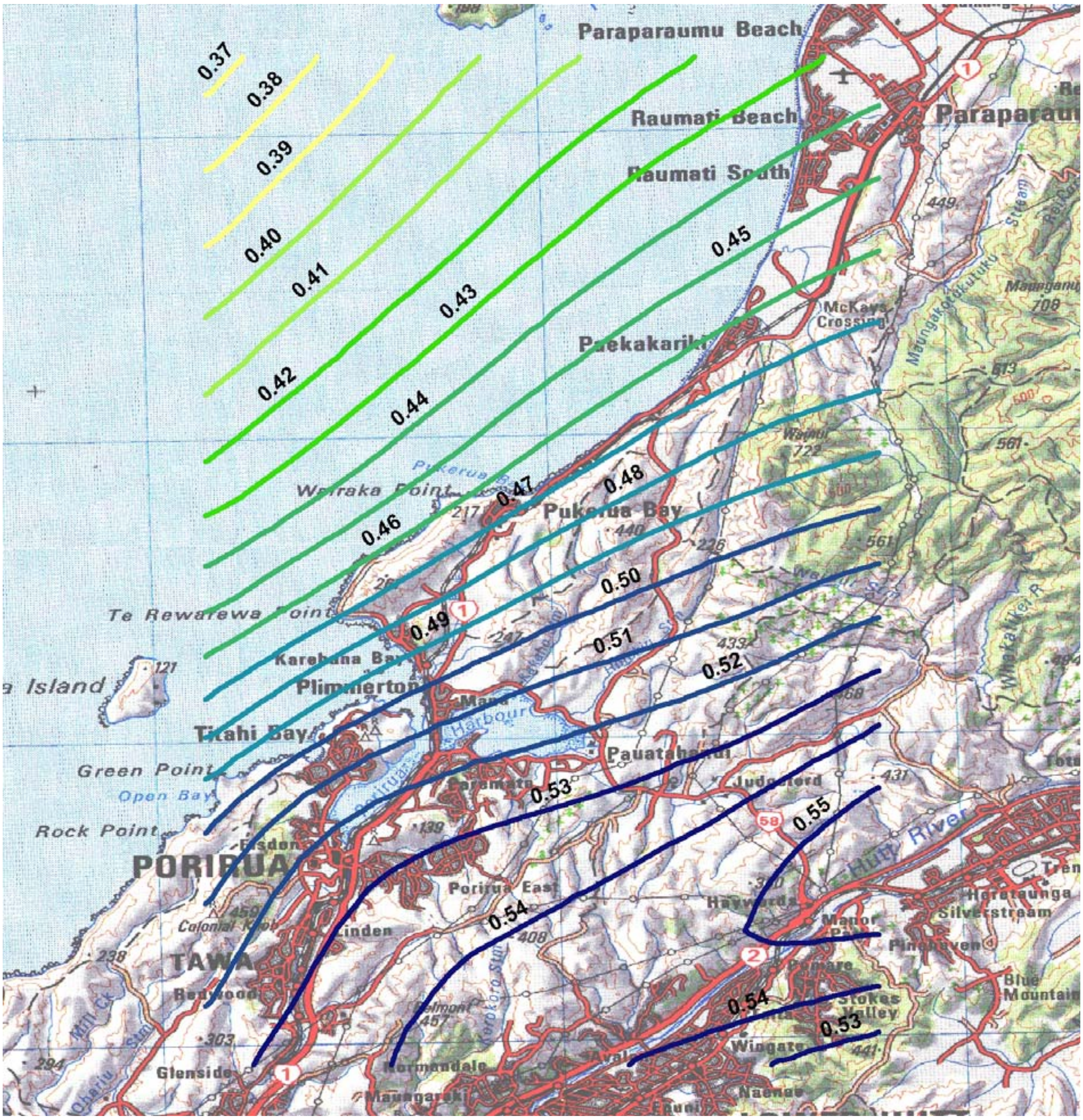



Figure A17 Contours of 500-year magnitude-weighted peak ground accelerations for shallow soil site conditions. Note that the contours are generally aligned in the direction of the Wellington Fault, which runs along the Hutt Valley, bending around in the south-western region to be aligned with the southern segment of the Ohariu Fault in the Porirua-Tawa area.

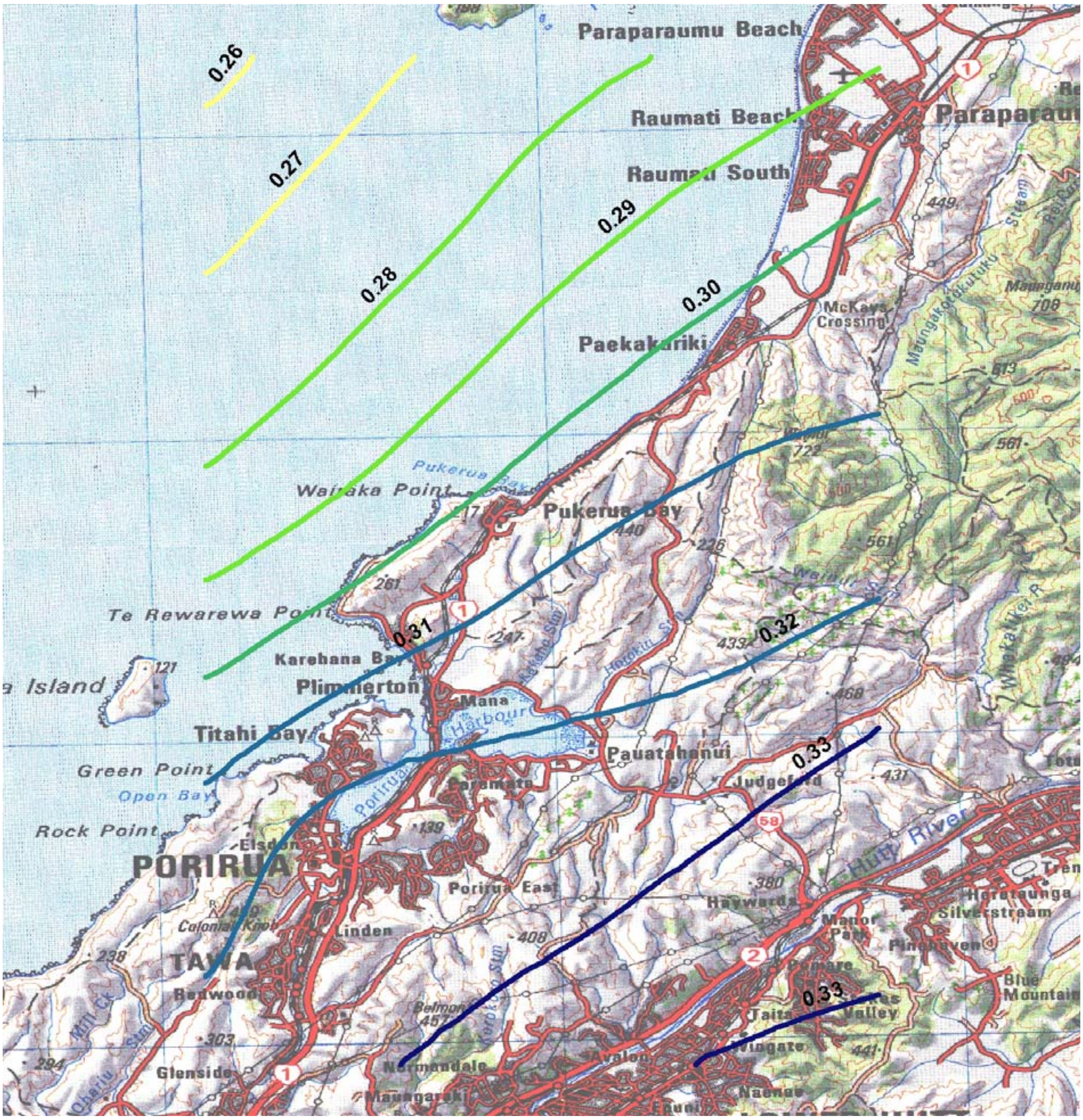



Figure A18    Contours of 500-year 5% damped response spectral accelerations for 1.5s period (SA(1.5s)) for shallow soil site conditions.

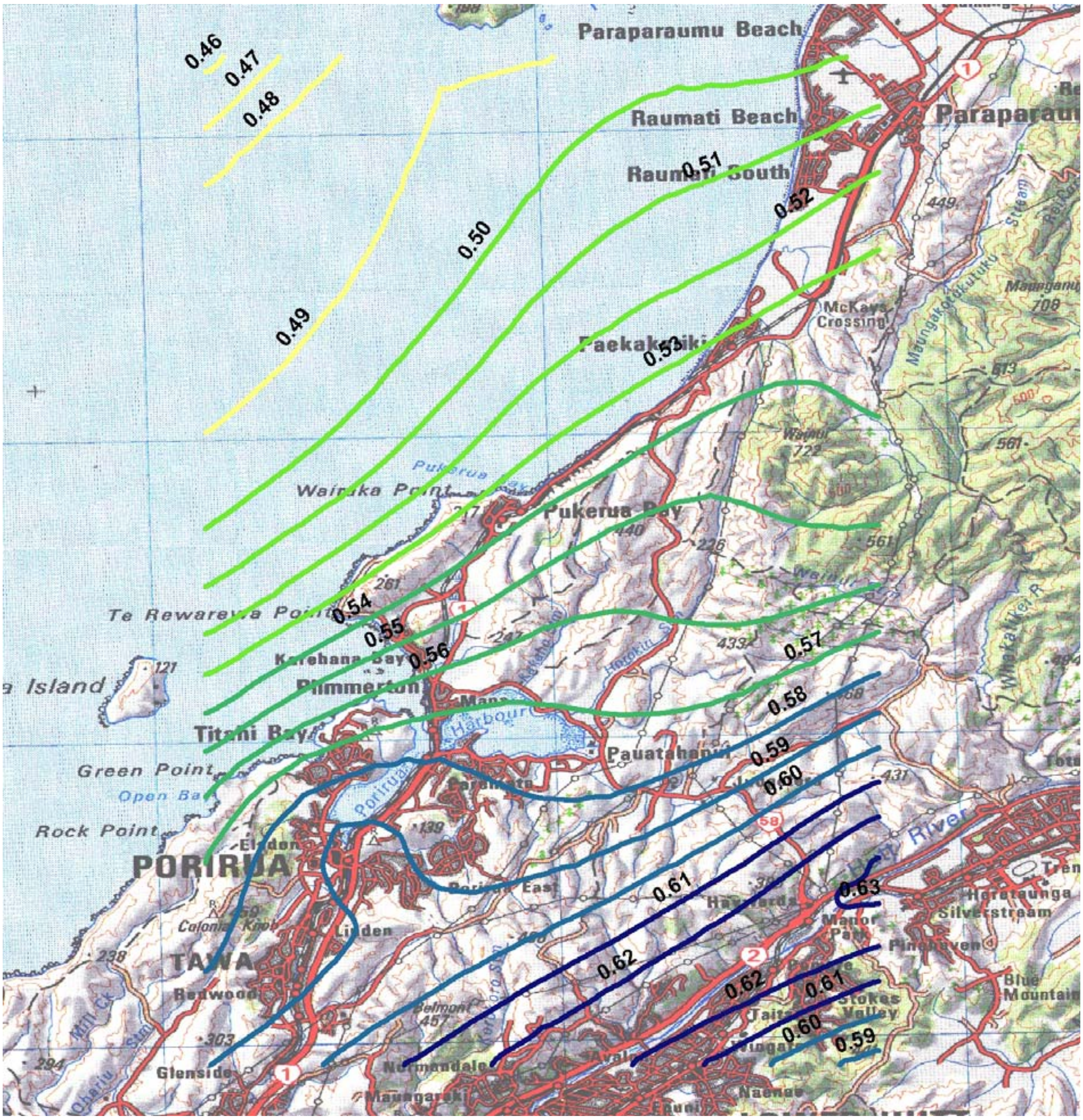



Figure A19 Contours of 2500-year 5% damped response spectral accelerations for 1.5s period (SA(1.5s)) for shallow soil site conditions.

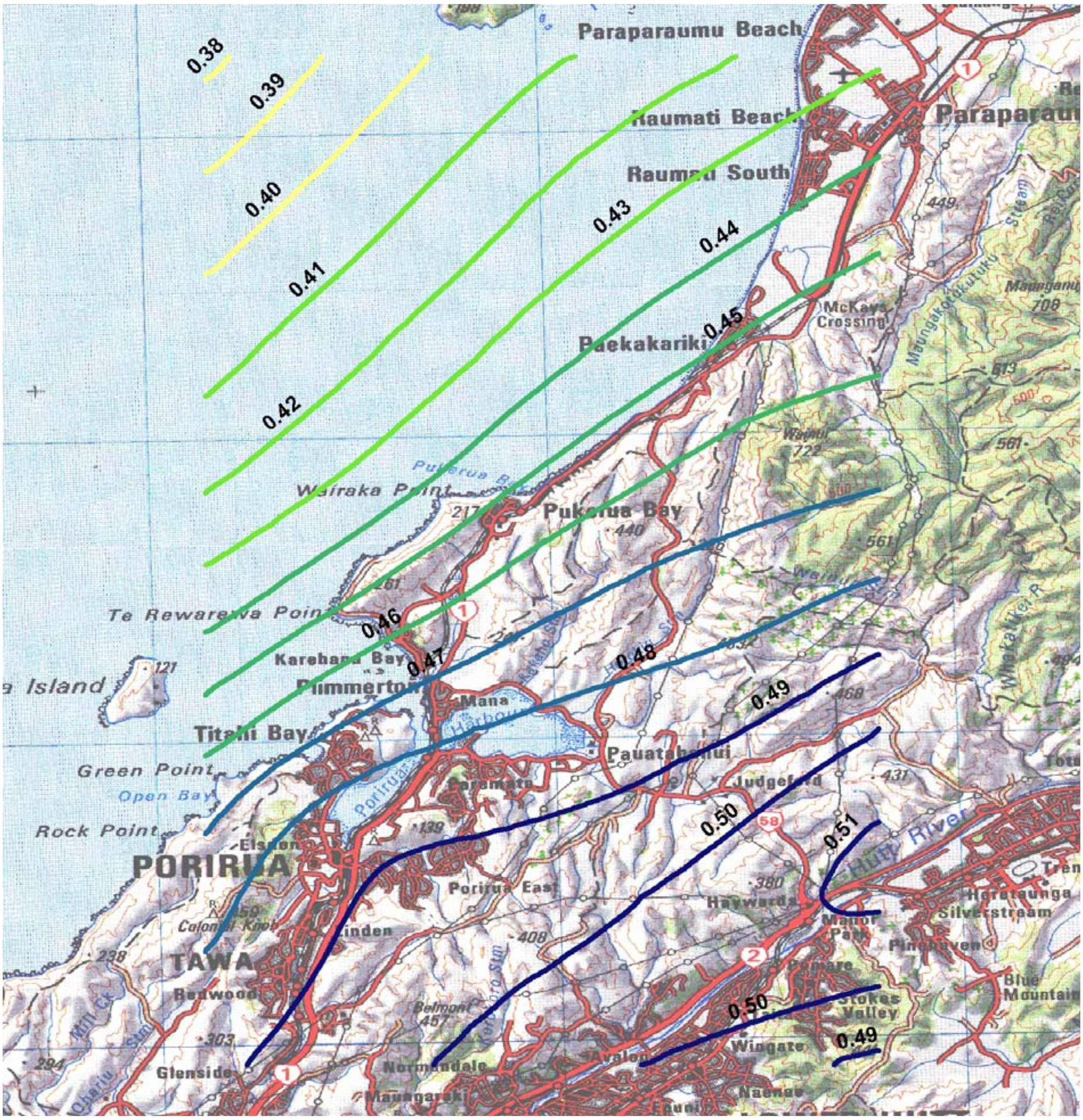



Figure A20 Contours of the equivalent of the NZS1170 Hazard Factor Z (i.e. 0.5 times the 500-year magnitude-weighted 5% damped response spectral accelerations for 0.5s period, SA(0.5s), for shallow soil site conditions).

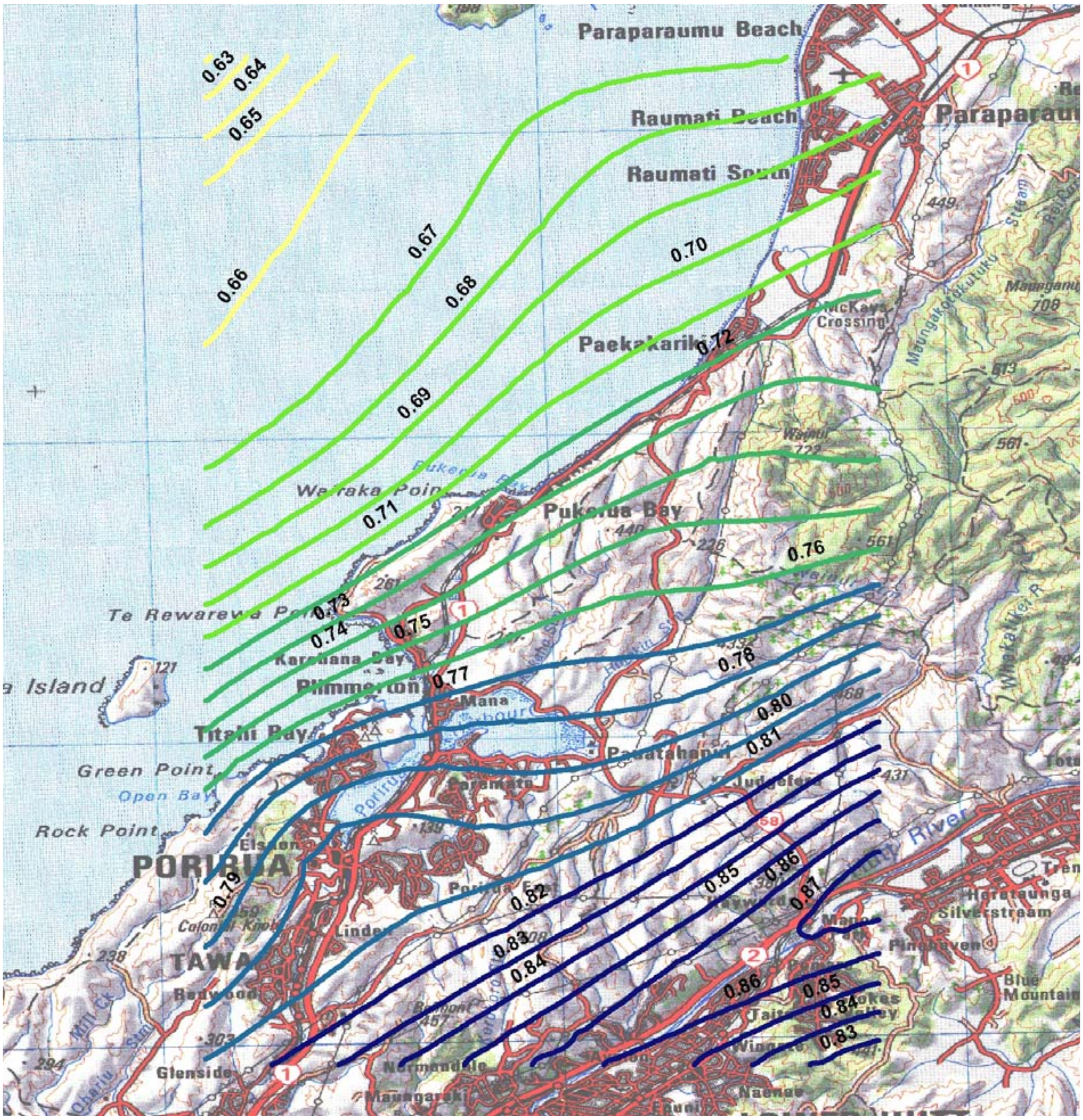



Figure A21 Contours of the equivalent of the product of the NZS1170 Hazard Factor Z and Return Period Factor R for a return period of 2500 years (i.e. 0.5 times the 2500-year magnitude-weighted 5% damped response spectral accelerations for 0.5s period, SA(0.5s), for shallow soil site conditions). Note that these values exceed the maximum required value of ZR=0.7 specified in NZS1170.

# A4 NEAR-FAULT FACTORS

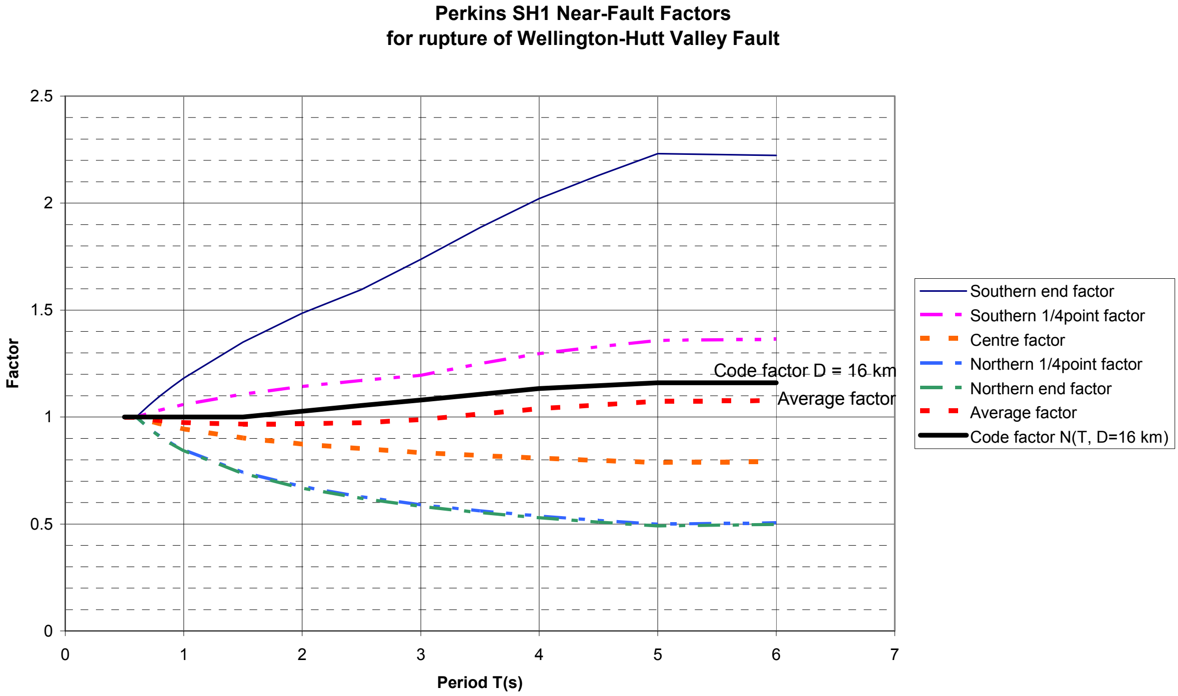



Figure A22 Near-Fault factors for the SH1 Perkins site for ruptures initiating at the quarter-points of the Wellington-Hutt Valley Fault segment, together with the average of the five factors and the NZS1170 factor N(T, D=16km) for the 16 km distance of the site from the Wellington Fault.

**Table A1** Average and Maximum Near-Fault Factors at SH1 Perkins for Rupture of the Wellington-Hutt Valley and Central Ohariu fault segments

| Period T | 0.5s | 0.75s | 1.0s | 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|---|---|---|
| Average Factor for Wellington-Hutt Valley Fault $NF_{ave}(T)$ | 1.0 | 0.99 | 0.97 | 0.97 | 0.97 | 0.99 | 1.04 | 1.07 |
| Maximum Factor for Wellington-Hutt Valley Fault $NF_{max}(T)$ x/L=0.875 | 1.0 | 1.08 | 1.18 | 1.35 | 1.48 | 1.74 | 2.02 | 2.23 |
| NZS1170 Factor for SH1 Perkins Code N(T, D=16 km) | 1.0 | 1.0 | 1.0 | 1.0 | 1.03 | 1.08 | 1.13 | 1.16 |
| Maximum Factor for Central Ohariu Segment D=1 km x/L=0.5 | 1.0 | 1.01 | 1.02 | 1.04 | 1.12 | 1.33 | 1.52 | 1.56 |

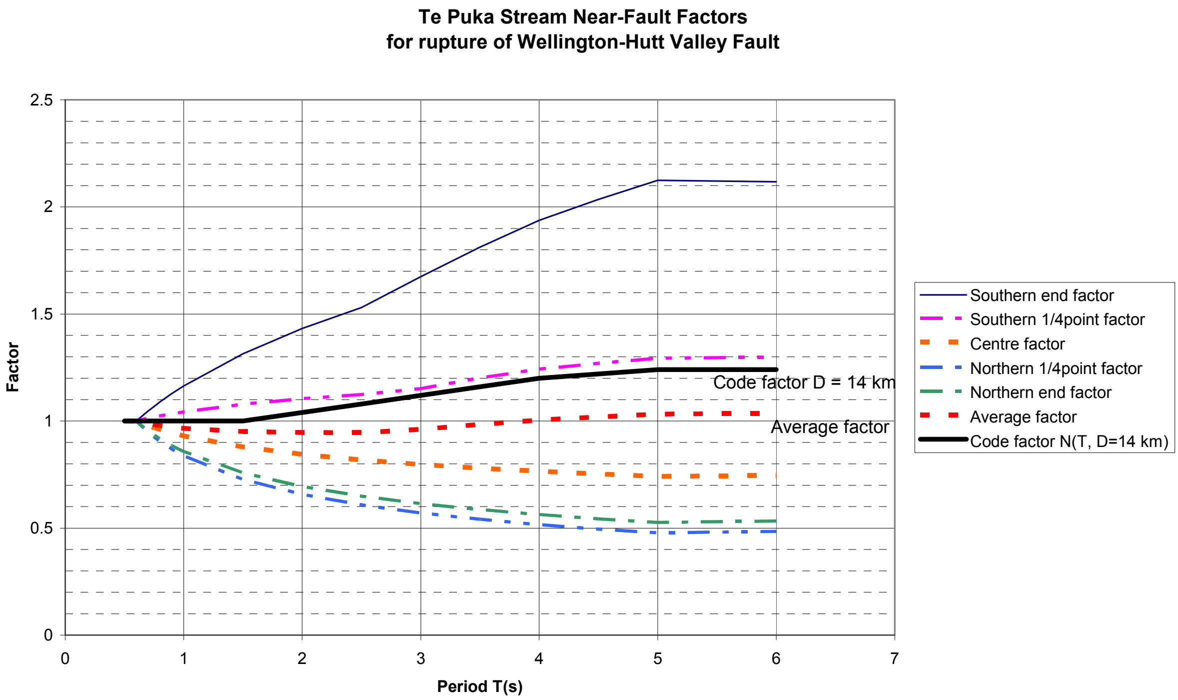



Figure A23 Near-Fault factors for the Te Puka site for ruptures initiating at the quarter-points of the Wellington-Hutt Valley Fault segment, together with the average of the five factors and the NZS1170 factor N(T, D=14km) for the 14 km distance of the site from the Wellington Fault.

**Table A2** Average and Maximum Near-Fault Factors at Te Puka for Rupture of the Wellington-Hutt Valley and Central Ohariu fault segments

| Period T | 0.5s | 0.75s | 1.0s | 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|---|---|---|
| Average Factor for Wellington-Hutt Valley Fault $NF_{ave}(T)$ | 1.0 | 0.98 | 0.97 | 0.95 | 0.95 | 0.96 | 1.01 | 1.03 |
| Maximum Factor for Wellington-Hutt Valley Fault $NF_{max}(T)$ x/L=0.84 | 1.0 | 1.07 | 1.16 | 1.31 | 1.43 | 1.67 | 1.94 | 2.12 |
| NZS1170 Factor for Te Puka Code N(T, D=14 km) | 1.0 | 1.0 | 1.0 | 1.0 | 1.04 | 1.12 | 1.20 | 1.24 |
| Maximum Factor for Central Ohariu Fault Segment D=0 km, x/L=0.61 | 1.0 | 1.03 | 1.07 | 1.15 | 1.31 | 1.66 | 1.97 | 2.06 |

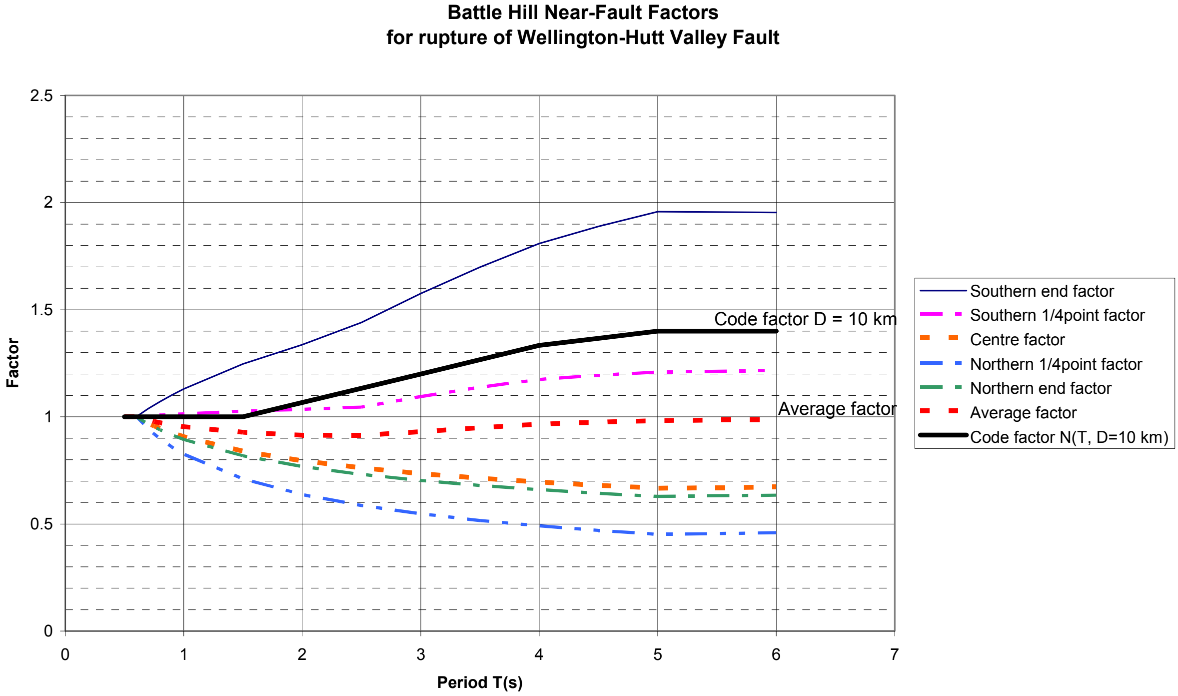



Figure A24 Near-Fault factors for Battle Hill for ruptures initiating at the quarter-points of the Wellington-Hutt Valley Fault segment, together with the average of the five factors and the NZS1170 factor N(T, D=10km) for the 10 km distance of the site from the Wellington Fault.

**Table A3** Average and Maximum Near-Fault Factors at Battle Hill for Rupture of the Wellington-Hutt Valley and Central Ohariu fault segments

| Period T | 0.5s | 0.75s | 1.0s | 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|---|---|---|
| Average Factor for Wellington-Hutt Valley Fault $NF_{ave}(T)$ | 1.0 | 0.98 | 0.95 | 0.93 | 0.91 | 0.93 | 0.97 | 0.98 |
| Maximum Factor for Wellington-Hutt Valley Fault $NF_{max}(T)$ x/L=0.76 | 1.0 | 1.05 | 1.13 | 1.25 | 1.34 | 1.58 | 1.81 | 1.96 |
| NZS1170 Factor for Battle Hill Code N(T, D=10 km) | 1.0 | 1.0 | 1.0 | 1.0 | 1.07 | 1.20 | 1.33 | 1.40 |
| Maximum Factor for Central Ohariu Fault Segment D=2 km, x/L=0.72 | 1.0 | 1.05 | 1.12 | 1.22 | 1.35 | 1.69 | 1.99 | 2.12 |

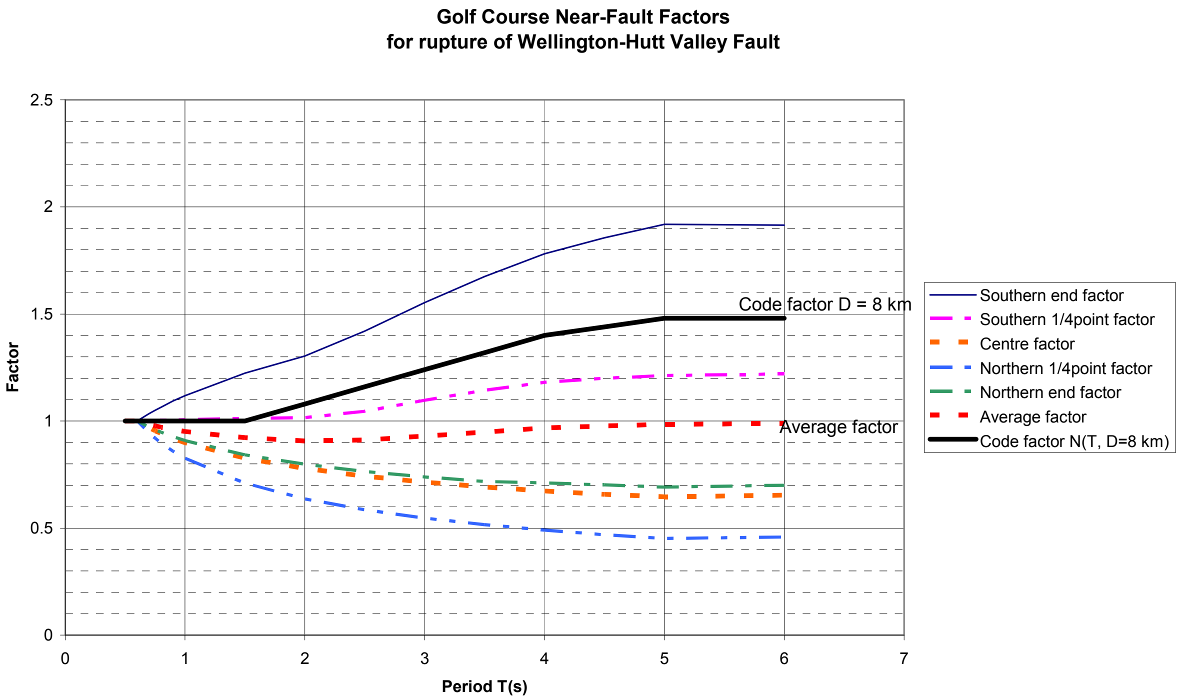



Figure A25 Near-Fault factors for the Golf Course for ruptures initiating at the quarter-points of the Wellington-Hutt Valley Fault segment, together with the average of the five factors and the NZS1170 factor N(T, D=8 km) for the 8 km distance of the site from the Wellington Fault.

**Table A4** Average and Maximum Near-Fault Factors at the Golf Course for Rupture of the Wellington-Hutt Valley and Central Ohariu fault segments

| Period T | 0.5s | 0.75s | 1.0s | 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|---|---|---|
| Average Factor for Wellington-Hutt Valley Fault $NF_{ave}(T)$ | 1.0 | 0.98 | 0.95 | 0.92 | 0.91 | 0.93 | 0.97 | 0.98 |
| Maximum Factor for Wellington-Hutt Valley Fault $NF_{max}(T)$ x/L=0.74 | 1.0 | 1.05 | 1.12 | 1.22 | 1.30 | 1.55 | 1.78 | 1.92 |
| NZS1170 Factor for the Golf Course Code N(T, D=8 km) | 1.0 | 1.0 | 1.0 | 1.0 | 1.08 | 1.24 | 1.40 | 1.48 |
| Maximum Factor for Central Ohariu Fault Segment D=3 km, x/L=0.79 | 1.0 | 1.06 | 1.15 | 1.28 | 1.41 | 1.77 | 2.09 | 2.26 |

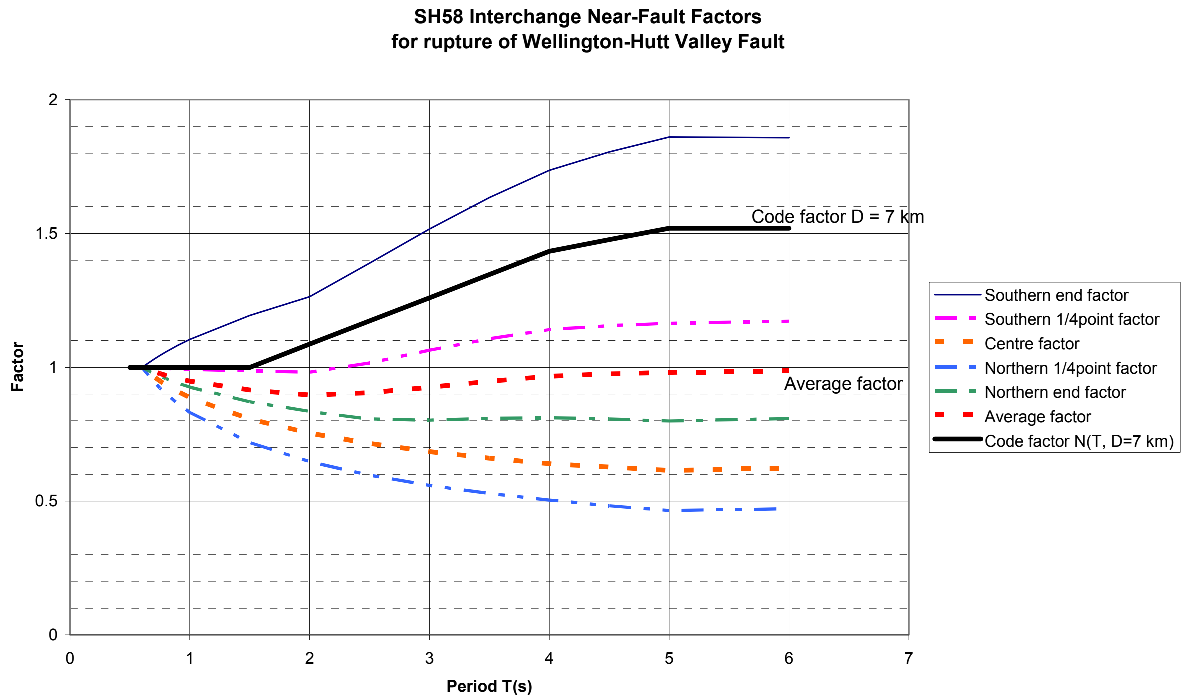



Figure A26 Near-Fault factors for the SH58 Interchange for ruptures initiating at the quarter-points of the Wellington-Hutt Valley Fault segment, together with the average of the five factors and the NZS1170 factor N(T, D=7 km) for the 7 km distance of the site from the Wellington Fault.

**Table A5** Average and Maximum Near-Fault Factors at the SH58 Interchange for Rupture of the Wellington-Hutt Valley and Central Ohariu fault segments

| Period T | 0.5s | 0.75s | 1.0s | 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|---|---|---|
| Average Factor for Wellington-Hutt Valley Fault $NF_{ave}(T)$ | 1.0 | 0.98 | 0.95 | 0.92 | 0.90 | 0.93 | 0.97 | 0.98 |
| Maximum Factor for Wellington-Hutt Valley Fault $NF_{max}(T)$ x/L=0.7 | 1.0 | 1.04 | 1.10 | 1.19 | 1.26 | 1.52 | 1.74 | 1.86 |
| NZS1170 Factor for the SH58 Interchange Code N(T, D=7 km) | 1.0 | 1.0 | 1.0 | 1.0 | 1.09 | 1.26 | 1.43 | 1.52 |
| Maximum Factor for Central Ohariu Fault Segment D=4 km, x/L=0.87 | 1.0 | 1.08 | 1.19 | 1.36 | 1.50 | 1.93 | 2.31 | 2.53 |

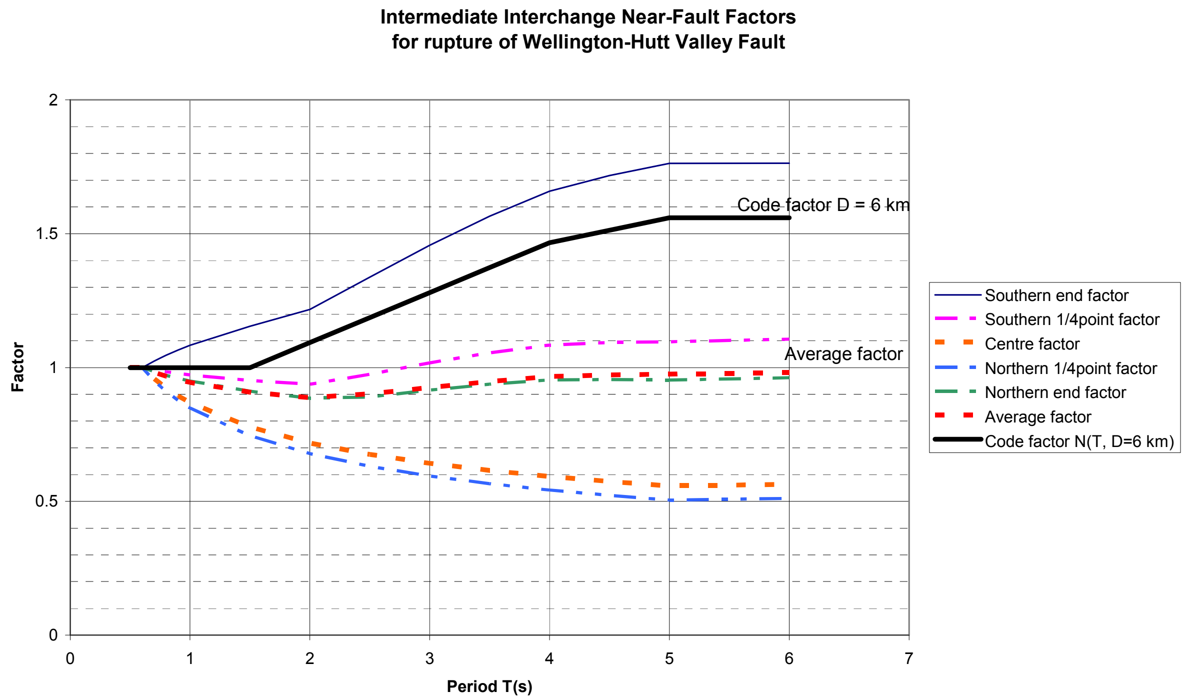



Figure A27 Near-Fault factors for the Intermediate Interchange for ruptures initiating at the quarter-points of the Wellington-Hutt Valley Fault segment, together with the average of the five factors and the NZS1170 factor N(T, D=6 km) for the 6 km distance of the site from the Wellington Fault.

**Table A6** Average and Maximum Near-Fault Factors at the Intermediate Interchange for Rupture of the Wellington-Hutt Valley and Central Ohariu fault segments

| Period T | 0.5s | 0.75s | 1.0s | 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|---|---|---|
| Average Factor for Wellington-Hutt Valley Fault $NF_{ave}(T)$ | 1.0 | 0.97 | 0.94 | 0.91 | 0.89 | 0.93 | 0.97 | 0.97 |
| Maximum Factor for Wellington-Hutt Valley Fault $NF_{max}(T)$ x/L=0.65 | 1.0 | 1.04 | 1.08 | 1.15 | 1.22 | 1.46 | 1.66 | 1.76 |
| NZS1170 Factor for the Intermediate Interchange Code N(T, D=6 km) | 1.0 | 1.0 | 1.0 | 1.0 | 1.09 | 1.28 | 1.60 | 1.72 |
| Maximum Factor for Central Ohariu Fault Segment D=4 km, x/L=0.75 | 1.0 | 1.10 | 1.25 | 1.50 | 1.71 | 2.29 | 2.82 | 3.17 |

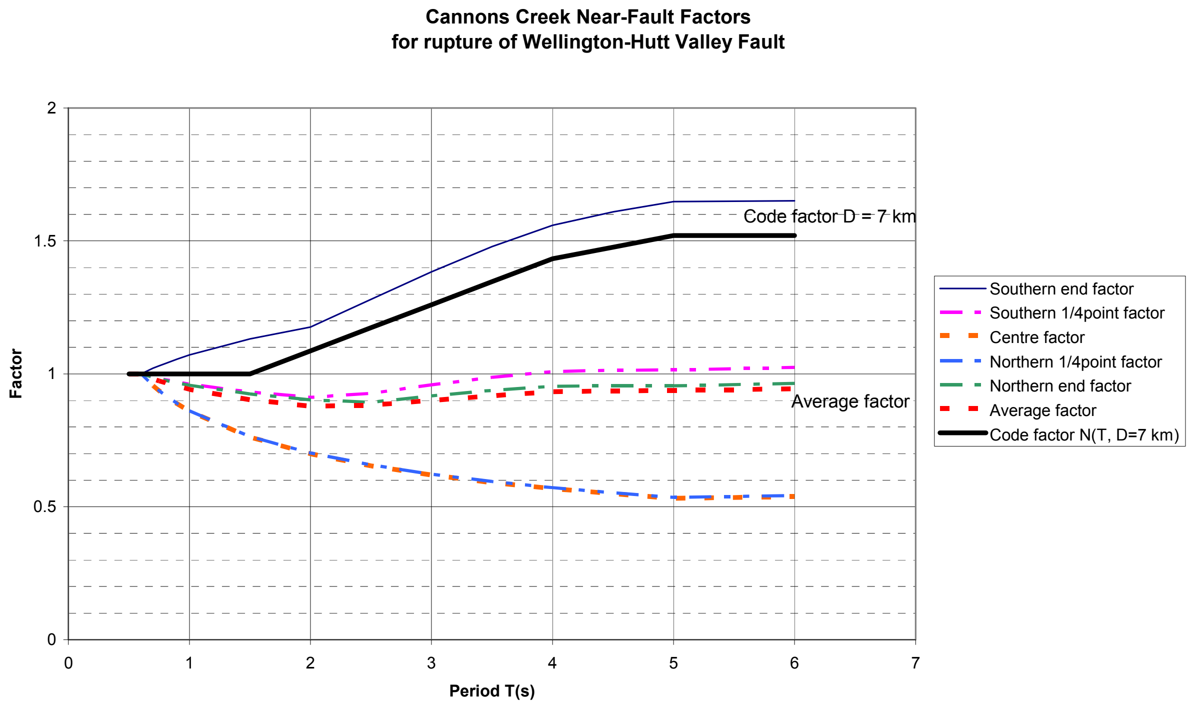



Figure A28 Near-Fault factors for Cannon's Creek for ruptures initiating at the quarter-points of the Wellington-Hutt Valley Fault segment, together with the average of the five factors and the NZS1170 factor N(T, D=7 km) for the 7 km distance of the site from the Wellington Fault.

**Table A7** Average and Maximum Near-Fault Factors at Cannon's Creek for Rupture of the Wellington-Hutt Valley and Central Ohariu fault segments

| Period T | 0.5s | 0.75s | 1.0s | 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|---|---|---|
| Average Factor for Wellington-Hutt Valley Fault $NF_{ave}(T)$ | 1.0 | 0.97 | 0.94 | 0.90 | 0.88 | 0.90 | 0.93 | 0.94 |
| Maximum Factor for Wellington-Hutt Valley Fault $NF_{max}(T)$ x/L=0.625 | 1.0 | 1.03 | 1.07 | 1.13 | 1.18 | 1.38 | 1.56 | 1.65 |
| NZS1170 Factor for Cannon's Creek Code N(T, D=7 km) | 1.0 | 1.0 | 1.0 | 1.0 | 1.09 | 1.26 | 1.43 | 1.52 |
| Maximum Factor for Central Ohariu Fault Segment D=4 km, x/L=1.0 | 1.0 | 1.10 | 1.25 | 1.50 | 1.72 | 2.32 | 2.87 | 3.23 |

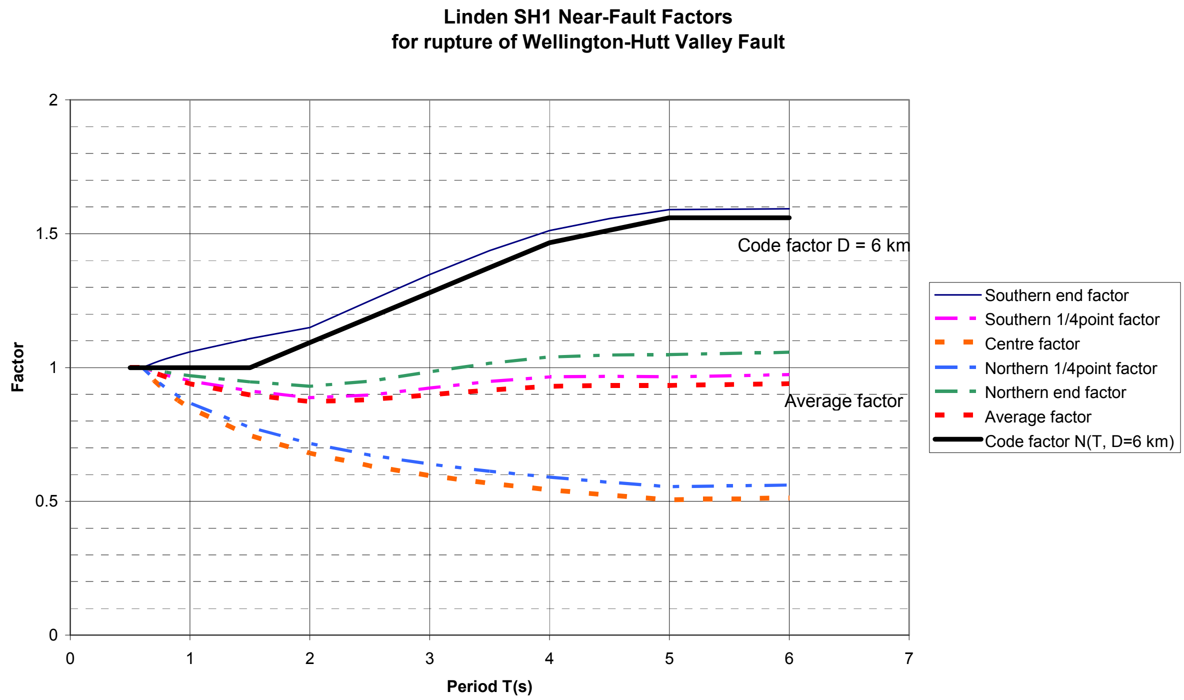



Figure A29 Near-Fault factors for the Linden SH1 Interchange for ruptures initiating at the quarter-points of the Wellington-Hutt Valley Fault segment, together with the average of the five factors and the NZS1170 factor N(T, D=6 km) for the 6 km distance of the site from the Wellington Fault.

**Table A8** Average and Maximum Near-Fault Factors at the Linden SH1 for Rupture of the Wellington-Hutt Valley and Central Ohariu fault segments

| Period T | 0.5s | 0.75s | 1.0s | 1.5s | 2s | 3s | 4s | ≥5s |
|---|---|---|---|---|---|---|---|---|
| Average Factor for Wellington-Hutt Valley Fault $NF_{ave}(T)$ | 1.0 | 0.97 | 0.94 | 0.90 | 0.88 | 0.90 | 0.93 | 0.93 |
| Maximum Factor for Wellington-Hutt Valley Fault $NF_{max}(T)$ x/L=0.6 | 1.0 | 1.03 | 1.06 | 1.11 | 1.15 | 1.35 | 1.51 | 1.59 |
| NZS1170 Factor for Linden SH1 Code N(T, D=6 km) | 1.0 | 1.0 | 1.0 | 1.0 | 1.09 | 1.28 | 1.47 | 1.56 |
| Maximum Factor for Ohariu South Fault Segment D=1.5 km, x/L=0.95 | 1.0 | 1.09 | 1.23 | 1.45 | 1.73 | 2.38 | 2.98 | 3.33 |

www.gns.cri.nz

Principal Location

1 Fairway Drive
Avalon
PO Box 30368
Lower Hutt
New Zealand
T +64-4-570 1444
F +64-4-570 4600

Other Locations

Dunedin Research Centre
764 Cumberland Street
Private Bag 1930
Dunedin
New Zealand
T +64-3-477 4050
F +64-3-477 5232

Wairakei Research Centre
114 Karetoto Road
Wairakei
Private Bag 2000, Taupo
New Zealand
T +64-7-374 8211
F +64-7-374 8199

National Isotope Centre
30 Gracefield Road
PO Box 31312
Lower Hutt
New Zealand
T +64-4-570 1444
F +64-4-570 4657