Appendix F

Laboratory Test Reports

PARTICLE SIZE ANALYSIS
TEST REPORT

Project:	**P2N Cycleway**
Location:	**Petone foreshore**
Client:	**NZTA**
Contractor:	**JAD Civil Design**
Sampled by:	**Aecom**
Date sampled:	**17.12.13**
Sampling method:	**Test pit**
Sample source:	**TP2 1.20m**
Sample description:	**GRAVEL: f-vc, brown, with sand**
Sample condition:	**As received**
Solid density	**n/a** t/m^3 **Assumed**
Water content as rec'd	**7.2** % **whole**

Report No:	**522900/1078**
Sample No:	**2-13/400**
Client Ref:	**60306339/7.02**

Sieve Analysis						Hydrometer Analysis			
Sieve Size (mm)	Passing (%)	Sieve Size (mm)	Passing (%)	Sieve Size (mm)	Passing (%)	Particle Size (mm)	Passing (%)	Particle Size (mm)	Passing (%)
100.0	**100**	9.50	**29**	0.425	**12**				
75.0	**88**	6.70	**25**	0.300	**11**				
53.0	**74**	4.75	**22**	0.212	**10**				
37.5	**61**	2.36	**18**	0.150	**9**				
19.0	**41**	1.18	**15**	0.106	**7**				
13.20	**34**	0.600	**13**	0.075	**6**				

Test Methods	**Notes**
Particle Size Analysis: NZS 4402 1986 Test 2.8.1 (Wet Sieve)	History: Air + Oven dried
Particle Size Analysis: NZS 4402 1986 Test 2.8.4 (Hydrometer)	Uncalibrated Sieve sizes: 0.212, 0.106mm.

Date Tested: 19.12.13
Date Reported: 13.1.14

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IANZ Approved Signatory
Designation *Technical Officer (MJ Mclachlan)*
Date : 13.1.14

IANZ ACCREDITED LABORATORY
All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

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PARTICLE SIZE ANALYSIS
TEST REPORT

OPUS

Project:	**P2N Cycleway**
Location:	**Petone foreshore**
Client:	**NZTA**
Contractor:	**JAD Civil Design**
Sampled by:	**Aecom**
Date sampled:	**17.12.13**
Sampling method:	**Test pit**
Sample source:	**TP2 1.80m**
Sample description:	**Gravel: f-vc, grey, with sand**
Sample condition:	**As received**
Solid density	**n/a** t/m^3 **Assumed**
Water content as rec'd	**11.4** % **whole**

Report No:	**522900/1078**
Sample No:	**2-13/401**
Client Ref:	**60306339/7.02**

Sieve Analysis						Hydrometer Analysis			
Sieve Size (mm)	Passing (%)	Sieve Size (mm)	Passing (%)	Sieve Size (mm)	Passing (%)	Particle Size (mm)	Passing (%)	Particle Size (mm)	Passing (%)
75.0	**100**	9.50	**53**	0.425	**12**				
53.0	**95**	6.70	**36**	0.300	**11**				
37.5	**91**	4.75	**28**	0.212	**9**				
26.5	**87**	2.36	**19**	0.150	**5**				
19.0	**81**	1.18	**15**	0.106	**4**				
13.20	**68**	0.600	**13**	0.075	**3**				

Test Methods	**Notes**
Particle Size Analysis: NZS 4402 1986 Test 2.8.1 (Wet Sieve)	History: Air + Oven dried
Particle Size Analysis: NZS 4402 1986 Test 2.8.4 (Hydrometer)	Uncalibrated Sieve sizes: 0.212, 0.106mm.

Date Tested: 19.12.13
Date Reported: 13.1.14

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Designation *Technical Officer (MJ Mclachlan)*
Date : 13.1.14

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CALIFORNIA BEARING RATIO (REMOULDED) TEST REPORT

OPUS

Project :	**P2N Cycleway**
Location :	**Petone foreshore**
Client :	**NZTA**
Contractor :	**JAD Civil Design**
Sampled by :	**Aecom**
Date sampled :	**17.12.13**
Sampling method :	**Test pit**

Report No:	**522900/1078**
Sample No:	**2-13/400**
Client Ref:	**60306339/7.02**

Test Results

Sample No.		**2-13/400**	-	-	-	-	-
Source:		**TP2 1.20m**	-	-	-	-	-
Sample description:		**GRAVEL: f-vc, brown, with sand**	-	-	-	-	-
History		**Air dried**	-	-	-	-	-
Passing 19mm	%	**41**	-	-	-	-	-
Lime/cement additive	%	-	-	-	-	-	-
Curing time	days	-	-	-	-	-	-
Surcharge mass	kg	**4**	-	-	-	-	-
Sample condition:		**Soaked**	-	-	-	-	-
Soaking time	days	**2**	-	-	-	-	-
Swell	%	**0.1**	-	-	-	-	-
W/c as rec'd (whole)	%	**7.2**	-	-	-	-	-
W/c as comp. (-19mm)	%	**7.0**	-	-	-	-	-
Dry density	t/m³	**2.02**	-	-	-	-	-
Compaction (NZ Heavy)	%	**95.0***	-	-	-	-	-
W/c after test	%	**9.0**	-	-	-	-	-
Penetration	mm	**2.5**	-	-	-	-	-
CBR value	%	**55**	-	-	-	-	-

Test Methods

CBR	NZS 4402: 1986 test 6.1.1, NZS 4407:1991 test 3.15
Water Content	NZS 4402: 1986 test 2.1, NZS 4407: 1991 test 3.1
Compaction	NZS 4402: 1986 test 4.1.3 (Vibrating hammer)

Notes:

* Estimated from one point NZ Heavy Compaction, indicating MDD = 2.13 t/m^3, OWC = 7.0%

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This report may only be reproduced in full

Date tested : 8-13.1.14
Date reported : 13.1.14

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Designation : *Technical Officer (MJ Mclachlan)*
Date : 13.1.14

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DRAFT

Appendix G

Site Walkover Observations and Photographs

Area ID No.	Chainage		length (m)	Observations	Photo
	start	end			
Area 1	3690	3730	40	0.2-0.3m rock elements in unreinforced concrete matrix existing railway embankment slope 45-55 deg Beach gently sloping. Cobbles and coarse gravel exposed beach area approx 10m Rock armour reutilisation: 0 elements	
Area 2	3730	3800	70	0.3-0.5m rock elements in unreinforced concrete matrix existing railway embankment slope 45-55 deg Beach gently sloping. Coarse gravel exposed beach area approx 5m Rock armour reutilisation: 0 elements Concrete blocks sizing 1.5x0.7mx0.6m at embankment toe (2 rows). Concrete includes cooble size elements and appear to be unreinforced. Some of the blocks are heavily eroded	

Area ID No.	Chainage start	Chainage end	length (m)	Observations	Photo
Area 3	3800	3840	40	0.2-0.3m rock elements in unreinforced concrete matrix Additional rock armour elemnts at toe. Sizing 400-1000mm existing railway embankment slope 45-55 deg Beach gently sloping. Cobbles and coarse gravel no exposed beach area Rock armour reutilisation: 10 elements	
Area 4	3840	3960	120	Additional rock armour protection on existing slope. Sizing 300-1200mm rock outcrops on beach and in shallow waters Overall embankment slope angle 30 deg Beach gently sloping. Coarse gravel Exposed beach area up to 15m Rock armour reutilisation: 40 elements	

Area ID No.	Chainage		length (m)	Observations	Photo
	start	end			
Area 5	3960	4095	135	0.2-0.3m rock elements in unreinforced concrete matrix slope angle 50/55deg Additional rock armour elemnts at toe. Sizing 350-1100mm Unreinforced masonry wall and RC beam on top of on a 15m section no exposed beach area Rock armour reutilisation: 15 elements Stormwater outlet diameter 800mm at chainage 3880	
Area 6	4095	4185	90	0.2-0.3m rock elements in unreinforced concrete matrix slope angle 50/55deg Additional rock armour elements at toe and on embankment slope. Sizing 250-1300mm Exposed beach area 5 to 8m Rock armour reutilisation: 15 elements	

Area ID No.	Chainage start	Chainage end	length (m)	Observations	Photo
Area 7	4185	4230	45	0.2-0.3m rock elements in unreinforced concrete matrix slope angle 40/45 deg No exposed beach area Stormwater outlet (diameter 600mm) and manhole at chainage 4200 Stormwater outlet (diameter 600mm) and manhole at chainage 4300 Rock armour reutilisation: 0 elements	
Area 8	4230	4350	120	rock armour and construction debris form the coastal protection Overall embankment slope angle 30 deg Reclaimed area on seaside of railway tracks 1 to 4m Exposed beach area 3 to 20m Beach is rock blocks, concret blocks and filled with cobbles and coarse gravel Rock armour reutilisation: 20 elements	

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Appendix H

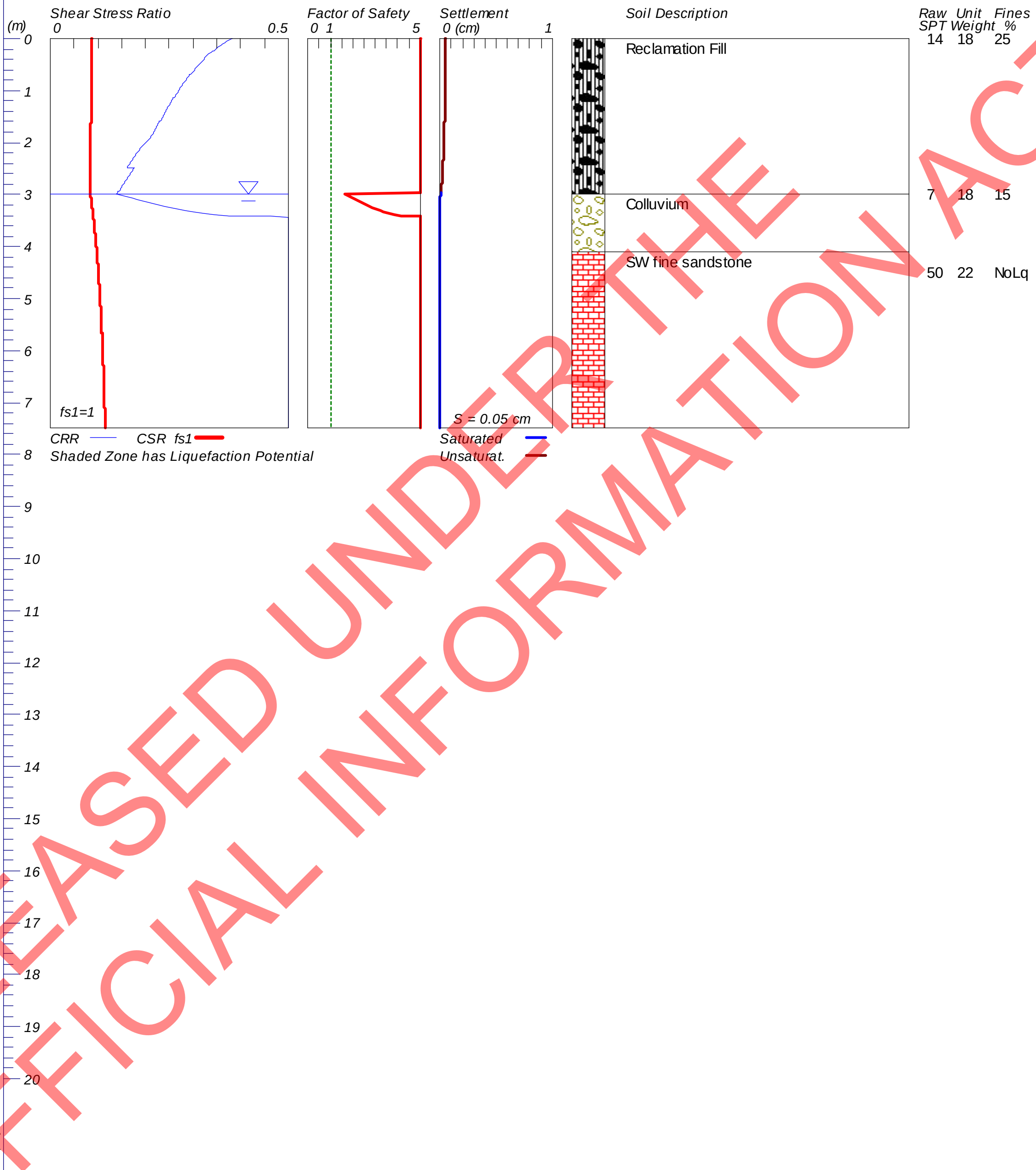
Liquefaction Analysis

LIQUEFACTION ANALYSIS

W2HVlink

Hole No.=DH02   Water Depth=3 m

Magnitude=7.5
Acceleration=0.133g

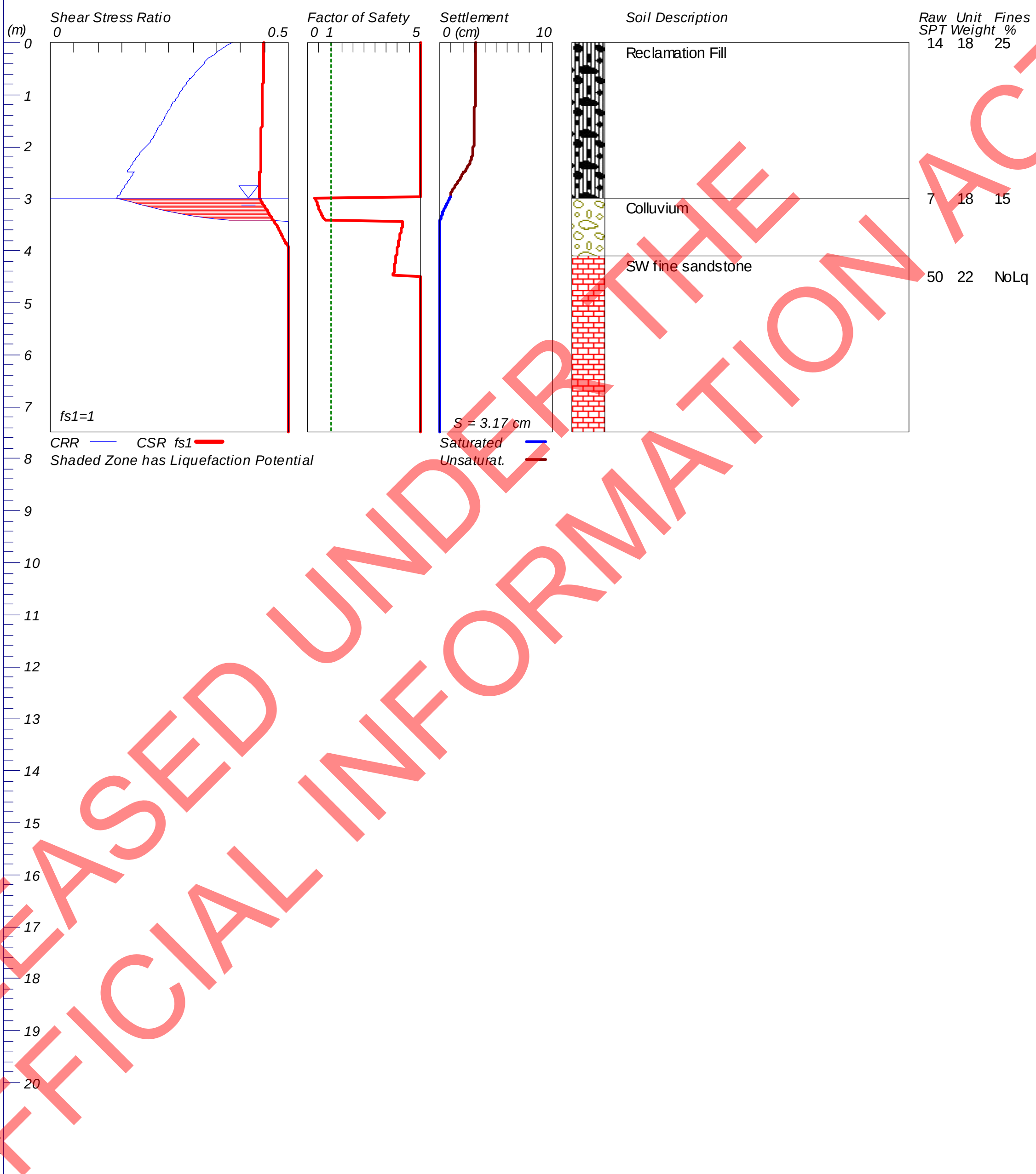


LIQUEFACTION ANALYSIS

W2HVlink

Hole No.=DH02 Water Depth=3 m

Magnitude=7.5
Acceleration=0.692g

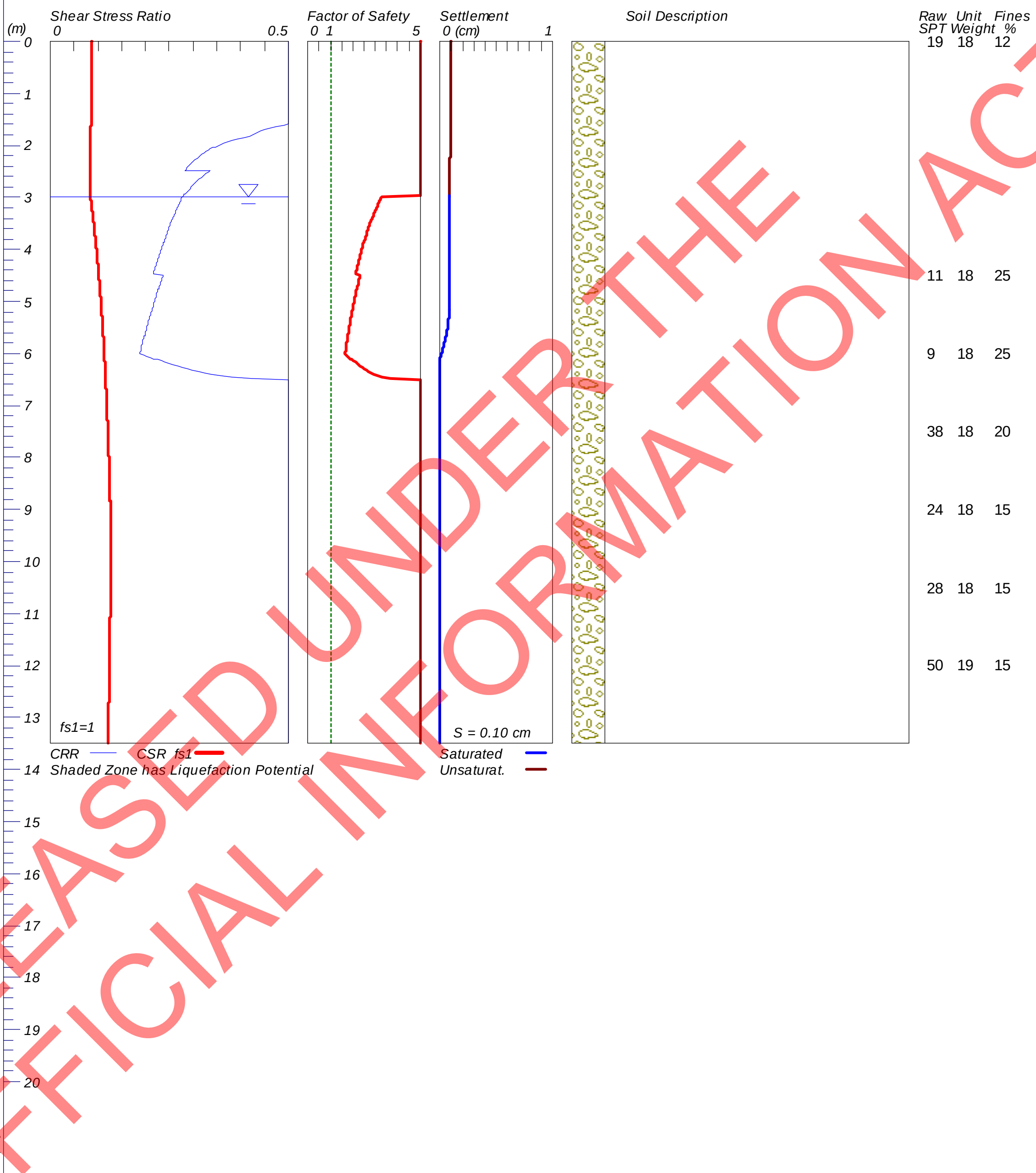


LIQUEFACTION ANALYSIS

P2N Cycleway

Hole No.=DH06 Water Depth=3 m

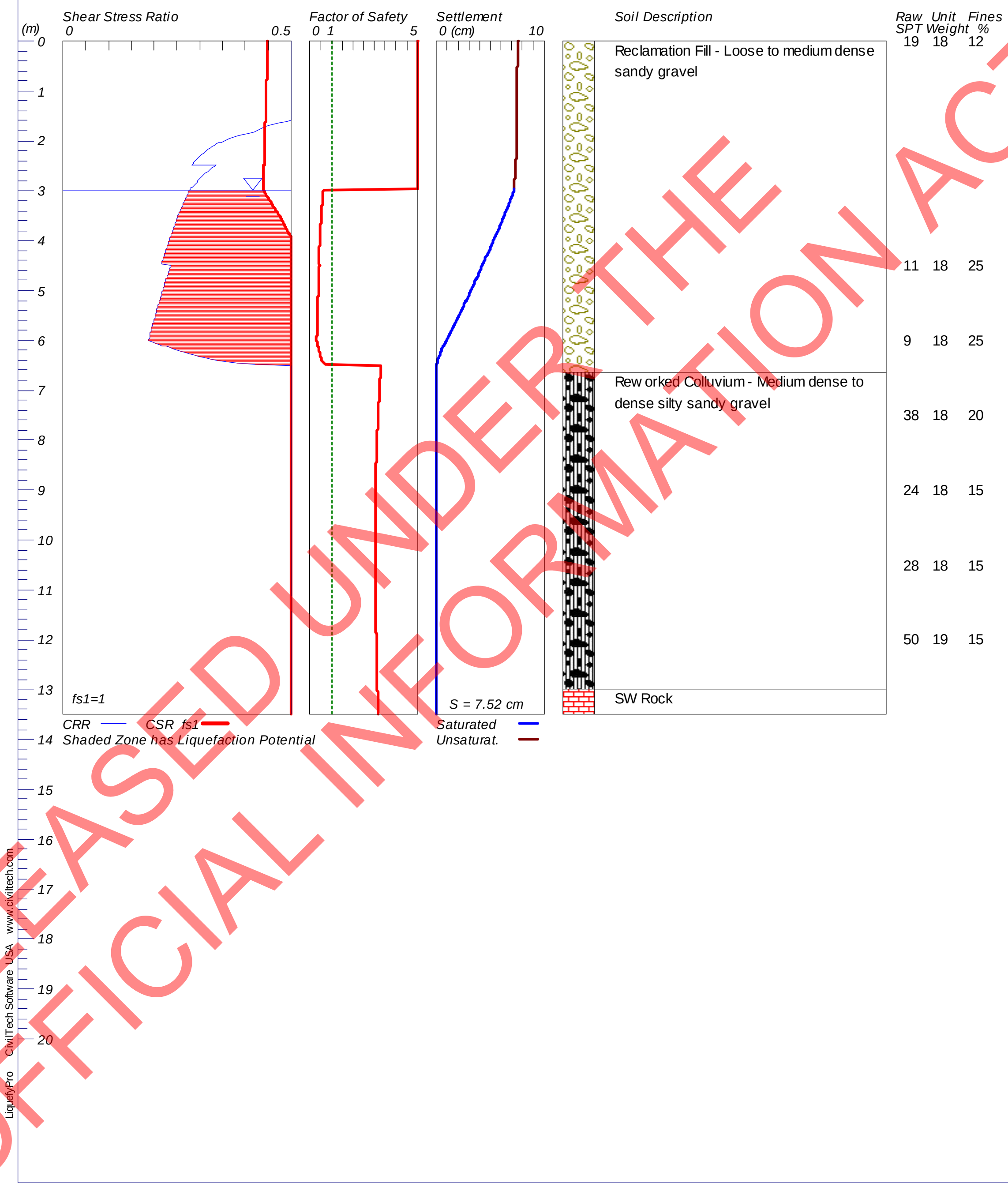
Magnitude=7.5
Acceleration=0.133g



LIQUEFACTION ANALYSIS

P2N Cycleway

Hole No.=DH06 Water Depth=3 m ***Magnitude=7.5***
Acceleration=0.692g

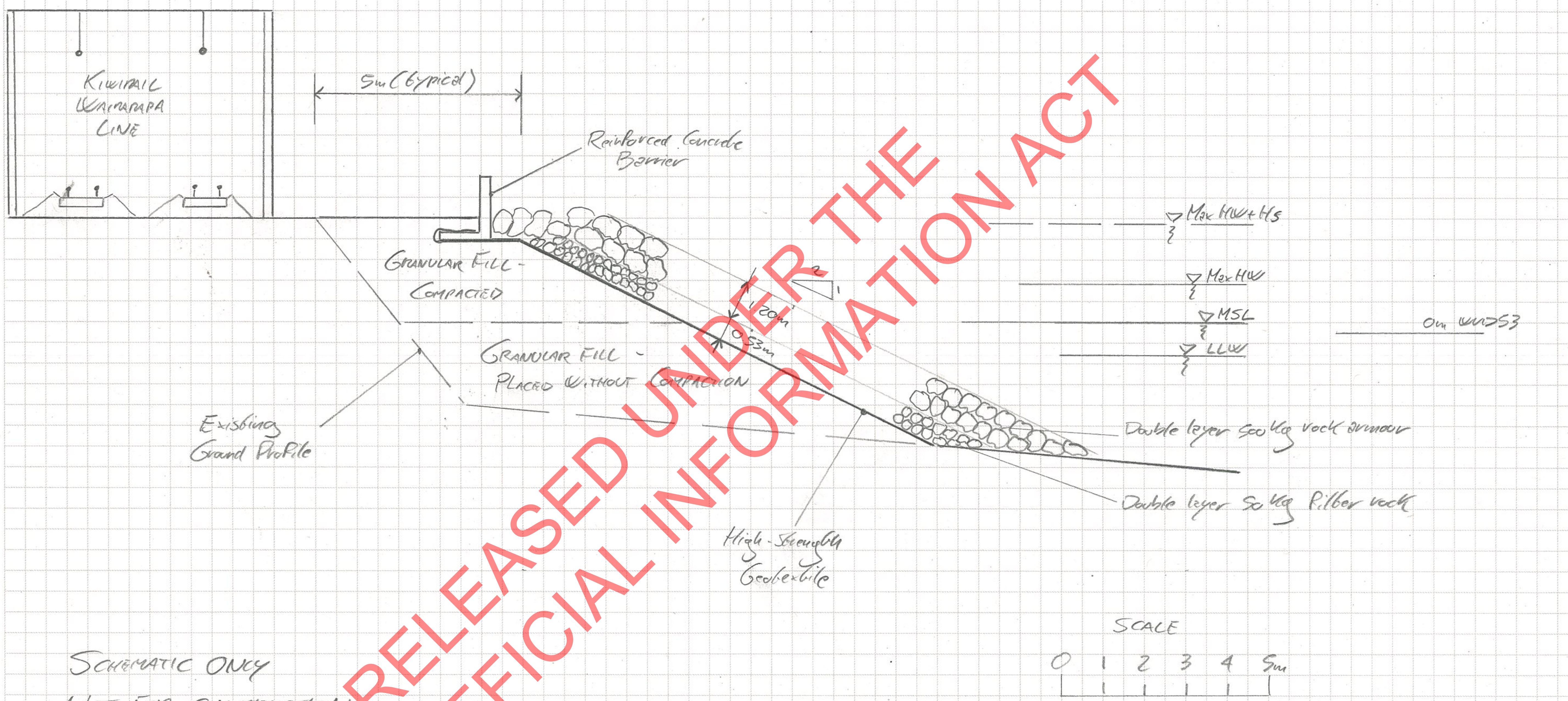


Appendix I

Proposed Preliminary Reclamation Cross-section

AECOM

Project: W2HV link
File/Ref No. 60306339
By: PGR
Page: 1/1
Date: 19/3/19



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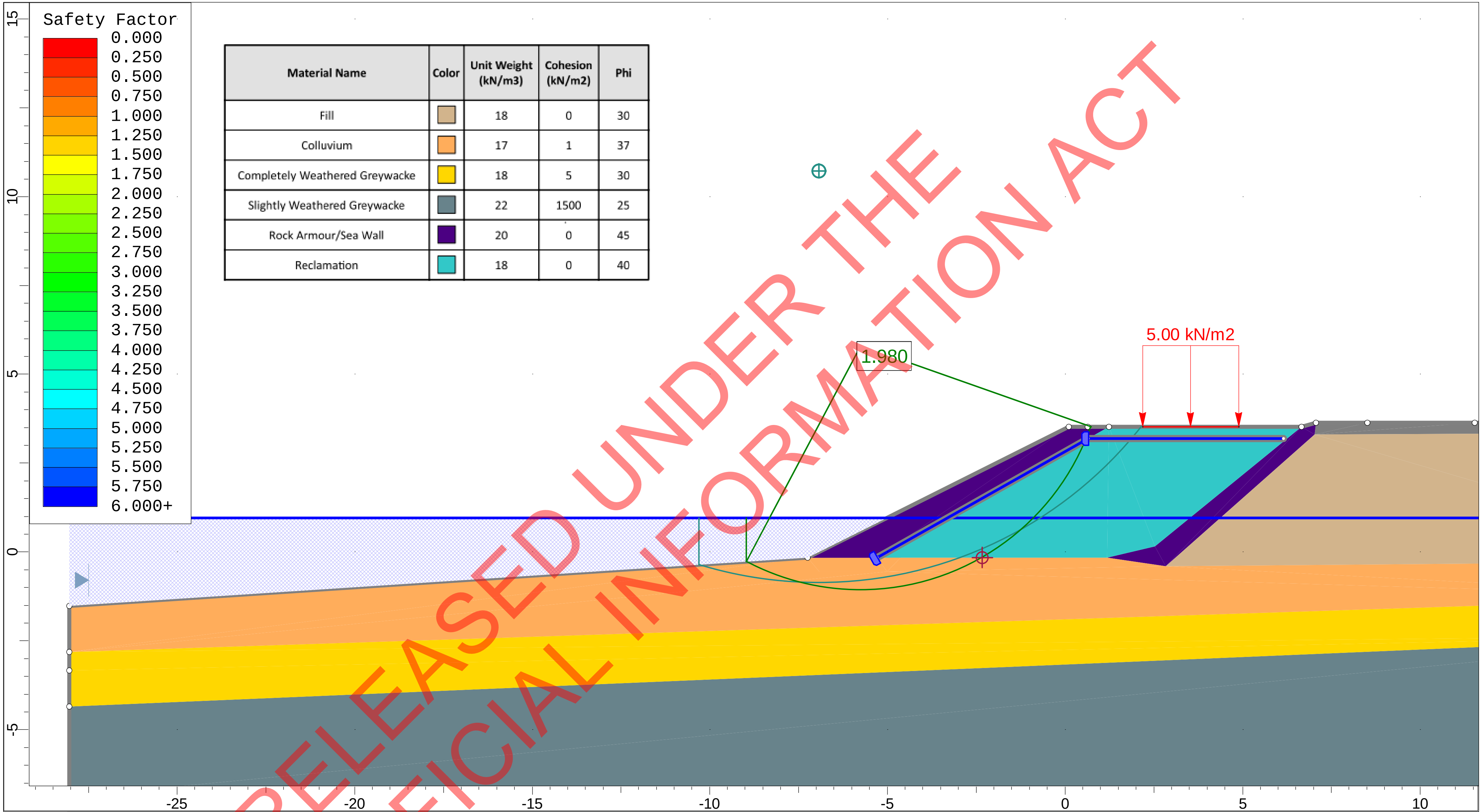
NOT FOR CONSTRUCTION

Appendix J

Slope Stability Analysis

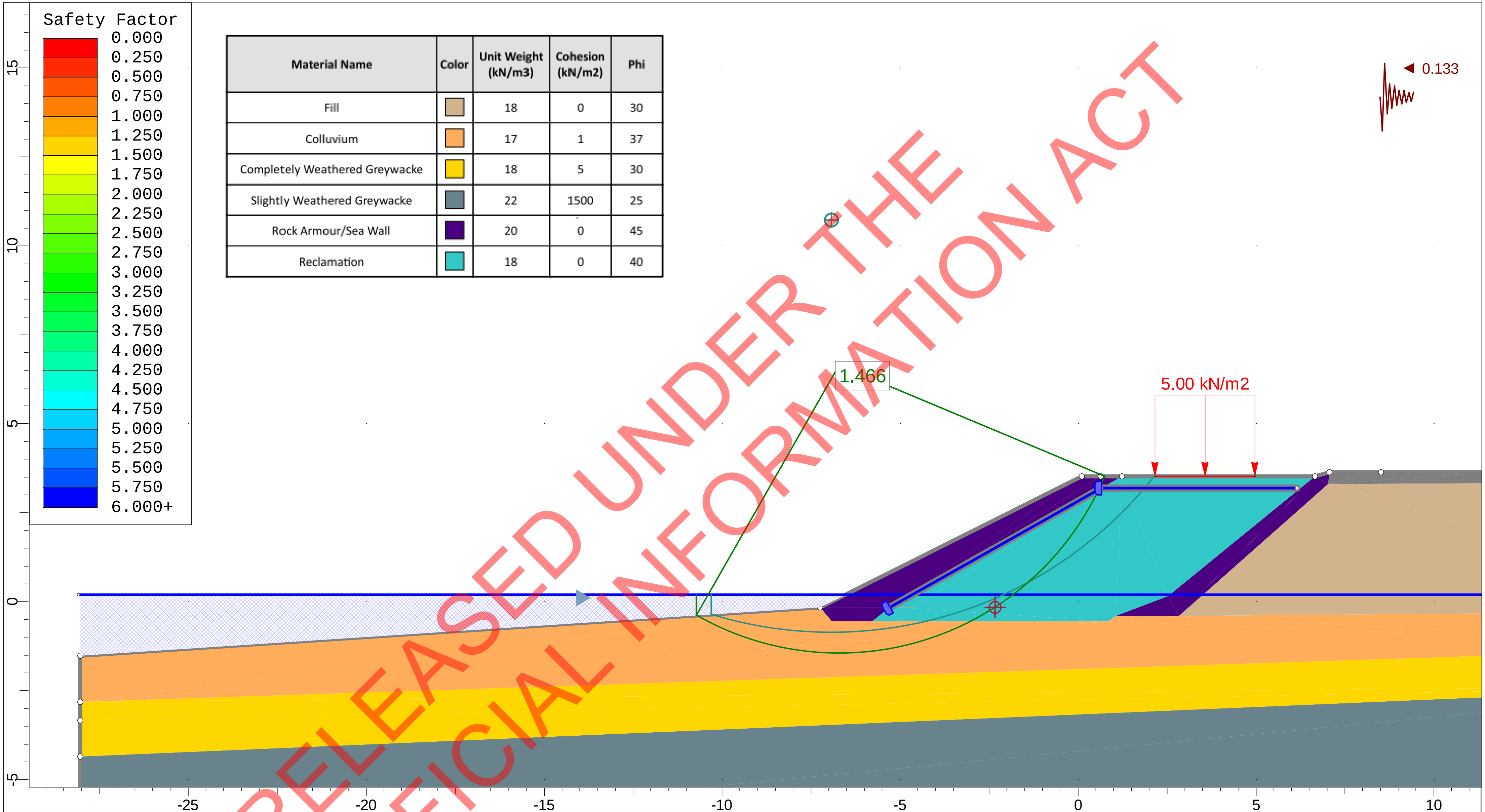


Material Name	Color	Unit Weight (kN/m3)	Cohesion (kN/m2)	Phi
Fill		18	0	30
Colluvium		17	1	37
Completely Weathered Greywacke		18	5	30
Slightly Weathered Greywacke		22	1500	25
Rock Armour/Sea Wall		20	0	45
Reclamation		18	0	40

rocscience

SLIDEINTERPRET 6.013

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description			
Drawn By		*Scale* 1:150	*Company*
Date 26/02/2014, 11:06:27 a.m.		*File Name* Chainage 3800 - static.slim	



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kN/m2)	Phi
Fill		18	0	30
Colluvium		17	1	37
Completely Weathered Greywacke		18	5	30
Slightly Weathered Greywacke		22	1500	25
Rock Armour/Sea Wall		20	0	45
Reclamation		18	0	40

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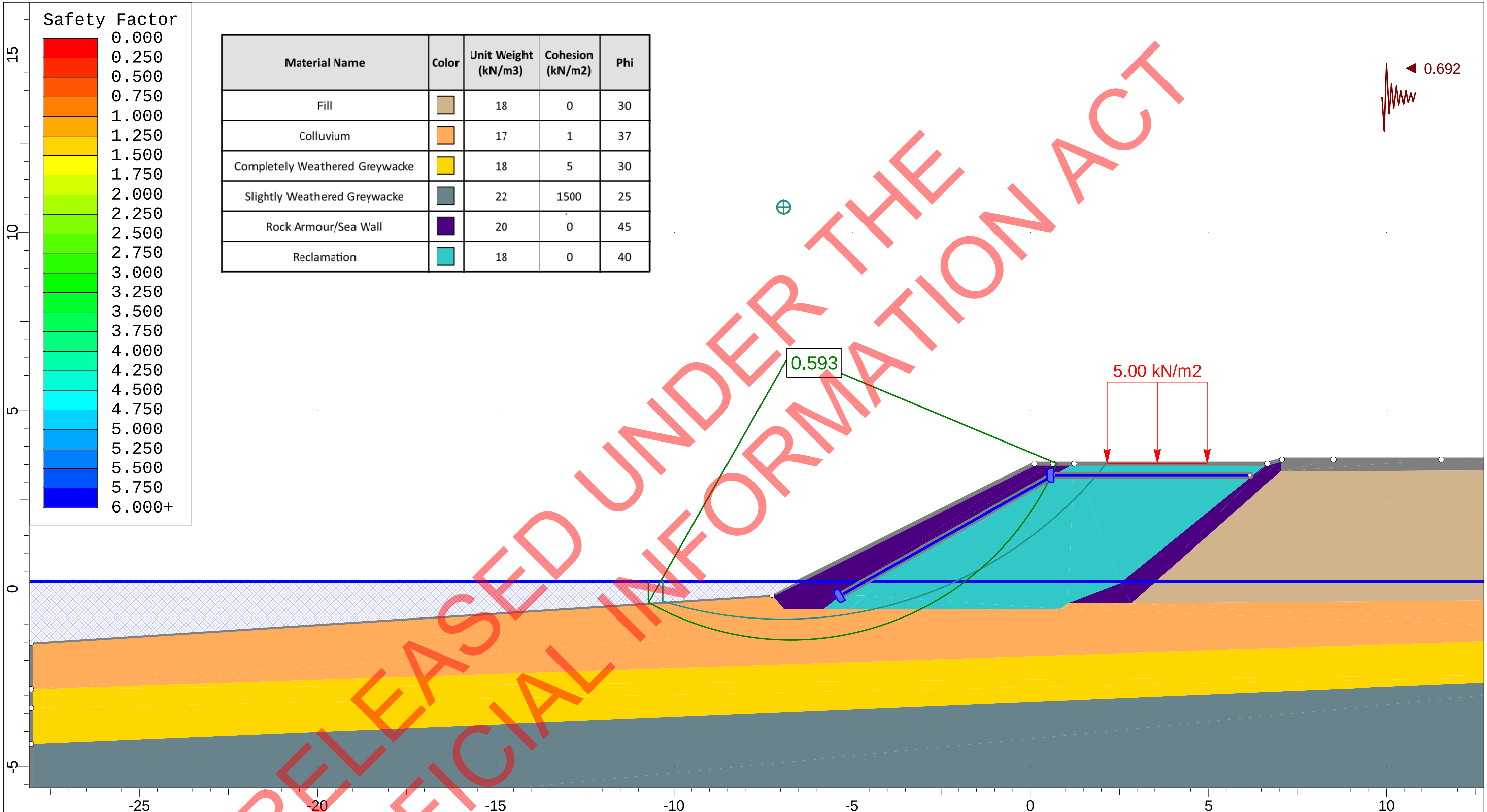
SLIDEINTERPRET 6.013

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description			
Drawn By		*Scale* 1:150	*Company*
Date 26/02/2014, 11:06:27 a.m.		*File Name*	Chainage 3800 - seismic SLS1 25.slim



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kN/m2)	Phi
Fill		18	0	30
Colluvium		17	1	37
Completely Weathered Greywacke		18	5	30
Slightly Weathered Greywacke		22	1500	25
Rock Armour/Sea Wall		20	0	45
Reclamation		18	0	40

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SLIDEINTERPRET 6.013

Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	
Drawn By	
Scale	1:150
Company	
Date	26/02/2014, 11:06:27 a.m.
File Name	Chainage 3800 - seismic ULS 1000.slim

Appendix P

Corridor Resilience

AECOM

New Zealand Transport Agency
19-Mar-2014

D R A F T

Wellington to Hutt Valley Cycle and Pedestrian Link

Project and Corridor Resilience

D R A F T

Wellington to Hutt Valley Cycle and Pedestrian Link

Project and Corridor Resilience

Client: New Zealand Transport Agency

Co No.: N/A

Prepared by

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19-Mar-2014

AECOM in Australia and New Zealand is certified to the latest version of ISO9001, ISO14001, AS/NZS4801 and OHSAS18001.

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Quality Information

Document	Wellington to Hutt Valley Cycle and Pedestrian Link
Ref	60306339
Date	19-Mar-2014
Prepared by	Matthew White
Reviewed by	James Hughes, Stuart Bettington

Revision History

Revision	Revision Date	Details	Authorised	
			Name/Position	Signature
1	12-Mar-2014	Internal reviews	Rob Napier Associate Director	
2	04-Apr-2014	Draft for Client Information	Rob Napier Associate Director	
3	28-Apr-2014	Updated draft for Client Information	Rob Napier Associate Director	
4	20-May-2014	Resilience costs updated	Rob Napier Associate Director	

D R A F T

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2.2.2 Storm Tides and Waves 4
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Executive Summary

A resilience workshop was held on 5 February 2014, attended by representatives from the NZ Transport Agency, KiwiRail, Wellington City Council, Hutt City Council and Greater Wellington Regional Council. The workshop involved a presentation on resilience theory, discussion on critical infrastructure elements and key hazards, and agreement on the major resilience objectives and focus areas for improving resilience.

This report follows the workshop, and recommends potential resilience parameters for the Petone to Ngauranga Corridor.

These parameters are divided into two general categories as follows:

1) Recommendations regarding design parameters and criteria for a range of agreed hazards

2) Recommendations regarding an ideal cross section dimension

RESILIENCE DESIGN PARAMETERS

The table below summarises the recommended design parameters.

Table E1 Summary of design parameters proposed

Hazard	Approach / criteria
Marine Conditions	Sea level rise, Sea level fluctuation, Tidal effects, Storm surge and wind effects, Datum adjustment, Sea level rise adjustment, Wave run-up, Wave setup: A total design level of **RL4.90** is recommended. The existing railway track and cycleway levels along the Petone to Ngauranga section are around RL4.40m above the Wellington Vertical Datum 1953. As such, the top of the sea wall would be around 500mm above this existing level.
Wind	The design wind speed shall be derived in accordance with AS/NZS 1170.2:2002 with the following specific requirements: 1) The design wind speed shall be taken as non-directional 2) The terrain Category Mz,cat shall be taken as not less than 2 (Exposed Rural Terrain)
Storms and Inland Flooding	Culverts to be designed for the 1 in 100 year ARI event with 500mm freeboard to the carriageway surface level. Bridges to be designed for the 1 in 100 year ARI event with 1200mm freeboard to the soffit Structural loading parameters to be as per requirements of the NZTA Bridge Manual
Earthquakes	Structures to be designed to earthquake loadings as per the NZTA Bridge Manual
Landslips	It is considered unlikely that any work will be required on the land-side of the corridor. During the workshop, it was suggested that landslips will not likely impact multiple lanes of the road, and as such resilience could be improved by facilitating access for rapid clearance of any slips (i.e. by widening the shoulder and providing and access road).

Note – specific design approaches to improve resilience for the various assets within the corridor have not been developed at this stage. These can be further considered at the detailed design stage, and may include the choice of materials specified, and specific design features.

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AN 'IDEAL' CORRIDOR CROSS SECTION

In relation to an 'ideal' cross section, two options were developed – each providing for: SH2 road carriageway, rail corridor, cycleway, a consolidated utility corridor, access for maintenance and wider shoulders.

Option 1 (refer Figure E1): the total width of the corridor is approximately 46.50m, with the additional footprint of the reclaimed land plus the sea wall embankment estimated to be around 12-15m wide.

Option 2 (refer Figure E2): the total width of the corridor is approximately 43.50m, with the additional footprint of the reclaimed land plus the sea wall embankment estimated to be around 8-10m wide.

Both of the above options would provide enhanced access for maintenance and improved ability to respond to a hazard event.

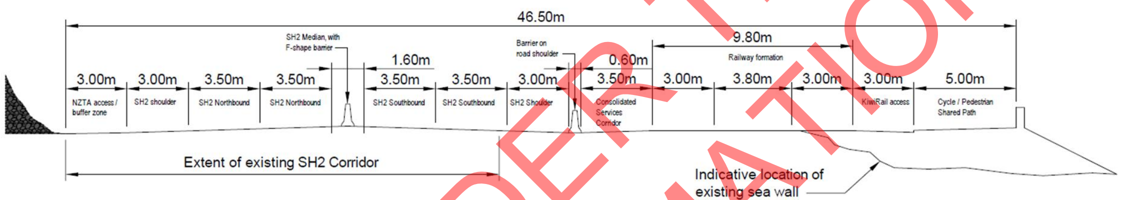

Figure E1 Option 1: Ideal Cross Section through the corridor

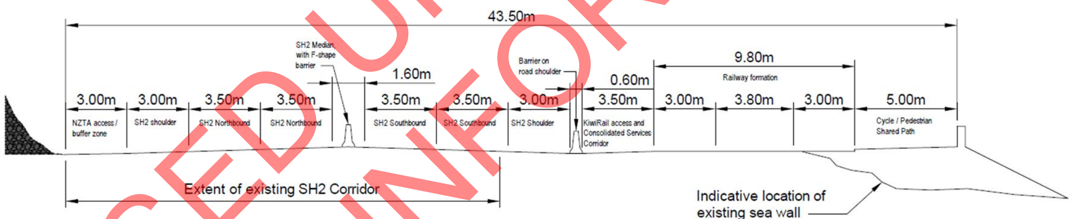

Figure E2 Options 2: Ideal Cross Section through the corridor

The cost comparisons between the reclamation options indicate that an extra 5m of land reclamation could cost in the order of $4M. Refer to section 4.0 for detail.

D R A F T

1.0 Introduction

The Wellington to Hutt Valley Shared Path (W2H) project is preparing scheme options for the construction of a high quality cycling and walking facility between Wellington and the Hutt Valley. The project outcome will be a Detailed Business Case for the section between Ngauranga and Petone, while considering connections north of Petone and south of Ngauranga.

We have prepared a shortlist of corridor options, based on the end-user requirements established during the project. To further develop these options, the project team has been asked to consider opportunities for improving transport resilience as part of the overall design approach.

A resilience workshop was held on 5 February 2014, attended by representatives from the NZ Transport Agency, KiwiRail, Wellington City Council, Hutt City Council and Greater Wellington Regional Council. The workshop involved a presentation on resilience theory, discussion on critical infrastructure elements and key hazards, and agreement on the major resilience objectives and focus areas for improving resilience.

As discussed at the workshop, infrastructure resilience can be divided into two dimensions: technical (or asset) resilience, and organisational (or operational) resilience (Hughes and Healy, 2014). Technical resilience can then be further divided into the following resilience *principles*: robustness, redundancy and safe-to-fail (Hughes and Healy, 2014).

The following two areas for improving resilience are the focus of this report:

Table 1 Summary of resilience focus

Resilience focus	Relates to:
1) Design Parameters and Criteria	- Technical (asset) resilience - robustness
2) Ideal Cross Section Dimension	- Technical (asset) resilience – safe-to-fail (relating to area available for maintenance and access should failure occur) - Organisational (operational) resilience – (relating to ability to maintain and respond to a failure or hazard event).

1.1 Purpose of this Report

This report addresses each of the items in Table 1, and more specifically:

In relation to 1) above; recommends which factors need to be considered within the design philosophy, appropriate 'resilience' design criteria for infrastructure elements, and which guidelines/ reports can be used to establish (justify) these design criteria.

In relation to 2) above; this report establishes a recommended 'ideal' corridor cross-section to facilitate access, maintenance and emergency response.

1.2 Resilience Workshop Outcomes

As mentioned above, a resilience workshop was held on 5 February 2014, attended by a range of stakeholders. Two key outcomes from the workshop were:

1) A list of known hazards which need to be considered to address resilience within the corridor, and;

2) Definition of critical assets within the corridor which would need protecting

1.2.1 Factors to Consider within the Corridor

Table 2 summarises the range of hazards identified and Table 3 describes the corridor elements which need to be considered when assessing resilience within the Petone to Ngauranga corridor. The first 6 are addressed in detail within Section 2.0, the remainder in Section 3.0.

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Table 2 Identified hazards

Hazard type
1. Tides – king tides and storm tides and waves
2. Sea level rise
3. Landslip
4. Earthquake
5. Inland storm / flooding
6. Wind (effect on poles/lighting/signage etc)
7. Mechanical issues (eg, derailments)
8. Operational incidents (e.g., trespassers in rail corridor)
9. Utility / service failure
10. Health and safety in general is an issue (for public and workers) due to poor access and lack of shoulders. It's a difficult place for emergency services to access.

Table 3 Corridor considerations by organisation

Organisation	Issues / Considerations
NZ Transport Agency	- Slopes adjacent to SH2 - Utilities within the road corridor - Emergency access either through this site, or to an incident in this location
KiwiRail / GWRC	- Rail bridge located at Ngauranga - Traction power system renewal - Cross-tie at Rocky Point (distributor / isolator for traction power supply) - Access to rail corridor in an emergency - Trespassers (operational issue) - Train derailments (operational issue) - Risk to rail; service disruption from traffic accidents and incidents on SH2 - Maintaining rail operations during maintenance work (operational issue) - Ongoing track and infrastructure deterioration from waves breaking over railway
WCC / HCC / GWRC	- Water main junctions
Other	- Sea wall - Consolidated utility corridor - BP service station, noted as potential fire risk

DRAFT

2.0 Resilience Design Parameters

This section details design parameters for the main hazards identified in the workshop. These parameters will be used to inform the project's *Design Philosophy Statement*.

2.1 Design Life

Even though the structure being considered is a pedestrian/bike path, this structure is to form the marine edge of a major transport corridor. For a normal marine structure a design life of 50 years would be considered suitable. In line with NZ government practice, consideration to longer time horizons when considering sea level rise (SLR) has been given.

2.2 Marine Conditions (Sea level rise, storm surge and waves)

This section summarises literature related to Annual Exceedence Probabilities (AEP) for storm surge effects, sea level rise and wave action effects in the Wellington area. The information has been obtained through reports sourced from NIWA, Wellington City Council, and Greater Wellington Regional Council. The results of this analysis will assist in establishing criteria for the design of the sea wall.

The design criteria that can be adopted for the structural and usability aspects of the structures impacted by marine loading are presented in Table 4.

Table 4 Parameters related to storm and flooding events

Element	Design Parameters	Source
Wave loads – Structural design	Ultimate 1/200 (0.5% AEP) No Damage 1/20 (5% AEP)	AS4997-2005 Guidelines for design of maritime structure
Overtopping – Structural design	Ultimate (0.5% AEP) $q>0.2m^3/m/s$ No Damage (5% AEP) $q<0.03m^3/m/s$	CUR - The Rock Manual
Overtopping – Serviceability*	Pedestrian (5% AEP) $q<0.005m^3/m/s$ Vehicles (5% AEP) $q<0.05m^3/m/s$	n/a*

* The overtopping parameter for serviceability of 5% AEP is deemed conservative, however is related to the assumed future water levels. Since climate change sea level rise is also being considered, this could be relaxed during detailed design.

Note the discharge values provided are not absolute and can be interpreted within a range depending on conditions.

To provide context, our team was recently involved in the design of the SH16 Causeway project in Auckland. For this project, NZTA commissioned NIWA to produce a combined sea level rise and storm surge (wave height) model. This model was used to assess the causeway options to ensure they would comply with the designated performance criteria.

The project adopted a minimum sea level rise value (for the Waitemata Harbour) of 0.5m above the 1990 mean sea level, plus future proofing within the design for an additional 0.9m.

In addition to sea level rise, the design also considered overtopping in terms of both discharge and volume. These parameters will be important to consider in final design, and criteria can be taken from Allsop et al (2005).

The following sections address sea level rise, storm surge and wave run-up in detail for the Wellington harbour.

2.2.1 Sea Level Rise

Bell and Hannah (2012) noted that Wellington Harbour is on course to record a sea level rise of 0.8m by the 2090s and a 1.0m approximate rise by 2115. This is in accordance with NZ Government guidance (the NZ Coastal Policy Statement and Ministry for the Environment statements). The authors state that these increases are similar to what is being used for planning in the United Kingdom and Australia.

The risk to assets is also discussed. Depending on the expected life of the asset, there is scope to design for lower or higher values for sea level rise; they suggest a range of values between **0.5m** and up to **2.0m**.

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For the purposes of this study (and the determination of an appropriate height of the sea wall), a sea level rise of 1.0m has been chosen. This corresponds to a 100 year timeframe – that is, to 2115.

It is recommended that the design of the wall itself is modular in nature, and provides opportunities to raise the wall in the future, should predictions be exceeded.

2.2.2 Storm Tides and Waves

For the purpose of quickly assessing the design level for a foreshore structure the assessment of the run-up level achieved by a combined water level and wave is a useful guide. To assess the 2% Exceedence run-up level ($Ru_{2\%}$) for a 100 year ARI storm event was adopted, as presented below.

A number of different NIWA reports exist which investigate the effects of tides, storm surge and waves within Wellington. Through discussions with Rob Bell from NIWA, it was agreed the most relevant was the 2006 report entitled, *Impacts of long term climate change on weather and coastal hazards for Wellington City* (Gorman et al, 2006).

This report develops joint probabilities of occurrence for sea level and offshore significant wave height (note this does not include *wave set up* or *wave run up* – which are covered below). The water level developed for this study is a storm tide level which includes the tides and the effect of storm surge (winds and barometric effects). The combined water level achieved by adding the tide and storm surge is commonly called storm tide. The probability of a very high tide level occurring concurrently with powerful storm surges is relatively low, and thus in the design 100 year ARI event data presented in Table 5, the water level (storm tide) associated with large waves is lower than the water level associated with small waves.

A simplified assessment of the combined impacts of waves and water level combined is to sum the wave height with the water level. The above report concludes that for a 1 in 100 year joint probability event, the maximum *combined* water level and wave height is 2.62m above. This is shown in the table below and is calculated by adding 1.4 (mean level of sea) and 1.22 (wave height).

The following important points are noted:

- This value includes storm tide comprising the combined influence of tides and storm surge (barometric and wind effects) but only includes a 0.4m of sea level rise
- Therefore an additional 0.6m of sea level rise needs to be added
- The water level needs an addition of 0.2m to convert MLOS to Wellington Vertical Datum (WVD-53)

Therefore, for the purposes of this study, the design still water level for a 100 year ARI event in 2115 is 1.4+0.6+0.2 = 2.2m WVD-53. When combined with the wave height this yields 3.4m WVD-53 (rounded). To this value, wave set up and run up need to be added to get a final level. It is noted that the wave setup and run up levels will vary depending on the near shore bathymetry and seawall construction.

Table 5 2100 joint probability return period wave and water levels for Petone area (NIWA, 2006)

Joint return (years)	Water level (m MLOS)	Wave height (m)
100	0	1.4
100	0.5	1.4
100	0.8	1.4
100	1	1.38
100	1.2	1.34
100	1.3	1.29
100	1.4	1.22
100	1.41	1.2
100	1.42	1.1
100	1.5	0.95
100	1.51	0.95
100	1.56	0.75
100	1.58	0.5
100	1.59	0.2
100	1.6	0.1
100	1.6	0

Note: MLOS = Mean Level of Sea

2.2.3 Wave Set up and Run up

Wellington Harbour has been subjected to large waves at various times. Aspects of the June 2013 storm were measured by NIWA scientists. Data from a wave buoy near Baring Head, which has been installed since 1995, revealed the storm generated the largest waves measured to date in on the coast[1]. Significant waves were also experienced in the inner harbour, which caused significant damage to the Petone to Ngauranga section of the Wairarapa Railway.

The process of waves breaking on the coast is described by two actions: firstly, as waves approach the coast they push up the water level as a result of the release of wave energy in shallow water (this is described as wave *setup*); secondly, as the wave breaks on the coast, it causes wave 'up-rush' on the beach (defined as wave *run-up).* Figure 1 shows these wave actions. If water depths in front of the seawall are too great to cause waves to break then there will be no wave setup. The wave run-up value is amplified if the beach or structure is steeply graded, smooth or has low permeability (e.g. stone pitch slopes).

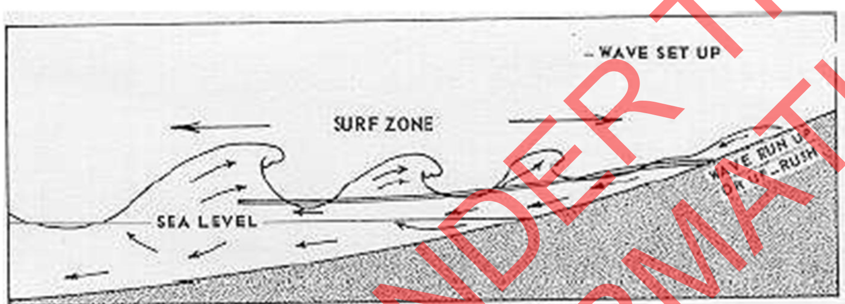

Figure 1 Diagram illustrating wave action at the coast (Source: Stephens el at, 2012)

An estimate of wave set-up of 0.2m was provided by NIWA (pers comm, Rob Bell). The adoption of this value is considered a conservative assumption as the vast bulk of the waves will not break on the foreshore associated with the design water depths being considered.

Calculations for wave run-up were undertaken and are summarised below. For estimating the run-up we use the wave length, wave period, significant wave height and structure slope and structure roughness. Because the wave climate impacting the seawall has a range of wave heights the run-up is typically described as an Exceedence probability. For this exercise, we will use a 2% AEP wave during the 1% joint probability event. This **100 year ARI $Ru_{2\%}$** event means that 1 wave in 50 during this extreme event will reach this level.

This is considered conservative and implies overtopping is unlikely (i.e. only in events greater than 2% AEP). During detailed design these parameters can be refined further and can allow for the impacts of structure face slope and material etc.

The calculation used to determine a wave run-up estimate for a 100 year ARI event (1% AEP event) is summarised below.

Significant wave height, Hs= 1.22m– refer Table 5.

Wave height exceeded by 2% of waves $H_{2\%}$ = 1.7m

Mean wave period, Tm= 3.4s

Assumed static water level, SWL= 2.2m WVD (2115)

Bed level -1mWVD (depth = 3.2m thus vast bulk of wave not breaking offshore of the structure)

Wave length at the seawall= 13.5m

Structure slope= 1 in 1 (steep)

Structure permeability = 0.2 (a low value, effectively impermeable)

[1] See www.niwa.co.nz/news/storm-and-snow-information-update

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Using Delft Hydraulics Equations (ref Coastal Engineering Manual equations VI-5-12 & VI-5-13)

$Ru_{2\%}$ = 2.5m above SWL

Therefore, based on the above calculations, a wave run-up value of 2.5m above the design water level has been calculated and adopted.

2.2.4 Summary of sea level rise, storm surge and wave run-up

Considering all of the above factors, we summarise the following parameters to be taken forward to establish design criteria. Some refinement may be required at final design stage, however these numbers are considered adequate for this level of design.

Table 6 Wellington Harbour – summary of design levels for establishing sea wall height

Component	Value	Source of data
Sea level rise	1.40m (defined by joint-probability analysis; including 0.40m for sea level rise)	*Impacts of long term climate change on weather and coastal hazards for Wellington City (NIWA, 2006)*
Sea level fluctuation		
Tidal effects		
Storm surge and wind effects		
Datum adjustment	0.20m (to adjust values to be in terms of WVD-53)	*Sea Level Variability and Trends: Wellington Region (NIWA, June 2012)*
Sea level rise adjustment	0.60m (to allow 1.0m total sea level rise, which is the current 2115 prediction)	*Sea Level Variability and Trends: Wellington Region (NIWA, June 2012)*
Wave run-up	2.50m (based on an offshore wave height of 1.22m hitting a sea wall sloped at 1:1)	Based on calculations
Wave setup	0.20m	Estimated value (Pers. Comm with Rob Bell, NIWA)
Recommended design level	**RL 4.90m**	

The existing railway track and cycleway levels along the Petone to Ngauranga section are around RL4.40m above the Wellington Vertical Datum 1953. As such, the top of the sea wall would be around 500mm above this existing level.

2.3 Wind

Wellington has been battered by high winds at various times, with major storms generating high wind speeds in 1961, 1965, 1967, 1968, 1974, 1977, 1985 and 2013. The 1968 storm, in which the Wahine disaster occurred, included a maximum ten-minute-average wind speed of 144km/h, which is the largest on record. A maximum ten-minute-average wind speed of 101km/h was recorded during the recent 2013 storm. [2]

The resilience workshop identified the potential effect of wind on poles, lighting componentry and signs.

Standard practice requires the design wind speed to be derived in accordance with AS/NZS 1170.2:2002. It is proposed the following specific requirements be utilised from this document.

1) The design wind speed shall be taken as non-directional

2) The terrain Category Mz,cat shall be taken as not less than 2 (Exposed Rural Terrain)

3) The structure importance factor for the signs shall be taken as 3, with a design wind event return period of 1000 years for a 50 year design life.

[2] http://www.niwa.co.nz/news/storm-and-snow-information-update

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Note – specific design approaches to improve resilience for assets such as poles/gantries have not been developed at this stage. These can be further considered at the detailed design stage, and may include the choice of materials specified, and specific design features.

2.4 Storms and Inland Flooding

Design parameters for flooding relate to both to conveyance of peak flows and impacts on structures. In terms of conveyance, both KiwiRail and the NZ Transport Agency use 1.0% AEP as a basis for design for conveyance of storm flood flows.

In terms structural impacts and freeboard above peak flows, the NZTA *Bridge Manual* gives clear guidance. This is summarised below

Climate change effects will need to be built into any flood estimation, in accordance with Ministry for the Environment guidance.

Table 7 summarises the design parameters at related to storm and flooding events.

Table 7 Parameters related to storm and flooding events

Element	Design Parameters	Source
Bridges (Importance Level 3)	Structural: 1/2500 (0.04% AEP) ULS Flood conveyance: 1/100 (1.0% AEP)	NZTA *Bridge Manual* specifies for wind, snow and floodwater actions
Culverts	Flood conveyance: 1/100 (1.0% AEP)	NZTA guidance
Earth slopes (fill slope > 6m high) (Importance Level 3)	1/1000 (0.1% AEP) ULS	NZTA *Bridge Manual*
Earth slopes (fill slope < 6m high, and all cut slopes) (Importance Level 3)	1/500 (0.2% AEP) ULS	NZTA *Bridge Manual*

Importance Level 3 is defined in AS/NZS 1170.0. This is also shown on p2-7 of the *Bridge Manual.*

An additional consideration is that of freeboard above the 1% AEP event. The NZTA Bridge Manual specifies a standard 600mm for bridges, and 1200mm if the upstream catchment could generate significant debris. This manual also specifies freeboard at culvert locations of 500mm to the carriageway level.

Recommendations are as follows:

- That the above parameters in Table 7 are used for detailed design of bridges and culverts.
- That freeboard for culverts is set as 500mm above the 1% AEP event
- That where possible, 1200mm freeboard be provided at bridges.

2.5 Earthquakes

The Petone to Ngauranga transport corridor is near major fault lines. A recent report by the Wellington Lifelines Group (Mowll, 2012), stated that a major earthquake event in the Wellington area may result in The Petone to Ngauranga section of SH2 being *'closed by large landslides for many weeks to months'*.

There is a significant amount of guidance available which describes earthquake loading for various structures. We have noted these design parameters in Table 8 below.

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Table 8 Parameters related to earthquake events

Element	Design Parameters	Source
Bridges (Importance Level 3)	1/2500 (0.04% AEP) ULS	NZTA *Bridge Manual*
Earth slopes (fill slope > 6m high) (Importance Level 3)	1/1000 (0.1% AEP) ULS	NZTA *Bridge Manual*
Earth slopes (fill slope < 6m high, and all cut slopes) (Importance Level 3)	1/500 (0.2% AEP) ULS	NZTA *Bridge Manual*
Earth slopes (fill slopes)	50% probability that movement will be less than 300mm in 1/1000 earthquake	*McKays to Peka Peka Scheme Assessment Report*
Earth slopes (fill slopes)	90% probability that movement will be less than 700mm in 1/1000 earthquake	*McKays to Peka Peka Scheme Assessment Report*

It is proposed that the parameters in Table 8 be utilised as design parameters.

It is noted that Tsunami do occur in Wellington Harbour, but these have not been actively included in this assessment. In the detailed assessment of the seawall design the impact of a low tsunami should be considered.

2.6 Landslips

At this stage, it is unlikely that any work will be required on the land-side of the corridor. During the workshop, it was suggested that landslips will not likely impact multiple lanes of the road, and as such resilience could be improved by facilitating access for rapid clearance of any slips (i.e. by widening the shoulder and providing and access road).

However, should remedial work be required as part of future design stages, there are potential approaches that could be used to identify, prioritise and mitigate risk.

In this event, an approach similar to that implemented recently for the Manawatu Gorge is considered appropriate (GNS, 2012). This approach would involve a risk assessment - considering the site geology, and investigating old landslips in the area and defining their impacts (where information is available). Where potential landslip sites are identified, mitigation measures would need to be defined to reduce the potential impact on SH2. These measures could include rock bolting and improvements to the slope face to minimise the potential of landslips.

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3.0 Ideal Corridor Cross Section

An ideal corridor cross section was discussed in the resilience workshop. Broad zones (widths) were discussed for SH2 and the rail corridor, and allowance was made for a consolidated utility corridor.

Following the workshop we carried out further investigation to clarify the widths required for each component within the corridor. This work involved using various standards to determine the road and rail corridor widths.

We have developed two options as shown in Figure 3 and Figure 4. Further details are provided below.

3.1 Road Parameters

Austroads guidance was used to define shoulder widths along SH2. Austroad's *Guide to Road Design, Part 3: Geometric Design* contains this paragraph:

> *A width of 2.5 m is needed to allow a passenger vehicle to stop clear of the traffic lanes. A width of 3.0 m allows a passenger vehicle to stop clear of the traffic lanes and provides an additional clearance to passing traffic. It also allows a truck to stop clear of the traffic lanes.*

It was recognised in the workshop that this section of SH2 carries a large proportion of HCVs. Furthermore, there was an incident on this section of SH2 in 2012 when a truck driver attempted to stop clear of the traffic lanes but ended up being clipped by a vehicle travelling in the left lane. For these reasons we have decided that, ideally, a 3.0m wide sealed shoulder would be suitable both on the northern and southern sides.

In addition, on the northern side of SH2, NZTA suggested that a 3.0m wide access corridor be installed, beyond the road shoulder. This would act as a secondary emergency access lane and also as a buffer or clear zone between the road carriageway and the cliff faces – in the event of rock fall from landslip or earthquake.

For shoulders on the median, the Austroads document defines the minimum shoulder width as 1.0m. However, the existing central median is not as wide as this. Therefore we will essentially keep the median the same width as existing. We have assumed that there is 0.5m of shoulder width on both sides of the road. Additionally, we have chosen to use an F-shape concrete median barrier within the median. This barrier will be TL-5 standard, and will be approx. 600mm wide.

3.2 Rail Parameters and Consolidated Services Corridor

KiwiRail's T200 Network Engineering Track Handbook was used to determine the widths between the rail tracks. Table 3 of this document defines minimum track centres. We selected 3.8m as the minimum distance between the railway tracks, as this refers to two mainline tracks outside station limits.

The structural gauge calls for any structures to be located a minimum of 2.75m from the rail centreline. New gantry poles would need to comply with this requirement.

The Queensland Government produced a document, *Design & Selection Criteria for Road/Rail Interface Barriers (June 2009)*, to guide barrier requirements when roads are located near rail corridors. This guide uses details such as the road category, the type of rail use (commuter or freight services) and the offset, measured from the road edgeline to 3m off the nearest rail track centreline, to determine barrier requirements. The nearer the two modes are to each other, the more stringent are the barrier requirements.

Given these criteria, one option is to place the consolidated services corridor (which was discussed at the workshop), between the rail corridor and the road shoulder. This increases the separation by 3.5m. So with a 3.0m wide road shoulder and the 3.5m services corridor, plus a 0.6m allowance for the roadside barrier, there is 7.1m offset between the road and rail corridors. Considering the design speed of SH2, and the principally commuter function of this section of the Wairarapa Railway, the guide defines the barrier between the two modes as a 1.5m high, TL-6 barrier. This requirement will need to be confirmed with the NZTA to ensure the barrier meets their requirements.

KiwiRail noted the need for a barrier, or similar, which would contain incidents on SH2. This is to protect rail services in the event of an incident, such as a diesel spillage, within the road corridor. AECOM can investigate designs which will contain spills on the SH2 carriageway, or at least stop them from entering the railway corridor.

A further KiwiRail requirement is for maintenance access along this corridor. This access road would need to be 3.0m wide. Two options were provided as shown in Figure 3 and Figure 4.

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- Option 1: access has been placed on the seaward side of the railway corridor.
- Option 2: access has been combined with the services corridor

Figure 2 Existing sea wall next to a shared path, New Plymouth

3.3 Parameters for Utility Services

Many utility services are defined as Lifeline Utilities as they provide essential infrastructure to the community. A number of utility services were identified as being located within the corridor – as summarised below:

- Water supply, wastewater and stormwater
- Petrol and diesel
- Electricity and gas, and
- Telecommunication utilities

Since these utilities are in many cases buried they can be overlooked. However, a review of available Lifeline Utility literature shows that considerable work has gone into identifying vulnerabilities associated with a range of events (flooding, earthquake, snow, tsunami and wind), and how to provide greater resilience to these services (see for example CELG (1997)). Specific recommendations on how to improve resilience of the individual utilities is outside the scope of this report however could be considered at a later date.

The provision of a consolidated services corridor will provide utility companies with opportunities to develop a migration strategy. This may also allow them to investigate options to build further robustness or redundancy into their systems. Moreover, placing services within a consolidated corridor will facilitate swift repairs without disrupting road and rail operations.

Two options are recommended for the service corridor:

- Option 1: 3.5m width between SH2 and the rail
- Option 2: 3.5m width of combined service and KiwiRail access corridor between SH2 and the rail

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3.4 Cross Section Summary

Figure 3 and Figure 4 below show the schematic ideal cross sections for options 1 and 2. Some items to note are:

Option 1:

- Total corridor width is approximately 48.90m
- For this cross section, the footprint of the *reclaimed* land plus the sea wall embankment is estimated to be around 12-15m wide.

Option 2:

- Total corridor width is approximately 43.50m
- For this cross section, the footprint of the *reclaimed* land plus the sea wall embankment is estimated to be around 8-10m wide.

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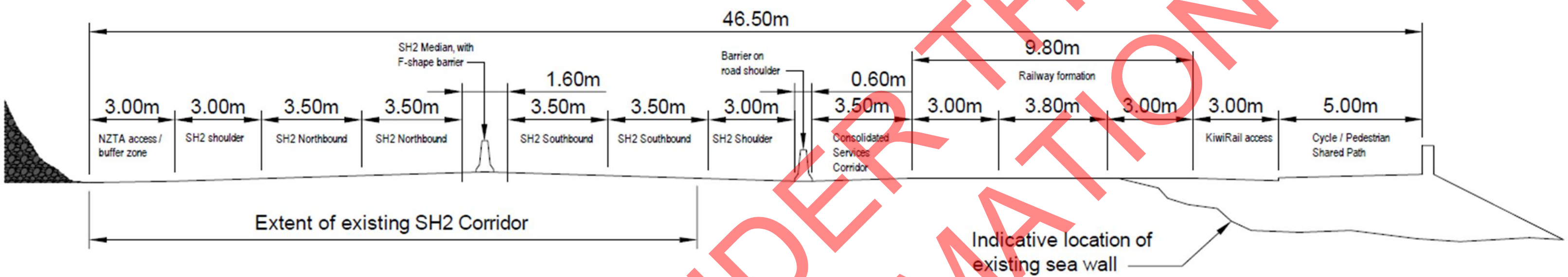

Figure 3 **Option 1: Ideal Cross Section through the corridor**

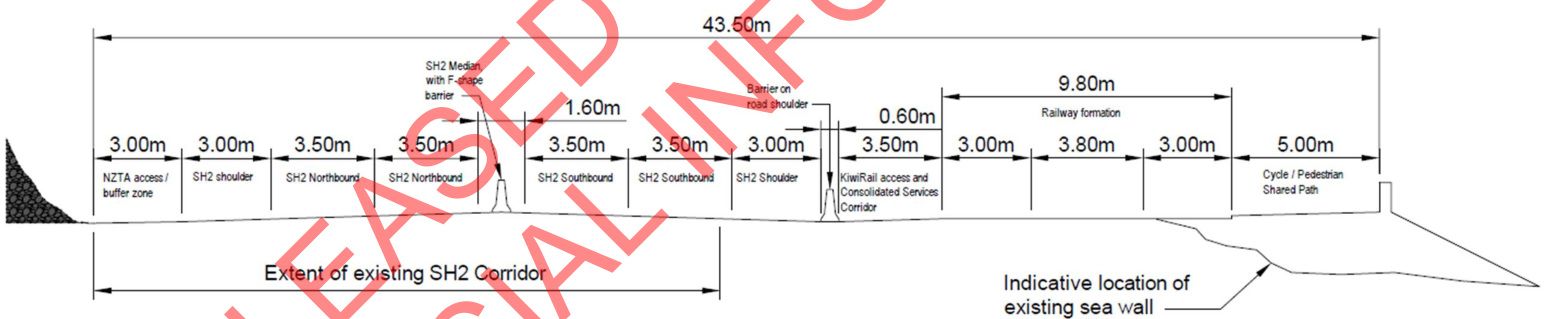

Figure 4 **Options 2: Ideal Cross Section through the corridor**

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3.5 Sea Wall Parameters

Our coastal specialists have defined the shape and material requirements for a new sea wall along the transport corridor. The new sea wall will use a slope of 1m (V): 2m (H). A geotextile will be used above the general fill material. A layer of rock will be placed on the geotextile, followed by larger rock armouring.

Figure 5 below shows a sketch of the proposed sea wall profile. Note that this sketch does not show the final expected width; this width will be determined during the scheme design process. Figure 2 shows an existing sea wall, next to a shared path, in New Plymouth. Note the path is at the same level as the top of the rock armour; which may mean that path users are not protected from spray from large waves.

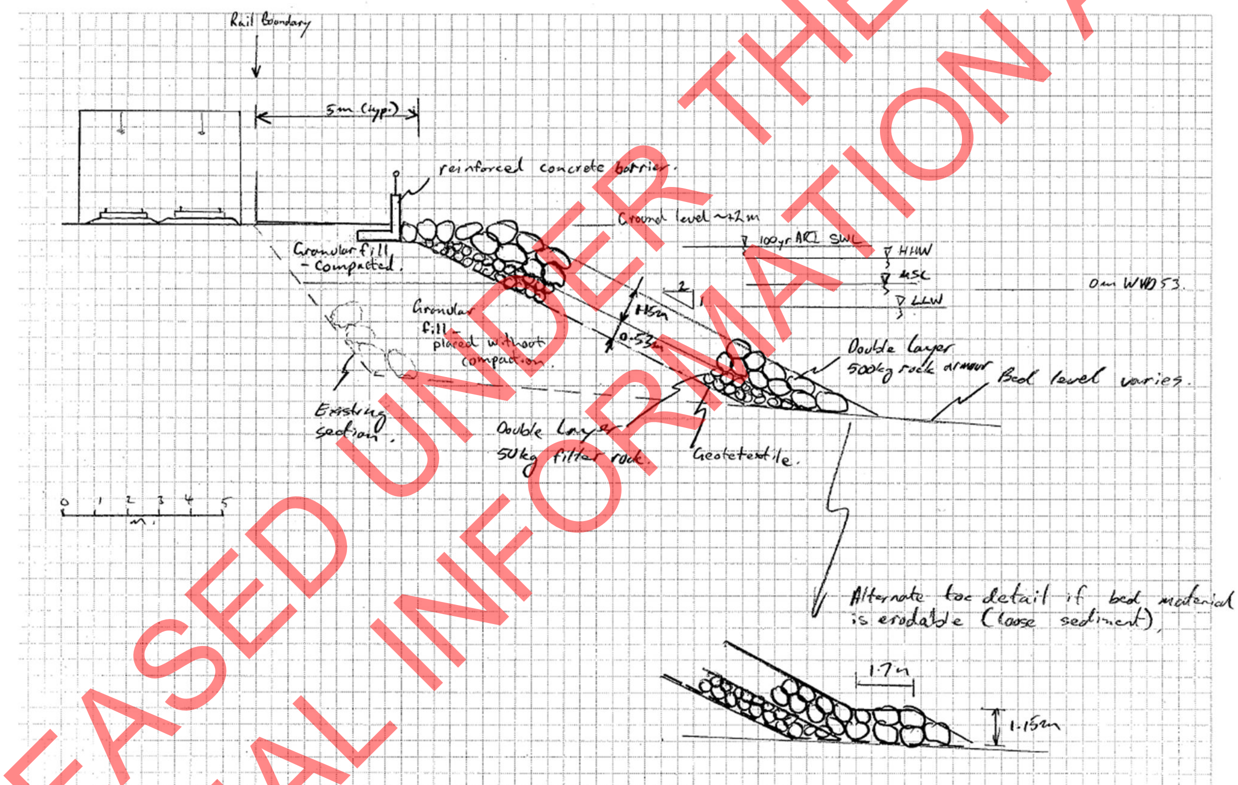

Figure 5 Sketch of proposed sea wall

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4.0 Reclamation Cost Options

The following high level cost estimates (feasibility base estimates) have been prepared to show the relationship between the cost of reclamation to support the current project (Option 3) and the cost of reclamation to support additional width for future corridor widening.

Table 9 shows that the cost reclamation to support the recommended option (Option 3) is approximately $10.4M. This would result in an average 5m effective width on the sea side of the rail corridor to support a 3m wide shared path.

The Resilience Report investigated an "Ideal Future" cross section for the corridor which suggested a total 46m wide cross section would bring the corridor up to 'standard' including a service lane for the rail corridor. Table 9 Cross Section A, shows that the reclamation required to support this option would cost in the order of $15.8M.

A further option to future proof the transport corridor, involving either a third rail track OR two additional motorway lanes, would require a total 51m wide cross section. Table 9 Cross Section B, shows that the reclamation required to support this option would cost in the order of $21.1M.

Option	Approximate Platform Achieved	Fill (m^3)		Cost / m^3	Reclamation Cost
		Rock Armouring	Remaining Bulk Fill		
A) Option 3 (allows 3m shared path)	5m	31,069m^3	60,566m^3	$113	$10.4M
B) Cross Section A (allows KiwiRail service lane)	10m	32,623m^3 (+0.5%)	119,754m^3	$103	$15.8M
C) Cross Section B (allows two traffic lanes or a third rail track)	15m	34,254m^3 (+0.5%)	178,865m^3	$99	$21.1M

Table 9: Reclamation Costs

Notes:

- Due to the shoreline profile or natural harbour contour, full reclamation increment (5, 10 or 15m) is not always required and the figure is an indication of the maximum reclamation width required.
- Option 3 – As per the current design Option 3, the cost of which is extracted from scheme level cost estimates. This achieves a 5m platform.
- **Cross Section A** provides a 10m wide platform, which would allow AUSTROAD compliant widths within a realigned corridor, including a KiwiRail service lane [consistent with the "Ideal Cross Section" as described in Section 3.0 and Figure 3 (Option 1)].
- **Cross Section B** provides a 15m wide platform, which would allow two traffic lanes or a third rail track if the corridor was realigned.

Disclaimers:

1) The rates used are based on experience and rates from other construction projects of a similar scale. Some savings can be made if the bulk fill is obtained from the proposed Petone to Granada project.

2) Costs for Cross Section B and C are based on total unit rates derived from Option 3.

3) Existing harbour bed gradient / fall is consistent (refer to Option 3 cross sections). As a result, assume rock armouring will increase by 5% due to bed gradient / fall.

4) The costs are for reclamation only and do not include costs associated with below or at-grade use of the reclaimed land.

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5.0 Summary and Conclusions

This report recommends potential resilience parameters for the Petone to Ngauranga Corridor. These parameters are divided into two general categories as follows:

1) Recommendations regarding design parameters and criteria for a range of agreed hazards

2) Recommendations regarding an ideal cross section dimension

The table below summarises the proposed design parameters.

Table 10 Summary of design parameters proposed

Hazard	Approach / criteria
Marine Conditions	Sea level rise, Sea level fluctuation, Tidal effects, Storm surge and wind effects, Datum adjustment, Sea level rise adjustment, Wave run-up, Wave setup: A total design level of **RL4.90** is recommended. The existing railway track and cycleway levels along the Petone to Ngauranga section are around RL4.40m above the Wellington Vertical Datum 1953. As such, the top of the sea wall would be around 500mm above this existing level.
Wind	The design wind speed shall be derived in accordance with AS/NZS 1170.2:2002 with the following specific requirements: 1) The design wind speed shall be taken as non-directional 2) The terrain Category Mz,cat shall be taken as not less than 2 (Exposed Rural Terrain)
Storms and Inland Flooding	Culverts to be designed for the 1 in 100 year ARI event with 500mm freeboard to the road carriageway level. Bridges to be designed for the 1 in 100 year ARI event with 1200mm freeboard to the soffit Structural loading parameters to be as per requirements of the NZTA Bridge Manual
Earthquakes	Structures to be designed to earthquake loadings as per the NZTA Bridge Manual
Landslips	It is considered unlikely that any work will be required on the land-side of the corridor. During the workshop, it was suggested that landslips will not likely impact multiple lanes of the road, and as such resilience could be improved by facilitating access for rapid clearance of any slips (i.e. by widening the shoulder and providing and access road).

Note – specific design approaches to improve resilience for the various assets within the corridor have not been developed at this stage. These can be further considered at the detailed design stage, and may include the choice of materials specified, and specific design features.

In relation to an ‘ideal’ cross section, two options were developed – each providing for: SH2 road carriageway, rail corridor, cycleway, a consolidated utility corridor, access for maintenance and wider shoulders.

Option 1: the total width of the corridor is approximately 46.50m, with the additional footprint of the reclaimed land plus the sea wall embankment estimated to be around 12-15m wide.

Option 2: the total width of the corridor is approximately 43.50m, with the additional footprint of the reclaimed land plus the sea wall embankment estimated to be around 8-10m wide.

Both of the above options would provide enhanced access for maintenance and improved ability to respond to a hazard event.

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