Appendix N Preliminary Structures Options Report

Ashburton-Tinwald Connectivity Preliminary Structure Options Report

PREPARED FOR ASHBURTON DISTRICT COUNCIL | JULY 2022

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EXECUTIVE SUMMARY

This report has been prepared in support of the Ashburton-Tinwald Connectivity (ATC) Detailed Business Case (DBC). As part of this DBC a new bridge is proposed crossing the Ashburton River approximately 0.8km downstream of the existing SH1 crossing. The proposed bridge crossing runs in line with Chalmers Ave connecting Ashburton to Tinwald. The total crossing length from stop bank to stop bank is around 650m.

The proposed crossing will comprise:

- A primary bridge crossing the main river channel (360m long)
- A secondary bridge (60m long) providing drainage of the Tinwald flood plain
- Earth fill approaches at each stop bank and within the heavy vegetation on the left bank

The key factors influencing design are detailed in Section 2 of this report and include:

Service requirements
• Dual lane carriageway and cycle lanes • Shared use (for walking and cycling) path on both sides of the roadway
Foundation conditions
• Well compacted sandy gravel from 4m below bed level to founding level • Some liquefaction risk above 4m, otherwise the deeper foundations are not susceptible
Urban design
• General Urban Design (UD) guidance for bridges includes consideration of location, context, views, underbridge/overbridge experience, form, proportion, light and shadow • Specific UD guidance for the Ashburton River includes consideration of existing upstream bridges, fitting within the local landscape, local ecology, and overall bridge form • Low maintenance design • Five key UD principles – connectivity, safety, choices, people, landscape/environment
Geometrics
• Longitudinal grade to suit stormwater containment and runoff to abutments for treatment before disposal
Hydrology
• Flood frequency estimation has been provided by ECAN and reviewed by Stantec • We have adopted Importance Level 4 (SLS2 = 100yr APE, ULS = 2500yr APE) • We have included allowance for climate change RCP6 • Stopbanks have been checked for a 200yr APE, plus climate change RCP8.5, plus a 0.5m freeboard • Riverbed level monitoring started in 1937 and indicate a currently stable bed with some scope for change, depending on future gravel extraction policy over the 100 year design life
Hydraulics (1D)
• Local hydraulics are heavily impacted by the TR bank tree block resulting in a strong hydraulic separation between the main river channel and the Tinwald flood plains • Typical bed roughness factors have been researched and adopted • Surveyed FWLs from the 2021 event were used to calibrate the 1D model, these show reasonable freeboard on the Ashburton side and marginal freeboard on the Tinwald side • Assessment of the SLS2 flood event with the new bridges in place, indicate that upstream water levels will rise by 156mm (for a shorter 330m long main bridge), extending approximately 200m upstream • Assessment of the ECAN stop bank design criteria (Q200 + RCP8.5) with the new bridges in place, indicate that the Ashburton side stopbank has compliant freeboard, but the Tinwald side stopbank does not • Assessment of the ULS flood event with the new bridges in place, indicate that flood water levels are marginally contained within the existing stopbanks, but are likely to be overtopping in other locations upstream and downstream of the modelled reach • Minimum soffit levels were determined for both the main and secondary bridge

HECRAS Analysis (2D)

- In order to address the limitation of the 1D modelling, a more detailed 2D hydraulic model was undertaken
- Results of this modelling will be provided upon completion

Scour

- A general long-term scour of +/- 0.50m is proposed
- Constriction scour (through the bridge constriction) is estimated to be 0.35m
- Local scour (at the piers) is estimated to be up to 6.45m
- Total design scour is estimated to be 7.30m below current bed levels

The various design options considered are covered under section 3, and include:

Bridge Length

- Single 420m bridge option discarded due to blockage of the Tinwald flood plain
- Dual bridge options of 360+60m and 330+60m considered and to be refined in detailed design

Super-Structure

- All concrete construction proposed for low maintenance and long service life
- Typical options include double hollow-core beams, single hollow-core beams, super-T beams

Flood Protection

- The Ashburton side stopbank has been assessed as compliant
- The Tinwald side stopbank (terrace) has been assessed as non-compliant and may require adjustment, relocation or a revised compliance level
- More detailed 2D hydraulic will provide a more reliable indication of flood water levels
- Further discussion will be required with ECAN to determine the best course of action

The preferred option is described and discussed under Section 4. It includes:

Bridge Length

- The dual bridge 360+60m has been adopted for this report as it provides a realistic upper limit of waterway requirements and may be refined to a shorter length
- The individual bridge lengths will be optimised in detailed design, pending more detailed hydraulic analysis and further discussions with ECAN

Super-Structure

- A 30m super-T option has been adopted for this report as it provides a conservative basis for costing and allows for longer spans if required
- Discussions with local contractors indicate that long span hollow-core option are often preferred due to simpler on-site works and reduced construction time
- Selection of the optimal super-structure will be determined in detailed design, pending refinement of carriageway and a detailed cost comparison of hollow-core vs super-T

Flood Protection

- Further discussions with ECAN should include:
  - Review of hydraulic modelling and agreement to new bridge lengths
  - Acceptance of Ashburton stopbank height as compliant
  - Consider options for Tinwald stopbank mitigation or modify compliance criteria

Alternative Design

- It is highly likely that alternative bridge designs will be offered during tender
- The detailed design phase should clearly identify the minimum design criteria to be achieved by alternative designs to ensure they are comparable with the conforming design

ABBREVIATIONS

AADT	Annual Average Daily Traffic
ADC	Ashburton District Council
AMP	Asset Management Plan
APE	Annual Probability of Exceedance
ATC	Ashburton-Tinwald Connectivity
BW	Back Water (analysis)
CALS	Collapse Avoidance Limit State
CBD	Central Business District
DBC	Detailed Business Case
DCLS	Damage Control Limit State
ECAN	Environment Canterbury
FWL	Flood Water Level
HECRAS	Hydrologic Engineering Center's River Analysis System
IL	Importance Level
LIDAR	Light Detection and Ranging
LOS	Level of Service
MBL	Mean Bed Level
NOR	Notice of Requirement
NZBM	NZTA Bridge Manual, Third edition, Amendment 4
RCP	Representative Concentration Pathway
SH1	State Highway 1
SLS	Serviceability Limit State
SPT	Standard Penetration Test
TCEV	Two-component Extreme Value distribution
TR/TL	True Right and True Left (of river)
ULDF	Urban and Landscape Design Framework
ULS	Ultimate Limit State
vpd	Vehicle movements per day
vph	Vehicle movements per hour

1. INTRODUCTION

1.1 GENERAL

This report has been prepared in support of the Ashburton-Tinwald Connectivity (ATC) Detailed Business Case (DBC). As part of this DBC a new bridge is proposed crossing the Ashburton River approximately 0.8km downstream of the existing SH1 crossing.

The proposed new bridge will provide:

- Improved connectivity between Ashburton and Tinwald
- Improved active mode options (cycling, walking, running)
- Emergency bypass if the SH1 bridge is damaged by future flood events
- A construction bypass when the SH1 bridge is eventually replaced

This report provides a preliminary assessment of the structural options under consideration, with the report content generally in alignment with that proposed by the Waka Kotahi Highway Structures Design Guide.

1.2 SITE DESCRIPTION

The proposed bridge crossing runs in line with Chalmers Avenue connecting Ashburton to Tinwald, as shown in Figure 1-1. The total crossing length from stop bank to stop bank is around 650m.

The proposed crossing will comprise:

- A primary bridge crossing the main river channel
- A secondary bridge or culvert providing drainage of the true right flood plain
- Earth fill approaches at each stop bank and within the heavy vegetation on the right bank

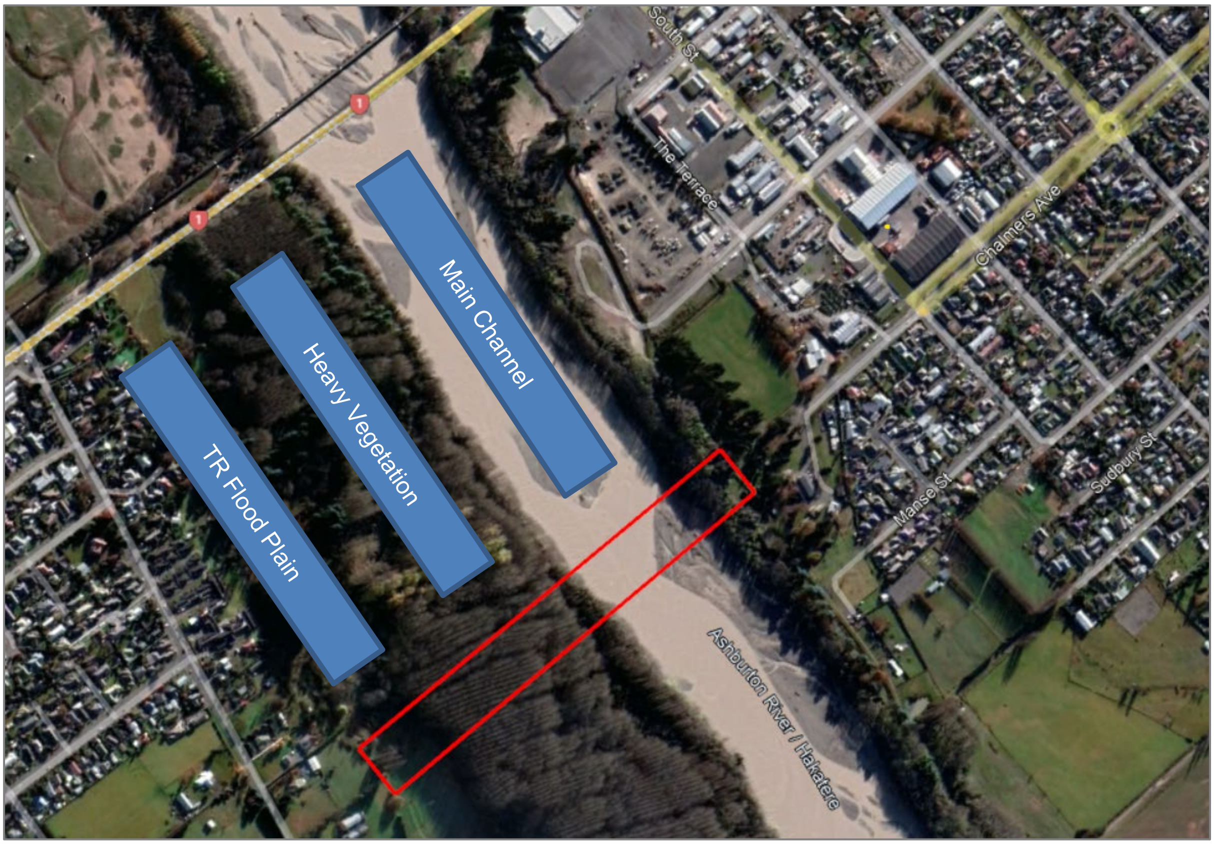


Figure 1-1: Proposed bridge crossing location

2. FACTORS INFLUENCING DESIGN

2.1 SERVICE REQUIREMENTS

The proposed crossing will provide the following level of service:

- District arterial connection as well as a temporary State Highway bypass/resilience option
- Dual traffic and cycle lanes with wide centreline treatment carriageway
- Shared use path on both sides
- Speed limit of 50 kph
- Traffic volume (AADT) of 4000+ vpd (estimated for year one after opening)
- Provision for current and future services (telecom, water, power, sewerage, etc)

The proposed bridge deck cross-section is shown below.

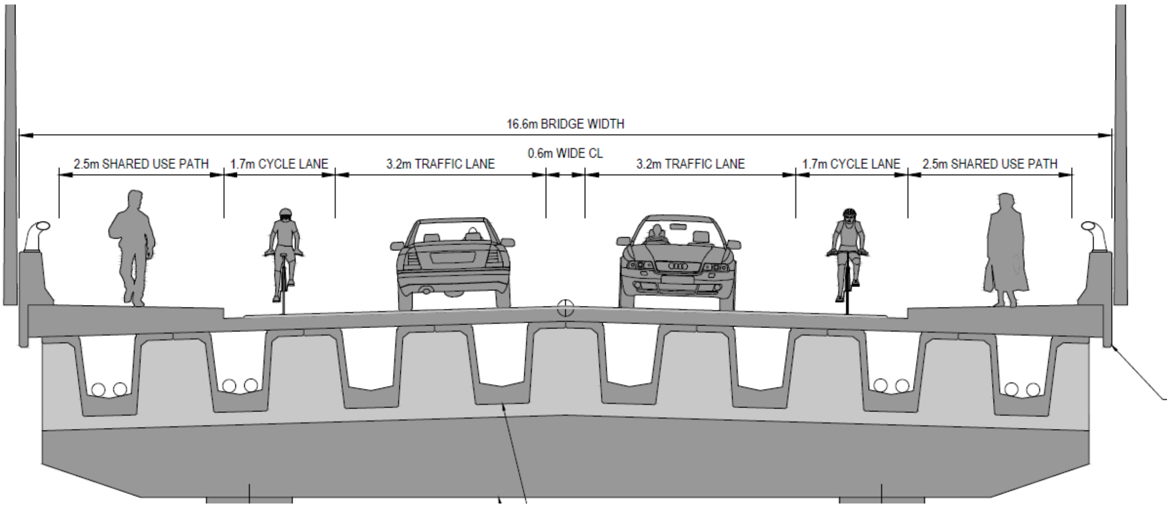


Figure 2-1: Proposed bridge cross section

2.2 FOUNDATION CONDITIONS

Geotechnical investigations were conducted to understand the site-specific conditions, to develop a ground profile, and to derive the preliminary design geotechnical input parameters, using the published correlations and our experiences from similar ground condition. The investigation consisted of:

- A borehole on each side of the main river channel to 20m below bed level
- Multiple test pits on the approach road alignment
- SPTs (standard penetration tests) at each borehole

Some of the key findings from this investigation are:

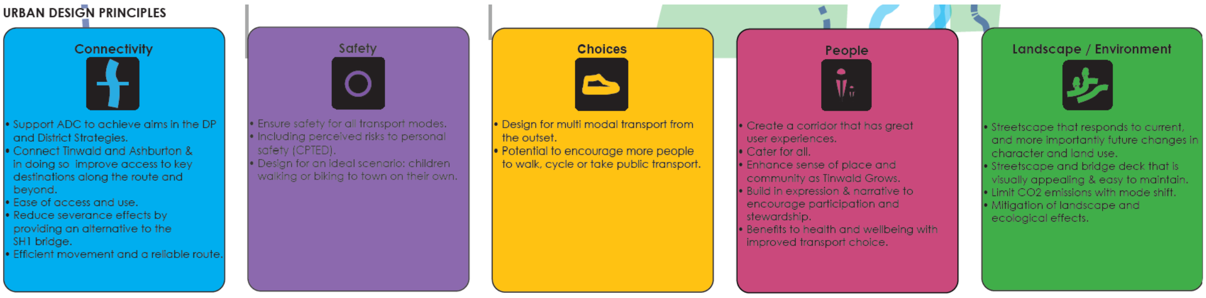
- The boreholes found well compacted sandy river gravels (N = 50+) from 4m below bed level down to founding level (10-15m below bed level).
- The liquefaction assessment concludes that liquefaction triggering is not anticipated under SLS seismic load conditions but is marginal under DCLS seismic load and anticipated under CALS seismic load condition. Soil strata lower than 4.0 – 5.0 m below ground level are not susceptible to liquefaction during any design earthquake events.
- For the flow failure condition (liquefied soil condition at the end of seismic event), the model predicted no major ground failure (flow failure).
- The preliminary pier foundation size of 2 No 1.5 m diameter cylindrical piers down to 15 m below bed level, is sufficient to support the proposed bridge vertically.

The results of this investigation are detailed in **Appendix C - Geotechnical Report**.

2.3 URBAN DESIGN

Urban design considerations for this project are detailed in the Ashburton River / Hakatere Second Bridge Urban and Landscape Design Framework (ULDF) report.

Five urban design principles are identified in the report:



URBAN DESIGN PRINCIPLES

Connectivity

- Support ADC to achieve aims in the DP and District Strategies.
- Connect Tinwald and Ashburton & in doing so improve access to key destinations along the route and beyond.
- Ease of access and use.
- Reduce severance effects by providing an alternative to the SH1 bridge.
- Efficient movement and a reliable route.

Safety

- Ensure safety for all transport modes.
- Including perceived risks to personal safety (CPTED).
- Design for an ideal scenario: children walking or biking to town on their own.

Choices

- Design for multi modal transport from the outset.
- Potential to encourage more people to walk, cycle or take public transport.

People

- Create a corridor that has great user experiences.
- Cater for all.
- Enhance sense of place and community as Tinwald Grows.
- Build in expression & narrative to encourage participation and stewardship.
- Benefits to health and wellbeing with improved transport choice.

Landscape / Environment

- Streetscape that responds to current, and more importantly future changes in character and land use.
- Streetscape and bridge deck that is visually appealing & easy to maintain.
- Limit CO2 emissions with mode shift.
- Mitigation of landscape and ecological effects.

Figure 2-2: Key urban design and landscape principles

2.3.1 Urban Design Guidance for Bridges

Bridging the Gap (Waka Kotahi New Zealand Transport Agency, 2013) provides guidance on urban design for road bridges. The guidance acknowledges that bridges exist to connect transport networks. However, they can also support linkages between communities, offer new opportunities for viewing and appreciating the landscape, and are strong landscape features in their own right. Bridges can have a significant impact on the driving as well as the viewing experience and good bridge design will enhance both. In the bridge design process, a key issue is often the balance between cost and design quality. Design quality is more than aesthetics. It includes appropriate form and scale for the specific location, amenity for road users and others who may travel over or under the bridge, accessibility for pedestrians and cyclists, the integration of abutments in the landscape, personal safety and resource efficiency. Cost should be considered over the life of the bridge and in relation to the environmental, social and cultural benefits offered. A well-considered bridge will create a well-connected transport facility which incorporates good landscape design and enhances views. It will make a positive contribution to surrounding communities as well as to road users.

Location: Bridge design starts with its location. Bridges that span waterways can dramatically change the landscape and bridges within or next to residential areas can appear out of scale and out of character. The role of the bridge in the overall project must be established from the early stages of route selection as it can influence the alignment.

Context: Bridges should complement their context. This means considering the topography, the rural or urban setting, any existing structures, visibility of the bridge and the distance and height to be spanned. Where a series of bridges will be seen in succession by road users, they should be consistent in form and recognizable as a 'family' of structures with individual variations reflecting the requirements of their specific settings. Feature bridges are suitable for special places, where they can act as landmarks.

Views: Bridges are both viewed objects and viewing platforms. The bridge can frame a new and unexpected vista contributing to appreciation of the surrounding landscape. Optimising views to, through and from the bridge will also help with orientation on the journey. This can be achieved by making the bridge design as slender and open as possible and minimising the height of solid barriers by using a top metal rail. Bridges that are highly visible from roads and public spaces should be designed for these views.

Underbridge experience: Where pedestrians and cyclists are likely to travel under a bridge, the treatment of the soffit, piers and abutments should provide a safe, convenient and attractive environment. In urban areas with high levels of foot traffic, the underbridge experience will be particularly important and justify architectural treatments and feature lighting.

Overbridge experience: Where a bridge provides an elevated viewpoint from which the wider landscape can be appreciated or crosses an important landscape feature (river, gully, etc), the overbridge experience should be carefully considered. This may involve using a metal top rail to minimise the height of solid bridge barriers and maximize the view from the bridge for motorists. If pedestrians and cyclists are likely to travel over the bridge, it may be desirable to provide space where they can safely stop and enjoy the view.

Form and proportion: The height of the bridge, number of supports, distribution of spans and size of the various components should be carefully considered to create a simple, elegant whole and to minimise the bridge profile. Structural integrity, where the forces at play in the bridge are clearly reflected in its design, generally results in pleasing composition.

Light and shadow: A play of light and shadow on a bridge can reduce the apparent mass and bulk of the structure and balance its vertical and horizontal proportions. Sloping all or part of the outer face of the parapet outwards to catch the sunlight, and recessing beams to create a shadow line, will reinforce the horizontal lines in the bridge. Surface texture on barriers and retaining walls will create a finer level of detail.

Texture: Barriers should have minimum embellishments, with any surface patterns reinforcing the clean lines of the bridge. Any textures on retaining walls and barriers should relate to the speed of travel. Abstract, repetitive patterns are suitable to add interest, while not distracting drivers. Where abutments will be visible by slow moving traffic, textures can be used on retaining walls to provide a finer level of detail and can reference the area's cultural or historical significance.

Colour: Colour offers opportunities to provide consistency to a family of bridges and to reinforce the landmark quality of a standalone structure. When used to highlight particular elements it should form part of a coherent, ordered composition. Colour must be used carefully as it draws the eye, especially in a rural setting.

Lighting and drainage: These bridge components must be considered early and integrated in the design of the structure. The external surface of the bridge should be free of drainage pipes or services. Lighting at night, like colour during the day, can be used to highlight all or parts of a feature bridge. Lighting design and selection should incorporate protection against vandalism.

Maintenance: It is important to select durable materials and finishes that do not significantly degrade in appearance over time. Where required, anti-graffiti coating should be applied as part of the bridge construction phase to the full extent of piers and barriers to prevent patchy application and appearance at later stages.

Barriers: Barriers must be designed to respond to the bridge setting and to achieve a smooth transition between the structure and its approach. Barriers should have continuous lines that are not obscured or interrupted by non-structural elements. Their depth must be carefully proportioned in relation to the deck and superstructure. Barriers should be extended past the abutments to anchor the bridge in the landscape. Sloping the top of the barrier inwards towards the deck will minimise water staining on the outer face of the barrier.

Abutments: Open abutments should generally be used in rural areas to optimise views of the landscape. Landscaped sloped abutments are less likely to attract graffiti than retaining walls. In urban settings or when the corridor width is constrained, near vertical or vertical retaining walls are the most practical abutment options. The design of these retaining walls must present a high quality appearance if visible to approaching traffic, pedestrians and cyclists.

Headstock: These substructure elements should not be designed in isolation. Their design is integral to the overall form of the bridge. Structural systems that eliminate the need for headstock can lead to simpler, more elegant solutions.

This guidance is relevant in that good urban design starts at the structure itself, and that urban design on the bridge isn't just 'dressing', it is an integrated multidisciplinary approach.

2.3.2 Landscape & Urban Design for the Ashburton River / Hakatere Bridges

Elements of landscape & urban design that are relevant to the bridge crossing include:

- The bridge will be the third in a series of bridges over the Ashburton River / Hakatere, with the rail bridge being the furthest west, the SH1 road bridge being in the middle, and the new bridge 800m east of the SH1 bridge. The bridges are to read as a series.
- That the bridge fits into the local landscape context – broadly the flat open Canterbury Plains.
- Landscape character influences the design of the bridge approaches, bridge itself and the under-bridge experience.
- The ecological context is considered in landscape and bridge design, especially with regard to using appropriate lighting and pier design to minimize effects on shorebird habitat, and to undertake restoration of planting in the riparian zone and river terraces of the Ashburton River / Hakatere. New embankments are also to be in native vegetation.
- The bridge form has a:
  - Modern highway design
  - Piers only as necessary to avoid excessive excavation and disturbance to river habitat
  - Abutments as natural 'spill through', potentially clad in local rock where planting isn't possible due to low light
  - Steel parapet rail to reinforce a light bridge form
  - A TL4 concrete parapet to tie into the series of bridges, and consider parapet design finishing
- The bridges may be gifted names by Te Runanga of Arowhenua. The Ashburton River / Hakatere is seen as a highway in itself, and the bridge is a Mōkihi (canoe) to traverse the river.

- Provision for pedestrians and cyclists to encourage transport choice.
- Adequate lighting for not only the vehicle lanes (alternating pattern), but at human scale to enhance the customer experience for people cycling and walking.
- Option for art and graphic designs to be added on the concrete barriers (internal and /or external faces), as well as treatment to the soffit, piers and abutments with patterns to be gifted and co-designed by Arowhenua local artists.
- Trails are provided to embankment approaches on both sides of the bridge to connect to existing river trails on the riverbanks.
- Focus areas / stopping places are to be provided at key points on the bridge approaches and under the bridge, that include, as a minimum:
    - access points to under bridge,
    - low planting and specimen trees,
    - wayfinding (recommend trail maps and trail markers),
    - resting places / seating,
    - cycle infrastructure, and
    - adequate lighting.

2.3.3 Low Maintenance Design

- The design is to be uncomplicated and coordinated, minimising the number of, and using local materials.
- Materials and finishes selected are robust and are sourced locally wherever possible.
- The design is to minimise opportunities for vandalism through CPTED measures. Graffiti deterrent will be through the use of textured finishes on concrete structures. Early reporting and removal will reinforce stewardship and low tolerance of graffiti. Where required by ADC Graffiti Guard can be added.
- Design and finishing for the bridge, culvert retaining walls and any other structures are precast concrete panel units ensuring uniformity and availability. Any patterns should be cut into, or sand blasted onto materials for permanence.

2.3.4 The Bridge & Urban Design Principles

In summary, how the following design interventions and features for the bridge achieve design principles:

Connectivity

- In connecting Tinwald & Ashburton, via an alternative route, the bridge and link road are key for ADC in achieving aims in the District Plan and District Strategies, particularly ‘Its our Place’, Walking & Cycling Strategy, Ashburton Transport Activity Management Plan, Ashburton River / Hakatere Shorebird Habitat Management Plan and the Ashburton District Sport & Recreation Strategy.
- Ensure ease of access and use by; providing regular crossing points, integrating activities and connecting people to networks though ‘Focus Areas’, providing a consistent unbroken path as a linear network.

Safety

- Bridge design to the Highway Structures Design Guide (Waka Kotahi, 2016), the Bridge Manual SP/M/022 (Waka Kotahi, 2013) and best practice structural design guidance.
- Landscape design responds to CPTED matters: integrating paths along the roads & bridge to avoid grade separation, that will benefit from informal surveillance; generous off road paths; proposes pedestrian level lighting on the road and bridge; long site lines using low planting & clear stem trees; promote high usage; and use hard wearing robust materials.

Choices

- Provide a choice of route for people travelling between Ashburton and Tinwald,
- Design for a change in transport mode with footpaths / shared paths.

People

- The Bridge, Roundabouts and crossing places become a focus of the corridor - where people can stop and interact with each other, with the natural environment and with amenities.
- Use materials, forms and plants that are of Mid Canterbury to enhance character and sense of place.

- Build in expression & Arowhenua narratives and names, gifted by Arowhenua into the bridge and roundabout islands.

Landscape / Environment

- Integrate bridge embankments and restore flood protection with native planting and rationalise access to existing trail networks,
- The bridge is part of a series of three, a modern highway bridge,
- Protecting bird habitat on the Ashburton / Hakatere River with relevant construction methodology and sensitive lighting.

2.4 GEOMETRICS

New bridge crossings require longitudinal drainage in order to carry rainfall run off to the abutments. The Waka Kotahi Bridge Inspection and Maintenance Manual, section 7.2.1, provides the following recommendations for deck drainage:

- Minimum longitudinal deck grade of 0.5%
- Minimum longitudinal gutter grade of 1%

A single vertical curve was adopted for the recently built Ashley Cones Bridge (Waimakariri District). Although the minimum deck and gutter grade limits are not met at the central spans, the deck grade increases gradually towards each abutment as the cumulative rainfall runoff increases. We have therefore adopted the following for this phase of the project:

- The primary bridge has a straight horizontal alignment
- The primary bridge has a vertical curve through the central spans transitioning to a target longitudinal grade of 1% minimum at each abutment
- The secondary bridge (Tinwald flood plain) has a target longitudinal grade of 0.5% at each abutment which could be either a single longitudinal grade or a peak at the central pier
- The formed approaches for both bridges have a combination of vertical and horizontal geometry to tie in with the Ashburton and Tinwald approach road alignments

Actual runoff flows and hydraulics should be further assessed as part of detailed design.

2.5 HYDROLOGY

2.5.1 Flood Estimation

This work uses material sourced from the Environment Canterbury Surface Water Archive, which is licensed under a Creative Commons Attributions 4.0 International license by Environment Canterbury (ECAN).

The Ashburton River catchment experienced a significant flood event on 30 May 2021. Following this event ECAN updated their flood estimation and provided the following data table to Stantec.

Table 2-1: ECAN 2021 flood frequency estimate for Ashburton River at SH1

Distribution	Max flow (year)	Flow estimate (m^3/s)								
		Mean annual flood	5 year	10 year	20 year	50 year	100 year	200 year	500 year	1000 year
TCEV (with historic events)	1794 (2021)	349	426	558	706	970	1275	1656	2190	2596
Rounded estimate		350	430	560	710	970	1280	1660	2190	2600

In addition to the above, ECAN also provided an extrapolated estimate of the 1500 year and 2500 year events derived from the graph below.

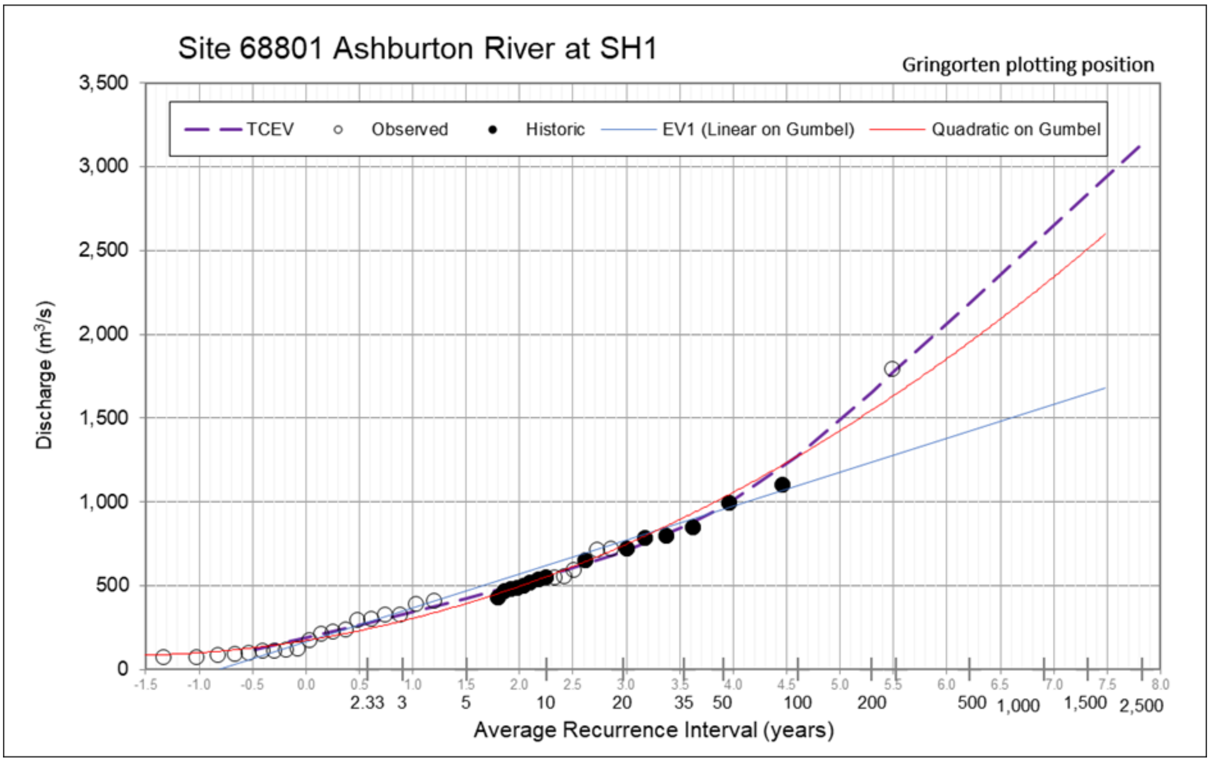


Figure 2-3: ECAN extrapolated flood event estimate

From this extrapolation ECAN estimated the following flood events:

- 1:1500 year – 2800 m^3/s
- 1:2500 year – 3100 m^3/s

As recommended by ECAN, Stantec carried out a review of the ECAN flood estimation which is attached in **Appendix A**. This review provided the following conclusions:

- The TCEV distribution results provided by ECAN are considered appropriate, and it is recommended that the 1:1500 and 1:2500 APE values of 2,800 m3/s and 3100 m3/s be adopted.
- Uncertainty associated with estimating extreme flood peaks is large and so an appropriate range of design flows should be considered during the design process.

2.5.2 Design Events

The NZTA Bridge Manual V3.4 (NZBM) Table 2.1 provides the annual probability of exceedance (APE) for various design events according to the Importance Level (IL) adopted. We have adopted IL4, based on:

- The bridge cost estimate is currently > $18M
- This structure will be of high importance to post-disaster recovery

For a permanent structure this requires consideration of the following design return periods for flood events:

- SLS1 (service limit state) flood water action of 1:25 year APE. For this design event the structure and approaches shall be designed to sustain no significant damage.
- SLS2 (service limit state) flood water action of 1:100 year APE. For this design event the structure shall pass the flood flow while complying with the minimum freeboard (1.2m for large waterways that may carry trees).
- ULS (ultimate limit state) flood water action of 1:2500 year APE. For this design event collapse of the structure shall be avoided and overtopping of the stop banks (secondary flow paths) need to be considered.

We have also spoken to ECAN regarding their stopbank design criteria. They have indicated that a reasonable design check to adopt would be:

- 1:200 year APE
- Plus 30% for climate change (RCP 8.5)
- Plus 0.5m freeboard

2.5.3 Climate Change

The effects of climate change are determined and based on the Ministry for the Environment Climate change projections for New Zealand. The NZBM advises that an RCP (representative concentration pathway) value of 6.0 is adopted with sensitivity testing to RCP 8.5.

We have therefore adopted the following values from the climate change projections:

- From Table 14, the land-average temperature increase in the years 2101-2120, relative to 1986-2120, is 2.32 degrees for RCP 6.0 and 3.13 degrees for RCP 8.5.
- From Table 13, the percentage increase in rainfall depth for a 100 year APE and a 12 hour storm duration (estimated time of concentration for this catchment) is 10.1%
- The following table summarises the various flood events.

Table 2-2: ECAN 2021 flood frequency estimate for Ashburton at SH1

Design Event APE	Design flood (m^3/s)	Including climate change adjustment RCP 6.0 (m^3/s)	Including climate change adjustment RCP 8.5 (m^3/s)
SLS1 (25yr)	750	927	1014
SLS2 (100yr)	1275	1576	1723
ECAN (200yr)	1656	n/a	2238
ULS (2500yr)	3100	3831	4189

It should be noted that the May 2021 flood event had a peak flow of 1794 m3/s, very close to the SLS2 design events with climate change included. ECAN carried out a post flood survey and picked up the indicative flood water level (based on flattened grass and deposited debris) which has been very useful for checking the results of the hydraulic analysis.

2.5.4 Riverbed Morphology

The Ashburton River has been the focus of several reports and studies produced by both ECAN and WKNZTA over the last 50 years. These two parties have commonly worked in opposing direction, specifically:

- ECAN have pursued gravel extraction as a means of lowering bed levels and increasing flood capacity
- WKNZTA have pursued cessation of some extraction in order to limit the risk of further degradation and undermining of the SH1 bridge foundation (shallow piled)

Further information can be found in the following documents:

- WSP, SH1S Ashburton River Bridge, Damage and Emergency Response Report, 2021
- OPUS, SH1S Ashburton River Bridge, Bridge Scour and Sediment Management Summary Report, 2013
- ECAN, Gravel Management Framework, 2012
- OPUS, Ashburton River Gravel Extraction – Impact on SH1 and South Island Main Trunk Rail Bridges, 2009
- OPUS, SH1: Ashburton River Bridge, Review of Scour Risk, 1994

The graph below shows historic variation in MBL (mean bed level). Between 1937 and 1981 the bed levels dropped in the order of 1-2m due to intensive gravel extraction. In 1979 rock protection was installed around the SH1 bridge piers to reduce the risk of undermining. From the 80’s until present day the majority of gravel extraction consents have been focus on Blands Reach (approximately 17km upstream of SH1), and as a result, bed levels around SH1 have stabilised.

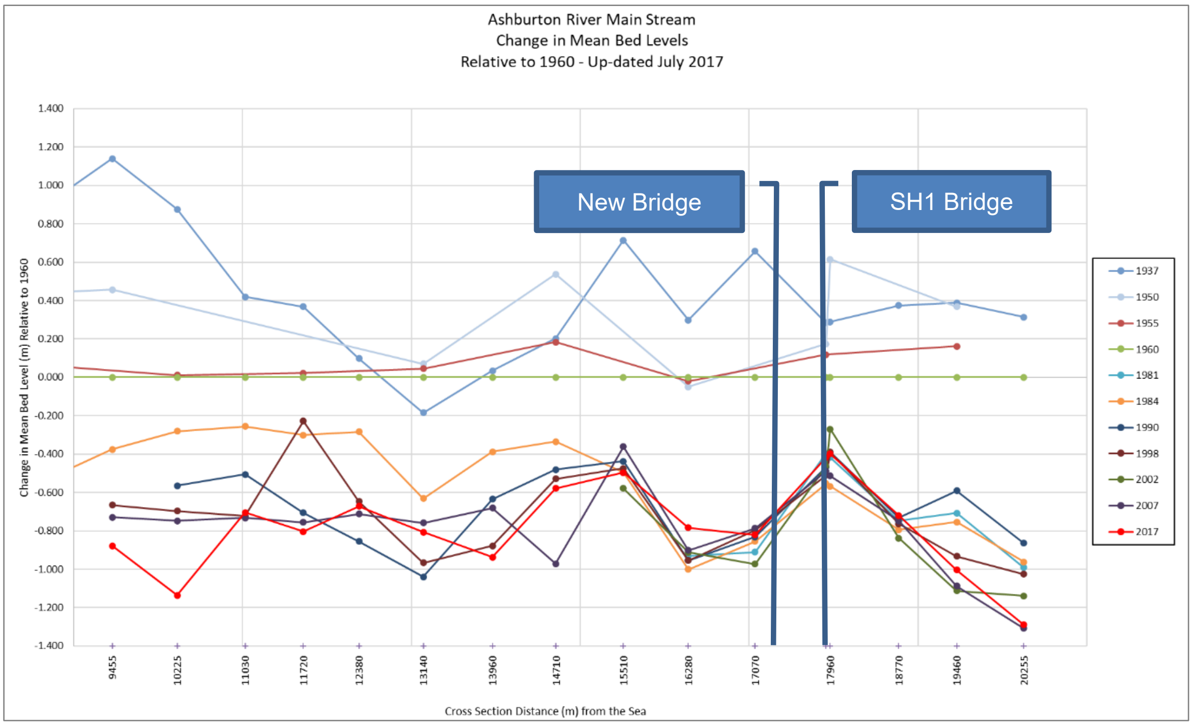


Figure 2-4: ECAN mean bed levels

To allow for future variations in bed level at the location of the proposed bridge we have adopted the following values for long term general scour:

- 0.5m of aggradation, allowing for a 50% return to 1937 bed levels
- 0.5m of degradation, allowing for intensive gravel extraction to restart after the SH1 bridge is eventually replaced

2.6 HYDRAULICS

2.6.1 Sections

Hydraulic sections have been produced at 100m intervals for 1km upstream and downstream. These sections are based on a combination of:

- Lidar ground levels (for the majority of the section data)
- ECAN survey levels (checking of LIDAR sections 800m upstream and 100m downstream of the new proposed bridge alignment)
- ECAN post flood survey levels (providing level data for the Tinwald stopbank)
- Stantec survey levels (checking of LIDAR section on the proposed bridge alignment)

The lidar data was heavily affected by tall tree blocks on both banks. Stantec staff carried out a site walk over to check the general ground alignment below the tree canopy level and determined the following guidelines for adjusting the sections:

- The TR main channel bank is typically 0.5m above the main channel maximum bed level
- The TL main channel bank is typically the same as the main channel maximum bed level
- Ground slope from the main channel to the Tinwald flood plain is typically flat or slightly graded, falling from the flood plain toward the TR main channel bank

The ECAN and Stantec surveyed cross sections allowed for direct comparison of the lidar (which does not pick up levels below water level) with actual bed levels. Comparison of these two data types indicated that actual surveyed MBL (mean bed level) in the main channel was typically 0.25m below that determined from lidar. A uniform 0.2m level reduction has therefore been applied to all lidar derived main channel bed levels in order to reflect the correct waterway area for hydraulic modelling.

Due to a short fall in the lidar data set, a significant proportion of the Tinwald flood plain was excluded from our hydraulic sections. We therefore adopted the ECAN post flood stopbank survey levels to manually extend each of the sections.

For modelling we have adopted the following Manning n roughness coefficients:

- Main river channel, n=0.03, from Hicks and Mason (Clarence River) rounded from 0.028
- Tinwald flood plain, n=0.035, from Eng Toolbox and Researchgate online
- Tinwald tree block, n=0.165, from US Geological Paper 2339
  - nb = 0.03 based value for firm soil
  - n1 = 0.015 degree of irregularity, badly sloughed and scalloped
  - n2 = 0.00 variation in channel, gradual
  - n3 = 0.02 obstructions, appreciable 15-50% of the cross section
  - n4 = 0.10 vegetation, very large trees intergrown with weeds and bush

The figure below shows the riverbed section at the proposed bridge location in blue. The stepped water level is shown in pink.

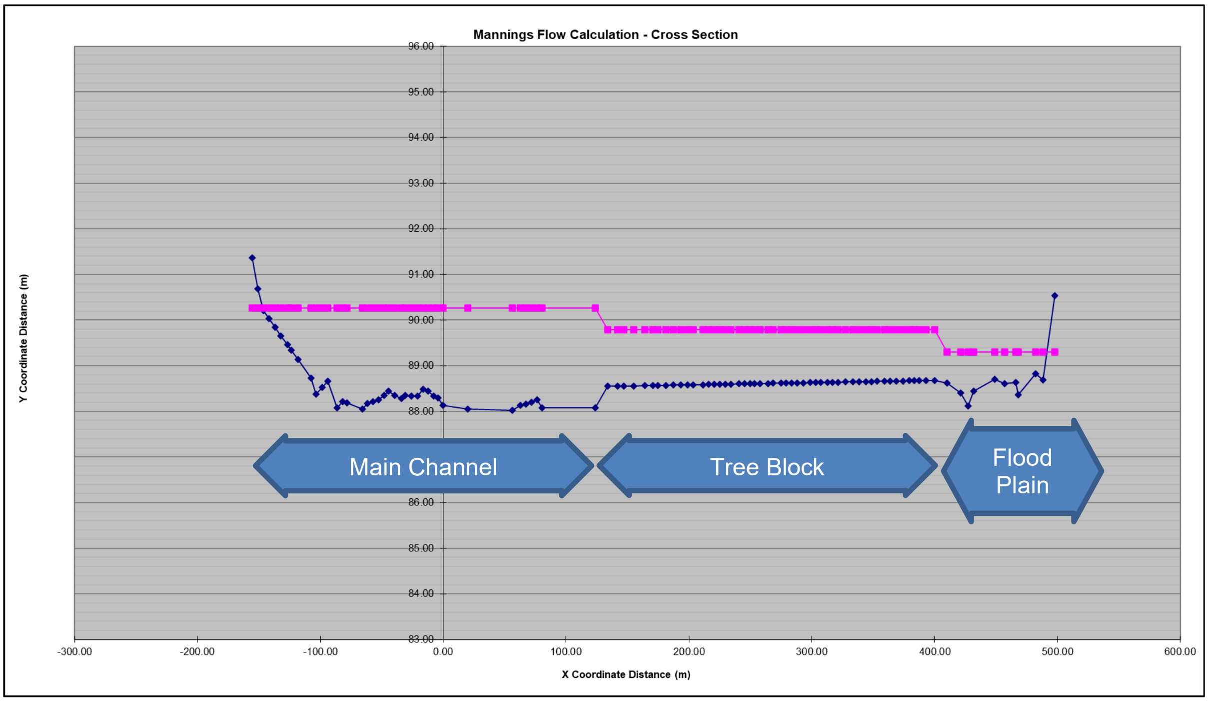


Figure 2-5: Typical hydraulic cross section at proposed bridge

2.6.2 Surveyed FWLs

Following the 2021 flood, ECAN surveyed the stopbanks and indicative flood water levels (FWLs). The results from this survey are shown in the figure below. From this graph the following observations were made:

- The Tinwald stopbank levels are generally around 1-1.5m lower than the Ashburton stopbank levels at the same chainage
- The Tinwald FWLs are generally around 0.6-2.0m lower than the Ashburton FWLs at the same chainage
- The flood freeboard on the Ashburton side varied from 0.6-2.0m
- The flood freeboard on the Tinwald side was 0.3-2.5m
- Two of the FWLs on the Tinwald side are higher than upstream FWLs, indicating that one or other is likely to be in error, in particular the point at CH-80m immediately upstream of the proposed bridge with only 0.3m freeboard (red circle)

Stantec visited the property at CH-80 and spoke with the owner (Cameron Ross) about flood water levels observed on the day of the flood. Cameron provided a photo indicating a minimum freeboard of 0.6m and highlighting errors in the surveyed FWLs.

Stantec also visited the property at CH+80 and spoke with the owner (Stuart Cross). Stuart provided photos and videos indicating a peak FWL of 88.9, in reasonable agreement with the graph below (orange circle).

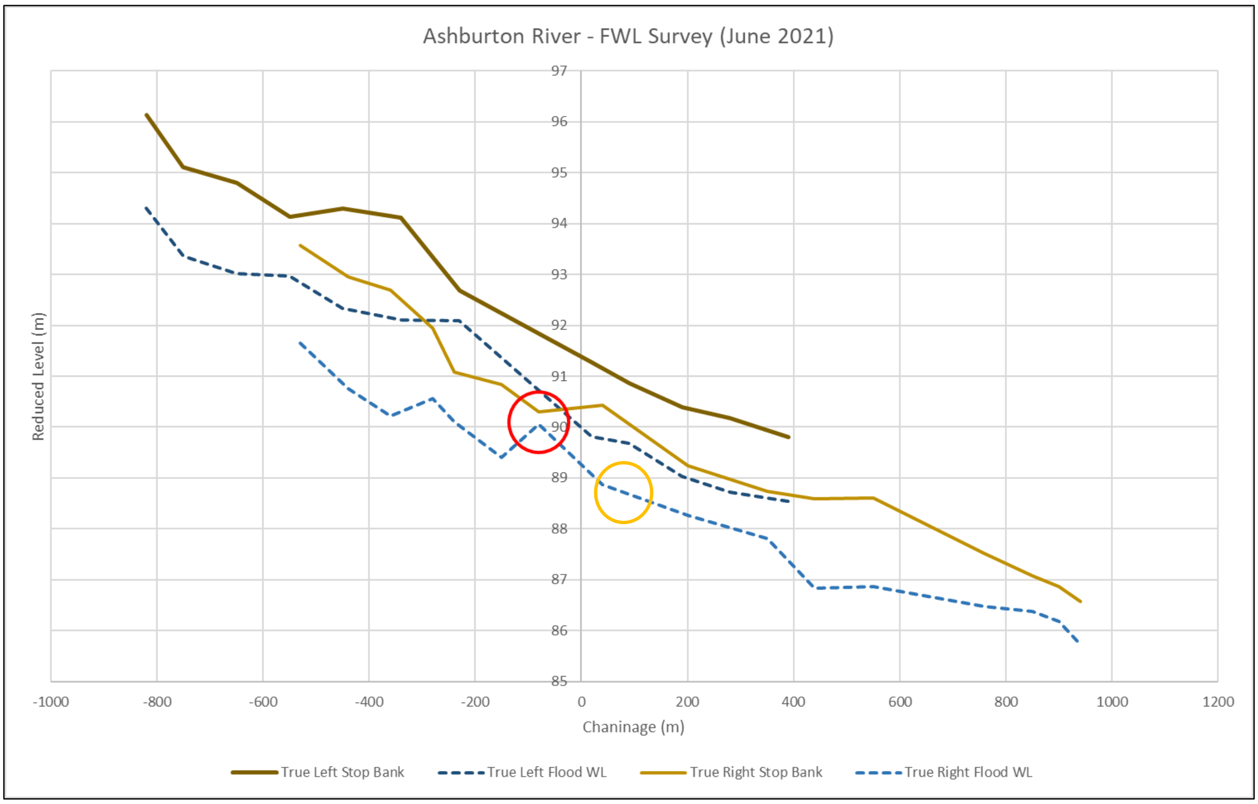


Figure 2-6: ECAN 2021 flood water level survey

One of the key observations from this survey data is that the Tinwald flood plain has FWLs that are significantly lower than those in the main channel of the river. The obvious conclusion from this is that the heavily vegetated block of trees on the south side of the river is acting as a hydraulic separation between the main channel and the flood plain. The flood plain is in effect providing a drainage path to remove any flow pushing through the tree block. The flood plain will therefore require adequate flow capacity through the new approach formation in order to avoid damming the flood plain.

2.6.3 Backwater Analysis (1D)

The sections derived above were used in a 1D backwater (BW) analysis. This analysis models the design flood events (Table 2-2) from 1km upstream to 1km downstream of the proposed new bridge alignment. To allow for the variation in flood water level between the main channel and the flood plain, the water level is stepped down up to 0.6m in the tree block and up to 1.2m in the Tinwald flood plain.

Calibration Check

The graph below shows initial 1D modelling and calibration check of the 2021 flood event peak flow (1794 cumecs), compared to the surveyed FWL indicators from ECAN. The new bridge is at chainage 0, denoted by the vertical red line. This shows reasonable agreement between the model and observations. The high surveyed FWL at CH-230 could be an error, given the very flat hydraulic grade back to CH-340.

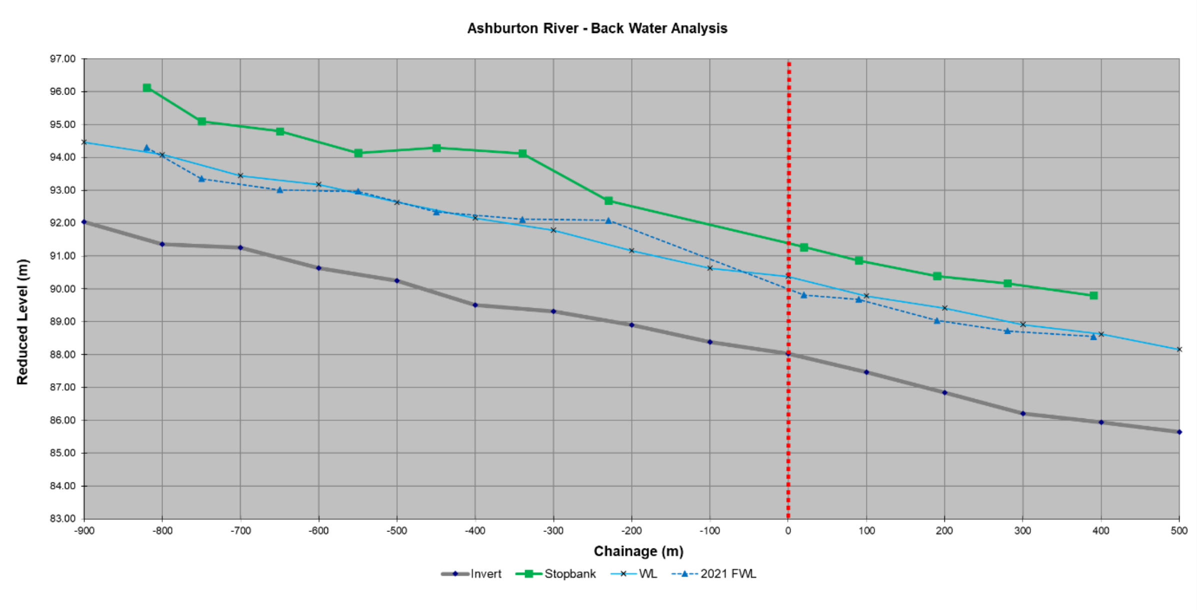


Figure 2-7: 1D modelling and calibration of the 2021 flood peak

SLS1 – Q25 + RCP6 (+ 360 bridge + 60 bridge)

The 1D model of the SLS1 design flow, including climate change RCP6 (927 cumecs), shows the following:

- Average water velocities of 2.1m/s under and the new bridge
- Peak water velocities of 2.6m/s in the deeper river channels

Under these conditions we would expect some gravel movement in the general bed width plus moderate localised scour at hard points, like piers and abutments. This is unlikely to pose any risk of significant damage and can be addressed in the detailed design with the foundation depth and rock protection works.

SLS2 – Q100 + RCP6 (+ 360 bridge + 60 bridge)

The graph below shows 1D modelling of the SLS2 design flow, including climate change RCP6 (1576 cumecs), showing FWLs with no bridge compared to FWLs with the new bridge in place. For this analysis we have adopted the following bridge lengths:

- Main channel bridge length of 360m (20m longer than the existing SH1 bridge upstream)
- Flood plain bridge length of 60m (assuming 10-15% of the total flow will drain to the flood plain)

This analysis shows that the new bridge creates a backwater effect, lifting the water level by 115mm at 100m upstream, reducing to near zero (3mm) at 200m upstream.

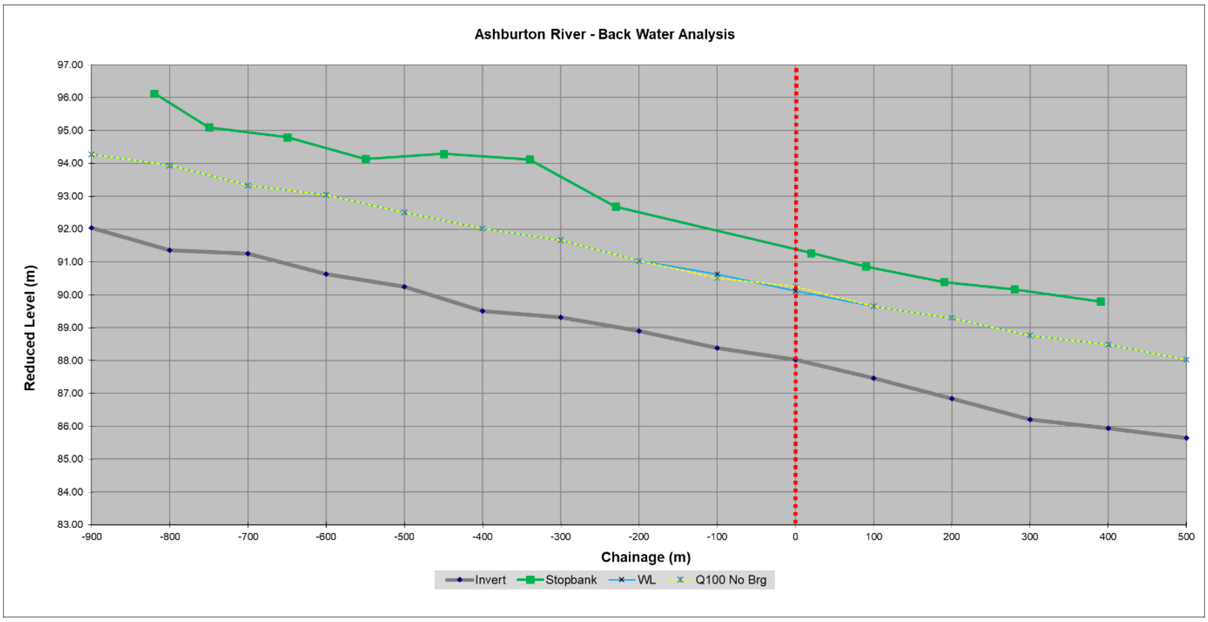


Figure 2-8: 1D modelling of backwater effect due to the Q100 plus new bridge

Further sensitivity testing was carried out as follows:

- Reducing the main channel bridge length to 330m creates an increased backwater effect, lifting the water level by 156mm at 100m upstream, reducing to 12mm at 200m upstream
- For the Q100 + RCP8.5 (1723 cumecs) with the 360m + 60m bridges, FWLs increase by around 90mm compared to RCP6. Back water effects from the 360m and 330m bridge options are similar to those for RCP6.

ECAN Stopbanks – Q200 + RCP8.5

The graph below shows 1D modelling of the ECAN stopbank design flow, including climate change RCP 8.5 (2238 cumecs), without the proposed bridge. This indicates a 400mm lift in water levels when compared to the Q100 + RCP6. ECAN have indicated they require a minimum freeboard of 0.5m for the Q200 + RCP8.5 and the model shows available freeboard on the Ashburton side of 0.7-2.0m over the 1200m length of surveyed stopbank, with the lowest freeboard being downstream of the proposed new bridge crossing (see orange circle).

With the shortest 330m bridge option in place over the main channel, this creates a backwater effect, lifting the water level by 282mm at 100m upstream. However, the model still shows available freeboard of 0.7-2.0m with this shorter 330m bridge in place. This indicates that the current Ashburton stopbanks are adequate for even the shortest bridge option and may not need to be adjusted to address the back water effect of the new bridge.

If we adopt a rough order 1m FWL difference between the main channel and the Tinwald flood plain, this indicates that freeboard to the Tinwald stopbank will be in the order of 0.0-0.4m. This falls below the 0.5m minimum freeboard and will require further discussion with ECAN and a more detailed hydraulic analysis to define the Tinwald flood water levels with improved accuracy.

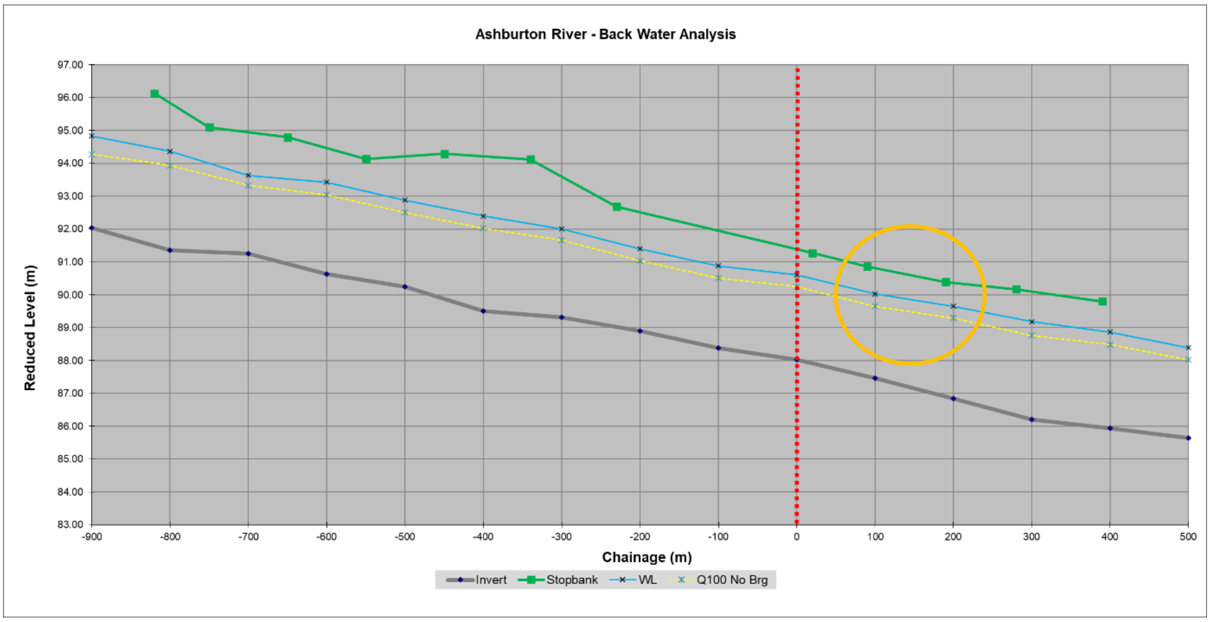


Figure 2-9: 1D modelling of Q200 + climate change RCP8.5 (no bridge)

ULS – Q2500 + RCP6 (+ 360 bridge + 60 bridge)

The graph below shows 1D modelling of the ULS design flow, including climate change RCP6 (3831 cumecs), showing FWLs with no bridge compared to FWLs with the new bridge in place. For this analysis we have adopted the following bridge lengths:

- Main channel bridge length of 360m (20m longer than the SH1 bridge upstream)
- Flood plain bridge length of 60m (assuming 10-15% of the total flow will drain to the flood plain)

This analysis shows that:

- The ULS design flow with no bridge is just contained within the existing Ashburton stop bank height
- The new bridge creates a backwater effect, lifting the water level by 409mm at 100m upstream, reducing to near zero (9mm) at 500m upstream
- The ULS design flow with the new bridge in place, and resulting backwater effect, is marginally contained within the existing stop bank height

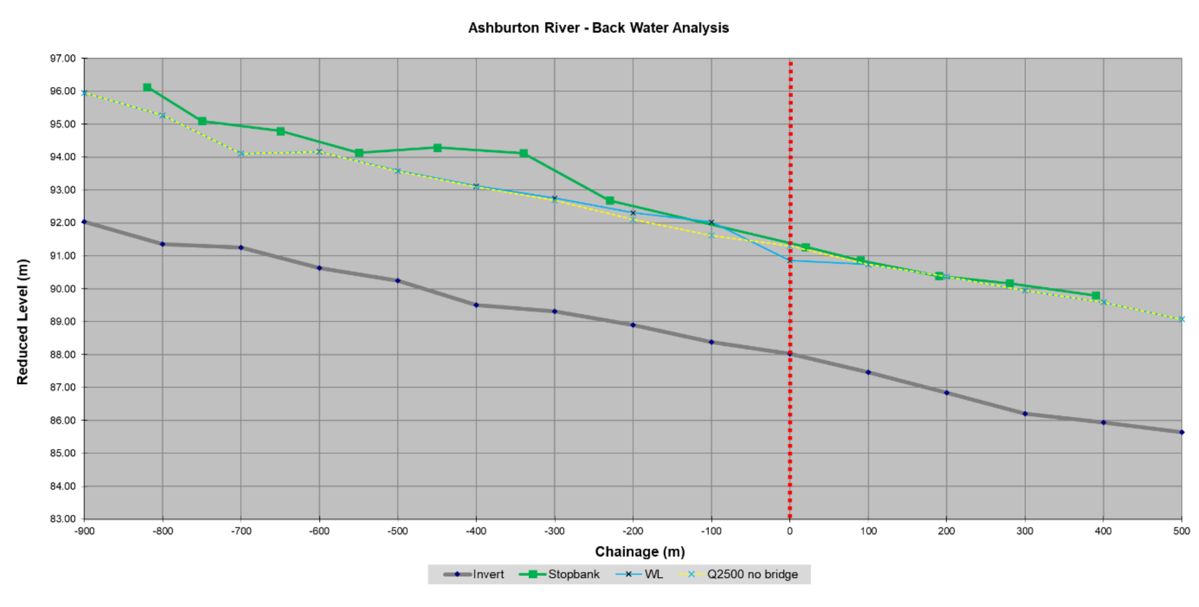


Figure 2-10: 1D modelling of backwater effect due to the Q2500 plus new bridge

Given this is our extreme ULS design case and the stop banks are on the limit of overtopping (and are likely to be overtopping in other locations upstream and downstream), we have not included any further sensitivity testing.

2.6.4 Summary

To summarise the above sub-sections, the 1D modelling indicates:

- reasonable calibration agreement with FWLs surveyed from the 2021 flood event
- that the SLS1 design flood event is unlikely to cause significant damage
- that the SLS2 design flood event will result in only minor backwater effects from constriction caused by the proposed new bridge
- that the ECAN stopbank design flood event has compliant freeboard on the Ashburton side (greater than 0.5m minimum), including allowance for any backwater effects from the new bridges
- that the ECAN stopbank design flood event does not have compliant freeboard on the Tinwald side (less than 0.5m minimum), including allowance for any backwater effects from the new bridges
- that the ULS design flood event is likely to result in only minor overtopping of the Ashburton stopbank

For determining primary bridge soffit freeboard we have adopted the following:

- Q100 RCP6 flood water level of RL90.25 from 1D model with no bridge (slightly higher than the surveyed FWL of RL89.82)
- Freeboard from table 2.5 of the NZBM
    - 1.2m minimum over the main channel (220m wide)
    - 0.6m minimum over the tree block and vegetation either side of the main channel
- Allowance for aggradation, 0.5m allowing for a 50% return to 1937 bed levels
- Adopted minimum soffit levels
    - RL91.95 minimum over the main channel (220m wide)
    - RL91.35 minimum over the tree block and vegetation either side of the main channel

2.6.5 HECRAS Analysis (2D)

PLACEHOLDER 2D analysis still being completed

2.7 SCOUR

2.7.1 General

Scour is typically made up of the following

- General scour, long term aggradation or degradation of the bed level, typically due to variations in gravel transport and extraction
- Constriction scour, due to narrowing of the waterway where the approaches of a bridge sit within the stop banks of a river
- Bend scour, due to directional changes in the river geometry, assumed to be zero for this project given the proposed bridge is on a straight section of riverbed
- Local scour, due to piers and debris rafts placed within the flowing channel

Total scour is a summation of all the above components.

We have carried out a preliminary scour assessment using the methodology given in the Austroads Waterway Design Guide and the NZBM. A more detailed assessment will be required as part of detailed design using the document ‘Scour’ (by Melville and Coleman).

2.7.2 General Scour

As discussed in section 2.5.4 above, since monitoring began in 1937, the riverbed level has dropped by around 1m at the location of the proposed bridge. However, the bed level has been maintained at a relatively constant level since 1981 due to careful limitation of gravel extraction consents downstream of Blands Reach.

For the purposes of this report, we have proposed adoption of general scour of 0.5m. This allows for gravel extraction to restart after the SH1 bridge is eventually replaced (to improve flood capacity) but assumes that extraction limits with still be actively monitored and controlled.

2.7.3 Constriction Scour

Constriction scour is caused by a localised narrowing of the waterway where the approaches of a bridge cut off a portion of the waterway width. This requires the wide-spread flood flow to accelerate through the narrower waterway leading to scour of the bed within the constriction under the bridge.

Following the simplified approach given in section 6.4.3 of the Austroads Waterway Design guidelines, this equates to:

Constriction scour, Ysc = 0.35m

2.7.4 Local Scour

Local scour is caused by flood water hitting a hard immovable object in the waterway. This creates turbulence and accelerated flow around the object resulting in a localised increase in scour.

Following the simplified approach given in section 6.4.4 of the Austroads Waterway Design guidelines, this equates to:

Local scour, Ysl = 3.84m

Local scour can also develop around a debris raft that attaches to one of the piers. This makes the effect pier width wider resulting in an increase in scour.

Following the equations given in section 2.3.5 of the NZBM, and adopting a rectangular raft, this equates to:

Equivalent pier width of 3.33m (cf 1.5m for the actual caisson diameter)

Local raft scour, Yslr = 6.45m

2.7.5 Summary

Based on summation of the above scour components, the piers on the proposed bridge will be subject to an estimated total scour made up as follows:

General scour	0.50 m
Constriction scour	0.35 m
Local raft scour	6.45 m
Total scour	7.30 m below current bed level

3. DESIGN OPTIONS

3.1 BRIDGE LENGTH

As part of our hydraulic modelling, we have considered a range of bridge length options, as detailed in the table below.

Table 3-1: Bridge length options considered

Option	Description/Comment
1. Single bridge – 420m	• Adopted from the previous consultant's design report • The bridge is positioned over the main river channel and also includes around 50% of the tree block on the true right bank • Provides a waterway width 24% greater than that of the SH1 bridge • Potentially blocks or limits drainage of the Tinwald flood plain
2. Double bridge – 360m + 60m	• A primary bridge (360m long) is positioned over the main river channel and also includes around 25% of the tree block • A secondary bridge (60m long) is positioned over the deeper natural channels in the Tinwald flood plain • Provides a total waterway width 24% greater than that of the SH1 bridge • Provides a main channel waterway width 6% greater than that of the SH1 bridge • Provides significant drainage and flow capacity for the Tinwald flood plain
3. Double bridge – 330 + 60m	• A primary bridge (330m long) is positioned over the main river channel and also includes around 15% of the tree block • A secondary bridge (60m long) is positioned over the deeper natural channels in the Tinwald flood plain • Provides a total waterway width 15% greater than that of the SH1 bridge • Provides a main channel waterway width 3% less than that of the SH1 bridge • Provides significant drainage and flow capacity for the Tinwald flood plain

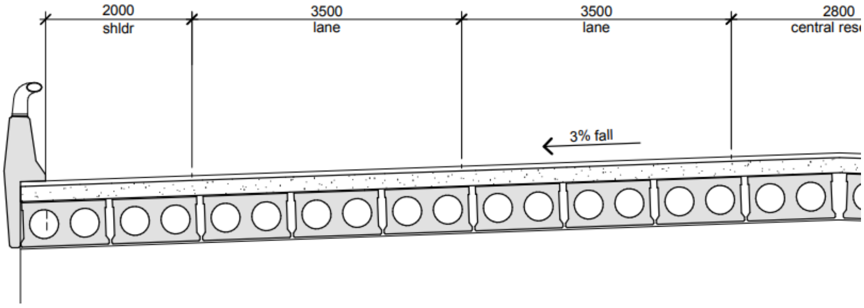
Both option 2 and 3 provide realistic crossing alternatives, however, we have adopted option 2 for the purposes of this report as it provides a more conservative waterway with scope to adjust the split between the primary and secondary bridge. Further work is recommended to optimise the proposed bridge lengths, specifically:

- A more detailed hydraulic assessment of the waterway to better understand the flow split between the main channel and the Tinwald flood plain
- Discussions with ECAN over other flood mitigation measures
- Discussions with ECAN and ADC regarding the risks of tree removal from the waterway

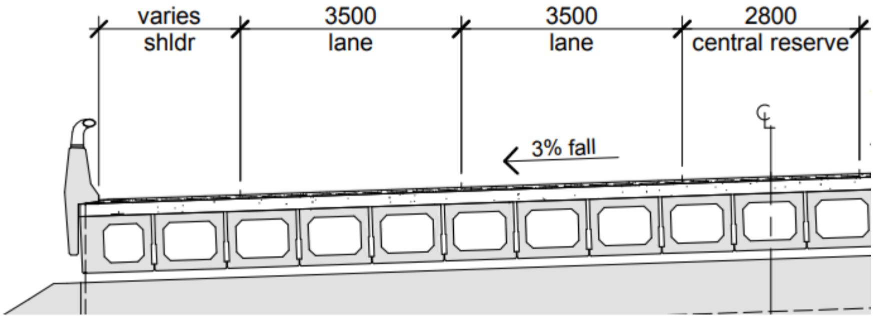
3.2 SUPER-STRUCTURE

Current bridge design tends to favour full concrete construction (over steel or timber) due to its long service life and low maintenance requirements. For bridges in 'easy' foundation conditions, such as in the Ashburton River, spans of 20-30m provide a reasonable optimisation between the cost of super-structure vs sub-structure.

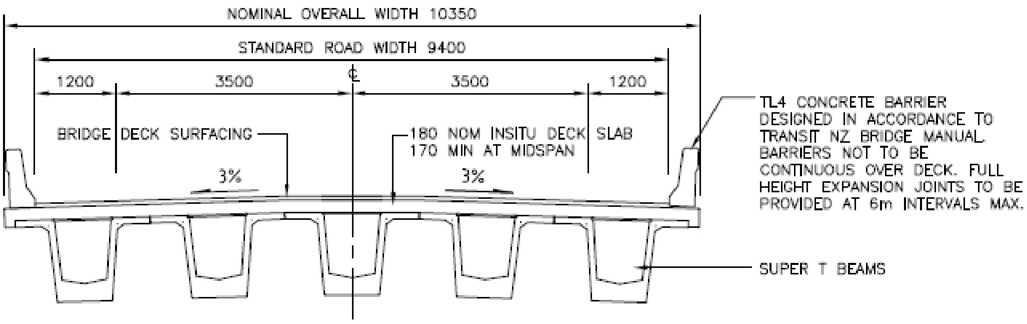
There are 3 commonly adopted options using standardly available precast units, shown in the figure below.



Double hollow core beams for spans from 10-18m



Single hollow core beams for spans from 16-25m



Super T beams for spans from 20-40m

Figure 3-1: Super-structure options

3.3 FLOOD PROTECTION

The flood event from May 2021 provided a good test of the flood containment measures on this part of the Ashburton River. The 2021 flood event peaked at 1794 cumecs, while the ECAN design flood for stopbanks is estimated to be 2238 cumecs (approximately 25% greater).

Our 1D hydraulic analysis indicated:

- that the ECAN stopbank design flood event has compliant freeboard on the Ashburton side (greater than 0.5m minimum), including allowance for any backwater effects from the new bridge
- that the ECAN stopbank design flood event does not have compliant freeboard on the Tinwald side (less than 0.5m minimum) in the current arrangement without the new bridges, and any backwater effects from the new bridge will further reduce the available freeboard

The Tinwald 'stopbank' is formed by a natural flood terrace that generally runs through private property. There is no simple means of adjusting the terrace height without removing boundary fences/hedges and moving a few storage sheds. Alternatively, a new stopbank alignment could be adopted, with construction of a formal stopbank closer to the main river channel, extending form the new proposed crossing location up to the State Highway bridge, where it can be more easily inspected and maintained.

4. PREFERRED OPTION

4.1 BRIDGE LENGTH

Both option 2 and 3 (Table 3-1: Bridge length options considered) provide realistic crossing alternatives, however, we have adopted option 2 for the purposes of this report as it provides a more conservative waterway (360m + 60m = 420m total) with scope to adjust the spit between the primary and secondary bridge.

Further work is recommended as part of detailed design to optimise the proposed bridge lengths, specifically:

- Refinement of the hydraulic assessment of the waterway to better understand the flow split between the main channel and the Tinwald flood plain
- Discussions with ECAN over other flood mitigation measures
- Discussions with ECAN and ADC regarding the risks of tree removal from the waterway

4.2 SUPER-STRUCTURE

All three options proposed (Figure 3-1: Super-structure options) provide viable structural options for a relatively straight forward crossing of this type.

Discussions with local contractors indicate that the full precast options (single and double HC units) are often preferred as they significantly reduce the need for insitu concrete on site, resulting in reduced construction time and less working at height issues. Hollow core units are also very efficient at sharing load to outer units, thereby reducing the peak structural demand with a single wheel track sitting on a single unit.

A recent example from the Christchurch Northern Corridor used 900 deep single hollow core beams with a span of 30m (usually limited to a 25m span). This was achieved by limiting the kerb-to-kerb carriageway to less than 9.7m, which only required the design to cater for 2 lanes of traffic, and then maximising load sharing to the outer units under the footpath. Our current proposed carriageway is 10.4m so this would require a reduction of just over 0.7m which may not be suitable to meet other design requirements.

The super-T beams are a deeper more efficient unit, more common in long span designs, but have the added complication of requiring construction of an insitu concrete deck over the precast units once they are in place. As the deepest of the 3 concrete options, they also require the approach road formation to be higher in order to achieve the same flood water level clearance from the beam soffit.

For the purposes of this report, we have adopted super-T construction, as this requires a more conservative (higher) road approach level and provides scope to move to a longer span option if desired. A 30m span was adopted as this sits in the usable range of both the super-T and the single hollow core options.

Further work is recommended as part of detailed design phase to optimize the proposed super-structure selection, specifically:

- Determine if the kerb-to-kerb carriageway can be reduced to 9.69m
- Determine if a single hollow core option is viable and if so to what maximum span
- Compare and optimise the foundation requirements between super-T and single hollow core (ie does the reduced weight of either option allow for a lighter less expensive sub-structure)

4.3 FLOOD PROTECTION

The current flood containment measures are assessed to be adequate on the Ashburton side but non-compliant on the Tinwald side. Further discussions are required with ECAN to review the results from our modelling and also determine what mitigation measures can be applied on the Tinwald side.

Further discussions with ECAN should include:

- Review of our hydraulic modelling and agreement to the acceptable length for both bridges (and total waterway provided) and determine what is an acceptable backwater effect for both the Ashburton and Tinwald stopbanks
- Acceptance/agreement that the existing Ashburton stopbank will be compliant with the proposed bridges in place and requires no adjustment
- Acceptance/agreement that the Tinwald terrace is currently non-compliant and will become further non-compliant with the proposed bridges in place
- Determine what mitigation measures can be applied to the Tinwald stopbank/terrace

4.4 ALTERNATIVE DESIGNS

The proposed bridge options are all relatively simple and modular in their construction. Other aspects of construction (e.g., site access, river diversion, foundation installation) should also be relatively straight forward. For bridges of this type, a small cost saving per span can multiply through to a significant cost reduction over the entre bridge length. It is therefore likely that some tenderers will offer alternative designs that achieve the same performance and service levels but at a more competitive rate.

To assist tenderers, and avoid unnecessary rework, the final design needs to be very clear in stating the design criteria and service limits that need to be met so that any alternative design is equivalent to the conforming design. This may include the following:

- Minimum waterway width
- Minimum waterway clearance (main channel)
- Foundation bearing limits and strength reduction factors
- Minimum foundation depth
- Minimum bed levels for seismic design checks
- Minimum bed levels for scour/raft design checks
- Minimum/maximum deck grades

4.5 COST ESTIMATE

As a basis for costing we have used the following:

SH74 Railway Overbridge

This is a multi-span dual-lane bridge constructed in 2005. The detailed rates from 2005 were adjusted by the NZTA Bridge Index but fell well short of current bridge prices (which appear to have been heavily affected by Covid and supply chain issues). A further 80% increase was needed in order to match current market rates.

Kuku East Overbridge

This is a single-span bridge constructed with 36m super-T beams and costed in May of 2022. The total cost was adjusted as follows:

- Significant ground improvements excluded
- Contractor’s margin included 11%
- On-site/off-site overheads included 13%
- Preliminary and general included 26%
- Adjusted for typical single span to multi-span savings 33%
- Indicative rate for Ashburton River Bridge is $4.31k/m^2

Cones Road Ashley River Bridge

This is a multi-span bridge constructed in the Waimakariri District in 2014. The bridge comprises 10 No spans of 30m, with a single hollow-core deck (12.7m wide) and twin caisson foundations at each pier – a very similar configuration to the bridge proposed for this project.

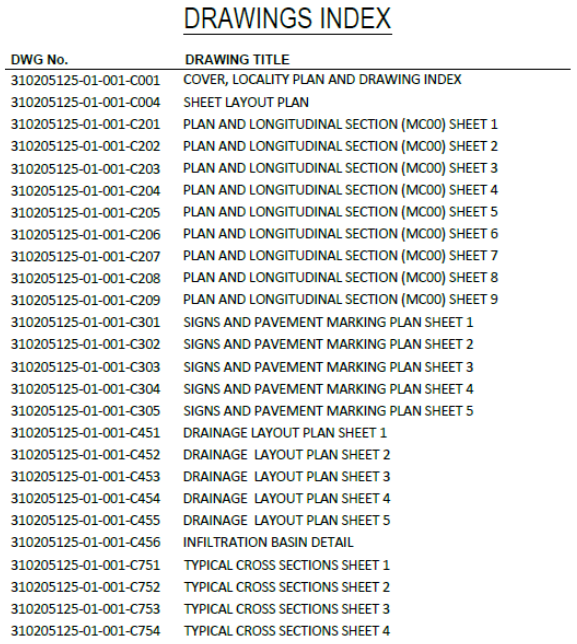
We contacted the contractor for this bridge (Concrete Structures, James Kelly) and requested an update of the Cones Road costs to meet the current market costs. They provided a bridge only estimate of $4.87 k/m^2.

For this bridge costing we have therefore adopted unit rate equivalent to $4.4-4.8 k/m^2. The detailed cost estimate is included with the DBC report.

5. DRAWINGS & DOCUMENTS

5.1 DRAWINGS

The following drawings are provided as part of detailed business case, and should be referred to as part of this preliminary structure options report:



DRAWINGS INDEX

DWG No.	DRAWING TITLE
310205125-01-001-C001	COVER, LOCALITY PLAN AND DRAWING INDEX
310205125-01-001-C004	SHEET LAYOUT PLAN
310205125-01-001-C201	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 1
310205125-01-001-C202	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 2
310205125-01-001-C203	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 3
310205125-01-001-C204	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 4
310205125-01-001-C205	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 5
310205125-01-001-C206	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 6
310205125-01-001-C207	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 7
310205125-01-001-C208	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 8
310205125-01-001-C209	PLAN AND LONGITUDINAL SECTION (MC00) SHEET 9
310205125-01-001-C301	SIGNS AND PAVEMENT MARKING PLAN SHEET 1
310205125-01-001-C302	SIGNS AND PAVEMENT MARKING PLAN SHEET 2
310205125-01-001-C303	SIGNS AND PAVEMENT MARKING PLAN SHEET 3
310205125-01-001-C304	SIGNS AND PAVEMENT MARKING PLAN SHEET 4
310205125-01-001-C305	SIGNS AND PAVEMENT MARKING PLAN SHEET 5
310205125-01-001-C451	DRAINAGE LAYOUT PLAN SHEET 1
310205125-01-001-C452	DRAINAGE LAYOUT PLAN SHEET 2
310205125-01-001-C453	DRAINAGE LAYOUT PLAN SHEET 3
310205125-01-001-C454	DRAINAGE LAYOUT PLAN SHEET 4
310205125-01-001-C455	DRAINAGE LAYOUT PLAN SHEET 5
310205125-01-001-C456	INFILTRATION BASIN DETAIL
310205125-01-001-C751	TYPICAL CROSS SECTIONS SHEET 1
310205125-01-001-C752	TYPICAL CROSS SECTIONS SHEET 2
310205125-01-001-C753	TYPICAL CROSS SECTIONS SHEET 3
310205125-01-001-C754	TYPICAL CROSS SECTIONS SHEET 4

DRAWINGS INDEX

DWG No.	DRAWING TITLE
310205125-01-001-C901	CROSS SECTIONS (MC00) SHEET 1
310205125-01-001-C902	CROSS SECTIONS (MC00) SHEET 2
310205125-01-001-C903	CROSS SECTIONS (MC00) SHEET 3
310205125-01-001-C904	CROSS SECTIONS (MC00) SHEET 4
310205125-01-001-C905	CROSS SECTIONS (MC00) SHEET 5
310205125-01-001-C906	CROSS SECTIONS (MC00) SHEET 6
310205125-01-001-C907	CROSS SECTIONS (MC00) SHEET 7
310205125-01-001-C908	CROSS SECTIONS (MC00) SHEET 8
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310205125-01-001-C921	CROSS SECTIONS (MC00) SHEET 21
310205125-01-001-C922	CROSS SECTIONS (MC00) SHEET 22
310205125-01-001-C923	CROSS SECTIONS (MC00) SHEET 23
310205125-01-001-C924	CROSS SECTIONS (MC00) SHEET 24
310205125-01-001-C925	CROSS SECTIONS (MC00) SHEET 25
310205125-01-001-C926	CROSS SECTIONS (MC00) SHEET 26
310205125-01-001-C927	CROSS SECTIONS (MC00) SHEET 27
310205125-01-001-C928	CROSS SECTIONS (MC00) SHEET 28
310205125-01-001-C929	CROSS SECTIONS (MC00) SHEET 29

5.2 DOCUMENTS

This report should be read in conjunction with the DBC. All other associated relevant documents are appended to this report.

Appendices

We design with community in mind

Appendix A Ashburton River Hydrology Review Memo

memo

To: Bryan Peters
From: Tom Kerr
CC: Ali Siddiqui
Date: 16/05/2022
Re: Ashburton-Tinwald Connectivity DBS – Ashburton at SH1 Hydrology Review
Ref: 310205125.100.0105

High Level Review of ECAN Design Flood Estimates

1:1500 and 1:2500 AEP floods

Recorded data at the Ashburton at SH1 flow gauge are from 1985 to present with a 7.8 year gap between September 1988 to June 1996. Historic floods from 1887 to 1978 are also available. The flow hydrograph is shown in Figure 1.

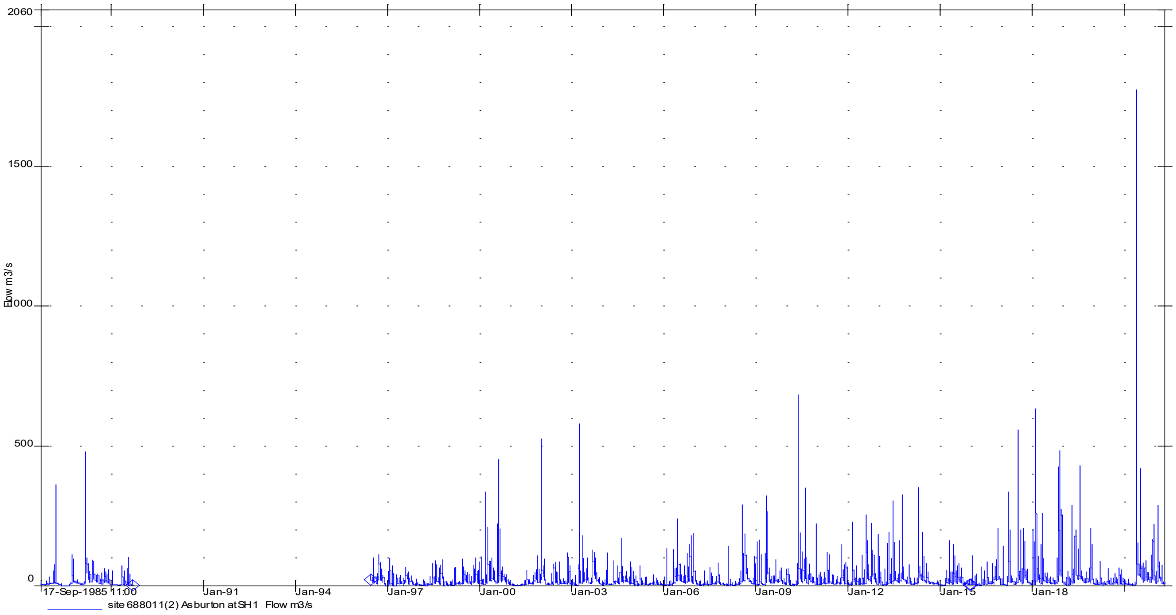

Figure 1: Ashburton at SH1 flow hydrograph

ECAN provided estimated design flood peaks for the Ashburton River at SH1 based on frequency analysis of recorded and historical data.

Preliminary ECAN results are shown in Table 1. It is understood the final version of the analysis will be reported by ECAN once a peer review of the work is completed.

Table 1: 2021 flood frequency estimate for Ashburton River at SH1

Distribution	Max flow (year)	Flow estimate (m³/s)							
		5 year	10 year	20 year	50 year	100 year	200 year	500 year	1000 year
TCEV (with historic events)	1794 (2021)	426	558	706	970	1,275	1,656	2,190	2,596
Rounded estimate		430	560	710	970	1,280	1,660	2,190	2,600

ECAN also provided estimates of 1:1500 and 1:2500 AEP flood peaks from extrapolation of the adopted TCEV flood frequency distribution and are listed in Table 2. Given the extent of extrapolation involved in estimating these extreme floods, ECAN advise caution in their use.

Table 2: 2021 flood frequency estimate for Ashburton River at SH1

Distribution	Max flow (year)	Flow estimate (m³/s)	
		1500 year	2500year
TCEV (with historic events)	1794 (2021)	2,800	3,100

The flood frequency analysis includes the May 2021 flood. The addition of this flood to the annual flood series increased estimated design flood peaks by 22% for the 1:100 AEP flood and 44% for the 1:1000 AEP event over previous estimates reported in 2017, (Tonkin and Taylor, 2017).

The Ashburton at SH1 flow record was provided by ECAN and Gumbel and GEV distributions fitted to the data. Neither distribution fit the annual flood series as well as the Two Component Extreme Value distribution provided by ECAN.

A marked tendency toward a two component distribution for south Canterbury East Coast rivers and a lesser tendency for North Canterbury East Coast rivers (including the Ashburton River) was reported in the paper *Two-component extreme value distribution applied to Canterbury annual maxima peaks,* (Connell & Pearson, 2001).

The TCEV was the preferred distribution for Ashburton at SH1 reported in *Flood frequency of Canterbury Rivers* (Tonkin and Taylor, 2017).

The use of the TCEV distribution for the Ashburton at SH1 annual series is therefore considered appropriate.

Ashburton at SH1 Flow Data Quality

The largest gaugings on record were measured during the May 2021 flood (1740 m^3/s and 1630 m^3/s). NIWA carried out surface velocity measurements with a handheld radar taken at over 30 locations across the length of the bridge. Estimated uncertainty is +- 200 m^3/s, (NIWA Client Report for ECAN, 2021). A slope area calculation was also undertaken following the flood which gave results consistent with the surface velocity measurement. These gaugings provide greater certainty related to the upper part of the stage - flow rating and the estimate of large floods at the site.

About 756 other gauging have been carried out at the Ashburton at SH 1 site with next highest measurement recorded as 516 m^3/s.

The reach at the flow gauge is a gravel bed channel and there are numerous rating changes as shown in Figure 2.

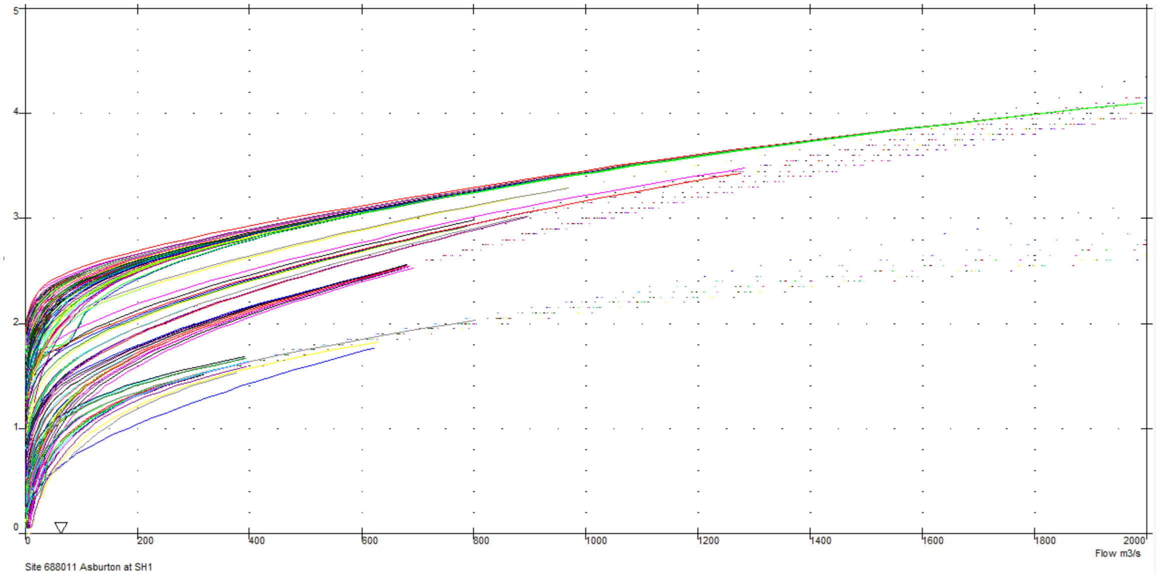

Figure 2: Ashburton at SH1 Ratings

Although the channel bed elevation at the gauge is not stable, there has been considerable effort put into maintaining stage–flow ratings. Uncertainty associated with flows recorded during the period of record is therefore likely to be relatively low.

The accuracy of historic flood peaks used in the frequency analysis is less certain. It isn't clear to what extent the flood peak at the SH1 bridge is affected by the development of upstream stopbanks over time. Nevertheless, the various distributions tested, including the TCEV distribution, fit historic flood peaks well. Results presented in (Tonkin and Taylor, 2017) include analysis with and without historic data. The difference between the two datasets is not great.

Comparison with Other Methods for Estimation of 1:1500 & 1:2500 AEP Floods

An alternative method of estimating extreme flood peaks is to interpolate between 1:50 and 1:100 AEP floods and the PMF.

The recommended approach to estimate the PMF is to apply the PMP to a calibrated rainfall runoff model of the catchment.

In the absence of a rainfall runoff model an estimate of the PMF for Ashburton at SH1 was transposed from a PMF estimate for Waimakariri at OHB reported in the Journal of Hydrology NZ, (Griffiths, Pearson, & Horell, 1989). The Waimakariri PMF of 8,000 m^3/s was scaled according to the difference in area as shown below.

$$Ashburton\ at\ SH1\ \ Q = Waimakariri\ at\ OHB\ Q\ x\left(\frac{1579}{3210}\right)^{0.8}$$

The calculation gives a PMF for Ashburton at SH1 of 4,535 m^3/s. The estimate is considered conservative as the headwaters of the Waimakariri catchment are at higher elevations than the Ashburton catchment. Based on HIRDS data, the Waimakariri receives 16% more rainfall than the Ashburton catchment during a 12 hour 1:100 AEP storm. See Figure 3 for map of catchment boundaries and a comparison of rainfall.

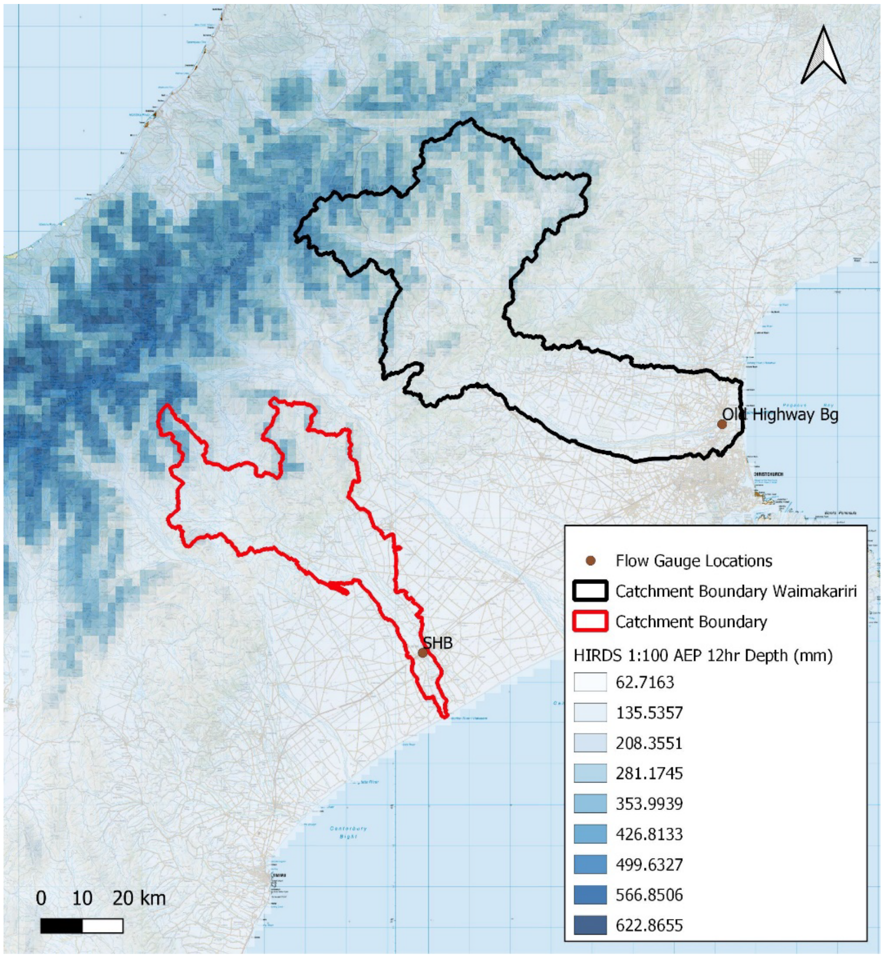

Figure 3: Ashburton and Waimakariri catchment boundaries and HIRDS rainfall

Nathan and Weinmann in Australian Rainfall and Runoff (ARR), (Ball, et al., 2019.) considered the AEP of the PMP to be solely a function of catchment area with small areas having larger AEP's. Chapter 3, Figure 8.3.2 gives an AEP for the 1579 km^2 Ashburton at SH1 catchment of 1:1,000,000 plus or minus two orders of magnitude.

Toward prediction of extreme rainfalls in NZ (Griffiths, McKerchar, & Pearson, 2014) estimates the AEP for a Christchurch PMP to be 1:10,000.

Figure 1 shows 1:50 AEP, 1:100 AEP flood values and the estimated PMF for Ashburton at SH1 with 1:10^4, 1:10^5 and 1:10^6 AEP's plotted. Also shown are TCEV values from Tables 1 and 2.

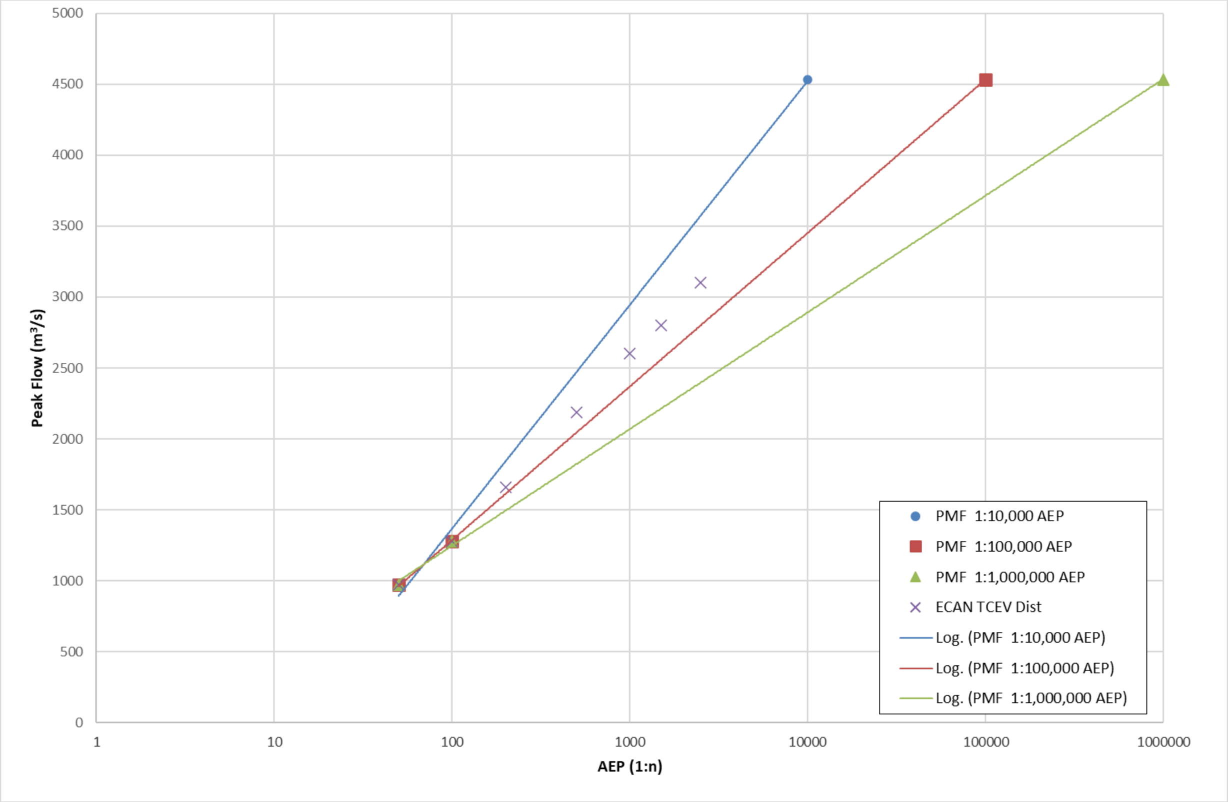

Figure 4: Interpolation Between 1:50 and 1:100 AEP and the PMF

The TCEV distribution values lie between the interpolated lines for 1:10,000 and 1:100,000 AEP PMF's which is considered at the more conservative end of the range.

ARR (Pilgrim, 1998) provide methods to interpolate between the 1:50 and 1:100 AEP event and the PMP and methods to estimate the AEP of the PMP. Flood estimates from Table 1 for the 1:50 and 1:100 AEP floods were used with a PMF of of 4535 m^3/s. This gave 1:1500 AEP and 1:2500 design flood estimates of 2800 m^3/s and 3000 m^3/s respectively based on a recommended AEP of the PMF of 10^6. Results are shown in Figure 5 and compare closely with TCEV distribution results.

The method assumes that interpolation of flood flows will be similar to that for rainfall.

Conclusion

The TCEV distribution results are considered appropriate and it is recommended that the 1:1500 and 1:2500 AEP values of 2,800 m3/s and 3100 m3/s be adopted.

As mentioned above, uncertainty associated with estimating extreme flood peaks is large and so an appropriate range of design flows should be considered during the design process.

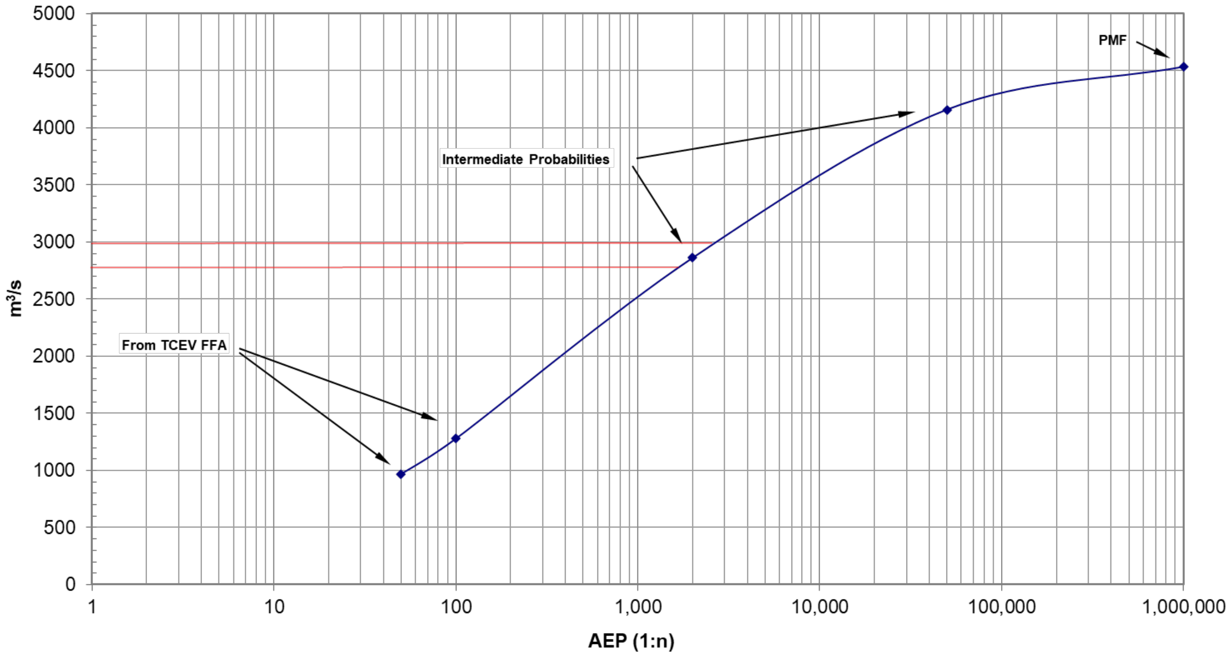

Figure 5: Interpolation between 1:50 and 1:100 AEP floods and the PMF

References

Connell, R., & Pearson, C. (2001). Two-component extreme value distribution applied to Canterbury annual maxima peaks. *Jounal of Hydrology NZ, 40(2)*, 105-127.

Griffiths, G., McKerchar, A., & Pearson, C. (2014). Towards prediction of extreme rainfalls in New Zealand. *Journal of Hydrology (NZ) 53 (1): 41-52*, 41 - 52.

Griffiths, G., Pearson, C., & Horell, G. (1989). Rainfall - Runoff Routing in the Waimakariri Basib, NZ. *Jounnal of Hydrology NZ, Vol 28 No. 2.*, 111- 122.

Tonkin and Taylor. (2017). *Flood frequency analysis for Canterbury Rivers.*

Appendix B Ashburton River 2D Model Memo

PLACEHOLDER

Appendix C ATC – Ashburton 2nd Bridge: Geotechnical Report

Stantec

Ashburton-Tinwald Connectivity - Ashburton 2nd Bridge: Geotechnical Report

This document was prepared by Stantec New Zealand ("Stantec") for the account of Ashburton District Council (the "Client"). The conclusions in the Report titled Geotechnical Report are Stantec's professional opinion, as of the time of the Report, and concerning the scope described in the Report. The opinions in the document are based on conditions and information existing at the time the document was published and do not consider any subsequent changes. The Report relates solely to the specific project for which Stantec was retained and the stated purpose for which the Report was prepared. The Report is not to be used or relied on for any variation or extension of the project, or for any other project or purpose, and any unauthorized use or reliance is at the recipient's own risk.

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Quality statement

Rev. no	Date	Description	Prepared by	Checked by	Reviewed by	Approved by
0	June 2022	For Comment	Alex Park	Steven Woods	Bryan Peters	Ali Siddique
1	July 2022	Final	Alex Park	Steven Woods	Bryan Peters	Ali Siddique

1 Scope of Work

This technical memo sets out the following geotechnical assessments for the proposed second Ashburton River crossing which consists of new road sections, both at grade and on fill embankments, a long section of bridge crossing the main river channel and a short section of bridge crossing a secondary flow channel. In particular it provides:

- Preliminary Liquefaction Assessment
- Preliminary Bridge Foundation Assessment
- Preliminary Embankment Stability Assessment
- Preliminary Road Design

2 Site Description

The proposed road and bridge alignment are located approximately 700 m to the south-east of the existing State Highway 1 river crossing. The south-west section extends approximately 1400 m through flat lifestyle block type properties from Grahams Road to Carters Terrace. The main river channel is approximately 200 m wide and marked by meandering flow banks, sediment banks and an extensive flood plain on the Tinwald side. On the eastern side of the river, the proposed road alignment will cross Maniaroto Park and connect into Chalmers Avenue approximately 250 m from the bank.

3 Geotechnical Investigations

The following geotechnical investigations have been undertaken at the immediate locations of the proposed infrastructure (bridge and road alignment) and are considered relevant for the design of the proposed infrastructure.

- Two (2) Machine boreholes (BH01 and 04) with Standard Penetration Testing (SPT) at 1.5 m vertical increments down to approximately 20.0 m below ground level (bgl)
- Seven (7) testpits on either side of riverbank along the proposed road alignment to the south-west.
- Two (2) transient falling head test at Test Pit (TP) 01 and TP 04

Figure 3-1 presents the investigation locations in relation to the proposed infrastructure. The Geotechnical factual report (Stantec , 2022) is prepared and attached in Appendix A.

Figure 3-1 Geotechnical investigation locations

4 Ground Model

All geotechnical investigations presented in Section 3 indicate good agreement in ground conditions at various locations across the site. A generalised profile has been adopted for the design of the bridges (based on the boreholes only) and is presented in Table 4-1.

Table 4-1 Typical ground model

Geological Unit	Description	Depth to base of layer (m)	Uncorrected SPT blow count, *N*	Note
1	SILT with some clay, trace sand and gravel; brown. Firm, moist, low plasticity; sand, fine to coarse; gravel, fine, sub-angular to sub-rounded.	0.9 (BH01) – 1.4 (BH04)	-	BH01 and 04
2-A	Sandy fine to coarse GRAVEL with trace silt; grey, brown. Medium dense to dense, wet to saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse.	2.8 (BH01) – 4.5 (BH04)	14 to 26	BH01 and 04
2-B	Silty fine to medium SAND with trace gravel; grey, brown. Dense, wet to saturated, poorly graded; gravel, fine, sub-angular to subrounded.	3.5 (additional layer found in BH01 only)	11	BH01 only
3	Sandy fine to coarse GRAVEL with some silt and trace cobbles; brown. Dense to very dense, saturated, well graded; gravel, subrounded; sand, fine to coarse; cobbles, up to 150mm.	3.5/4.5 – targeted depth (20)	50+	BH01 and 04

5 Groundwater

Groundwater was encountered typically at approximately 2.0 m depth and is inferred to be hydraulically connected to the adjacent river. Anecdotally springs have been identified by local landowners along the alignment of the bridge approaches and new road sections, which is inferred to be due to hydraulic connection through permeable lenses in the gravel unit to the river further up gradient.

To be able to estimate soil permeability, two transient falling head tests were undertaken at the locations of TP01 and TP04. The recorded data was plotted and is shown in Figure 5-1.

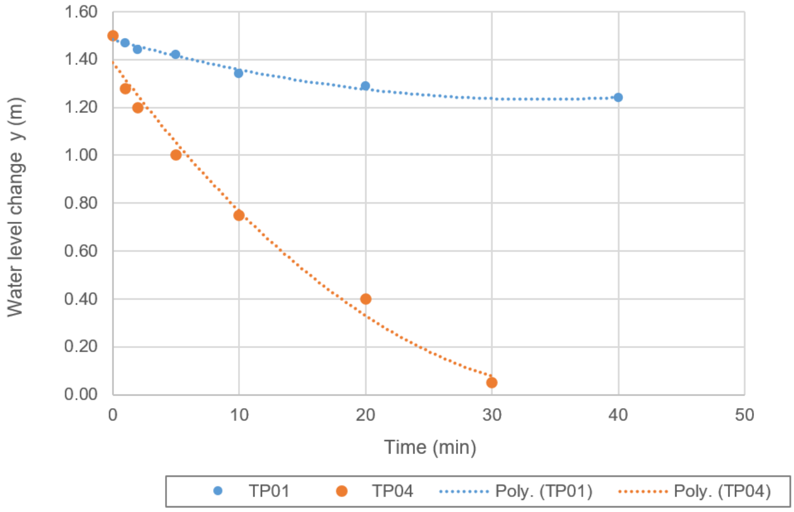


Figure 5-1 Hydraulic conductivity (falling head test results)

Using a published correlation (Stibinger, 2014), hydraulic conductivity of each test pit was calculated and shown in Table 5-1.

Table 5-1 Estimated hydraulic conductivity for each test pit

Test Pit	Hydraulic Conductivity (m/s)	Soil log field description
TP01	2.3E-6	Silty fine to coarse GRAVEL
TP04	3.2E-4	Sandy fine to coarse GRAVEL

6 Material Properties

Preliminary material properties for the soil strata described in Section 4 are presented in Table 6-1. Material properties have been selected from various published correlations between shear wave velocity and SPT blow counts, (WairB, DeJongJ, ShantzT, 2012), (Hatanka & Uchida, 1996), (Idriss & Boulanger, 2008), and (Olson & Stark, 2002), and our experience with similar soils and conditions.

Table 6-1 Summary of characteristic geotechnical material properties

Unit	Bulk Unit Weight, γ (kN/m^3)	Angle of Internal Friction, φ' (degrees)	Effective Cohesion, c'(kPa)	Liquefied strength ratio, $s_{u(LIQ)}/\sigma'_{v0}$
1	19	28	2	-
2-A	19	33	0	0.39 – 0.45
2-B	18	32	0	0.16 – 0.22
3	20	35	0	-

7 Seismic Ground Motion Parameters

The proposed infrastructure has been assigned Importance Level (IL) 4. The site subsoil class has been assessed by the definitions set out in NZS 1170.5 Structural Design Actions, Earthquake Actions as Class D – deep or soft soil.

The NZTA Bridge Manual (Waka Kotahi, 2018) requires the consideration of the following seismic load cases for an IL 4 bridge.

- Damage Control Limit State (DCLS): 1/2500-year earthquake
- Serviceability Limit State (SLS): Return period factor of DCLS/4:
- Collapse Avoidance Limit State (CALS): Return period factor of 1.5 x DCLS

Based on the methodology in Section 6.2.2 of the Bridge Manual, the design acceleration and magnitudes for earthquake loads are summarised in Table 7-1.

Table 7-1 Peak Ground Acceleration (PGA) derivation

Parameters	Description		
State	SLS	DCLS	CALS
Importance Level	4		
Annual Probability of Exceedance (APE) (1)	-	1/2500	-
Site Subsoil Class (2)	Class D – deep or soft soil		
Site Subsoil Class Factor, f (3)	1.0		
Return Period Factor (R_u) (4)	0.45	1.8	2.7
1000-year Return Period PGA Coefficient, $C_{0,1000}$ (5)	0.29 (Ashburton)		
Effective Magnitude (M_{eff}) (6)	6.1 (500 – 2500 yr)		
Unweighted Peak Horizontal Ground Acceleration (g) (7)	0.1	0.4	0.6

(1) Bridge Manual Table 2.1 DCLS Earthquake Action.

(2) NZS 1170.5:2004.

(3) Bridge Manual Section 6.2.2

(4) NZS 1170.5:2004 Table 3.5 and Bridge Manual Table 5.1 (DCLS / 4 and 1.5 · DCLS for SLS and CALS, respectively)

(5) Bridge Manual Section 6.2.2 Figure 6.1 (a).

(6) Bridge Manual Commentary Table C6.1 Ashburton.

(7) Bridge Manual Section 6.2.

8 Preliminary Liquefaction Assessment

A simplified liquefaction assessment has been undertaken using SPT data from both borehole investigations and the method set out by (Boulanger & Idriss, 2014), and (Zhang, Robertson, & Brachman, 2002). Ground water level has been set at 2.0 m for the assessment, and outputs from the assessment are included in Appendix B.

The assessment concludes that liquefaction triggering is not anticipated under SLS seismic load conditions (SLS not shown in graph for clarity), is marginal under the DCLS loading and is anticipated under CALS seismic load condition, as shown in Figure 8-1 in isolated shallow strata. Soil strata lower than 4.0 m below ground level are not susceptible to liquefaction during any design earthquake events.

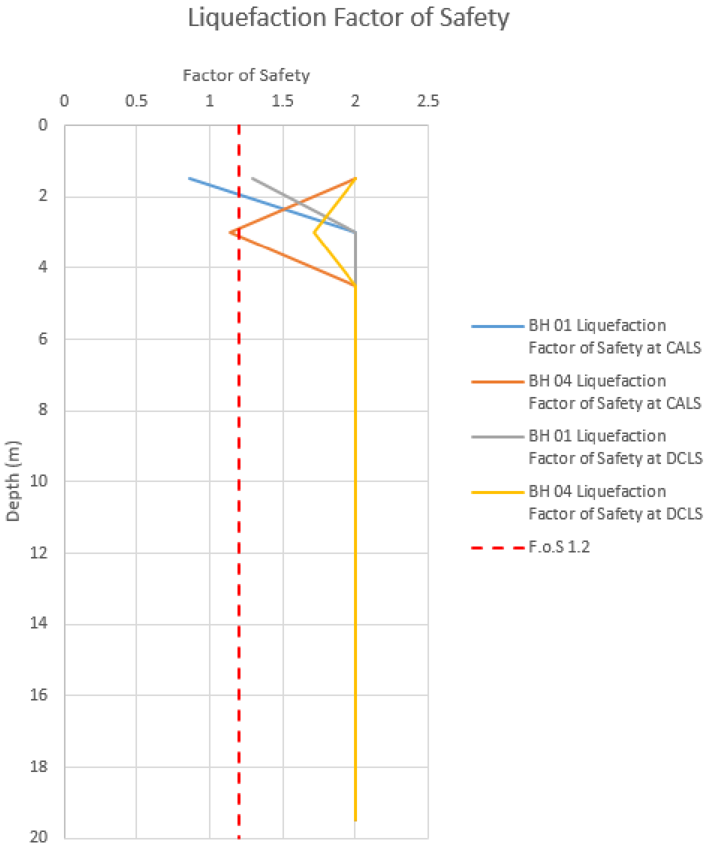


Figure 8-1 Liquefaction factor of safety for CALS and DCLS load combination cases

We note that:

- Due to uncertainty/discrepancy of data below 15.0 m bgl, liquefaction assessments typically limit the reporting to 15.0 m bgl. It is considered rare for liquefaction to occur below 15.0 m bgl due to general inhomogeneous soil profile.
- The simplified methods used here exclude any ground deformation induced by liquefaction manifestation (soil ejecta and ground distortion on surface).
- This assessment is a free-field assessment, implying that no loads from the proposed infrastructure are considered during assessment.

The Bridge Manual Section 6.3.3 stipulates that conducting shear wave velocity tests may be able to refine the liquefaction assessment further in the detailed design. To be able to estimate liquefaction triggering assessment within the river where access for drilling is difficult, we suggest conducting the shear wave velocity tests to build a ground profile along the proposed bridge alignment and to refine the liquefaction assessment prior to the commencement of the detailed design.

9 Preliminary Embankment Design

9.1 Geometry and Earthworks Considerations

The proposed design of the pavement indicated that a fill embankment is required to construct the bridge approach and abutment. A preliminary longitudinal section is shown in Figure 9.1-1 for the northern abutment of the main bridge. In final design the abutment is likely to be moved closer to the stopbank and may in fact sit inside the stopbank to make best use of the water way.

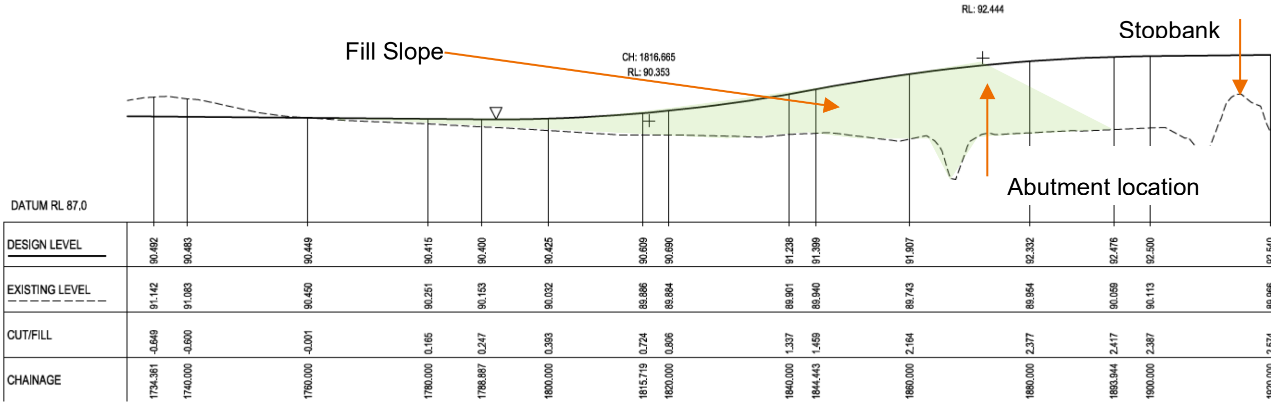


Figure 9.1-1 Preliminary longitudinal section showing at northern abutment (Ashburton)

We propose of applying a slope ratio of 3 Horizontal (H) : 1 Vertical (V) to form a bridge approach. Generally, 3H:1V slope achieves a stable slope in a long-term static condition and is acceptable for maintenance. We note that this is a generic slope and can be refined at the detailed design stage. Often the river side slope of spill through abutments can be as steep as 1:5H:1V.

For the simplicity of the assessment, we considered a set of stability checks along the bridge longitudinal orientation as this is typically the worst case. Stability assessments of the bridge transverse orientation can be checked at the detailed design stage and are considered unlikely to be critical. Only the northern approach to the main bridge has been considered, with similar conclusions expected at the other approaches and connecting embankments.

The embankment will be constructed across two broad ground conditions, namely:

- Across existing farmland.
- Across forested sections of the Ashburton River bed.

The first condition has been investigated by a series of test pits along the alignment, which identified typically 300mm topsoil overlying a silt layer up to approximately 2m thick. In these ground conditions preparation for filling is envisaged to consist of stripping topsoil to expose the underlying silt. Some settlement in the silt can be expected under embankment fill loads, however, given the silt is non saturated such settlement is expected to occur soon after construction such that it can be accommodated within the construction timeframe of the project.

The second condition occurs between the two bridge spans. Due to access constraints no investigations have been undertaken in this area, however, previous experience in similar river environments indicates that up to approximately 1m of sandy silt (deposited during river floods) can be expected overlying sandy gravel. Much of the overlying silt will be disturbed when trees and their roots are removed to facilitate construction and in effect this layer will need to be removed prior to embankment construction. Based on previous experience an average stripping depth of 750mm would be a reasonable allowance for preliminary design.

None of the stripped material from either section is anticipated to be suitable for use as embankment fill, although stripped topsoil could be re-used on external embankment faces assuming there are no contamination issues. It is envisaged that embankment fill would be sandy gravel sourced from nearby quarries that would also supply pavement layers.

Anecdotal evidence and observations indicate that springs could be encountered along the alignment. This is inferred to be due to permeable lenses in the gravel layer being hydraulically connected to the river up gradient. In practical terms addressing such springs will consists of localised excavation and replacement in any areas that the stripped surface is noticeably wet (and therefore soft) and the provision of subsoil drainage in the pavement layers discharging to the roadside drainage system.

9.2 Embankment Stability

Stability of the proposed embankment profiles have been assessed for four design cases:

- Static Case: Long-term static condition
- Seismic Case 1: Static soil conditions with DCLS and CALS events, representative of conditions early in an earthquake (pseudo static assessment)
- Seismic Case 2: Liquefied soil condition (shown in Table 6-1) with zero ground motions, representative of conditions at the end of an earthquake (flow failure)
- Seismic Case 3: Liquefied condition with horizontal earthquake load to obtain a yield acceleration, representative of conditions at starting of failure. These values were used to estimate lateral movement.

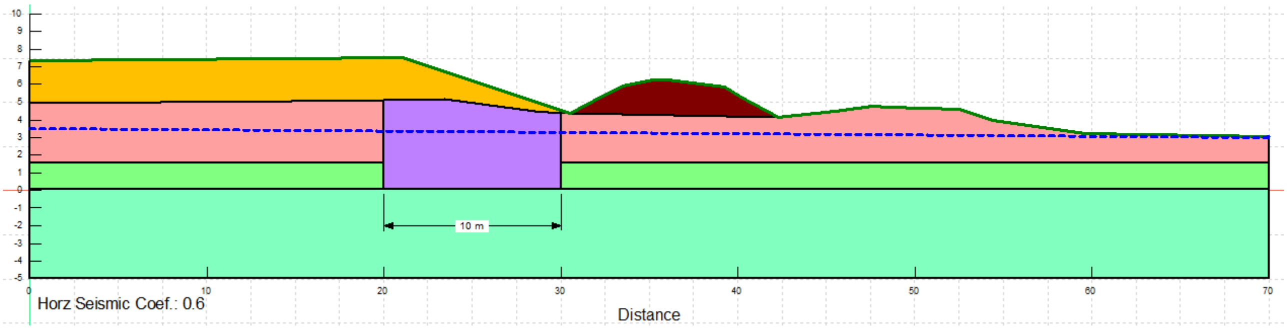
Stantec conducted stability modelling utilizing the commercially available SLOPE/W software package and a Morgenstern-Price/specified block type analyses. Table 9.2-1 presents geotechnical factors of safety and embankment deformation assessed for the design cases. Graphical outputs from the analysis package, showing the critical failure plane, are included in Appendix C.

Table 9.2-1 Embankment stability for each design case

Design Case		Factor of Safety	Acceleration (g)
Static Case		2.2	-
Seismic Case 1	DCLS	0.9	0.40
	CALS	0.6	0.60
Seismic Case 2 – liquefied soil condition		2.2	-
Seismic Case 3 – yield acceleration		1.0	0.21

Seismic case 1 – pseudo-static condition

For seismic case 1 (both DCLS and CALS events), the model predicted factor of safety less than 1 and therefore based on the requirements of the Bridge Manual (Waka Kotahi, 2018) further assessment of the potential for lateral spreading is required. Based on the value of yield acceleration and anticipated movement it is unlikely that the Bridge Manual Criteria would be met without some mitigation. From the bridge abutment toward land (where piled foundation is proposed), replacement of the compacted gravel fill to the bottom of liquefied layer (typically 5.0 m below ground level) was modelled and shown in Figure 9.2-1, with distances of 10 and 20m trialled.



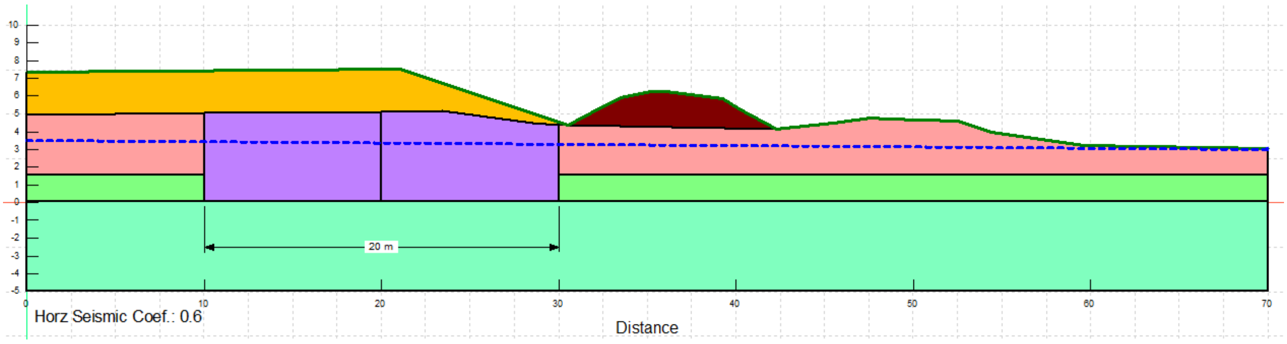


Figure 9.2-1 Mitigation models geometry

<u>Seismic Case 2 – liquefied soil condition</u>

For seismic case 2, this assessment represents the conditions at the end of an earthquake event, meaning no ground acceleration applied under liquefied soil condition. This assessment utilizes a reduction in the strength properties of the soil, typically SAND/Silty SAND/sandy SILT, due to liquefaction and has been quantified based on the methodology of Olson and Stark (Olson & Stark, 2002), which are introduced in Section 6. The model predicted the proposed embankment is stable (i.e, FOS>1) at the end of design seismic event (zero acceleration with reduced shear strength applied). This preliminary model is not able to capture the degree of deformation, however based on the seismic case 2 assessment, no major failure is anticipated at the end of the design seismic events.

<u>Seismic Case 3 – yield acceleration</u>

For seismic case 3, this assessment is to identify a triggering acceleration (yield) while shear strength is reduced by liquefaction. This assessment enables predicting lateral ground movement using the published correlations.

To be able to increase the resilience of the proposed embankment, replacing the liquefiable layer with the compacted gravel fill is implemented and the prediction is shown in Table 9.2-2.

Table 9.2-2 Embankment stability and yield acceleration (g) with mitigation options

Design Case		Factor of Safety	Yield Acceleration (g)
Seismic Case 1 – 10 m gravel fill replacement	DCLS	0.9	0.30
	CALS	0.6	
Seismic Case 1 – 20 m gravel fill replacement	DCLS	0.9	0.33
	CALS	0.6	

Table 9.2-2 shows that the treatments increase the yield acceleration and therefore reduce the estimated lateral displacement. With respect to this model, we note:

- Adopting a ground model with continuous and homogeneous liquefiable soil layers is very conservative and is unlikely in reality. Planned shear wave testing during detailed design will provide better information on the continuity of potentially liquefiable soils.
- The assumption of a continuous liquefiable layer will result in predictions of seismic ground oscillation that, based on further testing discussed above, are unlikely to be realistic.
- The resulting deformation assessment assumes that the soil is at residual liquefied strength throughout the earthquake event, which is a very conservative assumption.

The ground deformation limit for soil structure supporting or containing bridge abutments under the DCLS load combination is 25 mm (hazard factor, *z* < 0.3) per The Bridge Manual Table 6.1 (Waka Kotahi, 2018). Using Newmark's Sliding Block Analysis Methods 50th percentile displacement (Figure 9.2-2) shows the effect of applying the compacted gravel fill responses to the seismic induced lateral ground deformation.

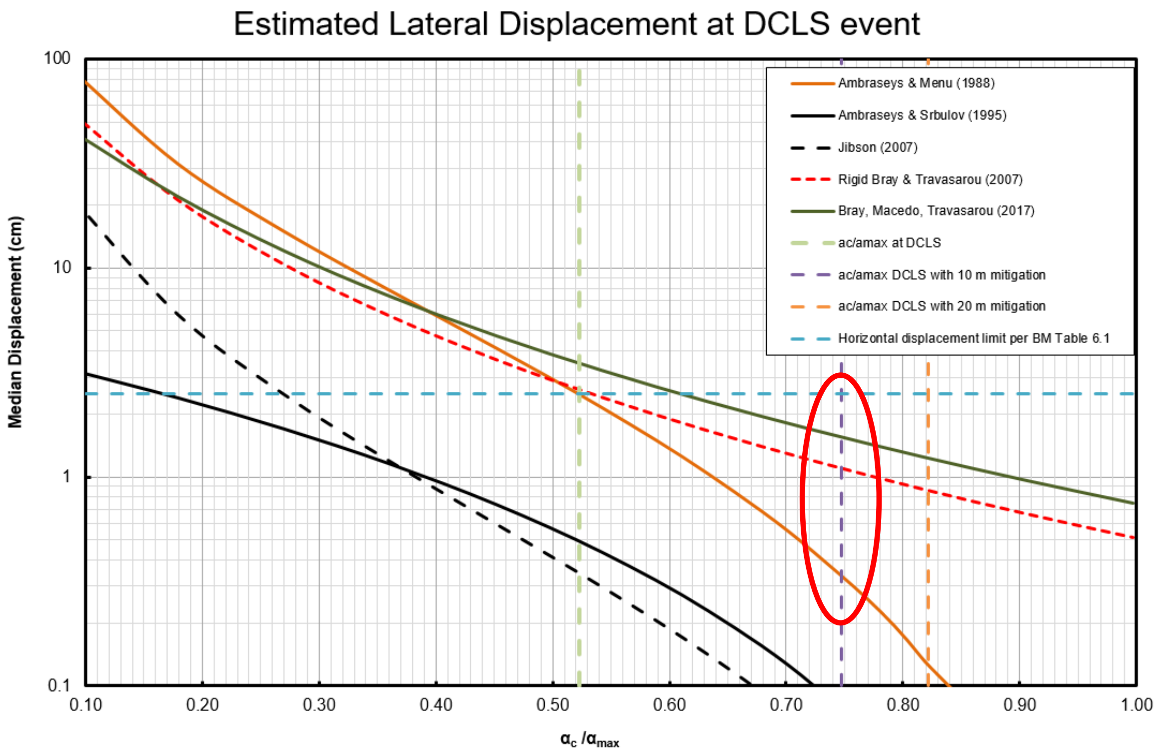


Figure 9.2-2 Prediction of lateral ground movement under DCLS with 10 and 20 m compacted gravel fill replacement.

Less than 25 mm of lateral ground movement is expected with 10 m or 20 m compacted gravel fill replacement shown in red circle in Figure 9.2-2, meeting the design requirement. As a conservative basis for the Preliminary Design a treatment zone of 20m has been applied to each of the four bridge abutments. This may require rebuilding of a short section of the existing stopbank.

10 Preliminary Pile Foundation Capacity

The preliminary bridge design intends to transmit the induced load from the bridge to lower levels in the soil mass via a piled or caisson style foundation structure. This piled foundation is considered a separated system to support the abutments independently. For the piers a preliminary caisson we have adopted a 1.5 m diameter cylinder (comprising reinforced concrete and steel liner), driven to 15 m below ground level. Two piles are intended per pier. A precast concrete plug will be driven at the end of each pile to effectively create a driven pile (with higher capacity). Pile capacity will be verified by construction stage drilling at each pier and based on energy formulae (such as the Hiley formula) and the observed plug set during driving.

Based on the piled foundation geometry and interpretation of borehole information, a preliminary pile vertical capacity assessment was conducted per the Bridge Manual Table 3.2 the load combinations and factors (Waka Kotahi, 2018) and Acceptable Solutions and Verification Methods (B1/VM4) (Ministry of Business, Innovation & Employment, 2019). A geotechnical strength reduction factor using the risk-based methodology for piled foundation was calculated per AS2159:2009 Section 4.3 (Australian Standard, 2009). The calculation is attached in Appendix D and it can be seen that the strength factor reduced pile capacity exceeds the load factor increased load per pile.

Table 10-1 Preliminary pile foundation assessment result

Factored total vertical load per pile (kN)	Factored vertical capacity per pile (kN)	
	Factored base resistance	Factored shaft resistance
11600	10100	1600
* Demand load combination 1A (Waka Kotahi, 2018) : 1.35 x Dead Load + 2.25 x Traffic Load * Strength reduction factor (Australian Standard, 2009) : 0.52		

We note that the clearance between the abutment and the end of the end span main girders in the longitudinal orientation shall be sufficient to accommodate (Waka Kotahi, 2018):

- 200% of the DCLS lateral ground movement estimation (section 9.2 in this report)
- 200% of the DCLS seismic movement of the superstructure (structure team to confirm in detailed design)
- 33% of the thermal movement (structure team to confirm in detailed design)

Where the bridge is supported by piles at the abutment, the piles shall be also protected from the displacement of the embankment by providing an adequate space to accommodate either:

- The CALS seismic lateral displacement and 200% of the DCLS lateral ground movement estimation
- or bridge shall be designed to withstand the forced induced by a CALS event and by two sequential DCLS design events.

The allowable displacement at the abutment and piled/caisson foundations will be further refined in the detailed design, considering bullet points listed above and separation achieved using a casing of larger diameter than the piles within the abutments.

Detailed assessment of lateral pile displacements has not been undertaken at this design stage as they are considered unlikely to be critical given the available depth of embedment. The critical case for lateral pile capacity is likely to be following river scour rather than seismic liquefaction, however, both cases can be checked during detailed design using the general methodology set out in NZTA Research Report 553.

11 Preliminary Road Design

The roads that connect the elevated embankment and bridge sections to the existing road network will be constructed across existing farmland. The ground conditions have been investigated by a series of test pits along the alignment, which identified typically 300mm topsoil overlying a silt layer up to approximately 2.0 m thick. The road formation is approximately at existing grade and in these ground conditions preparation for pavement is envisaged to consist of minimum stripping topsoil to expose the underlying silt. A subgrade CBR of 5 is an appropriate allowance for preliminary design of the pavement.

None of the stripped material from is anticipated to be suitable for use as embankment fill, although the stripped topsoil could be re-used on external embankment faces subject to any contamination issues.

Anecdotal evidence and observations indicate that springs could be encountered along the alignment. This is inferred to be due to permeable lenses in the gravel layer being hydraulically connected to the river up gradient. In practical terms addressing such springs will consists of localised excavation and replacement in any areas that the stripped surface is noticeably wet (and therefore soft) and the provision of subsoil drainage in the pavement layers discharging to the roadside drainage system to maintain control of moisture within the pavement.

12 Geotechnical Risks and Construction Considerations

This section presents the identification of construction and geotechnical risks considered relevant for the proposed works.

Table 12-1 Risks identified and their potential mitigations

Risk Identified	Proposed Potential Mitigation
The required excavations for gravel replacement may extend up to 4.0 to 5.0 m bgl. We expect that excavation into this material is likely to result in unstable open trenches at depths greater than 1 m bgl.	A form of shoring or battered slope will be required to support the excavations. Alternatively, sheet piles or trench boxes may also be suitable options. The Contractor will be required to manage and provide the stability of the trench during construction.
The groundwater information discussed in this report indicates the static groundwater level is likely to be at 1.5 to 2.5 m bgl, i.e., the excavation will encounter groundwater. Shallow groundwater levels combined with the expected sandy-gravelly granular ground materials can result in significant groundwater flows into trenches, through the trench walls and/or base of pits.	Higher pumping rates are likely to be required to establish a long-term drawdown modelled in this estimate. Installation of groundwater cut-off such as sheet piles may be required. The design of the groundwater cut-off will be provided by the Contractor, if required. Dewatering of excavations and discharge of dewatering will need managing by the Contractor.
No direct ground information on the in-river sections of the proposed bridge alignment.	We consider that the collated information is sufficient to inform the ground conditions to a sufficient level for preliminary design. To be able to estimate liquefaction triggering assessment within the river, we suggest conducting the shear wave velocity tests to build ground profile along the proposed bridge alignment and to refine the liquefaction assessment prior to the commencement of the detailed design.
Underground utility lines may be presented and may clash with the proposed design and/or construction.	The depths and locations of buried utilities should be confirmed before commencing any works.
Contaminated ground may be encountered impacting on the potential re-use of excavated ground.	A separate contamination assessment is being undertaken.
Groundwater encountered in localised springs along the scheme alignment.	Localised treatment and drainage as required.

13 Conclusion

Geotechnical investigations were conducted to understand the site-specific condition, to develop a ground profile, and to derive the preliminary design geotechnical input parameters, using the published correlations and our experiences from similar ground condition.

Based on the interpretation of the geotechnical investigation information, we conducted various geotechnical assessments, including:

- Preliminary liquefaction assessment
- Preliminary embankment assessment
- Preliminary pile foundation assessment
- Preliminary road assessment

The liquefaction assessment concludes that liquefaction triggering is not anticipated under SLS seismic load conditions but is marginal under DCLS seismic load and anticipated under CALS seismic load condition. Soil strata lower than 4.0 – 5.0 m below ground level are not susceptible to liquefaction during any design earthquake events.

The embankment assessment concludes that the proposed slope ratio of 3 Horizontal (H) : 1 Vertical (V) to form a bridge approach achieves a stable slope in a long-term static condition. For seismic assessment, we investigated three different phases; pseudo-static condition; flow failure condition; ground movement triggering condition.

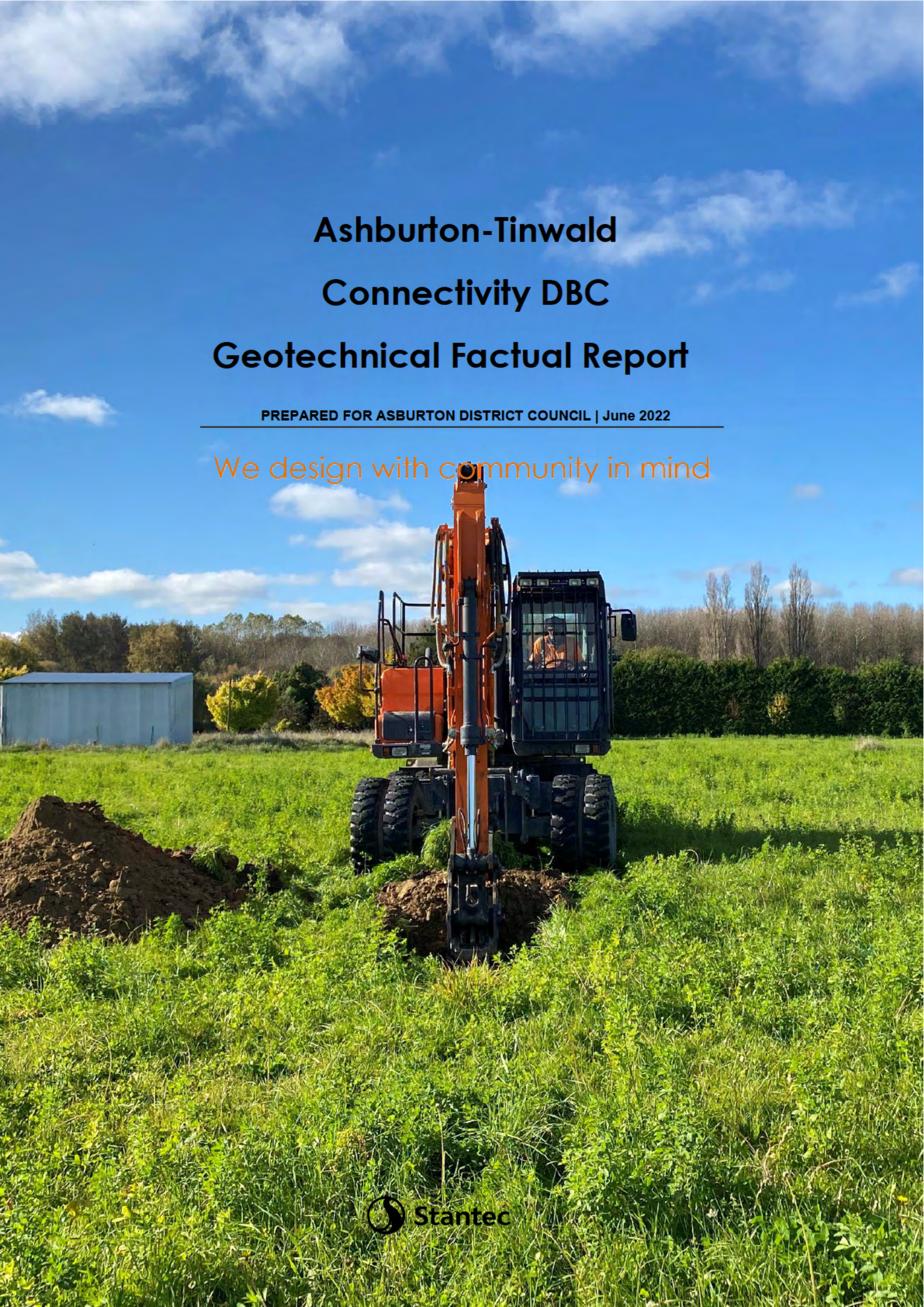
For the pseudo-static condition, less than 1 (factor of safety), it was estimated that for both DCLS and CALS events, further ground movement (lateral spreading) deformation assessment is required.

For the flow failure condition (liquefied soil condition at the end of seismic event), the model predicted no major ground failure (flow failure).

For the ground movement triggering condition, the in-situ material is not sufficiently resilient to lateral movement. Therefore, we propose replacing the in-situ material (down to the base of the liquefiable layer) with the compacted gravel fill for a distance of 20m back from each abutment. The model with the compacted gravel fill predicted a better seismic resilient response and this meets the design requirement set by the Bridge Manual.

The pile assessment concludes that the proposed pile geometry, 2 No 1.5 m diameter down to 15 m below ground level, is sufficient to support the bridge vertically. Based on the liquefaction assessment (no strata are expected to be liquefied below 4.0 to 5.0 m below ground level), lateral pile loading is not expected to be a critical case, however, the lateral pile capacity assessment will be assessed in detailed design, considering the superstructure load induced by seismic events and any loss of lateral support because of liquefaction and river scour.

Limited stripping on existing farmland is expected to be required to form fill embankments or road pavements but greater striping may be required in the existing river corridor. The material generated from stripping will not be suitable for structural fill but could be used as finishing layers on fill embankments. Structural fill form fill embankments will need to be imported to site, most likely sandy gravel from nearby quarries or river extraction operations.



Ashburton-Tinwald
Connectivity DBC
Geotechnical Factual Report

PREPARED FOR ASBURTON DISTRICT COUNCIL | June 2022

We design with community in mind

Stantec

Revision schedule

Rev No	Date	Description	Signature of Typed Name (documentation on file)			
			Prepared by	**Checked by**	**Reviewed by**	**Approved by**
1	June 2022	Factual Report	JG	BP	SW	AS

Quality statement

Project manager	Project technical lead
Ali Siddiqui	Steven Woods

PREPARED BY

CHECKED BY

REVIEWED BY

APPROVED FOR ISSUE BY

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Y P P
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Contents

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1 Introduction

Stantec have been engaged by the Ashburton District Council to prepare a detailed business case (DBC) and scheme design for the Ashburton-Tinwald Connectivity project. This includes geotechnical investigations and reporting to support the scheme design and consenting for the new bridge and road design.

Geotechnical site investigations were undertaken between 28th April and 3rd May 2022 which included two (2) boreholes and seven (7) test pits on either side riverbank and along the planned new road alignment to the southwest.

This Geotechnical Factual Reports details the site investigation locations, methodologies, and factual results from the 2022 site investigations.

Site plans and investigations logs are presented in the Appendices.

2 Site Description

The new road and urban bridge are proposed approximately 700m to the south-east of the existing State Highway 1 river crossing (Figure 2-1).

The south-western section extends approximately 1400m through flat lifestyle block type properties from Grahams Road to Carters Terrace. Between the western river terrace the existing river channel is densely vegetated with a river access track available from the south. The active river channel is approximately 200m wide and marked by meandering flow banks and sediment banks.

On the eastern side of the river, the proposed road will cross Maniaroto Park and connect into Chalmers Ave approximately 250m from the bank.

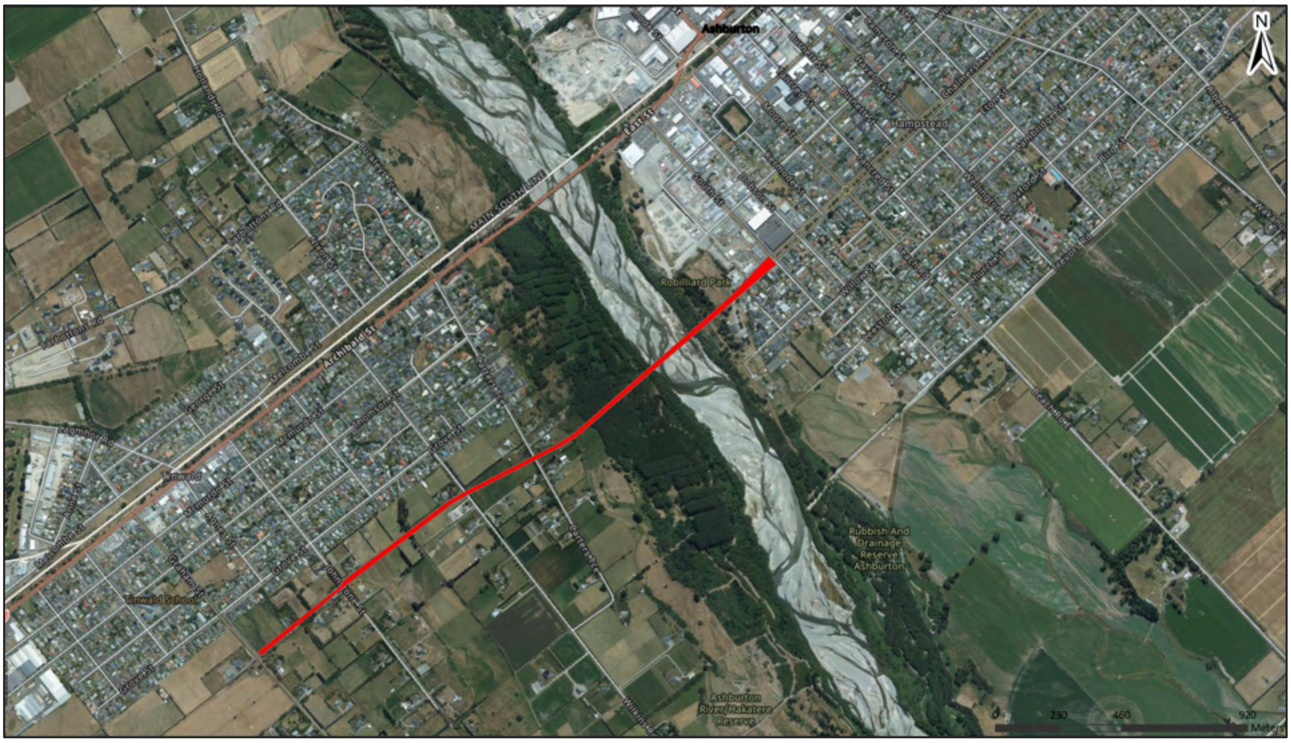


Figure 2-1: Site Location (approximate site extent in red)

3 Published Geology

The Geology of Christchurch QMAP (Forsyth, Barrell, & Jongens, 2008) indicates the site is underlain by indicates by Q1a (Holocene) and Q2a (late Pleistocene) river deposits, as shown on Figure 3-1.

Q1a River Deposits are mapped around the existing Ashburton River and Wakanui Creek channels. The published geological map describes this material as "*modern river floodplain/low-level degradation terrace. Unweathered, variably sorted gravel/sand/silt/clay. Surfaces <2-degree slope*".

Q2a River Deposits are mapped in the rest of the general area, including the existing river terraces. The map describes this material as "*unweathered, brownish-grey, variable mix of gravels/sand/silt/clay in low river terraces; locally up to 2m silt (loess) cap.*"

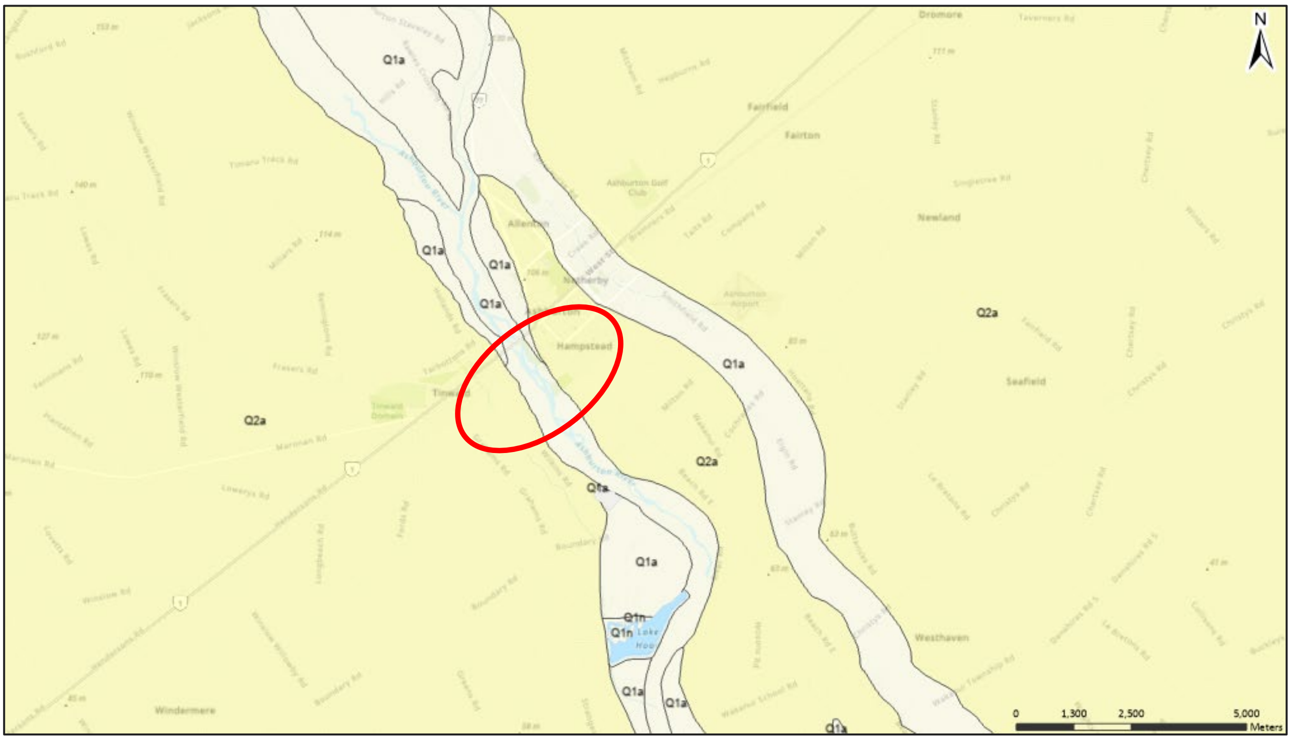


Figure 3-1: Mapped Geology (site in red)

4 Site Investigations

Site specific geotechnical investigations were completed between the 28th April and 3rd May 2022. This included:

- Two (2) sonic drilled boreholes.
- Seven (7) machine excavated test pits.

The machine investigations were undertaken by McMillans Drilling and Ashburton Contracting Ltd. and supervised by a Stantec engineering geologist. Underground services at each location were located by Underground Service Locaters Ltd. prior to drilling or excavation.

Additional boreholes and test pits (BH02, BH03, TP03) were planned, but not undertaken due to site access constraints at the time investigations were carried out.

The location details are summarized in Table 4-1 below and presented on the site plan in Appendix A

Table 4-1: Investigation Location Summary

ID	Type	Easting*	Northing*	Elevation (m RL)	Depth (m BGL***)
BH01	Borehole	1498995.93	5136623.85	88.8	20.19
BH04	Borehole	1499233.95	5136821.62	89.1	20.15
TP01	Test pit	1497629.66	5135665.67	89.5	3.00
TP02	Test pit	1497812.52	5135777.57	90.7	3.00
TP04	Test pit	1498164.82	5136051.8	90.4	2.80
TP05	Test pit	1498466.1	5136252.11	89.3	3.50
TP06	Test pit	1498725.3	5136392.62	90.6	3.30
TP07	Test pit	1499309.45	5136884.11	89.9	2.80
TP08	Test pit	1499260.14	5136874.94	89.9	2.80

*Coordinates in NZTM. Taken by hand-held GPS (+/- 3m accuracy)
*** Below Ground Level

4.1 Boreholes

Two (2) boreholes were undertaken between the 2nd and 3rd of May 2022 on the banks either side of the Ashburton River. The boreholes were undertaken to a maximum depth of 20.19m BGL and drilled using a sonic rig operated by McMillan Drilling. Core samples were logged in accordance with NZGS field description guidelines (NZGS, 2005) by the on-site geologist and boxed for storage.

In-situ Standard Penetrometer Testing (SPT) was undertaken at regular 1.5m intervals in accordance with NZS 4402:1988[1]. The SPT hammer had a calibration Energy Transfer Ratio (ETR) of 91.8%.

Both boreholes were backfilled with a combination of gravel and bentonite, and the surface reinstated to match natural conditions. All equipment and waste were removed off site once drilling was complete.

The borehole locations are presented on the site plan in Appendix A and the logs are presented in Appendix B

---

[1] Standards New Zealand, (1986). METHODS OF TESTING SOILS FOR CIVIL ENGINEERING PURPOSES (NZS 4402)

4.2 Test Pits

Seven (7) test pits were undertaken by Ashburton Contracting Ltd. between the 28th and 29th April 2022 on the south-western side of the of river between Grahams Rd and River, and on the north-eastern riverbank near Mania-O-Roto Scout Park.

The test pits were excavated to maximum depth of 3.50m BGL using a 12-tonne wheeled excavator with a 1m wide bucket. All test pits were supervised by a Stantec geologist who also conducted soil logging.

The test pit locations are presented on the site plan in Appendix A and the logs are presented in Appendix B

4.2.1 Drainage Pits

Two Transient Falling Head tests were undertaken adjacent to the locations of TP01 and TP04, records are presented in Tables 2 and 3 below.

Table 4-2: TP01 - Drain-away Test Results

Time (min)	Measured Water Level Depth from base of pit (m)
0.0000	1.500
1.0000	1.470
2.0000	1.440
5.0000	1.420
10.0000	1.340
20.0000	1.290
40.0000	1.240
66.0000	1.180

Table 4-3: TP04 - Drain-away Test Results

Time (min)	Measured Water Level Depth from base of pit (m)
0.0000	1.500
1.0000	1.280
2.0000	1.200
5.0000	1.000
10.0000	0.750
20.0000	0.400
30.0000	0.050

4.3 Groundwater Observations

Stantec field staff received anecdotal accounts of springs at various locations along the proposed alignment and, observed localised areas of vegetation consistent with high groundwater.

Groundwater observations were recorded at the time of investigation. We note that water levels were recorded in test pits immediately prior to backfilling and boreholes were dipped prior to resumption of drilling on day two and therefore may not accurately represent the groundwater level at the time of the investigations. Refer table 4-4 below.

Table 4-4: Stantec Groundwater Observations

Investigation	Depth to groundwater (meters below ground level)
TP01	2.7
TP02	2.6
TP04	2.5
TP05	3.1
TP06	3.0
TP07	2.8
TP08	2.5
BH01	2.7
BH02	1.5

5 Limitations

This report has been prepared for the Ashburton District Council in accordance with the generally accepted practices and standards in use at the time it was prepared. Stantec accepts no liability to any third party who relies on this report.

The information contained in this report is accurate to the best of our knowledge at the time of issue. Stantec has made no independent verification of this information beyond the agreed scope set out in the report.

The type, spacing and frequency of the investigations, sampling, and testing of materials were selected to meet the technical, financial, accessibility and time requirements agreed by the Client. Stantec accepts no liability for any unknown or adverse ground conditions.

Actual ground conditions encountered during site works may vary from those encountered during ground investigations. For example, subsurface groundwater conditions often change seasonally and over time. No warranty is expressed or implied that the actual conditions encountered will conform exactly to the conditions described herein. This report does not purport to describe all the site characteristics and properties. Subsurface conditions and testing relevant to construction works must be undertaken and assessed by any contractors as necessary for their own purposes.

Appendices

We design with community in mind

Appendix A Site Investigation Plan

Site Plan

Stantec Site Investigation Location Plan

Data Sources: {LINZ Data Service; https://data.linz.govt.nz/}
Basemap Service Credits: Map data © OpenStreetMap contributors, Microsoft, Facebook, Inc. and its affiliates, Esri Community Maps contributors, Map layer by Esri, Mackenzie DC, Maxar, LINZ, Stats NZ, Eagle Technology, Esri, HERE, Garmin, METI/ NASA, USGS

Appendix B Borehole Log

Stantec
Hazeldean Business Park, Level 3,
6 Hazeldean Road
Addington, Christchurch, New
Zealand, 8024

BOREHOLE LOG

Borehole ID
BH01
Sheet 1 of 2

Project Name: Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coordinates: 1498996 E 5136624 N (NZTM)	Total Depth: 20.19m
Client: Ashburton District Council		Elevation: 88 80 mRL (NZVD2016)	Logged By: BP
Description: South side of river on access track		Date: 02/05/2022 Start 03/05/2022 End	Checked By: SJ

Material Description
(Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)

Depth (m)	Geologic Unit	Material Description	Consistency/ Relative Density	Moisture Condition
0.00–(0.70)	CORE LOSS	Core loss		
(0.70)–(0.75)	ALLUVIUM	Organic SILT with some clay, trace sand and rootlets; dark brown. Firm, moist, low plasticity; sand, fine.	F	M
(0.75)–(0.90)	ALLUVIUM	SILT with some clay, trace sand and gravel; brown. Firm, moist, low plasticity; sand, fine to coarse; gravel, fine, sub-angular to sub-rounded.	F	M
(0.90)–(2.80)	ALLUVIUM	Sandy fine to coarse GRAVEL with trace silt; grey brown. Medium dense to dense, wet to saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse.	MD	W - S
(2.80)–(3.00)	ALLUVIUM	Silty fine to medium SAND with trace gravel; grey brown. Dense, wet to saturated, poorly graded; gravel, fine, sub-angular to sub-rounded.	D	W - S
(3.00)–(3.55)	ALLUVIUM	Fine to coarse SAND with some silt; brown. Dense, wet, well graded, slow dilatancy.	D	W
(3.55)–(6.08)	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt and trace cobbles; brown. Dense to very dense, saturated, well graded; gravel, sub-rounded; sand, fine to coarse; cobbles, up to 150mm.	MD	S
(6.08)–(7.18)	CORE LOSS	Core loss		
(7.18)–(7.60)	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt and trace cobbles; brown. Very dense, saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	VD	S
(7.60)–(8.60)	CORE LOSS	Core loss		
(8.60)–(8.70)	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt and trace cobbles; brown. Very dense, saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	VD	S
(8.70)–(9.12)	ALLUVIUM	Sandy SILT with minor gravel; grey. Dense, saturated, well graded; gravel, fine to medium, sub-angular to sub-rounded; sand, fine to coarse.	D	S
(9.12)–10.0	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt and trace cobbles; grey brown. Dense to medium dense, saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	D	S

Method	Core Recovery (%)	Depth (m)
SONIC	60	0.0–1.5
SPT	10	1.5–1.95
SONIC	100	1.95–3.0
SPT	100	3.0–3.45
SONIC	100	3.45–4.5
SPT	78	4.5–4.95
SONIC	100	4.95–6.0
SPT	97	6.0–6.45
SONIC	30	6.45–7.5
SPT	47	7.5–7.95
SONIC	41	7.95–9.0
SPT	78	9.0–9.45
SONIC	65	9.45–10.0

Elevation (m): 88, 87, 86, 85, 84, 83, 82, 81, 80, 79
Depth (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0

Sample and In Situ Testing

Depth (m)	Type	Results
1.5	SPT	2,3//3,2,3,3 **N=11**
3.0	SPT	5,9//9,12,9,10 **N=40**
4.5	SPT	3,10//12,15,11 ,15 **N=53**
6.0	SPT	9,13//18,18,24 **N=60+** for 210mm
7.5	SPT	10,16//15,17,1 7,11 **N=60+** for 275mm
9.0	SPT	3,4//4,7,9,10 **N=30**

Groundwater: ▼ 03-05-2022 (approx. 2.7 m)

Contractor	Inclination
McMillan Drilling Ltd.	90°
Method	**Direction**
SONIC	-
Plant	**Barrel Type**
Geoprobe 8140LC - Track	HQ

Remarks Backfilled with gravel from base of hole to 1.4m depth, bentonite from 1.4m to 0.2m and drill cuttings to surface.

Stantec
Hazeldean Business Park, Level 3,
6 Hazeldean Road
Addington, Christchurch, New Zealand, 8024

BOREHOLE LOG

Borehole ID
BH01
Sheet 2 of 2

Project Name: Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coordinates: 1498996 E 5136624 N (NZTM)	Total Depth: 20.19m
Client: Ashburton District Council		Elevation: 88 80 mRL (NZVD2016)	Logged By: BP
Description: South side of river on access track		Date: 02/05/2022 Start 03/05/2022 End	Checked By: SJ

Material Description
(Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)

Depth (m)	Geologic Unit	Material Description	Consistency/ Relative Density	Moisture Condition	Sample and In Situ Testing Type	Results
(10.64)	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt and trace cobbles; grey brown. Dense to medium dense, saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	D	S		
(11.74)	CORE LOSS	Core loss			SPT	6,11//13,18,17,12 **N=60**
(12.16)	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt and trace cobbles; grey brown. Dense to very dense, saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	D	S		
(13.00)		Silty fine to coarse GRAVEL with some sand, trace clay and cobbles; grey brown. Very dense, saturated, well graded; gravel, sub-rounded to rounded; sand, fine to coarse; cobbles, up to 100mm.	VD	W - S	SPT	4,5//18,22,20 **N=60+** for 225mm
(13.90)		Sandy fine to coarse GRAVEL with some silt and trace cobbles; grey brown. Very dense, saturated, well graded; gravel, sub-rounded; sand, fine to coarse.		S	SPT	3,17//20,18,22 **N=60+** for 225mm
(15.20)		Silty fine to coarse GRAVEL with some sand, trace clay and cobbles; grey brown. Very dense, wet to saturated, well graded; gravel, sub-rounded; sand, fine to coarse; cobbles, up to 100mm.				
(15.70)	CORE LOSS	Core loss			SPT	4,10//11,10,19,19 **N=59**
(16.10)	ALLUVIUM	Silty fine to coarse GRAVEL with some clay and sand, trace cobbles; light brown. Very dense, wet to saturated, well graded; gravel, sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	VD	S		
(18.70)		Sandy fine to coarse GRAVEL with some silt and trace cobbles; light brown. Dense to very dense, saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	D		SPT SPT	8,15//17,16,19,8 **N=60+** for 235mm 3,5//7,7,8,10 **N=32**
(20.19)		Silty Sandy fine to coarse GRAVEL with some clay, and trace cobbles; brown. Dense to very dense, saturated, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse; cobbles, up to 100mm.			SPT	9,14//18,20,20,2 **N=60+** for 230mm

Borehole terminated at 20.19m BGL due to Target depth

Method	Core Recovery (%)
SONIC	
SPT	78
SONIC	28
SPT	67
SONIC	100
SPT	93
SONIC	100
SPT	78
SONIC	84
SPT	65
SONIC	100
SPT	67
SONIC	100

Elevation (m): 78, 77, 76, 75, 74, 73, 72, 71, 70, 69
Depth (m): 10.5, 11.0, 11.5, 12.0, 12.5, 13.0, 13.5, 14.0, 14.5, 15.0, 15.5, 16.0, 16.5, 17.0, 17.5, 18.0, 18.5, 19.0, 19.5, 20.0

Other Observations | Groundwater | Installation/Backfill

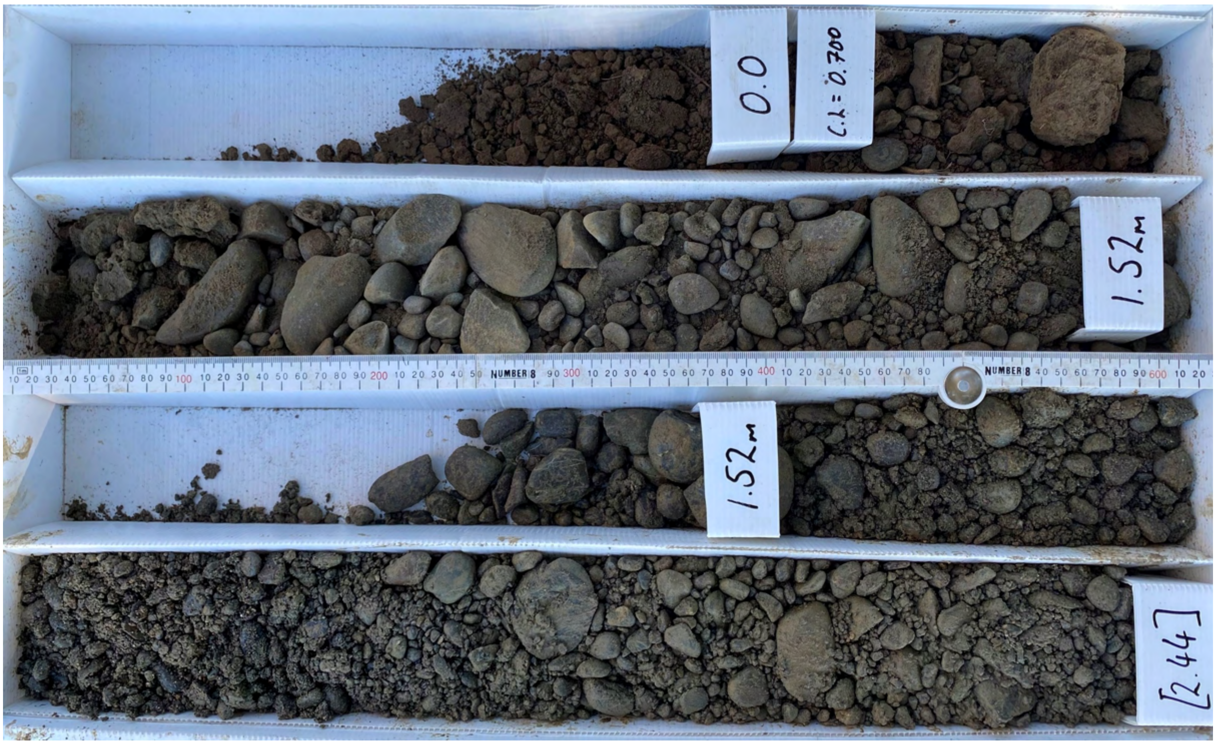
Contractor	Inclination
McMillan Drilling Ltd.	90°
Method	**Direction**
SONIC	-
Plant	**Barrel Type**
Geoprobe 8140LC - Track	HQ

Remarks Backfilled with gravel from base of hole to 1.4m depth, bentonite from 1.4m to 0.2m and drill cuttings to surface.

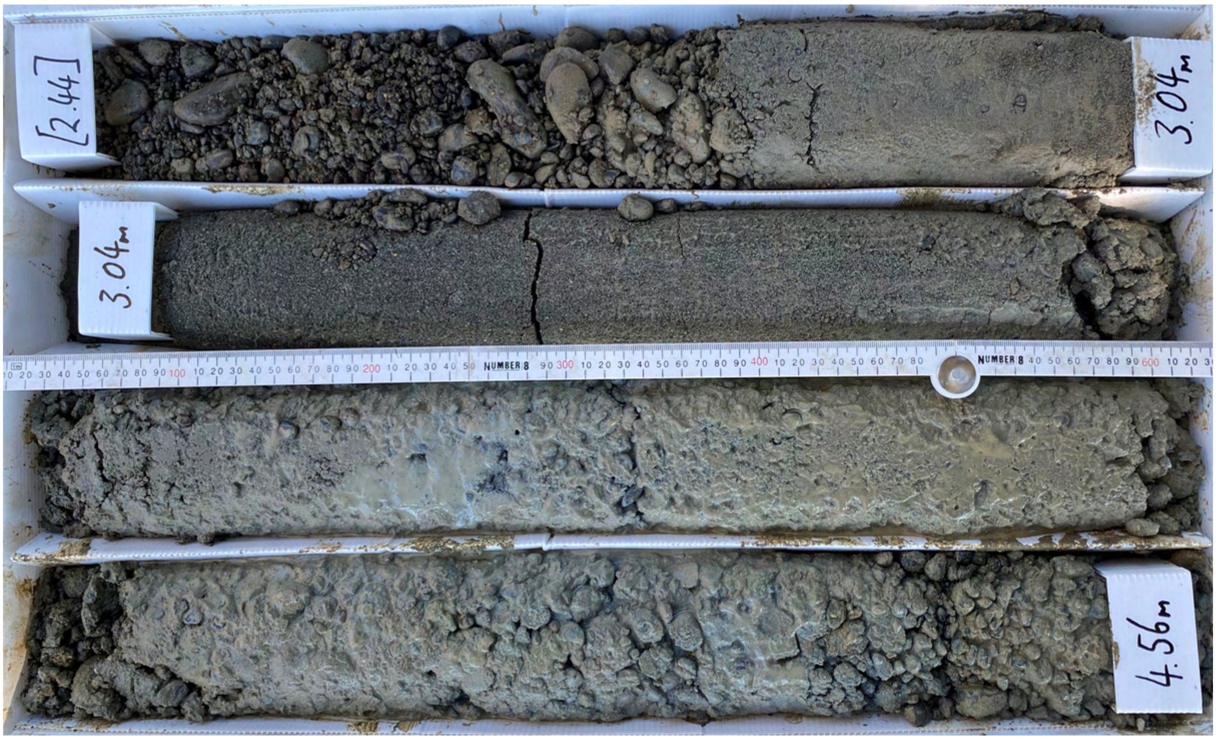
BOREHOLE PHOTOGRAPHS: BH01

PROJECT: Ashburton-Tinwald Connectivity DBC
CLIENT: Ashburton District Council
DESCRIPTION: South side of river on access track

PROJECT NO.: 310205125
DATE: 02/05/2022
COORDINATES: 1498996 E 5136624 N (NZTM)



Box 01 - 0.00m to 2.44m

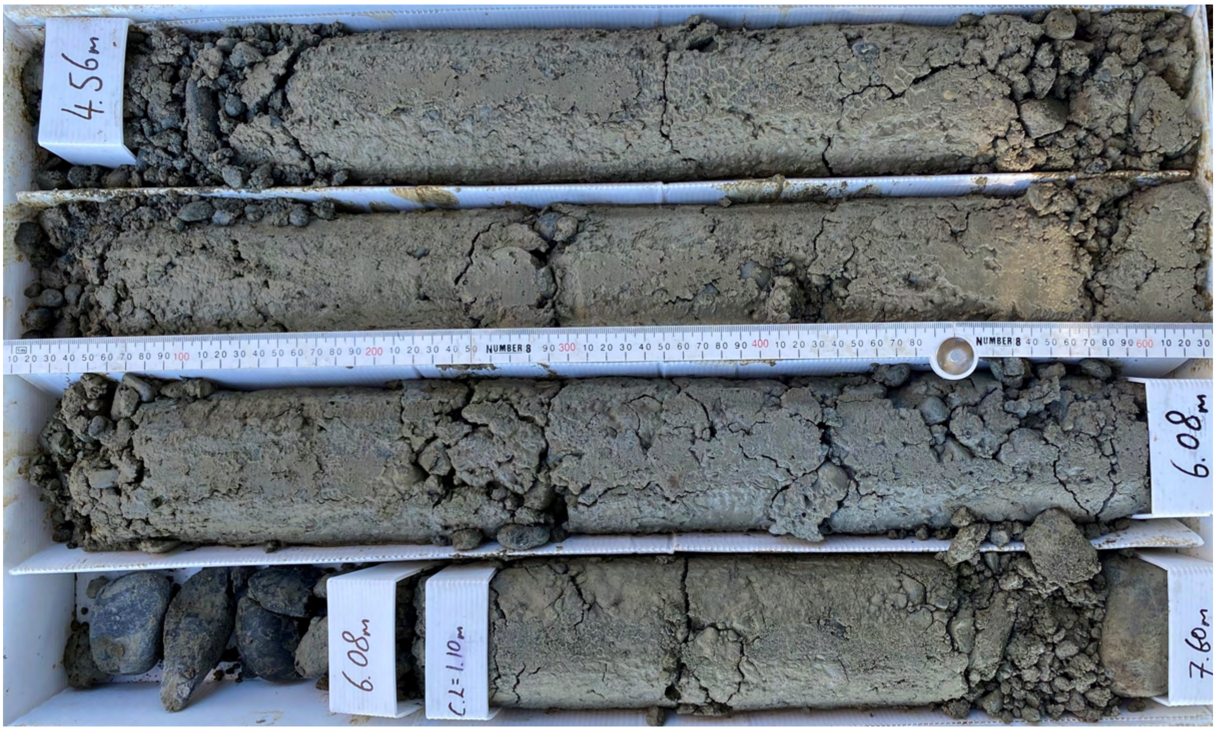


Box 02 - 2.44m to 4.56m

BOREHOLE PHOTOGRAPHS: BH01

PROJECT: Ashburton-Tinwald Connectivity DBC
CLIENT: Ashburton District Council
DESCRIPTION: South side of river on access track

PROJECT NO.: 310205125
DATE: 02/05/2022
COORDINATES: 1498996 E 5136624 N (NZTM)



Box 03 - 4.56m to 7.60m

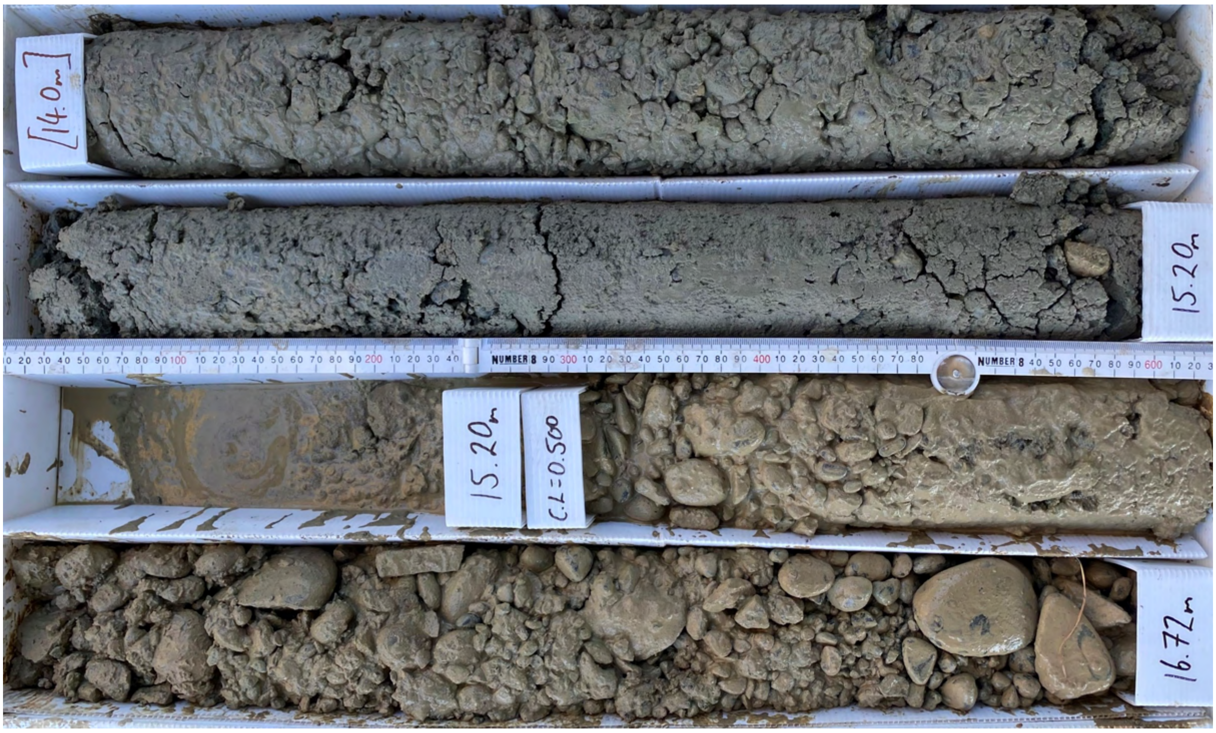
Box 04 - 7.66m to 12.16m

BOREHOLE PHOTOGRAPHS: BH01

PROJECT: Ashburton-Tinwald Connectivity DBC
CLIENT: Ashburton District Council
DESCRIPTION: South side of river on access track

PROJECT NO.: 310205125
DATE: 02/05/2022
COORDINATES: 1498996 E 5136624 N (NZTM)

Box 05 - 12.16m to 14.0m

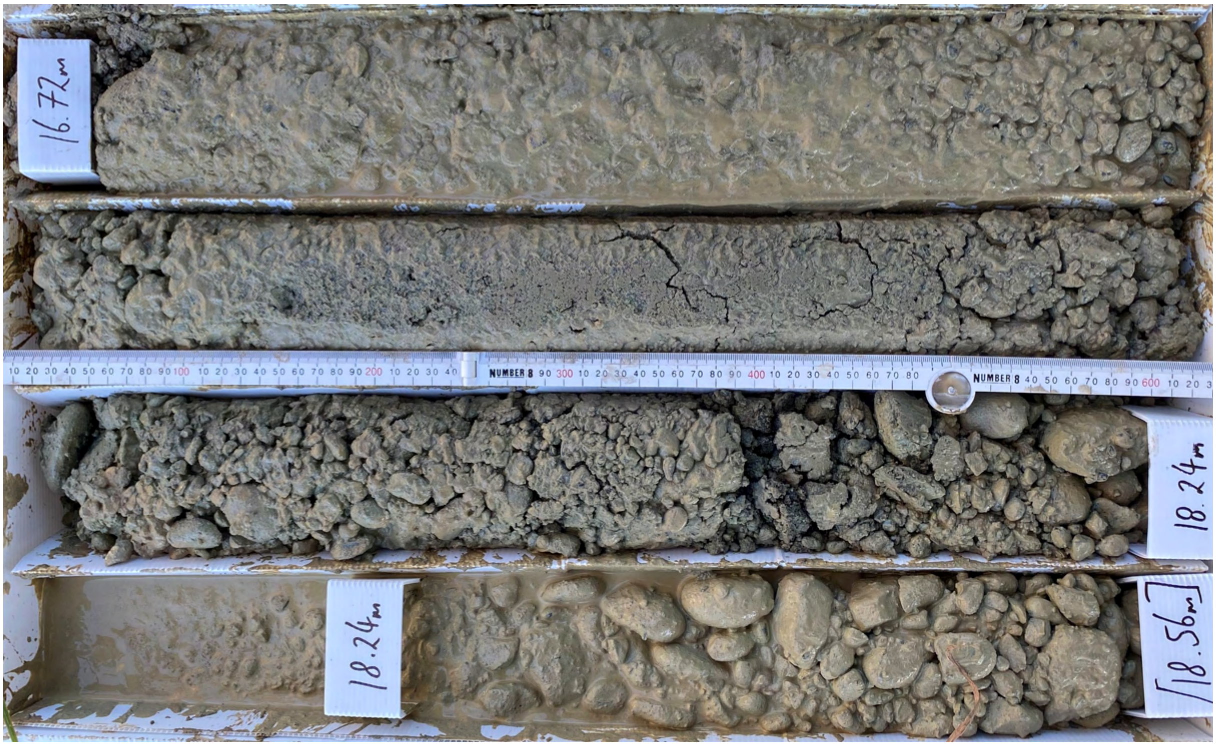


Box 06 - 14.0m to 16.72m

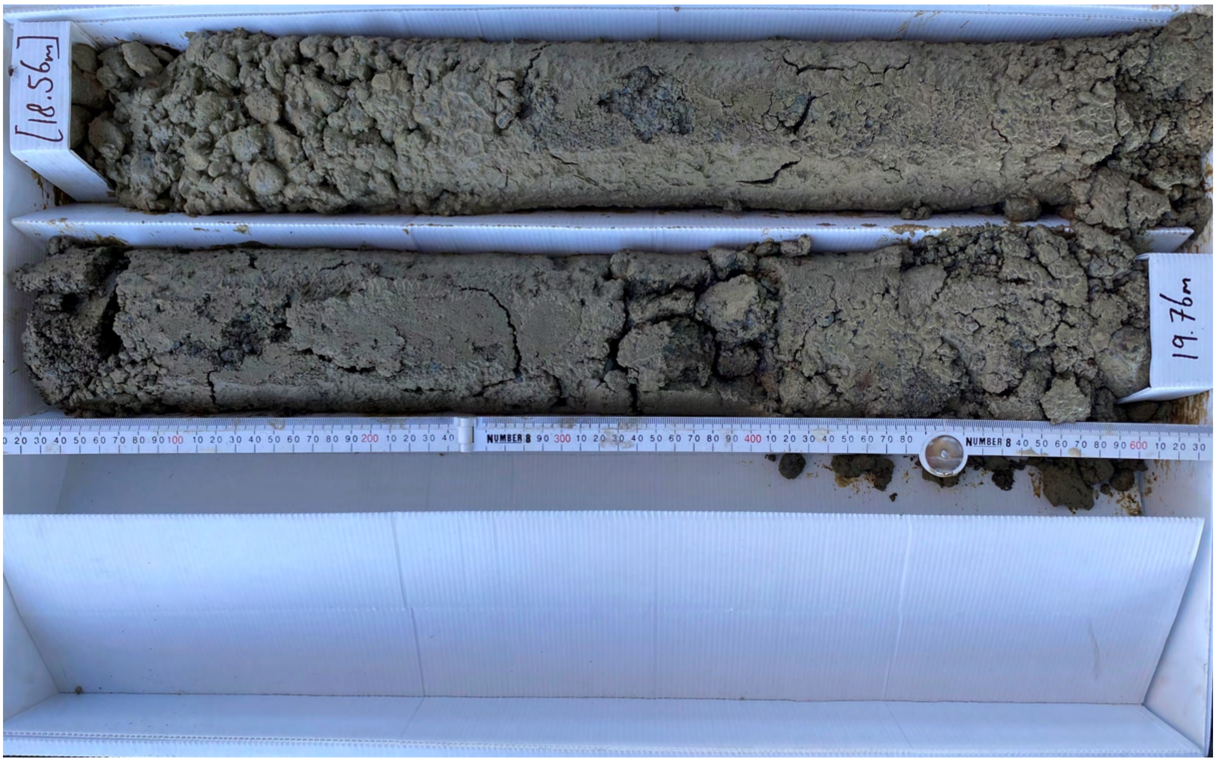
BOREHOLE PHOTOGRAPHS: BH01

PROJECT: Ashburton-Tinwald Connectivity DBC
CLIENT: Ashburton District Council
DESCRIPTION: South side of river on access track

PROJECT NO.: 310205125
DATE: 02/05/2022
COORDINATES: 1498996 E 5136624 N (NZTM)



Box 07 - 16.72m to 18.56m



Box 08 - 18.56m to 19.76m

Stantec	Hazeldean Business Park, Level 3, 6 Hazeldean Road Addington, Christchurch, New Zealand, 8024	**BOREHOLE LOG**	Borehole ID **BH04** Sheet 1 of 2

Project Name: Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coordinates:	1499234 E 5136822 N (NZTM)	Total Depth: 20.15m
Client: Ashburton District Council		Elevation:	89.10 mRL (NZVD2016)	Logged By: BP
Description: North side of bridge		Date:	28/04/2022 (Start) 29/04/2022 (End)	Checked By: SJ

Material Description

(Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)

Depth to base (m)	Geologic Unit	Material Description	Consistency/ Relative Density	Moisture Condition
(0.10)	TOPSOIL	Organic Clayey SILT with trace sand and rootlets; dark brown. Firm, moist, low plasticity, non-dilatant.		
(0.90)	ALLUVIUM	SILT, with minor clay and trace sand; brown. Firm, moist, low plasticity; sand, fine to course.	F	M
(1.40)	CORE LOSS	Core loss		
(2.80)	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt; grey brown. Dense, moist, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse.	D	M
(2.90)	CORE LOSS	Core loss		
(4.10)	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt; grey brown. Medium dense, moist, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse.	MD	W - S
(4.40)	CORE LOSS	Core loss		
(6.00)	ALLUVIUM	Sandy fine to coarse GRAVEL with some silt; grey brown. Medium dense to dense, moist, well graded; gravel, sub-angular to sub-rounded; sand, fine to coarse.	MD	W - S
(7.30)		Very dense	VD	
		Sandy fine to coarse GRAVEL with some silt, trace clay and cobbles; brown. Dense to very dense, saturated, well graded; gravel, sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	D	S

Method	Core Recovery (%)
SONIC	66
SPT	60
SONIC	100
SPT	40
SONIC	47
SPT	67
SONIC	100
SPT	71
SONIC	100
SPT	60
SONIC	100
SPT	82
SONIC	100

Sample and In Situ Testing

Type	Results
SPT	5,8//7,9,11,12 **N=39**
SPT	2,4//4,4,3,3 **N=14**
SPT	9,9//7,6,6,7 **N=26**
SPT	7,11//13,12,11,14 **N=50**
SPT	6,8//8,7,8,8 **N=31**
SPT	5,6//8,10,13,16 **N=47**

Other Observations

Groundwater: 29-04-2022

Installation/Backfill

Contractor	Inclination	Remarks
McMillan Drilling Ltd.	90°	Backfilled with gravel from base of hole to 9.3m depth and bentonite to surface.
Method	**Direction**	
SONIC	-	
Plant	**Barrel Type**	
Geoprobe 8140LS	HQ	

Stantec
Hazeldean Business Park, Level 3,
6 Hazeldean Road
Addington, Christchurch, New
Zealand, 8024

BOREHOLE LOG

Borehole ID
BH04
Sheet 2 of 2

Project Name:	Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coordinates: 1499234 E 5136822 N (NZTM)
Total Depth:	20.15m		
Client:	Ashburton District Council	Elevation:	89.10 mRL (NZVD2016)
Logged By:	BP		
Description:	North side of bridge	Date:	28/04/2022 Start 29/04/2022 End
Checked By:	SJ		

Material Description
(Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)

Elevation (m): 78, 77, 76, 75, 74, 73, 72, 71, 70, 69
Depth (m): 10.5, 11.0, 11.5, 12.0, 12.5, 13.0, 13.5, 14.0, 14.5, 15.0, 15.5, 16.0, 16.5, 17.0, 17.5, 18.0, 18.5, 19.0, 19.5, 20.0

Method	Core Recovery (%)
SONIC	
SPT	44
SONIC	100
SPT	36
SONIC	100
SPT	73
SONIC	100
SPT	100
SONIC	100
SPT	80
SONIC	100
SPT	62
SONIC	100
SPT	56

Geologic Unit	Material Description	Base depth	Consistency/ Relative Density	Moisture Condition
ALLUVIUM	Sandy fine to coarse GRAVEL with some silt, trace clay and cobbles; brown. Dense to very dense, saturated, well graded; gravel, sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	(12.50)	D	S
CORE LOSS	Core loss	(13.50)		
ALLUVIUM	Sandy fine to coarse GRAVEL with some silt, trace clay and cobbles; brown. Dense to very dense, saturated, well graded; gravel, sub-rounded; sand, fine to coarse; cobbles, up to 100mm.	(15.80)	D	S
ALLUVIUM	Sandy fine to coarse GRAVEL with some silt; grey. Very dense, saturated, poorly graded; gravel, sub-rounded to rounded; sand, fine to coarse.	(16.30)	VD	S
ALLUVIUM	Sandy fine to coarse GRAVEL with some silt and minor clay; light brown. Very dense, saturated, well graded; gravel, sub-rounded to rounded; sand, fine to course; cobbles, up to 100mm.	(20.15)	VD	S

Borehole terminated at 20.15m BGL due to Target depth

Sample and In Situ Testing: Type	Results
SPT	7,10//12,15,12,13 **N=52**
SPT	8,11//13,16,14,17 **N=60**
SPT	13,10//9,10,15,16 **N=50**
SPT	12,20//25,18,12,5 **N=60+** for 265mm
SPT	10,25//21,17,15,17 **N=60**
SPT	12,25//25,25,10 **N=60+** for 245mm
SPT	9,13//17,17,13,13 **N=60**

Other Observations | Groundwater | Installation/Backfill

Contractor	**Inclination**
McMillan Drilling Ltd.	90°
Method	**Direction**
SONIC	-
Plant	**Barrel Type**
Geoprobe 8140LS	HQ

Remarks Backfilled with gravel from base of hole to 9.3m depth and bentonite to surface.

BOREHOLE PHOTOGRAPHS: BH04

PROJECT: Ashburton-Tinwald Connectivity DBC
CLIENT: Ashburton District Council
DESCRIPTION: North side of bridge

PROJECT NO.: 310205125
DATE: 28/04/2022
COORDINATES: 1499234 E 5136822 N (NZTM)

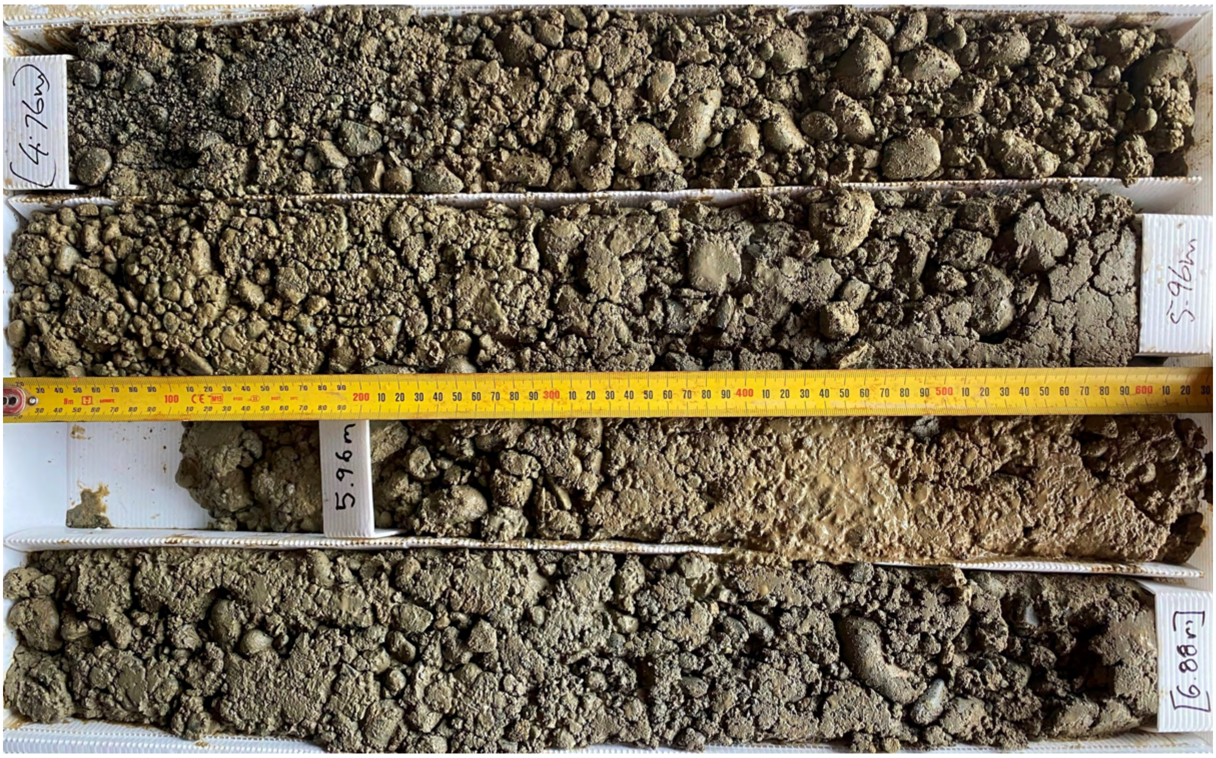
Box 01 - 0.00m to 2.32m

Box 02 - 2.32m to 4.76m

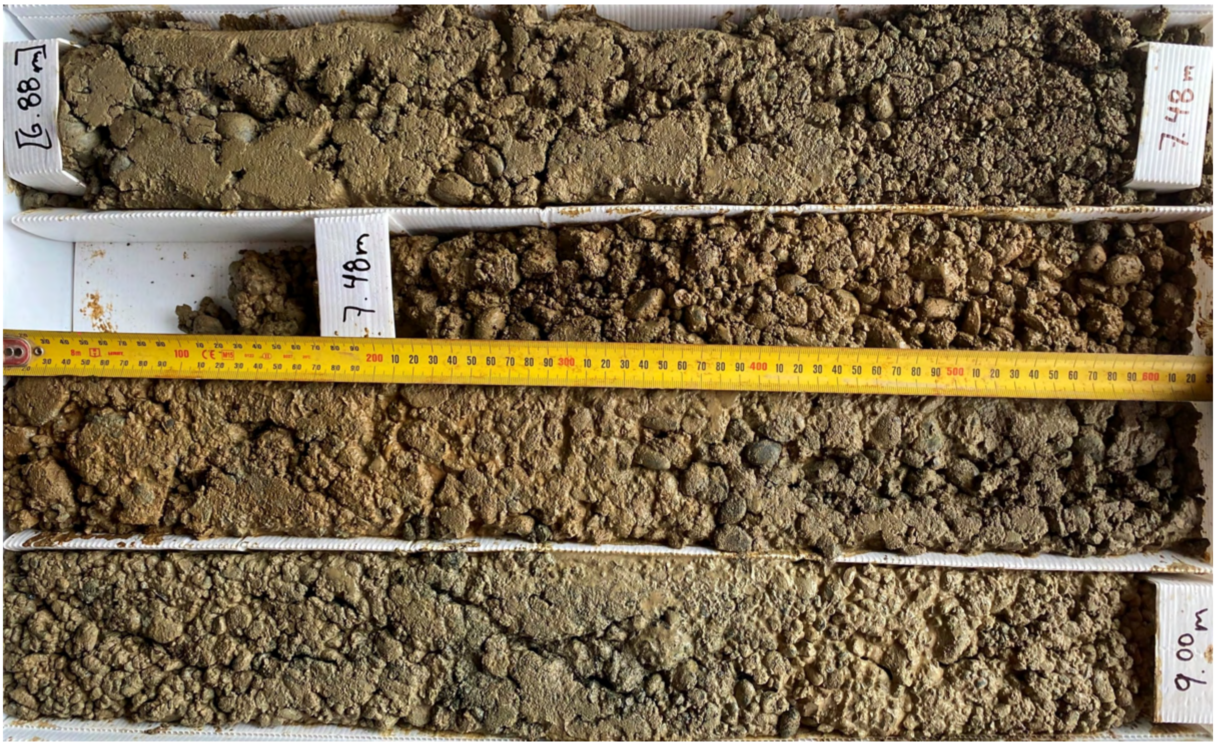
BOREHOLE PHOTOGRAPHS: BH04

PROJECT: Ashburton-Tinwald Connectivity DBC
CLIENT: Ashburton District Council
DESCRIPTION: North side of bridge

PROJECT NO.: 310205125
DATE: 28/04/2022
COORDINATES: 1499234 E 5136822 N (NZTM)



Box 03 - 4.76m to 6.88m



Box 04 - 6.88m to 9.00m

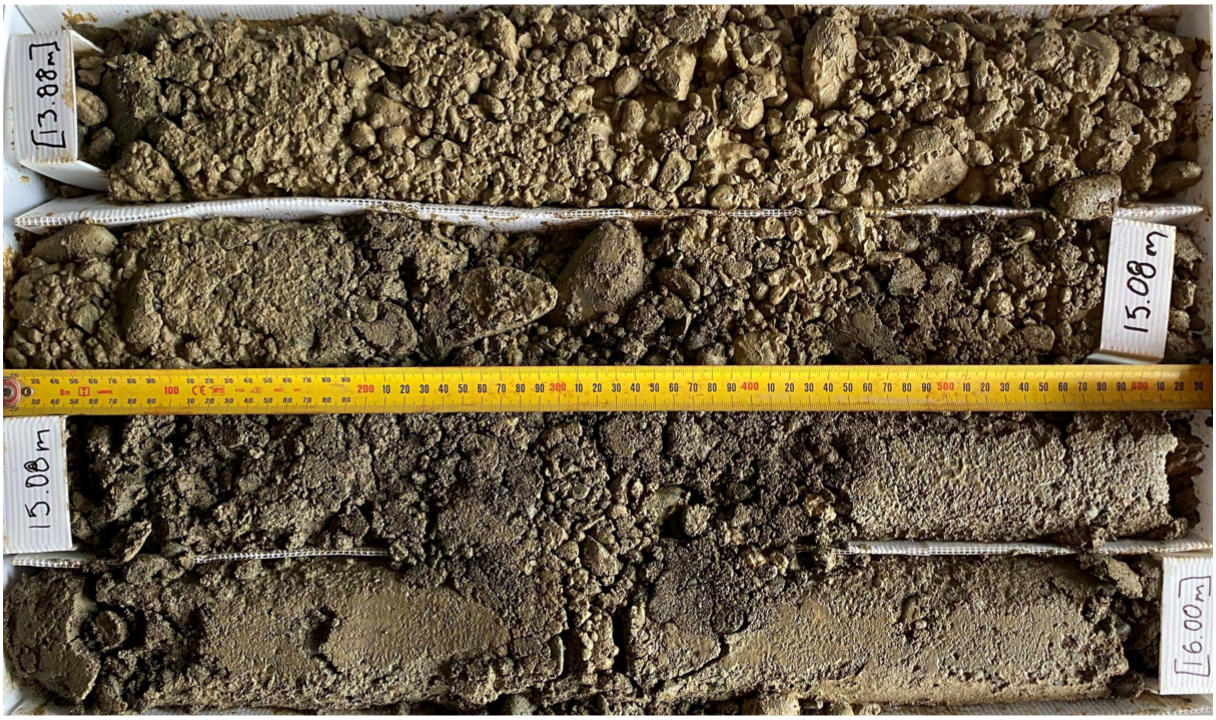
BOREHOLE PHOTOGRAPHS: BH04

PROJECT: Ashburton-Tinwald Connectivity DBC
CLIENT: Ashburton District Council
DESCRIPTION: North side of bridge

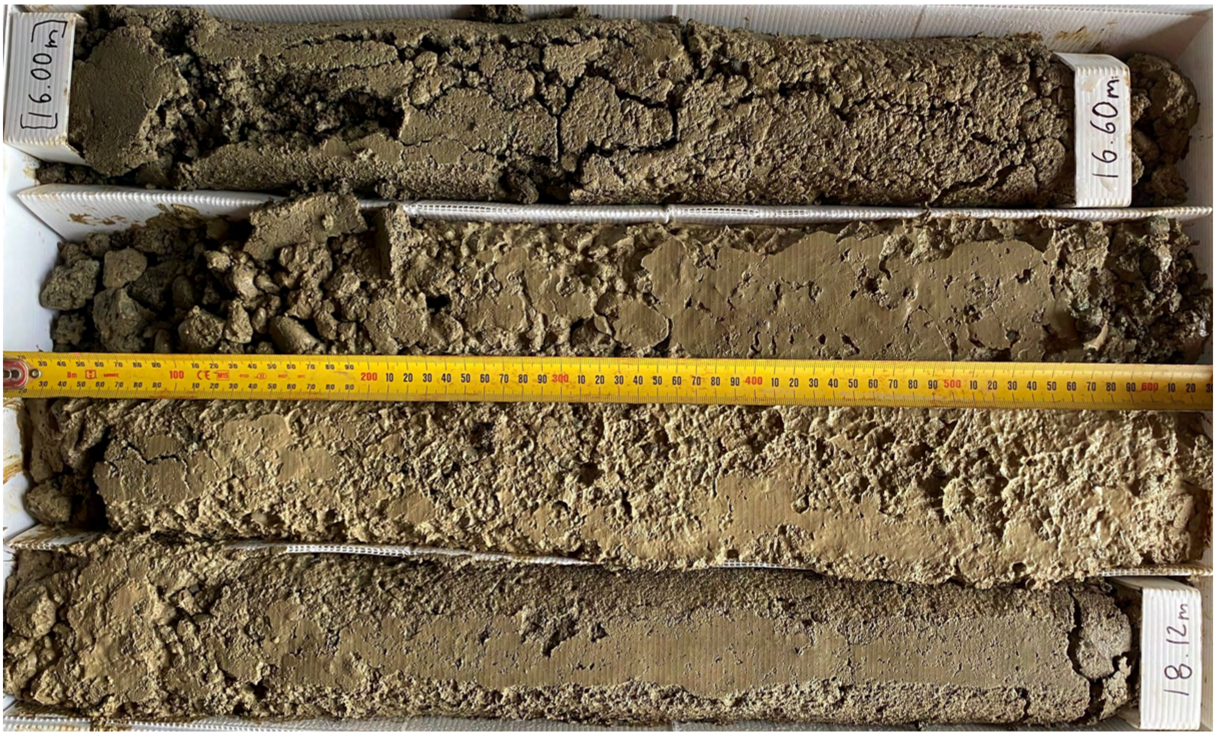
PROJECT NO.: 310205125
DATE: 28/04/2022
COORDINATES: 1499234 E 5136822 N (NZTM)

Box 05 - 9.00m to 10.84m

Box 06 - 10.84m to 13.88m



Box 07 - 13.88m to 16.00m



Box 08 - 16.00m to 18.12m

BOREHOLE PHOTOGRAPHS: BH04

PROJECT: Ashburton-Tinwald Connectivity DBC
CLIENT: Ashburton District Council
DESCRIPTION: North side of bridge

PROJECT NO.: 310205125
DATE: 28/04/2022
COORDINATES: 1499234 E 5136822 N (NZTM)

Box 09 - 18.12m to 19.64m

Appendix C Test Pit Logs

Stantec	Hazeldean Business Park, Level 3, 6 Hazeldean Road Addington, Christchurch, New Zealand, 8024	# TEST PIT LOG	Test Pit ID: **TP01** Sheet 1 of 1
Project Name:	Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coords: 1497630 E 5135666 N (NZTM)	Pit Dimensions: 1.5m x 3m
Client:	Ashburton District Council	Elevation: 89 50 mRL (NZVD2016)	Logged By: BP
Descrip ion:	Opposite 68 Grahams Road, Tinwald, Ashburton	Date: 29/04/2022 (Start) 29/04/2022 (End)	Checked By SJ

Elevation (m)	Depth (m)	Geologic Unit	Material Description (Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)	Legend	Consistency/ Relative Density	Moisture Condition	Samples	Shear Vane Reading (kPa)	Groundwater/ Seepages
	(0.30)	TOPSOIL	Organic SILT with some clay, trace rootlets; dark brown. Stiff, dry to moist, low plasticity, non-dilatant.		St	D - M			
89	0.5 (0.50)	ALLUVIUM	Clayey S LT; light orange brown. Stiff to very stiff, moist, low plasticity, non-dilatant.			M			
	(0.70)		Silty sandy fine to coarse GRAVEL with trace rootlets; yellow brown. Loosely packed, moist; gravel, sub-rounded; sand, fine to medium.		LP				
	1.0 (1.30)		Fine to medium SAND with minor silt, gravel and cobbles, trace rootlets; brown. Loosely packed, moist; gravel, fine to course, sub-rounded; obbles, up to 150mm.						
88	1.5 2.0 (2.10)		Silty sandy fine to coarse GRAVEL with minor cobbles; grey. Loosely packed, moist; gravel, sub-rounded; sand, fine to course; cobbles up to 200mm.						
87	2.5 (2.70)		Sandy fine to medium GRAVEL with minor silt and trace cobbles; grey. Loosely packed, wet to saturated; gravel, sub-rounded; sand, fine to course.			W - S			
	(3.00)					S			▼ 2.7m
86 85	3.0 3.5 4.0 4.5		Test Pit terminated at 3 00m BGL due to Collapse						

PHOTOGRAPHS/SKETCHES:

TP01 Photo 01

TP01 Photo 02

Equipment:	12T Excavator	Remarks: Encountered water inflow from ~2.7m depth. Standing water level at 2.9m prior to backfilling.
Contractor:	Ashburton Contracting Ltd	

Stantec
Hazeldean Business Park, Level 3, 6
Hazeldean Road
Addington, Christchurch, New
Zealand, 8024

TEST PIT LOG

Test Pit ID:
TP02
Sheet 1 of 1

Project Name:	Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coords:	1497813 E 5135778 N (NZTM)	Pit Dimensions: 1.5m x 3m
Client:	Ashburton District Council		Elevation:	90.70 mRL (NZVD2016)	Logged By: BP
Descrip ion:	68 Johnstone Street, Tinwald, Ashburton		Date:	24/04/2022 Start 24/04/2022 End	Checked By SJ

Material Description

(Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)

Elevation (m)	Depth (m)	Geologic Unit	Material Description	Legend	Consistency/ Relative Density	Moisture Condition	Samples	Shear Vane Reading (kPa)	Groundwater/ Seepages
		TOPSOIL	Organic SILT with minor clay and trace rootlets; dark brown. Firm, dry to moist, low plasticity, non-dilatant.	(0.30)	F	D - M			
90	0.5	ALLUVIUM	Clayey S LT; light brown. Stiff, moist, low plasticity, non-dilatant.	(0.85)	St	M			
	1.0 1.5		Sandy GRAVEL with some cobbles and trace silt; grey, loosely packed, moist; gravel, sub-rounded; sand fine to course; cobbles up to 150mm.	(1.50)	LP				
89 88	2.0 2.5 3.0		Becomes saturated	(3.00)		S			2.6m
87	3.5 4.0 4.5		Test Pit terminated at 3 00m BGL due to Collapse						

PHOTOGRAPHS/SKETCHES:


TP02 Photo 01


TP02 Photo 02

Equipment: 12T Excavator
Contractor: Ashburton Contracting Ltd

Remarks:
Encountered water inflow from ~1 5m depth. Standing water level at 2.6m prior to backfilling.

Stantec	Hazeldean Business Park, Level 3, 6 Hazeldean Road Addington, Christchurch, New Zealand, 8024	**TEST PIT LOG**	Test Pit ID: **TP04** Sheet 1 of 1

Project Name:	Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coords:	1498165 E 5136052 N	(NZTM)	Pit Dimensions: 1.5m x 3m
Client:	Ashburton District Council		Elevation:	90.40 mRL	(NZVD2016)	Logged By: BP
Descrip ion:	74 Wilkins Road, Tinwald, Ashburton		Date:	29/04/2022 Start	29/04/2022 End	Checked By SJ

Elevation (m)	Depth (m)	Geologic Unit	Material Description (Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)	Legend	Consistency/ Relative Density	Moisture Condition	Samples	Shear Vane Reading (kPa)	Groundwater/ Seepages
90	0.5	FILL	S LT with minor clay, trace gravel, cobbles, rootlets and rubbish fragments; dark brown. Firm, moist, low plasticity, non-dilatant; gravel, sub-rounded, greywacke; cobbles, up to 150mm; rubbish comprises plastic pieces.	(0.60)	F	M			
		ALLUVIUM	S LT with some clay; yellow brown. Stiff to very stiff, moist, low plasticity, non-dilatant. *- 0.70m - Becomes orange and grey mottled.*	(0.90)	St				
	1.0		S LT with trace clay; orange brown. Firm, moist, low plasticity.	(1.20)	F				
89 88	1.5 2.0		Sandy fine to coarse GRAVEL with trace silt and cobbles; grey. Loosely packed, moist to wet; gravel, sub-rounded; sand, fine to coarse; cobbles up to 200mm.	(2.50)	LP	M - W			
	2.5		*- 2.50m - Becomes saturated.*	(2.80)		S			▼ 2.5m
87 86	3.0 3.5 4.0 4.5		Test Pit terminated at 2 80m BGL due to Collapse						

PHOTOGRAPHS/SKETCHES:

TP04 Photo 01

TP04 Photo 02

Equipment:	12T Excavator	Remarks: Encountered water inflow from ~2 5m depth. Standing water level at 2.7m prior to backfilling.
Contractor:	Ashburton Contracting Ltd	

Stantec

Hazeldean Business Park, Level 3, 6
Hazeldean Road
Addington, Christchurch, New
Zealand, 8024

TEST PIT LOG

Test Pit ID:

TP05

Sheet 1 of 1

Project Name:	Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coords:	1498466 E 5136252 N (NZTM)
Pit Dimensions:	1.5m x 3m			
Client:	Ashburton District Council		Elevation:	89 30 mRL (NZVD2016)
Logged By:	BP			
Description:	58 Carters Terrace, Tinwald, Ashburton		Date:	28/04/2022 Start 28/04/2022 End
Checked By	SJ			

Material Description

(Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)

Elevation (m)	Depth (m)	Geologic Unit	Material Description	Legend	Consistency/ Relative Density	Moisture Condition	Samples	Shear Vane Reading (kPa)	Groundwater/ Seepages
	(0.30)	TOPSOIL	Organic SILT with minor clay; dark brown. Firm to stiff, dry to moist, low plasticity, non-dilatant.		F	D - M			
	(1.10)	ALLUVIUM	SILT, with some clay; light brown. Stiff, moist, low plasticity, non-dilatant. *- 0.80m - Orange brown mottled.*		St	M			
	(1.90)		Clayey SILT with trace sand and rootlets; orange grey, mottled brown. Very stiff, moist, low plasticity, non-dilatant; sand, fine to coarse.		VSt				
	(3.10)		Sandy fine to coarse GRAVEL with trace silt and cobbles; grey. Loosely packed, moist to wet; gravel, sub-rounded; sand, fine to course; cobbles, up to 200mm.		LP	M - W			
	(3.50)		Becomes saturated.			S			▼ 3.1m
			Test Pit terminated at 3 50m BGL due to Machine Limit						

PHOTOGRAPHS/SKETCHES:


TP05 Photo 01


TP05 Photo 02

Equipment: 12T Excavator

Contractor: Ashburton Contracting Ltd

Remarks:
Encountered water inflow from ~3.1m depth. Standing water level at 3.1m prior to backfilling.

Hazeldean Business Park, Level 3, 6 Hazeldean Road
Addington, Christchurch, New Zealand, 8024

TEST PIT LOG

Test Pit ID:
TP06
Sheet 1 of 1

Project Name:	Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coords:	1498725 E 5136393 N (NZTM)	Pit Dimensions: 1.5m x 3m
Client:	Ashburton District Council		Elevation:	90.60 mRL (NZVD2016)	Logged By: BP
Description:	61 Carters Terrace, Tinwald, Ashburton		Date:	28/04/2022 Start 28/04/2022 End	Checked By SJ

Material Description

(Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)

Elevation (m)	Depth (m)	Geologic Unit	Material Description	Legend	Consistency/ Relative Density	Moisture Condition	Samples	Shear Vane Reading (kPa)	Groundwater/ Seepages
	(0.20)	TOPSOIL	Organic SILT with some clay and trace rootlets; dark brown. Firm to stiff, moist, low plasticity, non-dilatant.		F	M			
	(0.40)	ALLUVIUM	SILT with some clay; light brown. Stiff to very stiff, low plasticity, non-dilatant.		St				
90	0.5 (0.90)		Fine to coarse SAND with some silt; light brown, brown mottle/ inclusions. Loosely packed, moist, poorly graded.		LP				
89	1.0 1.5 (1.90)		Clayey SILT; grey brown, brown mottle/ inclusions. Very stiff, moist, low plasticity; trace tree roots present.		VSt				
88	2.0 2.5 3.0 (3.30)		Sandy fine to coarse GRAVEL with trace silt and cobbles; brown. Loosely packed, moist to wet, well graded; gravel, sub-rounded; sand, fine to coarse; cobbles, up to 200mm.		LP	M - W			▼ 3.0m
87 86	3.5 4.0 4.5		Test Pit terminated at 3 30m BGL due to Machine Limit						

PHOTOGRAPHS/SKETCHES:


TP06 Photo 01


TP06 Photo 02

Equipment:	12T Excavator	Remarks: Encountered water inflow from ~3 0m depth. Standing water level at 3.0m prior to backfilling.
Contractor:	Ashburton Contracting Ltd	

Stantec

Hazeldean Business Park, Level 3, 6
Hazeldean Road
Addington, Christchurch, New
Zealand, 8024

TEST PIT LOG

Test Pit ID:
TP07
Sheet 1 of 1

Project Name:	Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coords:	1499309 E 5136884 N (NZTM)	Pit Dimensions: 1.5m x 3m
Client:	Ashburton District Council		Elevation:	89 90 mRL (NZVD2016)	Logged By: BP
Descrip ion:	Ashburton District Council Road Reserve		Date:	28/04/2022 Start 28/04/2022 End	Checked By SJ

Material Description

(Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)

Elevation (m)	Depth (m)	Geologic Unit	Material Description	Legend	Consistency/ Relative Density	Moisture Condition	Samples	Shear Vane Reading (kPa)	Groundwater/ Seepages
		FILL	Sandy fine to medium GRAVEL with minor silt, and trace rootlets; brown. Loosely packed, moist; gravel, angular to sub-rounded; sand, fine to course.	(0.10)					
		ALLUVIUM	Silty fine to medium SAND with trace gravel and rootlets; brown. Loosely packed, moist; gravel, fine to coarse, sub-rounded.	(0.40)		M			
89	0.5 1.0 1.5 2.0 2.5		Sandy fine to coarse GRAVEL with minor cobbles; grey brown. Loosely packed, moist to wet; gravel, sub-rounded; sand, fine to course; cobbles, up to 200mm.	(2.60)	LP	M - W			
88			S LT with some clay; orange brown, grey mottled. Firm to stiff, low plasticity, non-dilatant.	(2.80)	F	M			
87 86	3.0 3.5 4.0 4.5		Test Pit terminated at 2 80m BGL due to Collapse						2.8m

PHOTOGRAPHS/SKETCHES:

TP07 Photo 01

TP07 Photo 02

Equipment: 12T Excavator
Contractor: Ashburton Contracting Ltd

Remarks:
Encountered water inflow from ~2.7m depth. Standing water level at 2.75m prior to backfilling.

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Hazeldean Business Park, Level 3, 6
Hazeldean Road
Addington, Christchurch, New
Zealand, 8024

TEST PIT LOG

Test Pit ID:
TP08
Sheet 1 of 1

Project Name:	Ashburton-Tinwald Connectivity DBC	Project No. 310205125	Coords:	1499260 E 5136875 N (NZTM)	Pit Dimensions: 1.5m x 3m
Client:	Ashburton District Council		Elevation:	89 90 mRL (NZVD2016)	Logged By: BP
Description:	Southeast corner of Ashburton Domain, adjacent road reserve.		Date:	28/04/2022 Start 28/04/2022 End	Checked By SJ

Elevation (m)	Depth (m)	Geologic Unit	Material Description (Logging carried out in accordance with Guidelines for the Field Classification of Soil and Rock for Engineering Purposes, New Zealand Geotechnical Society, 2005)	Legend	Consistency/ Relative Density	Moisture Condition	Samples	Shear Vane Reading (kPa)	Groundwater/ Seepages
		TOPSOIL	Organic SILT with minor sand, and trace rootlets; dark brown. Firm, dry to moist, low plasticity, non-dilatant.	(0.10)	F	D - M			
	0.5	ALLUVIUM	SILT with trace clay and sand; brown. Dry to moist, stiff to very stiff, low plasticity, non-dilatant.	(0.80)	St				
89 88	1.0 1.5 2.0		Sandy fine to coarse GRAVEL with trace silt and cobbles; grey. Loosely packed, moist to wet; gravel, sub-rounded; sand, fine to coarse; cobbles, up to 200mm.	(2.50)	LP	M - W			
	2.5		Becomes Saturated.	(2.80)		S			2.5m
87 86	3.0 3.5 4.0 4.5		Test Pit terminated at 2 80m BGL due to Collapse						

PHOTOGRAPHS/SKETCHES:

TP08 Photo 01

TP08 Photo 02

Equipment: 12T Excavator
Contractor: Ashburton Contracting Ltd

Remarks:
Encountered water inflow from ~2 5m depth. Standing water level at 2.5m prior to backfilling.

DESIGN WITH
COMMUNITY
IN MIND

Communities are fundamental. Whether around the corner or across the globe, they provide a foundation, a sense of place and of belonging. That's why at Stantec, we always design with community in mind.

We care about the communities we serve—because they're our communities too. This allows us to assess what's needed and connect our expertise, to appreciate nuances and envision what's never been considered, to bring together diverse perspectives so we can collaborate toward a shared success.

We're designers, engineers, scientists, and project managers, innovating together at the intersection of community, creativity, and client relationships. Balancing these priorities results in projects that advance the quality of life in communities across the globe.

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SPT Based Liquefaction Analysis Calculation Sheet

Input Parameters			
α_{max} (g) =	0.60	Energy ratio	91.8
Earthquake magnitude, M =	6.10	Depth limit	20
Water table depth (m) =	2.70	Machine Hole:	BH01
Kulhawy & Mayne 1990 (EPRI EL-6800) S_u = N *	6.00	Free Face Depth	1
Change to new values to emulate preloading strength gain		Distance from FF	3
Embankment Surcharge — Embankment Height =	0.00	Slope grade (%)	33.5
Embankment Surcharge — Embankment Fill Unit Weight =	19.00	Settlement (cm)	1.54
Borehole diameter (mm) =	65	Lateral D. Index	3.82
Requires correction for sample liners (YES/NO):	NO	LPI	1

Summary Results

Top of ground RL	0
Water RL	-2.70

Information

Based on the Idris and Boulanger spreadsheet for SPT-based liquefaction triggering analysis of a single boring

Original data from Boulanger, Mejia, and Idriss (1997) "Liquefaction at Moss Landing During Loma Prieta Earthquake"

Revision 13

Date 26/11/2019

Lateral Displacement Based on Slope Grade (0.2% - 3.5%)	n/a
Lateral Displacement Based on Free Face Depth and Distance	10 cm

Power m that depends on the sand properties and relative density (Boulanger 2003b)	0.5

Borehole Data

SPT sample number	Depth (m)	Measured N	Unit Weight (kN/m³) γ	Soil type (USCS)	Top of Layer (m)	Bottom of Layer (m)	Flag "Clay" "Unsaturated" "Unreliable"	Fines content (%)
1	1.50	11	19.0	SM	1.5	2.3	Unreliable	5
2	3.00	40	19.0	GW	2.3	3.8	Unreliable	5
3	4.50	50	19.0	GW	3.8	5.3	Unreliable	5
4	6.00	50	19.0	GW	5.3	6.8	Unreliable	5
5	7.50	50	19.0	GW	6.8	8.3	Unreliable	5
6	9.00	30	19.0	GW	8.3	9.8	Unreliable	5
7	10.50	50	19.0	GW	9.8	11.3	Unreliable	5
8	12.00	50	19.0	GW	11.3	12.8	Unreliable	5
9	13.50	50	19.0	GW	12.8	14.3	Unreliable	5
10	15.00	50	19.0	GW	14.3	15.8	Unreliable	5
11	16.50	50	19.0	GW	15.8	17.3	Unreliable	5
12	18.00	32	19.0	GW	17.3	18.8	Unreliable	5
13	19.50	50	19.0	GW	18.8	20.0	Unreliable	5
14					20.0	20.0		
15					20.0	20.0		
16					20.0	20.0		
17					20.0	20.0		
18					20.0	20.0		
19					20.0	20.0		
20					20.0	20.0		
	20.00							

Estimations

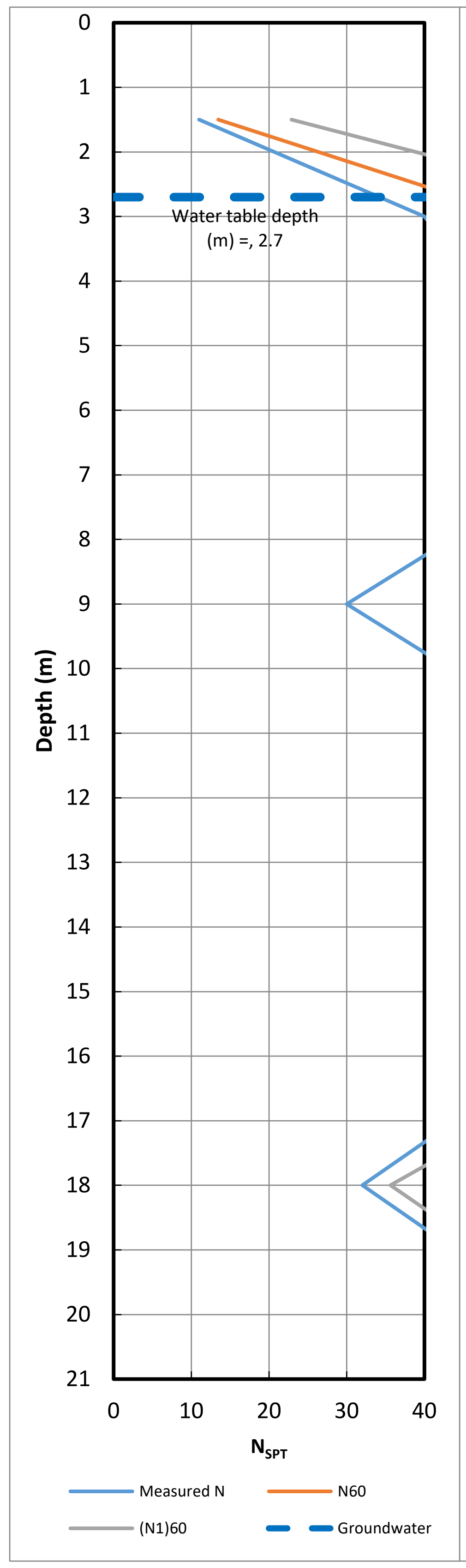
SPT sample number	S_u (kPa)	Stiffness Reduction Factor β	N_{60}	$(N_1)_{60}$	$CSR_{M,\sigma'v}$	$CRR_{M,\sigma'v}$	tanφ'	S_r/σ'_{v0} Sig Void R	S_r/σ'_{v0} Neg Void R	Global FS	Liquefaction FS
1		0.63	13.5	22.9	0.386	0.332	0.46	0.46	0.46	0.86	0.86
2		1.00	52.0	70.8	0.396	3.333	0.54	0.54	0.54	2.00	2.00
3		1.00	72.7	88.5	0.459	3.333	0.56	0.56	0.56	2.00	2.00
4		1.00	72.7	80.8	0.491	3.333	0.55	0.55	0.55	2.00	2.00
5		1.00	72.7	74.8	0.506	3.333	0.55	0.55	0.55	2.00	2.00
6		1.00	45.9	44.2	0.508	3.246	0.50	0.50	0.50	2.00	2.00
7		1.00	76.5	69.4	0.504	3.129	0.54	0.54	0.54	2.00	2.00
8		1.00	76.5	65.8	0.494	3.025	0.53	0.53	0.53	2.00	2.00
9		1.00	76.5	62.8	0.481	2.930	0.53	0.53	0.53	2.00	2.00
10		1.00	76.5	60.1	0.466	2.844	0.53	0.53	0.53	2.00	2.00
11		1.00	76.5	57.7	0.451	2.765	0.52	0.52	0.52	2.00	2.00
12		1.00	49.0	35.6	0.435	1.733	0.49	0.49	0.49	2.00	2.00
13		1.00	76.5	53.8	0.420	2.623	0.52	0.52	0.52	2.00	2.00
14		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
15		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
16		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
17		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
18		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
19		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
20		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00

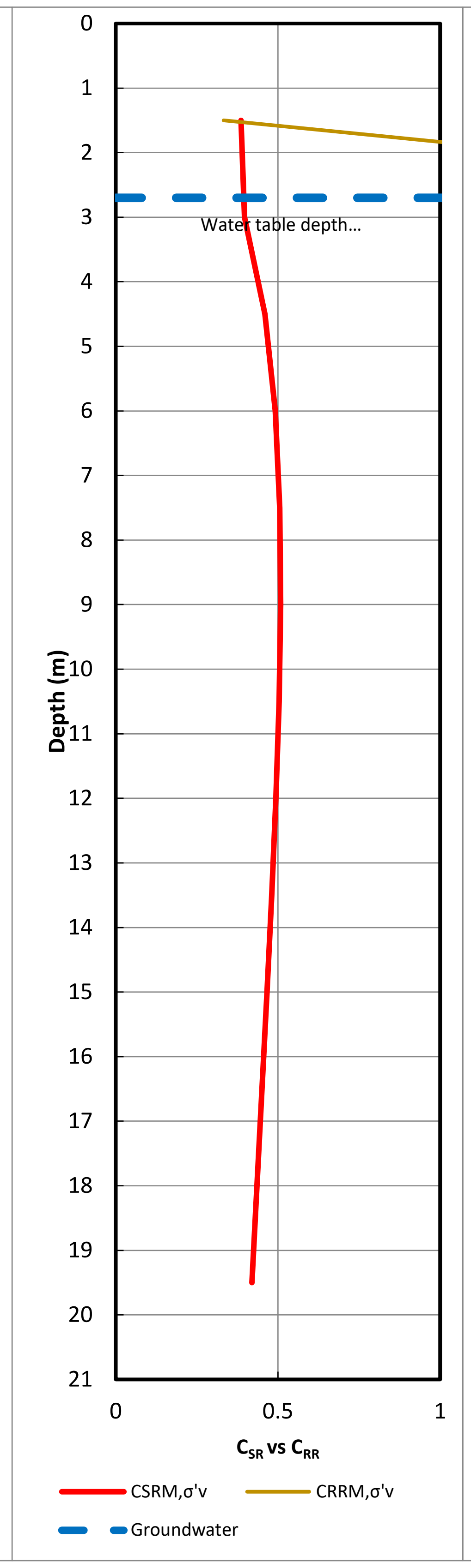
Settlement

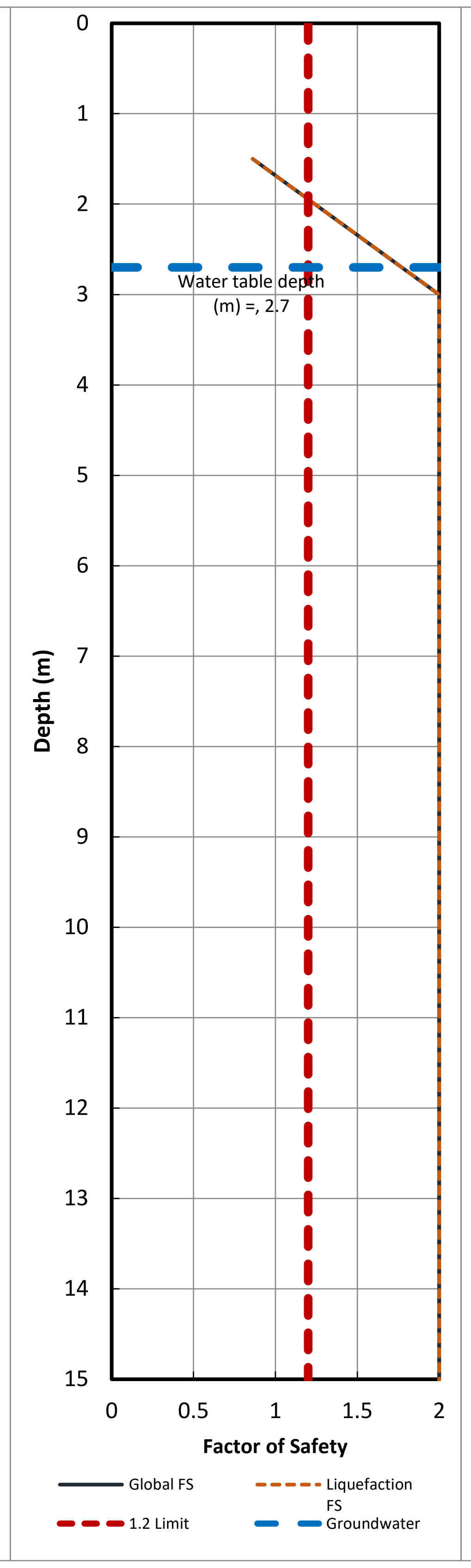
SPT sample number	Limiting shear strain γ_{lim}	Parameter F_a	Max shear strain γ_{max} (%)	DH_i (m)	$DLDI_i$ (cm)	Cumulative LDI (cm)	Vertical reconsol Strain e_V	DS_i (cm)	Cumulative Settlement (cm)
1	0.114	0.357	5.087	0.750	3.815	4	0.021	1.540	1.54
2	0.000	-3.365	0.000	1.500	0.000	0	0.000	0.000	0.00
3	0.000	-4.978	0.000	1.500	0.000	0	0.000	0.000	0.00
4	0.000	-4.265	0.000	1.500	0.000	0	0.000	0.000	0.00
5	0.000	-3.721	0.000	1.500	0.000	0	0.000	0.000	0.00
6	0.003	-1.124	0.000	1.500	0.000	0	0.000	0.000	0.00
7	0.000	-3.242	0.000	1.500	0.000	0	0.000	0.000	0.00
8	0.000	-2.929	0.000	1.500	0.000	0	0.000	0.000	0.00
9	0.000	-2.662	0.000	1.500	0.000	0	0.000	0.000	0.00
10	0.000	-2.432	0.000	1.500	0.000	0	0.000	0.000	0.00
11	0.000	-2.232	0.000	1.500	0.000	0	0.000	0.000	0.00
12	0.020	-0.480	0.000	1.500	0.000	0	0.000	0.000	0.00
13	0.000	-1.898	0.000	1.250	0.000	0	0.000	0.000	0.00
14	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00
15	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00
16	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00
17	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00
18	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00
19	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00
20	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00
					LDI =	4		S =	1.54

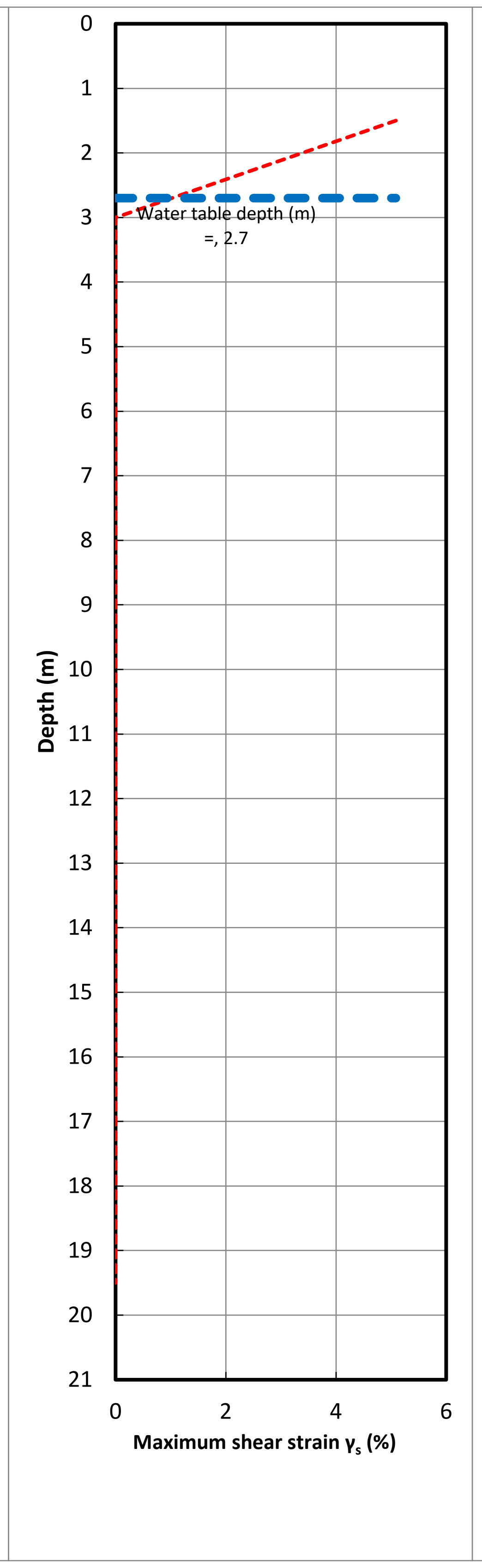
LPI

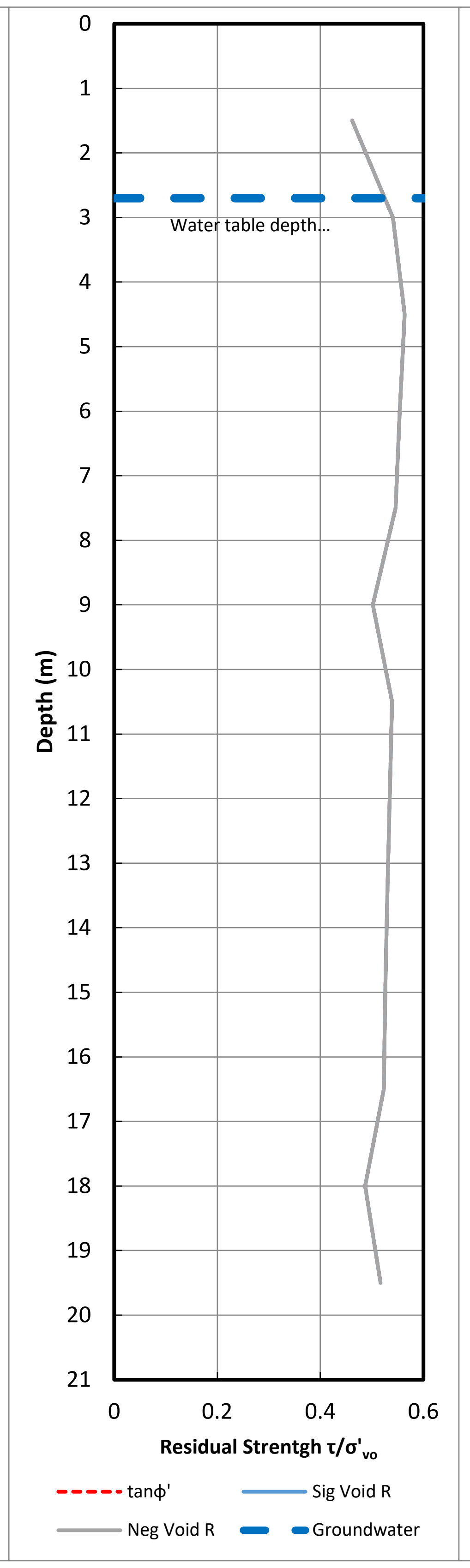
SPT sample number	W(z)	F1	F1W(z)	LPI
1	9.25	0.14	1.29	1
2	8.50	0.00	0.00	0
3	7.75	0.00	0.00	0
4	7.00	0.00	0.00	0
5	6.25	0.00	0.00	0
6	5.50	0.00	0.00	0
7	4.75	0.00	0.00	0
8	4.00	0.00	0.00	0
9	3.25	0.00	0.00	0
10	2.50	0.00	0.00	0
11	1.75	0.00	0.00	0
12	1.00	0.00	0.00	0
13	0.25	0.00	0.00	0
14	0.00	0.00	0.00	0
15	0.00	0.00	0.00	0
16	0.00	0.00	0.00	0
17	0.00	0.00	0.00	0
18	0.00	0.00	0.00	0
19	0.00	0.00	0.00	0
20	0.00	0.00	0.00	0

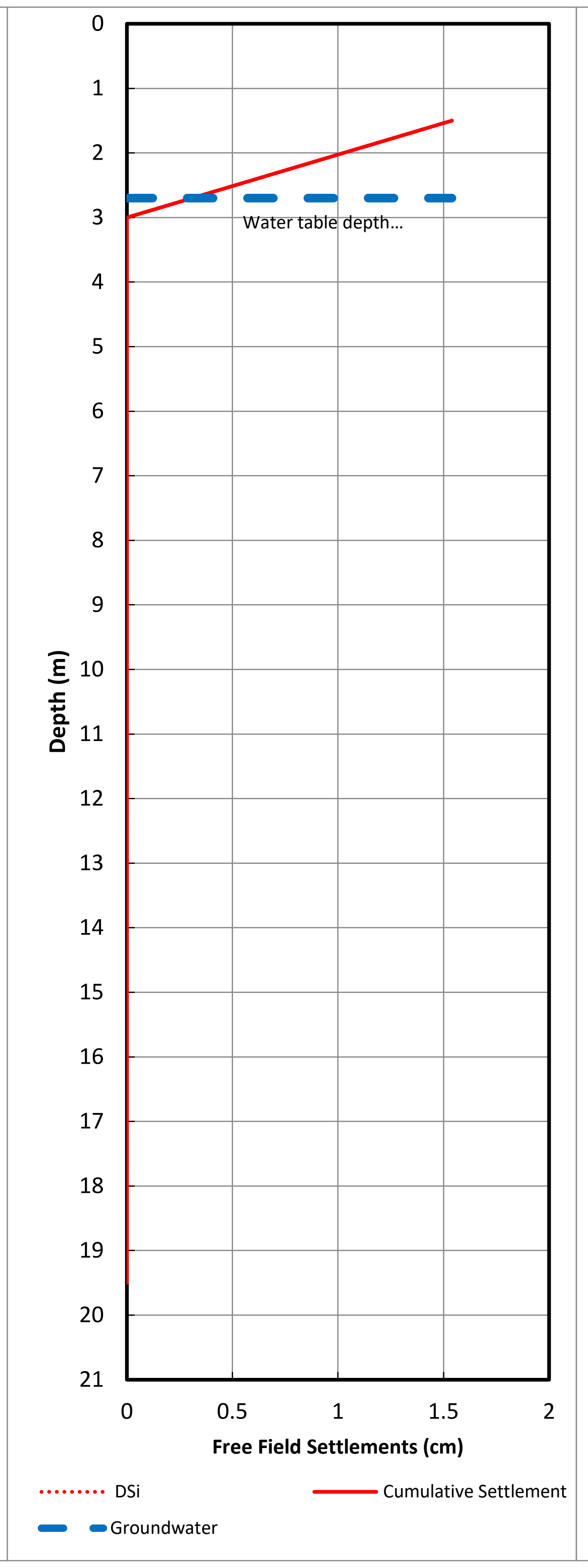


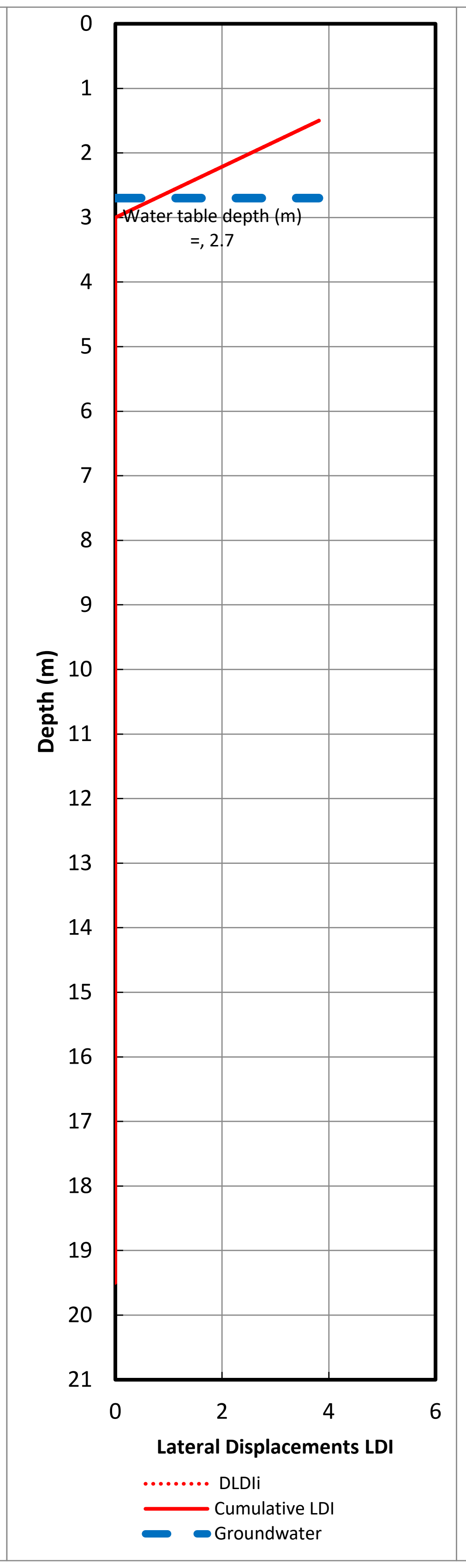


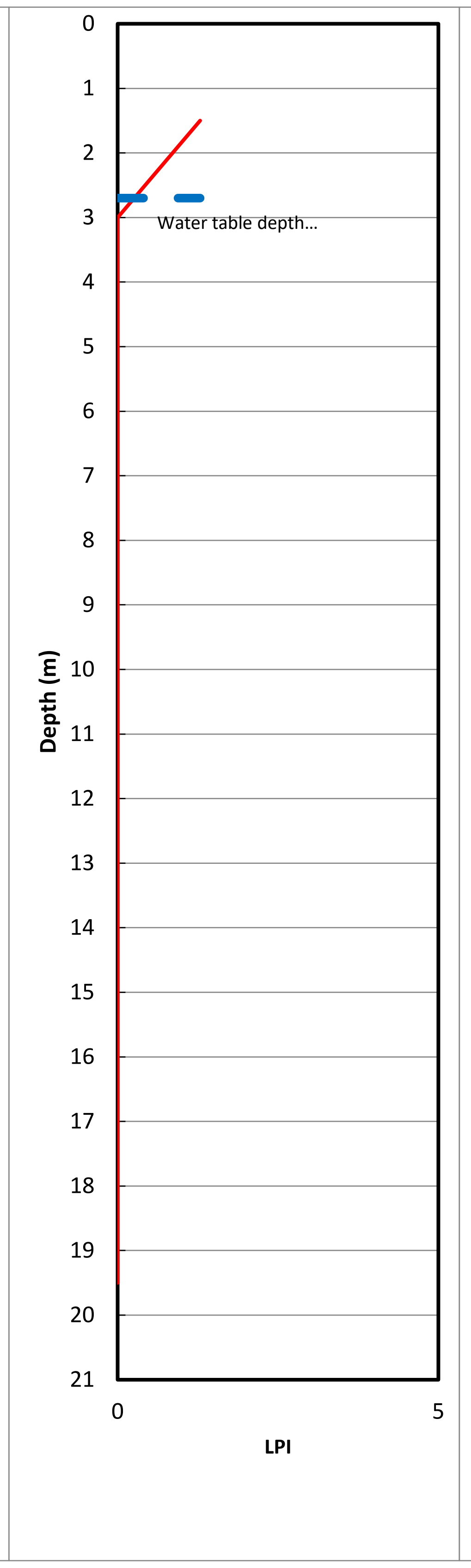


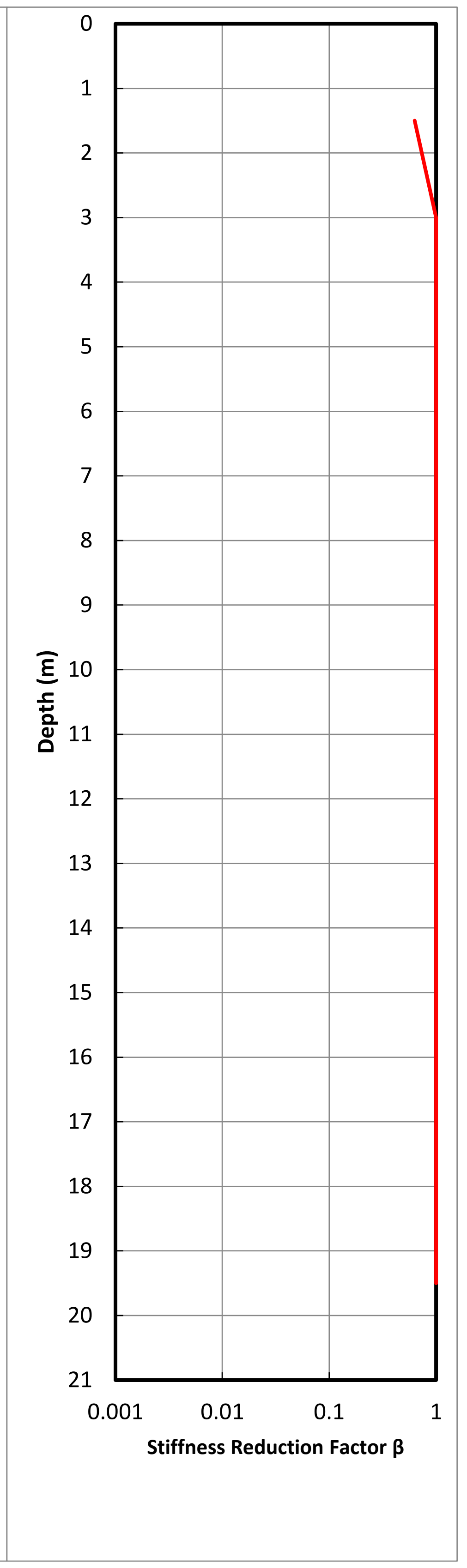












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SPT Based Liquefaction Analysis Calculation Sheet

Input Parameters			
α_{max} (g) =	0.60	Energy ratio	91.8
Earthquake magnitude, M =	6.10	Depth limit	20
Water table depth (m) =	1.50	Machine Hole:	BH 04
Kulhawy & Mayne 1990 (EPRI EL-6800) S_u = N *	6.00	Free Face Depth	1
Change to new values to emulate preloading strength gain		Distance from FF	3
Embankment Surcharge — Embankment Height =	0.00	Slope grade (%)	33.5
Embankment Fill Unit Weight =	18.00	Settlement (cm)	2.55
Borehole diameter (mm) =	65	Lateral D. Index	3.94
Requires correction for sample liners (YES/NO):	NO	LPI	0

Summary Results

Top of ground RL	0
Water RL	-1.50

Information

Based on the Idris and Boulanger spreadsheet for SPT-based liquefaction triggering analysis of a single boring

Original data from Boulanger, Mejia, and Idriss (1997) "Liquefaction at Moss Landing During Loma Prieta Earthquake"

Revision 13

Date 26/11/2019

Lateral Displacement Based on Slope Grade (0.2% - 3.5%)	n/a
Lateral Displacement Based on Free Face Depth and Distance	10 cm
Power m that depends on the sand properties and relative density (Boulanger 2003b)	0.5

Borehole Data

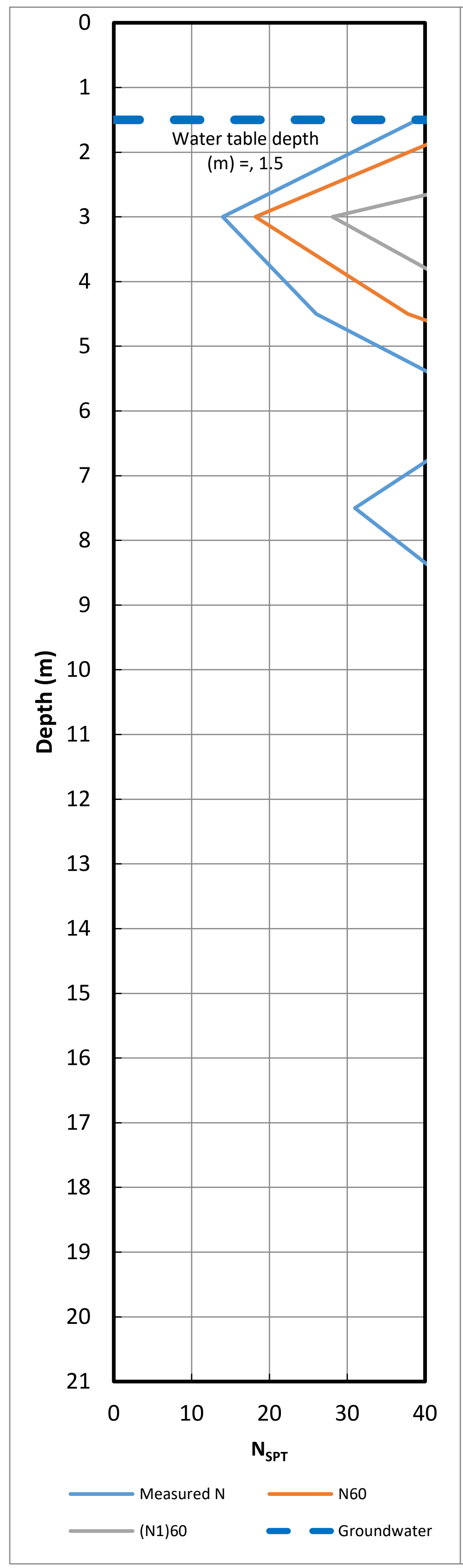
SPT sample number	Depth (m)	Measured N	Unit Weight (kN/m^3) γ	Soil type (USCS)	Top of Layer (m)	Bottom of Layer (m)	Flag "Clay" "Unsaturated" "Unreliable"	Fines content (%)
1	1.50	39	19.0	SM	1.5	2.3	Unreliable	5
2	3.00	14	19.0	SM	2.3	3.8	Unreliable	5
3	4.50	26	19.0	SM	3.8	5.3	Unreliable	5
4	6.00	50	19.0	GW	5.3	6.8	Unreliable	5
5	7.50	31	19.0	GW	6.8	8.3	Unreliable	5
6	9.00	47	19.0	GW	8.3	9.8	Unreliable	5
7	10.50	50	19.0	GW	9.8	11.3	Unreliable	5
8	12.00	50	19.0	GW	11.3	12.8	Unreliable	5
9	13.50	50	19.0	GW	12.8	14.3	Unreliable	5
10	15.00	50	19.0	GW	14.3	15.8	Unreliable	5
11	16.50	50	19.0	GW	15.8	17.3	Unreliable	5
12	18.00	50	19.0	GW	17.3	18.8	Unreliable	5
13	19.50	50	19.0	GW	18.8	20.0	Unreliable	5
14					20.0	20.0		
15					20.0	20.0		
16					20.0	20.0		
17					20.0	20.0		
18					20.0	20.0		
19					20.0	20.0		
20					20.0	20.0		
	20.00							

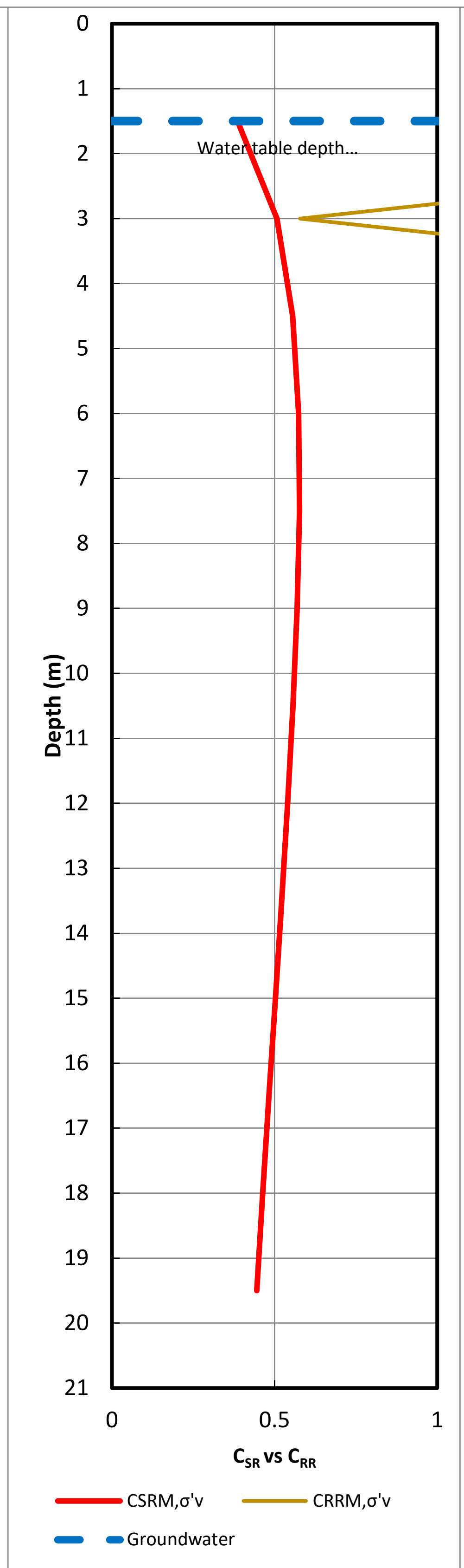
Estimations

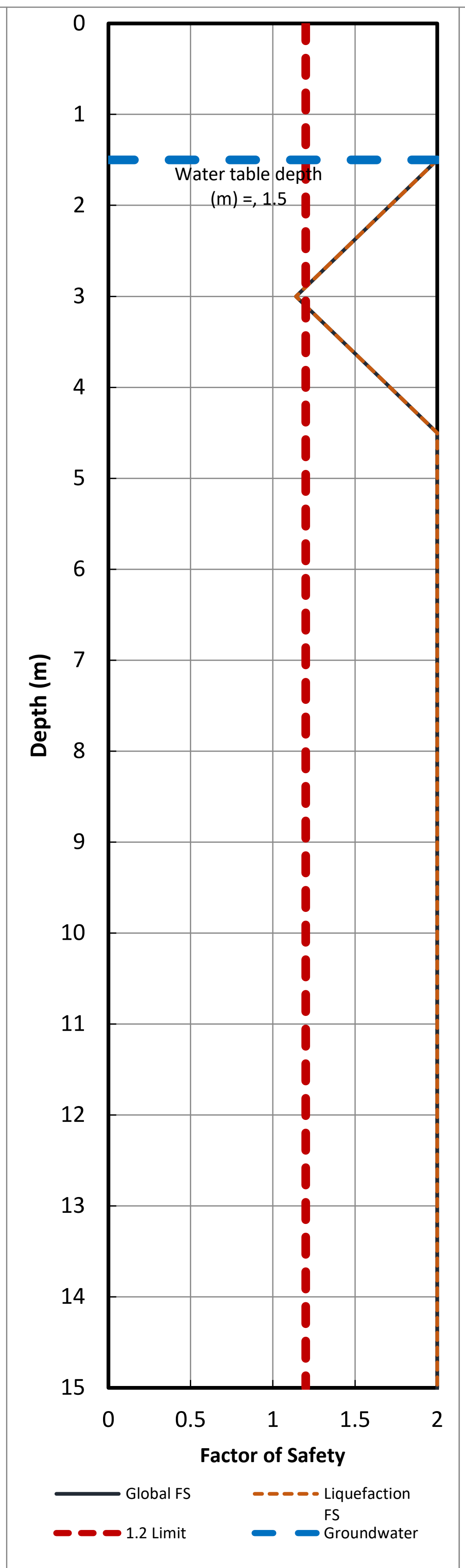
SPT sample number	S_u (kPa)	Stiffness Reduction Factor β	N_{60}	$(N_1)_{60}$	$CSR_{M,\sigma'v}$	$CRR_{M,\sigma'v}$	tanϕ'	S_r/σ'_{v0} Sig Void R	S_r/σ'_{v0} Neg Void R	Global FS	Liquefaction FS
1		1.00	47.7	81.2	0.386	3.333	0.55	0.55	0.55	2.00	2.00
2		1.00	18.2	28.1	0.507	0.578	0.47	0.37	0.47	1.14	1.14
3		1.00	37.8	50.7	0.556	3.333	0.51	0.51	0.51	2.00	2.00
4		1.00	72.7	87.5	0.574	3.333	0.56	0.56	0.56	2.00	2.00
5		1.00	45.1	49.6	0.577	3.333	0.51	0.51	0.51	2.00	2.00
6		1.00	71.9	73.4	0.570	3.333	0.54	0.54	0.54	2.00	2.00
7		1.00	76.5	73.1	0.557	3.228	0.54	0.54	0.54	2.00	2.00
8		1.00	76.5	69.0	0.540	3.113	0.54	0.54	0.54	2.00	2.00
9		1.00	76.5	65.5	0.522	3.010	0.53	0.53	0.53	2.00	2.00
10		1.00	76.5	62.5	0.502	2.917	0.53	0.53	0.53	2.00	2.00
11		1.00	76.5	59.8	0.483	2.832	0.53	0.53	0.53	2.00	2.00
12		1.00	76.5	57.5	0.464	2.754	0.52	0.52	0.52	2.00	2.00
13		1.00	76.5	55.4	0.445	2.681	0.52	0.52	0.52	2.00	2.00
14		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
15		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
16		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
17		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
18		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
19		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
20		1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00

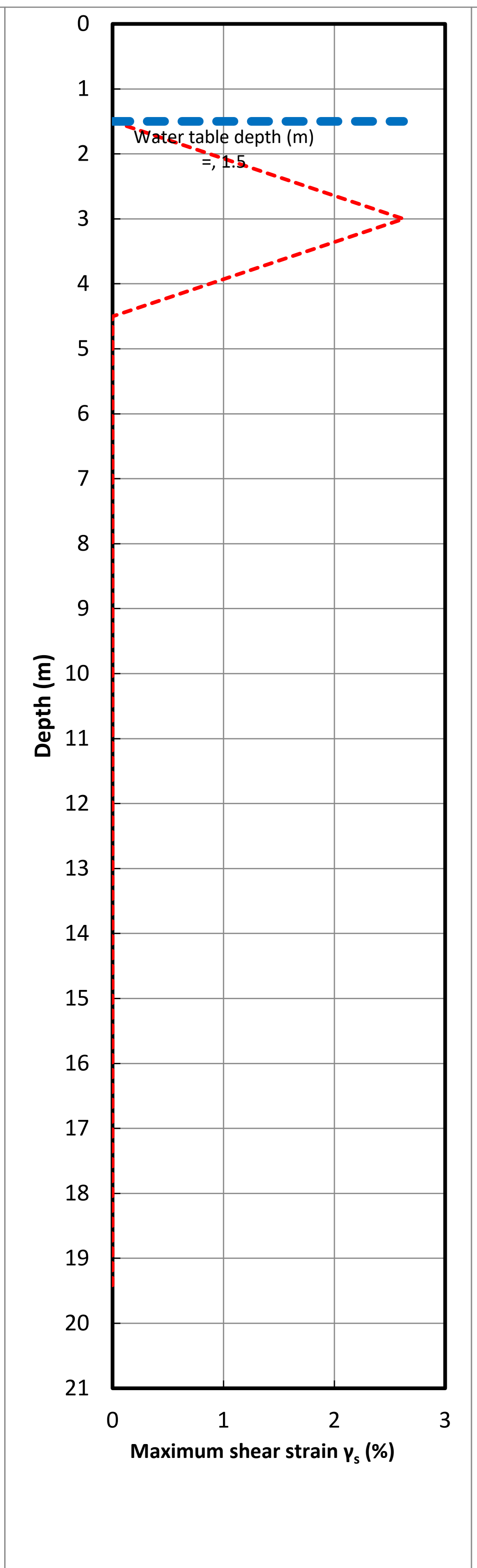
Settlement / **LPI**

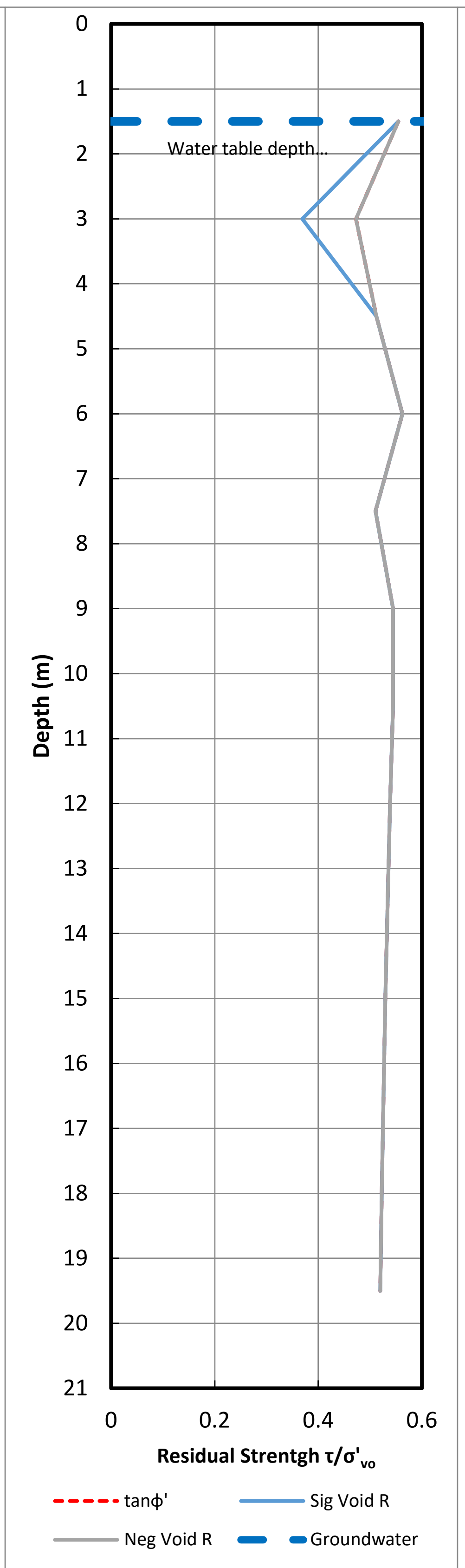
SPT sample number	Limiting shear strain γ_{lim}	Parameter F_a	Max shear strain γ_{max} (%)	DH_i (m)	$DLDI_i$ (cm)	Cumulative LDI (cm)	Vertical reconsol Strain e_V	DS_i (cm)	Cumulative Settlement (cm)	W(z)	F1	F1W(z)	LPI
1	0.000	-4.302	0.000	0.750	0.000	4	0.000	0.000	2.55	9.25	0.00	0.00	0
2	0.060	0.037	2.625	1.500	3.937	4	0.017	2.546	2.55	8.50	0.00	0.00	0
3	0.000	-1.648	0.000	1.500	0.000	0	0.000	0.000	0.00	7.75	0.00	0.00	0
4	0.000	-4.888	0.000	1.500	0.000	0	0.000	0.000	0.00	7.00	0.00	0.00	0
5	0.000	-1.557	0.000	1.500	0.000	0	0.000	0.000	0.00	6.25	0.00	0.00	0
6	0.000	-3.598	0.000	1.500	0.000	0	0.000	0.000	0.00	5.50	0.00	0.00	0
7	0.000	-3.572	0.000	1.500	0.000	0	0.000	0.000	0.00	4.75	0.00	0.00	0
8	0.000	-3.205	0.000	1.500	0.000	0	0.000	0.000	0.00	4.00	0.00	0.00	0
9	0.000	-2.897	0.000	1.500	0.000	0	0.000	0.000	0.00	3.25	0.00	0.00	0
10	0.000	-2.635	0.000	1.500	0.000	0	0.000	0.000	0.00	2.50	0.00	0.00	0
11	0.000	-2.409	0.000	1.500	0.000	0	0.000	0.000	0.00	1.75	0.00	0.00	0
12	0.000	-2.211	0.000	1.500	0.000	0	0.000	0.000	0.00	1.00	0.00	0.00	0
13	0.000	-2.037	0.000	1.250	0.000	0	0.000	0.000	0.00	0.25	0.00	0.00	0
14	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
15	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
16	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
17	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
18	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
19	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
20	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
					LDI =	4		S =	2.55				

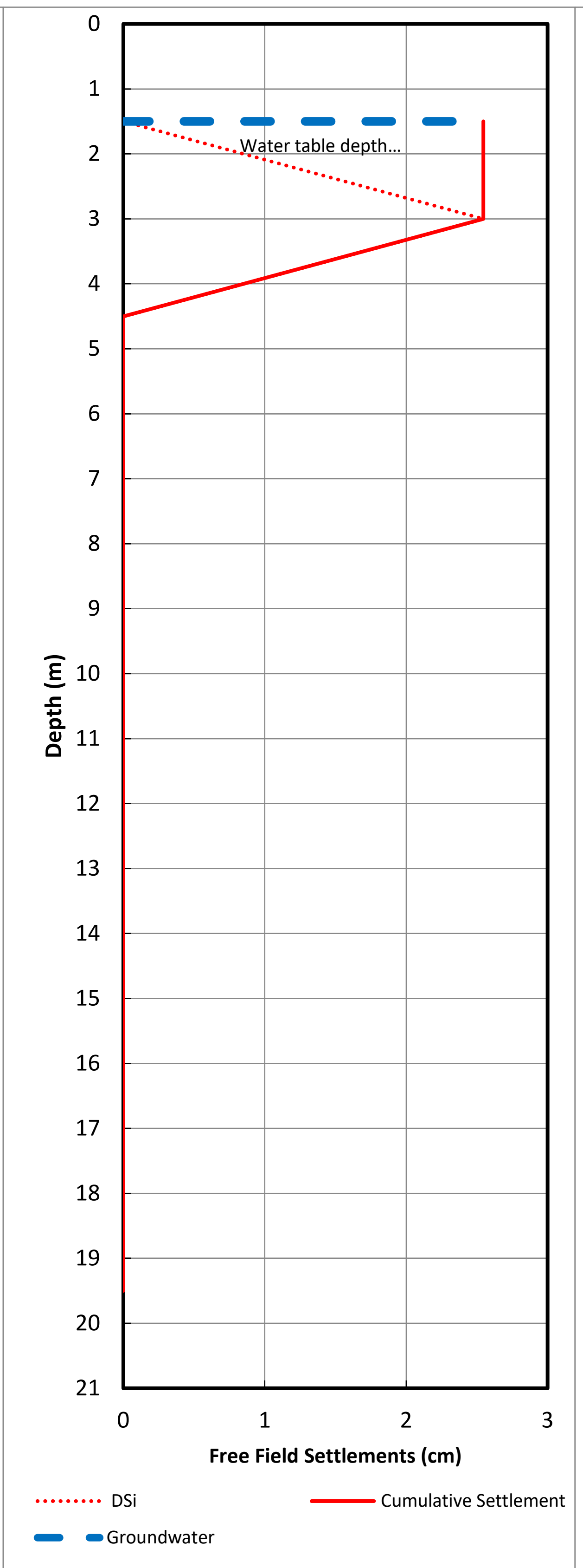


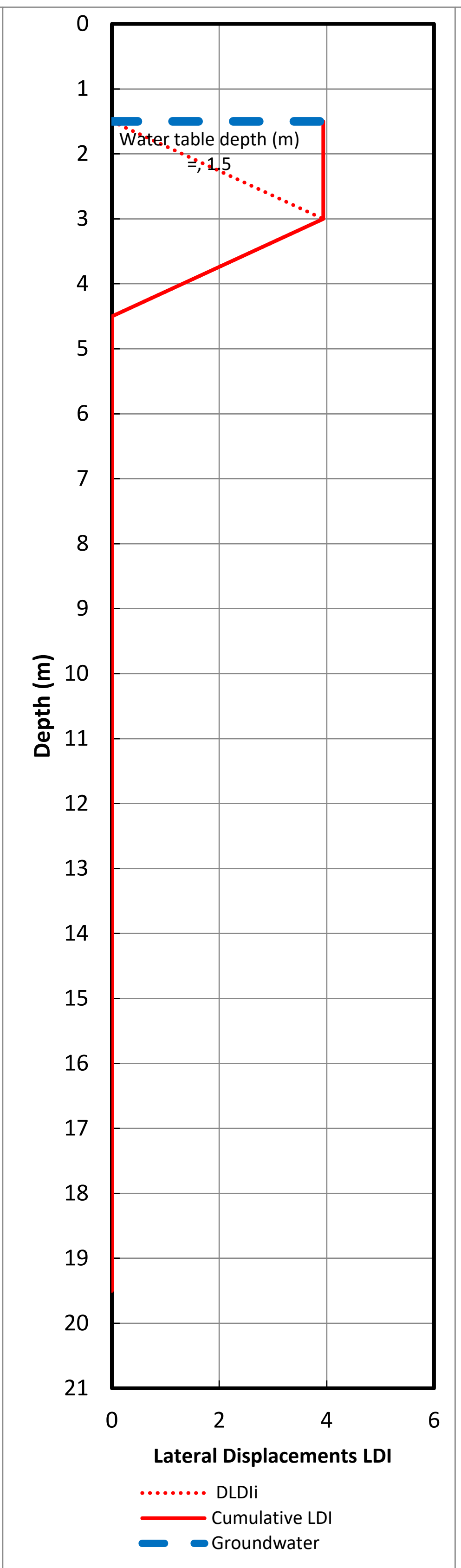


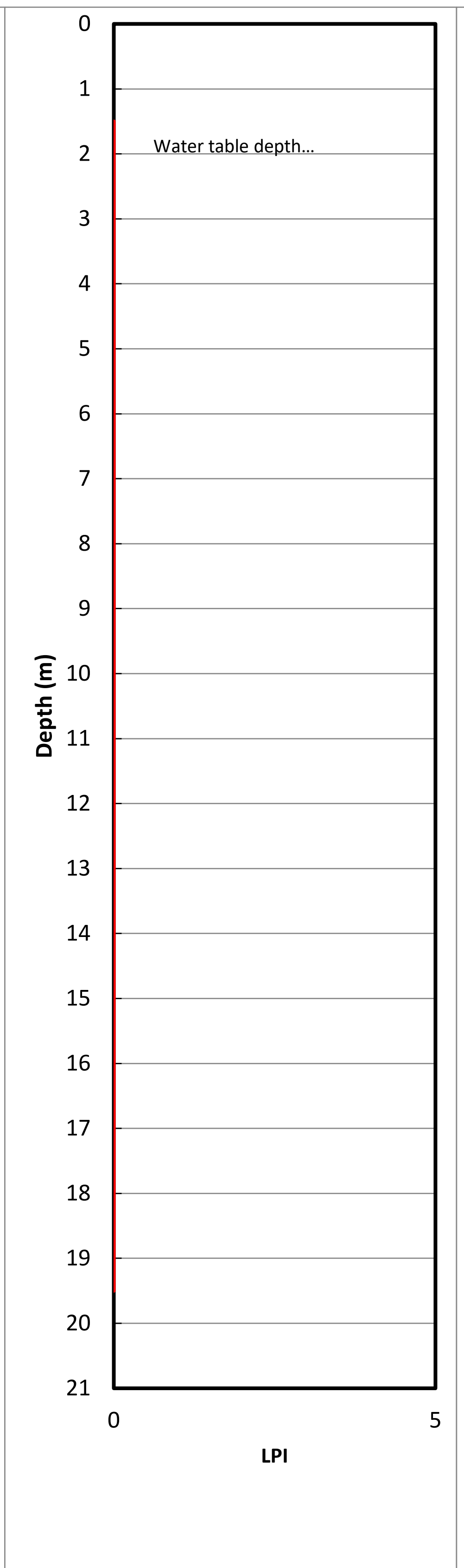


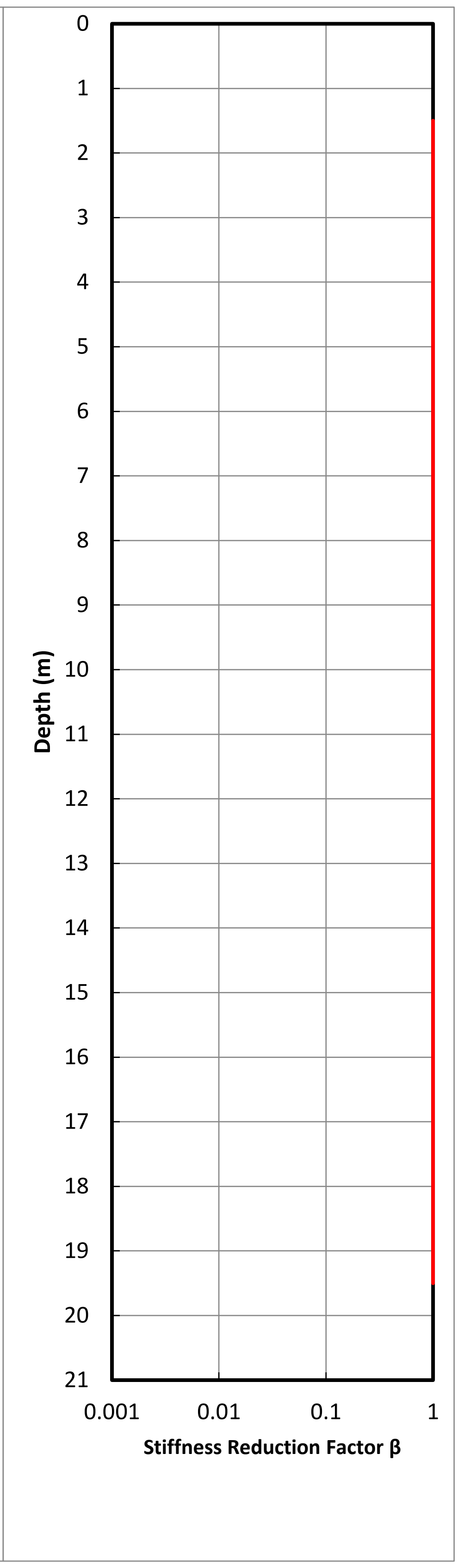












SPT Based Liquefaction Analysis Calculation Sheet

α_{max} (g) =	0.40	**Energy ratio**	**91.8**		
Earthquake magnitude, M =	6.10	**Depth limit**	**20**		
Water table depth (m) =	2.70	**Machine Hole:**	**BH01**	**Water RL**	-2.70
S_u = N *	6.00	**Free Face Depth**	**1**		
		Distance from FF	**3**		
Embankment Height =	0.00	**Slope grade (%)**	**33.5**		
Embankment Fill Unit Weight =	19.00	**Settlement (cm)**	**1.54**		
Borehole diameter (mm) =	65	**Lateral D. Index**	**1.29**		
Requires correction for sample liners (YES/NO):	NO	**LPI**	**0**		

Information

Based on the Idris and Boulanger spreadsheet for SPT-based liquefaction triggering analysis of a single boring

Original data from Boulanger, Mejia, and Idriss (1997) "Liquefaction at Moss Landing During Loma Prieta Earthquake"

Revision 13

Date

Lateral Displacement Based on Slope Grade (0.2% - 3.5%)

Lateral Displacement Based on Free Face Depth and Distance

Power m that depends on the sand properties and relative density (Boulanger 2003b) **0.5**

SPT sample number	Depth (m)	Measured N	Unit Weight (kN/m^3)	Soil type (USCS)	Top of Layer (m)	Bottom of Layer (m)	Flag "Clay" "Unsaturated" "Unreliable"	Fines content (%)	S_u (kPa)	Stiffness Reduction Factor β	N_{60}	$(N_1)_{60}$	$CSR_{M,\sigma'v}$	$CRR_{M,\sigma'v}$	tanϕ'	S_r/σ'_{v0}	S_r/σ'_{v0}	Global FS	Liquefaction FS	Limiting shear strain	Parameter	Max shear strain	DH_i	$DLDI_i$	Cumulative

Stantec

SPT Based Liquefaction Analysis Calculation Sheet

Input Parameters			
α_{max} (g) =	0.40	Energy ratio	91.8
Earthquake magnitude, M =	6.10	Depth limit	20
Water table depth (m) =	1.50	Machine Hole:	BH 04
Kulhawy & Mayne 1990 (EPRI EL-6800) S_u = N *	6.00	Free Face Depth	1
Change to new values to emulate preloading strength gain		Distance from FF	3
Embankment Surcharge: Embankment Height =	0.00	Slope grade (%)	33.5
Embankment Fill Unit Weight =	19.00	Settlement (cm)	2.55
Borehole diameter (mm) =	65	Lateral D. Index	0.88
Requires correction for sample liners (YES/NO):	NO	LPI	0

Summary Results

Top of ground RL	0
Water RL	-1.50

Information

Based on the Idris and Boulanger spreadsheet for SPT-based liquefaction triggering analysis of a single boring

Original data from Boulanger, Mejia, and Idriss (1997) "Liquefaction at Moss Landing During Loma Prieta Earthquake"

Revision 13

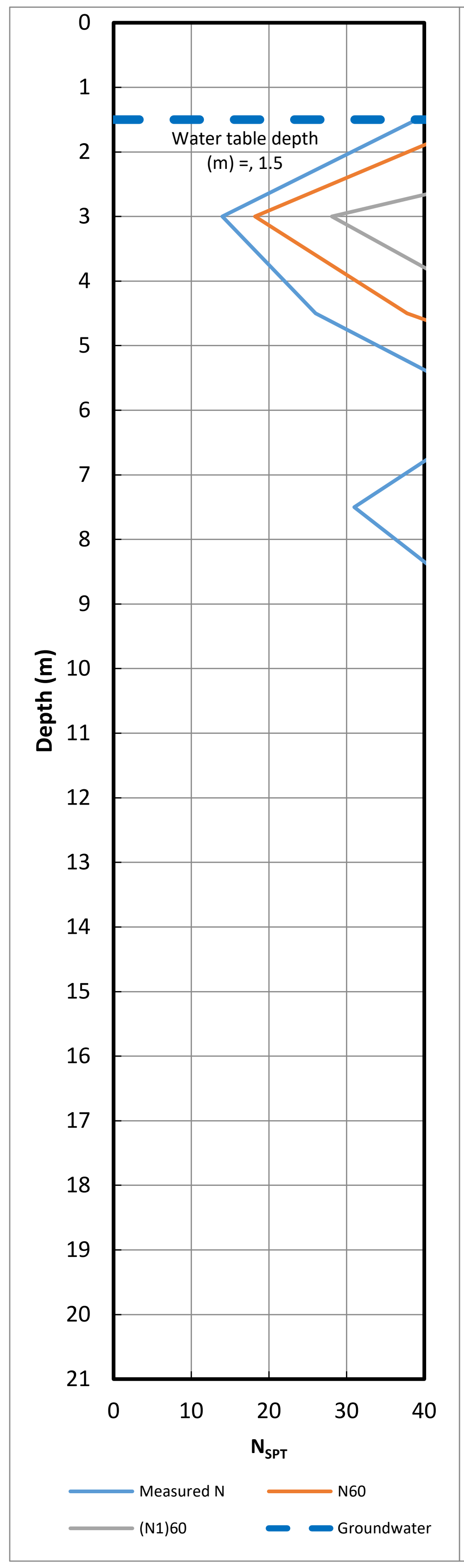
Date 26/11/2019

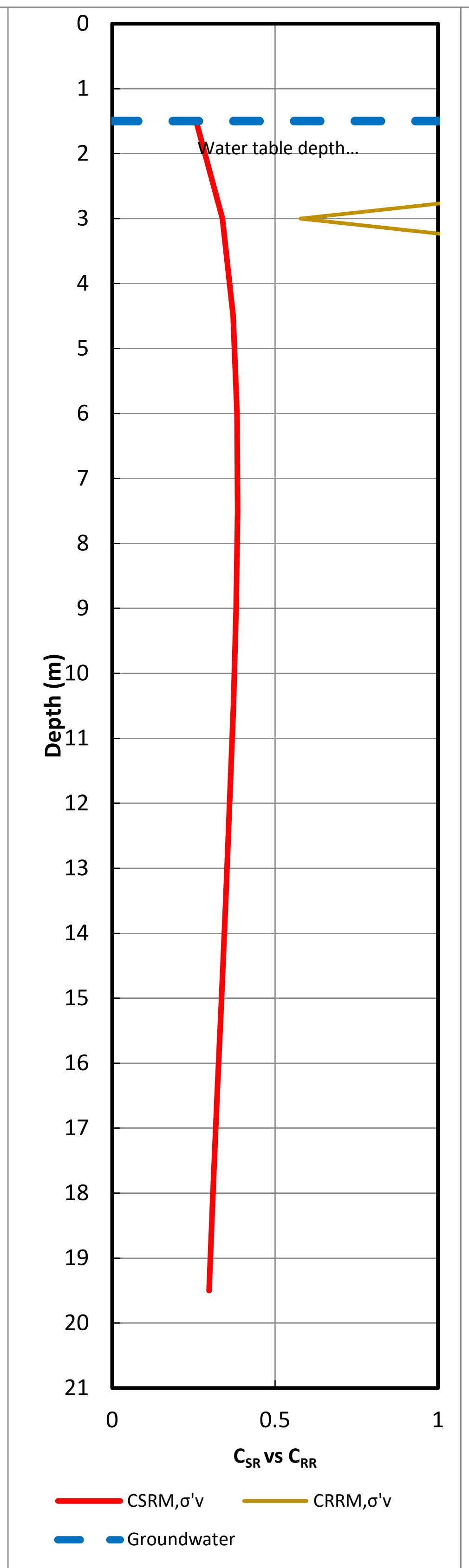
Lateral Displacement Based on Slope Grade (0.2% - 3.5%)	n/a
Lateral Displacement Based on Free Face Depth and Distance	2 cm

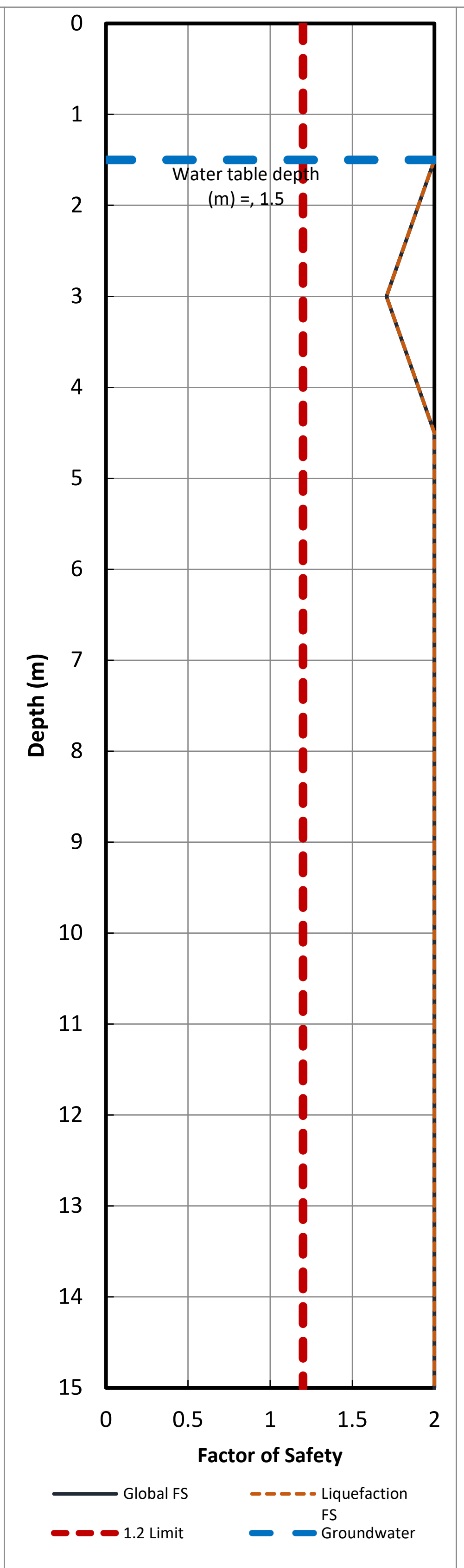
Power m that depends on the sand properties and relative density (Boulanger 2003b)	0.5

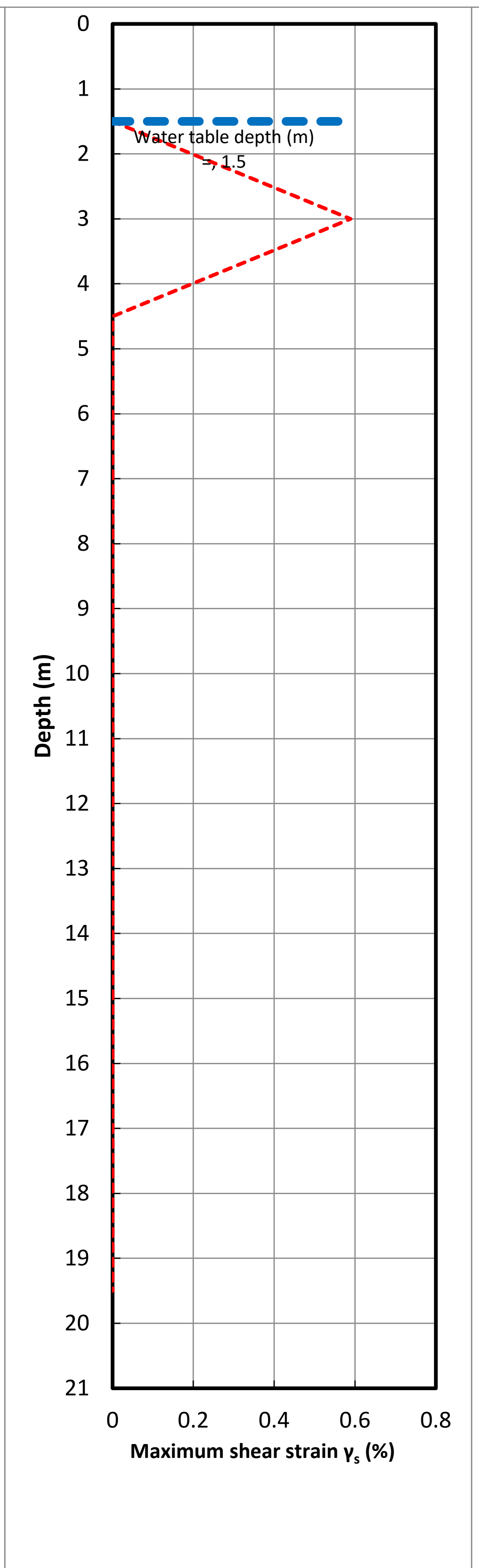
Borehole Data									Estimations										
SPT sample number	Depth (m)	Measured N	Unit Weight (kN/m^3)	Soil type (USCS)	Top of Layer (m)	Bottom of Layer (m)	Flag "Clay" "Unsaturated" "Unreliable"	Fines content (%)	S_u (kPa)	Stiffness Reduction Factor β	N_{60}	$(N_1)_{60}$	$CSR_{M,\sigma'v}$	$CRR_{M,\sigma'v}$	tanϕ'	S_r/σ'_{v0}	S_r/σ'_{v0}	Global FS	Liquefaction FS
			γ													Sig Void R	Neg Void R		
1	1.50	39	19.0	SM	1.5	2.3	Unreliable	5		1.00	47.7	81.2	0.258	3.333	0.55	0.55	0.55	2.00	2.00
2	3.00	14	19.0	SM	2.3	3.8	Unreliable	5		1.00	18.2	28.1	0.338	0.578	0.47	0.37	0.47	1.71	1.71
3	4.50	26	19.0	SM	3.8	5.3	Unreliable	5		1.00	37.8	50.7	0.371	3.333	0.51	0.51	0.51	2.00	2.00
4	6.00	50	19.0	GW	5.3	6.8	Unreliable	5		1.00	72.7	87.5	0.384	3.333	0.56	0.56	0.56	2.00	2.00
5	7.50	31	19.0	GW	6.8	8.3	Unreliable	5		1.00	45.1	49.6	0.385	3.333	0.51	0.51	0.51	2.00	2.00
6	9.00	47	19.0	GW	8.3	9.8	Unreliable	5		1.00	71.9	73.4	0.380	3.333	0.54	0.54	0.54	2.00	2.00
7	10.50	50	19.0	GW	9.8	11.3	Unreliable	5		1.00	76.5	73.1	0.372	3.228	0.54	0.54	0.54	2.00	2.00
8	12.00	50	19.0	GW	11.3	12.8	Unreliable	5		1.00	76.5	69.0	0.361	3.113	0.54	0.54	0.54	2.00	2.00
9	13.50	50	19.0	GW	12.8	14.3	Unreliable	5		1.00	76.5	65.5	0.349	3.010	0.53	0.53	0.53	2.00	2.00
10	15.00	50	19.0	GW	14.3	15.8	Unreliable	5		1.00	76.5	62.5	0.336	2.917	0.53	0.53	0.53	2.00	2.00
11	16.50	50	19.0	GW	15.8	17.3	Unreliable	5		1.00	76.5	59.8	0.322	2.832	0.53	0.53	0.53	2.00	2.00
12	18.00	50	19.0	GW	17.3	18.8	Unreliable	5		1.00	76.5	57.5	0.310	2.754	0.52	0.52	0.52	2.00	2.00
13	19.50	50	19.0	GW	18.8	20.0	Unreliable	5		1.00	76.5	55.4	0.297	2.681	0.52	0.52	0.52	2.00	2.00
14					20.0	20.0				1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
15					20.0	20.0				1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
16					20.0	20.0				1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
17					20.0	20.0				1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
18					20.0	20.0				1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
19					20.0	20.0				1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
20					20.0	20.0				1.00	0.0	0.0	2.000	2.000	0.36	0.36	0.36	2.00	2.00
	20.00																		

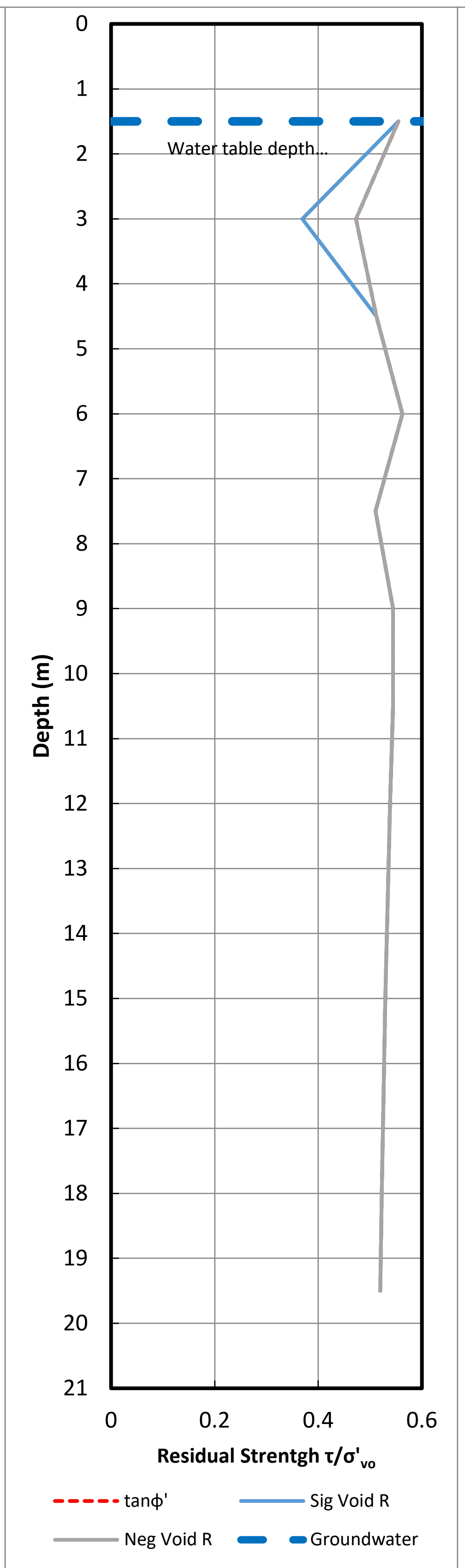
	Settlement										LPI			
SPT sample number	Limiting shear strain	Parameter	Max shear strain	DH_i	$DLDI_i$	Cumulative LDI	Vertical reconsol Strain	DS_i	Cumulative Settlement	W(z)	F1	F1W(z)	LPI
	γ_{lim}	F_a	γ_{max} (%)	(m)	(cm)	(cm)	e_V	(cm)	(cm)				
1	0.000	-4.302	0.000	0.750	0.000	1	0.000	0.000	2.55	9.25	0.00	0.00	0
2	0.060	0.037	0.589	1.500	0.883	1	0.017	2.546	2.55	8.50	0.00	0.00	0
3	0.000	-1.648	0.000	1.500	0.000	0	0.000	0.000	0.00	7.75	0.00	0.00	0
4	0.000	-4.888	0.000	1.500	0.000	0	0.000	0.000	0.00	7.00	0.00	0.00	0
5	0.000	-1.557	0.000	1.500	0.000	0	0.000	0.000	0.00	6.25	0.00	0.00	0
6	0.000	-3.598	0.000	1.500	0.000	0	0.000	0.000	0.00	5.50	0.00	0.00	0
7	0.000	-3.572	0.000	1.500	0.000	0	0.000	0.000	0.00	4.75	0.00	0.00	0
8	0.000	-3.205	0.000	1.500	0.000	0	0.000	0.000	0.00	4.00	0.00	0.00	0
9	0.000	-2.897	0.000	1.500	0.000	0	0.000	0.000	0.00	3.25	0.00	0.00	0
10	0.000	-2.635	0.000	1.500	0.000	0	0.000	0.000	0.00	2.50	0.00	0.00	0
11	0.000	-2.409	0.000	1.500	0.000	0	0.000	0.000	0.00	1.75	0.00	0.00	0
12	0.000	-2.211	0.000	1.500	0.000	0	0.000	0.000	0.00	1.00	0.00	0.00	0
13	0.000	-2.037	0.000	1.250	0.000	0	0.000	0.000	0.00	0.25	0.00	0.00	0
14	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
15	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
16	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
17	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
18	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
19	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
20	0.500	0.948	0.000	0.000	0.000	0	0.000	0.000	0.00	0.00	0.00	0.00	0
					LDI =	1		S =	2.55				

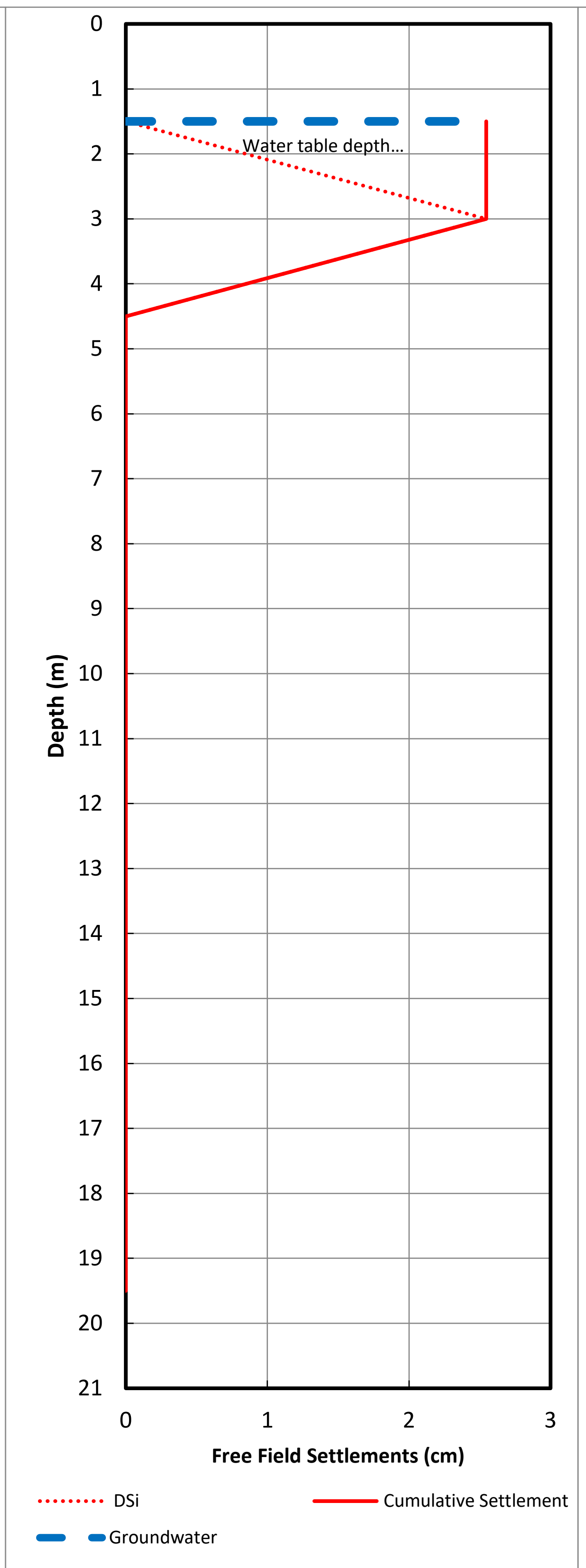


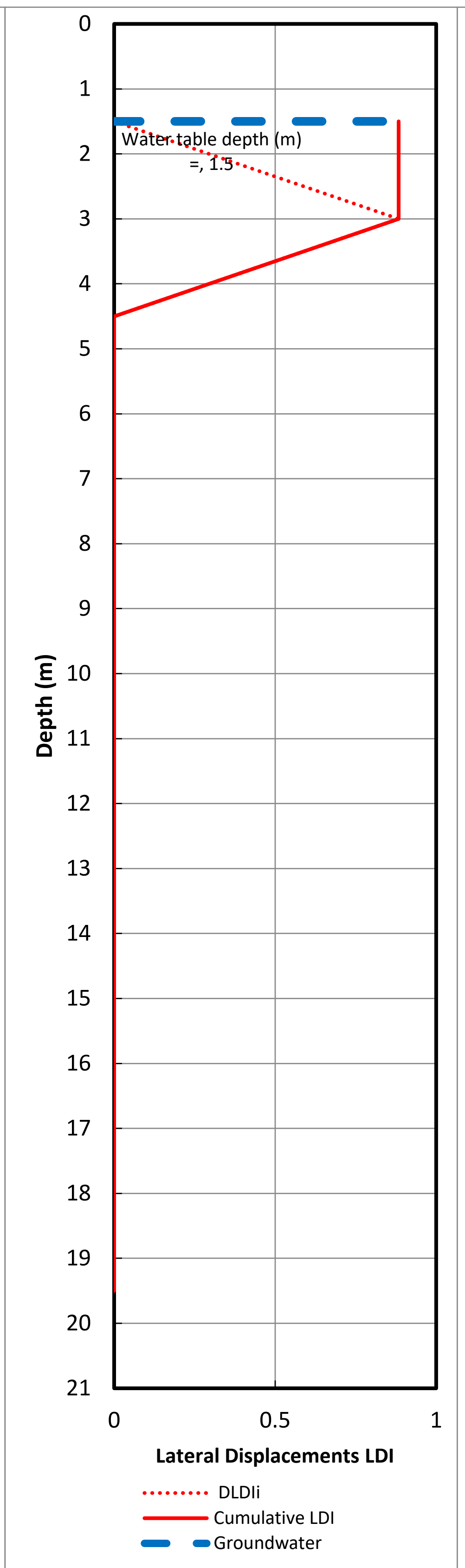


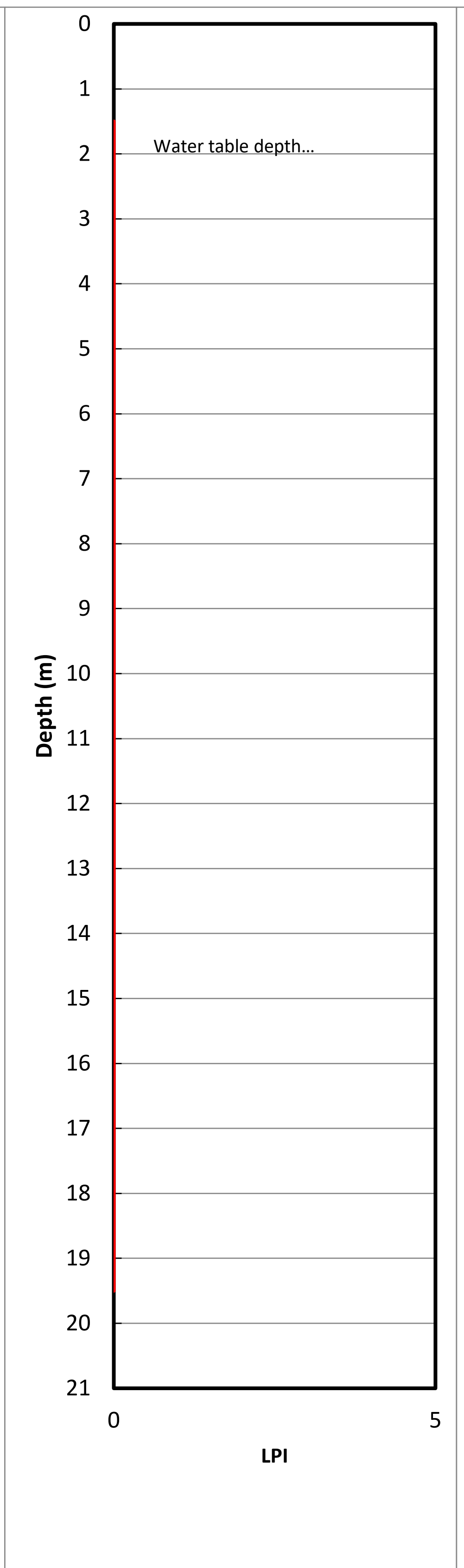


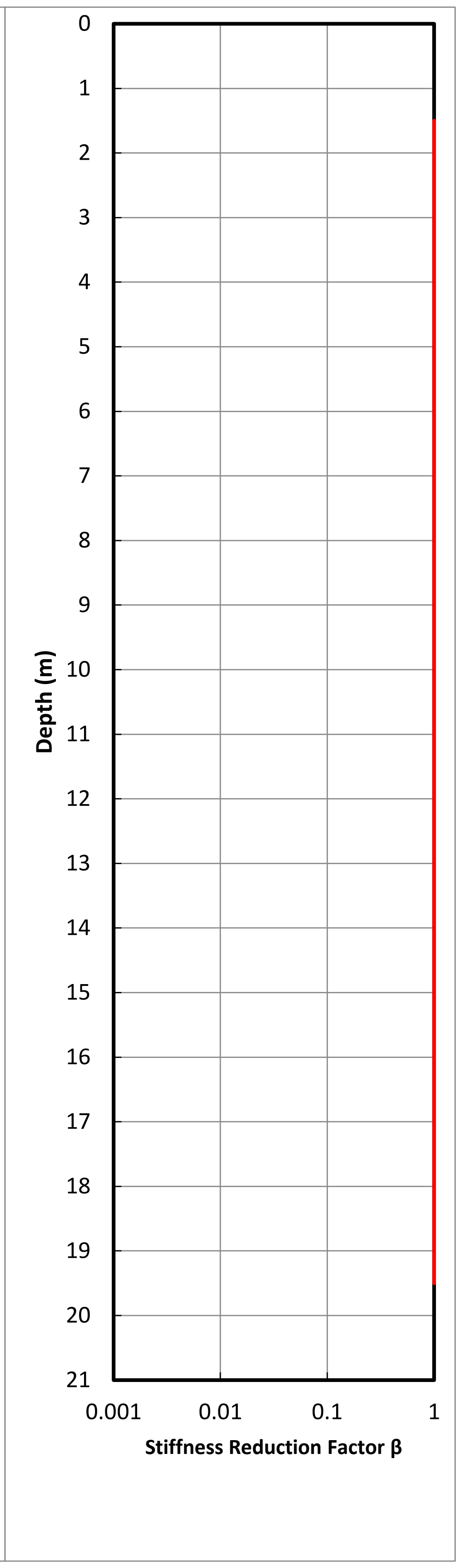








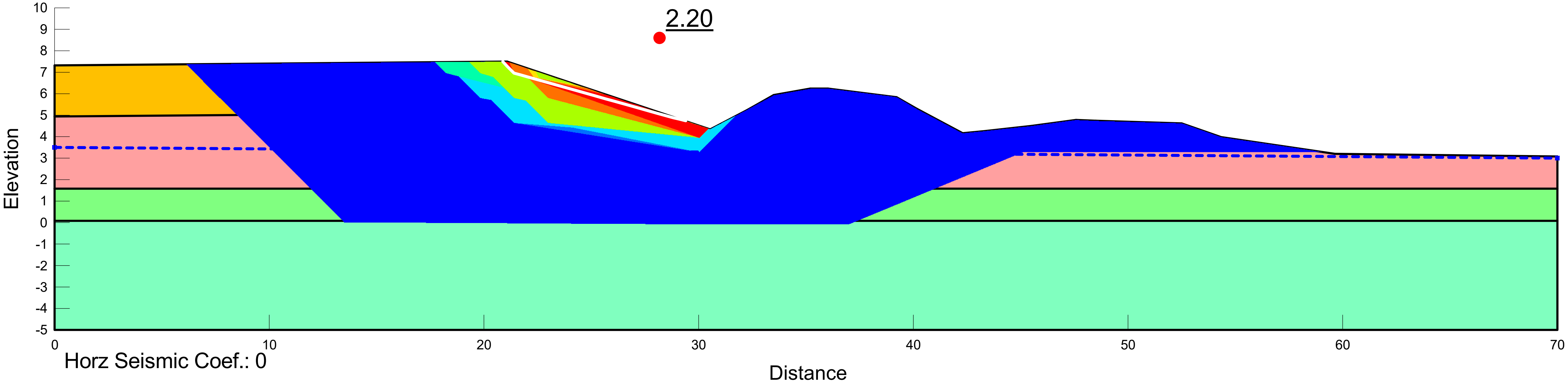




Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟥	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

🟥	2.20 - 2.30
🟧	2.30 - 2.40
🟨	2.40 - 2.50
🟩	2.50 - 2.60
🟩	2.60 - 2.70
🟩	2.70 - 2.80
🟩	2.80 - 2.90
🟦	2.90 - 3.00
🟦	3.00 - 3.10
🟦	≥ 3.10

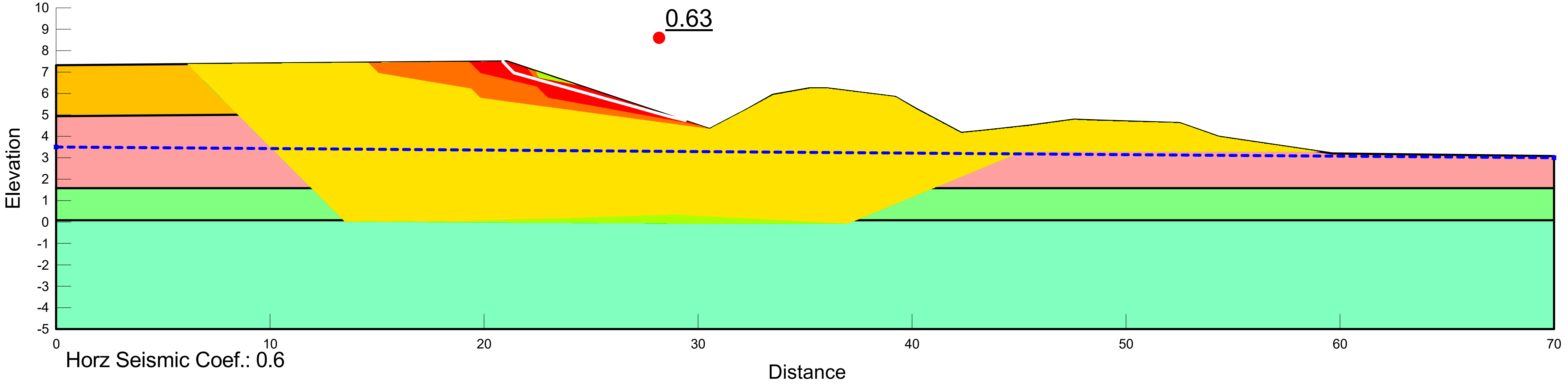


01_Northern abutment static
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

🟥	0.63 - 0.73
🟧	0.73 - 0.83
🟨	0.83 - 0.93
🟩	0.93 - 1.03
🟩	1.03 - 1.13
🟩	1.13 - 1.23
🟩	1.23 - 1.33
🟦	1.33 - 1.43
🟦	1.43 - 1.53
🟦	≥ 1.53



Horz Seismic Coef.: 0.6

02_Northern abutment PS CALS 0.6g

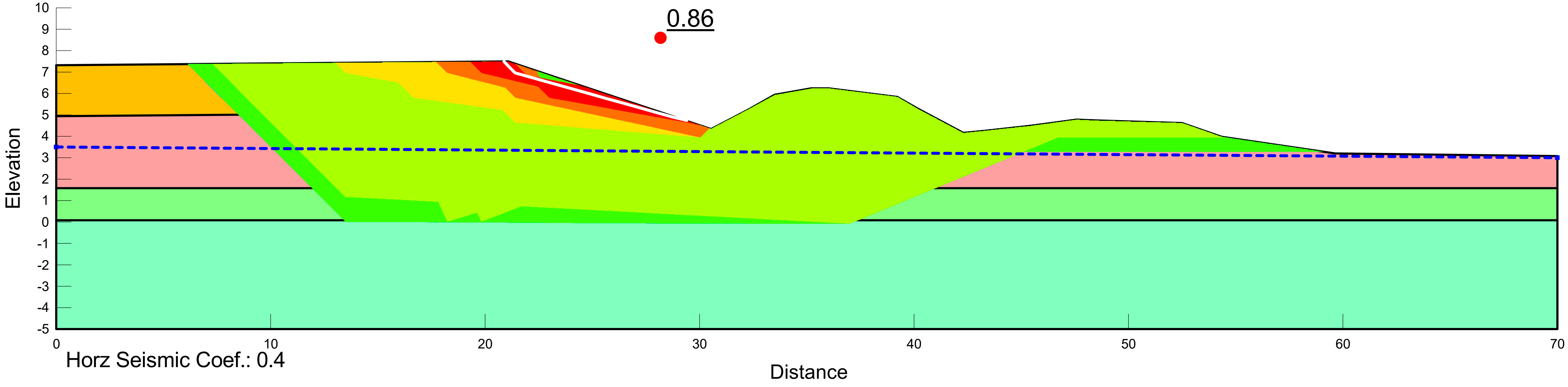
Preliminary Embankment Design_rev2.gsz

22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

🟥	0.86 - 0.96
🟧	0.96 - 1.06
🟨	1.06 - 1.16
🟩	1.16 - 1.26
🟩	1.26 - 1.36
🟩	1.36 - 1.46
🟩	1.46 - 1.56
🟦	1.56 - 1.66
🟦	1.66 - 1.76
🟦	≥ 1.76



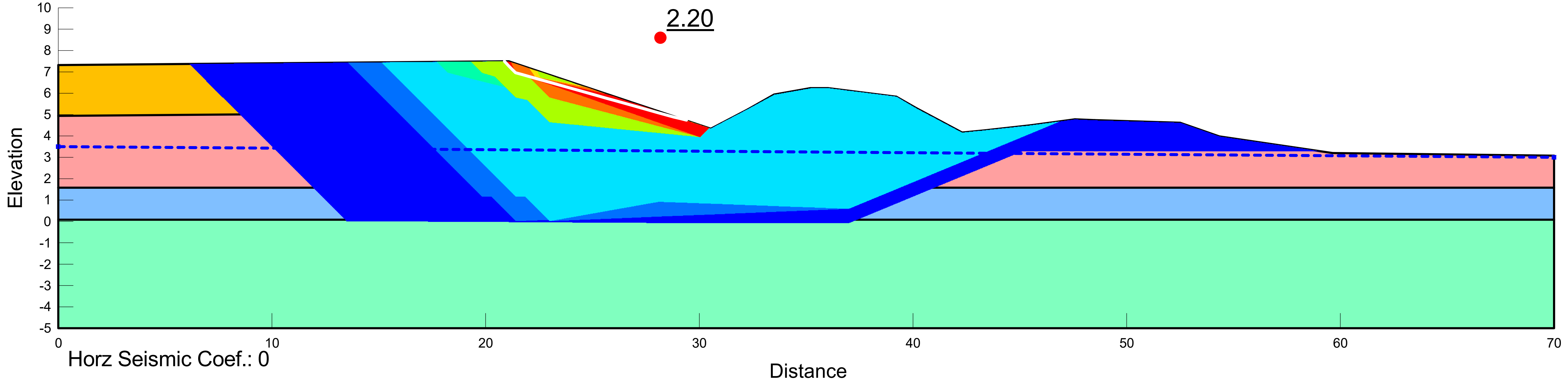
Horz Seismic Coef.: 0.4

03_Northern abutment PS DCLS 0.4g
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezor Line
🟧	Emabnkment	Mohr-Coulomb	20			0	35	0	1
🟥	Stopbank	Mohr-Coulomb	18			2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19			0	33	0	1
🟦	Unit 2 - Liquefied	SHANSEP	8	3	0.2				1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20			0	34	0	1

Factor of Safety

🟥	2.20 - 2.30
🟧	2.30 - 2.40
🟨	2.40 - 2.50
🟩	2.50 - 2.60
🟩	2.60 - 2.70
🟩	2.70 - 2.80
🟩	2.80 - 2.90
🟦	2.90 - 3.00
🟦	3.00 - 3.10
🟦	≥ 3.10



04_Northern abutment Flow failure

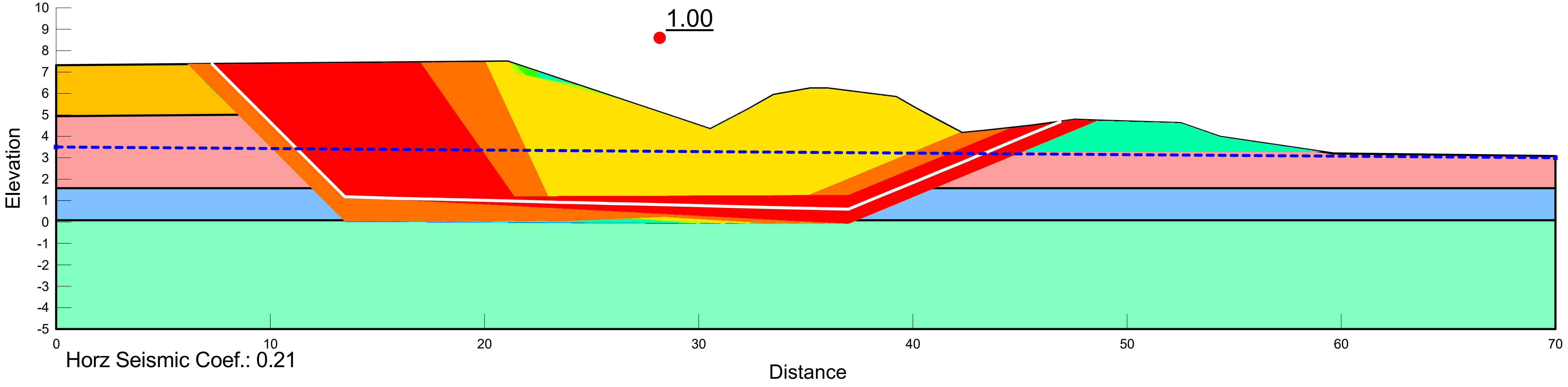
Preliminary Embankment Design_rev2.gsz

22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezor Line
🟧	Emabnkment	Mohr-Coulomb	20			0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18			2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19			0	33	0	1
🟦	Unit 2 - Liquefied	SHANSEP	8	3	0.2				1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20			0	34	0	1

Factor of Safety

🟥	1.00 - 1.10
🟧	1.10 - 1.20
🟨	1.20 - 1.30
🟩	1.30 - 1.40
🟩	1.40 - 1.50
🟩	1.50 - 1.60
🟩	1.60 - 1.70
🟦	1.70 - 1.80
🟦	1.80 - 1.90
🟦	≥ 1.90

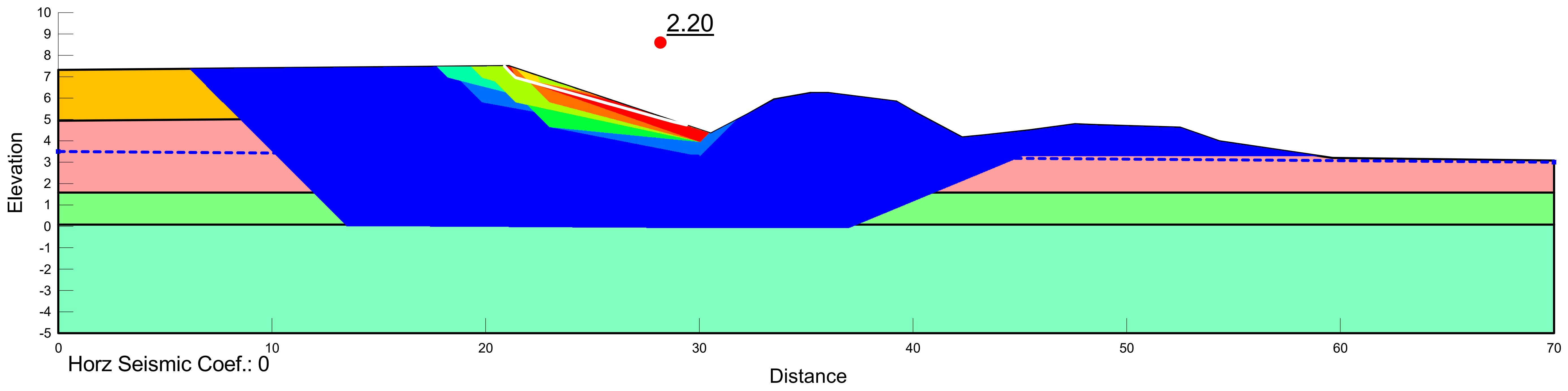


05_Northern abutment Yield Acc
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20	0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

🟥	2.20 - 2.30
🟧	2.30 - 2.40
🟨	2.40 - 2.50
🟩	2.50 - 2.60
🟩	2.60 - 2.70
🟩	2.70 - 2.80
🟩	2.80 - 2.90
🟦	2.90 - 3.00
🟦	3.00 - 3.10
🟦	≥ 3.10

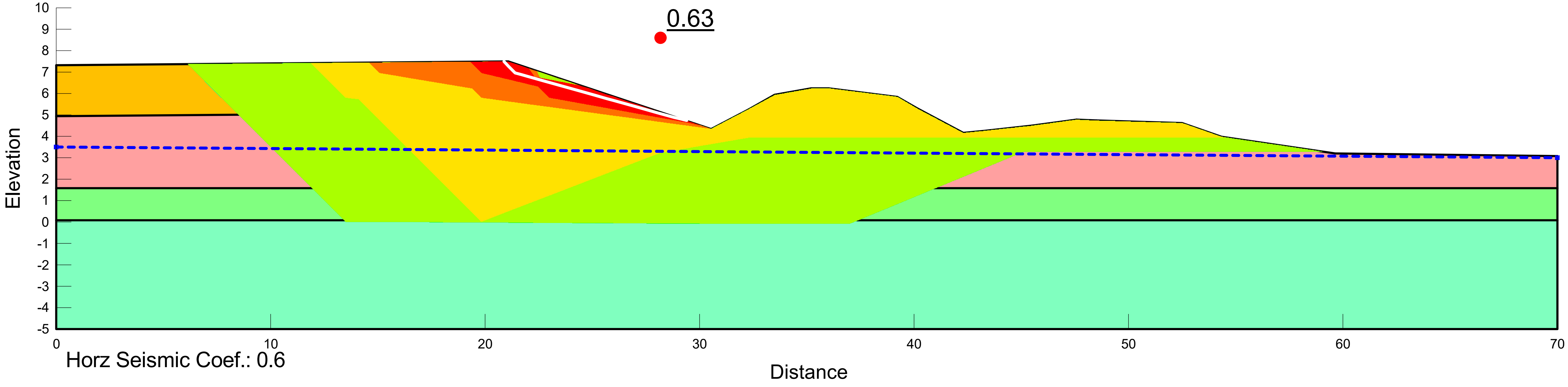


06_Northern abutment static	
Preliminary Embankment Design_rev2.gsz	
22/06/2022	1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20	0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

🟥	0.63 - 0.73
🟧	0.73 - 0.83
🟨	0.83 - 0.93
🟩	0.93 - 1.03
🟩	1.03 - 1.13
🟩	1.13 - 1.23
🟩	1.23 - 1.33
🟦	1.33 - 1.43
🟦	1.43 - 1.53
🟦	≥ 1.53

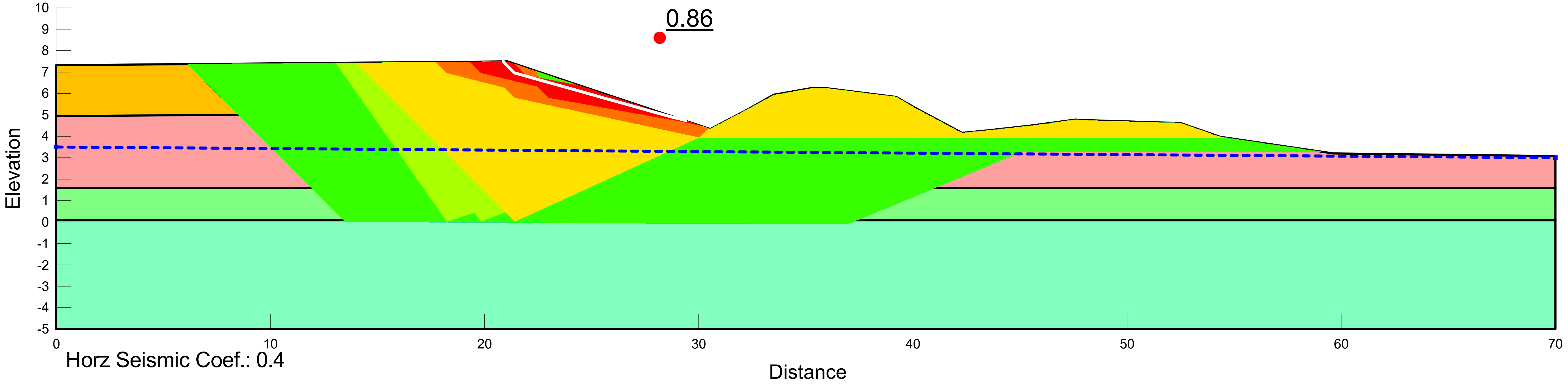


07_Northern abutment PS CALS 0.6g
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20	0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

🟥	0.86 - 0.96
🟧	0.96 - 1.06
🟨	1.06 - 1.16
🟩	1.16 - 1.26
🟩	1.26 - 1.36
🟩	1.36 - 1.46
🟩	1.46 - 1.56
🟦	1.56 - 1.66
🟦	1.66 - 1.76
🟦	≥ 1.76

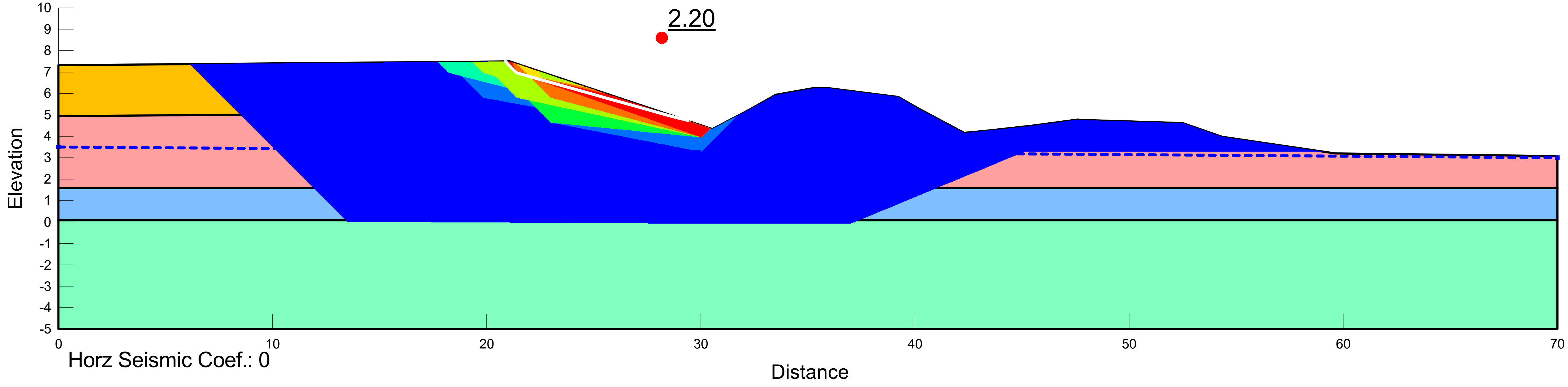


08_Northern abutment PS DCLS 0.4g
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezom Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20			0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20			0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18			2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19			0	33	0	1
🟦	Unit 2 - Liquefied	SHANSEP	8	3	0.2				1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20			0	34	0	1

Factor of Safety

- 🟥 2.20 - 2.30
- 🟧 2.30 - 2.40
- 🟨 2.40 - 2.50
- 🟩 2.50 - 2.60
- 🟩 2.60 - 2.70
- 🟩 2.70 - 2.80
- 🟩 2.80 - 2.90
- 🟦 2.90 - 3.00
- 🟦 3.00 - 3.10
- 🟦 ≥ 3.10

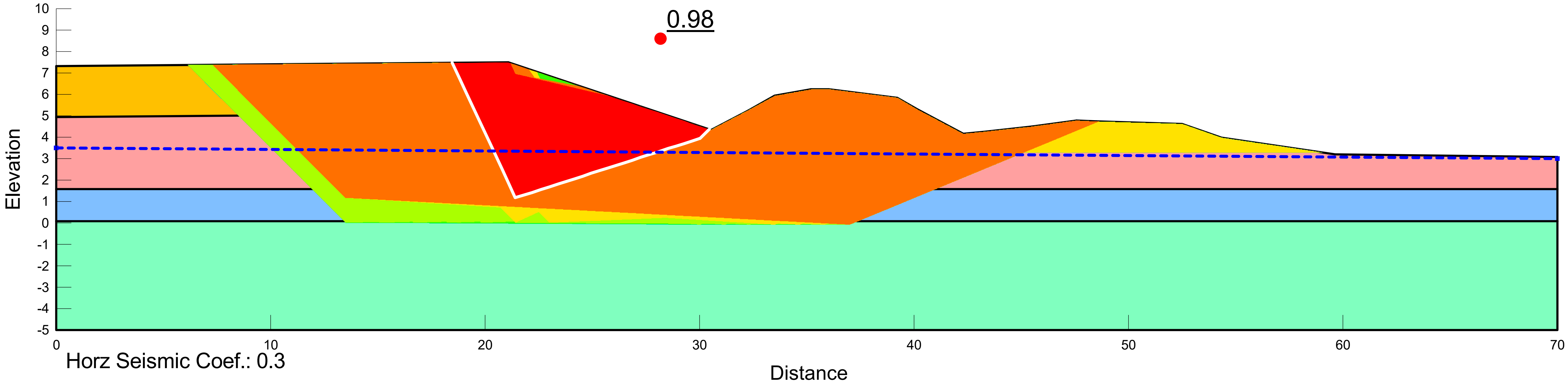


09_Northern abutment Flow failure	
Preliminary Embankment Design_rev2.gsz	
22/06/2022	1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20			0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20			0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18			2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19			0	33	0	1
🟦	Unit 2 - Liquefied	SHANSEP	8	3	0.2				1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20			0	34	0	1

Factor of Safety

🟥	0.98 - 1.08
🟧	1.08 - 1.18
🟨	1.18 - 1.28
🟩	1.28 - 1.38
🟩	1.38 - 1.48
🟩	1.48 - 1.58
🟩	1.58 - 1.68
🟦	1.68 - 1.78
🟦	1.78 - 1.88
🟦	≥ 1.88



Horz Seismic Coef.: 0.3

10_Northern abutment Yield Acc

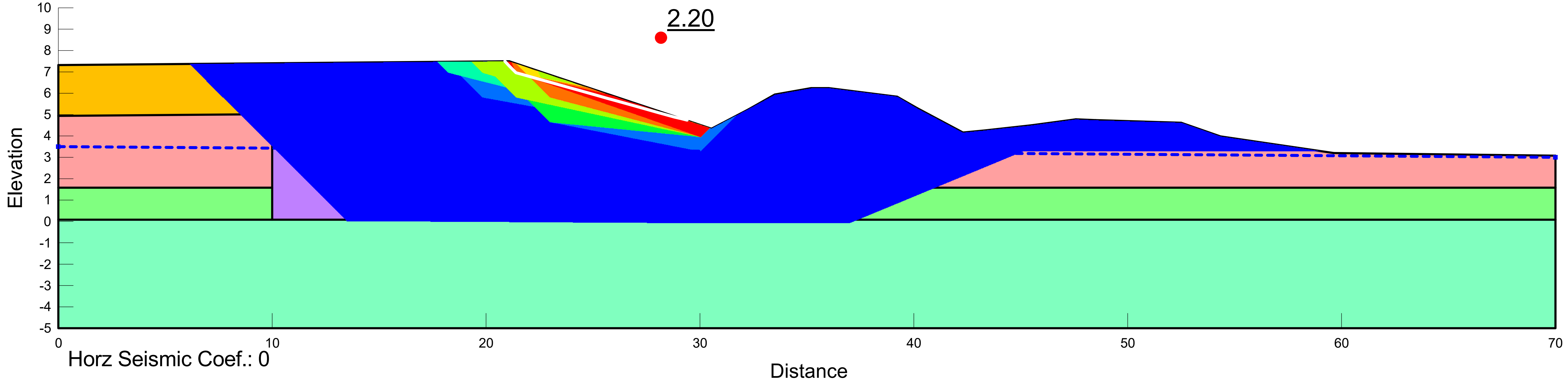
Preliminary Embankment Design_rev2.gsz

22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20	0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

- 🟥 2.20 - 2.30
- 🟧 2.30 - 2.40
- 🟨 2.40 - 2.50
- 🟩 2.50 - 2.60
- 🟩 2.60 - 2.70
- 🟩 2.70 - 2.80
- 🟩 2.80 - 2.90
- 🟦 2.90 - 3.00
- 🟦 3.00 - 3.10
- 🟦 ≥ 3.10



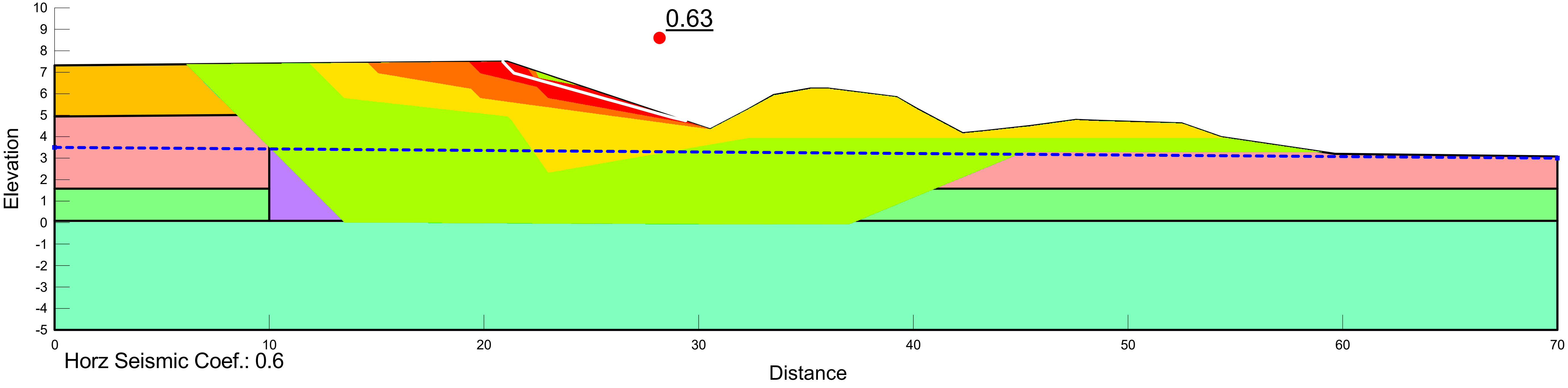
Horz Seismic Coef.: 0

11_Northern abutment static
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	**Name**	**Material Model**	**Unit Weight (kN/m³)**	**Effective Cohesion (kPa)**	**Effective Friction Angle (°)**	**Phi-B (°)**	**Piezometric Line**
🟪	Compacted GRAVEL	Mohr-Coulomb	20	0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

🟥 0.63 - 0.73
🟧 0.73 - 0.83
🟨 0.83 - 0.93
🟩 0.93 - 1.03
🟩 1.03 - 1.13
🟩 1.13 - 1.23
🟩 1.23 - 1.33
🟦 1.33 - 1.43
🟦 1.43 - 1.53
🟦 ≥ 1.53

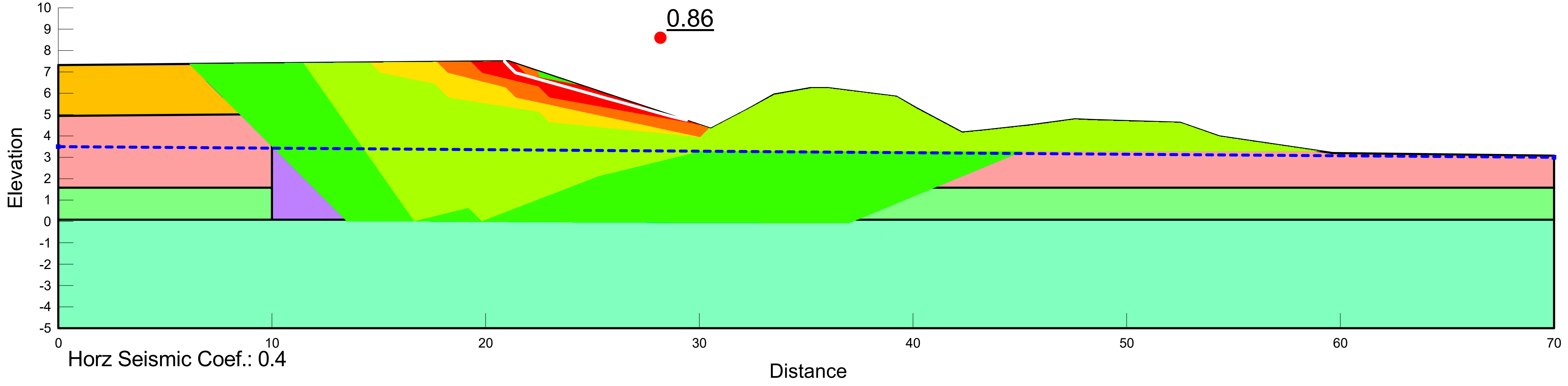


12_Northern abutment PS CALS 0.6g
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20	0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20	0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18	2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19	0	33	0	1
🟩	Unit 2 - Silty SAND	Mohr-Coulomb	18	0	32	0	1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20	0	34	0	1

Factor of Safety

🟥 0.86 - 0.96
🟧 0.96 - 1.06
🟨 1.06 - 1.16
🟩 1.16 - 1.26
🟩 1.26 - 1.36
🟩 1.36 - 1.46
🟩 1.46 - 1.56
🟦 1.56 - 1.66
🟦 1.66 - 1.76
🟦 ≥ 1.76

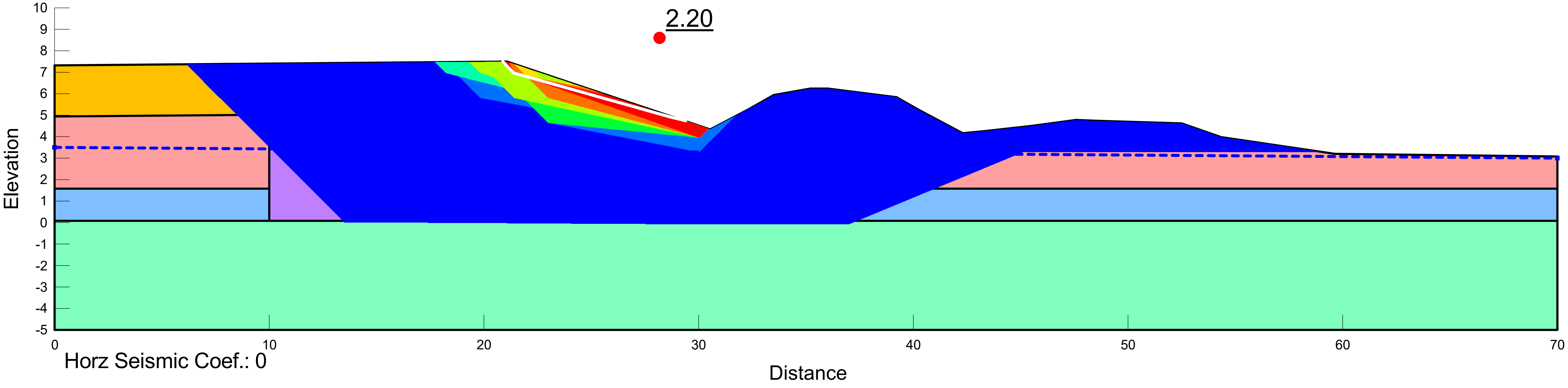


13_Northern abutment PS DCLS 0.4g
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezom Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20			0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20			0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18			2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19			0	33	0	1
🟦	Unit 2 - Liquefied	SHANSEP	8	3	0.2				1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20			0	34	0	1

Factor of Safety

🟥	2.20 - 2.30
🟧	2.30 - 2.40
🟨	2.40 - 2.50
🟩	2.50 - 2.60
🟩	2.60 - 2.70
🟩	2.70 - 2.80
🟩	2.80 - 2.90
🟦	2.90 - 3.00
🟦	3.00 - 3.10
🟦	≥ 3.10

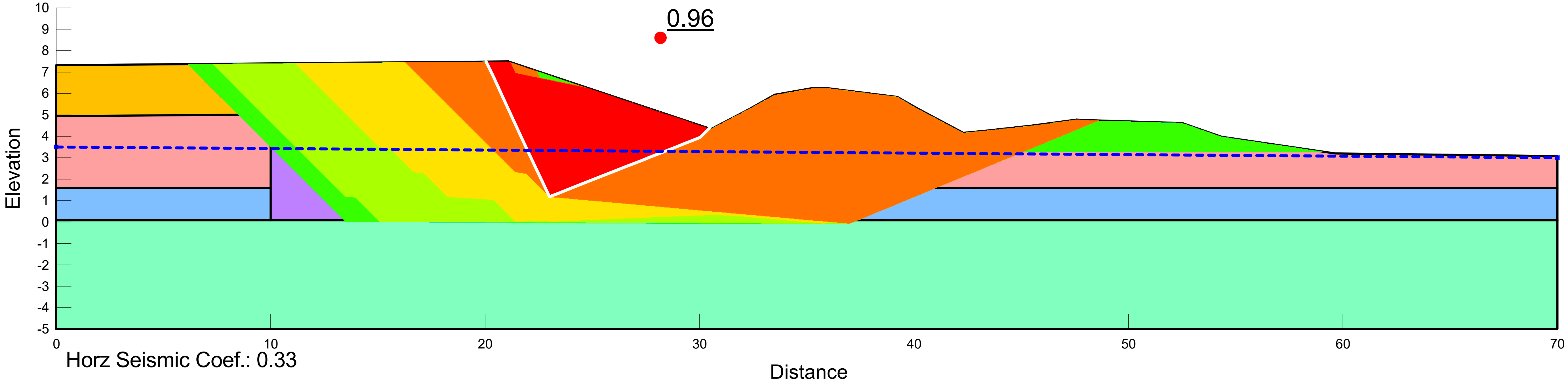


14_Northern abutment Flow failure
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Color	Name	Material Model	Unit Weight (kN/m³)	Minimum Strength (kPa)	Tau/Sigma Ratio	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
🟪	Compacted GRAVEL	Mohr-Coulomb	20			0	35	0	1
🟧	Emabnkment	Mohr-Coulomb	20			0	35	0	1
🟫	Stopbank	Mohr-Coulomb	18			2	28	0	1
🟥	Unit 1 - Sandy GRAVEL	Mohr-Coulomb	19			0	33	0	1
🟦	Unit 2 - Liquefied	SHANSEP	8	3	0.2				1
🟩	Unit 3 - Dense sandy GRAVEL	Mohr-Coulomb	20			0	34	0	1

Factor of Safety

🟥	0.96 - 1.06
🟧	1.06 - 1.16
🟨	1.16 - 1.26
🟩	1.26 - 1.36
🟩	1.36 - 1.46
🟩	1.46 - 1.56
🟩	1.56 - 1.66
🟦	1.66 - 1.76
🟦	1.76 - 1.86
🟦	≥ 1.86



Horz Seismic Coef.: 0.33

15_Northern abutment Yield Acc
Preliminary Embankment Design_rev2.gsz
22/06/2022 1:200

Ashburton 2nd Bridge Vertical Pile Capacity Check

Static Check

Number of Piles 2

Single Pile Vertical load (Using NZTA Bridge Manual Table D2 - Combination 1A (Worst Case))

Load Type	Load Magnitude (kN)	Load Factor	Design Load (kN)	
DL	12683	1.35	17100	
LL	2220	2.25	6100	
Imapct Load Factor (BP)		1.22		
		Total Vertical Load	**23200**	**kN**
		Vertical Load Per Pile	**11600**	**kN**

Single Pile Vertical Capacity (using B1/VM4 driven pile methodology)

Description	Ultimate Capacity (kN)	Design Capacity (kN)			
Base Bearing (kN)	19385	10100			*Calculation shown in base resistance tab*
Skin Friction (kN)	3128	1600			*Calculation shown in shaft resistance tab*
Sensitive check β Method	4953	2600			*Calculation shown in sensitivity check tab, cohesionless β Method*
Total Capacity	**22512**	**11700**	**kN**	**OK**	

Strength Reduction Factor 0.52 *Calculation attached as per AS2159-2009*

Base Resistence Calculation on pile (single) - B1/VM4 Method

Unit	Bottom of Layer Depth (m)	SPT	Y (kN/m^3)
Gravel Compacted Fill	1.5	50	20
Gravel	3.0	40	19
Gravel	4.5	50	19
Gravel	6.0	50	19
Gravel	7.5	50	19
Gravel	9.0	30	19
Gravel	10.5	50	19
Gravel	12.0	50	19
Gravel	13.5	50	19
Gravel	15.0	50	19
Gravel	16.5	50	19
Gravel	18.0	32	19
Gravel	19.5	50	19

Replacing topsoil with gravel compacted fill

H_{water}	1.5	m	
Y' @ targeted depth	9	kN/m^3	
c' @ targeted depth	0	kPa	
ø' @ targeted depth	34	°	
D_b	1.5	m	
A_b	1.8	m^2	
q'	151.5	kPa	*Water level at 1.5 m bgl*
N_q	70		*Figure 4. B1/VM4*
N_Y	45		*Figure 3. B1/VM4*
V_{bu}	**19385**	**kN**	

$$V_{bu} = (9c^{\prime} + q^{\prime}N_q + 0.6\,D_b\,\Gamma N_{\gamma})\,A_b$$

Figure 4: **N_q Values for Pile Foundations**
Paragraph 4.1.3

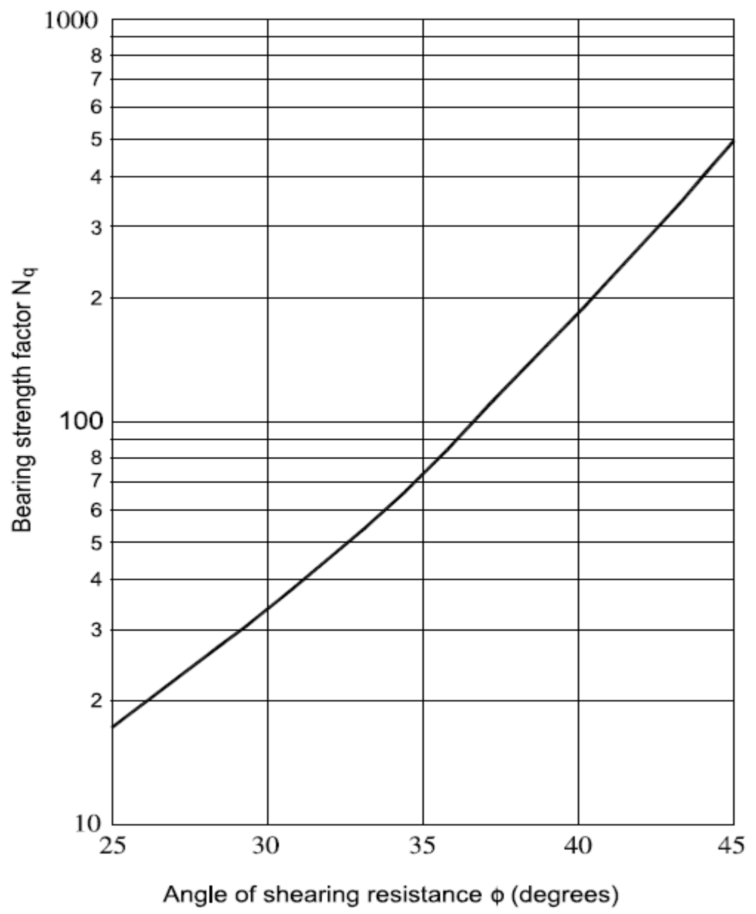
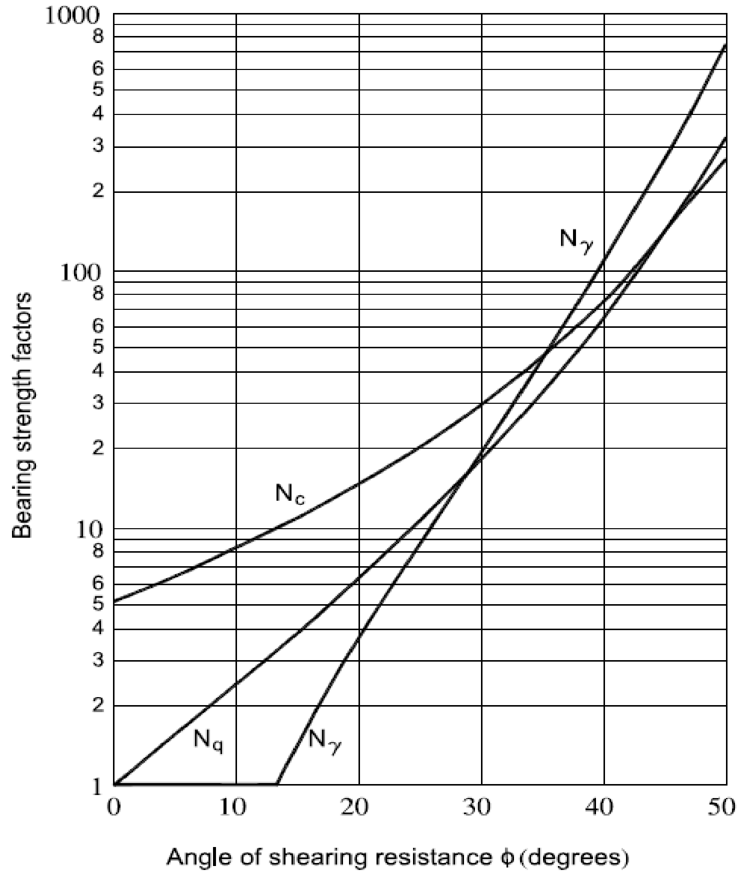


Figure 3: **Bearing Strength Factors**
Paragraphs 3.3.2 and 4.1.3



For ϕ = 0: N_c= 5.14, N_q= 1, N_γ= 0.

Shaft Resistance (Single Pile Analysis)

Ref. New Zealand Building Code (2000). Clause B1, Verfication Method 4.

Pile Type	Concrete-cassion composite pile	
Unit Weight (above GWL)	20.0	kN/m3
Unit Weight (below GWL)	19.0	kN/m3
Pile Diameter	1.5	m
GW Depth	1.5	m
Pile Circumference	4.7	m

Factored Dead Load	17100 kN
Factored Base Resistance	10100 kN

Top of Layer (m)	Base of Layer (m)	Liquefiable	Soil Cohesion	Depth to Midpoint of Layer (m)	Layer thickness (m)	Horizontal Effective Sress Factor (Ks)	Angle of Shearing Resistance (degrees)	Angle of Shearing Resistance (degrees)	Undrained Shear strength (Su)	Adhesion Factor	σ_{vo} (kPa)	σ_{vo}' (kPa)	Skin Friction Load (kN)
0.0	1.5	No	No	0.75	1.5	1.0	20	0.34907	0	-	15	15	39
1.5	3.0	No	No	2.25	1.5	0.5	20	0.34907	0	-	44	37	47
3.0	4.5	No	No	3.75	1.5	1.0	20	0.34907	0	-	73	51	130
4.5	6.0	No	No	5.25	1.5	1.0	20	0.34907	0	-	101	64	166
6.0	7.5	No	No	6.75	1.5	1.0	20	0.34907	0	-	130	78	201
7.5	9.0	No	No	8.25	1.5	0.5	20	0.34907	0	-	158	92	118
9.0	10.5	No	No	9.75	1.5	1.0	20	0.34907	0	-	187	106	272
10.5	12.0	No	No	11.25	1.5	1.0	20	0.34907	0	-	215	120	308
12.0	13.5	No	No	12.75	1.5	1.0	20	0.34907	0	-	244	133	343
13.5	15.0	No	No	14.25	1.5	1.0	20	0.34907	0	-	272	147	379
15.0	16.5	No	No	15.75	1.5	1.0	20	0.34907	0	-	301	161	414
16.5	18.0	No	No	17.25	1.5	0.5	20	0.34907	0	-	329	175	225
18.0	19.5	No	No	18.75	1.5	1.0	20	0.34907	0	-	358	189	485
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!
				0	0			0	0	-	0	0	#VALUE!

Rd

Steel Caisson

3128 kN

Ultimate Skin Friction (not considering negative portion from static loading)

Key

Horizontal Effective Stress Factor (Ks)	Table 2, Page 60
Angle of Shearing Resistance (δ')	Table 2, Page 60
Adhesion Factor (α)	Figure 5, Page 62

Effective Stress Method (β -method)

Top Layer Depth (m)	Bottom Layer Depth (m)	Mid point (m)	Total overburden pressure, σ_v (kPa)	Effective overburden pressure, σ_v' (kPa)	Skin friction, f_s (kPa)	Cumlative Skin friction, R_s (kN)	Uplift Skin friction, f_{st} (kPa)	Cumlative Uplift Skin friction, R_{st} (kN)
0.0	1.5	0.75	15	15	4	16	3	11
1.5	3.0	2.25	44	37	11	116	7	78
3.0	4.5	3.75	73	51	15	266	10	178
4.5	6.0	5.25	101	64	19	474	13	316
6.0	7.5	6.75	130	78	23	740	16	493
7.5	9.0	8.25	158	92	27	1064	18	709
9.0	10.5	9.75	187	106	31	1446	21	964
10.5	12.0	11.25	215	120	36	1885	24	1257
12.0	13.5	12.75	244	133	40	2383	26	1589
13.5	15.0	14.25	272	147	44	2938	29	1959
15.0	16.5	15.75	301	161	48	3552	32	2368
16.5	18.0	17.25	329	175	52	4223	35	2816
18.0	19.5	18.75	358	189	56	4953	37	3302

Friction angle (degrees) = 34

Input required for yellow cells

K_o = 0.44

K_p = 3.54 (Rankine)

K_a = 0.28 (Rankine)

β = 0.30

Pile diammeter (m) = 1.5

GW = 1.5 m bgl

Unit weight above GW, γ (kN/m^3) = 20

Unit weight below GW, γ (kN/m^3) = 19

UltimateTotal Axial Compression R_s (kN) = 4953

0 Note: Compression Skin Friction values not taken in total capacity as multiple voids will be surrounding t

DETERMINATION OF GEOTECHNICAL REDUCTION FACTOR IN ACCORDANCE WITH AS2159-2009

Project Name:	Ahsburton 2nd Bridge
Project Number:	310205125
Engineer:	Alex Park
Date:	2022.06.07

Table 4.3.2(A)

Risk Factor	**Weighing Factor (w_i)**	**Typical description of risk circumstances for individual risk rating (IRR)**			**Attributed Risk Rating**	**Note**
		1 (Very Low Risk)	**3 (Moderate)**	**5 (Very High Risk)**		
Site						
Geological complexity of site	2	Horizontal strata, well-defined soil and rock characteristics	Some variability over site, but without abrupt changes in stratigraphy	Highly variable profile or presence of karstic features or steeply dipping rock levels or faults present on site, or combinations of these	**2 (Low)**	
Extent of ground investigation	2	Extensive drilling investigation covering whole site to an adequate depth	Some boreholes extending at least 5 pile diameters below the base of the proposed pile foundation level	Very limited investigation with few shallow boreholes	**2 (Low)**	
Amount and quality of geotechnical data	2	Detailed information on strength compressibility of the main strata	CPT probes over full depth of proposed piles or boreholes confirming rock as proposed founding level for piles	Limited amount of simple in situ testing (e.g., SPT) or index tests only	**3 (Moderate)**	
Design						
Experience with similar foundations in similar geotechnical conditions	1	Extensive	Limited	None	**2 (Low)**	
Method of assessment of geotechnical parameters for design	2	Based on appropriate laboratory or in situ tests or relevant existing pile load test data	Based on site-specific correlations or on conventional laboratory or in situ testing	Based on non-site-specific correlations with (for example) SPT data	**3 (Moderate)**	
Design method adopted	1	Well-established and soundly based method or methods	Simplified methods with well-established basis	Simple empirical methods or sophisticated methods that are not well established	**3 (Moderate)**	
Method of utilizing results of in situ test data and installation data	2	Design values based on minimum measured values on piles loaded to failure	Design methods based on average values	Design values based on maximum measured values on test piles loaded up only to working load, or direct measurements used during installation, and not calibrated to static loading case	**3 (Moderate)**	
Installation						
Level of construction control	2	Detailed with professional geotechnical supervision, construction processes that are well established and relatively straightforward	Limited degree of professional geotechnical involvement in supervision, conventional construction procedures	Very limited or no involvement by designer, construction processes that are not well established or complex	**3 (Moderate)**	
Level of performance monitoring of the supported structure during and after construction	0.5	Detailed measurements of movements and pile loads	Correlation of installed parameters with on-site static load tests carried out in accordance with this Standard	No monitoring	**3 (Moderate)**	

System Redundancy	**Low Redundancy**	3
Type of Testing	**No testing**	0
Percentage of piles tested (p)	**0%**	

Average Risk Rating (ARR)	2.7
Overall Risk Category	Low to moderate
Basis Geotechnical Strength Reduction Factor φ_{gb}	0.52
Intrinsic Test Factor φ_{tf}	0.52
Testing Benefit Factor K	0.00

Φ_g	0.52

CREATING COMMUNITIES

Communities are fundamental. Whether around the corner or across the globe, they provide a foundation, a sense of belonging. That's why at Stantec, we always **design with community in mind**.

We care about the communities we serve—because they're our communities too. We're designers, engineers, scientists, and project managers, innovating together at the intersection of community, creativity, and client relationships. Balancing these priorities results in projects that advance the quality of life in communities across the globe.

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