REPORT

Nelson City Council

Tahunanui Area Liquefaction Assessment

Report prepared for:

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Executive summary

The key findings from this preliminary liquefaction assessment of the Tahunanui Area in Nelson are:

- The Study Area is underlain by silt and sand dominant sediments to depths of 8 m bgl (below existing ground level) in the south-east and up to 14 m depth in the north-west. In general, between 50% and 80% of this layer is assessed to be liquefaction susceptible.
- Gravel dominant layers inter-bedded with silt and sand was encountered below the silt and sand dominant sediments to the maximum depth of investigations (26.6 m bgl in the north-western part of the Study Area).
- Based on the currently-available geotechnical investigation data we assess the subsoil classification of the Study Area to be “Class C - Shallow Soil” in terms of NZS 1170:2004.
- Preliminary (i.e. with no correction to account for soil plasticity) analyses of CPT results indicate total liquefaction induced settlements are likely to be between 5 and 25 mm during an SLS (Serviceability Limit State) seismic event, and, between 130 mm and 290 mm during an ULS seismic event.
- The soils that are predicted to liquefy generally comprise sands to non-plastic silts. Visual assessment of the core which was recovered from the machine boreholes indicates that none of these potentially liquefiable soils are likely to have sufficient plasticity to resist liquefaction.
- Analysis of the CPT results using the recently developed Liquefaction Severity Number (LSN) methodology indicates that collateral damage due to liquefaction is likely to vary across the Study Area, and, the level of damage is likely to be similar to that observed recently in areas zoned as TC2 and TC3 land in Christchurch.
- During a ULS (Ultimate Limit State) seismic event lateral spread displacements in the order of 100-200 mm are expected within 100-200 m of any sharp changes in elevation (such as adjacent to tidal channels). This lateral spread displacement is typically expected to reduce to less than 50mm a distance 200 to 300 m back from the water’s edge.
- Some waterside properties where unfavourable topographic and geotechnical conditions are present may exhibit more than 300 mm lateral spread as a result of ULS levels of seismic shaking. Such a level of lateral spread is likely to result in high levels of building damage. It is likely that the lateral spreading risk on such properties cannot be readily mitigated on a site by site basis.
- Mitigation measures are available that can be incorporated into new building development, building upgrading, and as part of infrastructure renewal.
- For areas showing characteristics of TC2 and TC3 land, and in the absence of national guidelines, future site investigations for development should follow the guidelines of MBIE for investigating TC2 and TC3 land.
- As part of the current RMA review, there is likely to be more clarity to local bodies regarding hazard identification, and their likely obligations regarding investigation and documentation of hazards within the District Plan. Information currently available from this study can be provided to land owners via property Land Information Memoranda (LIM’s).

1 Introduction

Tonkin & Taylor Ltd (T&T) has been engaged by Nelson City Council (NCC) to undertake a preliminary assessment of the liquefaction potential of sediments underlying the Tahunanui area of Nelson City.

Authority to proceed with this report was provided in writing by Martin Workman of NCC on 30 March 2013. T&T's Letter of Engagement dated 28 February 2013 sets out the scope of works and conditions of engagement.

The area of land which is the subject of this report comprises the low-lying flat to gently sloping land at Tahunanui as shown on Figure A1. Henceforth this area of land is referred to as the Study Area. It must be appreciated that other areas in the Nelson urban area may also be subject to a liquefaction risk.

T&T have previously prepared and issued a draft report entitled 'Tahunanui Liquefaction Potential Geotechnical Desk-top Assessment', dated 28 March 2013 (T&T ref. 871023). This report was based on a desk-top study of available information relevant to the Study Area.

Subsequent to issue of the above report, T&T have completed a preliminary site investigation of the Study Area comprising two (2) boreholes, ten (10) Cone penetrometer (CPT) tests, and; a MASW geophysical (Multi-channel Analysis of Surface Waves) survey. The conclusions in this report may differ from those given in the March 2013 desk-top assessment report as this report incorporates the results of site-specific geotechnical investigations. The logs for boreholes BH1 and BH2, the results of the MASW survey, and the results of the CPT liquefaction assessment are attached in Appendix D.

As mentioned in the previous paragraph, the results from 10 CPT tests have been utilized to assess the liquefaction potential within the Study Area. A further test (CPT4 as shown on Figures A1 to A3) was carried in the southern part of the Study Area. However, Quality Assurance (QA) checks carried out during testing showed errors outside of acceptable limits. As a result of this the data from this test on liquefaction potential has not been included as part of this assessment. However, the results of the test were in general agreement with the other test carried out within the Study Area. Groundwater level information from this test is not subject to these errors and has been included on Figure A1.

We wish to stress that the level of intrusive geotechnical investigation which has been completed to date is not sufficient to allow zoning of the liquefaction risk to be completed. However, the currently-available geotechnical data is considered sufficient to conclude that the Tahunanui area is subject to variable degrees of liquefaction induced land damage under earthquake shaking that is likely to be experienced during normal residential building lifetimes.

2 Liquefaction description

Appendix B includes a detailed description of the process of liquefaction and its effects. This section summarises that detailed description.

Liquefaction is where loose soils below the groundwater level loose strength and stiffness in response to an applied cyclic force, like earthquake shaking (refer Appendix B). Liquefaction can cause damage to land, buildings and infrastructure. Only some soil types are susceptible to liquefaction and only some earthquakes are strong enough to cause liquefaction. Geotechnical investigations and analysis can be applied to estimate the likelihood and consequence of liquefaction making up the risk of liquefaction for a specific site.

2.1 Susceptible soils

Liquefaction only occurs in some soils. Liquefaction susceptible soils typically have the following characteristics:

- Non-cohesive
- Loose to medium dense
- Saturated (beneath the water table)
- Not very high permeability.

In general:

- Sands and non-plastic silts are most susceptible to liquefaction
- Gravels can liquefy if they have a low permeability matrix or confining layers top and bottom
- Clays are generally too cohesive to liquefy.

The distinction between silts that are liquefiable or not are described as either being:

- “Sand-like behaviour” and therefore susceptible to liquefaction
- “Clay-like behaviour” and therefore not susceptible to liquefaction.

The NZ Geotechnical Society “Guideline for the identification, assessment and mitigation of liquefaction hazards” (NZGS, 2010) provides further criteria for the assessment of liquefaction susceptible soils. Particular guidance is provided for fine grained soils (silts etc.).

Section 5 reports on the liquefaction susceptibility of the Study Area.

2.2 Triggering

The intensity and duration of earthquake shaking required to cause (trigger) liquefaction of susceptible soil varies depending on the density and fines content of the soil. The likelihood (return period) of earthquake shaking to trigger liquefaction is assessed by considering:

- The local seismic hazard. The likelihood (return period) of earthquakes of various duration (magnitude) and intensity (peak ground acceleration, PGA).
- Field penetration test (CPT and SPT) and fines content results for the soil, and available empirical relationships between these results and the magnitude and PGA to trigger liquefaction.

Section 5.7 reports on the assessed trigger for liquefaction of the Tahunanui Area.

2.3 Liquefaction effects

There is a number of liquefaction effects each of which affect buildings and infrastructure differently. These include:

- Surface ejection of soil and water (Sand Boils)
- Buoyancy effects in buried pipes, tanks, chambers and basements
- Reduced bearing capacity of foundations
- Settlement
- Lateral spreading.

Appendix B provides details of these effects. The degree to which these effects relate to a particular site depends on the site specific ground conditions.

Section 5 describes the consequences of liquefaction for the Tahunanui Area and Section 6 discusses mitigation options.

3 Site conditions

3.1 Landform features

The study area comprises generally flat to gently north-west sloping land lying at an elevation generally between 14 m and 19 m (Nelson City Datum) which is 2-7 m above mean sea level. It is characterised by:

- Relic beach ridges that are semi-continuous, generally east-west trending lines topography raised 1 m to 7 m above the adjacent land,
- Back beach estuarine areas of low elevation topography between relic dune ridges, locally tidal and /or swampy,
- An abandoned sea cliff rising to between 3 m and 6 m above the beach and estuarine deposits and forming the southern and south-eastern margin of the study area,
- Gently inclined fan surfaces that pro-grade locally from the base of the Tahunanui Hills onto the eastern part of the area.

In places, particularly within the Airport site, the natural ground surface has been altered by earthworks and reclamation.

3.2 Subsurface geology

The geology has been mapped (refer to Figure A1, Appendix A) as consisting of Holocene beach sands and gravels (Tahunanui Sands and Rabbit Island Gravel) and Late Pleistocene Gravels (Stoke Fan Gravel).

Our drilling has indicated a sequence of loose sands and silty sands overlying loose to moderately dense or stiff sandy silts and silty gravels to depths varying from 15 m to more than 20 m. These soils overlie very dense to hard silt and gravel.

The soils which are present within the upper 20 m to 25 m of the soil profile are inferred to comprise Tahunanui Sands and Stoke Fan Gravel. These materials are assessed to have variable liquefaction susceptibility, as set out in Table 2 below.

The following is a summary of the key geotechnical characteristics which have been inferred from the data which is currently available:

- The Tahunanui Sands generally comprise loose sandy and non-plastic silty sediments.
- In general, the thickness of the Tahunanui Sands was found to increase to the north-west, reaching an inferred depth of 14.8 m at CPT6.
- The MASW survey indicted a low velocity layer of material was present at depths between
  3 m and 6 m depth across large parts of the north-west of the Study Area.
- Analysis of CPT results indicated that the marginal marine /estuarine sediments underlying the Tahunanui Sands also contain silty and sandy layers that were predicted to liquefy under a ULS earthquake scenario.
- Analysis of CPT results indicated that 50-80% of the total thickness of Tahunanui Sands is predicted to liquefy in a future ULS earthquake scenario.
- The MASW showed that the upper 10 m of sediments at the eastern extent of MASW Line 4 (<ch. 980 m) has a higher seismic velocity than indicated in the rest of the MASW survey. This may indicate that these sediments are stronger and less susceptible to liquefaction than those further to the west.

Table 1 below presents the generalised soil profile which was encountered during the preliminary geotechnical investigation and provides a summary of the typical soil properties related to liquefaction potential.

Table 1 – Generalised soil profile

Inferred Geology	Typical depth to top of layer (m)	Typical thickness (m)	Typical CPT q_c (MPa)	Liquefaction Susceptibility (Refer Sections 2.1 and 6.2)		
				Material Description	SLS Seismic Event (1/25 AEP*)	SLS Seismic Event (1/500 AEP*)
Tahunanui Sands	0 to 1	8-15	5 to 15 (typically 12)	Sandy SILT to silty SAND	Moderate	High
			15-25	Silty GRAVEL and GRAVEL lenses	Low	Moderate
Stoke Fan Gravel	8 to 15	8 - >10	3-13	Sandy SILT and silty SAND	Low	Moderate
			20-30	Silty GRAVEL	Low	Low

AEP = Annual Probability of Exceedence

3.3 Groundwater

Groundwater levels have been inferred from measurements that have been made at the locations shown on Figure A1 attached in Appendix A. The following preliminary conclusions have been made regarding the site groundwater level:

- Groundwater levels recorded in CPTs ranged between 0.7 m and 2.3 m depth.
- Groundwater levels are very flat and generally fall to the north-west.
- Areas of elevated groundwater levels are present within the Study Area beneath areas of elevated topography, i.e. the sand dune deposits at Nelson Golf Club and on the fan deposits in the east of the Study Area.

3.4 Existing land use and infrastructure

The Study Area can be divided broadly into three zones of development as shown on Figure A1, and are described below:

- The Residential Zone is situated in the central and northern areas of the Study Area, and occupies approximately 1/5th of its total area. The land in this area is already fully developed apart from Centennial Park. This area also includes the Tahunanui Drive section of State Highway 6 (SH6)
- The Industrial Zone occupies the south-eastern part of the Study Area and comprises approximately 1/4th of the total Study Area. There is still a small area of undeveloped industrial zoned land in the southern extent of the Study Area. This area also includes the Whakatu Drive section of State Highway 6 (SH6).

The above areas are already largely developed, and contain a significant amount of NCC owned assets, i.e. roads and services, as well as private infrastructure.

- NCC owned land comprises the majority of the western and northern parts of the Study Area, and is made up of:
- Nelson Airport in the south-west,
- Nelson Golf Course in the west,
- The Tahunanui Holiday Park to the west of the residential area, and,
- The Tahunanui Recreation Area in the north.

The majority of this land currently comprises grassed fields. However, some significant infrastructure in the form of buildings and hard-standing areas is present within all of these areas (most notably Nelson Airport). No physical investigations have been carried out in Area 3 (NCC owned land) as the purpose of this investigation was to establish the liquefaction potential in the built up areas which are described in 1 and 2 above. However, Area 3 (which includes Nelson Airport) is likely to be subject to similar geological conditions to those encountered within the Study Area.

4 Earthquake scenarios

4.1 General

New Zealand Standard, NZS1170.5:2004 Structural Design Actions Part 5 Earthquake Actions, clause 2.1.4 specifies that in order to meet the requirements of the New Zealand Building Code, design of structures is to allow for two earthquake scenarios:

1. (SLS) “Serviceability limit states for earthquake loading are to avoid damage to .... The structure and non-structural components that would prevent the structure from being used as originally intended without repair after the SLS1 earthquake ... “.
2. (ULS) “Ultimate limit state for earthquake loading shall provide .... Avoidance of collapse of the structural system ... or loss of support to parts... damage to non-structural systems necessary for emergency building evacuation that renders them inoperative.”

The earthquake magnitude at the epicentre (M) and peak ground acceleration (PGA) have been proposed for evaluation of liquefaction potential in the Study Area is presented in Table 2 below. These earthquake magnitudes and accelerations are based on seismic hazard coefficients (Z factors) for various areas within New Zealand prescribed in NZS1170.5. It must be appreciated that these earthquakes are remote from the site, and are theoretical earthquakes used for modelling purposes.

Table 2 - Design earthquake scenarios

Design Case	Peak Ground Acceleration (PGA) (g)	Magnitude (M)	Annual Probability of exceedence
SLS	0.09g(1)	7.5 (2)	1/25
ULS	0.36g(1)	7.5 (2)	1/500

Notes:

(1) Assumes Seismic Subsoil Class C and

(2) Magnitude M = 7.5 reflects the magnitude weighting used for the calculation of PGA in NZS1170.5:2004.

PGA has been assessed based on NZS1170.5: 2004 for the following:

Building design life	50 years
Building importance level	2
Return period factor	1.0 for 500 years and 0.25 for 25 years.
Sub-soil class	C (Shallow soils)
Hazard factor	0.27 (Nelson)

4.2 Importance category

A Building Importance Level of 2 (IL2) as defined in NZS1170.5:2004 has been used for this study, as the large majority of buildings within the Study Area fall into this category (single family dwellings).

Buildings where larger numbers of people can congregate (i.e. churches, health-care facilities, air-port terminals, and large commercial and industrial buildings) or buildings that perform a special function (i.e. post-disaster functions, or that contain hazardous waste) should be designed with a higher IL category and as a result must be designed for a higher return period seismic event.

4.3 Site subsoil classification

A sites response to an earthquake is partly dependant on the depth of weak soils that underlie the site. In our draft desk-top study report, we assumed that the Site Subsoil Class (in terms of NZS1170:2004 – Structural Design Actions) as likely to be consistent with a Class C (Shallow Soil) classification. Basic descriptions of each category are given below:

Class A: Strong Rock

Class B: Rock

Class C: Shallow Soil Sites

Class D: Deep or Soft Soil Sites

Class E: Very Soft Soil Sites

NZS1170:2004 gives guidelines for ascertaining a site's likely Site Subsoil Class, one of which is estimates of shear-wave velocity travel times from bedrock to the ground surface. The MASW survey carried out within the Study Area gives preliminary information on the shear-wave velocity of the soils beneath the site.

Based on the most conservative estimate of the seismic velocities from the MASW survey, the site is consistent with a Class C (Shallow Soil Sites) classification.

Information on the depth to bedrock within the Study Area is limited. The maximum depth to bedrock recorded within the Study Area is 33.5 m (logged as Moutere Gravel) as recorded in a historic borehole drilled adjacent to CPT9 (DH13 from the Geo-logic report listed in Section 4 of this report) in the north-western part of the Study Area. Our geological model of the Study Area (refer Section 3.1 of this report) suggests that the bedrock depth increases to the north-west. If bedrock is significantly deeper than logged in DH13 (i.e. greater than 40 m) elsewhere within the Study Area such areas may be classified as Class D (Deep or Soft Soil Sites). Although this is not likely, our assessment suggests that any Class D areas that do exist are likely to be present along the north-west margin of the Study Area adjacent to the Blind Channel (Waimea River channel).

5 Liquefaction assessment

5.1 Key documents

We have referenced the following key documents during the liquefaction assessment:

- New Zealand Geotechnical Society (NZGS) – 'Guidelines for Geotechnical Earthquake Engineering Practice in New Zealand' dated July 2010. This provides a basis for the assessment of liquefaction potential.
- Ministry of Business Innovation and Employment (MBIE) – 'Guidances: Repairing and rebuilding houses affected by the Canterbury earthquakes' - Part A, dated December 2012. It comments on site investigation techniques, liquefaction assessment, land classification and advisory recommendations. Although prepared specifically for Canterbury and for residential subdivision purposes, the general principals presented are considered relevant to other regions. At this time there is no equivalent national document.
- Standards New Zealand. NZS1170.5:2004 Structural Design Actions – Part 5: Earthquake Actions – New Zealand. This is a New Zealand Standard providing procedures for the determination of earthquake actions on structures in New Zealand (earthquake hazard).
- Dr M.R. Johnston – 'Preliminary Assessment of the Liquefaction Hazard in Tasman and Nelson Regions' - dated June 2011.
- Geo-logic Limited – 'Liquefaction Hazard Review- Drill Hole Data Compilation NELSON' dated January 2013.

5.2 SLS / ULS liquefaction induced settlements

Seismic liquefaction occurs when excess pore pressures are generated in loose, saturated, generally cohesionless soil during earthquake shaking, causing the soil to undergo a partial to complete loss of shear strength. Such a loss of shear strength can result in settlement and/or horizontal movement (lateral spreading) of the soil mass. The occurrence of liquefaction is dependent on several factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and elevation of the groundwater table.

Analyses were performed to evaluate the liquefaction potential of the loose to medium dense sands and non-plastic/low plasticity silts found in the borehole and CPT soundings utilising the methods recommended by Cetin et al. (2004) and Moss, et al. (2006). The two earthquake scenarios described above, and assumed groundwater levels of between 0.7 m and 2.3 m bgl were assumed in our analyses, based on the data collected during our site investigation.

The seismic settlements of the liquefiable layers identified were computed using the methodology published by Ishihara and Yoshimini and are summarised below in Table 3.

Table 3 – Summary of liquefaction-induced free-field settlements inferred from analysis of the CPT Data

Location	Computed total liquefaction-induced settlement (mm)*	
	SLS (M=7.5, PGA=0.09g)	ULS (M=7.5, PGA=0.36g)
CPT-1	8	127
CPT-2	12	266
CPT-3	11	185
CPT-5	5	157
CPT-6	18	285
CPT-7	11	239
CPT-8	23	278
CPT-9	18	245
CPT-10	13	255
CPT-12	6	144
TOTAL RANGE	5-23	127-285
AVERAGE	13	218

A detailed summary of the liquefaction analysis results and output is presented in Appendix D.

The methodology used to obtain the above total settlement figures may be conservative as no correction has been made for soil plasticity. Therefore the values given above in Table 3 are generally expected to represent an upper bound of the total settlement likely at the test locations. If soils which have been predicted to liquefy under the analysis used in this report contain some clay-sized particles, and therefore have some degree of plasticity, their liquefaction potential will be lower than that predicted by the above analysis.

Examination of the core recovered from BH1 and BH2 identified no soil layers that were predicted to liquefy in the adjacent CPT probe had enough plasticity to resist liquefaction.

5.3 Assessed thickness of liquefaction and settlement

The following Figures in Appendix A summarise key results from the liquefaction analysis;

- Figure A2 “Liquefaction settlement and Thickness”, reports the following;
    - Free field calculated settlements for SLS and ULS events. These are reported for each CPT.
    - The cumulative thickness of liquefaction (CLT) for layer 2 (above 4 m depth) and layer 3 (below 4 m depth) are reported for ULS and SLS events.
- Figure A3– “Liquefaction profile with depth”, shows a graphical output of the soil layers that may potentially liquefy. These bar charts are shown in plan view on the page and are for each CPT.

The general conclusions of the liquefaction analysis (refer Figures A2 and A3 in Appendix A) are as follows;

- The groundwater level above which liquefaction is not expected to occur is typically between 0.7 and 2.3 m below ground surface level. Liquefaction could be expected to occur with a cumulative thickness of:
  - a) Serviceability limit state (1/25 AEP) earthquake (SLS)

    Cumulative layer thickness: 0 - 0.2 m
  - b) Ultimate limit state (1/500 AEP) earthquake (ULS)

    Cumulative layer thickness: 5.4 – 14.9 m
- Calculated free field settlements, i.e. settlements which could be expected on ground which is not surcharged by buildings or other near surface load. Surface loads could result in higher settlements.
  - a) Serviceability limit state (1/25 AEP) earthquake (SLS)

    Free-field Settlements: 2 mm to 25 mm
  - b) Ultimate limit state (1/500 AEP) earthquake (ULS)

    Free-field Settlements: 130 mm to 290 mm

5.4 MBIE - Foundation technical categories

As part of the Christchurch Earthquake recovery process, the MBIE has developed a classification system of ‘Technical Categories’ (TC1 to TC3 in Table 16.1 attached in Appendix C) to categorize the expected site response in areas of varying liquefaction hazard. Whilst this classification system was designed to classify land in Christchurch, there is currently no other system in use in New Zealand. Accordingly, we have classified the Study Area based on the MBIE guidance. For a description of Technical Categories refer to Table 16.1 from the MBIE guidance attached in Appendix C.

The above anticipated free-field settlement for the SLS earthquake scenario is consistent with a MBIE Technical Category 2 (TC2) classification. Anticipated free-field settlements for the ULS earthquake scenario are consistent with a MBIE Technical Category 3 (TC3) classification.

It is regarded as good practice to adopt the most severe classification where an area displays effects consistent with two or more Technical Categories, accordingly we assess that the site response in a future earthquake is likely to be consistent with a TC3 classification.

5.5 New Zealand Geotechnical Society Classification

In terms of the New Zealand Geotechnical Society[1] guidelines, the level of liquefaction estimated to occur at the site can be considered to correspond to a *Liquefaction Performance Level* of L3 to L4 (‘High’ to ‘Severe’) under ULS loading, and L1 to L2 (‘Mild’ to ‘Moderate’) under SLS loading. These performance levels are defined as follows:

Performance Level L0 - (Insignificant) “No significant excess pore pressures.”

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[1] New Zealand Geotechnical Society, Geotechnical earthquake engineering practice, Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards, July 2010

Performance Level L1 - (Mild) “Limited excess pore pressures without complete liquefaction; relatively small deformation of the ground with relatively small settlements (few tens of millimetres).”

Performance Level L2 - (Moderate) “Liquefaction occurs in layers of limited thickness (small proportion of the deposit); ground deformation results in differential settlements.”

Performance Level L3 – (High) “Liquefaction occurs in significant portion of the deposit resulting in differential movements, large settlements (few hundreds of millimetres) and lateral displacements.”

Performance Level L4 – (Severe) “Complete Liquefaction develops in most of the deposit resulting in very large settlements (total and differential) and lateral displacements of the ground.”

Performance Level L5 – (Very Severe) “Liquefaction resulting in lateral spreading.”

5.6 Liquefaction Severity Number (LSN)

The Liquefaction Severity Number (LSN) assessment methodology was developed by T&T on the behalf of the Earthquake Commission (EQC). Its purpose is to enable a more accurate prediction of the likely damage at the ground surface as a result of various seismic scenarios.

The closer a liquefiable layer is to the ground surface, the more likely it is to cause damage to surface structures during liquefaction. The LSN assessment methodology takes into account the depth and thickness of liquefiable layers in addition to their proximity to the ground surface, as well as crust thickness, varying soil conditions, shaking intensity, shaking duration and groundwater levels. The assessment output is an overall “LSN” rating for each earthquake scenario. Table 4 below summarises the anticipated ground effects for each range of LSN.

Table 4 – Summary of LSN and expected post-earthquake damage on the ground surface

LSN Range	Expected ground surface damage
0-10	Little to no expression of liquefaction, minor effects
10-20	Minor expression of liquefaction, some sand boils
20-30	Moderate expression of liquefaction, with some sand boils and structural damage
30-40	Moderate to severe expression of liquefaction, settlement can cause structural damage
40-50	Major expression of liquefaction, undulations and damage to ground surface, severe total and differential settlements of structures
>50	Severe damage, extensive evidence of liquefaction as surface, severe total and differential settlements affecting structures, damage to services.

* Table based on Table 13.1 from T&T report ‘Liquefaction Vulnerability Study’

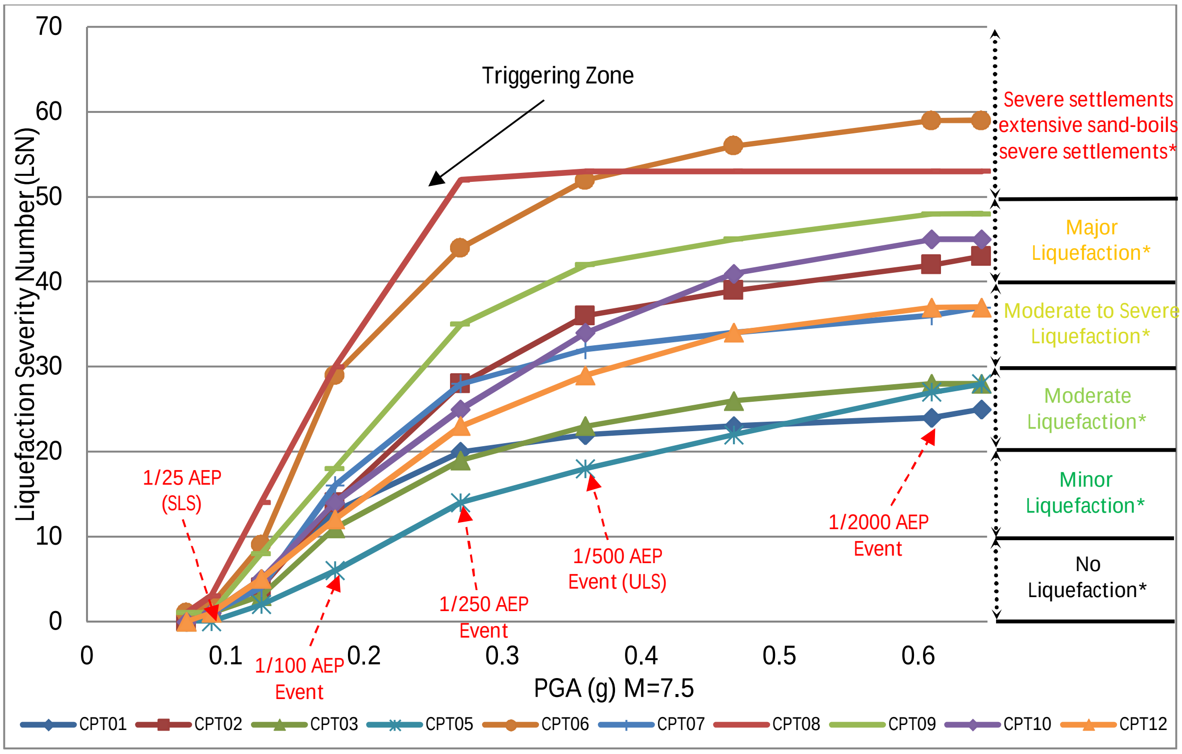
Table 5 below summarises the Liquefaction Severity Number that has been calculated using the ten CPT’s that were conducted during this investigation for both SLS and ULS earthquake scenarios. These are also summarized in plan view on Figure A2 in Appendix A.

Table 5 – Summary of LSN values calculated for the subject site

Earthquake Scenario	Liquefaction Severity Number (LSN)									
	CPT1	CPT2	CPT3	CPT5	CPT6	CPT7	CPT8	CPT9	CPT10	CPT12
SLS PGA = 0.09g	0	0	0	0	0	0	1	0	12	1
ULS PGA = 0.36g	22	36	23	18	52	32	53	42	34	29

5.7 Liquefaction trigger

Analysis has been undertaken to assess the trigger for liquefaction of susceptible soils in the Study Area by applying the cone penetration test (CPT) results. Published methods for liquefaction assessment (Idriss & Boulanger, 2008) and settlement (Zhang, Robertson, & Brachman, 2002) were applied. Figure 1 presents the assessed LSN for each CPT test for various return periods of earthquake shaking. More intense earthquake shaking (higher return period) will trigger liquefaction of more dense soils and thus result in a greater LSN.



* Predominant performance as per Table 5.

Figure 1 –LSN versus PGA at Tahunanui

With reference to Figure 1 the potential for liquefaction, thickness of potentially liquefied soil, is relatively small for the SLS seismic event (0.09g). However, at a slightly higher level of shaking (0.18g, 1/100 AEP event) the assessment indicates substantially more liquefaction is triggered. CPT6 and CPT8, which are located furthest to the west, show the highest LSN's. This is likely due to a combination of the thickness of liquefiable Tahunanui Sands increasing to the west, and the proximity of the groundwater table to the surface in these tests.

5.7.1 Influence of groundwater level on Liquefaction Severity

As discussed in Section 2 and Appendix B, liquefaction occurs beneath the groundwater table. The LSN is heavily influenced by the proximity of the groundwater table to the ground surface, especially where liquefiable sediments are present near the surface.

The groundwater data collected during our site investigation was gathered during CPT testing in April/May 2013, and as such gives no information on the variation in groundwater levels from winter to summer. It is likely that winter groundwater levels will be higher than those recorded during our investigation in late summer/autumn.

Historic groundwater data gathered during the desk-top assessment (and shown on Figure A1 in Appendix A) is in general accordance with groundwater levels recorded in our CPT tests. It is not known what time of year much of the historic groundwater data was collected therefore little can be inferred about the likely level of summer/winter groundwater level variance.

Collection of further groundwater data will provide a more accurate picture of the liquefaction susceptibility of the Study Area.

5.7.2 Historic events

During the last 170 years Nelson has experienced shaking from large earthquakes on at least four occasions that would have had the potential to initiate liquefaction (it is generally accepted that felt shaking intensities of MM7 or greater on the Modified Mercalli Scale are required to produce liquefaction).

The following is a list of earthquakes, assessed Magnitudes (M) at the epicentre, and Modified Mercalli felt intensities (MM) in Nelson that had the potential to initiate liquefaction within Study Area. A table giving descriptions of the various Mercalli Felt Intensities is attached in Appendix C.

- 1848 Marlborough Earthquake – MM 7
- 1855 Wairarapa Earthquake – MM 7
- 1868 Cape Farewall Earthquake – MM 7
- 1929 Murchison Earthquake – MM 8

We are not aware of any observations of liquefaction within the Study Area as a result of any of these earthquakes. All of the above earthquakes occurred prior to 1930. However, there appears to have been only sparse development of the low-lying flat land at Tahunanui prior to the 1930s.

We note that the 1968 Inangahua Earthquake produced likely MM 6 levels of shaking. This is unlikely to have produced liquefaction at the Study Area.

5.8 Consequences of liquefaction

Section 2 and Appendix B generally describe the possible consequences of liquefaction.

Liquefaction and associated ground damage could be expected within the Study Area as a consequence of a 1/100 AEP (0.18g, M7.5) seismic event or more intense shaking. Table 6 outlines the expected consequences of liquefaction for the Study Area.

Table 6 - Summary of potential consequences of liquefaction within the Study Area

Effects	Consequences	
	SLS	ULS
Sand Boils	Localised minor sand boils possible	Possibility of widespread sand boils across the Study Area. Sand boils can result in damage to all surface structures including paved surfaces.
Buoyancy and uplift of buried pipes and manholes	Unlikely	This is likely for parts of the Study Area where pipes and manholes are at or below the groundwater level. Preliminary assessment indicates this is potentially a significant area of the Study Area.
Bearing failure of shallow foundations and associated subsidence	Localised bearing failure of foundations possible	Widespread bearing capacity failure of shallow foundations likely. Likely to be a significant issue for more heavily loaded foundations.
Free-field settlement of ground surface	Minor (Refer section 5.3)	Free-field liquefaction induced ground surface settlements of typically 130 mm to 290 mm are currently predicted. Larger settlements may occur where surface loads are applied such as at foundation locations. Increased vulnerability to flooding (lowered ground surface level). Differential settlements could result in damage to underground services and paved surfaces (falls on pipes and surfaces) and to buildings. Increased vulnerability to liquefaction (crust thinning).
Lateral spreading	Not expected	Lateral displacement of up to 300mm or more could occur adjacent to the water's edge. Based on observations made after the Canterbury earthquakes his lateral displacement should reduce to less than 50 mm 100 to 300 metres back from the water's edge. Lateral spreading has the potential to result in severe cracking and damage to paved surfaces, buildings and buried services. Lateral spreading may also result in Increased vulnerability to flooding due to narrowed waterways.

Lateral spreading was the most damaging effect of liquefaction experienced in Christchurch in terms of damage to foundations and infrastructure.

Some waterside properties, where unfavourable topographic and geotechnical conditions are present, may exhibit more than 300 mm lateral spread as a result of ULS levels of seismic shaking. Such a level of lateral spread is likely to result in high levels of building damage. It is likely that the lateral spreading risk on such properties cannot be readily mitigated on a site by site basis.

The above summary of potential consequences is generalised. Any specific development proposed within the Study Area will require detailed site-specific geotechnical investigation and assessment of liquefaction consequences.

6 Guidelines for future development

6.1 Possible mitigation measures

6.1.1 Foundation mitigation options

This information is provided to assist NCC to assess what foundation options should be specified at the building consent phase for new dwellings and alterations to existing dwellings.

Appropriate foundations for individual sites within the Study Area will depend on the findings of site investigations carried out on each site.

6.1.1.1 Recommended foundation types

Section 15 of the Ministry for Business Innovation and Employment (MBIE) guidance document states that the following foundation types are appropriate for TC3 land:

- Deep piles
- Site ground improvements
- Surface structures and shallow foundations.

These are discussed with reference to the Study Area below.

6.1.1.2 Deep piles

Deep piles are not considered to be economically feasible within the Tahunanui Area, as no dense gravel layer of sufficient thickness was identified during our investigation that could serve as a founding layer for such piles.

6.1.1.3 Site ground improvements

Localised ground improvement may be feasible within the Study Area. Section 15.3 of the MBIE guidance currently gives the following general methods for ground improvement:

- Densification of either the crust layer and/or the deeper liquefiable soils. This includes methods such as compaction, excavation and replacement/recompaction, vibro-flotation, pre-loading, dynamic compaction (DC) and rapid impact compaction.
- Crust strengthening/stabilisation by permeation grouting, stabilizing mixing or replacement.
- Deep strengthening using deep soil/cement piles, jet grouting, stone columns, close spaced timber or precast piles.
- Containment by ground reinforcement or curtain walls.
- Drainage using stone columns or earthquake drains.

These methods are generally regarded as being suitable for sites where liquefaction susceptible soils are generally < 10 m below the ground surface. Although the majority of liquefiable sediments within the Study Area are within 10 m of the ground surface (Tahunanui Sands) our investigations indicate that a significant thickness of liquefiable sediments is present below this depth. This means that the first two methods described here (Methods 1 and 2) are not likely to reduce the settlements anticipated in a ULS event to within acceptable limits. The proximity of the groundwater table to the surface in parts of the Study Area will limit the depth to which these methods can be applied.

Methods 3 to 5 are usually relatively expensive and are unlikely to be economically feasible on a single site residential section. However, these may be appropriate for industrial and larger residential developments.

Some or all of these methods may require resource consent. In particular, noise and vibration effects should be considered.

6.1.1.4 Surface structures and shallow foundations

The following are considered suitable foundation options for sites consistent with a TC2/TC3 classification. These solutions are generally designed to provide a greater degree of resilience in the event of an SLS type earthquake scenario by being readily repairable. These are listed below in order of suitability for the Study Area:

- A suspended floor supported on a 'waffle-slab' type thickened reinforced concrete pad foundation, with a minimum 600 mm air gap between the concrete slab and the suspended floor. The concrete slab could in turn be supported on a hard-fill raft with 2 layers of heavy-duty, bi-directional Geogrid® or similar geo-textile to mitigate the risk of sand-boils adversely affecting the waffle-slab. The provision of 'slab piles' and Bowmac® type brackets between the waffle-slab and suspended floor would enable the building to be easily re-levelled following a future severe seismic event.
- A rib-raft concrete slab with slab thickenings below load-bearing walls, supported on a hard-fill raft with 2 layers of heavy-duty, bi-directional Geogrid® or similar geo-textile to resist any lateral spreading effects, and an underlying layer of Bidim® A19 cloth to prevent sand-boils penetrating the hard-fill raft.
- An enhanced foundation slab supported on a hard-fill raft with two layers of heavy-duty, bi-directional Geogrid or similar geo-textile to resist any lateral spreading effects and an underlying layer of Bidim A19 cloth to prevent sand-boils penetrating the hard-fill raft.

6.1.2 Implications for infrastructure

As discussed in Table 6 liquefaction induced differential settlements are likely to cause damage to infrastructure such as roading, buried service lines and manholes under a ULS earthquake scenario. As in parts of Christchurch following the 2011 earthquakes, the following effects are likely to occur at Tahunanui during an ULS seismic event:

- Buoyancy of manholes and services leading to damage and a loss of fall to sewer and stormwater services, rupture of pipes, and popping of manholes where they are below the water table.
- Ejection of sand beneath paved surfaces and differential settlement of pavements which would lead to significant damage to sub-grades and pavement surfaces and extensive pot-holing of the road surface.
- The proximity of the liquefiable sediments to the surface indicates that a loss of bearing capacity leading to both sinking and leaning of street lighting and power poles is also likely.
- Non NCC in-ground infrastructure such as power and telephone services is also likely experience significant disruption in a ULS seismic event.

To reduce the likelihood of sand boils compromising the integrity of the base-course beneath new pavements, a layer of non-woven, heavy-duty Bidim® cloth should be placed on the sub-grade prior to placement of the base-course.

Thickening of the hard-fill base-course layer placed beneath new pavements is expected to increase their resilience against differential settlement due to seismically induced liquefaction. A minimum hard-fill depth in the order of 500 mm, and/or the provision of a cement-stabilised sub-grade, is expected to result in satisfactory levels of post-liquefaction damage and serviceability.

Anchoring of manholes to resist uplift forces generated by liquefaction (i.e. ground anchors or similar) may be considered to reduce damage to manholes during future large seismic events. The length of these anchors will be governed by the depth of any potentially liquefiable layer. As such ground anchors in excess of 15 to 20 metres length may be required in some parts of the Study Area.

6.1.3 Mitigation of lateral spreading hazard

As discussed in Section 5.8, lateral spreading was the most damaging effect of liquefaction in Christchurch in terms of damage to foundations and infrastructure.

Lateral spreading of the order anticipated to occur within the Study Area during a ULS earthquake event (100-200 mm, and in extreme cases up to 300 mm within 100-200 m of any sharp changes in elevation - refer Table 6) is unlikely to be remediable on an individual site basis.

Perimeter treatments have been designed for use in areas of Christchurch identified to have a significant risk of lateral spreading. These treatment methods are designed reduce the risk of lateral spreading by strengthening the land immediately adjacent to the ‘free edge’ (such as a river bank) and effectively retaining it. A brief description of various mitigation options for lateral spreading is given below:

1. Stone columns – This involves drilling of large diameter holes, and replacement of site soils with granular material in a grid type pattern over a certain width back from the ‘free edge’
2. Soil densification – Various methods exist to density soils to increase their strength and hence resistance to liquefaction and lateral spreading. One of these that has been utilised in areas of Christchurch is vibro-compaction, which involves a large steel probe that penetrates the soil and compacts it though vibration as it is withdrawn from the ground.
3. Soil grouting – This involves injection of grout into the ground to effectively cement the soils together, increasing their strength.

These treatment methods can be expensive and to date – within New Zealand - have only been employed in Christchurch (as well as overseas). Also, to be effective they must be carried out over a wide area, and hence are not suitable for individual site remediation. Recent experience in Christchurch is that costs associated with construction of stone columns and CFA piles have dropped significantly as contractors become more experienced in carrying out this type of work. Also if these treatment measures are carried out over large areas, economies of scale will further reduce per square metre rates.

Further assessment (including assessment of topography and bathymetry adjacent to water bodies) is required before further recommendations can be given as to the suitability of the above methods for treatment of the lateral spreading risk within the Study Area.

6.2 Site investigation requirements

As discussed previously in this report, our preliminary assessment indicates the seismic performance of the land which underlies the Tahunanui Area is expected to be consistent with an MBIE TC2/TC3 categorisation. The MBIE guidance document gives appropriate levels of site investigation for the different Technical Categories (TC1 – TC3) of land.

MBIE guidance states the following for TC1 and TC2 type land:

‘For land that fits the characteristics of TC1 and TC2, the Ministry guidelines require as a minimum a shallow investigation to be carried out at each house site (similar to a normal NZS 3604 – type investigation), and as a minimum four test locations for each house site would be required. The Geotechnical Engineer may judge it appropriate to carry out deeper or more intense investigations than this, particularly for TC2-like land if the previous subdivision consent level of investigation indicated a high variability in the assessed liquefaction potential.’

The following guidance is given by the MBIE for TC3 type sites:

‘For building sites on TC3-like land, deep investigations and liquefaction assessments as outlined in ‘Interim Guidance for Repairing and Rebuilding Foundations in Technical Category 3’ (Appendix C to ‘Guidance on repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence’) should be initiated, as well as a shallow investigation as judged necessary by the Geotechnical Engineer’

Table C3.1 from Appendix C of this document (Table 5.2 of the ‘Guidance on repairing and rebuilding houses affected by the Canterbury earthquake’, dated December 2012’) gives guidance on the appropriate level of investigations required for various repair and rebuild scenarios within TC3 land, and is reproduced in Appendix C of this report.

We recommend that Table C3.1 be utilised as a guideline for assessing the level of investigations required for building consents relating to new dwellings and alterations to existing dwellings. Although this table was compiled for use in repairing and rebuilding foundations in Canterbury that were already damaged by liquefaction, the guidance for rebuilt foundations (i.e. the second half of the table) is considered suitable for the scoping of geotechnical investigations for building consent purposes in the Tahunanui Area.

These recommendations will also apply as a minimum for investigations for commercial or industrial developments although additional site -specific geotechnical investigations may be considered appropriate by the geotechnical engineer.

6.3 Property Title / LIM tags

In order to ensure that developers and potential purchasers are made aware of potential liquefaction risks NCC may consider placing an advisory note on any Land Information Memorandum (LIM) sought in respect of a property within the Study Area. An example of wording that may be placed on a LIM is:

“This property is situated an in area that has been identified as being underlain by soils that have the potential to liquefy during seismic shaking. Liquefaction induced by future significant seismic events is likely cause differential settlements that may result in damage to structures within the Study Area. Lateral spreading may also occur within up to 300 m from free water bodies (eg stream banks, coastline). All new foundations that are proposed within the Study Area should be investigated and designed by a Chartered Professional Engineer who specialises in the field of Geotechnical Engineering and should give due consideration to the guidelines for geotechnical investigations contained within “Guidance on repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence, dated December 2012, or any subsequent updated guidance published by MBIE. The design of any new foundations must be in accordance with current MBIE guidelines for TC3

(Technical Category) land and be such that they can accommodate future movement of the ground surface that is triggered by a significant seismic event."

6.4 Statutory requirements for future development

The Resource Management Act (1991) is currently under review, with changes likely in the near future as to how natural hazards are defined and addressed by the Act. These changes have been promulgated in response to the Canterbury earthquakes and will provide greater clarity to local bodies in fulfilling their obligations under the Act. Notwithstanding this NCC currently have functions under the Act to control the effects of the use of land for the avoidance or mitigation of natural hazards. Currently this is achieved via policy statements and provision in the District Plan. Subject to the amendments to the Act being sought there may be a shift from assessing natural hazard likelihood to assessing natural hazard risk.

NCC will need to consider the liquefaction risk within the framework of the revisions to the Act and this may impact on what specific District Plan provisions are likely to be required, and what specific works and restrictions will be needed at subdivision consent stage (Section 106).

NCC will also need to consider the liquefaction and lateral spreading risk in administering the Building Act. While these hazards are not specifically mentioned in the Act (Clause 71), compliance with the Building Code is required to satisfy the requirements of the Act. The Building Code (Section B1) requires all building works to be designed to accommodate the loads (including earthquake) that they are likely to experience throughout their lives without causing risk to life or loss of amenity. The site investigation recommendations provided by MBIE and foundation treatments set out in Section 6 above are considered a means by how compliance can be achieved with the requirements with the Building Code. However, where severe lateral spreading is risk is identified it may not be feasible to undertake works on a site by site basis that can demonstrate compliance with the Building Code.

6.5 Further investigations

Our preliminary assessment is based on the results of a MASW survey, 10 CPT tests, and two (2) boreholes, and it must be appreciated that actual conditions away from test locations may vary from those assumed here.

The classification of any land within the Study Area as being consistent with a TC3 level of seismic performance has potentially significant financial and insurance implications for land owners and stakeholders.

Our assessment indicates that there is a significant amount of variation in liquefaction potential within the Study Area. However, due to site investigation constraints there are significant areas where no CPT testing was carried out such as:

(i) the industrial area south of Quarantine Road, and;
(ii) the residential area east of Roto Street and north of Parkers Road.

The current intensity of geotechnical investigations does not allow sub-zoning of the liquefaction and lateral spreading risk within the Study Area. It is possible that there is a significantly smaller thickness of liquefiable sediments along the eastern margin of the Study Area, where groundwater tables are deeper, and sediments are less liquefiable.

We recommend that further geotechnical investigations and detailed engineering assessment be carried out by NCC prior to any statutory provisions being included in the

District Plan. This will allow a more robust zoning exercise to be carried out to more accurately assess areas of higher and lower liquefaction risk.

The following scope of work is recommended to allow the above:

- Obtain and review any additional available data e.g. from Nelson Airport.
- One (1) to two (2) days of CPT testing (6 to 12 tests) dependant on the level of sub-zoning that NCC would like to achieve.
- Installation and monitoring of piezometers in the CPT holes to provide information on the groundwater level across the site, and variation in summer and winter groundwater levels. A good time of year to carry out this work would be late winter (prior to September) when groundwater levels are likely to be at their highest.
- Assess the CPT results for estimated settlements under SLS and ULS events.
- Produce a zoning map in conjunction with NCC that differentiates areas of higher and lower liquefaction potential within the Study Area.
- Produce draft conditions for property files and provisions in the District Plan (including overlays) for approval by council.

7 Applicability

This report has been prepared for the benefit of Nelson City Council with respect to the particular brief given to us and it may not be relied upon in any other context or for any other purpose without our prior review and written agreement.

All recommendations and opinions which are contained in this report are based on subsurface data from a geophysical survey, boreholes and cone penetration tests. The nature and continuity of subsoil away from the test locations are inferred and it must be appreciated that actual conditions could vary from the assumed model.

All recommendations and opinions which are contained in this report are preliminary in nature and subject to confirmation by detailed geotechnical investigation and engineering assessment.

Mitigation options and a description of the associated residual risk are presented in this report to assist Nelson City Council to develop preliminary appropriate constraints on land development. This information is not of appropriate detail to enable detailed economic evaluation of a subdivision development or design of liquefaction mitigation measures.

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Tonkin & Taylor Ltd

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MJL

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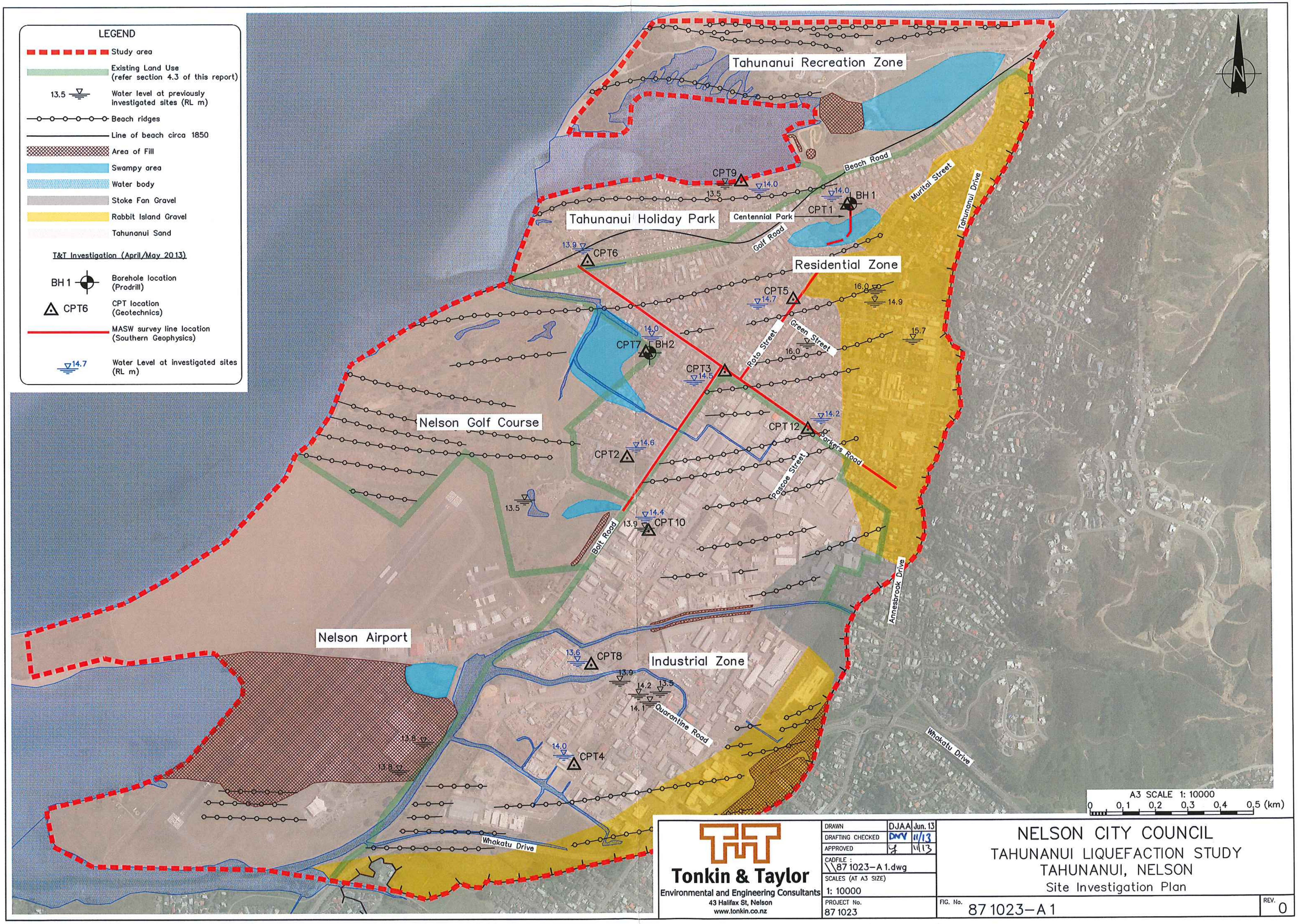
Appendix A: Tonkin & Taylor Figures

- Figure 871023-A1 – Site Investigation Plan
- Figure 871023-A2 - Liquefaction Settlement and Thickness
- Figure 871023-A3 – ULS Liquefaction Potential with Depth



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NELSON CITY COUNCIL
TAHUNANUI LIQUEFACTION STUDY
TAHUNANUI, NELSON
Site Investigation Plan

FIG. No. 871023-A1

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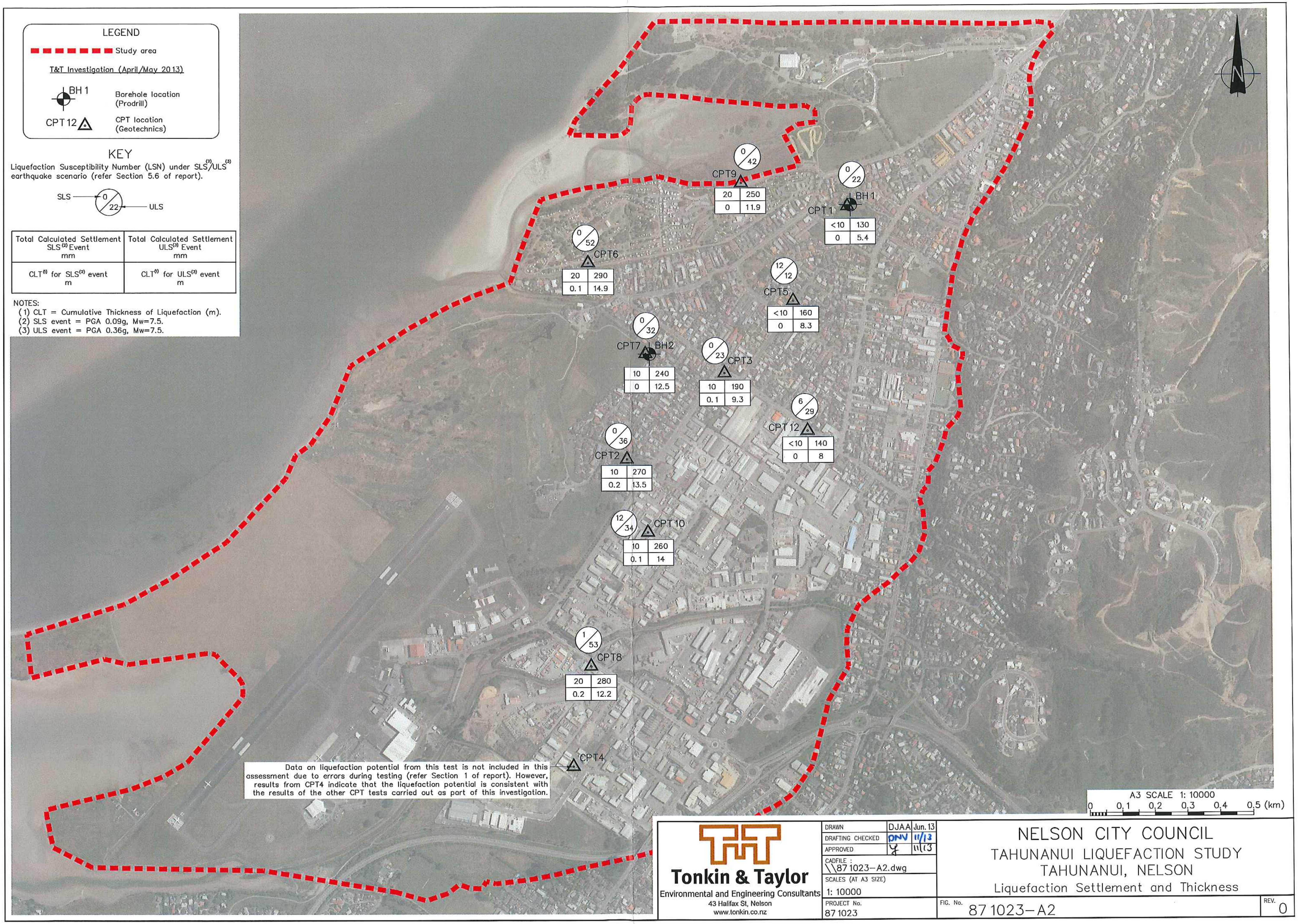

Data on liquefaction potential from this test is not included in this assessment due to errors during testing (refer Section 1 of report). However, results from CPT4 indicate that the liquefaction potential is consistent with the results of the other CPT tests carried out as part of this investigation.

LEGEND

- Study area

T&T Investigation (April/May 2013)

- BH 1 — Borehole location (Prodrill)
- CPT 12 — CPT location (Geotechnics)

KEY

Liquefaction Susceptibility Number (LSN) under SLS[2]/ULS[3] earthquake scenario (refer Section 5.6 of report).

SLS 0/22 ULS

Total Calculated Settlement SLS[2] Event mm	Total Calculated Settlement ULS[3] Event mm
CLT[1] for SLS[2] event m	CLT[1] for ULS[3] event m

NOTES:
(1) CLT = Cumulative Thickness of Liquefaction (m).
(2) SLS event = PGA 0.09g, Mw=7.5.
(3) ULS event = PGA 0.36g, Mw=7.5.

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NELSON CITY COUNCIL
TAHUNANUI LIQUEFACTION STUDY
TAHUNANUI, NELSON
Liquefaction Settlement and Thickness

FIG. No. 871023-A2
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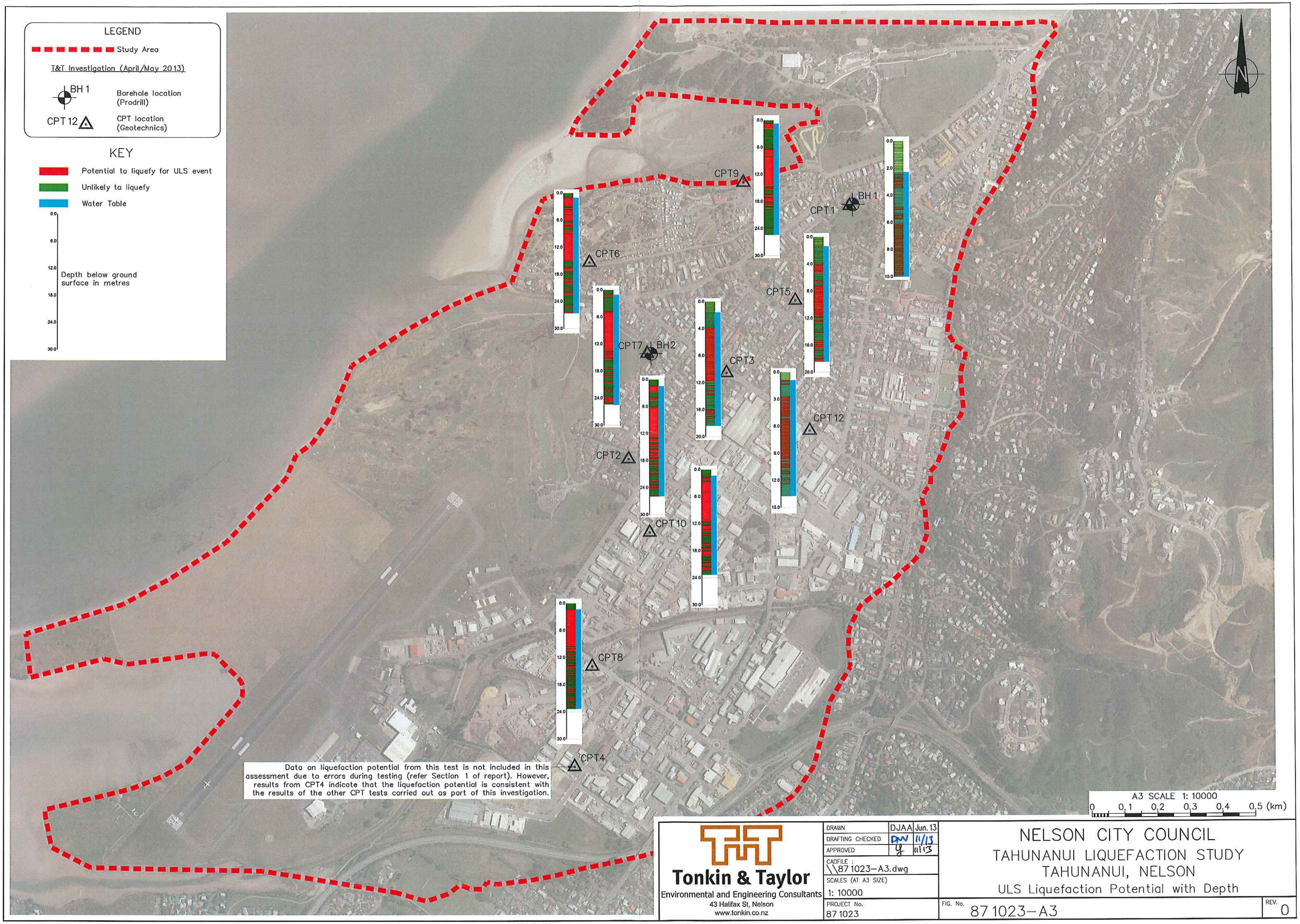

Data on liquefaction potential from this test is not included in this assessment due to errors during testing (refer Section 1 of report). However, results from CPT4 indicate that the liquefaction potential is consistent with the results of the other CPT tests carried out as part of this investigation.

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NELSON CITY COUNCIL
TAHUNANUI LIQUEFACTION STUDY
TAHUNANUI, NELSON
ULS Liquefaction Potential with Depth

FIG. No. 87 1023–A3

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Appendix B: Liquefaction Description

Liquefaction Description

B1 Process

The process of liquefaction is described by B1 and the commentary below.

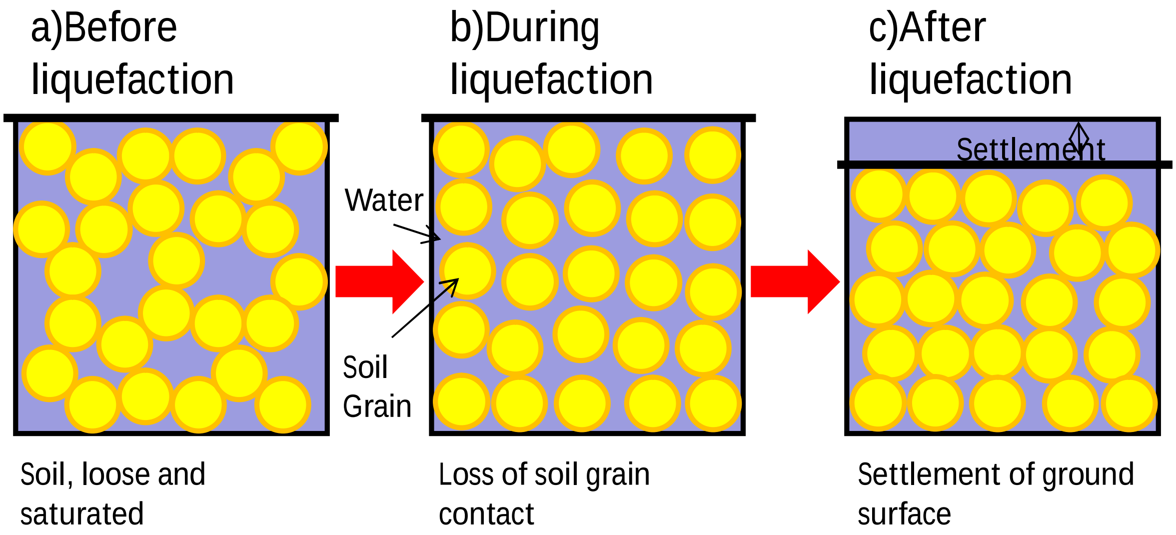


Figure B1: Liquefaction process

During Shaking

The support of the overlying ground is transferred from the soil grains to the water between the soil grains. The result is a large increase in water pressure and a loss of soil shear strength (i.e. it becomes like a viscous liquid).

After Shaking

The high water pressures result in water and soil escaping to the surface as sand boils (See Section B4.1). The soil grains reorient into a denser configuration. This densification in conjunction with the expulsion of soil and water to the surface, results in settlement.

B2 Susceptible soils

Liquefaction only occurs in some soils. Liquefaction susceptible soils are typically:

- Non-cohesive
- Loose to medium dense
- Saturated (beneath the water table)
- Not very high permeability.

In general:

- Sands and non-plastic silts are most susceptible to liquefaction
- Gravels can liquefy if they have a low permeability matrix or confining layers top and bottom
- Clays are too cohesive to liquefy.

The distinction between silts that are liquefiable or not are described as either being:

- "Sand-like behaviour" and therefore susceptible to liquefaction

- “Clay-like behaviour” and therefore not susceptible to liquefaction

The NZ Geotechnical Society “Guideline for the identification, assessment and mitigation of liquefaction hazards” (NZGS, 2010) provides further criteria for the assessment of liquefaction susceptible soils. Particular guidance is provided for fine grained soils (silts etc.).

B3 Triggering

The intensity and duration of earthquake shaking required to cause (trigger) liquefaction of susceptible soils (Refer Section B2) varies depending on the density and fines content of the soil. The likelihood (return period) of earthquake shaking triggering liquefaction is assessed by considering:

- The local seismic hazard. The likelihood (return period) of earthquakes of various duration (magnitude) and intensity (peak ground acceleration, PGA).
- Field penetration tests (CPT, and SPT – Standard Penetration Tests) and fines content results for the soil, and available empirical relationships between these results and the magnitude and PGA to trigger liquefaction.

B4 Liquefaction effects

There is a number of liquefaction effects each of which affect buildings and infrastructure differently. The risk of earthquake induced damage can be accepted, mitigated or avoided.

B4.1 Surface ejection of soil and water (Sand Boils)

Liquefied soils often release their water pressures to the surface. This is particularly evident where the crust of non-liquefied soil is relatively thin. This can result in water and soil being ejected to the surface. These are observed as “sand boils” or mini-volcanos. This flow of water and soil to the surface can damage floor slabs, pavements and services. Of the effects of liquefaction, sand boils is typically the most damaging to residential developments. It results in; uneven subsidence of the ground surface, and damage to buildings, paved surfaces and infrastructure.

Figure B2: Surface ejection of soil and water as sand boils

B4.2 Buoyancy

Increased groundwater pressures due to seismic shaking can cause buoyancy forces on structures and services. These forces, along with the reduced strength of liquefied soils can lead to uplift of pipes, manholes, chambers and swimming pools extending below the groundwater level.

Figure B3: Manhole uplifted due to buoyancy

B4.3 Bearing capacity failure

Liquefaction causes a loss of soil strength and stiffness resulting in reduced support (bearing capacity) to shallow foundations. This can result in subsidence of both shallow and deep foundations if the liquefiable layer is directly beneath the foundation.

B4.4 Lateral spreading

Lateral spreading is the displacement of the ground horizontally with shaking. Associated vertical displacement can also occur. Lateral spreading can occur on sloping or unrestrained ground (ground adjoining a river, foreshore or other free face). It is as a result of ground sliding on a liquefied layer during and possibly after (flow failure) the shaking. Sloping ground only needs to be very gentle for lateral spreading to occur. Lateral spreading in the order of meters can occur immediately adjoining a free face, and can be in the order of tens of millimetres at 100 m distance. However, in Christchurch where there was gently sloping ground back from the free face, displacements of hundreds of millimetres at 100 meters back from the free face were observed.

Figure B4: Lateral spreading showing horizontal and vertical displacement of land causing damage to structures

B4.5 Settlement

As discussed in Section B1, settlement can occur from densification of the liquefied soil layer and from expulsion of water and soil to the surface. Settlements of the ground surface are broken down into two components:

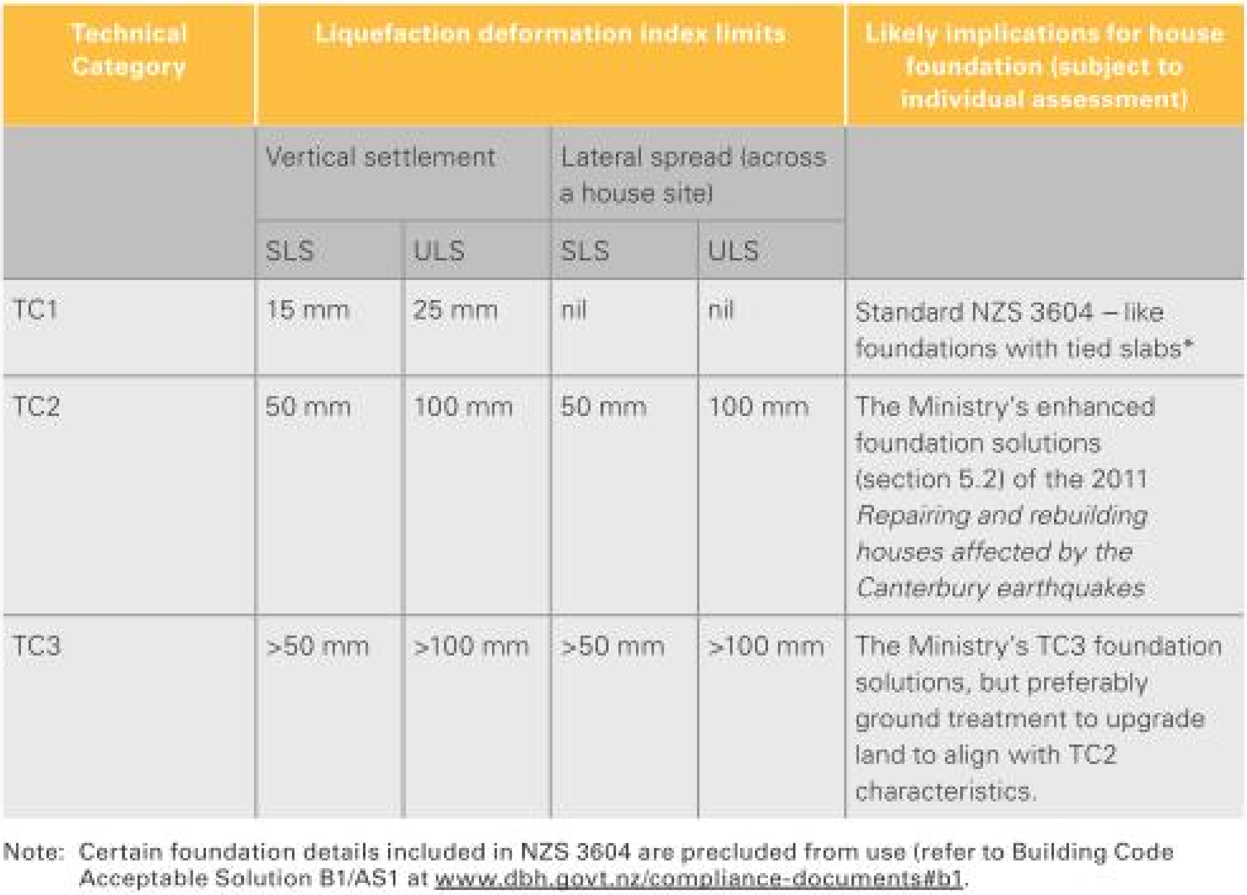
- Total settlement – General overall settlement of the area.
- Differential settlement – The difference in settlement between points within the area.
- Settlement of a few hundred millimetres can occur depending on the thickness, depth and density, of the liquefied layer. Settlement can cause damage to buildings and infrastructure.
- In Christchurch damage as a result of settlement was relatively small compared to that attributed to sand boils and loss of support to foundations.

Appendix C: Ministry of Business Innovation and Employment (MBIE) and Technical Guidance

C1 Broad Classification of Land

MBIE Guidance: Repairing and rebuilding houses affected by the Canterbury earthquakes, December 2012, Part O – Subdivisions table 16.1 broadly classifies land on the basis of assessed liquefaction deformations and types foundations (technical categories) required to address these deformations. Table 16.1 is reproduced below.

Table 16.1: Liquefaction deformation limits and house foundation implications



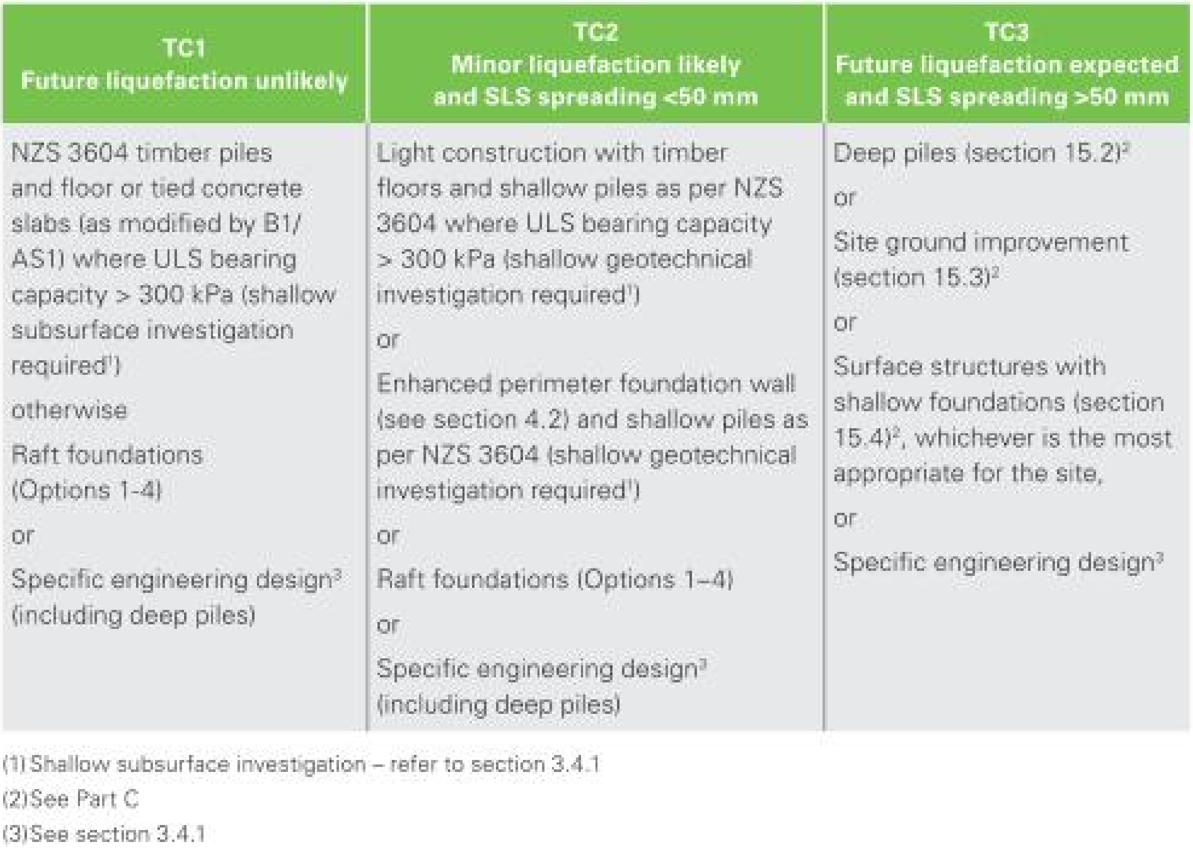
Technical Category	Liquefaction deformation index limits				Likely implications for house foundation (subject to individual assessment)
	Vertical settlement		Lateral spread (across a house site)		
	SLS	ULS	SLS	ULS	
TC1	15 mm	25 mm	nil	nil	Standard NZS 3604 – like foundations with tied slabs*
TC2	50 mm	100 mm	50 mm	100 mm	The Ministry's enhanced foundation solutions (section 5.2) of the 2011 *Repairing and rebuilding houses affected by the Canterbury earthquakes*
TC3	>50 mm	>100 mm	>50 mm	>100 mm	The Ministry's TC3 foundation solutions, but preferably ground treatment to upgrade land to align with TC2 characteristics.

Note: Certain foundation details included in NZS 3604 are precluded from use (refer to Building Code Acceptable Solution B1/AS1 at www.dbh.govt.nz/compliance-documents#b1.

C2 Technical Categories and Foundation Solutions

Part A of the MBIE guidance provides information on suitable foundations for the various technical categories. The descriptions of these suitable foundations are outlined in Part A Table 5.1 of the guidance which is produced below.

Table 5.1: Summary of proposed foundation solutions for rebuilt foundations or new foundations on the flat



TC1 **Future liquefaction unlikely**	**TC2** **Minor liquefaction likely and SLS spreading <50 mm**	**TC3** **Future liquefaction expected and SLS spreading >50 mm**
NZS 3604 timber piles and floor or tied concrete slabs (as modified by B1/AS1) where ULS bearing capacity > 300 kPa (shallow subsurface investigation required[1]) otherwise Raft foundations (Options 1-4) or Specific engineering design[3] (including deep piles)	Light construction with timber floors and shallow piles as per NZS 3604 where ULS bearing capacity > 300 kPa (shallow geotechnical investigation required[1]) or Enhanced perimeter foundation wall (see section 4.2) and shallow piles as per NZS 3604 (shallow geotechnical investigation required[1]) or Raft foundations (Options 1–4) or Specific engineering design[3] (including deep piles)	Deep piles (section 15.2)[2] or Site ground improvement (section 15.3)[2] or Surface structures with shallow foundations (section 15.4)[2], whichever is the most appropriate for the site, or Specific engineering design[3]

(1) Shallow subsurface investigation – refer to section 3.4.1

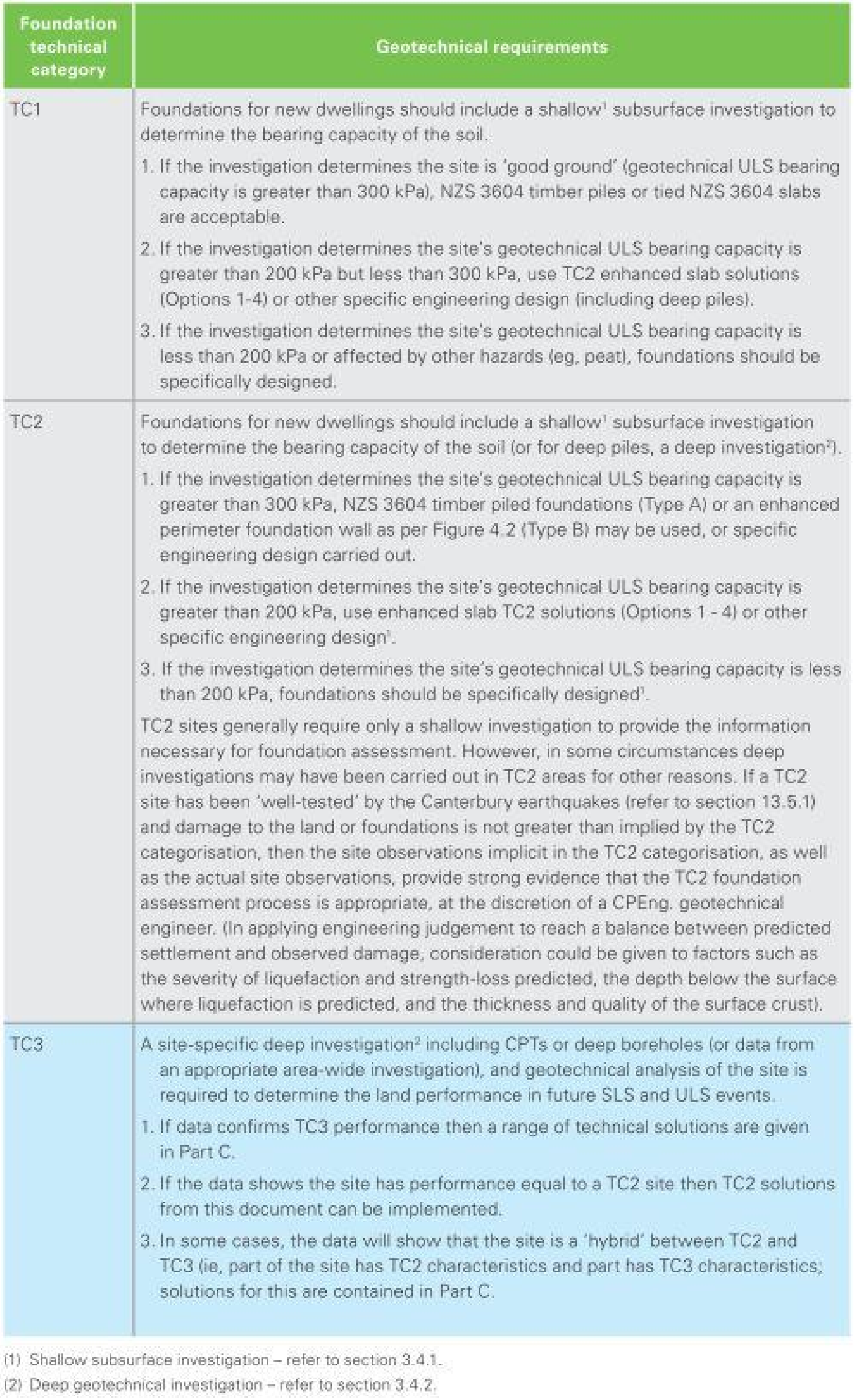
(2)See Part C

(3)See section 3.4.1

C3 Investigation requirements for TC3 Land

The MBIE guidance ‘Guidances: Repairing and rebuilding houses affected by the Canterbury earthquakes - Part A (dated December 2012) gives guidance on repairing and rebuilding foundations. Table 5.2 of the guidance which is reproduced below as Table C3.1 provides information on suitable investigation levels for various rebuilding and repair scenarios.

Table C3.1: Geotechnical requirements for rebuilt or new foundations on the flat



Foundation technical category	Geotechnical requirements
TC1	Foundations for new dwellings should include a shallow[1] subsurface investigation to determine the bearing capacity of the soil. 1. If the investigation determines the site is ‘good ground’ (geotechnical ULS bearing capacity is greater than 300 kPa), NZS 3604 timber piles or tied NZS 3604 slabs are acceptable. 2. If the investigation determines the site’s geotechnical ULS bearing capacity is greater than 200 kPa but less than 300 kPa, use TC2 enhanced slab solutions (Options 1-4) or other specific engineering design (including deep piles). 3. If the investigation determines the site’s geotechnical ULS bearing capacity is less than 200 kPa or affected by other hazards (eg, peat), foundations should be specifically designed.
TC2	Foundations for new dwellings should include a shallow[1] subsurface investigation to determine the bearing capacity of the soil (or for deep piles, a deep investigation[2]). 1. If the investigation determines the site’s geotechnical ULS bearing capacity is greater than 300 kPa, NZS 3604 timber piled foundations (Type A) or an enhanced perimeter foundation wall as per Figure 4.2 (Type B) may be used, or specific engineering design carried out. 2. If the investigation determines the site’s geotechnical ULS bearing capacity is greater than 200 kPa, use enhanced slab TC2 solutions (Options 1 - 4) or other specific engineering design[1]. 3. If the investigation determines the site’s geotechnical ULS bearing capacity is less than 200 kPa, foundations should be specifically designed[1]. TC2 sites generally require only a shallow investigation to provide the information necessary for foundation assessment. However, in some circumstances deep investigations may have been carried out in TC2 areas for other reasons. If a TC2 site has been ‘well-tested’ by the Canterbury earthquakes (refer to section 13.5.1) and damage to the land or foundations is not greater than implied by the TC2 categorisation, then the site observations implicit in the TC2 categorisation, as well as the actual site observations, provide strong evidence that the TC2 foundation assessment process is appropriate, at the discretion of a CPEng. geotechnical engineer. (In applying engineering judgement to reach a balance between predicted settlement and observed damage, consideration could be given to factors such as the severity of liquefaction and strength-loss predicted, the depth below the surface where liquefaction is predicted, and the thickness and quality of the surface crust).
TC3	A site-specific deep investigation[2] including CPTs or deep boreholes (or data from an appropriate area-wide investigation), and geotechnical analysis of the site is required to determine the land performance in future SLS and ULS events. 1. If data confirms TC3 performance then a range of technical solutions are given in Part C. 2. If the data shows the site has performance equal to a TC2 site then TC2 solutions from this document can be implemented. 3. In some cases, the data will show that the site is a ‘hybrid’ between TC2 and TC3 (ie, part of the site has TC2 characteristics and part has TC3 characteristics; solutions for this are contained in Part C.

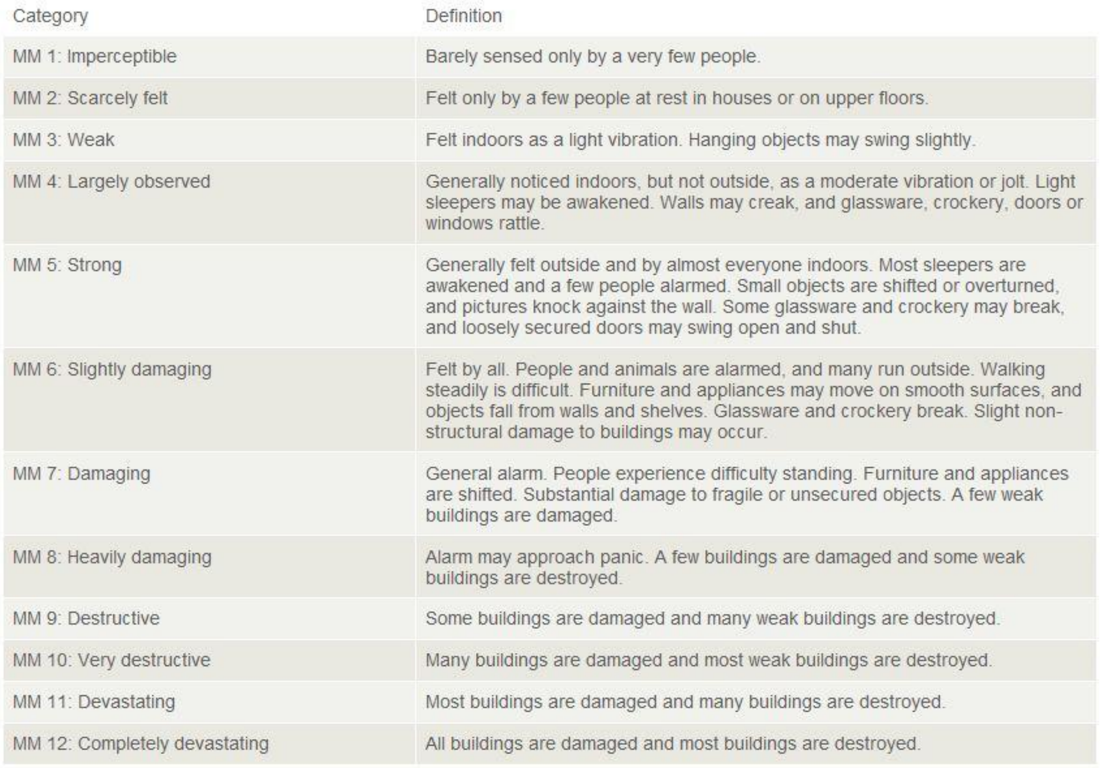
(1) Shallow subsurface investigation – refer to section 3.4.1.

(2) Deep geotechnical investigation – refer to section 3.4.2.

C4 Modified Mercalli Felt Intensity Scale

The modified Mercalli Scale for classifying earthquakes based on reports of felt intensities of shaking has been reproduced below. The descriptions in the table below have been adapted by GNS (Geological and Nuclear Sciences) to account for New Zealand conditions.

It is generally accepted that felt intensities of MM7 to MM8 are required to produce liquefaction.



Category	Definition
MM 1: Imperceptible	Barely sensed only by a very few people.
MM 2: Scarcely felt	Felt only by a few people at rest in houses or on upper floors.
MM 3: Weak	Felt indoors as a light vibration. Hanging objects may swing slightly.
MM 4: Largely observed	Generally noticed indoors, but not outside, as a moderate vibration or jolt. Light sleepers may be awakened. Walls may creak, and glassware, crockery, doors or windows rattle.
MM 5: Strong	Generally felt outside and by almost everyone indoors. Most sleepers are awakened and a few people alarmed. Small objects are shifted or overturned, and pictures knock against the wall. Some glassware and crockery may break, and loosely secured doors may swing open and shut.
MM 6: Slightly damaging	Felt by all. People and animals are alarmed, and many run outside. Walking steadily is difficult. Furniture and appliances may move on smooth surfaces, and objects fall from walls and shelves. Glassware and crockery break. Slight non-structural damage to buildings may occur.
MM 7: Damaging	General alarm. People experience difficulty standing. Furniture and appliances are shifted. Substantial damage to fragile or unsecured objects. A few weak buildings are damaged.
MM 8: Heavily damaging	Alarm may approach panic. A few buildings are damaged and some weak buildings are destroyed.
MM 9: Destructive	Some buildings are damaged and many weak buildings are destroyed.
MM 10: Very destructive	Many buildings are damaged and most weak buildings are destroyed.
MM 11: Devastating	Most buildings are damaged and many buildings are destroyed.
MM 12: Completely devastating	All buildings are damaged and most buildings are destroyed.

Appendix D: Investigation data

- Borehole logs - BH1 & BH2
- MASW Survey results
- CPT Liquefaction Assessment Results (CPT-1–CPT-3, CPT-5–CPT-10 & CPT 12)
- Engineering Terminology log sheet

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BOREHOLE LOG

BOREHOLE No:BH1
Hole Location: Refer site plan. Centennial Park.
SHEET 1 OF 3

PROJECT: Tahuna Liquefaction
LOCATION: Nelson
JOB No: 871023

CO-ORDINATES 529670.62 mN
1620420.96 mE
R.L. 16.40 m
DATUM NZTM/NZGD 2000

DRILL TYPE: Sonic Rig
DRILL METHOD: SONIC
DRILL FLUID: None

HOLE STARTED: 30/4/13
HOLE FINISHED: 30/4/13
DRILLED BY: Prodrill Ltd
LOGGED BY: FAW
CHECKED: myl

GEOLOGICAL — GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.

ENGINEERING DESCRIPTION — SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.

DEPTH (m)	GEOLOGICAL UNIT	WATER	CORE RECOVERY (%)	METHOD	CLASSIFICATION SYMBOL	MOISTURE CONDITION	STRENGTH/DENSITY CLASSIFICATION	SOIL DESCRIPTION
0.0	TOPSOIL		100	SONIC DRILLING	ML	M	S	0.0-0.08m: Organic SILT with some sand and roots [Topsoil]
0.08	FILL						F	0.08m: SILT with some gravel and clay, minor sand; grey brown with orange mottle. Firm, moist to wet, low palsticity.
0.7	BURIED TOPSOIL FILL				GM		L	0.7m: Buried TOPSOIL remnant. 0.7-1.2m: Silty GRAVEL with minor clay; brown. Loose, moist, non-plastic.
1.2					ML		F	1.2-1.5m: SILT with some gravel and clay; brown. Firm, moist, low plasticity.
1.5					GM			1.5-1.9m: Silty GRAVEL with some clay, minor cobbles; brown. Firm, moist, non-plastic.
1.9	UNKNOWN - CORE LOSS. FILL?	31/4/13-10.00am	0	SONIC				1.9-3.0m: Core loss, cobble blocked core barrel.
3.0	ESTUARINE DEPOSIT		100	SONIC DRILLING	GP	S	L	3.0m: Fine to medium SAND with rare gravel; grey. Loose, saturated, non-plastic, dilatant (rare shell fragments).
4.4					SP GP SP			4.4-4.5m: Lens of fine to medium SAND with some gravel, minor shell fragments; grey. Loose, saturated, non-plastic, dilatant. Gravel is fine to coarse, sub-rounded clasts. 4.6m: Fine SAND with minor to some silt, rare gravel; grey. Loose, saturated, non-plastic, dilatant. Rare shell fragments.
7.0					SM			7.0m: Silty fine SAND, rare gravel and shell fragments; grey. Loose, saturated, non-plastic, dilatant.
8.4					SM		F	8.4m: Sandy SILT, rare shell fragments; grey. Firm, saturated, non-plastic, dilatant.
9.25	PORT HILLS FORMATION DERIVED - Marginal Marine Deposit.				GM		D	9.25m: 100mm Cobble Gravelly SILT with some sand; greenish grey. Dense, moist (saturated), non-plastic.

Log Scale 1:50

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BOREHOLE LOG

BOREHOLE No:BH1
Hole Location: Refer site plan.
Centennial Park.

SHEET 2 OF 3

PROJECT: Tahuna Liquefaction
LOCATION: Nelson
JOB No: 871023

CO-ORDINATES 529670.62 mN
1620420.96 mE
R.L. 16.40 m
DATUM NZTM/NZGD 2000

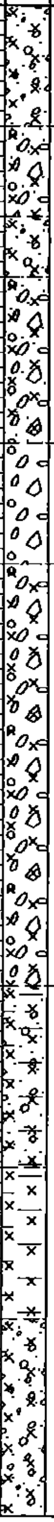
DRILL TYPE: Sonic Rig
DRILL METHOD: SONIC
DRILL FLUID: None

HOLE STARTED: 30/4/13
HOLE FINISHED: 30/4/13
DRILLED BY: Prodrill Ltd
LOGGED BY: FAW
CHECKED: mp

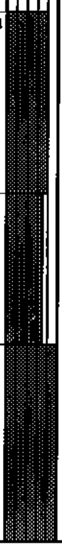
GEOLOGICAL									ENGINEERING DESCRIPTION						
GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.		
PORT HILLS FORMATION DERIVED - Marginal Marine Deposit.			100	SONIC DRILLING						SM	S	D	Sandy SILT, as above.		
									10.8	GM			10.8m: Silty GRAVEL with some sand, minor clay; greenish grey. Dense, moist (saturated), non-plastic.		
									11.4			L	11.4-11.8m : Zone of saturated, softened core, drill disturbed? Loose, saturated, low plasticity (Gravelly SILT with some clay).		
									11.8			D			
									12.2				12.2-12.9m: Core disturbed, too dense for sonic rig.		
									12.9	GP			12.9-13.2m: GRAVEL (possible drilling disturbance).		
									13.5			L/MD	13.5-13.7m: Loose to medium dense.		
									13.7	GM		D	13.7m: Silty GRAVEL to gravelly SILT with minor to some sand; greenish grey. Dense, moist (saturated), non-plastic.		
									15				15-15.3m: Core drilling disturbed.		
									15.5				15.5-16.5m: Core disturbed. Fried by sonic rig - too dense?		
									16.5	GM		St/VSt	16.5m: SILT with minor to some gravel, rare coal fragments, minor sand and clay; dark grey. Stiff to very stiff, moist (saturated), non-plastic (Swamp Derived Port Hills Gravel?)		
									17.7			St	17.7m: SILT with minor clay; dark grey. Stiff, moist (saturated), non-plastic.		
									18.7			VSt/H	18.7-21.0m: Gravelly SILT with some sand; green grey. Very stiff to hard, moist (saturated), non-plastic.		
									19.2				19.2m: Disturbed core, saturated, loose to medium dense?		

R.L. (m): 6, 5, 4, 3, 2, 1, 0, -1, -2, -3
DEPTH (m): 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

GRAPHIC LOG



SHEAR STRENGTH (kPa) 10 25 50 100 200
COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250
DEFECT SPACING (mm) 50 250 1000 2000



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BOREHOLE LOG

BOREHOLE No:BH1
Hole Location: Refer site plan.
Centennial Park.
SHEET 3 OF 3

PROJECT: Tahuna Liquefaction	LOCATION: Nelson	JOB No: 871023
CO-ORDINATES 529670.62 mN 1620420.96 mE	DRILL TYPE: Sonic Rig	HOLE STARTED: 30/4/13
	DRILL METHOD: SONIC	HOLE FINISHED: 30/4/13
R.L. 16.40 m		DRILLED BY: Prodrill Ltd
DATUM NZTM/NZGD 2000	DRILL FLUID: None	LOGGED BY: FAW CHECKED:

GEOLOGICAL											ENGINEERING DESCRIPTION						
GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250	DEFECT SPACING (mm) 50 250 1000 2000	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
PORT HILLS FORMATION DERIVED - Marginal Marine Deposit.								-4			GM	S	VSt/H				18.7-21.0m: Gravelly SILT with some sand; green grey. Very stiff to hard, moist (saturated), non-plastic. Gravelly SILT, as above.
									21								END OF BOREHOLE AT 21.0m.
								-5									
									22								
								-6									
									23								
								-7									
									24								
								-8									
									25								
								-9									
									26								
								-10									
									27								
								-11									
									28								
								-12									
									29								
								-13									
									30								

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BOREHOLE LOG

BOREHOLE No:BH2
Hole Location: Refer site plan.
Nelson Golf Course.
SHEET 1 OF 3

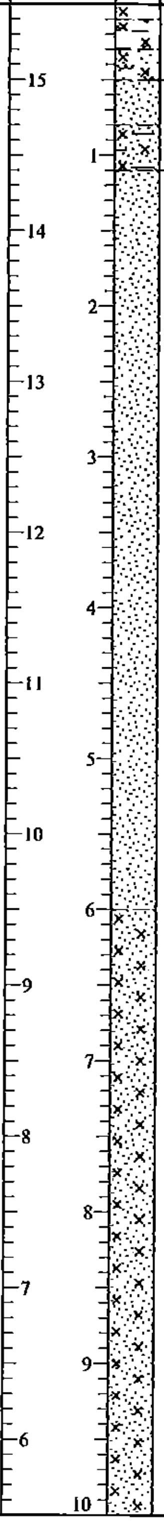
PROJECT: Tahuna Liquefaction		LOCATION: Nelson	JOB No: 871023
CO-ORDINATES	5429215.42 mN 1619797.38 mE	DRILL TYPE: Sonic Rig	HOLE STARTED: 30/4/13
		DRILL METHOD: SONIC	HOLE FINISHED: 1/5/13
R.L.	15.50 m		DRILLED BY: Prodrill Ltd
DATUM	NZTM/NZGD 2000	DRILL FLUID: None	LOGGED BY: FAW CHECKED:

GEOLOGICAL — **ENGINEERING DESCRIPTION**

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION. | FLUID LOSS | WATER | CORE RECOVERY (%) | METHOD | CASING | TESTS | SAMPLES | R.L (m) | DEPTH (m) | GRAPHIC LOG | CLASSIFICATION SYMBOL | MOISTURE CONDITION / WEATHERING | STRENGTH/DENSITY CLASSIFICATION | SHEAR STRENGTH (kPa) 10 25 50 100 200 | COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250 | DEFECT SPACING (mm) 50 250 1000 2000

SOIL DESCRIPTION
Soil type, minor components, plasticity or particle size, colour.

ROCK DESCRIPTION
Substance: Rock type, particle size, colour, minor components.
Defects: Type, inclination, thickness, roughness, filling.



Geological unit	Classification symbol	Moisture condition	Strength/density
TOPSOIL	ML	M	S
FILL			
BURIED TOPSOIL			F
BEACH SAND DEPOSIT	SM		
TAHUNANUI SANDS - ESTUARINE DEPOSIT	SW	S	L

WATER: 2/5/13-8.00am

CORE RECOVERY (%): 100

METHOD: SONIC DRILLING

0.0-0.1m: Organic SILT. Soft, wet [Topsoil]

0.1-0.3m: Clayey SILT with some gravel; light brown. Soft, wet, low plasticity.

0.3-0.5m: Sandy SILT with some organics; dark brown. Firm, moist, non-plastic. Sand is fine.

0.5-0.8m: Fine SAND; gold brown. Loose, moist. [Beach deposit]

0.8-1.1m: Silty fine SAND with some clay; grey brown. Loose, moist, low plasticity.

1.1-1.5m: Fine SAND; light grey. Loose, saturated, non-plastic, dilatant.

1.5m: Fine to medium SAND; grey. Loose, saturated, non-plastic, dilatant.

6m: Fine SAND with minor to some silt, with rare shell fragments; dark grey. Loose, saturated, non-plastic, dilatant.

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BOREHOLE LOG

BOREHOLE No:BH2
Hole Location: Refer site plan.
Nelson Golf Course.
SHEET 2 OF 3

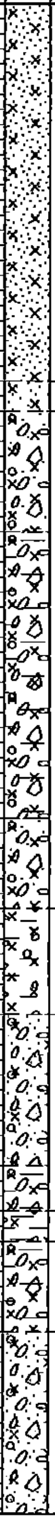
PROJECT: Tahuna Liquefaction
LOCATION: Nelson
JOB No: 871023

CO-ORDINATES 5429215.42 mN
1619797.38 mE
R.L. 15.50 m
DATUM NZTM/NZGD 2000

DRILL TYPE: Sonic Rig
DRILL METHOD: SONIC
DRILL FLUID: None

HOLE STARTED: 30/4/13
HOLE FINISHED: 1/5/13
DRILLED BY: Prodrill Ltd
LOGGED BY: FAW
CHECKED: mp

CORE RECOVERY (%): 100
METHOD: SONIC DRILLING



GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	CLASSIFICATION SYMBOL	MOISTURE CONDITION	STRENGTH/DENSITY CLASSIFICATION	SOIL DESCRIPTION
TAHUNANUI SANDS - ESTUARINE DEPOSIT		S	L	Fine SAND, as above.
				10.5m: Becoming silty fine SAND; grey. Loose, saturated, non-plastic, dilatant.
	SM		S/F	12.5-12.7m: Sandy SILT; grey. Soft to firm, saturated, non-plastic, dilatant.Sand is fine.
MARGINAL ESTUARINE DELTA DEPOSIT	GM		L	12.7-13.5m: Silty, fine to coarse GRAVEL with some sand, minor clay; grey. Loose, saturated. Gravel is sub-angular to angular; sand is fine.
DELTAIC MARGINAL ESTUARINE DEPOSIT			D	13.5m: Silty GRAVEL with minor to some clay; green grey/brown. Dense, saturated, non-plastic.
			L/MD	14.3-15.4m: Loose to medium dense?
			L	15.4-16.0m: Silty, sandy GRAVEL with minor clay; grey. Loose, saturated, non-plastic.
	ML		VSt	16.0m: SILT with some clay, minor gravel; grey. Very stiff, wet (saturated), low plasticity [Estuarine deposit?]
	GM		L	16.7-17.7m: Sandy GRAVEL with minor clay and silt; brown grey. Loose, saturated, non-plastic.
			L/MD	17.7-18.8m: Silty GRAVEL with minor clay and sand; brown grey. Loose to medium dense, saturated, non-plastic.
	CL			18.0-18.2m: Gravelly, silty CLAY; light grey with orange mottle. Very stiff, saturated, low to moderate plasticity.
	GM			
	GM			18.8-20.0m: Silty, sandy GRAVEL with minor clay; grey brown. Loose to medium dense, saturated, non-plastic.

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BOREHOLE LOG

BOREHOLE No:BH2
Hole Location: Refer site plan.
Nelson Golf Course.
SHEET 3 OF 3

PROJECT: Tahuna Liquefaction	LOCATION: Nelson		JOB No: 871023
CO-ORDINATES 5429215.42 mN 1619797.38 mE	DRILL TYPE: Sonic Rig	HOLE STARTED: 30/4/13	
	DRILL METHOD: SONIC	HOLE FINISHED: 1/5/13	
R.L. 15.50 m		DRILLED BY: Prodrill Ltd	
DATUM NZTM/NZGD 2000	DRILL FLUID: None	LOGGED BY: FAW	CHECKED: MJL

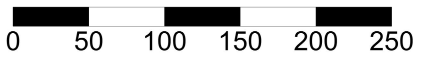
GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SOIL DESCRIPTION Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
DELTAIC MARGINAL ESTUARINE DEPOSIT			100	SONIC DRILLING					20.0	ML	S	VSt	20.0m: SILT with some clay; blue grey mottled orange. Very stiff, moist (saturated), non-plastic.
								-5					
									21				21.0m: Rare gravel.
								-6					
									21.9	MH		St	21.9m: Clayey SILT; blue grey mottled orange. Stiff, moist (saturated), low plasticity.
									22.4	GM		D	22.4-22.5m: Silty GRAVEL with some clay; brown. Medium dense, wet, non-plastic.
								-7	22.5				END OF BOREHOLE AT 22.5m.
									23				
								-8					
									24				
								-9					
									25				
								-10					
									26				
								-11					
									27				
								-12					
									28				
								-13					
									29				
								-14					
									30				

GRAPHIC LOG; SHEAR STRENGTH (kPa) 10 25 50 100 200; COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250; DEFECT SPACING (mm) 60 250 600 1000 2000

T+T DATATEMPLATE.GDT David

Log Scale 1:50

		CHAINAGE (max. segment length 200m):
MASW LINE:	1	
SEGMENTS:	1a	0m - 91.2m
	CENTENNIAL ROAD	91.2m - 139.2m
	1b	139.2m - 199.2m
MASW LINE:	2	
SEGMENTS:	2a	0m - 147.2m
	GREEN STREET	147.2m - 196.4m
	2b	196.4m - 387.4m
MASW LINE:	3	
SEGMENTS:	3a	0m - 130.3m
	GOLF ROAD	130.3m - 193.9m
	3b	193.9m - 393.9m
	3c	393.9m - 557.8m
	ROTO STREET	557.8m - 613.5m
	3d	613.5m - 813.5m
	3e	813.5m - 917.5m
	MURITAI STREET	917.5m - 981.2m
	3f	981.2m - 1165.5m
MASW LINE:	4	
SEGMENTS:	4a*	0m - 200m
	4b	200m - 400m
	4c	400m - 485m
	*SEGMENT 4a includes data gap over Cohen Place	



SCALE (m)

N

TITLE- **MASW LINE LOCATIONS**

LOCATION- **TAHUNA, NELSON**

NOTES- Coordinates: NZ2000 TM Grid.
Background: LINZ Topo50 map
Line labels show the chainage along the line.
Each point represents the middle of the 24 channel array.

A3

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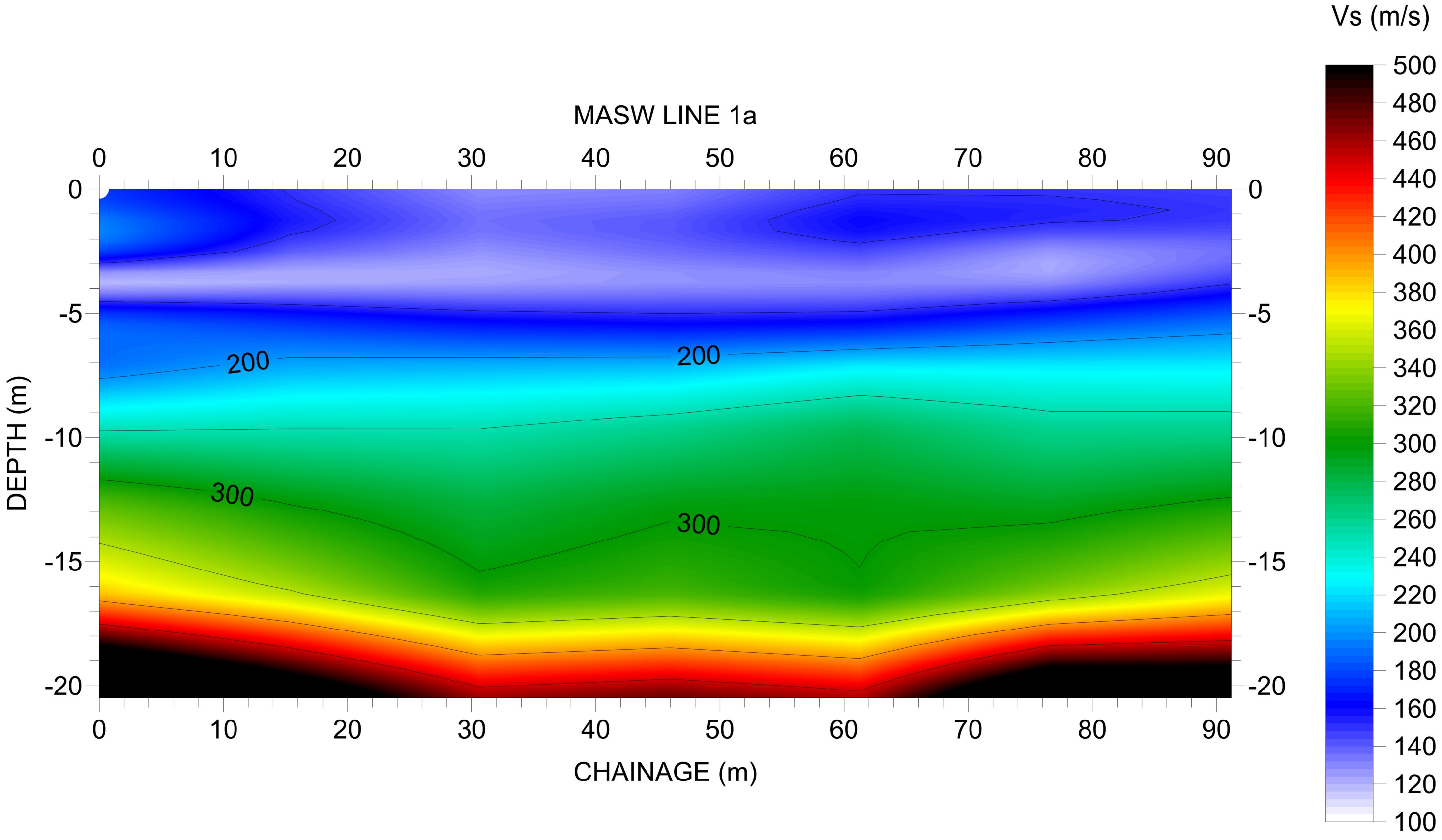

TITLE- **MASW LINE 1a (0m - 91.2m)**

LOCATION- **Tahuna, Nelson**

NOTES
Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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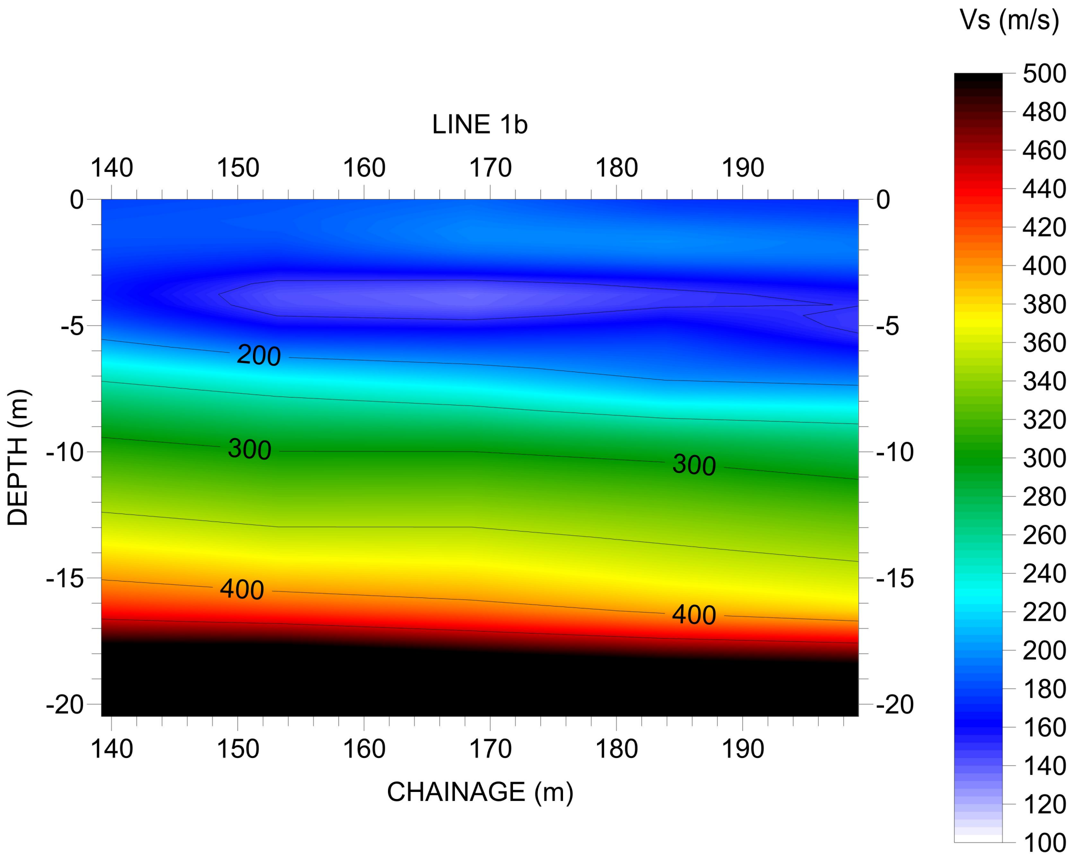

TITLE- **MASW LINE 1b (139.2m - 199.2m)**

LOCATION- **Tahuna, Nelson**

NOTES
Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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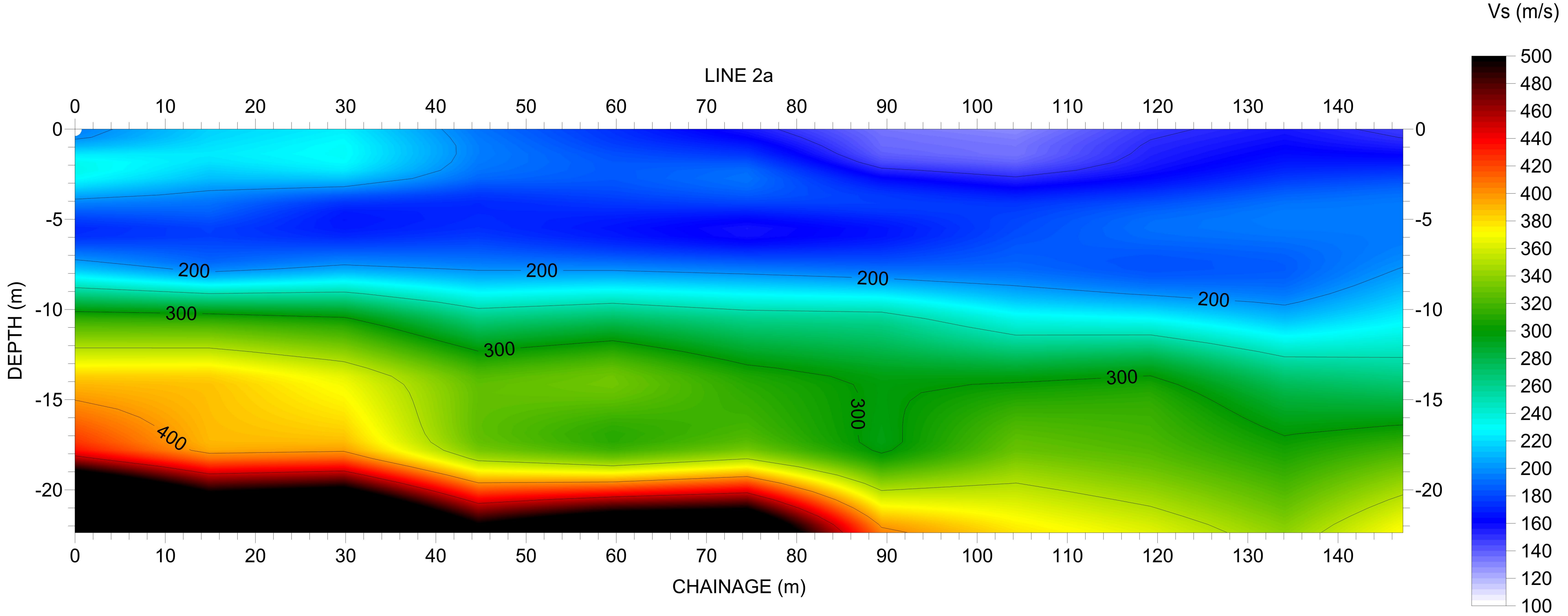

TITLE- **MASW LINE 2a (0m - 147.2m)**

LOCATION- **Tahuna, Nelson**

NOTES

Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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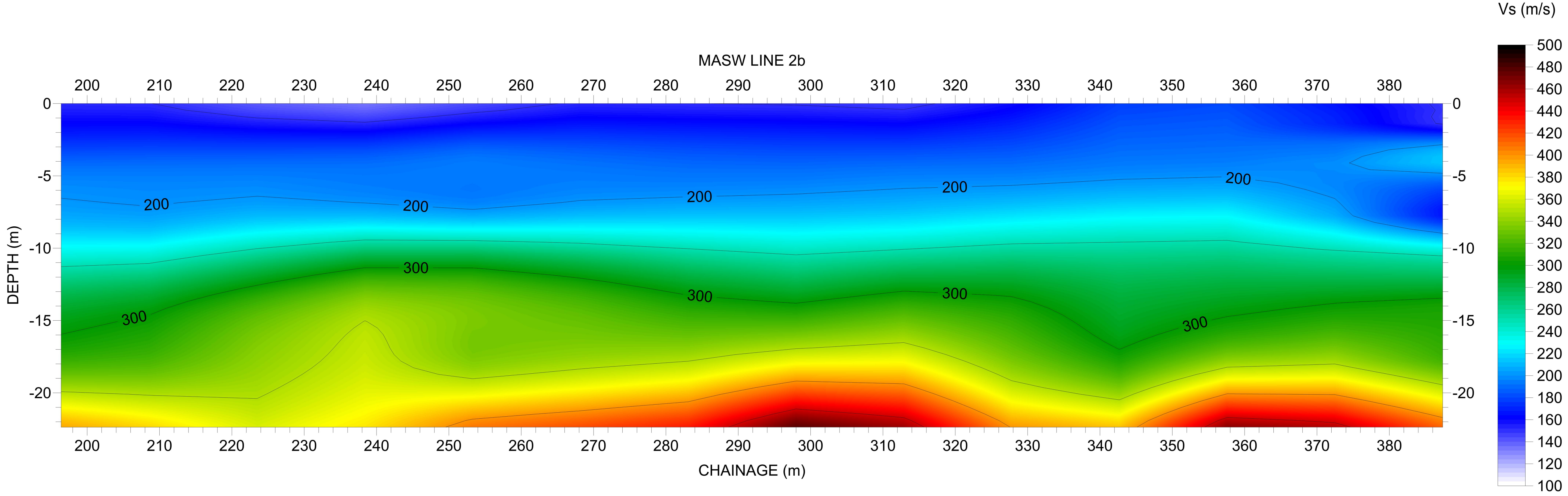

TITLE- **MASW LINE 2b (196.4m - 387.4m)**

LOCATION- **Tahuna, Nelson**

NOTES
Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

Southern Geophysical Ltd
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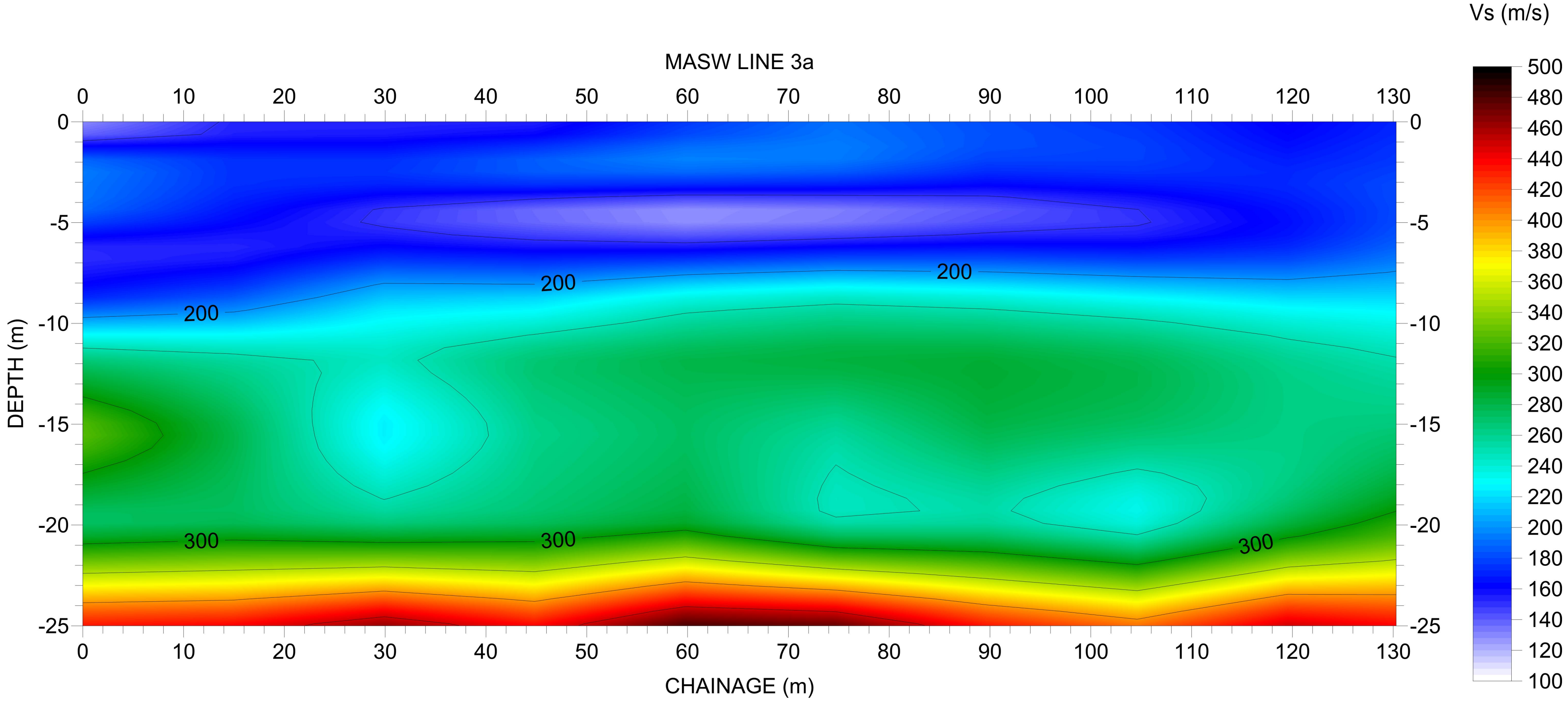

TITLE- **MASW LINE 3a (0m - 130.3m)**

LOCATION- **Tahuna, Nelson**

NOTES Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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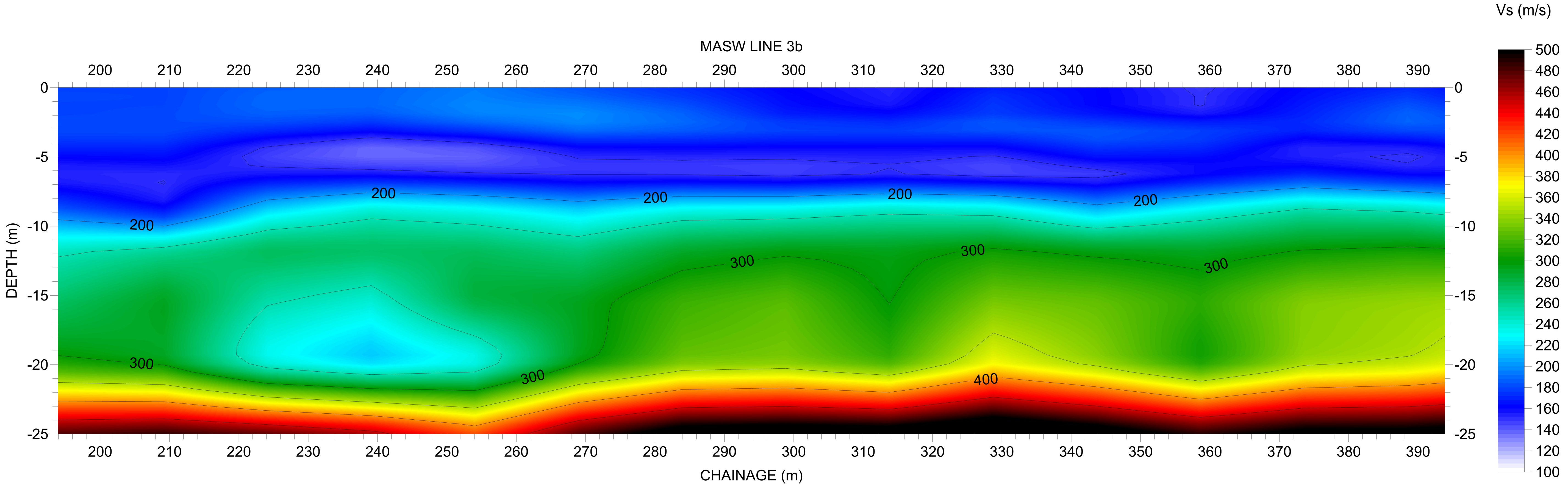

TITLE- **MASW LINE 3b (193.9m - 393.9m)**

LOCATION- **Tahuna, Nelson**

NOTES

Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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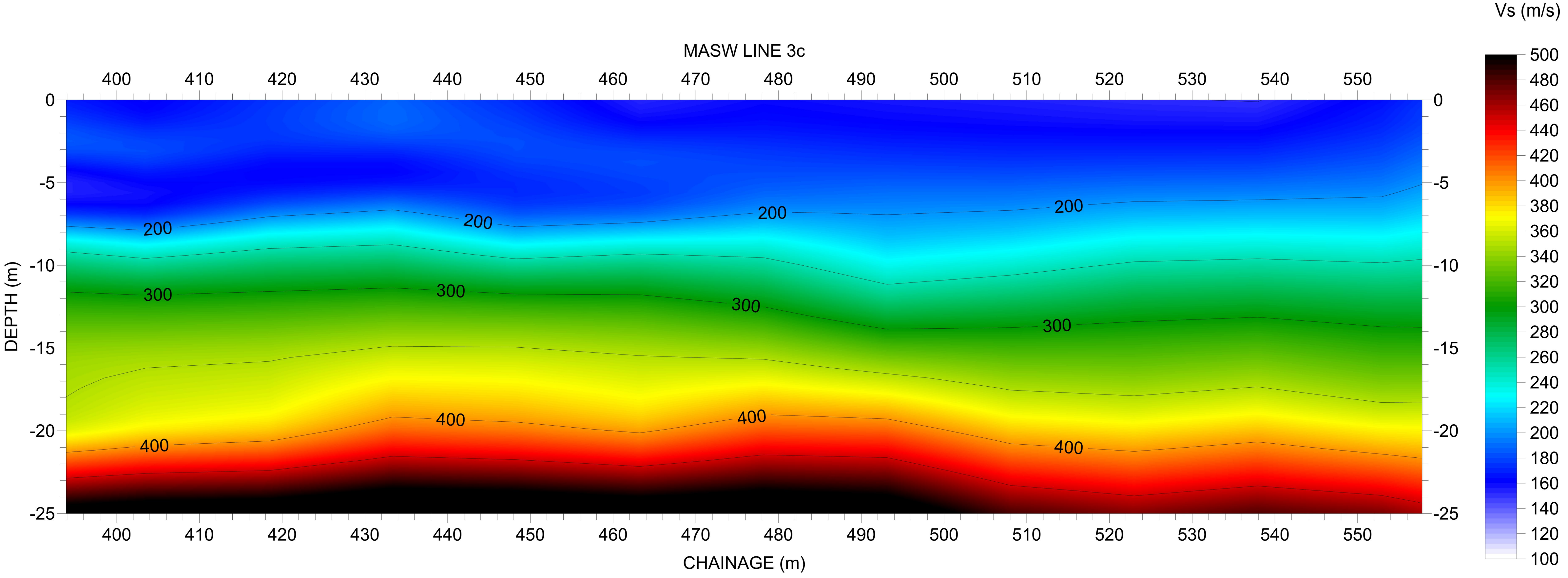

TITLE-	**MASW LINE 3c (393.9m - 557.8m)**
LOCATION-	**Tahuna, Nelson**

NOTES

Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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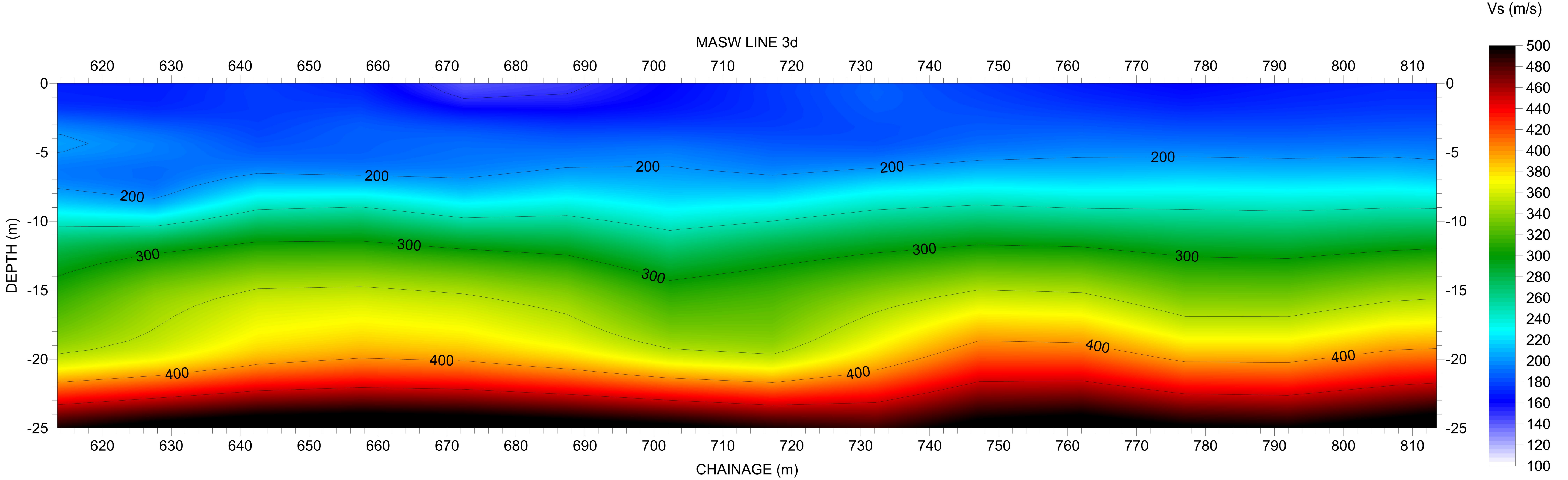

TITLE- **MASW LINE 3d (613.5m - 813.5m)**

LOCATION- **Tahuna, Nelson**

NOTES

Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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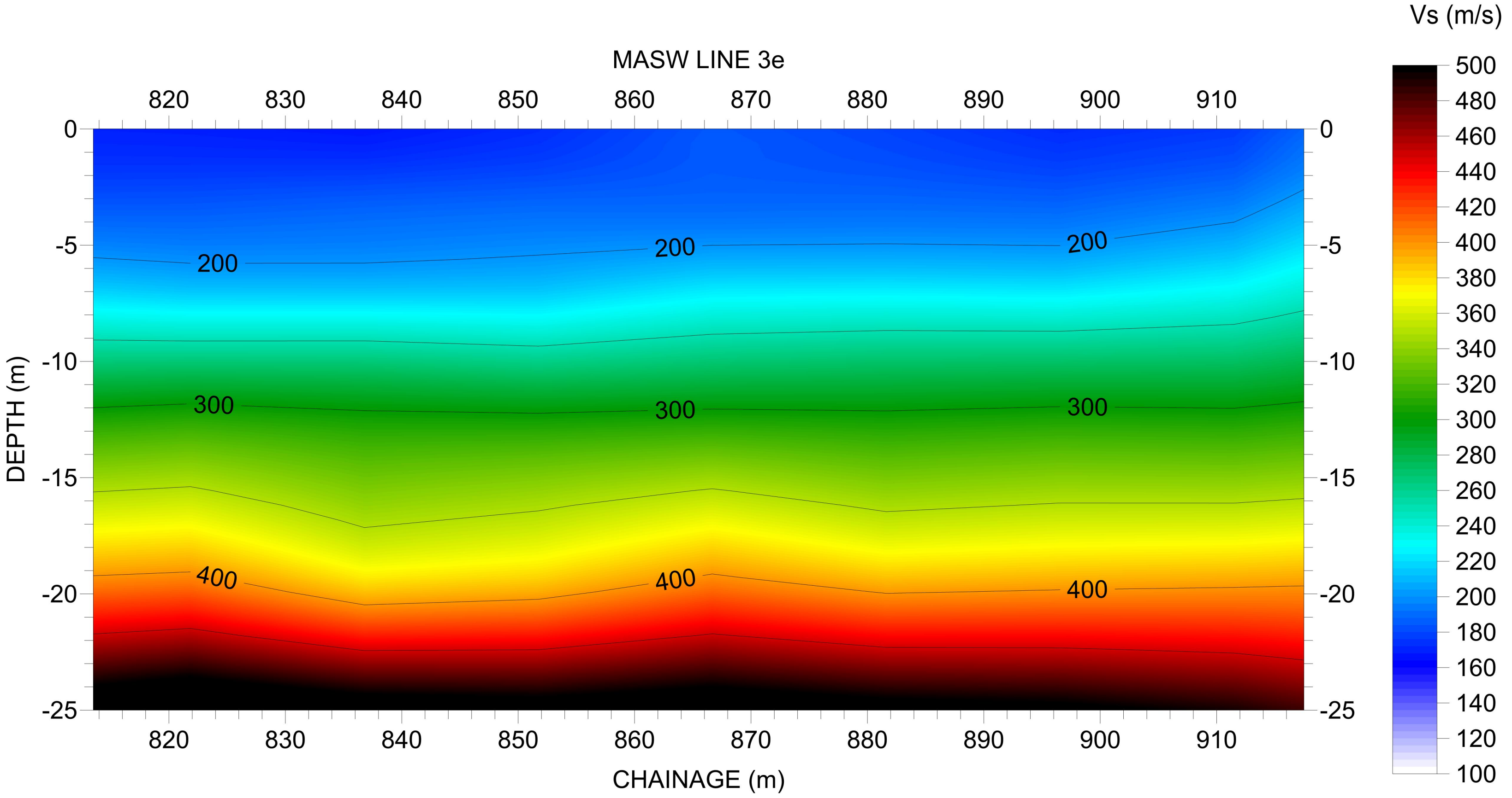

TITLE- **MASW LINE 3e (813.5m - 917.5m)**

LOCATION- **Tahuna, Nelson**

NOTES Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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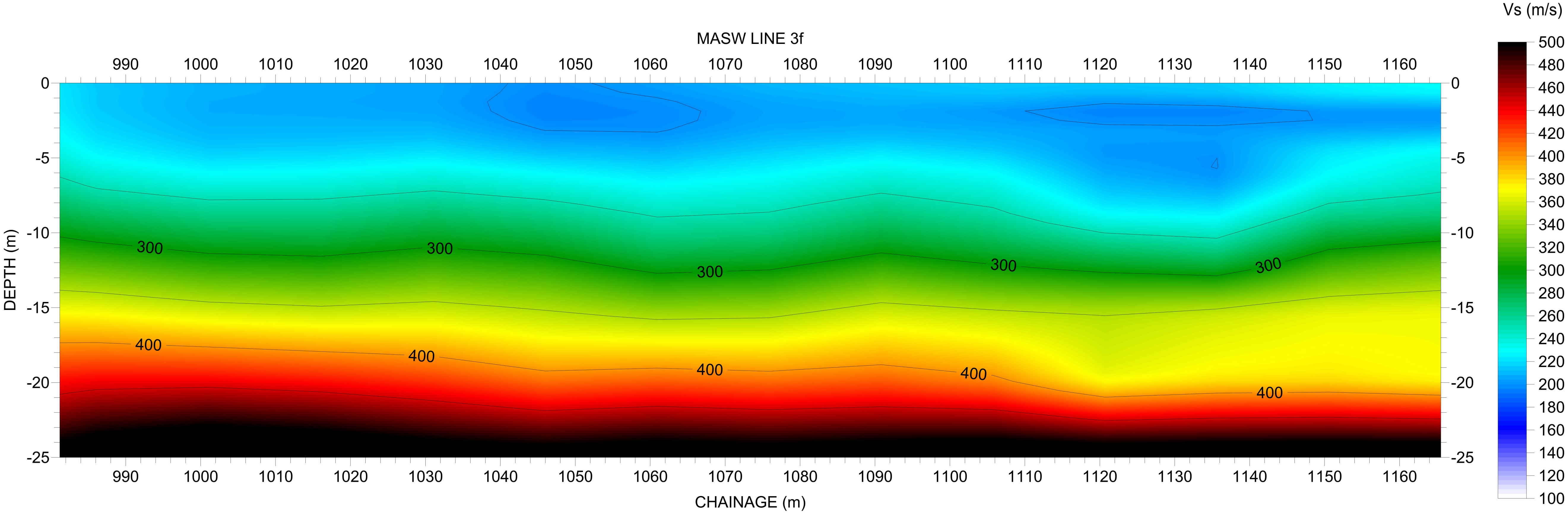

TITLE-	**MASW LINE 3f (981.2m - 1165.5m)**	
LOCATION-	**Tahuna, Nelson**	

NOTES

Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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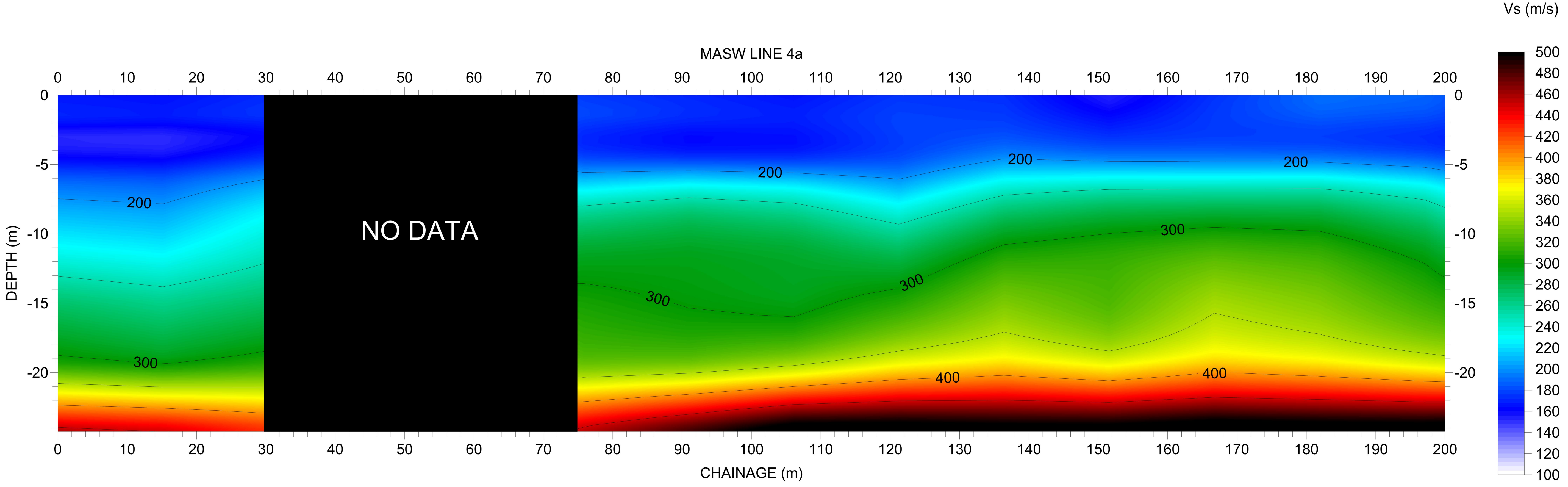

TITLE- **MASW LINE 4a (0m - 200m)**

LOCATION- **Tahuna, Nelson**

NOTES
Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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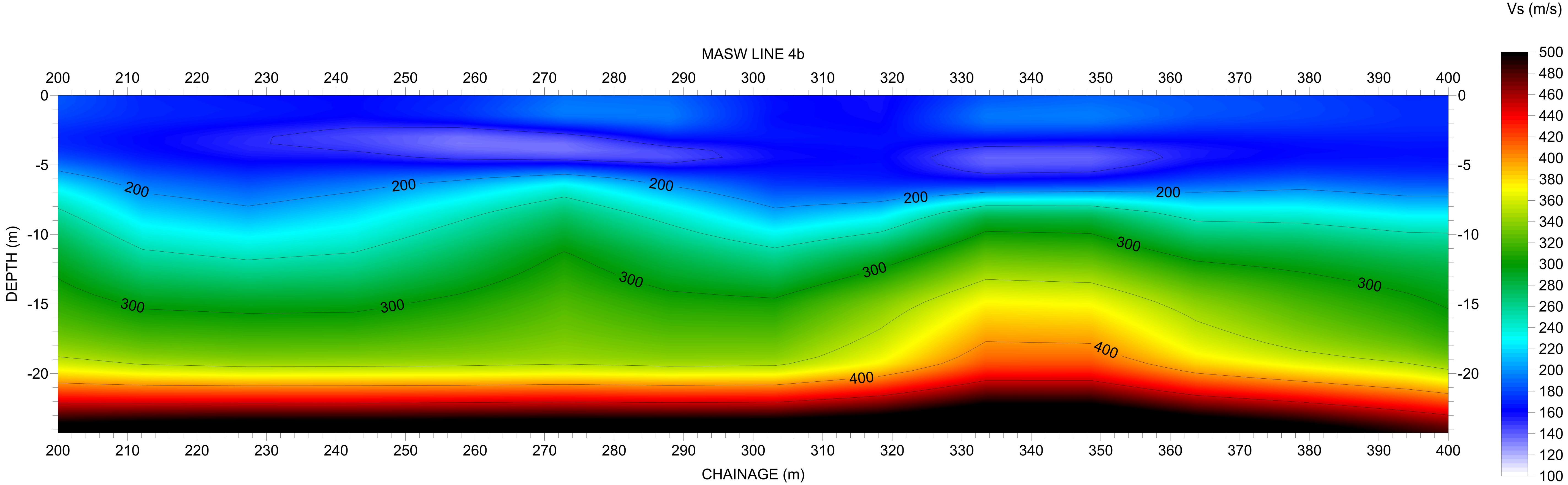

TITLE- **MASW LINE 4b (200m - 400m)**

LOCATION- **Tahuna, Nelson**

NOTES

Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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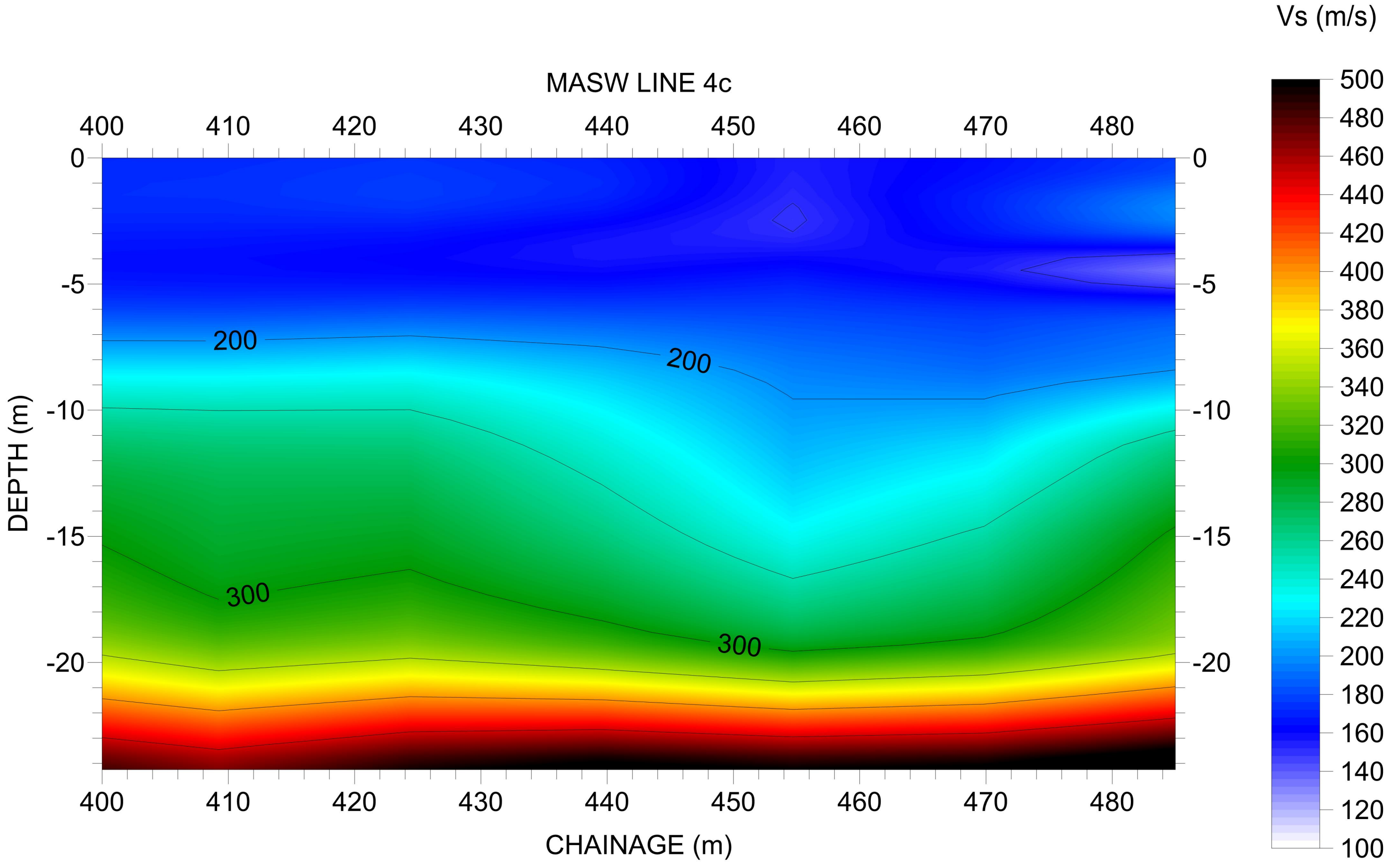

TITLE- **MASW LINE 4c (400m - 485m)**

LOCATION- **Tahuna, Nelson**

NOTES

Contour intervals of 50 m/s (Vs).
Refer to site map for location.

A3

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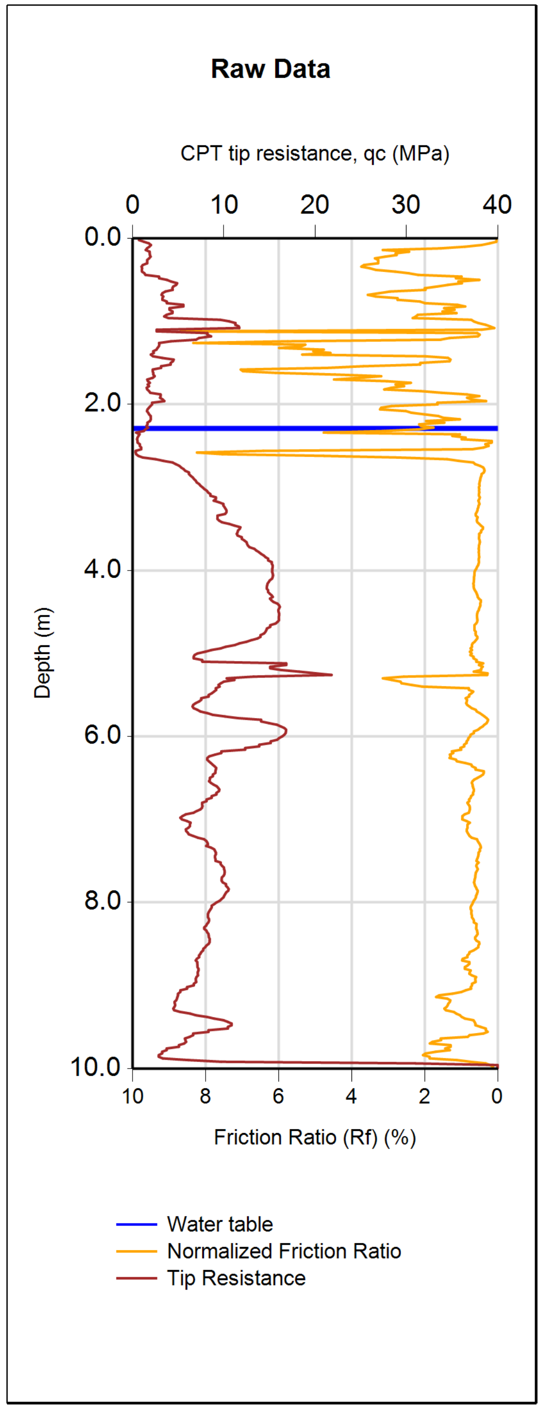

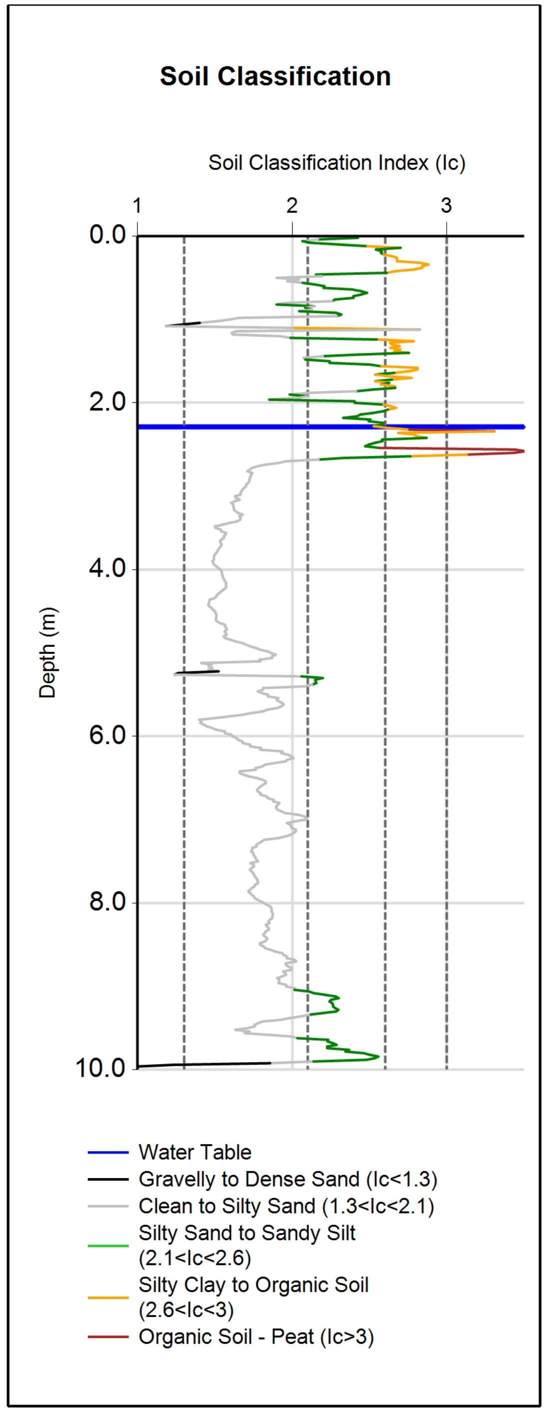

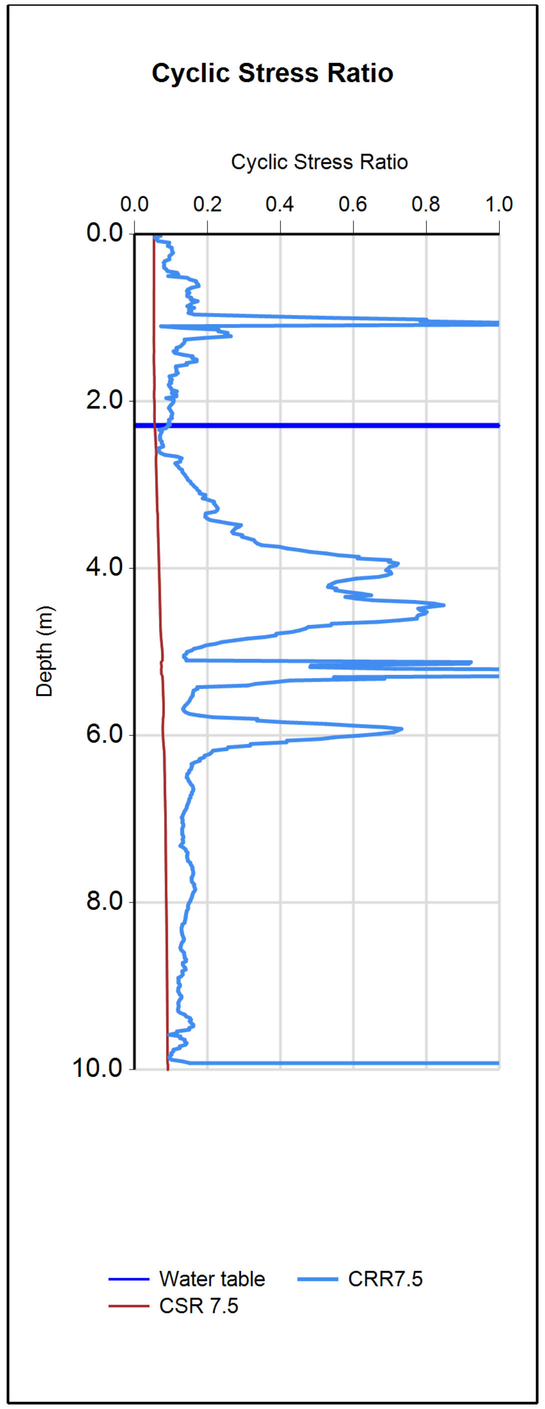

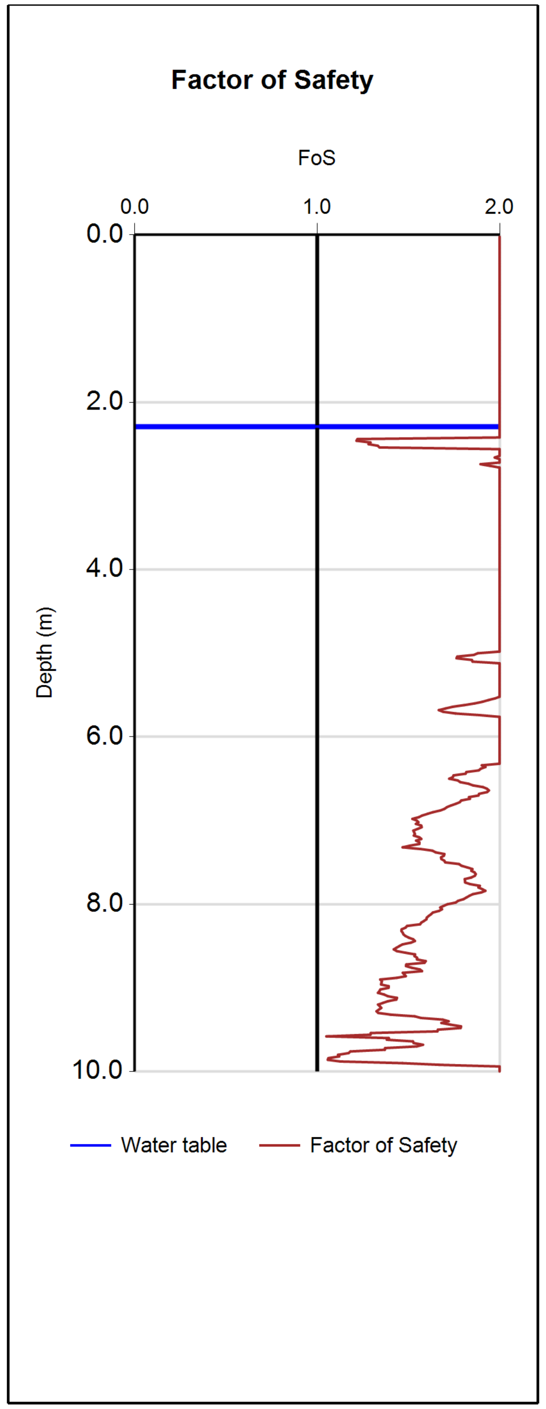

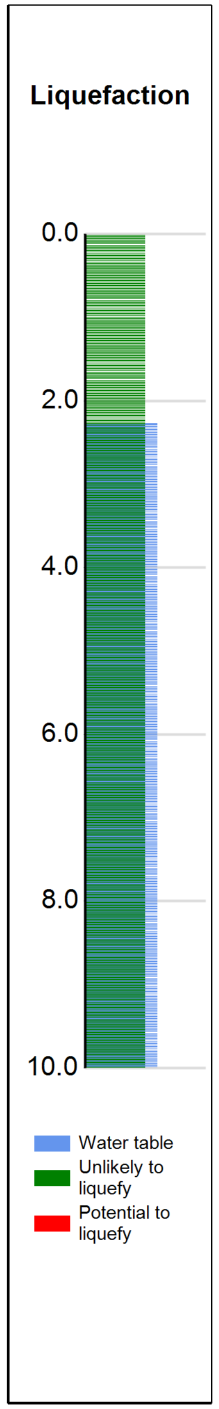

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	ɣ (kN/m³)
INPUT:	CPT1	20770	8/05/2013	User Specified	7.5	0.09	2.3	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	8	0	0	1	10

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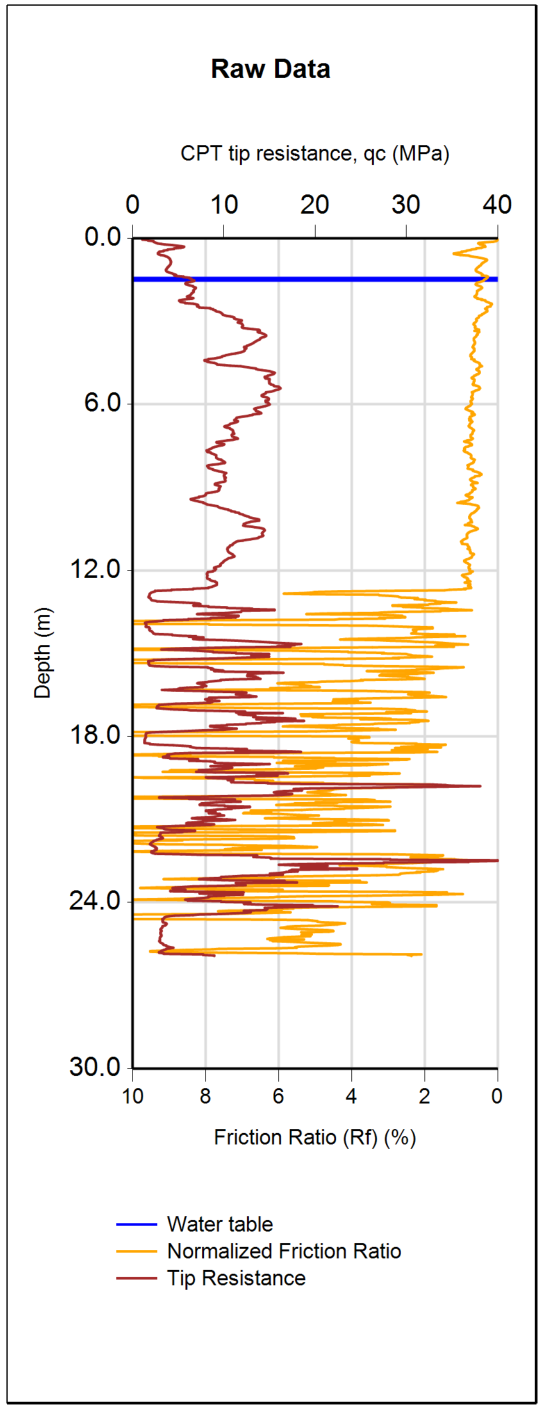

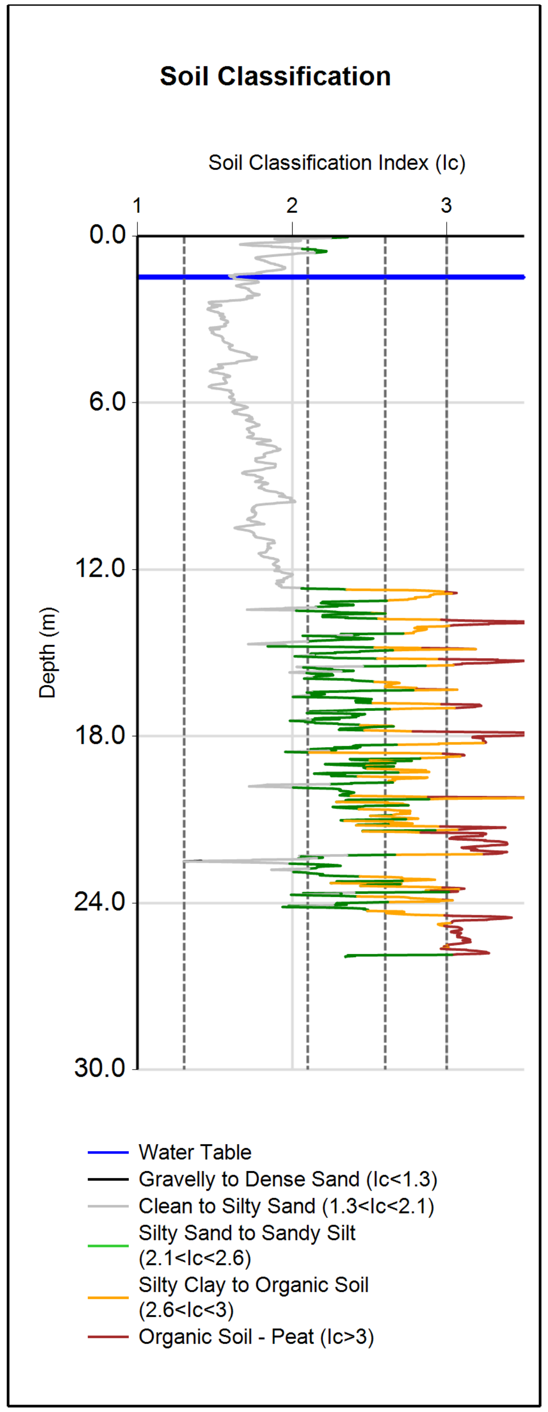

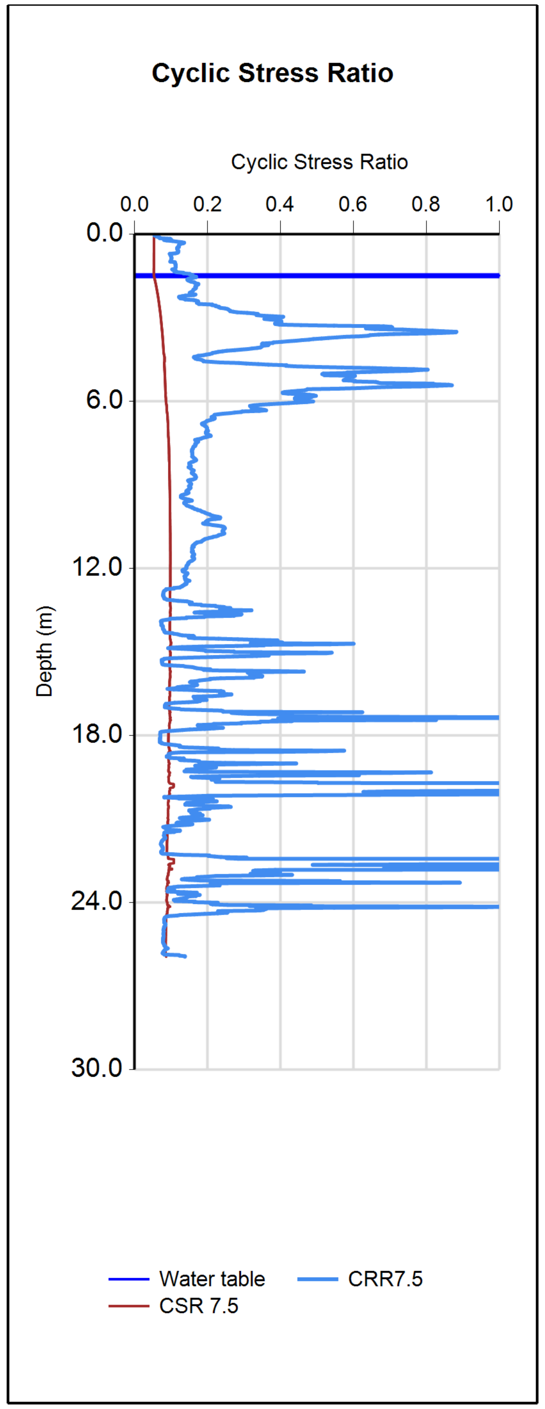

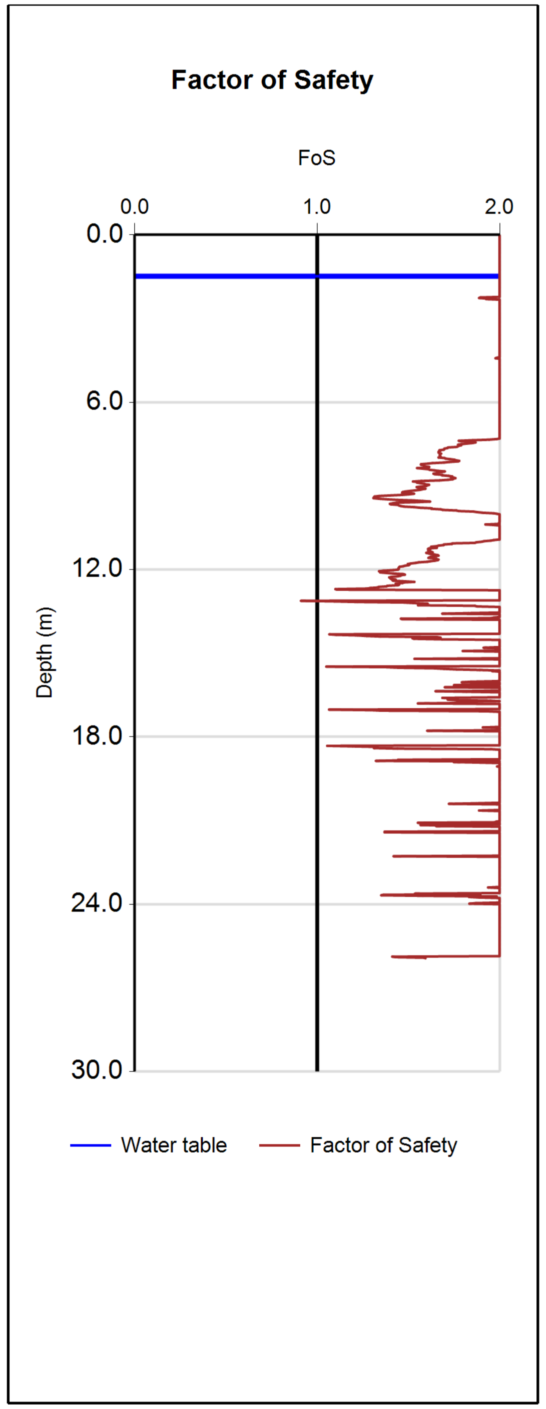

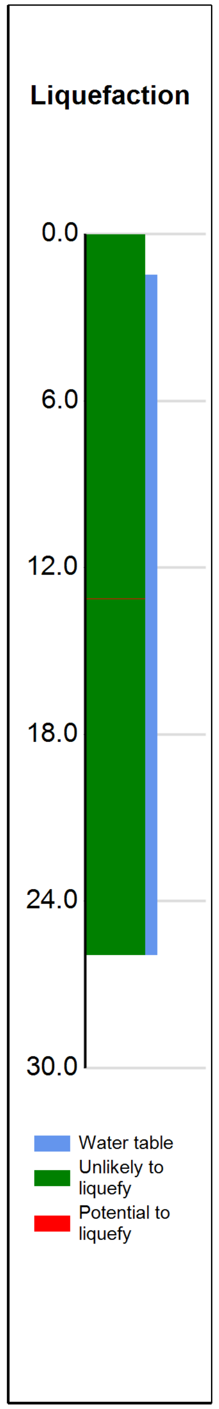

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT2	20879	7/05/2013	User Specified	7.5	0.09	1.5	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	12	0	0	1	25.9

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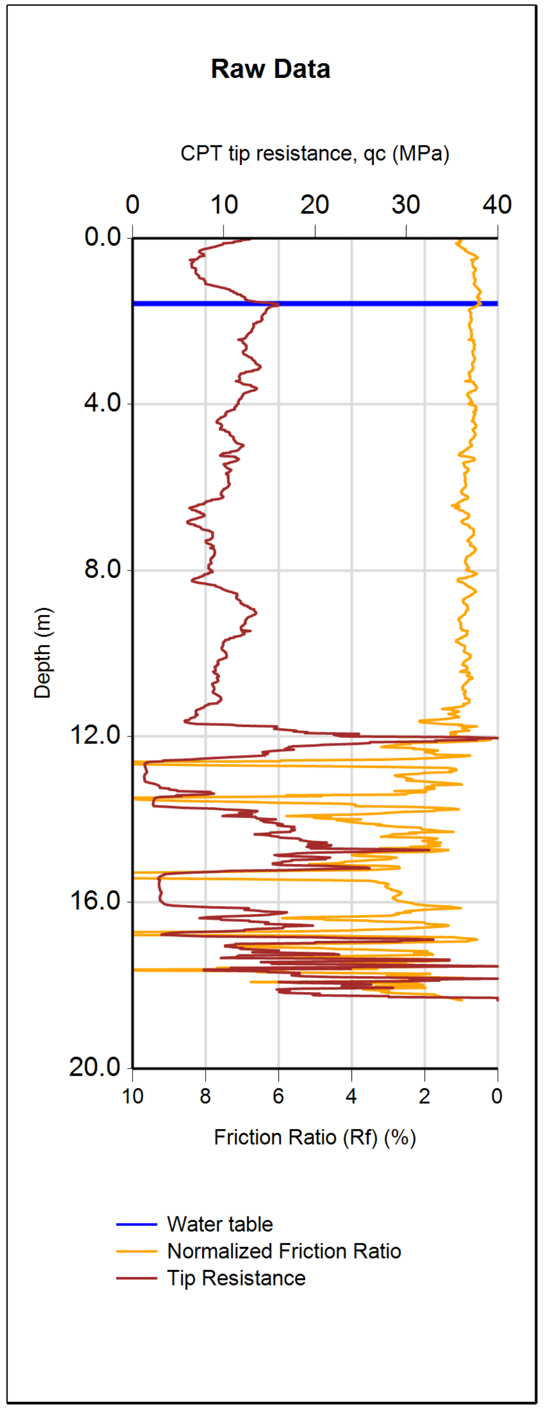

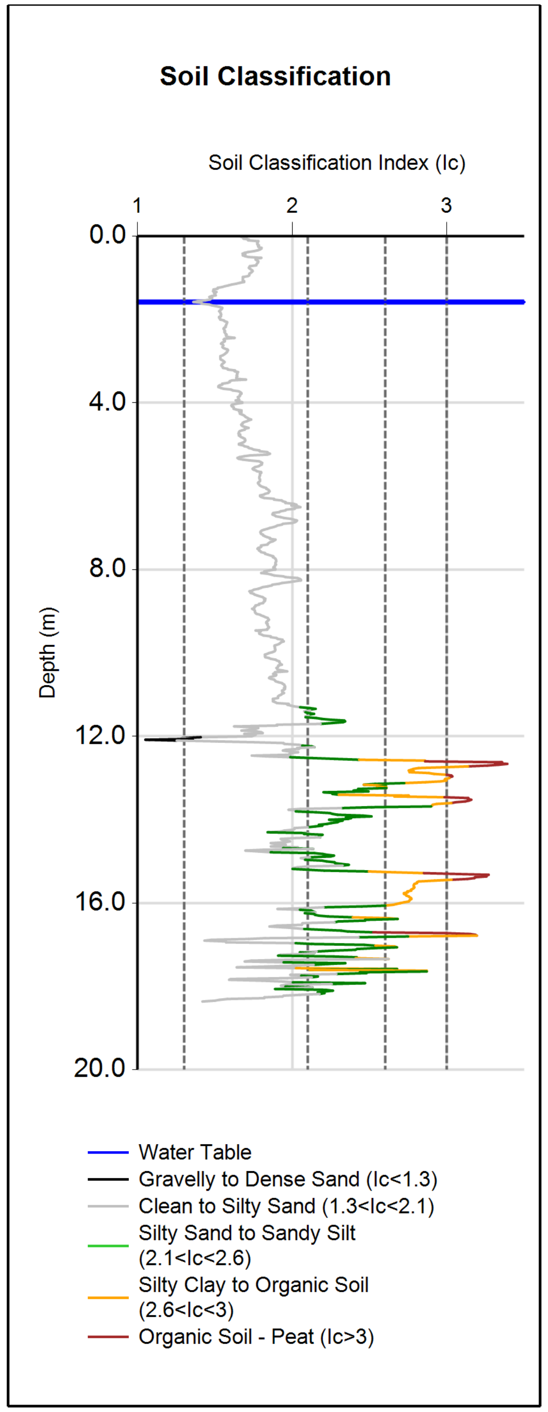

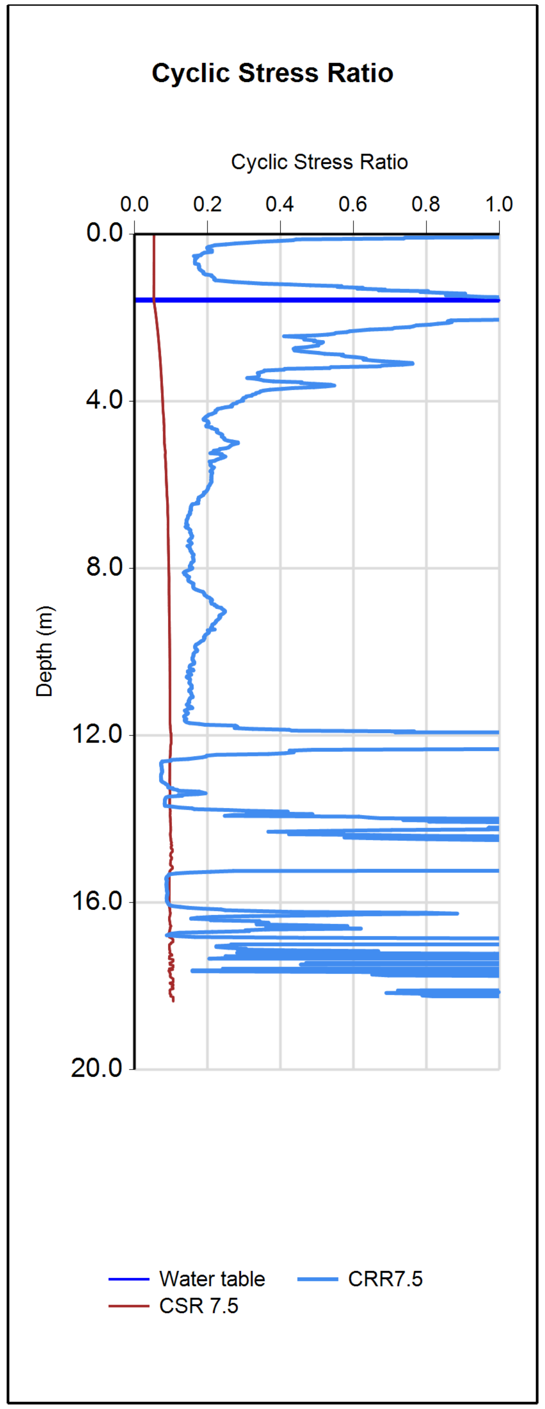

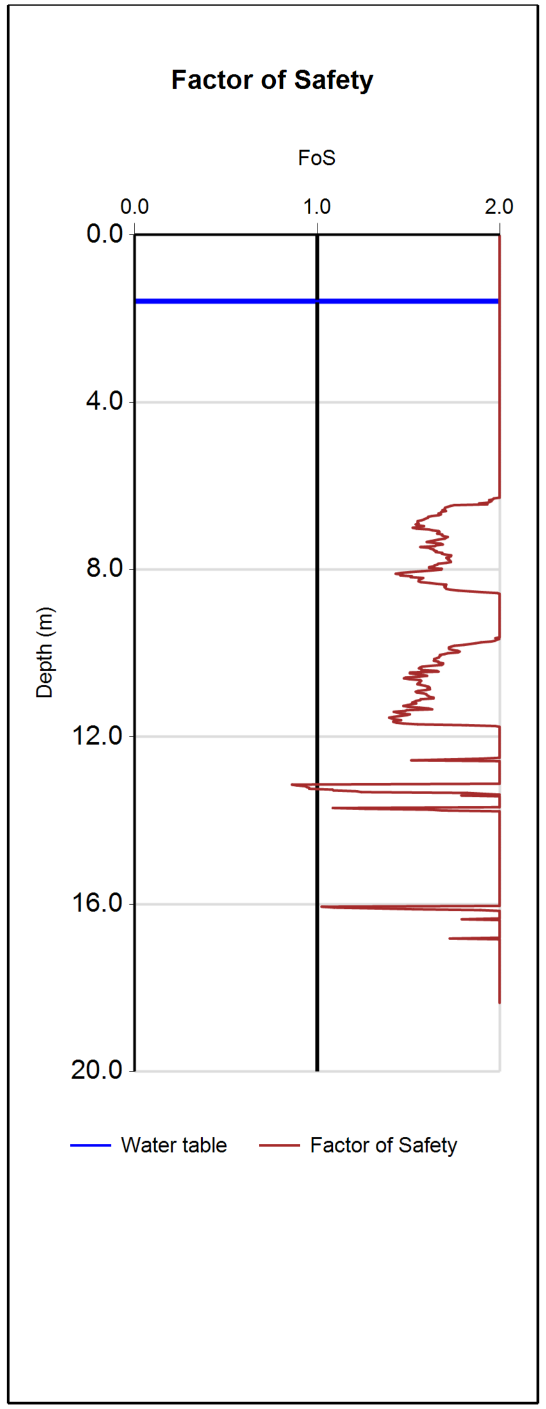

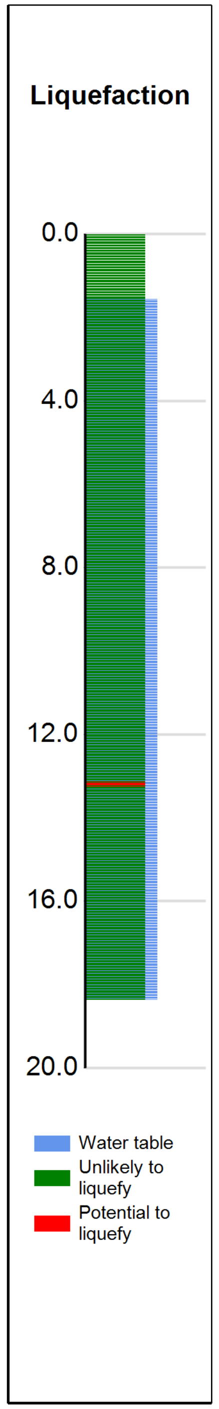

										(Assumed predrill values)			
	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	Qc (MPa)	Fs (MPa)	ɣ (kN/m³)
INPUT:	CPT3	20861	4/05/2013	User Specified	7.5	0.09	1.6	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	11	0.1	0	1	13.2

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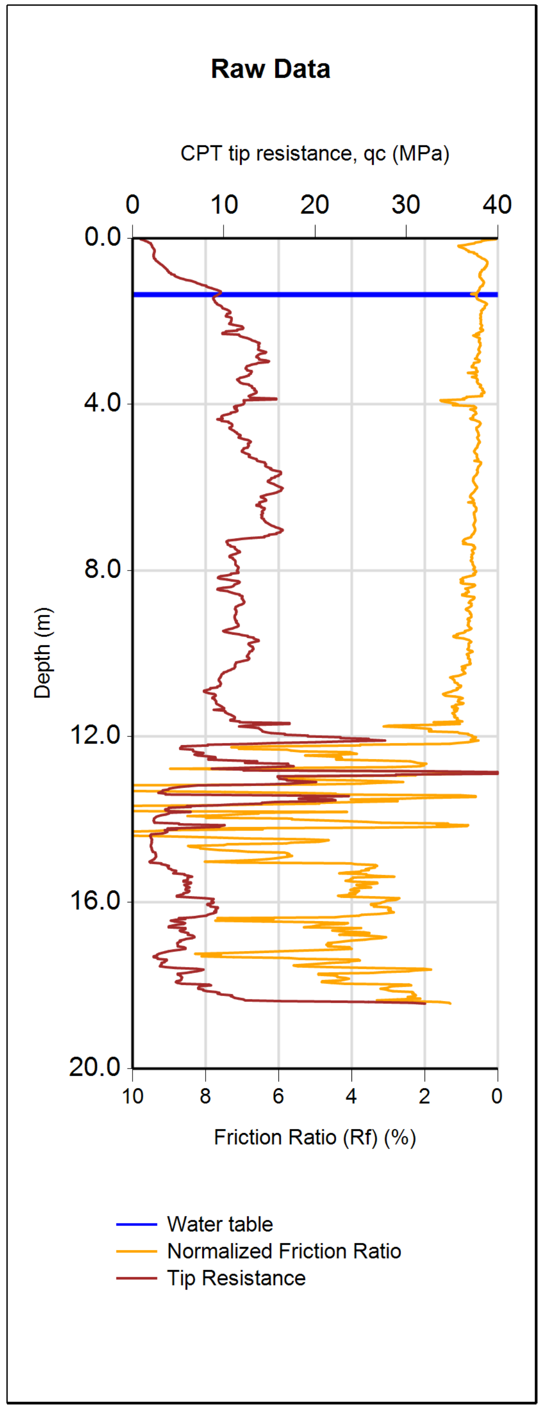

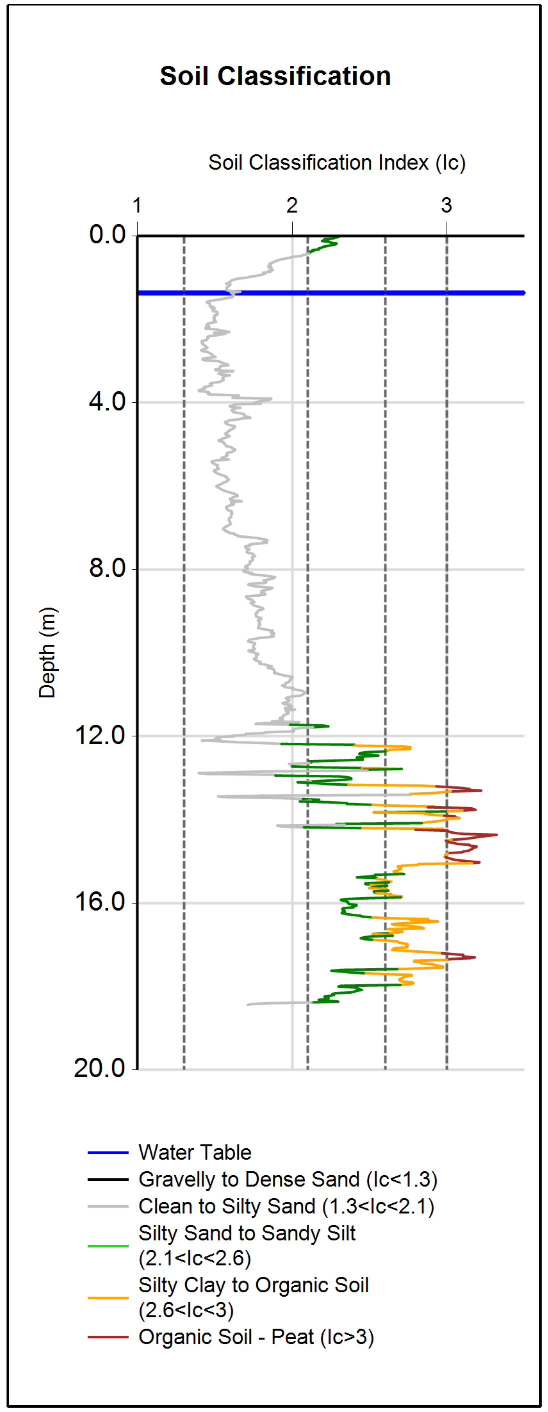

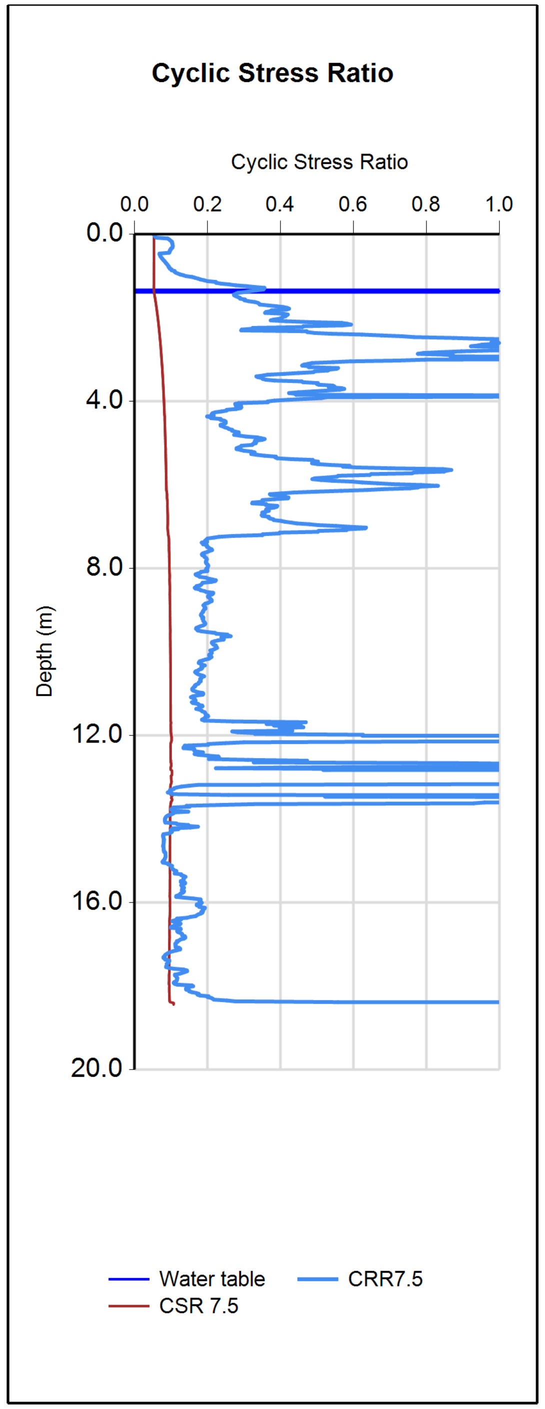

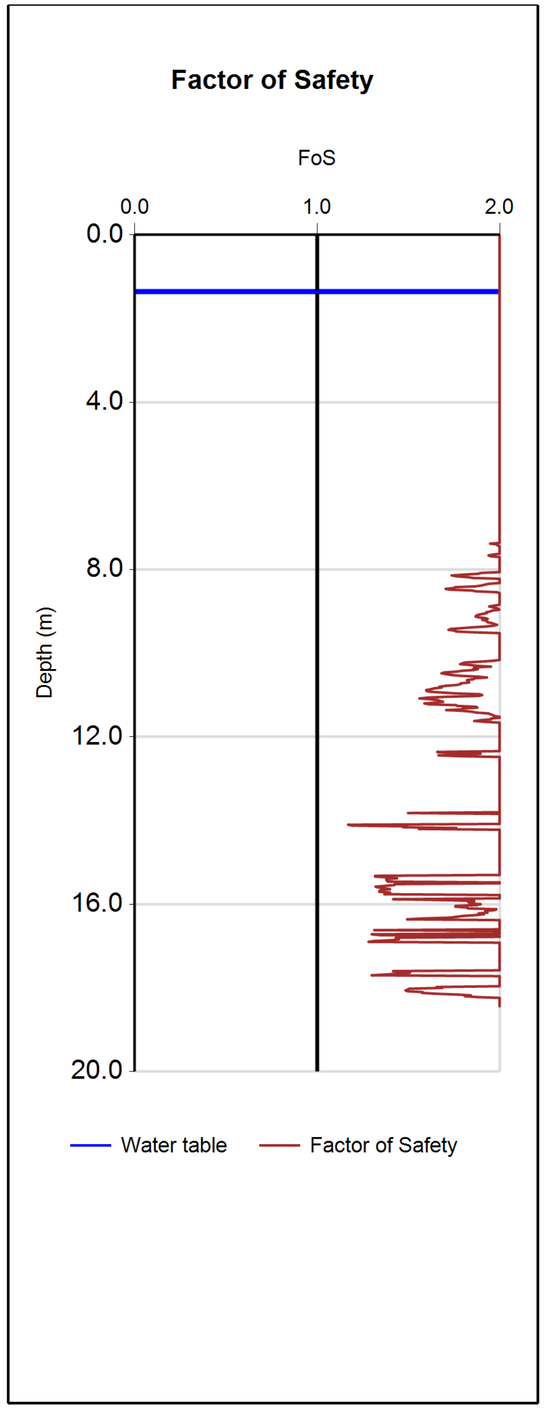

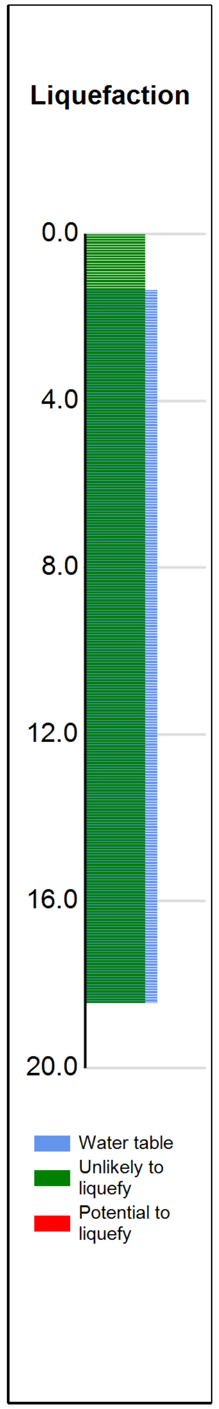

(Assumed predrill values)

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT5	20866	4/05/2013	User Specified	7.5	0.09	1.4	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	5	0	0	0	18.4

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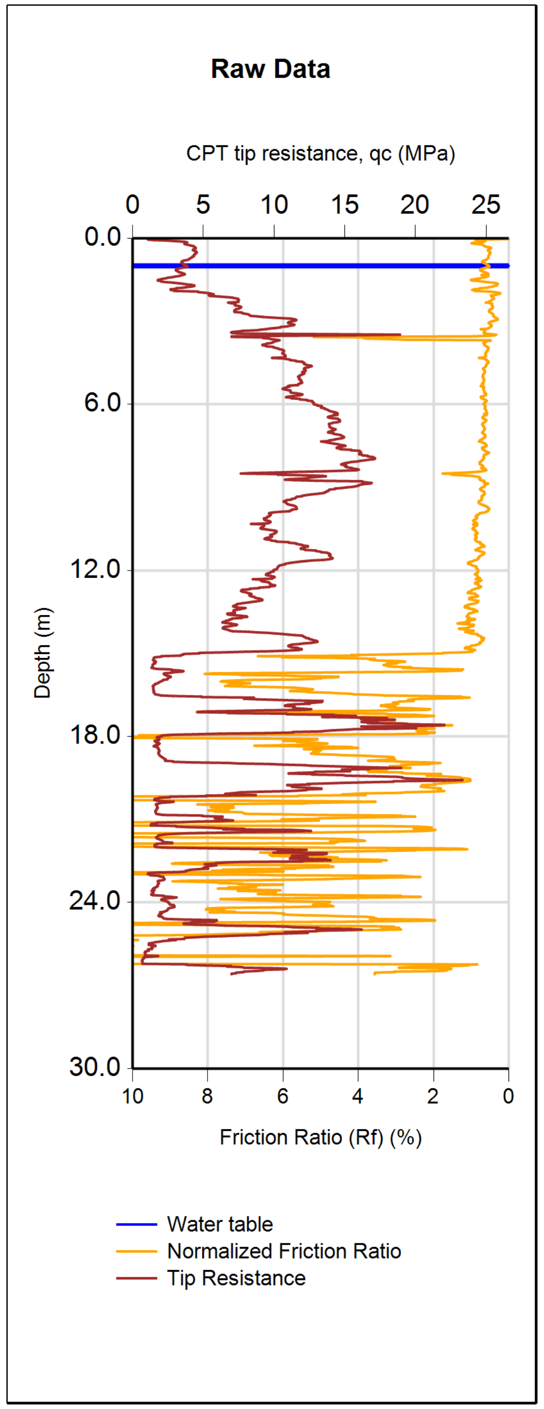

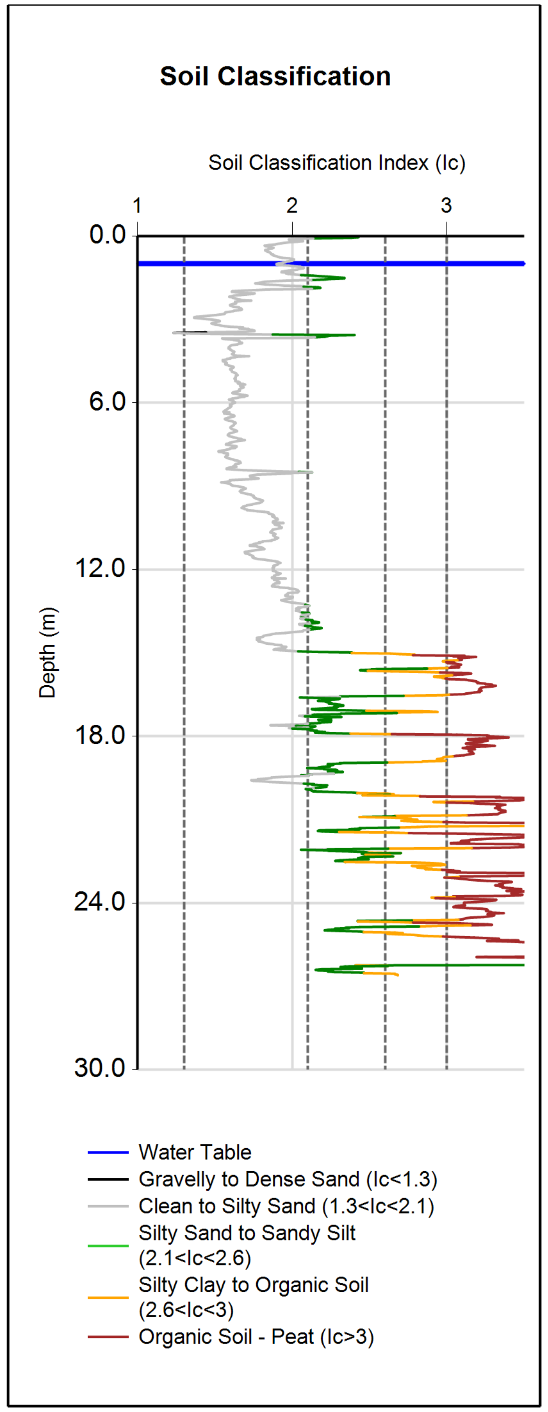

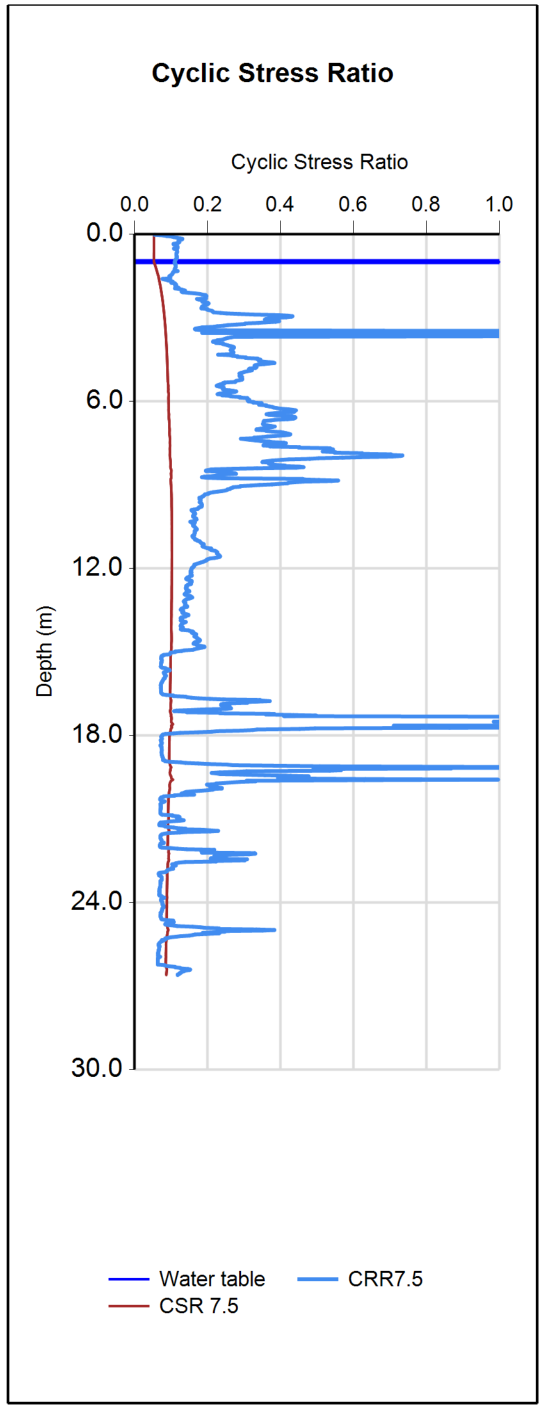

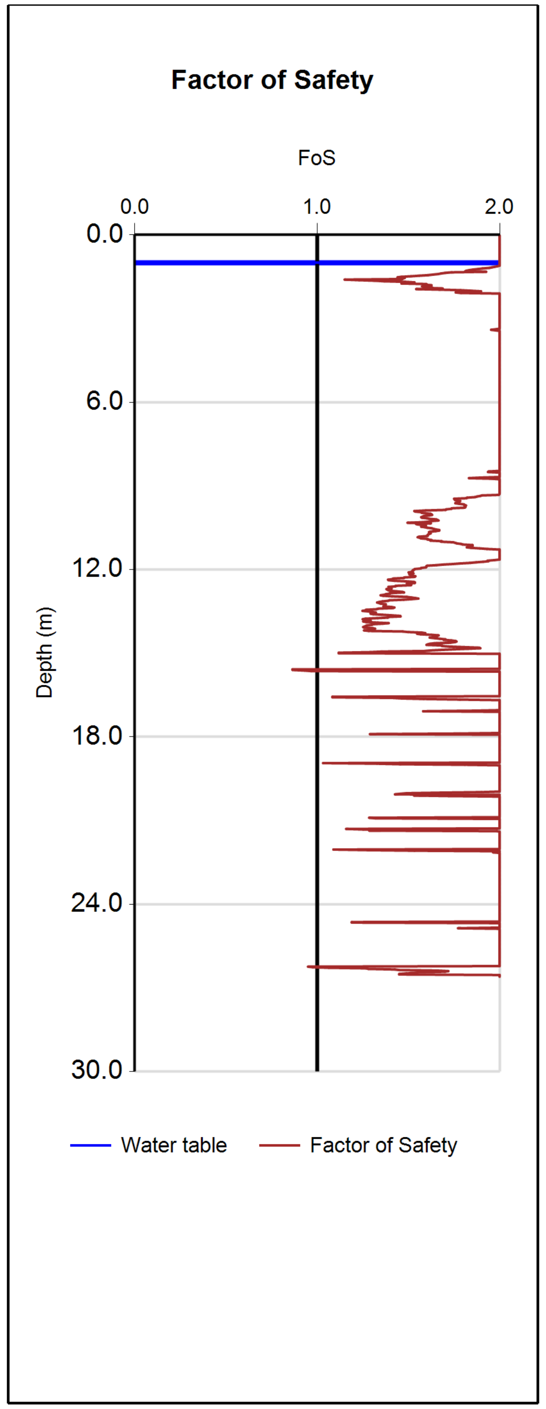

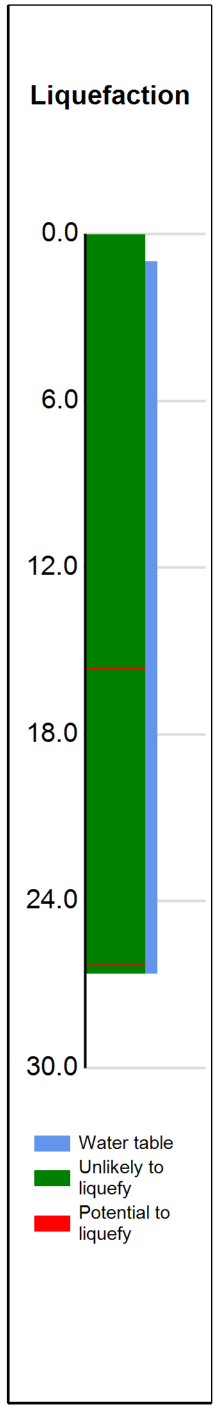

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	ɣ (kN/m³)
INPUT:	CPT6	20867	7/05/2013	User Specified	7.5	0.09	1.0	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	18	0.1	0	2	26.2

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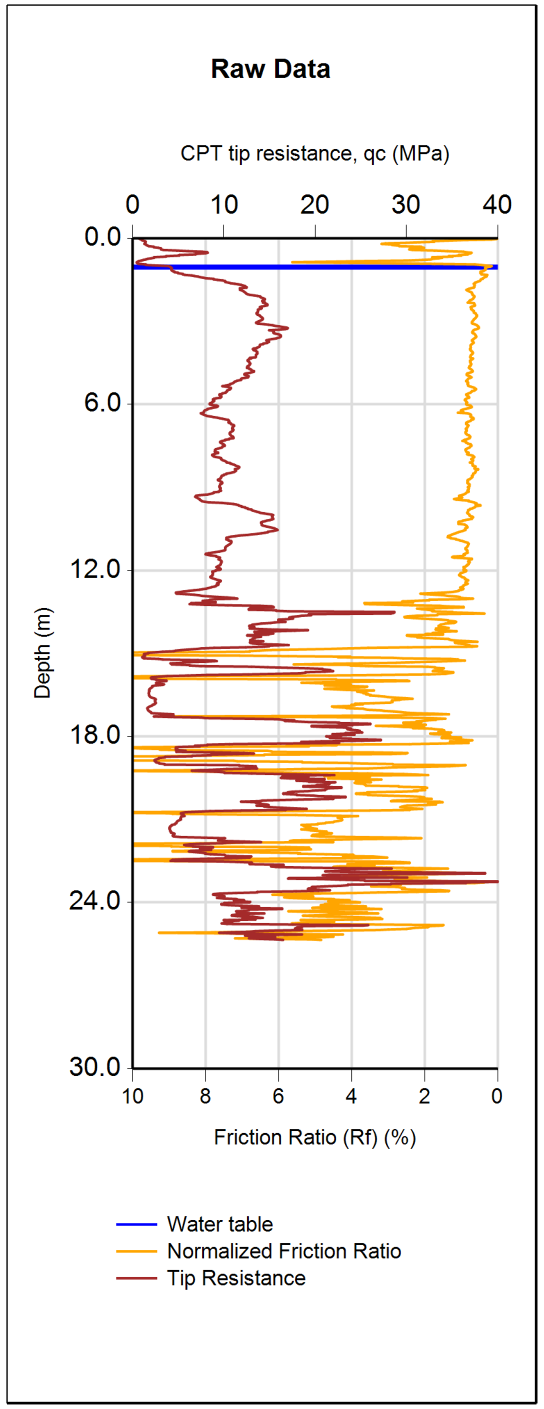

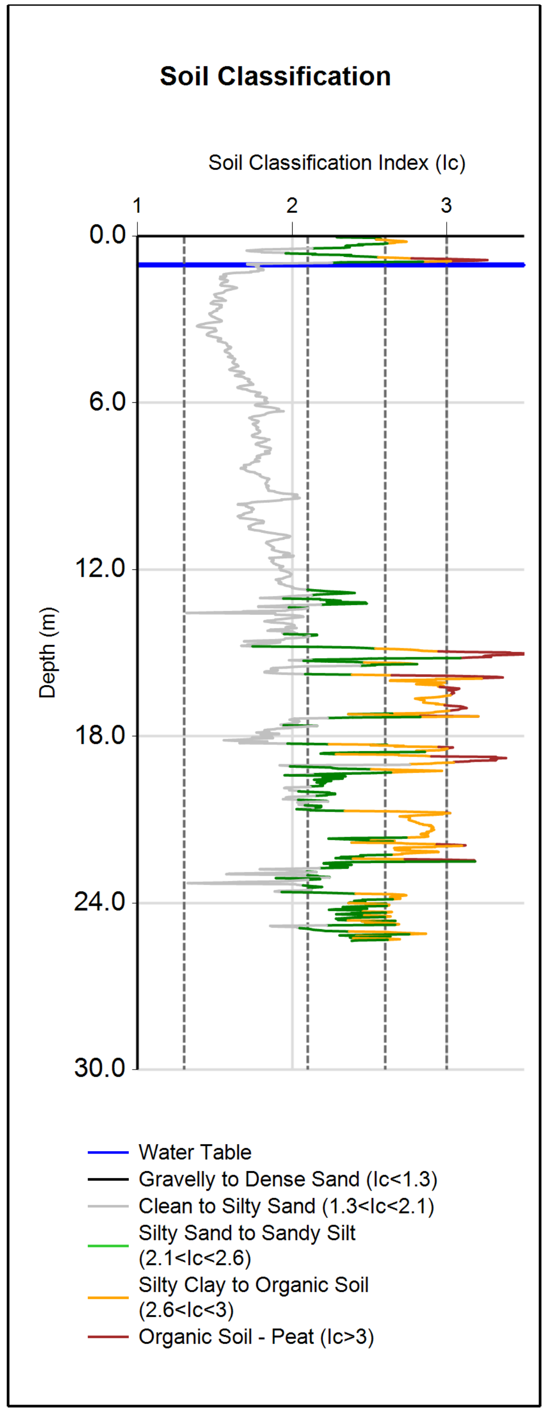

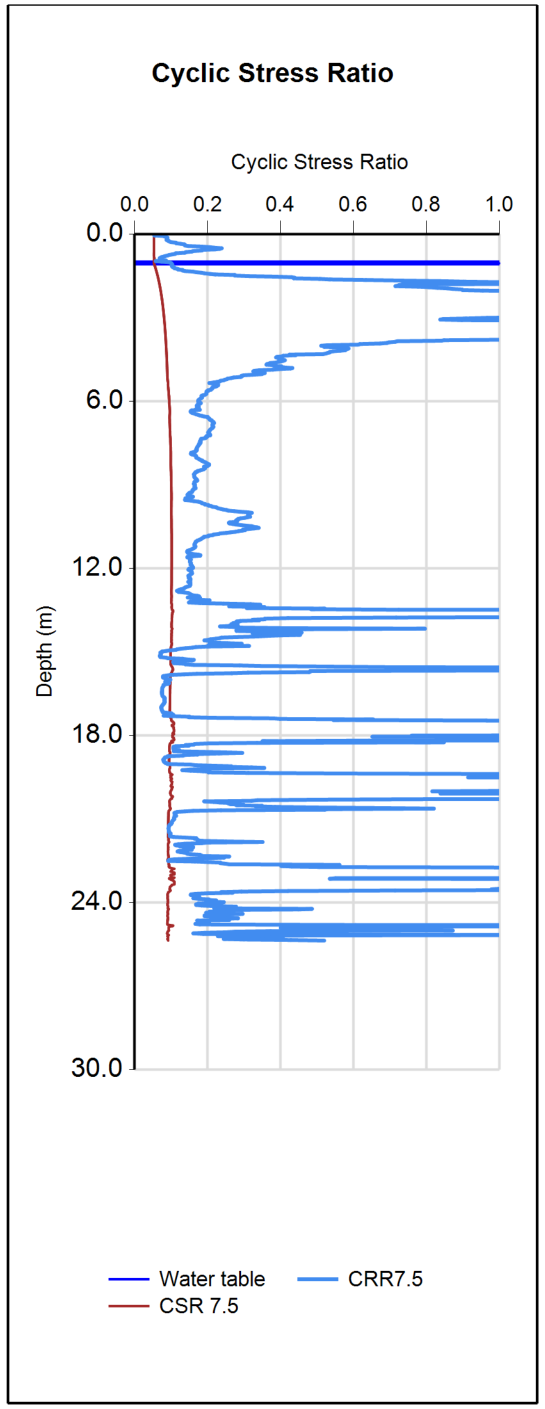

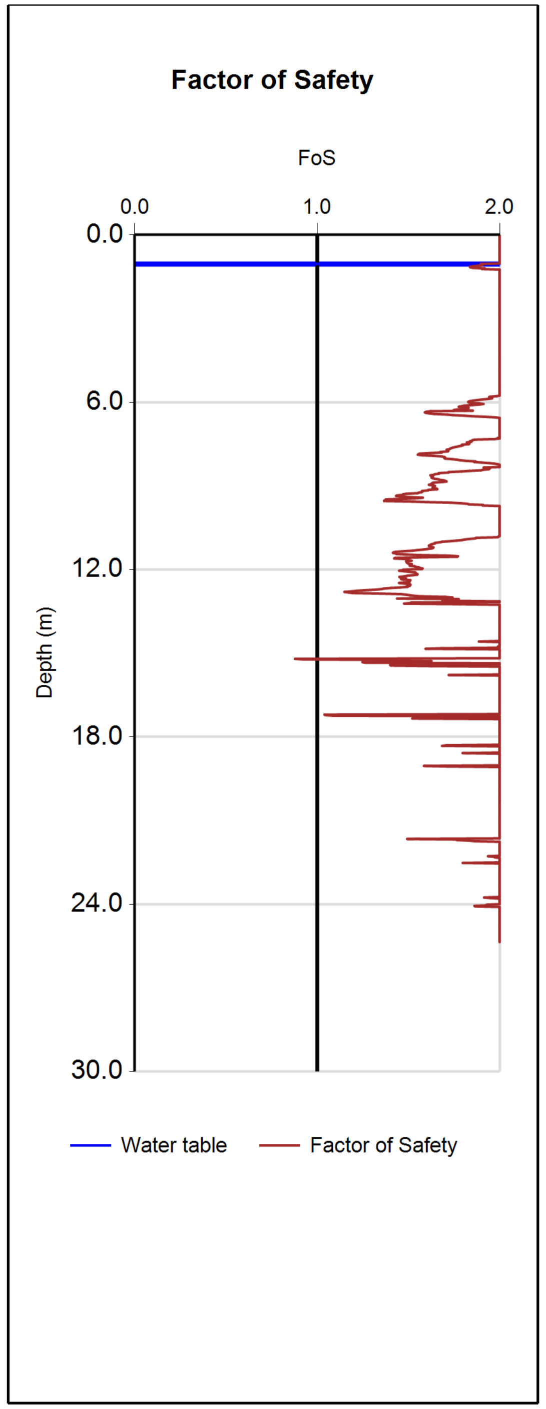

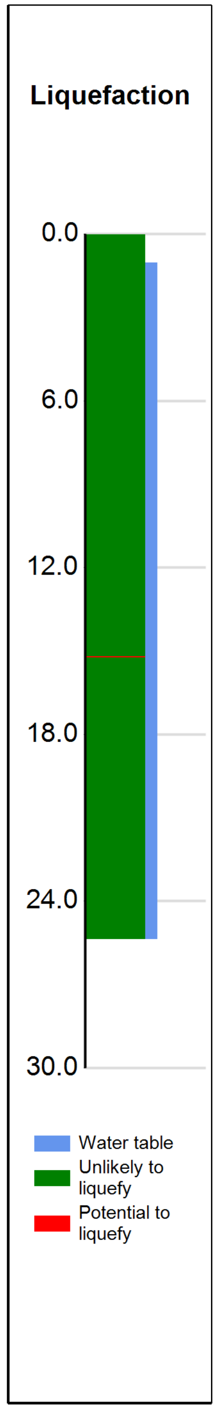

(Assumed predrill values)

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	Qc (MPa)	Fs (MPa)	ɣ (kN/m³)
INPUT:	CPT7	20868	8/05/2013	User Specified	7.5	0.09	1.0	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	11	0	0	1	25.4

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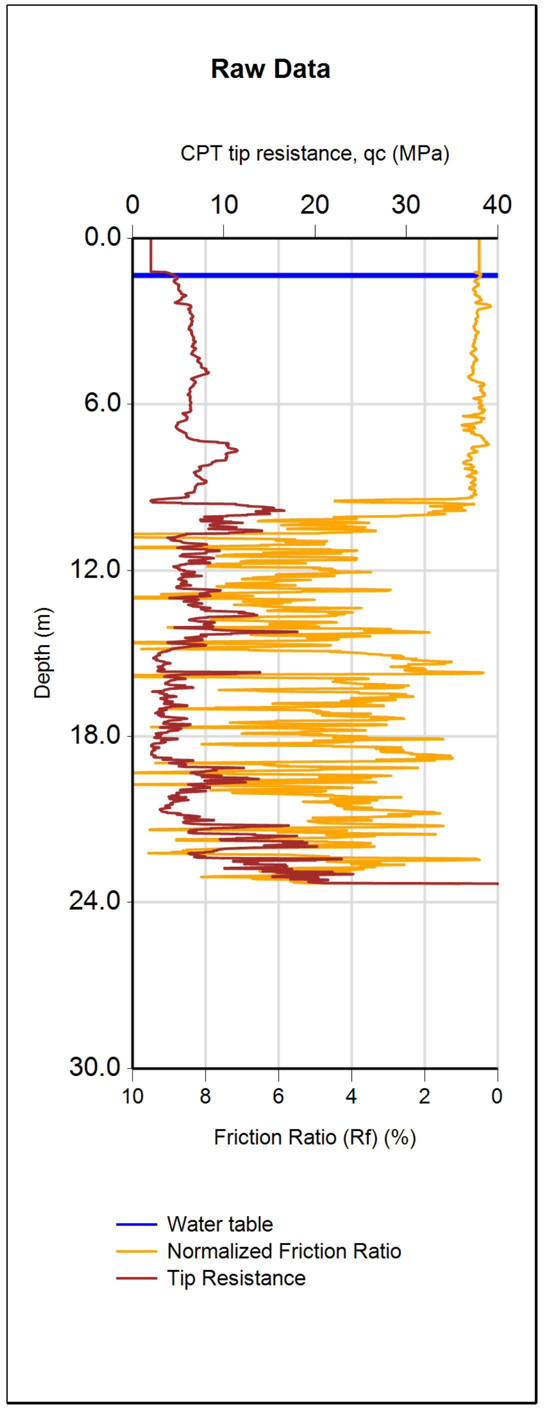

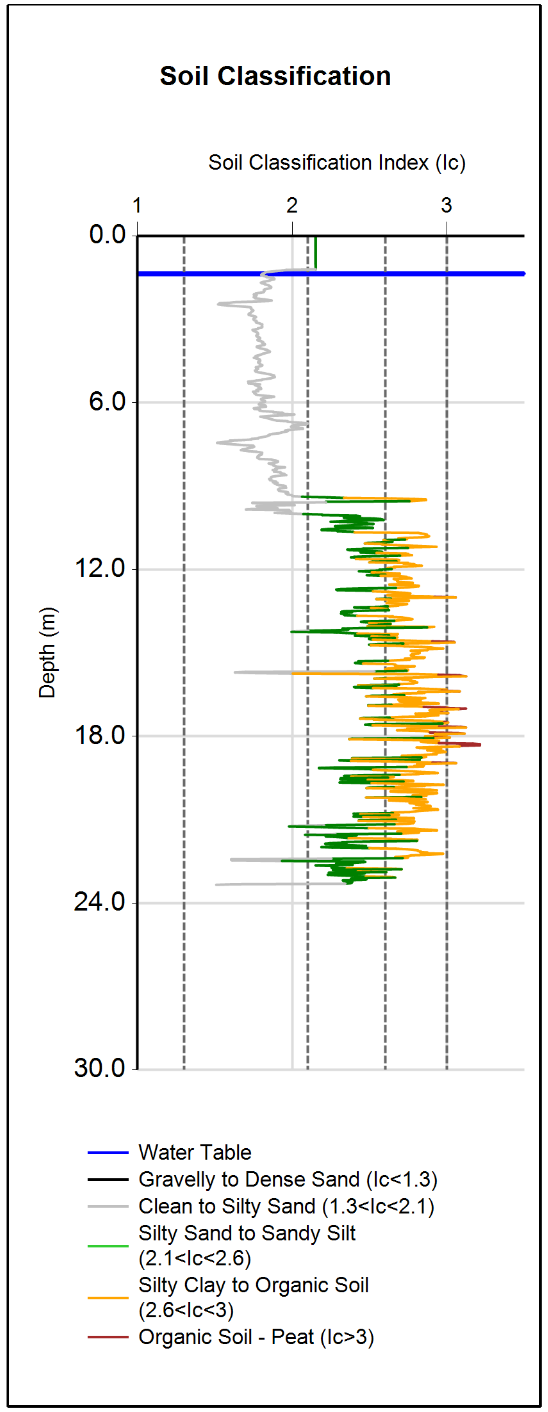

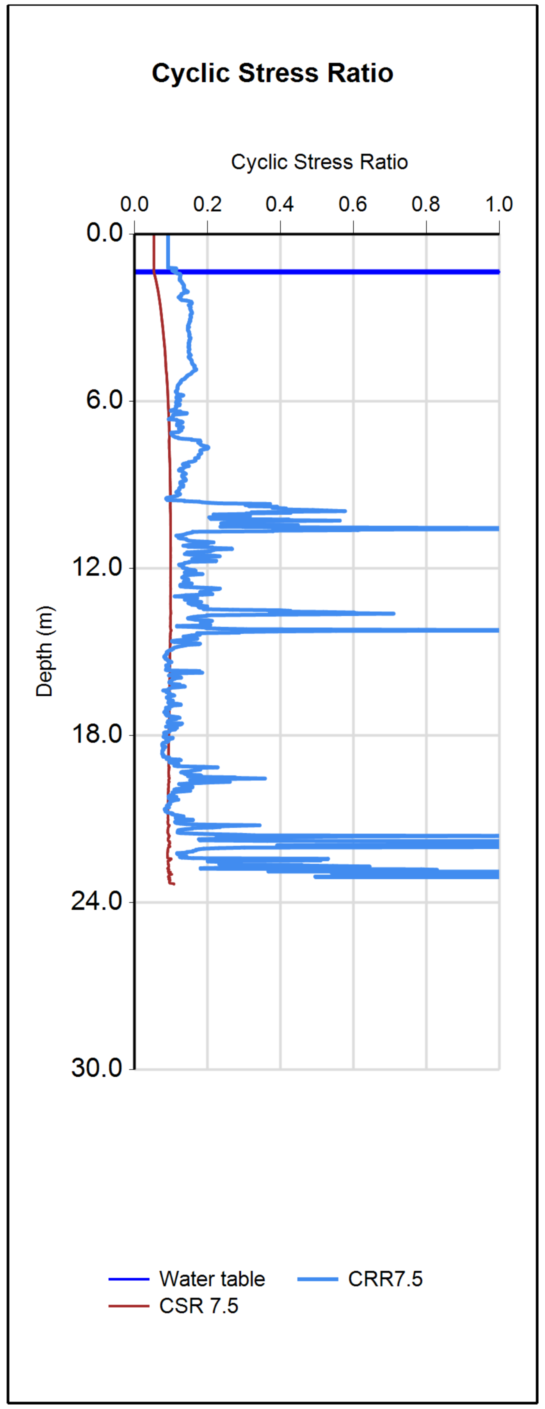

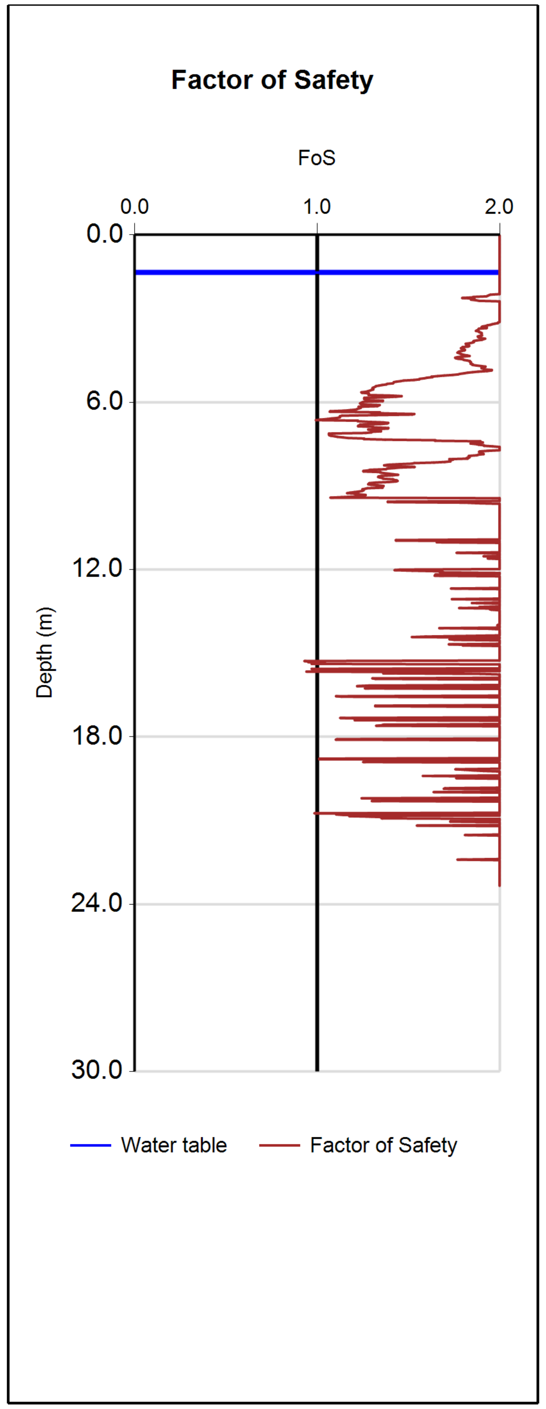

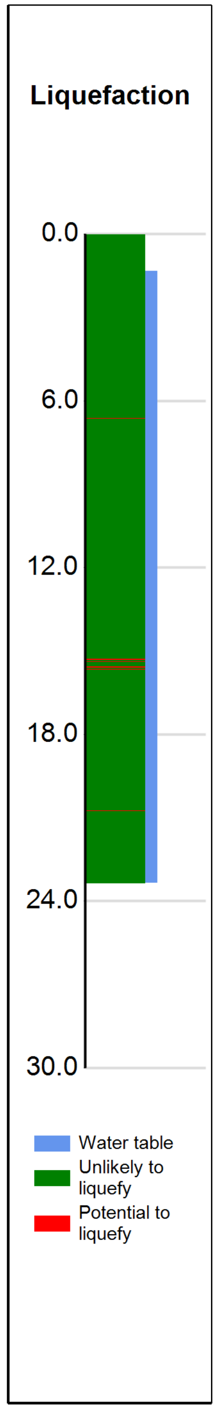

(Assumed predrill values)

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT8	20870	7/05/2013	User Specified	7.5	0.09	1.4	IB	ZRB	1.2	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	23	0.2	0	3	15.4

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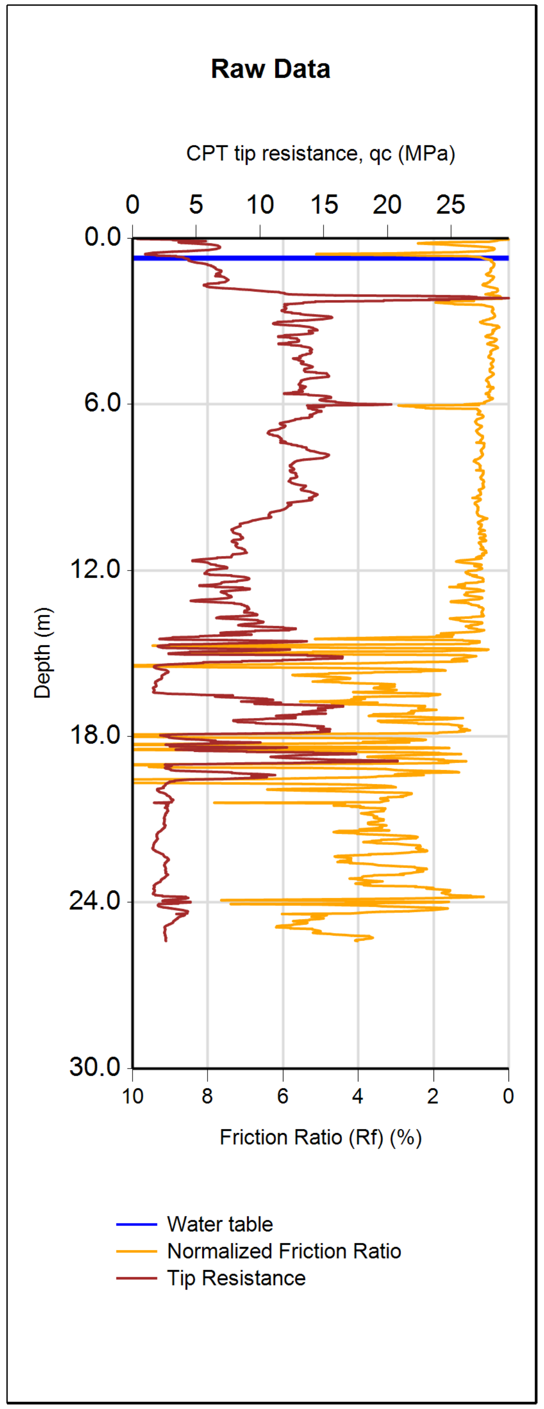

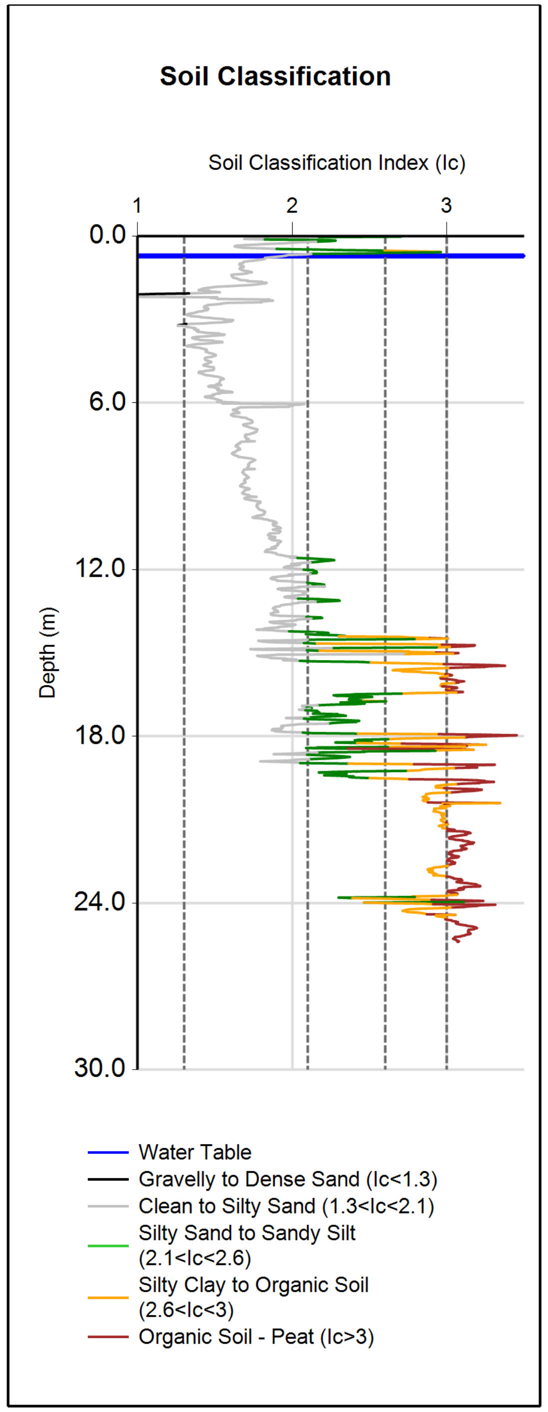

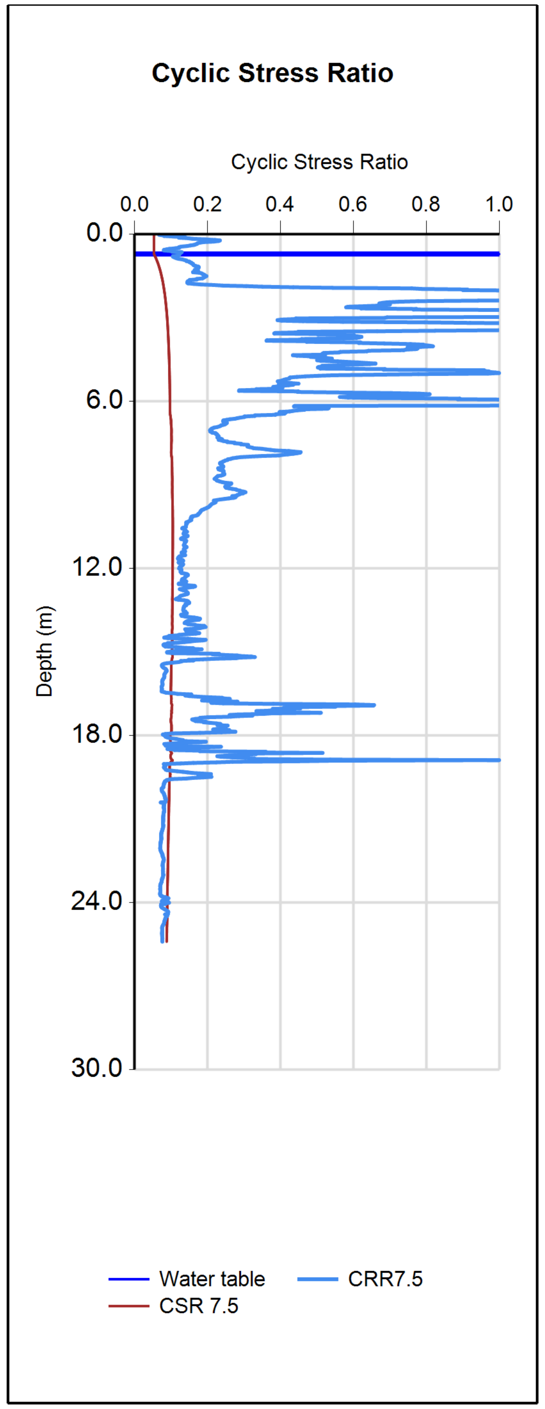

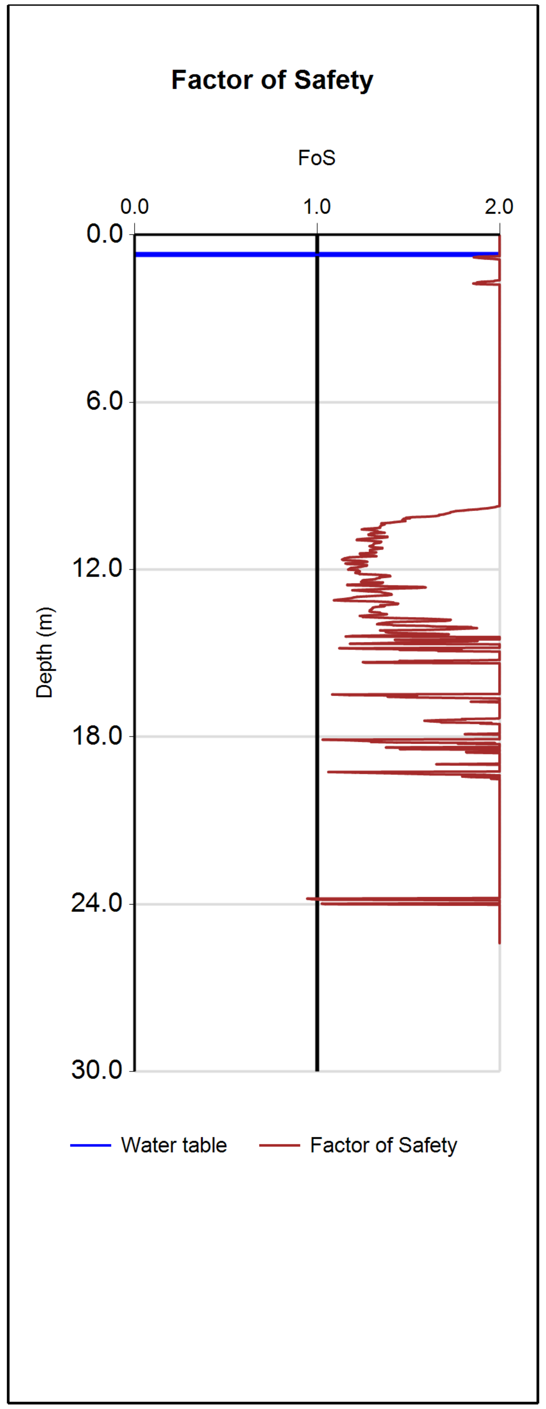

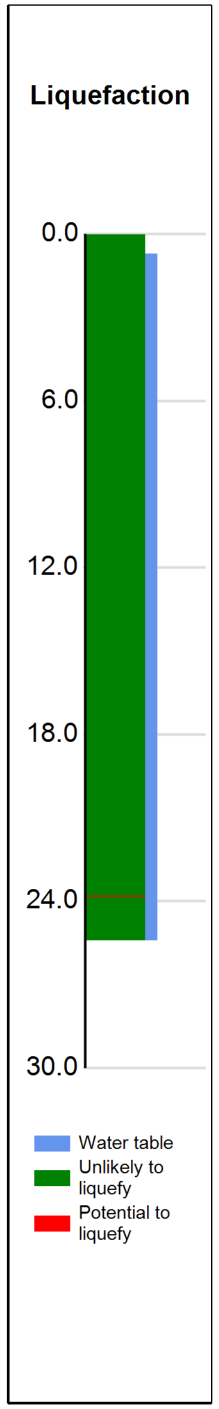

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	ɣ (kN/m³)
INPUT:	CPT9	20873	7/05/2013	User Specified	7.5	0.09	0.7	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	18	0	0	1	25.4

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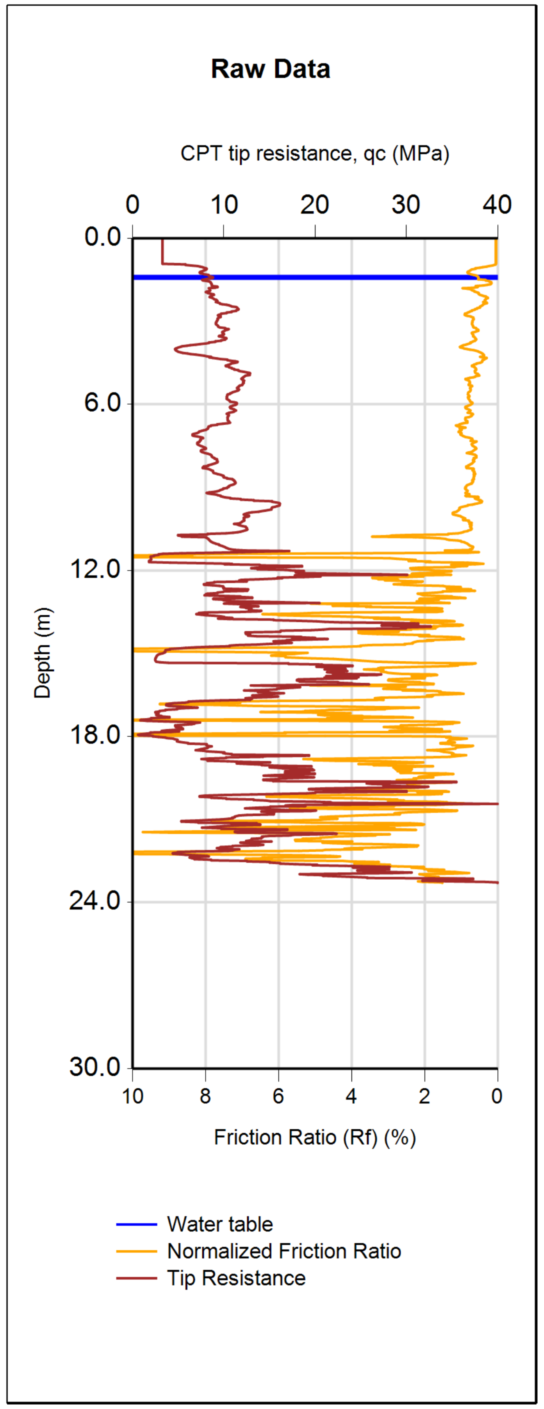

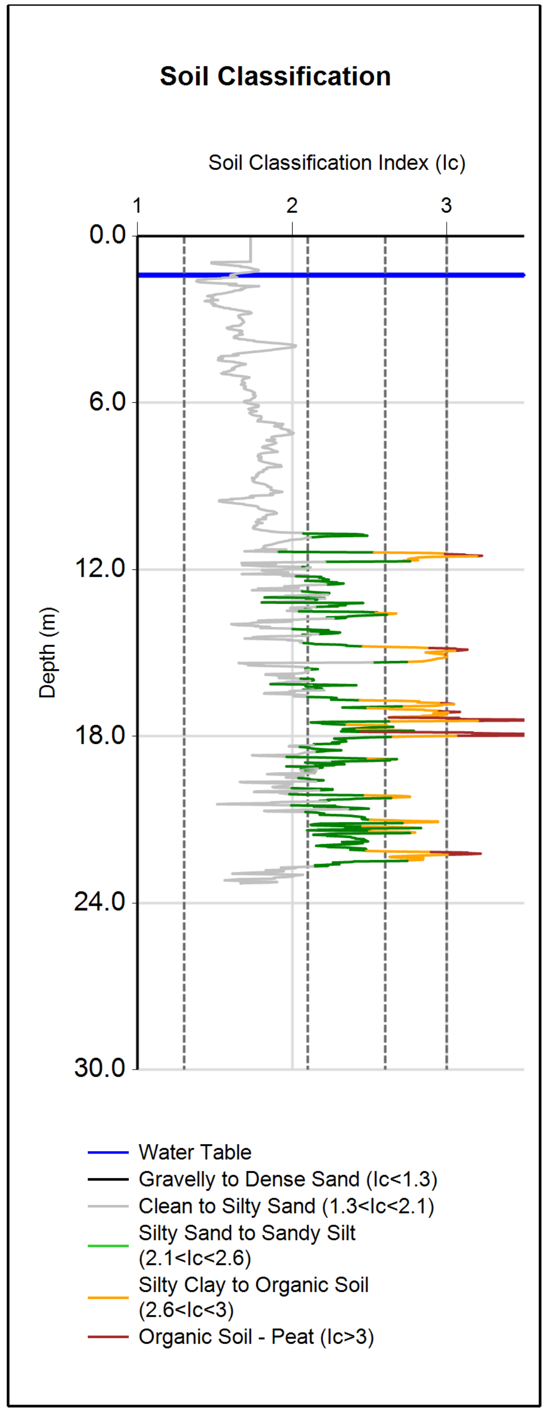

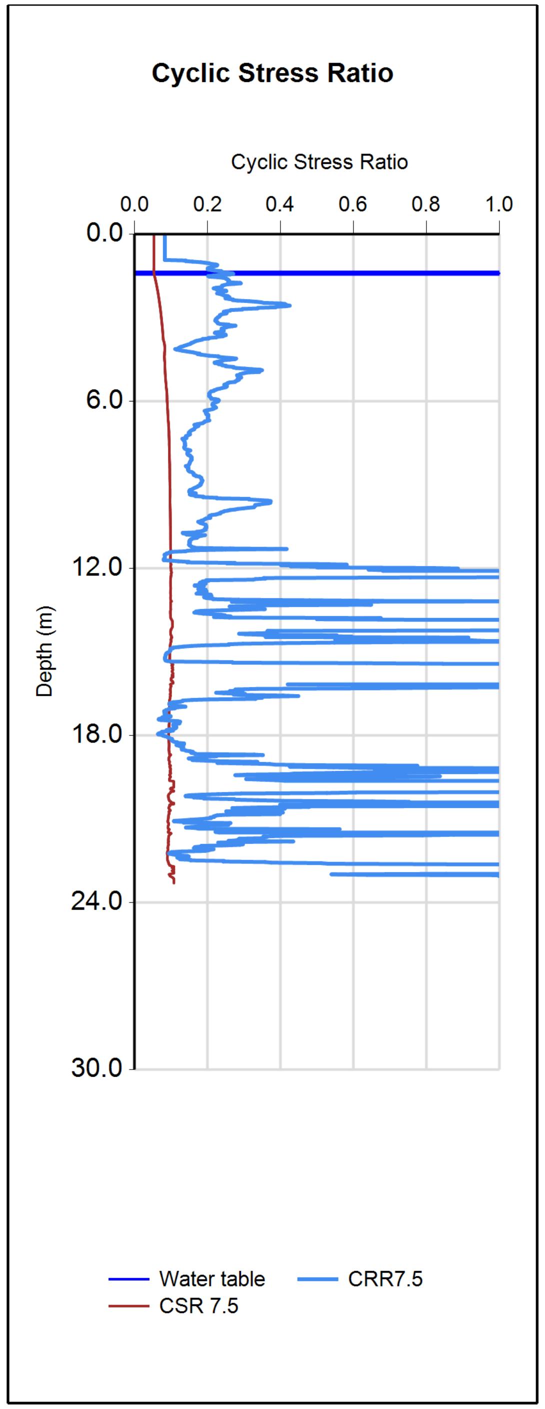

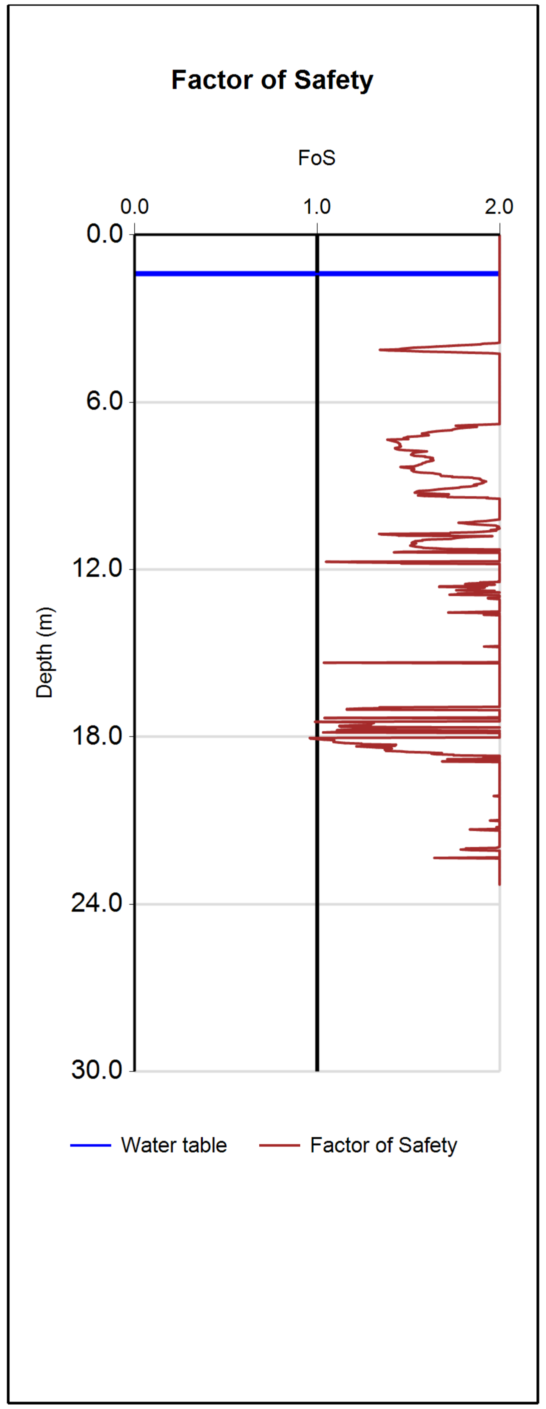

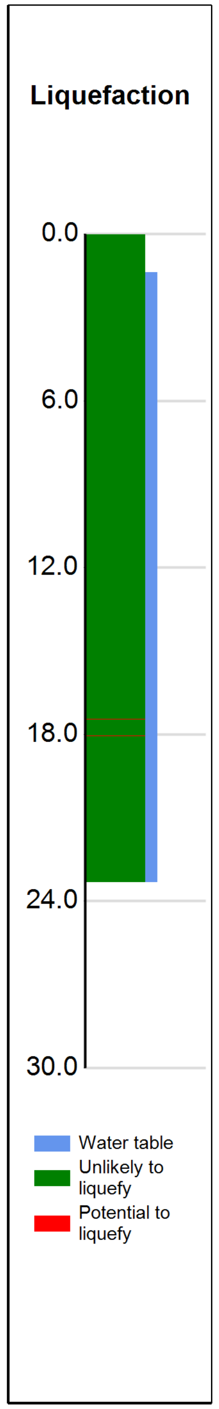

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT10	20875	4/05/2013	User Specified	7.5	0.09	1.4	IB	ZRB	0.9	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	12	0.1	0	1	23.3

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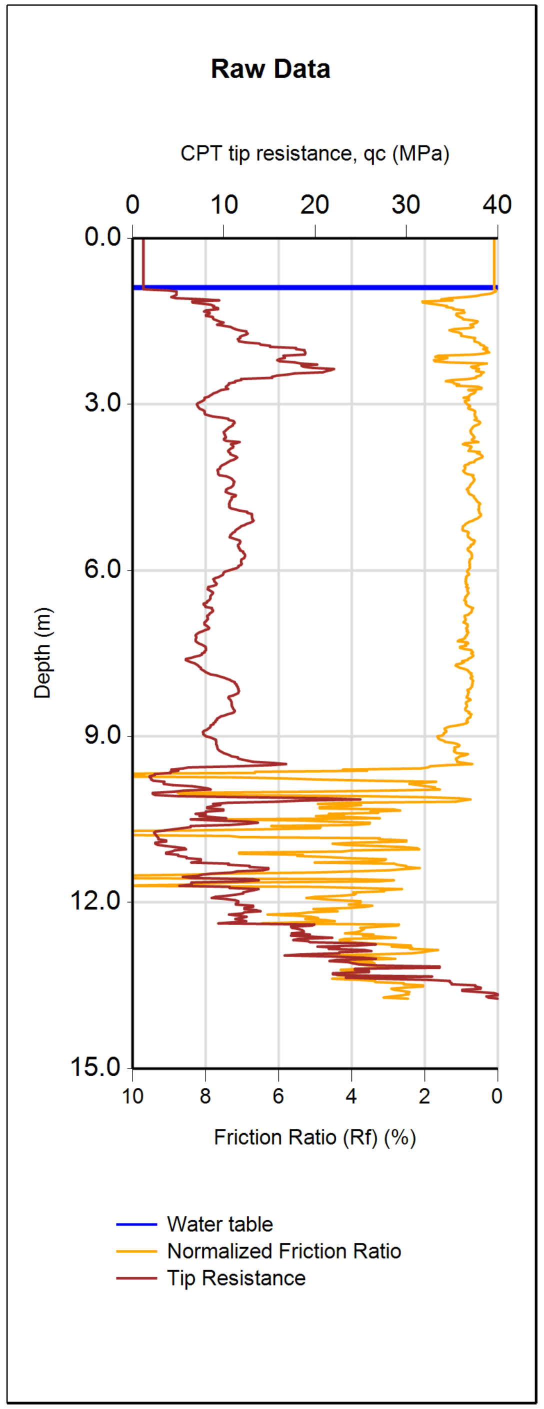

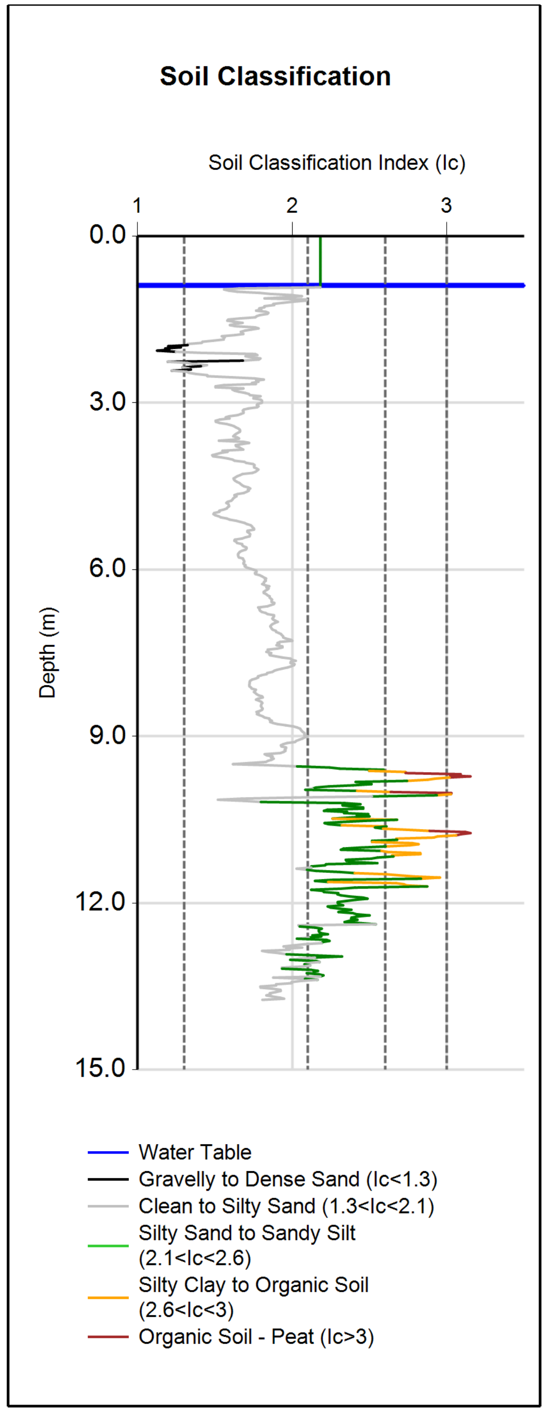

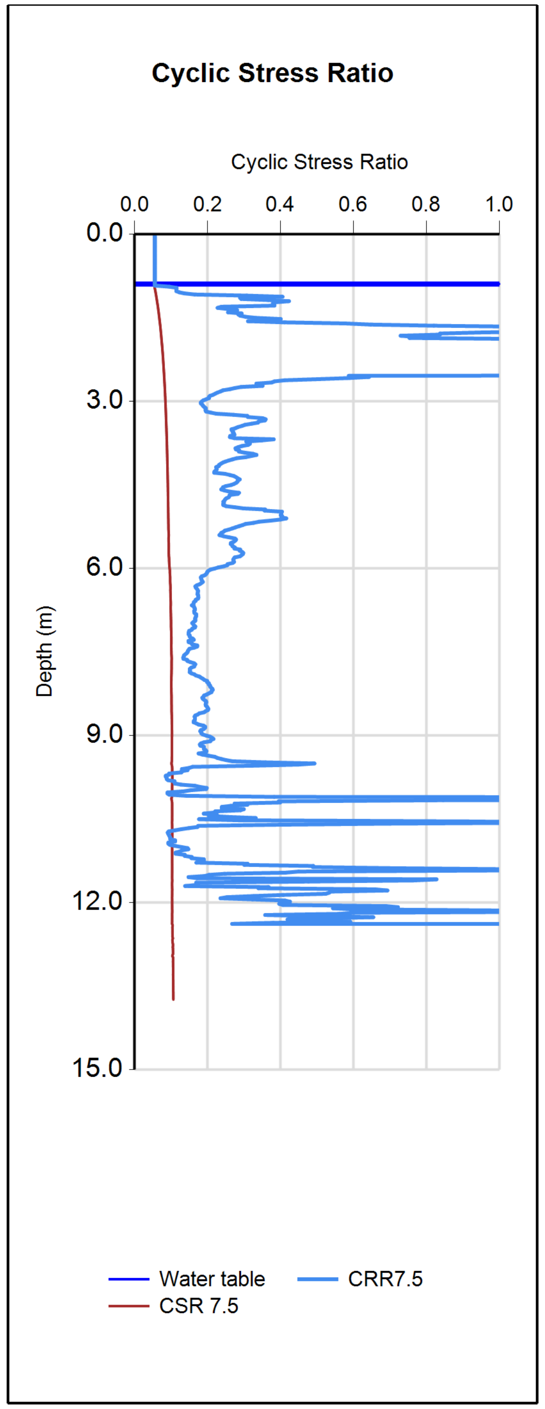

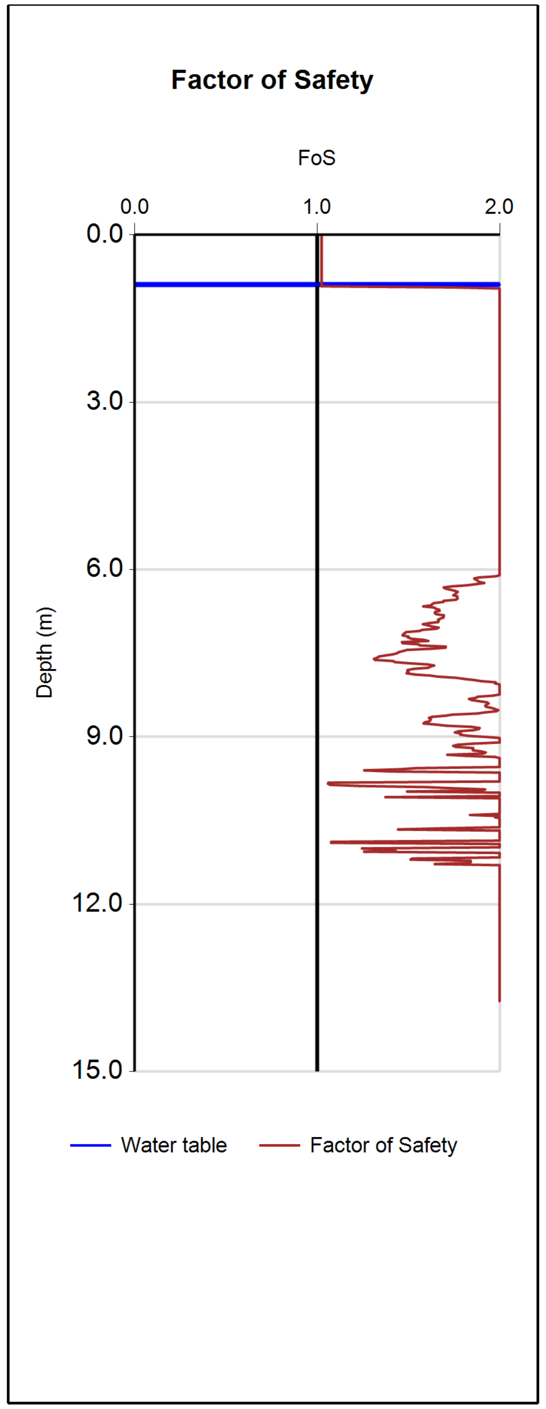

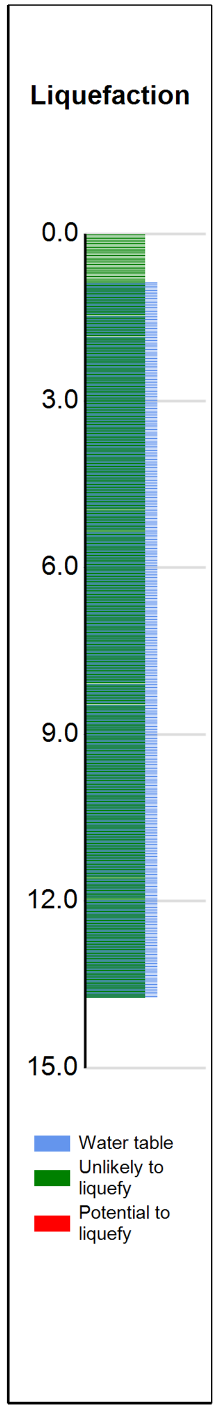

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT12	20877	4/05/2013	User Specified	7.5	0.09	0.9	IB	ZRB	0.9	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	6	0	0	1	13.7

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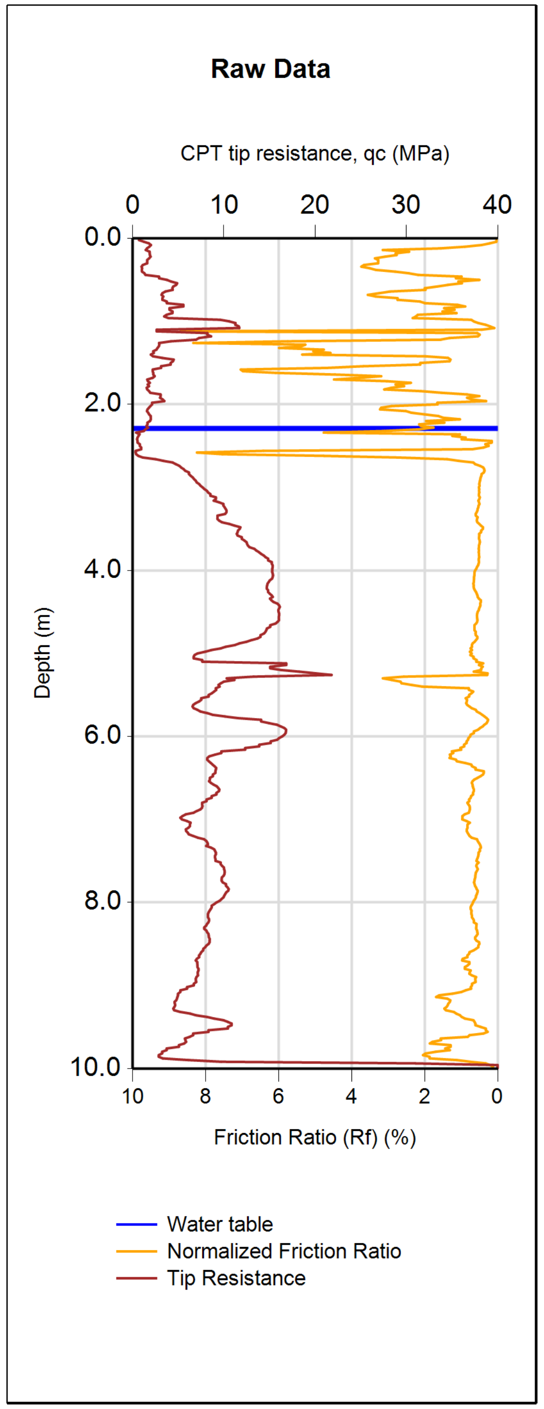

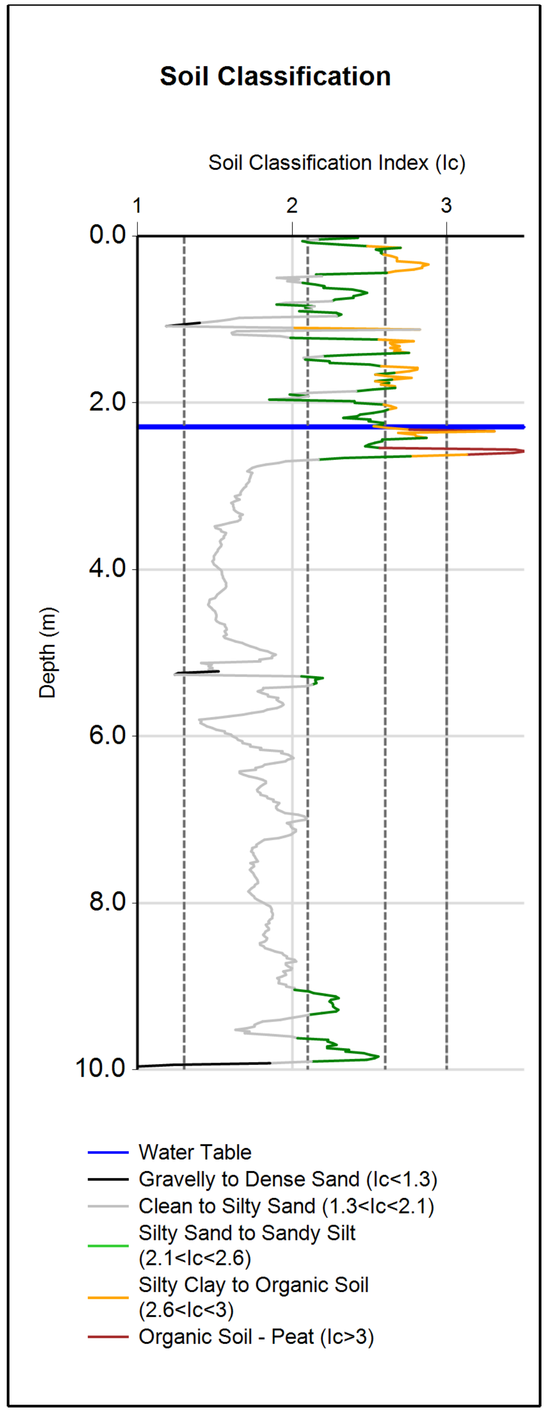

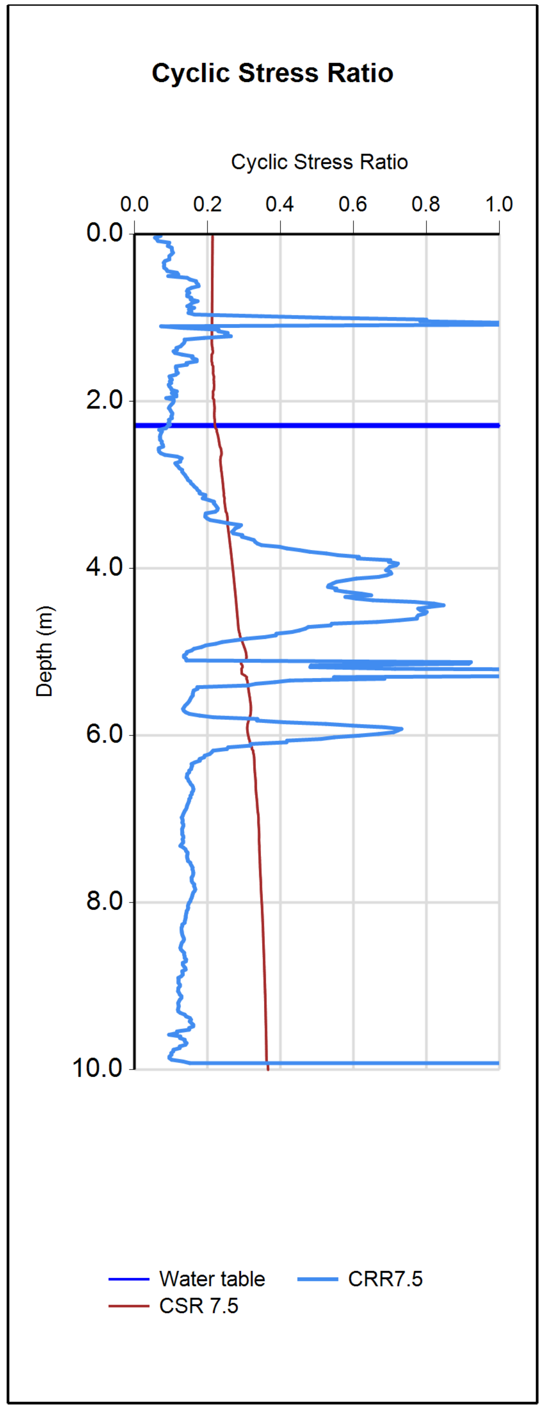

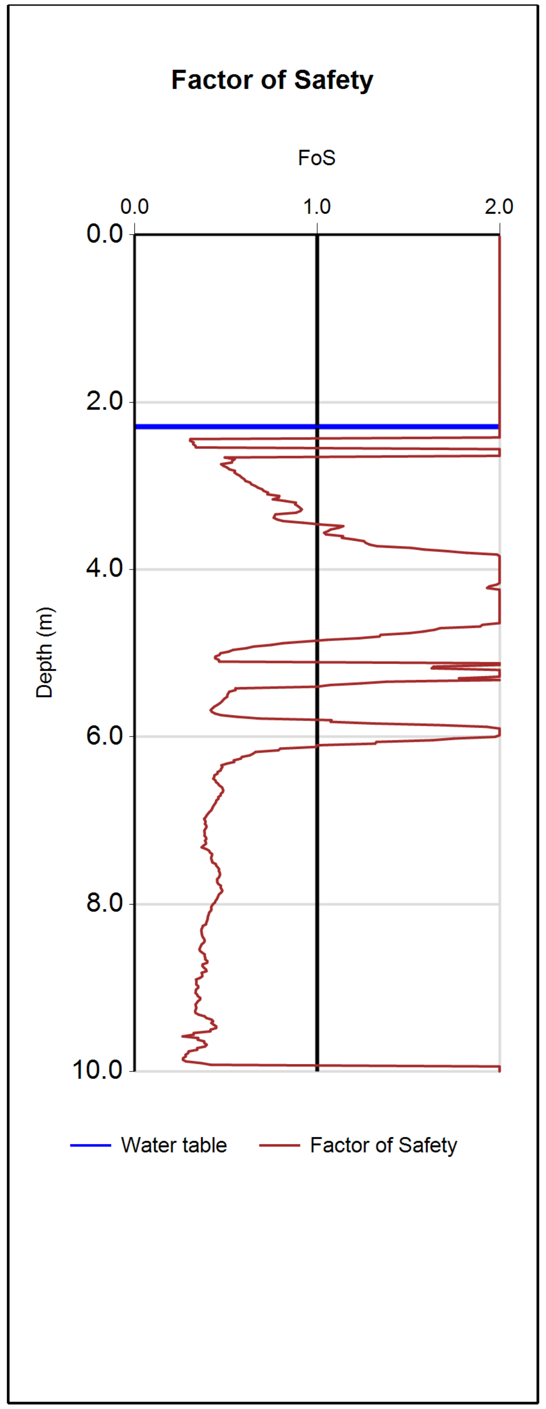

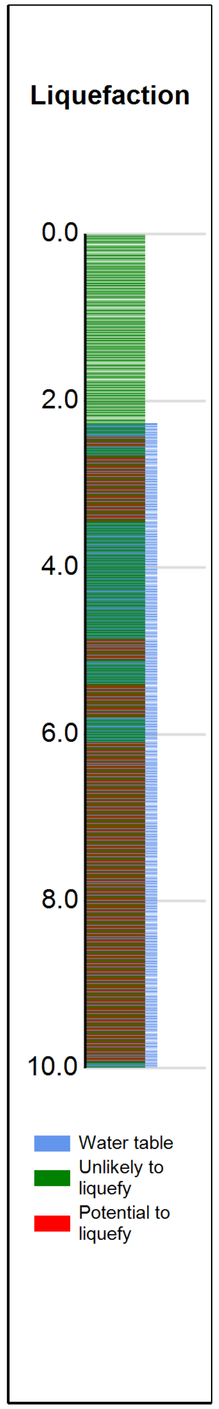

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT1	20770	8/05/2013	User Specified	7.5	0.36	2.3	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	127	5.4	18	22	2.5

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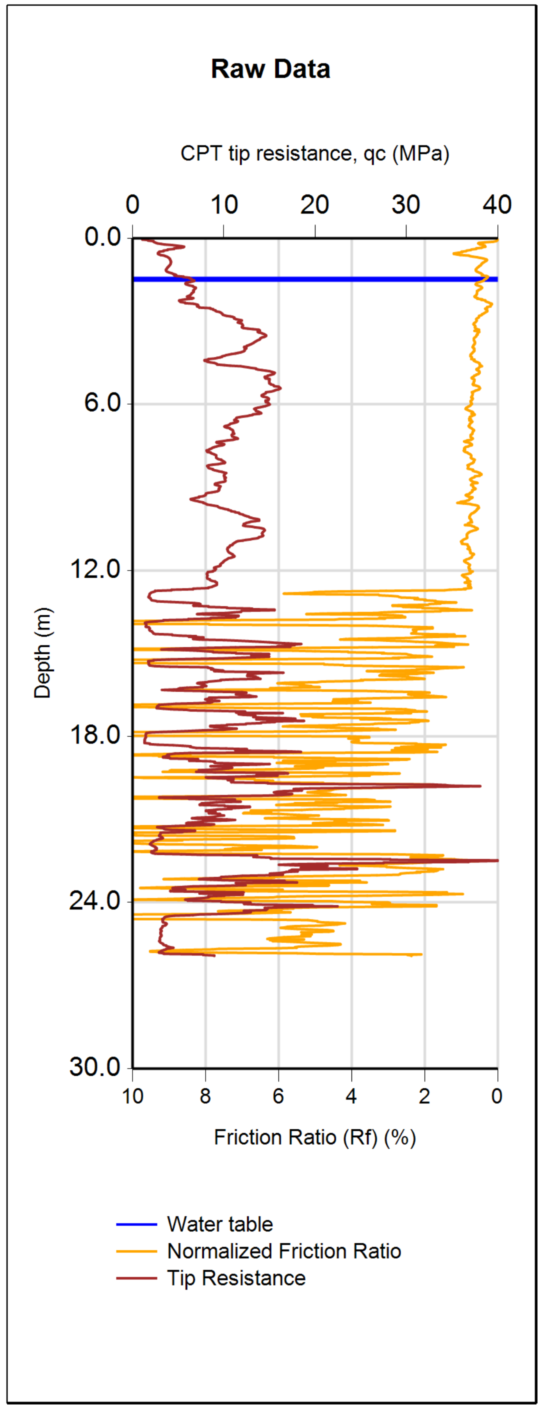

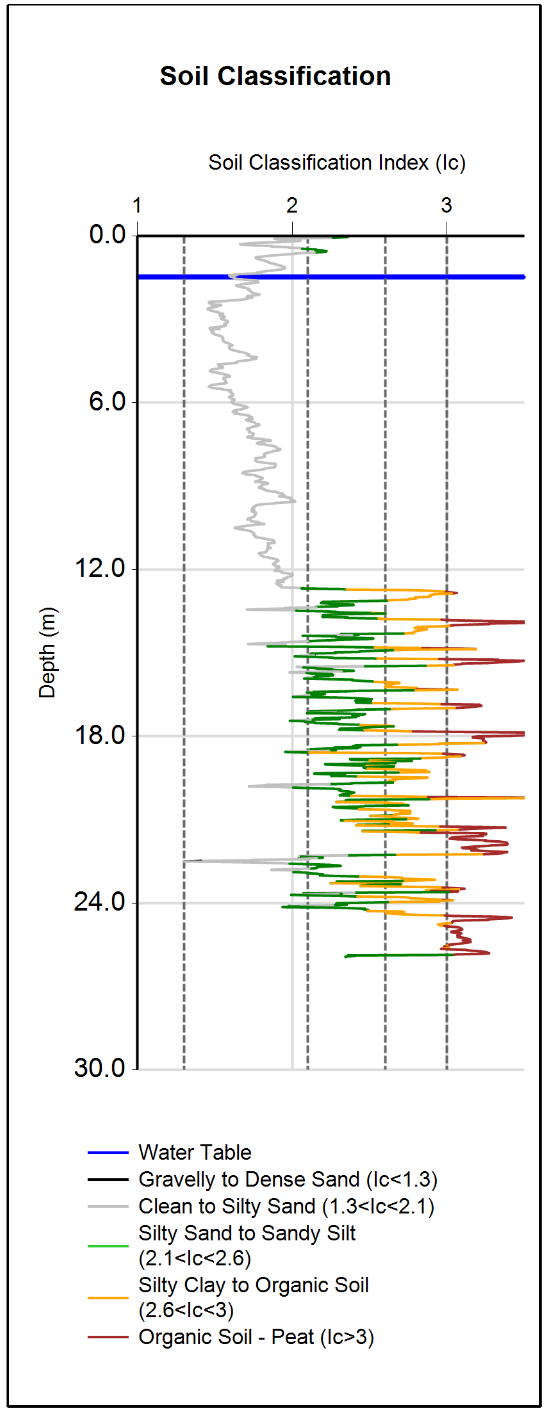

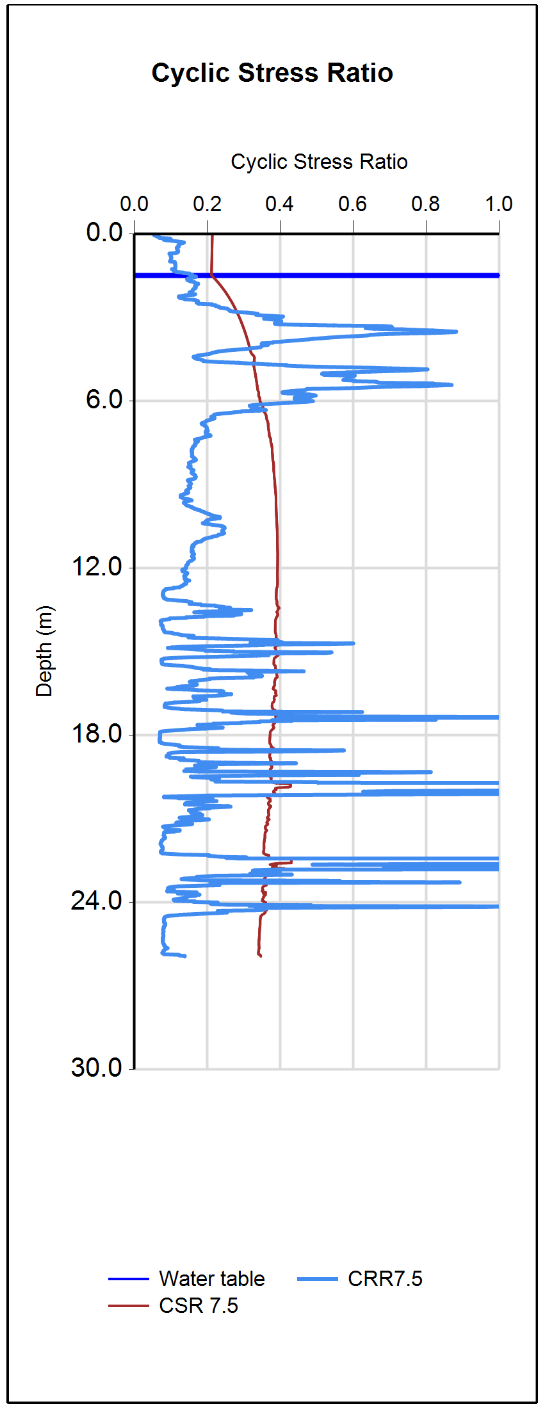

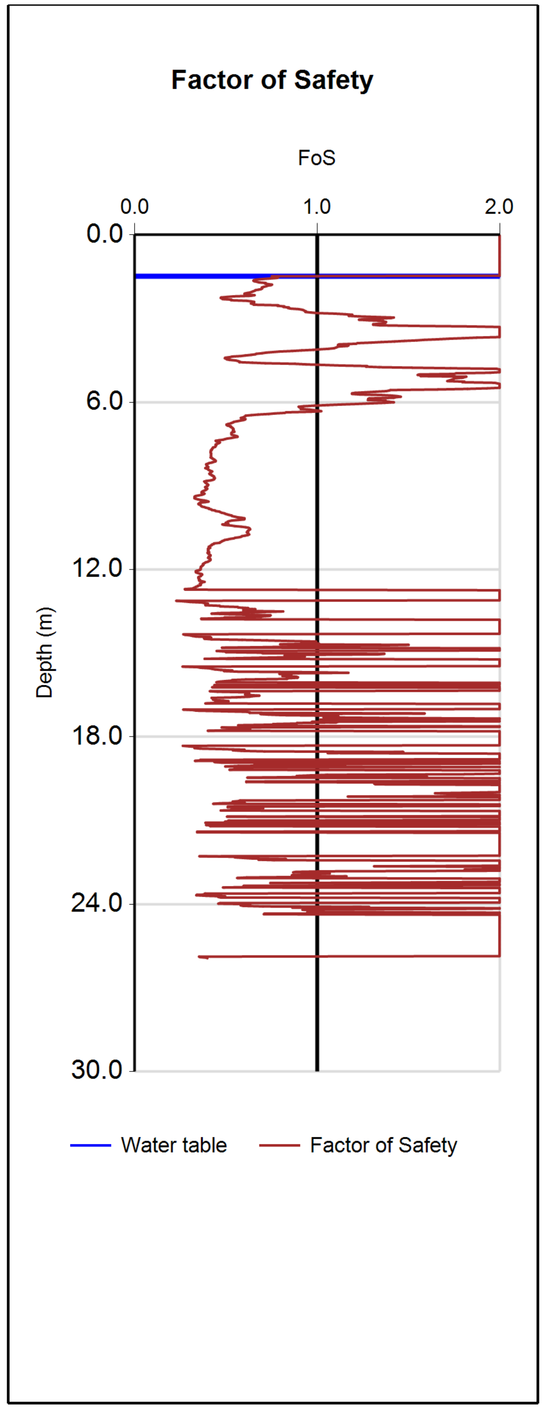

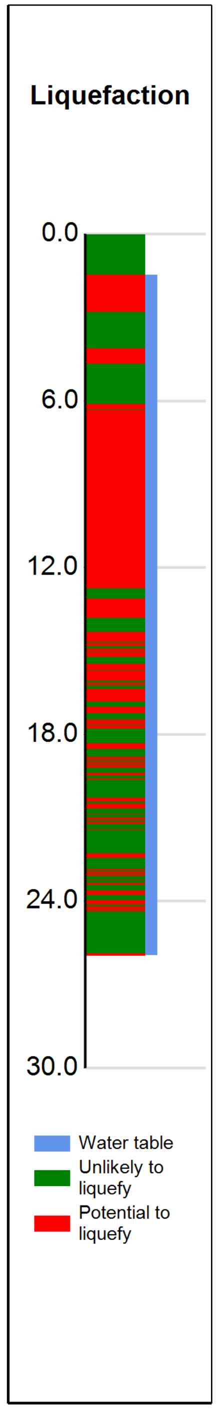

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT2	20879	7/05/2013	User Specified	7.5	0.36	1.5	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	266	13.5	26	36	1.6

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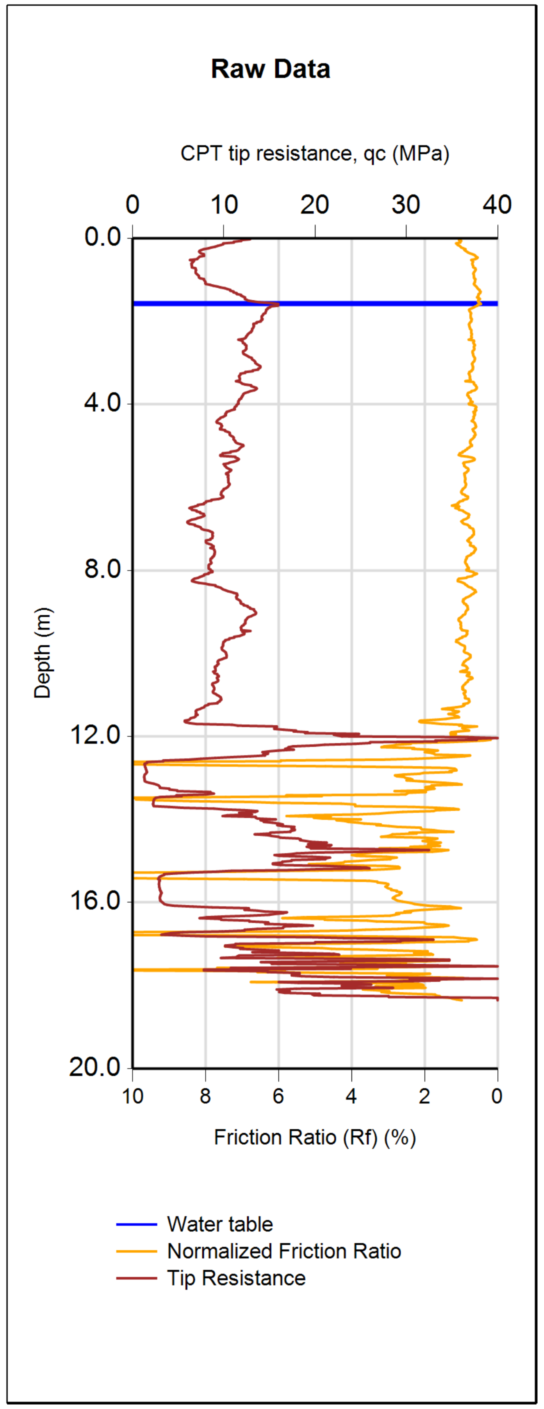

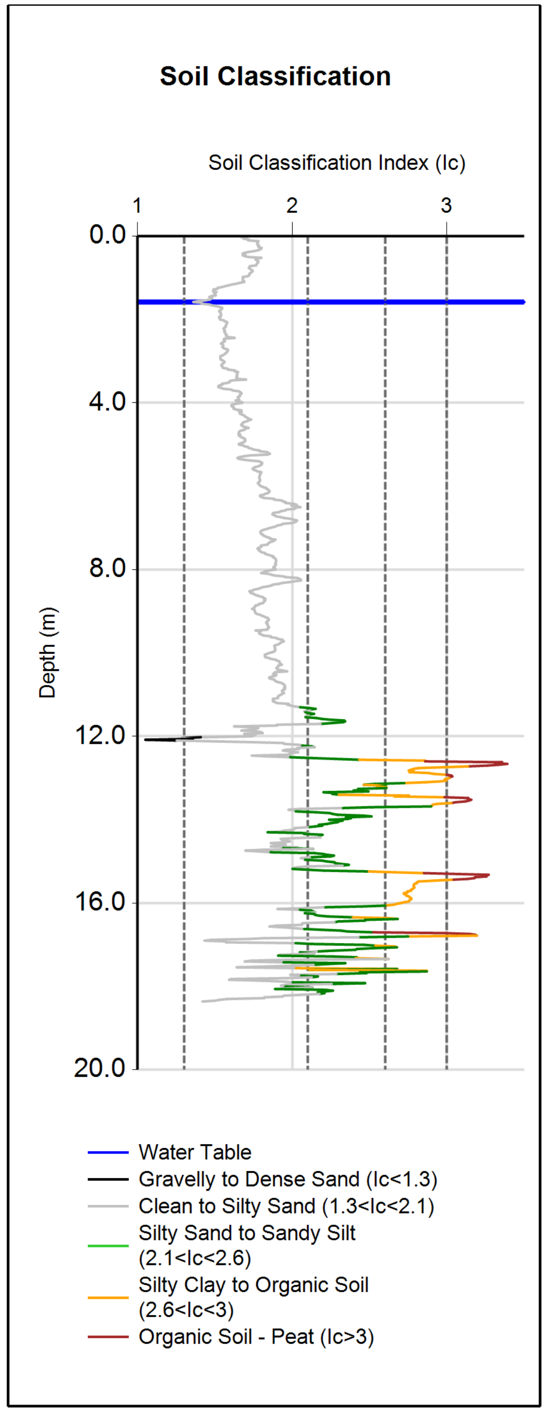

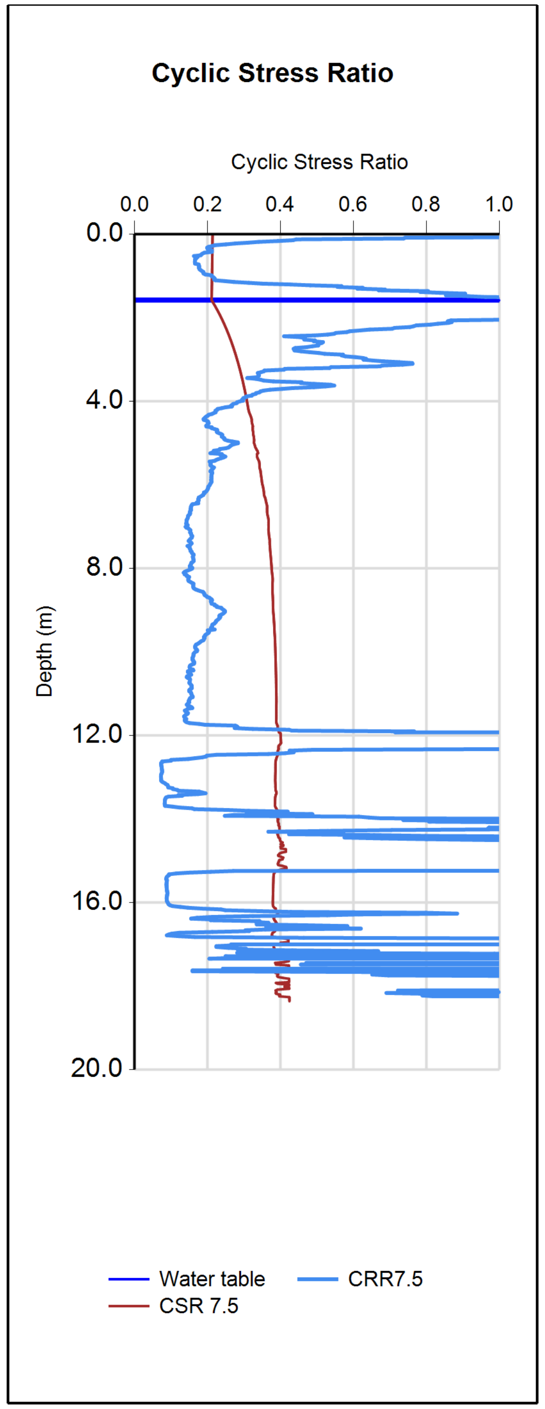

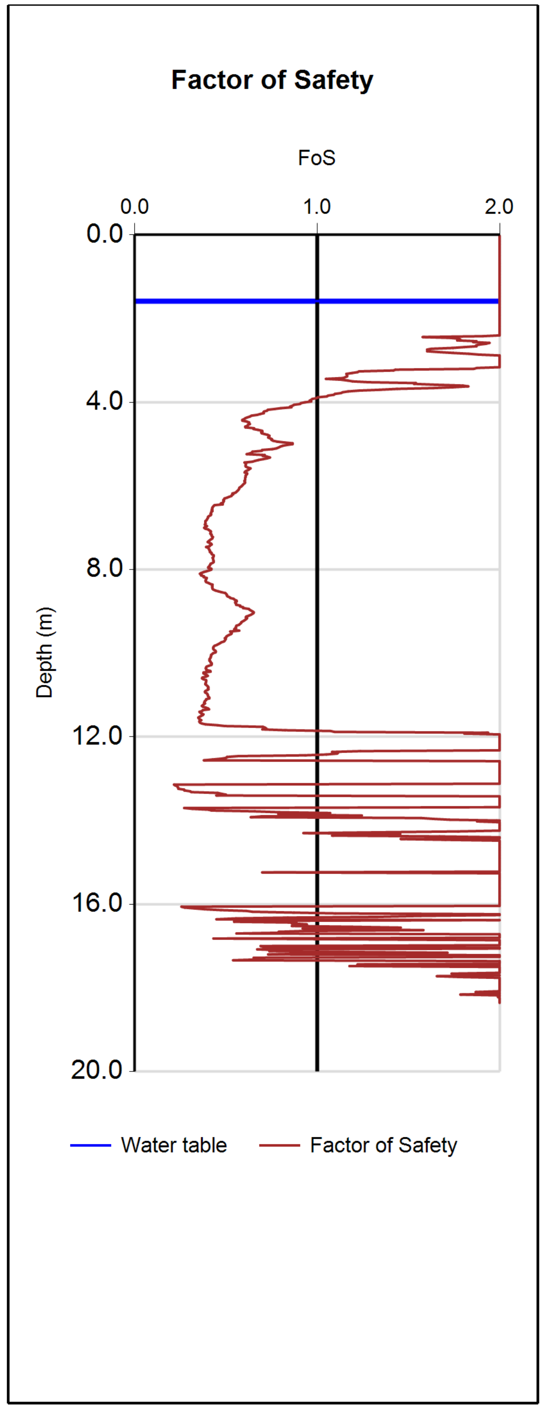

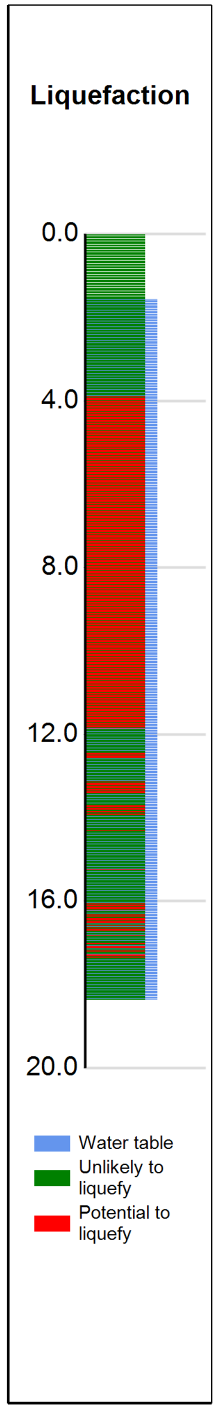

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT3	20861	4/05/2013	User Specified	7.5	0.36	1.6	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	185	9.3	24	23	4

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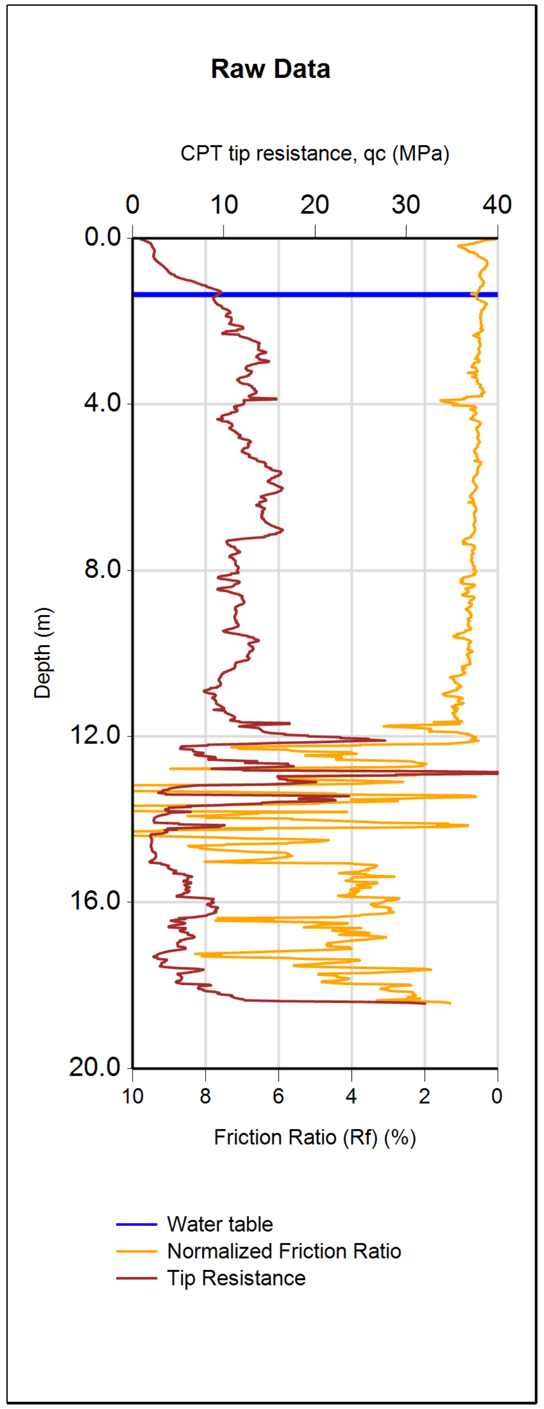

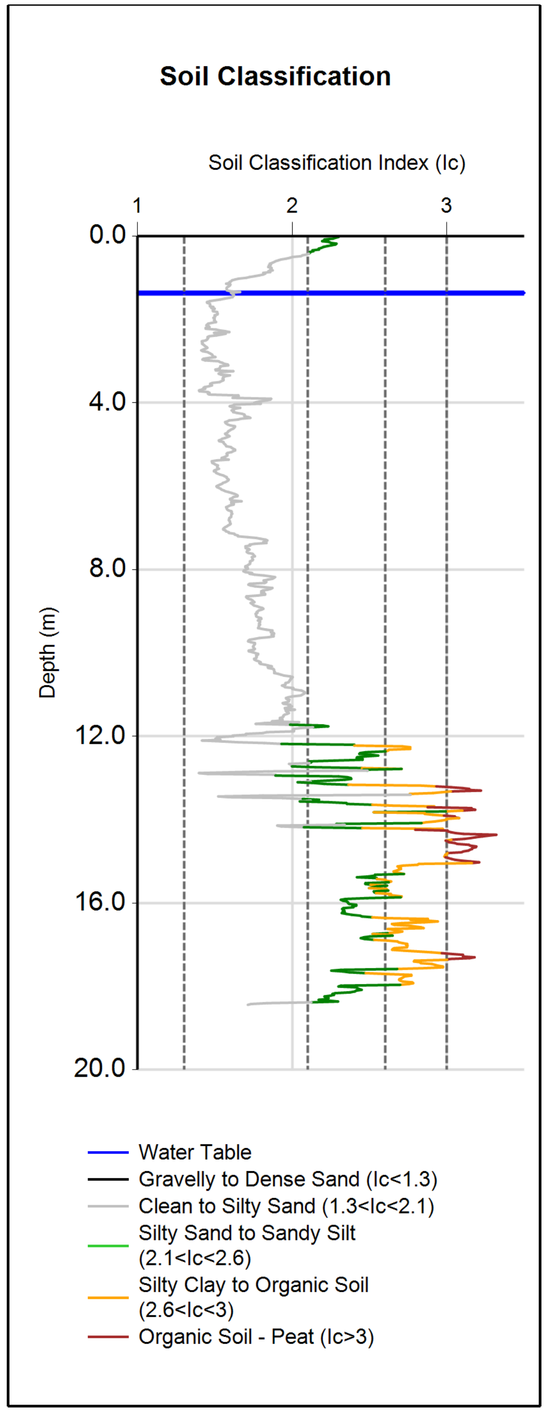

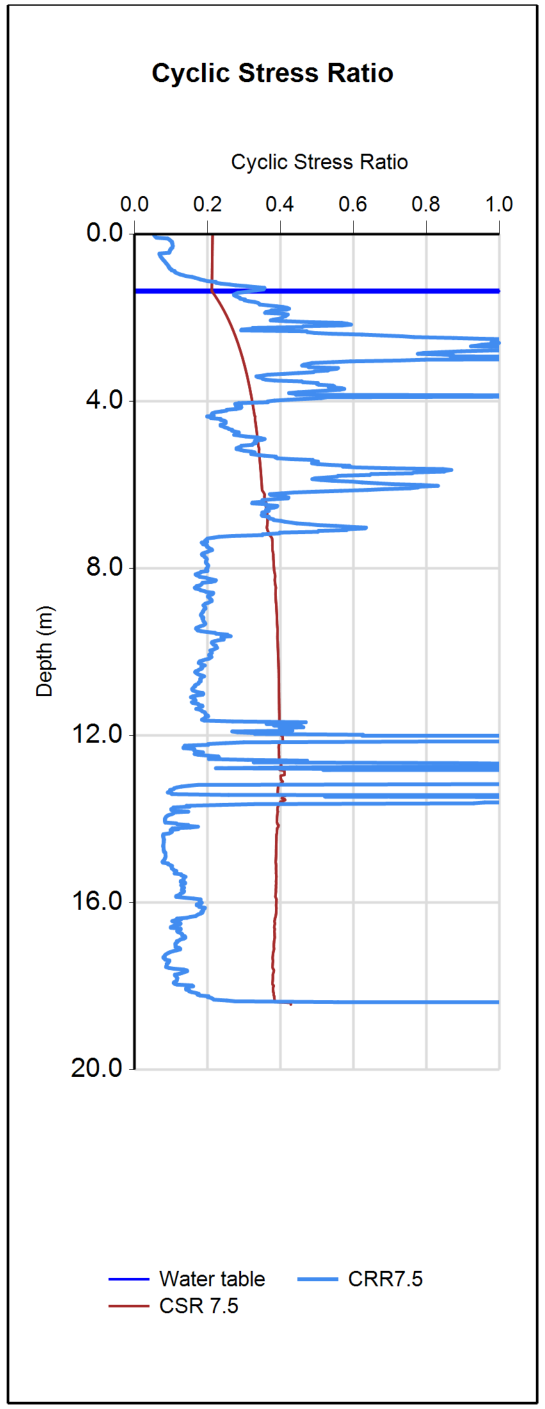

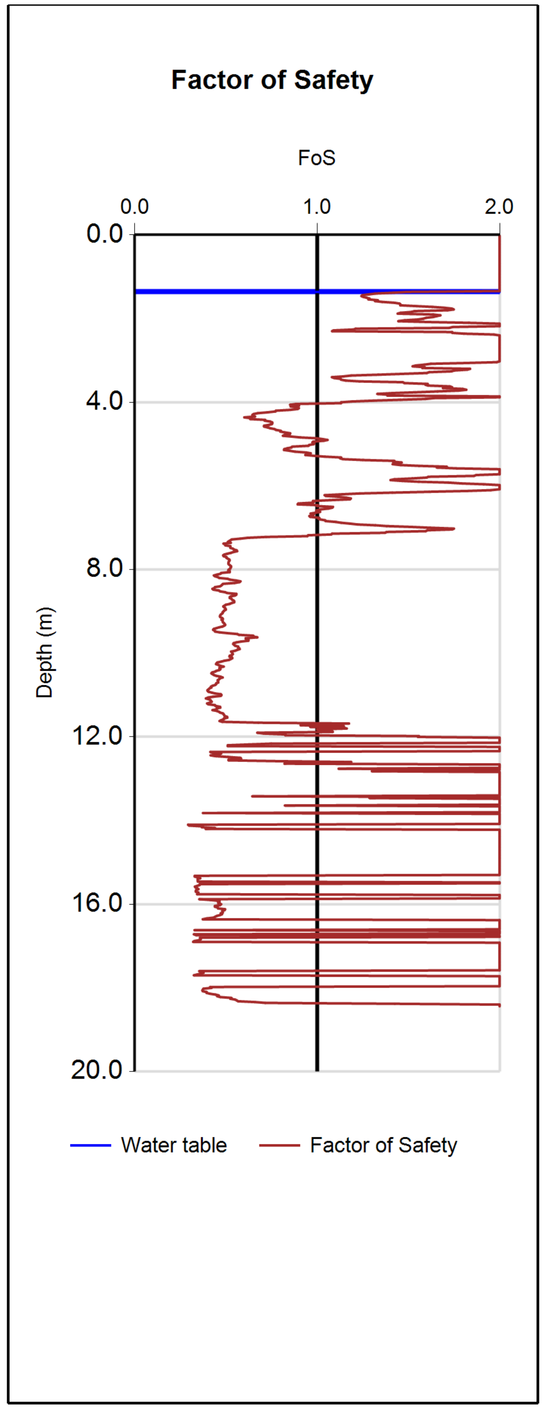

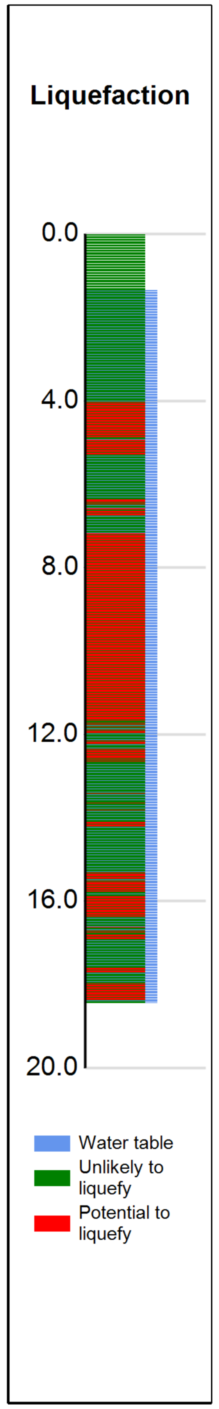

											(Assumed predrill values)		
	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT5	20866	4/05/2013	User Specified	7.5	0.36	1.4	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	157	8.3	16	18	4.1

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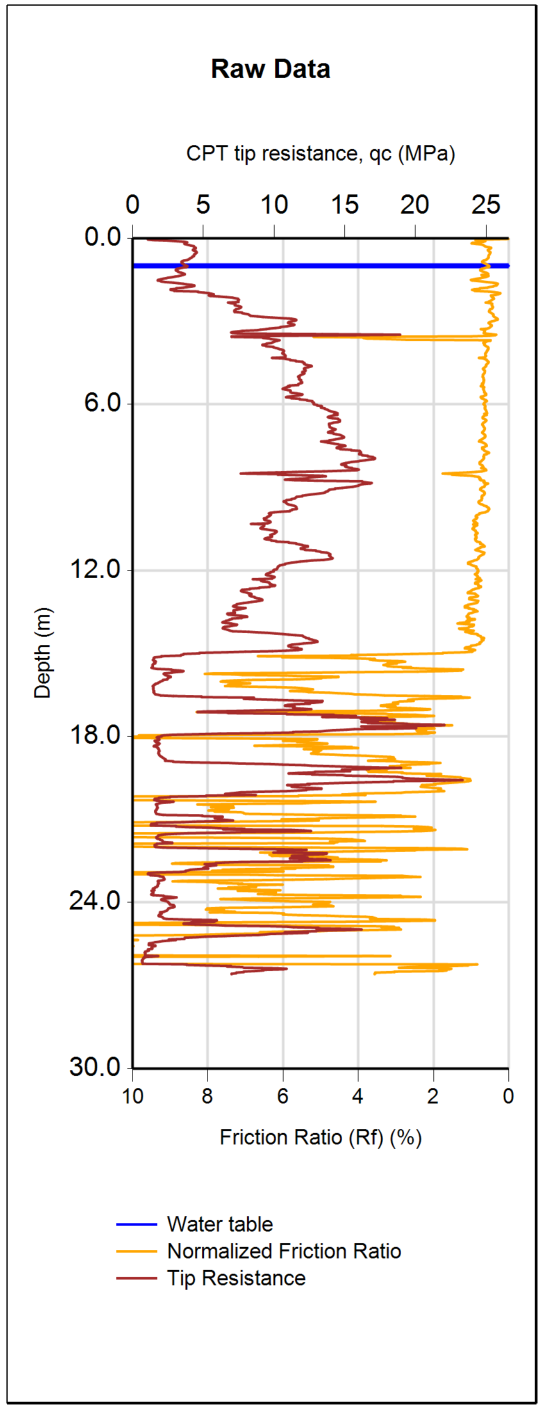

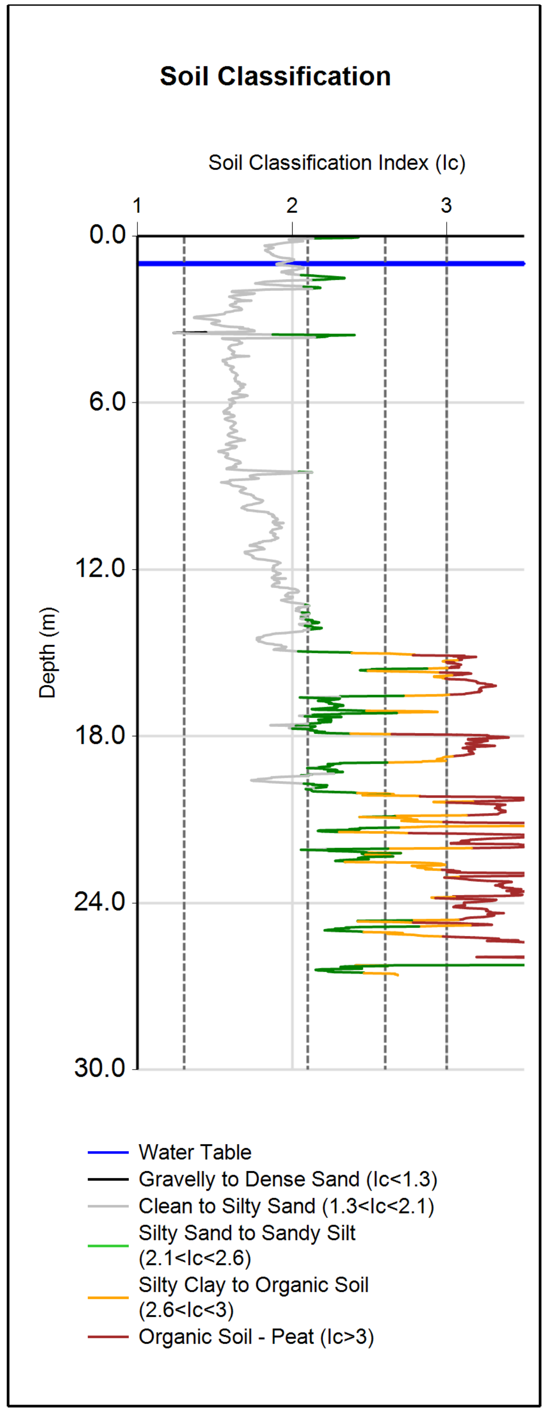

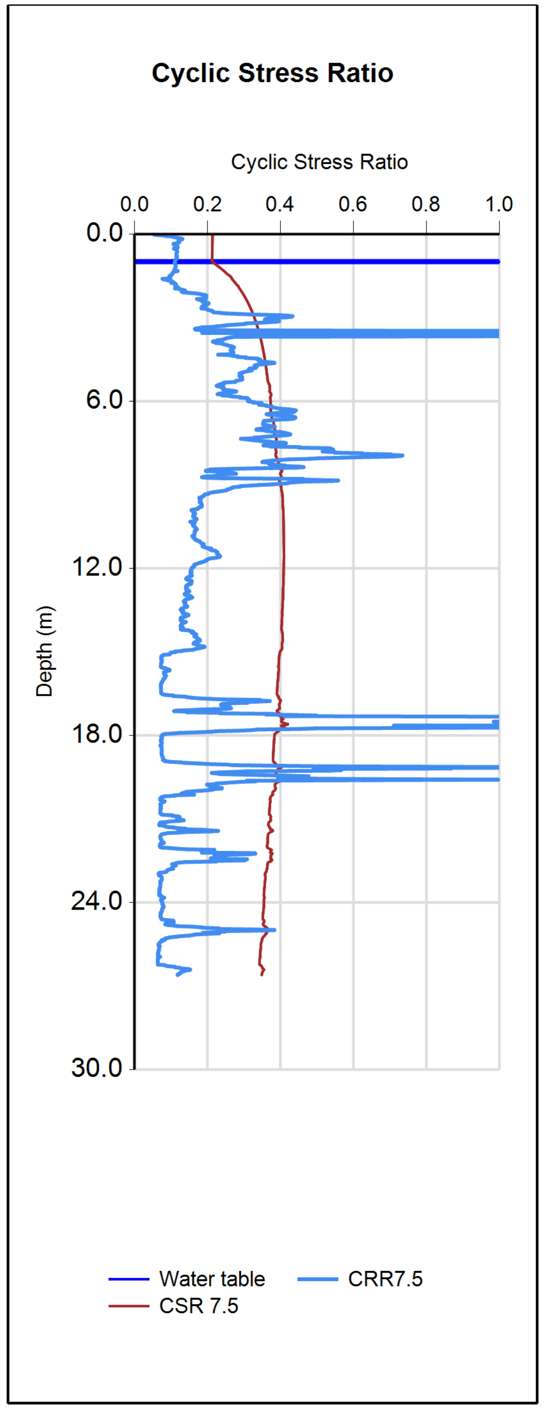

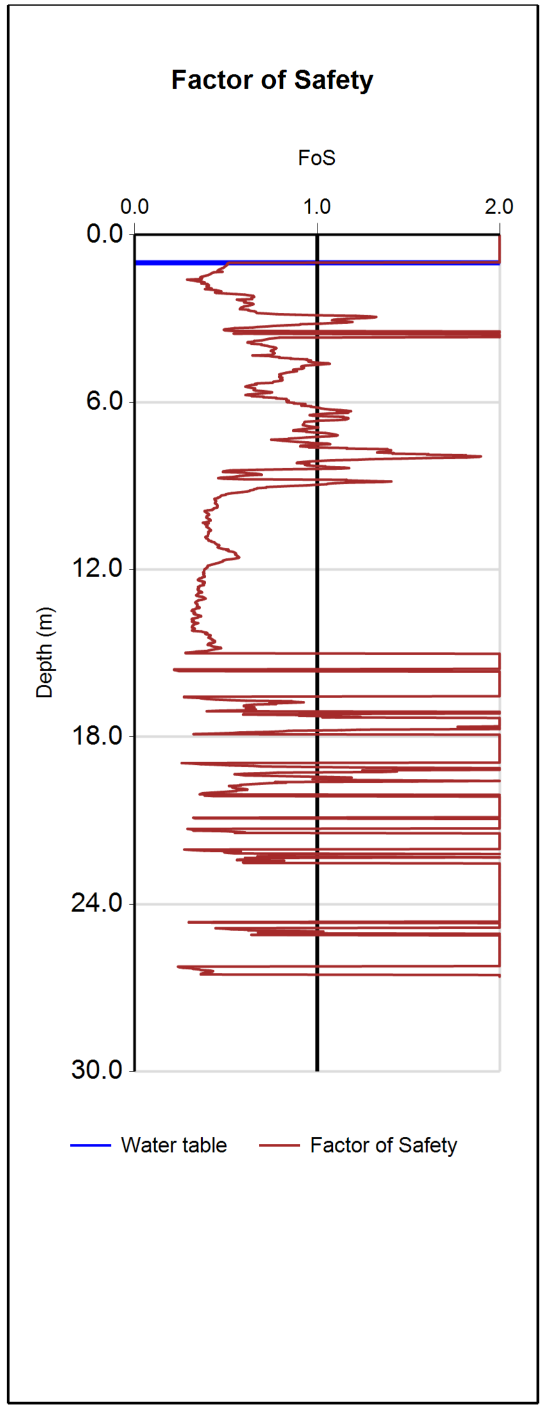

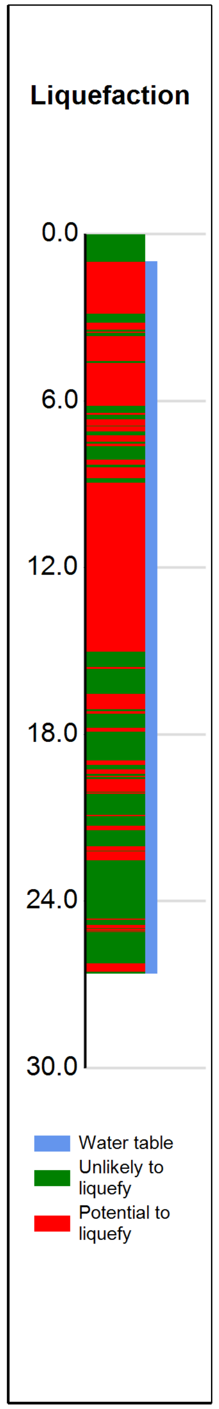

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT6	20867	7/05/2013	User Specified	7.5	0.36	1.0	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	285	14.9	29	52	1.1

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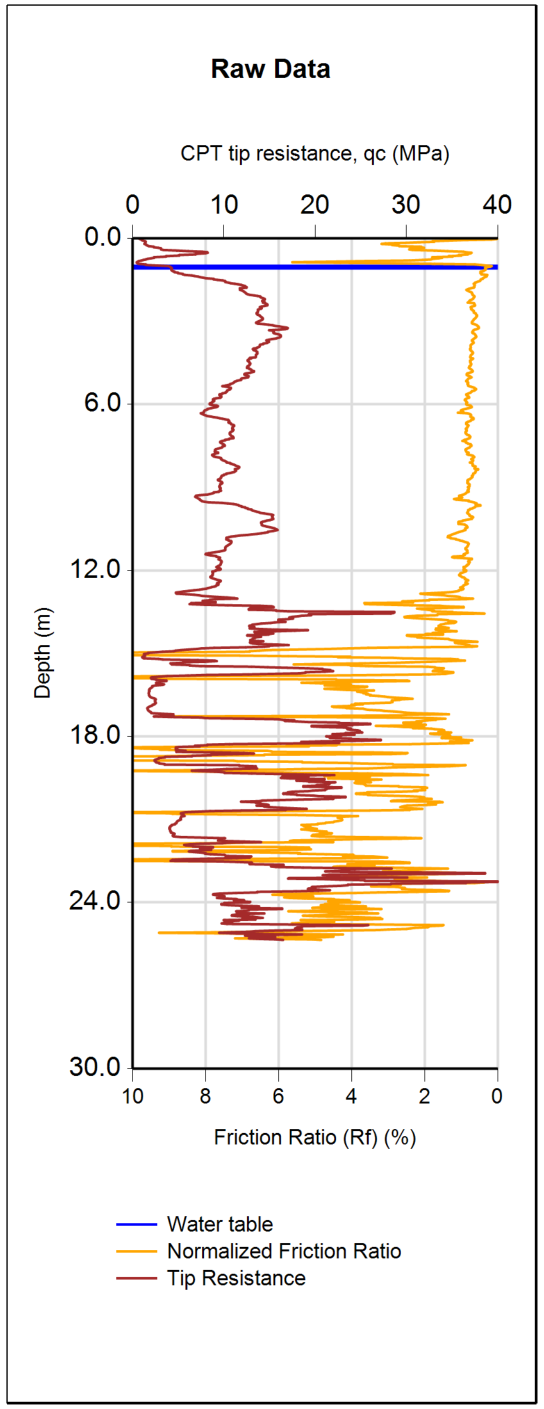

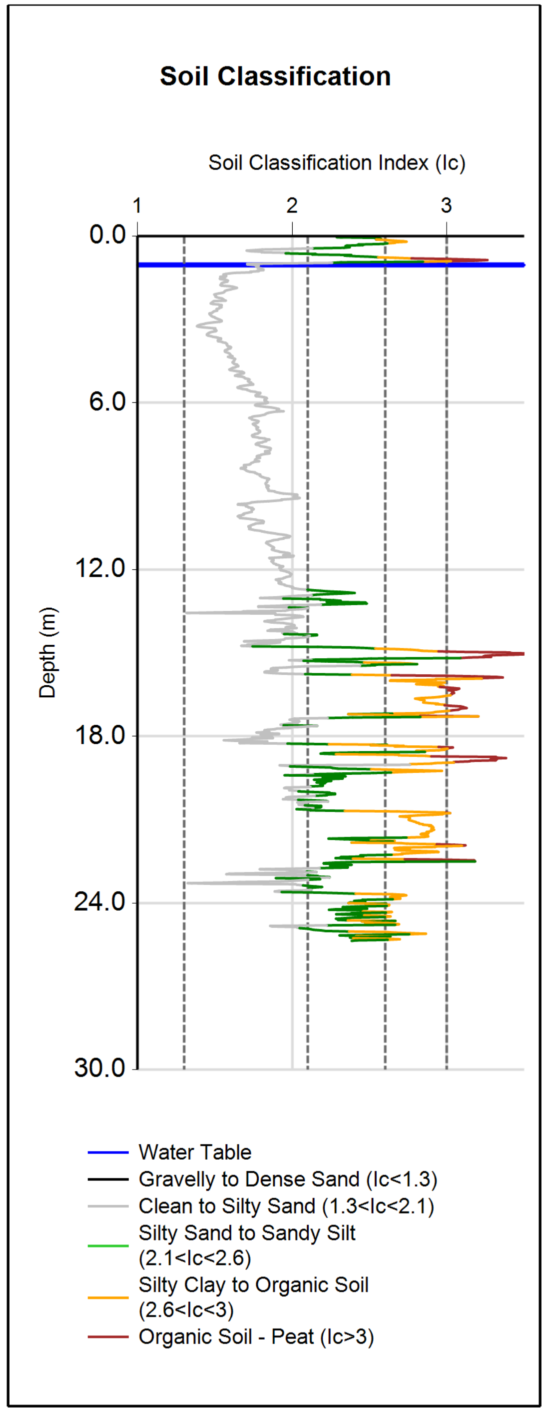

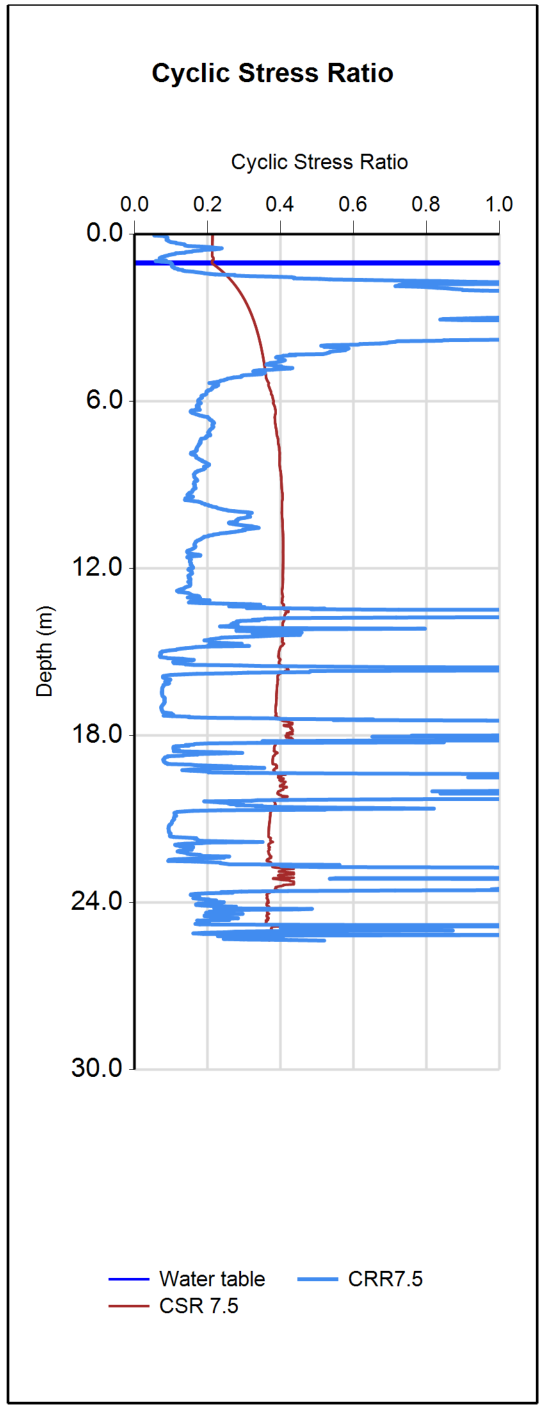

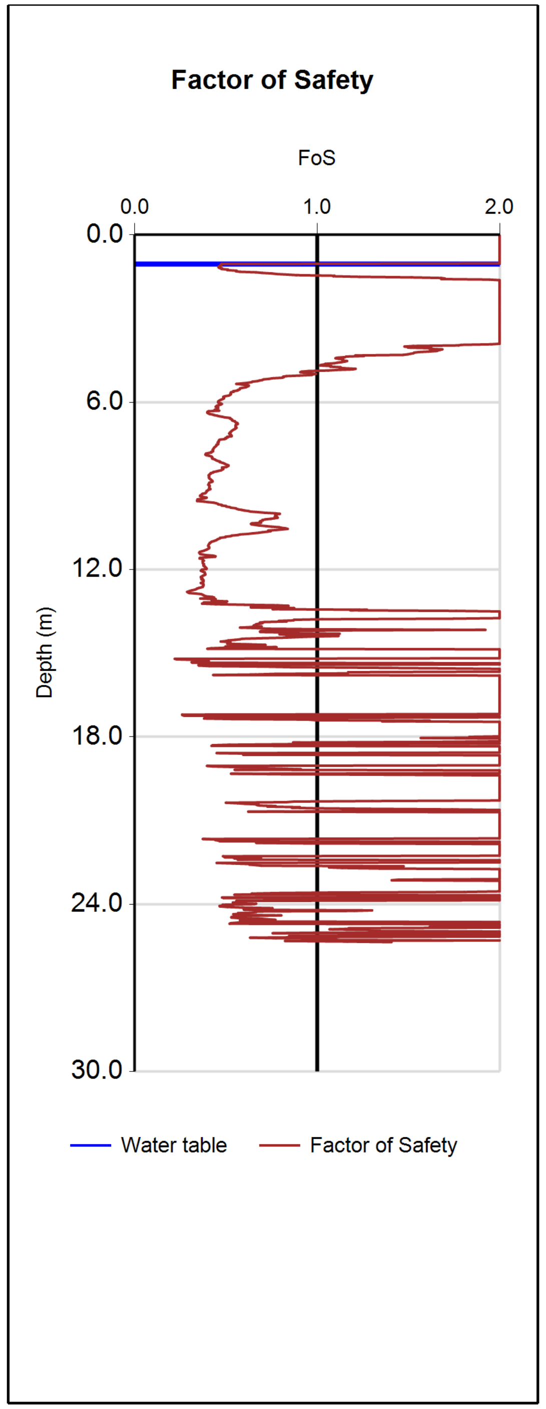

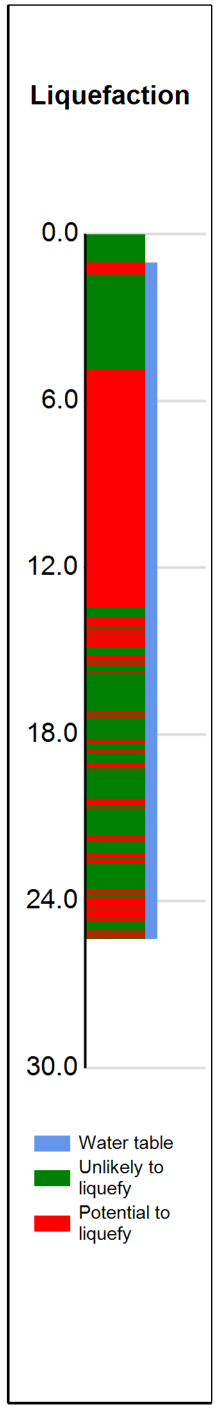

(Assumed predrill values)

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT7	20868	8/05/2013	User Specified	7.5	0.36	1.0	IB	ZRB	0	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	239	12.5	26	32	1.1

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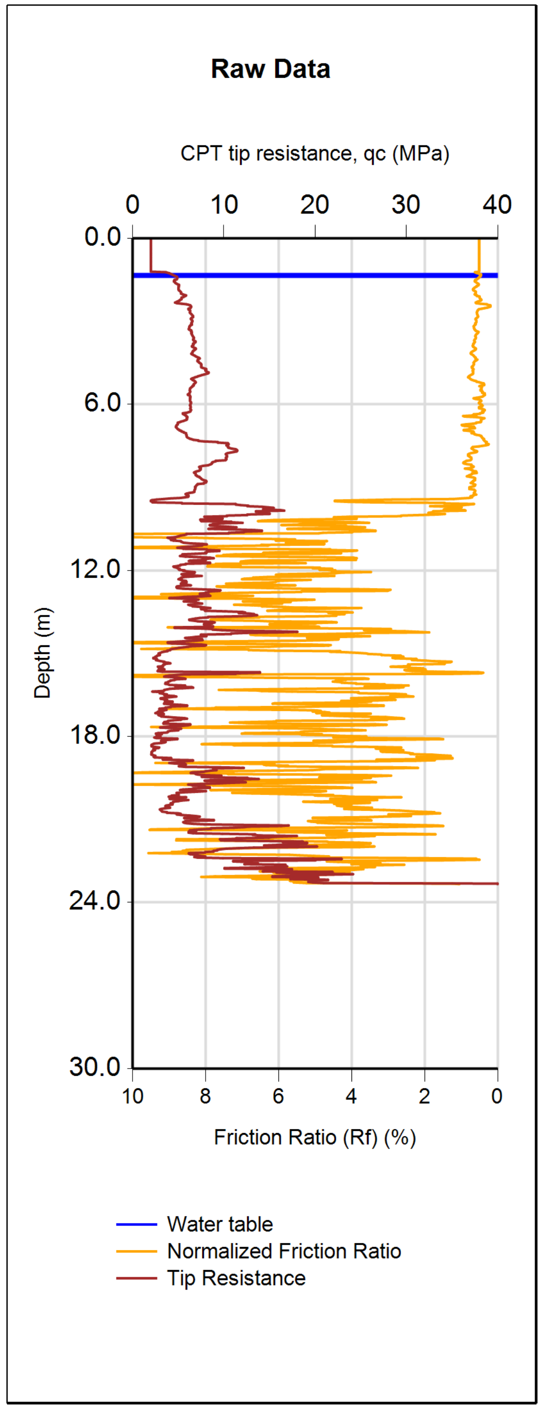

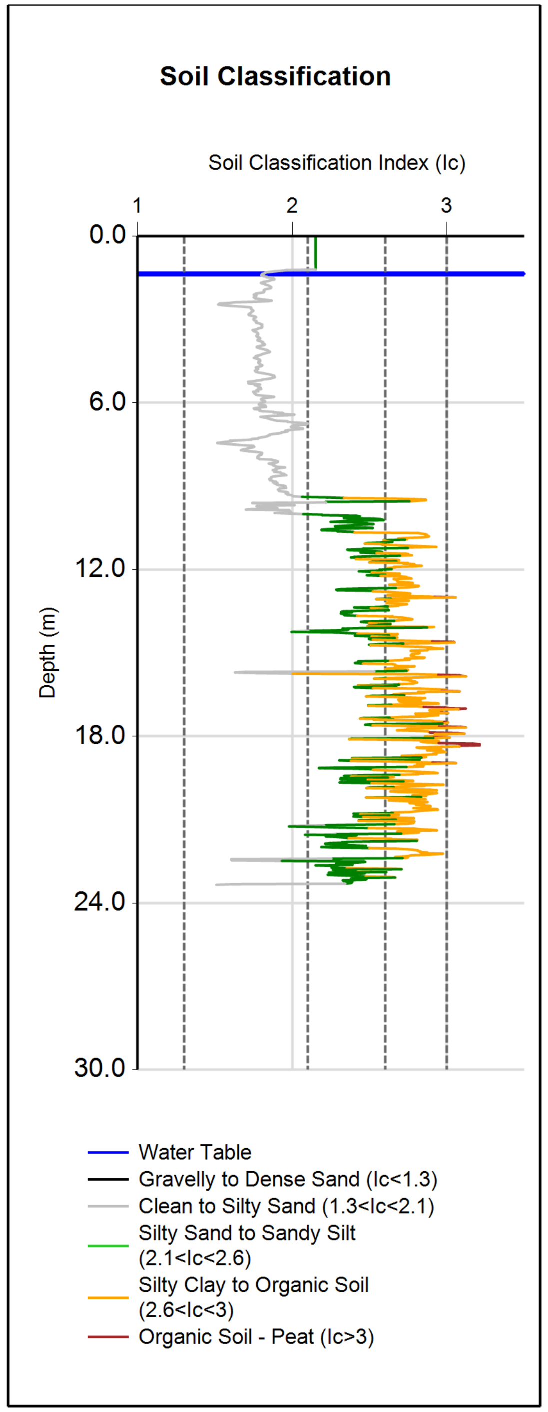

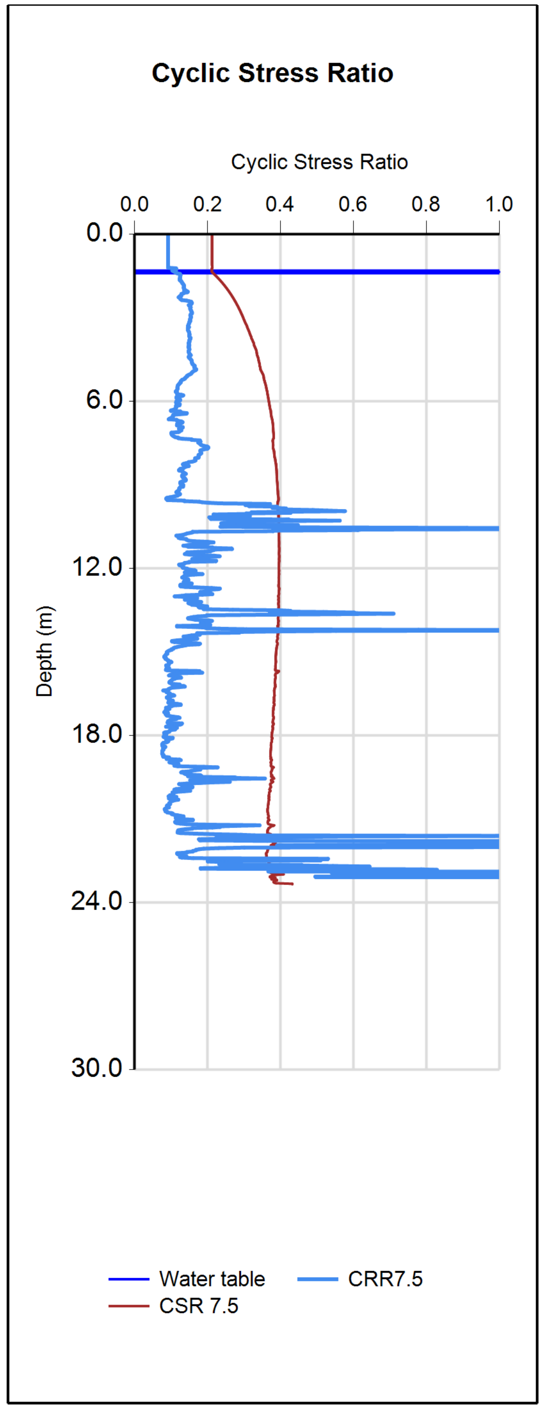

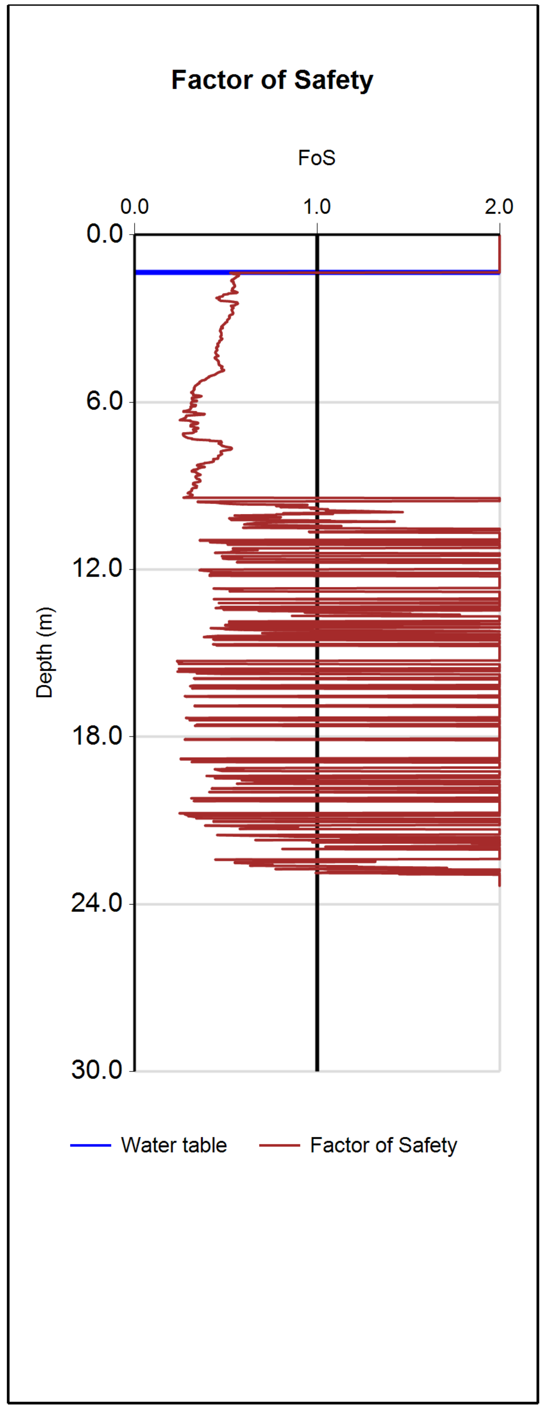

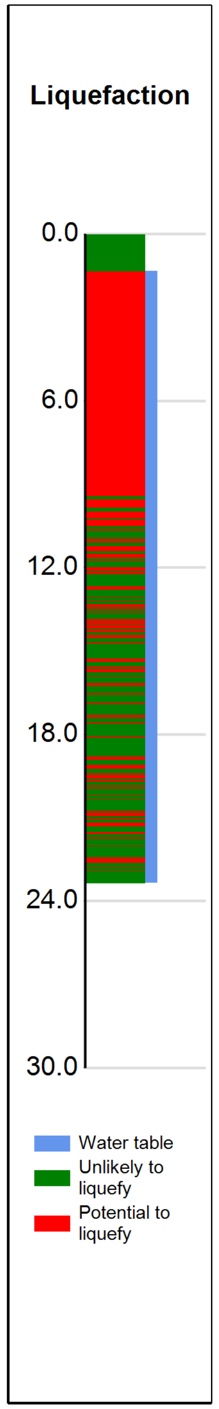

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT8	20870	7/05/2013	User Specified	7.5	0.36	1.4	IB	ZRB	1.2	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	278	12.2	38	53	1.4

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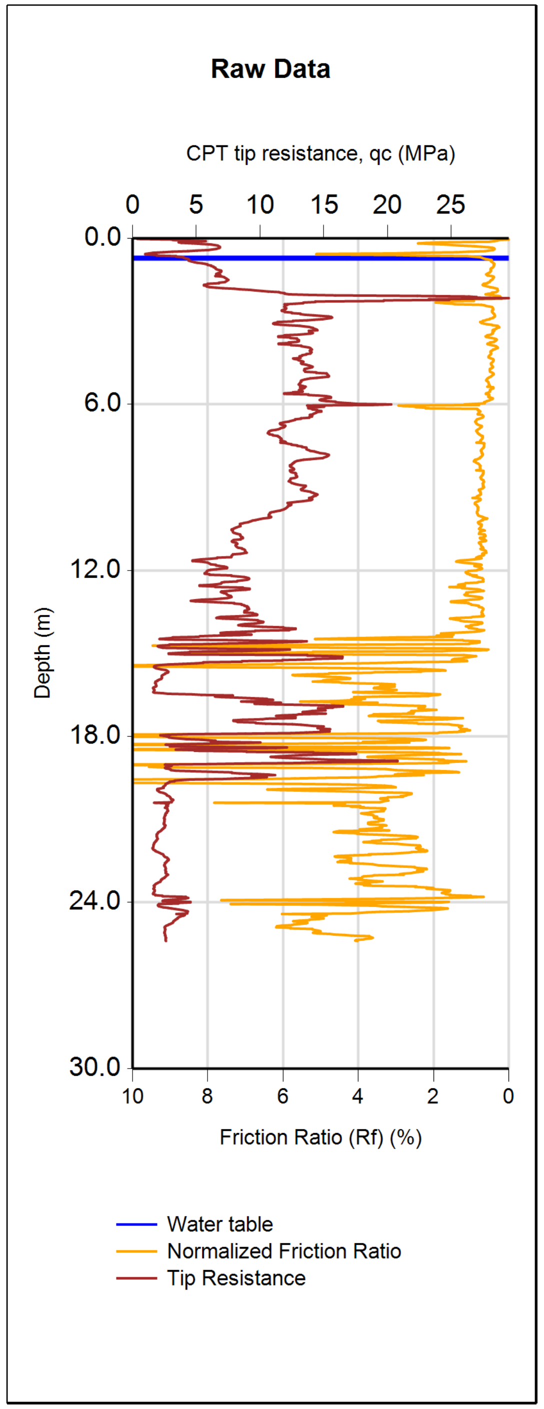
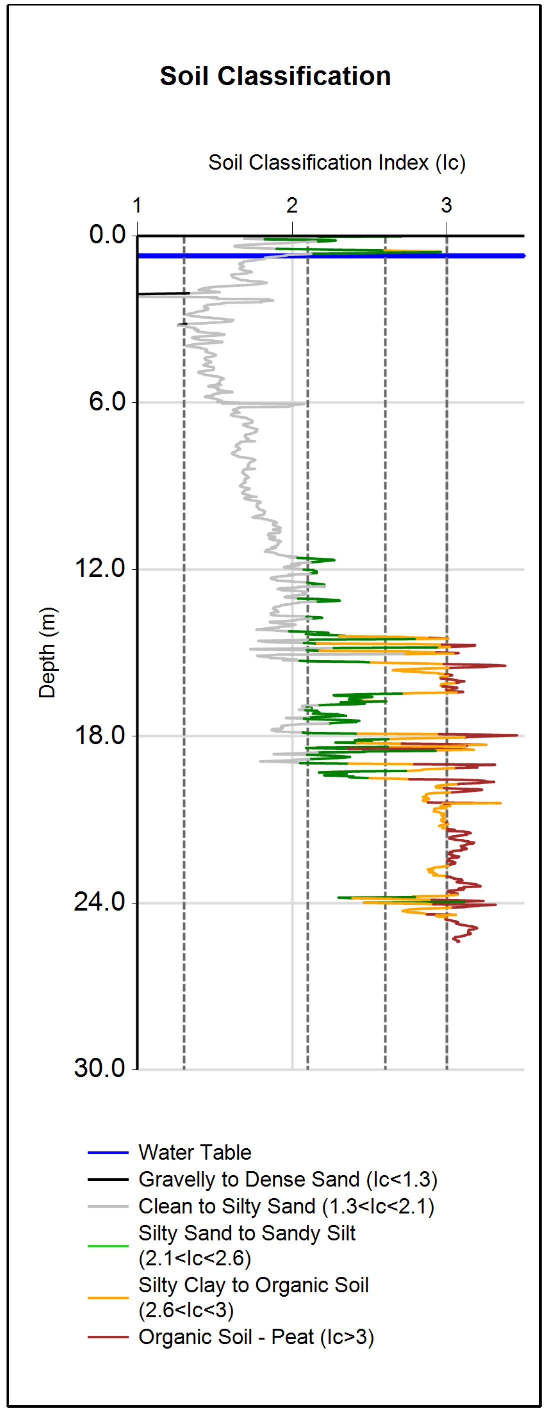
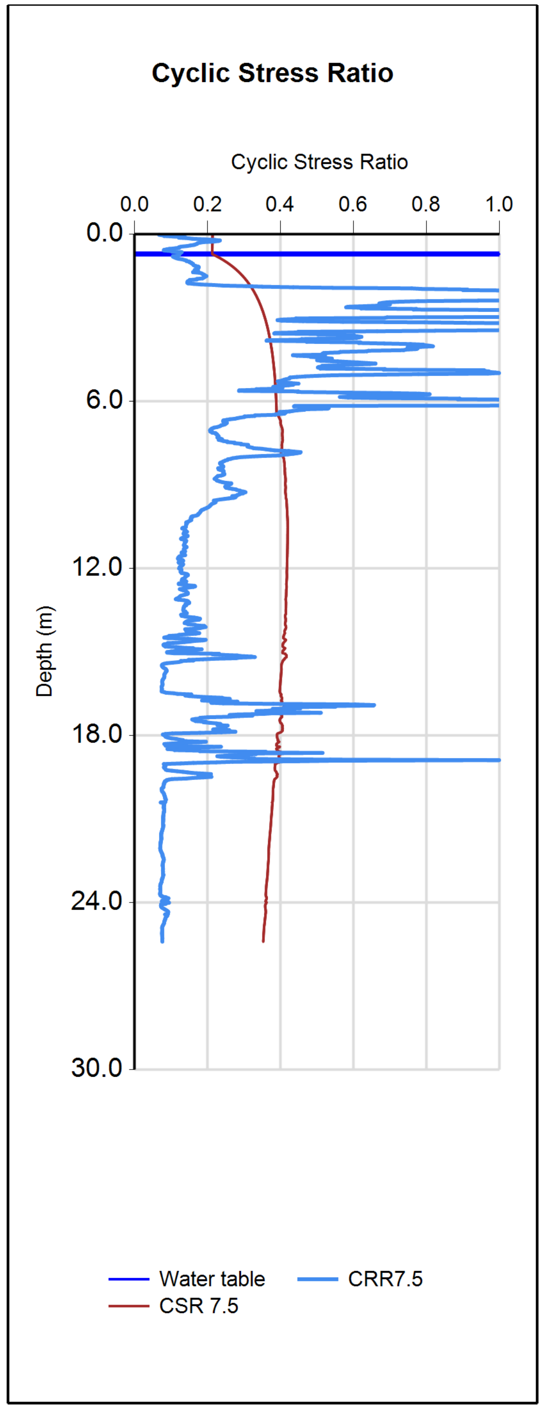
Raw Data



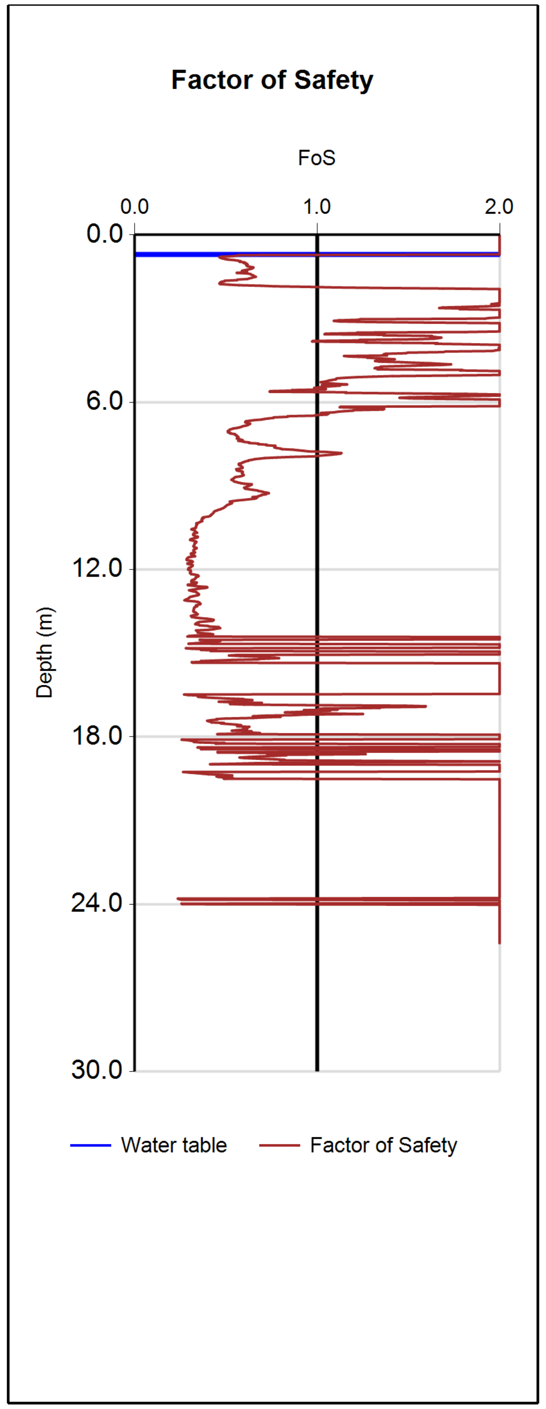
Soil Classification



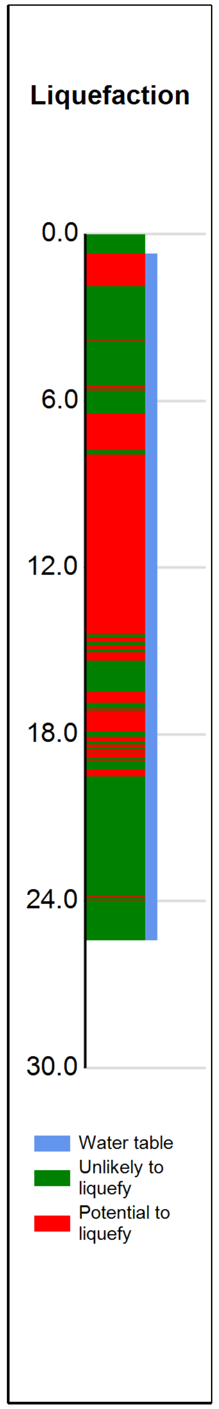
Cyclic Stress Ratio



Factor of Safety



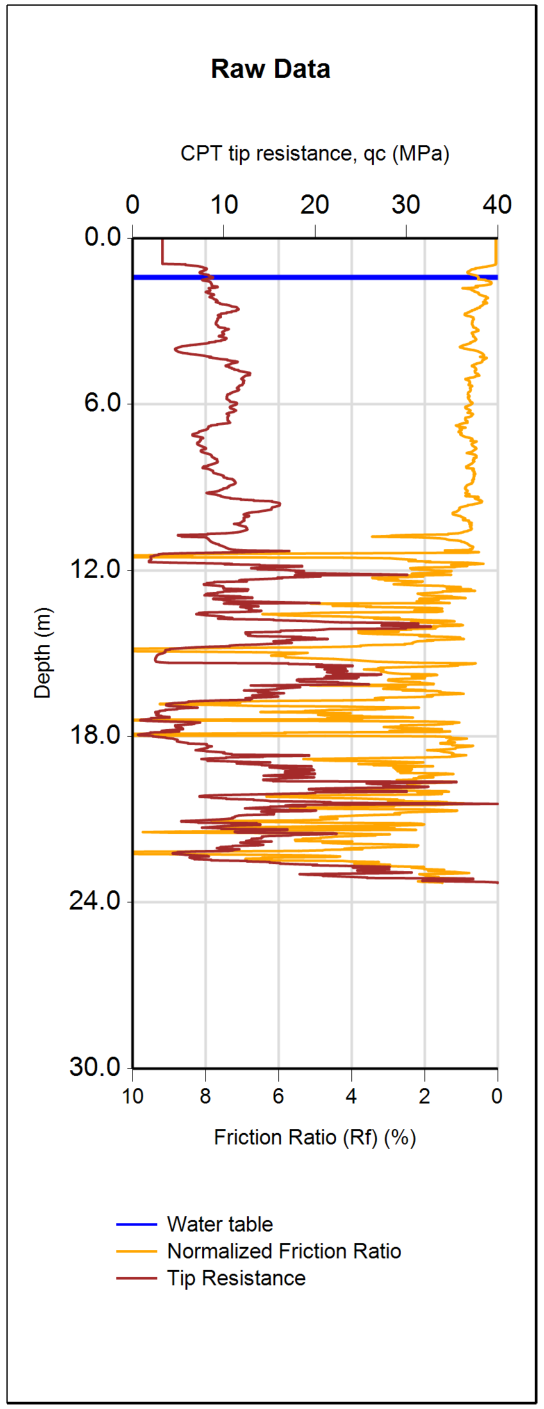
Liquefaction

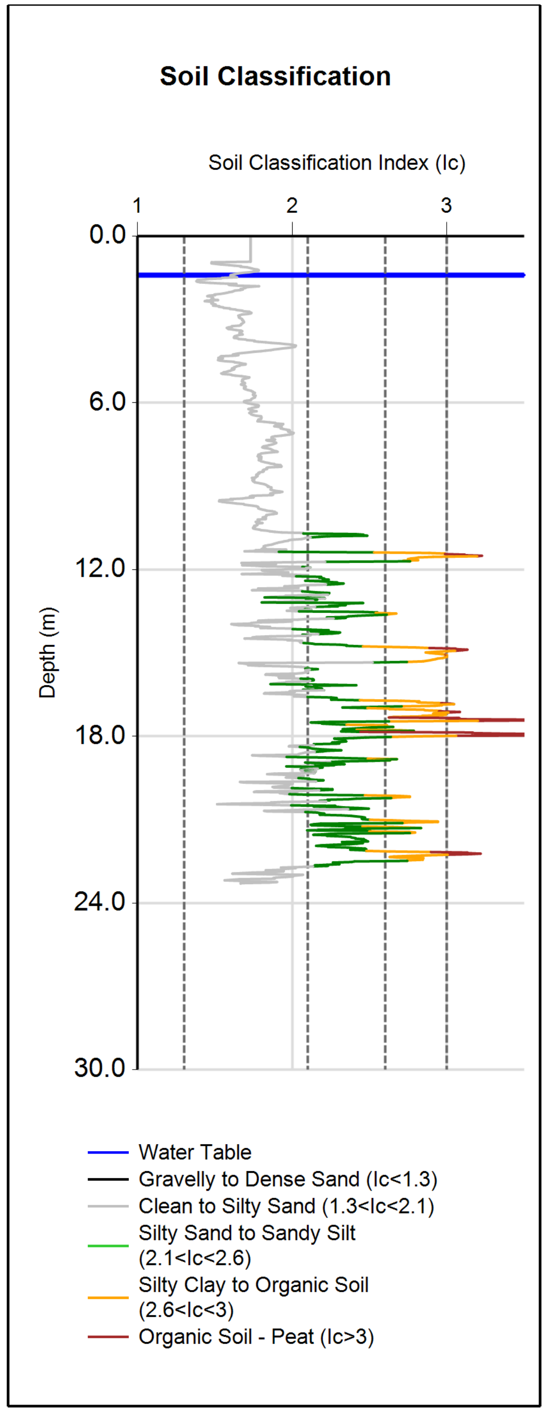


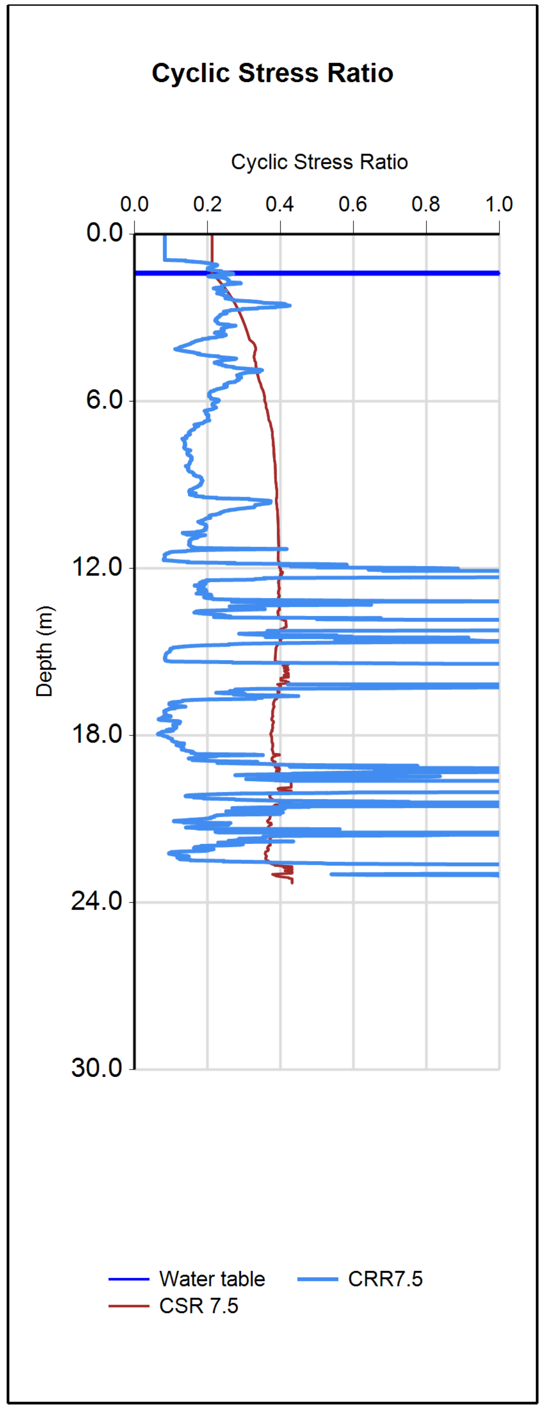
	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT9	20873	7/05/2013	User Specified	7.5	0.36	0.7	IB	ZRB	0	2	0.01	18

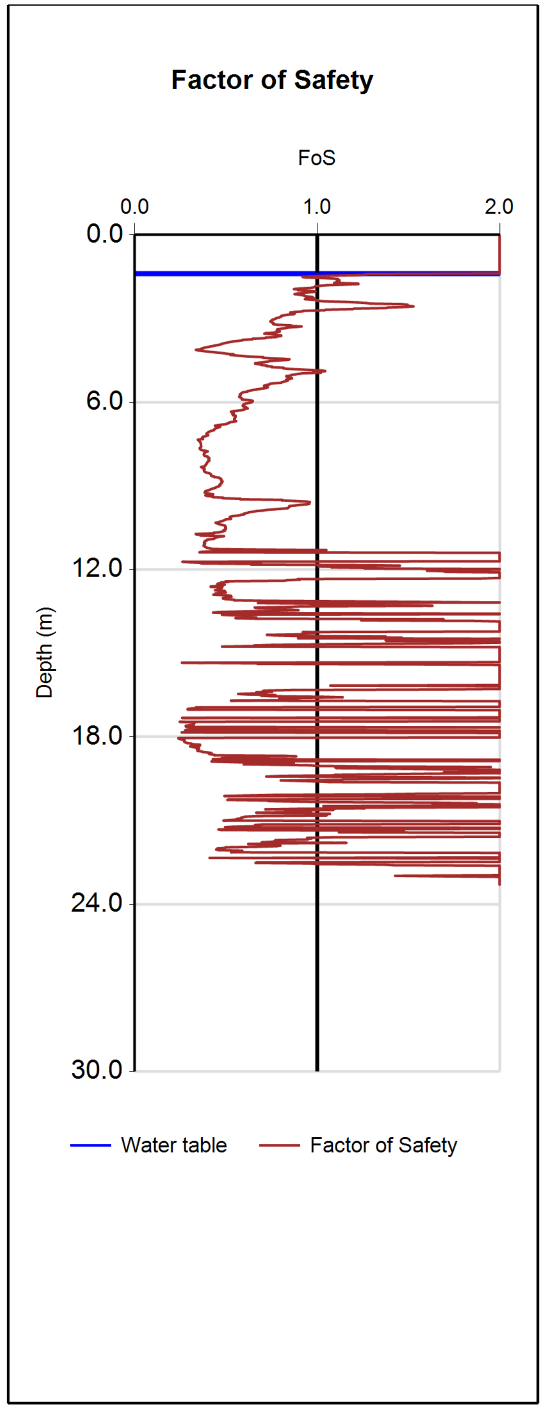
	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	245	11.9	25	42	0.8

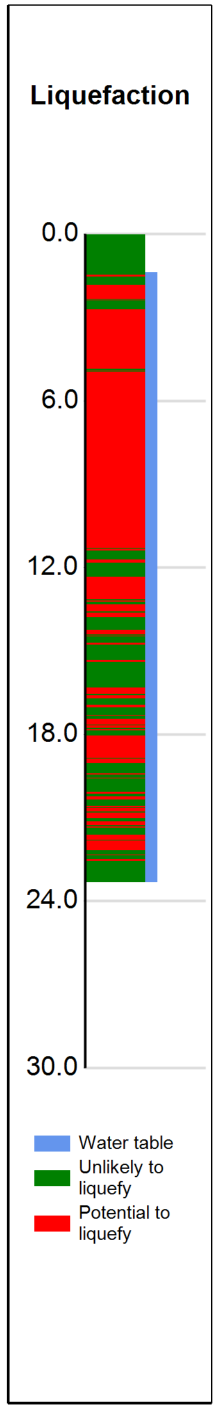
Tonkin & Taylor **Tonkin & Taylor** Environmental and Engineering consultants V1.2	CLIENT, PROJECT **Nelson City Council** **Tahunanui Liquefaction Assessment**	LOCATION Tahunanui. Nelson	DATE 10/06/2013 ANALYSED mjl CHECKED
	TITLE **Liquefaction Assessment**	JOB NUMBER **871023**	PAGE 9 of 11 pages











(Assumed predrill values)

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT10	20875	4/05/2013	User Specified	7.5	0.36	1.4	IB	ZRB	0.9	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	255	14	27	34	1.9

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CLIENT, PROJECT
Nelson City Council
Tahunanui Liquefaction Assessment

TITLE
Liquefaction Assessment

LOCATION
Tahunanui. Nelson

JOB NUMBER
871023

DATE	10/06/2013
ANALYSED	mjl
CHECKED	
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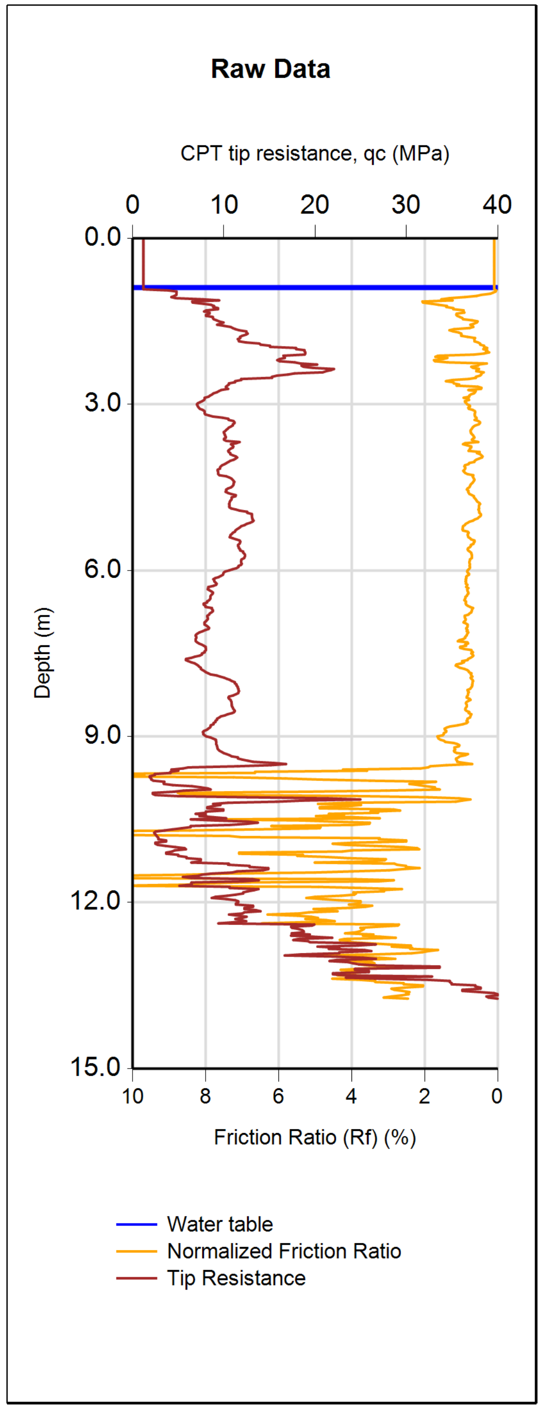

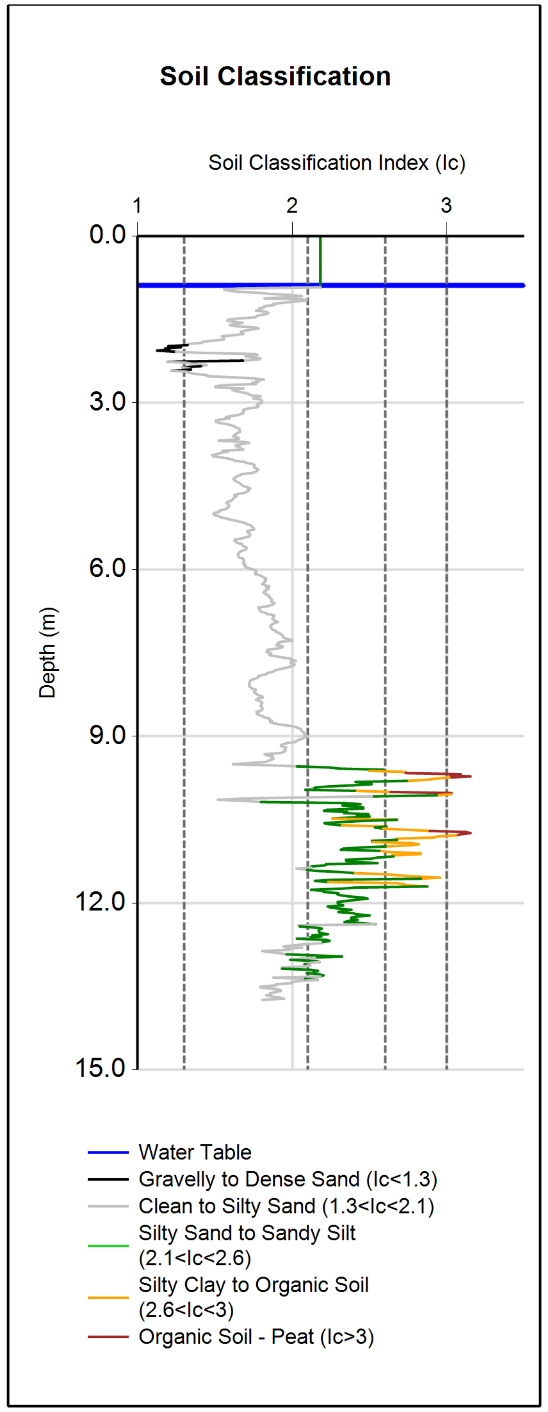

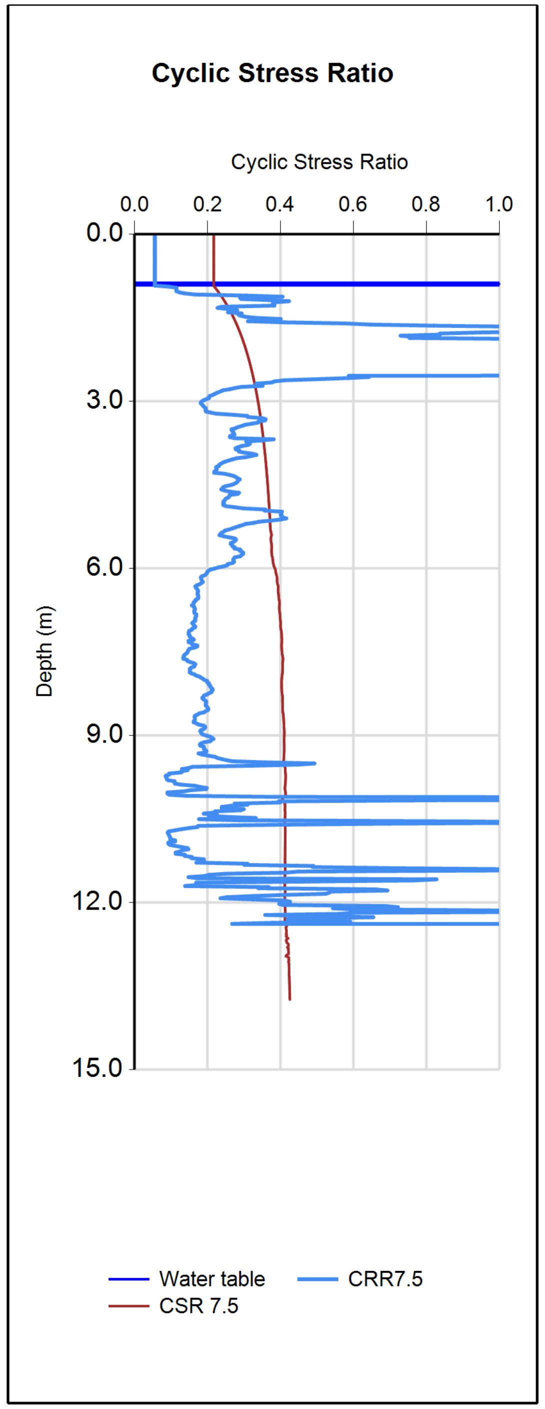

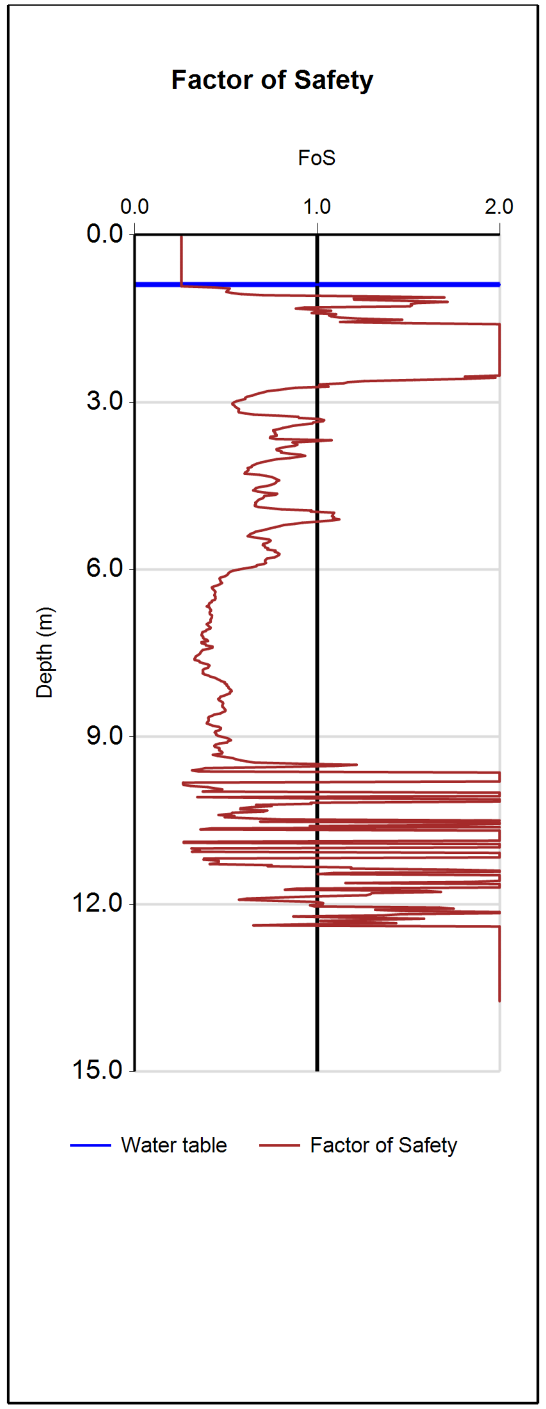

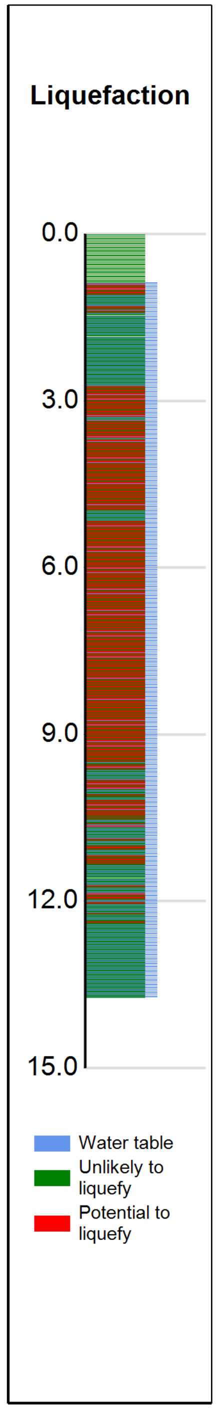

	CPT Name	Database ID	Investigation Date	Event and PGA	Magnitude	PGA (g)	GWD (m)	Trigger Method	Settlement Method	Pre-drill Depth (m)	(Assumed predrill values) Qc (MPa)	Fs (MPa)	γ (kN/m³)
INPUT:	CPT12	20877	4/05/2013	User Specified	7.5	0.36	0.9	IB	ZRB	0.9	2	0.01	18

	S - Calculated deformation (mm)	CTL - Cumulative Thickness of Liquefaction (m)	LPI - Liquefaction Potential Index	LSN - Liquefaction Severity Number	CT - Crust Thickness (m)
OUTPUT:	144	8	22	29	1

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CLIENT, PROJECT

Nelson City Council

Tahunanui Liquefaction

TITLE

Liquefaction Assessment

LOCATION

Tahunanui, Nelson

JOB NUMBER

871023

DATE	5/07/2013
ANALYSED	mjl
CHECKED	
PAGE	1 of 3 pages

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Engineering Log Terminology

GENERAL

Soil and rock descriptions follow the "Guidelines for the field classification and description of soil and rock for engineering purposes" by the New Zealand Geotechnical Society (2005). Refer to this document for methods of field determination.

Water

Water level on date shown	▼
Water Inflow	▶
Water outflow	◀

Core recovery

Expressed as percentage of the length of the core run recovered.

Drilling method/casing

Shows drilling method and depth of casing.

Common types:

OB	Open barrel
W	Wash
HQ3	HQ triple tube
PQ3	PQ triple tube coring
HSA	Hollow Stem Auger
WS	Window Sampler

Graphic logs

The graphic log shows soil and rock types. The defect log indicates the location, orientation and abundance of defects of all types.

Typical material symbols:

ω ω	Organic material	V V	Igneous rock
≡	Clay	☰	Mudstone
x x	Silt	xxxx	Siltstone
▒	Sand	▦	Sandstone
⁂	Gravel or Conglomerate	≈	Metamorphic Rock

Tests

- N=22:SPT uncorrected blow count for 300 mm
- 75/12:Undrained shear strength (peak /residual as measured by field vane.

Laboratory test(s) carried out:

PMT	Pressuremeter test
LT	Lugeon test
LV	Laboratory vane
AL	Atterburg limits
UU	Undrained triaxial
PSD	Particle size
c' Ø'	Effective stress
CONS	Consolidation
DS	Direct shear
COMP	Compaction
UCS	Unconfined compression
IS	Point load

Installation type

◫	Standpipe	▥	Slotted standpipe
◆	VWP	■	Bentonite seal
▒	Filter pack		

Sample type

⊟	Spt	◩	Other
◧	Thin-wall tube	☒	Core or Sample loss
◫	Bulk sample		

SOIL DESCRIPTION

Moisture content

D	Dry, looks and feels dry
M	Moist, no free water on hand when remoulding
W	Wet, free water on hand when remoulding
S	Saturated, free water present on sample

Consistency/undrained shear strength

		S_u (kPa)
VS	Very soft	< 12
S	Soft	12 to 25
F	Firm	25 to 50
St	Stiff	50 to 100
VSt	Very stiff	100 to 200
H	Hard	> 200

Density index

		SPT(N) - uncorrected
VL	Very loose	0 to 4
L	Loose	4 to 10
MD	Medium dense	10 to 30
D	Dense	30 to 50
VD	Very dense	> 50

Proportional terms definition (Coarse soils)

Fraction	Term	% of soil mass	Example
Major	(UPPER CASE)	Major constituent	Gravel
Subordinate	(lower case)	> 20	Sandy
Minor	with some... with minor...	12 - 20 5 - 12	with some sand with minor sand
	with trace of... (or slightly)...	< 5	with trace of sand (slighly sandy)

Grain size criteria

Type	Coarse								Fine	
	Boulders	Cobbles	Gravel			Sand			Silt	Clay
			Coarse	Medium	Fine	Coarse	Medium	Fine		
Size range (mm)	200	60	20	6	2	0.6	0.2	0.06	0.002	

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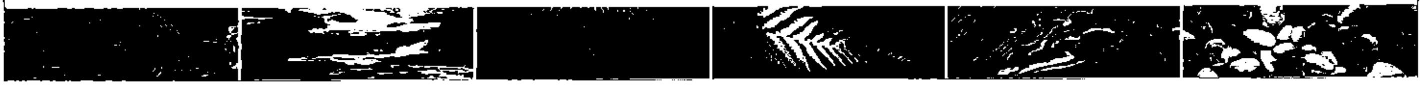
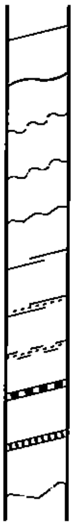


Engineering Log Terminology

ROCK DESCRIPTION

Signifcant defects

B	Bedding	
J	Joint	
Sc	Schistosity	
Cl	Cleavage	
Bz	Broken zone/crushed zone	
Ft	Fault	
Fg	Fault with gauge	
SZ	Shear zone	
Iz	Infilled seam	
XD	Extremely weathered seam	
DD	Drilling - induced defect	



Weathering

UW	Unweathered
SW	Slightly weathered
MW	Moderately weathered
HW	Highly weathered
CW	Completely weathered
RW	Residual soil

Defect shape

ST	Stepped
UN	Undulating
PL	Planar

Roughness of defect surface

R	Rough
SM	Smooth
SL	Slickensided

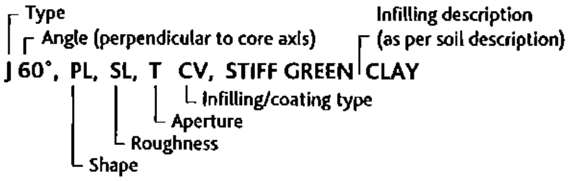
Field strength

		UCS (MPa)	$I_{s(50)}$ (MPa)
EW	Extremely weak	< 1	N/A
VW	Very weak	1 - 5	N/A
W	Weak	5 - 20	N/A
MS	Moderately strong	20 - 50	1 - 2
S	Strong	50 - 100	2 - 5
VS	Very strong	100 - 250	5 - 10
ES	Extremely strong	> 250	> 10

Aperture

		Apperture (mm)
T	Tight	nil
VN	Very narrow	0 - 2
N	Narrow	2 - 6
MN	Moderately narrow	6 - 20
MW	Moderately wide	20 - 60
W	Wide	60 - 200
VW	Very wide	> 200

Defect coding



J 60°, PL, SL, T CV, STIFF GREEN CLAY

Defect Orientation: for vertical unoriented boreholes defect orientation is measured normal to core axis e.g horizontal = 0°. For angled boreholes defect orientation is measured relative to core axis e.g parallel to core axis = 0°.

Infillings and coatings

CG	Clay gouge	Joints have openings between opposing faces of intact rock substance in excess of 1 mm filled with clay gouge. Clay is generally described in terms os soil properties.
CV	Clay veneers	Joints contain clay coating whose maximum thickness does not exceed 1 mm. Note: Describe clay in terms of soil properties.
PL	Penetrative limonite	Joint traces are marked in terms of well defined zones of slightly to moderately weathered ferruginised rock-substance within the adjacent rock.
FeSt	Limonite stained	Joint surfaces are stained or coated with limonite, although the rock substance immediately adjacent to the joints is fresh.
CT, SC	Coated	Joints exhibit coatings other than clay or limonite, e.g. Carbonate (CT) or Silica (SC).
CL, CS, CC	Cemented	Joints are cemented with limonite (CL), Silica (CS), or Carbonates (CC).
CN	Clean	Joint surface show no trace of clay, limonite, or other coatings.