APPENDIX TWELVE

PRELIMINARY GEOTECHNICAL ASSESSMENT

The Wellington Company
19-Jan-2016

Shelly Bay Development

Preliminary Geotechnical Assessment Report

Shelly Bay Development

Preliminary Geotechnical Assessment Report

Client: The Wellington Company

Co No.: 903151

Prepared by

AECOM New Zealand Limited
Level 3, 80 The Terrace, Wellington 6011, PO Box 27277, Wellington 6141, New Zealand
T +64 4 896 6000 F +64 4 896 6001 www.aecom.com

19-Jan-2016

Job No.: 60480847

Quality Information

Document	Shelly Bay Development
Ref	60480847
Date	19-Jan-2016
Prepared by	[redacted]
Reviewed by	[redacted]

Revision History

Revision	Revision Date	Details	Authorised	
			Name/Position	Signature
	19-Jan-2016	For Client Comment	[redacted] Geotechnical Manager - Ground Engineering & Tunnelling	[redacted]

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Executive Summary

AECOM New Zealand Ltd. (AECOM) have been contracted by The Wellington Company Ltd. (TWC) to provide multidisciplinary and design consultancy services, as part of the initial technical investigation and high level concept design validation, for a combined residential & commercial development at Shelly Bay & Mount Crawford, Wellington.

Residential properties, including houses, townhouses and apartment buildings up to 2, 3 and 7 storeys each, respectively, are proposed. The development will also include construction of a variety of commercial and retail facilities, including large office and retail developments up to 1,400m^2, as well as several hotels up to 6-7 storeys each. The existing offshore wharf and jetty structures are to be rejuvenated to create a ferry terminal and marina, and a cable car terminal and track is to be built upon the hillside to serve new properties upon Mount Crawford itself.

AECOM have scoped and supervised a preliminary phase of geotechnical investigation across the project site, including boreholes, inspection pits and cone penetration (CPT) testing. This report presents the findings and interpretation of the geotechnical investigations undertaken by AECOM at Shelly Bay, provides a geological model for the site, and preliminary engineering parameters for each stratum identified.

The site occupies two adjacent bays located in Wellington Harbour, each of which was progressively infilled during the Holocene Epoch with marginal marine sediments, most typically comprising fine sand. More recently, development of the area in the mid-19th to 20th century as a military installation has led to the placement of reclamation fill across much of the site area on top of these marine sediments. Completely weathered greywacke (colluvium) underlies the marine sediment and reclamation fill, in turn overlying more competent greywacke bedrock which also forms Mount Crawford, the steep hillsides of which border the site to the east.

A geohazard assessment has also been carried out to identify geotechnical and geological issues which may impact upon the development. This assessment has considered hazards such as tsunami inundation and ground fault rupture, as well as liquefaction, lateral spreading and rock slope instability. The marine sediments which underlie much of the site have been found to be susceptible to liquefaction, and vertical settlements of up to 250mm have been estimated in the southern bay where these deposits are encountered to their greatest extent. Elsewhere, such settlements are generally around 50 – 60mm in magnitude.

Recommendations for foundations for onshore structures, marine infrastructure (including seawalls, the marina, wharf and beach), requirements for slope stability measures and other site infrastructure (i.e. roads, paving and utilities) have been made upon the basis of the geohazard assessment. Foundations for onshore structures are likely to comprise a combination of shallow pad or strip footings where bedrock is encountered close to the surface; where liquefiable materials are present, piled foundations extending to bedrock are likely to be required, especially for heavier structures such as the multi-storey hotel. Ground improvement may also be required to mitigate against the risk posed by lateral spreading during a seismic event.

A structural assessment of the existing marina in 2010 suggests that the structure is in a state of disrepair, and is likely to require a major overhaul. Large numbers of the existing piles are likely to require replacement or retrofitting as a minimum. An alternative option may be to install steel sheet piles around the existing structure and backfill with further reclamation fill, largely demolishing the existing structure in the process.

Whilst some of the existing sea walls appear in good condition, others are not and some have even undergone partial collapse. In general, the seawalls are not considered to offer significant resilience to lateral spread, and may have been founded directly upon liquefiable sediment. These features may require retrofit or complete replacement.

There are a number of rock slopes around the site. A detailed discontinuity survey of unfavourable discontinuities of each, and subsequent analysis, has confirmed the potential for continued failures from these outcrops. The most common failures are likely to be relatively small (up to 0.1m^3), but rarer, larger failures (up to 10m^3) are also possible under adverse conditions in a few areas. Netting and rock bolting is recommended to remove the hazard posed by such failures to end users of the development.

Additional geotechnical investigation will be required prior to detailed design, and recommendations have been made in this report on a structure and area specific basis across the site.

1.0 Introduction

1.1 General

AECOM New Zealand Ltd. (AECOM) have been contracted by The Wellington Company Ltd. (TWC) to provide multidisciplinary and design consultancy services, as part of the initial technical investigation and high level concept design validation, for a combined residential & commercial development at Shelly Bay & Mount Crawford, Wellington (hereafter ‘the site’).

1.2 Proposed Development

The development proposed by TWC is outlined in detail in the Shelly Bay & Mount Crawford Masterplan (Ref. 1). An extract of the development proposal showing prominent details across the site is included in Appendix A.

The majority of existing structures at the site are likely to be demolished as part of the development, with only a few elements retained for refurbishment. Residential properties, including houses, townhouses and apartment buildings up to 2, 3 and 7 storeys each, respectively, are proposed. The development will also include construction of a variety of commercial and retail facilities, including large office and retail developments up to 1,400m^2, as well as several hotels up to 6-7 storeys each.

The development will also entail construction of a cable car terminal and track in the adjacent hillside to serve new residential properties upon Mount Crawford, as well as refurbishment of the existing offshore pier and wharf structures, in order to create a new ferry terminal. The existing beach to the south of the site area is also to be replenished with additional sand and extended.

1.3 Scope of Works

The geotechnical Scope of Works in support of the development is as follows;

- Carry out an initial desk based study of the site and surrounding area;
- Carry out a site walkover, including geological mapping and discontinuity survey(s) of prominent features, such as rock outcrops, across the site area;
- Plan, scope, supervise and interpret an initial phase of intrusive geotechnical site investigations across the site;
- Provide a geological ground model for the site;
- Provide geotechnical and seismic design parameters;
- Identify potential geohazards at the site, assess their likelihood of occurrence & severity, and the resulting qualitative risk to the development and end users;
- Provide preliminary recommendations for the following:
  - Foundations for onshore buildings throughout the development,
  - The need for and preliminary scoping of slope stabilisation works in the terrain surrounding the development;
  - Requirements for marine infrastructure, including the ferry wharf, marina, and land reclamation for the proposed beach;
  - Recommendations for other site infrastructure, such as roadways, paving, and utilities;
  - Recommendations for mitigation or remedial measures with respect to geohazards identified during the site investigations;
  - Requirements and preliminary scoping of additional geotechnical investigations for detailed design stages.
- Prepare and deliver a Preliminary Geotechnical Assessment Report (PGAR) summarising the findings and recommendations of the above investigations.

2.0 Site Description

2.1 Site Description

Shelly Bay is located 4km to east of Wellington City, and upon the western edge of the Miramar Peninsula. A general location plan of the site is shown in Appendix A.

The site comprises two adjacent infilled bays bordered to the east by the steep, densely vegetated slopes of Mount Crawford, and to the west by Wellington Harbour. Mount Crawford rises steeply at a slope of between 30 up to 70 degrees, and to a maximum height of 163m above sea level.

The site is almost 5 hectares in plan area, and comprises mostly flat terrain across each bay. A satellite image of the site, dated 2013, is shown in Figure 1. There are approximately 43 buildings across the site, including several pier and wharf structures at the headland between the two bays. These structures are associated with historical usage of the site as a military installation in the late 19th century through to the mid-20th century; many remain in active use, though some structures, particularly the pier and wharf, are in various states of disrepair. The site is intersected by several roads, most notably Massey Road and Shelly Bay Road, as well as several car parks.

2.2 Geological Setting

2.2.1 Solid Geology

Figure 2 shows an extract from the geological survey map of the Miramar Peninsula (Ref. 2).

Shelly Bay & Mount Crawford are underlain by Rakaia Terranes; Triassic rock types which are part of the wider Torlesse Supergroup. The Rakaia Terrane is part of a group of greywacke rocks terranes, which characteristically comprises late Carboniferous to late Trassic, quartzfeldspathic, metamorphosed sandstone and mudstone sequences together with poorly bedded sandstone with minor coloured mudstone of marginal marine to submarine origin.

In the Wellington Area, greywacke rocks are known to comprise monotonous, complexly folded and steeply dipping sequences of uniformly low-grade metamorphosed tubidites consisting of cyclical sedimentary units of sand grading up to mud.

2.2.2 Quaternary Deposits

Above the greywacke basement rock, each of the bays at the site has been progressively infilled by colluvium (completely weathered greywacke) originating from the surrounding slopes, as well as natural marginal marine sediments of Holocene age. More recently, reclamation fill, associated with the development of the area as a naval station in the late 19th & early 20th century, has also been placed across much of the area to create an artificial shoreline, sitting above the layers of colluvium and marginal marine sediments.

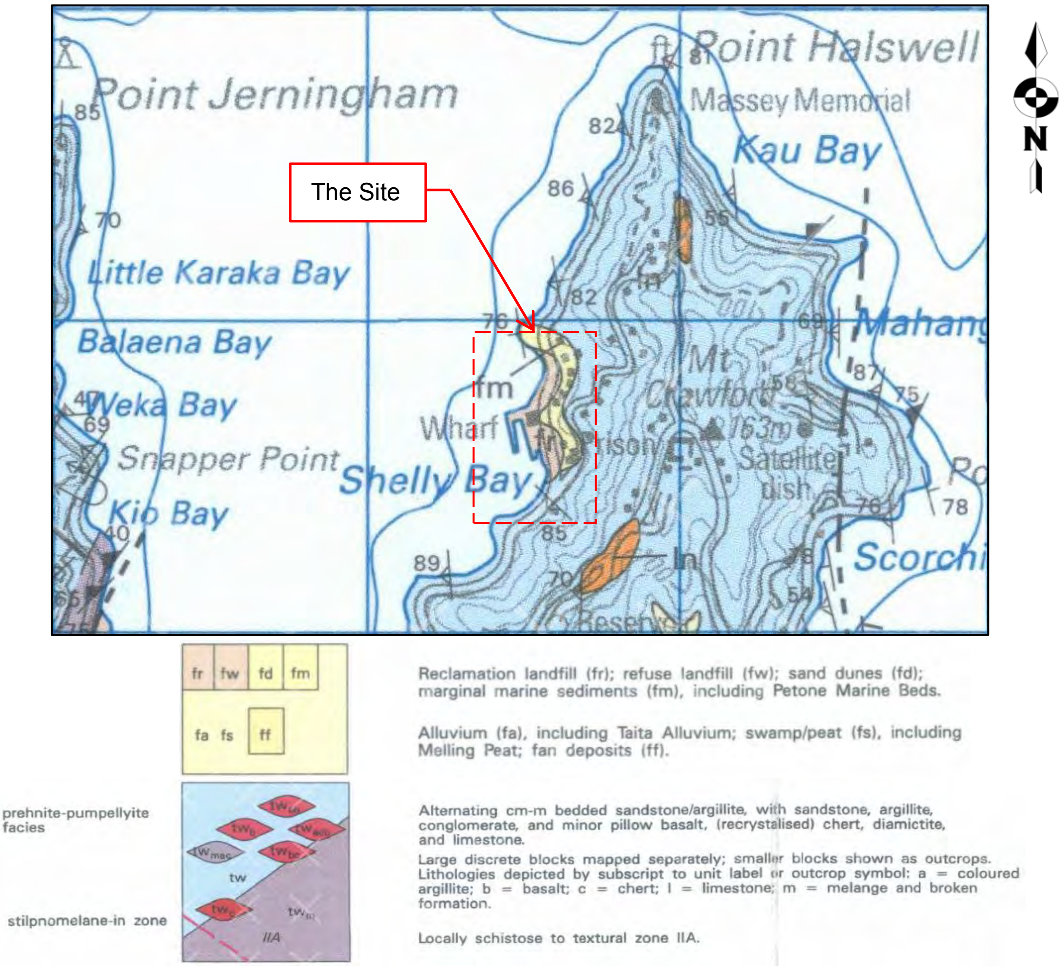
2.3 Seismicity

The site is located within 20km of 2 major faults, as identified in NZS 1170.5 (Ref. 3).

The active Wellington Fault, which runs in a southwest to northeast orientation, lies within 5 km to the west of the site. The Wairarapa Fault is also located approximately 19km to the east of the site, and beyond the Rimutaka Range.

The geological map also indicates a number of faults within approximately 800m to 2km of the site, such as the Seatoun and Evans Bay Faults, respectively. However, for the purposes of determining seismic spectra for design, these features are not considered to be major faults.

Figure 1 **Aerial Photograph, Shelly Bay, 2013 (Ref. 4)**



Reclamation landfill (fr); refuse landfill (fw); sand dunes (fd); marginal marine sediments (fm), including Petone Marine Beds.

Alluvium (fa), including Taita Alluvium; swamp/peat (fs), including Melling Peat; fan deposits (ff).

Alternating cm-m bedded sandstone/argillite, with sandstone, argillite, conglomerate, and minor pillow basalt, (recrystalised) chert, diamictite, and limestone.

Large discrete blocks mapped separately; smaller blocks shown as outcrops. Lithologies depicted by subscript to unit label or outcrop symbol: a = coloured argillite; b = basalt; c = chert; l = limestone; m = melange and broken formation.

Locally schistose to textural zone IIA.

Figure 2 **Geological Map of Shelly Bay, Mount Crawford & Surrounding Area (Ref. 2)**

3.0 Geotechnical Investigations

3.1 Desk Study

A desk study was conducted in tandem with the field works, and included appraisal of the following sources of information;

- A review of the geological maps and memoirs available for the Miramar Peninsula and greater Wellington region;
- A search for historical site investigation records within the public domain using the Greater Wellington Regional Council GIS viewer;
- Aerial photography and topographical data available online through Wellington City Council Webmaps;
- Review of historical design and construction drawings for the roadway, seawalls and buildings across the site, including the areas of reclamation, wharf and slipway structures, respectively;
- Retrieval and review of geotechnical investigation data for the Shed 8 area conducted in 2007 and 2015, respectively, and held by Tonkin & Taylor (T&T).

3.2 Site Walkover & Survey

An initial, general walkover was conducted at the site on the 9th December 2015. The primary objective of this walkover was to investigate prospective geotechnical investigation locations and potential access issues, prior to the intrusive geotechnical works being carried out.

A second walkover took place on 18th January 2016, and included more detailed inspection of the slopes around the site, included nine rock outcrops. Detailed mapping of rock discontinuities was also undertaken across three of these features for further analysis, and scoping of requirements for slope remediation.

3.3 Geotechnical Investigations

Intrusive geotechnical investigations were carried out across the site, as summarised below in Table 1.

Table 1 Summary of Geotechnical Investigations

Type	ID	Northing [mN]	Easting [mE]	Depth [mbgl]	Reason for Termination
Borehole	DH01	5426871	1752549	19.68	Rock head proven.
	DH02	5426889	1752628	4.6	Rock head proven.
	DH03	5427090	1752594	10.78	Rock head proven.
	DH04	5427135	1752586	16.63	Rock head proven.
Cone Penetration Test	CPT1	5426848	1752593	6.6	Refusal within colluvium.
Trial Pit	TP4	5427031	1752539	2.2	Rock head proven.
	TP5	5427077	1752605	2.4	Rock head proven.
	TP6	5427114	1752612	1.9	Rock head proven.

The site investigation coordinates are given in terms of the NZTM2000 datum, and have been approximated by taking measurements from landmarks in the vicinity of each investigative location (e.g. a kerb line, manhole cover or other distinctive feature easily distinguishable on the most recent aerial photographs of the site). Site investigation locations are shown upon the SI Location Plan & Geological Map in Appendix A.

Trial pits and cores recovered from the boreholes were logged by an AECOM geotechnical engineer in accordance with the procedures outlined in the NZ Geotechnical Society Guideline, ‘Field Description of Soil and

Rock'. The cores were also photographed and placed in core boxes for storage. All cores are stored at Griffiths Drilling NZ Limited's yard in Wellington.

The borehole logs and core photographs are presented in Appendix C. The trial pit logs are presented in Appendix D, and the CPT log in Appendix E.

3.3.1 Access Restrictions

Limited access to the areas surrounding Shed 8 during the site investigation works meant that a number of investigative locations could not be completed. As a consequence, several proposed borehole and trial pit locations, which would have otherwise been completed within this area, were relocated or cancelled over the course of the site works. In some instances, a borehole was carried out in an area where a CPT test had originally been proposed. The prevalence of shallow rock in some areas of the site (such as the northern bay) evidenced during the course of the trial pit excavations also meant that carrying out CPT testing in these areas would add relatively little value to the boreholes already completed by this stage in the investigation.

As a result, only one CPT test was completed, whilst two trial pits (TP1 & TP2) scheduled in the vicinity of Shed 8 were cancelled. A third trial pit (TP3) encountered a disused concrete culvert at around 300mm below ground level, and which had not been detected during the buried service location survey carried out prior to the geotechnical investigations. The ground above the culvert was reinstated and the trial pit subsequently cancelled.

4.0 Geological Model & Preliminary Design Parameters

4.1 Geological Model

A geological model of the site has been developed on the basis of the findings of the desk study, site visits and intrusive investigations outlined in Section 3.0.

In general, ground conditions consist of reclamation fill, often overlying marginal marine sediments on top of colluvial material (completely weathered greywacke rock) and highly to moderately weathered greywacke.

The depth to competent rock varies across each bay. As would be expected, however, the depth to rock head below ground level increases with proximity to the foreshore, and decreases towards the back of each bay and with decreasing proximity from the base of Mount Crawford, where the rock head 'daylights'.

A number of geological sections have been prepared to illustrate the geological model in each bay, and are presented in Appendix A. General ground conditions are summarise in Table 2 below.

Table 2 Geological Summary

Soil Unit & Typical Description		Depth to the Top of Layer [mbgl]	Layer Thickness [m]	SPT 'N' Value [Blows/300mm]		Cone Resistance, q_c [MPa]	
				Range	Average	Range	Average
1a	Silty GRAVEL, some cobbles and minor boulders, sometimes in a sandy or silty matrix. **[Reclamation Fill]**	0.0	1.7 – 3.0	5 - 15	11	2 - 20	8
1b	GRAVEL and COBBLES in a silty matrix. Some gravel and boulders of concrete. Wood fragments, iron pins, brick and ceramic fragments. **[Demolition Fill]**	0.0	0.3 – 1.5	10	10	N/A	
2a	Fine SAND with some shell fragments and minor silt. **[Marginal Marine Deposits]**	0.5 – 3.9	2.5 – 7.5	2 – 24	17	2 – 5	3
2b	With lenses of very soft, highly plastic SILT. **[Marginal Marine Deposits]**	4.7	1.3	< 2		Not encountered	

Soil Unit & Typical Description		Depth to the Top of Layer	Layer Thickness	SPT 'N' Value [Blows/300mm]		Cone Resistance, q_c [MPa]	
3a	Sandy SILT with some gravel **[Colluvium; completely weathered greywacke]**	11.4	5	8 - 14	10	20 - 35	25
3b	Highly weathered, very weak, silty fine SANDSTONE **[Greywacke]**	1.5 – 5.5	6	9 - 50	26	N/A	
3c	Moderately weathered, very weak, silty fine SANDSTONE and sandy SILTSTONE **[Greywacke]**	11.5 - 16.3	N/A	50 +		N/A	

4.2 Groundwater

Groundwater strikes were recorded in a number of trial pits, and groundwater measurements taken in several boreholes, as summarised below in Table 3.

Measurements in DH02 were taken at least 24 hours after drilling had finished, in order to allow the local groundwater table to restabilise following artificial introduction of water into the bore as part of the sonic drilling process.

Table 3 Groundwater Recordings

Location	Depth [mbgl]
TP5	1.8
TP6	1.9
DH02	0.7

Due to the coastal environment, it is anticipated that the groundwater level close to the foreshore will be related to the sea level and tidal variations. Tidal effects will decrease moving inland.

An estimation of the likely groundwater table across the site is included on the geological sections shown in Appendix A. Along the foreshore, a design static groundwater level of 1 - 2m depth may generally be assumed for the preliminary liquefaction assessment. However, it is anticipated that that there will be a general flow of groundwater from the hillside of Mount Crawford and towards the sea, and that this depth may reduce further inland. Groundwater level adopted for design purposes should therefore be selected on a location specific basis where this is relevant.

4.3 Geotechnical Parameters

Geotechnical parameters for the units identified in Table 2 are presented below in Table 4.

Table 4 Geotechnical Parameters, Soil

Soil Unit & Typical Description		Unit Weight [kN/m^3]	Undrained Shear Strength [kPa]	Effective (Drained) Parameters		Unconfined Compressive Strength, q_u [MPa]	Drained Young's Modulus, E' * [MPa]
				Friction Angle [Degrees]	Cohesion [kPa]		
1a	Silty GRAVEL, some cobbles and minor boulders, sometimes in a sandy or silty matrix. **[Reclamation Fill]**	19	-	35	-	-	40

Soil Unit & Typical Description		Unit Weight [kN/m^3]	Undrained Shear Strength [kPa]	Effective (Drained) Parameters		Unconfined Compressive Strength, q_u [MPa]	Drained Young's Modulus, E' * [MPa]
				Friction Angle [Degrees]	Cohesion [kPa]		
1b	GRAVEL and COBBLES in a silty matrix. Some gravel and boulders of concrete. Wood fragments, iron pins, brick and ceramic fragments. **[Demolition Fill]**	19	-	35	-	-	40
2a	Fine SAND with some shell fragments and minor silt. **[Marginal Marine Deposits]**	17	-	30	-	-	30 – 50
2b	With lenses of very soft, highly plastic SILT. **[Marginal Marine Deposits]**	16	10	-	-	-	2 – 12
3a	Sandy SILT with some gravel **[Colluvium; completely weathered greywacke]**	18	-	32	2	-	30 – 50
3b	Highly weathered, very weak, silty fine SANDSTONE **[Greywacke]**	19	-	35	20	-	150
3c	Moderately weathered, very weak, silty fine SANDSTONE and sandy SILTSTONE **[Greywacke]**	20	-	-	-	2	250 – 400

** Values of Young's Modulus provided are appropriate for 0.1% axial strain*

4.4 Site Classification & Seismic Hazard Spectra

The site is divisible into two subsoil classes, owing to the varying depth to greywacke bedrock across the site.

Close to the shorefront, Subsoil Class C (Shallow Soil) is judged as being appropriate, whilst towards the rear of each bay, and as the depth of competent rock reduces to less than around 2 to 3 metres, Class B (Rock) is suitable. An indicative boundary line separating these two zones is shown in Appendix A, and is based upon the boreholes undertaken by AECOM in December 2015, by T&T in 2007 & 2015, and historical data showing the extent of reclamation fill and rock outcropping in the vicinity of Shed 8. This line is indicative only.

Parameters for the calculation of Peak Ground Acceleration (PGA) for horizontal loading are given in Table 5 below. PGA is then calculated from the following;

$$C(T) = C_h(T)ZRN(T,D) \quad (1)$$

On the basis of the Shelly Bay & Mount Crawford Masterplan (Ref. 1), the site has been classed as Importance Level 2. This is considered appropriate for the majority of structures throughout the site, however where larger structures (such as the 6 storey hotel) are proposed, then an Importance Level of 3 may be warranted and should be adopted if, for example, the cumulative plan area of the structure exceeds 10,000m^2, or if any of the other criteria warranting an Importance Level of 3 as outlined in Ref. 15 are met. The Importance Level for each structure should be re-evaluated as the masterplan evolves, and prior to detailed design once final building forms are known.

Table 5 Seismic Parameters, Horizontal Loading Spectrum, Subsoil Class B & C

Common Parameters	**Symbol**	**SLS**	**ULS**
Annual Probability of Exceedance		1/25	1/500
Return Period Factor	R_s or R_u	0.25	1.00
Structural Importance Level		2	
Design Working Life		50 years	
Hazard Factor	Z	0.40	
Near Fault Factor	N(T,D)	1.00	
Subsoil Class B	**Symbol**	**SLS**	**ULS**
Spectral Shape Factor	$C_h(T)$	1.00	
Peak Ground Acceleration, Horizontal Loading	PGA	0.10g	0.40g
Subsoil Class C	**Symbol**	**SLS**	**ULS**
Spectral Shape Factor	$C_h(T)$	1.33	
Peak Ground Acceleration, Horizontal Loading	PGA	0.13g	0.53g

5.0 Geohazard Assessment

5.1 Overview

The following section discusses and quantifies (where appropriate) geohazards identified across the site area during the desk study and field works, respectively.

A geohazard is best defined as a geological state with the potential to cause damage or harm to human life, property and both the natural and built environment.

The following geohazards are anticipated to have some level of impact upon the design of the proposed development at the site, and are discussed in the following subsections;

- Earthquake induced hazards, including:
    - fault rupture,
    - ground shaking amplification,
    - soil liquefaction and lateral spread;
- Tsunami inundation;
- Rock falls.

5.2 Surface Fault Rupture

In sufficiently large or shallow earthquakes, the fault rupture may propagate up to the ground surface. In addition to being strongly shaken, any buildings situated on or near the fault rupture have the potential to suffer substantially more damage or collapse – particularly if the foundations are offset and the building straddles the fault trace. An example of Surface Fault Rupture observed after the 2010 Canterbury Earthquake is shown below in Figure 3.

The Ministry for the Environment (Ref. 5) recommend a minimum avoidance zone of 20 metres either side around surface traces of mapped faults or the likely fault rupture zone, though this should be increased depending upon the complexity of the fault system, or uncertainty regarding the location or extent of the fault trace at the ground surface.

The closest mapped fault is the Seatoun Fault, some 800m to the east of the site. It should also be noted that there is some evidence of relative movement in several of the rock outcrops surveyed around the site (Section 5.7). The potential for a splay or 'offshoot' fault to rupture across the site cannot therefore be ruled out; however, the same could be said for the majority of the Wellington CBD.

Figure 3 Surface Fault Rupture following 2010 Canterbury Earthquake (Ref. 6)

5.3 Ground Shaking Amplification

There are two mechanisms by which the intensity of ground shaking may be amplified, resulting in larger peak accelerations at the ground surface, and larger seismic demands upon buildings in the vicinity.

The first mechanism is amplification of the seismic waves generated by the fault rupture as a consequence of soft and loose soils overlying bedrock. The geotechnical investigations conducted at the site have highlighted the potential for sporadic layers of very soft material; in DH03, for example, a layer of very soft, highly plastic silt (Unit 2b) was encountered. However, this was the only such occurrence of such soft material in any of the boreholes, and the thickness of this unit was relatively thin; only 1.3 metres in total. It is therefore considered unlikely that there will be any substantial amplification of ground shaking as a result of soft deposits across the site.

Topographical features may also act to amplify the intensity of ground shaking. For slope angles of less than about 15 degrees, such effects are minimal; however, where slopes are significantly steeper, peak ground accelerations may be increased by as much as 20 – 40%. This amplification is typically concentrated in the immediate vicinity of the slope crest, and diminishes with increasing distance from it (Ref. 7). Rather than being considered a specific hazard to the development, this is better classed as a design consideration and should be considered during detailed design.

5.4 Tsunami Inundation

A number of the faults in the Greater Wellington region include an offshore component. Should rupturing of the fault take place offshore or within Wellington Harbour, then the location of the site on the coast places the development at risk of inundation from the resulting earthquake-triggered tsunami. Submarine landslides in the Cook Strait may also potentially generate a tsunami.

The most significant fault rupture in the Wellington area in recent history took place in 1855 on the Wairarapa Fault, some 19km to the west of the site. This rupture generated a tsunami with a maximum run-up of 5m in several locations in Wellington City. In Lambton Quay, the tsunami was also up to 2.5m in height, whilst waves continued to sweep around Wellington Harbour and Cook Strait for more than 12 hours following the event (Ref. 8).

GNS have developed tsunami hazard curves for several major cities in New Zealand, including Wellington. For a return period of 500 years (corresponding to that of the design ULS seismic event), the maximum amplitude of the tsunami wave may be between 5 – 7 metres, though it should be noted that this modelling is highly probabilistic and intended to give a general indication as to the severity of such an event.

Nevertheless, in the event of a future fault rupture offshore, and with sufficient energy to generate a tsunami, it is considered highly likely that the resulting wave will completely inundate both of the bays at the site. This is reflected in the evacuation planning and zonation of the area (Ref. 12).

5.5 Seismic Liquefaction

5.5.1 General

Liquefaction occurs when cyclic deformations generated by an earthquake cause an increase in pore water pressure in lower density sands and silts. When the pore water pressure equals in-situ applied pressure, loss in strength occurs (liquefaction) leading to ground deformation and, potentially, loss of bearing capacity. The presence of significant pore water pressure within the soil is essential for liquefaction and generally material above the water table is not susceptible to liquefaction. The susceptibility of a soil is a function of particle size distribution, groundwater level, soil density and loading. Liquefaction is a transient effect and strength is regained to some degree following the event as pore water pressures dissipate.

During earthquake shaking, soil particles may dislodge and reorganise into a denser state, whether above or below the groundwater table, though typically effects are more pronounced below the groundwater table. Densification of discrete layers accumulated over the full depth soil profile, as well as ejection of material, can also result in significant ground surface settlement.

5.5.2 Evaluation

A liquefaction analysis has been carried out using the results from the in-situ geotechnical testing, and the CLiq (Version 1.7.6.34 by Geologismiki, 2006) and LiquefyPro software programs, respectively. To this end, only those investigative locations where potentially liquefiable soils were observed during the fieldworks were considered in the analysis, including DH01, 03 & 04, and CPT1.

Groundwater level was taken at between 0.5m to 2mbgl, depending upon investigative location considered. Peak Ground Acceleration is taken as calculated in Table 5 and for Class C – Shallow Soil.

The following assumptions and options were also selected in conducting the liquefaction assessment based upon the CPT test (and using CLiq);

- Liquefaction Criteria is after the Idriss & Boulanger (I&B 2014) method;
- Settlements are calculated after Zhang et al. (2002 & 2004)
- Fines correction after Robertson & Wride 1998 is adopted; and
- Clay-like material softening behaviour has been applied.

Where liquefaction susceptibility was based upon results of SPT testing (and LiquefyPro), the following assumptions and options were selected;

- Liquefaction settlements are calculated after Ishihara & Yoshimine,
- Fines correction after Idriss & Seed is adopted during liquefaction,

- A hammer energy ratio correction of 1.25 is applied to raw SPT blowcounts, as appropriate for an Automatic Trip Hammer,
- Additional corrections for borehole diameter and sampling method are set to unity.

5.5.3 Results

A summary of the magnitude of liquefaction-induced vertical ground settlement is given in Table 6.

Table 6 Magnitude of Liquefaction – Induced Vertical Settlements

Investigation ID	Design Groundwater Level [m]	Total Ground Settlement (mm)	
		1/25 Year Return Period (SLS)	1/500 Year Return Period (ULS)
CPT1	1	Negligible (< 10)	< 50
DH01	2		180 – 250
DH03	2		< 55
DH04	2		< 65

5.5.4 Discussion

It may be seen from the above results that soil liquefaction in an SLS event is likely to have minimal impact upon the development, with settlements of less than 10mm generally predicted across the site.

The magnitude of settlement predicted in the ULS event at each investigative location is somewhat larger, and generally correlates directly with the extent to which the Marginal Marine Sediments are encountered in each borehole – though the groundwater level in the vicinity also influences the extent of liquefiable materials. The analysis also indicates that, rather than liquefaction presenting as discrete intervals of liquefiable material in this unit, the entire strata has the potential to liquefy.

As a result, liquefaction induced settlements are seen to peak at DH01 and where Unit 2a was around 7 – 8m in thickness; conversely, at DH03 and DH04, where this unit was less than 2 metres in thickness, settlements are notably less.

5.6 Lateral Spread

5.6.1 General

Lateral spreading of ground can occur in liquefied soil where there is a slope or a 'free face' (e.g., shoreline) towards which the ground may displace. Lateral spread of the ground occurs under static loading condition (post-earthquake) when the gravitational driving force of the ground due to the slope or free face gradient exceeds the shearing resistance of the liquefied soil. Lateral displacements are greatest towards the free face and diminish with distance back from the free face. Lateral displacements can be highly destructive for infrastructure, with effects of lateral spread potentially extending hundreds of metres back from the free face.

Instability of a quayside wall bounding reclaimed land alongside Wellington Centerport was observed following the 21st July 2013, M6.5 Seddon Earthquake. The existing coastal protection, and part of the reclaimed area, was lost to sea, as shown in Figure 4. In this instance, effects of lateral spread were observed up to approximately 150 metres back from the face of the quayside wall (Ref. 9).

Figure 4 **Effects of Liquefaction and Lateral Spreading upon Quayside Wall, Wellington, 2013 (Ref. 9).**

Lateral spreading at the site has been assessed at the location of DH01 and CPT1 using empirical methods (including the CLiq software, and Ref. 13). The following inputs and assumptions have also been considered to give a preliminary assessment of lateral spreading risk at the site;

- A free face height of 2.5m. This has been assessed from topographical data of the area, as well as historical construction drawings of the seawalls and bathymetry data available in the vicinity;
- Distance from the free face varies from 5m (DH01) to 30m (CPT1);
- Distance to source earthquake of 4km, assuming that rupturing takes place upon the Wellington Fault.

5.6.2 Results

Results of the lateral spread analysis are shown below in Table 7.

Table 7 **Empirical Estimation of Lateral Spread**

Location	Distance from shoreline [m]				
	5m	10m	20m	30m	40m
	Estimated Lateral Spread [m]				
DH01	1.5	1.0	0.7	0.5	0.4
CPT1	-	-	0.9	0.7	0.5

The analysis indicates that ULS lateral spread may be in the region of 700mm to over 1.5 metres, depending upon proximity to the free face. This estimation is based upon empirical methods only, and should be taken as an indication that significant lateral spread is likely to occur, rather than a precise calculation of the exact magnitude.

More detailed geometric information, as well as offshore geotechnical investigation, is required to determine the bathymetry and gradient of the seabed, as well as the thickness and extent of the liquefiable material offshore. This should be acquired and this risk more thoroughly addressed and quantified during detailed design.

Owing to the generally negligible liquefaction settlements predicted during the SLS level event, negligible lateral spread is inferred during the SLS.

5.7 Slope Stability

5.7.1 Site Survey

A site walkover was conducted on 18th January 2016 to supplement geological and geotechnical data procured from the geotechnical investigations, as well as to investigate significant rock features and slopes in the area surrounding the site for potential signs of instability.

In total, 9 distinct slopes were inspected, as shown below in Figure 5; an interpretive geological map of the site is also included in Appendix A.

3 sites in total (Slopes 1, 5 & 7) were also subject to detailed discontinuity mapping, either as a result of visibly unfavourable discontinuities 'daylighting' across the outcrop, visual evidence of large or recent debris falls, and where access to the feature on foot was possible. A detailed site walkover and observations matrix has been compiled for each slope and is included in Appendix B. General observations from the inspection are discussed and analysed in the following sections.

Notes
Contours are at 5m intervals, and derived from data captured as part of the Wellington City Rural Aerial Photography Project 2004.

Figure 5 Location of Slope Inspections at Shelly Bay

5.7.2 Summary of Observations

5.7.2.1 Geology

The rock outcrops slopes surrounding the site area comprise interbedded sequences of greywacke rock, consisting of highly to moderately weathered fine sandstone and fine sandy siltstone. In many locations, the crest of the slope was also covered in a thin cover of topsoil and completely weathered greywacke (colluvium) material, and which was frequently covered by dense scrub/bush and pine trees with visibly extensive root systems.

5.7.2.2 Modes of Failure

In general, many of the rock slopes inspected displayed unfavourable discontinuities which are anticipated to result in the future development of wedge and planar type failures, with toppling type failures also possible, but less common. Such failures are likely to be triggered by normal weathering processes, and are also likely exacerbated in several areas by the presence of large root systems which penetrate into the more competent rock from the colluvium overburden, and dislodge intact blocks through 'root jacking'. The presence of such root systems will also create enhanced pathways for rainwater to penetrate into the slope during periods of prolonged or heavy rainfall. Seismic activity will also, of course, also increase the frequency with which such failures occur.

At the majority of slopes, debris volumes were substantially less than 0.5m^3, with only a few discrete blocks of very weak to moderately strong greywacke up to 400mm across present in the resulting slides, and only at some sites. However, at slope 5, a much larger, albeit older debris flow, potentially up to 10m^3 in volume was observed, with intact boulders of moderately strong to strong greywacke rock up to 900mm across present in the debris pile. This is shown below in Figure 6(a).

Limited shallow translational failures in the superficial cover of soil overlying the greywacke rock were observed during the walkover and survey. However, the dense cover of vegetation and generally difficult access to the higher areas of Mount Crawford means that the possibility of such slope failures elsewhere cannot be discounted. It is likely that the dense vegetation covering much of the hillside has acted in part to stabilise this shallow surface layer, however such failures are very common in slopes of similar geology and topography in the Greater Wellington region, and are often triggered by periods of intense rainfall or seismic activity. Consideration should be given to the potential for such failures during detailed design, if significant removal of vegetation from slopes is required. One such failure, at Slope 8, is illustrated below in Figure 6(b).

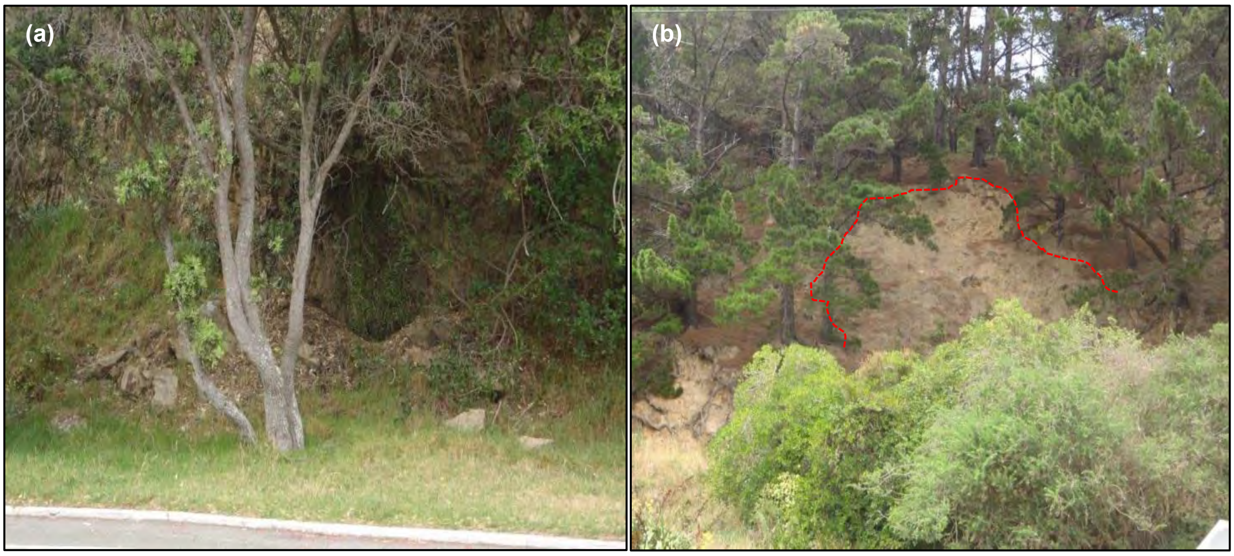


Figure 6 (a) Rock fall debris at toe of Slope 5; (b) Extent of shallow surface failure above greywacke outcrop at Slope 8

5.7.3 DIPS Analysis

The software DIPs was used to investigate which failure modes are kinematically admissible in each rock slope. DIPS graphically represents the surveyed rock discontinuities in a stereographic projection to allow identification of potential failure modes.

Typical DIPS analysis outputs are shown below to illustrate the failure mechanisms associated with each kinematic analysis. A DIPs analysis was carried out using rock discontinuity data taken from the 3 slopes surveyed during the site walkover, to investigate which failure modes within the rock mass are kinematically admissible, and confirm site observations.

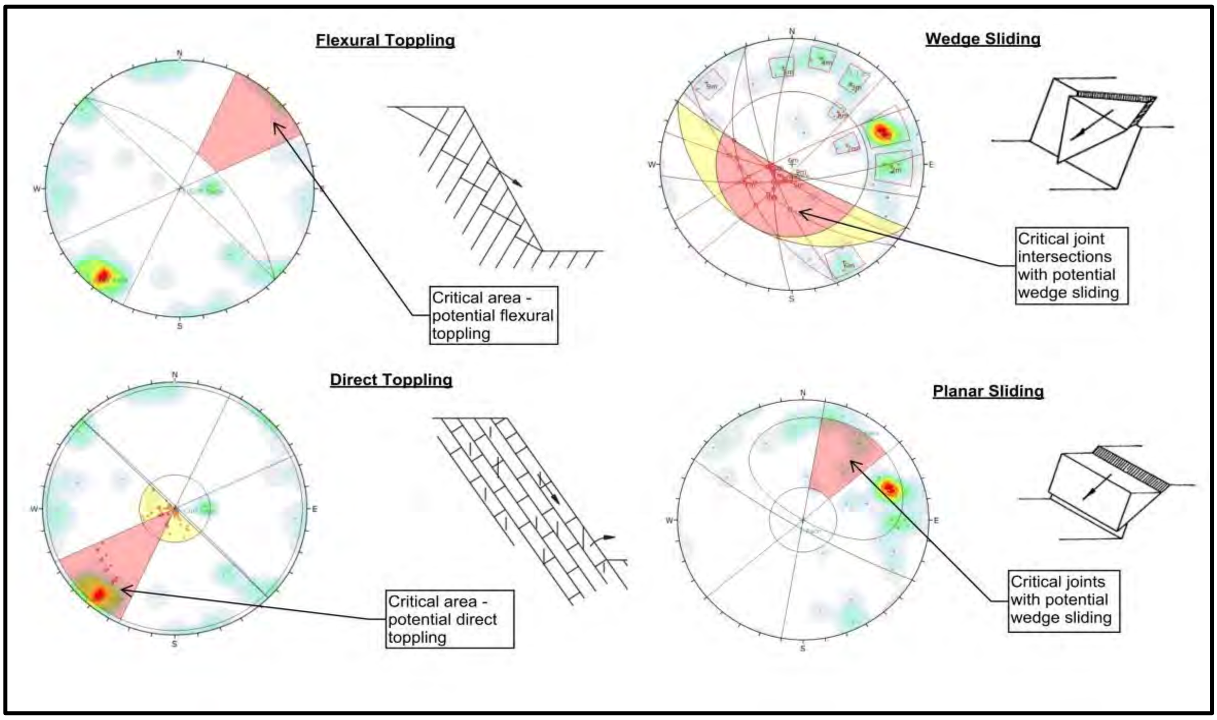


Figure 7 **Illustration of a DIPS Kinematic Analysis**

Toppling describes the possibility of individual rock blocks or slabs to topple over and in most cases result in rock falls or ravelling.

Planar Sliding and Wedge Sliding describe the possibility of rock blocks or slabs to slide along one or multiple (intersecting) planes. In order to evaluate the possibility of these failure modes friction components and geometric constraints are considered in the DIPS analysis.

While DIPS shows the kinematically possible failure mechanisms, it does not give an indication of the factor of safety against failure or the scale of failures.

Results from the DIPS analysis for the 3 slopes surveyed during the site walkover are shown in Table 8. Detailed output is included in Appendix F.

Table 8 **DIPS Analysis – Results: Slope 1, 5 & 7**

Kinematic Failure Mode	Percentage Critical Planes or Intersections (%)			
	Slope 1	Slope 5 – Face 1	Slope 5 – Face 2	Slope 7
Planar Sliding	24%	37%	24%	25%
Wedge Sliding	22%	59%	40%	36%
Flexural Toppling	0%	10%	5%	25%
Direct Toppling	24%	37%	29%	31%

5.7.4 Discussion

The result of the kinematic analyses is that unfavourable discontinuity orientations exist at all sites to varying degrees. It should be noted that critical intersections for toppling and wedge failure modes are based on intersections of all mapped discontinuities at the slope sections. The analyses assume indefinite persistence and therefore wedge sliding potential is likely to be overestimated.

With respect to the conditions observed on site, and in particular the frequency with which recent and older failures were observed, their relative sizes and total volumes of debris, this is likely indicative that small failures up to 0.125m^3 in volume will continue indefinitely as a consequence of the mechanisms described in Section 5.7.2.2; that is, weathering, root jacking, periods of prolonged rainfall and periodic seismic activity. Larger falls, possibly up to 3m^3 cannot be discounted, but are perhaps possible at only at a few slopes (such as Slope 5) and are generally considered to be rarer occurrences, more likely to be triggered by adverse conditions such as seismic activity.

Regrading of the slopes for construction purposes should carefully consider and design slopes accordingly so as not to create a face geometry which is more likely to result in more substantial rock falls from each face.

6.0 Geotechnical Risk Register & Development Hazard Map

A qualitative risk assessment has been carried out considering the results and interpretation of the geotechnical field works and analysis presented in Section 5.0. The likelihood of each geohazard and the potential impact upon the end users of the development have been considered in order to evaluate the risk associated with each.

Table 9 and Table 10 below show the matrix used to generally assess risk level, and the risk assessment outcomes respectively. The risk assessment methodology is included in Appendix G.

Table 9 Risk Level Matrix (Based upon Ref. 10)

		Impact					
		Catastrophic	Disastrous	Major	Medium	Low	Minor
Likelihood	Almost Certain	Very High	Very High	Very High	High	High	Moderate
	Very Likely	Very High	Very High	High	High	Moderate	Low
	Likely	Very High	High	High	Moderate	Low	Low
	Possible	Very High	High	Moderate	Low	Very Low-Low	Very Low
	Unlikely	High	Moderate	Low	Very Low	Very Low	Very Low
	Rare	Moderate	Low	Very Low	Very Low	Very Low	Very Low

Table 10 Risk Assessment

ID	Geohazard	Potential Effects	Likelihood	Severity	Risk
1	Surface Fault Rupture	– Large vertical and lateral displacements at ground surface – Substantial damage to foundations, buildings and infrastructure within immediate vicinity of surface fault trace	Rare	Catastrophic	Moderate
2	Tsunami Inundation	– Devastating inundation of low lying land – Flooding of basements, scouring and undermining of buildings, – Exposure and damage of underground services – Bodily movement of lighter structures and property (e.g. vehicles)	Rare	Catastrophic	Moderate
3	Liquefaction	– Differential settlement (sinking or tilting) of structures on liquefiable material, – Damage to underground services, – Deformation of surface infrastructure (i.e. roadways)	Possible	Major	Moderate
4	Lateral Spread	– Lateral movement of soil masses towards shoreline, – Differential settlement (sinking or tilting) of structures, – Spreading of foundations, – Substantial damage to and/or collapse of aging coastal infrastructure (e.g. seawalls)	Possible	Major	Moderate

ID	Geohazard	Potential Effects	Likelihood	Severity	Risk
5	Slope Instability	**Small Rock/Debris Falls** Up to 0.125m^3 Rocks piling up behind or entering property boundary Potential for minor damage or moderate injury to property and end users	Very Likely	Low	Moderate
		Large Rock Falls Up to 10m^3 More likely to result in significant damage and injury to property and end users	Possible	Major	Moderate

A Development Hazard & Recommendations Map overlay has been created using extracts of the Shelly Bay Masterplan Document (Ref. 1), which zones the above hazards, indicating which areas of the site are susceptible to each. This Map is included in Appendix A.

Overall, the risk level is considered normal for a large development site in Wellington.

Recommendations for design, and in order to address and mitigate the risk posed by each of the above hazards are indicated upon the Development Hazard Map, and discussed in greater detail in the following Section.

7.0 Design Recommendations

7.1 Onshore Building Foundations

For those areas marked in green on the Development Hazard Map in Appendix A, static settlements and liquefaction susceptibility are anticipated to be low, and competent greywacke bedrock is likely to be located at shallow depths (up to 2 – 3 metres) below existing ground level. Building foundations are therefore likely to consist of predominantly shallow pad and strip foundations; however, where larger building footprints are proposed, localised short piles may also be required to control differential settlement, owing to the nature of the rock head profile which tends to dip downwards across each bay from the base of Mount Crawford towards the shoreline.

Those areas marked in red are considered susceptible to seismic liquefaction and lateral spread; shallow pad and strip foundations are therefore unlikely to control or prevent damage, even for relatively light structures (i.e. timber framed buildings of 2 storeys or less), such as the 2 bedroom apartment buildings proposed along the shoreline in the northernmost bay. However, the relatively shallow depths to competent bedrock and non-liquefiable material in the northernmost bay (around 6 – 7 metres) are likely to mean that piles are again a viable option economically. However, additional piles or ground improvement will be required to resist the effects of lateral spread for structures placed close to the foreshore, and this is likely to add extra cost to the foundations of each building.

Competent bedrock was found to be deeper below ground level in the southernmost bay. Larger structures, such as the 6 storey hotel, should also be founded upon piles which penetrate to bedrock. Such piles are likely to be at least 10 – 12m long, or possibly longer, depending upon structural requirements and the exact depth to competent greywacke rock within the building footprint. Caution should be exercised for those structures which straddle the headland between the two bays and extend into the southern bay, as these buildings are likely to be founded partially upon shallow bedrock as well as liquefiable material. This is indicated by the yellow shaded area upon the Site Hazard Map.

7.2 Marine Infrastructure

7.2.1 Marina and Ferry Wharf

On the basis of the Masterplan (Ref. 1), it is proposed that the existing wharf in its entirety be redeveloped into a ferry wharf and small craft marina.

A (structural) engineering assessment was carried out upon the existing structure in November 2010 (Ref. 11). This included a visual inspection of the supporting piles from the surface to seabed by a team of divers, who rated each pile on a scale from 1 (good) to 5 (no integrity). The scale employed is as shown below in Table 11.

Table 11 Wharf Pile Grading System (Ref. 11)

Grade	Description	Piles per Grade
1	**Good** Pile capable of taking significant portion of design load, estimate 80 – 95% of design load	62
2	**Minimal necking** Pile capable of taking minor portion of design loads. Estimate 60 – 85% of design load.	
3	**Under half worn** Pile capable of taking minor portion of design loads. Estimate 40 – 60% of design load. Caution required.	132
4	**More than half worn** Pile must be treated with considerable caution and thoroughly inspected before loading.	63
5	**Broken/missing/no integrity** Pile is of no structural value.	41
	Total:	298

Out of a total 298 piles inspected, almost 80% were rated at grade 3 or below; this implies that some 45% of the piles are incapable of carrying 40-60% of their design load, with a further 35% of the total piles inspected are incapable of carrying less than 40% of their design load. In lieu of a further detailed assessment, consideration of actual design loadings upon the wharf and potential proof-load testing of several piles, it is unlikely that the wharf as-is is suitable for reuse, without some form of remedial works or intervention.

One solution for rejuvenation of the wharf may be to construct a reinforced concrete or steel sheet pile cofferdam around the perimeter of the existing structure, which is subsequently backfilled with reclamation fill. This may allow for only limited demolition/removal of the existing structure to be carried out, rather than complete removal, prior to construction of the new facility.

A second alternative would then be to partially or completely remove and replace the existing structure with a similar structure comprising reinforced concrete piles and deck, respectively. This may involve replacement of individual piles with new timber or concrete sections, or retrofitting of existing piles. Other structural elements, such as the deck, may also require replacement, though this will be the subject of a later report by the structural/civil discipline. A specialist wharf and marine structures designer is required and should be engaged for further assessment, and any design will need to be carried out in cooperation between the marine engineer, structural engineer and geotechnical engineer.

Due to the long wave run distance from the northwest of the site, the wave height is likely to exceed levels appropriate for small craft to moor. If a piled wharf structure similar to the current arrangement is preferred, then skirting is likely to be required as a minimum to reduce the wave heights within the marina. This will significantly increase the lateral load demand upon the structure, but can be accommodated during the detailed design. In this respect, a beneficial combination may be the construction of a cofferdam type structure towards the proposed ferry dock, which would double as protection for the marina behind. The Wharf alongside Shed 8 may also benefit from a change from piled pier to sheet pile seawall, including additional reclamation fill.

It is considered likely that redevelopment of the wharf structure will require additional geotechnical investigation, some of which may need to be carried out over water. Requirements for additional geotechnical investigation are discussed in Section 8.0.

7.2.2 Sea walls

There are several different configurations of seawall and coastal protection around the site. Whilst some of these appear to be in good condition, others are in various states of disrepair or have undergone collapse, as shown below in Figure 8. In general, many of the walls were judged as being at the end of their useful life, with 30% requiring repair or retrofit, and 20% requiring complete replacement. Several sections of sea wall, particularly around the Shed 8 area, could not be accessed or inspected visually.

Review of construction drawings of several seawalls in the southern bay show only thin concrete covers with a greywacke boulder facing; backfill to the wall is likely demolition or reclamation fill. Whilst some of these structures are founded directly onto bedrock, others appear to have been built directly onto the 'beach'. This implies that the sea walls are founded directly upon unit 2a, which was been identified as being susceptible to liquefaction in

Section 5.5. As a result, such structures will offer limited resilience to the effects of lateral spread and are likely to be severely damaged in a ULS level event.

It is uneconomical to design new or retrofit existing seawalls to resist lateral spread, as the extent of movement is too significant to be retained by such a relatively small structure. Instead, building foundation design should take into account the likely magnitude of lateral spread, and ground improvement around foundations of buildings at significant risk (i.e. those close to the shoreline) should be adopted or additional piles provided, as suggested in Section 7.1. This could be combined with the seawall retrofit or redesign for certain structures.

The seawall design should also consider sea level rise associated with climate change; based upon estimations by Tonkin & Taylor (Ref. 14), a 0.5m rise over the course of 50 years is suggested as a preliminary estimation. The seawalls should therefore be designed for overtopping as a result of sea level rise and the associated effects of climate change (e.g. increase in frequency of heavy swells); this may be acceptable in some areas of the site where structures are positioned some distance from the seawalls and unlikely to be influenced. In other areas, however, a staged or simply a higher seawall may be required to mitigate the risk.

Stone revetment and rock armour type designs are likely to be given priority for seawall design at the site as these are relatively economical designs, and match current seawall appearances around the bays. Seawall design will also vary depending upon the marina design, as the configuration of the seawalls may also influence wave heights in some areas of the site.

Figure 8 **General impression of existing seawalls around the site: (a) Location Plan; (b) Concrete/greywacke boulder facing founded directly onto bedrock; (c) damage to existing seawall in southern bay; (d) collapse of seawall in vicinity of Shed 8.**

7.2.3 Beach Expansion

The expansion of the existing beach to the south of the site should consider the potential for the material placed to be subsequently removed as a result of erosional processes in the adjacent bay. A specialist marine engineering assessment is likely to be required to design the beach expansion, and should include an assessment of the ocean currents and migration rates, options for migration mitigation, beach sand grading and consideration of the preferred beach layout.

Depending upon the mechanisms and rates of erosion, wooden groynes could be placed along the beach, or a breakwater or similar structure could be placed along the western flank of the bay, to improve retention of placed material.

7.3 Slope Stability

Based upon the detailed survey and rock discontinuity survey, it is considered advisable to carry out some form of remedial works across each of the prominent rock slopes surveyed and discussed in Section 5.7. The rough order extent of the remedial works has been estimated as 60% of the current rock slopes across the site area, and is shown indicatively on the Development Hazard Map in Appendix A

The precise extent of such works will require confirmation during detailed design, and should consider the requirements for removal of vegetation across each slope, as well as the geometry to which each slope requires to be regraded. Optimisation of the rock slope geometry using further DIPS analyses will minimise the amount of failures likely to originate from a given slope, if further cuts are required for structures around the site.

Where rock slope failures continue to be predicted with respect to the proposed geometry of each slope, the most economical form of remediation is likely to be high strength netting secured to the slope with a grid of rock bolts at approximately 2m centres; additional discrete bolts may also be deployed. Similar remedial works have been employed in the greywacke bedrock present across the greater Wellington region with apparent success; an image of a rock bolt netting on Birdwood Street, Karori, is shown below in Figure 9.

Figure 9 **Rock netting designed by AECOM and installed on Birdwood Street, Wellington, 2013.**

Where good separation is maintained between the rock slopes and structures, a rock ditch or catch fence could be provided as an alternative to netting to arrest and debris becoming dislodged from the slope face. Existing debris patterns, such as that shown at Slope 5 in Figure 6(a), could be used as a guide for sizing rock ditch width in this instance.

In either case, where substantial vegetation is required to be removed from the slopes as part of the development, scaling works should also be carried out to remove the remaining superficial layer of completely weathered greywacke and topsoil from the slope surface, as this material will be prone to shallow translational failures if it is allowed to become saturated during periods of prolonged rainfall, or as a result of seismic activity. The exposed greywacke surface may then require netting as shown in Figure 9. Localised shotcrete and concrete buttresses may also be required to maintain rock slope stability.

7.4 Site Infrastructure

7.4.1 Roads & Paving

The existing reclamation fill across the site is likely to provide a suitable subgrade for the construction or rerouting of roads and paving proposed. This is evidenced by the apparently good condition of the existing roads and car parks across the site, though traffic levels through the area are likely to increase with the commissioning of the development.

Consideration should be given to rerouting the stream, which currently drains from the gully in the southeast of the site (shown on the geological interpretive map in Appendix A), into a culvert below the existing road level. The existing drain beneath the structure in this location is in a state of considerable disrepair, and the constant flow of surface water across the road has caused substantial localised damage to the pavement, as per Figure 10 below.

Figure 10 Road damage due to surface water from gully runoff

7.4.2 Service Corridors

Connections of structures to external services (e.g. water, sewerage and power) should be made using flexible connections in order to avoid damage as a consequence of liquefaction induced differential settlement between the structures and surrounding ground, and to generally increase resilience of the development to a seismic event.

Service conduits should also not enter buildings via concrete slab foundations or pile cap, and the connection should instead be made through the external walls of each building. This will ensure that the service conduits are readily accessed and repairable, should they rupture as a result of a seismic event, or otherwise.

8.0 Additional Geotechnical Investigations

8.1 Investigation Requirements

It is considered advisable to carry out an additional phase of site investigation prior to detailed design, and once the layout of the development and nature of each structure has been finalised. Recommendations are summarised in Table 12 and discussed below.

Table 12 Recommendations for Additional Geotechnical Investigation

Development	Site Location	Hazard Map Zone	Recommended Investigations
3 bedroom townhouse	South Bay	Yellow	1 borehole, aligned with centre of gully feature
Retail, Café, Fish & Chips/Micro Brewery	South Bay	Red	Max. 2 CPTs within general footprint of building cluster
120 Bed Hotel – 6 Levels, Restaurant	South Bay/Headland	Yellow	1 borehole; 2 CPT tests around southern perimeter/footprint.
2 Bedroom apartments with 1 bed units underneath – 2 levels	North Bay	Red	2 CPTs either side of DH04 location.
Wharf, marina, (& potential breakwater site)	Headland, South Bay	N/A	2 – 3 boreholes and 4 CPT tests, concentrated around southern end of promenade and marina.

Where structures are proposed that may straddle two adjacent zones identified upon the Development Hazard and Recommendations Map, it would also be of considerable value to perform one borehole in the centre of the structure, and one or more CPTs around the perimeter of the building. This will allow determination of the likely dip of the rock head, as well as determination of the extent of any liquefiable material across the building footprint. This is of particular importance for the 6 storey hotel and restaurant, respectively, which are likely to straddle zones of shallow bedrock and liquefiable material. In this instance, the borehole is recommended so that targeted undisturbed samples of the bedrock can be retrieved for strength testing (e.g. UCS tests). Classification testing in the liquefiable material (e.g. particle size distribution tests) would also be of benefit.

The other structures proposed in the red and potentially liquefiable zones are generally likely to be only one or two storeys high. Targeted CPT testing around the building cluster is therefore likely to suffice for establishing depth to bedrock and extent of liquefiable material within the footprint of each structure.

For marine structures, a phase of offshore investigation should also be carried out. This should consist of predominantly CPT testing, as the potential for reclamation or demolition fill which might otherwise inhibit progression of the CPT below ground level is low, and liquefiable marine sediments are likely to be present directly at the seabed and overlying greywacke bedrock. These CPTs will also allow extent of liquefiable strata offshore to be more precisely determined for the purposes of lateral spread analyses in the northern and southern bays, respectively, and 2 – 3 boreholes would also be of benefit as part of this phase of investigation.

In performing CPT testing, it is recommended that equipment with a large self/dead-weight be adopted to perform the tests. The reclamation fill present across much of the site comprises coarse gravel and cobbles, which may inhibit penetration of the cone if pushed by a smaller machine relying upon screw augers to generate thrust/resistance to early cone refusal.

8.2 Post-Investigation Processes and Multi-Disciplinary Involvement

Following completion and interpretation of the additional geotechnical investigations, the following processes & disciplines will need to be engaged to advance the detailed design of the development;

- Geotechnical foundation design should be carried out in cooperation with a structural engineer responsible for the overall building design,
- A marine engineer should be engaged for the wharf and beach design, respectively, and detailed geotechnical design will also be required for the wharf piles and cofferdam elements,
- A detailed geotechnical assessment and design will be required for the existing seawalls and rock slopes,
- Infrastructure assessment and design, including construction and modernisation of new and existing gas, electricity, and communication networks will be required across the site,
- Building services assessment and design, including air conditioning, piping, etc. for each structure will be required,
- Civil engineering services will also be required for road and stream realignment design.

9.0 References

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10.0 Limitations

Recommendations and opinions contained in this report are based upon limited site investigations and observations. Inferences of ground conditions over the site are made on the basis of investigation results using geological principles and engineering judgement. However, it is possible that ground conditions over the site may vary and therefore it is not possible to guarantee the continuity of the ground conditions away from test locations.

Information in this report is not sufficient for detailed design. Further investigations, potentially including collection of bathymetry metocean data for offshore structural design are required. Where details of the proposed development change from that shown and assumed in this report, certain elements and recommendations may require reassessment.

This report has been prepared for the particular project described in the brief to us, and no responsibility accepted for the use of any part of this report in any other context or for any other purpose.

Appendix A

Site Location Plans, Maps & Drawings

1) Regional Site Location Plan

2) Extract from Shelly Bay & Mount Crawford Masterplan

3) SI Location Plan & Interpretive Geological Map

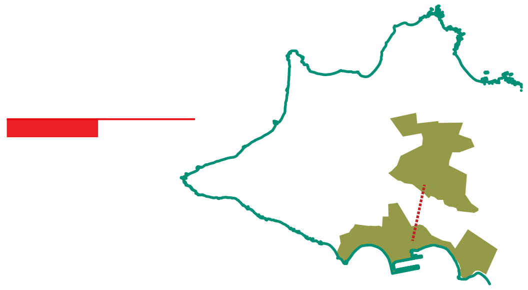
4) Geological Cross Sections

5) Development Hazard & Recommendations Map

Appendix A Site Location Plans & Drawings

1. Bedroom Townhouses with 1 bedroom units under – 3 Levels
2. 3 Bedroom Townhouses – 2 Levels
3. 2 Bedroom Apartments with 1 bed units under – 2 Levels
4. 2 and 3 Bedroom Apartment Building – 7 Levels
5. 2 and 3 Bedroom Apartment Building – 6 Levels
6. Courtyard / Plaza with Carparking below
7. Retail
8. Ferry Terminal
9. Ferry Wharf
10. Marina- 46 Berth
11. Hotel Conference Rooms / Back of House

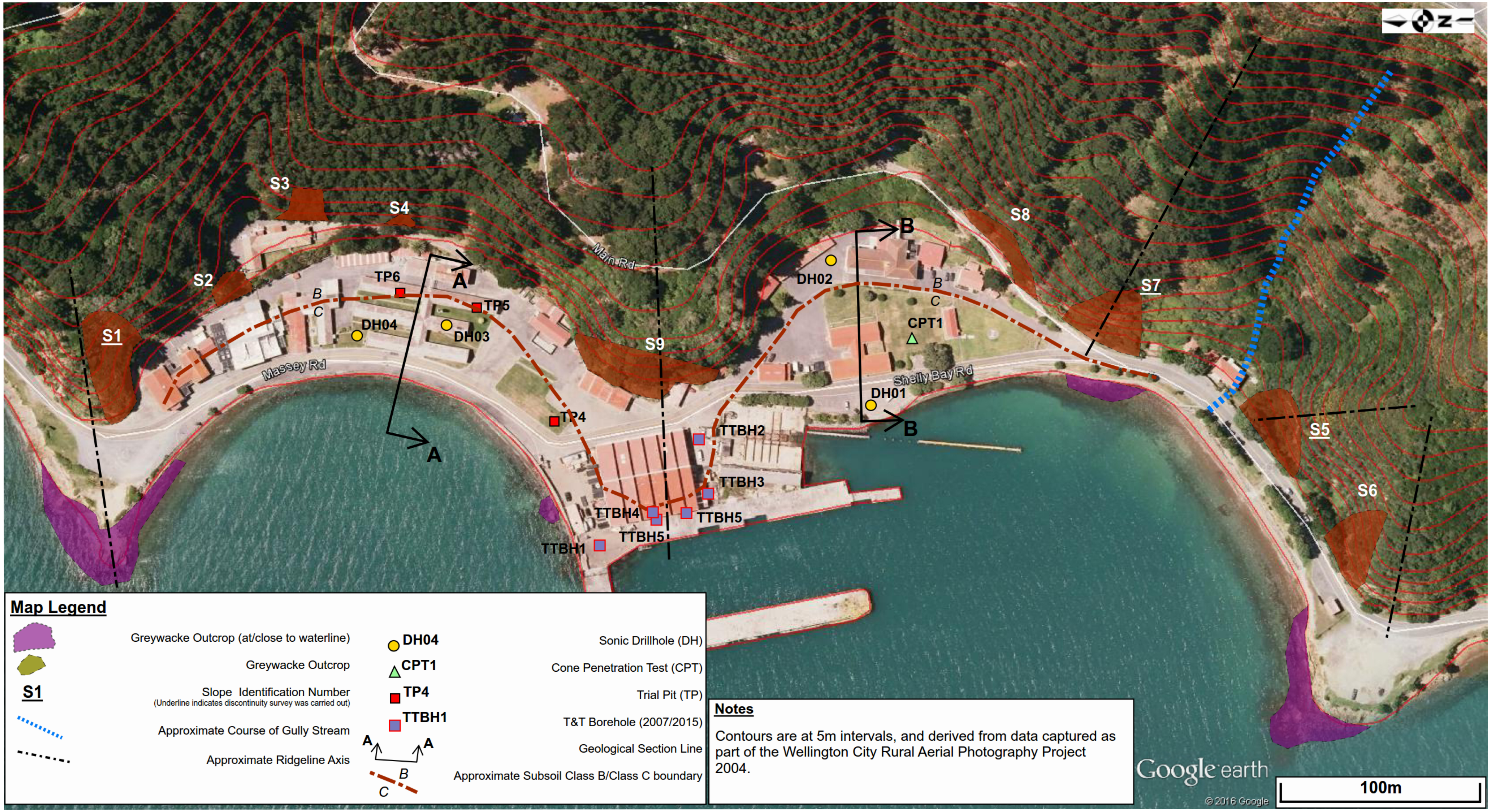
12. 120 Bed Hotel – 6 Levels
13. Restaurant
14. Cafe
15. Fish and Chips / Micro Brewery
16. Artists Quarter – Mixed Retail and Artists
17. Cable Car Terminal
18. Plaza with Retail under
19. Boutique Hotel
20. 3 Bedroom Townhouses- 3 Levels
21. 4 Bedroom Houses-3/4 Levels
22. 4 Bedroom Houses – 3 Levels
23. Gateway Pavilion



SHELLY BAY | MT. CRAWFORD MASTER PLAN

CONNECTED AND SHARED

[AUG 2015]



Map Legend

▨	Greywacke Outcrop (at/close to waterline)
▨	Greywacke Outcrop
S1	Slope Identification Number (Underline indicates discontinuity survey was carried out)
┈	Approximate Course of Gully Stream
─·─	Approximate Ridgeline Axis
● DH04	Sonic Drillhole (DH)
△ CPT1	Cone Penetration Test (CPT)
■ TP4	Trial Pit (TP)
■ TTBH1	T&T Borehole (2007/2015)
A A	Geological Section Line
B C	Approximate Subsoil Class B/Class C boundary

Notes

Contours are at 5m intervals, and derived from data captured as part of the Wellington City Rural Aerial Photography Project 2004.

FOR INFORMATION ONLY

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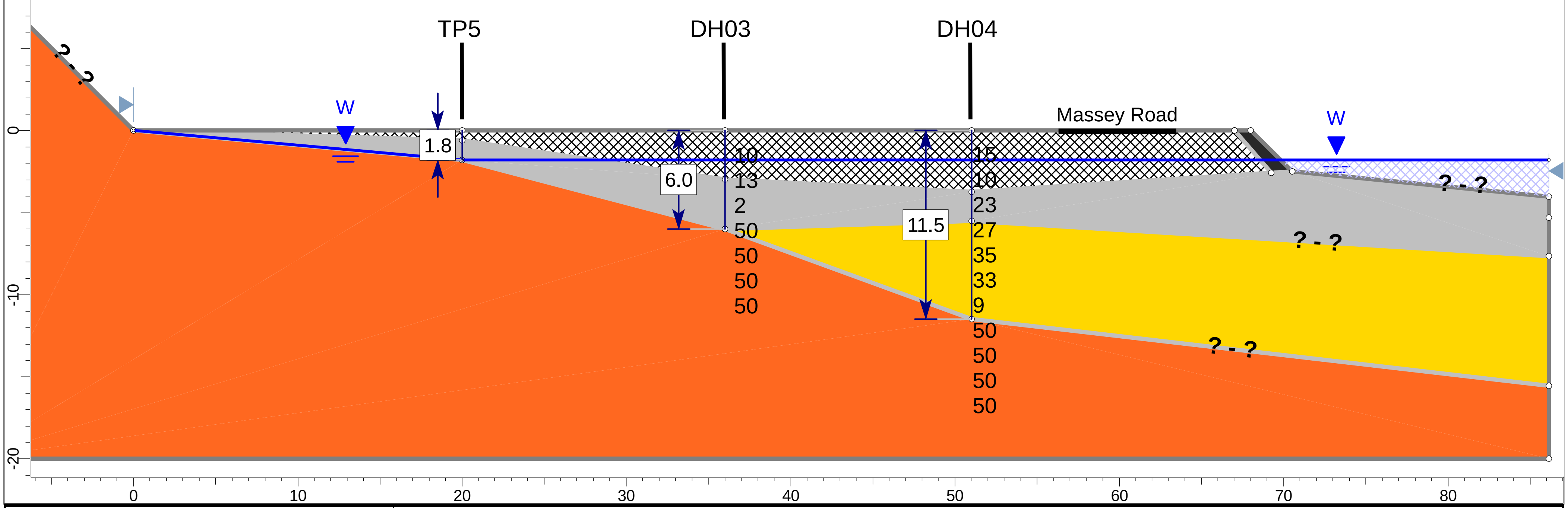
SHELLY BAY
SI Location Plan & Interpretive Geological Map
January 2016

Notes:

Cross section is indicative only, and not intended to give exact levels to rock head or any other stratum for design purposes.

Uncorrected SPT blowcounts shown next to each borehole.

Material Name	Color
Moderately Weathered GREYWACKE	🟧
Completely Weathered GREYWACKE [Colluvium]	🟨
Marginal Marine Sediments	⬜
Reclamation/Demolition Fill	▩
Sea Wall	⬛



Project	Shelly Bay
Analysis Description	Geological Cross Section A-A
Drawn By	TK
Company	AECOM
Date	25/01/2016, 4:51:38 p.m.
File Name	

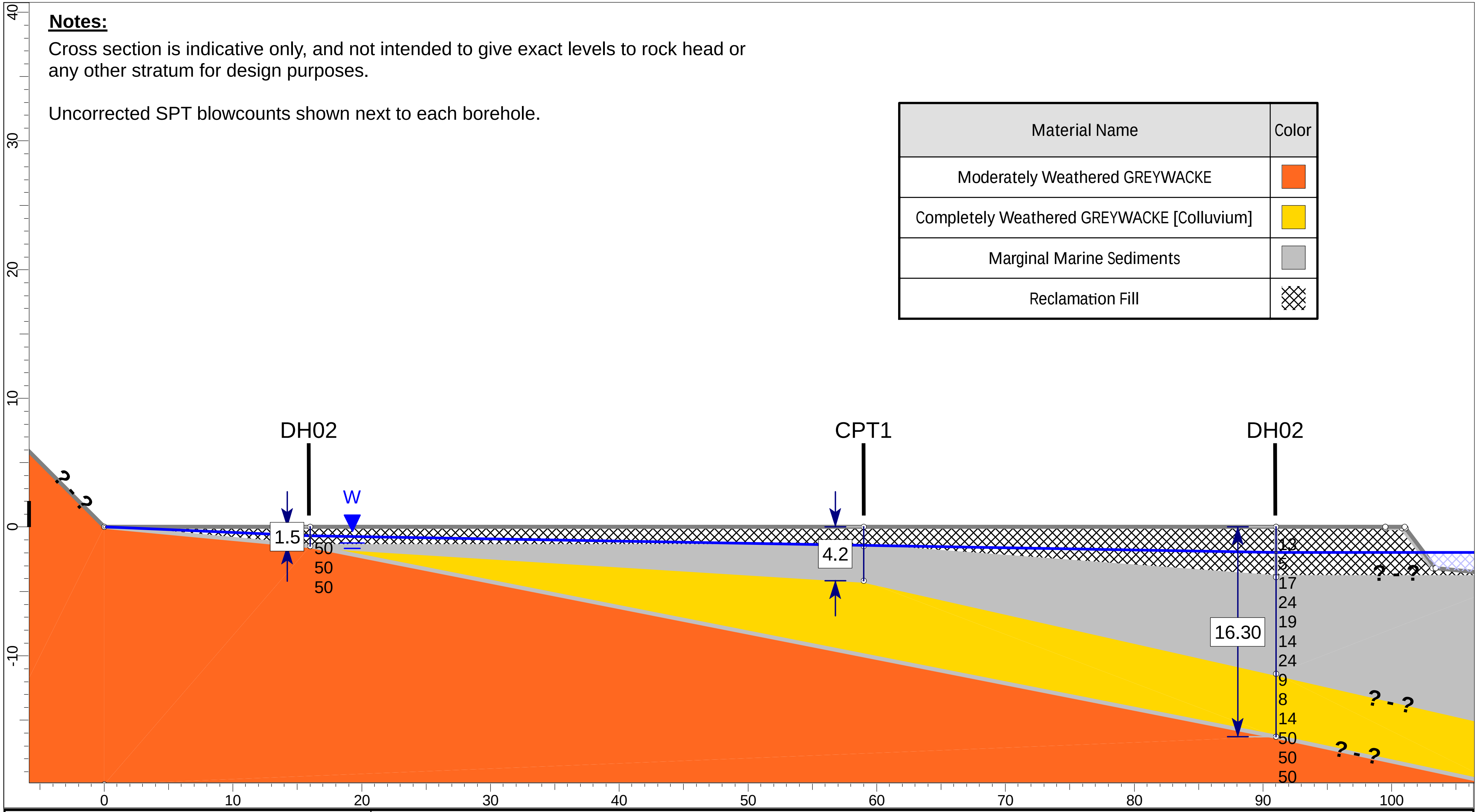
SLIDE 6.004

Notes:

Cross section is indicative only, and not intended to give exact levels to rock head or any other stratum for design purposes.

Uncorrected SPT blowcounts shown next to each borehole.

Material Name	Color
Moderately Weathered GREYWACKE	🟧
Completely Weathered GREYWACKE [Colluvium]	🟨
Marginal Marine Sediments	⬜
Reclamation Fill	▩



Project
Shelly Bay

Analysis Description
Geological Cross Section B-B

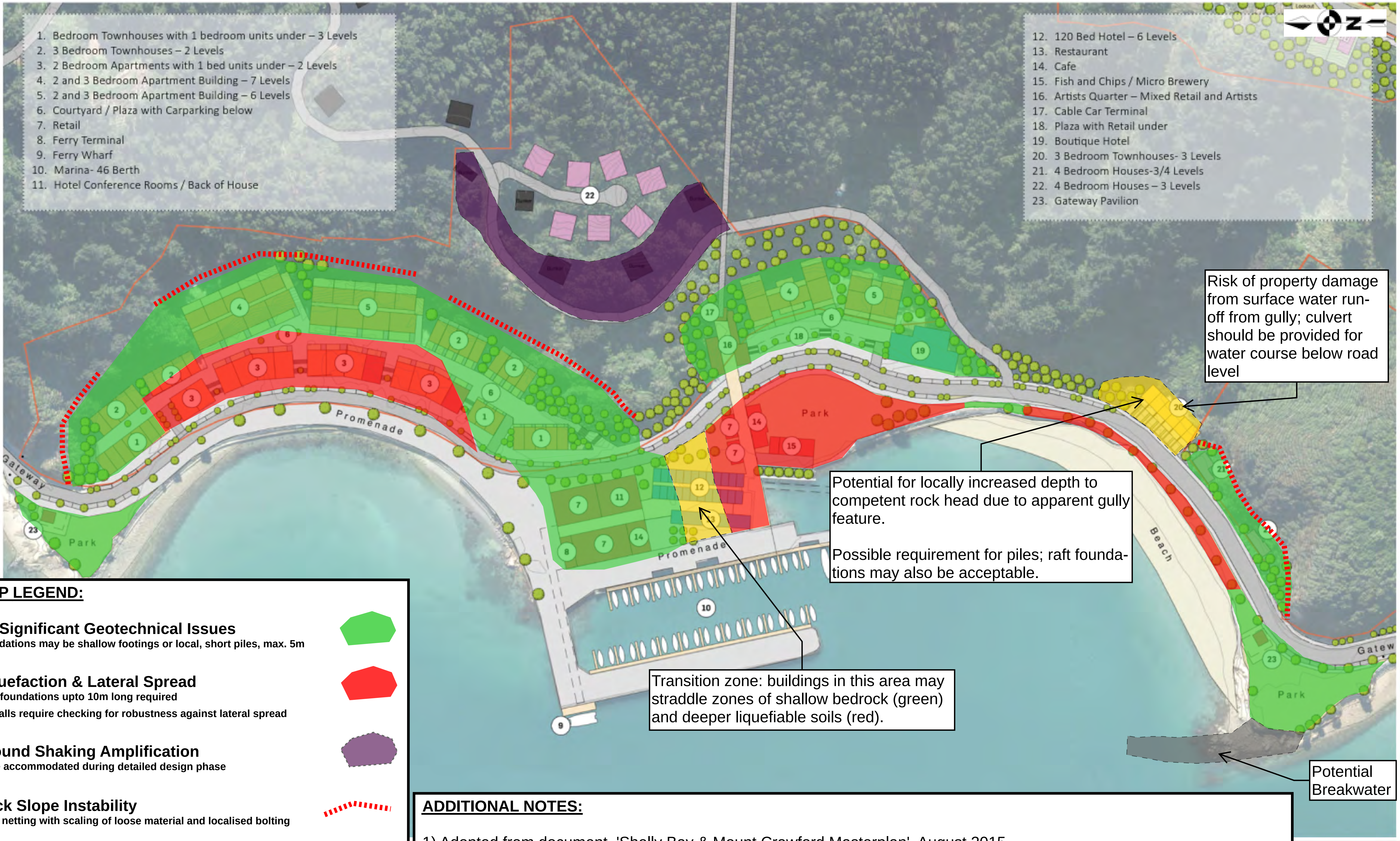
Drawn By
TK

Company
AECOM

Date
25/01/2016, 5:50:43 p.m.

File Name

SLIDE 6.004



1. Bedroom Townhouses with 1 bedroom units under – 3 Levels
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20. 3 Bedroom Townhouses- 3 Levels
21. 4 Bedroom Houses-3/4 Levels
22. 4 Bedroom Houses – 3 Levels
23. Gateway Pavilion

Risk of property damage from surface water run-off from gully; culvert should be provided for water course below road level

Potential for locally increased depth to competent rock head due to apparent gully feature.

Possible requirement for piles; raft foundations may also be acceptable.

Transition zone: buildings in this area may straddle zones of shallow bedrock (green) and deeper liquefiable soils (red).

Potential Breakwater

MAP LEGEND:

No Significant Geotechnical Issues 🟩
Foundations may be shallow footings or local, short piles, max. 5m

Liquefaction & Lateral Spread 🟥
Piled foundations upto 10m long required
Seawalls require checking for robustness against lateral spread

Ground Shaking Amplification 🟪
To be accommodated during detailed design phase

Rock Slope Instability ┅
Rock netting with scaling of loose material and localised bolting

Areas of Uncertainty 🟨
See specific call-out notes
Further geotechnical investigation required

ADDITIONAL NOTES:

1) Adopted from document, 'Shelly Bay & Mount Crawford Masterplan', August 2015.
2) Tsunami risk not shown; entire area should be considered at risk of inundation.
3) Ground rupture risk not shown, as this is considered to be a low risk item.
4) Extent of each colour coded zone is indicative only, and may vary slightly from that shown as geotechnical data is refined.

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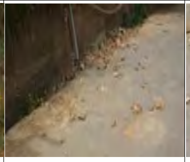
SHELLY BAY
Development Hazard & Recommendations Map
January 2016

Appendix B

Slope Survey Observations Matrix

			Shelly Bay Rock Slope Inspection Matrix (To be read in conjunction with AECOM Shelly Bay PGAR, January 2016)				Survey Photographs					
Slope ID	Height [m]	Inclination [Degrees]	Geological Description	Overburden	Vegetation	Discontinuity Survey Conducted?	Feature	General Form, Prominent Features & Details			Apparent Failures	
1	10 - 12	65	Moderately weathered greywacke. Outcrops of fine SANDSTONE often massive with no apparent discontinuities. Otherwise generally closely to very closely spaced, moderately narrow to very narrow discontinuities with undulating to planar surfaces	Cover of completely weathered greywacke, at slope toe and crest, respectively.	Dense bush coverage over slope crest. Some small vegetation across slope face, frequent root systems with evidence of root jacking.	Yes	 View of feature from south, looking north	 Plane of relative movement (possible faulting); evidence of crushed material close to feature.	 Live tree roots within slope face, root jacking mechanism likely/evident.	 Cave at base, likely requiring infill.	 Discrete blocks, upto 300mm, moderately strong	 Otherwise small, upto 100mm, very weak to weak debris.
2	12 - 15	60	Moderately to highly weathered greywacke. Closely spaced, moderately wide to narrow, discontinuities with undulating to planar surfaces.	Cover of completely weathered greywacke, at slope crest.	Yes, shallow vegetation and substantial root structures throughout (though many have been felled or appear to be dead)	No - difficult access	 View of feature, looking east	 Extensive (dead) root system and clear, loose blocks in-situ	 Side-on view of slope crest, looking north	 Slumping within colluvium/completely weathered greywacke cover.	 Failure onto roadway at base of feature. Small debris slides < 0.15m^3 volume, individual blocks are very weak to weak, moderately weathered greywacke, < 100mm maximum size.	
3	Section 1, 3m tall Section 2, above Section 1, 25m - 30m tall	Section 1, 65 degrees Section 2, 45 degrees	Moderately to highly weathered greywacke. Closely spaced to very closely spaced, moderately narrow to very narrow discontinuities with undulating to stepped surfaces.	Thin veneer of topsoil/completely weathered greywacke at top of Section 1, continuing behind slope and likely increasing in thickness.	Yes,dense cover of bush at crest of Section 1, with some root systems evident. Dense coverage of pine trees across Section 2	No	 View of Section 1 & 2, looking north/northeast along roadway. Section 2 contintutes to horizon.	 View of Section 1 only, looking south/southeast along roadway	 Live root system, potential for root jacking of blocks.		 Failure onto roadway at base of feature. Some discrete blocks, upto 400mm, moderately strong to strong greywacke. Small debris flows < 0.1m^3 volume, comprising very weak to weak greywacke.	
4	5	65	Moderately weathered greywacke. Closely spaced to very closely spaced, moderately narrow to very narrow discontinuities with planar and stepped surfaces.	Highly weathered layer at crest, with thin veneer of topsoil.	Dense, shallow bush (ferns, etc) at crest. Single mature tree at toe/road level. Dense coverage of pine trees across slope behind feature.	No	 View of feature, looking south/southeast along roadway, downhill	 Detail of discontinuities				
5	20	70	Moderately weathered greywacke. Slope has round holes with 'pitted' like quality high upon face.	Occasional, thin veneer of superficial soil across face. Also appears to be deposit of overburden, presumably colluvium, extending back from slope crest, as evidenced by presence of vegetation.	Frequent, shallow vegetation and grass across face, as well as numerous areas of mature vegetation growth (trees) across face. Slope crest features dense cover of bush.	Yes	 View of feature from adjacent beach, looking south	 Close - up of moderately to highly weathered material approaching crest.	 Outcrops surveyed at toe of slope; debris visible in foreground		 Large debris flows, <3m^3, vegetation growth across debris flow suggests these are not recent failures.	 Boulders, strong greywacke, upto 900mm across present in debris.

6	10	55	Moderately weathered greywacke. Closely spaced, moderately narrow to very narrow, stepped discontinuities.	Occasional, thin veneer of superficial soil across face. Also appears to be deposit of overburden, presumably colluvium, extending back from slope crest, as evidenced by presence of vegetation.	Frequent bush and mature vegetation, such as trees, present over upper portion of slope face.	No - difficult access, limited structures currently proposed in vicinity						
							View of feature from adjacent roadway, looking south		Development of wedge failures within rock mass		Debris flows 1up to m^3, Boulders upto 400mm	
7	20	75	Moderately weathered greywacke. Closely spaced, moderately narrow, undulating to planar discontinuities.	Occasional, thin veneer of superficial soil across face. Also appears to be deposit of overburden, presumably colluvium, extending back from slope crest, as evidenced by presence of vegetation.	Frequent, shallow vegetation and grass across face, as well as numerous areas of mature vegetation growth (trees) across face. Slope crest features dense cover of bush.	Yes						
							View of feature from adjacent roadway, looking southeast	Outcrop at slope toe				
8	10	75	Moderately weathered greywacke. Very closely to extremely closely spaced, moderately narrow to moderately wide, undulating discontinuities.	Thin veneer of topsoil/completely weathed greywacke and topsoil, continuing behind slope and likely increasing in thickness.	Frequent, shallow vegetation and grass across face, as well as numerous areas of mature vegetation growth (trees) across face. Slope crest features dense cover of bush.	No						
							View of feature from adjacent roadway, looking southwest	Plane of relative movement (dip/dip dir; 053/045), evidence of crushed material. Roots follow plane of weakness.	Visible bedding, moderately thick, very steeply inclined		Shallow slide in topsoil/completely weathered greywacke.	Debris < $0.5m^3$ topsoil/completely weathered greywacke, fragments of highly - moderately weathered, very weak greywacke
9	10	50	Moderately weathered greywacke. Fine sandstone often massive in nature, with discontinuties only appearing around slope toe.	Thin veneer of topsoil/completely weathed greywacke and topsoil, continuing behind slope and likely increasing in thickness.	Frequent, shallow vegetation and grass across face, as well as numerous areas of mature vegetation growth (trees) across face. Slope crest features dense cover of bush.	No - difficult access						
							View of feature from corner of old Transfield Depot, looking northeast	View of upper slope, over top of Transfield Depot			Superficial debris piled up behind pipework and building	

Appendix C

Borehole Logs

TERMINOLOGY AND SYMBOLS

Drilling / Investigation Methods

CFHSA	- Continuous Flight Hollow Stem Auger.
CFSSA	- Continuous Flight Solid Stem Auger.
DC	- Dynamic Coring (eg Terrier Rig).
DCP	- Dynamic Cone Penetrometer.
HA	- Hand Auger.
HQ3	- HQ Triple Tube.
HQWL	- HQ Wire Line.
HWOB	- Heavy Weight Open Barrel.
NQ3	- NQ Triple Tube.
NQWL	- NQ Wire Line.
OB	- 100mm diameter Open Barrel.
OB70	- 70mm diameter Open Barrel.
PERC	- Percussion.
PQ3	- PQ Triple Tube.
PQWL	- PQ Wire Line.
RC	- Reverse Circulation.
RCDHH	- Reverse Circulation Down Hole Hammer.
SPT	- Standard Penetration Test.
SPERC	- Sonic Percussion.
PT	- Push Tube Sample
VAC EX	- Vacuum Excavation.
WASH	- Wash Drilling.

Test Results

SPT "N" value; uncorrected blow count for 300 mm penetration
/# / # / # / # / # blows per 75 mm penetration

ss - Standard Penetration Test - split spoon
sc - Standard Penetrattion Test - solid cone
SUOW - Sunk Under Own Weight

Vane Shear Strength Tests

/ # Vane shear strenght test results given as peak / remoulded shear strengths (kPa). Test as per NZGS Guideline, 2001.

$^{\#}$ = Vane test performed on core recovered prior to extrusion from core barrel.
* = Vane test performed on excavated material of suitable size.

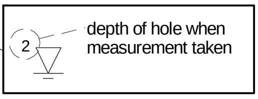
UTP - Unable to penetrate.

Piezometer Installation

Standpipe	▯	Grout	▦
Slotted Standpipe	▤	Cement	▨
Drill Cuttings	⬚	Gravel Pack Filer	●
Bentonite	■	Sand Pack Filter	⁙

Groundwater Records

Water Level (Static)	▼
Water Level (During Drilling)	▽
Water Inflow/Seep	▷
Water Outflow	◁
Complete Water Loss	◀
Regain Circulation	▶



Samples

PT	- Thin Wall Push Sample
U	- Undisturbed
D	- Disturbed (Core)
B	- Disturbed (Pit)

ROCK DESCRIPTIONS

Relative Strength

		USC (MPa)
ES	- Extremely strong	> 250
VS	- Very Strong	100 - 250
S	- Strong	50 - 100
MS	- Moderately Strong	20 - 50
W	- Weak	5 - 20
VW	- Very Weak	1 - 5
EW	- Extremely Weak	< 1

Weathering

UW	- Unweathered
SW	- Slightly Weathered
MW	- Moderately Weathered
HW	- Highly Weathered
CW	- Completely Weathered

SOIL DESCRIPTIONS

Consistency Cohesive Soils

	Su (kPa)
Very Soft	< 12
Soft	12 - 25
Firm	25 - 50
Stiff	50 - 100
Very Stiff	100 - 200
Hard	200 - 500

Relative Density Non-cohesive soils

	SPT "N" Value (uncorrected)
Very Loose	< 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

Rock Defect Abbreviations

Defect Type
J = Joint
Slk = Slickenside
BP = Bedding Plane Defect
SZ = Shear Zone
FZ = Fracture Zone
WZ = Weak Zone
F = Fracture
BkJ = Broken Joint
L = Lamination
HJ = Healed Joint
DB = Drilling Break

Defect Apperance
BkJ = Broken Joint
L = Lamination
HJ = Healed Joint
DB = Drilling Break
R = Rough
vR = Very Rough
Sm = Smooth
T = Tight
Pl = Planar
Cn = Clean
Bed = Bedding
\\ = Parallel
Ud = Undulating
St = Stepped
Op = Open
Pol = Polished
H = Healed

Infill Material
Mn = Manganese
Fe = Iron Oxide
Qtz = Quartz
S = Sand
Gr = Graphite
Ch = Chlorite
NF = No Infill
Co = Coalified
Py = Pyrite
Slt = Silt
CC = Calcite
Cb = Carbonaceous
Cl = Clay
V = Veneer
Calc = Calcareous

Graphic Log (typical symbols)

⬚	Organic Material	☰	Mudstone
☰	Clay	⊠	Siltstone
×	Silt	⁙	Sandstone
⁙	Sand	⌄	Volcanic Rock
○	Gravel / Cobbles	⊠	No recovery

Rock Classification Abbreviations

GSI = Geological Strength Index
RQD = Rock Quality Designation
Jn = Joint Set Number
Jr = Joint Roughness Number
Ja = Joint Alteration Number

Soil and rock descriptions generally as in "Guidelines for the Field Description of Soil and Rock for Engineering Purposes" by the NZ Geotechnical Society Inc, December 2005.

LOG OF DRILLHOLE

HOLE IDENTIFICATION **DH01**

Client The Wellington Company Ltd.
Project Shelly Bay Development
Project number 60480847

Co-ordinates 1752549mE 5426871mN
Orientation -90° Elevation (Approx)
Location Shelly Bay, Wellington
Feature Shoreline car park, adjacent to Officer's Mess Quarters (HQ).

GEOLOGICAL DESCRIPTION

Unit	Geological description
FILL	0m: Reclamation Fill
FILL	2.9m: Core Loss
FILL	3m: Reclamation Fill
Marginal Marine Deposits	3.9m: Marine Sediments comprising fine sand and silt with intact shells and shell fragments.
RAKAIA TERRANE	11.4m: Colluvium [Completely weathered greywacke].
RAKAIA TERRANE	16.3m: Moderately weathered, brown, silty fine SANDSTONE [Greywacke]. Very weak, very closely spaced joints.

Test Records

Shear Vane residual - peak 0 - 200 kPa; N Values 0 - 50; Core Loss/Lift 0 - 100%

Test Records	Drilling Method Casing remarks
	VAC EX
ss 3,2,3, 2,3,5 N=13	SPT
	Sonic
ss 1,1,1, 1,1,2 N=5	SPT
	Sonic
ss 3,4,4, 4,4,5 N=17	SPT
	Sonic
ss 4,5,6, 5,6,7 N=24	SPT
	Sonic
ss 5,4,5, 4,5,5 N=19	SPT
	Sonic
ss 3,4,3, 3,4,4 N=14	SPT
	Sonic
ss 2,3,5, 6,7,6 N=24	SPT
	Sonic
ss 3,2,1, 3,3,2 N=9	SPT
	Sonic
ss 1,3,2, 2,3,1 N=8	SPT
	Sonic
ss 2,3,3, 3,4,4 N=14	SPT
	Sonic
ss 4,3,9, 30,11 for 35mm N>50	SPT
	Sonic
ss 2,9,28, 22 for 75mm N>50	SPT
	Sonic
ss 5,18,40, 10 for 15mm N>50	SPT

Depth: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21

Graphic Log

Instrumentation

SOIL PROPERTIES

Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc

0m: Vacuum excavation, no recovery.

1.5m: Sandy GRAVEL with some silt; brown. Medium dense, moist. Sand is fine to coarse. Gravel is fine to coarse, angular to subrounded, moderately weathered, moderately strong, greywacke.

2.8 to 2.9m: Layer of cobbles; brown. Dry. Moderately weathered, moderately strong greywacke.

2.9m: Core Loss

3m: Sandy GRAVEL with some silt; brown. Loose, moist. Sand and gravel as described above.

3.9m: Fine SAND with some shell fragments and minor silt; grey. Medium dense, moist.

10.9m: With only minor intact shells/shell fragments.
11m: Grading to silty, low plasticity.

11.4m: Sandy SILT with some gravel; brown-grey. Soft to firm, moist, low plasticity. Sand is fine. Gravel is fine to medium, angular to sub-angular, moderately to highly weathered, very weak to weak greywacke.

16m: Grading to stiff.

17 to 17.5m: Recovered as gravel in a sandy silty matrix; brown. Stiff, wet, low plasticity. Sand is fine. Gravel is fine to medium, angular to subangular, very weak greywacke. (Drilling induced).

18.3 to 18.9m: As above.

19.1 to 19.3m: As above; loose, dry.

DH01 terminated at 19.68m
Target Depth

Date Time (m)

Hand Held Shear Vane

vane shear strength per NZGS guideline

Remarks
Coordinates in terms of NZTM2000 and are approximate.
Groundwater not encountered.

Casing Details
Depth Diameter

Date logged 15/12/2015
Logged TK
Checked RBG

Driller Griffiths Drilling
Drill Rig Crawler Sonic
Started 14/12/2015
Finished 15/12/2015
Core Boxes 6

Box: 1 of 6 - Depth: 1.50m to 4.95m of 19.68m
Date Drilled 14/12/2015 to 15/12/2015

Box: 2 of 6 - Depth: 4.95m to 7.95m of 19.68m
Date Drilled 14/12/2015 to 15/12/2015

PHOTOGRAPHIC LOG OF DRILLHOLE

Project Shelly Bay Development
Location Shelly Bay, Wellington

HOLE IDENTIFICATION **DH01**

Box: 3 of 6 - Depth: 7.95m to 10.95m of 19.68m

Date Drilled 14/12/2015 to 15/12/2015

Box: 4 of 6 - Depth: 10.95m to 13.95m of 19.68m

Date Drilled 14/12/2015 to 15/12/2015

PHOTOGRAPHIC LOG OF DRILLHOLE

Project Shelly Bay Development
Location Shelly Bay, Wellington

HOLE IDENTIFICATION **DH01**

Box: 5 of 6 - Depth: 13.95m to 16.84m of 19.68m

Date Drilled 14/12/2015 to 15/12/2015

Box: 6 of 6 - Depth: 16.84m to 19.68m of 19.68m

Date Drilled 14/12/2015 to 15/12/2015

AECOM

LOG OF DRILLHOLE

HOLE IDENTIFICATION	**DH02**

Client	The Wellington Company Ltd.
Project	Shelly Bay Development
Project number	60480847

Co-ordinates	1752628mE	5426889mN
Orientation	-90°	Elevation (Approx)
Location	Shelly Bay, Wellington	
Feature	Car park adjacent to South Bay Officer's Mess Garages.	

GEOLOGICAL DESCRIPTION	Test Records Shear Vane residual - peak 0 - 200 kPa	N Values 0 - 50	Drilling Method Casing remarks	Core Loss/Lift 0 - 100%	Depth	Graphic Log	SOIL PROPERTIES Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc	Instrumentation
0m: Reclamation Fill			VAC EX		1		0m: Vacuum excavation, no recovery.	
RAKAIA TERRANE 1.5m: Highly weathered, very weak, brown, silty fine SANDSTONE [Greywacke].	ss 4,9,12, 12,14,12 for 65mm N>50		SPT				1.5m: Recovered as fine to coarse GRAVEL with minor cobbles in a fine silty sandy matrix; light brown. Dense; dry. Gravel is fine to coarse, angular to subangular, greywacke. Gravel crumbles under firm finger pressure to fine silty sand.	
			Sonic		2			
	ss 8,23,43, 7 for 15mm N>50		SPT		3			
			Sonic		4		3.8 to 4.6m: With minor coarse gravel of moderately weathered, moderately strong greywacke.	
	ss 20,30 for 25mm N>50		SPT					
							DH02 terminated at 4.6m Target Depth	

Date Time (m)	Remarks Coordinates in terms of NZTM2000 and are approximate. Groundwater not encountered.	Driller Griffiths Drilling	Started 15/12/2015
		Drill Rig Crawler Sonic	Finished 15/12/2015
Hand Held Shear Vane *vane shear strength per NZGS guideline*	Casing Details Depth Diameter	Date logged 15/12/2015 Logged TK Checked RBG	Core Boxes 1 Page 1 of 2

Box: 1 of 1 - Depth: 1.50m to 4.60m of 4.60m

Date Drilled 15/12/2015 to 15/12/2015

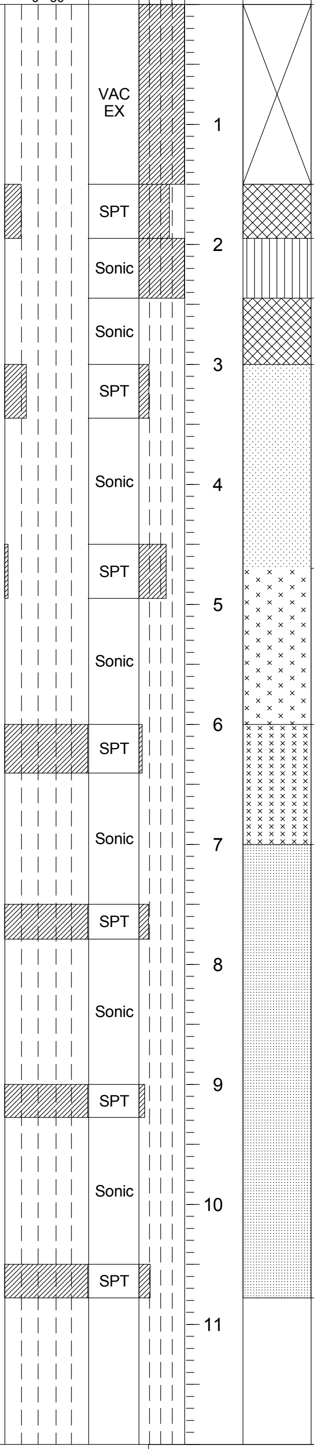
LOG OF DRILLHOLE

HOLE IDENTIFICATION **DH03**

Client The Wellington Company Ltd.
Project Shelly Bay Development
Project number 60480847

Co-ordinates 1752594mE 5427090mN
Orientation -90° Elevation (Approx)
Location Shelly Bay, Wellington
Feature Footprint of demolished Airmen's Accommodation Building.

	GEOLOGICAL DESCRIPTION	Test Records (N Values)	Drilling Method	SOIL PROPERTIES
FILL	0m: Demolition Fill		VAC EX	0m: Vacuum excavation, no recovery.
	1.5m: Reclamation Fill	ss 3,3,3, 3,2,2 N=10	SPT	1.5m: GRAVEL and COBBLES; light brown. Loose, moist. Cobbles and gravel are angular to subangular, moderately strong to strong greywacke. Gravel is fine to coarse.
	1.95m: Core Loss		Sonic	1.95m: Core Loss
	2.45m: Reclamation Fill		Sonic	2.45m: Soil description as above. 2.65 to 3m: In a sandy matrix with some silt.
Marginal Marine Deposits	3m: Marine Sediments comprising fine sand and silt with intact shells and shell fragments.	ss 3,4,3, 3,3,4 N=13	SPT	3m: Fine SAND with some wood fragments and minor silt; grey. Medium dense, moist.
			Sonic	3.9 to 3.95m: Large root fragment, partially decomposed. No odour. 3.95 to 4.7m: Grading to a fine sandy SILT with some shell fragments.
		ss 3,2,1, 0,1,0 N=2	SPT	4.7m: SILT; grey. Very soft, saturated, highly plastic. Recovered as a slurry.
			Sonic	5.4m: Dilatant, free water appears on surface when tapped/shaken in hand.
RAKAIA TERRANE	6m: Moderately weathered, grey-brown, fine to medium sandy SILTSTONE [greywacke]. Very weak, closely to very closely spaced joints. 6.5 to 7m: Grading to a silty fine to medium SANDSTONE.	ss 3,6,9, 11,18,12 for 30mm N>50	SPT	
			Sonic	
	7m: Moderately weathered, light brown, silty fine SANDSTONE [greywacke]. Very weak, closely spaced joints.	ss 6,12,18, 32 for 65mm N>50	SPT	7.15 to 7.3m: Recovered as fine to medium gravel with minor cobbles in a fine sandy silty matrix; light brown. Loosely packed; dry. Gravel is highly weathered, very weak, fine to medium sandstone. (drilling induced).
			Sonic	8.5 to 8.7m: Recovered as fine to medium gravel with minor cobbles in a fine sandy silty matrix; light brown. Loosely packed; dry. Gravel as described above (drilling induced).
		ss 6,7,32, 18 for 60mm N>50	SPT	
			Sonic	10 to 10.5m: Recovered as fine to medium gravel with minor cobbles in a fine sandy silty matrix; light brown. Loosely packed; dry. Gravel as described above (drilling induced).
		ss 9,19,24, 26 for 55mm N>50	SPT	
				DH03 terminated at 10.78m Target Depth



SOIL PROPERTIES: Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc

Instrumentation

Date Time (m)

Hand Held Shear Vane
vane shear strength per NZGS guideline

Remarks
Coordinates in terms of NZTM2000 and are approximate.
Groundwater not encountered.

Casing Details
Depth Diameter

Date logged 16/12/2015
Logged TK
Checked RBG

Driller Griffiths Drilling
Drill Rig Crawler Sonic
Started 15/12/2015
Finished 16/12/2015
Core Boxes 3

Page 1 of 3

Date Printed: 22/01/2016

Box: 1 of 3 - Depth: 1.50m to 5.20m of 10.78m

Date Drilled 15/12/2015 to 16/12/2015

Box: 2 of 3 - Depth: 5.20m to 8.00m of 10.78m

Date Drilled 15/12/2015 to 16/12/2015

PHOTOGRAPHIC LOG OF DRILLHOLE

Project Shelly Bay Development
Location Shelly Bay, Wellington

HOLE IDENTIFICATION **DH03**

Box: 3 of 3 - Depth: 8.00m to 10.78m of 10.78m

Date Drilled 15/12/2015 to 16/12/2015

AECOM

LOG OF DRILLHOLE

HOLE IDENTIFICATION **DH04**

Client The Wellington Company Ltd.
Project Shelly Bay Development
Project number 60480847

Co-ordinates 1752586mE 5427135mN
Orientation -90° Elevation (Approx)
Location Shelly Bay, Wellington
Feature Adjacent to W/O and SNCO's Mess Building.

GEOLOGICAL DESCRIPTION

Unit	Description
FILL	0m: Topsoil
FILL	0.3m: Core Loss
FILL	0.64m: Reclamation Fill
Marine	3.75m: Marine Sediments comprising fine sand and silt with intact shells and shell fragments.
RAKAIA TERRANE	5.5m: Highly weathered, extremely weak, silty fine SANDSTONE [greywacke].
RAKAIA TERRANE	11.5m: Moderately weathered, light brown, silty fine SANDSTONE [greywacke]. Very weak, closely spaced joints.

Test Records

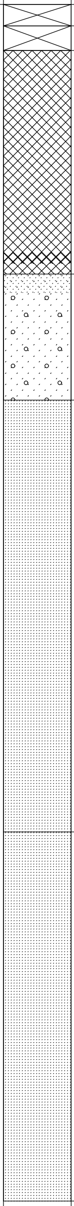
Shear Vane residual - peak 0 - 200 kPa; N Values 0 - 50

Drilling Method (Casing remarks)	Test Records
DUG	
Sonic	
SPT	ss 3,3,2, 3,3,7 N=15
Sonic	
SPT	ss 2,2,4, 3,1,2 N=10
Sonic	
SPT	ss 3,7,7, 6,5,5 N=23
Sonic	
SPT	ss 5,8,6, 8,6,7 N=27
Sonic	
SPT	ss 4,6,10, 9,7,9 N=35
Sonic	
SPT	ss 6,14,7, 10,7,9 N=33
Sonic	
SPT	ss 2,1,2, 1,2,4 N=9
Sonic	
SPT	ss 4,7,22, 26 for 35mm N>50
Sonic	
SPT	ss 4,30,20 for 25mm N>50
Sonic	
SPT	ss 9,20,50 for 55mm N>50
Sonic	
SPT	ss 16,34 for 35mm N>50

Core Loss/Lift 0 - 100%

Depth 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19

Graphic Log



SOIL PROPERTIES

Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc

0m: (Hand excavated).

0.3m: Core Loss

0.64m: Gravelly SILT with some sand; brown. Soft to firm, moist, high plasticity. Sand is fine. Gravel is fine to coarse, angular to subrounded, moderately weathered, weak to moderately strong greywacke.

3.45 to 3.6m: Grading to saturated.

3.75m: Fine to medium SAND with some shell fragments; light grey. Medium dense, moist.

4m: Silty GRAVEL with some sand; light grey. Medium dense, wet. Gravel is fine to coarse, angular to subangular greywacke.

5 to 5.5m: Grading to light brown.

5.5m: Recovered as fine to coarse GRAVEL in a fine silty sandy matrix; light brown. Medium dense; dry. Gravel is angular to subangular, extremely weak greywacke. Gravel crumbles under firm finger pressure to fine silty sand.

11.6 to 13.5m: Recovered as fine to coarse GRAVEL in a fine silty sandy matrix; light brown. Loosely packed; dry. Gravel is angular to subangular, weak greywacke. Gravel crumbles under firm finger pressure to fine silty sand. (Drilling induced).

14.6 to 15m: As above.

16 to 16.5m: As above; gravel is coarse

DH04 terminated at 16.63m
Target Depth

Instrumentation

Date Time (m)

Hand Held Shear Vane

vane shear strength per NZGS guideline

Remarks
Coordinates in terms of NZTM2000 and are approximate.
Groundwater not encountered.

Casing Details
Depth Diameter

Date logged 17/12/2015
Logged TK
Checked RBG

Driller Griffiths Drilling
Started 16/12/2015
Drill Rig Crawler Sonic
Finished 17/12/2015
Core Boxes 6

Date Printed: 22/01/2016

PHOTOGRAPHIC LOG OF DRILLHOLE

Project Shelly Bay Development
Location Shelly Bay, Wellington

HOLE IDENTIFICATION **DH04**

Box: 1 of 6 - Depth: 0.30m to 3.45m of 16.63m

Date Drilled 16/12/2015 to 17/12/2015

Box: 2 of 6 - Depth: 3.45m to 6.45m of 16.63m

Date Drilled 16/12/2015 to 17/12/2015

Box: 3 of 6 - Depth: 6.45m to 9.45m of 16.63m

Date Drilled 16/12/2015 to 17/12/2015

Box: 4 of 6 - Depth: 9.45m to 12.26m of 16.63m

Date Drilled 16/12/2015 to 17/12/2015

PHOTOGRAPHIC LOG OF DRILLHOLE

Project Shelly Bay Development
Location Shelly Bay, Wellington

HOLE IDENTIFICATION **DH04**

Box: 5 of 6 - Depth: 12.26m to 14.60m of 16.63m

Date Drilled 16/12/2015 to 17/12/2015

Box: 6 of 6 - Depth: 14.60m to 16.63m of 16.63m

Date Drilled 16/12/2015 to 17/12/2015

DRILLHOLE LOG SOIL SHELLYBAY.GPJ BASE.GDT 22/01/16

Appendix D

Trial Pit Logs

AECOM

LOG OF TEST PIT

INSPECTION PIT IDENTIFICATION **TP4**

Client The Wellington Company Ltd.
Project Shelly Bay Development
Project number 60480847

Co-ordinates 1752539mE 5427031mN
Orientation -90° Elevation (Approx)
Location Shelly Bay, Wellington
Feature Adjacent to Transfield Depot.

Depth		GEOLOGICAL DESCRIPTION Weathering, Colour, Fabric, Rock Name, Strength, Discontinuities, Lithological Features (bedding, foliation, mineralogy, cement, etc)	Test Records	Sampling	Dynamic Cone Penetrometer (Blows per mm) 2 4 6 8	SOIL PROPERTIES Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc Depth Related Remarks DEFECT DESCRIPTION (Joints, Bedding Seams, Shatter, Shear and Crush Zones, Foliation, Schistosity, Attitude, Spacing, Continuity, Roughness, Infilling, etc.)	Graphic Log		Instrumentation
0.0–0.3	TOPSOIL	0m: Topsoil				0m: SILT with minor sand and minor gravel; light brown. Loosely packed, dry. Sand is fine. Gravel is fine to medium, angular greywacke. 0.3m: With minor glass and brick fragments.			
0.3–0.5	FILL	0.3m: Reclamation Fill				0.3m: GRAVEL and COBBLES in a sandy matrix with minor silt; brown. Loosely packed, dry. Gravel and cobbles are angular to subangular greywacke. Gravel is fine to medium.			
0.5–2.0						0.5m: BOULDERS, COBBLES and GRAVEL in a silty matrix with minor sand; brown. Loosely packed, moist. Boulders, cobbles and gravel are angular to subangular, moderately weathered greywacke. Gravel is fine to coarse.			
2.0–2.2	Marine	2m: Marginal Marine Sediments				2m: Sandy SILT with intact shells and shell fragments; dark grey. Loose, moist. Sand is fine.			
2.2						TP4 terminated at 2.2m Unable to advance as too difficult to excavate			

Depth scale: 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4

For explanation of symbols and observations, see key sheet

FLUID DEPTHS DURING DRILLING

Date Time	Drilled Depth (m)	Casing Depth (m)	Fluid Depth (m)

Hand Held Shear Vane

Vane shear strength per NZGS guideline

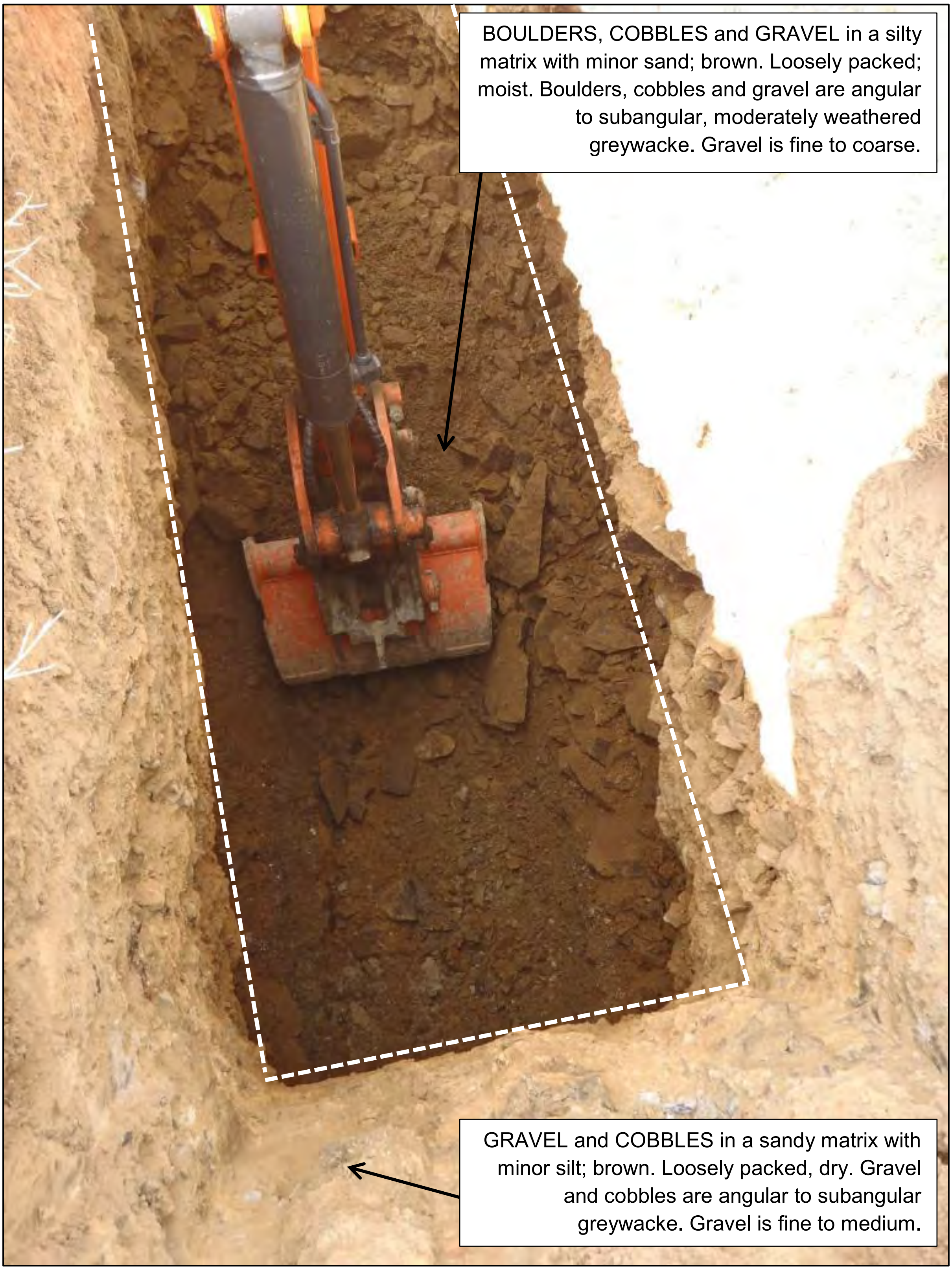
Length
Width
Stability Stable

Excavation Method 3.5 Tonne Excavator
Orientation B -90°

Remarks
Coordinates in terms of NZTM2000 and are approximate.
Trial pit terminated upon establishing greywacke basement.
Hole backfilled with spoil upon completion.
No groundwater encountered.

Started 17/12/2015
Finished 17/12/2015
Date logged 17/12/2015
Logged TK
Checked RBG

22/01/2016



BOULDERS, COBBLES and GRAVEL in a silty matrix with minor sand; brown. Loosely packed; moist. Boulders, cobbles and gravel are angular to subangular, moderately weathered greywacke. Gravel is fine to coarse.

GRAVEL and COBBLES in a sandy matrix with minor silt; brown. Loosely packed, dry. Gravel and cobbles are angular to subangular greywacke. Gravel is fine to medium.

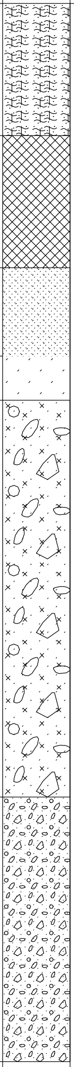
AECOM

LOG OF TEST PIT

INSPECTION PIT IDENTIFICATION **TP5**

Client	The Wellington Company Ltd.
Project	Shelly Bay Development
Project number	60480847
Co-ordinates	1752605mE 5427077mN
Orientation	-90° Elevation (Approx)
Location	Shelly Bay, Wellington
Feature	Footprint of demolished Airmen's Accommodation Building.

Depth		GEOLOGICAL DESCRIPTION Weathering, Colour, Fabric, Rock Name, Strength, Discontinuities, Lithological Features (bedding, foliation, mineralogy, cement, etc)	Test Records	Sampling	Dynamic Cone Penetrometer (Blows per mm) 2 4 6 8	SOIL PROPERTIES Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc Depth Related Remarks DEFECT DESCRIPTION (Joints, Bedding Seams, Shatter, Shear and Crush Zones, Foliation, Schistosity, Attitude, Spacing, Continuity, Roughness, Infilling, etc.)	Graphic Log		Instrumentation
0.0–0.3	TOPSOIL	0m: Topsoil				0m: Gravelly SILT; light brown. Loose, dry. Gravel is angular to subangular, fine to medium.			
0.3–0.6	FILL	0.3m: Demolition Fill				0.3m: GRAVEL and COBBLES in a silty matrix with some intact shells and shell fragments; light brown. Loosely packed, dry. Cobbles and gravel are angular, moderately weathered, strong greywacke. Gravel is fine to coarse. Some coarse gravel to cobble sized fragments of brick, concrete and ceramic; minor fragments of wood, 0.5 to 0.6m in length; iron pins. 0.5 to 0.6m: Concrete boulder, 400mm diameter.			
0.6–0.9	Marine	0.6m: Marginal Marine Sediments				0.6m: Fine to medium SAND with minor gravel and some rootlets; black. Loose, moist. Gravels are subangular to subrounded, fine to medium, greywacke. 0.8m: Coarse SAND; brown. Loose, moist.			
0.9–1.8	RAKAIA TERRANE	0.9m: Highly weathered, brown, silty fine SANDSTONE [greywacke].				0.9m: COBBLES and GRAVEL in a sandy silty matrix with minor boulders; grey-brown. Loosely packed; moist. Gravel is fine to coarse. Gravel, cobbles and boulders are angular to subrounded, moderately weathered greywacke.		1.8 ▽	
1.8–2.4		1.8m: Moderately weathered, brown, fine SANDSTONE [greywacke].				1.8m: Recovered as angular to subangular COBBLES and fine to coarse GRAVEL in a sandy matrix with some boulders.			
2.4						TP5 terminated at 2.4m Unable to advance as too difficult to excavate			



For explanation of symbols and observations, see key sheet

FLUID DEPTHS DURING DRILLING

Date Time	Drilled Depth (m)	Casing Depth (m)	Fluid Depth (m)
17/12/2015 00:00	1.80	-	1.8

Hand Held Shear Vane

Vane shear strength per NZGS guideline

Length

Width

Stability Stable

Excavation Method 3.5 Tonne Excavator

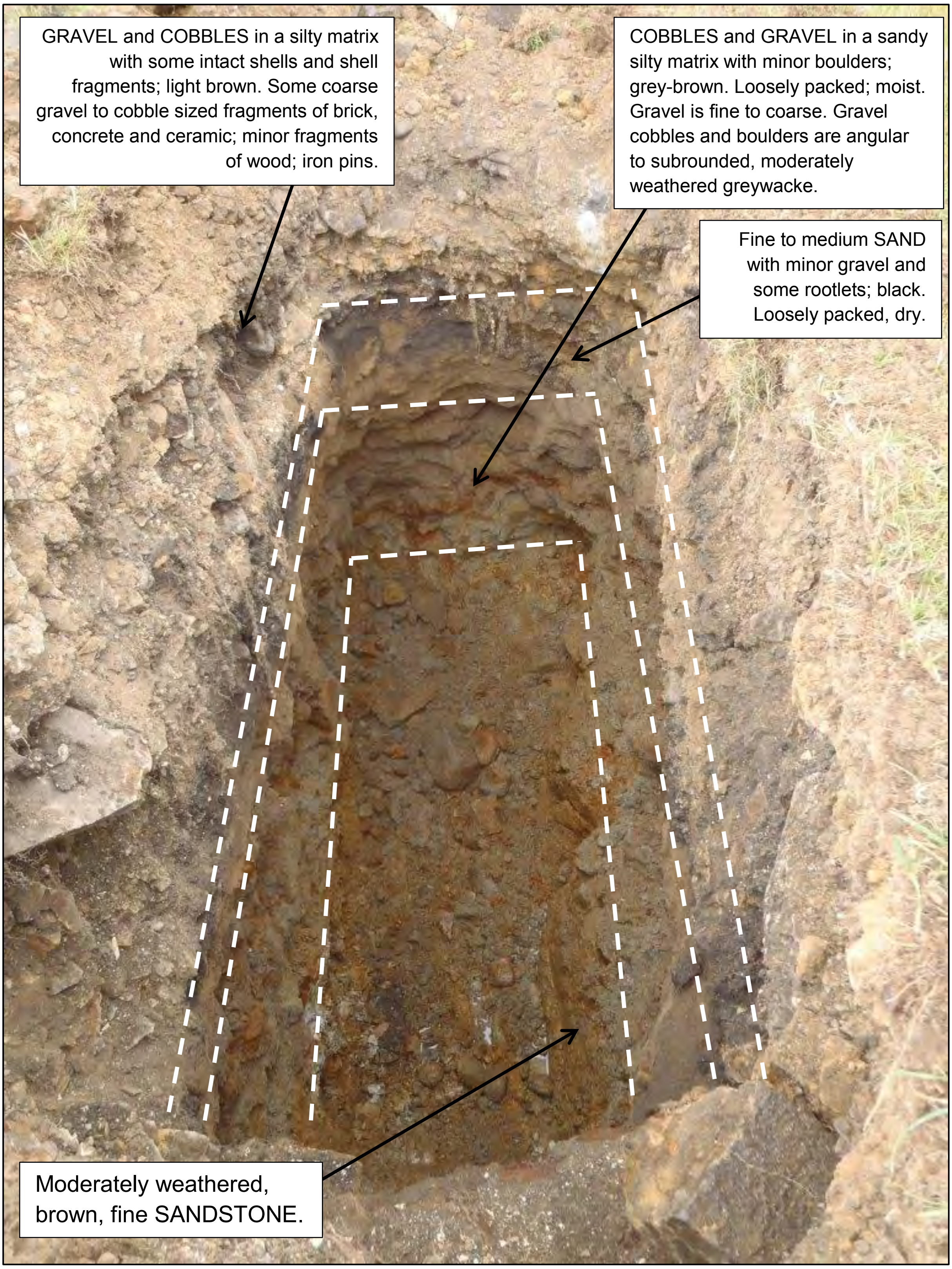
Orientation B -90°

Remarks

Coordinates in terms of NZTM2000 and are approximate.
Trial pit terminated upon establishing greywacke basement.
Hole backfilled with spoil upon completion.

Started 17/12/2015
Finished 17/12/2015
Date logged 17/12/2015
Logged TK
Checked RBG

22/01/2016



GRAVEL and COBBLES in a silty matrix with some intact shells and shell fragments; light brown. Some coarse gravel to cobble sized fragments of brick, concrete and ceramic; minor fragments of wood; iron pins.

COBBLES and GRAVEL in a sandy silty matrix with minor boulders; grey-brown. Loosely packed; moist. Gravel is fine to coarse. Gravel cobbles and boulders are angular to subrounded, moderately weathered greywacke.

Fine to medium SAND with minor gravel and some rootlets; black. Loosely packed, dry.

Moderately weathered, brown, fine SANDSTONE.

LOG OF TEST PIT

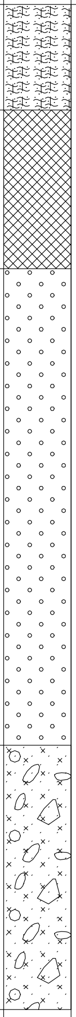
INSPECTION PIT IDENTIFICATION **TP6**

Client	The Wellington Company Ltd.
Project	Shelly Bay Development
Project number	60480847

Co-ordinates	1752612mE 5427114mN
Orientation	-90° Elevation (Approx)
Location	Shelly Bay, Wellington
Feature	Footprint of demolished Airmen's Accommodation Building.

Depth		GEOLOGICAL DESCRIPTION Weathering, Colour, Fabric, Rock Name, Strength, Discontinuities, Lithological Features (bedding, foliation, mineralogy, cement, etc)	Test Records	Sampling	Dynamic Cone Penetrometer (Blows per mm) 2 4 6 8	SOIL PROPERTIES Subordinate MAJOR minor; colour; structure. Strength; moisture condition; grading; bedding; plasticity; sensitivity; major fraction description; subordinate fraction description; minor fraction description etc Depth Related Remarks DEFECT DESCRIPTION (Joints, Bedding Seams, Shatter, Shear and Crush Zones, Foliation, Schistosity, Attitude, Spacing, Continuity, Roughness, Infilling, etc.)	Graphic Log		Instrumentation
0	TOPSOIL	0m: Topsoil				0m: Gravelly SILT with some rootlets; light brown. Loosely packed, dry. Gravel is fine to medium, subangular to rounded, moderately weathered, moderately strong greywacke.			
0.2	FILL	0.2m: Reclamation Fill				0.2m: Silty GRAVEL with some cobbles and rootlets and minor boulders; light brown. Loosely packed, dry. Cobbles and gravel are angular, moderately weathered strong greywacke. Gravel is fine to coarse.			
0.5	Marine Sediments	0.5m: Marginal Marine Sediments				0.5m: GRAVEL and shell fragments with minor sand and minor intact shells; black. Loosely packed, moist. Gravel is fine to coarse, sub-rounded to rounded. Sand is medium to coarse. Shell fragments; white, grade as fine to coarse sand; intact shells up to 20mm in size; trace fine purple shell fragments.			
1.4	RAKAIA TERRANE	1.4m: Highly weathered, brown, silty fine SANDSTONE [greywacke].				1.4m: COBBLES and GRAVEL in a sandy silty matrix with minor boulders; grey-brown. Loosely packed; moist. Gravel is fine to coarse. Gravel, cobbles and boulders are angular to subrounded, moderately weathered greywacke.		1.9 ▽	
1.9						TP6 terminated at 1.9m Unable to advance as too difficult to excavate			

Depth scale: 0.2, 0.4, 0.6, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.4



For explanation of symbols and observations, see key sheet

FLUID DEPTHS DURING DRILLING

Date Time	Drilled Depth (m)	Casing Depth (m)	Fluid Depth (m)
17/12/2015 00:00	1.90	-	1.9

Hand Held Shear Vane

Vane shear strength per NZGS guideline

Length

Width

Stability Stable

Excavation Method 3.5 Tonne Excavator

Orientation B -90°

Remarks

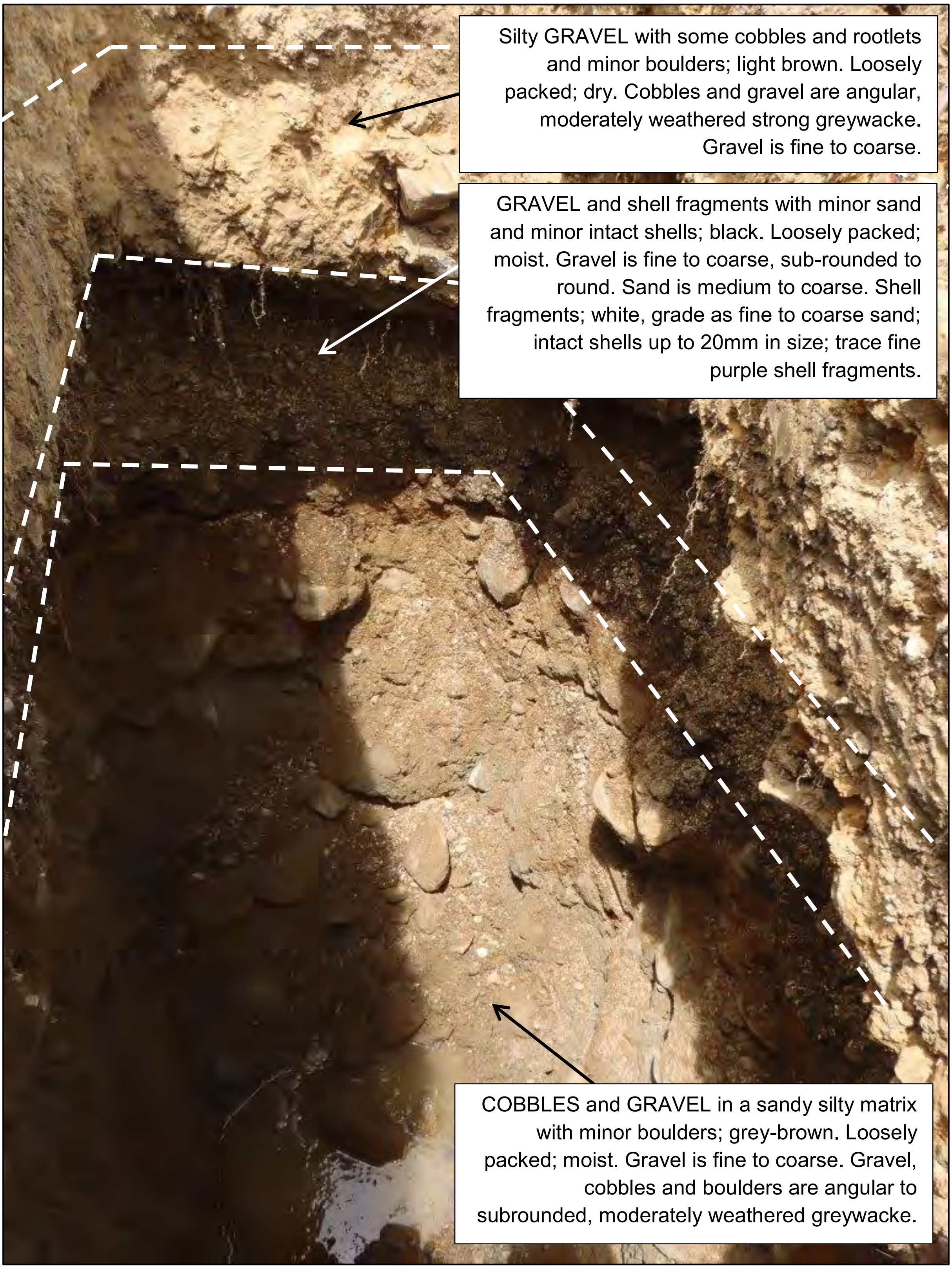
Coordinates in terms of NZTM2000 and are approximate.
Trial pit terminated upon establishing greywacke basement.
Hole backfilled with spoil upon completion.

Started 17/12/2015
Finished 17/12/2015
Date logged 17/12/2015
Logged TK
Checked RBG

Page 1 of 1

22/01/2016

Shelly Bay – TP5 Test Pit photograph



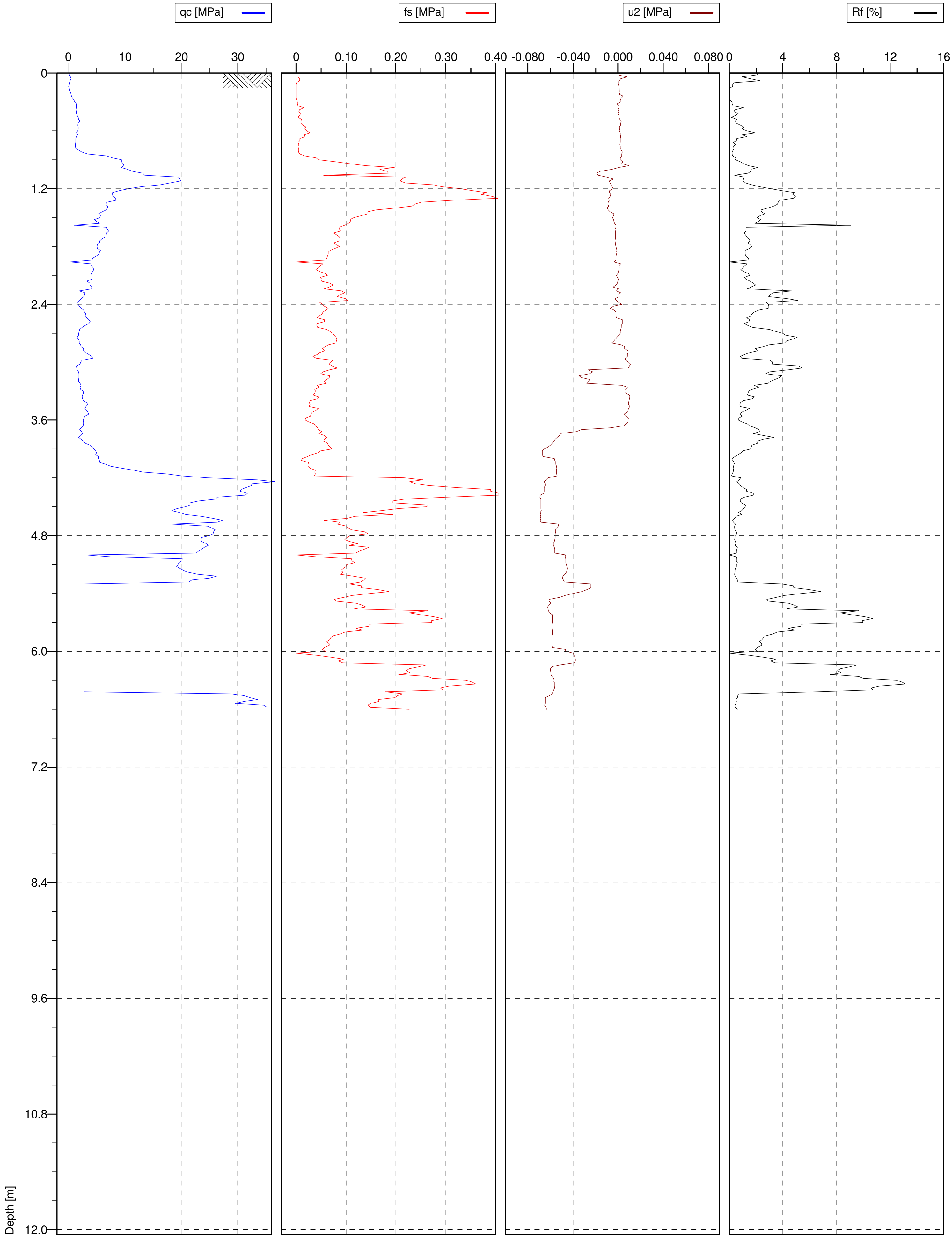
Silty GRAVEL with some cobbles and rootlets and minor boulders; light brown. Loosely packed; dry. Cobbles and gravel are angular, moderately weathered strong greywacke. Gravel is fine to coarse.

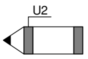
GRAVEL and shell fragments with minor sand and minor intact shells; black. Loosely packed; moist. Gravel is fine to coarse, sub-rounded to round. Sand is medium to coarse. Shell fragments; white, grade as fine to coarse sand; intact shells up to 20mm in size; trace fine purple shell fragments.

COBBLES and GRAVEL in a sandy silty matrix with minor boulders; grey-brown. Loosely packed; moist. Gravel is fine to coarse. Gravel, cobbles and boulders are angular to subrounded, moderately weathered greywacke.

Appendix E

CPT Logs





GRIFFITHS DRILLING

Cone No: 4818
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Shelly Bay	Position: S41"17.724', E174"49.351'	Ground level: 0.00	Test no: CPT1
Project ID: CPT1	Client: AECOM	Date: 17/12/2015	Scale: 1 : 50
Project: SHELLY BAY		Page: 1/1	Fig:
		File: Shelly Bay CPT1.cpt	

Appendix F

Analysis Output

1) Liquefaction Analysis (LiquefyPro & CLiq)

2) DIPs Discontinuity Analysis, Slope 1, 5 & 7

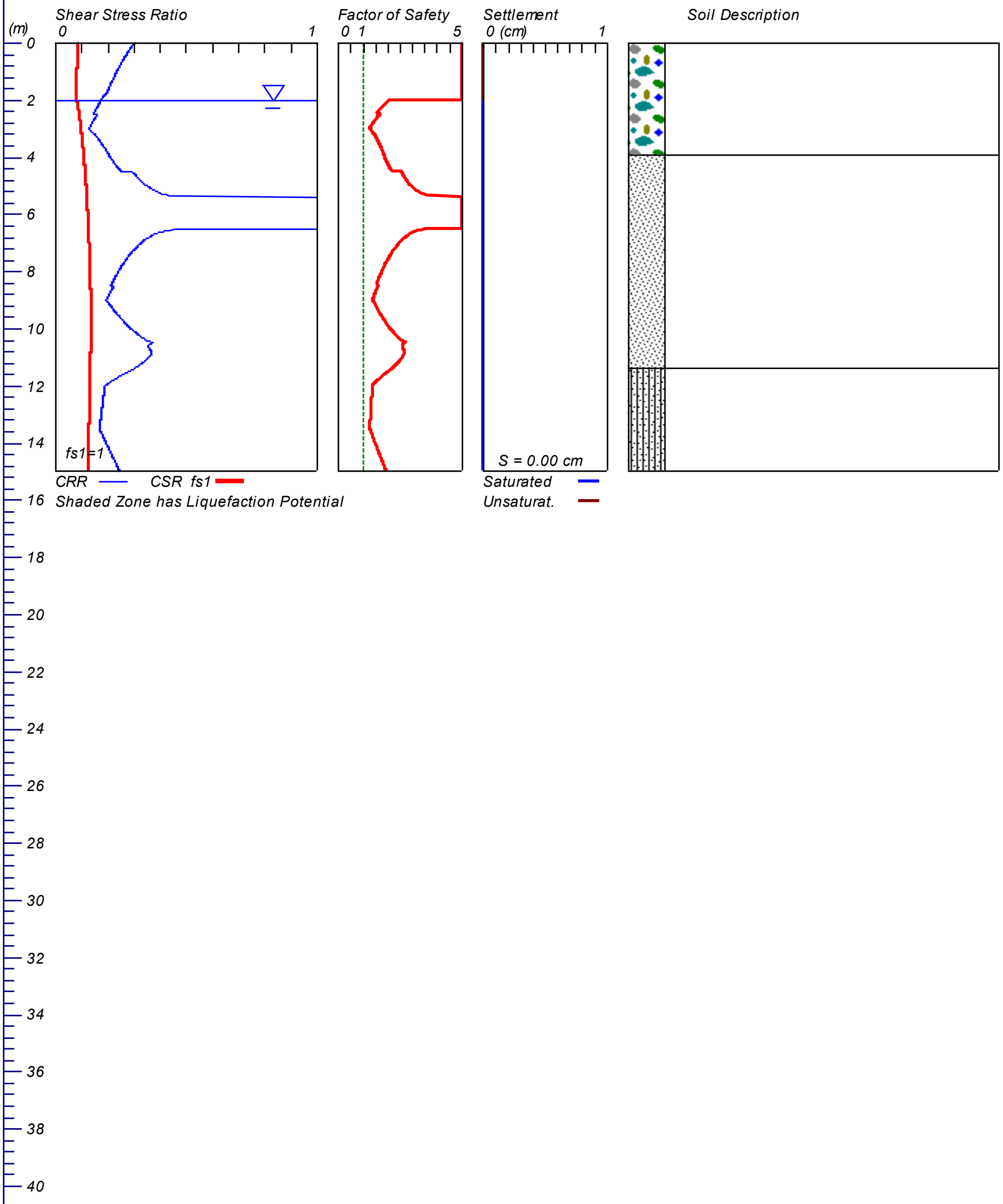
LIQUEFACTION ANALYSIS

DH01
SLS

Shelly Bay

Hole No.=DH01   ***Water Depth=2 m***

Magnitude=7.5
Acceleration=0.13g



Raw SPT	Unit Weight	Fines %
13	19	20
5	19	20
17	17	5
24	17	5
19	17	5
14	17	5
24	17	5
9	18	55
8	18	55
14	18	55

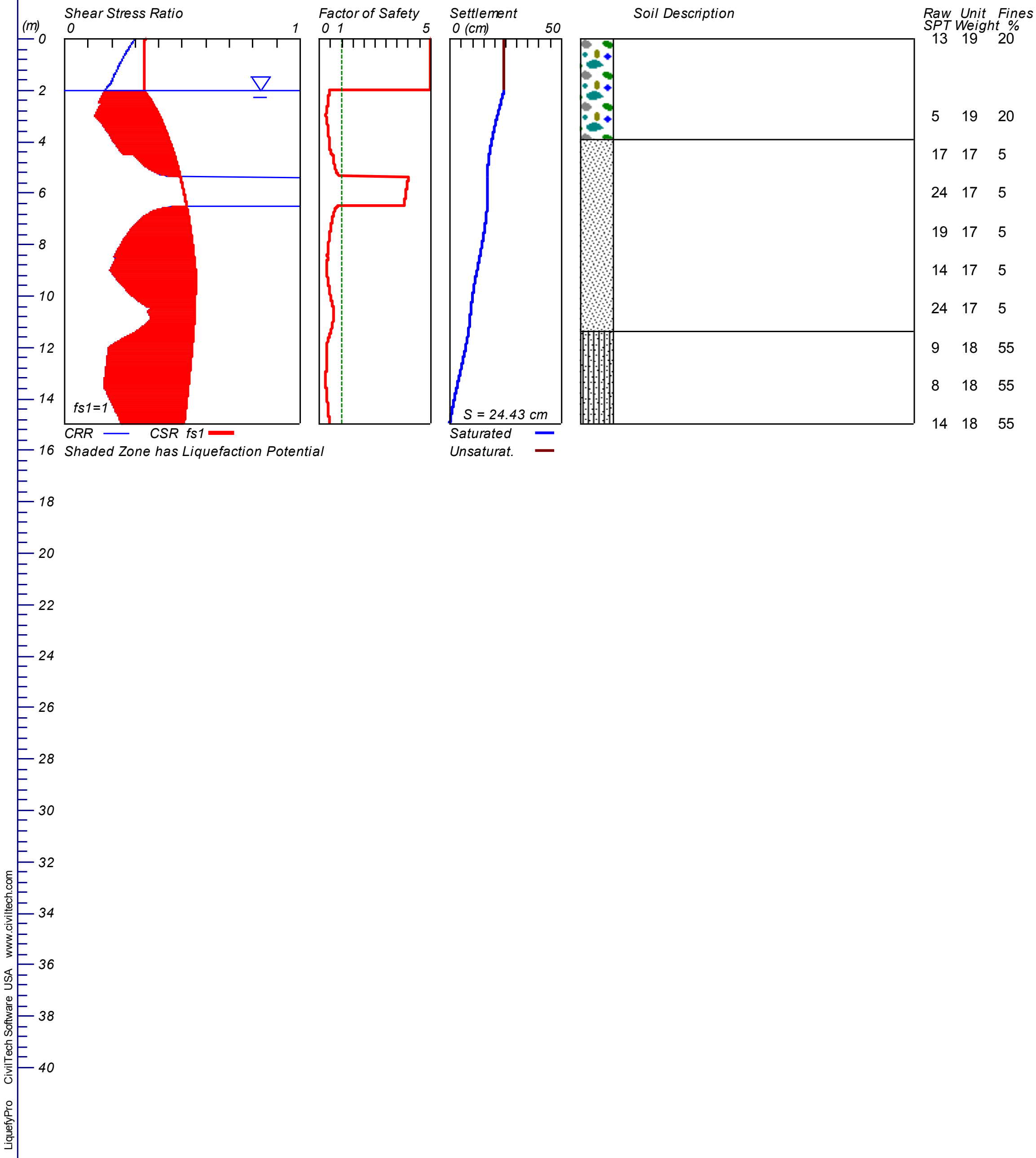
LIQUEFACTION ANALYSIS

DH01
ULS

Shelly Bay

Hole No.=DH01   Water Depth=2 m

Magnitude=7.5
Acceleration=0.53g



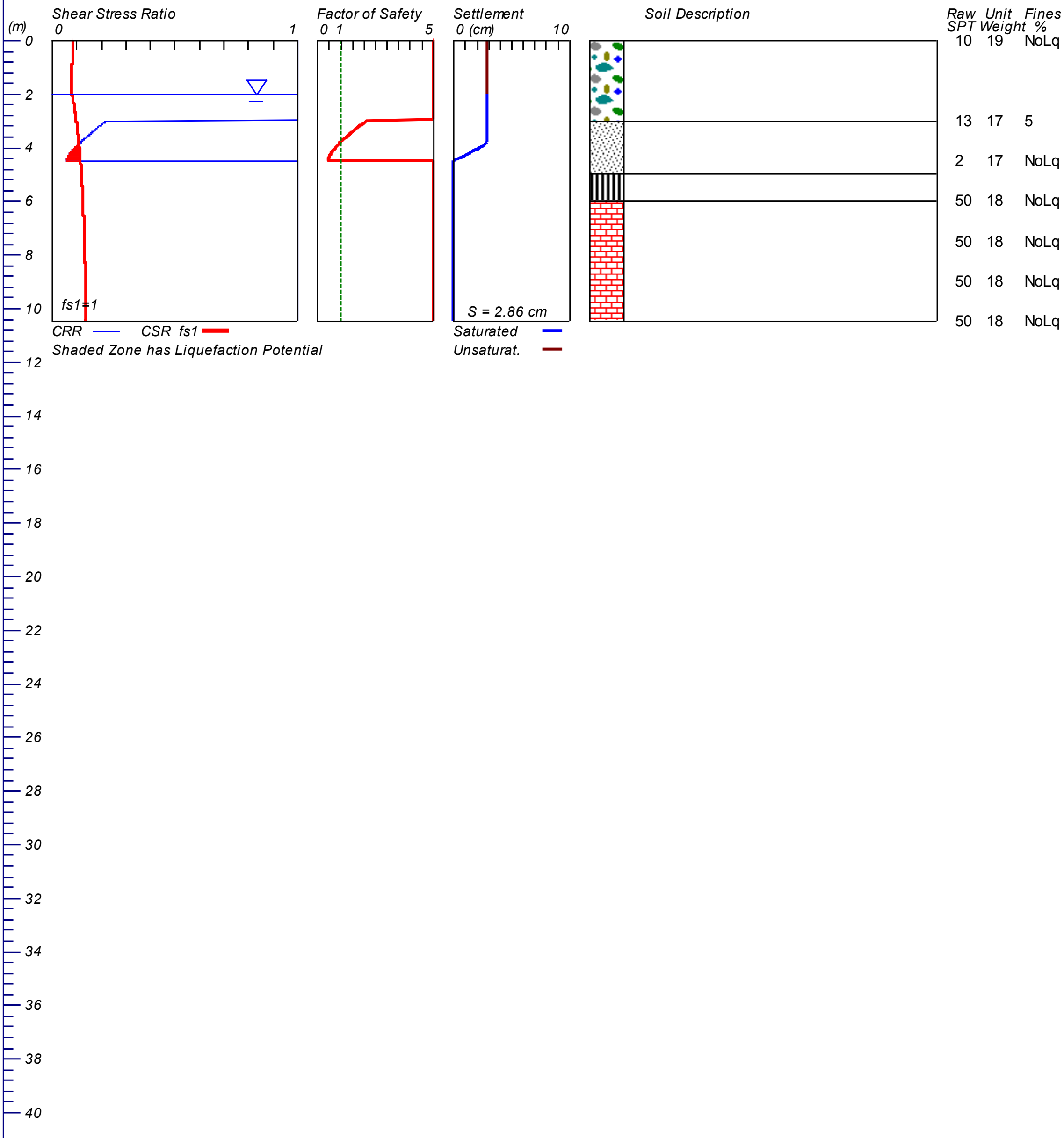
LIQUEFACTION ANALYSIS

DH03
SLS

Shelly Bay

Hole No.=DH03 Water Depth=2 m

Magnitude=7.5
Acceleration=0.13g



LiquefyPro CivilTech Software USA www.civiltech.com

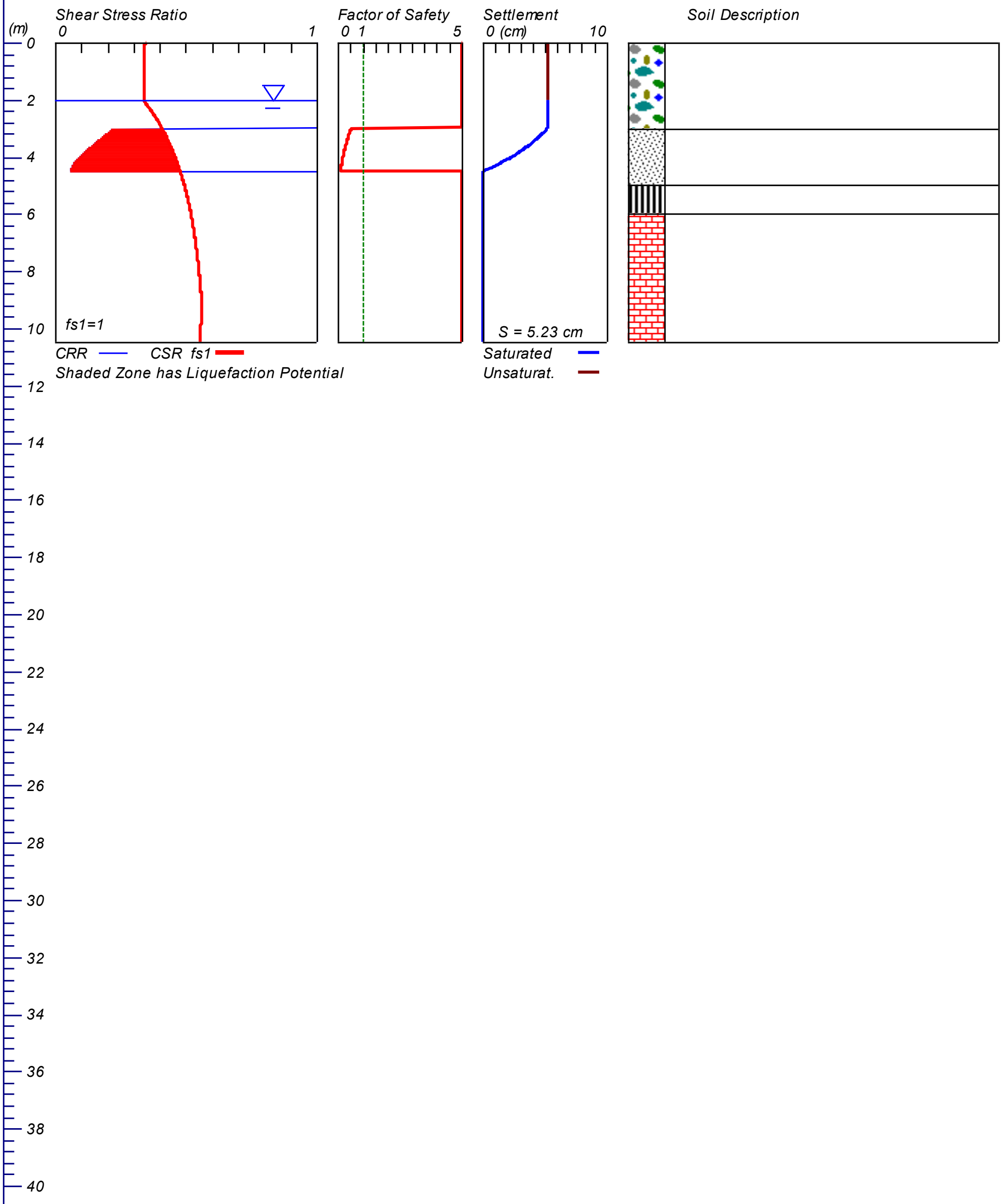
DH03
ULS

LIQUEFACTION ANALYSIS

Shelly Bay

Hole No.=DH03 Water Depth=2 m

Magnitude=7.5
Acceleration=0.53g



Raw SPT	Unit Weight	Fines %
10	19	NoLq
13	17	5
2	17	NoLq
50	18	NoLq
50	18	NoLq
50	18	NoLq
50	18	NoLq

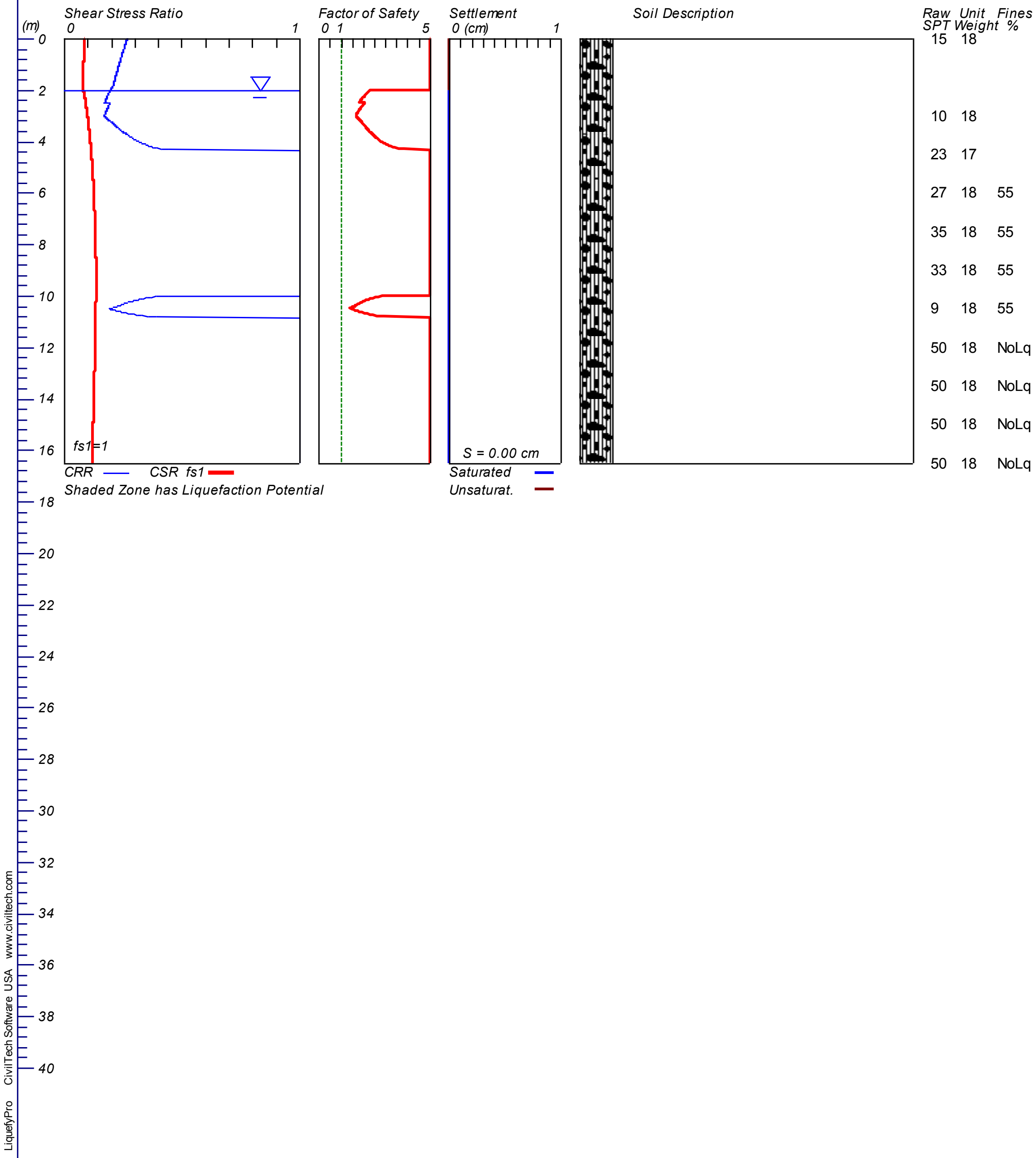
DH04
SLS

LIQUEFACTION ANALYSIS

Shelly Bay

Hole No.=DH04 Water Depth=2 m

Magnitude=7.5
Acceleration=0.13g



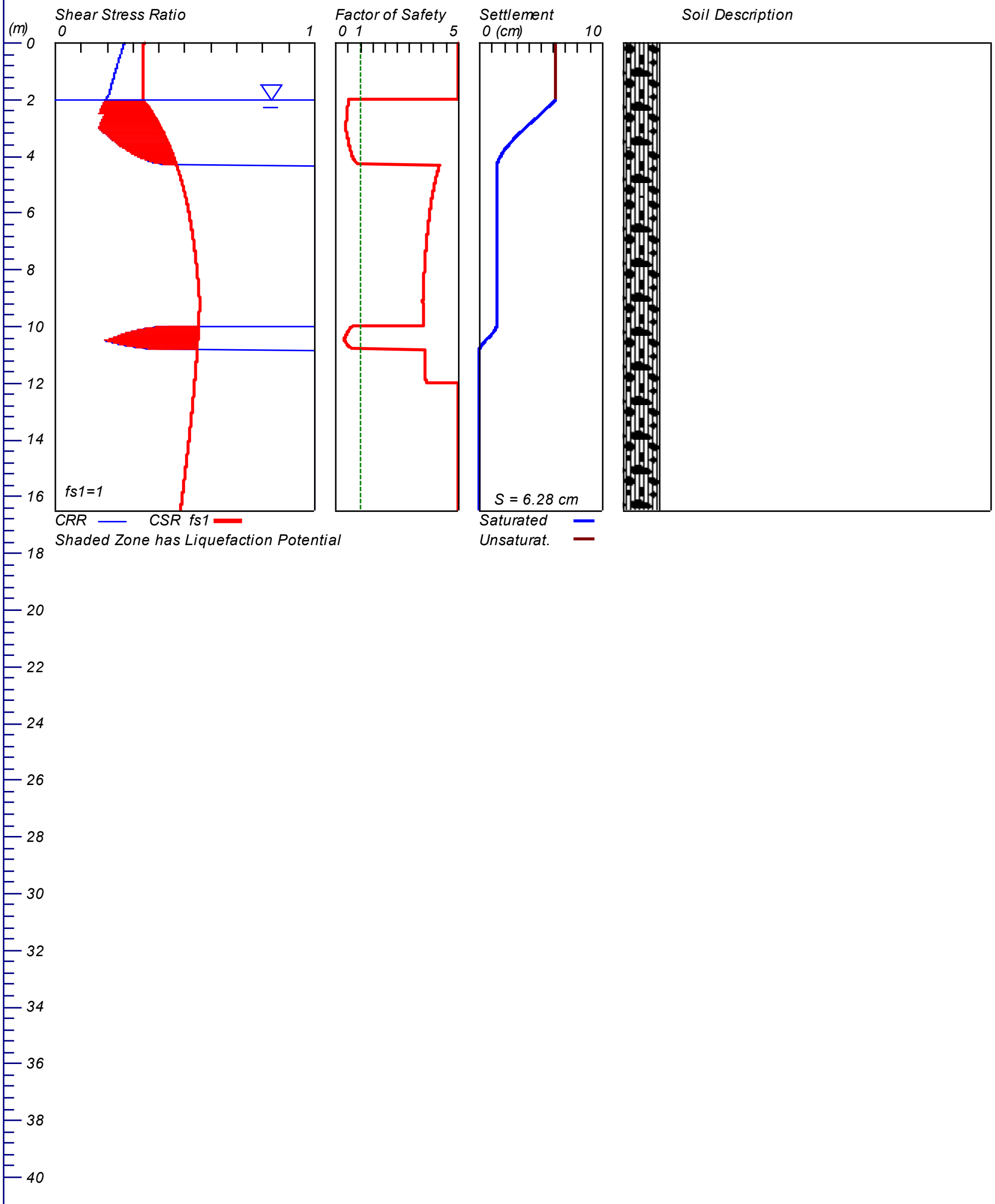
DH04
ULS

LIQUEFACTION ANALYSIS

Shelly Bay

Hole No.=DH04   Water Depth=2 m

Magnitude=7.5
Acceleration=0.53g



Raw SPT	Unit Weight	Fines %
15	18	
10	18	
23	17	
27	18	55
35	18	55
33	18	55
9	18	55
50	18	NoLq
50	18	NoLq
50	18	NoLq
50	18	NoLq

AECOM NZ LTD
121 Rostrevor Street
Hamilton
www.aecom.com

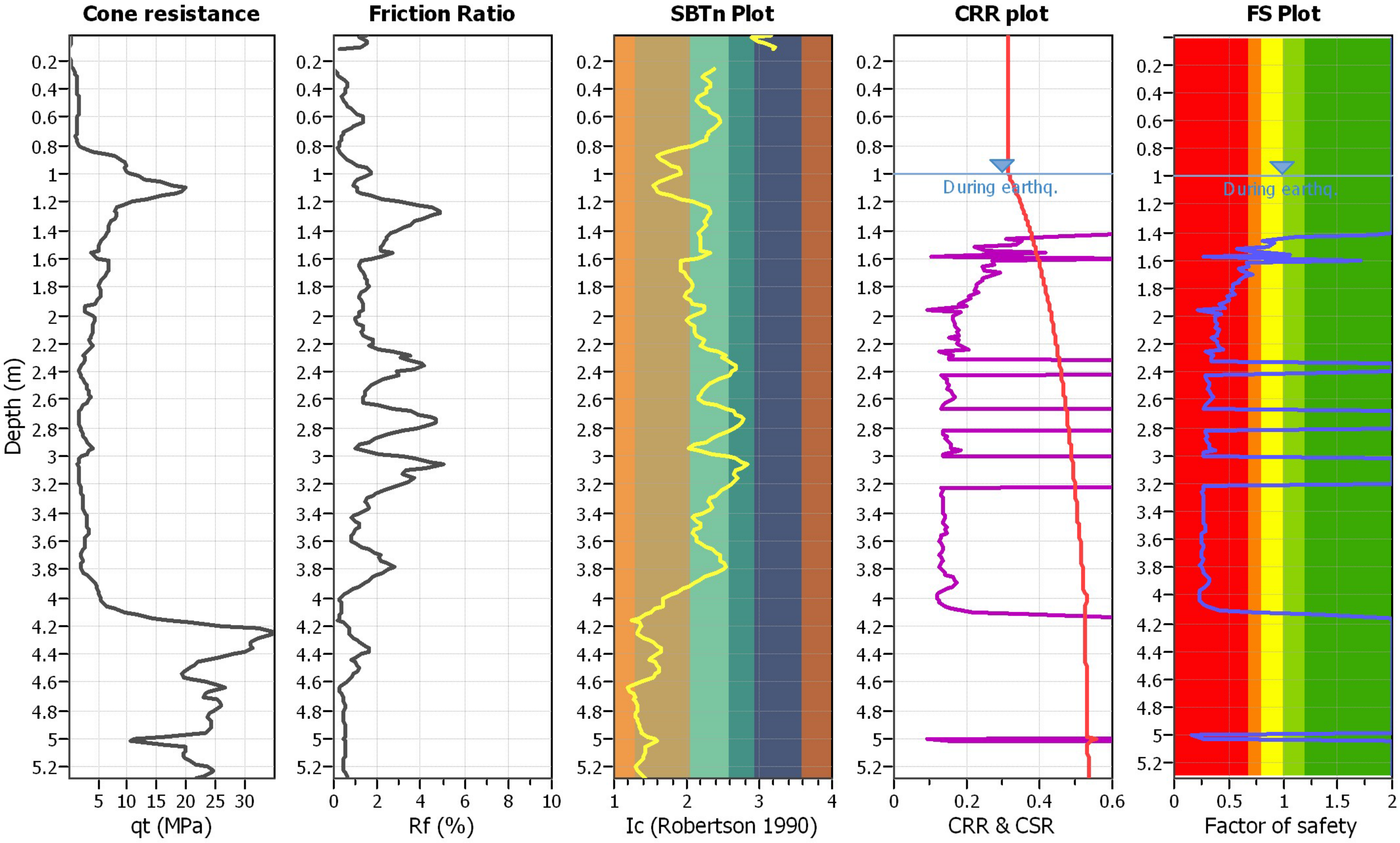
LIQUEFACTION ANALYSIS REPORT

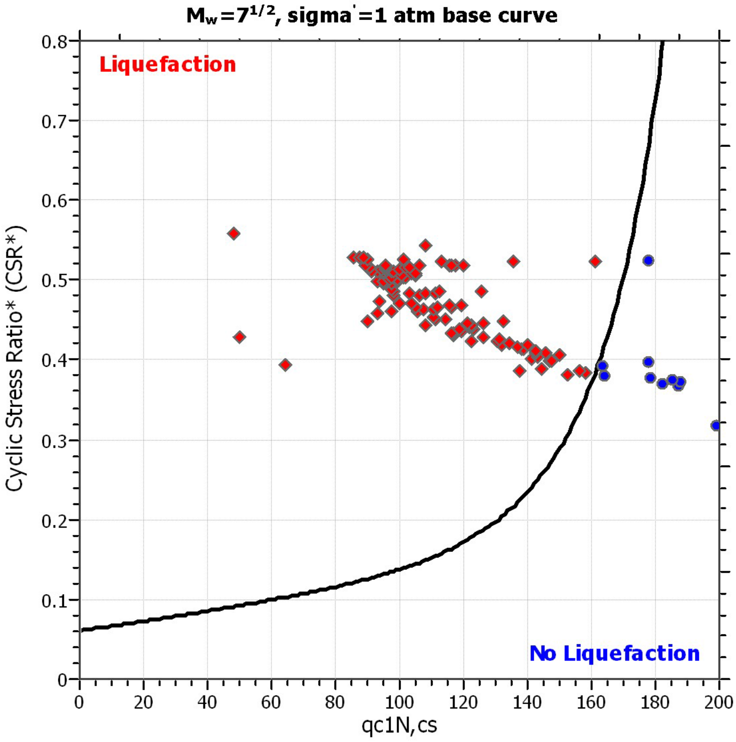
Project title : **Location :**

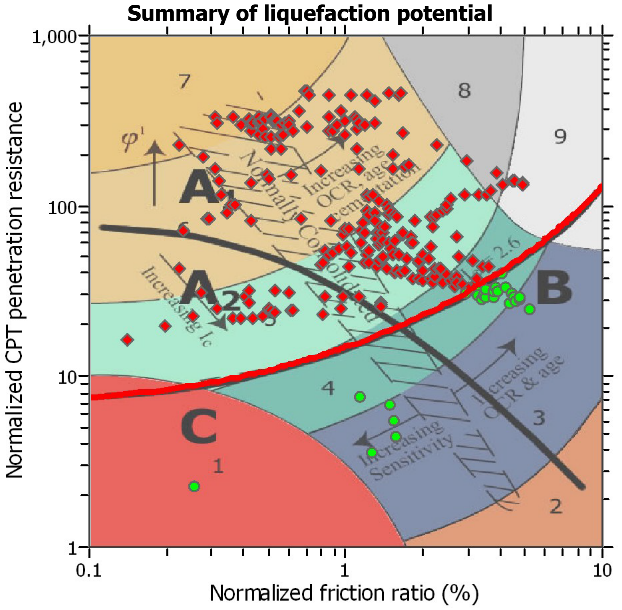
CPT file : Shelly Bay CPT1, ULS

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.53	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

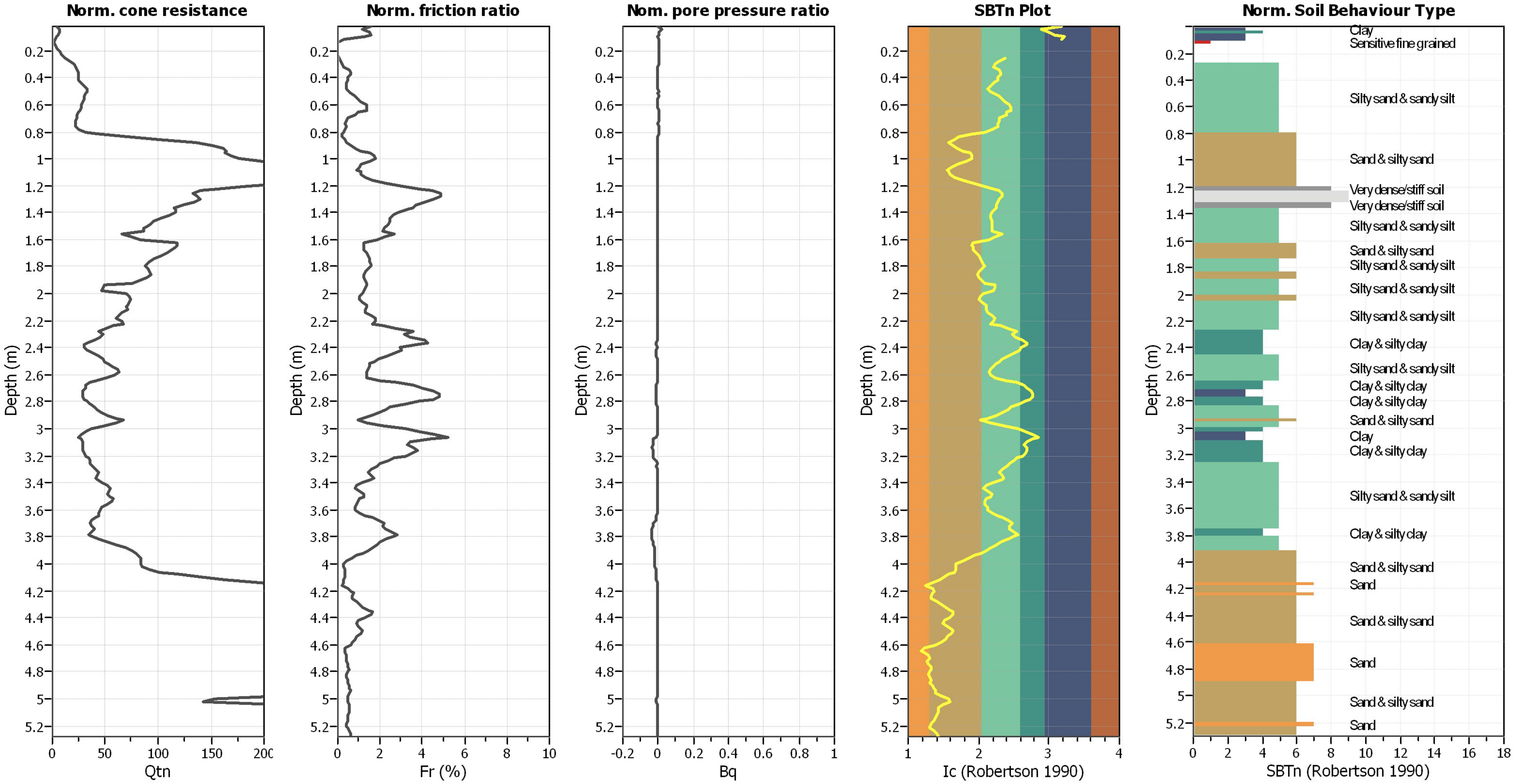






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.50
Peak ground acceleration: 0.53
Depth to water table (insitu): 1.00 m

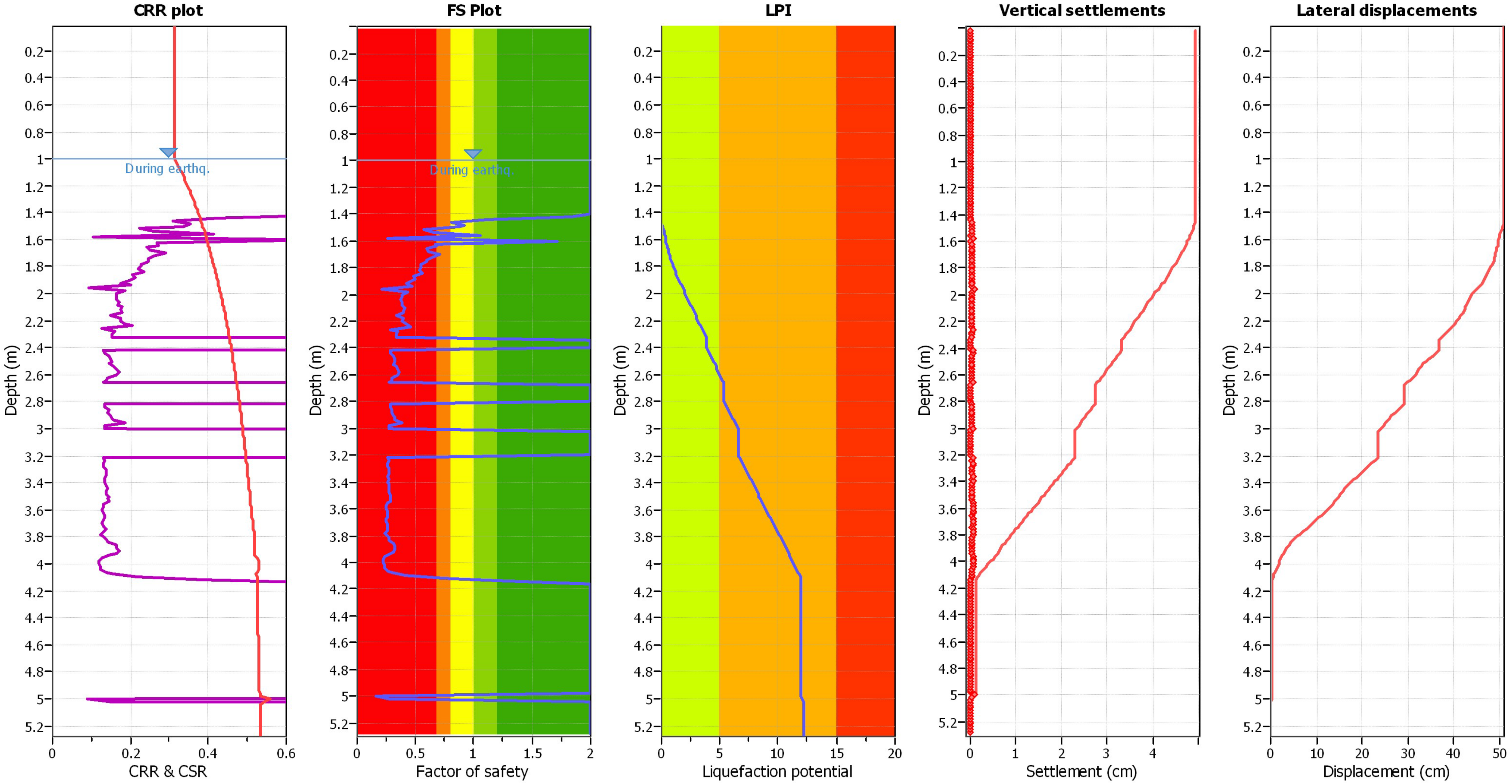
Depth to GWT (erthq.): 1.00 m
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: No
K_σ applied: Yes
Clay like behavior applied: Sand & Clay
Limit depth applied: No
Limit depth: N/A

SBTn legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)
Fines correction method:	B&I (2014)
Points to test:	Based on Ic value
Earthquake magnitude M_w:	7.50
Peak ground acceleration:	0.53
Depth to water table (insitu):	1.00 m
Depth to GWT (erthq.):	1.00 m
Average results interval:	3
Ic cut-off value:	2.60
Unit weight calculation:	Based on SBT
Use fill:	No
Fill height:	N/A
Fill weight:	N/A
Transition detect. applied:	No
K_σ applied:	Yes
Clay like behavior applied:	Sand & Clay
Limit depth applied:	No
Limit depth:	N/A

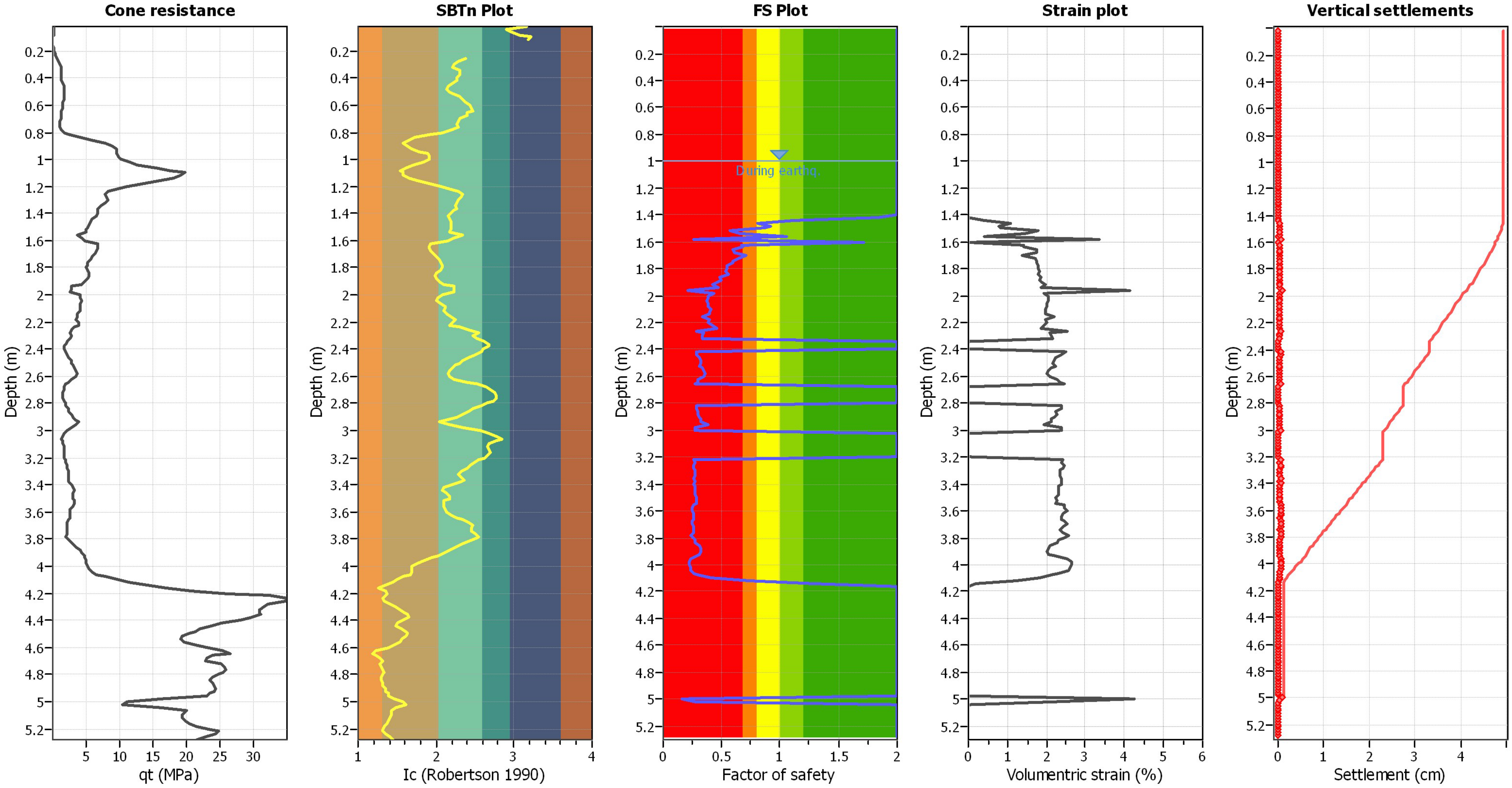
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

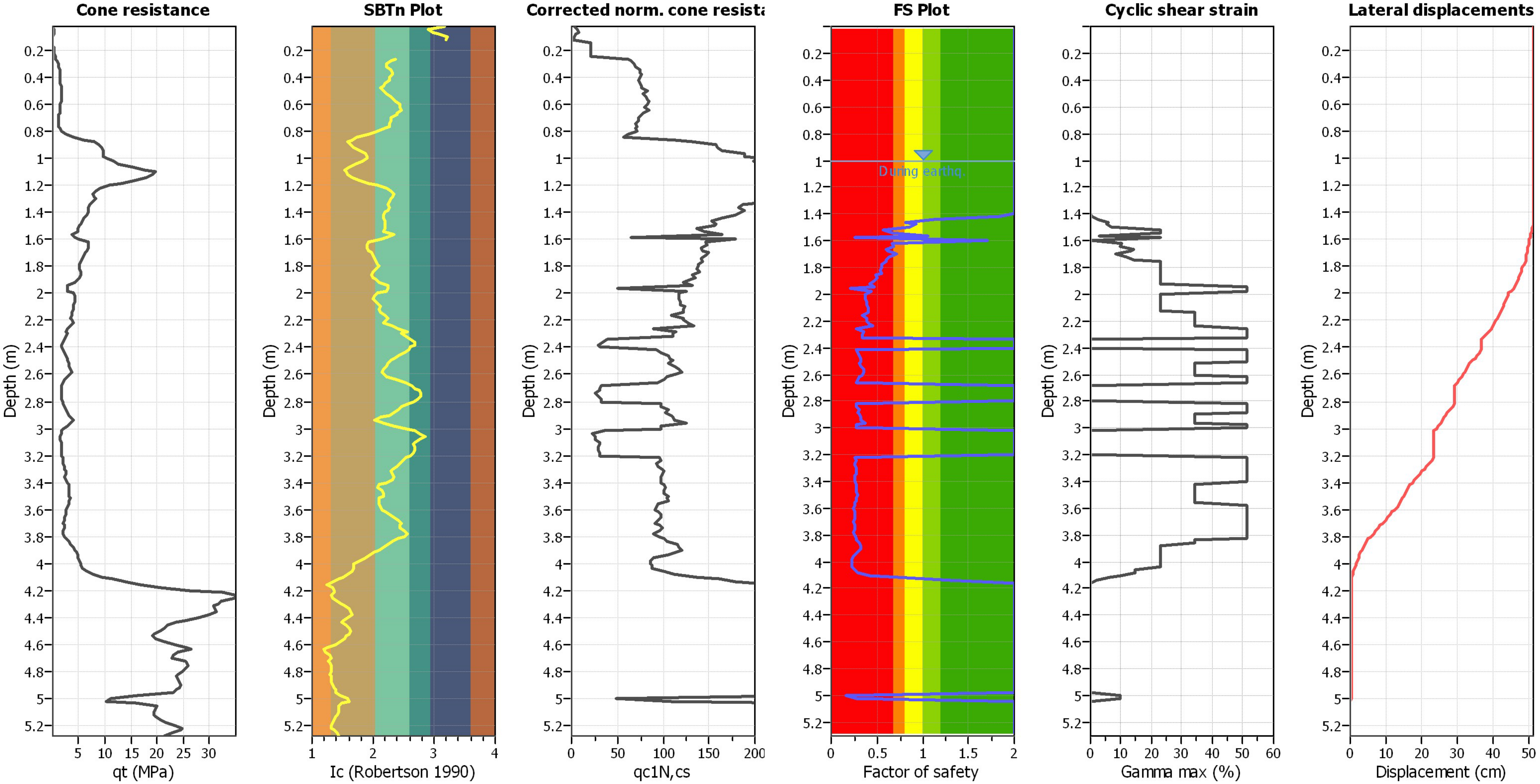
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

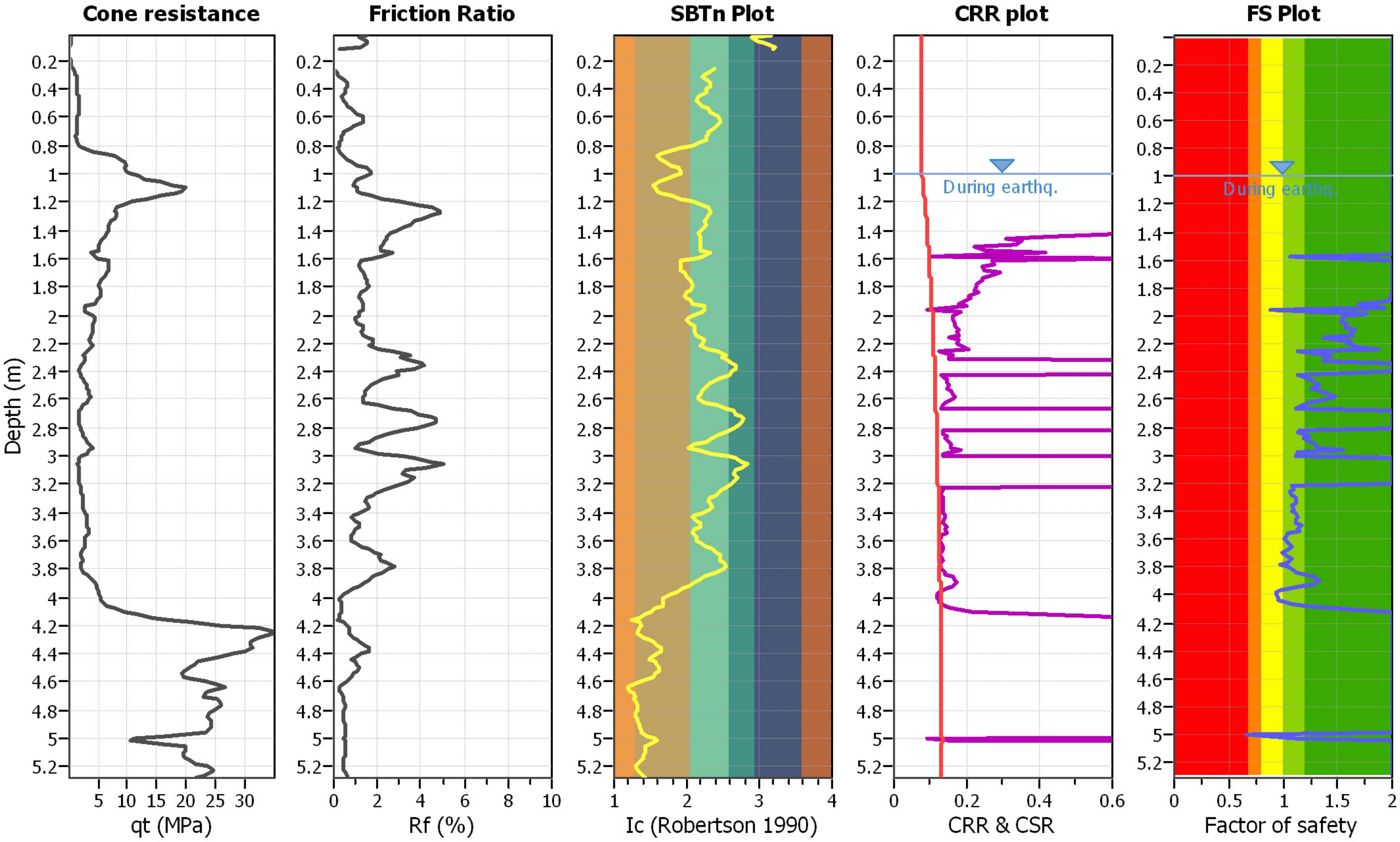
LIQUEFACTION ANALYSIS REPORT

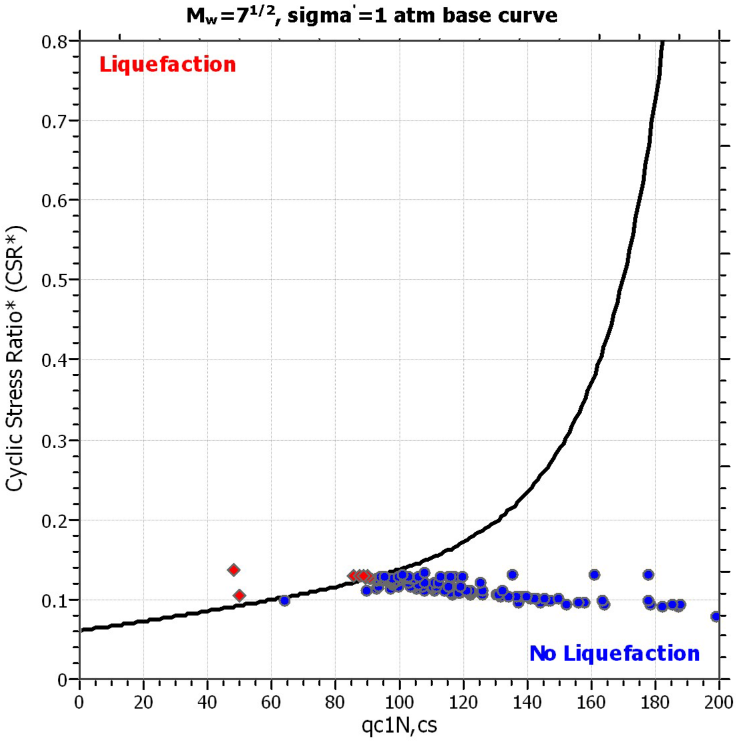
Project title : **Location :**

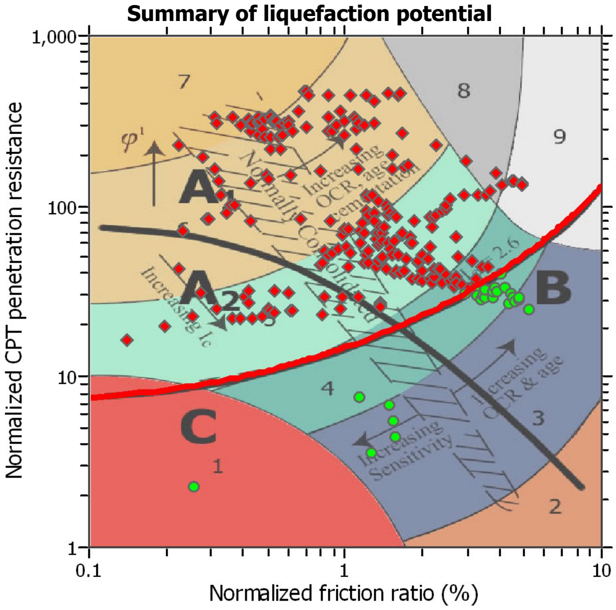
CPT file : Shelly Bay CPT1, SLS

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.00 m	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.13	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

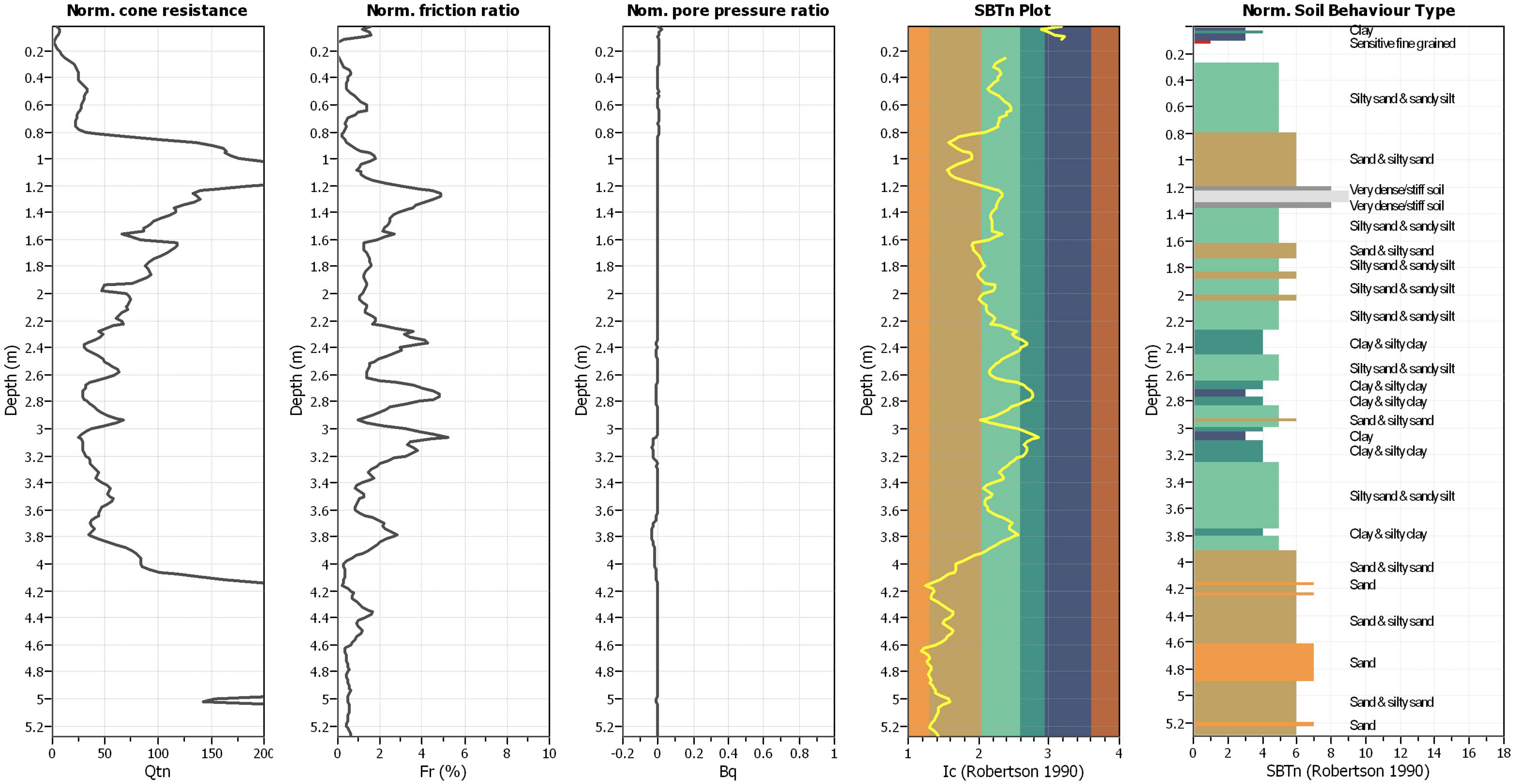






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



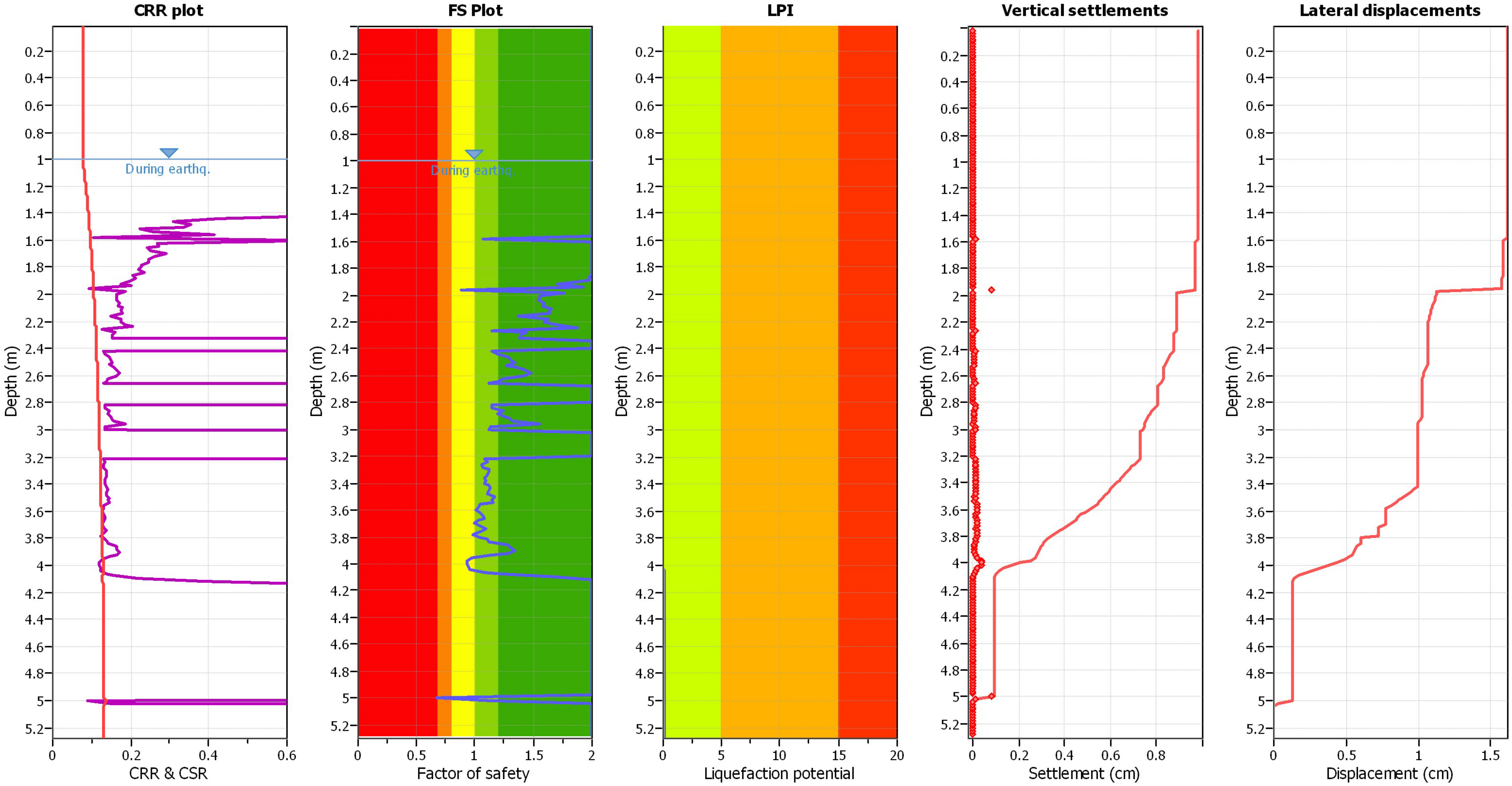
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

SBTn legend

🟥	1. Sensitive fine grained	■	4. Clayey silt to silty	■	7. Gravely sand to sand
🟫	2. Organic material	🟩	5. Silty sand to sandy silt	■	8. Very stiff sand to
🟦	3. Clay to silty clay	🟫	6. Clean sand to silty sand	⬜	9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	1.00 m	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.13	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 m	Fill height:	N/A	Limit depth:	N/A

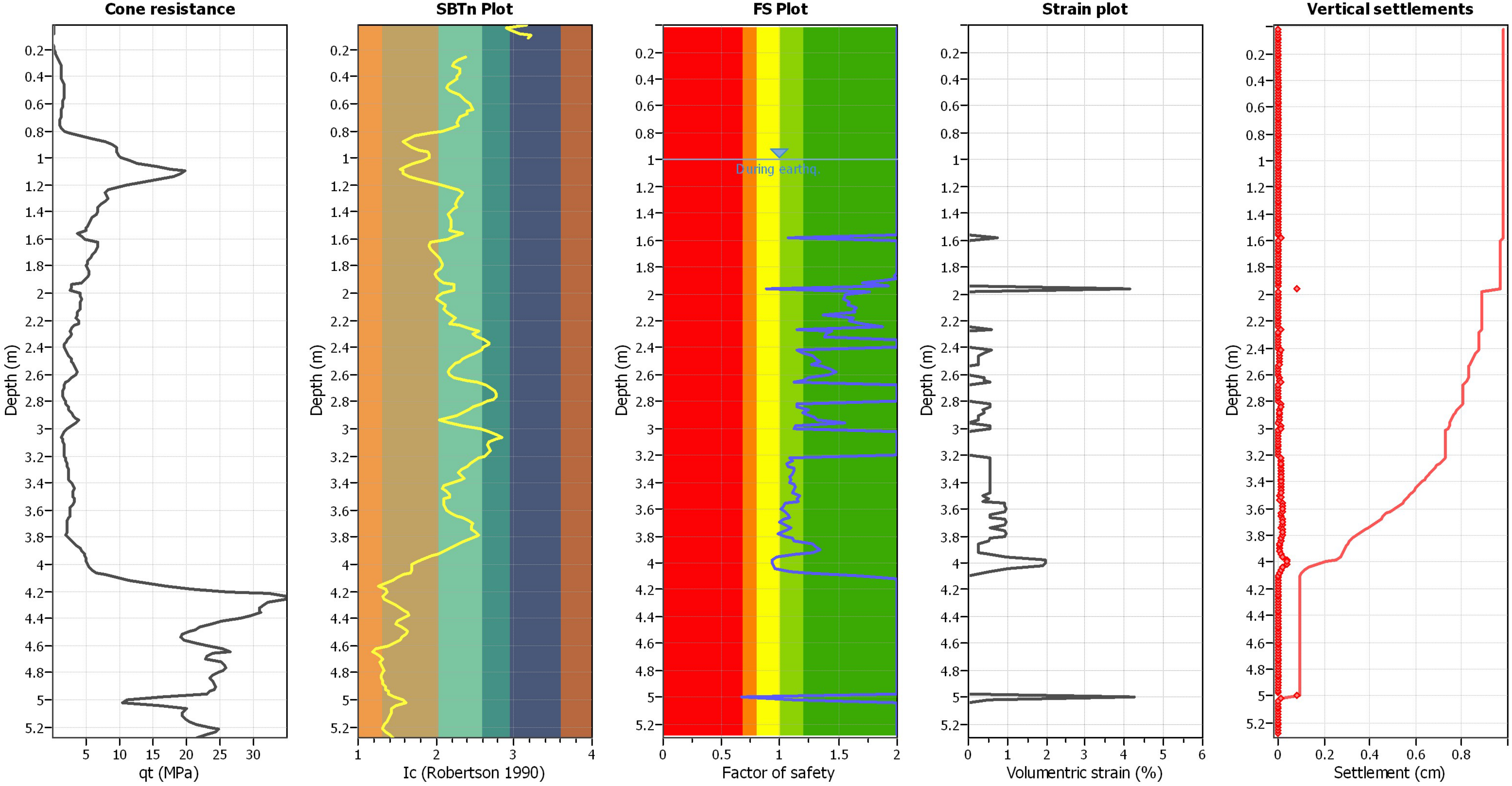
F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

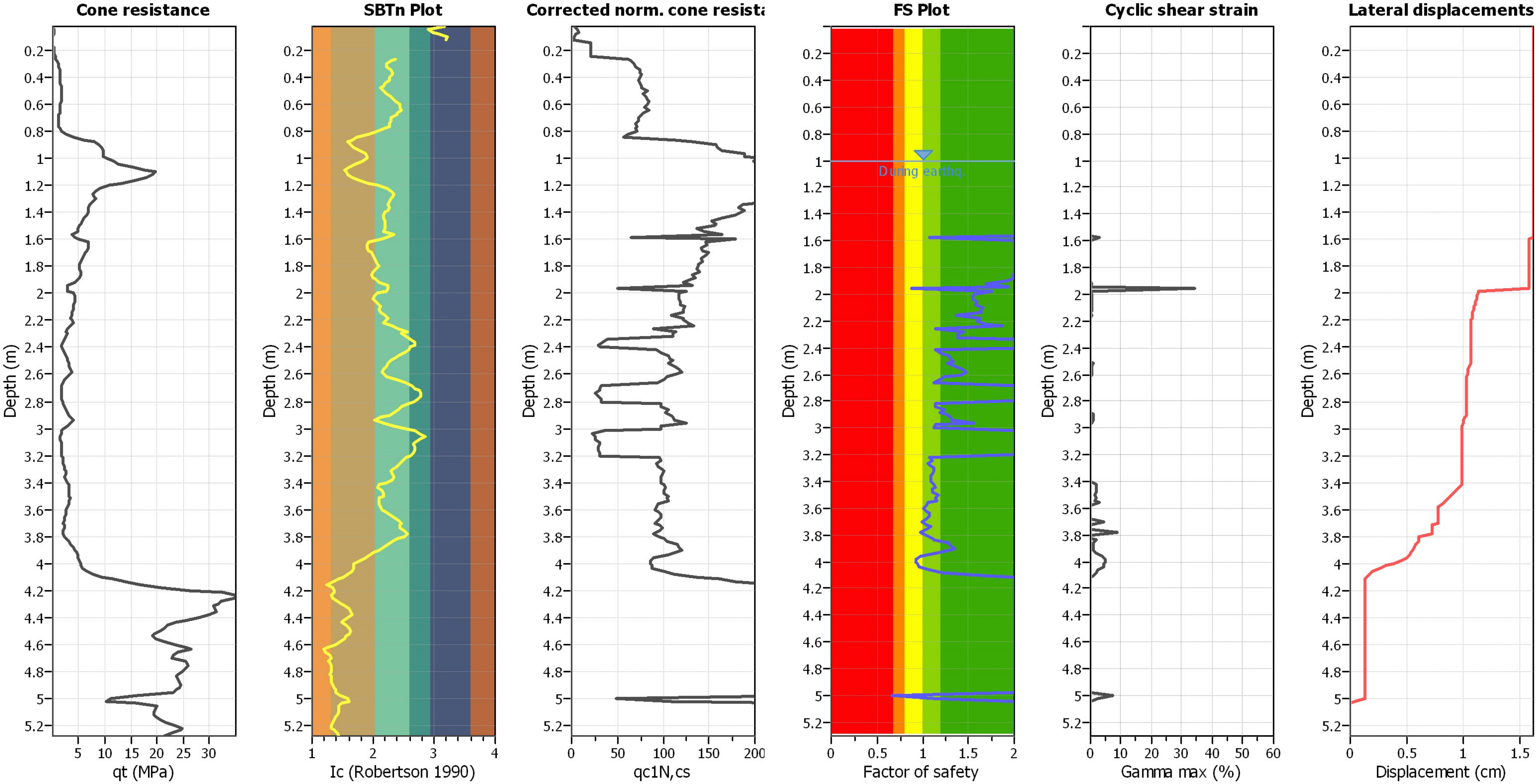
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

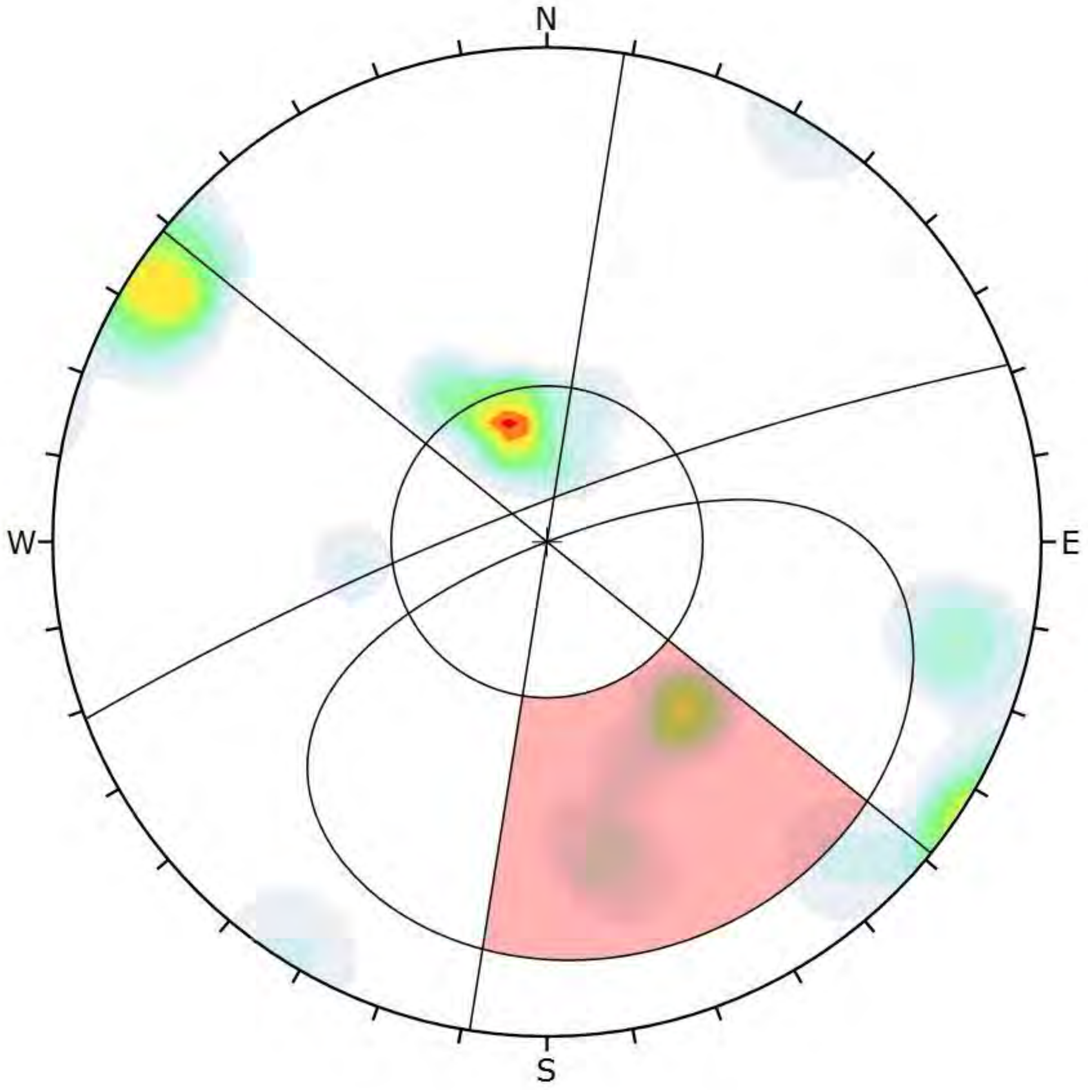
Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index



Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

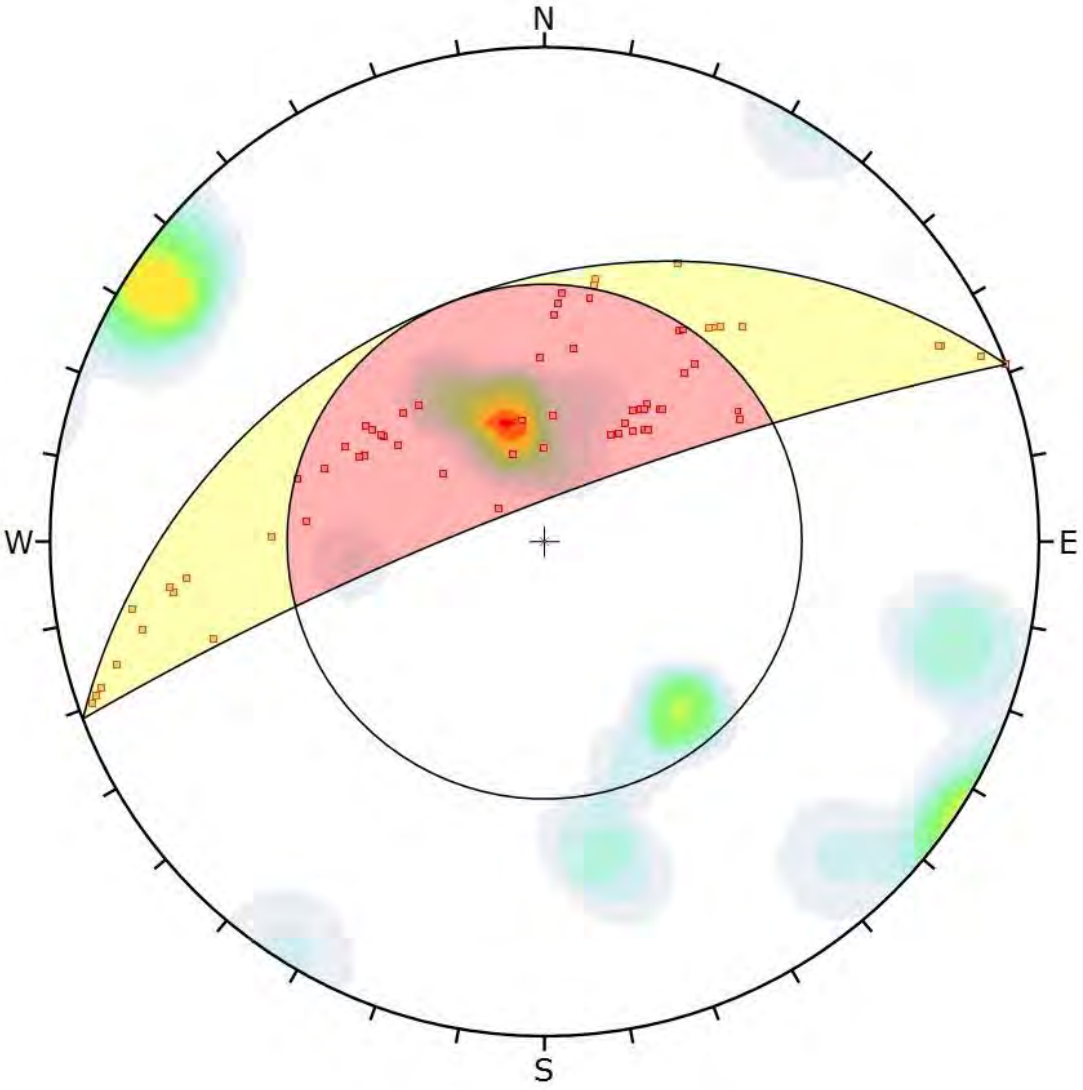
Maximum Density	18.00%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding
Slope Dip	81
Slope Dip Direction	339
Friction Angle	35°
Lateral Limits	30°

	Critical	Total	%
Planar Sliding (All)	6	25	24.00%

Plot Mode	Pole Vectors
Vector Count	25 (25 Entries)
Hemisphere	Lower
Projection	Equal Angle

Slope 1
Planar Sliding



Symbol	Feature
□	Critical Intersection

Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

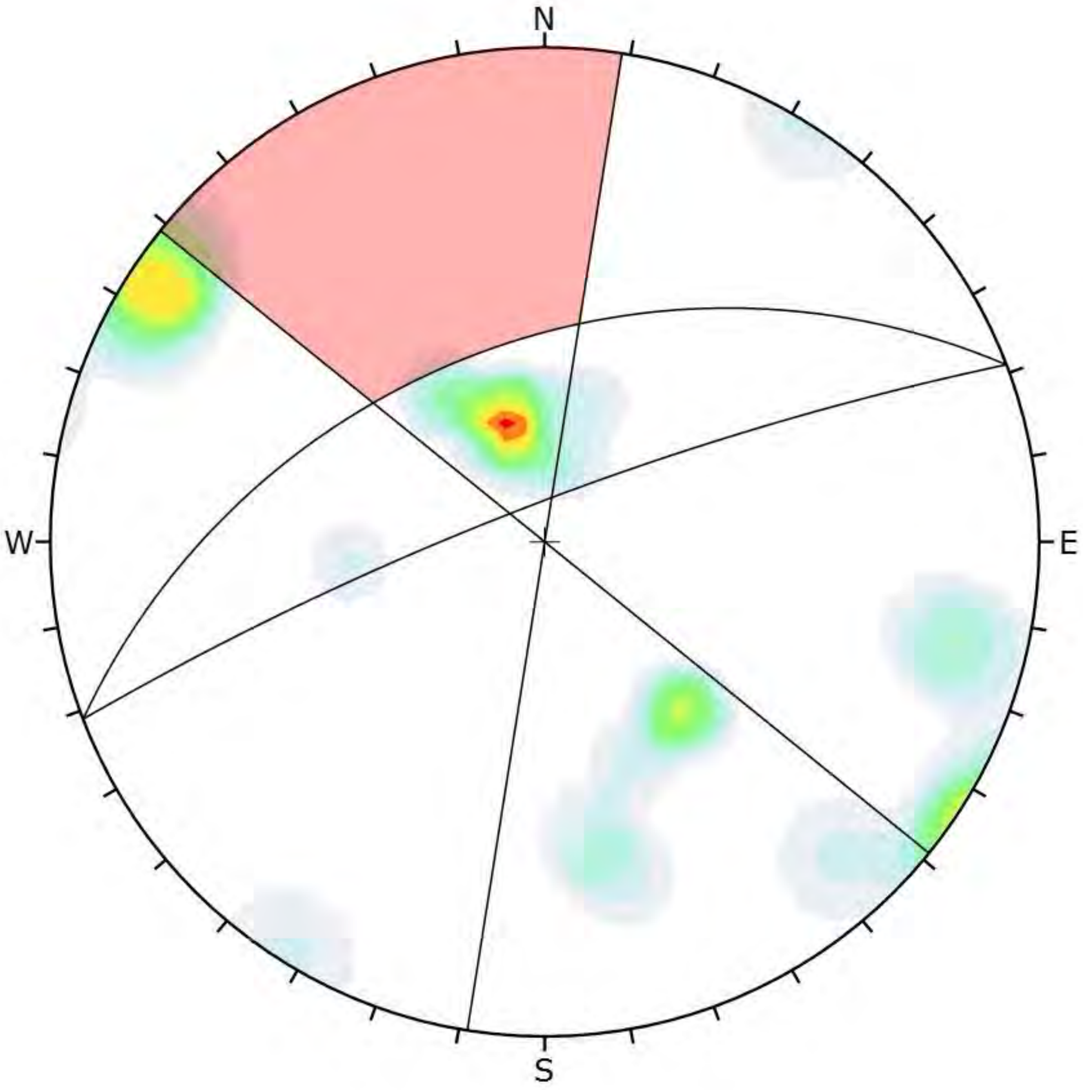
Maximum Density	18.00%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Wedge Sliding
Slope Dip	81
Slope Dip Direction	339
Friction Angle	35°

	Critical	Total	%
Wedge Sliding	67	300	22.33%

Plot Mode	Pole Vectors
Vector Count	25 (25 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	300
Hemisphere	Lower
Projection	Equal Angle

Slope 1
Wedge Sliding



Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

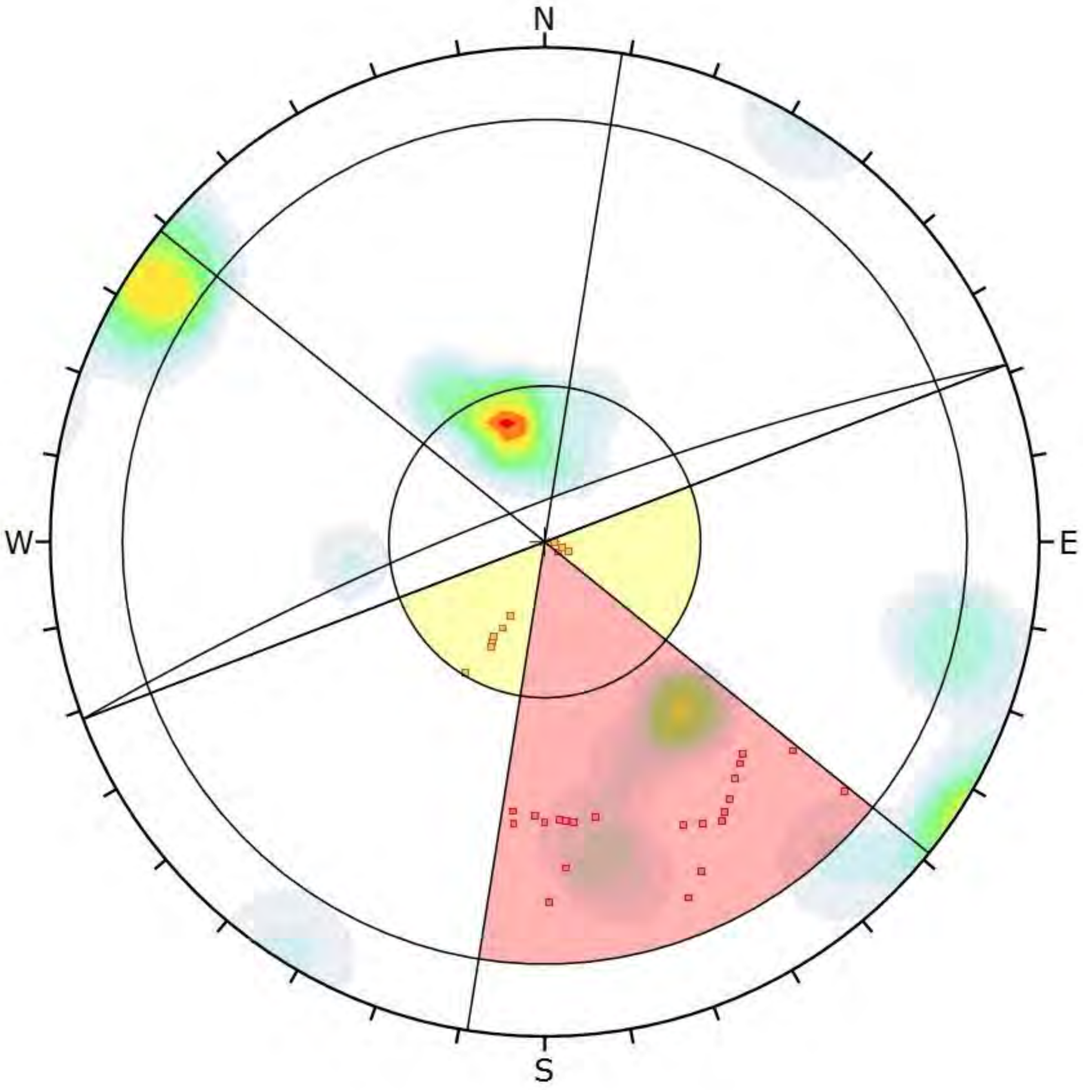
Maximum Density	18.00%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Flexural Toppling
Slope Dip	81
Slope Dip Direction	339
Friction Angle	35°
Lateral Limits	30°

	Critical	**Total**	**%**
Flexural Toppling (All)	0	25	0.00%

Plot Mode	Pole Vectors
Vector Count	25 (25 Entries)
Hemisphere	Lower
Projection	Equal Angle

Slope 1
Flexural Toppling



Symbol	Feature
▫	Critical Intersection

Color	Density Concentrations
	0.00 - 1.90
🟦	1.90 - 3.80
🟦	3.80 - 5.70
🟩	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟨	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

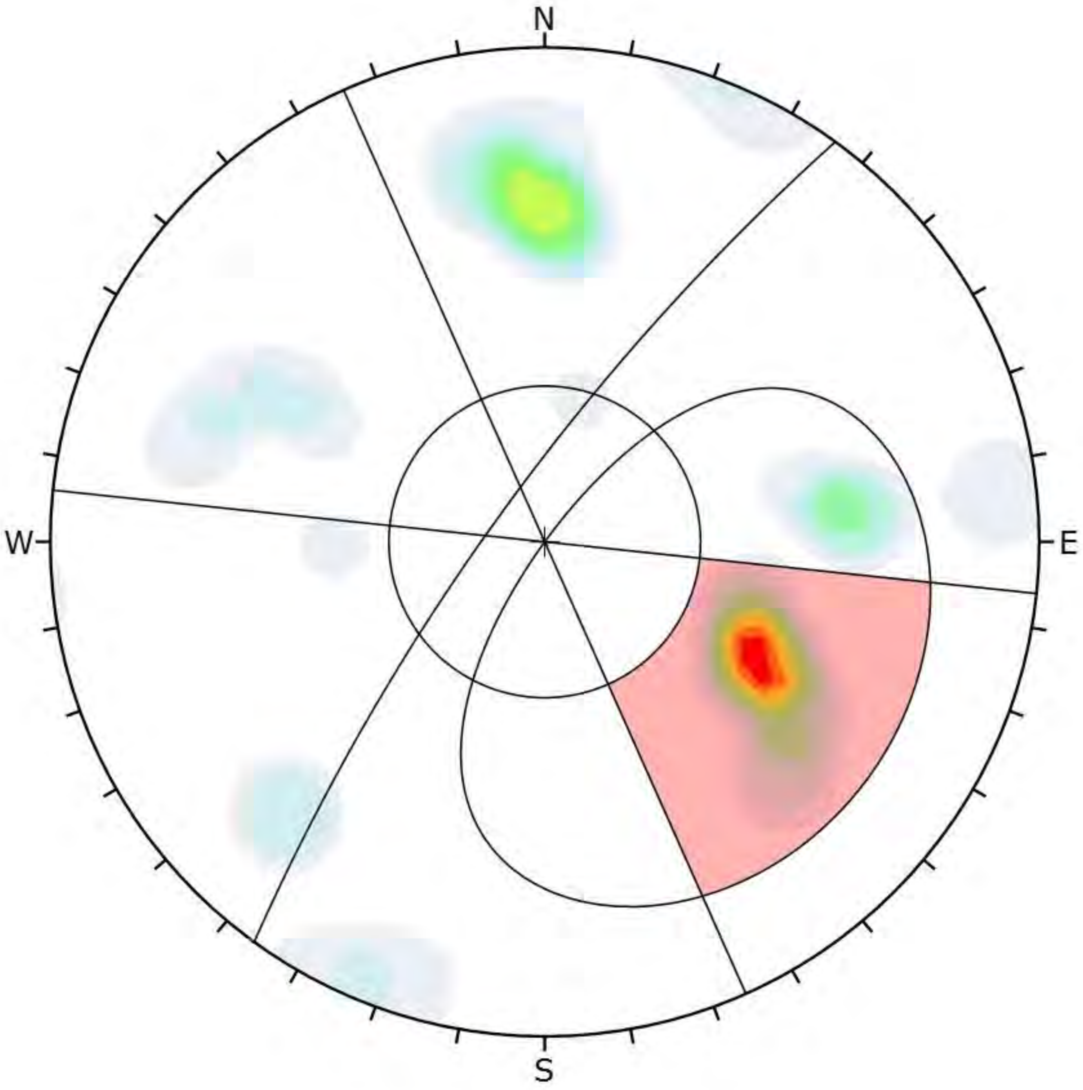
Maximum Density	18.00%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	81
Slope Dip Direction	339
Friction Angle	35°
Lateral Limits	30°

	Critical	Total	%
Direct Toppling (Intersection)	22	300	7.33%
Oblique Toppling (Intersection)	11	300	3.67%
Base Plane (All)	6	25	24.00%

Plot Mode	Pole Vectors
Vector Count	25 (25 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	300
Hemisphere	Lower
Projection	Equal Angle

Slope 1
Direct Toppling



Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

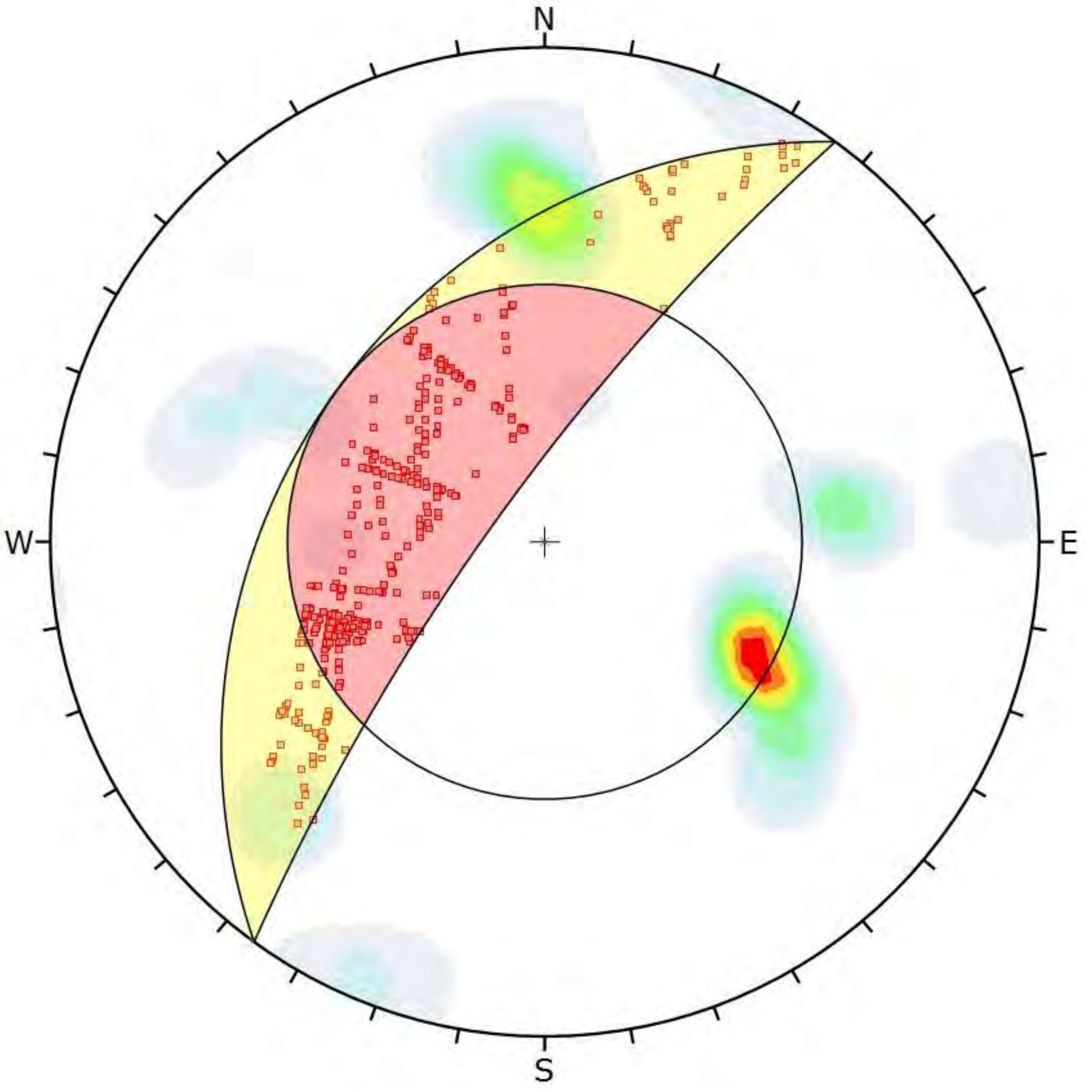
Maximum Density	18.41%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding
Slope Dip	78
Slope Dip Direction	306
Friction Angle	35°
Lateral Limits	30°

	Critical	**Total**	**%**
Planar Sliding (All)	14	37	37.84%

Plot Mode	Pole Vectors
Vector Count	37 (37 Entries)
Hemisphere	Lower
Projection	Equal Angle

Slope 5, Face 1
Planar Sliding



Symbol	Feature
□	Critical Intersection

Color	Density Concentrations
	0.00 - 1.90
🟦	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

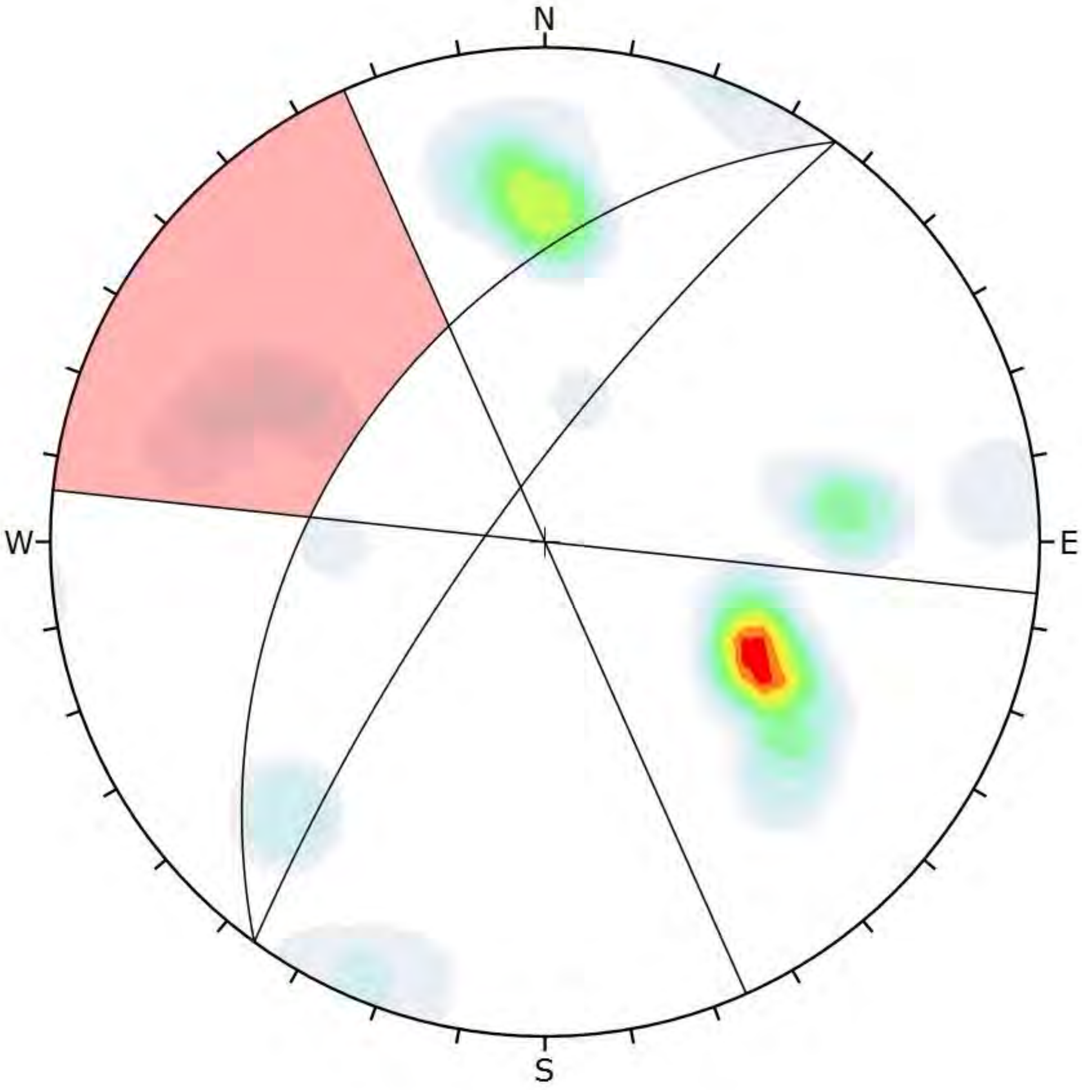
Maximum Density	18.41%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Wedge Sliding
Slope Dip	78
Slope Dip Direction	306
Friction Angle	35°

	Critical	Total	%
Wedge Sliding	394	666	59.16%

Plot Mode	Pole Vectors
Vector Count	37 (37 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	666
Hemisphere	Lower
Projection	Equal Angle

Slope 5, Face 1
Wedge Sliding



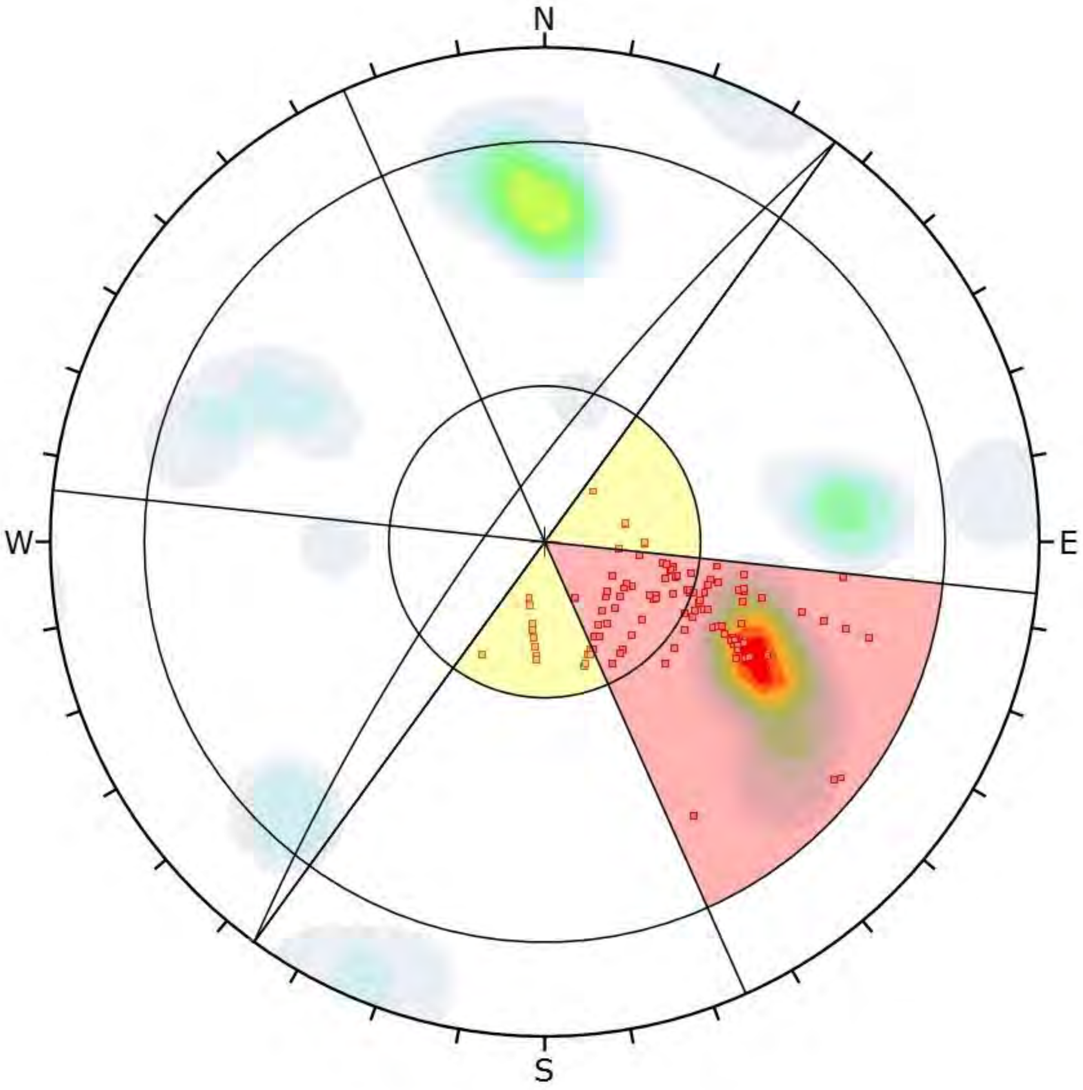
Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

Maximum Density	18.41%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Flexural Toppling		
Slope Dip	78		
Slope Dip Direction	306		
Friction Angle	35°		
Lateral Limits	30°		
	Critical	**Total**	**%**
Flexural Toppling (All)	4	37	10.81%

Plot Mode	Pole Vectors
Vector Count	37 (37 Entries)
Hemisphere	Lower
Projection	Equal Angle

Slope 5, Face 1
Flexural Toppling



Symbol	Feature
▫	Critical Intersection

Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

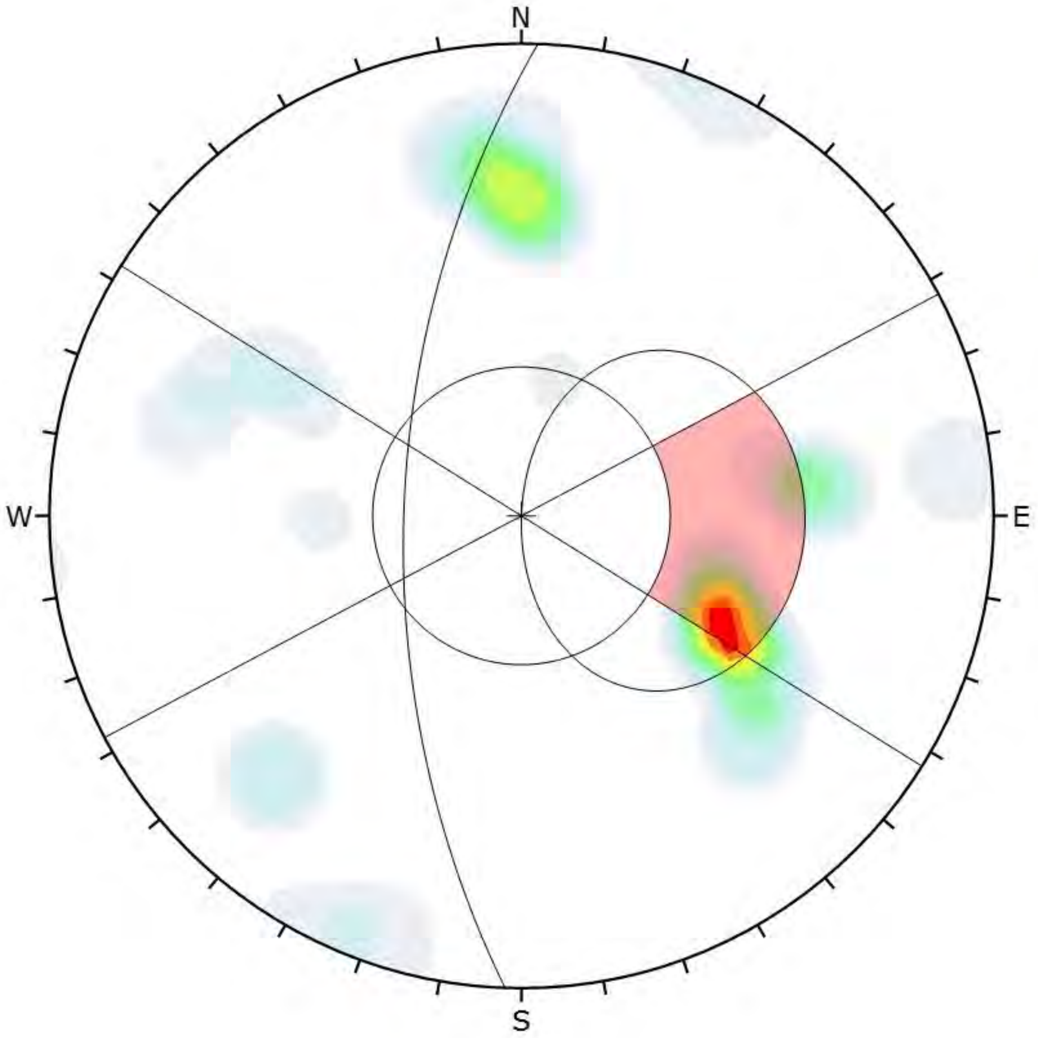
Maximum Density	18.41%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	78
Slope Dip Direction	306
Friction Angle	35°
Lateral Limits	30°

	Critical	Total	%
Direct Toppling (Intersection)	92	666	13.81%
Oblique Toppling (Intersection)	21	666	3.15%
Base Plane (All)	14	37	37.84%

Plot Mode	Pole Vectors
Vector Count	37 (37 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	666
Hemisphere	Lower
Projection	Equal Angle

Slope 5, Face 1
Direct Toppling



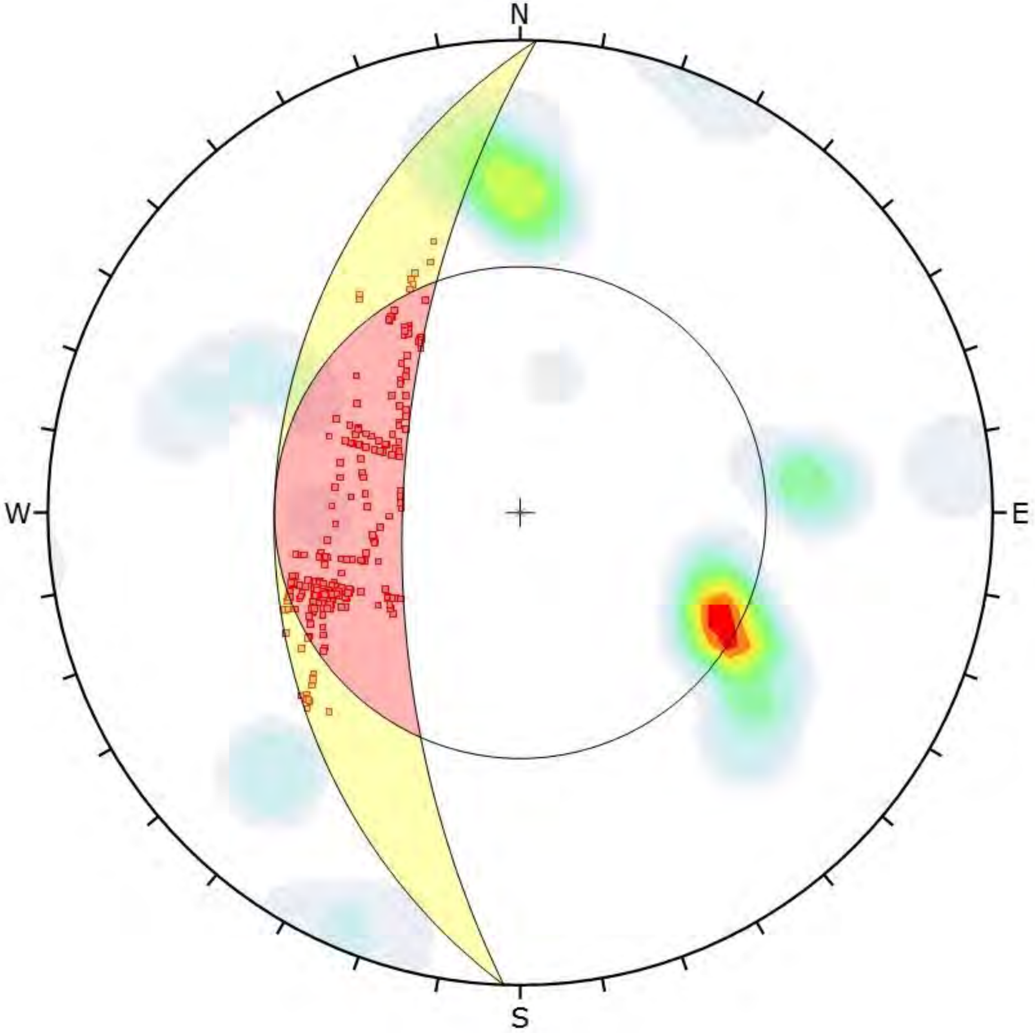
Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

Maximum Density	18.41%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding			
Slope Dip	62			
Slope Dip Direction	272			
Friction Angle	35°			
Lateral Limits	30°			
		Critical	**Total**	**%**
Planar Sliding (All)		9	37	24.32%

Plot Mode	Pole Vectors
Vector Count	37 (37 Entries)
Hemisphere	Lower
Projection	Equal Angle

Slope 5, Face 2
Planar Sliding



Symbol	Feature
□	Critical Intersection

Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

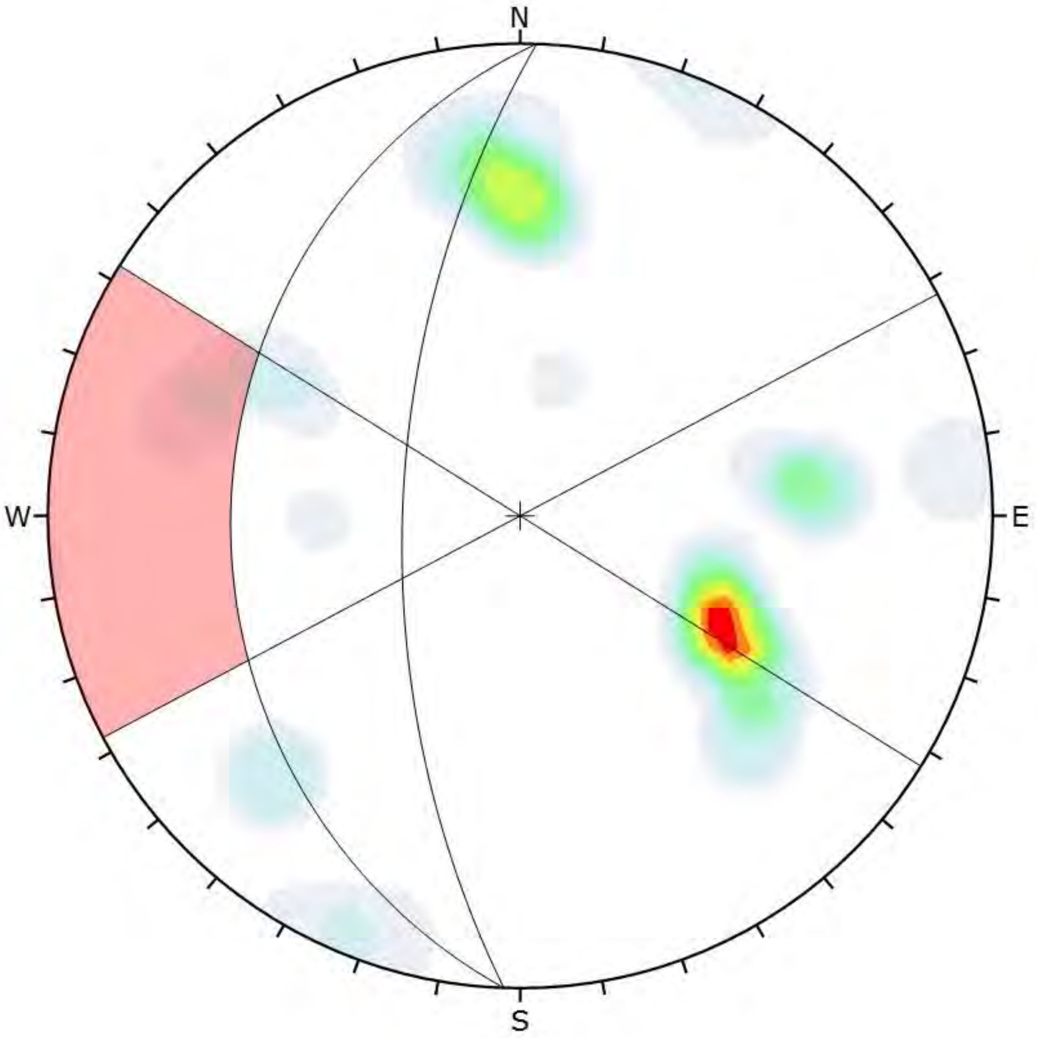
Maximum Density	18.41%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Wedge Sliding
Slope Dip	62
Slope Dip Direction	272
Friction Angle	35°

	Critical	Total	%
Wedge Sliding	270	666	40.54%

Plot Mode	Pole Vectors
Vector Count	37 (37 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	666
Hemisphere	Lower
Projection	Equal Angle

Slope 5, Face 2
Wedge Sliding



Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

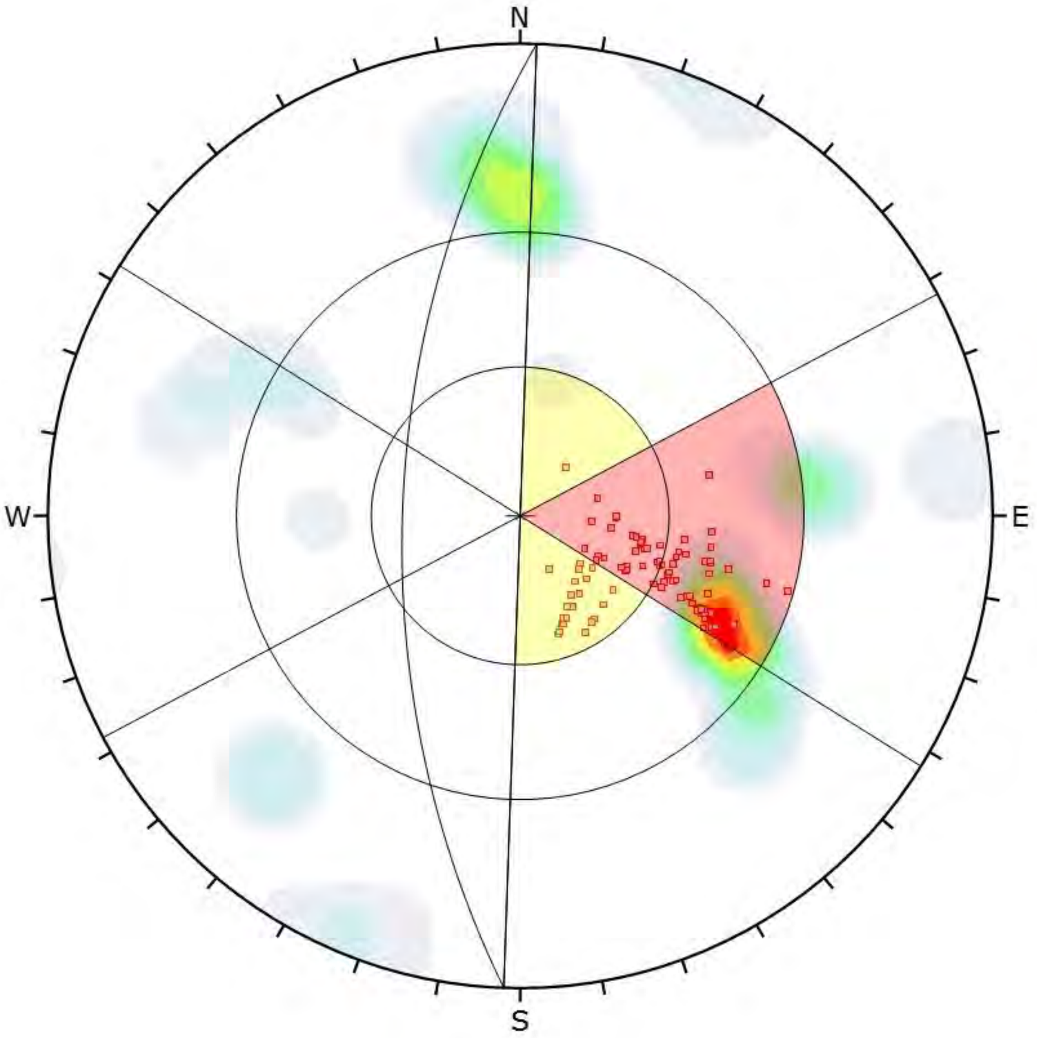
Maximum Density	18.41%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Flexural Toppling
Slope Dip	62
Slope Dip Direction	272
Friction Angle	35°
Lateral Limits	30°

	Critical	Total	%
Flexural Toppling (All)	2	37	5.41%

Plot Mode	Pole Vectors
Vector Count	37 (37 Entries)
Hemisphere	Lower
Projection	Equal Angle

Slope 5, Face 2
Flexural Toppling



Symbol	Feature
□	Critical Intersection

Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

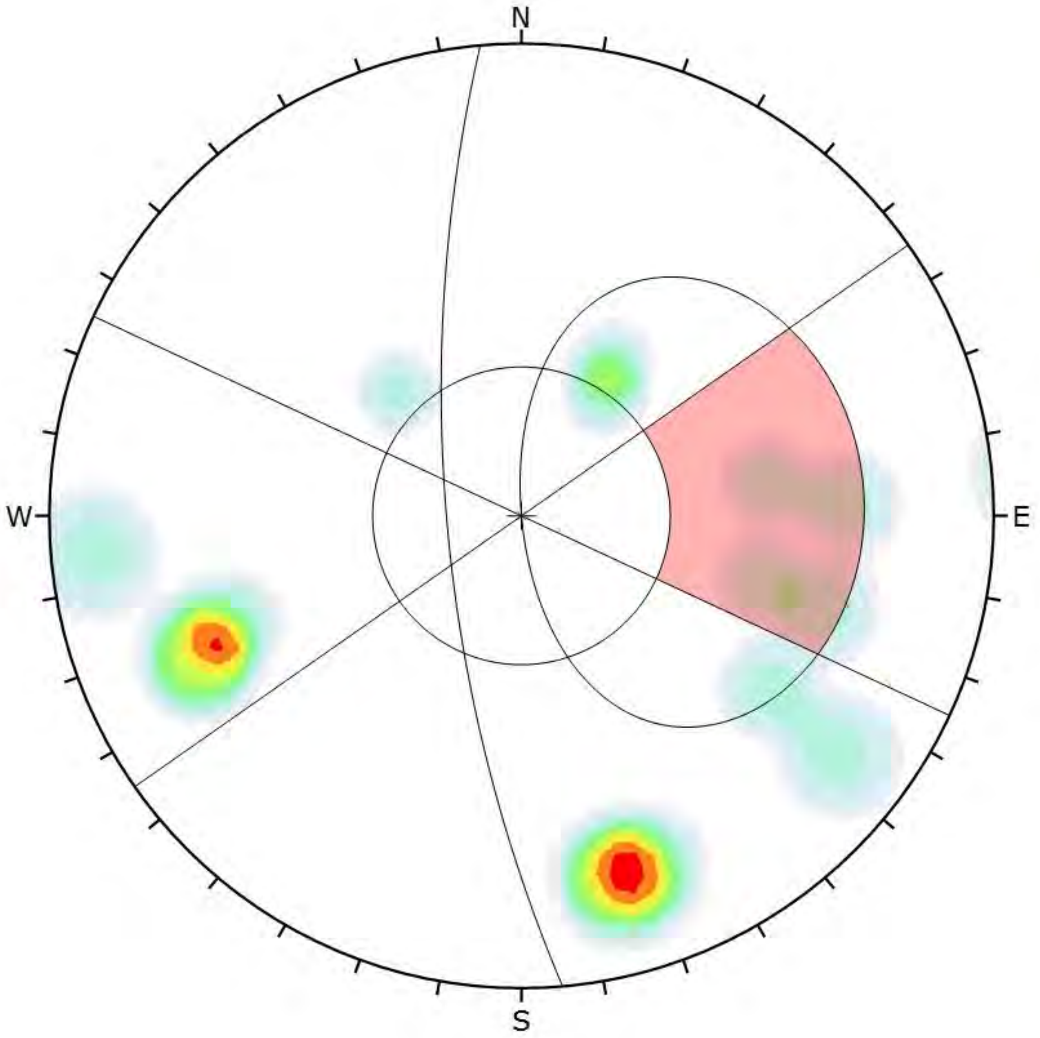
Maximum Density	18.41%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	62
Slope Dip Direction	272
Friction Angle	35°
Lateral Limits	30°

	Critical	Total	%
Direct Toppling (Intersection)	72	666	10.81%
Oblique Toppling (Intersection)	23	666	3.45%
Base Plane (All)	11	37	29.73%

Plot Mode	Pole Vectors
Vector Count	37 (37 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	666
Hemisphere	Lower
Projection	Equal Angle

Slope 5, Face 2
Direct Toppling



Color	Density Concentrations
⬜	0.00 - 1.90
🟦	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

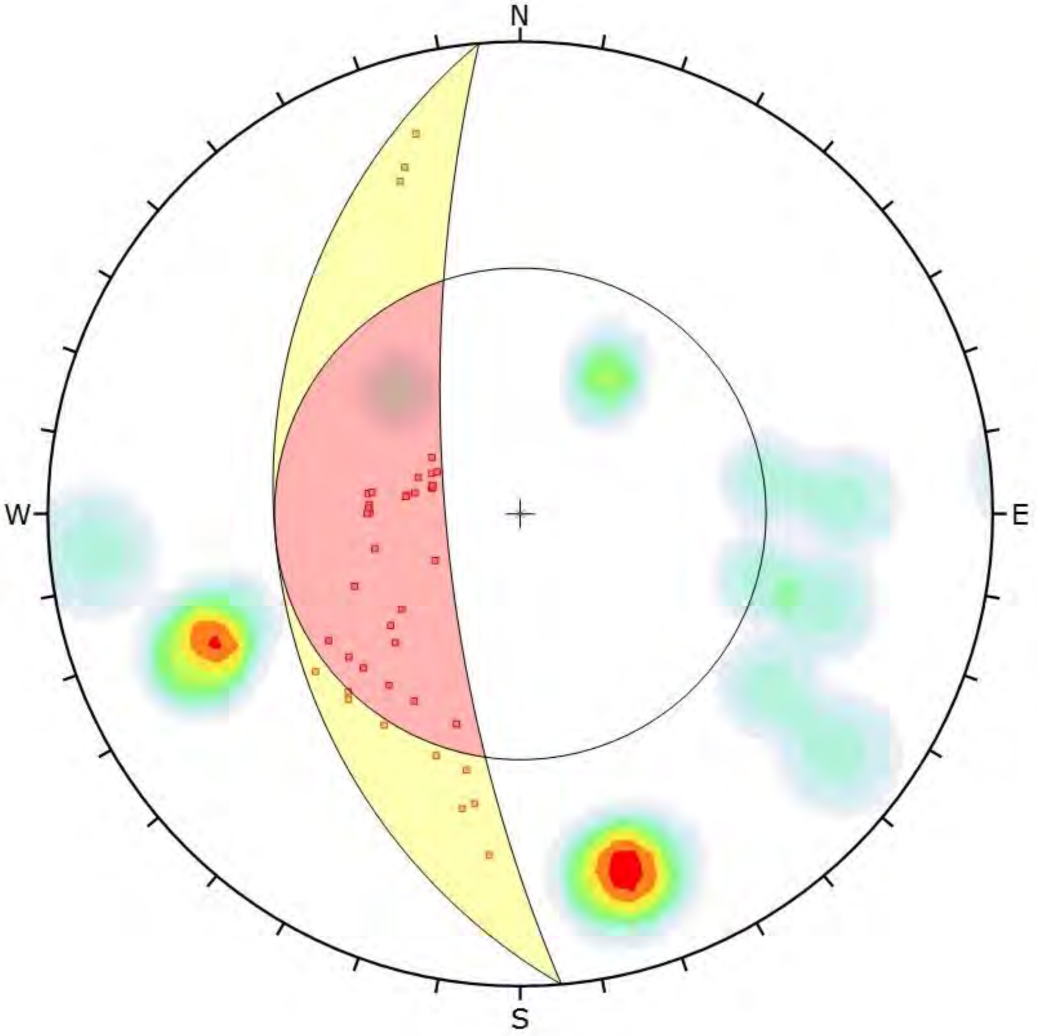
Maximum Density	18.01%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding
Slope Dip	72
Slope Dip Direction	265
Friction Angle	35°
Lateral Limits	30°

	Critical	**Total**	**%**
Planar Sliding (All)	4	16	25.00%

Plot Mode	Pole Vectors
Vector Count	16 (16 Entries)
Hemisphere	Lower
Projection	Equal Angle

Slope 7
Planar Sliding



Symbol	Feature
□	Critical Intersection

Color	Density Concentrations
⬜	0.00 - 1.90
⬜	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

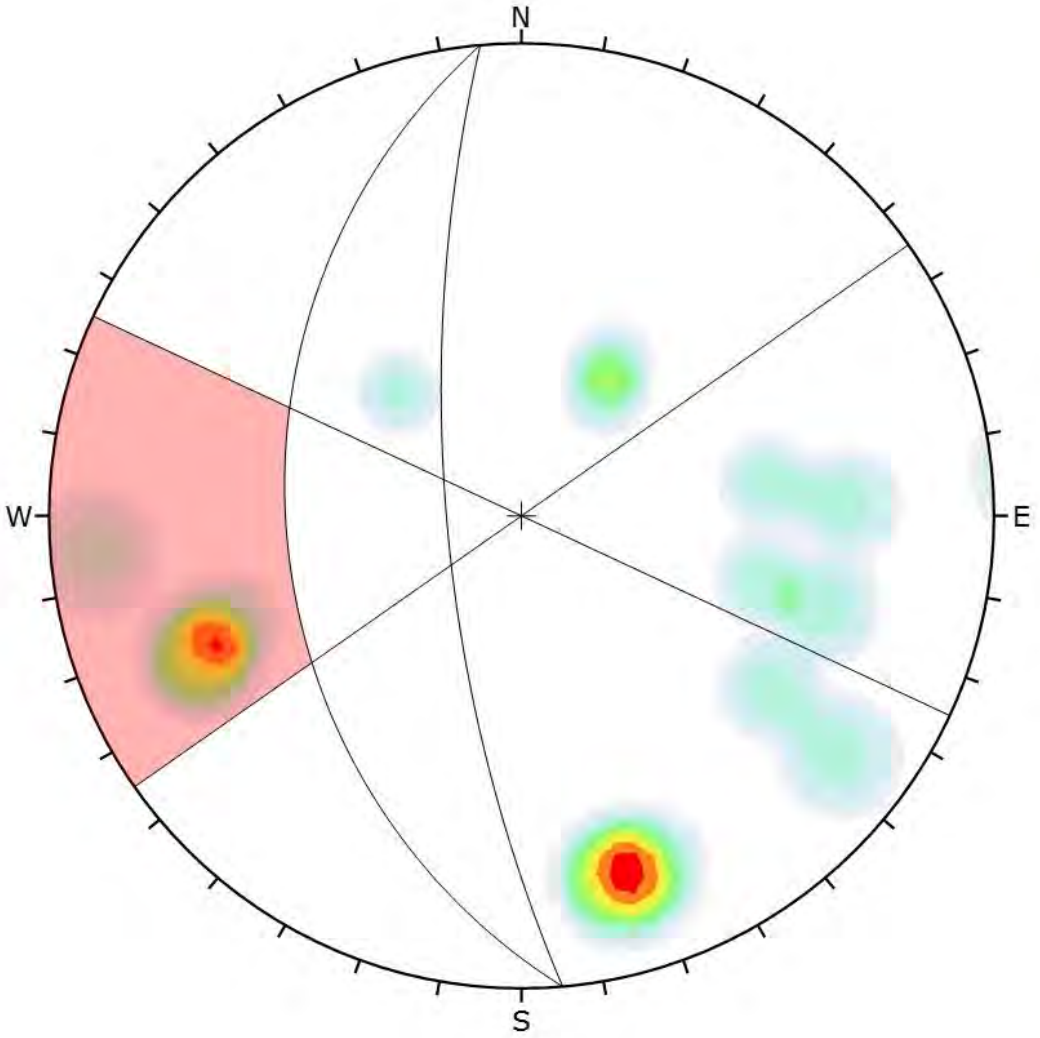
Maximum Density	18.01%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Wedge Sliding
Slope Dip	72
Slope Dip Direction	265
Friction Angle	35°

	Critical	Total	%
Wedge Sliding	43	120	35.83%

Plot Mode	Pole Vectors
Vector Count	16 (16 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	120
Hemisphere	Lower
Projection	Equal Angle

Slope 7
Wedge Sliding



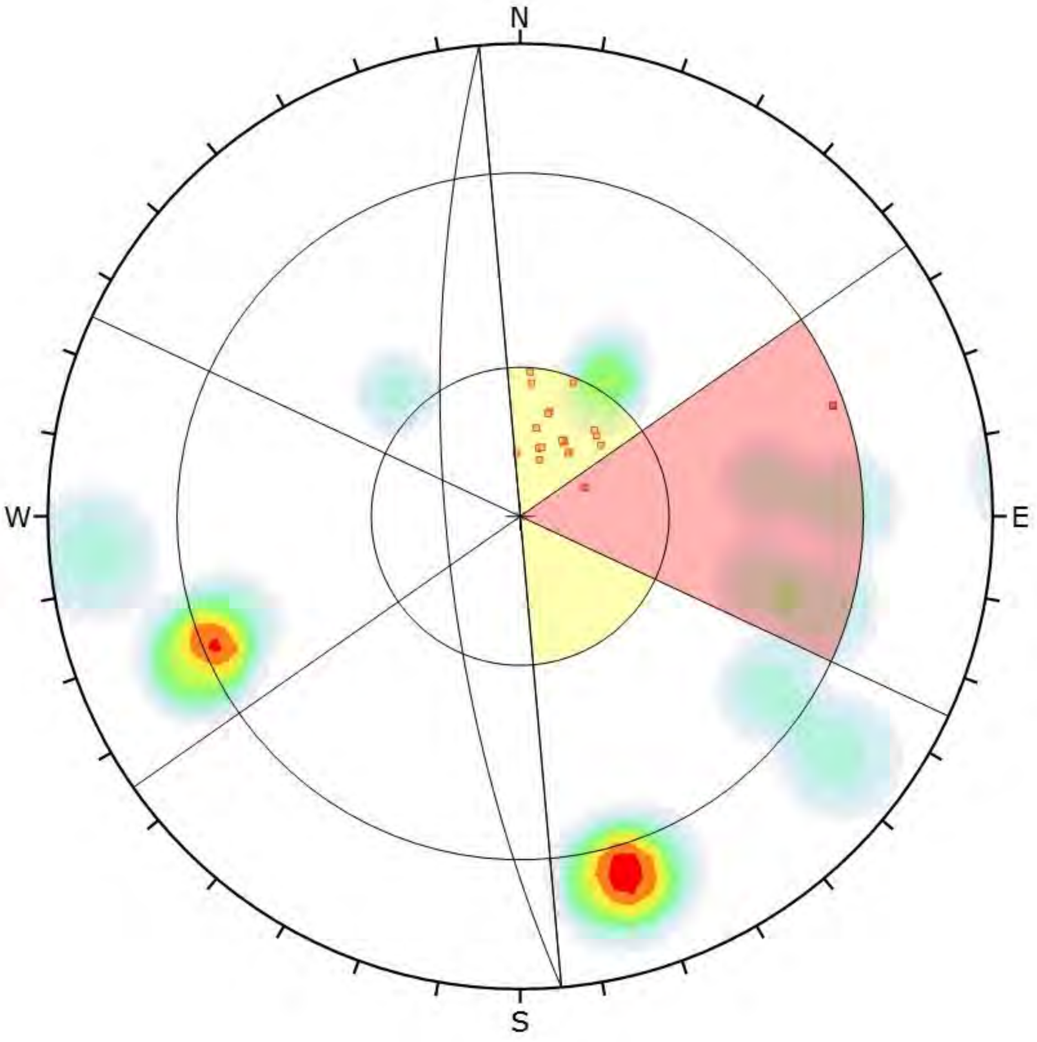
Color	Density Concentrations
⬜	0.00 - 1.90
🟦	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

Maximum Density	18.01%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Flexural Toppling		
Slope Dip	72		
Slope Dip Direction	265		
Friction Angle	35°		
Lateral Limits	30°		
	Critical	**Total**	**%**
Flexural Toppling (All)	4	16	25.00%

Plot Mode	Pole Vectors
Vector Count	16 (16 Entries)
Hemisphere	Lower
Projection	Equal Angle

Slope 7
Flexural Toppling



Symbol	Feature
□	Critical Intersection

Color	Density Concentrations
⬜	0.00 - 1.90
🟦	1.90 - 3.80
🟦	3.80 - 5.70
🟦	5.70 - 7.60
🟩	7.60 - 9.50
🟩	9.50 - 11.40
🟩	11.40 - 13.30
🟨	13.30 - 15.20
🟧	15.20 - 17.10
🟥	17.10 - 19.00

Maximum Density	18.01%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	72
Slope Dip Direction	265
Friction Angle	35°
Lateral Limits	30°

	Critical	Total	%
Direct Toppling (Intersection)	3	120	2.50%
Oblique Toppling (Intersection)	20	120	16.67%
Base Plane (All)	5	16	31.25%

Plot Mode	Pole Vectors
Vector Count	16 (16 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	120
Hemisphere	Lower
Projection	Equal Angle

Slope 7
Direct Toppling

Appendix G

Risk Assessment Methodology

Measures of Likelihood

Level	Descriptor	Description	Annual Probability of Occurrence
A	Almost Certain	The event is on-going, or is expected to occur during the next year	100%
B	Very Likely	The event is expected to occur.	20% to 100%
C	Likely	The event is expected to occur under somewhat adverse conditions	5% to 20%
D	Possible	The event is expected to occur under adverse conditions	1 to 5%
E	Unlikely	The event is expected to occur under high to extreme conditions	0.2 to 1%
F	Rare	The event could occur under extreme conditions	Less than 0.2%

Measures of Consequence

Level	Descriptor	Example Descriptions (Damage to Private Property)
1	Catastrophic	Large scale damage to multiple properties
2	Disastrous	Large scale damage involving private property and dwelling requiring major engineering works for stabilisation
3	Major	Extensive damage to property but dwelling not involved
4	Medium	Moderate damage to private land
5	Low	Limited damage to private land
6	Minor	No damage

Risk Matrix

		Consequences to Property/Assets					
		1: Catastrophic	2: Disastrous	3: Major	4: Medium	5: Low	6: Minor
Likelihood	A – Almost Certain	VH	VH	VH	H	H	M
	B – Very Likely	VH	VH	H	H	M	L
	C – Likely	VH	H	H	M	L	L
	D – Possible	VH	H	M	L	VL-L	VL
	E – Unlikely	H	M	L	VL	VL	VL
	F –Rare	M	L	VL	VL	VL	VL

Risk Level Implications

Risk Level		Implications for Risk Management
VH	Very High Risk	Detailed investigation, design, planning and implementation of treatment options to reduce risk to acceptable levels: May involve very high costs.
H	High Risk	Detailed investigation, design, planning and implementation of treatment options to reduce risk to acceptable levels.
M	Moderate Risk	Broadly tolerable provided treatment plan is implemented to maintain or reduce risks, May require investigation and planning of treatment options.
L	Low Risk	Acceptable. Treatment requirements to be defined to maintain or reduce risk
VL	Very Low Risk	Acceptable. Manage by normal maintenance procedures