17 February 2023

PROPOSED RESIDENTIAL SUBDIVISION

TEI TEI DRIVE, OHAKUNE

GEOTECHNICAL INTERPRETATIVE REPORT

Cheal Consultant Ltd

TGA2022-0238AC Rev 0

TGA2022-0238AC		
Date	**Revision**	**Comments**
8 February 2023	A	Initial draft for internal review
17 February 2023	0	Final issued to client

	Name	**Signature**	**Position**
Prepared by	Ellis Evens		Project Engineering Geologist
Reviewed by	Kirstin Brown		Senior Geotechnical Engineer
Authorised by	Dave Morton		Principal Geotechnical Engineer CMEngNZ, CPEng

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Appendices

1 INTRODUCTION

CMW Geosciences (CMW) was engaged by Cheal Consultant Ltd (Cheal), on behalf of Kāinga Ora, to carry out a geotechnical investigation of a site located at the southern end of Tei Tei Drive, Ohakune, which is being considered for a residential development comprising approximately 145 to 160 residential allotments with associated accessways and reserve areas.

The scope of work and associated terms and conditions of our engagement were detailed in our services proposal letter referenced TGA2022-0238AA Rev 1, dated 29 August 2022 and Kainga Ora's Form of Agreement for Consultancy Services referenced AR110717, dated October 2022.

The purpose of this report is to advance the existing Preliminary Feasibility Assessment Report prepared by Cheal[1]. This Geotechnical Interpretative Report describes the ground conditions encountered, presents the results of a geohazards assessment and provides mitigation strategies and geotechnical recommendations with respect to the proposed development.

The results of the geotechnical investigation were provided in the CMW Geotechnical Factual Report dated 17 February 2023 (ref: TGA2022-0238AB Rev 1).

2 SITE DESCRIPTION

2.1 Site Location

The site comprises an area of approximately 10Ha and is legally described as Lot 2 DP 54909. It is located south of the Ohakune township as shown on Figure 1 below.

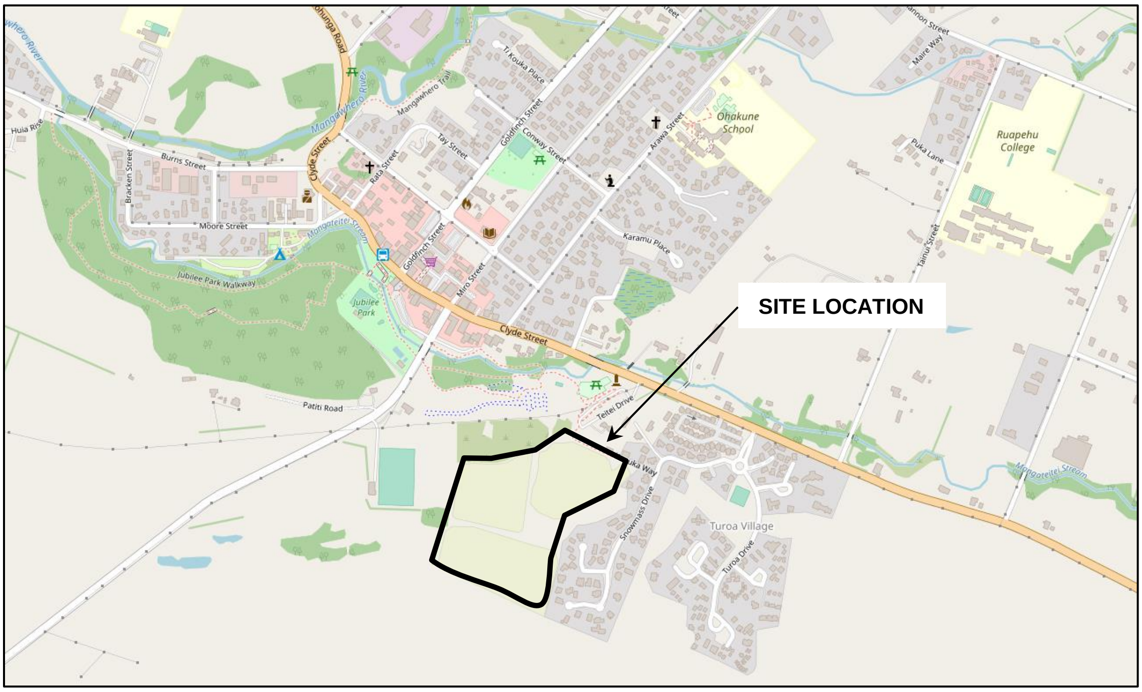


Figure 1: Site Location Plan (Openstreetmap.org)

[1] Cheal report titled 'Preliminary Feasibility Assessment for Tei Tei Drive, Ohakune (ref. 200901L2, dated 18 December 2020)

2.2 Landform

The current general landform, together with associated features located within and adjacent to the site is presented on the attached Geotechnical Investigation Plan (***Drawing 01)***.

The site comprises gently sloping topography, with elevations ranging from RL 593m (Tuhirangi Circuit 2000) in the northeast corner to the lowest portion of the site, located centrally along the western boundary, at RL 587m. The site is bound by a residential housing area to the east, Rochfort Park to the west, a reserve to the north and farmland to the south. The site can be accessed via Tei Tei Drive to the north.

Open drains run east to west through the centre of the site, and north to south along the western boundary, both of which discharge into low lying area within Rochfort Park to the west. Areas of swampy ground were observed across some of the low-lying portions of the site, specifically, centrally along the eastern boundary and in the north-western part of the site, as shown on ***Drawing 01***.

3 HISTORIC AERIAL PHOTOGRAPHS

A review of available historic aerial photographs[4] show that in 1943 (earliest available image) the site comprised grassed farmland. At this time, an east to west orientated channel ran across the centre of the site. A shallow, north-east to south-west orientated gully, encroaches onto the site from western site boundary. Two north-south orientated shallow vegetated channels were also present during this time, extending from the southern boundary towards the centre of the site.

By 1967 one of the shallow vegetated channels in the southern part of the site had been infilled and much of the site re-contoured. An open drain had been formed in place of the east-west orientated channel. Earthwork had occurred in the north-western corner of the site, with a channel infilled.

By 2002, the second north-south orientated channel had also been infilled.

Between 2010 and 2014 the north-east to south-west orientated gully had been infilled. The site has remained unchanged since this time and present day.

4 PROPOSED DEVELOPMENT

At the time of undertaking this investigation and of writing this report the project was in the early stages of planning and it is anticipated that this report would assist in the development of a scheme plan for the site.

The Kāinga Ora Concept Drawings[5] provided (***Appendix A***), present two preliminary scheme plan options for the proposed development. Option one comprises 145 residential allotments and includes the construction of swales to control stormwater runoff within the development. Option two comprises 157 residential allotments and relies solely on converting the existing east-west orientated open drain into a stormwater swale to help control stormwater runoff.

Specific details for the disposal of concentrated stormwater are yet to be confirmed with Cheal currently assessing options for the site, however we understand it is likely to utilise the existing open drains on the site. We understand wastewater will connect to the existing council reticulation.

We have prepared this report on the basis that the future development will broadly comprise minor cuts and fills in the order of 1.0 metre, to form a near level building platforms.

[4] Retrolens website, sourced from http://retrolens.nz and licensed by LINZ CC-BY 3.0
[5] KOHC Concept Drawings, dated 21 September 2022

5 FIELD INVESTIGATION

5.1 Previous Investigation

Cheal previously completed a geotechnical investigation for the site with the results presented in the Preliminary Feasibility Assessment Report. The geotechnical investigation was undertaken in December 2020 and comprised the following:

- Thirteen test pits to depths of between 2.2m and 3.8m;
- Four hand auger boreholes to depths of between 0.9m and 1.1m.

The approximate locations of the respective investigation sites referred to above are shown on the Geotechnical Investigation Plan as ***Drawing 01.***

5.2 Recent Investigation

Following a dial before you dig search, and onsite service location, the field investigation was carried out on October and December 2022. All fieldwork was carried out under the direction of CMW Geosciences in general accordance with the NZGS specifications[7] and logged in accordance with NZGS guidance[8]. The scope of fieldwork completed was as follows:

- A site walkover to assess the general landform and site conditions;
- Six machine boreholes, denoted MBH01 to MBH06, were drilled to depths of up to 15m (or refusal) to visual observe the ground profile across the site;
- Seven Cone Penetrometer Tests (CPT), denoted CPT01 to CPT07, were pushed to refusal depths of up to 2.8m. The CPTs terminated prior to target depths due to equipment refusal, attributed to the presence of boulders in the soil profile;
- Eight test pits, denoted TP01 to TP08, were excavated using a 12-tonne hydraulic excavator fitted with a 1.4m wide toothed rock bucket to depths of between 1.5m and 3.2m below existing ground levels. TP01 to TP03, TP05, TP06 and TP08 were terminated due to refusal, while TP04 and TP07 were terminated at their target depth.

The approximate locations of the respective investigation sites referred to above are shown on the Geotechnical Investigation Plan as ***Drawing 01***. Test locations were measured using handheld GPS and elevations were inferred from the Cheal topographic survey plan[9].

The Engineering logs and CPT traces are presented in the CMW Geotechnical Factual Report referenced above.

[7] NZ Geotechnical Society (2017) NZ Ground Investigation Specification, Volume 1 – Master Specification
[8] NZ Geotechnical Society (2005), Field Description of Soil and Rock, Guideline for the field classification and description of soil and rock for engineering purposes.

[9] Drawing titled 'Topographical Survey of Lot 2 DP 54909 (ref. 220528-TP01, Rev A, dated 24 November 2022)

6 GROUND MODEL

6.1 Published Geology

Published geological maps[10] for the area depict the regional geology as comprising "*Undifferentiated late Quaternary river gravel, sand and fan deposits*" and "*Debris-hyperconcentrated flow and fluvial deposits*" of the Waimarino Formation, as illustrated in Figure 2 below.

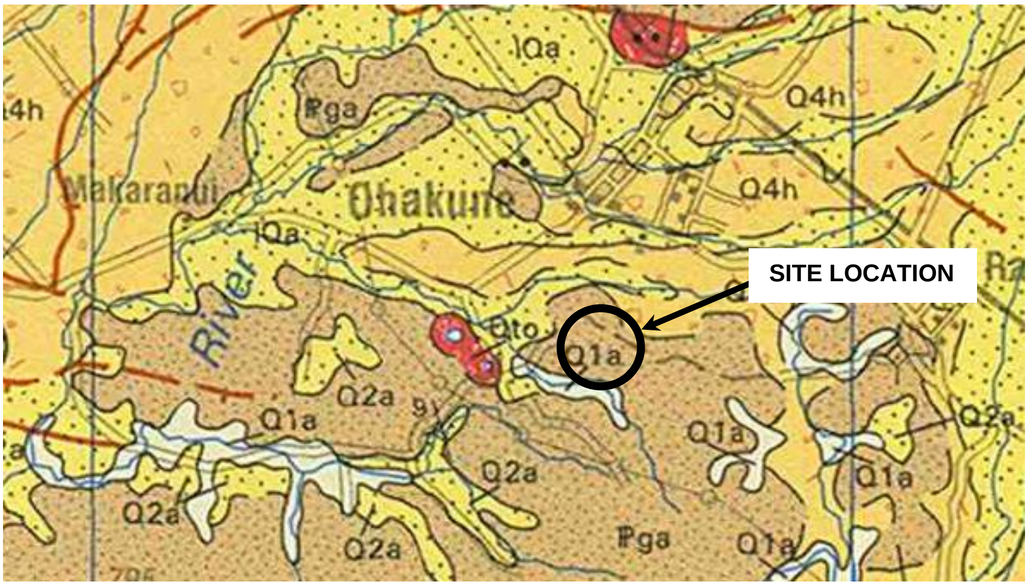


Figure 2: Regional Geology (GNS Geological Map 7)

The published geology may be mantled by volcanic ash deposits. Some superficial depths of fill may be present as a result of previous and existing land use activities.

6.2 Stratigraphic Units

The ground conditions encountered and inferred from the investigation were generally consistent with the published geology for the area and can be generalised according to the following subsurface sequences. The distribution of the various units encountered is presented in the appended Geological Cross Section A and B, on ***Drawing 03 and 04***.

6.2.1 Topsoil

Topsoil, was encountered at all investigation locations to depths of between 0.1m and 0.4m below ground level (bgl).

6.2.2 Existing Fill

Uncontrolled fill associated with the infilling of former gullies and channels, as described in Section 3 above, was encountered beneath the topsoil in Cheal TP13, HA15, HA16 & HA17 and CMW TP02. This material comprised brown, soft to stiff silt, locally containing organic fibres. Soil strengths are based on field assessments as no undrained shear strengths were recorded in this material.

This uncontrolled fill was encountered to depths of between 0.3m and 0.6m (up to 0.5m thick). The approximate extent (based on review of historic aerial photographs and site investigation data) of areas of the site where uncontrolled fill is likely to be encountered is shown on ***Drawing 02***.

[10] Leonard, Begg, Wilson (2010), Geology of the Taranaki Area, GNS 1:250 000, Geological Map 7

6.2.3 Volcanic Ash Deposits

Volcanic Ash deposits comprising orange, brown stiff to very stiff clayey silts were encountered beneath the topsoil and mantle the site. These soils extend to depths of between 0.5m and 1.9m below existing ground level (bgl). Undrained shear strength values range from 66kPa to >200kPa (the upper limit of the device), but are generally above 120kPa.

6.2.4 Fan Deposits

Fan deposits underlie the Volcanic Ash deposits and comprise of the following units:

- Medium dense to dense gravelly sands interbedded with stiff to very stiff clayey silts and sandy silts were encountered from immediately beneath the Volcanic Ash layer across the northern and eastern parts of the site, and extended to depth of up to 11.5mbgl. SPT N values range from 13 to 40.
- Medium dense to dense sandy gravels and gravelly sands underlie the Volcanic Ash layer in the south-western part of the site and underlies the above unit in the north-eastern part of the site. These soils were encountered to depths of up to approximately 10mbgl. SPT N values range from 8 to greater than 50 but are generally above 30.
- A layer of stiff to hard sandy silt underlies the above units in the western part of the site and was encountered within BH02 in the north-eastern part of the site. It was encountered between depths of approximately 7mbgl and 13.5mbgl. SPT N values recorded in this material are variable and range between 4 and greater than 50.
- Dense to very dense sandy gravels and gravelly sands were encountered below 9m depth in MBH02 to MBH06 and were inferred to extend beyond the termination depth of the boreholes. SPT N values range between 22 and greater than 50 but are generally greater than 50. A basaltic andesite boulder was encountered in MBH01 between 10.3mbgl and 10.5mbgl where the borehole was terminated due to equipment refusal.

6.3 Groundwater

During our investigation, which was completed in late spring and early summer conditions (October-December 2022), groundwater was encountered within the CPTs, test pits and machine boreholes at the depths provided in ***Appendix B***.

The depths recorded are highly variable, ranging from 0.2m to 2.8m below existing ground level. This indicates perched groundwater tables are present, forming as a result of the layered stratigraphy of contrasting permeabilities.

We have assumed the regional groundwater level is approximately 1.5m to 2.0m below existing ground level based on the water levels observed within the open drains running through the site.

We expect that seasonal fluctuations in groundwater levels are also likely to occur.

7

7 GEOHAZARDS ASSESSMENT

7.1 Seismicity

A seismic assessment has been carried out in general accordance with NZGS guidance[11]. The ultimate limit state (ULS) and serviceability limit state (SLS) peak ground accelerations (PGAs) were assessed based on a 50-year design life and Importance Level (IL) 2 buildings (residential buildings), in accordance with the New Zealand Building Code.

The recommended PGA values for geotechnical assessment at this Site are presented in Table 2 below. Structural designers working on this Site should assess seismic parameters in accordance with NZS1170:2004.

Table 2: Design Peak Ground Acceleration (PGA) for Various Limit States

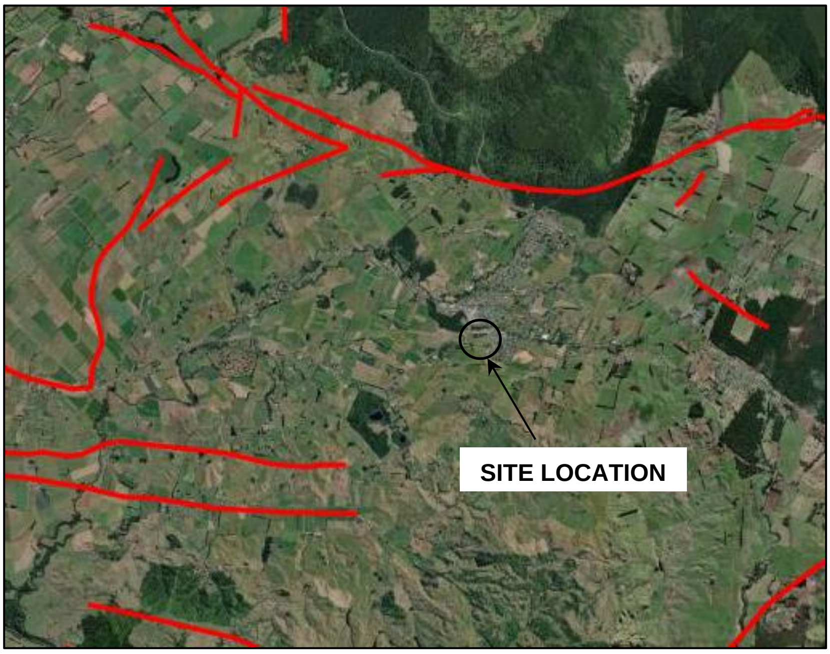
Limit State	AEP	R	PGA(g)[1]	$Magnitude_{eff}$
SLS	1/25	0.25	0.10	6.1
IL2 ULS	1/500	1.0	0.39	

Note: R = return period factor; AEP = annual exceedance probability; [1]As per Appendix A1 of NZGS Module 1

7.2 Fault Rupture

The Institute of Geological and Nuclear Science (GNS)[12] and Horizons Regional Council Natural Hazard Viewer[13] show several active faults to the north, south, east, and west, as shown on Figure 3 below. These are approximately 1km to 5km away from site. The closest known active fault to the site is an unnamed fault 1.1km to the north.

The distance to the nearest fault suggests a low risk of fault rupture affecting the subject site.



[11] NZ Geotechnical Society publication "Earthquake geotechnical engineering practice, Module 1: Overview of the standards", (November 2021)

[12] GNS, 'New Zealand Active Faults Database' (2022)

[13] https://experience.arcgis.com/experience/3f7b4ec2f6f14503af1146ce412de39e/page/Fault-Lines/

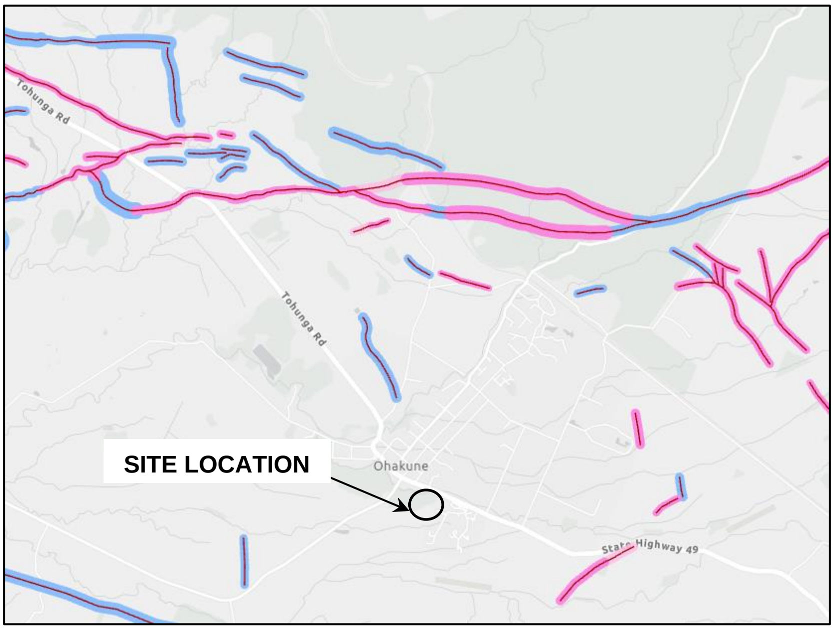


Figure 3: Active Faults (3a - GNS Active Fault Database, 3b – Horizons Regional Council Natural Hazard Viewer)

7.3 Liquefaction

Liquefaction occurs in loose saturated cohesionless soils that are subject to cyclic shear loading during an earthquake. This process leads to pore pressure build-up, soil grains moving into suspension and temporary loss of strength causing vertical and lateral ground deformation.

In accordance with MBIE/NZGS guidance[14] the liquefaction susceptibility of the soils at this site was assessed with respect to geological age and compositional (soil fabric and density) criteria.

Only the saturated soils below the groundwater table were modelled as being susceptible to liquefaction. Typically the soils encountered below the groundwater table are granular and therefore based on their composition are considered susceptible to liquefaction.

Conventionally CPT traces are used to carry out liquefaction analyses, however given that all the CPTs terminated at shallow depths (typically at depths less than 2.0mbgl), SPT N-values were used to undertake specific liquefication analyses for the site. Based on experience on other sites within the region with similar ground conditions, liquefaction is unlikely to be triggered for the design PGA in soils with SPT N-values greater than 30.

As outlined in Section 5.2, the SPT N-values within the granular fan deposits below the groundwater table are variable but typically are greater than 30 with isolated layer with lower SPT N-values.

Quantitative liquefaction susceptibility analyses was carried out using the computer software package LiqSVs 2.0, based SPT N-values obtained from the machine boreholes and in accordance with Boulanger & Idriss (2014) method. Results demonstrate that liquefaction triggering occurs where SPT N-values are less than approximately 12 to 15, which as described above are generally limited to isolated layers within the soil profile with significantly higher N values above and below.

Select LiqSVs 2.0 outputs are presented in ***Appendix C*** and are summarised below:

- Under the SLS seismic case, estimated liquefaction induced settlements are to be less than 10mm;

[14] Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards", (November 2021)

- Under the ULS seismic case, liquefaction induced settlements of between approximately less than 10mm and 40mm when considering a 10m depth limit to establish ULS index settlement for comparison with the MBIE guidance documents.
- The layers considered susceptible to liquefaction are isolated and are typically less than 2.0m thick and at least 4.5m below the existing ground surface.

Recommendations to mitigate potential ULS liquefaction settlement effects on the proposed development are provided below in Section 9.

7.4 Lateral Spread

Following the onset of liquefaction, the liquefied soils behave as a very weak undrained material, which can give rise to lateral spreading where a free face is present within, or in the vicinity, of the site or across sloping ground.

When assessing lateral spread, Chu et al (2006)[15] found that predicted values better matched observed values when liquefaction calculations in the soil profile were limited to a depth of twice the free face height (2H).

For lateral spread to occur, liquefaction must develop within shallow continuous soil layers that extend a sufficient length and width towards a free face or across sloping ground. Giving consideration to the liquefaction analysis results, ground model and site topography, the liquefiable layers within the upper 4m of the soil profile as shown to be relatively thin and non-continuous layers. As such, the risk lateral spread is considered to be low.

7.5 Slope Stability

The surrounding landform and landform across the site is near level to gently sloping, and on this basis the risk of deep-seated land instability impacting the proposed development is considered low. Quantitative site-specific stability analyses were therefore not considered necessary for the site.

Consideration will need to be given to the open drains located across the site. The gradient of the batters into the drains are currently very steep standing near vertical. Guidance on possible mitigation measures are provided in Section 7.2.

7.6 Load Induced Settlement

Load-induced settlements occur in subsoils that are subject to static loading (eg. by filling and / or building loads) where the magnitude of settlement is governed by the soil stiffness and the load applied.

The soils encountered beneath the site generally, with the exception of the former gullies / channels, comprise stiff to very stiff silts and medium dense to very dense sands and gravels, which are not considered to be prone to significant or excessive static settlements under typical residential development loads.

The uncontrolled fill encountered within the former gullies / channels, as shown on ***Drawing 02***, is variable in strength and organic content, and future loads may induce settlements that are likely to be greater than the recommended tolerances provided in the NZ Building Code. Mitigation options are discussed in Section 8.3.3.

Provided the recommendations outlined in Section 8.3 below are followed, the risk of excessive static settlements below the proposed development is considered to be low and should comply with the maximum differential settlement criteria of 25mm over 6m under the serviceability limit state scenario as recommended in Appendix B of Section B1VM4 of the NZ Building Code.

[15] Chu, D.B., Stewart, J.P., Youd, T.L., & Chu, B L. (2006). Liquefaction-induced lateral spreading in near-fault regions during the 1999 Chi- Chi, Taiwan earthquake. Journal of Geotechnical and Geoenvironmental Engineering, December 2006, 1549-1565.

7.7 Sensitive Soils

The Volcanic Ash deposits that will be encountered within the earthworks cuts / exposed at or immediately below design subgrade level are sensitive to remoulding and can have a high natural moisture content. Care will be required to avoid over working and trafficking of these materials, and to protect them from moisture ingress. Further recommendations are provided in Section 7.3.

8 GEOTECHNICAL RECOMMENDATIONS

8.1 Seismic Site Subsoil Category

The geological strata encountered beneath the site comprise soil strength materials, which with respect to the seismic site subsoil category defined in Section 3.1.3 of NZS1170.5, is defined as having an unconfined compressive strength (UCS) < 1MPa. However, the depth to bedrock at this location has not been confirmed.

Based on these ground conditions the seismic site subsoil category may be either Class C (shallow soils) or Class D (deep soils) in accordance with NZS1170.5. Deeper investigations would be needed to confirm the specific subsoil category.

For geotechnical analyses, based on our understanding of the local geology and in accordance with the current MBIE guidance, the site should be assessed for Class C conditions.

We recommend that structural designers working on this site considered both Class C and Class D conditions and choose the more conservative case for their design unless additional deeper investigation is carried out.

8.2 Slope Stability Management

The risk of deep-seated slope instability affecting the development has been assessed as low, however, buildings that may extend to within several metres to the crest of the open drains may be susceptible to soil creep or some slope instability, especially where unmaintained drain banks become eroded and unstable.

Based on the current landform, a building restriction line (BRL) set back at least 5 metres from the crest of the open drains, has been nominated for the site. The Slope Instability Hazard Zone, as shown on ***Drawing 02,*** is defined by this BRL. All structures requiring building consent must be located outside the Slope Instability Hazard Zone unless supported by further geotechnical investigation and/or assessment by a Chartered Professional Geotechnical Engineer;

Alternatively, the following mitigation measure may be considered to reduce the risk of slope instability on the proposed development:

- Regrade the banks to improve long-term stability;
- Open drains may be piped and backfilled as part of the subdivision earthworks;
- Support the banks with specifically designed walls.

8.3 Earthworks

8.3.1 General

All earthwork activities must be carried out in general accordance with the requirements of NZS 4431[16] and the requirements of the District and Regional Council under the guidance of a Chartered Professional Geotechnical Engineer.

8.3.2 Material Suitability / Conditioning

Within cut areas the natural subgrade is expected to comprise sensitive silt and clay of the volcanic ash deposits. We expect that excavation of these materials will be readily achieved with normal earthworks plant, such as scrapers and excavators.

Whilst these materials are considered generally suitable to use for the construction of engineer certified fills, their relatively high sensitivity means that they have a narrow range of moisture contents in which they can be successfully earthworked.

Particular care must therefore be exercised by the earthworks contractor to optimise the moisture condition of these soils to enable compaction to certifiable standards. This is likely to require disking of the soils in both cut and fill areas with adequate allowance for conditioning in dry summer conditions. It is also noted that timeframes for earthworks may be lengthened considerably if intermittent rainfall occurs through the summer months.

8.3.3 Subgrade Preparation

Preparation of the firm to stiff / medium dense subgrade beneath the proposed fill areas should comprise stripping of all vegetation, topsoil, any pre-existing fill materials or weak colluvium followed by benching of the exposed subgrade where natural slopes beneath the fill exceed gradients of nominally 1:5 (vertical to horizontal). The subgrade should then be scarified and moisture conditioned where necessary and then proof rolled to verify the subgrade stiffness and consistency.

The uncontrolled fill encountered within the former gullies / channels, as shown on ***Drawing 02***, is variable in strength and organic content. As such these areas of the site will need to be over-excavated and replaced with engineered fill. The subgrade should be observed by the project geotechnical engineer prior to fill placement.

The subgrade should then be proof rolled under the guidance of a geotechnical engineer to verify the subgrade density and consistency. Where any particularly loose / soft materials are encountered during the proof rolling process, they should be undercut and removed prior to placing engineered fill.

8.3.4 Earthworks Cut/Fill Batter Stability

Earthworks will be required to from level building platforms and access roads.

Self-supporting long term cut and fill batters should be formed no steeper than 1(V):2.5(H). Fill batters can be formed to a maximum of 3m height, cut batters should be up to a maximum of 1.0m, due to shallow groundwater. Alternatively, batters can be supported by specifically designed retaining walls.

All formed batters should be covered and then grassed as soon as practicable following construction to reduce the effects of surficial scour/rilling.

[16] Standards New Zealand (2022) Engineered fill construction for lightweight structures, NZS 4431:2022, NZ Standard

8.3.5 Stockpiles

Careful consideration must be given to the location of temporary topsoil / unsuitable stockpiles to ensure that they are not located immediately above steep or unstable slopes (open drains).

The location of all temporary stockpiles must be approved by the Geotechnical Engineer prior to placement. Where stockpiles cannot be avoided above sloping ground they should be placed over a wide area with the height restricted under the direction of the Geotechnical Engineer.

8.3.6 Compaction

All earthfill must be placed, spread and compacted in controlled lifts under the supervision of a Chartered Professional Geotechnical Engineer.

Cut material is expected to comprise of volcanic ash deposits, which should be suitable for reuse as Engineer Certified Fill. Soil textures and moisture contents will however vary widely and careful management, conditioning and compaction control will be required.

All earthfill must be placed to ensure adequate knitting of successive fill lifts by ripping any natural subgrade or fill surfaces that have become dry prior to placing the following fill lift

8.3.7 Quality Control

The stripping of topsoil, undercutting of pre-existing uncontrolled fill materials, where required from across the site must be subject to observation by the project geotechnical engineer to ensure that all unsuitable materials have been removed.

The source and / or type of material used for engineered fill will dictate the type of quality control testing undertaken.

For granular (sand and gravel) fill materials, testing following compaction should be principally in terms of the maximum dry density within the appropriate water content range, which may be calibrated with a dynamic cone (Scala) penetrometer test that is then used as the primary testing measure. Where the source or quality of fill changes, re-calibration will be required.

Where silts and clays are used as fill alternative test criteria using vane shear strength and air voids should be used.

Further earthworks recommendations will be provided during detailed design once the cut and fill depths are known with the view to provide a geotechnical earthworks specification prior to earthwork commencing onsite.

8.3.8 Temporary Sediment Retention Ponds

Temporary sediment retention ponds may be required to store stormwater for significant periods (several months to years) and therefore their construction should be subject to design and observation input from the geotechnical engineer. As a minimum, the following input is recommended from the project geotechnical engineer:

- Advise on pond locations with respect to land stability and seepage potential;
- Structural design of pond fill embankments including key and compaction specification;
- Observe embankment subgrade conditions and advise on undercut requirements;
- Earthfill QA / QC testing of all embankment materials to ensure compliance with specification.

When decommissioning temporary sediment ponds, all water softened material in the bases and sides of the ponds shall be removed and undercut to the satisfaction of the Geotechnical Engineer. Backfilling of temporary ponds shall be to the compaction standard for general filling unless otherwise specified.

8.4 Civil Works

8.4.1 Subgrade CBR

The subdivision roading is expected to be constructed in a combination of both cut and fill areas. For preliminary design purposes, we recommend a CBR of 3% for the natural silty/clay soils and 5% for the fill soils.

It is recommended that soaked CBR laboratory testing and a programme of penetration resistance testing is carried out at routine intervals along road alignments, as part of the road pavement design prior to road construction to confirm CBR values.

The Volcanic Ash Deposit soils, are highly sensitive and degrade rapidly with trafficking. Where traffic can be left off these materials, they are moisture conditioned, recompacted at optimum moisture contents and located at least 1m above the peak winter water table, there could be some opportunity to use them as a pavement subgrade material. However, this may not be considered practical, and allowance should be made to undercut these materials and replace with a subgrade improvement layer (SIL).

8.4.2 Service Trenches

Most of the materials expected to be exposed during the excavation of service trenches should be readily removed using an excavator.

Trench collapse may pose problems in areas wherever service trenches extend below the water table, specifically if excavations extend below the volcanic ash deposits and into the sand/gravel Fan deposits. In this instance trench support is likely to be required. Temporary dewatering, in the form of regularly spaced sump pumps or well point dewatering spears may also be required.

At the completion of the development, Specific Design Zones for services will be applied to protect future foundations from settlement from poorly compacted trench backfill and to prevent new loads crushing service pipes. This is a restriction on building foundations within the 45 degree zone of influence from pipe inverts.

8.4.3 Retaining Walls

Specific engineer designed retaining walls may be required to support cuts and/or fills as part of the proposed development.

All retaining walls should be designed by a suitably qualified and experienced Chartered Professional Engineer taking into consideration undrained (short term) and drained (long term) ground conditions, seismic loads, groundwater conditions, surcharges above and toe slopes.

Recommended geotechnical design parameters for retaining wall are presented in **Error! Reference source not found.**3 below. These design parameters assume a horizontal ground surface above and below the retaining structure.

It is noted that some ground movement will occur behind temporary or permanent retaining walls. By definition, movement of the wall must occur to fully mobilise the active and passive earth pressure coefficients provided in Table 3 above. The extent of this movement is dependent on the height of retaining, type of wall selected and construction methodology. This must be considered during the design and construction of the retaining walls to ensure adjacent facilities are not adversely affected.

At the completion of the development, Specific Design Zones are expected to be applied to protect retaining walls from future overloading at the crest or undermining at the toe that could lead to instability. These zones typically extend the same distance as the wall height and where they are present above a wall, require deepening of foundations unless the wall has been designed for future foundation loads. Where they are present below a wall, careful consideration needs to be given to location, depth and timing of any future excavations.

Table 3: Retaining Wall Design Parameters

Soil Unit	ϒ (kN/m^3)	Ø' (deg)	c' (kPa)	K_0	Su (kPa)	No wall friction		Wall friction	
						K_a	K_p	K_a	K_p
Ashfall Deposits – Stiff to very stiff clayey/sandy silt	16	30	3	0.5	100	0.33	3.0	0.28	3.88
Fan Deposits – Medium dense to dense gravelly sand	17	35	1	0.5	-	0.27	3.69	0.23	5.17

Notes:

1. Refer Section 6.2 for definition of soil unit levels.
2. ϒ – soil unit weight; Ø' - angle of internal soil friction; Su – undrained shear strength; K0 - coefficient of earth pressure at rest, Ka - coefficient of active earth pressure, K_{ae} – coefficient of seismic earth pressure (at 100% ULS PGA), K_p - coefficient of passive earth pressure.
3. Values of K_0 are based on initial conditions following construction of the retaining wall.
4. The retaining wall designer must adopt the appropriate K_o, K_a and K_p parameters relevant to the actual construction method adopted and considering any additional surcharge effects.
5. Earth pressure coefficients with wall interface friction assume δ=2/3Ø for the active case and δ=1/3Ø for the passive case.
6. The above parameters are based on the conditions of a horizontal ground surface above and below the retaining structure. Applicable surcharge loads behind the wall must also be considered in the design.

8.4.4 Stormwater Soakage

Due to the cohesive nature of the surficial site soils and high groundwater table, disposal of concentrated stormwater flows via soakage to ground is not considered suitable. Specific details for the disposal of concentrated stormwater are yet to be confirmed with Cheal currently assessing options for the site, however we understand it is likely to utilise the existing open drains on the site.

9 FOUNDATION RECOMMENDATIONS

9.1.1 Liquefaction Mitigation

The liquefaction-induced settlements outlined in Section 7.3 above, meet the requirements of a Technical Category 2 (TC2) foundation solution as outlined in the MBIE Canterbury Rebuild Guidelines[17] as defined in ***Figure 4*** below.

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

Figure 4: Canterbury Rebuild Guidance Part A, Table 5.1: Summary of proposed foundation solutions for rebuilt foundations or new foundations on the flat

9.1.2 Bearing Capacity

Once bulk earthworks are completed in accordance with the recommendations provided in Section 7.3 above, a preliminary geotechnical ultimate bearing pressure of 300kPa should be available within both the natural cut ground and engineered fill areas, provided the short axis plan dimension of perimeter and internal beams does not exceed nominally 500mm.

There may be areas where localised variations in shear strength within the natural cut ground occur, particularly where the depth of cut varies across the building platforms. Further confirmation of available bearing pressures will be addressed at the time of post earthworks soil testing.

As required by section B1/VM4[18] of the New Zealand Building Code Handbook, the following strength reduction factors must be applied to all recommended geotechnical ultimate soil capacities in conjunction with their use in factored design load cases:

- 0.8 for load combinations involving earthquake overstrength;
- 0.5 for all other load combinations.

[17] Repairing and Rebuilding House affected by the Canterbury Easrthquakes", (December 2012)
[18] Ministry of Business, Innovation and Employment (2019) *Acceptable Solutions and Verification Methods for NZ Building Code Clause B1 Structure,* B1/VM4, Amendment 19

10 FURTHER WORK

The site investigation works were carried out prior to the development of the civil engineering drawings including any cut/fill earthworks and finalised scheme plan. Additional geotechnical inputs to support the design and construction of the proposed residential development at this site may include, but not limited to:

- Additional shallow investigations to further define the extent and thickness of the uncontrolled fill within the former gullies / channels;
- Additional machine borehole tests across the site to facilitate soils sampling for laboratory testing and further define subsoils across the site with the view to refine predicted liquefaction inducted settlements;
- Laboratory testing of potentially liquefiable soil layers to further assist with liquefaction analyses;
- Laboratory testing for earthworks including, standard compaction testing, solid densities and moisture contents in proposed borrow materials;
- Geotechnical analyses and reporting suitable to accompany a Resource Consent application to Ruapehu District Council.

11 CLOSURE

Additional important information regarding the use of your CMW report is provided in the '*Using your CMW Report*' document attached to this report.

This report has been prepared for use by Cheal Consultant Ltd and Kāinga Ora in relation to the Tei Tei Drive, Ohakune project in accordance with the scope, proposed uses and limitations described in the report. Should you have further questions relating to the use of your report please do not hesitate to contact us.

Where a party other than Cheal Consultant Ltd and Kāinga Ora seeks to rely upon or otherwise use this report, the consent of CMW should be sought prior to any such use. CMW can then advise whether the report and its contents are suitable for the intended use by the other party.

USING YOUR CMW GEOTECHNICAL REPORT

Geotechnical reporting relies on interpretation of facts and collected information using experience, professional judgement, and opinion. As such it generally has a level of uncertainty attached to it, which is often far less exact than other engineering design disciplines. The notes below provide general advice on what can be reasonably expected from your report and the inherent limitations of a geotechnical report.

Preparation of your report

Your geotechnical report has been written for your use on your project. The contents of your report may not meet the needs of others who may have different objectives or requirements. The report has been prepared using generally accepted Geotechnical Engineering and Engineering Geology practices and procedures. The opinions and conclusions reached in your report are made in accordance with these accepted principles. Specific items of geotechnical or geological importance are highlighted in the report.

In producing your report, we have relied on the information which is referenced or summarised in the report. If further information becomes available or the nature of your project changes, then the findings in this report may no longer be appropriate. In such cases the report must be reviewed, and any necessary changes must be made by us.

Your geotechnical report is based on your project's requirements

Your geotechnical report has been developed based on your specific project requirements and only applies to the site in this report. Project requirements could include the type of works being undertaken; project locality, size and configuration; the location of any structures on or around the site; the presence of underground utilities; proposed design methodology; the duration or design life of the works; and construction method and/or sequencing.

The information or advice in your geotechnical report should not be applied to any other project given the intrinsic differences between different projects and site locations. Similarly geotechnical information, data and conclusions from other sites and projects may not be relevant or appropriate for your project.

Interpretation of geotechnical data

Site investigations identify subsurface conditions at discrete locations. Additional geotechnical information (e.g. literature and external data source review, laboratory testing etc) are interpreted by Geologists or Engineers to provide an opinion about a site specific ground models, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist due to the variability of geological environments. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. Interpretation of factual data can be influenced by design and/or construction methods. Where these methods change review of the interpretation in the report may be required.

Subsurface conditions can change

Subsurface conditions are created by natural processes and then can be altered anthropically or over time. For example, groundwater levels can vary with time or activities adjacent to your site, fill may be placed on a site, or the consistency of near surface conditions might be susceptible to seasonal changes. The report is based on conditions which existed at the time of investigation. It is important to confirm whether conditions may have changed, particularly when large periods of time have elapsed since the investigations were performed.

Interpretation and use by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical report. To help avoid misinterpretations, it is important to retain the assistance of CMW to work with other project design professionals who are affected by the contents of your report. CMW staff can explain the report implications to design professionals and then review design plans and specifications to see that they have correctly incorporated the findings of this report.

Your report's recommendations require confirmation during construction

Your report is based on site conditions as revealed through selective point sampling. Engineering judgement is then applied to assess how indicative of actual conditions throughout an area the point sampling might be. Any assumptions made cannot be substantiated until construction is complete. For this reason, you should retain geotechnical services throughout the construction stage, to identify variances from previous assumption, conduct additional tests if required and recommend solutions to problems encountered on site.

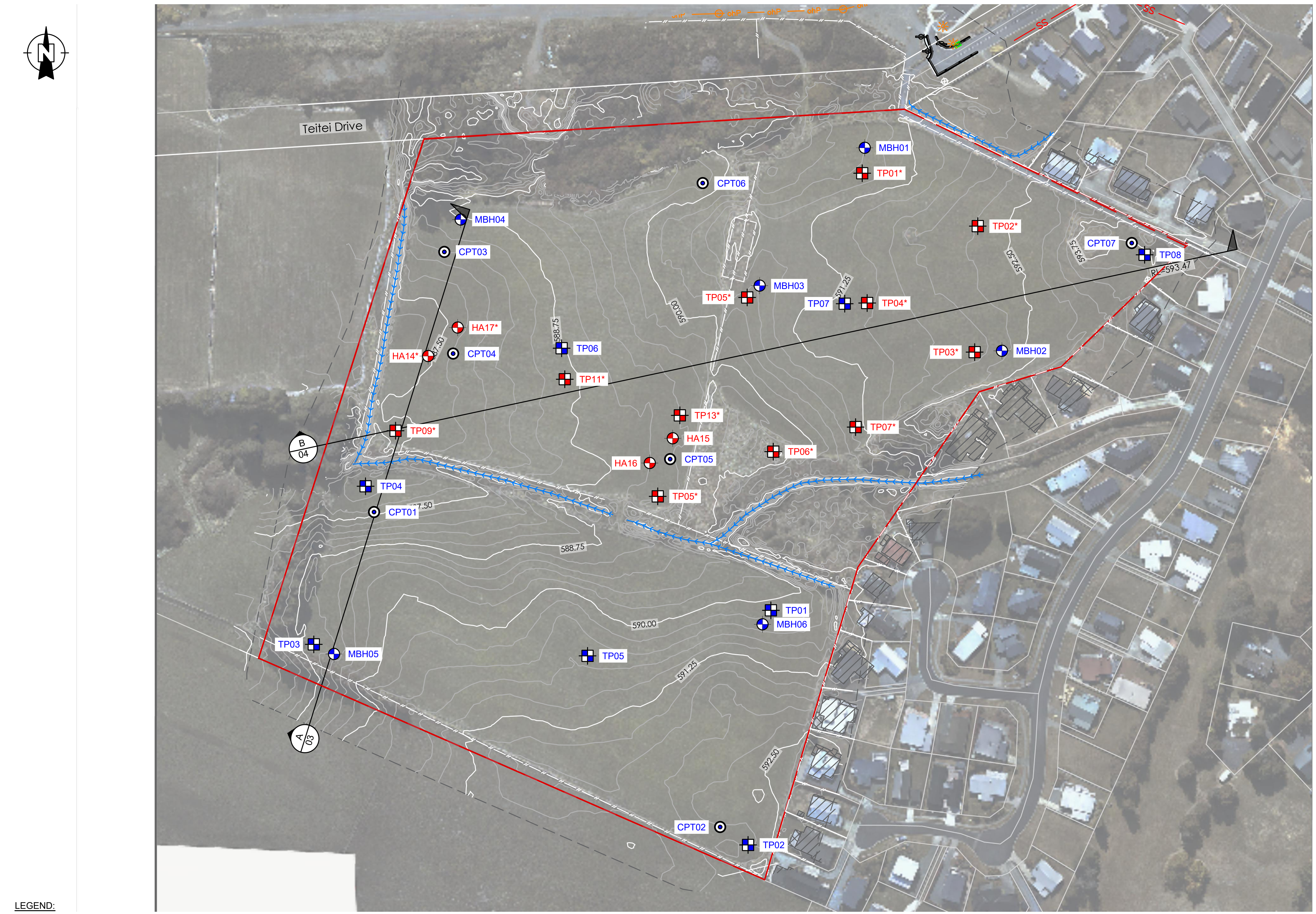
A Geotechnical Engineer, who is fully familiar with the site and the background information, can assess whether the report's recommendations remain valid and whether changes should be considered as the project develops. An unfamiliar party using this report increases the risk that the report will be misinterpreted.

Environmental Matters Are Not Covered

Unless specifically discussed in your report environmental matters are not covered by a CMW Geotechnical Report. Environmental matters might include the level of contaminants present of the site covered by this report, potential uses or treatment of contaminated materials or the disposal of contaminated materials. These matters can be complex and are often governed by specific legislation.

The personnel, equipment, and techniques used to perform an environmental study can differ significantly from those used in this report. For that reason, our report does not provide environmental recommendations. Unanticipated subsurface environmental problems can have large consequences for your site. If you have not obtained your own environmental information about the project site, ask your CMW contact about how to find environmental risk-management guidance.

Drawings



LEGEND:

	SITE BOUNDARY
MBH01	MACHINE BOREHOLE (MBH) LOCATION (CMW, 2022)
CPT01	CONE PENETROMETER TEST (CPT) LOCATION (CMW, 2022)
TP01	TEST PIT (TP) LOCATION (CMW, 2022)
TP01*	TEST PIT (TP) LOCATION (CHEAL, 2020)
HA01*	HAND AUGER (HA) LOCATION (CHEAL, 2020)

NOTES:

1. BASE PLAN ADAPTED FROM RUAPEHU DISTRICT COUNCIL MAPS. & DATA PROVIDED BY CHEAL CONSULTANTS.
2. CONTOURS ARE IN 0.25m INTERVALS IN TERMS OF TUHIRANGI CIRCUIT 2000.
3. TEST LOCATIONS ARE APPROXIMATE ONLY.

1:1500 — 0 15 30 45 60 75 m

CMW Geosciences

CLIENT: CHEAL CONSULTANTS LTD	DRAWN: HR	PROJECT: TGA2022-0238	
PROJECT: TEITEI DRIVE, OHAKUNE	CHECKED: EE	DRAWING: 01	
	REVISION: 0	SCALE: 1:1500	
TITLE: GEOTECHNICAL INVESTIGATION PLAN	DATE: 16/02/2023	SHEET: A3 L	

C:\Users\HyshamRasheed\CMW Geosciences Pty Ltd\Tauranga Office - TGA2022-0238 Tei Tei Drive, Ohakune\Drawings\TGA2022-0238 Site-plan_recover.dwg

LEGEND:

- SITE BOUNDARY
- OPEN DRAIN
- SLOPE INSTABILITY ZONE
- EXPECTED EXTENT OF UNCONTROLLED FILL ASSOCIATED WITH PREVIOUS INFILL WORKS

NOTES:

1. BASE PLAN ADAPTED FROM RUAPEHU DISTRICT COUNCIL MAPS. & DATA PROVIDED BY CHEAL CONSULTANTS.
2. CONTOURS ARE IN 0.25m INTERVALS IN TERMS OF TUHIRANGI CIRCUIT 2000.
3. TEST LOCATIONS ARE APPROXIMATE ONLY.

0 15 30 45 60 75 m

1:1500

CMW Geosciences

CLIENT: CHEAL CONSULTANTS LTD	DRAWN: HR	PROJECT: TGA2022-0238
PROJECT: TEITEI DRIVE, OHAKUNE	CHECKED: EE	DRAWING: 02
	REVISION: 0	SCALE: 1:1500
TITLE: GEOTECHNICAL HAZARD PLAN	DATE: 16/02/2023	SHEET: A3 L

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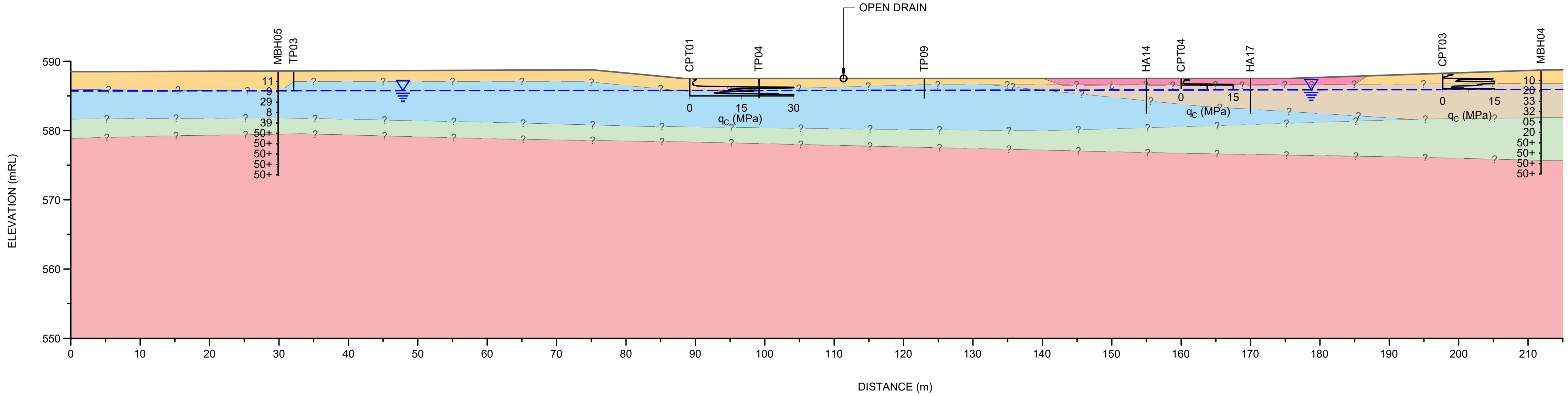


LEGEND:

- V.STIFF CLAYEY SILT **(FAN DEPOSITS)**
- SOFT TO STIFF SILT, LOCALLY ORGANIC **(UNCONTROLLED FILL)**
- INTERBEDDED M.DENSE - DENSE GRAVELLY SAND & STIFF TO V.STIFF SANDY SILT **(FAN DEPOSITS)**
- M.DENSE TO DENSE GRAVELLY SAND & SANDY GRAVEL **(FAN DEPOSITS)**
- VERY STIFF TO HARD SILT **(FAN DEPOSITS)**
- V.DENSE SANDY GRAVEL & GRAVELLY SAND **(FAN DEPOSITS)**
- APPROX GROUNDWATER LEVEL
- EXISTING GROUND SURFACE
- APPROX GEOLOGICAL BOUNDARY

NOTES:

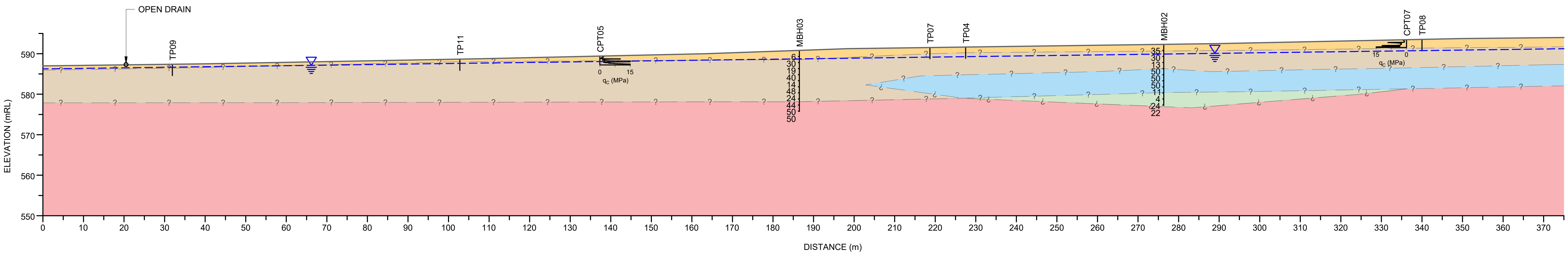
1. GEOLOGY INTERPRETED FROM NEARBY INVESTIGATIONS.
2. TEST LOCATIONS ARE APPROXIMATE ONLY.

0 7.5 15 22.5 30 37.5 m

1:750

CMW Geosciences

CLIENT:	CHEAL CONSULTANTS LTD	DRAWN: HR	PROJECT: TGA2022-0238
PROJECT:	TEITEI DRIVE, OHAKUNE	CHECKED: EE	DRAWING: 03
		REVISION: 0	SCALE: 1:750
TITLE:	GEOLOGICAL CROSS-SECTION A	DATE: 16/02/2023	SHEET: A3 L



LEGEND:

- V.STIFF CLAYEY SILT **(FAN DEPOSITS)**
- INTERBEDDED M.DENSE - DENSE GRAVELLY SAND & STIFF TO V.STIFF SANDY SILT **(FAN DEPOSITS)**
- M.DENSE TO DENSE GRAVELLY SAND & SANDY GRAVEL **(FAN DEPOSITS)**
- VERY STIFF TO HARD SILT **(FAN DEPOSITS)**
- V.DENSE SANDY GRAVEL & GRAVELLY SAND **(FAN DEPOSITS)**
- APPROX GROUNDWATER LEVEL
- EXISTING GROUND SURFACE
- APPROX GEOLOGICAL BOUNDARY

NOTES:

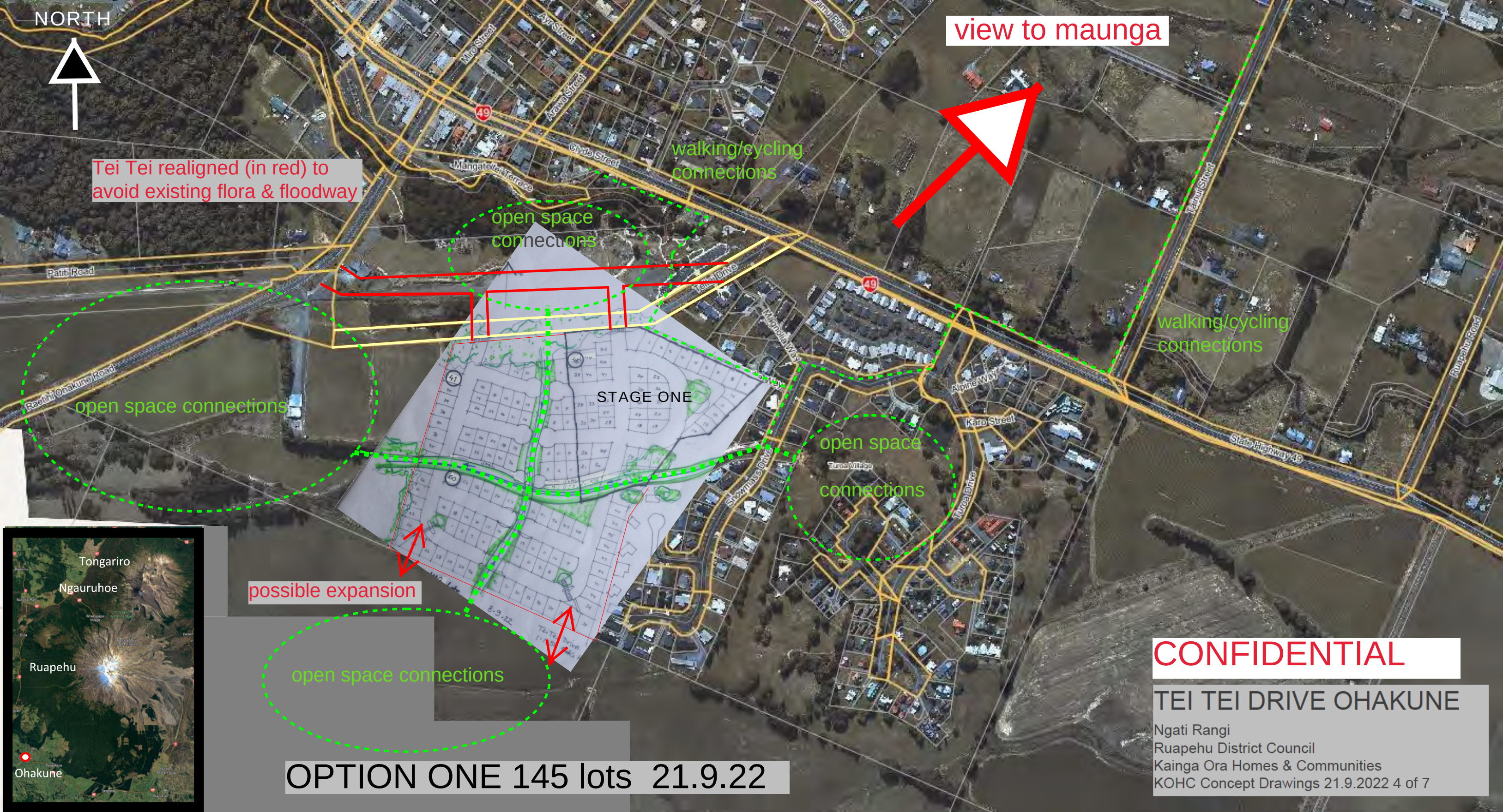
1. GEOLOGY INTERPRETED FROM NEARBY INVESTIGATIONS.
2. TEST LOCATIONS ARE APPROXIMATE ONLY.

1:1000 — 0 10 20 30 40 50 m

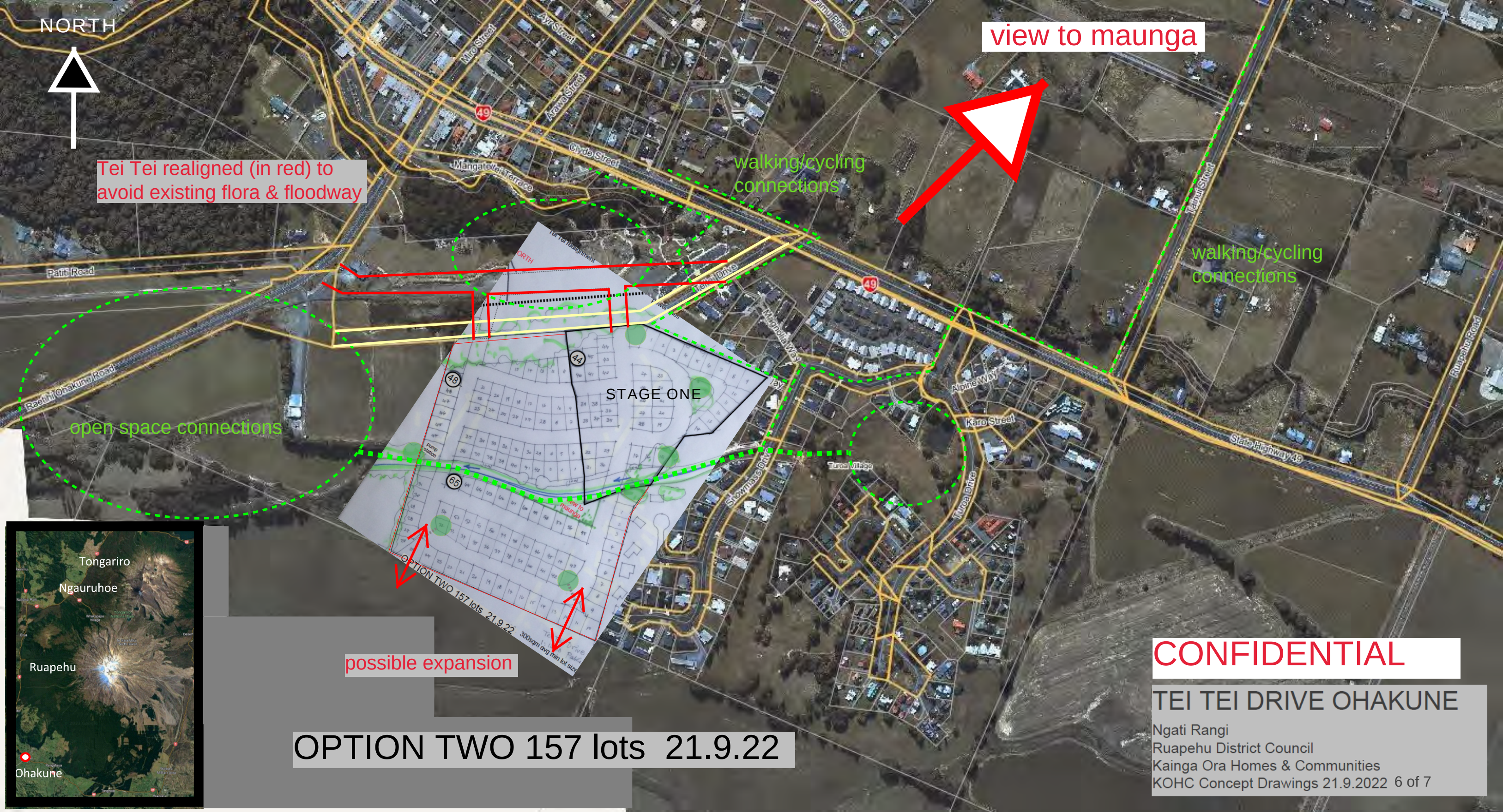
CMW Geosciences

CLIENT:	**CHEAL CONSULTANTS LTD**	DRAWN: HR	PROJECT: TGA2022-0238
PROJECT:	**TEITEI DRIVE, OHAKUNE**	CHECKED: EE	DRAWING: 04
		REVISION: 0	SCALE: 1:1000
TITLE:	**GEOLOGICAL CROSS-SECTION B**	DATE: 16/02/2023	SHEET: A3 L

Appendix A: Cheal 'Scheme plan sketches'


NORTH
view to maunga
Tei Tei realigned (in red) to
avoid existing flora & floodway
walking/cycling
connections
open space
connections
walking/cycling
connections
open space connections
STAGE ONE
open space
connections
possible expansion
open space connections
Patiti Road
Raetihi Ohakune Road
Clyde Street
Mangateitei Terrace
Tei Tei Drive
Miro Street
Ayr Street
Arawa Street
Tainui Street
Karo Street
Alpine Way
Snowmass Drive
Turoa Village
Turoa Drive
State Highway 49
Ruapehu Road
Tongariro
Ngauruhoe
Ruapehu
Ohakune
OPTION ONE 145 lots 21.9.22
CONFIDENTIAL
TEI TEI DRIVE OHAKUNE
Ngati Rangi
Ruapehu District Council
Kainga Ora Homes & Communities
KOHC Concept Drawings 21.9.2022 4 of 7


CONFIDENTIAL
power lines
Tei Tei realignment
NORTH
cycleway / walkway
STAGE ONE
45
40
60
SW run-off into swale
& pedestrian landscape
view to
maunga
pump
station
300sqm avg min lot size
18m road reserve
OPTION ONE 145 lots 21.9.22
TEI TEI DRIVE OHAKUNE
Ngati Rangi
Ruapehu District Council
Kainga Ora Homes & Communities
KOHC Concept Drawings 21.9.2022
5 of 7


NORTH
view to maunga
Tei Tei realigned (in red) to
avoid existing flora & floodway
walking/cycling
connections
walking/cycling
connections
STAGE ONE
open space connections
OPTION TWO 157 lots 21.9.22
possible expansion
Tongariro
Ngauruhoe
Ruapehu
Ohakune
Miro Street
Ayr Street
Arawa Street
Clyde Street
Mangawhero Terrace
Pattil Road
Raetihi Ohakune Road
Alpine Way
Karo Street
Turoa Village
State Highway 49
Ruapehu Road
OPTION TWO 157 lots 21.9.22
300sqm avg min lot size
CONFIDENTIAL
TEI TEI DRIVE OHAKUNE
Ngati Rangi
Ruapehu District Council
Kainga Ora Homes & Communities
KOHC Concept Drawings 21.9.2022 6 of 7


CONFIDENTIAL
power lines
Tei Tei realignment
NORTH
STAGE ONE
44
48
65
pump station
view to maunga
300sqm avg min lot size
18m road reserve
OPTION TWO 157 lots 21.9.22
TEI TEI DRIVE OHAKUNE
Ngati Rangi
Ruapehu District Council
Kainga Ora Homes & Communities
KOHC Concept Drawings 21.9.2022 7 of 7

Appendix B: Groundwater Level Data

Groundwater Depths Encountered		
Location	**Depth (mbgl)**	**Approx. Elevation of Test (m RL)**
MBH01	0.80	589
MBH02	2.40	588
MBH03	2.10	588
MBH04	2.80	587
MBH05	2.80	587
MBH06	1.50	589
CPT01	1.10	589
CPT02	1.60	588
CPT04	0.20	589
CPT05	0.8	588
CPT06	1.30	589
CPT07	1.5	589
TP01	1.5	585
TP02	1.5	585
TP03	1.5	582
TP04	2.5	583
TP05	N/A - TP Refused at 1.5m	584
TP06	0.9	584
TP07	1.6	586
TP08	N/A - TP Refused at 2.5m	589
Cheal TP01	N/A - TP Refused at 2.2m	587
Cheal TP02	3.1	587
Cheal TP03	3.3	587
Cheal TP04	3.1	587
Cheal TP05	2.5	586
Cheal TP06	1.5	585
Cheal TP07	2.8	586
Cheal TP08	2.8	585
Cheal TP09	2.5	583
Cheal TP10	N/A – TP refused at 3.0m	584
Cheal TP11	1.3	584
Cheal TP12	1.7	585
Cheal TP13	1.5	585
Cheal HA14	0.7	583
Cheal HA15	0.5	585
Cheal HA16	0.9	584

Appendix C: LiqSVs outputs

CMW Geosciences
A3/63 Apollo Drive
Rosedale, Auckland 0632
https://www.cmwgeosciences.com/

SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH01

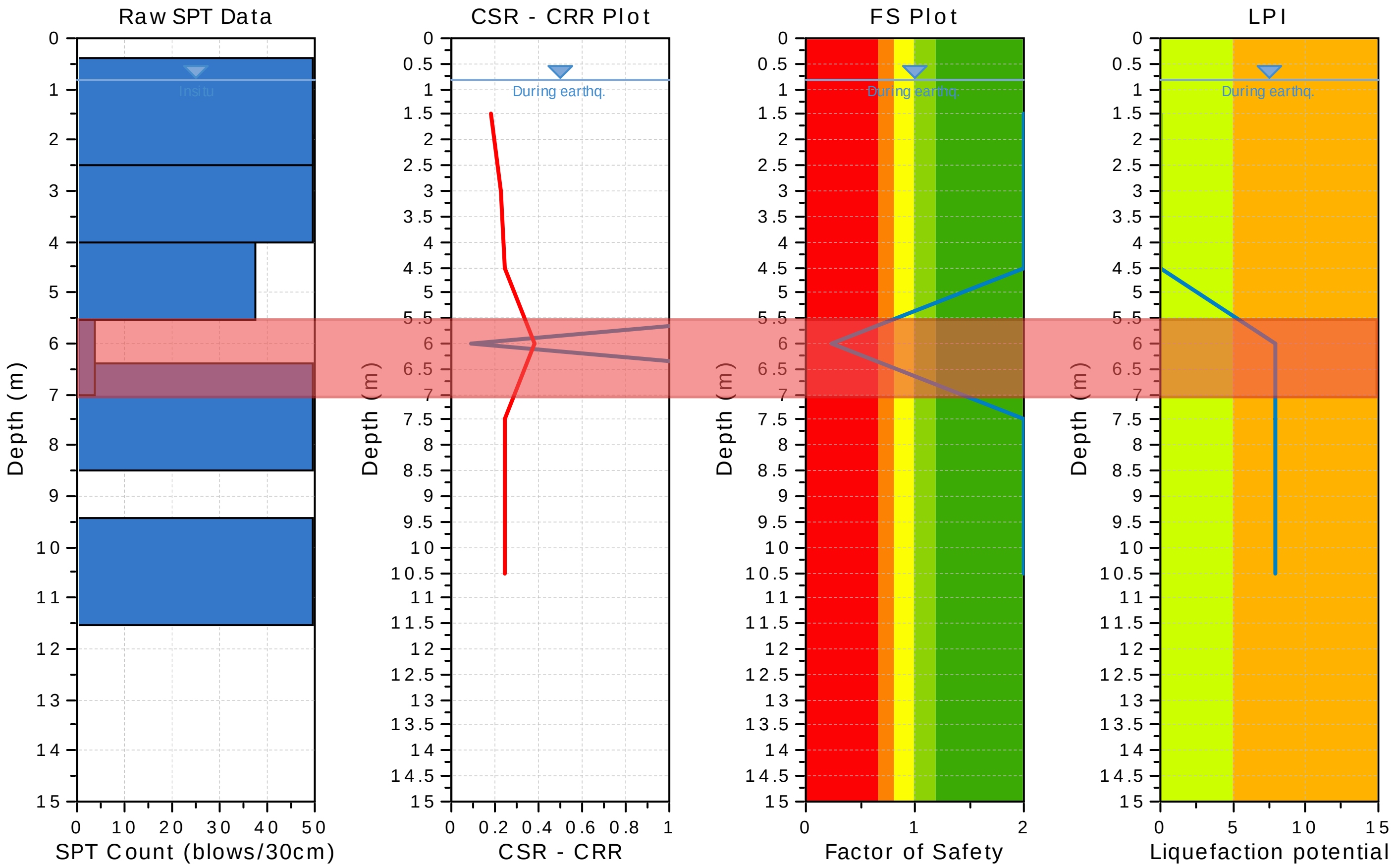
Location : Tei Tei Drive Ohakune

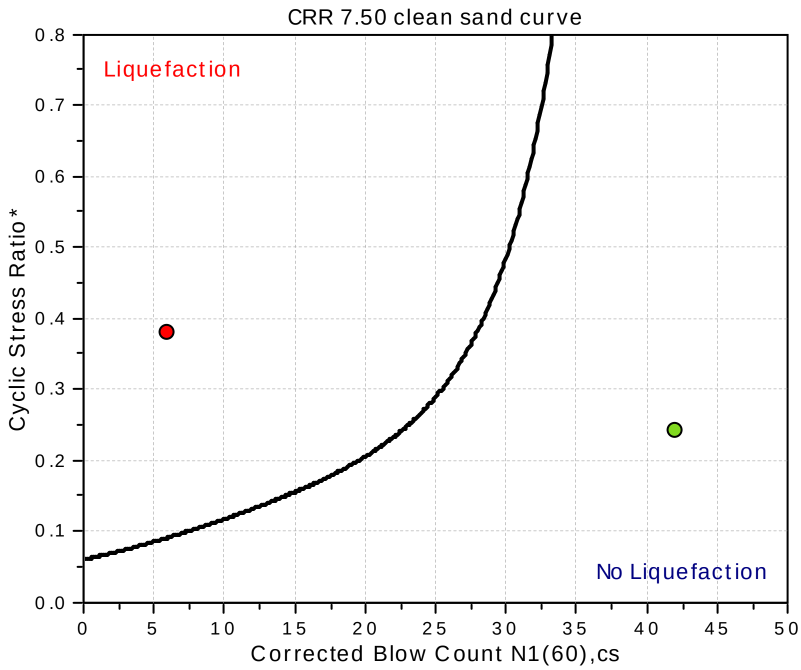
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	0.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	0.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.36 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





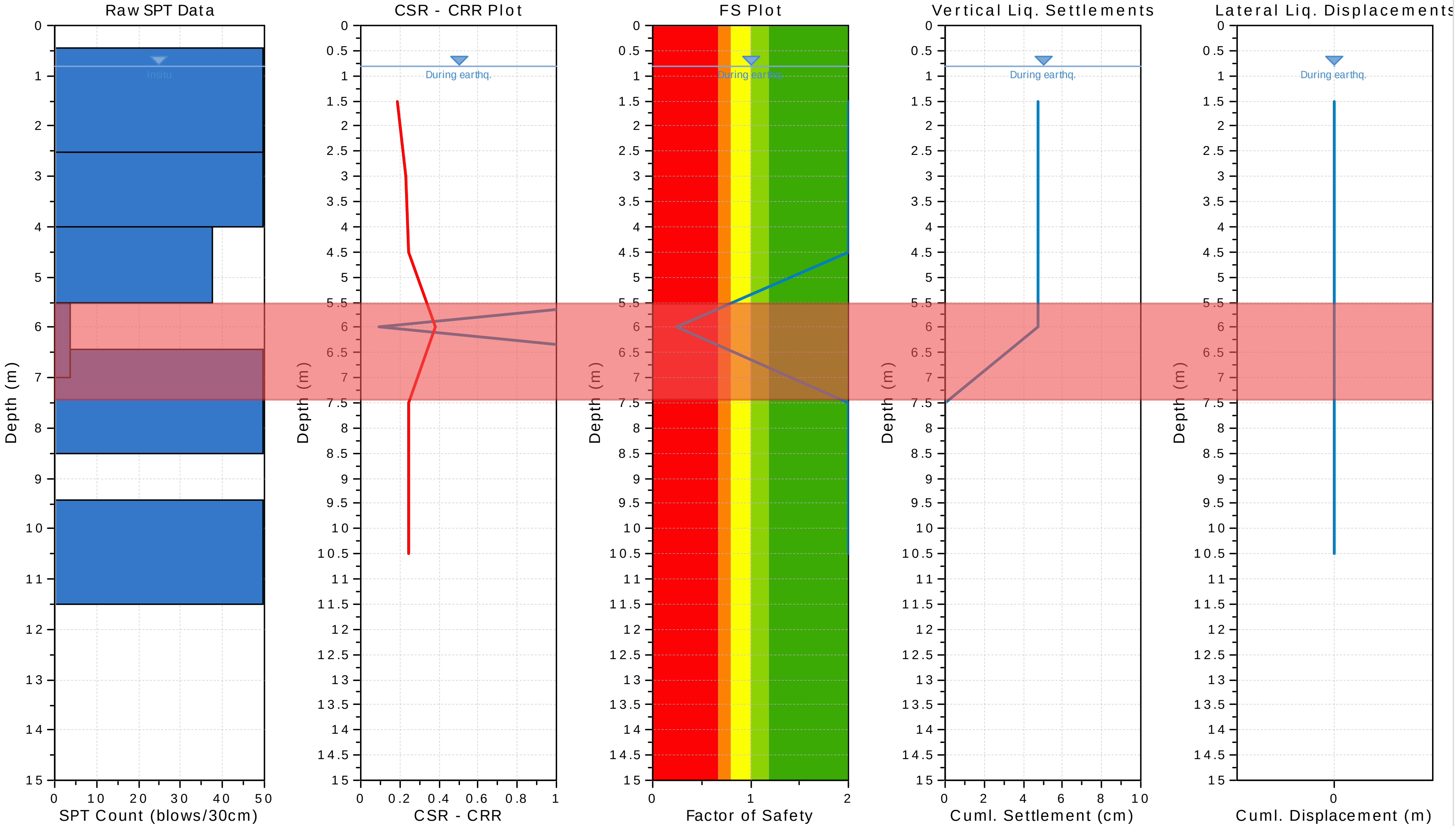
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



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A3/63 Apollo Drive
Rosedale, Auckland 0632
https://www.cmwgeosciences.com/

SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH02

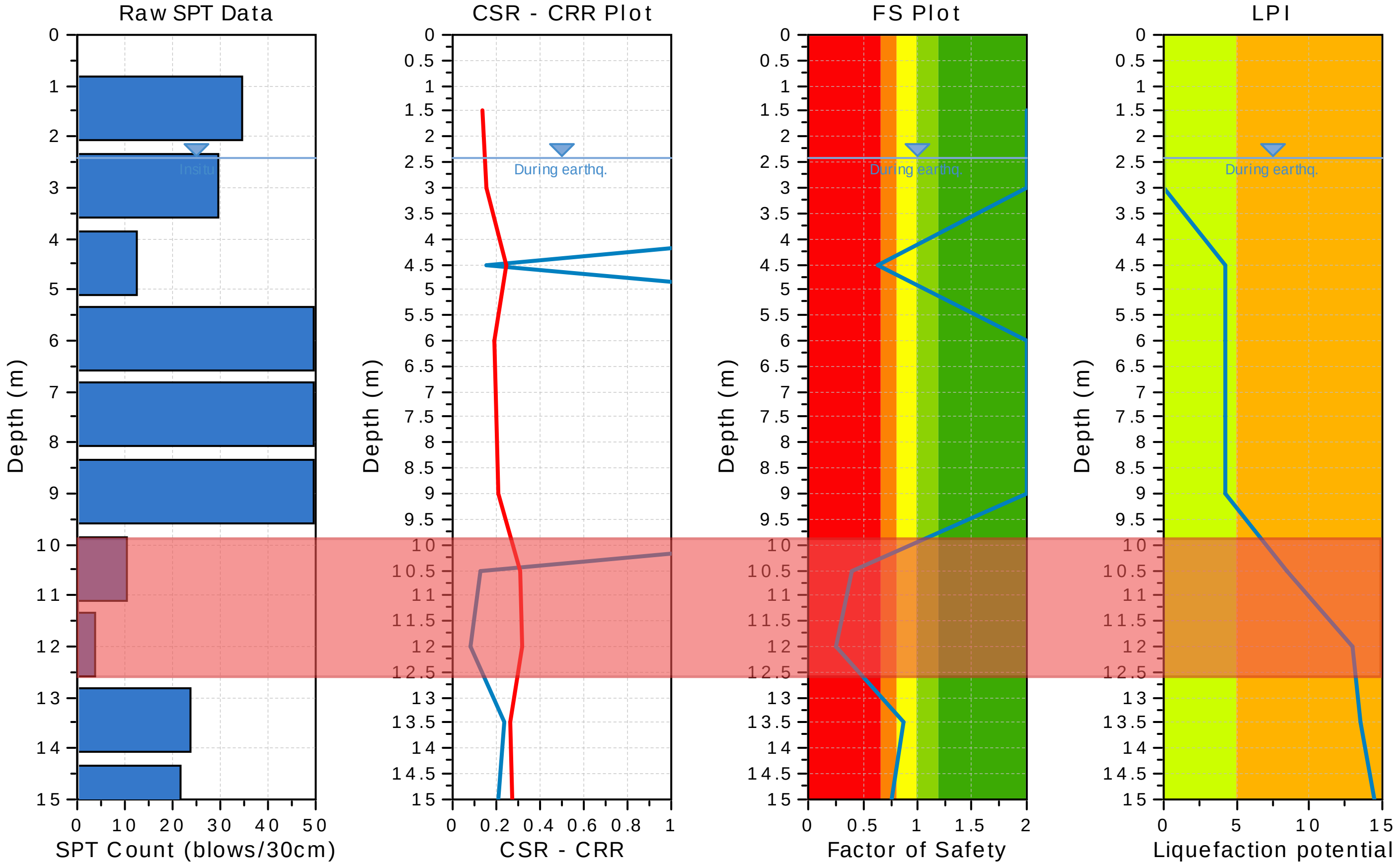
Location : Tei Tei Drive Ohakune

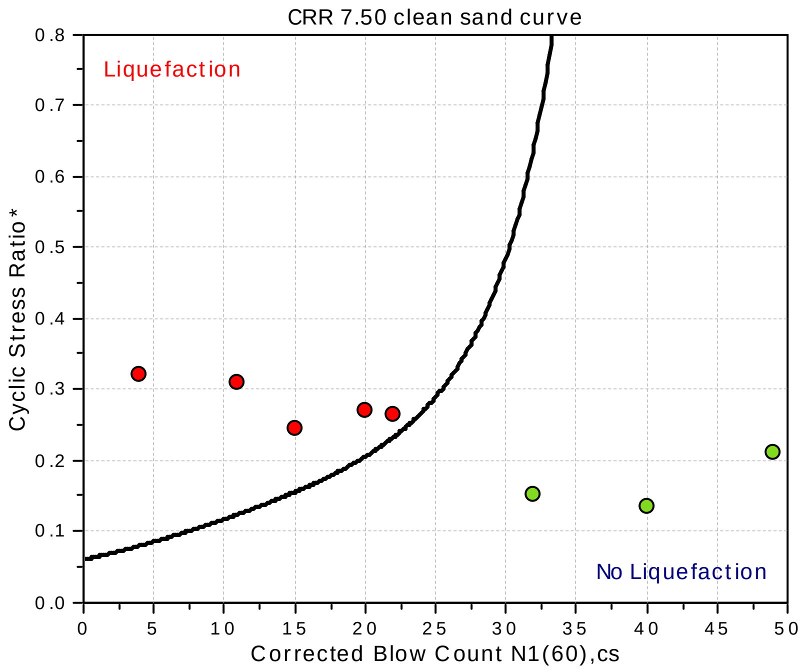
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.40 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.40 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.36 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





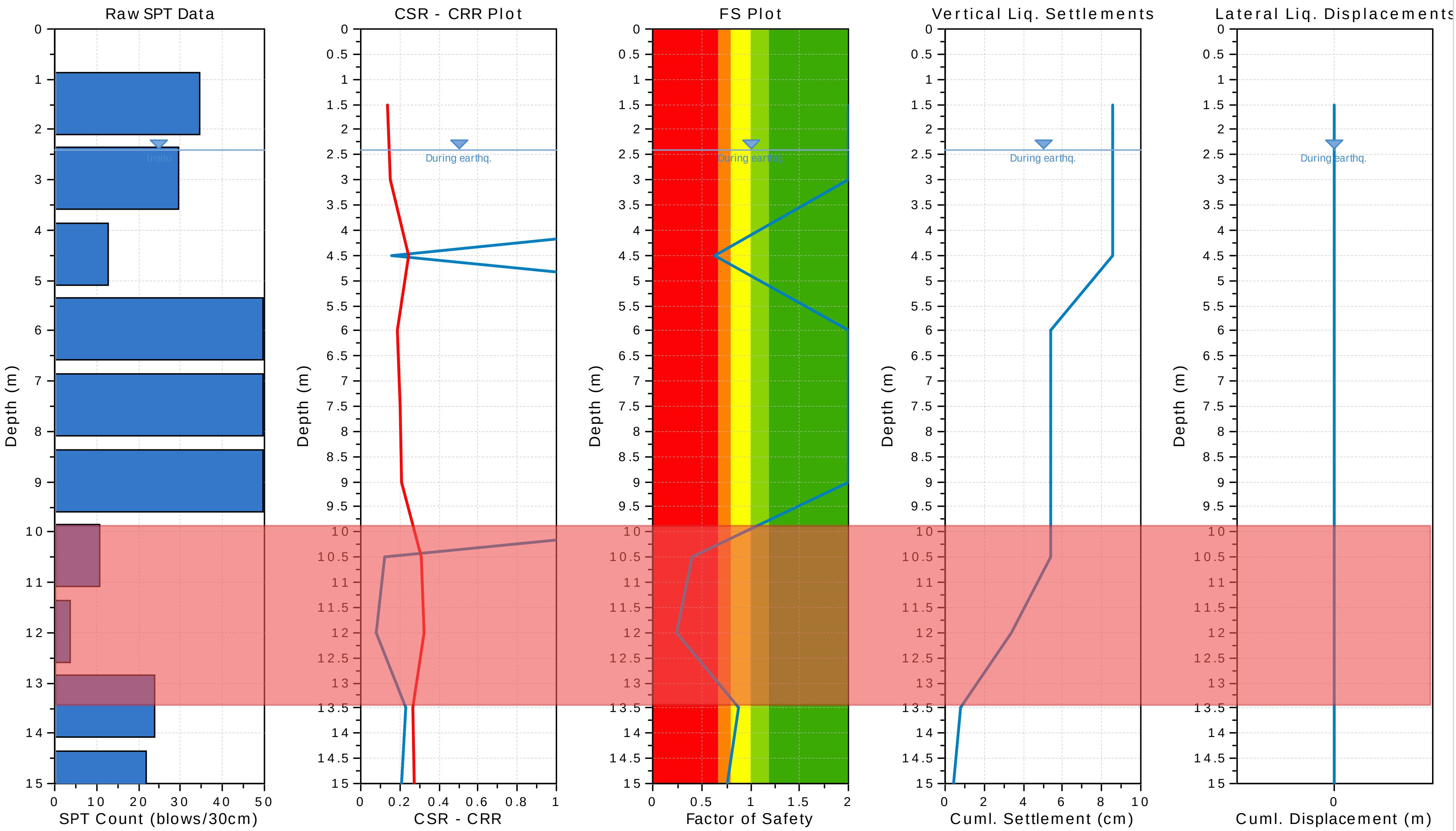
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



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Rosedale, Auckland 0632
https://www.cmwgeosciences.com/

SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH03

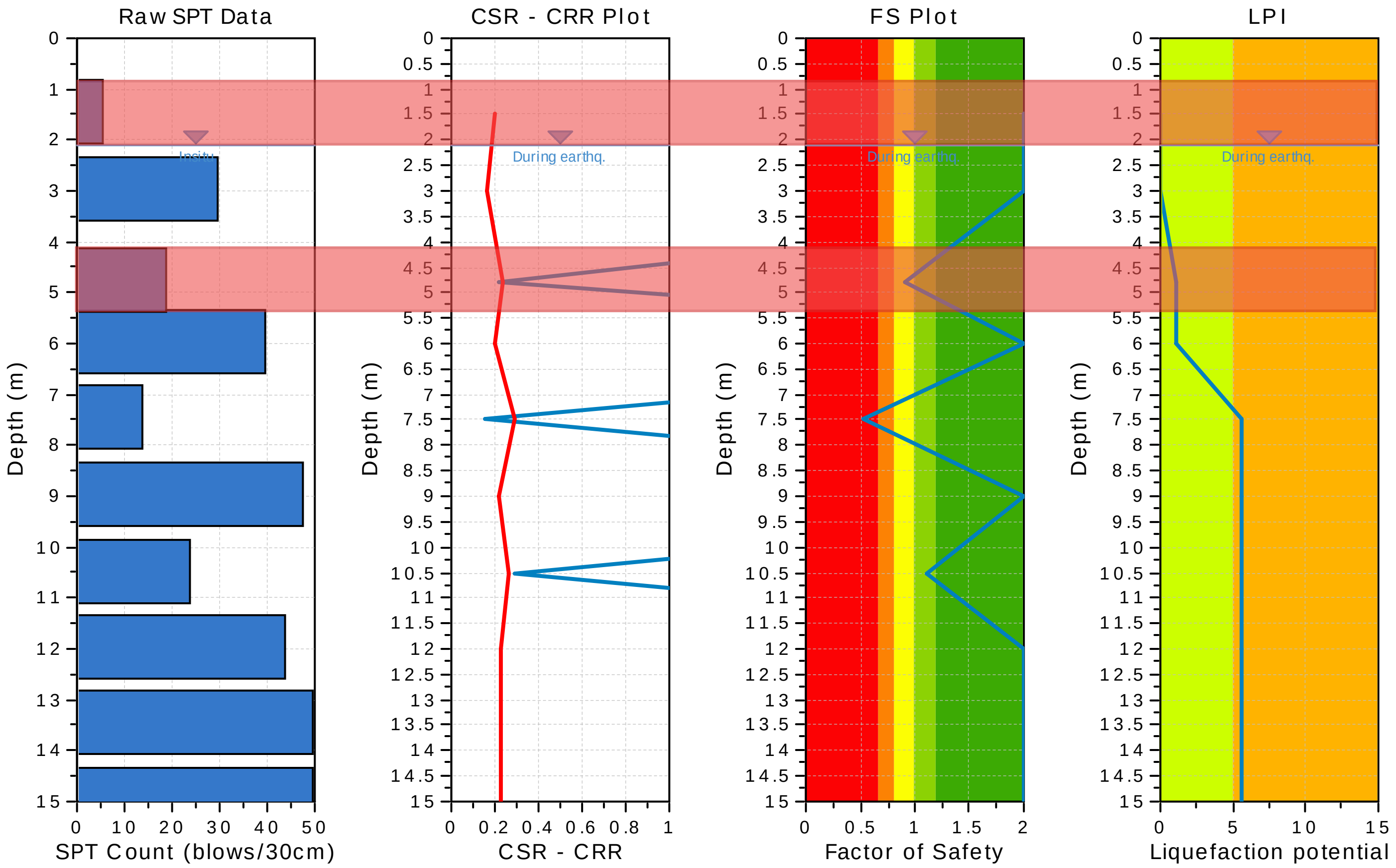
Location : Tei Tei Drive Ohakune

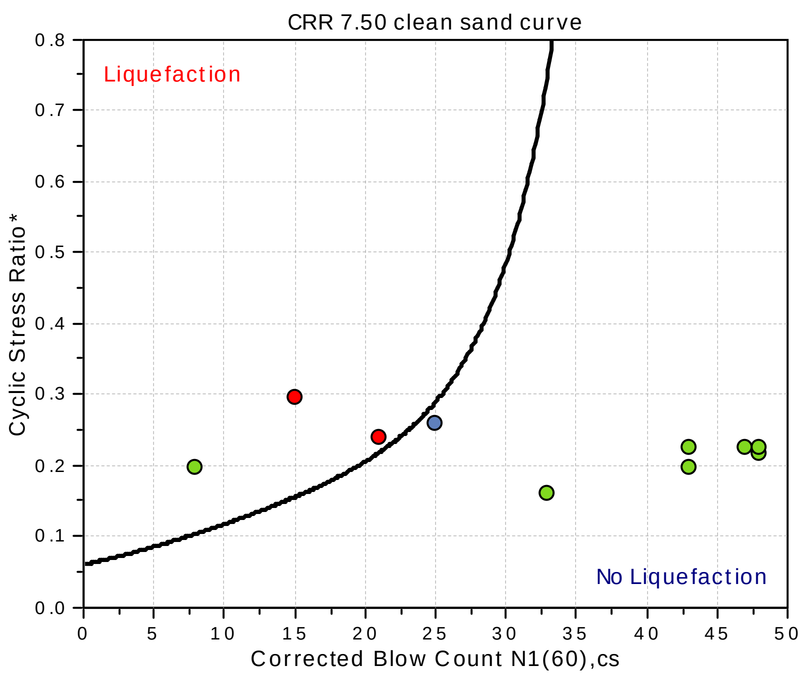
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.10 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.10 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.36 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





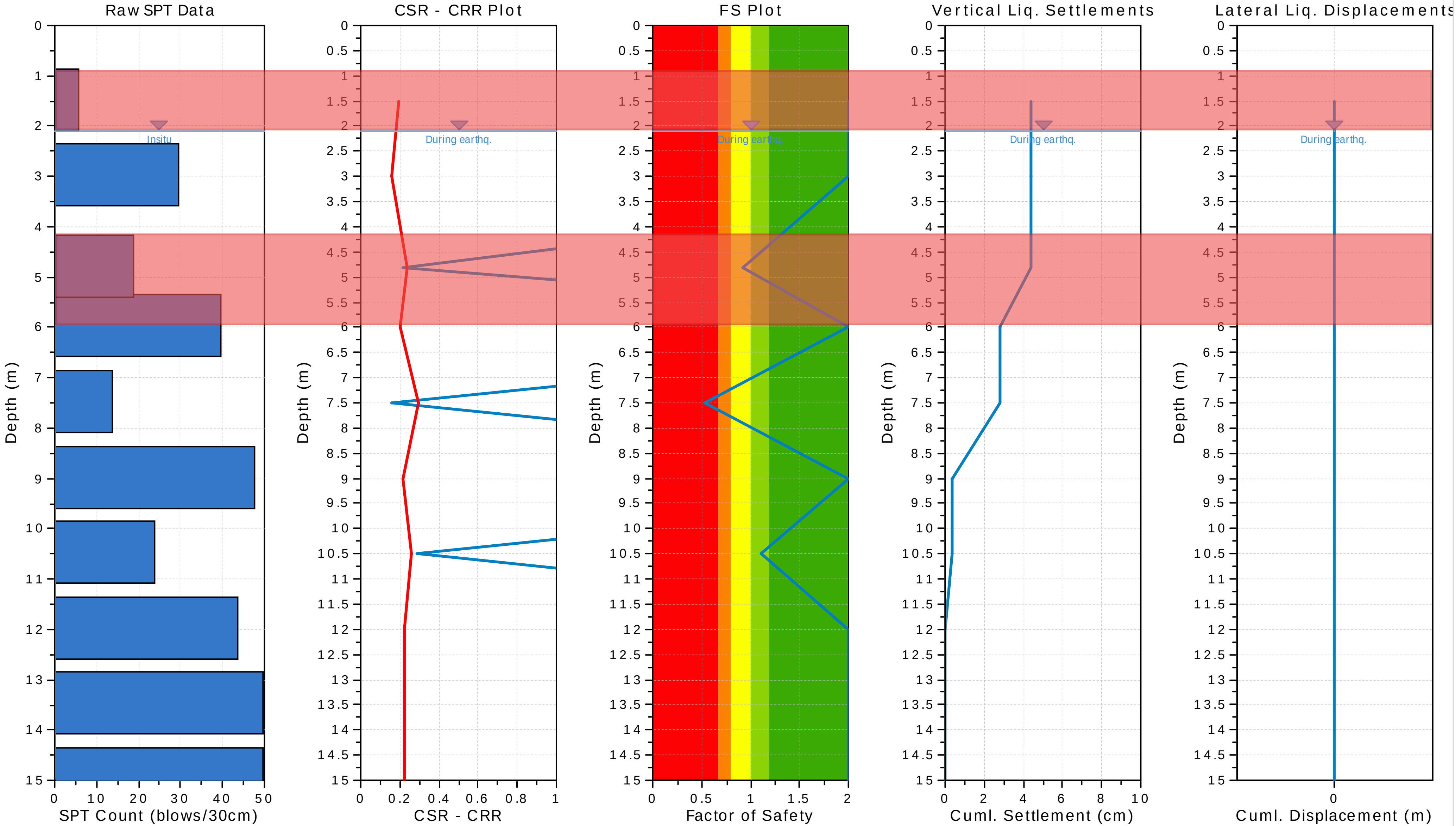
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



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Rosedale, Auckland 0632
https://www.cmwgeosciences.com/

SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

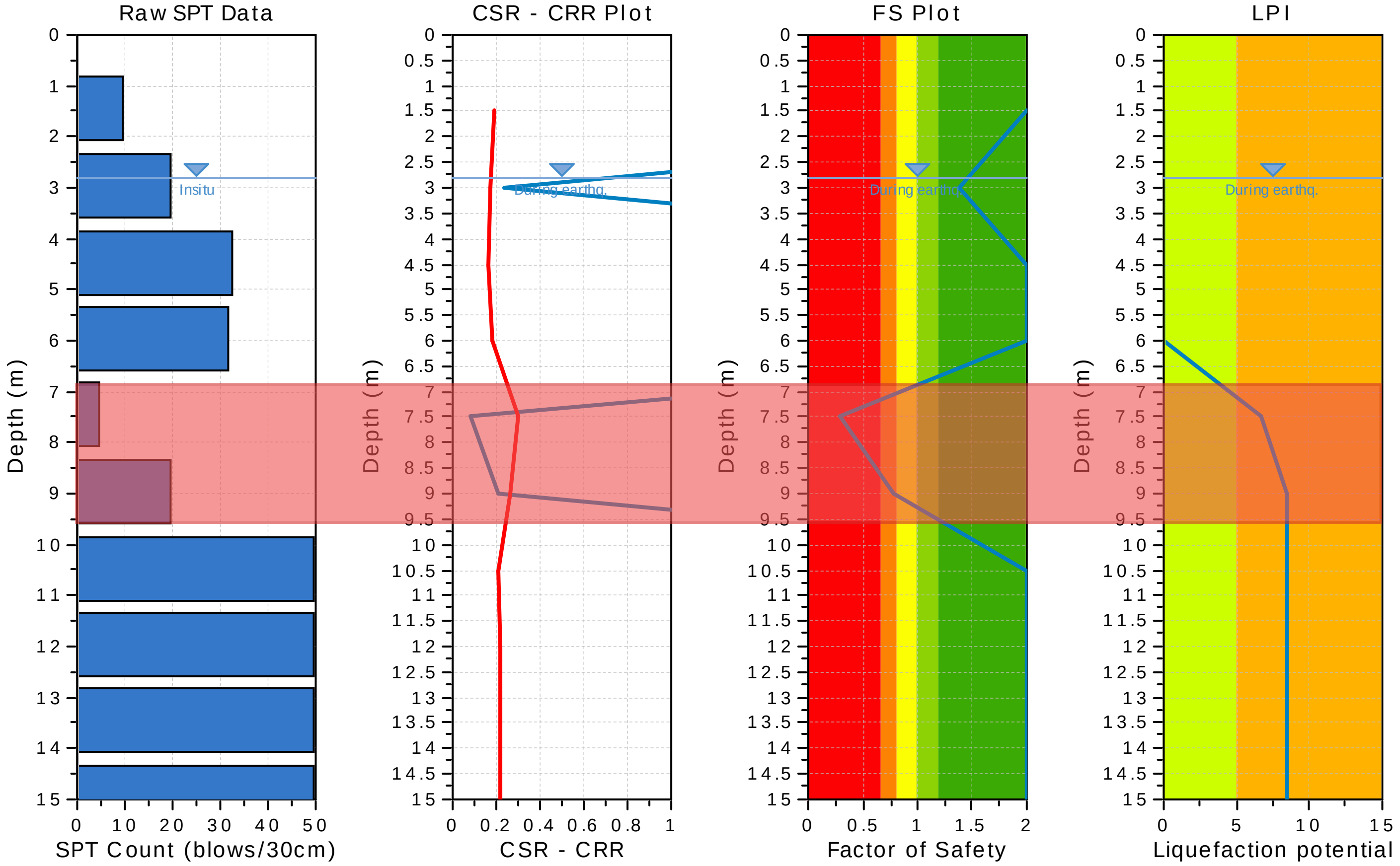
SPT Name: MBH04

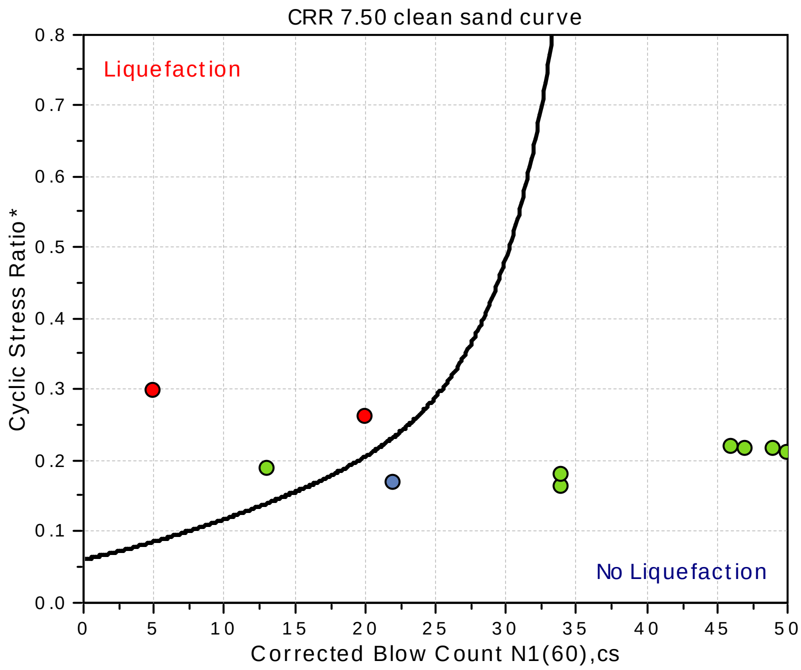
Location : Tei Tei Drive Ohakune

Legend:
Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.36 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		

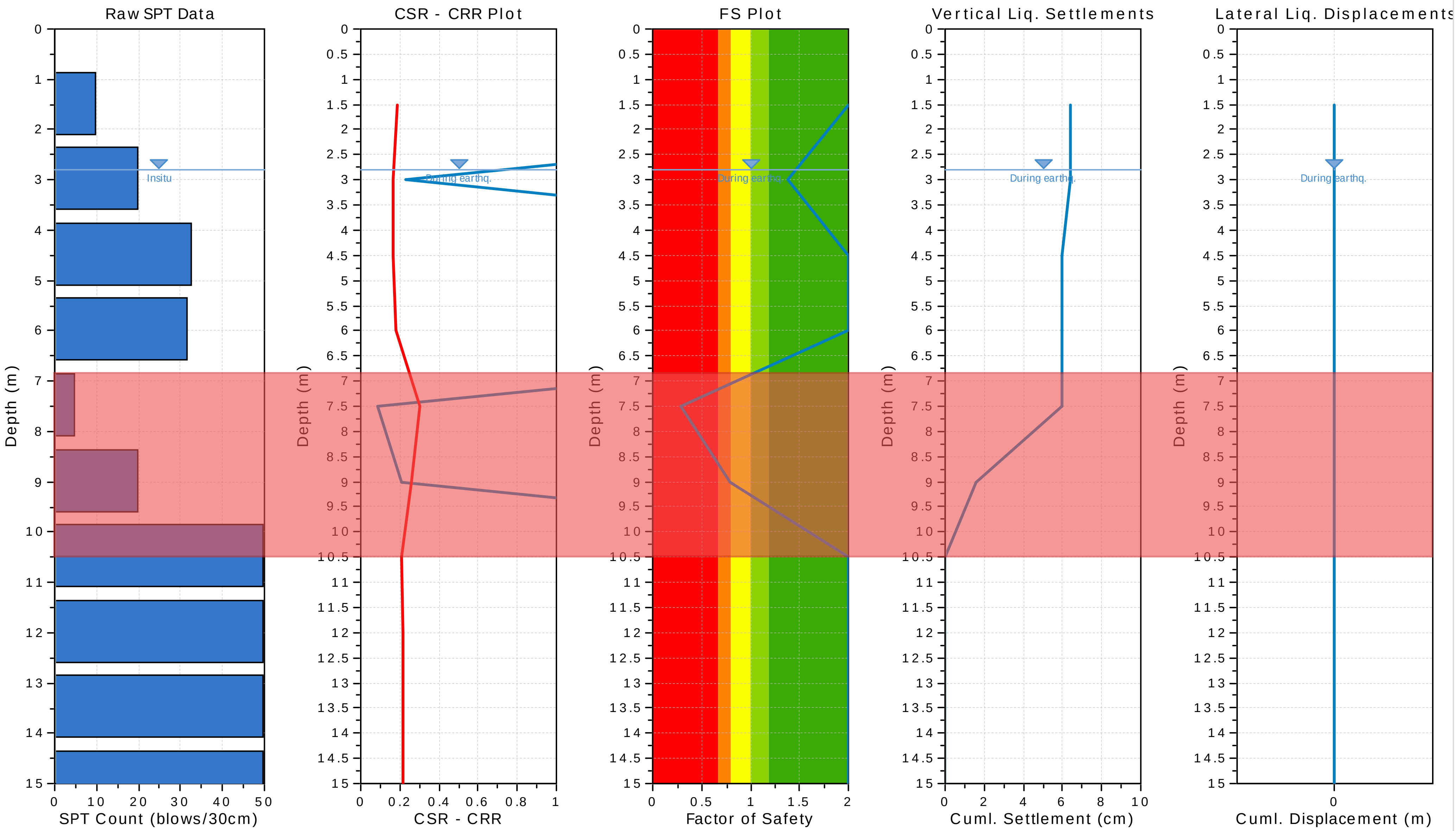




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



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SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH05

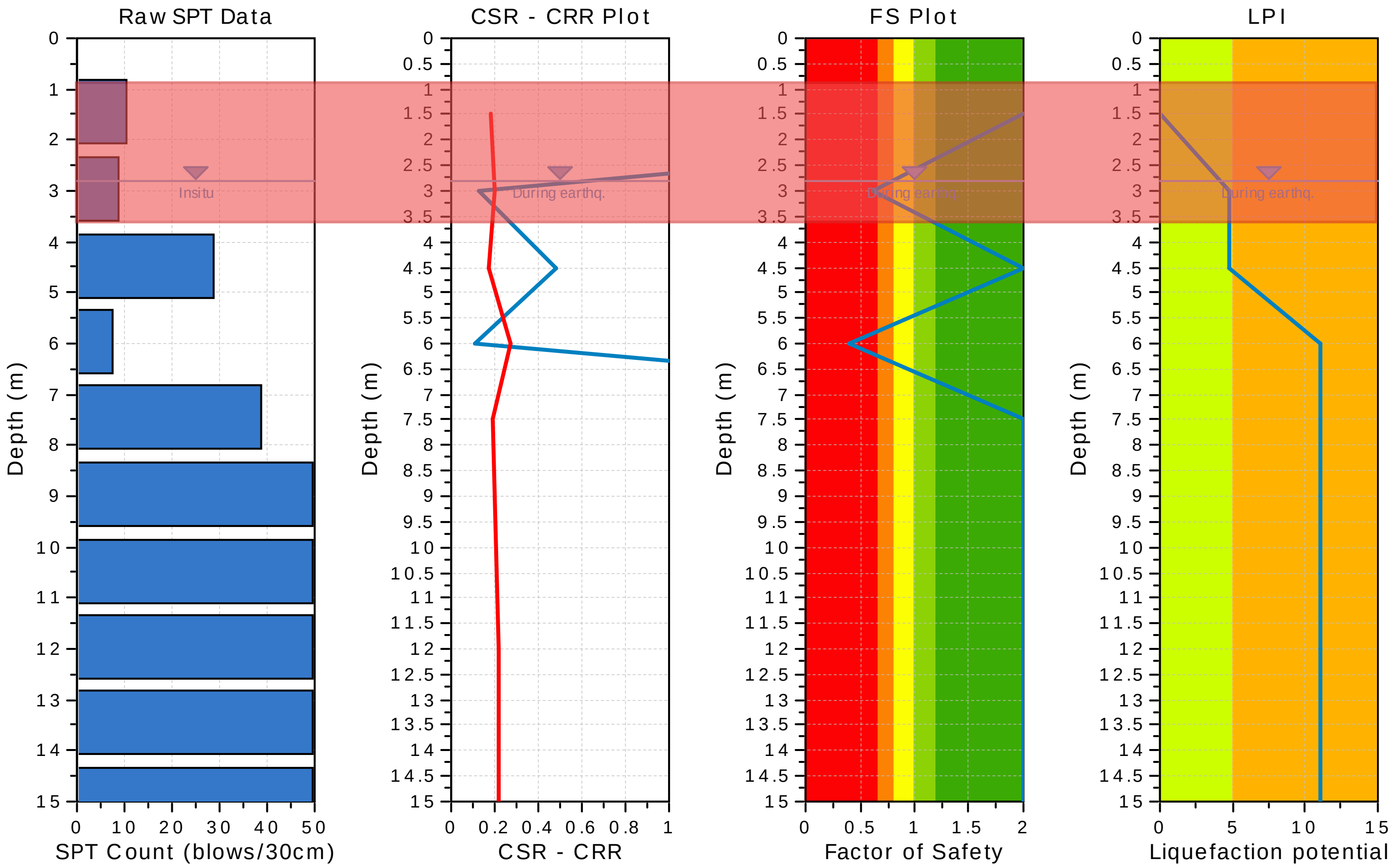
Location : Tei Tei Drive Ohakune

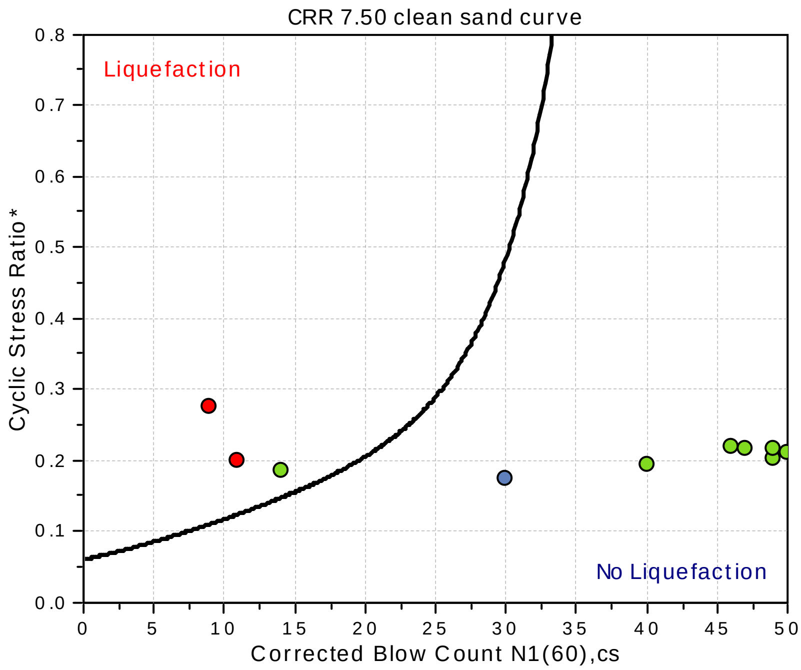
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.36 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





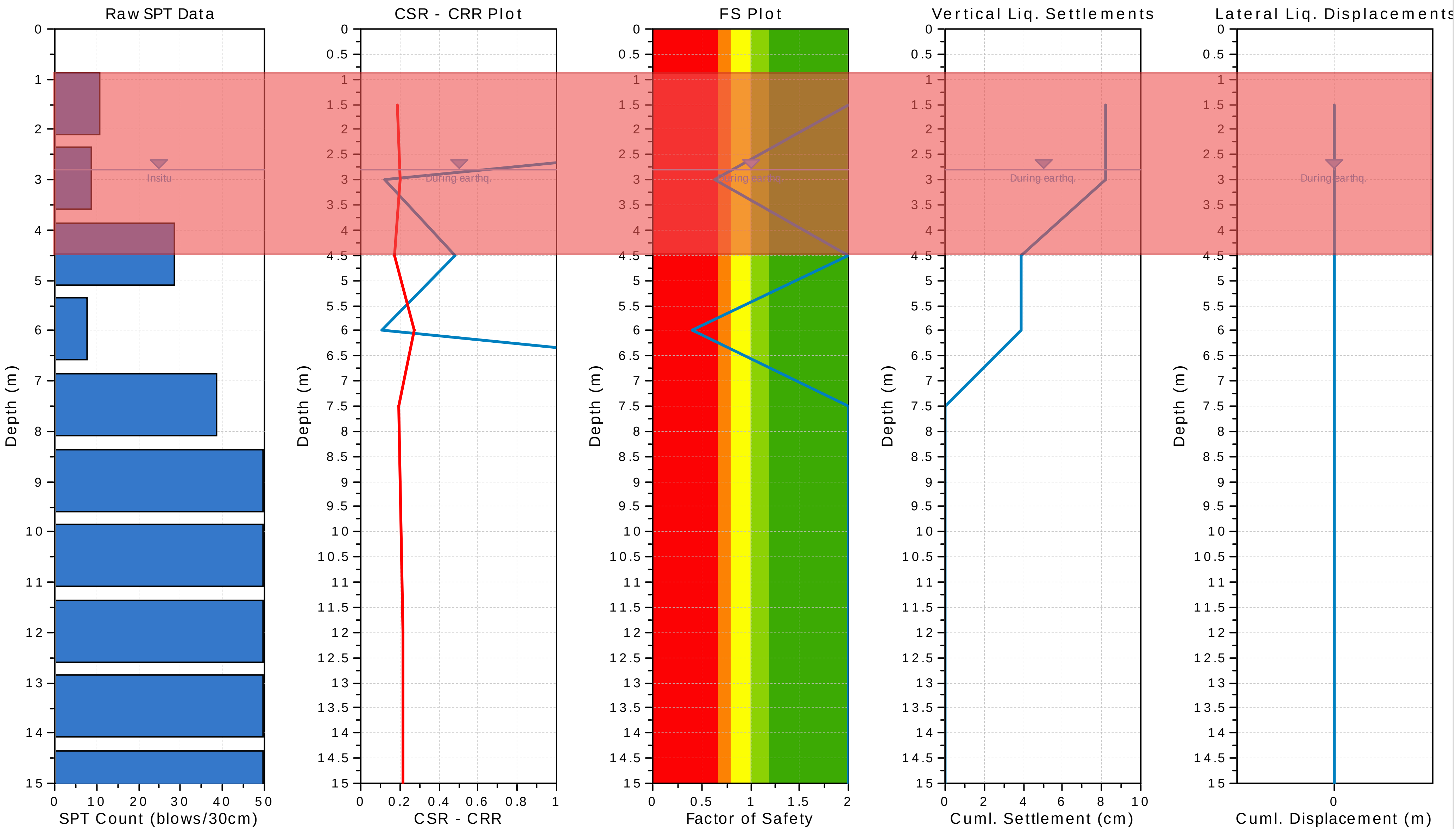
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



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SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH06

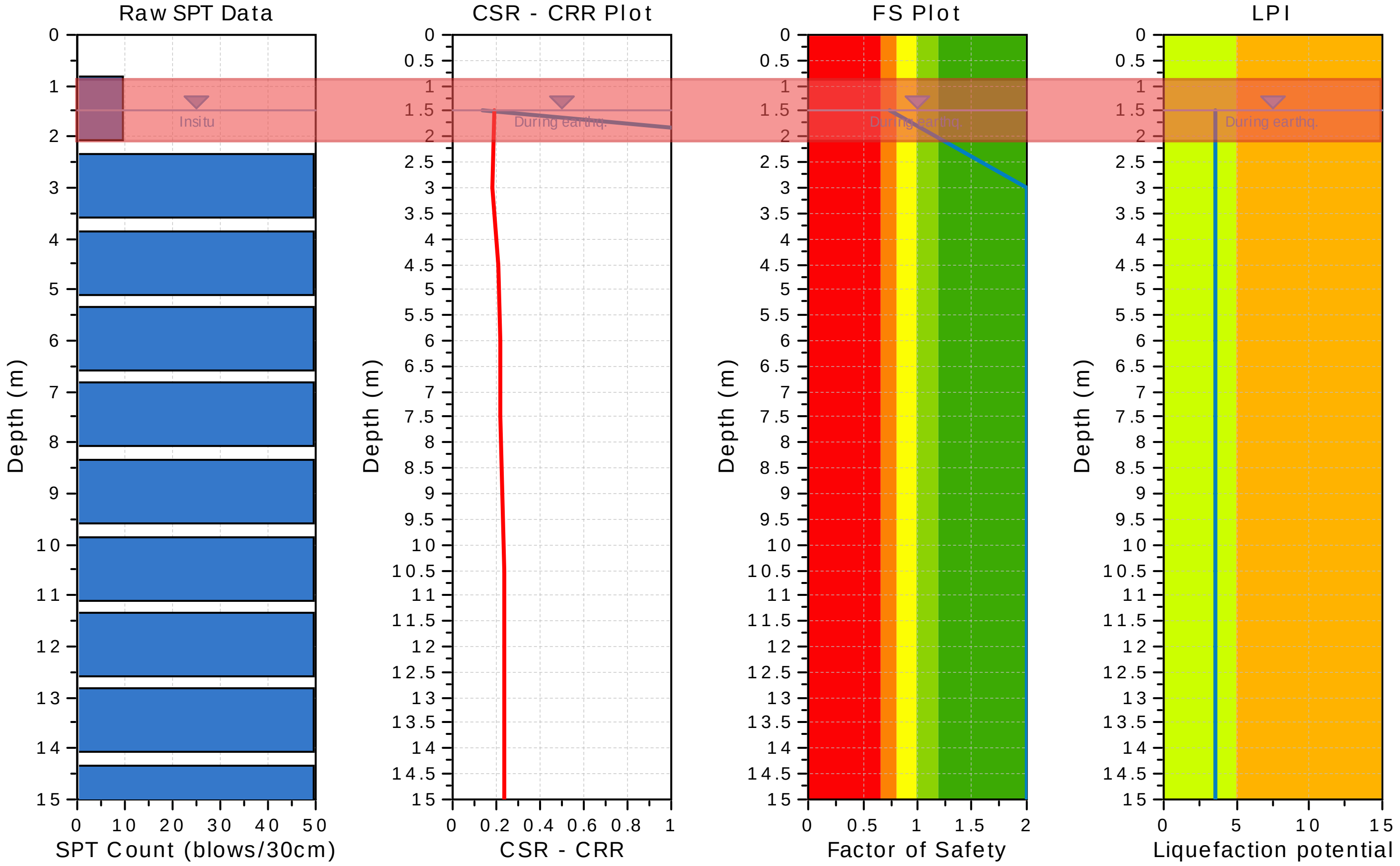
Location : Tei Tei Drive Ohakune

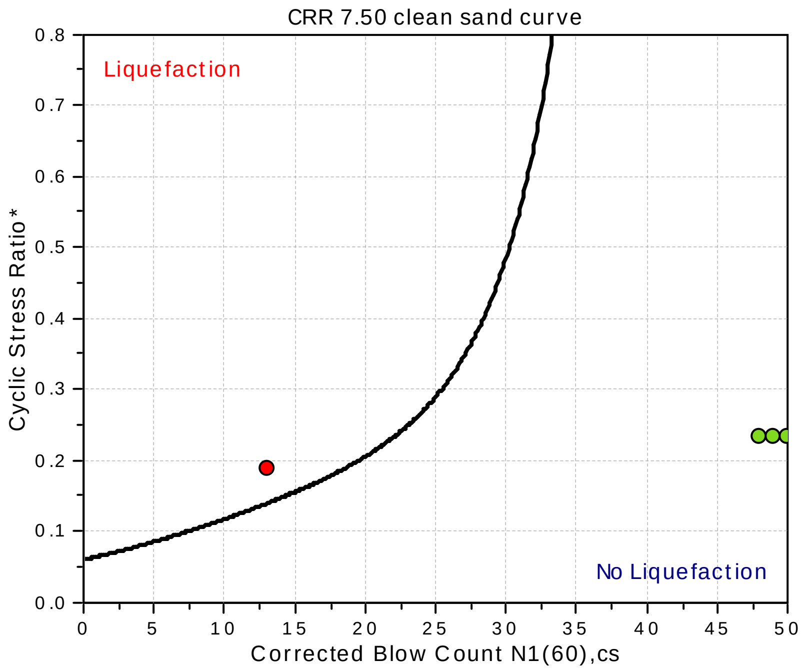
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	1.50 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	1.50 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.36 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





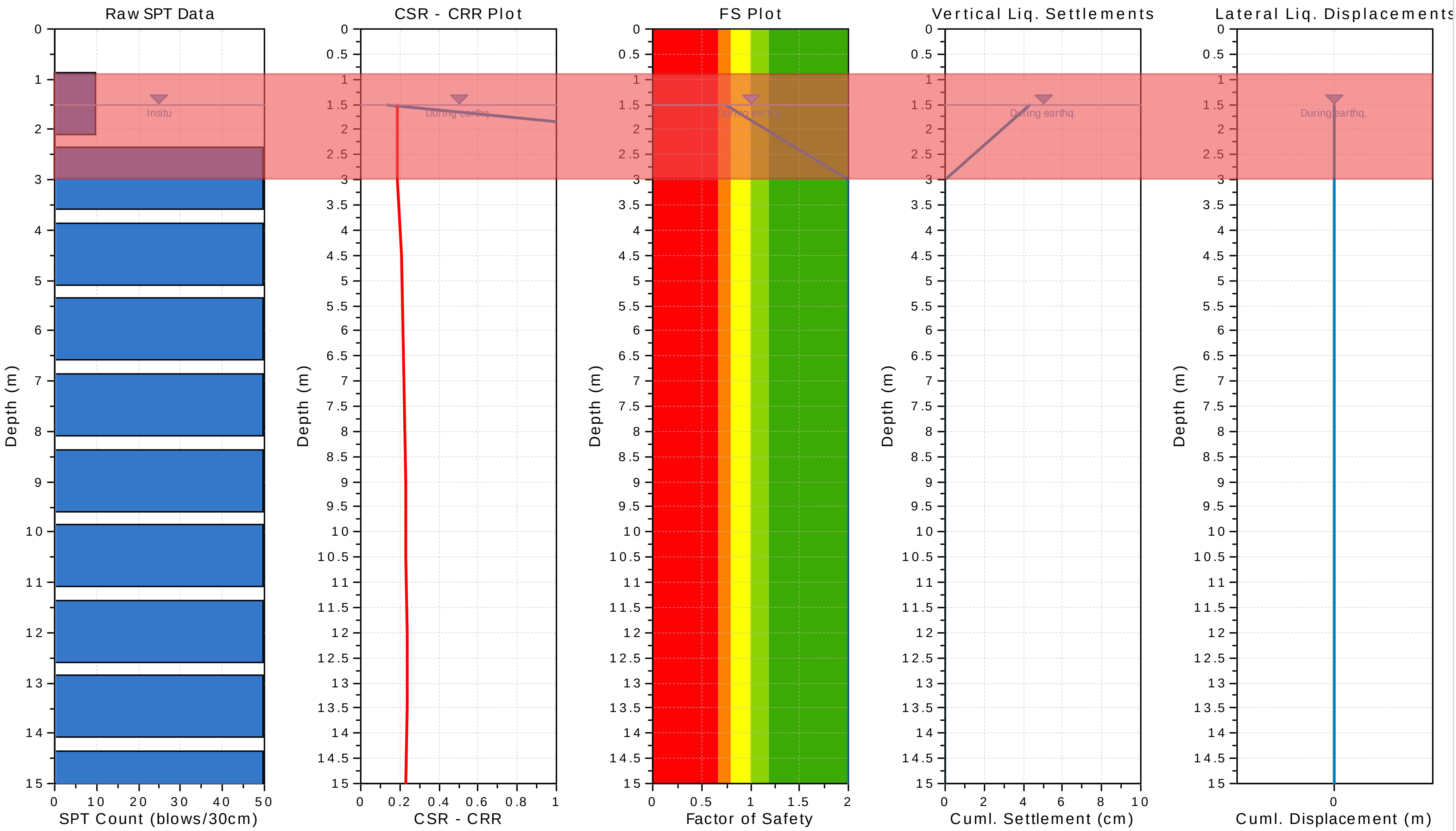
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



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SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH01

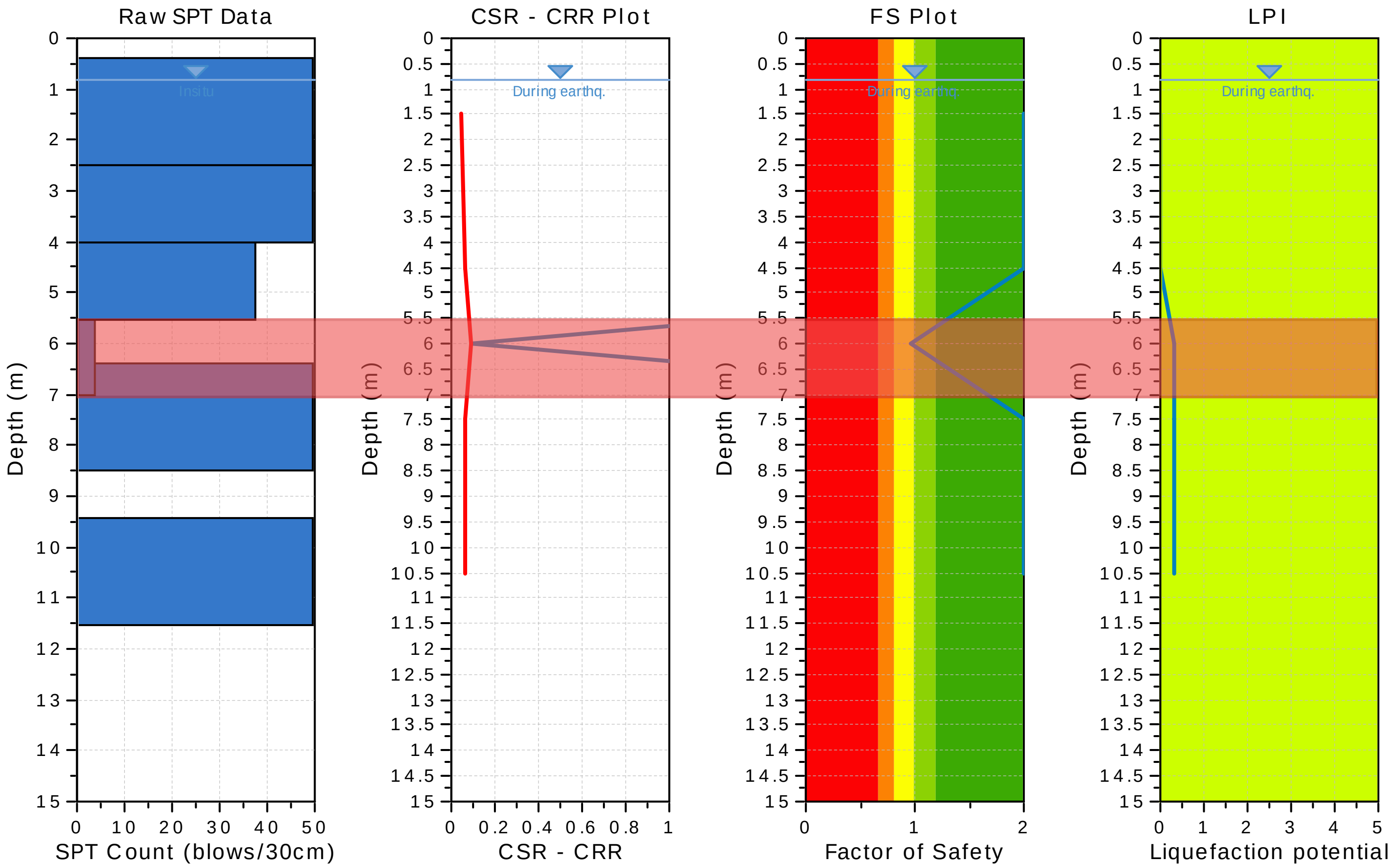
Location : Tei Tei Drive Ohakune

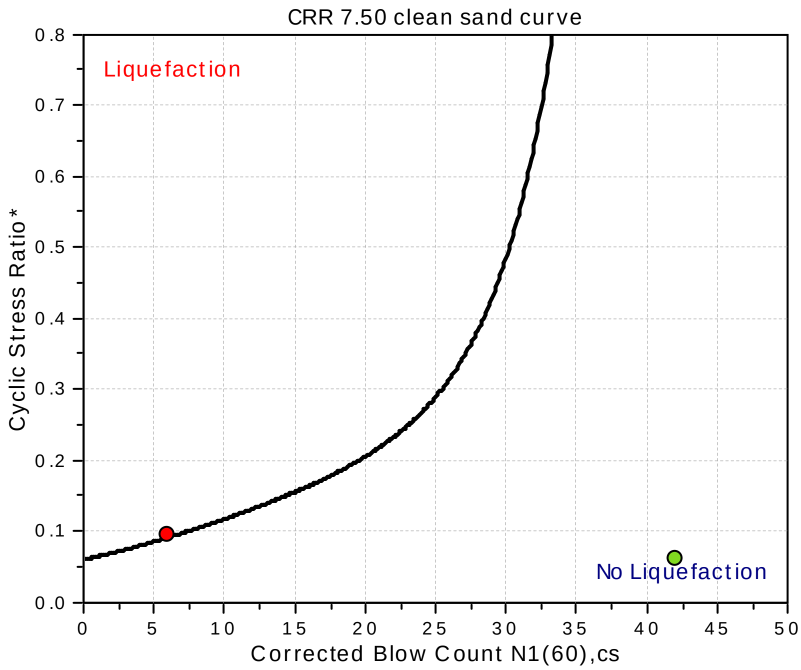
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	0.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	0.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.09 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		

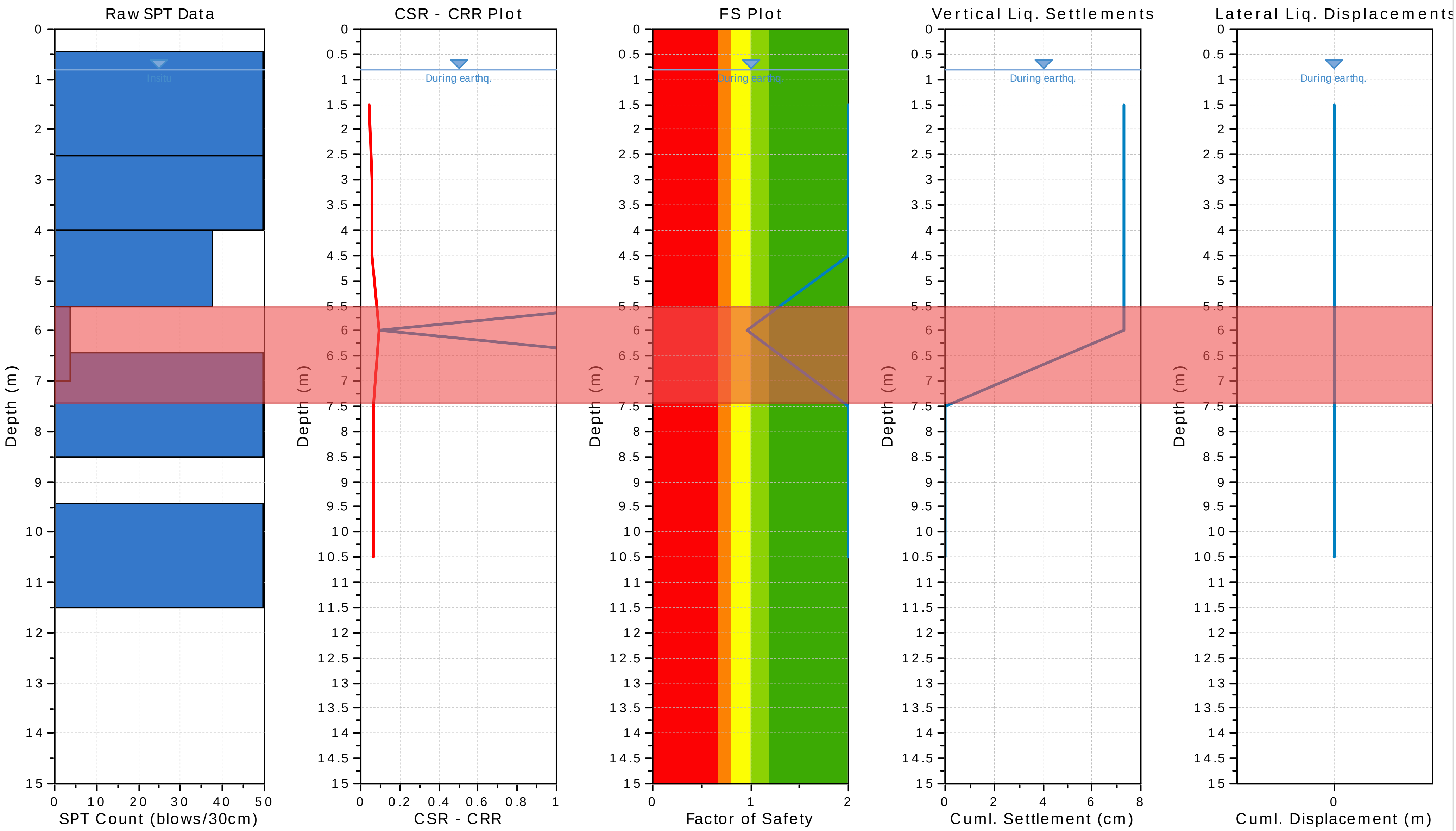




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



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SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH02

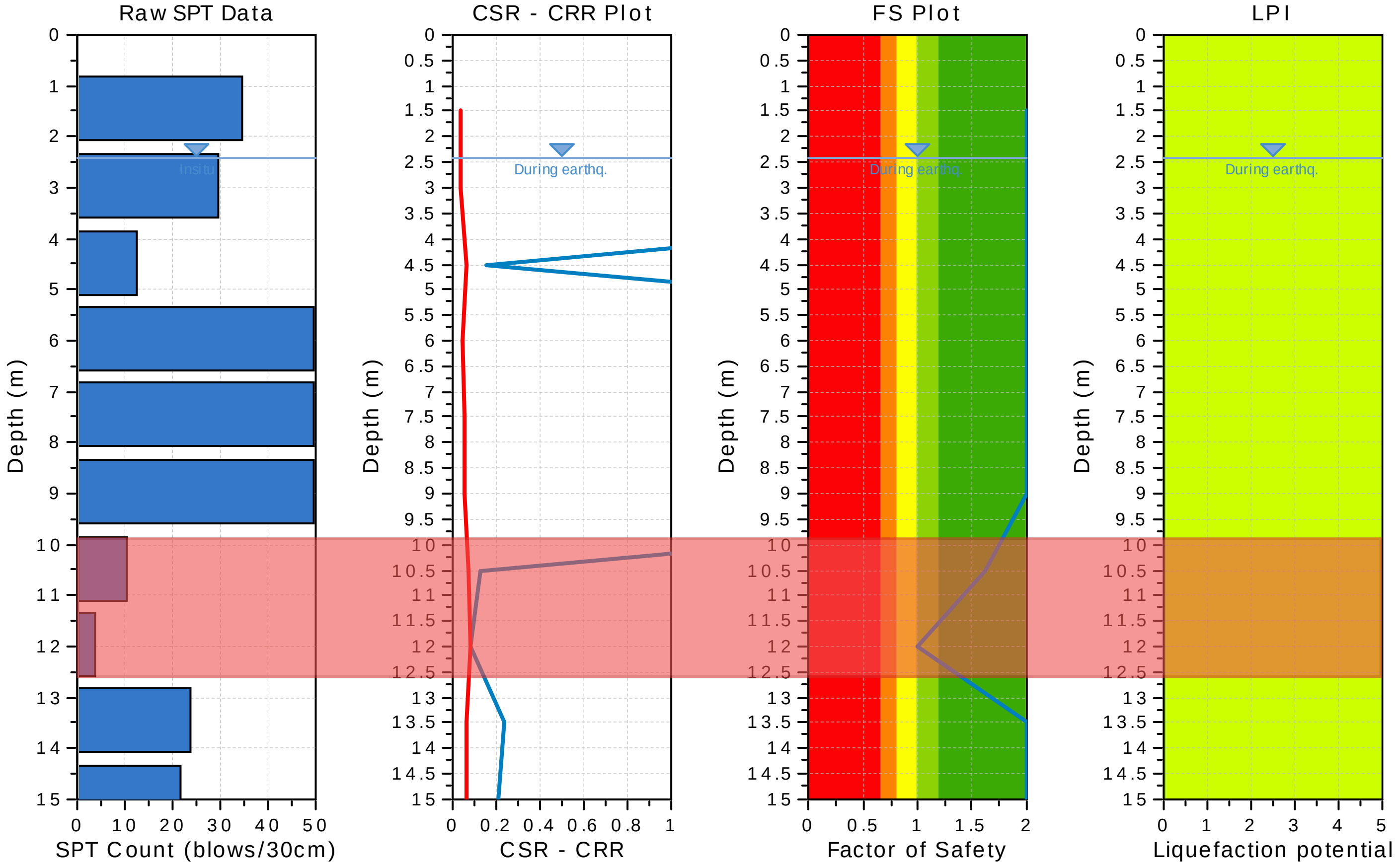
Location : Tei Tei Drive Ohakune

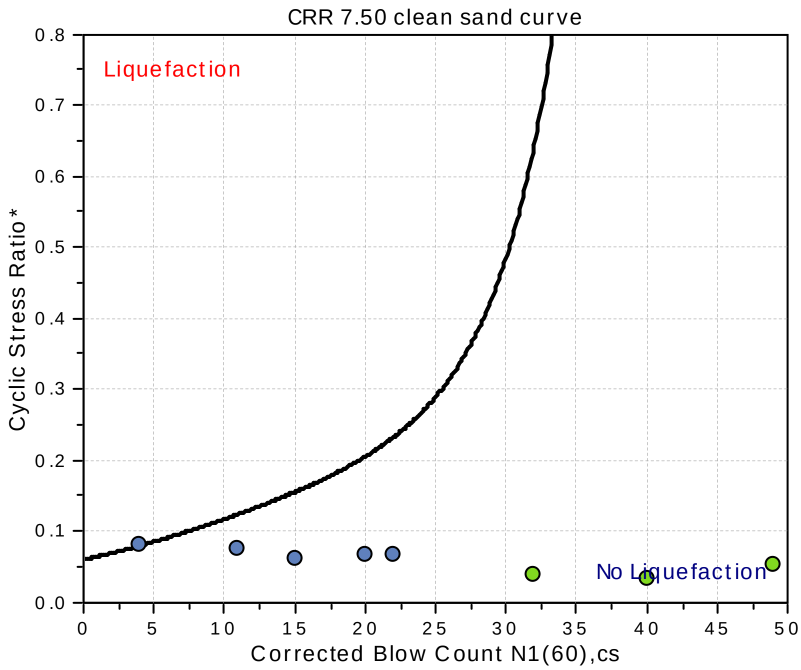
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.40 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.40 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.09 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		

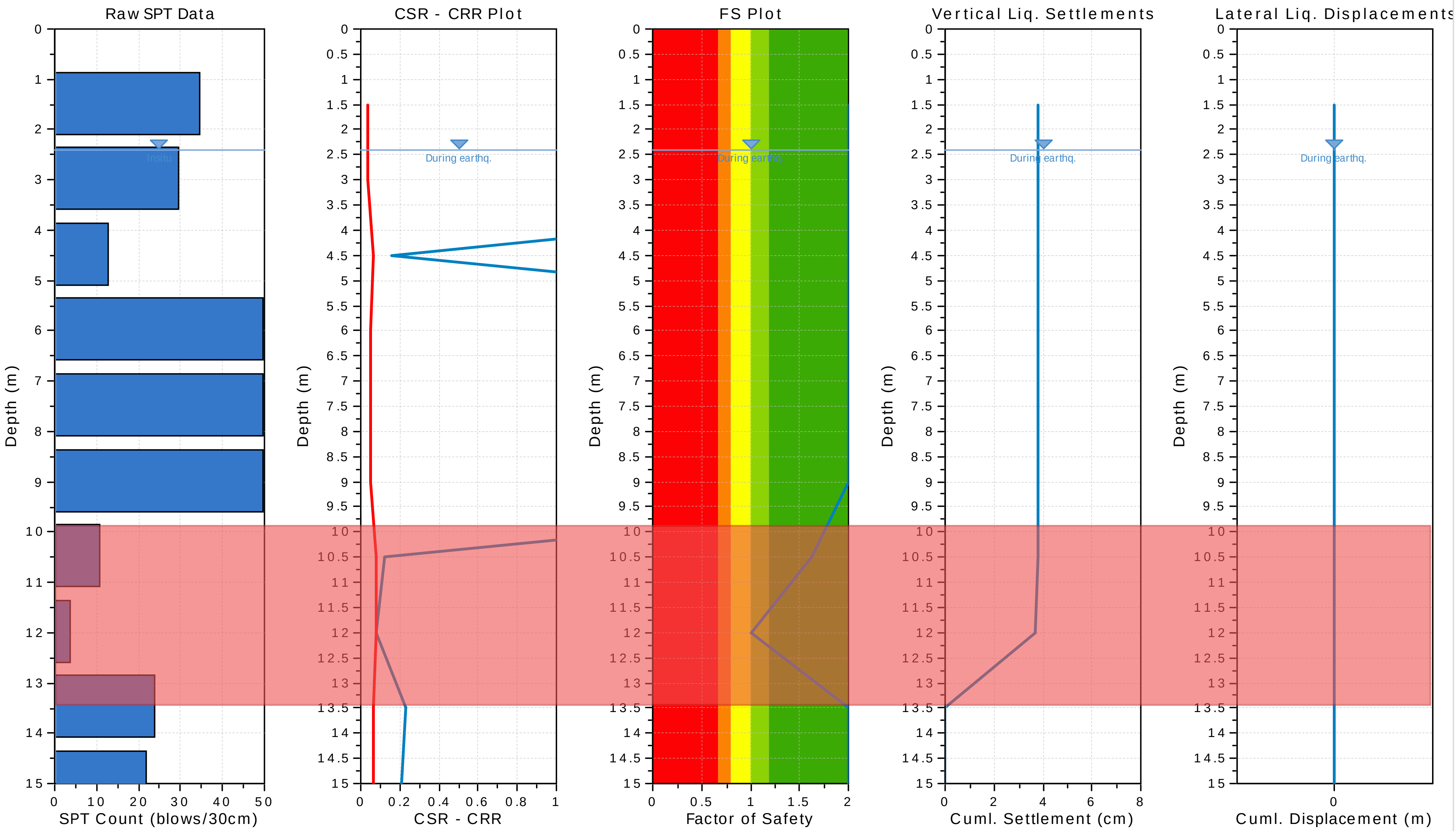




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



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SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH03

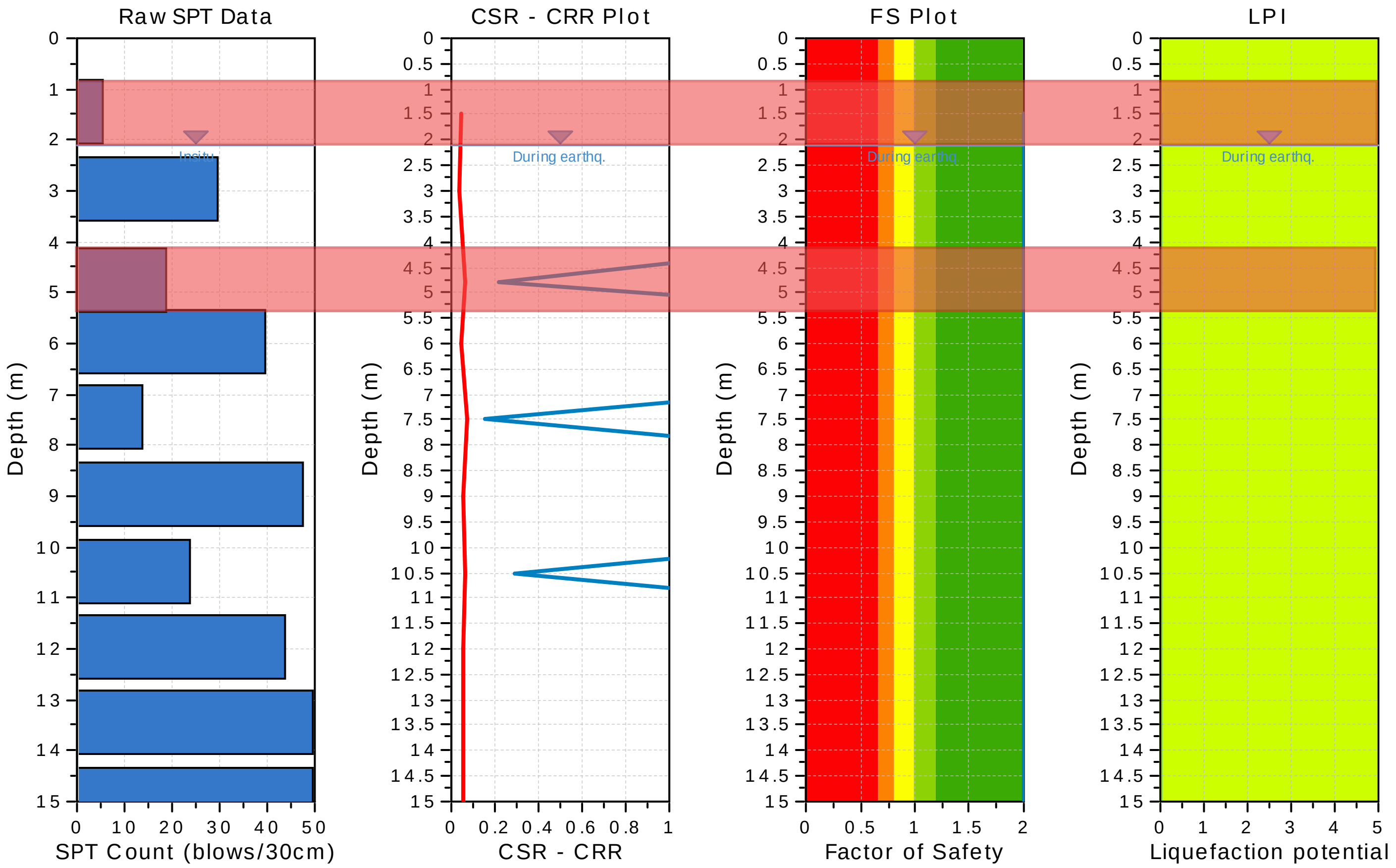
Location : Tei Tei Drive Ohakune

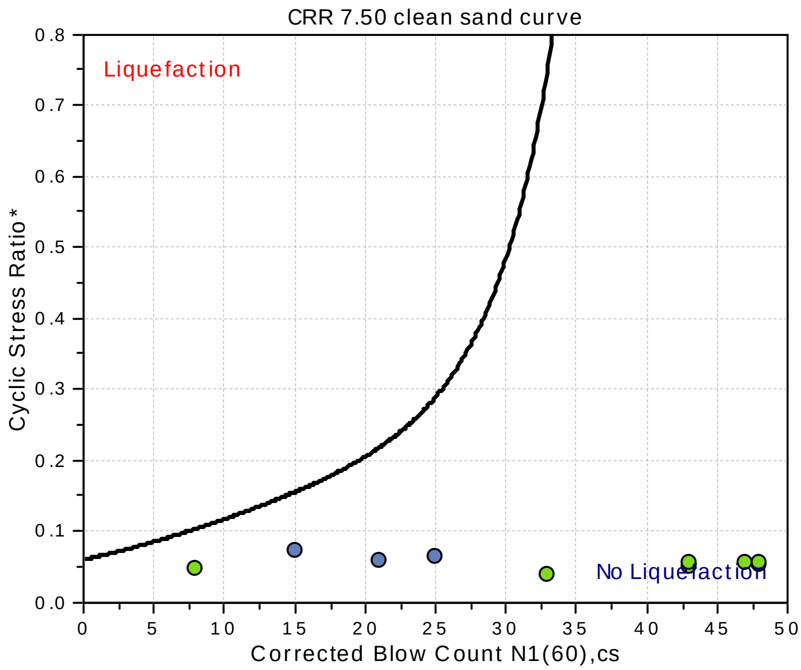
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.10 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.10 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.09 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





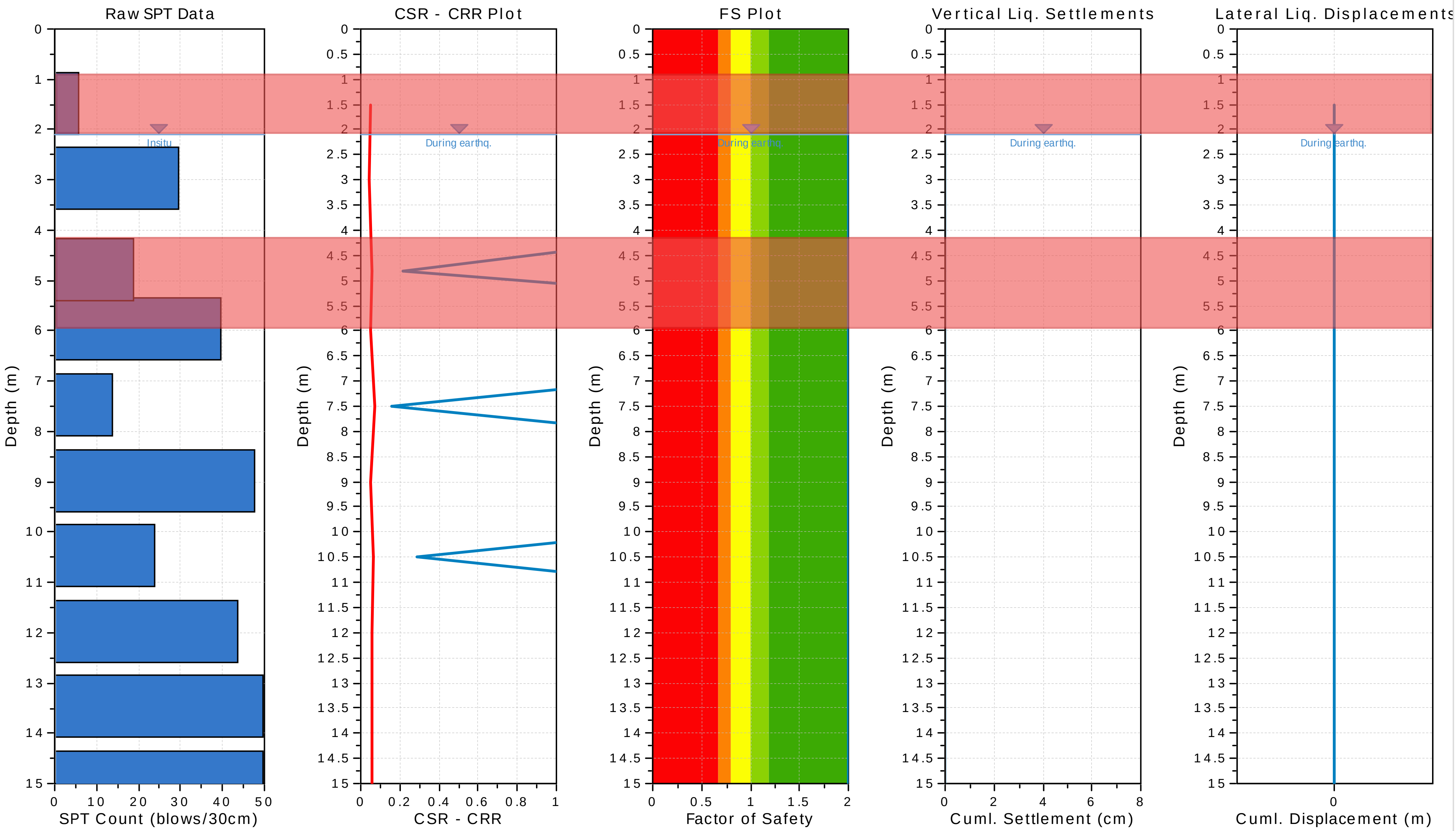
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



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SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH04

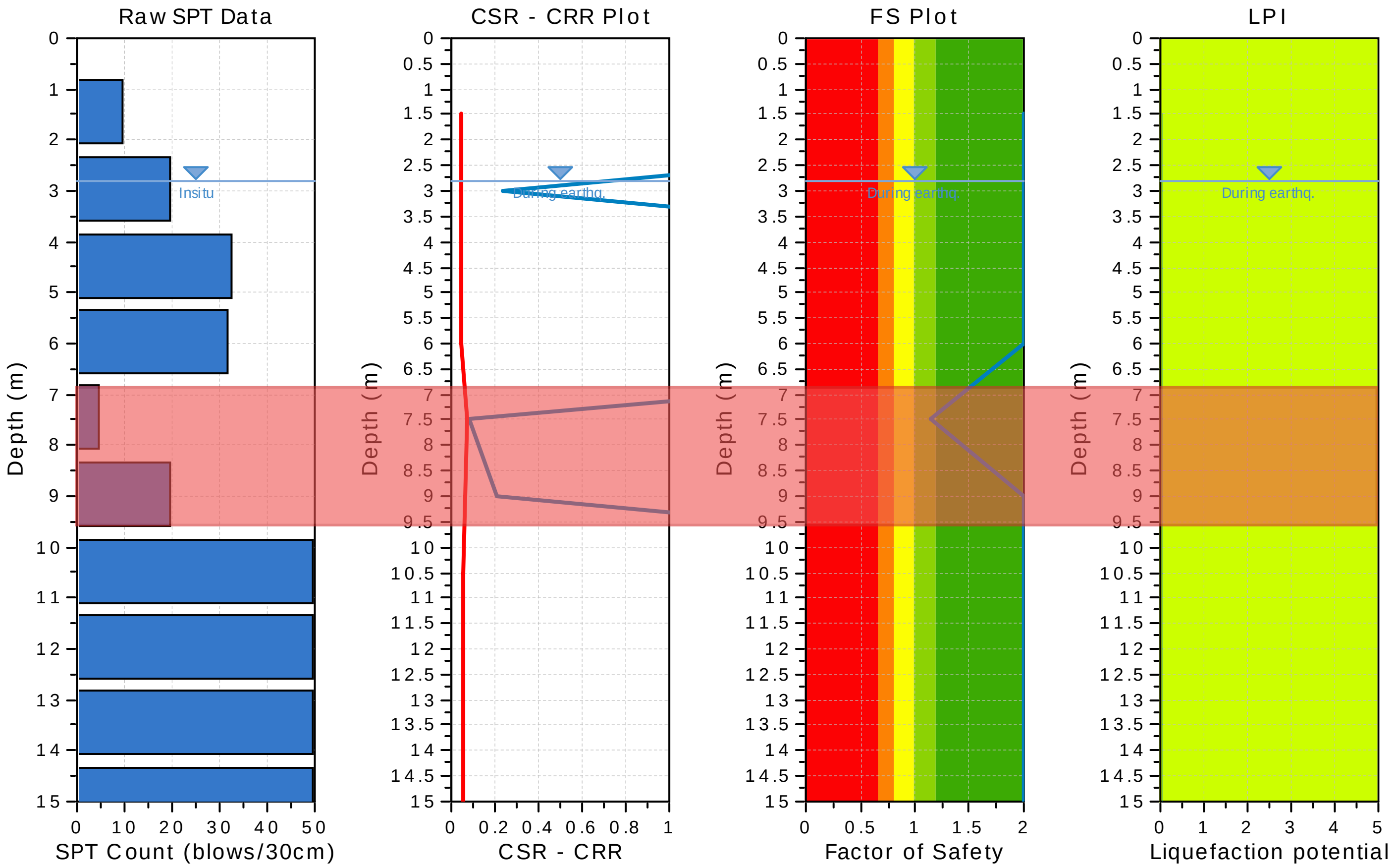
Location : Tei Tei Drive Ohakune

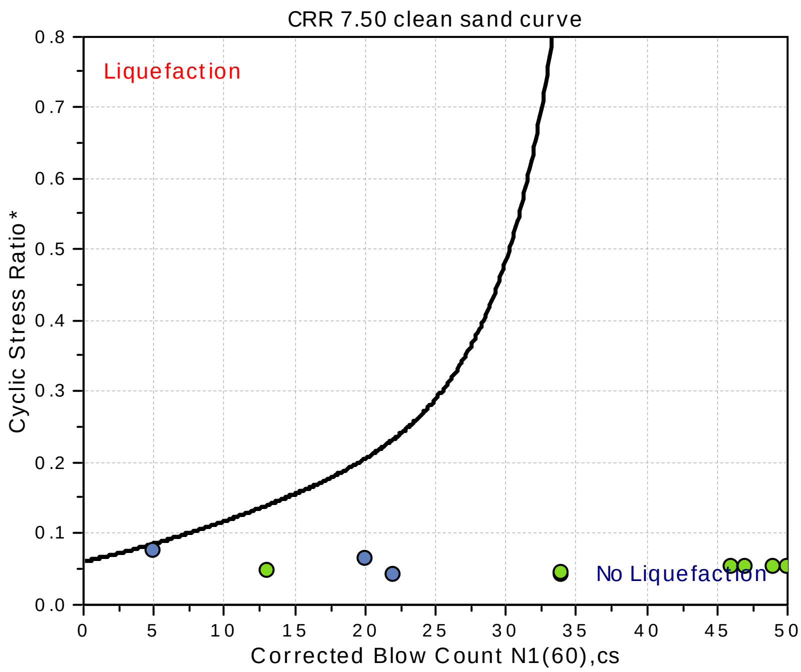
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.09 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





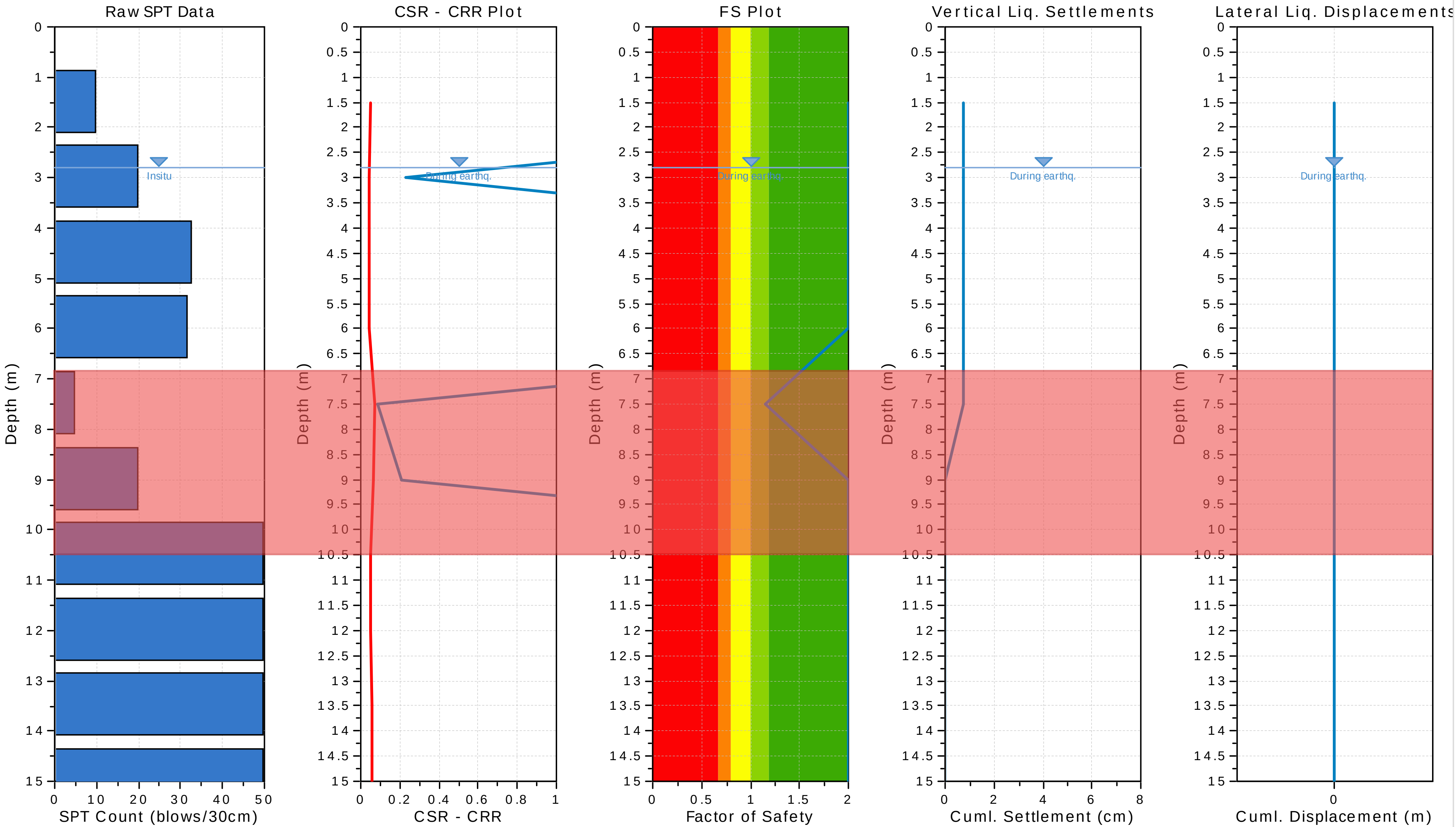
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



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SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH05

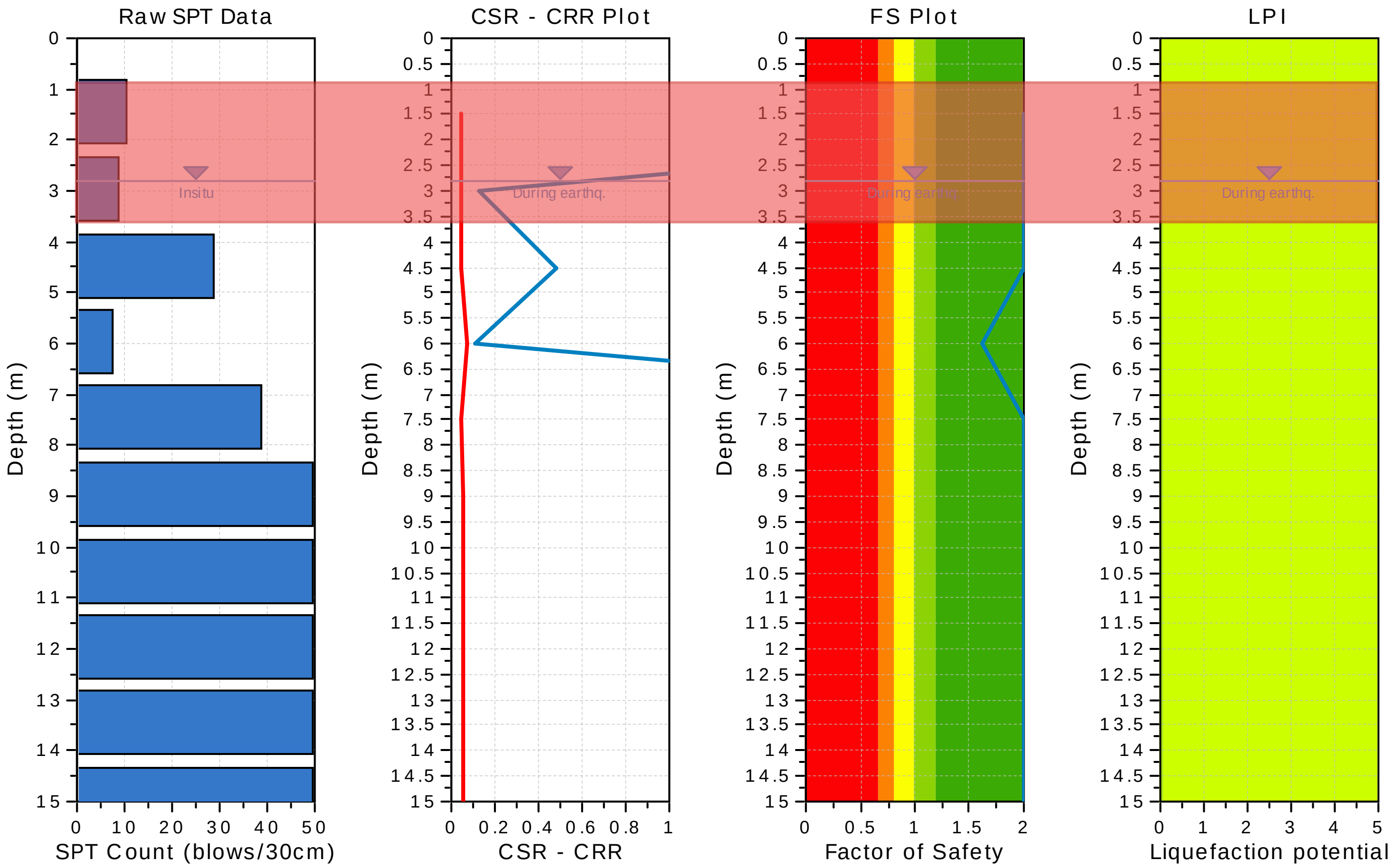
Location : Tei Tei Drive Ohakune

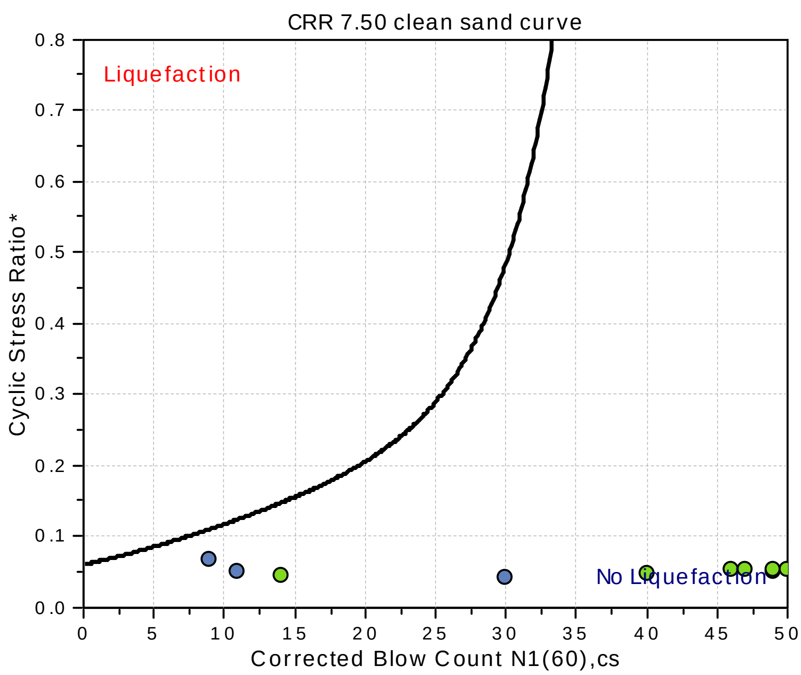
Legend:

Cohesive material not susceptible to liquefaction

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.09 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		

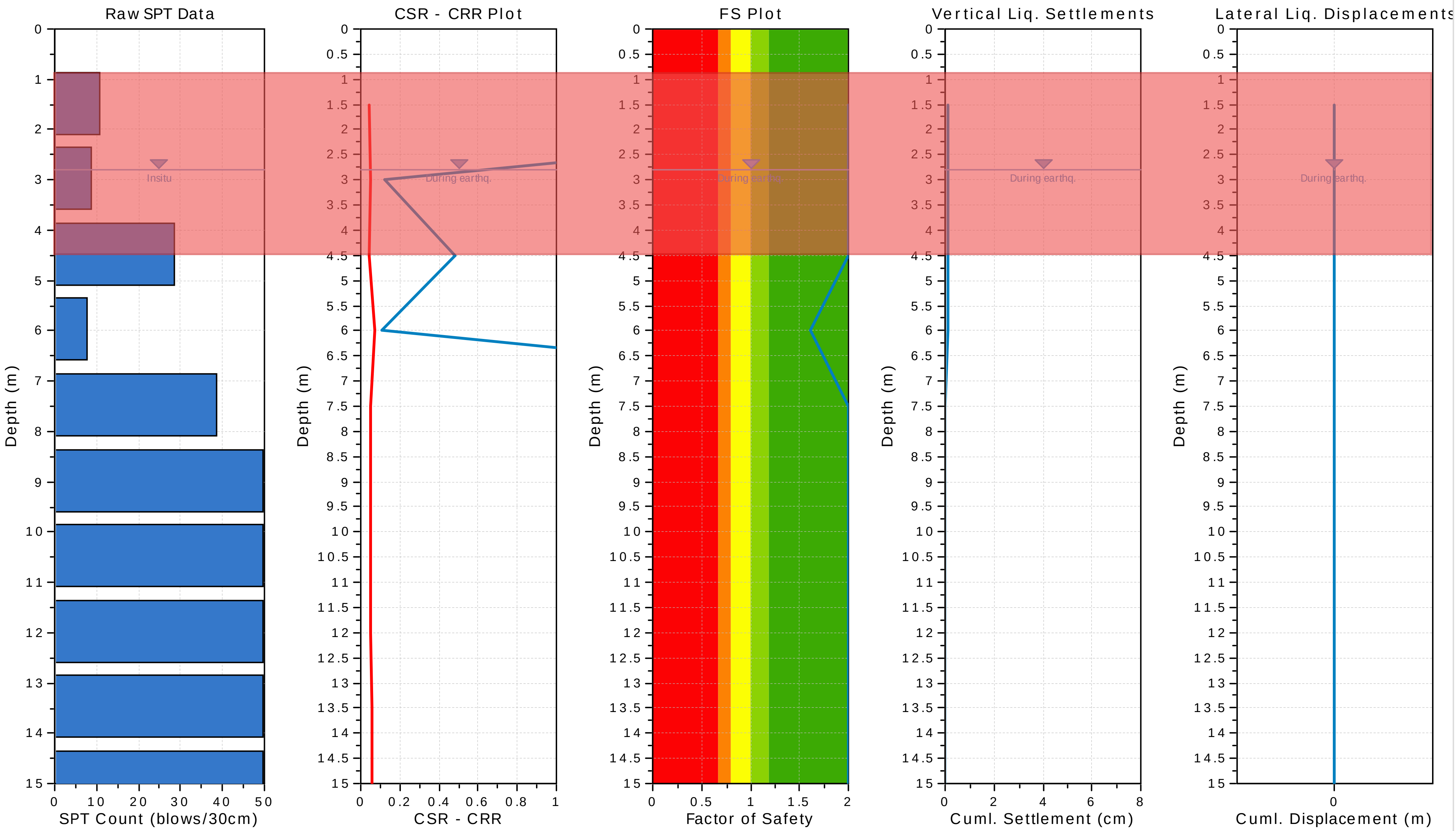




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



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SPT BASED LIQUEFACTION ANALYSIS REPORT

Project title : TGA2022-0238

SPT Name: MBH06

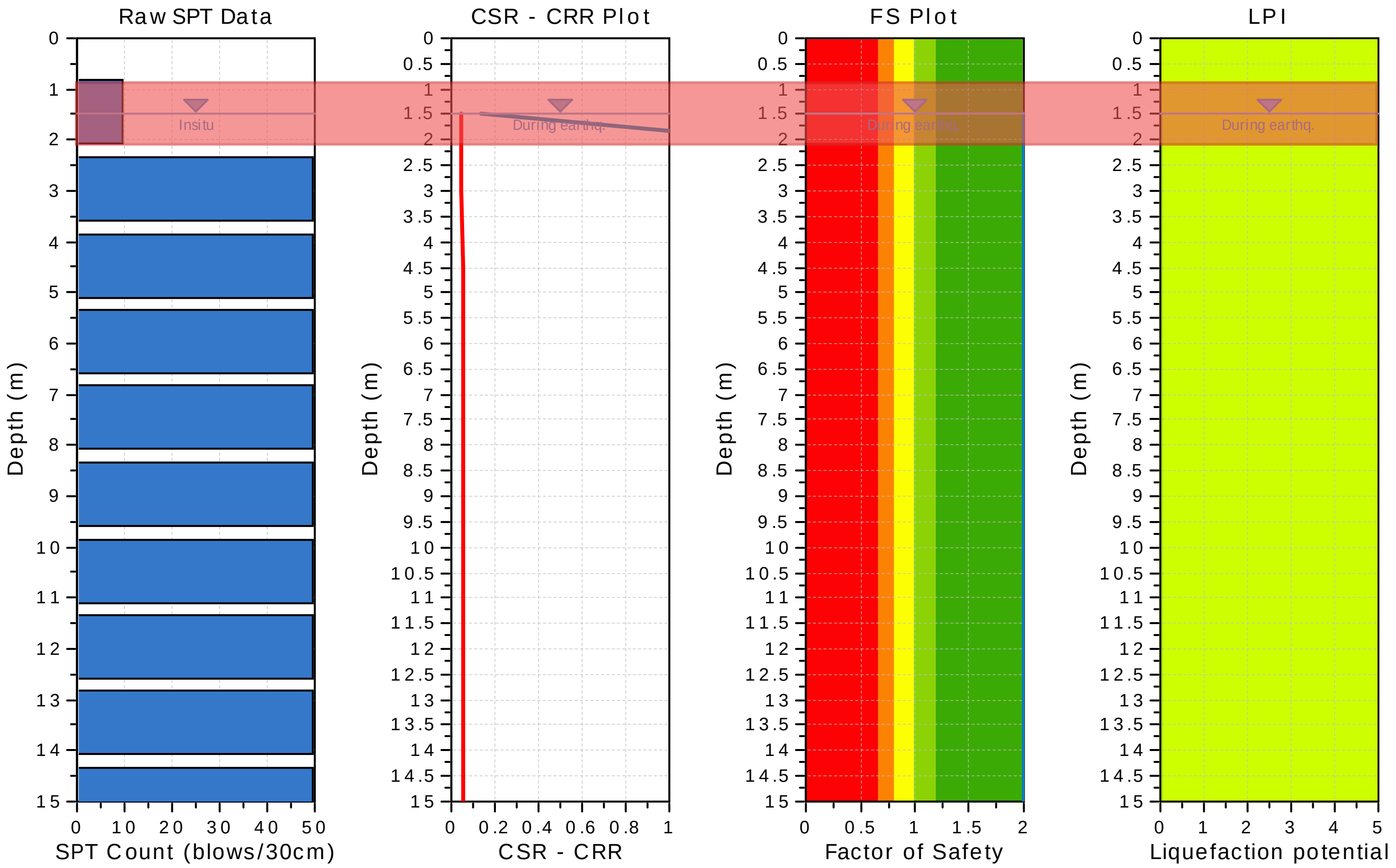
Location : Tei Tei Drive Ohakune

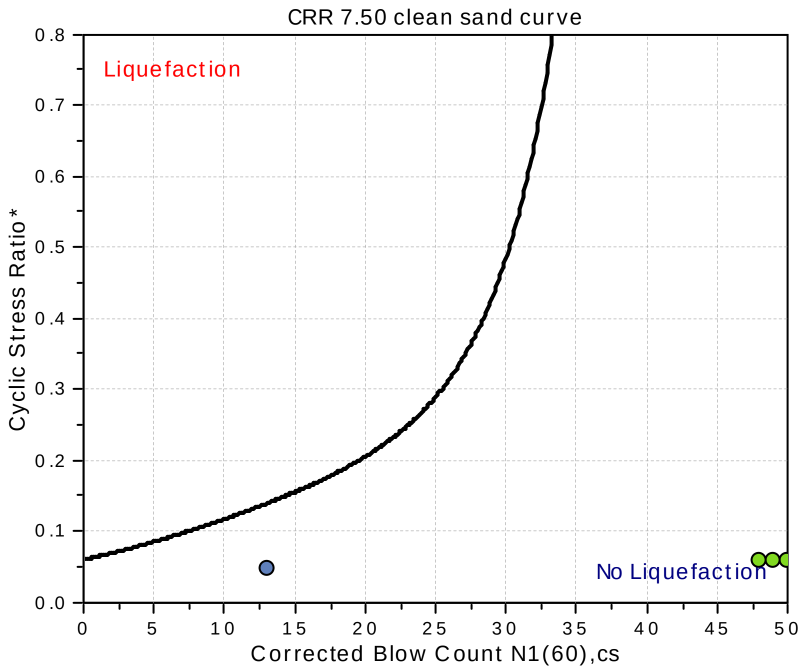
:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	1.50 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	1.50 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.09 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		

Legend:

Cohesive material not susceptible to liquefaction





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

:: Overall Liquefaction Assessment Analysis Plots ::