MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HIKINA WHAKATUTUKI

Worked Examples

Assessment of foundation solutions for residential technical category 3 properties

These worked examples are to accompany revised Section 15.3 and Appendix C4 of MBIE Guidance for repairing and rebuilding houses in the Canterbury region following the 2010-2011 earthquake sequence.

Introduction

The following worked examples illustrate the basic steps involved to use the guidance document to assess a residential property located on Technical Category 3 (TC 3) category land, and select an appropriate foundation/shallow ground improvement solution to mitigate future liquefaction settlement in order to meet the general requirements of the Building Code. **These examples are hypothetical and not specific to a particular property. Further, the solutions provided represent only one of several potential possibilities, and are for illustrative purposes only**. This document is intended only to supplement the oral training presentations provided by MBIE on 30 April 2015 and is subject to change.

The guidance documents are intended to provide a relatively simple ground assessment/foundation selection framework, and therefore may in some cases (particularly the deep ground improvement options), on average result in more conservative solutions than might be achieved with specific engineering design. However, it is recognised that the Design Engineer may wish to consider factors not explicitly addressed in the guidance where doing so might achieve significant cost savings over a simple guidance solution. Examples of such factors and their application through engineering judgement are included as commentary at the end of the worked examples.

Key design considerations

There are many items to be considered as part of geotechnical assessment/foundation design for a TC3 property. The following are some of the key considerations to keep in mind when performing a geotechnical assessment for residential foundation design:

a) **Available information:** In Christchurch, a significant amount of geotechnical investigation information, reports and maps is available on the Canterbury Geotechnical Database (CGD) accessible at https://canterburygeotechnicaldatabase.projectorbit.com.

b) **Land/structure performance during CES:** Observations of the land and building/foundation damage may help inform the assessment process for a particular site.

c) **Ground investigations:** The guidance document outlines recommendations for what constitutes adequate site-specific ground investigations (refer to Sections 13 and 15.3.4).

d) **Groundwater depth:** Groundwater depth is used for liquefaction triggering and settlement calculations. The predicted performance of the site, and therefore the ground improvement, may be quite sensitive to the groundwater depth assumed for design.

e) **Liquefaction assessment procedure:** There was a 2014 update (by the authors) to the Idriss and Boulanger (2008) liquefaction triggering assessment procedure for CPTs. The updated procedure (referred to herein as BI2014) is used for the worked examples. Q&A 50 and 51 on the MBIE website (Clarifications and updates to the Guidance 'Repairing and rebuilding houses affected by the Canterbury earthquakes' – Issue 7, October 2014) also discuss the use of BI2014

f) Constructability: The ground improvement options outlined in Section 15.3 and Appendix C4 of the guidance document have different constraints related to noise, vibration, access and excavation below groundwater which need to be taken into account for a particular site.

g) Total settlement: Liquefaction-induced ground surface settlement will result in lower post-event floor levels. Therefore, the Design Engineer is encouraged to consider the potential effects of total as well as differential liquefaction-induced settlement. For example, might the predicted liquefaction-induced settlement result in the floor level being substantially below design flood level?

Typical steps in site-specific geotechnical assessment for a TC 3 residential property

Following are the typical steps taken as part of a site-specific geotechnical assessment for a TC 3 property where liquefaction mitigation is presumed to be required:

1. Desktop study
2. Site walkover
3. Site-specific geotechnical investigation
4. Liquefaction triggering assessment
5. Liquefaction settlement analysis
6. Selection of foundation/ground improvement system

The worked examples discussed below follow these general steps.

Worked example 1

Desktop study

The purpose of the desktop study is to determine some basic information about the site with respect to subsurface conditions, land/structure performance during the CES, levels of ground shaking during the CES and other information that may help inform the overall geotechnical assessment. Most of the information for a desktop study for sites in Christchurch is expected to be available on the CGD. However, the property owner, their insurer or others working on the project may also have useful information.

Technical category

The example property located well within TC3; however, many TC3 properties will be located near boundaries with TC2 land. The boundaries are lines on a map for the purposes of categorising land based on its generally assessed susceptibility to liquefaction. However, any given property may have ground conditions that are different from its TC category. Figure 1 shows the example site area within the TC3 boundary.

Figure 1. Example site area identified as TC 3 land

Nearby Geotechnical investigations

Figure 2 shows the Borehole (BH) and Cone Penetration Test (CPT) locations within the vicinity of the project site as shown on the CGD. A large number of ground investigations have been carried out in TC3 areas, and it is possible that a particular property may already have one or more investigations located on it. This information can help inform a general characterisation of the ground conditions in the project vicinity.

Figure 2. CGD screenshot showing existing geotechnical investigations

Black maps, geological maps and LiDAR

The greater Christchurch area contains various streams, rivers, swamps and estuary land. Some of these are still present in the landscape; however, many others have been covered over. These features can have a significant impact on the seismic performance of land. Black maps, geological maps and LiDAR elevation models are useful tools to help determine whether the project site may be located within or near one or more of these features. Figure 3 shows a portion of eastern Christchurch with some of the features that might be present on a property. For this example, it is assumed that no buried streams or swamp/estuary land exist on the site.

Figure 4 shows a 1:25000 geological map of Christchurch in the east. For this example, the subject property is assumed to be underlain by dominantly sand of fixed and semi fixed dunes and beaches of the Christchurch formation.

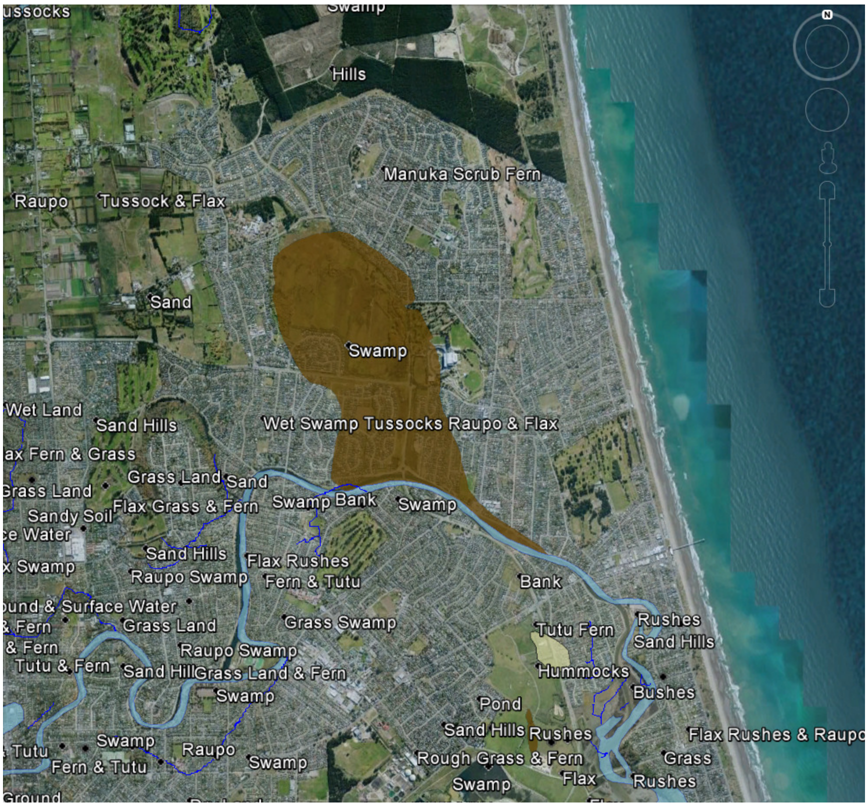


Figure 3. Screenshot of the Black Maps layer on the CGD

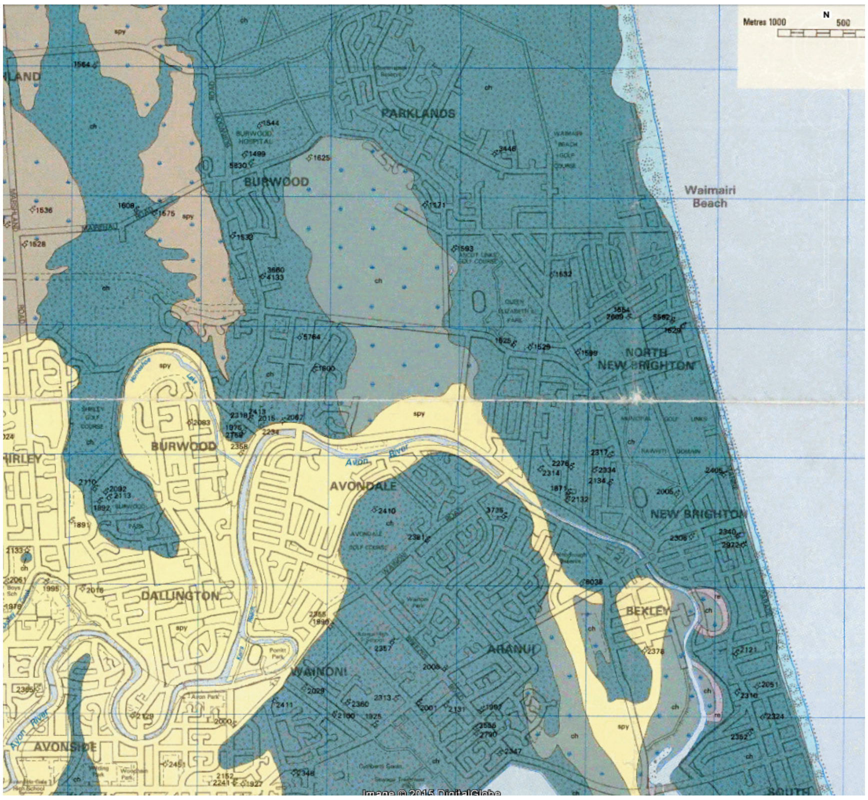


Figure 4. Christchurch 1:25000 geological map (Brown and Weber, 1992), east Christchurch

LiDAR can help highlight features within the site area that may impact seismic ground performance such stream channels and general slope of the ground around the site. Figure 5 shows an example of LiDAR from September 2011 (1 m raster).

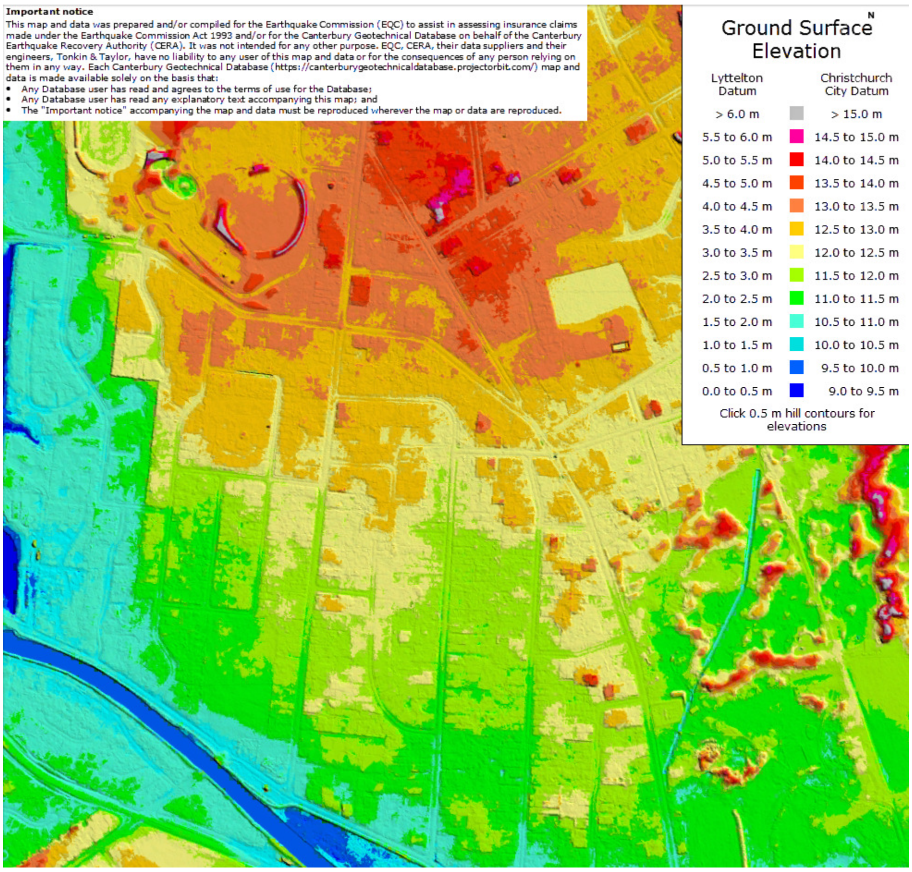


Figure 5. September 2011 (1m raster) LiDAR digital elevation model

Estimated ground shaking during CES events

As part of the site assessment, it is useful to consider what level of ground shaking likely occurred in the project site area. The CGD presents contours of conditional median horizontal peak ground acceleration (PGA) for the main CES events (from Bradley and Hughes, 2012). This information can be used to help assess the liquefaction triggering that may have occurred in a given earthquake. The PGA contours are associated with a conditional standard deviation to reflect the uncertainty associated with the contour model.

Figure 6 presents the PGA contours and standard deviations across the example site area in the 22 February 2011 event.

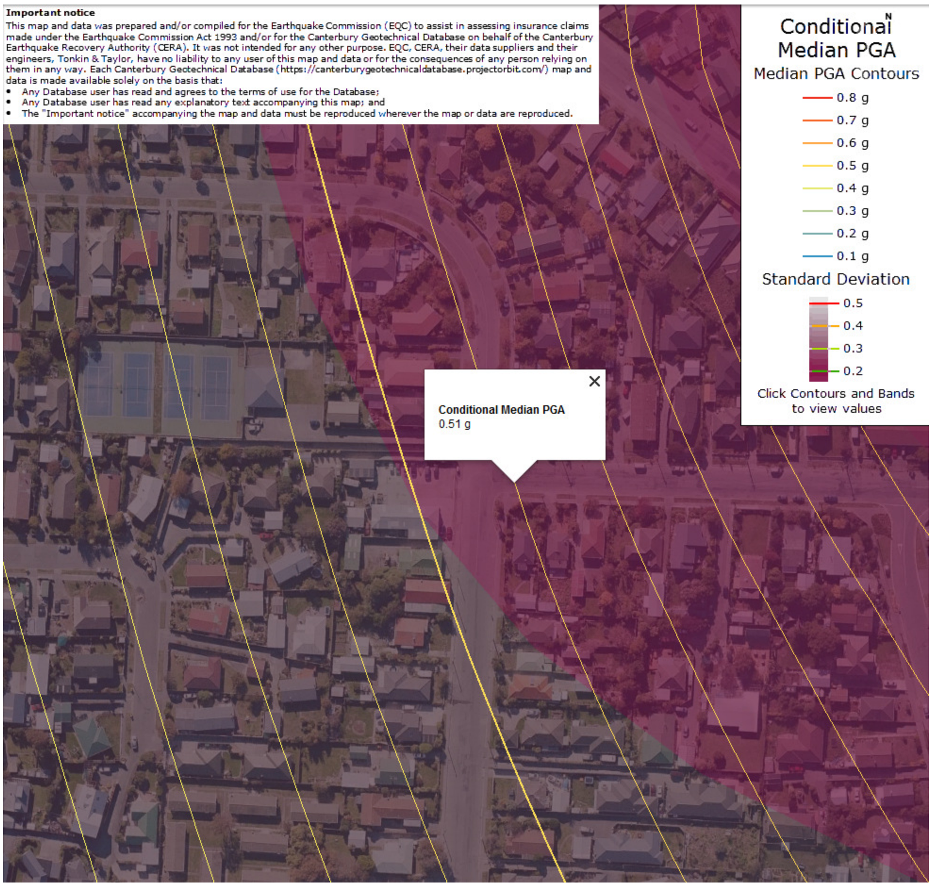


Figure 6. Bradley and Hughes (2012) contours of conditional median Peak Ground Accelerations (PGA) for liquefaction assessment for the February earthquake

The conditional PGAs and standard deviations for the example site, for each of the four main CES earthquakes are summarised in Table 1.

Table 1 – Event magnitude, peak ground acceleration and conditional standard deviation values for 4 main CES events (Bradley and Hughes, 2012)

Earthquake event	Magnitude, M	Peak ground acceleration, PGA	Conditional standard deviation
September 2010	7.1	0.18g	0.250
February 2011	6.2	0.51g	0.250
June 2011	6.0	0.23g	0.275
December 2011	5.9	0.36g	0.375

Aerial photos

Aerial photos taken of Christchurch after each of the four main earthquakes can be useful for helping to assess whether surface manifestation of liquefaction (ie, ejecta, ground cracking, slumping, etc) occurred on or near the project site. Figure 7 shows the aerial photograph of the example site area from 4 September 2010, taken several hours after the earthquake. No liquefaction ejecta is visible on the roads or properties. Figure 8 shows the aerial photograph of the same area taken on 24 February 2011; two days after the February earthquake. Notable liquefaction ejecta is visible on the roads and tennis courts; including the road in front of the example project site.

Figure 7. Aerial photo taken 4 September 2010 post earthquake

Figure 8. Aerial photo taken 24 February 2011

Land damage maps

Observations of surface manifestation of liquefaction were mapped (residential property only – shown on the CGD) by the EQC based on both interpretation of aerial photographs taken shortly after the four main earthquakes, and on ground observations from EQC personnel canvassing neighbourhoods on foot and by car. Not all properties were inspected between each pair of consecutive earthquakes (e.g. between 04 Sept 2010 and 22 Feb 2011). Also, it is noted on the CGD that "…some observations following the 22 Feb 2011 and 13 Jun 2011 earthquakes could have been induced by preceding earthquakes." These maps are not meant to be complete, however, they can give a good indication of general land performance within a given area.

Figure 9 and 10 shows the recorded land damage across the example project site area after the 04 September 2010 and 22 February 2011 earthquakes. Figure 9 is from an interpretation of the aerial photography as the EQC land-based observation mapping doesn't cover the site area. For this example, no land damage was mapped as a result of the September earthquake.

Figure 9. Interpreted land damage from aerial after the 04 September 2010 earthquake

Figure 10 presents the results of land based observation mapping of the project site area after the 22 February 2011 earthquake. The example site is located in an area mapped has having "*no lateral spreading but minor to moderate quantities of ejected material*" but close to an area mapped as having large quantities of ejected material. Owner/tenant knowledge and records for a specific property may reveal more detailed information about observed damage.

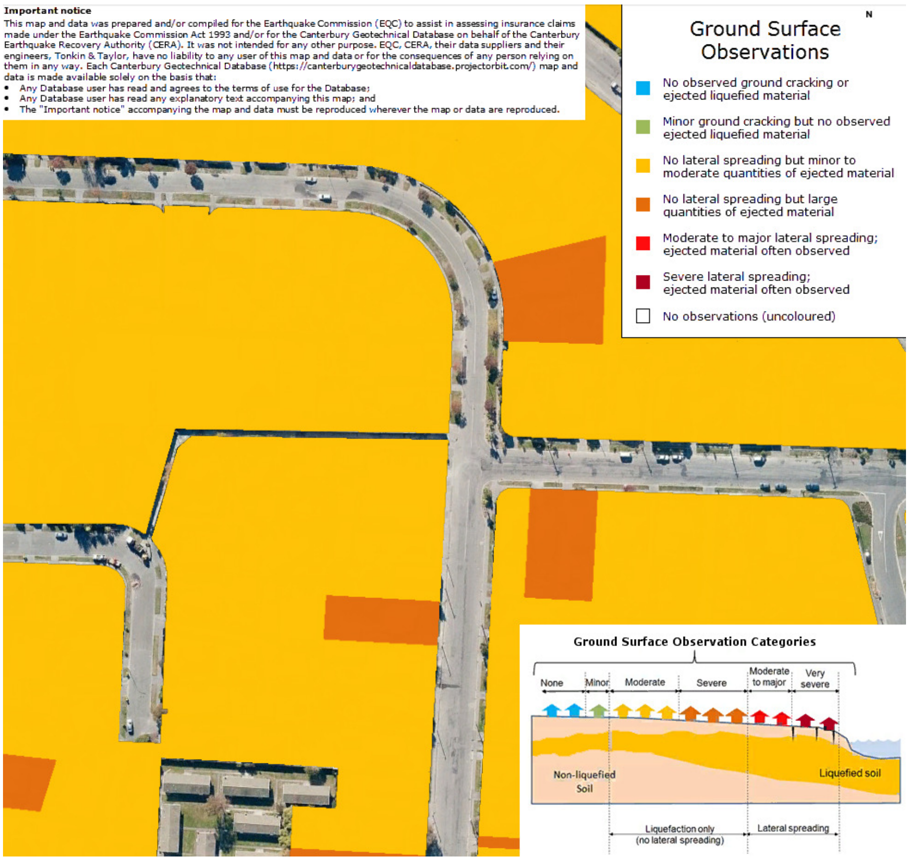


Figure 10. Land damaged mapped after the 22 February 2011 earthquake

Another dataset available on the CGD that may help identify changes in ground surface elevation as a result of the earthquakes is vertical elevation change based on aerial LiDAR. The LiDAR difference maps include elevation changes both with and without the tectonic component. In general, the aerial LiDAR datasets should not be used to accurately assess elevation change across a single house or residential property. The accuracy of the vertical LiDAR elevation difference maps is +/- 100 mm and areas of the maps contain anomalous readings for a variety of reasons. In particular, the accuracy of the LiDAR data to assess elevation changes specifically as a result of the 04 September 2010 earthquake is considered limited because the accuracy of the pre-earthquake digital elevation model (DEM) is less accurate than the post-earthquake DEMs.

Recognising these limitations, the maps of vertical elevation change with the tectonic component removed can be a good indicator of liquefaction-induced ground surface settlement across a given area after the February, June and December 2011 earthquakes.

Figure 11 indicates the LiDAR-derived vertical elevation change across the project site area from period shortly after the 04 September 2010 earthquake to May 2011 (i.e., after the 22 February, but before the 13 June 2011 earthquakes). The example site is located in an area shown to have a vertical elevation change (without tectonic component) of -0.1 to -0.2 m.

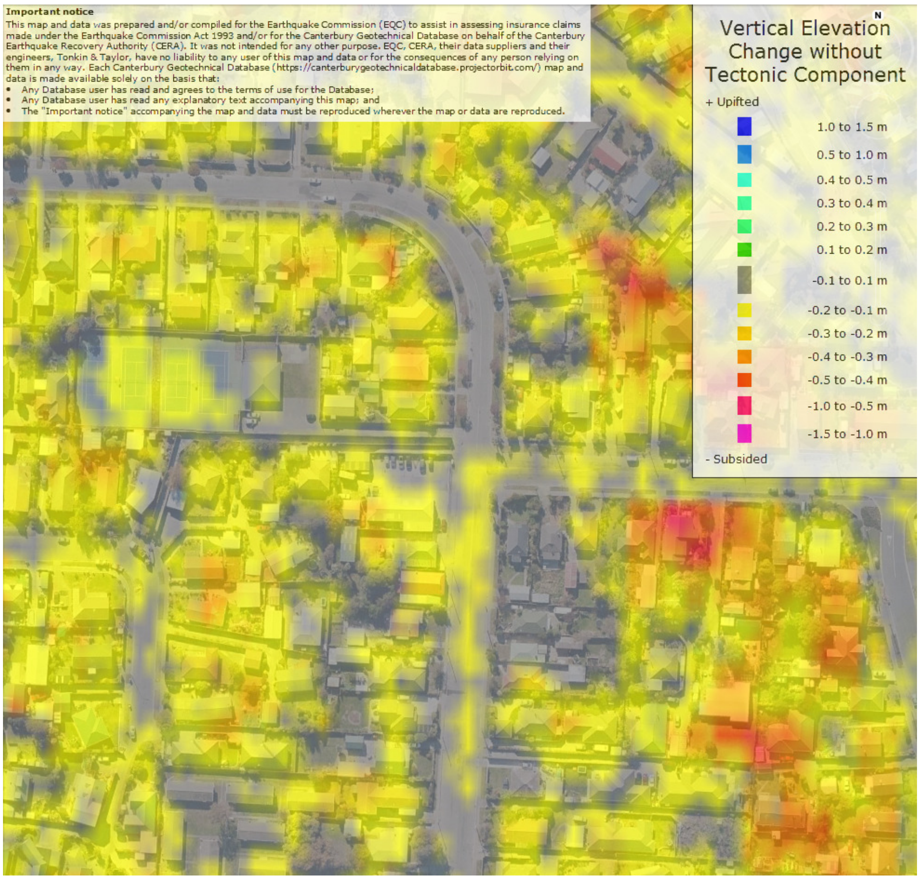


Figure 11. Non-tectonic vertical elevation change for the 22 February 2011 earthquake

Global lateral movement and lateral stretch

A detailed discussion of how to identify and address the potential effects of lateral stretch and global lateral movement on foundation design are beyond the scope of this discussion. However, detailed information on seismically induced lateral ground movement can be found in Section 12 of the Guidance Document.

For this example, it is assumed that the site is not subject to lateral stretch or consequential global lateral movement.

Site walk over

A site walk over provides the Design Engineer an opportunity to confirm the information found in the desktop study and look for site-specific evidence of land/structure damage (or lack thereof). Questions to consider during the site walkover may include:

1. How did the land perform during the earthquakes?
    a. Is there evidence of liquefaction ejecta?
    b. Is there evidence of ground cracking? Typically, hard features such as driveways and kerbs show up lateral strains in the ground.
    c. Is there water ponding on the ground surface?
    d. How did the neighbours' land and adjacent road reserve perform?
    e. Can the owner / tenant recall which of the main events caused land damage?

2. How did the structure perform during the earthquakes?
    a. What was the foundation system?
    b. Was the structure relatively light or heavy?
    c. What type of damage occurred?
    d. Where was the damage over the building footprint?
    e. Were service connections to the building damaged or repaired?
    f. How did neighbouring houses perform?
    g. Can the owner provide further record of the structural damage?

The site walkover is also a good time to determine suitable locations for site-specific geotechnical investigations. These should be placed with the location of the new building footprint in mind and take into account where the worst damage may have occurred.

Site-specific investigations

The MBIE Guidance Document (Section 13) indicates 2 CPTs per site for rebuilt foundations (minimum depth of 15 m), unless area-wide geotechnical investigation results are considered adequate. This is at the sole discretion of the geotechnical professional. However, in Section 15.3.4, site-specific investigations are recommended for projects involving ground improvement.

It may be necessary to predrill through dense gravels via a machine borehole if shallow CPT refusal occurs. In particularly difficult ground conditions, machine boreholes may need to be utilised and SPT N values used for liquefaction analyses. Such an analysis is beyond the scope of this training document; however the use of SPT-based liquefaction analyses are well documented (reference – Idriss and Boulanger, 2008 and BI2014)

The example site contains an occupied house, and one CPT (CPT A) was already completed as part of a previous investigation. A second CPT (CPT B) is located on the opposite corner of the house for the assessment as shown in Figure 12. To the extent possible, the locations of the investigation points should be located within or as close as possible to the proposed building footprint, and "bracket" the footprint. There may be a need for additional investigations as determined by the Design Engineer, for example to investigate a lateral spreading issue or for a secondary structure or particularly large primary structure.

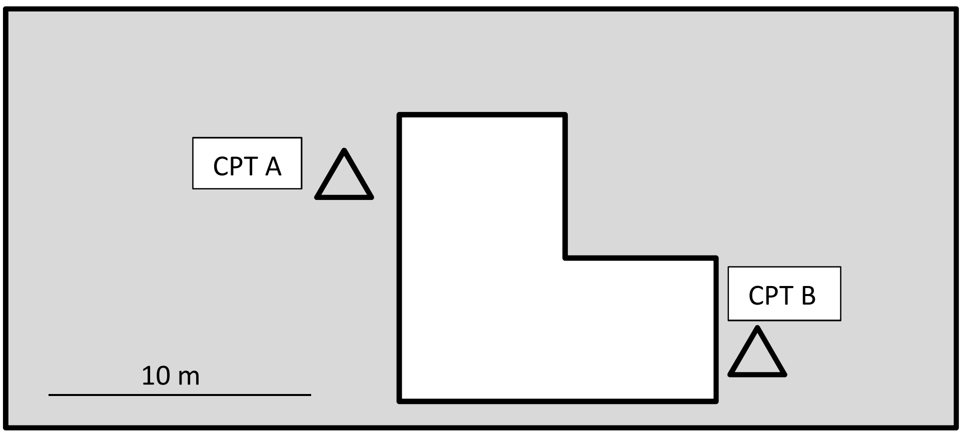


Figure 12. Locations of site-specific CPT investigations CPT A and CPT B

Liquefaction triggering analysis

Groundwater depth

The depth to groundwater assumed for the liquefaction triggering analysis is very important as the liquefaction potential of many sites may change significantly based on the depth assumed. An area-wide groundwater model has been developed for Christchurch and is available on the CGD. For this example liquefaction analysis, depth to groundwater is taken as the median groundwater level (refer to Figure 13). For back-calculation of the liquefaction potential for a particular CES event, the CGD also contains event-specific groundwater depths.

Figure 14 presents the water levels measured over time in a groundwater monitoring well located near the example site. These are useful to assess the fluctuation in level and confirm the assumed design depth.

For this example a median groundwater depth of 1.0m below ground level is assumed.

It is recommended that the potential effects of fluctuation in groundwater level be assessed (by reference to the 85th percentile level). If the higher level would potentially have a significant effect on the ground improvement design, then it is recommended that it be adopted for design purposes.

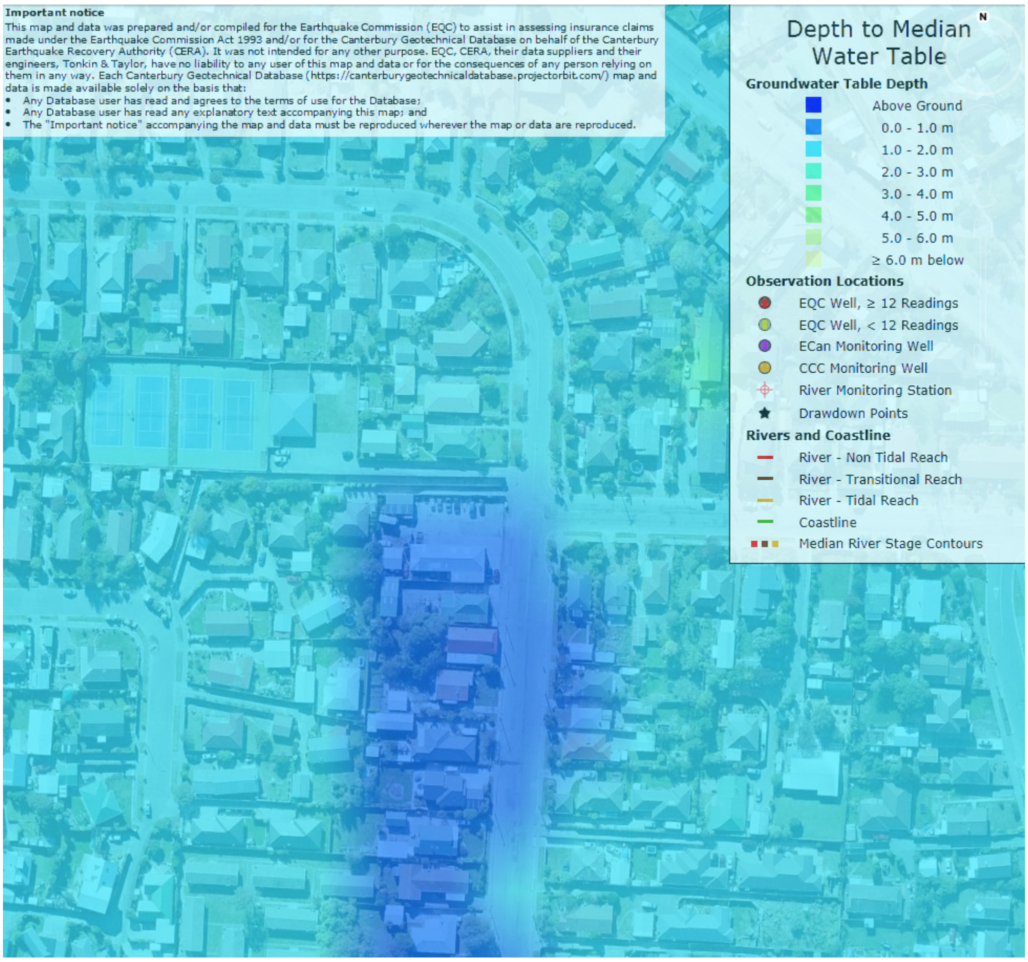


Figure 13. Depth to median groundwater model surface

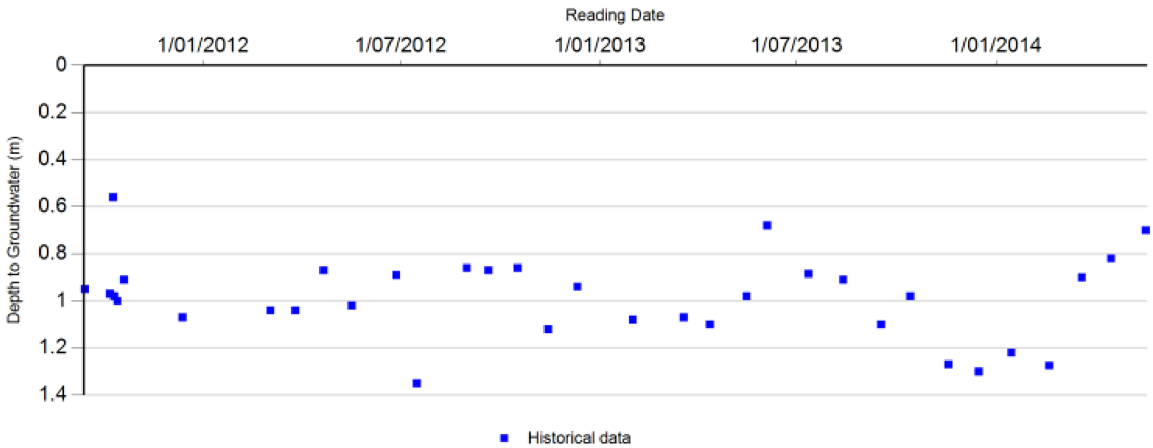


Figure 14. Groundwater level at red monitoring well

Liquefaction triggering calculation

For this example, the liquefaction triggering potential of the two on-site CPTs was assessed using the BI2014 CPT-based simplified procedures as discussed previously. The guidelines previously recommended the use of the Idriss and Boulanger (2008) CPT-based simplified procedure in combination with the fines content correlation by Robertson and Wride (1998). BI2014 contains significant updates to their earlier methodology including: 1) incorporation of 50 Christchurch liquefaction case histories to the international case history database; 2) a major revision to the calculation of the magnitude scaling factor (MSF); 3) inclusion of a fines content/I_c relationship; and, 4) three probabilities of liquefaction – 15%, 50% and 85%.

Liquefaction assessment should only be performed by competent practioners that are familiar with the method being used and in particular, how various inputs may affect the results as well as the limitations of the method. Commercial available software such as CLIQ has made CPT-based liquefaction triggering and settlement calculations a relatively simple exercise. However, it has not reduced the need for the practioner to have a thorough knowledge of the principles of liquefaction assessment, and to clearly understand how the software works.

The key input parameters in this example assessment are:

- Groundwater depth of 1.0 m (median value).
- PGA and magnitude (M) pairs. When using the BI2014 simplified method, the following PGA/M pairs are to be used for liquefaction assessment as explained in MBIE Q&A 50:

	SLS Case 1	SLS Case 2	ULS
PGA	0.19 g	0.13 g	0.35 g
Magnitude, M	6.0	7.5	7.5

- Soil behaviour index (I_c) cut-off value for clay-like behaviour to differentiate liquefaction susceptible and non-susceptible material – 2.6.
- Fines content/I_c relationship curve fitting parameter (C_{FC}) – 0.0.
- Probability of liquefaction (P_L) – 15 %

As noted in Q&A 50, C_{FC} and P_L values of 0.0 and 15% should be utilised unless site-specific observations and laboratory testing from on-site samples confirms that the use of a P_L of 50% and/or different C_{FC}.

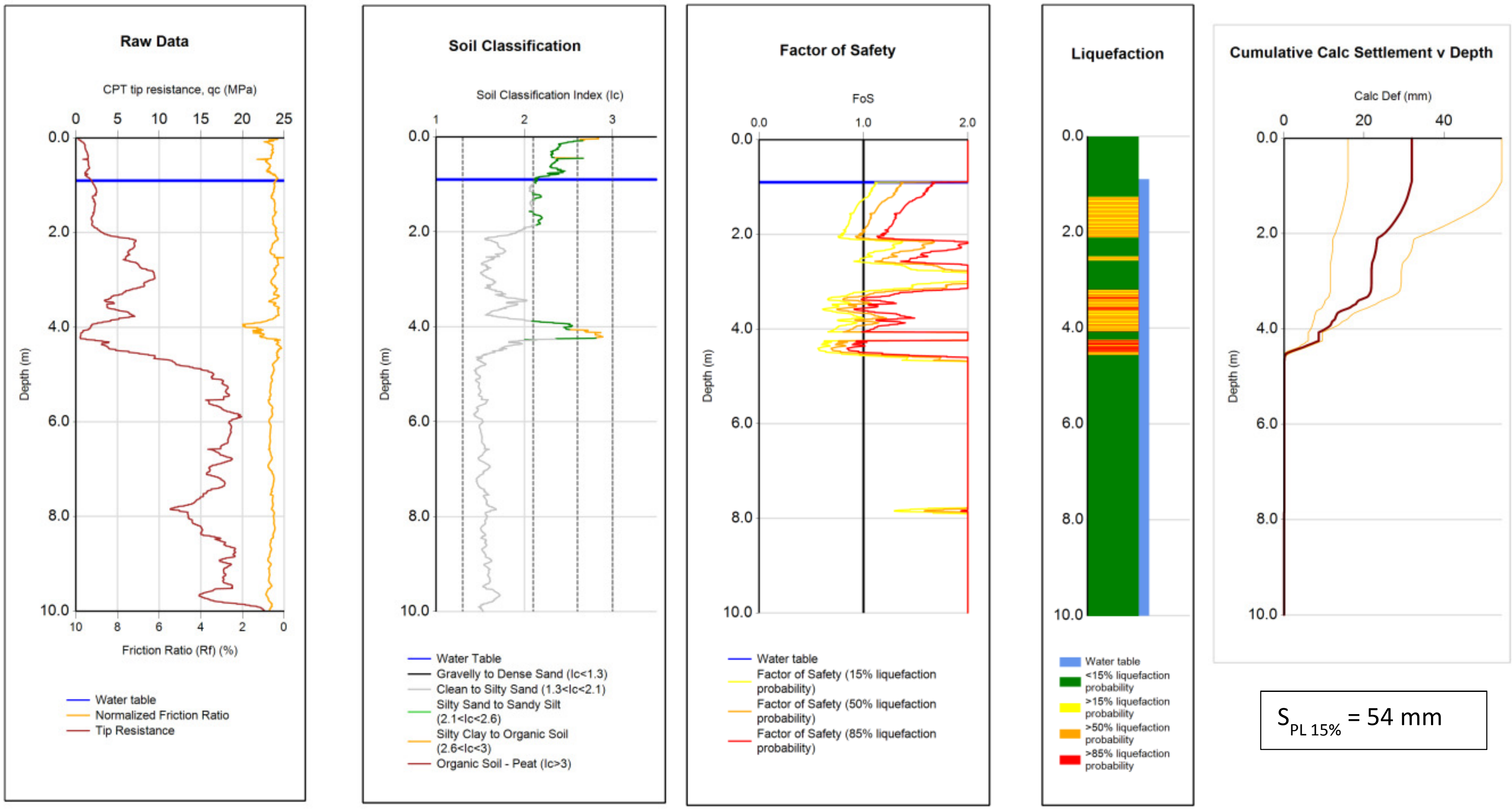
Note that for relatively loose soils, the M6.0 case will nearly always control (relative to M7.5) at the SLS shaking level. This is due to the incorporation of soil density in the calculation of the MSF in BI2014.

It is recommended in Section 15.3.9 of the Guidance Document that the liquefaction potential also be checked at an "intermediate" level (referred to herein as "ILS") of ground shaking – nominally the level associated with a 100-year return period event. In Christchurch, this is often taken as aM7.5/0.20 g event. As noted previously, a lower magnitude event resulting in the same level of ground shaking as a M7.5 event will often result in more triggering of liquefaction, hence greater settlement within loose soil profiles. Therefore, it is recommended that the IL liquefaction potential also be assessed using a M6 event with a PGA of 0.30 g.

It is not a requirement for general geotechnical reporting, that an intermediate level of shaking be analysed. However, for ground improvement projects it is considered good practice to assess the effect of this level of ground shaking; particularly to identify where in the soil profile the liquefaction occurs. This exercise may help inform the selection and design of the ground improvement.

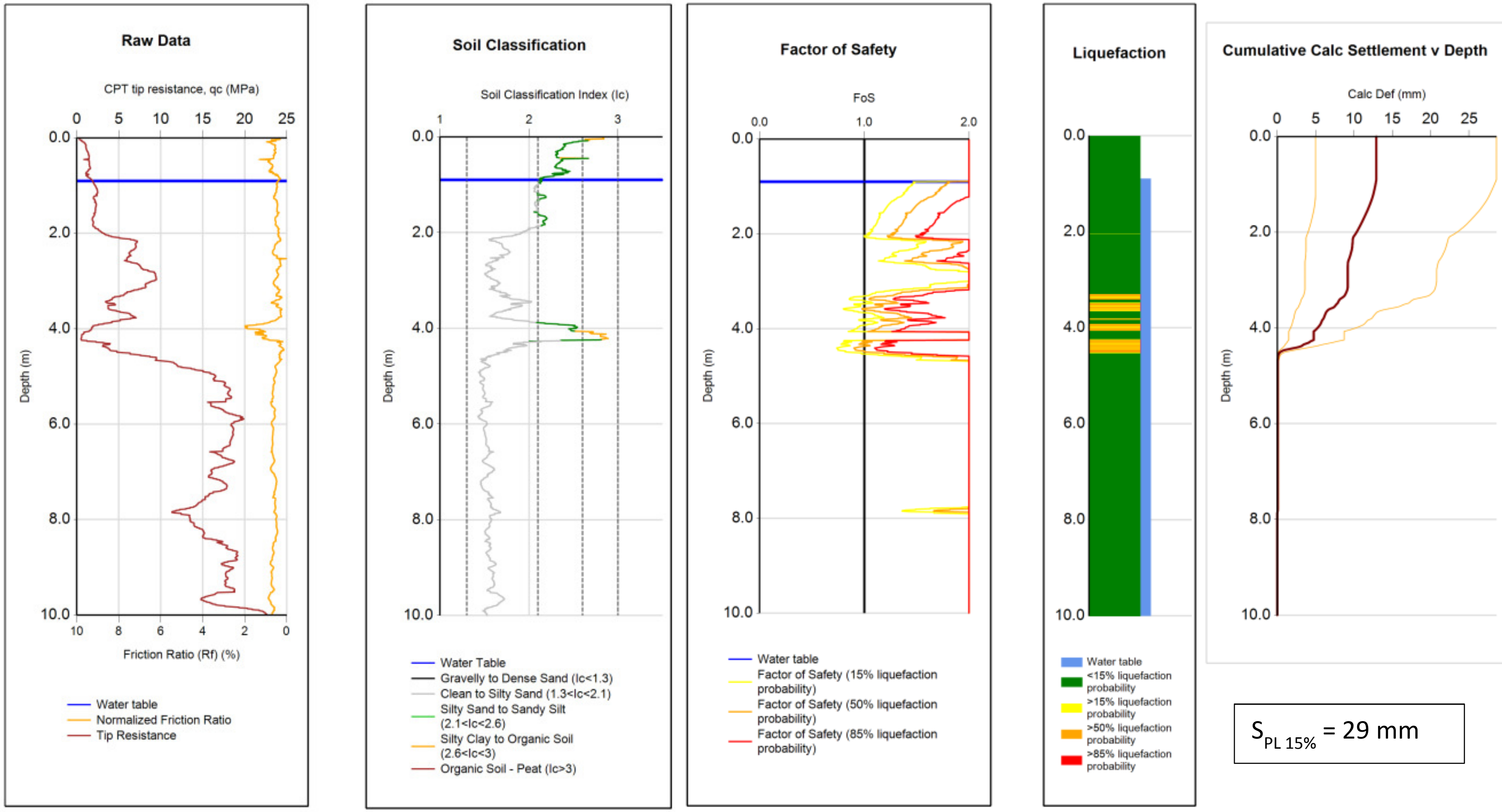
Figures 15 and 16 (a through d) present the graphical results of the liquefaction analyses of both CPT soundings for the three design cases plus the IL case described above. For the ILS case, only the M6 analysis (larger settlements than M7.5 scenario) is presented. Also shown are the CPT tip resistance (q_c), friction ratio (R_f) and I_c values, as well as the calculated liquefaction-induced settlements (discussed in Section 2.5).

The plots of I_c and factors of safety against liquefaction (FS_{liq}) are useful for assessing where in the soil profile, liquefaction is predicted to occur, as well as the thickness of the susceptible layers. For example, in CPT A it is clear that the majority of the liquefaction is predicted to occur above a depth of about 4.5 m, with a significant portion of the liquefaction occurring above a depth of 2 m. The depth of predicted liquefaction is a key parameter to consider when selecting shallow ground improvement and foundation options as discussed in Section 2.6.



$S_{PL\ 15\%}$ = 54 mm

Figure 15 a). CPT A: SLS, M = 6, PGA = 0.19g



$S_{PL\ 15\%}$ = 29 mm

Figure 15 b). CPT A: SLS, M = 7.5, PGA = 0.13g

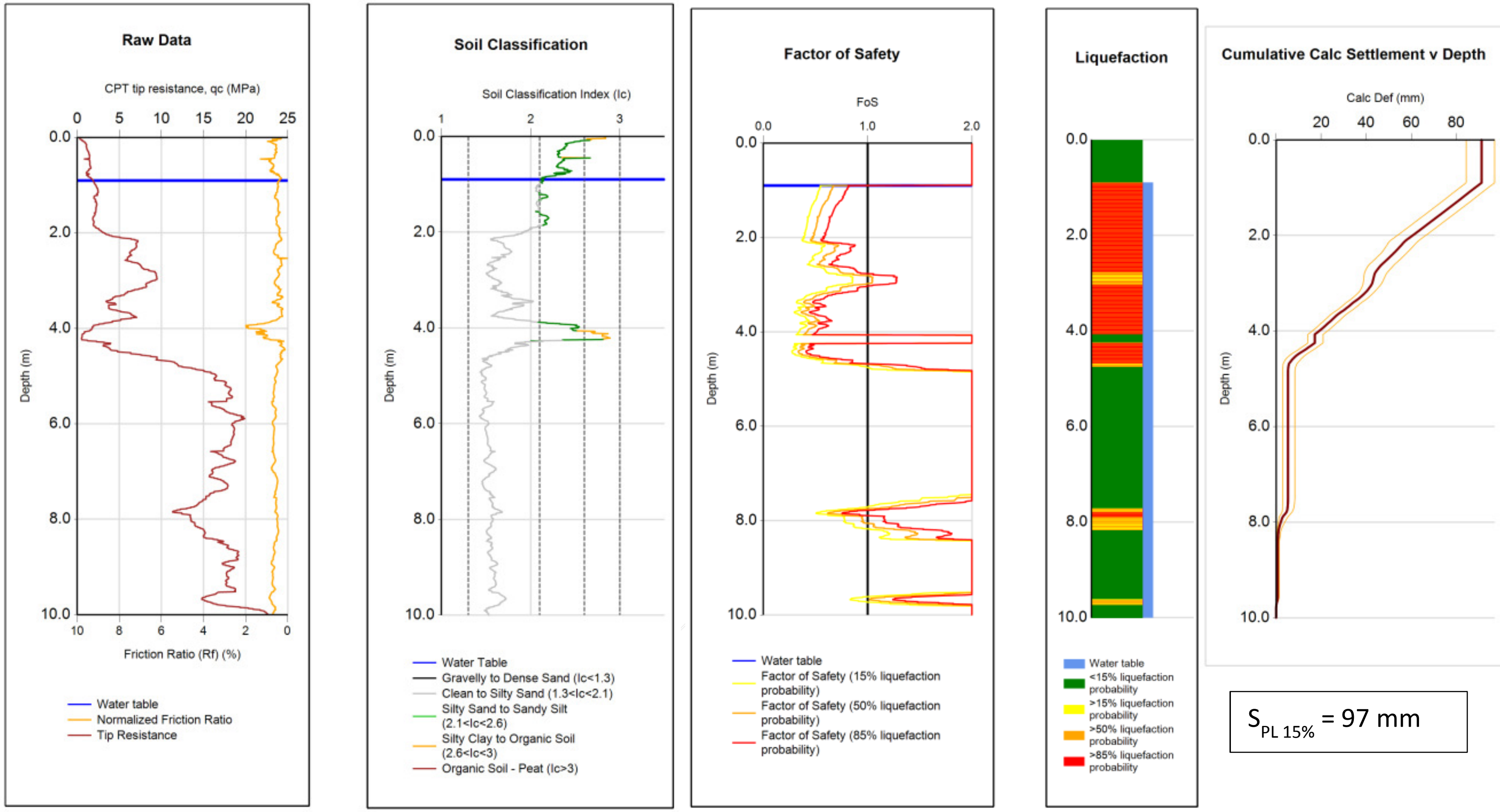
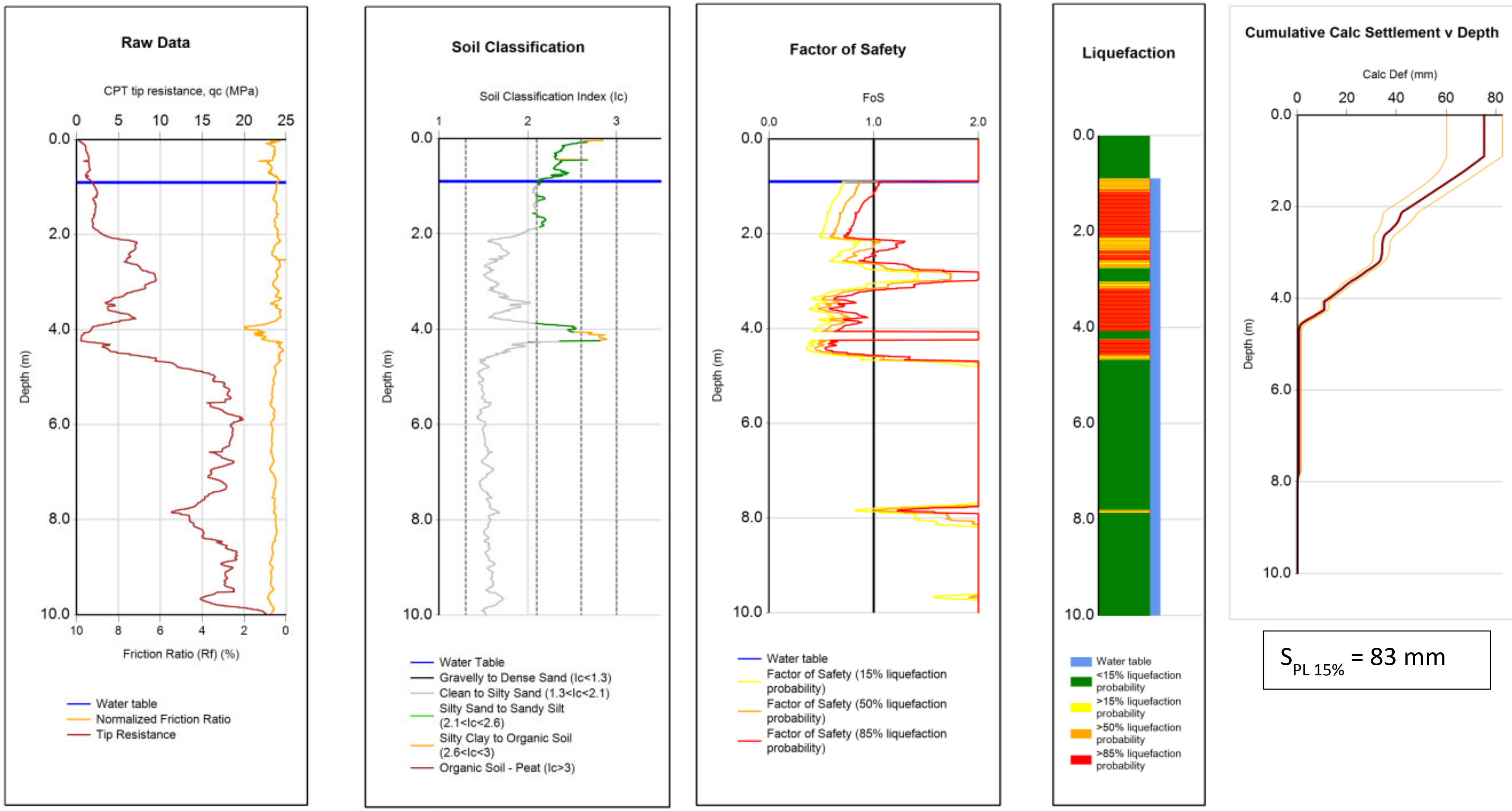
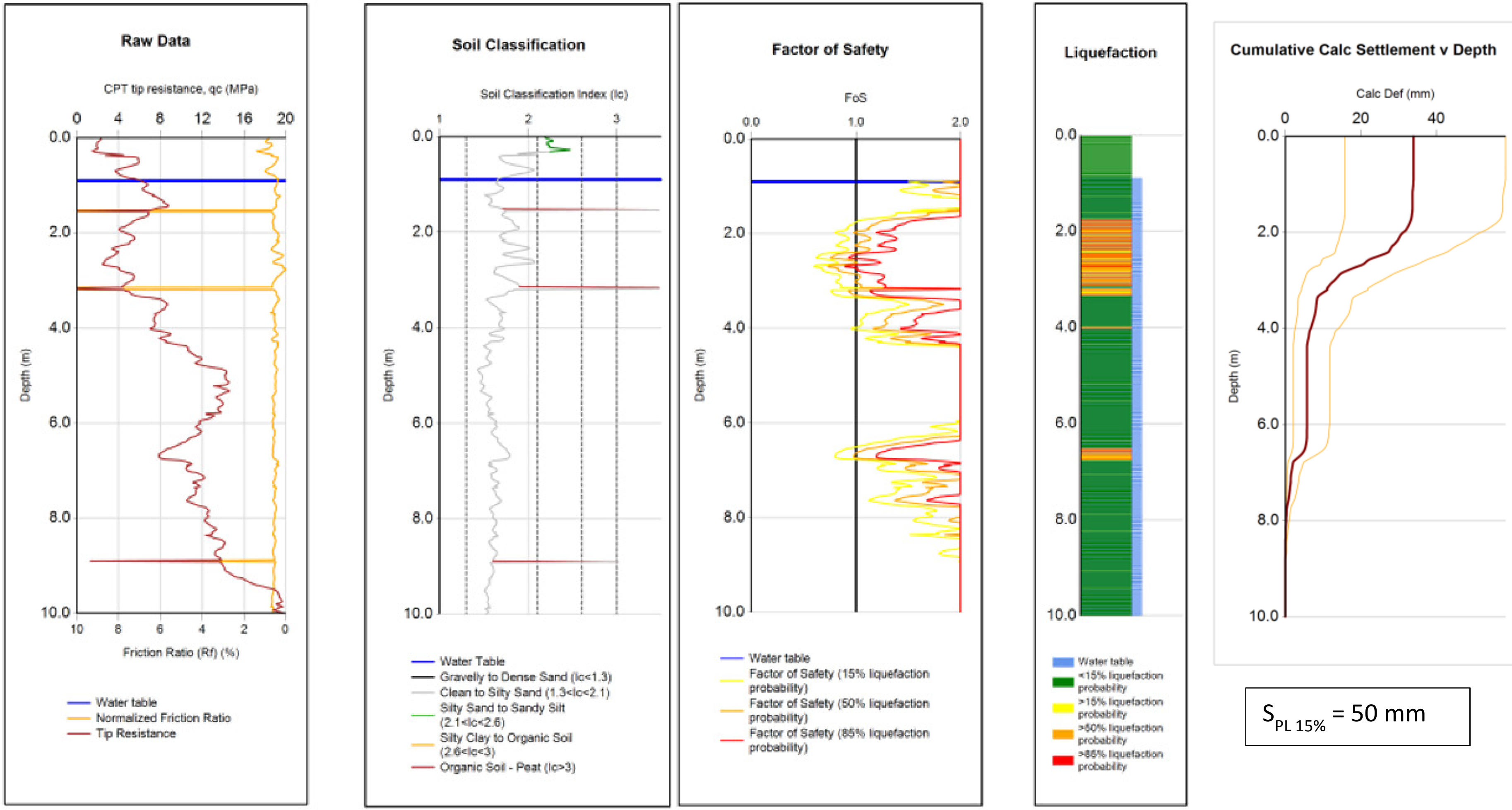


Figure 15 c). CPT A: ULS, M = 7.5, PGA = 0.35g

Figure 15 d). CPT A: ILS, M = 6.0, PGA = 0.30g

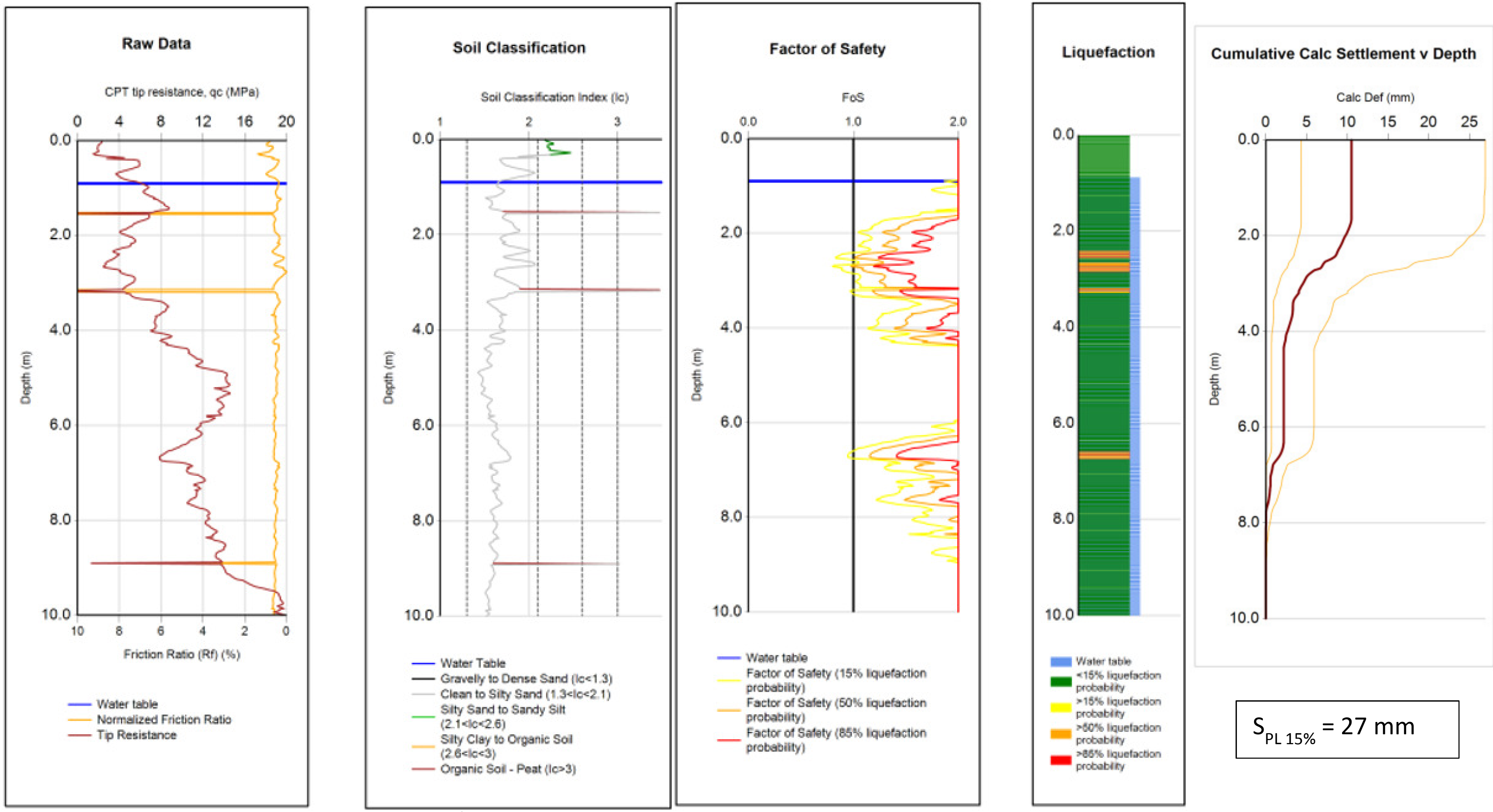


$S_{PL\ 15\%}$ = 83 mm



$S_{PL\ 15\%}$ = 50 mm

Figure 16 a). CPT B: SLS, M = 6, PGA = 0.19g



$S_{PL\ 15\%}$ = 27 mm

Figure 16 b). CPT B: SLS, M = 7.5, PGA = 0.13g

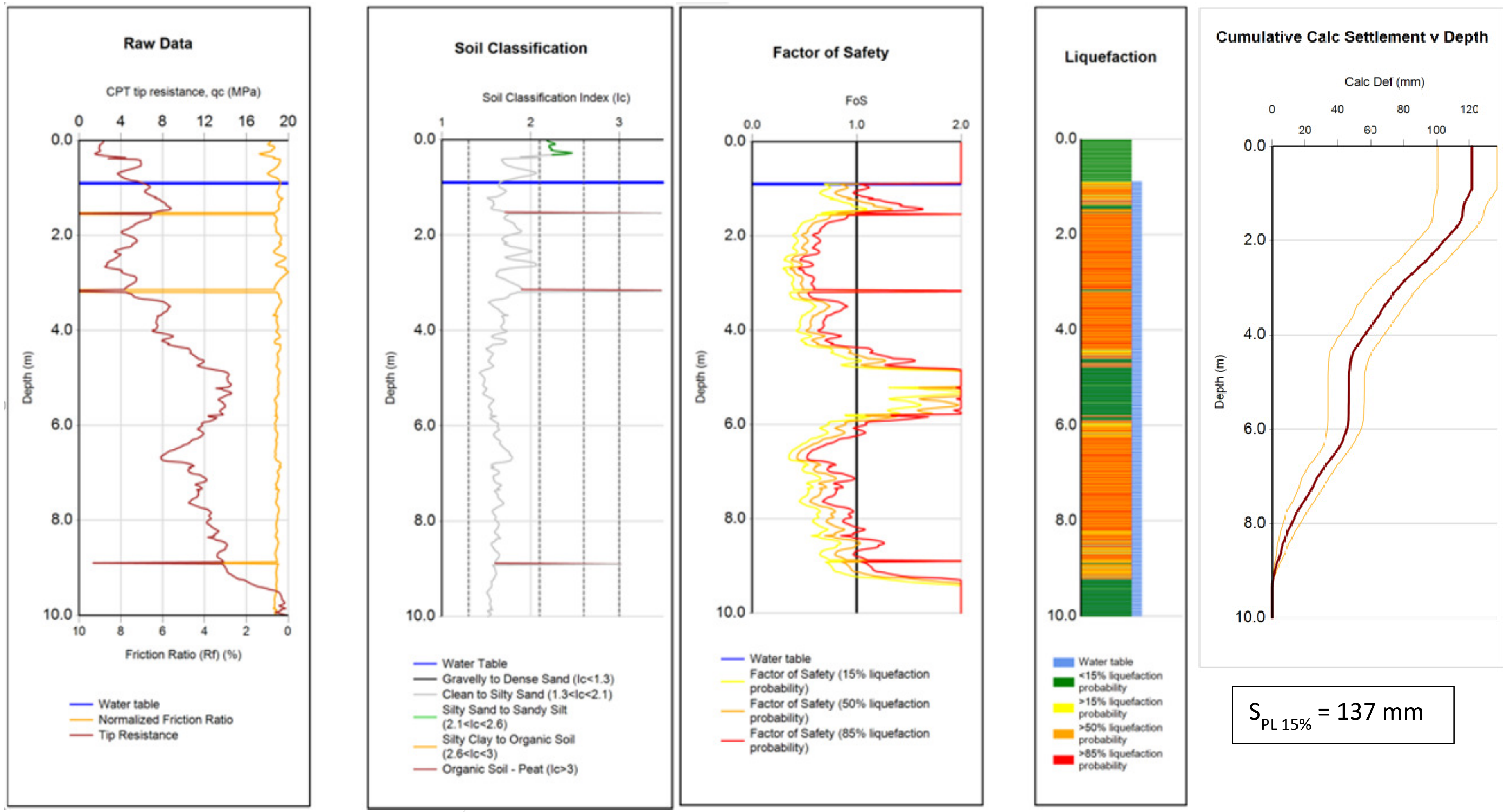
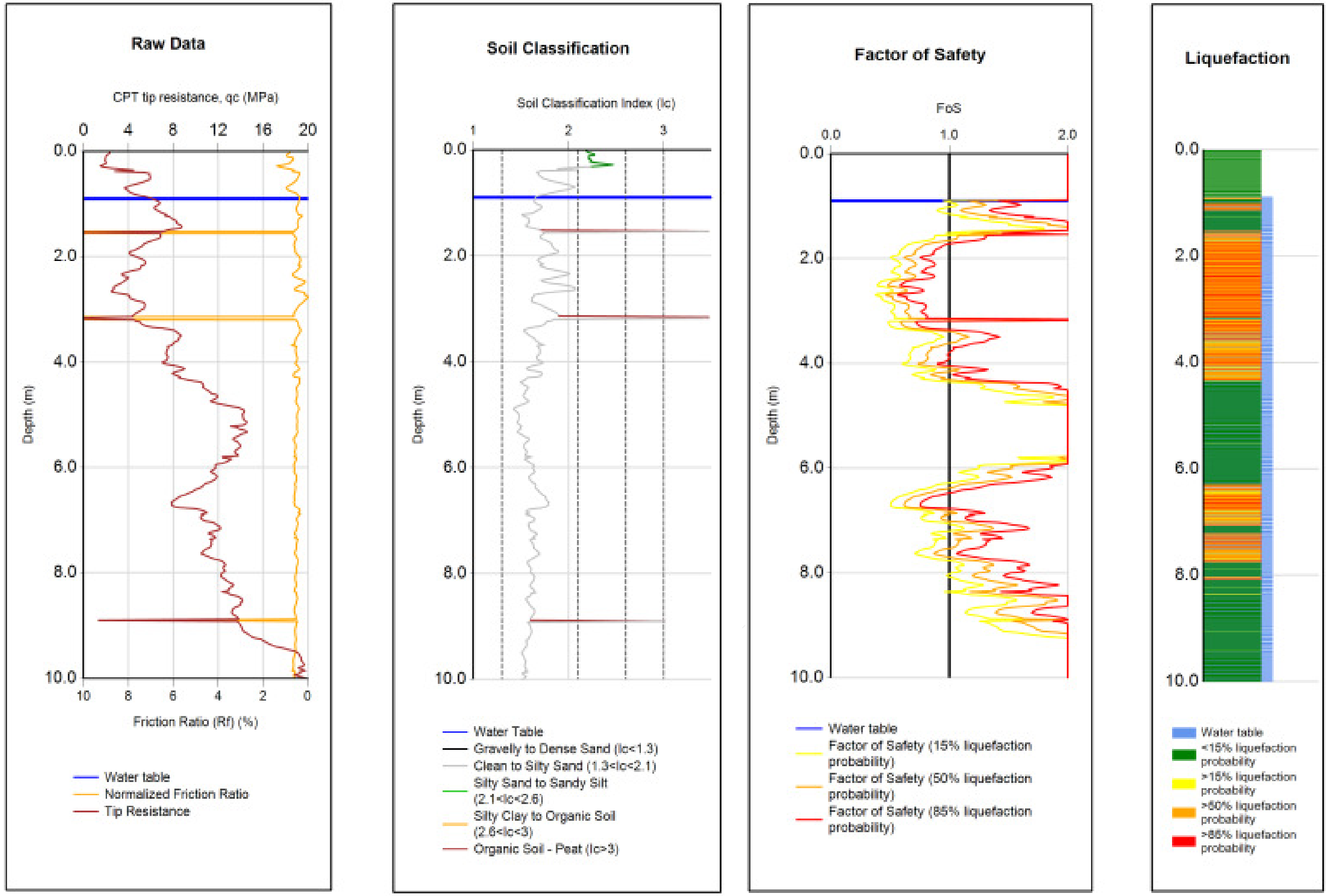


Figure 16 c). CPT B: ULS, M = 7.5, PGA = 0.35g



$S_{PL\ 15\%}$ = 94 mm

Figure 16 d). CPT B: ILS, M = 6.0, PGA = 0.30g

Liquefaction settlement

Calculated settlement

To calculate the liquefaction-induced settlement from a CPT-based triggering analyses, the methodology by Zhang et al. (2002) was utilised as per the MBIE guidance. Most commercial liquefaction software that performs CPT-based liquefaction triggering analysis also computes the associated settlements using the Zhang et al. (2002) methodology.

The Guidance Document specifies that liquefaction settlement be computed over the upper 10 m of the soil profile (refer to Sections 12.3 and 13.5), and it is this “index number” that forms the basis for informing the selection of the recommended foundation and shallow ground improvement options.

For CPT A and CPT B the free-field ground surface settlements computed over the top 10 m of the soil profile are presented in Table 2.

Table 2 – Computed liquefaction settlements over upper 10 m from CPT A and CPT B (mm)

	CPT A	CPT B
SLS Case 1 M = 6.0, PGA = 0.19 g	54	50
SLS Case 2 M = 7.5, PGA = 0.13 g	29	27
ULS, M = 7.5, PGA = 0.35 g	97	137
ILS, M = 6.0, PGA = 0.30 g	83	94

The SLS Case 1 settlements are larger than those for SLS Case 2, hence the Case 1 values will be adopted for foundation design. The SLS index number is less than 100 mm which is defined as “minor to moderate” (refer to Table 12.5 of the Guidance Document).

Figures 15 and 16 illustrate where in the soil profile the predicted liquefaction settlement occurs. The amount of liquefaction-induced settlement computed for a particular liquefiable layer is dependent on soil density, but also directly proportional to the thickness of that layer.

Comparison to observed site/structure performance

Where there is sufficient information available, the performance of the site should be compared with the calculated free-field liquefaction settlements. At this site, there are no recorded observations of surface expression of liquefaction (ejecta, lateral stretch/spread, differential settlement of house, etc) as a result of the 04 September 2010 earthquake. For the purpose of this example, it also assumed that there was no structural damage related to foundation movement/settlement.

Section 13.5.1 of the Guidance Document provides criteria to assess if the site has been “sufficiently tested at SLS” by the CES. A site is considered sufficiently tested if one of the following criteria are met:

- The site PGA from one of the CES earthquakes (scaled to a M7.5 event) is at least 170% of the $PGA_{SLS, M7.5}$. For this assessment, the SLS magnitude/PGA pair to be used is 7.5/0.13g; or,
- the 10^{th} percentile site PGA is greater than the $PGA_{SLS, M7.5}$.

This can be useful to assess whether the calculated SLS settlements are consistent with what actually occurred at the site.

The site PGA is defined as the mean conditional PGA by Bradley and Hughes (2012). The MSF used in the Guidance Document for this assessment is the one proposed by Idriss and Boulanger (2008) and that equation is used here for consistency. For this site, the following are the calculations for site PGA estimated to result from the 04 September 2010 earthquake:

- Mean site PGA = $PGA_{50,M7.1}$ = 0.18g
- Scaled site PGA = $PGA_{50,M7.5}$ = $PGA_{50,M7.1}$/MSF where MSF = 1.11 (Idriss & Boulanger, 2008)
- $PGA_{50,M7.5}$ = 0.16g which is 123% of PGA_{SLS} < 170% - **not sufficiently tested**
- 10th percentile site PGA = $PGA_{10, M7.1}$ = $PGA_{50,M7.1}$ * exp(-1.28*σ_n)
  where σ_n = conditional standard deviation of site PGA = 0.250

  $PGA_{10, M7.1}$ = 0.16g x exp(-1.28 x 0.250) = 0.12g

  $PGA_{10, M7.5}$ = 0.12/1.11 = 0.11g – **not sufficiently tested**

Selection of foundation / ground improvement system

Based on the results of the liquefaction analysis, the site is confirmed to be TC 3 on the basis of the SLS settlements, and a significant portion of the predicted SLS liquefaction potential is located within the top 4 m of the site profile. The site was not “sufficiently tested” in the 04 September 2010 earthquake and there does not appear to have been any observations of “consequential liquefaction,” in that event.

It is noted that the predicted ULS liquefaction is extensive throughout the soil profile and it begins at a depth of about 1.0 m (the depth of the assumed groundwater level). The predicted total ULS settlements are considered acceptable with respect to impact on other aspects of the project (e.g., drainage, flooding for example). The liquefaction estimated for the ILS event is relatively close to the level expected at ULS.

Obvious surface manifestation of liquefaction (ejecta, ground surface settlement) was observed in the 22 February 2011 earthquake; including moderate levels of liquefaction close to the subject site. The site was estimated to have experienced a scaled mean ground shaking in the February event of close to that of ULS level.

On the basis of the above information, the foundation solution selected for this site is a TC2 foundation combined with shallow ground improvement. Due to the relatively clean sand profile, all 8 of the shallow ground improvement options would be technically suitable. The groundwater level is very shallow, however, and the clean sands may result in unstable excavation walls. Therefore, the excavate and replace options (G1a, G1d and G2a) are deemed likely to be more costly.

The site has houses on either side and behind so RIC and DC (options G1b and G1c, refer MBIE Guidance Chap 15.3) are also likely to be unsuitable because of potential vibration and noise issues. It is noted that either of these methods work well in clean sands and would be well-suited when several properties are being treated simultaneously.

Out of the 8 options listed in Section 15.3, option G5a is selected as the most suitable option. Driven timber piles (G5b) would also work but may not be as effective as shallow stone columns at achieving consistent densification throughout the depth of improvement. Either conventional vibroflot stone columns or “rammed aggregate piers™ (RAP™) would be expected to work well in the clean sands.

A 4 m depth of improvement would remove most or all of SLS as well as the shallow ULS and ILS liquefaction potential. The base of the improvement zone may terminate in a liquefiable layer, however, the treatment depth could likely be extended to a depth of 5 m for relatively little additional cost. The computed post-improvement liquefaction settlements would be less than about 60 mm at ULS and less than about 10 mm at SLS.

Figure 17 illustrates what a 5 m deep zone of ground improvement might look like relative to the anticipated liquefaction profile for SLS and ULS levels of shaking.

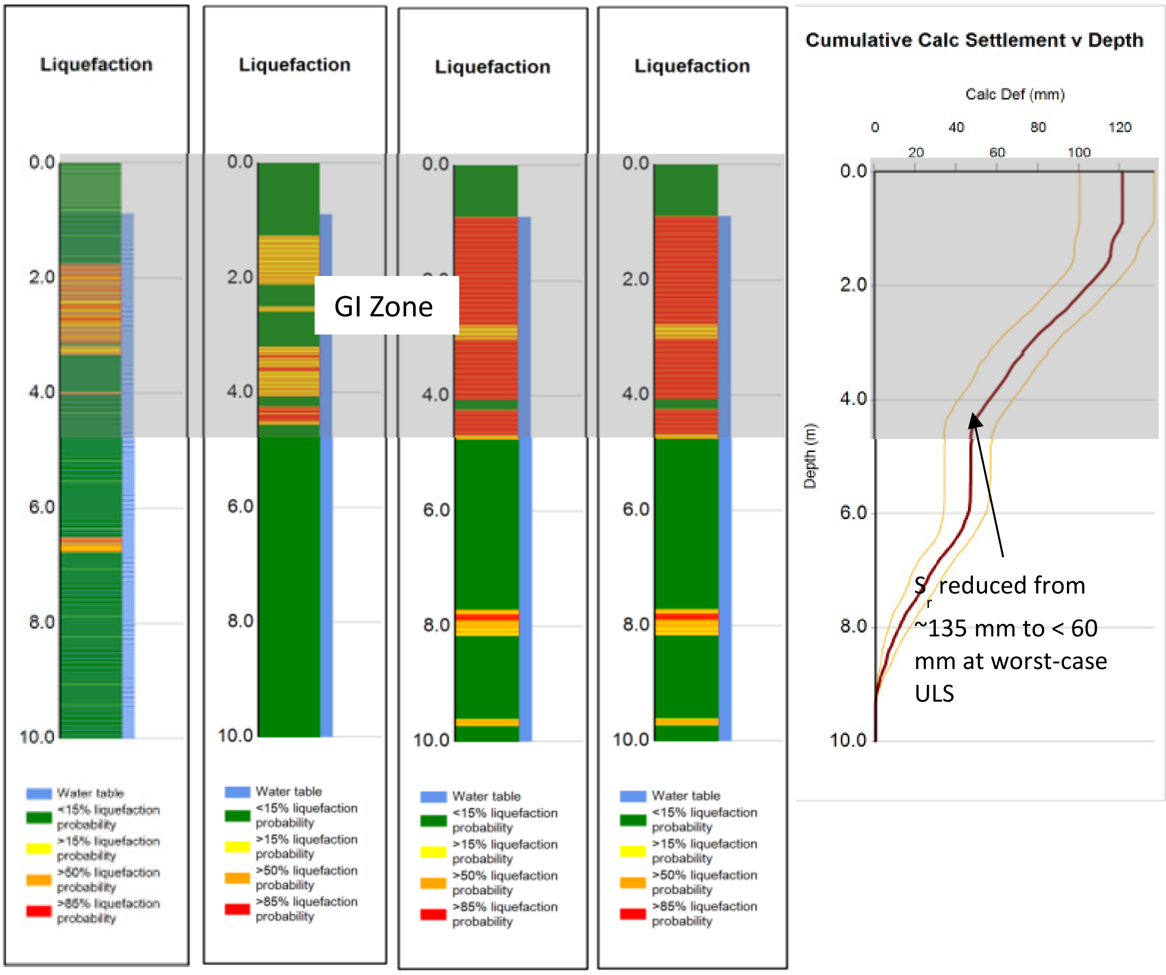


Figure 17 – 5 m deep zone of ground improvement and effect on predicted liquefaction settlement

Quality control (QC) during construction of ground improvement and post-improvement verification testing is important to confirm that the design intent has been met. Appendix C4 of the Guidance Document presents recommendations for construction QC and verification testing, including the minimum construction documentation that the Design Engineer should receive. A comprehensive discussion of construction QC and verification testing is beyond the scope of this document. However, the reader is urged to carefully read the revised Appendix C4 as well as other appropriate reference material and incorporate adequate QC on their projects.

In this example, it is likely that the cost of the ground improvement can be reduced by refining the design minimum area replacement ratios (ARR) and implementing post-improvement verification testing to confirm that the required densification has been achieved. However, for a shallow stone column installation, there are also minimum ARR that can be used in lieu of verification testing (for clean sand sites).

Specific design and specification of ground improvement is beyond the scope of this document. However, the design is not difficult, particularly for clean sands. There are several good technical references covering the design and construction of various ground improvement methods.

Worked example 2

This worked example steps through an assessment of an example property in St Albans in the central west of Christchurch. It is intended to illustrate some of the potential complexities associated with assessing a "silty" site as a potential candidate for shallow ground improvement.

The main steps of the assessment are the same as outlined for Example 1. For brevity, the results of the desktop study and geotechnical investigations are briefly summarised and the majority of this example focusses on the site liquefaction assessment and selection of the preferred ground improvement option.

Summary of findings from desktop study and site-specific geotechnical investigation

The summary of the findings from the desktop study are:

- Site is categorised as TC3 but is located at the boundary with a large area of TC2 land.
- No extraordinary geotechnical issues found (ie lateral spreading, buried channels) on or adjacent to the site.
- No observed land/structure damage in 04 Sept 2010 earthquake on site or surrounding area.
- Site neighbourhood contains areas ranging from no surface expression of liquefaction observed to minor to moderate surface expression of liquefaction in 22 Feb 2011 event. Some minor ejecta was observed on the project site.
- Some damage to house foundation (noting this was a Type 2 foundation) with a broken concrete ring beam which may necessitate a complete rebuild – we have assumed a rebuild is necessary for this example.
- No disruption to site services was experienced and the house remained occupied well after the earthquakes.
- LiDAR data indicates that the entire neighbourhood around the site may have subsided between 0.1 and 0.4 m (net of tectonic movement) as a result of entire CES.
- Median GWL at 1.0 m
- Site was "sufficiently tested" in both the 04 Sept 2010 and 22 Feb 2011 main earthquakes.

The site-specific geotechnical investigation found:

- The site soil profile consists primarily of sandy silt and silty sand, with an approximately 4.5 m thick crust comprised mainly of non-liquefiable clayey silt / organic silt (potentially some peat).
- An approximately 0.5-1.0 m thick layer of silty sand was found at a depth of 2.0 m in both site CPT soundings.
- Between depths of about 4.5 and 10 m, the average I_c value is about 2.3 with a range of approximately 2.1 to 2.8 and some thin layers of 1.8 and 3+ materials.
- The stratigraphy of the 2 site CPTs is very similar.

Estimated ground shaking during the CES

The conditional median PGAs and standard deviations for the example site, for each of the four main CES earthquakes are summarised in Table 3.

Table 3 – Event magnitude, peak ground acceleration and conditional standard deviation values for 2 largest CES events (Bradley and Hughes, 2012)

Earthquake event	Magnitude, M	Peak ground acceleration, PGA	Conditional standard deviation
September 2010	7.1	0.21g	0.325
February 2011	6.2	0.30g	0.350
June 2011	6.0	0.18g	0.375
December 2011	5.9	0.18g	0.375

Liquefaction triggering analysis

As for the previous example, the liquefaction triggering potential of the two on-site CPTs was assessed using the BI2014 CPT-based simplified procedure – including the BI2014 FC/I_c correlation. The depth to groundwater was taken as 1.0 m (median value).

The other key input parameters in this example assessment are the same as for Example 1, namely:

Three levels of ground shaking:

	SLS Case 1	SLS Case 2	ULS
PGA	0.19 g	0.13 g	0.35 g
Magnitude, M	6.0	7.5	7.5

- Soil behaviour index (I_c) cut-off value for clay-like behaviour to differentiate liquefaction susceptible and non-susceptible material – 2.6.
- Fines content/I_c relationship curve fitting parameter (C_{FC}) – 0.0 (no lab testing available)
- Probability of liquefaction (P_L) – 15 %

The ILS level of ground shaking assessed as part of Example 1 (M6/PGA = 0.30 g) was also assessed in this example.

Figures 18 and 19 (a through d) present the graphical results of the liquefaction analyses of both CPT soundings for the three design cases plus the IL case (M6/PGA = 0.30 only), as well as the calculated liquefaction-induced settlements.

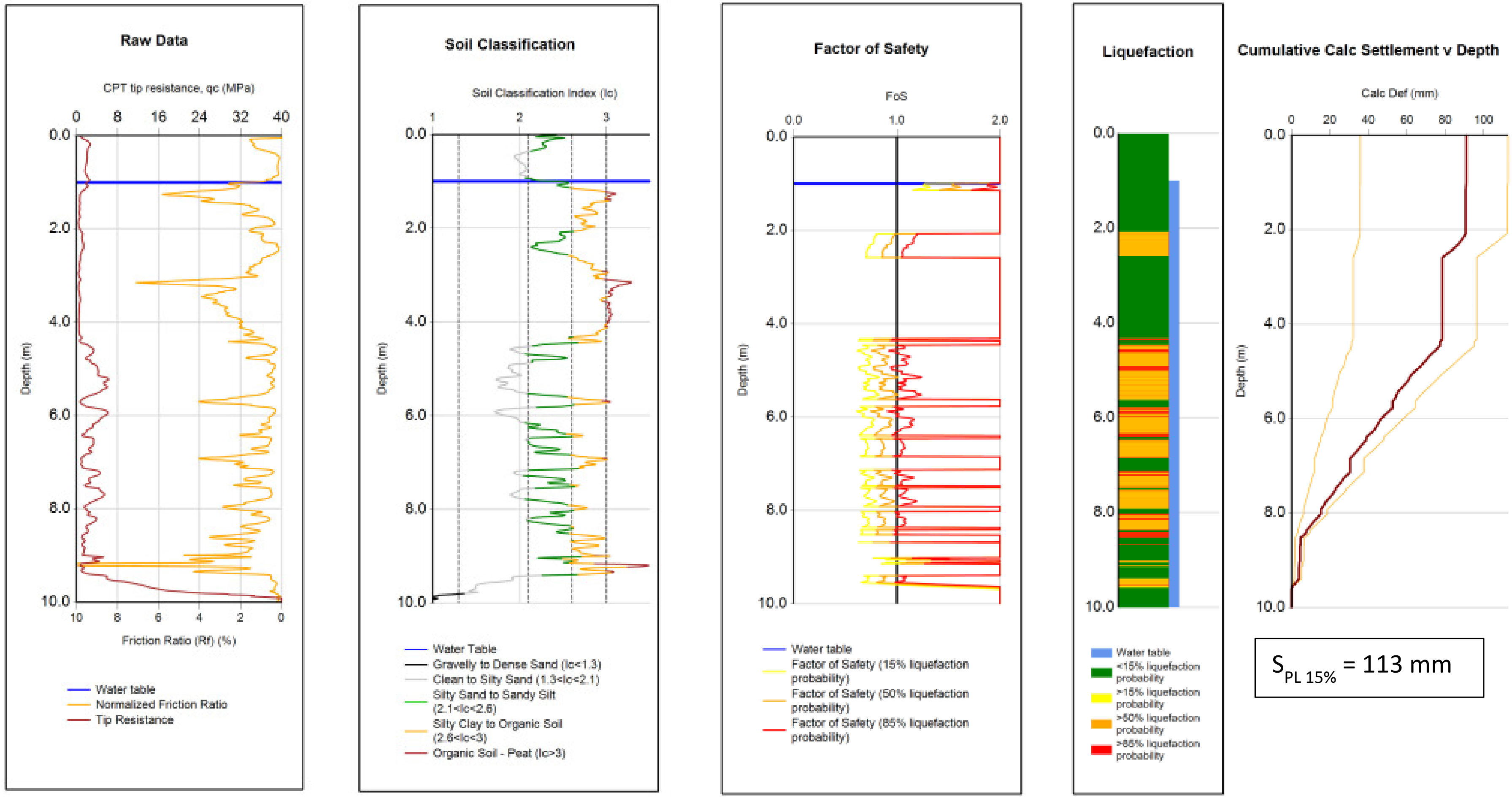
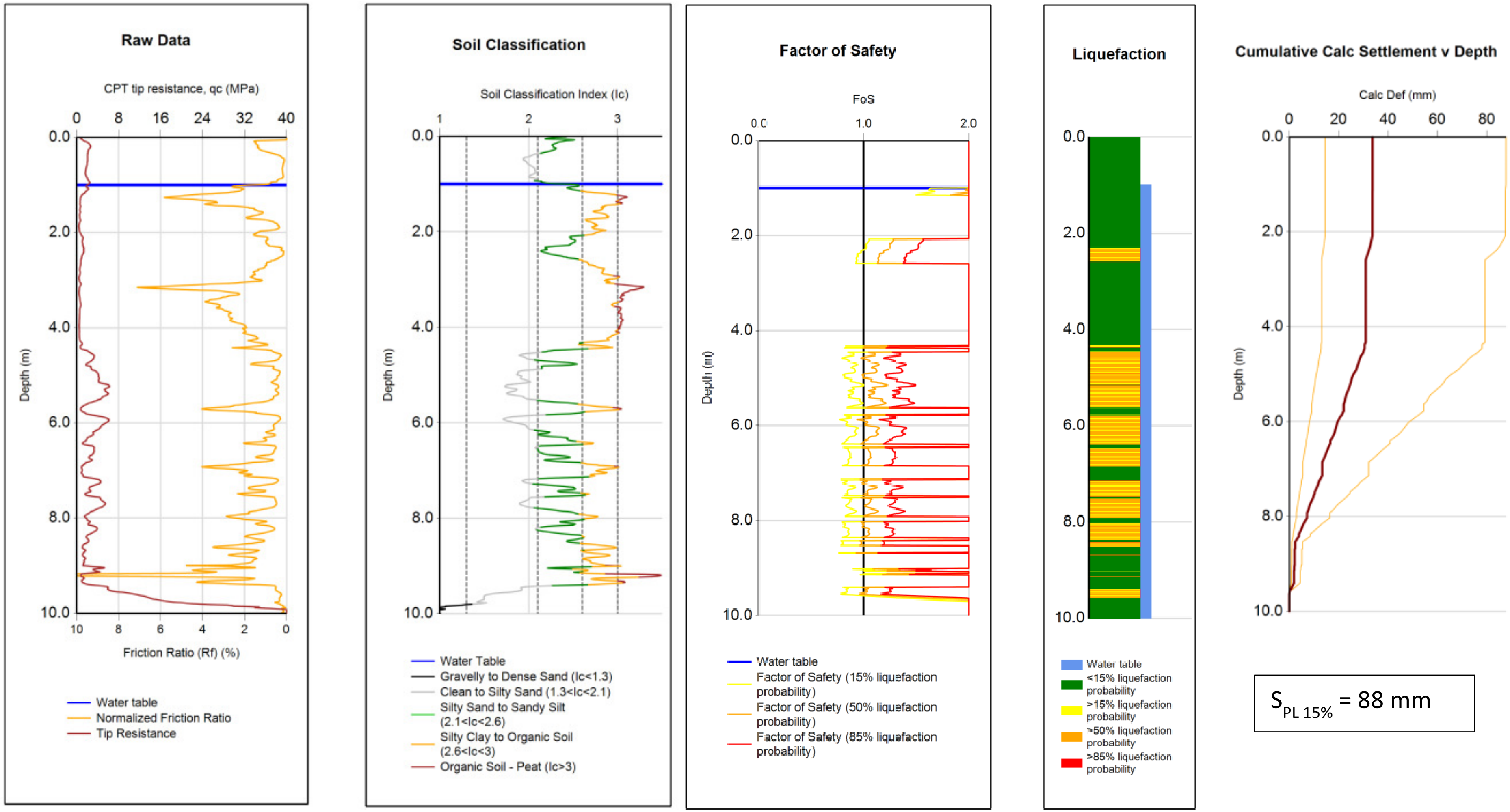


Figure 18 a). CPT A: SLS, M = 6, PGA = 0.19g



$S_{PL\ 15\%}$ = 88 mm

Figure 18 b). CPT A: SLS, M = 7.5, PGA = 0.13g

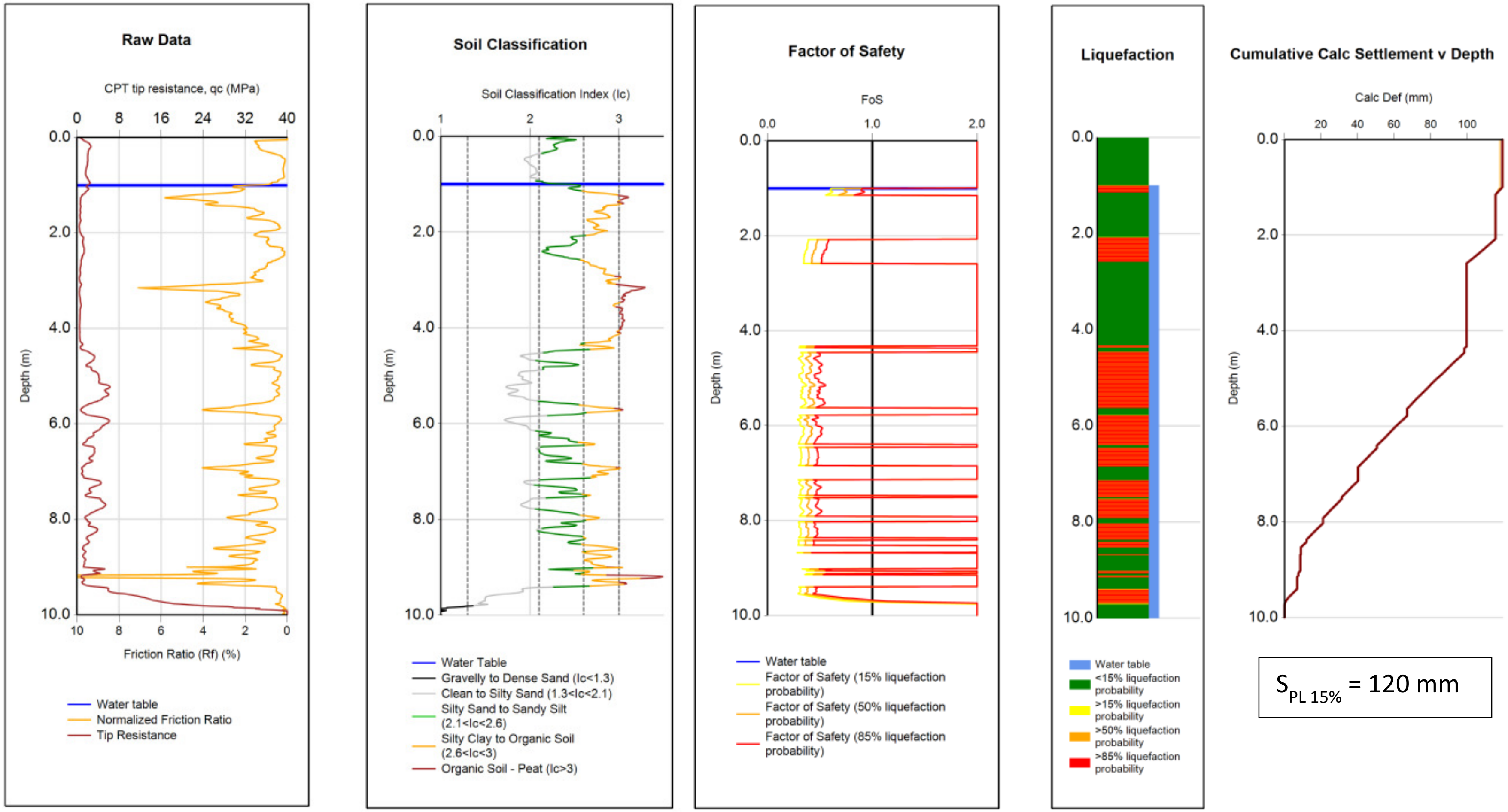
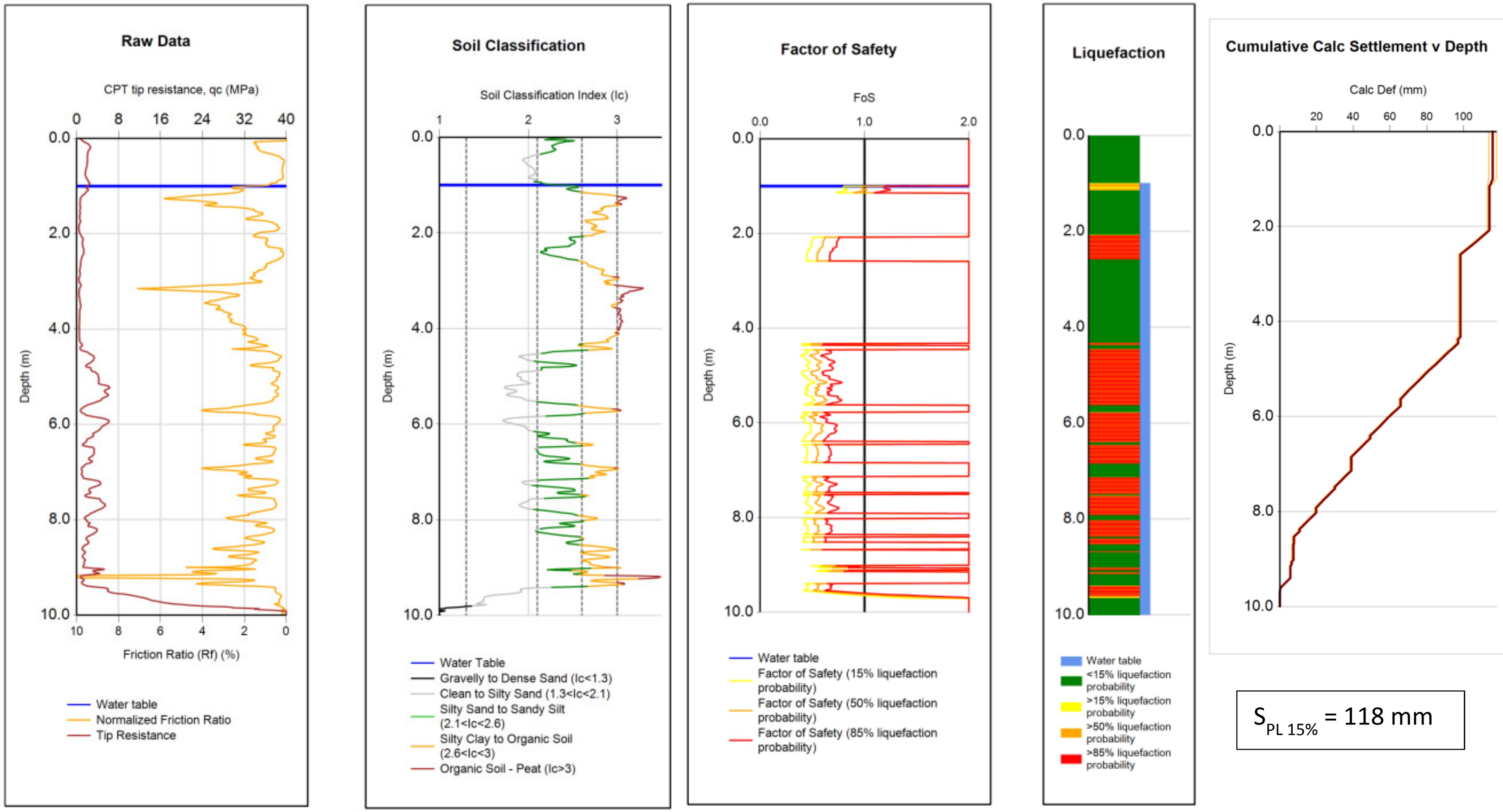


Figure 18 c). CPT A: ULS, M = 7.5, PGA = 0.35g

Figure 18 d). CPT A: ILS, M = 6.0, PGA = 0.30g



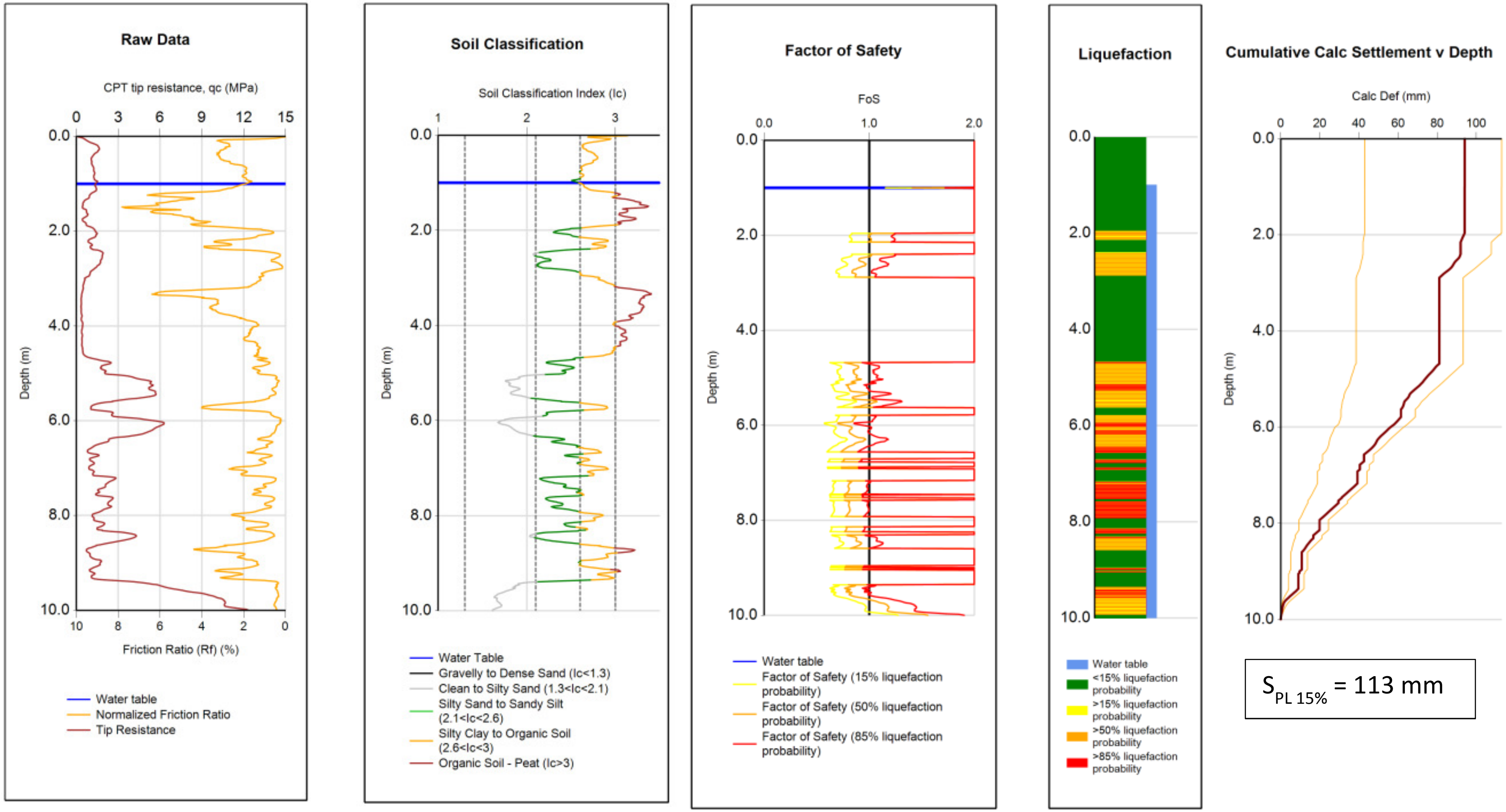


Figure 19 a). CPT B: SLS, M = 6, PGA = 0.19g

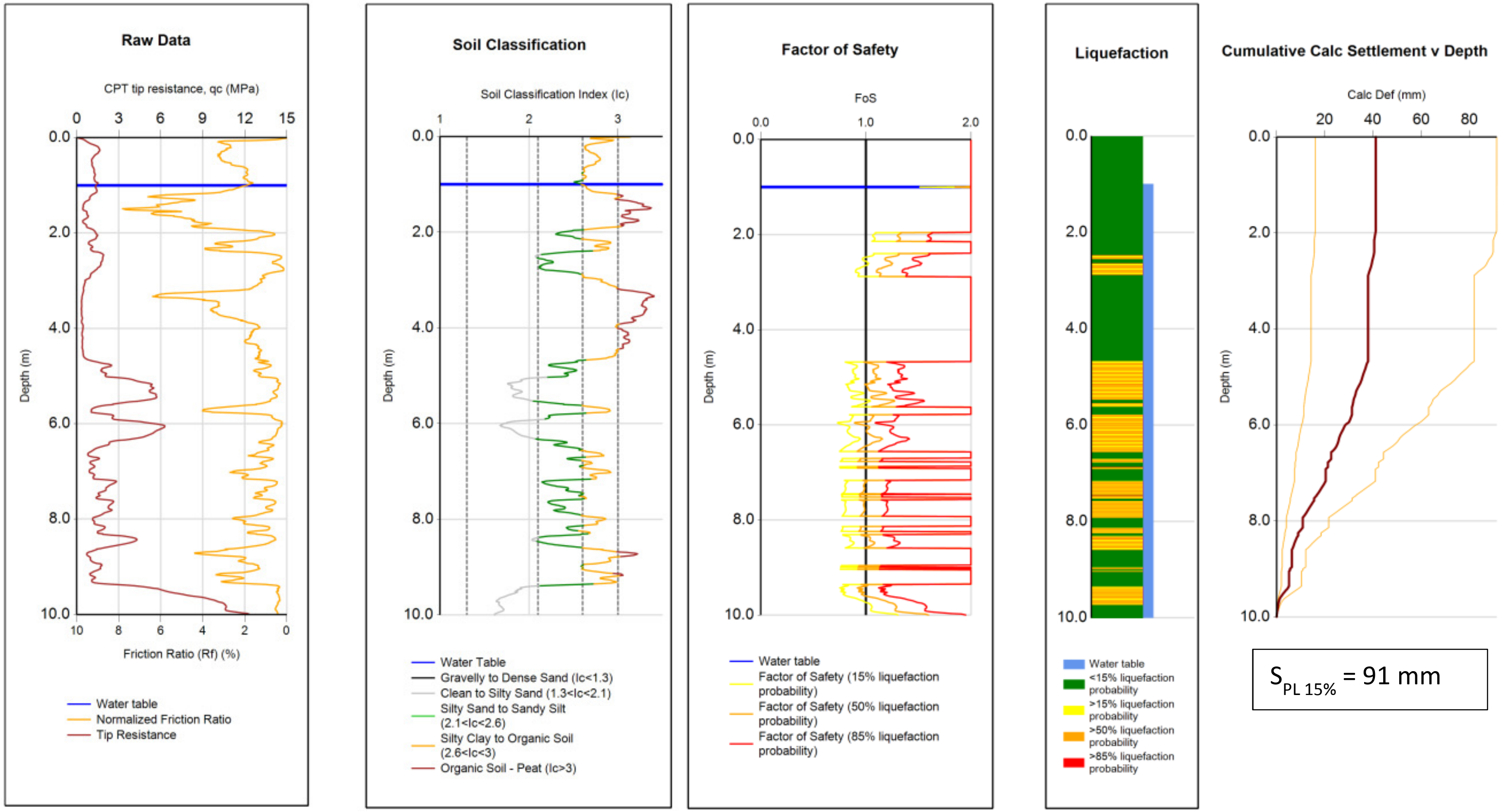


Figure 19 b). CPT B: SLS, M = 7.5, PGA = 0.13g

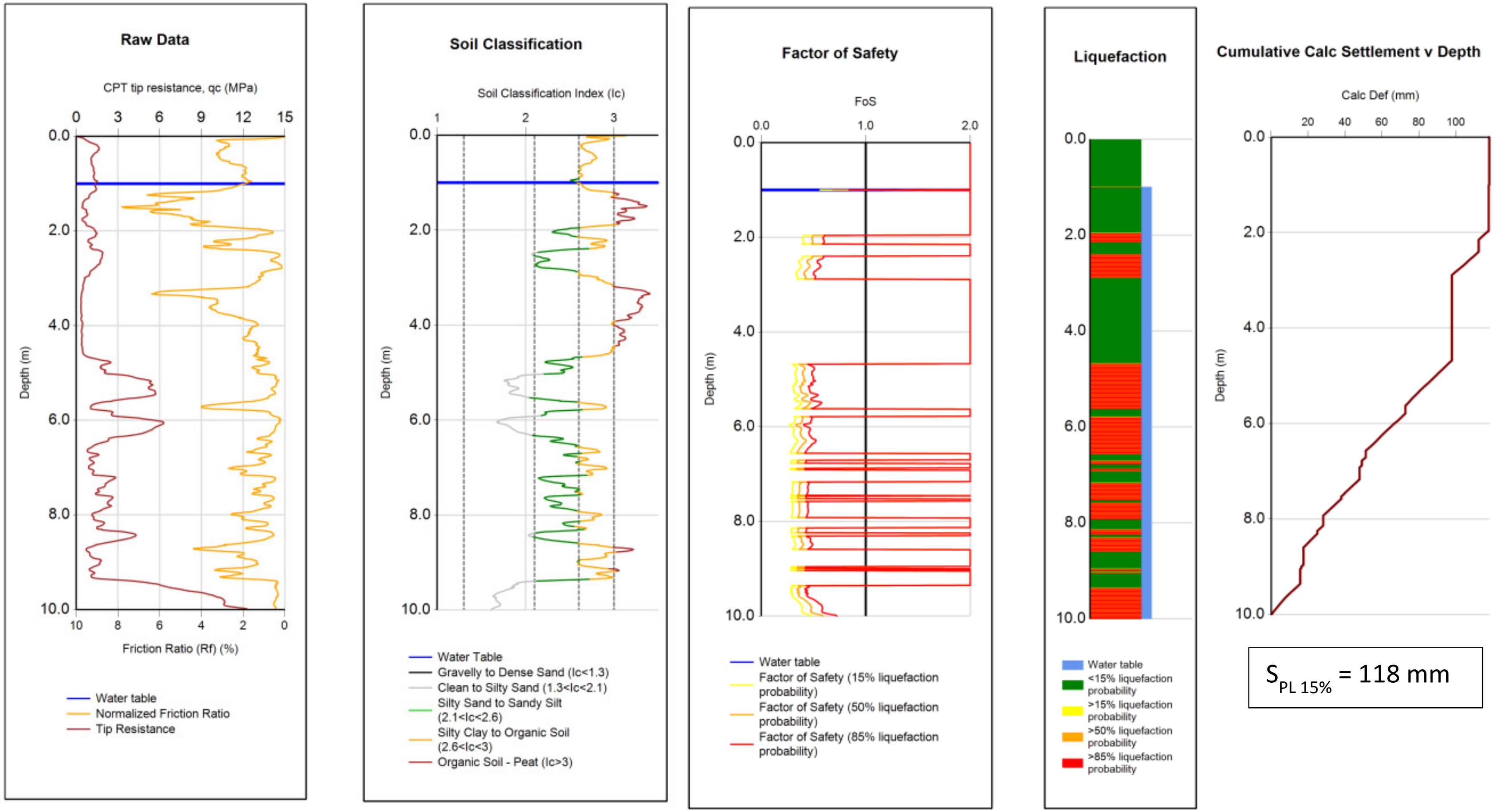
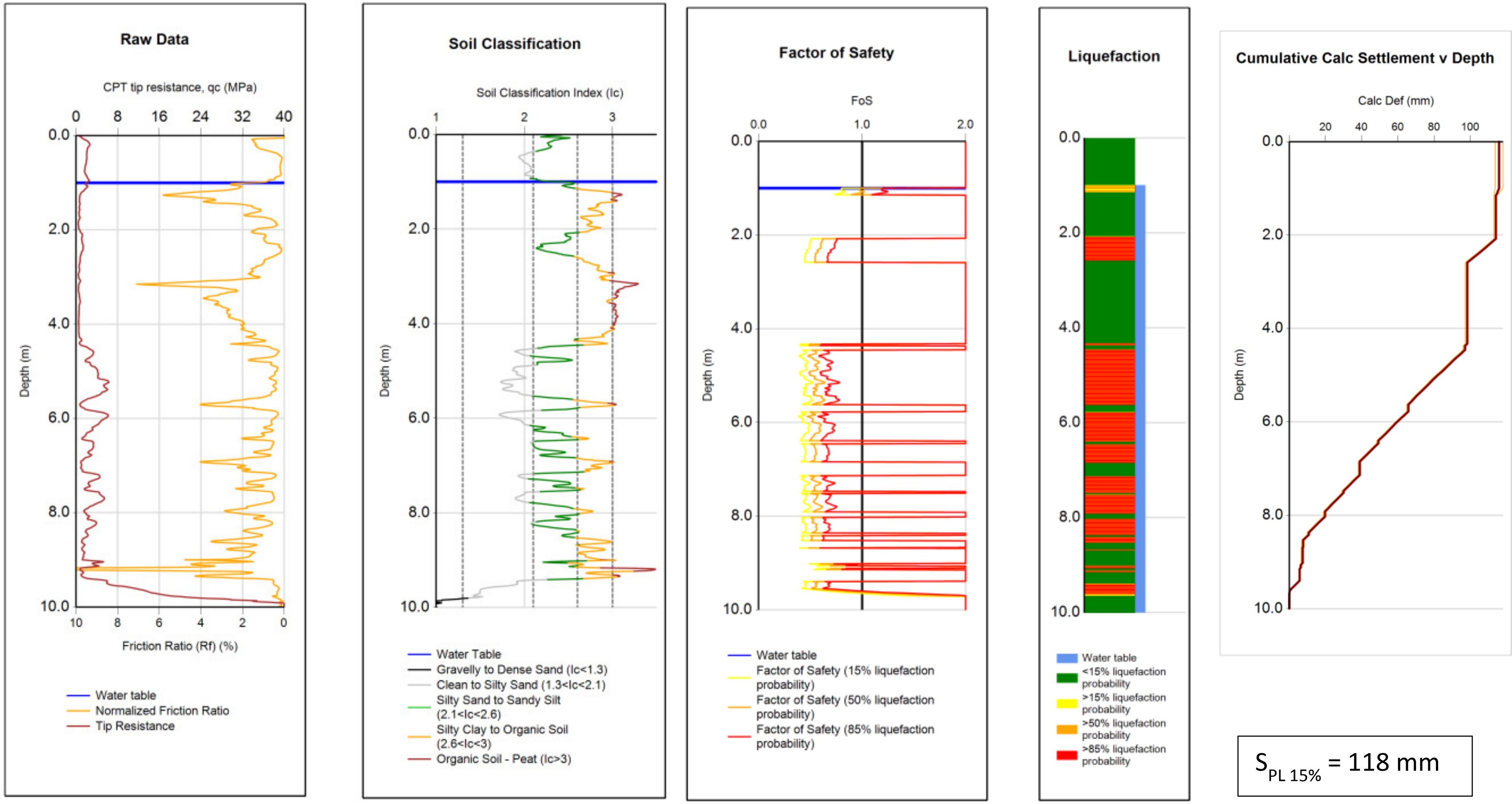


Figure 19 c). CPT B: ULS, M = 7.5, PGA = 0.35g



$S_{PL\ 15\%}$ = 118 mm

Figure 19 d). CPT B: ILS, M = 6.0, PGA = 0.30g

Liquefaction settlement

Calculated settlement

The liquefaction-induced settlements were calculated from a CPT-based triggering analyses using the methodology by Zhang et al. (2002) for Example 1. For CPT A and CPT B, the free-field ground surface settlements computed over the top 10 m of the soil profile are presented in Table 4.

Table 4 – Computed liquefaction settlements over upper 10 m from CPT A and CPT B (mm)

	CPT A	CPT B
SLS Case 1 M = 6.0, PGA = 0.19 g	113	113
SLS Case 2 M = 7.5, PGA = 0.13 g	88	91
ULS, M = 7.5, PGA = 0.35 g	120	118
ILS, M = 6.0, PGA = 0.30 g	118	118

The SLS Case 1 settlements are larger than those for SLS Case 2, hence the Case 1 values will be adopted for foundation design. The SLS index number is greater than 100 mm which is defined as "potentially significant" (refer to Table 12.5 of the Guidance Document).

Figures 18 and 19 illustrate where in the soil profile the predicted liquefaction settlement occurs. As shown in both the graphical liquefaction outputs and by the computed settlements above, the liquefaction profiles of the two CPTs are very similar. The majority of the predicted liquefaction occurs below a depth of about 4.5 m, with some liquefaction occurring within a 0.5 to 1 m thick layer beginning at a depth of about 2 m. The layers predicted to liquefy do not change significantly across the three levels of shaking assessed; however the likelihood of liquefaction (i.e., the P_L) of the layers increases at ILS and ULS.

Comparison to observed site/structure performance

It is noted that the predicted liquefaction settlements for the site are relatively large; even at the SLS level of shaking. The first step in comparing observed site performance with predicted performance is to determine whether the site was "sufficiently tested" as described for Example 1. The results of that analysis indicate that the site met the criteria of "sufficiently tested" for the 22 Feb 2011 earthquake, but fell just short of the criteria for the 04 Sept 2010 event. While not presented here, a liquefaction analysis of the site CPTs using the conditional median site PGAs indicated about 100 mm of settlement from each of these earthquakes (assuming a P_L of 15% and C_{FC} of 0.0). In terms of median conditional PGA, the February earthquake was in the order of an ILS-level event.

The fact that the site area experienced ground shaking in the order of SLS to ILS during the CES, yet only the 22 February event resulted in surface expression of liquefaction and some foundation damage, suggests that the site ground performance may have been better than predicted by the liquefaction analyses. It does not necessarily mean that liquefaction did not occur.

The liquefaction analyses suggest that the site in theory could have undergone total free-field ground surface settlements of around 200 mm as a result of the combined September and February earthquakes. The LiDAR data for the site area also indicates that the neighbourhood where the site is located may have experienced settlements of this magnitude. However, <u>the consequences</u> of this settlement to the land and house at the site were relatively minor – some minor ejecta and a compromised ring foundation (Type 2) as a result of the February event.

Refined liquefaction assessment

Due to the disparity between the site performance and the predicted liquefaction settlements associated with the level of shaking experienced, the liquefaction potential was further assessed using a P_L of 50% as discussed in Q&A 51. While no lab data is available for the site, the CPT data clearly indicates that the soils are relatively silty. Recent research indicates that a C_{FC} of 0.2 is probably appropriate for silty soils in many areas of Christchurch; including the area where the site is located (Lees *et al* 2015). Therefore, liquefaction analyses incorporating a C_{FC} of 0.2 were also conducted. While the C_{FC} value cannot be changed without technical justification (in the form of site-specific laboratory data), the additional analyses indicate the sensitivity of the liquefaction potential to the fines content.

The results of the additional liquefaction analyses are tabulated below to illustrate how a change in C_{FC} (if supported by lab testing) can affect the computed settlements:

Table 3 – Sensitivity of computed liquefaction settlements with P_L = 50% and C_{FC} = 0.2 – (SLS Case 2: M6.0/PGA – 0.19 g) (mm)

Case	CPT A	CPT B
P_L = 15%, C_{FC} =0.0	113	113
P_L = 50%, C_{FC} =0.0	91	94
P_L = 15%, C_{FC} =0.2	83	85
P_L = 50%, C_{FC} =0.2	51	59

The refined liquefaction analysis combined with site observations are considered to form a reasonable basis for concluding that future SLS liquefaction settlement is likely to be less than 100 mm.

Selection of foundation / ground improvement system

The results of the liquefaction analysis (i.e., based on a P_L of 50%) confirm that the site is TC 3 on the basis of the SLS settlements. The site appears to have a relatively thick, silty crust with only limited liquefaction potential at a depth of between about 2-2.5 and 3 m. It is noted that the predicted liquefaction is extensive throughout the soil profile below this crust (depth about 4.5 m). For this example, the predicted total ULS settlements are assumed to be acceptable with respect to impact on other aspects of the project (e.g., drainage, flooding for example) on the basis that the site and surrounding area appears to have settled relatively uniformly during the CES with relatively little consequential damage.

Obvious surface manifestation of liquefaction (ejecta, ground surface settlement) was observed in the 22 February 2011 earthquake; including moderate levels of liquefaction close to the subject site. The site was estimated to have experienced scaled mean ground shaking in the September 2010 and February 2011 events close to that of SLS and ILS level, respectively.

On the basis of the above assessment, the foundation solution selected for this site is a TC2 foundation combined with shallow ground improvement. Deep improvement is not considered necessary as total settlement is not a major concern and a sufficiently stiff shallow improvement combined with a robust TC2 foundation is anticipated to adequately mitigate potential differential settlement.

Due to the relatively silty nature of the site soils, in situ densification options (G1b – DC, G1c – RIC, shallow stone columns - G5a) are unlikely to be effective to adequately densify the liquefiable layer located at a depth of 2 to 3 m. Timber driven piles are also unlikely to achieve the necessary improvement of this layer. Shallow stone columns or RAP™ could potentially be used to stiffen the silty crust and thereby reduce the potential for damaging differential settlement (e.g., flexural distortion), but would likely require a relatively high ARR thus driving up cost.

A shallow raft (i.e., G1a, G1d, G2a, G2b) might also be an acceptable solution however the shallow groundwater may increase construction complexity and therefore cost. A raft would also need to be relatively stiff to mitigate the potential differential settlements associated with a large liquefaction event.

Considering the various advantages and limitations of each method, the option selected for this example is a 1.2 m thick geogrid-reinforced cement stabilised raft (option G2a). An in situ mixed soil cement raft would also be acceptable; particularly if shallow groundwater makes site excavation and fill placement difficult without dewatering. However, assuming that groundwater will not be a significant issue, the likely lower cost and the ability to easily confirm the constructed product makes the ex situ mixed raft the preferred option. Either G2 improvement option, if constructed well and with sufficient cement, will form a very stiff element that will resist flexural distortion due to liquefaction-induced settlement of the underlying soils; particularly when combined with a stiff TC2 foundation. The raft performance could be further enhanced by extending the raft 2 m beyond the perimeter foundation line though this is not required for building consent.

Quality control and other considerations

Quality control (QC) during construction of an ex situ mixed soil-cement raft is critical. The stiffness and strength of the raft is dependent on the soil having sufficient cement to bond strongly with the soil particles. Silty/clayey soils require considerably more cement than do clean sands to form an adequate bond. Appendix C4 provides guidance regarding minimum cement content, however, laboratory testing is recommended for silty/clayey soils to confirm that the design strength/stiffness can be achieved.

Careful control of soil moisture during mixing, and thorough mixing of the cement throughout the soil mass are also very important. High moisture contents or mixing of soil below the water table will require higher cement contents to control hydration. It is also important that the mixing process break down soil clumps which, if left intact, can result in uncemented weak zones within the raft. It is also very important to achieve sufficient compaction of the soil mass. The raft should be constructed in a series of 200 mm thick loose lifts, each one compacted to the required relative compaction prior to placing the next lift.

Proper placement of the geogrid is key for the grid to adequately resist future flexural distortion of the raft. The grid should be installed in accordance with the product manufacturer's instructions. This generally includes ensuring that the grids are placed level and are pulled tight as the overlying fill is placed in order to remove any wrinkles or folds.

QC testing for an ex situ mixed raft primarily consists of field compaction testing during construction, and periodic sampling of mixed material for laboratory strength/stiffness testing. This is discussed in some detail in the Appendix C4 method statement.

References

Bradley, B. A. and Hughes, M. (2012). Conditional Peak Ground Accelerations in the Canterbury Earthquakes for Conventional Liquefaction Assessment. Technical report prepared for Department of Building and Housing. 13 April.

Brown, L.J., Weeber, J.H. Geology of the Christchurch urban area. Scale 1:25,000, 1992.

Boulanger, R.W., and Idriss, I.M. (2014). CPT and SPT Based Liquefaction Triggering Procedures. Report No. UCD/CGM-14/01. Center for Geotechnical Modeling. University of California at Davis. April.

Idriss, I. M. and Boulanger, R. W. (2008). Soil Liquefaction during Earthquakes. Monograph MNO-12, Earthquake Engineering Research Institute.

Lees, J., van Ballegooy, S., F.J. Wentz F.J., (2015) Liquefaction Susceptibility and Fines Content Correlations of Christchurch Soils, 6th ICEGE Conference Christchurch New Zealand, 1-4 November 2015.

Ministry of Business Innovation and Employment (2012). Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence. December.

Zhang, G., Robertson, P.K., and Brachman R.W.I. (2002). Estimating liquefaction-induced ground settlements from CPT for level ground. Canadian Geotechnical Journal. Vol. 39 pp 1168-1180.

[Version: Rev 4. – Nov 2014]