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Evaluating Fully Nonlinear Effective Stress Site Response Analyses using Records from the Canterbury Earthquake Sequence

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ABSTRACT

The capability of one-dimensional (1D) seismic site response analyses to capture the seismic response of potentially liquefiable soils during strong shaking is examined. Specifically, the 1D fully nonlinear effective stress analysis program *DeepSoil* (V5.1) is employed to calculate the generation, dissipation, and redistribution of excess pore water pressure during earthquake strong shaking. The recorded strong motion data from the greater Christchurch area for six events of the 2010-2011 Canterbury earthquake sequence as well as the extensive site investigation data that have been obtained for this area were used to perform the seismic site response analyses. A comparison of calculated pseudo-acceleration response spectra from analyses to response spectra from recorded surface motions shows that for most cases a reasonable "fit" can be achieved, especially for the events that produced lower intensity ground motions. However, a common observation for the analyses conducted in this study is the relatively minor differences in the calculated acceleration response spectra at the surface for total stress nonlinear analyses and effective stress nonlinear analyses. The effective stress analyses were able to capture the generation of excess pore water pressures within critical layers, but this modeled response had a minimal effect on the calculated surface motions of the effective stress analyses compared to the total stress analyses conducted in this study.

ACKNOWLEDGEMENTS

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[No publications were produced by this research at the time of this report.]

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1. INTRODUCTION

The 2010-11 Canterbury earthquake sequence devastated much of the city of Christchurch, New Zealand and the surrounding land. Liquefaction during the 4Sep10 (M_w7.1) Darfield event affected approximately 10% of the area of Christchurch, while the 22Feb11 (M_w6.2) Christchurch event affected over 50% of the developed land (see Figure 1). Including these two events, there were a total of seven events with moment magnitude (M_w) greater than or equal to 5.5 between 4 September 2010 and 23 December 2011 that caused varying degrees of liquefaction in and around Christchurch. Moreover, the M_w 4.7 event that occurred on 26 December 2010 triggered isolated cases of liquefaction. Figure 2 shows the spatial distribution of seismicity that occurred during the Canterbury earthquake sequence (up to 17 December 2012), with the locations of the events of interest for this study shown.

The Canterbury earthquake sequence provides a unique opportunity to examine seismic soil response during strong shaking, particularly as it relates to the effects of liquefaction. During the events discussed above, some sites within Christchurch liquefied as many as five times, other sites only liquefied once or twice, and other sites never experienced soil liquefaction. The effects of liquefaction on critical infrastructure, including buildings and lifelines, during this Canterbury earthquake sequence were dramatic. Liquefaction during several earthquakes damaged reinforced concrete and steel buildings, masonry buildings, industrial facilities, and timber framed residential structures, as well as lifelines, including water supply, wastewater, natural gas, and transportation networks.

By taking advantage of the dataset provided by these earthquakes, engineers' understanding with regards to the response of critical infrastructure during major shaking events with extensive and damaging liquefaction can be improved. Understanding the effects of liquefaction, however, starts with being able to capture reliably the seismic response of sites that develop significant pore water pressures during earthquake shaking. This report capitalizes on the data provided by the vast network of strong motion stations operated by GeoNet throughout the greater Christchurch area and the site investigation data that has been collected by researchers and practitioners throughout Christchurch. The presented research focuses on evaluating the capabilities of one-dimensional, fully nonlinear effective stress seismic site response analytical procedures to model the seismic response of sites with and without significant shaking-induced pore water pressure generation.

This report starts with an overview of the dataset that was used to conduct the research followed by an explanation of the input motions that were used for seismic site response analyses. Next, details regarding the completed seismic site response analyses are presented; the parameters used as inputs for the one dimensional (1D) seismic site response program *DeepSoil* (V5.1, Hashash 2012) are discussed with a focus on the inputs for effective stress analyses. A brief discussion of the results and trends observed from a representative set of analyses is presented, and finally, closing remarks are made regarding the overall research and outcomes as well as guidance for future work.

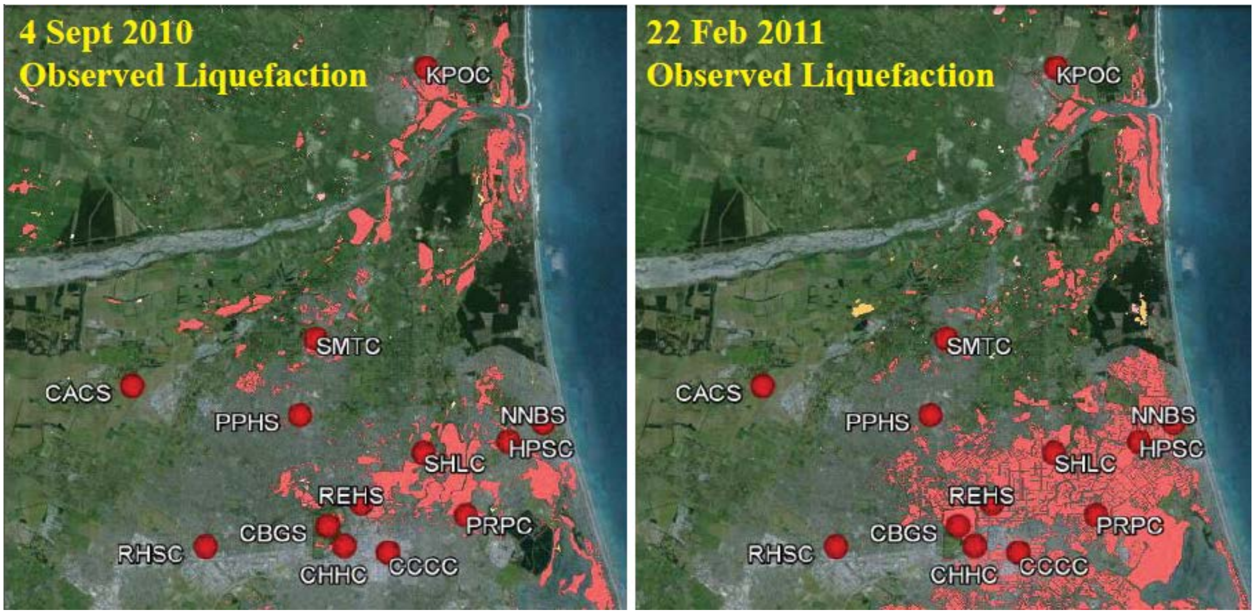

Figure 1: Observed liquefaction maps for 4Sep10 and 22Feb11 events; obtained from the Canterbury Geotechnical Database (21Aug14); *GNS Science Post 4 Sept 2010 Observations--Liquefaction Map;* also plotted with strong motion station sites examined in this study

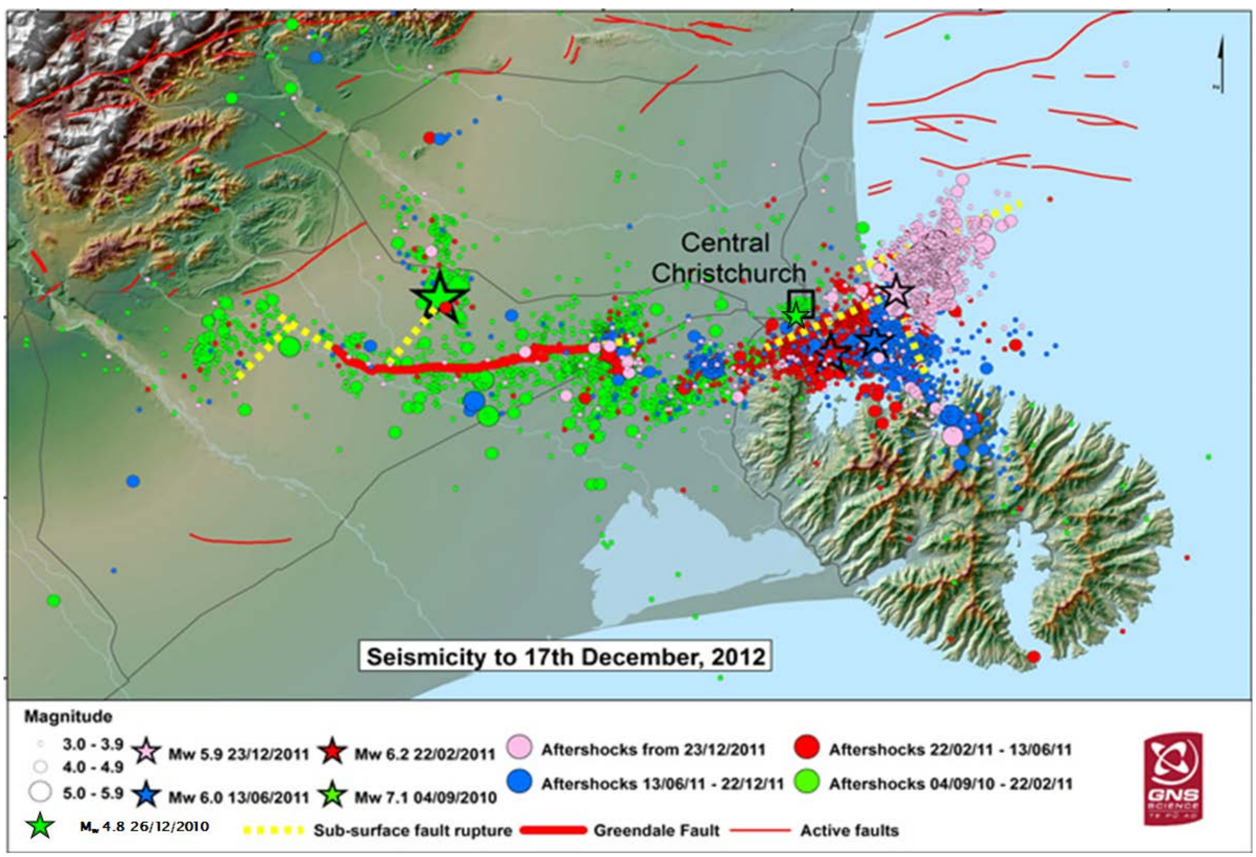

Figure 2: Location of 4Sep2010 Darfield main shock and subsequent aftershocks up to 17 Dec12; obtained from GNS Science website on 21Aug14 (http://www.gns.cri.nz/)

2. CANTERBURY EARTHQUAKE SEQUENCE

2.1 Introduction

The 2010-2012 Canterbury earthquake sequence of events was recorded by the vast network of strong motion stations operated by GeoNet. These strong motion records coupled with the large amount of site investigation data that has been conducted in and around Christchurch (and specifically at the location of strong motion stations (SMS)), before, during, and after the Canterbury earthquake sequence provide the basis for conducting the seismic site response analyses in this study. This section summarizes the pertinent information and data that was used to perform the analyses discussed in the latter portion of this report.

2.2 Overview of Strong Shaking Events

Seven events between 4 September 2010 and 23 December 2011 had an M_w greater than or equal to 5.5 for the Canterbury earthquake sequence. Of these seven events, seismic records from five earthquakes were examined in this study. In addition to these events with $M_w \geq 5.5$, the earthquake that occurred on 26 December 2010 with M_w4.7 was included in this study. These events were chosen based on the goals of the research, which was to evaluate the capabilities of a commonly used 1D fully nonlinear effective stress seismic site response analysis program (e.g., *DeepSoil*) to model soil response during strong shaking and potential liquefaction events. During these earthquakes, some sites within Christchurch liquefied several times, other sites only liquefied once or twice, and other sites never experienced soil liquefaction.

Table 1 provides the relevant information for each of the events studied. The horizontal acceleration records and respective response spectra from each event at each strong motion station were important in evaluating the performance of the analyses presented in the latter part of this report. All records were rotated to fault normal and fault parallel components based on the strike values listed in Table 1. Processed acceleration records for the 4Sep10 Darfield and 22Feb11 Christchurch events were provided to the authors by the Pacific Earthquake Engineering Research (PEER) center via personal communication. Records for these events can also be obtained from the GeoNet website (http://www.geonet.org.nz/), which is where the acceleration records for all other events were obtained. Appendix A (electronic) provides the rotated (fault normal and fault parallel), horizontal surface recordings at the strong motion stations studied for the six events of interest.

2.3 Overview of Strong Motion Stations and Event-Specific Parameters

Figure 1 above provides a map overview of the locations of the strong motion stations of interest. The stations to be studied were chosen on the basis of availability and proximity of site investigation data to accurately represent the subsurface profile of each station site. Furthermore, some of these sites showed signs of liquefaction in multiple events, which was evident in surface manifestation (i.e., soil ejecta; see Wotherspoon et al. 2013) or distinct features of the surface recording (this idea will be discussed further in a subsequent subsection of the report). Table 2 provides a detailed overview of relevant event parameters for each strong motion station for the events studied.

Table 1: Event characteristics for Canterbury earthquake sequence

Event	Date	NZ Local Time	M_w (USGS)	Hypocentral Latitude	Hypocentral Longitude	Strike (°)	Dip (°)	Z_{tor} (km)
1	4-Sep-10	04:35:46	7.1	-43.5382	172.1635	85.1	82.2	0.0
2	26-Dec-10	10:30:15	4.7	-43.5544	172.6615	74	84	2.0
3	22-Feb-11	12:51:42	6.2	-43.5644	172.6915	50	64	0.5
4	13-Jun-11	14:20:50	6.0	-43.5638	172.7431	162	67	1.41
5	23-Dec-11	12:58:36	5.8	-43.4862	172.7957	45	63	0.0
6	23-Dec-11	14:18:02	5.9	-43.5300	172.7428	57	51	1.47

Notes:

1) Moment magnitudes obtained from GeoNet (www.geonet.org.nz) regional Centroid Moment Tensor (CMT) solutions (Ristau 2008).
2) Strike, dip, and Z_{tor} values are based on Metadata received from Bradley (2013) via personal communication, except for the 22Feb11 event; the 22Feb11 values are based on Bradley and Cubrinovski (2011)

Table 2: Characteristics of event parameters at the strong motion stations studied

Station	4Sep11 M_w7.1		22Feb11 M_w6.2		13Jun11 M_w6.0		23Dec11 M_w5.8		23Dec11 M_w5.9		26Dec10 M_w4.7	
	PGA(g)	rup^2(km)	PGA(g)	rup(km)	PGA(g)	rup^2(km)	PGA(g)	rup(km)	PGA(g)	rup^2(km)	PGA(g)	rup^2(km)
CACS	0.2	11.7	0.21	12.8	0.14	16.2	0.07	19.4	0.08	16.7	0.02	13.1
CBGS	0.16	14.4	0.5	4.7	0.16	7.6	0.16	12.9	0.21	10.2	0.27	4.4
CCCC	0.22	16.2	0.43	2.8	-	-	0.13	11.1	0.18	8.7	0.23	2.6
CHHC	0.17	14.7	0.37	3.8	0.22	6.8	0.17	12.5	0.22	10.0	0.16	3.5
HPSC	0.15	21.7	0.22	3.9	0.26	5.5	0.2	6.12	0.26	3.2	0.05	6.6
KPOC	0.34	27.6	0.2	17.4	0.1	19.4	-		-	-	0.01	19.8
NNBS	0.21	23.1	0.67	3.8	0.2	5.6	-		-	-	0.04	7.8
PPHS	0.22	15.3	0.21	8.6	0.12	10.4	0.12	13.4	0.14	10.5	0.09	8.2
PRPC	0.21	19.3	0.63	2.5	0.34	3.7	0.29	8.1	-	-	0.09	3.7
REHS	0.25	15.8	0.52	4.7	0.26	6.8	0.2	11.5	0.25	8.8	0.25	4.4
RHSC	0.21	10.0		6.5	0.19	11.8	0.16	17.2	0.16	14.6	-	-
SHLC	0.18	18.6	0.33	5.1	0.18	6.3	0.26	9.1	0.28	6.1	0.16	5.6
SMTC	0.18	17.5	0.16	10.8	0.09	12.0	0.07	13.2	0.15	10.4	0.03	10.5

Notes:

1) PGA values from Bradley et al. (2014) for Darfield, Christchurch, 13Jun11, and 23Dec11 (M_w5.9) events; values for 23Dec11 (M_w5.8) and 26Dec10 events are from metadata provided by Bradley (2013) pers. comm.
2) R_{rup} values from Bradley et al. (2014) for Darfield, Christchurch, 13Jun11, and 23Dec11 (M_w5.9) events; values for 23Dec11 (M_w5.8) and 26Dec10 events are from metadata provided by Bradley (2013) pers. comm.

The 4Sep10 Darfield event had the largest moment magnitude (M_w) of all the events shown in Table 1. However, it was the 22Feb11 Christchurch event that caused the most intense shaking in the greater Christchurch area, which is evidenced by the generally higher recorded peak ground accelerations for the stations presented in Table 2. The relatively lower source-to-site distances of the Christchurch event compared to the Darfield main shock provide some explanation for the more intense shaking, which is illustrated in Table 2 (see Kaiser et al. 2012 for further discussion).

2.3.1 Subsurface Characterization of Strong Motion Stations

Over 10,000 cone penetration tests (CPT) alone have been completed in the greater Christchurch area. Much of these site investigation data can be obtained directly through the Canterbury Geotechnical Database (CGD; accessed 2014). In addition to these data, much of the site investigation information necessary to characterize the strong motion stations of interest was obtained through the work of other researchers, particularly Prof. Wotherspoon of the Univ. of Auckland and his research collaborators (see Wotherspoon et al. 2013). CPT data and surface wave testing results were the primary sources of information used for characterizing the subsurface of the strong motion station sites. Table 3 provides a summary of the site investigation data for each site.

A simplified subsurface profile for Christchurch is shown in Figure 3. It can be seen that in general the subsurface is comprised of surficial deposits varying in thickness from less than 10 m to over 40 m. These materials form the Springston Formation (primarily alluvial gravels, sands, and silts) in the western area of Christchurch and the Christchurch Formation (comprised of estuarine, lagoon, beach, dune, and coastal swamp deposits of sand, silt, clay, and peat) in the eastern part of the city (Cubrinovski et al. 2011). Below these deposits lies the well graded Riccarton gravel layer (Brown and Weeber 1992). Most site investigations characterize only the soils that overlie the dense Riccarton gravel. The assumed depth to the Riccarton gravel layer for each site studied is also listed in Table 3.

For the analyses presented in later sections, unit weights for all soils above the ground water table were assumed to be about 17.3 kN/m^3 and about 19.6 kN/m^3 below the ground water table. Event specific groundwater table depths were obtained from the Canterbury Geotechnical Database.

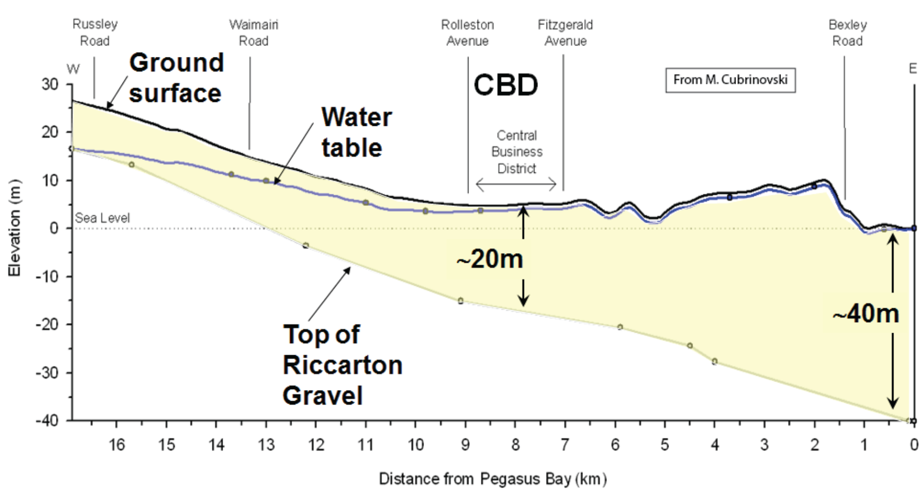

Figure 3: Simplified subsurface profile for Christchurch (from Cubrinovski et al. 2011.)

Table 3: Strong Motion Station Data Used for Site Characterization

Station ID	Latitude	Longitude	Depth to Riccaton Gravel(m) (1)	Available Information		
				Geophysics	CPTu (4)	Boreholes (4)
CACS	-43.4832	172.5300	6/14	SW(2)	-	BH-11529A
CBGS	-43.5293	172.6199	21.0	SW(2),	CBGS_CPT1(2)	CBGS_BH1(2), BH11793(CGD)
CCCC	-43.5381	172.6474	25.0	SW(2),	CPT484(CGD),CPT24862(CGD), CPT24865(CGD)	BH1759(CGD)
CHHC	-43.5359	172.6275	22.0	SW(2),	CPT425(CGD), CPT12257(CGD), CPT12258(CGD)	BH1756 (CGD),BH12255(CGD), BH26682 (CGD)
HPSC	-43.5016	172.7022	36.0	SW (3)	CPT47(CGD), CPT89(CGD),CPT18940(CGD)	BH16910 (CGD)
KPOC	-43.3764	172.6637	18.5	SW(2)		KPOC_BH1(2)
NNBS	-43.4954	172.7180	41.0	SW(2),	CPT33695(CGD),CPT1461(CGD) , CPT17254(CGD)	BH30210 (CGD), BH2685 (CGD),BH30211(CGD)
PPHS	-43.4928	172.6069	20.0	SW(2)	CPT1497(CGD)	BH34717(CGD),
PRPC	-43.5258	172.6828	28.0	SW(2),	CPT1396(CGD), PRPC_CPT2(2)	BH23529 (CGD)
REHS	-43.5219	172.6351	20.0	SW(2) ,	REHS_CPT1(2), REHS_CPT2(2),CPT386(CGD), CPT9215(CGD),CPT9217(CGD)	BH1735 (CGD), BH21735 (CGD)
RHSC	-43.5362	172.5644	11/16	SW(2), SW (3)	-	BH11529 (CGD)
SHLC	-43.5053	172.6634	27.0	SW(2),)	CPT626(CGD),CPT17584(CGD)	BH20985(CGD), BH20992(CGD), BH23531(CGD)
SMTC	-43.4675	172.6139	28.0	SW(2)	-	BH14315(CGD)

Notes

1) From Wootherspoon et al.(2013) and Bradley (2014) pers. comm.
2) From Wotherspoon et al. (2013).
3) From Wood et al. (2011)..
4) CGD: Canterbury Geotechnicall Database (2014).

Appendix B.1 provides processed CPT data for each strong motion station. The information obtained from CPT data was crucial not only in estimating the physical properties of the subsurface materials through relative correlations, but also in defining the stratigraphy of the sites. Furthermore, the McGann et al. (2014) Christchurch specific CPT-V_s correlation (McGann et al. 2014 a-c) provided the primary means for estimating the V_s profiles of the strong motion station sites. Assumed shear wave velocity profiles for each strong motion station site are provided in Appendix B.2. Table 3 shows that no CPT data were available at the time of writing for the strong motion station sites at CACS, RHSC, SMTC and KPOC (primarily due to the presence of near surface gravelly soil). For such cases V_s profiles calculated via the surface wave testing results of Wood et al. (2011) and provided in Wotherspoon et al. (2013) were used for site characterization.

Researchers are continuing to work at characterizing the subsurface profiles of the strong motion station sites throughout Christchurch. The authors were made aware of new subsurface investigation data for the strong motion station sites within the Central Business District of Christchurch (Wotherspoon et al. 2014) at the time of writing. This information is not incorporated into the analyses presented; however, a preliminary examination of these recent data by the authors shows mainly differences in the interpreted shear wave velocity of the Riccarton gravel, which we found had minor effects on the results presented in this study

2.4 Evidence of Liquefaction in Acceleration Time Histories

A prominent feature of several recordings from the Christchurch event, as well as a select number of records from the other events studied, is the presence of high frequency "spikes" in the acceleration time histories in the later part of the records (after initial S-wave arrivals). These spikes are indicative of the temporary stiffening of materials undergoing large cyclic shear strains as a result of soil liquefaction during strong shaking. This temporary increase in shear strength and stiffness allows for the propagation of high frequency energy (Bradley and Cubrinovski 2011). Figure 4 provides an example of an acceleration time history that exhibits such acceleration spikes. For further discussion on the evidence of liquefaction in earthquake records the reader is directed to Zeghal and Elgamal (1994).

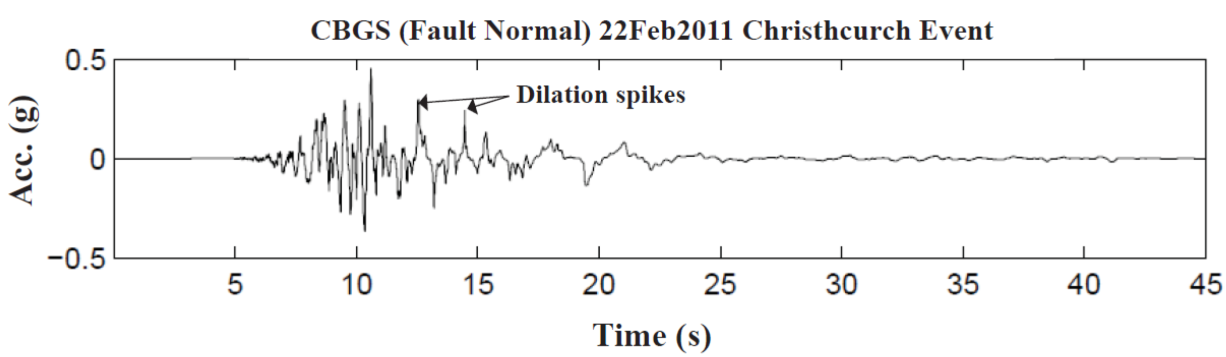

Figure 4: Example of dilation spikes for recording at CBGS (Fault Normal direction) for the 22 February 2011 Christchurch event

3. DECONVOLVED INPUT BASE MOTIONS

3.1 Brief Geologic Overview

All of the strong motion station sites of interest are situated within the Canterbury Plains. The general geology of this area comprises of distinct layers of gravels interbedded with layers of finer sediments (i.e., silts, sands, and even clays and peats) to a depth of over 500 m below the ground surface. Brown and Weeber (1992), Brown and Wilson (1988), and Forsyth et al. (2008) provide in-depth discussions about the geology of the Canterbury Plains and the general Christchurch area. Figure 5 provides a simplified geologic profile of the Christchurch region.

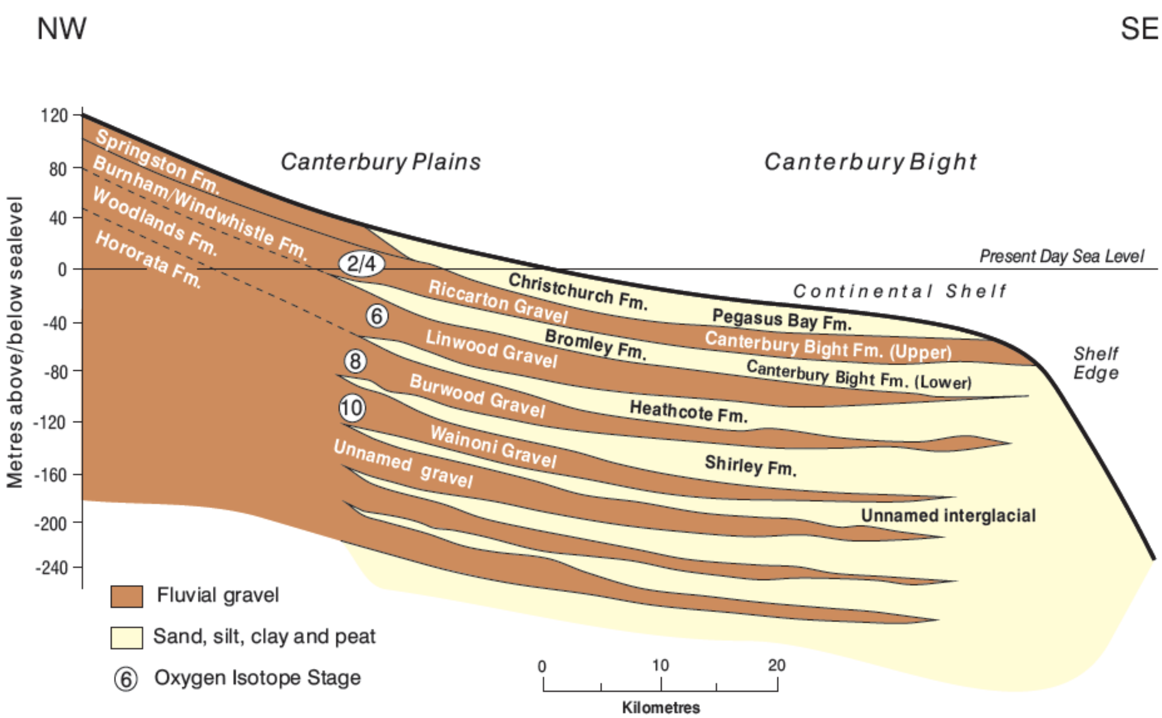

Figure 5: Geologic Cross section of the Christchurch area (from Forsyth et al. 2008; orig. from Brown & Weeber (1992) and Browne & Naish (2003))

Depth to "basement" rock for soils underlying the Canterbury Plains can be over 2 km below ground surface (Brown et al. 1995; Hicks 1989). Figure 6 provides contours of depth to basement rock for the Christchurch area. These deep sediment deposits coupled with the presence of the volcanic rock that makes up the Port Hills and Banks Peninsula to the southeast of central Christchurch create a basin structure. This basin structure leads to distinct signatures in the recorded motions of the seismic events outlined previously. In particular, for the Darfield and Christchurch earthquakes, indications of basin generated surface waves and wave guide effects are evident through large amplitude, long period recorded ground motions (e.g., Bradley and Cubrinovski, 2011, Bradley 2012, and Bradley et al., 2014).

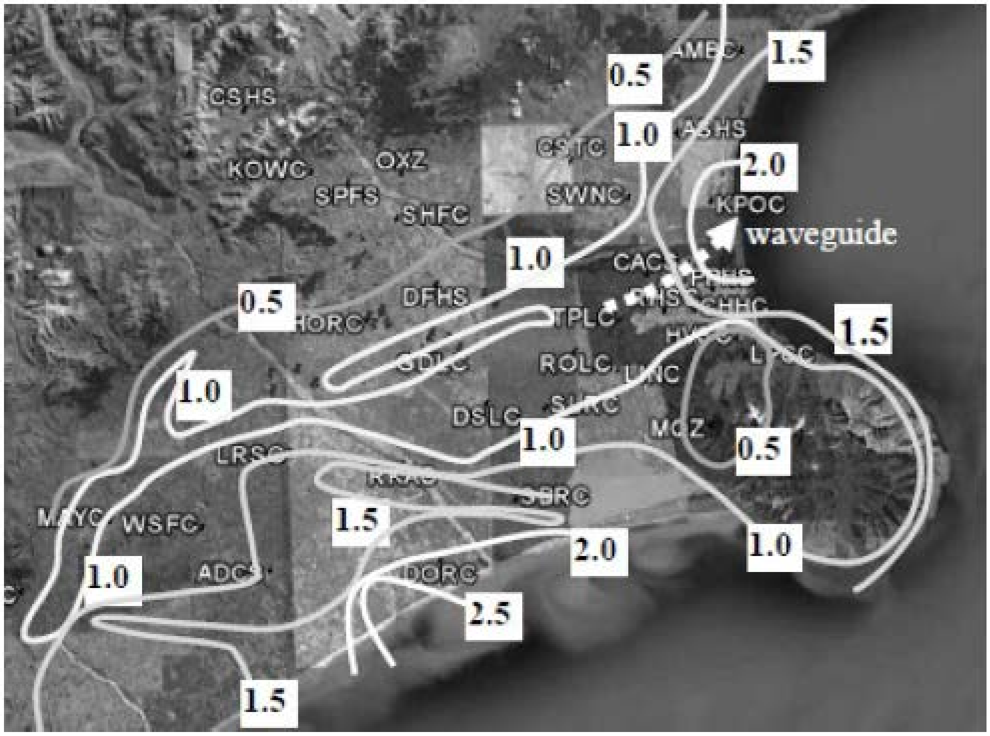

Figure 6: Depth to basement rock for Christchurch; contour depths are shown in kilometers (figure obtained from Bradley 2012)

3.2 Lack of Representative "Rock" Motions for Christchurch Basin

The deep basin structure that underlies the studied sites makes the selection of representative "rock" input motions difficult due to the absence of outcropping "rock" recordings on the north side of the Port Hills (i.e., Canterbury Plains side of the Port Hills). The Lyttelton Port strong motion station (LPCC) has a V_{s30} of about 792 m/s (Wood et al. (2011)), placing it in the category of engineering bedrock (i.e., B/C rock boundary for $V_s \approx 760$ m/s; see ASCE 7-10); however, the location of LPCC with respect to the locations of the events of interest and seismic energy propagation from these events make it a non-ideal input motion for seismic site response analyses in the Christchurch area (e.g., LPCC is located on the southern side of the Port Hills as opposed to the northern side as well as the hanging wall of the 22Feb11 Christchurch event as opposed to the footwall).

3.3 Deconvolution of Surface Motions

With the lack of representative "rock" input motions, as well as the difficulty in accurately characterizing the stratigraphy beneath the studied sites to a depth of engineering bedrock, deconvolving recorded surface motions becomes an attractive alternative. The basic concept of deconvolution consists of inputting an outcropping motion at the surface of a 1D soil column and using an equivalent linear analysis to calculate the acceleration time history at a point beneath the ground surface. This within motion can be converted to an outcropping motion and used as an input motion at the base of subsequent convolution analyses. Figure 7 provides an illustration of the deconvolution process. For further discussion on the topic of deconvolution of surface motions see: Kramer (1996), Silva (1988), and Idriss and Akky (1979).

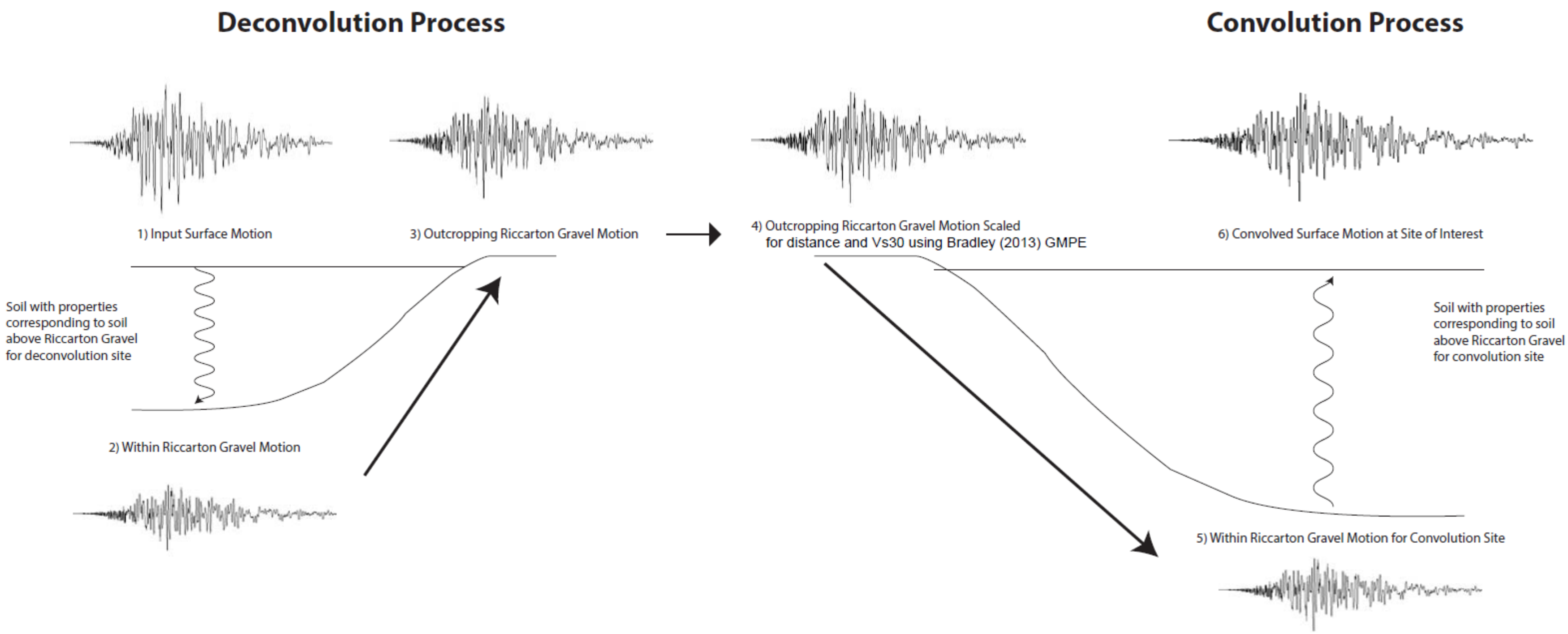

Figure 7: Overview of deconvolution process to obtain input motions for seismic site response analyses (convolution analyses)

3.3.1 Specific Deconvolution Process Followed

As stated, an equivalent linear analysis allows for the deconvolution of surface motions due to the linear solution process allowing for a motion to be transferred from any one point to another in a 1D soil column. Silva (1988) outlines a procedure to help avoid the situation of unrealistic motions being calculated at depth due to the propagation of the total surface motion via an equivalent linear analysis. The steps suggested to perform deconvolution that are recommended in Silva (1988) were followed by this study; they are:

1. Low pass (LP) filter recorded surface motion at 15 Hz and scale by 0.87
    a. SeismoSignal™ was used to perform 4th order, LP Butterworth filter
2. Input motion at surface using an equivalent linear analysis
3. Obtain motion from layer of interest at depth
4. Obtain the final iteration values of shear modulus reduction ($G/G_{max)}$ and material damping (λ) for each layer during the deconvolution process
5. Using a linear analysis with the final values of G/G_{max} and λ from step 4 for each layer, perform the deconvolution process again by placing the LP filtered (15Hz) full surface motion (i.e., not scaled by 0.87) at the surface and obtain the "final", outcropping, deconvolved motion

SHAKE2000 (Ordonez 2000) was utilized to perform all deconvolution analyses. This software does not allow the user to view the final G/G_{max} and λ values that result from a deconvolution analysis. To obtain these values for each layer of the 1D column being analyzed, an extra step to the above procedure must be included where the obtained motion from step 3 must be inserted at the depth of the soil column and a "typical" convolution analysis must be performed (i.e., the reverse of steps 3 and 2).

3.3.2 Riccarton Gravel as Half-Space for Analyses

The Riccarton gravel layer shown in Figures 3 and 5 was used as the half-space for deconvolution analyses and subsequent seismic site response analyses. The impedance contrast between the stiff Riccarton gravel and the softer overlying surficial deposits as well as the presence of this layer throughout the Christchurch region were two of the main reasons for choosing this layer as the half-space for analyses. As discussed above, the results of the deconvolution procedure were outcropping Riccarton gravel motions to be used as inputs for seismic site response analyses.

3.3.3 Deconvolution at Selected Strong Motion Station Sites (CACS and RHSC)

The strong motion station sites used for the deconvolution procedure were Canterbury Aero Club station (CACS) and the Riccarton High School station (RHSC); the locations of these station are shown in Figure 1. These sites are located on soil that did not show surface manifestations of liquefaction (Figure 1) during any of the events of interest, and are believed to have shown minimal nonlinear response during shaking; these points are important to note as an equivalent linear solution to the seismic site response cannot fully capture the nonlinear response of soils, but is necessary for deconvolution. Furthermore, as can be seen in Table 3, the depth to the Riccarton gravel layer for these sites is the lowest among the 13 strong motion station sites listed, which requires the surface motion to be deconvolved over a relatively shallow profile.

For the deconvolution process, the empirically based normalized shear modulus reduction and material damping relationships from Darendelli (2001) were used for all material above the Riccarton gravel. The V_s profiles assumed for each site are shown in Appendix B. It should be

noted that two different Vs profiles were considered for both CACS and RHSC to complete the deconvolution in order to account for the epistemic uncertainty in the site characterization of these stations.

3.4 Scaling of Motions for Seismic Site Response Analyses at Strong Motion Stations

The deconvolved Riccarton gravel motions were scaled using the New Zealand specific ground motion prediction equation outlined in Bradley (2013). By using the same source parameters for a given event (i.e., M_w, Z_{tor}, dip, etc.) and changing the distance parameter (R_{rup}) and the V_{s30} between the sites where the deconvolution was performed and the sites where convolution analyses were carried out allows for an average scale factor across all periods to be calculated. The parameter V_{s30} was used as a proxy to scale between the different assumed values of V_s of the Riccarton gravel (see Appendix B). Table 4 provides the scale factors for each station and event.

Table 4: Scale factors for input motions at strong motion stations for seismic site response analyses

Station	4Sep11 M_w7.1		22Feb11 M_w6.2		13Jun11 M_w6.0		23Dec11 M_w5.8		23Dec11 M_w5.9		26Dec10 M_w4.7	
	CACS	**RHSC**	**CACS**	**RHSC**	**CACS**	**RHSC**	**CACS**	**RHSC**	**CACS**	**RHSC**	**CACS**	**RHSC**
CBGS	0.85	0.76	2.15	1.22	2.08	1.48	1.61	1.40	1.69	1.45	3.20	--
CCCC	0.78	0.69	2.74	1.55	--	--	1.90	1.65	1.98	1.70	4.59	--
CHHC	0.93	0.83	2.61	1.47	2.53	1.79	1.90	1.65	1.94	1.67	4.32	--
HPSC	0.63	0.56	2.48	1.40	2.80	1.98	3.37	2.92	3.96	3.39	2.42	--
KPOC	0.47	0.42	0.73	0.42	0.82	0.58	--	--	--	--	0.47	--
NNBS	0.61	0.54	2.57	1.45	2.83	2.00	--	--	--	--	2.11	--
PPHS	0.81	0.73	1.44	0.82	1.69	1.19	1.57	1.36	1.75	1.50	1.93	--
PRPC	0.78	0.70	3.16	1.78	3.77	2.67	3.05	2.64	--	--	4.55	--
REHS	0.92	0.82	2.45	1.38	2.66	1.89	2.21	1.92	2.30	1.97	3.98	--
SHLC	0.68	0.61	2.07	1.17	2.42	1.71	2.31	2.01	2.62	2.25	2.65	--
SMTC	0.76	0.68	1.25	0.71	1.47	1.04	1.69	1.46	1.77	1.52	1.44	--

4. SEISMIC SITE RESPONSE ANALYSES OVERVIEW

4.1 Introduction

The following section details the procedures of the 1D seismic site response analyses that were completed for this study. Nonlinear effective stress analyses, nonlinear total stress analyses, and equivalent linear analyses were performed for each station listed in Table 2 for each event of interest (see Table 1) that was recorded (i.e., if a particular event was not recorded for a particular station, no analysis was carried out for that combination of station and event). The site investigation data discussed in section 2.3.1 and provided in Appendix B was used to help correlate and estimate the inputs for the analyses.

4.2 Representation of Strain Dependent Soil Response

4.2.1 Shear Modulus Reduction and Material Damping Curves

As with the deconvolution analyses, the work of Darendeli (2001) was used to estimate normalized shear modulus reduction and damping curves for each site where seismic site response analyses were completed. The input parameters used for the Darendeli (2001) model for each layer of each strong motion station site are organized in table form in Appendix C. In general, the soils for each subsurface profile were considered to be non-plastic (PI=0), and normally consolidated (OCR=1). The mean confining pressure for each layer (σ'_m) was determined based on the unit weights mentioned in section 2.3.1 (i.e., 17.3 kN/m^3 above GWT and 19.6 kN/m^3 below GWT) and a K_o of 0.5 (i.e., $\sigma'_m = 2/3\sigma'_v$).

4.2.2 Strength Correction of Shear Modulus Reduction and Material Damping Curves

In general, for the materials studied within this research, the Darendeli (2001) relationship tended to underestimate the assumed shear strength of a soil, though it did sometimes overestimate the shear strength. Figure 8 illustrates this idea by showing the assumed shear strength, as well as implied shear strength from the Darendeli (2001) relationship for a specific case. In the illustrated case it can be seen that the Darendeli (2001) relationship significantly underestimates the shear strength of the soil (see Stewart and Kwok, 2008 and Stewart et al., 2008 for further discussion on this topic).

To remedy the potential misrepresentation of soil shear strength, Stewart and Kwok (2008) suggested a procedure to transition from an empirically based shear modulus reduction curve (or one based on material-specific testing, if available) to a strength based shear modulus reduction curve at a specified strain level. An updated procedure was proposed by Yee et al. (2013) that allows for a hybrid shear modulus reduction curve (backbone curve) to be calculated. The Yee et al. (2013) procedure was used to modify all shear modulus reduction curves calculated by the Darendeli (2001) relationship. Due to a lack of published guidance in correcting the material damping curve to capture large strain behavior, a hybrid damping curve was calculated that transitioned from the damping curve calculated from the Darendeli (2001) relationship to a strength based material damping curve via a linear (in semi-log space) approximation. Figure 8 illustrates the corrections made to the shear modulus reduction and material damping curves for the performed seismic site response analyses.

A fitting procedure of the target shear modulus reduction and material damping curves is employed within *DeepSoil*. This fitting was done using the MRDF procedure proposed by Phillips and Hashash (2009), which is implemented in *DeepSoil*. The procedure proposed by Hashash et al. (2010) was followed for all nonlinear site response analyses to ensure that the

implied shear strength of the fitted shear modulus reduction curve for a given material was approximately equal to the assumed shear strength. This procedure requires an iterative adjustment of the target shear modulus reduction curve to capture the assumed shear strength for a given material.

4.3 Overview of Seismic Site Response Analyses

DeepSoil was used to complete all 1D seismic site response analyses, including equivalent linear, nonlinear total stress, and nonlinear effective stress analyses. For more information on the details of this software see the *DeepSoil* user manual and tutorial (V5.1, Hashash 2012). This subsection details the procedures followed to complete each type of analysis.

4.3.1 Equivalent Linear Seismic Site Response Analyses

The strength corrected shear modulus reduction curve calculated using the Yee et al. (2013) procedure for each layer of a given 1D soil profile was used directly for all equivalent linear analyses. The benefit of being able to enter discrete points to define the shear modulus reduction curves for an equivalent linear analysis (as opposed to having to fit a target curve for a nonlinear analysis) allows for this direct input. Material damping curves for equivalent linear analyses were the same material damping curves used in nonlinear site response analyses (defined using discrete points). These material damping curves were the result of fitting the strength corrected shear modulus reduction and damping curves for nonlinear analyses, which is discussed more below. The reason for the use of the fitted material damping curve versus the material damping curve obtained directly from the strength correction outlined in the previous subsection is due mostly to the hyperbolic representation of the material damping curve that the fitted procedure provides, as well as a better representation of material damping at large strains via the fitting procedure implemented in *DeepSoil*.

The ratio of effective shear strain to maximum shear strain (effective shear strain ratio) proposed by Idriss and Sun (1992) was used. This expression relates the effective shear strain ratio to an earthquake's magnitude via the following expression: $\gamma_{eff} = (M_w\text{-}1)/10$. The frequency independent complex shear modulus (Hashash 2012) was used in the equivalent linear analyses completed with *DeepSoil*. All input motions were input as outcropping motions with a damping ratio of 2% and a unit weight of 21.2 kN/m^3 used for the half-space.

4.3.2 Total Stress Nonlinear Seismic Site Response Analyses

The strength corrected hybrid shear modulus reduction and material damping curves described previously were used as target curves to define the dynamic properties for each soil layer of a given site. These target curves were fitted using the MRDF-UIUC pressure dependent hyperbolic fitting procedure implemented in *DeepSoil*. This fitting procedure allows for the best fit of the target shear modulus reduction and material damping curves to be found, but also introduces the reduction factor proposed by Phillips and Hashash (2009).

After the fitting procedure was carried out, implied shear strengths were checked to ensure that the assumed shear strength was being captured. If the implied shear strength was incorrect, an iterative procedure to adjust the target curves was completed for each layer according to the procedure outlined in Hashash et al. (2010) until the implied shear strength was within +/-5% of the assumed value for a shear strength greater or equal to 20 kPa or +/- 1 kPa for a shear strength value less than 20 kPa.

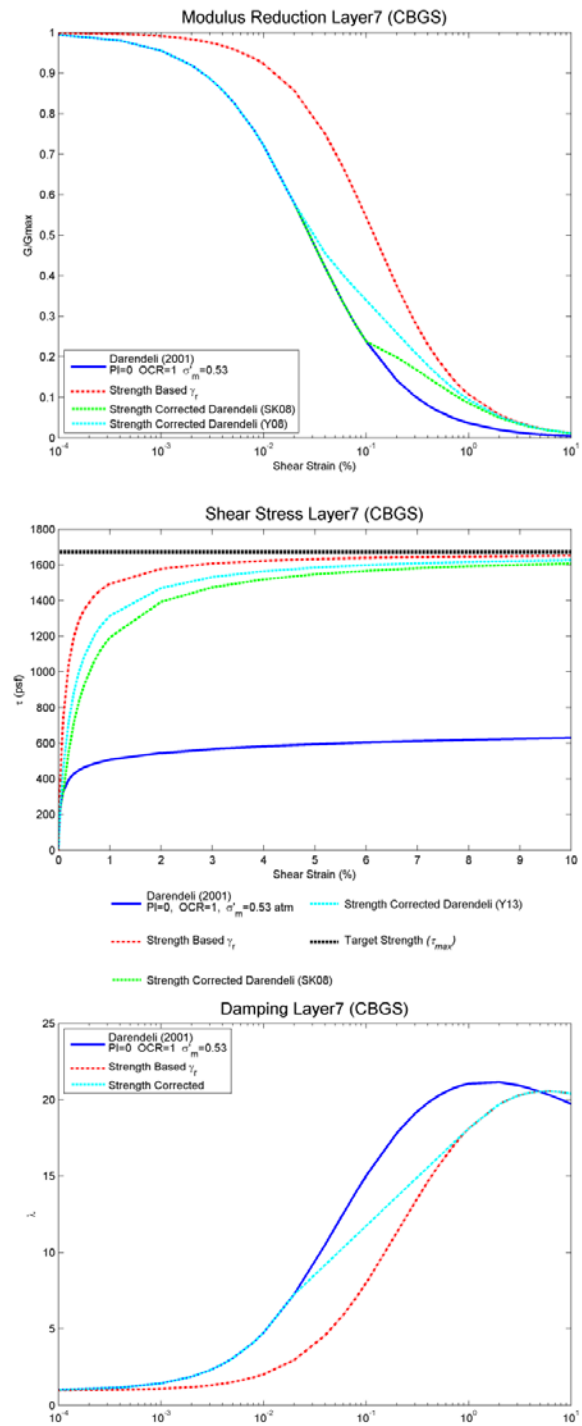

Figure 8: Adjustment of target curves for shear modulus reduction and material damping; shear stress vs. shear strain shown for reference

The frequency independent small strain damping formulation proposed by Phillips and Hashash (2009) was used to calculate viscous damping for all nonlinear site response analyses. This damping formulation removes many of the limitations of the traditional, frequency dependent Rayleigh Damping formulation (Hashash 2012).

4.3.3 Effective Stress Nonlinear Seismic Site Response Analyses

The parameters found using the procedures outlined above for representing the dynamic soil properties of a site for total stress nonlinear seismic site response analyses were used for the effective stress analyses. The effective stress nonlinear seismic site response analyses take into account the generation and build-up of pore water pressure during seismic shaking, as well as the dissipation and redistribution of excess pore water pressures. The generation of excess pore water pressures leads to a reduction in the stiffness and strength of a soil. This reduction can be incorporated into a nonlinear analysis through the incorporation of a degradation parameter in the estimation of shear modulus and shear strength.

This study consisted of primarily non-plastic, cohesionless soils. These soils were modelled using the pore water pressure model for sands originally developed by Dobry et al. (1985) and presented in Matasovic (1993). Equation 1 provides the governing equation of this model:

$$u_N^* = \frac{p*f*F*N*\left(\gamma_c-\gamma_{tvp}\right)^s}{1+f*F*N*\left(\gamma_c-\gamma_{tvp}\right)} \tag{1}$$

The parameters for the pore water pressure model would ideally be selected based on curve fitting site specific, undrained, cyclic test results. Due to a lack of site specific information for the strong motion stations, the *D-MOD2000* manual (Matasovic and Ordonez 2012) and the work of Carlton (2014) was used in the selection of parameters. Based on this information, the following logic was used in the selection of parameters:

- f was assumed to be 1 for all analyses; i.e., only 1D pore water pressure generation was considered
- p was assumed to be 1 for all analyses
- γ_{tvp} was chosen based on the value of shear strain (γ) at G/G_{max}=0.65
- In general, F was chosen based on the guidance of the correlation presented by Carlton (2014) that uses the soil information from Matasovic and Ordonez (2012) to correlate F to shear wave velocity (V_s in m/s). In some particular cases F was adjusted to prevent unrealistic large shear strains, or unexpected high pore water pressure generation (e.g., layers of silty clay material above the Riccaton gravel). The correlation is as follows:
  $$F = 3810 * V_s^{-1.55}$$
- s was chosen based on a similar correlation from Carlton (2014) which relates this parameter to fines content (FC in percent)
  $$s = (FC+1)^{0.1252}$$

Where FC was correlated from CPT data using the mean minus one standard deviation of the Christchurch, NZ specific correlation proposed by Robinson et al. (2013).

Furthermore, the additional exponent v considered by Matasovic (1993) and Matasovic and Vucetic (1993) in the calculation of the degradation parameter was incorporated into the

analyses. A value of v equal to 3.8 was assumed for all materials based on the work of Matasovic (1993).

Pore water dissipation and redistribution can be accounted for simultaneously with the generation of excess pore water pressure in a nonlinear effective stress analysis. The dissipation and redistribution of excess pore water pressure is modeled using Terzaghi's 1D consolidation theory. The solution of *DeepSoil* assumes dissipation only in the vertical direction (Hashash 2012). This model requires only the specification of the coefficient of consolidation (c_v), which was estimated based on CPT correlations of soil permeability (k) and constrained modulus (M) so that:

$$c_v = \frac{k * M}{\gamma_w} \quad (2)$$

The effective stress parameters used for all nonlinear effective stress seismic site response analyses are tabulated and provided in Appendix C.

5. RESULTS AND DISCUSSION

5.1 Introduction

This section provides an overview of select results from the seismic site response analyses. Key observations and trends supported by the results are presented. The section is organized so that it progresses from discussing general trends noticed from event-to-event to discussing specific observations made for various combinations of studied sites and events. Selected results are presented to illustrate these observations; Appendices D-F provide a comprehensive presentation of the results of all of the analyses performed as part of this study.

A total of eight input motions were used for five of the six events studied to complete the seismic site response analyses. For the 26Dec11 event only 4 input motions were considered as no recording was taken at RHSC for this event (i.e., only inputs from deconvolution at CACS were used). These input motions corresponded to two possible subsurface profiles at each of the two strong motion station sites used for deconvolution (CACS and RHSC) in two directions (fault normal and fault parallel). It was found that for a given deconvolution site, the input motions resulting from the consideration of two different soil profiles yielded similar response spectra at the ground surface for the analyses completed. Therefore, the results that correspond to the input motions from the deconvolution at CACS for the *Woth1* shear wave velocity profile and RHSC for the *Woth1* shear wave velocity profile (see Appendix B) are focused on below.

Representative soil profiles were chosen based on the site investigation data discussed previously. Using these respective site investigations for each strong motion station site, liquefaction triggering analyses were carried out using the Boulanger and Idriss (2014) liquefaction triggering procedure (see Appendix E for results of the triggering analyses). These liquefaction triggering analyses provided a guide for identifying layers that should have or should not have experience significant generation of pore water pressure during strong shaking. To this extent, the liquefaction triggering analyses provided a means to assist with selecting final parameter values used in the nonlinear effective stress analyses at the strong motion station sites, specifically the F and s parameters for pore water pressure generation, as well as the coefficient of consolidation (c_v) for dissipation and redistribution of excess pore water pressures.

5.2 Pseudo-Spectral Acceleration Comparison Between Event Analyses

The strong motion station recordings for each event of interest provide a means for assessing the results of the calculated motions at the surface of each site of interest for all seismic site response analyses completed using *DeepSoil* (including equivalent linear (EQL), nonlinear total stress (TS), and nonlinear effective stress analyses (ES)). Pseudo-acceleration response spectra (5% damped; abbreviated as response spectra) were used to simplify the comparisons between recorded surface motions and those calculated from analyses. Residuals were calculated between recorded and calculated spectral acceleration values on a period-by-period basis to quantify the "fit" of the response spectra calculated from analyses to those of recorded surface motions. The residuals are calculated as:

$$\delta = \ln\left(S_{a_{recorded}}\right) - \ln(S_{a_{predicted}})$$

A presentation and discussion of the salient observations of spectral acceleration residuals plotted as a function of period for a select number of events is provided below. Appendix F provides all plots of residuals as a function of period for all analyses.

5.2.1 Spectral Acceleration Comparisons for 22 February 2011 Christchurch Event

Figure 9 provides a plot of the residuals as a function of period for analyses completed using the *CACS Woth1 FN* (fault normal) input motion for the Christchurch event. One of the most noticeable characteristics of the results presented in Figure 9 is the underestimation of the spectral accelerations for periods less than 0.1 s (i.e., positive δ values), especially for the nonlinear total stress and effectives stress analyses. This underestimation is not the case for station sites HPSC and SMTC, where the response spectra values from analyses are consistently higher than the recorded spectra values across the considered period range (i.e., negative δ values). Additionally, the residuals from the total stress nonlinear analyses and the effective stress nonlinear analyses are similar, while there are noticeable differences for the residuals from equivalent linear analyses compared to the nonlinear analyses.

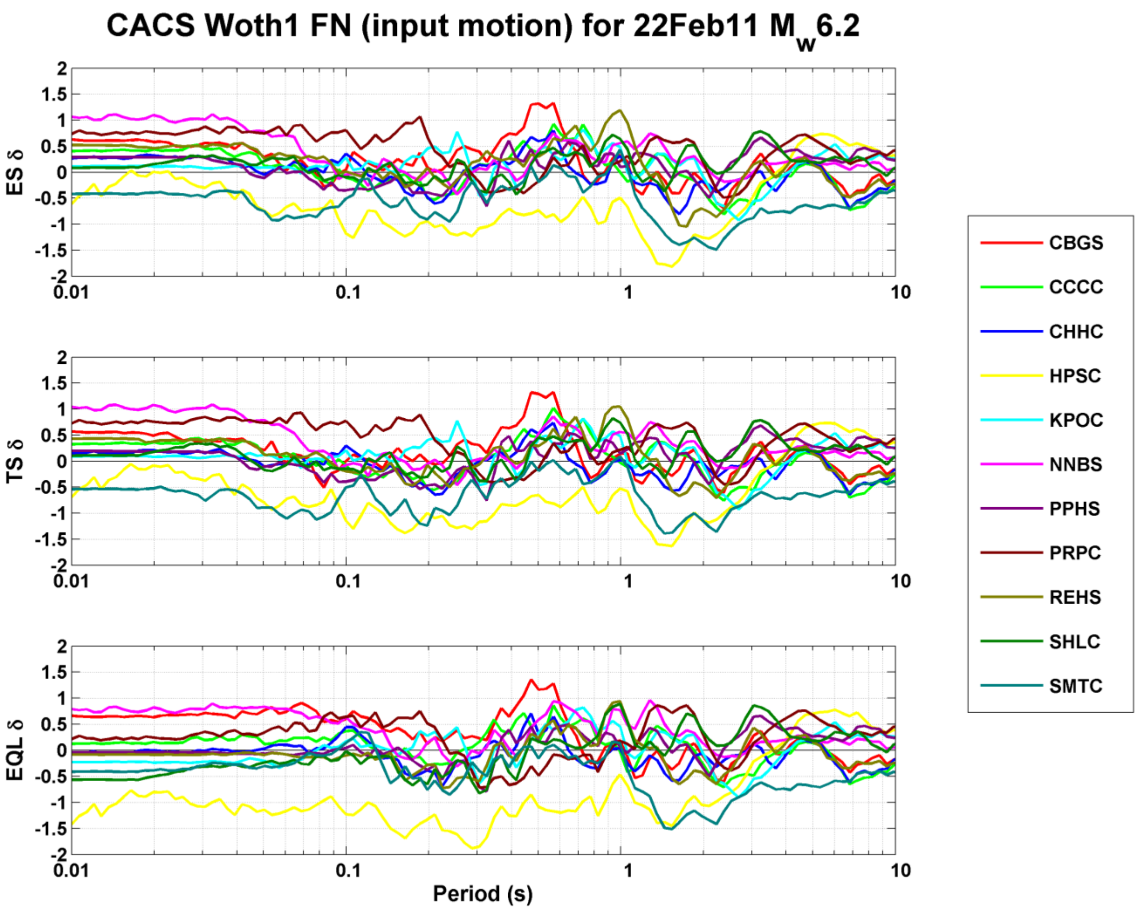

Figure 9: Residuals for all analyses using the CACS Woth1 FN input motion for the 22 Feb 2011 Christchurch event; EQL: equivalent linear, TS: nonlinear total stress, and ES: nonlinear effective stress analyses are shown

Figure 10 shows the residual plots for the Christchurch event for the *CACS Woth1 FP* input motion (i.e., same profile used for deconvolution to obtain the input motion for the results shown in Figure 10 but with the fault parallel surface motion deconvolved instead of the fault normal).

Similar trends as observed with the *CACS Woth1 FN* are observed with *CACS Woth1 FP* regarding the consistent underestimation of spectral accelerations in the short period range (i.e., less than 0.1 s). The variation of residuals from site-to-site in the higher period range (i.e., greater than 1 s) is more pronounced than those shown in Figure 9. The similarity between the total stress and effective stress nonlinear analyses results can be seen in Figure 10, as with the results for the fault normal direction shown in Figure 9.

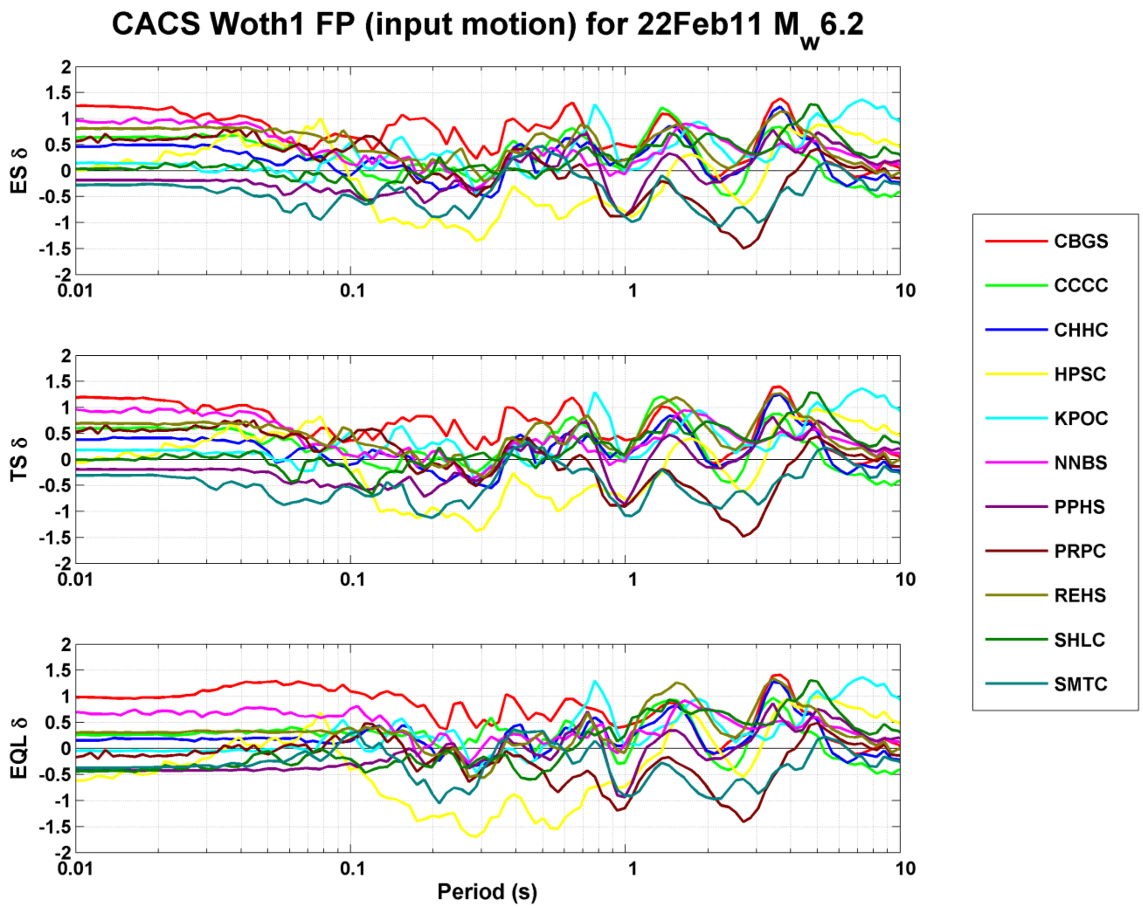

Figure 10: Residuals for all analyses using the CACS Woth1 FP input motion for the 22 Feb 2011 Christchurch event; EQL: equivalent linear, TS: nonlinear total stress, and ES: nonlinear effective stress analyses are shown

5.2.2 Spectral Acceleration Comparisons for 4 September 2010 Darfield Event

Figure 11 provides the results for analyses completed in the fault normal direction of shaking using the *CACS Woth1 FN* input motion. It can be seen that for these analyses, there is a consistent positive "bump" in the residuals across all stations for periods between 1 s and approximately 5 s. This consistent underestimation of the spectral accelerations within this period range could be due to the inability of the input motion to replicate the forward directivity effects experienced at the strong motion stations throughout Christchurch during the Darfield event. For the analyses where *RHSC Woth1 FN* was used as an input motion (see Appendix F), there is a consistent underestimation of the spectral acceleration values for periods greater than 1

s, but on a much less pronounced scale compared to the results shown in Figure 11. The residuals for all analyses using the *RHSC Woth1 FN* input motion are centered on a value of δ≈0.2-0.3. The consistently lower residuals in the higher period range for the *RHSC Woth1 FN* input motion suggest that it was better able to capture the long-period part of the seismic response at the strong motion stations for the Darfield event (i.e., relative to the *CACS Woth1 FN* input motion).

The response spectra calculated from the seismic site response analyses for the strong motion station site KPOC severely underestimates the recorded response spectra (Figure 11). This underestimation could be due to both the large forward directivity and basin-generated surface wave effects experienced at KPOC during the Darfield event, especially for the higher period response (see Bradley 2012), and the inability for analyses with both input motions in the fault normal direction to capture these effects. Furthermore, for both sets of analyses performed with the fault normal inputs discussed above, the spectral accelerations for periods greater than 4 s are overestimated for the analyses completed at HPSC (increasingly negative residual past 4 s). This station is an exception to the trends shown at all other stations for this period range; this exception could be due to the proximity of this station to a free-face (the Avon River runs within 50m of HPSC), which could have affected the site's response during seismic shaking.

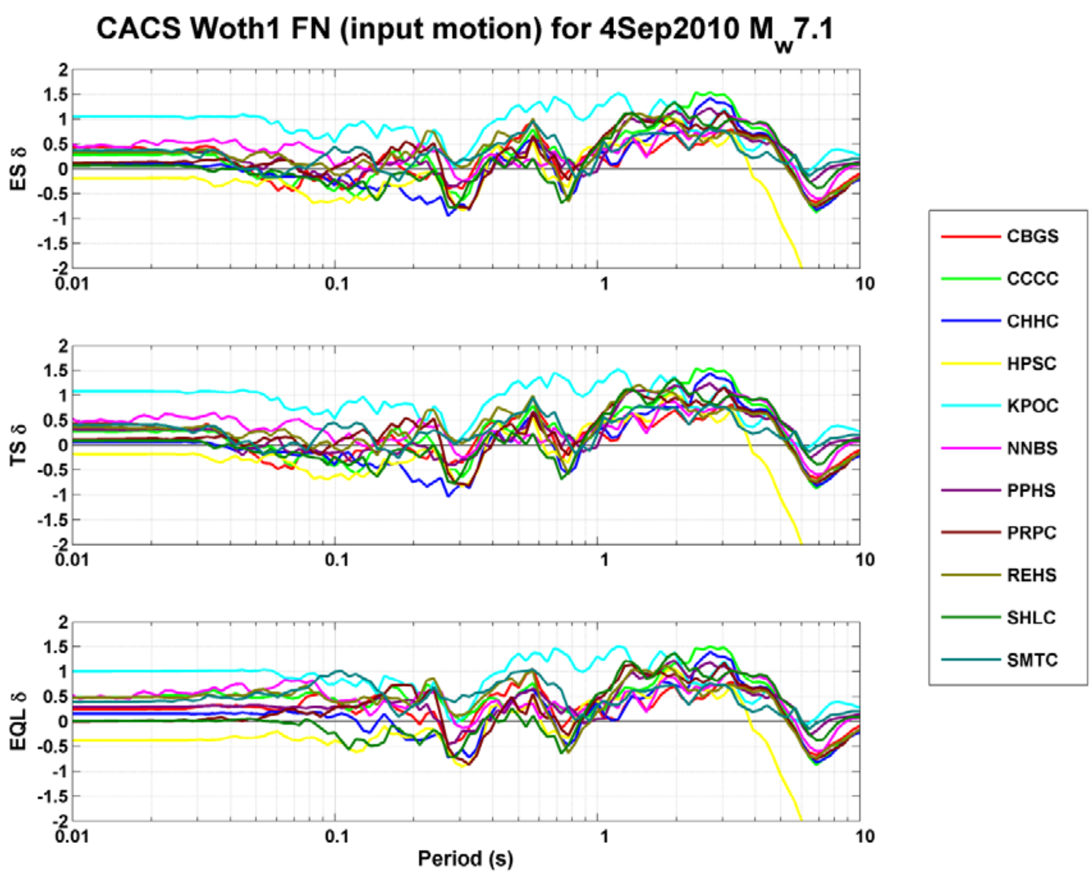

Figure 11: Residuals for all analyses using the CACS Woth1 FN input motion for the 4 Sep 2010 Darfield event; EQL: equivalent linear, TS: nonlinear total stress, and ES: nonlinear effective stress analyses are shown

5.2.3 Spectral Acceleration Comparisons for Other Events

The results presented for the two largest events (Darfield and Christchurch events) show clear trends from station-to-station, an example of which is the consistent underestimation of the spectral acceleration values in the lower period range (less than 0.1 s) for the Christchurch event. In general, the results from analyses for other events capture the short period response better. Furthermore, the overall fit is better across the entire range of periods considered for the four, lower magnitude events. Figure 12 provides the results for the 13Jun11 (M_w6.0) event using the *RHSC Woth1 FP* input motion, which illustrates this observation regarding fit for the lower magnitude events.

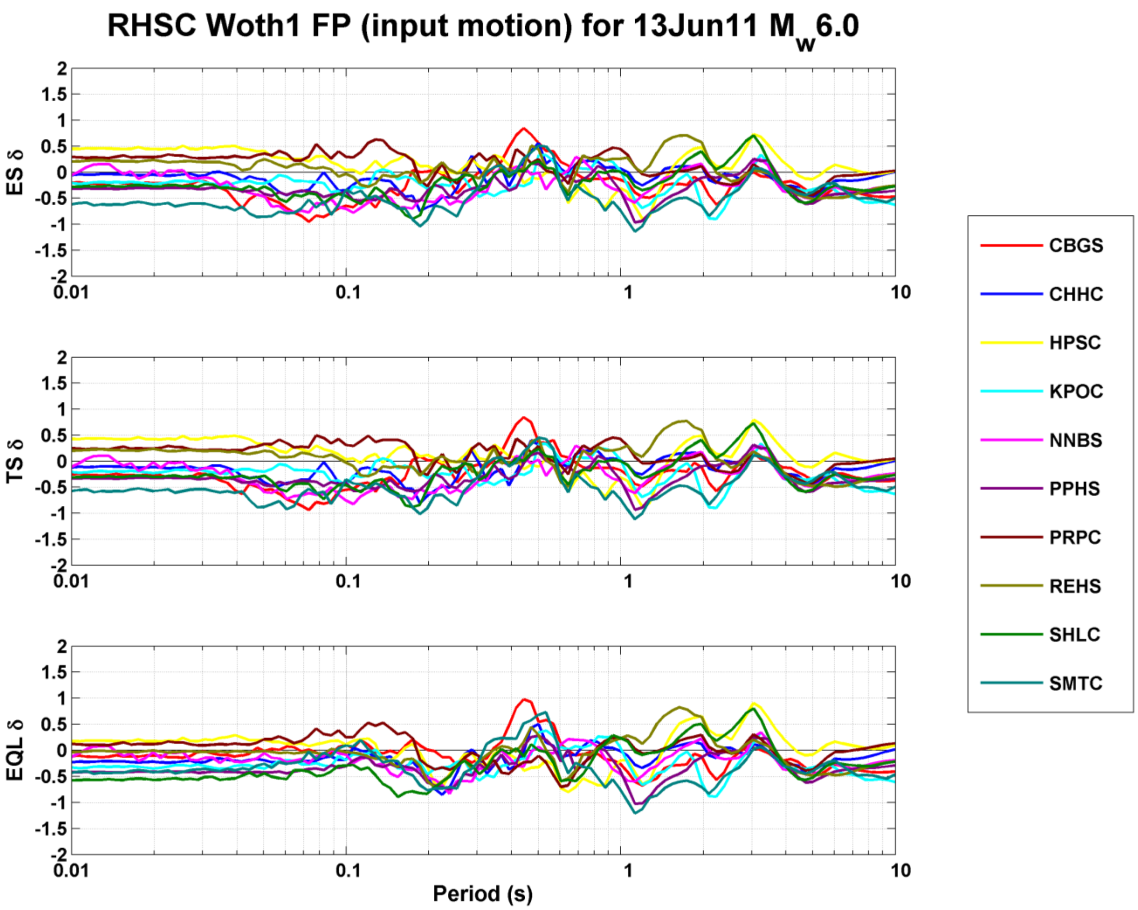

Figure 12: Residuals for all analyses using the RHSC Woth1 FP input motion for the 13 June 2011 event; EQL: equivalent linear, TS: nonlinear total stress, and ES: nonlinear effective stress analyses are shown

5.3 Observations for Selected Analyses

5.3.1 CHHC for 23 December 2011 (M_w5.8) Event—Fault Parallel

Figure 13 shows the acceleration response spectra for the analyses performed in the fault parallel direction for CHHC for the 23 Dec 2011 (M_w5.8) event compared to the acceleration response spectrum calculated from the recorded surface motion. The recorded motion's spectral shape is captured well by the analyses. Furthermore, the acceleration response spectrum from the

recorded surface motion tends to lie between the estimated spectral accelerations for the two sets of analyses that consider two different input motions (i.e., the analyses capture the recorded spectrum in an average sense). Evidence for both of these observations is provided by the average residuals (δ_{ave} is the arithmetic mean of residuals across all periods considered) being relatively close to zero for all analyses.

The response spectra shown in Figure 13 are similar (essentially identical for the effective stress compared to the total stress analysis for the same input motion), which is evidenced by the similar δ_{ave} values (approximately the same for ES and TS analyses for a given input motion) for each set of analyses (ES, TS, and EQL) for a given input motion. The similarity between nonlinear and equivalent linear analyses is expected since relatively low shear strains were estimated by the analyses for the illustrated case, as well as a lack of significant excess pore water pressure generated during shaking. These observations are illustrated in Figure 14 as a plot of maximum shear strain and r_u with depth for both input motions. The relatively low maximum calculated shear strains and excess pore water pressures result in a lower degree of nonlinear response in the *DeepSoil* simulations.

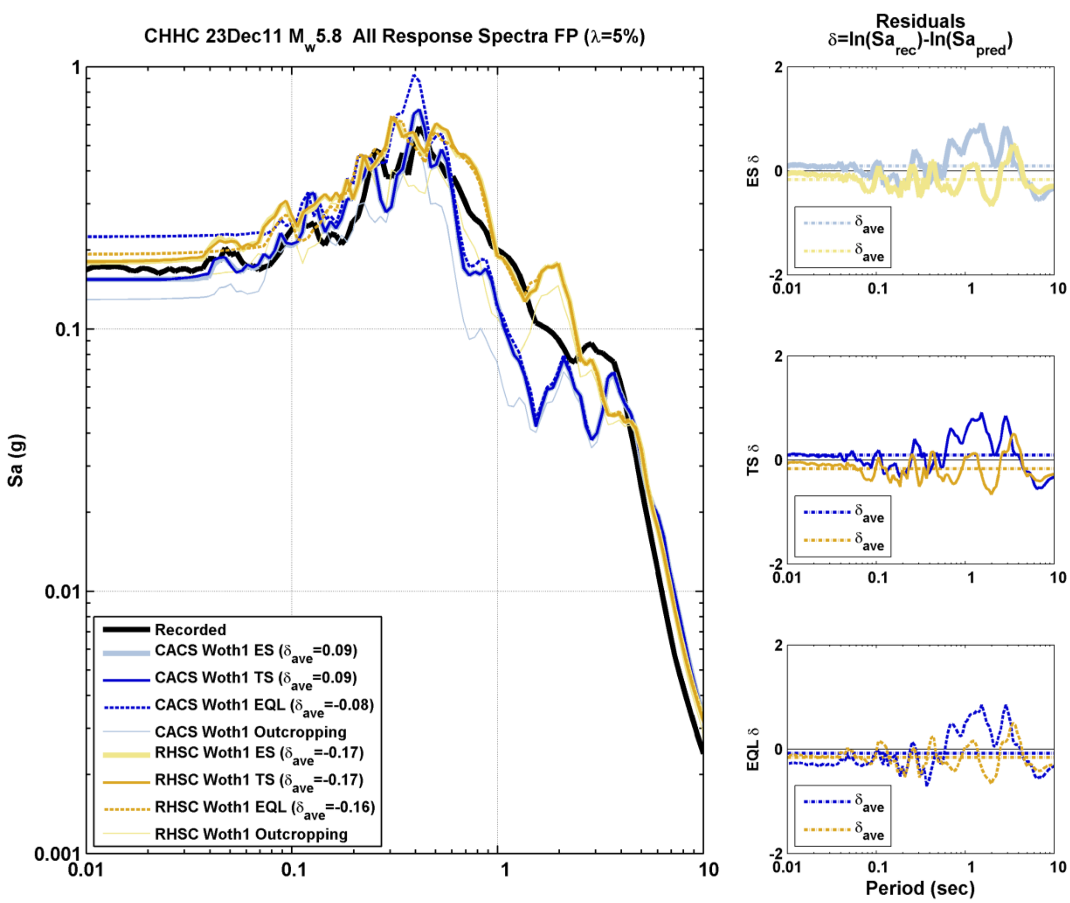

Figure 13: Acceleration response spectra comparisons and residuals for CHHC (FP) for the 23 Dec 2011 (Mw5.8) event; EQL: equivalent linear, TS: nonlinear total stress, and ES: nonlinear effective stress analyses are shown

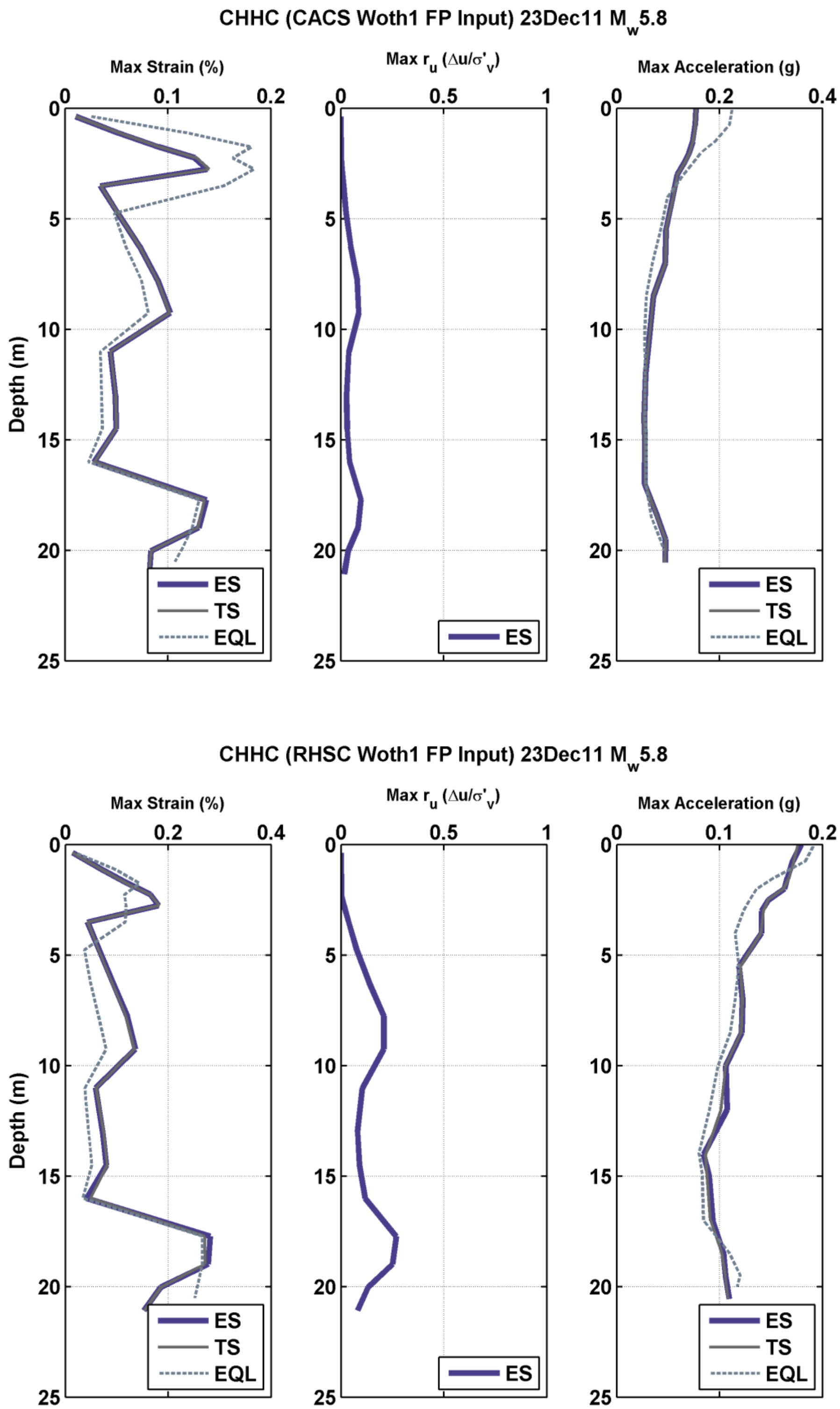

Figure 14: Maximum strain, r_u, and acceleration values with depth for all analyses completed at CHHC for 23Dec11 (Mw5.8) event using CACS Woth1 FP and RHSC Woth1 FP inputs

5.3.2 CBGS for 22 February 2011 Christchurch Event—Fault Normal

In contrast to the results shown previously for the 23Dec11 (M_w5.8) event, the results for the Christchurch event often showed different patterns in the comparisons of the response spectra calculated from analyses compared to those calculated from recorded surface motions. Figure 15 shows the results for the strong motion station CBGS for the Christchurch event. While the response is captured relatively well for periods of 1 s to 10 s, there is a general underestimation of the recorded acceleration response spectrum, especially in the lower period range (consistent with the overall trends outlined in section 5.2.1). Furthermore, the overall spectral shape is not captured. One of the most interesting observations is the similarity between calculated response spectra for both nonlinear total stress and effective stress analyses for a given input motion. This is evidenced by the similar δ_{ave} values calculated for the total and effective stress analyses for a given input motion. Figure 16 shows the maximum shear strain, r_u, and accelerations with depth for the same analyses presented in Figure 15. Though the acceleration response spectra for the surface motions are similar for these analyses for a given input motion, the maximum shear strain with depth is noticeably different, especially for depths greater than about 9 m below ground surface for analyses using the *CACS Woth1 FN* input motion. At a depth of about 9.8 m below the ground surface (bgs), the r_u calculated for the effective stress analyses using both input motions is in the range of 0.75 to about 0.9, which should cause a reduction in both the shear strength and stiffness of the soil modeled at this depth. Figure 17 provides an illustration of the fact that the effective stress analysis does show a markedly different shear stress versus shear strain plot compared to the total stress analysis for the layer of soil that experiences the greatest excess pore water pressure generation (Layer 10 centered at 9.75 m bgs). Figure 18 shows the excess pore water pressure ratio (r_u) time history for the same layer.

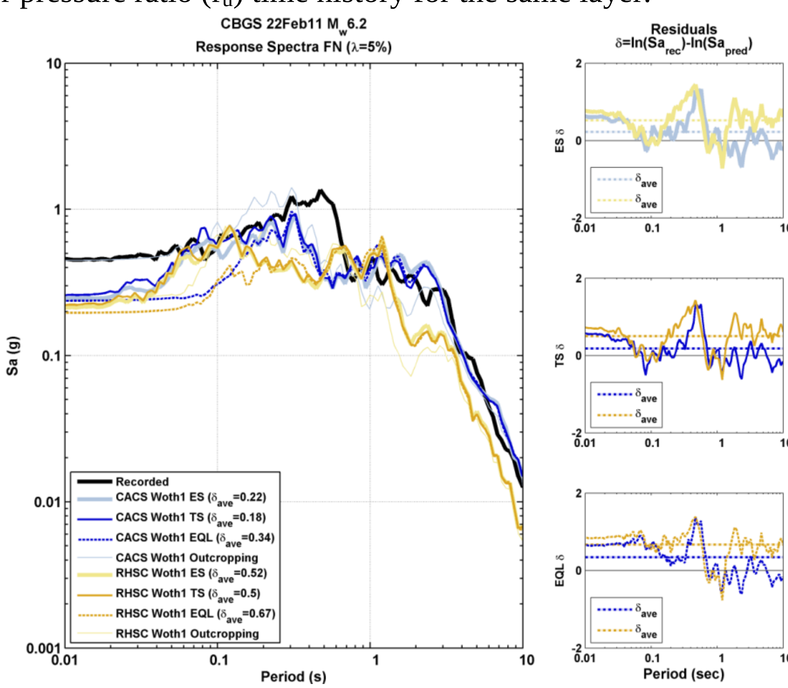

Figure 15: Acceleration response spectra comparisons and residuals for CBGS (FN) for the Christchurch event; EQL: equivalent linear, TS: nonlinear total stress, and ES: nonlinear effective stress analyses are shown

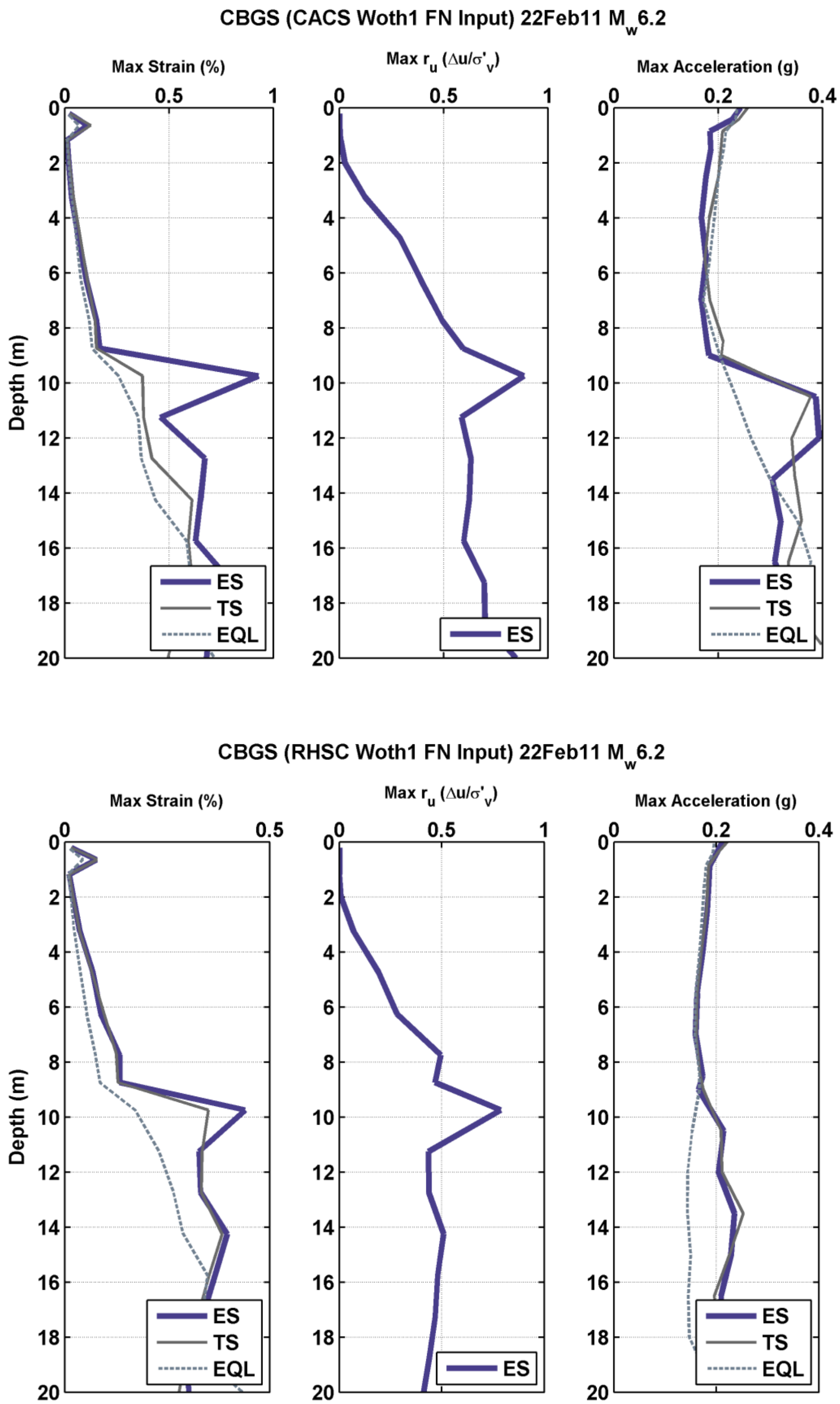

Figure 16: Maximum strain, ru, and acceleration values with depth for all analyses completed at CBGS for the Christchurch event using CACS Woth1 FN and RHSC Woth1 FN inputs

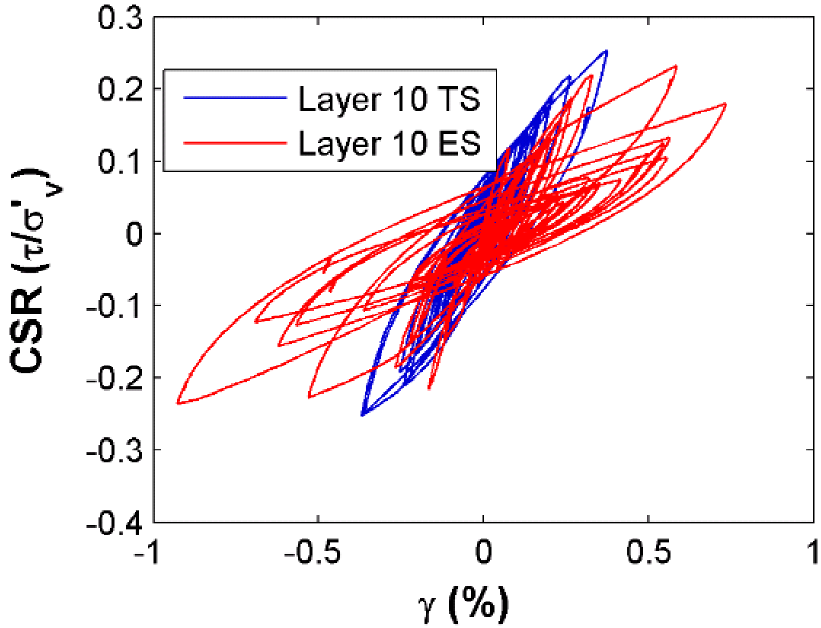

Figure 17: Cyclic stress ratio vs. shear strain for Layer 10 (centered at 9.75m bgs) for CBGS for the Christchurch event using the CACS Woth1 FN input motion for total stress (TS) and effective stress (ES) nonlinear seismic site response analyses

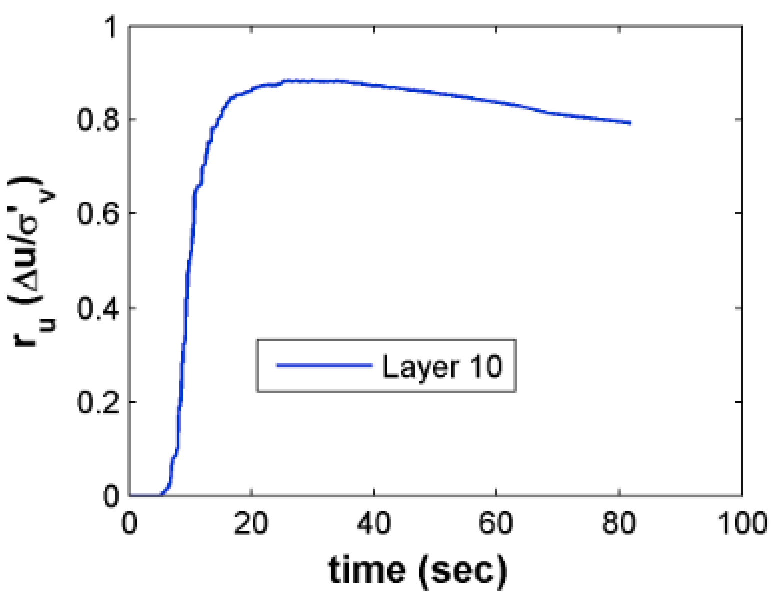

Figure 18: Excess pore water pressure ratio (r_u) vs. time for Layer 10 (centered at 9.75 m bgs) for CBGS for the Christchurch event using the CACS Woth1 FN input motion for the effective stress nonlinear seismic site response analysis

5.3.3 CCCC for 22 February 2011 Christchurch Event—Fault Parallel

Based on the liquefaction triggering results provided in Appendix E, much of the soil in the upper 10 m of the subsurface profile at CCCC has a FS_{liq} less than one based on the input parameters used for the Christchurch event (i.e., PGA, M_w, etc.—see Tables 1 and 2). One would then expect to see maximum values of r_u close to 1.0. Figure 20 shows that the effective stress analyses for CCCC in the fault parallel direction do indeed calculate a r_u value in excess of 0.5 for much of the soil down to a depth of 10 m bgs, and even close to 1.0 for the layer centered just above 5.0 m bgs. As shown in Figure 19, the response spectra from analyses for this case compare relatively well to the recorded acceleration response spectrum. Though the spectral acceleration values are underestimated for the short period range (T≤0.1 s), the intermediate and long period spectral values match the recorded acceleration response spectrum well, as indicated by the residual plots shown in Figure 19. Furthermore, the spectral shape of the acceleration response spectrum of the recorded surface motion and those from the completed analyses are similar.

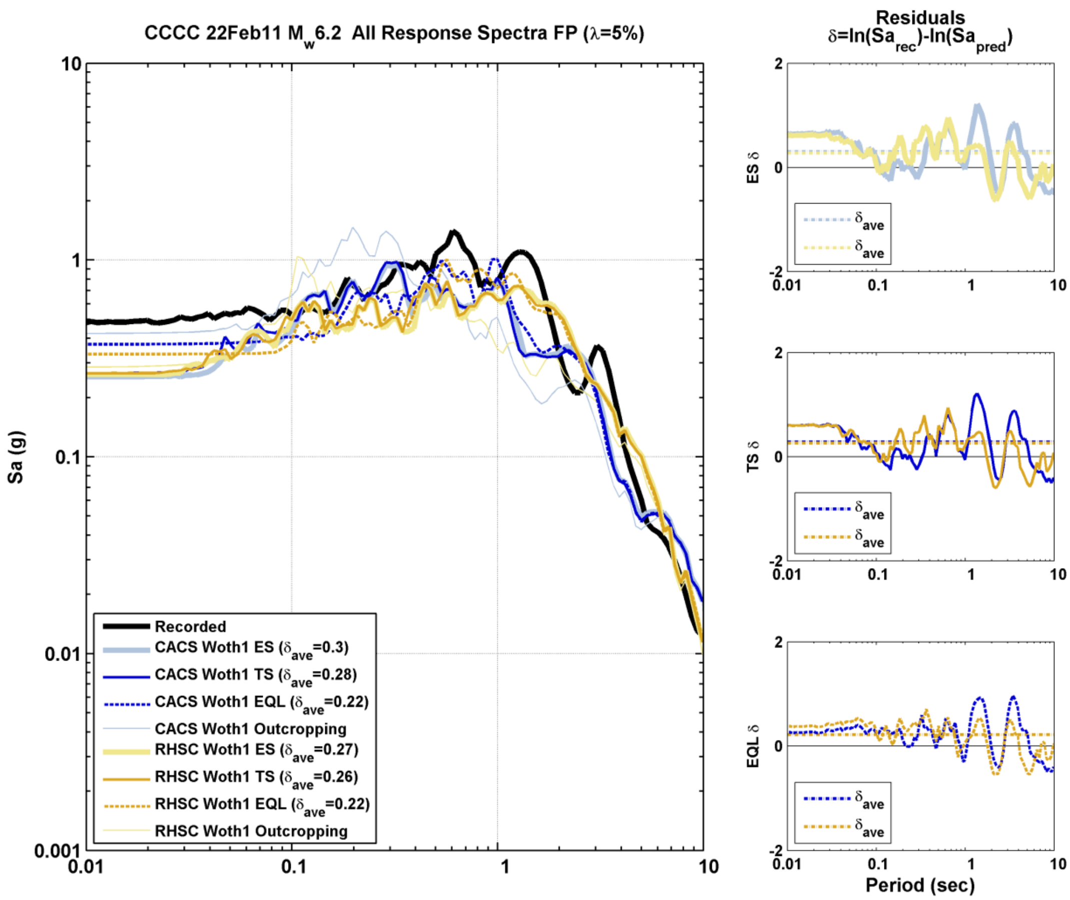

Figure 19: Acceleration response spectra comparisons and residuals for CCCC (FP) for the Christchurch event; EQL: equivalent linear, TS: nonlinear total stress, and ES: nonlinear effective stress analyses are shown

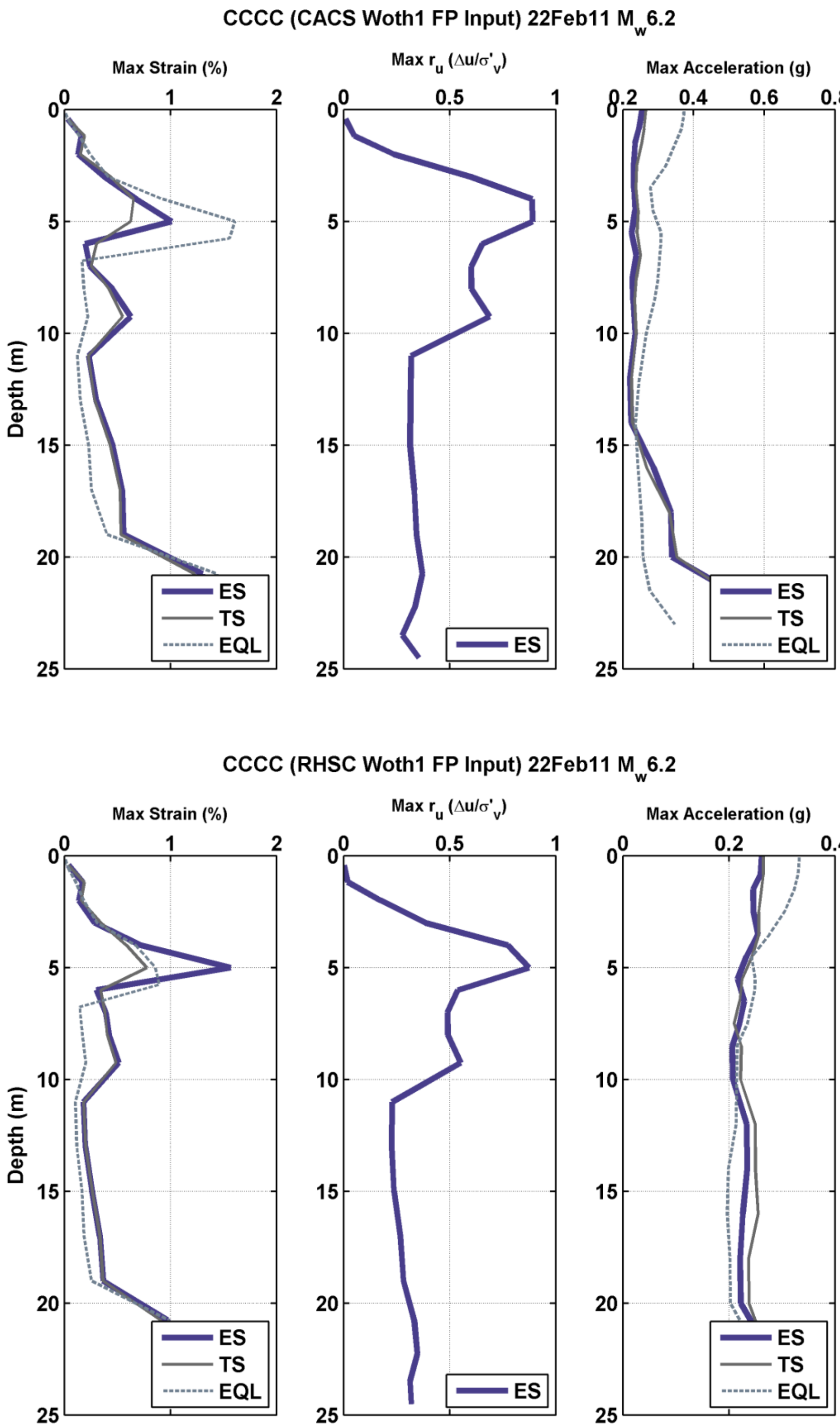

Figure 20: Maximum strain, ru, and acceleration values with depth for all analyses completed at CCCC for Christchurch event using CACS Woth1 FP and RHSC Woth1 FP inputs

5.3.4 CHHC for 4 September 2010 Darfield Event—Fault Normal

As discussed in Section 5.2.2, the Darfield event had significant forward directivity effects on many of the recorded motions in Christchurch. These effects are illustrated for CHHC in Figure 21 as one is able to see a distinct peak in the long period portion of the acceleration response spectrum for the recorded event in the fault normal direction. The presence of this long period amplification is not completely captured by the analyses, especially for those corresponding to the use of the *CACS Woth1 FN* input motion. However, it can be seen that the average residuals for the nonlinear analyses are below 0.1 for both input motions in the fault normal direction. This low average residual is due to the compensation of the underestimation of the spectral accelerations corresponding to higher periods to the overestimation of spectral acceleration values in the 0.03-0.3 s range. Furthermore, even though the results corresponding to the *CACS Woth1 FN* input motion underestimate the long period spectral acceleration values more than those that correspond to the *RHSC Woth1 FN* input motion, the average residuals for the *CACS Woth1 FN* results have a lower average residual.

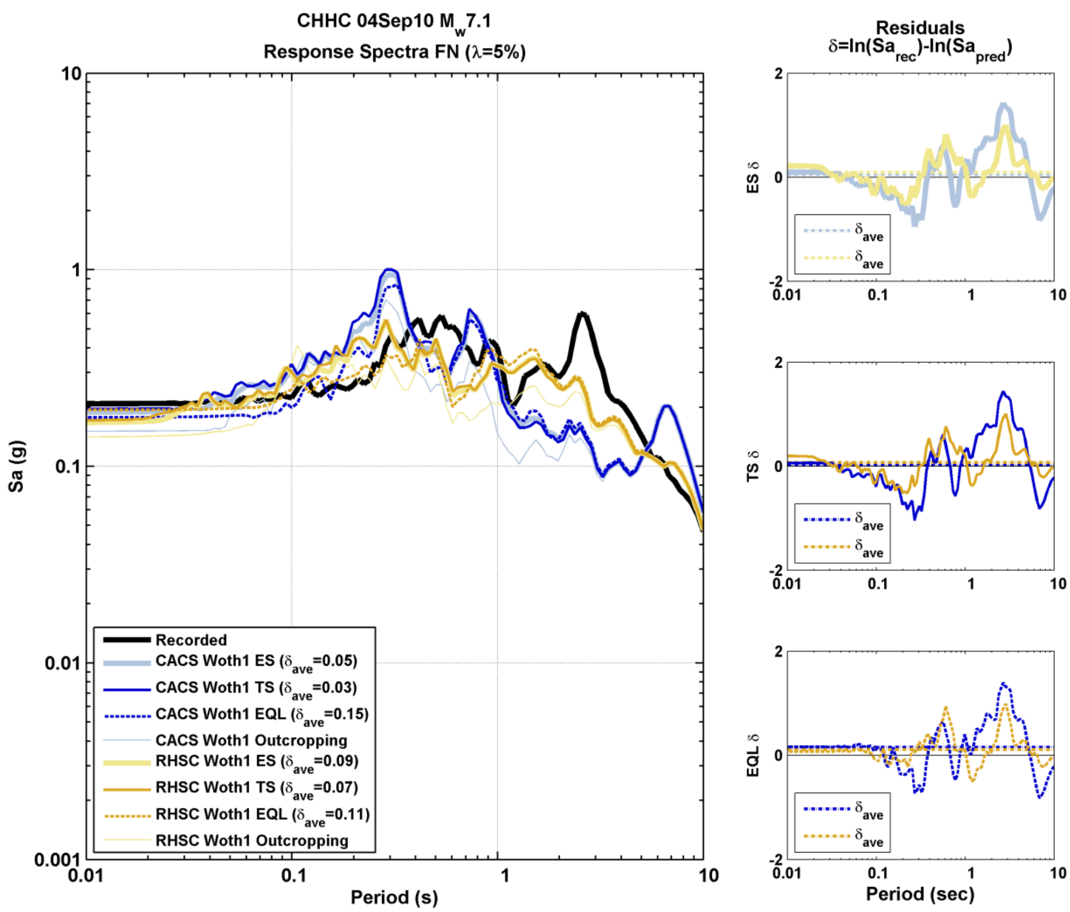

Figure 21: Acceleration response spectra comparisons and residuals for CHHC (FN) for the Darfield event; EQL: equivalent linear, TS: nonlinear total stress, and ES: nonlinear effective stress analyses are shown

5.3.5 HPSC Effective Stress Analysis for Combined 23Dec11 Events

Approximately eighty minutes elapsed between the beginning of the M_w5.8 event and the beginning of the M_w5.9 event on 23 December 2011. The closeness of these two events with regards to timing raises the question as to how much the soil response during the second event was affected by the earlier event. Analyses were performed for the earlier M_w5.8 event that occurred on 23 Dec 2011 to identify a site that was significantly affected by the first event to conduct an effective stress analysis that used the combined input motions from both 23 Dec 2011 events. The analyses of the strong motion station site HPSC indicated a significant generation of excess pore water pressure during the M_w5.8 event in the upper 3-8 m of the soil profile. This site was chosen to conduct a nonlinear effective stress analysis that utilized the combined input motion for the two back-to-back 23 Dec 2011 events.

Figure 22 shows the time history for the excess pore water pressure ratio (r_u) for two layers of the HPSC profile (Layers 4 and 7) that was a result of considering the combined *CACS Woth1 FN* input motion. It can be seen that based on the input parameters used for the effective stress analysis, the excess pore water pressures generated during the first event do not fully dissipate before the start of the second event. Though these excess pore water pressures do affect the calculated stress-strain behavior of the soil, it is shown in Figure 23that the resulting acceleration response spectrum of the calculated surface motion is minimally affected.

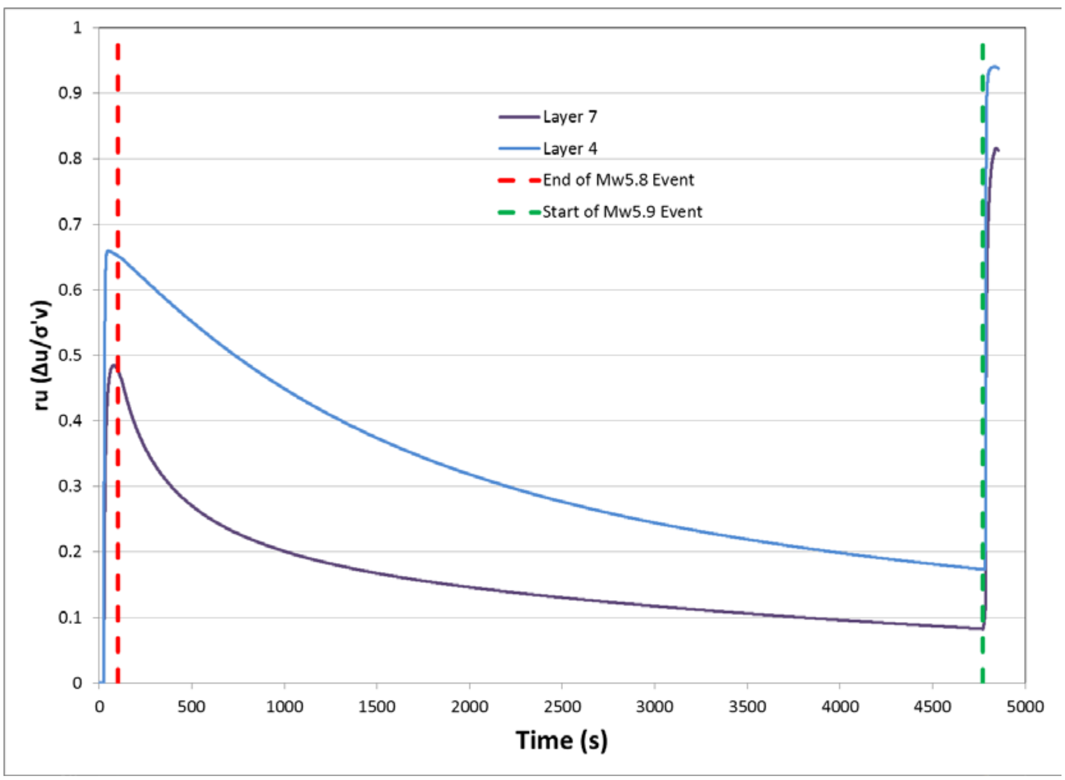

Figure 22: Excess pore water pressure ratio (r_u) with time for the combined *CACS Woth1 FN* input motion for the 23 Dec 2011 event at HPSC

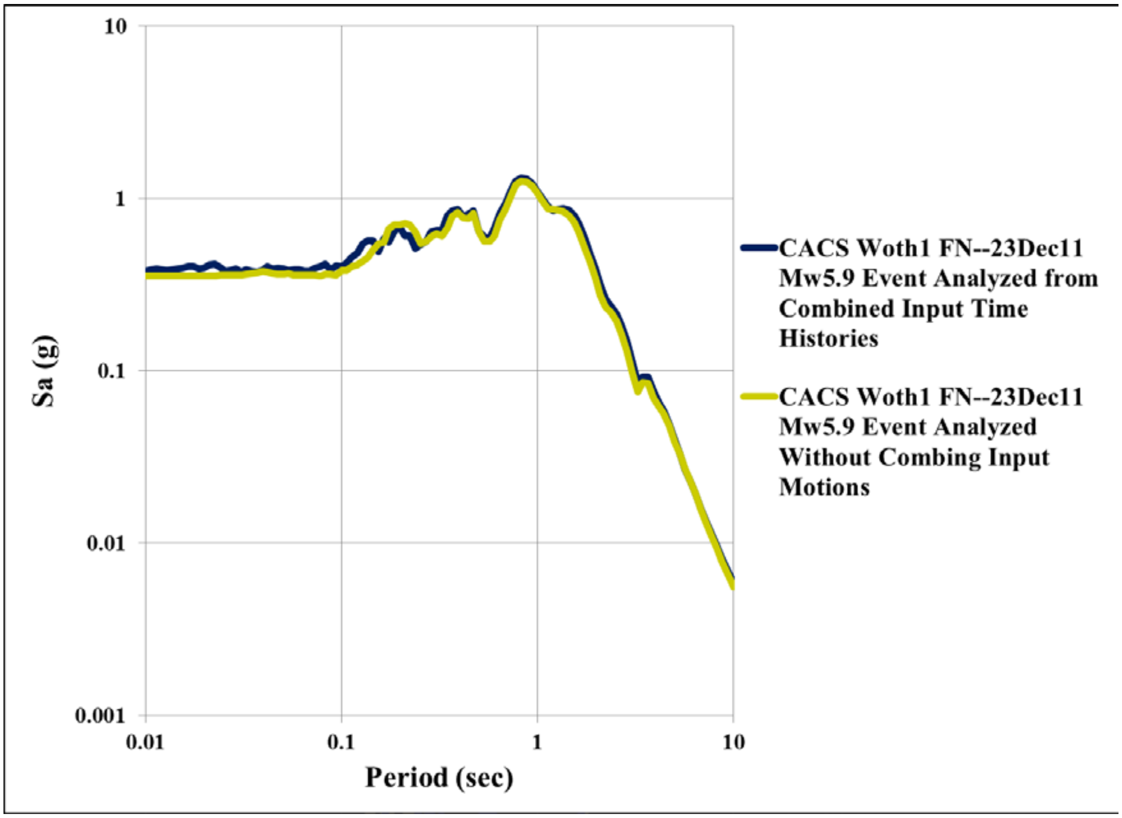

Figure 23: Comparison of acceleration response spectra (λ=5%) for effective stress analyses using the CACS Woth1 FN input motion for the 23 Dec 2011 (Mw5.9) event at HPSC considering a combined input motion for both 23Dec11 events and an uncombined input motion

6. CONCLUSIONS

The 2010-2012 Canterbury earthquake sequence provides an exceptional opportunity to investigate how the same ground responded to several significant earthquakes that delivered different intensities and durations of strong shaking. The Canterbury earthquake sequence was recorded at several strong motion stations from the GeoNet strong motion network operated in the Christchurch area. Some of these strong motion station sites experienced soil liquefaction multiple times, while other sites never experienced soil liquefaction. Thus, the recordings at GeoNet strong motion stations provide a unique and robust dataset for evaluating the capabilities of fully nonlinear one-dimensional effective stress seismic site response analyses.

This study considered five (5) of the seven (7) events with magnitude greater than or equal to M_w5.5 from the Canterbury earthquake sequence. In addition to these events with $M_w \geq 5.5$, the earthquake that occurred on 26 December 2010 with M_w4.7 was also included. Thirteen (13) strong motion stations were selected based on an examination of recorded events and the availability of close site investigation data. CPT data and surface wave testing results along with boreholes completed near strong motion station sites were the primary sources of information for characterizing the soil profiles at the strong motion station sites. CPT data were also used to help define the shear wave velocity profiles by means of the McGann et al. (2014) Christchurch specific CPT-Vs correlation.

The deep basin structure that underlies the studied sites made the selection of "rock" input motions challenging due to the absence of representative recorded outcropping rock motions. For this reason, deconvolution of select sites was carried out to provide input motions for subsequent seismic site response analyses. The Riccation gravel layer, which is present throughout the subsurface of Christchurch, was chosen as the half-space for deconvolution (and subsequent convolution analyses) due to the relatively high impedance contrast between this gravel and the overlying surficial material. The stations selected for performing the deconvolution were CACS and RHSC. These stations were chosen for deconvolution due to the relatively stiff subsurface soil and consequently minimal nonlinear response of the soil during strong shaking.

The software *Deepsoil* (*V5.1*) was used to evaluate the capabilities of fully nonlinear one-dimensional effective stress seismic site response analyses. For completeness and comparative purposes total stress nonlinear analyses and equivalent linear analyses were also performed at the remaining eleven (11) strong motion station sites for each event studied. Comparison of the recorded surface motions and the calculated surface motions from conducted seismic site response analyses via pseudo-acceleration response spectra are made for all of the events and sites of interest. Furthermore, an examination of calculated shear strains and excess pore water pressures for each analysis is made. A select number of results are discussed within the report; a more comprehensive synopsis of the results is provided for all analyses within Appendix D.

Comparisons of pseudo-acceleration response spectra of surface motions calculated from analyses to spectra of recorded motions for the 13Jun11 (M_w6.0), 23Dec11 (M_w5.8), and 23Dec11 (M_w5.9) events show a reasonable "fit" across a broad range of periods. The same observation is true for the 26Dec10 (M_w4.7) event except for the results of analyses at CBGS. The analyses performed at CBGS for the 26Dec10 event underestimated severely the recorded response spectra. These stark differences between recorded spectra and those calculated from analyses for CBGS are most likely due to the inability of the input motion to capture the soil response during shaking at this particular station (i.e., CBGS experienced the highest peak ground acceleration of the sites studied for the 26Dec10 (M_w4.7) event—see Table 2).

For the four events mentioned, the peak ground acceleration is often slightly overestimated for each analysis, especially for equivalent linear analyses. The spectral shape of the calculated surface motions are often well represented by the results of these analyses. A brief examination of the potential for the 23 Dec 2011 (M_w5.8) event to affect the calculated results of the 23 Dec 2011 (M_w5.9) event was presented. As these two events were separated by approximately eighty minutes, some of the excess pore water pressures generated during the analysis of the earlier event were not fully dissipated at the start of the second event. These residual pore water pressures had a minimal effect on the calculated surface acceleration response spectrum for the effective stress analysis presented.

Acceleration response spectra comparisons for the Darfield event show that the complexity of near-source, forward directivity and most likely basin-generated surface wave effects were often difficult to capture with the input motions used. The high intensity of shaking witnessed for the 22 Feb 11 Christchurch event at many of the strong motion station sites in the short period range was not captured fully by the seismic site response analyses. The general underestimation of spectral acceleration values for short periods could be a product of the input motions or the representation of the dynamic soil properties at the sites. For the effective stress analyses, the inability of the pore water pressure generation model to fully represent the dilation of soils that experience large shear strains due to soil liquefaction may be a reason for this underestimation.

Overall, the effective stress analyses performed well with regards to calculating the generation of pore water pressure. Liquefaction triggering analyses were conducted for each site of interest and for each event of interest using the Boulanger and Idriss (2014) CPT based liquefaction triggering procedure. Soil layers with FS_{liq} less than one typically displayed maximum r_u values in excess of 0.5 to 0.6 during the effective stress analyses. However, the generation of excess pore water pressures for the analyses considered did not lead to a drastic difference between the calculated acceleration response spectra at the surface for total stress and effective stress fully nonlinear analyses.

Recommendations resulting from this study regarding future work to advance the profession include: 1) the installation of strong motion stations on outcropping "rock" sites on the north side of the Port Hills to obtain "rock" acceleration time histories during future strong shaking events; 2) the installation of at least one down-hole array in Christchurch to measure accelerations (and pore water pressures) at various depths within a soil profile to assist in the calibration of input parameters for future fully nonlinear seismic site response analyses; 3) the generation of synthetic "rock" or "Riccarton gravel" motions for Christchurch to be used as representative input motions for seismic site response analyses; and 4) the implementation of a pore water pressure model that more accurately models the dilative response of some liquefiable soils during strong shaking.

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APPENDIX A

Surface Recorded Ground Motions at Strong Ground Motion Stations.

Available at:

https://www.dropbox.com/sh/086h0k8u5ilfysv/AABJbkjiJ8xjD9AqlwnGuj7Oa?n=314408476

APPENDIX B

B.1 CPT Data at Strong Ground Motion Stations.

B.2 Shear Wave Velocity Profiles for Seismic Site Response Analyses.

CPT: CBGS_CPT1(Wotherspoon,2013)

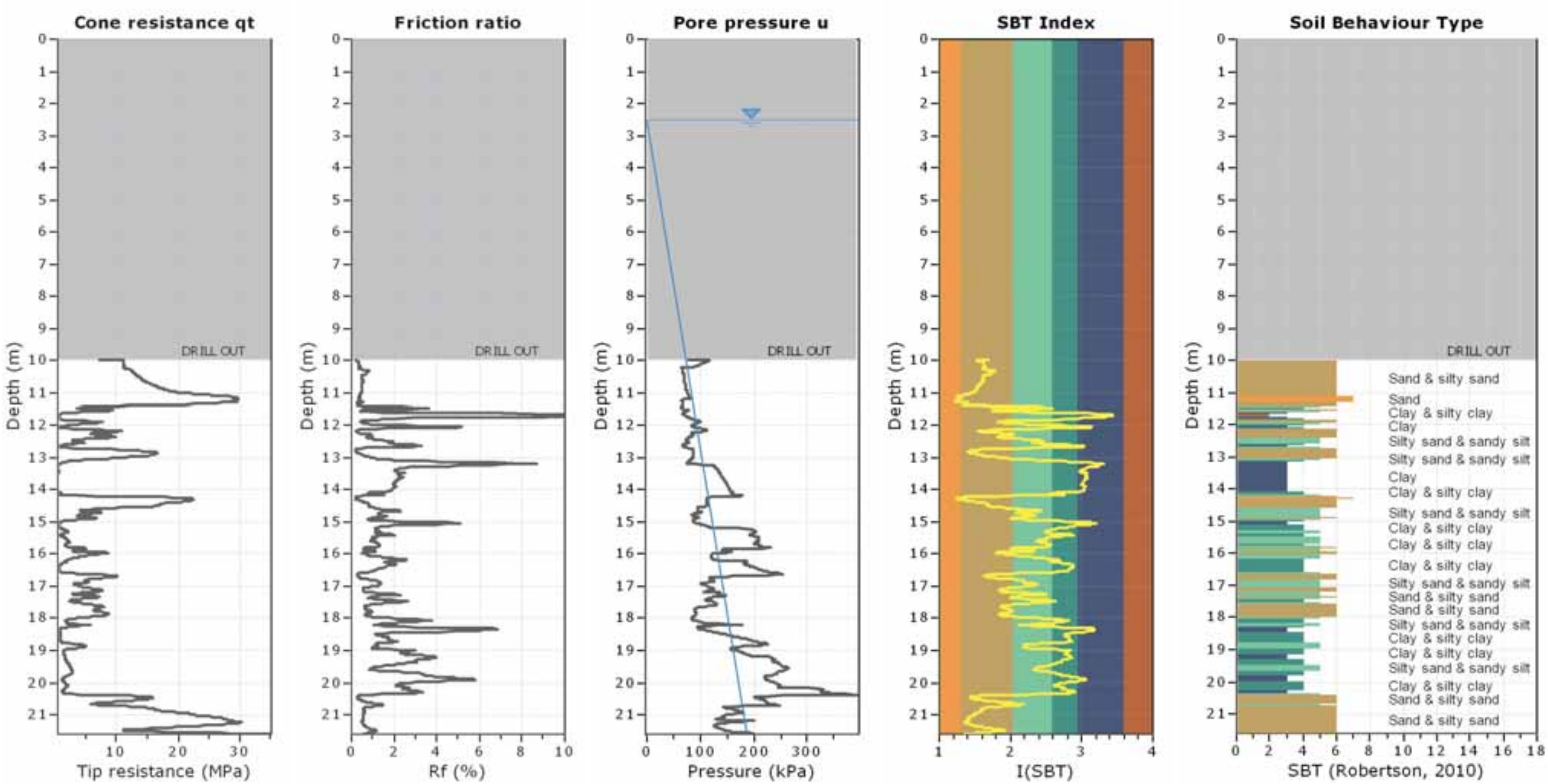
Total depth: 21.60 m

Coords: S 43.5293, E 172.6198

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



SBT legend

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

CPT: CBGS_CPT1(Wotherspoon,2013)

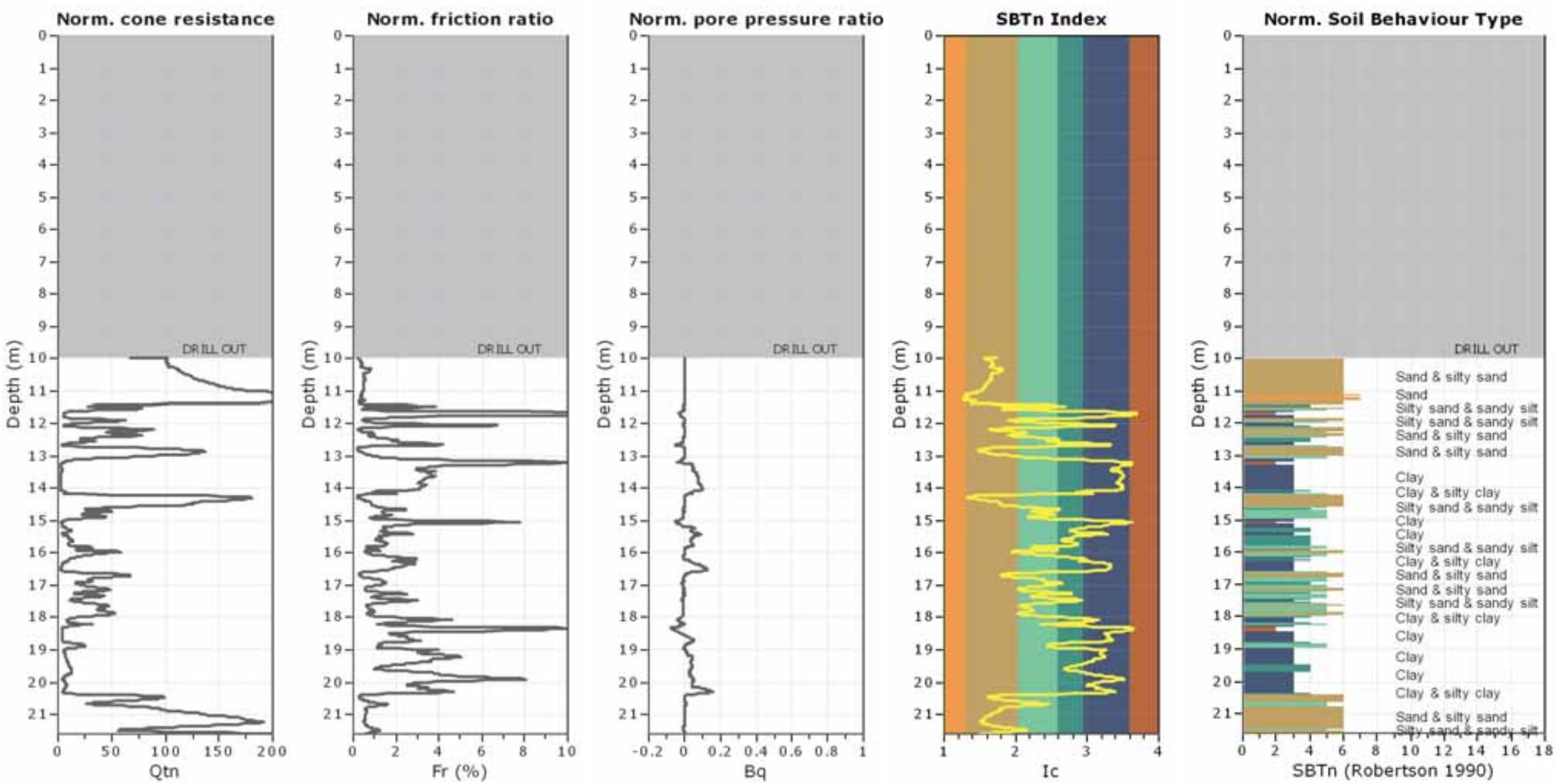
Total depth: 21.60 m, Date: 07/07/2014

Coords: S 43.5293, E 172.6198

Cone Operator: Uknown

Project: Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence

Location: Christchurch, New Zealand



SBTn legend

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

CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 21/08/2014, 01:30:58 a.m.

Project file: C:\Users\User\Desktop\summer\USGS Site Response Project\Soil Profiles and Info\CBGS\Site Investigation Data\CBGS CPT\test_2.cpt

CPT: CCCC_CPT484(CGD)

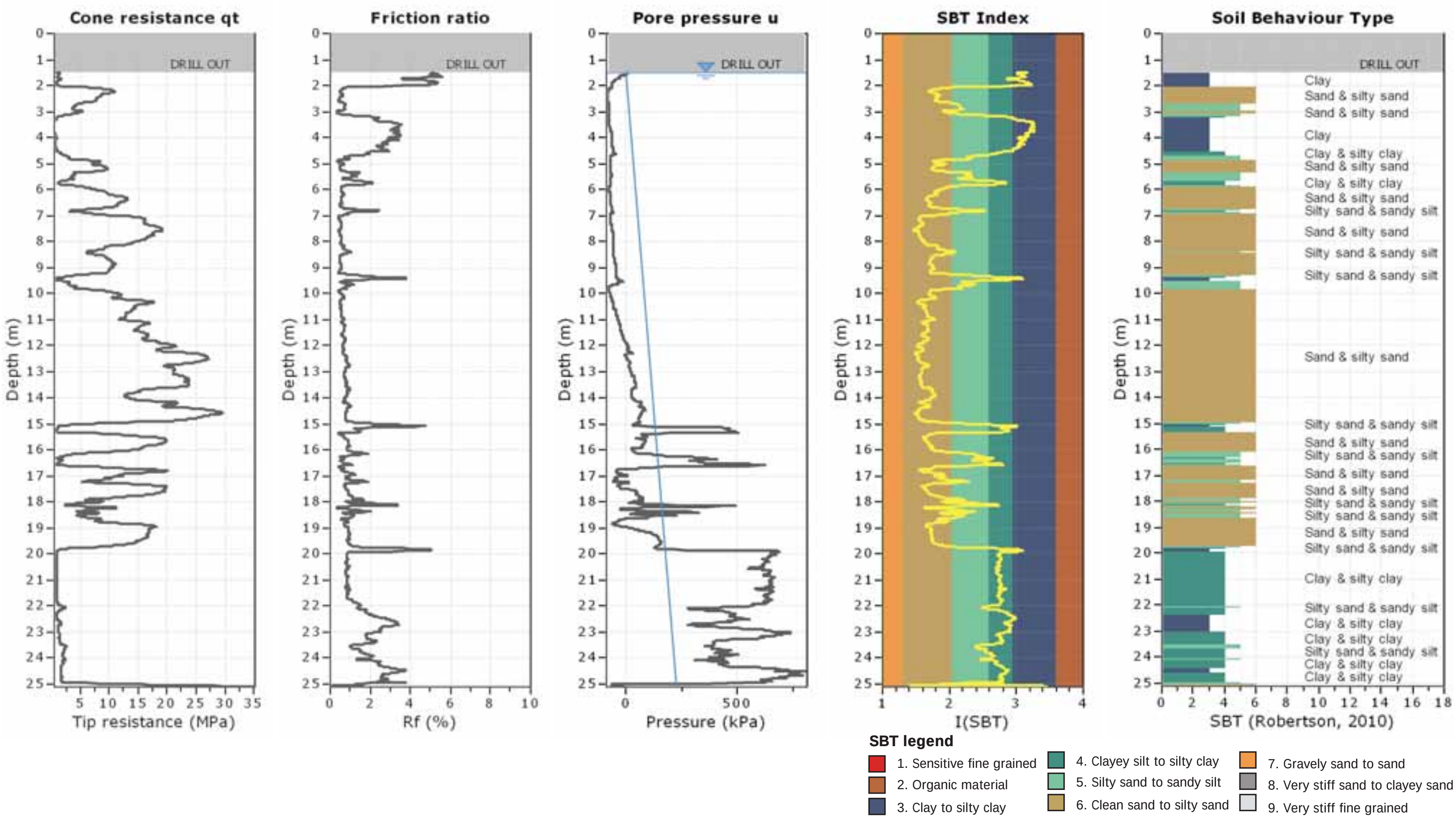
Total depth: 25.11 m

Project: **Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence**

Coords: S 43.5379, E 172.6479

Location: **Christchurch, New Zealand**

Cone Operator: Uknown



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Project file: C:\Users\User\Desktop\summer\USGS Site Response Project\Soil Profiles and Info\CCCC\Proceced Data\CPT_484\cpt_484_2.cpt

CPT: CCCC_CPT484(CGD)

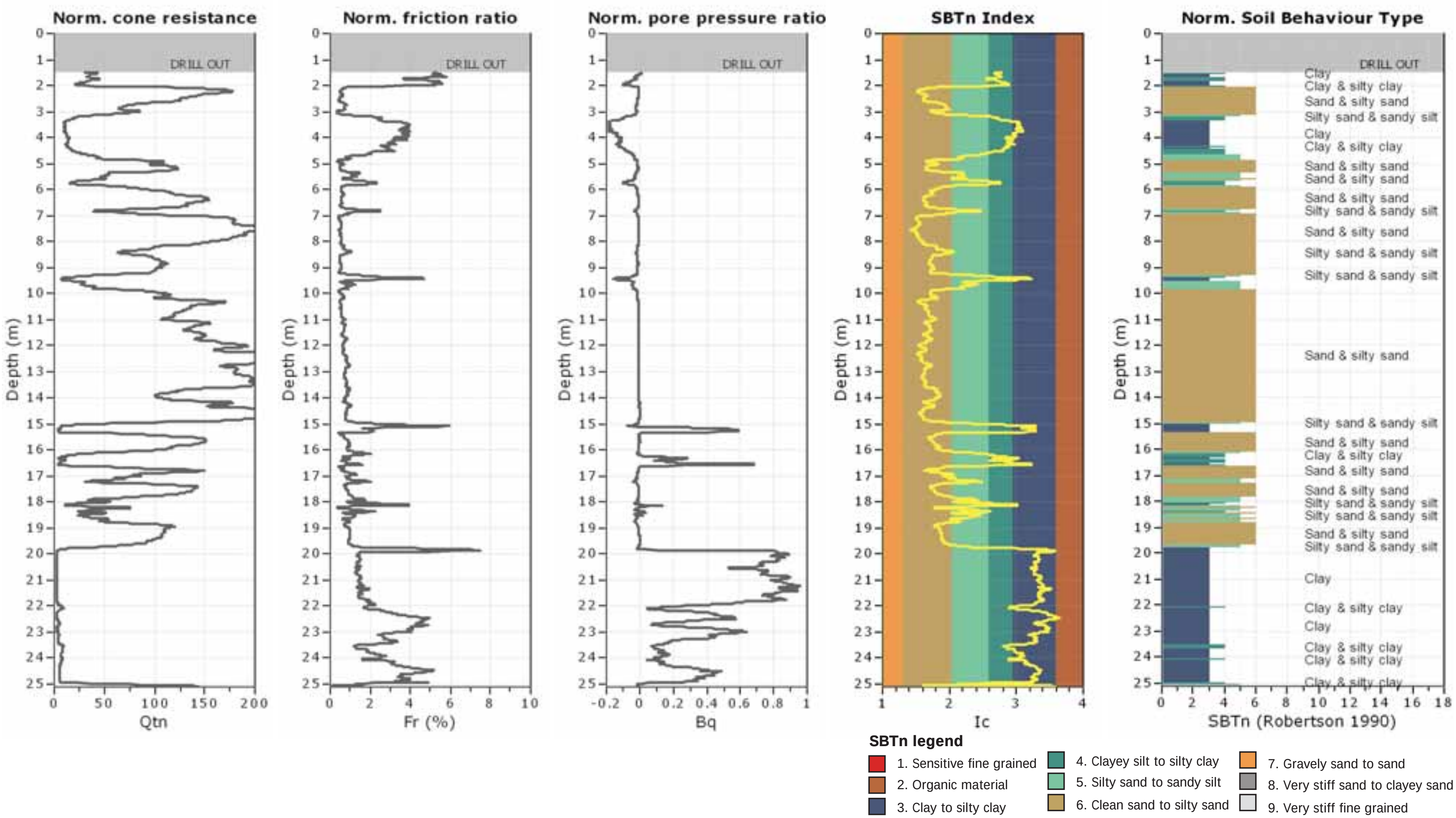
Total depth: 25.11 m, Date: 05/07/2014

Coords: S 43.5379, E 172.6479

Cone Operator: Uknown

Project: Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence

Location: Christchurch, New Zealand



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CPT: CHHC_CPT425(CGD)

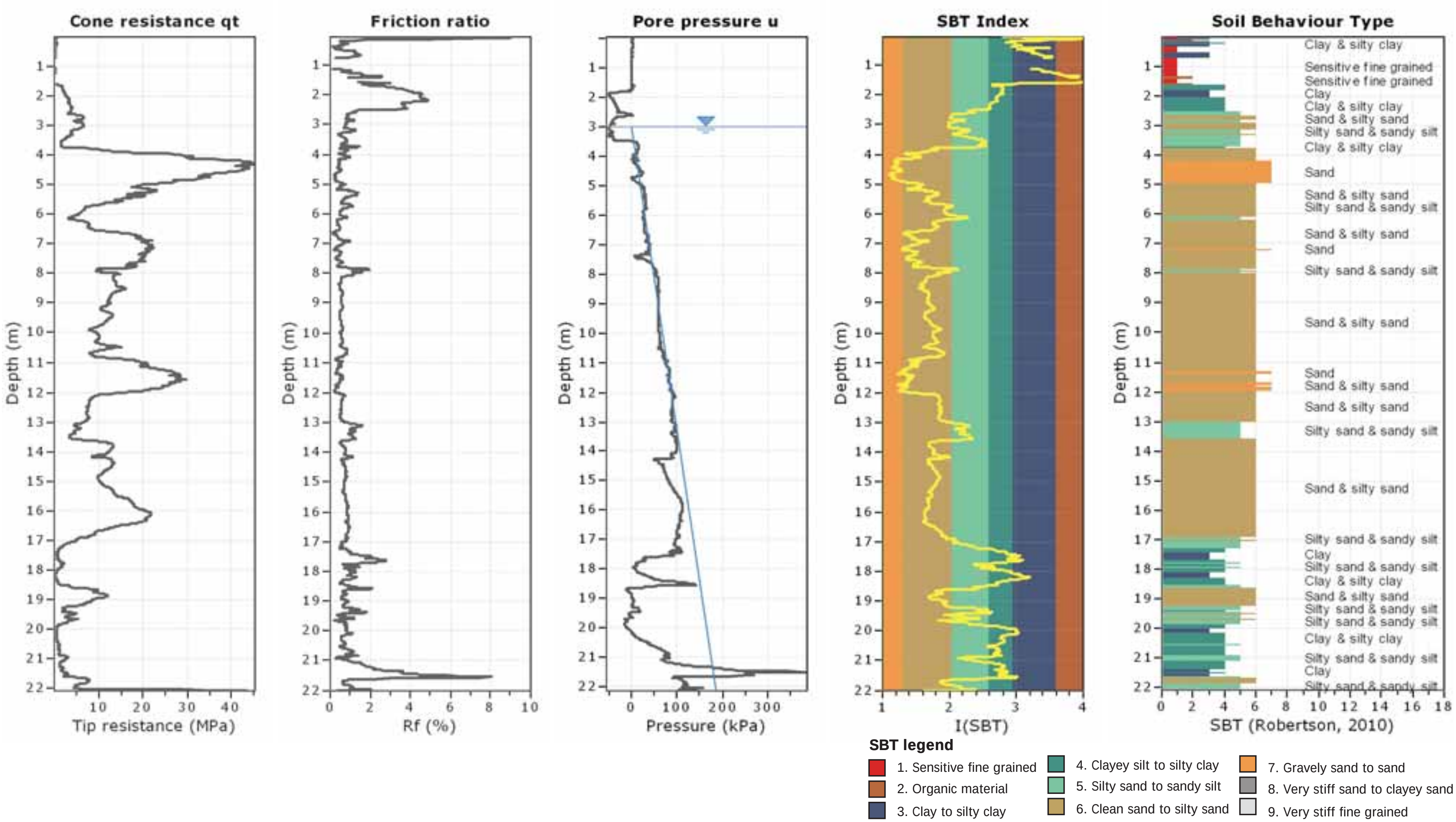
Total depth: 22.08 m

Coords: S 43.5354, E 172.6275

Cone Operator: Uknown

Project: Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence

Location: Christchurch, New Zealand



CPT: CHHC_CPT425(CGD)

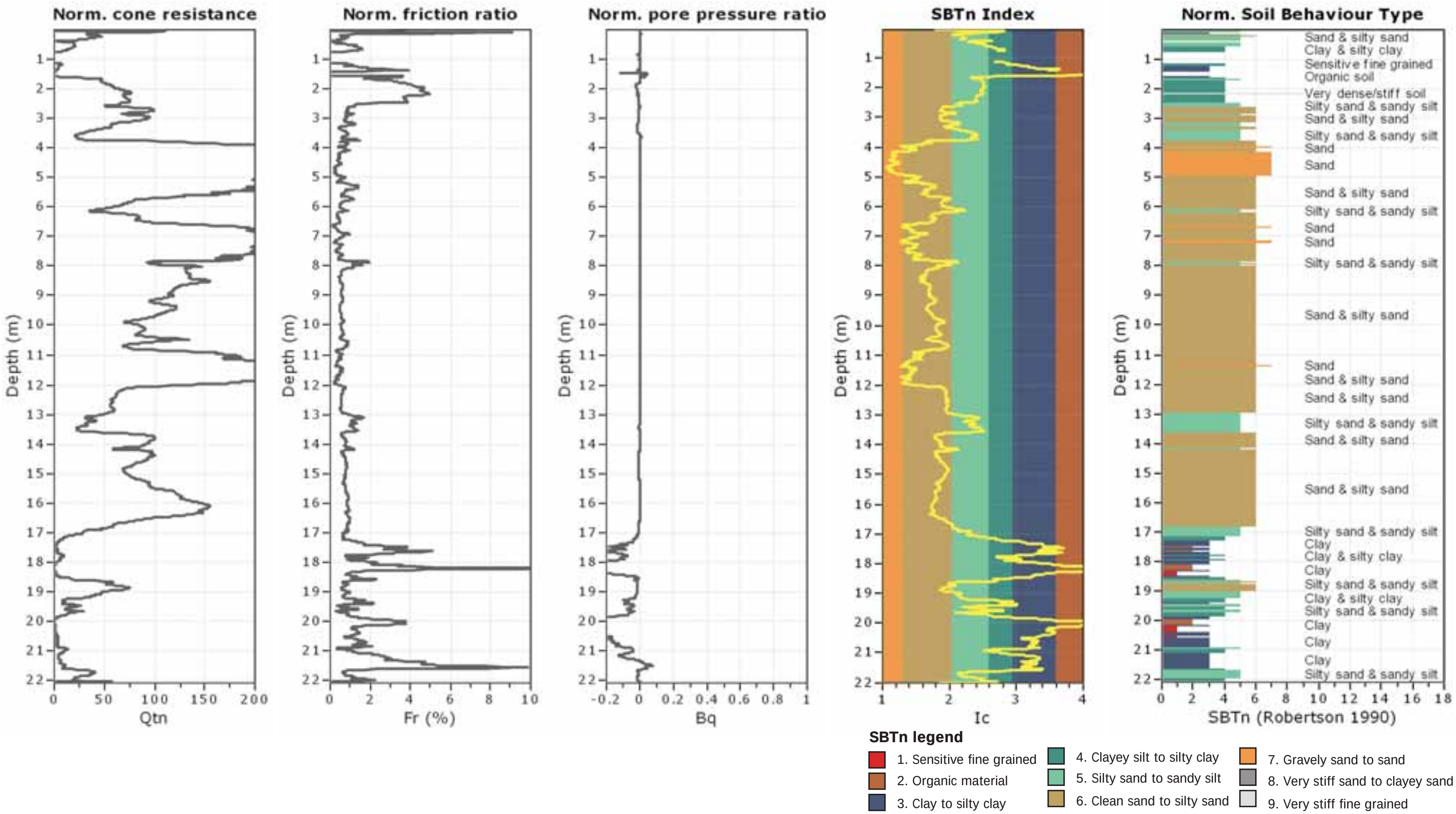
Total depth: 22.08 m, Date: 09/07/2014

Coords: S 43.5354, E 172.6275

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



CPT: HPSC_CPT89(CGD)

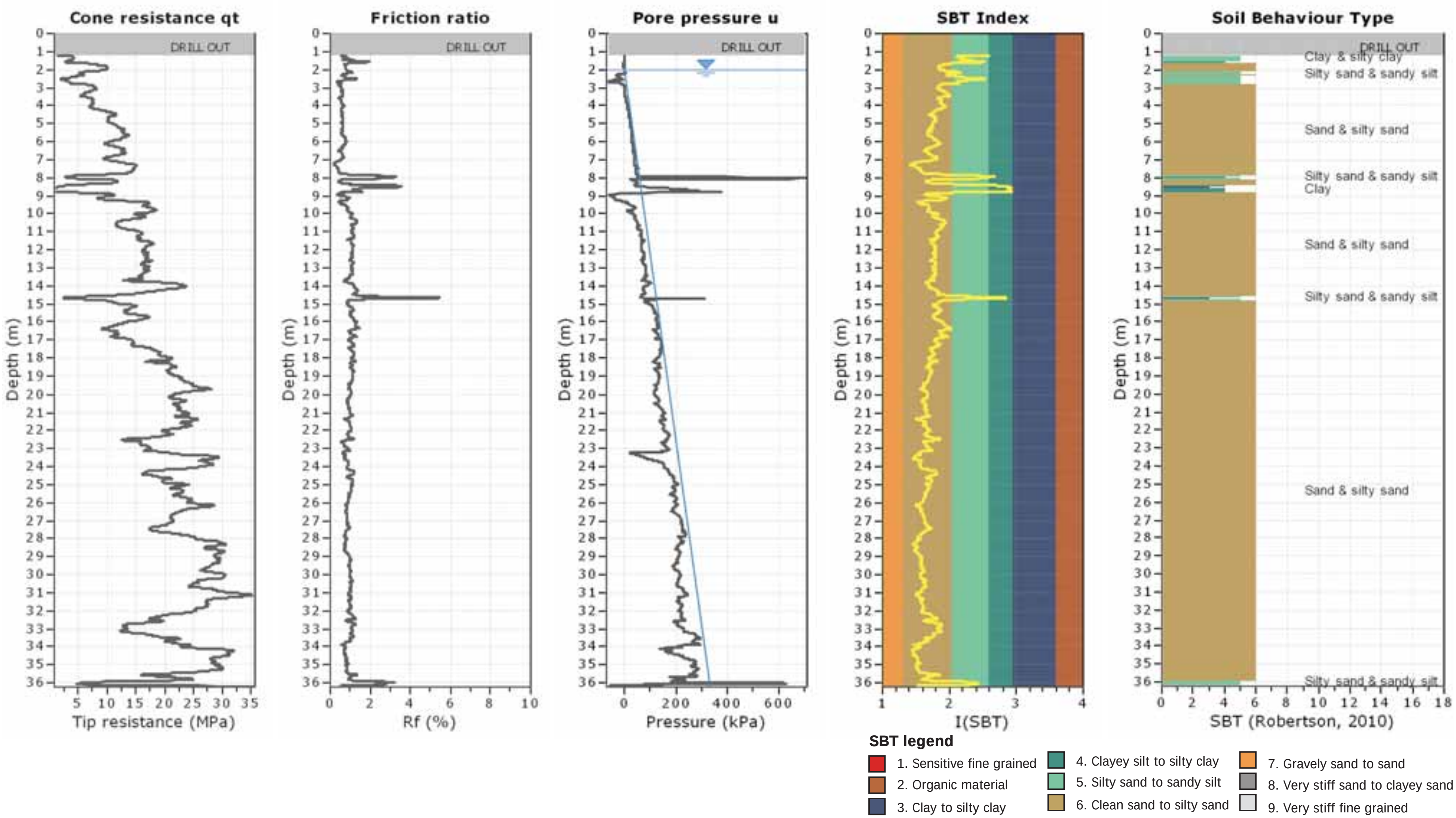
Total depth: 36.23 m

Coords: S 43.5014, E 172.7021

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Stress Site Response Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



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Project file: C:\Users\User\Desktop\summer\USGS Site Response Project\Soil Profiles and Info\HPSC\Proceeced Data\CPT89(13m)\CPT89(13m)_2.cpt

CPT: HPSC_CPT89(CGD)

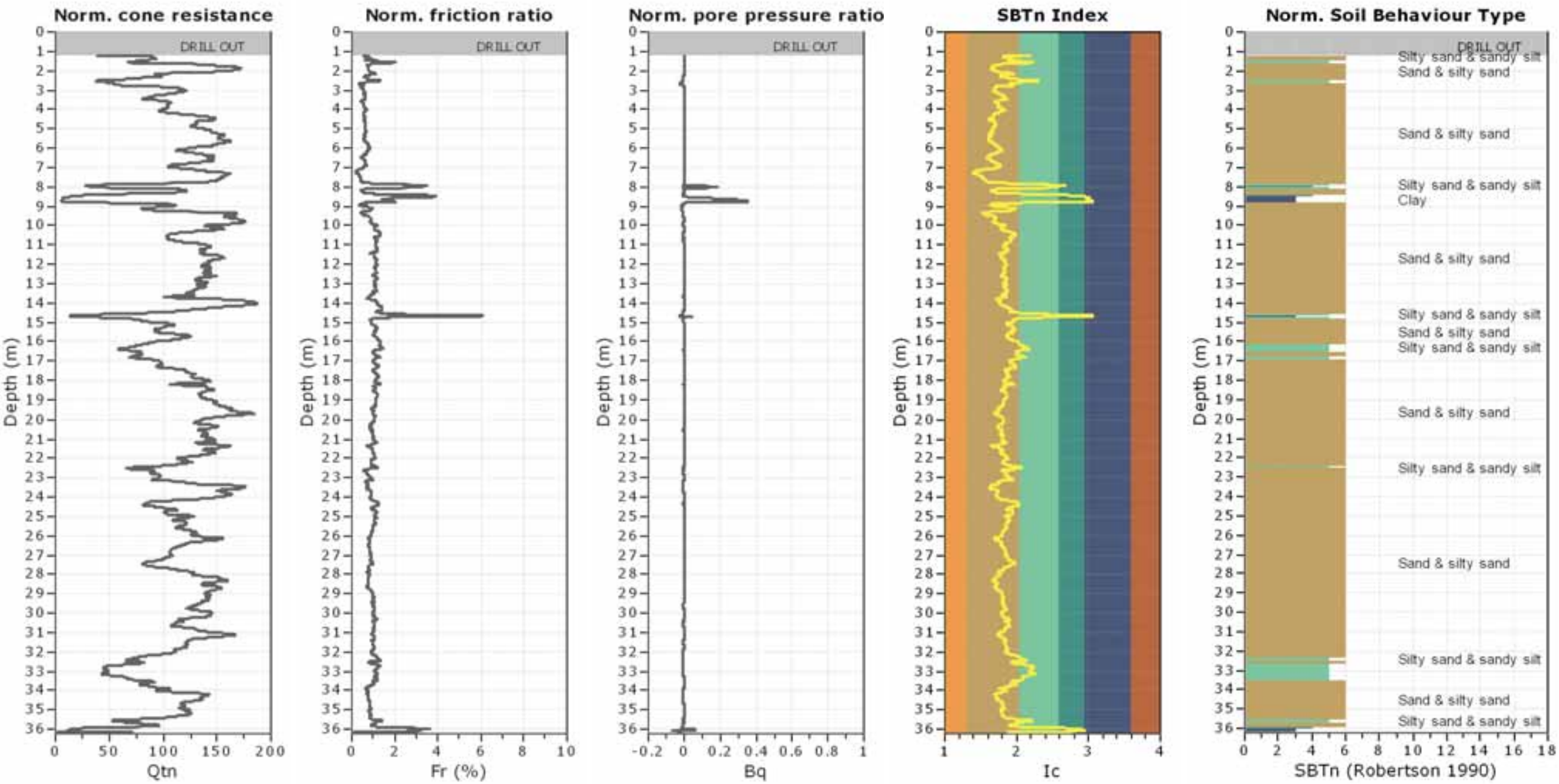
Total depth: 36.23 m, Date: 12/07/2014

Coords: S 43.5014, E 172.7021

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Stress Site Response Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



SBTn legend

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

CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 21/08/2014, 04:31:30 p.m.

Project file: C:\Users\User\Desktop\summer\USGS Site Response Project\Soil Profiles and Info\HPSC\Proceeced Data\CPT89(13m)\CPT89(13m)_2.cpt

CPT: NNBS_CPT33695(CGD)

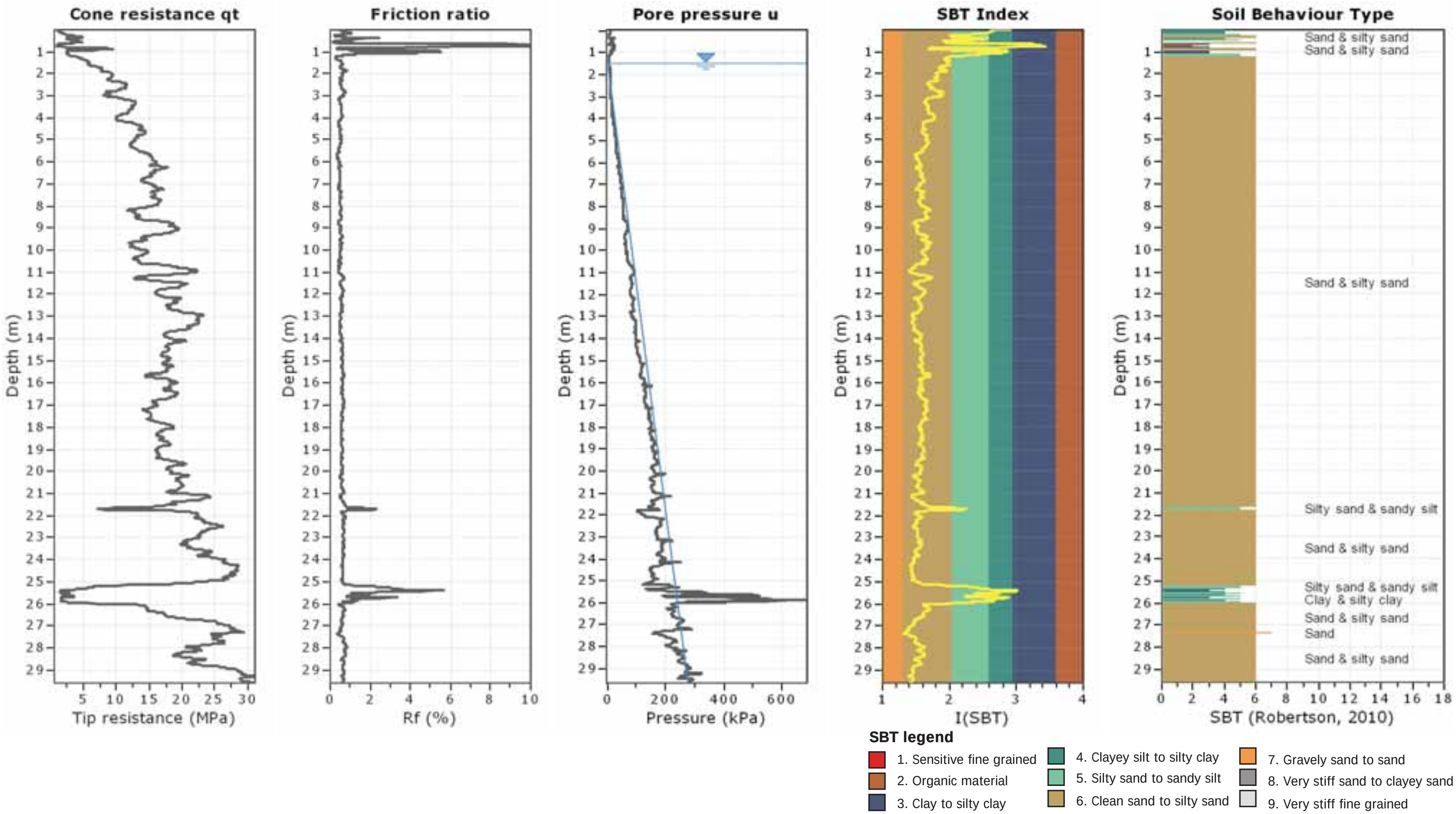
Total depth: 29.59 m

Coords: S 43.4953, E 172.7181

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Stress Site Response Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**

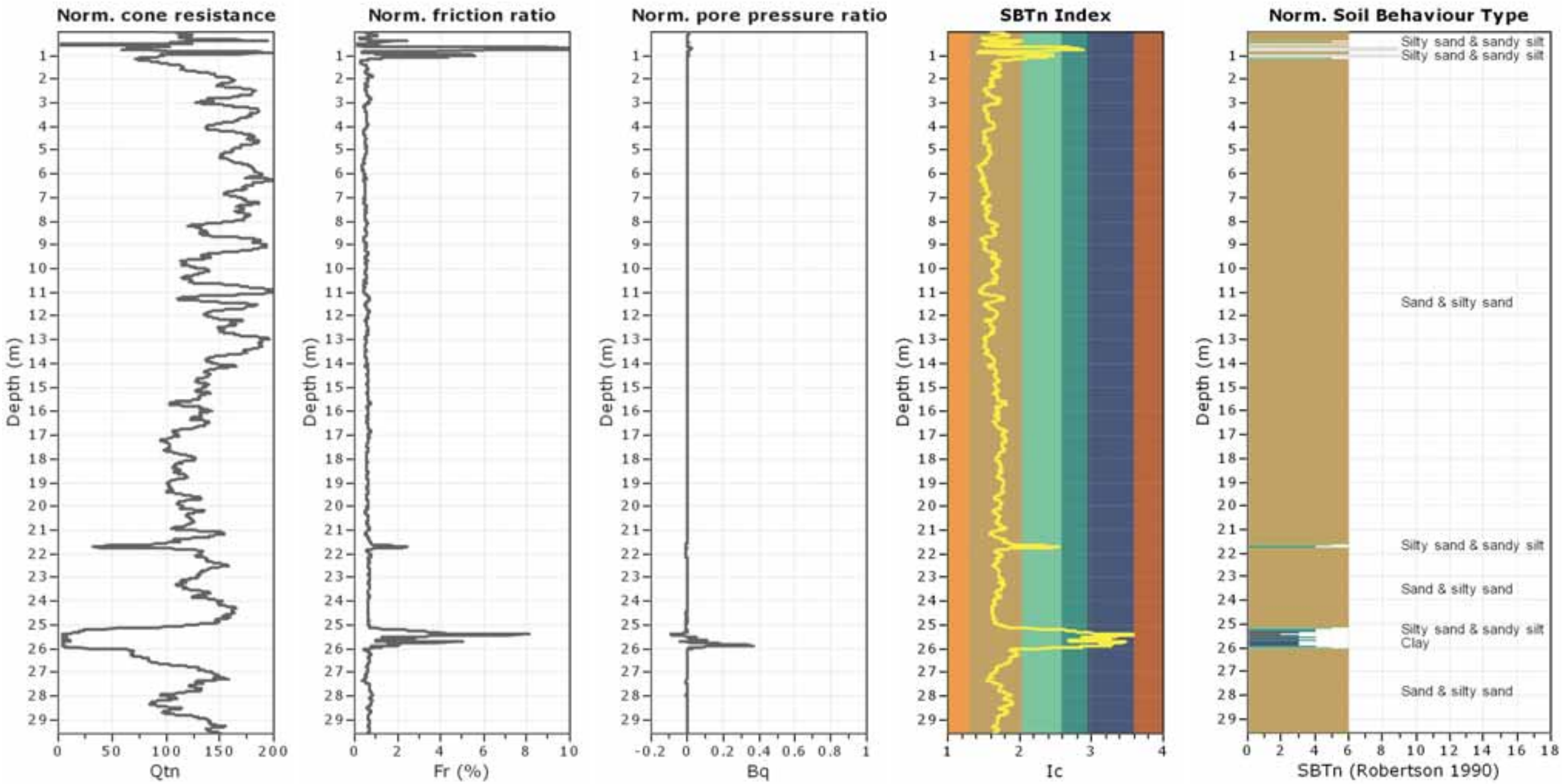


CPT: NNBS_CPT33695(CGD)

Total depth: 29.59 m, Date: 22/07/2014

Project: Evaluating Fully Nonlinear Stress Site Response Computer Programs using Records from the Canterbury Earthquake Sequence Coords: S 43.4953, E 172.7181

Location: Christchurch, New Zealand Cone Operator: Uknown



SBTn legend

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

CPT: PPHS_CPT1497(CGD)

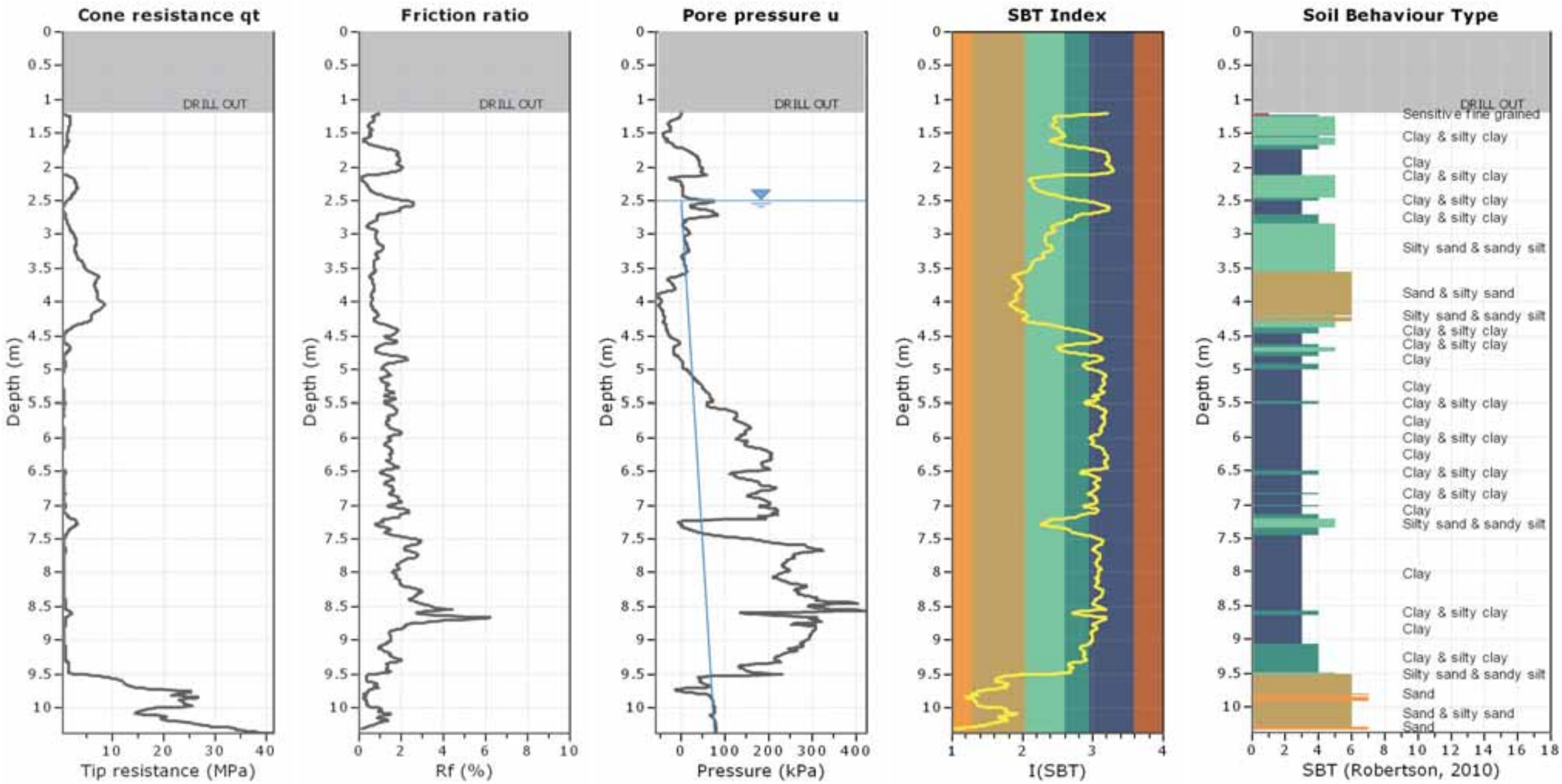
Total depth: 10.38 m

Coords: S 43.4932, E 172.6067

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Stress Site Response Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



SBT legend

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

CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 21/08/2014, 04:28:47 p.m.

Project file: C:\Users\User\Desktop\summer\USGS Site Response Project\Soil Profiles and Info\PPHS\Proceced Data\CPT1497(125m)\cpt1497(125m)_2.cpt

CPT: PPHS_CPT1497(CGD)

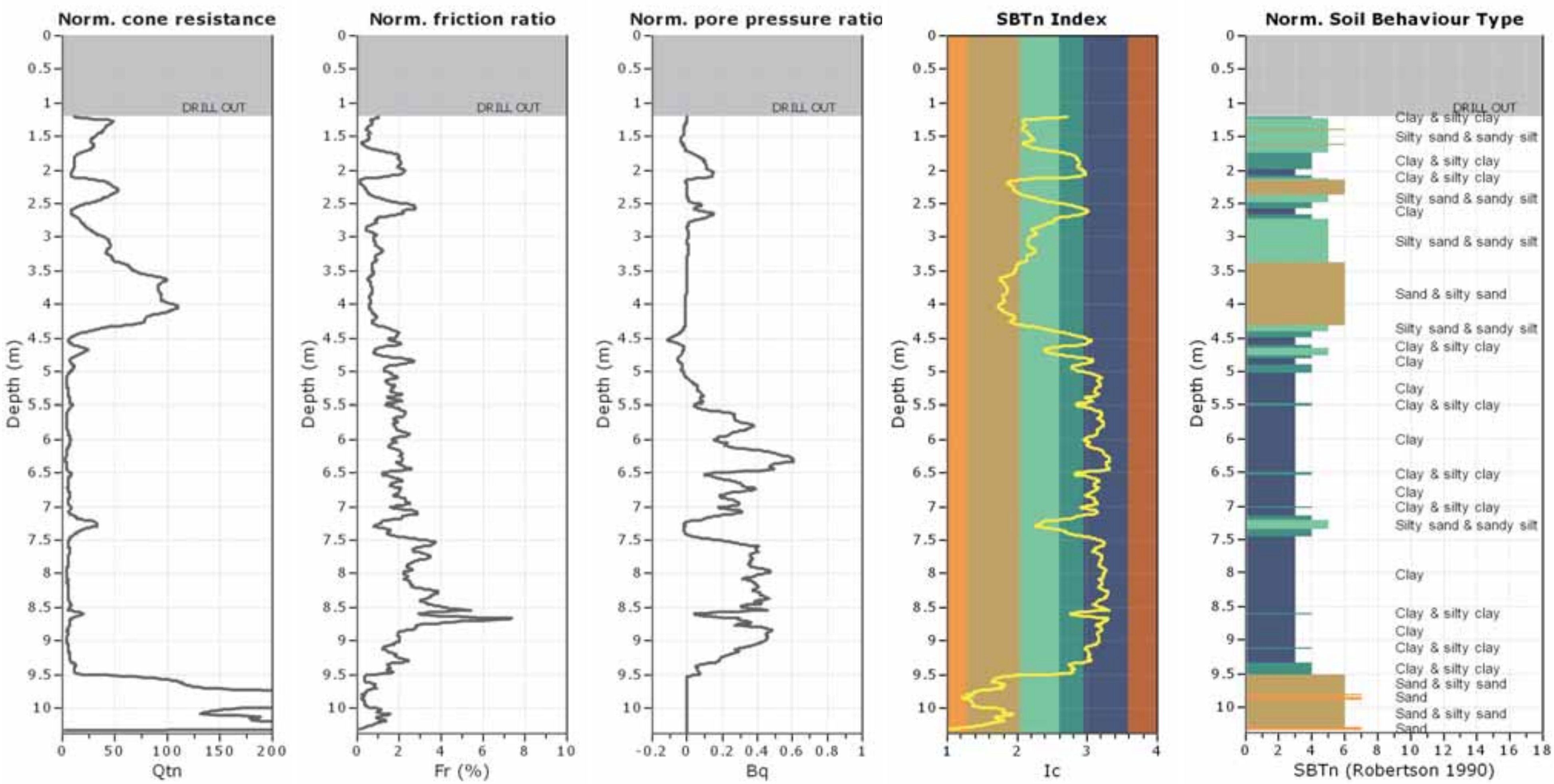
Total depth: 10.38 m, Date: 13/07/2014

Coords: S 43.4932, E 172.6067

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Stress Site Response Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



SBTn legend

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

CPT: PRPC_CPT1396 (CGD)

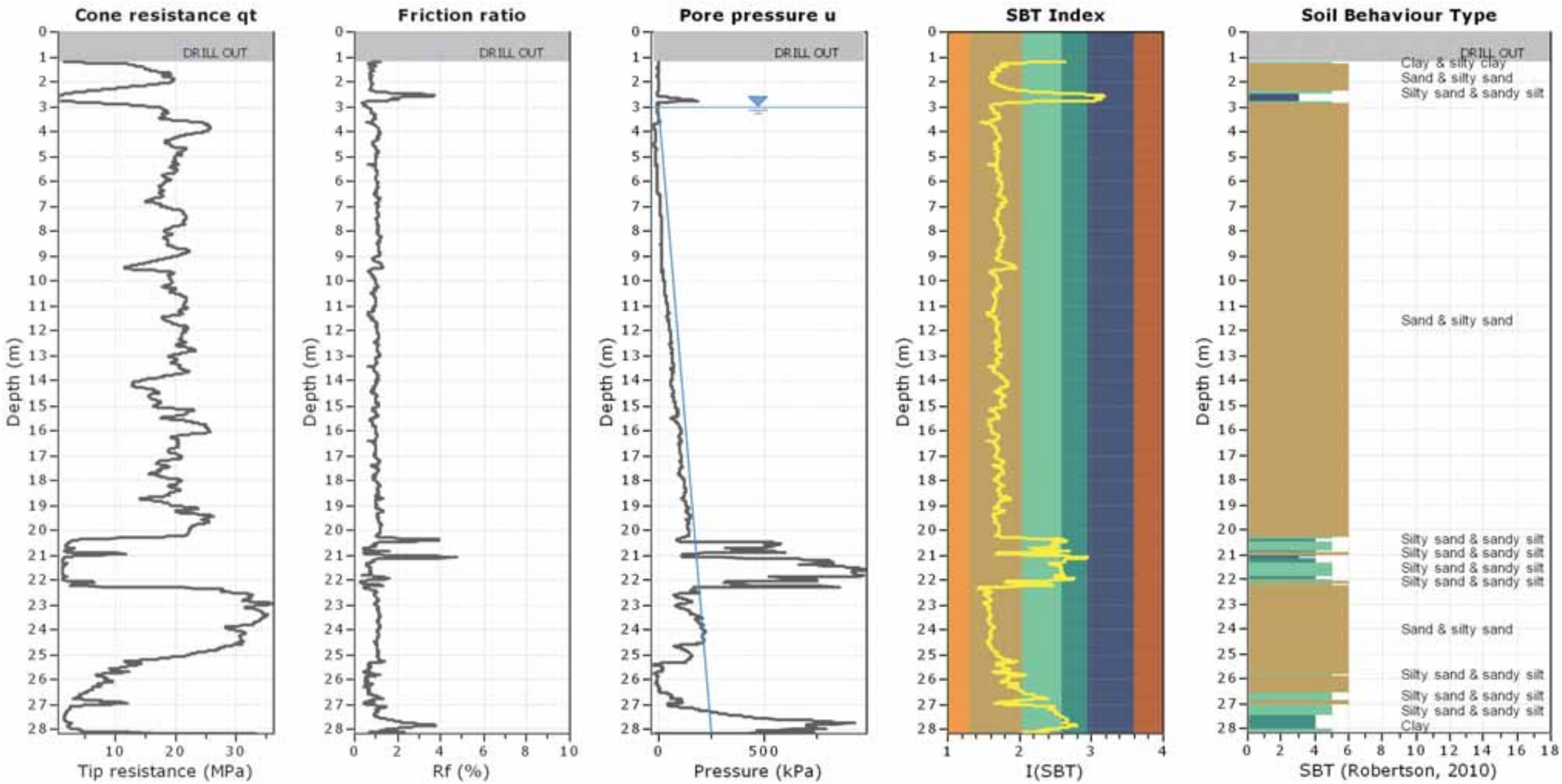
Total depth: 28.16 m

Coords: S 43.5259, E 172.6828

Cone Operator: Uknown

Project:

Location:



SBT legend

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

CPT: PRPC_CPT1396 (CGD)

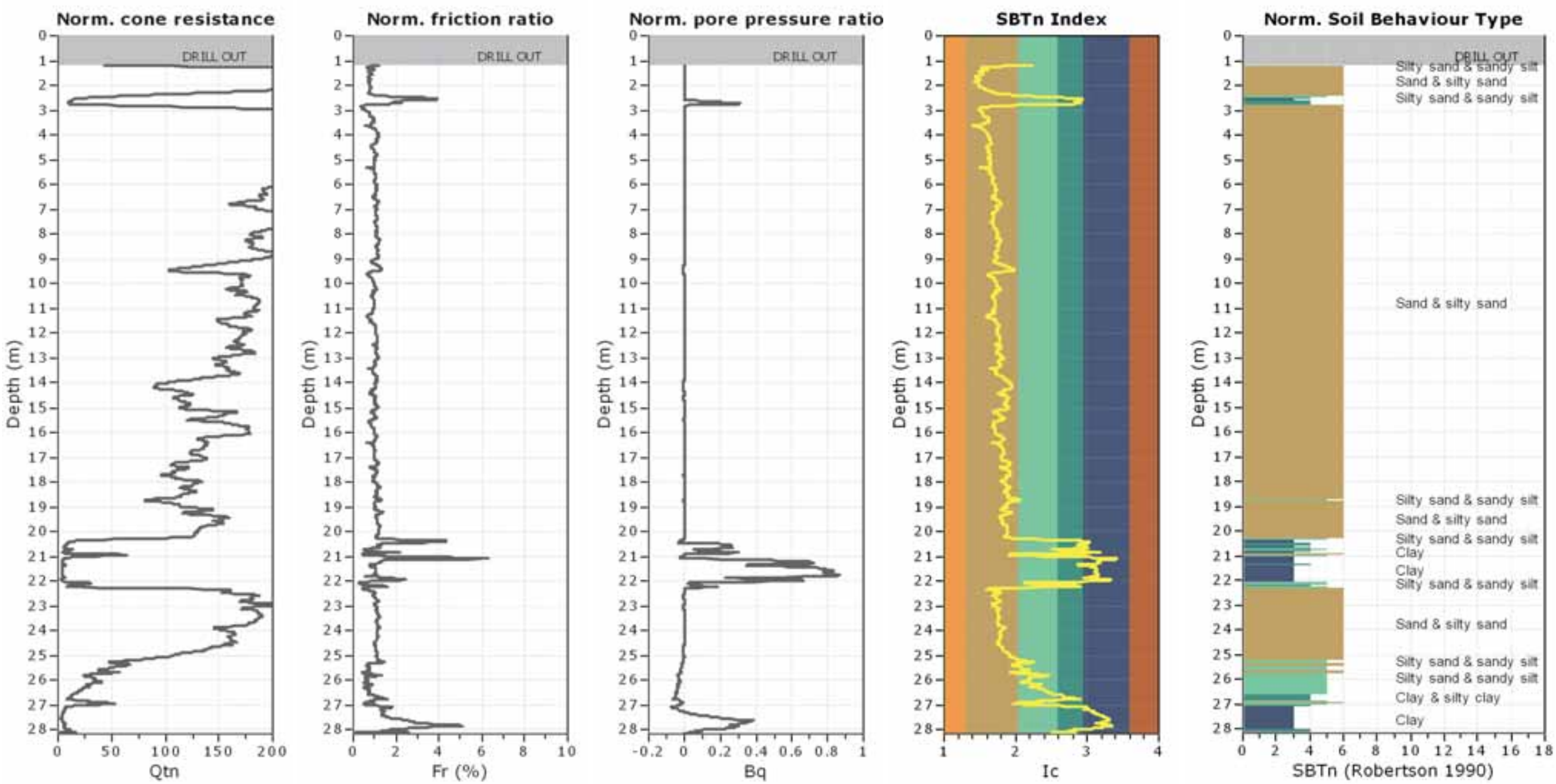
Total depth: 28.16 m, Date: 18/07/2014

Coords: S 43.5259, E 172.6828

Cone Operator: Uknown

Project:

Location:



SBTn legend

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

CPT: REHS_CPT2 (Wotherspoon,2013)

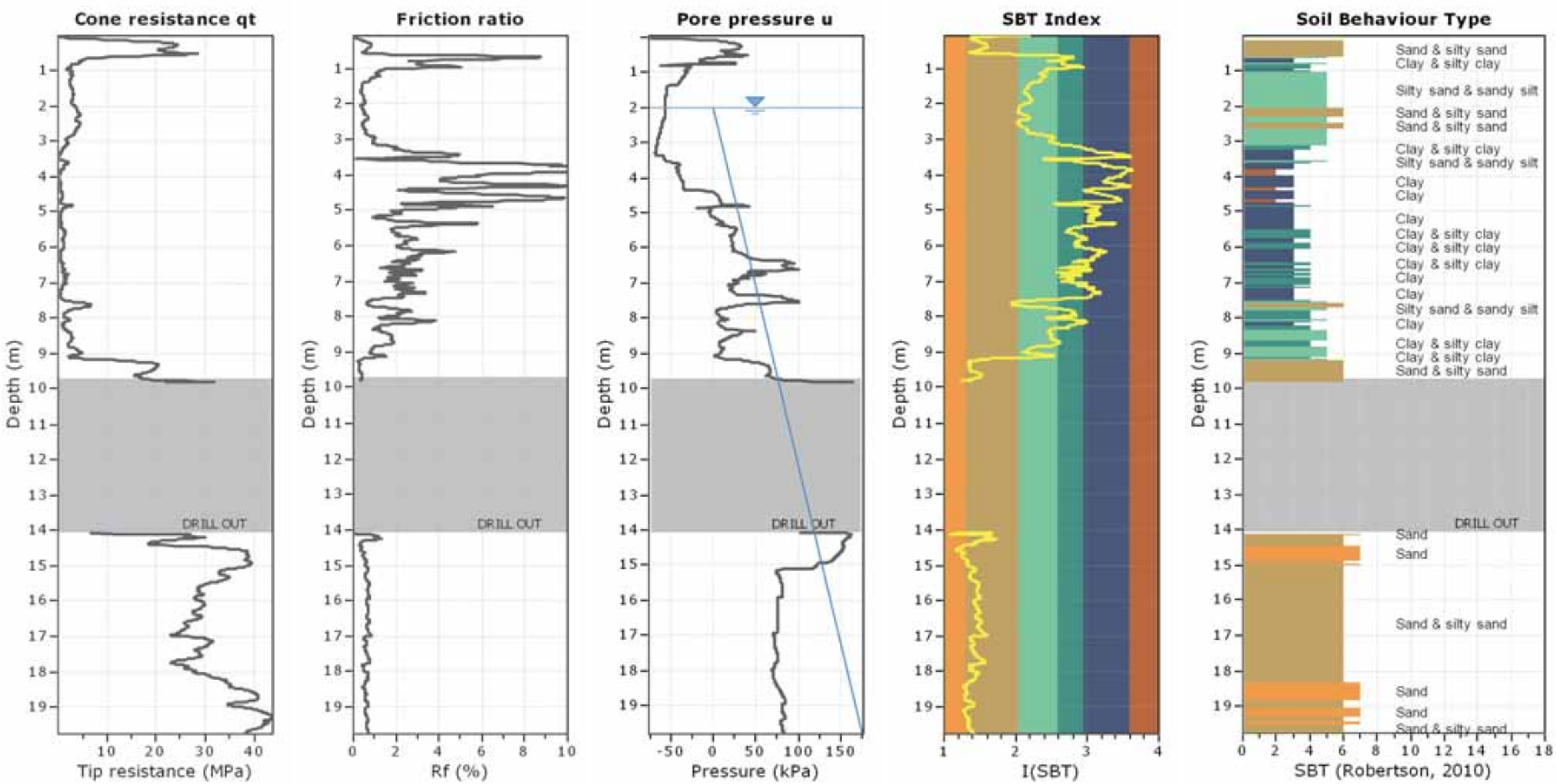
Total depth: 19.76 m

Coords: S 43.5220, E172.6351

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



SBT legend

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

CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 21/08/2014, 01:28:45 a.m.

Project file: C:\Users\User\Desktop\summer\USGS Site Response Project\Soil Profiles and Info\REHS\Proceced Data\Wotherspoon\REHC_CPT1_CPT1a_2.cpt

CPT: REHS_CPT2 (Wotherspoon,2013)

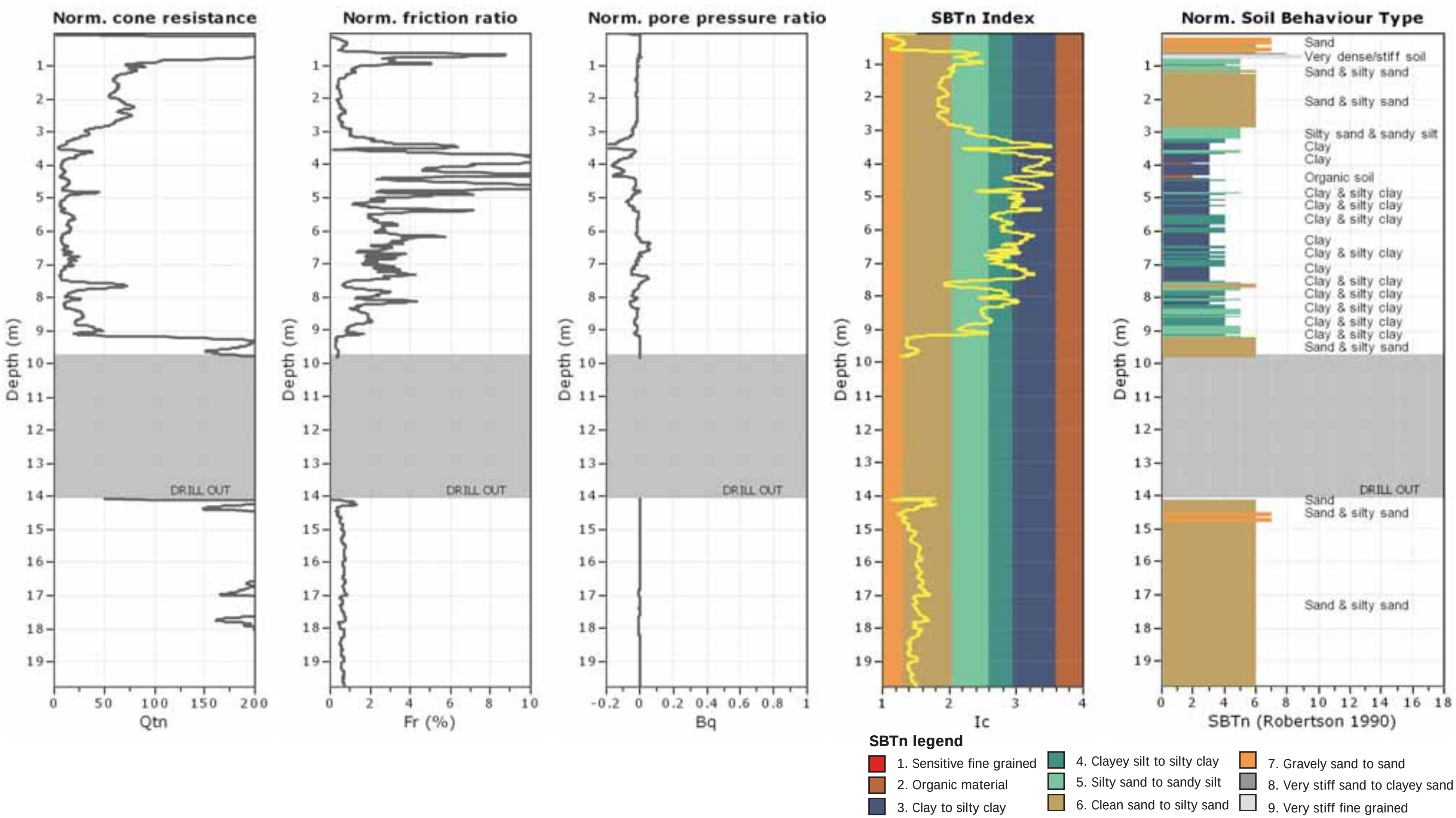
Total depth: 19.76 m, Date: 06/07/2014

Coords: S 43.5220, E172.6351

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



CPT: SHLC_CPT626(CGD)

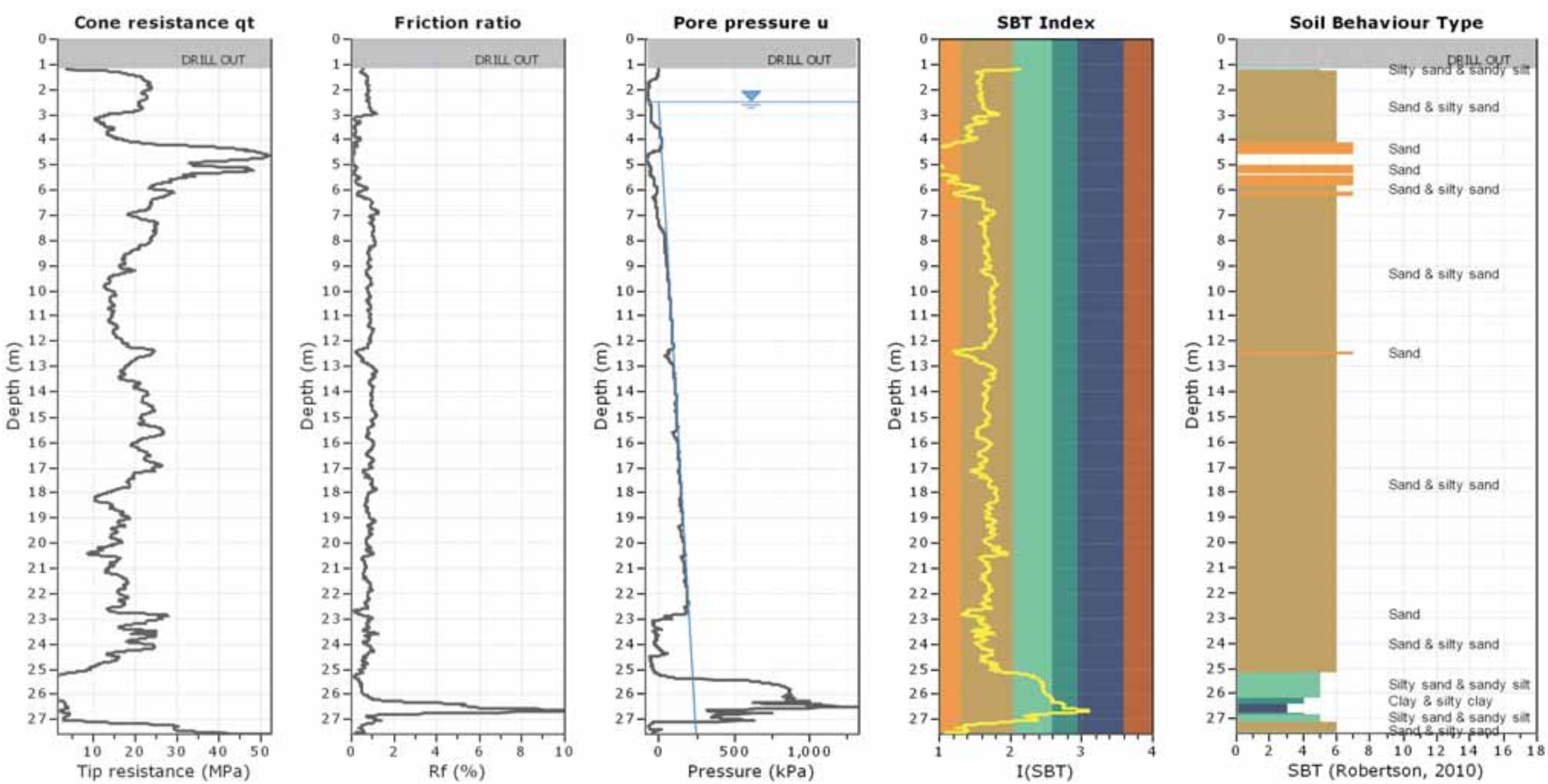
Total depth: 27.58 m

Coords: S 43.5054, E 172.6628

Cone Operator: Uknown

Project: **Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence**

Location: **Christchurch, New Zealand**



SBT legend

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

CPT: SHLC_CPT626(CGD)

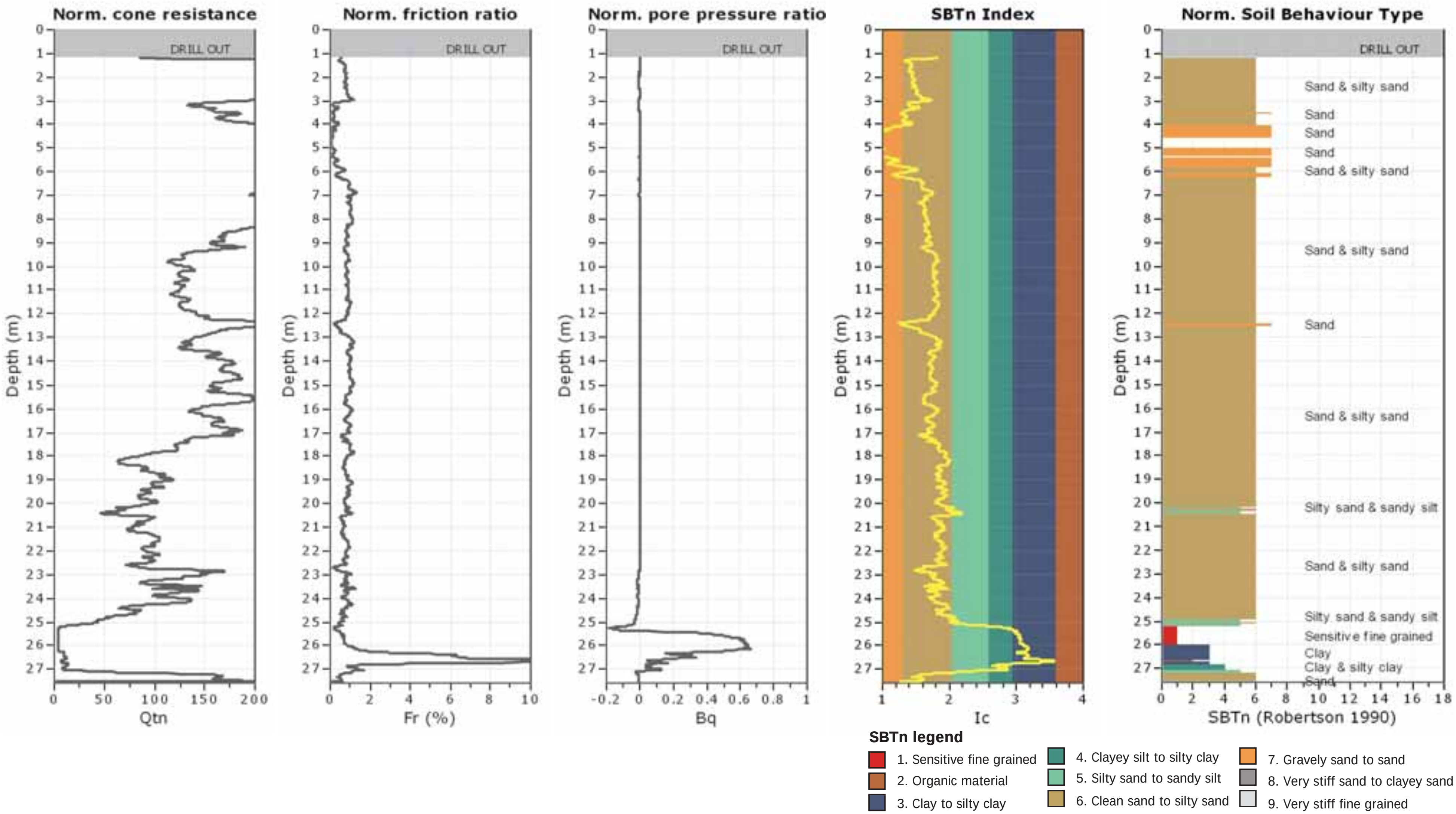
Total depth: 27.58 m, Date: 11/07/2014

Project: **Evaluating Fully Nonlinear Effective Stress Site Resonse Computer Programs using Records from the Canterbury Earthquake Sequence**

Coords: S 43.5054, E 172.6628

Location: **Christchurch, New Zealand**

Cone Operator: Uknown



CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 21/08/2014, 04:39:13 p.m.

Project file: C:\Users\User\Desktop\summer\USGS Site Response Project\Soil Profiles and Info\SHLC\Proceced Data\CPT626(50m)\CPT626(50m)_2.cpt

APPENDIX B.2

Shear Wave Velocity Profiles for Seismic Site Response Analyses.

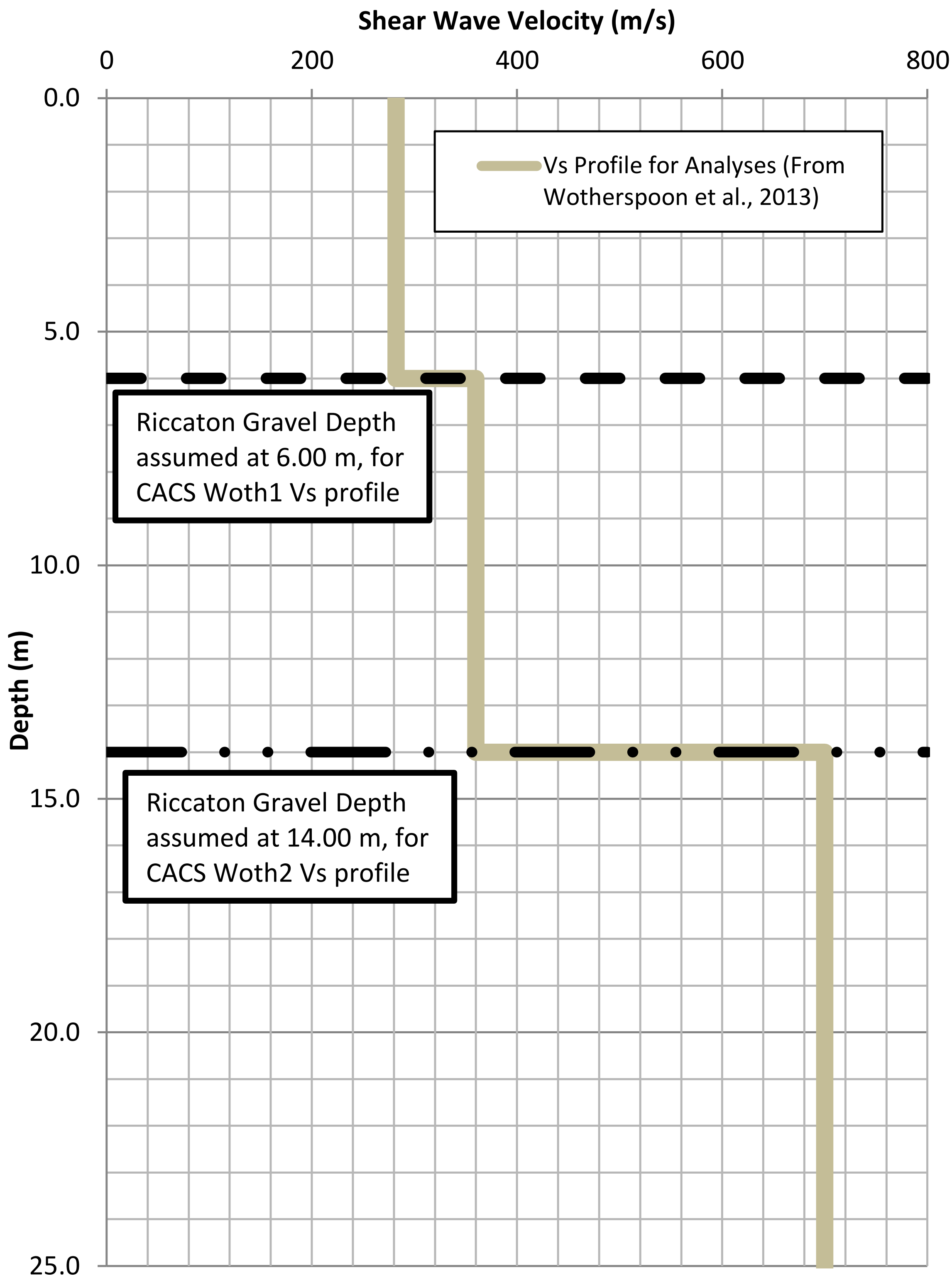


Figure B.2.1: Shear Wave velocity profile for CACS strong ground motion station.

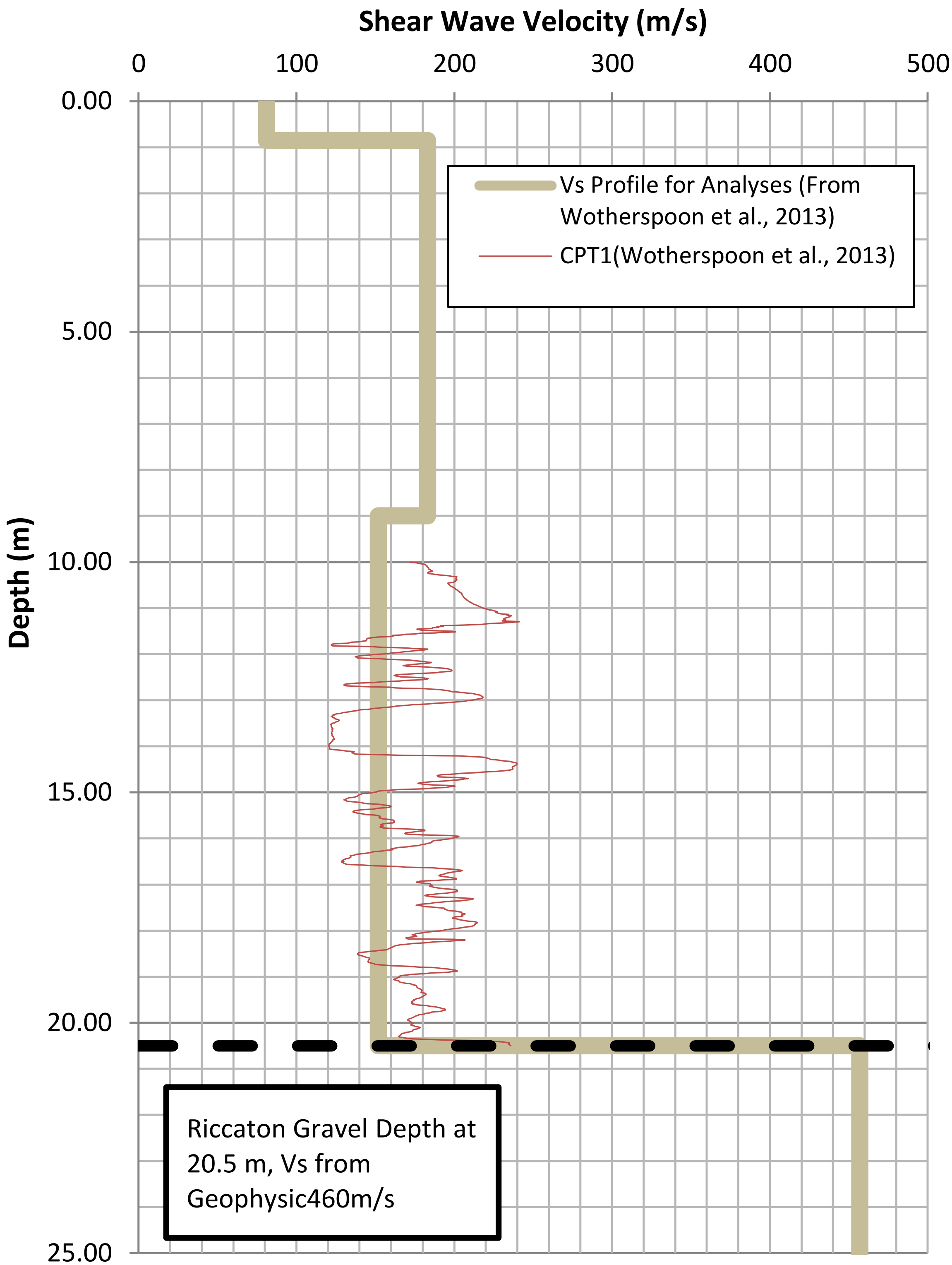


Figure B.2.2: Shear Wave velocity profile for CBGS strong ground motion station.

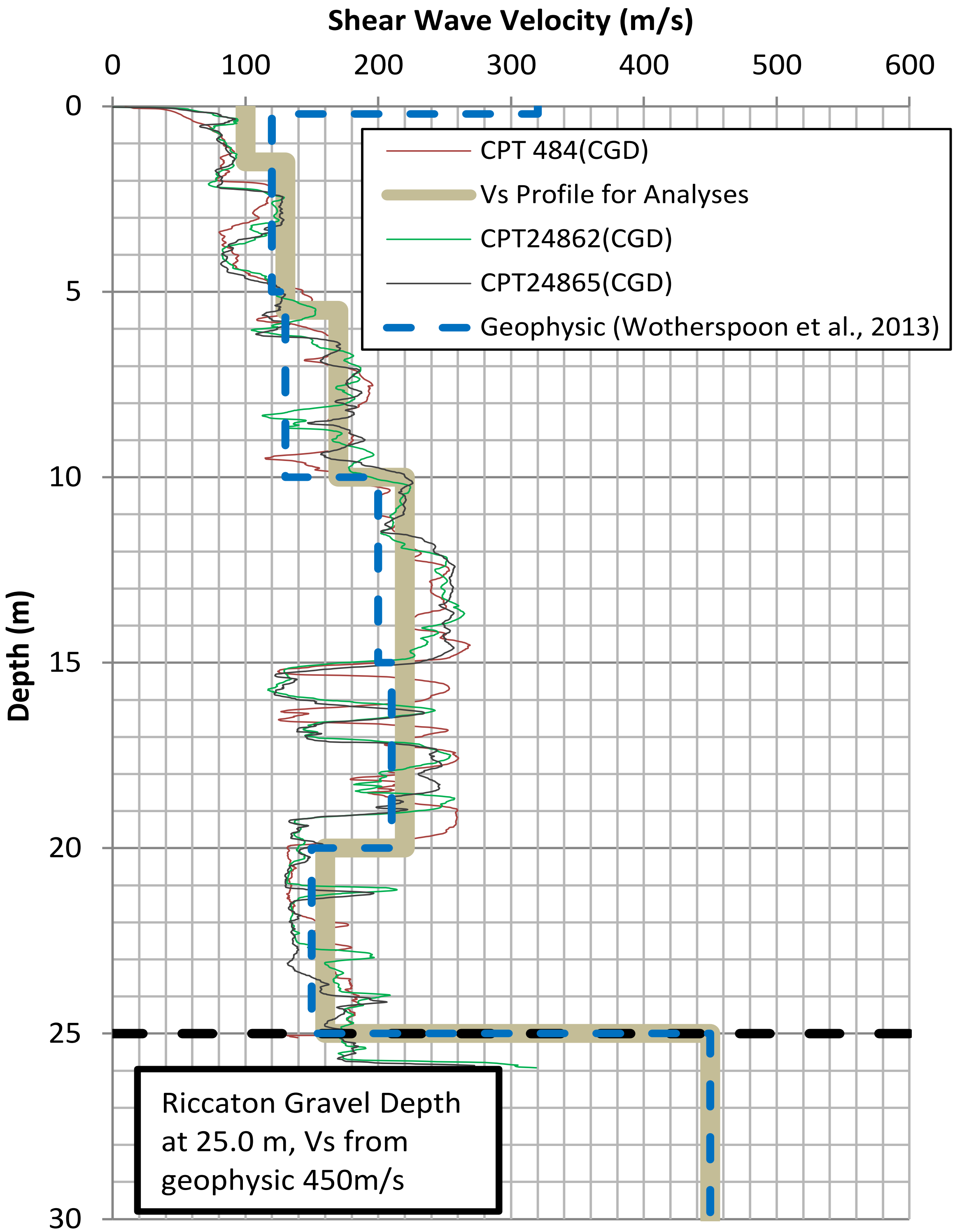


Figure B.2.3: Shear Wave velocity profile for CCCC strong ground motion station.

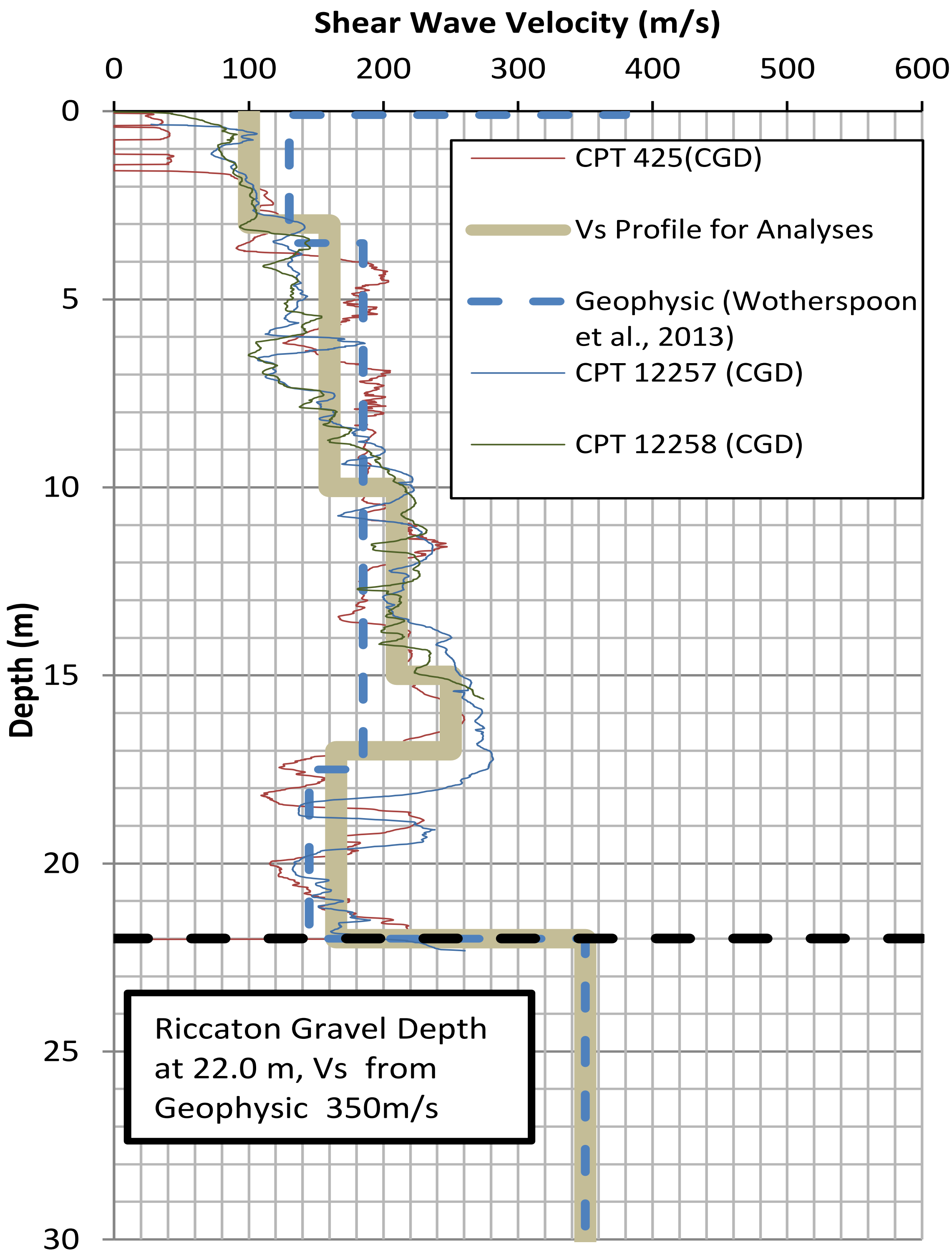


Figure B.2.4: Shear Wave velocity profile for CHHC strong ground motion station.

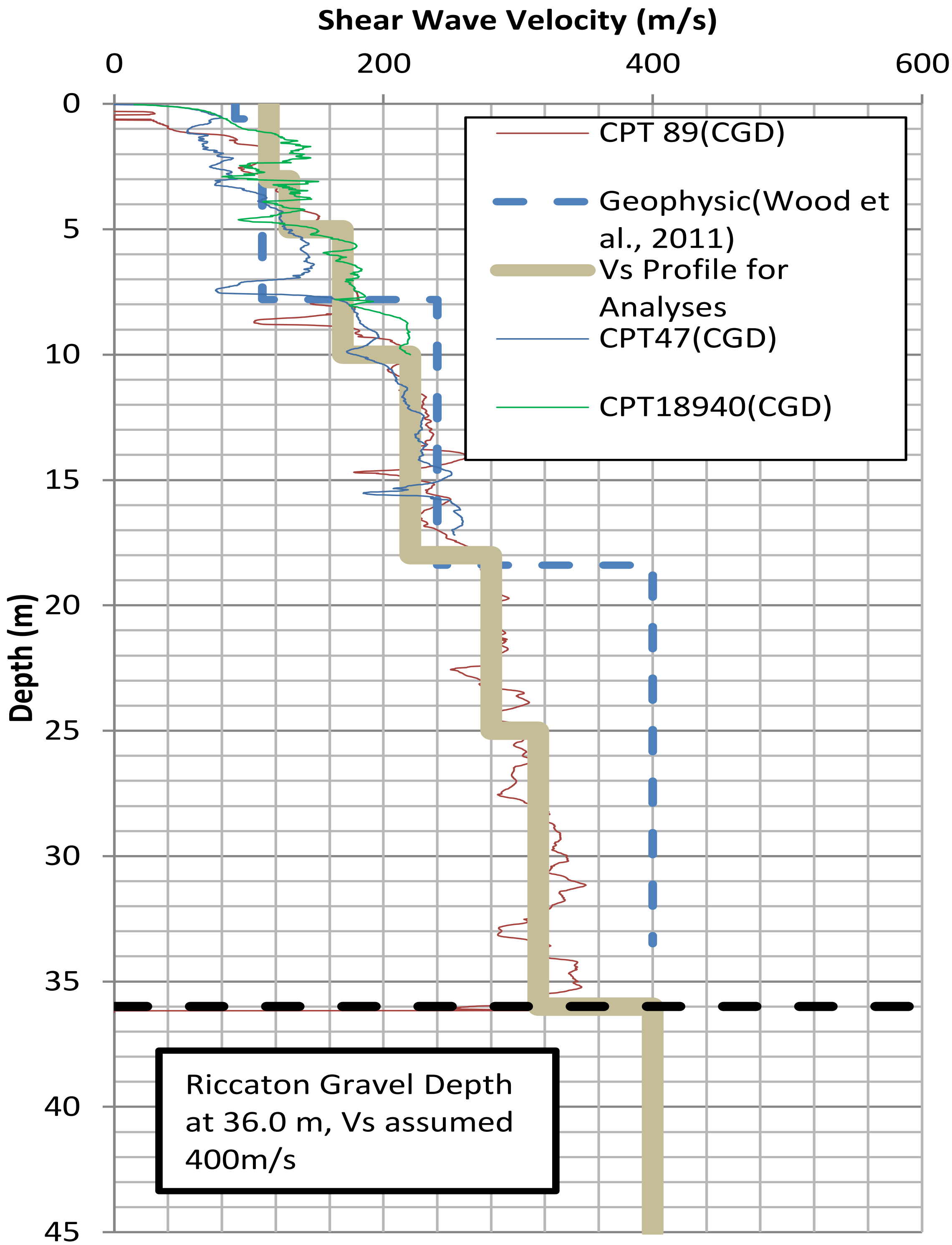


Figure B.2.5: Shear Wave velocity profile for HPSC strong ground motion station.

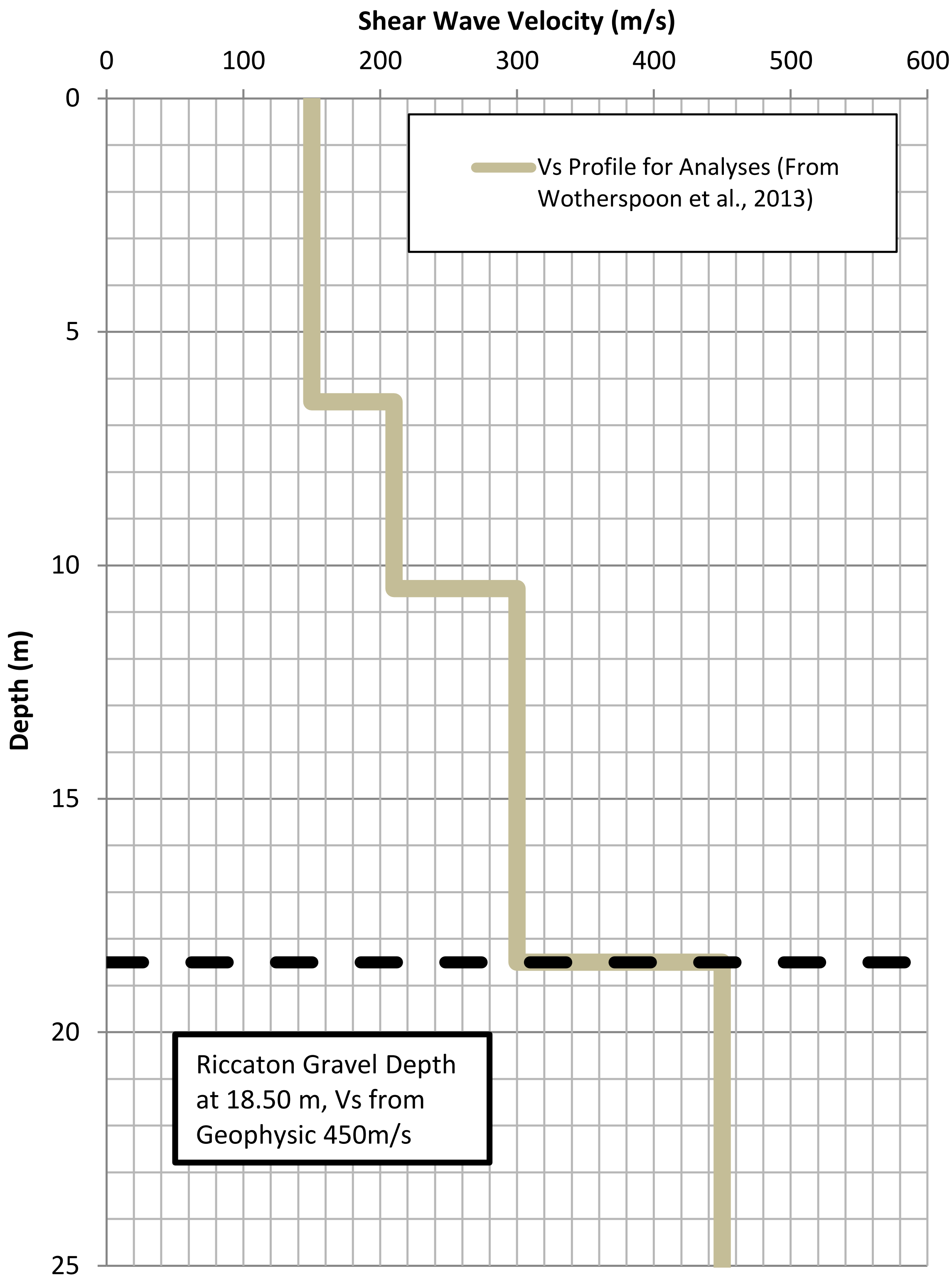


Figure B.2.6: Shear Wave velocity profile for KPOC strong ground motion station.

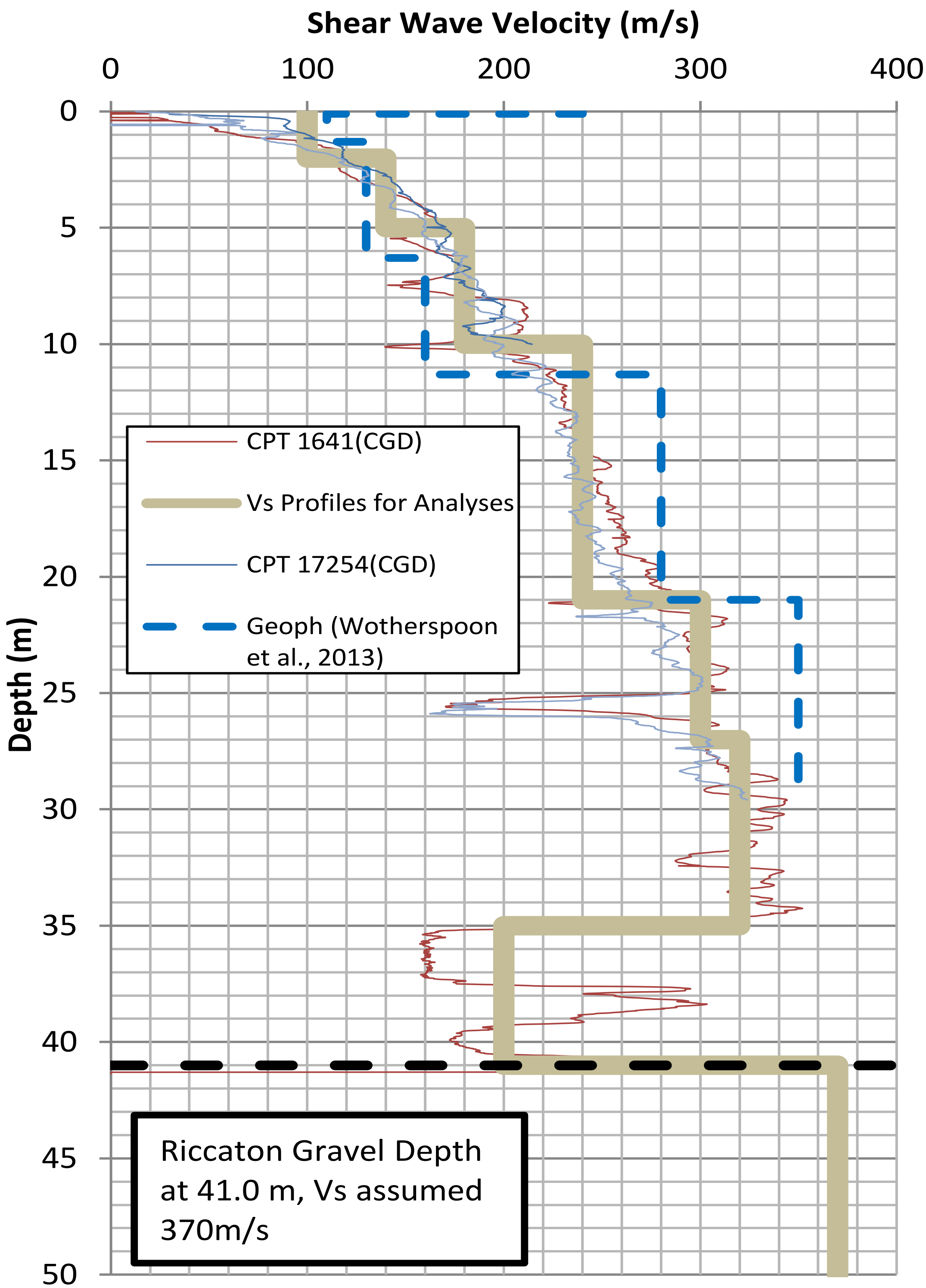


Figure B.2.7: Shear Wave velocity profile for NNBS strong ground motion station.

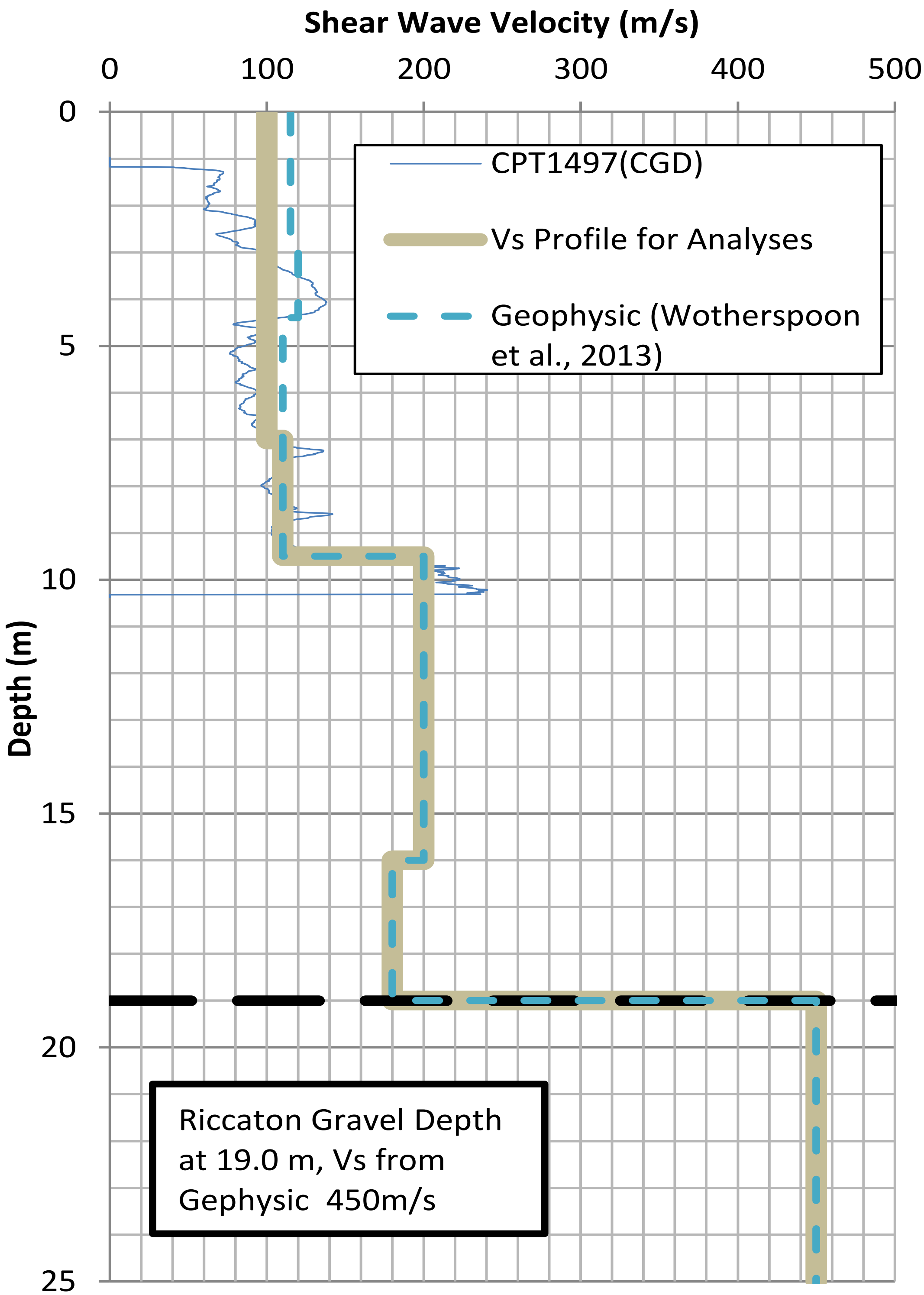


Figure B.2.8: Shear Wave velocity profile for PPHS strong ground motion station.

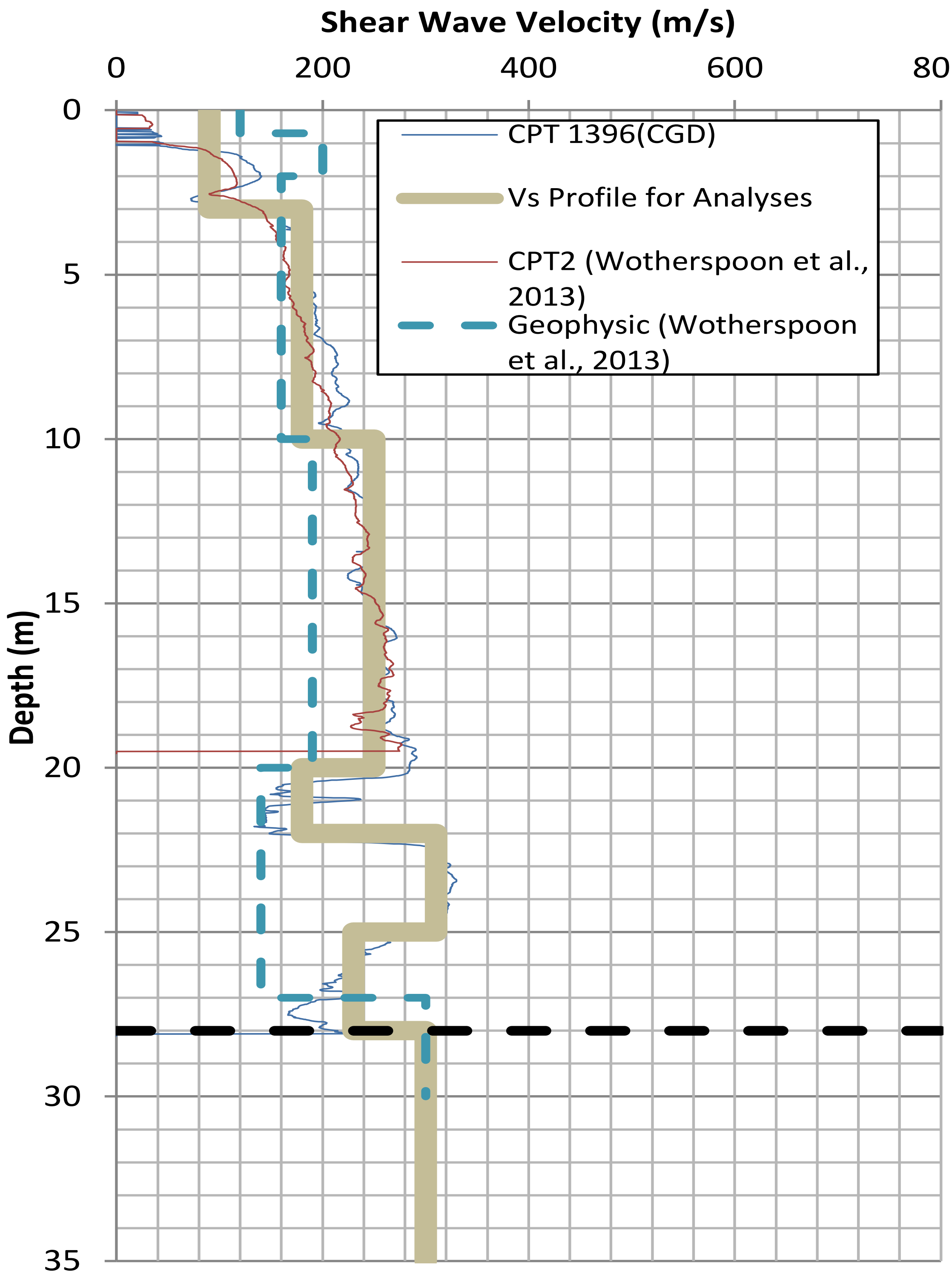


Figure B.2.9: Shear Wave velocity profile for PRPC strong ground motion station.

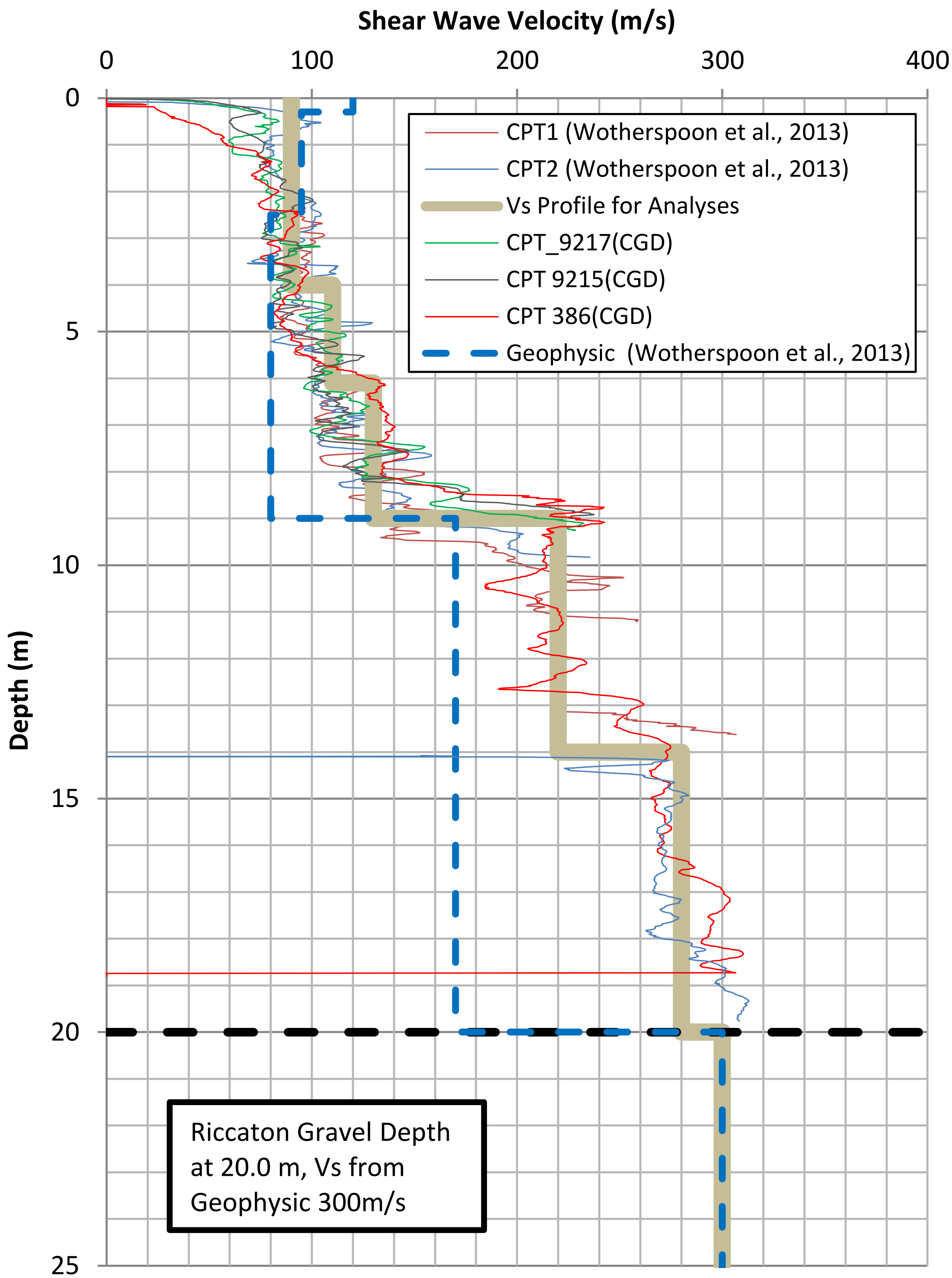


Figure B.2.10: Shear Wave velocity profile for REHS strong ground motion station.

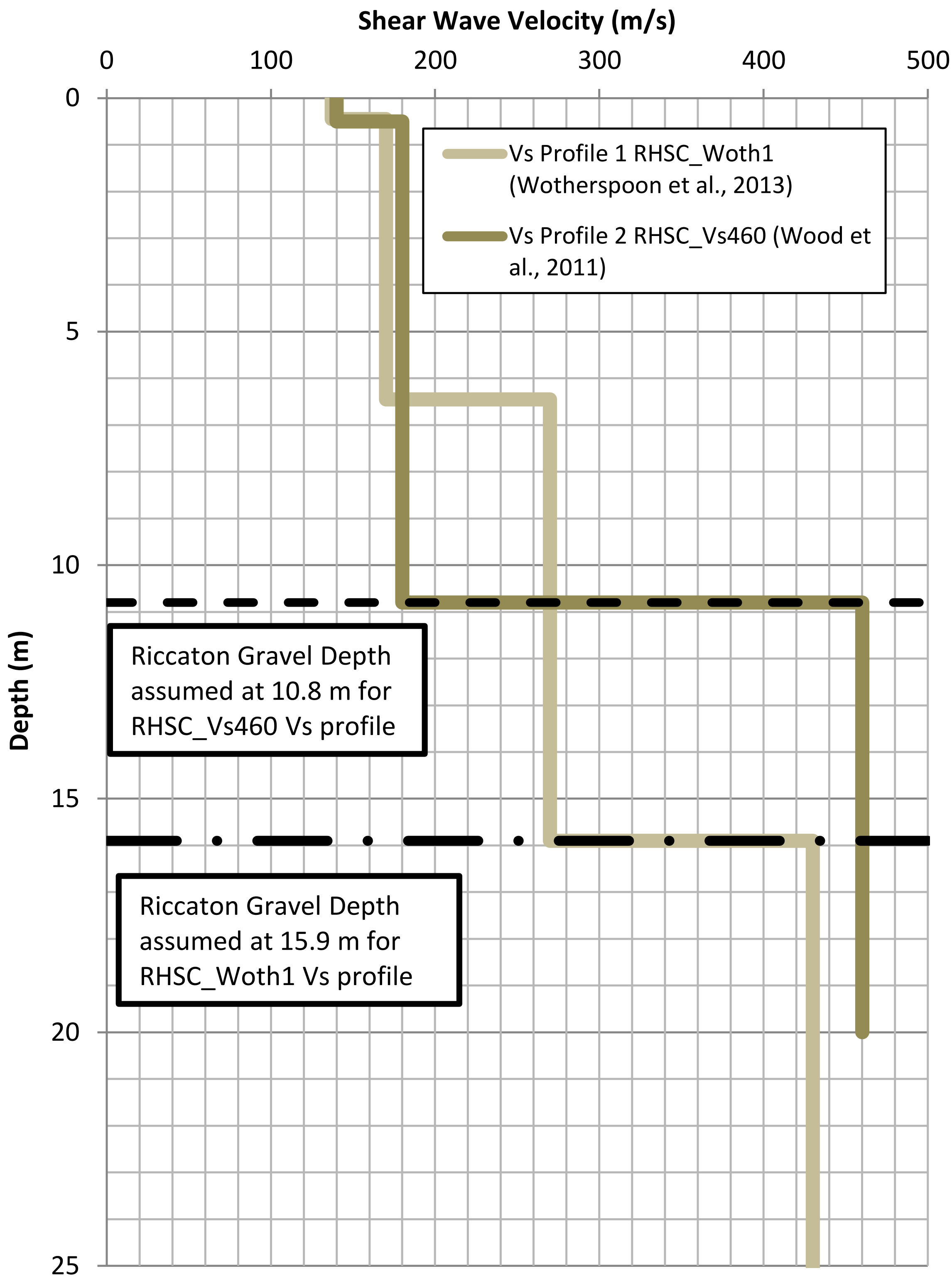


Figure B.2.11: Shear Wave velocity profile for RHSC strong ground motion station.

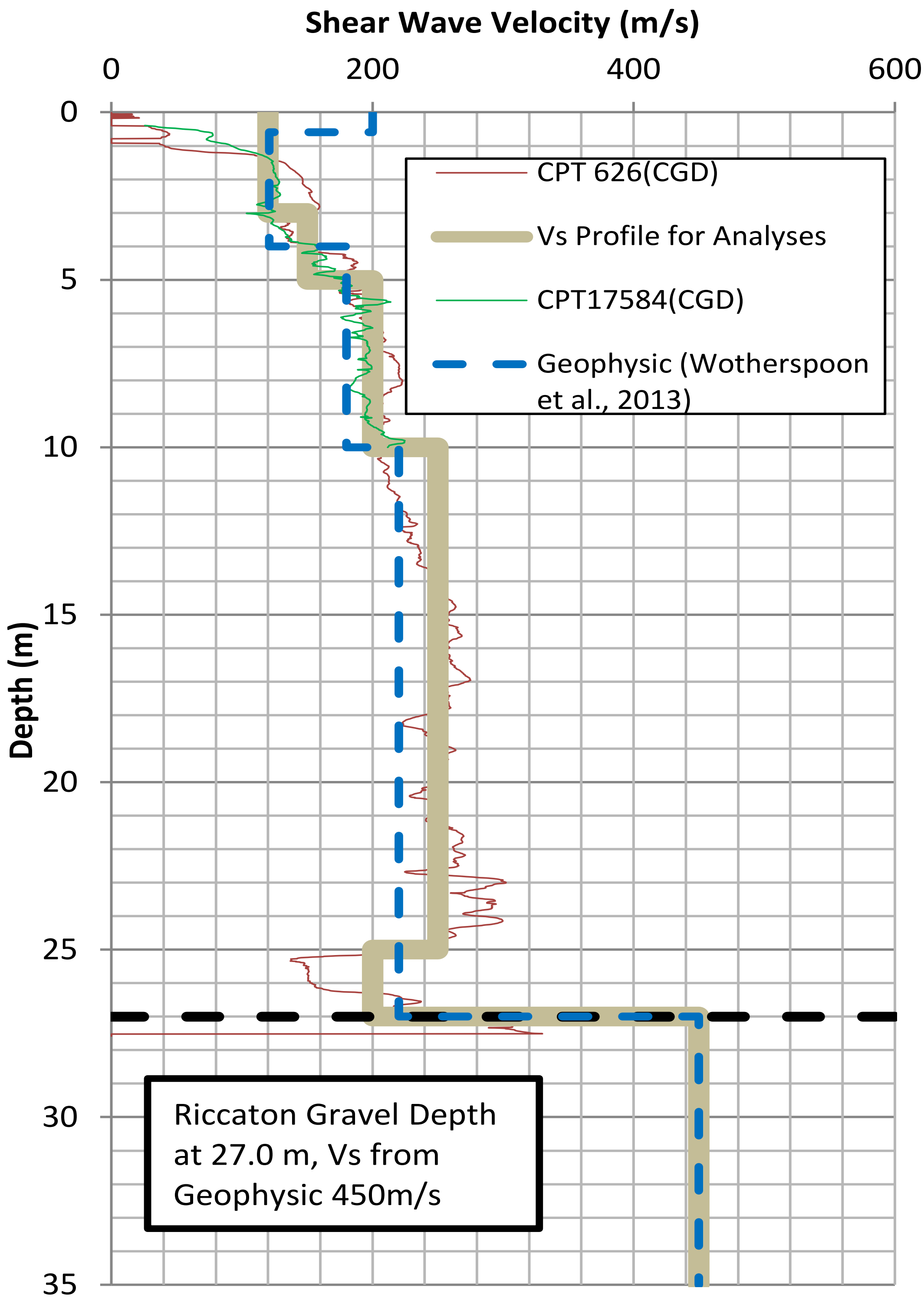


Figure B.2.12: Shear Wave velocity profile for SHLC strong ground motion station.

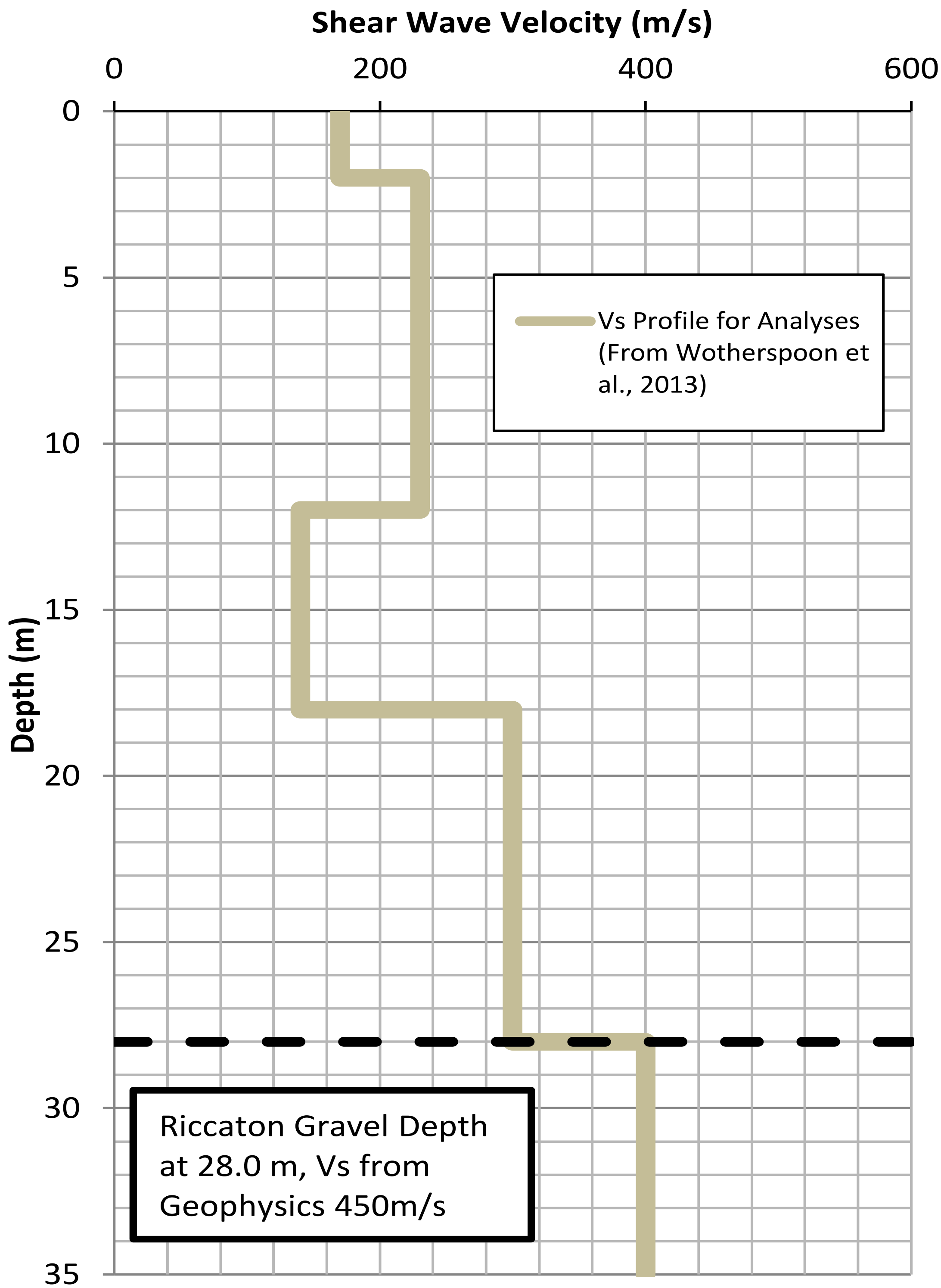


Figure B.2.13: Shear Wave velocity profile for SMTC strong ground motion station.

APPENDIX C

Parameters for Seismic Site Response Analyses.

Table C.1 Parameters for Seismic Site Response analyses CBGS SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	0.42	17.3	81	2.220	0.017	0.18	1.47	0.84	0.00	0.00	0.99	1	1	1	3.000	1.000	0.005	3.8		9.30E-03	0.76	0.32	1.15
2	0.42	17.3	81	1.513	0.031	0.18	1.55	0.74	0.00	0.00	0.99	1	1	1	3.000	1.000	0.007	3.8		9.30E-03	0.79	0.24	1.55
3	0.65	17.3	183	1.409	0.028	0.18	1.46	0.90	0.00	0.00	0.99	1	1	1	1.186	1.000	0.009	3.8		9.30E-03	0.59	0.20	3.25
4	1.00	17.3	183	1.169	0.042	0.18	1.55	0.86	0.00	0.00	0.99	1	1	1	1.186	1.000	0.012	3.8		9.30E-03	0.76	0.33	1.00
5	1.50	19.6	183	1.038	0.045	0.18	1.50	0.83	0.00	0.00	0.99	1	1	1	1.186	1.000	0.013	3.8		9.30E-03	0.68	0.22	1.60
6	1.50	19.6	183	0.933	0.054	0.18	1.53	0.80	0.00	0.00	0.99	1	1	1	1.186	1.000	0.014	3.8		9.30E-03	0.79	0.29	0.85
7	1.50	19.6	183	0.886	0.061	0.18	1.55	0.78	0.00	0.00	0.99	1	1	1	1.186	1.000	0.016	3.8		9.30E-03	0.71	0.20	1.35
8	1.50	19.6	183	0.806	0.063	0.18	1.53	0.75	0.00	0.00	0.99	1	1	1	1.186	1.000	0.016	3.8		9.30E-03	0.78	0.24	1.45
9	0.50	19.6	183	0.769	0.054	0.18	1.31	0.74	0.00	0.00	0.99	1	1	1	1.186	1.000	0.016	3.8		9.30E-03	0.80	0.24	1.35
10	1.50	19.6	152	0.730	0.072	0.18	1.52	0.72	0.00	0.00	0.99	1	1	1	1.582	1.000	0.017	3.8		9.30E-03	0.85	0.28	1.05
11	1.50	19.6	152	0.694	0.055	0.18	1.20	0.71	0.00	0.00	0.99	1	1	1	1.582	1.673	0.018	3.8		2.79E-06	0.83	0.25	2.10
12	1.50	19.6	152	0.674	0.080	0.18	1.50	0.71	0.00	0.00	0.99	1	1	1	1.582	1.673	0.019	3.8		2.79E-06	0.84	0.25	2.10
13	1.50	19.6	152	0.637	0.079	0.18	1.44	0.69	0.00	0.00	0.99	1	1	1	1.582	1.673	0.019	3.8		2.79E-06	0.85	0.25	2.00
14	1.50	19.6	152	0.598	0.087	0.18	1.46	0.68	0.00	0.00	0.99	1	1	1	1.582	1.673	0.020	3.8		2.79E-06	0.91	0.29	2.34
15	1.50	19.6	152	0.583	0.089	0.18	1.43	0.68	0.00	0.00	0.99	1	1	1	1.582	1.673	0.021	3.8		2.79E-06	0.92	0.30	2.21
16	1.50	19.6	152	0.552	0.104	0.18	1.53	0.66	0.00	0.00	0.99	1	1	1	1.582	1.673	0.021	3.8		2.79E-06	0.94	0.30	2.27
17	1.00	19.6	152	0.520	0.095	0.18	1.37	0.65	0.00	0.00	0.99	1	1	1	1.582	1.673	0.022	3.8		2.79E-06	0.99	0.33	2.70

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). “DeepSoil V5.1, User Manual and Tutorial 2002-2012.” Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). “Recent advances in non-linear site response analysis.” In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.2 Parameters for Shear Strength Correction CBGS Station.

For Use in Strength Correction							
Layer No.	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	OCR	PI	σ' (atm)	τ (kPa)
1	0.42	17.3	81	1	0	0.0	3.7
2	0.42	17.3	81	1	0	0.1	11.0
3	0.65	17.3	183	1	0	0.2	20.3
4	1.00	17.3	183	1	0	0.3	34.6
5	1.50	19.6	183	1	0	0.5	50.6
6	1.50	19.6	183	1	0	0.6	65.3
7	1.50	19.6	183	1	0	0.8	80.1
8	1.50	19.6	183	1	0	0.9	94.8
9	0.50	19.6	183	1	0	1.0	104.7
10	1.50	19.6	152	1	0	1.1	89.5
11	1.50	19.6	152	1	0	1.3	97.4
12	1.50	19.6	152	1	0	1.4	100.8
13	1.50	19.6	152	1	0	1.6	111.2
14	1.50	19.6	152	1	0	1.7	121.5
15	1.50	19.6	152	1	0	1.9	131.8
16	1.50	19.6	152	1	0	2.0	142.1
17	1.00	19.6	152	1	0	2.1	156.4

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.3 Parameters for Seismic Site Response Analyses CCCC SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	0.85	17.3	100	1.850	0.020	0.18	1.38	0.84	0.00	0.00	0.99	1	1	1	3.000	1.379	0.007	3.8		4.65E-03	0.70	0.26	1.60
2	0.65	17.3	100	1.273	0.032	0.18	1.38	0.77	0.00	0.00	0.99	1	1	1	3.000	1.504	0.009	3.8		4.65E-03	0.78	0.26	1.20
3	1.00	19.6	130	1.175	0.040	0.18	1.56	0.83	0.00	0.00	0.99	1	1	1	2.015	1.592	0.011	3.8		4.65E-03	0.72	0.26	1.35
4	1.00	19.6	130	1.063	0.046	0.18	1.52	0.78	0.00	0.00	0.99	1	1	1	2.015	1.636	0.011	3.8		4.65E-03	0.71	0.21	1.45
5	1.00	19.6	130	0.979	0.051	0.18	1.49	0.77	0.00	0.00	0.99	1	1	1	2.015	1.464	0.012	3.8		3.72E-02	0.81	0.29	0.80
6	1.00	19.6	130	0.896	0.056	0.18	1.55	0.74	0.00	0.00	0.99	1	1	1	2.015	1.464	0.013	3.8		3.72E-02	0.81	0.26	1.45
7	1.00	19.6	170	0.919	0.054	0.18	1.52	0.81	0.00	0.00	0.99	1	1	1	1.330	1.350	0.014	3.8		9.30E-04	0.77	0.30	0.90
8	1.00	19.6	170	0.869	0.058	0.18	1.52	0.80	0.00	0.00	0.99	1	1	1	1.330	1.350	0.016	3.8		3.72E-01	0.85	0.36	0.60
9	1.00	19.6	170	0.830	0.064	0.18	1.53	0.78	0.00	0.00	0.99	1	1	1	1.330	1.350	0.017	3.8		3.72E-02	0.87	0.36	0.55
10	1.50	19.6	170	0.802	0.060	0.18	1.40	0.78	0.00	0.00	0.99	1	1	1	1.330	1.350	0.018	3.8		9.30E-03	0.87	0.36	0.55
11	2.00	19.6	220	0.796	0.067	0.18	1.56	0.81	0.00	0.00	0.99	1	1	1	0.892	1.252	0.018	3.8		9.30E-01	0.72	0.24	1.05
12	2.00	19.6	220	0.749	0.072	0.18	1.53	0.80	0.00	0.00	0.99	1	1	1	0.892	1.252	0.019	3.8		9.30E-01	0.78	0.28	0.80
13	2.00	19.6	220	0.731	0.075	0.18	1.53	0.81	0.00	0.00	0.99	1	1	1	0.892	1.252	0.021	3.8		9.30E-04	0.76	0.28	0.85
14	2.00	19.6	220	0.693	0.074	0.18	1.53	0.78	0.00	0.00	0.99	1	1	1	0.892	1.415	0.020	3.8		2.79E-03	0.71	0.20	1.60
15	2.00	19.6	220	0.662	0.078	0.18	1.50	0.77	0.00	0.00	0.99	1	1	1	0.892	1.415	0.021	3.8		2.79E-04	0.73	0.20	1.45
16	1.50	19.6	160	0.541	0.106	0.18	1.53	0.66	0.00	0.00	0.99	1	1	1	0.551	1.782	0.022	3.8		2.79E-06	0.93	0.29	2.29
17	1.50	19.6	160	0.527	0.077	0.18	1.20	0.66	0.00	0.00	0.99	1	1	1	0.551	1.782	0.023	3.8		2.79E-06	0.96	0.32	2.50
18	1.00	19.6	160	0.501	0.119	0.18	1.53	0.65	0.00	0.00	0.99	1	1	1	0.551	1.782	0.024	3.8		2.79E-06	0.98	0.33	2.65
19	1.00	19.6	160	0.495	0.123	0.18	1.55	0.65	0.00	0.00	0.99	1	1	1	0.551	1.782	0.024	3.8		2.79E-06	0.99	0.33	2.59

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). "DeepSoil V5.1, User Manual and Tutorial 2002-2012." Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). "Recent advances in non-linear site response analysis." In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.4 Parameters for Shear Strength Correction CCCC Station.

For Use in Strength Correction							
Layer No.	**Thickness (m)**	**γ (Kn/m^3)**	**Vs (m/s)**	**OCR**	**PI**	**σ' (atm)**	**τ (kPa)**
1	0.85	17.3	100	1	0	0.1	6.6
2	0.65	17.3	100	1	0	0.2	15.3
3	1.00	19.6	130	1	0	0.3	23.5
4	1.00	19.6	130	1	0	0.4	32.4
5	1.00	19.6	130	1	0	0.5	40.7
6	1.00	19.6	130	1	0	0.6	48.3
7	1.00	19.6	170	1	0	0.7	56.0
8	1.00	19.6	170	1	0	0.8	63.7
9	1.00	19.6	170	1	0	0.9	71.4
10	1.50	19.6	170	1	0	1.0	75.3
11	2.00	19.6	220	1	0	1.2	105.1
12	2.00	19.6	220	1	0	1.4	122.2
13	2.00	19.6	220	1	0	1.6	122.9
14	2.00	19.6	220	1	0	1.8	138.0
15	2.00	19.6	220	1	0	2.0	153.1
16	1.50	19.6	160	1	0	2.1	157.5
17	1.50	19.6	160	1	0	2.3	168.2
18	1.00	19.6	160	1	0	2.4	177.1
19	1.00	19.6	160	1	0	2.5	184.3

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.5 Parameters for Seismic Site Response Analyses CHHC SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	0.75	17.3	100	1.946	0.019	0.18	1.44	0.89	0.00	0.00	0.99	1	1	1	3.000	1.537	0.006	3.8		1.86E-04	0.63	0.23	2.50
2	0.75	17.3	100	1.324	0.036	0.18	1.58	0.78	0.00	0.00	0.99	1	1	1	3.000	1.537	0.009	3.8		1.86E-04	0.76	0.26	1.30
3	0.50	17.3	100	1.096	0.043	0.18	1.46	0.74	0.00	0.00	0.99	1	1	1	3.000	1.537	0.011	3.8		1.86E-04	0.93	0.38	0.60
4	0.50	17.3	100	0.998	0.048	0.18	1.49	0.71	0.00	0.00	0.99	1	1	1	3.000	1.537	0.011	3.8		1.86E-04	0.83	0.25	1.96
5	0.50	17.3	100	0.837	0.057	0.18	1.46	0.68	0.00	0.00	0.99	1	1	1	3.000	1.317	0.013	3.8		4.65E-03	0.90	0.28	2.51
6	1.00	19.6	160	0.978	0.050	0.18	1.53	0.81	0.00	0.00	0.99	1	1	1	1.461	1.317	0.014	3.8		9.30E-03	0.77	0.30	0.90
7	1.50	19.6	160	0.920	0.058	0.18	1.55	0.78	0.00	0.00	0.99	1	1	1	1.461	1.317	0.015	3.8		9.30E-03	0.71	0.20	1.40
8	1.50	19.6	160	0.869	0.060	0.18	1.49	0.78	0.00	0.00	0.99	1	1	1	1.461	1.317	0.016	3.8		9.30E-03	0.73	0.22	1.15
9	1.50	19.6	160	0.780	0.064	0.18	1.49	0.74	0.00	0.00	0.99	1	1	1	1.461	1.317	0.016	3.8		9.30E-02	0.81	0.25	1.30
10	1.50	19.6	160	0.734	0.059	0.18	1.32	0.72	0.00	0.00	0.99	1	1	1	1.461	1.317	0.017	3.8		9.30E-02	0.83	0.25	1.20
11	2.00	19.6	210	0.758	0.071	0.18	1.53	0.80	0.00	0.00	0.99	1	1	1	0.958	1.317	0.019	3.8		9.30E-02	0.84	0.34	0.60
12	2.00	19.6	210	0.727	0.062	0.18	1.37	0.78	0.00	0.00	0.99	1	1	1	0.958	1.317	0.019	3.8		9.30E-02	0.73	0.22	1.60
13	1.00	19.6	210	0.697	0.074	0.18	1.50	0.77	0.00	0.00	0.99	1	1	1	0.958	1.317	0.019	3.8		9.30E-02	0.75	0.23	1.40
14	2.00	19.6	250	0.710	0.080	0.18	1.56	0.81	0.00	0.00	0.99	1	1	1	0.731	1.317	0.021	3.8		9.30E-02	0.72	0.23	1.15
15	1.45	19.6	145	0.519	0.118	0.18	1.55	0.63	0.00	0.00	0.99	1	1	1	1.393	1.464	0.022	3.8		9.30E-03	0.99	0.30	2.60
16	1.05	19.6	145	0.511	0.121	0.18	1.53	0.63	0.00	0.00	0.99	1	1	1	1.393	1.537	0.023	3.8		9.30E-03	0.99	0.30	2.55
17	1.05	19.6	165	0.536	0.112	0.18	1.56	0.66	0.00	0.00	0.99	1	1	1	1.393	1.673	0.022	3.8		9.30E-06	0.94	0.29	2.26
18	1.05	19.6	165	0.530	0.097	0.18	1.40	0.66	0.00	0.00	0.99	1	1	1	1.393	1.734	0.023	3.8		9.30E-06	0.94	0.30	2.25

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). "DeepSoil V5.1, User Manual and Tutorial 2002-2012." Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). "Recent advances in non-linear site response analysis." In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.6 Parameters for Shear Strength Correction CHHC Station.

For Use in Strength Correction							
Layer No.	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	OCR	PI	σ' (atm)	τ (kPa)
1	0.75	17.3	100	1	0	0.1	4.7
2	0.75	17.3	100	1	0	0.2	14.1
3	0.50	17.3	100	1	0	0.3	22.0
4	0.50	17.3	100	1	0	0.4	28.2
5	0.50	17.3	100	1	0	0.5	36.9
6	1.00	19.6	160	1	0	0.6	46.9
7	1.50	19.6	160	1	0	0.7	56.9
8	1.50	19.6	160	1	0	0.8	64.0
9	1.50	19.6	160	1	0	1.0	83.7
10	1.50	19.6	160	1	0	1.1	96.1
11	2.00	19.6	210	1	0	1.3	114.5
12	2.00	19.6	210	1	0	1.5	118.3
13	1.00	19.6	210	1	0	1.6	134.5
14	2.00	19.6	250	1	0	1.8	151.8
15	1.45	19.6	165	1	0	2.0	154.6
16	1.45	19.6	165	1	0	2.1	165.7
17	1.05	19.6	165	1	0	2.2	163.0
18	1.05	19.6	165	1	0	2.3	170.5

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.7 Parameters for Seismic Site Response Analyses HPSC SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	1.00	17.3	115	1.811	0.025	0.18	1.64	0.90	0.00	0.00	0.99	1	1	1	2.437	1.705	0.007	3.8		9.30E-05	0.60	0.22	3.25
2	1.00	17.3	115	1.206	0.039	0.18	1.52	0.78	0.00	0.00	0.99	1	1	1	2.437	1.464	0.010	3.8		9.30E-05	0.74	0.24	1.30
3	0.50	19.6	115	1.018	0.050	0.18	1.52	0.75	0.00	0.00	0.99	1	1	1	2.437	1.464	0.013	3.8		9.30E-05	0.89	0.35	0.60
4	0.50	19.6	115	0.968	0.050	0.18	1.49	0.72	0.00	0.00	0.99	1	1	1	2.280	1.464	0.012	3.8		9.30E-05	0.79	0.23	2.20
5	1.00	19.6	130	0.930	0.054	0.18	1.55	0.74	0.00	0.00	0.99	1	1	1	2.015	1.252	0.013	3.8		9.30E-02	0.83	0.27	1.45
6	1.00	19.6	130	0.868	0.058	0.18	1.52	0.72	0.00	0.00	0.99	1	1	1	2.015	1.252	0.014	3.8		9.30E-02	0.85	0.28	1.30
7	1.50	19.6	170	0.884	0.057	0.18	1.52	0.80	0.00	0.00	0.99	1	1	1	1.330	1.252	0.016	3.8		9.30E-02	0.87	0.38	0.60
8	1.50	19.6	170	0.829	0.059	0.18	1.50	0.77	0.00	0.00	0.99	1	1	1	1.330	1.252	0.016	3.8		9.30E-02	0.77	0.25	1.50
9	1.50	19.6	170	0.796	0.069	0.18	1.56	0.78	0.00	0.00	0.99	1	1	1	1.330	1.317	0.018	3.8		9.30E-02	0.87	0.36	0.55
10	0.50	19.6	170	0.747	0.070	0.18	1.52	0.74	0.00	0.00	0.99	1	1	1	1.330	1.317	0.017	3.8		9.30E-02	0.81	0.25	1.30
11	2.00	19.6	220	0.784	0.058	0.18	1.35	0.81	0.00	0.00	0.99	1	1	1	0.892	1.317	0.019	3.8		9.30E-02	0.72	0.24	1.10
12	2.00	19.6	220	0.741	0.076	0.18	1.55	0.80	0.00	0.00	0.99	1	1	1	0.892	1.317	0.020	3.8		9.30E-02	0.77	0.27	0.85
13	2.00	19.6	220	0.714	0.076	0.18	1.50	0.80	0.00	0.00	0.99	1	1	1	0.892	1.317	0.021	3.8		9.30E-02	0.77	0.27	0.83
14	2.00	19.6	220	0.675	0.079	0.18	1.55	0.77	0.00	0.00	0.99	1	1	1	0.892	1.317	0.020	3.8		9.30E-02	0.73	0.20	1.50
15	2.50	19.6	280	0.686	0.074	0.18	1.49	0.84	0.00	0.00	0.99	1	1	1	0.614	1.317	0.022	3.8		9.30E-02	0.76	0.31	0.85
16	2.50	19.6	280	0.650	0.069	0.18	1.32	0.83	0.00	0.00	0.99	1	1	1	0.614	1.317	0.023	3.8		9.30E-02	0.87	0.40	0.55
17	2.00	19.6	280	0.635	0.080	0.18	1.52	0.81	0.00	0.00	0.99	1	1	1	0.614	1.317	0.022	3.8		9.30E-02	0.73	0.25	1.21
18	3.00	19.6	315	0.633	0.073	0.18	1.32	0.84	0.00	0.00	0.99	1	1	1	0.511	1.317	0.025	3.8		9.30E-02	0.73	0.27	1.05
19	3.00	19.6	315	0.603	0.089	0.18	1.49	0.83	0.00	0.00	0.99	1	1	1	0.511	1.317	0.026	3.8		9.30E-02	0.82	0.35	0.70
20	3.00	19.6	315	0.599	0.071	0.18	1.23	0.84	0.00	0.00	0.99	1	1	1	0.511	1.317	0.027	3.8		9.30E-02	0.73	0.28	1.00
21	2.00	19.6	315	0.574	0.093	0.18	1.55	0.81	0.00	0.00	0.99	1	1	1	0.511	1.317	0.025	3.8		9.30E-02	0.71	0.23	1.15

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). "DeepSoil V5.1, User Manual and Tutorial 2002-2012." Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). "Recent advances in non-linear site response analysis." In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.8 Parameters for Shear Strength Correction HPSC Station.

For Use in Strength Correction							
Layer No.	**Thickness (m)**	**γ (Kn/m^3)**	**Vs (m/s)**	**OCR**	**PI**	**σ' (atm)**	**τ (kPa)**
1	1.00	17.3	115	1	0	0.1	6.0
2	1.00	17.3	115	1	0	0.3	20.4
3	0.50	19.6	115	1	0	0.4	33.8
4	0.50	19.6	115	1	0	0.5	37.8
5	1.00	19.6	130	1	0	0.5	45.3
6	1.00	19.6	130	1	0	0.6	53.6
7	1.50	19.6	170	1	0	0.8	63.9
8	1.50	19.6	170	1	0	0.9	76.3
9	1.50	19.6	170	1	0	1.0	74.0
10	0.50	19.6	170	1	0	1.1	96.9
11	2.00	19.6	220	1	0	1.3	107.2
12	2.00	19.6	220	1	0	1.5	123.7
13	2.00	19.6	220	1	0	1.6	125.9
14	2.00	19.6	220	1	0	1.8	145.9
15	2.50	19.6	280	1	0	2.1	175.3
16	2.50	19.6	280	1	0	2.3	195.9
17	2.00	19.6	280	1	0	2.5	214.5
18	3.00	19.6	315	1	0	2.8	235.1
19	3.00	19.6	315	1	0	3.1	259.9
20	3.00	19.6	315	1	0	3.3	255.6
21	2.00	19.6	315	1	0	3.6	294.6

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.9 Parameters for Seismic Site Response Analyses KPOC SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	1.00	17.3	150	1.752	0.022	0.18	1.53	0.95	0.00	0.00	0.99	1	1	1	1.614	1.537	0.007	3.8		9.30E-03	0.62	0.27	3.25
2	1.50	19.6	150	1.280	0.034	0.18	1.46	0.87	0.00	0.00	0.99	1	1	1	1.614	1.251	0.011	3.8		9.30E-03	0.64	0.22	2.10
3	1.00	19.6	150	1.113	0.041	0.18	1.49	0.83	0.00	0.00	0.99	1	1	1	1.614	1.251	0.012	3.8		9.30E-03	0.71	0.25	1.36
4	1.50	19.6	150	1.004	0.050	0.18	1.50	0.78	0.00	0.00	0.99	1	1	1	1.614	1.251	0.013	3.8		9.30E-03	0.73	0.22	1.30
5	1.50	19.6	150	0.892	0.055	0.18	1.52	0.75	0.00	0.00	0.99	1	1	1	1.614	1.251	0.014	3.8		9.30E-03	0.80	0.26	1.45
6	2.00	19.6	210	0.902	0.053	0.18	1.47	0.83	0.00	0.00	0.99	1	1	1	0.958	1.251	0.016	3.8		9.30E-03	0.66	0.19	1.85
7	2.00	19.6	210	0.832	0.062	0.18	1.52	0.81	0.00	0.00	0.99	1	1	1	0.958	1.251	0.017	3.8		9.30E-03	0.74	0.26	1.05
8	2.50	19.6	300	0.827	0.056	0.18	1.50	0.89	0.00	0.00	0.99	1	1	1	0.551	1.251	0.018	3.8		9.30E-03	0.59	0.18	3.10
9	2.50	19.6	300	0.780	0.058	0.18	1.40	0.87	0.00	0.00	0.99	1	1	1	0.551	1.251	0.019	3.8		9.30E-03	0.61	0.19	2.25
10	3.00	19.6	300	0.732	0.070	0.18	1.52	0.86	0.00	0.00	0.99	1	1	1	0.551	1.251	0.021	3.8		9.30E-03	0.68	0.24	1.32

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). "DeepSoil V5.1, User Manual and Tutorial 2002-2012." Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). "Recent advances in non-linear site response analysis." In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.10 Parameters for Shear Strength Correction KPOC Station.

For Use in Strength Correction							
Layer No.	**Thickness (m)**	**γ (Kn/m^3)**	**Vs (m/s)**	**OCR**	**PI**	**σ' (atm)**	**τ (kPa)**
1	1.00	17.3	150	1	0	0.1	7.7
2	1.50	19.6	150	1	0	0.3	21.1
3	1.00	19.6	150	1	0	0.4	33.0
4	1.50	19.6	150	1	0	0.5	46.5
5	1.50	19.6	150	1	0	0.7	59.7
6	2.00	19.6	210	1	0	0.8	75.2
7	2.00	19.6	210	1	0	1.0	92.9
8	2.50	19.6	300	1	0	1.2	125.3
9	2.50	19.6	300	1	0	1.5	149.9
10	3.00	19.6	300	1	0	1.7	177.0

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.11 Parameters for Seismic Site Response Analyses NNBS SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	1.00	17.3	100	1.772	0.018	0.18	1.19	0.86	0.00	0.00	0.99	1	1	1	3.000	1.537	0.007	3.8		9.30E-03	0.63	0.21	2.30
2	0.50	17.3	100	1.195	0.034	0.18	1.37	0.74	0.00	0.00	0.99	1	1	1	3.000	1.251	0.010	3.8		9.30E-02	0.94	0.39	0.60
3	0.50	19.6	100	1.059	0.045	0.18	1.50	0.71	0.00	0.00	0.99	1	1	1	3.000	1.251	0.011	3.8		9.30E-02	0.84	0.25	2.10
4	1.00	19.6	140	1.098	0.045	0.18	1.55	0.80	0.00	0.00	0.99	1	1	1	1.796	1.251	0.012	3.8		9.30E-02	0.71	0.22	1.45
5	1.00	19.6	140	0.999	0.053	0.18	1.55	0.77	0.00	0.00	0.99	1	1	1	1.796	1.251	0.013	3.8		9.30E-02	0.79	0.26	0.95
6	1.00	19.6	140	0.906	0.046	0.18	1.34	0.74	0.00	0.00	0.99	1	1	1	1.796	1.251	0.013	3.8		9.30E-02	0.83	0.28	1.40
7	1.75	19.6	180	0.917	0.055	0.18	1.53	0.81	0.00	0.00	0.99	1	1	1	1.217	1.251	0.015	3.8		9.30E-02	0.77	0.30	0.90
8	1.50	19.6	180	0.837	0.063	0.18	1.55	0.78	0.00	0.00	0.99	1	1	1	1.217	1.251	0.016	3.8		9.30E-02	0.87	0.36	0.55
9	1.75	19.6	180	0.795	0.062	0.18	1.52	0.77	0.00	0.00	0.99	1	1	1	1.217	1.251	0.016	3.8		9.30E-02	0.76	0.24	1.43
10	2.25	19.6	240	0.799	0.055	0.18	1.35	0.84	0.00	0.00	0.99	1	1	1	0.779	1.350	0.018	3.8		9.30E-02	0.75	0.29	1.00
11	2.25	19.6	240	0.767	0.063	0.18	1.41	0.83	0.00	0.00	0.99	1	1	1	0.779	1.350	0.020	3.8		9.30E-02	0.67	0.20	1.60
12	2.25	19.6	240	0.723	0.077	0.18	1.55	0.81	0.00	0.00	0.99	1	1	1	0.779	1.350	0.021	3.8		9.30E-02	0.74	0.25	1.00
13	2.25	19.6	240	0.667	0.077	0.18	1.55	0.78	0.00	0.00	0.99	1	1	1	0.779	1.350	0.020	3.8		9.30E-02	0.81	0.30	0.85
14	2.00	19.6	240	0.646	0.080	0.18	1.52	0.78	0.00	0.00	0.99	1	1	1	0.779	1.350	0.021	3.8		9.30E-02	0.83	0.32	0.75
15	2.50	19.6	300	0.669	0.078	0.18	1.50	0.84	0.00	0.00	0.99	1	1	1	0.551	1.350	0.023	3.8		9.30E-02	0.71	0.26	1.15
16	1.50	19.6	300	0.651	0.079	0.18	1.47	0.84	0.00	0.00	0.99	1	1	1	0.551	1.350	0.024	3.8		9.30E-02	0.74	0.28	0.95
17	1.00	19.6	300	0.643	0.081	0.18	1.47	0.84	0.00	0.00	0.99	1	1	1	0.551	1.655	0.024	3.8		2.79E-06	0.73	0.27	1.05
18	1.00	19.6	300	0.620	0.078	0.18	1.37	0.83	0.00	0.00	0.99	1	1	1	0.551	1.379	0.025	3.8		9.30E-02	0.87	0.39	0.60
19	3.00	19.6	320	0.624	0.074	0.18	1.32	0.84	0.00	0.00	0.99	1	1	1	0.499	1.379	0.026	3.8		9.30E-02	0.73	0.27	1.05
20	3.00	19.6	320	0.595	0.090	0.18	1.56	0.81	0.00	0.00	0.99	1	1	1	0.499	1.379	0.024	3.8		9.30E-02	0.70	0.21	1.45
21	2.00	19.6	320	0.583	0.091	0.18	1.55	0.81	0.00	0.00	0.99	1	1	1	0.499	1.379	0.025	3.8		9.30E-02	0.69	0.21	1.35
22	1.50	19.6	200	0.475	0.086	0.18	1.17	0.68	0.00	0.00	0.99	1	1	1	1.034	1.734	0.027	3.8		9.30E-07	0.93	0.31	2.20
23	1.50	19.6	200	0.470	0.118	0.18	1.44	0.68	0.00	0.00	0.99	1	1	1	1.034	1.734	0.028	3.8		9.30E-07	0.93	0.31	2.23
24	1.00	19.6	200	0.450	0.140	0.18	1.56	0.66	0.00	0.00	0.99	1	1	1	1.034	1.636	0.028	3.8		9.30E-07	0.94	0.30	2.36
25	2.00	19.6	200	0.445	0.141	0.18	1.55	0.66	0.00	0.00	0.99	1	1	1	1.034	1.734	0.029	3.8		9.30E-07	0.95	0.31	2.30

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). “DeepSoil V5.1, User Manual and Tutorial 2002-2012.” Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). “Recent advances in non-linear site response analysis.” In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.12 Parameters for Shear Strength Correction NNBS Station.

For Use in Strength Correction							
Layer No.	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	OCR	PI	σ' (atm)	τ (kPa)
1	1.00	17.3	100	1	0	0.1	6.5
2	0.50	17.3	100	1	0	0.2	20.0
3	0.50	19.6	100	1	0	0.3	29.8
4	1.00	19.6	140	1	0	0.4	36.0
5	1.00	19.6	140	1	0	0.5	45.0
6	1.00	19.6	140	1	0	0.6	54.0
7	1.75	19.6	180	1	0	0.7	63.0
8	1.50	19.6	180	1	0	0.9	76.9
9	1.75	19.6	180	1	0	1.0	90.8
10	2.25	19.6	240	1	0	1.2	104.1
11	2.25	19.6	240	1	0	1.4	122.7
12	2.25	19.6	240	1	0	1.7	141.3
13	2.25	19.6	240	1	0	1.9	159.8
14	2.00	19.6	240	1	0	2.1	177.4
15	2.50	19.6	300	1	0	2.3	195.9
16	1.50	19.6	300	1	0	2.5	212.4
17	1.00	19.6	300	1	0	2.6	207.4
18	1.00	19.6	300	1	0	2.7	231.0
19	3.00	19.6	320	1	0	2.9	247.5
20	3.00	19.6	320	1	0	3.2	272.3
21	2.00	19.6	320	1	0	3.4	292.9
22	1.50	19.6	200	1	0	3.6	266.1
23	1.50	19.6	200	1	0	3.8	276.8
24	1.00	19.6	200	1	0	3.9	285.8
25	2.00	19.6	200	1	0	4.0	296.5

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.13 Parameters for Seismic Site Response Analyses PPHS SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	1.00	17.3	100	1.749	0.022	0.18	1.41	0.86	0.00	0.00	0.99	1	1	1	3.000	1.251	0.007	3.8		9.30E-04	0.62	0.20	2.85
2	1.00	17.3	100	1.164	0.036	0.18	1.37	0.75	0.00	0.00	0.99	1	1	1	3.000	1.415	0.010	3.8		9.30E-04	0.88	0.34	0.75
3	0.50	17.3	100	0.994	0.050	0.18	1.52	0.71	0.00	0.00	0.99	1	1	1	3.000	1.415	0.012	3.8		9.30E-04	0.83	0.25	1.95
4	1.00	19.6	100	0.912	0.052	0.18	1.46	0.69	0.00	0.00	0.99	1	1	1	3.000	1.415	0.012	3.8		9.30E-04	0.84	0.23	2.00
5	1.00	19.6	100	0.811	0.002	0.18	0.17	0.66	0.00	0.00	0.99	1	1	1	3.000	1.415	0.013	3.8		9.30E-04	0.92	0.28	2.63
6	0.50	19.6	100	0.779	0.061	0.18	1.41	0.66	0.00	0.00	0.99	1	1	1	3.000	1.705	0.014	3.8		2.79E-06	0.94	0.30	2.35
7	1.00	19.6	100	0.725	0.074	0.18	1.53	0.65	0.00	0.00	0.99	1	1	1	1.000	1.705	0.015	3.8		2.79E-06	0.98	0.33	2.95
8	1.00	19.6	100	0.698	0.068	0.18	1.38	0.65	0.00	0.00	0.99	1	1	1	1.000	1.705	0.016	3.8		2.79E-06	0.99	0.34	2.80
9	1.00	19.6	110	0.698	0.070	0.18	1.41	0.66	0.00	0.00	0.99	1	1	1	1.000	1.705	0.016	3.8		2.79E-06	0.96	0.32	2.60
10	0.50	19.6	110	0.659	0.060	0.18	1.22	0.65	0.00	0.00	0.99	1	1	1	1.000	1.705	0.017	3.8		2.79E-06	0.99	0.33	2.68
11	1.00	19.6	110	0.625	0.089	0.18	1.52	0.63	0.00	0.00	0.99	1	1	1	1.000	1.705	0.017	3.8		2.79E-06	0.99	0.32	3.05
12	1.50	19.6	200	0.752	0.066	0.18	1.47	0.75	0.00	0.00	0.99	1	1	1	1.034	1.000	0.017	3.8		4.65E-02	0.78	0.24	1.39
13	1.50	19.6	200	0.715	0.063	0.18	1.34	0.74	0.00	0.00	0.99	1	1	1	1.034	1.000	0.018	3.8		4.65E-02	0.80	0.24	1.35
14	1.50	19.6	200	0.692	0.057	0.18	1.22	0.74	0.00	0.00	0.99	1	1	1	1.034	1.000	0.019	3.8		4.65E-02	0.80	0.24	1.15
15	2.00	19.6	200	0.647	0.080	0.18	1.49	0.72	0.00	0.00	0.99	1	1	1	1.034	1.000	0.020	3.8		4.65E-02	0.87	0.30	1.40
16	1.50	19.6	180	0.632	0.070	0.18	1.34	0.72	0.00	0.00	0.99	1	1	1	1.217	1.636	0.020	3.8		9.30E-07	0.84	0.27	1.50
17	1.50	19.6	180	0.617	0.083	0.18	1.47	0.72	0.00	0.00	0.99	1	1	1	1.217	1.636	0.021	3.8		9.30E-07	0.85	0.27	1.45

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). “DeepSoil V5.1, User Manual and Tutorial 2002-2012.” Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). “Recent advances in non-linear site response analysis.” In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.14 Parameters for Shear Strength Correction PPHS Station.

For Use in Strength Correction							
Layer No.	**Thickness (m)**	**γ (Kn/m^3)**	**Vs (m/s)**	**OCR**	**PI**	**σ' (atm)**	**τ (kPa)**
1	1.00	17.3	100	1	0	0.1	6.3
2	1.00	17.3	100	1	0	0.3	18.8
3	0.50	17.3	100	1	0	0.4	28.7
4	1.00	19.6	100	1	0	0.5	37.1
5	1.00	19.6	100	1	0	0.6	44.6
6	0.50	19.6	100	1	0	0.7	50.1
7	1.00	19.6	100	1	0	0.7	51.7
8	1.00	19.6	100	1	0	0.8	58.6
9	1.00	19.6	110	1	0	0.9	65.5
10	0.50	19.6	110	1	0	1.0	70.7
11	1.00	19.6	110	1	0	1.1	75.8
12	1.50	19.6	200	1	0	1.2	120.6
13	1.50	19.6	200	1	0	1.3	135.3
14	1.50	19.6	200	1	0	1.5	150.1
15	2.00	19.6	200	1	0	1.7	167.3
16	1.50	19.6	180	1	0	1.8	129.2
17	1.50	19.6	180	1	0	2.0	139.5

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.15 Parameters for Seismic Site Response Analyses PRPC SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	0.75	17.3	90	1.980	0.019	0.18	1.41	0.87	0.00	0.00	0.99	1	1	1	3.000	1.537	0.006	3.8		9.30E-05	0.61	0.20	3.00
2	0.75	17.3	90	1.272	0.037	0.18	1.47	0.72	0.00	0.00	0.99	1	1	1	3.000	1.317	0.009	3.8		4.65E-01	0.79	0.22	1.35
3	0.75	17.3	90	0.943	0.005	0.18	0.33	0.65	0.00	0.00	0.99	1	1	1	3.000	1.251	0.011	3.8		4.65E-01	0.96	0.31	3.06
4	0.75	17.3	90	0.842	0.004	0.18	0.26	0.65	0.00	0.00	0.99	1	1	1	3.000	1.636	0.012	3.8		1.86E-05	0.96	0.30	2.75
5	1.75	19.6	180	0.965	0.054	0.18	1.59	0.81	0.00	0.00	0.99	1	1	1	1.217	1.297	0.014	3.8		5.58E-01	0.74	0.26	1.10
6	1.75	19.6	180	0.875	0.043	0.18	1.20	0.80	0.00	0.00	0.99	1	1	1	1.217	1.297	0.016	3.8		5.58E-01	0.86	0.37	0.60
7	1.75	19.6	180	0.816	0.062	0.18	1.53	0.77	0.00	0.00	0.99	1	1	1	1.217	1.297	0.016	3.8		5.58E-01	0.76	0.24	1.51
8	1.75	19.6	180	0.762	0.058	0.18	1.35	0.75	0.00	0.00	0.99	1	1	1	1.217	1.297	0.017	3.8		5.58E-01	0.80	0.26	1.25
9	1.00	19.6	180	0.743	0.069	0.18	1.52	0.74	0.00	0.00	0.99	1	1	1	1.217	1.297	0.017	3.8		5.58E-01	0.80	0.24	1.30
10	2.50	19.6	250	0.777	0.056	0.18	1.35	0.84	0.00	0.00	0.99	1	1	1	0.731	1.297	0.019	3.8		5.58E-01	0.77	0.32	0.85
11	2.50	19.6	250	0.746	0.066	0.18	1.43	0.83	0.00	0.00	0.99	1	1	1	0.731	1.297	0.020	3.8		5.58E-01	0.69	0.22	1.30
12	2.50	19.6	250	0.707	0.080	0.18	1.55	0.81	0.00	0.00	0.99	1	1	1	0.731	1.297	0.022	3.8		5.58E-01	0.72	0.23	1.15
13	2.50	19.6	250	0.661	0.077	0.18	1.52	0.80	0.00	0.00	0.99	1	1	1	0.731	1.297	0.021	3.8		5.58E-01	0.78	0.29	0.95
14	1.00	19.6	180	0.568	0.096	0.18	1.49	0.69	0.00	0.00	0.99	1	1	1	0.524	1.690	0.022	3.8		9.30E-06	0.89	0.27	1.45
15	1.00	19.6	180	0.562	0.092	0.18	1.44	0.69	0.00	0.00	0.99	1	1	1	0.524	1.690	0.022	3.8		9.30E-06	0.88	0.27	2.20
16	3.00	19.6	310	0.650	0.072	0.18	1.35	0.84	0.00	0.00	0.99	1	1	1	0.524	1.297	0.024	3.8		9.30E-01	0.74	0.28	1.00
17	1.50	19.6	230	0.602	0.089	0.18	1.49	0.77	0.00	0.00	0.99	1	1	1	0.832	1.592	0.024	3.8		9.30E-03	0.73	0.21	1.35
18	1.50	19.6	230	0.584	0.095	0.18	1.52	0.75	0.00	0.00	0.99	1	1	1	0.832	1.592	0.024	3.8		9.30E-05	0.75	0.20	1.25

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). "DeepSoil V5.1, User Manual and Tutorial 2002-2012." Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). "Recent advances in non-linear site response analysis." In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.16 Parameters for Shear Strength Correction PRPC Station.

For Use in Strength Correction							
Layer No.	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	OCR	PI	σ' (atm)	τ (kPa)
1	0.75	17.3	90	1	0	0.1	4.2
2	0.75	17.3	90	1	0	0.2	17.5
3	0.75	17.3	90	1	0	0.3	30.2
4	0.75	17.3	90	1	0	0.5	33.6
5	1.75	19.6	180	1	0	0.6	58.0
6	1.75	19.6	180	1	0	0.8	71.5
7	1.75	19.6	180	1	0	1.0	87.0
8	1.75	19.6	180	1	0	1.1	102.5
9	1.00	19.6	180	1	0	1.2	105.8
10	2.50	19.6	250	1	0	1.3	121.3
11	2.50	19.6	250	1	0	1.6	143.5
12	2.50	19.6	250	1	0	1.8	154.3
13	2.50	19.6	250	1	0	2.1	174.9
14	1.00	19.6	180	1	0	2.2	164.0
15	1.00	19.6	180	1	0	2.3	171.1
16	3.00	19.6	310	1	0	2.5	221.8
17	1.50	19.6	230	1	0	2.7	194.2
18	1.50	19.6	230	1	0	2.9	204.5

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.17 Parameters for Seismic Site Response Analyses REHS SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	0.30	17.3	90	2.442	0.014	0.18	1.40	0.90	0.00	0.00	0.99	1	1	1	3.000	1.000	0.005	3.8		9.30E-03	0.61	0.23	3.25
2	0.55	17.3	90	1.613	0.029	0.18	1.56	0.81	0.00	0.00	0.99	1	1	1	3.000	1.251	0.008	3.8		9.30E-03	0.87	0.40	0.65
3	0.65	17.3	90	1.225	0.037	0.18	1.50	0.74	0.00	0.00	0.99	1	1	1	3.000	1.251	0.009	3.8		9.30E-03	0.95	0.39	0.60
4	0.50	17.3	90	1.035	0.045	0.18	1.46	0.68	0.00	0.00	0.99	1	1	1	3.000	1.251	0.103	3.8		9.30E-03	0.86	0.23	2.45
5	1.00	19.6	90	0.950	0.006	0.18	0.35	0.68	0.00	0.00	0.99	1	1	1	2.500	1.350	0.012	3.8		9.30E-03	0.86	0.23	2.30
6	1.00	19.6	90	0.850	0.005	0.18	0.29	0.66	0.00	0.00	0.99	1	1	1	1.000	1.673	0.013	3.8		4.65E-05	0.93	0.29	2.70
7	1.00	19.6	110	0.878	0.046	0.18	1.29	0.71	0.00	0.00	0.99	1	1	1	1.000	1.705	0.013	3.8		4.65E-05	0.81	0.23	1.90
8	1.10	19.6	110	0.809	0.062	0.18	1.47	0.69	0.00	0.00	0.99	1	1	1	1.000	1.673	0.014	3.8		4.65E-05	0.88	0.28	2.31
9	0.90	19.6	130	0.816	0.059	0.18	1.46	0.72	0.00	0.00	0.99	1	1	1	1.000	1.673	0.015	3.8		4.65E-05	0.83	0.25	1.35
10	1.00	19.6	130	0.767	0.055	0.18	1.31	0.71	0.00	0.00	0.99	1	1	1	2.015	1.592	0.016	3.8		4.65E-05	0.87	0.28	1.10
11	1.00	19.6	130	0.733	0.058	0.18	1.29	0.69	0.00	0.00	0.99	1	1	1	2.015	1.566	0.016	3.8		4.65E-05	0.88	0.27	2.15
12	0.70	19.6	220	0.843	0.058	0.18	1.52	0.86	0.00	0.00	0.99	1	1	1	0.892	1.350	0.017	3.8		9.30E-01	0.68	0.24	1.40
13	0.90	19.6	220	0.817	0.056	0.18	1.38	0.83	0.00	0.00	0.99	1	1	1	0.892	1.148	0.018	3.8		9.30E-01	0.69	0.23	1.30
14	0.90	19.6	220	0.788	0.070	0.18	1.56	0.81	0.00	0.00	0.99	1	1	1	0.892	1.148	0.019	3.8		9.30E-01	0.74	0.26	1.00
15	0.90	19.6	220	0.744	0.067	0.18	1.49	0.77	0.00	0.00	0.99	1	1	1	0.892	1.148	0.018	3.8		9.30E-01	0.73	0.21	1.60
16	0.90	19.6	220	0.728	0.060	0.18	1.34	0.77	0.00	0.00	0.99	1	1	1	0.892	1.148	0.018	3.8		9.30E-01	0.75	0.23	1.40
17	0.70	19.6	220	0.716	0.073	0.18	1.53	0.77	0.00	0.00	0.99	1	1	1	0.892	1.148	0.019	3.8		9.30E-01	0.76	0.23	1.30
18	2.50	19.6	280	0.732	0.068	0.18	1.49	0.84	0.00	0.00	0.99	1	1	1	0.614	1.148	0.020	3.8		9.30E-01	0.77	0.31	0.85
19	2.50	19.6	280	0.708	0.074	0.18	1.49	0.83	0.00	0.00	0.99	1	1	1	0.614	1.148	0.022	3.8		9.30E-01	0.69	0.22	1.30
20	1.00	19.6	280	0.666	0.079	0.18	1.56	0.80	0.00	0.00	0.99	1	1	1	0.614	1.148	0.021	3.8		9.30E-01	0.74	0.25	1.20

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). "DeepSoil V5.1, User Manual and Tutorial 2002-2012." Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). "Recent advances in non-linear site response analysis." In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.18 Parameters for Shear Strength Correction REHS Station.

For Use in Strength Correction							
Layer No.	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	OCR	PI	σ' (atm)	τ (kPa)
1	0.30	17.3	90	1	0	0.0	2.6
2	0.55	17.3	90	1	0	0.1	8.3
3	0.65	17.3	90	1	0	0.2	15.9
|	0.50	17.3	90	1	0	0.3	24.1
5	1.00	19.6	90	1	0	0.4	29.5
6	1.00	19.6	90	1	0	0.5	35.4
7	1.00	19.6	110	1	0	0.6	42.2
8	1.10	19.6	110	1	0	0.7	49.5
9	0.90	19.6	130	1	0	0.8	56.4
10	1.00	19.6	130	1	0	0.9	65.3
11	1.00	19.6	130	1	0	1.0	72.4
12	0.70	19.6	220	1	0	1.1	78.5
13	0.90	19.6	220	1	0	1.1	100.7
14	0.90	19.6	220	1	0	1.2	108.4
15	0.90	19.6	220	1	0	1.3	133.6
16	0.90	19.6	220	1	0	1.4	142.4
17	0.70	19.6	220	1	0	1.5	150.3
18	2.50	19.6	280	1	0	1.6	166.0
19	2.50	19.6	280	1	0	1.9	190.6
20	1.00	19.6	280	1	0	2.1	207.8

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.19 Parameters for Seismic Site Response Analyses SHLC SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	1.00	17.3	120	1.780	0.019	0.18	1.25	0.92	0.00	0.00	0.99	1	1	1	2.281	1.537	0.007	3.8		9.3E-03	0.63	0.26	2.60
2	0.50	17.3	120	1.256	0.038	0.18	1.55	0.77	0.00	0.00	0.99	1	1	1	2.281	1.251	0.010	3.8		7.4E+00	0.78	0.26	1.20
3	0.50	19.6	120	1.132	0.042	0.18	1.52	0.77	0.00	0.00	0.99	1	1	1	2.281	1.251	0.011	3.8		7.4E+00	0.80	0.28	0.91
4	1.00	19.6	120	1.015	0.041	0.18	1.32	0.74	0.00	0.00	0.99	1	1	1	2.281	1.251	0.012	3.8		9.3E-01	0.92	0.37	0.55
5	1.00	19.6	150	1.021	0.048	0.18	1.49	0.78	0.00	0.00	0.99	1	1	1	1.614	1.251	0.013	3.8		7.4E+00	0.73	0.22	1.30
6	1.00	19.6	150	0.946	0.054	0.18	1.52	0.77	0.00	0.00	0.99	1	1	1	1.614	1.251	0.014	3.8		7.4E+00	0.79	0.27	0.85
7	2.00	19.6	200	0.918	0.055	0.18	1.53	0.81	0.00	0.00	0.99	1	1	1	1.034	1.251	0.015	3.8		7.4E+00	0.72	0.24	1.20
8	1.50	19.6	200	0.860	0.060	0.18	1.53	0.81	0.00	0.00	0.99	1	1	1	1.034	1.251	0.016	3.8		9.3E-01	0.75	0.27	0.90
9	1.50	19.6	200	0.808	0.067	0.18	1.56	0.80	0.00	0.00	0.99	1	1	1	1.034	1.251	0.018	3.8		7.4E-01	0.84	0.35	0.65
10	2.50	19.6	250	0.809	0.061	0.18	1.52	0.86	0.00	0.00	0.99	1	1	1	0.731	1.251	0.018	3.8		4.6E-01	0.68	0.24	1.40
11	2.50	19.6	250	0.763	0.059	0.18	1.32	0.83	0.00	0.00	0.99	1	1	1	0.731	1.251	0.020	3.8		4.6E-01	0.65	0.18	1.85
12	2.50	19.6	250	0.714	0.076	0.18	1.53	0.81	0.00	0.00	0.99	1	1	1	0.731	1.251	0.021	3.8		9.3E-01	0.74	0.26	0.90
13	2.50	19.6	250	0.691	0.072	0.18	1.40	0.81	0.00	0.00	0.99	1	1	1	0.731	1.350	0.022	3.8		1.9E-01	0.71	0.23	1.10
14	2.50	19.6	250	0.647	0.078	0.18	1.50	0.80	0.00	0.00	0.99	1	1	1	0.731	1.350	0.021	3.8		2.8E-01	0.79	0.29	0.90
15	2.50	19.6	250	0.618	0.074	0.18	1.37	0.78	0.00	0.00	0.99	1	1	1	0.731	1.350	0.022	3.8		2.8E-01	0.81	0.30	0.80
16	2.00	19.6	200	0.540	0.098	0.18	1.44	0.69	0.00	0.00	0.99	1	1	1	1.034	1.690	0.024	3.8		4.6E-06	0.87	0.27	2.20

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). “DeepSoil V5.1, User Manual and Tutorial 2002-2012.” Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). “Recent advances in non-linear site response analysis.” In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.20 Parameters for Shear Strength Correction SHLC Station.

For Use in Strength Correction							
Layer No.	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	OCR	PI	σ' (atm)	τ (kPa)
1	1.00	17.3	120	1	0	0.1	6.3
2	0.50	17.3	120	1	0	0.2	22.2
3	0.50	19.6	120	1	0	0.3	29.8
4	1.00	19.6	120	1	0	0.4	36.6
5	1.00	19.6	150	1	0	0.5	45.8
6	1.00	19.6	150	1	0	0.6	55.0
7	2.00	19.6	200	1	0	0.7	73.7
8	1.50	19.6	200	1	0	0.9	84.8
9	1.50	19.6	200	1	0	1.0	95.1
10	2.50	19.6	250	1	0	1.2	109.0
11	2.50	19.6	250	1	0	1.5	130.3
12	2.50	19.6	250	1	0	1.7	159.9
13	2.50	19.6	250	1	0	2.0	161.2
14	2.50	19.6	250	1	0	2.2	181.1
15	2.50	19.6	250	1	0	2.5	201.0
16	2.00	19.6	200	1	0	2.7	211.2

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

Table C.21 Parameters for Seismic Site Response Analyses SHLC SMS.

Layer #	Thickness (m)	γ (Kn/m^3)	Vs (m/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (Mpa)	Beta	s	b	d	Max Ru	PWP Model	f/s/f	p/r/Dr(%)	F/A/FC(%)	s/B/-	g/C/-	v/D/v	-/g/-	Cv (m^2/sec)	P1	P2	P3
1	1.25	17.3	170	1.691	0.025	0.18	1.62	0.95	0.00	0.00	0.99	1	1	1	1.330	1.251	0.008	3.8		9.30E-02	0.61	0.26	3.25
2	1.25	17.3	170	1.167	0.036	0.18	1.43	0.83	0.00	0.00	0.99	1	1	1	1.330	1.251	0.011	3.8		9.30E-02	0.70	0.24	1.49
3	1.00	19.6	230	1.059	0.034	0.18	1.22	0.90	0.00	0.00	0.99	1	1	1	0.832	1.251	0.014	3.8		9.30E-02	0.63	0.24	2.15
4	2.00	19.6	230	0.978	0.040	0.18	1.22	0.87	0.00	0.00	0.99	1	1	1	0.832	1.251	0.016	3.8		9.30E-02	0.64	0.22	2.15
5	2.25	19.6	230	0.875	0.054	0.18	1.49	0.84	0.00	0.00	0.99	1	1	1	0.832	1.251	0.016	3.8		9.30E-02	0.77	0.32	0.85
6	2.25	19.6	230	0.815	0.056	0.18	1.37	0.81	0.00	0.00	0.99	1	1	1	0.832	1.251	0.018	3.8		9.30E-02	0.72	0.24	1.10
7	2.00	19.6	230	0.760	0.074	0.18	1.56	0.80	0.00	0.00	0.99	1	1	1	0.832	1.251	0.019	3.8		9.30E-02	0.80	0.30	0.75
8	1.00	19.6	140	0.608	0.094	0.18	1.53	0.66	0.00	0.00	0.99	1	1	1	1.796	1.592	0.019	3.8		9.30E-04	0.96	0.32	2.35
9	1.00	19.6	140	0.575	0.099	0.18	1.53	0.65	0.00	0.00	0.99	1	1	1	1.796	1.592	0.020	3.8		9.30E-04	0.97	0.31	2.45
10	1.00	19.6	140	0.564	0.105	0.18	1.53	0.65	0.00	0.00	0.99	1	1	1	1.796	1.592	0.021	3.8		9.30E-04	0.99	0.34	2.87
11	1.00	19.6	140	0.559	0.091	0.18	1.37	0.65	0.00	0.00	0.99	1	1	1	1.796	1.251	0.021	3.8		9.30E-02	0.98	0.34	3.15
12	1.00	19.6	140	0.529	0.077	0.18	1.20	0.63	0.00	0.00	0.99	1	1	1	1.796	1.251	0.022	3.8		9.30E-02	0.99	0.32	3.15
13	1.00	19.6	140	0.525	0.119	0.18	1.53	0.63	0.00	0.00	0.99	1	1	1	1.796	1.592	0.023	3.8		9.30E-04	0.99	0.32	3.19
14	2.50	19.6	300	0.670	0.058	0.18	1.17	0.83	0.00	0.00	0.99	1	1	1	0.551	1.251	0.023	3.8		9.30E-02	0.83	0.36	0.65
15	2.50	19.6	300	0.648	0.078	0.18	1.53	0.81	0.00	0.00	0.99	1	1	1	0.551	1.251	0.021	3.8		9.30E-02	0.72	0.24	1.25
16	2.50	19.6	300	0.621	0.083	0.18	1.52	0.80	0.00	0.00	0.99	1	1	1	0.551	1.251	0.023	3.8		9.30E-02	0.75	0.26	1.00
17	2.50	19.6	300	0.593	0.076	0.18	1.35	0.78	0.00	0.00	0.99	1	1	1	0.551	1.251	0.024	3.8		9.30E-02	0.80	0.29	0.80

Notes:

1) γ: Specific unit weight, Vs: Shear wave velocity, Cv: Coefficient of consolidation.
2) For explanation of the rest of parameters see Hashash, Y.M.A. (2012). “DeepSoil V5.1, User Manual and Tutorial 2002-2012.” Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, and Hashash, Y.M.A., Groholski, D.R. Phillips, C.A., (2010). “Recent advances in non-linear site response analysis.” In proceedings of Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, May 24-29, San Diego, CA. and

Table C.22 Parameters for Shear Strength Correction SHLC Station.

For Use in Strength Correction							
Layer No.	**Thickness (m)**	**γ (Kn/m^3)**	**Vs (m/s)**	**OCR**	**PI**	**σ' (atm)**	**τ (kPa)**
1	1.25	17.3	170	1	0	0.1	10.8
2	1.25	17.3	170	1	0	0.3	33.9
3	1.00	19.6	230	1	0	0.5	54.0
4	2.00	19.6	230	1	0	0.7	68.8
5	2.25	19.6	230	1	0	0.9	89.7
6	2.25	19.6	230	1	0	1.1	111.8
7	2.00	19.6	230	1	0	1.3	132.7
8	1.00	19.6	140	1	0	1.5	115.2
9	1.00	19.6	140	1	0	1.6	122.9
10	1.00	19.6	140	1	0	1.6	130.5
11	1.00	19.6	140	1	0	1.7	138.2
12	1.00	19.6	140	1	0	1.8	145.9
13	1.00	19.6	140	1	0	1.9	153.6
14	2.50	19.6	300	1	0	2.1	213.8
15	2.50	19.6	300	1	0	2.4	238.4
16	2.50	19.6	300	1	0	2.6	263.0
17	2.50	19.6	300	1	0	2.8	287.6

Notes:

1) γ: Specific unit weight.
2) Vs: Shear wave velocity.
3) OCR: Overconsolidation ratio
4) PI: Plasticity Index.
5) σ': Mean effective stress.
6) τ : Shear Strenght.

APPENDIX D

Results for Seismic Site Response Analyses at Strong Ground Motion Stations

Available at:

https://www.dropbox.com/s/8ljjkbmgbnr5v1z/AppendixD2.pdf?n=314408476

D.1 Results for 22 February 2011 M_w 6.2 Event.

D.2 Results for 04 September 2010 M_w 7.1 Event.

D.3 Results for 13 June 2011 M_w 6.0 Event.

D.4 Results for 23 December 2011 M_w 5.8 Event.

D.5 Results for 23 December 2011 M_w 5.9 Event.

D.6 Results for 26 December 2010 M_w 4.7 Event.

APPENDIX E

Liquefaction Triggering Analyses.

E.1 Liquefaction Triggering Analyses 22 February 2011 Event.

E.2 Liquefaction Triggering Analyses 04 September 2010 Event.

E.3 Liquefaction Triggering Analyses 26 December 2010 Event.

APPENDIX E.1

Liquefaction Triggering Analyses 22 February 2011 Event.

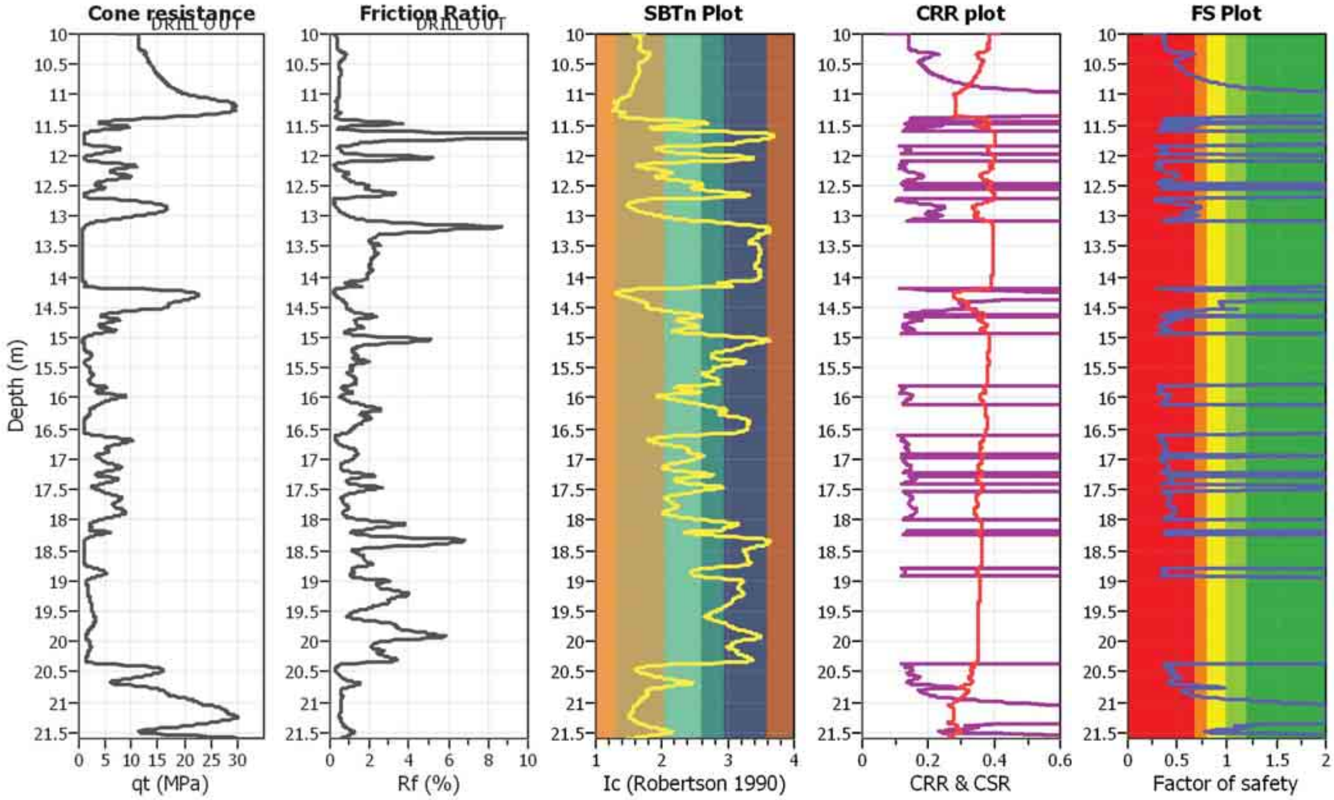
LIQUEFACTION ANALYSIS REPORT

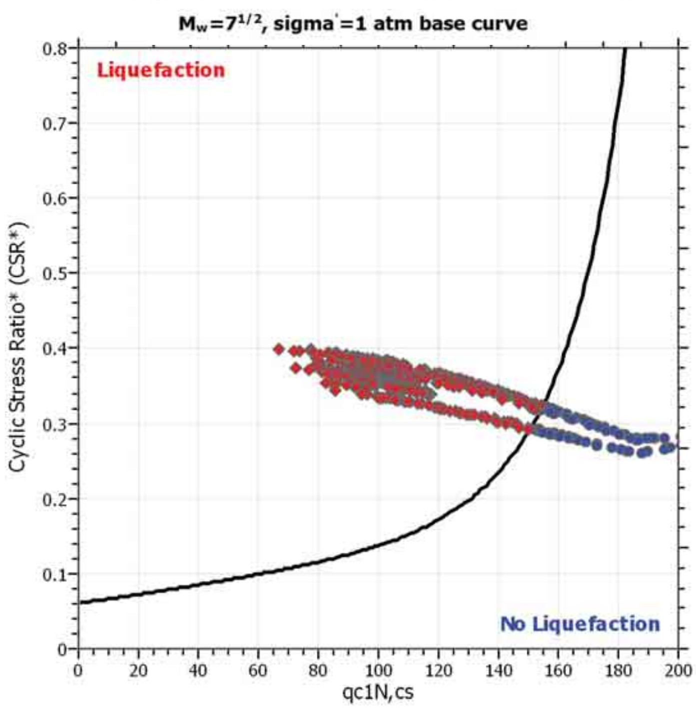
Project title : G13AP00029 **Location : Christchurch, New Zealand**

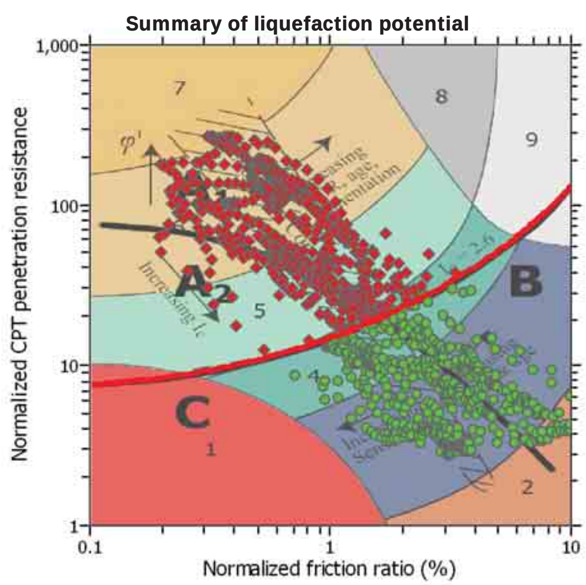
CPT file : CBGS_CPT1(Wotherspoon,2013)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

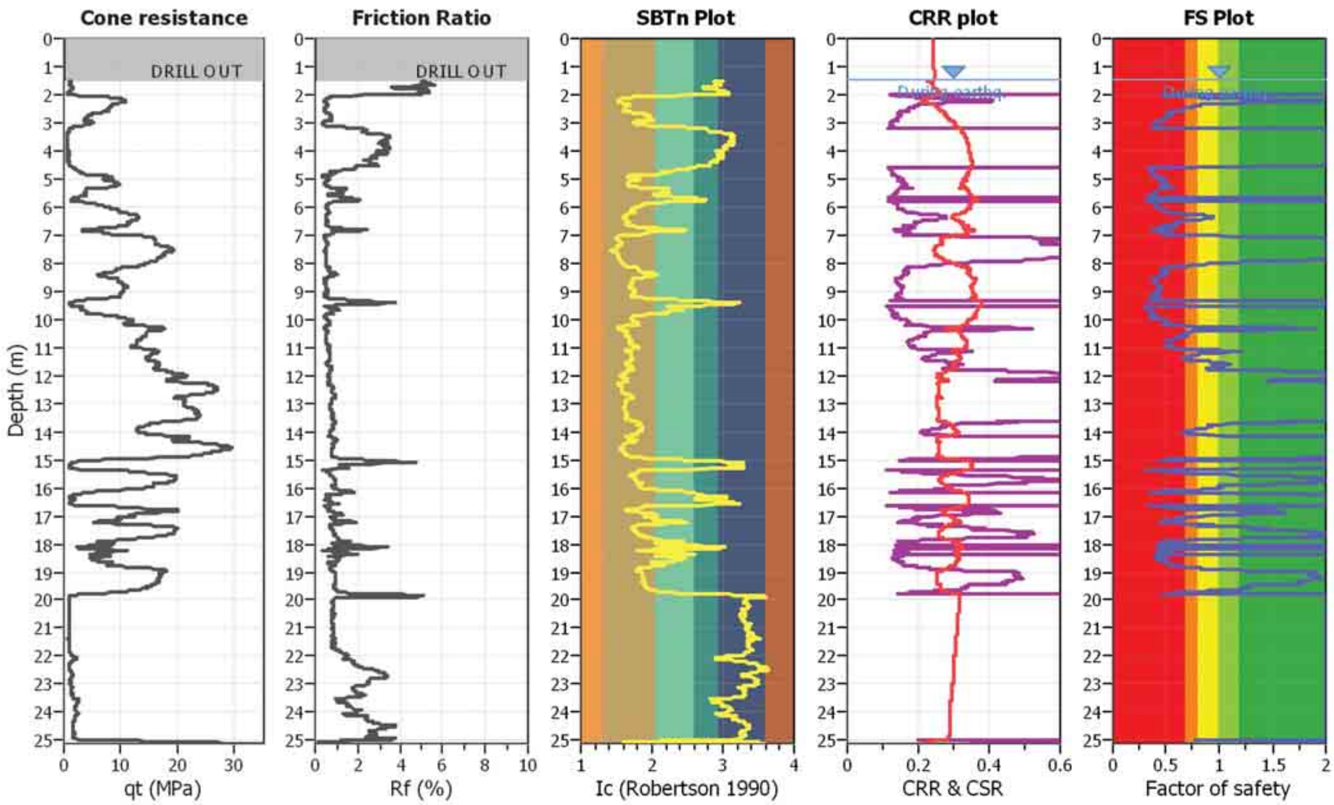
LIQUEFACTION ANALYSIS REPORT

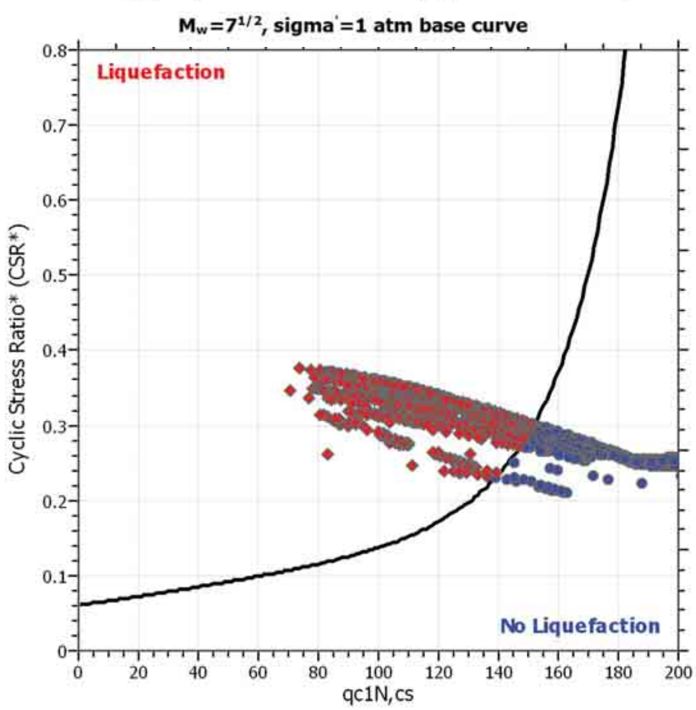
Project title : G13AP00029 **Location : Christchurch, New Zealand**

CPT file : CCCC_CPT484(CBD)

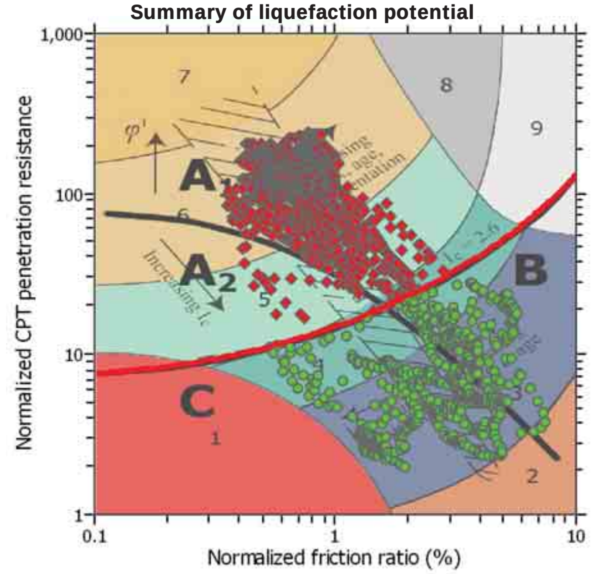
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.43	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		





Summary of liquefaction potential



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

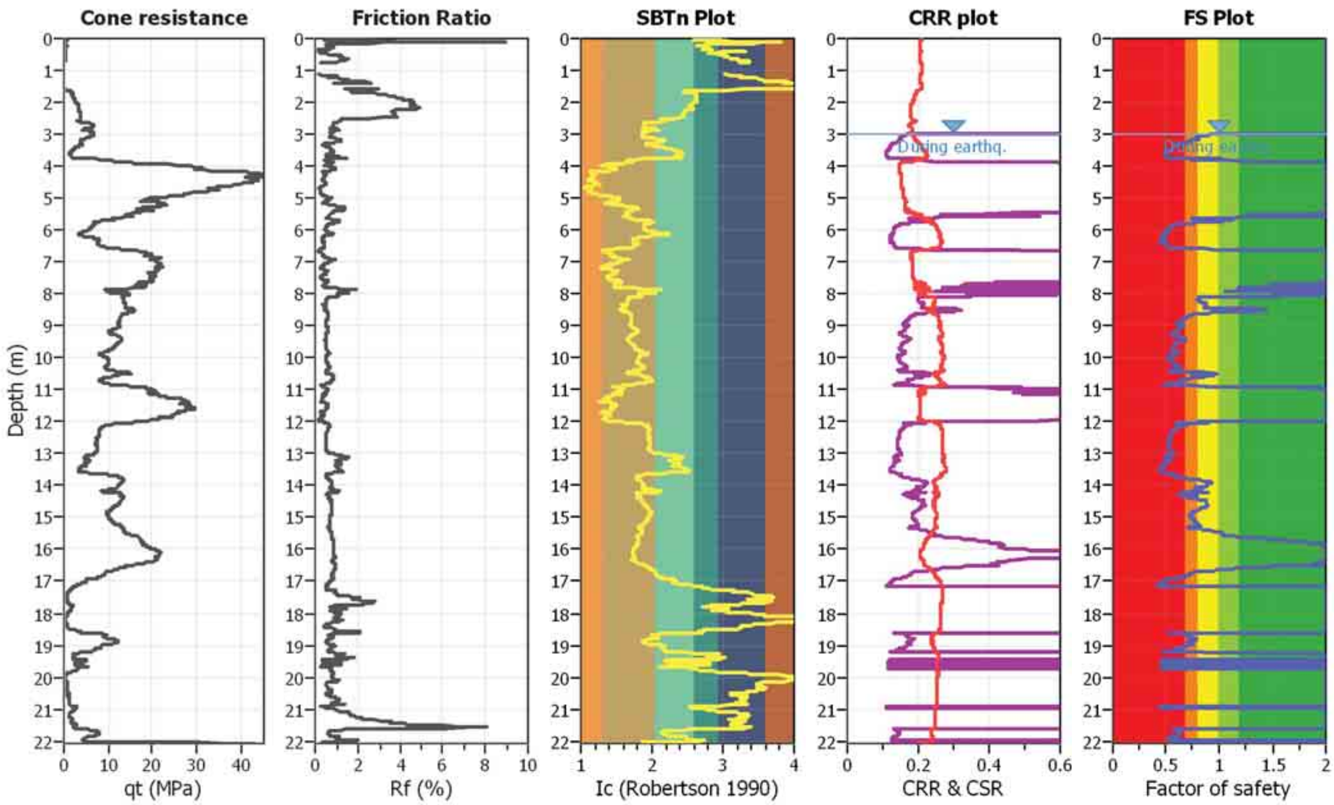
LIQUEFACTION ANALYSIS REPORT

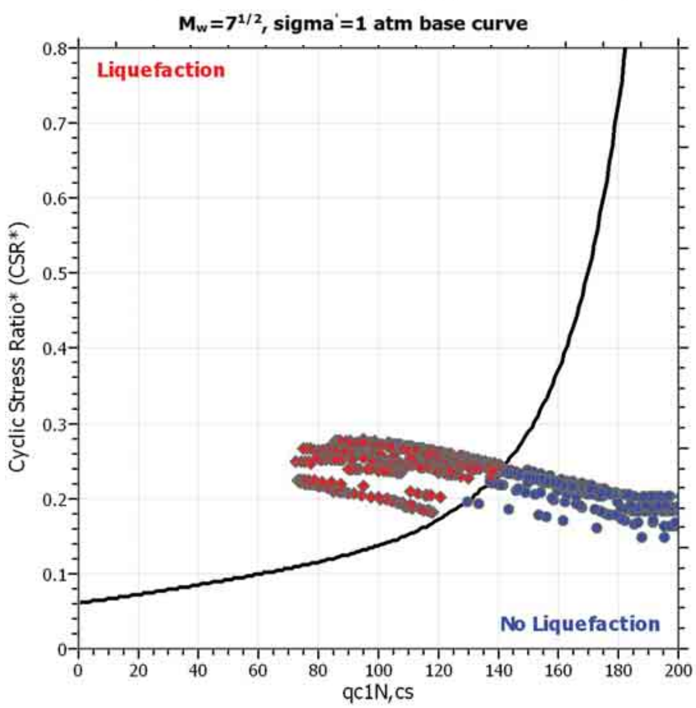
Project title : G13AP00029 **Location : Christchurch, New Zealand**

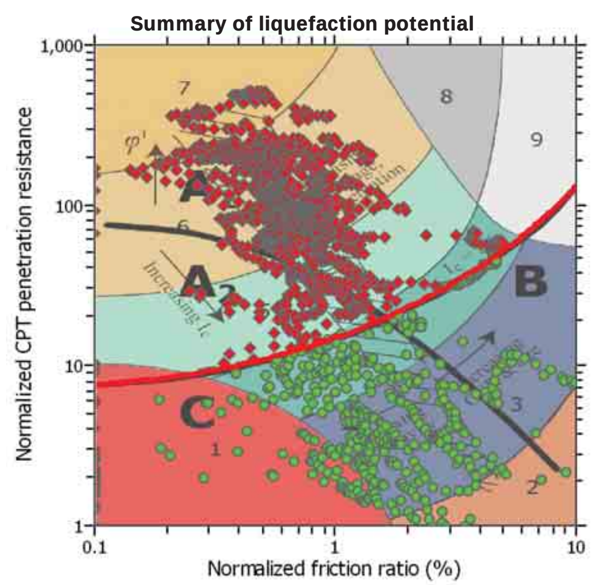
CPT file : CHHC_CPT425(CBD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	3.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	3.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.37	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

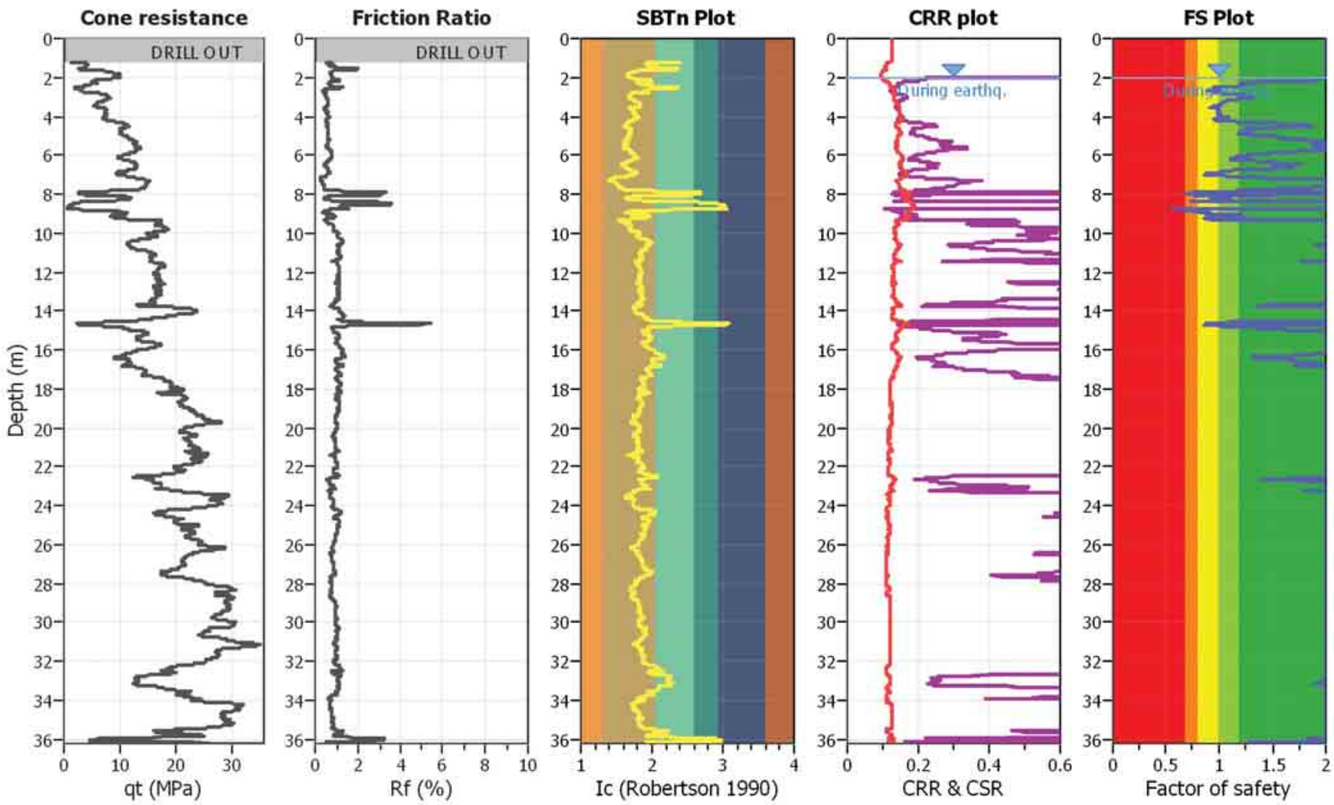
LIQUEFACTION ANALYSIS REPORT

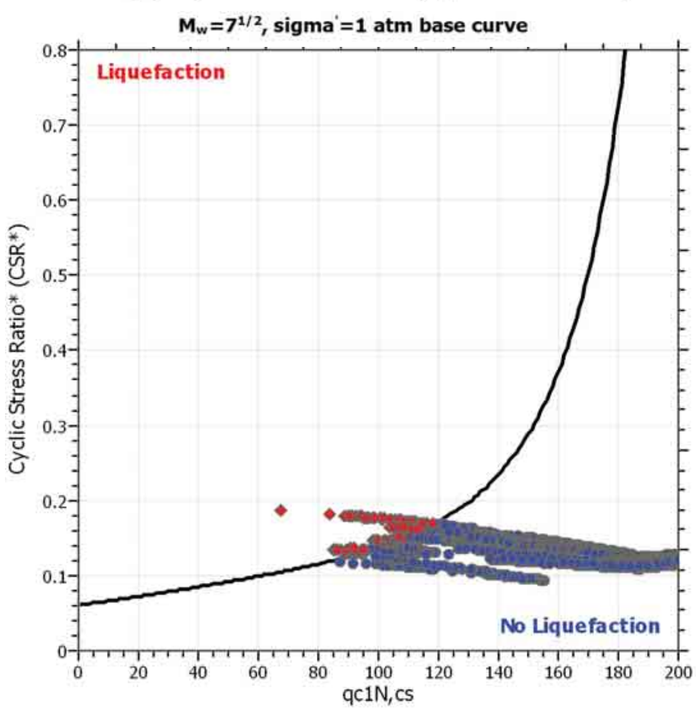
Project title : G13AP00029 **Location : Christchurch, New Zealand**

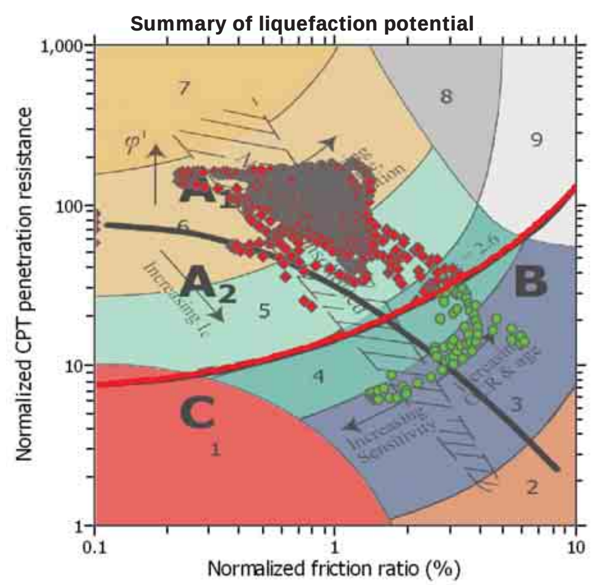
CPT file : HPSC_CPT89(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.22	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

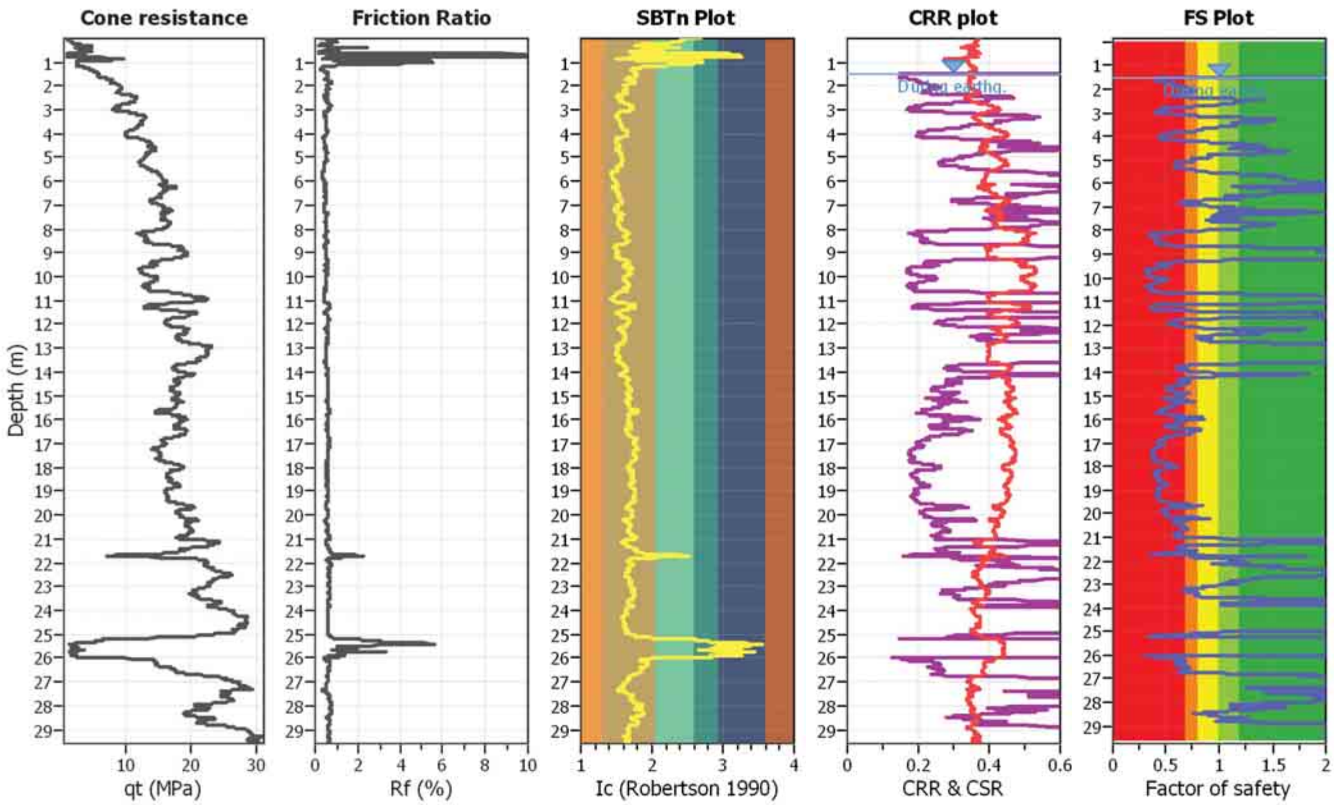
LIQUEFACTION ANALYSIS REPORT

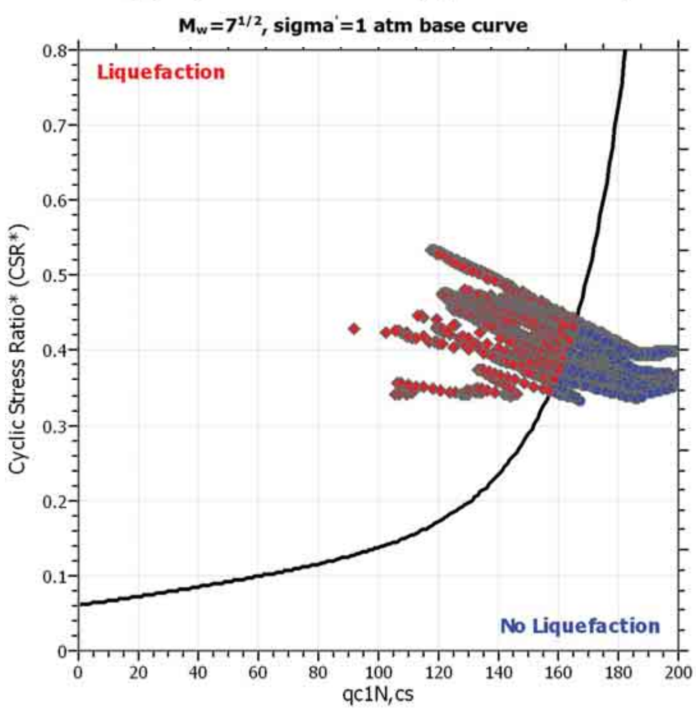
Project title : G13AP00029 **Location : Christchurch, New Zealand**

CPT file : NNBS_CPT33695(CGD)

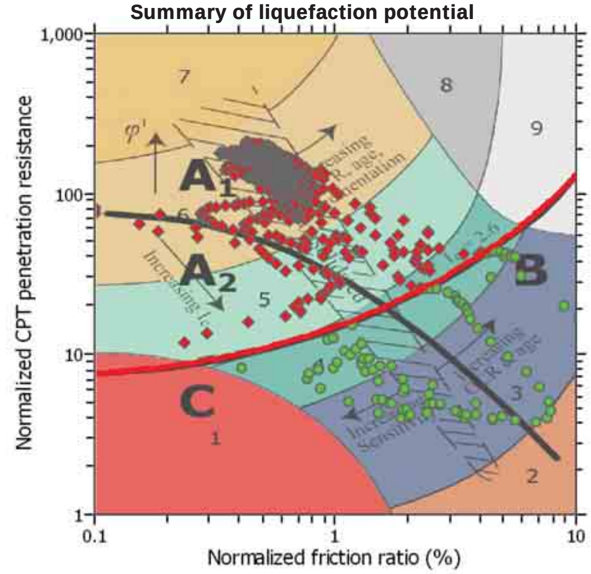
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.67	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		





Summary of liquefaction potential



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

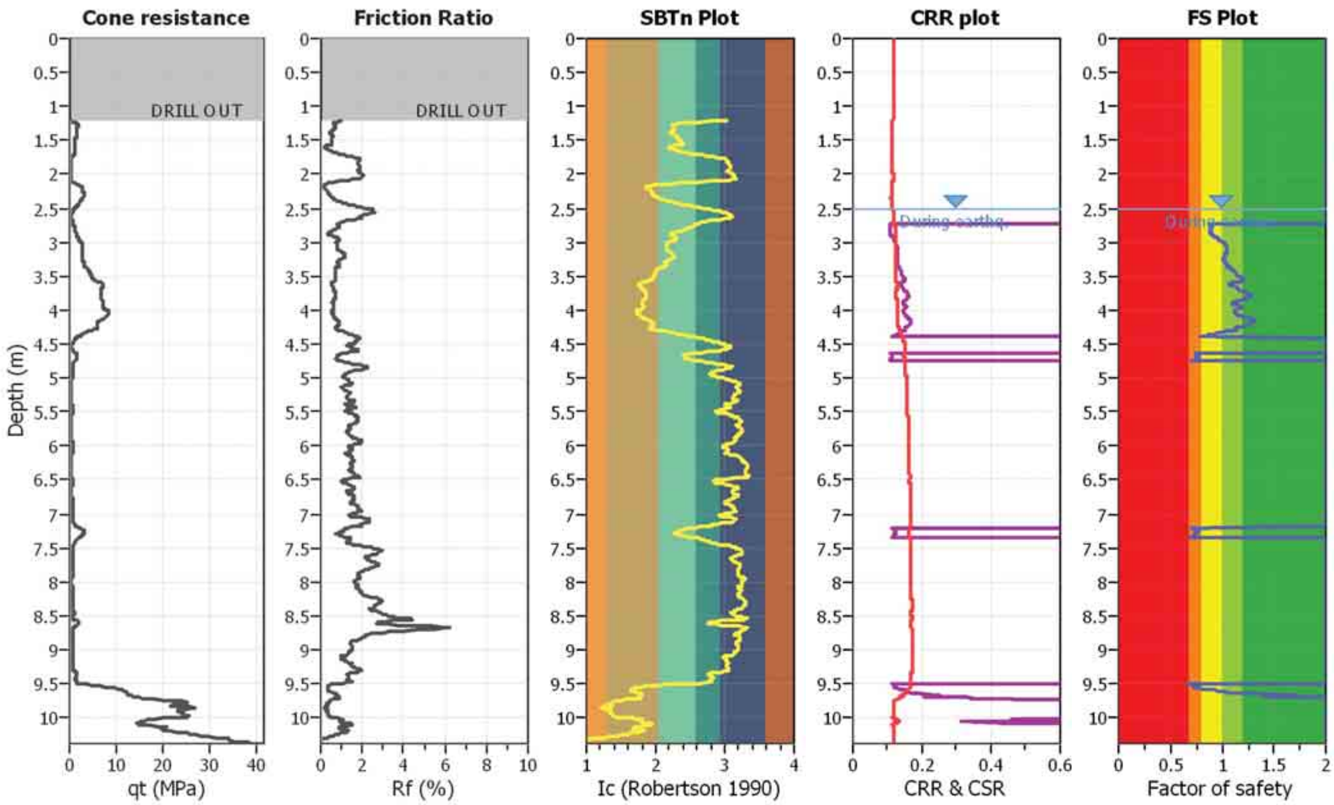
LIQUEFACTION ANALYSIS REPORT

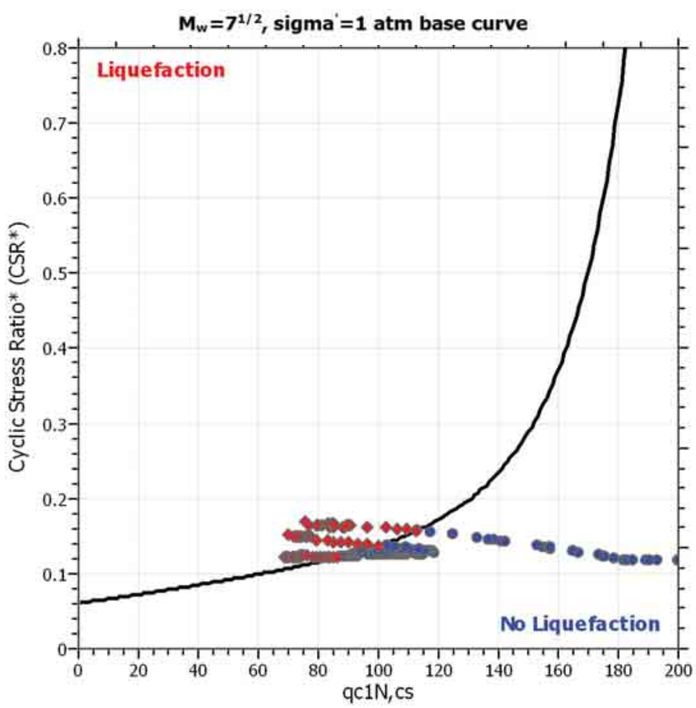
Project title : G13AP00029 **Location : Christchurch, New Zealand**

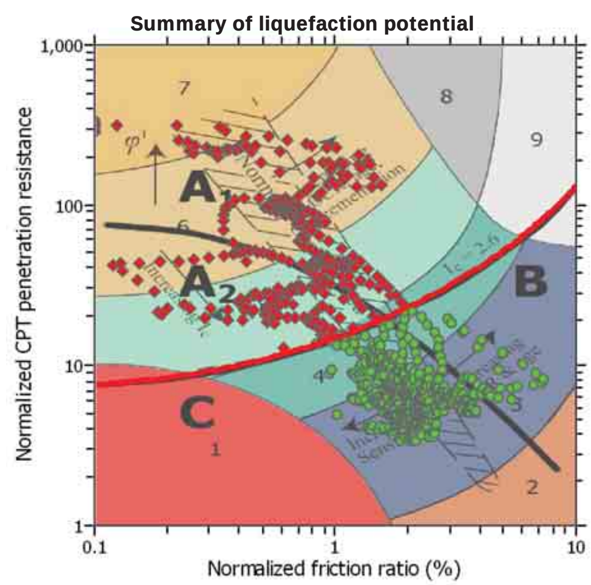
CPT file : PPHS_CPT1497(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.21	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		







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

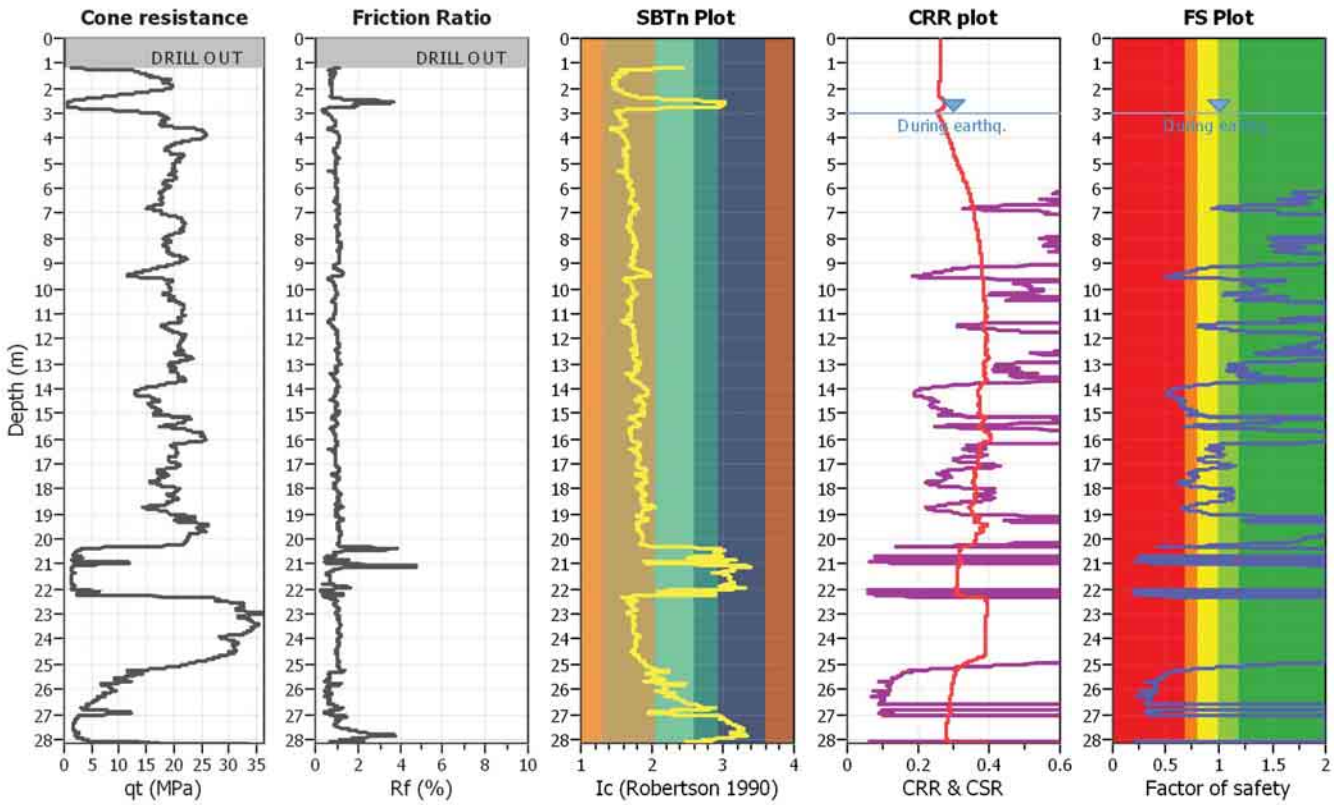
LIQUEFACTION ANALYSIS REPORT

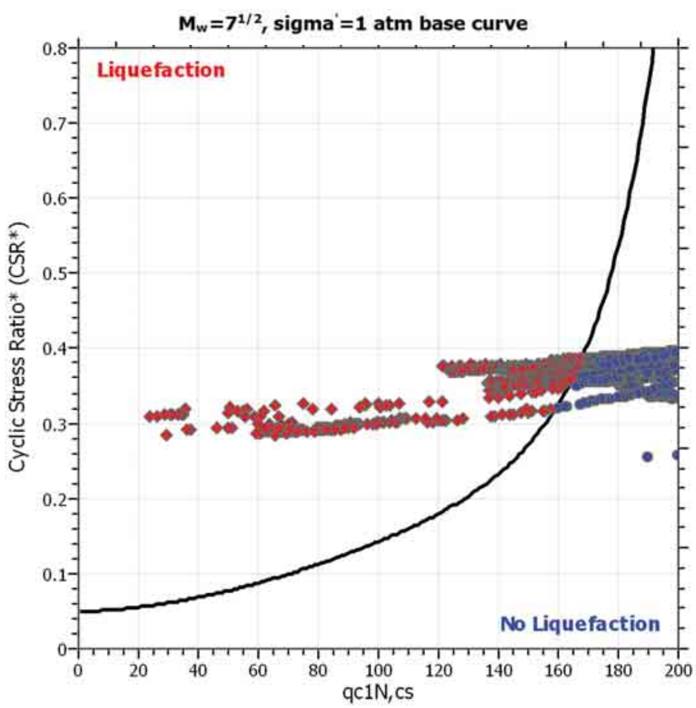
Project title : G13AP00029 **Location : Christchurch**

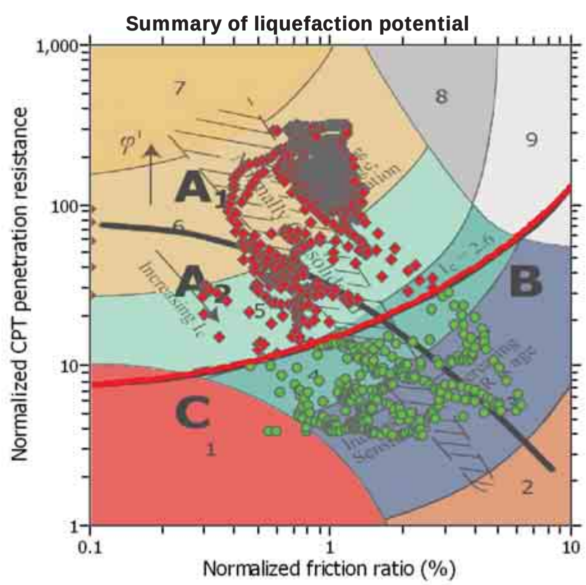
CPT file : PRPC_CPT1396 (CGD)

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	3.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	3.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.63	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

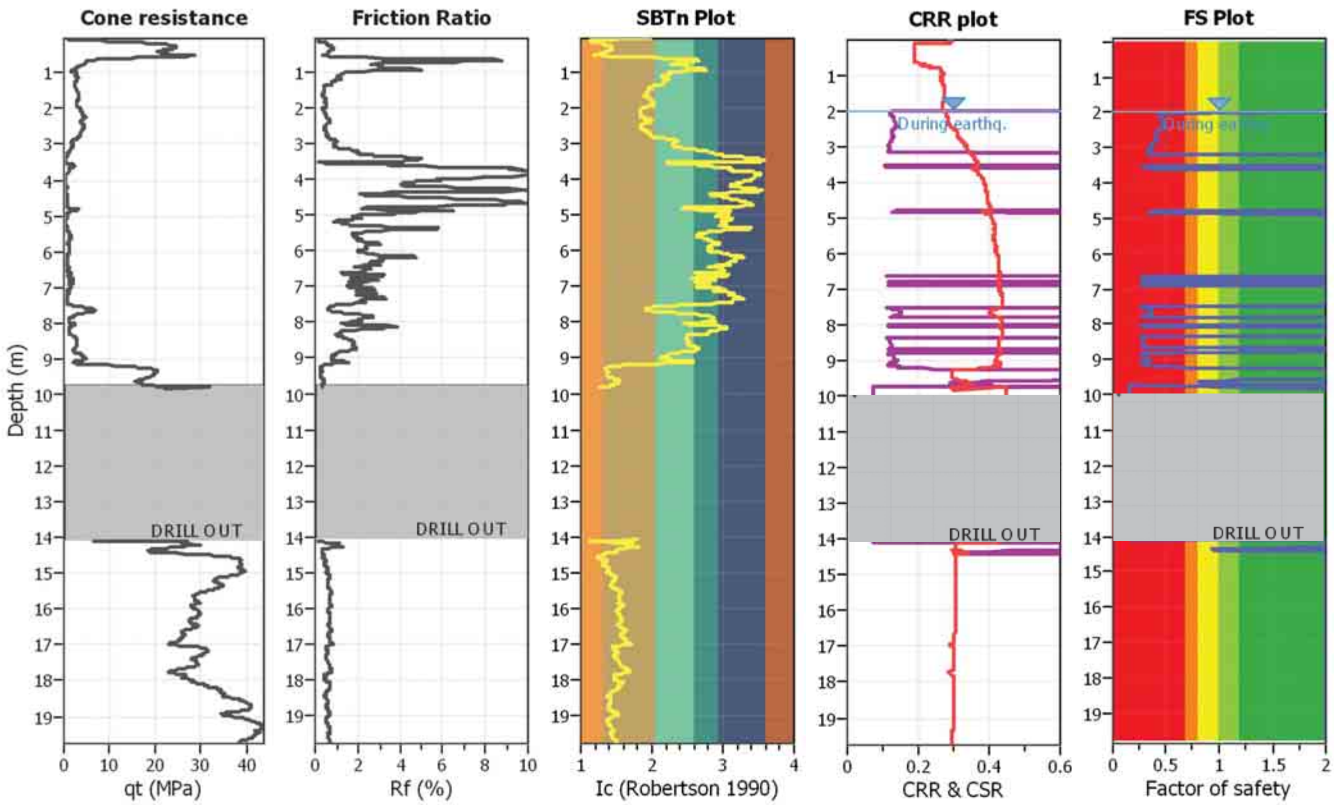
LIQUEFACTION ANALYSIS REPORT

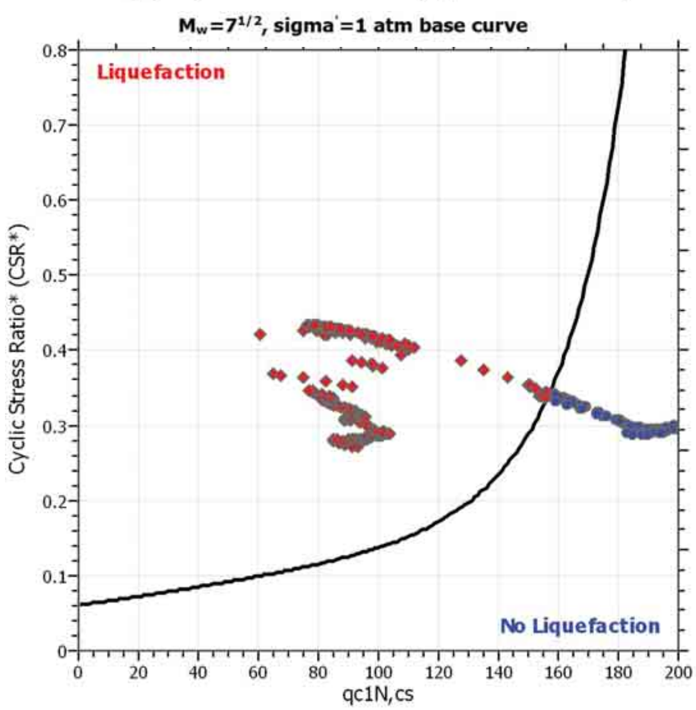
Project title : G13AP00029 **Location : Christchurch, New Zealand**

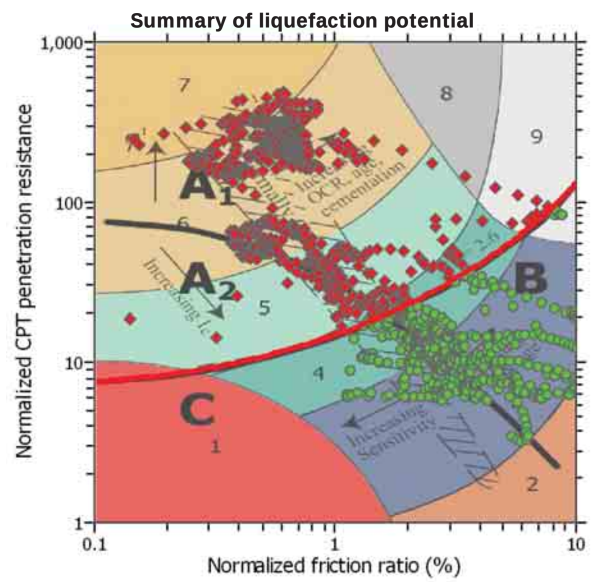
CPT file : REHS_CPT2 (Wotherspoon,2013)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.52	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

LIQUEFACTION ANALYSIS REPORT

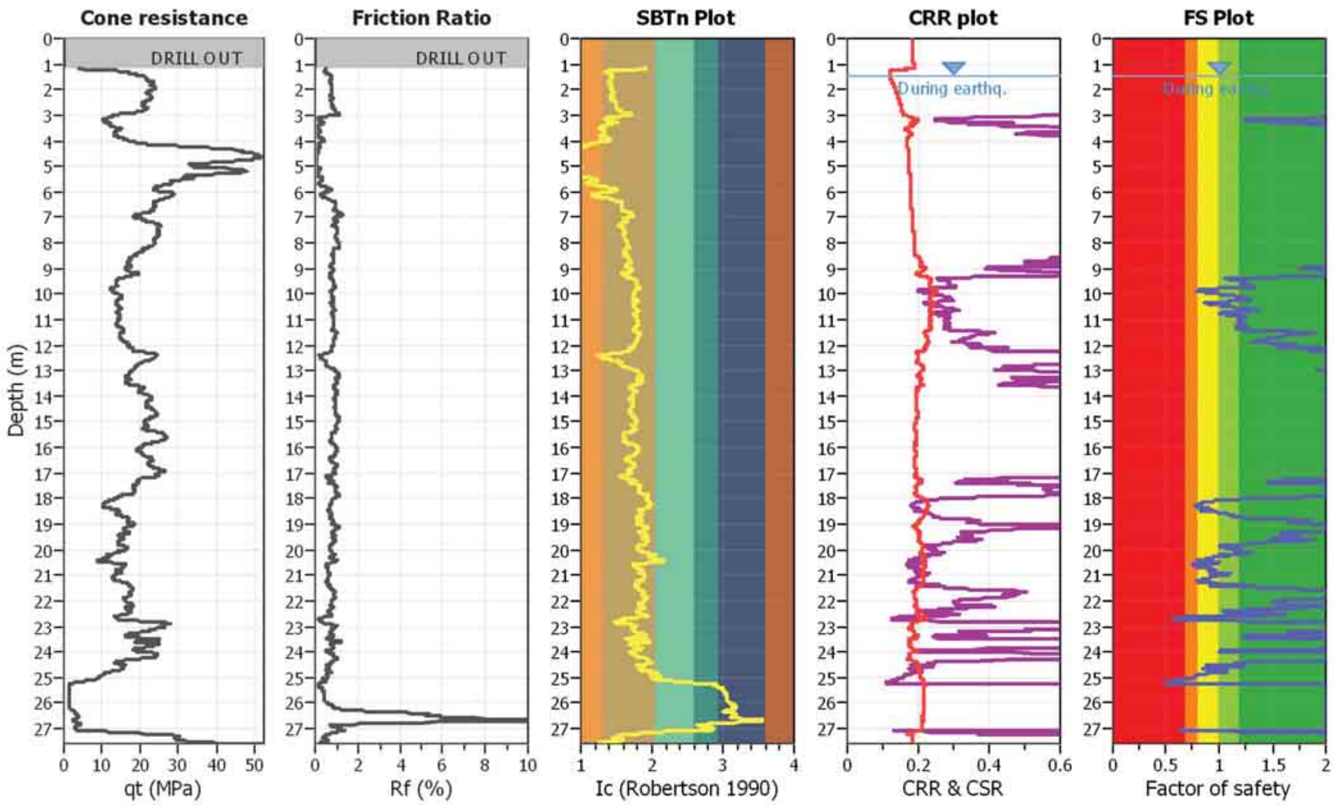
Project title : G13AP00029

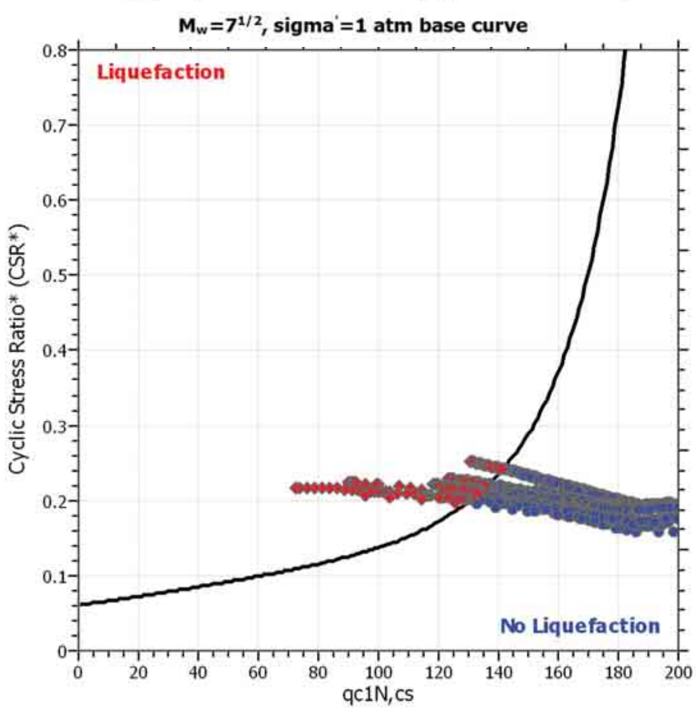
Location : Christchurch, New Zealand

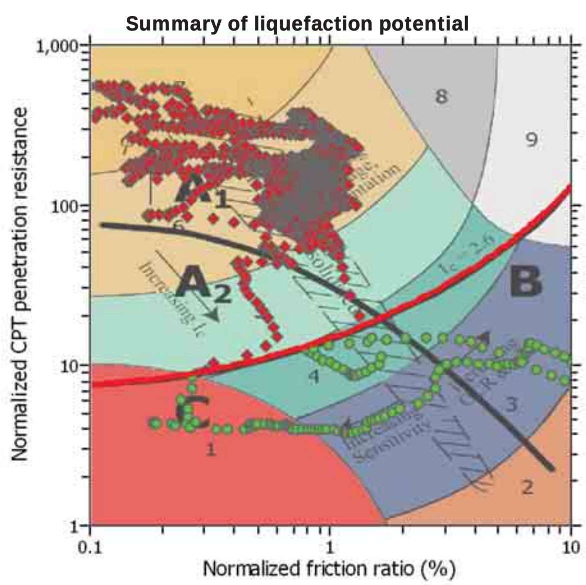
CPT file : SHLC_CPT626(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.33	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

APPENDIX E.2

Liquefaction Triggering Analyses 04 September 2010 Event.

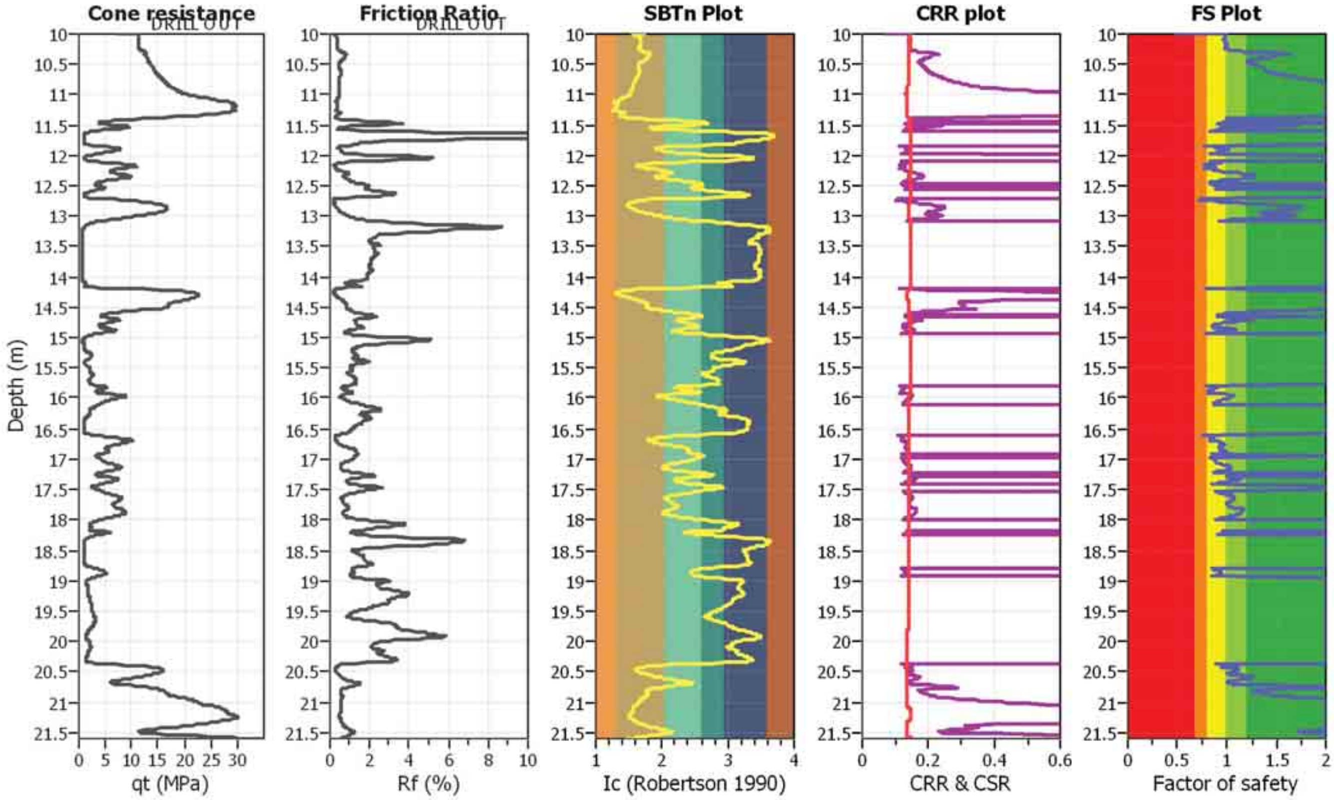
LIQUEFACTION ANALYSIS REPORT

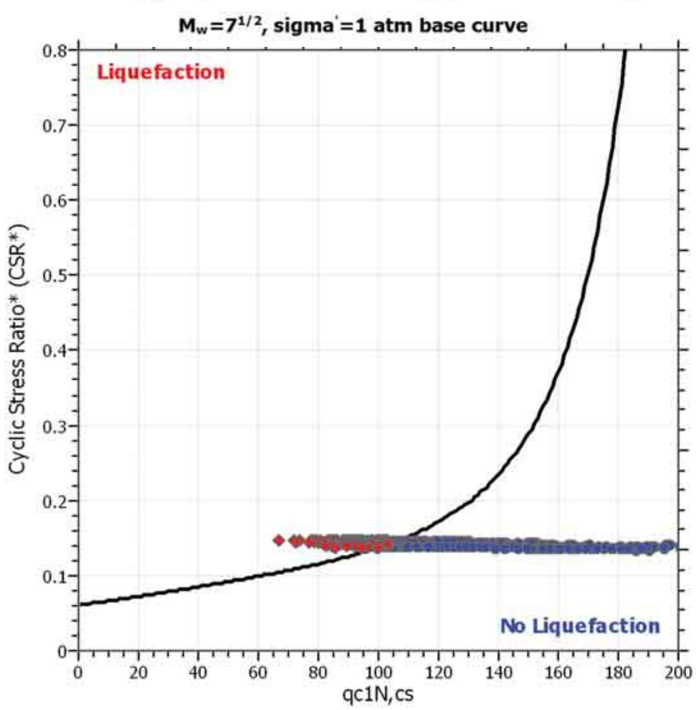
Project title : G13AP00029 **Location : Christchurch, New Zealand**

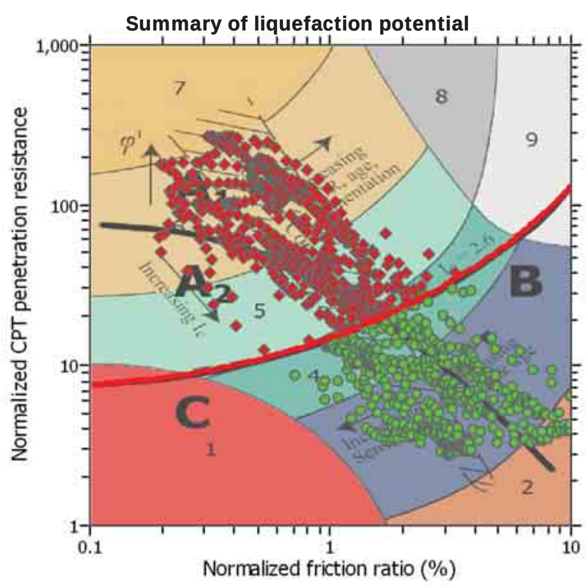
CPT file : CBGS_CPT1(Wotherspoon,2013)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.16	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		







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

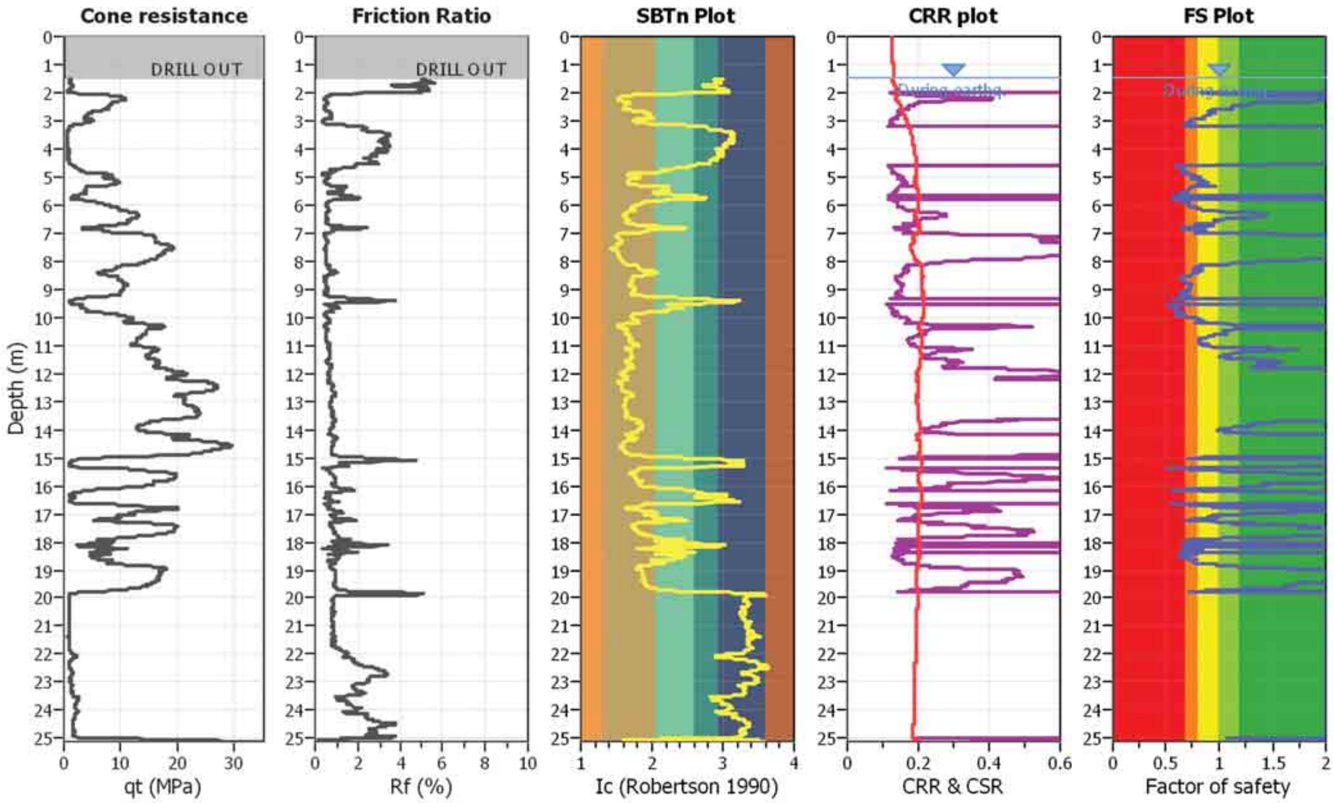
LIQUEFACTION ANALYSIS REPORT

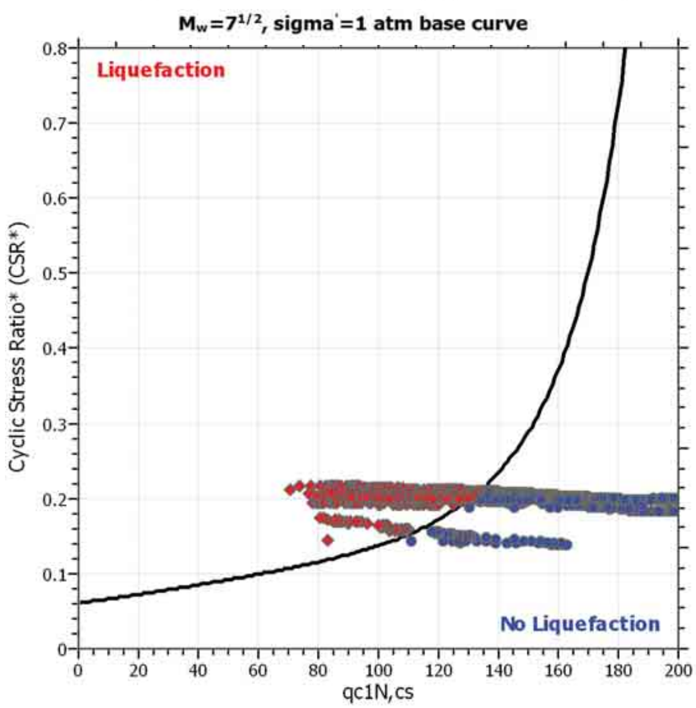
Project title : G13AP00029 **Location : Christchurch, New Zealand**

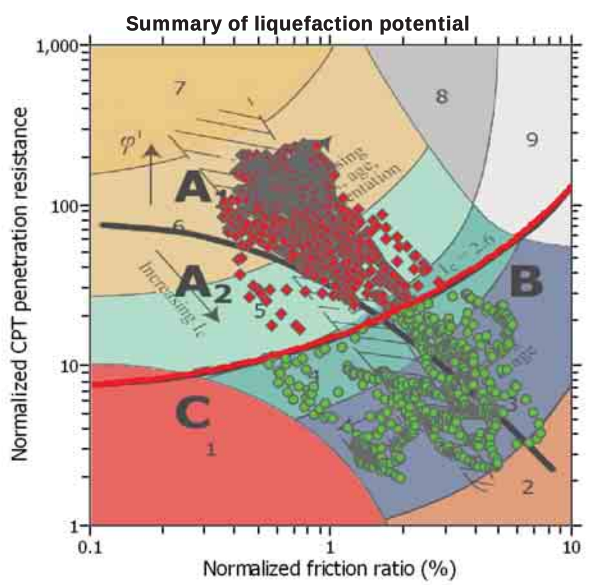
CPT file : CCCC_CPT484(CBD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.22	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		







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

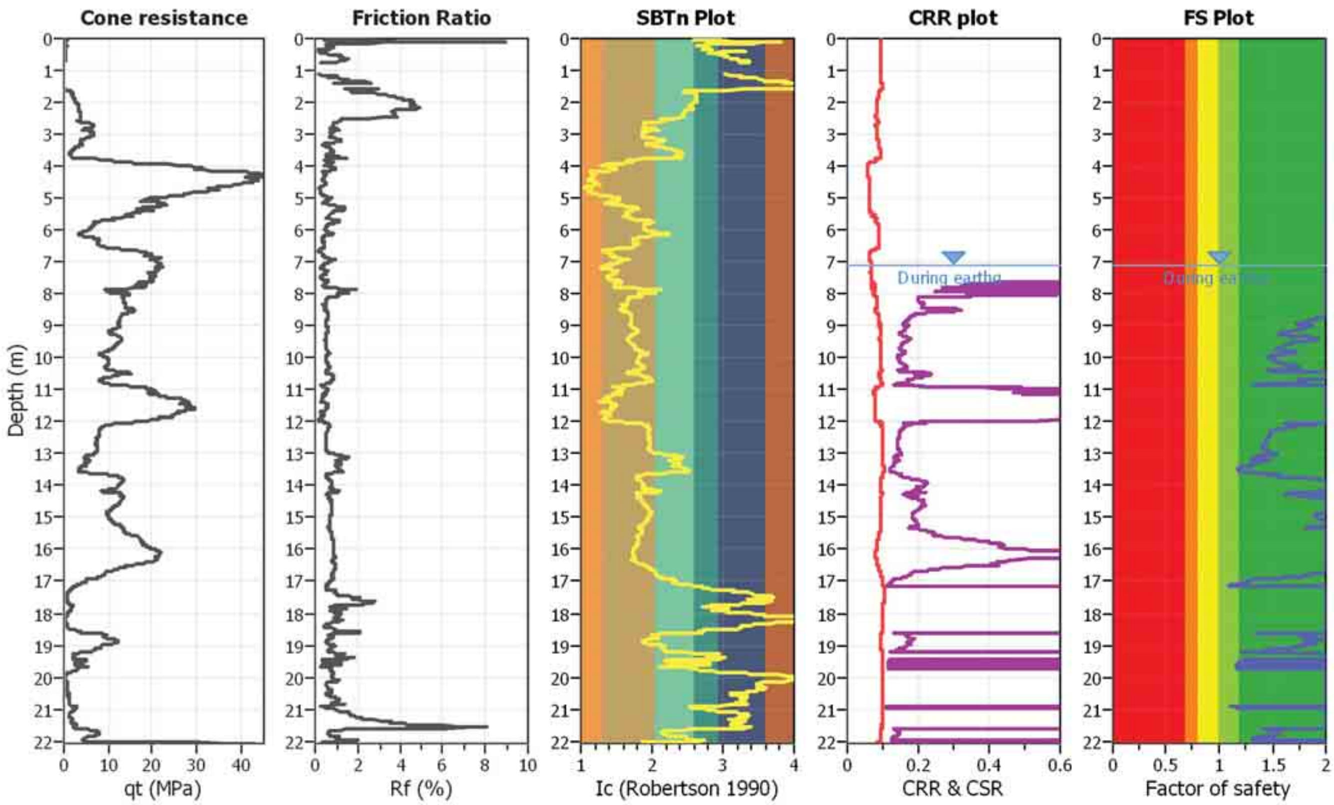
LIQUEFACTION ANALYSIS REPORT

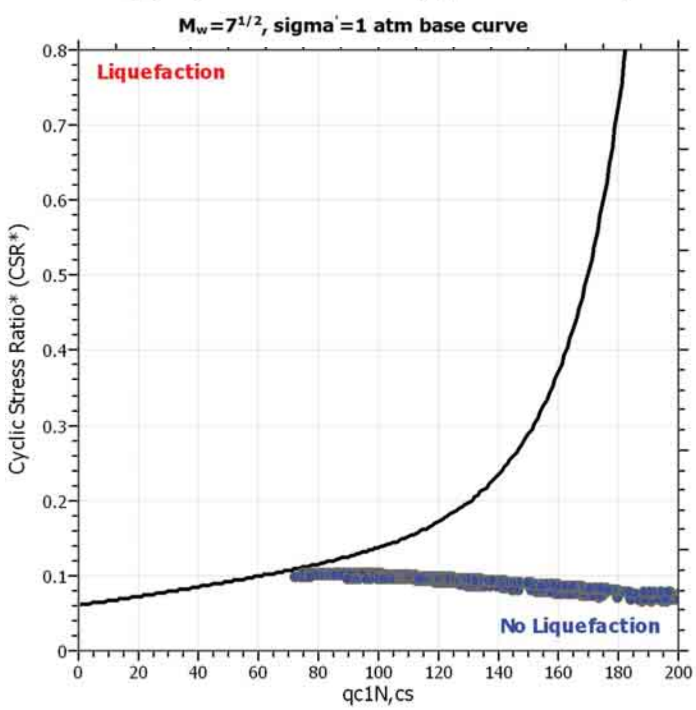
Project title : G13AP00029 **Location : Christchurch, New Zealand**

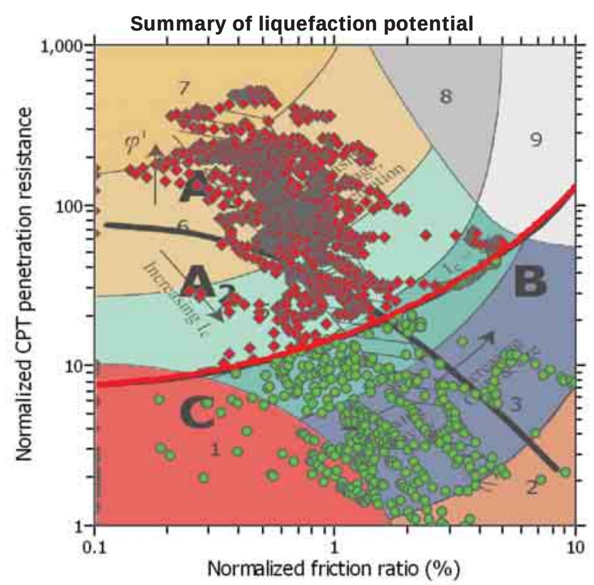
CPT file : CHHC_CPT425(CBD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	3.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.10 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.20	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.17	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		







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

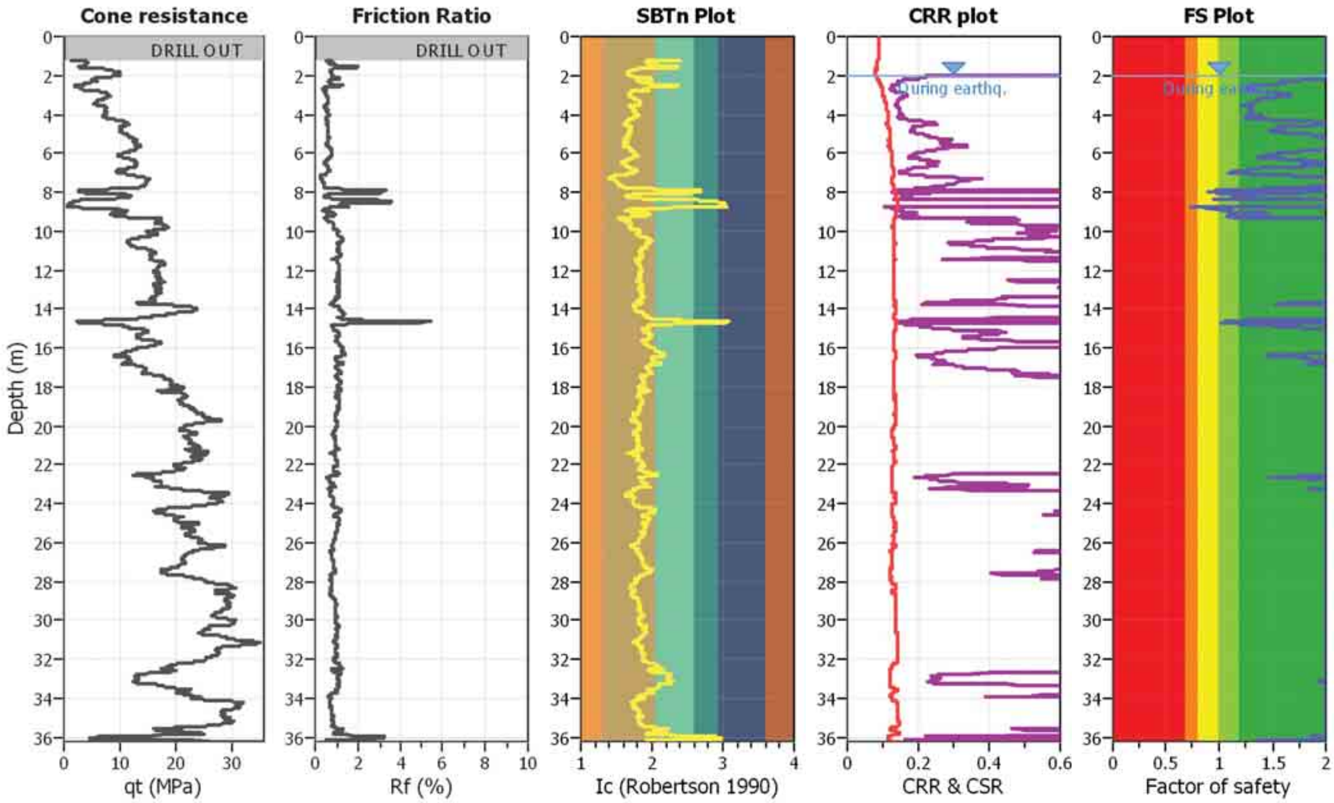
LIQUEFACTION ANALYSIS REPORT

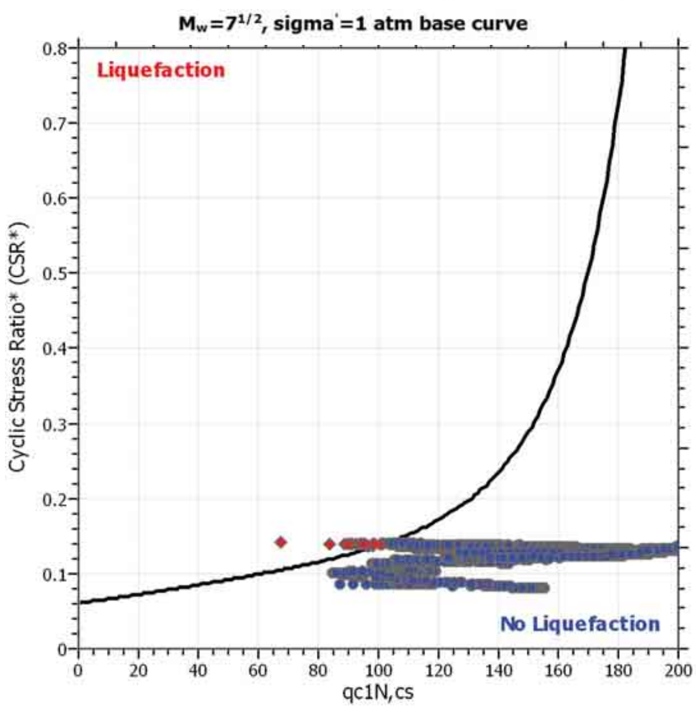
Project title : G13AP00029 **Location : Christchurch, New Zealand**

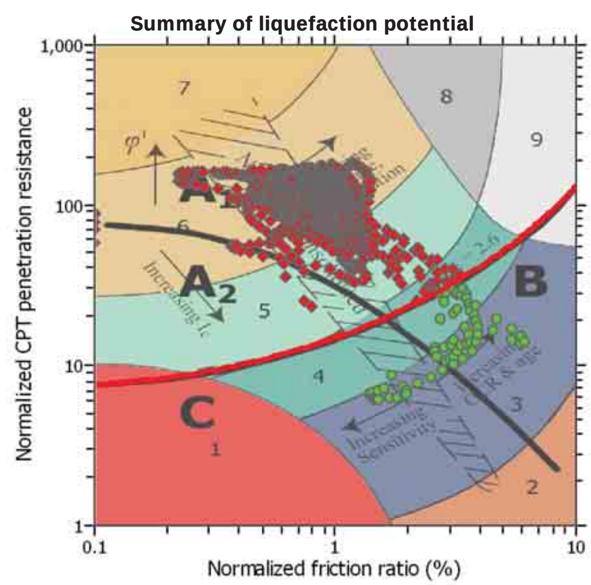
CPT file : HPSC_CPT89(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.15	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

GEOLOGISMIKI
Geotechnical Software

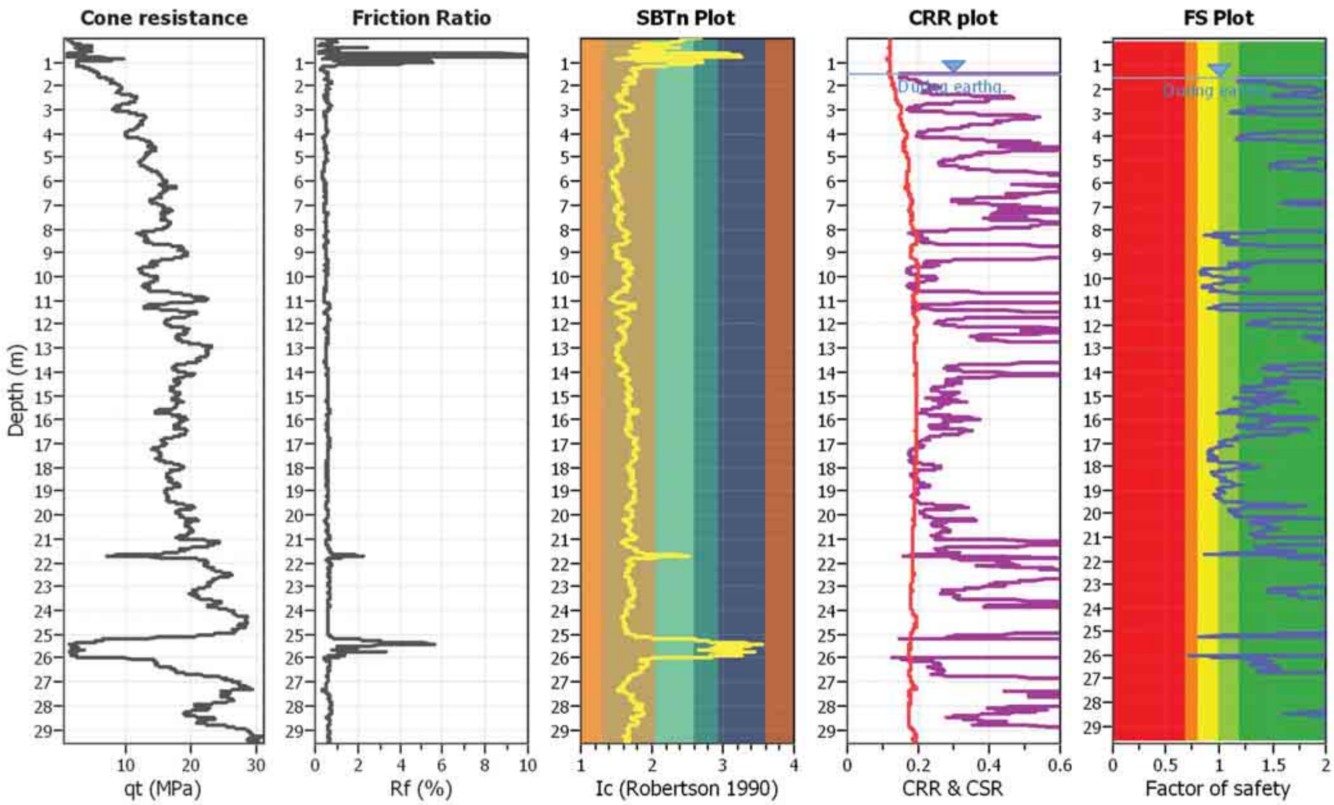
LIQUEFACTION ANALYSIS REPORT

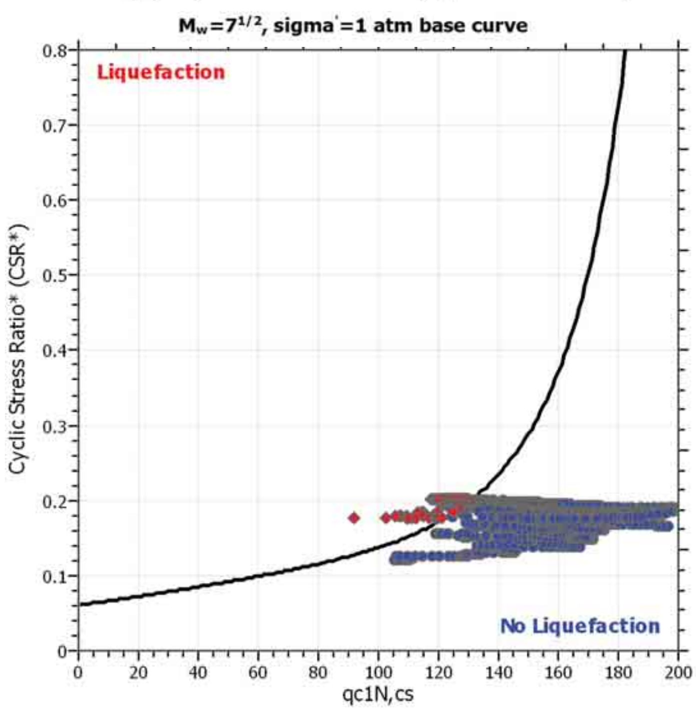
Project title : G13AP00029 **Location : Christchurch, New Zealand**

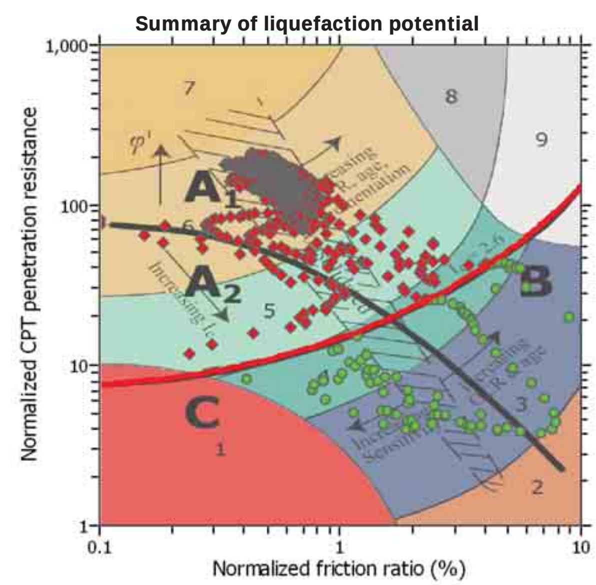
CPT file : NNBS_CPT33695(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.21	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

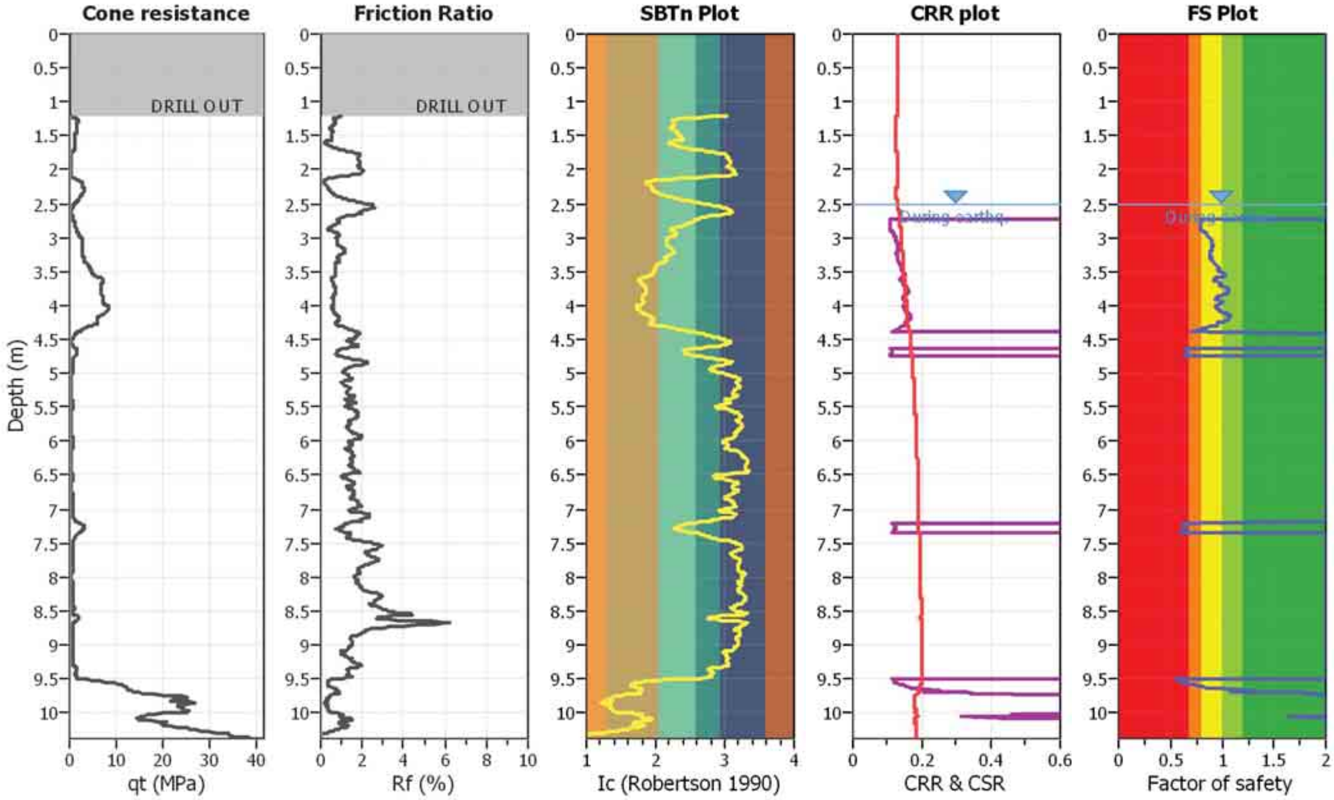
LIQUEFACTION ANALYSIS REPORT

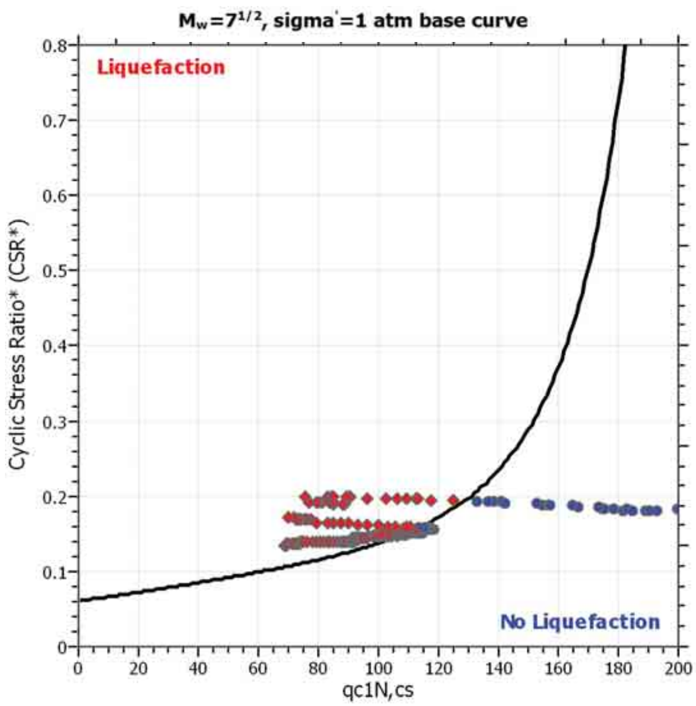
Project title : G13AP00029 **Location : Christchurch, New Zealand**

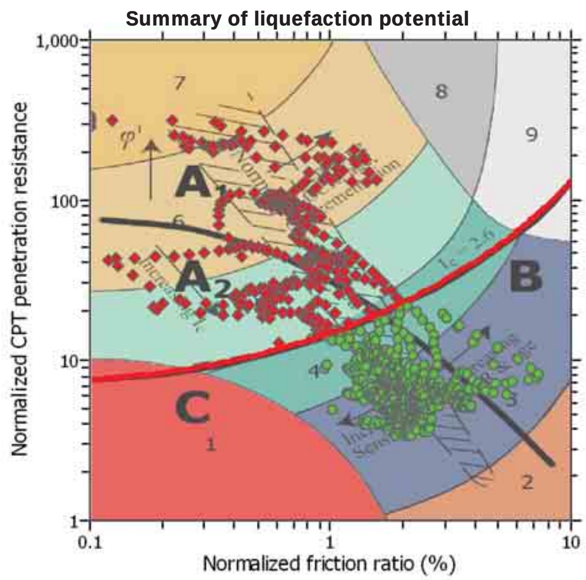
CPT file : PPHS_CPT1497(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.22	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		







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

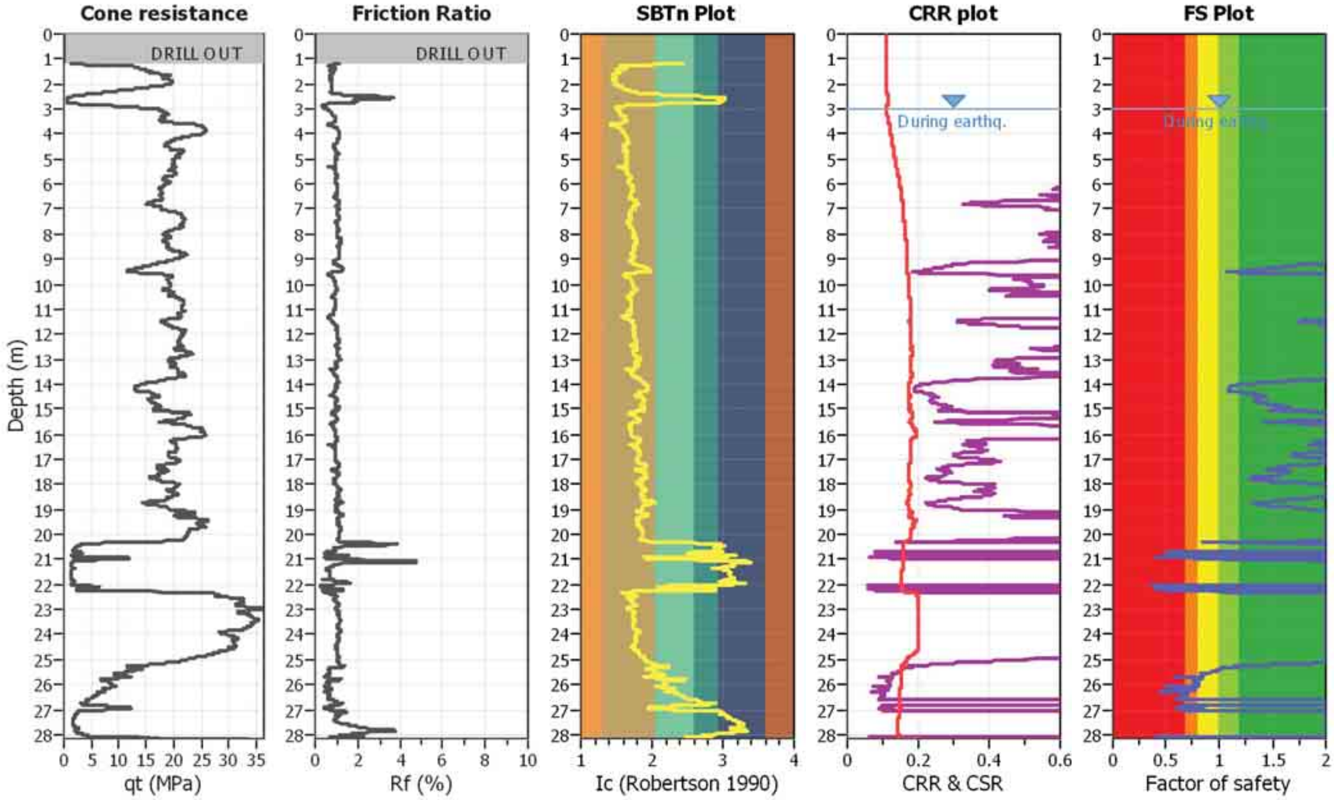
LIQUEFACTION ANALYSIS REPORT

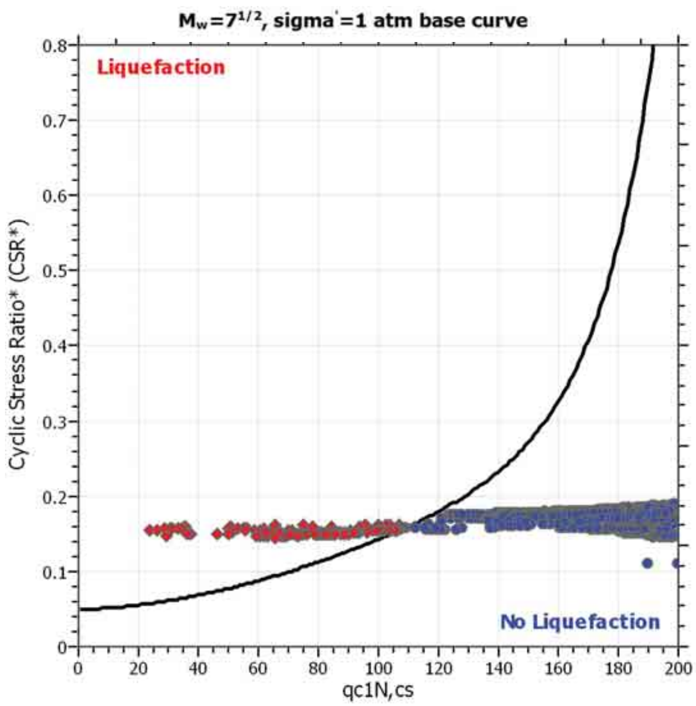
Project title : G13AP00029 **Location : Christchurch**

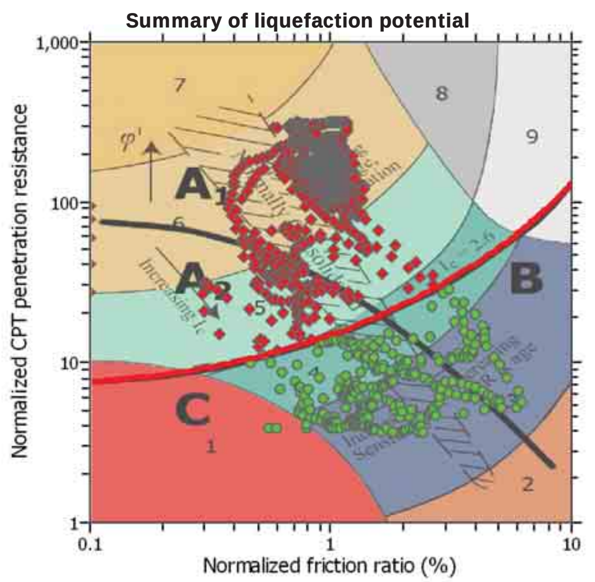
CPT file : PRPC_CPT1396 (CGD)

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	3.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	3.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.21	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

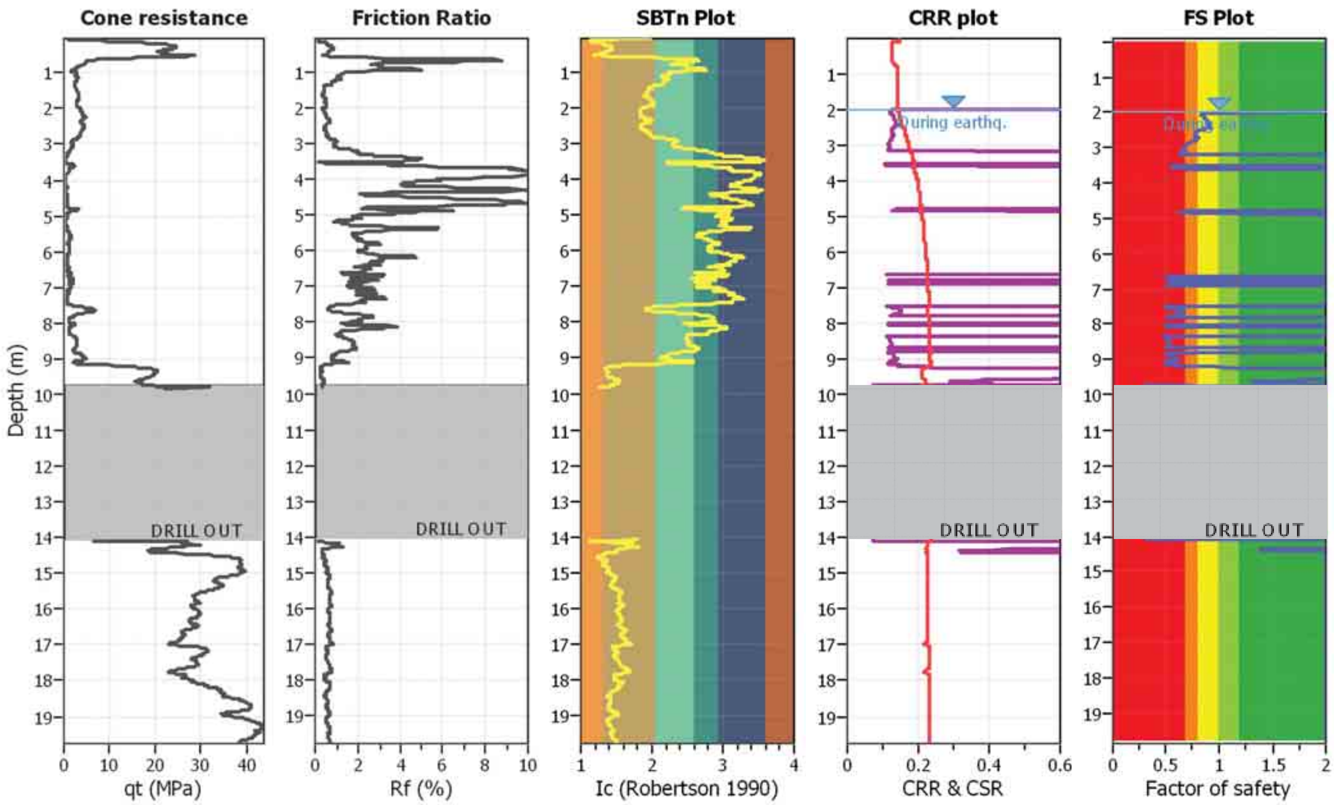
LIQUEFACTION ANALYSIS REPORT

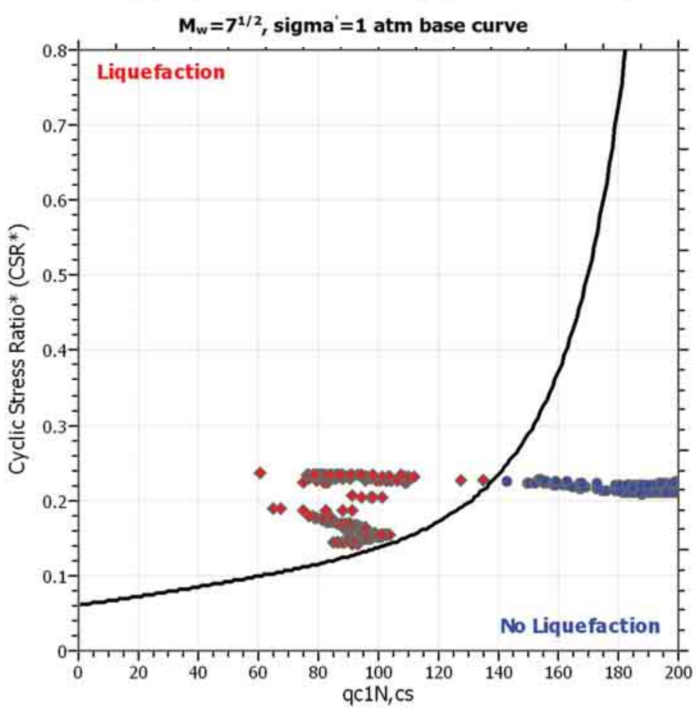
Project title : G13AP00029 **Location : Christchurch, New Zealand**

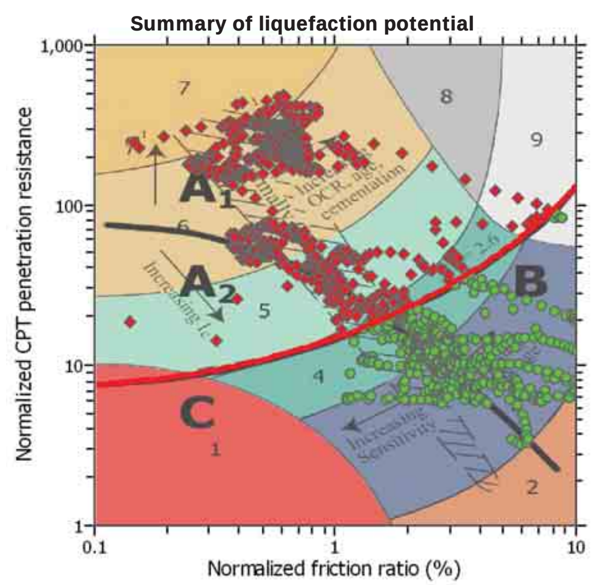
CPT file : REHS_CPT2 (Wotherspoon,2013)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.25	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

GEOLOGISMIKI Geotechnical Software

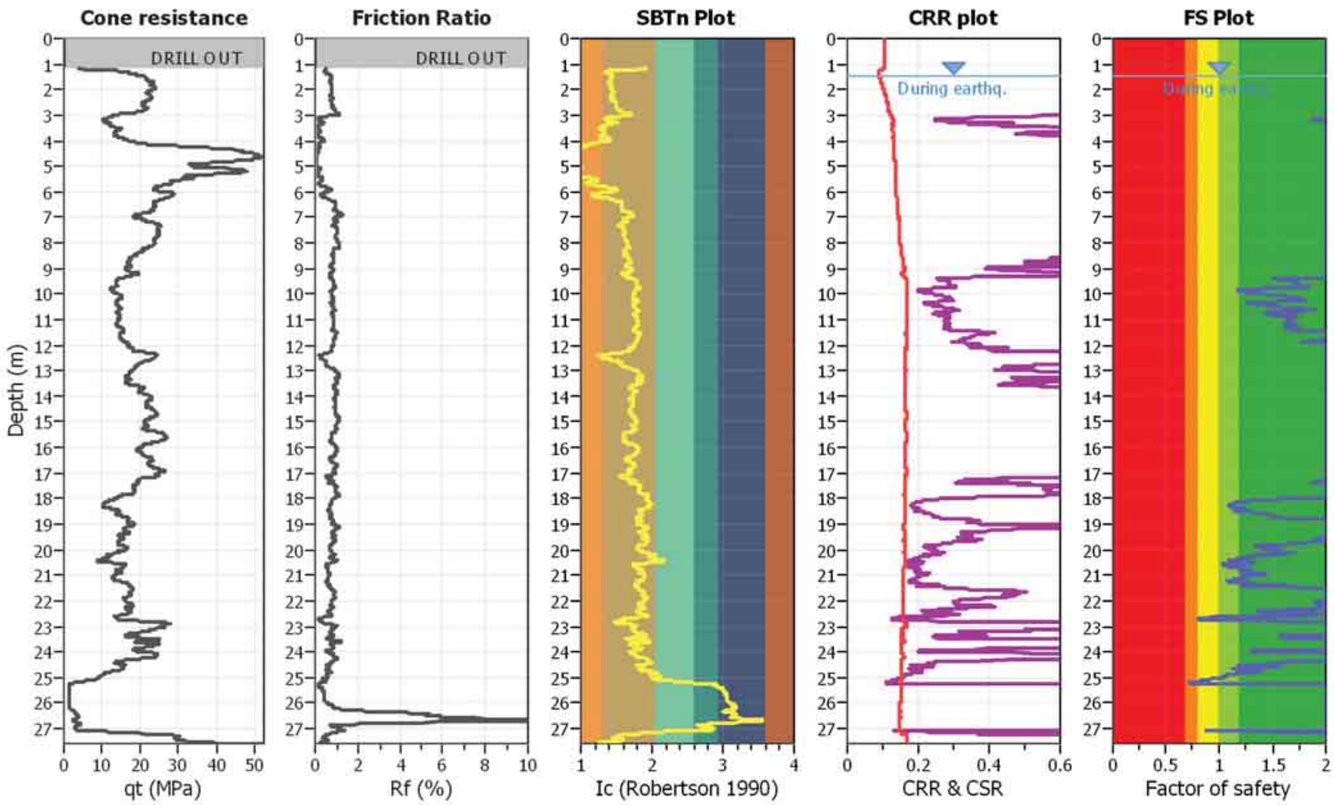
LIQUEFACTION ANALYSIS REPORT

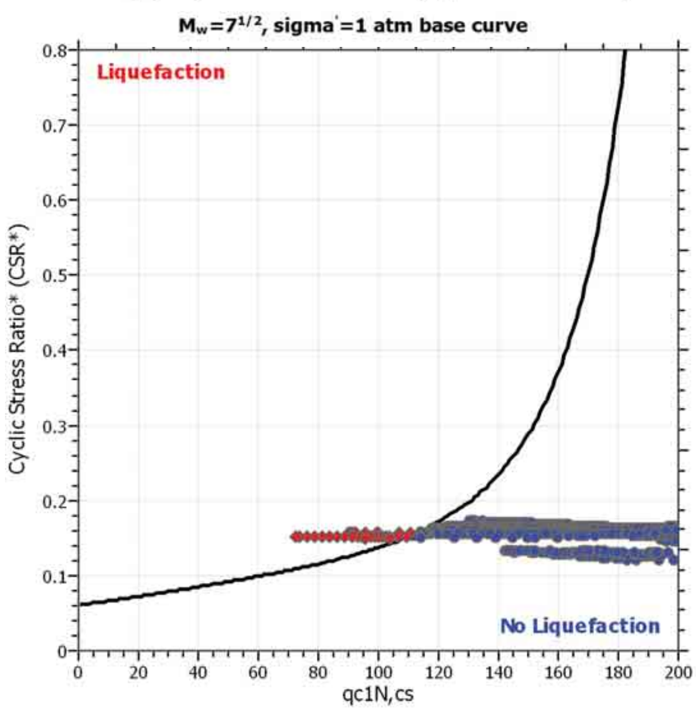
Project title : G13AP00029 **Location : Christchurch, New Zealand**

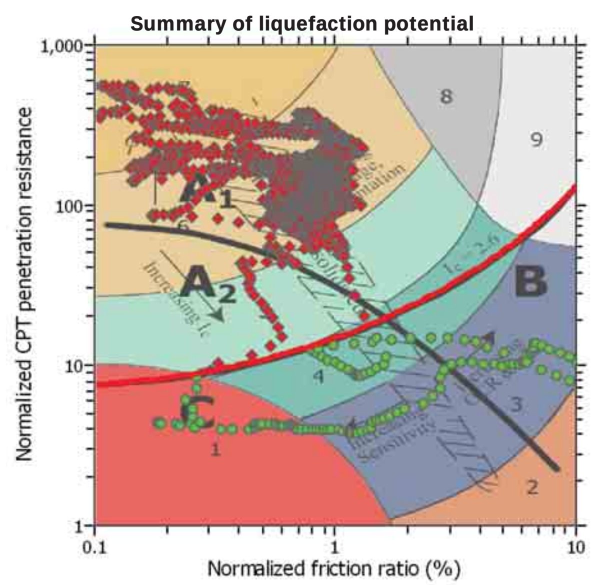
CPT file : SHLC_CPT626(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.18	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

APPENDIX E.3

Liquefaction Triggering Analyses 26 December 2010 Event.

GEOLOGISMIKI
Geotechnical Software

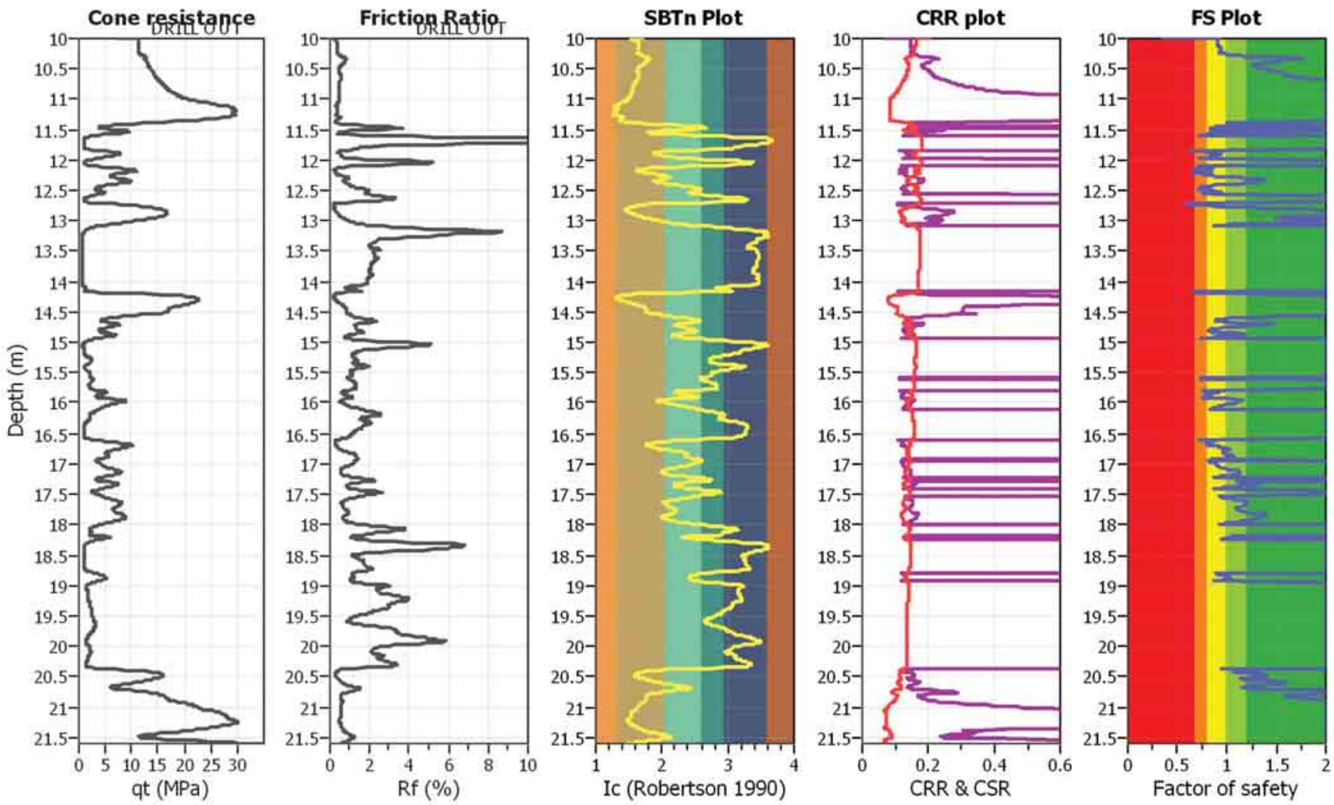
LIQUEFACTION ANALYSIS REPORT

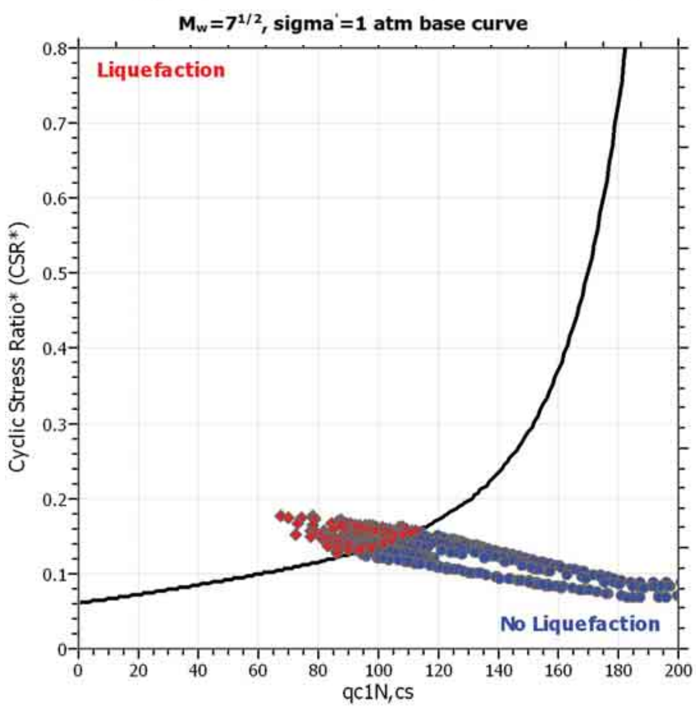
Project title : G13AP00029 **Location : Christchurch, New Zealand**

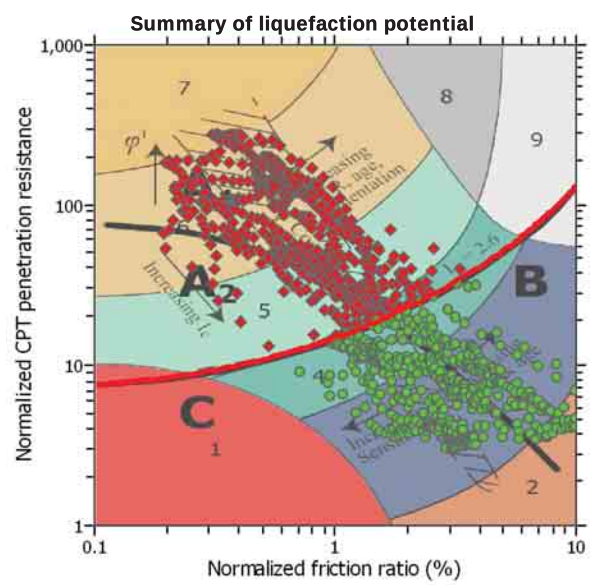
CPT file : CBGS_CPT1(Wotherspoon,2013)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.27	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		







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

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Geotechnical Software

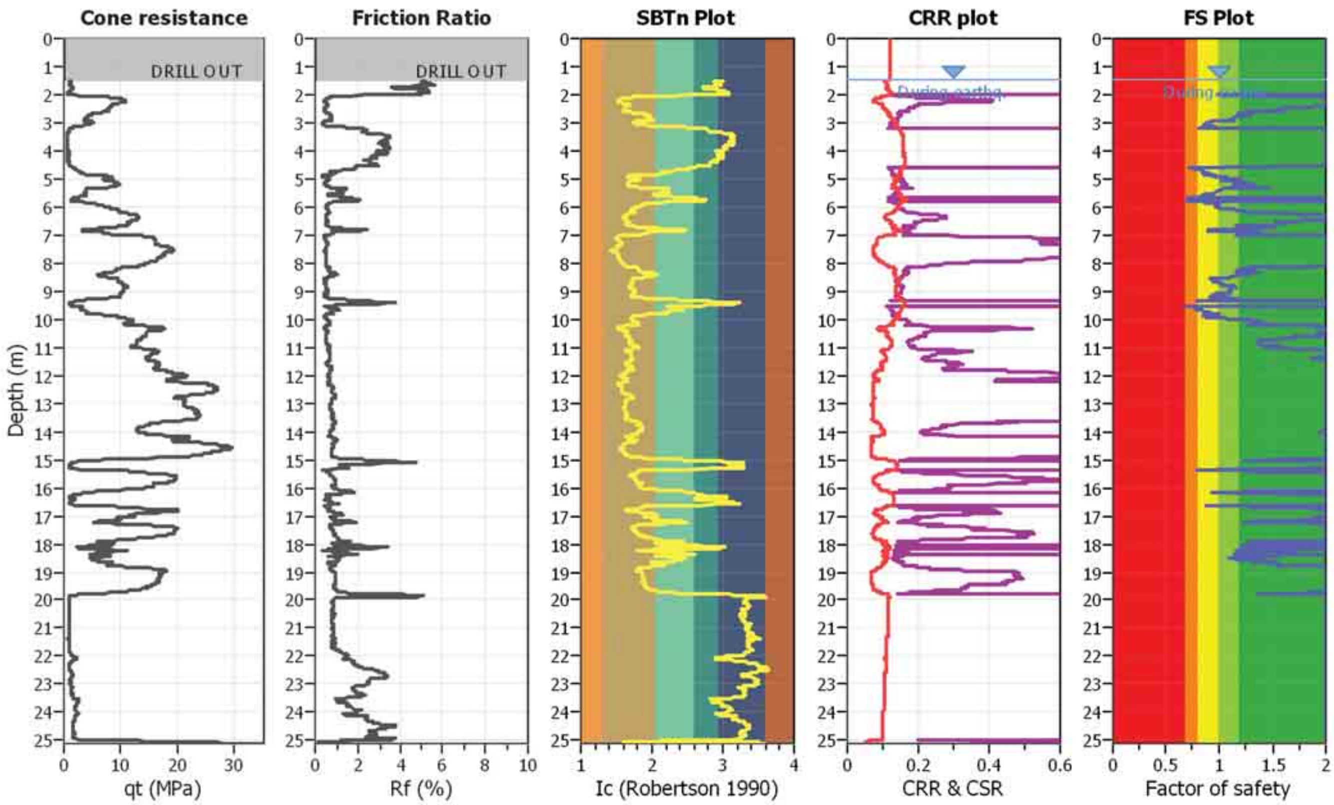
LIQUEFACTION ANALYSIS REPORT

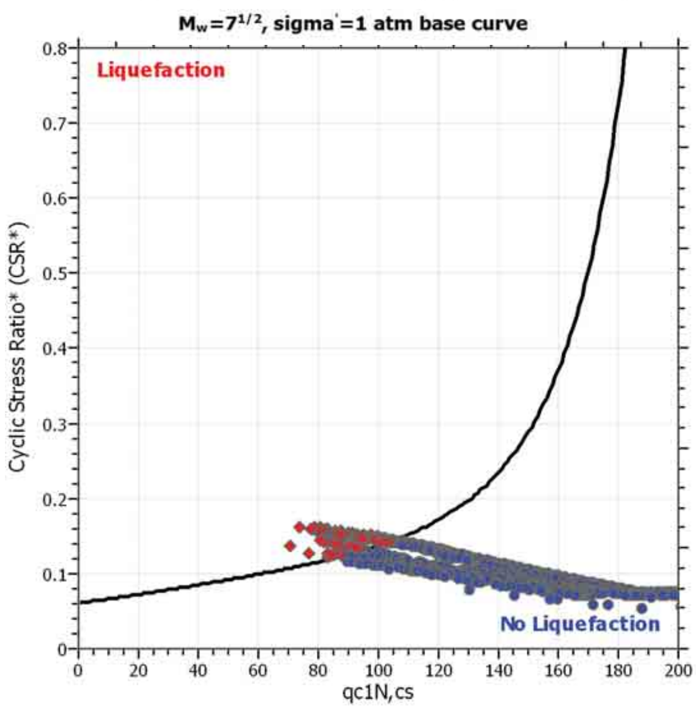
Project title : G13AP00029 **Location : Christchurch, New Zealand**

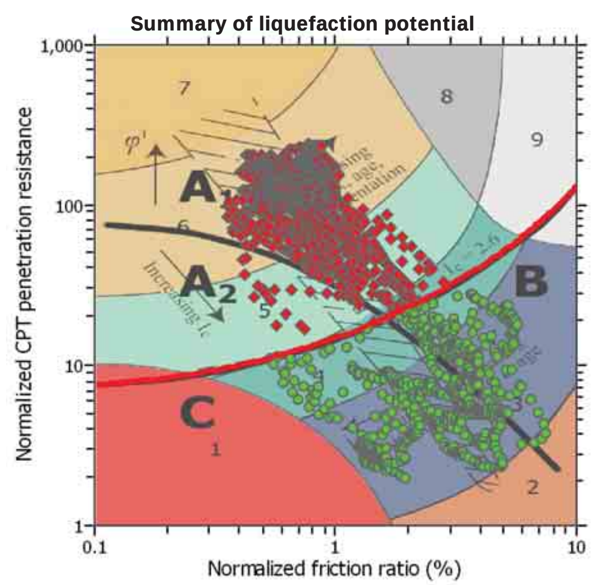
CPT file : CCCC_CPT484(CBD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.23	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

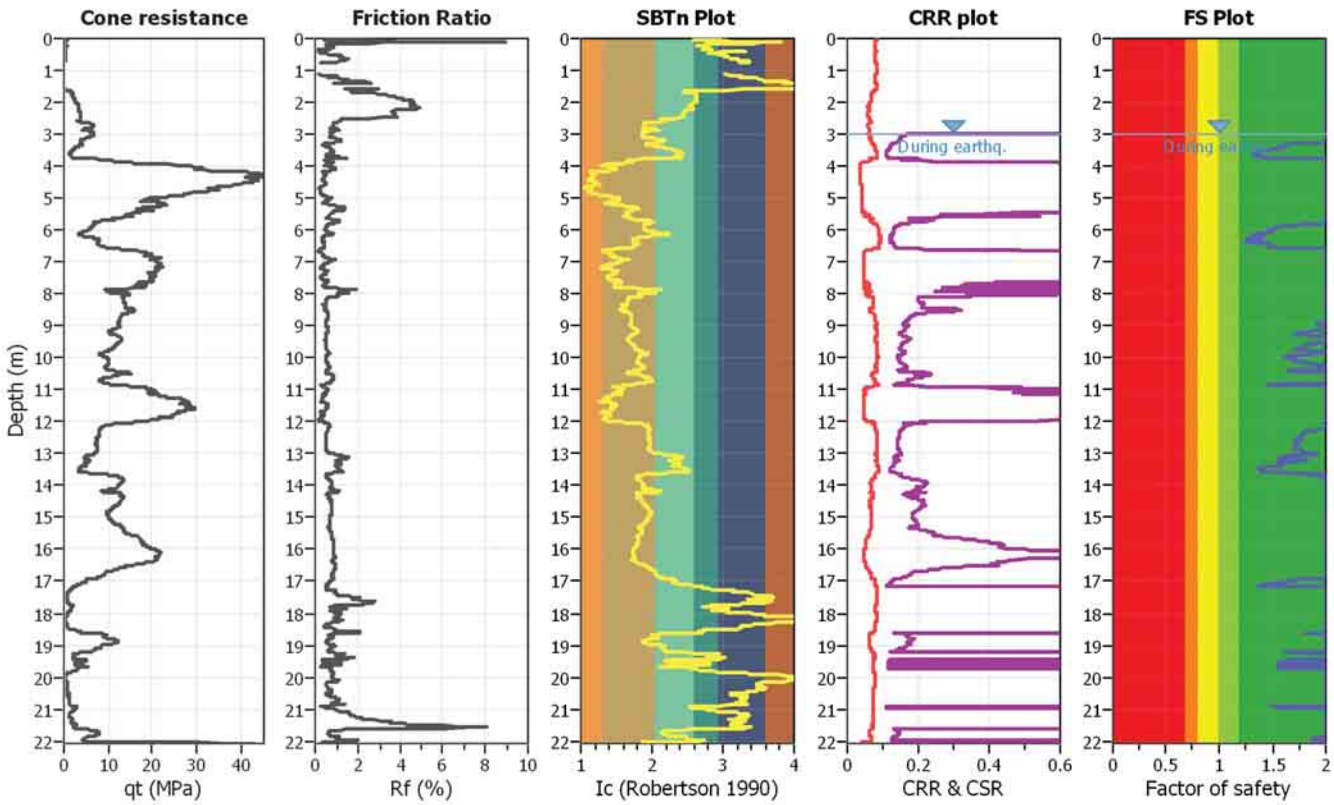
LIQUEFACTION ANALYSIS REPORT

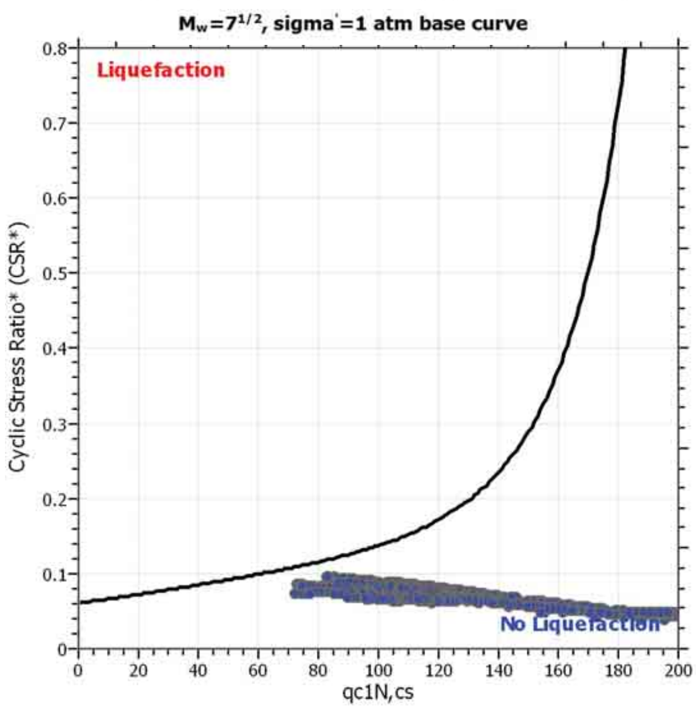
Project title : G13AP00029 **Location : Christchurch, New Zealand**

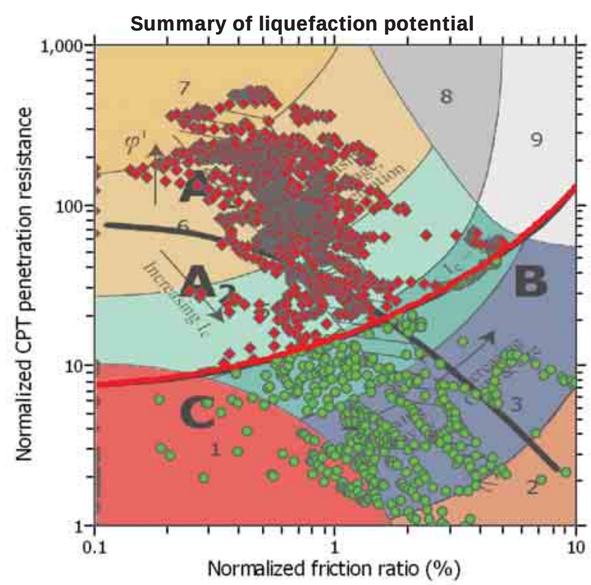
CPT file : CHHC_CPT425(CBD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	3.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	3.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.16	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

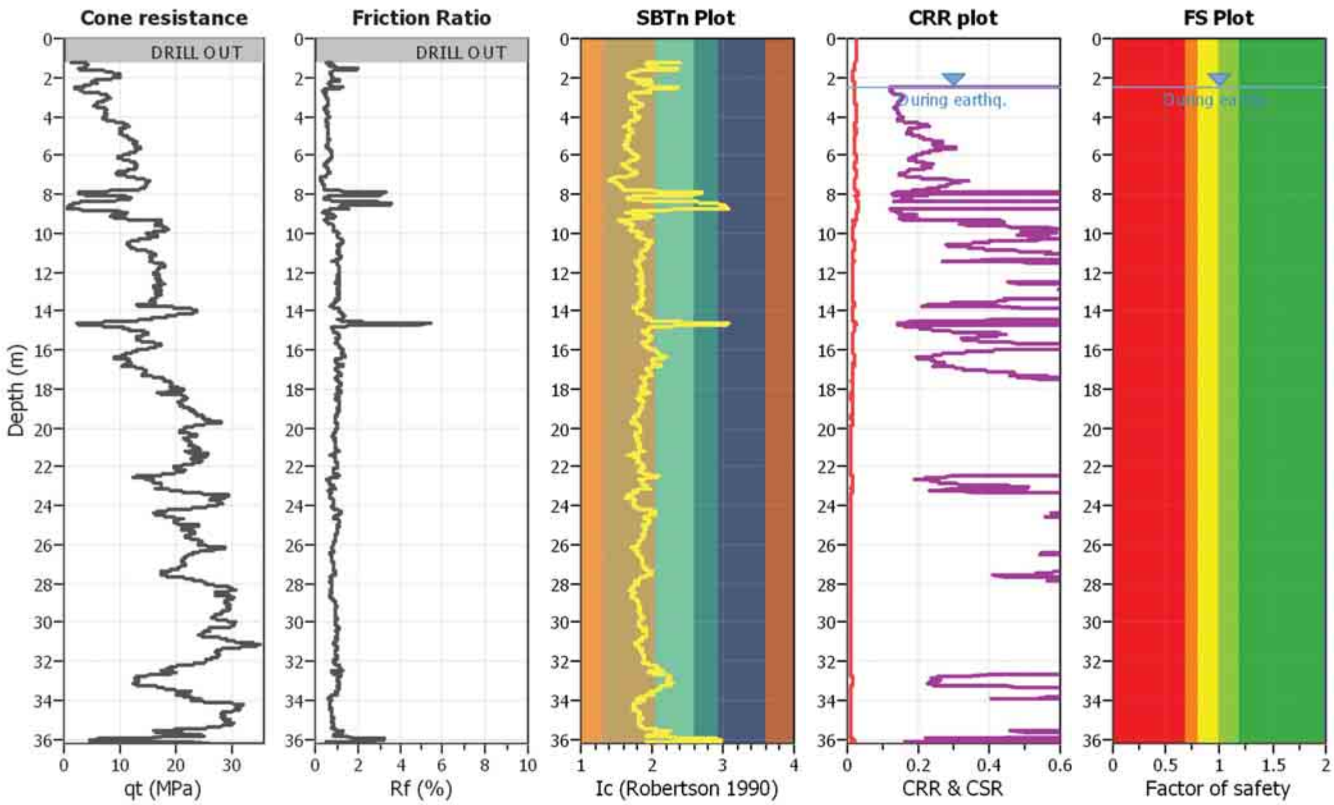
LIQUEFACTION ANALYSIS REPORT

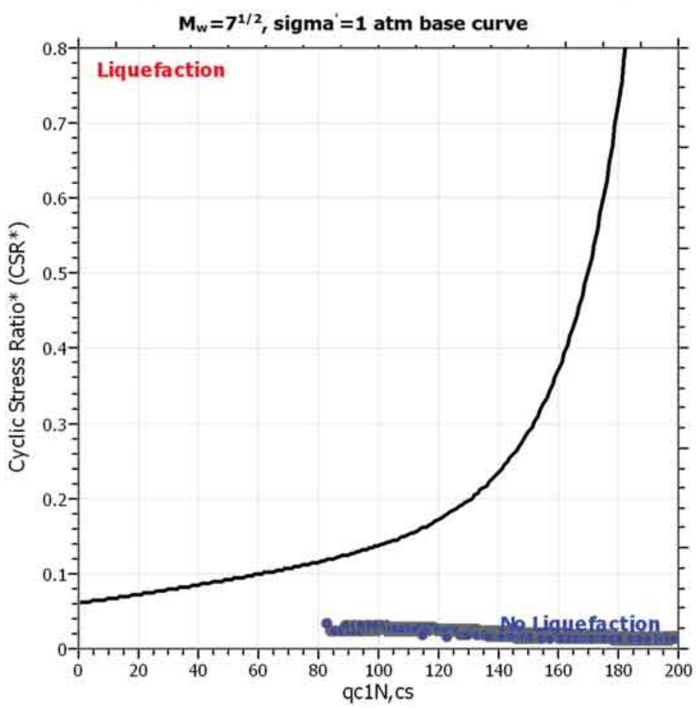
Project title : G13AP00029 **Location : Christchurch, New Zealand**

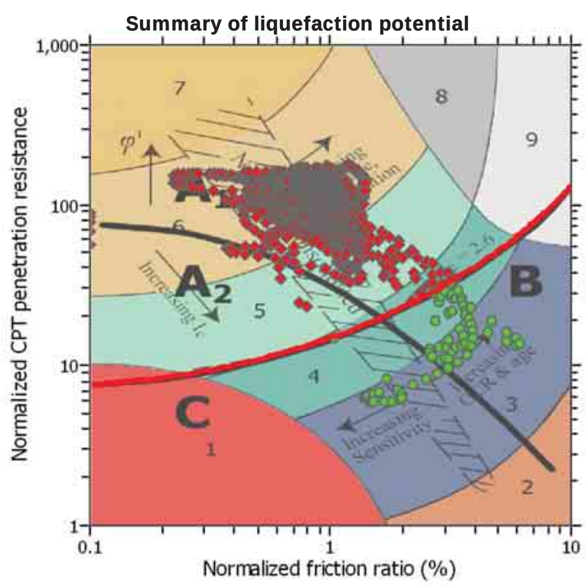
CPT file : HPSC_CPT89(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.05	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		







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

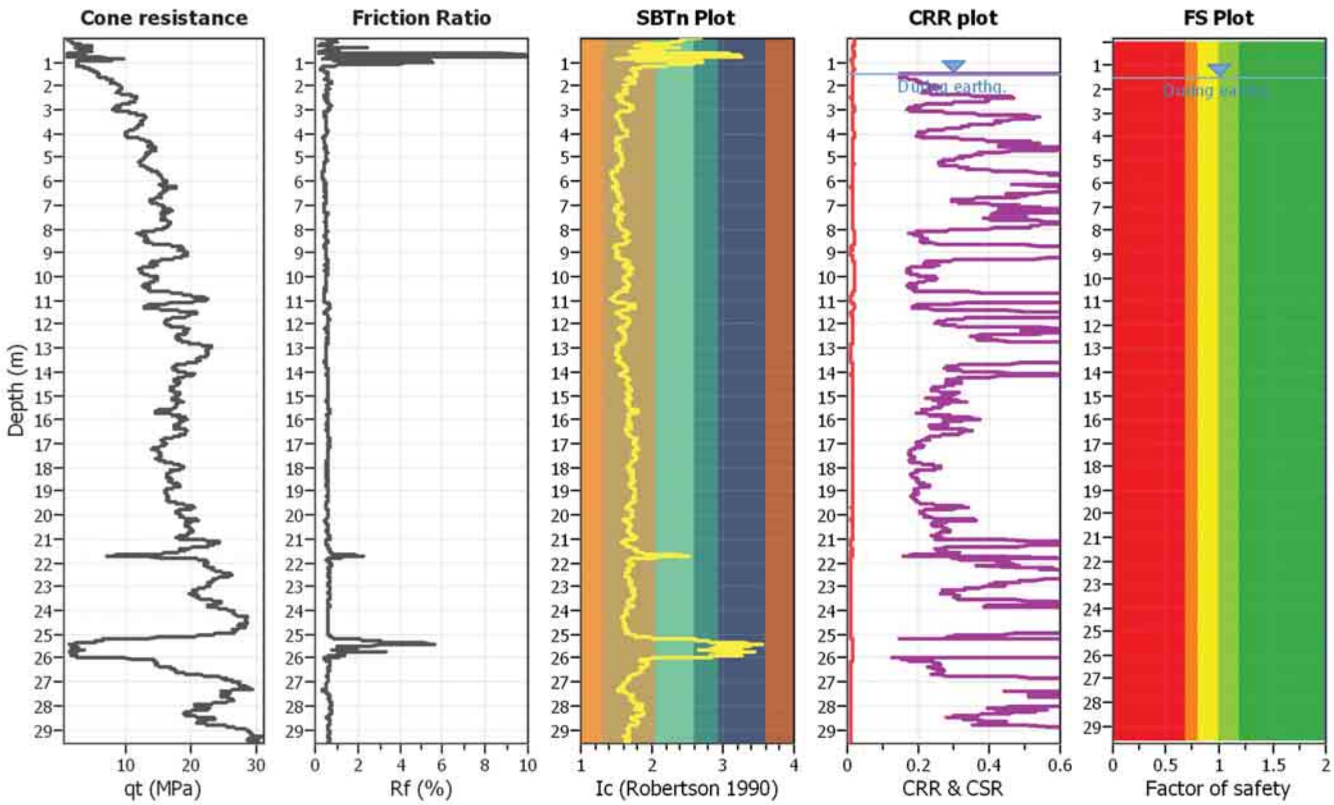
LIQUEFACTION ANALYSIS REPORT

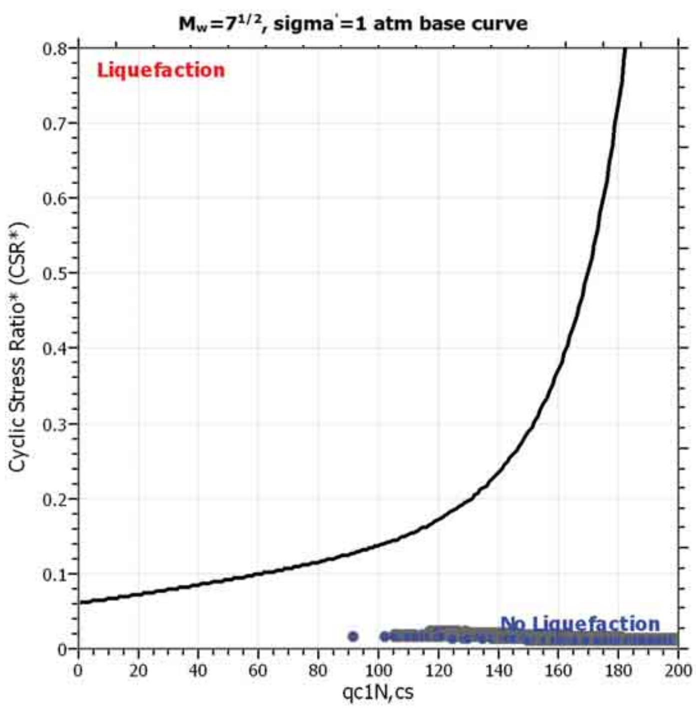
Project title : G13AP00029 **Location : Christchurch, New Zealand**

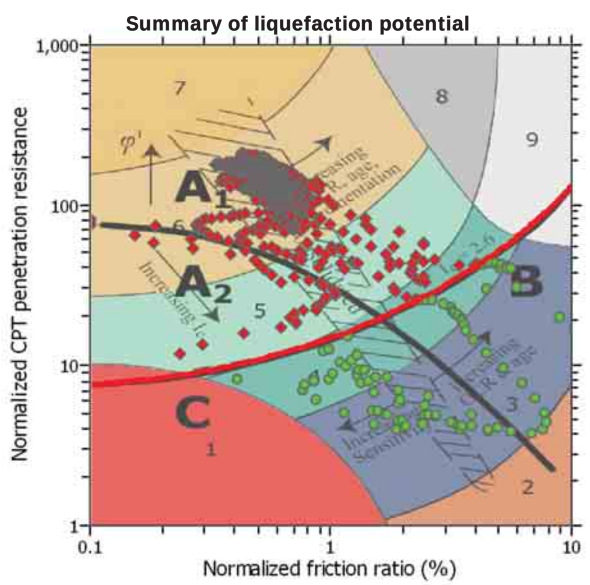
CPT file : NNBS_CPT33695(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.04	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

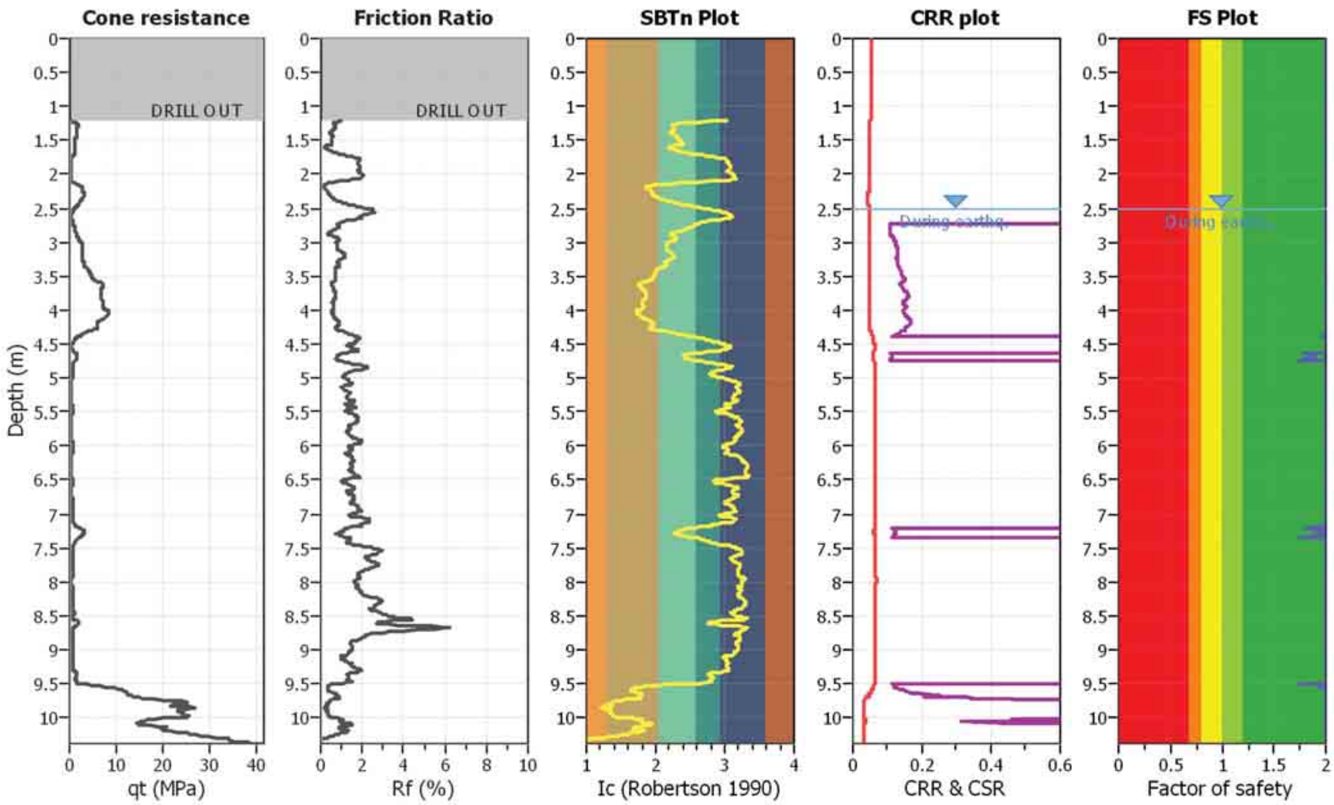
LIQUEFACTION ANALYSIS REPORT

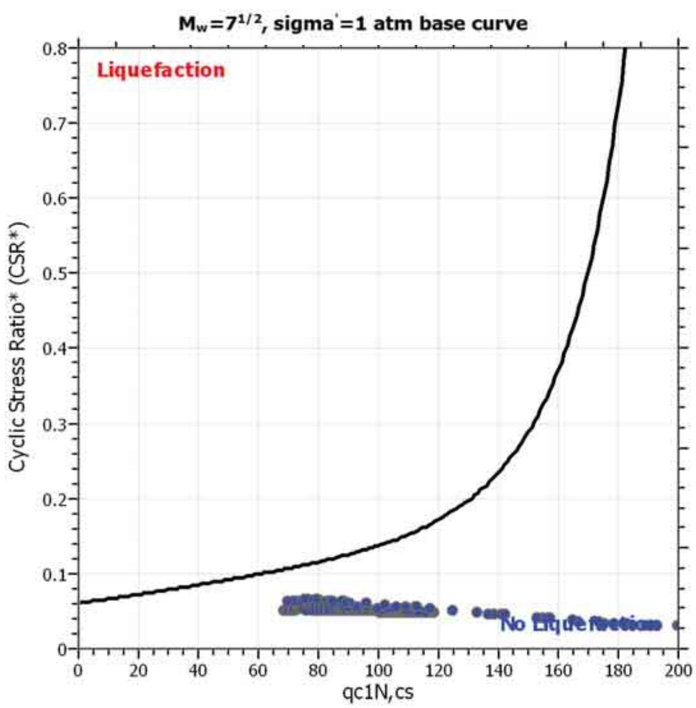
Project title : G13AP00029 **Location : Christchurch, New Zealand**

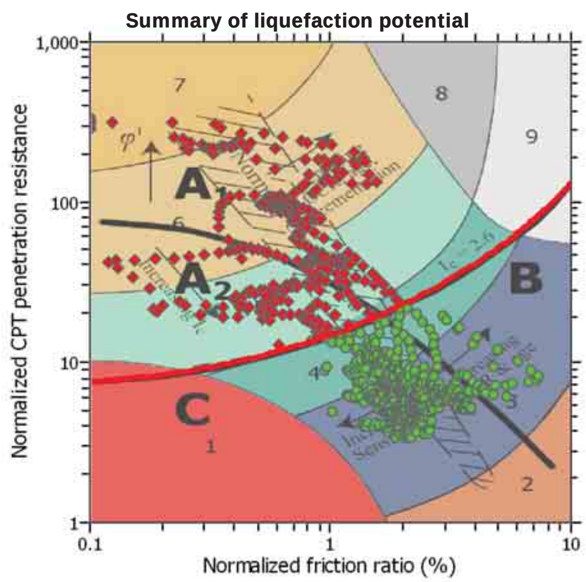
CPT file : PPHS_CPT1497(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.10	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

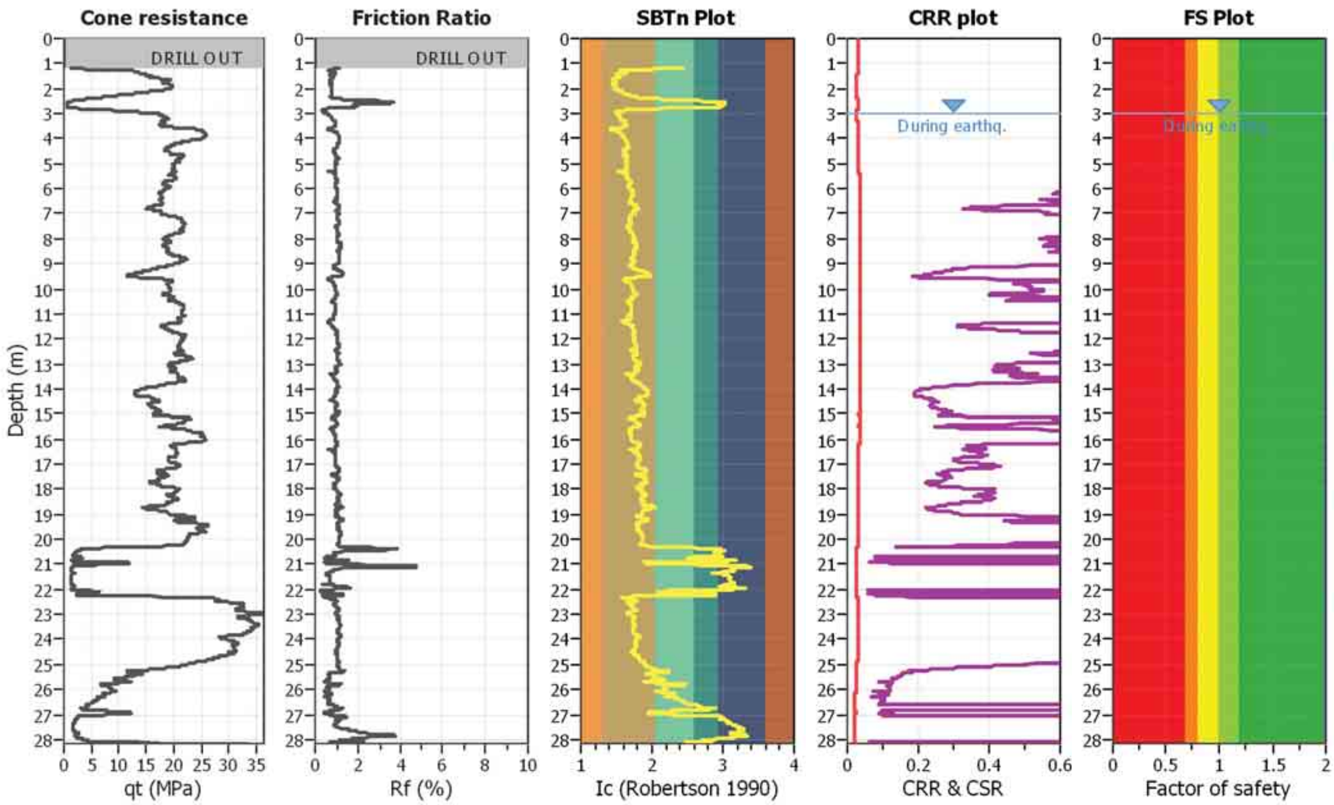
LIQUEFACTION ANALYSIS REPORT

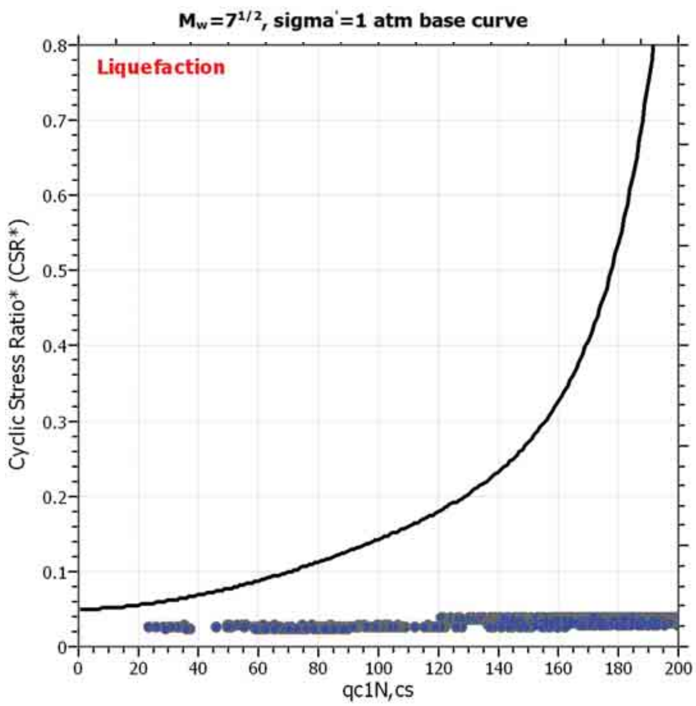
Project title : G13AP00029 **Location : Christchurch**

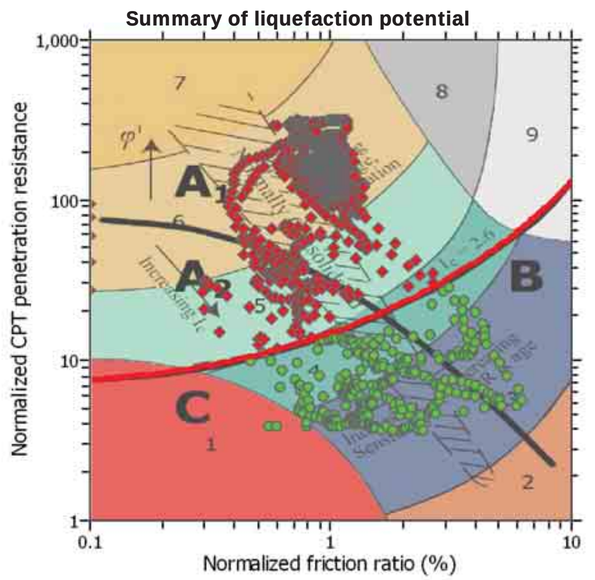
CPT file : PRPC_CPT1396 (CGD)

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	3.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	3.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.09	Unit weight calculation:	19.00 kN/m³	K_σ applied:	Yes		







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

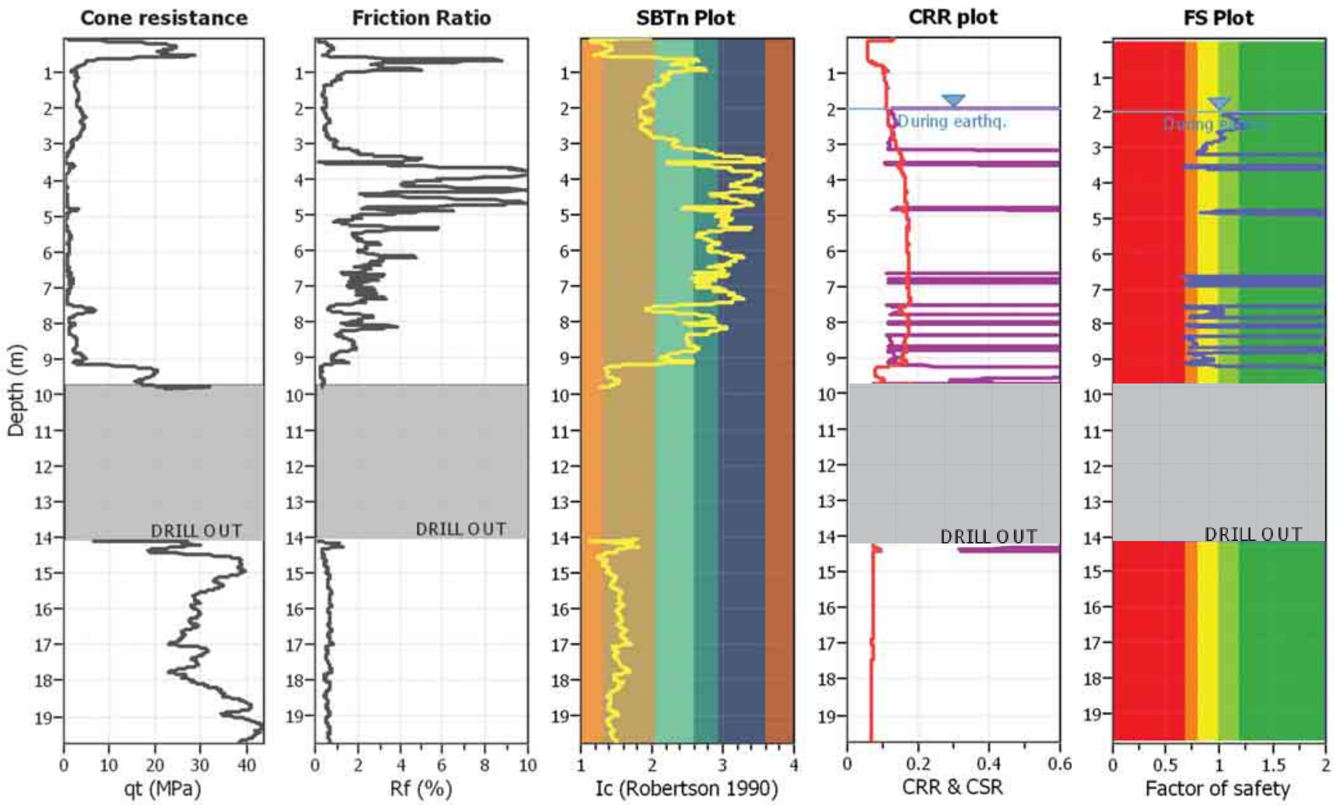
LIQUEFACTION ANALYSIS REPORT

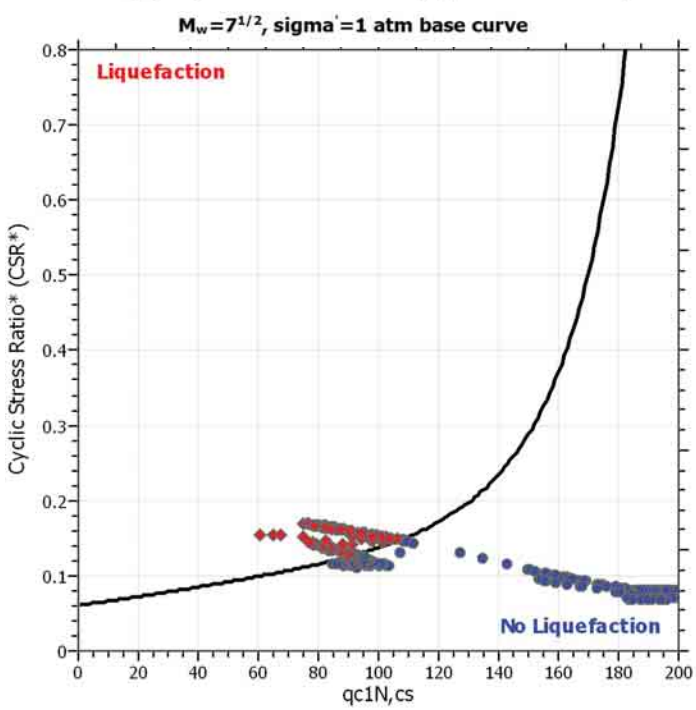
Project title : G13AP00029 **Location : Christchurch, New Zealand**

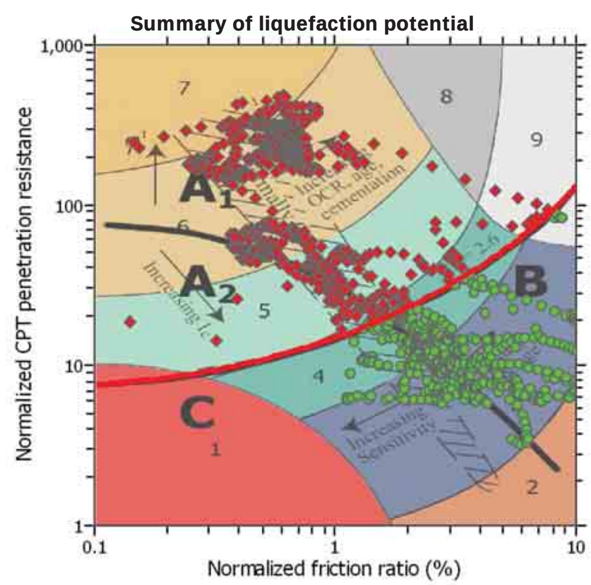
CPT file : REHS_CPT2 (Wotherspoon,2013)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.25	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

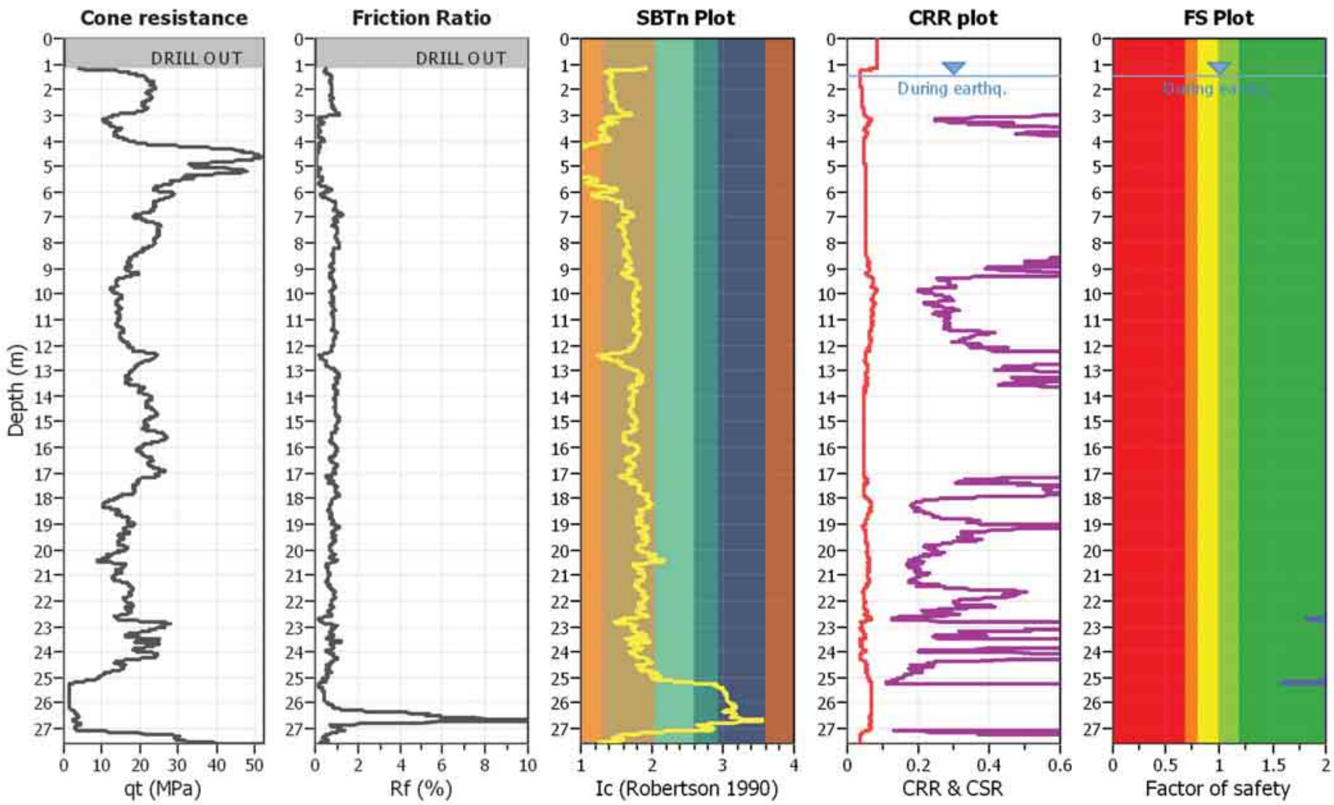
LIQUEFACTION ANALYSIS REPORT

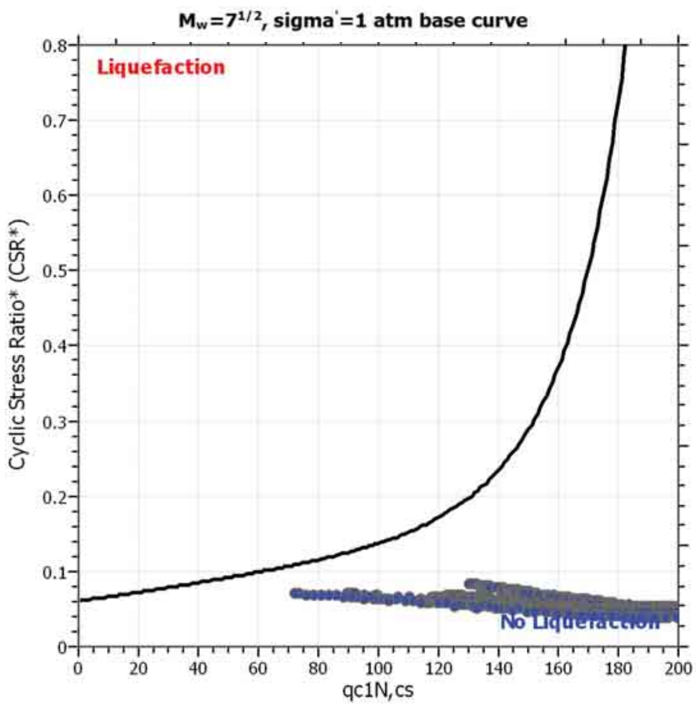
Project title : G13AP00029 **Location : Christchurch, New Zealand**

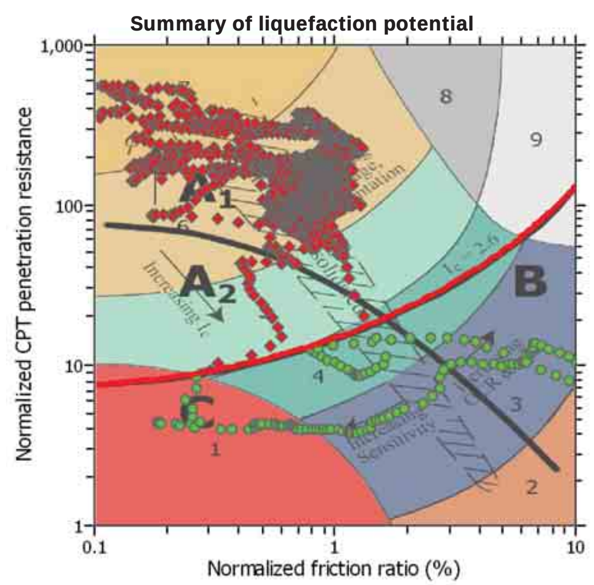
CPT file : SHLC_CPT626(CGD)

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	4.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.16	Unit weight calculation:	19.00 kN/m^3	K_σ applied:	Yes		







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

APPENDIX F

Residuals Between Actual Recorded Response Spectra and Seismic Site Response Analysis Estimated Response Spectra

F.1 Residuals for 22 February 2011 M_w 6.2 Event.

F.2 Residuals for 04 September 2010 M_w 7.1 Event.

F.3 Residuals for 13 June 2011 M_w 6.0 Event.

F.4 Residuals for 23 December 2011 M_w 5.8 Event.

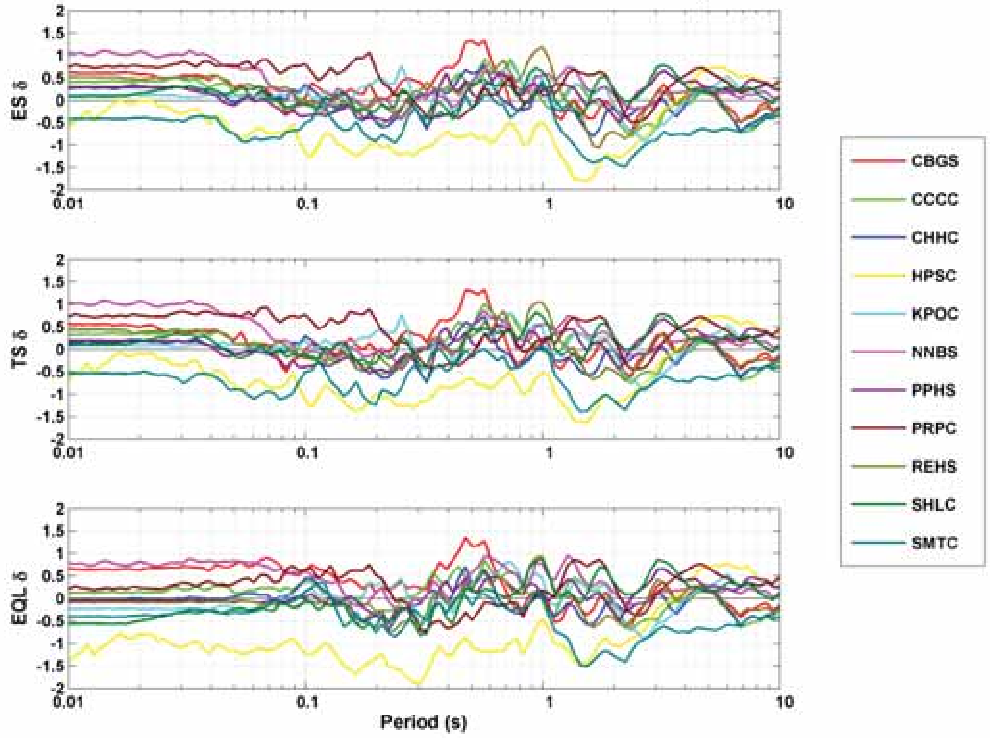
F.5 Residuals for 23 December 2011 M_w 5.9 Event.

F.6 Residuals for 26 December 2010 M_w 4.7 Event.

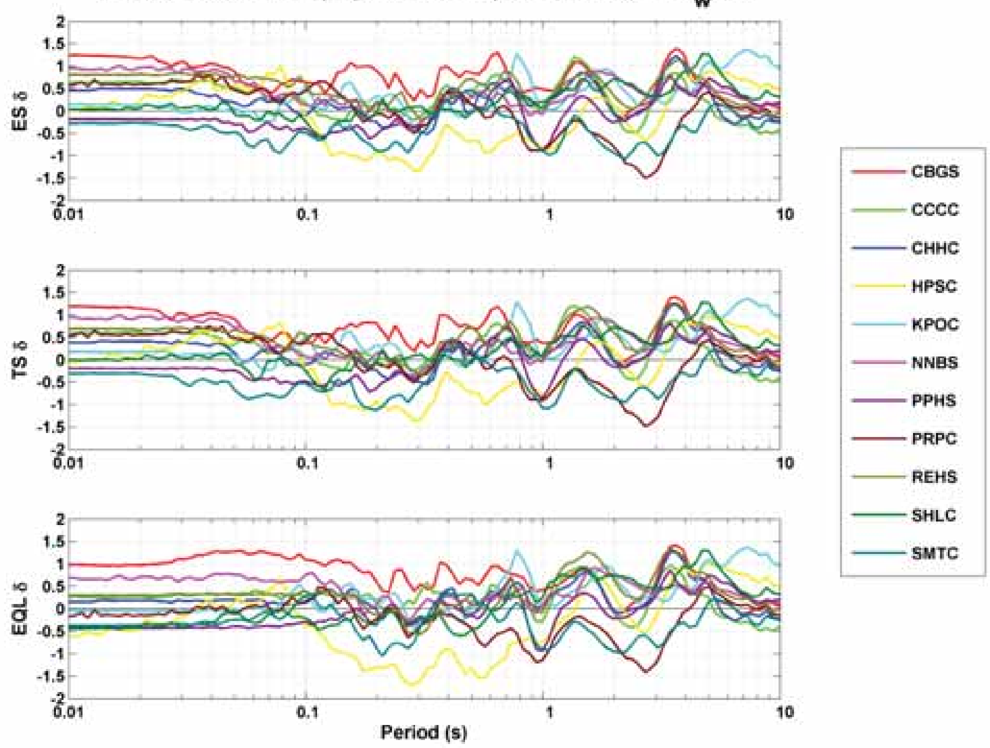
APPENDIX F.1

Residuals for 22 February 2011 M_w 6.2 Event.

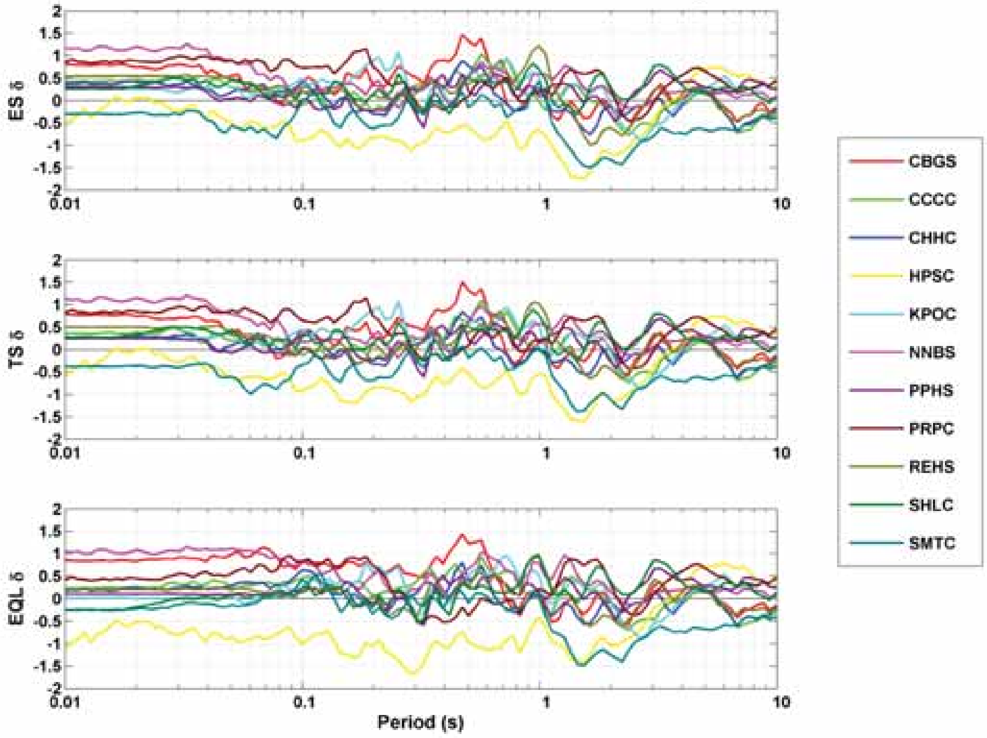
CACS Woth1 FN (input motion) for 22Feb11 M_w6.2



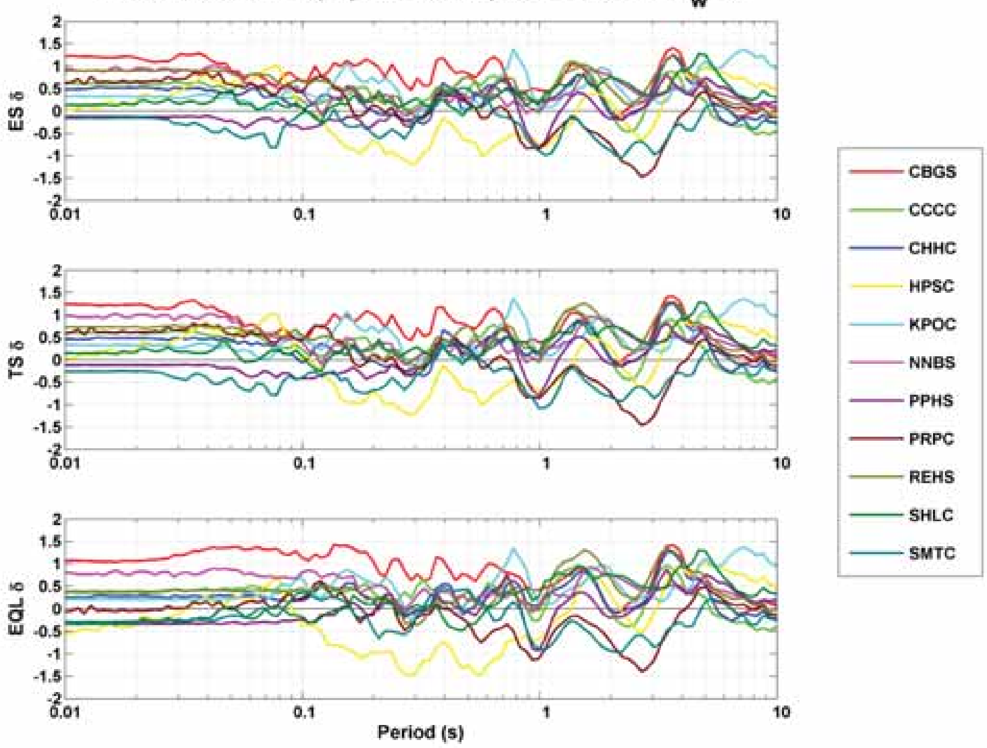
CACS Woth1 FP (input motion) for 22Feb11 M_w6.2

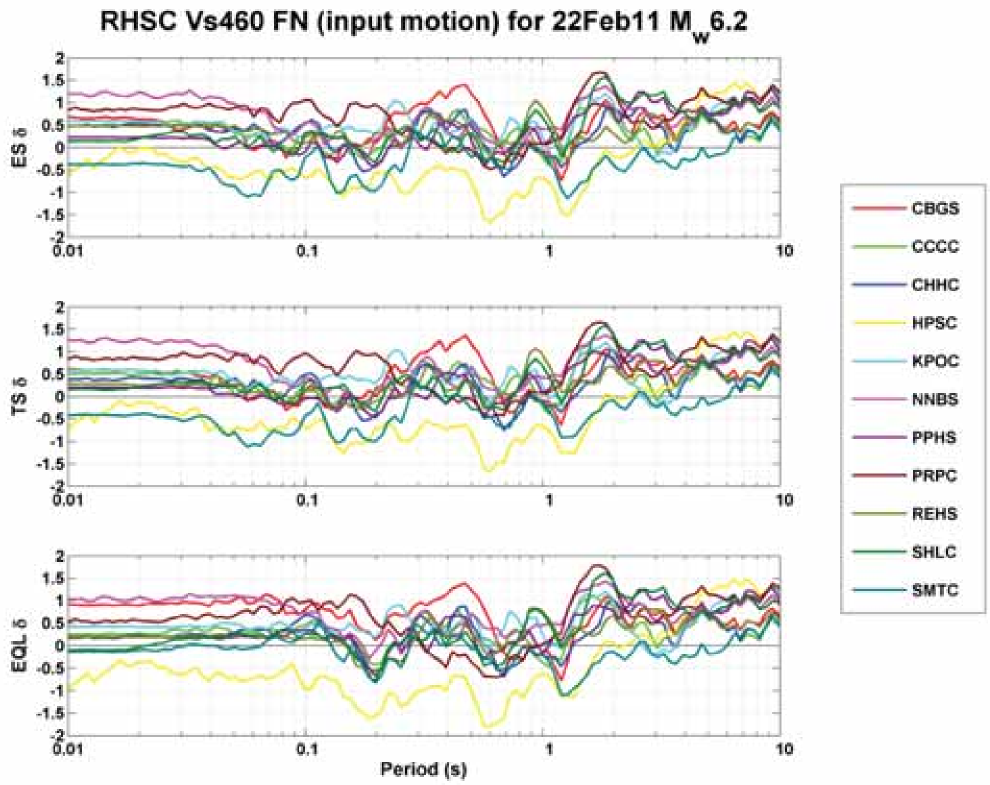


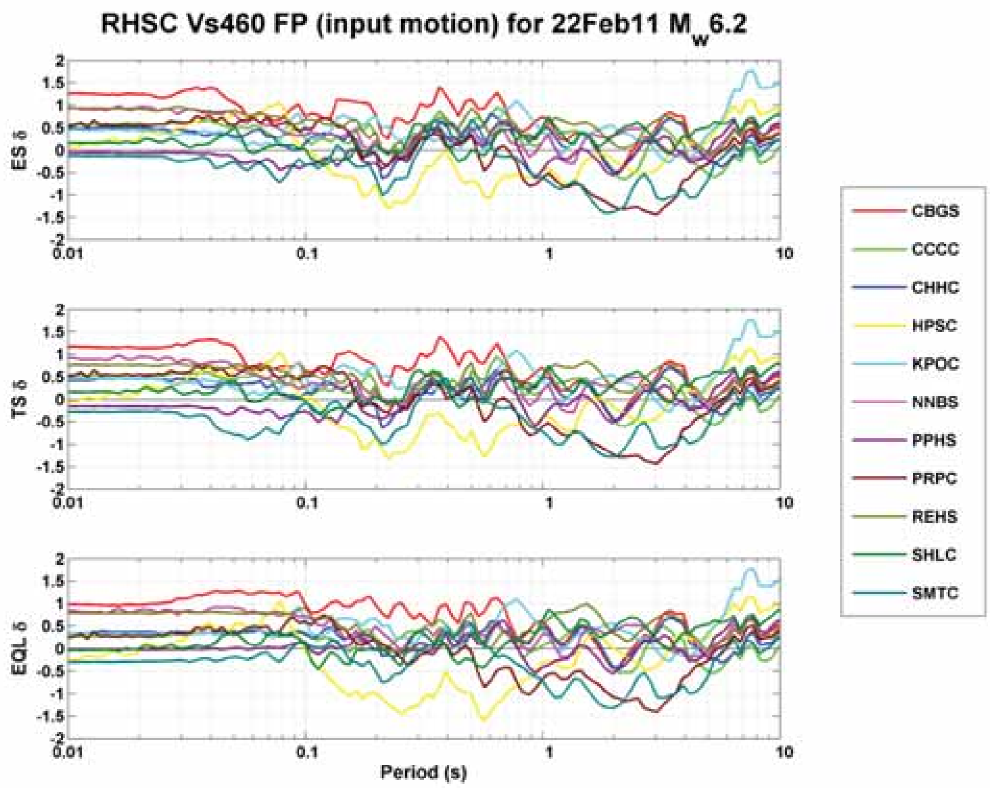
CACS Woth2 FN (input motion) for 22Feb11 M_w6.2

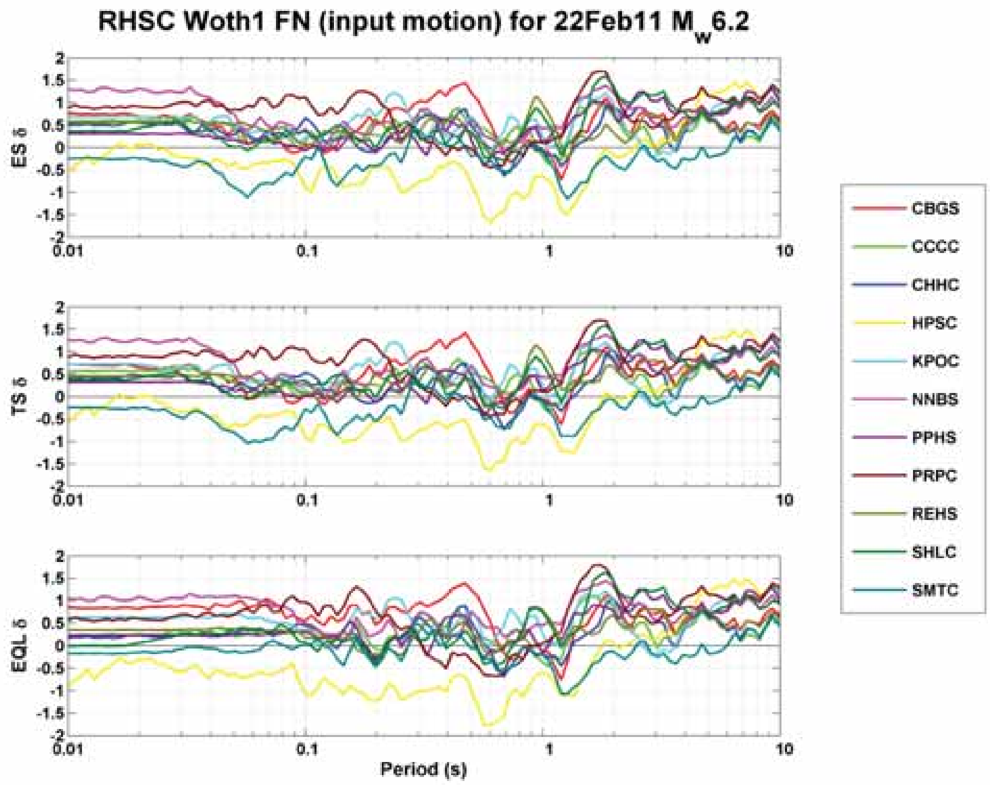


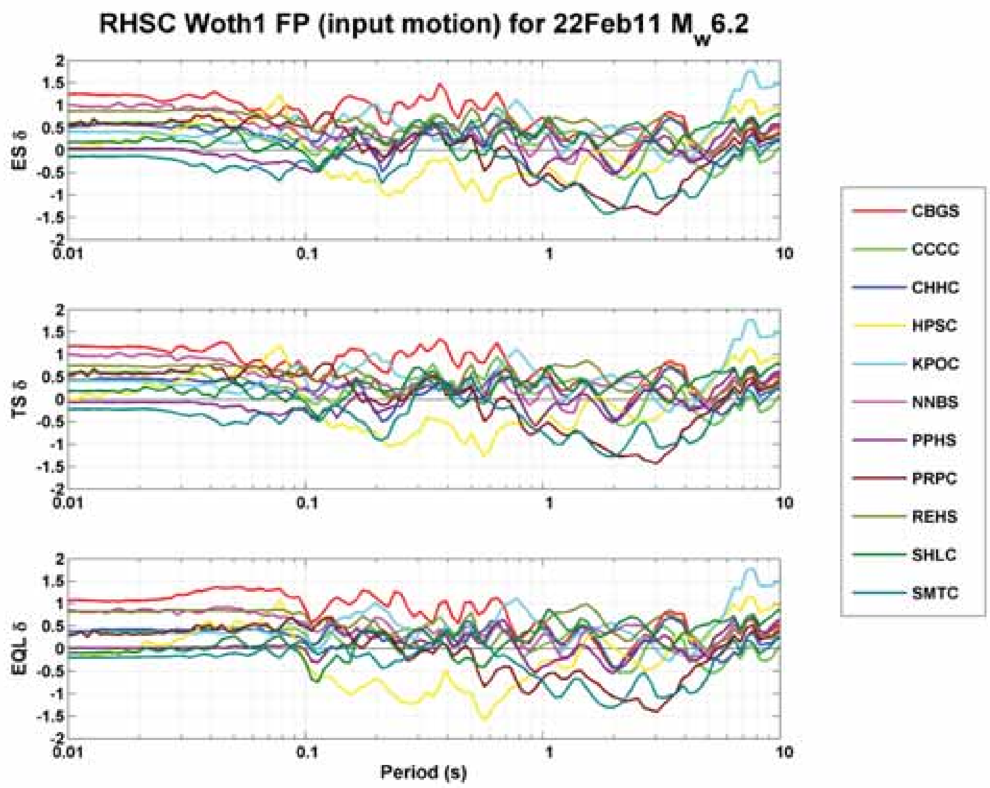
CACS Woth2 FP (input motion) for 22Feb11 M_w6.2




RHSC Vs460 FN (input motion) for 22Feb11 M$_w$6.2
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC


RHSC Vs460 FP (input motion) for 22Feb11 M$_w$6.2
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC

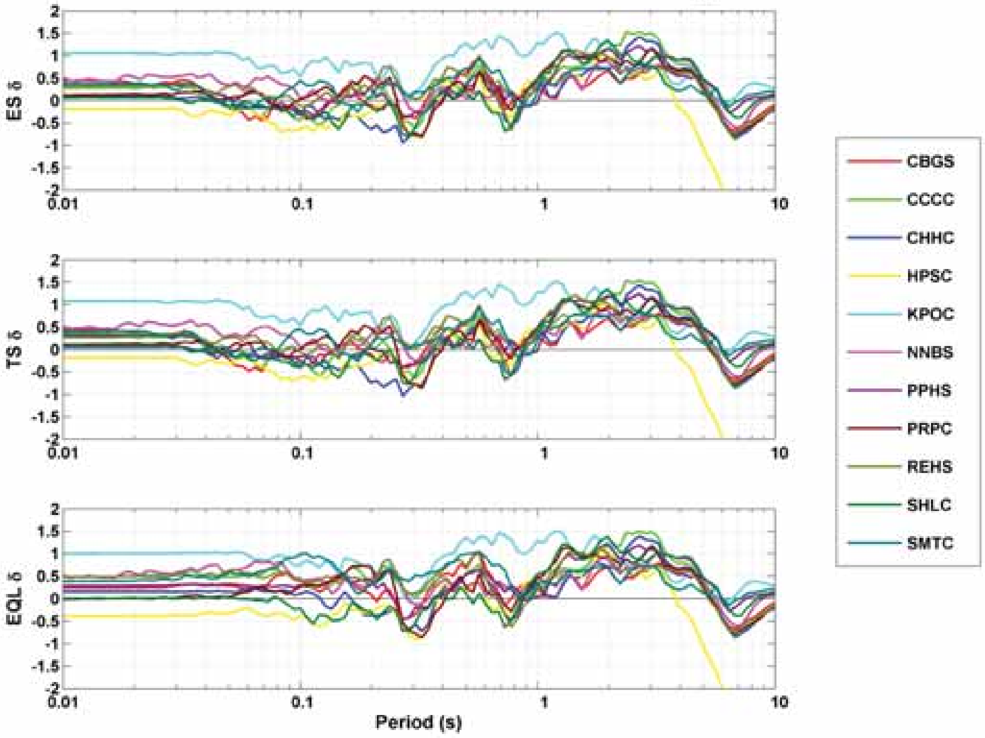

RHSC Woth1 FN (input motion) for 22Feb11 Mw6.2
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC


RHSC Woth1 FP (input motion) for 22Feb11 Mw6.2
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC

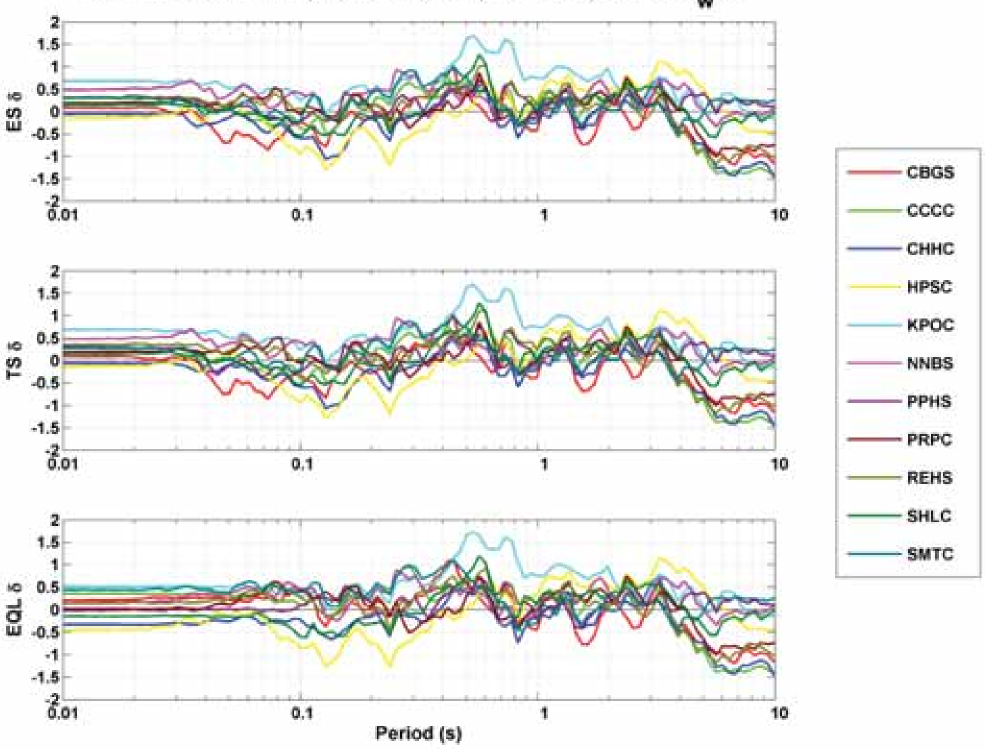
APPENDIX F.2

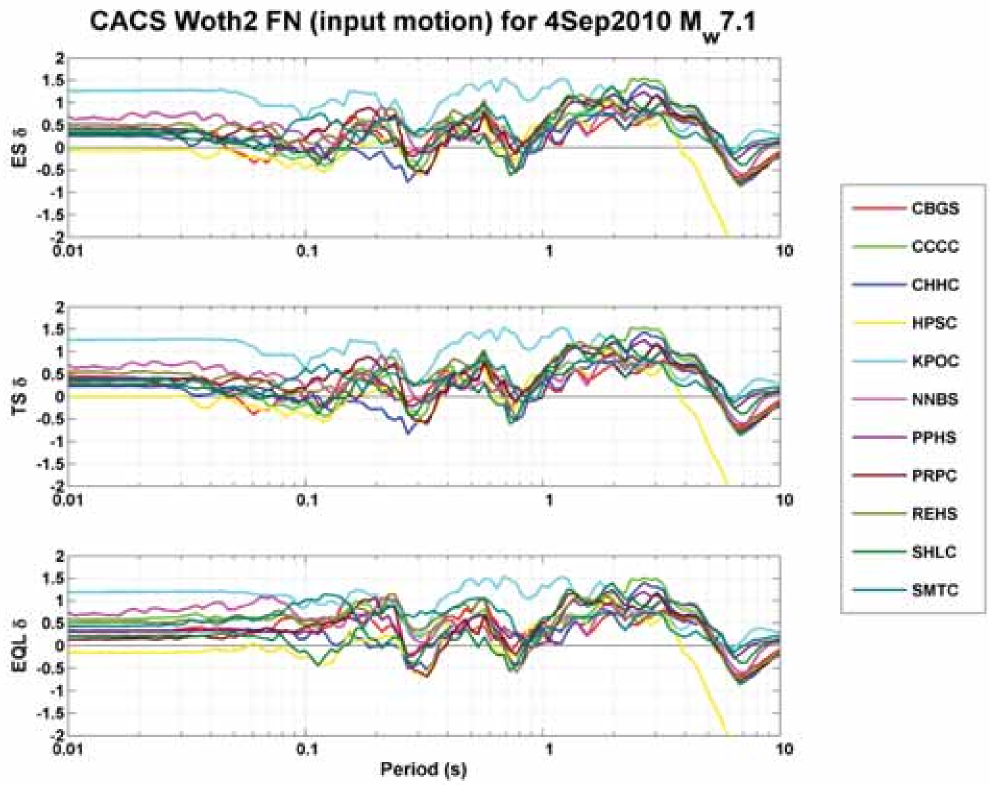
Residuals for 04 September 2010 M_w 7.1 Event.

CACS Woth1 FN (input motion) for 4Sep2010 M_w7.1

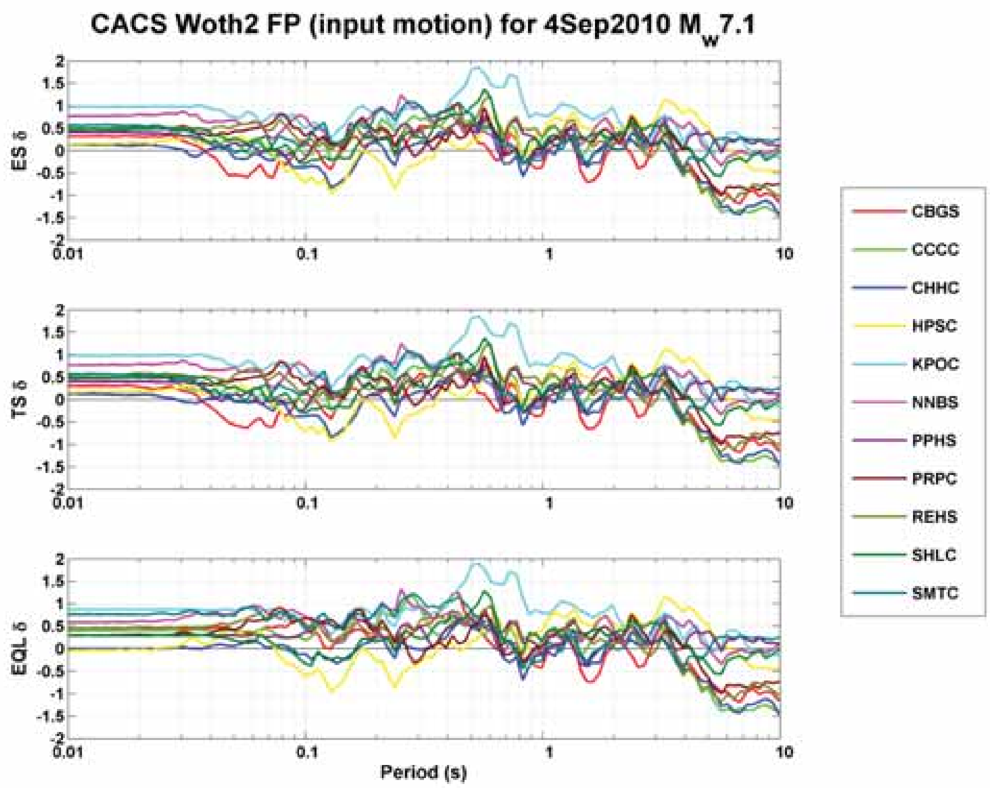


CACS Woth1 FP (input motion) for 4Sep2010 M_w7.1

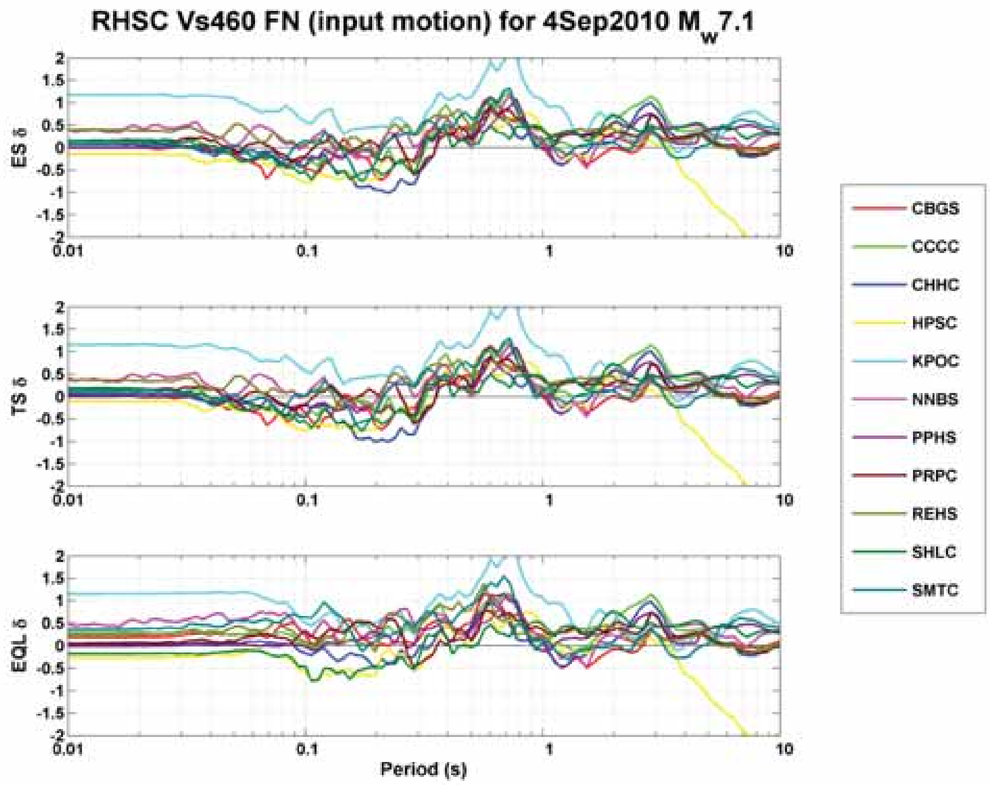


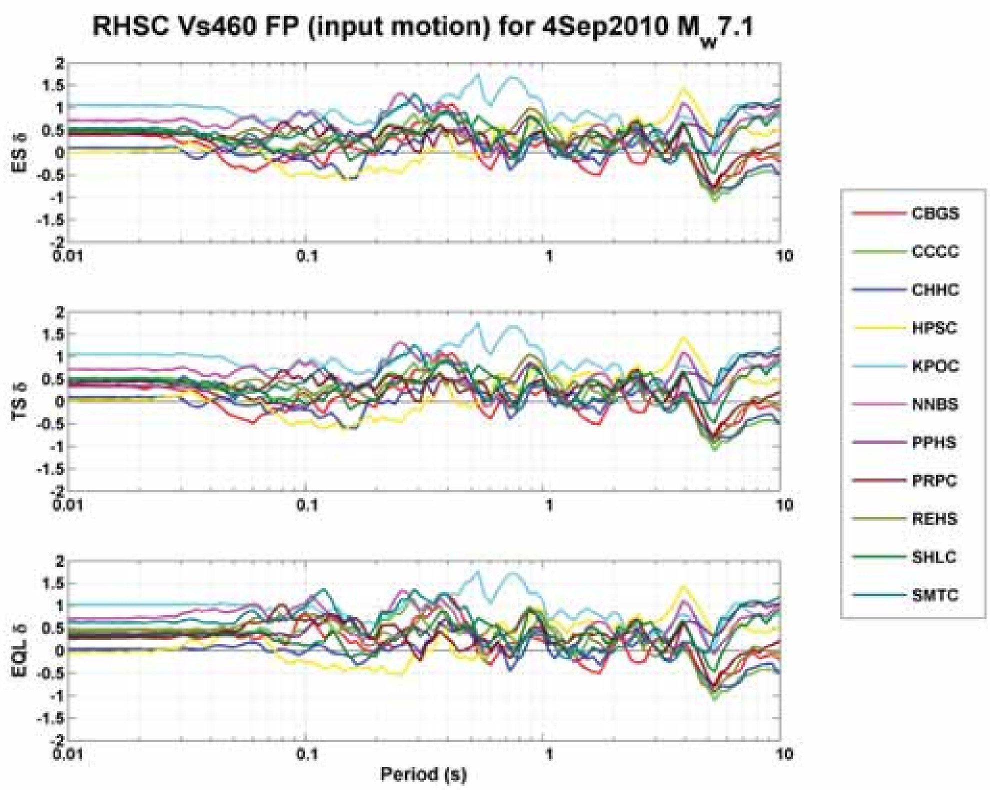


CACS Woth2 FN (input motion) for 4Sep2010 M_w7.1
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC

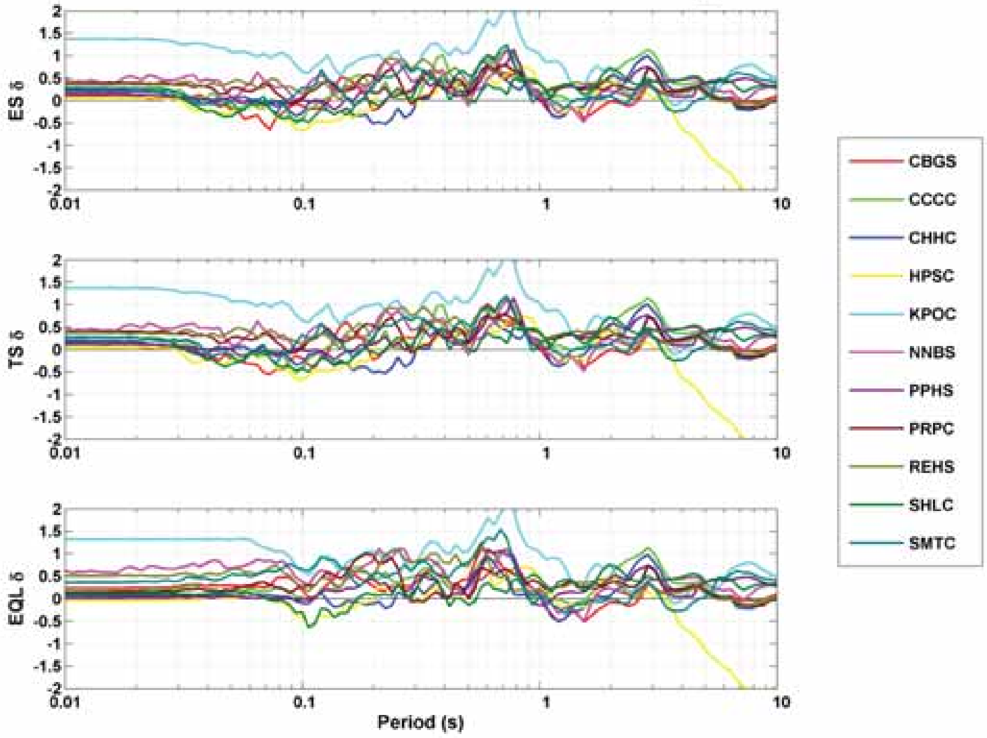


CACS Woth2 FP (input motion) for 4Sep2010 M_w7.1
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC

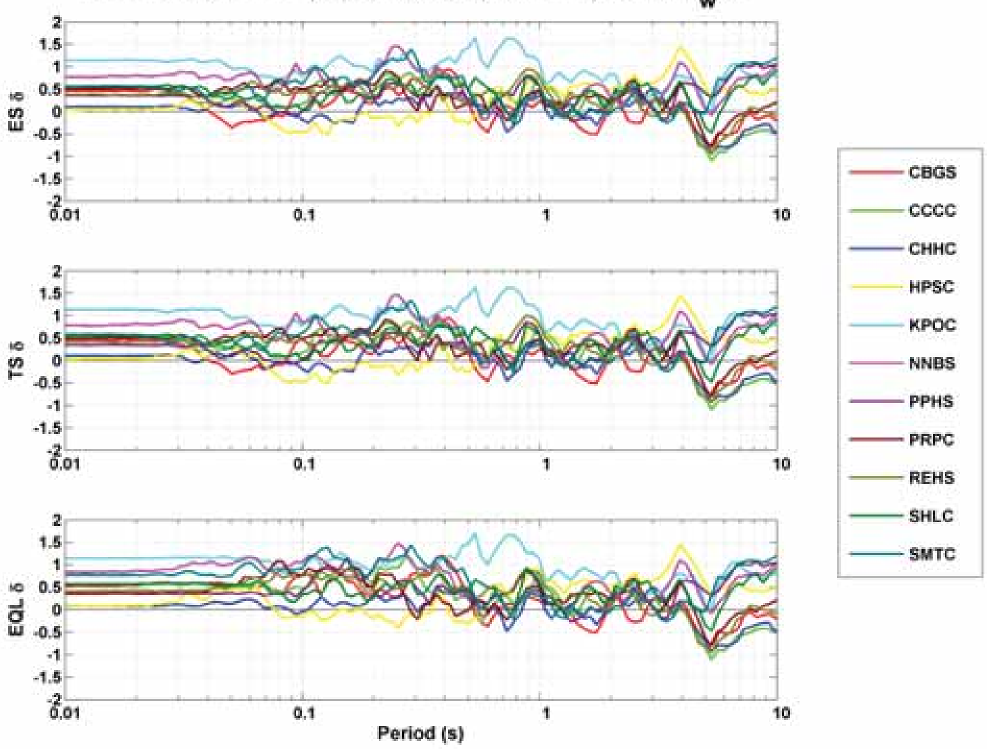

RHSC Vs460 FN (input motion) for 4Sep2010 Mw7.1
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC


RHSC Vs460 FP (input motion) for 4Sep2010 Mw7.1
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC

RHSC Woth1 FN (input motion) for 4Sep2010 M_w7.1



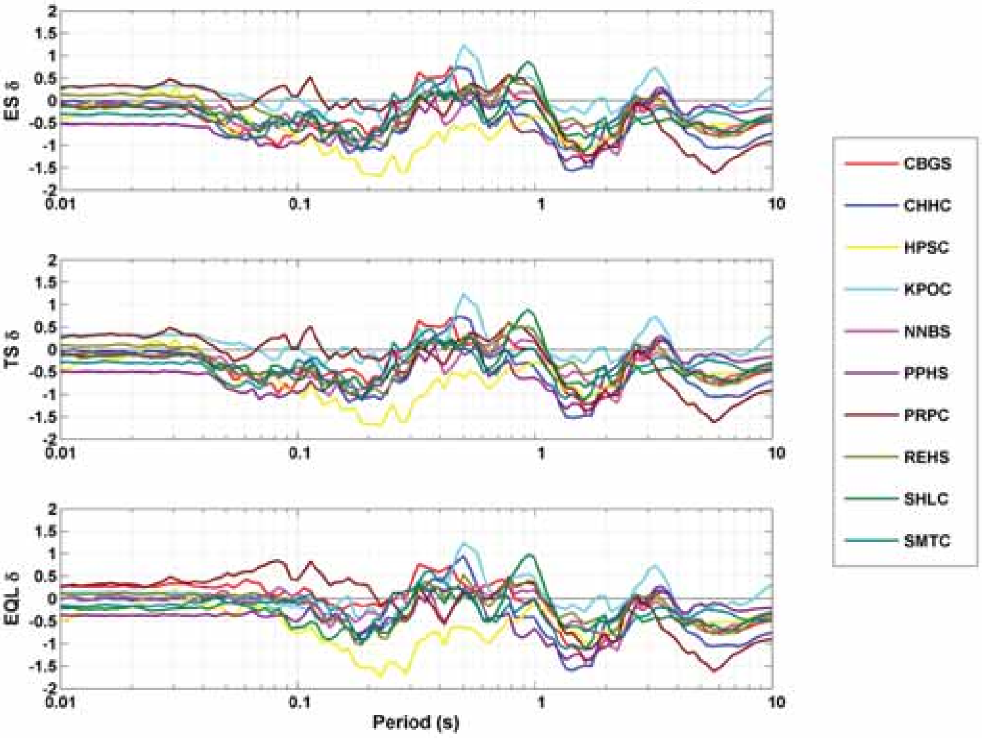
RHSC Woth1 FP (input motion) for 4Sep2010 M_w7.1



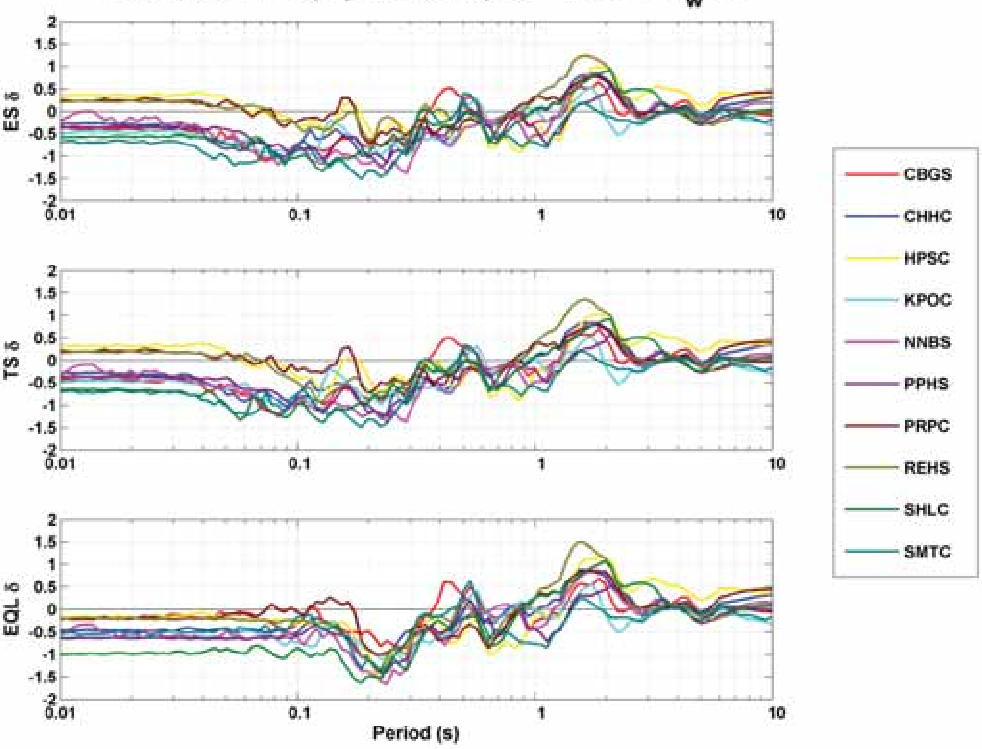
APPENDIX F.3

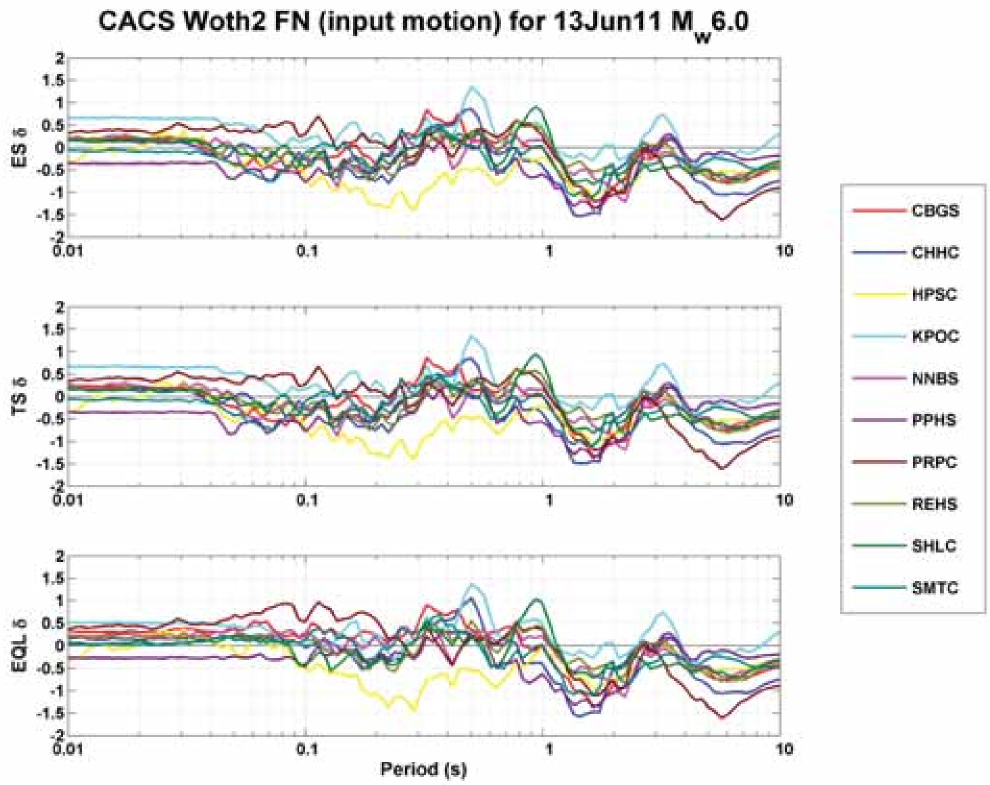
Residuals for 13 June 2011 M_w 6.0 Event.

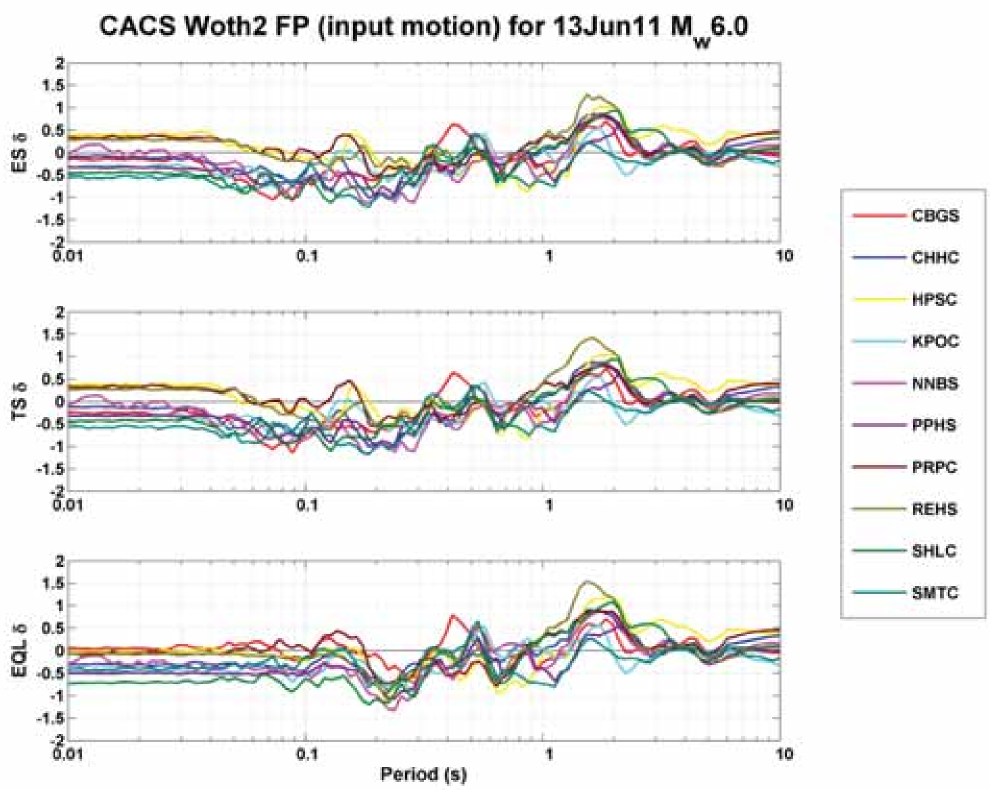
CACS Woth1 FN (input motion) for 13Jun11 M_w6.0

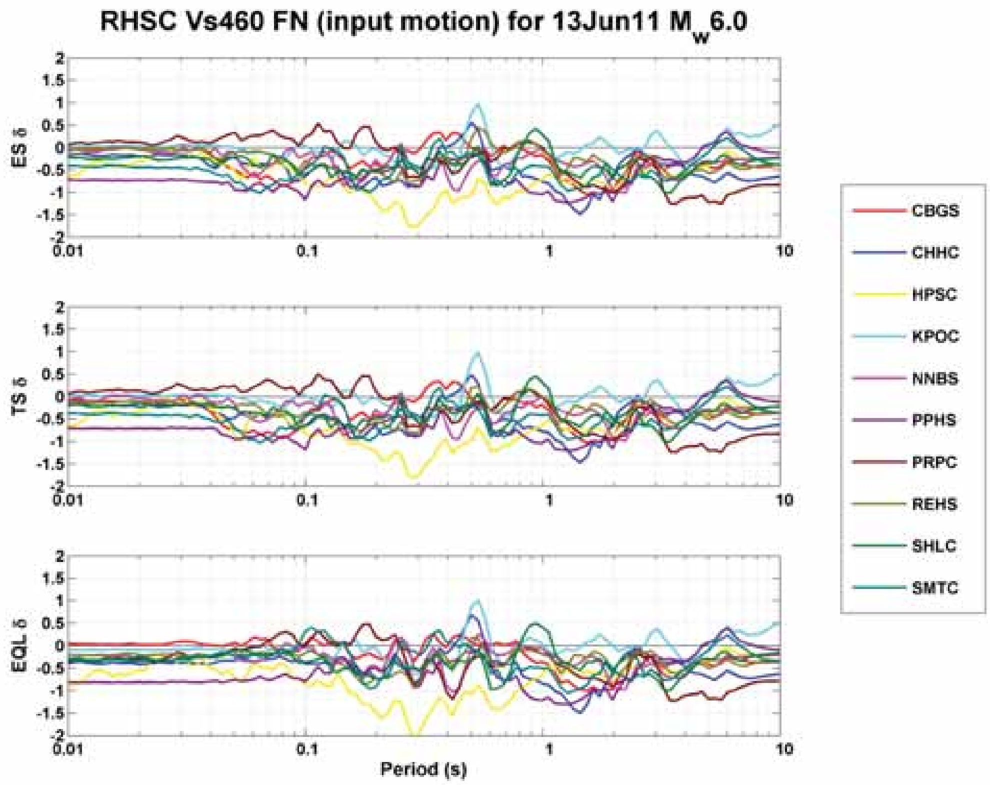


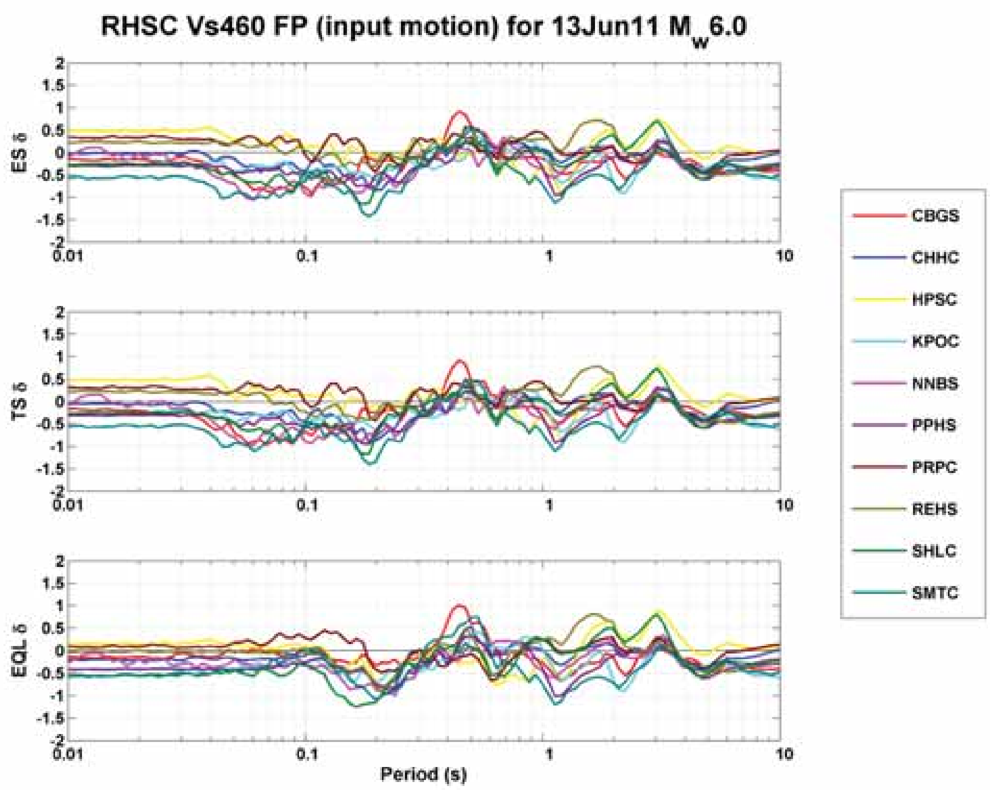
CACS Woth1 FP (input motion) for 13Jun11 M_w6.0



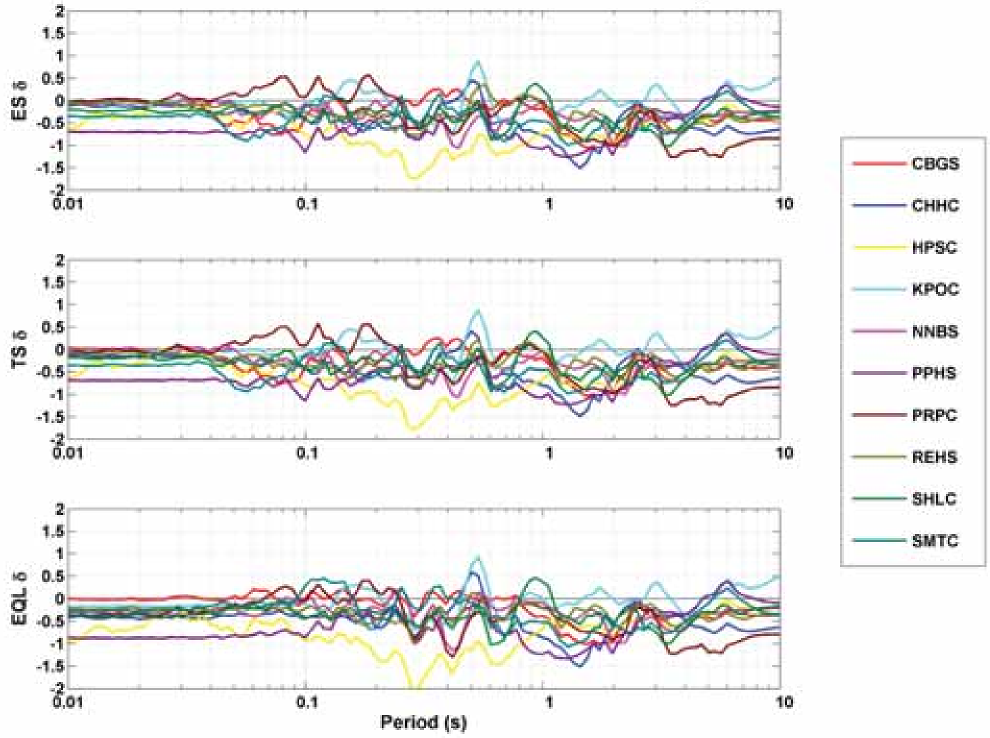

CACS Woth2 FN (input motion) for 13Jun11 Mw6.0
ES δ
TS δ
EQL δ
Period (s)
CBGS
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC


CACS Woth2 FP (input motion) for 13Jun11 Mw6.0
ES δ
TS δ
EQL δ
Period (s)
CBGS
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC

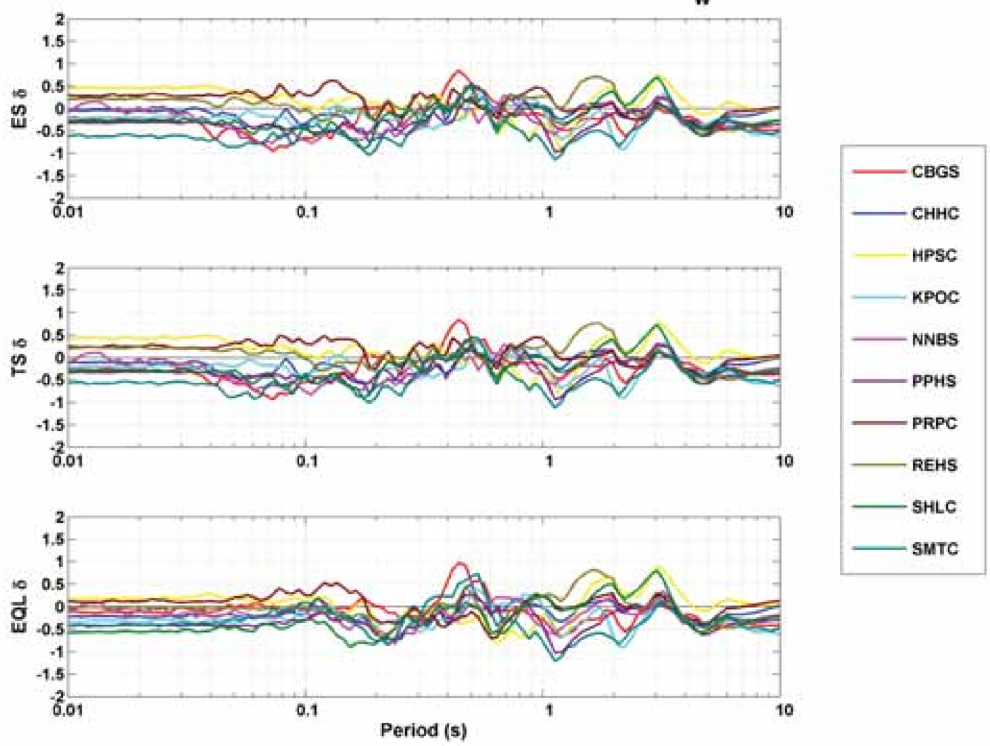

RHSC Vs460 FN (input motion) for 13Jun11 M_w6.0
ES δ
TS δ
EQL δ
Period (s)
CBGS
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC


RHSC Vs460 FP (input motion) for 13Jun11 M_w6.0
ES δ
TS δ
EQL δ
Period (s)
CBGS
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC

RHSC Woth1 FN (input motion) for 13Jun11 M_w6.0



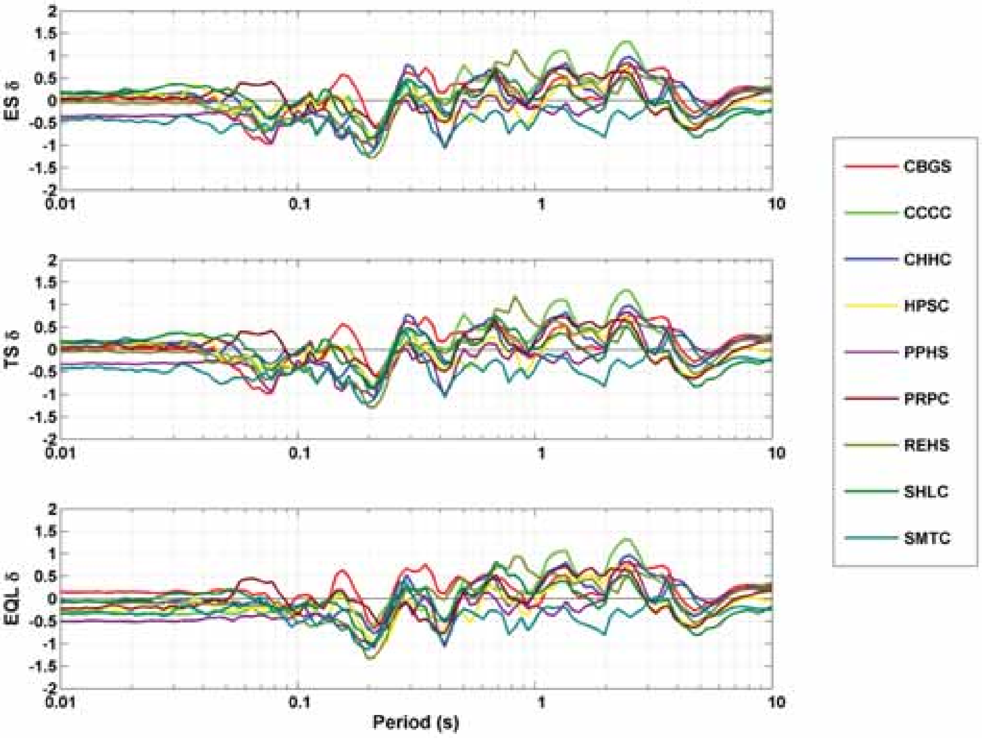
RHSC Woth1 FP (input motion) for 13Jun11 M_w6.0



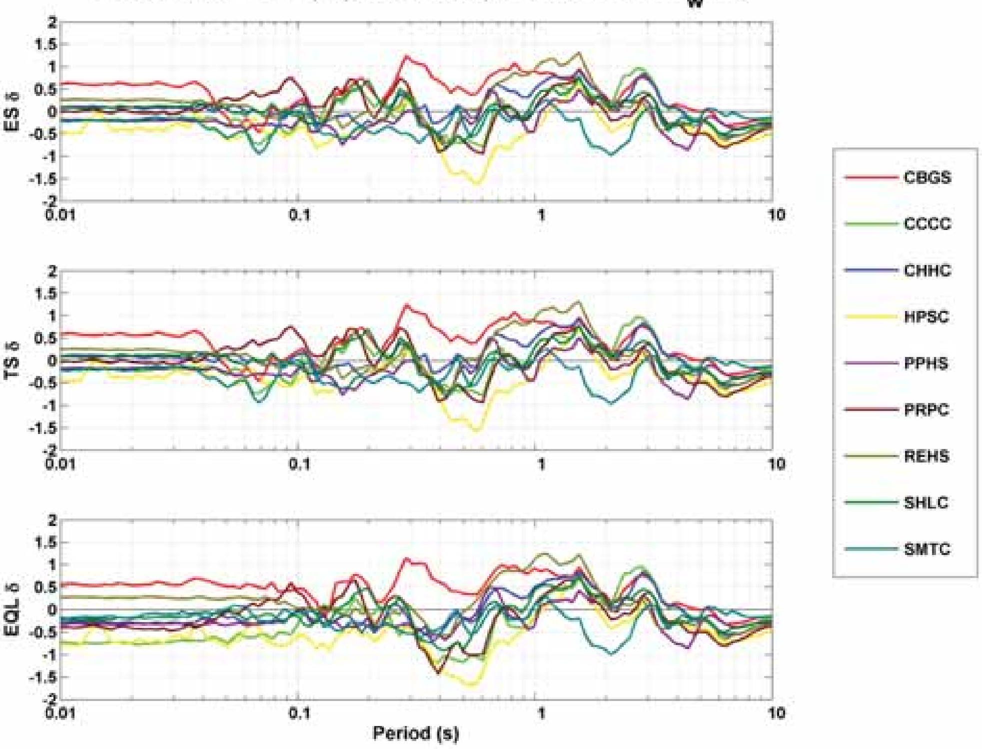
APPENDIX F.4

Residuals for 23 December 2011 M_w 5.8 Event.

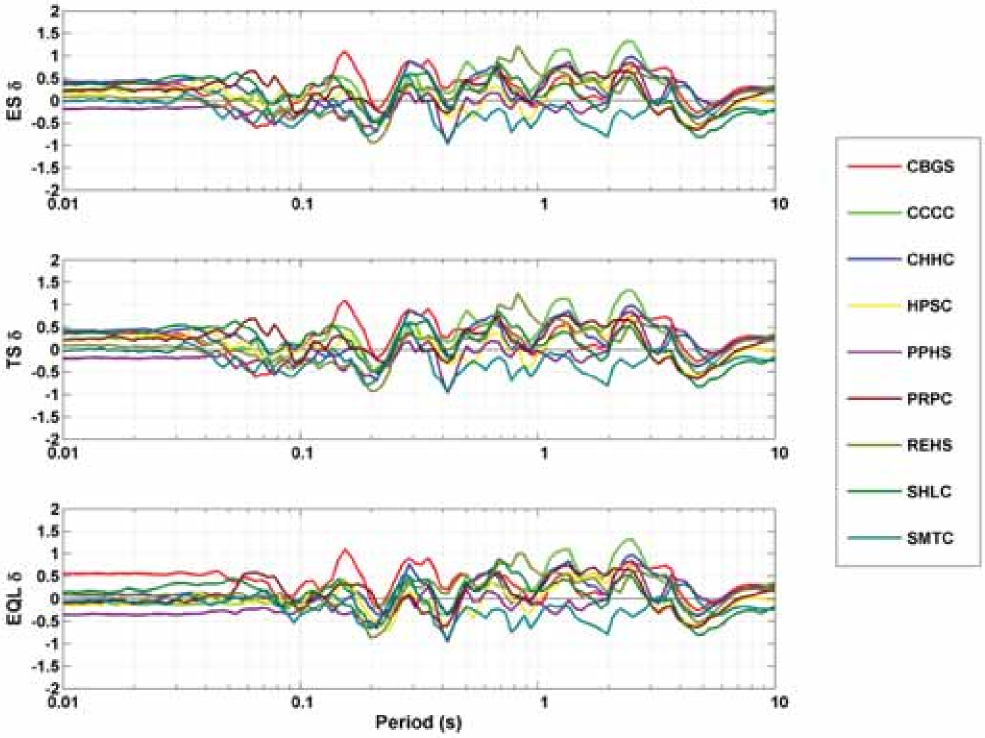
CACS Woth1 FN (input motion) for 23Dec11 M_w5.8



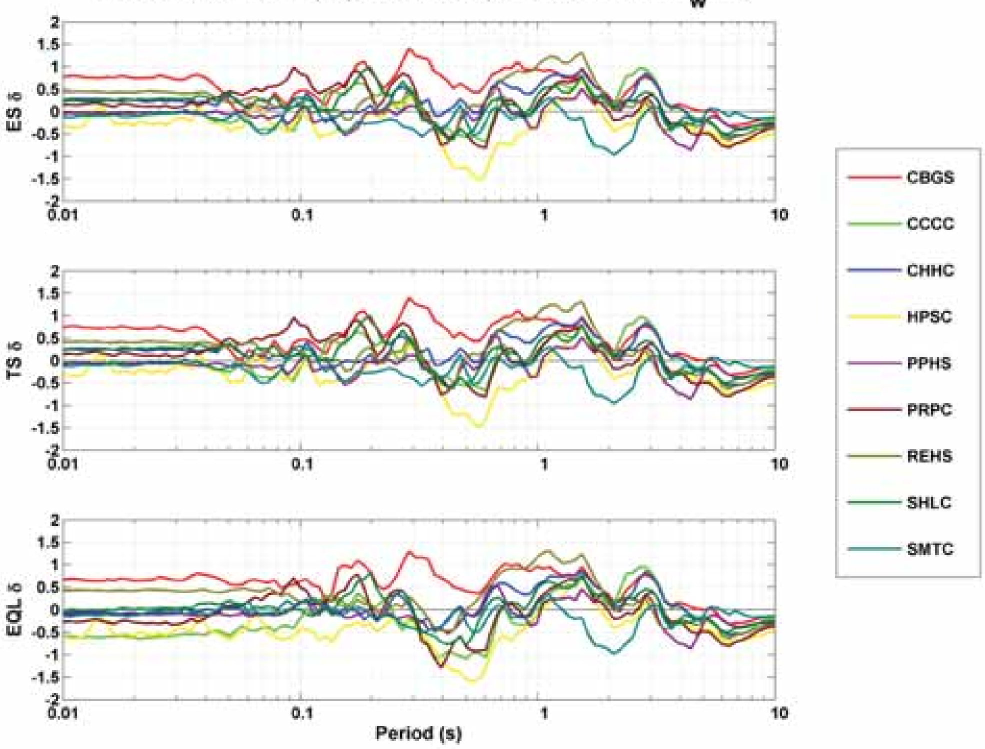
CACS Woth1 FP (input motion) for 23Dec11 M_w5.8

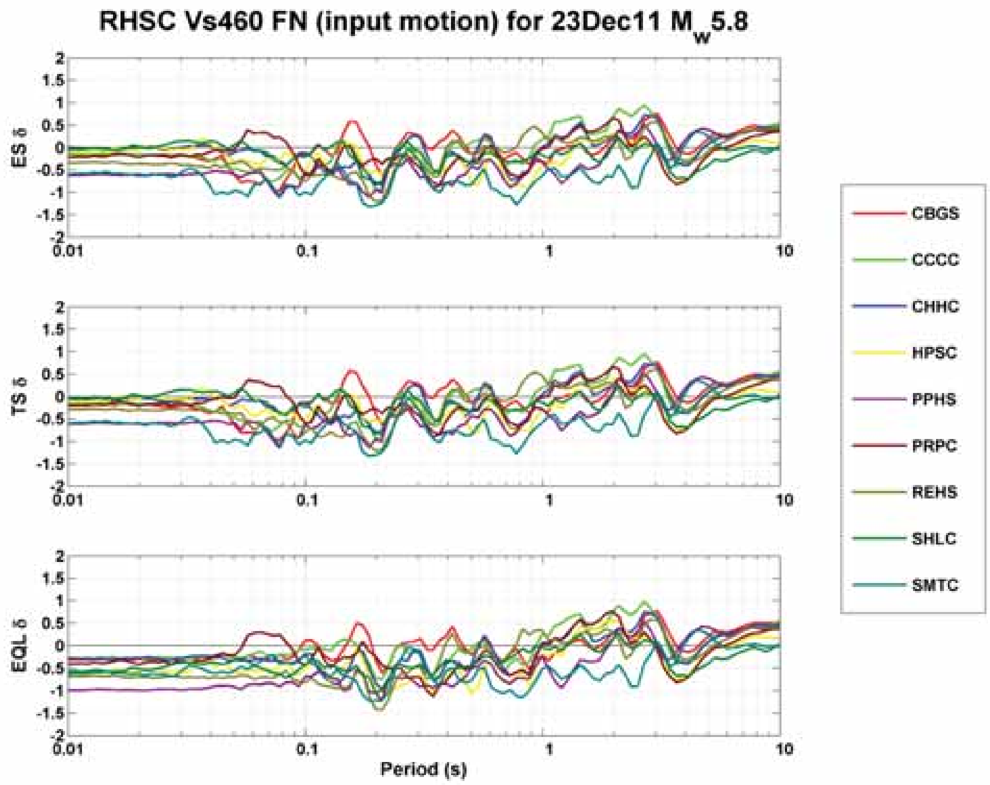


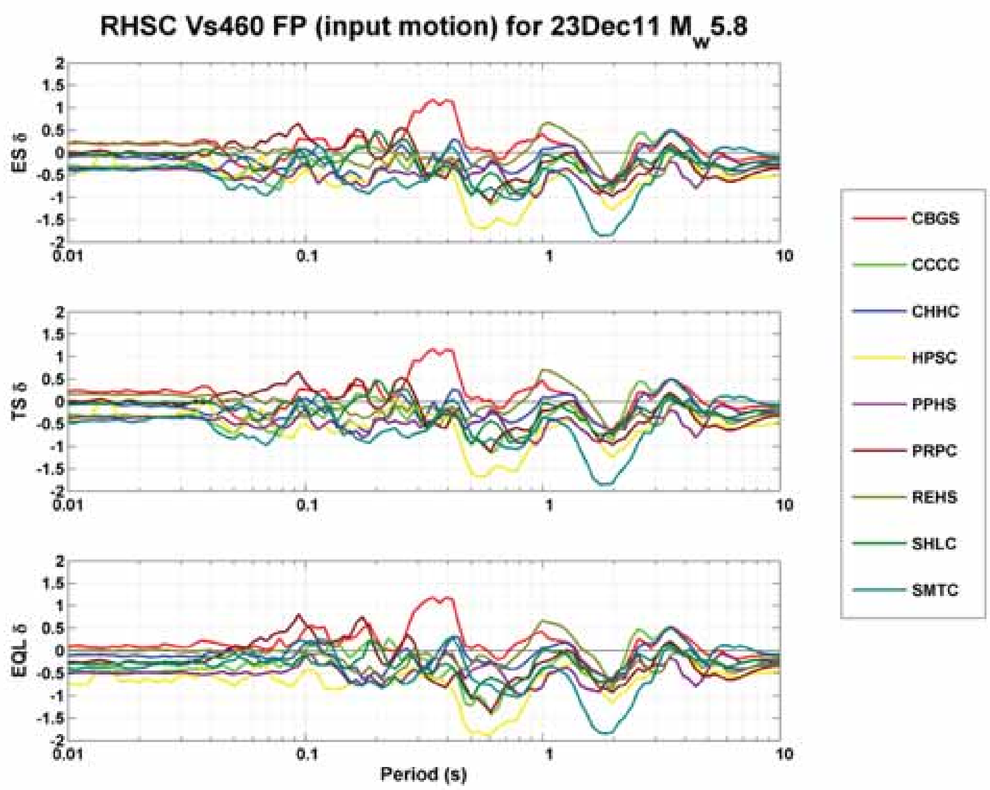
CACS Woth2 FN (input motion) for 23Dec11 M_w5.8



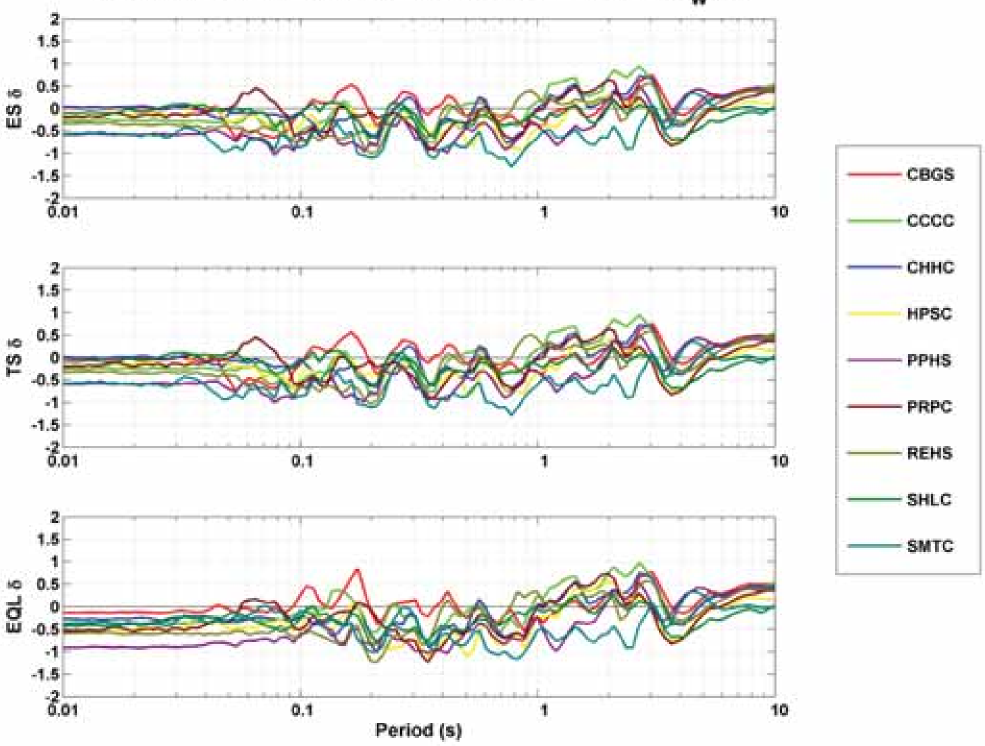
CACS Woth2 FP (input motion) for 23Dec11 M_w5.8



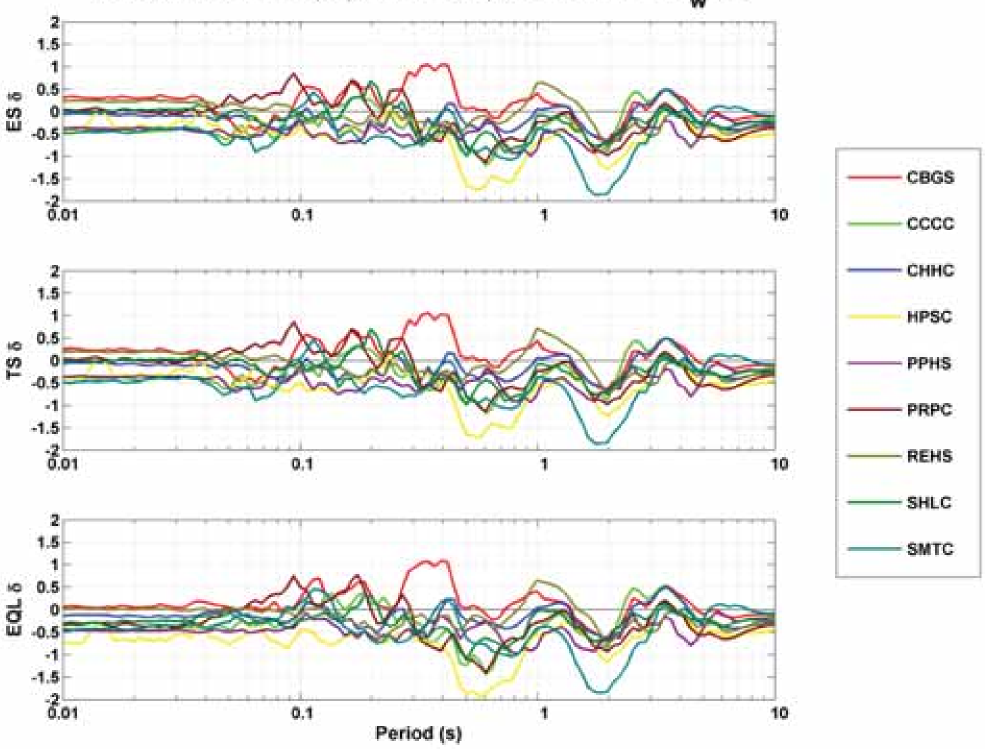

RHSC Vs460 FN (input motion) for 23Dec11 M_w5.8
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
PPHS
PRPC
REHS
SHLC
SMTC


RHSC Vs460 FP (input motion) for 23Dec11 M_w5.8
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
PPHS
PRPC
REHS
SHLC
SMTC

RHSC Woth1 FN (input motion) for 23Dec11 M_w5.8

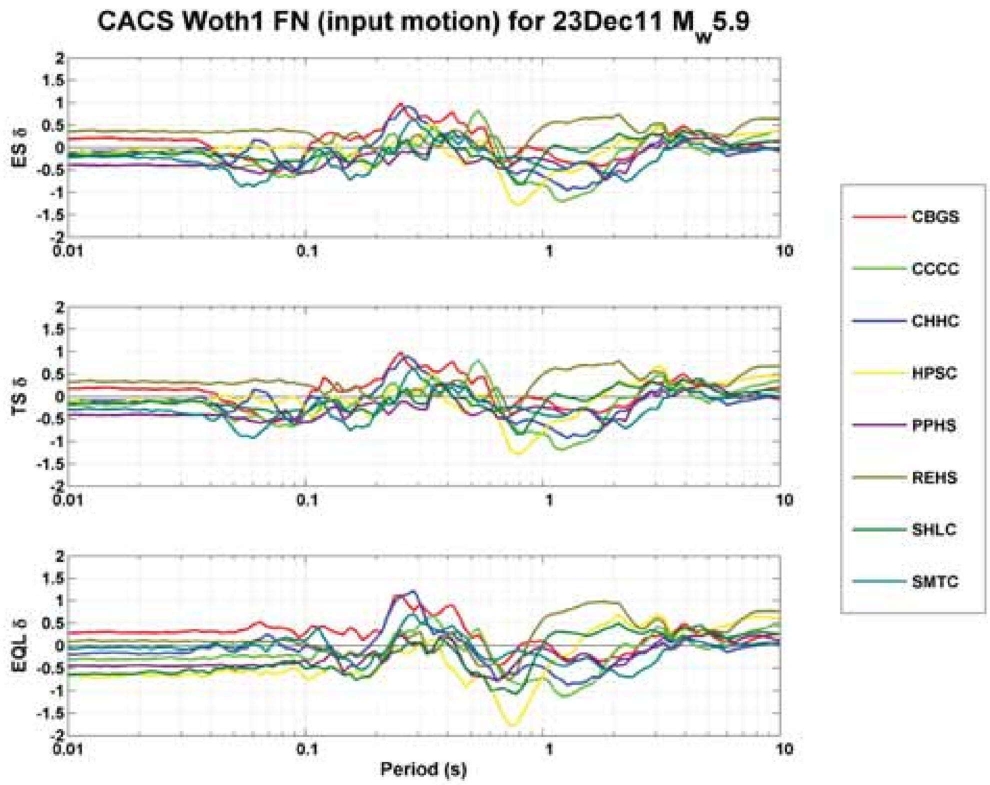


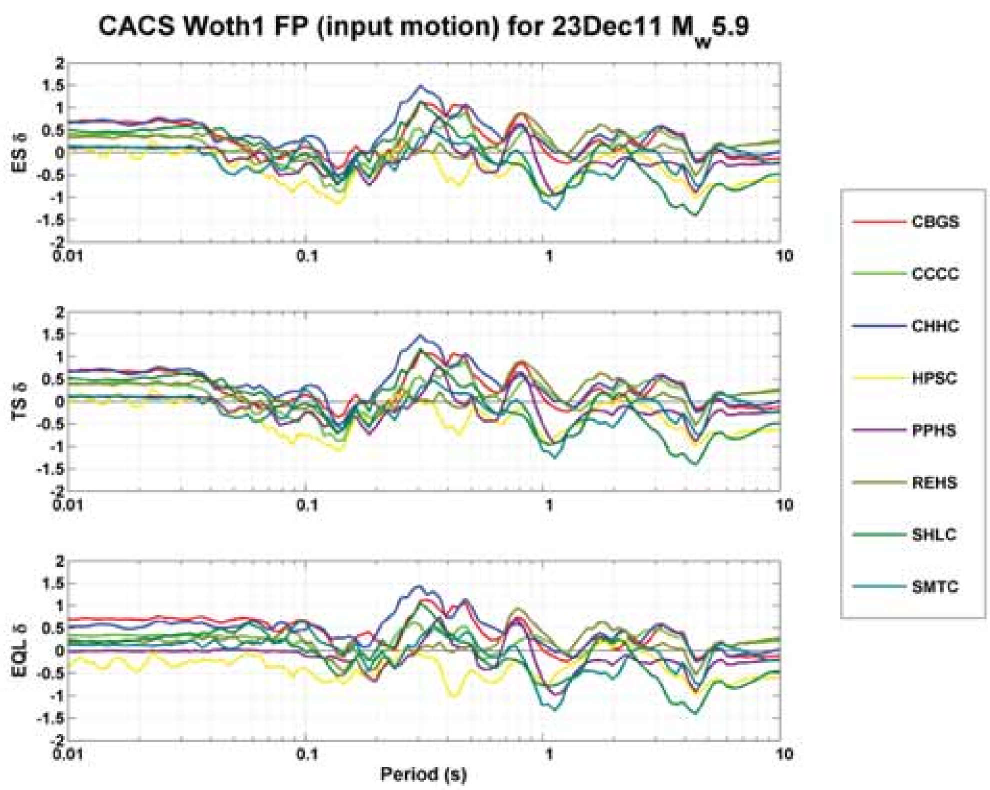
RHSC Woth1 FP (input motion) for 23Dec11 M_w5.8



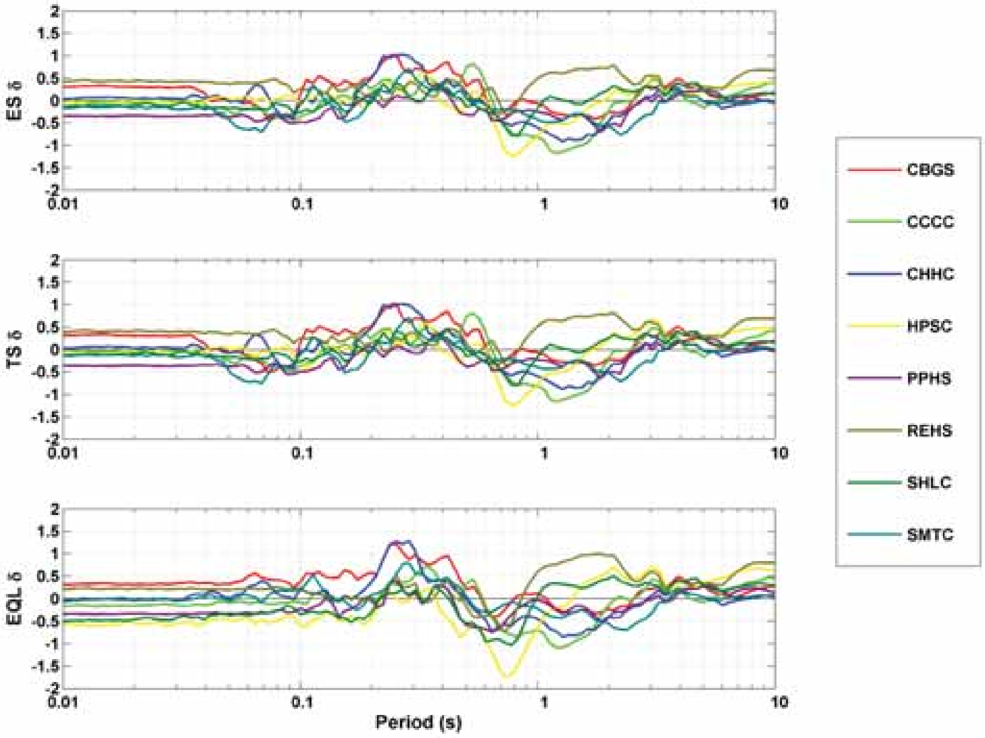
APPENDIX F.5

Residuals for 23 December 2011 M_w 5.9 Event.

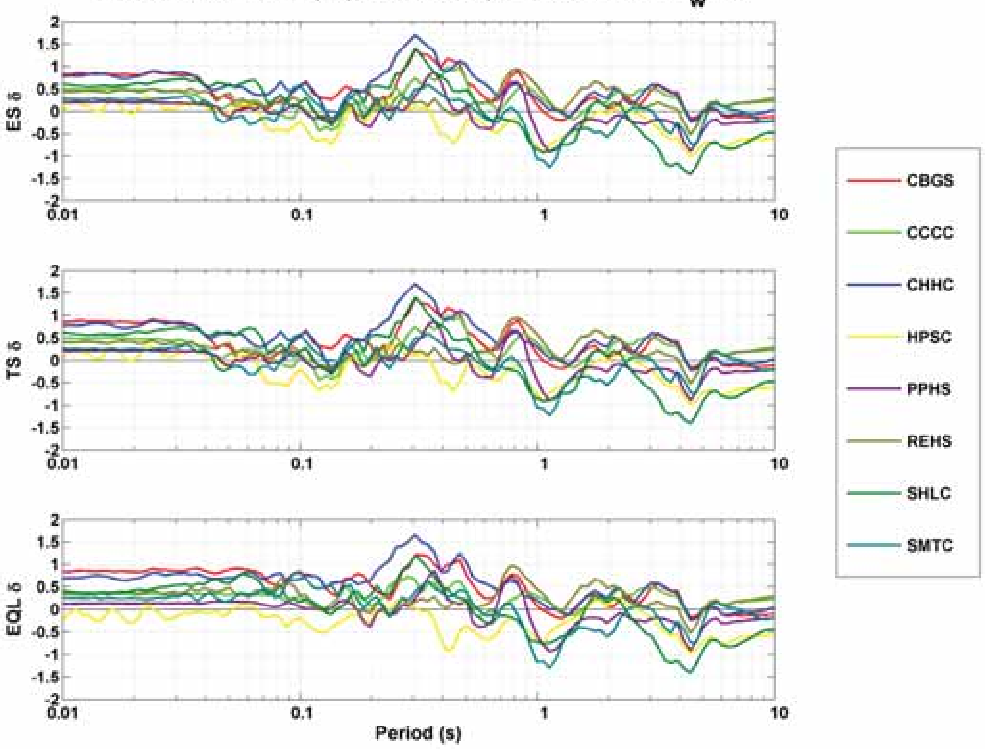

CACS Woth1 FN (input motion) for 23Dec11 M_w5.9
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
PPHS
REHS
SHLC
SMTC

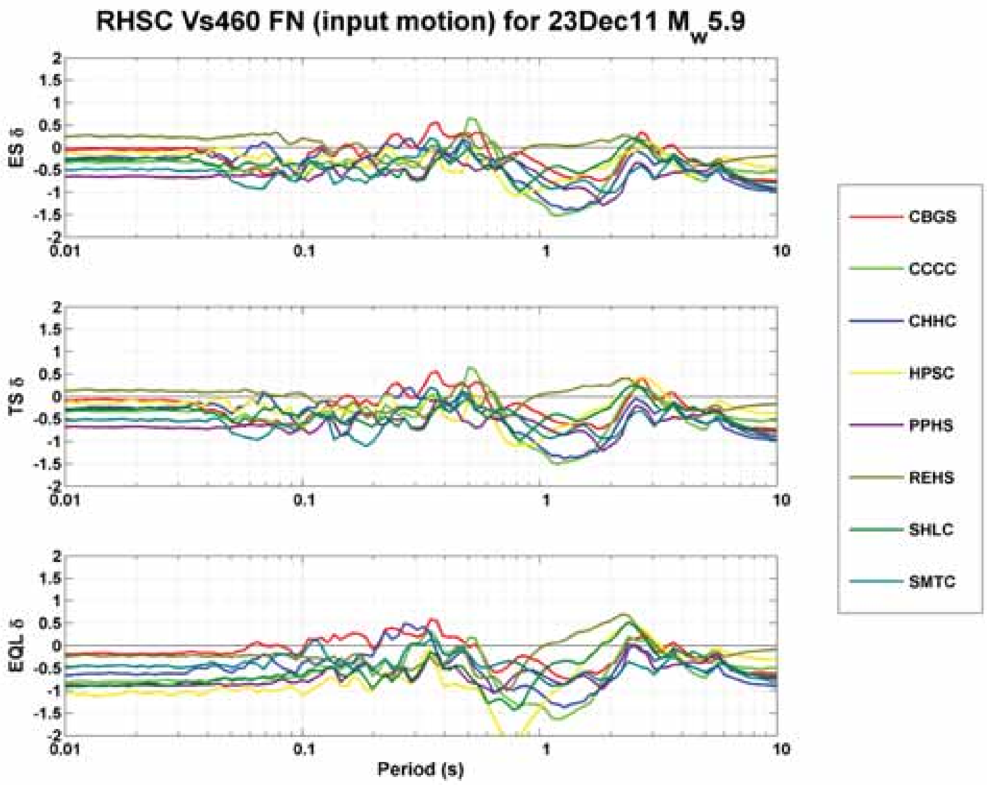

CACS Woth1 FP (input motion) for 23Dec11 M_w5.9
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
PPHS
REHS
SHLC
SMTC

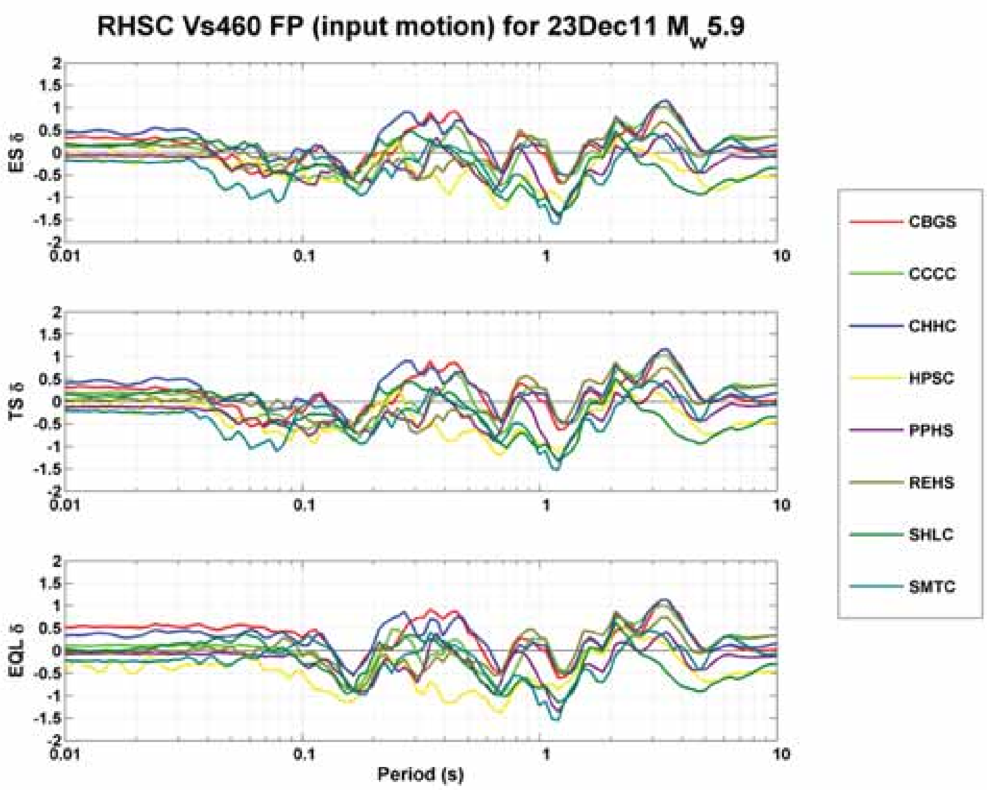
CACS Woth2 FN (input motion) for 23Dec11 M_w5.9

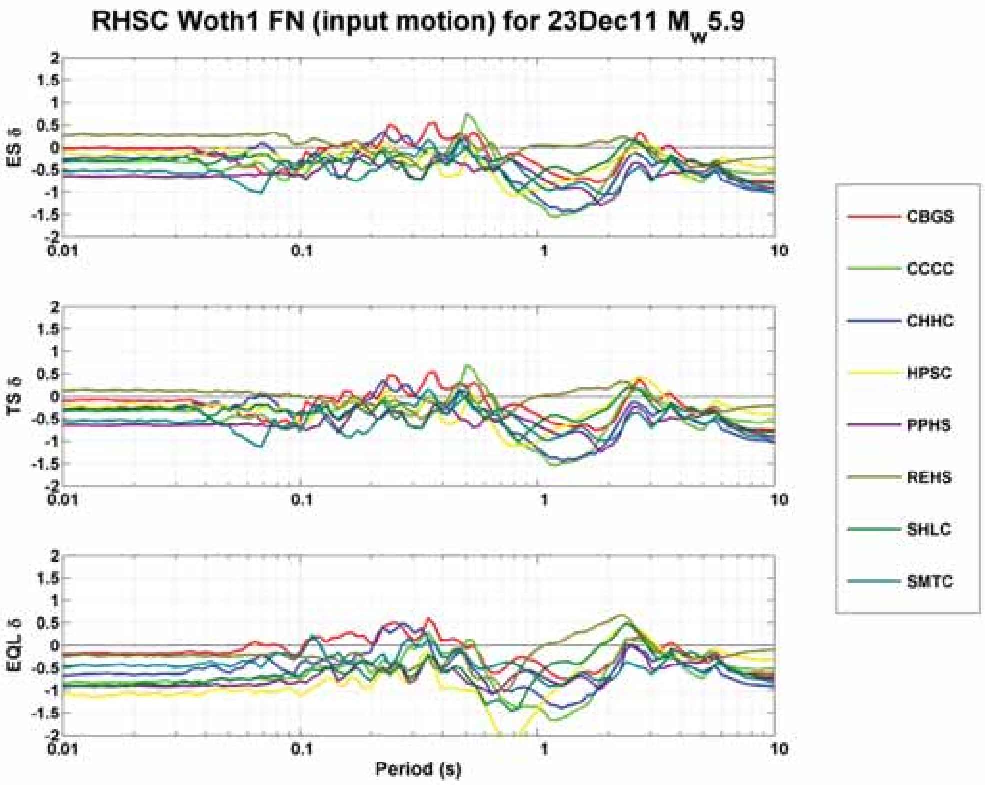


CACS Woth2 FP (input motion) for 23Dec11 M_w5.9

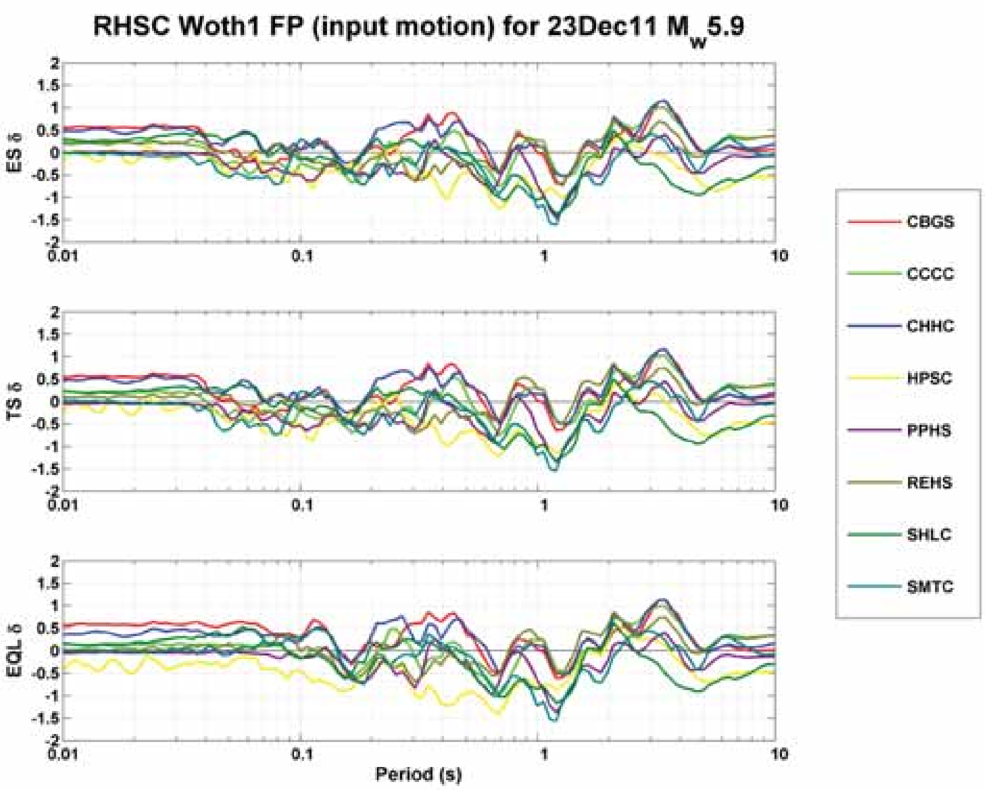



RHSC Vs460 FN (input motion) for 23Dec11 M_w5.9
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
PPHS
REHS
SHLC
SMTC


RHSC Vs460 FP (input motion) for 23Dec11 M_w5.9
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
PPHS
REHS
SHLC
SMTC



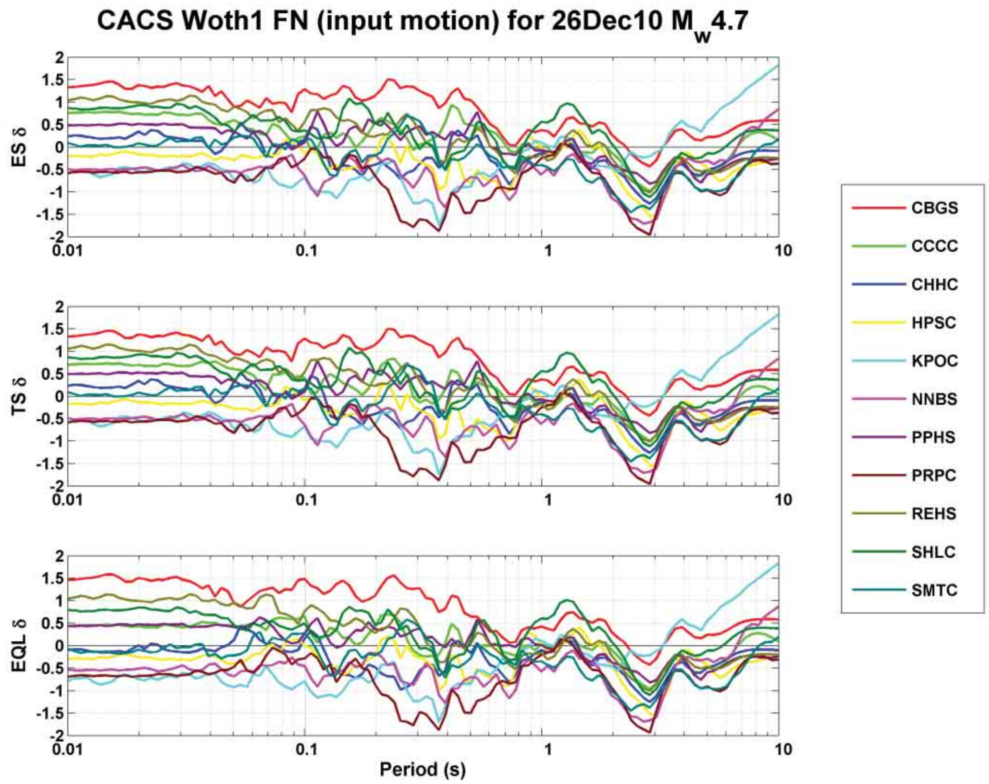
RHSC Woth1 FN (input motion) for 23Dec11 Mw5.9
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
PPHS
REHS
SHLC
SMTC

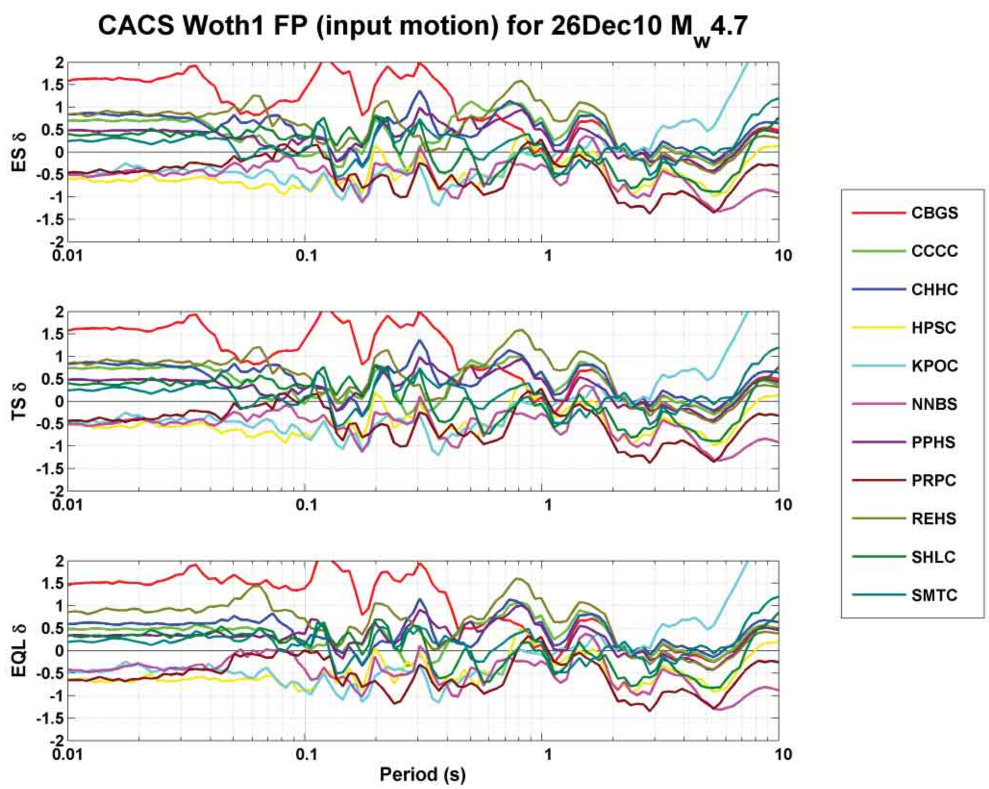


RHSC Woth1 FP (input motion) for 23Dec11 Mw5.9
ES δ
TS δ
EQL δ
Period (s)
CBGS
CCCC
CHHC
HPSC
PPHS
REHS
SHLC
SMTC

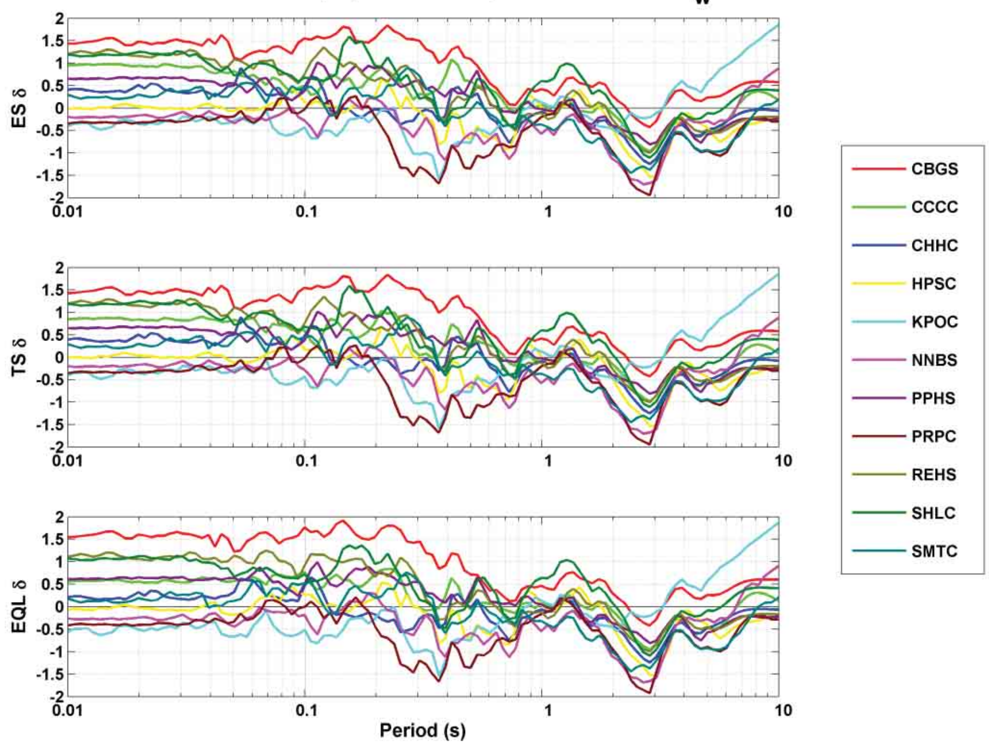
APPENDIX F.6

Residuals for 26 December 2010 M_w 4.7 Event.


CACS Woth1 FN (input motion) for 26Dec10 M_w4.7
ES δ
TS δ
EQL δ
Period (s)
0.01
0.1
1
10
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC


CACS Woth1 FP (input motion) for 26Dec10 M_w4.7
ES δ
TS δ
EQL δ
Period (s)
0.01
0.1
1
10
CBGS
CCCC
CHHC
HPSC
KPOC
NNBS
PPHS
PRPC
REHS
SHLC
SMTC

CACS Woth2 FN (input motion) for 26Dec10 M_w4.7



CACS Woth2 FP (input motion) for 26Dec10 M_w4.7