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A CRITICAL COMPARISON BETWEEN STRESS AND ENERGY BASED METHODS FOR THE EVALUATION OF LIQUEFACTION POTENTIAL

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Abstract: *Selected liquefaction case histories in New Zealand during recent earthquakes were analysed using the conventional SPT and CPT stress based methods, and the energy based method recently proposed by Kokusho (2013). Several sites in the wider Christchurch region were examined considering strong motions from the 2010-2011 Canterbury Earthquake Sequence. The liquefaction potential was also examined at three sites in the Wellington and Marlborough regions for the 2013 M_w6.6 Lake Grassmere and 2016 M_w7.8 Kaikoura earthquakes. The methods were compared in terms of the critical liquefaction depth and layer thickness, data scatter and number of false-negative predictions. The Kokusho energy based method performed satisfactorily in assessing the liquefaction potential at the case history sites, giving comparable results to the stress based methods. Furthermore, the Kokusho method succeeded in identifying liquefaction potential at several sites in Christchurch where false-negative predictions were shown in the CPT stress based method.*

1. Introduction

Soil liquefaction, as an earthquake-induced geohazard in loosely deposited, saturated soils of low plasticity, remains a major challenge to the built environment today. Hitherto, different approaches exist to assess the likelihood of liquefaction. The existing liquefaction assessment methodologies can be distinguished in three categories depending on whether they consider the induced shear stresses, shear strains or dissipated energy within a soil profile.

The most commonly used methods are the simplified stress based ones which are grounded on comprehensive review of worldwide liquefaction cases (Idriss & Boulanger, 2008; Boulanger & Idriss, 2014), and are thus popular in conventional practice. Yet, false-negative predictions have been reported (Green *et al.*, 2014; Tsaparli *et al.*, 2018), when using stress based methods, implying an overestimation of the factor of safety in sites with liquefaction manifestation. The question of whether liquefaction can be more accurately predicted by considering characteristics other than stresses has thus led to the development of alternative methodologies.

Energy based approaches, which take into account the unique relationship between pore-pressure build-up and the associated accumulation of dissipated energy, have gained popularity (Chen *et al.*, 2005; Davis & Berrill, 1996) in recent years. They have the advantage of using the entire time series of a ground motion instead of a single intensity measure, such as the peak ground acceleration. Recently, Kokusho and his co-workers (Kokusho, 2013; Kokusho & Mimori, 2015) proposed a new energy based liquefaction evaluation scheme which compares the “strain energy capacity” within a soil deposit with the energy induced by the upward incident wave for a given ground motion. This paper applies the Kokusho method at several sites considering case histories with documented field records related to recent earthquakes in New Zealand. The performance of the Kokusho method is evaluated in comparison with the widely used stress-based methods and against the field observations.

2. Methodology

The Kokusho's (2013) energy based method was applied to analyse liquefaction cases triggered by recent earthquakes in New Zealand. Ground profiles in the greater region of Christchurch

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which displayed liquefaction of varying severity during the 2010-2011 Canterbury Earthquake Sequence are reviewed and evaluated for their factor of safety against liquefaction. In particular false-negative case histories (Green *et al.*, 2014; Tsaparli *et al.*, 2018) based on stress based methods were re-examined. The method is also applied to sites in the vicinity of Wellington during the 2016 M_w7.8 Kaikoura earthquake, forming another set of case histories. SPT and CPT (Idriss & Boulanger, 2008) stress based methods are also used to evaluate the liquefaction potential for each site. The extensive network of strong motion stations, abundant geotechnical investigation data and well documented liquefaction records of the country make it an exceptional opportunity to assess the performance of the stress and energy based procedures.

In the Kokusho method (2013), the factor of safety against liquefaction is defined as the accumulated energy ratio *AER*, which is expressed as the strain energy capacity *WH* over the final upward energy E_{uf} of a layer. The upward energy computation, which represents the "demand", results from the integration of particle velocity time series of the upward travelling wave. The upward and downward components of an input motion are separated using Fourier Transform and a recursion formula which involves the strain-dependent shear modulus and damping ratio. The evaluation of the "capacity" term *WH* stems from the results of undrained triaxial tests on remoulded Futtsu sand samples under cyclic loading, where the dissipated energy per unit volume, normalised by the effective confining stress, $\Delta W/\sigma'_c$ was found to have a unique correlation with the strain amplitude and excess pore pressure generation, irrespective of the relative density, fines content, number of cycles and applied stress ratio (Kokusho, 2013). Assuming that the abovementioned relationship is also valid in natural sand deposits, Kokusho (2013) correlated the normalised dissipated energy required to trigger liquefaction to field SPT N values.

3. Case Studies of the 2010-2011 Canterbury Earthquake Sequence

The two main events of the 2010-2011 Canterbury Earthquake Sequence (CES) were considered in this study, namely the M_w7.1 Darfield earthquake on 4 September 2010 and the M_w6.2 Christchurch earthquake on 22 February 2011.

Eight sites, as listed in **Table 1**, were chosen out of the 25 case histories established from CPT profiling in Green *et al.* (2014). These locations are within the network of four strong motion stations (SMS), including Christchurch Cathedral College (CCCC), Holverstone Drive Pumping (HPSC), Kaiapoi North School (KPOC) and Shirley Library (SHLC) stations. Case history selection criteria similar to that in Anatolakis (2016) were adopted and are listed as follows:

(i) The site is sufficiently close to a strong motion station;

(ii) Sites with different levels of liquefaction damage in the Darfield and Christchurch earthquakes. Preferably, the site liquefied in either one of the ground motions;

(iii) Sites with lateral spreading are excluded

(iv) SPT data can be found within 200 m of the site;

(v) The dataset should cover a reasonable range of ground motions so that different motion characteristics and broader distribution of peak ground acceleration can be examined.

SMS code	CPT Site code in Green *et al.* (2014)
CCCC	Z4-4
HPSC	AVD-07, BUR-46
KPOC	KAN-09, KAN-19, KAS-08, KAS-20
SHLC	SHY-09

Table 1. List of CPT sites chosen from Green et al. (2014)

In addition, liquefaction assessments were performed at three sites where false-negative prediction was obtained with the simplified CPT procedure in Wotherspoon *et al.* (2015), as highlighted by Tsaparli *et al.* (2018). These are strong motion stations in central Christchurch, namely CCCC, HPSC and Pages Road Pumping (PRPC) stations.

3.1 Ground conditions

The surface geology is made up of the Springston Formation, comprising alluvial deposits of about 20 m thick; the Christchurch Formation, associated with estuarine, swamp, dune soils of varying grain size and of thickness up to 40 m; and the Riccarton Gravel Formation, extending hundreds of metres depth before meeting the bedrock (Orense *et al.*, 2014; Green *et al.*, 2014). The ground water table is as shallow as 0 to 2 m below the surface in the central and eastern Christchurch (Orense *et al.*, 2014).

The soil profiles were based on investigation logs of CPT, SPT and any available test data retrieved from the Geotechnical Database of New Zealand (NZGD, 2016). It is assumed that the soil stratigraphy is uniform and can be represented by horizontal layers within a small extent from the CPT sites. Thus, any nearby SPT are considered in the liquefaction assessment in spite of the slight distance between the CPT and SPT boreholes.

For each individual site, the soil profile was initially divided into layers of either one or two meters thickness. Measured SPT-N values were corrected for overburden pressure and fines content. It is assumed that the energy efficient coefficient C_E and other procedural correction factors are one due to the lack of testing details. The corrected N-values were then assigned to the layers according to the depth. For cases where the SPT-N value corresponded to a depth different from the mid-depth of a layer in the adopted profile, interpolation and engineering judgement were exercised to infer the N-value. The depth of groundwater table was based on the CPT records. The dry soil unit weight was taken as 17 kN/m^3 in layers above groundwater table, and the saturated unit weight of 19.5 kN/m^3 was applied to layers below (Green *et al.*, 2014).Since the soil deposits in Christchurch tend to be unconsolidated and young, the overconsolidation ratio *OCR* was taken as one. The plastic index *PI* was assigned to be zero to account for the dominating presence of sand.

The fines content (*FC*) is accounted in the step of adjusting the penetration resistances into the condition of equivalent clean sand. *FC* measurements were used for the correction of SPT N-values when available, otherwise, an average value was predicted using CPT based correlations. To estimate *FC*, the locally calibrated expression (Equation 1) of Robinson et al (2014) was used, as cited in Green *et al.* (2014). When the normalised sleeve friction *F* is smaller than 0.5% (*F*<0.5%) and the soil behaviour type index I_c is between 1.64 and 2.36 (1.64< I_c <2.36), a maximum value of 5% is imposed on *FC* (*FC*≤5%) to prevent wrongly classifying very loosely deposited clean sands as dense sands with fines. Green *et al.* (2014) commented that the prediction errors in the traditional CPT (Idriss & Boulanger, 2008) method were independent of the *FC* expression, confirming the applicability of the Christchurch-specific correlation.

$$FC = \begin{cases} 0, & \text{if } I_c \leq 1.75 \\ 76.9 I_c - 136.5, & \text{if } 1.75 < I_c < 3.07 \\ 100, & \text{if } 3.07 \leq I_c \end{cases} \quad (1)$$

The shear wave velocity (V_s) profile for each site to be evaluated is necessary when using the Kokusho (2013) approach. V_s profiles based on surface wave testing at a number of strong motion stations can be found in Wood *et al.* (2011) and are used when analysing the three selected stations. Meanwhile, since V_s had not been measured directly close to the remaining examined CPT sites, a CPT based empirical expression relating the measured cone resistance q_c and sleeve friction f_s (kPa) to V_s was adopted. The correlation was developed for non-gravel near-surface soil and was calibrated using CPT data in Christchurch by McGann *et al.* (2015). The proposed model is:

$$V_s = 18.4 \cdot q_c^{0.144} \cdot f_s^{0.0832} \cdot z^{0.278} \text{ (m/s)} \quad (2)$$

where *z* is the depth below ground level (m). The shear velocity at a certain percentile *x* can be derived from the median V_s in Equation 2, considering the standard deviation in Equation 3 and standard normal variate z_x in Equation 4:

$$\sigma_{In(V_s)} = \begin{cases} 0.162, & \text{if } z \leq 5\text{m} \\ 0.216 - 0.0108z, & \text{if } 5 < z < 10\text{m} \\ 0.108, & \text{if } z \geq 10\text{m} \end{cases} \quad (3)$$

$$V_{sx} = V_s \exp(z_x \cdot \sigma_{In(V_s)}) \text{ (m/s)} \quad (4)$$

Since penetration values can be problematic for shallow layers just below the ground surface, a lower bound of V_s was selected of 80 m/s, with reference to the observed minimum in the V_s data of Wood *et al.* (2011) and McGann *et al.* (2015). For depths greater than the CPT penetration depth and up to the bedrock depth (top of the Riccarton Gravel Formation), the V_s profile of the nearby motion station was used. Subsequently, the average shear wave velocity for the upper 30 m strata $V_{s,30}$ was calculated and used as a proxy for nonlinearity as explained in the following sections.

3.2 Site Response Analysis

One-dimensional equivalent linear analysis was performed with STRATA (Kottke et al 2013) to determine the strain-dependent soil properties, including the damping ratio and the reduced V_s. In the absence of site specific data the Darendeli & Stoke (2001) curves were used to represent the stiffness degradation and damping curves of the considered soil layers in STRATA. It should be noted that these empirical curves have been based on test data corresponding to shear strains generally lower than 1%. Vucetic (1994) defined the strain level beyond which soils display high nonlinearity and significant inelastic deformations, as the volumetric cyclic strain threshold γ_{tv}. The threshold γ_{tv} can be as low as 0.01% in clean sands to slightly above 0.1% in soils of higher plasticity. This clearly challenges the validity of equivalent linear analysis when the computed strain falls in the range of medium to large strains. (Papaspiliou *et al.*, 2012) highlighted these shortcomings and further noted the convergence issues of the equivalent linear methodology when used for sandy deposits subjected to high intensity records.

3.3 Strain Compatible V_s

To address the shortcomings of the equivalent linear methodology in the large strain range, an “acceptability test” was introduced in the one-dimensional equivalent linear site response analysis in the earlier studies of Anatolakis (2016). The results of the site response analysis were disregarded for strains greater than 0.6%. In these cases, a strain proxy relation proposed by Gueguen (2016) was used to predict an uniform strain in the whole soil profile.The strain proxy γ_{proxy} is a ratio of the peak ground velocity *PGV* to $V_{s,30}$ for a given ground profile.

$$\gamma_{proxy} = PGV / V_{s,30} \quad (5)$$

3.4 Scaling of Input Motion

Scaling has to be performed to approximate the ground motion at locations away from strong motion stations . Employing the generated contours of conditional peak ground acceleration in Christchurch region during the two earthquakes (Green *et al.*, 2014), the ground motion for a site was obtained by scaling the horizontal major component of the closest station's motion data. In this way, the resulting *PGA* at a site equals to the value computed in the contours. Even though the minor component of horizontal motion may be of large amplitude, only the motion in the major horizontal axis was considered in this study. The Kukosho (2013) method does not address the issue of minor horizontal component when calculating the upward seismic energy, probably for simplicity. With this caution in mind, the energy demand was computed for the scaled major axis horizontal ground acceleration time history.

3.5 Liquefaction Evaluation at a Selected Site: CPT site BUR-46

To demonstrate the application of the Kokusho (2013) methodology an example site, BUR-46, is detailed herein. A small amount of liquefaction ejecta and small ground cracking were observed at that site in the Darfield earthquake. Severe surface manifestation in the form of sand boiling and sand blisters was reported at this location after the Christchurch earthquake. The AER of the two events was very similar at this site. In both earthquakes, the *AER* was less than or close to unity in the upper 10 m of soil (**Figure 1a**). As the water table depth is 1.3 m, the first layer L1 (z = 0 to 2 m) may be considered unsaturated and not taken as a critical layer. It follows that the lowest *AER* was found in L3 (z = 4 to 6 m), being 0.21 and 0.19 in the Darfield and Christchurch earthquakes respectively. The N-value is the smallest in L3 among the layers below L1, while L3 seems to have a low relative density as estimated from the CPT profile, around 40%. The liquefaction sequence, in the ascending order of AER, was L3, L2, L5 and L4 (marginally safe). Although liquefaction was predicted with the Kokusho method (2013), the *AER* profile did not differentiate the severity of liquefaction observed during the two events. A thicker layer of liquefiable soil may be expected from the Christchurch motion as liquefaction was more serious.

The SPT stress based method (Idriss & Boulanger, 2008) was applied to the same soil model. Although a profile of stress reduction coefficient r_d can be obtained from the site response analysis, the empirical expression for r_d was adopted in the SPT analysis to examine the performance of the method in its usual, simplified form. Hence, r_d is 0.61 for the Darfield and 0.52 for the Christchurch events. The magnitude scaling factor *MSF* was determined as 1.11 for the Darfield earthquake and 1.41 for the Christchurch earthquake. The cyclic stress ratio *CSR* was higher in the Christchurch due to the greater *PGA* at the location. As it can be seen in **Figure 1b**, no liquefaction was predicted in the Darfield event. For the Christchurch earthquake, the *FoS* was lower than one in L3 and L5, whereas L4's *FoS* was slightly larger than one. L3 was the most vulnerable layer, with its FoS estimated as 1.1 in the Darfield earthquake and 0.7 in the Christchurch earthquake.

The factor of safety profile of the CPT method (Idriss & Boulanger, 2008) was generated and is given in **Figure 1c**. The site was predicted to be susceptible to liquefaction during both earthquakes. The Christchurch scenario would trigger more prominent liquefaction impact as the thickness of liquefied soil was about 6.3 m, comparing to the 3.3m-thick liquefiable layer in the Darfield event. A critical layer at depth of 5 to 9 m was obvious in the Christchurch *FoS* profile, while a thinner layer between 5 and 8 m was identified as critical in the Darfield profile. The results were comparable to that of Green *et al.* (2014) where the identified critical layer was between 5.75 to 8.75 m due to its low soil behaviour index and relative density.

Upon examination, the CPT stress based method's results matched the liquefaction observation at site BUR-46. The Kokusho method although predicted liquefaction occurrence it could not distinguish between the two strong events. The SPT method indicated the presence of liquefiable layers in the Christchurch event, but the *FoS* was unconservative for the Darfield event.

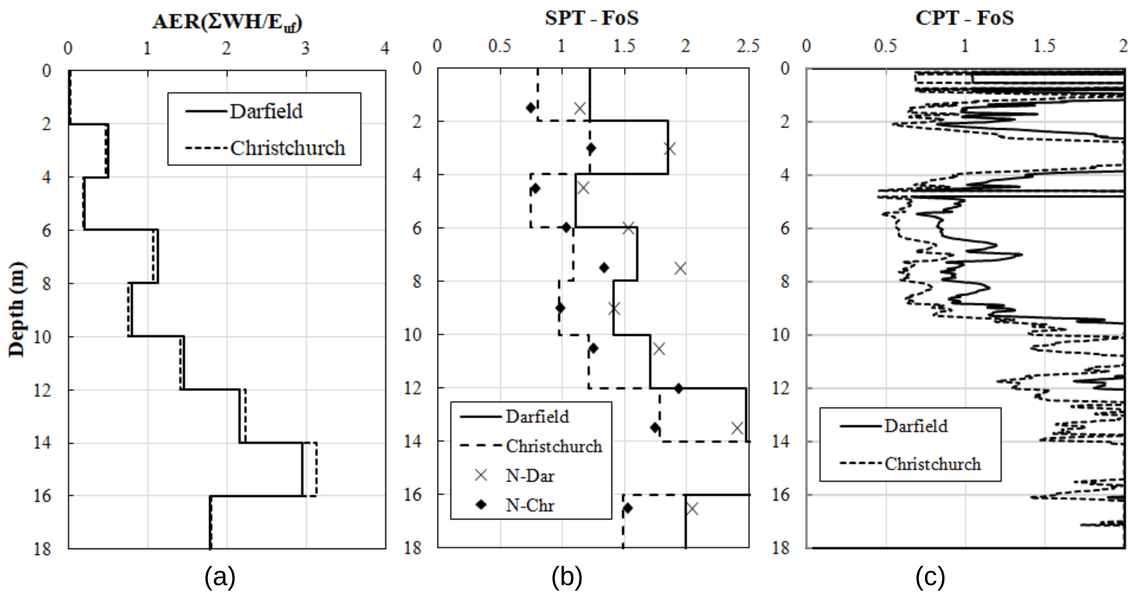


Figure 1. Factor of safety against liquefaction at BUR-46. (a) AER profile of Kokusho method, (b) SPT method: the lines refer to FoS calculation based on N-value assigned to the uniformly-layered model built for the Kokusho method; whereas the scatter points are the results based on corrected N-value at a depth as in the measurement, (c) CPT method

4. Case Studies of the 2016 Mw7.8 Kaikoura Earthquake

New Zealand was threatened again by liquefaction when the M_w7.8 Kaikoura earthquake shook the eastern side of the upper South Island on 14 November 2016. The earthquake reconnaissance report of Cubrinovski and Bray (2017) provides documentation of the liquefaction occurrence. With the available data, three sites in Wellington and Marlborough region were considered suitable as case studies for the purposes of this study.

Based on the GeoNet database, another significant earthquakes struck the area before the Kaikoura event; the M_w6.6 Lake Grassmere earthquake on 16 August 2013 (GNS, 2014; Cubrinovski & Bray, 2017). Since no liquefaction was reported at the three selected sites during the Lake Grassmere event, these scenarios can be considered as non-liquefaction case histories.

Cubrinovski and Bray (2017); Cubrinovski *et al.* (2018) scrutinised the soil characteristics from CPT soundings and documented the liquefaction evidences at CentrePort, an important infrastructure that was built by reclaiming in the Wellington Harbour with quarry and 10 to 20m thick hydraulic fills. However, these post-earthquake CPT data are not yet available in the NZGD database as of April 2019. The selection of case study sites was at CentrePort also constrained by large-scale lateral spreading which took place and only a few spots demonstrated solely sand ejecta. One site, B1-02 located in the northern side of a buried concrete seawall of the port and which CPT profile was presented in Cubrinovski *et al.* (2018) seems to have suffered mainly settlement and insignificant lateral displacement. The nearby station CPLB is part of the instrumentation of the BNZ CentrePort Building array and it was possible to obtain the relevan ground motions from GeoNet.

Two additional sites in Blenheim of Marlborough were identified as appropriate case studies. They refer to Location 1 and 2 in urban Blenheim in the reconnaissance report; it is noted that the loose sands and silts in the eastern part of the township are under high groundwater table of 1 to 2 m depth only, and that liquefaction occurred in many historic earthquakes (Cubrinovski & Bray, 2017). According to the classification in Wotherspoon *et al.* (2015), cracking and sand ejecta surveyed at the two locations may be described as 'minor liquefaction'. As ground shaking in the Blenheim township is monitored by the station Blenheim Marlborough Girls College (MGCS), location 1 and 2 were named as MGCS-Loc1 and MGCS-Loc2 respectively. Since the Kokusho (2013) method was developed for SPT blow counts only, a SPT-CPT correlation was used in the absence of SPT data in the surroundings of sites B1-02 and MGCS-Loc1. The revised version proposed by Lunne et al. (1997) based on Roberstson et al. (1986) is often employed to correlate CPT penetration resistance to SPT N-values (corrected N_{60}), and its applicability to the examined energy based method was investigated by Anatolakis (2016). The Lunne et al. (1997) SPT-CPT expression takes the following form:

$$\frac{q_c/P_a}{N_{60}} = 8.5(1 - \frac{I_c}{4.6}) \tag{6}$$

where *Pa* is the atmospheric pressure. The equivalent SPT-N profile was denoted as CPT-N profile in the subsequent analyses.

5. Comparisons between SBM and EBM

The performance of different liquefaction assessment methods can be compared by examining the predicted critical layer. Apart from the depth, the seismic demand and capacity of the critical stratum were analysed. The outcome of the Kokusho energy based method, SPT and CPT stress based methods at all examined sites are discussed herein.

5.1 Evaluation of the Critical Layer

Green *et al.* (2014) described different combinations of stratigraphy features when selecting a critical layer. Severe surface manifestation usually requires a thick, loose stratum to liquefy; moderate liquefaction implies the liqeufiable layer may be thinner or denser. A number of possibilities appear when the liquefaction is minor. A thin, loose layer with high liquefaction potential, or a thick, dense layer near the ground of marginal FoS can be sufficient to trigger minor liquefaction. What is more, even when the safety factor is high for the entire profile, a shallow layer that develops significant pore pressure can cause minor surface manifestation. This means engineering judgement must be exercised when assigning the critical layer. It is noteworthy that the cumulative nature of energy ratio in the Kokusho method gives a sequence of liquefaction. The layer of the lowest accumulated energy ratio was primarily considered as the critical in the majority of case histories.

Depending on the location, two different layers may be of concern under the Darfield and Christchurch motions for the selected sites. In general, the CPT predictions of this study agreed with the interpretation of Green *et al.* (2014). The Kokusho method using the SPT-N values tended to indicate a liquefiable zone similar to that predicted by the SPT stress based method. At sites SHY-09, AVD-07, KAS-20 and Z4-4, the critical depth differed from the CPT results. When the correlated N-values from CPT resistances were employed in the Kokusho method, a more comparable estimation of critical depth was observed with that of the CPT stress based method.

Details of the critical depth for the three special cases where the CPT method did not identify a critical layer in contrast to the surface observation in the Christchurch earthquake, are provided in **Table 2**. The Kokusho method, using either SPT-N or CPT-N, suggested a different liquefaction potential profile. Except for the output of the Kokusho method with CPT-N at site CPT1396, the AER of the critical strata listed was below or equal to unity. The Kokusho method predicted liquefaction at a shallower depth which is more consistent with the field observations.

Site	Critical depth (m)				
	CPT by Wotherspoon et al. (2015)	**EBM with SPT-N**	**EBM with CPT-N**	**SPT**	**CPT**
CPT484	8-9	6-8 $\boxed{4\text{-}6}$	4-6	4-6	8-9
CPT89	8-9	2-4	2-4	2-4	8-9
CPT1396	25-27	6-8	8-10 $\boxed{16\text{-}18}$	6-8	25-27 $\boxed{14\text{-}15}$

*The identified critical depth for the Christchurch earthquake is in a $\boxed{\text{box}}$ when it is different from that for the Darfield earthquake.

Table 2. Critical depth at the examined sites CPT484 for station CCCC, CPT89 for station HPSC and CPT1396 for station PRPC

The depth of the critical layer at sites in Wellington and Blenheim during the Kaikoura and Lake Grassmere earthquake is given in **Table 3**. It is intriguing to see that despite CPT data were utilised as equivalent N-values in the Kokusho and SPT methods at site B1-02, the predictions are not identical as at site MGCS-Loc1.

Site	Critical depth (m)			
	EBM with SPT-N	**EBM with CPT-N**	**SPT**	**CPT**
B1-02	(-)	2-4 $\boxed{10\text{-}12}$	10-12	2-8.5 $\boxed{4\text{-}5}$
MGCS-Loc1	(-)	0-2	0-2	0-3 $\boxed{0\text{-}1.5}$
MGCS-Loc2	3-4	1-2	3-4	1.6-2.4

*The identified critical depth for the Lake Grassmere earthquake is in $\boxed{\text{box}}$ when it is different from that under the Kaikoura earthquake.

Table 3. Critical depth at the examined sites in Wellington and Blenheim

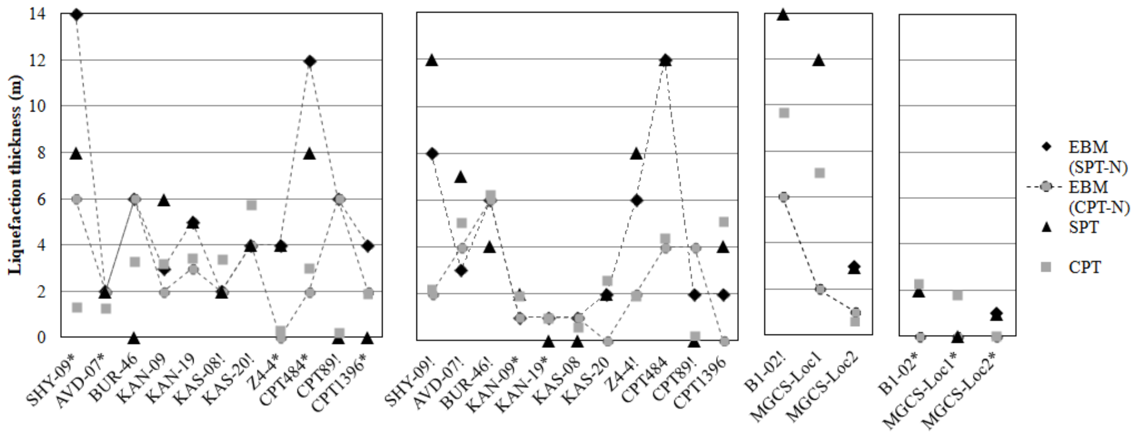
6.2 Predicted Liquefaction Layer Thickness

The considered methods of analyses are compared in Figure 2 in terms of the predicted thickness of the critical layer. Large discrepancies were found at sites SHY-09 and CPT484 for the Darfield and Christchurch earthquake scenarios (see **Figure 2a and 2b**). The results of the Kokusho method using SPT-N overestimated the liquefaction thicknesses at these locations, where no liquefaction was observed in the Darfield event. For cases of moderate or severe liquefaction manifestation (denoted with an asterisk), the liquefaction thickness was generally consistent..

For the sites B1-02 and MGCS-Loc1, the variation in liquefiable stratum thickness predicted by different approaches was obvious during the Kaikoura earthquake (**Figure 2c**). Conservative results were obtained with the stress based methods at the non-liquefaction cases under the Lake Grassmere motion as shown in **Figure 2d.**

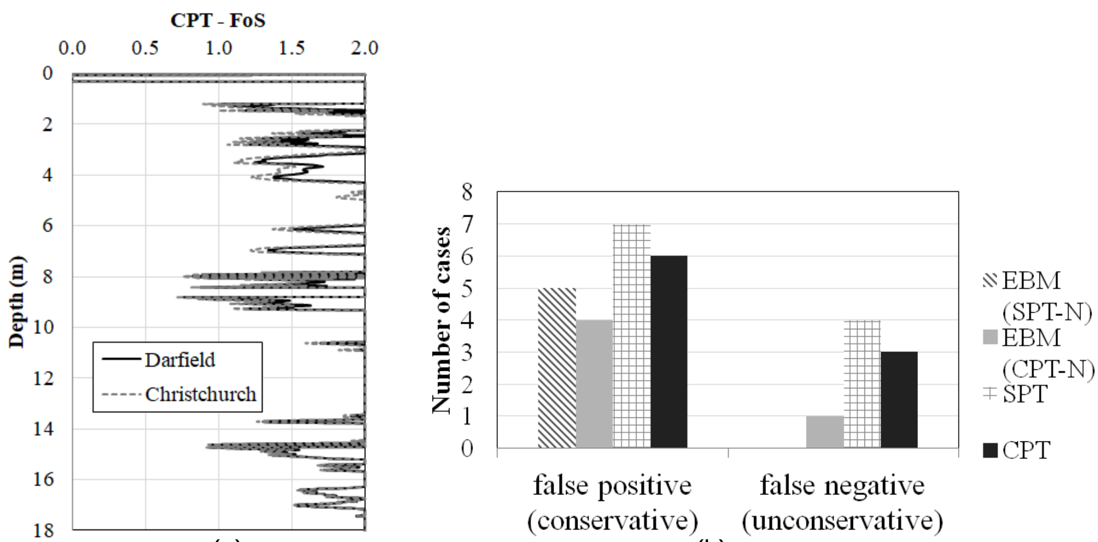
6.3 Overall Trends

Based on the critical depth and liquefaction thickness, false predictions associated with each method can be identified. Here, a non-liquefied case is noted as false positive if the predicted safety factor is lower than unity. As previously explained, false negative case histories are cases with observed liquefaction yet the ground profiles are assessed with factor of safety higher than one as a whole. For instance, in the CPT based FoS profile for HPSC-CPT89 (see **Figure 3a**) the factor of safety was below unity only for a small layer thickness and therefore it is considered as a no-liquefaction prediction. The number of conservative assessment results (false positive) was similar amongst the examined procedures, out of the total 28 case histories (see **Figure 3b**). The Kokusho method performed well in avoiding false-negative predictions. Only one liquefaction case history was found non-conservative under the evaluation of the Kokusho method using CPT equivalent N values, whereas the number of non-conservative cases was higher in both the SPT and CPT method.



(a) (b) (c) (d)

*Figure 2. Thickness of liquefiable layer at sites in wider Christchurch during the (a) Darfield and (b) Christchurch earthquakes, in Wellington and Blenheim during the (c) Kaikoura and (d) Lake Grassmere earthquakes. Note: * is to denote no liq. while ! refers to moderate or severe liq Minor liq. was observed at sites that do not have a symbol.*



(a) (b)

Figure 3. (a) CPT-based factor of safety against liquefaction at HPSC-CPT89. (b) Number of false predictions in the case histories

Furthermore, the results of the examined liquefaction assessment methods are evaluated in terms of liquefaction resistance and seismic demand. A set of penetration resistance and seismic demand corresponding to that determined at the critical depth was extracted out of the analysis output for each case history. From the plots of $CSR_{M7.5}$ (cyclic stress ratio corrected to magnitude M7.5) against adjusted N-value $(N_1)_{60cs}$ in **Figure 4a**, the data of the SPT method demonstrated significant scatter. In **Figure 4b**, the data of adjusted q_{c1Ncs} corresponding to liquefaction observation were mostly above from the empirical triggering $CRR_{M7.5}$ (cyclic resistance ratio corrected to magnitude M7.5) curve. Compared to the SPT results, the CPT data set was closer to the $CRR_{M7.5}$ curve derived from previous CPT based case studies. There seems to be apparent discrepancy between **Figure 3** and **Figure 4** in representing the number of false negative prediction if the data points below the empirical curve in **Figure 4** are to be evaluated. The points of **Figure 4** were computed as a quantitative measure of the soil penetration resistance and seismic demand at the critical layer, irrespective of its thickness. It is acknowledged that the aspect of liquefaction thickness prediction was not incorporated in the case history data plots and hence false negative cases could not be represented clearly. In those false negative case histories counted in **Figure 3**, the estimated liquefaction thickness, as shown in **Figure 2**, was very small in contrast to the field observations.

The graphs of the Kokusho method using SPT-N (**Figure 5a**) and that using the CPT-correlated N value (**Figure 5b**) displayed a considerable varied pattern. The scatter of liquefaction cases data was reduced in the latter plot, suggesting that it may be possible to delineate an upward energy threshold to separate the liquefaction from non-liquefaction observations. However, the final upward energy was not corrected for a reference earthquake magnitude. Identifying a similar limiting to the $CRR_{M7.5}$ curve for the Kokusho method requires further investigation.

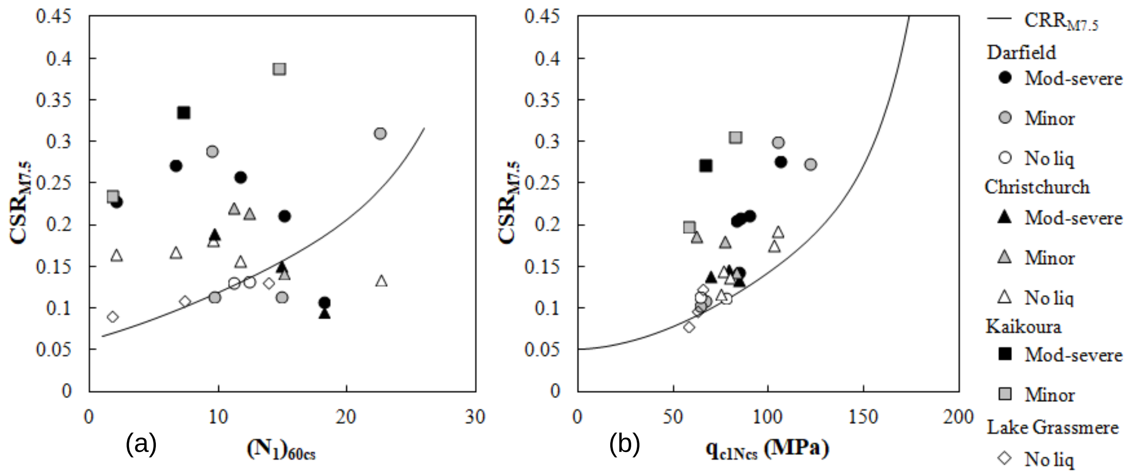


Figure 4. Case history data for the (a) SPT and (b) CPT methods

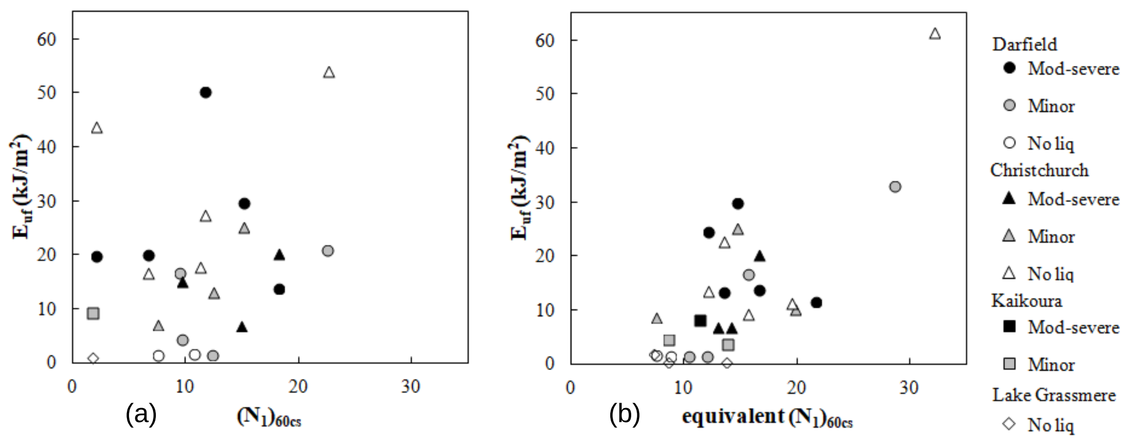


Figure 5. Case history data for the Kokusho method using (a) SPT-N value and (b) CPT equivalent N-value

6.4 Discussion

The comparison presented above is largely influenced by the reliance of surface liquefaction observation. This has been a longstanding major limitation in liquefaction evaluation, as non-liquefaction cases might actually have a certain liquefied stratum but not visual damage. The accuracy or effectiveness of an assessment procedure should be considered together with all the assumptions made.

A number of correlations are involved in both the stress and energy based methods. Fine content profiling is essential in every approach. While the SPT based method can be benefited from *FC* measurement on retrieved samples, this piece of information is often unavailable along the whole profile. CPT-FC correlations may be brought into such situation to improve our interpretation. When implementing the Kokusho method in the various case studies, the shear wave velocity was derived from CPT data. A check may be carried out to investigate how sensitive the predictions of Kokusho method are to the upper and lower bounds adopted in the V_s correlation. The use of equivalent N-value with the Kokusho method was also studied, and remarkable differences were found in the predictions associated with the two sets of N-values. Due to the discretisation of soil model in the Kokusho method, a single value of penetration resistance is needed.

The seismic demand exerted into a soil deposit is calculated based on different principles. The *CSR* formula in the SPT and CPT methods is calibrated from past case histories, while the upward energy is calculated using the motion series and output of one-dimensional equivalent linear site response analysis in the Kokusho method. The stress based methods are likely to be favoured for their simplicity, more well-defined framework and much wider case history database. Nevertheless, the energy demand computation can be more representative of the motion characteristics and dynamic soil behaviour.

6. Conclusions

This study presents liquefaction case histories during the Darfield and Christchurch earthquakes of the 2010-2011 Canterbury Earthquake Sequence, as well as during the 2013 Lake Grassmere and 2016 Kaikoura earthquakes. In total, eleven sites in the wider Christchuch area were analysed for the first two events. Three sites in Wellington and Marlboroguh regions were examined for the last two earthquakes. These case history sites were chosen for their proximity to strong motion stations and availability of site investigation data.

The predictions from the three considered liquefaction assessment approaches were compared and contrasted, in terms of liquefaction critical depth, layer thickness, data scatter and number of false predictions. For methods using SPT data, i.e. the SPT stress based and Kokusho method with liquefaction resistance derived directly from SPT-N values, the case history data showed significant scatter. The CPT stress-based predictions were found to be close to the existing $CRR_{M7.5}$ curve, while the assessment results of the Kokusho method using CPT-N correlation seems to suggest a limiting curve of final upward energy for liquefaction occurrence based on the case histories in New Zealand.

The use of the Kokusho method may be revised with the introduction of CPT penetration resistances. More case analyses will help understand the sensitivity of the method to different parameters so that a more refined procedure can be developed.

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References

Anatolakis, G. (2016) *A critical comparison of stress and energy based methods for evaluation of liquefaction potential (MSc thesis)*. Imperial College London.

Boulanger, R. & Idriss, I. (2014) CPT and SPT based liquefaction triggering procedures. *Report No. UCD/CGM-14/01*.

Chen, Y.R., Hsieh, S.C., Chen, J.W. & Shih, C.C. (2005) Energy-based probabilistic evaluation of soil liquefaction. *Soil Dynamics and Earthquake Engineering*. 25, 55–68.

Cubrinovski, M. & Bray, J.D. (2017) *Geotechnical Reconnaissance of the 2016 Mw7.8 Kaikoura, New Zealand Earthquake*. [Online]. Available from: www.geerassociation.org.

Cubrinovski, M., Bray, J.D., de la Torre, C., Olsen, M., et al. (2018) Liquefaction-induced damage and CPT characterization of the reclamations at Centreport, Wellington. *Bulletin of the Seismological Society of America*. 108, 1695–1708.

Davis, R.O. & Berrill, J.B. (1996) Liquefaction susceptibility based on dissipated energy: a consistent design methodology. *Bulletin of the New Zealand Society for Earthquake Engineering*. 29 (2), 83–91.

GNS (2014) *GeoNet*. [Online]. 2014. Available from: http://www.geonet.org.nz.

Green, R.A., Cubrinovski, M., Cox, B., Wood, C., et al. (2014) Select Liquefaction Case Histories from the 2010-2011 canterbury earthquake sequence. *Earthquake Spectra*. 30

(1), 131–153.
Idriss, I.M. & Boulanger, R.W. (2008) *Soil liquefaction during earthquakes*. Engineering Monographs on Miscellaneous Earthquake Engineering Topics ; MNO-12. Oakland, CA, Earthquake Engineering Research Institute.
Kokusho, T. (2013) Liquefaction potential evaluations: energy-based method versus stress-based method. *Canadian Geotechnical Journal*. 50, 1088–1099.
Kokusho, T. & Mimori, Y. (2015) Liquefaction potential evaluations by energy-based method and stress-based method for various ground motions. *Soil Dynamics and Earthquake Engineering*. 75, 130–146.
Kottke, A.R., Wang, X. & Rathje, E.M. (2013) *Technical Manual for Strata*. [Online]. Available from: http://www.gnu.org/licenses/.
Lunne, T., Robertson, P.K. & Powell, J.J.M. (1997) *Cone penetration testing in geotechnical practice*. New York, Blackie Academic & Professional .
McGann, C.R., Bradley, B.A., Taylor, M.L., Wotherspoon, L.M., et al. (2015) Development of an empirical correlation for predicting shear wave velocity of Christchurch soils from cone penetration test data. *Soil Dynamics and Earthquake Engineering*. 75, 66–75.
National Research Institute for Earth Science and Disaster Resilience of Japan (2018) *Strong-motion seismograph networks (K-net, KiK-net)*. [Online]. 2018. Available from: http://www.kyoshin.bosai.go.jp [Accessed: 2 July 2018].
NZGD (2016) *New Zealand Geotechnical Database - MBIE & EQC*. [Online]. 2016. Available from: https://www.nzgd.org.nz.
Orense, R.P., Hickman, N.A., Hill, B.T. & Pender, M.J. (2014) Spatial evaluation of liquefaction potential in Christchurch following the 2010/2011 Canterbury earthquakes. *International Journal of Geotechnical Engineering*. 8 (4), 420–425.
Papaspiliou, M., Kontoe, S. & Bommer, J.J. (2012) An exploration of incorporating site response into PSHA-Part I: Issues related to site response analysis methods. *SOIL DYNAMICS AND EARTHQUAKE ENGINEERING*. 42, 302–315.
Robinson, K., Cubrinovski, M. & Bradley, B.A. (2014) Lateral spreading displacements from the 2010 Darfield and 2011 Christchurch earthquakes. In: *Proc. 19th NZGS Geotechnical Symposium*. 2014 Wellington, New Zealand. p.
Tsaparli, V., Kontoe, S., M G Taborda, D. & M Potts, D. (2018) Liquefaction triggering due to compressional waves: validation through field records. In: *16th European Conference on Earthquake Engineering*. June 2018 Thessaloniki. p.
Vucetic, M. (1994) Cyclic Threshold Shear Strains in Soils. *Journal of Geotechnical Engineering*. 120 (12), 2208–2228.
Wood, C.M., Cox, B.R., Wotherspoon, L.M. & Green, R.A. (2011) Dynamic site characterization of Christchurch strong motion stations. *Bulletin of the New Zealand Society for Earthquake Engineering*. 44 (4), 195–204.
Wotherspoon, L., Orense, R., Bradley, B. & Wood, C.M. (2015) *Geotechnical Characterization of Christchurch Strong Motion Stations V3. Earthquake Commission Report (Project No. 12/629)*. [Online]. Available from: doi:10.13140/RG.2.1.2376.0720.