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Liquefaction assessment by seismic dilatometer test (SDMT) after 2010-2011 Canterbury earthquakes (New Zealand)

S. Amoroso[1], P. Monaco[2], K.M. Rollins[3], M. Holtrigter[4], A. Thorp[5]

ABSTRACT

The 2010-2011 Canterbury earthquakes (New Zealand) provided many case histories of liquefaction and lateral spreading. In the aftermath several seismic dilatometer (SDMT) tests were performed in Christchurch within the scope of the Ground Improvement Trials Report for the Earthquake Commission (EQC). SDMT-based liquefaction analyses were conducted to validate the use of existing correlations for deriving the cyclic resistance ratio *CRR* from the horizontal stress index K_D and to increase the CRR-K_D case history database. The results obtained at different sites show a variable trend and point out the need for further investigation on the influence of factors, such as the fines content, not taken into account by current CRR-K_D correlations.

Introduction

The seismic dilatometer (SDMT) provides two parallel independent estimates of the liquefaction resistance (cyclic resistance ratio *CRR*), one from the shear wave velocity V_S and the other from the DMT horizontal stress index K_D. The use of V_S for evaluating *CRR* is well known. CRR-K_D correlations have been developed in the last two decades, stimulated by the recognized sensitivity of K_D to a number of factors which are known to influence liquefaction resistance and are difficult to sense by other tests. These include stress history, prestraining/aging, structure (though it is not possible to separate the individual contribution of each factor), and by its correlation with the state parameter (see Monaco et al. 2005). The paper illustrates and comments on the liquefaction assessment by SDMT test based on results obtained in Christchurch, New Zealand, in combination with the other available investigation data. The site campaigns were conducted within the scope of the Ground Improvement Trials Project (EQC 2013) for EQC, MBIE, NEES, and U.S. NSF. The study was commissioned in response to the need for ground improvement following the 2010-2011 earthquakes.

Site investigations by SDMT

In December 2013 SDMTs were carried out at 16 different trial test sites. The test locations are plotted in Figure 1, superimposed on a map that identifies liquefaction and lateral spreading.

[1]Researcher, Istituto Nazionale di Geofisica e Vulcanologia, L'Aquila, Italy, sara.amoroso@ingv.it
[2]Assistant Professor, Dept. Civil, Arch. Env. Engineering, University of L'Aquila, Italy, paola.monaco@univaq.it
[3]Full Professor, Dept. Civil Env. Engineering, Brigham Young University, Provo, Utah, USA, rollinsk@byu.edu
[4]Director, Ground Investigation Ltd, Aukland, New Zealand, marco@g-i.co.nz
[5]Technical Manager, Ground Investigation Ltd, Aukland, New Zealand, alan@g-i.co.nz

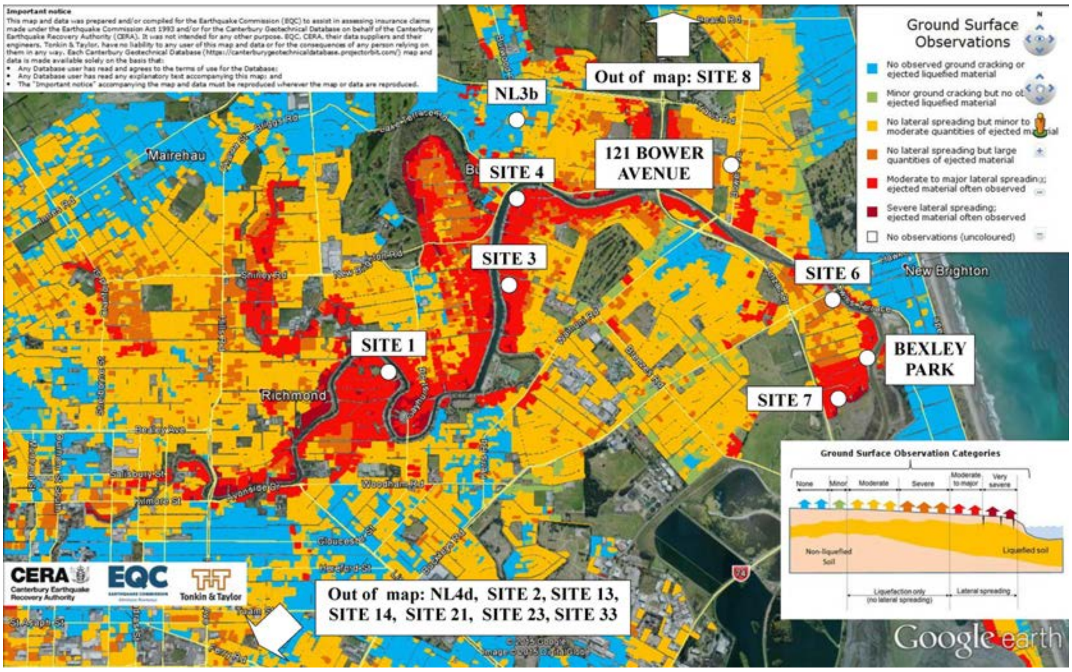

Figure 1. "Liquefaction and Lateral Spreading Observations" following the 22nd February 2011 earthquake (Canterbury Geotechnical Database - CGD 2013) with location of SDMT test sites.

This paper is focused on site investigations and liquefaction analyses performed at Site 3 (Wainoni) and Site 7 (Waireka Lane). Both sites are close to the Avon River and composed of sands and silty sands that liquefied during the 2010-2011 earthquake sequence. Site 3 was classified as affected by "No lateral spreading but minor to moderate quantities of ejected material" in the 4th September 2010 earthquake and by "Moderate to major lateral spreading; ejected material often observed" in the 22nd February 2011 earthquake. Site 7 is described as affected by "Severe lateral spreading; ejected material often observed" in both earthquakes. Stretching and cracking due to lateral spreading could create pathways for the liquefied material to eject through to the ground surface. Therefore the prediction of vulnerability to the liquefaction hazard by simplified procedures could be misleading, since it does not include the damage caused by the lateral spreading hazard. It is debatable whether or not lateral spreading case histories should be included in a liquefaction triggering database (Green et al. 2014). The limited case history database of current DMT-based methods may possibly include cases where liquefaction manifested in the form of lateral spreading; therefore such methods were applied at all sites investigated by SDMT in the Christchurch area. Site 3 and Site 7 were selected because of the difference in the profiles of K_D, which influences the liquefaction response. It is expected that the values of K_D obtained in the SDMT soundings were not significantly influenced by lateral spreading for Site 3. The location of site investigations at Site 3 and Site 7 is shown in Figures 2a and 2b. The results of piezocone (CPTu) and cross hole (CH) tests, as well as borehole logs (BH) and grain size distribution curves, were downloaded from the CGD (2015). The results obtained from CPTu and SDMT are summarized in Figures 3 and 4, which show the borehole logs with the values of fines content (*FC*), and the profiles with depth of: the corrected cone resistance q_t and the soil behavior type index I_c obtained from CPTu; the material index I_D, the horizontal stress index K_D and the constrained modulus M_{DMT} (Marchetti 1980); the lateral earth pressure coefficient K_0 evaluated from DMT and CPT for freshly deposited sands (Baldi et al. 1986); the shear wave velocity V_S measured by SDMT and CH. At Site 3 CPTu (I_c) and SDMT (I_D) compared well with BH, while at Site 7 the borehole log shows high fines content ($FC \approx$ 28-90%). The sandy silt layers have a plasticity index $PI \approx$ 0-8%, natural water content w_c $\approx$ 28-30%, liquid limit LL $\approx$ 28%). They were then assigned a "sand-like" liquefaction behaviour.

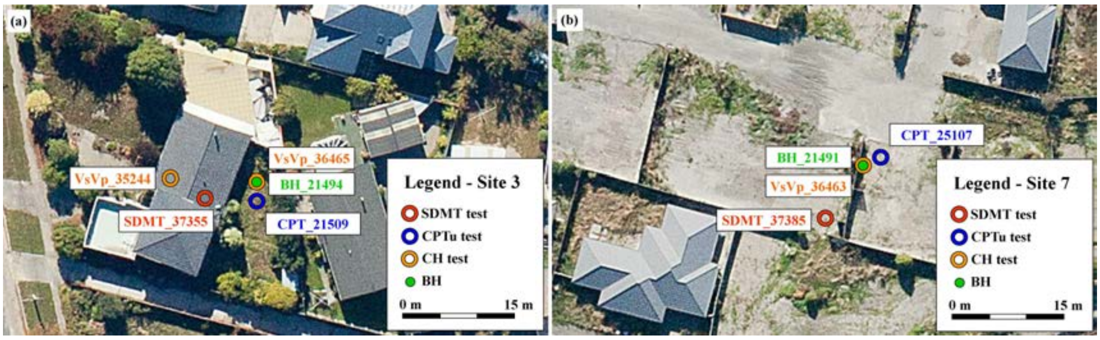

Figure 2. Site investigation performed at Site 3 - Wainoni (a) and Site 7 - Waireka Lane (b).

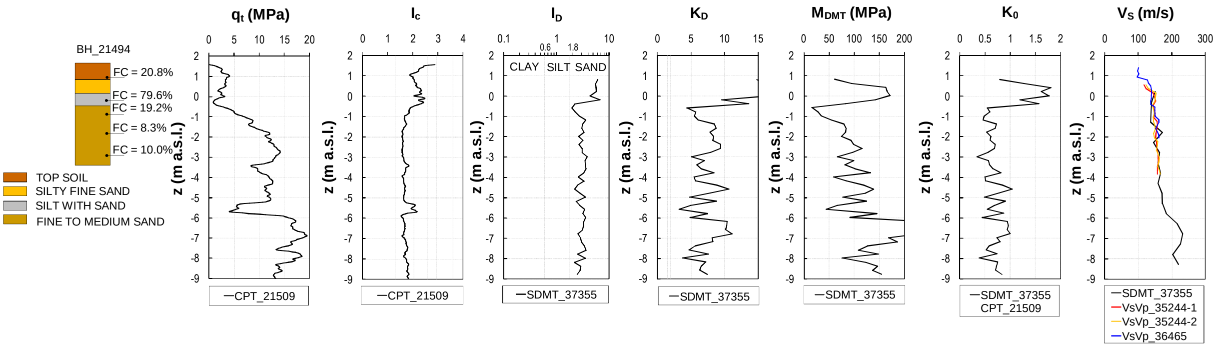

Figure 3. Borehole log, CPTu and SDMT results at Site 3 - Wainoni.

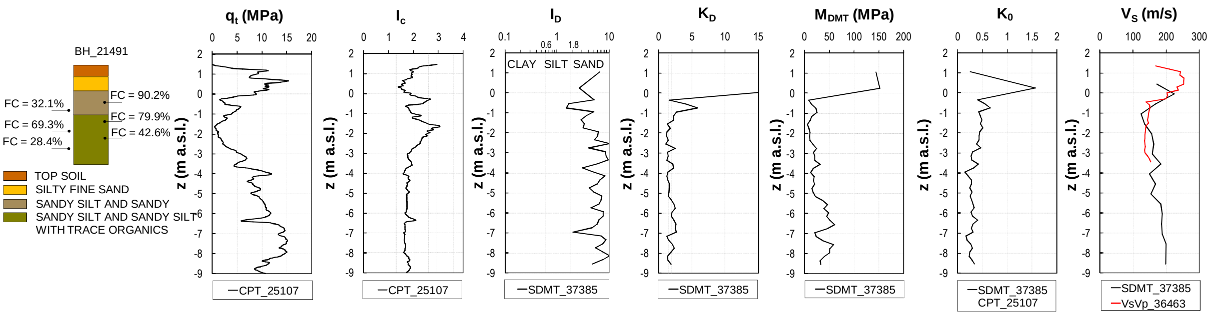

Figure 4. Borehole log, CPTu and SDMT results at Site 7 - Waireka Lane.

Liquefaction analyses

The liquefaction analyses were carried out according to the "simplified procedure". The cyclic stress ratio *CSR* was estimated by Seed and Idriss (1971) formulation. Magnitude Scaling Factor

(MSF) and shear stress reduction coefficient *(r_d)* were evaluated according to Idriss and Boulanger (2008) for CPTu and DMT data, and according to procedures proposed by Andrus and Stokoe (2000) and Kayen et al. (2013) for V_S measurements. The peak horizontal acceleration a_{max} was defined for the 4th September 2010 and the 22nd February 2011 earthquakes (Bradley and Hughes 2012a, 2012b). Table 1 summarizes the parameters that identify the two main shocks at Site 3 and Site 7, including the moment magnitude M_w. The groundwater table levels (*GWT*) in Table 1 were provided by Tonkin and Taylor Ltd (2013) and CGD (2014).

Table 1. Seismic actions and groundwater table levels at Site 3 and Site 7.

	Site 3 - SDMT_37355 (1.22 m asl)		**Site 7 - SDMT_37385 (1.45 m asl)**	
Earthquake	04/09/2010	22/02/2011	04/09/2010	22/02/2011
M_w	7.1	6.2	7.1	6.2
a_{max} (g)	0.19	0.41	0.19	0.57
GWT (m asl)	0.32	0.42	0.29	0.45

The cyclic resistance ratio *CRR* was derived from CPTu and SDMT results, using correlation with the normalized cone tip resistance q_{c1N} (Idriss and Boulanger 2008), the horizontal stress index K_D (Monaco et al. 2005, Tsai et al. 2009, Robertson 2012) and the overburden stress corrected shear wave velocity V_{S1} (Andrus and Stokoe 2000, Kayen et al. 2013), introducing the fines content *FC* obtained from sieve analyses, in combination with *FC* estimated from CPT. The *CRR*-q_{c1N}, *CRR*-K_D and *CRR*-V_{S1} correlations were corrected introducing a partial saturation factor (*PSF*), inferred from compression wave velocity V_P (Tsukamoto et al. 2002). V_P measurements from CH tests are available on the CGD (2015), and *PSF* values were evaluated by the Ground Improvement Trials Project (EQC 2013). The influence of the *PSF* on the results was found to be negligible at Site 3 and very small at Site 7, where the increase in *CRR* in the topmost 4.50 m was 11 to 26%. Results of the liquefaction analyses for the 22nd February 2011 earthquake are illustrated in Figures 5 and 6. Each diagram shows the profiles with the absolute elevation (z) of: the soil behavior type index I_c or the material index I_D; the parameter used in each case for evaluating *CRR*: q_{c1N}, V_{S1} or K_D; *CSR*, divided by *MSF*, compared to *CRR*; the liquefaction safety factor F_L = *CRR* / (*CSR* / *MSF*). The comparison of the results obtained at Site 3 (Figure 5) indicate disagreement between the three methods. q_{c1N} indicates liquefaction at depths from 1 to -1 m and in thin layers from -3.5 to -4 m and -5 to -6 m; V_{S1} predicts liquefaction from -2 to -6 m and in a thin layer at about -1 m. The *CRR*-K_D correlations by Tsai et al. (2009) and Robertson (2012) predict liquefaction only in a few thin isolated layers at various depths, while the Monaco et al. (2005) correlation appears to overpredict *CRR*. On the other hand, at Site 7 (Figure 6) all methods based on K_D indicate extended liquefaction from -1 to -8 m, in broad agreement with q_{c1N}, while V_{S1} suggests that much of this layer is not liquefiable. A possible reason for the higher K_D (and K_0) found at Site 3, is that these sand layers could have become denser due to settlement as a result of repeated liquefaction events, but not dense enough to prevent liquefaction occurrence in a subsequent strong earthquake. Such increase in relative density D_R and K_0 may have affected K_D more than q_{c1N} or V_S, in agreement with the available experience (see e.g. Marchetti et al. 2001). Another factor to analyze is the influence of fines content, not taken into account by current *CRR*-K_D correlations, based on clean uncemented sand which may then underestimate *CRR* in the case of high *FC*. At Site 7 *CRR* from K_D appears to be lower compared to *CRR* from q_{c1N}. The sensitivity of K_D to changes in K_0 may be greater than the sensitivity of liquefaction resistance to changes in K_0. Some of the discrepancies appear to be associated with layers having higher K_0 values while in layers with lower K_0 values, typical of normally consolidated soil, the agreement seems to improve. Similar concerns have been raised about the influence of K_0 on liquefaction resistance and penetration resistance for SPT and CPT tests (Harada et al. 2008). This issue is particularly important for cases involving ground improvement where both K_0 and D_R increase and is a topic for further study.

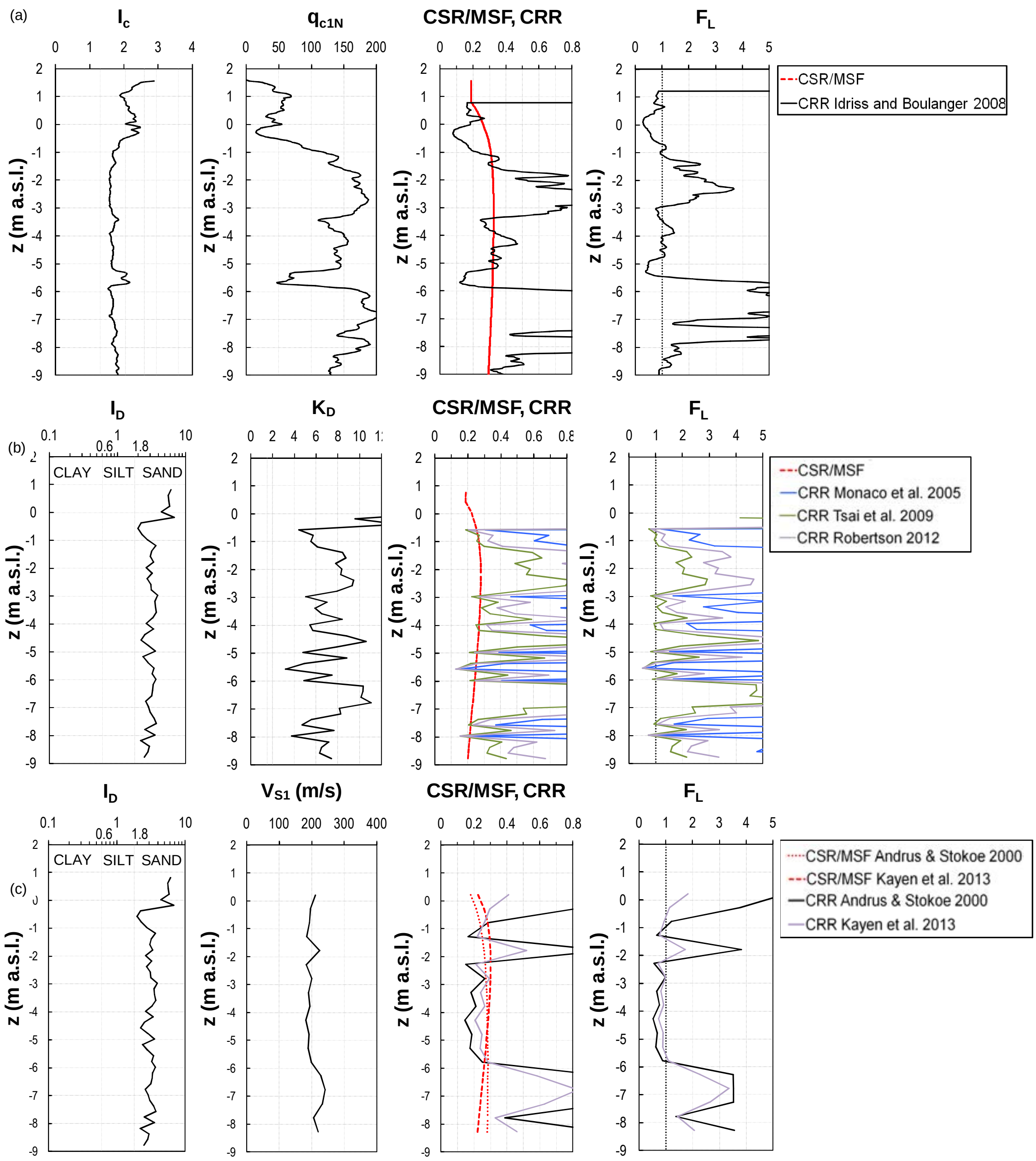

Figure 5. Site 3 (SDMT_37355-CPT_21509)-Liquefaction analyses by CPT (a), DMT (b), V_S (c).

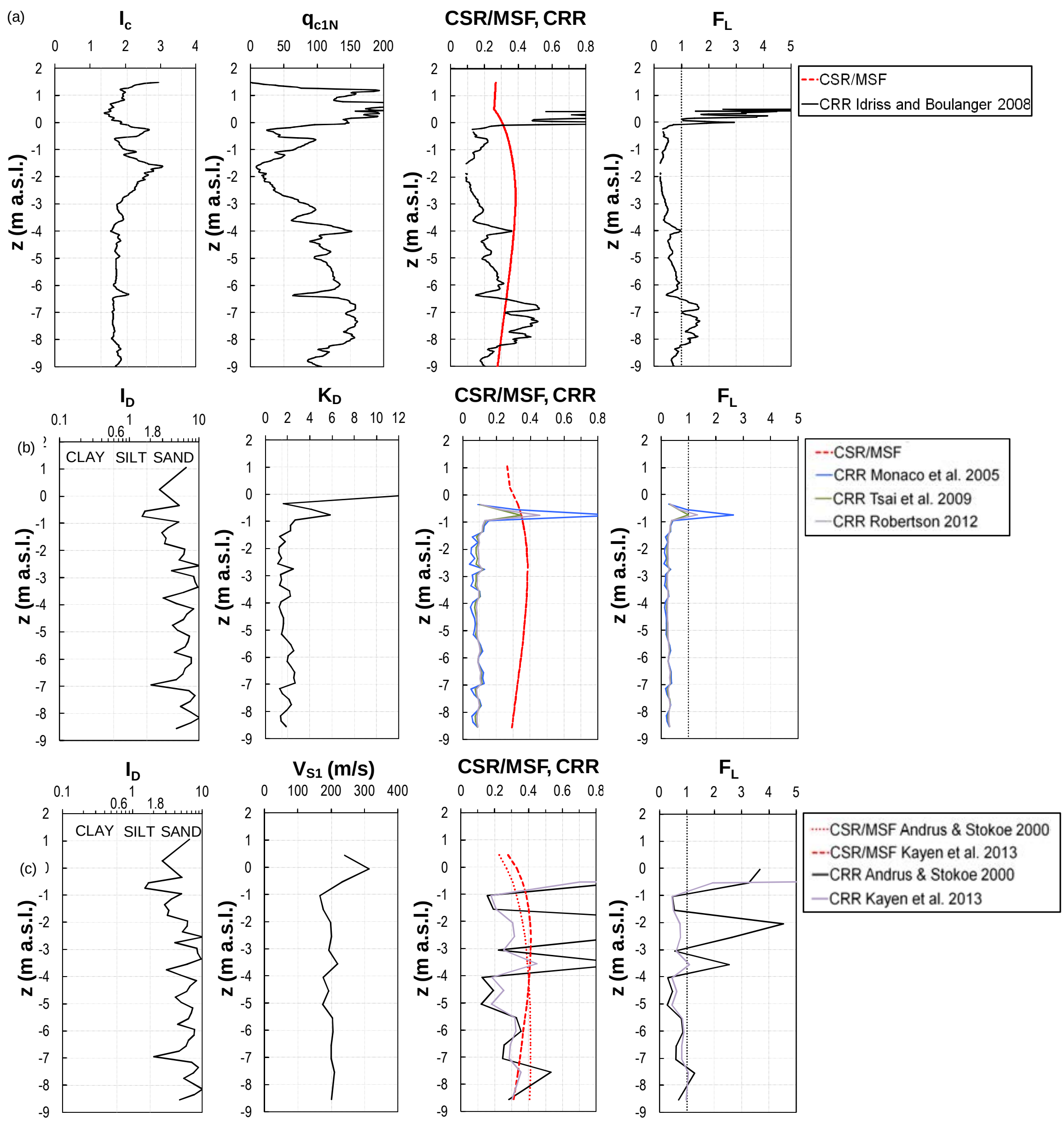

Figure 6. Site 7 (SDMT_37385-CPT_25107)-Liquefaction analyses by CPT (a), DMT (b), V_S (c).

Integral liquefaction vulnerability indicators were also calculated, namely the liquefaction potential index *LPI* (Iwasaki et al. 1982) and the liquefaction severity number *LSN* (Tonkin and Taylor Ltd 2013) considering the postliquefaction volumetric strain ε_v (Zhang et al. 2002) and estimating the equivalent clean sand normalized cone resistance $(q_{c1N})_{cs}$ using correlations with K_D and V_S data from Robertson (2012). A comparison between the actual damage observations, and the *LPI* and *LSN* values calculated by CPT (Idriss and Boulanger 2008), V_S (Andrus and Stokoe 2000), and DMT (Tsai et al. 2009) is presented in Table 2. It should be reminded *LPI* and *LSN* are not intended to be reliable indicators of vulnerability in case of significant lateral spreading, where alternate measures that include consideration of lateral spreading are required to make an appropriate assessment of liquefaction land damage (van Ballegooy et al. 2014).

Table 2. Comparison of *LPI* and *LSN* and actual liquefaction damage observations. The colour scale fits for the observed damage: blue is underpredict, red is over predict, green is reasonable.

	Site 3		Site 7	
Earthquake	04/09/2010	22/02/2011	04/09/2010	22/02/2011
Observed damage	No lateral spreading but minor to moderate quantities of ejected material	Moderate to major lateral spreading; ejected material often observed	Severe lateral spreading; ejected material often observed	Severe lateral spreading; ejected material often observed
Predicting damage using CPT data	*LPI* = 2.98	*LPI* = 9.94	*LPI* = 5.99	*LPI* = 11.29
	LSN = 66.88	*LSN* = 105.76	*LSN* = 66.40	*LSN* = 81.79
Predicting damage using V_S data	*LPI* = 1.69	*LPI* = 10.29	*LPI* = 2.48	*LPI* = 17.46
	LSN = 13.15	*LSN* = 20.91	*LSN* = 11.26	*LSN* = 29.27
Predicting damage using DMT data	*LPI* = 0.08	*LPI* = 2.51	*LPI* = 20.26	*LPI* = 44.19
	LSN = 1.60	*LSN* = 9.03	*LSN* = 89.63	*LSN* = 94.27

Conclusions

The results provided by current SDMT-based simplified procedures at different sites in Christchurch show a variable trend, which depicts the need for further investigation on possible influencing factors (e.g. fines content, cementation, mineralogy, grain shape etc.), as well as on the influence of lateral spreading. The Canterbury earthquakes liquefaction case history database offers a valuable opportunity to enlarge the data set used to develop the existing CRR-K_D correlations. It appears that no current method can accurately predict liquefaction at every site. It is suggested that an approach involving various investigation methods is used for higher risk projects. Further research is required to understand better the influence of K_0.

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