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

S. Amoroso¹, P. Monaco², K.M. Rollins³, M. Holtrigter⁴, A. Thorp⁵

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.

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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.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Site 3 - SDMT_37355 (1.22 m asl)</th>
<th>Site 7 - SDMT_37385 (1.45 m asl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_w</td>
<td>04/09/2010 22/02/2011</td>
<td>04/09/2010 22/02/2011</td>
</tr>
<tr>
<td>a_{max} (g)</td>
<td>0.19 0.41</td>
<td>0.19 0.57</td>
</tr>
<tr>
<td>GWT (m asl)</td>
<td>0.32 0.42</td>
<td>0.29 0.45</td>
</tr>
</tbody>
</table>

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 $K_D$. 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).
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 $\varepsilon_v$ (Zhang et al. 2002) and estimating the equivalent clean sand normalized cone resistance ($q_{c1N}$) 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 Ballegoooy 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.

<table>
<thead>
<tr>
<th></th>
<th>Site 3</th>
<th>Site 7</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Earthquake</strong></td>
<td>04/09/2010</td>
<td>04/09/2010</td>
</tr>
<tr>
<td></td>
<td>22/02/2011</td>
<td>22/02/2011</td>
</tr>
<tr>
<td><strong>Observed damage</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>No lateral spreading but minor to moderate quantities of ejected material</td>
<td>Moderate to major lateral spreading; ejected material often observed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Severe lateral spreading; ejected material often observed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Severe lateral spreading; ejected material often observed</td>
</tr>
<tr>
<td><strong>Predicting damage using CPT data</strong></td>
<td>LPI = 2.98</td>
<td>LPI = 9.94</td>
</tr>
<tr>
<td></td>
<td>LSN = 66.88</td>
<td>LSN = 105.76</td>
</tr>
<tr>
<td><strong>Predicting damage using V₅ data</strong></td>
<td>LPI = 1.69</td>
<td>LPI = 10.29</td>
</tr>
<tr>
<td></td>
<td>LSN = 13.15</td>
<td>LSN = 20.91</td>
</tr>
<tr>
<td><strong>Predicting damage using DMT data</strong></td>
<td>LPI = 0.08</td>
<td>LPI = 2.51</td>
</tr>
<tr>
<td></td>
<td>LSN = 1.60</td>
<td>LSN = 9.03</td>
</tr>
</tbody>
</table>

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-KD 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 K₀.

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