Earthquake-induced liquefaction triggering of Christchurch sandy soils

M.L. Taylor, M. Cubrinovski & B.A. Bradley

Department of Civil Engineering, University of Canterbury, Christchurch

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ABSTRACT: The empirical liquefaction triggering chart of Idriss and Boulanger (2008) is compared to direct measurements of the cyclic resistance of Christchurch silty sands via undisturbed and reconstituted lab specimens. Comparisons suggest that overall there is a reasonable agreement between the empirical triggering curve and the interpreted test data. However, the influence of fines on cyclic resistance appears to be over-predicted by the empirical method, particularly for non-plastic silty sands that are commonly encountered in flood over-bank deposits in Christchurch and nearby settlements.

1 INTRODUCTION

Liquefaction results in a significant loss of strength of saturated loose to medium dense cohesionless soils. It was a major contributing factor to the high cost of the damage to land, buildings and infrastructure associated with the recent 2010-2011 Canterbury earthquake sequence (CES).

The evaluation of liquefaction triggering in response to earthquake-induced ground motion (cyclic) loading is routinely evaluated using an empirical method that relies on proxy indicators of both cyclic demand and resistance from estimated surface peak ground acceleration (PGA) and measured ground penetration resistance (SPT or CPT resistance) following historical earthquake events where observations of liquefaction were documented (referred to as the "simplified" or *Seed-Idriss* method (Seed and Idriss 1971; Seed et al. 1985; Youd et al. 2001)). The interpreted relationship between demand and resistance is inferred from those few case histories that lie on the lower-bound range of cyclic demand for a given penetration resistance. Boulanger and Idriss (2014) have collated some 252 and 253 case histories for the development of SPT and CPT-based empirical triggering evaluation methods respectively. Despite the large number of case histories gathered in the last 50 years, the number of events that significantly influence the position of the curve remains relatively small; particularly the case for dense soils, and soils with significant 'fines' content (silt and clay fraction).

There is an inherent epistemic uncertainty in empirical liquefaction datasets, which researchers have attempted to capture via a rigorous *probabilistic* evaluation of the case history data and the selection of the position of the triggering curve (e.g. Seed et al. (2003); Boulanger and Idriss (2012)). It has been established that the *deterministic* empirical liquefaction triggering curve commonly adopted for assessing a factor of safety against liquefaction triggering, FS_{Liq}, corresponds to a probability of liquefaction, P_L, of ~15%.

The Ministry of Business Innovation and Employment (MBIE) has proposed the Idriss and Boulanger (2008) (**IB08**), and the more recent Boulanger and Idriss (2014) (**BI14**) revisions of the simplified method, as appropriate for the evaluation of liquefaction hazard in the rebuild of Canterbury (MBIE 2012). The latter incorporates many new case histories including 50 from the Canterbury Earthquake Sequence (CES), revisions to the interpretation of some earlier case histories, and the influence of fines on liquefaction triggering.

In this paper the direct testing on undisturbed and reconstituted specimens of Christchurch sandy soils are compared to the **IB08** empirical liquefaction triggering chart. The results of penetration testing close by the sampling holes were used to develop a correlation between penetration testing and soil relative density D_R, including the influence of soil gradation characteristics. An initial comparison was presented in Taylor et al. (2013b), which is revised, extended and summarised in this paper.

2 DIRECT TESTING OF THE CYCLIC STRENGTH OF CHRISTCHURCH SANDS

Recently, direct laboratory testing of the cyclic resistance of Christchurch sandy soils has been conducted, including consideration of the influence of fines content (*FC*). Cubrinovski et al. (2010) presented undrained cyclic triaxial testing (CTX) on reconstituted specimens of typical Christchurch sands of the Springston Formation (*Sp. Fm.*) derived from flood over-bank deposits, *Fitzgerald Bridge Mixtures* (FBM) and *Pinnacle Sand Mixtures* (PSM)– both comprising fine sand with varying proportions of non-plastic silt (*FC* range 0 – 30 %). Immediately following the recent earthquakes, undisturbed sampling using a novel Gel-push (GP) method was performed at two sites in the Central Business District (CBD) of Christchurch (Taylor et al. 2012). These samples were subjected to cyclic tri-axial testing and feature important natural fabric and structure effects on the cyclic response (Taylor et al. 2013a). The soils tested included clean marine sands (SP) of the Christchurch Formation (*Ch. Fm.*), and silty sands (SM) and sandy silts (ML) of the *Sp. Fm.*, with predominantly non-plastic fines between 0 and ~98% (refer Fig. 1).

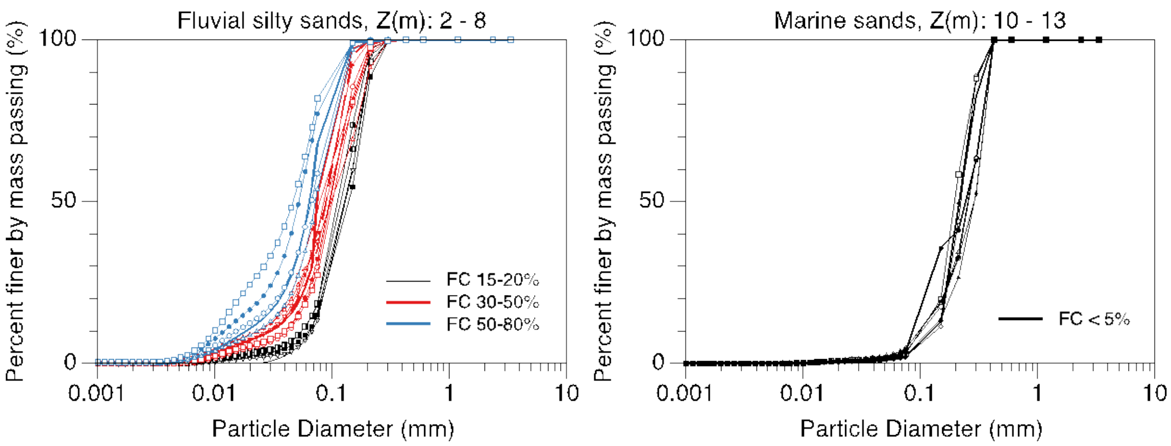

Figure 1. Particle size distribution curves for GP specimens obtained from site ‘K1’ in the Christchurch CBD. Left: Springston Formation silty sands, Right: Christchurch Formation clean sands.

The results of CTX testing are interpreted by constructing a cyclic strength curve (cyclic stress ratio *CSR* vs. N_c, where N_c is the number of cycles to cause ‘liquefaction’, typically defined using a strain-based proxy of 5 % axial strain in double amplitude). Ideally three or more CTX tests are performed on specimens with consistent material properties and state, under different cyclic stress amplitudes, are used to construct the *CSR* vs. N_c curve. An example cyclic strength curve is shown in Figure 2.

The intercept of this curve at 15 cycles, i.e. CRR_{15}, may be plotted on the liquefaction triggering chart (CRR_{15} vs. SPT $(N_1)_{60}$ or CPT q_{c1N}). This requires correlation between penetration testing and the relative density, D_R of the soil, or performing penetration testing close to the sampling borehole in the case of undisturbed specimens. It also requires the *CRR* derived from the laboratory test to closely replicate the cyclic resistance of the soil in the field.

3 INTERPRETATION OF CYCLIC STRENGTH TESTING

The natural variations present in undisturbed specimens add complexity in characterising cyclic strength, with specimens exhibiting variation in soil gradation, state, layering and fabric with depth. The adopted approach involved using fines content (*FC*) to group the undisturbed specimens by ‘soil type’, similar to USCS classification (SP, SM, ML), and correcting *CSR* values for the field condition and normalising confining stress to 1 atmosphere (using the **IB08** K_σ factor). The K_σ factor considers the influence of changes in state on the dilatancy of the soil, as it affects the cyclic resistance.

Corrections for the field condition incorporate differences in the stress-regime of the testing (isotropic, uni-directional cyclic triaxial) vs. the field condition (anisotropic, bi-directional simple shear).

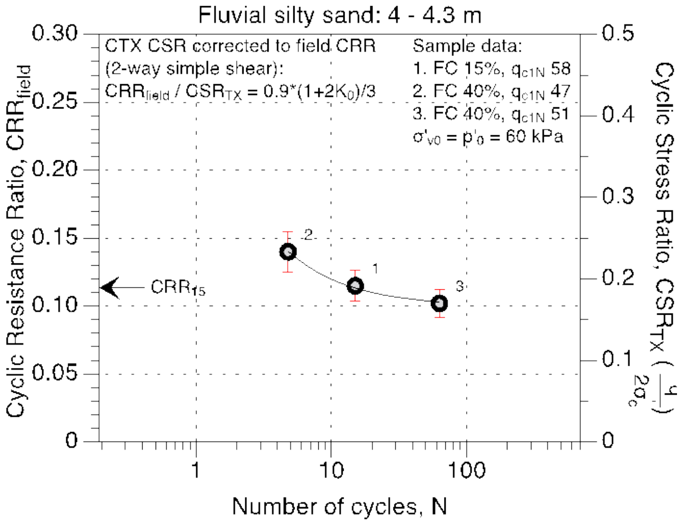

Figure 2. Example cyclic strength curve derived from three cyclic triaxial tests on undisturbed specimens of Christchurch silty sand. Vertical axes display test measured cyclic strength (right axis), and interpretation of the equivalent field cyclic resistance (left axis). Intercept at 15 cycles (*CRR_{15}*) is indicated. Presumed error associated with correction to field resistance (+/- 10%) shown as red bars.

The typical correction factor, C_r applied is expressed as follows (Yoshimi et al. 1994; Idriss and Boulanger 2008):

$$C_r = \frac{CRR_{\text{field}}}{CRR_{\text{TX}}} = 0.9\frac{(1 + 2 \times K_0)}{3} \tag{1}$$

Where K_0 is the at-rest earth pressure coefficient ($K_0 = \sigma_h / \sigma_v$). For normally consolidated sands, K_0 is typically 0.45 - 0.5, resulting in a C_r factor of ~ 0.6. A wide range of C_r are reported in the literature, with C_r shown to be a function of state, fabric and soil type (Tatsuoka et al. 1986; Vaid and Sivathayalan 1996; Jefferies and Been 2006). For soils with states dense of critical, it has been observed that C_r may vary by typically ± 10 %. The influence of fabric and soil type was found to be too complex to consider in a simple relationship, but Equation (1) was found to be typically conservative and thus adopted for the interpretation of the Christchurch sands test data. The ± 10 % range was retained as the expected uncertainty with the correlation (all samples had in-situ states dense of critical). Further testing using a cyclic direct simple shear device would be needed to confirm the relationship. The example cyclic strength curve in Figure 2 shows corrected field cyclic resistance including the anticipated error associated with the correction.

Once corrected in this manner, tests performed on undisturbed samples with similar soil type (using *FC* range as a proxy) and also density (using q_{c1N} as a proxy after grouping by *FC*) were grouped together and cyclic strength curves constructed (Fig. 3). Selected 'representative' specimens from each *FC* range were subjected to repeated tests conducted on moist-tamped (MT) reconstituted specimens. These were likewise corrected to equivalent field CRR_{15} values.

4 ESTIMATING NORMALISED PENETRATION TEST VALUES.

In the process of testing undisturbed GP specimens, the density of the specimen is obtained by measuring the post-consolidation void ratio, e_c, and post-test measurements of the void ratio limits, e_{max} and e_{min} for the sample (Japanese Standard maximum and minimum dry density method). Thus the relative density, D_R, of each sample may be obtained:

$$D_R = \frac{e_{max} - e_c}{e_{max} - e_{min}} \tag{2}$$

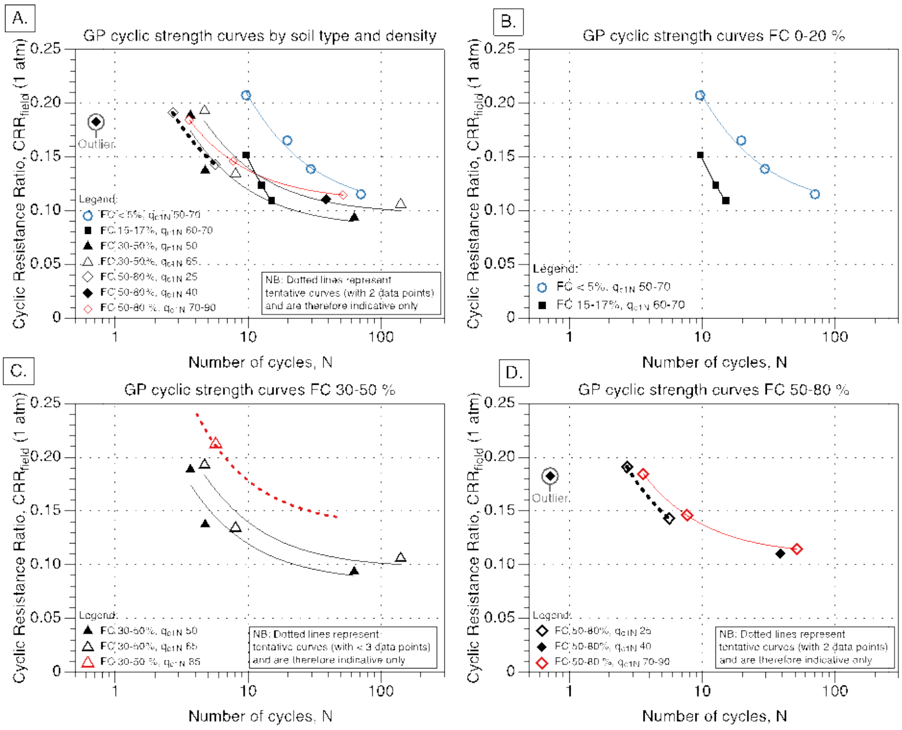

Figure 3. GP sample cyclic strength curves developed by soil type: geological formation, fines content (*FC*) and penetration resistance in that order, data points corrected for the field condition and to 1 atmosphere. A: All data points and curves. B: Selected curves for soils with *FC* 0-20%, C: Selected curves for soils with *FC* 30-50%, D: Selected curves for soils with 50-80%.

Cubrinovski and Ishihara (1999) present a correlation between SPT N and D_R using a data set of 61 high quality frozen samples ranging in gradation from silty sands (up to FC 14 %), to sands and gravels. They represent the relationship between stress-normalised (to 1 atm) SPT blow-count at a standard Japanese SPT energy ratio of 78%, $(N_1)_{78}$ and the square of relative density, $D_R^{\,2}$ in the following form:

$$C_D = \frac{(N_1)_{78}}{D_R^{\,2}} \tag{3}$$

They showed that the compressibility factor, C_D, is a function of the granular characteristics of the soil, in particular *void ratio range,* $(e_{\max} - e_{\min})$, expressed with the following relationship form:

$$C_D = \frac{\alpha}{(e_{\max} - e_{\min})^{\beta}} \tag{4}$$

where α and β are curve fitting coefficients, and for their dataset 9 and 1.7 respectively were suggested. In this study the GP sample data were compared to this relationship through use of a published correlation between SPT $(N_1)_{60}$ and CPT q_c (Jefferies and Davies 1993), making use of soil behaviour type index, I_c:

$$(N_1)_{78} = \frac{(N_1)_{60}}{C_E} = \left(\frac{q_{c1N}}{C_E \cdot 8.5\left(1 - \frac{I_c}{4.6}\right)} \right) \tag{5}$$

where q_{c1N} is the stress-normalised cone resistance, and C_E the energy ratio between North American and the Japanese standard SPT, equal to the ratio 78/60.

The C_D values calculated from sample D_R and CPT-based estimates of $(N_1)_{78}$ are compared to the published dataset in Figure 4A, and generally follow the same trend. Differences observed may be on account of both soil characteristics but also compression occurring due to sampling disturbance. Samples were excluded from the correlation if they were considered to be of lower quality. The published dataset contains few specimens with a void ratio range > 0.6, while there are a significant number are in this study. A site-specific correlation presented in the figure provides α and β coefficients of 7 and 1.657 respectively, with a coefficient of determination, R^2 of 0.62. For the silty sands, a better correlation was obtained with *FC* rather than void ratio range (refer Fig. 4B), possibly as the naturally obtained soils were fairly uniformly graded, and thus *FC* has a high correlation with both median and the effective grain sizes (D_{50} and D_{10} respectively). It has an R^2 of 0.95 and the following form:

$$C_D = 30.75 \exp\left(-\frac{FC}{54.38}\right) \tag{6}$$

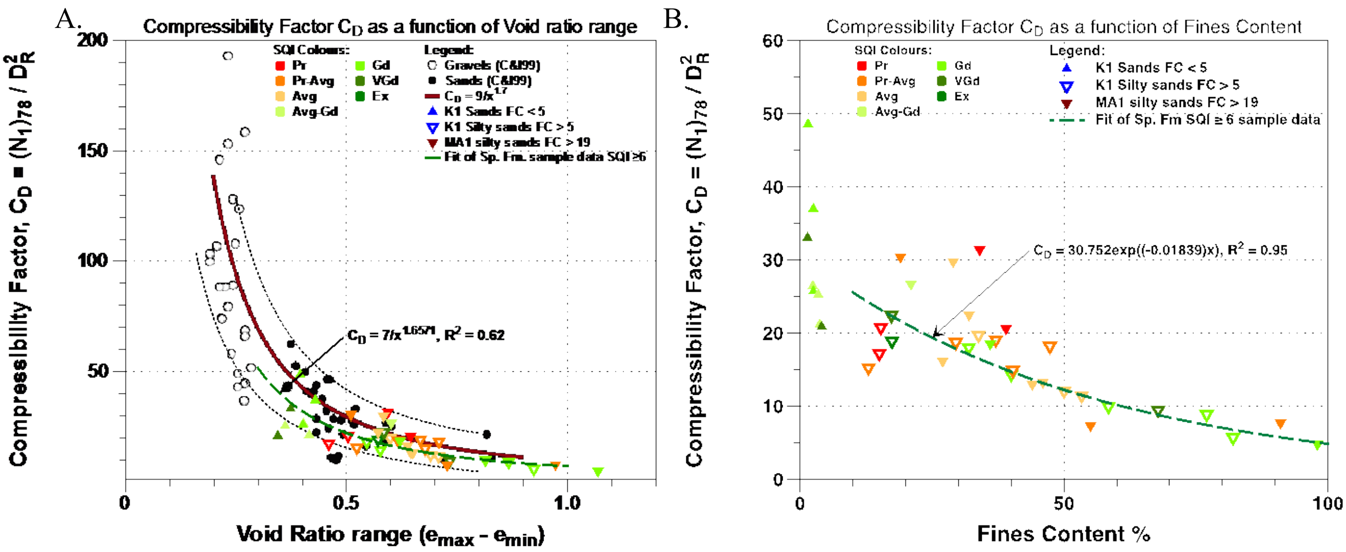

Figure 4. Compressibility factor, C_D as a function of void ratio range (left) and fines content (right). Data points are coloured by specimen quality index (SQI). Relationships established for 'better than average quality' (green shaded). Data from Cubrinovski and Ishihara (1999) in grey circles.

A site-specific *FC*-I_c correlation for Christchurch silty sands ($FC > 10$ %) was also used with equation 5, it is based on high quality GP samples with adjacent CPT test to the sample hole, it has an R^2 of 0.53, and the following form:

$$I_c = 1.79 + \frac{FC}{133.583} \tag{7}$$

Equations 3, 5, and 7 together allow for the estimate of a corresponding q_{c1N} for Christchurch silty sands from known specimen D_R and soil gradation proxy *FC*:

$$q_{c1N} = \left(C_E \cdot C_D \left(\frac{D_R}{100}\right)^2\right) \cdot \left(5.2 - \frac{FC}{72.5}\right) \tag{9}$$

where C_D is from equation (6) for silty sands and (4) for clean sands, and C_E remains 78/60.

The uncertainty in the estimate may be captured by considering the uncertainty associated with the correlation between C_D and soil index properties (*FC* or void ratio range).

5 COMPARISON OF CHRISTCHURCH SOIL CYCLIC STRENGTH DATA

The cyclic resistance derived from cyclic triaxial tests performed on either GP or MT samples, regardless of gradation have been plotted on the empirical liquefaction triggering chart (CRR_{15} vs. q_{c1N}). Due to the limited GP sample test data, MT reconstituted specimens have been used to supplement the Christchurch silty-sand dataset and provide a comparison to the empirical liquefaction triggering curve. The MT data comprises the results presented by Rees (2010) (specimens of FBM, and PSM soils). Further testing of MT specimens of selected representative soils among the range of GP specimens obtained in this study have also been included in this comparison.

Figure 5 presents four **IB08** triggering curve plots, with data separated by soils type. The appropriate triggering curve for the corresponding soil test data is shown in red, with the deterministic triggering curve representing P_L of 15% (lowest curve corresponding to 'median - σ'), while P_L of 50% and 85% are also shown corresponding to 'median' and 'median + σ' respectively. The case-history data of Moss (2003) are presented in black dots, while blue dots are from Christchurch case histories collated by Green et al. (2014).

The following observations are made:

- For clean sands (Fig. 5A), the GP sample data point (blue triangle) lies on the P_L of 50% curve (solid red curve). The reconstituted (MT) data points (red and black) of similar q_{c1N} position slightly lower, closer to P_L of 15% (lower red-dashed curve). MT data at lower q_{c1N} are positioned above the P_L of 85% curve (upper red-dashed curve). Six case history data points from Canterbury Earthquake Sequence (CES) plot below the P_L of 50% curve, with three lying on the P_L of 15% curve. Given the uncertainty with the position of the data points the new results offer no improvement over the deterministic **IB08** triggering curve which offers reasonable prediction for Christchurch soils with low/no fines over the range of densities considered in these tests.
- For silty sands with *FC* 10-20% (Fig. 5B), the GP sample data (two data points – blue triangles) plot below but close to the P_L of 15% triggering curve for soils with FC 15%. Two points from the global dataset (black dots) plot on or below this curve, while a further three points from the CES case histories plot below this curve (due to the uncertainty in I_c-FC correlation used to estimate FC for these case histories these data-points may not be reliable). The MT specimens plot close to the P_L of 50% triggering curve for soils with FC 15%, and uniformly sit above the GP specimens. The data suggests good agreement with the **IB08** triggering curve. Field case-histories and tests on undisturbed specimens indicate slightly lower cyclic resistance than assumed in the simplified method.
- For silty sands with *FC* 30-50% (Fig. 5C), the non-plastic GP samples (blue triangles, pointing upwards) lie well below the **IB08** deterministic triggering curve (P_L of 15%) for soils with $FC \geq 35\%$. This is supported by MT specimens between *FC* 25% and 40% which generally lie on or below the P_L of 15% triggering curve. Again GP specimens generally have lower cyclic resistance than MT specimens of the same *FC* and D_R. Few case history data points in this FC range exist in either the global database, or the recently acquired CES case histories. The majority of points lie above and to the left of the **IB08** FC ≥ 35% deterministic triggering curve. However, two CES case histories and one from the international database do plot well below the triggering curve, seemingly in agreement with the test data. A single GP sample (blue triangle pointing downwards) plots in agreement with the P_L of 50% triggering curve. This sample exhibited slight plasticity (PI 8) suggesting the higher cyclic resistance of soils with significant fines may account for this effect.
- For sandy silts with FC 50-100% (Fig. 5D), the corresponding IB08 triggering curve is also for FC ≥ 35%, which represents the upper-limit for the influence of fines on liquefaction resistance considered in the method. Three GP specimens are plotted in various positions due to variation in plasticity. No/ low plasticity (PI < 5) specimens (GP and MT) plot at or below the P_L of 50% triggering curve, while a moderate plasticity specimen (PI 12) plots well above the P_L of 85% triggering curve. No case histories from the CES were interpreted as having $FC > 50\%$, however four data points from the international case history database plot below the P_L of 15% triggering curve. Overall the specimens with low/no plasticity typical of Christchurch silty sands lie close to the deterministic triggering curve.

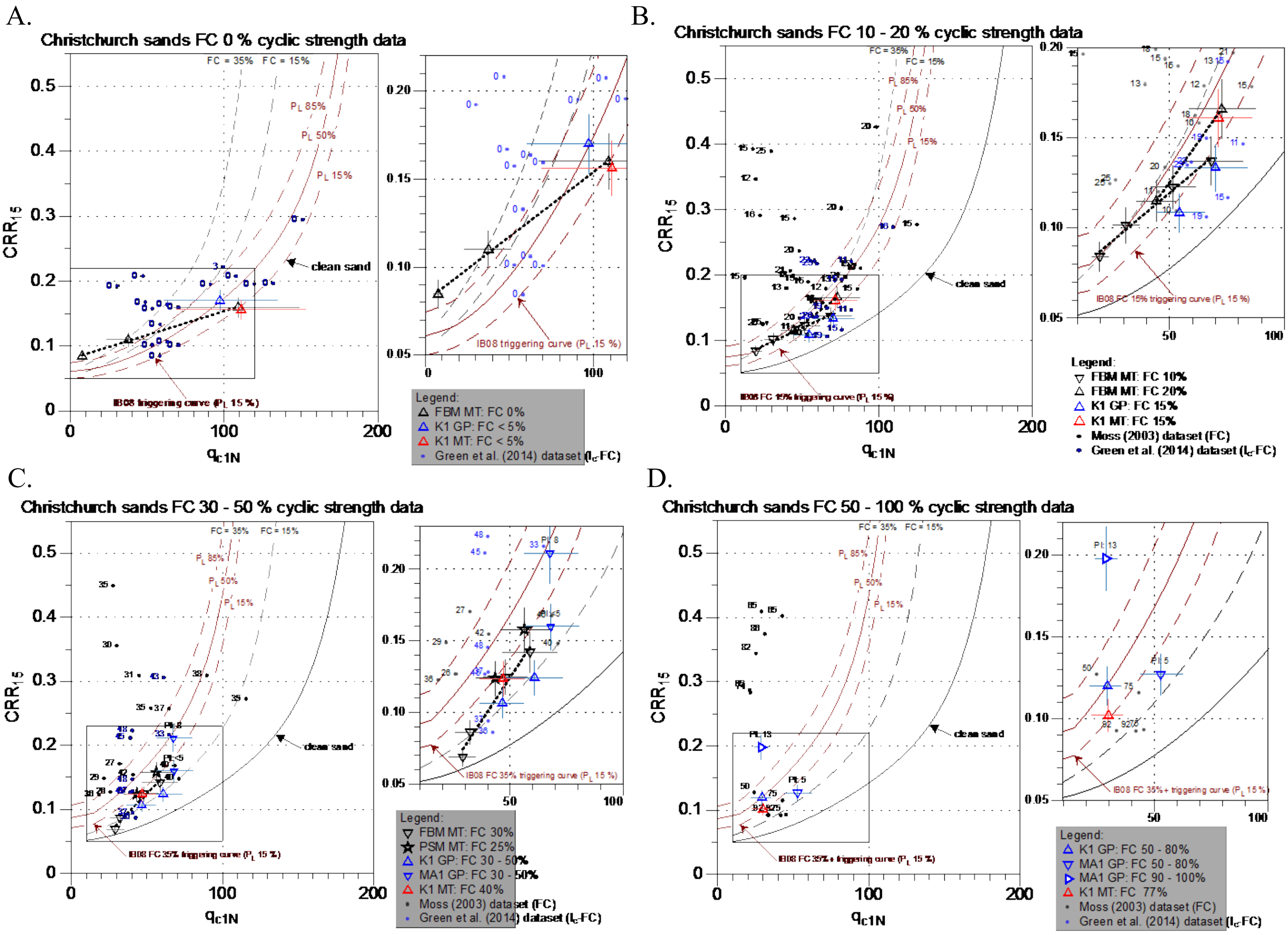

Figure 5. Comparison of cyclic strength of GP and MT specimens with the liquefaction triggering chart (IB08). A) FC 0%, B) FC 10-30%, C) FC 30-50%, D) FC 50-100%. International case history data from Moss (2003) and from the Canterbury Earthquake Sequence from Green et al. (2014) also plotted.

6 DISCUSSION AND CONCLUSIONS

This paper presents the results of direct testing of the cyclic resistance of undisturbed specimens of Christchurch silty sands in the laboratory, interpreted so as to compare directly to the empirical liquefaction triggering chart of Idriss and Boulanger (2008) (**IB08**) used in conventional engineering practice ("simplified method"). The interpretation involves corrections for field conditions in-situ confining stress. An estimate of the equivalent normalised cone resistance, q_{c1N}, is presented by using high-quality GP sample data and nearby CPT data to develop a relationship between q_{c1N} and soil D_R as a function of soil gradation characteristics. A series of plots present the comparison wherein the probability of liquefaction is used to highlight the expected range of uncertainty in the empirical triggering curve. The **IB08** triggering curve appears to provide a reasonable estimate of the triggering curve for Christchurch sands, acknowledging that the deterministic triggering curve is moderately conservative (P_L of 15%) to account for inherent uncertainties. However, it appears for soils with significant non-plastic fines, typical of many soils in Christchurch and surrounding regions where flood over-bank deposits are silty and exhibit low/no plasticity, that the influence of fines content to increase the resistance for a given penetration resistance may be significantly over-predicted by the method. The influence of plasticity is known to significantly increase cyclic resistance, and was identified in some of the test results presented.

For sites with significant non-plastic fines, therefore, the inherent conservatism of the triggering curve model is fully utilised to obtain a reasonable estimate of the liquefaction resistance, leaving little room for model error. For engineering design, a more conservative accounting for the influence of non-plastic fines may be warranted than considered by the Idriss and Boulanger method. Further research into the influence of fines on the cyclic resistance of Christchurch sands is on-going at University of Canterbury.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

Boulanger RW, Idriss IM 2012. Probabilistic Standard Penetration Test-Based Liquefaction-Triggering Procedure. *Journal of Geotechnical and Geoenvironmental Engineering* 138(10): 1185-1195.

Boulanger RW, Idriss IM 2014. *CPT and SPT Based Liquefaction Triggering Procedures*. April 2014 ed. Davis, CA., Dept. Civil & Environmental Engineering, University of California at Davis.

Cubrinovski M, Ishihara K 1999. Empirical correlation between SPT N-value and relative density for sandy soils. *Soils and Foundations* 39(5): 67-71.

Cubrinovski M, Rees S, Bowman E 2010. Effects of non-plastic fines on liquefaction resistance of sandy soils. In: Ansal A ed. *Earthquake Engineering in Europe. Geotechnical, Geological, and Earthquake Engineering 17*. London & New York, Springer.

Green RA, Cubrinovski M, Cox B, Wood CM, Wotherspoon L, Bradley BA, Maurer B 2014. Select Liquefaction Case Histories from the 2010-2011 Canterbury Earthquake Sequence. *Earthquake Spectra* 30(1): 131-153.

Idriss IM, Boulanger RW 2008. *Soil Liquefaction during Earthquakes*. Oakland, CA, Earthquake Engineering Research Institute.

Jefferies M, Davies M 1993. Use of CPTU to estimate equivalent SPT N60. *Geotechnical Testing Journal* 16(4): 458-468.

Jefferies M, Been K 2006. *Soil liquefaction : a critical state approach*. London ; New York, Taylor & Francis. xxxi, 479 p. p.

MBIE 2012. Assessing, repairing and rebuilding foundations in TC3. *Repairing and rebuilding houses affected by the Canterbury earthquakes*. 3rd ed. Wellington, Ministry of Business Innovation and Employment. Pp. 110.

Moss RES 2003. CPT-based Probabilistic Assessment of Seismic Soil Liquefaction Initiation. *Doctor of Philosophy thesis*, University of California, Berkeley.

Rees S 2010. Effects of fines on the undrained behaviour of Christchurch sandy soils. *Doctor of Philosophy thesis*, University of Canterbury, Christchurch. 282 p.

Seed HB, Idriss IM 1971. Simplified procedure for evaluating soil liquefaction potential. *Journal of the Soil Mechanics and Foundations Division, ASCE* 107(SM9): 1249-1274.

Seed HB, Tokimatsu K, Harder LF, Riley MC 1985. Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations. *Journal of geotechnical engineering* 111(12): 1425-1445.

Seed RB, Cetin KO, Moss RES, Kammerer AM, Wu J, Pestana JM, Riemer MF, Sancio RB, Bray JD, Kayen RE and others 2003. Recent advances in soil liquefaction engineering: A unified and consistent framework. *26th Annual ASCE Los Angeles Geotechnical Spring Seminar*. H.M.S. Queen Mary, Long Beach, California.

Tatsuoka F, Ochi K, Fujii S, Okamoto M 1986. Cyclic Undrained Triaxial and Torsional Shear Strength of Sands for Different Sample Preparation Methods. *Soils and Foundations* 26(3): 23-41.

Taylor ML, Cubrinovski M, Haycock I 2012. Application of new 'Gel push' sampling procedure to obtain high quality laboratory test data for advanced geotechnical analyses. *2012 New Zealand Society for Earthquake Engineering Conference*. Christchurch, New Zealand, NZSEE.

Taylor ML, Cubrinovski M, Bradley BA 2013a. Cyclic strength testing of Christchurch sands with undisturbed samples. *2013 New Zealand Society for Earthquake Engineering Conference*. Wellington, NZSEE (Oral Paper No. 11).

Taylor ML, Cubrinovski M, Bradley BA, Horikoshi K 2013b. The undrained cyclic strength of undisturbed and reconstituted Christchurch sands. In: Chin CY ed. *Proceedings 19th NZGS Geotechnical Symposium.*, New Zealand Geotechnical Society

Vaid YP, Sivathayalan S 1996. Static and cyclic liquefaction potential of Fraser Delta sand in simple shear and triaxial tests. *Canadian Geotechnical Journal* 33(2): 281-289.

Yoshimi Y, Tokimatsu K, Ohara J 1994. In situ liquefaction resistance of clean sands over a wide density range. *Géotechnique* 44(3): 479-494.

Youd TL, Idriss IM, Andrus RD, Arango I, Castro G, Christian JT, Dobry R, Liam Finn WD, Harder L.F, Jr., Hynes ME and others 2001. Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. *Journal of Geotechnical and Geoenvironmental Engineering* 127(10): 817-833.