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Effect of Liquefaction Triggering Uncertainty on Liquefaction Consequence

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ABSTRACT

Boulanger and Idriss (2014) included liquefaction assessment triggering uncertainty, P_L, in their liquefaction assessment methodology. It was developed from case history data and provides correlations for P_L of 15, 50 and 85%. This paper comprises two parts. The first part of this paper compares the predicted extent and severity of liquefaction for P_L = 15, 50 and 85%, using the Christchurch Geotechnical Database (CGD) Cone Penetration Test (CPT) dataset, with the observed extent and severity of liquefaction following each of the main events in the 2010 to 2011 Canterbury Earthquake Sequence (CES). The second part of this paper examines the sensitivity of the predicted CPT-based liquefaction vulnerability for P_L = 15, 50 and 85% for different levels of seismic hazard (i.e., the 25, 100 and 500 year return period ground motions). The results show that the observed extent and severity of liquefaction following each of the main CES events is enveloped within the range of predicted extent and severity for the P_L = 15 and 85% cases. The results also show that the predicted liquefaction extent and severity in Christchurch at the 25 and 100 year return period ground motions is very sensitive to the P_L parameter, whereas at the 500 year return period ground motions the predicted liquefaction extent and severity is generally not sensitive to the P_L parameter.

Introduction & Background

Detailed land damage mapping was undertaken after each of the four main earthquake events in the 2010 – 2011 Canterbury Earthquake Sequence (CES) is discussed in van Ballegooy et al. (2014b and 2015b). In addition, an extensive geotechnical investigation programme was undertaken following the CES, including 15,000 Cone Penetration Tests (CPTs) and 3,000 boreholes (as at June 2014) to better understand the subsurface conditions beneath the residential suburbs in Christchurch. This data was collected for the purposes of assessment of the liquefaction vulnerability to as part of the recovery process. This geotechnical dataset is available from the Canterbury Geotechnical Database (CGD) which can be accessed from the following link: https://canterburygeotechnicaldatabase.projectorbit.com

Liquefaction vulnerability evaluations in the Christchurch area have made use of four CPT-based vulnerability parameters including; one-dimensional post-liquefaction reconsolidation settlement (S_{V1D}), Liquefaction Potential Index (LPI), modified Liquefaction Potential Index (LPI_{ISH}) and the Liquefaction Severity Number (LSN). These liquefaction vulnerability parameters all use a liquefaction triggering analysis as one step in their calculation. The ability of the S_{V1D}, LPI, LPI_{ISH} and LSN parameters, in combination with some common liquefaction triggering correlations, to reasonably predict the observed liquefaction-induced damage on a regional scale was evaluated by van Ballegooy et al. (2014b, 2015a and 2015b). Their conclusions pertinent to this study included: (1) the Boulanger and Idriss (2014) liquefaction triggering procedure

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produced slightly better correlations with observed liquefaction-induced damage for each liquefaction vulnerability parameter compared with the other commonly used methods, and (2) the LSN liquefaction vulnerability parameter provided a more consistent correlation with the observed liquefaction-induced land damage compared with the other liquefaction vulnerability parameters.

The liquefaction assessment triggering uncertainty, P_L, is incorporated in the Boulanger and Idriss (2014) liquefaction triggering assessment methodology. Standardised magnitude 7.5 Cyclic Resistance Ratio ($CRR_{M7.5,\ \sigma'_v = 1atm}$) equations are given for three values of P_L (i.e., 15, 50 and 85%). Figure 1 shows the band width of P_L (denoted by the transition zone). When the normalised clean sand CPT tip resistance (q_{c1NCS}) for a soil layer at a given Cyclic Stress Ratio (CSR) plots above the transition zone liquefaction triggering is likely (> 85%) and when they plot below the transition zone, liquefaction triggering is unlikely (< 15%). A comparative regional study has been carried out to evaluate the effect of P_L on liquefaction consequence assessments, using the Boulanger and Idriss (2014) liquefaction triggering method and the LSN vulnerability parameter[4].

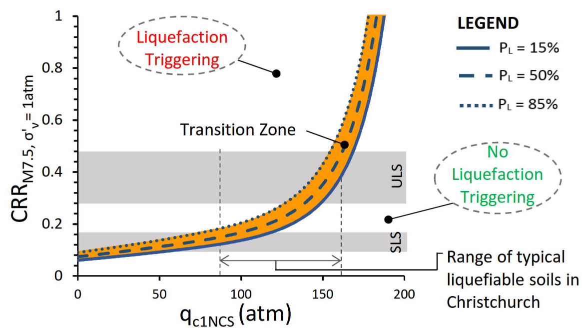


Figure 1: The Boulanger and Idriss (2014) CRR lines for P_L = 15, 50 and 85%.

The P_L = 15% line is typically used for design purposes when the other uncertainties in Peak Ground Acceleration (PGA), Magnitude (M), depth to groundwater, soil Fines Content (FC) and a soil behaviour type index (I_c) threshold (above which the soil behaviour is considered too clay rich to liquefy) are neglected. However, this will give poor correlations in an assessment context, because over a particular range of ground motions this approach is generally conservative. By definition the P_L = 50% line should give an unbiased and more realistic prediction (on average). Therefore, this paper presents a back analysis using the CES event-based peak ground acceleration models from Bradley and Hughes (2012) shown in the first row of Figure 2 and the respective event-specific depth to groundwater surfaces for the September 2010, 22 February and 13 June 2011 earthquakes and compares it to the mapped land damage (van Ballegooy et al. 2015b). Subsequently, the paper looks at the likelihood of liquefaction occurring across Christchurch for the 25, 100 and 500 year return period ground motions by comparing the deterministic triggering analyses using the median groundwater surface from van Ballegooy et al. (2014a) using P_L = 15, 50 and 85%. These three ground motions are often referred to as the

[4] Observed liquefaction related land damage during the CES generally correlate with the following LSN ranges: 0 to 16 correlates with none-to-minor liquefaction related land damage, 16 to 25 correlates with minor-to-moderate liquefaction related land damage and more than 25 correlates with moderate-to-severe liquefaction land damage.

Serviceability Limit State (SLS), Intermediate Limit State (ILS) and the Ultimate Limit State (ULS) respectively.

The ground motions for the SLS, ILS and ULS design cases are specified in the MBIE (2012 & 2014) guidelines. For the SLS and ILS cases, the design PGA values are 0.19g and 0.30g for a M6 earthquake and for ULS are 0.35g for a M7.5 earthquake. Studies undertaken by van Ballegooy et al. (2015b) show that the ULS M7.5 0.35g ground motions results in similar LSN values compared to M6 0.52g ground motions, when using the Boulanger and Idriss (2014) liquefaction triggering methodology. Therefore, for simplicity, the ULS case has been modelled using M6 0.52g motions.

The LSN parameter was computed at each CPT location using the respective groundwater surfaces based only on the top 10 m of any CPT sounding (as discussed in van Ballegooy et al., 2015b). In order to apply the various P_L cases to a regional study of 15,000 CPT (available from the CGD), assumptions have been made including; no liquefaction occurs where $I_c > 2.6$; the FC for each soil layer at each CPT location was estimated in accordance with the Boulanger and Idriss (2014) method-specific FC-I_c correlation assuming a default C_{FC} fitting parameter of zero; and that the Cyclic Resistance Ratio (CRR) of the soil estimated from the CPT traces has not been affected by the CES. The last assumption is based on studies comparing the assessed CRR of CPT undertaken prior to the CES compared with adjacent CPT undertaken following the CES. These studies did not find evidence that the soil CRR had changed as a result of the CES (Tonkin & Taylor, 2015).

Results and Analyses

Maps of liquefaction severity observations during the September 2010, February 2011 and June 2011 events are shown in the top row of Figure 2. Areas are separated into those with no visible liquefaction effects, minor-to-moderate liquefaction effects and moderate-to-severe liquefaction effects. The February 2011 event caused the greatest amount of liquefaction damage, with slightly less damage in the June 2011 events, and the least amount of damage in the September 2010 event. The spatial distribution of the calculated LSN for each event, based on a P_L = 15, 50 and 85% is shown in the maps presented in the second, third and bottom rows of Figure 2 respectively.

The maps show that the areas with high LSN values generally correlate with areas where there was moderate-to-severe liquefaction effects, whereas areas with low LSN values generally correlate with areas where there was no observed liquefaction-induced land damage. P_L = 15 and 85% generally lead to an over- and under-prediction, respectively, of liquefaction severity in comparison to observed land damage. There are areas (e.g. northwest of the Central Business District, CBD) in which the use of P_L = 15% clearly leads to an over-prediction of liquefaction severity in comparison with observations. The LSN maps indicate that the observed land damage in eastern Christchurch is more aligned with P_L = 15% LSN values whereas observed land damage in western Christchurch in more aligned with P_L = 85% LSN values. However, there are some localised exceptions, in particular south of the CBD, where the deterministic prediction based on P_L = 85% yields greater LSN values relative to the observed land damage. This is likely due to the swampy nature of the subsurface material and presence of fine grained materials meaning that in reality it is likely to be less susceptible to liquefaction. Across all three events, it is possible to conclude that areas indicating high LSN at P_L = 85% are areas where one can have high certainty for liquefaction damage to occur (except south of CBD as mentioned prior).

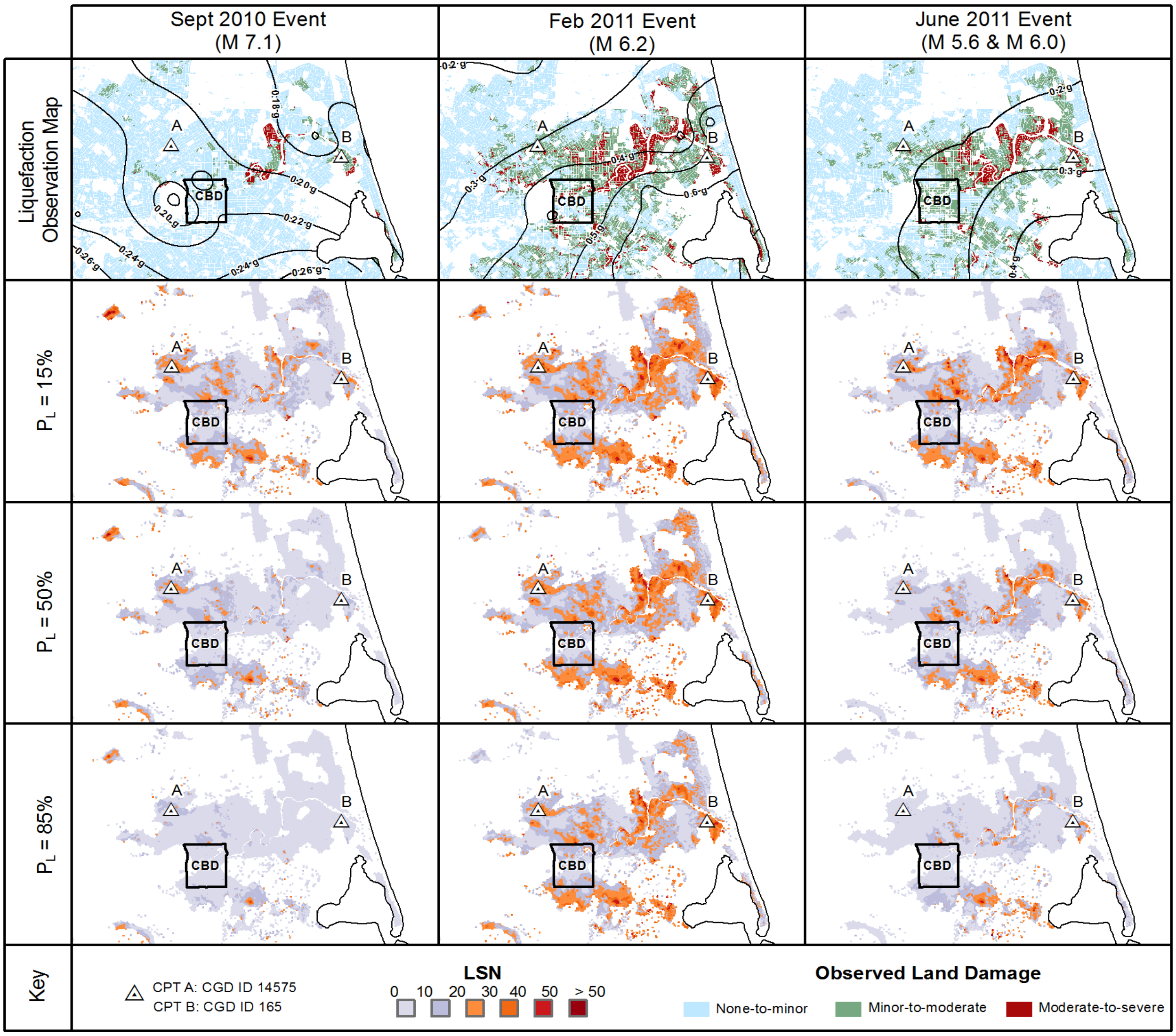
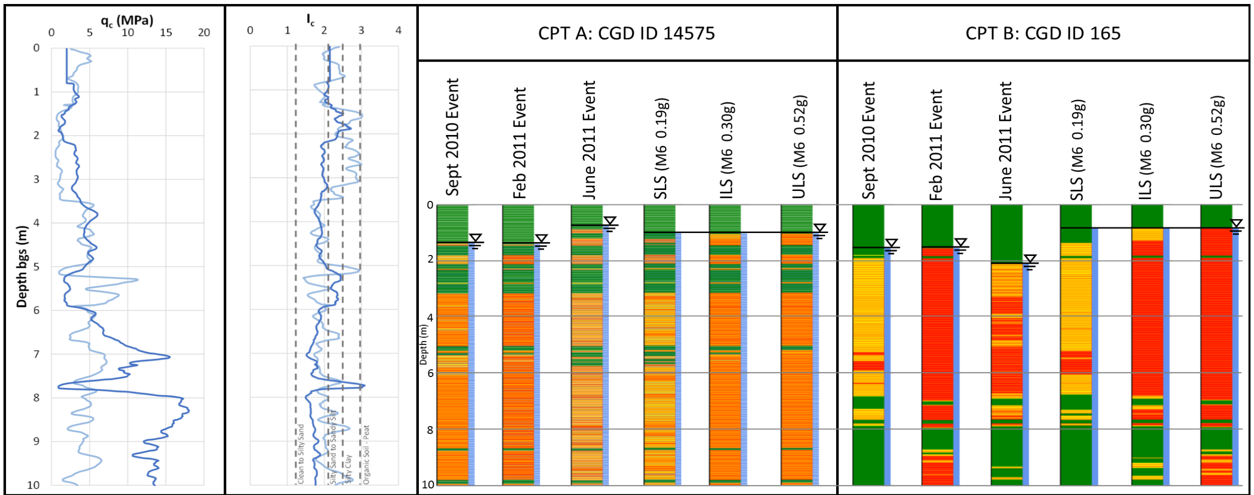


Figure 2: Maps of liquefaction severity observations across the CES overlaid with PGA contours (top row). Maps of calculated LSN for the September 2010, February 2011 and June 2011 earthquake events using P_L = 15% (2nd row), P_L = 50% (3rd row) and P_L = 85% (bottom row).

Figure 3 examines the effect of P_L on the various soil layers of two CPT (identified as CPT A and B in Figure 2) at various ground motion scenarios. The soil layers in the green colour are identified as unlikely to liquefy using P_L = 15% and the soil layers in the red colour are identified as likely to liquefy using P_L = 85%. The yellow and orange layers are in the transition zone (Figure 1).



		CPT A: Sept 2010 Event	CPT A: Feb 2011 Event	CPT A: June 2011 Event	CPT A: SLS (M6 0.19g)	CPT A: ILS (M6 0.30g)	CPT A: ULS (M6 0.52g)	CPT B: Sept 2010 Event	CPT B: Feb 2011 Event	CPT B: June 2011 Event	CPT B: SLS (M6 0.19g)	CPT B: ILS (M6 0.30g)	CPT B: ULS (M6 0.52g)
LSN	P_L = 15%	36	36	42	37	44	45	30	43	33	42	55	57
	P_L = 50%	32	36	24	26	43	45	17	43	37	25	50	57
	P_L = 85%	23	34	10	11	39	44	7	42	16	10	44	56
Observed Land Damage		None-to-Minor	Minor-to-Moderate	Minor-to-Moderate	N/A			Minor-to-Moderate	Minor-to-Moderate	Moderate-to-Severe	N/A		

Figure 3: Liquefaction triggering analysis results for two example CPTs (shown in Figure 2).

Figure 3 shows that there is a high certainty that liquefaction triggering occurred over a relatively thick layer in CPT B in the February and June 2011 events. Conversely, less certainty exists about the layer thickness of the liquefying soil at CPT A during the September 2010 event, particularly because no visual manifestation of damage was observed at the ground surface. The calculated LSN parameters at each earthquake scenario for the different P_L values are summarised in Figure 3 for the two CPT. Based on the corresponding land damage observations, P_L = 85% appears to be a better fit for CPT A and P_L = 50% appears to be a better fit for CPT B.

Following the event-specific regional liquefaction analysis, an analysis at the design earthquake motions was undertaken. Maps of calculated LSN at P_L = 15, 50 and 85% are presented in rows 1, 2 and 4 of Figure 4, respectively, for the SLS, ILS and ULS ground motions (left, center and right hand columns respectively). Difference maps of the calculated LSN for $LSN_{PL=15\%}$ - $LSN_{PL=50\%}$ and $LSN_{PL=15\%}$ - $LSN_{PL=85\%}$ are shown in rows 3 and 5 respectively. As expected, the difference maps show that the calculated LSN at P_L = 50% is smaller throughout the whole area for all three ground motions and smaller again for the P_L = 85% case when compared to the P_L = 15% case, which is typically adopted in deterministic design-based calculations. The difference is much more significant at the SLS and ILS ground motions compared to the ULS ground motions. In large parts of the city, the LSN difference for the P_L = 50% case at SLS is in the order of 10 LSN points. At ILS, the difference is in the order of 2 to 5 points. At ULS, it is between 0 to 2 points in the central and western parts of Christchurch and 2 to 5 points in the eastern Christchurch suburbs. Given that the absolute LSN values at SLS are lower than at ULS, the percentage difference in LSN between SLS and ULS is even more significant.

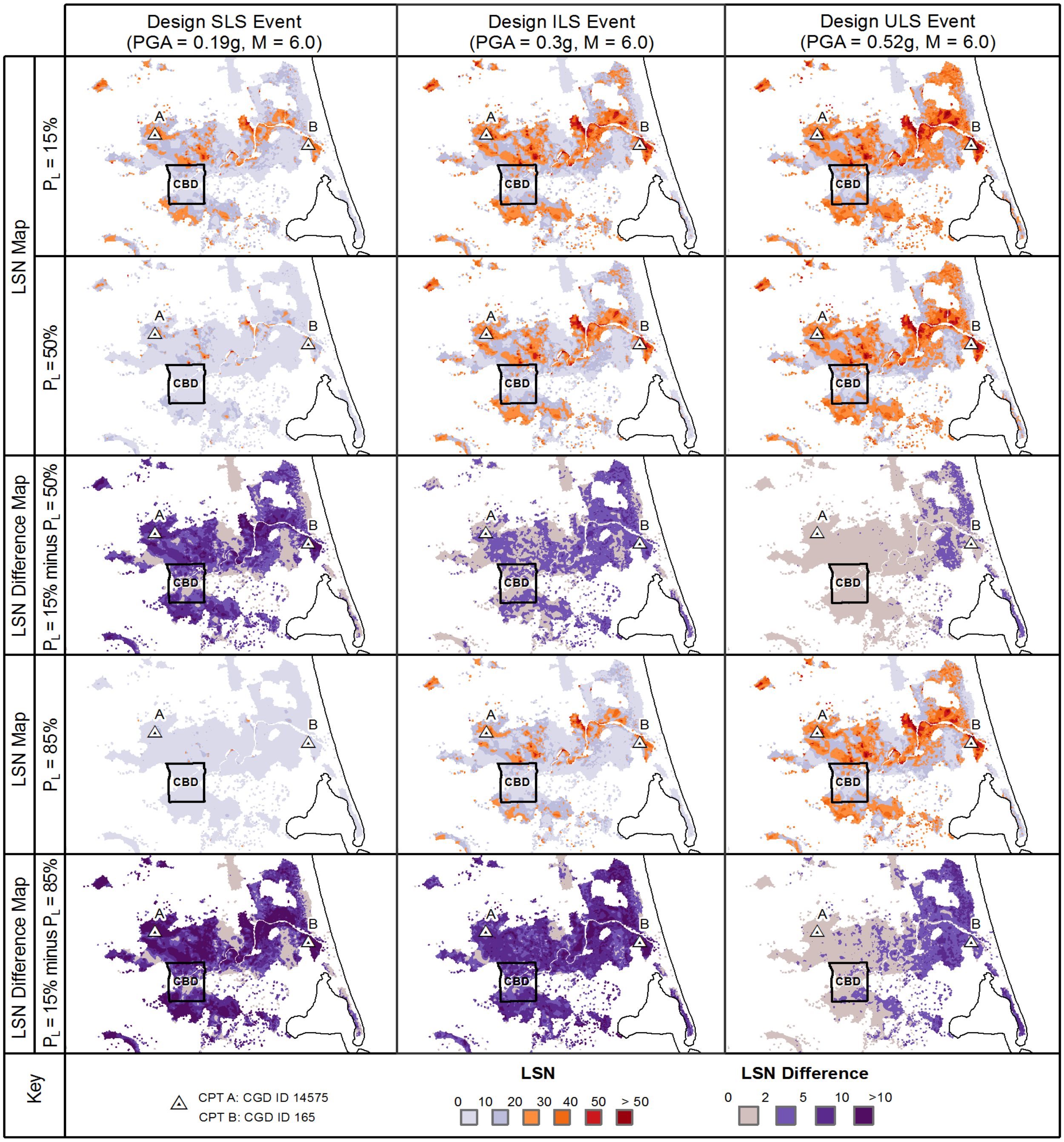


Figure 4: Calculated LSN maps at P_L = 15, 50 and 85% (rows 1, 2 and 4 respectively). Rows 3 and 5 show difference maps of the calculated LSN for $LSN_{P_L=15\%}$ - $LSN_{P_L=50\%}$ and $LSN_{P_L=15\%}$ - $LSN_{P_L=85\%}$ respectively.

The sensitivity of LSN to P_L at SLS, ILS and ULS ground motions is also demonstrated by LSN vs M6 PGA sensitivity curves shown in Figure 5 for three simplified soil profiles representing loose, medium dense and dense sand (with q_{c1NCS} values of 80, 120 and 160 atm respectively). These sensitivity curves help explain some of the observed differences in LSN for the various P_L cases presented in Figure 4. A uniform soil profile of loose sand (left-hand graph of Figure 5) has a very wide band of calculated LSN for the various P_L cases at the SLS ground motions (e.g. LSN at P_L = 15% is around 55 whereas LSN at P_L = 85% is around 20). On the other hand, there is a narrow band of calculated LSN for the various P_L cases at the ILS and, in particular, ULS ground motions. This is because at larger ground motions the CSR increases and hence the likelihood of liquefaction increases (refer to Figure 1). A soil profile of medium dense sand (middle graph of Figure 5) has a narrow band of calculated LSN for the various P_L cases at the SLS and ULS ground motions and a wider band of calculated LSN for the various P_L cases at the ILS ground motions. The characteristics of the dense sand soil profile (right-hand graph of Figure 5) are very different. LSN values are near zero at the SLS and ILS motions because at these levels of shaking, liquefaction is not triggered by the P_L = 15% case, whereas liquefaction is triggered at the ULS motions resulting in sensitivity to P_L.

The observations of sensitivity of LSN to P_L from Figure 5 translate to the LSN difference maps in Figure 2. At SLS, there are large differences in LSN values between P_L = 15, 50 and 85% across the majority of the city, because, in general, the upper soils are loose to medium dense. However, at ULS, there are insignificant differences between the three P_L cases in western and central Christchurch whereas there are subtle differences in eastern Christchurch where the underlying soils from 5 to 10 m depth are generally denser. These results show geotechnical engineers need to consider the use of P_L for liquefaction assessment. Liquefaction assessments in Christchurch at ULS ground motions are reasonably insensitive to P_L. However, at the SLS ground motions, liquefaction assessments are very sensitive to P_L, and care should be exercised relative to the soil profile to avoid over-conservative foundation design.

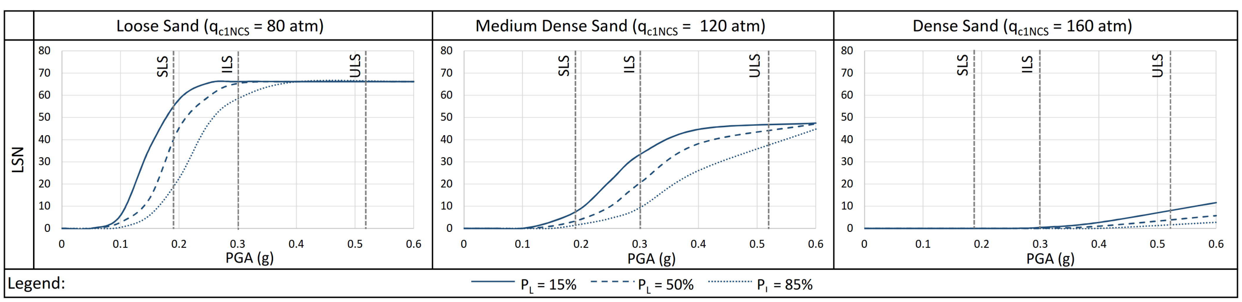


Figure 5: LSN vs M6 PGA sensitivity curves at P_L = 15, 50 and 85%.

Explicit Consideration of Liquefaction Triggering Uncertainties

The problems noted in the previous sections with respect to which value of P_L should be used in liquefaction design and assessment stem from the attempt to deterministically treat a problem which clearly contains significant uncertainties. In particular, the principal two problems with the (conventional) approach to liquefaction triggering consequences previously discussed are:

1. When computing liquefaction severity for a given level of ground shaking, a single deterministic percentile of the liquefaction triggering curve is considered, therefore ignoring the significant uncertainties in the triggering correlations.
2. Because liquefaction consequences are considered separately at discrete levels of ground shaking, and a single percentile of the triggering correlation is considered, then there is no

way in which to quantify the actual likelihood of a specific level of LSN being exceeded. For example, if a specific site has LSN = 20 based on the 100 year return period ground motion for the P_L = 50% case, then the annual likelihood that LSN = 20 is exceeded is not $1/100$.

The above two problems also exist in equivalent applications in structural earthquake engineering (Bradley, 2013). In order to overcome these problems it is necessary to explicitly consider the uncertainty in the liquefaction triggering correlations and also to make use of multiple points on the ground motions hazard curve (which defines the likelihood of certain levels of ground shaking). In this manner, the likelihood of exceeding a specific level of LSN = x can be computed from:

$$\lambda_{LSN}(x) = \sum P(LSN > x | PGA = y) |d\lambda_{PGA}(y)| \tag{1}$$

where $P(LSN > x | PGA = y)$ is the probability that $LSN > x$ given $PGA = y$, which is obtained from the soil profile and liquefaction triggering relationship (including its uncertainty); and $|d\lambda_{PGA}(y)|$ is the increment of the seismic hazard curve between the different levels of PGA that are considered in the summation. By plotting spatially distributed values of LSN corresponding to specific likelihoods of exceedance (e.g. a map of LSN for λ_{LSN}=1/100, i.e. the 100 year return period) forward-predictions of liquefaction severity manifestations can be made in a probabilistically-consistent manner by appropriately accounting for the uncertainties which have been shown to be significant in the earlier sections of this paper.

Conclusions

The Boulanger and Idriss (2014) CPT-based liquefaction triggering assessment method show that the observed extent and severity of liquefaction following for the CES events is enveloped within the range of predicted extent and severity for the P_L = 15 and 85% cases. Observed land damage in eastern Christchurch generally correlates with P_L = 15%, whereas western Christchurch correlates more closely using P_L = 85%. Geotechnical engineers use P_L = 15% when undertaking a liquefaction assessment. There are a number of areas in Christchurch this approach over predicts the liquefaction vulnerability relative to the observed CES land damage on a regional basis. The different P_L curves could help engineers understand the degree of potential over-estimation of assessed liquefaction. It is important for engineers to consider this to avoid incorporating excessive levels of conservatism into foundation design on land susceptible to liquefaction.

Acknowledgments

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