

# NONLINEAR SHEAR STRESS REDUCTION FACTOR ( $r_d$ ) FOR ASSESSMENT OF LIQUEFACTION POTENTIAL IN CHRISTCHURCH CENTRAL BUSINESS DISTRICT

James N. Dismuke<sup>1</sup>

## SUMMARY

Simplified procedures for evaluating liquefaction triggering potential use the nonlinear shear stress reduction factor,  $r_d$ , to estimate the peak earthquake-induced cyclic shear stress within the soil strata. Previous studies have derived  $r_d$  by considering the response of representative ground profiles subjected to input ground motions with a range of ground motion characteristics. In this study, site-specific  $r_d$  for serviceability limit state (SLS) and ultimate limit state (ULS) design ground motions are developed using site response models of the Christchurch Central Business District (CBD). The site response models are generated for typical geologic conditions of Christchurch CBD with shear wave velocity,  $V_s$ , profiles developed from the results of multichannel analysis of surface waves (MASW) surveys conducted across Christchurch CBD. A total of 528 simulations were conducted using 1D nonlinear time domain site response analyses using a suite of input ground motions that are representative of controlling ground motion scenarios for seismic hazard of Christchurch. The results of the ground response analyses are used to determine Christchurch CBD-specific  $r_d$  relationships for liquefaction triggering assessments. The proposed relationships provide a better estimate of the cyclic stress ratios induced below Christchurch CBD when subjected to design SLS and ULS ground motions as compared to typical practice using generic liquefaction assessment methodologies.

## INTRODUCTION

Simplified procedures for evaluating liquefaction triggering potential account for the interaction of earthquake ground motion and site response in the development of cyclic shear stress at a specific depth of interest using the shear stress reduction factor,  $r_d$ . Following the original simplified method proposed by Seed and Idriss [1],  $r_d$  relates peak ground acceleration, PGA, and maximum cyclic shear stress for a rigid soil column,  $\tau_{rigid}$ , to earthquake-induced cyclic shear stress for a deformable soil column,  $\tau_{cyc}$ , as:

$$r_d = \frac{\tau_{cyc,z}}{\tau_{rigid}} \quad (1)$$

where  $\tau_{cyc,z}$  is the computed peak cyclic shear stress at depth  $z$ ; and  $\tau_{rigid}$  is equal to  $\sigma_v \cdot \text{PGA}$ .

$r_d$  is a nonlinear function of soil stratigraphy, dynamic soil properties (eg. low strain shear modulus and soil damping), and input earthquake ground motion. Relationships for  $r_d$  have evolved since the development of the simplified method. The two current widely used relationships, Idriss [2] and Cetin and Seed [3] were developed using site response analyses to determine the earthquake-induced cyclic shear stress for a range of subsurface models and input ground motions. Although these generic  $r_d$  relationships provide acceptable accuracy for most liquefaction triggering assessments, site-specific seismic ground response analyses are a more accurate

method to assess  $r_d$  and the cyclic shear stress induced by specific ground motions. However, site-specific analyses are often saved for projects of higher importance, generally because of cost considerations.

In Christchurch, current practice for assessment of liquefaction triggering uses magnitude-weighted design PGA with earthquake magnitude of M7.5 (MBIE [4]). This practice is appropriate for ensuring that PGA and duration weighting factor are compatible with the design seismic hazard, but does not accurately represent the seismic ground response because a) the Christchurch seismic hazard is dominated by ground motion scenarios with magnitude less than M7.5, b) the ground motions used to derive generic  $r_d$  relationships do not necessarily have spectral compatibility with Christchurch design seismic hazards, and c) and the effect of local site conditions are not explicitly considered. The influence of these factors on  $\tau_{cyc}$  was explored in this study, and it was found that typical practice in Christchurch using magnitude-weighted-design PGA and M7.5 over predicts  $\tau_{cyc}$  and the cyclic stress ratio, CSR.

## $r_d$ RELATIONSHIPS USED IN PRACTICE IN 2013

The original  $r_d$  relationship proposed by Seed and Idriss [1] was developed using a suite of equivalent linear site response analyses (i.e. SHAKE, Schnabel *et al.* [5]) using simple site conditions. This relationship was the state-of-the-practice throughout the 1970s, 1980s, and 1990s, and was used to

<sup>1</sup>Senior Geotechnical Engineer, Golder Associates, Christchurch (member)

derive the legacy liquefaction triggering relationship discussed in Youd *et al.* [6]. Although it was recognized that  $r_d$  was dependent on several factors, the original relationship was defined in terms of depth only.

Idriss [2] conducted equivalent linear site response analyses on various soil profiles to expand the work of Golesorkhi [7], who investigated the effect of magnitude, site response, and input ground motions on  $r_d$ . Golesorkhi [7] evaluated  $r_d$  using equivalent linear site response analyses, as well as two different fully nonlinear site response analysis methods, and found that each of the three methods were typically in agreement. Idriss [2] extended the work of Golesorkhi [7] through further equivalent linear site response analyses using additional geologic models and input ground motions. The  $r_d$  relationship eventually presented by Idriss [2] was defined in terms of depth and magnitude, and corresponded to 67th percentile values so that  $r_d$  for M7.5 was compatible with the long-standing Seed and Idriss [1] relationship. The derivation of the Idriss [2]  $r_d$  relationship is discussed in greater detail by Idriss and Boulanger [8]. The  $r_d$  proposed by Idriss [2] were used in development of the liquefaction triggering procedure by Idriss and Boulanger [9].

Cetin and Seed [3] performed a suite of equivalent linear site response analyses using several soil profiles based on actual case histories. The profiles were subjected to input ground motions selected to cover a wide range of ground motion characteristics. This study sought to use realistic geologic profiles so that the proposed  $r_d$  would be forward compatible with  $r_d$  determined using site-specific site response analyses. Cetin and Seed [3] describe  $r_d$  using magnitude, depth, and soil stiffness via the average shear wave velocity in the upper 12.2 m,  $V_{s,12.2}$ . The resulting  $r_d$  were used in derivation of liquefaction triggering assessment methodologies of Cetin *et al.* [10], Moss *et al.* [11], and Kayen *et al.* [12].

### APPROACH

This study presents a set of  $r_d$  developed from the results of site response analyses conducted using geotechnical models and input ground motions compatible with geotechnical conditions and seismic hazards for Christchurch CBD. In this study, typical geologic profiles were developed to represent a range of geologic conditions underlying Christchurch CBD. Small-strain shear stiffness,  $G_{max}$ , for site response analyses were determined from the results of multichannel analysis of surface waves (MASW) surveys conducted across Christchurch CBD. Analyses of the MASW data was used to estimate shear wave velocity,  $V_s$ , profiles, to determine  $G_{max}$  for the alluvium underlying Christchurch CBD. A set of site response models created from the geologic and  $V_s$  profiles were then subjected to a suite of input ground motions that represent the controlling ground motion scenarios for the Christchurch seismic hazard. The site response analyses were conducted using the 1D nonlinear time domain site response analysis program, DEEPSOIL (Hashash *et al.* [13]). The results of the ground response analyses were used to determine  $r_d$  for serviceability limit state (SLS) and ultimate limit state (ULS) liquefaction triggering assessments in Christchurch CBD.

### GEOLOGIC MODELS

Christchurch CBD is underlain by 20 to 30 m of interbedded coarse- and fine-grained alluvium, which overlies a sequence of very dense gravels that are several hundreds of metres thick (Brown and Weeber [14]). The surficial geologic units

comprise deposits of the Christchurch and Springston Formations. These deposits are Holocene-aged alluvium and comprise various mixtures of sand and silt, although gravels, organic, and clayey soils are frequent. The Christchurch and Springston Formations are interbedded throughout the surface alluvium corresponding to periods of sea level rise (the Christchurch Formation was deposited in a coastal environment while the Springston Formation was deposited in a fluvial environment); however a notable transition between Christchurch and Springston Formations occurs roughly between about 5 and 10 m below Christchurch CBD. This transition is commonly marked by the presence of either dense sand and gravel or soft cohesive soil with various amounts of organics. The bottom of the Christchurch and Springston Formations comprises soft silts and clays with various amounts of organics that are up to 5 m or more in thickness (Tonkin and Taylor [15]). The soft silts and clays separate the surface alluvium from a deep sequence of dense gravels that underlies Christchurch CBD.

The general conditions described above are adopted for this study to create typical geologic profiles, denoted Profile Types C, G, S for site conditions with a 4 m thick transition between Christchurch and Springston Formations comprising clay, gravel, or sand, respectively. The transition is also modelled as occurring over 2 thinner layers in Profile Types CC, CG, CS, GC, GG, GS, SC, and SG, where the first letter designates the composition of the upper transition layer and the second letter designates the composition of the lower transition layer. The dense gravels underlying Christchurch CBD were modelled at a depth of 26 m in all profiles. Although the depth of the gravels varies roughly between about 20 and 30 m,  $r_d$  computed in this study are not sensitive to the modelled depth. Figure 1 summarizes the geologic models.

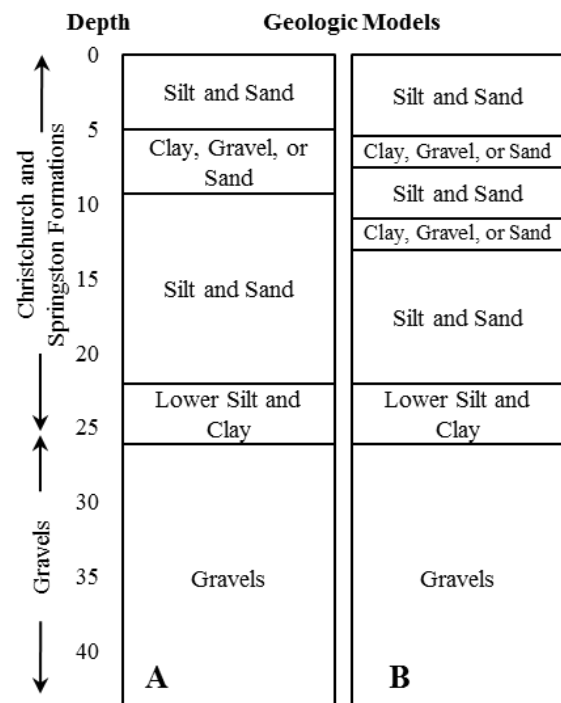


Figure 1: Geologic models used in this study (A) Profile Types C, G, and S, and (B) Profiles Types CC, CG, CS, GG, GC, GS, SC, and SG.

## SHEAR WAVE VELOCITY PROFILES

$V_s$  profiles of Christchurch CBD were developed in order to estimate  $G_{max}$  for site response analyses.  $V_s$  is a small strain property that is related to  $G_{max}$  and mass density,  $\rho$ , as:

$$V_s = \sqrt{\frac{G_{max}}{\rho}} \quad (2)$$

Development of  $V_s$  profiles used the results of MASW surveys conducted for the post-Canterbury Earthquake response, which are available from the Canterbury Geotechnical Database (CERA [16]). The extent of MASW survey is shown on Figure 2. Discretised MASW data was downloaded from the Canterbury Geotechnical Database (37,045 data in total) and grouped into 39 overlapping depth bins of varying thickness, each containing about 1,800 - 2,000 data. A lognormal distribution was fit to the  $V_s$  data in each bin. The  $V_s$  data, and computed median,  $\pm 1$  standard deviation, and  $\pm 2$  standard deviation  $V_s$  are shown at the mean depth of each bin on Figure 3.

The standard deviation of  $\ln V_s$  is greatest between depths of about 5 m to 10 m, as shown on Figure 4. This is consistent with the location of the interface between Springston Formation and Christchurch Formation where the soil conditions vary broadly from soft fine-grained soil to dense gravel. The minimum standard deviation is between 15 m and 25 m, which is a depth range where the lower silt and clay are generally consistent across Christchurch CBD. The standard deviation used to compute the smoothed profiles on Figure 3 was taken as a constant equal to  $\ln V_s$  of 0.3. A constant value was used for simplicity, while providing a reasonable fit to the data. The smoothed  $V_s$  profiles shown on Figure 3 can be programmed in a spreadsheet as:

$$\ln V_s = \max(4.97, 0.570 \cdot \ln z + 3.98) + N \cdot 0.3 \quad (3)$$

where the coefficient of 4.97 corresponds to median  $V_s$  of 144 m/s near the ground surface;

$z$  is depth below ground surface in metres; and

$N$  is the number of standard deviations.

The MASW surveys were conducted by others after the 2010 - 2011 Canterbury Earthquakes, and were done so in a manner to collect a large quantity of information rapidly. As such, the MASW data does not provide consistently accurate site-specific information. However, from the general standpoint used to develop the  $V_s$  profiles, the overall body of MASW data compares well with typical  $V_s$  profiles determined for similar subsurface conditions, as shown on Figure 5. For example, Toro [17] analysed 164 soil profiles used to develop the Geomatrix site classification scheme (summarized in Table 1), as well as 557 profiles of varying average  $V_s$  in the upper 30 m,  $V_{s,30}$ . Kottke and Rathje [18] report the median  $V_s$  and standard deviation of the models developed by Toro (1995), which compare well with the  $V_s$  profiles developed in this study:

- The proposed median -2 standard deviation profile has  $V_{s,30}$  of 138 m/s and is comparable to the Toro [17]  $V_{s,30} < 180$  m/s profile.
- The proposed median  $\pm 1$  standard deviation profiles have  $V_{s,30}$  of 185 m/s to 332 m/s and are comparable to the Geomatrix C and D and Toro [17]  $V_{s,30}$  180 m/s to 360 m/s profiles.

- The proposed median +2 standard deviation profile has  $V_{s,30}$  of 445 m/s and is comparable to the Toro [17]  $V_{s,30}$  360 m/s to 720 m/s profile.

**Table 1. Geomatrix site classification, third letter description.**

Third Letter	Site Description	Comments
A	Rock	Instrument is found on rock material ( $V_s > 600$ m/s) or less than 5 m of soil overlying rock material.
B	Shallow (stiff) soil	Instrument is founded in/on a soil profile up to 20 m thick overlying rock.
C	Deep narrow soil	Instrument is found in/on a soil profile at least 20 m thick overlying rock material in a narrow canyon or valley.
D	Deep broad soil	Instrument is found in/on a soil profile at least 20 m thick overlying rock material in a broad valley.
E	Soft deep soil	Instrument is found in/on a deep soil profile that exhibits low average shear wave velocity ( $V_s < 150$ m/s).

## SITE RESPONSE MODELS

Models for site response analyses were developed for typical geologic conditions using the  $V_s$  profiles determined in this study. The basis for assigning  $V_s$  in the site response models is highlighted on Figure 6, where the  $V_s$  profiles determined in this study are compared with  $V_s$  determined using correlation with CPT data (Robertson [19]). As expected, a majority of the correlated  $V_s$  fall within  $\pm 1$  standard deviation of the median values determined in this study. Clayey soil (assumed for CPT data with  $I_c$  greater than 2.6), is generally softer with  $V_s$  between the median value and median -2 standard deviations; and gravelly soil (assumed for data with  $I_c < 1.31$ ), is generally denser with  $V_s$  in the range of the median value to median +2 standard deviations. These observations were adopted for seismic ground response analyses as follows:

- Median values of  $V_s$  were used for the interbedded Christchurch and Springston Formations;
- +1 standard deviation  $V_s$  values were used for upper gravels;
- -1 standard deviation  $V_s$  values were used for upper and lower silt and clay; and
- Median values of  $V_s$  were used for lower gravels.

Dynamic properties for the interbedded Christchurch and Springston Formations and the upper and lower silt and clay were assigned shear modulus degradation and damping curves following Darendeli [20]. Consideration was given to the shear strength of soils at high strains according to the recommendation of Hashash *et al.* [21]. Shear modulus degradation and damping curves of Menq [22] were selected for the upper and lower gravels. Table 2 summarizes the  $V_s$  and dynamic properties used in the site response analyses.

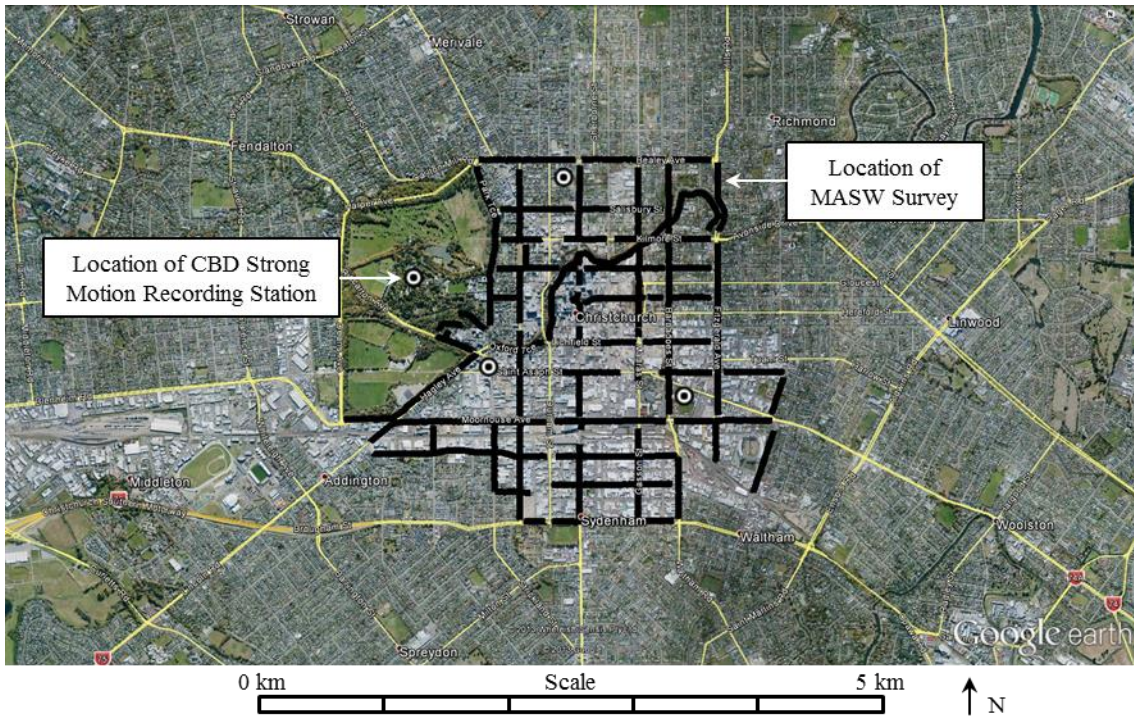


Figure 2: Location of MASW surveys and CBD strong motion recording stations (Google Earth and Canterbury Geotechnical Database).

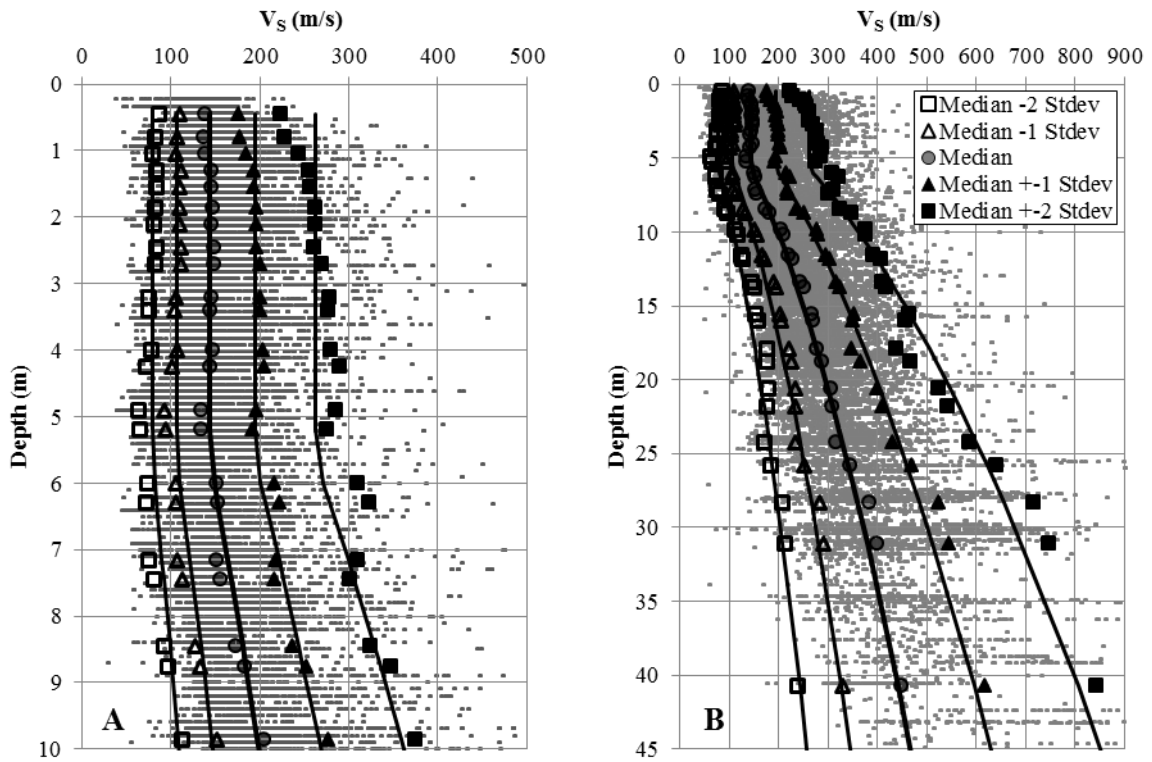


Figure 3:  $V_s$  data and smoothed  $V_s$  profiles (A) Depth 0 m to 10 m and (B) Depth 0 m to 45 m.

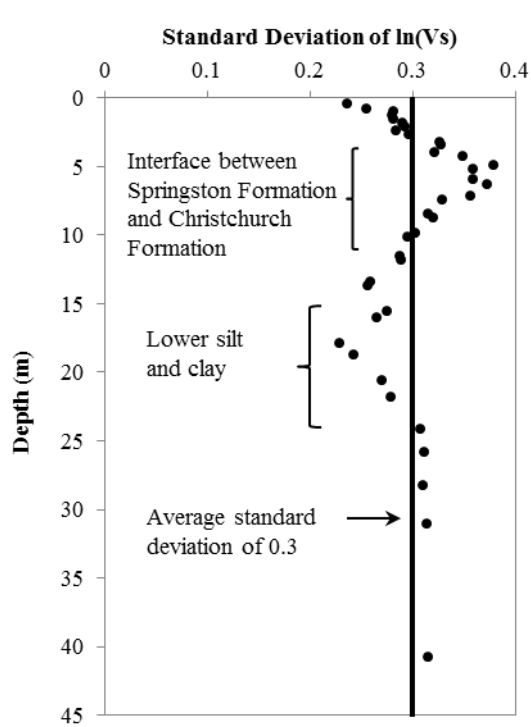


Figure 4: Standard deviation of  $\ln V_s$ .

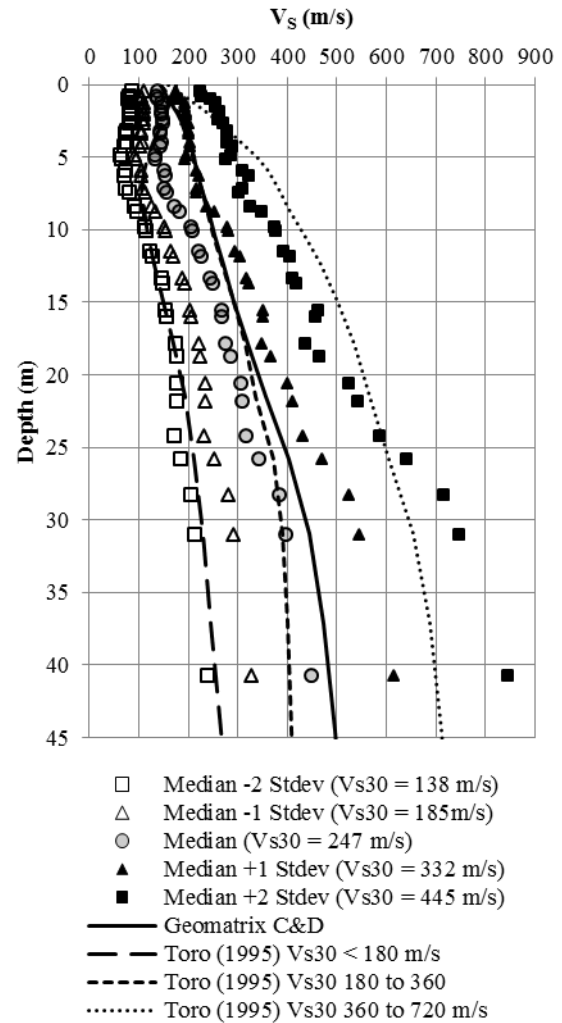


Figure 5: Comparison of  $V_s$  data with  $V_s$  profiles computed for similar geologic profiles.

Table 2. Site response model parameters.

Geologic Unit	$V_s$	Dynamic Properties	Model Parameters
Silt and Sand (interbedded alluvium)	Median values	Darendeli (2001)	Nonplastic; $K_0 = 0.5$ ; Over-consolidation ratio (OCR) = 1
Upper Gravel	+1 standard deviation values	Menq (2003)	$K_0 = 0.5$ ; Coefficient of Uniformity ( $C_u$ ) = 50
Upper Clay	-1 standard deviation values	Darendeli (2001)	Plasticity Index = 20; $K_0 = 0.5$ ; OCR = 1
Lower Silt and Clay	-1 standard deviation values	Darendeli (2001)	Plasticity Index = 30; $K_0 = 0.5$ ; OCR = 1
Lower Gravels	Median values	Menq (2003)	$K_0 = 0.5$ ; $C_u = 50$

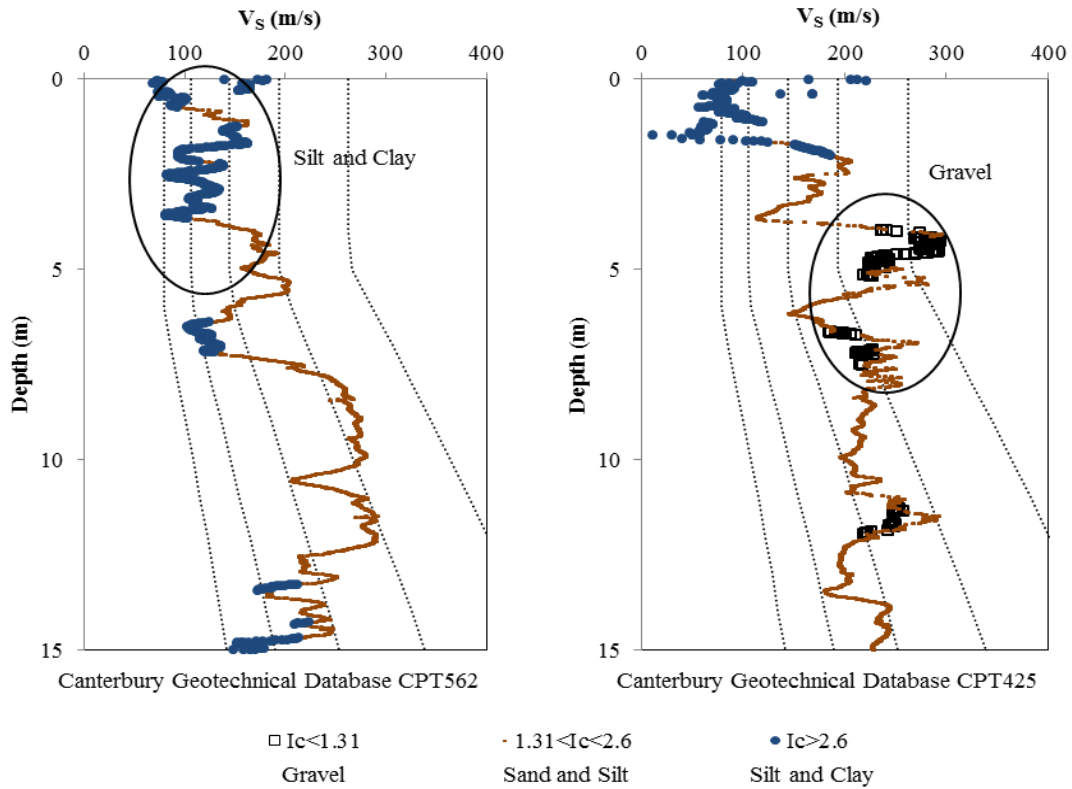


Figure 6: Comparison of proposed  $V_s$  profiles with  $V_s$  determined using CPT correlation.

**GROUND MOTIONS**

Ground motions for the site response analyses were selected to be representative of 1) local Christchurch practice, 2) the probabilistic ground motion hazard for Christchurch, and 3) New Zealand Standard NZS1170.5 [23] Typical practice in Christchurch uses the recommended ground motions presented in MBIE [4], which comprise PGA of 0.13g for the SLS (25-year return period) and 0.35g for the ULS (500-year return period). The recommended PGAs are weighted to M7.5, and were developed for NZS1170.5 Site Class D. Table 3 summarizes the NZS site classifications for reference. Although MBIE [4] is a guidance document for residential projects, these ground motions are typically adopted for non-residential projects in Christchurch.

The probabilistic ground motion hazard for Christchurch is dominated by three earthquake scenarios: the ‘Local’ Scenario is a M 5 – 5.5 earthquake at distance less than 20 km; the ‘Foothills’ Scenario is a M 6.9 – 7.4 earthquake at distance of

less than 50 km, and the ‘Alpine Fault’ Scenario is a M 8 earthquake at distance of 75 to 150 km (Stirling *et al.* [24]). Examination of the magnitude and distance de-aggregation plot for the 475-year return period PGA ( $\approx$  ULS design ground motion for Christchurch) shown on Figure 7 indicates that the Local Scenario dominates the ULS hazard, with lesser contribution from the Foothills Scenario and marginal contribution from the Alpine Fault Scenario for PGA. The de-aggregation plot for the 25-year return period ground motion ( $\approx$  SLS design ground motion for Christchurch) is not available in Stirling *et al.* [24] and [25]; however the SLS hazard is assumed to be strongly controlled by the Local Scenario given the relatively low return period. The Alpine Fault Scenario is assumed to be too rare to contribute significantly to the SLS hazard.

Table 3. Summary of NZS 1170.5 site classification.

Site Class	Site Description	Partial Description of Site Class Criteria
A	Strong rock	Average $V_{s,30}$ of 1500 m/s.
B	Rock	Average $V_{s,30}$ of 360 m/s.
C	Shallow soil	Low amplitude natural period less than or equal to 0.6 s.
D	Deep or soft soil	Low amplitude natural period greater than to 0.6 s.
E	Very soft soil	More than 10 m of soil with $V_s$ of 150 m/s or less.

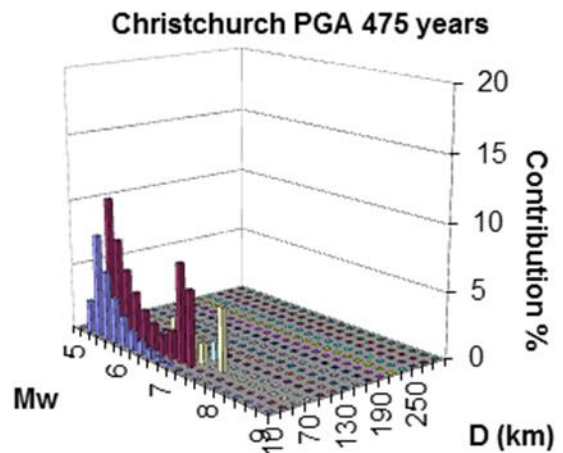


Figure 7: Christchurch 475-year return period PGA de-aggregation (Stirling *et al.*, 2012).

Input ground motions for the site response analyses were selected to represent each of the three earthquake scenarios that contribute to the Christchurch CBD seismic hazard. The PEER Ground Motion Database (PEER [26]) was queried to find input acceleration-time histories consistent with the spectral shape of NZS1170.5 Class A and B (rock) to ensure spectral compatibility over the periods of interest. Preference was given to magnitude and distance combinations matching the three earthquake scenarios discussed above. A total of 12 ground motion records were selected, corresponding to 4 records for each of the three earthquake scenarios.

The selected ground motions were linearly scaled to be compatible with the non-weighted design SLS and ULS ground motions. The scaling factors used for Alpine Scenario at ULS are greater than the value of 3 recommended in the NZS1170.5 because ground motion records matching the target response spectra over the periods of interest at the ULS hazard level for this scenario are sparse. As discussed, seismic hazard in Christchurch is codified as M7.5-weighted PGA. In order to evaluate the effect of magnitude on  $r_d$ , the magnitude weighting was removed by computing aggregate magnitude weighting factors for SLS and ULS separately using the PGAs and magnitude-weighted-PGAs presented in Stirling *et al.* [24]. The weighting factors were applied in reverse to the SLS and ULS ground motions to remove the magnitude weighting. Table 4 summarizes the selected ground motions and linear scale factors. The selected ground motions cover the NZS1170.5 Class A and B response spectra over the periods of interest, as shown on Figure 8.

#### NONLINEAR SITE RESPONSE ANALYSES

Site response analyses were performed using the 1D nonlinear time domain site response analysis program DEEPSOIL (Hashash *et al.* [13]). Both horizontal components of each of the 12 ground motion records (24 acceleration-time histories) were run through the eleven models using the SLS and ULS scale factors summarized in Table 4. Outcrop ground motions were applied directly to flexible bedrock in the models, as recommended by Stewart *et al.* [27]. Frames A and B of Figure 9 depicts the geometric mean of the 5% damped ground surface acceleration response spectra for all 528 simulations (264 simulations each for SLS and ULS). The ground surface spectral acceleration computed for SLS input ground motions is amplified from the input ground motion between periods of about 0.2 and 2 seconds, whereas for the larger amplitude, ULS input ground motions, the ground surface response is attenuated at short periods and amplified between periods of about 0.5 and 2 seconds.

The computed ground response is consistent with observed ground surface response from the 2010 – 2011 Canterbury Earthquake Sequence. Figure 10 depicts 5% damped ground surface acceleration response spectra normalized by the PGA of the input ground motion for the four strong motion stations located within the CBD and for several recent earthquakes: the M5.3 13 June 2011 Foreshock is representative of SLS design level (Frame A), and the M7.1 04 September 2010, M6.2 22 February 2011 Christchurch Earthquake, M6.0 13 June 2011 Main shock, and M5.9 23 December 2011 earthquakes are representative of ULS design level (Frame B). PGA for both computed and observed response is amplified at SLS, but is attenuated at ULS. In either case, the spectral shape of the computed spectra closely resembles the spectral shape of the observed spectra.

The “input” motion used to normalize the observed ground surface response in Christchurch CBD was recorded at the strong ground motion recording station at Lyttelton Port Company, station designation LPCC (Geonet [28]). LPCC is located about 11 kilometres southeast of the CBD and is

situated on NZS1170.5 Class B (Rock) conditions. The PGA used to normalize the response spectra was modified from the observed PGA at LPCC to account for the difference in source-to-site distance. The residuals for the observed PGAs at LPCC were back-calculated using Bradley [29] for the five earthquakes considered, and then applied in forward-prediction of peak bedrock acceleration below the CBD. LPCC is situated on the hanging wall of the Port Hills fault (the source of the 22 February 2011 Christchurch Earthquake), while Christchurch CBD is on the footwall. However, LPCC is the only strong motion station near Christchurch CBD situated on engineering bedrock, and provides a means to compare the nonlinear ground response experienced in Christchurch.

#### EQUIVALENT LINEAR SITE RESPONSE ANALYSES

Sensitivity studies using equivalent linear site response analyses were conducted to explore the assumptions made to create the simplified geologic profiles adopted in the nonlinear analyses. The equivalent linear site response analyses were conducted using the computer program STRATA (Rathje and Kottke [30]), which allows stochastic variation of the input parameters in the analysis through Monte Carlo simulation. STRATA constructed random site conditions from the lognormal distribution of  $V_s$  described above in order to model variability in stiffness of the interbedded Christchurch and Springston Formations. STRATA constructed 100 site realizations for each of the 11 geologic models, which were analysed for all 24 input ground motions at SLS (26,400 simulations). A unique set of 100 site realizations were also created and run at ULS (26,400 simulations).

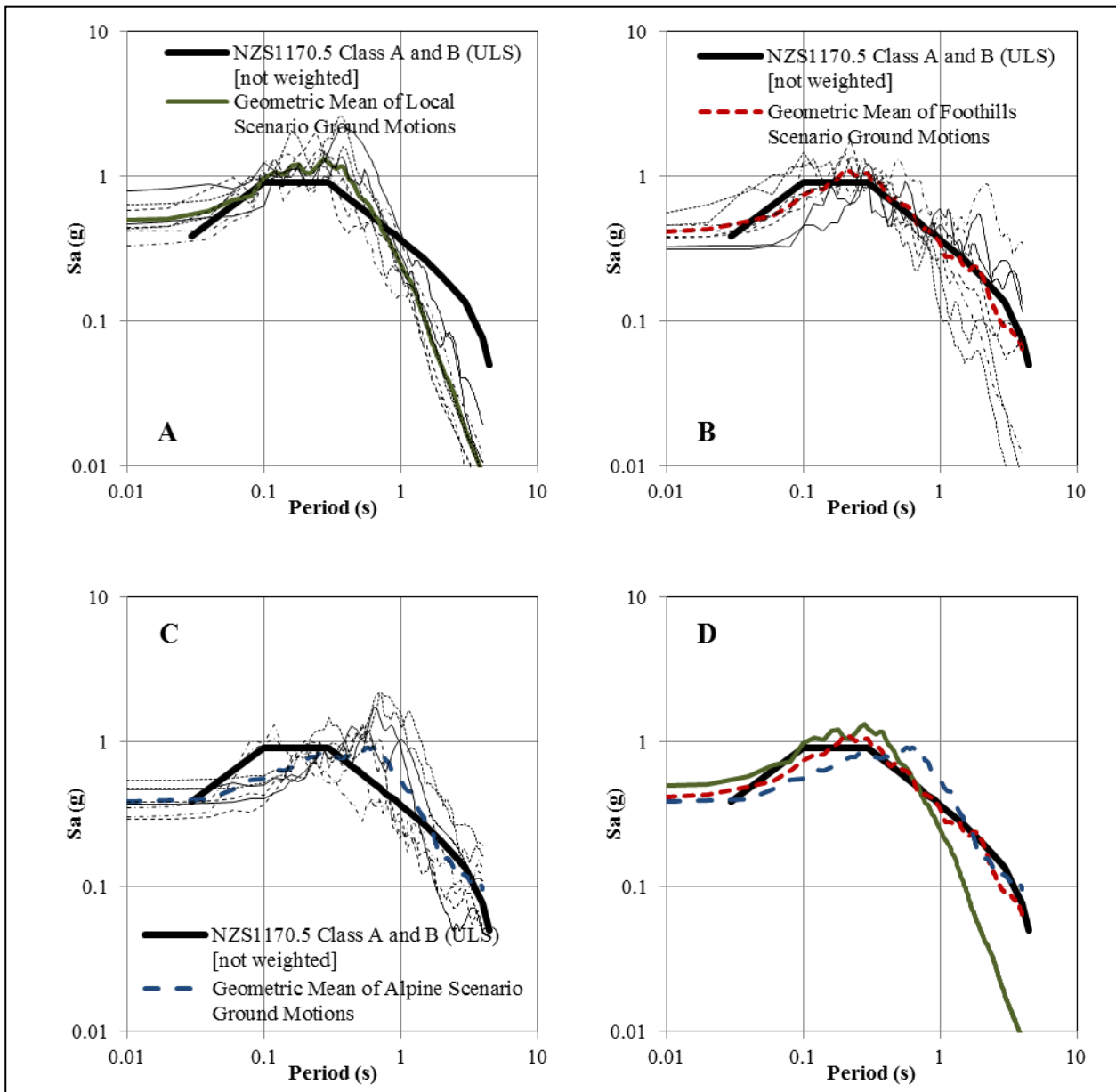
The predicted ground surface response shown on Figure 9 for SLS (Frame A) is reasonably consistent between equivalent linear (STRATA) and nonlinear (DEEPSOIL) analyses; however, the results of ULS analyses (Frame B) are not consistent. Equivalent linear analyses for ULS input motions indicated greater amplification at periods less than about 2 seconds compared to the nonlinear analyses. The analysis results are also compared to observed ground motions from multiple events within the 2010 – 2011 Earthquake Sequence on Figure 10. The response spectra shown on Figure 10 were normalized by the input PGA, as discussed above. Both analysis methods predict amplification of PGA at SLS comparable to the ground motion observed during the 13 June 2011 Foreshock. In contrast, nonlinearity in the ground response observed during the 04 September 2010, 22 February 2011, 13 June 2011 Main shock, and 23 December 2011 earthquakes resulted in attenuated PGA. This behaviour was captured well by nonlinear analyses, but was not evident in equivalent linear analyses. However, if a true back calculation of ground response were the focus of a site-specific study it is reasonable to expect that an equivalent linear model could be calibrated to better approximate the observed ground response than what is predicted by the sensitivity analyses presented in this paper. Although equivalent linear and nonlinear analysis methods both provide reasonable estimates of ground response of Christchurch CBD, the results of the nonlinear analyses conducted for this study are better matched, on average, to observed performance than the results of equivalent linear analyses.

#### PROPOSED CHRISTCHURCH-CBD-SPECIFIC $r_d$

Christchurch-CBD-specific  $r_d$  were developed using the results of the nonlinear site response analyses.  $r_d$  was determined according to Eq. 1, and the computed  $r_d$  are shown on Figure 11.

**Table 4. Input ground motions.**

Scenario	Earthquake	M	Distance (km)	$V_{s,30}$ (m/s)	SLS Scale Factor	ULS Scale Factor
Local	Coyote Lake (NGA 145)	5.74	6.1	597	1	2.85
	Coalinga-04 (NGA 404)	5.18	<10	376	1	2.85
	Whittier Narrows-02 (NGA 707)	5.27	<15	550	1	2.85
	Sierra Madre (NGA 1647)	5.61	18.7	379	1	2.85
Foothills	Loma Prieta (NGA 736)	6.93	41.0	450	1	2.85
	Northridge-01 (NGA 1005)	6.69	31.5	376	1	2.85
	Kocaeli, Turkey (NGA 1162)	7.51	32.4	425	1	2.85
	Chi-Chi, Taiwan (NGA 1478)	7.62	42.7	423	1	2.85
Alpine	Kocaeli, Turkey (NGA 119)	7.51	142.3	660	1	4.28
	Chi-Chi, Taiwan (NGA 1425)	7.62	94.8	474	1	4.28
	Denali, Alaska (NGA 2110)	7.90	141.3	425	1	4.28
	Denali, Alaska (NGA 2115)	7.90	127.0	376	1	4.28



**Figure 8:** Summary of 5% damped horizontal acceleration response spectra for input ground motions for (A) Local Scenario, (B) Foothills Scenario, (C) Alpine Fault Scenario, and (D) comparison of all scenarios.

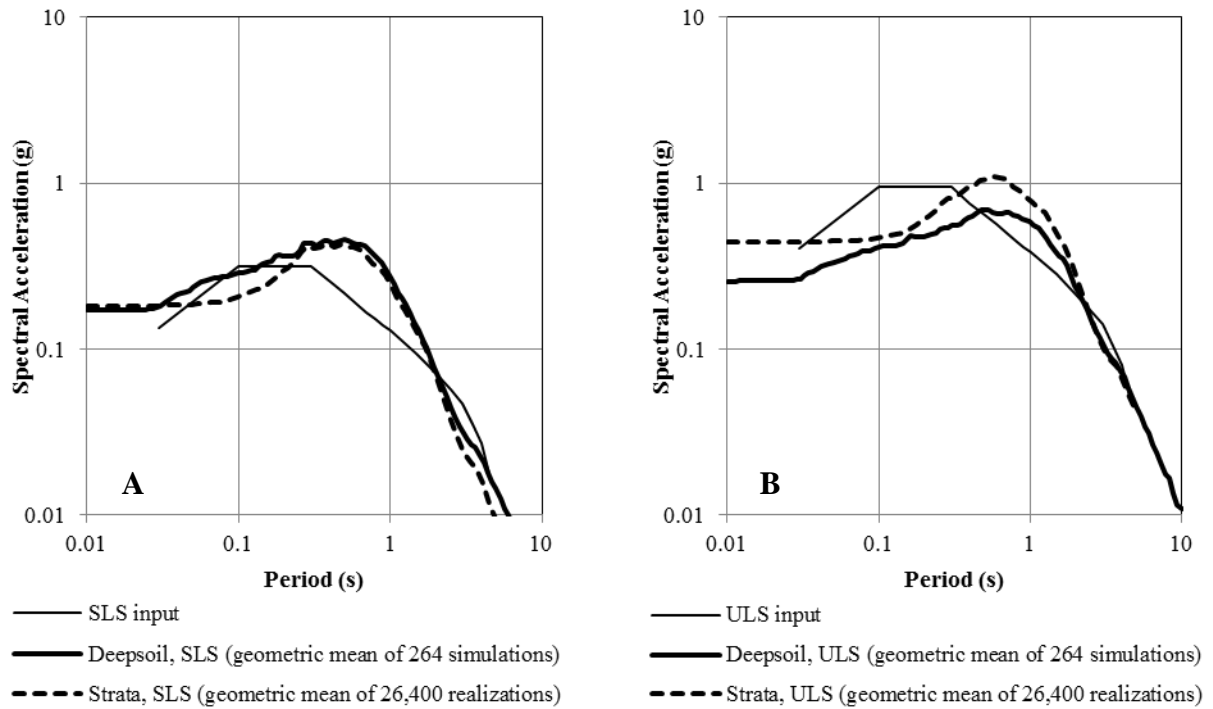


Figure 9: Comparison of computed ground surface response spectra (A) SLS and (B) ULS.

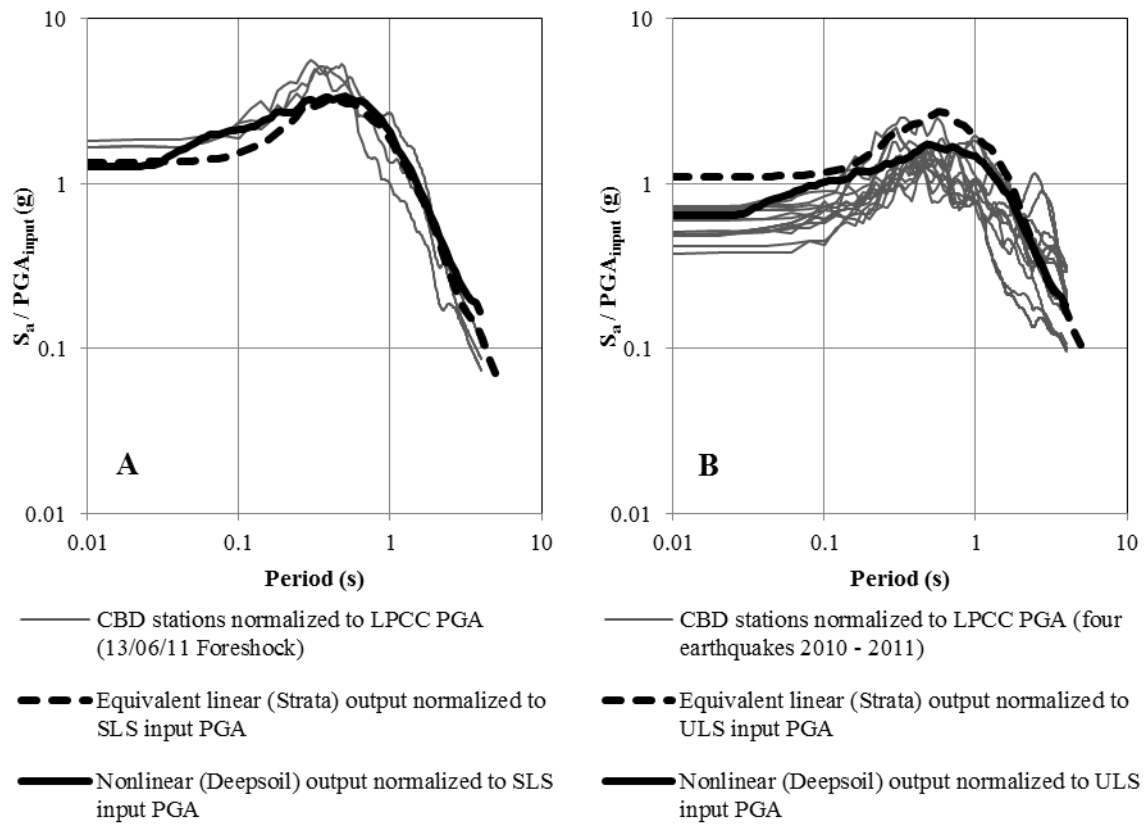
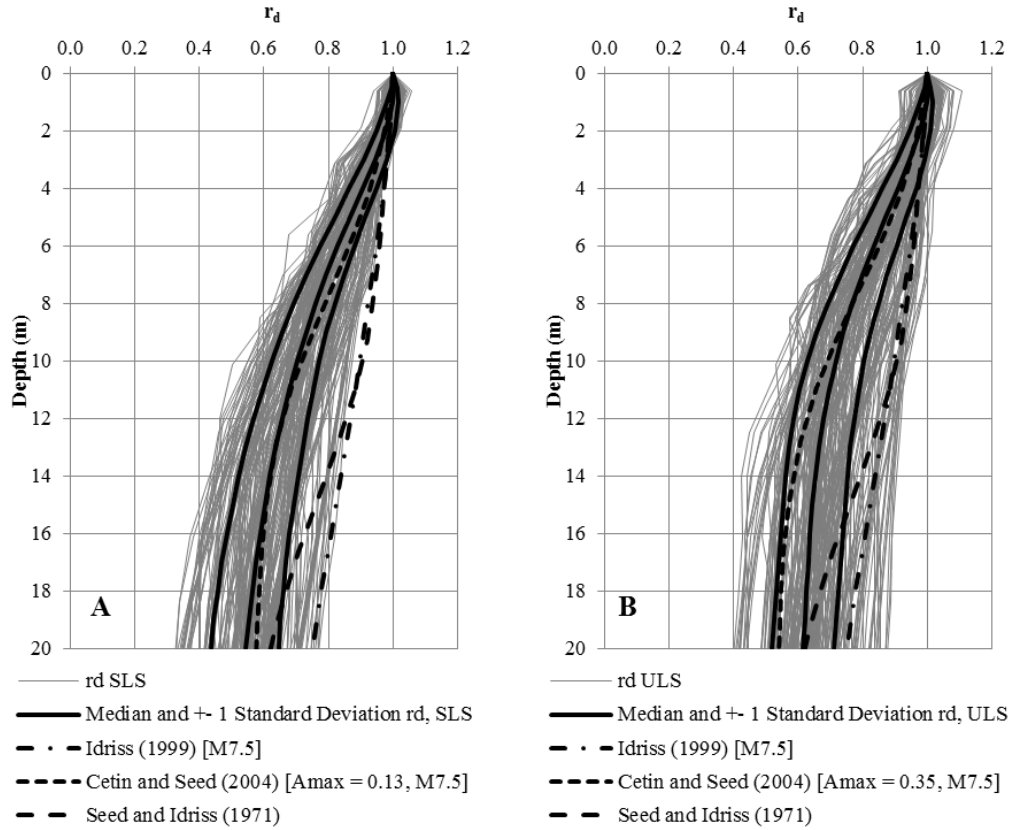


Figure 10: Comparison of ground surface response spectra normalized by input PGA (A) computed SLS with observed response spectra from 13 June 2011 Foreshock and (B) computed ULS with observed response spectra from 4 September 2010, 22 February 2011, 13 June 2011, and 23 December 2011 Earthquakes.



**Figure 11:**  $r_d$  computed for Christchurch CBD overlain with median and  $\pm 1$  standard deviation  $r_d$  for (A) SLS input ground motions and (B) ULS input ground motions.

Results from Local and Foothills Scenarios were used in determination of  $r_d$  for SLS,  $r_{d,SLS}$ . The Alpine fault scenario is assumed to be too rare to contribute significantly to the SLS hazard, and was not used in determination of  $r_{d,SLS}$ .

In contrast, results from all scenarios run at ULS were used in determination of  $r_d$  for ULS,  $r_{d,ULS}$ .  $r_{d,SLS}$  and  $r_{d,ULS}$  correspond to the median value and have  $R^2$  of 0.82 and 0.78 for SLS and ULS, respectively.  $r_{d,SLS}$  and  $r_{d,ULS}$  may be used directly in computation of CSR with magnitude-weighted PGA and can be programmed into a spreadsheet as:

$$r_{d,SLS} = -8.017E-8z^6 + 5.729E-6z^5 - 1.634E-4z^4 + 2.345E-3z^3 - 1.637E-2z^2 + 1.375E-2z + 1 \quad (4)$$

$$r_{d,ULS} = -4.475E-08z^6 + 3.420E-06z^5 - 1.082E-04z^4 + 1.755E-03z^3 - 1.344E-02z^2 + 9.049E-03z + 1 \quad (5)$$

where  $z$  is depth below ground surface in metres.

The  $r_d$  of Idriss [2] and Cetin and Seed [3] are shown on Figure 11 in comparison with the proposed Christchurch-CBD-specific  $r_d$ . Using M7.5 to determine  $r_d$  with Idriss [2] overpredicts  $r_d$  in Christchurch on average by about 15 percent compared to  $r_{d,SLS}$  and  $r_{d,ULS}$ . Using M7.5 and the magnitude-weighted SLS PGA (0.13g) with  $r_d$  of Cetin and Seed [3] is reasonable compared to  $r_{d,SLS}$ . The use of M7.5 and magnitude-weighted ULS PGA (0.35g) with  $r_d$  of Cetin and Seed [3] is reasonable in the upper 10 m, but underpredicts  $r_d$  up to about 10 percent for depths greater than 10 m compared to  $r_{d,ULS}$ .

The computed  $r_d$  for Local, Foothills, and Alpine Fault scenario input ground motions are shown on Figures 12, 13 and 14, respectively.  $r_{d,SLS}$  and  $r_{d,ULS}$  are superimposed on Figures 12, 13 and 14 for reference. Examination of Figures 12, 13 and 14 indicates that Local Scenario input ground motions yield  $r_d$  slightly lower than  $r_{d,SLS}$  and  $r_{d,ULS}$ , but

Foothills and Alpine Fault Scenario ground motions yield  $r_d$  slightly greater than  $r_{d,SLS}$  and  $r_{d,ULS}$ .

## CONCLUSIONS

The effect of seismic ground response in determination of the CSR for simplified liquefaction triggering calculations can be approximated using the nonlinear shear stress reduction factor,  $r_d$ . Available generic relationships parameterize  $r_d$  using earthquake magnitude, amplitude of ground shaking, and site conditions. Current practice in Christchurch relies on magnitude-weighted-design PGA and employs M7.5 for liquefaction triggering calculations. This practice is appropriate for ensuring that PGA and duration weighting factor are compatible with the design seismic hazard, but does not accurately represent the seismic ground response because a) the Christchurch seismic hazard is dominated by ground motion scenarios with magnitude less than M7.5, b) the ground motions used to derive generic  $r_d$  do not necessarily have spectral compatibility with Christchurch design seismic hazards, and c) the effect of local site conditions are not explicitly considered. The influence of these factors on  $\tau_{cyc}$  and  $r_d$  was explored in this study, and it was found that typical practice in Christchurch using magnitude-weighted-design PGA and M7.5 over predicts CSR. New  $r_d$  relationships for Christchurch CBD are proposed; the differences between

proposed  $r_d$  with the relationships of Idriss [2] and Cetin and Seed [3] are due to two key factors:

1. Nonlinear seismic ground response analyses were used in development of the proposed Christchurch-CBD-specific  $r_d$ , but previous relationships were developed using equivalent linear site response analyses. Equivalent linear analysis methods are suitable for lower amplitude ground motions, such as the SLS hazard level, but nonlinear analyses can provide a better estimate of the nonlinear response of the soils at larger amplitude ULS ground motions. This is demonstrated by comparison of the ground surface spectral response determined by nonlinear and equivalent analyses with observed response recorded in Christchurch during the 2010 – 2011 Canterbury Earthquake Sequence.
2. A narrow range of subsurface conditions and input ground motions were considered in estimation of Christchurch-CBD-specific  $r_d$ , whereas a broad range of subsurface conditions and input ground motions were used by Idriss [2] and Cetin and Seed [3]. The resulting range of uncertainty in proposed Christchurch-CBD-specific  $r_d$  is smaller on average than what is obtained using generic  $r_d$  relationships, largely because of the focused set of geologic and geotechnical models and ground motion inputs used in the current study.

Simplifying assumptions regarding the geologic models, shear wave velocity profiles, and dynamic soil properties were made in order to maintain the Christchurch-CBD-wide scope of the study. Although there is uncertainty in  $r_d$  due to the geologic and geotechnical interpretation, variation of these parameters has small effect on the computed  $r_d$ . A site-specific analysis, as opposed to the regional-specific approach used in this study, can reduce this uncertainty to provide a more accurate estimate of  $r_d$ . Similarly, 2D and 3D effects, such as directivity

and wave-guide effects, were observed in ground motions recorded in Christchurch (Bradley and Cubrinovski [31]), but these effects are not modelled. Consideration of these phenomena could result in a model that predicts ground response closer to the observed response than what is predicted in this study.

In summary, the proposed Christchurch-CBD-specific  $r_d$  provide an improved estimate of the nonlinear seismic ground response for evaluating CSR for liquefaction triggering analyses in Christchurch CBD. The proposed  $r_d$  can be used directly in liquefaction triggering assessments using magnitude-weighted-design PGA in lieu of  $r_d$  computed for M7.5.

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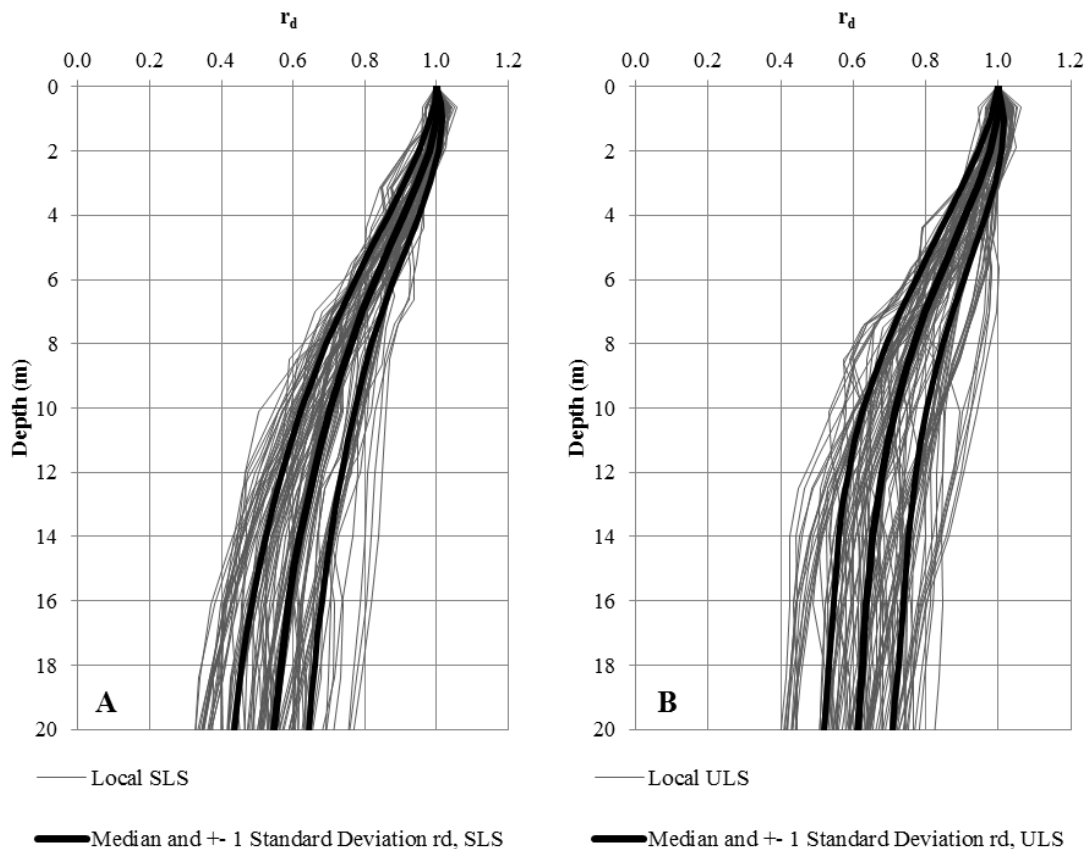


Figure 12:  $r_d$  computed for Christchurch CBD Local Scenario for (A) SLS input ground motions and (B) ULS input ground motions.

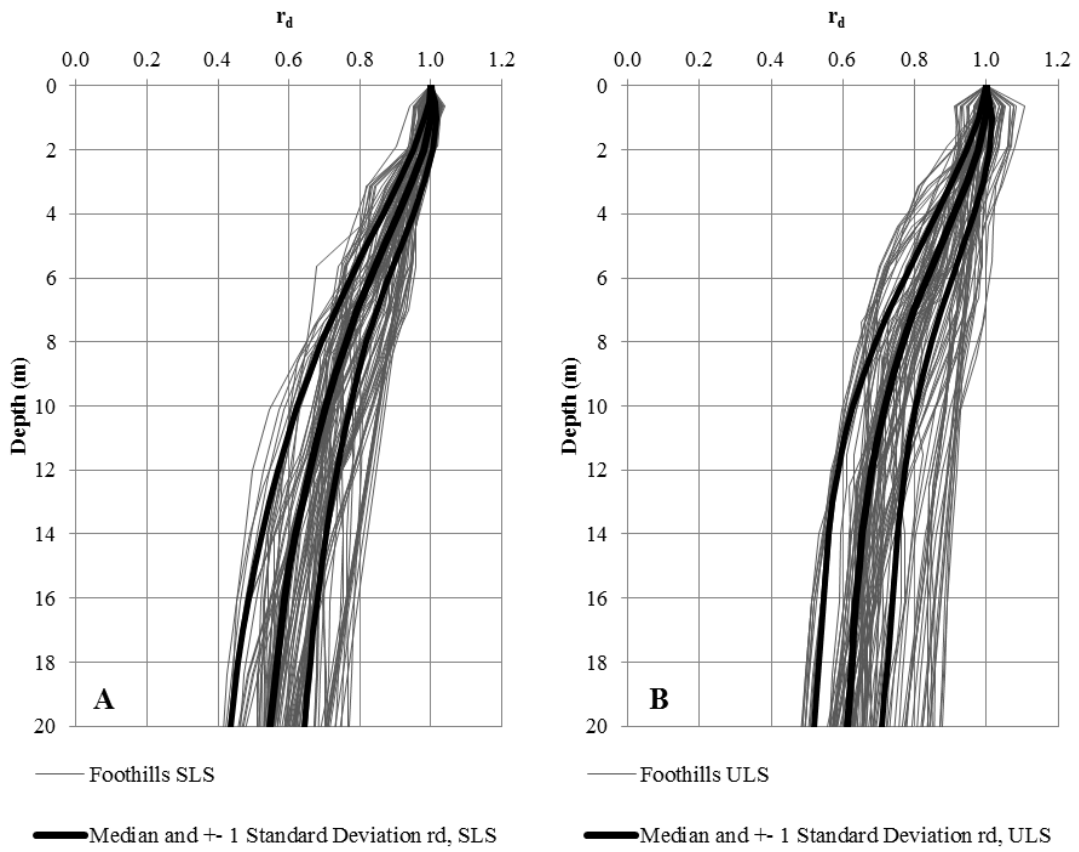


Figure 13:  $r_d$  computed for Christchurch CBD Foothills Scenario for (A) SLS input ground motions and (B) ULS input ground motions.

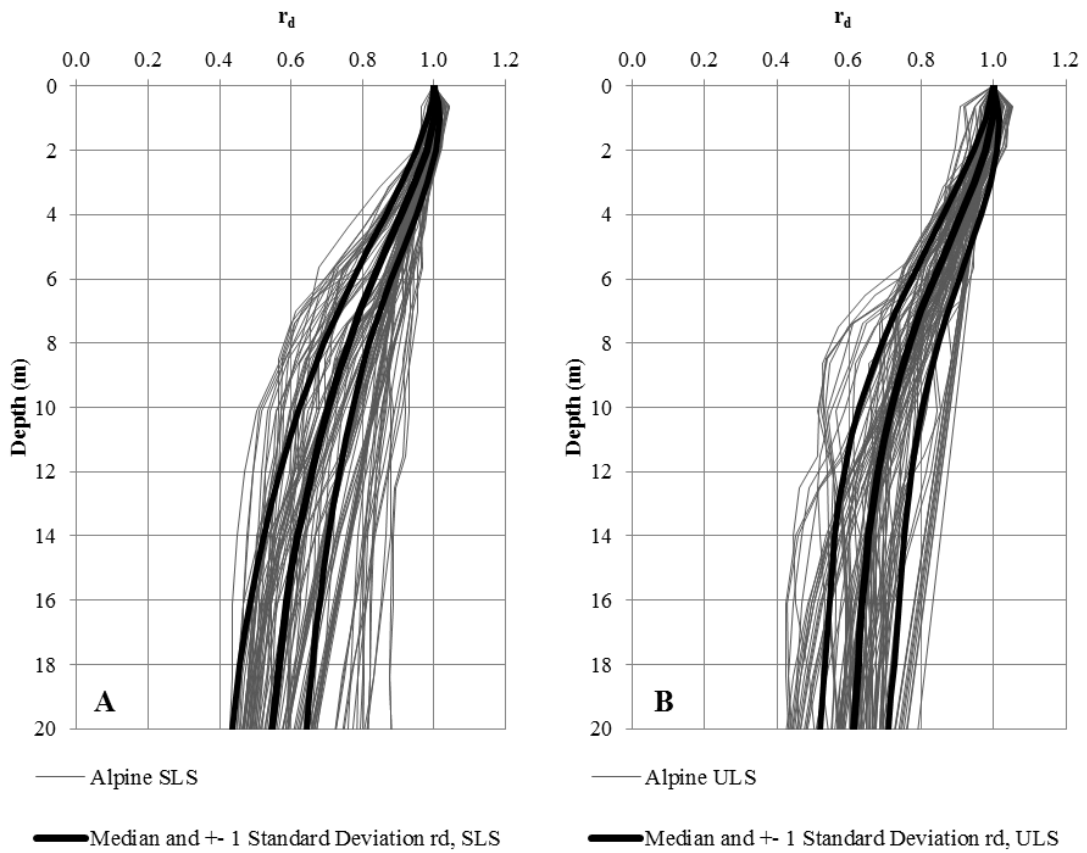


Figure 14:  $r_d$  computed for Christchurch CBD Alpine Fault Scenario for (A) SLS input ground motions and (B) ULS input ground motion.

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