

# Seismic Ground Response at a Deep Soft Soil Site: Comparing 1D Numerical Solutions with Observed Data from the Christchurch Earthquakes

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## ABSTRACT

The earthquake hazard for an engineered structure is influenced by the dynamic response of the soils that underlie the site. During seismic shaking, bedrock motions are altered when travelling through surficial soils. Analytical methods to model these effects have developed over the last few decades. This paper compares the observed ground response at a deep soil site at Lyttelton Harbour, Christchurch with the calculated response using both equivalent-linear and non-linear 1D analytical methods. Ground motion records from two seismographs were utilised; one located on a nearby rock outcrop and the other at the surface of a 60m deep soft soil profile. The observed ground response indicates the presence of non-linear, hysteretic and 3-dimensional effects at the site. Comparisons between the observed and calculated acceleration response spectra, peak ground accelerations and site periods are made. The non-linear analysis demonstrates an overall good fit to the observed data, and is shown to outperform the NZS1170.5 subsoil class method for most periods, emphasising the importance of carrying out a ground response analysis for important structures.

*Keywords:* site response, ground response, deep soils, soft soils, seismic hazard, local site effects

## 1 INTRODUCTION

When a rupture occurs along a fault line, energy waves are released that travel in all directions through the bedrock. When these energy waves reach the softer surficial soils they begin to slow and travel towards the surface. Depending on the ground profile, the physical characteristics of each soil layer and the strength and direction of shaking, the resulting ground motions and forces on a structure at the surface can be very different to a rock outcrop site or even another nearby soil site.

The 1985 Mexico City earthquake is an example of when ground response played a significant part in the destruction of a populated area. The unconsolidated deposits within the City Lake Zone had a natural site period of about 2 seconds, resulting in amplification of earthquake waves at this period. The 6- to 15-storey structures that had been built upon the lakebed also had a fundamental period near 2 seconds. This aligning of the structural and site periods created the 'double-resonance effect' and the large seismic forces that resulted in the collapse or irreparable damage to many of the buildings in the area, and the deaths of thousands (Seed 1989).

Over the last few decades, analytical solutions to estimate site specific ground response have evolved and many numerical codes are available to the engineering industry. Although these codes are readily available, they are not commonly used within New Zealand due to the requirement of additional ground information that is not routinely investigated and the provisions of the New Zealand loadings standard NZS1170.5 (2004) which provides a simplified method for estimating design spectra based on discrete "subsoil classes".

The existing literature has reported on case studies comparing 1D ground response analyses with observed motions (e.g. Rollins, et al. 1994; Baturay and Stewart 2003). Varying success of the analyses have been reported often depending on the presence of 2D/3D physical effects that cannot be captured in the assumptions of the analytical codes.

This paper uses strong ground motion records from Lyttelton Port, Christchurch collected from the Canterbury Earthquake Sequence of 2010-2011 to compare the ground response measured at the port with the predicted ground response using common 1D equivalent-linear and non-linear codes.

## 2 BACKGROUND

### 2.1 Lyttelton Port

Lyttelton has long been the main port of the Christchurch area and is located on the southern slopes of the Port Hills, which effectively separate Lyttelton Harbour from the city of Christchurch. The port overlies deep marine sediments that in turn overlie the volcanic basalt and andesite bedrock that form the Port Hills to the north. Much of the port development has also been constructed on reclaimed land.

### 2.2 NZS1170.5 site subsoil classification

The New Zealand Standard NZS1170.5 incorporates ground response into design spectra by means of Spectral Shape Factors,  $C_h(T)$  for different subsoil classes. These subsoil classes range from Class A/B (rock) to Class C (shallow soils), Class D (deep/soft soils) and Class E (very soft soils). Sites are classified based on either the shear wave travel time to bedrock, the undrained shear strength or SPT-N values of the soils, or simply the depth to rock. Based on low SPT-N values for the marine sediments, the subject site is Class E (very soft soils). These shape factors consider the ground response for a large number of sites that fit within each classification 'bucket' and therefore provide an average response that may not adequately represent the site specific response.

Additionally, unlike other international standards (e.g. IBC 2009; NBC 2015) NZS1170.5 keeps a constant shape factor regardless of the level of shaking. As a result, the shape factors are required to envelope the non-linear response of both low and high levels of shaking, often leading to significant over-prediction of large amplitude motions for soft soil sites.

### 2.3 Common ground response analytical methods

The most common methods of ground response analysis are based on the assumption of one-dimensional propagation of shear waves. For sites where 2D/3D effects are considered not to be significant, this assumption is broadly valid due to the refraction of shear waves towards the ground surface as they travel through increasingly softer surficial soils. The two most common methods are summarised below and in Figure 1.

**1D equivalent-linear (EL) analyses** are the easiest to implement and work by analysing ground motion records in the frequency domain. Irregular input bedrock motions are deconvoluted (by way of Fourier transform) into a series of harmonic sinusoidal waves. A closed-form solution and linear-elastic model (a constant value of soil stiffness and damping is assumed for each layer) is then used to calculate the amplification/damping for each harmonic load. The calculated responses are combined to obtain the irregular response at the ground surface. The assumed constant stiffness and damping values are then updated based on the calculated soil shear-strain history and the process is repeated until strain compatible stiffness and damping constants are obtained.

**1D non-linear (NL) analyses** require additional input parameters and work by solving the dynamic equation of motion at each time step to model the propagation of shear waves through the soil profile (i.e. in the time domain). The soil column is discretised into individual layers using a multi-degree-of-freedom lumped parameter model. A corresponding mass and non-linear hysteresis spring represent each layer. The instantaneous spring stiffnesses are updated at each time increment to incorporate soil non-linearity and damping.

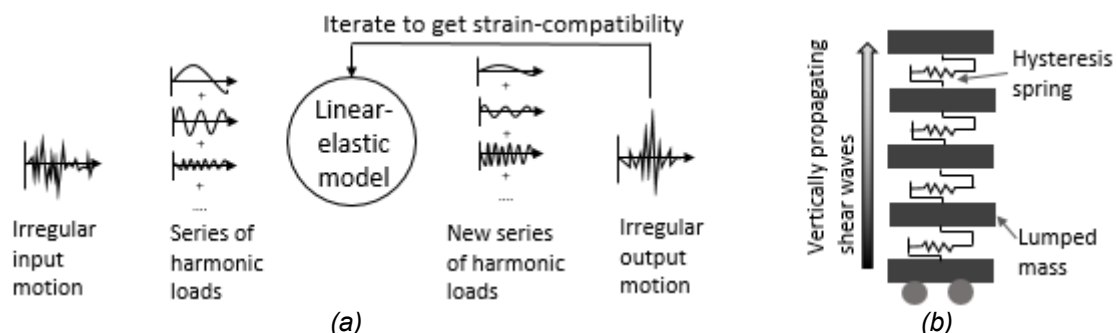


Figure 1. Diagram of (a) equivalent-linear (EL); and (b) non-linear (NL) ground response analysis methods

### 3 GROUND MOTION RECORDS

Ground motion records from three earthquakes were sourced from the GeoNet (2016) database and are summarised in Table 1. Each earthquake record was analysed separately for its orthogonal horizontal directions. i.e. a total of six outcrop motions and six deep soil profile motions from the three earthquakes were used.

Rock outcrop ground motions were obtained from the “LPCC” record station located east of the port at the base of the Port Hills. The LPCC record station is located on a shallow weathered rock profile and the ground motions were accordingly deconvolved to bedrock motions using an equivalent-linear analysis.

The deep soil profile motions were obtained from the “LPOC” record station located near the western end of the port on reclaimed land. The accelerograms at both stations are roughly aligned north-south and east-west.

Table 1. Earthquake ground motion records

Earthquake	Magnitude, $M_w$	Epicentral distance and bearing from LPOC station
22 February 2011 (Christchurch)	6.2	4km NW
13 June 2011 (Christchurch)	6.0	5km N
23 December 2011 (Christchurch)	5.9	10km N

### 4 SITE CHARACTERISATION

#### 4.1 Small-strain shear modulus and shear wave velocity

The geological profile beneath the Lyttelton Port LPOC record station has been obtained from borehole logs and CPT soundings and is summarised in Table 2.

Characterisation of the small-strain shear modulus,  $G_{max}$  of the soils and rock underlying a site is an integral part of ground response analysis. Data from a downhole shear wave velocity survey at a nearby location was used to estimate the average shear wave velocity,  $V_s$  (which is directly related by  $G_{max} = \rho V_s^2$ ) for each soil unit. These shear wave velocities were compared with published CPT- $V_s$  correlations (PEER 2012) which were found to be in relative agreement with the adopted values.

Table 2. Ground profile and soil parameter values

Layer No.	Unit	Soil description	Depth of unit (m)	$V_s$ (m/s)	Non-linear $G/G_{max}$ and $\epsilon$ curve
1a	Reclamation fill	Loose silty GRAVEL	0 to 6	130	Vucetic and Dobry (1991) PI=15
1b	Reclamation fill	Medium dense silty GRAVEL	6 to 13	230	
2a	Harbour muds	Soft to firm SILT	13 to 27	180	
2b	Harbour muds	Firm to stiff SILT	27 to 35	250	
2c	Harbour muds	Stiff SILT	35 to 43	230	
3	Harbour sands	Medium dense to very dense SAND	43 to 61	420	Vucetic and Dobry (1991) PI=0
4	Colluvium	COBBLES and BOULDERS	61+	830	EPRI (1993) Rock – 37m to 76m depth

#### 4.2 Non-linear parameters

Geotechnical input parameters required for ground response analysis include modulus degradation ( $G/G_{max}$ ) and damping ( $\epsilon$ ) curves. These curves define the non-linear soil stiffness and hysteretic damping behaviour as a function of the shear strain of the soil. Vucetic and Dobry (1991) showed that the plasticity index (PI) is the main factor controlling the  $G/G_{max}$  and  $\epsilon$  curves for a wide variety of soils ranging from clays to sands. The published curves in Figure 2 have been selected based on soil descriptions in borehole logs as well as historic atterburg limit tests of the soils. Curves adopted for each soil layer are summarised in Table 2.

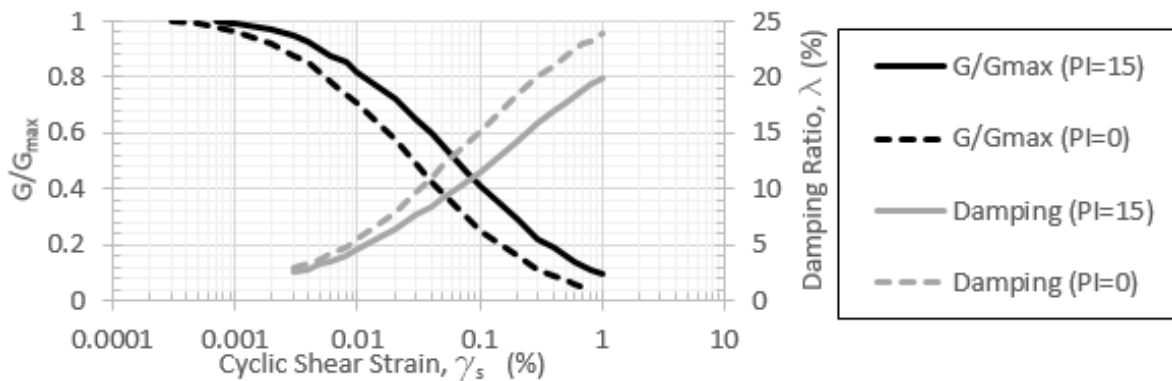


Figure 2. Non-linear  $G/G_{max}$  and damping ( $\epsilon$ ) curves assumed for the reclamation fill and harbour sediments (after Vucetic and Dobry 1991).

## 5 METHODOLOGY

The SHAKE2000 and DMOD2000 program codes were used for the EL and NL analysis respectively. A ‘blind’ analysis was undertaken (i.e. no calibration of the input parameters was carried out against the observed motions). The LPCC rock motions were applied as outcrop motions to the base of the 1D soil columns.

The non-linear DMOD2000 analysis assumed a total-stress analysis without cyclic softening or build-up of pore-water pressures. The DMOD2000 code can account for cyclic stiffness degradation, however Vucetic and Dobry (1991) showed that generally, the  $G/G_{max}$  curve is affected far more by a variation in the assumed PI of the soil. Given the lack of cyclic loading test data and the relative uncertainty in the assumed  $G/G_{max}$  and  $\epsilon$  curves for the soils, approximation and modelling of cyclic degradation in DMOD2000 was considered unnecessary and was not investigated further.

## 6 RESULTS AND DISCUSSION

### 6.1 Peak ground accelerations (PGA)

The EL and NL analyses both predicted soil hysteretic damping for PGAs (Figure 3), as shown by the EL/NL data points plotting below the grey dashed line. The observed PGAs were similar to those predicted, with the NL method generally giving slightly better predictions.

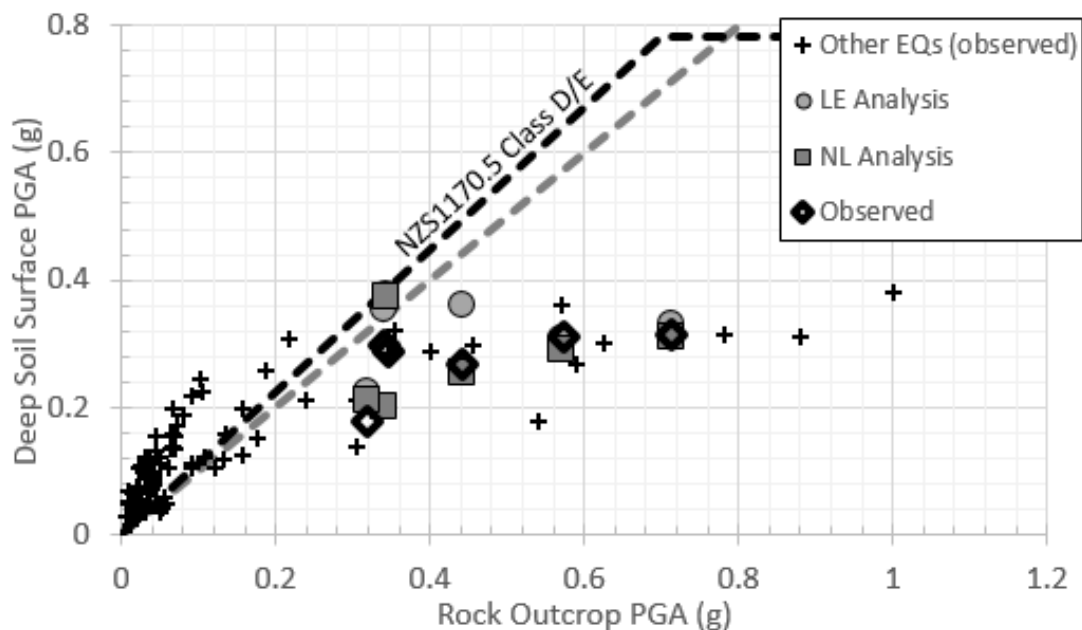


Figure 3. Non-linear response in peak ground accelerations (PGA) at the Lyttelton Port site.

These non-linear ground response effects are supported by the PGA records from other earthquakes recorded between 22 February 2011 and 30 September 2013, which are included in Figure 3. The observed data show clear non-linear effects at the site beyond an outcrop PGA of approximately 0.2g.

It is worthwhile to note that NZS1170.5 Class D/E ( $C_h(0)=1.12$ ) under-predicts PGA for low levels of shaking and over-predicts PGA for large levels of shaking at the site. This is partly because NZS1170.5 considers a spectral shape factor independent of the level of shaking. This may have consequences for assessments based on PGA such as liquefaction triggering, seismic earth pressures on retaining walls or slope stability.

## 6.2 Acceleration response spectra

Although de-amplification of PGAs is observed in these results, ground response effects may result in amplified spectral accelerations at periods greater than  $T=0$ . The 5%-damped acceleration response spectra from the EL/NL analyses are presented in Figure 4(a) together with the observed ground motion response spectra. The observed ground response was slightly different in each direction, as discussed later. Figure 4(a) therefore presents the geometric mean spectra for comparison.

The calculated response for both the EL and NL methods provide reasonable visual matches to the observed spectra for periods shorter than about one second. For periods longer than one second the June record provided a very close fit however the February and December records both under-predicted the observed response.

The larger observed response for periods beyond one second may be due to the occurrence of liquefaction or physical processes that are not captured within the simplifications of a 1D analysis, such as topographical or basin effects (Baturay and Stewart 2003; Afshari and Stewart 2015). However, they may also be a result of the uncertainties in the soil and rock model. Most notably the assumed shear wave profile and non-linear  $G/G_{max}$  and  $\epsilon$  curves. A parametric study or calibration of the input parameters would need to be undertaken to confirm if the 1D analysis is capable of capturing the observed response.

## 6.3 Ratio of response spectra and site period

Ratio of response spectra (RRS) are plots of the ratio between the ordinates of the surface response spectra (with ground response effects) and the ordinates of the outcrop spectra i.e.  $SA_{site}/SA_{outcrop}$ . They provide a means of investigating the ground response at each spectral period. The geometric mean RRS for the Lyttelton Port analysis are presented in Figure 4(b), and several observations are made from them.

### 6.3.1 Site period

The high-amplitude site period (as shown by the peaks in the RRS) is the period at which the maximum amplification of ground response is observed during a large earthquake. This can vary from the low-amplitude site period calculated as 'four times the shear wave travel time to rock' due to the non-linear softening of soils under high strains. Correctly predicting this site period is critical to ground response analysis as was shown in the 1985 Mexico earthquake. Structures with periods at or near the site period can expect to attract potentially significant seismic forces. Calculated high-amplitude site periods agree well with the observed data. For both the February and June records a period at about 1.7 seconds (based on the geometric mean) is observed. For the December record, it is less clear due to different values depending on the direction of shaking, as discussed later. It is evident from the RRS that the NL method provides a closer fit to the observed data for most periods and for all three earthquakes.

Although the calculated site period provides a close match to the observed data, the magnitude of the amplification was under-estimated in all cases. This may be due to topographical or basin effects as previously mentioned; although it would be expected that any resonance of motions due to 3D effects would likely be at a period different to that of the 1D site period, but not necessarily.

### 6.3.2 Non-linear effects at periods $T > 0$

Non-linear effects at high frequencies (i.e. PGA) have already been discussed. However, it can also be observed by way of the RRS that these non-linear effects extend to periods up to about one second (both observed and calculated). This is evident in the smaller short period RRS values for the February record which had the highest level of shaking, and the largest short period RRS values for the December record which had the lowest level of shaking. It is interesting to note however that the peaks of the RRS curves appear to increase with increased levels of shaking.

### 6.3.3 Comparisons with NZS1170.5

Because NZS1170.5 assumes a shape factor,  $C_h(T)$  for ground response effects that is independent of the level of shaking, it is possible to plot the equivalent RRS curve that is assumed for each subsoil class. This is simply the shape factor ordinates for the Site Class divided by the ordinates for a Class B (rock) site. The equivalent RRS curve for a NZS1170.5 Class E site is included in Figure 4(b) for comparison with the site specific ground response.

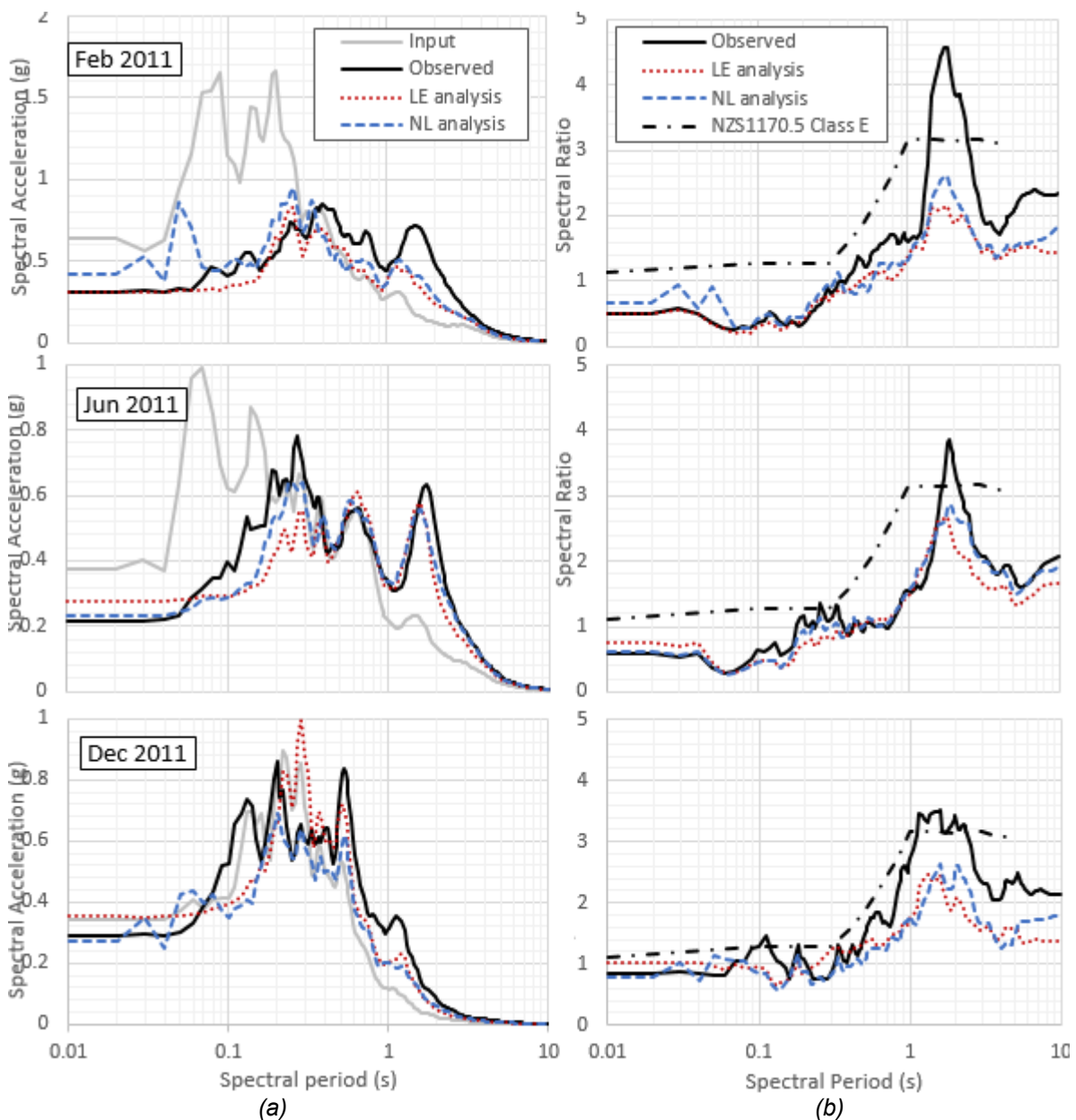


Figure 4. (a) 5% damped geometric mean acceleration response spectra. (b) Geometric mean ratio of response spectra ( $SA_{site}/SA_{outcrop}$ ).

For the December record, the site specific RRS appears to be similar to the NZS1170.5 Class E curve for periods shorter than 3 seconds. For the larger February and June records, the observed site specific RRS are lower than the Class E curve (for periods other than the site period). This is also the case for the calculated responses which provide a much better fit to the observed data over the NZS1170.5 Class E shape factor. This is likely due to the non-linear effects as discussed previously, that NZS1170.5 does not provide for.

#### 6.4 Topographical and basin effects

The ground response analysis presented here has assumed one-dimensional propagation of shear waves only. In reality, the ground response will be influenced by three-dimensional factors such as earthquake source or directivity effects, surface waves, topography or basin effects. However, the extent of these effects are difficult to estimate without detailed investigations and models. The observed ground motion data shows some variation in the ground response in each direction of shaking and this may indicate the presence of 3D effects which could account for some of the differences between the calculated and observed ground responses.

The RRS from the observed ground motions is presented in Figure 5 for the two orthogonal directions of motion recorded. If the ground response was truly one dimensional, it would be expected that the peaks for each direction would be about equal. However, it is observed that the spectral peak in the east-west direction of shaking is at a longer period than for north-south shaking.

The difference might be the result of topographical effects at either the *LPCC* or *LPOC* stations. The *LPOC* station is located on a quay made of reclaimed land behind a rock-fill buttress that runs roughly north-south. If ridge amplification effects due to the elevated land were present, it would be expected that the natural period in the east-west direction would be longer than the north-south direction, and this is what is observed in Figure 5.

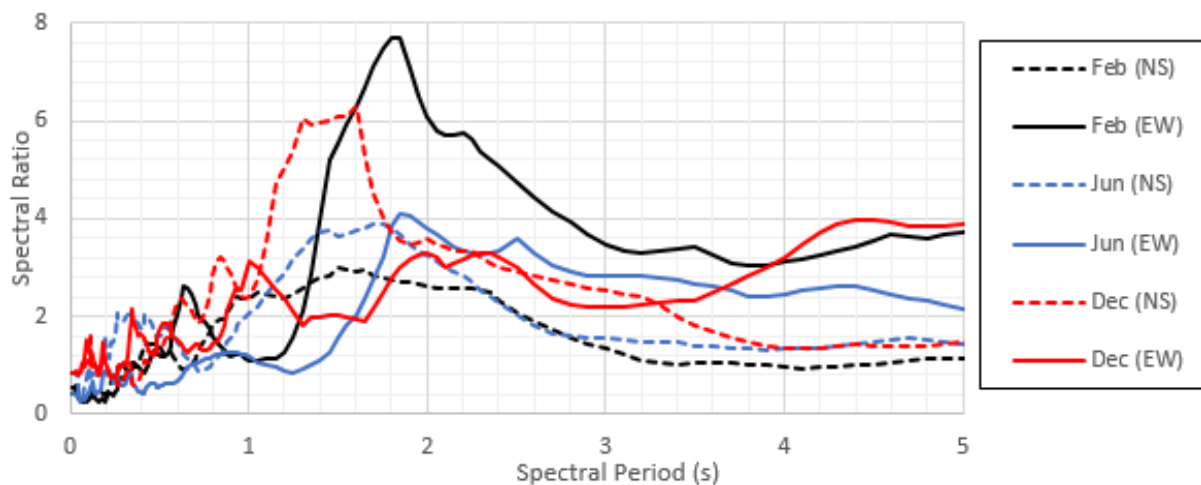


Figure 5. RRS for observed ground motions in each direction of shaking ( $SA_{LPOC}/SA_{LPCC}$ ).

It is also interesting to note that the February record results in the largest amplification across the three records for the east-west direction, but shows the smallest amplification in the north-south direction. The converse is true for the December record. The reason for this is unknown but could be due to earthquake source or directivity effects.

## 7 CONCLUSIONS

- i. Non-linear effects at short spectral periods occur at the port for high levels of shaking. Both the EL and NL methods are able to capture these observations unlike NZS1170.5, which significantly over-predicts PGAs and short period responses for high levels of shaking such as the February 2011 earthquake.
- ii. The observed response at the port site varies between the north-south and east-west directions of shaking. This variation was not seen in the EL and NL methods and indicate the presence of some 2D/3D effects, although the significance or cause of the variation is unclear.

- iii. The calculated response using both the EL and NL methods resulted in acceleration response spectra that provides reasonable matches to the observed spectra for periods less than one second. For periods greater than one second the observed response is generally larger, possibly due to liquefaction or 2D/3D effects not captured in the 1D analysis. A parametric study or calibration of the assumed input parameters would need to be undertaken to confirm if the 1D analysis is capable of capturing the observed response.
- iv. The high-amplitude site period during large levels of shaking corresponds to the period of maximum response amplification and is critical for the design of structures with a similar period. The EL and NL methods both show good agreement with the observed site period, with the NL method providing a slightly better fit. However, both methods under-estimate the maximum amplification at this period.
- v. The site specific ground response analysis provides a much better fit to the observed response when compared with the NZS1170.5 subsoil class method for all periods except near the site natural period (although this may be able to be improved with calibration of the model). This is particularly the case for high levels of shaking (with non-linear effects) such as the February and June 2011 earthquake, and is due to the inherent simplifications within the subsoil class method. NZS1170.5 significantly over-predicts the seismic hazard at the Lyttelton Port deep soft soil site (Class E) for long return period earthquakes. It is recommended that a site specific ground response study be considered for important structures at sites with similar conditions.

## 8 ACKNOWLEDGEMENTS

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