RETAINING STRUCTURES PERFORMANCE AGAINST SEISMIC LOADS FOR AUSTRALIAN TYPE GROUND MOTIONS

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ABSTRACT

The conventional design of retaining structures against seismic loads is based on the Mononobe Okabe type design rules that are specified in many international seismic codes. Attempts had been made by researchers to apply these rules to a range of retaining walls. However, the problem with these conventional methods is that they are grounded in highly seismic area type ground motions which have considerably different frequency content to Australian type ground motions.

This paper presents the results of dynamic time history analyses of gravity retaining walls subjected to Australian type earthquake ground motion. These are based on FLAC non-linear dynamic modelling of a range of retaining walls (e.g. bridge abutments, etc). Comparisons are made with results of conventional Mononobe Okabe type design rules for retaining walls with varying lateral and bending stiffnesses.

1 INTRODUCTION

The conventional design of retaining structures against seismic loads is based on the Mononobe Okabe type design rules that are specified in many international seismic codes. Attempts had been made by researchers to apply these rules to a range of retaining walls. However, the problem with these conventional methods is that they are grounded in highly seismic area type ground motions which have considerably different frequency content to Australian type ground motions. This paper presents a study of dynamic time history analyses of retaining walls subjected to Australian type earthquake ground motion.

2 DESIGN BEDROCK GROUND MOTION

The Australian bedrock ground motion is defined by the normalised response spectra for different site sub-soil class as in the Australian Earthquake Loading Standard AS1170.4 (2007) with a scale seismic hazard factor (Z). Z represent the effective peak ground acceleration (PGA) in gravity, g. Figure 1 shows the design rock type (Class B) response spectrum for a hazard factor of 0.08 g, such as Sydney having a 10% chance probability of being exceeded in the next 50 years. For comparison purposes, the UBC (1997) design rock response spectrum for California is also shown on this figure.

It is a common practice for engineers to use the code defined response spectrum for seismic design. However, site specific probabilistic seismic hazard assessments (PSHA) are carried out when the available seismic code is considered overly conservative for certain structural periods or a hazardous or important facility is being designed. Arup has carried out a number of PSHA for different regions of seismicity. These regions include East Coast of Australia, Hong Kong, Korea, Japan and Philippines and they are also presented in Figure 1. In general, the Arup PSHA studies are comparable with the code spectra presented in Figure 1 for moderate to high seismicity regions.

By normalising all the response spectra shown in Figure 1 to a value of 1 at 0.01 seconds as shown in Figure 2, it is found that the frequency content of the Arup PSHA response spectra is quite different between the high and moderate seismicity regions. The scaled spectral mapped accelerations for New York at 0.2 second and 1 second are also presented in Figure 2 and they are similar to the Arup PSHA results of the moderate seismicity region. Interestingly, the design AS1170.4 rock type response spectral shape is found to be similar to the California UBC type until 1.5 seconds. The frequency content reduces after 1.5 second, as the Australian design has a constant spectral displacement after 1.5 seconds but the California design remains at a constant spectral velocity (see Figure 3 and Figure 4).

In this study, the Australian code design and the Arup PSHA for the East Coast of Australia earthquake ground motion shapes have both been used to assess the dynamic response on a retaining wall. The California type of earthquake ground motion has also been assessed for comparison.

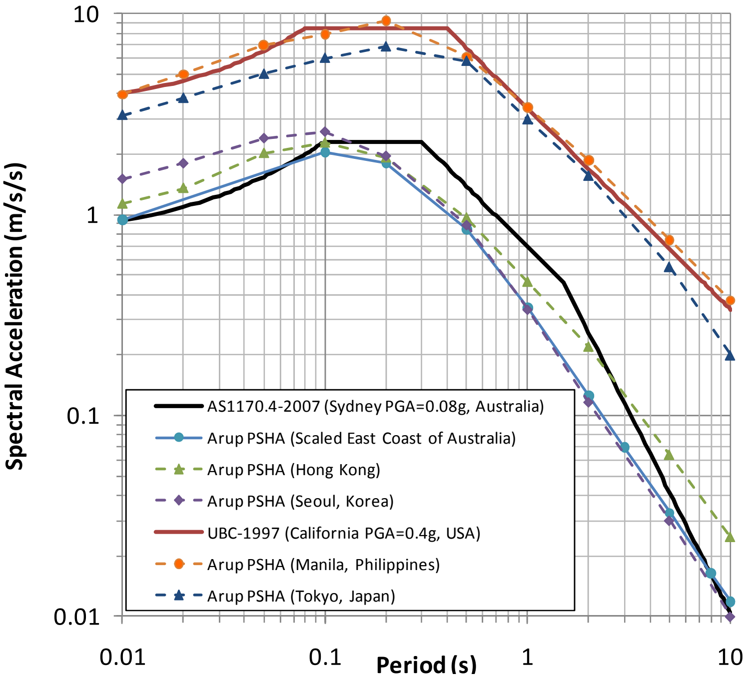

Figure 1: Bedrock Acceleration Response Spectra (10% chance in next 50 years).

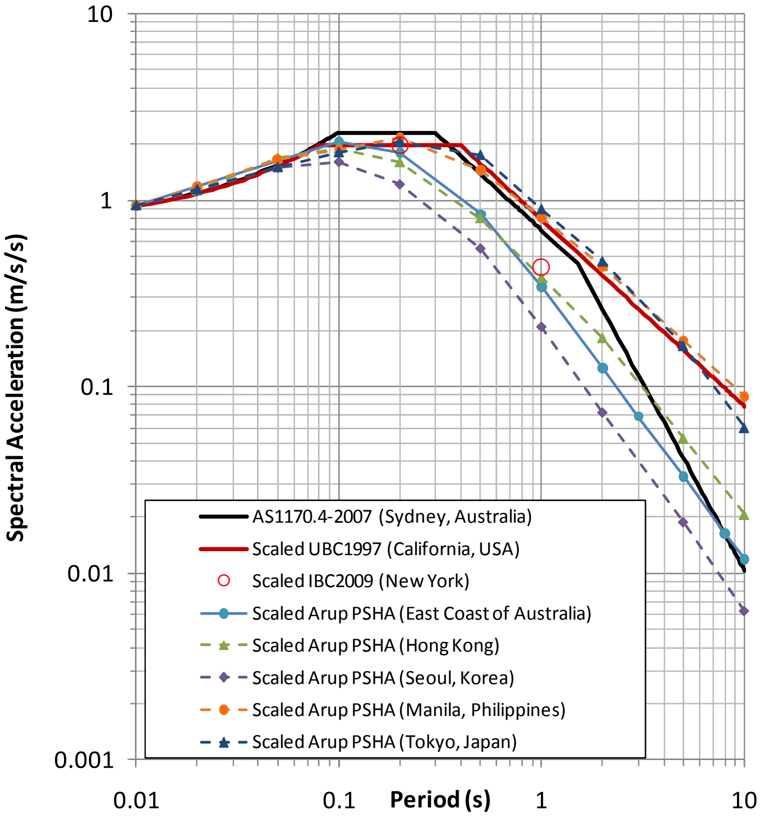

Figure 2: Normalised Bedrock Acceleration Response Spectra.

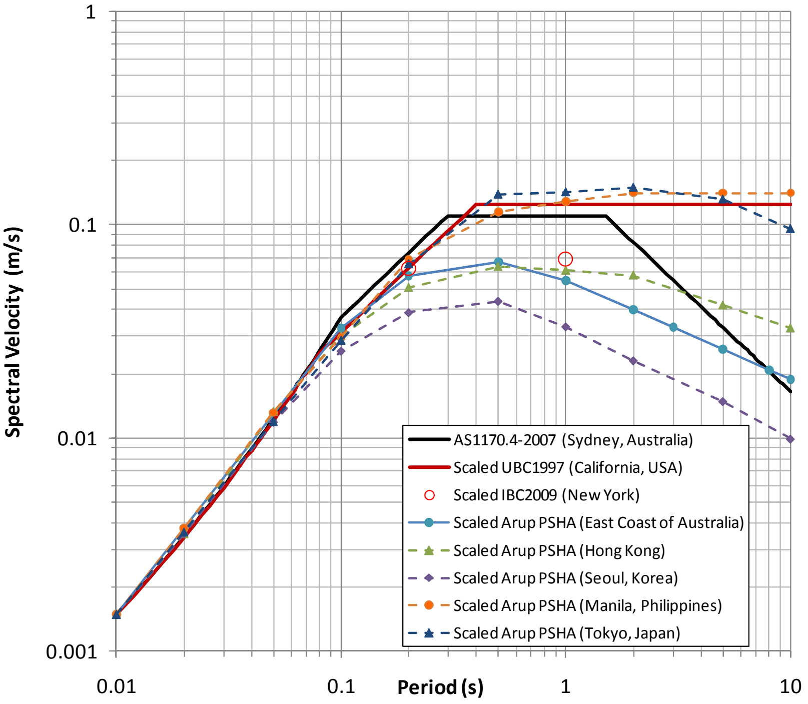

Figure 3: Normalised Bedrock Velocity Response Spectra

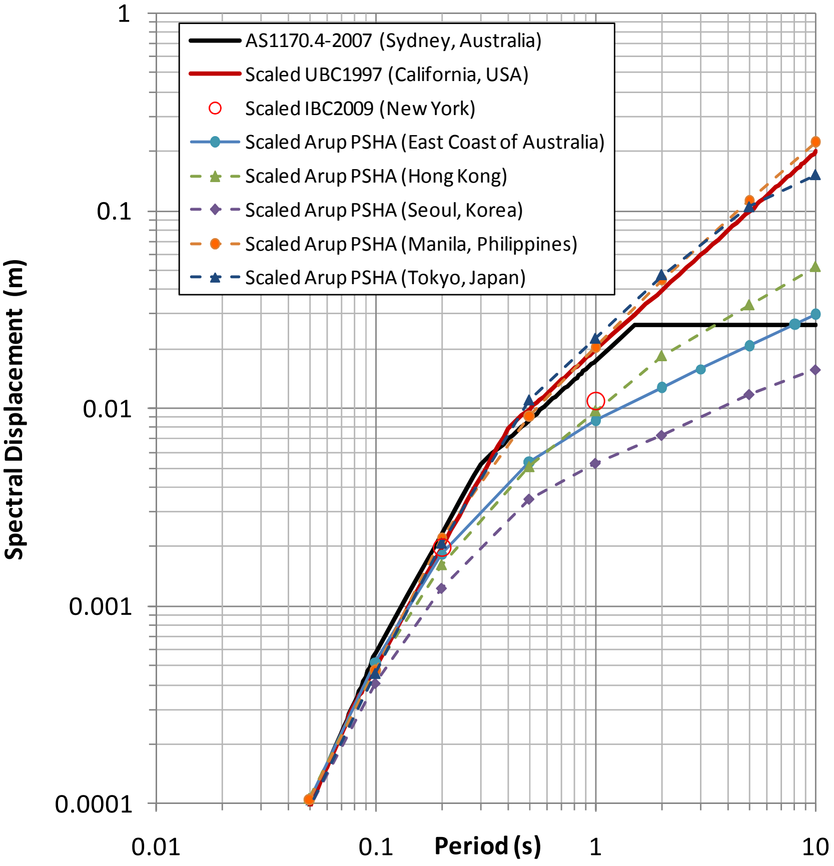

Figure 4: Normalised Bedrock Displacement Response Spectra

3 NON-LINEAR ANALYSIS

The dynamic analysis option of FLAC Version 6.0 permits two-dimensional, plane-strain or axisymmetric, non-linear dynamic analysis. The calculation is based on the explicit finite difference scheme to solve the full equations of motion, using lumped grid point masses derived from the real density of surrounding zones. This formulation can also be coupled to a structural element model, permitting analysis of 2 dimensional dynamic soil-structure interaction. The application of FLAC dynamic has been widely used in the international engineering practice.

In the late 1980's, Arup developed a simple explicit non-linear one-dimensional finite difference program, Oasys SIREN, to analyse the response of a one dimensional soil column subjected to an input dynamic motion at its base. Detailed calibration analyses with post-earthquake data undertaken using Oasys SIREN are described by Henderson *et al.* (1990) and Heidebrecht *et al.* (1990). Theoretically the soil model used in Oasys SIREN more accurately reflects actual hysteretic soil behaviour when compared to pseudo non-linear soil models used in many other dynamic response programs e.g. SHAKE (Schnabel *et al.*, 1972). However, the limitation of this program is a one dimensional solver which cannot take into account the two dimensional geometry required for slope analyses and Mohr-coulomb type of soil failure. In the next section, comparisons of soil surface response between FLAC (2-D) and Oasys SIREN are presented.

4 ANALYTICAL MODEL

4.1 MODEL GEOMETRY AND PARAMETERS

A 5 m high cantilever retaining wall is used in this study to assess three types of seismic ground motion all scaled to a PGA of 0.08 g:

1. Arup PSHA response spectrum for the East Coast of Australia
2. Australian Standard AS1170.4 design response spectrum
3. California UBC design response spectrum

The FLAC model is presented in Figure 5. It is a 5 m thick homogenous soil layer with earthquake motion input underneath the soil layer. A 5 m high cantilever retaining wall is attached to the soil grid. The toe rotation of the retaining wall is fixed. The grid size is 0.3 m x 0.3 m, which has been tested to be insensitive to further reduction of grid size. The fixity of boundaries is shown in Figure 5. A free-field boundary is applied to right hand side of the model boundary during the dynamic analysis to avoid wave reflection. A model damping of 0.5% Rayleigh damping at 3Hz is adopted. The soil model is assumed to be dry. The degradation of the soil shear modulus as a ratio of the small strain shear modulus G_0 is shown as a function of shear strain in Figure 6. The soil and wall properties are summarised in Table 1.

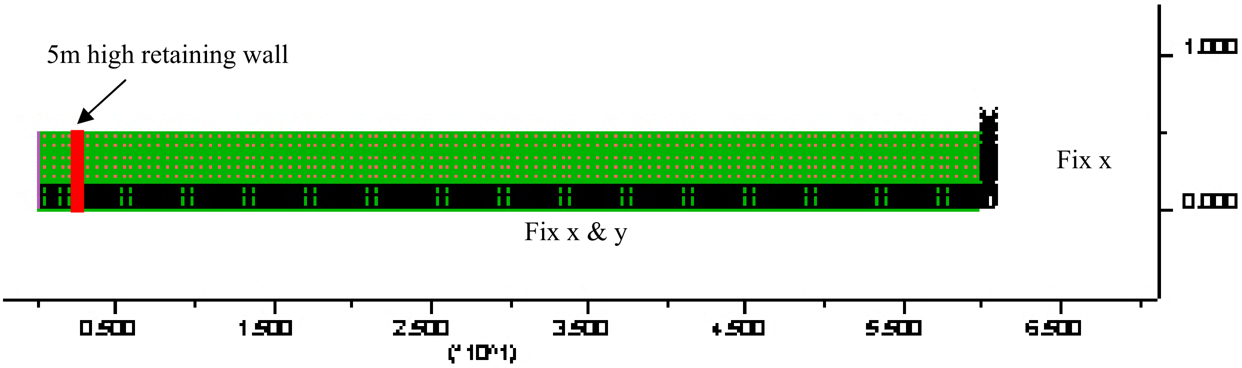

Figure 5: FLAC Geometry.

Table 1 Summary of soil and wall properties

Properties	Soil	Wall
Density	1900kg/m^3	2400kg/m^3
Initial Bulk Modulus	165 MPa	
Initial Shear Modulus	76 MPa	
Elastic Modulus	-	20GPa for high stiffness wall (Grade 30 Concrete) 2GPa for low stiffness wall
Shear Wave Velocity	200 m/s	-
Soil Model	Mohr-Coulomb Model	Elastic
Friction angle	33 degrees	-
Cohesion	5kPa	-
Shear Degradation Curve (Figure 6)	Sand (Upper bound curve of Seed & Idriss, 1970)	-

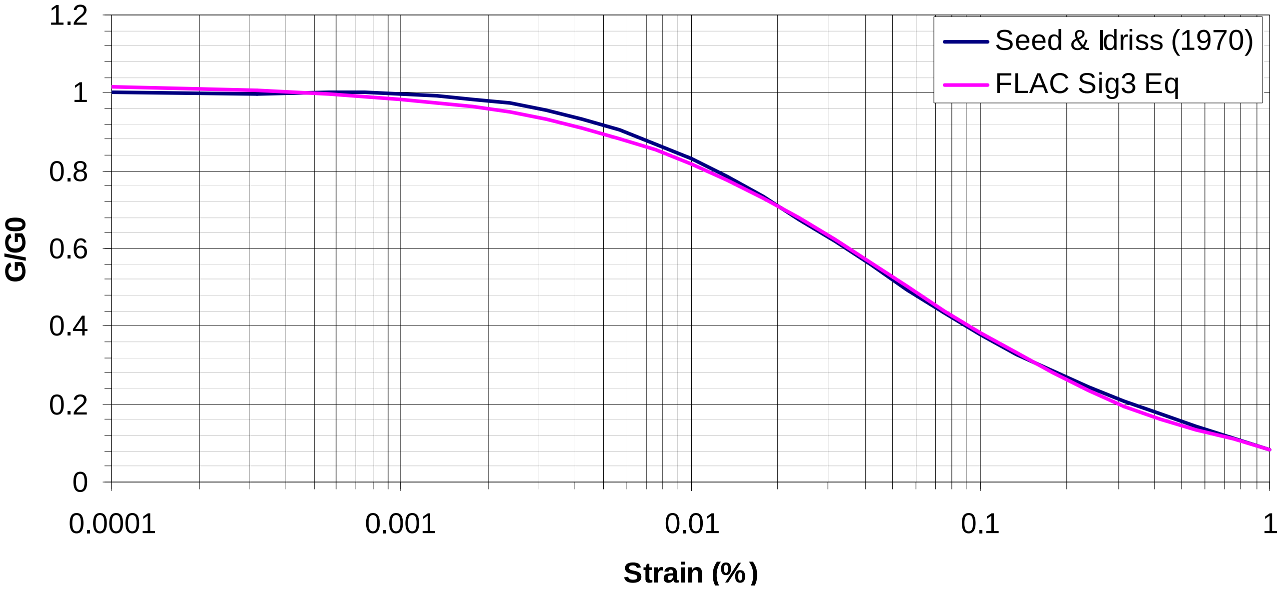

Figure 6: Soil Shear Modulus Degradation Curve.

4.2 INPUT EARTHQUAKE TIME HISTORY

The 1986 Chelfant Valley Earthquake (USA) with a magnitude of 5.8 and a distance of 26 km was used to represent an earthquake scenario event for Australia with a 10% chance probability being exceeded in the next 50 years. This selected horizontal earthquake time history was spectrally matched to the selected target response spectra in this study as shown in Figure 7. A time domain method was used as the favoured approach for spectral matching. Wave packets are added to the recorded time history within small and discontinuous time windows where there is significant amplitude of motion of the same period. The time domain method better preserves the original features of the recorded time histories. The computer program RSPMATCH (Hancock *et al*., 2006) was used for this study. The spectrally matched time histories are shown in Figure 8.

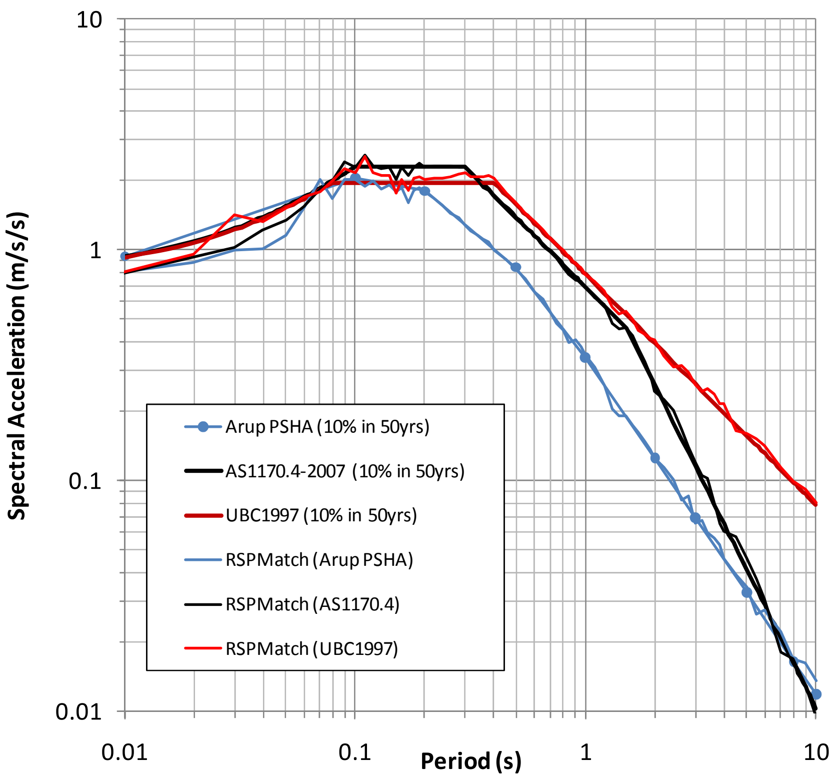

Figure 7: Spectrally Matched Response Spectra

4.3 PARTIAL TRANSMISSION IN BEDROCK

In order to model the partial transmission of waves into the bedrock underlying the soil layer, the principal proposed by Papastamatiou (1973) is used in the following equation.

$$v_b = v_s + \frac{\tau}{v_r \rho_r}$$

In this method the input rock motion is modified so that v_b is the derived base velocity, v_s is the specified input velocity time history for the bedrock outcrop, τ is the shear stress in the lowest soil element, v_r is the shear wave velocity of the bedrock (1000 m/s) and ρ_r is the bedrock density (2500 kg/m^3).

Oasys SIREN allows the partial transmission of waves using this theory. The calculation of this partial transmitting motion is carried out by Oasys SIREN using the spectrally matched time history as the input base excitation. The soil height, shear wave velocity and shear modulus degradation curve used in the SIREN model are the same input as the FLAC analysis. The calculated acceleration time history at the bottom layer of the 5m soil in SIREN is then used as the input of the FLAC base excitation.

The spectrally matched time history and the calculated time history with the partial transmission of waves for the Arup PSHA, AS1170.4 and UBC types ground motions are presented in Figure 8. The peak acceleration is slightly reduced after the partial transmitting process.

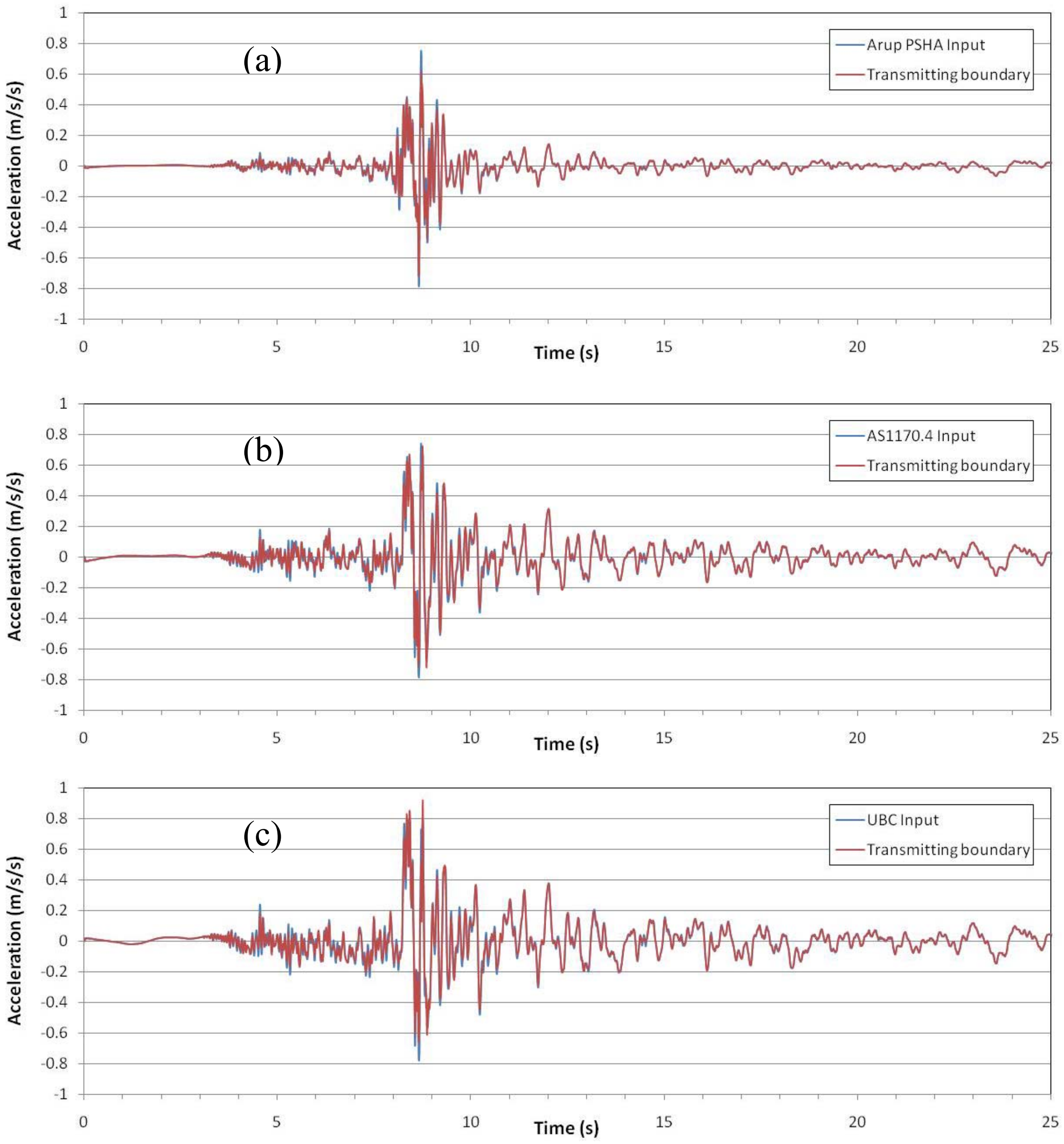

Figure 8: Spectral Matched Bedrock Time History.

5 RESULTS

5.1 COMPARISON OF SURFACE SOIL RESPONSE SPECTRAL RATIO AND TIME HISTORY BETWEEN FLAC-2D AND OASYS SIREN

The free field ground surface soil response spectrum obtained from Oasys SIREN has been compared with the results of FLAC dynamic by using the same input parameters as described in Section 4. Figure 9 shows the soil amplification ratio of the spectrum at ground level over the input spectrum obtained from Oasys SIREN and FLAC. They match quite well, except at very low period. Oasys SIREN gives higher range of amplification factors at the low period as result of the undamped resonance effect at the first layer of soil node and the results can be discarded. The ground surface response time histories obtained from Oasys SIREN and FLAC are presented in Figure 10. The results show similar soil response in the time histories obtained from Oasys SIREN and FLAC.

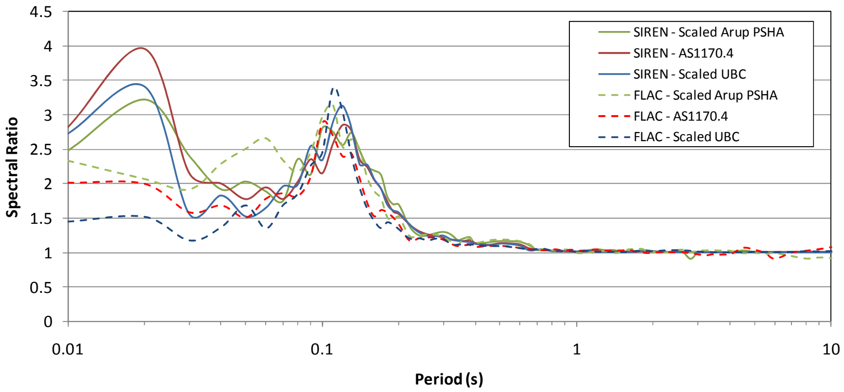

Figure 9: Ground Surface Soil Response Spectral Ratio.

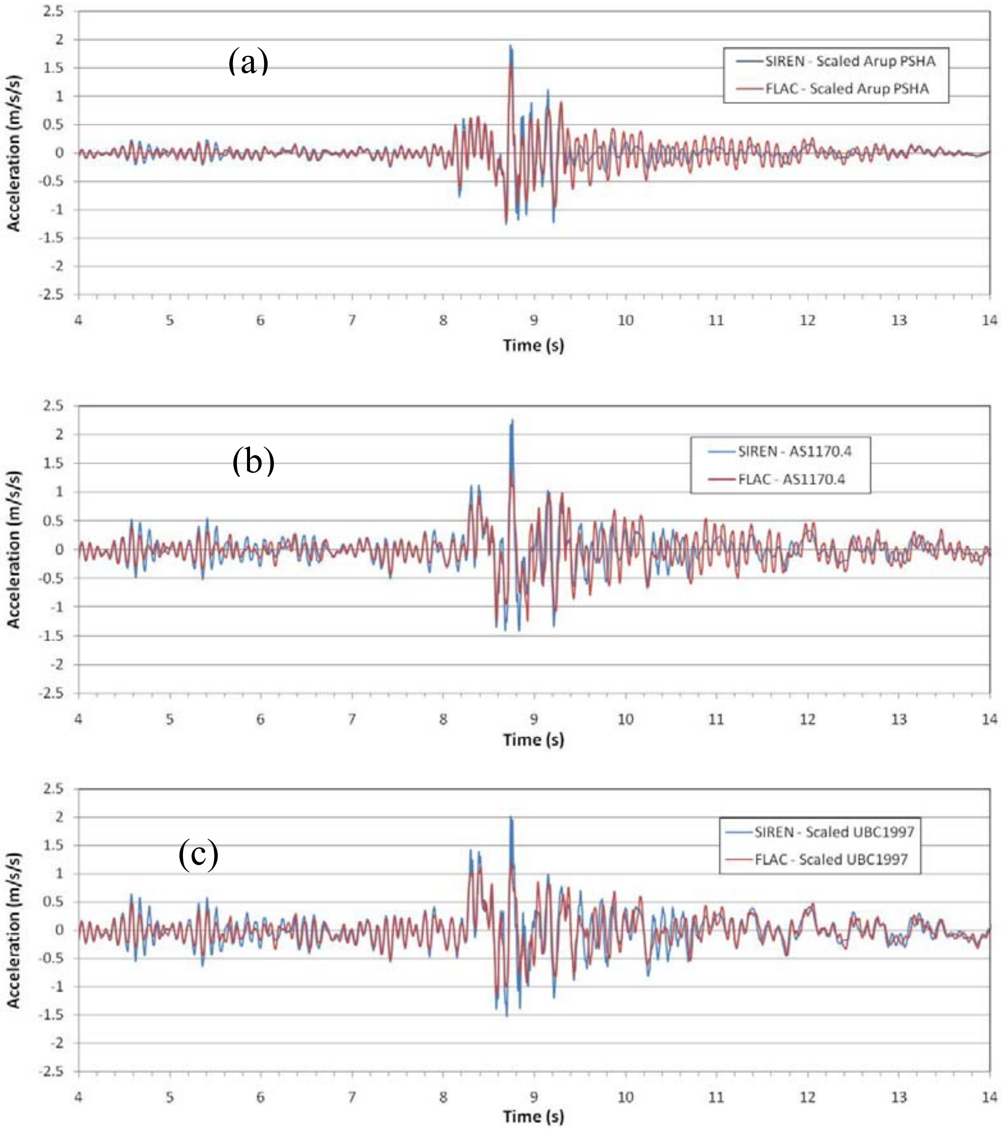

Figure 10: Surface Response Time History

5.2 DYNAMIC RESPONSE OF WALL LATERAL DISPLACEMENT

Eurocode 8 Part 5 gives a very useful summary of how to calculate seismic earth pressures acting onto retaining walls. For walls that are very flexible or able to move forwards during the earthquake event they propose the conventional method of calculating the seismic active earth pressure using the Mononobe Okabe methodology and for fixed walls the simplified Wood's method (1973) where the soil force increase $\Delta P = \gamma H^2 k_h$ and γ is the soil unit weight, H the height of the wall and k_h the peak horizontal acceleration as a fraction of gravity. To determine this additional earth pressure profile, the recommendation of Matthewson *et al.* (1980) is often adopted, where the dynamic earth pressure decreases linearly from the top of the wall with the top pressure being three times that at the base.

A common difficulty is how to assess whether a retaining wall is effectively flexible or rigid whereas it is often somewhere in between. To explore this question a series of "FLAC Dynamic" analyses have been carried out as presented in Section 4. Two wall stiffnesses (EI) of 167 MNm^2/m and 1670 MNm^2/m have been assumed to represent walls ranging from relatively flexible to relatively stiff.

Figure 11 shows the relative wall top displacement time history plots for the high and low stiffnesses walls. The maximum displacement of the wall top under dynamic loading is 1.3 mm, 2.6 mm and 3.7 mm for the high EI value subjected to the Arup PSHA, AS1170.4 and UBC types of ground motions respectively. The maximum deflection for the low EI value is 3.5 mm, 6.1 mm and 8.1 mm for the Arup PSHA, AS1170.4 and UBC types of ground motions respectively. The results show that the maximum lateral displacement of wall top is increasing with the increase of the displacement content of response spectrum from the input ground motions.

For comparison the deflection of the top of the walls under static loading was 0.25mm and 1.1mm for the high and low EI values respectively. It can be seen from Figure 11 that the dynamic motion increases the wall displacements to be 5, 10 and 15 times these original values for high EI value and increases to be 4, 6 and 8 times these original values for low EI value for the Arup PSHA, AS1170.4 and UBC types of ground motions respectively.

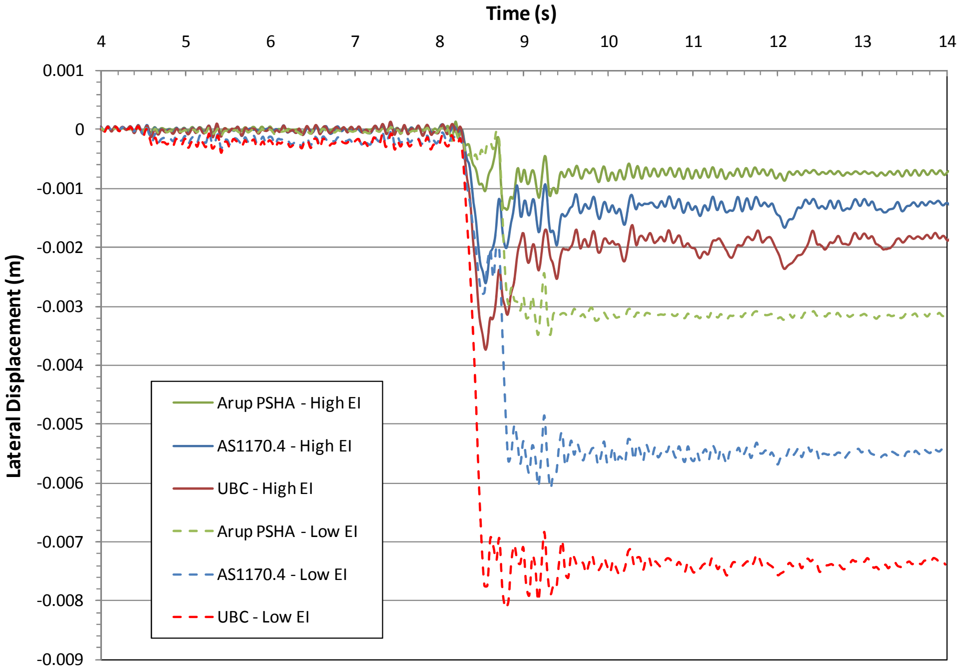

Figure 11: Relative Wall Top Time History to Input Motion

5.3 DYNAMIC RESPONSE OF WALL BENDING MOMENT

Figure 12 shows the predicted wall base bending moments for the three input ground motions. The bending moments at the base of the wall predicted by the Mononobe Okabe method for flexible walls and by the Wood's method for rigid walls with average and maximum soil amplification on PGA (calculated by FLAC) are also shown. The UBC type ground motion is seen to agree well with the Mononobe Okabe method for low stiffness wall as expected since this method is grounded in highly seismic area type ground motions. However, the results show that the Arup PSHA motion gives less bending moment for low stiffness wall than the value obtained from the Mononobe Okabe method. Therefore, it indicates that the dynamic response of a retaining wall is affected by the frequency content of the input ground motions.

Interestingly, the Wood's method agrees well with that predicted for stiff wall when Arup PSHA motion is used. Also, the values obtained from the AS1170.4 and UBC are somewhat much higher than that predicted by the Wood's method for high stiffness wall.

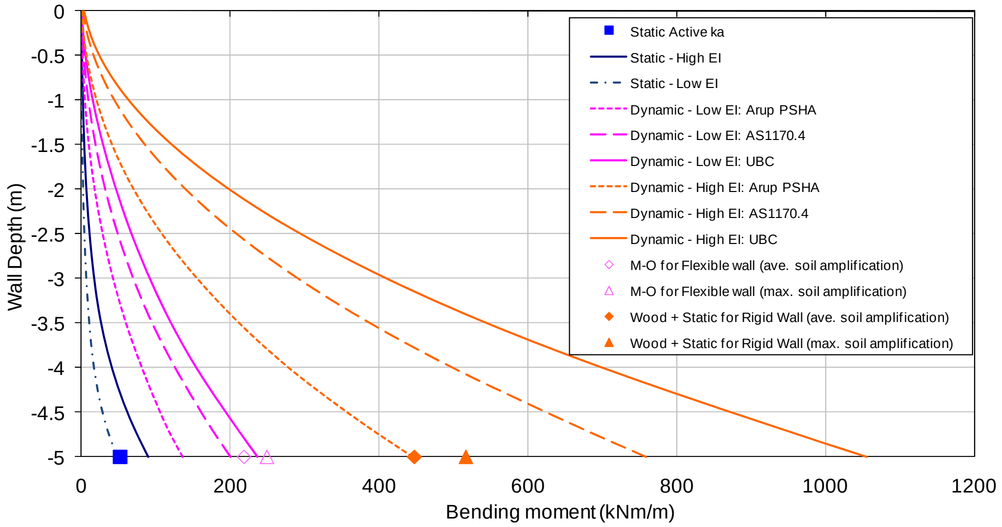

Figure 12: Bending Moment Diaphragm

6 CONCLUSION

These preliminary calculations show that for both high and low stiffnesses walls, the Arup PSHA type ground motion for the East Coast of Australia scaled to 0.08 g gives lower bending moment of wall base than the design Australian standard scaled to 0.08 g. The UBC type ground motion, which has larger long period energy which also scaled to 0.08 g gives even larger values. The UBC type ground motion is seen to agree well with the conventional Mononobe Okabe method for low stiffness wall. It follows that the dynamic response of a retaining wall is affected by the frequency content of the input ground motions and is not just dependent on peak ground acceleration as is assumed by conventional design method.

This study also shows that conventional simplified Wood's method (1973) for retaining wall design may be reasonable for the Arup PSHA type ground motion but possibly optimistic for regions of high seismicity and the use of Australian design response spectrum.

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