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BEHAVIOR OF CANTILEVER RETAINING WALLS UNDER SEISMIC CONDITIONS

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ABSTRACT

Analyzing the behavior of earth retaining structures under seismic conditions has been very important issue due to their wide applications in several infrastructural applications and other structures. The problem of instability of walls is mainly related to earth pressure distribution on the wall and the response of wall against the earth pressure, especially, under dynamic loading condition. Soil – wall interaction is an important property which governs the dynamic behavior of the wall. Even after a large number of studies, the dynamic behavior of soil-wall system is still not completely elucidated. The current study will be helpful in improvising the design of retaining walls allowing them to perform well.

The objective of this paper is to study the dynamic behavior of a retaining wall along with the earth pressure distribution of soil in seismic conditions. Among various types of retaining structures, cantilever retaining wall was adopted for the present study. Numerical model was developed using finite element method based package to simulate the dynamic behavior of the cantilever retaining wall. The developed numerical model was verified for validation with the available physical model studies in the literature. The validated numerical model was then subjected to seismic records with different predominant frequency. The methodology followed in developing the numerical model and results obtained from the parametric studies are discussed in this paper.

Keywords: seismic analysis, retaining wall, soil-structure interaction, finite element analysis

INTRODUCTION

There have been several damages to retaining walls due to earthquakes in many parts of the world. Damages to retaining walls used for slope stability, road and railway embankments, bridge abetments and in nuclear power reactors are highly sensible as the damages to these structures are related to a large number of casualties. For many decades, the seismic analysis of retaining walls has been based on the simple extension of Coulomb's limit equilibrium analysis, also known as M-O method, by treating the dynamic forces from a seismic event as quasi-static forces acting along the failure plane. Dynamic lateral earth pressures behind a soil retaining wall subjected to an intensive ground motion can be significant and cannot be determined accurately using pseudo-static approach. These additional earth pressures can result in damage to or collapse of the structure.

Dynamic earth pressures depend on a large number of parameters such as backfill density, angle of internal friction of soil, structural design of wall, ground motion parameters like peak ground acceleration, duration of strong motion and predominant frequency of the earthquake. The predominant

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frequency of earthquake plays a vital role in behavior of a retaining wall during a seismic event. Dynamic response of retaining walls to ground motion has been the subject of several studies including both physical modeling and mathematical modeling. To conceptualize the real behavior, a large number of studies have been done using the centrifuge tests (Dewooker et al. (2000)) and shake table tests. Watanabe et al. (2003) conducted a series of shaking table tests with irregular excitation to establish practical design procedures to evaluate seismic stability of different types of retaining walls against high seismic loads. Reinforced-soil retaining wall models with a rigid full-height facing exhibited a ductile behavior. Although physical modeling predict the exact behavior of wall but the expenses and low reusability of model make this method less desirable and create a need for mathematical modeling methods.

Clough and Duncan (1971) developed a method for finite element simulation of realistic behavior of interface between retaining wall and backfill soil. Cameron and Green (2004) conducted a non-linear numerical analysis using FLAC to evaluate the effect the soil-structure system flexibility on the magnitude distribution of lateral seismic earth pressures on cantilever retaining walls. Psarropoulos et al. (2005) developed a method for finite element analysis of rigid and flexible retaining walls to analyze more realistic solutions that are not amenable to analytical solution, by modeling the soil as visco-elastic continuum, solutions were converged to analytical solutions of Wood (1975) for a rigid wall and of Velestos and Younan (1997) for a wall having a rotational or structural flexibility at its base. Madabhushi and Zeng (2007) performed a finite element simulation of a flexible, cantilever retaining wall with dry and saturated backfill under earthquake loading and compared the results with that of a centrifuge.

A little can be found in the literature that specifically addresses the influence of frequency of system as a parameter. Hatami and Bathurst (2000) studied the effect of structural design on fundamental frequency of reinforced-soil retaining walls and showed that the resonance frequency of wall were dependent on the ground motion intensity and to a lesser extent, on the height to width ratio of the backfill. Theodorakopoulos et al. (2001) studied the soil displacements and stresses, wall pressures and resultant forces as well as the pore water pressure and their variation with frequency, hysteric damping, porosity and permeability to assess the relative importance of the parameters on the response. It was observed that the resonance frequency of the horizontal displacement, the stresses and the resultant forces increase with increasing porosity and permeability. A few other studies have been performed with emphasize on structural frequency and resonance frequency of walls.

This study aims to study the behavior of an L-shaped precast cantilever retaining wall with dry backfill subjected to several ground motions having different predominant frequency. The physical model of cantilever retaining wall tested by Watanabe et al. (2003) was simulated in finite element model and the results were compared to validate the model. The finite element software package Abaqus® v6.8 was used to carry out the numerical experiments. The program is widely being used for both linear and non-linear numerical simulations in various engineering fields as it is quite fast in non-linear analysis.

NUMERICAL MODEL

Physical model of cantilever retaining wall tested by Watanabe et al. (2003), shown in Fig. 1-(a), was adopted as the typical geometry for the generation of 2D retaining wall model. Total height of the wall was 530 mm and the bottom width at the base of the cantilever retaining wall was 230 mm. The subsoil and backfill layers were made of air-dried Toyoura sand (G_s = 2.648, e_{max} = 0.977 and e_{min} = 0.609) with average relative density of 90%. A surcharge of 1 kPa was applied on the surface of the soil backfill.

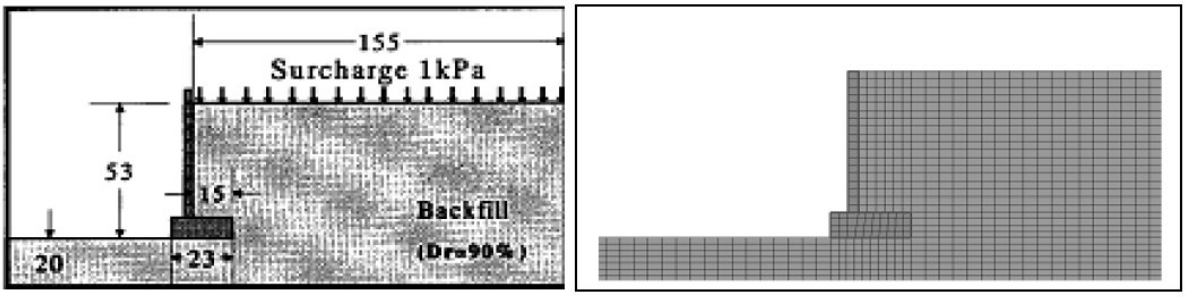

Figure 1- (a) Physical model of retaining wall (Watanabe et. al., 2003), (b) Finite element model

Finite element mesh developed for analysis is shown in Fig. 1-(b). First order, reduced integration, four nodded plain strain elements were used and nodal definitions are referred to a global rectangular coordinate system centered at the lower left corner of the wall. Prior to earthquake excitation, the system is subjected to gravity loading due to its self-weight and surcharge pressure. In next step system is subjected to 1995 ‘Kobe’ earthquake excitations.

Soil constitutive model and Material Properties

The wall was modeled as a continuous concrete panel with elastic properties having unit weight as ρ_c = 25 KN/m^3. The modulus of elasticity and Poisson’s ratio of the concrete was E = 25000 MPa and υ = 0.15 respectively. The soil was modeled as frictional, elastic-plastic model with Mohr-Coulomb failure criterion. Mohr-Coulomb plasticity model provided in Abaqus® was used with a bi-linear soil hardening rule to define the expansion of yield surface. The unit weight of the soil was 19.5 KN/m^3, modulus of elasticity E_s= 60 Mpa and poisson’s ratio υ = 0.33. The reference friction angle of soil was ϕ = 37° and dilation angle ψ = 10°. Nominal cohesion yield stress for the soil was adopted as 1 kPa.

The wall-soil interaction was modeled using the surface-to-surface interaction module in software package with friction angle between concrete and soil ϕ_i = 37° and damping coefficient ξ_i = 0.1.

Problem boundaries and analysis steps

The analysis was performed in three steps to simulate the stages in field. A general static, time independent step was defined for gravity and surcharge loading while the wall was braced horizontally using rigid external supports to achieve static equilibrium. The wall support was then released as done in field. After the static equilibrium was achieved, the system was subjected to seismic loading with a dynamic-implicit step. The acceleration record shown in the Fig. 2 was applied horizontally to all nodes at the bottom and side boundaries of the backfill region. The application of uniform acceleration at the vertical boundary was based on the assumption of uniform distribution of horizontal acceleration over the depth of the backfill away from the wall. Kobe earthquake data modified to have a predominant frequency of 5 Hz was used to validate model with the results obtained by Watanabe et al. (2003). Fig 2 shows the modified Kobe record with 0.81g PGA (Peak Ground Acceleration). For parametric study 5 different seismic inputs modified to have 0.3g PGA with their original predominant frequencies were used. Table 1 shows peak ground acceleration values and predominant frequencies of the 5 seismic records used.

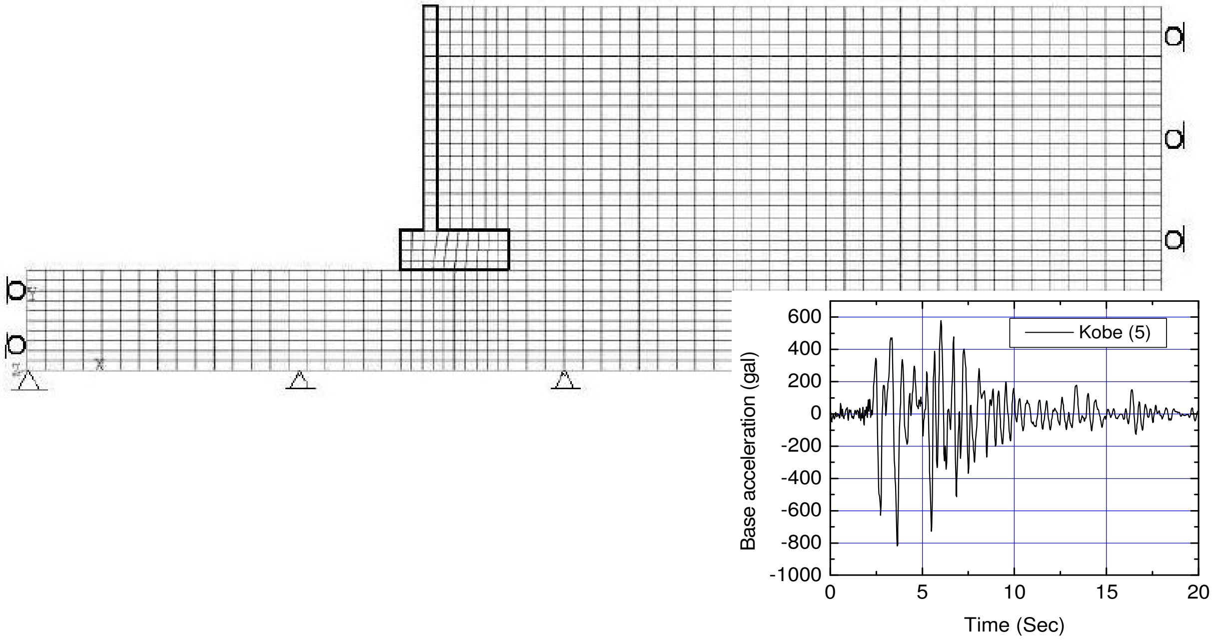

Figure 2 – Finite element mesh and boundary conditions of the model for static step

Table 1 – Earthquake data

Model	Earthquake	Peak ground acceleration	Predominant frequency
1	Kobe	0.81g	5
2	Emaryville	0.3g	0.847
3	ElCentro	0.3g	2.0
4	Friuli	0.3g	3.846
5	Sakaria	0.3g	6.25
6	Parkfield	0.3g	12.5

RESPONSE OF WALL MODEL

Validation of the model

A plot of wall top residual displacement with seismic coefficient (K_h) values, corresponding to different PGA ($K_h g$), was obtained from the numerical simulations. A comparison between numerical modeling results and physical modeling results reported by Watanabe et al. (2003) has been shown in Fig 3. Wall top displacement values of both studies were comparable up to K_h=0.4. Therefore for further parametric studies all seismic inputs were modified to have a PGA=0.3g with different predominant frequencies to study the effect of the predominant frequency of earthquake.

Results of parametric studies

Lateral earth pressures (lateral normal stress on wall) after different seismic events have been compared in Fig. 4. The very first thing to observe is the earth pressure profile obtained along the height of the wall was non linear. As compared to 'at rest' earth pressure, the post earthquake earth pressures have lower values. During a seismic event, earth pressure increases due to ground acceleration which tends to shift or translate and topple the wall from its rest position. As wall was allowed to tilt, this excess pressure was mobilized and hence earth pressure reduces from actual value and wall gets a permanent deformation after earthquake.

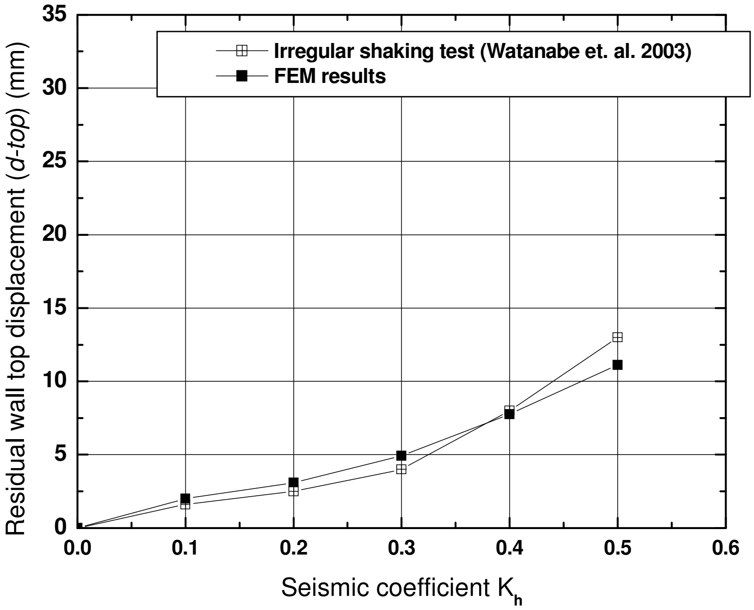

Figure 3 - Comparison of FEM results with shaking table test results

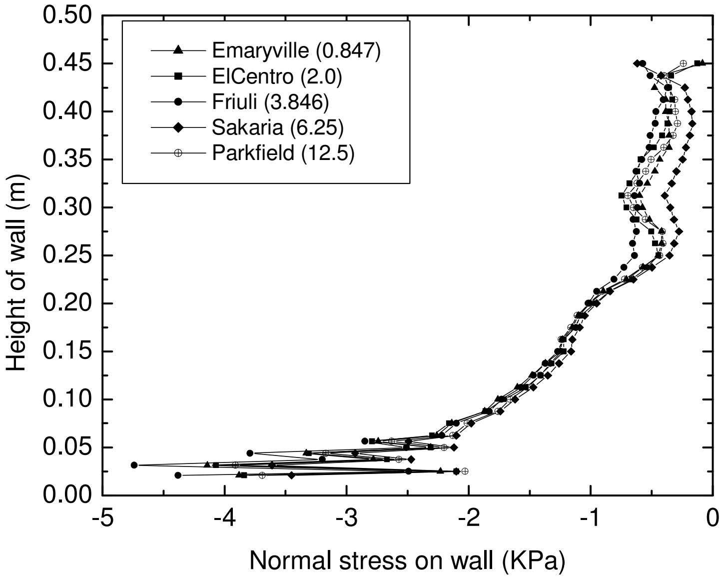

Figure 4 – Lateral normal stress distribution along height of the wall (Negative values indicate compressive behavior)

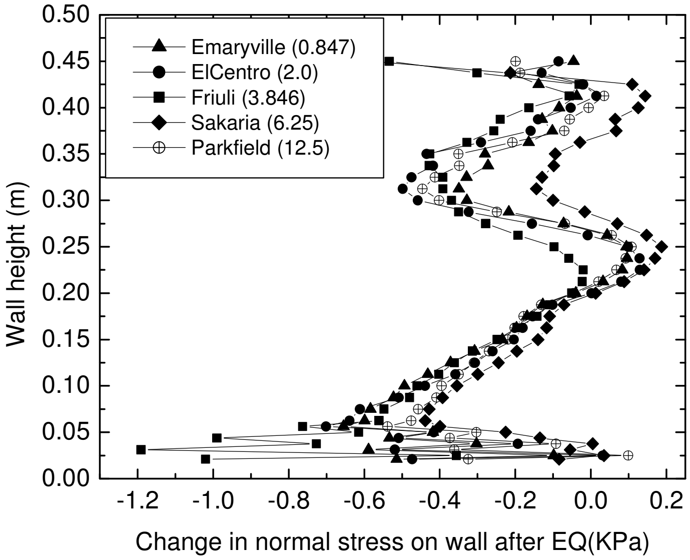

Figure 5 - Change in lateral normal stress along height of the wall (Negative values indicate compressive behavior)

The changes in earth pressures (stress), from the initial rest position, due to dynamic event have been shown in Fig. 5. The change in earth pressures due to backfill were measured by observing the lateral normal stress on wall and have been presented in Fig. 5. These values are negative and show decrement in earth pressures as the excess earth pressures have been mobilized due to displacement and tilt of wall. Starting from bottom of wall, stresses developed after each earthquake have almost similar profile up to height 0.2 m and varied from 600 Pa to 800 Pa at this height. But from 0.2 m to top of the wall, normal stresses varied and high predominant frequency seismic event generated lower stress values after the earthquake. Again up to a height of 0.2 m, this change is having similar pattern for all events but above this height the change in stresses is very abrupt.

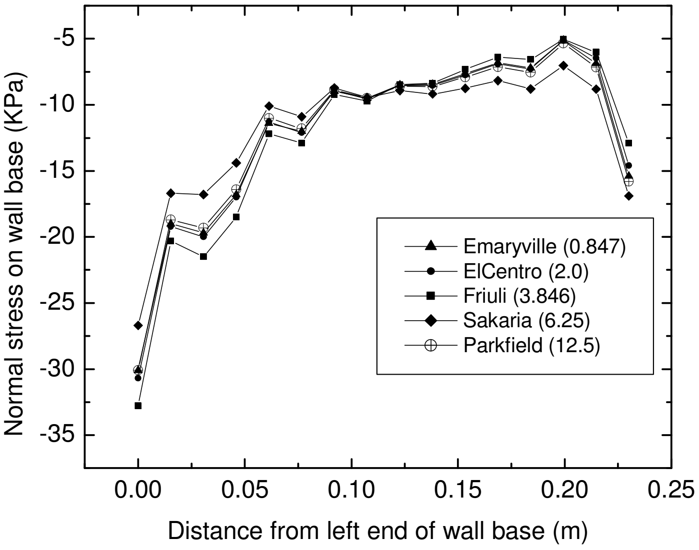

Figure 6 - Normal stress distribution on wall base after earthquakes

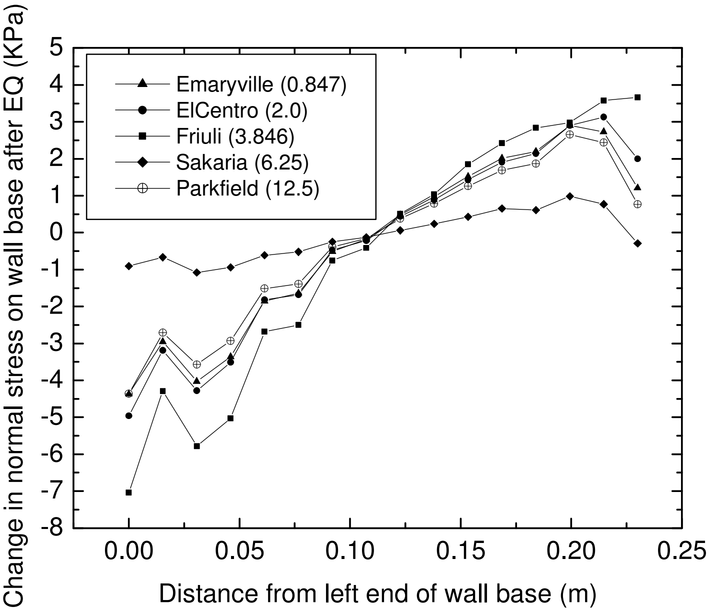

Figure 7 - Change in normal stress distribution on wall base after earthquakes

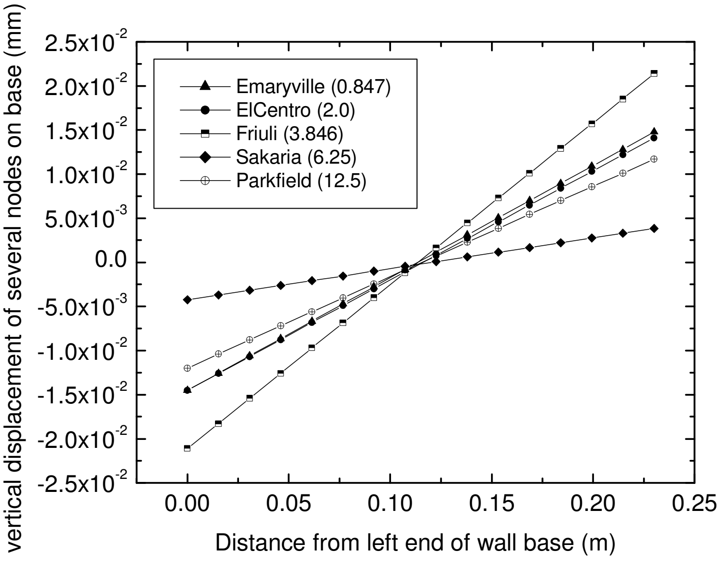

Figure 8 – Vertical displacement of wall base for different earthquakes

Normal stresses on the base of the wall due to seismic events of different predominant frequencies have been shown in Fig. 6., indicating the stress on the base-toe side is higher than the stress on base heel side. Fig. 7 shows a better prediction of stress increments on base due to different seismic events. Negative change in stress values show increment according to the principal axes of the FE model. For different predominant frequency events, stresses on wall base increased from an average value of 1 kPa to 6.5 kPa. The change pattern clearly shows that the wall was not tilted about the toe but it tilted about a point somewhere in between and the position of that point varied with the frequency of the seismic event. These values can further be verified by calculating tilt in wall base. Fig. 8 shows the vertical displacement of wall base for different earthquakes that can provide the amount of tilt in wall base which can be calculated by measuring displacement of various nodes on wall base along vertical direction.

Effect on residual wall top displacement

Residual relative wall top displacements for various seismic events were calculated by measuring the lateral displacement of wall top node relative to lateral displacement of wall base node. As shown in Fig. 9, for very low frequencies, the residual wall top displacement was significantly large (about 18 mm) and for very high frequency this value was quite low (about 9 mm). For middle ranged frequencies, the wall top displacement values were almost comparable.

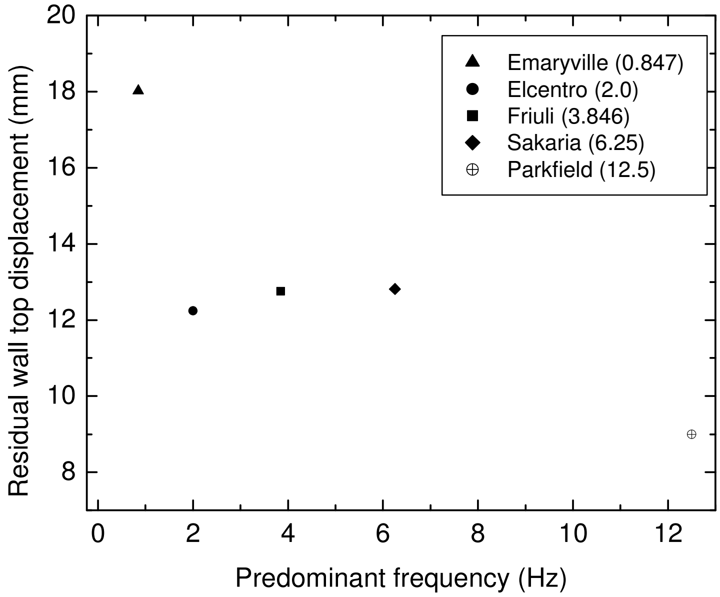

Figure 9 - Residual wall top displacement for different predominant frequencies

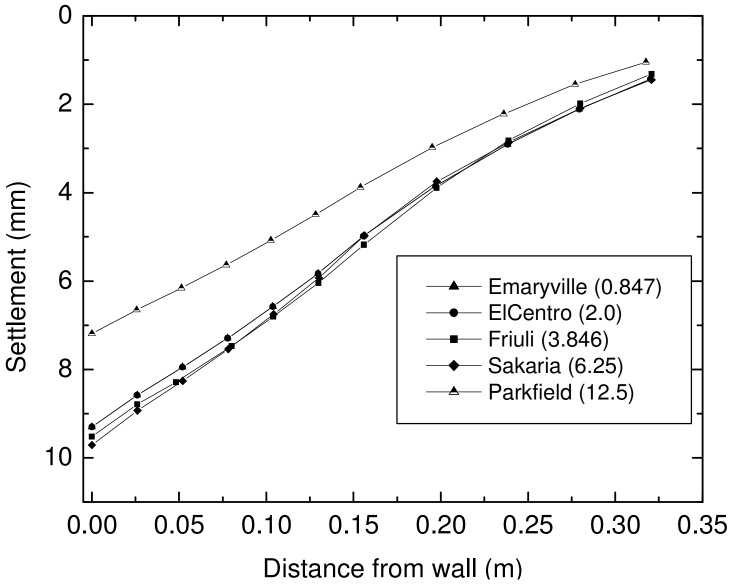

Figure 10- Far field settlement of soil for earthquakes of different predominant frequencies

Far field settlement of soil
The far-field settlement of soil was measured at different nodes on surface of soil which are depicted in Fig. 10. Soil near the wall settled more as compared to soil far away from the wall. After a significant distance the settlement of soil was diminished, which can be considered as the top width of the sliding soil wedge. The profiles of far-field settlement for low frequency values are comparable but at high frequency the settlement was less than those for low frequencies. All settlement profiles approached to zero settlement at almost same distance therefore the slope of settlement profile for high frequency values are less than that of low frequency settlement profiles. Fig. 11 shows the nodal displacement vectors and displacement contours of the model wall after the excitation.

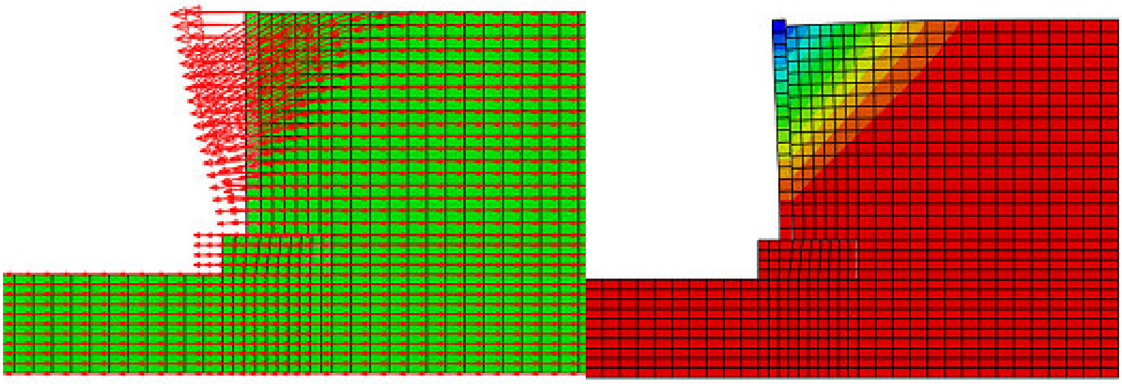

Figure 11 - Lateral displacements of soil and wall nodes during a seismic event. (a) Nodal displacement vectors, (b) Displacement contours

Stresses produced in the wall
Earth pressures developed during seismic events were balanced by the cantilever action of the wall. This action produces high stress concentration near the joint of wall base and stem making it vulnerable to failure (Fig. 12). Mises equivalent stress (*S, Mises*) developed in the wall near wall base-stem joint during this study have been compared in Fig. 13.

Mises equivalent stress (*S, Mises*) is an invariant calculated by Abaqus to represent deviatoric stress developed in the analysis and is given by Eqn.1(Abaqus 2008).

$$q\ (S, Mises) = \sqrt{\frac{3}{2}\,(\,S : S\,)} \qquad (1)$$

where *S* is deviatoric stress. Stresses developed for low predominant frequency earthquake are higher than those of higher frequencies. As the predominant frequency increases the stresses produced on wall base-stem joint reduce (Fig. 13) which supports the behavior shown by the finite element model for residual wall top displacement (Fig. 9), far field settlement of soil (Fig 10) and normal stresses developed on the wall and base.

CONCLUSIONS

The wall-soil system tested by Watanabe et al. (2003) was simulated into finite element model to study dynamic earth pressures developed during different seismic events. Primary results produced by the FE model were comparable to the physical model results and for static conditions the results were approximately matching with the theoretical values. After discussing the results, following conclusions were drawn.

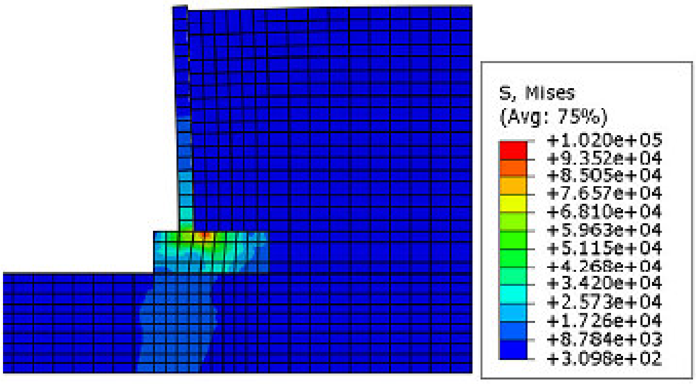

Figure 12 - Mises equivalent stress contours developed after Friuli earthquake

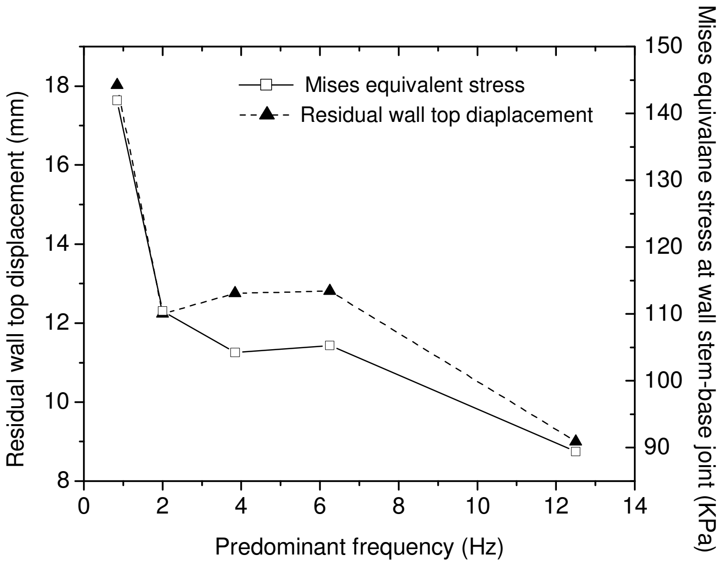

Figure 13 - Variation of Mises equivalent stresses developed at wall stem-base joint and residual wall top displacement with predominant frequencies of earthquakes

1. The lateral earth pressure profile for different predominant frequencies was similar up to a certain height and then varied abruptly. This can be explained by the formation of the sliding soil wedge at the backfill.
2. The vertical stresses at wall base increased on the toe side and consequently reduced towards the heel side due to tilting of wall base caused by lateral seismic loads on wall stem.
3. The residual wall top displacement was high for very low frequency (0.847 Hz) seismic loads and low for high frequency (12.5 Hz) seismic loads and showed no significant variation for middle ranged (2.0, 3.846 and 6.25 Hz) frequency earthquakes.

4. Soil near the wall settled due to shaking of wall forming the sliding soil wedge near the wall and this settlement reduced on going away from the wall. Far-field settlement was almost same for all but for very high seismic loads it reduced. It followed the residual wall top displacement profile of the wall. As for very high frequency the wall top displacement was very less, the soil near the wall was less vulnerable to settlement.
5. Stresses developed at the stem-base joint of the wall varied significantly with the predominant frequency of the seismic events. Due to cantilever action of the wall, earth pressures acting on wall base and stem caused the wall stem-base joint to develop tensile stresses. The variation of this stress followed the residual wall top displacement profile. For very high predominant frequency (12.5 Hz) seismic loads, the wall top displacement and the stresses developed in the wall stem-base joint were high and for very low frequency (0.847) seismic loads both the parameter were quite low.

The results presented in this paper are based on the finite element simulations on the small physical model size retaining wall model. Further simulations are needed to be performed on full size model wall with more realistic stress levels.

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