SEISMIC RESPONSE OF NON-DISPLACING BASEMENT WALLS: NUMERICAL VERIFICATION OF CENTRIFUGE EXPERIMENTS

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ABSTRACT

A numerical analysis of the seismic response of two non-displacing basement walls, stiff and flexible, is performed. The walls are founded on a uniform layered dry medium-dense sand stratum. A nonlinear inelastic constitutive model with kinematic hardening and an associative flow rule is applied to capture the soil behavior. Particular attention is paid to the wall-soil interface as it is properly modeled to allow separation and sliding. What is more, to avoid the box effect and therefore simulate realistically the radiation damping, vertical free-field columns (which are connected with the main soil domain through normal and shear dampers) are introduced on each horizontal side of the model to secure the undisturbed transmission of up-coming seismic waves.

The results are compared with the centrifuge results of the first experiment of Mikola and Sitar (2013). To investigate thoroughly the dynamics of the system, series of parametric analyses are performed. Both the test and our analyses utilize several near-fault ground motions recorded in Kocaeli 1999, Kobe 1995 and Loma Prieta 1989 earthquakes. Results are presented in terms of acceleration time histories, acceleration, velocity and displacement spectra, dimensionless wall moment distributions, and dynamic soil pressures. Comparison with Mononobe-Okabe method and Veletsos & Younan analytical solution is performed as well, and practical conclusions are drawn.

Keywords: Non-displacing walls; Numerical Verification; Centrifuge; Earthquake Motions; Dynamic Pressures

1. INTRODUCTION

In state-of-practice, a non-displacing basement wall is formed by a vertical part connected to a slab foundation and being secured by using struts, while its stability mechanism is based on the action of the backfill soil. A special type of basement walls is the non-displacing, which restrain the retained earth by the passive resistance provided by the opposite wall. Usually, the adjacent vertical parts are braced with struts. The major advantage of a non-displacing basement wall is their robust construction, so they are recommended to use next to adjacent buildings because horizontal displacements are (relatively) small. However, the control of lateral wall displacement is the major design objective. The magnitude of horizontal wall deflections depends on the passive earth resistance mobilization and on the stiffness of the struts that are used.

1.1 Non-Displacing Basement Walls

Basement walls, in practice, are designed with limit equilibrium methods. The Mononobe-Okabe method (1926), an extension of Coulomb’s method, is the earliest and most widely used analytical

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method. It gives the total active thrust acting on the wall by applying a pseudo-static inertial force on the soil wedge. Despite its known drawbacks, the classic pseudo-static Mononobe-Okabe (M-O) formula is still the main method proposed for the analysis of such walls. Since then, numerous analytical, experimental, and numerical studies have been published for the dynamic behavior of retaining walls. The M-O method had been modified and simplified by Seed and Whitman (1970). Richards and Elms (1979) determined permanent inelastic outward displacements, and Nadim and Whitman (1983) permanent sliding and rotation using the Newmark sliding block concept. Veletsos and Younan (1994) modelled the soil as an elastic medium and obtained elastodynamic solutions. Several other studies have also appeared, among which Al-Homoud and Whitman (1999), Wu and Prakash (1999), Green and Ebeling (2002), Cameron and Green (2004), Gazetas et al. (2004), Huang (2005), Psarropoulos et al. (2005), and Dakoulas and Gazetas (2008). In parallel, a significant effort was made in numerical study of seismic earth pressures in centrifuge experiments, as referred, by Ortiz et al. (1983), Cai and Bathurst (1995), Zeng (1998), Theodorakopoulos et al. (2001), Nakamura (2006), Madabhushi and Zeng (2007), Al Atik and Sitar (2010), and most recently by Mikola and Sitar (2013).

1.2 Scope

The scope of this paper is to shed more light into fundamental aspects of seismic response of non-displacing basement walls subjected to near-fault ground shaking, by numerical verification of the centrifuge experiments of Mikola and Sitar (2013); experiments which will be presented in the following section.

2. THE CENTRIFUGE EXPERIMENT

Two centrifuge experiments were performed by Mikola and Sitar (2013) on the dynamic centrifuge at the Center for Geotechnical Modeling at the University of California, Davis. The centrifuge has a radius of 9.1 m, and an available bucket area of 4 m$^2$ as pictured in Figure 1.

![Figure 1](image1.png)

Figure 1. The large centrifuge payload bucket at the Center for Geotechnical Modeling at U.C. Davis.

The shaking table can operate up to a maximum centrifugal acceleration of 75 g. For the particular experiments, the centrifugal acceleration used in was 36 g. The first centrifuge experiment, named ROOZ01, was performed on uniform dense sand, whereas the second centrifuge experiment, ROOZ02, on a two-layer sand model. Centrifuge data were taken from NESShub, but nowadays are accessible from DataCenterHub of Purdue University (https://datacenterhub.org/dv_dibbs/view/1316:dibbs/experiments_dv/?filter=experiments.title|Experiment-1:%20Seismic%20earth%20pressures%20on%20braced%20wall%20in%20sand%20(Rooz01)).

In the two previous studies, the results of the displacing cantilever wall of the second experiment (Garini et al. 2016) and the results of the U-shaped cantilever retaining wall (Tsantilas et al. 2017) were verified numerically. In this study, it is interesting to verify numerically the results of the non-displacing basement walls of the first experiment, as depicted in Figure 2.
The ROOZ01 model consisted of two retaining walls, a stiff basement wall and a flexible one. The structures were founded on approximately 12.5 m of dry medium-dense sand (Dr = 75%) and support a dry medium-dense sand backfill (Dr = 75%) as can be seen in Figure 3.

The model soil was dry Nevada Sand. Retaining structures were constructed of T6061 aluminum plate. The non-displacing basement walls were constructed by three plates: two parallel plates which were bolted to a base plate, and they were braced with six struts in two different heights. Geometry and dimensions of these retaining structures in prototype scale is pictured in Figure 3. Twelve shaking events were applied to the ROOZ01 model in flight. The excitations examined herein, are: two shaking events from the Yarimca recorded ground motion during the 1999 Kocaeli earthquake, the Takatori 090 component from the 1995 Kobe earthquake, and one shaking event from the Santa Cruz shaking by the 1989 Loma Prieta excitation. Three of them are presented in Figure 4. Further information for the examined excitations can be found in the report of Mikola and Sitar (2013).

Six types of electronic transducers were employed for measuring in the experiment: accelerometers, strain gages, pressure transducers, linear potentiometers, linear variable differential transformer, and load cells. With these devices were evaluated the acceleration on the retaining wall and backfill soil, the bending strains and deflections, the backfill settlements, the lateral earth pressures acting on the
non-displacing basement walls and the axial force in horizontal struts.

![Figure 4. Acceleration time histories of the examined excitations [(a) Santa Cruz, (b) Yarimca 060, and (c) Yarimca 330].](image)

**3. VERIFICATION: FINITE ELEMENT MODEL AND MATERIALS**

A 2-D plane-strain finite element model was constructed using the ABAQUS finite element commercial code. The discretization consists of four-noded quadrilateral, plane-strain elements as shown in Figure 5. Interface between walls and soil appropriately modeled as tensionless but frictional; it is simulated with special elements that allow separation and sliding, the latter controlled by coefficient of friction $\mu$. To capture radiation damping, normal and shear viscous elements $\rho V_S$ and $\rho V_P$ (per unit area) are placed at the vertical boundaries between the soil domain and the vertical free-field columns, which are introduced on each side in order to have proper transmission of up-coming waves thus, avoiding the box effect.

![Figure 5. The ABAQUS finite element model configuration.](image)
The geometrical limits of the model are 36.4 m behind the outer wall plates of each basement wall. This dimension was selected after various sensitivity runs performed to determine the effect of the boundaries on the results. The soil properties are: ρ = 1.695 Mgr/m³, E_{max} = 396 MPa, φ = 35°, ψ = 5°. A small value of cohesion was adopted in order to eliminate problems that the zero cohesion of sand was creating at the very top of the soil, because of the relative small value of Young’s Modulus at this region. Some of the soil properties are taken equal to the initial input parameters for the UBCHYST soil properties in the numerical model of Mikola and Sitar (2013), and the rest of them are given specific values in order to simulate the physical properties of such soils and the different achieved density. The wall is modeled with solid elements and its behavior presumed to be elastic. The coefficient of friction is μ = 0.45 between the retaining walls and the soil.

Soil behavior is described by a refined soil model developed by Gerolymos et al. (2006) and Anastasopoulos et al. (2011), utilized through a subroutine attached to ABAQUS. It models the nonlinear soil inelasticity through a simple kinematic hardening with Von Mises failure criterion and an associative flow rule. The evolution law consists of two components: a nonlinear kinematic hardening component describing the translation of the yield surface in stress space, and an isotropic hardening component, which defines the size of the yield surface as a function of plastic deformation.

For the purposes of the present study, model parameters were systematically calibrated, according to the G-γ and ξ-γ curves employed by Mikola and Sitar (2013). A value of model's parameter, λ, was found to provide a reasonable fit to G-γ curves. The computed τ-γ (shear stress-shear strain) hysteresis loops (because of the adoption of the Masing criterion for loading-unloading) result in overestimating the hysteretic damping for large shear strain amplitudes (γ ≈ 10°). Details for the validation of the model can be found in the afore-cited references.

4. VERIFICATION RESULTS

The verification is performed in terms of acceleration time histories (and their corresponding acceleration and displacement elastic spectra), static soil pressure distribution along the depth of the south (left) and north (right) walls of the stiff basement retaining structure, and dynamic earth pressure distribution along the depth of the right stem of the stiff basement wall. All results are presented in terms of prototype units and they refer to characteristic points shown in Figures. The nomenclature of these points is the same with the one in the report of Mikola and Sitar (2013).

For the sake of brevity, only a minimum of all the parametric results are presented below. Figure 6 illustrates the acceleration time histories and their corresponding acceleration and displacement elastic spectra in points A37, A16 induced by the Santa Cruz excitation. The black solid line corresponds to centrifuge and the red solid line to our numerical analysis. The very good agreement between the experiment and the analytical response is evident. Not only the maxima are captured but also the smaller details of the motion. Also, the frequency content of the accelerograms is reproduced as well. The agreement of the frequency-amplitude is portrayed better in terms of the response spectra of Figure 6. Either at the wall point (A37) or the backfill soil (A16), numerical response is very close to the experimental one, and this is true not only for the Santa Cruz excitation but for Yarimca too, as it is depicted in Figure 7.

Figure 7 illustrates the acceleration time histories and their corresponding acceleration and displacement elastic spectra in points A38, A28 induced by the Yarimca 060 excitation. Either at the wall point (A38) or the backfill soil (A16), numerical response is very close to the experimental one. In terms of static earth pressure distribution, our numerical analysis results showed also good agreement with the experimental ones. In Figure 8 the normalized static normal soil pressures σ_{st}/γ H, along the left wall of the stiff structure, are depicted. The circles correspond to centrifuge results and the solid lines correspond to our analysis. With blue color are shown the results from the right (north) wall and with red color the results from the left (south) stem. Notice that our analysis with the constitutive soil model shows almost the same results with that from the experimental response.
The dynamic earth pressure distributions are presented in Figure 9, 10 and 11, where the theoretical pressure distributions using the Mononobe-Okabe (M-O) method was computed assuming \( k_h = 100\% \text{PGA} \). Notice that \( \Delta \sigma_{AE} \) is the horizontal seismic pressure minus the static component. Observe that in Figure 9 the true experimental response, with the light orange circles, shows a similar trend to the trend of dynamic pressures taken from the analysis of the constitutive soil model, whereas the interpreted dynamic pressure distribution is very close to the pseudo-static approach of the M-O method. Figure 10 and 11 depict also the aforementioned behavior, but in less observed manner.
Figure 6. Comparison of acceleration time histories, their respective acceleration and displacement spectra, at two characteristic points [Point A37 left, Point A16 right]. In black is pictured the centrifuge experiment results, and with red the analytical ones. [Excitation: Santa Cruz].
Figure 7. Comparison of acceleration time histories, their respective acceleration and displacement spectra, at two characteristic points [Point A38 left, Point A28 right]. In black is pictured the centrifuge experiment results, and with red the analytical ones. [Excitation: Yarimca 060].
Figure 8. Normalized depth of the wall versus normalized static normal soil pressure [Excitation: Santa Cruz].

Figure 9. Normalized depth of the wall versus normalized dynamic normal soil pressure [Excitation: Santa Cruz].
It has to be mentioned, that in the experiment, soil pressures, $p$, were recorded by earth pressure sensors and were filtered using a low-pass Butterworth filter to reduce noise. In the experiment report by Mikola and Sitar (2013) it is stated that the pressure transducers, employed to measure $p$, have a manufacturer stated frequency response up to 100 Hz, which is sufficient for static earth pressures but they have difficulties to capture dynamic earth pressures because centrifuge scaling requires a sensor with at least 500-700 Hz frequency sensitivity to fully record dynamic earth pressures. That is the reason why dynamic earth pressures were interpreted (according to Mikola and Sitar (2013) report) from the load cells as well as strain gage measurements. Maybe this is a reason of the experimental versus numerical soil pressures difference.

It is important to note, that all of the data represent the earth pressures and earth pressure distributions at the point of maximum dynamic moment, which does not necessarily correspond to the maximum observed earth pressure, as it is also noted by Mikola and Sitar (2013).

The M-O method is great in comparing the behavior of gravity walls, in contrast with the Veletsos and Younan analytical solution that is the most appropriate method for comparing the results both of the specific experiment and of our analytical case. The aforementioned adequacy of the Veletsos and Younan analytical solution can be inferred by the below remarks.

Even though the experimental results of Mikola and Sitar (2013) show to converge adequately the M-O distribution, our results from F.E. analysis are in strict disagreement not only the values but even more significantly in the shape of pressures along the wall.

Notice in Figure 9, 10 and 11 that Veletsos and Younan (1994) pressure distribution (indicating the same pattern of wall deformation) is in full agreement with our results.
Figure 11. Normalized depth of the wall versus normalized dynamic normal soil pressure [Excitation: Yarimca 330].

5. CONCLUSIONS

The paper verified numerically the seismic response of non-displacing basement walls comparing them with experimental centrifuge results conducted by Mikola and Sitar (2013). 2-D analyses were conducted and the response of the two retaining walls was investigated for the Yarimca components (060 and 330) and Santa Cruz recorded ground motion. Agreement was obtained for detailed acceleration time histories at characteristic points of the wall and the backfill soil and for static soil pressure distribution. Yet, the dynamic soil pressure distributions presented differences between the analysis and the experiment, mainly due to the pressure transducers frequency limitation (this experimental shortcoming has been noted in the report of Mikola and Sitar (2013)).

6. REFERENCES


