Seismic Earth Pressures on Deep Stiff Walls

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Abstract
Experimental and numerical studies of the seismic response of a deep, stiff basement structure were motivated by the fact that the current seismic design methodologies based on the work of Wood (1973) and Ostadan (2005) predict very large dynamic forces in areas of high seismicity. The experimental program consisted of a geotechnical centrifuge model with a basement structure embedded in cohesionless backfill. The numerical analyses sought to replicate the results of the centrifuge experiment and to validate the use of numerical analyses for the prediction of expected behavior. Overall, the results of this study show that the Mononobe-Okabe method of analysis provides a reasonable estimate of the expected response of stiff basement structures provided depth-averaged design accelerations are considered.

Introduction
The introduction of more stringent seismic design provisions in recent updates of design codes, e.g. IBC 2012 and FEMA 750, has increased the demand on seismic design of retaining walls and basement structures and, hence, there is a need for appropriate analysis and design methodology. While not all codes are prescriptive in specifying a particular methodology, the most commonly recommended analyses for non-yielding or "rigid" walls (e.g., embedded structures and basement walls) are based on an elastic solution developed by Wood (1973). More recently, Ostadan (2005) proposed a simplified method that has the Mononobe-Okabe (M-O) method, based on work by Okabe (1924) and Mononobe & Matsuo (1929), as a lower bound and the Wood (1973) solution as an upper bound, which can be as much as 2 to 2.5 times greater than the M-O method. The principal problem for a designer is that at high design accelerations, > 0.5 g, these methods predict very large dynamic forces for non-yielding walls, which appear unrealistic in view of actual experience in recent earthquakes.

Sitar et al. (2012) present a detailed review of the different methods of analysis and their underlying assumptions and, therefore, they are addressed only briefly in the context of deep stiff walls. Recent experimental and numerical results are presented to show that the previously mentioned analysis methods do not adequately represent the actual seismic demand and that they are indeed conservative using current design recommendations.

Methods of Analysis and Design
The Ostadan (2005) "Rigid Wall Approach" (as defined in FEMA 750), is the latest recommended design methodology, and it depends on the characteristics of the ground motion, the backfill, and the embedded structure. The Mononobe-Okabe (M-

O) method or, particularly in the US, the Seed & Whitman (1970) simplified method provide a lower bound estimate of the Ostadan (2005) approach. Both methods assume a Coulomb wedge that behaves as a rigid body with no phase difference between the response of the soil and the structure. The M-O method assumes that the combined static and dynamic resultant force acts at 1/3H and is given by Equation 1

$$P_{ae} = 0.5\gamma H^2(1 - k_v)K_{ae} \quad (1)$$

where K_{ae} is given by Equation 2

$$K_{ae} = \frac{\cos^2(\phi - \theta - \beta)}{\cos(\theta)\cos^2(\beta)\cos(\delta + \beta + \theta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right]^2} \quad (2)$$

and γ is the unit weight of the soil, H is the height of the wall, ϕ is the angle of internal friction of the soil, δ is the angle of wall friction, β is the slope of the wall relative to the vertical, i is the slope of the backfill, $\theta = \tan^{-1}(k_h/(1 - k_v))$, k_h is the horizontal acceleration (in g), and k_v is the vertical acceleration (in g). Note that this formulation is valid for the particular case of cohesionless backfill soil ($c = 0$) and no surcharge ($q = 0$).

A major limitation of Equation 2 (and hence, Equation 1) is that it increases exponentially and does not converge if $\theta < \phi - \beta$, which for typical values of ϕ means that accelerations in excess of 0.7g. The Seed & Whitman (1970) method seeks to remedy this issue by separating the total force on the wall into static and dynamic components such that (Equation 3)

$$P_{ae} = P_a + P_{ae} = \frac{1}{2}\gamma H^2 K_a + \frac{1}{2}\gamma H^2 \Delta K_{ae} = \frac{1}{2}\gamma H^2(K_a + \Delta K_{ae}) \quad (3)$$

where K_a is Coulomb's coefficient of static earth pressure and $\Delta K_{ae} \approx 0.75k_h$ is the dynamic increment for a vertical wall ($\beta = 0$), horizontal backfill slope ($i = 0$), and $\phi = 35°$. Based on shaking table experiment by Matsuo (1941), Seed & Whitman (1970) further suggested that the dynamic load increment acts at a height 0.5H to 0.67H above the base of the retaining structure, which led to the "inverted triangle" interpretation of the dynamic earth pressure. Lastly, Seed & Whitman (1970) recommended that 85% of the PGA should be used in seismic design of retaining walls since the peak ground acceleration occurs only for an instant. The forces considered in the Seed & Whitman (1970) method, as well as the M-O method, are shown in Figure 1.

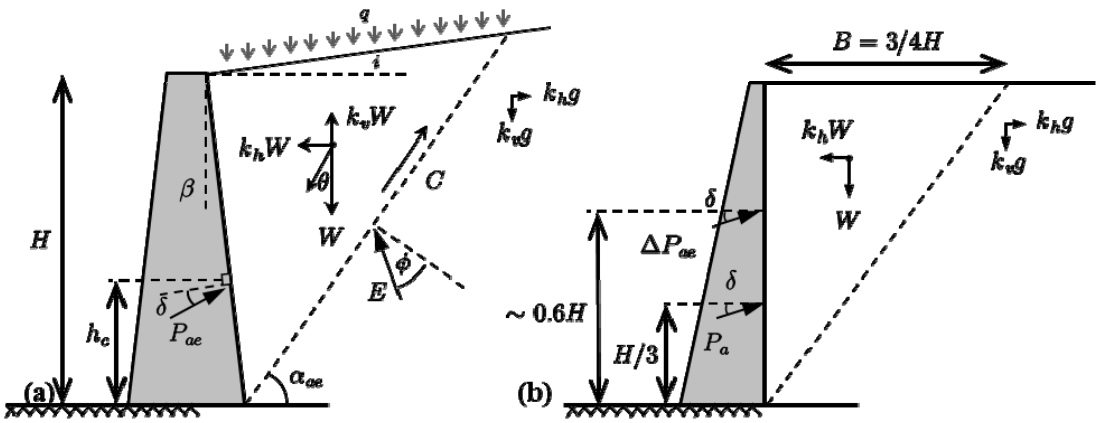

Figure 1: Force diagrams used in a) Okabe (1924) and b) Seed & Whitman (1970) (adapted from Candia & Sitar (2013))

Mononobe & Matsuo (1932) observed that stiffer structures rigidly attached at the base experience higher seismic loads by granular backfill. This problem was first addressed analytically by Wood (1973) who modeled linearly elastic soil in a container with rigid walls and a rigid base as shown in Figure 2a. As can be seen in Figure 2b, the computed dynamic stress increment is zero at the base and maximum at the top of the backfill with the recommended point of application of the resulting force at ~0.6H. The dynamic thrust, ΔP_E, for a uniform, constant seismic coefficient k_h applied throughout the backfill acting on a smooth rigid wall is shown in Equation 4, where Whitman (1991) concluded that the value of F is approximately equal to unity. The Wood (1973) solution provides an upper bound estimate of the Ostadan (2005) approach.

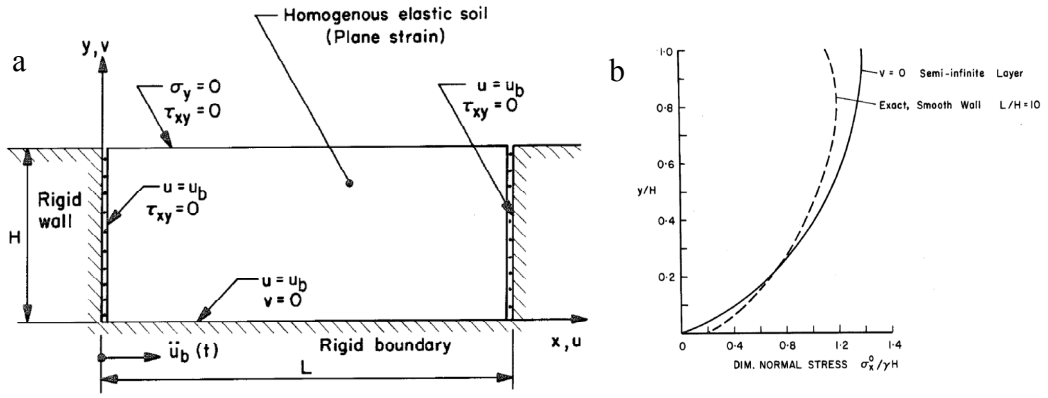

Figure 2: a) Geometry of the problem solved by Wood (1973); b) Computed dynamic stress increment

$$\Delta P_E = \Delta K_{ae} = F k_h \gamma H^2 \approx k_h \gamma H^2 \quad (4)$$

Experimental Study

While the field observations following earthquakes are very valuable, one of the main limitations is that most common information on the actual design and construction is

lacking. Hence, except in rare cases, e.g. Clough & Fragaszy (1977), a rigorous back analysis of the observed performance has not been possible. Therefore, scale model physical experiments are essential in order to be able to evaluate the validity of the various assumptions and the applicability of the various methods of analysis. To this end, the authors conducted a set of centrifuge experiments on a very stiff braced deep wall, 13.3 m deep and founded on 5.5 m of medium dense sand in prototype dimensions, using the facilities at the Center for Geotechnical Modeling at the University of California, Davis. The structure consisted of two thick walls with three levels of stiff cross braces, as shown in Figure 3. The bracing was instrumented with load cells in order to obtain a direct measurement of loads because the readily available earth pressure sensors, while providing satisfactory relative values, do not provide reliable absolute values. Consequently, the structure was very stiff, albeit not completely rigid. Other instrumentation included accelerometers and LVDT's to measure site response and to measure transient and permanent deformations. All tests were performed at 36g. Details of the experiment and the results are discussed in Wagner & Sitar (2013) and Sitar & Wagner (2015), respectively.

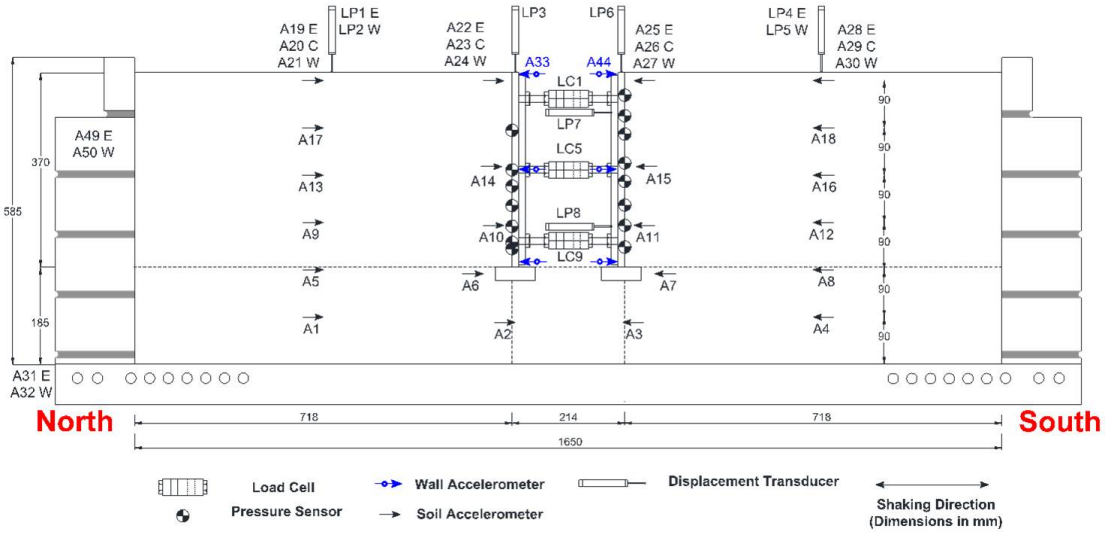

Figure 3: Layout of the centrifuge model of a deep stiff structure

Numerical Study

A numerical model was developed in FLAC$^{2\text{-}D}$ to simulate the centrifuge experiment in an attempt to replicate the response (Figure 4). The dimensions of the soil domain and the structure were the same as those of the prototype dimensions in the centrifuge experiment. The boundary conditions were specified as a rigid base and the sides of the model were attached to simulate the flexible shear beam container. Gravity was increased in stages to simulate the buildup of stresses as in the centrifuge experiment. The two-dimensional, total stress soil model UBCHyst (Naesgaard, 2011) was used to model the non-linear response of the soil. The shear modulus degradation characteristics of the UBCHyst soil model were calibrated to match Darendeli (2001) curves. Input ground motions were the same as those recorded at the base of the centrifuge experiment. Interface elements were used to model the connection between the soil grid and the structure.

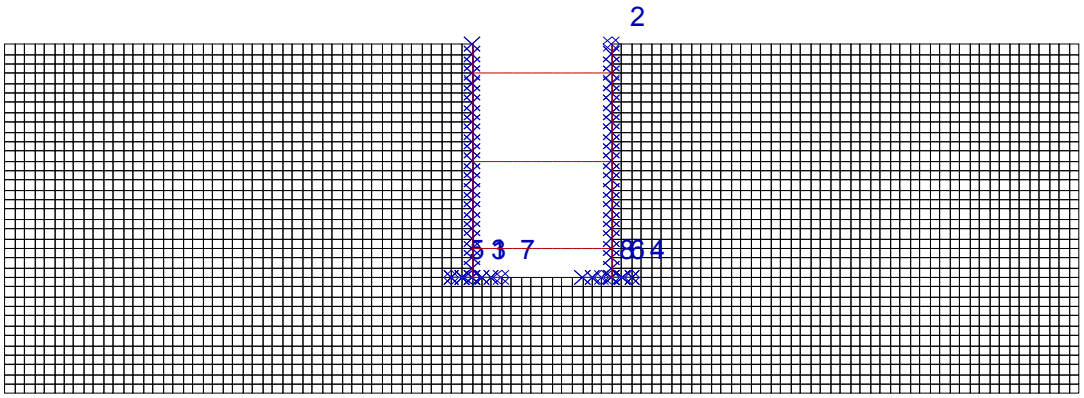

Figure 4: Two-dimensional finite difference mesh developed in FLAC

Results of the Experimental and Numerical Studies

The typical representation of the previously discussed conventional analysis procedures is to view the results in terms of the seismic coefficient ΔK_{ae}. Figure 5 is a summary of data obtained from the centrifuge experiment and numerical simulation, as well as results from previous centrifuge work by Mikola & Sitar (2013) and Candia & Sitar (2013) on shorter basement walls in cohesionless and cohesive backfill, respectively. The M-O, Seed & Whitman (1970), and Wood (1973) solutions are also shown for comparison.

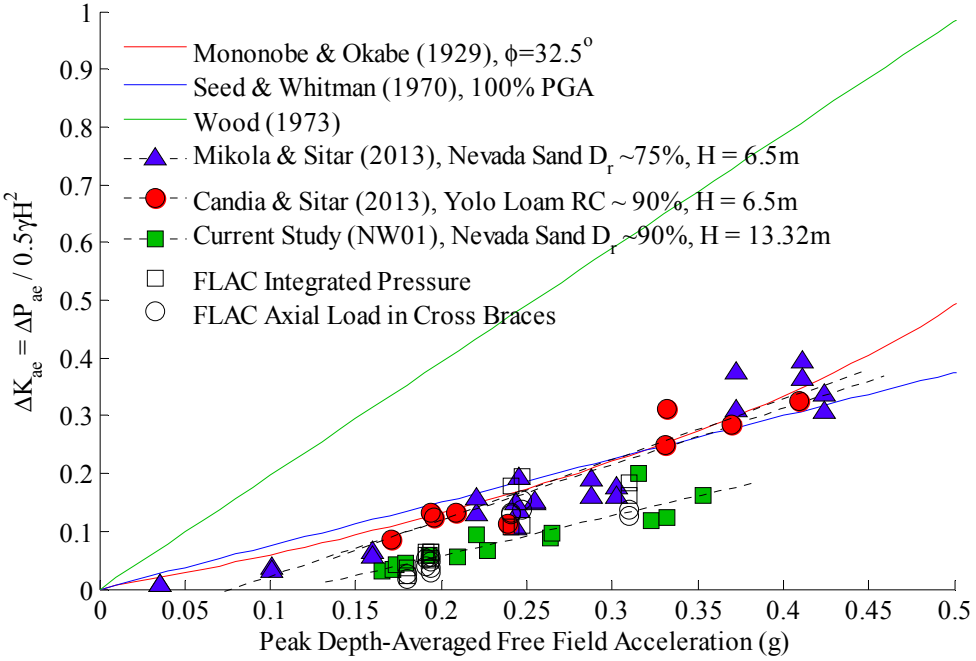

Figure 5: Seismic earth pressure coefficients from centrifuge model tests for stiff walls and numerical simulations for deep, stiff walls

Considering the original formulation of the M-O and Seed & Whitman (1970)

methods, it is apparent that an average acceleration is implied in the analysis since the seismic coefficient represents the entire rigid mass of soil in the Coulomb failure wedge.

Therefore, to maintain consistency with the predictive methods it seems appropriate to use an average acceleration measured over the depth of the assumed failure wedge. In our study the acceleration value is computed by taking the average of all accelerations measured throughout the depth of the basement structure in the free field at every instance in time, then computing the peak value of the new acceleration record. In this manner, the depth of embedment is implicitly included in the analysis as acceleration at depth is considered and the phase difference between the ground motion in the upper and lower parts of the backfill retained by the structure is included. Using this depth-averaged acceleration measure as a correlation parameter, the experimental and numerical results for shallow basement walls and deep, stiff basement walls agree much more closely with each other, with the M-O and Seed & Whitman (1970) methods providing a reasonable average estimate for the seismic earth pressure resultant.

An alternative method to evaluate the data is to consider the dynamic earth pressure distribution. The dynamic earth pressure increment envelopes interpreted from the load cells in the centrifuge experiment and the interface elements in the numerical model are shown for the Loma Prieta SC-1 (Figure 6) and Kobe TAK090-3 (Figure 7) ground motions. The predicted dynamic earth pressure using the M-O and the Seed & Whitman (1970) methods as well as the static at-rest earth pressure are shown for comparison. The dynamic increment for the M-O method was calculated by computing the total earth pressure (Equation 1) and subtracting the static component of the earth pressure. Note that for the Kobe TAK090 ground motion, the M-O method does not have a solution, as the horizontal seismic coefficient is too large. The value of friction angle was chosen based on the static at-rest pressure resultant measured in the load cells in the centrifuge experiment. The Wood (1973) solution would predict a dynamic earth pressure resultant over twice as large as the Seed & Whitman (1970) method. For the high intensity ground motions that are shown, the predicted dynamic earth pressure is nearly equal to the static at-rest earth pressure. This represents a significant fraction of the total load, assuming the total load to be the sum of static and dynamic components as in the Seed & Whitman (1970) method.

The results of the centrifuge experiments and the numerical analyses show that the dynamic earth pressure increments are essentially uniform with depth corresponding to $\Delta\sigma/\gamma H \approx 0.1$. The measured dynamic earth pressure increments are slightly larger than predicted by the M-O method at shallow depths (~0.1-0.2H), but significantly lower at depths greater than ~0.3H. In comparison, the Seed & Whitman (1970) "inverted triangle" significantly over predicts the dynamic earth pressure increment at shallow depths and the predicted pressure distribution does not match those observed in centrifuge experiments and numerical analyses. Additionally, the magnitude of the observed dynamic earth pressure resultant is considerably smaller than would be predicted when using either the M-O or Seed & Whitman (1970) methods with the recommended design accelerations.

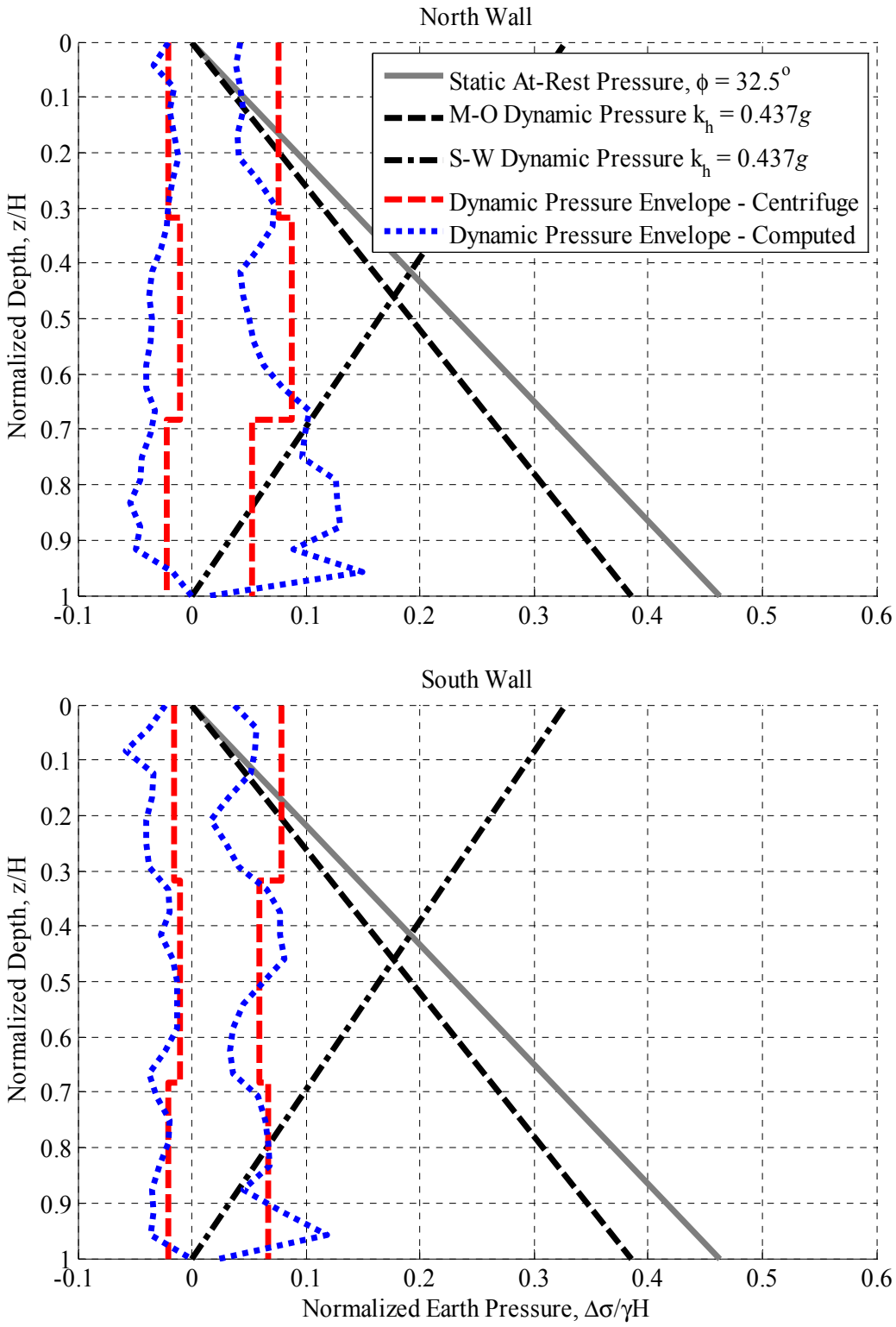

Figure 6: Dynamic earth pressure increment distribution for Loma Prieta SC-1 ground motion as a function of depth from centrifuge experiment and numerical simulation in FLAC on a deeply embedded structure (top) north wall (bottom) south wall

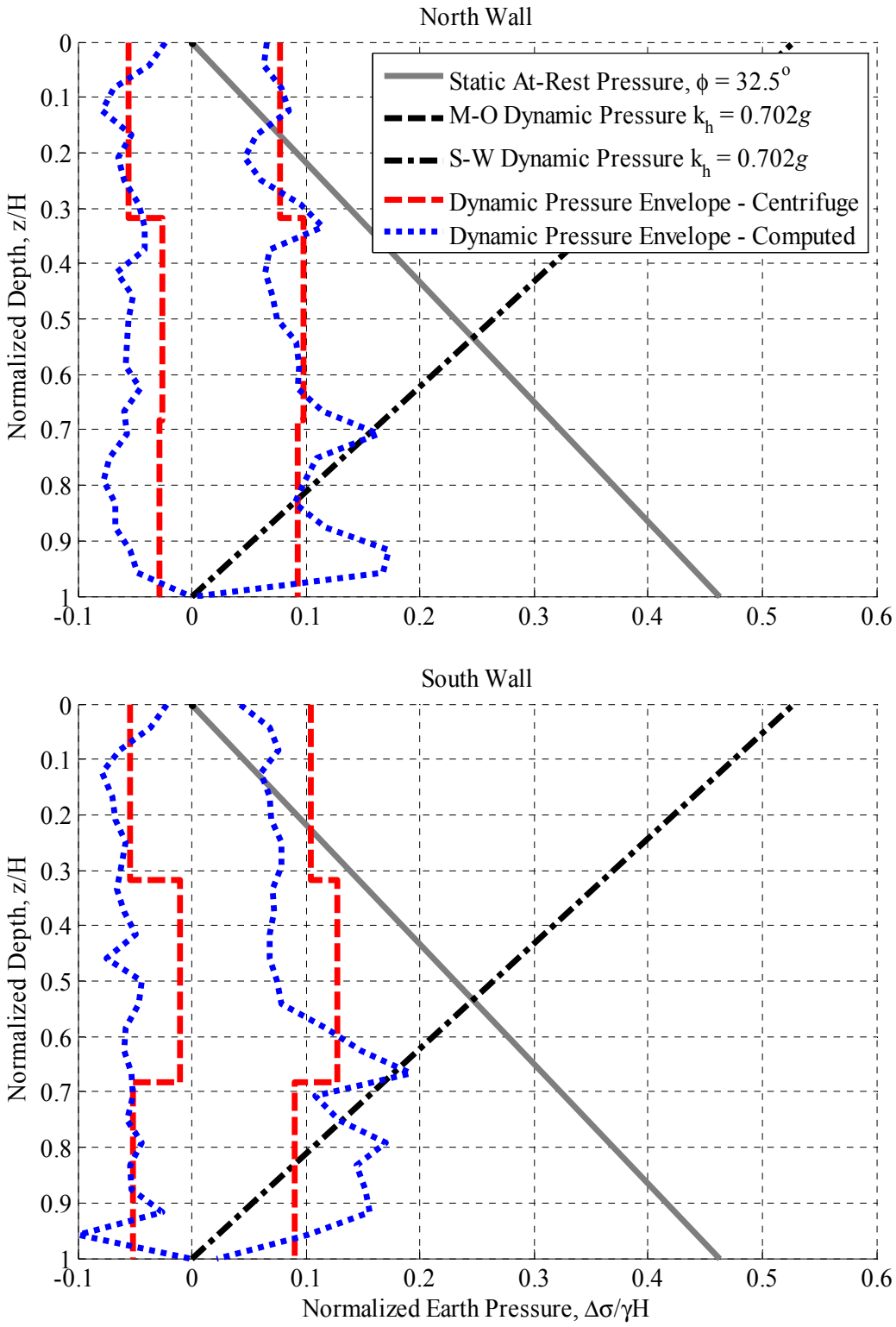

Figure 7: Dynamic earth pressure increment distribution for Kobe TAK090-3 ground motion as a function of depth from centrifuge experiment and numerical simulation in FLAC on a deeply embedded structure (top) north wall (bottom) south wall

Conclusions

A review of traditional methods of analysis shows that the selection of the acceleration measure is important for predicting the seismic loads on basement walls. Typically, the peak acceleration at the surface or the peak input acceleration is used as the design acceleration. This is a reasonable design choice for shallow retaining structures founded in shallow deposits as the phase lag between the top and bottom of the wall is essentially negligible and the amplification is not especially large. However, for deeper walls the differences in phase and amplification of the motion from the base of the structure to the surface deviate from the assumptions in traditional analyses. In fact, NCHRP Report 611 (2008) acknowledged the issue of depth dependency and proposed height-dependent seismic design coefficients for use in the Mononobe-Okabe method. Observations of seismic performance of conventional basement structures provide further support for these conclusions.

The experimental results show that the traditionally used Mononobe-Okabe and Seed & Whitman (1970) methods of analysis provide a reasonable average estimate for predicting seismic loads on retaining structures provided a depth-averaged acceleration measure is used as the seismic coefficient. On the other hand, there is no evidence to support the further use of the Wood (1973) solution and its derivatives. The results also show that seismic earth pressures increase only moderately with depth (if at all) and are a small fraction of the static pressure at depth. Further, the seismic earth pressure increment distributions assumed in the Mononobe-Okabe and Seed & Whitman (1970) methods do not match the observed uniform distribution for basement structures, as observed in the centrifuge experiment and the numerical simulation.

Acknowledgments

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