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Seismic Soil-Structure-Interaction and Lateral Earth Pressures on Buried Reservoir Structures

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ABSTRACT: This paper summarizes the results from a centrifuge experiment conducted at the University of Colorado at Boulder to evaluate seismic soil-structure-interaction, lateral seismic earth pressures, and dynamic response of equivalent model structures representing buried water reservoirs during a selected suite of input earthquake motions. This paper presents a summary of the design and planning, instrumentation challenges, and test results for the first baseline experiment in a series of centrifuge tests. The preliminary results indicate that underground structures similar in type to those tested in this study can experience seismic lateral earth pressures of engineering importance. The insight from these tests is useful for understanding the performance of underground reservoir structures worldwide and is applicable to an entire class of *unyielding* underground structures that are restrained at their roof and floor levels.

INTRODUCTION

The Los Angeles Department of Water and Power (LADWP) is currently replacing many of its open reservoirs with buried, reinforced-concrete structures to improve water quality. These underground structures in Southern California need to be designed to safely withstand seismic loading. The current state of practice for evaluating lateral seismic earth pressures on underground structures is based on simplified procedures or numerical tools that have not been calibrated or validated adequately against physical model studies. These buried reservoirs fit into a class of underground structures that have limited deformation and lateral movement capability and are restrained at the roof and floor levels, which does not match conditions from which common simplified procedures were derived.

A reliable evaluation of the seismic performance of relatively stiff underground structures that are restrained at the top and bottom presents a critical gap in the field of earthquake engineering. This typically leads to a conservative design that may unnecessarily increase construction cost, or an inadequate design followed by earthquake-induced damage. An example of earthquake-induced damage occurred during the 1971 San Fernando Earthquake where the walls of a large reinforced concrete underground reservoir at the Balboa Water treatment facility failed due to increased earth pressures (Wood, 1973).

Underground structures such as reservoirs, bunkers, box culverts, basement walls, and retaining walls don't experience free-vibration like building structures, because their movement is restricted by that of the surrounding soil. A key design factor for these structures is the magnitude and distribution of seismic lateral earth pressures and bending moments. The current state of practice for assessing seismic earth pressures on *yielding* or *displacing* underground structures relies heavily on methods proposed by Mononobe-Okabe (Okabe 1926; Mononobe and Matsua 1929) and Seed-Whitman (1970). Recently, a number of centrifuge experiments were performed by Al Atik (2008) and Mikola (2012) primarily on *yielding* retaining walls to provide additional insight on the seismic response of these underground structures. The response of relatively stiff, *unyielding* underground structures during earthquake loading has not been sufficiently evaluated experimentally in order to validate the numerical tools used in design.

The model structures tested represent prototype reinforced concrete buried reservoirs to be constructed in Southern California by LADWP. The proposed reservoirs include 35 to 40-ft high walls that will be buried and restrained against rotational movement at the top and bottom by the reservoir roof and floor, preventing excessive deformation.

This paper describes the first centrifuge experiment with the baseline model structure and presents some initial results in terms of dynamic earth pressures in comparison with the existing analytical methods commonly used in practice. This experiment serves as a fundamental study of the trends in static and dynamic lateral earth pressures acting on buried structures undergoing a range of earthquake motions.

BACKGROUND

The current state of practice for evaluating the seismic response of underground structures relies heavily on simplified procedures or numerical tools that have not been verified adequately against physical model studies or case histories, particularly for *unyielding* structures, leading to significant uncertainties. Hence, a series of dynamic centrifuge tests were planned to evaluate lateral seismic earth pressures and structural response for a range of reduced scale underground structures, with variations in the structural properties, ground motions, soil properties, and container boundary conditions. The physical model tests were designed to investigate a certain class of *unyielding* underground structures that have lateral translation and rotational restraints at the top and bottom from the roof and floor. These structures have limited movement but can flex and deform due to soil pressures and inertial forces relative to their structural stiffness.

The most common analytical methods used to evaluate dynamic lateral earth pressures on retaining structures may be categorized based on *yielding* and *unyielding* walls. Methods developed by Mononobe and Okabe (1926-1929) and Seed-Whitman (1970) were developed to evaluate dynamic earth pressures on *yielding* retaining structures. The Mononobe-Okabe (M-O) method is based on a Coulomb limit equilibrium earth pressure theory, except that it includes horizontal and vertical inertial forces due to seismic loading. They assumed that total (static and dynamic) lateral earth pressures increase with depth in a triangular fashion, and the resultant force is applied at 1/3H above the base, where H is the total height of the wall. A major assumption in this method is that the wall yields (or displaces) sufficiently to produce minimum active pressure condition. The Seed-Whitman (S-W) method is similar to M-O, but it separates the total lateral earth pressure coefficient, $K_{ae,}$ into an active static lateral earth pressure coefficient, K_a, and a dynamic earth pressure coefficient increment, ΔK_{ae}. Their suggested dynamic earth pressure profile is an inverted triangle with the resultant force applied at 0.6H above the base.

Analytical methods for *unyielding* walls (e.g., Woods 1973; Davis 2003) apply to underground structures that do not displace sufficiently to create active pressures that result from a limit-equilibrium soil condition. Wood (1973) is commonly used as an upper-bound estimate of the seismic earth pressure increments on *unyielding* underground structures. This method was developed for infinitely rigid restrained walls having a fixed base, with a linear elastic soil backfill. The method does not take into account wave propagation in its estimation of dynamic pressure. Hence, the Wood (1973) method is expected to be valid for cases where there is negligible dynamic amplification, which is when the frequency of motion is less than about half of the fundamental frequency of the backfill (Ebeling and Morrison, 1992). For walls with very long backfills, the dynamic thrust from Wood's method is applied at 0.63H above the base of the wall (Ebeling and Morrison, 1992).

The equivalent dynamic coefficient increment of lateral earth pressure, ΔK_e, as a function of the horizontal peak ground acceleration (PGA) at the base of the wall is shown in Figure 1 for the three methods discussed above. The ΔK_e estimated from Wood's method (i.e., for *unyielding* walls) is significantly greater than the M-O and S-W methods (i.e., for *yielding* walls), as expected. Recent tests done by Mikola (2012) for restrained basement walls, however, showed that the Seed and Whitman (1970) method provides a reasonable upper bound for the measured dynamic earth pressure increment in the centrifuge for PGAs ranging from 0.1 to 0.65g. Although the M-O, S-W, and Wood methods are often used in practice, their applicability for a wide range of structures, soil conditions, and ground motion characteristics (other than just the PGA) is not well understood; especially for the class of underground structures investigated in the current study.

EXPERIMENTAL SET-UP

The centrifuge testing program at the University of Colorado, Boulder consisted of a total of 116 shaking tests on 7 different model configurations. The information presented in this paper is for Test-1A. The instrumentation layout and test configuration for Test-1A is shown in Figure 2. The model was spun to 60 g of centrifugal acceleration calculated at the mid-depth of the soil profile. This test was conducted using a transparent flexible shear beam (FSB) type container developed by

Ghayoomi et al. (2012) to reduce boundary effects. All dimensions presented in this paper are in the prototype scale, unless stated otherwise.

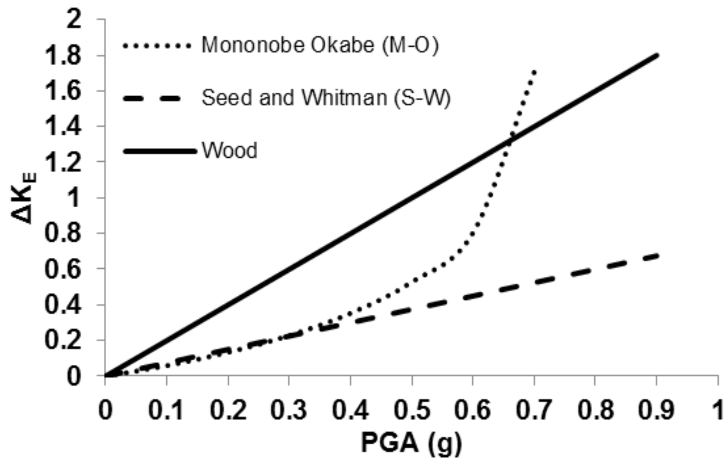

FIG. 1. Comparison of the dynamic coefficient increment (ΔK_e) of lateral earth pressure based on three methods: M-O, S-W, and Wood.

Data was acquired using accelerometers (A1-A16 in Figure 2), LVDTs (D1-D7), tactile pressure sensors, and strain gages. LVDTs were used to measure the settlement of soil and structure as well as the lateral displacement of structure and the FSB container frames. Eight strain gages were installed on each wall of the structure to measure bending strains. Two high speed tactile pressure sensors manufactured by Tekscan Inc. were used to measure total pressures on each side of the wall; one sensor to measure earth pressures on the upper half and the other for the lower half of the wall. Each tactile sensor used had 14 rows by 14 columns of sensels (measuring points) totalling to 196 sensels with dimensions of 5.1 mm by 5.1 mm each. Each of the 196 sensels recorded pressure data at a rate of 4000 samples/sec during the dynamic centrifuge tests. Older tactile pressure sensors with lower sampling rate capabilities were not able to capture the full amplitude of dynamic pressure especially at high frequencies, partially due to their low sampling rate causing signal aliasing and partially due to the frequency response of the sensor itself (Dashti et al. 2012). The tactile sensors used in this research had a sufficiently high sampling rate and were dynamically calibrated using a digital filter to recover the full amplitude of pressure at higher frequencies of interest in centrifuge. The sensors were also statically calibrated with the expected interface conditions (detailed by Gillis 2013).

This paper only reports results from accelerometers and tactile pressure sensors on the north wall of the structure. The remaining measurements will be further evaluated and reported in future work.

Soil Properties

Nevada Sand was chosen for use in the testing program, since it is a relatively well-characterized, uniform, fine, angular sand. A relative density (D_r) of approximately 60% was selected for testing. Table 2 summarizes the properties of Nevada Sand obtained at the University of Colorado before centrifuge testing.

Structure Properties

The model structure was placed in the middle of the FSB container and the ends spanned the width of the container. To simulate plain strain conditions, the friction between the structure and the FSB container were minimized by placing thin sheets of Teflon at the interface with the container. The structures were designed to have the stiffness, mass, and natural frequency of equivalent reinforced concrete reservoirs considered for design. No water was placed in the structure, representing an empty reservoir. Table 2 shows the prototype dimensions of the baseline, equivalent model structure used in the test. The structure was made by welding together pieces of AISI 1018, cold drawn steel with a mass density = 7870 kg/m^3, modulus of elasticity = 211 GPa, and Poisson's ratio (ν) = 0.29. The natural frequency of the structure ranged from 3.9 to 4 Hz obtained both experimentally and confirmed numerically.

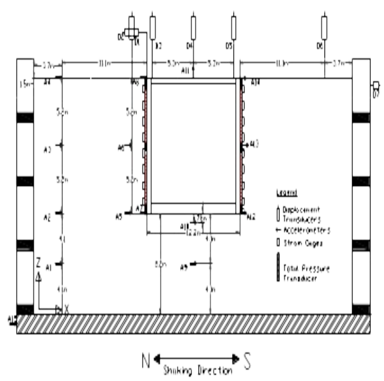

FIG. 2. Instrumentation layout for Test-1A (dimensions shown in prototype scale for a spin acceleration of 60 g). N and S represent North and South.

Table 1. Nevada Sand Properties.

Specific Gravity	2.65 (assumed)
Maximum Dry Unit Weight	16.39 kN/m^3
Minimum Void Ratio	0.586
Minimum Dry Unit Weight	14 kN/m^3
Maximum Void Ratio	0.852

Table 2. Prototype Dimensions of the Equivalent Model Structure.

Base Thickness	0.69 m	**Length**	17.45 m
Roof Thickness	0.37 m	**Width**	12.16 m
Wall Thickness	0.57 m	**Height**	10.44 m

PRELIMINARY EXPERIMENTAL RESULTS

Accelerations

The input ground motions were measured by A-15 located at the base of the FSB container (see Figure 2). The centrifuge testing program used a suite of recorded earthquake motions with a range of characteristics, in addition to sinusoidal motions applied to the base of the model container. However, only 1 record is evaluated as part of this paper, the Northridge-1 motion, the key properties of which are summarized in Table 3. The "free-field" ground motion amplification in the model away from the reservoir structure is shown in Figure 3 (see Ghayoomi et al. 2012 for the effects of boundary conditions on free-field motions). The ground motion amplification at the base and middle of the structure wall and the adjacent soil are shown in Figure 4. Typically, accelerations on the structural wall were greater than those of the adjacent soil.

The recorded base acceleration time history by A15 was filtered for each of the ground motions using a 5^{th} order, acausal, Butterworth low-pass filter with a prototype scale corner frequency of 18 Hz for noise reduction, as well as a 5^{th} order Butterworth high-pass filter with a prototype scale corner frequency of 0.1 Hz to remove the long period drift in the acceleration records that would appear in the computed velocity and displacement time histories after integration of accelerations.

Table 3. Properties of the Achieved (recorded) Base Earthquake Motion (units in prototype scale).

Ground Motion Name	PGA (g)	Arias Intensity I_a (m/s)	Significant Duration D_{5-95} (s)	Mean Period T_m (s)	Predominant Period T_p (s)
Northridge-1*	0.34	1.5	15.5	0.7	0.37

*Northridge: 1994 M 6.7 Northridge Earthquake, Sylmar Converter Station, NSC52 Record
An iterative procedure was used to obtain a reasonable match between the desired earthquake records and the achieved accelerations.

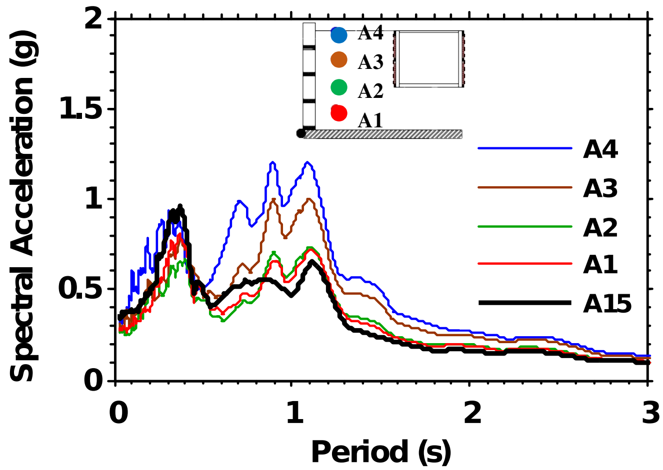

FIG 3. 5%-damped spectral accelerations in the free-field during the Northridge-1 event.

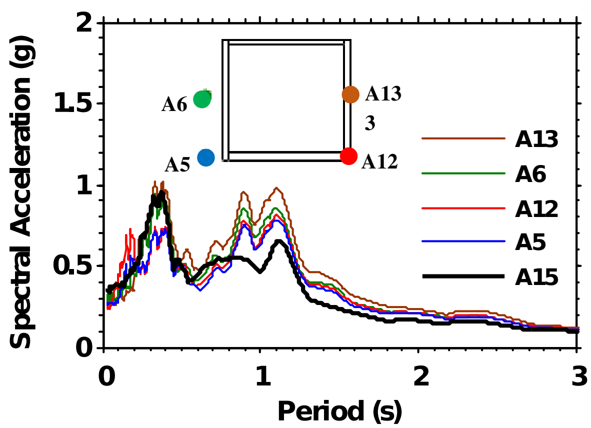

FIG. 4. 5%-damped spectral accelerations on and adjacent to the structure during the Northridge-1 event.

Static and Dynamic Earth Pressures

The pressure readings from the fourteen sensels in each row of the tactile sensors were averaged so that the data could be reduced to a total of 28 rows for easier and more reliable analysis (e.g., Gillis 2013). Rows 15, 16, and 28 from the bottom pressure sensor were excluded from the figures because they malfunctioned during the test. The at-rest lateral earth pressures recorded by the Tekscan sensors before and after each ground motion were compared to theoretical at-rest (K_o condition) and active (K_a condition) earth pressures as shown in Figure 5. The static, at-rest earth pressures were calculated for a backfill friction angle of approximately 35° (Arumoli, 1992) using Jaky's equation (1-sinϕ') (Jaky, 1944) for normally consolidated soils. A static pressure coefficient of K_o=0.6 was also plotted since it matches better with the tactile pressure data and is commonly used as a conservative estimate in design. The recorded at-rest earth pressures increased linearly on the upper half, but are more scattered with a general nonlinear increase in pressure towards the bottom of the wall.

During the dynamic portion of the centrifuge test, total (static + dynamic) lateral pressure time histories were recorded by the tactile pressure sensors. The total thrust acting on the wall at a given time was calculated by fitting the data corresponding to the total pressure profile recorded on the wall at that time with a fourth order polynomial (e.g., $\Delta\sigma_E = -0.0504(z/H)^4 + 0.9512(z/H)^3 - 6.813(z/H)^2 + 23.2284(z/H) - 5.6223$, obtained at 6.69 s) and integrating that profile over the wall height. The time history of the total, resultant force or thrust obtained in this manner during the Northridge-1 event is shown in Figure 6. The total earth pressure envelope acting on the wall at the time of peak resultant force (t = 6.69 s in this example) is shown in Figure 7. There is a steady increase in total pressure for the upper half of the wall which tends to reduce relative to the static envelope in the lower half of the wall.

The dynamic pressure increment was calculated by subtracting the static (before shake) pressure from the total pressure and plotted in Figure 8 at the time of maximum total resultant force and fitted with a fourth order polynomial line. The measured dynamic pressure increment is compared to the analytical methods of M-O, S-W, and Wood as shown in Figure 8. The analytical equations used in Figure 8 have not been repeated here due to space limitations, but can be found in the corresponding references. The horizontal seismic coefficient (kh) in the analytical methods was taken as 100% of the average PGA obtained by accelerometers A1 and A4 in the free-field. The dynamic coefficient of lateral earth pressure, ΔKae = Kae- Ka, used in the M-O and S-W methods, assumes the dynamic increment of pressure is added to static active pressure (for yielding structures). As shown in Figure 8, the measured dynamic pressure increments appeared to fall roughly between the lower-bound methods (M-O and S-W) and the upper bound method by Wood (L/H=1.5) for all wall heights.

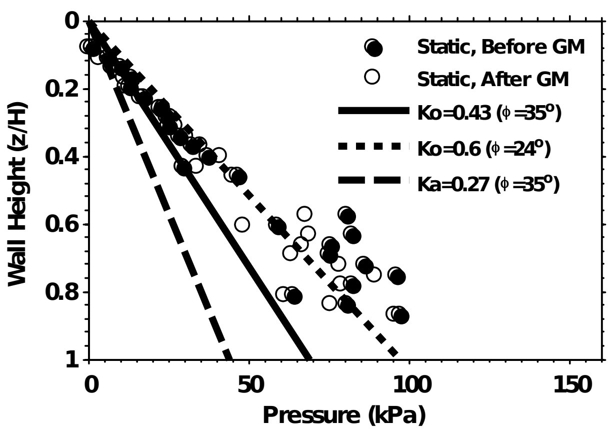

FIG 5. Static lateral earth pressure profiles measured by tactile pressure sensors before and after the Northridge-1 ground motion (GM).

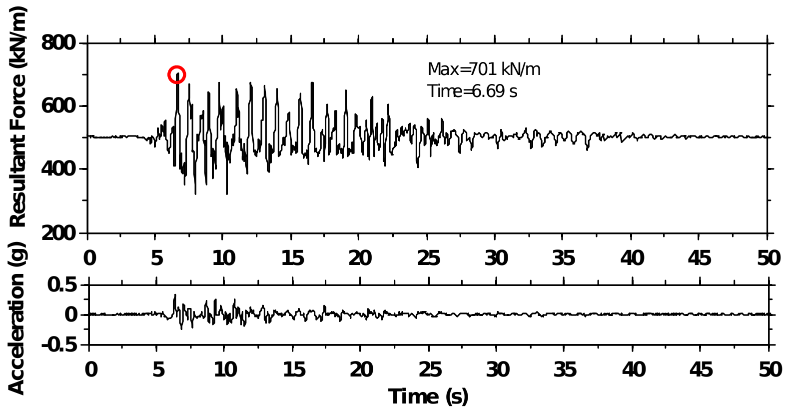

FIG. 6. The total resultant force time history acting on the reservoir wall during the Northridge-1 ground motion.

SUMMARY AND CONCLUSIONS

This paper presents some initial results of a centrifuge testing program to evaluate the fundamentals of the seismic response and lateral earth pressures acting on a relatively stiff, *unyielding* underground structure buried in a medium dense to dense, dry cohesionless soil shaken by earthquake ground motions. The experiments were conducted in an FSB container using Nevada Sand with an initial D_r = 60%. The models were heavily instrumented to measure total lateral earth pressures, bending strains, accelerations, and displacements at key locations with respect to the model structure. High-speed tactile pressures sensors were used to measure total earth pressures, which need to be handled with great care due to their fragility.

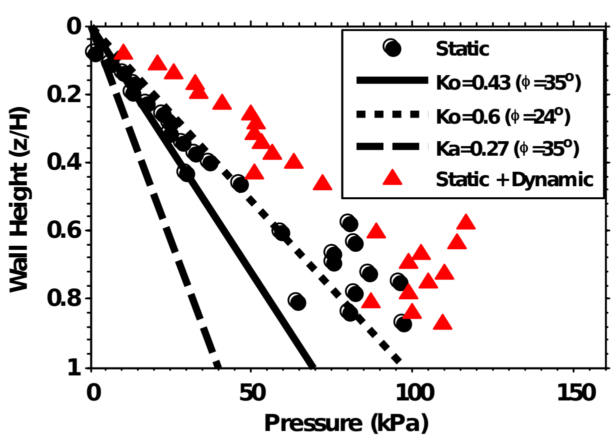

FIG. 7. Static and total pressure profiles measured by tactile pressure sensors at the time of maximum resultant force during the Northridge-1 Motion.

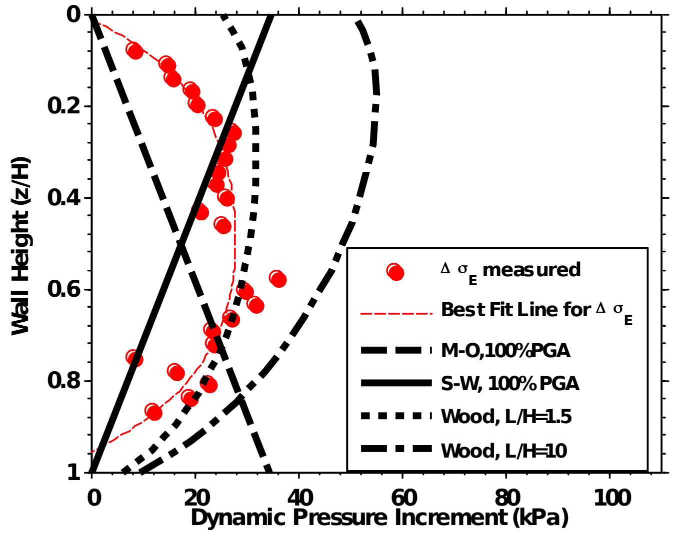

FIG. 8. The dynamic pressure increment ($\Delta\sigma_E$) profile measured by tactile pressure sensors on the reservoir wall during the Northridge-1 motion at the time of maximum resultant force, compared with analytical methods.

The static lateral earth pressure profiles matched well with theoretical methods for a 10.4 m high wall. It was also shown that the dynamic pressure increment ($\Delta\sigma_E$) profile roughly fell within the range of pressures predicted by the upper bound method proposed by Wood for *unyielding* structures and the lower bound methods proposed by M-O and S-W for *yielding* structures. This is an important finding for engineering practice because it shows how the infinitely rigid model of Wood (1973) over-estimates and the limit-state methods of S-W (1970) and M-O (1926-1929) under-estimate the measured pressures on the class of underground restrained structures tested in this study. These are preliminary test results that require further review and evaluation of the collected data from several experiments to confirm their reproducibility. The performed centrifuge experiments at the University of Colorado combined with parallel nonlinear numerical simulations will provide valuable insight into the influence of wall stiffness, cover soil, structure base fixity conditions, varying backfill type, and ground motion characteristics on the seismic response of *unyielding* underground structures having lateral translational and rotational restraints at the top and bottom.

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REFERENCES

Al Atik, L. (2008). Experimental and Analytical Evaluation of Seismic Earth Pressures on Cantilever Retaining Structures. *PhD Thesis*, University of California, Berkeley, CA.

Dashti, S., Gillis, K., Ghayoomi, M., and Hashash, Y. (2012). "Sensing of Lateral Seismic Earth Pressures in Geotechnical Centrifuge Modeling." 15th World Conf. on Earthquake Eng., Lisbon, Portugal.

Davis, C.A. (2003). "Lateral Seismic Pressures for Design of Rigid Underground Lifeline Structures." Proc. of the 6th U.S. Conference on Lifeline Earthquake Engineering. Monograph 25. ASCE. pp. 1001-1010.

Ebeling, M.E. and Morrison, E.M. (1992). The Seismic Design of Waterfront Retaining Structures. Technical Report ITL-92-11. U.S. Army Corps of Engineers, Washington, D.C.

Ghayoomi, M., Dashti, S., and McCartney, J.S. (2012). "Effect of Boundary Conditions on the Performance of a Transparent Flexible Shear Beam-Type Container." *2nd Int. Conf. on Perf.-Based Design EQ Geotech. Eng.*, Taormina, Italy.

Gillis, K. (2013). Tactile Pressure Sensor Calibration Methods and Data Analysis for Geotechnical Centrifuge Modeling. *MS Thesis*, University of Colorado, Boulder, CO.

Jaky, J. (1944), "The Coefficient of Earth Pressure at Rest", Journal for Society of Hungarian Architects and Engineers, October, pp. 355-358.

Mikola, R. (2012). *Seismic Earth Pressures on Retaining Structures and Basement Walls in Cohesionless Soils*. *PhD Thesis*, University of California, Berkeley, CA.

Mononobe, N. and Matsuo M. (1929). "On the Determination of Earth Pressures during Earthquakes." *Proc. World Engineering Congress*, Vol. 9, 179-187.

Okabe S. (1926). "General Theory of Earth Pressure," *Journal of the Japanese Society of Civil Engineers*, Tokyo, Japan, (12)

Seed, H.B. and Whitman, R.V. (1970). "Design of Earth Retaining Structures for Dynamic Loads." *ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures*, Cornell Univ., Ithaca, New York, 103–147.

Wood, J.H. (1973). *Earthquake Induced Soil Pressures on Structures*. *PhD Thesis*, California Institute of Technology, Pasadena, CA.