On seismic response of retaining structures

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ABSTRACT: Recent changes in codes and improved understanding of strong ground motions have led to increased demands in the seismic design of retaining structures. Methods for evaluating the seismically induced lateral earth pressures gradually evolved from the seminal Japanese work performed in the 1920's. The resulting design procedures suggest large dynamic loads indicating that older retaining structures may be significantly under-designed. However field evidence from recent major earthquakes fails to show any significant problems with the performance of retaining structures designed for static earth pressures only. Results of a series of centrifuge experiments performed by the authors indicate that seismically induced lateral earth pressures are significantly less than those estimated using the most current design methods. Specifically, the data show that the earth pressure distribution remains roughly triangular, increasing with depth, and the maximum dynamic moments on the retaining structure are to a large extent caused by the moment of inertia of the structures themselves. Most importantly, there is no evidence for the development of a failure wedge postulated in the M-O method of analysis and, hence, the basis for the continued use of the M-O method has to be reexamined in light of the preponderance of field evidence and experimental data.

1 INRODUCTION

The dynamic response of even the simplest type of retaining wall is a complex soil-structure interaction problem. Wall movements and dynamic earth pressures depend on the response of the soil underlying the wall, the response of the backfill, the inertial and flexural response of the wall itself, and the nature of input motions. The problem of seismically induced lateral earth pressures on retaining structures has received significant attention from researchers over the years. The pioneering work was performed in Japan following the Great Kanto Earthquake of 1923 by Okabe (1926) and Mononobe & Matsuo (1929). The method proposed by these authors and currently known as the Mononobe-Okabe (M-O) method is based on the Coulomb's theory of static earth pressures and is today, with its derivatives, the most commonly used approach to determine seismically induced lateral earth pressures. Later studies provided design methods mostly based on analytical solutions or experimental programs. While many studies have been conducted on the subject of seismic earth pressures over the last eighty years, to date, there seems to be no general agreement on a seismic design method for retaining structures or whether seismic provisions should be applied at all.

Given the importance of the seismically induced lateral earth pressures problem in the design of retaining structures in seismically active areas, an experimental study was undertaken aimed at improving our understanding of the seismic response of U-shaped cantilever walls retaining dry medium dense sand deposits. Herein we present a brief review of relevant existing studies on dynamic earth pressures including evidence from recent earthquakes highlighting the field performance of retaining walls. Observations from recently performed centrifuge tests are then presented to elucidate the factors controlling the seismic performance of cantilever retaining structures.

2 BACKGROUND

Since the pioneering work of Mononobe & Matsuo (1929) and analytical work of Okabe (1926), researchers have developed a variety of analytical and numerical models to predict the dynamic behavior of retaining walls or performed various types of experiments to study the mechanisms behind the development of seismic earth pressures on retaining walls.

2.1 *The Mononobe Okabe Method and its Derivatives*

The M-O method developed by Okabe (1926) and Mononobe & Matsuo (1929) is an extension of Coulomb's static earth pressure theory to include the inertial forces due to the horizontal and vertical backfill accelerations. The M-O method was developed for dry cohesionless backfill retained by a gravity wall and is mainly based on the following assumptions (Seed & Whitman 1970):

1 The wall yields sufficiently to produce minimum active pressure and the soil is assumed to satisfy the Mohr-Coulomb failure criterion;
2 When the minimum active pressure is attained, a soil wedge behind the wall is at the point of incipient failure, and the maximum shear strength is mobilized along the potential sliding surface; and
3 The soil wedge behaves as a rigid body, and accelerations are constant throughout the mass.

The theoretical underpinnings of the M-O method were developed by Okabe (1926). To validate the analytical method developed by Okabe (1926), Mononobe & Matsuo (1929) carried out experiments on dry, relatively loose sand in a rigid shaking table container in order to measure dynamic earth pressures on retaining walls. The Mononobe & Matsuo (1929) experimental configuration is presented in Figure 1. The experiments consisted of rigid base sand boxes with two vertical doors hinged at their base and hydraulic pressure gauges at their tops to measure the horizontal pressure exerted on the walls. The modeled walls were of 1.2 and 1.8 m height. The sand boxes were set on rollers and horizontal simple harmonic motion was imparted by means of a winch driven by an electric motor. Mononobe & Matsuo (1929) obtained experimental results consistent with the Okabe (1926) theoretical solution and their proposed seismic earth pressure theory became known as the M-O method.

While these experiments were very meticulous and pioneering in their scope, their applicability is limited by the fact that 1-g shaking table experiments on frictional material cannot be simply scaled to taller structures because of the stress dependency

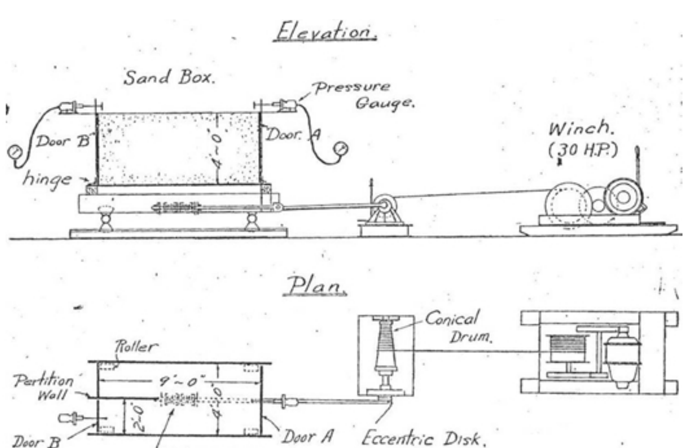

Figure 1. Mononobe & Matsuo (1929) experiments setup

of the material properties. Consequently, Mononobe and Matsuo's results are strictly correct only for the tested geometry and material, i.e. walls up to 1.8 m height with relatively loose granular backfill.

Results from various later experimental programs aimed at determining dynamic earth pressures on retaining walls have been similarly reported in the literature based on 1-g shaking table experiments. Results of such experiments generally suggested that the M-O method predicts reasonably well the total resultant thrust but that its point of application should be higher than one third the height of the wall above its base. However, as with the Mononobe & Matsuo (1929) experiments, the accuracy and usefulness of the 1-g shaking table experiments are limited due to the inability to replicate in-situ stress conditions, especially for granular backfills. More importantly, the observed amplification of ground motion and the observed increase in earth pressure upwards is the direct result of the physical layout of the geometry of the shaking table box and properties of the sand and not necessarily representative of field response.

2.2 *Analytical Methods*

Analytical solutions for the dynamic earth pressures problem can be divided into three broad categories depending on the magnitude of the anticipated wall deflection. These categories include rigid-plastic, elastic, and elasto-plastic methods. Relatively large wall deflections are usually assumed for rigid plastic methods while very small deflections are assumed for elastic methods. Elasto-plastic methods, appropriate for moderate wall deflections, are usually developed using finite element analysis.

Numerical modeling efforts have been applied to verify the seismic design methods in practice and to provide new insights into the problem. While elaborate finite element techniques and constitutive models are available in the literature to obtain the soil pressure for design, simple methods for quick prediction of the maximum soil pressure are rare. Moreover, while some of the numerical studies reproduced experimental data quite successfully; independent predictions of the performance of retaining walls are not available. Hence, the predictive capability of the various approaches is not clear. It is important to note that analytical methods for computing seismic earth pressures are usually based on idealized assumptions and simplifications that do not necessarily represent the real retaining structures-backfill seismic behavior. Therefore, such methods often result in overly conservative estimates of dynamic earth pressures.

2.3 *Dynamic Centrifuge Testing*

The basic principle behind centrifuge testing in geotechnical engineering is to create a stress field in a model that simulates prototype conditions. This allows the investigation of phenomena that otherwise would be possible only on full-scale prototypes. The major advantage of dynamic centrifuge modeling is that scaling is relatively straight forward and correct strength and stiffness can be readily reproduced for a variety of soils. Thus, in granular soils, for a reduced scale model with dimensions 1/N of the prototype and a gravitational acceleration during spinning that is N times the acceleration of gravity, the soil in the model will have same strength, stiffness, stress, and strain as the prototype. Thorough discussions of centrifuge scaling laws are given by Kutter (1995).

Dynamic centrifuge tests on model retaining walls with dry and saturated cohesionless backfills have been performed by Ortiz (1983), Bolton & Steedman (1985), Zeng (1990), Steedman & Zeng (1991), Stadler (1996), and Dewoolkar et al. (2001). The majority of these dynamic centrifuge experiments used sinusoidal input motions and pressure cells to measure earth pressures on the walls. While most of these researchers observed a general agreement between the maximum measured forces and the M-O predictions, the point of application of the dynamic thrust remained uncertain. Moreover, Stadler (1996) observed that the incremental dynamic lateral earth pressure profile ranges between triangular and rectangular and suggested that using a reduced acceleration coefficient of 20–70% of the original magnitude with the M-O method provides good agreement with the measured forces.

Nakamura (2006) performed a series of dynamic centrifuge experiments to study the seismic behavior of gravity retaining walls and investigate the accuracy of the M-O assumptions. His study presented invaluable insights into the seismic behavior of the gravity wall-backfill system. The configuration of the Nakamura (2006) centrifuge model is presented in Figure 2. Nakamura (2006) studied the displacement, acceleration and earth pressures responses in order to understand the seismic behavior of the wall/backfill system. His conclusions can be summarized as follows:

1. Contrary to the M-O rigid wedge assumption, the part of the backfill that follows the displacement of the retaining wall deforms plastically while sliding down;
2. While the M-O theory assumes that no phase difference occurs between the motion of the retaining wall and backfill, Nakamura (2006) experimentally observed that the acceleration is transmitted instantaneously through the retaining wall and then transmitted into the backfill; and
3. The M-O theory assumes that seismic earth pressures increase when the inertia force acts in the active direction on the wall/backfill system. In reality, dynamic earth pressures and inertia forces are not in phase. Dynamic earth pressure increment is around zero when the inertia force is maximum and vice versa.

3 FIELD PERFORMANCE

The performance of retaining structures and basement walls during earthquakes greatly depends on the presence of liquefaction-prone loose cohensionless backfills. Case histories from recent major earthquakes show that retaining structures supporting loose, saturated, liquefiable, cohesionless soils are quite vulnerable to strong seismic shaking. On the other hand, flexible retaining walls supporting dry cohesionless sands or cohesive, clayey soils have performed particularly well during earthquakes. It is important to note that many of these retaining structures were not designed for seismic

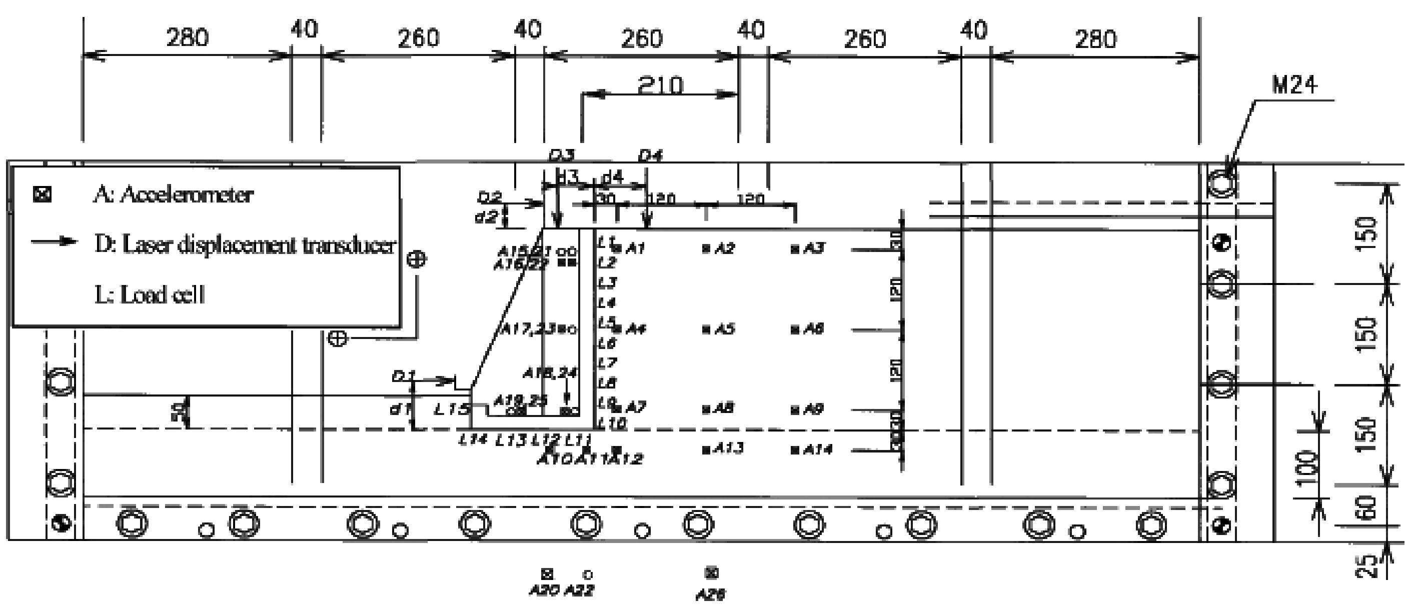

Figure 2. Nakamura (2006) centrifuge model, horizontal shaking direction, dimensions in mm

loading and others were designed for base accelerations not more than 20% of the peak accelerations that they actually experienced during the earthquake.

Clough & Fragaszy (1977) investigated the seismic performance of open channel floodway structures in the Greater Los Angeles area during the 1971 San Fernando Earthquake. The floodway structures studied consisted of open U-shaped channels with wall tops set flush to the ground surface as shown in Figure 3. The backfill soil consisted of dry medium-dense sand with an estimated friction angle of 35°. The structures were designed for a conventional Rankine static triangular earth pressure distribution, and no seismic provisions were applied in the design. The cantilever walls were damaged during the earthquake, with the typical mode of failure as shown in Figure 3.

Clough & Fragaszy (1977) performed pseudo-static analyses and shear wave propagation studies, and concluded that "conventional factors of safety used in design of retaining structures for static loadings provide a substantial strength reserve to resist seismic loadings. Peak accelerations of up to 0.5 g were sustained by the floodways with no damage even though no seismic loads were explicitly considered in the design." The relationship between wall damage and ground acceleration obtained by Clough & Fragaszy (1977) is shown in Figure 4.

During the 1994 magnitude 6.7 Northridge Earthquake, numerous "temporary" anchored walls were subjected to acceleration levels in excess of 0.2 g and in some cases as large as 0.6 g. Lew et al. (1995) described four such prestressed-anchored piled walls in the greater Los Angeles area with excavation depths of 15 to 25 m and supporting relatively stiff soils. The authors reported that the measured deflections of these walls did not exceed 1 cm and that no significant damage was observed.

During the 1995 magnitude 7 Kobe Earthquake in Japan, a wide variety of retaining structures most of them located along railway lines were put to test. Gravity-type retaining walls such as masonry, unreinforced concrete and leaning type were heavily damaged. On the other hand, reinforced-concrete walls experienced only limited damage. Koseki et al. (1998) presented preliminary evaluations of the internal and external stability of several damaged retaining walls during the Kobe earthquake. The aim of their study was to improve the current design procedures that are mostly based on the M-O theory. Koseki et al. (1998) concluded that a horizontal acceleration coefficient based on a reduced value of the measured peak horizontal acceleration (60 to 100% of peak ground acceleration) is appropriate for use with the M-O method.

During the 1999 magnitude 7.6 Chi-Chi Earthquake in Taiwan, flexible reinforced-concrete walls and reinforced-soil retaining walls performed

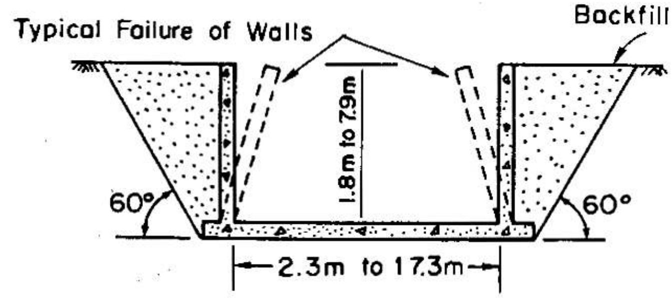

Figure 3. Section through open channel floodway and typical mode of failure due to earthquake shaking (Clough & Fragaszy 1977)

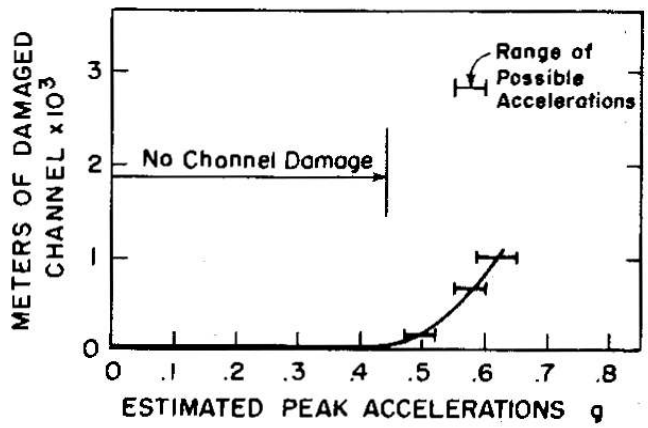

Figure 4. Relationship between channel damage and peak accelerations (Clough & Fragaszy 1977).

relatively well. Similarly, Gazetas et al. (2004) reported that during the 1999 magnitude 5.9 Athens Earthquake several metro stations were being constructed. Although the retaining structure of the Kerameikos metro station was not designed for seismic shaking, it was able to withstand nearly 0.5 g of peak ground acceleration during the earthquake with no visible damage. Maximum wall displacements were estimated to have been on the order of few centimeters.

Most recently, observations of the seismic response of retaining structures during the great 5-12 Sichuan Earthquake showed excellent performance of all types of retaining structures. Figure 5 shows a simple cobble and mortar retaining wall for an unfinished bridge abutment in Hanwang, which experienced very significant shaking that caused extensive damage to other types of structures in the vicinity. This type of retaining structure is extensively used throughout the region and, as far as it could be ascertained, none of the structures were designed for the severity of shaking that they experienced. Nevertheless, no evidence of significant damage was observed during post-earthquake reconnaissance.

Overall, the case histories show that retaining structures perform quite well under seismic loading, even if they were not specifically designed to handle dynamic loads. These observations clearly run

Figure 5. Unfinished cobble and concrete gravity retaining structure in Hanwang, Sichuan Province, China, following the 5-12 Sichuan Earthquake.

counter to the accepted theories of seismically induced earth pressures.

4 OBSERVATIONS FROM THE DYNAMIC CENTRIFUGE EXPERIMENTS

4.1 *Model Configuration*

In our study we performed a series of centrifuge experiments on the 400g-ton dynamic centrifuge at the Center for Geotechnical Modeling at the University of California, Davis. The centrifuge has a radius of 9.1m, a maximum payload of 4,500Kg, and an available bucket area of $4m^2$. The shaking table has a maximum payload mass of 2,700Kg and a maximum centrifugal acceleration of 80g. Additional technical specifications for the centrifuge and the shaking table are available in the literature (Kutter et al., 1994, Kutter, 1995). The centrifugal acceleration used in this experiment was 36g. All test results are presented in terms of prototype units unless otherwise stated.

The models for these experiments were constructed in a rectangular flexible shear beam container with internal dimensions of 1.65m long by 0.79m wide by approximately 0.58m deep. The model container is designed such that its natural frequency is less than the initial natural frequency of the soil in order to minimize boundary effects.

In prototype scale, the models consist of approximately 12.5m of dry medium dense sand overlaid by two U-shaped retaining wall structures, stiff and flexible, spanning the width of the container. Both structures support dry medium dense sand backfill. The first centrifuge experiment was performed on a two-layer sand model with sand backfill and foundation having relative density of 61% and 73%, respectively. The second centrifuge experiment was performed on a uniform density sand model with relative density of 72%. Dry pluviation was used to place the sand in different layers underneath and behind the structures. Lead was added to the structures in small pieces of 1 in^2 each in order to match the masses of the prototype reinforced concrete structures. Figure 6 presents the configuration of the second centrifuge model.

4.2 *Instrumentation*

The centrifuge models were densely instrumented in order to collect accurate and reliable measurements of accelerations, displacements, shear wave velocities, strains, bending moments and earth pressures. Horizontal and vertical accelerations in the soil and on the structures were measured using miniature ICP and MEMS accelerometers. Soil settlement and structures' deflection and settlement were measured at different locations using a combination of spring loaded LVDTs and linear potentiometers. Shear wave velocities in the soil underneath and behind the structures were measured using piezo-ceramic bender elements and mini-shear air hammers. The locations of accelerometers, bender elements, air hammers, and displacement transducers for the second centrifuge experiment are shown in Figure 6.

Accurate measurement of lateral earth pressure distribution was the major goal of this study. Lateral stress measurements in laboratory experiments are usually made using pressure cells. Such measurements are not always reliable due to possible cell/soil reaction, the relative stiffness of the cell with respect to surrounding soil and arching effects. Therefore, three different sets of independent instruments were used in this study. The lateral earth pressures were directly measured in the two centrifuge experiments using flexible tactile pressure Flexiforce sensors manufactured by Tekscan. Lateral earth pressures on the south stiff and north flexible walls were also calculated by double differentiating the bending moments measured by the strain gages mounted on the model walls. Finally, direct measurements of the total bending moments at the bases of the south stiff and north flexible walls were made using force-sensing bolts at the wall-foundation joints.

4.3 *Shaking Events*

Multiple shaking events covering a wide range of predominant periods and peak ground accelerations were applied to each model in flight at a centrifugal acceleration of 36g. The shaking was applied parallel to the long sides of the container. The shaking events consisted of step waves, ground motions recorded at the Santa Cruz (SC) and the Saratoga West Valley College (WVC) stations during the 1989 Loma Prieta Earthquake, ground motions recorded at the Port Island (PI) and Takatori (TAK) stations during the 1995 Kobe Earthquake, and ground

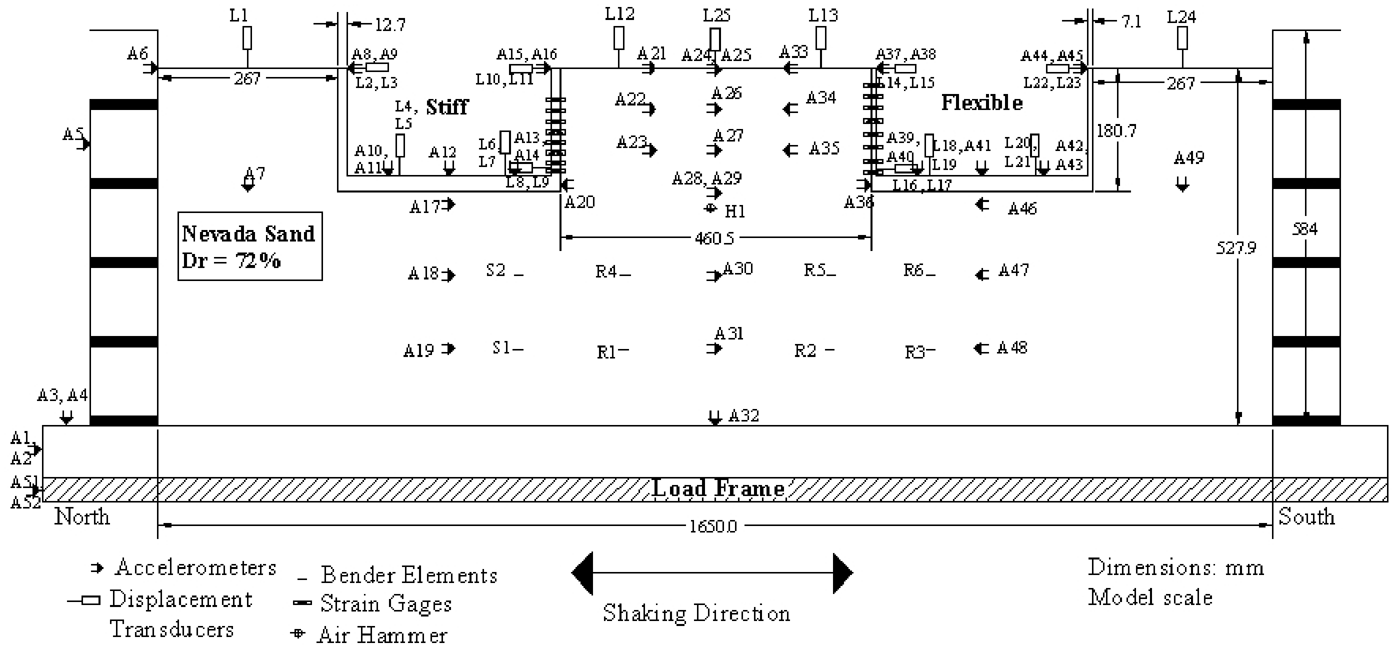

Figure 6. Model configuration for the second centrifuge experiment in profile view

motions recorded at the Yarmica (YPT) station during the 1999 Kocaeli Earthquake. The detailed description of the input ground motions can be found in Al Atik & Sitar (2007).

4.4 *Observations on the Seismic Response of the Retaining Wall-Backfill Systems*

Data collected during the centrifuge experiments were used to study the seismic behavior of the wall-backfill systems and to evaluate the hypothetical assumptions of the M-O theory. While the M-O theory assumes that the seismic response occurs simultaneously and uniformly in the backfill and the retaining walls, experimental results showed that the inertial force does not in fact occur at the same time in the backfill and the walls. Moreover, accelerations are not uniform in the backfill or on the walls.

The M-O theory further assumes that dynamic earth pressures and inertial forces simultaneously take their maximums and stability analyses of retaining walls are conducted for such case. Comparisons of the dynamic wall moments, dynamic earth pressures and inertial forces acting on the walls suggest that when the inertial force is at its local maximum, the dynamic wall moment (due to dynamic earth pressures and wall inertia) is at its local maximum too but the dynamic earth pressure increment is at its local minimum or around zero. On the other hand, when the dynamic earth pressure is at its local maximum, the inertial force and dynamic wall moment reach their local minimum values or zero. Figure 7 presents a close-up comparison of the dynamic wall moments (due to dynamic earth pressure and wall inertia) interpreted from a strain gage measurements to the dynamic earth pressures recorded by a Flexiforce sensor on the south stiff and north flexible walls during a Loma Prieta, Santa Cruz shaking event from the second centrifuge experiment. Figure 7 illustrates the out of phase relation between dynamic earth pressures and dynamic wall moments on the stiff and the flexible walls.

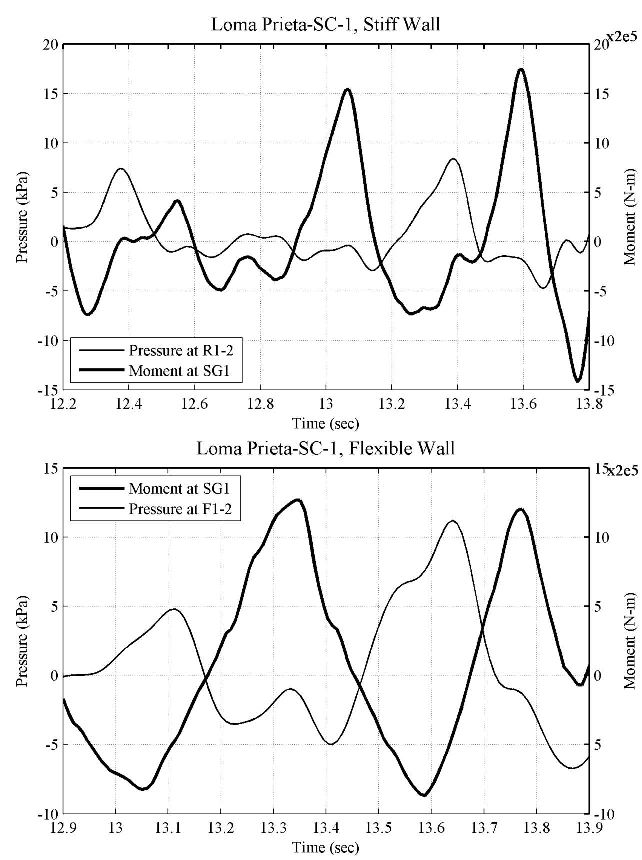

Figure 7. Comparison of dynamic wall moments and dynamic earth pressures on the south stiff and north flexible walls

4.5 *Total Earth Pressure Distribution and Moment Generation*

Figure 8 shows the maximum total earth pressure profiles recorded by the Flexiforce sensors and interpreted from the strain gage measurements from a centrifuge experiment on a stiff and a more flexible wall for the Loma Prieta-SC-2 and KocaeliYPT060-3 shaking events. Figure 8 also shows the total pressure distributions estimated by the M-O method using the measured peak ground accelerations at the top of the soil in the free field. As shown in Figure 8, results from the centrifuge experiments consistently demonstrate that the maximum dynamic earth pressure increases with depth and can be reasonably approximated by a triangular distribution analogous to that used to represent static earth pressures (Al Atik, 2008, Al Atik & Sitar, 2008a). The magnitude of seismic earth pressures depends on the magnitude and intensity of shaking, the density of the backfill soil, and the flexibility of the retaining walls. As can be seen from these results, the traditional M-O and the Seed & Whitman (1970) methods currently used in practice provide overly conservative estimates of the maximum induced seismic earth pressures (Al Atik, 2008, Al Atik & Sitar, 2008b).

While it is traditional in geotechnical practice to work in terms of earth pressures, it is the moment distribution and the moment at the base of the wall that is of paramount interest from structural design point of view. An important contribution to the overall dynamic wall moments is the mass of the wall itself. While Richards & Elms (1979 and 1980) make a strong case for the consideration of the inertial forces due to the mass of the retaining structure in the design of gravity walls, cantilever walls have not received similar attention. Results from the centrifuge experiments presented here show that the wall inertial moments contribution to the overall dynamic wall moments is substantial and should be accounted for separately. Moreover, wall inertial moments are generally in phase with dynamic wall moments. This suggests that dynamic wall moments are largely influenced by the inertia of the wall itself.

As shown in Figure 7, the moments generated by the earth pressure on the wall are out of phase with the dynamic wall moments. As a result, the current trend to design retaining walls for maximum dynamic earth pressures and maximum wall inertia is overly conservative and does not reflect the true seismic performance of the backfill-wall systems. Since wall inertial forces and dynamic earth pressures are not in phase, their cumulative effect results in reduced overall moments acting on the walls.

5 CONCLUSIONS

A review of the history of the development of methods for the estimation of seismically induced lateral

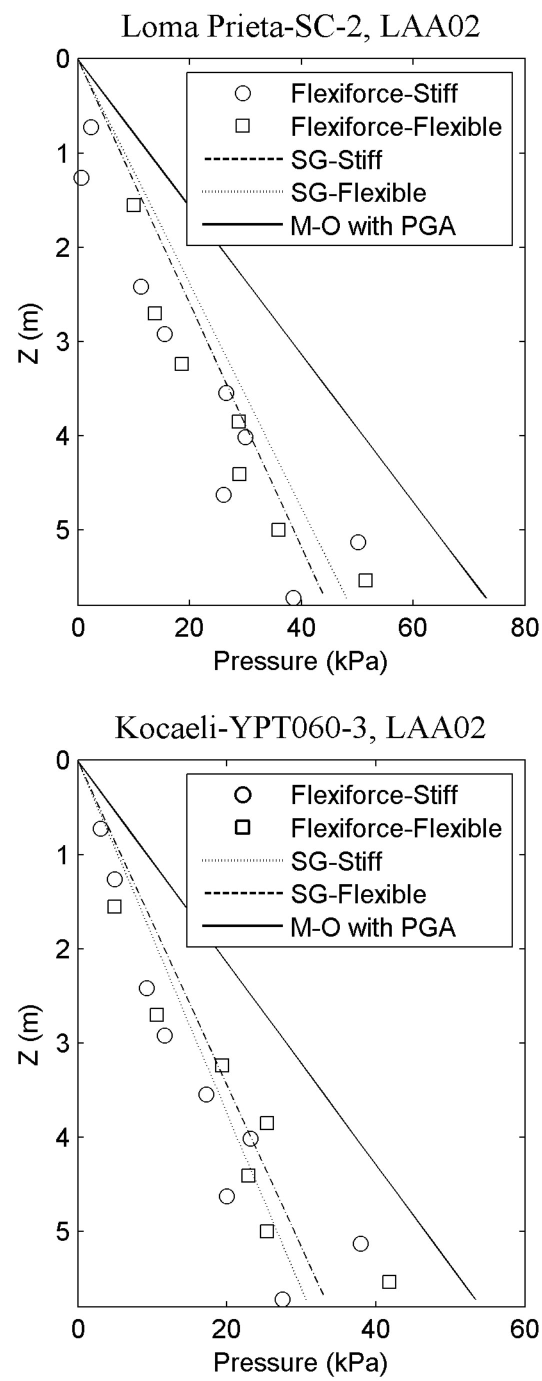

Figure 8. Maximum total earth pressure profiles measured and estimated using the M-O method on the south stiff and north flexible walls during Loma Prieta-SC-2 and Kocaeli-YPT060-3

earth pressures suggests that the experimental basis for the current design methodology based on the M-O method is not representative of the actual field response. This conclusion is supported by observed excellent performance of various types of retaining structures in recent earthquakes, which suggest that retaining structures underdesigned with respect to seismic forces perform well under seismic loading with peak acceleration in excess of 0.5 g.

Similarly, the results from a series of centrifuge experiments designed to evaluate the magnitude and distribution of dynamic earth pressures on retaining walls show that the mechanism of development of dynamic earth pressures is quite different from the simple limit equilibrium wedge assumption inherent in the M-O method. Specifically, the earth pressure distribution increases downward and the moments on the walls are to a large extent due to the contribution of the moment of inertia of the walls themselves. In addition, the moments generated by wall inertial forces and earth pressures are out of phase producing much lower overall moments than would be predicted by the M-O method. Retaining walls should therefore be designed for these reduced dynamic wall moments that include the combined effects of the inertial forces on the wall and the backfill.

Finally, given the apparent shortcomings of the assumptions inherent in the current analysis and design methods, there is a need for a complete and thorough re-examination of the entire methodology and philosophy for estimation of seismic earth pressures in different soil conditions and for different types of structures.

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