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Steve,

Here's the Ostadan paper.

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# LATERAL SEISMIC SOIL PRESSURE

# AN UPDATED APPROACH

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## INTRODUCTION

The effect of ground motion on retaining walls was recognized by Okabe (1924) and Mononobe and Matsuo (1929) following the great Kanto Earthquake of 1923 in Japan. The method proposed by Mononobe and Okabe, currently known as the M-O method, was based on the Coulomb's theory of static soil pressure developed more than 200 years ago. In the last 30 years, a great deal of research work both in the analytical and in experimental areas has been performed to evaluate the adequacy of the M-O method or to extend the method for specific applications. Discussion of the all the research work on the seismic soil pressure is extensive and is beyond the scope of this study. Rather, only the milestones that have influenced the design practice are described below.

### Seed and Whitman (1970)

In 1970, the M-O method and the associated analytical relationships were simplified by Seed and Whitman (1970) for design of earth retaining structures for dynamic loads. Using the charts, the designer only needs to know the basic properties of the backfill (the angle of internal friction) and the peak ground acceleration to obtain the seismic soil pressure. As suggested by Seed and Whitman, the basic assumptions used in the development of the M-O method should always be considered in design applications. These assumptions are:

- The backfill materials are dry cohesionless materials.
- The retaining wall yields equally and sufficiently to produce minimum active soil pressure.
- The active soil pressure is associated with a soil wedge behind the wall which is at the point of incipient failure and the maximum shear strength is mobilized along the potential sliding surface.
- The soil behind the wall behaves as a rigid body and the acceleration is uniform in the soil wedge behind the wall.

### Whitman et al. (1979, 1990, 1991)

The effect of some of the limiting assumptions used in the M-O method above has been investigated by, among others, Whitman et al. (1979, 1990, 1991) and Nadim and Whitman (1984). The non-yielding wall conditions and the amplifications of the motion in the soil mass were found to be significant in some cases. However, no practical tools were proposed for design applications to circumvent the limiting assumptions used in the M-O method. Judging from the results of model tests by several researchers, Whitman (1990) found that use of the M-O method for design of relatively simple gravity walls up to 30 ft high is acceptable. However, for higher walls and non-yielding walls, he recommended a more careful analysis be performed.

**Richards and Elms (1979)**

One of the more important recent developments in characterizing the seismic soil pressure for retaining walls was the work performed by Richards and Elms (1979). Using the M-O method and the Newmark's sliding-block analogy, the authors proposed a displacement-controlled method which incorporates basic ground motion parameters (maximum acceleration and maximum velocity) and reduces the seismic soil pressure based on the acceptable amount of the wall movement. In practice, the method is currently used for designing walls for which limiting horizontal displacements are of no concern.

**Wood (1973)**

While the M-O method was developed for yielding walls, Wood (1973) developed an equivalent static elastic solution for seismic soil pressure for non-yielding walls. The solution is based on finite element analysis of a soil-wall system for a wall resting on a rigid base and a uniform soil layer behind the wall. In general, Wood's solution amounts to a lateral force that acts about 0.63 times the height of the wall above the base of the wall which corresponds approximately to a parabolic distribution of soil pressure unlike M-O's inverted triangular distribution. Wood's solution predicts seismic soil pressure larger (by a factor of 2 to 3) than the pressure predicted by the M-O method. The elastic solution proposed by Wood has been adopted by ASCE Standards for Nuclear Structures (1986) and has been used in many applications. Wood's solution requires knowledge of the maximum ground acceleration along with the density and Poisson's ratio of the soil to obtain the seismic soil pressure behind the wall.

**Matsuzawa et al. (1984), Ishibashi et al. (1985)**

To address saturated backfill conditions and to include the hydrodynamic forces, the M-O method was extended by Matsuzawa et al. (1984) and Ishibashi et al. (1985). A comprehensive summary of the all the M-O based methods and their applications to various retaining wall conditions are documented in a recent US Army publication (Ebeling and Morrison, 1992).

**Veletsos et al. (1994a, 1994b)**

More recently, Veletsos and Younan (1994a, 1994b) developed an analytical model to compute seismic soil pressure for rigid vertical walls resting on a rigid base. The proposed model is based on the series of elastically supported semiinfinte horizontal bars with distributed mass to model the soil medium behind the wall. The model was developed for vertically propagating shear waves with the assumption that horizontal variation of vertical displacements in the soil medium is negligible. In this model, contrary to Wood's equivalent static solution, amplification of motion in the soil medium behind the wall is considered. The model highlights the effects of several parameters including the frequency of vibration on the seismic soil pressure magnitude and distribution. The model was subsequently expanded for application to cylindrical vaults and storage buildings (Veletsos and Younan, 1994c; 1995).

### Significance of Seismic Soil Pressure in Design

Seed and Whitman (1970) summarized damage to wall structures during earthquakes. Damage to retaining walls with saturated backfills is typically more dramatic and is frequently reported in the literature. However, damage reports of walls above the water table are not uncommon. A number of soil retaining structures were damaged in the San Fernando earthquake of 1971. Wood (1973) reports that the walls of a large reinforced concrete underground reservoir at the Balboa Water Treatment Plant failed as a result of increased soil pressure during the earthquake. The walls were approximately 20 ft high and were restrained by the top and bottom slabs.

Damage has been reported for a number of underground reinforced concrete box-type flood control channels. Richards and Elms (1979) report damage to abutment of bridges after the 1968 earthquake in Inangahua, New Zealand. Out of the 39 bridges inspected, 24 showed measurable movement and 15 suffered damage on bridge abutments. In the Madang earthquake of 1970 in New Guinea, the damage patterns were similar. Out of 29 bridges repaired, some experienced abutment lateral movements as much as 20 inches. Reports on failed or damaged bridge abutments indicate mainly settlement of the backfill and pounding of the bridge superstructure against the abutment in longitudinal and transverse directions.

Nazarian and Hadjian (1979) also summarized damage to soil-retaining structures during past earthquakes. Damage to bridges has also been reported from various earthquakes including 1960 Chilean, 1964 Alaskan, 1964 Nigata, 1971 San Fernando, and 1974 Lima. Most of the reported damage can be attributed to the increased lateral pressure during earthquakes.

Numerous damage reports are available from recent earthquakes which report damage to the embedded walls of buildings. However, contribution of the seismic soil pressure to the damage can not be quantified since the embedded walls often carry the inertia load of the superstructure with cracks extending in all directions in the walls of the buildings. On the other hand, simple structures, such as underground box-type structures, retaining walls, and bridge abutments have suffered damage due to the increased soil pressure. All of these reports and others not mentioned highlight the significance of using appropriate seismic soil pressure in design.

## RECENT EXPERIMENTS AND OBSERVATIONS

### Lotung Experiment

Soil-structure interaction (SSI) effects play a significant role in the dynamic response of critical structures and internal components. Recognizing these effects, the Electric Power Research Institute (EPRI) with the cooperation from Taiwan Power Company (TPC) and the United States Nuclear Regulatory Commission (NRC) sponsored a large-scale experiment in the earthquake active area of Lotung, Taiwan. The objective

of the experiment was to evaluate the SSI analyses methodologies and to reduce uncertainties in the design. In this experiment, a 1/4-scale containment model was constructed. Instrumentation was installed both in the containment model and at the site. Since completion of the model and its instrumentation in October 1985, a number of recordings from earthquakes ranging in Richter magnitude 4.5 to 7.0 have been made at the site. The information on site condition, soil properties, and structural drawings were distributed to selected teams from the industry and academia (a total of 13 groups one which was Bechtel) to predict the responses on a round-robin basis. The results of this extensive experiment and follow up studies are published in several EPRI reports (EPRI, 1989; EPRI, 1991). The senior author also participated in the studies performed by Bechtel.

The Lotung site is a relatively flat with a relatively soft surface layer with thickness of 200 ft to 260 ft (60 m to 80 m) overlying deep alluvium stratum. The soil properties in terms of low-strain shear and compression wave velocities were measured at the site. The shear wave velocity is about 100 m/sec increasing to 250 m/sec at the depth. In addition cyclic laboratory testing was performed on soil samples and the strain-dependent soil properties were obtained.

The instrumentation for the experiment is extensive and consists of accelerometers and pressure gages in the model and in the free-field. Pressure gages were installed beneath the basemat for monitoring uplifting and bonding/de-bonding of the basemat from the supporting soil layer. In addition, pressure gages were also installed on the perimeter of the containment shell to measure seismic lateral soil pressure.

A number of earthquakes up to magnitude 7 were recorded at the site. For the purpose of this study, only the records from earthquake event LSST07 are used. The LSST07 event occurred on May 20, 1986 at about 40 miles(66.2 km) from the Lotung experiment. This event had a Richter magnitude of 6.5. The peak ground acceleration in the free-field at the ground surface were 0.16g, 0.21g, and 0.04g in the east-west, north-south, and vertical directions, respectively.

A typical recorded rime histories of seismic soil pressure is shown in Figures 1. Most time histories show a drift in the response and substantial residual pressure at the end of the shaking. Some of the pressure time histories have also been examined by Chang et al. (1990). As suggested by Chang et al., the drift in the time history and the residual pressure are attributed to the compaction of the backfill material during shaking and particle re-arrangement of the materials in the soil near the instrument. For this reason the recorded pressure time histories were corrected to eliminate the drift and the residual pressure in order to obtain the peak transient stresses. The corrected pressure time history is also shown in Figure 1 with positive sign indicating pressure and negative sign indicating extension.

The seismic soil pressure shown in Figure 1 is the normal stress component with the direction normal to the body of the containment shell in the North-South direction. The magnitude of the stress is a function of the relative motion of the containment and the surrounding soil and the soil properties. In the Lotung experiment, the relative motion was caused primarily by the rigid body rocking motion of the containment shell. To evaluate the effect of rocking motion on the lateral seismic soil pressure, frequency contents of the rocking motion are compared with the frequency contents of the pressure time history at one location, as shown in Figure 2. Comparison of the pair of spectra shows that, while the nature of the spectral amplitudes are different and are expected to have different amplitudes, the frequency content of the two motions are very similar, particularly at the rocking frequency of the containment shell (2.2 Hz).

The overall comparison of the results (see Ostadan and White, 1997) indicates that the seismic soil pressure is caused by the relative motion of the structure with respect to the surrounding soil and as such it is a SSI response. This implies that the seismic soil pressure will not only be affected by the soil properties and the characteristics of the ground motion, but also the structural properties as well as the size of the structure and its foundation embedment.

Finally, the result of the SSI analysis using the computer program SASSI (Lysmer et al., 1981) in terms of seismic soil pressure was obtained and compared with the recorded pressure in terms of spectral amplitudes in Figure 3

## Other Observations From Recent Field and Experimental Data

In recent years, several field and laboratory experiments have been conducted to resolve the complexities associated with the seismic soil pressure and to develop a more realistic design parameter for the design of embedded structures. A summary of the selected recent investigations is presented below.

### Case 1 - Deeply Embedded Reactor Building

Hirota et al. (1992) have collected and studied the soil pressure data from instrumented buildings since 1989. Specifically, the data from a deeply embedded reactor building (embedment depth of 120 ft) in a suburb of Tokyo have been presented and evaluated. The data from a total of eight earthquake records are presented. The principal conclusions of the study are as follows:

- The seismic soil pressure is significantly affected by the low-frequency content of the earthquake motion.
- Comparison of the pressure time history with the derived relative displacement time history between the structure and the far-field shows similar characteristics in phase and amplitude.

**Case 2 - Deeply Embedded Structure**

Matsumoto et al. (1991) and Watakabe et al. (1992) present the results of a study using the recorded data for a deeply embedded building in a suburb of Tokyo. The site consists of a soft alluvial layer with a thickness of 120 ft underlain by a much stiffer formation. The shear wave velocity of the upper layer ranges from 300 ft/sec to 1000 ft/sec. The building foundation rests on the stiff formation. The records from a total of 21 earthquakes have been collected and examined. The main points of the investigation are as follows:

- Frequency content of the soil pressure was examined by comparing the normalized response spectra of the soil pressure with the normalized velocity spectra of the motion in the soil layers at the respective elevations. The shapes of the normalized spectra closely matched.
- The finite element method employed was able to predict the soil-interaction effects. This conclusion confirms the use of finite element and soil-structure interaction techniques to predict seismic soil pressure.

**Case 3 - Underground LNG Storage Tanks**

Koyama et al. (1988,1992) collected and examined the earthquake and seismic soil pressure records from two large scale Liquid Natural Gas (LNG) underground storage tanks. The instrumented tanks are large diameter concrete tanks (200 ft diameter, 120 ft high). The site soil is a medium dense sand with a shear wave velocity of 1300 ft/sec. Over the 8-year period, records from 70 earthquakes have been collected and examined. The authors concluded that the seismic soil pressure is strongly correlated to the acceleration and the relative displacement of the tank and the ground.

In addition to the field experiments, a number of laboratory tests have been recently performed Kazama and Inatoi (1988) and Itoh and Nogami (1990). Evaluation of the test results showed that:

- The dynamic soil pressure is amplified near the resonant frequency of the backfill sand.
- The effect of soil nonlinearity on the peak dynamic pressure can be observed by increasing the amplitude of the vibration.
- The dynamic soil pressure distribution is consistent with the relative displacement between the ground and the caisson.
- Finite element analysis methods are able to reproduce the measured data.
- At the soil column resonant frequency, the seismic soil pressure acts in the direction of the basement movement to drive the structure, whereas at the structural resonant frequency, the dynamic pressure acts in the opposite direction of the basement movement to restrain the movement of the structure.

### Recognition of the Problem and Objective of the Study

In spite of the much better understanding of the soil-wall interaction behavior that have evolved over the years, the M-O method continues to be widely used despite many criticisms and its limitations. As stated above, the method was developed for gravity retaining walls with cohesionless backfill materials. In design applications, however, the M-O method or any of its derivatives is commonly used for below ground building walls. In this regard, the M-O method is one of the most abused methods in the geotechnical practice.

In view of the overwhelming information and evidence on the dynamic behavior of buildings, some of which was outlined above, the United States Nuclear Regulatory Commission (US NRC, 1991) recently issued a position paper on the subject of the seismic soil pressure. Pertinent excerpts are quoted as follows:

> "The use of the M-O method of analysis to compute pressure on embedded walls of structures like the nuclear island (NI) structure of ….. is not considered appropriate since the development of the limit conditions in the soil requires wall movements which are most likely inappropriate for SSI conditions anticipated. The M-O approach will generally lead to a lower bound estimate for soil loads (using active state conditions in the soil) since the soil in the active wedge is assumed to transfer part of the load to the soil below through its own shear strength..."

It is the objective of this study to develop a simple and practical method to predict lateral seismic soil pressure for building walls.

- The walls of the buildings are often of the non-yielding type. The movement of the walls is limited due to the presence of the floor diaphragms, and displacements to allow development of the limit-state conditions are unlikely to develop during the design earthquake.
- The frequency content of the design motion is fully considered. Use of a single parameter as a measure of design motion such as peak ground acceleration may misrepresent the energy content of the motion, at frequencies important for soil amplifications.
- Appropriate soil properties are included in the analysis. For soil dynamic problems, the most important soil property is the shear wave velocity followed by the material damping, Poisson's ratio, and density of the soil.
- The method is flexible to allow for consideration of soil nonlinear effect where soil nonlinearity is expected to be important.
- The interaction between the soil and the building is represented. This includes consideration for the building rocking motion, amplification and variation of the motion in the soil, geometry, and embedment depth of the building.

## SIMPLIFIED METHOD TO PREDICT LATERAL SEISMIC SOIL PRESSURE FOR BUILDING WALLS ON ROCK OR FIRM FOUNDATIONS

In this section, the dynamic characteristics of lateral seismic soil pressure for buildings with basemat resting on rock or firm soil layers are examined and a simplified method for predicting seismic soil pressure is presented. It is assumed that the building walls are effectively rigid. The condition that the basemat rests on a firm soil layer also simplifies the problem in that the rocking vibration of the buildings becomes insignificant. With this assumption, the embedment ratio of the building (embedment depth to basemat width) will not play a role in the results. The extension of the method for buildings embedded in deep soil layers is presented in the next section.

To investigate the characteristics of the lateral seismic soil pressure, a series of seismic soil-structure interaction analyses was performed using the Computer Program SASSI. A typical SASSI model of a building basement is shown in Figure 4. The embedment depth is designated by H and the soil layer is identified by the shear wave velocity, Vs, the Poisson's ratio, $\nu$, total mass density, $\rho$, and the soil material damping, $\beta$. The basemat is resting on rock or a firm soil layer. A column of soil elements next to the wall is included in the model in order to retrieve the pressure responses from the results.

For this analysis, the acceleration time history of the input motion was specified at the top of the rock layer corresponding to the basemat elevation in the free-field. In order to characterize the dynamic behavior of the soil pressure, the most commonly used wave field consisting of vertically propagating shear waves was specified as input motion. The frequency characteristics of the pressure response were examined using harmonic shear waves for a wide range of frequencies. For each harmonic wave, the amplitude of the normal soil pressure acting on the building wall at three locations (Elements 2, 10, and 15 in Figure 4) was obtained. The pressure responses are presented in terms of pressure transfer function amplitudes which are the ratio of the amplitude of the seismic soil pressure in the respective element to the amplitude of the input motion (1g harmonic acceleration) in the free-field for each harmonic frequency. The analyses were performed for a building with embedment of 50 ft and soil shear wave velocities of 500, 1000, 1500, and 2000 ft/sec, all with the Poisson's ratio of 1/3. The material damping in the soil was specified to be 5%. The transfer function results for Element 2 (see Figure 4) are shown in Figure 5. As shown in this figure, the amplification of the pressure amplitude takes place at distinct frequencies. These frequencies increase as the soil shear wave velocity increases. The amplitude of soil pressure at low frequency was used to normalize the amplitude of the pressure transfer functions for each element. The frequency axis was also normalized using the soil column frequency which was obtained from the following relationship:

$$f = Vs / (4 \times H) \qquad (1)$$

In the above equation, Vs is the soil shear wave velocity and H is the embedment depth of the building. The normalized transfer functions are shown in Figure 6. As shown in this figure, the amplification of the pressure is about the same for all the shear wave velocities considered. In all cases the maximum amplification takes place at the frequency corresponding to the soil column frequency. Similarly, the results for points in the mid-height and bottom of the wall were examined (Ostadan and White, 1997). These results also showed the same characteristics described above.

Examining the dynamic characteristics of the normalized pressure amplitudes (such as those shown in Figure 6), it is readily evident that such characteristics are those of a single degree-of-freedom (SDOF) system. Each response begins at a value of one and increases to a peak value at a distinct frequency and subsequently reduces to a small value at high frequency. Dynamic behavior of a SDOF system is completely defined by the mass, stiffness and associated damping constant. It is generally recognized that response of a SDOF system is controlled by the stiffness at low frequency, by damping at resonant frequency, and by the inertia at high frequencies.

Following the analogy for a SDOF system and in order to characterize the stiffness component, the pressure amplitude at low frequencies for all elements (Elements 1 through 15 in Figure 4) was obtained and plotted as shown in Figure 7 in terms of the normalized height (Y/H, H=50 ft; Y is the distance from the base of the wall as shown in Figure 4). The pressure amplitudes at low frequency are almost identical for the wide range of the soil shear wave velocity profiles considered. The sudden increase shown at the top of the profile is due to the zero stress boundary condition near the ground surface and can be improved if finer elements are used. However, it is also generally recognized that soils particularly at shallow depths with low confining pressure have low shear strength and are subject to softening during vibration. For this reason, the normalized pressure profile was adjusted to have a vertical tip as shown in Figure 7. The shape of the normalized pressure will be used as a basis to determine seismic soil pressure along the height of the building wall. This will be discussed after the seismic soil pressure is examined for cases in which input motion is specified at the ground surface level.

A similar series of parametric studies were also performed by specifying the input motion at the ground surface level (Ostadan and White, 1997). The results of these studies also showed that the seismic soil pressure in normalized form can be represented by a single degree-of-freedom (SDOF) system. For both cases considered, the low frequency pressure profiles depict the same distribution of the pressure along the height of the wall as shown in Figure 7. This observation is consistent with the results of the analytical model developed by Veletsos and Younan (1994a). Since all the soil-structure interaction analyses were performed for the Poisson's ratio of 1/3, the pressure distribution was adjusted for the soil's Poisson's ratio using the factor recommended by Veletsos and Younan (1994a). The $\psi_\nu$ factor is defined by:

$$\psi_\nu = \frac{2}{\sqrt{(1-\nu)(2-\nu)}} \tag{2}$$

For the Poisson's ratio of 1/3 used in the analysis, $\psi_\nu$ is 1.897. Use of $\psi_\nu$ in the formulation allows correction of the soil pressure amplitude for various Poisson's ratios. The adjusted soil pressure distribution is also shown in Figure 7. Using the adjusted pressure distribution, a polynomial relationship was developed to fit the normalized pressure curve. The relationship in terms of normalized height, y = Y/H (Y is measured from the bottom of the wall and varies from 0 to H), is as follows:

$$p(y) = -.0015 + 5.05y - 15.84y^2 + 28.25y^3 - 24.59y^4 + 8.14y^5 \tag{3}$$

The area under the curve can be obtained from integration of the pressure distribution over the height of the wall. The total area is 0.744 in terms of normalized wall height or 0.744H for the wall with the height H.

Having obtained the normalized shape of the pressure distribution, the amplitudes of the seismic pressure can be also obtained from the concept of a SDOF. The response of a SDOF system subjected to earthquake loading is readily obtained from the acceleration response spectrum of the input motion at the damping value and frequency corresponding to the SDOF. The total load is subsequently obtained from the product of the total mass times the acceleration spectral value at the respective frequency of the system.

To investigate the effective damping associated with the seismic soil pressure amplification and the total mass associated with the SDOF system, the system in Figure 4 with wall height of 50 ft and soil shear wave velocity of 1500 ft/sec was subjected to six different input motions in successive analyses. The motions were specified at the ground surface level in the free-field. The acceleration response spectra of the input motions at 5% are shown in Figure 8. The motions are typical design motions used for analyses of critical structures. From the set of six shown in Figure 8, two motions labeled EUS local and distant are the design motions for sites in Eastern US with locations close and far away from a major fault. The ATC S1 motion is the ATC recommended motion for S1 soil conditions. The WUS motion is the design motion for a site close to a major fault in Western US. The RG1.60 motion is the standard site-independent motion used for nuclear plant structures. Finally, the Loma Prieta motion is the recorded motion from the Loma Prieta earthquake scaled to 0.3g maximum acceleration. This motion is used in the analysis as described in later sections. All motions are scaled to 0.30g and limited to frequency cut-off of 20 Hz for use in the analysis. The cut-off frequency of 20 Hz reduces the peak ground acceleration of the EUS local motion to less than 0.30g due to high frequency content of this motion as shown in Figure 8.

The maximum seismic soil pressure values at each depth obtained from the analyses for the various input motions are shown in Figure 9. The amplitudes of the pressure vary from one motion to the other with larger values associated with use of RG1.60 motion. Using the pressure profiles in Figure 9, the lateral force acting on the wall for each input motion was computed. The lateral force represents the total inertia force of a SDOF for which the system frequency is known. The system frequency for the case under consideration is the soil column frequency which is 7.5 Hz based on Eqn (1). The total force divided by the spectral acceleration of the system at 7.5 Hz at the appropriate damping ratio amounts to the mass of the SDOF. To identify the applicable damping ratio, the acceleration response spectrum of the free-field response motions at the depth of 50 ft were computed for all six motions shown in Figure 8 for damping ratios of 5, 10, 20, 30, 40, 50, and 60 percents. Knowing the total force of the SDOF, the frequency of the system, and the input motion to the SDOF system, the relationship in the form proposed by Veletsos and Younan (1994a) was used to compute the total mass and the damping of the SDOF system. For the total mass, the relationship is

$$m = 0.50 \times \rho \times H^2 \times \psi_\nu \qquad (4)$$

where $\rho$ is the mass density of the soil, H is the height of the wall, and $\psi_\nu$ is the factor to account for the Poisson's ratio as defined in Eqn (2). In the analytical model developed by Veletsos and Younan, a constant coefficient of 0.543 was used in the formulation of the total mass. Study of the soil pressure transfer functions and free-field response motions at the depth of 50 ft showed that spectral values at the soil column frequency and at 30% damping have the best correlation with the forces computed directly from the SSI analysis. In the Veletsos and Younan's model, a damping of 27.5 + $\beta$ percent has been proposed where $\beta$ is the material damping of the soil (%). For the case of 5% soil material damping, the proposed spectral damping amounts to 32.5%. However, as shown by Ostadan and White (1997), the spectral values of the various motions considered are insensitive to the spectral damping ratios at the soil column frequency of 7.5. The various motions, however, have significantly different spectral values at the soil column frequency. This observation leads to the conclusion that while the frequency of the input motion particularly at the soil column frequency is an important component for magnitude of the seismic soil pressure, the spectral damping ratio selected is much less important in terms of pressure amplitudes. The role of soil material damping is discussed by Ostadan and White (1997).

### Simplified Method: Computational Steps

To predict the lateral seismic soil pressure for below ground building walls resting on firm foundation and assuming rigid walls, the following steps should be taken:

1. Perform free-field soil column analysis and obtain the ground response motion at the depth corresponding to the base of the wall in the free-field. The response motion in terms of acceleration response spectrum at 30% damping should be obtained. The free-field soil column analysis may be performed using the Computer Program SHAKE (Schnabel et al., 1972) with input motion specified either at the ground surface or at the depth of the foundation basemat. The choice for location of control motion is an important decision that needs to be made consistent with the development of the design motion. The location of input motion may significantly affect the dynamic responses of the building and the seismic soil pressure amplitudes.

2. Use Eqn (4) to compute the total mass for a representative SDOF system using the Poisson's ratio and mass density of the soil.

3. Obtain the lateral seismic force from the product of the total mass obtained in Step 2 and the acceleration spectral value of the free-field response at the soil column frequency obtained at the depth of the bottom of the wall (Step 1).

4. Obtain the maximum lateral seismic soil pressure at the ground surface level by dividing the lateral force obtained in Step 3 by the area under the normalized seismic soil pressure, 0.744 H.

5. Obtain the pressure profile by multiplying the peak pressure with the pressure distribution relationship shown in Eqn (3).

One of the attractive aspects of the simplified method is its ability to consider soil nonlinear effect. The soil nonlinearity is commonly considered by use of the equivalent method and the strain-dependent soil properties. Depending on the intensity of the design motion and soil properties, the effect of soil nonlinearity can be important in changing the soil column frequency and therefore, amplitude of the spectral response at the soil column frequency.

### Accuracy of the Simplified Method

The simplified method outlined above was tested for building walls with heights of 15, 30 and 50 ft using up to six different time histories as input motion. The results computed directly with SASSI are compared with the results obtained from the simplified solution. A typical comparison is shown in Figure 10. More extensive validation of the method is presented by Ostadan and White (1997).

### Comparison to Other Commonly Used Solutions

The seismic soil pressure results obtained for a building wall 30 ft high embedded in a soil layer with shear wave velocity of 1000 ft/sec using the M-O, Wood and the proposed simplified methods are compared in

Figure 11. For the simplified method, the input motions defined in Figure 8 were used. The M-O method results in the smallest pressure values. This is understood since this method relies on the wall movement to relieve the pressure behind the wall. Wood's solution generally results in the maximum soil pressure and is independent of the input motion as long as the peak acceleration is 0.3 g. The proposed method results in a wide range of pressure profiles depending on the frequency contents of the input motion, particularly at the soil column frequency. For those motions for which the ground response motions at the soil column frequency are about the same as the peak ground acceleration of the input motion, e.g., RG1.60 motion, the results of the proposed method are close to Wood's solution. Similar trend in the results is observed if sum of the lateral forces and the overturning moments from the above three methods are compared (Ostadan and White, 1997).

The simplified method was extended for application to soil layered system and soil deposits with parabolic distribution of the shear modulus. The extended method and its verification are discussed by Ostadan and White (1997).

## SIMPLIFIED METHOD TO PREDICT LATERAL SEISMIC SOIL PRESSURE FOR BUILDINGS IN DEEP SOIL SITES

One of the distinct dynamic characteristics of a building in a deep soil site is its rocking vibration which has a significant role on distribution of the pressure depending on the embedment ratio (embedment depth versus plan dimensions), dynamic properties of the soil, and frequency contents of the ground motion under consideration.

Mita and Luco (1989) have reported the harmonic response of an embedded square foundation subjected to vertically propagating shear waves. The results adopted from the authors but modified to reflect the same nomenclature used in this report are shown in Figure 12. The results are for a square foundation with plan dimensions of 2B x 2B and embedment depth H. The halfspace is characterized by the shear wave velocity of Vs. The free-field motion has a unit amplitude at the ground surface at each harmonic frequency. The horizontal translational motion of the foundation (D) at the middle point corresponding to the basemat motion and the normalized rocking motion represented in terms of HxT are shown in terms of dimensionless frequency ratio $a_1 = \omega x H/Vs$ where T is the angle of rocking rotation and $\omega$ is the circular frequency at each harmonic frequency under consideration. The dimensionless frequency is a measure of the harmonic shear wave length as compared to the embedment depth H. The free-field motion corresponding to the basemat depth (depth of H) in the free-field shows decreasing amplitude with increasing frequency. At the soil column frequency of $f = Vs/(4xH)$, the dimensionless frequency a1 is 1.57 at which the amplitude of the free-field motion is zero. The foundation motion is a function of the frequency of vibration and the embedment ratio (H/B).

In order to examine the effect of rocking motion on seismic soil pressure, a series of SSI analyses were performed using the soil shear wave velocities of 500, 1000, 1500, and 2000 ft/sec. In all cases, the wall height considered was H=50 ft but the foundation width (2B) was changed successively from 50 ft, to 100 ft, 200 ft, and to 400 ft, resulting in embedment ratios of B/H=0.5, 1, 2, and 4. The input motion was specified at the basemat level in the free-field. A typical result in terms of amplitude of pressure transfer function is shown in Figures 13. For each soil case, the results from all three elements are clustered together with the same peak frequency which leads to the conclusion that (1) the soil column frequency continues to be the most significant frequency for the response in terms of maximum value of the seismic soil pressure, and (2) the frequency of the peak response is not affected by the embedment ratio. However, the distribution of the maximum soil pressure in terms of amplitude of the pressure in Elements 2, 5, and 10 is significantly affected by the rocking motion of the building and thus the embedment ratio. The effects of rocking motion on distribution of maximum seismic soil pressure for four different aspect ratios are shown in Figure 14. As shown, for buildings with narrow width, the rocking motion tends to reduce the amplitude of the soil pressure at top of the wall.

The results of the parametric studies performed for deep soil sites were also examined in detail. Limitation of space prohibits detail discussion of the studies performed. The computational steps for deep soil sites are, however, similar to the rigid case and consist of the following:

1. Perform free-field soil column analysis and obtain the response motion in terms of acceleration response spectrum at 30% damping at the depth corresponding to the basemat elevation in the free-field.
2. Obtain the soil column frequency using Eqn (1) and obtain the spectral value at the soil column frequency using the results of Step 1.
3. Use the following relationship to obtain the lateral force acting on the wall:

$$F = \alpha \times \rho \times H^2 \times Sa \times \Psi_v \qquad (5)$$

where $\rho$ is the mass density of the soil, H is height of the wall, Sa is the spectral value of the free-field response obtained in Step 2, and $\Psi_v$ is the function that considers the effect of soil Poisson's ratio and can be obtained using Eqn (2). In order to represent the effect of the embedment ratio and reduction of soil pressure due to rocking motion as well as its increase beyond the rigid base cases for wide buildings, the parameter $\alpha$ is defined in the equation above. This parameter was determined from back-calculation of the lateral force obtained from soil pressure and the shear stress under the basemat to hold the equilibrium of forces in the horizontal direction. Using the results of the all the parametric studies, the following values were obtained for $\alpha$:

| Embedment Ratio, B/H | Parameter α |
|---|---|
| 0.50 | 0.27 |
| 1.0 | 0.43 |
| 2.0 | 0.62 |
| 4.0 | 0.92 |

4. Obtain the maximum soil pressure by dividing the lateral force obtained from Step 3 to the area under the soil pressure curve provided in Eqns(6) through (9) below depending on the embedment ratio. For an embedment ratio that falls in between the ratios considered, use interpolation.

**Embedment ratio of B/H=0.50**

$p(y) = -2.58y^3 + 0.32\,y^2 + 2.46\,y - 0.03$ (6)

Maximum pressure at the depth y = 0.625

Area under the curve = 0.632H

Point of application for resultant force, Y = 0.55H

**Embedment ratio of B/H=1.0**

$p(y) = 0.60y^3 - 3.09y^2 + 3.34y - 0.025$ (7)

Maximum pressure at the depth y = 0.625

Area under the curve = 0.77H

Point of application for resultant force, Y = 0.58H

**Embedment ratio of B/H=2.0**

$p(y) = -1.33y^4 + 4.38y^3 - 5.66y^2 + 3.44y + 0.17$ (8)

Maximum pressure at top of the wall y = 1

Area under the curve = 0.832H

Point of application for resultant force, Y = 0.57H

**Embedment ratio of B/H=4.0**

$p(y) = -0.085y^2 + 0.47y + 0.61$ (9)

Maximum pressure at top of the wall y = 1

Area under the curve = 0.82H

Point of application for resultant force, Y = 0.54H

5. Multiply the maximum lateral soil pressure from Step 4 by the relationships provided in Eqns (6) through (9) to get the pressure distribution depending on the embedment ratio of the foundation under

consideration. Judgment should be exercised to obtain the distribution for embedment ratios in between the four embedment ratios considered above.

The simplified method for deep soil sites was also tested extensively for a wide range of soil properties and foundation embedment ratios (Ostadan and White, 1997).

A comparison of the simplified method with the M-O and Wood's methods for a building with four different embedment ratios is shown in Figure 15. The results clearly demonstrates the effect of the rocking motion on distribution of the seismic soil pressure.

## SUMMARY AND CONCLUSION

The Mononobe-Okabe (M-O) method was developed in the 1920's. Since then, a great deal of research work has been performed to evaluate its adequacy and to improve it. The method is, strictly speaking, applicable to gravity retaining walls which, upon experiencing seismic loading, undergo relatively large movement to initiate the sliding wedge behind the wall and to relieve the pressure to its active state. Unfortunately, the method has been and continues to be used extensively for embedded walls of the buildings as well. Recent field observations and experimental data, along with enhancements in analytical techniques have shown that hardly any of the assumptions used in the development of the M-O method are applicable to building walls. The data and the follow up detail analysis have clearly shown that the seismic soil-pressure is an outcome of the interaction between the soil and the building during the seismic excitation and as such is function of all parameters that affect soil-structure interaction (SSI) response. Some of the more recent observations and experimental data were presented in the paper. The new understanding of the attributes of seismic soil pressure prompted the United Sates Nuclear Regulatory Committee (NRC) to reject the M-O and the M-O based methods for application to critical structures. At this time, while elaborate finite element techniques are available to obtain the soil pressure for design, no simple method has been proposed for quick prediction of the maximum soil pressure, thus hindering the designer's ability to use an appropriate method in practice. To remedy this problem, the current research was conducted to develop a simple method which incorporates the main parameters affecting the seismic soil pressure for buildings.

Using the concept of the single degree-of-freedom, a simplified method was developed to predict maximum seismic soil pressures for buildings resting on firm foundation materials. The method incorporates the dynamic soil properties and the frequency content characteristics of the design motion in its formulation. It was found that the controlling frequency that determines the maximum soil pressure is that corresponding to the soil column adjacent to the embedded wall of the building. The proposed method requires the use of conventionally-used simple one-dimensional soil column analysis to obtain the relevant soil response at the base of the wall. More importantly, this approach allows soil nonlinear effects to be

considered in the process. The effect of soil nonlinearity can be important for some applications depending on the intensity of the design motion and the soil properties. Following one-dimensional soil column analysis, the proposed method involves a number of simple hand calculations in order to arrive at the distribution of the seismic soil pressure for design. The accuracy of the method relative to the more elaborate finite element analysis was verified for a wide range of soil properties, earthquake motions, and wall heights.

The method was extended to include buildings on deep soil sites . The complexity of the seismic soil pressure for such cases is compounded by the rocking motion of the structure. The rocking motion is in turn, a function of soil properties, frequency content of the design motion, and embedment ratio of the structure. A wide range of parametric studies were performed that cover many practical cases. The steps for the analysis are similar to the steps outlined for buildings on rock except that an appropriate pressure distribution curve should be selected to observe the effect of the embedment ratio. Similarly, the accuracy of the proposed method was verified against a more detailed SSI analysis.

## REFERENCES

ASCE 4.86 (1986). "Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures," Published by the American Society of Civil Engineers.

Chang, C. Y., Power, M. S., Tang, Y. K., and Tang, H. T. (1990). "Analysis of Dynamic Lateral Earth Pressures Recorded on Lotung Reactor Containment Model Structure," Proceedings of 4th US National Conference on Earthquake Engineering, May 20-24, Palm Springs, CA.

Ebeling, R. M., and Morrison, Jr., E. E. (1992). "The Seismic Design of Waterfront Retaining Structures," US Army Technical Report ITL-92-11, Prepared by Information Technology Laboratory, Department of the Army, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.

EPRI (1989). "Proceedings: EPRI/NRC/TPC Workshop on Seismic Soil-Structure Interaction Anslysis Techniques Using Data From Lotung, Taiwan," Electric Power Research Institute Publication No. NP-6154, 2 Volumes, March.

EPRI (1991). "Post-Earthquake Analysis and Data Correlation for the 1/4-Scale Containment Model of the Lotung Experiment," Electric Power Research Institute Publication No. NP-7305SL, October.

Hirota, M., Sugimoto, M., and Onimaru, S. (1992). "Study on Dynamic Earth Pressure Through Observation," Proceedings of 10th World Conference on Earthquake Engineering, July, Madrid, Spain.

Ishibashi, I., Matsuzawa, H., and Kawamura, M. (1985). “Generalized Apparent Seismic Coefficient for Dynamic Lateral Earth Pressure Determination,” Proceedings of 2nd International Conference on Soil Dynamics and Earthquake Engineering, the Queen Elizabeth II, New York to Southampton, June/July.

Itoh, T., and Nogami, T. (1990). “Effects of Surrounding Soils on Seismic Response of Building Basements,” Proceedings of 4th US National Conference on Earthquake Engineering, May 20-24, Palm Springs, CA.

Kazama, M., and Inatomi, T. (1988). “A Study on Seismic Stability of Large Embedded Rigid Structures,” Proceedings of Ninth World Conference on Earthquake Engineering, August 2-9, Tokyo-Kyoto, Japan.

Koyama, K., Watanabe, O., and Kusano, N. (1988). “Seismic Behavior of In-Ground LNG Storage Tanks During Semi-Long Period Ground Motion,” Proceedings of 9th World Conference on Earthquake Engineering, August 2-9, Tokyo-Kyoto, Japan.

Koyama, K., Kusano, N., Ueno, H. , and Kondoh, T. (1992). “Dynamic Earth Pressure Acting on LNG in-Ground Storage Tank During Earthquakes,” Proceedings of 10th World Conference on Earthquake Engineering, July, Madrid, Spain.

Matsuzawa, H., Ishibashi, I., and Kawamura, M. (1984). “Dynamic Soil and Water Pressures of Submerged Soils,” ASCE, Journal of Geotechnical Engineering, Vol. 111, No. 10, September.

Matsumoto, H., Arizumi, K., Yamanoucho, K., Kuniyoshi, H., Chiba, O., and Watakabe, M. (1991). “Earthquake Observation of Deeply Embedded Building Structure,” Proceedings of 6th Canadian Conference on Earthquake Engineering, June, Toronto, Canada.

Mita, A. and Luco, J. E. (1989). “Impedance Functions and Input Motions For Embedded Square Foundations,” ASCE, Journal of Geotechnical Engineering, Vol. 115, No. 4, April.

Mononobe, N. and Matuo, H. (1929). “On the Determination of Earth Pressures During Earthquakes,” Proceeding of World Engineering Congress, Tokyo, Vol. 9, Paper 388.

Nadim, F. and Whitman, R.V. (1984). “Coupled Sliding and Tilting of Gravity Retaining Walls During Earthquakes,” Proceedings of 8th World Conference on Earthquake Engineering, July, San Francisco, CA.

Nazarian, H., and Hadjian, A. H. (1979). “Earthquake Induced Lateral Soil Pressure on Structures,” ASCE, Journal of Geotechnical Engineering, Vol. 105, No. GT9, September.

Nuclear Regulatory Commission (NRC, 1991), Proposed Staff Position on Dynamic Lateral Earth Pressures on Earth Retaining Walls and Embedded Walls of Nuclear Power Plant Structures, Civil Engineering and Geoscience Branch, Division of Engineering, Office of Nuclear Reactor Regulation.

Okabe, S. (1924). “General Theory of Earth Pressures and Seismic Stability of Retaining Wall and Dam,” Journal of Japan Society of Civil Engineers, Vol. 12, No. 1.

Lysmer et al. (1981). “SASSI - A System for Analysis of Soil-Structure Interaction,” Report No. UCB/GT/81-02, University of California, Department of Civil Engineering, Berkeley, California.

Ostadan, F., White, W. H. (1997). “Lateral Seismic Soil Pressure-An Updated Approach,” Bechtel Technical Grant Report, Bechtel Corporation, San Francisco, California.

Richards Jr., R. and Elms, D. (1979). “Seismic Behavior of Gravity Retaining Walls,” ASCE, Journal of Geotechnical Engineering, Vol. 105, No. GT4, April.

Schnabel, P. B. , Lysmer, J., Seed, H. B., ”SHAKE - A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites,” Earthquake Engineering Research Center, University of California, Berkeley, Report No. EERC 72-12, December 1972.

Seed, H. B., and Whitman, R. V. (1970). “Design of Earth Retaining Structures for Seismic Loads,” ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, June.

Veletsos, A, and Younan, A. H. (1994a). “Dynamic Soil Pressure on Rigid Vertical Walls”, Earthquake Engineering and Soil Dynamics, Vol. 23, 275-301.

Veletsos, A, and Younan, A. H. (1994b). “Dynamic Modeling and Response of Soil-Wall Systems,” ASCE, Journal of Geotechnical Engineering, Vol. 120, No. 12, December.

Veletsos, A, and Younan, A. H. (1994c). “Dynamic Soil Pressures on Rigid Cylindrical Vaults,” Earthquake Engineering and Structural Dynamics, Vol. 23, 645-669

Veletsos, A, and Younan, A. H. (1995). “Dynamic Modeling and Response of Rigid Embedded Cylinders,” ASCE, Engineering Mechanics, October.

Veletsos, A, Parikh, V. H., Younan, A. H., and K. Bandyopadhyay (1995). "Dynamic Response of a Pair of Walls Retaining a Viscoelastic Solid," Brookhaven National Laboratory, AUI, Department of Advanced Technology, Associated Universities, Inc., UPTON, New York.

Watakabe, M., Matsumoto, H., Fukahori, Y., and Shikama, Y., Yamanouchi, K., and Kuniyoshi, H. (1992). "Earthquake Observation of Deeply Embedded Building Structure," Proceedings of 10th World Conference on Earthquake Engineering, July, Madrid, Spain.

Whitman, R. V. (1979). "Dynamic Behavior of Soils and Its Application to Civil Engineering Projects," Proceedings of 6th Pan American Conference, Lima, Peru.

Whitman, R. V., (1990). "Seismic Design and Behavior of Gravity Retaining Walls," Proceedings of a Specialty Conference on Design and Performance of Earth Retaining Structures, ASCE, Cornell University, June 18-21.

Whitman, R. V., and Christian, J. T. (1990). "Seismic Response of Retaining Structures," Pola Workshop Meeting, San Pedro, CA.

Whitman, R. V. (1991). "Seismic Design of Earth Retaining Structures," Proceedings of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, St. Louis, Missouri.

Wood, J. H. (1973). "Earthquake Induced Soil Pressures on Structures," Doctoral Dissertation, EERL 73-05, California Institute of Technology, Pasadena, CA.