SEISMIC SOIL PRESSURE FOR BUILDING WALLS— AN UPDATED APPROACH

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Abstract—*The Mononobe-Okabe (M-O) method of predicting dynamic earth pressure developed in the 1920s in Japan continues to be widely used despite many criticisms and its limitations. The method was developed for gravity walls retaining cohesionless backfill materials. In design applications, however, the M-O method and its derivatives are 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 recognition of the M-O method's limitations, a simplified method was recently developed to predict lateral seismic soil pressure for building walls. The method is focused on the building walls rather than retaining walls and specifically considers the dynamic soil properties and frequency content of the design motion in its formulation.*

Keywords—*Mononobe-Okabe, SASSI2000, seismic, soil pressure, SSI*

INTRODUCTION

The Mononobe-Okabe (M-O) method of predicting dynamic earth pressure was developed in the 1920s. [1, 2] Since then, a great deal of research has been performed to evaluate its adequacy and develop improvements. This research includes the work by Seed and Whitman [3], Whitman et al. [4, 5, 6], Richard and Elms [7], and Matsuzawa et al. [8]. A good summary of the various methods and their application is reported in [9]. Most developments cited above are based on the original M-O method. The M-O method is, strictly speaking, applicable to soil 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 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 subsequent detailed analysis have clearly shown that the seismic soil pressure is a result of the interaction between the soil and the building during the seismic excitation and as such is a function of all parameters that affect soil-structure interaction (SSI) responses. Some of the more recent observations and experimental data, including an expanded discussion of the method presented herein, are reported in [10].

The major developments that consider the soil-wall interaction under dynamic loading are those by Wood [11] and Veletsos et al. [12, 13]. The solution by Wood commonly used for critical facilities [14] is, in fact, based on static "1 g" loading of the soil-wall system and does not include the wave propagation and amplification of motion. The recent solution by Veletsos et al. is a much more rigorous solution and considers the effects of wave propagation in the soil mass. The solution, however, is complex and lacks simple computational steps for design application. The effect of soil non-linearity and incorporation of harmonic solution for application to transient design motion require additional development with an associated computer program not currently available.

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 that incorporates the main parameters affecting the seismic soil pressure for buildings. This paper presents the development of the simplified method and a brief summary of its extensive verification. Its application for a typical wall is demonstrated by a set of simple numerical steps. The results are compared with the commonly

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ABBREVIATIONS, ACRONYMS, AND TERMS

ATC	Applied Technology Council
M-O	Mononobe-Okabe
NEHRP	National Earthquake Hazard Reduction Program
NRC	Nuclear Regulatory Commission
SASSI	System for Analysis of Soil-Structure Interaction
SDOF	single degree of freedom
SSI	soil-structure interaction
TF	transfer function

In recent years, the understanding of the attributes of seismic soil pressure has improved significantly.

used methods such as the M-O method and the solution by Wood.

The proposed method has been adopted and recommended by the National Earthquake Hazard Reduction Program (NEHRP). [15]

Significance of Seismic Soil Pressure in Design: Recent Observations

Seed and Whitman [3] 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, reports of damage to walls above the water table are not uncommon. A number of soil-retaining structures were damaged in the San Fernando earthquake of 1971. Wood [11] 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 top and bottom slabs.

Damage has been reported for a number of underground reinforced concrete box-type flood control channels. Richards and Elms [7] report damage to bridge abutments after the 1968 earthquake in Inangahua, New Zealand. Out of the 39 bridges inspected, 24 showed measurable movements and 15 suffered abutment damage. In the Madang earthquake of 1970 in New Guinea, the damage patterns were similar. Of the 29 bridges repaired, some experienced abutment lateral movements of as much as 20 in. 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 [16] also summarized damage to soil-retaining structures during past earthquakes. Damage to bridges has also been reported from various earthquakes, including 1960 Chile, 1964 Alaska, 1964 Nigata, 1971 San Fernando, and 1974 Lima. Most of the reported damage can be attributed to the increased lateral pressure during earthquakes.

Numerous reports are also available from recent earthquakes that report damage to the embedded walls of buildings. However, it is not possible to quantify the contribution of seismic soil pressure to the damage because the embedded walls often carry the inertia load of the superstructure, which is combined with seismic soil pressure load contributing to the damage. 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.

In recent years, the understanding of the attributes of seismic soil pressure has improved significantly. This is mainly due to extensive field and laboratory experiments and data collected from instrumented structures, as well as to the improvement in computational methods in handling the SSI problems. Recent experiments and analyses of the recorded response of structures and seismic soil pressure have been reported in numerous publications. [17–24] These observations confirm that seismic soil pressure is caused by the interaction of the soil and structure and is influenced by the dynamic soil properties, the structural properties, and the characteristics of the seismic motion. The new insight prompted the US Nuclear Regulatory Commission (NRC) to reject the M-O and M-O-based methods for application to critical structures.

SIMPLIFIED METHODOLOGY

The simplified methodology presented in this paper focuses on the building walls rather than soil-retaining walls and specifically considers the following factors:

- Deformation of the walls is limited due to the presence of the floor diaphragms and the internal cross walls, and the walls are considered rigid and non-yielding.
- The effect of wave propagation in the soil mass and interaction of the soil and wall are considered.
- The frequency content of the design motion is fully considered. Use of a single parameter, such as the peak ground acceleration, as a

measure of design motion may misrepresent the energy content of the motion at frequencies important for soil pressure.

- Applicable dynamic soil properties, in terms of soil shear wave velocity and damping, are included in the analysis.
- The method is flexible to allow for consideration of soil nonlinear effect where soil nonlinearity is expected to be significant.

It is recognized that the seismic soil pressure is affected not only by the kinematic interaction of the foundation, but also by the inertia effect of the building. The mass properties of buildings vary significantly from one to another. The proposed solution is limited to prediction of seismic soil pressure as affected by the kinematic interaction effects of the building, consistent with the inherent assumption used in the current methods. Experience from numerous rigorous SSI analyses of buildings confirms that using the proposed solution can adequately predict the amplitude of the seismic soil pressure for many buildings even when the inertia effect is included. Some local variation of soil pressure may be present, depending on the layout of the interconnecting slabs and the interior cross walls to the exterior walls, and the relative stiffness of the walls and the soil.

To investigate the characteristics of the lateral seismic soil pressure, a series of seismic soil-structure interaction analyses was performed using the Computer Program SASSI2000. [25] A typical SASSI model of a building basement is shown in **Figure 1**. The embedment depth is designated by *H* and the soil layer is identified by shear wave velocity *Vs*, Poisson's ratio ν, total mass density ρ, and soil material damping β. The basemat is resting on rock or a firm soil layer. A column of soil elements next to the wall is explicitly modeled to retrieve the pressure responses from the solution. The infinite extent of the soil layers on the sides of the building, as well as the half-space condition below the building, are internally modeled in SASSI in computing the impedance functions for the structural nodes in contact with soil.

The assumption of a firm soil or a rock layer under the basemat eliminates the rocking motion of the foundation. For deep soil sites, and depending on the aspect ratio of the foundation, the rocking motion can influence the magnitude and distribution of soil pressure. Due to space limitation, the extension of the method for deep soil sites is not presented in this paper. A detailed discussion is reported in [10].

For the SASSI 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. 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

The NRC rejects the conventional M-O method for application to critical structures.

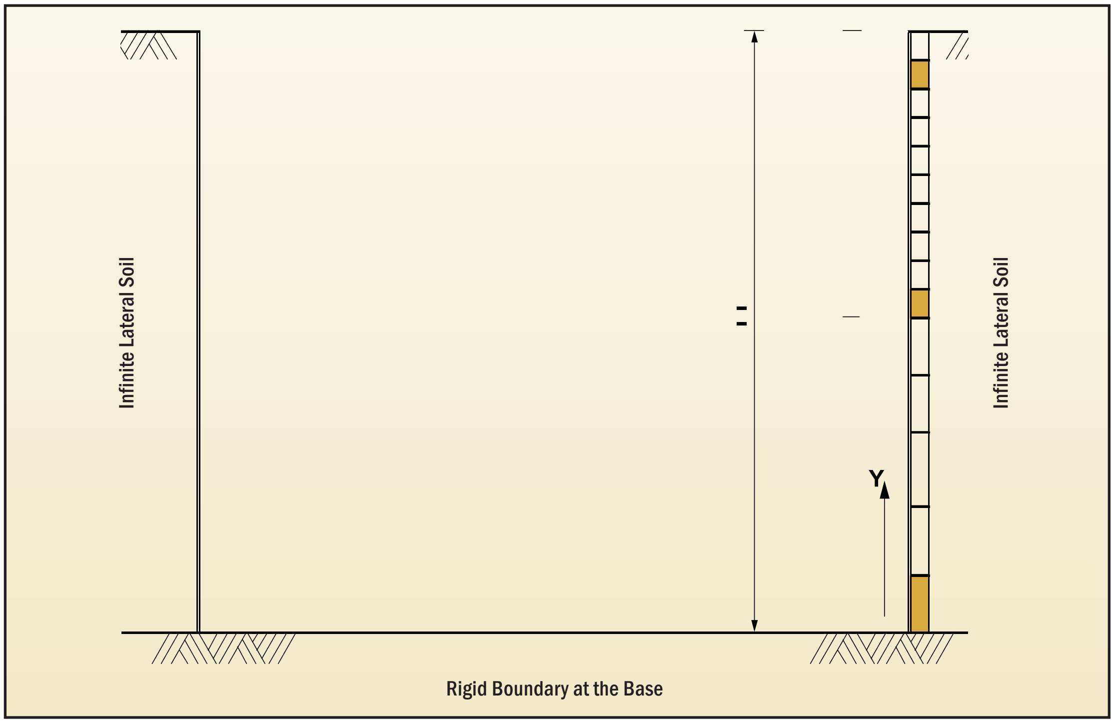

Figure 1. Typical SASSI Model of the Foundation

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 several locations along the wall was monitored. To evaluate the frequency contents of the pressure response, the pressure transfer function (TF) amplitude was obtained. This consists of the ratio of the amplitude of the seismic soil pressure to the amplitude of the input motion (1 g harmonic acceleration in the free-field) for each harmonic frequency. The analyses were performed for a building with embedment of 15.2 m (50 ft) and soil shear wave velocities of 152, 305, 457, and 610 m/sec (500, 1,000, 1,500, and 2,000 ft/sec), all with the Poisson's ratio of 1/3. The material damping in the soil was specified to be 5 percent. The transfer function results for a soil element near top of the wall are shown in **Figure 2**. 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 dynamic characteristics of the seismic soil pressure are the same as the SDOF system.

To evaluate the frequency characteristics of each transfer function, the frequency axis was also normalized using soil column frequency *f*, which was obtained from the following relationship:

$$f = \frac{V_s}{4H} \qquad (1)$$

In the above equation, *Vs* is the soil shear wave velocity and *H* is the embedment depth of the building. The amplitude of soil pressure at low frequency was used to normalize the amplitude of the pressure transfer functions for all frequencies. The normalized transfer functions are shown in **Figure 3**. As can be seen, the amplification of the pressure and its frequency characteristics are about the same for the range of the shear wave velocities considered.

In all cases, the maximum amplification takes place at the frequency corresponding to the soil column natural frequency. The same dynamic behavior was also observed for all soil elements along the height of the walls.

Examining the dynamic characteristics of the normalized pressure amplitudes (such as those shown in Figure 3), it is readily evident that such characteristics are those of a single-degree-of-freedom (SDOF) system. Each response begins with a normalized value of one, increases to a peak value at a distinct frequency, and subsequently reduces to a small value at high frequency. Dynamic behavior of an SDOF system is completely defined by the mass, stiffness, and associated damping constant. It is generally recognized that response of an SDOF system is controlled by stiffness at low frequency, by damping at resonant frequency, and by inertia at high frequencies.

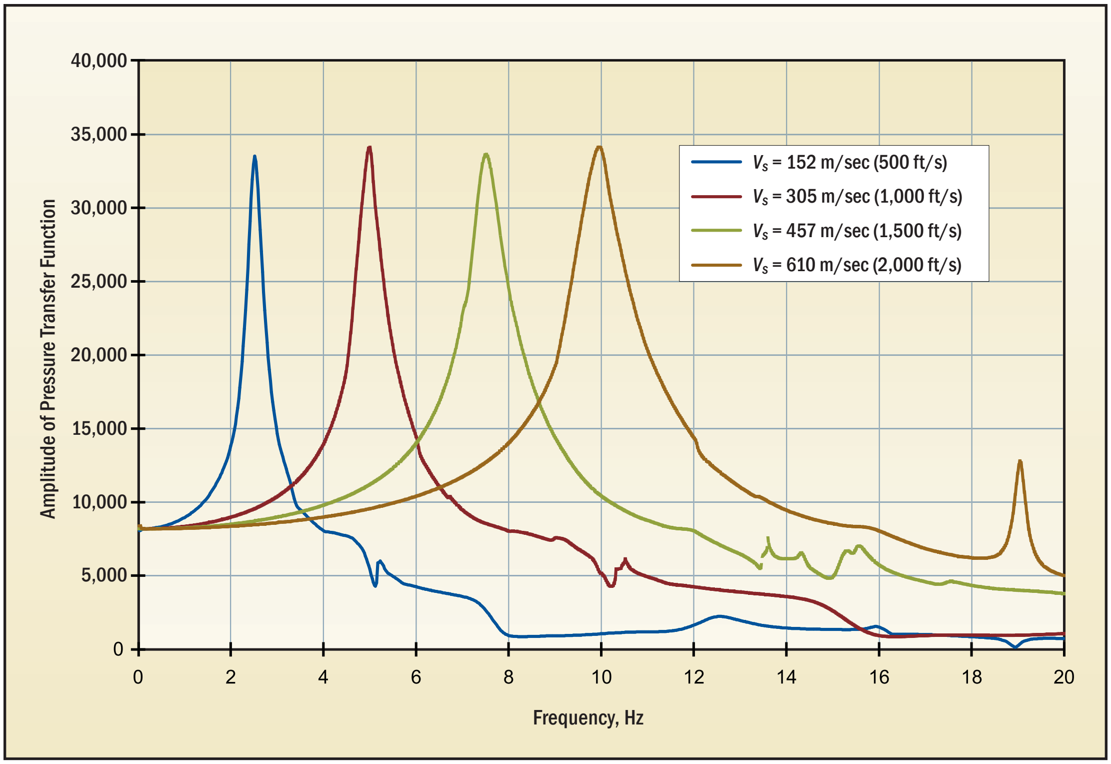

Figure 2. Typical Transfer Functions for Soil Pressure Amplitude

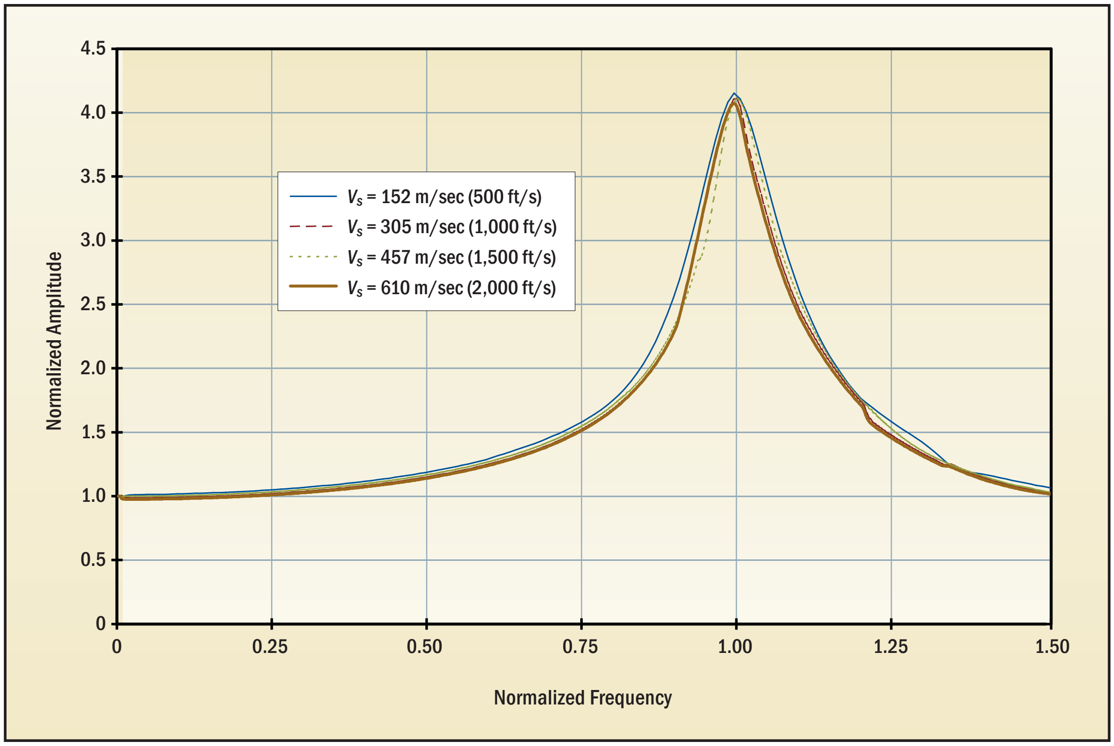

Figure 3. Normalized Transfer Functions

The analogy of an SDOF is used to formulate the new method for predicting seismic soil pressure.

Following the analogy for an SDOF system and to characterize the stiffness component, the pressure amplitudes at low frequencies for all soil elements next to the wall were obtained. The pressure amplitudes at low frequency are almost identical for the wide range of the soil shear wave velocity profiles considered, due to the long wave length of the scattered waves at such low frequencies. The shape of the normalized pressure was used as a basis for determining seismic soil pressure along the height of the building wall.

A similar series of parametric studies was also performed by specifying the input motion at the ground surface level [10]. The results of these studies also showed that the seismic soil pressure, in normalized form, could be represented by an SDOF system. For all cases considered, the low frequency pressure profiles depict the same distribution of the pressure along the height of the wall. This observation is consistent with the results of the analytical model developed by Veletsos et al. [12, 13] Since the SSI 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 et al. The Ψ_ν factor is defined by:

$$\Psi_\nu = \frac{2}{\sqrt{(1-\nu)(2-\nu)}} \qquad (2)$$

For the Poisson's ratio of 1/3, Ψ_ν is 1.897. Use of Ψ_ν in the formulation allows correction of the soil pressure amplitude for various Poisson's ratios. The adjusted soil pressure profile is compared with the normalized solution by Wood and the M-O method in **Figure 4**. In the proposed method, the maximum soil pressure is at the top of the wall. This is due to amplification of

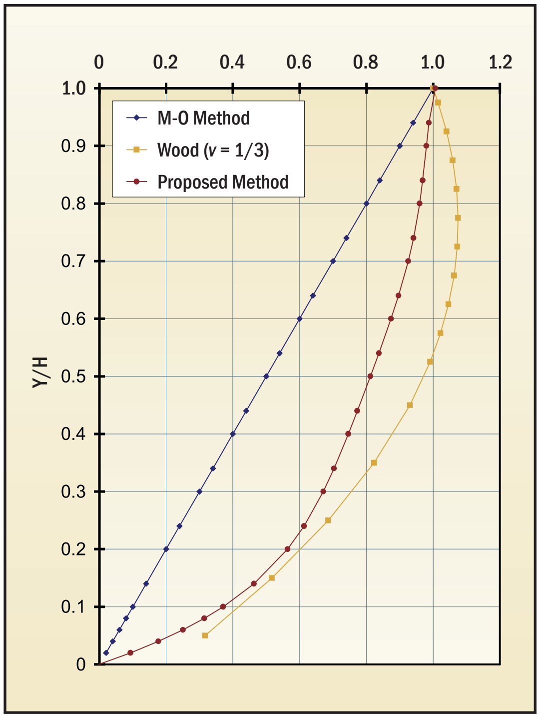

Figure 4. Comparison of Normalized Pressure Profiles

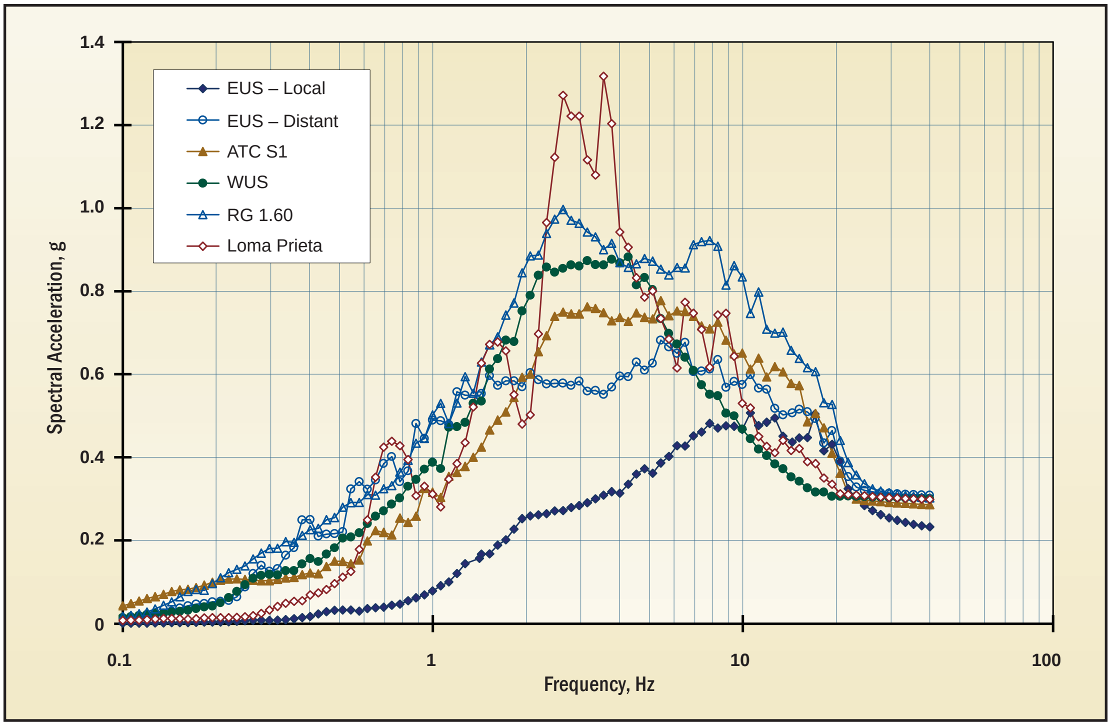

Figure 5. Motions Used in the Study

The proposed method is verified for typical design motions used for critical facilities.

the motion in the soil with highest amplification at ground surface level. This effect was not considered in the Wood solution.

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:

$$p(y) = -0.0015 + 5.05y - 15.84y^2 + 28.25y^3 - 24.59y^4 + 8.14y^5 \quad (3)$$

The area under the curve can be obtained by integrating the pressure distribution over the height of the wall. The total area is 0.744 H for a wall with a height of H. Having obtained the normalized shape of the pressure distribution, the amplitudes of the seismic pressure can also be obtained from the concept of an SDOF. The response of an SDOF system subjected to earthquake loading is readily obtained from the acceleration response spectrum of the input motion at the damping value and the 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 1 with wall height of 15 m (50 ft) and soil shear wave velocity of 457 m/sec (1,500 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 percent damping are shown in **Figure 5**.

The motions are typical design motions used for analyses of critical structures. From the set of six motions shown in Figure 5, two motions labeled EUS local and distant are the design motions for a site in the Eastern United States with locations close and far away from a major fault. The Applied Technology Council (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 the Western United States. The RG 1.60 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. All motions are scaled to 0.30 g and limited to frequency cut-off of 20 Hz for use in the analysis. This cut-off frequency reduces the peak ground acceleration of the EUS local motion to less than 0.30 g due to the high frequency content of this motion.

From the SASSI analysis results, the maximum seismic soil pressure from each element along the wall height was obtained for each of the input motions. The amplitude of the pressure changes from one motion to the other, with larger values associated with use of RG 1.60 motion. Using the computed pressure profiles, the lateral force acting on the wall for each input motion was computed. The lateral force represents the total inertia force of an 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 Equation 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 a depth of 15 m (50 ft) was computed for a wide range of damping ratios. 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 et al. [12] was used to compute the total mass and the damping of the SDOF system. For the total mass, the relationship is

$$m = 0.50\,\rho\,H^2\,\Psi_v \qquad (4)$$

where ρ is the mass density of the soil (total weight density divided by the acceleration of gravity), H is the height of the wall, and Ψ_v is the factor to account for the Poisson's ratio as defined in Equation 2. In the analytical model developed by Veletsos et al., a constant coefficient of 0.543 was used in the formulation of the total mass. Study of the soil pressure transfer functions and the free-field response motions at a depth of 15 m (50 ft) showed that spectral values at the soil column frequency at 30 percent damping have the best correlation with the forces computed directly from the SSI analysis. The high value of 30 percent damping is due to the radiation damping associated with soil-wall interaction. However, the spectral values of the motions at the depth corresponding to the base of the wall in the free-field are insensitive to the spectral damping ratios at the soil column frequency due to the dip in the response motion that appears in the acceleration response spectra at the soil column frequency (soil column missing frequency). The various motions, however, have significantly different spectral values at the soil column frequency, depending on the energy content of the design motion 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 a much less sensitive parameter. In practice, it is often warranted to consider the variation of soil properties typically using the best estimate and the lower and upper bound range of soil velocity profile. This, in effect, shifts the soil column frequency to a wider range.

Computational Steps

To predict the lateral seismic soil pressure for below-ground building walls resting on a firm foundation and assuming rigid walls (no significant deformation), the following steps should be taken:

1. Perform free-field soil column analysis and obtain the 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 percent damping should be obtained. The free-field soil column analysis may be performed using the Computer Program SHAKE [26] 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 response of the building and the seismic soil pressure amplitudes.
2. Use Equations 4 and 2 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 from Step 4 by the pressure distribution relationship shown in Equation 3.

One of the attractive aspects of the simplified method is its ability to consider soil nonlinear effects. Soil nonlinearity is commonly

The proposed method is based on a set of simple computational steps to develop an accurate seismic soil pressure profile for design.

considered by use of the equivalent linear method and the strain-dependent soil properties. Depending on the intensity of the design motion and the soil properties, the effect of soil nonlinearity can be important in changing the soil column frequency and, therefore, the amplitude of the spectral response at the soil column frequency.

The simplified method was verified extensively for a wide range of soil properties, wall heights, and design motions.

Accuracy of the Simplified Method

The simplified method outlined above was tested for building walls with the embedment depths of 4.6, 9, and 15.2 m (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. To depict the level of accuracy, typical comparisons for a 4.6 m (15 ft) wall with shear wave velocity of 152 m/sec (500 ft/sec) using the ATC motion, a 9.2 m (30 ft) wall with soil shear wave velocity of 305 m/sec (1,000 ft/sec) using the Loma Prieta motion, and a 15.2 m (50 ft) wall with soil shear wave velocity of 457 m/sec (1,500 ft/sec) using the EUS local motion are shown in **Figures 6, 7, and 8**, respectively. In all cases, the soil material damping of 5 percent and Poisson's ratio of 1/3 were used. These comparisons show a relatively conservative profile of seismic soil pressure predicted by the simple method as compared with a more rigorous solution. A comprehensive validation of the proposed method is presented in [10].

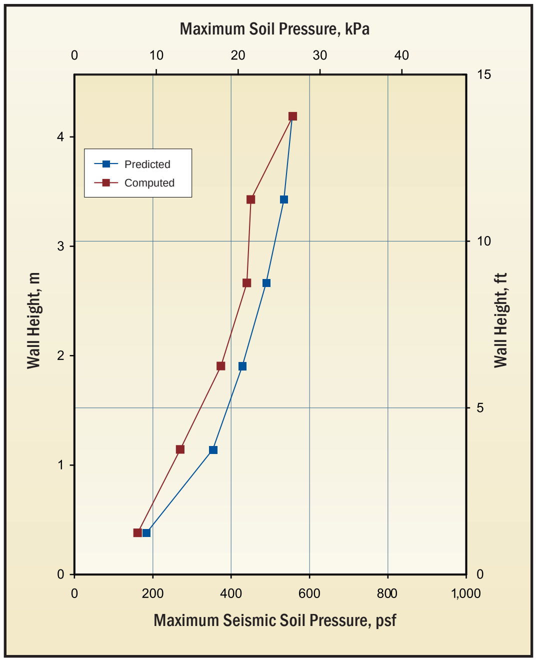

Figure 6. Predicted and Directly Computed Seismic Soil Pressure, 4.6 m (15 ft) Wall, V_s = 152 m/sec (500 ft/sec), ATC Motion

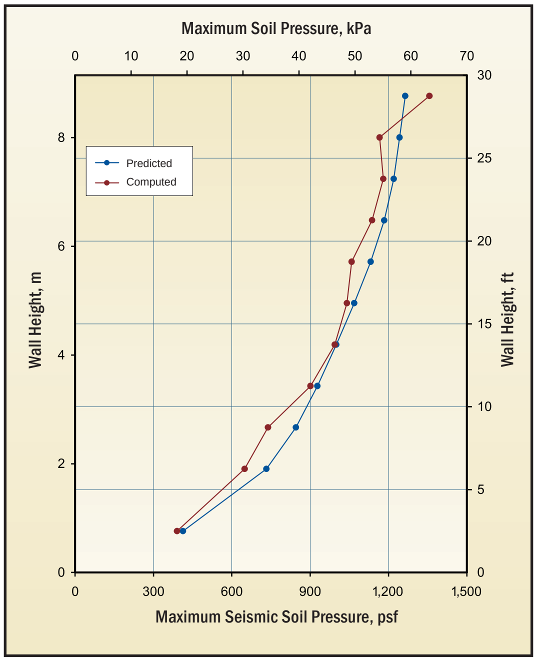

Figure 7. Predicted and Directly Computed Seismic Soil Pressure, 9.2 m (30 ft) Wall, V_s = 305 m/sec (1,000 ft/sec), Loma Prieta Motion

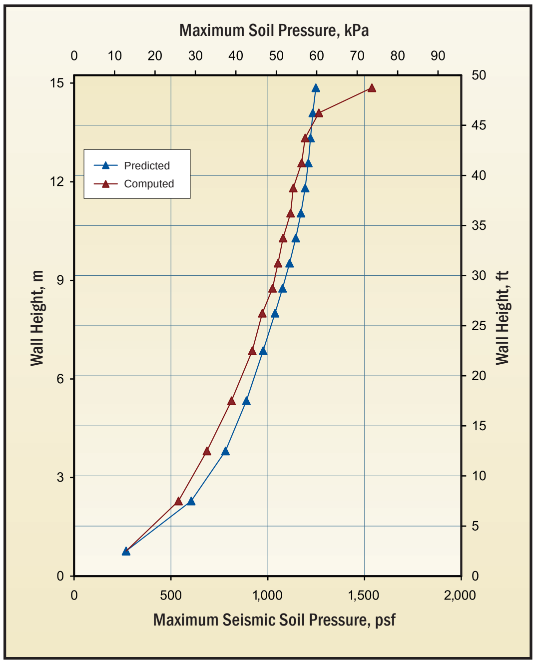

Figure 8. Predicted and Directly Computed Seismic Soil Pressure, 15.2 m (50 ft) Wall, V_s = 457 m/sec (1,500 ft/sec), EUS Local Motion

Comparison with Other Commonly Used Methods

The seismic soil pressure results obtained for a building wall 9.2 m (30 ft) high embedded in a soil layer with shear wave velocity of 305 m/sec (1,000 ft/sec) using the M-O, Wood, and proposed simplified methods are compared in Figure 7. For the simplified method, the input motions defined in Figure 5, all scaled to 0.30 g peak ground acceleration, were used.

The same soil shear wave velocity was used for all motions to compare the effects of frequency content of each motion on the pressure amplitude. In real application, the average strain-compatible soil velocity obtained from the companion free-field analysis would be used.

The M-O method and the Wood solution require only the peak ground acceleration as input, and each yields one pressure profile for all motions. For the M-O method, it is commonly assumed (although specified by neither Mononobe nor Okabe) that the seismic soil pressure has an inverted triangular distribution behind the wall. As shown in **Figure 9**, the M-O method results in lower pressure. This is understood, since this method relies on the wall movement to relieve the pressure behind the wall. The Wood solution generally results in the maximum soil pressure and is independent of the input motion as long as the peak acceleration is 0.30 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., RG 1.60 motion, the results of the proposed method are close to those of the Wood solution. There is a similar trend in the results of the various methods in terms of the magnitude of the total lateral force and the overturning moment.

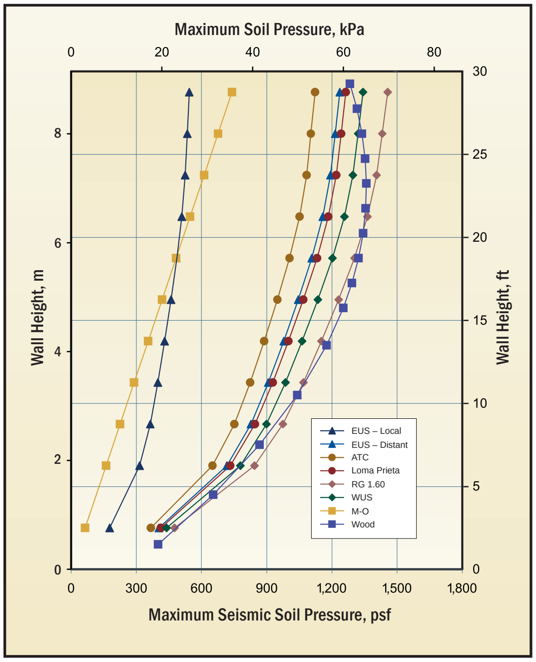

Figure 9. Maximum Seismic Soil Pressure for the 9.2 m (30 ft) Wall

CONCLUSIONS

Using the concept of an SDOF system, 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 of the design motion in its formulation. It was found that the controlling frequency that determines the maximum soil pressure is the one 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 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 simplified method has been adopted by design codes and standards such as the NEHRP standards and the ASCE standards for nuclear structures. ■

The simplified method has been adopted by design codes and standards, e.g., ASCE4-09 and NEHRP standards.

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REFERENCES

[1] N. Mononobe and H. Matuo, "On the Determination of Earth Pressures During Earthquakes," *Proceedings of World Engineering Congress*, Tokyo, Japan, Vol. 9, Paper 388, 1929.

[2] S. Okabe, "General Theory of Earth Pressures and Seismic Stability of Retaining Wall and Dam," *Journal of Japan Society of Civil Engineers*, Vol. 12, No. 1, 1924.

[3] H.B. Seed and R.V. Whitman, "Design of Earth Retaining Structures for Seismic Loads," ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell University, Ithaca, New York, June 22–24, 1970.

[4] R.V. Whitman, "Seismic Design and Behavior of Gravity Retaining Walls," *Proceedings of ASCE 1990 Specialty Conference on Design and Performance of Earth Retaining Structures*, Cornell University, Ithaca, New York, June 18–21, 1990, pp. 817–842, access via <http://cedb.asce.org/cgi/WWWdisplay.cgi?9002388>.

[5] R.V. Whitman and J.T. Christian, “Seismic Response of Retaining Structures,” POLA Seismic Workshop, San Pedro, California, 1990.

[6] R.V. Whitman, “Seismic Design of Earth Retaining Structures,” *Proceedings of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St. Louis, Missouri, March 11–15, 1991, pp. 1767–1778.

[7] R. Richards, Jr., and D.G. Elms, “Seismic Behavior of Gravity Retaining Walls,” ASCE, *Journal of Geotechnical Engineering*, Vol. 105, No. GT4, April 1979, pp. 449–464.

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

[9] R.M. Ebeling and E.E. Morrison, Jr., “The Seismic Design of Waterfront Retaining Structures,” US Army Corps of Engineers, Technical Report ITL-92-11, 1992.

[10] F. Ostadan and W.H. White, “Lateral Seismic Soil Pressure – An Updated Approach,” Bechtel Technical Grant report, 1997.

[11] J.H. Wood, “Earthquake Induced Soil Pressures on Structures,” Doctoral Dissertation, EERL 73-05, California Institute of Technology, Pasadena, California, 1973.

[12] A. Veletsos and A.H. Younan, “Dynamic Soil Pressure on Rigid Vertical Walls,” *Earthquake Engineering and Soil Dynamics*, Vol. 23, 1994, pp. 275–301.

[13] A. Veletsos and A.H. Younan, “Dynamic Modeling and Response of Soil-Wall Systems,” ASCE, *Journal of Geotechnical Engineering*, Vol. 120, No. 12, December 1994.

[14] ASCE 4-98, “Seismic Analysis of Safety-Related Nuclear Structures and Commentary,” American Society of Civil Engineers, 1998.

[15] NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2000 Edition, FEMA 369, March 2001.

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

[17] Electric Power Research Institute, “Proceedings: EPRI/NRC/TPC Workshop on Seismic Soil-Structure Interaction Analysis Techniques Using Data From Lotung, Taiwan,” EPRI Publication No. NP-6154, Two Volumes, March 1989.

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

[19] M. Hirota, M. Sugimoto, and S. Onimaru, “Study on Dynamic Earth Pressure Through Observation,” *Proceedings of 10th World Conference on Earthquake Engineering*, Madrid, Spain, July 1992.

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

[21] M. Watakabe, H. Matsumoto, Y. Fukahori, Y. Shikama, K. Yamanouchi, and H. Kuniyoshi, “Earthquake Observation of Deeply Embedded Building Structure,” *Proceedings of 10th World Conference on Earthquake Engineering*, Madrid, Spain, July 1992.

[22] T. Itoh and T. Nogami, “Effects of Surrounding Soils on Seismic Response of Building Basements,” *Proceedings of 4th US National Conference on Earthquake Engineering*, Palm Springs, California, May 20–24, 1990.

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

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

[25] J. Lysmer, F. Ostadan, and C.C. Chen, “SASSI2000 – A System for Analysis of Soil-Structure Interaction, Department of Civil Engineering, University of California, Berkeley, 1999.

[26] P.B. Schnabel, J. Lysmer, and H.B. Seed, “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.

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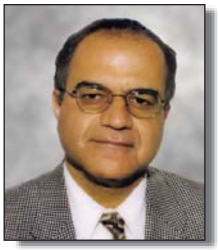
BIOGRAPHY



Farhang Ostadan, a Bechtel Fellow, has more than 25 years of experience in geotechnical and geotechnical earthquake engineering and foundation design. As chief soils engineer for Bechtel, he has overall responsibility for this discipline and manages the efforts of a large and diverse group of geotechnical specialists in locations across the US and around the globe. His project oversight responsibilities range from major transportation projects to petrochemical, nuclear, and power- and energy-related projects.

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