Seismic Earth Pressures on Deep Building Basements

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Abstract

The International Building Code and ASCE 7-05 require that earth retaining structures and basement walls be designed for seismic earth pressures. Although there are many documented failures of retaining structures during earthquakes, almost all are associated with some form of soil-related failure in loose or poorly compacted soils in waterfront or marine locations or associated with embankments, slope instability or liquefaction. On the other hand, there have been no reports of damage to building basement walls as a result of seismic earth pressures in recent United States earthquakes including the 1971 San Fernando, 1987 Whittier Narrows, 1989 Loma Prieta and 1994 Northridge earthquakes, or in the 1995 Kobe, Japan or 1999 Chi Chi, Taiwan earthquakes. However, despite the absence of compelling damage or failure due to seismic earth pressures, inclusion of seismic earth pressures is required in the design of earth retaining structures and basement walls in the current United States building code. Most geotechnical engineers estimate seismic earth pressures using the Mononobe-Okabe method of analysis developed in the 1920s based on model tests of walls with sand backfill on a small shake table. The results from the original Mononobe-Okabe method have been compared to more recent tests which allow superior geometric and material property scaling using wall and soil models shaken in a centrifuge. The centrifuge tests strongly suggest that the Mononobe-Okabe methodology does not properly model full scale conditions and may be extremely conservative in the predicted seismic earth pressures. In addition, many geotechnical engineers are uncertain about the various inputs to the Mononobe-Okabe method which adds more unpredictability in the reported results. The applicability of the Mononobe-Okabe method to non-sandy backfill is also an issue. Based on the recent research, provisional recommendations for the design of building basement walls are presented, and the impact on the structural design of the basements is discussed.

Introduction

The building code is generally a set of model code regulations that are designed to safeguard the public health and safety in all communities, large and small. The building code establishes minimum regulations for building systems using both prescriptive and performance-based provisions. For structural design, the building code prescribes minimum structural loading requirements for use in the design and construction of buildings and structural components. In dealing with soils and foundations, the building code provides criteria for the geotechnical and structural considerations in the selection and installation of adequate support for the loads transferred from the structure above and from the soil onto the structure (if applicable). The building code provisions are based on years of experience, observation, and judgment. In the case of seismic provisions, observations of damage or failure usually bring new regulations to prevent and mitigate such conditions in future construction. Although there is little or no evidence that significant damage or failure has occurred in deep building basements, the building code has evolved to require that building basements be designed for seismic earth pressures.

Performance of Deep Basement Walls in Recent Earthquakes

A summary of reports of damage to walls in recent earthquakes has been presented in Lew, Sitar and Al Atik (2010). Although there are reports of damage and failure of retaining walls due to earthquakes in the United States, the distress has been attributed to some form of soil or foundation failure, such as slope instability or soil liquefaction. There have been no reports of damage to building basement walls as a result of seismic earth pressures in recent U.S. earthquakes including the 1971 San Fernando, 1987 Whittier Narrows, 1989 Loma Prieta and 1994 Northridge earthquakes.

Similarly, while there are many failures of walls during foreign earthquakes outside of the United States, almost all are associated with some form of soil-related failure with many in marine or waterfront structures (Whitman, 1991; Huang, 2000; Tokida et al., 2001; Abrahamson et al., 1999). There was significant damage to subway stations in Kobe, Japan in the 1995 Hyogoken-Nambu earthquake (Iida, Hiroto, Yoshida and Iwafuji, 1996); however, there was no reported damage to building basements. The damage to and collapse of the Daikai Subway Station in Kobe appears related to the soil and high ground-water conditions at the site which strongly suggest that soil liquefaction had a significant role in the failure (Lew, Sitar and Al Atik, 2010). Also, Iida et al. reported that the subway station was not designed for racking conditions due to earthquake loading and information presented in the paper indicates that the concrete subway structure did not have sufficient ductility as columns had very minimal lateral ties. There were reports of damage to basements in two recent earthquakes in Turkey. Gur et al. reported that basement damage occurred in a half-buried basement of a school building during 1999 Düzce earthquake; the half-buried basement was surrounded by partial height earth-retaining concrete walls and there were windows between the top of the earth-retaining walls and the beams at the top of the basement. The exterior basement columns failed in shear at the level of the windows; although Gur et al. reported that damage occurred to masonry infill walls in the basement of the building, there was no mention of damage to the earth-retaining concrete walls of the basement. Gur et al. also reported on light damage to lateral basement walls of a building in the 2003 Bingöl, Turkey earthquake; the buildings experienced significant structural damage and collapse above the basement and the maximum horizontal ground accelerations in Bingöl were reported as being 0.55g.

Although not building basement walls, Clough and Fragaszy (1977) reported on a study of floodway channels in the San Fernando Valley that experienced strong ground motions from the February 9, 1971 San Fernando earthquake. They reported that no damage occurred to cantilever channel walls until accelerations of about 0.5g were reached, which was a surprisingly large value of acceleration in view of the fact that the walls were not explicitly designed for seismic loadings.

Observations were also made of a few deep basement walls in Chile after the February 27, 2010 magnitude 8.8 Offshore Maule earthquake. No damage was observed by the first author. Figure 1 shows a portion of the undamaged basement wall of the 55-story Torre Titanium La Portada in Santiago at its lowest subterranean level of -7. There was no observed or reported damage in any of the seven subterranean levels.

Figure 1 Level -7 Basement Wall of Torre Titanium La Portada in Santiago, Chile after February 27, 2010 earthquake

Figure 2 shows the undamaged basement wall of the Echeverria Izquierdo building, also in Santiago, after the

February 27, 2010 earthquake; this building has nine subterranean levels below grade. There was no observed or reported damage to any of the nine subterranean levels.

Figure 2 Level -9 Basement Wall of Echeverria Izquierdo Building in Santiago, Chile after February 27, 2010 earthquake

It was reported by Professor G. Rodolfo Saragoni of the University of Chile that there were no observations of damage to basement walls in any major buildings in Chile in the earthquake (Saragoni, 2010).

Building Code Provisions Requiring Design for Seismic Earth Pressures in the United States

The current edition of the International Building Code (IBC, 2009) adopts by reference the seismic requirements of the Minimum Design Loads for Buildings and Other Structures (commonly known as "ASCE 7-05") published by the American Society of Civil Engineers (2006). ASCE 7-05 states that all earth retaining structures assigned to Seismic Design Category D, E or F should determine the lateral earth pressures due to earthquake ground motion in accordance with Section 11.8.3, which simply states that the geotechnical investigation report should include "…the determination of lateral pressures on basement and retaining walls due to earthquake motions."

Despite the lack of compelling evidence that seismic earth pressures are a major concern to deep building basements, how is it that the building code in the United States now requires consideration of seismic earth pressures?

The answer may go back to a Specialty Conference on Lateral Stresses in the Ground and Design of Earth-Retaining Structures held in 1970 containing state-of-the-art papers. One of these papers was the landmark paper on "Design of Earth Retaining Structures for Dynamic Loads" by Seed and Whitman (1970) which brought awareness of seismic earth pressures to the geotechnical community.

The first regulatory document that incorporated the concept of seismic earth pressures was the California Building Code (CBC), which was based on the Uniform Building Code. The CBC had jurisdiction over hospitals and public schools (K-12 and community colleges), as well as state-owned public buildings, but did not apply to other buildings and structures in California. The CBC did have provisions that included the consideration of the seismic increment of active earth pressure. As early as the 1980s, the California amendments to the Uniform Building Code (UBC) had provisions mandating that the seismic increment of active earth pressure should be applied to buildings with walls that retain earth having exterior grades on opposite sides differing by more than 6 feet; this provision is shown below from Section 2312 (e) 1 E of the California amendments to the 1988 UBC:

> *Seismic increment of active earth pressure. Where buildings provide lateral support for walls retaining earth, and the exterior grades on opposite sides of the building differ by more than 6 feet, the load combination of the seismic increment of active earth pressure due to earthquake acting on the higher side, as determined by a civil engineer qualified in soil engineering plus the difference in active earth pressures shall be added to the lateral forces provided in this section.*

The identical language was still present in the 2001 edition of the CBC (California amendments to the 1997 UBC) (California Building Standards Commission, 2002 and International Conference of Building Officials, 1997). In addition, the 2001 edition of the CBC had the following amendment to Section 1611.6 of the 1997 UBC regarding retaining walls:

> *Retaining walls higher than 12 feet (3658 mm), as measured from the top of the foundation, shall be designed to resist the additional earth pressure caused by seismic ground shaking.*

From the context of these two CBC amendments to the UBC, the former amendment clearly refers to building basement walls and the latter amendment refers to free-standing retaining walls as UBC Section 1611.6 describes the features of a retaining wall in some detail.

The California consideration of seismic earth pressures, despite its limited reach, probably had an influence on the development of national guidelines being developed under the National Earthquake Hazards Reduction Program (NEHRP). The "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450)," 2003 Edition, Part 1 – Provisions, also known as the FEMA 450 report (Building Seismic Safety Council, 2004a), was intended to form the framework for future model building codes in the United States. The provisions did not contain any explicit recommended provisions for accounting of seismic earth pressures for design of retaining walls in the recommended provisions. However, Part 2 – Commentary of the FEMA 450 report (Building Seismic Safety Council, 2004b) contains almost four pages of commentary on the consideration of lateral pressures on earth retaining structures. Section 7.5.1 of the commentary states that "In addition to the potential site hazards discussed in *Provisions* Sec. 7.4.1, consideration of lateral pressures on earth retaining structures shall be included in investigations for Seismic Design Categories D, E, and F." The NEHRP provisions were an important resource to the development of ASCE 7-05, which is referenced in the IBC.

State of Practice for Evaluation of Seismic Earth Pressures on Building Basement Walls

As mentioned above, the initial impetus for ultimate inclusion of seismic earth pressures into the present building code provisions probably dates back to the Seed and Whitman (1970) paper which essentially brought to the forefront the concept of designing for loads on walls due to earthquakes. In this paper, they highlighted the so-called Mononobe-Okabe seismic coefficient analysis (Mononobe and Matsuo, 1929 and Okabe, 1926). This method has been the predominant method used by geotechnical engineers to evaluate seismic earth pressures.

The Mononobe-Okabe (M-O) method is based on Mononobe and Matsuo's (1929) experimental studies of a small scale cantilever bulkhead hinged at the base with a dry, medium dense cohesionless granular backfill excited by a one gravity (1g) sinusoidal excitation on a shaking table. The test set up is shown in Figure 3. Note that the walls are hinged at the base and are not allowed to move laterally.

The M-O method assumes that the Coulomb theory of static earth pressures on a retaining wall can be modeled to include the inertial forces due to ground motion (in the form of horizontal and vertical acceleration) in the retained earth as shown in Figure 4.

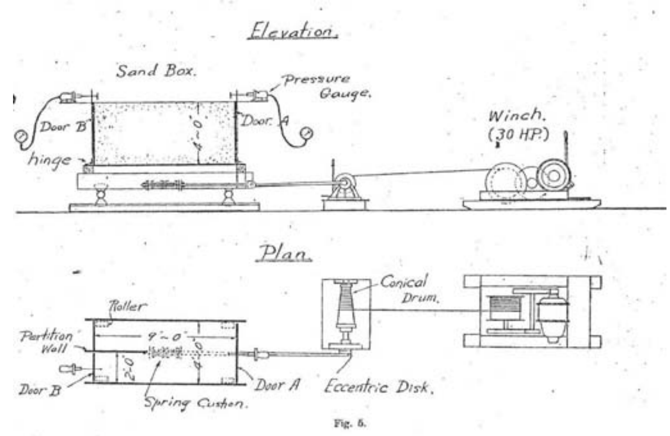

Figure 3 Test Setup for Shake Table Test (After Mononobe and Matsuo, 1929)

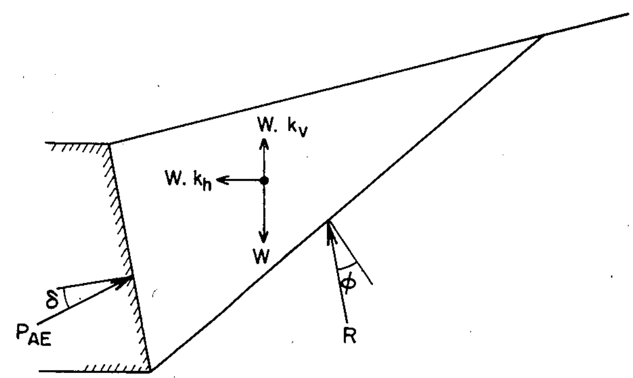

Figure 4 Forces considered in the Mononobe-Okabe Analysis (after Seed and Whitman, 1970)

The M-O method was developed for dry cohesionless materials with the following assumptions:

1. The wall yields sufficiently to produce minimum active pressures.
2. When the minimum active pressure is attained, a soil wedge behind the wall is at a point of incipient failure and the maximum shear strength is mobilized along the potential sliding surface.
3. The soil behind the wall behaves as a rigid body so that accelerations are uniform throughout the mass. The effect of the earthquake motions is represented by inertia forces $W \cdot k_h$ and $W \cdot k_v$, where W is the weight of the wedge of soil and k_h and k_v are the horizontal and vertical components of the earthquake accelerations at the base of the wall.

Thus, the active pressure during the earthquake, P_{AE}, is computed by the Coulomb theory except that the additional forces, $W \cdot k_h$ and $W \cdot k_v$, are included. For the critical sliding surface, the active pressure is expressed in the following equation:

$$P_{AE} = (1/2)\,\gamma H^2\,(1-k_v)\,K_{AE} \tag{1}$$

where

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

θ = $\tan^{-1}[k_h / (1-k_v)]$

γ = unit weight of soil

H = height of wall

ϕ = angle of internal friction of soil

δ = angle of wall/soil friction

i = slope of ground surface behind wall

β = slope of back of wall with respect to vertical

k_h = horizontal ground acceleration/g

k_v = vertical ground acceleration/g

Seed and Whitman state that Mononobe and Okabe apparently considered that the total pressure computed by their analytical approach would act on the wall as the same location as the initial static pressure; i.e., the resultant would act at a height of H/3 above the base.

Seed and Whitman also state in their state-of-the-art paper that for most earthquakes, "…the horizontal acceleration components are considerably greater than the vertical acceleration components…" Thus they concluded that k_v could be neglected for practical purposes. For practical purposes, Seed and Whitman proposed to separate the total maximum earth pressure into two components – the initial static pressure on the wall and the dynamic pressure increment due to the base motion. The total dynamic earth pressure coefficient, K_{AE}, could be written as:

$$K_{AE} = K_A + \Delta K_{AE} \tag{2}$$

and the dynamic lateral force component would be:

$$\Delta P_{AE} = (1/2)\,\gamma H^2\,\Delta K_{AE} \tag{3}$$

Seed and Whitman gave an approximation for ΔK_{AE} as:

$$\Delta K_{AE} \sim (3/4)\,k_h \tag{4}$$

Then the simplified dynamic lateral force component on yielding walls is given by:

$$\Delta P_{AE} \sim (1/2)\,(3/4)\,k_h\,\gamma H^2 = (3/8)\,k_h\,\gamma H^2 \tag{5}$$

where k_h is the "horizontal ground acceleration divided by gravitational acceleration." This simplified equation is also presented in the FEMA 450 report commentary (BSSC, 2004b). It is recommended that k_h be taken as equal to the site acceleration that is consistent with the design ground motions as defined in the provisions of FEMA 450 (i.e., $k_h = S_{DS}/2.5$); where S_{DS} is the design, 5-percent-damped, spectral response acceleration parameter at short periods (i.e., period of 0.2 seconds). Seed and Whitman recommended that the resultant dynamic thrust be applied at 0.6H above the base of the wall (i.e., similar to an inverted triangular pressure distribution).

In contrast to the M-O method which is a limit-equilibrium force approach, other methods of analysis based on tolerable displacements are also available. These methodologies, however, are not as widely used. For nonyielding walls, Whitman (1991) recommended the approach of Wood (1973) who analyzed the response of a rigid nonyielding wall retaining a homogeneous linear elastic soil and connected to a rigid base. Whitman recommended that the point of application of the dynamic thrust also be taken at a height of 0.6H above the base of the wall with the dynamic thrust on a nonyielding wall, ΔP_E, taken as:

$$\Delta P_E = k_h\,\gamma H^2 \tag{6}$$

The present state-of-practice for evaluation of seismic earth pressures on building basement walls by geotechnical engineers in the United States is generally to rely upon an analysis based on the Mononobe-Okabe (M-O) method of analysis regardless of whether the wall is considered yielding or nonyielding. It could be argued that deep building basement walls are constructed in open excavations that generally are shored which cause the retained soils to be in a yielded (active) condition already. The reasons for using the M-O method appear to be the simplicity of the method requiring only knowledge of the wall and backfill geometry, the soil's angle of internal friction, and the horizontal and vertical ground acceleration.

Is the Mononobe-Okabe Method Applicable to Building Basement Walls?

Although the Mononobe-Okabe method appears simple to use, the validity of the method for evaluation of seismic earth pressures has been questioned by some. Also, the M-O method contains some limiting assumptions and there are questions about the proper input into the method.

The original tests that formed the basis for the M-O method were conducted on a sand filled box shaking table with hinged doors (which were the "walls") as shown in Figure 3. One of the basic questions that arise is: Do the conditions in the M-O test properly model a real building basement wall?

The configuration of the "walls" in the Mononobe and Matsuo (1929) test apparatus do not model the building basement wall condition properly. Listed below are some of the physical incongruities:

1. The walls in the Mononobe and Matsuo test are hinged at the bottom of the wall, thus allowing only for rotation and not for horizontal movement.
2. The walls in the Mononobe and Matsuo test have a free edge at the top, not a fixed or a pinned edge as is the case in the intermediate or top levels of a building basement wall.
3. The physical scaling of the test wall may not be applicable to a full size basement wall.

Ostadan and White (1998) have stated that "…the M-O method is one of the most abused methods in the geotechnical practice." Ostadan and White listed some reasons why they believe the M-O method is abused:

1. The walls of buildings are often of the non-yielding type. Wall movement may be limited due to the presence of floor diaphragms and displacements to allow limit-state conditions are unlikely to develop during the design earthquake.
2. The frequency content of the design ground motion is not fully considered since a single parameter (peak ground acceleration) may misrepresent the energy content of the motion at frequencies important for soil amplifications.
3. Appropriate soil properties are not considered as they are for soil dynamic problems, the most important property is the shear wave velocity, followed by the material damping, Poisson's ratio, and then the density of the soil.
4. Soil nonlinearity effects are not considered.
5. Soil-structure interaction (SSI) is not considered, such as building rocking motion, amplification and variation of the motion in the soil, geometry, and embedment depth of the building.

Despite the differences between the model cantilevered wall and actual building basement walls, the Mononobe-Okabe method continues to be used in practice and its use is actually encouraged by documents such as FEMA 450.

Areas of Confusion in Using the Mononobe-Okabe Method

A major area of confusion to geotechnical consultants is what to specify as the ground acceleration in the M-O method. Whitman (1991) had recommended that except where structures were founded at a sharp interface between soil and rock, the M-O method should be used with the actual expected peak acceleration. In keeping with this view, the seismic coefficient, k_h, is being recommended in future NEHRP documents to be equal to the site peak ground acceleration that is consistent with the design earthquake ground motions. In high seismic regions, such as California, these peak ground motions could easily exceed 0.5g. However, Kramer (1996) refers to the M-O method as a "pseudostatic procedure" and these accelerations as "pseudostatic accelerations." Arulmoli (2001) comments on the use of the M-O method and states that it has limitations, including the observation that the M-O method "blows up" for cases of large ground acceleration. In practice, many geotechnical engineers have been using a seismic coefficient that is less than the expected peak ground acceleration for the design of building basement walls and other walls. The reasons for the reduced value of the seismic coefficient compared to the peak ground acceleration are due to the following considerations:

1. The M-O method is a pseudo-static method of analysis, similar to many traditional slope stability methods that use a pseudo-static coefficient to represent earthquake loading.
2. There should be an intuitive reduction based upon the use of an effective ground acceleration rather than an isolated peak ground acceleration (to take into effect the "repeatable" ground motion).
3. There should be a reduction to account for the averaging of the lateral forces on the retaining wall over the height of the wall (because of the potentially out-of-phase nature of the ground movement as shear waves propagate vertically through the backfill soil; this effect increases with increasing height of the wall and reduced stiffness of the retained soils).

The justification for many geotechnical engineers for the use of a reduced seismic coefficient comes from a Federal Highway Administration (FHWA) design guidance document for design of highway structures (Kavazanjian, Matasović, Hadj-Hamou, and Sabatini, 1997). In this document, it is stated that "…for critical structures with rigid walls that cannot accommodate any deformation and partially restrained abutments and walls restrained against lateral movements by batter piles, use of the peak ground acceleration divided by the acceleration of gravity as the seismic coefficient may be warranted." The document goes on to further state that "…however, for retaining walls wherein limited amounts of seismic deformation are acceptable…, use of a seismic coefficient between one-half to two-thirds of the peak horizontal ground acceleration divided by gravity would appear to provide a wall design that will limit deformations in the design earthquake to small values." Thus many geotechnical engineers have been using a seismic coefficient of one-half of the horizontal peak ground acceleration.

Another area of confusion for geotechnical engineers is how to account for cohesion in the backfill or retained earth behind the building basement wall. The assumption in the M-O method is that the backfill material is a medium dense cohesionless soil. However, it is commonplace to have backfill material or retained earth that has some cohesion and the M-O method simply does not account for any cohesion at all following Coulomb's assumptions. All geotechnical engineers know that cohesion in the soil can reduce the static lateral earth pressures and that some excavations can stand vertically without support if there is sufficient cohesion in the soil. It seems logical that since soil cohesion reduces the active lateral earth pressure, it would also reduce the lateral seismic pressures. A very recent National Cooperative Highway Research Program (NCHRP) report (Anderson, Martin, Lam and Wang, 2008) provides guidance for use of the M-O method for soils with cohesion. Anderson et al. state that most natural cohesionless soils have some fines content that often contributes to cohesion, particularly for short-term loading conditions. Similarly, cohesionless backfills (for highway structures) are rarely fully saturated, and partial saturation would provide for some apparent cohesion, even for clean sands.

Figures 5 through 8 present active earth pressure coefficient charts for four different soil friction angles with different values of cohesion for horizontal backfill, assuming no tension cracks and wall adhesion. These charts show that a small amount of cohesion would have a significant effect in reducing the dynamic active earth pressure for design. Figures 5 and 6 were provided by Dr. Geoffrey R. Martin (2010) and Figures 7 and 8 are found in Anderson et al. (2008).

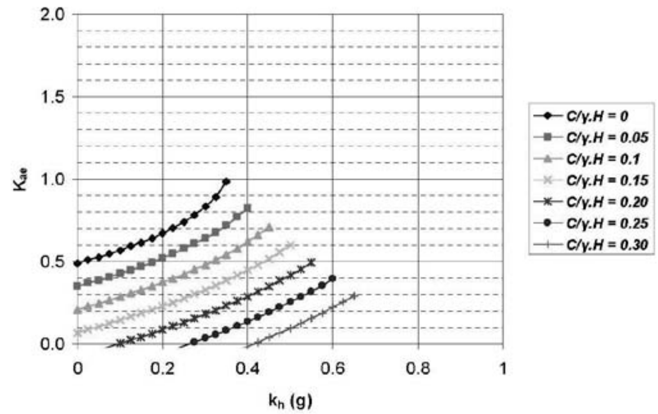

Figure 5 Seismic coefficient chart for c-ϕ soils for angle of internal friction of 20 degrees (Courtesy of Dr. Geoffrey R. Martin)

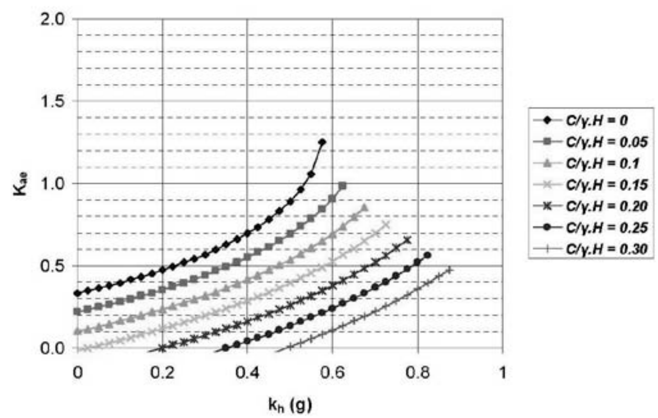

Figure 6 Seismic coefficient chart for c-ϕ soils for angle of internal friction of 30 degrees (Courtesy of Dr. Geoffrey R. Martin)

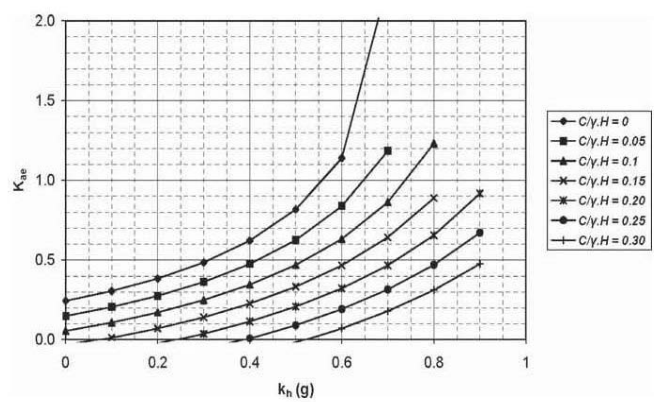

Figure 7 Seismic coefficient chart for c-ϕ soils for angle of internal friction of 35 degrees (after Anderson et al., 2008).

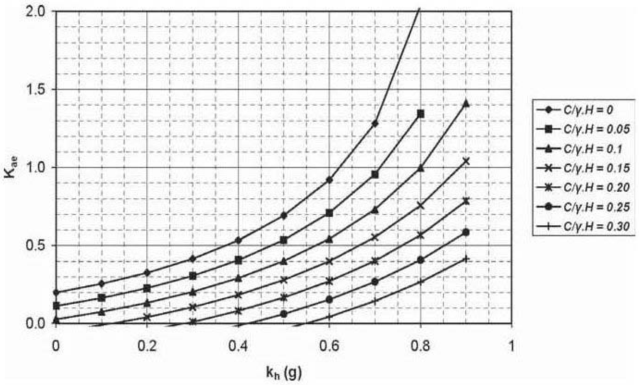

Figure 8 Seismic coefficient chart for c-ϕ soils for angle of internal friction of 40 degrees (after Anderson et al., 2008).

Validity of the Mononobe-Okabe Method

The Mononobe-Okabe method is based on the response of a small scale cantilever bulkhead that is hinged at the bottom which retained a dry, medium dense cohesionless backfill, and was excited by a one gravity (1g) sinusoidal input on a shaking table that was 4 feet high, 4 feet wide, and 9 feet long, as shown in Figure 3. It is natural to ask the following questions: Can the M-O method be applied to large building basement walls that may be an order of magnitude larger (or greater) in height? Were the conclusions in developing the M-O method based on observations that can be extrapolated to larger structures? Was the backfill material the suitable material to use in the test? Questions can be raised regarding the validity using the M-O method for basement walls.

Concerned about proper scaling of results in smaller model tests, researchers have turned to centrifuge testing which can simulate correct boundary and load conditions on large prototype structures. Centrifuge testing allows for creating a stress field in a model that simulates prototype conditions in that proper scaling will provide correct strength and stiffness in granular soils. The granular soils, when having a scale model with dimensions of 1/N of the prototype and a gravitational acceleration during spinning of the centrifuge at N times the acceleration of gravity, will have the same strength, stiffness, stress and strain of the prototype (Kutter, 1995).

An early centrifuge test of a cantilever retaining wall subjected to a model acceleration history similar to the characteristics of real earthquake ground shaking was conducted by Ortiz, Scott and Lee (1983) to verify the M-O theory. An important conclusion was that "it is difficult or impossible to achieve in a (one-g) shaking table a pressure distribution which can be related quantitatively to that of the full-scale situation." Ortiz et al. also use dimensional analysis to show that "true representation of the dynamic prototype behavior cannot be attained in a (one-g) shaking table experiment, utilizing a reduced scale model and same soil as the prototype." An important finding of Ortiz et al. was that "…under dynamic loading, the resultant acts very near to the where the static one acted." They also concluded that "…the earth pressure distributions are not linear with distance down the wall although a linear earth pressure distribution seems to be a reasonable "average" for the actual."

In Japan, Nakamura (2006) also sought to reexamine the M-O theory by centrifuge testing. An important finding by Nakamura was that the earth pressure distribution on the model gravity retaining wall is not triangular (as assumed by M-O), and that its size and shape will change with time. Nakamura also found that the earth pressure distribution for an input motion that was based on actual earthquake ground shaking was different from the distribution for sinusoidal shaking. The earth pressure in the bottom part of the wall, which greatly contributes to the total earth pressure, is not as great in earthquake loading as it is for sinusoidal loading. Nakamura stated that the earth pressure increment is around zero when considering earthquake-type motions, with the earth pressure nearly equal to the initial value prior to shaking when the inertia force is maximum. Nakamura's tests show that the earth pressure distributions at the time of maximum moment in the gravity wall generally increases with depth.

Another centrifuge study was conducted by Al Atik and Sitar (2007) on model cantilever walls with medium dense dry sand backfill. Al Atik and Sitar found that the maximum dynamic earth pressures increase with depth and can be reasonably approximated by a triangular distribution analogous to that used to represent static earth pressure. They also found that the seismic earth pressures can be neglected at accelerations below 0.4g and stated that the data suggest that even higher seismic loads could be resisted by cantilever walls designed to an adequate factor of safety. As the tests were conducted with medium sand backfill, they state that a severe loading condition may not occur in denser granular materials or materials with some degree of cohesion. Al Atik and Sitar also found that the maximum moment in the wall and the maximum earth pressure were out of phase and did not occur at the same time. Based on their research, Al Atik and Sitar (2009, 2010) developed relationships for the "Dynamic Increment in Earth Pressure Coefficient, ΔK_{ae}," as defined by Seed and Whitman (1970) computed from the dynamic earth pressures at the time that maximum wall moments based on strain gauge data occur as shown in Figure 9. This research illustrates that the seismic earth pressures in the M-O method are very conservative if the actual peak ground acceleration is used.

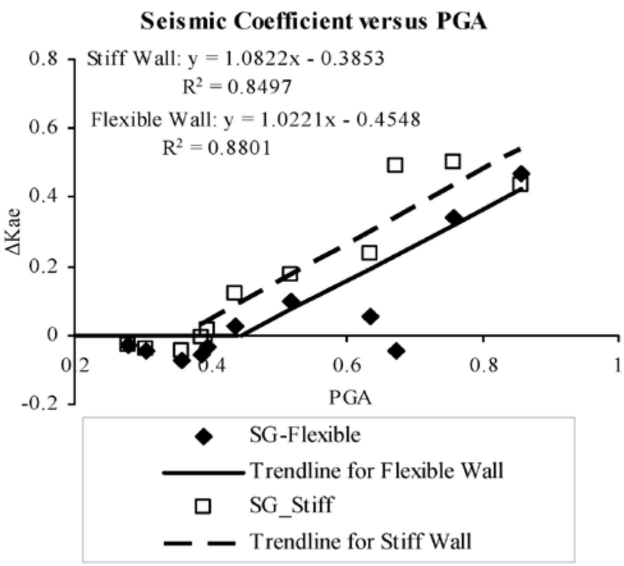

Figure 9 Dynamic Increment in Earth Pressure Coefficient, ΔK_{ae}, computed at maximum dynamic wall moments based on strain gauge data (after Al Atik and Sitar, 2009)

One issue that needs to be addressed is the moment of inertia of the wall which can contribute to dynamic wall moments. This should not be a concern for building basement walls as they generally are very constrained by floor systems and interior walls that prevent much movement of the walls that would contribute to inertial forces. However, this should be a concern for free standing walls and should be accounted for in the design.

Thus the validity of the Mononobe-Okabe method is severely questioned by the results of these various centrifuge studies. These studies also strongly suggest that the seismic earth pressures predicted by the M-O method can be very conservative. Also the location of the resultant of the static and seismic earth pressures is closer to the one-third height from the base of the wall and not in the upper wall as recommended by many researchers.

Provisional Recommendations for Design of Building Basement Walls

Although there is evidence that seismic earth pressures may not actually develop as predicted by the M-O method, it may be premature to recommend that seismic earth pressures be neglected in design altogether. It would be prudent to wait upon further research that may be conducted to confirm the observations and conclusions that have been made by recent researchers. In the interim, presented below are provisional recommendations for the evaluation of seismic earth pressures for building basement walls.

It should be noted that the current International Building Code requires that basement walls be designed for at-rest earth pressures for static conditions. The M-O method on the other hand is based on computing active lateral earth pressures in combination with the seismic lateral earth pressure. Thus, the seismic increment of lateral earth pressure computed by the M-O method is intended to be the increased earth pressure above the active lateral earth pressure and not the at-rest pressure. As such, any computed seismic increment of lateral earth pressure should not be added to the static (at-rest) lateral earth pressures. For seismic conditions, the M-O method may be used to evaluate the seismic earth pressures; however, the combination should be made with the active pressures. These pressures should be treated as a separate condition for earthquake loading whereas the at-rest earth pressures are strictly for static loading only. Recent research suggests that the earth pressure distribution under seismic loading is very similar to a fluid distribution (i.e., triangular distribution), like static earth pressure.

Presented below are general provisional recommendations for building basement walls founded in non-saturated conditions with level ground or retained earth conditions:

- If the depth of the basement wall is less than 12 feet, the evaluation of seismic earth pressures is not necessary provided the walls are designed for a static factor of safety of at least 1.5. As described in the following section, this static factor of safety is satisfied when a load factor of 1.6 is used in loading combination for lateral earth pressures as is currently prescribed by the code.
- The seismic increment of earth pressure may be neglected if the maximum ground acceleration is 0.4g or less.
- If a seismic increment of earth pressure is determined separately by the M-O method, it should be added to the active earth pressure and not to the at-rest static earth pressure.
- If the backfill or retained earth materials are cohesive (including cemented soils and stiff clays), the NCHRP design charts (shown in Figures 5 to 8) may be used to determine the seismic coefficient, K_{AE}, in the M-O method. The horizontal ground acceleration, k_h, may be taken as one-half of the PGA, where PGA is the maximum ground acceleration in gravity.
- If the backfill or retained earth materials are cohesionless, the "Dynamic Increment in Earth Pressure Coefficient," ΔK_{AE}, may be determined directly from the Figure 9 for use in Equation (3). As an alternative, the horizontal ground acceleration may be conservatively estimated from Table 1.

- The location of the resultant of the active and seismic earth pressures may be taken at the one-third point from the base of the wall.

Table 1 Horizontal Ground Acceleration for Cohesionless Backfill or Retained Earth (1)

Peak Ground Acceleration (g)	Recommended k_h
< 0.4	0
0.4	0.25 PGA
0.6	0.5 PGA
1.0	0.67 PGA

(1) For other levels of peak ground acceleration, interpolation of the tabulated values may be used.

Comments on Factored Loads Using Strength Design or Load and Resistance Factor Design

The International Building Code prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and ground water pressure, the IBC prescribes the basic combinations shown in equations (7) and (8) below. Equation (9) indicates the IBC loading combination including earthquake and live load components:

$$1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \quad \text{[IBC Eq. 16-2]} \quad (7)$$

$$0.9D + 1.0E + 1.6H \quad \text{[IBC Eq. 16-7]} \quad (8)$$

$$1.2D + 1.0E + f_1 L + f_2 S \quad \text{[IBC Eq. 16-5]} \quad (9)$$

where

D = dead load
E = earthquake load
F = load due to fluids with well-defined pressures and maximum heights
f_1 = 1 for floors in public assemblies, live loads exceeding 100 psf and garage live load and
= 0.7 for other live loads
f_2 = 0.7 for roof configurations that do not shed snow and,
= 0.2 for other roof configurations
H = load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L = live load
L_r = roof live load
R = rain load
S = snow load
T = self-straining force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement or combinations thereof
W = wind load

From equation (7) it is evident that H, when due to lateral earth pressure, is treated in the same manner as the live load with a load factor of 1.6 for static loading conditions. The intent is to use a static lateral earth pressure in this equation which for most building basement walls will be the at-rest earth pressure. Therefore, from a static design perspective, the building basement walls have a factor of safety of at least 1.6 on the at-rest earth pressure. This satisfies the recommendation made in the previous section with regards to a minimum safety factor of 1.5.

Eq. (8) gives the load combination for seismic loading and lateral soil pressure while Equation (9) depicts the load combination including seismic and live loads. In comparing Eqs. (7) and (9) it is evident that a reduced live load factor (0.5 for typical range of live load and 1.0 for large live loads) is considered when live load combination with seismic loading is considered. The reason for this is the transitory nature of the seismic loading and the low likelihood of the two load maxima occurring simultaneously. A similar type of approach is warranted for load combinations including both the static soil pressures and the seismic increment of the soil.

If the Mononobe-Okabe analysis is used to determine the lateral seismic earth pressure, the lateral earth pressure should consist of the static active earth pressure and the seismic increment of earth pressure as discussed in the previous section. Presumably, the load factor of 1.6 in Eq. (8) would be applicable to the total earth pressure in this case. However, as noted above, a reduced load factor would be appropriate when considering the transitory nature of the seismic component and the low likelihood of the load maxima occurring simultaneously. Accordingly a lower load factor of 1.0 is proposed to be applied to the seismic increment component of earth pressure while the 1.6 load factor is applied to the static active pressure component. To facilitate such loading combination the geotechnical engineers would have to separate earth pressure components attributable to the active earth pressure condition and the seismic increment of earth pressure when using the M-O method.

Conclusions and Summary

When considering the load conditions given in IBC, it appears that building basement walls analyzed and designed using at rest pressures in accordance with the load combination in Eq. (7) may be adequate for seismic earth pressure loading without further analysis. The reason is the different types of earth pressures that must be considered for static versus seismic conditions. As noted above for the seismic load condition represented by Eq. (8), the active earth pressure combined with the seismic increment of earth pressure needs to be considered. Active earth pressures are typically much smaller than at-rest pressures which are commonly on the order of 1.6 to 2.0 times more. Thus as basement walls are conservatively designed for at-rest static pressures using loading combination in Eq. (7) it is very likely that the loading combination in Eq. (8) which is based on active pressures will be automatically satisfied unless the seismic increment of earth pressure is unusually large. With recent research (reported above) indicating that the seismic earth pressures are not as great as indicated by current practice, it would appear that building basement walls retaining level unsaturated earth materials may be considered adequate when just designed for at-rest earth pressures as stipulated in the IBC. Consequently, the current requirement in the seismic provisions to consider seismic earth pressures for such walls may be unnecessary. In retaining walls designed with active pressures, the addition of the seismic increment of soil using loading combination Eq. (8) should still be a consideration and will likely dictate the design of the wall, However, when applying Eq. (8) in this condition, it is recommended that a reduced load factor of 1.0 be used for the seismic increment component of soil in combination with a 1.6 load factor applied to the active pressure component These load factors will more appropriately represent the transitory nature of seismic loading and the low likelihood of load maxima occurring at the same time. To facilitate such loading combinations, the geotechnical engineers would have to separate earth pressure components attributable to the active earth pressure condition and the seismic increment of earth pressure when using the Mononobe-Okabe method.

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