# Seismic Earth Pressures: Fact or Fiction?

**Conference Paper** *in* Geotechnical Special Publication · July 2010

DOI: 10.1061/41128(384)66



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# Seismic Earth Pressures: Fact or Fiction

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## ABSTRACT

The current state of practice in the United States as implemented in the International Building Code now requires that structures with subterranean walls be designed for seismic earth pressures in addition to the normal static earth pressures. Since many design issues in the building code are introduced because of observed failure or deficiencies during earthquakes, the requirement to design for seismic earth pressures is peculiar because there is little or no evidence that any failures in engineered subterranean structures have occurred in past or even recent earthquakes. This paper examines how seismic earth pressures entered into the design practice and reviews some of the methodologies used to estimate seismic earth pressures in current engineering practice in the United States.

## INTRODUCTION

The building code is a guiding document used to design and construct buildings to protect the public from man-made and natural hazards for an acceptable amount of risk. Many of the seismic provisions in the building code are a result of poor performance or observations from past and recent earthquakes. Examples of seismic provisions introduced into the building code as a result of poor performance have included ductile detailing of concrete, proper anchorage of floor and roof diaphragms in tilt-up buildings, consideration of liquefaction potential and mitigation, precautions about steel moment resisting frames, and accounting for near-source directivity effects near active earthquake faults. A list of changes to the U.S. building codes in response to observed earthquake performance was published in the SEAOC Blue Book (2009). However, damage attributable to seismic earth pressures have not been observed in United States earthquakes, yet provisions have crept into the building code with significant design and cost impact.

## BRIEF HISTORY OF SEISMIC EARTH PRESSURE CODE PROVISIONS IN THE UNITED STATES

There were no specific requirements for the seismic increment of active earth pressure to be applied to walls retaining earth in any of the model building codes in the United States through 2003; this would include the Uniform Building Code, National Building Code, and Southern Building Code, which were all ultimately supplanted by the International Building Code (IBC). The 2006 edition of the IBC was the first national building code to include provisions to consider seismic earth pressures on earth retaining walls.

**California Building Code**

The California Building Code (CBC), which was based on the Uniform Building Code, did have provisions that included the issue of the seismic increment of active earth pressure. The CBC had jurisdiction over hospitals and public schools, as well as State of California public buildings. 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 (International Conference of Building Officials, 1988):

> ***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 ICBO, 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.

**NEHRP Recommended Provisions**

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. It did not, however, 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 other hazards to be investigated are slope instability, liquefaction, and surface rupture due to faulting or lateral spreading, all as a result of earthquake motions.)

The FEMA 450 commentary states that "…increased lateral pressures on retaining structures during earthquakes have long been recognized; however, design procedures have not been prescribed in U.S. model building codes." The commentary notes that waterfront structures have often performed poorly in major earthquakes due to excess pore water pressure and liquefaction conditions developing in relatively loose, saturated granular soils based on a paper by Whitman (1991). The commentary also mentions that damage reports for structures away from waterfronts are generally limited with only a few cases of stability failures or large permanent movements, also according to Whitman.

The FEMA 450 commentary provides a discussion of the seismic design analysis of retaining walls for two categories of walls:

- "yielding" walls – walls that can move sufficiently to develop minimum active earth pressures
- "nonyielding" walls – walls that do not satisfy the movement condition

For yielding walls, the FEMA 450 commentary states that there is consensus in the geotechnical engineering practice that a simplified Mononobe-Okabe seismic coefficient analysis reasonably represents the dynamic (seismic) lateral earth pressure increment for yielding retaining walls (Mononobe and Matsuo, 1929; Okabe, 1926). The commentary presents an equation for evaluation of the dynamic incremental component ($\Delta P_{AE}$) proposed by Seed and Whitman (1970):

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

where $k_h$ is the "horizontal ground acceleration divided by gravitational acceleration." The commentary recommended that $k_h$ be taken equal to the site peak ground acceleration that is consistent with the design earthquake ground motions as defined in the Provisions of FEMA 368 ($k_h = S_{DS}/2.5$). The resultant dynamic thrust was recommended to act at $0.6H$ above the base of the wall (which would be an inverted trapezoidal pressure distribution). It should be noted for the record that the Mononobe-Okabe theory also considers the vertical ground acceleration, denoted as $k_v$. Seed and Whitman (1970) had determined that the vertical ground acceleration, $k_v$, could be neglected for practical purposes because they made the observation that for most earthquakes, "…the horizontal acceleration components are considerably greater than the vertical acceleration components…"

For nonyielding walls, the FEMA 450 commentary presents an equation developed by Wood (1973) for a rigid nonyielding wall retaining a homogeneous linear elastic soil and connected to a rigid base. The dynamic thrust, $\Delta P_E$, is approximately:

$$\Delta P_E = k_h \gamma H^2$$

As for yielding walls, the point of application of the dynamic thrust is typically taken at a height of 0.6*H* above the base of the wall.

The FEMA 450 commentary suggests that dynamic earth pressure solutions would range from the Mononobe-Okabe solution as a “lower” bound to the Wood solution as an “upper” bound.

Although the FEMA 450 report has an extensive commentary on consideration of increased lateral pressures on retaining walls during earthquakes, it does not provide any insight or guidance on what situations should be considered, especially in the case of nonyielding walls not connected to a rigid base. The commentary does not provide recommendations on the height of the retained earth (for “retaining” walls or level of unbalanced earth in the case of opposite building walls retaining earth, such as given in the earlier versions of the California Building Code.

**ASCE/SEI 7-05 Minimum Design Loads**

Minimum Design Loads for Buildings and Other Structures were published as ASCE Standard ASCE-SEI 7-05 (commonly referred to as ASCE 7-05) (American Society of Civil Engineers, 2006). For all earth retaining structures assigned to Seismic Design Category D, E, or F, lateral earth pressures due to earthquake ground motion are to be determined in accordance with Section 11.8.3 of ASCE 7-05. Section 11.8.3 just states that the geotechnical investigation report shall include: “The determination of lateral pressures on basement and retaining walls due to earthquake motions.” A similar terse recommendation was also in the earlier edition of ASCE 7-02.

**International Building Code**

The 2006 edition of the International Building Code (IBC) adopts by reference the seismic requirements of ASCE 7-05. The 2009 IBC does not change this practice. Thus the requirements for seismic design pressures mandated by ASCE 7-05 are part of IBC.

## DAMAGE TO BUILDING BASEMENT WALLS IN EARTHQUAKES

Although there are many reports of damage to earth retaining walls during earthquakes, almost all of the reports are for either poorly constructed non-engineered walls or walls that failed because of a soil-related failure, with many being in a marine or waterfront environment. Based on a search of literature by the authors, no reports of any damage to building basement walls retaining earth have been found for the 1971 San Fernando, 1987 Whittier Narrows, 1989 Loma Prieta, and 1994 Northridge earthquakes in the United States. Also, reports of damage to building basement walls in foreign earthquakes are few.

**United States Experience**

It is the authors' personal experience and the experience of colleagues in geotechnical engineering that engineered building basement walls did not experience damage in the recent United States earthquakes.

An extensive report on damage observed in the San Fernando earthquake of February 9, 1971 was published by the United States Department of Commerce National Oceanic and Atmospheric Administration (NOAA) (Murphy, 1973). In this report, the only reported damage to a building basement wall occurred at the Olive View Medical Center, Medical Treatment and Care Unit. A basement or retaining wall on the lowest level experienced pounding from movement of the structure against the wall, disturbing the soil behind the wall and also causing tension cracks on the inside (compression) face of the cantilever retaining wall; movement at the top of the wall was reported to be as much as 6 inches. Clough and Fragaszy (1977) reported on a study of floodway channels in the Los Angeles area that also experienced the San Fernando earthquake. They reported that no damage occurred to 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.

Damage to building basement walls was not reported in the two volumes of Earthquake Spectra (Earthquake Engineering Research Institute, 1988a and 1988b) which presented observations from the Learning from Earthquakes (LFE) program on the October 1, 1987 Whittier Narrows earthquake in Southern Califorina. Whitman (1991) also cited a reference on the behavior of ten tied-back walls in the Whittier Narrows earthquake that had no evidence of loss of integrity.

During the post-earthquake reconnaissance of the October 17, 1989 Loma Prieta earthquake by the Earthquake Engineering Research Institute (EERI), there were no observations or reports of damage to building basement wall structures (Benuska, 1990). A survey of mechanically-stabilized walls (for highways) was also cited by Whitman (1991); in the Loma Prieta earthquake region, no evidence of significant residual movements was observed in mechanically-stabilized walls.

Numerous geotechnical researchers and practitioners performed extensive reconnaissance of the effects of the January 17, 1994 Northridge earthquake (moment magnitude 6.6). No reports of damage to building basement walls were reported by Stewart et al. (1994), Hall (1995), and Holmes and Somer (1996). Lew, Simantob and Hudson (1995) reported that several deep excavations in Los Angeles secured with soldier beams and tieback anchors experienced no failures or excessive deflections. There are two examples of buildings in Los Angeles that retained a significant difference in height of soil from one side to the other: UCLA Boelter Hall and a 55-story office building in Downtown Los Angeles. Both buildings experienced strong ground motions during the Northridge earthquake. Boelter Hall has one wing constructed into a hillside and has 3 stories are below grade on the east side of the wing and it is daylighted on the west side, having an approximately 35 feet of unbalanced earth loading. The California Strong Motion Instrumentation Program had a free field ground response instrument on the UCLA campus and it is reported that the peak ground acceleration for one horizontal component at that instrument was

0.66g in the Northridge earthquake (Shakal et al., 1994). There were no reports of damage to the basement wall of Boelter Hall.

The 55-story office building is also constructed into a hillside having a base podium structure that extends about 100 feet below grade on the east and about 45 feet below grade on the west; there is approximately 55 feet of unbalanced earth retention from east to west. Because the 100-foot tall basement wall was constructed by slope cutting the natural materials at an inclination of 2/3:1 (horizontal to vertical) and then backfilling against the basement wall with soil, the basement wall was designed for lateral earth pressures consisting of a triangular distribution of earth pressure equivalent to that developed by a fluid having a density of 32 pounds per cubic foot. The peak ground accelerations during the Northridge earthquake within a few blocks of the 55-story office building were on the order of 0.2g (Shakal et al., 1994). There were no reports of damage to the basement wall of this building.

**Experience outside the United States**

Some comment is also necessary regarding the few retaining structures with documented significant movement that are away from waterfronts that were described in the paper by Whitman (1991) mentioned earlier. The retaining structures with significant movement that are cited include a few cantilever retaining walls, gravity walls, and a bridge abutment. Some of the failures were attributed to liquefaction. The references quoted by Whitman also mention retaining structures (away from waterfronts) that were not affected by earthquake. In one instance, there were low retaining walls in Tokyo where extensions were added to make higher retaining walls; these walls had no damage during earthquakes despite calculations by the Mononobe-Okabe formula that would have predicted failure. Whitman also reported that despite extensive earthquake damage to port facilities at Akita, Japan, 24 reinforced earth walls in the area performed well. Thus only a few actual cases of retaining structures with significant movement are documented. It is significant to observe that there are no reports of damage to building basement walls retaining earth in any of the references cited by Whitman.

A review of case history reports on geotechnical aspects of the January 17, 1995 Hyogoken-Nambu earthquake that devasted Kobe, Japan (Japanese Geotechnical Society, 1996) provided much evidence of failures and large displacements in waterfront walls and freestanding retaining walls supporting embankments, however, no evidence of damage to building basement walls was reported. However, there was some damage to subway stations in Kobe with the most severe damage to the Daikai Subway Station, part of the Kobe Rapid Transit Line (Iida, Hiroto, Yoshida and Iwafuji, 1996). Less severe damage occurred at four other stations in Kobe and at other locations in the subway system.

Iida et al. report that the Daikai station is the first subway structure completely damaged during an earthquake. The Daikai station was completed in 1964 and used the cut-and-cover method of construction. The station is about 120 meters long. Most of the Daikai station is a reinforced box type frame with columns at the center of the box and passenger platforms on the two sides. The box is about 17 meters wide and about 7.2 meters high (outside dimensions). The thickness of the overburden soils was about 4.8 meters above the one-story portion of the station. A small portion of the station was two stories with the upper floor serving as a ticket concourse and the lower floor with the passenger platforms. The two-story portion of the station also has center columns and is wider with additional column lines matching the exterior wall lines of the one-story portion of the station. The two-story portion is about 26 meters wide and about 10.1 meters high (outside dimensions). The thickness of the overburden soils above the two-story portion of the station is about 1.9 meters.

The most severe damage in the station occurred in the longer one-story portion of the station with failures of the center columns resulting in the ceiling slab subsiding along with the overburden soils above the station. Many cracks were also observed in the longitudinal walls as well as the few transverse walls at the ends the station and at the areas where the two-story portion abuts the one-story portions. Most of the columns failed at the base and Iida et al. opine that the initial mechanism of failure was from a combination of shear and bending moment. Once the initial damage occurred, the axial capacity of the columns was reduced which resulted in the complete failure of most of the center columns resulting in collapse of the ceiling of the station. The collapse of the center columns and ceiling caused cracking and tilting of the longitudinal walls; separations were seen near the top of the walls and near a bottom haunch for the station platforms.

An examination of the photographs in the article by Iida et al. reveals that the columns had very minimal lateral ties indicating that the columns would exhibit nonductile behavior. From the discussions in the paper, it appears that the station box structure was not designed for racking conditions due to earthquake, a practice that is common in design of subway stations in the United States for such systems such as the Bay Area Rapid Transit District in the San Francisco Bay area and the Metro Rail System in Los Angeles.

The Iida et al. paper does not mention the possibility that liquefaction may have occurred at the Daikai station. An examination of the soil profiles of boreholes drilled before and after the earthquake reveals that Holocene age sand materials are present in the vicinity of the station. One borehole drilled adjacent to the station after the earthquake encountered fill materials consisting of sandy soil. Soil profiles shown in the Iida et al. paper show that the standard penetration test (SPT) blowcounts (N-values) range from below 10 to above 30 in the Holocene sand materials, with many values in the 10 to 20 range. In the sandy fill soils, the N-values were typically about 10. The ground water level was reported to be about 3 meters in 1959 and between 6 to 8 meters in February 1995. Although Iida et al. do not mention the possibility of liquefaction, the data about the N-values and ground water levels strongly suggest that liquefaction may have occurred in the soils around the station, especially in fill materials adjacent to the station. This liquefaction may have contributed to the structural failures of the Daikai station whereby a liquefied soil exerts higher lateral pressures, even without directly considering the effects of lateral ground motions.

Damage to building basement walls were not reported in the EERI reconnaissance report for the August 17, 1999 Kocaeli, Turkey earthquake (Youd, Bardet and Bray, 2000).

Huang (2000) and Tokida et al. (2001) reported on the various types of soil retaining structures damaged by the September 21, 1999 Chi-Chi, Taiwan earthquake;

they both reported on damage to gravity-type retaining walls, wrap-around type geosynthetics-reinforced soil retaining walls, and segmental retaining walls with no damage to cantilever type retaining walls. However, a careful review of the damaged retaining structures shows that most, if not all, were located on steep slopes and their failures involved some combination of bearing capacity failure, overturning due to inadequate base width, slope instability above the walls, and in several cases direct fault offset. There was no mention of building basement walls in the two papers or in Abrahamson et al. (1999).

Rathje et al. (2006) reported that there was an absence of damage to basement walls and retaining walls in Duzce as a result of the November 12, 1999 Duzce, Turkey earthquake. However, Gur et al. (2009) does report that basement damage occurred in a school building in the Duzce earthquake. It was reported that a four-story school building had damage concentrated in the half-buried basement surrounded by partial height earth-retaining concrete walls. There were windows between the earth-retaining walls and the beams at the top of the basement and the exterior basement columns, which were captive along their weak axis, failed in shear. Gur et al. also report that the displacement demand was high enough to result in severe damage to masonry infill walls in the basement of the school building, but did not report about any damage to the earth-retaining concrete walls of the basement. The maximum horizontal ground accelerations near the school was reported as being 0.51g (east-west) and 0.41g (north-south). Gur et al. also report that in the May 1, 2003 Bingöl, Turkey earthquake, there was light damage to lateral basement walls even though the buildings had severe structural damage or collapse; the maximum horizontal ground accelerations in Bingöl were reported as being 0.28g (east-west) and 0.55g (north-south).

### Summary

There are only few instances of documented damage to building basement walls due to seismic earth pressure in the United States or outside of the United States. The few reported instances that have occurred outside of the United States have only been minor in the damage amount. The available literature does not indicate that damage to building basement walls is a prevalent or even an occasional concern.

## CURRENT STATE OF PRACTICE

Despite the lack of compelling damage that can be attributed to seismic earth pressures, the IBC Code (through the provisions of ASCE 7) requires the “determination” of seismic earth pressures for the design of earth retaining structures. The impetus for ultimate inclusion of seismic earth pressures probably dates back to the Seed and Whitman (1970) paper which essentially brought to the forefront the so-called Mononobe-Okabe seismic coefficient analysis (Mononobe and Matsuo, 1929 and Okabe, 1926). The interest aroused by this state-of-the-art paper sparked many researchers to conduct analytical, laboratory, and field analyses of the behavior of earth retaining structures to earthquake ground motions. Many of these studies, usually based on the same or similar assumptions made in the Mononobe-Okabe method, concluded that seismic earth pressures would be significant on earth retaining structures. On the basis of these studies and not actual experience, the concept of seismic earth pressures became an issue of concern that eventually led to its inclusion in the seismic design regulations embodied in the current ASCE 7 and IBC publications.

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. The reasons for using the M-O method may 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. Although other methods may be used in practice, the M-O method is the most common method of analysis by far.

## USE OF THE MONONOBE-OKABE METHOD OF ANALYSIS

Despite the appearance of simplicity of the M-O method, suffice it to say that there is confusion among geotechnical practitioners regarding the evaluation of seismic earth pressures using this method for building basement walls. Part of the confusion stems from whether the M-O method is actually applicable for the intended analysis. The 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 granular backfill excited by a sinusoidal excitation on a shaking table.

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 1.

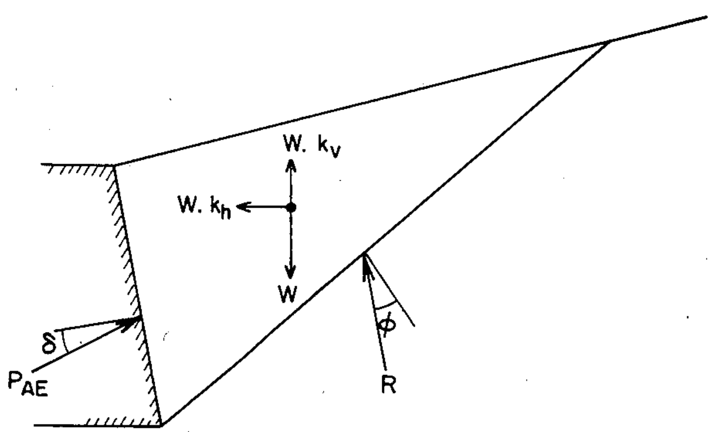


Fig. 1 Forces considered in the Mononobe-Okabe Analysis (after Seed and Whitman, 1970)

Seed and Whitman (1970) endorse the use of the method for gravity walls and list 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.

Despite these assumptions, the M-O method continues to be used for below ground structures. Ostadan and White (1998) have stated that "…the M-O method is one of the most abused methods the geotechnical practice." Ostadan and White list 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 as 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 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.

An area of abuse or perhaps more correctly misuse, 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 the same vein, 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.

A study by the Washington State Transportation Center (Fragaszy, Denby, Higgins and Ali, 1987) on the seismic response of tieback retaining walls found that the M-O method overpredicted the dynamic soil pressures by a significant amount except for a small interval when compared with a finite element model using the full peak ground acceleration. Although a tieback retaining wall is not the same as a basement wall, tiebacks may be used for temporary shoring before the permanent basement wall is constructed and there may be application in this case as the shoring is usually not completely de-tensioned and left in place.

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 reason for the reduced value of the seismic coefficient compared to the peak ground acceleration is 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 a 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 out-of-phase nature of the ground movement as shear waves propagate vertically through the backfill soil).

The justification many geotechnical engineers use for using 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 from 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.

Probably the biggest abuse of the M-O method is its application to retained earth that is not a truly cohesionless backfill. 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 2 and 3 present active earth pressure coefficient charts for two different soil friction angles with different values of cohesion for horizontal backfill, assuming no tension cracks and wall adhesion.

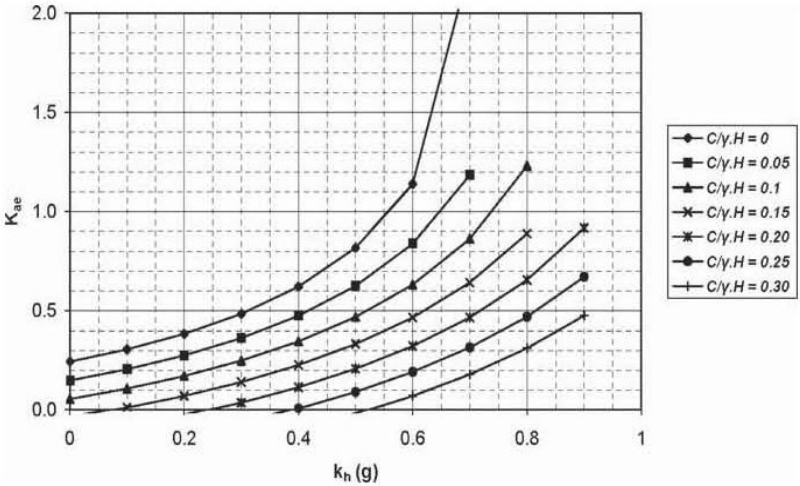


Fig. 2. Seismic coefficient chart for c-ϕ soils for angle of internal friction of 35 degrees (after Anderson et al., 2008).

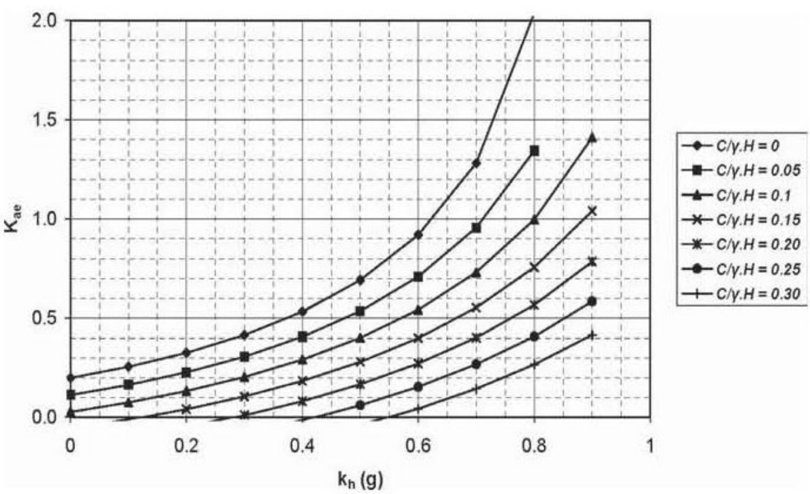


Fig. 3. Seismic coefficient chart for c-ϕ soils for angle of internal friction of 40 degrees (after Anderson et al., 2008).

These two charts show that a small amount of cohesion would have a significant effect in reducing the dynamic active earth pressure for design.

It should be noted that neglecting the vertical ground acceleration in the M-O analysis, as suggested by Seed and Whitman (1970), may be unconservative in cohesive soils. Recent events such as the 1994 Northridge, 1999 Chi-Chi (Taiwan), and 2008 Great Wenchuan (China) earthquakes have given recordings where the vertical ground motion components are comparable or even greater than the horizontal ground motion components. While some failures of retaining structures occurred in the epicentral region with high vertical ground motions in the Chi-Chi event, the absence of damage to retaining structures was striking in the 1994 Northridge and the 2008 Great Wenchuan events which also contained significant vertical components. Most recently, Gazetas et al. (2009) show that vertical accelerations have no influence on purely frictional analysis of sliding block motion using the Chi-Chi data, which is consistent with current analysis methods.

There are many reasons why the Mononobe-Okabe method is being used, misused and abused by geotechnical engineers in estimating seismic earth pressures on building basement walls. Geotechnical engineers are drawn to this method because of its simplicity, however, there are many assumptions that have to be made and some of the assumptions may simply not be applicable. The inclusion of cohesion in determining the M-O seismic increment of earth pressure may give more “reasonable” results. However, there is a lack of guidance as to what is a correct or reasonable seismic earth pressure.

## RECENT RESEARCH ON SEISMIC LATERAL PRESSURES

As mentioned previously, the original experimental tests that formed the basis of the M-O method were based on the response of a small scale cantilever bulkhead supporting a dry, medium dense cohesionless backfill, excited by a sinusoidal input on a shaking table with accelerations up to 0.3g. Many of the researchers that followed have used similar experimental set-ups. However, the applicability of the test results from a small scale test based on idealized sinusoidal loading to full size structures has been called into question with new advances in testing, especially with the emergence of centrifuge testing.

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 was conducted by Ortiz, Scott and Lee (1983) to verify the M-O theory. One important finding in this study is the conclusion 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.” Ortiz et al. also found that there was good agreement between the M-O theory and the centrifuge experiment that the point of application of the resultant of the static and dynamic earth pressure; i.e., the resultant was at about the one-third of the wall height above the base of the wall.

A more recent study by Nakamura (2006) also sought to reexamine the Mononobe-Okabe theory by centrifuge testing. An important finding by Nakamura was that the earth pressure distribution on the model retaining wall is not triangular (as assumed by M-O), and that its size and shape change with time. Nakamura also found that the earth pressure distribution for an input motion that was based on an actual earthquake time history 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 an earthquake 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.

Al Atik and Sitar (2007) also performed centrifuge experiments on model cantilever walls with medium dense dry sand backfill. Al Atik and Sitar found that the maximum dynamic earth pressures increase with depth that 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 0.3g and state 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.

## CONCLUSIONS

Despite the absence of compelling or even minimal evidence of structural distress or failure of building basement walls in earthquakes, the state of practice as dictated by the current building code (IBC) and engineering practice standards (ASCE 7-05) requires the consideration of seismic earth pressures for buildings and structures that have retained earth materials. Observations of the behavior of walls during earthquakes suggest that structural performance is quite good except for cases where there may be loss of strength in the soil due to liquefaction or other processes.

Because of the simplicity of the method, geotechnical engineers have generally been trying to apply the Mononobe-Okabe method of analysis to evaluate the seismic earth pressures for the building code’s design earthquake criteria. Whether or not the Mononobe-Okabe method is really suitable for the evaluation of building basement walls may be debatable because the tests were made for cantilever walls retaining medium dense sand. As building basement walls are generally braced at several levels, comparisons with cantilever walls may be difficult. Also, medium dense sand backfill may not be representative of most of the retained earth behind building basement walls. Some attempts have been made to account for cohesion in the retained earth which will reduce the intensity of the seismic earth pressures. The profession has also struggled with the appropriate value of the seismic coefficient to use as high ground accelerations give very high seismic earth pressures that do not seem reasonable.

Recent research using centrifuge testing brings into question the validity of the M-O method. The applicability of the original test on a small scale shaking table with medium dense sand backfill excited by a sinusoidal wave to a large building basement wall appears to be suspect, if not valid at all. Centrifuge testing indicates model walls that have been properly scaled subjected to more realistic earthquake ground motions do not appear to experience large seismic earth pressures and the results indicate that the M-O method is very conservative, if just not applicable. However, centrifuge testing does indicate that the location of the resultant of the static and seismic earth pressures appears to match the M-O method at the one-third height above the base of the wall and is not located on the upper part of the wall as suggested by some researchers.

It appears that the current design practice for seismic earth pressures on building basement walls is conservative, uneconomical, and perhaps unnecessary. More importantly, the design practice is mostly based on experimental data that were extrapolated beyond the limits of their applicability.

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