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CURRENT PRACTICES FOR ANALYSIS OF LATERAL SEISMIC EARTH PRESSURES

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Abstract

Retaining and basement walls are structurally designed to resist lateral earth pressures from the retained soil mass behind the walls. Walls are also designed to resist other induced pressures, such as those induced by nearby surcharge loads as well as those induced by seismic ground motions. The understanding of lateral seismic earth pressures on retaining walls has advanced in recent years with developments in numerical modeling, case history data, and experimental physical testing. These advancements include how to define the horizontal seismic coefficient, distribution of the seismic earth pressure, point of application of the seismic incremental force, and the differences in analyses for yielding and non-yielding retaining walls. Based on a comprehensive literature review, this paper discusses the methodologies recommended for analysis of lateral seismic earth pressures from recent studies and presents a compiled, one-stop-shop for the current standards set forth by various jurisdictions, professional associations, and research programs. The goal of this paper is to present the state-of-the-practice and provide practicing engineers with an understanding of the progress made in estimating lateral seismic earth pressures on retaining structures.

Keywords: seismic; earth pressures; retaining walls; geotechnical

1. Introduction

Research into the understanding and analysis of seismic lateral earth pressures was pioneered after the Great Kanto Earthquake of 1923. The innovative yet "simplistic" Mononobe-Okabe (M-O) method came about as a result of this research performed and published by Okabe in 1926 [1] and Mononobe and Matsuo in 1929 [2]. The M-O method has since become the most widely used method for estimation of lateral seismic earth pressures acting on retaining walls. Since the induction of the M-O method, several studies have been performed and technologies have been implemented which have advanced the general understanding of dynamic lateral earth pressures. However, the M-O method is still a standard frequently used in practice today. Furthermore, while these recent studies have been beneficial in understanding the limitations of the somewhat dated M-O method and updates to analyzing dynamic lateral earth pressures have been proposed, there has been little consensus on what should be implemented by engineers in practice.

Due to the historical use of the M-O method and lack of consensus on how to improve on this method, practicing engineers often resort to the use of the M-O method for both yielding and non-yielding walls with variable "tweaks" such as those proposed by Seed and Whitman in 1970 [3]. These "tweaks" include which value of the spectral acceleration to use, how the pressure distribution is applied to the retaining wall, and the location of application of the seismic resultant force. As different resources recommend different variations of the M-O method or different methodologies completely, engineers throughout the practice provide variable recommendations for seismic lateral earth pressures on retaining walls. This paper discusses the current methods available to engineers and used in practice to date for analysis of seismic lateral earth pressures on both yielding and non-yielding retaining walls, takes a look at how these methods align with existing case and experimental studies, provides a discussion on what various professional organizations and agencies are currently recommending, and, lastly, provides a summary on what updates are being used in practice currently so that engineers may be able to implement these updates into their engineering practice.

2. Methodologies for Estimation of Seismic Lateral Earth Pressure – Yielding Walls

Due to the difference in structural response, retaining walls are typically divided into two general categories: yielding and non-yielding walls. Yielding walls are walls that are typically unrestrained and can displace enough to develop active earth pressures. Non-yielding walls are walls that are typically restrained, such as basement walls, and do not displace sufficiently. Therefore, at-rest earth pressures are used for non-yielding walls in design.

The following sections discuss the methodologies available and used in practice for analysis of seismic lateral earth pressures on yielding retaining walls for cohesionless and cohesive backfill soils. Methodologies used for non-yielding walls are discussed in Section 3 of this paper.

2.1 Yielding Walls with Cohesionless Backfills

2.1.1 Mononobe-Okabe (M-O) Method

The M-O methodology [1],[2] is based on Coulomb's sliding wedge theory for static lateral earth pressures for active conditions as well as the results of shake table experiments. This method presents an equation for estimating the total active lateral thrust force (i.e. static active plus seismic force) and was originally developed for gravity retaining walls. The total active lateral thrust is estimated based on the unit weight and internal angle of friction of the backfill soil, height of the wall, vertical and horizontal coefficients of acceleration (k_v and k_h), friction angle between the soil and the wall material, slope angle of the inner face of the wall, and backfill slope angle. The M-O method proposed that this total active lateral thrust has a point of application above the base of the wall of one-third the height of the wall corresponding to an upright triangular pressure distribution. However, as will be discussed in subsequent sections on this paper, arguments have been made to raise the point of application of the lateral seismic force associated with the M-O method and inconsistencies in these recommended application heights have led practitioners to apply the seismic lateral force at wall heights above the base ranging from 1/3H to 2/3H. This range of application heights have also resulted in variations of the distribution of the dynamic earth pressure with practitioners generally recommending an upright triangular distribution, inverted triangular distribution, or a uniform distribution.

This pioneering method is still commonly used by engineers in practice for a wide array of wall types, backfill conditions, and slope conditions despite several limiting assumptions. The assumptions valid for the M-O method include the following: 1) the backfill soil is dry, cohesionless, homogenous, and perfectly plastic; 2) the wall is long enough to neglect the effect of the wall end; 3) the wall yields sufficiently to mobilize active earth pressures; and 4) the failure plane in the backfill soil goes through the heel of the wall.

For simpler wall designs, the M-O method has generally been thought of as a reasonable, upper-bound estimate to account for dynamic forces in retaining wall design. However, engineers often apply this method where the fundamental assumptions of the M-O method do not apply. For instance, the M-O method is often applied to non-yielding walls or in cases where cohesive or layered soils are present behind the wall. Furthermore, the M-O method can result in an "infinite" earth pressure on the wall if a steep backslope is combined with relatively high seismic accelerations, making this method inappropriate in these cases.

Also of debate with the M-O method is the value to use for the coefficient of horizontal acceleration, designated as k_h, in calculation of the total active lateral thrust. Typically, the consensus on the coefficient of vertical acceleration, k_v, is to assume this value as zero, consistent with the M-O method. The M-O method then proposed to use the design peak ground acceleration (PGA) value as the coefficient of horizontal acceleration, k_h. However, in practice, the coefficient of horizontal acceleration is debated to range from one-third of the design PGA to 100 percent of the PGA, with other common values used in practice including one-half and two-thirds of the PGA. The fraction of the PGA recommended to be used to calculate the k_h value changes based on the code, jurisdiction, or engineer involved.

2.1.2 Seed and Whitman Method (1970)

The Seed and Whitman 1970 method [3] is based on the M-O method with some adjustments. Seed and Whitman proposed that the total active lateral thrust can be separated into two components: the static active lateral earth pressure and the seismic lateral earth pressure in which the static active lateral force plus the seismic lateral force equals to the total active lateral thrust. This notion was subsequently applied to the M-O method in which the coefficient of the total lateral active thrust (K_{AE}) is calculated using the M-O equation and subtracted by the static coefficient of active earth pressure (K_A) to calculate the change in the lateral active earth pressure coefficient due to the dynamic acceleration (ΔK_{AE}). The ΔK_{AE} value is used to calculate the seismic incremental lateral earth pressure force. However, Seed and Whitman proposed a simplified method to estimate the ΔK_{AE} value based on a parametric sensitivity analysis of the M-O method in which ΔK_{AE} is generally about three-fourths times the k_h value. Seed and Whitman proposed to use this ΔK_{AE} value to calculate the seismic incremental lateral force.

Of significance with the Seed and Whitman 1970 method [3] is their recommendation for the k_h value and the point of application of the seismic incremental lateral force. Seed and Whitman noted that the PGA occurs for one instant in time and, therefore, does not have enough duration to cause wall movements of significance. As a result, Seed and Whitman recommended a value of 85 percent of the design PGA be used for the k_h value. Furthermore, based on shake table results by others, Seed and Whitman recommended the seismic incremental force be applied at approximately 0.6H above the wall base inherently increasing the moment that this force induces. This recommendation by the Seed and Whitman 1970 method [3] has stuck around with the application of the M-O method resulting in some practitioners recommending a point of application at about 0.6H corresponding to an inverted triangular distribution.

2.1.3 Mylonakis Method (2007)

Another method was proposed by Mylonakis et al. in 2007 [4] that is based on the theory of discontinuous stress fields to provide a simple, closed-form, stress limit solution. This method uses parameters associated with the weight and friction of the backfill soil, slope of the wall and backfill, wall roughness, surface surcharge, and coefficients of horizontal and vertical seismic accelerations. These parameters are proposed to be altered from their static state based on the angle that the seismic thrust is being imposed. Simplified expressions, graphs, and charts are presented by Mylonakis et al. 2007 [4] that can be used to estimate the coefficient of total seismic active earth pressure, K_{AE}. Furthermore, as this solution is based on a pseudo-static analysis (as is the M-O solution), the stresses in the soil are assumed to vary linearly with depth. Therefore, Mylonakis et al. 2007 [4] proposed an upright triangular distribution with the resultant seismic thrust force applied at H/3 above the base of the wall.

This solution was found to be conservative by generally overestimating the seismic active earth pressure compared to rigorous numerical analyses. Although this method provides a stress-based, conservative solution, it has been found to be overly conservative and has gained little traction in comparison to the M-O method [1], [2] and the Seed and Whitman 1970 [3] updates and is less commonly used in engineering practice.

2.1.4 Anderson et al. (2008) (NCHRP Report 611)

The 2008 National Cooperative Highway Research Program (NCHRP) Report 611 [5] presents important advancements that are now being used in practice today. The first of these advancements is presentation of the Generalized Limit Equilibrium (GLE) procedure which was first illustrated by Chugh in 1995 [6] and studied by several others. The GLE method involves using limit equilibrium slope stability software, such as SLIDE (software by RocScience) or Slope/W (software by GeoStudio), and evaluating the total seismic active force by applying an external point load on the wall face corresponding to a safety factor of 1.0 for pseudo-static conditions. The GLE method can handle complex slope geometry, the inclusion of cohesion in backfill soils, and layered or saturated soils, but requires the wall and slope geometry to be known at the time of earth pressure analysis. Per the NCHRP 611 Report [5], the point load is recommended to be applied at 0.5H of the retained soil height, but can be examined anywhere from H/3 to 2/3H of the retained soil height to examine the maximum force required to achieve a safety factor of 1.0. The angle of this applied force is based on the wall-soil friction angle where a horizontal load can be used if assuming a smooth wall.

Another important advancement that the NCHRP Report 611 [5] introduces is the height-dependent PGA to be used for earth pressure analyses. For wall heights greater than 20 feet, as presented in the report, the factor alpha is applied to the site's PGA value (modified for site class). Alpha is computed based on the height of the wall and the spectral acceleration at a period of 1 second, S_1. For wall heights less than 20 feet, the NCHRP Report 611 [5] suggests that the value of alpha is about 1.

The NCHRP 611 Report [5] concludes that the M-O method may be used for most wall applications in idealized situations and that the GLE method may be used for more complex wall designs. The NCHRP Report 611 [5] then recommends different fractions of the height-dependent PGA to be used dependent on the wall type. For instance, for soil nail walls, non-gravity cantilever walls, and anchored walls, the value of the site-adjusted PGA to be used for the seismic coefficient may be reduced by half if permanent displacements of 1 to 2 inches are acceptable. If displacements need to be limited, the full value of the height-dependent PGA is recommended to be used.

2.1.5 Brandenberg et al. (2020) NEHRP Method

The 2009 National Earthquake Hazards Reduction Program (NEHRP) Seismic Provisions [7] required that seismic lateral earth pressures be accounted for in design of retaining and basement walls for specified seismic design categories, but did not provide guidance as to what method to use for this requirement. Part 3 of the 2009 NEHRP Seismic Provisions provided Resource Paper 12 [7] discussed the typical methods that can be used for analyses of seismic lateral earth pressures. In particular, Resource Paper 12 [7] discussed the M-O method for yielding walls and the Wood 1973 [8] method for non-yielding walls for simpler wall designs and discussed the GLE method for more complex wall designs. This resource paper also touched on displacement-based approaches, but the more commonly used M-O, Wood, and GLE methods remained to be widely used as was permitted by the 2009 NEHRP provisions [7].

In the recently published 2020 NEHRP Recommended Seismic Provisions [9], Resource Paper 4 [10], based on the publications of Brandenberg and others in 2015 and 2017 [11], [12], commentary is provided on the fundamental flaws and limitations of the currently used pseudo-static formulations (i.e. M-O method, Wood method, etc.) and a displacement-based approach is presented for yielding and non-yielding walls in any soil type for evaluation of seismic lateral earth pressures.

The displacement-based approach included in the 2020 NEHRP Seismic Provisions Resource Paper 4 [10] presents a kinematic solution which considers wall flexibility and can be used for non-rigid or rigid, free-standing walls. The kinematic solution presented in the 2020 NEHRP Provisions Resource Paper 4 [10] provides a way to calculate the normalized seismic earth pressure resultant force and normalized height of application of the resultant force, both dependent on the wall flexibility. The seismic earth pressure resultant force is normalized by a stiffness intensity parameter, ground surface displacement parameter, and the wall height while the height of application is normalized by the wall height. In order to de-normalize the seismic earth pressure force, Resource Paper 4 [10] presents a series of equations and figures to estimate the stiffness intensity parameter and ground surface displacement. These values are estimated based on several variables including the peak ground velocity (PGV), mean site period and associated angular frequency, average shear wave velocity of the backfill soil, and soil density and Poisson's ratio of the backfill soil.

Although this method is billed as a "simplified" method, it requires some parameters to be known which are generally unknown or unfamiliar for most geotechnical engineers when developing seismic lateral earth pressures, such as the shear wave velocity profile of the backfill soil. Therefore, while this method has been recently recommended by the 2020 NEHRP Seismic Provisions [9], [10], hesitancy is anticipated due to the relative complexity of this method when compared to the more traditional pseudo-static approaches.

2.2 Yielding Walls with Cohesive Backfills

The methodologies discussed in the subsequent sections are for estimation of seismic lateral earth pressures in cohesive soils. It has also been noted in Section 2.1 that some of those methods are applicable to both cohesionless and cohesive soils.

2.2.1 Okabe (1926) and Others

The Okabe 1926 equation [1] accounts for soil cohesion by including a cohesive term within the coefficient of the total lateral active thrust, K_{AE}, expression. Okabe's methodology [1] assumes that the seismic lateral earth pressure increases linearly with depth while the earth pressure due to cohesion is uniform with depth resulting in a point of application of the total seismic lateral force to be higher than H/3. This method has been shown to result in "infinite" values when dealing with high cohesion values, sloping backfills, or high seismic loads.

Subsequent studies were completed by several researchers based on Okabe's groundbreaking work. These studies built on Okabe's methodology but are not as commonly used in practice today. Prakash and Saran in 1966 [13] presented plots for determining the seismic lateral earth pressure of a c-ϕ soil by superimposing gravitational, surcharge, and cohesive effects resulting in a conservative approach. The Prakash and Saran method [13] accounted for tension cracks that may develop in cohesive backfill soils and found that effects associated with cohesion were essentially unaltered due to dynamic loading. Another study of note is that of Chen and Liu published in 1990 [14], in which both linear and log-spiral failure surfaces were examined for cohesive backfills and translational wall movements, but this method required a complex numerical solution and was not necessarily practical. With a lack of consensus in what methodology and parameters to use for seismic lateral earth pressures, particularly for materials with a cohesion component, these and other methodologies have not been widely used in practice.

2.2.2 Candia and Sitar (2013)

The Candia and Sitar 2013 methodology [15] is based on a series of centrifuge testing and numerical models using cohesive backfills. The results of these experiments were similar to the results of the centrifuge testing and numerical modeling published by Geraili and Sitar in 2013 [16] with cohesionless backfill soils. Therefore, the Candia and Sitar 2017 method [15] proposed simple linear expressions to be used for evaluation of the seismic lateral earth pressure increment for both cohesionless and cohesive backfills. For yielding cantilever walls with level backfill, a simple expression is recommended in which 0.42 times the free-field PGA is recommended for calculation of the ΔK_{AE} value to be used to calculate the seismic lateral incremental force. For yielding cantilever walls with sloping backfill, the Candia and Sitar method [15] proposed that the ΔK_{AE} value be 0.7 times the free-field PGA value. These expressions represent the 95 percent confidence level of the centrifuge testing results.

Furthermore, for cantilever walls in cohesive backfill conditions with sloping conditions, the effect of cohesion can increase the point of application of the resultant seismic force to up to about 0.4H above the base of the wall. The Candia and Sitar method [15] presents an expression for determining the point of application of the seismic resultant force based on the cohesive properties of the soil. This expression yields results ranging from 0.33H to 0.4H with a result of 0.33H for cohesionless backfills.

While the centrifuge testing and numerical modeling performed by Candia and Sitar [15] is valuable in further understanding how walls react under seismic loading, these simplified expressions don't have a way to account for layered soils, different slope angles, or different values of cohesion.

3. Methodologies for Estimation of Seismic Lateral Earth Pressure – Non-Yielding Walls

Although case studies and experimental studies are relatively limited for seismic earth pressures on retaining walls, even less case studies and full-scale experiments on non-yielding walls or basement walls have been conducted. This is partially a result of basement walls typically performing relatively well in seismic events. Nonetheless, there have been studies in which analytical or experimental methods for estimating the seismic lateral earth pressure on non-yielding walls have been proposed. The following sections discuss the methodologies available and used in practice for analysis of seismic lateral earth pressures on non-yielding retaining walls for cohesionless and cohesive backfill soils.

3.1 Non-Yielding Walls in Cohesionless Backfills

3.1.1 Wood Method (1973)

The Wood 1973 method [8] used an elastic-form solution, instead of plasticity-based solution (like those proposed for yielding walls), to estimate the seismic force applied on a perfectly rigid wall. The seismic force was determined by Wood to be approximately twice that of the solution proposed by the M-O method and, therefore, Wood proposed the seismic incremental force be equal to the coefficient of horizontal seismic acceleration, k_h, multiplied by the unit weight of the soil behind the rigid wall and the height of the wall squared. This seismic incremental force was recommended to be applied at a height of approximately 0.6H above the base of the wall by the Wood method [8]. Furthermore, it should also be noted that the Wood solution [8] does not account for the effects of the superstructure connected to the top of the wall.

3.1.2 2020 NEHRP Method

As discussed in Section 2.1.5, the method presented in Resource Paper 4 of the 2020 NEHRP Seismic Provisions [10] presents a kinematic solution that accounts for wall flexibility and can be applied to both rigid and non-rigid free-standing walls in all soil types. Section 2.1.5 provides further discussion on this method.

3.2 Non-Yielding Walls in Cohesive Backfills

3.2.1 Candia and Sitar (2013)

The linear expressions presented in the Candia and Sitar 2013 method [15] for cohesive and cohesionless backfill soils based on a series of centrifuge testing and numerical modeling also included an expression for non-yielding basement walls. The expression presented by Candia and Sitar [15] proposed the ΔK_{AE} value to be 0.68 of the free-field PGA value with the application of the resultant seismic incremental force at H/3.

4. Field Case and Experimental Studies

Several studies on the field performance of retaining walls after seismic events as well as experimental studies using shake tables or centrifuge models have been performed throughout the years since the experimental work published by Mononobe and Matsuo in 1929 [2]. The following sections discuss these field cases and experimental studies and the results observed from these studies in further detail.

4.1 Observations from Field Case Studies

In the Candia 2013 publication [17], a comprehensive review of observations of the field performance of retaining structures after earthquakes is presented. In this review, most of the observed retaining wall failures due to an earthquake event are associated with liquefaction of saturated backfills and supporting ground [18], [19]. Otherwise, retaining structures have performed within acceptable design levels as observed during the following seismic events: 1989 Loma Prieta, 1994 Northridge, 1999 Turkey, 2008 Wenchuan, 2010 Chile, 2010-11 New Zealand, and 2011 Japan [20]. In the 1999 Chi-Chi earthquake, failure of different types of retaining structures was observed, but most of these failures were located on steep slopes. Some leaning-type gravity walls collapsed due to bearing capacity failures and stress concentrations at footings. When liquefaction and lateral spreading were observed, the 1995 Kobe earthquake resulted in failure of some free-standing retaining walls and waterfront structures [21], but no damage was observed to building basement walls. Case histories have historically showed that retaining structures have generally performed within acceptable levels in low to moderate seismic activity, even in cases where seismic design was not included [3][18], [22]. This is further supported by experimental studies, as discussed in Section 4.2, and some of the methodologies presented in Sections 2 and 3 of this paper consider these observations by proposing the seismic earth pressure is negligible for low design ground motions.

4.2 Observations from Experimental Studies

Several experimental studies have been performed using retaining wall models of different sizes sitting on earthquake simulators using 1-g shake tables or within centrifuge environments. The retaining wall experimental models in these studies were either fixed to the base of the shake table or founded on soil.

Mononobe and Matsuo [2] used a small sand box filled with loose sand and simple harmonic simulated ground motions under a 1-g environment. The model wall was hinged at the base so the wall could tilt outwards. The results of these experiments agreed with the analytical work published by Okabe [1] and the M-O method was developed by modifying the Coulomb theory on static active earth pressure based on these experimental studies as discussed in Section 2.1.1 of this paper [1], [2].

Similar experiments using sand boxes under 1-g loading have been performed by different researchers with conclusions consistent with the M-O method [23], [24], [25], [26].

Additionally, dynamic centrifuge experiments have been developed, and shake table testing under centrifuge environments were used to capture the effects of confining stresses in the dynamic response of model retaining walls. One of the first to use this type of experimental testing, Ortiz and others in 1983 [27] performed centrifuge experiments to study the response of flexible cantilever walls on medium dense sand. Experimental results were also consistent with the M-O method but were limited to the medium dense, dry, cohesionless soil used in the experiments. Bolton and Steedman [28], [29] performed centrifuge experiments of micro concrete cantilever walls retaining a dry cohesionless backfill. Results were also consistent with the M-O method and indicated that wall inertial forces played an important role in these types of walls. Later studies published by Munaf in 1998 [30] and Koseki et al. in 1998 and 2001 [31], [32] reported that the resultant force of the static earth pressure acting on a conventional retaining wall model tested in a centrifuge environment using tilting tests was, in a broad sense, comparable with the theoretical total dynamic value based on the M-O method. Nakamura in 2006 [33] reported results of centrifuge testing of free-standing gravity retaining walls which were also not consistent with the M-O method.

Dewoolkar and others in 2001 [34] studied the effects of liquefiable backfills on fixed-base cantilever walls under harmonic accelerations using centrifuge testing. Results indicated that excess pore pressure and inertial effects play an important role on the total seismic lateral earth pressure. Watanabe and others in 1999 and 2003 [35], [36] showed that the seismic earth pressure is largely affected by the dynamic structural response of retaining walls further highlighting the importance that wall type plays. Additional centrifuge testing results from Watanabe and others in 2011 [37] revealed that the seismic active earth pressure was significantly smaller than that obtained from the M-O method, especially under a large seismic load. The results of these centrifuge experiments and other experiments generally showed the traditional methods being used in practice were typically overpredicting seismic earth pressure on retaining walls.

Shake table tests under 1-g loading using large or full-scale rigid or laminar boxes have also been used to study seismic lateral earth pressures. Wilson and Elgamal in 2010 [38] published the results of shake table tests which studied the response of a vertical reinforced concrete test wall section. They measured the dynamic earth pressure and concluded that accurate consideration of the backfill shear strength and the wall-soil interaction may lead to more realistic estimates of dynamic earth pressures. Ebeido and others in 2019 [39] published the results of different tests using the world's largest outdoor shake table and large laminar and rigid boxes. Their large-scale test results indicate that at high peak accelerations, recorded earth pressures exceeded values suggested by Seed and Whitman [3] by up to approximately 30 percent. However, at low peak accelerations, recorded values were lower than suggested by Seed and Whitman [3], further complicating the use of traditional analytical methods.

Professor Nicholas Sitar at the University of California Berkeley developed an aggressive and comprehensive research program using centrifuge testing to evaluate seismic lateral earth pressures on retaining structures and applicability and validity of the M-O theory and other traditional methods [15], [16], [20], [40], [41], [42], [43]. These studies generally concluded that the distribution of seismic earth pressure generally increases with depth for yielding walls and can be reasonably modeled as having an upright triangular distribution with the point of application of the seismic resultant force generally at H/3 for cohesionless backfills. These studies also were consistent with the observations found in case histories in which, for walls designed using an adequate static factor of safety, these walls were able to adequately resist seismic accelerations up to 0.3g and even up to 0.4g for yielding cantilever walls. These studies also concluded that the M-O method, Mylokanis method, and Wood method are generally overly conservative for walls subject to high seismic accelerations. Furthermore, the centrifuge experimental study published by Wagner and Sitar in 2016 [41] showed that for deep stiff walls, such as the conditions often seen for basement walls, the M-O method [1], [2] and Seed and Whitman 1970 [3] method provide a reasonable estimate if adjusting the value of the horizontal seismic coefficient for depth of the wall and that the Wood 1973 method [8] should not be used for basement walls.

5. Standards and Recommendations Set Forth by Building Codes, Professional Organizations, and Public Agencies

5.1 2019 California Building Code (CBC 2019)

The 2019 California Building Code (CBC) [44], which is based on the 2018 International Building Code (IBC) [45] and the 2016 American Society of Civil Engineers (ASCE) Standard 7 (ASCE 7-16) [46], requires that, for structures in Seismic Design Categories D, E, and F, retaining walls retaining more than six feet of soil be designed including the seismic lateral earth pressure. The 2019 CBC [44] specifies that the calculation of the dynamic seismic lateral earth pressure on basement and retaining walls be based on the *design* ground motions, however, no requirements are specified with regards to the methodology to use for calculation of the seismic lateral earth pressure.

5.2 SEAOC (2019)

The Structural Engineers Association of California (SEAOC) provides recommendations for seismic design of cantilevered, free-standing retaining walls and basement walls [47]. For these walls, if the seismic earth pressure increment is calculated separately by the M-O method, it is recommended to be added to the static active earth pressure and not to the at-rest static earth pressure, even if designing a non-yielding, restrained wall. Additionally, the location of the resultant of the active and seismic earth pressures is recommended to be taken at one-third the height of the wall from the wall base. For basement walls, SEAOC [47] recommends that the relationship between the direction of ground motion and the resulting seismic forces should be taken into consideration to calculate whether seismic earth pressures should be included in the base shear and overturning moments.

5.3 County of Los Angeles (2017)

The County of Los Angeles [48] recommends that the total seismic load should be the sum of the static and dynamic load increments with the dynamic load increment, ΔP_{AE}, calculated consistent with the equations presented in the Candia and Sitar 2013 method [15] as discussed in Section 2.2.2 of this paper.

The County of Los Angeles recommends that the PGA value used for calculation of the dynamic load increment be calculated as $S_{DS}/2.5$, where S_{DS} is the short-period design spectral acceleration calculated per ASCE 7-16 [46], [48]. For cohesionless soils, the point of application of the dynamic load increment is recommended to be H/3. For soils with cohesion, the point of application may vary between 0.37H to 0.40H.

5.4 Caltrans (2020)

Per Caltrans guidelines [49], which are based on guidelines set forth by the American Association of State Highway and Transportation Officials (AASHTO) [50], the site-specific seismic design of ordinary earth retaining systems is not mandatory when the site-adjusted PGA per Caltrans standards is less than or equal to

0.6g and allowable wall displacement are allowed to be up to 5 inches. Allowable analysis methods for estimation of the seismic lateral earth pressure include the M-O method and the GLE method. When using a conventional method of analysis, Caltrans guidelines [49] specifies the fraction of the PGA value to use for the horizontal seismic coefficient, k_h, dependent on the allowable displacement of the retaining wall structure. For instance, Caltrans Standard Plans provide standard wall designs which assume an expected mean displacement of up to 5 inches using a k_h value of 0.2g for a PGA value of 0.6g (i.e. k_h is equal to 1/3*PGA for PGA values up to 0.6g in the Standard Plans). If desired expected displacements are to be less than 2 inches, Caltrans [49] recommends the k_h value to be equal to or greater than 0.5*PGA with the seismic force calculated as the larger of $P_{IR}+0.5P_{AE}$, $0.5P_{IR} + P_{AE}$, and $P_{IR}+P_a$. where P_{AE} is the resultant seismic active lateral earth pressure force, P_{IR} is the horizontal inertial force, and P_a is the static active lateral earth pressure force. No displacement analysis is required when using k_h values equal to or greater than 0.5*PGA. When k_h is less than 0.5*PGA, the P_{AE} force should be added to the P_{IR} force and a simplified displacement analysis should be performed to estimate the expected mean displacement of the wall.

5.5 Eurocode 8 (1998)

The Eurocode 8 [51] recommends that retaining wall structures be designed using the resultant force of the static active plus the dynamic earth pressures. The point of application of the dynamic lateral force is recommended to be applied at mid-height of the wall in the absence of a more detailed study considering the relative stiffness, type of wall movements, and relative mass of the retaining structure. For walls free to rotate about their toe, the dynamic force may be taken to act at the same point as the static force (i.e. H/3). Annex E of Eurocode 8 [51] describes in detail the recommended procedure for a simplified analysis based on the M-O method. Both horizontal and vertical ground motions are recommended to be considered for the M-O method. For retaining structures more than 10 meters high, Eurocode 8 [51] recommends a free-field, one-dimensional analysis of vertically propagating waves be performed and a more refined estimate of the average value of the peak horizontal soil accelerations along the height of the structure be calculated.

5.6 New Zealand Geotechnical Society, Inc. (2017)

The New Zealand Geotechnical Society (NZGS) 2017 publication [52] sets forth seismic performance guidelines for new retaining structures based on wall type. Six different wall types are identified in this publication including: retaining wall integral to a building, retaining wall supporting a building, retaining wall downslope and supporting building foundations, retaining wall upslope and within 1.5H of a building, retaining walls for access and services to a building (e.g., driveways), and other cases. The calculation of the design horizontal acceleration to be used for analysis of the seismic lateral earth pressure includes a topographic amplification factor and a wall displacement factor. Retaining structures of low to moderate risk and of simple form may be designed to resist earthquake loading by considering a simplified pseudo-static horizontal acceleration. High-risk retaining structures are recommended to be subject to a more sophisticated analysis. Flexible walls are recommended to be designed assuming development of active earth pressures behind the wall while stiff walls are designed using higher pressures derived from the inertia of the retained soil mass. Per the NZGS guidelines [52], flexible walls are to be designed using the M-O method with the seismic active earth pressure acting at H/3 above the base of the wall. In stiff walls that deflect between 0.1 to 0.2 percent of their wall height, the NZGS guidelines [52] recommends that the dynamic seismic increment can be taken as 0.6 times the horizontal seismic coefficient, k_h, multiplied by the unit weight of the backfill soil and height of the wall squared with uniform pressure distribution. The NZGS guidelines [52] recommend that the pressure distribution of a stiff wall change from a uniform distribution to a triangular distribution for walls with deflections increasing above 0.3 percent of their wall height.

6. Conclusions

While the work of Okabe [1], Mononobe and Matsuo [2], and others has significantly advanced seismic design, significant research, experimental and analytical technologies, and the understanding of seismic ground motions have significantly progressed since the induction of the M-O method. Due to this progress, conclusions can be made that the more traditional methods of analysis for estimation of the seismic lateral earth pressure are somewhat outdated and generally overestimate the seismic earth pressure on retaining structures, particularly at sites with high design ground motions. While the GLE method has been a significant contribution to considering more complex wall conditions, practitioners have been reluctant to consistently apply this method based on the need for modeling software and identification of specific wall geometry. The latest trend in seismic design of retaining walls is advancing towards the use of displacement-based methodologies, such as that proposed by Brandenberg et al. [10].

While the use of displacement-based methods proposes a more updated, fundamentally sound approach to estimation of seismic lateral earth pressures, these methods have only recently been introduced and have been slow to be implemented in engineering practice. As more research may be needed to further gain consensus for use of a displacement-based method, the M-O method and/or other more traditional methods are likely to continue to be used. While using the traditional methods of analysis, the recent recommendations published from several experimental studies, professional organizations, and government agencies may be implemented. Consensus in being reached in the engineering community that an upright triangular distribution is more applicable for the seismic lateral earth pressure on retaining walls with the seismic incremental force applied at H/3. Furthermore, while there is still a general lack of agreement with regards to what value to use for the horizontal seismic coefficient, it should be noted that consensus has been to use design earthquake level values adjusted for site class and that experiments have shown that a height-adjusted seismic acceleration value may be more appropriate.

Further research is still needed to reach a consensus on updated methods to use for analysis of seismic lateral earth pressures on retaining walls, however, this paper summarizes the conclusions on recent studies and the current recommendations set forth by some governing agencies and organizations so that engineers may make a more informed decision when providing recommendations for seismic design of retaining walls.

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8. References

[1] Okabe S (1926): General theory of earth pressure and laboratory testing on seismic stability of retaining walls. Proc. Japanese Society of Civil Engineers, Vol. 12, No. 1, pp. 123-134 (in Japanese).

[2] Mononobe N, Matsuo M (1929): On the determination of earth pressures during earthquakes. Proceedings, World Engineering Congress 9: 179-187.

[3] Seed HB, Whitman RV (1970): Design of earth retaining structures for dynamic loads. ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell Univ., Ithaca, USA.

[4] Mylonakis G, Kloukinas P, Papantonopoulos C (2007): An alternative to the Mononobe–Okabe equations for seismic earth pressures, Soil Dyn. Earthquake Eng., 27, 957-969.

[5] Anderson DG, Martin GR, Lam I, Wang JN (2008): Seismic analysis and design of retaining walls, slopes and embankments, and buried structures. NCHRP Rep, 611.

[6] Chugh AK (1995): A Unified Procedure for Earth Pressure Calculations. Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Volume III, St. Louis, USA.

[7] Building Seismic Safety Council (2009): NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750). 2009 Edition.

[8] Wood JH (1973): Earthquake induced soil pressures on structures, *Report No. EERL 73-05*, California Institute of Technology, Pasadena, CA.

[9] Building Seismic Safety Council (2020): NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-2082-2), 2020 Edition, Volume I-II.

[10] Brandenberg SJ, Durante, MG, Stewart J (2020): Resource Paper 4 Seismic Lateral Earth Pressures. NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, Volume II: Part 3 resource Papers, September 2020.

[11] Brandenberg SJ, Mylonakis G, Stewart JP (2015): Kinematic framework for evaluating seismic earth pressures on retaining walls. J. Geotech. Geoenviron. Eng. 141(7), 04015031.

[12] Brandenberg, SJ, Mylonakis G, Stewart JP (2017): Approximate solution for seismic earth pressures on rigid walls retaining inhomogeneous elastic soil, Soil Dyn. Earthquake Eng., 97, 468–477.

[13] Prakash S, Saran S (1966): Static and dynamic earth pressures behind retaining walls. Proceedings of the Third Symposium on Earthquake Engineering, University of Roorkee, Roorkee, India, Vol. 1, pp. 277-88.

[14] Chen WF, Liu XL, 1990. *Limit analysis in soil mechanics*. Elsevier, Amsterdam, The Netherlands.

[15] Candia Agusti, G. and Sitar, N. "Seismic Earth Pressures on Retaining Structures in Cohesive Soils." Report submitted to the California Dept. of Transportation (Caltrans), Report No. UCB GT 13-02, August 2013, 161p.

[16] Geraili Mikola R, Sitar N (2013): Seismic Earth Pressures on Retaining Structures in Cohesionless Soils. Report submitted to the California Department of Transportation (Caltrans), Report No. UCB GT 13-01, March 2013.

[17] Candia Agusti G (2013): Experimental and Numerical Modeling of Seismic Earth Pressures on Retaining Walls with Cohesive Backfills. PhD Dissertation, Berkeley, USA.

[18] Gazetas G, Psarropoulos P, Anastasopolous, I, Gerolymos N (2004): Seismic behavior of flexible retaining systems subjected to short-duration moderately strong excitation. Soil Dyn. and Earthquake Engineering (24), 537-550.

[19] Shirato M, Fukui J, Koseki J (2006): Current Status of Ductility Design of Abutment Foundations Against Large Earthquakes. *Soils and Foundations*, 46(3), 377–396.

[20] Sitar N, Geraili Mikola R, Candia Agusti G (2012): Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls. Keynote Lectures from GeoCongress 2012.

[21] Koseki J, Tatsuoka F, Munaf Y, Tateyama M, Kojima K (1998): A modified procedure to evaluate active earth pressure at high seismic loads, Soils and Foundations, Special Issue on Geotechnical Aspects of the January 17, 1995 Hyogoken-Nambu Earthquake, 2, 209–216.

[22] Lew M, Simantob E, Hudson ME (1995): Performance of Shored Earth Retaining Systems during the January 17, 1994, Northridge Earthquake. Proceedings of the Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St Louis, USA.

[23] Ishii Y, Arai H, Tsuchida H (1960): Lateral earth pressure in an earthquake. Proceedings of the second World Conference on Earthquake Engineering: Tokyo and Kyoto, Japan, July 11-18, 1960. pp. 211-230.

[24] Matsuo H, O'Hara S (1960): Lateral earth pressure and stability of quay walls during earthquakes. Proceedings of the Second World Conference on Earthquake Engineering, Tokyo, Japan.

[25] Sherif MA, Fang YS (1984): Dynamic earth pressures on walls rotating about the top. Soils and Foundations 24(4), 109-117.

[26] Ishibashi I, Fang YS (1987): Dynamic earth pressures with different wall movement modes, Soils and Foundations. Japanese Soc. of Soil Mechanics and Foundation Engineering, 27(4), 11–22.

[27] Ortiz LA, Scott RF, Lee, J (1983): Dynamic centrifuge testing of a cantilever retaining wall. Earthquake Engineering and Structural Dynamics, 11, 251–268.

[28] Bolton MD, Steedman RS (1982): Centrifugal testing of micro-concrete retaining walls subject to base shaking. Proc. Conf. on Soil dynamics and Earthquake Engineering, 311-329.

[29] Bolton MD, Steedman RS (1984): The behavior of fixed cantilever walls subject to lateral shaking, Proceedings of Symposium on the Application of Centrifuge Modeling to Geotechnical Design, 302-314.

[30] Munaf Y (1998): Study on seismic performance of soil retaining walls by tilting and shaking table tests, Doctoral thesis, University of Tokyo.

[31] Koseki J, Munaf Y, Tatsuoka F, Tateyama M, Kojima K, Sato T (1998): Shaking table and tilt table tests of geosynthetic-reinforced soil and conventional retaining wall, Geosynthetics International, 5(1–2), 73–96.

[32] Koseki J, Watanabe K, Tateyama M, Kojima K (2001): Seismic earth pressures acting on reinforced-soil and conventional type retaining wall, Proc. of the International Symposium on Earth Reinforcement, 393–398.

[33] Nakamura S (2006). Reexamination of Mononobe-Okabe theory of gravity retaining walls using centrifuge model tests. Soils and foundations, 46(2), 135-146.

[34] Dewoolkar MM, Ko HY, Pak RY (2001): Seismic behavior of cantilever retaining walls with liquefiable backfills. Journal of Geotechnical and Geoenvironmental Engineering, 127(5), 424-435.

[35] Watanabe K, Maeda T, Kobayashi Y, Towhata I (1999): Shaking table tests on seismic earth pressure exerted on retaining wall model, Proc. of the 2nd Int. Conf. on Earthquake Geotechnical Engineering, Lisbon, 1, 297–302.

[36] Watanabe, K., Munaf, Y., Koseki, J., Tateyama, M. and Kojima, K. (2003): Behaviors of several types of model retaining walls subjected to irregular excitation, Soils and Foundations, 43(5), 13–27.

[37] Watanabe K, Koseki J, Tateyama M (2011): Seismic Earth Pressures exerted on Retaining Walls under a Large Seismic Load. Soils and Foundations, Vol. 51, No. 3, 379-394.

[38] Wilson P, Elgamal A (2010): Large Scale Experimental and Numerical Dynamic Earth Pressure Investigation. Proc. Earth Retention 2010, Bellevue, USA.

[39] Ebeido A, Zayed M, Kim K, Wilson P, Elgamal A (2019): Large Scale Geotechnical Shake Table Testing at the University of California San Diego, Proceedings of the 2nd GeoMEast International Congress and Exhibition on Sustainable Civil Infrastructure.

[40] Al Atik L, Sitar N (2010): Seismic Earth Pressures on Cantilever Retaining Structures. J. Geotech. and Geoenvir. Engrg., ASCE, 136 (10), 1324-1333.

[41] Wagner N, Sitar N (2016): On seismic response of stiff and flexible retaining structures. Soil Dynamics and Earthquake Engineering, 91, 284–293.

[42] Candia Agusti G, Geraili Mikola R, Sitar N (2016): Seismic response of retaining walls with cohesive backfill: Centrifuge model studies. Soil Dynamics and Earthquake Engineering, 90, 411–419,

[43] Geraili Mikola R, Candia Agusti G, Sitar N (2016): Seismic Earth Pressures on Retaining Structures and Basement Walls in Cohesionless Soils. J. Geotech. Geoenviron. Eng., 10, 1061

[44] California Building Standards Commission (2019): 2019 California Building Code. California Code of Regulations, Title 24.

[45] International Code Council (2018): International Building Code.

[46] American Society of Civil Engineers (2016): Minimum Design Loads and Associated Criteria for Buildings and other Structures, ASCE/SEI 7-16.

[47] Structural Engineers Association of California (2019): Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls. The SEAOC Blue Book: Seismic Design Recommendations, Sacramento, CA.

[48] County of Los Angeles (2017): Design of Retaining Walls. Building Code Manual, No. 35, 1807.2, Article 1.

[49] California Department of Transportation (2020): Seismic Design Criteria for Earth Retaining Systems, Structure Technical Policy.

[50] American Association of State Highway and Transportation Officials (2017), “AASHTO LRFD Bridge Design Specifications,” 8th Edition.

[51] European Committee for Standardization (1998): Eurocode 8: Design of structures for earthquake resistance - Part 5: Foundations, retaining structures and geotechnical aspects. European Standard EN-1998-1.

[52] New Zealand Geotechnical Society (2017): Earthquake Geotechnical Engineering Practice, Module 6: Earthquake resistant retaining wall design. Ministry of Business, Innovation and Employment.