Seismic Design of Restrained Rigid Walls

Conception Sismique de Murs Rigide de Soutien

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ABSTRACT: In current practice, the increment of seismically induced earth pressure on a rigid, non-yielding wall is generally taken as the product of seismic coefficient (k_H) and the soil mass behind the wall, $\Delta P_E = k_H \cdot \gamma H^2$, as developed by Wood (1973) and modified by Whitman (1991). Wood's study and most of the research thenceforth were based on the assumption that the wall and the retained soil are connected to a rigid base. This assumption neglects the fact that comparing to the mass mobilized by an earthquake, the size of the wall and its retained soil are relatively small. In other words, even though the wall is connected to a relatively rigid base, the base exhibits certain movement during an earthquake. In addition, there is much disagreement in current practice related to the value of k_H used with respect to the relationship to peak ground acceleration (PGA). In this study, a series of elasto-plastic pseudostatic finite element analyses were performed to assess the appropriateness of the Wood (1973) equation for determining the seismically induced lateral earth pressures on the stem of the restrained wall, and relationships were established between k_H and PGA based on the momentum conservation law. The results indicate that the increment of seismic earth pressure acting on non-yielding wall is a function of k_H and the supporting condition. They also indicate that a value of k_H of 25% of PGA seems reasonable and somewhat conservative for the design of normal structures.

RÉSUMÉ: L'augmentation de pression des terres induite par un séisme sur un mur rigide et résistant à une rupture est généralement calculée comme le produit du coefficient sismique (k_H) et de la masse de sol retenu derrière le mur, $\Delta P_E = k_H \cdot \gamma H^2$, comme développé par Wood (1973) et modifié par Whitman (1991). L'étude de Wood et la plupart de ses recherches ont été basées sur l'hypothèse que le mur et le sol qu'il retient sont connectés à une base rigide. Cette hypothèse néglige le fait qu'en comparaison de la masse mobilisée par un tremblement de terre, la taille du mur et du sol qu'il retient sont relativement petites. En d'autres termes, même si le mur reste connecté à une base relativement rigide, la base subit certains déplacements durant un tremblement de terre. Des désaccords existent sur la valeur de k_H par rapport à l'accélération maximale du sol (PGA). Dans cette étude, une série d'analyses aux éléments finis élasto-plastiques pseudostatiques a été réalisée pour évaluer la validité de l'équation de Wood (1973) pour déterminer les pressions sismiques latérales induites sur le pied du mur de soutènement et la relation entre k_H et PGA a été examinée en se basant sur la loi de conservation des moments. Les résultats indiquent que l'augmentation de pression terrestre sismique agissant sur le mur est une fonction de k_H et de la condition de support. Il est également établi qu'une valeur de k_H de 25% du PGA semble raisonnable et également conservative pour la conception de structures normales.

KEYWORDS: earth pressure, restrained, seismic, non-yielding wall, pseudostatic seismic coefficient, peak ground acceleration

1 INTRODUCTION

Since the late 1920s, seismic earth pressure acting on retaining walls has been widely studied by researchers. Okabe (1926) pioneered the research by introducing pseudostatic force into Coulomb earth pressure theory. Mononobe and Matsuo (1929) finalized Okabe's theory and established the well-known Mononobe-Okabe (M-O) method, which continues to be widely used in current practice despite many criticisms and its limitations. Similar research and simplifications have been conducted since then (Seed and Whitman, 1970). The M-O method is based on an important assumption that the wall structure displaces a sufficient amount to develop a fully plastic stress state in the soil near the wall, and thus is generally applied for cantilever walls. However, some wall structures, such as massive gravity walls founded on rock or basement walls or bridge abutments restrained on the top and bottom, do not move sufficiently to mobilize the shear strength of the soil and thus the M-O method cannot be directly applied. Motivated by the lack of well-defined design procedures and design data for evaluating seismic earth pressures on such wall structures, Wood (1973) analyzed the response of a homogeneous linear elastic soil trapped between two rigid walls connected to a rigid base and obtained an equation for seismic earth pressure as $\Delta P_E = F k_H \cdot \gamma H^2$. Whitman (1991) suggested the value of F approximately equal to unity. While a great deal of additional research has been performed since the 1970s (Veletsos and Younan, 1994; Wu and Finn, 1999; Ostadan, 2004; Maleki and Mahjoubi, 2010), Wood's equation is still recommended by various authorities (FEMA, 2003; FHWA, 2009) in current practice.

Wood's study and most of the research thenceforth (Ostadan, 1998, 2004; Maleki & Mahjoubi, 2010) were based on the assumption that the wall and the retained soil are connected to a rigid base. This assumption neglected the fact that, compared to the mass mobilized by an earthquake, the size of the wall and its retained soil are relatively small. In other words, even if the wall is connected to a relatively rigid base, the base exhibits certain movement during an earthquake event, and the wall will also move together with the supporting base, even it is "locally" restrained. This will certainly result in changes in earth pressures on the back of the wall.

Another important issue in the seismic earth pressure equations is the lack of clarity of the pseudostatic seismic coefficient (k_H). Conflicts are even exhibited in the specifications. For example, ASSHTO (2010) suggests k_H equal to the half of PGA while the NCHRP report (Anderson et al.,

2008) and FHWA (Berg et al., 2009) recommended k_H be approximated to the site corrected PGA for walls less than approximately 6 meters. Few publications can be found that contain detailed discussions on this issue. Therefore, this paper will present an approach for estimating seismic earth pressures acting on rigid walls by considering more general base and wall conditions and to provide a more detailed discussion with respect to the relationship between k_H and PGA by considering the momentum equivalent. Simplified seismic earth pressure equations directly related to PGA are proposed for use by engineers in their daily practice.

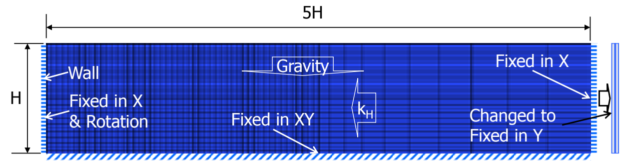
2 PSEUDOSTATIC NUMERICAL SIMULATION

Although response analysis is a good method for the evaluations of seismic earth pressures, it is usually complicated and not suitable for routine practice. In this study, pseudostatic numerical simulations were initially performed for various conditions. Simplified equations were then derived based on the numerical simulation results.

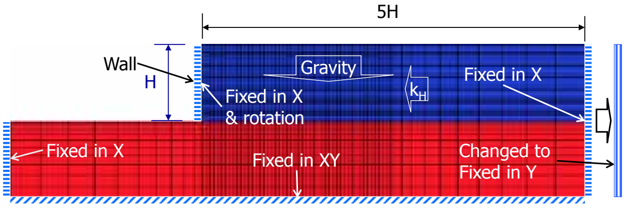
2.1 *Finite element method (FEM) modeling*

Three cases were considered, 1) a restrained rigid wall and its retained soils supported by a rigid base, 2) a restrained rigid wall and its retained soils supported by a non-rigid base, and 3) a rigid wall and its retained soils supported by a non-rigid base, where a *restrained rigid wall* refers to a wall without horizontal as well as rotational movement and a *rigid wall* refers to a wall restricted in rotational movement only. Comparing the scale of the wall and its retained soil mass, the mass mobilized by an earthquake is much larger and the area of movement is much broader. As such, Case 3 is considered to be more representative to real situation.

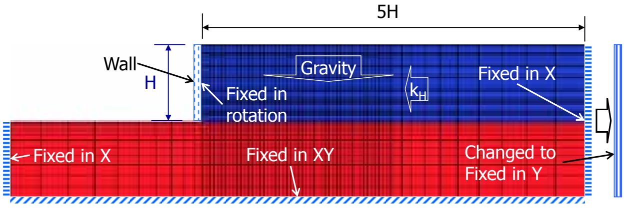
The finite element models for the three cases are shown in Figure 1. In Cases 1 and 2, the wall was completely fixed both in the *x*-direction and rotation. In Case 3, the wall was only fixed for rotational movement and was allowed to move horizontally together with the deformation of the base at the bottom of the wall. To delimit the boundary effects, a model width was taken as 5 times the wall height. Furthermore, the right-side boundary was switched from fixed in the *x*-direction under gravity load to free in *x*-direction when inertial force was applied. Details are shown in Figure 1.



a) Restrained rigid wall on rigid base



b) Restrained rigid wall on non-rigid base



c) Rigid wall on non-rigid base

Figure 1. Finite element modeling

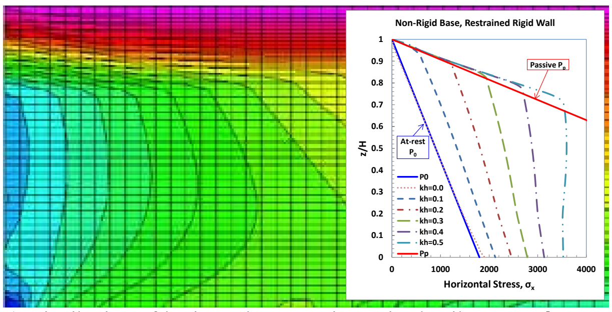
Soil retained by the wall was assumed to be elasto-plastic material and modeled using Mohr-Coulomb failure criteria. The initial modulus of elasticity of this layer was chosen such that it represents a dense sand material. Internal frictional angles varying from 30 to 38 degrees were analyzed to confirm the effect of strength parameters. To model the non-rigid base, a layer of material with a modulus of elasticity corresponding to soft bedrock material was utilized below the wall and its retained soils. The depth of this layer was taken as equal to the height of the wall. To shorten the calculation time, the material comprising this layer was assumed to be linear elastic.

After finishing the FEM modeling, the calculations were performed in steps. The stress field under gravity force was calculated in the first step. Inertial forces were then applied by adding horizontal seismic coefficients in the subsequent steps. The right-side boundary was switched from fixed in the *x*-direction and free in the *y*-direction to free in the *x*-direction and fixed in the *y*-direction so that no tension would be created in the soils near the right-side boundary.

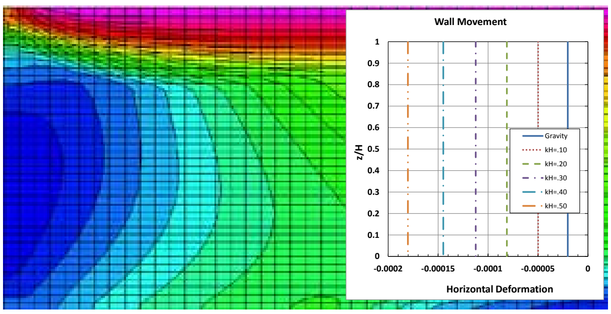
A commercial finite element analysis program, Strand7, was utilized in the analysis.

2.2 *Earth pressures under pseudostatic load*

The typical horizontal stress distributions for a seismic coefficient of $k_H = 0.5$ in the retained soil mass behind the wall are shown as color contours in Figure 2 for a) a restrained rigid wall on non-rigid base and b) a rigid wall on non-rigid base, respectively. Figure 2 a) also shows the horizontal stress, or earth pressure, distribution behind the wall for a seismic coefficient that varied from 0 to 0.5. It can be seen that the calculated stress distribution under $k_H = 0.0$ (gravity only) is consistent with at-rest earth pressure calculated by the equation, $(1 - sin\phi)\gamma H$. It was also noticed that the distributions of seismic earth pressures fall into a zone which is defined with the at-rest earth pressure as the lower boundary and with the passive earth pressure as the upper boundary. Theoretically, this is true. Considering the relative movement, an applied inertial force should be equivalent to passive wall movement. As such, an increase in inertial force will eventually result in a passive-type failure within the soil mass.



a) Distribution of horizontal stresses in retained soil mass at $k_H = 0.5$ and pressure distributions on the wall for various k_H



b) Distribution of horizontal stresses in retained soil mass at $k_H = 0.5$ and wall movement for various k_H

Figure 2. Typical results of pseudostatic finite element analysis, a) restrained rigid wall on non-rigid base; b) rigid wall on non-rigid base

The total seismic earth pressure, P_{0E}, was obtained by integrating the horizontal stress distributions as shown in the *xy*-graph of Figure 2 a). By subtracting the total at-rest earth pressure, the total increment of seismic earth pressure, ΔP_{0E}, was then calculated. Figure 3 shows the normalized ΔP_{0E} versus seismic coefficient for internal frictional angles varying from 30 to 38 degrees for Case 3. It can be seen that the effect of shear strength of the retained soil to the normalized ΔP_{0E} is insignificant.

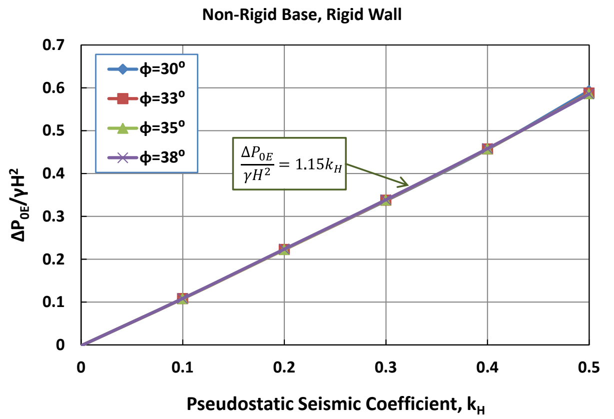


Figure 3. The relationship of normalized total increment of seismic earth pressure versus seismic coefficient for various strength parameters

Figure 4 shows the normalized ΔP_{0E} versus seismic coefficient for all of the three cases. For each case, the multiple points at the same seismic coefficient indicate the results of different strength parameters. It is clear that although based on different approaches, the results for the case of a restrained rigid wall connected to a rigid base are consistent with those obtained by Wood (1973). However, if the rigid wall and its retained soil are supported on a non-rigid base, the seismic earth pressure increment will be approximately 15 to 17% higher. Considering the anticipated real movement of the wall system during an earthquake, it is recommended to calculate the seismic earth pressure increment using the following equation:

$$\frac{\Delta P_{0E}}{\gamma H^2} = 1.15k_H \qquad (1)$$

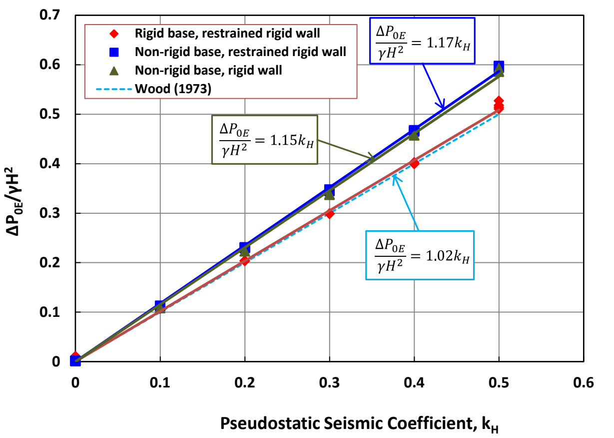


Figure 4. The relationship of normalized total increment of seismic earth pressure versus seismic coefficient for various conditions.

2.3 *Thrust point*

To design the stem of the wall, the distribution of earth pressure is important. Several studies were intended to provide such a distribution (Wu and Finn, 1999; Ostadan, 2004; Maleki and Mahjoubi, 2010). However, in routine practice, a total force and its thrust point are easier to handle and calculate.

The equivalent thrust point was calculated based on the calculated earth pressure distributions such as shown in the *xy*-graph of Figure 2 a). Figure 5 shows the normalized thrust point of the total earth pressure (including static and seismic) as a function of the seismic coefficient. The thrust point moves upward with the increase in seismic coefficient and can be conservatively approximated by the following equation.

$$z/H = 0.55(k_H + 0.1)^{0.2} \qquad (2)$$

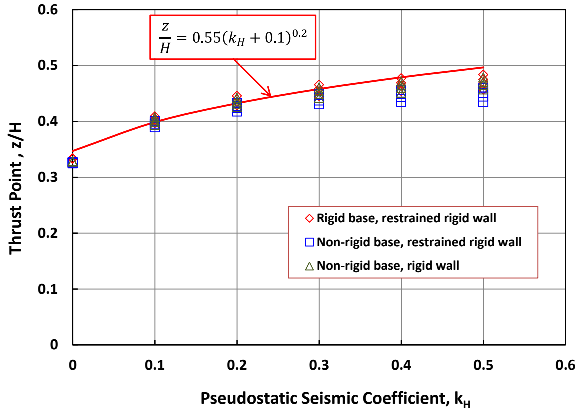


Figure 5. The relationship of normalized thrust point of total earth pressure versus seismic coefficient

3 SEISMIC COEFFICIENT

Due to its irregularity, direct use of the acceleration time history of an earthquake is usually difficult. Researchers typically use equivalent approaches to deal with the problem. For example, a harmonic wave is utilized in the laboratory for cyclic shear tests and an equivalent uniform value equal to 65% of peak cyclic stress is used in simplified liquefaction analysis procedures. In the case of seismic earth pressure, a pseudostatic seismic coefficient (k_H) is usually utilized. However, significant variations exists in k_H, varying from 1/3 PGA (Kramer, 1996) to as high as 100% of PGA (FEMA, 2003; HCHRP, 2008).

Yi (2011) proposed a method to establish the relationship between k_H and PGA based on momentum equivalent: that is, the total momentum created by irregular acceleration, $a(t)$, to a soil mass, m, should be equivalent to that created by the seismic coefficient, k_H:

$$\sum ma(t)\Delta t = \sum mk_H\Delta t \qquad (3)$$

At any time *t*=*T*, Equation (3) can be rewritten as

$$(k_H)_T = \sum a(t)\Delta t/T \qquad (4)$$

Figure 6 shows the normalized equivalent k_H for 35 acceleration time history records from 14 earthquakes where PGA is varying from 0.04g to 1.15g. The results indicate that, except for the El Centro Earthquake record (EW), the equivalent k_H is generally less than 0.25 PGA.

Based on the results of centrifuge model tests, Al Atik and Sitar (2010) obtained a relationship between dynamic earth pressure coefficients, ΔK_{ae}, and PGA, as shown in Figure 7, by back-calculations. By introducing Equation (1), this relationship could be modified to a relationship with k_H as shown on the second *y*-axis in Figure 7. A line for $k_H = 0.25PGA$ is also plotted in this figure. It can be seen that even taking 25% of PGA, it may still be conservative. This is consistent with Figure 6, that 0.25 is generally the maximum value for most acceleration records.

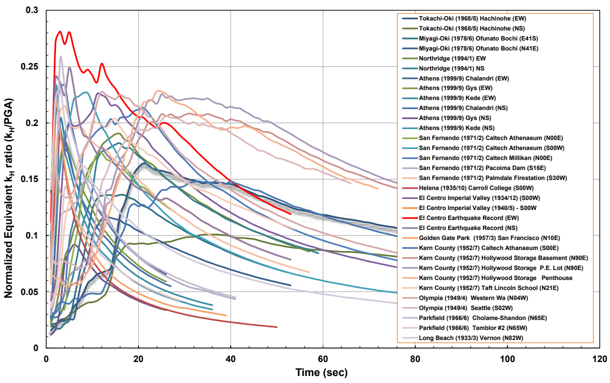


Figure 6. Normalized equivalent k_H versus time for 35 acceleration records from 14 earthquakes.

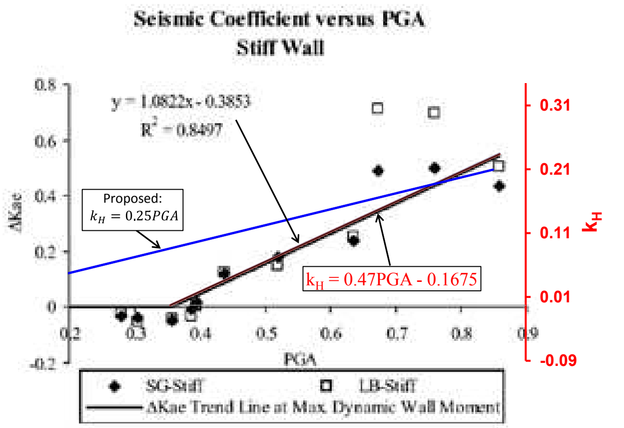


Figure 7. Back-calculated dynamic earth pressure coefficients at time of maximum dynamic wall moments on stiff walls as function of peak ground acceleration measured at the top of soil in free field (after Al Atik and Sitar, 2009) with modification to relationship with k_H.

4 STATIC DESIGN VS. SEISMIC DESIGN

It is important to note that a wall designed for static lateral forces usually includes a factor of safety of 1.5. The factor of safety is then reduced to 1.1 for the occasional, temporary forces due to earthquakes. Therefore, a statically designed structure can withstand a seismic load of

$$F_{seismic} = 1.5/1.1\, F_{static} = 1.36 F_{static} \tag{5}$$

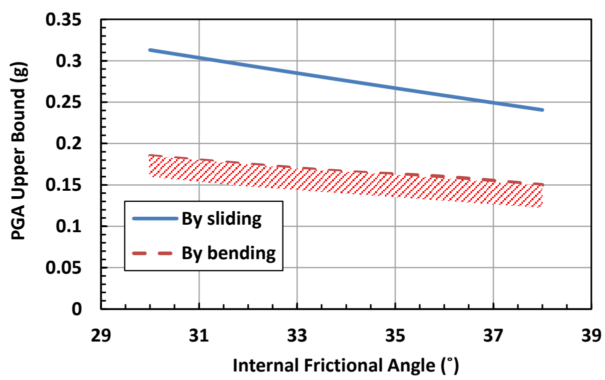


Figure 8. Upper bound of PGA covered by static design versus internal frictional angle

In other words, if the increment of seismic load is less than 36% of the static load, the static design will be adequate for the seismic condition. By introducing Equations (1) and (2), we can obtain the upper boundaries of PGA that are covered by static design as shown in Figure 8.

5 CONCLUSION AND RECOMMENDATIONS

While much research has been conducted on seismic earth pressures in the last 80 years, various questions are still arising. The author of this paper intended to have a more detailed examination of the seismic earth pressures of restrained walls as well as the seismic coefficient. Based on the results of serial elasto-plastic finite element analyses and the examination of the relationship between the seismic coefficient (k_H) and the peak ground acceleration (PGA), it is concluded that the increment of total seismic earth pressure acting on restrained walls be calculated using the following equation:

$$\Delta P_{0E} = 0.3 \cdot PGA \cdot \gamma H^2 \tag{6}$$

The thrust point of the total earth pressure (including static and seismic) can be calculated from Equation (2). If the peak ground acceleration is less than that graphed in Figure 8, seismic design may not be necessary.

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