# Seismic assessment of basement walls for different design criteria

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ABSTRACT
Current state of practice for design of basement walls in Vancouver is based on Mononobe-Okabe (M-O) theory using a code based Peak Ground Acceleration (PGA)=0.46g (NBCC 2005). Under the previous version of the building code walls were designed for a PGA=0.24g (NBCC 1995). A series of dynamic numerical analyses is being conducted to study the seismic performance of basement walls designed using M-O earth pressures derived using both NBCC (1995) and NBCC (2005) PGA. The walls are subjected to seven ground motions spectrally matched to the UHS prescribed by NBCC (2005) for Vancouver. The analyses show that current engineering practice for designing basement walls based on the M-O approach and using the PGA of the current hazard level (NBCC 2005) may be conservative. In the analyses walls designed using M-O earth pressures with PGA=0.24g (hazard level specified by NBCC 1995) result in acceptable drift except perhaps at the first story where the drifts reach 3%.

RÉSUMÉ
La conception de murs de sous-sol à Vancouver est actuellement basée dans sur la théorie Mononobe-Okabe (M-O) utilisant un code basé sur l'accélération maximale du sol (PGA) = 0.46 g (NBCC 2005). Dans la version précédente du code du bâtiment, les murs étaient conçus pour un PGA=0.24g (NBCC 1995). Des séries d'analyses numériques dynamiques sont en cours afin d'étudier la performance sismique des murs de sous-sol conçus en utilisant les pressions des terres M-O utilisant le PGA de NBCC (1995) et NBCC (2005). Les murs sont soumis à sept mouvements du sol spectralement adaptés au UHS prescrit par NBCC (2005) pour Vancouver. Les analyses montrent que la pratique en cours du génie lors de la conception des murs de sous-sol basée sur l'approche M-O et utilisant le PGA du présent niveau de risque (NBCC 2005) peut être trop conservatrice. Dans les analyses murs conçus en utilisant la pression des terres M-O avec un PGA = 0.24 (niveaux de risque spécifiés par NBCC 1995) présentent des niveaux de dérives acceptables, sauf peut-être au premier étage où les dérives atteignent 3%.

## 1 INTRODUCTION

Structural and geotechnical engineers have long relied upon the use of the Mononobe-Okabe (M-O) method (Mononobe and Matsuo, 1929 and Okabe, 1924) for determining seismic lateral pressures acting on retaining walls. Based on the Coulomb theory and the M-O method the active static and the total (static and earthquake induced) active lateral thrusts on a wall are given by $P_A=\gamma H^2 K_A/2$ and $P_{AE}=\gamma H^2 K_{AE}/2$, respectively, applicable for a dry granular backfill where $K_A$ and $K_{AE}$ are the active earth pressure coefficient without and with the earthquake effect, $\gamma$ is the soil unit weight, and $H$ is the retaining wall height. For a straight wall with level backfill, $K_A$ and $K_{AE}$ are mainly functions of the friction angle of the soil and the angle of wall friction. The $K_{AE}$ is also a function of the horizontal and vertical coefficients of Peak Ground Acceleration (PGA). The M-O method only provides the lateral thrust, $P_{AE}$. It does not explicitly indicate anything about the distribution of lateral earth pressure from seismic events. Several studies have been conducted for investigating the distribution of the lateral earth pressures and the point of application of the resultant lateral forces, depending on the mode of deformation of the wall (e.g., see Seed and Whitman 1970, Sherif et al. 1982, and Sherif and Fang 1984). A current simplified method often used for basement walls in practice in the Vancouver area for finding the distribution of the total lateral earth thrust is as follows. The static active lateral thrust $P_A$ is distributed linearly along the wall height as a triangle with zero pressure at the surface. The dynamic component of active lateral thrust ($\Delta P_{AE}=P_{AE}-P_A$) is distributed as an inverse triangle along the height of the wall with zero pressure at the base of the wall (Seed and Whitman 1970). These distributions of lateral earth pressures are then used by structural engineers to find the design moments and shears in the walls and eventually simplified design of the walls.

For the previous version of the building code (prior to 2005) the PGA for Vancouver, British Columbia used in the M-O method was 0.24g which had an exceedance rate of 10% in 50 years. Since 2005 PGA=0.46g was often used for design. This has an exceedance rate of 2% in 50 years. Using the M-O method with the higher PGA resulted in a demand increase of about two over the previous code version and raised some concerns from design engineers on the applicability of the M-O method for the basement walls, especially when considering the generally good performance of basement walls (which were often designed for much lower earth pressures) in past earthquakes. In addition, the M-O limit equilibrium-based method was originally developed for rigid retaining walls with sufficient rigid body displacements to mobilize the active wedge in the soil. In reality, however, certain types of retaining walls, such as basement walls, have variable degree of flexibility and deformation at different depths.

Recently, the Structural Engineers Association of British Columbia (SEABC) initiated a task force to review current design procedures for basement walls. The first step in the study was to evaluate how walls designed using M-O earth pressures with PGA=0.24g would behave when subjected to the new hazard of PGA=0.46g. A series of dynamic numerical analyses, using the computer program FLAC, was conducted to study the response of basement walls to seismically induced lateral earth pressures, taking into account the flexibility and potential yielding of the wall components (Ahmadnia et al., 2011). In that study, the performance of the basement wall subjected to three ground motions representing the current hazard level prescribed by NBCC 2005 was assessed. Simulations in that study showed a large drift ratio in the wall at the top basement level for one of the applied ground motions. In the extension of this study, the present paper examines in more detail the performance of a basement wall designed using M-O earth pressures with a 10% probability of exceedance in 50 years (using PGA = 0.24g) and also basement wall designed using M-O earth pressures with a 2% probability of exceedance in 50 years (using PGA=0.46g) subjected to seven ground motions representing the current hazard level prescribed by NBCC 2005. In addition to these two walls, the performance of a third wall (W3) is also examined. This wall was designed for an earth pressure that was the same as case W2 (MO earth pressure with PGA for 2% probability of exceedance in 50 years) but with the dynamic portion of the earth pressure reduced by a factor of 1.3. This is to account for the wall having a structural overstrength ratio of R0=1.3. The dynamic analysis results are compared to those of pseudo-static M-O method in terms of the resultant forces and the lateral earth pressure distributions on the walls. The results indicate that the flexibility of the walls has important effects on the distribution of the seismic lateral pressures. This paper outlines some of the significant findings in this study.

## 2 WALL DESIGN

Three basement walls were designed by structural engineers for this study, using earth pressures derived from the pseudo-static M-O method. PGA values of 0.24g and 0.46g were used for design of walls W1 and W2. These PGA values correspond to the probability of exceedance of 10% in 50 years (NBCC1995) and 2% in 50 years (NBCC2005), respectively. The lateral earth pressure distributions (including static and seismic components) for walls W1 and W2 are shown in Figures 1(b) and 1(c), respectively. The M-O pressure for wall W2 was modified at the top using a "passive" cut off, with a coefficient of about 3 for the passive pressure. The maximum of these total pressures or 1.5 times the static active pressures are used to find the design moments at

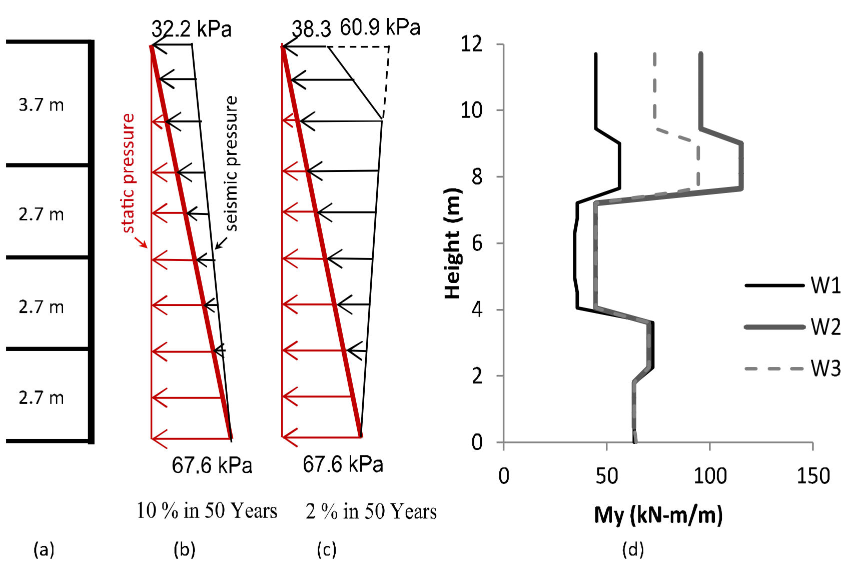


Figure 1: (a) The geometry of the basement wall with floor heights, (b) lateral earth pressure distribution for wall W1 designed to PGA=0.24g, (c) lateral earth pressure distribution for wall W2 designed to PGA=0.46g, (d) resulting moment capacity distribution (real member's strength) along the height of the walls W1-W3.

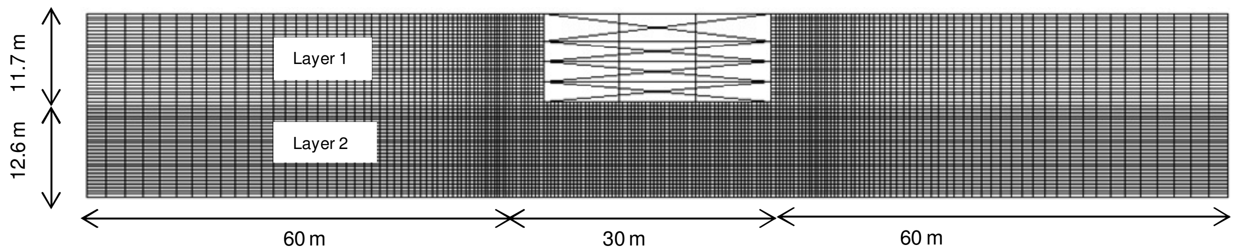

Figure 2: FLAC model.

each depth of the wall and then the wall was designed by the structural engineer for these moments in the usual manner using factored resistances of $0.85f_y$ and $0.65f'_c$ for steel and concrete, where $f_y$ and $f'_c$ are the corresponding nominal yield strengths. Wall W3 was designed for a reduced seismic earth pressure to take into account a structural overstrength factor of $R_0$=1.3. Finally approximately 1.3 times of the above mentioned design moments were used as the moment capacity (real member's strength) profile along the height of each wall for evaluating the response of the walls to a suite of seven ground motions, as explained in the next section. The profiles of the moment capacity along the height of these three walls are shown in Figure 1(d). Uniform properties of Moment of Inertia, $I$=0.0013 $m^4$/m, cross-section area, $A$=0.25 $m^2$/m, and Young's modulus, $E$=2.74×10$^7$ kN/m$^2$ are considered along the height of all basement walls.

## 3 METHOD OF ANALSIS

Nonlinear seismic response of the basement wall is analyzed by using the two-dimensional finite difference computer program FLAC 6.00 (Itasca, 2008). In order to ensure the proper initial stress distribution on the basement the actual construction sequence is modeled. First, a 24.3 m deep and 150 m wide layer of soil is created and brought to equilibrium under gravity forces. The model consists of two soil layers that will be discussed further in the next section. A part of the upper soil layer is then excavated in lifts to a depth of 11.7 m and a width 30m. As each lift was excavated, lateral pressures (shoring) are applied to retain the soil. Then the basement wall is constructed and the static analysis is repeated to establish the equilibrium static stress condition prior to the subsequent dynamic analysis. The flexural behavior of the walls is modeled by elastic-perfectly plastic beam model with yield moments equal to the corresponding yield moment values as shown in Figure 1(d). Finally, a gap between the basement and soil is backfilled. Then the shoring pressures are removed, allowing the load from the soil to transfer to the basement wall. The FLAC model used for analysis is depicted in Figure 2. This model was subjected suite of seven input ground motions at the base.

## 4 PROPERTIES OF SOIL

Constitutive response of the soil is modeled by a Mohr-Coulomb material model. The required model parameters, elastic bulk and shear moduli, cohesion, and friction and dilation angles of soil are constant with depth in each layer. The properties for soil layers 1 and 2 (see Figure 2) are presented in Table 1. Maximum elastic shear modulus, $G_{max}$, is assumed to change with depth using the relation proposed by Robertson et al. (1992) using constant stress corrected shear wave velocity of Vs1 of 200 and 400 m/s in soil layers 1 and 2, respectively. An equivalent shear modulus was determined using Shake analyses (Idriss 1992) with representative modulus reduction curves for the soils (sand lower bound, Seed & Idriss 1970). The average equivalent shear modulus was found to be 30% of the maximum elastic shear modulus, i.e. $G_{eq}$=0.3$G_{max}$. A nominal Rayleigh damping of 5% is also added for numerical stability.

## 5 GROUND MOTIONS

Ground motions were selected from the Pacific Earthquake Engineering Research (PEER) strong ground

Table 1: Soil material properties.

| Soil layer | Density (kg/m$^3$) | $Vs_1$ (m/s) | $G_{eq}/G_{max}$ | Poisson's ratio | Cohesion (kPa) | Friction angle (degrees) | Dilations angle (degrees) |
|---|---|---|---|---|---|---|---|
| 1 | 1950 | 200 | 0.3 | 0.4 | 0 | 33 | 0 |
| 2 | 1950 | 400 | 0.3 | 0.4 | 20 | 40 | 0 |

Table 2: List of selected ground motions.

| Ground Motion | NGA No. | Event Name | Year | Station | Magnitude | $Vs_{30}$ (m/s) |
|---|---|---|---|---|---|---|
| G1 | NGA57 | San Fernado | 1971 | Old ridge Route | 6.61 | 450.3 |
| G2 | NGA78 | San Fernado | 1971 | Palmdale Fire | 6.61 | 452.9 |
| G3 | NGA164 | Imperial Valley | 1979 | Cerro Prieto | 6.53 | 659.6 |
| G4 | NGA739 | Loma Preita | 1989 | Anderson Dam | 6.93 | 488.8 |
| G5 | NGA755 | Loma Preita | 1989 | Cyote Lake dam | 6.93 | 597.1 |
| G6 | NGA952 | Northridge | 1994 | Beverly Hills | 6.69 | 545.7 |
| G7 | NGA1787 | Hector Mine | 1999 | Hector | 7.13 | 684.9 |

motion database (PEER 2011). Based on the results of de-aggregation of the NBCC Uniform Hazard Spectrum (UHS), site class C for Vancouver, candidate input motions are selected in the magnitude range M=6.5–7.5 and the distance range 10–30 km using the program Design Ground Motion Library, DGML (Wang et al., 2009). Table 2 shows the list of seven ground motions selected for this study. The selected ground motions are linearly scaled to match UHS using the computer program DGML and then spectrally matched to UHS in the period range of 0.02–1.7sec using computer program SeismoMatch (Seismosoft 2009).

## 6 RESULT OF ANALYSIS

The time histories of the resultant forces on the three walls are shown in Figures 3(a-c). The resultant lateral force at each time is calculated by integrating the lateral earth pressure distribution along the height of the wall. The presented lateral earth force time histories in Figures 3(a-c) are from the analyses using G1 ground motion. The M-O method for the same hazard level gives approximately the same peak resultant force shown by the solid red lines in Figures 3(a-c). Figure 3(a) also shows the peak M-O force for a PGA of 0.24g, which corresponds to NBCC (1995), as a baseline for comparison. Many existing walls have been designed for this level of shaking and their performance at the current hazard level in NBCC (2005) is a matter of concern. This was a partial motivation for the present study.

The analysis results show that the maximum resultant earth force for each of the studied walls (W1-W3) subjected to ground motion G1 occurs at about the peak of the input motion G1 (t=2.8 sec). The pressure patterns at the instance of occurrence of maximum lateral earth force on the three walls are shown in Figures 3(d-f). Also shown for comparison is the earth pressure profile obtained using the M-O method with the 2% in 50 year return period PGA (0.46g). The distributed pressure

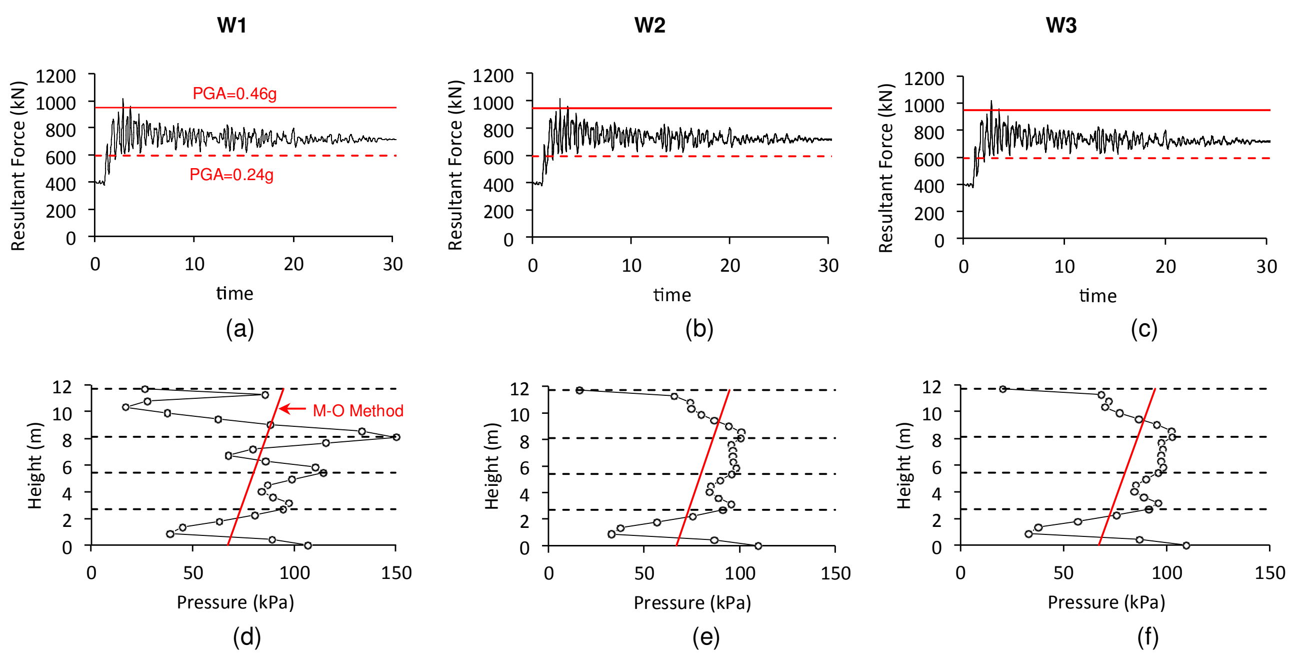


Figure 3: (a-c) Time histories of the resultant lateral earth forces on walls W1-W3 for ground motion G1, and (d-f) the pressure patterns on the three walls at the instance of occurrence of maximum lateral earth force on the walls. The red lines represent the calculated earth pressures using the simplified M-O based method with PGA=0.46g.

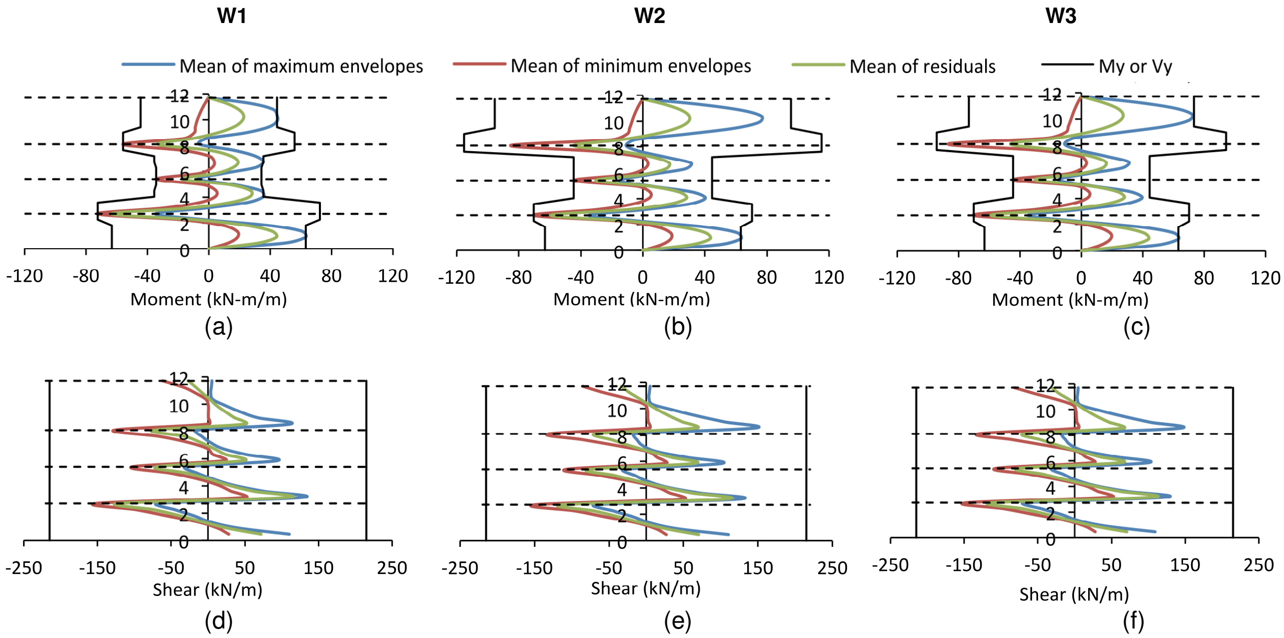


Figure 4: The averages of bending moment and shear envelopes, the residual bending moment and shear profiles for the G1-G7 ground motions, and the moment capacity (yield moment and yield shear) profiles for the basement walls W1-W3.

on the wall at the moment of peak horizontal force is more concentrated around the floor levels than between the floor levels. This is more evident in all levels for W1.

The average bending moment envelopes and shear envelopes over the wall height from the analyses using G1-G7 ground motions on walls W1-W3 are presented in Figures 4(a-c) and 4(d-f), respectively. Averages of both maximum and minimum envelopes are plotted for each wall. The limiting values of yield moment (My) and shear strength (Vy), shown in black in this figure, correspond to the moment and shear capacity (real member's strength) of each wall, as explained in section 2. Figure 4(a) shows that the basement wall W1 designed for a hazard level of 10% probability of exceedance in 50 years yields in moment at the mid-height of each basement story and also at each floor level. Figure 4(b) however shows that the basement wall W2 designed for a hazard level of 2% probability of exceedance in 50 years just yields at the mid-height of the bottom story and also at the lowest two floor levels. Figure 4(c) shows that the basement wall W3 designed with reduced seismic load to account for over strength factor of $R_0$=1.3 behaves similarly to W2 but only barely yields at the mid-height of the top story.

The shear envelopes in Figures 4(d-f) show that the shear demand is considerably less than the shear capacity along the height of the wall.

Given that the walls yield in moment, it is very important to monitor the resulting deformations and drift ratios of the wall. Drift ratio is a common parameter for assessing the performance of a structure. It is usually defined as the relative displacement between floor levels divided by storey height. Drift ratio in this paper is defined in the following equation with the associated displacements patterns are shown in Figure 5.

$$\text{Drift ratio} = \left[u_{\text{wall}} - \frac{u_{\text{floor,top}} + u_{\text{floor,bottom}}}{2}\right] / (0.5h)$$

This definition is consistent with the definition of hinge rotation used by Task Committee on Blast Resistance Design (TCBRG 1997). This committee related hinge rotation to structural performance. They specified two performance categories which may apply to basement walls; low and medium response categories. The Low Response Category is defined as 2% drift ratio: "localized building/component damage. Building can be used; however repairs are required to restore integrity of structural envelope. Total cost of repairs is moderate". The Medium Response Category is defined as 4% drift ratio: "widespread building/component damage. Building cannot be used until repaired. Total cost of repairs is

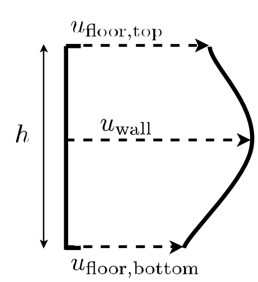


Figure 5: Definition of drift ratio for each story of the basement wall.

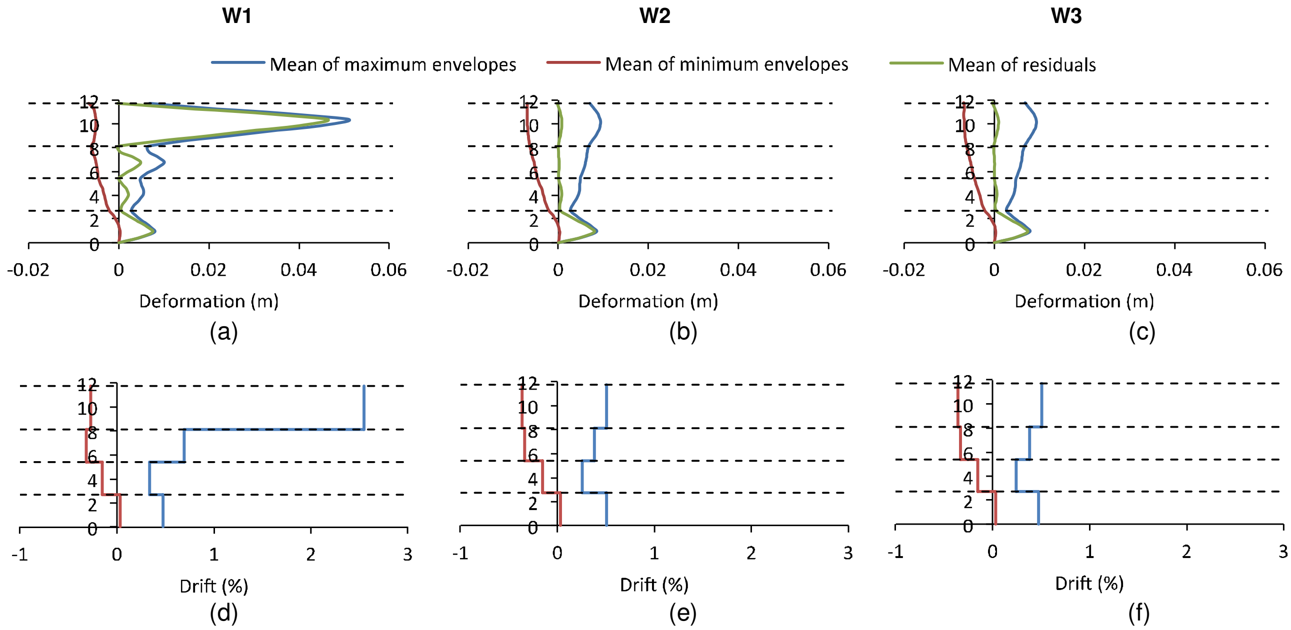


Figure 6: (a-c) Mean envelopes of maximum, mean envelopes of minimum, and mean residual wall deformations (displacements relative to the base of the basement wall), and (d-f) mean envelop of maximum and mean envelopes of minimum drift ratios for the G1-G7 ground motions in the basement walls W1-W3.

significant".

The wall displacements (relative the base of the walls) are shown in Figure 6(a-c) and the associated drift ratios in Figures 6(d-f). The results suggest that the walls designed for PGA=0.46g using the M-O approach are probably overdesigned because the drift ratios are less than about 0.5%. For the wall designed for PGA=0.24g the performance seems adequate except for the first level basement where the drift ratio approached 3%. At the other basement levels the drift ratio is less than 0.5%. Based on the current numerical analyses, for a hazard level of 2% in 50 years in Vancouver, design to the associated PGA=0.46g may not be warranted. An additional series of analyses is being carried out to determine at what hazard level the PGA should be selected for the M-O method to give an economical design for the 2% in 50 years hazard. In effect, we are establishing a database for determining an appropriate effective PGA for use in M-O analysis. A problem in defining satisfactory performance in terms of drift ratios is the lack of any performance standards for basement walls in terms of drift ratios.

## 7 SUMMARY AND CONCLUSION

The aim of this study was 1) to assess the behavior of existing walls designed using M-O earth pressures using PGA from a seismic event with 10% probability of exceedance in 50 years (NBCC1995) but subjected to a suite of earthquakes representative of an event with 2% probability in 50 years (NBCC 2005), 2) to assess the behavior of walls designed using M-O earth pressures using PGA from a seismic event with 2% probability of exceedance in 50 years (NBCC 2005) and subjected to a suite of earthquakes representative of an event with 2% probability in 50 years (NBCC 2005), and 3) to assess the performance of walls designed using M-O earth pressures using PGA from a seismic event with 2% probability of exceedance in 50 years (NBCC 2005) but with the dynamic portion reduced by a factor of $R_0$=1.3 to account for overstrength in the wall and subjected to a suite of earthquakes representative of an event with 2% probability in 50 years (NBCC 2005). These three basement walls were designed by the structural engineer according to the current state of practice based on earth pressures derived using a simplified M-O based method. Those walls were then modeled in FLAC and subjected to seven ground motions representing the hazard level prescribed by NBCC 2005. The ground motions were matched to Vancouver Uniform Hazard Spectrum in the period range 0.05–1.5 sec.

The analyses show that the computed peak force on the wall is very close to the M-O maximum force with PGA=0.46g. The point of application of the resultant force on the wall is consistently at about 0.5H from the base of the wall (Ahmadnia et al. 2011). This is the location usually assumed in British Columbia for application of M-O total (static and dynamic) resultant force. However, the pressure distributions from the dynamic analyses were radically different from the linear distribution typically assumed in the practice. At the location of floor slabs, a high concentration of lateral pressure is observed in wall W1 but is not very evident in walls W2 and W3. It should be noted that wall W1 was only designed to resist the forces from 10% in 50 year and is more flexible than walls W2 and W3. The lateral earth pressure is reduced between floor slabs. Results of analysis showed that wall W1 yields at mid elevation and also yields at all floor levels. The drift envelope for wall (W1) shows that the mean drift ratio approached 3% at the first level of the basement wall. According to task committee on Blast Resistance Design (TCBRG 1997), the hinge rotation of 3% corresponds to "medium response" meaning building cannot be used until repaired with significant total costs. The hinge rotations on other levels correspond to "low response" meaning the moderate repair cost. The hinge rotation for the wall W2 at all levels is less than "low response" limit meaning an acceptable performance. Comparing the result of analyses for wall W2 and W3 in Figures 6(b,c,e,f) shows that these two walls have approximately the same seismic performances in terms of displacement and drift ratio. This can be explained by comparing the mean envelopes of moments in Figures 4(b,c). As these two plots suggest, walls W2 and W3 yield in the in the mid-height of the bottom story and remain elastic in the mid-heights of the middle stories. In the top story, wall W2 remains elastic and wall W3 “barely” reaches the yield strength (in contrast Figure 4(a) shows that wall W1 yields significantly at the same location).

In summary the analyses showed that, even though the peak earth pressures from the dynamic numerical analysis were similar to those calculated using the M-O approach with PGA, basement walls could be designed with a reduced pressure and still have acceptable performance. This is because during the earthquake the weaker basement wall segments between floors shed load to the stiffer and stronger segments located where walls were buttressed by adjacent floors. This suggests that the current practice of applying the M-O approach with the 2% in 50 years hazard PGA to the weaker wall segments between floors may be overly conservative.

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