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Seismic Performance of Deep Basement Walls

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ABSTRACT

This paper reports on continuation of an ongoing research at the University of British Columbia (UBC) on seismic design of basement walls. The current state of practice for the seismic design of basement walls in British Columbia is based on the Mononobe-Okabe (M-O) method, using a Peak Ground Acceleration (PGA) mandated by the National Building Code of Canada (NBCC). Despite the absence of compelling damage or failure due to seismic earth pressures in the past earthquakes, the Structural Engineers Association of British Columbia initiated a task force to review the current seismic design procedures for basement walls. For this purpose a series of dynamic nonlinear soil-structure interaction analyses were recently conducted to evaluate the seismic performance of typical 4-level basement walls designed based on the state of practice in Vancouver. It was shown that under the code mandated demand the wall designed for 50-60% PGA results in a satisfactory performance in terms of drift ratio. This paper presents the recent extensions of the study to deeper basement walls founded on a different soil profile. Also a more representative nonlinear hysteretic model is used to characterize the hysteretic stress-strain response of soil. The results provide further evidence for evaluating the recommended fraction of code mandated PGA that may be used with the M-O method for acceptable seismic performance of basement walls.

Introduction

The seismic response of basement walls is a complex soil-structure interaction problem that depends on many different factors such as the nature of the input motion, dynamic response of the backfill soil, and flexural response of the wall. The current state of practice for seismic design of basement walls in the United States (Lew et al. 2010; Lew 2012) as well as in British Columbia (DeVall et al. 2010) is generally based on the studies of Okabe (1924) and Mononobe and Matsuo (1929) and their interpretations by Seed and Whitman (1970), which is generally known as the Mononobe–Okabe (M–O) method. In this limit-equilibrium force method, the earthquake thrust acting on the wall is a function of the Peak Ground Acceleration (PGA).

The seismic hazard level in the 1995 edition of the National Building Code of Canada (NBCC, 1995) had a probability of exceedance of 10% in 50 years, with the corresponding PGA of 0.24g for Vancouver. The more recent editions of the NBCC (2005, 2010) mandate a considerably

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different seismic hazard level with probability of exceedance of 2% in 50 years, which leads to almost doubling the PGA (0.46g). The current design PGA leads to very large seismic forces that make the resulting basement walls more expensive. Despite the absence of compelling damage or failure due to seismic earth pressures in the past earthquakes (Lew et al., 2010), the Structural Engineers Association of British Columbia (SEABC) initiated a task force to review current seismic design procedures for deep basement walls in Vancouver.

The main objective of this paper is to provide further evidence for evaluating the recommended fraction of code mandated PGA that may be used with the M-O method for acceptable seismic performance of basement walls (Taiebat et al., 2013, 2014). To this end two basement walls with different heights (4-level and 6-level basement walls) have been designed by members of SEABC using the current state of practice for four different fractions of the code PGA for Vancouver. Also a more representative nonlinear hysteretic soil model compared to the one used in earlier studies of the authors is used to simulate the hysteretic stress-strain response of soil. The performance of these basement walls in the term of drift ratio have been numerically studied under fourteen seismic events matched to the Uniform Hazard Spectrum (UHS) enforced by the NBCC (2010) for Vancouver. Results of the study are presented and discussed in this paper.

Seismic Design of the Basement Walls

Seismic design of basement walls is commonly based on the active thrust calculated from the M-O method. The M-O method is a limit-equilibrium force approach, developed by modification of Coulomb's theory for active (or passive) pressures. The M-O method provides only the total lateral force during earthquake and it does not explicitly indicate anything about the distribution of lateral earth pressure from seismic events. For practical purposes, Seed and Whitman (1970) proposed to separate the total (static and dynamic) active lateral force, P_{AE}, into two components, the initial static component, P_A, and the dynamic increment due to the base motion, ΔP_{AE}, where $P_{AE} = P_A + \Delta P_{AE}$. The static thrust, calculated from the Coulomb theory, is to be applied at $H/3$ from the base of the wall, resulting in a triangular distribution of pressure. As Seed and Whitman (1970) stated, most of the investigators agree that the increase in lateral pressure due to the shaking is greater near the top of the wall and the resultant increment in force acts at a height varying from $H/2$ to $2H/3$ above the base of the wall.

In this study 4-level and 6-level basement walls with total heights of 11.7m and 17.1m, are investigated as shown in Figs. 1 and 2. To study the effect of the pseudo-static horizontal acceleration used in the M-O method, each basement wall was designed by the structural engineers for four different fractions of the current code PGA (0.46g). Following NBCC (2010), structural engineers used two load combinations as (1) $1.5P_A$ and (2) $P_{AE} = P_A + \Delta P_{AE}$ to design both 4-level and 6-level basement walls for different fractions of code PGA. Details about the calculation of moment capacities at different elevations along the height of the walls are discussed and outlined in Taiebat et al. (2014). In Consistent with the four scenarios of lateral earth pressure, corresponding to four different fractions of PGA for each wall presented in Figs. 1 and 2, four levels of yielding moment are calculated and presented in Fig. 3.

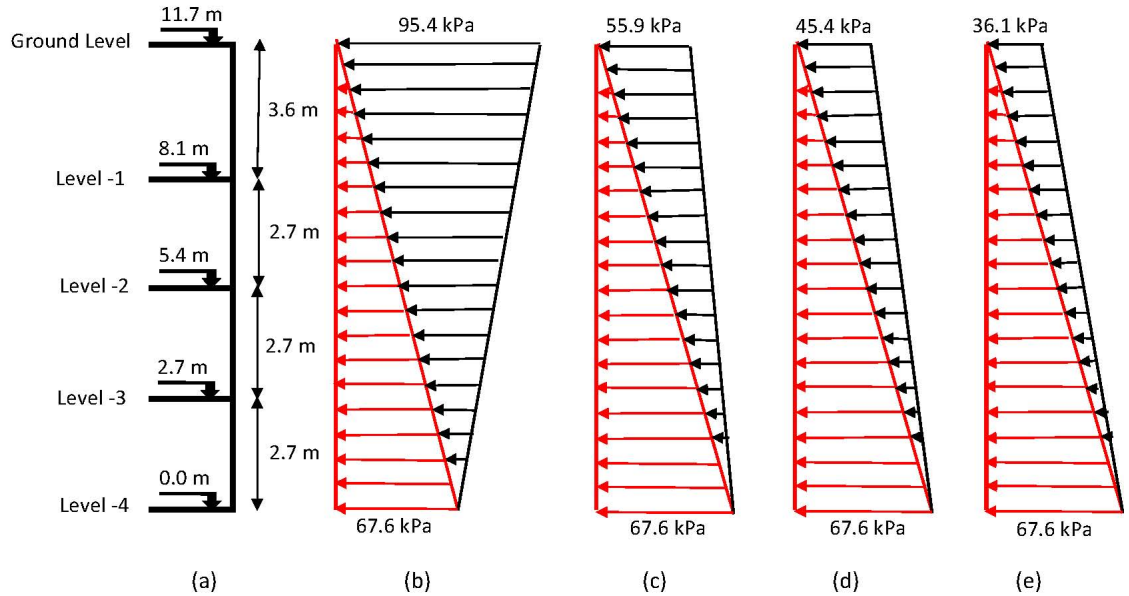

Figure 1: (a) Floor heights in the 4-level basement wall, and the design lateral earth pressure distributions using the M-O method with (b) 100% PGA, (c) 70% PGA, (d) 60% PGA, and (e) 50% PGA, where PGA=0.46g based on the NBCC (2010) for Vancouver.

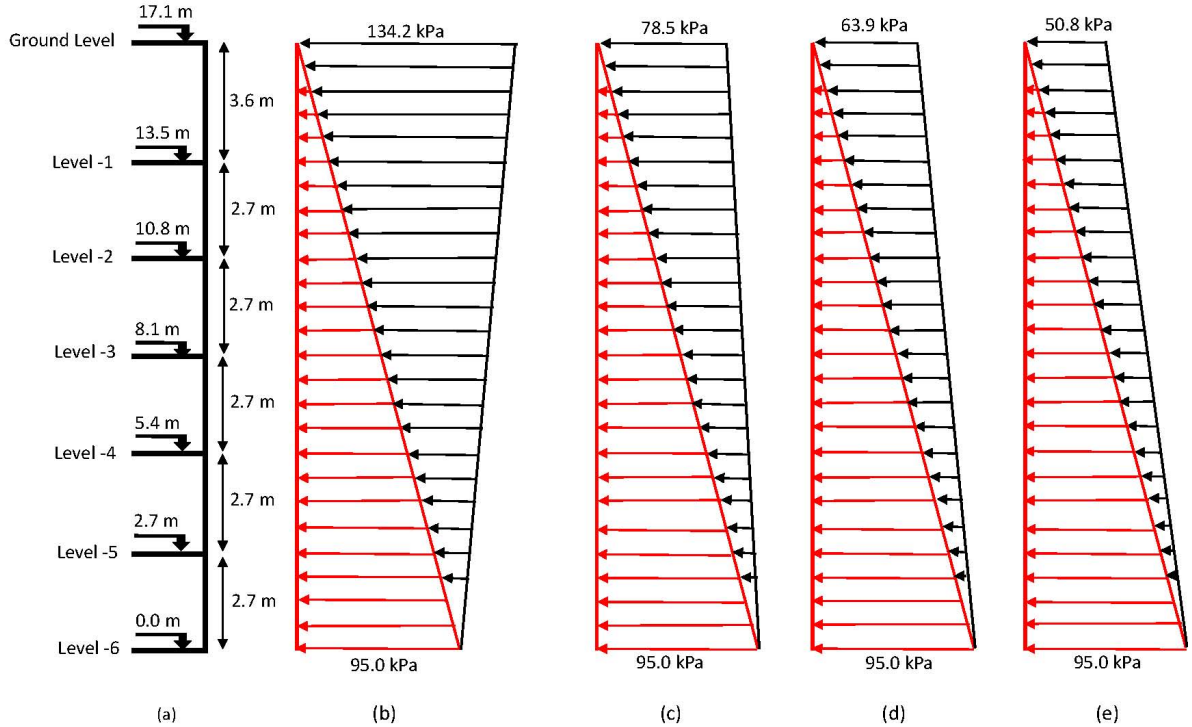

Figure 2: (a) Floor heights in the 6-level basement wall, and the design lateral earth pressure distributions using the M-O method with (b) 100% PGA, (c) 70% PGA, (d) 60% PGA, and (e) 50% PGA, where PGA=0.46g based on the NBCC (2010) for Vancouver.

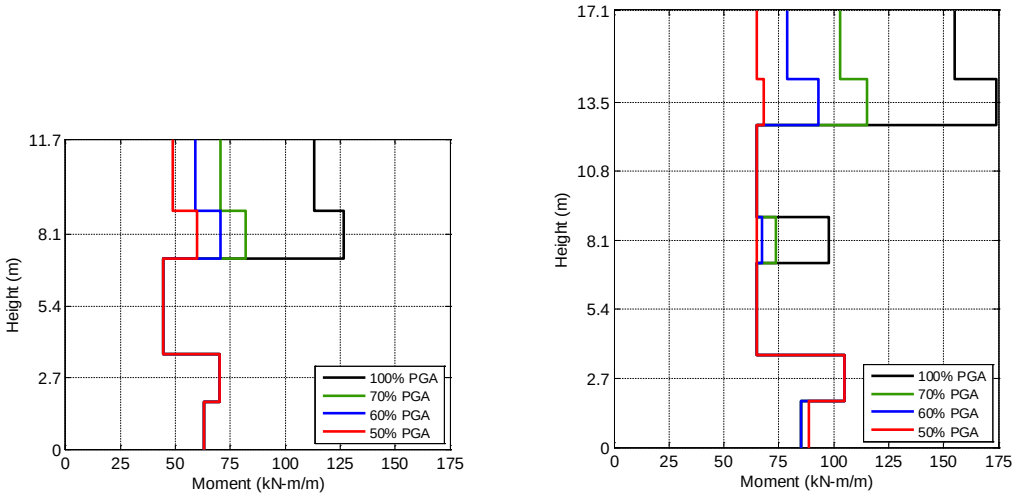

Figure 3: Moment capacity distribution along the height of the 4-level and 6-level basement walls designed for four different fractions of code PGA (0.46g).

Numerical Model Building

A series of nonlinear two-dimensional finite difference analyses using FLAC 2D have been conducted to model the seismic behavior of the 4-level and 6-level basement walls designed for various fractions of the NBCC (2010) PGA for Vancouver. The description of the boundary condition, construction simulation, structural and interface elements can be found in the companion paper (Taiebat et al. 2014). The 2D models of the 4-level and 6-level basement walls are presented in Fig. 4.

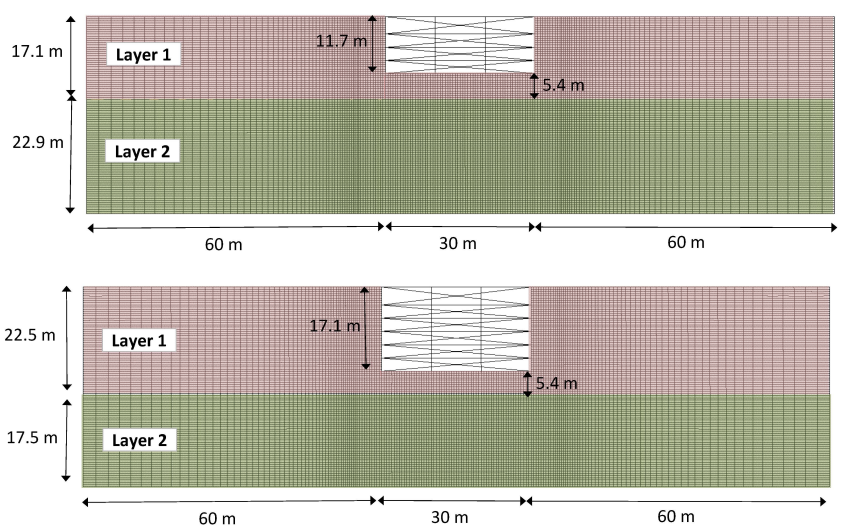

Figure 4: 4-level and 6-level basement wall models in FLAC 2D.

Constitutive model and calibration of soil parameters

In consultation with geotechnical engineers, the soil properties listed in Table 1 are suggested for the two layers of soil in Fig. 4. In this table V_{s1} is a normalized shear wave velocity based on the suggestion of Robertson et al. (1992), which is a function of the effective overburden stress.

Table 1: Soil layer material properties.

Soil layer	Density (kg/m^3)	V_{s1} (m/s)	G_{max} (MPa)	Mohr-Coulomb				UBCHYST				
				υ	Coh. (kPa)	ϕ (°)	ψ (°)	hrm	hdfac	hrf	hn1	hn
1	1950	200	17-143	0.28	0	33	0	0.5	0	0.98	1.0	3.3
2	1950	400	580-885	0.28	20	40	0	0.5	0	0.85	1.5	2.0

Two layers of soil are modeled with UBCHYST soil model, which is a two dimensional nonlinear hysteretic model developed at the University of British Columbia for dynamic analysis as a FISH source (Naesgaard 2011). In order to improve the efficiency of the code, the model was converted to C^{++} and compiled as a DLL file (Mikola and Sitar, 2012), as used in this study. In UBCHYST the tangent shear modulus (G_t) is a function of the peak shear modulus (G_{max}) times a reduction factors which are a function of the developed stress ratio which varies throughout the loading cycle to generate nonlinear hysteretic stress-strain loops. In this model the magnitude of the stress ratio is limited by a Mohr-Coulomb failure envelope. The UBCHYST model parameters are calibrated by comparing the modulus degradation and damping curves resulting from simulations with the model to those published by Darendeli (2001). To this end, for each layer of soil presented in Table 1, an initial estimate of values was made based on a series of sensitivity analysis conducted on each parameter. Then an element cyclic simple shear (CSS) test using UBCHYST constitutive model was conducted in FLAC at different depth of the model for fifteen shear strain amplitudes, ranging from 0.0001% to 1%, to generate modulus and damping curves. The UBCHYST parameters were adjusted in a way to result the best match to the Darendeli modulus reduction and damping curves. Fig. 5 illustrates the modulus reduction and damping of the first soil layer at different confining pressures compared to the laboratory results of Darendeli. As it is shown in this figure the model overestimates the damping response at medium to large shear strains (> 0.1%). Mikola and Sitar (2012) had drawn the same conclusion and related it to the width of the hysteresis loop in the UBCHYST model.

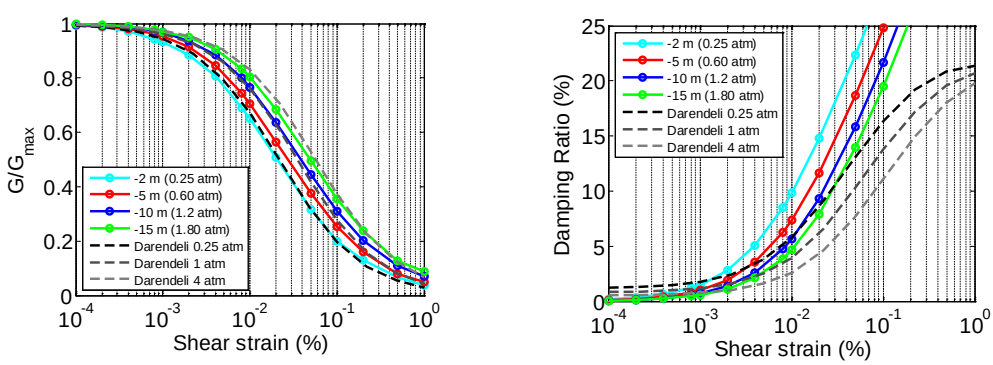

Figure 5: Modulus reduction and damping curves estimated by FLAC at different depth of the first layer of soil using UBCHYST model.

Ground motion selection and scaling

Based on the results of de-aggregation of the Uniform Hazard Spectrum of Vancouver, searching criteria was set as the magnitude range of 6.5 to 7.5, with the closest distance of 10-30 km of the causative fault plane from the earthquake sites. Selection of the candidate ground motions was done based on the best linearly matched motions to the UHS of Vancouver for firm ground (soil class C, V_s =360-760 m/s) proposed by the NBCC (2010), in the period range of 0.02-1.7 sec.

Table 2: List of selected ground motions.

Event	**Year**	**Station**	**Mag.**	**Mechanism**	**V_{s30} (m/s)**
Friuli- Italy-01	1976	Tolmezzo	6.5	Reverse	424.8
Tabas- Iran	1978	Dayhook	7.35	Reverse	659.6
New Zealand-02	1987	Matahina Dam	6.6	Normal	424.8
Loma Prieta	1989	Coyote Lake Dam (SW Abut)	6.93	Reverse-Oblique	597.1
Loma Prieta	1989	San Jose - Santa Teresa Hills	6.93	Reverse-Oblique	671.8
Northridge-01	1994	LA - UCLA Grounds	6.69	Reverse	398.4
Hector Mine	1999	Hector	7.13	Strike-Slip	684.9

Also in order to eliminate the potential bias towards one specific event, no more than two ground motions were selected from the same seismic event. Table 2 shows the list of the selected seven ground motions. It should be mentioned that both Fault-Normal and Fault-Parallel components of each motion were used in this study, resulting in a total of fourteen ground motions. the selected motions were spectrally matched to the NBCC (2010) UHS of Vancouver in the period range of 0.02-1.7 sec, and then using the computer program SeismoSignal (Seismosoft, 2009) they were baseline corrected with a linear function and filtered with a band pass Butterworth filter with cut-off frequencies of 0.1 Hz and 25 Hz.

Simulation Results

The maximum resultant drift ratio along the height of the wall is a parameter which is evaluated in this section. For this purpose, the recommendation of ASCE task committee on design of blast-resistant buildings in petrochemical facilities (ASCE-TCBRD, 2010) is used as the performance criteria. To the best knowledge of the authors, there is no other report on the acceptable drift ratios for constrained walls with distributed lateral loading. In this recommendation the drift ratio of a basement wall at the middle of each storey is calculated as the difference between the displacement of the wall at that level and the average displacements of the wall at the top and bottom of the storey divided by half of the storey height. The ASCE specified a low response category as “Localized component damage with moderate cost of repairs”. The response limits associated with this category for the reinforced concrete wall panels (with no shear reinforcement) is 1.7% drift ratio.

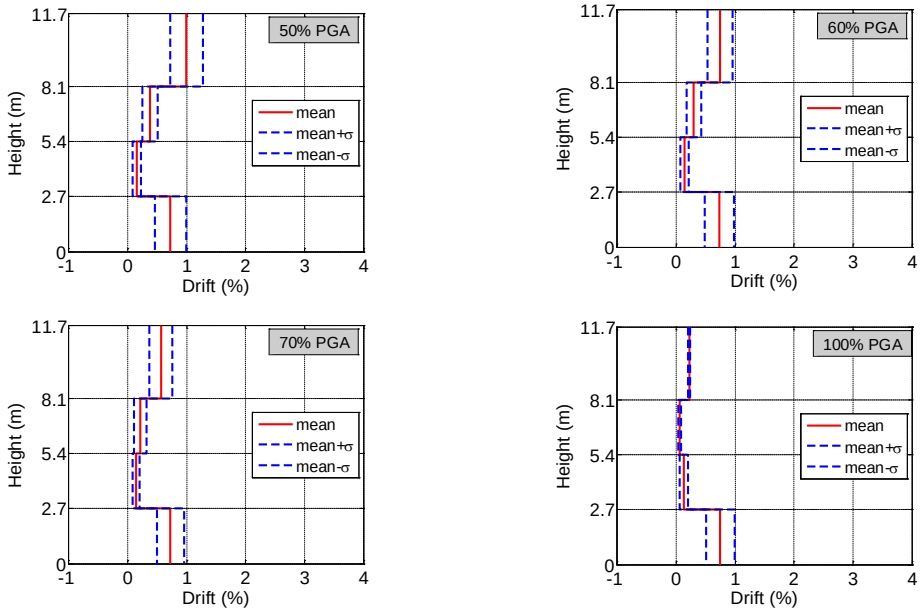

Figure 7: Average of maximum drift ratios along the height of the 4-level basement wall, designed for four different fractions of the code PGA subjected to fourteen ground motions.

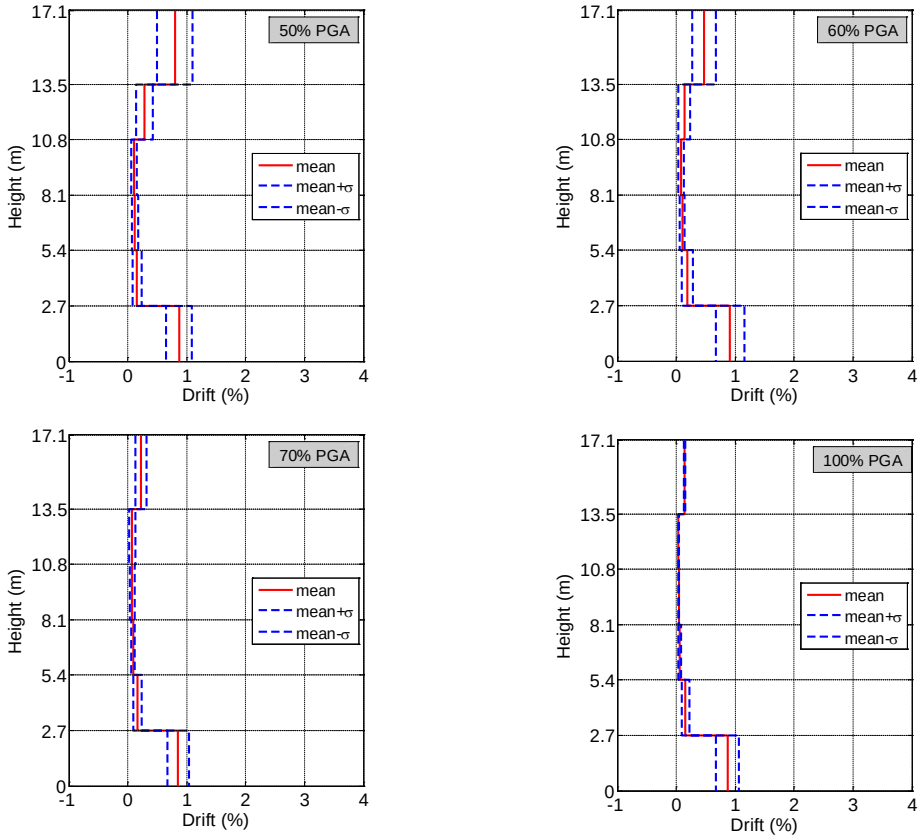

Figure 8: Average of maximum drift ratios along the height of the 6-level basement wall, designed for four different fractions of the code PGA subjected to fourteen ground motions.

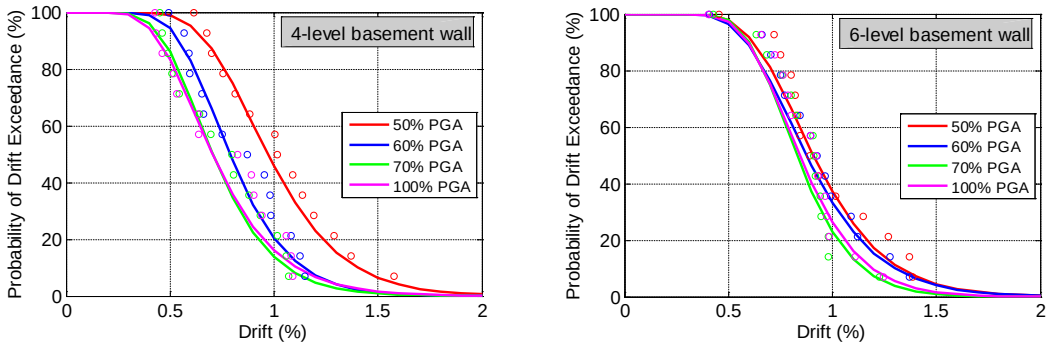

Figure 9: Exceedance probability of drift ratio for 4-level and 6-level basement walls designed for different fractions of code PGA.

Figs. 7 and 8 show the maximum drift ratio along the height of the 4-level and 6-level basement walls, each designed for four different fractions of code PGA, and subjected to fourteen seismic events. In these plots the mean value corresponds to the average of the maximum drift ratio from fourteen seismic events. Assuming normally distributed drift ratios, mean ± standard deviation (σ) represents the first standard deviation with 68% chance that the mean falls within the range of standard error. According to the adopted performance criterion for drift ratio, in the present problem the response of the top and bottom levels of the basement walls need careful consideration. The results of both 4-level and 6-level basement walls suggest that the performance of the walls designed for even 50-60% of the code PGA for Vancouver seem adequate. The exceedance probability of drift ratios from a certain value for both 4-level and 6-level basement walls, each designed for different fractions of code PGA is presented in Fig. 9.

Conclusions

In the present study the seismic performance of the typical 4-level and 6-level basement walls, subjected to the seismic demand in Vancouver were examined. Both walls were designed according to the state of practice in Vancouver for different fractions of NBCC (2010) PGA. The nonlinear seismic performance of these walls suggest that the behavior of the top and bottom basement levels are critical. Based on the proposed acceptance criteria for drift ratio by ASCE-TCBRD (2010) one can conclude that designing the basement walls for full PGA is over conservative. This is while the behavior of both 4-level and 6-level basement walls designed for 50%–60% code PGA result in satisfactory performances when subjected to the current seismic hazard level in Vancouver with a 2% chance of being exceeded in 50 years.

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