Effect of Aftershock Ground Motions on the Seismic Response of Low-rise Buildings

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SUMMARY:
Over an extended lifespan, some buildings can be subjected to strong ground shaking from several earthquake events. Structures with damage caused by a strong mainshock could experience more severe damage when subjected to several aftershocks. Prediction of this aftershock damage is challenging. There is documented evidence of buildings during the last 40 years in California, Chile, Central America, Europe and Japan where aftershocks have induced further damage and in some cases collapse of buildings. This paper studies the effect of aftershock ground motions on the retrofit design of low-rise school buildings in British Columbia, on West Coast of Canada. The school building models are subjected to suites of ground motions of varying intensity in performing back to back non-linear dynamic analysis. Several ground motions are included in each sequence, which are selected from events of similar characteristics to the ones expected in British Columbia. The results of this study provide insight on the potential for earthquake damage that is attributed to ground motion sequences on buildings. The study highlights differences in seismic response of buildings based on shaking intensity and the seismic resistance system of the buildings.

Keywords: Ground Motion Sequences, Aftershocks, Seismic Response Low-rise Buildings

1. INTRODUCTION

In May 2004 The Ministry of Education of British Columbia designated a budget of $1.5 billion towards the seismic mitigation program of public K-12 school buildings. Many studies to develop guidelines for the innovative and cost effective seismic retrofit program are being developed through collaboration of Association of Professional Engineers and Geoscientists of British Columbia (APEGBC) and the Department of Civil Engineering at the University of British Columbia (UBC).

The Province of British Columbia in Canada is located in a seismic prone region, with multiple earthquake sources, which are subduction, subcrustal and crustal (Atkinson, 2005). This poses the threat that several earthquake events could originate from the different sources throughout the lifespan of building structures. Under the seismic mitigation program for school buildings studies are being conducted to better understand the ability that low-rise schools buildings would have to survive seismic demands from ground motion sequences and develop post-earthquake evaluation guidelines.

In this paper a deterministic assessment of the seismic response of a low-rise school building structural system to crustal earthquake ground motion sequences is performed. The selected reinforced concrete prototype is typically used in many school buildings across British Columbia.

2. METHODOLOGY

The methodology implemented for this study includes the following steps.

2.1 Definition of mathematical model of structure: A stick model for a low-rise reinforced concrete building is developed and used throughout the parametric study.

2.2 Selection of seed ground motions: A suite of ground motion sequences recorded during past earthquakes is selected. The selection is performed at several stations to capture interevent intensity variability.

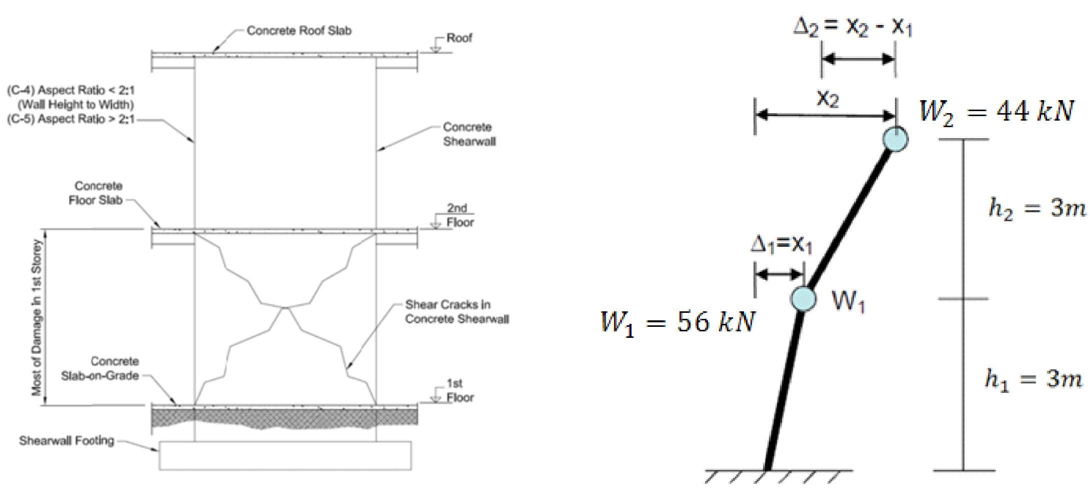
2.3 Scaling of seed ground motions: Each sequence of ground motions is uniformly scaled to different intensity levels.

2.4 Back to back nonlinear response history analysis: Back to back nonlinear response history analyses (NHRA) are conducted using the mathematical model and the suite of ground motion sequences previously defined. This approach enables to inherit the properties of the mathematical model across the shaking sequence.

2.5 Iterative analysis: An iterative approach is implemented by running numerous back to back NRHA and applying a marginal increment or decrement of the wall model lateral force resistance. This is done until the minimum resistance required to undergo the sequence with a maximum interstory drift ratio of 6% is found. The same approach is implemented for all the different intensities of ground shaking considered.

3. CASE STUDY DESCRIPTION

A two storey squat reinforced concrete shear wall was used as the physical model of the case study and is shown in Figure 1a. Alongside in Figure 1b is the corresponding stick model representation. The behaviour of the squat wall is dominated by shear deformation because of the low aspect ratio (less than 2). Failure modes of this prototype are diagonal tension, diagonal compression followed by flexure. Sliding base shear can occur in some cases.



(a) (b)

Figure 1. Squat RC shear wall prototype and stick model representation.

The cyclic envelope adopted for the squat wall NRHA is a backbone curve comprised by four linear segments as shown in Figure 2. The wall yields at a low interstorey drift ratio (0.1%), yielding is followed by a plateau that extends until 1% interstorey drift ratio. The next segment has a negative slope branch accounting for stiffness and strength degradation. The residual strength is 40% of the yield resistance (SRG1, 2011). Cyclic tests conducted at the University of Ottawa, Canada confirm the drop in resistance at 1% drift ratio and then followed by a stable response (Saatcioglu, 1992).

The nonlinear response history analysis (NRHA) were performed using program CANNY. The lateral force resistance was modelled through a quadrilinear skeleton curve CA4. The hysteretic response of the model considered stiffness and strength degradation. The P-Delta effects were included in mathematical model as well.

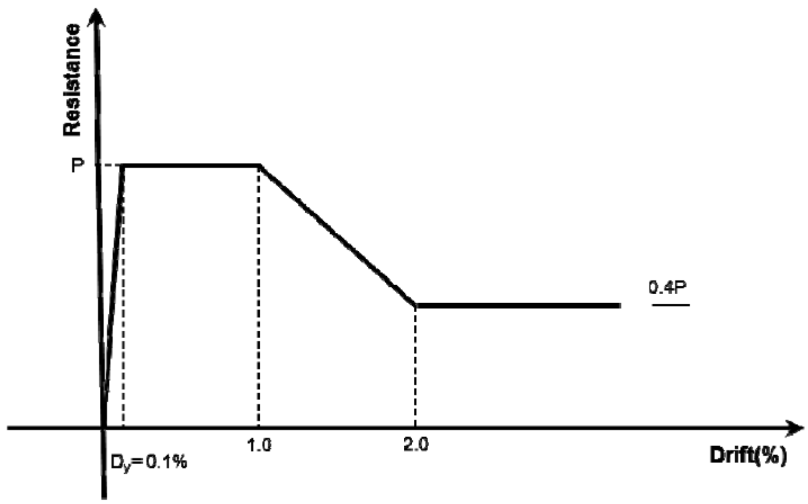


Figure 2. Squat RC shear wall backbone curve.

The seismic input was defined using recorded ground motion sequences from past earthquakes. Seed ground motion sequences recorded at different stations of the GeoNet strong motion network (GNS, 2011) in New Zealand were selected for this study. The ground motions were recorded during recent crustal earthquake events in Christchurch and Canterbury for the period of September 2010 to December 2011. In total each sequence includes four strong ground motion records which were recorded at epicentral distances that range from 9 to 58 km and are listed in Table 3.1. Events are identified from 1 to 4 when the table is read from left to right. The east components of ground motion are being used in this paper.

The predominant soil conditions in the sites of the selected strong motion stations are soft soils classified as Geomatrix "D" type, the surficial geology being flat alluvial plain deposits (floodplain of Waimakariri River).

Table 3.1. Summary of ground motion records selected from GeoNet

Station	Epicentral Dis. (km)	Epicentral Dis. (km)	Epicentral Dis. (km)	Epicentral Dis. (km)
DFHS	9	49	52	58
CMHS	36	6	10	18
CBGS	36	9	11	17
Date ->	Sep 3 - 2010	Feb 21 - 2011	June 13 - 2011	Dec 23 – 2011
Magnitude ->	Mw 7.1	Ml 6.3	Mw 6.0	Mw 5.8

The earthquake of September 3^{rd} 2010 originated along an unknown fault, which has been named Greendale fault. The epicentre of this event was located to the west of the three stations listed above. Earthquake events of 2011 originated eastward to Greendale fault. The response spectra in Figures 3, 7 and 9 show that shaking sequences are strongly influenced by the locations of the different epicentres.

4. RESULTS AND DISCUSSION

The response spectra of the DFHS sequence in Table 3.1 is shown in Figure 3. This sequence can be seen to have a very strong ground motion for the first event in September 3rd 2010, and the subsequent three ground motions to have an average intensity of 8% of the first event.

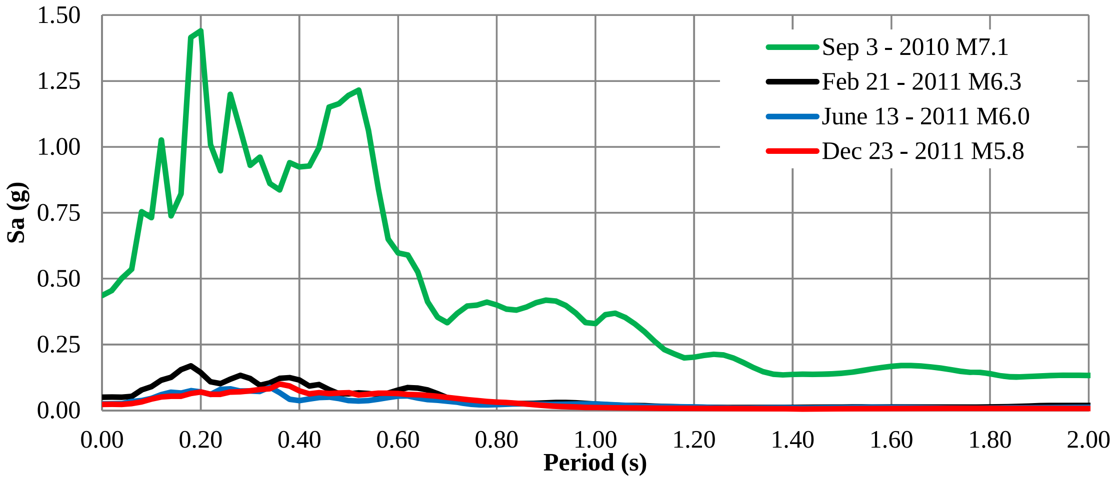


Figure 3. Response spectra at Darfield High School (DFHS) station (5% damping ratio).

The ground shaking sequence of Figure 3 is uniformly scaled to have seven different shaking intensity scenarios as indicated in the colour scale of Figure 6. These seven sequences are used to determine the minimum required lateral resistances for the RC wall model to have a response that remains below the target drift ratio throughout each sequence. Figure 4 and 5 overlay on the backbone curve the sequence of first storey peak interstorey drift ratios for two models at different shaking intensities.

Figure 4 shows that at 50% of shaking intensity a model with a resistance of 4.0%W is required. In addition under stronger shaking this model cannot reach the onset point of stiffness and strength degradation (1%). This is compared with the response in Figure 5 where the wall model (14.66%W) is able to undergo stronger shaking and larger interstorey drift ratios without experiencing instabilities. The model resistance in Figure 4 is 68% of the residual resistance of the model shown in Figure 5.

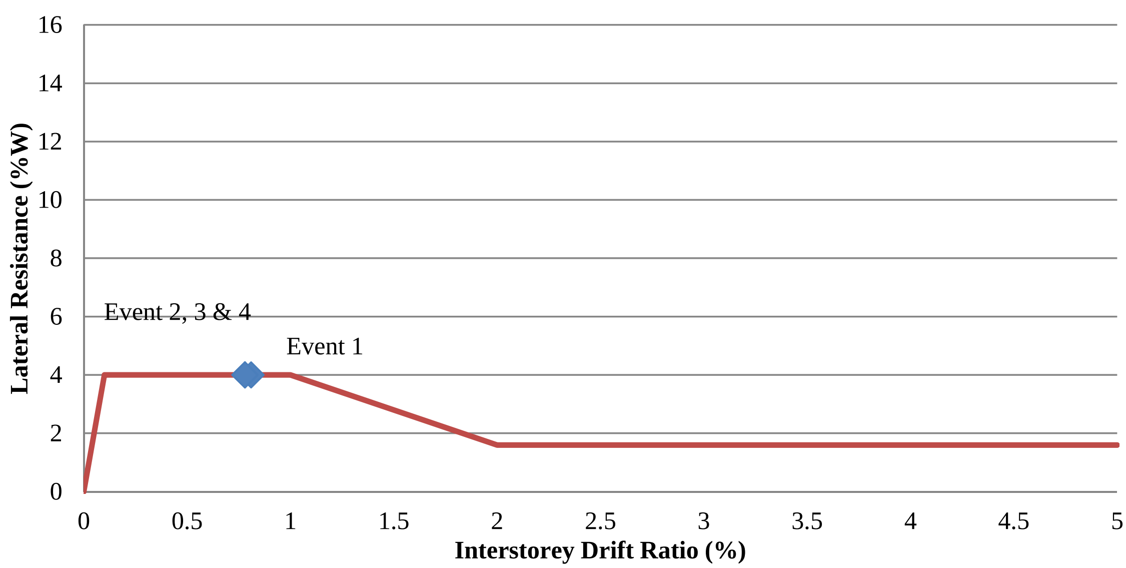


Figure 4. Sequence of first storey peak interstorey drift ratio on backbone curve for P= 4.0%W.

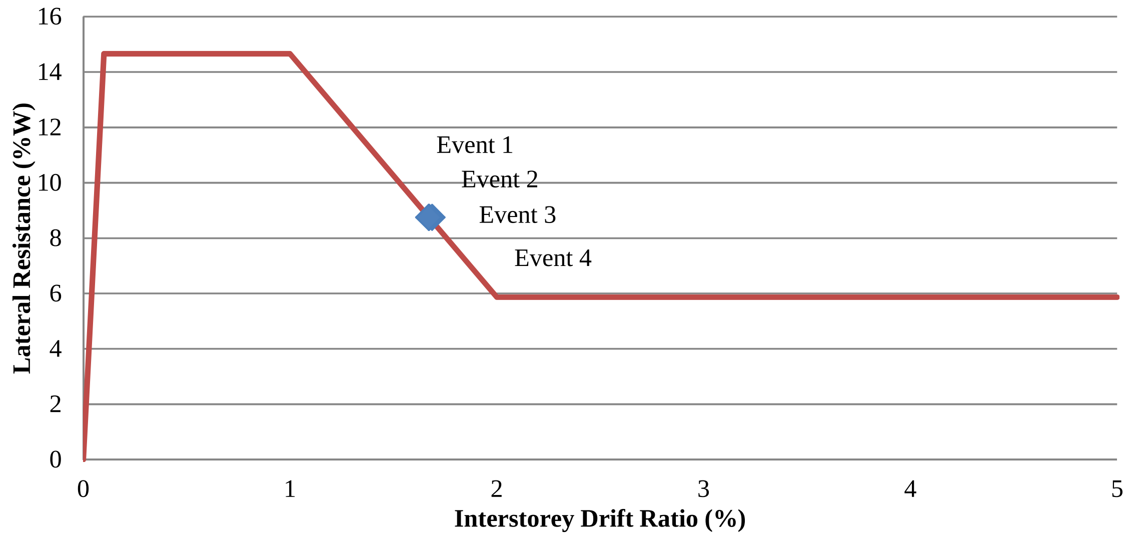


Figure 5. Sequence of first storey peak interstorey drift ratio on backbone curve for P= 14.66%W.

Figure 6 summarizes the responses for different shaking intensity scenarios. For a given intensity and resistance the peak first storey interstorey drift ratio at each one of the four events is shown. In the lower end of the shaking intensity it can be observed that weak walls with resistance less than 8%W the maximum interstorey drift is not much greater than 1%. For these walls any stronger shaking escalates into very large deformations were P-Delta effects are dominant.

As the resistance is built across the different wall models their ability to sustain stronger shaking and to undergo larger interstorey drifts in excess of 1% increases. Since the remaining events in the sequence are weak they do not contribute significant demands.

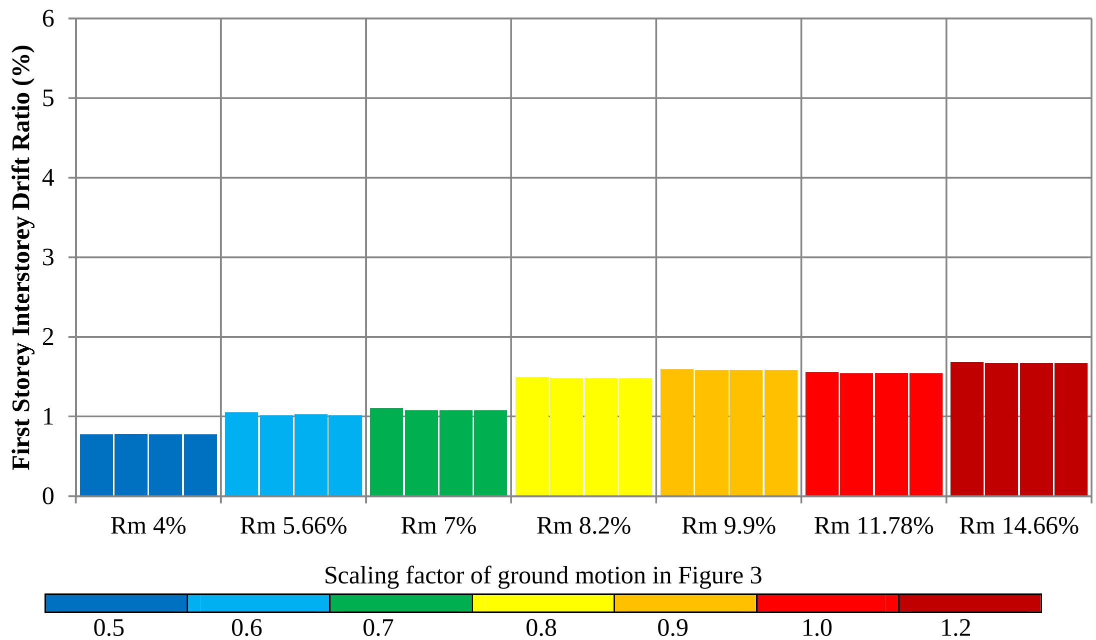


Figure 6. Lateral resistance (%W) required to surviving shaking sequence vs. interstorey drift ratio.

The response spectra of CMHS sequence in Table 3.1 is shown in Figure 7. This sequence has the events of September 2010 and February 2011 with quite strong shaking. The latter is a broadband spectrum with three distinct peaks at 0.2 seconds, 0.48 seconds and 0.92 seconds. The spectral ordinates of the events of June and December on average are smaller than the previous events.

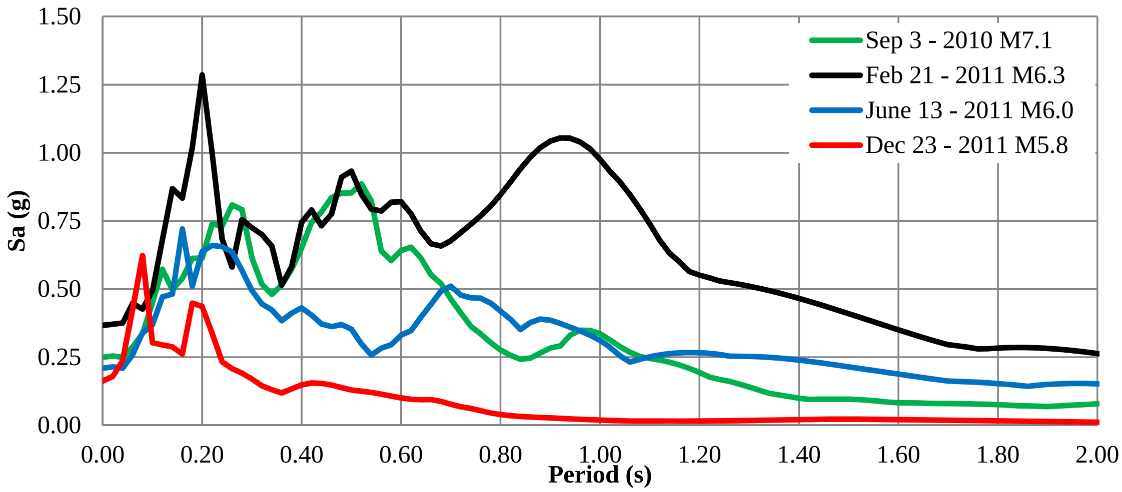


Figure 7. Response spectra at Cashmere High School (CMHS) station (5% damping ratio).

Figure 8 shows that during the first event the interstorey drift ratios are very low. However the second shaking induces an interstorey drift in excess of 1%, this beyond the onset point of strength and stiffness degradation in Figure 2. Thus the remaining ground motions drive the system up to interstorey drifts of 4.45%. It is seen that for the wall models having larger strength the marginal increment in the interstorey drifts due to the fourth ground motion is not as large as the increment due to the third shaking, this is expected due to its low intensity.

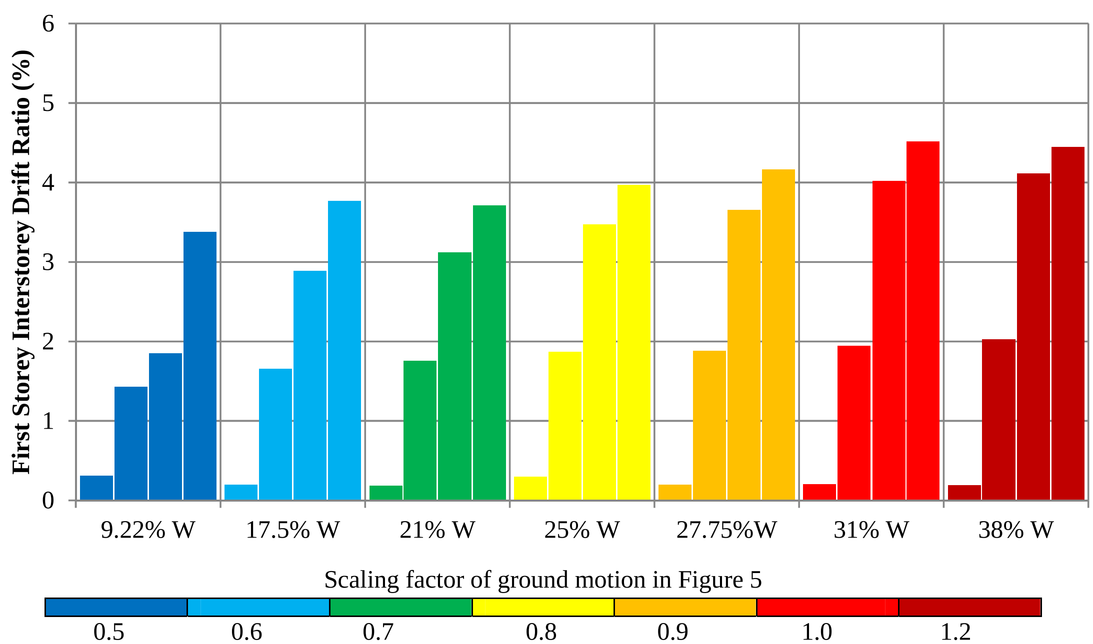


Figure 8. Lateral resistance (%W) required to surviving shaking sequence vs. interstorey drift ratio.

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The response spectra of CBGS sequence in Table 3.1 is shown in Figure 9. In this case the event recorded in February 2011 is stronger than the other events. This spectrum is also broadband. The other events are of comparable intensity.

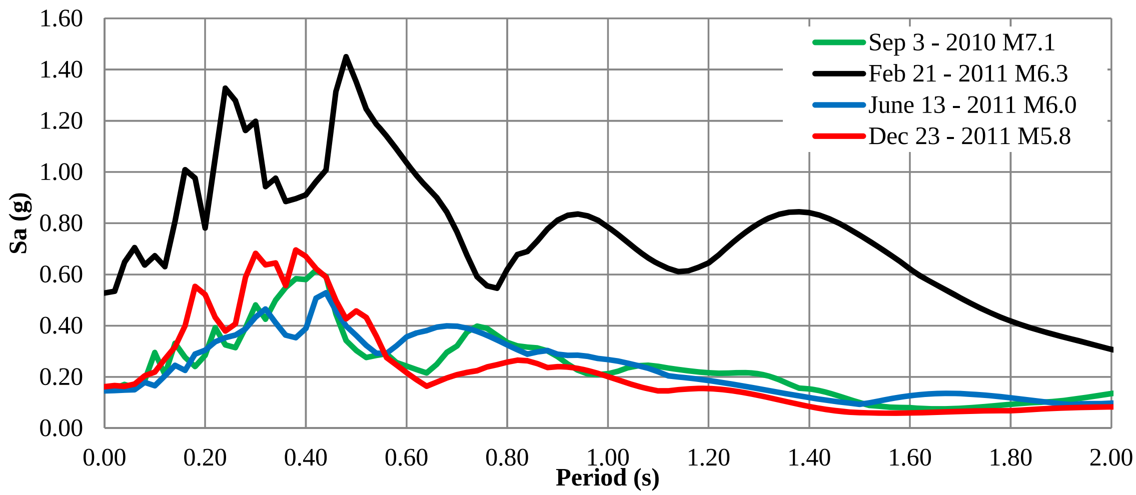

Figure 9. Response spectra at Christchurch Botanic Gardens (CBGS) station (5% damping ratio).

Same as previous case Figure 10 shows that during the first event the interstorey drift ratios are very low, furthermore wall models remain elastic. The response of the wall to the second event reach interstorey drift in excess of 2% in most cases, this is beyond the onset point of residual strength in Figure 2. The remaining ground motions produce interstorey drifts of up to 5.5%. The marginal increment of interstorey drift ratios due to ground motions 3 and 4 are comparable.

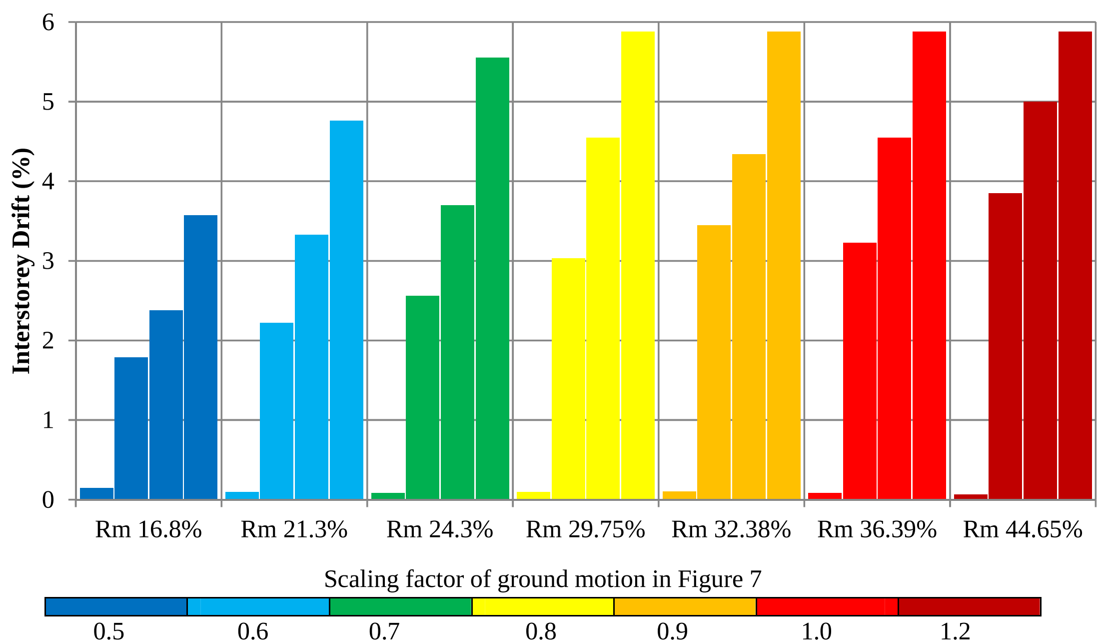

Figure 10. Lateral resistance (%W) required to surviving shaking sequence vs. interstorey drift ratio.

5. CONCLUSIONS

It is seen from response spectra in Figure 3, 7 and 9 that the ground motions from events 2, 3 and 4 at the CBGS and CMHS stations have stronger shaking than at DFHS station. A reason for this is the geographical clustering of the events of February, June and December which were closer to CBGS and CMHS stations. The fact that the east components of ground motion were used also contributes to stronger shaking for these events given that directionality of ground motion was predominant along east-west, whereas the event 1 had stronger shaking along north south direction.

Besides having stronger shaking at CBGS and CMHS, the ground motions generate quite different spectral shapes when compared to DFHS station. The events recorded at stations CBGS and CMHS have wider broadband frequency content when compared to the DFHS station. For the parametric study conducted herein the consequence of this difference in the ground motions is that higher lateral resistances are warranted. These ranged from 9% to 45% W for the CBGS and CMHS sequences, whereas for the DFHS they ranged from 4% to 14.66%W.

Higher lateral resistances allow the models to undergo larger interstorey drifts in a progressive manner throughout the shaking sequences as is expected. In Figure 6 two different behaviours are observed, weak wall models exhibit “brittle” behaviour being able to undergo moderate interstorey drift ratios whereas stronger wall models behave in a more ductile manner and are able to undergo larger interstorey drift ratios in a stable manner.

The results presented in this paper provide through deterministic scenarios estimates of the seismic performance of squat RC shear walls buildings when subject to ground shaking sequences. It is concluded that systems with low resistance (less than 8%) are quite vulnerable and even at small interstorey drift ratios could become unstable. Their ability to survive sequences of strong ground shaking is not reliable. In performing post-earthquake evaluation such structures should to be red tagged even if not apparent damage is observable. Further studies will be conducted for the development of post-earthquake evaluation guidelines of low-rise building schools in British Columbia.

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REFERENCES

Atkinson G. M. (2005). Ground Motions for Earthquakes in Southwestern British Columbia and Northwestern Washington: Crustal, In-Slab, and Offshore Events. *Bulletin of the Seismological Society of America* **95**: 1027-1044

GNS. (2011). Geological Nuclear Sciences. Available at: http://www.geonet.org.nz/resources/basic-data/strong-motion-data/

Saatcioglu, M. (1992). Hysteretic shear response of low rise walls. Concrete shear in earthquake, National Science Foundation, U.S.

SRG1, (2011). Seismic Retrofit Guidelines, first edition. Ministry of Education of British Columbia - Association of Professional Engineers and Geoscientists of British Columbia - The University of British Columbia, Canada.