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INVESTIGATION OF THE COLLAPSE RISK OF CONCRETE SHEARWALL BUILDINGS UNDER LONG DURATION GROUND MOTIONS

M. Fairhurst[1], A. Bebamzadeh[2], C. E. Ventura[3]

[1] *Graduate Research Assistant, Civil Engineering, UBC Vancouver, fairhurstmike@gmail.com*
[2] *Research Associate, Civil Engineering, UBC Vancouver, armin@civil.ubc.ca*
[3] *Professor, Civil Engineering, UBC Vancouver, ventura@civil.ubc.ca*

Abstract

Recent megathrust earthquakes in Tohoku, Japan (M_w 9.0, 2011), El Maule, Chile (M_w 8.8, 2010), and Sumatra, Indonesia (M_w 9.1, 2004) have served as reminders that large magnitude, long duration earthquakes are possible in subduction zones around the world. Reconnaissance teams have repeatedly reported large levels of damage from these events, due in part to the high number of load reversal cycles. Additionally, many experimental tests indicate that load duration and number of cycles are highly important to the observed damage. Most studies using numerical models of structures also agree that ground motion duration is of significant importance when considering cumulative damage measures. However, currently, no such provisions for shaking duration are required in most building codes. This may be in part due to the conflicting results of studies which consider the effect of ground motion duration on peak structural response (drift or displacement).

This paper aims to reduce the uncertainty in the effect of ground motion duration on the collapse risk of mid- and high-rise concrete shearwall buildings in South-western British Columbia. This area is affected by the Cascadia Subduction Zone, which is capable of producing magnitude 9.0+ earthquakes. A 20 storey archetype building model is considered analyzed with several suites of ground motions. The suites of ground motions are selected to represent long- and short-duration motions. Both NBCC code-level and analyses and collapse level analyses are performed. The results of this study will be useful in developing future design provisions for worldwide subduction tectonic zones.

Keywords: Megathrust earthquakes, reinforced concrete, ground motion duration

1. Introduction

Currently, the effect of ground motion duration on the damage or collapse probability of structures is not well established. This is in part due to different conclusions reached by researchers studying ground motion duration or number of cycles using observational evidence, experimental testing, and numerical modeling. Some studies [1, 2] have observed greater damage caused by the rapid energy release in short duration crustal events, while others suggest that the large number of cycles from subduction interface events cause greater damage [3].

Hancock and Bommer [4] provide a comprehensive state-of-knowledge review on the effect of ground motion duration on structural damage. In their review they note that most studies using cumulative damage or displacement measures find a correlation between ground motion duration and structural damage. However, studies that use extreme responses (such as maximum interstorey drift or displacement) generally do not find correlations between duration and damage. This, however, may be to a lack of cyclic degradation in the models employed in many of these studies. Additionally, most of the experimental tests reviewed showed a high degree of relevance of ground motion duration and number of cycles on specimen damage [4].

More recently, a study by Chandramohan et al. [5] found that the probability of structural collapse increases with increasing ground motion duration when records have equivalent response spectra. A similar study by Raghunandan and Leil [6] on the collapse risk of structures in the Cascadia Subduction Zone showed that longer duration events can cause collapse at lower intensity levels compared to shorter events. Both of these studies were done using reinforced concrete moment frame numerical models which included cyclic strength and stiffness degradation.

In this paper the effect of ground motion duration on concrete shearwall buildings is investigated. The prototype model employed is a 20 storey reinforced concrete shearwall building with coupled walls designed for Vancouver, BC, Canada using the 2010 National Building Code of Canada (NBCC). Cyclic and in-cycle degradation is accounted for in the coupling beams as well as the material models used in fiber sections of the shearwalls. Two suites of spectrally equivalent records are run at various levels of shaking, from code design level all the way to collapse level. The code level motions are ran to see of ground motion duration effects typical code design, while collapse level motions are ran to determine if the collapse risk of the structure is correlated with ground motion duration.

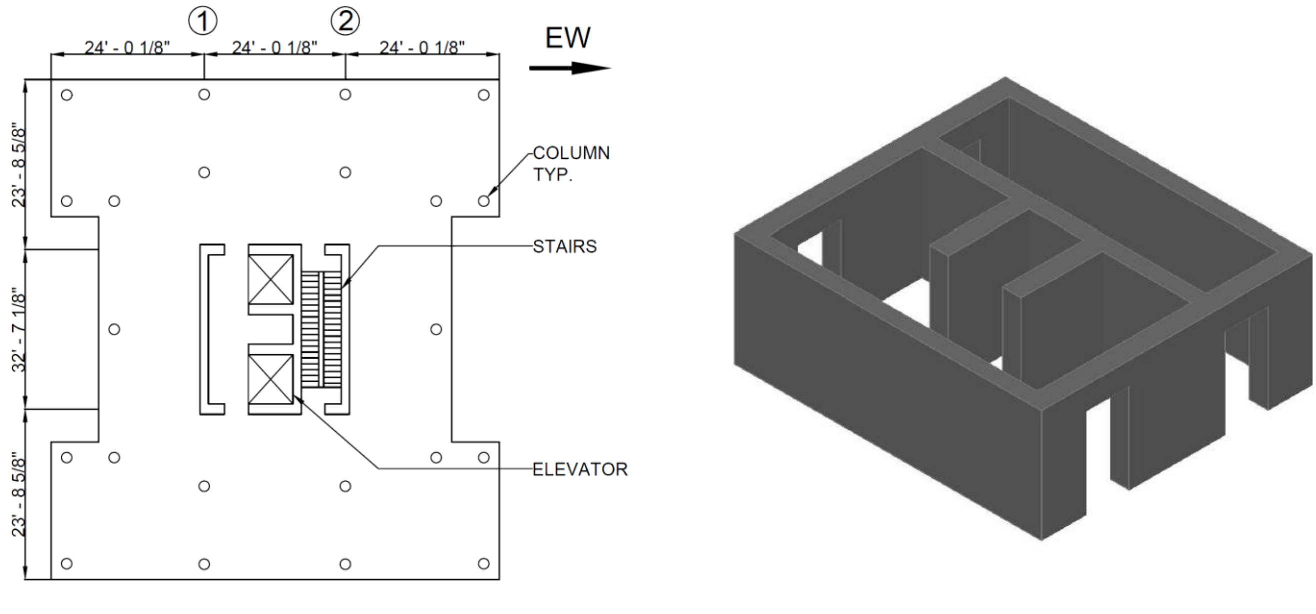
2. Prototype Building Model

The prototype building for this study is a typical 20 storey reinforced concrete shearwall building designed for Vancouver, BC, according to the 2010 National Building Code of Canada (NBCC). The lateral load resisting system comprises of three interior reinforced concrete shearwalls which comprise the elevator and stair core of the building. The gravity resisting system comprises of perimeter and exterior circular columns and 8” slabs at each level. The prototype building layout was based off of a 20 storey concrete building designed for another study [7]. The structural system and core are illustrated in Fig. 1.

The floor area is about 5200 ft^2 per storey and the weight is calculated as 0.21 kip/ft^2 (approximately 10 kN/m^2).

Reinforcement in the shearwalls beams is illustrated in Fig. 2. The reinforcement in the walls along lines 1 and 2 is the same. The walls are connected by 2’ deep header beams (Fig. 3) which are reinforced by transverse 15M stirrups spaced at 4”.

Based on the NBCC Vancouver 2010 spectrum, with an R_dR_o = 2.0*1.4, the design base shear is approximately 12% of the total weight of the structure. Only the softer EW direction was considered for analysis.



(a) **(b)**

Fig. 1 – Prototype building: (a) typical floor plan and (b) illustration of core

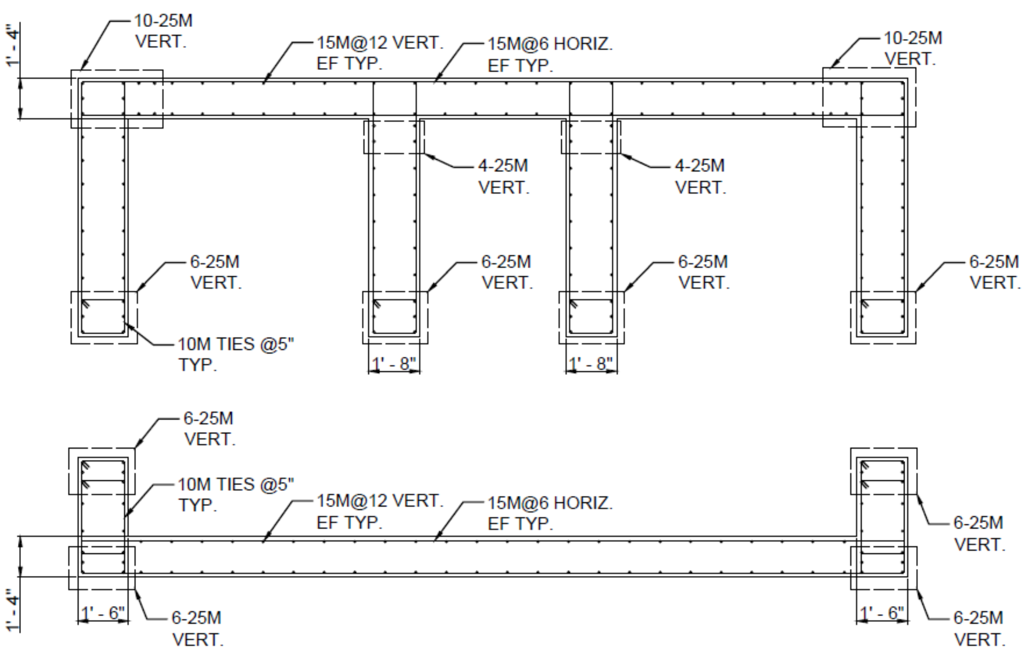


Fig. 2 – Reinforcement of the interior shearwalls

3.2 OpenSees Model

A numerical model of this building was developed in OpenSees [8]. The three interior shearwalls were modeled using fiber elements with a displacement-based formulation. The header beams were modeled with elastic beam elements with a cracked section modulus ($I_{cracked} = 0.35I_{gross}$) to account for bending deformations. Between the header beam elements and wall elements, rigid beams were modeled to account for the physical width of the walls. A shear hinge was modeled at the midpoint of each beam to account for the shear yielding and nonlinearity in the elements.

The header beams are 20” wide by 24” deep with 15M stirrups spaced at 4” as shown in Fig. 3. The nonlinear shear hinge properties were calibrated to a reverse-cyclic test on a similar beam performed by Galano and Vignoli [9]. The nonlinear hinges used the Pinching4 material model to capture pinching, in-cycle degradation, and cyclic stiffness and strength degradation [10]. The Pinching4 numerical model comparison to the test results is presented in Fig. 4.

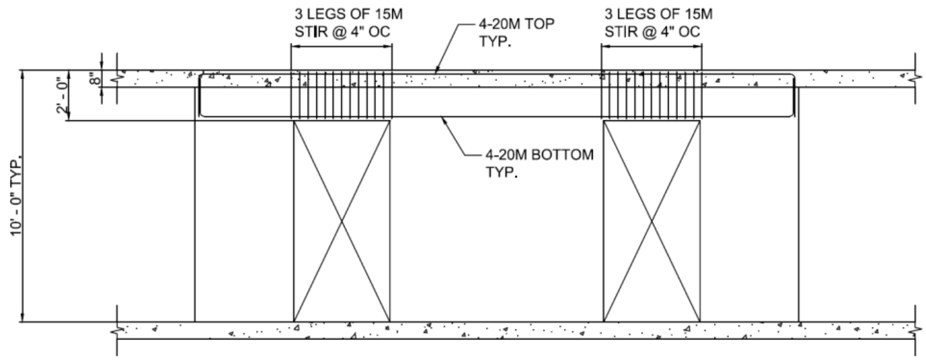


Fig. 3 – Elevation view of slabs and header beams

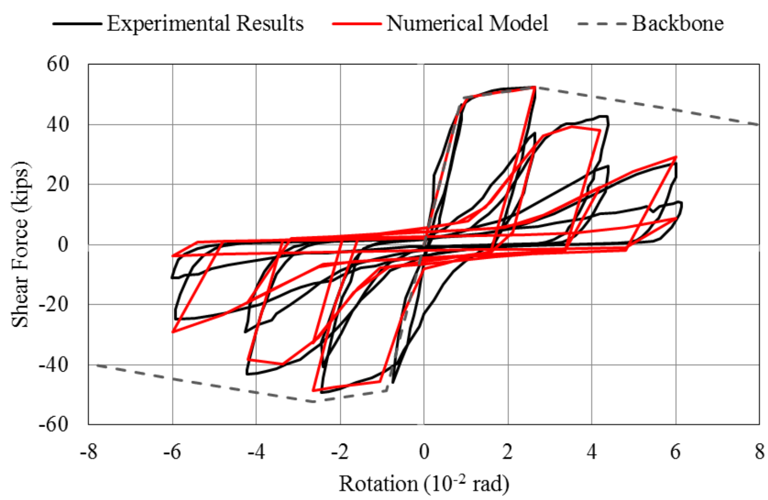


Fig. 4 – Nonlinear shear hinge model

Concrete was modeled with the Concrete02 material model [11]. Confinement was accounted for using the Mander et al. relationship [12]. Reinforcing steel was modeled using the Bilin material model to account for cyclic strength degradation [13] observed in cyclic testing protocols [14].

Elastic shear hinges were modeled at the midpoint of each wall in each storey to account for shear deformations in the walls. The hinges were modeled with stiffness equal to the cracked shear area multiplied by the shear modulus divided by the storey height. To account for the loss of shear area due to cracking, the gross area was multiplied by 0.1 [15].

The gravity system was not explicitly modeled, but to account for the second order effects of the weight carried by the gravity system a leaning (or PDelta) column was included in the model. The weight of the structure not applied directly on the shearwalls was applied on the leaning column, which was pinned at the base and constrained to the walls at each storey level. Rigid diaphragm constraints were applied at each level. An illustration of the OpenSees model is presented in Fig. 5.

Damping was applied as 2.5% Rayleigh damping in the first and third modes. The first three periods were 1.87, 0.53, and 0.27 seconds, respectively.

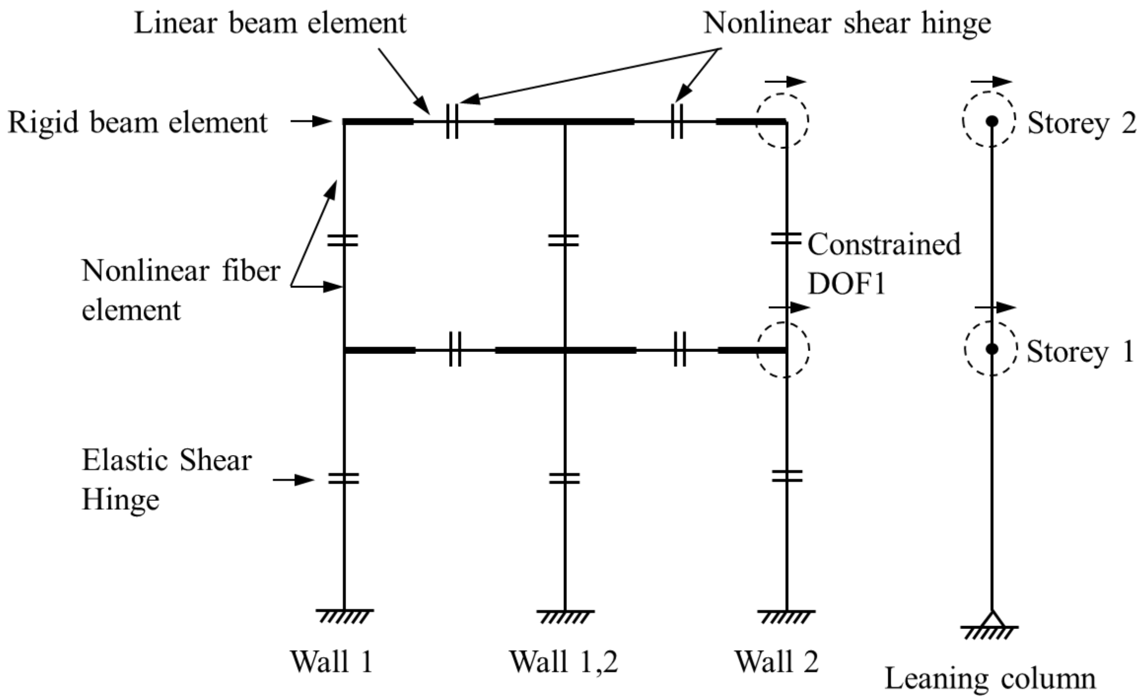


Fig. 5 – Illustration of OpenSees numerical model

3. Ground Motions Suites

Two suites of ground motions suites were selected in order to investigate the effect of shaking duration. The records were chosen from the PEER NGA-West1 database [16] as well as a database comprising several historic subduction interface events [17, 18].

In order to quantify record duration 5-95% significant duration (D_{5-95}) was adopted. D_{5-95} corresponds to the duration in the ground motion record between 5% and 95% of the total energy accumulation, where energy is expressed as Arias intensity (the time integration of the acceleration squared).

3.1 Long Duration Suite

The first suite comprised 30 long duration records selected and linearly scaled to the Vancouver 2010 design spectrum. The events and records selected are summarized in Table 1. Most of the events are from large magnitude crustal events and subduction interface events.

Table 1 – Long duration suite summary

Event	Type	Magnitude	Year	Number of Stations	Stations
Chi-Chi, Taiwan	Crustal	7.6	1999	6	CHY046, TCU056, TCU106, TCU120, TCU122, TCU138
Hokkaido, Japan	Subduction	8.0	2003	5	HKD079, HKD084, HKD099, HKD103, HKD109
Imperial Valley, Ca.	Crustal	6.5	1979	1	DELTA
Kobe, Japan	Crustal	6.9	1995	1	SAKAI
Landers, Ca.	Crustal	7.3	1992	2	DLT, MVH
El Maule, Chile	Subduction	8.8	2010	4	ANT, LACH, stgocentrol, stgoflorida
Tohoku, Japan	Subduction	9.0	2011	11	FKS012, GNM013, IBR009, KNG205, MYG016, TKY004, TKY005, TKY024, TKY025

3.2 Short Duration Suite

For each record in the long duration suite, a spectrally equivalent short duration earthquake record was selected by minimizing the mean squared error between the spectra of the two records. Fig. 6 illustrates an example of a long record and spectrally equivalent short duration record.

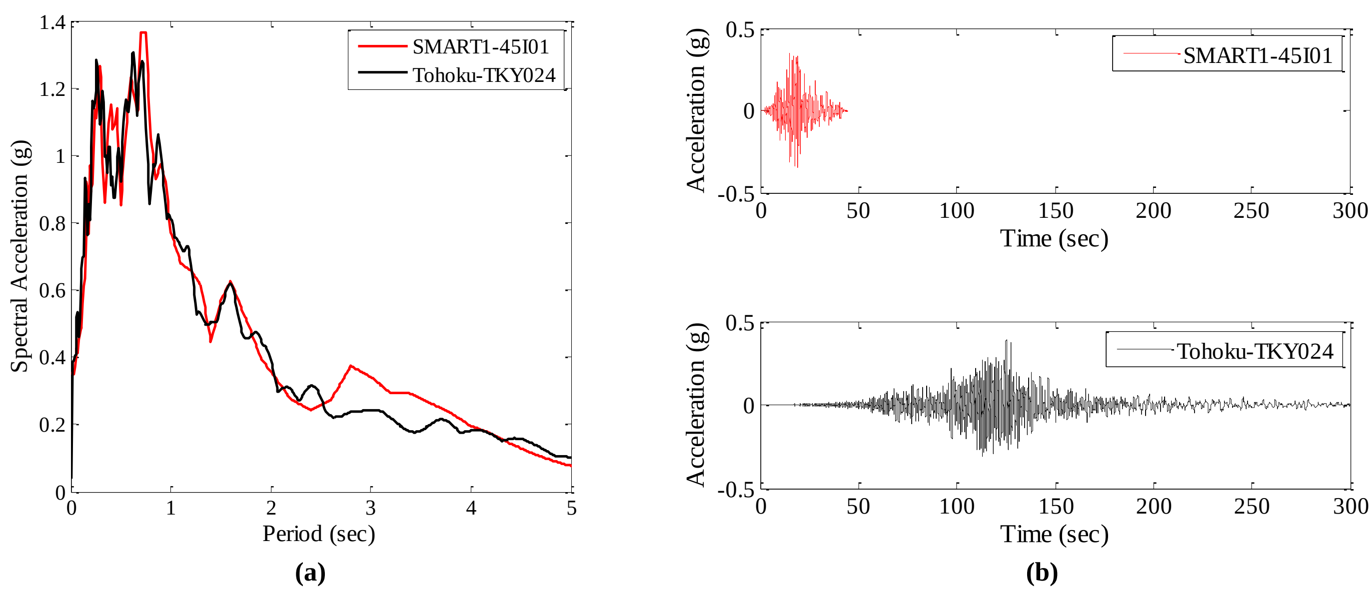


Fig.6 – Example spectrally equivalent records (a) spectrum and (b) time histories

Table 2 summarizes the selected short duration record sets. Fig. 7 presents the spectra of the two suites along with the Vancouver 2010 design spectrum. Fig. 8 compares the significant duration of the two suites.

Table 2 – Short duration suite summary

Event	Type	Magnitude	Year	Number of Stations	Stations
Chi-Chi, Taiwan	Crustal	7.6	1999	11	CHY024, NSY, TCU029, TCU051, TCU053, TCU054, TCU072, TCU089, CHY028, CHY035
Gazli, Uzbekistan	Crustal	6.8	1976	1	KARAKYR
Greece	Crustal	6.2		1	KALAMATA
Imperial Valley, Ca.	Crustal	6.5		4	BRA, E03, E04, E06
Loma Prieta, Ca.	Crustal	6.9	1989	1	HDA
Manjil, Iran	Crustal	7.4	1990	1	184
Northridge, Ca.	Crustal	6.7	1994	3	CCN, LDM, RRS
San Salvador	Crustal	5.8		1	NGI
S. Fernando Valley	Crustal	6.6	1971	1	PDL
SMART, Taiwan	Crustal	7.3		4	40E01, 45I01, 45O02
Victoria, Mexico	Crustal	6.3	1980	1	CHIHUAHUA
Westmoreland, Ca.	Crustal	5.9	1981	1	PTS

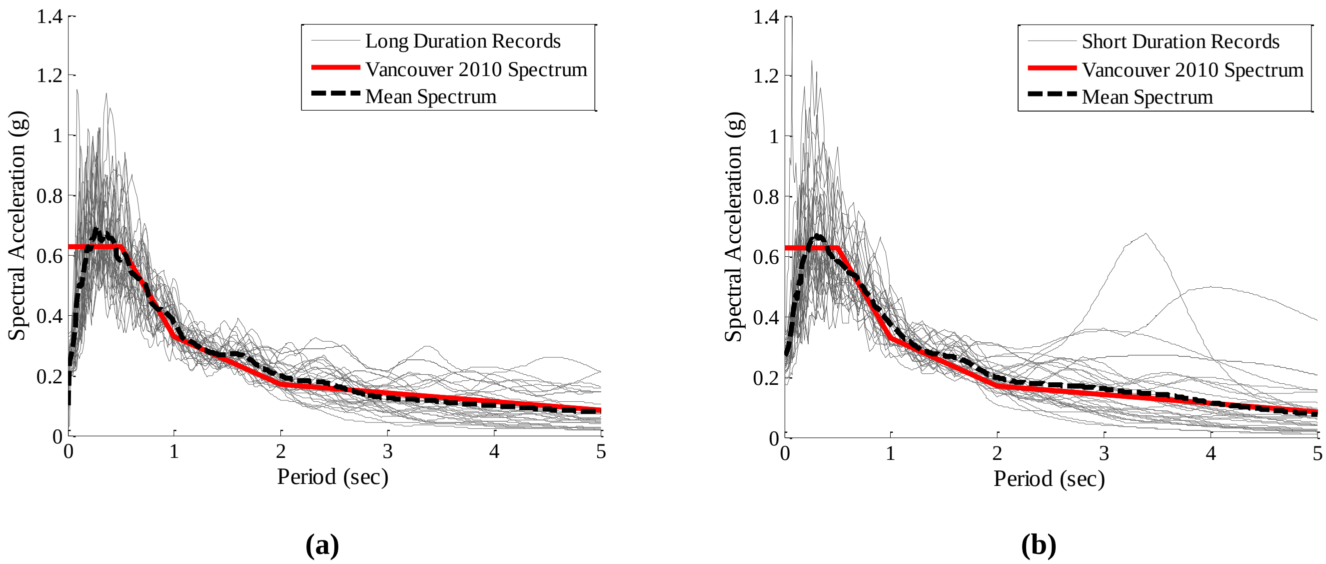


Fig. 7 – Spectra of (a) long duration suite and (b) short duration suite

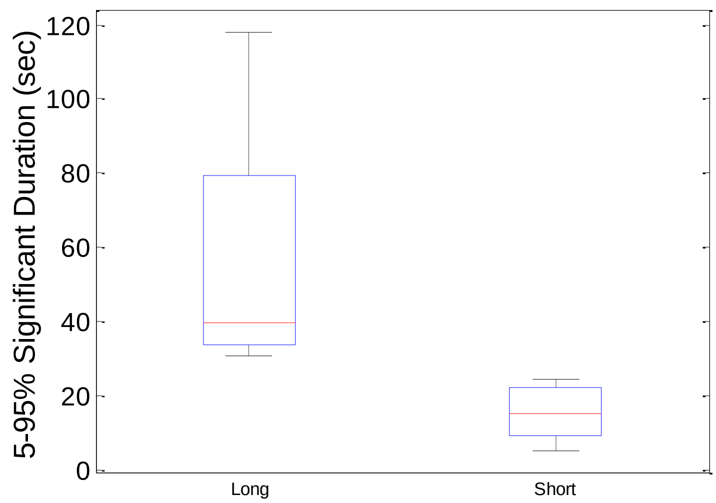


Fig. 8 – Significant duration ($D_{5\text{-}95}$) statistics comparison

4. Results

4.1 Code Level Analysis

The NBCC specifies a ground motion shaking level with a 2% in 50 year probability of exceedance for code-level analysis. Accordingly, the two suites of ground motions were first scaled to the Vancouver NBCC 2010 design spectrum (see Fig. 7) and used to analyze the model using nonlinear time history analysis. The resulting maximum displacements, interstorey drifts, and storey accelerations are shown in Figs. 9, 10, and 11, respectively.

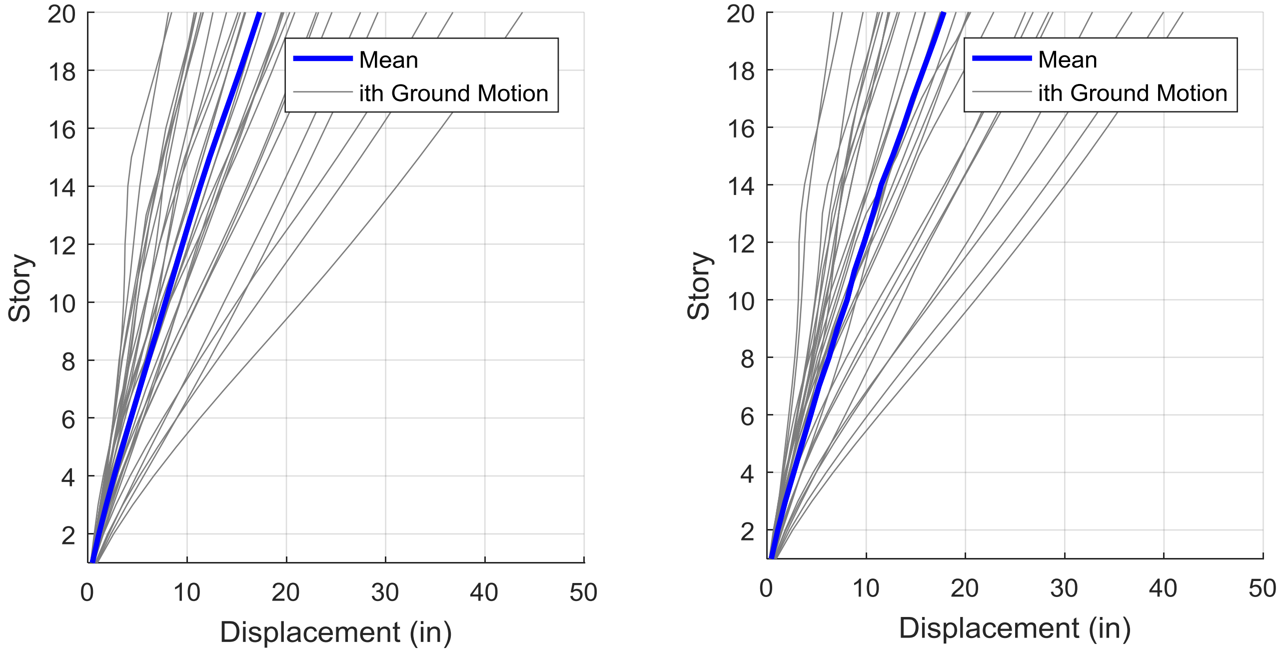
The NBCC 2010 uses interstorey drifts as a surrogate for structural damage, and limits regular buildings to a maximum interstorey drift of 2.5% of the storey height. As seen in Fig. 10, on average, neither suite exceeds this drift limit.

The results for these two suites of ground motions are similar. The longer duration suite does not significantly increase displacement or acceleration demands on the structure. This may be because the overall damage (interstorey drift levels) is quite low at this level of shaking (the maximum is only about 1% for both motion suites). At these lower levels of damage, the amount of degradation in the walls and header beams will be quite low, which will nullify the effect of the ground motion duration.

The energy response of the structure was also computed during the time history analysis using the restoring force vector and incremental displacement vector. Fig. 12 compares the energy response calculated in the structure at this level of shaking for the two motion suites. Despite producing similar peak displacement and acceleration results, the long duration suite has much high energy demands, which may lead to more structural and nonstructural damage in the building.

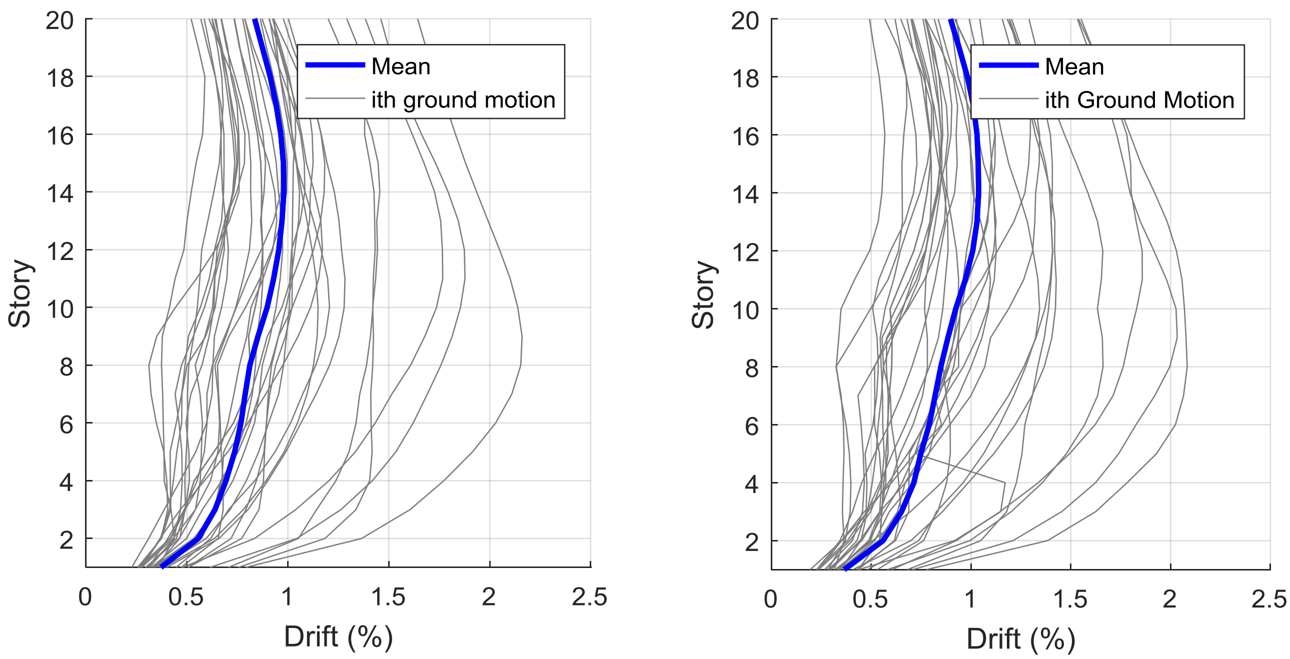
4.2 Collapse Level Analysis

Next, in order to determine if long duration ground motions increase the collapse risk of reinforced concrete shearwall structures at higher shaking levels, the two suites of ground motions were incrementally scaled up until collapse was reached. Collapse is defined as excessive interstorey drifts or numerical instability. Fig. 13 presents the fragility curves derived for the two ground motion suites.



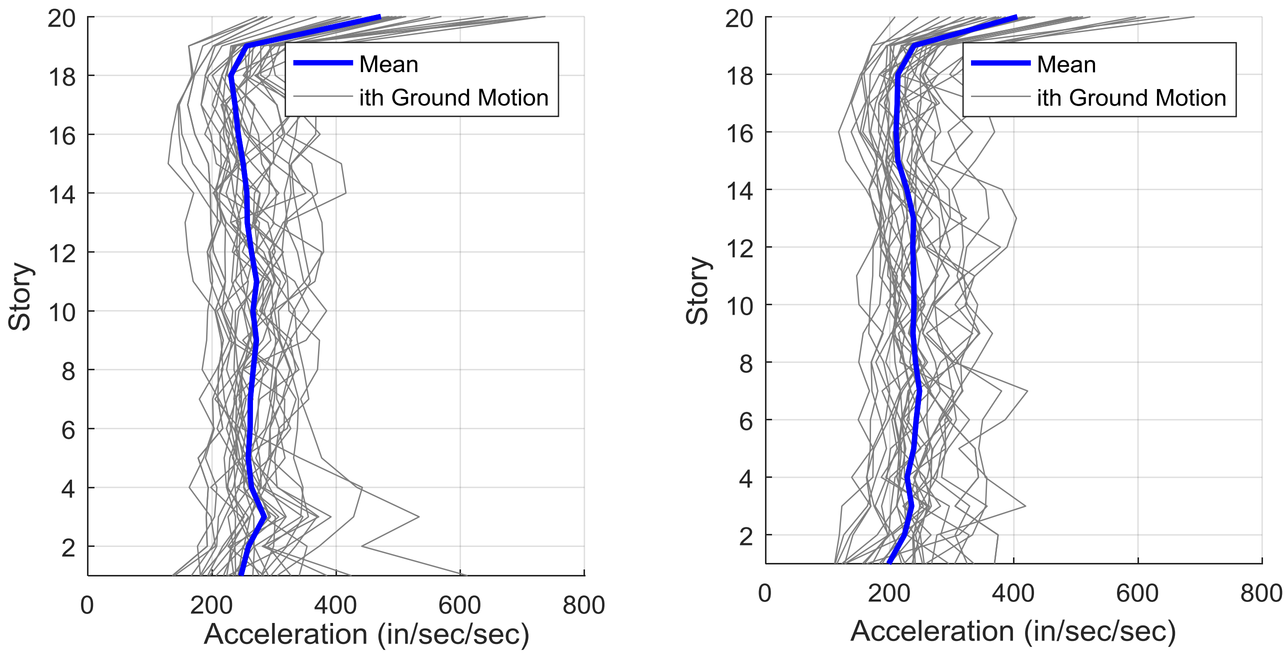
(a) **(b)**

Fig. 9 – Displacement results at code shaking level for a) long duration suite and b) short duration suite



(a) **(b)**

Fig. 10 – Interstorey drift results at code shaking level for a) long duration suite and b) short duration suite



(a) **(b)**

Fig. 11 – Acceleration results at code shaking level for a) long duration suite and b) short duration suite

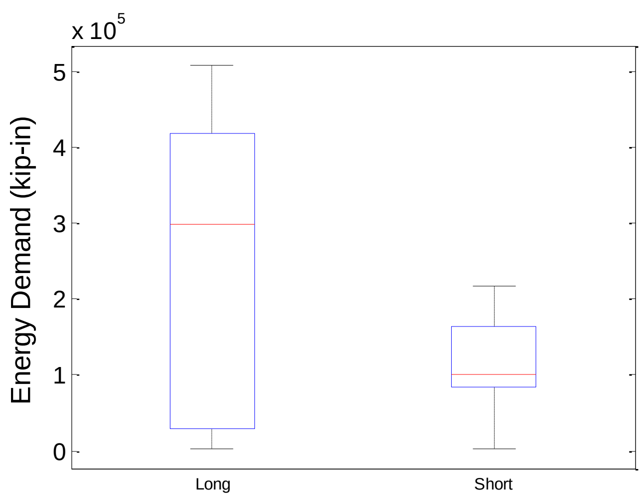


Fig. 12 – Energy demand statistics comparison

As seen in Fig. 13, at lower scaling levels, up to 150% (100% is the design scaling level according to the NBCC 2010 for Vancouver, BC) there is little to no probability of collapse. Up to 250% scaling, the short duration records tend to have a higher probability of collapse. Both suites reach 50% probability of collapse at around the 250% scaling level. Above this level, the long duration suite has a higher probability of collapse.

It should be noted that the short suite has a higher standard deviation compared to the long duration suite. This is because a small number of records caused collapse at low scaling levels, while the majority required very large scaling factors. This could be due to very strong pulses in several of these records which caused premature collapse. On the other hand, the long duration suite, most of which did not have strong pulses, all tended to cause collapse near the same scaling level.

If we define the collapse margin ratio (CMR) as the ratio between scaling level where 50% probability of collapse is observed and the design scaling level [19], then both suites yield the same CMR. Accordingly, we can conclude that when considering maximum interstorey drift as a surrogate for collapse, the factor of safety against collapse for this structure is not affected by ground motion duration.

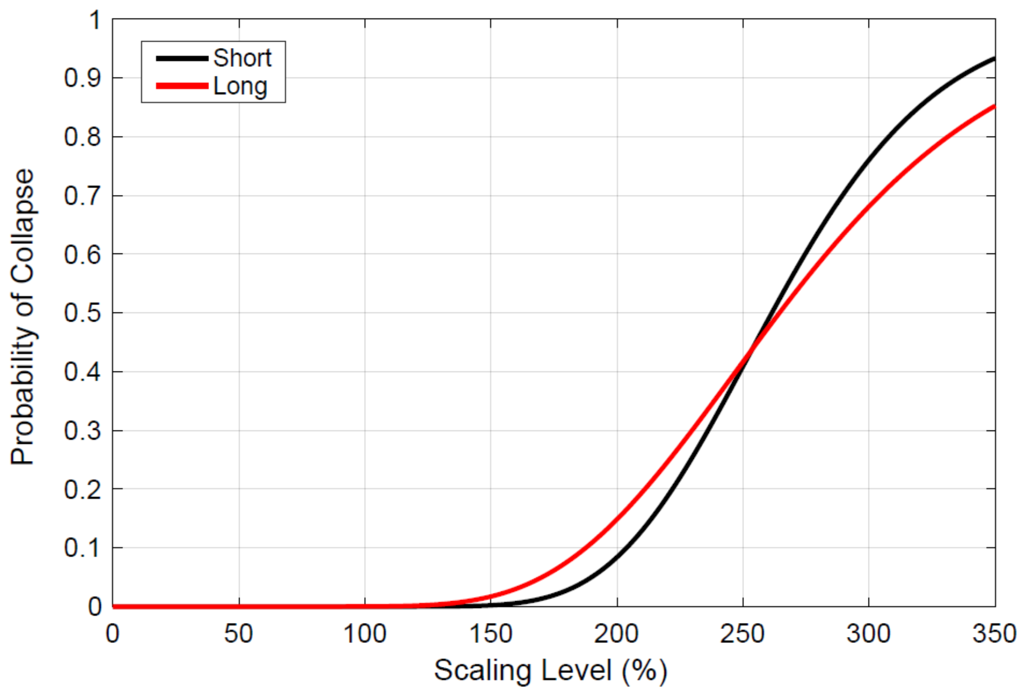


Fig. 13 – Fragility results for long and short duration motion suites

5. Conclusions

In this study a 20 storey reinforced concrete shearwall building was analyzed using two sets of records: a long duration suite, and a spectrally equivalent short duration suite. At a code level of shaking (2% in 50 year probability of exceedance), little damage was observed and the peak responses of the structure were not influenced by ground motion duration.

When an incremental dynamic analysis was performed and the ground motion suites were scaled to very high levels of shaking, the collapse margin ratio was not affected by ground motion duration. Most of the short duration records required very large scaling factors before collapse was observed; however, a several of the records had significant acceleration pulses which cause premature collapse. This resulted in a similar median probability of collapse between the two record suites.

Based on these results, ground motion duration does not seem to be a significant factor in the peak response of this type of structure at the code prescribed level of shaking. Also duration did not tend to affect the safety margin against collapse of the prototype building. However, it was observed that duration was correlated with cumulative damage measures, energy demands in this case, which has also been widely recognized in the literature [4]. This means that while collapse probabilities and peak responses may not be sensitive to shaking duration, the amount of damage in the structure may still be affected.

6. References

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