Tenth U.S. National Conference on Earthquake Engineering
Frontiers of Earthquake Engineering
July 21-25, 2014
Anchorage, Alaska

SEISMIC PERFORMANCE OF SHEAR WALL BUILDINGS WITH GRAVITY-INDUCED LATERAL DEMANDS

M. Baradaran Shoraka[1], M. R. Dupuis[2], J. Macauley[3], K. J. Elwood[4], D. L. Anderson[4], and R. Simpson[5]

ABSTRACT

Building structural systems are commonly idealized as two distinct systems: a seismic force resisting system (SFRS), designed to resist lateral demands during strong ground shaking, and a gravity system, designed to support gravity loads and detailed to withstand imposed lateral deformations during seismic response without loss of gravity-load support. However, current architectural trends have resulted in modern buildings with inclined facades and irregularities in the gravity system that applies gravity-induced lateral demands to the SFRS. Any impact the gravity system may have on the response of the SFRS is not currently considered during the design process or in current building codes. This paper summarizes the results of a study to identify if there are behavioral trends not recognized within the scope of current building codes. To this end, a nonlinear, parametric study was conducted in a structural analysis platform to investigate the inelastic response of concrete shear wall buildings including gravity-induced lateral demands with a range of design characteristics at various hazard levels. The results demonstrate that a seismic ratcheting effect can develop and amplify inelastic displacement demands that lead to an increase in the structural collapse metrics. The effect is significantly more prevalent in coupled shear walls compared with cantilevered shear walls. An irregularity class to address buildings with gravity-induced lateral demands on the seismic force resisting system is proposed for the National Building Code of Canada.

[1]Post-doctoral Researcher, Dept. of Civil Engineering, University of British Columbia, Vancouver, BC, V6T 1Z4
[2]Design Engineer, Read Jones Christoffersen Ltd, Vancouver, BC, V6H 3X8
[3]Design Engineer, Glotman·Simpson Consulting Engineers, Vancouver, BC, V6J 1N5
[4]Professor, Dept. of Civil Engineering, University of British Columbia, Vancouver, BC, V6T 1Z4
[5]Principal, Glotman·Simpson Consulting Engineers, Vancouver, BC, V6J 1N5

Baradaran Shoraka M, Dupuis M.R., Macauley J., Elwood K.J., Anderson D.L., and Simpson R., Seismic Performance of Shear Wall Buildings with Gravity-Induced Lateral Demands. *Proceedings of the 10th National Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Anchorage, AK, 2014.

Seismic Performance of Shear Wall Buildings with Gravity-Induced Lateral Demands

M. Baradaran Shoraka[1], M.R. Dupuis[2], J. Macauley[3], K. J. Elwood[4], D. L. Anderson[4], and R. Simpson[5]

ABSTRACT

Building structural systems are commonly idealized as two distinct systems: a seismic force resisting system (SFRS), designed to resist lateral demands during strong ground shaking, and a gravity system, designed to support gravity loads and detailed to withstand imposed lateral deformations during seismic response without loss of gravity-load support. However, current architectural trends have resulted in modern buildings with inclined facades and irregularities in the gravity system that applies gravity-induced lateral demands to the SFRS. Any impact the gravity system may have on the response of the SFRS is not currently considered during the design process or in the current building codes. This paper summarizes the results of a study to identify if there are behavioral trends not recognized within the scope of current building codes. To this end, a nonlinear, parametric study was conducted in a structural analysis platform to investigate the inelastic response of concrete shear wall buildings including gravity-induced lateral demands with a range of design characteristics at various hazard levels. The results demonstrate that a seismic ratcheting effect can develop and amplify inelastic displacement demands that lead to an increase in the structural collapse metrics. The effect is significantly more prevalent in coupled shear walls compared with cantilevered shear walls. An irregularity class to address buildings with gravity-induced lateral demands on the seismic force resisting system is proposed for the National Building Code of Canada.

Background and Motivation

Unique architectural features and other irregularities in the gravity system that apply gravity-induced lateral demands (GILDs) on the seismic force resisting system (SFRS) are increasingly being incorporated in new buildings in Canada. The impact of such gravity-induced lateral demands on the seismic response of the building is currently not considered in the NBCC [1] or other international building codes. These permanent demands have raised concerns, due to the perceived potential for a ratcheting effect to occur during seismic loading [2]. The results of recent studies have demonstrated that a seismic ratcheting effect can develop, amplifying inelastic displacement demands, and potentially leading to collapse in strong ground shaking [3,4,5]. The following briefly summarises a parametric study on concrete core wall buildings, 5 to 50 stories by Dupuis et al. [5].

Dupuis et al. [5] considered two types of SFRSs; cantilevered shear walls and coupled

[1]Post-doctoral Researcher, Dept. of Civil Engineering, University of British Columbia, Vancouver, BC, V6T 1Z4
[2]Design Engineer, Read Jones Christoffersen Ltd, Vancouver, BC, V6H 3X8
[3]Design Engineer, Glotman·Simpson Consulting Engineers, Vancouver, BC, V6J 1N5
[4]Professor, Dept. of Civil Engineering, University of British Columbia, Vancouver, BC, V6T 1Z4
[5]Principal, Glotman·Simpson Consulting Engineers, Vancouver, BC, V6J 1N5

shear walls. OpenSees [6] is used for the nonlinear analyses and both SFRS systems are modeled in two dimensions. All inelastic behavior in the cantilevered shear wall was assumed to occur in a plastic hinge at the base of the wall. In the coupled shear walls, in addition to the base of the wall, the coupling beams were also modeled using nonlinear fiber elements based on modeling recommendations from ATC 72-1 [7]. A coupling ratio (ratio of overturning moment resisted by yielding of all coupling beams to the design overturning moment) of 0.7 was used for the coupled wall models. The impact of the gravity system on the SFRS is expressed as a demand ratio, α, where α is the ratio of the permanent static moment demands at the base of the shear wall, M_{GILD}, to the base yield strength required to resist seismic demands, M_y:

$$\alpha = \frac{M_{GILD}}{M_Y} \tag{1}$$

Demand ratios ranging from 0.0 (i.e. representing no permanent demand on the SFRS) to 0.8 were considered by Dupuis et al. [5]. A parameter – the relative amplification factor, β - was introduced to relate performance metrics in a building with GILDs to an equivalent building without GILDs, namely:

$$\beta = \frac{\Delta\,(R_{\alpha=0},\alpha)}{\Delta\,(R_{\alpha=0},\alpha=0)} \tag{2}$$

where Δ is the maximum roof drift, and $R_{\alpha=0}$ is the relative strength factor defined as the ratio of the maximum elastic base overturning moment (estimated based on the 2% in 50 years spectrum used for design in NBCC [1]) , $M_{E\,max}$, to the yield strength, M_y, of the SFRS assuming no GILD. Using a set of ten ground motions scaled to the UHS for Vancouver, the study by Dupuis et al. [5] sought to define a maximum allowable value of α above which standard elastic design procedures can no longer reliably estimate the deformation demands on building structures with GILDs and nonlinear analysis is required to accurately capture the performance of the building. The limits on α were found to be dependent on the hysteretic characteristics of the SFRS (i.e. fat hysteresis in coupled shear walls or flag shaped in cantilevered shear walls with high axial loads). Dupuis et al. found that the amplification in displacement demands, β, was not sensitive to the period of the building or relative strength factor, $R_{\alpha=0}$. Results from the Dupuis et al study are summarized in Figure 1 to assist in selecting appropriate limits for α. Figure 1 compares the displacement amplification, β, of coupled and cantilevered shear walls for different values of α. The figure provides the median β value (solid line) for the worst performing building (i.e. considering all periods and $R_{\alpha=0}$) and the maximum β (dashed line) for all ground motions and buildings considered. The higher slopes noted for the coupled wall building indicate that the limit on α should be lower for coupled walls compared with cantilevered walls. These results were reviewed by the Standing Committee on Earthquake Design of the Canadian Commission on Building and Fire Codes in the development of a new irregularity clause for GILD. Based on the judgment of the Committee, it was decided to limit the median amplifications of displacements due to GILDs to less than 25% (i.e. $\beta < 1.25$). Consulting Figure 1, the selected limit on β suggests allowable α values of 0.06 and 0.2 for coupled and cantilevered shear wall buildings, respectively. A new irregularity type has been proposed for the 2015 NBCC whereby a building with α values above these limits would be required to be assessed using nonlinear analyses to demonstrate that a ratcheting behavior during seismic response is not anticipated. Readers are referred to Dupuis et al. [5] for more information on the above study and the proposed irregularity clause.

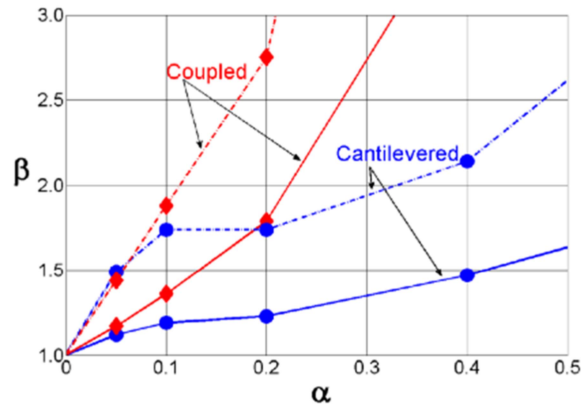


Figure 1. Deformation amplifications corresponding to the median β (solid line) for the worst performing building (considering all periods and $R_{\alpha=0}$) and the maximum β (dashed line) for all ground motions and buildings considered [5].

The study presented in this paper will extend the Dupuis et al. study [5] by considering the following:

- Updating the numerical model to have a better estimation of the displacement amplifications due to the GILD on the SFRS;
- Considering additional case studies for design and loading conditions to study the influence of GILDs on the response of core wall buildings;
- Using collapse fragilities (i.e. the response of buildings with GILDs in various hazard levels) to better understand the impact of GILDs at different levels of ground shaking.

Numerical Model

The first step in extending the previous study is to update the numerical model of the SFRS. As explained in the previous section, the numerical models used in Dupuis et al. [5] have assumed that all inelastic behavior in the wall to occur in a plastic hinge at the base of the wall. However, recent studies have indicated that yielding is likely to occur at upper stories in tall core wall buildings [7]. Realistic modeling of core (cantilevered or coupled) walls is critical to achieving good estimates of deformation demands. The numerical models in the current study have the following additional features:

- Displacement-based nonlinear beam-column (fiber) elements with material regularization [8] are used to capture the nonlinear flexural response of the shear walls over the full height of the buildings.
- Reinforcing steel in walls is modeled as a hysteretic material in OpenSees. The post-yield slope and the corresponding strain values of bar buckling and bar rupture differ along the length of the element (based on the material regularization rules of [8]). It is assumed that bar buckling occurs at a strain equal to concrete crushing. Strength loss after bar buckling is modeled at a slope of $-0.02E_s$ until steel reaches $0.2f_y$ [9]. Strength loss after bar fracture

is modeled at a slope of approximately $-0.06E_s$ to $-0.08E_s$ [10]. Models used in Dupuis et al did not include material strength degradation.

- Nonlinear (bi-linear) shear modeling is used to capture softening effects after shear cracking. Cracking is established based on Eq. 11-27 of ACI 318-11 [11] and a post-cracking slope of 0.01E is adopted [10]. An uncoupled shear model is adopted in the current study.

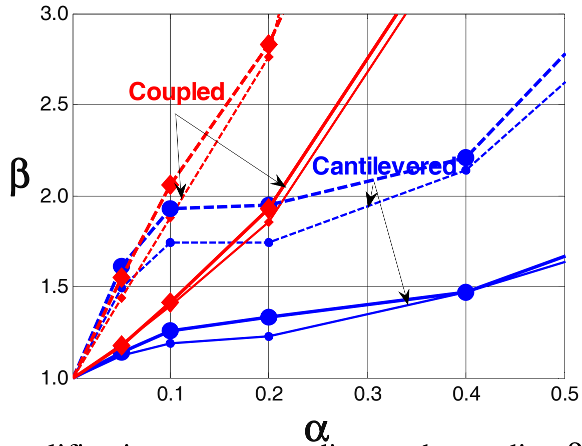


Figure 2. Deformation amplifications corresponding to the median β (solid line) for the worst performing building (considering all periods and $R_{\alpha=0}$) and the maximum β (dashed line) for the updated numerical model (thick lines) and the original model [5] (thin lines).

In addition to the above changes to the numerical model, the 22 pairs of far field FEMA P695 [12] ground motions were used for the current study instead of the ten crustal ground motions used in [5]. Figure 2 compares the response of coupled and cantilevered shear walls for different values of α for both the updated numerical model and the original model used in [5]. Overall the results are similar, indicating that the simpler models used in the Dupuis study were generally adequate to capture the response of buildings with GILD. Using the same criteria (i.e. $\beta < 1.25$) for the updated numerical model in Figure 2 the selected limit on β suggests allowable α values of 0.06 and 0.15 for coupled and cantilevered shear wall buildings, respectively. As seen in Figure 2, nonlinearity has a greater impact on the cantilevered shear wall. This is because of the greater possibility of mid-height yielding in cantilevered shear walls compared to the coupled cases.

Case Studies

The Dupuis et al study selected specific design characteristics for the archetype buildings used to select limits on the allowable GILD. Additional design and loading conditions in real buildings can influence the response of buildings with GILDs. To further understand the influence of these varying design and loading conditions on the response of buildings with GILD, four additional case studies were completed. For comparison, the results from the updated numerical model presented in the previous section (Figure 2) will be used as the base model.

Strengthened Coupled Beams

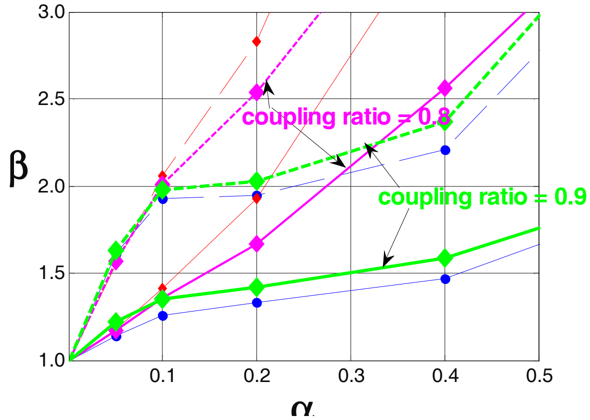
The first study was conducted to investigate the performance of the building with stronger coupling beams, and thus a greater coupling ratio. The strength of the coupling beams was increased and the flexural strength at the base of the shear walls was decreased - an approach that allowed the shear walls to maintain constant overturning moment resistance, while achieving coupling ratios of 0.8 and 0.9. By strengthening coupling beams, additional nonlinear response was shifted to the base of the shear walls. One of the key conclusions from the study by Dupuis et al, was that seismic ratcheting was promoted by yield of the coupling beams. By shifting inelasticity from the coupling beams to the base of the shear walls, the seismic ratcheting phenomenon was partially mitigated resulting in a decrease in the median β, as shown in Figure 3a (note that the thin lines in the figure show the results from the updated numerical model with a coupling ratio of 0.7). In the case for a coupling ratio of 0.9 the response of the coupled shear walls and the cantilevered shear walls are almost identical. Therefore, this value of 0.9 could be used as a limit to distinguish between the coupled and cantilevered shear walls in the new irregularity clause proposed for the 2015 NBCC.

Influence of Axial Load

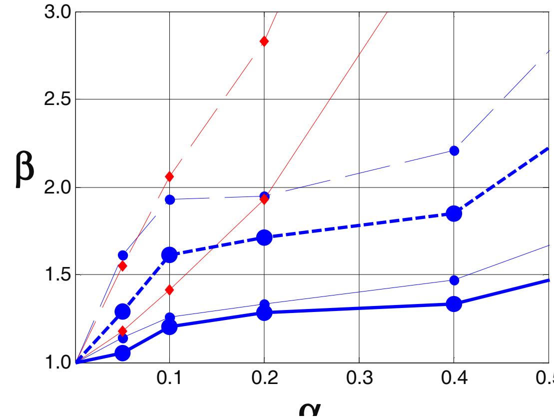
Dupuis et al. [5] have highlighted the influence of the hysteretic behavior exhibited by the SFRS on the extent of the seismic ratcheting phenomenon. When the hysteretic behavior was flag-shaped (i.e. cantilevered shear walls), the GILD had a much smaller influence; causing only mild to moderate increases in inelastic displacement demands; however, the shape of the hysteresis is influenced by the axial load on the cantilevered wall, with higher axial loads leading to a more flag-shaped hysteretic response. The Dupuis et al study conservatively assumed that the core only supported the weight of the wall itself. Here we consider the influence of increasing the axial load on the core wall. 20% of the slab weight in addition to the wall weight is considered as the axial load on the wall in this case study. The results in Figure 3b indicate that the increase of axial load mitigated the seismic ratcheting phenomenon on the cantilevered shear walls. On the contrary, the increase of axial load on the coupled shear walls was negligible. This response is due to the fact that additional axial load on the cantilevered shear walls will increase the pinching of the hysteretic response and therefore will decrease the impact of the GILD on the building response. Increasing the axial load on the coupled walls will not influence the response of the coupling beams and hence there is no reduction in the ratcheting behavior.

Gravity System Irregularity

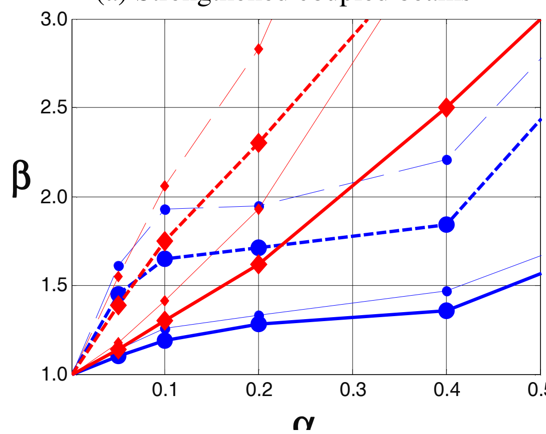
In the third study, three different irregularities in the gravity system were considered: inclined columns partway up the building (over the ground floor lobby and the second floor), eccentric floor spans (i.e. a gravity system that applies greater dead and live loads to one side of the shear wall than the other), and a mid-height constant moment (e.g. a large cantilevered or transfer girder supported by the SFRS). All three irregularities were selected to achieve the same GILDs at the base of the concrete shear walls, i.e. the same α. The results are shown in Figure 3c, 3d, and 3e for the three cases, respectively. All three cases are compared to the original case where the gravity system is assumed to be inclined columns over the full height of the building. The results indicate that for the inclined columns partway up the building case, the GILD (regardless of the SFRS system) will result in smaller amplifications in displacement demands. On the contrary, the eccentric floor spans and the mid-height constant moment cases will result in larger amplifications in displacement demands due to GILDs compared to original case. The results from these three cases imply that the greater the overall distribution of bending moments on the SFRS due to the GILD, the greater the amplifications on the displacement demands.



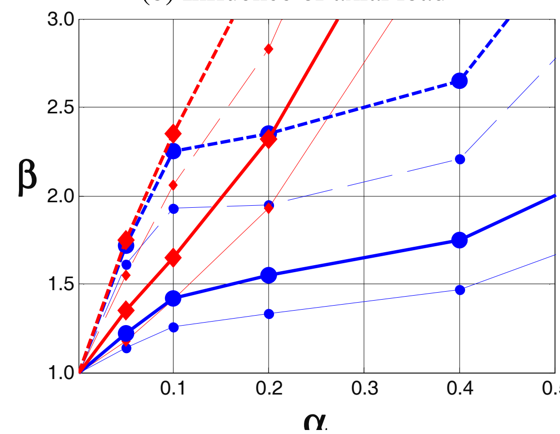
(a) Strengthened coupled beams



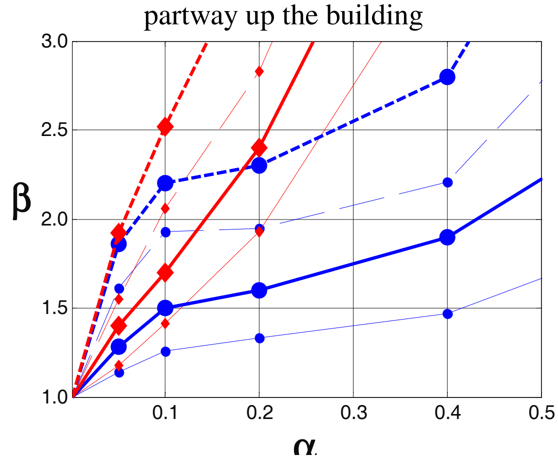
(b) Influence of axial load



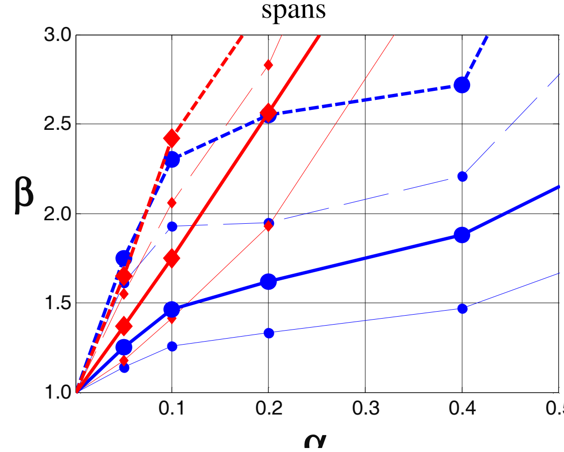
(c) Gravity system irregularity - inclined columns partway up the building



(d) Gravity system irregularity - eccentric floor spans



(e) Gravity system irregularity - mid-height constant moment



(f) Subduction ground motions

Figure 3. Deformation amplifications corresponding to the median β (solid line) for the worst performing building (considering all periods and $R_{\alpha=0}$) and the maximum β (dashed line) for the different case studies (the thinner lines present the base case in Figure 2).

Subduction Ground Motions

The fourth study was conducted with subduction ground motion records used in the British Columbia schools retrofit project [13], each linearly scaled to induce the same maximum elastic base overturning moments as the Vancouver UHS. This scaling approach was adopted to allow direct comparison of response quantities with the results from the ground motions discussed above. The results are presented in Figure 3f. As seen in this figure, the subduction ground motions have a greater amplification on the building response likely owing to their longer duration and greater number of cycles. The strength degradation included in the numerical model enables the analysis to capture the impact of the number of cycles on the building response. The results from this case study highlight the need to be cautious with the design of buildings with GILDs in a region with a prominent subduction earthquake hazard.

Collapse Fragilities

The Dupuis et al study and the case studies in the previous section have only considered the performance of buildings subjected to 2%/50 year ground motions (i.e. MCE event). Considering the sensitivity of systems with GILDs to collapse, there is a need to use Incremental Dynamic Analysis (IDA) to estimate the probability of collapse and select limits on the GILDs based on a consistent probabilistic framework for all SFRS with and without GILD.

IDA results using the FEMA P695 far field ground motions are described in this section. Collapse fragilities are provided, enabling a comparison of the collapse probability results for different case studies. Three additional case studies are considered in this section, each with a relatively narrow scope, focusing on a single building configuration: the 30 story cantilevered and coupled shear wall buildings with $R_{\alpha=0} = 2.0$ and GILDs corresponding to $\alpha = 0.4$. The spectral acceleration at MCE for this building in Vancouver is 0.17g.

The conditional probability of collapse (or collapse fragilities) determined from the results of the IDA for the base case and both SFRS (cantilevered and coupled shear walls) are shown in Figure 4a. The FEMA P695 methodology defines collapse fragility in terms of the value of the median collapse intensity (S_{CFT}[T]), adjusted by the spectral shape factor (*SSF*), and the value of total system collapse uncertainty (β_{TOT}). The total system collapse uncertainty, β_{TOT}, used to plot the collapse fragilities was determined based on an assessment of the quality ratings associated with the design requirements, test data, and nonlinear models, as well as record-to record uncertainty. For the concrete archetypes design requirements, test data were assessed as "Good", and nonlinear modeling was assessed as "Fair" (according to the definitions of FEMA P695), resulting in a total system collapse uncertainty of β_{TOT}=0.6. Figure 4a also includes adjustment of the median (S_{CFT}[T]) for the SSF.

Figure 4a clearly indicates that the GILD significantly increases the probability of collapse for both cantilevered and coupled shear wall systems. However, the impact is more prominent for the coupled system. This is consistent with the observations from the previous section that the coupled shear wall system has more impact on the building response compared to the cantilevered system. Collapse evaluation results are summarized in Table 1 for the different case studies explained below. In this table the *ACMR* (adjusted collapse margin ratio, *SSF* x

CMR [12]) and the probability of collapse at the MCE level are also reported. Based on the FEMA P695 evaluation process, it is suggested that buildings will be considered safe (from a collapse point of view) whenever the probability of collapse at MCE is less than 10% and the *ACMR* (for β_{TOT}=0.6) over 2.16 [12]. Based on these collapse criteria, all four buildings under the base case category are considered to have an acceptable collapse performance. In fact, the results suggest that the design of core wall buildings according to NBCC 2010 and CSA A23.3-04 [14] result in very low probabilities of collapse, regardless of the GILD. Note that the NBCC does not use the probability of collapse to select seismic performance factors (i.e. R_d and R_o). Additionally, the minimum longitudinal reinforcement criteria in CSA A23.3-04 usually governs in the high-rise shear wall buildings designed in a moderate hazard region (i.e. Vancouver). This additional reinforcement will also translate in higher collapse capacity.

Subduction Ground Motions

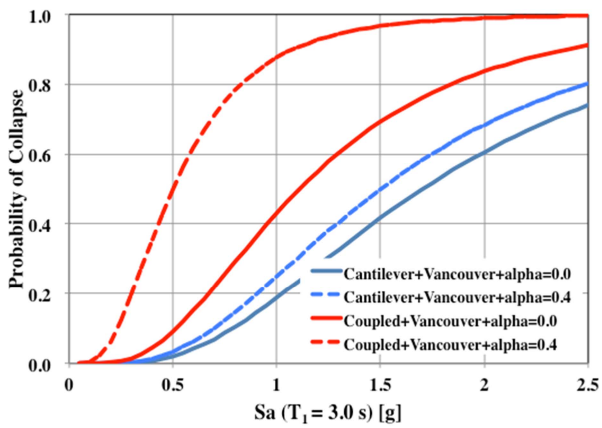
The second study was conducted with the same ten subduction ground motion records described in the previous section. The results are shown in Figure 4b. The collapse fragilities indicate a decrease in the median collapse capacity by nearly a factor of 2 for systems with GILD when subduction ground motions are considered, while there is limited impact on the collapse capacity for systems with GILD. The collapse evaluations in Table 1 indicate that the coupled shear wall case with α=0.4, exhibits an unacceptable collapse performance when subduction ground motions are considered. This emphasizes the vulnerability of systems with GILD in regions susceptible to long duration subduction ground motions.

Table 1. Summary of key collapse evaluation results for different cases

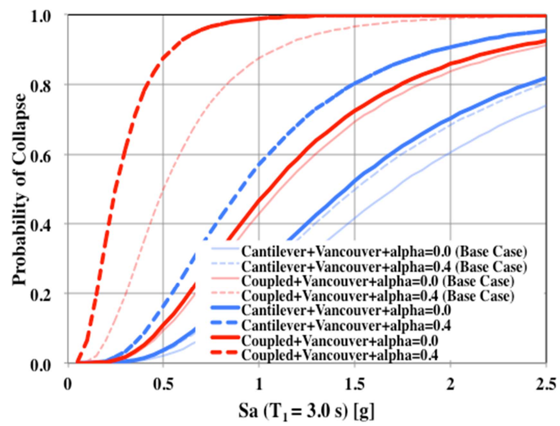
SFRS	Gravity System PLD (α)	Key Collapse Evaluation Results			
		ACMR	SSF-Adjusted Collapse Fragility		Probability of Collapse
			$\hat{S}_{CFT}(T)$ [g]	β_{TOT}	
Base Case					
Cantilever	0	10.00	1.70	0.6	0.0%
Cantilever	0.4	8.82	1.50	0.6	0.0%
Coupled	0	6.47	1.10	0.6	0.1%
Coupled	0.4	2.94	0.50	0.6	3.6%
Subduction GMs					
Cantilever	0	8.53	1.45	0.60	0.0%
Cantilever	0.4	5.29	0.90	0.60	0.3%
Coupled	0	6.18	1.05	0.60	0.1%
Coupled	0.4	1.47	0.25	0.60	26.0%
Mid-Way Constant Moment					
Cantilever	0	10.00	1.70	0.60	0.0%
Cantilever	0.4	6.43	1.09	0.60	0.1%
Coupled	0	6.47	1.10	0.60	0.1%
Coupled	0.4	1.76	0.30	0.60	17.2%

Gravity System Irregularity

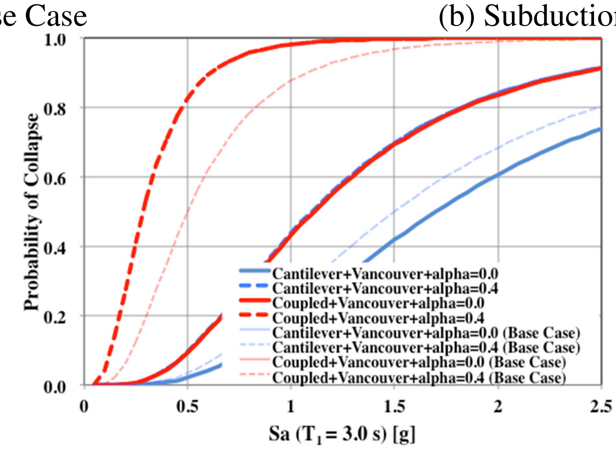
In the third study, eccentric floor spans was considered as the irregularity in the gravity system. The comparison to the original case is shown in Figure 4c. As seen in this figure, the eccentric floor span irregular system has a higher probability of collapse. This is consistent with the results obtained from the previous section. Similar to the subduction ground motion case, the coupled shear wall case with α=0.4, exhibits an unacceptable collapse performance.



(a) Base Case



(b) Subduction ground motions



(c) Gravity system irregularity - eccentric floor spans

Figure 4. Collapse fragilities for different cases (the thinner lines in b and c present the collapse fragilities for the base cases in a).

Conclusions

This study has demonstrated that gravity-induced lateral demands (GILDs) acting on a SFRS due to irregularities in the gravity system can lead to a ratcheting of displacement demands and potentially collapse. Considering cantilevered and coupled shear walls, the study has demonstrated that systems with a flag-shaped hysteretic response (e.g. tall cantilevered walls with high axial loads) are less prone to ratcheting compared with systems with the more typical fat hysteresis usually considered to be desirable for energy dissipation. Four different case studies demonstrated that subduction records with numerous cycles can result in greater amplifications in displacement demands compared to the original case, especially for coupled shear wall buildings. Gravity systems inducing a mid-height constant moment and eccentric floor plans imposing gravity loads on one side of the core wall can also lead to high amplifications in displacement demands. These results were considered over a range of building heights and strength reduction factors at the MCE (2%/50yr) hazard level. These conclusions are also supported by the results of incremental dynamic analysis displaying an increase in collapse probability for systems with GILDs This limited collapse study emphasizes the need for further consideration of the impacts of GILDs on the seismic response of structures. In particular, a wider variety of structural systems (e.g. moment frames) designed for different seismic hazards should be considered in future studies.

Acknowledgments

The authors would like to gratefully recognize funding by the Natural Science and Research Council of Canada to make this study possible. The authors thank members of the Standing Committee on Earthquake Design and practicing engineers in Vancouver for their valuable input.

References

1. National Building Code of Canada (NBCC), 2010. Issued by the *Canadian Commission on Building and Fire Codes, National Research Council of Canada*, Ottawa, ON, 2010.
2. Canterbury Earthquake Royal Commission. *Final Report: The Performance of Christchurch CBD Buildings*, Volume 2, Christchurch, New Zealand, 2012.
3. Best T., Elwood K.J., and Anderson D. *Inclined Columns and how they Influence the DynamicResponse of Structures*. M.Eng project, University of British Columbia, Vancouver, BC, 2011.
4. Dupuis M., *Seismic Performance of Buildings with Permanent Lateral Demands*, University of British Columbia, Vancouver, BC, MASc thesis, 2012.
5. Dupuis M., Best T., Elwood K.J., and Anderson D.L. Seismic Performance of Buildings with Gravity-induced lateral demands on SFRS, submitted to *Can. J. of Civil Eng*., 2013.
6. Open System for Earthquake Engineering Simulation (OpenSees). OpenSees v2.2.0 & v2.3.1, University of California, Berkeley, CA, 2012.
7. Applied Technology Council (ATC). Modeling and acceptance criteria for seismic design and analysis of tall buildings, Report No. ATC-72-1, Redwood City, CA, 2010.
8. Pugh, J. S. *Numerical simulation of walls and seismic design recommendations for walled buildings*, University of Washington, Seattle, WA, Ph.D. dissertation, 2012.
9. Kunnath, S.K., Heo, Y. and Mohle, J.F. Nonlinear Uniaxial Material Model for Reinforcing Steel Bars. *Journal of Structural Engineering*, ASCE, 134 (4), 333-343, 2009.
10. National Institute of Standards and Technology (NIST). *Applicability of nonlinear multiple-degree-of-freedom modeling for design*. Report NIST GCR 10-917-9, prepared by the NEHRP Consultants Joint Venture, Gaithersburg, 2010.
11. American Concrete Institute. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*. American Concrete Institute, Farmington Hills, MI, 2011.
12. FEMA. *Quantification of Building Seismic Performance Factors* (FEMA P695 Report), prepared by Applied Technology Council for the Federal Emergency Management Agency. Washington, D.C.: FEMA, 2009.
13. Bebamzadeh, A., Ventura, C.E., Pandey, B.H., & Finn, L.W.D. Design Drift Limits for Performance-based Seismic Assesment and Retrofit Design of School Buildings in British Columbia, Canada. Proceedings of the15th World Conference on Earthquake Engineering, Lisbon, Portugal, 2012.
14. Canadian Standards Association. Design of Concrete Structures: CAN/CSA-A23. 3-04: a National Standard of Canada (approved July 2007),(reaffirmed 2010). Canadian Standard Association, 2010.