



Investigating The Seismic Response of Structural Walls Using Nonlinear Static and Incremental Dynamic Analyses

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ABSTRACT

Structural walls commonly used as efficient structural elements to resist lateral and vertical loads. Diverse performance of bearing wall system in past earthquakes, motivates investigation on the adequacy of current seismic design provision for these walls. This study considers seismic performance of model walls of bearing wall and building frame systems designed as ordinary and special structural walls. Performance of the model walls are evaluated through static pushover and incremental dynamic analyses. Results show superior performance of the bearing wall system, which is in odd with small response modification factor given in the current design codes.

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NOMENCLATURE

A_f	Floor area	R	Response modification factor
A_w	Wall area	t	thikness
E_c	concrete modulus of elasticity	V_d	Design base shear force
E_s	steel modulus of elasticity	V_{max}	Maximum base shear force
f_c	concrete compression strength	Greek Symbols	
f_y	bending reinforcement yeild stress	Ω_0	Over strength factor
f_{ys}	shear reinforcement yeild stress	ρ	rebar percentage

1. INTRODUCTION

Reinforced concrete structural walls are frequently used in multistory buildings to resist the lateral and vertical loads. Building codes (e.g. ASCE 7 [1]) considers diverse response for structural walls in the bearing wall system and building frame system. The main geometrical difference between these structural systems is the ratio of the wall area to floor area. This ratio for the bearing wall system is about 5-6%, while for the building frame system is about 1-2%. In fact while

structural walls in the building frame system is only part of lateral force resisting system, in the bearing wall system they are also part of vertical force resisting system.

After good performance of bearing walls in 1985 Chile earthquake, research on the seismic performance of these structural elements is increased. Wallace and Mohle [2] investigated the required design and detailing requirements for these walls. They concluded that for symmetrically reinforced walls with rectangular cross section, no confinement is required and confinement requirment of the ACI is generally quite conservative. The February 27th 2010 Maule earthquake measured with surface magnitude of 8.8 caused wide spread

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damage in the bearing walls. Damages includes but not limited to crushing and spalling of concrete and buckling of vertical reinforcement [3]. This under-performance again attracted attention of researchers to the seismic performance of the bearing wall systems (common in Chile) especially in comparison with the building frame system which is more common in the United States.

Paulay [4] noting that the yield displacement of structural walls is insensitive to the reinforcement area, showed that it is possible to distribute different fraction of lateral loads to different elements with disregard for element stiffness, where calculations is accompanied with large errors in the element stiffness estimation.

Dashti and Dhakal [5] simulated seismic performance of structural walls designed using NZS:3101-2006 [6], ACI-318-11 [7] and EC8 [8]. They found that smaller confinement length requirement in EC8 is accompanied with substantial reduction in curvature and displacement ductility of the walls designed by this code, while the performance of the walls designed according to ACI-318 and NZS:3101 seem satisfactory. This sensitivity of the performance to the confinement is in odd with the findings of Wallace and Moehle [2].

Islam and Saito [9] developing an estimate of confinement requirements in the structural walls, proposed a method for seismic retrofit of the structural walls using carbon fiber sheets.

Dhakal et al. [10] developing different approach for modeling of shear walls, found that commonly accepted hypothesis of plane sections remain plane after deformation, is not applicable for large drifts. They also noted that floor-slab interaction with structural wall and shear-flexure interaction could have large effect on the response.

Seo et al. [11] developing seismic fragility curves for drift limits associated with different performance levels of FEMA 356 [12], compared the seismic performance of structural walls as evaluated using FEMA 356 and Los Angeles Tall Building Structural Design Council approaches. They find that the first and second floors are the most vulnerable stories. They also reported up to 54% underestimation of the drifts as evaluated by response spectrum method in comparison with nonlinear response history method.

Creagh et al. [13] experimentally investigated the effect of loading history on the response of the shear wall boundary elements. They concluded that pre-tensioning due to cyclic loading, which leads to yielding of reinforcing bars could have devastating effect on the cyclic response of the member.

Parra and Mohle [14] investigated the lateral buckling mechanism of structural walls. They concluded that crushing of concrete rather than

softening due to tensile loading triggers the lateral buckling of the shear wall.

Wallace [15] reviewing observed damages of structural walls in Chile 2010 and New Zealand 2011 pointed out some sources for this design inadequacies. Possible reasons for this inferior performance include, underestimating the need for confinement in boundary elements (especially in nonsymmetric walls), splice of longitudinal bars leading to smaller plastic hinge length, brittle failure of structural walls in compression controlled state and buckling of longitudinal reinforcements.

Hagen [16] used ASCE 41-06 [17] analysis methods to study the performance of a 6 stories office building with lateral load resisting system comprised of slender structural walls. He found satisfactory performance meeting the basic safety objective of ASCE 41-06.

Gogus and Wallace [18] investigated the seismic safety of reinforced concrete walls adopting FEMA P695 [19] methodology. They considered 20 special and 20 ordinary archetypes designed according to ASCE 7 and ACI 318 requirements. They evaluated adjusted collapse margin ratio for these archetypes using incremental dynamic analysis method. They concluded that for archetype with height to length ratio larger than 3, the response modification coefficients in ASCE 7 could be increased. Their results also indicate that the increase in the axial load of the structural wall does not have meaningful impact on the seismic safety of the archetypes. They also reported good displacement ductility even for walls with high axial load (with axial stress on the order of $0.2f_c$) [20]. These findings show good seismic safety of the structural walls and the possibility for increase in their response modification factor.

Tiong et al. enhanced seismic performance of low ductility precast walls employing base isolation [21]. They reported low efficiency of base isolation in improving seismic behavior of these types of structures.

Tiong et al. [21] developed a model for seismic performance of lap spliced columns and studied the effect of lap splice on the seismic performance of reinforced concrete frame structures. They concluded that lap splice significantly increases the probability of exceeding collapse limit state.

This paper compares the seismic safety of the structural walls designed as part of the bearing wall and building frame systems. The next section describes the measures adopted for modeling seismic response of the walls. Then the efficiency of the approach adopted for nonlinear modeling is verified against available experimental results. Finally the seismic performance of eight structural walls designed as part of different lateral load resisting systems are evaluated using static pushover and incremental dynamic analyses.

2. MODELING

Analyses are done employing OpenSees [22] finite element program, which provides an open source platform with extensive material and element library. Shear walls are modelled using fiber beam-column element nonlinearBeamColumn in OpenSees element library. Using enough number of Gauss points along member length, this element is able to capture spread of plasticity along member length. Fiber section discretization of the wall section employs concrete01 uniaxial material property for concrete and Steel02 uniaxial material property for reinforcing bars. These material properties are numerically inexpensive and suitable for studying the nonlinear behavior in structures with extensive cracking.

There are more sophisticated models for concrete confinement in the literature, however as noted by Wallace and Moehle [2], it is anticipated that confinement will not have large impact on the response of structural walls with symmetrical geometry and reinforcement arrangement. For this reason, simpler modified Kent-Park method is used to account for confinement. Considering aspect ratio of the model walls, flexure controls the response and there is no possibility of shear failure. Existing elements in the OpenSees library are not able to account for the flexure-shear interaction in cyclic loading. For this reason and considering also the aspect ratio of the model walls, this study ignores flexure-shear interaction.

3. VERIFICATION

To verify nonlinear modeling approach, wall RW2 from experimental tests of Thomsen and Wallace is used [23]. Figure 1 gives geometry, reinforcement and loading arrangement of the specimen. Lateral load on the specimen is applied via a single concentrated load at top of the specimen. Stiff pedestal (Figure 1) which is used to anchor the specimen to the strong floor also acts as fixed foundation for the wall. The wall is a one-quarter scale of actual wall which is loaded as cantilever beam with constant axial load of about $0.07f_cA_g$.

Table 1 give material properties used in OpenSees simulations. For this simulation to achieve better approximation of actual response, cracking in concrete is modeled employing Concrete02 material property. nonlinearBeamColumn in OpenSees element library is used and the wall is simulated using 4 elements and each element with 5 Gauss points.

Figure 2 compares the experimental results with the analytical model. As could be seen, there is a good correlation in terms of strength and pinching between hysteretic response of the specimen and analytical simulation.

4. MODEL WALLS

Structural walls are assumed to be 10 stories with typical length and story height of 4 and 3.5 m. The walls are designed to comply with seismic design category C requirements of ASCE/SEI 7-10 and ACI 318-14. Design is done using uniform and lumped bars arrangements. The walls designed as special or ordinary structural wall as part of two seismic force resisting systems: the building frame system (BFS) and the bearing wall system (BWS).

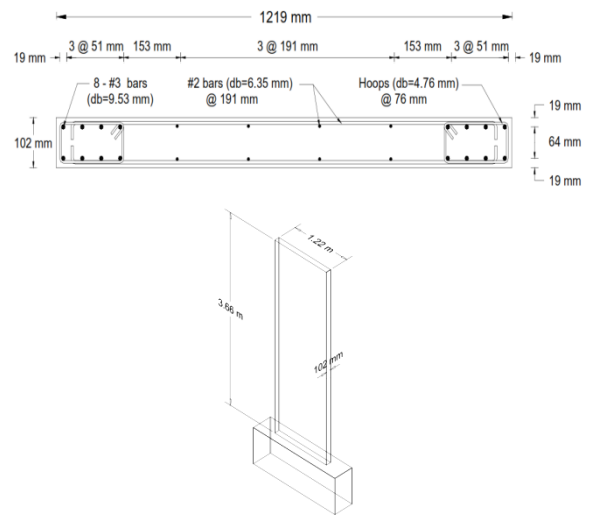


Figure 1. Section details of analytical RW2 [25]

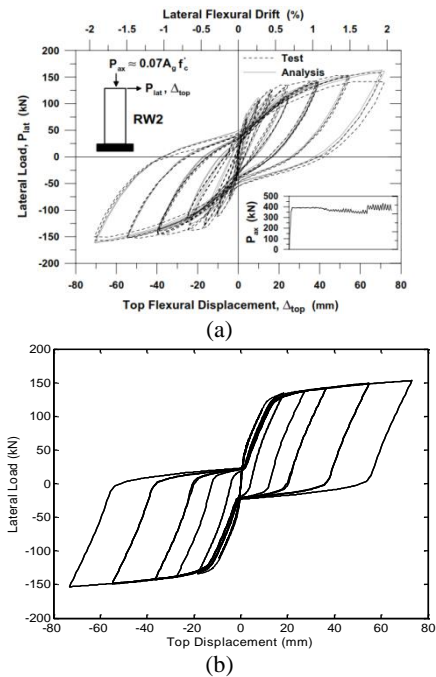


Figure 2. Comparison between experimental and numerical results, a) experimental results [23], b) numerical analysis (this study).

TABLE 1. Numerical values of the parameters used in OpenSees model for verification

Material	Parameter	RW2	
		Boundary (confined)	Web (Unconfined)
Concrete in Compression	f_c (MPa)	47.6	42.8
	f_t (MPa)	2.0	2.0
	ϵ_0	0.0033	0.0021
	E_c (GPa)	31.0	31.0
	$\epsilon_{cracking}$	0.00008	0.00008
#3 Rebar in Tension	σ_y (MPa)	395.0	
	E_0 (GPa)	200.0	
	$E_0/E_{str. hard.}$	0.0185	
#2 Rebar in Tension	σ_y (MPa)		336
	E_0 (GPa)		200
	$E_0/E_{str. hard.}$		0.0350

Specifications of the walls including wall designation, seismic and gravitational tributary area, design response modification coefficient and base shear are tabulated in Table 2. Material properties given in Table 3. Assumed uniform dead and live loads are 6 and 2 KN/m², respectively. Design information for each model wall are presented in Table 4. Each wall element is simulated using 10 elements and each element with 5 Gauss points.

5. RESULTS

5.1. Nonlinear Static Analysis To investigate the nonlinear behavior of the structural walls, pushover analysis is used with an inverted-triangular lateral load

pattern. Figure 3 shows the results of pushover analyses for the model walls. As could be seen, for the walls with similar design ductility and seismic force resisting system, cases with the lumped distribution of longitudinal reinforcement have slightly larger ductility than those with uniform distribution.

Table 5 also gives displacement ductility evaluated for each model as ratio of drift at descending branch corresponding to 15 percent reduction in strength to yield displacement. As it is evident, changing the distribution of the reinforcements from lumped to uniform has largest effect in the walls designed as BFS, with change of as much as 59 percent (from UOF to LOF), while the change in the displacement ductility for the walls designed as BWS is negligible.

Further examination of Figure 3 also shows that the model walls designed as BWS, generally provide poor ductility as compared to walls designed as BFS. Interesting point is that the model walls of BWS designed as special shear walls have smaller displacement ductility compared to the walls designed as ordinary wall. Both of these observations could be explained by considering the value of axial stress on the walls. Larger number of walls in the case of BWS and larger response modification factor for the walls designed as special structural wall, gives rise to smaller thickness of the wall and considering that all of the walls have nearly same vertical load, this leads to larger axial stress on the wall. As expected larger axial stress on the wall reduces the available ductility. Table 5 also gives target displacements for the model walls evaluated using ASCE/SEI 41 displacement method. Large stiffness of the walls resulted in relatively small target displacements and consequently small or even no strength deterioration is evident for displacements about structure's target displacement.

TABLE 2. Designation, seismic force resisting system, design base shear and response modification coefficients of the models. Notations: Building frame system (BFS), Bearing wall system (BWS), Ordinary shear wall (OSW), Special shear wall (SSW)

Designation	UOF	LOF	UOB	LOB	USF	LSF	USB	LSB
Bar arrangement	Uniform	Lumped	Uniform	Lumped	Uniform	Lumped	Uniform	Lumped
Building system	(BFS)	(BFS)	(BWS)	(BWS)	(BFS)	(BFS)	(BWS)	(BWS)
Resisting system	OSW	OSW	OSW	OSW	SSW	SSW	SSW	SSW
Grav. Trib. area (m ²)	30	30	30	30	30	30	30	30
Seis. Trib. area (m ²)	67.5	67.5	30	30	67.5	67.5	30	30
R	5	5	4	4	6	6	5	5
Vd (kN)	8919	8919	10124	10124	6614	6614	7597	7597

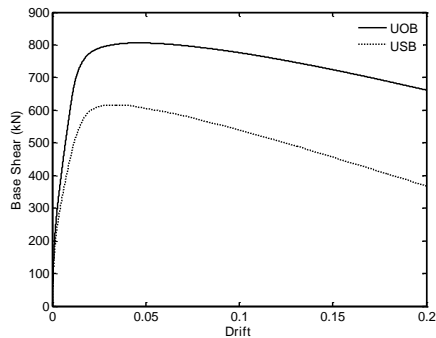
TABLE 3. Material properties considered in this study

f_c (MPa)	E_c (GPa)	f_y (MPa)	f_{ys} (MPa)	E_s (GPa)
28	26.46	400	300	199.99

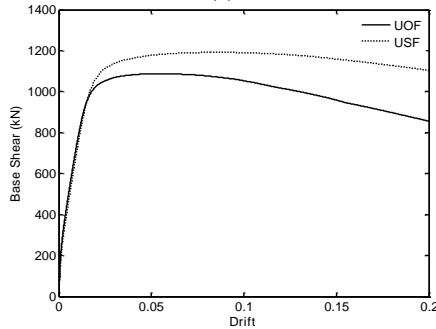
As anticipated design ductility (special or ordinary) of walls and bar arrangement has no significant effect on the calculated target displacement. In the BWS due to smaller seismic tributary area (larger stiffness) target displacement is smaller than that for BFS.

TABLE 4. Design information of the model walls

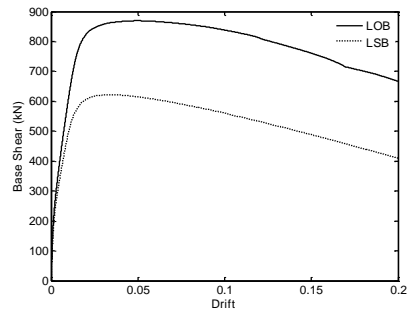
Designation	Story	t (m)	Total	$\rho\%$ boundary	web	Confined zone length (m)	$\frac{A_w}{A_f}$
UOF	1-4	0.35	1.39	-	-	-	0.021
	5-7	0.30	1.41	-	-	-	0.018
	8-10	0.25	1.49	-	-	-	0.015
LOF	1-4	0.35	1.74	2.99	0.99	-	0.021
	5-7	0.30	1.30	2.81	0.91	-	0.018
	8-10	0.25	1.35	3.48	1.07	-	0.015
UOB	1-4	0.30	1.71	-	-	-	0.044
	5-7	0.25	1.09	-	-	-	0.037
	8-10	0.20	1.04	-	-	-	0.030
LOB	1-4	0.30	1.32	2.53	0.67	-	0.044
	5-7	0.25	0.97	2.03	0.62	-	0.037
	8-10	0.20	0.93	2.01	0.56	-	0.030
USF	1-4	0.30	2.53	-	-	1.00	0.018
	5-7	0.25	1.61	-	-	0.70	0.015
	8-10	0.20	1.54	-	-	-	0.012
LSF	1-4	0.30	2.1	3.27	1.16	0.90	0.018
	5-7	0.25	1.52	2.51	1.09	0.60	0.015
	8-10	0.20	1.32	2.32	0.98	-	0.012
USB	1-4	0.25	1.25	-	-	0.90	0.037
	5-7	0.20	1.19	-	-	0.75	0.030
	8-10	0.15	1.17	-	-	-	0.022
LSB	1-4	0.25	1.11	2.04	0.62	0.70	0.037
	5-7	0.20	0.93	2.01	0.57	0.50	0.030
	8-10	0.15	0.83	2.05	0.52	-	0.022



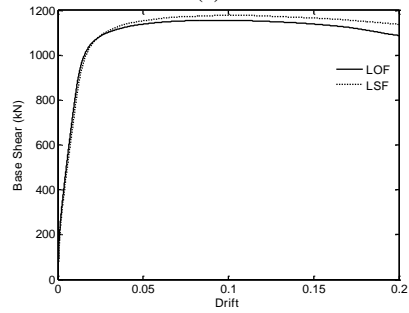
(a)



(b)



(c)



(d)

Figure 3. Results of pushover analyses for different model walls analysis

TABLE 5. Results of pushover analyses.

Designation	Drift at target displacement	V_{max} (kN)	Drift at $0.85V_{max}$	Displacement ductility	Ω_0
UOF	0.0031	1088	0.17	18.9	1.39
LOF	0.0030	1155	0.27	30.1	1.41
UOB	0.0021	806	0.16	17.8	1.59
LOB	0.0021	869	0.18	20.1	1.72
USF	0.0031	1177	0.26	29.0	1.42
LSF	0.0031	1192	0.33	36.8	1.44
USB	0.0024	616	0.11	12.3	1.62
LSB	0.0023	622	0.12	13.4	1.64

As could be seen, performance at target displacement will be essentially linear, which is an indication of design adequacy. As discussed in the previous paragraphs, while changing design ductility from ordinary to special, greatly improves ductility in the BFS, this is not the case for the BWS. On the other hand, the walls in BWS have larger over-strength values, which is mainly due to adoption of code's minimum dimension and reinforcement in the design.

Figure 4 depicts the effect of confinement on the post peak response of the different walls. As it is evident, confinement is able to improve post peak response of the shear walls in BFS, while it has no significant impact on the walls in BWS. This is mainly due to smaller confinement length (Table 4) and smaller wall thickness in the BWS. The resulting small area of confined concrete is unable to significantly affect the nonlinear response of the wall.

5. 2. Incremental Dynamic Analysis (IDA) To develop fragility curves, incremental dynamic analyses (IDA) are done on the model structures. Analyses are done for sixteen far field ground motions listed in Table 6. These ground motion records have been selected from the list of far field ground motions suggested by FEMA P695 [19]. As could be seen, for some events, the list includes two records.

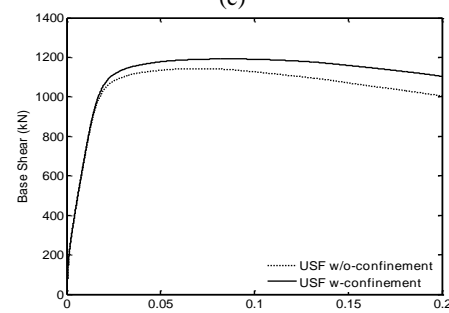
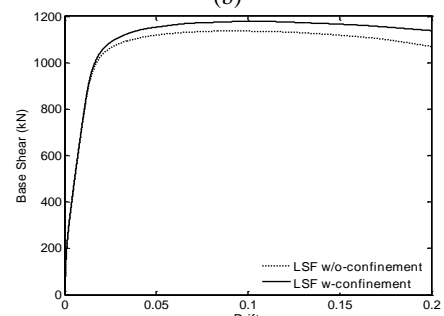
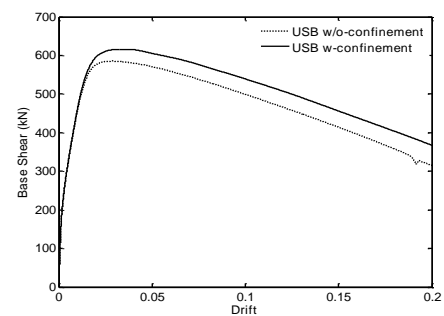
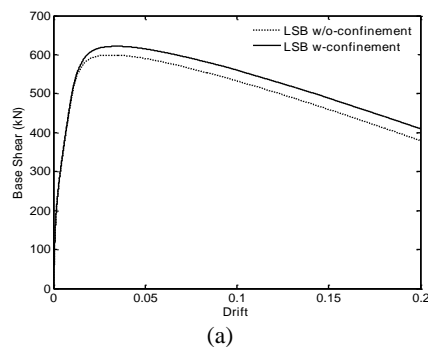


Figure 4. Effect of confinement for different model walls designed as special shear wall

Due to change in the record stations, in FEMA P695 these records are assumed as independent ones and consequently it is anticipated that their use will not introduce bias in the results.

TABLE 6. List of ground motion records used in IDA analyses

Earthquake			Recording station	Recorded motions	
M	Year	Name	Name	PGA _{max} (g)	PGV _{max} (cm/s)
6.7	1994	Northridge	Beverly Hills-Mulhol	0.52	63
7	1992	Cape Mendocino	Rio Dell OverPass	0.55	44
6.5	1979	Imperial Vally	Delta	0.35	33
6.5	1979	Imperial Vally	El Centro Array #11	0.38	42
6.9	1995	Kobe, Japan	Nishi-Akashi	0.51	37
6.9	1995	Kobe, Japan	Shin-Osaka	0.24	38
7.5	1999	Kocaeli, Turkey	Duzce	0.36	59
7.5	1999	Kocaeli, Turkey	Arcelik	0.22	40
7.3	1992	Landers	Yermo Fire Station	0.24	52
7.3	1992	Landers	Coolwater	0.42	42
6.9	1989	Loma Prieta	Capitola	0.53	35
6.5	1987	Superstition Hills	El Centro Imp. Co.	0.36	46
6.5	1987	Superstition Hills	Poe Road (temp)	0.45	36
7.6	1999	Chi-Chi, Taiwan	CHY101	0.44	115
6.6	1971	San Fernado	LA-Hollywood Stor	0.21	19
6.5	1976	Friuli, Italy	Tolmezzo	0.35	31

Figure 5 summarizes results of IDA in terms of fragility curves for life safety (LS) and collapse prevention (CP) limit states (due to limitation in space, results for Immediate Occupancy (IO) limit state are omitted). Drift corresponding to these limit states are assumed to be 0.5, 1 and 2 percent, respectively for IO, LS and CP limit states [17]. This figure shows meaningful increase in seismic safety due to change in the design ductility from ordinary to special structural wall in all of the model walls and limit states.

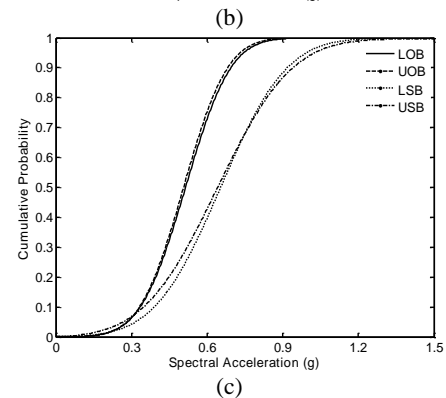
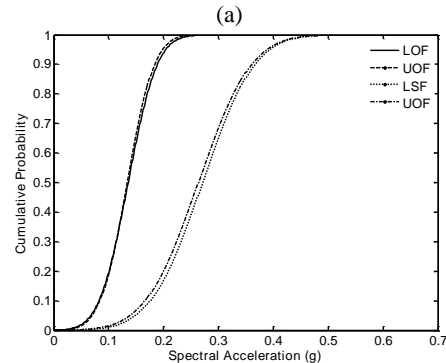
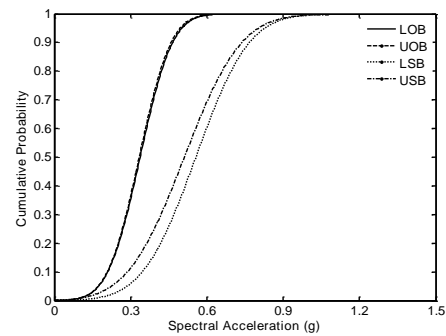
Table 7 gives 10 and 20 percent probability of failure for different model structures. As it is evident, in collapse prevention limit state, change in design ductility from ordinary to special results in about 50 percent increase in the corresponding spectral acceleration for the walls in the BFS and between 10 to 20 percent for the walls in the BWS. In the case of BWS this is in odd with the results of pushover analysis, where even there is reduction in the displacement ductility for changing design ductility from ordinary to special, as discussed in section 5.1.

Table 7 also shows that although walls in BFS gives higher displacement ductility, they have smaller spectral acceleration corresponding to 10 and 20 percent

probability of failure (in all of the limit states), as compared to the walls in BWS.

TABLE 7. 10 and 20 percent probability of failure for the model walls in IDA

Designation	IO		LS		IO	
	10%	20%	10%	20%	10%	20%
UOF	0.043	0.052	0.084	0.100	0.136	0.162
LOF	0.041	0.05	0.082	0.100	0.144	0.171
UOB	0.106	0.129	0.209	0.252	0.33	0.393
LOB	0.106	0.129	0.210	0.254	0.334	0.395
USF	0.085	0.100	0.166	0.200	0.199	0.244
LSF	0.087	0.103	0.176	0.209	0.211	0.26
USB	0.164	0.208	0.285	0.363	0.348	0.448
LSB	0.191	0.233	0.346	0.418	0.39	0.480



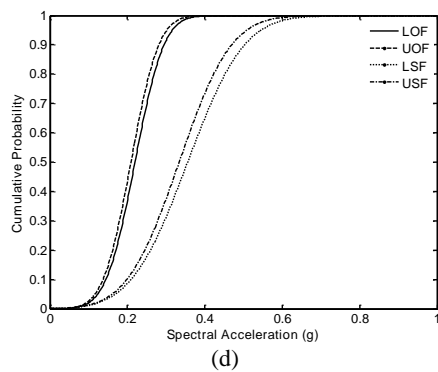


Figure 5. Fragility curves for the model walls, a) LS-BWS, b) LS-BFS, c) CP-BWS, d) CP-BFS

To have a better insight on the results of IDA analyses, Figure 6 gives the median response and its scatter in terms of spectral acceleration corresponding to 16%, median and 84% fractals. Again, better seismic safety of BWS is evident in the figure for all of the limit states.

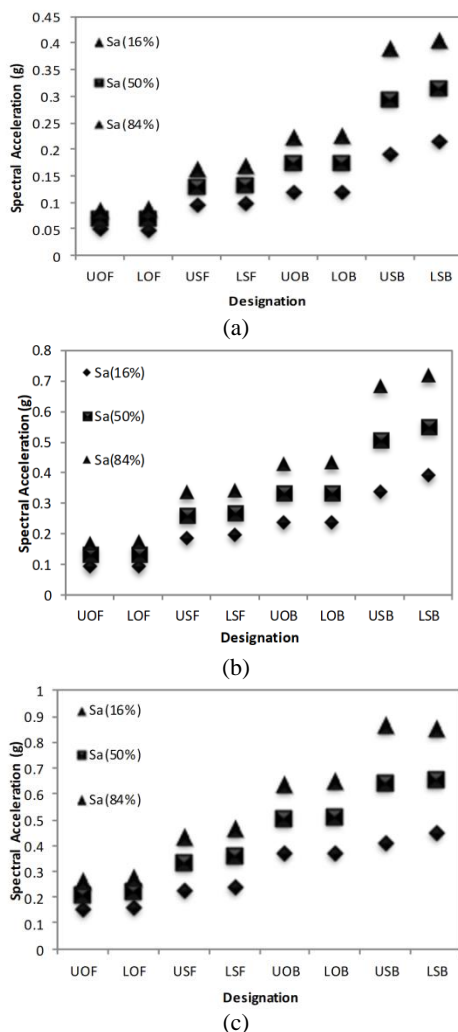


Figure 6. The spectral acceleration values for 16, 50 and 84 percent fractals for different limit states, a) IO, b) LS, c) CP

6. CONCLUSION

Seismic safety of structural walls as part of bearing wall system and building frame system designed as ordinary and special shear walls are investigated through static pushover and incremental dynamic analyses. Results confirm superior seismic safety of the bearing wall systems compared to the building frame system, which is in odd with current code prescribed response modification factor for these systems. Analyses also show that confinement have no significant impact on the response of the structural walls symmetric in reinforcement and geometry and thin in thickness.

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Investigating The Seismic Response of Structural Walls Using Nonlinear Static and Incremental Dynamic Analyses

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دیوارهای سازه‌ای معمولاً به عنوان اجزایی کارا برای بارهای جانبی و قائم مورد استفاده قرار می‌گیرند. عملکردهای متفاوت دیوارهای باربر در زلزله‌های گذشته، لزوم بررسی ایمنی دیوارهای طراحی شده براساس ضوابط آیین‌نامه‌های طراحی را در پی دارد. در این مطالعه عملکرد دیوارهای سیستم دیوار باربر و سیستم قاب ساختمانی طراحی شده براساس ضوابط شکل‌پذیری ویژه و معمولی بررسی شده است. عملکرد دیوارهای مورد مطالعه با استفاده از آنالیز استاتیکی و دینامیکی افزاینده مورد بررسی قرار گرفته است. نتایج، گویای عملکرد قابل قبول سیستم دیوار باربر است که با ضرایب رفتار پائین‌تر این سیستم در آیین‌نامه‌های طراحی هم‌خوانی ندارد.

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