PERFORMANCE-BASED SEISMIC ASSESSMENT OF BUILDINGS WITH CURVED SHEAR WALLS

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ABSTRACT: Nowadays, use of curvilinear architecture in tall buildings design is becoming increasingly important, so as structural engineers, we need to study these curved configurations from a structural point of view. This paper summarizes the results of a performance-based assessment of buildings with curved shear walls. The assessment was carried out using nonlinear static pushover analysis of Four (G+10 Typ.) concrete buildings which were divided into two categories. Each category included two buildings. The main parameter was the shape of the shear wall in plan and all the other parameters as the reinforcement ratio, the concrete volume, and position of shear walls were the same. A comparison was made between different plans with straight and curved shear walls. First, a modal analysis was carried out to determine the building's dynamic characteristics. Then an ASCE7-16-based [1] equivalent static procedure was performed to approach a fair design data based on ACI318-14 Code [2] and to determine the design base shear required by the ASCE7-16 [1]. After that the four buildings were subjected to a monotonic pushover loading in triangular vertical pattern using software PERFORM 3D [3]. The response modification factor, R, was then calculated based on the displacement ductility and the overstrength of the different buildings.

The results of the analysis have shown that buildings with curved shear walls have larger response modification factor and are more energy dissipating than buildings with straight shear walls.

KEYWORDS: Shear walls; Response spectrum; Pushover analysis; Response modification factor; Ductility; Overstrength.

1. INTRODUCTION

Structural design procedure begins by selecting a structural system adequate for the performance goals of strength, stiffness, and ductility within the constraints of architectural requirements. Alternative structural configurations should be discussed during the concept development stage [4]. The architectural trend of high rise buildings is moving towards the curved and aerodynamic form and geometry [5]. These curvilinear configurations should be studied from a structural point of view. This study provides an assessment and comparison between curved and straight shear walls which might help structural designers when they encounter curved structural forms.

The seismic assessment was carried out using pushover analysis in which the structure was subjected to a monotonically increasing lateral force pattern. The lateral force pattern can be uniform, triangular, based on code equivalent lateral forces, based on the fundamental mode of vibration, or any other vertical pattern; the triangular load pattern was used. The drive for the pushover analysis was to assess the anticipated performance of a structural system by approximately calculating its strength and deformation capacities for the different performance levels and then comparing these capacities with the earthquake demand [6]. Current seismic design codes philosophy mostly counts on the energy dissipated through inelastic deformation of the structure and provide a reduced earthquake forces by a response modification factor, R, that accounts for strength and ductility of the structure.

In this study, four (G+10 Typ.) buildings with straight and curved shear walls were designed according to the provisions of ASCE7-16 [1] and ACI318-14 [2]. The four buildings were then subjected to a monotonically increasing triangular lateral load pattern and then the base shear-roof drift ratio curve was used to assess the energy dissipation capacity and to calculate the response modification factor.

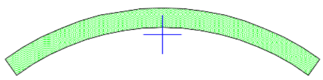
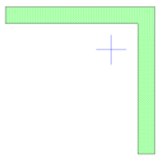
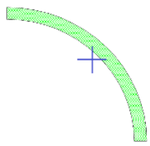
2. DISCRIPTION OF STUDIED BUILDINGS

Four buildings having the same plan dimensions (20m*20m) were considered. Those buildings had constant perimeter column module of 5m. the perimeter columns sections (65cm*65cm) and the slab thickness (22cm) are constant for all four buildings. Figure 1 shows the structural plans of the considered buildings. The studied parameter was the shape of the shear wall, so there are four shear wall sections corresponding to the four buildings. The shear walls geometry and design longitudinal reinforcement ratio are grouped in Table 1. The reinforcement is assumed to be uniformly distributed across the section.

3.MOMENT-CURVATURE RELATIONSHIPS FOR WALLS

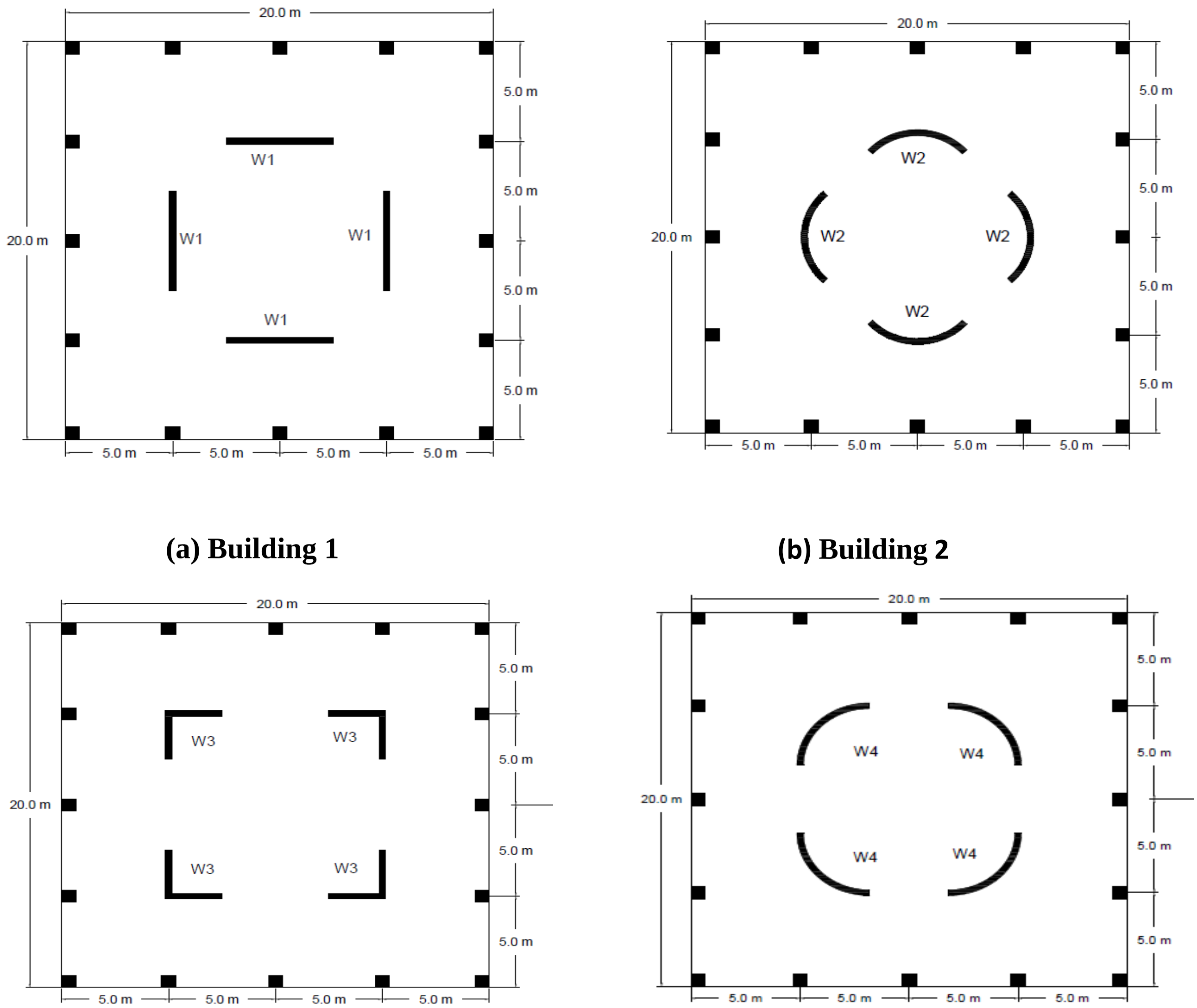
In order to preliminarily assess the strength and ductility of the shear wall elements, a moment-curvature analysis was made at two levels of axial load; 5% and 10% of ultimate axial load capacity of the walls. The analysis was made for moment directions similar to directions in which the shear walls are bent in the four buildings. The CSI, Computers and Structures Incorporation, Section Designer utility was implemented for the development of such curves. Figure 2 shows the moment-curvature curves of the studied walls. W1-5% means Wall W1 with an axial load level 5% of ultimate axial load capacity. The moment direction referred to in the moment-curvature is measured from the positive x-axis of the cross section. The cross section axes are shown in blue in the in-plan shape of the wall shown in Table 1. Theoretical moment-curvature relationships for reinforced concrete sections, showing the available flexural strength and ductility, can be acquired

Table 1 Geometry and reinforcement ratio of the studied shear walls

Wall	Geometry				μ^* (constant for all stories)
	Length (Centerline) (m)	Shape in plan	Radius (m)	Thickness (mm)	
W1	5.0		∞	500	3%
W2	5.0		3	500	3%
W3	5.0 (2.5 m for each wing)		∞	500	3.2%
W4	5.0 (Quarter of a circle)		3.18	500	3.2%

* Longitudinal Reinforcement ratio

provided that the stress-strain relationships for the concrete and reinforcing bars are identified and assuming the classical bending theory assumption that plane sections remain plane after bending. The moments and curvatures associated with increasing flexural deformations of the member may be calculated for a predetermined level of axial load by incrementing the curvature and satisfying the requirements of strain compatibility and equilibrium of forces [7]. Moment curvature analysis develops the curvatures related to a range of moments for a reinforced concrete cross section based on the principles of strain compatibility and equilibrium of forces[8]. The bending moment-curvature relationships of reinforced concrete sections are the basis for nonlinear seismic analysis of reinforced concrete structures. Curvature indicates the gradient of the strain distribution and varies with neutral axis position and the concrete and steel strains. The analysis procedure to get the moment-curvature relationship for a given level of axial load starts with incrementing the strain in the extreme compression fiber, $\varepsilon_{cm,}$ and then adjusting the neutral axis depth, c, until equilibrium equations are satisfied. After that the moment and curvature can be calculated from equilibrium and compatibility conditions respectively[9].



(a) Building 1

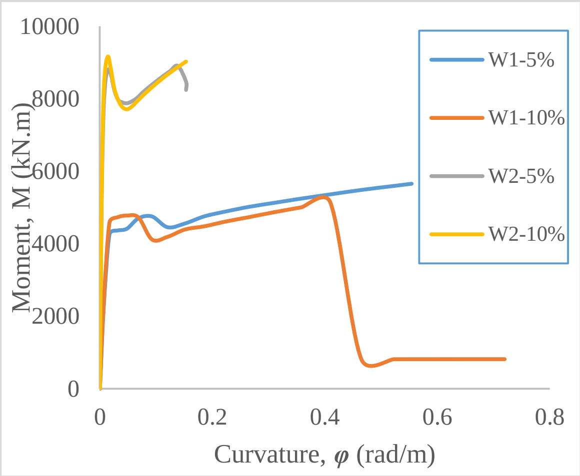
(b) Building 2

(c) Building 3

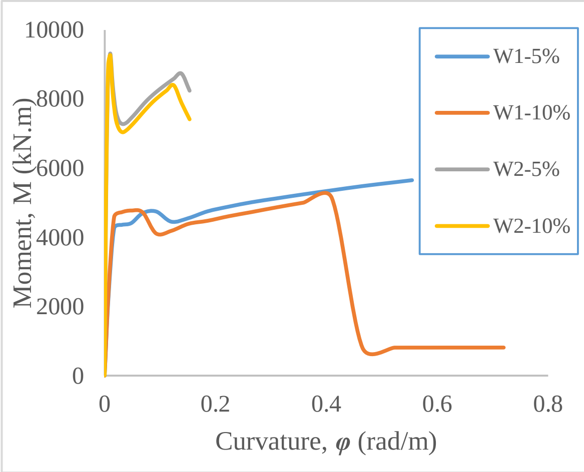
(d) Building 4

Figure 1 Structural plans of the studied buildings

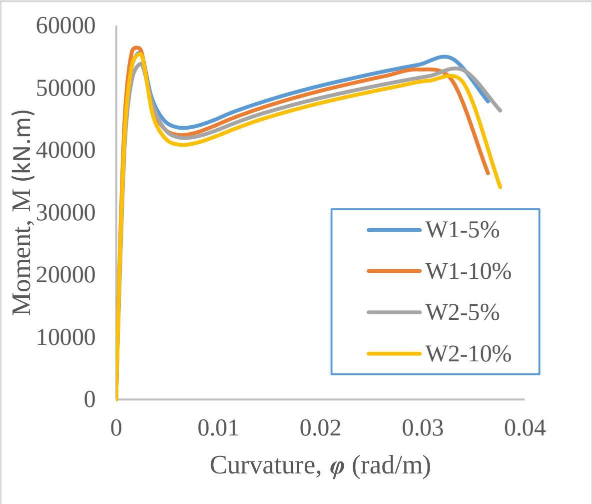
As can be concluded from Figure 2, curved shear walls in building 2 showed higher moment capacity than straight walls in building 1 when summing all moment capacities of individual walls in the same orientation as the shear walls are oriented within the building. In the same analogy, it can be shown that shear wall 4 showed higher moment capacity than shear wall 3.



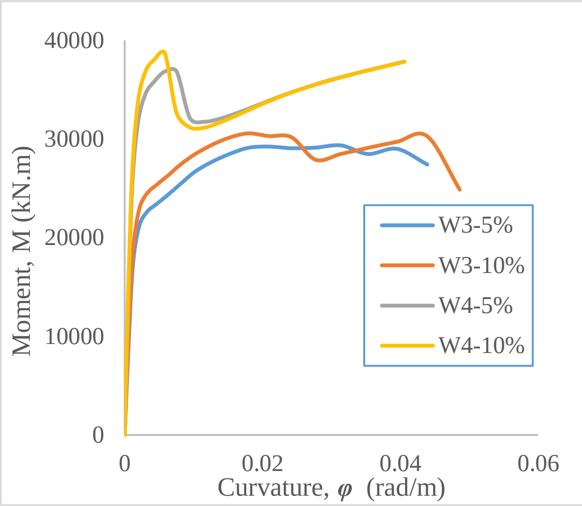
(a) M-ϕ curves (moment angle = 90^o)



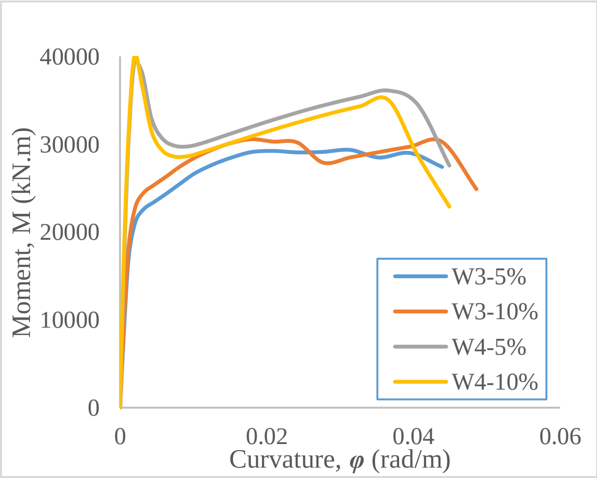
(b) M-ϕ curves (moment angle = -90^o)



(c) M-ϕ curves (moment angle = 0^o)



(d) M-ϕ curves (moment angle = 90^o)



(e) M-ϕ Curves (moment angle=0^o)

Figure 2 Moment-curvature relationships of studied buildings' walls

4. DESIGN OF BUILDINGS ACCORDING TO ASCE7-16 [1] AND ACI318-14 [2]

The seismic loads were calculated according ASCE7-16. Site class D and a location of the building in California, USA at latitude 38.123^0 and longitude of 121.123^0 was assumed. Then United Stated Geological Survey maps (USGS maps) through the web-based utility was used to generate the design response spectrum and to get the parameters used in static equivalent lateral force procedure. These parameters were: short period spectral Acceleration S_s = 0.634 g; long period spectral acceleration S_1 = 0.272 g; and the fundamental time period $T_a = 0.0488(h_n)^{0.75}$ = 0.687 second. The lateral load resisting system is shear walls, hence the response modification factor R = 5, the displacement amplification factor C_d = 4.5, and the overstrength factor Ω_o = 2.5. Occupancy category I, dead load = 1.5 kN/m^2, live load = 3 kN/m^2 and a mass source of dead load plus 0.25 times of the live load were used in the design procedure. Figure 3 shows the design response spectrum developed by the USGS web-based maps.

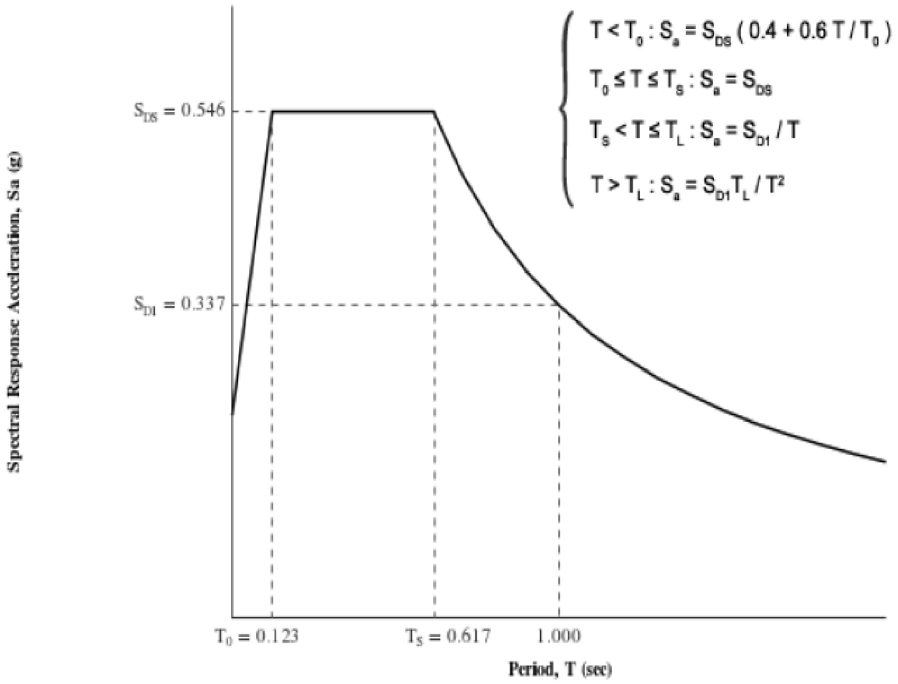


Figure 3 Design response spectrum developed by USGS online application

Using the static equivalent lateral load pattern developed according to ASCE7-16 [1]. CSI ETABS software was used to analyze and design the considered buildings' shear walls. It is common practice to neglect the lateral strength of columns in the design of shear wall buildings due to the complex modelling requirements of the slab-to-column connection. The columns were assumed pinned to the slabs, so the shear walls carry all lateral loads. The buildings with straight walls were only designed and the design reinforcement ratios for shear walls were put in the curved shear walls. The design longitudinal reinforcement ratios along with the thicknesses of walls and their geometry are shown in Table 1. The linear elastic storey displacements of the four buildings are shown in Figure 4. Also a modal analysis was performed to compare the modal time periods of the buildings. Table 2 shows the ASCE7-16 [1] and ETABS periods of the considered structures.

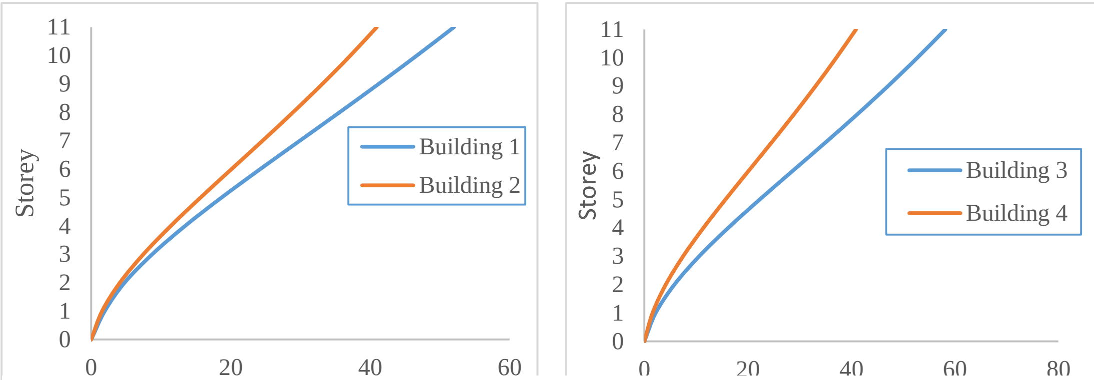


Figure 4 Storey drifts of Studied buildings

Table 2 Code and ETABS periods of the buildings

Building	Code Period ASCE7-16, second	ETABS Period, second		
		Mode 1	Mode 2	Mode 3
Building 1	0.687	0.994	0.994	0.944
Building 2	0.687	0.888	0.888	0.847
Building 3	0.687	1.064	1.064	0.941
Building 4	0.687	0.89	0.89	0.82

5. PERFORM 3D MATHEMATICAL MODEL

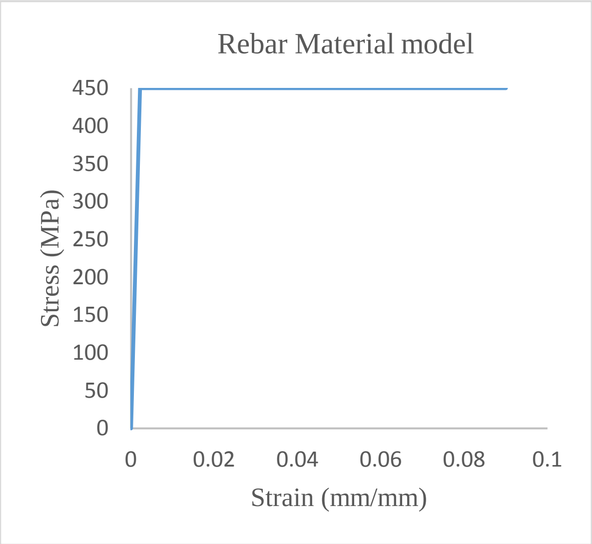
PERFORM 3D [3] is a nonlinear analysis and performance-based assessment finite element software developed by Professor Graham Powell; Professor of Emeritus University of California at Berkeley. Traditionally, seismic design was strength-based using linear elastic analysis and accounting for the inelastic behavior during a seismic event by using behavior factors as the response modification factor, R, in ASCE7 [1] standard and almost all today's seismic codes. Displacement based design is surely a better choice as it considers nonlinearity in material and geometry of a structure. Recently, displacement based design codes like ASCE41 [10] have been well developed. ASCE41 standard is mainly for seismic retrofit of existing structures but it can absolutely be used for design of new buildings. PERFORM 3D allows the usage of the principles of displacement based design and ASCE41.

Shear walls were modelled in PERFORM 3D using shear wall element. Shear wall element in PERFORM 3D is not a typical finite element, it is more likely to be an engineering element. It is a fiber based element which models reinforced concrete sections as discrete fibers of concrete and reinforcing bars. It has three uncoupled components: [1] axial and bending along strong axis which can be elastic or inelastic; [2] axial and bending along weak axis, this is always elastic but it can be modelled as inelastic using other category of elements called general wall elements; and [3] shear component which can be elastic or inelastic. The columns were modelled as elastic segments with end moment releases for the reason that it is a typical engineering practice to neglect the lateral strength of columns and to depend only upon walls in resisting lateral loads and also the objective of this study is to study the effect of changing the shape of the shear wall which is not affected as long as all the buildings have all other parameters fixed.

The adopted material models for unconfined concrete is the Mander[7] unconfined concrete model idealized to meet PERFORM 3D input requirements as shown in Figure 5 Tension strength of concrete is neglected. The Adopted reinforcing steel material is elastic perfectly plastic as shown in Figure 5.

6.PUSHOVER ANALYSIS

The four buildings were subjected to a monotonically increasing triangular load pattern with displacement control increments until the first concrete or rebar fiber



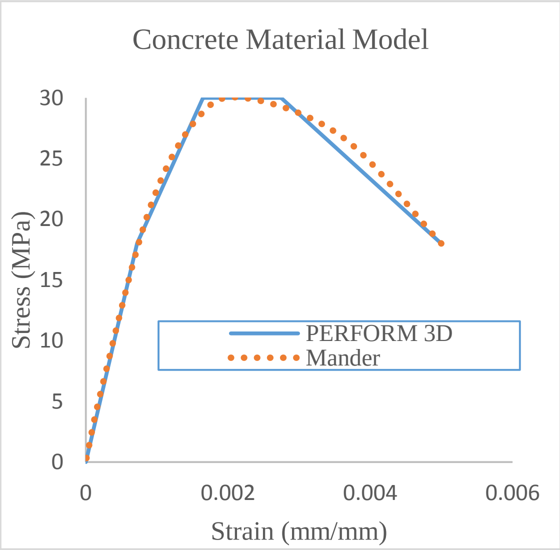


Figure 5 Concrete and rebar material uniaxial stress-strain curves

full softening behavior of the whole structure, however for design purposes, this has little or no importance. The result of the pushover analysis was the capacity curve which is a relationship between the base shear and the roof drift ratio. The capacity curve was then approximated to a bilinear idealization based on two criteria: the first is equal area under both the capacity and idealized curves; the second is the minimization of the area between the capacity curve and the idealized curve. The aim of this idealization was identifying a yield point in order to calculate the overall displacement ductility of the buildings. Figure 6 shows the capacity and bilinear idealization curves along with the equivalent static design base shear. The area under the base shear-roof drift ratio (capacity curve) shown in Figure 7 may be used as a measure for comparing the overall structure's dissipating energy capacity.

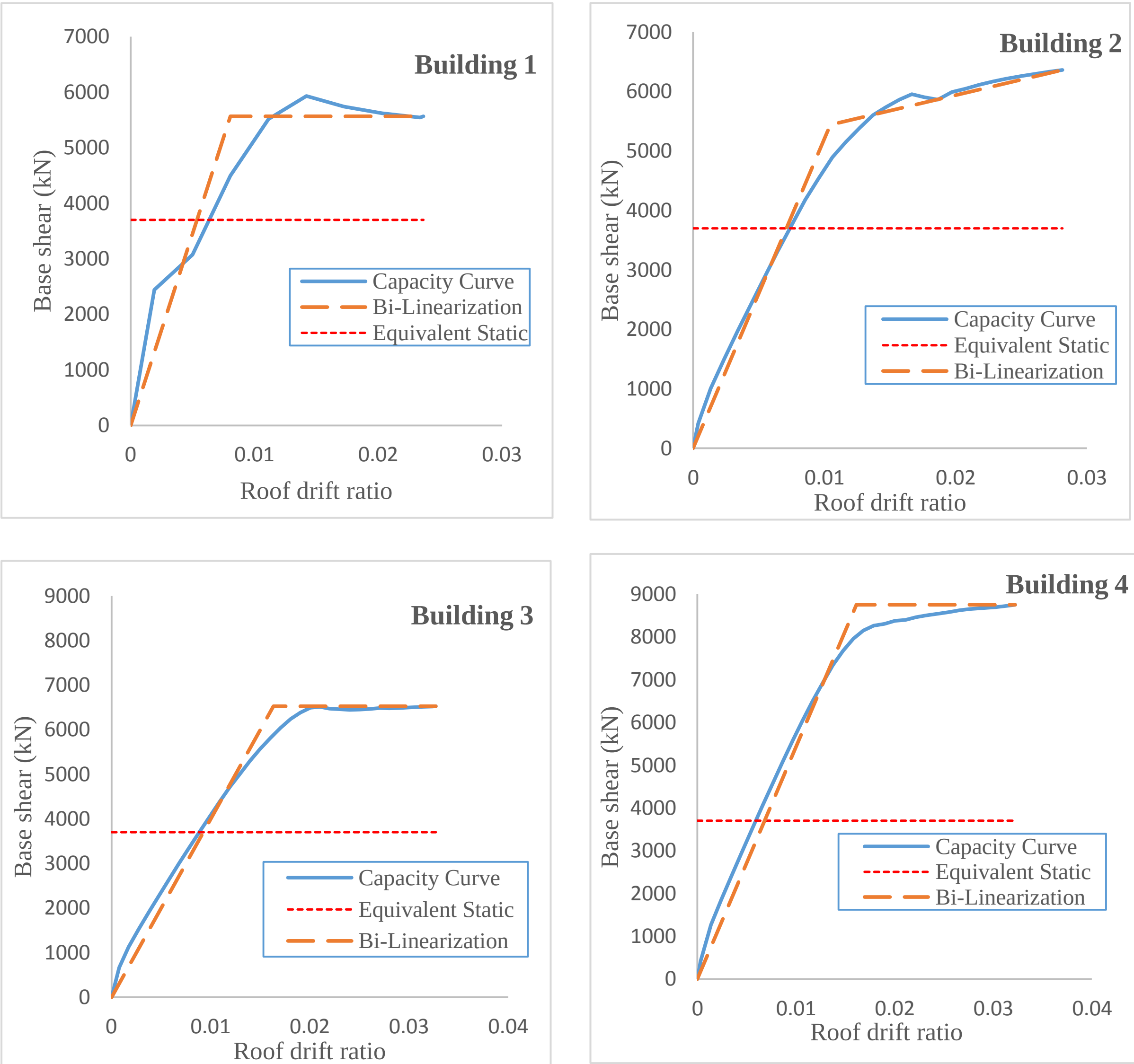


Figure 6 Pushover Analysis Results

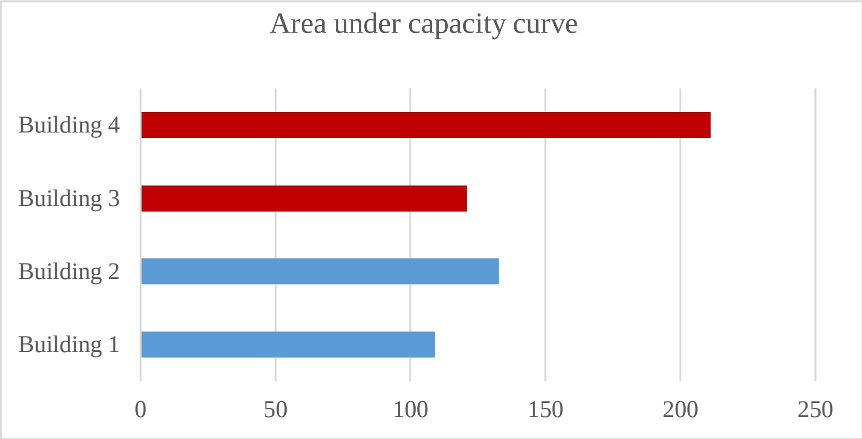


Figure 7 Area Under Capacity Curves

7. RESPONSE MODIFICATION FACTOR

Almost all design codes and standards today use the response modification factors to account for the inelastic deformations which contribute to energy dissipation in a structure. Linear elastic analysis and force-based design remains the basis of seismic design in spite of the growing momentum in using displacement-based design which provides a measure of structural damage, so evaluation of the response modification factors is vital[11]. Many research work investigated the various components of the response modification factor, R. The formulation given in Equation (1) is adopted [11].

$$\mathbf{R = R_s R_\mu R_R} \qquad \mathbf{(1)}$$

Where R_s is the overstrength factor; R_μ is the ductility factor; R_R is the redundancy factor. A fourth factor which accounts for the viscous damping may be added to account for the energy dissipated by damping devices. Figure 8 shows the definition of the design base shear, $\boldsymbol{V_d}$, ultimate base shear, $\boldsymbol{V_u}$, ultimate displacement parameter, $\Delta_{\boldsymbol{u}}$**,** and yield displacement parameter, $\Delta_{\boldsymbol{y}}$.

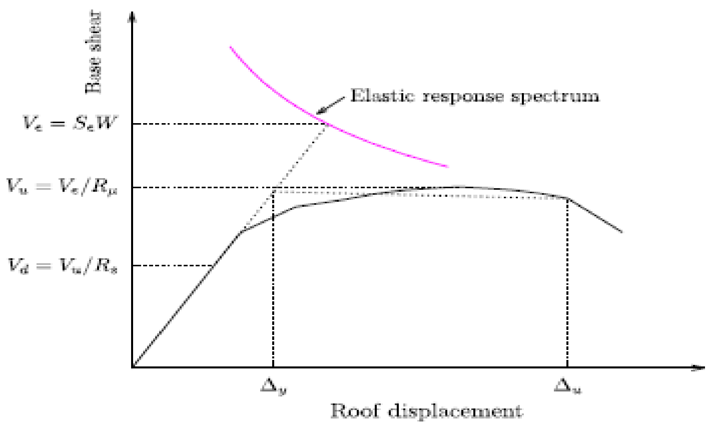


Figure 8 Capacity curve and Bi-linearization [12]

7.1 Overstrength factor, R_s

The overstrength factor accounts for the reserve strength in a structure due to several reasons: the strength reduction factors; strain hardening in materials; load factors; minimum dimensions and actual rebar sizes; and the conservatism of code method in determining the seismic loads. It is the ratio between the ultimate base shear capacity of the structure, V_u, to the base shear that was used in design, V_d, that was calculated using the provisions of ASCE7-16 [1].

$$R_s = \frac{V_u}{V_d} \qquad (2)$$

$$V_d = C_s W \qquad (3)$$

7.2 Ductility factor, R_μ

The ductility factor measures the post yield displacement capacity of the structure. It measures the overall nonlinear behavior of a structure, due to the hysteretic dissipated energy [13]. The ductility factor, in the medium and long period zones is marginally reliant on the period, and is approximately equal to the displacement ductility. However, in the short-period region, the ductility factor depends on the period[14]. R_μ also depends on the post yield hardening ratio. Many proposals have been made for the ductility factor. The proposal made by Krawinkler and Nassar[15] is adopted. The ductility factor, R_μ, can be calculated as follows:

$$R_\mu = [c(\mu - 1) + 1]^{1/c} \qquad (4)$$

where μ is the displacement ductility of the overall response of the structure represented usually by the relationship between base shear and roof drift or drift ratio. It is the ratio between the ultimate displacement parameter and the yield displacement parameter. The displacement is expressed in this work as the building's drift ratio. If Δ_u is the ultimate drift ratio and Δ_y is the yield drift ratio, then the displacement ductility, μ, can be expressed as:

$$\mu = \frac{\Delta_u}{\Delta_y} \qquad (5)$$

The c factor considers the dependence of R_μ on the period and post yield hardening ratio, α.

$$c(T, \alpha) = \frac{T^a}{1+T^a} + \frac{b}{T} \qquad (6)$$

a and b are regression parameters that depend on the post yield hardening ratio, α. It ranges from 0% to 10%. For α = 0%, a = 1.0, b = 0.42. For α = 10%, a = 0.8, b = 0.29.

7.3 Redundancy factor, R_R

A redundant seismic system is composed of multiple lines of resisting frames. It is highly encouraged to have a redundant system as it increases the ductility of the system prevents fast brittle collapse of the structure. It should be noted that redundancy is not studied in this paper and the redundancy factor is taken equal to 1.0. It is also very important to notice that the studied structures are simple and have small number of lateral load resisting framing lines which may results in somehow underestimating the response modification factor. However, it is not the purpose of this paper to evaluate the response modification factors provided in the codes but to give a comparison between the planar and curved walls with respect to ductility and overstrength.

Table 3 shows the parameters used in the evaluation of the ductility factor and the overstrength factor and hence the response modification factor. The values of response modification factor using Krawinkler and Nassar [15] approach is illustrated in Figure 9.

Table 3 Design base shear, ultimate base shear, ultimate drift ratio, yield drift ratio, and displacement ductility of studied buildings

	V_d (kN)	V_u (kN)	Δ_u (Ultimate Drift Ratio)	Δ_y (Yield drift ratio)	μ (Displacement ductility)
Building 1	3701	5567.025	0.023627	0.008059	2.931697
Building 2	3701	6362.224	0.0281395	0.0104518	2.692308
Building 3	3701	6526.266	0.03269	0.016309	2.00443
Building 4	3701	8750.35	0.032213	0.016139	1.995972

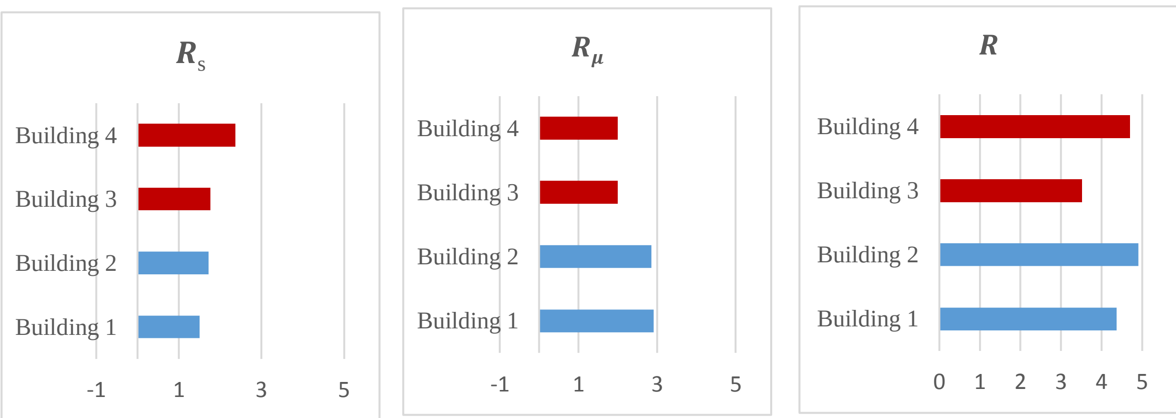


Figure 9 Overstrength Factor, Ductility factor and the response modification factor

8. CONCLUSIONS

Based on this study, the following conclusions can be drawn:

a. Moment-curvature analysis represents a good insight into the nonlinear behavior of slender shear walls in high rise buildings.
b. The code-based response modification factor may need to be re-evaluated based on shape and in-plan distribution of shear walls.
c. Based on the values of the response modification factor, curved shear walls show larger energy dissipation capacity than straight shear walls.
d. Based on the linear elastic lateral displacement resulting from ASCE7-16 equivalent static procedure and on the building natural period, buildings with curved walls have higher stiffness than those with straight walls.
e. When architecturally possible, it is recommended for structural engineers to use curved, and curvilinear shear walls in high rise buildings as they provide higher stiffness and strength.

9. REFERENCES

[1] A. S. of C. Engineers, “Minimum design loads and associated criteria for buildings and other structures,” 2016.

[2] A. C. I. Committee, A. C. Institute, and I. O. for Standardization, “Building code requirements for structural concrete (ACI 318-14) and commentary,” 2014.

[3] I. . CSI (Computers and Structures, “Perform 3D, Nonlinear Analysis and Performance Assessment for 3D Structures User Guide, Version 7.0.0.” Computers and Structures, Inc. Berkeley, CA, USA, 2018.

[4] B. S. Taranath, *Reinforced concrete design of tall buildings*. CRC press, 2009.

[5] M. Alaghmandan, P. Bahrami, and M. Elnimeiri, “The Future Trend of Architectural Form and Structural System in High-Rise Buildings,” *Archit. Res.*, vol. 4, no. 3, pp. 55–62, 2014.

[6] H. Krawinkler and G. Seneviratna, “Pros and cons of a pushover analysis of seismic performance evaluation,” *Eng. Struct.*, vol. 20, no. 4–6, pp. 452–464, 1998.

[7] J. B. Mander, M. J. N. Priestley, and R. Park, “Theoretical stress-strain model for confined concrete,” *J. Struct. Eng.*, vol. 114, no. 8, pp. 1804–1826, 1988.

[8] C. D. of Transportation, *CALTRANS Seismic Design Criteria*, Version 1. 2013.

[9] R. (Robert) Park, *Reinforced concrete structures* . New York: Wiley, 1975.

[10] ASCE, *ASCE/SEI 41-13 : American Society of Civil Engineers : seismic evaluation and retrofit of existing buildings*. 2013.

[11] A. Whittaker, G. Hart, and C. Rojahn, "Seismic response modification factors," *J. Struct. Eng.*, vol. 125, no. 4, pp. 438–444, 1999.

[12] A. Mondal, S. Ghosh, and G. R. Reddy, "Performance-based evaluation of the response reduction factor for ductile RC frames," *Eng. Struct.*, vol. 56, pp. 1808–1819, 2013.

[13] M. Mahmoudi and M. Zaree, "Determination the response modification factors of buckling restrained braced frames," *Procedia Eng.*, vol. 54, pp. 222–231, 2013.

[14] T. Vidic, P. Fajfar, and M. Fischinger, "Consistent inelastic design spectra: strength and displacement," *Earthq. Eng. Struct. Dyn.*, vol. 23, no. 5, pp. 507–521, 1994.

[15] A. A. Nassar, J. D. Osteraas, and H. Krawinkler, "Seismic design based on strength and ductility demands," in *10th World Conference on Earthquake Engineering*, 1992, vol. 10, pp. 5861–5866.