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Evaluation of Seismic Collapse Safety of Concrete Frame Structures Reinforced with Shape Memory Alloy

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Abstract: Superelastic Shape Memory Alloys (SMAs) are gradually gaining attention for application in various industries like aircraft, telecommunication, medicine, construction etc. When used as reinforcement in structures like buildings or bridges, its distinctive ability to undergo large deformation and regain its original shape upon unloading or by heating helps to minimize the residual drift of that structure and help maintain serviceability even after a large earthquake. In this study the seismic performance of concrete frames reinforced with SMAs is being assessed in terms of Collapse Margin Ratio (CMR), which is the ratio of the ground motion intensity that causes median collapse, to the Maximum Considered Earthquake (MCE) ground motion intensity at the fundamental period of the building frame and then compared with the performance of concrete frames reinforced with steel rebar only. Collapse safety assessment is performed using nonlinear incremental dynamic analysis with pre-defined ground motions record set that are systematically scaled to increasing intensities until median collapse is established. For analysis, three different storeys (3, 6 and 8) of reinforced concrete (RC) buildings, each with two different reinforcement detailing are considered: (i) steel reinforcement (Steel) only and (ii) SMA rebar used in plastic hinge region of all beams and in ground floor columns (SMA-BM-CM). Results indicate significant increase in collapse capacity for low and high rise concrete frame structure while using SMA as reinforcement.

Keywords: superelastic shape memory alloy (SMA), seismic, incremental dynamic analysis (IDA), collapse safety, collapse margin ratio (CMR).

1. Introduction

The primary emphasis of different seismic design building code is to provide the building with adequate safety against collapse to protect the life of building inhabitants when subjected to a large earthquake. But actual collapse safety that can be achieved through current building codes is still not properly defined (Haselton et al. 2011), which requires large scale test up to collapse limit. Though the life safety of the inhabitants and prevention of collapse is ensured in the current building code practice, the reliability in achieving the objectives cannot be measured. Now a day's there is a trend among researchers to design the structures based on performance criteria that can be quantified (Ghobarah, 2001). In this study the performance of reinforced concrete building has been assessed in terms of Collapse Margin Ratio (CMR) according to FEMA P695 (2009) guidelines. CMR is the ratio of the 5% damped spectral acceleration of the collapse level ground motion, S_{CT} to the 5% damped spectral acceleration of Maximum Considered Earthquake (MCE) ground motions, S_{MT} at the fundamental period of the seismic force resisting system of interest (FEMA 2009). Collapse level ground motion refers to the intensity that would result in median collapse of the seismic force resisting system. Median collapse occurs when one half of the ground motion record sets used for analyzing caused the building to collapse (FEMA 2009). Three different storeys (3, 6 and 8) of reinforced concrete (RC) buildings, each with two different reinforcement detailing are considered for nonlinear analysis: (i) steel reinforcement (Steel) only and (ii) SMA rebar used in the plastic hinge region of all beams and only ground floor columns (SMA BM CM).

2. Research Significance

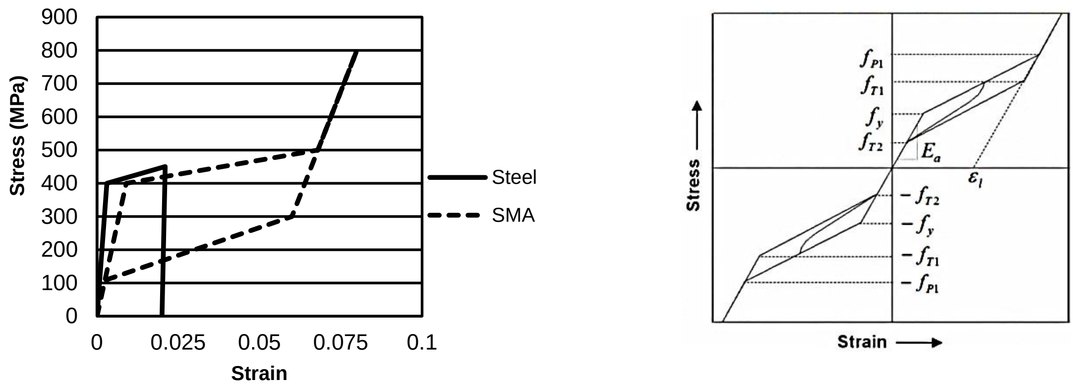
Recent experimental and analytical investigations have showed various uses of SMA in civil infrastructure for improving the performance in terms of residual drift, energy dissipation, and seismic isolation while subjected to an earthquake. (Dolce, Cardone et al. 2004, Wilde et al. 2000, Attanasi and Auricchio, 2011). Saiidi and Wang (2006) demonstrated an experiment on residual deformation of RC column reinforced with shape memory alloy in its plastic hinge region which results in almost complete recovery of post yield deformation. Youssef et al. (2007) and Alam et al. (2007, 2008) have investigated beam column joint reinforced with SMA in its plastic hinge region, which showed much better performance than the beam column joint reinforced with only mild steel in terms of residual displacement. Nehdi et al. (2010) also utilized SMA as reinforcement in beam-column joint whereas the other parts were reinforced with FRP rebar and tested under reversed cyclic loading where the SMA could regain all its plastic deformation. Saiidi et al. (2007) investigated the efficiency of RC beams reinforced with SMA. Experimental results showed greater energy dissipation capacity with less amount of permanent deformation when subjected to an earthquake. Alam et al. (2009) compared the performance of two eight-story RC frames reinforced with shape memory alloy in its plastic hinge region. The analysis showed that the concrete frames reinforced with SMA results in lesser residual drift compared to the frames with mild steel only. Zafar and Andrawes (2012) have performed incremental dynamic analysis on building frame reinforced with SMA-FRP hybrid rebar. Results showed that using SMA-FRP rebar in plastic hinge region increase the seismic performance in terms of energy dissipation, ductility and residual drift. For the last few decades many analytical and experimental investigation have been carried out to utilize the unique properties of SMA to control the drift of RC structure during earthquakes; however, little or no work has been directed towards the evaluation of SMA RC building collapse capacity. The impact of SMA in the plastic hinge region of beam or column or both on the collapse safety of a building is yet a question.

3. Overview of Methodology

Here six building frames have been analyzed and designed according to the National Building code of Canada (NBCC, 2005) considering the design spectrum of Vancouver. The building frames have been modeled in Seismostrut V6 (Seismostrut 2012), a finite element software. Plastic hinge model with rotational spring to account for slippage between SMA and steel has been considered for modeling the frame (Alam et al. 2008). Collapse assessment is performed by using nonlinear incremental dynamic analysis (IDA) on the building frames. A set of twenty predefined ground motion time histories has been used where each record has been scaled to the design spectrum for IDA. From IDA 5% damped spectral acceleration corresponding to collapse for each ground motion has been determined. Results from IDA have been used to plot a lognormal fragility curve to identify the median collapse level intensity or spectral acceleration S_{CT}. Ratio of median collapse intensity, S_{CT} to Maximum Considered Earthquake (MCE) demand S_{MT} is defined as collapse margin ratio (CMR) which has been used to assess the collapse safety of the building reinforced with mild steel and SMA.

4. Modeling of Superelastic Shape Memory Alloy

Among different compositions of SMA, Ni-Ti has been mostly used for civil infrastructure as well as in this study because of its ability to recover strain, superelasticity and extremely good resistance to corrosion (Alam et al. 2007). Figure 1 (a) depicts the idealized stress-strain behavior of superelastic SMA and mild steel under axial forces which shows that during unloading SMA experiences negligible residual deformation when it is within its superelastic strain range whereas steel experiences large residual deformation.



(a) (b)

Figure 1: (a) Stress-strain behavior of superelastic SMA and Steel, (b) 1D-Superelastic model of SMA incorporated in FE Packages Seismostruct (2012)

Here SMA has been used as longitudinal reinforcement in beam and column, so one-dimensional phenomenological models are considered for modeling the frame structures (Alam et al. 2012). Figure 1 (b) shows the 1D-superelastic model of SMA incorporated in FE Packages e.g. in Seismostruct V6 (Seismostrut 2012). Seven model-calibrating parameters that are required to fully describe the mechanical characteristics of SMA in SeismoStruct (2012) are the Modulus of elasticity, E_a; austenite-to-martensite starting stress, f_y ; austenite-to-martensite finishing stress, f_{P1}; martensite-to-austenite starting stress, f_{T1} ; martensite-to-austenite finishing stress, f_{T2}; superelastic plateau strain length, ε_1; and specific weight.

5. Nonlinear Model Development

In this paper buildings of three different number of storeys- 3, 6 and 8 each with two different types of reinforcement configuration- (i) steel reinforcement (Steel) only and (ii) SMA rebar used in plastic hinge region of all beams and only ground floor columns (SMA-BM-CM), have been analyzed according to National Building Code of Canada (NBCC 2005) and designed as per CSA A23.3-04 (CSA 2004) considering as moderately ductile moment resisting frames (Alam et al. 2012). The design base shear is calculated considering the building to be located in Vancouver on site class C. The overstrength factor R_o and ductility factor R_d is kept constant for all the building models. Table 1 shows the material properties used for the design and analysis of the buildings (Alam et al. 2012).

All the buildings are regular in geometric plan having 5 bays of 5m in both directions and the storey height is 3 m for all the buildings. From the flexural and shear stress of the structures the section size and amount of reinforcement are designed as per Canadian standards (CSA 2004). Tables 2 and 3 show the member sizes and the reinforcement detailing of beams and columns, respectively. The 20 columns located along the perimeter of the buildings are designated as C2, and the remaining interior 16 columns are designated as C1. In case of SMA-BM-CM model SMA bars have been used in the plastic hinge regions of the beam and ground floor column and mild steel reinforcements have been used in the remaining parts of the beams and columns. The plastic hinge length of the beam-column is estimated using an analytical expression proposed by Paulay and Priestley (1992).

[1] $l_p = 0.08L + 0.022 d_b f_y$

Where L is the length of the member, d_b represents the rebar diameter in mm and f_y is the yield strength of the rebar in MPa.

Table 1: Material properties used in the finite element program (Alam et al. 2012)

Material	Mechanical Property	Value
Concrete	Compressive strength (MPa)	35
	Tensile strength (MPa)	3.5
Steel	Strain at peak stress (%)	0.2
	Modulus of elasticity (MPa)	200,000
	Yield strength (MPa)	400
	Strain hardening parameter	0.5
SMA	Modulus of elasticity (MPa)	60,000
	Austenite to martensite starting stress (MPa)	400
	Austenite to martensite finishing stress (MPa)	500
	Martensite to Austenite starting stress (MPa)	300
	Martensite to Austenite finishing stress (MPa)	100
	Super elastic plateau strain length (%)	6

Table 2: Beam section and reinforcement detailing (Alam et al. 2012)

Storey Id.	Beam Id.	Size (mm x mm)	Section ID					
			Section 1-1		Section 2-2		Section 3-3	
			Top	Bottom	Top	Bottom	Top	Bottom
3-storey	B1	300x450	3-20M	3-20M	3-20M	3-20M	3-20M	3-20M
6-storey	B1	300x500	3-25M	5-20M	3-25M	5-20M	3-25M+2-20M	3-20M
	B2	300x500	3-20m	3-20M	3-20M	3-20M	3-20M	3-20M
8-storey	B1	300x500	3-25M	5-20M	3-25M	5-20M	3-25M+2-20M	3-20M
	B2	300x500	3-20M	3-20M	3-20M	3-20M	3-20M	3-20M

Figure 2 shows the reinforcement detailing of a typical beam. From eigenvalue analysis the fundamental period of each building frame has been determined. The design spectral acceleration, S (T) for individual fundamental period has been calculated from design spectrum of Vancouver (NBCC, 2005) which has been used to revise the design base shear and the number of reinforcement in beam and column. For the collapse simulation of the frame structure fibre modeling technique (Seismostrut 2012) has been used to incorporate the material inelasticity and geometrical nonlinearity. Concrete model has been defined by using the Mander et al. (1988) constitutive relationship and the cyclic response by Martinez-Rueda and Elnashai (1997), and for steel bilinear kinematic strain hardening model has been used. For defining SMA the model provided by Auricchio and Sacco (1997) has been used. The beam and column were divided longitudinally into 8 and 4 elements, respectively where two of the beam elements near beam column joint and one element of column near foundation represent the plastic hinge region. In plastic hinge region the SMA rebar are considered to be connected to mild steel with mechanical couplers (Alam et al. 2010). The bond slip relationship has been incorporated in the joint with a rotational spring (Alam et al. 2008).

Table 3: Column section and reinforcement detailing (Alam et al. 2012)

Storey ID	Floor level	Description	Column ID	
			C1	C2
3-Storey	Up to roof	Size (mm x mm)	375x375	300x300
		Main reinforcement	8-15M	4-20M
6 -Storey	Up to 3rd floor	Size (mm x mm)	450x450	300x300
		Main reinforcement	8-25M	6-20M
	3rd floor to roof	Size (mm x mm)	450x450	300x300
		Main reinforcement	8-20M	4-20M
8-Storey	Up to 3rd floor	Size (mm x mm)	500x500	300x300
		Main reinforcement	8-25M	6-25M
	3rd floor to roof	Size (mm x mm)	500x500	300x300
		Main reinforcement	6-25M	6-20M

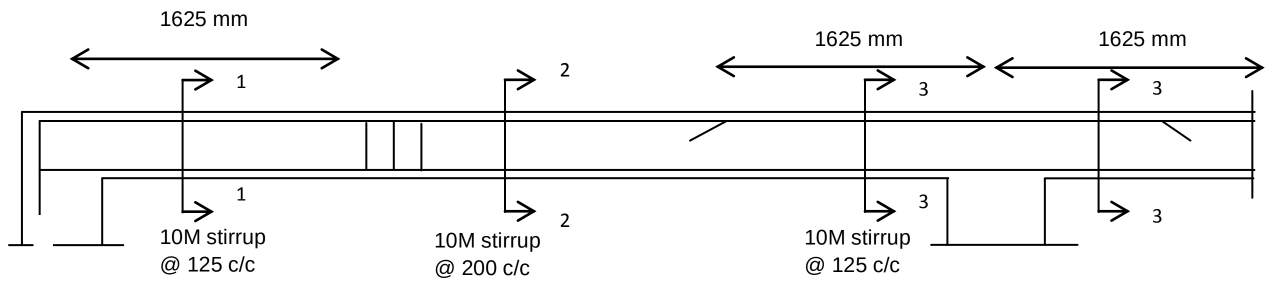


Figure 2: Typical longitudinal section of beam and reinforcement configuration

Table 4: Ensemble of ground motion records

Earthquake ID			Recording Station	Epicentral	PGA_{max}	PGV_{max}	PGV/PGA
M	Year	Name	Name	Distance (Km)	(g)	(cm/s.)	(sec)
6.7	1994	Northridge	Beverly Hills - Mulhol	13.3	0.42	58.95	0.145
7.3	1992	Landers	Yermo Fire Station	86	0.24	52	0.214
6.7	1994	Northridge	Canyon Country-WLC	26.5	0.41	42.97	0.107
7.3	1992	Landers	Coolwater	82.1	0.28	26	0.092
7.1	1999	Duzce, Turkey	Bolu	41.3	0.73	56.44	0.079
6.9	1989	Loma Prieta	Capitola	9.8	0.53	35	0.068
7.1	1999	Hector Mine	Hector	26.5	0.27	28.56	0.11
6.9	1989	Loma Prieta	Gilroy Array #3	31.4	0.56	36	0.066
6.5	1979	Imperial Valley	Delta	33.7	0.24	26	0.111
7.4	1990	Manjil, Iran	Abbar	40.4	0.51	43	0.084
6.5	1979	Imperial Valley	El Centro Array #11	29.4	0.36	34.44	0.096
6.5	1987	Superstition Hills	El Centro Imp. Co.	35.8	0.36	46	0.132
6.9	1995	Kobe, Japan	Nishi-Akashi	8.7	0.51	37.28	0.075
6.5	1987	Superstition Hills	Poe Road (temp)	11.2	0.45	36	0.082
6.9	1995	Kobe, Japan	Shin-Osaka	46	0.24	38	0.158
7.0	1992	Cape Mendocino	Rio Dell Overpass	22.7	0.39	44	0.116
7.5	1999	Kocaeli, Turkey	Duzce	98.2	0.31	59	0.192
7.6	1999	Chi-Chi, Taiwan	CHY101	32	0.35	71	0.204
7.5	1999	Kocaeli, Turkey	Arcelik	53.7	0.22	17.69	0.082
7.6	1999	Chi-Chi, Taiwan	TCU045	77.5	0.47	37	0.079

6. Nonlinear Dynamic Analysis

Incremental dynamic analysis of 6 frame structures have been performed using a set of 20 ground motion records (Table 4). Here, only far field ground motion data are used that are recorded at location greater than or equal to 10 km from the epicenter. All the ground motions are representatives of strong earthquake event having PGA>2.0 or of magnitude M > 6.5 which are collected from Pacific Earthquake Engineering Research Center Database (PEER 2007). Strong magnitude earthquakes with longer duration are chosen because they govern the collapse of the building and they will also shake the building for longer period, which is important for collapse safety evaluation (FEMA 2009). The record set is first scaled to the design response spectrum for Vancouver as per NBCC (2005) which is then systematically scaled to increasing intensity until the building collapse limit is reached. The scaled response spectra along with the code defined response spectra are shown in Figure 3.

Nonlinear dynamic time history analysis is performed for the entire building frames to calculate the median collapse capacity (Vamvatsikos and Cornell 2002). The median collapse occurs when half of the ground motion record set cause the building to collapse. The analysis result for each frame has been plotted in terms of 5% damped spectral acceleration, S_a versus the maximum interstory drift. Each single line in Figure 5 represents the change of interstory drift with increasing intensity of an individual ground motion. For assessing the collapse capacity, side sway collapse is defined as the point in IDA curve where the building frame become unstable and the interstory drift increases without any bound for a small increase in response spectral acceleration (Haselton et.al. 2011, ATC 2012). A large number of ground motions are selected for the analysis to measure the record to record variability which is later incorporated to develop the collapse fragility curve which relates the ground motion intensity to the probability of collapse (Ibarra et al. 2002). Results for incremental dynamic analysis of 6 building frames are shown in Figure 4.

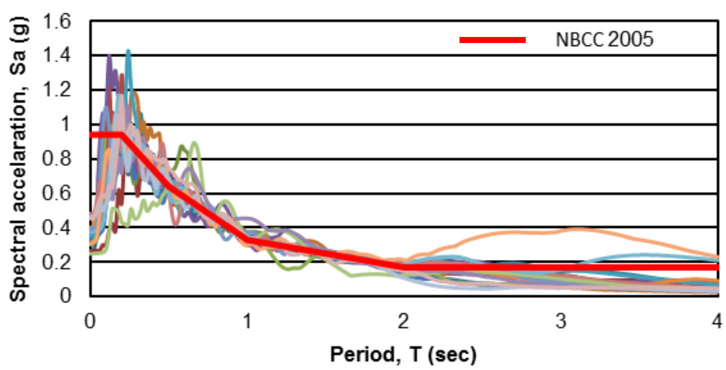


Figure 3: Scaled response spectrum for 20 ground motion record set

7. Calculation of Collapse Margin Ratio

From incremental dynamic analysis the collapse level intensity for each ground motion can be calculated considering the collapse limit state which is used to develop a collapse fragility curve that is defined by a cumulative distribution function. The Collapse fragility curve is used to define the median collapse capacity of the building frame which is actually the intensity of ground motion above which 50% of the ground motion data from record set will cause the building to collapse. The Collapse Margin Ratio (CMR) is calculated by dividing the median collapse level intensity or spectral acceleration (S_{CT}) by Maximum Considered Earthquake (MCE) intensity (S_{MT}) for the fundamental period of the building model.

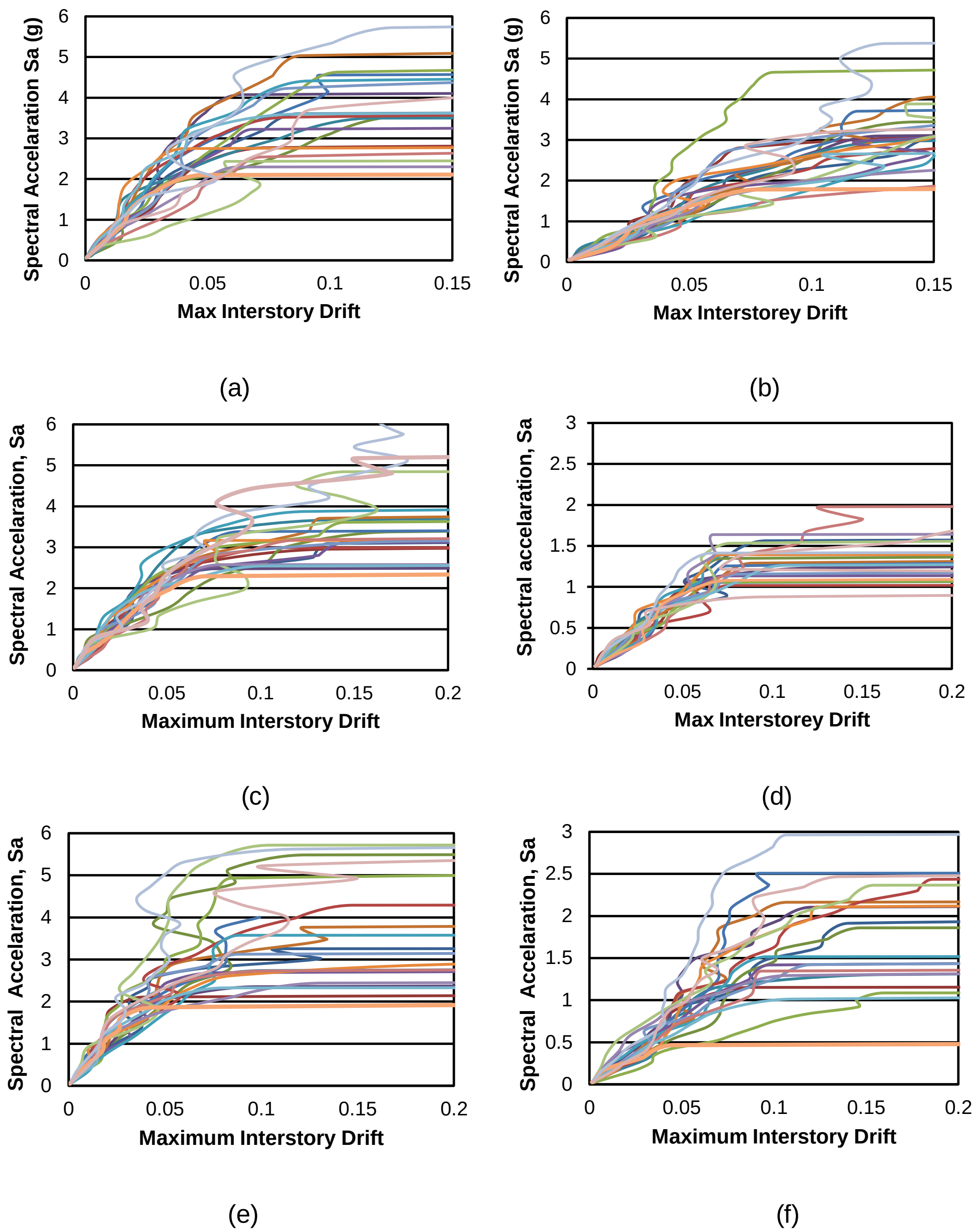


Figure 4: IDA response plot for 3 storey building frame (a) Steel, (b) SMA-BM-CM; for 6 storey building frame (c) Steel, (d) SMA-BM-CM; and for 8 storey building frame (e) Steel, (f) SMA-BM-CM

The two collapse fragility parameters are the median spectral acceleration and logarithm standard deviation or dispersion or record to record variability, β_{RTR} (ATC 2012). The median value of 5% damped spectral acceleration at collapse can be computed by arranging the collapse level spectral acceleration in ascending order and selecting the acceleration midway between the 10th and 11th values. The dispersion value, β_{RTR} is taken as a fixed value of 0.4 assuming the ductility factor of all the building frames greater than 3 (FEMA 2009) as it is not the governing factor among other uncertainty that affect the CMR value. The following fragility function for collapse expresses the conditional probability of exceeding the collapse capacity for a given level of ground motion intensity (Ibarra et.al.2002)

[2] $$F_{c,S_{a,c}}(x) = P\left[S_a \geq S_{a,c} \mid S_a = x\right] = P\left[S_{a,c} \leq x\right]$$

Where, $F_{c,S_{a,c}}$ represents the value on the fragility curve at any spectral acceleration, x. The collapse fragility curve is expressed as the probability that the capacity of the system ($S_{a,c}$) is less than or equal to the demand (S_a = x). Figure 5 shows the cumulative distribution plot for each building frame considered, that is obtained by fitting a lognormal distribution of the collapse level points from the incremental dynamic analysis.

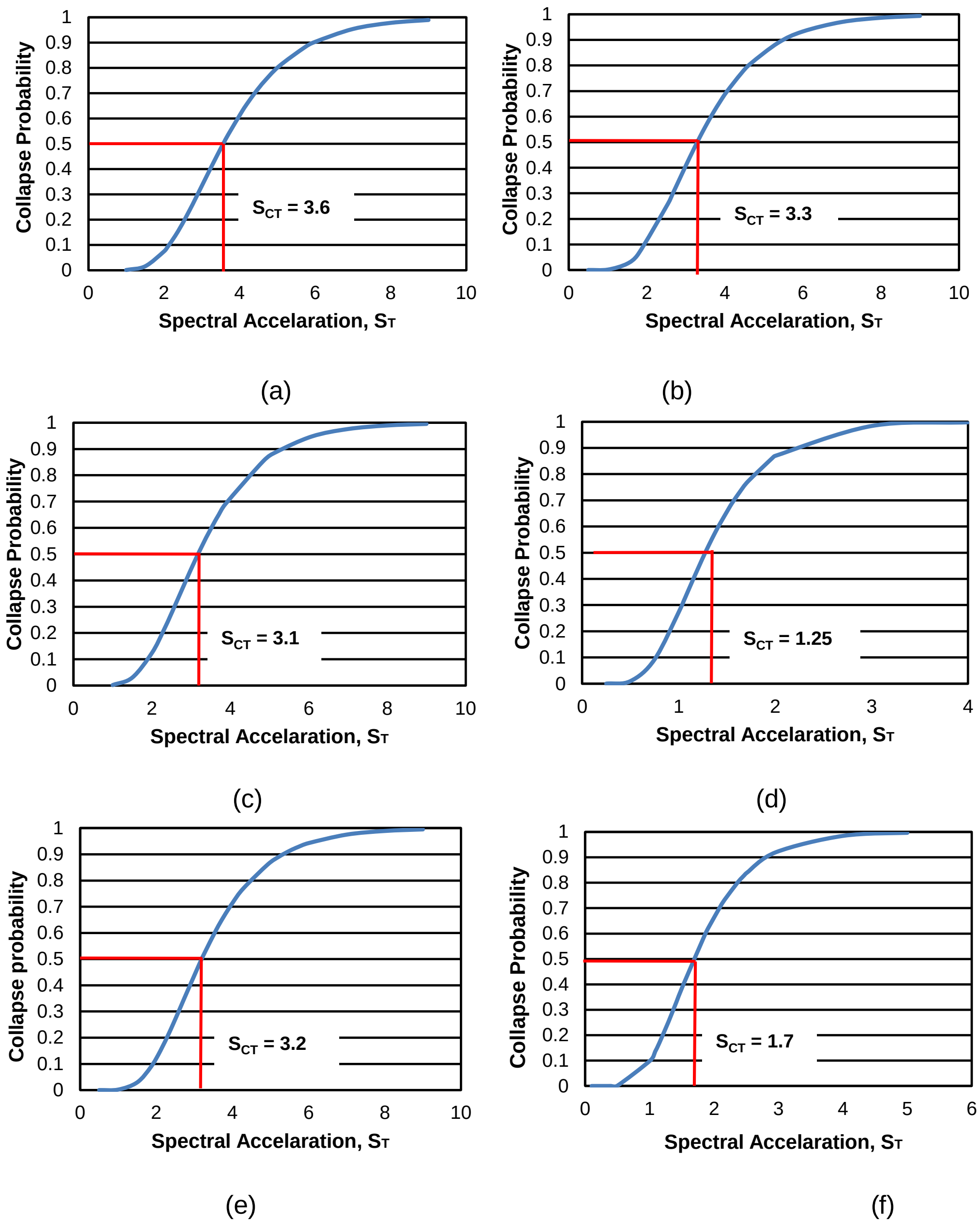


Figure 5: Collapse fragility curves for 3 storey building frame (a) Steel, (b) SMA-BM-CM; 6 storey building frame (c) Steel, (d) SMA-BM-CM; and 8 storey building frame (e) Steel, (f) SMA-BM-CM

8. Results and Discussion

Table 5 shows the summary of the calculated results. From the CMR value it can be concluded that for 3 and 8 storied buildings, the use of shape memory alloy in the plastic hinge region of all the beams and only ground floor columns has increased the collapse safety of the building considerably. Whereas for 6 story building frame collapse capacity has decreased while using SMA as reinforcement compared to building reinforced with mild steel only. The increase in collapse capacity for using SMA is 21.6% and 5% for 3 storied and 8 storied building, respectively compared to steel reinforced building. Unlikely, for 6 storied building frame collapse capacity decrease up to 23% for using SMA in the plastic hinge regions of beams and in ground floor columns. There is a trend of increase in collapse capacity with higher number of stories in case of regular concrete building. For the case of building reinforced with SMA the medium rise structures seem to have higher collapse risk than the regular one.

Table 5: Collapse Margin Ratio (CMR) for different frame structures

	3 Storey		6 Storey		8 Storey	
	Steel	SMA-BM-CM	Steel	SMA-BM-CM	Steel	SMA-BM-CM
Fundamental Period, T (sec)	0.33	0.54	0.51	0.99	0.65	1.32
Median Collapse Level, S_{CT}	3.6	3.3	3.1	1.25	3.2	1.7
MCE Demand, S_{MT}	1.22	0.92	0.95	0.5	0.83	0.42
Collapse Margin Ratio (CMR)	2.96	3.6	3.3	2.53	3.9	4.1

9. Conclusion

In this study, the performance of SMA reinforced concrete building frame structure has been judged in terms of Collapse Margin Ratio (CMR) which is a direct indication of the collapse capacity or collapse safety of the considered building type or proposed seismic force resisting system. Higher CMR value indicates safer building structures against collapse when subjected to an earthquake. In this paper, the CMR values for buildings with three different storeys (3, 6 and 8 storey) have been evaluated, where each frame had two different reinforcement configurations: (i) steel in all members, and (ii) SMA only in the plastic hinge region of beams and ground columns, and steel in other regions. In case of 3 and 8 storied buildings the collapse capacity increase significantly while using SMA in plastic hinge region of beams and ground floor columns with mild steel in other portions except the 6 storey frame. However, before making any generalized statement about the midrise building frames further studies should be carried out with more analysis of taller buildings. There are other factors that are responsible for changing the collapse capacity of the building such as the type of building frame used (e.g. space frame or perimeter frame), irregular bay spacing, and consideration of soil structure interaction, which is beyond the scope of this current paper. Future work will be carried out to understand the effect of using SMA on collapse capacity of building considering different frame types, possible structural irregularities, different design codes and soil structure interaction.

References

Alam, M.S., Moni, M. and Tesfamariam, S. 2012. Seismic overstrength and ductility of concrete buildings reinforced with superelastic shape memory alloy rebar. Engineering Structures, 34: 8-20

Alam, M.S., Youssef, M.A., Nehdi, M. 2010. Exploratory investigation on mechanical anchors for connecting SMA bars to steel or FRP bars. Materials and Structures, 43: 91-107.

Alam, M.S., Nehdi, M. Youssef, M.A. 2009. Seismic performance of concrete frame structures reinforced with superelastic shape memory alloys. Smart Structures and Systems, 5: 565-585.

Alam, M.S., Youssef, M.A. and Nehdi, M. 2008. Analytical prediction of the seismic behaviour of superelastic shape memory alloy reinforced concrete elements. Engineering Structures. 30: 3399–3411

Alam, M.S., Youssef, M.A. Nehdi, M. 2007. Seismic behaviour of concrete beam-column joints reinforced with superelastic shape memory alloys. *In* 9th Can Conf on Earthq Eng, June, Canada 2007, Paper no. 1125, 10

ATC, 2012. Development of next generation performance-based seismic design procedures for new and existing buildings, Report No. ATC-58, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

Attanasi, and Auricchio, F. 2011. Innovative superelastic isolation device, Journal of Earthquake Engineering; **15**(S1): 72-89.

Auricchio, F. and Sacco, E. 1997. A one-dimensional model for superelastic shape memory alloys with different elastic properties between austenite and martensite. International Journal of Non-Linear Mechanics, **32**(6): 1101–1114.

CSA A23.3-04 (2004). Design of Concrete Structures (Canadian Standards Association, Rexdale, Ontario, Canada).

Dolce, M. Cardone, D. Marnetto, R. Mucciarelli, M. Nigro, D. Ponzo, F.C. and Santarsiero, G. 2004. Experimental static and dynamic response of a real RC frame upgraded with SMA re-centering and dissipating braces. Proc. of the 13th World Conf. on Earthquake Engineering, Canada, Paper no. 2878.

FEMA P695. 2009. Quantification of building seismic performance factors. Federal Emergency Management Agency, Washington DC.

Ghobarah, A. 2001. Performance-based design in earthquake engineering: state of development. Engineering Structures, **23:** 878-884

Haselton, C.B. Liel, A.B. Deierlein, G.G. Dean, B.S. and Chou, J.H. 2011. Seismic collapse safety of reinforced concrete buildings I: assessment of ductile moment frames. Journal of Structural Engineering, **137**: 481-191.

Ibarra, L. Medina, R. and Krawinkler, H. 2002. Collapse assessment of deteriorating SDOF systems. Proceedings, 12th European Conference on Earthquake Engineering, London, Elsevier Science Ltd, paper no.665.

Mander, J.B. Priestley, M.J.N. Park, R. 1988. Theoretical stress-strain model for confined concrete. Journal of Structural Engineering, **114**(8): 1804-1826.

Martinez-Rueda, J.E. Elnashai, A.S. 1997. Confined concrete model under cyclic load. Materials and Structures, **30** (197): 139-147

Nehdi, M., Alam, M.S., Youssef, M.A. 2010. Development of corrosion free concrete beam-column joint with adequate seismic energy dissipation. Engineering Structures, 32(9): 2518-2528.

NBCC (2005) National Building Code of Canada, Canadian Commission on Building and Fire Codes. National Research Council of Canada, Ottawa.

Paulay, T. Priestley, M.N.J. Seismic design of reinforced concrete and masonry buildings. John Wiley & Sons, Inc., New York, 1992.

PEER, 2007, PEER NGA Database, Pacific Earthquake Engineering Research Center, University of California, Berkeley, California, Available at http://peer.berkeley.edu/nga/.

Saiidi, M.S. Zadeh, M.S. Ayoub, C. Itani, A. 2007. Pilot study of behavior of concrete beams reinforced with shape memory alloys. J Mater Civil Eng, ASCE **19**: 454–461.

SeismoStruct user Manual, (2012) Version 6. Pavia, Italy. Seismo-Soft Inc. Supporting Services.

Vamvatsikos, D. and Cornell, C.A. 2002. Incremental dynamic analysis. Earthquake Engineering and Structural Dynamics, **31**(3): 491-514.

Wilde, K. Gardoni, P. and Fujino, Y. 2000. Base isolation system with shape memory alloy device for elevated highway bridges. Eng Struct **22**: 222–229.

Youssef, M.A. Alam, M.S. Nehdi, M. 2007. Experimental investigation on the seismic behaviour of beam-column joints reinforced with superelastic shape memory alloys. J Earthq Eng **12**:1205-1222.

Zafar, A. and Andrawes, B. 2012. Incremental Dynamic Analysis of concrete moment resisting frames with shape memory composited bar. Smart Materials and Structures. 21(2012) 025013. 14pp