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SEISMIC PERFORMANCE OF MODULAR STEEL FRAMES EQUIPPED WITH SHAPE MEMORY ALLOY BRACES

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ABSTRACT

Modular steel buildings (MSBs) are widely used for one to six storey schools, apartments, and similar buildings, where repetitive units are required. Modular units are first built and finished under a controlled manufacturing environment. They are then transported to the building site, where they are connected horizontally and vertically. The lateral load resisting system for MSBs usually relies on steel braced frames, which dissipate the seismic energy through steel yielding. This behaviour leads to residual drifts complicating the repair of seismically damaged buildings or rendering them as irreparable. Systems that can minimize the seismic residual drifts are thus needed. Superelastic Shape Memory Alloys (SMAs) have the ability to undergo large plastic deformations and recover them upon unloading. Their utilization in steel structures can significantly reduce seismic residual deformations, which will facilitate post-seismic retrofitting. The purpose of this study is to examine the seismic performance of modular steel braced frame (MSBF) that utilizes SMA braces. A six-storey buckling restrained MSBF was considered as a case study. Nonlinear dynamic analysis was conducted to compare the seismic performance of this MSBF when it is fitted with steel and SMA braces. The use of SMA braces was found to improve the seismic performance of MSBs in terms of maximum residual inter storey drift (MRID) and damage scheme.

Keywords: - Modular steel building, Shape memory alloy, Seismic performance, Inter-storey drift, Residual drift.

1. INTRODUCTION

Different lateral load resisting systems such as moment resisting frames and braced frames are used in steel structures. In conventional steel braced frames, the seismic energy is dissipated through yielding of the tension braces and buckling of the compression braces. Several studies were conducted to assess the seismic performance of steel braced frames (Sabelli, R., 2001; Tremblay and Robert, 2001).

In order to overcome the buckling of conventional steel braces, researchers explored the use of special bracing systems such as buckling restrained braces, tension only braces, and self-centering shape memory alloy (SMA) braces. SMAs based on Nickel Titanium (NiTi) are the most suitable combination of material properties for most commercial applications .The two fundamental and characteristic properties of SMA are: The shape memory effect (SME) and superelasticity (SE). The shape memory effect is the ability of the material to recover from large mechanically induced strain (8%) via moderate increase in temperature. Super elasticity is the ability of the material in a high temperature regime to support relatively high strains in a loading process and by means of a hysteresis loop, recover its original shape when load is removed. The super-elastic SMA braces were found to be very effective in reducing the seismic damage and the associated residual deformations (McCormick et al., 2007; Kari et al., 2011; Asgarian et al., 2011; Eatherton et al., 2014). This behaviour was attributed to the unique property of superelastic SMAs, which allows maintaining the original shape regardless of the plastic deformations occurring during the earthquake. Although existing literature provides few research data on use of SMA in regular steel frames, no research was found on their potential applications in modular steel buildings.

The demand for modular steel buildings (MSBs) is increasing because of improved quality, fast on-site installation, and lower cost of construction. One to six storey MSBs usually rely on bracing elements for lateral stability. Figure 1 shows a plan view of a typical MSB along with the horizontal and vertical connections between the modules (Annan et al, 2009a). Lawson and Richards (2010) reviewed recent modular technologies and proposed a design method for high-rise-modular buildings while accounting for installation and construction tolerance. However, they did not examine their dynamic response under seismic loading. Annan et al. (2009a, 2009b, 2009c) investigated the seismic performance of modular steel braced frames (MSBFs). They emphasized that the seismic performance of MSBFs is significantly different from regular steel braced frames. Such difference is attributed to the existence of ceiling beams, the eccentricity developed at the joints as the braces do not intersect at a single working point, the semi-rigid connections between the columns of a module and the ones above them.

This study examines the seismic performance of MSBFs that utilize superelastic SMA braces. A six-storey MSB designed by Annan et al. (2009c) was chosen as a case study. The seismic performance of this MSB considering two types of braces, steel and superelastic SMA, was investigated in terms of maximum inter-storey drift (MID), maximum residual inter-storey drift (MRID), and damage distribution.

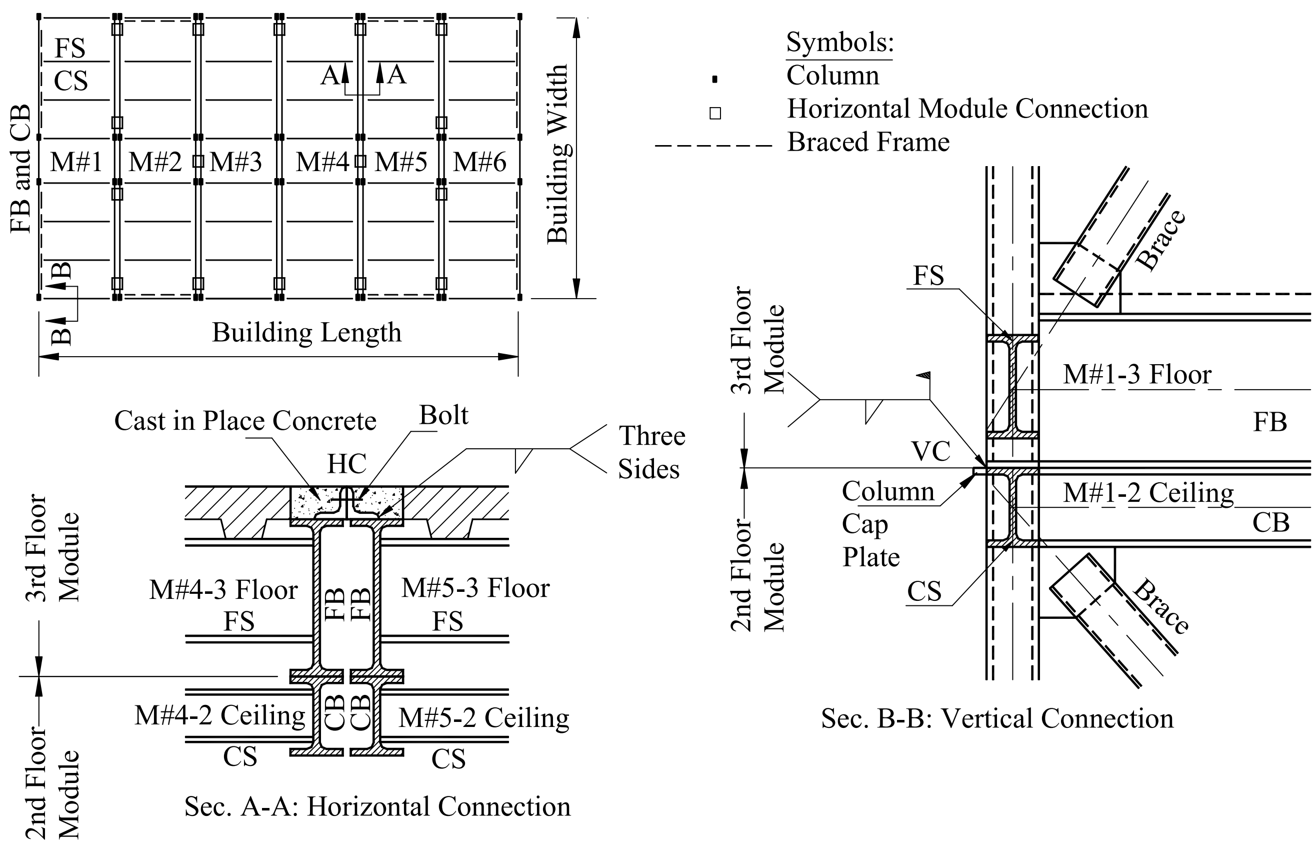


Figure 1: Typical plan and sections for a modular steel building (Annan et al., 2009a)

2. MODULAR STEEL BRACED FRAME

Annan et al. (2009c) designed a six-storey MSBF according to the Canadian standard (CSA, 2001) and the National Building Code of Canada (NBCC, 2005). The MSB has six modular units, as shown in Fig. 2, which are connected horizontally. The lateral response of the MSBs in the N-S direction is considered in this study. The lateral forces in this direction are resisted by the two external braced frames. Table 1 shows sections of the MSBF.

3. FINITE ELEMENT MODELING OF MSBFS

The modeling technique used to model the six-storey MSBF incorporated the unique detailing of the MSB. As the structure is symmetric, a two-dimensional (2D) frame is modelled using the software SeismoStruct. The model is based on the fibre -based modelling approach to represent the cross section of the various structural members, where each fibre is given a uniaxial stress–strain relationship. Beams and columns are modelled using force-based inelastic-frame elements considering distributed plasticity. The mass of the building is converted into lumped masses. The beams to columns connections are considered rigid as they are achieved by direct welding of the members. During construction, since the inner face of the columns are not accessible, only the outer faces of the upper and lower frame units are connected using field welding (Figure 1). The vertical joints between the modules were simulated as pin connections to allow for independent rotation of the upper and lower modules. The steel braces and the SMA braces of the MSBF were modelled using inelastic truss elements. Braces were assumed to be buckling restrained. The P-Δ effect is included in the analysis.

Menegotto-Pinto hysteretic material model (1973) is assumed for the steel members. The model parameters are as follows: yield stress of 350 N/mm^2, elastic modulus of 200 kN/mm^2, and strain hardening of 3%. The SMA material model proposed by Aurichio (1997) and implemented by Fugaza (2003) was adopted in this study. The model assumes a constant stiffness for both the fully austenitic and fully martensitic behavior. The SMA model parameters are provided in Table 2. They were selected based on the studies conducted by DesRoches et al. (2004). Rayleigh damping was used in the model with a damping coefficient of 5%.

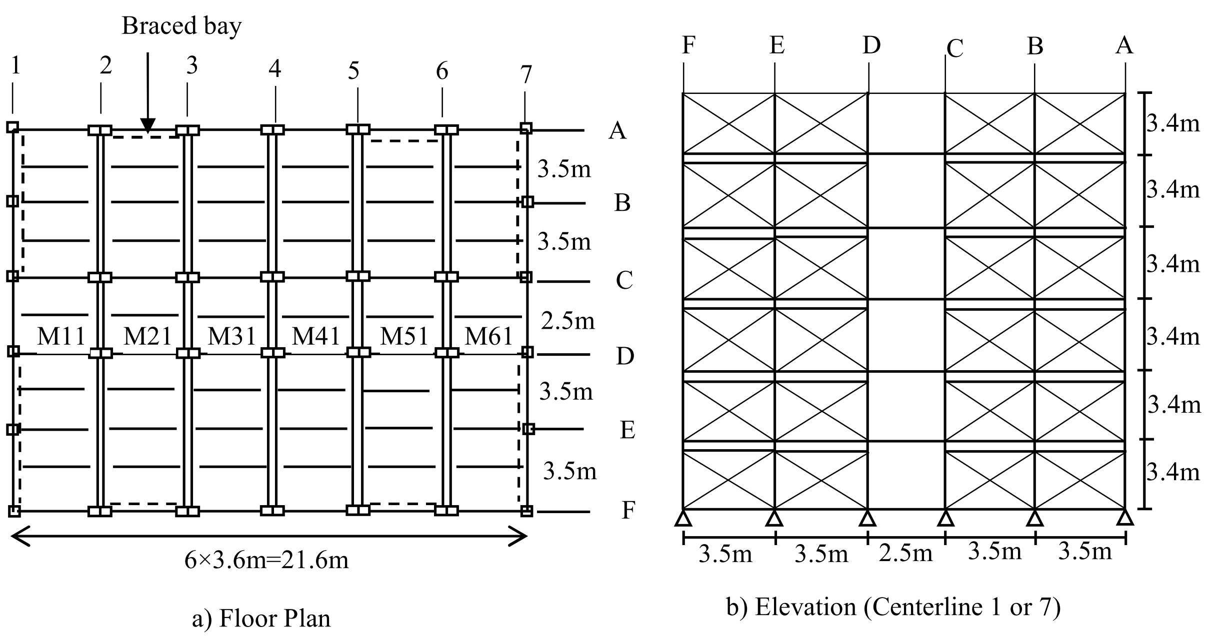


Figure 2 Six-storey modular steel braced frames

Table 1: Section properties of MSBF

Beam		Column		Braces	
Storey/Floor	Sections	Storey	Sections	Storey	Sections
Roof	W100×19	Storey 6	HS 102×102×6	Storey 6	HS 76×76×5
Floor 6	W250×33	Storey 5	HS 178×178×6	Storey 5	HS 102×102×6
Floor 5	W250×33	Storey 4	HS 203×203×10	Storey 4	HS 102×102×6
Floor 4	W250×33	Storey3	HS 305×305×10	Storey3	HS 102×102×6
Floor 3	W250×33	Storey 2	HS 305×305×13	Storey 2	HS 102×102×6
Floor 2	W250×33	Storey 1	HS 305×305×13	Storey 1	HS 102×102×6
Floor 1	W250×33				
Ceiling	W100×19				

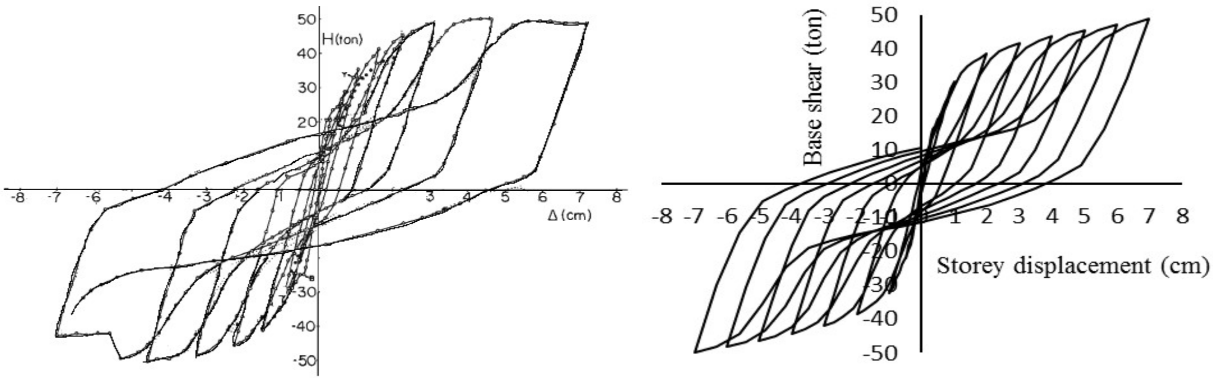
Table 2: Material properties of SMA

Material properties	Value
Modulus of elasticity E (MPa)	55,000
Austenite –to-martensite starting stress (MPa)	420
Austenite –to-martensite finishing stress (MPa)	520
Martensite-to-austenite starting stress (MPa)	310
Martensite-to-austenite finishing stress (MPa)	240
Superelastic plateau strain length (%)	6

3.1 Validation of FE modeling technique

A concentrically braced steel frame tested by Wakabayashi et al. (1974) was modelled using the technique explained in the previous section. Braces were modelled using inelastic frame elements that incorporated buckling behaviour by assuming an initial geometric imperfection. As the experimental cyclic load curve was not available, the cyclic load for numerical simulation was developed based on the experimental maximum storey displacement shown in the Figure 3(a). The numerical and the experimental results are shown in Figure 3. The FE model provided reasonable predictions of the frame behaviour in terms of maximum base shear and residual drift.

Annan et al. (2009a) conducted an experimental study to assess the hysteretic characteristics of MSBFs under cycling loading. The frame, Figure 4, was modelled while accounting for the unique details of the modular frame. Figure 5 shows the model. The heavy lines represent rigid elements. Member M1 is a short extension of the column of the lower module to connect to the upper module. Its height represents the 150 mm vertical clearance required for fire proofing between the two storeys. The vertical joint, j5, was simulated as a pin connection to allow independent rotation of the upper and lower modules. Figure 6 compares the experimental base shear versus drift (%) curve with the numerical ones. The maximum base shear of FE model at 3.5% drift is only 6.67% lower than the experimental value, which proves that the adopted modeling technique can reasonably predict the seismic behaviour of the MSBFs.



a) Experimental result (Wakabayashi et al., 1974) b) Numerical simulation

Figure 3: Comparison of numerical and experimental responses

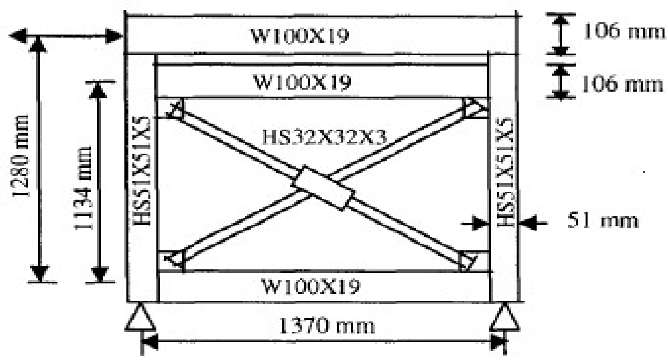
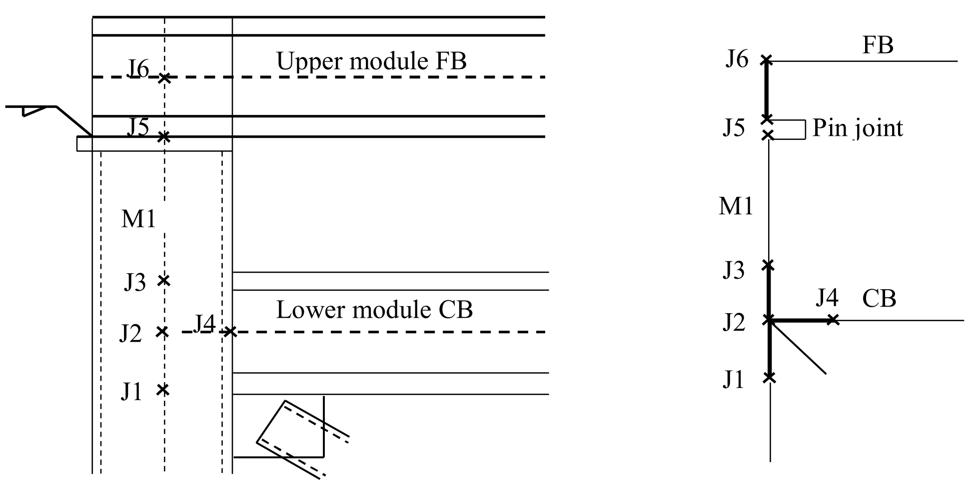
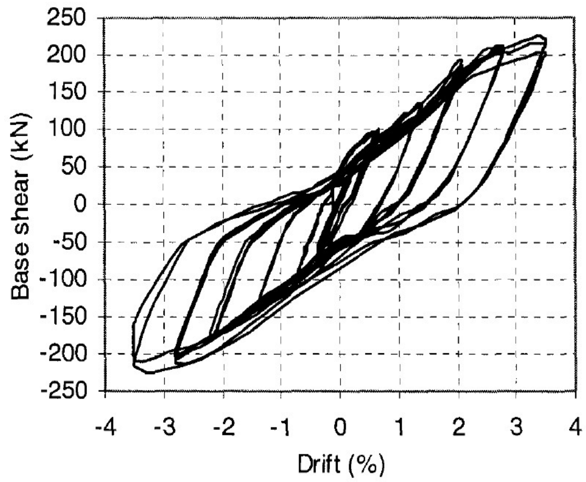


Figure 4: Geometry of MSBF tested by Annan et al. (2009a)

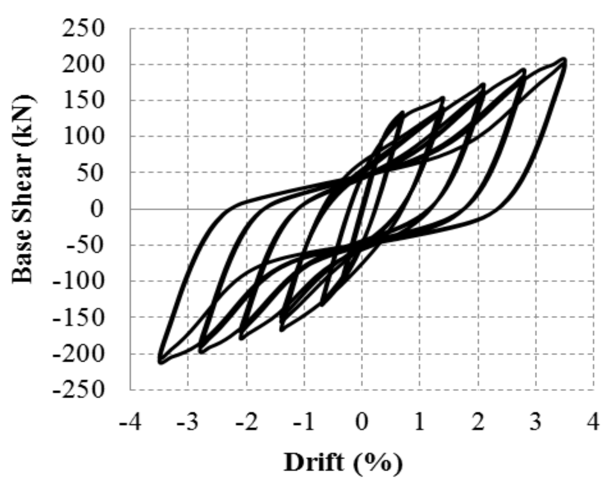


Vertical connection representation Model representation

Figure 5: Model of vertical connection of MSBF



a) Experimental result (Annan et al., 2009a)



b) Numerical simulation

Figure 6: Comparison of experimental and numerical results

4. MODULAR STEEL BRACED FRAME WITH SMA BRACES

The steel braces of the MSBF were replaced with the super-elastic SMA braces in such a way that the overall frame system would have the same natural period of vibration as the MSBF. This was achieved by selecting the area and length of SMA braces so that the frame had the same initial stiffness and yield forces as the designed steel bracing system at each storey. The same design philosophy was used by other researchers (Asgarian et al., 2011; Hu et al., 2014; McCormick et al., 2007; Moradi et al., 2014). SMA braces were modelled using inelastic truss elements connected to rigid elements as shown in Figure 7. The same beam and column sizes of MSBF with steel braces are maintained in the SMA-MSBF.

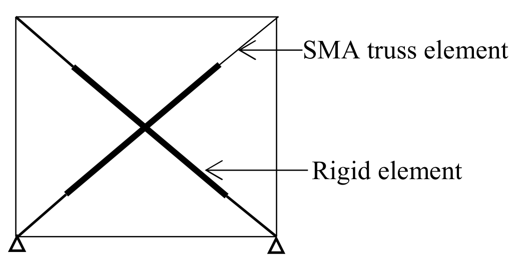


Figure 7 Modeling of SMA braces

5. DYNAMIC ANALYSIS

First, Eigen value analysis was performed to determine the natural period of vibrations and mode shapes of frames. The first and second natural periods of vibrations are 0.54 second and 0.19 second, respectively. Incremental dynamic analysis (IDA) was then conducted considering two different ground motions. The ground motions are scaled at different intensities to capture the structural response up to collapse. Characteristics of selected ground motions from PEER ground motion database are listed in Table 3.

IDA analysis was first conducted for the MSBF with steel braces (Steel-MSBF). The failure of the frame was assumed when all of the columns in one storey reach the yield state. The seismic intensity is expressed in term of the spectral acceleration at the first period of vibration [Sa (T1, 5%)]. After that the steel braces were replaced by the SMA braces. Nonlinear dynamic time history analysis of MSBF equipped with SMA braces (SMA-MSBF) was performed considering the ground motion intensity at which the Steel-MSBF reached failure.

Table 3: Characteristics of ground motions

Earthquake	Ms magnitude	Station	PGA(g)
Imperial Valley	6.9	El Centro Array #13	0.139
Loma Prieta	7.1	Capitola	0.451

Source: PEER ground motion database, http://peer.berkeley.edu/svbin

6. RESULTS OF THE DYNAMIC ANALYSIS

MID and MRID of Steel-MSBF and SMA-MSBF are listed in Table 4 along with the earthquake intensity. It is observed that the storey experiencing the MID is not always the storey experiencing the MRID. The MRID of Steel-MSBF varied from 0.32% to 0.57% indicating a severely damaged frame. Although incorporating SMA braces increased the MID up to 8.7%, it reduced the MRID up to 98.6%. The distribution of ID and RID along the building height are compared in Figures 8 and 9. It is observed that first three storeys of Steel-MSBF experienced higher IDs and RIDs as compared to the top three storeys. Incorporating SMA braces were more effective in reducing RID for these damaged storeys.

6.1 Damage distribution

The seismic performance of members of the MSBF was evaluated according to FEMA 356 (2000). The damage distribution is shown in Figures 10 and 11. Yielding of the columns and beams are presented by solid black circle while yielding of the braces is represented by heavy lines. Yielding of the beams, columns, and braces were observed at all storeys. Yielding of the short columns connecting the modules vertically was observed for both records; however it did not result in a local failure mechanism. The exterior columns and the columns of the unbraced bays experienced more damage than the remaining columns. All of the interior columns in the unbraced bays failed due to Imperial earthquake. Severe damage was observed up to the second storeys for both frames as four columns of each storey failed. Incorporating SMA braces reduced the damage of the top storeys as compared to Steel-MSBF. ID and RID distribution along the frame height, which are shown in Figures 8 and 9, agree with the observed damage distribution. The yield distribution suggests good distribution of energy dissipation along the height and width of both modular braced frames.

Table 4: MID and MRID at collapse

Ground motion	Sa(T1,5%) at collapse	Steel-MSBF		SMA-MSBF	
		MID (%)	MRID (%)	MID (%)	MRID (%)
Imperial	3.84g	3.37 (1st storey)	0.57 (2nd storey)	3.67 (2nd storey)	0.12 (3rd storey)
Loma	3.95g	3.33 (1st storey)	0.32 (2nd storey)	3.38 (1st storey)	0.005 (1st storey)

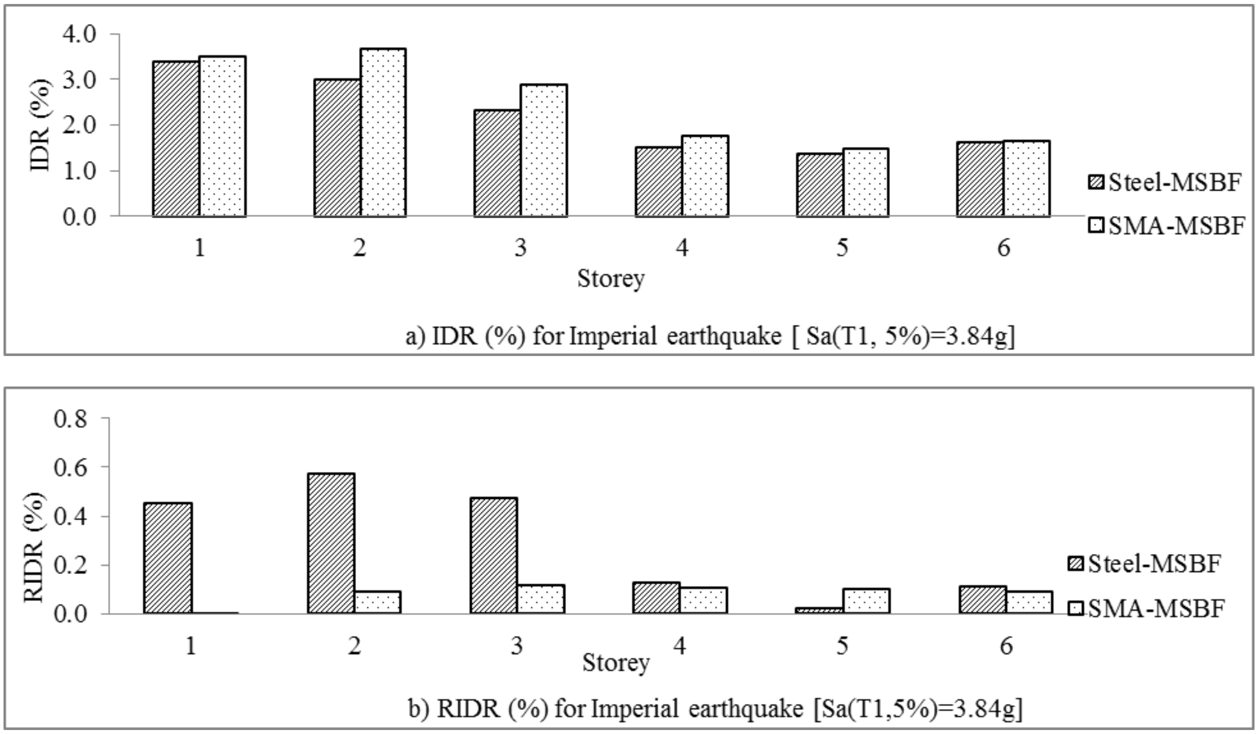


Figure 8: IDR and RIDR distribution for different storeys due to Imperial earthquake

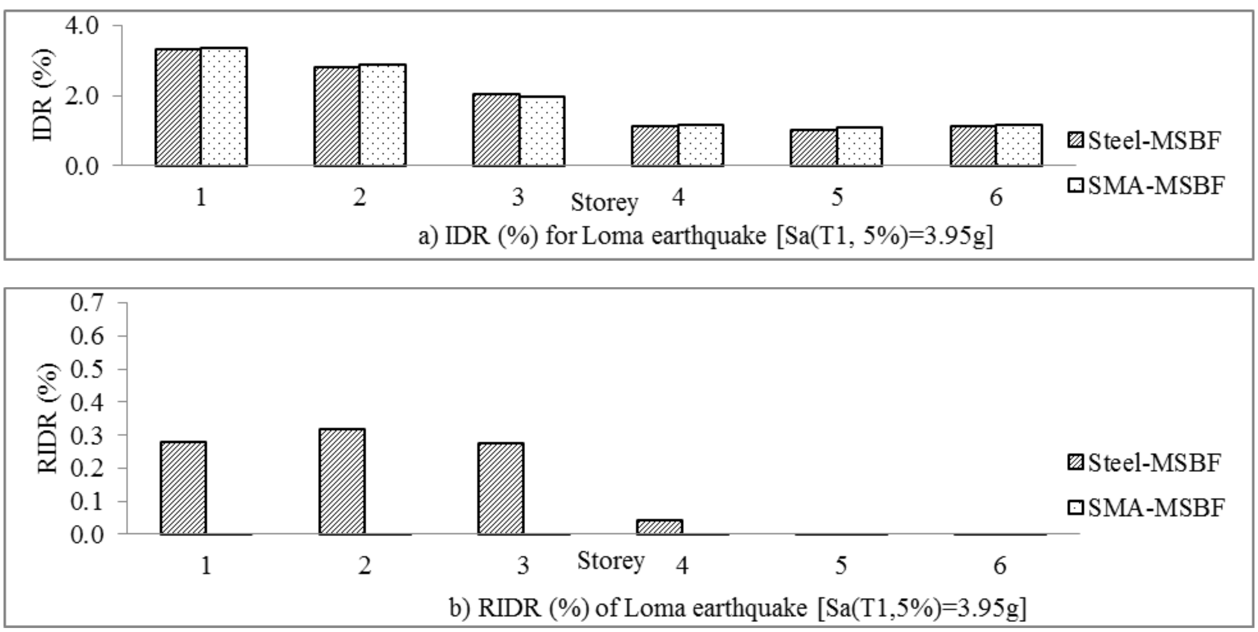
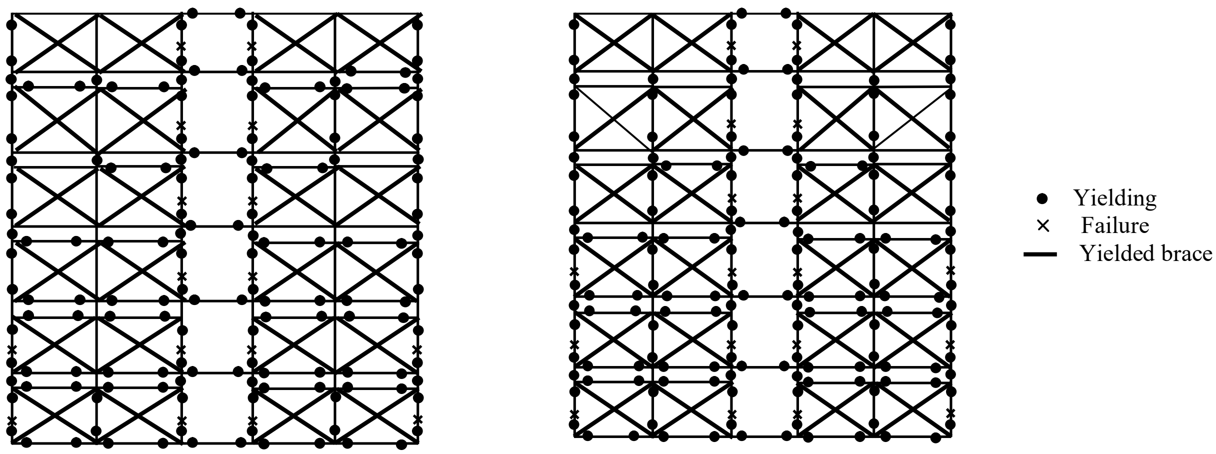
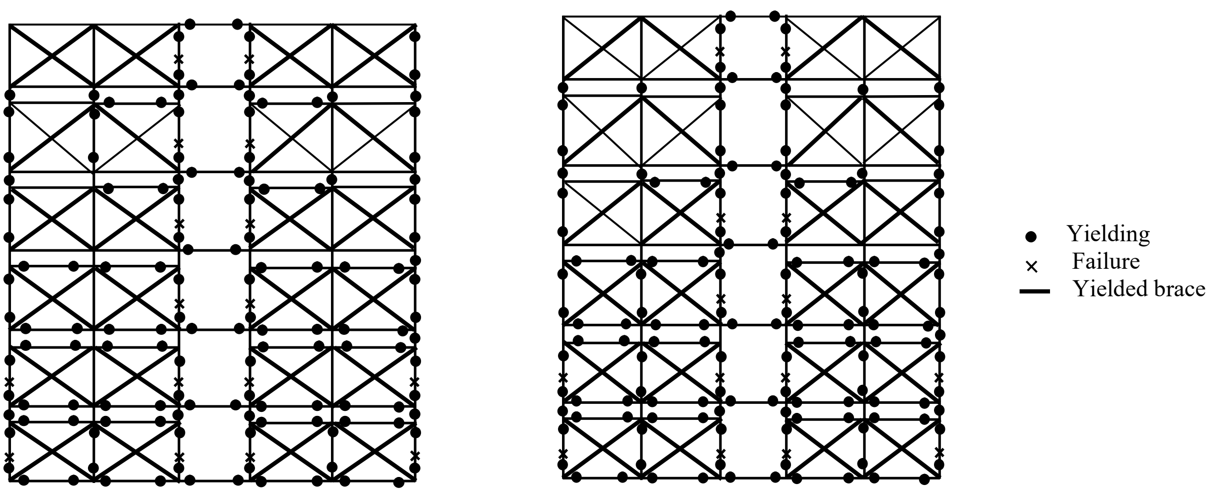


Figure 9: IDR and RIDR distribution for different storeys due to Loma earthquake



a) Steel-MSBF b) SMA-MSBF

Figure 10: Damage distribution due to Imperial earthquake [Sa(T1, 5%)=3.84g]



a) Steel-MSBF b) SMA-MSBF

Figure 11: Damage distribution due to Loma earthquake [Sa(T1, 5%)=3.95g]

7. SUMMARY AND CONCLUSION:

This study investigated the seismic performance of MSBF with steel braces and SMA braces. The behaviour of SMA-MSBF was evaluated and compared with the Steel-MSBF in terms of MID, MRID and damage distribution. It was observed that incorporating SMA braces did not significantly increase the MID. Due to the re-centering capability of super-elastic SMA braces; the MRID was reduced up to 98%, which clearly shows that SMA braces have the potential to reduce seismic damage and retrofitting cost.

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