Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 488

SEISMIC PERFORMANCE OF SHAPE MEMORY ALLOY REINFORCED CONCRETE FRAMES

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ABSTRACT

Superelastic Shape Memory Alloys (SMAs) are unique materials which can regain their original length upon unloading. Utilizing superelastic SMA bars in Reinforced Concrete (RC) frames results in dissipating the earthquake energy while minimizing the seismic residual deformations. In this study, the critical sections of a six-storey steel RC building are defined using five different earthquake records. The building was then redesigned using SMA bars. Seven alternative designs were explored representing using the SMA bars at the plastic hinge zones of the critical beam/column sections and/or at the beam sections adjacent to the critical columns. Each design alternative was subjected to dynamic analysis using the chosen five records scaled to the intensity causing severe damage to the steel RC frame. It was concluded that using SMA bars at the critical beam sections and at the beam sections adjacent to the critical columns results in minimal local damage and residual drifts.

Introduction

The post-earthquake serviceability of Reinforced Concrete (RC) structures is jeopardized due to the residual lateral deformations. Recent research has focused on reducing residual lateral deformations using re-centering devices (Valente 1999), passive energy dissipating devices (Clark 1995), and Shape Memory Alloys (SMAs) (Alam 2009).

Super-Elasticity (SE) is a distinct property that makes SMA a smart material. A SE SMA can recover inelastic strains by stress removal (DesRoches 2004). Among various composites, Ni-Ti SMA is found to be the most appropriate SMA for structural applications because of its large recoverable strain, SE, energy dissipation, and exceptionally good corrosion resistance. The phase change of this alloy can be stress-induced at room temperature if the alloy has the appropriate formulation and treatment (DesRoches 2002). Fig. 1 shows a simplified model for the SMA stress–strain relationship (Elbahy 2009). The model consists of four linear branches. The parameters used to define the loading plateau of this model are f_{cr} (austenite, AST, to martensite, MRT, starting stress); f_{P1} (AST to MRT finishing stress); ε_{p1} (MRT stress induced strain); $f_{y\text{-}SMA}$ (real yielding stress) and modulus of elasticity E_{cr}. The unloading plateau is defined by two parameters: f_{T1} (MRT to AST starting stress) and f_{T2} (MRT to AST finishing

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stress). For structural applications, it is recommended to design SMA RC sections to behave within the superelastic range shown in Fig. 1 (Youssef 2008). Thus the yield stress recommended for the design should be taken equal to f_{cr} (Elbahy 2009).

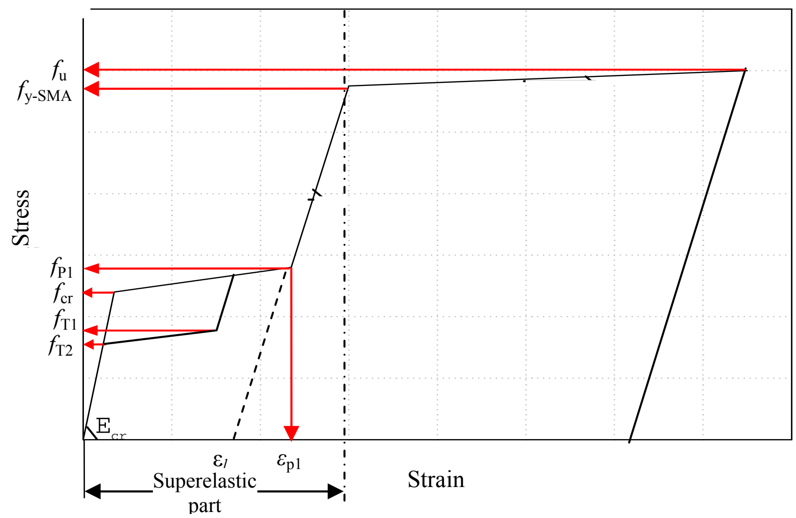


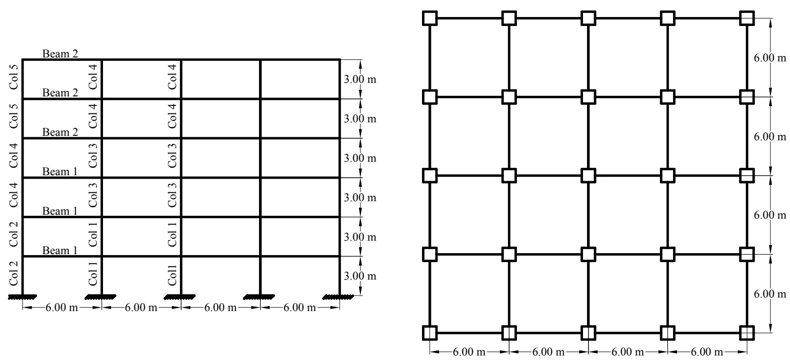
Figure 1. Typical stress-strain model for SMA.

Since SMA is an expensive material, it was not until recently that it found its way as reinforcing bars in RC structures. The experimental results of Sakai (2003) showed that mortar beams reinforced with SMA wires recover most of their inelastic deformations after releasing the load causing failure to the beams. Using shake table tests, Saiidi (2006) showed that RC columns reinforced with SMA bars in the plastic hinge area are able to recover most of their post-yield deformation and can withstand earthquakes with higher amplitude than steel RC columns. The experimental and analytical work of Youssef (2008) and Alam (2008) showed that beam-column joints with SE SMA bars are advantageous to steel RC joints in case of cyclic loading. Alam (2009) conducted dynamic analysis on an eight-storey concrete frame reinforced with SMA bars at the plastic hinge areas of all beams. This frame had reduced Residual Inter-storey Drifts (RIDs) than a steel RC frame.

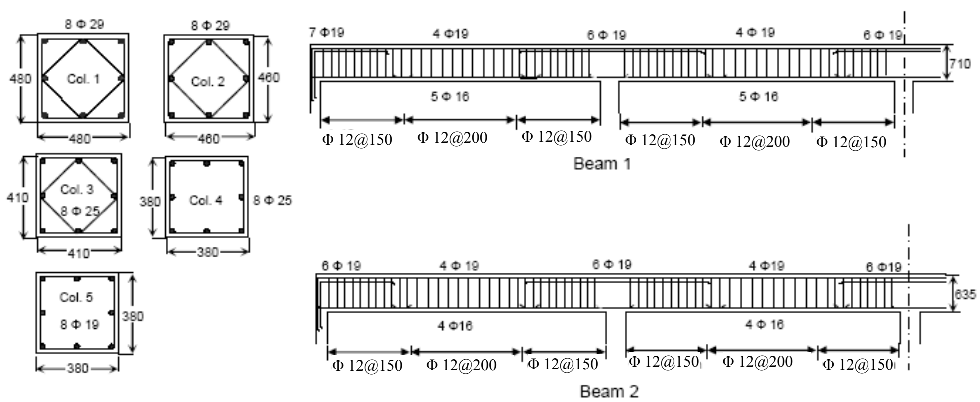
Because of the relatively high cost of SMA bars, this paper aims at reducing the amount of SMA bars to be used in a typical RC frame while keeping the benefit of reducing the RIDs. Nonlinear dynamic analyses are performed for a regular steel RC framed building using five earthquake records up reaching severe damage state. The building is then redesigned using SMA bars in the identified critical locations. Seven different arrangements for the SMA bars are selected resulting in seven different frames. Nonlinear dynamic analyses are then conducted using same records with the same intensities to select the frame which has the best seismic performance in terms of the amount and severity of damage, the Maximum Inter-storey Drift (MID) and the Maximum Residual Inter-storey Drift (MRID).

Steel RC Frame

A symmetric six-storey RC office building has been selected for this study. The selected dimensions and layout of the building are shown in Fig. 2. The building is designed according to the regulations of the International Building Code (IBC 2006) and the ACI requirements (ACI 318 2005) for both gravity and seismic loads assuming that it is located in California. The concrete unconfined compressive strength and the reinforcing steel yield strength are assumed 28 MPa and 400 MPa, respectively. The dead loads include the weight of the structural elements and the masonry walls. The live load is considered 4.8 kN/m^2. The lateral load resisting system is composed of five special moment frames. Section dimensions and reinforcement details for a typical moment frame (Frame 1) are given in Fig. 2b.



a) Plan and Elevation



b) Cross sections of beams and columns

Fig. 2 Six-storey building

The finite element program Seismostruct (Seismosoft 2008) was utilized to perform the nonlinear dynamic analyses. The program is capable of representing spread of inelasticity within the member length using the fiber analysis approach and can be used to predict the nonlinear response of moment frames under static or dynamic loading. Concrete, steel and SMA material models are available in the program library. The program has been tested and validated by Saiidi (2006) and Alam (2008 and 2009).

As the structure is symmetric, a 2D model is used. Beams and columns are modeled using cubic elasto-plastic elements. To match the distribution of longitudinal and transverse reinforcements and to monitor the progress of local damage, beams and columns are divided into six and three elements, respectively. The frame beams are modeled as T-sections assuming an effective flange width equal to the beam width plus 14% of the clear beam span (Jeong 2005). The beam-column connections are modeled using rigid elements as shown in Fig. 3.

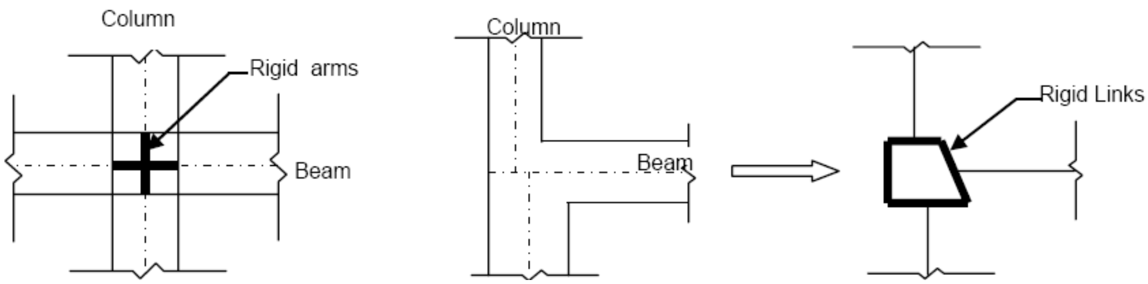


Figure 3. Rigid arms for modeling of beam-column connections

Local and Global Damage Criteria

Local yielding of elements is defined when the tensile strain in the longitudinal reinforcement reaches the yield strain (0.002 for steel and 0.007 for SMA). Local failure of concrete members is defined using the concrete crushing strain. The crushing strain is expected to depend on the type of concrete, the level of confinement, and the level of axial force. The crushing strain was found to be varying from 0.015 to 0.05 for confined concrete (Paulay 1992). It occurs when the stirrups reach their fracture strength (Pauley 1992), and is given by Eq. 1.

$$\varepsilon_{cu(\text{confined concrete})} = \varepsilon_{cu(\text{unconfined concrete})} + \frac{1.4\rho_s f_y \varepsilon_{sm}}{k_h f_c'} \tag{1}$$

where ρ_s is the ratio of the volume of transverse reinforcement to the volume of concrete core measured to outside of the transverse reinforcement, f_y is the steel yielding stress, ε_{sm} is the steel strain at maximum tensile stress and K_h is the confinement factor. The value of $\varepsilon_{cu(\text{unconfined concrete})}$ is assumed 0.0035 in this paper.

The collapse limit for a RC frame has been defined by majority of researchers using a single value of MID or RID. This definition has led to a wide range of proposed values for MID at collapse varying from 2% (Sozen 1981) to 6% (Roufaiel 1983). Unlike MID, only few researchers worked on defining damage levels using RIDs. Stephens (1987) showed that buildings will be critically damaged at 1% RID. FEMA 273 (1997) defined collapse limits to be at 3% RID or 4% MID. In this paper, rather than using a building collapse limit defined using a single value of drift, a building Severe Damage State (SDS) is used. The SDS is assumed to occur when four columns in the same storey reach the crushing state.

Dynamic Analysis of the Steel RC Frame

Selection of Ground Motion Records

Five earthquakes records are selected to conduct the dynamic analysis. These records cover a wide range of ground motion frequencies represented by the ratio between the peak ground acceleration and the peak ground velocity (A/v ratio). The characteristics of the chosen records are presented in table 1. Using a reliable method to scale the selected ground motion records is critical to conduct dynamic analysis. Available methods include scaling based on: peak ground acceleration, peak ground velocity, and the 5% damped spectral acceleration at the structure's first-mode period [Sa(T1, 5%)]. Using Sa(T1,5%) to scale the records was found to be a reliable method (Shome 1999 and Vamvatsikos 2002). Each earthquake is scaled to different Sa(0.501 second, 5%) levels and then used in the dynamic analysis.

Table 1. Earthquake records

Earthquake	**Date**	**Ms Magnitude**	**Station**	**PGA (g)**	**A/v**
Northridge USA	17/1/94	6.7	Arleta-Nordhoff	0.340	Inter.
Imperial Valley USA	15/10/79	6.9	El Centro Array #6 (E06)	0.439	Low
Loma Prieta USA	18/10/89	7.1	Capitola (CAP)	0.530	High
Whittier USA	1/10/87	5.7	Whittier Dam	0.316	High
San Fernando	9/2/71	6.6	Pacoima Dam	1.23	Inter.

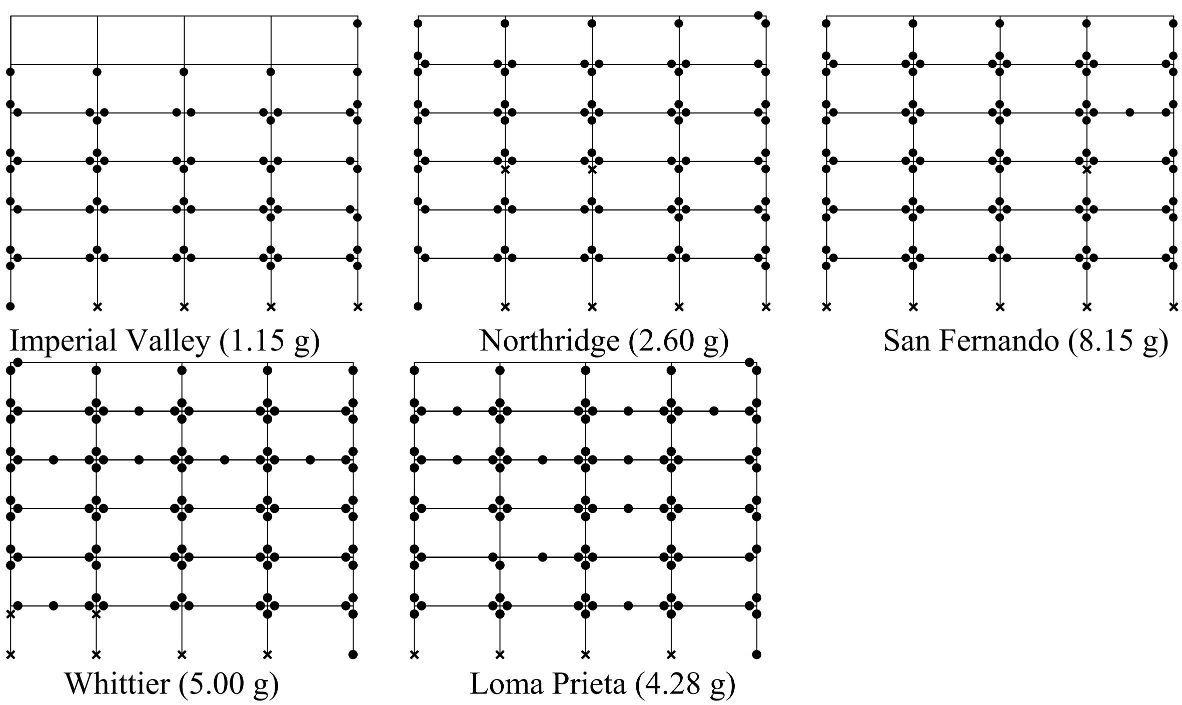


Figure 4. Damage Scheme of Steel RC frame at SDS

Table 2 Sa, MID, and MRID for the steel RC frame at SDS

	Earthquake record				
	Northridge	Imperial Valley	Loma Prieta	Whittier	San Fernando
Sa (g)	2.6	1.15	4.28	5.00	8.15
MID (%)	5.13	4.36	5.00	6.25	5.25
MRID (%)	3.00	2.68	2.72	2.47	2.60

Discussion

The damage schemes at SDS under the effect of the selected records are shown in Fig. 4. Table 2 presents values for Sa, MID, and MRID at SDS. It can be observed that crushing is concentrated at the lower ends of the first storey columns. Most of the beams and columns experienced some degree of yielding. The 4th floor beams experienced the highest damage as they sustained yielding at their mid-spans under the effect of Whittier, Loma Prieta, and San Fernando earthquakes. The 5th floor beams sustained yielding at their mid-spans under the effect of Whittier and Loma Prieta ground motions. Table 2 shows that the MIDs and MRIDs at SDS are varying from 4.36% to 6.25% and from 2.47% to 3.00%, respectively.

SMA RC Frames

In this section, the analyzed steel RC frame is redesigned by replacing the steel at the critical sections defined in the previous section with SMA bars. Seven alternative locations for SMA bars are tested. The selected positions for the SMA bars are shown in Fig. 5. These alternatives (Frames 2 to 8) are representing the location of observed yielding in beams (frame 2), the most critical beams and/or columns (frames 3, 4 and 5), the beams adjacent to the critical columns (frame 6), and combinations of the most critical sections (beams or columns) with the beams adjacent to the critical columns (frames 7 and 8).

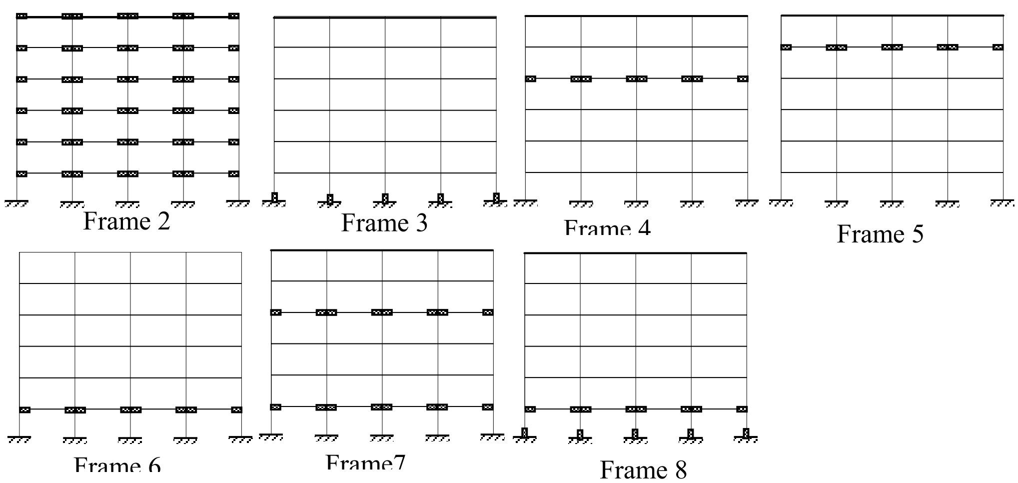


Fig. 5 Locations of SMA bars in the selected SMA-RC frames

For each frame, the SMA RC sections are redesigned using the method proposed by Elbahy (2009). The concrete maximum strain corresponding to the peak moment is assumed to be equal 0.00350 for beam sections and 0.00255 for column sections (Elbahy 2009). These values are used to calculate the stress block parameters and the required area of SMA. The SMA yielding stress is assumed 401 MPa (Alam 2009). The length of the plastic hinge (L_p) is calculated using Eq. 2 that was proposed by Paulay (1992) and recommended for SMA RC elements by Alam (2008).

$$L_p = 0.08 \, . \, L + 0.022 \, . \, d_{sma} \, . \, f_{cr} \qquad (2)$$

where L is the element length from the face of the beam-column joint to the inflection point,

d_{sma} is the SMA bar diameter, and f_{cr} is the yielding stress of SMA bars. The plastic hinge length is calculated as 390 mm and 373 mm, measured from the face of the column, for 19 mm and 29 mm bars, respectively. Mechanical couplers are assumed to connect SMA with regular steel bars as recommended by Youssef (2008) and Saaidi (2006).

Each SMA RC frame is subjected to the selected five earthquake records scaled to the intensity causing SDS of the steel RC frame. In the following sections the seismic performance of these buildings is evaluated.

Damage Schemes

The damage schemes for the seven frames in case of Whittier record are illustrated in Fig 6. For all frames, yielding occurred in most beams and columns. Yielding is observed at mid-spans of some of the beams. As the SDS is defined in this paper by crushing of four columns at the same storey level, Fig. 6 shows that all the frames except frame 7 can be considered at SDS in case of Whittier record.

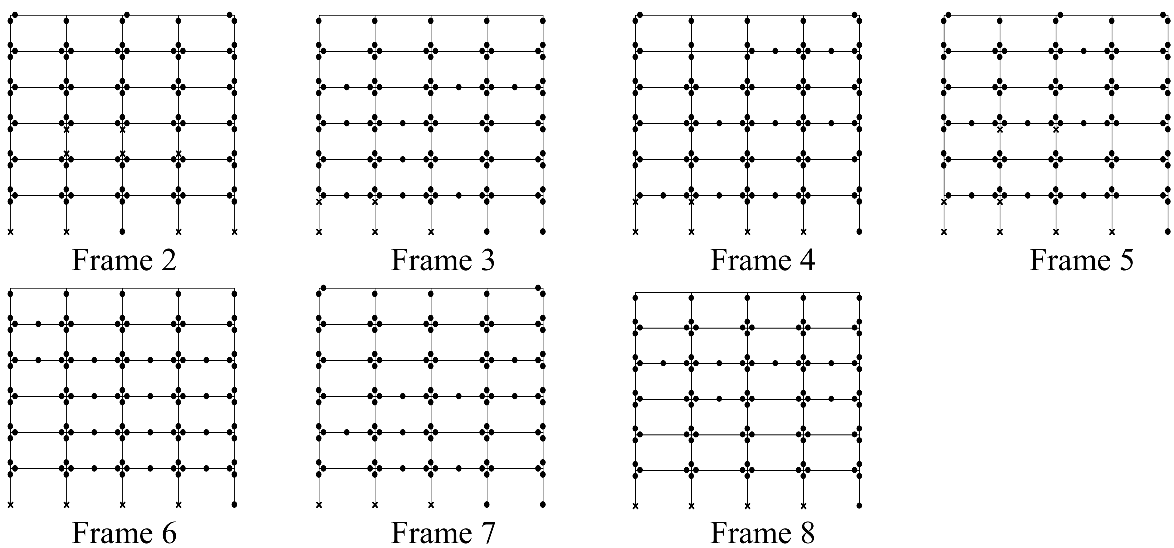


Fig. 6 Frames damage in case of Whittier record at Sa = 5.0 g

Frame 7 is a combination of using SMA at beams adjacent to the critical columns (at the end of the first floor beams) with using SMA bars at the critical beams (the fourth floor beams). The damage scheme of Frame 7 shows that crushing is observed at only three columns at the first storey level and no crushing is observed elsewhere in the building. This is due to the re-centering effect of SMA which has reduced the residual strains. On contrary to this observation, it can be observed from the damage scheme of Frame 2, with SMA bars used at all the beams ends (SMA used at 48 sections), that crushing of the third storey columns could not be prevented. This is due to the large amount of SMA bars that have led to increasing the maximum deformations. Thus, it can be concluded that the damage schemes are affected by the location and the amount of SMA bars. Similar damage scenarios are observed for other earthquake records. Frame 7 had the best damage scheme as it can tolerate higher earthquake intensities.

Maximum and Residual Drifts

The average ratios of the MID and the MRID for all the studied frames at the Sa value causing SDS of the steel frame are calculated and illustrated in Fig. 7. The steel-RC frame has the lowest MID (5.20%) and Frame 2 (SMA used at 48 sections) has the highest MID (6.42%). All the other frames have values of MID that ranges from 5.57% to 5.77%. The increase in MID values is mainly resulting from the modulus of elasticity of SMA that is about one third the steel modulus of elasticity. The results show that MID demands are affected by the amount of SMA bars regardless of their location.

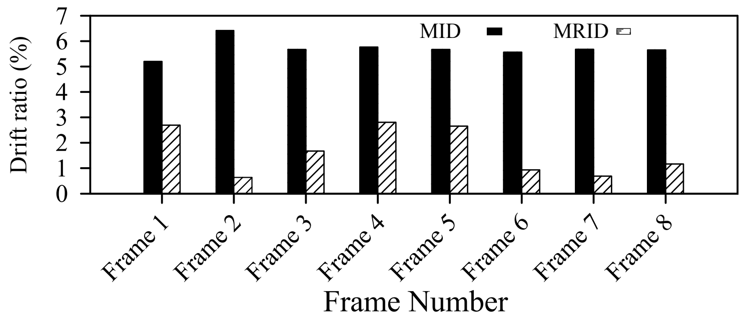


Figure 7 Average values of MID and MRID ratios at Sa causing SDS of the steel frame

The average values of MRID demands presented in Fig. 7 show a different scenario. The location of the SMA bars greatly affects the MRID demand. The reduction in the MRID relative to the steel frame is 76.24%, 74.54%, 65.38%, ,56.87%, 37.78%, and 1.56% for Frames 2, 7, 6, 8, 3, and 5, respectively. For frame 4, the MRID demands have increased by about 4.08%.

It can be concluded that Frame 7 has the best configuration of SMA bars as: (1) it has reasonable amount of SMAs (SMA at 16 sections), (2) its MRID is significantly less than that of the steel RC frame, (3) its MID is comparable to that of the steel RC frame, and (4) it has the best damage scheme (Fig 6) and can tolerate higher earthquake intensities before reaching SDS.

Conclusions

This paper aims at optimizing the use of smart material, SMA, in RC frames to achieve the best seismic performance by enhancing the building damage scheme and by lowering the MRID demands. A six-storey RC frame building, located in highly seismic zone, is considered as a case study.

The dynamic analysis showed that the steel-RC frame SDS has resulted from crushing of the first storey columns. The highest damage has occurred in the beams of the 4^{th} and the 5^{th} floors. The results of the building deformations showed that the MID representing the SDS varied between 4.36% and 6.25% and the MRID obtained from the analyses varied between 2.47% and 3.00%.

After defining the position of the critical sections in the building, seven different designs that utilize SMA bars are examined using the same earthquake records scaled to the Sa value causing SDS of the steel-RC frame. The total amount of used SMA bars, regardless of their locations, is found to have a major effect on the MID demand. On the contrary, the location of the SMA bars greatly affects the MRID demand. The best seismic performance is achieved when

SMA bars are used at the critical beams (4th floor beams) and at the ends of the beam adjacent to the critical columns (1st floor beams). This performance is characterized by the lowest number of crushed columns, a minor increase in MID relative to the steel RC frame, a significant reduction in MRID values, and an increase in the PGA value causing SDS to the steel RC frame.

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