Seismic Assessment of Concentrically Braced Steel Frames with Shape Memory Alloy Braces

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Abstract: The use of special concentrically braced frames has increased since the 1994 Northridge and 1995 Hyogoken–Nanbu Earthquakes. However, past performance suggests limited ductility and energy dissipation in braced frame systems due to buckling of conventional braces. In order to address this limitation, three- and six-story concentrically braced frames with superelastic shape memory alloy (SMA) braces are studied to evaluate their seismic performance in comparison to traditional systems. SMAs are unique metallic alloys that have the ability to undergo large deformations while reverting back to their original undeformed shape providing recentering capabilities to the braced frame. Detailed analytical models of the frames with SMA braces are developed and two suites of ground motions are used to evaluate the structures with respect to interstory drift and residual drift. The results suggest that the SMA braces are effective in limiting interstory drifts and residual drifts during an earthquake, in part, due to the recentering nature of superelastic SMAs.

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Introduction

Steel frame structural systems have been widely used in the United States for mid- to high-rise structures. A large majority of these systems built before 1994 consisted of steel moment-resisting frames to provide lateral resistance during an earthquake. The occurrence of the 1994 Northridge Earthquake and 1995 Hyogoken–Nanbu (Kobe) Earthquake caused unexpected damage to many of these systems due to fracture of welded beam-to-column connections resulting in unacceptably large lateral displacements (Nakashima et al. 1998). In order to prevent future problems associated with geometric nonlinearities and brittle fracture of the beam–column connection in steel moment-resisting frames, research in the United States has focused on understanding the nonlinear and brittle performance of these steel frame structures. Significant efforts were undertaken to develop different connection geometries and configurations to mitigate these problems (Nakashima et al. 2000). The problems associated with steel moment-frame systems led, also, to a search for alternative economical lateral load resisting systems, such as the use of concentrically braced frames, and more recently special concentrically braced frames (Sabol 2004). Concentrically braced structures continue to be used as lateral load resisting systems with expected increases in their use as new systems and design approaches are developed. Although there has been an increase in the use of braced frame systems, damage during past earthquakes suggests that braced systems may perform poorly due to limited ductility and energy dissipation of the bracing system, failure of the connection between the braces and the frame, and asymmetric behavior of the brace in tension and compression (Sabelli et al. 2003). A lack of knowledge in regards to the behavior of concentrically braced steel frame systems has prompted efforts to characterize the performance of such structures and develop more reliable design standards.

As a further result of the Northridge and Hyogoken–Nanbu Earthquakes, the structural engineering community has focused on a more performance-based seismic design approach in order to prevent a reoccurrence of similar damage and economic losses in future earthquakes. Performance-based design provides engineers with the means to design and analyze building structures such that they have a predictable and reliable performance in the event of an earthquake (Hamburger et al. 2003). The Federal Emergency Management Agency (FEMA)/SAC steel project resulted in the development of guidelines for the design of steel moment frames, which limited interstory displacement to given performance levels (FEMA 2000). However, such a comprehensive study has yet to be completed for concentrically braced frame systems. There exists a need to investigate improved design and retrofit measures for current concentrically braced frame systems to ensure that these systems fit in accordance with performance-based design parameters.

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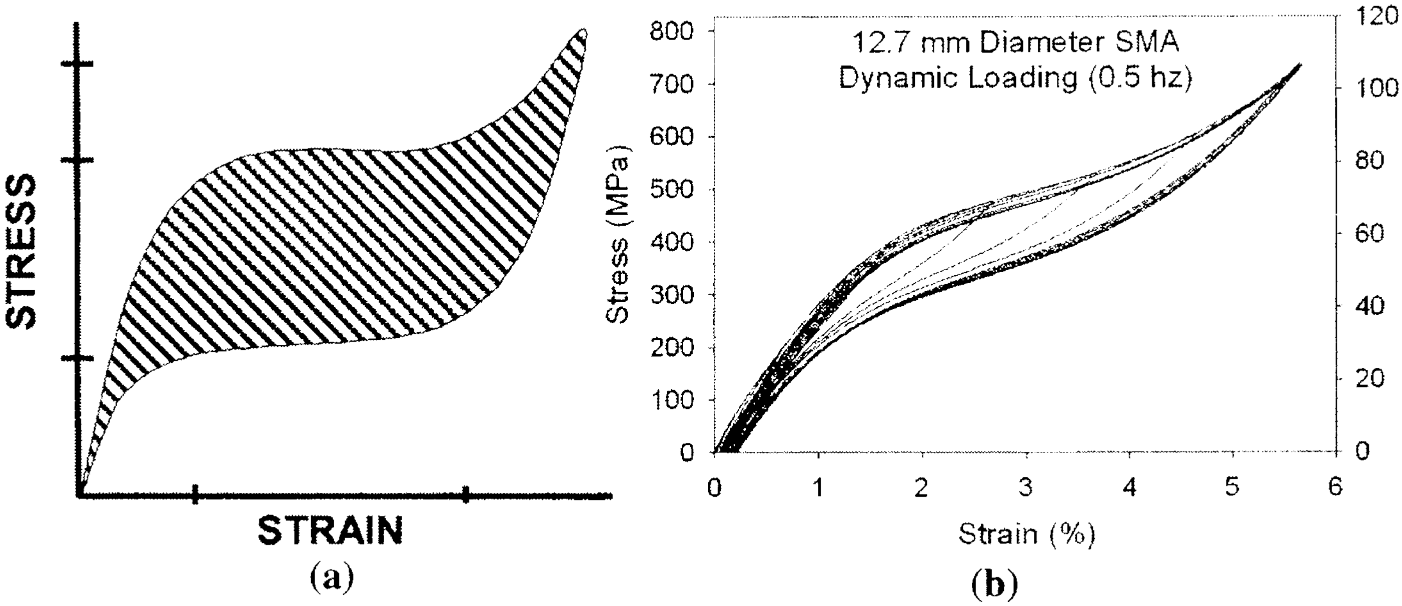

Fig. 1. (a) Idealized superelastic SMA behavior; (b) experimental results from cyclic tensile tests on a 12.7 mm diameter SMA bar (DesRoches et al. 2004)

One means of improving the performance of concentrically braced frame systems in terms of limiting interstory drift levels is through the use of innovative materials in the bracing system. In particular, superelastic shape memory alloys (SMAs) have been shown to develop a flag-shape hysteresis under cyclic axial loading, which can provide both recentering and supplemental energy dissipation to a structural system, resulting in limited interstory drifts and decreases in permanent displacement of the structure. The objective of this study is to assess the performance of both a conventional, concentrically-braced system configuration and a concentrically-braced system with superelastic SMAs through a series of nonlinear time history analyses to determine the potential of using SMAs in earthquake engineering applications for the dynamic control of building structures.

Shape Memory Alloys

Shape memory alloys are a class of metallic alloys that display several characteristics not present in traditional civil engineering materials. At the macroscopic level, SMAs feature two unique properties: the *shape memory effect* (SME) and the *superelastic effect* (SE). The SE is related to the ability to recover large deformations after the removal of the load, while the SME refers to the ability to regain the original shape through heating (Duerig et al. 1990). As a result of these properties, in recent years SMAs have attracted significant attention from the scientific community with several applications being developed in the aerospace, automotive, and biomedical fields.

The ability of SMAs to recover their shape is related to the reversible martensitic phase transformation that results from a solid-to-solid diffusionless process between a crystallographically more-ordered phase, austenite, and a crystallographically less-ordered phase, martensite. The austenitic phase tends to be stable at low stresses and high temperatures, while the martensitic phase is stable at high stresses and at low temperatures. The SME occurs when the ambient temperature is below the martensite finish temperature of the material resulting in the material being in its twinned martensite phase. Upon uniaxial deformation, the twinned martensite undergoes a detwinning process along with the accumulation of residual deformation. This deformation can only be recovered by heating the material above the austenite finish temperature causing a phase transformation to the high temperature austenite phase and a recovery of the residual deformation. During cooling, the material returns to its original, more stable, low temperature twinned martensite phase. Alternatively, the SE occurs when the ambient temperature is above the austenite finish temperature of the material resulting in the SMA being in its austenite phase upon deformation. For this case, the SMA undergoes a martensitic phase transformation to stress-induced detwinned martensite upon reaching the forward transformation stress. Since the stress-induced martensite is only stable due to the applied load, upon unloading, the material reverts back to its original austenite phase with little or no residual deformation and without requiring the application of heat. The martensite start, martensite finish, austenite start, and austenite finish temperatures can be altered through the manufacturing and processing of the SMA to provide either a SME specimen or a SE specimen for a desired temperature range (Duerig et al. 1990).

Superelastic shape memory alloys have shown particular promise in engineering applications because of the ability to recover their shape without the need for an external heating source. In general, superelastic SMAs provide several properties ideal for earthquake engineering applications as shown in Fig. 1. These properties include repeatable recentering capability, loading plateaus which limit force transfer to other members of the structure at intermediate strain levels, supplemental damping attributed to the flag-shape hysteresis, stiffening at large strain levels due to the formation of stress-induced martensite, and excellent low- and high-cycle fatigue properties. This unique behavior has led to the belief that SMAs can reliably be used to control the response of a structure during seismic events.

Previous Research of Shape Memory Alloys

Recent interest in the development of new technologies to limit interstory and residual drifts in civil engineering structures as a result of a seismic event has led to several numerical and experimental studies which have highlighted the possibility of utilizing SMAs as a promising innovative material for the dynamic control of buildings (Dolce et al. 2000; Baratta and Corbi 2002; Bruno and Valente 2002; Han et al. 2003; Fukuta et al. 2004; Ocel et al. 2004; Tamai et al. 2004) and bridges (Wilde et al. 2000; Li et al. 2004; Andrawes and DesRoches 2005). A large majority of these applications have implemented SMAs in passive control devices in order to gain the benefits of both supplemental damping attributed to the hysteretic behavior and the recentering capability unique to SMAs. Many of these studies have shown a wide variety of uses for SMAs which have provided promising results.

In bridge systems, Wilde et al. (2000) proposed a new isolation system which combined both laminated rubber bearings and SMA bars to provide both damping and isolation between the pier and the superstructure. Analytical results showed that the proposed isolation system provides a better performance in attenuating damage energy than the conventional lead–rubber systems. The use of superelastic SMA bars as restrainers was studied by Andrawes and DesRoches (2005) to prevent unseating of the bridge deck from the pier. The recentering capability and high fatigue-resistance of SMAs has led to studies on their use in cable stay bridge systems (Li et al. 2004). The investigation of a combined cable–SMA damper system showed an ability of the SMA damper to suppress cable vibration in dominant modes depending on the position of the damper along the cable.

Along with the use of SMAs in bridge systems, there have been a significant number of developments involving the use of SMAs in building structures. Bruno and Valente (2002) presented a comparative analysis between both typical rubber devices used for base isolation and those implementing SMAs which accounted for the occurrence of structural damage, non-structural damage, and damage to contents. The results showed that the SMA base isolation system was more effective in reducing structural vibrations. A more recent study by Masuda et al. (2004) analytically looked at isolation devices using SMA wires to provide energy dissipation.

As the use of SMAs in building systems has gained interest, more unique strategies taking advantage of their properties also have been conceived. The ability of partially restrained beam–column connections using SMA rods to limit interstory drift angles and maximum connection rotation have been considered by Ocel et al. (2004) and Taftali et al. (2004). Tamai et al. (2004) has suggested the use of SMA rods for anchoring column base plates in order to prevent deterioration of the restoring force due to large rocking displacements at the foot of building frame structures during an earthquake. Others have proposed the use of SMA rods in joint beams connecting reinforced concrete coupling walls to provide recentering and prevent large permanent deformations (Fukuta et al. 2004).

Besides isolation systems and those more innovative strategies presented, bracing systems have been one of the more common areas where the use of SMAs has been considered for frame structures. As a result of the Memory Alloys for New Seismic Isolation and Energy Dissipation Devices (MANSIDE) project, Dolce et al. (2000) developed SMA braces using martensitic and/or superelastic wires to provide energy dissipation and/or recentering. More recently, studies by Baratta and Corbi (2002) and Han et al. (2003) determined the effectiveness of SMA braces in a simple portal frame and a small scale two-story structure, respectively. Although many studies have focused on SMA isolation and bracing systems, detailed information on the extent of their ability to control structural response during an earthquake in a variety of building configurations is still lacking.

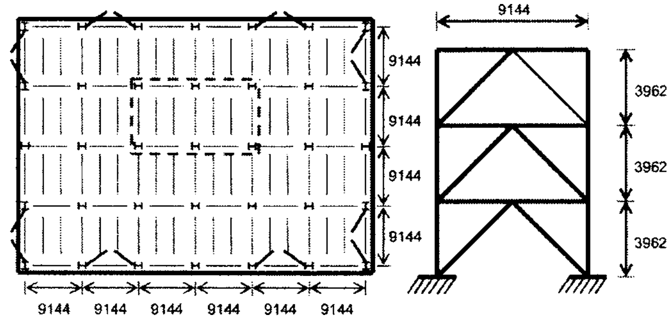

Fig. 2. Plan and elevated view of the three-story frames (mm) with the penthouse indicated with a dashed line

Analysis

The interest in concentrically braced frames as an alternative to steel moment frames and the need to better understand their behavior under a performance-based framework provide an opportunity to understand in detail the benefits of using SMA braces to improve their performance. The following section outlines an analytical study designed to highlight the benefits of SMAs in a concentrically-braced frame.

Frame Characteristics

Among the several steel braced frames presented by Sabelli (2001), two chevron (inverted V) braced buildings were selected to compare conventional steel braced systems with superelastic SMA braced systems. All of the structures considered in this study had designs governed by the 1994 Uniform Building Code and 1997 National Earthquake Hazard Reduction Program Recommended Provisions for Seismic Regulations for New Buildings and Other Structures for a building constructed in the Los Angeles area assuming a 10% probability of exceedence in 50 years ground motion. The two structures consisted of a three-story building and a six-story building. Both buildings are designed with traditional chevron braced bays along the perimeter to provide lateral load resistance during a seismic event. A total of four braced bays in each direction for the three-story building and six braced bays in each direction for the six-story building were incorporated into the design of the lateral load resisting system in order to prevent an increase in the braced design forces due to the reliability factor. A plan view of the three- and six-story buildings along with the braced bay dimensions corresponding to each are found in Figs. 2 and 3. Further details corresponding to the building design can be found in the work by Sabelli (2001) and Sabelli et al. (2003).

The chevron braced frames have spans of 9.14 m (30 ft) with story heights of 3.96 m (13 ft), except for the first story of the six-story frame which has a story height of 5.49 m (18 ft). The base of the structure is considered fixed. The members of the framed system are designed consistent with the AISC Load and Resistance Factor Design Code and Seismic Provisions for Structural Steel Buildings. The columns are continuous for the three-

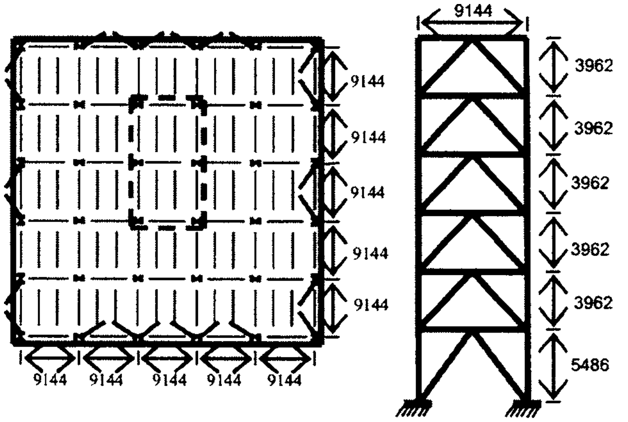

Fig. 3. Plan and elevated view of the six-story frame (mm) with the penthouse indicated with a dashed line

story frames and assumed spliced between the fourth and fifth floors for the six-story frames, whereas the beams are continuous across the 9.14 m (30 ft) span. Member sizes for the conventional steel braced systems are shown in Tables 1 and 2 for the three- and six-story frames, respectively. Because of the unbalanced load placed on the beams resulting from buckling of the compression brace in a chevron braced system, all beams, excluding the roof beam, were designed to account for the post-elastic behavior mode. Standard hollow-tube sections were used for all of the conventional steel braces.

The chevron braced frames with the SMA bracing system have the same beam and column design as the conventional steel braced system. The steel braces are replaced with superelastic SMA bar segments connected to the frame through rigid elements in order to reduce the length of SMA required as seen in Fig. 4. The SMA bars consist of a single diameter in the range of 12.7–31.75 mm (0.5–1.25 in.) and are connected to the bracing system to act in parallel with each other. A sufficient number of bars are used to provide the required cross section. Any deformation is thus transferred to the SMAs which, as a consequence, are the only structural elements able to sustain any significant axial strain. Although somewhat idealized, this is the general behavior for which the actual SMA bracing system would be designed. The

Table 1. Three-Story Buckling-Allowed Model Information

Story	Element size: Column	Beam	Brace
1	W12×96	W30×90	HSS 8×8×12
2	W12×96	W27×84	HSS 8×8×12
3	W12×96	W18×46	HSS 6×6×3/8

Table 2. Six-Story Buckling-Allowed Model Information

Story	Element size: Column	Beam	Brace
1	W14×211	W36×150	HSS 10×10×1/2
2	W14×211	W30×116	HSS 8×8×1/2
3	W14×211	W30×116	HSS 8×8×1/2
4	W14×211	W30×116	HSS 8×8×1/2
5	W14×211	W30×99	HSS 6×6×1/2
6	W14×211	W27×94	HSS 5×5×1/2

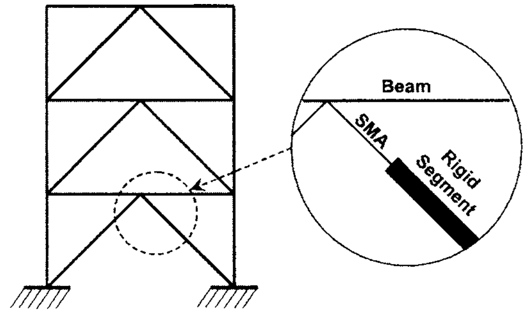

Fig. 4. Detail of the SMA brace installed in the three-story frame

SMA braces are modeled based on the uniaxial tests carried out by DesRoches et al. (2004), who studied the cyclic properties of large diameter superelastic SMA bars [see Fig. 1(b)]. Although these tests only looked at tensile loadings, past analytical studies have typically modeled SMA behavior as symmetric. Experimental studies of small SMA specimens conducted by those in the material science field have suggested an increase in the forward transformation stress occurs during compressive loadings with similar shape recovery still being obtained (Frick et al. 2004). The current model cannot account for this asymmetry providing an area of future improvement, but captures the general expected behavior of the SMA bars, albeit somewhat idealized. The material properties selected for the numerical simulations are those obtained from the dynamic tests, in order to correctly consider the reduced energy dissipation capability of such materials at high frequency loads (Dolce and Cardone 2001; DesRoches et al. 2004). The SMA braces are designed to provide the same initial axial stiffness and yield strength as the steel braces. As a result, both the conventional steel braced frames and the SMA braced frames have the same natural periods of 0.46 and 0.74 s for the three-story and six-story frames, respectively. Moreover, both the steel and SMA braces will yield at the same force level. Table 3 provides the length and cross-sectional area of the SMA elements. For this particular study and for comparison purposes, it is assumed that all SMA bars are encased in grouted tubes and are unbonded, allowing them to undergo compressive loads without buckling.

Analytical Model

In order to determine the dynamic performance of the braced frames, nonlinear dynamic time history analyses are carried out using the Open System for Earthquake Engineering Simulation (OpenSEES) (McKenna and Fenves 2004) analysis platform. As

Table 3. Geometrical Characteristics of SMA Elements Related to the Frames with Buckling-Allowed Braces

	Frame 3BA		Frame 6BA	
Story	Length (mm)	Area (mm^2)	Length (mm)	Area (mm^2)
1	504	5,259	595	6,701
2	504	5,259	504	5,259
3	504	2,953	504	5,259
4	—	—	504	5,259
5	—	—	504	3,795
6	—	—	504	3,070

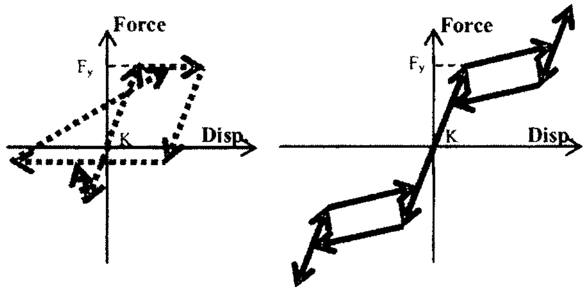

Fig. 5. Force–displacement relationship of: (a) buckling-allowed steel elements; (b) SMA members

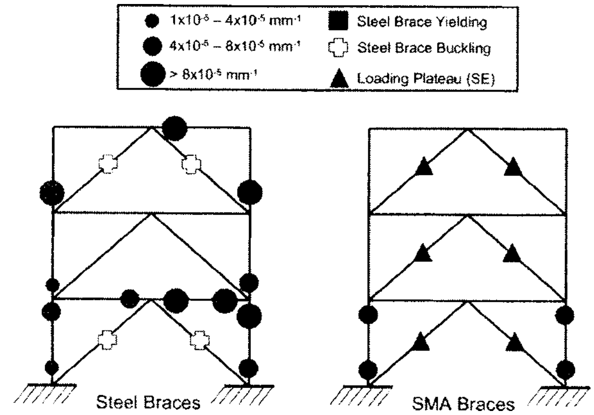

Fig. 6. Member yielding during the LA16 ground motion for the steel and SMA braced frames

suggested by the symmetry of the structures, only one braced bay is modeled. The floor masses used in the analyses to account for horizontally acting inertial forces are taken as the total mass of each floor divided by the number of braced bays in the lateral load resisting frame for each principal direction.

Beams and columns are modeled using elements with fiber sections. Although the frames themselves were not designed as moment resisting, fixed connections are assumed at points where gusset plates are attached at both the top and bottom of the beam providing some fixity to the connection. This is done, in part, for consistency purposes since this is the same approach taken by Sabelli (2001) and Sabelli et al. (2003) during the study of these same frames. Hinge connections are assumed at the roof level beam and column connection since no gusset plates are present. Braces are pinned at both ends so as to impose axial loads only. Global P–Δ effects are taken into consideration, but for the purpose of this analysis any added lateral stiffness from the gravity load frame is not considered. As commonly done for code-designed steel structures (Sabelli 2001; Sabelli et al. 2003), a damping coefficient of 5% is assumed.

In order to model the material properties of the beams and columns, a typical uniaxial bilinear force–deformation relationship was considered. To model the behavior of the braces, an accurate hysteretic model accounting for strength and stiffness degradation, negative post-buckling tangent stiffness, and pinching of the hysteresis was implemented into OpenSEES [Fig. 5(a)]. This model is similar to that which was used by Sabelli (2001) and represents typical brace behavior where buckling is allowed, but it does not account for the onset of brace fracture. Mechanical properties of the structural steel are provided in the work by Sabelli (2001) and Sabelli et al. (2003).

The superelastic behavior of the SMA braces is simulated using a modified (Fugazza 2003) one-dimensional constitutive model proposed by Auricchio and Sacco (1997) as can be seen in Fig. 5(b). Its formulation, developed in the small deformation regime, relies upon the assumption that the relationship between stresses and strains is represented by a series of straight lines, whose extension depends on the transformation experienced. In agreement with approaches taken by previous studies, further assumptions are that no strength degradation occurs during cycling and that the austenite and martensite branches have the same Young's modulus (Bruno and Valente 2002; Andrawes et al. 2004). Although strength degradation does occur during initial cycling, studies of single and multiple degree-of-freedom systems by Andrawes (2005) suggest that this has little effect on the overall performance of the structural system. Further, it can be assumed that the bars are previously mechanically trained in order to minimize any property degradation due to cycling (McCormick et al. 2005). Details in regards to the model formulation and the integration technique can be found in the work by Fugazza (2003).

Earthquake Ground Motions

Forty ground motion records were used to analyze the behavior of both the conventional braced steel frames and the SMA braced frame systems. These ground motions consisted of two suites of 20 ground motions corresponding to a seismic hazard level of 2 and 10% probability of exceedence in 50 years, respectively. The ground motions were developed from both historical records and from simulations for the Los Angeles area as part of the FEMA/SAC Steel Project study on steel moment-resisting frames (Somerville et al. 1997). No further scaling of the ground motions was conducted for this analysis in order to better gauge the response of the two systems under different spectral accelerations at the fundamental period of the frames.

Case Study: Three-Story Braced Frame

In order to determine the effects of using a SMA bracing system, a detailed comparative study of the three-story braced frame with conventional steel braces or SMA braces is conducted for a single ground motion. The 6.7 magnitude 1994 Rinaldi RS Northridge ground motion (LA16) is chosen for this case study which is part of the suite consisting of a seismic hazard level of 10% probability of exceedence in 50 years. The spectral acceleration associated with the first deformation mode for both of the braced three-story frames is approximately 1.11 g, which is slightly less than the average spectral acceleration imposed by each individual ground motion of the 10% probability of exceedence ground motion suite used in this paper. This case study is meant to provide an understanding of the behavior of the two systems during a typical earthquake. As the behavior of the frame structures is highly ground motion dependent, a more precise comparison of the two systems with respect to the response of the structures to the two suites of ground motions follows.

The effect of using a SMA bracing system on the other members of the braced frame is shown in Fig. 6, where column and beam members which have undergone yielding during the ground motion are depicted as solid dots with the size of the dots representing the extent of the yielding based on the curvature. The conventional steel braced frame shows yield points at every floor level along the continuous columns due to nonuniform interstory

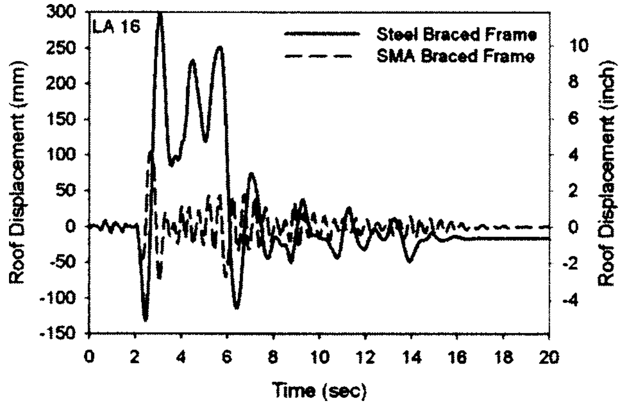

Fig. 7. Displacement time histories of the top floor when the three-story building undergoes the LA16 earthquake

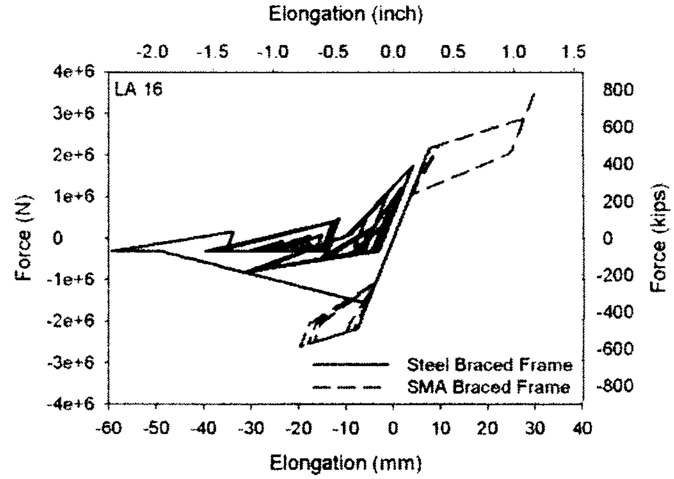

Fig. 8. Force–displacement relationship exhibited by the left brace of the first floor when the three-story building is hit by the LA16 earthquake

drifts along the height of the structure. The first and third floor braces experience buckling under the uniaxial compressive loading suggesting large permanent frame displacements. The center of the first floor and third floor beams also experienced yielding as a result of vertical displacement of the beam due to the chevron brace configuration and the unbalanced load associated with brace buckling, while the second floor experiences only a relatively small amount of damage. For the case of the SMA braced frame, yielding occurs at the base and first floor level of the continuous columns with all other large deformation being concentrated in the uniaxial deformation of the SMA braces. The ability of more members in the SMA braced frame to remain elastic can be attributed to the increased stiffness of the SMA braces and a more uniform distribution of the interstory drifts along the height of the structure. The loss of brace capacity due to buckling and the onset of permanent deformation in tension of the steel braces can result in large demands placed on the other members of the frame system.

In order to examine the response of the overall structure for the two bracing systems, the story drifts and residual drifts are compared. Fig. 7 provides the time history for the top floor displacement of both frames. The peak roof displacements for the steel braced and SMA braced system are approximately 299 and 106 mm (11.8 and 4.2 in.), respectively, representing roof drifts of 2.5 and 0.89%. The large drifts associated with the steel structure can be attributed to the buckling of the steel braces and the onset of permanent deformation in the columns as was discussed previously. The stiffening of the SMA braces and recentering capability decreased the peak roof displacement by approximately 65%. The use of conventional steel braces resulted in a residual roof displacement of approximately 16 mm (0.63 in.), whereas the SMA braced system only had 0.10 mm (0.004 in..) of residual displacement. The ability of superelastic SMA elements to recenter the system strongly reduces the permanent deformation of the frame system.

As suggested by Sabelli et al. (2003), a further measure of the potential for concentrated damage in a given floor can be obtained from the column drift ratio. The column drift ratio refers to the difference in drift values (relative displacement divided by the story height) for adjacent floors where larger values result in higher flexural demands and larger column rotations. The higher the value of the column drift ratio, the more the continuous column needs to bend at a given floor level. The three-story conventional steel braced structure had a large column drift ratio of 6.68% as compared to the SMA braced structure which had a column drift ratio of only 0.58%. The large drift ratio for the steel braced structure suggests a significant amount of rotation at the floor level and the possibility of higher mode responses. The column drift ratio for the SMA braced structure implies that a more uniform distribution of the interstory drifts occurs along the height of the structure.

As the beam and column members are the same in both of the frames and the yield force and initial stiffness of both types of bracing systems are constant, the difference in the response must be attributed to the different behavior of the steel braces as compared to the superelastic SMA braces. The uniaxial force-displacement plots for one of the first floor braces in each braced system are shown in Fig. 8. Large deformations occur in the steel braces after buckling, 60 mm (2.4 in.), whereas the SMA members allow for much lower displacements, 30 mm (1.2 in.), due to the recentering capability guaranteed by the superelastic effect. The ability of the superelastic SMA bracing system to provide recentering, undergo compressive loadings, and stiffen at large strains results in a decrease in maximum and residual drifts. Further, a comparison of the hysteretic loops of the two types of braces suggests that the supplemental damping provided by the SMAs is comparable to the postbuckling energy dissipation provided by the steel braces as a result of the reduced load capacity of the steel braces.

Results and Discussion

A more comprehensive study of both the three-story and six-story frames is conducted to verify that the findings of the case study are consistent over the suite of ground motions. It is necessary to understand the response of the conventional steel bracing system and the SMA bracing system relative to each other and the effect that the different seismic hazard levels have on the bracing systems performance in order to have confidence in the predicted behavior of these systems in real world structures. Attention is focused both on the maximum interstory drift and the residual roof displacement, two quantities traditionally considered to judge the seismic performance of structures undergoing dynamic loads (Hamburger et al. 2003). The average maximum interstory and residual drifts for each floor level will also be considered to

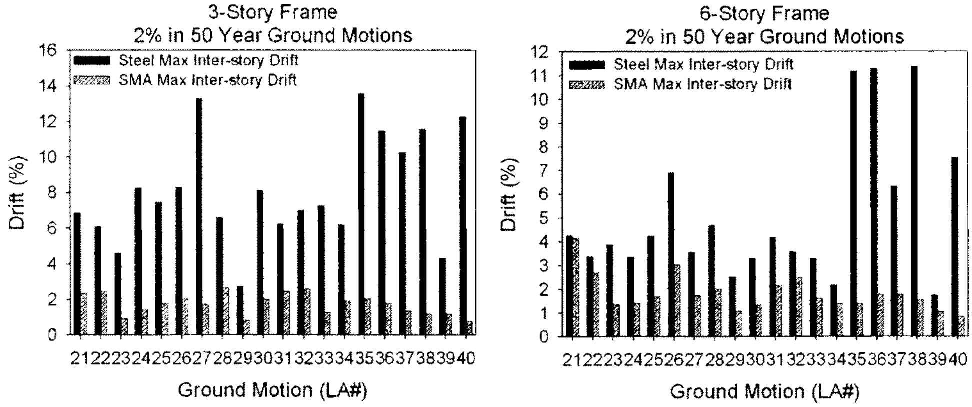

Fig. 9. Maximum interstory drift of the three- and six-story frames for the 2% probability of exceedence in 50 years ground motions

gain an understanding of which floor levels are placed under the highest demands for the given bracing systems.

The maximum interstory drift and residual roof displacement provide good measures of the performance of a structure during an earthquake and the resulting damage to both structural and non-structural components. Figs. 9 and 10 show the maximum interstory drift and residual drifts for the three- and six-story frames with either conventional steel braces or the superelastic SMA braces for the 2% probability of exceedence in 50 years suite of ground motions. In all cases, the maximum interstory drift was lower for the SMA braced structure compared with the steel braced structure. It should be noted that for some ground motions, the conventional steel braced frame underwent significantly large interstory drifts, suggesting the possibility of stability problems associated with P–Δ effects. As these frames were designed based on 10% probability of exceedence in 50 year ground motions, the large drifts as a result of the higher seismic hazard are not surprising. The average peak interstory drifts for the steel braced structures were 8.1 and 4.7% for the three- and six-story frames, whereas the average maximum interstory drifts for the SMA braced structures were considerably less, 1.8 and 1.9%. On average, use of SMA braces reduced the maximum interstory drift as compared to the use of conventional steel braces by approximately 76 and 56% for the three- and six-story frames, respectively. The results suggest that the ability of the SMA brace hysteresis to pass through the origin during cycling while undergoing large deformations reduces the maximum interstory drift and subsequently the demand on the column members at each floor level.

The use of SMA braces also reduces the residual roof displacement in almost all cases as shown in Fig. 10. On average, this reduction was approximately 91% for both the three- and six-story structures as compared to the residual roof displacements of the steel braced frames. The recentering capability of the SMAs thus reduces inelastic deformation in the columns of the braced frames for both the three- and six-story structures. For both the three- and six-story structure, it is important to note that the LA32 ground motion resulted in larger residual roof displacements in the SMA braced system as compared to the steel braced system suggesting that certain ground motion parameters may have an effect on the performance of the SMA bracing system. Although the steel braced system did show lower residual roof drifts for a small number of the cases, the average residual roof drift for the steel braced structures was 1.8 and 0.57%, whereas the SMA braced structure had residual roof drift values of 0.15 and 0.2% for the three- and six-story frames, respectively.

Table 4 provides the mean and standard deviation values of the maximum interstory drift and residual roof drift for the four structures studied with respect to the seismic hazard level. The results show a consistent behavior of the SMA braces with respect to seismic hazard and structure height. The mean value and standard deviation of the maximum interstory drift and residual roof drift increase slightly between the 10% in 50 years and 2% in 50 years ground motions as is expected with the increased seismic hazard.

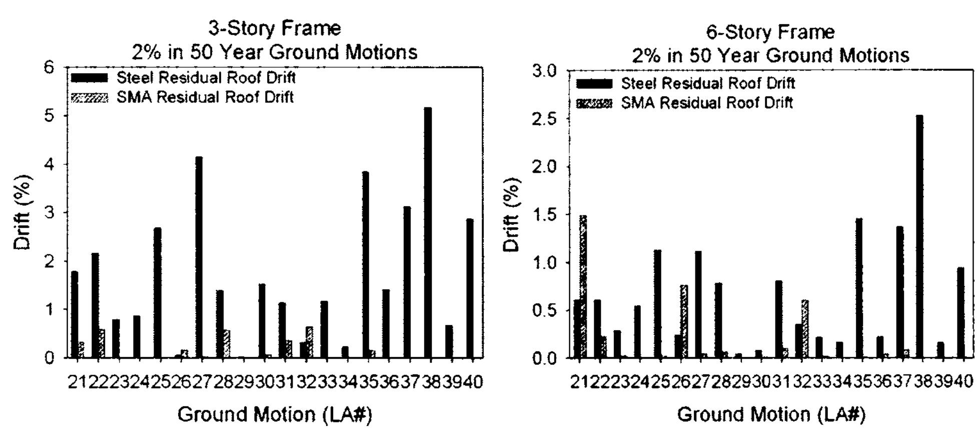

Fig. 10. Residual roof drift of the three- and six-story frames for the 2% probability of exceedence in 50 years ground motions

Table 4. Mean and Standard Deviation (between Brackets) of Maximum Interstory Drift and Residual Drift of the Top Floor for the 2 and 10% Probability of Exceedence in 50 Years Records

	Maximum interstory drift				Residual drift			
	2% in 50 years		10% in 50 years		2% in 50 years		10% in 50 years	
	Three-story (%)	Six-story (%)	Three-story (%)	Six-story (%)	Three-story (%)	Six-story (%)	Three-story (%)	Six-story (%)
SMA	1.75	1.90	0.99	0.91	0.15	0.20	0.004	0.006
	(0.60)	(0.78)	(0.40)	(0.31)	(0.23)	(0.39)	(0.005)	(0.006)
Steel	8.13	4.67	3.57	1.97	1.78	0.57	0.25	0.12
	(3.03)	(2.70)	(1.61)	(0.68)	(1.45)	(0.45)	(0.33)	(0.12)

In all instances, the mean maximum interstory drift and residual roof drift decreased with the use of the SMA braces. Based on the results, conventional steel braces perform better in the six-story building as compared with the three-story building where both the maximum interstory drifts and residual roof drifts were smaller for the six-story structures. This occurrence can most likely be attributed to the smaller demands placed on each brace due to the inherent flexibility of taller buildings resulting in a smaller percentage of the conventional steel braces undergoing buckling. The difference in the performance of the SMA braces and conventional steel braces also tended to decrease with the increase in building height.

A comparison of the average maximum and residual interstory drifts with respect to story level can be seen in Fig. 11 for the six-story steel and SMA braced frames undergoing the 2% in 50 years ground motions. The conventional steel braced frame shows large structural demands placed on the first and sixth stories of the structure due to local bending action in the columns as a result of large interstory drifts. The demand on the SMA braced structure remains almost constant over the frame height. Most of the residual drift for the conventional steel braced frame can be attributed to deformation in the first story, whereas the SMA braced structure showed almost equal permanent displacement at each story level. These findings suggest that the most economical bracing system may be one which takes advantage of the recentering behavior of SMA braces for the first and second floors and uses conventional braces for each subsequent floor. Although, the large interstory drift at the first floor level may also be a function of the larger story height and the use of a combined SMA and conventional steel bracing system may result in a shift in where the deformation demands are largest. Studies are currently being undertaken to address these issues and explore these options. Further, in order to make such a mixed system viable, design standards must also be developed in order to prevent the formation of undesirable deformation modes.

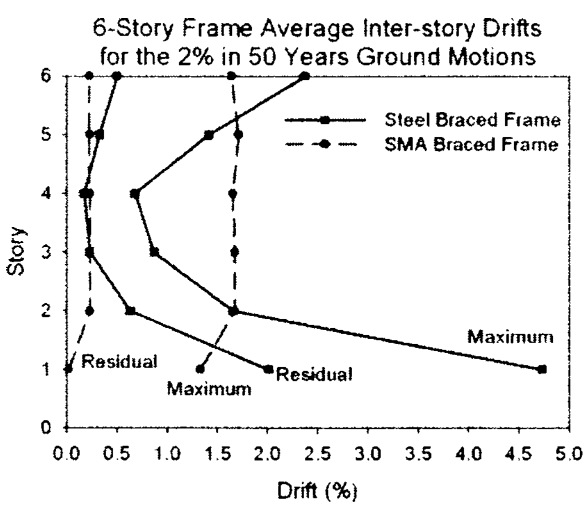

Fig. 11. Maximum and residual interstory drifts with respect to each floor level for the six-story frame undergoing the 2% probability of exceedence ground motions

Conclusions

This study involved an extensive analysis of both three- and six-story concentrically braced frame systems implementing either conventional steel braces or superelastic shape memory alloy braces. Through nonlinear dynamic time history analyses, the effect of the unique flag shape and recentering behavior of the SMA braces on the performance of the concentrically braced frames was evaluated and compared to the performance of the conventional steel bracing system with respect to interstory drift levels and residual roof drift. In order to accurately compare the two systems, the SMA braces are designed such that the yield force and initial stiffness is equal to that of the conventional steel braced system resulting in the same natural period for the two three-story frames and the two six-story frames, respectively. The major results from this study are summarized below:

- Column drift ratios for the conventional steel braced frame are significantly higher than those for the SMA braced frame suggesting much larger flexural demands on the continuous column members.
- The ability of the SMA braces to provide recentering leads to smaller overall elongation in the bracing members. Also, the buckling behavior of the steel braces in compression contributes to the large permanent deformations seen in the conventional steel braces due to a decrease in load carrying capacity in compression.
- For both the three- and six-story frames, the use of SMA braces on average resulted in significantly smaller maximum interstory drift values and residual roof drift values for both the 2 and 10% probability of exceedence in 50 years ground motions. Although, the difference in the maximum interstory drift and residual drift values decreased with an increase in story height suggesting that the recentering capability of the SMA braces is most effective for shorter buildings.
- The study of the mean interstory drifts of the six-story frame shows that a large percentage of the overall residual drift in the conventional steel braced frame occurred in the first and second stories. The results for the upper stories are equivalent to those experienced by the SMA braced frame suggesting that

SMA braces may be most beneficial in the lower levels of tall buildings while conventional steel braces can still be used in the upper stories. Further studies are currently being completed pursuant with this conclusion.

The results of this study suggest promise for implementing new, innovative materials in concentrically braced frames in order to address the lack of ductility and energy dissipation in traditional concentrically braced frames during past earthquakes. Further work still is necessary in order to verify experimentally these analytical results, while future work will also consist of developing more accurate models based on experimental testing of actual SMA systems incorporating large diameter bars. The capacity of such systems must also be studied in order to gage limit states in conjunction with performance-based design techniques.

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