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DEVELOPMENT OF SELF-CENTERING STEEL MOMENT-RESISTING FRAMES FOR DAMAGE-FREE SEISMIC RESISTANT BUILDINGS

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ABSTRACT

Self-centering steel moment resisting frames (SC-MRFs) for seismic resistant buildings have been developed in the past decade as an alternative to conventional welded steel moment-resisting frames (WMRFs). While conventional WMRFs are expected to sustain cyclic inelastic deformations in critical regions of the beams during a moderate or severe earthquake, SC-MRF systems are designed to exhibit similar stiffness and strength as WMRFs and provide stable energy dissipation, while sustaining only minor damage to structural elements and essentially no residual drift under the design basis earthquake. In this paper, the concepts and mechanics of an SC-MRF are presented, followed by an overview of the experimental and numerical research that has been conducted to develop and validate SC-MRFs. The main issues that are central to the seismic design and performance of these SC-MRFs are discussed. Finally, critical aspects related to the practical implementation of SC-MRFs are outlined and a discussion of the steps that are still needed for practical implementation is presented.

Introduction

Recent major earthquakes (e.g., 1994 Northridge Earthquake, 1995 Kobe Earthquake, and 1999 Chi-Chi Earthquake) have shown that improved steel beam-to-column moment connections are needed [Ricles et al. 2002a, Roeder 2002]. Conventional welded steel moment-resisting frames (WMRFs) are expected to sustain cyclic inelastic deformation in critical regions of members during a moderate or severe earthquake. This damage can result in significant structural damage and residual drift after the earthquake. This residual drift contributes significantly to total building losses from earthquakes. Miranda [2009] found that 50% of buildings were demolished following major earthquakes because of excessive residual drifts. Choi et al. [2009] reported on a comprehensive study of the nonlinear response of steel building frames ranging from 2 to 12 stories designed according to ASCE-7 [2005] subjected to the design basis earthquake (DBE) ground motions. The results of the study indicate that these structures are expected to be unusable because of residual drift which is expected to be consistently greater than 0.5%.

To avoid this damage and residual drift, post-tensioned beam-column connections for self-centering steel moment-resisting frames (SC-MRFs) have been developed. SC-MRF systems

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are designed to exhibit similar stiffness and strength as conventional WMRFs and provide stable energy dissipation, while sustaining only minor damage to structural elements and essentially no residual drift. SC-MRF systems use post-tensioned steel strands or bars to provide a clamping force to connect the beams to the columns instead of welded beam-column joints. The behavior of these connections is characterized by gap opening and closing at the beam-column interface. Energy dissipation occurs in special devices designed for the beam-column connection regions.

In this paper, the basic concept and mechanics of an SC-MRF is first presented, followed by an overview of the experimental and numerical research that has been conducted to develop and validate SC-MRFs. The main issues that are central to the seismic design and performance of these SC-MRFs are then discussed. Finally, critical aspects related to the practical implementation of SC-MRFs are outlined and a discussion of the steps that are still needed for practical implementation is presented.

Concept and Mechanics of SC-MRFs

Fig. 1 shows a schematic elevation of an SC-MRF. An SC-MRF has post-tensioned (PT) steel beam-to-column moment connections, where high-strength steel PT strands or bars run parallel with the beams across multiple bays and are anchored as shown in Fig. 1. Energy dissipation (ED) devices are placed in the PT connections to provide energy dissipation. Together, the PT strands and the energy dissipation devices provide the flexural resistance of the connections. As noted above, the behavior of a PT connection is characterized by gap opening and closing at the beam-column interface. The gap opening results in a softening of the lateral force-lateral drift behavior of the SC-MRF, thereby limiting the level of forces in the beams and columns of the SC-MRF. These members are designed to resist these forces elastically.

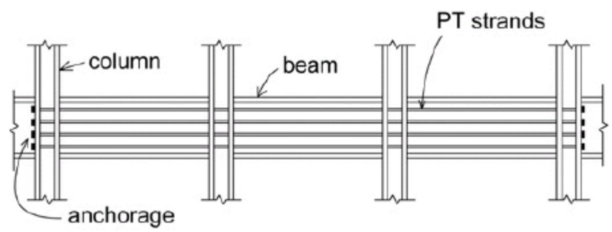

Figure 1. Schematic elevation of a self-centering post-tensioned frame

Many types of energy dissipation devices have been used, including bolted top and seat angles [Garlock et al. 2005], confined energy dissipating bars yielding in tension and compression [Christopoulos et al. 2002b], and steel plates yielding in tension and compression [Chou et al. 2006]. These energy dissipation devices were attached to both the top and bottom flanges of the beams. Rojas et al. [2005] studied a PT friction damped connection, with friction devices located along the top and bottom flanges of the beams. Wolski et al. [2009] studied another friction device, called the bottom flange friction device (BFFD), which is attached to only the beam bottom flange to avoid interference with the floor slab, and to make it easy to repair. Kim and Christopoulos [2008] proposed a bolt-prestressed friction mechanism with a frictional interface consisting of stainless steel and nonasbestos organic break-lining pads connected to the top and bottom flanges to dissipate seismic energy. Fig. 2 shows at PT beam-to-column connection with a web friction device (WFD) for energy dissipation. Brass plates are sandwiched between the friction channels and beam to provide reliable friction. Slotted holes are used in the beam web to accommodate the travel of the friction bolts during the gap opening and closing of the connection. More details of this connection can be found in [Lin et al. 2009].

As noted above, the behavior of a PT connection is characterized by a gap opening at the beam-column interface. The conceptual moment-relative rotation (M-θ_r) behavior of a WFD connection shown in Fig. 2(a) is shown in Fig. 2(b). Initially the connection has stiffness similar to that of a conventional welded moment connection. After the connection moment M reaches the imminent gap opening moment at event 2 (M_{IGO}), the beam tension flange loses contact with the shim plate at the column face and gap opening occurs. M then increases with increases in the PT strand force due to gap opening (event 2 to 3). Excessive gap opening will eventually yield the PT strands at event 4. Unloading between events 5 and 6 reduces θ_r to zero as the beam tension flange comes in contact with the shim plate at the column face. Further unloading (between events 6 and 7) decreases M to zero as the beam tension flange fully compresses against the shim plate. The connections with the other energy dissipation devices have similar hysteretic behaviour [e.g. Ricles et al. 2001, Christopoulos et al. 2001].

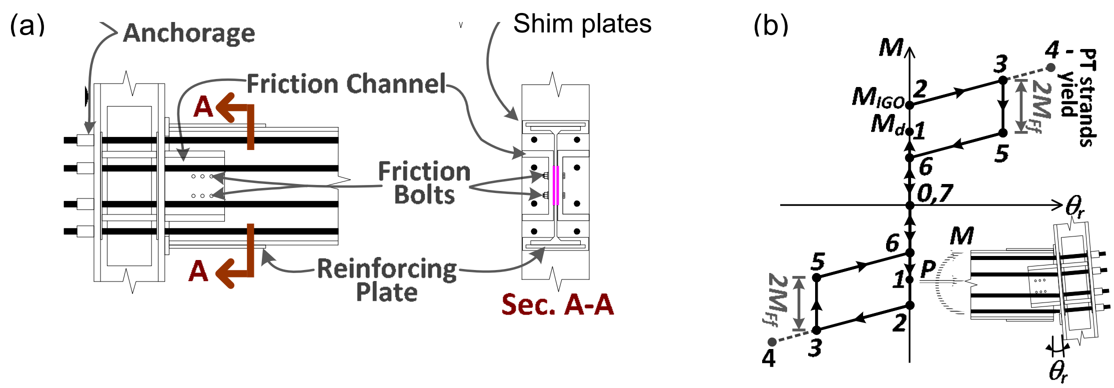

Figure 2. PT connection with WFD: (a) details, and (b) conceptual M-θ_r behavior

After M_{IGO}, M is equal to Pd_2+F_fr. P is the axial force in the beam, F_f is the force in the energy dissipating device, d_2 is the distance from the beam cross section centroid to the center of rotation (COR) of the connection, and r is the distance from F_f to the COR. The COR is at the point of contact of the beam compression flange with the column. P includes the PT force, T, and the effects of interaction between the SC-MRF and floor system (e.g., floor diaphragm forces) [Garlock et al. 2007a, Garlock and Li 2008]. T depends on the initial PT force, T_o, and the strand elongation due to θ_r.

Gap opening at the connections after decompression causes an SC-MRF to "expand" as shown, for a four-bay frame in Fig. 3. In the deformed position, the distance between the column centerlines is larger than in the original undeformed position due to connection gap opening. The expansion of an SC-MRF must be accommodated by the floor system and the collector elements that transmit inertial forces from the floor system to the PT frame. In addition, the PT frame must be designed to accommodate the forces that develop as the floor system partially restrains the expansion. This effect is more pronounced on the first floor where the fixity of the base columns increases the restraint to this frame expansion. Christopoulos [2002] and Garlock and Li [2008] proposed methods to estimate the increased axial compression in beams that results from restraint of the frame expansion.

Garlock et al. [2007a,b] and Garlock and Li [2008] developed a floor diaphragm system consisting of composite steel beam throughout most of the floor (see Fig. 4(a)). A system of non-composite flexible collector beams is used near the perimeter SC-MRFs to transfer the inertia forces into these frames while allowing the PT connections to develop gap opening. Kim and Christopoulos [2009a] and Garlock and Li [2008] proposed sliding details along the boundaries of the slabs that allow for the gap openings to be accommodated. A similar system was studied by Lin et al. [2009] where the floor diaphragm system consists of a composite floor slab attached to one bay of each SC-MRF in each perimeter frame to avoid restraining gap opening of the SC-MRF connections (see Fig. 4(b)).

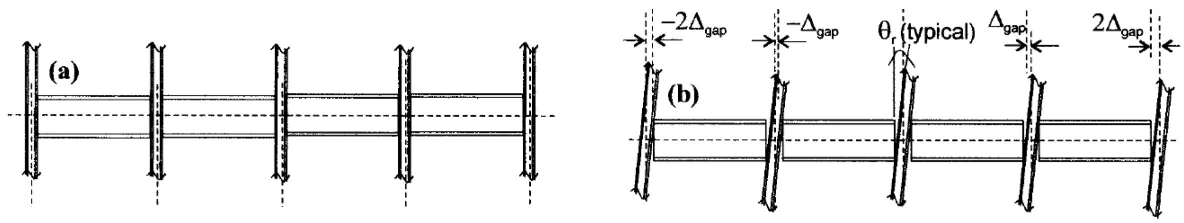

Figure 3. Elevation of one floor of a SC-MRF: (a) undeformed, (b) deformed configurations.

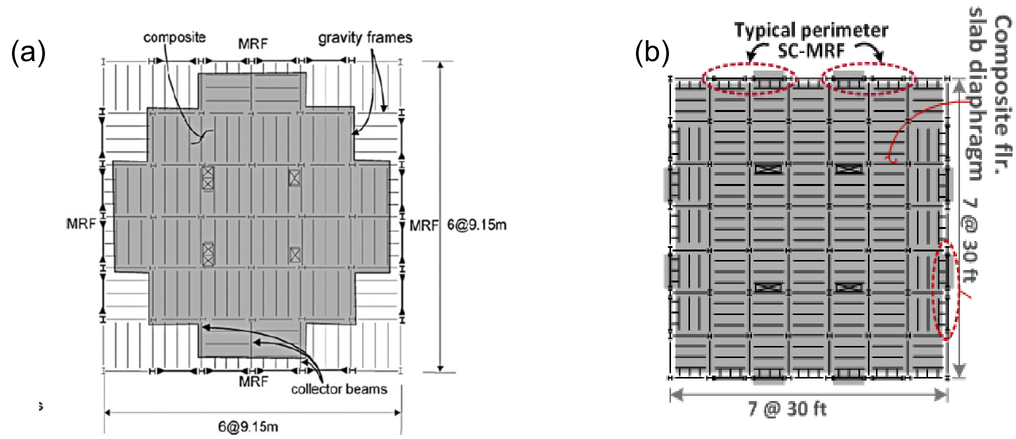

Figure 4. Floor diaphragm systems for buildings with perimeter SC-MRFs: (a) composite floor slab with flexible collector beams; (b) composite floor slab attached to one bay of each SC-MRF

Performance-Based Design of SC-MRFs

A performance-based design (PBD) procedure proposed for SC-MRFs [Garlock et al. 2007a] considers two levels of seismic input, namely the DBE and maximum considered earthquake (MCE). The DBE has two-thirds the intensity of the MCE [FEMA 2000] and an approximate 10% probability of being exceeded in 50 years. The MCE has a 2% probability of being exceeded in 50 years. Under the DBE, an SC-MRF is designed to achieve immediate occupancy (IO) performance [FEMA 2000], with limited structural and nonstructural damage. Under the MCE, an SC-MRF is designed to achieve collapse prevention (CP) performance [FEMA 2000]. Fig. 5 shows the design objectives and limit states of an SC-MRF with WFDs in a conceptual base shear-roof drift (V-θ_{roof}) response. The design objectives are similar for SC-MRFs with other energy dissipating devices [e.g., Garlock et al. 2007a].

The PBD procedure permits the use of an equivalent lateral force analysis of the SC-MRF, using an analysis model with rigid beam-column connections that is subjected to the design forces defined in ASCE7-05 [2005] with a response modification factor R equal to 8. The design moment M_{des} from this analysis is used to establish an initial value of M_{IGO} approximately equal to $0.95M_{des}$. The effective energy dissipation ratio of the SC-MRF connections, $\beta_E = M_{Ff}/M_{IGO}$, is used to establish the proportion of M_{IGO} provided by M_{Ff}. To provide the SC-MRF with sufficient energy dissipation to achieve a satisfactory seismic response, Seo and Sause [2005] recommend that $\beta_E \geq 0.25$, however to enable connection re-centering, $\beta_E \leq 0.40$. The PBD procedure includes numerous steps to control the limit states shown in Fig. 5. In this procedure, estimates of the θ_r demand under the DBE and MCE are critical for determining whether these limit states are reached. The details of the PBD procedure are given in Garlock et al. [2007a].

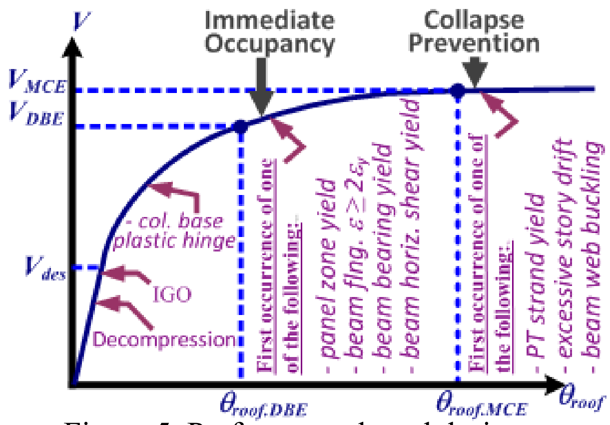

Figure 5. Performance based design objectives.

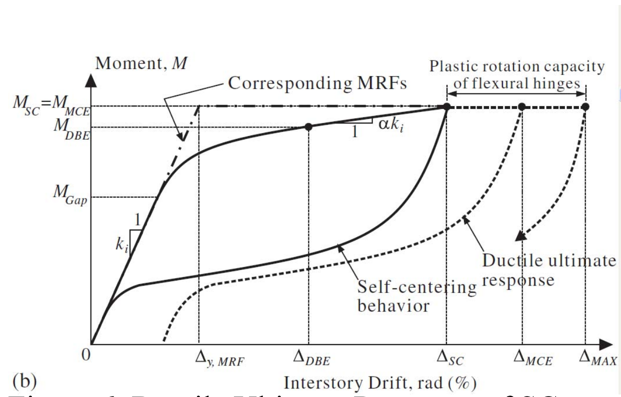

Figure 6. Ductile Ultimate Response of SC Steel Frames through Beam Flexural Yielding.

Kim and Christopoulos [2009a] propose an alternate design procedure that results in SC-MRFs with maximum story drifts similar to those of WMRFs. This procedure is based on results from a study by Christopoulos et al. [2002a] on the seismic response of SDOF self-centering systems that showed that despite their reduced energy dissipation when compared to full elastoplastic systems, they sustain similar maximum deformations when designed for the same target strength as elastoplastic systems. This procedure aims to design an SC-MRF that responds in the self-centering range with no damage and minimal residual drift under DBE level earthquakes, but allows the SC-MRF to form ductile mechanisms (thus eliminating the possibility of PT steel yielding or web buckling) under larger earthquakes with significant inelastic deformation reserve through flexural yielding of the beams at carefully detailed locations. Fig. 6 illustrates the intended response of an SC-MRF designed according to the proposed procedure. Kim and Christopoulos [2009b] proposed detailing requirements for beam webs to ensure that flexural hinging in the beams can occur prior to beam web buckling.

Experimental Studies of SC-MRFs

PT seismic-resistant steel connections were conceived in the late 1990's [Garlock et al. 1998, Christopoulos et al. 2001]. The first experiments performed on PT steel connections were presented by Ricles et al. [2002b] and Christopoulos et al. [2002b]. These test results provided experimental verification of the PT connection concept with controlled gap opening and closing under cyclic loading. A series of full-scale tests were subsequently performed by Garlock et al. [2005] to evaluate the PT connection concept at full scale. The PT connections studied by Garlock et al. [2005] used angles for energy dissipation devices, while the connections studied by Christopoulos et al. [2002b] used confined energy dissipating bars designed to yield in tension and compression. Chou et al. [2006] conducted experiments on PT connections with steel plates yielding in tension and compression to provide energy dissipation. These energy dissipation devices were attached to both the top and bottom flanges of the beam. Wolski et al. [2009] performed experiments on a PT-connection with a bottom flange friction device (BFFD). The BFFD was attached only to the beam bottom flange to avoid interference with the floor slab. Recent tests by Kim and Christopoulos (2009b] validated new PT self-centering friction damped (SCFR) steel moment-resisting (interior and exterior) full-scale connections. The test results showed that self-centering moment connections with the proposed friction dampers were capable of developing stiffness and strength similar to that of welded connections. The test specimens were tested twice to demonstrate that the cyclic response of the second run was, for all practical purposes, identical to those of the first run (see Fig. 7). This indicated that the proposed system is capable of a full recovery of strength, stiffness, and ductility by simply de-stressing and prestressing the bolts in the friction devices. When tested beyond the self-centering limit (i.e., beyond the expected MCE level), the proposed connections were shown to exhibit a ductile response with the formation of flexural hinges in the beams, thus avoiding the sudden loss of strength and stiffness that occurs when the post-tensioning elements are overloaded or when the beams buckle under excessive combined axial loads and bending.

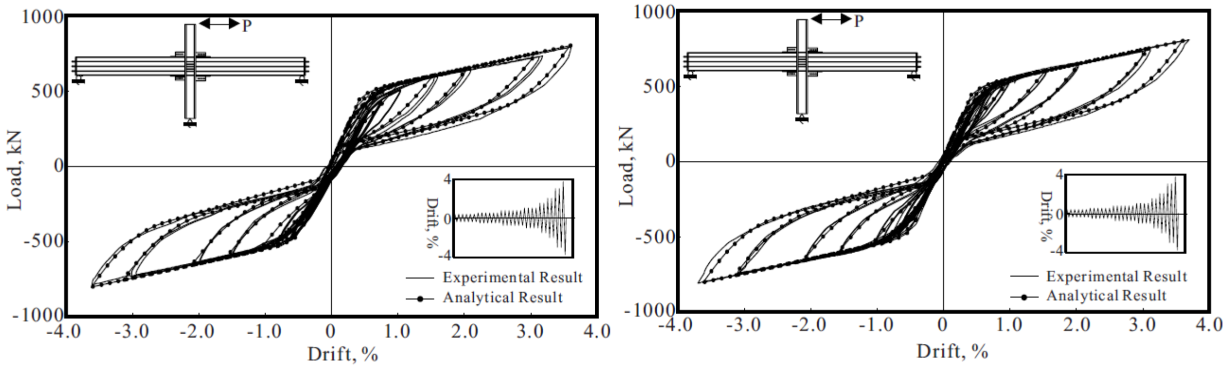

Figure 7: Full-Scale testing of interior SCFR frame (1st run (left), 2nd run (right).

Lin et al. [2009] conducted experiments on an SC-MRF *system* (i.e. including the floor diaphragm). The PBD procedure described above was used to design SC-MRFs with WFDs for the 7x7-bay 4-story prototype building shown in plan in Fig. 8(a). The building is located on a stiff soil site in the Los Angeles area. Each perimeter frame has two 2-bay SC-MRFs with WFDs. Fig. 8(a) shows the floor diaphragm is attached to one bay of each SC-MRF in each perimeter frame to avoid restraining gap opening of the SC-MRF connections.

The test frame was a 0.6-scale model of one SC-MRF from the prototype building, as shown in Fig. 8(b). The test frame has A992 steel members which were scaled down from the prototype SC-MRF. The DBE design demands for roof drift (θ_{roof}), story drift (θ_s), and connection relative rotation (θ_r) are 0.026, 0.039, and 0.031 rads, respectively. The MCE design demands are 0.039, 0.059, and 0.047 rads., respectively. The initial PT forces T_o, see Fig. 8(b), are less than 45% of strand ultimate strength (T_u). Design values of β_E are shown in Fig. 8(b).

A series of earthquake simulations were performed on the test frame. Ground motions at the frequently occurring earthquake (FOE), DBE, and MCE level were used. During the simulations lateral force was applied at each level through a simulated floor diaphragm attached near the middle of the North-bay beam by a hydraulic actuator as shown in Fig. 8(b). The hybrid simulation method was used for the simulations, in which the test frame was the experimental substructure and the gravity load bearing system, gravity loads, and the seismic mass tributary to the test frame were included in the analytical substructure. The explicit unconditionally stable CR integration algorithm [Chen and Ricles 2008] was used to solve the equations of motion.

Fig. 9(a) shows floor displacement time histories from a DBE level simulation. The maximum residual story drift is 0.00074 rads. in the first story, which demonstrates the self-centering capability of the SC-MRF. This residual drift is due to yielding in the columns at the ground level. Table 1 shows the maximum story drift during the DBE simulation $\theta_{s\,max}$ is 3.9% rads., which equals the design demand for the DBE given earlier. The maximum $\theta_{r\,max}$ is 3.8%

rads., which is slightly larger than the design demand of 3.1% rads. for the DBE. The maximum PT force is $0.65T_u$, where T_u is the PT strand tensile strength. No yielding occurred in the PT strands. The change in PT force, ΔT, is less than 1.5% of T_u. The self-centering behavior of the SC-MRF beam-column connections is illustrated by the typical M-θ_r response from the DBE simulation for the North-end connection of the 3rd floor South-bay beam (denoted 3FSN), shown in Fig. 9(b). The behavior differs in the positive and negative moment directions due to the effects of the floor diaphragm forces on the beam axial force. β_E estimated from the DBE simulation results is around 30%. Overall, the DBE performance objectives were achieved.

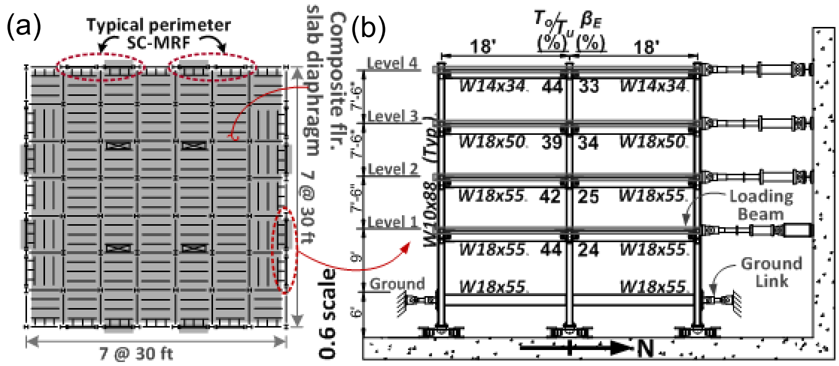

Figure 8. Schematic of (a) plan of prototype, and (b) elevation of 0.6-scale 4-story 2-bay SC-MRF test frame (note: 1′ = 1ft. = 305mm; 1″= 1in. = 25.4mm).

Other system studies include those by Wang and Filiatrault [2008], who performed shake table tests on a 3-story 2-bay SC-MRF and compared the response with that of a 3-story 2-bay conventional WMRF. Their results showed that the SC-MRF had improved performance compared to the WMRF. This study also showed that the local effect of the slab on the response of the PT connections was negligible, even when debris from the damaged concrete slab was introduced at the gap opening of the connection.

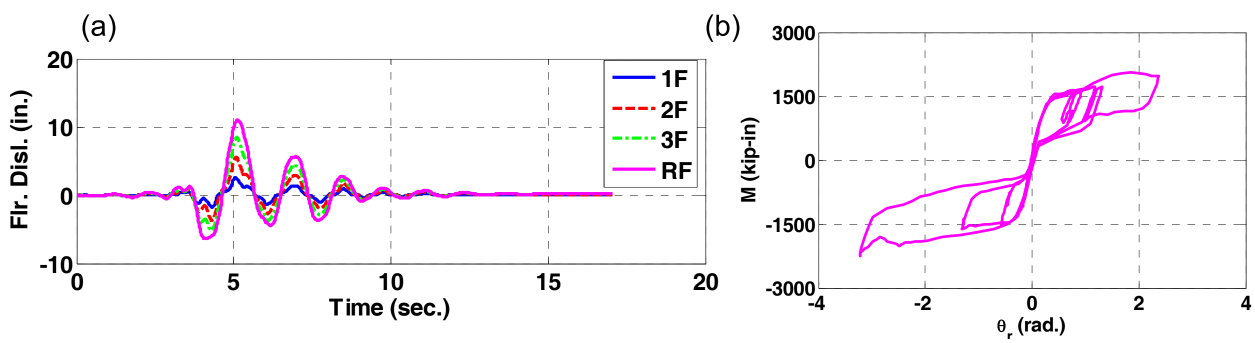

Figure 9. DBE simulation results: (a) floor displacements; (b) moment-θ_r of 3FSN connection

Table 1. Test results from DBE simulation

Level	$\theta_{s\,max}$ (rads.)	$\theta_{r\,max}$ (rads.)	T_{max}/T_u (%)	$\Delta T/T_u$ (%)
RF	0.039	0.038	65	-0.7
3F	0.035	0.034	60	-0.7
2F	0.035	0.031	61	-1.3
1F	0.021	0.025	59	-1.0

Numerical Studies of SC-MRFs

The initial numerical studies of SC-MRFs were performed by Ricles et al. [2001], who designed and investigated the seismic behavior of a six-story, six-bay SC-MRF with angles as energy dissipating devices. Analytical results showed that in terms of maximum story drifts and residual drifts, the seismic performance of an SC-MRF connections exceeds the performance of a a conventional WMRF. Similar conclusions were reached by Christopoulos [2002] where a 6-story SC-MRF with confined yielding ED bars was shown to sustain similar maximum deformations as an equivalent conventional WMRF without sustaining any notable residual deformations under DBE level ground motions. Kim and Christopoulos [2009b] investigated the response of SC-MRFs designed according to the ductile ultimate response approach proposed by Kim and Christopoulos [2009a] (i.e., intended to develop a ductile ultimate mechanism) under MCE level ground motions. Rojas et al. [2005] investigated the seismic response of SC-MRFs with PT friction damped connections (PFDCs). In terms of strength, story drifts, and local deformations the MRFs with the PFDCs showed good seismic performance under both DBE and MCE level ground motions. Kim and Christopoulos [2008] presented three levels of modeling techniques of increasing complexity that are capable of capturing the response of SC-MRFs. A simplified sectional analysis procedure, a lumped plasticity spring frame model, and a nonlinear solid finite element analysis to predict the response at ultimate deformation levels were outlined. The models were validated against numerous experimental results.

Currently, Herning et al. [2009a, 2009b] have subjected three prototype (3-, 9-, and 20-story) SC-MRF buildings to several natural ground motions and one thousand artificial ground motions to evaluate the design procedure proposed by Garlock et al. [2007a], which was developed using limited nonlinear time history simulation data. The results indicate that the probability of exceeding the anticipated design demand values (such as θ_{roof}, θ_s, and θ_r) varies among the prototypes with the 9-story having the largest exceedance probability. A revised relationship for predicting θ_r from θ_s is proposed.

Research Needs for Practical Implementation of SC-MRFs

In the performance-based design of SC-MRFs, estimates of the θ_r demand under the selected seismic input levels are critical for determining whether the limit states considered in the design process are reached. Additional research is needed to provide better estimates for the θ_r demand associated with a level of seismic input, considering the uncertainty associated with ground motions, structural system configuration, and the probability of exceeding selected limit states. There is a need for further numerical studies to calibrate design procedures that can be included in design codes and used widely by practicing engineers. Additional work is needed to validate the 3-D response of SC-MRFs, especially with respect to the slab and column restraint of the frame elongation, and to account for the effect of deformations imposed in the direction perpendicular to that of the SC-MRFs.

Most of the studies to date on SC-MRFs have considered the performance of the system under the DBE and MCE, where selected limit states are considered. There is limited knowledge about the collapse potential of SC-MRFs. Research is needed to fill this gap in knowledge. In addition, continued development of structural concepts and details is needed to ensure that SC-MRFs are ductile at the ultimate limit state and that they are similar to conventional WMRFs when considering collapse prevention. Further studies are needed to establish the long-term reliability of PT strands and bars elements in buildings, especially with respect to fire resistance.

Finally, a detailed cost analysis of SC-MRF buildings compared to conventional WMRF buildings is needed. The analysis should focus primarily on the initial cost of designing, building and erecting these structures, but should also consider the consequences of the expected seismic response on the cost of repair and loss of operations for different levels of seismic hazard. This analysis is needed to determine if SC-MRF systems are a cost effective solution to significantly increase the resilience of steel structures against earthquakes.

Summary and Conclusions

An overview of the development of steel self-centering post-tensioned MRFs that has taken place over the past decade was outlined in this paper. The concept and the mechanics governing the response of SC-MRFs was first presented, followed by a review of performance-based design methodologies that have been proposed for the design of such structures. An overview of some of the main large-scale experimental validations that have been carried out to develop and validate the response of SC-MRFs was provided. A summary of the main numerical studies that have been carried out on SC-MRFs as well as some key results on the predicted response of buildings incorporating SC-MRFs was also presented. Key results from the one of the first large-scale tests of a system incorporating SC-MRFs were also presented. These results have confirmed that SC-MRFs offer a viable alternative to traditional welded MRFs: they can achieve similar stiffness and strength while eliminating residual deformations that are expected in conventional steel-framed structures. The paper also outlines a number of research areas that still need to be addressed to facilitate the implementation of SC-MRFs in practice.

Acknowledgements

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