COMPARISON OF CONVENTIONAL AND SELF-CENTERING STEEL FRAME WITH BRACINGS

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This paper focuses on seismic design, assessment and comparison of conventional steel moment-resisting frame with bracings and self-centering steel frame with bracings. A prototype building was selected and designed as a conventional frame according to Eurocode 8 and as a self-centering frame. The self-centering frame is designed to utilize the same cross-section as the conventional one, while the post-tensioning connection is developed based on an iterative pushover analysis, conducted at the early phase of the design process to estimate rotations and axial forces in post-tensioned (PT) connections and to provide comparable shear strength to the conventional frame. To compare the performance of the both systems, a nonlinear dynamic analysis is conducted using a set of 30 ground motions, scaled to represent the frequently occurring earthquake (FOE), design-based earthquake (DBE), and maximum considered earthquake (MCE). Seismic analyses results show that the conventional and the self-centering frame have comparable peak story displacements and highlight the potential of the second one to eliminate or reduce damage and residual displacements.

Keywords: Conventional steel systems, self-centering systems, residual displacement.

1. INTRODUCTION

Conventional seismic-resistant systems such as steel moment resisting frames (MRFs) or concentrically braced frames (CBFs) are currently designed to develop significant inelastic deformations in the main structural members (i.e., beams and columns and/or braces) under strong earthquakes [1]. This design approach offers certain advantages, such as achieving acceptable seismic performance in terms of life safety and cost-effectiveness. Designing a structure to remain elastic during a strong earthquake would require oversized structural components, making elastic systems not justified both economically and due to increased acceleration. However, allowing inelastic deformations in main structural elements can lead to challenges in repairing damage, residual drifts, and consequently, higher repair

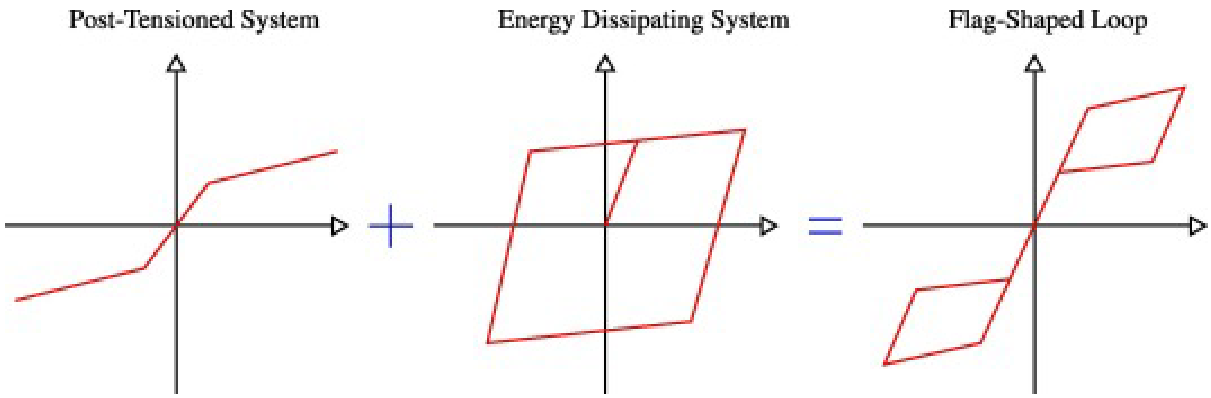

Figure 1. Formation of a flag-shaped hysteresis loop [10]

costs and extended downtime while the building is out of service. A study conducted in Japan by McCormick et al. [2] which examined 12 steel-framed buildings affected by the 1995 Hyogoken-Nanbu earthquake, concluded that in cases where residual inter-story drifts exceeded 0.5% it was more cost-effective to demolish and rebuild the structures rather than repair them due to the high repair costs and the financial losses associated with keeping the building closed during repairs. These losses emphasize the importance of implementing more resilient structures that are less vulnerable and easier to repair after strong earthquakes, with the aim of minimizing or even preventing economic seismic losses. Steel self-centering frames with post-tensioned (PT) beam-column connections are a type of resilient seismic-resistant structure that prevent inelastic deformations in beams and reduce or eliminate residual drifts. These systems usually use energy dissipation devices which are activated when gaps open and can be easily replaced if damaged.

In this paper, a comparison between the conventional steel frame and self-centering steel frame where the dissipation of the energy is designed to be through the braces under tension is presented. For this purpose, a prototype steel building is designed with two different lateral load-resisting systems, i.e. conventional MRF with bracings and corresponding self-centering steel frame. To achieve a fair comparison, both seismic resistant frames are designed using the same structural member dimensions which results in two systems having very similar initial stiffness and periods of vibration.

2. SELF-CENTERING SYSTEMS

In seismic design, while life safety remains a priority, modern expectations, particularly in developed countries, demand buildings to maintain almost full functionality after an earthquake. It's been discussed that residual deformations are commonly seen as an unwanted effect of seismic loads, prompting researchers to develop methods to predict and reduce them. However, a more ambitious goal is to eliminate these residual deformations and return the structure to its original position after the end of a seismic action. This idea has led to the development of systems that return the structure to its original position, or the so-called self-centering systems. These systems are characterized with post-tensioned steel strands that remain elastic throughout seismic loading, providing an elastic restoring force and energy dissipation mechanism. Energy dissipation occurs through specialized dissipaters or elements designed to undergo inelastic behavior during rocking, while the beams and columns remain elastic. The combination of these two hysteretic behaviors creates the "flag-shaped" hysteresis loop, offering both energy dissipation and self-centering during cyclic loading, Figure 1.

Furthermore, if we compare the nonlinear response of the conventional yielding system and self-centering system, Figure 2 there are few main differences, 1) the flag-shaped hysteresis inherently has less energy dissipation per cycle, half at most; 2) the flag-shaped hysteresis has more frequent stiffness changes within one nonlinear cycle than the elastoplastic hysteresis and 3) The flag-shaped hysteresis returns to the zero-force, zero-displacement point at every cycle whereas yielding of the elastoplastic system at every cycle may lead to cumulative "crawling" of the response in one direction.

3. PROTOTYPE BUILDING AND DESIGN OF SEISMIC-RESISTANT FRAMES

The case study is a four-story building with a square plan of four bay by four bay, and a total length of 24.00 m x 24.00 m. The story height is equal to 3.0 m except for the first floor, which is 4.00 m high. The lateral force resisting system is placed at the perimeter of the plan of the buildings, consisting of two seismic frames in longitudinal direction and 2 in transverse direction. The interior frames are assumed to be gravity frames, and their lateral load resisting capacity is neglected. Consequently, the tributary area for seismic masses defers from the tributary area for the gravity loads in a way that the first one takes into account half the mass whereas the second takes into consideration the half bay mass, as described on figure 3. The perimeter frame is designed as a steel MRF with braces and as a self-centering frame with braces.

3.1 DESIGN OF A CONVENTIONAL FRAME

The design of the structure is done according to the provisions of EN 1993 and EN 1998-1 and it is carried out using commercial software. The model represents the distribution of stiffness and mass so that all significant deformation shapes and inertia forces are properly accounted for under seismic action. The models used to perform the designs are based on the centerline dimensions of the steel MRFs without accounting for the finite panel zone dimensions. The columns are considered continuous through each floor beam whereas the braces are pinned. All beam-column connections have been considered fully strength and fully rigid while all floors are assumed made of composite slabs with profiled steel sheeting that should be designed to resist the vertical loads and to behave as horizontal rigid diaphragms able to transmit the seismic actions to the seismic resistant frames. Masses were considered lumped in a selected master joint for each floor, because the floor diaphragms may be taken as rigid in their planes. The building satisfies the criteria for regularity both in plan and in elevation.

For the spectrum analysis, it is required to consider a number of vibration modes that satisfy either of the conditions of EC8. In this case, the first two mode shapes are considered (Tx1=0.6 sec and Tx2=0.17 sec). The SRSS (Square Root of the Sum of the Squares) method is used to combine the modal maxima, since the first and the second modes of vibration in X direction are independent (T2 ≤ 0.9T1).

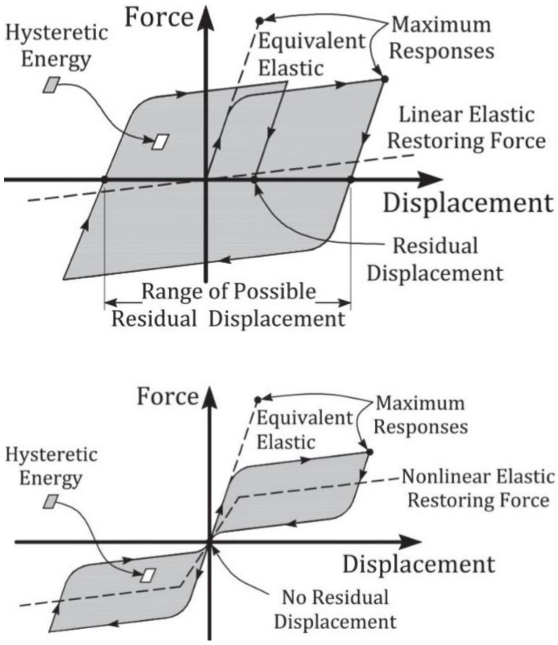

Figure 2. Idealized seismic response of yielding system (up) and self-centering system (down) [4]

The steel MRF is designed as medium-ductility class according to EC8 [1]. The material for all frame elements is S275 steel with an over-strength factor γ_{ov} = 1.25. The chosen member sections are standard metric sections which are commercially available. The gravity loads are taken as approximative values for administrative/residential building, 5.50 kN/m^2 and 3.0 kN/m^2, for the dead and live load, respectively.

The DBE (10% probability of exceedance in 50 years) is expressed by the type 1 EC8 design spectrum for peak ground acceleration equal to 0.3g, ground type B, importance factor II, and behavior factor q equal to 4 (for moment resisting frames combined with concentric bracings). To meet the damage limitation requirement given ductile non-structural elements, the allowable peak story drift, θmax, under the frequently occurred earthquake (10% probability of exceedance in 10 years) is equal to 0.75% according to Eurocode [1]. The frequently occurred earthquake has an intensity of 40% the DBE, i.e. the v reduction factor is equal to 0.4 according to EC8 [1]. For all the steel MRFs, the story drift sensitivity coefficient θ that accounts for P-Δ effects is limited below 0.20. The maximum considered earthquake is assumed to have an intensity equal to 150% the DBE intensity.

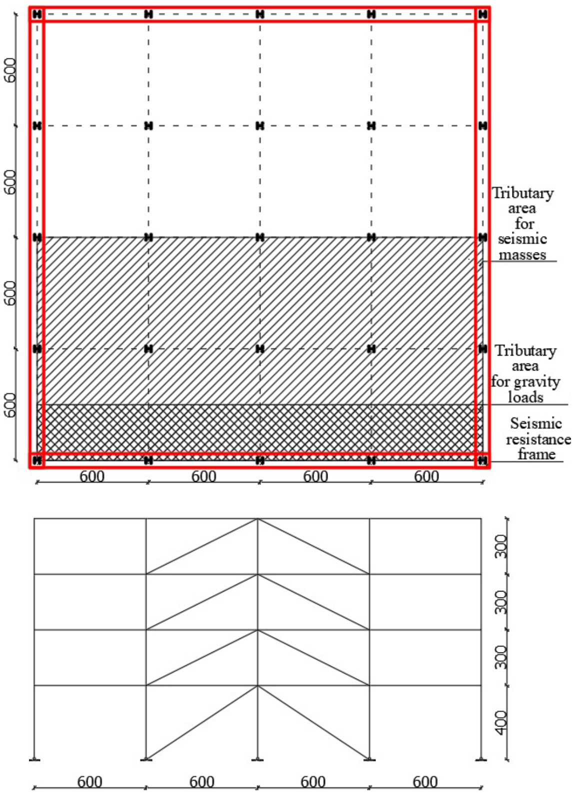

Figure 3. Plan and cross-section of the seismic resistance frame

Both flexural and shear checks are done for the verification of the beams belonging to external and internal bays according to the following equations (Eq.1 - Eq.3).

$$\frac{M_{Ed}}{M_{pl,Rd}} \leq 1 \tag{1}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0.50 \tag{2}$$

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 0.15 \tag{3}$$

The columns are also checked against axial forces, bending moments and shear forces calculated according to [1] and Eq.4 - Eq.6:

$$M_{Ed} = M_{Ed,G} + 1.1 * \gamma_{ov} * \Omega * M_{Ed,E} \tag{4}$$

$$V_{Ed} = V_{Ed,G} + 1.1 * \gamma_{ov} * \Omega * V_{Ed,E} \tag{5}$$

$$N_{Ed} = N_{Ed,G} + 1.1 * \gamma_{ov} * \Omega * N_{Ed,E} \tag{6}$$

where NEd,G, MEd,G, and VEd,G are the design values of the axial force, bending moment, and shear force due to non-seismic actions; γ_{ov} is the material overstrength factor that is equal to 1.25; and Ω is an overstrength factor which is calculated as the minimum of the ratios of the plastic moment resistance to the internal bending moment under the seismic action of all beams. Design details of the

Table 1. Cross-section of the steel frame elements

Story	Structural elements			P_{T0}
	Columns	Beams	Braces	[kN]
1	HE200B HE220B	IPE 300	160.160. 12.5	580
2	HE200B HE220B	IPE 300	140.140. 10	580
3	HE200B	IPE 300	140.140. 8	580
4	HE200B	IPE 270	120.120. 6	540

conventional frame are provided in Table 1. The braces are designed according to the rules described in Eurocodes.

The weak beam-strong column capacity design rule is enforced by satisfying the following condition:

$$\frac{\Sigma M_{Rc}}{\Sigma M_{Rb}} \geq 1.3 \tag{7}$$

3.2 DESIGN OF A SELF-CENTERING FRAME

As mentioned before, the PT connection of the self-centring frame is designed using the same cross-sections for the structural elements as the conventional frame. In such a way, frame with the same or very close initial stiffness and period of vibration as the conventional one is obtained, but with different type of lateral-load resisting system and consequently with different structural performance under strong ground motions.

The main accent in the design of a self-centering frames is in the design of the post-tensioning connection between the column and the beam which is designed to "open" for a certain moment known as moment of decompresion. To achive this, the PT force in the cables that are supposed to replace the moment connection at the initial condition shoud be properly designed. According to the equations and recommendations of Garlock [5], the ratio of the decompression moment and the plastic moment of the beam should have value less than one and higher than 0.5 so that the self-centering will be posible, Eq.8.

An iterative procedure (trial and error) was made to obtain ratio of these moments and a ratio of 0.55 was defined. For this purpose, a pushover analysis was conducted at the early phase of the design process and the pushover curves of the two frames were compared, so that the self-centering frame has base share strength comparable to that of the conventional frame. It should be mention that when calculating the decompression moment, the contribution of the two prestressed cables is taken into account.

$$0.5 \leq \frac{M_d}{M_{pl,b}} < 1 \tag{8}$$

$$M_d = P_{T0} * \frac{b_h}{2} = 0.55 * M_{pl,b} \tag{9}$$

After decompression, the gap opening results in an increase in the post-tensioning force, PT, which can be calculated from the equations derive by Christopoulos [4].

$$P_T = P_{T0} + 2K_{PT}(1 - 1/\Omega)b_h\theta \tag{10}$$

$$\Omega = 1 + \frac{K_b}{K_c + 2K_{PT}} \tag{11}$$

Where: K_b, K_c and K_{PT} are the axial stiffness's of the beam, column and PT elements, respectively (AE/L).

The diameter of the cable is calculated according to the Eq.10-12, where $f_{y,PT}$ is 1770 MPa. The required cross-section of the diameter can be assumed so that the value of the initial prestressing force is half the value of the yield force of the cables ($P_{T0}/P_{Ty} \approx 0.5$) which approximately ensures that PT bars avoid yielding under large rotations in the PT connections [7]. Also, the area of the cables should be verified for the designed drift demand for the frame.

$$P_{Ty} = 0.5 * f_{y,PT} * \pi * d_{PT}^2 \tag{12}$$

4. NONLINEAR MODELS OF THE FRAME

To investigate the seismic performance, two-dimensional nonlinear analytical models of the conventional and of the self-centering frame were developed for nonlinear dynamic analyses in OpenSees [11]. The conventional frame is modelled according to the guidelines for conventional frames [11] whereas for the modeling of the self-centering frame experimental results were used [3] [10]. The SC model uses *force-based non-linear beam-column elements* to represent the braces, columns and beams. An initial camber was applied at the mid-point of the brace to simulate the effects of buckling, whereas the beam and the columns remain in elastic range. The PT connection is modeled using truss elements with applied initial PT force utilizing the *"initStrain"* material. For simulating the rocking connection ENT (*"elastic no-tension"*) material is used.

5. NONLINEAR DYNAMIC ANALYSIS AND COMPARISON OF THE RESULTS

5.1 GROUND MOTIONS AND PROCEDURE FOR DYNAMIC ANALYSES

A set of 30 recorded ground motions, developed by the INNOSEIS project [9] was used for nonlinear dynamic time-history

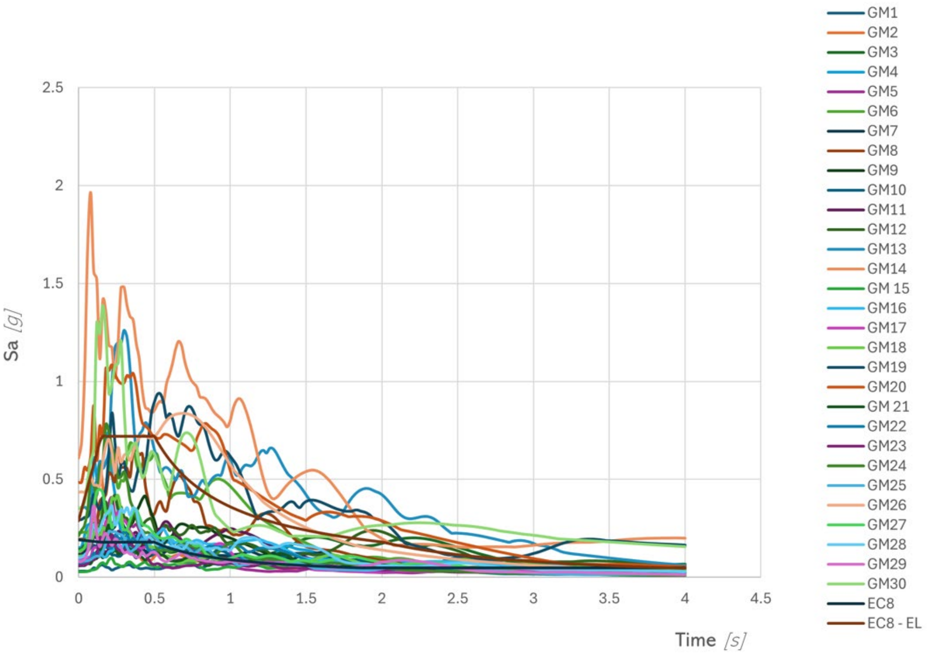

Figure 4. Acceleration response spectra of the ground motions considered in this study (unscaled)

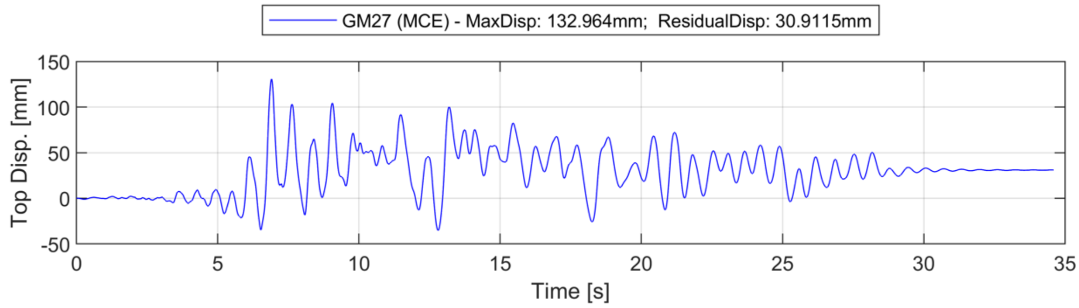

Figure 5. Time history plot of top displacement for MCE for conventional frame

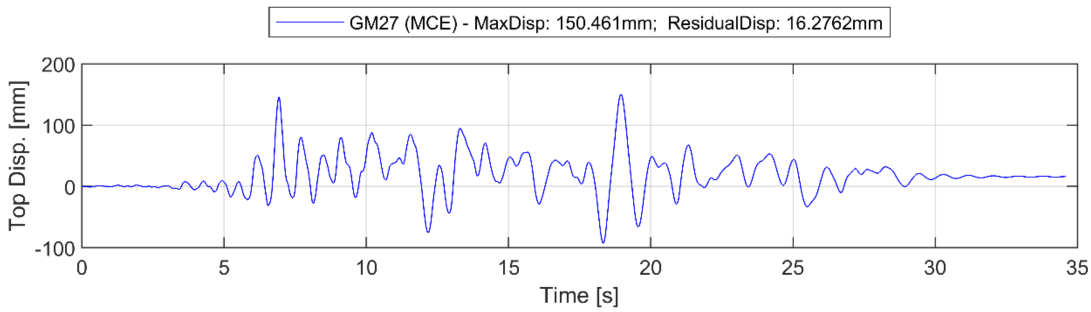

Figure 6. Time history plot of top displacement for MCE for self-cantering frame

analyses. The ground motions were scaled to FOE, DBE (Figure 4) and MCE, where the seismic intensity was represented by the 5% spectral acceleration, Sa, at T (0.6sec) of the frame models.

The Newmark method with constant acceleration is used to integrate the equations of motion. The Newton method with tangent stiffness is used to minimize the unbalanced forces within each integration time step. A Rayleigh damping matrix is used to model the inherent 3.3% critical damping at the first two modes of vibration. Each dynamic analysis was extended beyond the actual earthquake time to allow for damped free vibration decay and accurate residual drifts calculation.

5.2 SESMIC ASSESSMENT

The results of the 90 nonlinear response-history analyses for the two design cases were post-processed and the results are presented as the mean, median and standard deviation values for the maximum displacement and residual displacement, in table 2 and table 3, for the conventional and for the self-centering frame, respectively. A comparison is shown for one earthquake excitation – ground motion 27 (Figure 5 and 6), which reflects the statistical trend, which is that the self-centering system plays a role generally only during the maximum expected earthquake (MCE) and in that case it eliminates or reduces the residual deformations more than 50%.

Table 2. Mean, median and STD for the maximum and residual displacement for conventional frame

		Mean	Median	STD
FOE	Max. disp. [mm]	33.8	33.4	2.6
	Res. disp. [mm]	0.4	0.4	0.2
DBE	Max. disp. [mm]	85.6	83.3	17.0
	Res. disp. [mm]	11.1	11.6	7.0
MCE	Max. disp. [mm]	163.5	137.7	99.7
	Res. disp. [mm]	34.4	22.0	35.9

Table 3. Mean, median and STD for the maximum and residual displacement for conventional frame

		Mean	Median	STD
FOE	Max. disp. [mm]	33.1	32.6	2.4
	Res. disp. [mm]	1.1	1.1	0.9
DBE	Max. disp. [mm]	114.1	104.1	45.1
	Res. disp. [mm]	12.0	9.6	9.2
MCE	Max. disp. [mm]	204.8	180.5	123.7
	Res. disp. [mm]	12.7	9.1	16.5

6. SUMMARY AND CONCLUSIONS

In this paper, a comparison between a conventional moment resisting frame with braces and self-centering frame with braces is presented. For that purpose, the design procedure according to Eurocode 3 and 8 is given for the conventional frame. The design of the self-centering frame is carried out so that the same cross-section of the conventional frame is used, and a design for the posttensioning connection is done according to the recent research and guidelines for PT connections. In such a way, frame with the same or very close initial stiffness and period of vibration as the conventional one is obtained, but with different type of lateral-load resisting. Thus, the frames will have different structural performance under strong ground motions. To compare the different behavior, nonlinear dynamic analysis is performed for a set of 30 ground motions, scaled for FOE, DBE and MCE.

The results show that self-centering mechanism plays a significant role only for the case of maximum considered earthquake and it generally eliminates or reduce the residual displacement for at least 50% and consequently decrease the probability for non-repairability. The reason why this applies only for MCE is the conservative design of EC8, particularly the FOE design drift limits. Thus, for low to moderate earthquakes the frames behave almost elastic, and the energy dissipation and self-centering mechanisms do not make a difference.

Also, the maximum displacements for the SC frame are slightly higher. The results confirmed findings of previous studies regarding peak and residual displacements (Karavasilis and Seo 2011) indicating that for structures with a period exceeding 0.5 seconds, energy dissipation does not play a significant role [6]. Furthermore, the conventional frame experience damage in the beams for DBE and MCE, whereas the self-centering frame is damage free in the beam because of the rocking mechanism, but experience damage in the braces.

Based on aforementioned outcomes and previous work showing that self-centering and conventional systems of the same strength and period of vibration have similar seismic drifts when the self-centering system is designed with increased energy dissipation capacity and post yield stiffness [6], it can be concluded that the self-centering frame with braces may have better performance if equipped with energy dissipation devices acting as fuses for the braces.

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