Comparison of the Seismic Response of Steel Building Incorporating Self-Centering Energy Dissipative Braces, Buckling Restrained Braced and Moment Resisting Frames

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ABSTRACT :

The seismic response of special moment-resisting frames (SMRF), buckling restrained braced (BRB) frames and self-centering energy dissipating (SCED) braced frames is compared when used in building structures of 2 to 12 stories in height. The study involves pushover analysis as well as 2D and 3D nonlinear time history analysis for two ground motion hazard levels. The SCED and BRB braced frames generally experienced similar peak interstory drifts. The SMRF system had larger interstory drifts than both braced frames, especially for the shortest structures. The SCED system exhibited a more uniform distribution of the drift demand along the building height and was less prone to the biasing of the response in one direction due to P-Delta effects. The SCED braced frames also had significantly smaller residual lateral deformations. The two braced frame systems experienced similar interstory drift demand when used in torsional irregular structures.

KEYWORDS: Bracing member, Residual drifts, Self-centering, Torsion, Yielding,

1. INTRODUCTION

The purpose of this study is to compare the seismic response of three non-linear seismic force resisting systems that may be used in low- to mid-rise steel building structures. These systems are the special moment-resisting frame (SMRF), the buckling restrained braced frame (BRB frame) and the self-centering energy-dissipative braced frame (SCED braced frame). All three systems provide energy dissipation as well as a force limitation mechanism (a structural fuse) when subjected to earthquake loads. This allows the buildings using these systems to be designed according to capacity design principles to limit stresses in all members of the structure.

The self-centering energy dissipative (SCED) brace is an advanced cross-bracing system that has recently been developed at the University of Toronto and École Polytechnique de Montréal (Christopoulos et al. 2008). The advantage of the SCED brace system is that, unlike other comparable advanced bracing systems that dissipate energy, such as the buckling restrained brace system, it has a self-centering capability that reduces or eliminates residual building deformations after major seismic events. This leaves the main structural components of the building intact, undamaged and in-place, allowing the building to be re-occupied relatively soon following a seismic event. This is an attractive feature in seismic design because sometimes the financial losses due to repair and disruption time following an earthquake, especially those due to residual deformations, can be more significant than the structural damage itself.

A thorough explanation of the SCED brace concept and validation can be found in Christopoulos et al. (2008). A schematic of the SCED brace showing its basic function is given in Figure 1(a). The system consists of two rigid longitudinal members that are abutted by end plates on each end. These end plates are connected together by pre-tensioned tendons. These tendons provide the self-centering capability of the brace. Since the end plates are not connected to the longitudinal elements, the tendons are in tension regardless of the direction of axial deformation. Between the two longitudinal members, there exists some type of energy-dissipative mechanism either in the form of a friction, viscous, or yielding device. The resulting hysteretic behavior of the brace is shown in Figure 1(b).

Recently, Tremblay et al. (2008) performed an extensive analytical study to assess the seismic response of SCED braced frames. The response of these frames was also compared to that of BRB frames. According to these results, the residual deformations for SCED braced frame systems are negligible under low and moderate hazard levels and are reduced considerably under maximum considered earthquake level. However, this analytical study was limited to two-dimensional model structures.
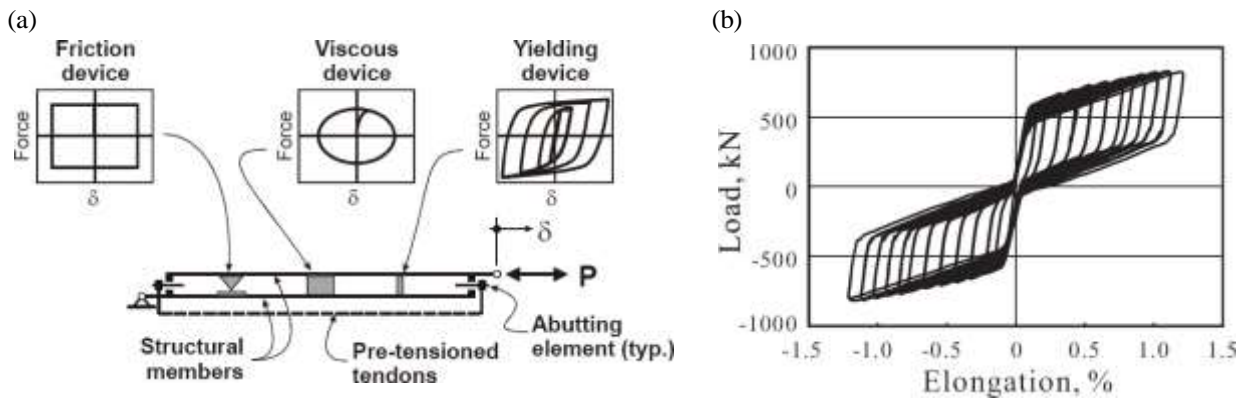

Figure 1 (a) Concept of SCED System; (b) Hysteretic Response of SCED Prototype under Quasi-Static Axial Loading (from Christopoulos et al. 2008)

2. DESIGN OF MODEL STRUCTURES

1.1. Description of Model Structures

A typical steel frame office building was chosen to compare the response of moment-resisting frames (SMRF), buckling restrained braced frames (BRB frames) and self-centering energy-dissipative braced frames (SCED braced frames). This building was assumed to be located in downtown Los Angeles, California, where seismic loads are expected to govern the design of the lateral force resisting system. The plan of the office building consists of three 9.14 m bays in the north-south direction and five 9.14 m bays in the east-west direction. The lateral force-resisting system in the east-west direction consists of two SMRFs with three bays each (along the north and south edges of the building). The lateral force-resisting system in the north-south direction consists of two BRB or SCED braced frames located in the center bays of the outer north-south frames. To investigate the effects of torsion, the location of these two braced frames in the east-west direction is varied to produce a torsional eccentricity for some of the analyses. For the case where there is no significant torsion (besides the accidental torsion), the two braced frames are located along the east and west edges of the building. To induce torsion, one of the frames is relocated further inward by one bay (for a 20% eccentricity offset) or two bays (for a 40% eccentricity offset). Six different building heights were evaluated: 2, 4, 6, 8, 10 and 12 stories. Overall this resulted in 36 complete building designs (two brace types, three eccentricity values and six heights).

1.2. Design of Buildings According to ASCE 7-05

The building structures were designed using the loading defined by ASCE 7-05 (ASCE, 2005). The steel members were designed according to the provisions of ANSI/AISC 360-05 (AISC, 2005a) and ANSI/AISC 341-05 (AISC, 2005b). This design process included consideration of gravity, wind and seismic loadings on the structures. For the seismic design, a first trial design was obtained using the equivalent lateral force procedure in ASCE 7-05. Using this preliminary design, a full 3D modal response spectrum analysis was performed for each building, including in-plane torsion effects for the braced frames with offsets. The design values from the response spectrum analysis were scaled to ensure that the base shear was not less than 85% of the base shear from the equivalent static force procedure (as required by ASCE 7-05). The story drift limits, P-Delta effects and the effect of accidental torsion on the buildings were then calculated and the accidental torsional loads were added to the structure. The structural members were redesigned according to these new loads and the seismic detailing of all members according to AISC/ANSI 341-05 was completed. These steps were then repeated

beginning with a rerun of the modal response spectrum analysis until convergence of the design had been reached. A complete description of the design method and results, including the explicit consideration of capacity design principles can be found in Choi et al. (2008).

3. MODELING OF STRUCTURES

Two-dimensional and three-dimensional models of the buildings were produced. These models were created in the nonlinear frame analysis software Ruaumoko (Carr, 2005). They were then subjected to push-over analyses and full 2D and 3D time-history analyses under two suites of earthquake records. Both types of models considered the effect of a non-linear geometry (P-delta effects) and the 3D models considered the effect of torsion on the structure due to the irregular location of the braced frames. Several modeling assumptions were made in the creation of the nonlinear models. In all the models, the concrete slabs for each floor were not modeled explicitly. Instead, rigid diaphragms were employed to ensure that each floor translates and rotates as a rigid body. For all models, large displacement analysis was used to account for geometric nonlinearities. In Ruaumoko, this consists of a general large displacement analysis that updates the full geometry of the model at every time step.

The 2D models are simplifications of the corresponding 3D building designs. Each 2D model is for a single frame (braced frame or moment resisting frame) and therefore models the lateral load resisting system of half of a building, including half of the weight of the building. These models include all of the beams, columns and bracing members that are required for the lateral load resisting system itself. In addition, the models contain leaning columns that effectively represent the rest of the building half. These leaning columns are loaded vertically with all of the dead and live load that is not directly surrounding the lateral load resisting frame in the building. In this way, P-Delta effects arising from the deformation of the entire building are taken into account. The 3D models are direct translations of the buildings designed including all members and loadings.
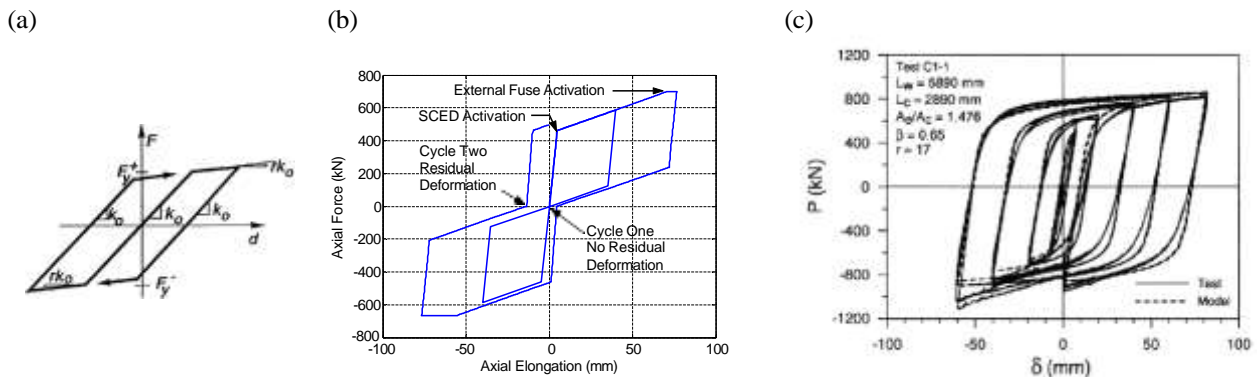
(a)   (b)   (c)

Figure 2 (a) Ruaumoko Bilinear Hysteresis (Carr, 2005)
(b) SCED Brace Hysteresis Including Activation of External Friction Fuse
(c) BRB Hysteresic Model: Comparison Between Test and Predicted Responses (Tremblay et al., 2004)

The nonlinear hystereses that were used for each different type of nonlinear element are shown in Figure 2. The steel beam and column moment yielding behavior was modeled using a simple bilinear hysteresis (Figure 2(a)). The SCED brace behavior was modeled using a combined hysteresis using a flag-shaped hysteresis and a bilinear hysteresis in series, giving the total hysteresis shown in Figure 2(b). This bilinear hysteresis was added to the SCED brace model in order to simulate the effect of an external friction fuse in series with the SCED brace itself. The purpose of this fuse is to limit the maximum load in the SCED brace and allow capacity design principles to be applied to the designs of the SCED braced frames. The BRB behavior was modeled using a Ramberg-Osgood Hysteresis. Figure 2(c) shows a comparison of the hysteretic model response versus the real response of a BRB based on tests (Tremblay et al., 2004). For the steel building models, a global damping of 3% of critical was chosen as a reasonable value. In most cases, this was implemented in the models using a

Rayleigh damping model with 3% damping in modes 1 and 2. However, for the models with SCED braces, a uniform damping model was used that results in 3% uniform damping in all modes of vibration. This was required to avoid the development of unrealistic damping forces in the SCED external friction fuse upon sliding.

4. PUSH-OVER ANALYSIS RESULTS

Pushovers were applied to the models using a dynamic time-history force-control loading using a slow ramp loading function. Loads were applied along the height of the building with magnitudes corresponding to combined mode shapes of the building's response. The 2D top story displacement *vs* base shear pushover curves for the 6- and 12-story models are shown in Figure 3 for each of the lateral load resisting system. The points corresponding to the first yielding of members and the achievement of maximum interstory drift are indicated on each curve. The interstory drift limit for these two structures is 2.0%. In the figures, V_d corresponds to the design base shear, Δ_R is the roof displacement and h_n is building height.

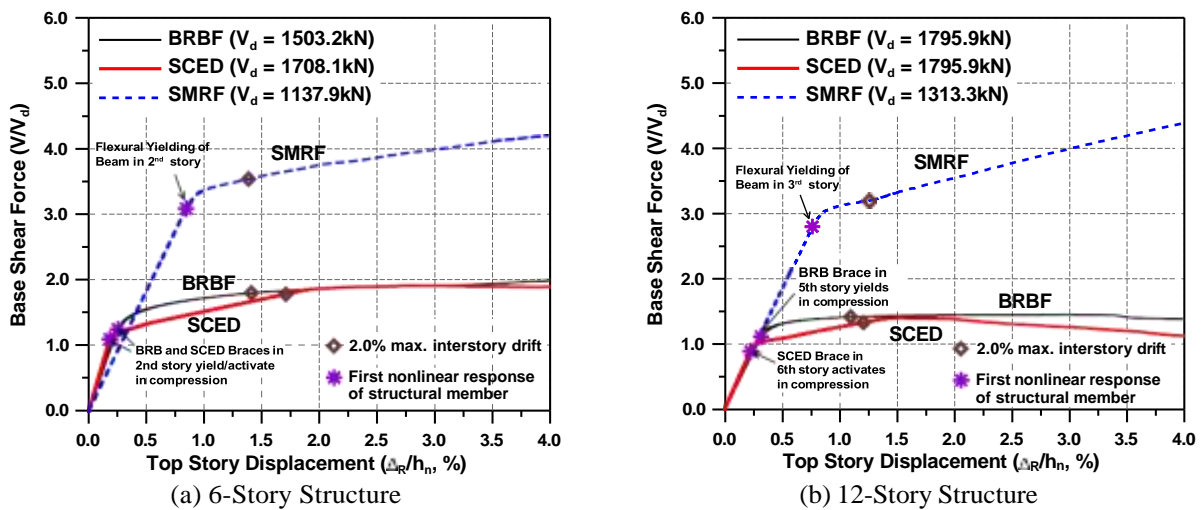(a) 6-Story Structure   (b) 12-Story Structure
Figure 3: Push-over curves for 6- and 12-story 2D Models

The figures show that the yield strength of SMRF systems is much larger than that of the braced frames relative to the design base shear. This is because the SMRF frames have considerably more material overstrength built into their design than braced frames. On the other hand, for the braced frame pushovers, the strength of brace members themselves directly controls the response of the braced frames since they are the only nonlinear elements present. In this study, the ratio of strength demand to capacity is kept above 0.95 for BRBs and SCED braces. The corresponding ratio of the actual first yield/activation strength to the required design strength, for SMRFs, BRB frames and SCED frames are about 3.1~3.4, 1.3~1.6 and 1.0~1.2, respectively.

The displacement profiles for the SCED frames and BRB frames are similar in general. However, there is a more well-defined activation ('yield') point for the SCED braced frames than for the BRB frames because the SCED braces were modeled using effectively bilinear hysteresis models whereas the BRBs were modeled using a hysteresis that has a smooth transition between the initial stiffness and the yielded stiffness. The taller models exhibit negative post-yield stiffnesses for the braced frames at high drifts due to P-Delta effects.

For the 6-story structure, the pushover results from the 2D model and the 3D model with no braced frame offset were compared to verify the validity of 2D modeling method. The pushover curves from 2D and 3D models are almost identical, confirming that the responses of 2D structures can adequately represent the response of 3D structures that have no mass and stiffness irregularities.

5. GROUND MOTIONS CONSIDERED FOR TIME-HISTORY ANALYSIS

The earthquake records that were used for this study were originally developed as part of the SAC steel project (Somerville et al., 1997) and were chosen to correspond with the design response spectrum for the Los Angeles region where the model buildings are located. The SAC ground motions used in this study correspond to site class D in ASCE 7-05. Two different levels of seismic ground motions were considered: one level to represent the design level earthquake (with earthquake records chosen to correspond to the design response spectrum), and one level to represent the maximum credible earthquake or MCE (with earthquake records chosen to correspond to the MCE response spectrum which is assumed to be 3/2 times the design response spectrum). For California, the design level earthquakes have a 10% probability of exceedance in 50 years and the maximum credible earthquakes have a 2% probability of exceedance in 50 years. For each hazard level, 20 earthquake records (10 sets) were used.

6. MAXIMUM INTERSTORY DRIFTS

The average maximum interstory drift values for the 2D models are compared in Figure 4 for the design-level seismic hazard (10% in 50 years) and the maximum considered seismic hazard (2% in 50 years). For the design-level hazard, the SCED and SMRF systems have the largest response for the shortest buildings, but the systems tend to converge to approximately the same maximum interstory drift and variation as the building height increases. The SCED and SMRF systems especially seem to converge to a similar response. Above 8 stories in building height, the average maximum interstory drift response is more or less the same for the three systems, with the BRB system showing a higher variation in the response than the other two. For the MCE earthquakes, the SCED has the largest response for the 2-story building, but quickly shows a large reduction in the maximum response as the building height increases. The SCED system also shows a marked decrease in the variability of the response as the height increases, performing much better than the other two systems. Again, above 8 stories, the SMRF and BRB systems show similar levels of average maximum response, but the SMRF outperforms the BRB frame for the tallest systems subjected to the maximum considered seismic loadings.
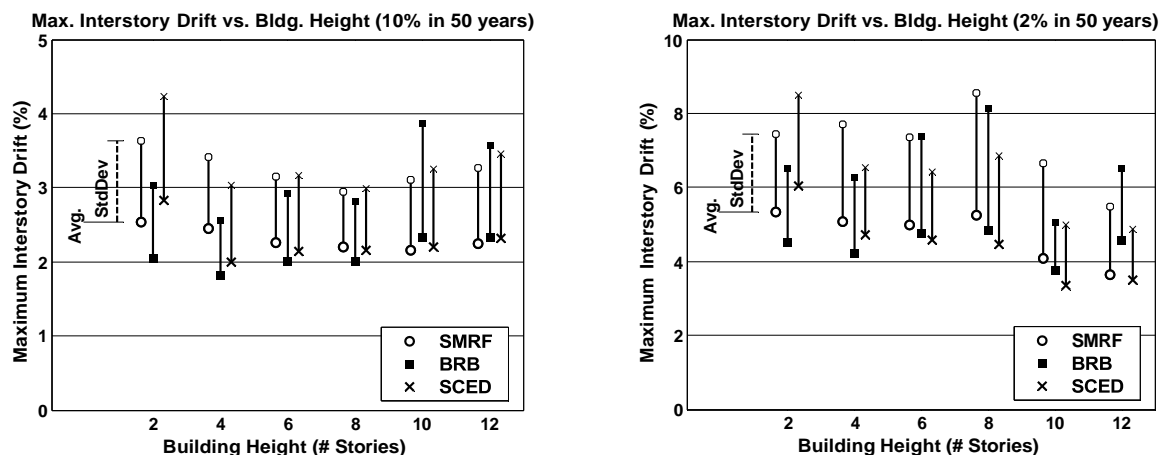

Figure 4 Mean Maximum Interstory Drift vs. Building Height and Type (Avg. and Avg. + One Std. Deviation)

Since the SCED braced frame tends to exhibit poor behaviour for low-rise 2-story buildings for both seismic hazard levels, it has been suggested by Tremblay et al. (2008) that a reduced response reduction factor (R-factor) may be warranted for the design of these low-rise frames. Tremblay et al. conducted 2D nonlinear analyses on SCED and BRB frames and concluded that 2- and 4-story SCED and BRB frames would benefit from a reduction in the response reduction factor from 7 to 5. This reduction resulted in stiffer buildings and lower lateral deformations for both frames.

7. MAXIMUM RESIDUAL INTERSTORY DRIFTS

Currently, there is no set of defined relationships between the level of residual drift and the consequence of that residual drift. However, a relationship may be discussed qualitatively. Residual drifts less than 0.2% can be considered to be insignificant since this is the acceptable out-of-plumb tolerance for new construction according to AISC specifications. For residual drifts less than 0.5%, most likely no special intervention would be necessary. For residual drifts between 0.5% and 1.0%, a special assessment by a structural engineer should be required before the building may be reoccupied. For residual drifts between 1.0% and 1.5%, intervention would likely be required consisting of either straightening of the building structure or reinforcing the building in its deformed position. This would also be required from an occupancy point of view since a 1.0% drift would be noticeable to occupants especially, for example, in doorways. The cost of such intervention would likely be high, probably on the order where replacement of the building would be a viable economic alternative. Lastly, any residual drift greater than 1.5% would likely be a total loss from an economic point of view.

The average and average plus one standard deviation values for the maximum residual interstory drift are compared for all of the 2D models subjected to both seismic hazard levels in Figure 5. This figure shows that the BRB buildings clearly have the largest residual drifts and also the largest variations in the residual drift values. The drifts also tend to increase as the building height increases (except for the tallest buildings when subjected to the 2% in 50 year excitations). The SMRF buildings have similar maximum residual interstory drifts (and variations of the drift) to the BRB buildings for buildings lower than 8 stories. For the tallest buildings, though, the residual drift response diverges, with the SMRF buildings showing significantly lower residual drifts than the BRB buildings for the 10 and 12 story heights. The SCED brace systems generally performed far better than the other two systems with respect to the residual drift, with average drifts generally much lower and smaller variation in those drift values as well for both hazard levels. The only exception is for 2-story buildings, where all three systems have approximately the same drifts and variations. For 2-story models subjected to the 2% in 50 year excitations, the SCED brace systems seem to show even larger drifts than the other two systems. Again, this problem may be mitigated by reducing the response modification factor for the design of the 2-story SCED building.
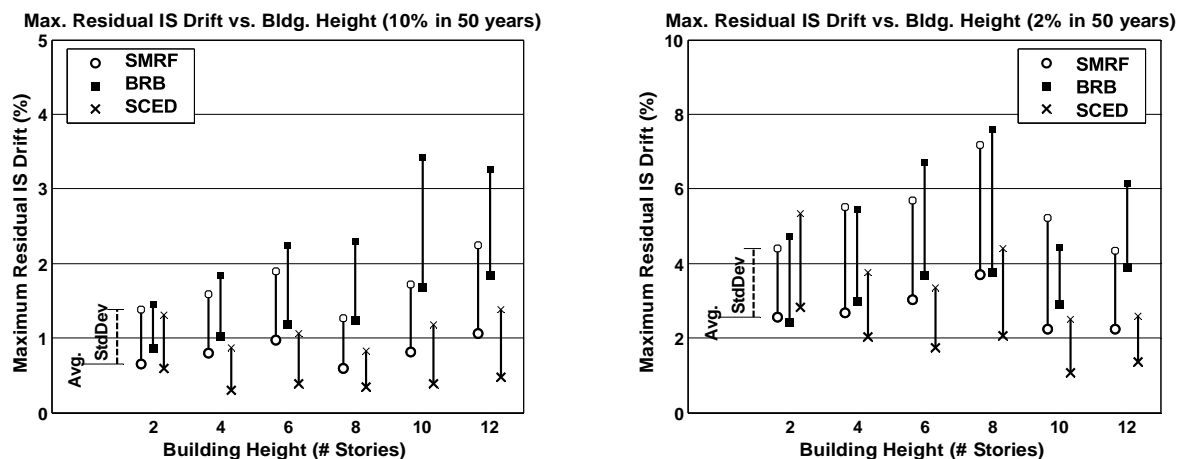

Figure 5 Mean Maximum Residual Interstory Drift *vs* Building Height and Type
(Avg. and Avg. + One Std. Deviation)

In general, the self-centering behavior of the SCED brace minimizes residual drift even when the external fuse activates because the residual drift of the SCED brace is equal to only the activation displacement of the external fuse which is over and above the activation displacement of the brace itself (which is recoverable through self centering). This is opposed to the BRB and SMRF frames where almost the entire displacement of the system results in an unrecoverable deformation until the system is loaded in the opposite direction.

8. TORSIONAL RESPONSE

The effect of torsion on the maximum interstory drift and maximum residual interstory drift response of the braced frame structures for the design-based seismic hazard is shown in Figure 6. For the mean maximum interstory drift (Figure 6(a)), the effect of the stiffness irregularity (frame eccentricity) is not large. This is attributed to the fact that ASCE 7-05 provisions for in-plane torsion were included in design. The drift magnitudes are also similar to the results of the 2D time-history analyses. The variations of the interstory drift responses were similar for both systems (as shown by the mean plus one standard deviation lines in the figures). The SCED frame system experiences larger drifts for the tallest buildings when the torsional effect is included in the analysis, although in general, both systems exceed the drift limit for those tall buildings. In general, the 2-story SCED frame also exceeds the drift limit, which was also the case for the 2D analysis.

The residual story drift ratio tends to decrease as torsional irregularity increases (Figure 6(b)). This trend is likely due to a restoring force provided by the SMRF frames that are perpendicular to the braced frames in the 3D models. As the torsional irregularity increases, deformation of the SMRF frames makes up a larger percentage of the total lateral deflection in the braced frame direction, since the amount of twist of each story increases and the total drift remains more or less constant. Since the SMRF frames remain mostly linear due to their higher yield forces, they provide an effective twisting restoring force which reduces the overall residual drifts in the braces. This restoring force effect is likely exacerbated by the fact that the earthquake excitation is applied in the braced frame direction only in the 3D models, meaning that the SMRF is likely experiencing much less inelasticity than it would be for an earthquake excitation in an arbitrary direction. This effect was previously encountered by Pettinga et al. (2007). Except for the 2 story SCED brace structure, which performs similarly to the BRB frames, the residual drifts of the SCED braced frames are generally less than 0.5%, whereas residual drifts of BRB frames generally range between 0.5% and 1.5%.
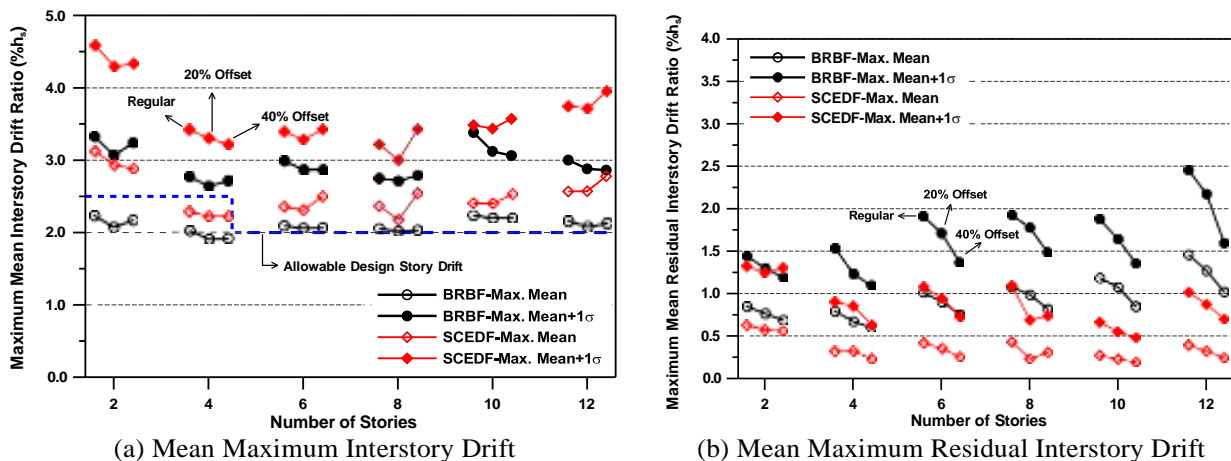

(a) Mean Maximum Interstory Drift   (b) Mean Maximum Residual Interstory Drift
Figure 6 Torsional Response for Maximum and Residual Interstory Drifts *vs* Building Height and System Type
(Avg. and Avg. + One Std. Deviation) – Both for 10% in 50 Year Hazard Level

9. CONCLUSIONS

For the design-based excitations (with a probability of exceedance of 10% in 50 years), the SCED braced frames and the BRB frames showed similar levels of displacements and interstory drifts for the suite of earthquakes that was considered. The SMRFs had larger displacements and interstory drifts than both braced frames and the difference was most pronounced for the shorter buildings. Under the maximum credible earthquake excitation (with a probability of exceedance of 2% in 50 years), the SCED braced frames showed lower displacements and interstory drifts than either the BRB frames or the SMRFs. In general, the BRB frames performed better than the SMRFs. However, for the taller buildings, soft-story formation and P-Delta effects in the BRB frames caused the displacement and interstory drift responses to deteriorate to the point where the maximum values

were significantly worse than the SMRFs exhibited. In addition, the SCED brace's high post-activation stiffness and self-centering behaviour caused a more uniform distribution of the drift demand along the height of the building, allowing the SCED braced frame to avoid the formation of a soft-story even for the tallest buildings considered, and prevented the biasing of the response in one direction due to P-Delta effects.

Under design-based excitations, the SCED braced frames experienced negligible residual drifts, while the BRB frames and the SMRFs experienced significant permanent drifts after the excitations with an average magnitude around 1.2%, a value which is both noticeable to occupants and would warrant expensive repair. Under the maximum considered excitations, activation external fuses of the SCED braces caused residual drifts in the SCED braced frames equal to approximately 1% to 2%. These are significant drifts, but the performance criterion of life-safety for the structure is well outside this limit. For the non-self-centering systems (the BRB and the SMRF) the residual drifts after resulting from the MCE level excitation were, on average, between 2% and 4%. A residual drift greater than 1.5% likely represents a total loss of the building from an economic point of view, with repairs to the building probably costing more than the value of the building.

The main purpose of the 3D tests was to determine the effect of building torsion on the different systems. The analysis results show that the maximum interstory drifts for both braced frame systems with different irregularities did not exceed the allowable design story drift in general because the effect of horizontal torsional irregularity was considered in the design process. The residual drift responses tended to decrease as irregularity increased due to a restoring force caused by the SMRFs in the perpendicular direction to the frames.

The results of this study demonstrate the benefits of using the self-centering energy dissipative (SCED) braces as an alternative to either buckling restrained braces (BRBs) or special moment resisting frames (SMRFs). The displacement and drift responses for all of the systems are comparable, but the SCED braces provide the added benefit of self-centering behaviour, which limits P-Delta effects in tall structures and the costly residual deformations in the structure after an earthquake.

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