시간이력감쇠기를 가진 강골조의 지진저항성능

Earthquake resistant performance of steel frame with hysteretic damper

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Abstract

This paper highlights research being conducted to identify ground motion and structural characteristics that control the response of concentrically braced frames using hysteretic damper, unbonded brace, and to identify improved design procedures and code provisions. The focus of this paper is on the seismic response of six story concentrically braced frames utilizing hysteretic damper. A brief discussion is provided regarding the mechanical properties of such braces and the benefit of their use. Results of detailed nonlinear dynamic analyses are then examined for specific cases to characterize the effect on key response parameters of structural configurations and proportions.

요 지

이 논문은 이력감쇠기, 즉 unbonded brace를 이용한 폐브레이싱된 골조의 거동을 지배하는 구조적 특성을 연구하였으며, 이를 이용한 골조의 설계지침 개선을 제시코자 하였다. 본 연구에서는 시간이력댐퍼를 이용한 6층 폐브레이싱된 골조의 지진거동을 예시로 제시하였다. 또한 좌굴방지 브레이싱의 기계적 특징과 장점에 대해 간략히 서술하였다. 골조의 형상과 특성의 중요 파라메타의 효과를 살펴보기 위해 비선형동적해석을 수행, 비교 분석하였다.

Keywords : hysteretic damper, concentrically braced frame, seismic response, unbonded brace

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1. Introduction

Steel moment-resisting frames are susceptible to large lateral displacements during severe earthquake ground motions, and require special attention to limit damage to nonstructural elements as well as to avoid problems associated with P-D effects and brittle or ductile fracture of beam to column connections[(1)]. As a consequence, engineers in the US have increasingly turned to concentrically braced steel frames as an economical means for resisting earthquake loads. However, damage to concentrically braced frames in past earthquakes, such as the 1985 Mexico[(2)], 1989 Loma Prieta[(3)], 1994 Northridge [(4)], and 1995 Hyogo-ken Nanbu[(5)] earthquakes, raises concerns about the ultimate deformation capacity of this class of structure.

Individual braces often possess only limited ductility capacity under cyclic loading[(6)]. Brace hysteretic behavior is unsymmetric in tension and compression, and typically exhibits substantial strength deterioration when loaded monotonically in compression or cyclically.

Because of this complex behavior, actual distributions of internal forces and deformations often differ substantially from those predicted using conventional design methods[(7),(8)]. Design simplifications and practical considerations often result in the braces selected for some stories being far stronger than required, while braces in other stories have capacities

very close to design targets. This variation in story capacity, together with potential strength losses when some braces buckle prior to others, tend to concentrate earthquake damage a few "weak" stories. Such damage concentrations place even greater burdens on the limited ductility capacities of conventional braces and their connections. It has also been noted that lateral buckling of braces may cause substantial damage to adjacent nonstructural elements.

Prompted by these observations and concerns, seismic design requirements for braced frames have changed considerably during the 1990s, and the concept of special concentric braced frames has been introduced[(9)]. Considerable research has also been initiated improve the performance of concentrically braced frames through the introduction of new structural configurations. During the past decade, there have also been parallel advances in research related to characterizing the seismic hazard at a site, simulating seismic response, and theories for characterizing seismic performance in probabilistic terms. As such, a review of the overall seismic performance characteristics of concentrically braced frames designed to current standards is timely.

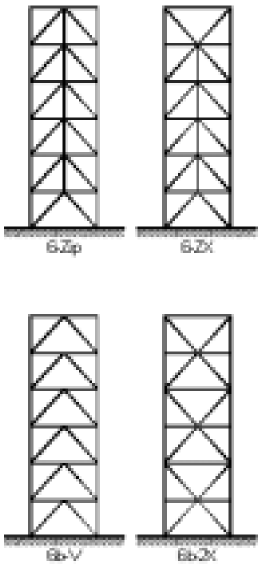


Fig. 1 Concentric braced frame systems

The goal of the overall project described in this paper is to investigate the system level performance of concentrically braced buildings subjected to seismic loads with the intention of understanding the structural and ground motion characteristics that control behavior, and to assess and, where necessary, propose improved design and analysis procedures.

A series of nonlinear dynamic analyses has been carried out examining the behavior of concentrically braced frames having conventional braces, high performance hysteretic braces. Some of the basic configurations being studied are shown in Figure 1. This paper highlights results obtained for frames utilizing buckling - restrained braces.

2. Behavior of Buckling-Restrained Braces

Since many of the potential performance difficulties with conventional concentrically braced frames rise from the difference between the tensile and compression capacity of the brace, and the degradation of brace capacity under compressive and cyclic loading, considerable research has been devoted to development of braces that exhibit more ideal elasto-plastic

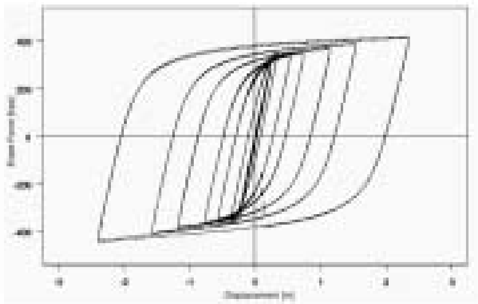


Fig. 3 Axial Force-Displacement Plot for Buckling Restrained Brace with Steel Core Unbonded from Mortar Filled Steel Tube

behavior. One means of achieving this is through metallic yielding, where buckling in compression is restrained by an external mechanism. A number of approaches to accomplish this have been suggested (see Fig. 2) including enclosing a ductile metal (usually steel) core (rectangular or cruciform plates, circular rods, etc.) in a continuous concrete filled tube, within a continuous steel tube, a tube with intermittent stiffening fins, and so on. The assembly is detailed so that the central yielding core can deform longitudinally independent from the mechanism that restrains lateral and local buckling.

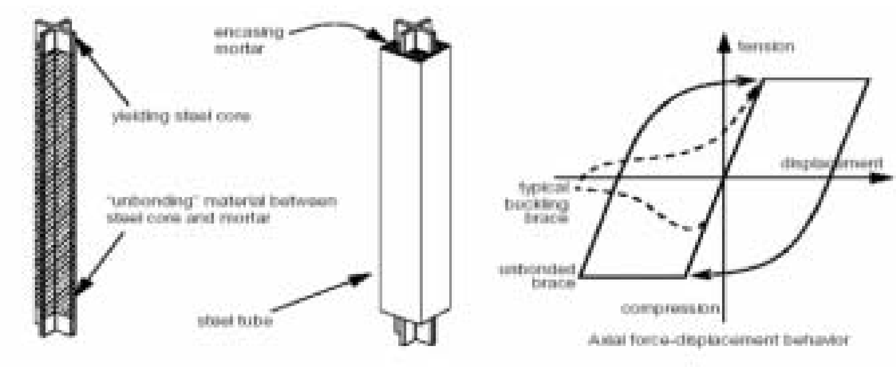


Fig. 2 Some schematic details used for buckling restrained braces

Through appropriate selection of the strength of the material, and the areas and lengths of the portions of the core that are expected to remain elastic and to yield, a wide range of brace stiffnesses and strengths can be attained. Since lateral and local buckling behavior modes are restrained, large inelastic capacities are attainable. Theoretically based methods have been developed to design the restraining media[10]. The inelastic cyclic behavior of several types of buckling-restrained braces have been reported[11,12]. These tests typically (see Fig. 3) result in hysteretic loops having nearly ideal bilinear hysteretic shapes, with moderate kinematic and isotropic hardening evident. Interestingly, the difference between the tensile and compressive strength of steel results in greater strength of the buckling restrained braces in compression than in tension (differences up to 10% have been reported [Clark, 2000])

3. Numerical Study

3.1 Model Buildings

To assess the performance of concentrically braced frames, a series of six-story braced frame buildings were designed for a site in metropolitan Los Angeles. The buildings were designed according to the 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 302/303)[13]. The building configurations and non-seismic loading conditions were identical to those utilized in the development of the FEMA 350 guidelines for moment resisting frames, so that comparisons to moment frame behavior could be made[14].

Results are presented here only for systems with buckling-restrained braces oriented in a stacked chevron (inverted V-) pattern. In the design, buckling-restrained braces were envisioned as having an unbonded, yielding steel core within a mortar filled steel tube. However, nearly any unbonded-brace having equivalent properties may be assumed. A572 Gr.50 steel was used for all beams and columns.

The six-story building design is adapted from the FEMA model building design criteria for the nine-story building. This height structure was added to the example studies, as it is a very common height for braced frame structures in the western US. The six story building has a typical 13-foot (4 m) story-height, but with an 18 feet (5.5 m) height at the first story. Its nominal plan dimensions are 154 feet by 154 feet (46.9m by 46.9 m); 30-foot by 30-foot (9.1 m by 9.1) bays are employed. Floors and roof have a 3-inch (76-mm) metal deck with normal-weight concrete topping. A small penthouse is located on the roof. Twelve bays of bracing are provided; six in each direction. Again, the number of braced bays was selected to prevent an increase in member design forces due to the r factor. These are located on the on the perimeter; bracing is located in non-adjacent bays. Both frame and non-frame columns are spliced mid-height at the fourth story.

In the design of the model buildings using FEMA 302/303, the equivalent static lateral force procedure was employed based on a response-spectrum corresponding to a hazard of 10% chance of exceedence in a fifty year period. A Response Modification Factor (R) of six was considered; a parallel design was also done using a Response Modification Factor of eight.

A System Over-strength Factor (Wo) of 2 was used. Since code displacement criteria were not expected to control the design of these systems, and the Deflection Amplification Factor (Cd) remains to be defined for these systems, drifts under static design forces were calculated, but not used to limit the design. The buildings were designed consistent with Seismic Use Group I and Seismic Design Category D with an Importance Factor of 1.0. Site Class D (firm soil) was used for determining the response spectrum in conjunction with acceleration data obtained from seismic hazard maps prepared by the US Geological Survey. For the determination of design forces, the building period and the force distribution over the building height was determined using the approximate methods provided in the provisions (where period is based on building height, and lateral forces are distributed in proportion to elevation), rather than by employing a more realistic dynamic analysis. Braces were designed for the force calculated based on the computed equivalent static base shear. Brace sizes were set to within two percent of the computed required cross-sectional area (based on a nominal yield stress of 36 ksi (248 MPa) for the yielding core); no strength-reduction factor, f, was used.

Table 1 Member Properties

story	Buckling-Restrained Braces		Beams	Columns
	Tension Capacity (Kips)	Axial Stiffness (Kip/in)		
6	173	888		
5	288	1432		W14X132
4	317	1566		
3	349	1707	W14X48	
2	389	1886		
1	511	1907		W14X211

The brace stiffness was calculated assuming a yielding length of 70% of the brace length and cross-sectional area of the non-yielding zone of three to six times that of the yielding zone; this is consistent with current design practice. Sizes of members determined for model 6vb2 are shown in Table 1.

3.2 Analytical Modeling Assumptions

Only a single braced bay was modeled and analyzed for each frame configuration. Although the frames were not explicitly designed to be moment resisting, all beam to column connections with gusset plates attached were modeled as being fixed. Possible contributions of the floor slabs to the beam stiffness and strength were ignored. Beams were assumed inextensible in the analyses. Columns were modeled as having a fixed base. The foundation was modeled as being rigid; footing up-lift was not permitted. Braces were modeled as pin-ended members.

The floor level masses used in the analysis to account for horizontally acting inertia forces was taken as the total mass of the each floor divided by the number of braced bays used in the building in each principal direction.

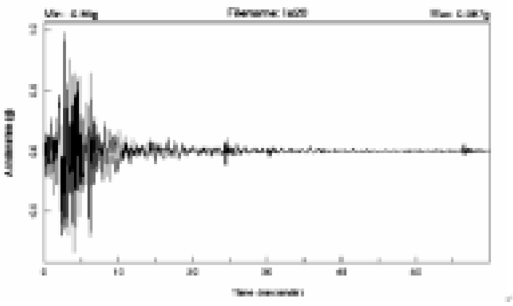


Fig. 4 1986 North Palm Springs earthquake acceleration data

Global P-D effects were considered based on his mass. Since only horizontal ground excitations were considered, local tributary masses were not distributed along the floors. An effective viscous damping coefficient of 5% was assumed, according to common practice for code designed steel structures.

The analyses were carried out using the nonlinear dynamic analysis computer program SNAP-2DX (Rai, 1996). The buckling-restrained braces were modeled using a simple truss element with ideal bilinear hysteretic behavior, exhibiting no stiffness or strength degradation.

The models were analyzed using the suites of ground motions developed previously by Somerville and others for use in the FEMA project on steel moment-resisting frames [15],[16]. These suites consist of 20 horizontal

ground acceleration records (two components for each of ten physical sites) adjusted so that their mean response spectrum matches the 1997 NEHRP design spectrum (as modified from soil type of SB-SC to soil type SD and having a hazard specified by the 1997 USGS maps for downtown Los Angeles). For this study, the earthquake suites corresponding to downtown Los Angeles, California, were selected for seismic hazard levels corresponding to a 50%, 10% and 2% probability of exccedence in a 50 year period. These acceleration time histories were derived from historical recordings or from simulations of physical fault rupture processes.

3.4 Analysis and discuss the result

In this section, the response results for a six story braced frame model(Fig. 5), designed with an R factor of eight (Model 6vb2) are described in detail for the specific case of one of the records in the 10% in 50 year hazard suite. The record in question is designated LA20, and was derived from a near-fault site during a moderate magnitude event, the 1986 North Palm Springs earthquake(Fig. 4). For the 10% in 50 year probability of exceedence, this record has been amplitude scaled to 0.98g. For this severe record, the peak roof displacement computed is 11.93 in. (300 mm), corresponding to an average maximum interstory drift of only about 1.2 %. The maximum interstory drift ratio that occurs at any level during the earthquake is 2.3%, suggesting that some concentration of damage occur within one or more stories.

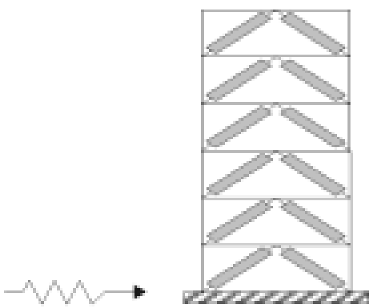


Fig. 5 Configuration of chevron braced frame

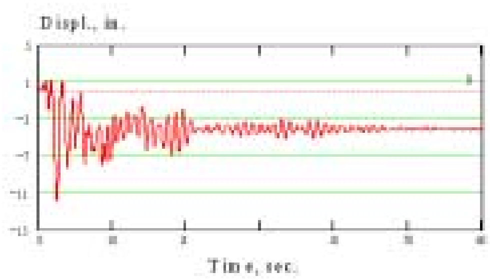
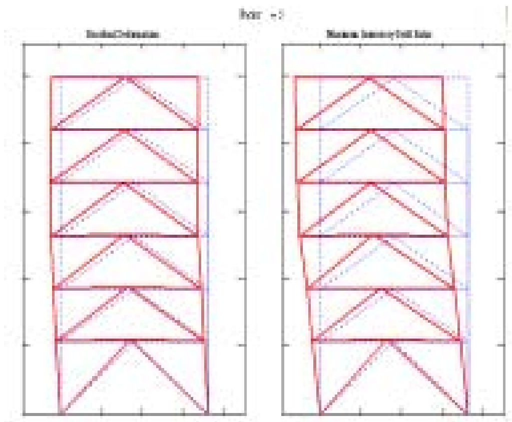


Fig. 6 Roof displacement time history for Model 6vb2 to the LA20 record

The permanent displacement offset that can be seen in the roof level displacement time history (Fig. 6) suggests that considerable inelastic action does occur during this earthquake. As will be elaborated on in the next section, peak roof displacements ranged from 5.48 in to 16.6 in (140 mm to 422 mm) for the records considered in this suite; averaging 9.74 in (247 mm). Thus, the response to this record is well above average.

An examination of the displaced shape of the building when the maximum roof response occurs (Fig. 7) suggests a relatively uniform distribution of interstory drift over the height, with higher than average drifts in the lower three stories, and lower than average values in the upper three stories. Similarly, the residual displacements retained in the structure are nearly uniform over the full height (Fig. 7). It should be noted that the maximum residual displacement remaining at the roof level at the end of the earthquake is 4.95 in. (124 mm), corresponding to a average permanent drift ratio of about 0.5%. The peak residual drift in any story is slightly less than 1% for this earthquake.



a. Permanent Displacements b. Maximum Displacements

Fig. 7 Displaced shapes of Model 6vb2 resulting from LA20

The bar graph of the maximum lateral displacement and residual lateral displacement is shown in Fig. 8.

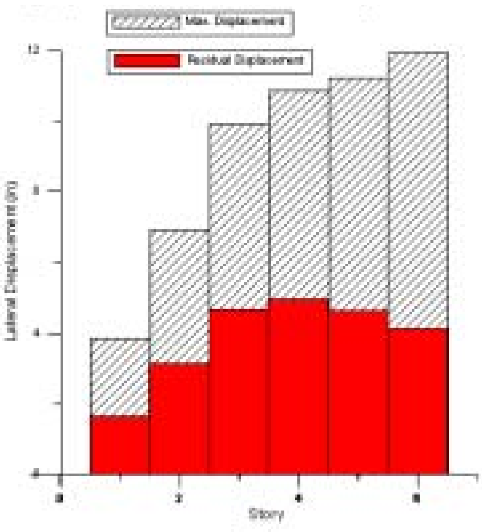


Fig. 8 Comparison of lateral displacement

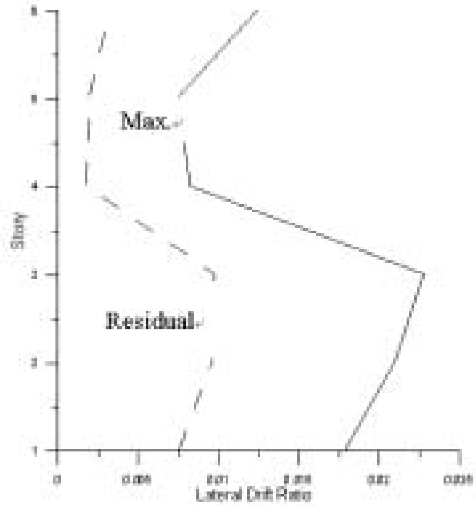


Fig. 9 Maximum lateral drift ratio

The ratio of maximum and residual lateral displacement for each floor is as follow; 43% for first floor, 45% for second floor, 47% for third floor, 46% for 4th floor, 41% for 5th floor, 35% for roof. Its clear from Fig. 8, the all of those ratio are under 50%. It represents that this braced frame has somewhat less inelastic deformation after occurring the maximum lateral displacement subject too earthquake shaking. Similarly, as can be seen in Fig. 9, all residual drift ratio for each floor are under 50% of the maximum drift ratio, so this plot also show the same tendency as Fig. 8

The severity of the inelastic response can be better visualized from Fig. 10, which plots for each story in the braced bay the relation

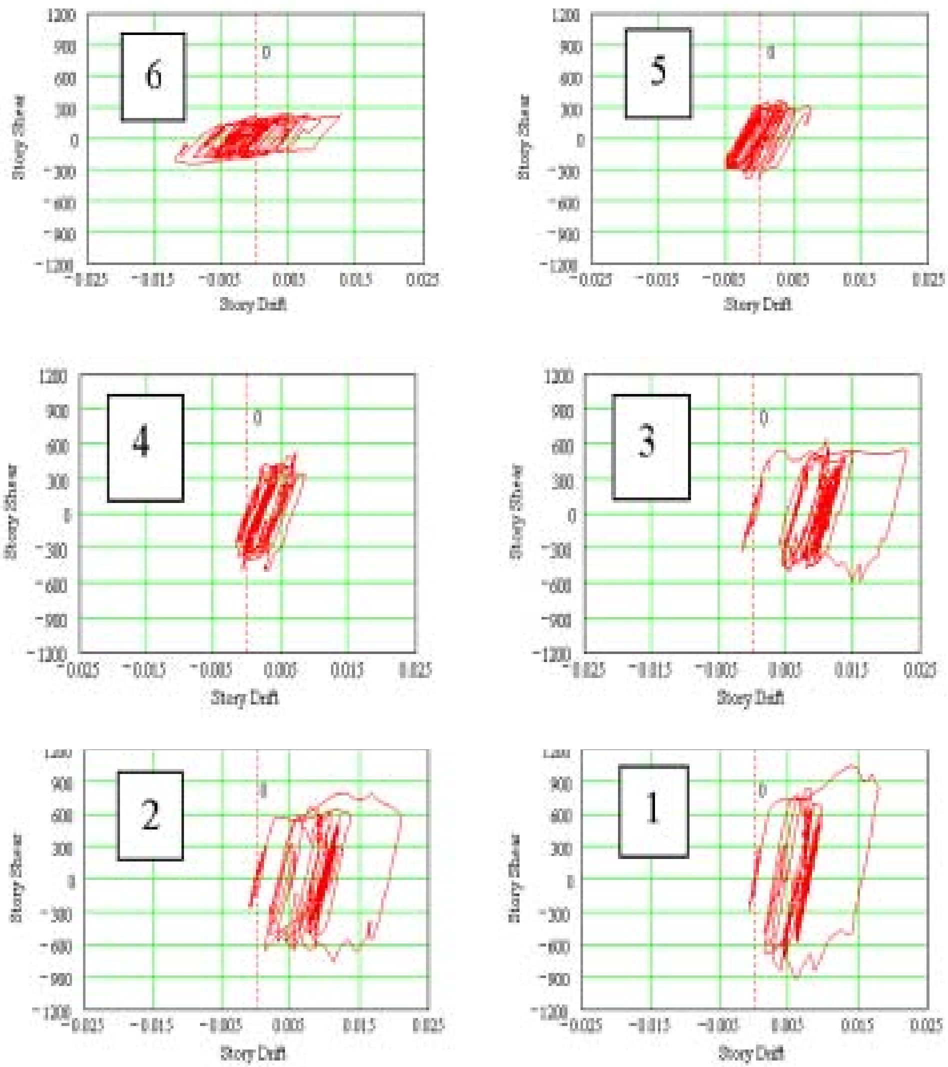


Fig. 10 Story level relations between story shear and interstory drift

between story shear and interstory drift.

As can be seen in this figure, there is substantial yielding. However it is significant to note that this is nearly uniformly distributed over the height of the structure, and in spite of the relatively small post-yield stiffness of the structure resulting from the modeling assumptions, that there is little tendency to concentrated damage at weaker stories that yield substantially more than other stories. Another parameter used in this study to assess the tendency to concentrate damage in a floor, and to place significant flexural demands on the columns and beams, is the column rotation angle, defined herein as the difference in drift ratios for adjacent stories for a floor level. This change in drift represents the need for the column to bend or kink at the floor level. For this building, the maximum value of the column drift ratio was 1.6%, suggesting that at some time during the response, significant local bending action is demanded of the frame. However, at the instant of maximum displacements, such large differences in interstory drift do not occur.

The maximum brace ductility demand (computed as change in brace length divided by the yield displacement in tension) is 15.3 in extension and 9.4 in contraction. This difference is associated in large part with the difference in the tensile and compressive capacities of the braces. The worst cumulative ductility (taken as the plastic deformations occurring in a brace summed over all cycles throughout the entire response history, in either tension or compression, divided by the tensile yield displacement of the brace plus unity) demanded for any brace in the frame is 127. Test data suggest that such demands are well within the capacity of many types of buckling-restrained braces.

It is apparent that there is some vertical movement (displacement) at the center of the beam. Because the buckling-restrained braced considered are slightly stronger in compression than in tension, the tendency is for the flexible beam considered in this analysis to displace upward as the tensile brace will yield before the compression brace.

For this earthquake, the center of the beam deflects upward 1.08 in. (27 mm) (and only 0.01 in. (0.3 mm) downward). While this is a small displacement over the 30 ft (9 m) span of the beam, nearly 90% of the peak value remains after the earthquake, and it represents a large fraction (about 2/3) of the worst interstory drift developed at any level. It should be noted that the braces were intentionally modeled to maximize this behavior. Inclusion of more realistic beam and brace properties would be expected to reduce this vertical movement

4. Concluding Remarks

An extensive analytical investigation of the seismic response of concentrically braced steel frames has been undertaken. Results have identified a number of important parameters associated with the ground motion intensity and characteristics as well was with the structural configuration, proportioning and modeling that have important impacts on computed response. Results presented in this paper have focused on applications of buckling restrained bracing members. Results from this phase of the overall study indicate:

1) Buckling-restrained braces provide an effective means for overcoming many of the potential problems associated with special concentric braced frames. To accentuate potential difficulties with this system, numerical modeling and design assumptions were intentionally selected to maximize predicted brace demands and the formation of weak stories.
2) None the less, the predicted behavior is quite good, with significant benefits relative to conventional braced frames and moment resisting frames.
3) Response appears to be sensitive to proportioning suggesting that further improvements in response may be obtained by better estimation of a structures dynamic properties.

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