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SEISMIC TESTING OF STEEL BRACED FRAMES WITH ALUMINUM SHEAR YIELDING DAMPERS

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ABSTRACT

A shake table study of single-bay two-story model of conventional ordinary concentric braced frame (OCBF) and aluminum shear-link enabled braced frame (SLBF) was conducted to evaluate the performance of shear-link as energy dissipation device. The 1:12 reduced scale models were subjected to Taft ground motion of increasing peak ground accelerations, representing seismic loads of increasing severity. Similitude laws for the reduced models were satisfied and time, frequency and acceleration values were scaled. The test indicated that SLBF frame attracted about 47 to 64% less base shear compared to OCBF. Similar trend was noticed for overturning moments, and floor acceleration as well. However, floor displacements were greater by about 3 to 14% for SLBFs due to reduced stiffness caused by yielding of shear-links. Significant amount of energy was absorbed by aluminum shear-links leading to satisfactory response upto scaled PGA of 1.7*g* of the Taft motion, while the OCBF frame could not survive the scaled PGA of 0.8*g*. These tests also helped validate the design methodology developed for proportioning aluminum shear-link enabled frames.

Introduction

Under seismic action, reliance for survival of fixed-base structures is placed on its ability to dissipate seismic energy, which occurs while undergoing large inelastic deformations in specially detailed regions of beams and column bases of the gravity load system. With the use of energy dissipation devices (EDDs), which can be easily replaced, it is possible to prevent accumulation of inelastic deformation in the main gravity load resisting members and localization of the damage induced. The basic function of EDDs is to reflect and/or absorb a portion of the input energy, and thereby reducing the energy dissipation demand on primary structural members and minimizing possible structural damage.

A widely considered strategy for the dissipation of energy in the structure during an earthquake is through the inelastic deformation of metallic devices [Soong and Dargush 1997]. The shear yielding of low yielding alloy metals, such as aluminum, has been found to be very ductile and large inelastic deformations are possible without tearing or buckling. The yielding in shear mode maximizes the material participating in plastic deformation without excessive localized

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strains. In this regard I-shaped shear-links of low yield ductile alloys of Aluminum have been found to be excellent energy dissipative devices limiting the energy dissipation demand on structural members of the primary structure [Rai and Wallace 1998, Matteis and Mazzolani 2007, Sahoo and Rai 2009] as shown in Fig. 1. Further, the addition of aluminum shear-links to an ordinary chevron braced frame (OCBF) has been shown to improve the seismic performance remarkably. The analytical study indicated that addition of shear-links leads to considerable reductions in the base shear acted as a damper by dissipating considerable amount of seismic energy induced in the structure. A number of element tests on the reduced and full scale shear-links showed satisfactory performance over a wide range of frequencies [Rai and Wallace 1998, Jain and Rai 2008, Banerjee 2005, and Banerjee 2006]. However, no system test by means of fixing the shear-link within a frame has been conducted to verify the effectiveness of shear-link braced frames (SLBF).

Earthquake simulation tests are an invaluable source of information for the understanding of the behavior of the structural systems in the nonlinear range. Shaking table tests were conducted to evaluate the load resistance mechanism, failure/damage pattern and the hysteretic behavior of shear-link systems and provide the data for developing suitable design procedures for proportioning various elements of the overall system.

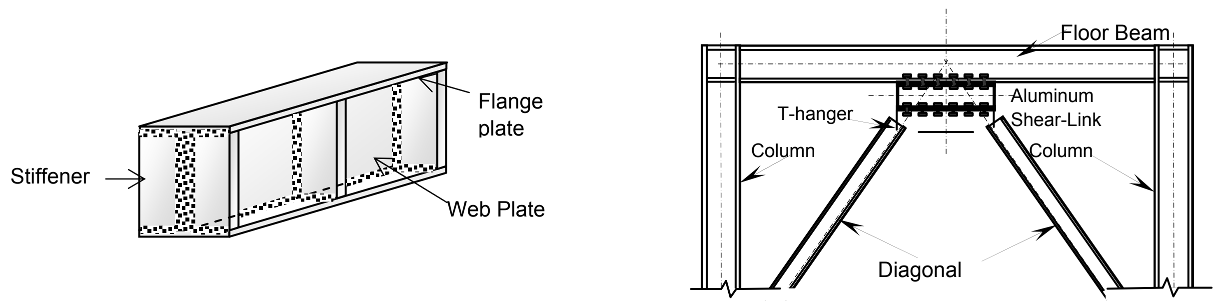


Figure 1 Schematic diagram of typical Shear-link and its arrangement in shear-link brace frame system (SLBF).

OBJECTICVE AND RESEARCH MOTIVE

The primary objective of this research effort is to study the performance of the SLBF, designed as per the simplified method developed by Rai and Wallace (1998), using shake table experiments. A 1:12 reduced scale model was fabricated with due care of dynamic similitude relations and earthquake simulation tests were conducted. The performance of the SLBF was evaluated in terms of floor accelerations, base shears, overturning moments, and hysteretic response of shear-links. Similarly, OCBF model having same details as that of SLBF model was tested in order to compare the performance with the SLBF.

ALUMINUM SHEAR LINK AS SEISMIC DAMPER

Typical shear-link with two panels is shown in Fig. 1, which is fabricated from thin plates forming its flanges, web and stiffeners. The aluminum shear-link is designed to yield in shear mode to limit the maximum lateral force which is transmitted to the primary structure and to provide significant energy dissipation potential during the earthquake ground shaking by means of inelastic deformation (damping device). In addition, significant amount of strain hardening of aluminum alloys allows the shear-links to resist additional lateral loads after first yield and thus absorbing additional deformations in shear-links of other stories in a multi-storied structure. Consequently, the inelastic activities are spread out across various bays and stories of the building structure. These properties make the aluminum shear-link attractive for both new buildings and improvement to existing structures. Aluminum links should be placed strategically in the structure to yield in shear; for example, in ordinary chevron braced frames it is placed in between the diagonal braces and the floor beam as shown in Fig. 1 [Rai and Wallace 1998].

EARTHQUAKE SIMULATOR TESTING

Prototype Building

A two-storey community building assumed to be located in seismic zone V on the soil profile Type I (Rock, or hard soil) of IS 1893(Part 1) [BIS 2002] was considered for analysis. In the plan, the building is 36 m long in the E-W direction (6 bays @ 6m) and 18 m (3 bays @ 6m) wide in the N-S direction. In the elevation, floor to floor heights are 4.5 m and the building is assumed to possess no irregularity of any kind. In the N-S direction, six bracing frame system were designed to provide the code level lateral resistance. The building was assumed to have a dead load and live load of 3.5 kPa and 3 kPa, respectively, on roof and floor. The six bracing frame systems at the middle bay in N-S direction are designed as SLBF systems and all the other interior frames were designed to resist only gravity loads associated with their tributary areas.

Model

The experimental model represents a portion of the building which corresponds to two sets of isolated braced frames (SLBF or OCBF) designed to resist inertial forces developed due to loads on corresponding tributary areas, as shown in Fig. 2. The assumption is justifiable considering that the excitation is only in one direction. For a reliable correlation with the prototype, it is important that the modeling is appropriate with respect to governing similitude relations [Mills et. al 1979]. Based on the size and capacity of the shake table an optimum length scaling ratio of 1:12 was adopted. An acceleration ratio of 2 was adopted to reduce the required mass for limited space available in the model. For adequate dynamic simulation, acceleration, time and frequency scale ratios were modified according to applicable similitude relations, as shown in Table 1.

Table 1. Dynamic similitude rules adopted for modeling

Parameters	S_l	S_E	S_a	ρ	S_ω	S_M
Value	$1/12$	1	2	$S_E/S_l S_a = 6$	$S_l\sqrt{\rho/S_E} = 0.204$	$\rho S_l^3 = 0.00347 = (1:288)$

S_l : Length scale ratio, S_E : Modulus scale ratio, S_a : Acceleration scale ratio, ρ : Equivalent mass density, S_ω : Modified time scale ratio, and S_M : Modified mass scale ratio

Tubes sections were used for frame members because of ease in fabrication. The dimensions of the shear link required were computed from the formulation developed by Rai and Wallace (1998). Braces were proportioned such that they should not buckle before the shear stress in the links reached the failure shear stress. Pipe sections were chosen for braces because the fracture life and ductility are greater than those of rectangular tube sections. In this present study, the braces and other members of the OCBF were kept the same, as in the SLBF, to facilitate a direct comparison. All the beam and columns were cut to the required lengths from the corresponding tube sections, and connected using pinned connections. The connection details have been dealt in detail by Annam (2007). All the columns were connected to base beam through pin connection. The schematic diagram of the model is illustrated in Fig. 3, where SL1 and SL2 represent the aluminum shear link at ground story and first story levels.

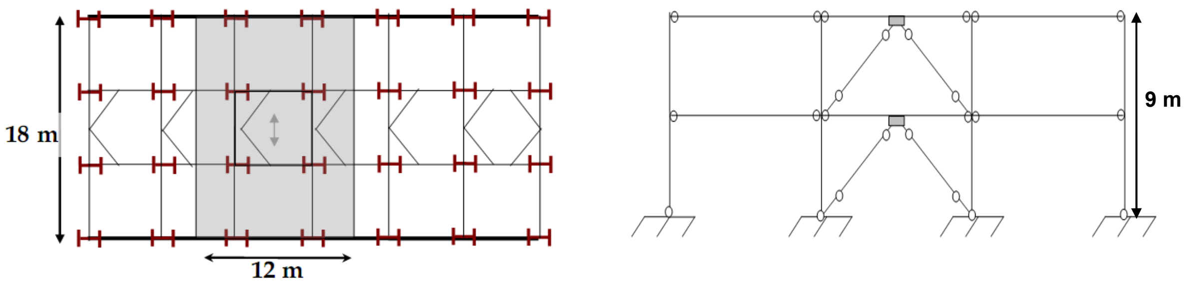


Figure 2 (a) Portion of Building and its Tributary Loading Area of the Prototype Considered for the Model (b) Cross-sectional view of the frame under consideration.

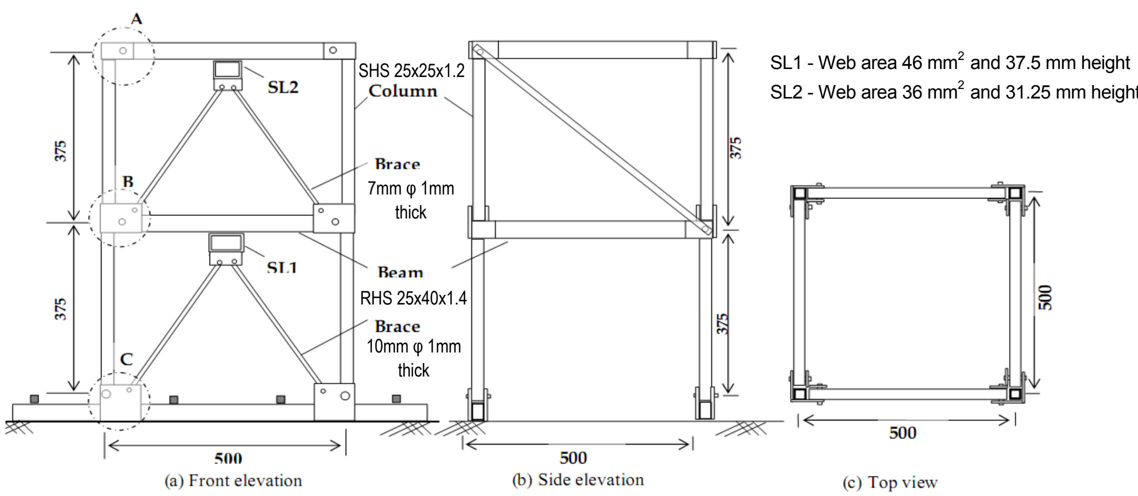
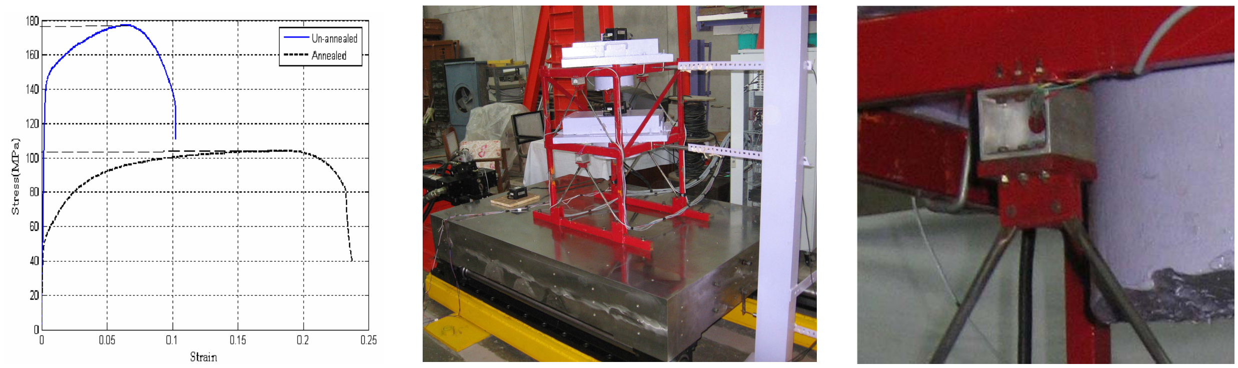


Figure 3 Model specimen frame and its members

Fabrication of Shear-link

The energy dissipation capacity of the shear panels depends strongly on the mechanical properties of the material. A highly ductile material is needed to meet the large inelastic strain demand required in these applications. In this investigation, specimens were fabricated from commonly used alloy of aluminum 1100-O. These were machined from a larger piece because of greater accuracy needed for very thin web (1.11-1.28 mm). To relieve the initial stresses and stresses developed close to the surface during machining, the shear-links were heat treated. The mechanical properties of the material used in the test specimens were obtained from uni-axial tension tests on un-annealed and annealed coupons. The coupons are made from the strips, which are cut out of the solid square section. The 0.2% proof stress for unannealed 1100-O was about 150 MPa while after annealing the yield stress reduced to 53.6 MPa, as shown in Fig 4 (a).



(a) (b) (c)

Figure 4 (a) Tensile stress-strain curve for aluminum alloy used (b) Specimen mounted over the shake table (c) Close up view of shear link.

Test Set-up

The uni-axial shake table used for the out-of-plane testing has the dimensions of 1.8 m x1.2 m (length x width) with 1.8 m length in the direction of motion. [Sinha and Rai 2009] The test specimens were instrumented with accelerometers, transducers and strain gauge to measure the required response. Two accelerometers, one at each floor level were placed to measure the floor's absolute accelerations. Similarly, table acceleration was measured using the accelerometer attached to it. Displacement transducers (LVDTs) were used to measure floor displacements. At the mid-length of each column and brace, strain gages were used to measure axial force. A sampling period of 0.0026 s (393 Hz) was adopted for acquisition of data through a high performance Data acquisition system. Figure 4 (b) represent specimen mounted on the shake table.

Loading History and Earthquake Simulation

A series of earthquake simulation tests were performed by using the uni-axial shaking table. The target accelerogram of the table was the recorded component of the 1952 Taft earthquake (TaftN21E with PGA of 0.15g). A comparison of acceleration response spectrum of Taft ground motion is performed with response spectrum used for the design of the prototype and model as per IS 1893 (BIS 2002). The scaled Taft motion with PGA equal to 0.2g matched reasonably well with the code spectrum in highest seismic zone as shown in Fig. 5 and it can be considered to represent the 'intensity' or 'severity' implied by the design-basis earthquake (DBE). As a result, the test run TAFT04 will be taken to represent DBE level ground motion and others test runs can be expressed as percentage of DBE, for example, TAFT08 can be referred as 200%DBE. The 150%DBE can be regarded as the code-level design earthquake with importance factor of 1.5 used for structures with higher consequences of failure, such as hospitals, schools, monuments, etc. Similarly, 200%DBE can be considered to represent maximum credible earthquake (MCE), whereas the 400%DBE is likely to represent a 'catastrophic' level earthquake. The prototype and model structure were designed for 150%DBE as they were intended to serve a community building.

Based on the similitude rules the time axis was compressed by a scale factor of $1/\sqrt{24}$ (Table 1). The original Taft ground motion, scaled motion and their response spectra are shown in Fig. 5. For several runs of time scaled Taft motion, only PGA was varied in increments of 0.1g, starting with 0.1g and carried up to the failure of specimen. A white noise test (PGA=0.05g) was also conducted to investigate the change in the stiffness properties of the model after each earthquake simulation test.

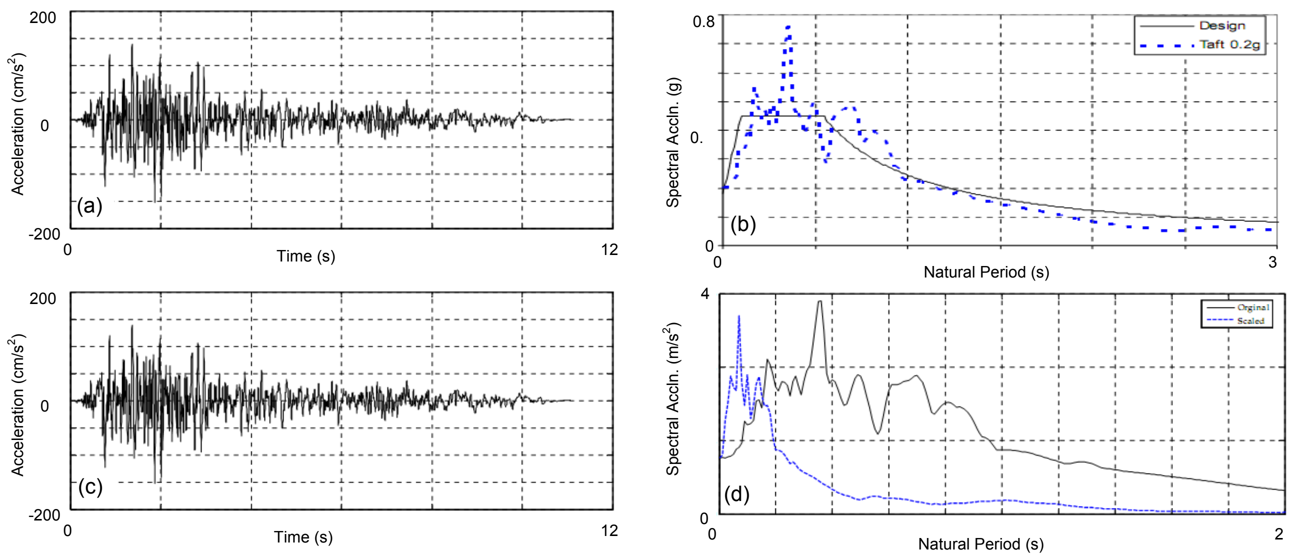


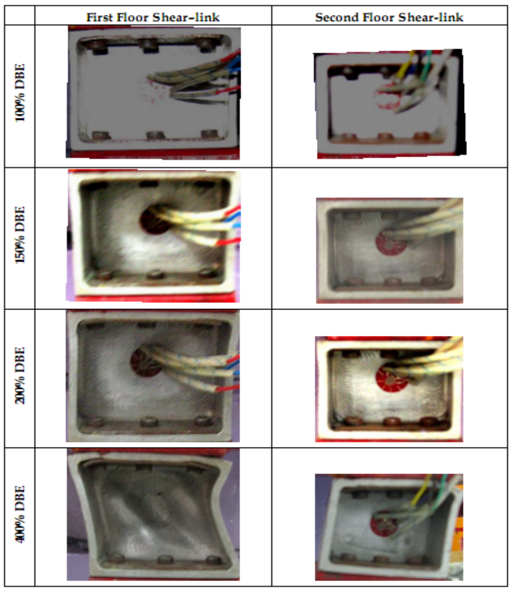
Figure 5. (a) Original Taft N21E component of 1952 Taft earthquake (b) Comparison of design response spectra and Taft 0.2g response spectrum (c) Scaled motion (d) Response spectra comparison of the original and scaled ground motion.

EVALUATION OF EARTHQUAKE TEST RESULTS

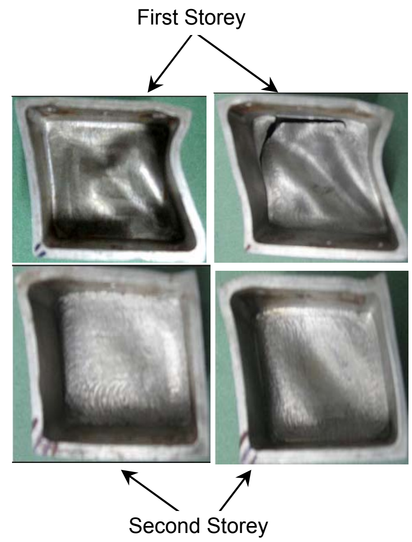
Overall Behavior and Failure Mechanism

The response of a system to dynamic excitation depends on dynamic properties, such as fundamental period. The observed natural periods obtained from forced vibration test for SLBF and OCBF were 7.3 Hz and 9.8 Hz, respectively. For earthquake excitation below 100% DBE (i.e., model PGA of 0.4g), no inelastic activity or damage was noticed for both OCBF and SLBF. However, at 125%DBE, elastic buckling of first floor braces in OCBF was observed along with a premature failure of pin connection (Event 1). The pin was replaced and further tests were continued. The permanent bending of second floor braces of OCBF occurred at 200%DBE. During the test run corresponding to model PGA of 1.1g (i.e., 275% DBE), the second story brace fractured which seriously undermined the lateral load capacity of OCBF and further test runs were not carried out.

In contrast, for the same level of excitation (275% DBE), only inelastic activity in SLBF was the buckling of the first floor shear-links, whereas all other members were in elastic region. The buckling of second floor shear-links was noticed for 375%DBE. The SLBF continued to sustain even higher levels of excitation and when the test was stopped at 425%DBE, severe buckling had taken place in shear-links of both stories. However, no other member of the frame had experienced any inelastic deformation or damage. The buckled shear-links after the last test run TAFT17 corresponding to model PGA of 1.7g are shown in Fig. 6. At the end of the test, no tearing of web was observed, though very deep buckles (folds) were present in the web. Web tearing in one of the first floor shear-link occurred suddenly when it was being removed from the frame, possibly due to readjustment of internal stresses at the release of restraint.



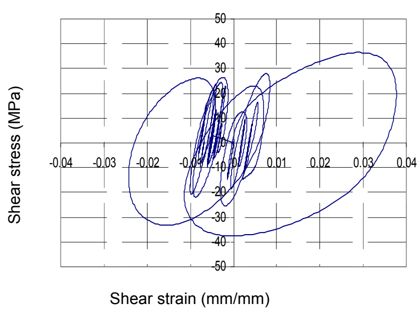
(a)



(b)

Figure 6. (a) Deformed configurations of shear-links at different level of excitation and (b) Deformed shear-links after the TAFT17 Test (425% DBE)

Fig. 6 shows the deformed configuration of shear-links at various levels of excitation. As discussed above, until 200%DBE, the first story links suffered significant plastic shear distortion, while the second story links 'nearly' maintained the original geometry. Significant yielding and buckling of web, flanges and end stiffness was noticed at the excitation level of 400%DBE. Despite such visible distress, the links maintained structural integrity and that of the SLBF at such a high intensity of shaking.



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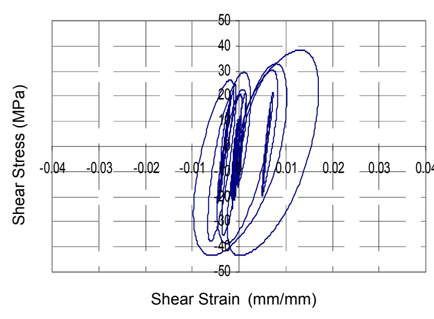


(b)

Figure 7. Hysteresis response of shear-links at 200%DBE (a) SL-1, and (b) SL-2

Shear stress in shear links was obtained from the horizontal component of brace forces divided by link web area, whereas shear strains were directly measured using strain rosette in the web. At 100%DBE, shear-links experienced very little or no inelastic deformations, which increased only marginally at 150%DBE. Shear-links in the first-story experience larger shear stress in comparison to second story links. Further, even at 200%DBE, the maximum shear strain was about 4% in first storey and about 2% in the upper storey. Throughout all test runs, shear links at different floors have shown the stable hysteresis loops. (Refer Fig 7)

Acceleration Response

Peak accelerations at different floors of SLBF system are substantially smaller than those observed for OCBF system for all acceleration levels as shown in Fig. 8. In other words, the inertial forces developed at different floors, which need to be resisted by the bracing system were smaller in SLBF frame in comparison to OCBF system. The peak values in the SLBF system increased at progressively decreasing rate. The SLBF system had shown better performance in reducing the peak floor accelerations during the tests of higher PGAs from 0.9*g* to 1.1*g* (corresponding to 225% to 275%DBE).

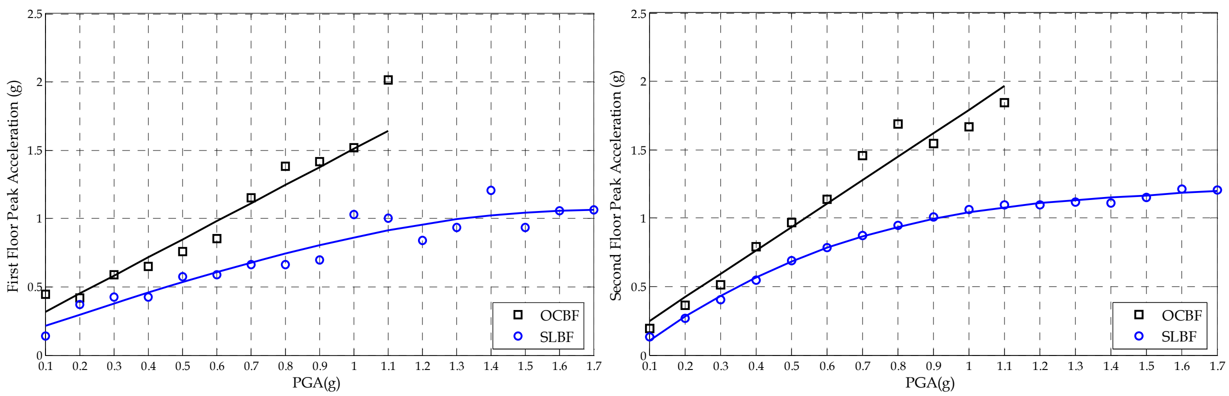


Figure 8 Comparison of peak accelerations of OCBF and SLBF (a) First floor (b) Second floor

Base Shear and Overturning Moments

Base shears were determined from the first storey brace forces which were derived from the axial strains measured using strain gauges. Similarly, base overturning moments (OTM) were derived from the column axial forces measured from axial strains in the first story columns.Base shears indicate the global seismic demand imposed on the model structure for various excitation levels of the TAFT ground motion. As noted in the case of floor acceleration response, the general characteristics of force quantities, base shear and OTMs, do not change significantly with increase in earthquake excitation level. Further, the nature of time-histories of base shear and OTMs are nearly similar, indicating that columns primarily resisted OTMs, while the lateral shear was almost entirely resisted by the braces and/or shear-links.

The SLBF system attracted lower base shear than OCBF during all simulation tests and the reduction ranged from about 47% to 64%. Moreover, it can be observed in Fig. 9 that the SLBF system attracted base shear at a progressively decreasing rate. This can be attributed to the fact that the further yielding of shear links at both floor levels made the SLBF system more flexible, and consequently, it attracted base shear at much decreased rate and was limited by the post-yield/buckling capacity of shear-links.

Similarly, the SLBF system was subjected to lesser overturning moments than the OCBF system. Reduced maximum base overturning moments resulted in lesser peak forces in the primary members. It is clear from these observations that SLBF system is capable of limiting the forces transferred to the primary structural members, at all levels of ground motion. As in the case of base shear, the overturning moment also increased at a progressively decreasing rate with increasing severity of ground motion as shown in Fig. 9. These results clearly indicate the effectiveness of shear-links in limiting the seismic demand on the structure as a whole and in the primary structural member such as columns and braces.

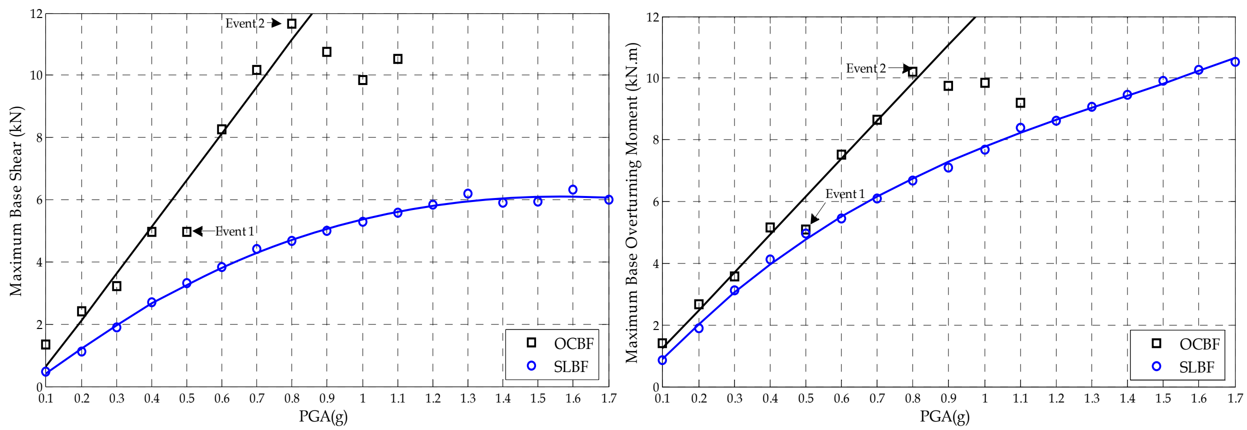


Figure 9 (a) Comparison of Maximum Base Shears of OCBF and SLBF, and (b) Comparison of Maximum Base Overturning Moments of OCBF and SLBF

Floor Drifts and Floor Displacements

The first floor storey drifts of SLBF were always greater than the OCBF system for all test runs, where as the second storey drifts of SLBF were not larger than those of OCBF. The first floor storey drifts of SLBF were always greater than the OCBF system for all test runs, whereas the second storey drifts of SLBF were not larger than those of OCBF (Fig. 10). However, the floor displacements for SLBF are larger than those of OCBF indicating that the most of the lateral deformations of the frame was concentrated in the first storey shear-links in case of SLBFs. Further, the increased storey drifts were not so large to cause extensive non-structural damages; they were about 1% at 150%DBE which increased to about 1.3% for 200%DBE excitation. The SLBF system had increased floor displacements in comparison to OCBF by a factor of 1.03 to 1.14.

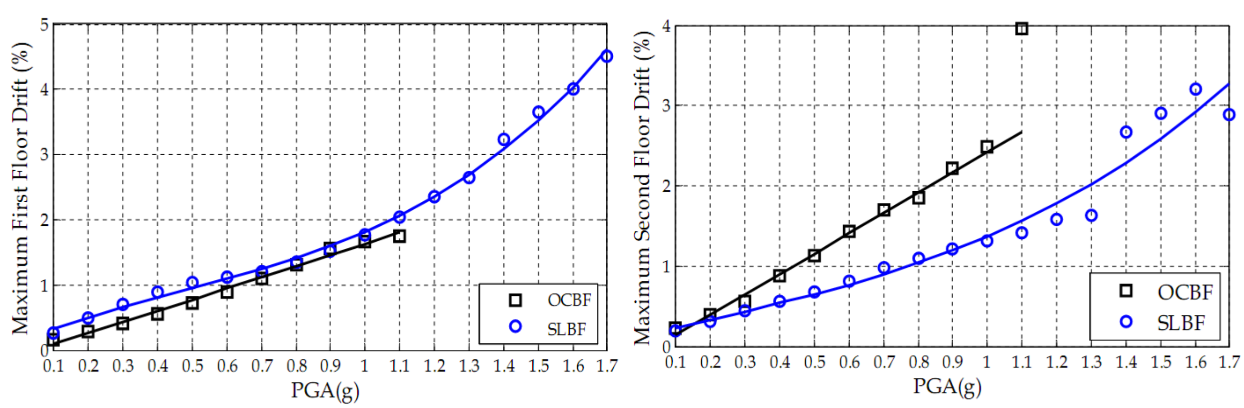
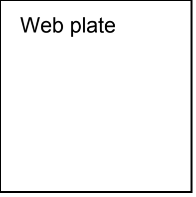


Figure 10 Comparison of peak drifts of OCBF and SLBF (a) First floor and (b) Second floor



SUMMARY AND CONCLUSIONS

Reduced scale models (1:12) of ordinary concentric braced frame (OCBF) and frame supplemented with aluminum shear link (SLBF) were subjected to a series of scaled Taft ground motion with increasing severity, and the results were evaluated and compared:

- The SLBF system attracted less base shear during simulation tests compared to OCBF. Peak base shears were observed to be progressively decreasing with increasing severity (PGA) of ground motion with maximum reduction of about 50%. Similarly, overturning moments and floor accelerations were substantially smaller in SLBF than that observed for OCBF system.
- In SLBF, all inelastic activities were confined to shear-links as expected, while the other structural members remained in the elastic range even upto 1.7*g* PGA of simulated motions. In comparison, braces of OCBF buckled permanently and deformed at much lower PGA of 1.1*g*.

Acknowledgments

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