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# SEISMIC ASSESSMENT OF THE "STRONGBACK" SYSTEM

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## *Abstract*

Conventional steel braced frames have a tendency to form weak stories during strong earthquake shaking, concentrating damage in a few stories while the rest of the frame contributes little to the structure's ability to dissipate energy. A two-story, one-bay "strongback" (SB) braced frame was tested under quasi-static cyclic loading conditions to assess the system's ability to mitigate this weak story behavior as part of a strategy to retrofit an existing building. The SB system employs an elastic mast that is pinned at its base to the foundation and runs over the full height of the structure. The mast imposes a displaced shape that increases linearly with height, resulting in nearly uniform drift demands in all stories. The test specimen arranged the braces in a "lambda" configuration, with a single buckling restrained brace (BRB) in the bottom story that acts as an energy dissipating "fuse" and two HSS braces that are part of a relatively strong vertically-oriented truss, or "mast." Test results show that the SB test was effective in impeding the formation of a weak story mechanism and in mobilizing the reserve strength of other structural components even after BRB fracture. Numerical results were able to capture the overall response of the frame, including the fracture of the BRB. Based on the results from this experimental test, a three-story SB system was analyzed using OpenSEES to improve understanding this system's behavior under a suite of 240 ground motions at three different hazard levels for a site in Oakland, CA.

*Keywords: earthquake engineering, structural engineering, concentrically braced frames, strongback system, full-scale testing*

## 1. Introduction

Concentric braced frames (CBFs) have been popular in the United States for both new construction and seismic retrofit for decades. In high seismic regions, current design guidelines require special provisions to ensure that steel braced frames exhibit a ductile response in the event of a strong earthquake. For example, Special Concentric Braced Frames (SCBFs) rely on the inelastic deformations of the braces to dissipate energy. This is primarily accomplished through tension yielding and compression buckling of the braces. Detailing requirements, such as the maximum slenderness ($kl/r$) and maximum width-to-thickness ($b/t$) ratios, are imposed to aid the braces in developing an adequate hysteretic response. Capacity design principles are additionally employed to avoid premature failures at the connections and yielding in the beams from brace buckling in V- or inverted V-brace configurations.

Yet despite these efforts to improve braced frame behavior, conventional concentric braced frames are consistently vulnerable to weak story mechanisms [1, 2, 3, 4, 5, 6, 7, 8, 9, 10]. Weak stories in CBFs often stem from the poor hysteretic response of the braces. The compression capacity of a buckled brace degrades with increasing inelastic deformations upon subsequent cycles. Thus, buckling of a brace in a story causes it to become relatively weaker than the stories that have remained elastic. This relative reduction in story shear strength and stiffness promotes larger amounts of damage and drift in stories with earlier or larger inelastic deformations, as shown schematically in Fig. 1(a).

The concentration in demand from a weak story triggers greater localized structural and nonstructural damage. These localized demands can cause premature brace fracture, heightened fracture and damage to gusset plates and adjacent beams and columns, significant residual displacements, and possible collapse. The amplified damage in a few levels can further make repairs technically difficult or even economically infeasible, negatively impacting the performance of the system.

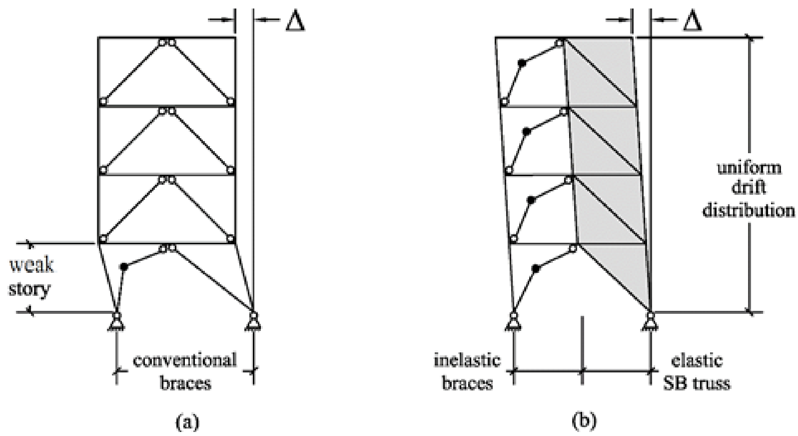



Fig. 1 – Examples of plastic mechanism: (a) conventional braced frame; (b) SB system.

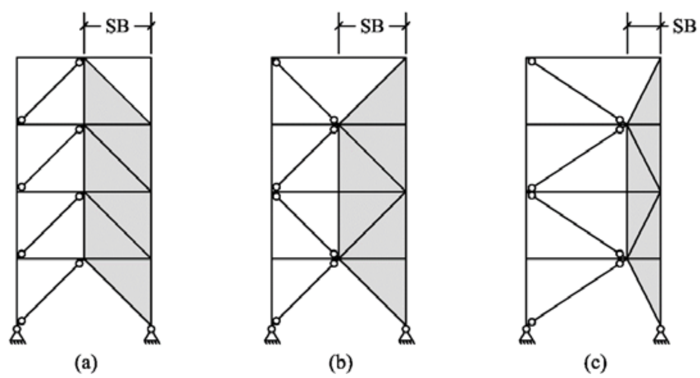



Fig. 2 – Possible SB configurations: (a) “chevron” SB; (b) double story X SB; (c) offset double story X SB

Weak stories develop from the system’s inability to compensate for the loss of story shear capacity when a brace in a story buckles. Thus, if a uniform drift distribution could be imposed over the entire height of the structure, local damage could be reduced, making it not only safer, but also more reasonable to repair after an intense, very rare earthquake.

Many researchers have explored various methods to reduce concentrations of damage in braced fames. Several approaches include: (i) the use of slender braces with relatively large tension-to-compression capacities with the ability to re-distribute the forces from the compression brace to the tension brace [11, 5]; (ii) providing a “back-up” system that utilizes framing action to carry the loss of local story shear capacity upon brace bucking, as in a dual system [12, 13, 14]; (iii) implementing a zipper frame that includes a vertical tie with an undersized beam to induce inelastic behavior in adjacent stories upon brace buckling [15, 16, 17, 18, 19, 20] and (iv) detailing the columns in both the lateral and gravity systems to help carry the load upon brace buckling, as in the continuous column concept [21].

But these methods have their drawbacks. The slender braces used in approach (i), for instance may result in large beam sizes and substantial overstrength, impacting the size of the columns, foundations, and surrounding structural elements. While dual systems have been recognized in building codes for several decades, it is unclear how strong and stiff the backup frame should be to achieve a desired performance goal [15]. Subsequent research of the zipper frame [18, 17, 19] found that it is difficult to find the appropriate member sizes needed to obtain the desired response. Finally, the distributed nature of the continuous column raises a number of design issues related to seismic detailing of the gravity load system and could potentially complicate the distribution of lateral forces to the columns in the gravity load system.

Thus, there was space to implement an adaptation of these concepts. The "strongback" approach implemented in this study represents a modified extension of past research, including the zipper frame [15], tied eccentrically braced frame [22, 23], and elastic truss system [5, 24, 25]. The strongback (SB) system forces a nearly uniform drift distribution through the use of a "mast" constructed within the bay of a conventional concentrically braced frame. While the elastic "mast" could be designed in either steel or concrete – and configured as a deep column or shear wall with a pinned base – the strongback mast shown in Fig. 1(b) is characterized by a vertical elastic steel truss that is integrated within the configuration of a traditional concentric braced frame. While the SCBF shown in Fig. 1(a) may concentrate the lateral displacement of the structure in a single story, the SB configurations in Fig. 2 are proportioned to distribute story drift demands in a uniform fashion over the height of the structure; thereby mitigating the development of localized demands that could lead to a weak story.

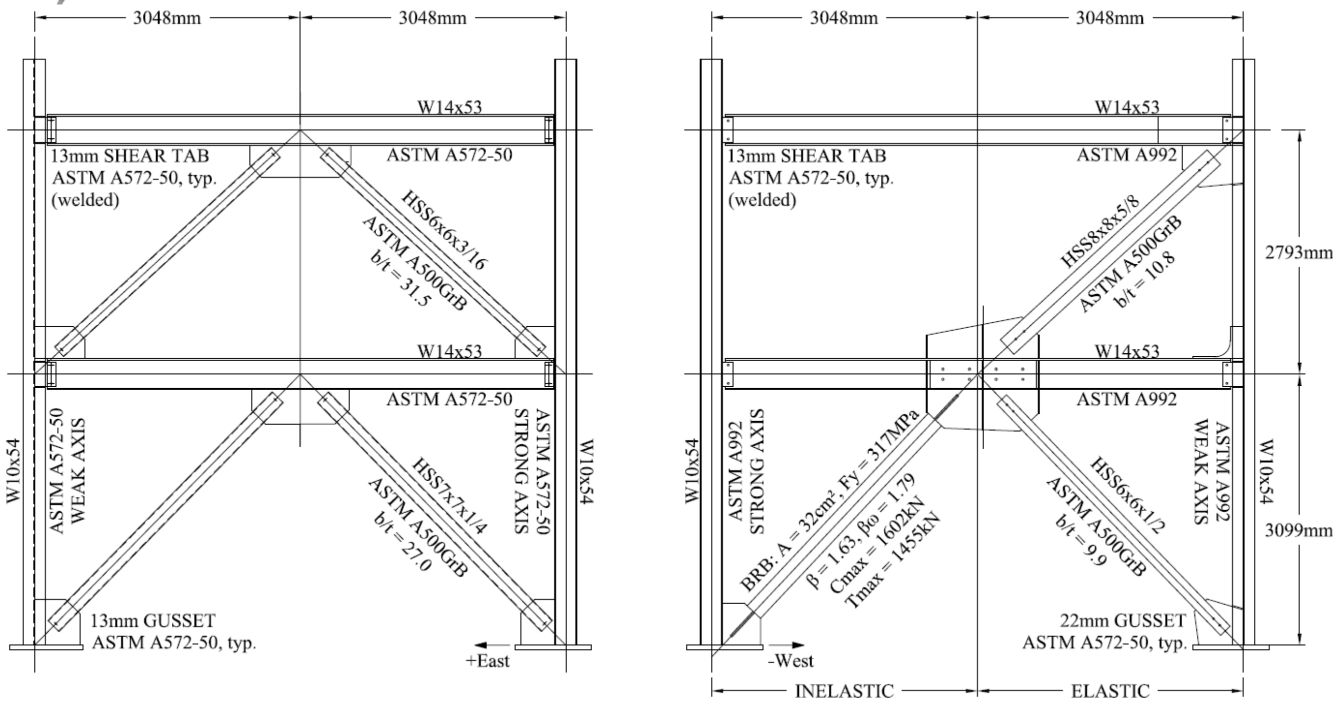
The strongback is not intended by itself to provide supplemental lateral resistance to the structure. Rather the strongback members within the shaded areas of Fig. 2 are intended to remain elastic. The base of the strongback mast is then supported by a column with adequate axial load capacity and limited bending strength and high rotational capacity; e.g. a pinned base connection or a column oriented in weak axis bending. The structure outside of this elastic "strongback" is then designed and detailed to yield, controlling the inelastic behavior in the system through either buckling restrained braces (BRBs) or conventional brace yielding and buckling behavior. Other possible SB configurations are shown in Fig. 2.

The benefit of the strongback system lies in its ability to engage the entire building to resist seismic demands. Instead of only engaging a few stories, as in a weak story mechanism, the strongback is able to average damage across multiple stories and possibly reduce the influence of higher mode effects. Since every story is engaged, the system can be designed to be redundant, permitting a more reliable redistribution of the forces after the loss of one of the inelastic braces. While the strongback portion of the system may require extra steel to remain elastic, this cost could be balanced by an allowance of fewer inelastic braces and the utilization of the same brace cross section and connection details at every story.

Research to date on "masted" systems like the SB system has focused primarily on applications to new construction. Moreover, few experimental studies have examined the efficacy of such systems, as most investigations have focused on computational simulation of the seismic response [26, 22, 5, 24, 27, 28, 29, 30]. Thus, A full-scale experimental test of a two-story, one bay strongback retrofit strategy was undertaken for this study to: (1) evaluate the behavior of a strongback braced frame under quaststatic, cyclic loading conditions; (2) establish whether weak story behavior could be mitigated through the use of a strongback; and (3) develop and calibrate analytical models to simulate a range of observed SB system behavior. Additional work is currently underway to develop and refine design methods for the strongback system using the results of the experimental test and ongoing numerical analyses.

## 2. Experimental Test

The strongback experimental test was the third in a series of experiments carried out at the University of California, Berkeley to assess and mitigate vulnerable seismic behavior in older braced frames designed prior to 1988. The first two specimens studied were representative of typical one bay, two story concentric braced frames incorporating braces fabricated from square HSS sections, as labeled as NCBF-B-1 and 2 in Fig. 3(a). While the two initial test specimens were vulnerable to a variety of local failure modes, both test specimens were consistently limited by weak story behavior. Thus, the third test specimen (NCBF-B-3SB) consisted of a retrofit scheme aimed at mitigating this weak story mechanism through the use of a strongback. The details of all three tests can be seen in Table 1. A schematic of the plastic mechanisms for each of the three tests can be seen in Fig. 4. A mores detailed discussion of all three experimental tests can be found in Simpson et al. [31].




(a) (b)

Fig. 3 – Test specimen schematic with dimensions, materials, and member sizes: (a) NCBF-B-1 & 2; (b) NCBF-B-3SB.

Table 1 – Summary of experimental test specimens.

| Specimen name | Description | Maximum base shear kN (kips) | Roof Drift at Yield[b] | Maximum roof drift[a] | Weak-story location |
|---|---|---|---|---|---|
| NCBF-B-1 | Baseline NCBF specimen | 1722 (387) | 0.41% | 0.44% | Second story |
| NCBF-B-2 | NCBF-B-1 repair: | 2412 (542) | 0.51% | 0.77% | First story |
| | (i) CFT braces; | | | | |
| | (ii) Net section reinforcement | | | | |
| NCBF-B-3SB | NCBF retrofit: SB system | 2323 (522) | 0.21% | 2.0% | -[c] |

[a] Maximum roof drift prior to observable strength degradation where the measured base shear dropped below 80% of the specimen‘s maximum capacity; [b] Yield corresponds to the first signs of dominant nonlinear behavior such as brace buckling or yielding; [c] No weak-story behavior.

## 2.1 Test Specimen Design

As a hypothetical retrofit, the design of the NCBF-B-3SB test specimen was based on the original design of the two previous tests. The beams, columns, and shear tabs were considered to be from the original NCBF design and were not modified for the strongback retrofit, minimizing the potential need for demolition and shoring in an actual retrofit situation. The original "chevron" braces were removed and replaced with new braces in a re-oriented, "lambda" configuration. New gusset connections were designed for the ends of all bracing members using current AISC provisions and basic capacity design principles, employing force distributions from free body diagrams and the Uniform Force Method at applicable connection regions.

The final lambda configuration consisted of two halves (Fig. 3(b)):

1. The column, braces, and half beam on the west (right) side of the frame were designed to remain essentially elastic throughout the test. Extensive plastic rotations were anticipated at the base of the west column, hence the column was oriented in weak axis bending to mimic a pinned base. The west

column and braces were intended to act like the strong backbone (or "strongback") for the system and distribute story drifts nearly uniformly over the height of the structure.

2. The lateral load resisting system on the east (left) half of the frame consisted of a single Buckling Restrained Brace (BRB) that acted as the primary energy dissipating device in the system. Other plastic deformations were expected at the ends of the east lower level half beam, the base of the east column, and the east shear tab connections at the lower and roof beams.

Both the elastic and inelastic halves of the system were based on a plastic analysis of the expected failure mechanism, similar to that shown in Fig. 5. The elastic strongback was further designed to be 1.1 times the maximum force that could be delivered to it by the BRB. The test followed a modified loading protocol similar to the buckling-restrained brace loading sequence found in Chapter K3.4c of the AISC 341 Seismic Provisions.

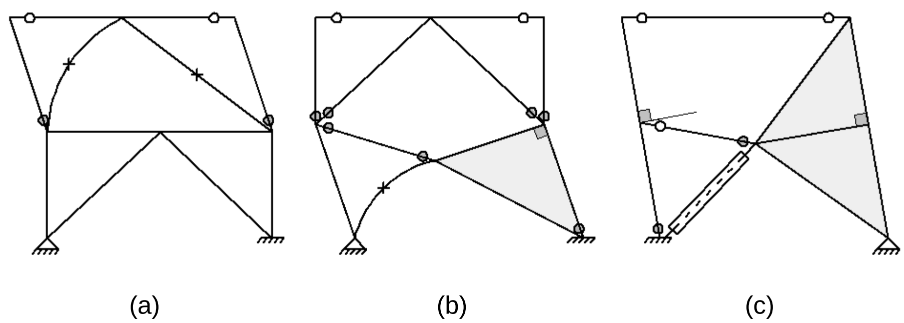



Fig. 4 – Schematic of plastic mechanisms of the test specimens:
(a) NCBF-B-1; (b) NCBF-B-2; (c) NCBF-B-3SB.

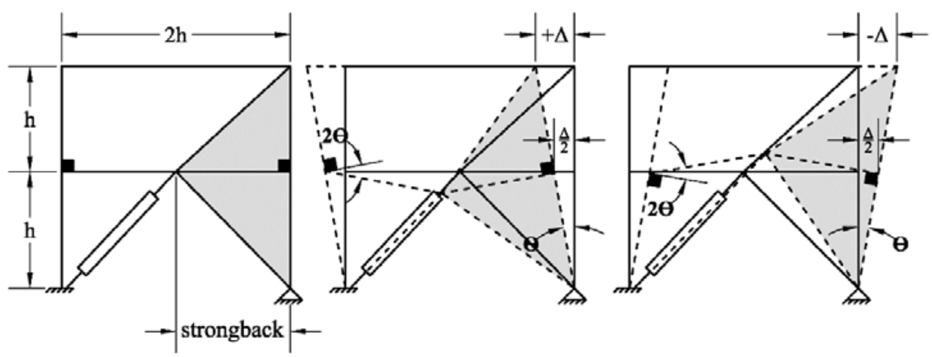



Fig. 5 – Idealized kinematic relations of strongback test with a lambda configuration (NCBF-B-3SB).

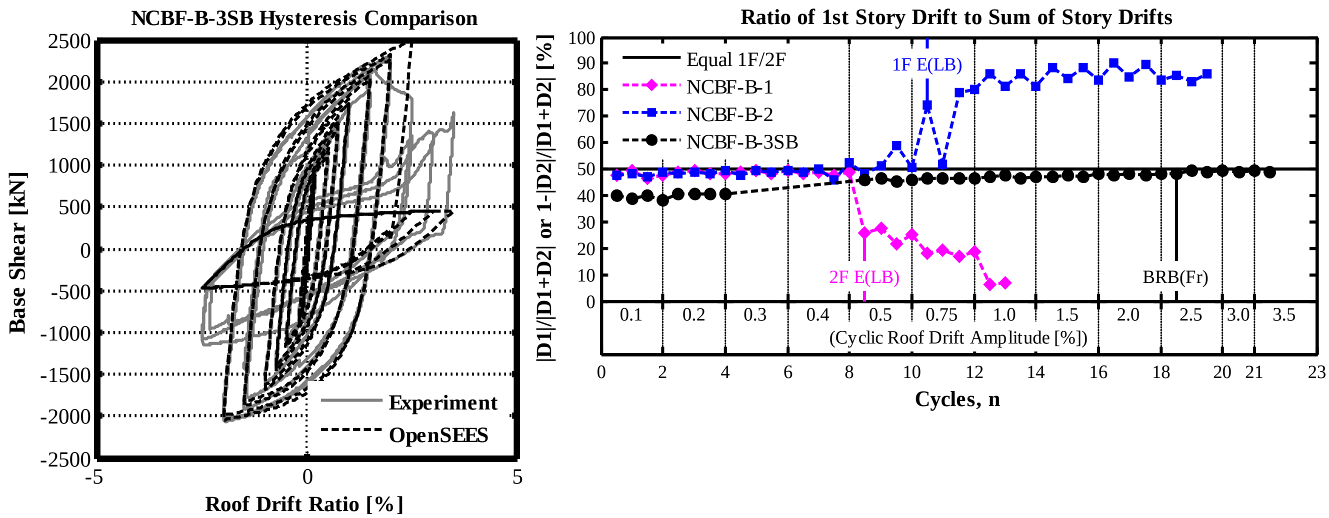
## 2.2 Experimental Results

The SB retrofit was successful in limiting a weak story mechanism, maintaining a uniform drift distribution over the full frame height during the entire test up to a 3.5% roof drift ratio. After exceeding a targeted roof drift ratio of 2% and satisfying the BRB acceptance criteria in AISC 341-10, the BRB bulged and ruptured during the first quarter cycle to a roof drift of 2.5%. In spite of this fracture, the strongback continued to avoid the formation of a weak story during several subsequent cycles up to 3.5% roof drift, as shown by the black dotted line of Fig. 6(a).

Note from the simple kinematic considerations of Fig. 5 that the plastic and shear tab rotations at the east end of the first floor beam and the strains in the inelastic brace for the lambda configuration are about double the rotations and strains of a conventional chevron SCBF configuration with the same lateral displacement. In light of these rotational demands, fracture at both the shear tab location and the BRB core were observed during the test. Thus, even though the BRB satisfied the AISC 341-10 BRB testing requirements, special attention should be placed in the design of these regions and elements due to the local concentration of inelastic demands caused by the geometry of the lambda strongback configuration.

The hysteretic loops of the NCBF-B-3SB retrofit through the 2% roof drift cycles were very full and stable. While the stiffness and strength of the frame was reduced after BRB fracture, the frame was still able to dissipate energy through smaller but stable and full hysteretic loops produced by the stiffness and strength of the beam and remaining portions of the lateral resisting frame.

Plots of the ratio of the first story drift to the sum of the story drifts at peak cyclic amplitudes for all three tests can be seen in Fig. 6(b). The second and first story for the NCBF-B-1 and NCBF-B-2 tests respectively, contribute disproportionately more to the total displacement after the start of their strength deterioration, illustrating the weak story behavior observed during both tests. In contrast, the NCBF-B-3SB test specimen exhibits similar drift ratios in both stories throughout the entire test regardless of the direction of loading, varying little from the solid line at the 50% ratio in the plot that indicates nearly equal story drift levels in the first and second stories, showing that no weak story formed for the entire test.




(a) (b)

Fig. 6 – (a)NCBF-B-3SB comparison of numerical and experimental results of global hysteresis;
(b) Comparison of the weak story tendency at peak cyclic amplitudes (Fr = fracture; LB = local buckling).

## 2.3 Numerical Calibration

A Numerical model of the experimental test was developed using the structural analysis program OpenSEES [31]. The assumptions used in modeling this numerical model are listed under section 5. Plots of the hysteretic loops from the experimental tests and numerical models are overlaid for comparison purposes in Fig. 6(a). The solid grey line in the figures represents the experimental test and the dotted black line reflect the output from the OpenSEES models. The elastic behavior of the NCBF-B-3SB test specimen is well matched by the numerical model. The stiffness in both the model and experiment are very similar and the hysteretic loops match well up to BRB fracture. While the BRB fracture was well captured by the numerical model, after fracture of the BRB, the hysteretic loops no longer match as well. This is because the BRB contributes nothing to the frame after it fractures in the numerical model. In the case of the experiment, some reserve capacity was observed in the steel core as the two fractured ends came in contact in compression and pulled apart in tension. This reserve capacity from this contact was not modeled by the OpenSEES model.

## 3. Model Building

This study assessed the seismic response of a three-story steel strongback system. The basic building plan and dimensions can be seen in Fig. 7. The building has regularly spaced gravity framing that is simply pinned at the foundation. The strongback lateral-resisting frames were spaced around the perimeter of the building. The building was designed by a professional engineering design firm [32] to meet the minimum code requirements for a commercial office building. This building is representative of many locations in California, and the assumed design parameters can be seen in Table 2.

Member sizes for the strongback frame can be seen in Table 3. Sizes for the inelastic braces, selected as BRBs, were based on a response-modification factor, $R$, of 7, typical of buckling-restrained brace frame with pinned connections. The beam sizes were selected for a member strength of 1.1 times the force delivered by the maximum expected capacity of the BRBs based on a plastic analysis and the expected kinematic response of the building. Beam plastic hinging was then designed to occur prior to plastic hinging at the base of the columns. To address the beam-column connection and BRB fracture seen during the experimental test, the strongback centerline was shifted to the third point of the beam length to allow for a greater yield length for the BRB and to decrease the amount of rotation seen by the beam-column connections in the inelastic part of the frame; see Fig. 7. Note that the design of the strongback is currently being simplified, refined, and optimized, and further improvements of the design process are being investigated.

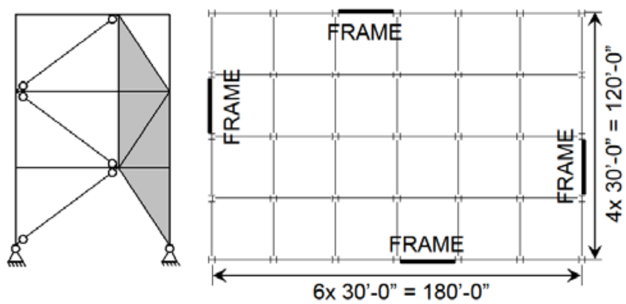



Fig. 7 – Model building floor plan and elevations.

Table 2 – Design parameters

| | |
|---|---|
| Building Location: Oakland, CA | |
| Seismic Design Category | D |
| Occupancy Category | II (office) |
| Importance Factor | 1.0 |
| Short Period Spectral Acceleration, $S_s$ | 2.2g |
| 1s Period Spectral Acceleration, $S_1$ | 0.74g |
| Soil Type D ($F_a = 1.0, F_v = 1.5$) | |
| Response Modification Factor, $R$ | 7 |
| Base Shear, $V_b$ | 0.21W |
| Period, $T$ | 0.6s |

## 4. Ground Motions

The set of ground motions used for the dynamic time history analysis were selected to match the uniform hazard spectrum and associated causal events for a site in Oakland, CA [33]. Forty three-component (vertical, fault-normal, and fault-parallel) ground motions records were selected to be representative of three different hazard levels (50%/50 years, 10%/50 years, and 2%/50 years). Fig. 8 shows that were was good agreement between the median of the selected ground motions for the 10%/50 year and 2%/50 year and the code-based design MCE and DBE response spectra used to design the building.

Table 3 – Member sizes.

| Structural element | 1st floor |
|---|---|
| Columns[1] | W14x132 |
| Beams | W18x97 |
| BRB core area | $10 in^2$ |
| Elastic brace | W12x120 |
| Vertical tie | HSS10x10x5/8 |

[1] Oriented in weak-axis bending.

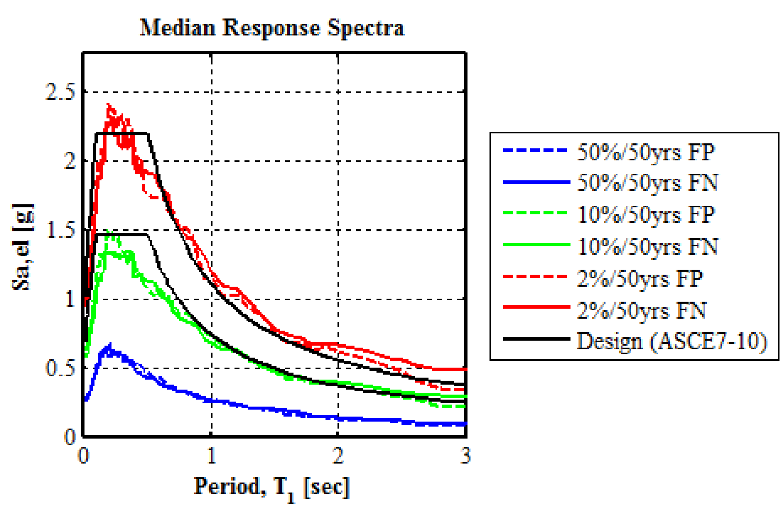



Fig. 8 – Median pseudo-acceleration of records and code-stipulated design spectra.

## 5. Analysis Model and Methods

The numerical model was implemented in OpenSEES [31]. Assumptions made in the development of the numerical models are outlined below.

1. The model was simplified as a two-dimensional model with all braces oriented to buckle in plane.
2. All brace and beam-to-column connections were represented by ideal "pins". Even though studies have shown that pins at the gusset plate connections may be an over-simplification [8, 34], this simplification was deemed acceptable for this study.
3. It was assumed that a "pin"-like connection would also be provided at the beam outside the gusset plates; e.g. as in the experimental tests done by Lai et al [9]. Rigid end zones were used to represent member depth and gusset plate length. These zones were modeled as elastic and given a moment of inertia and area to be ten times the member framing into it.
4. The weak-axis columns were assumed fixed at the base.
5. The BRBs were represented by a co-rotational truss element. Steel4 based on a Menegotto-Pinto hysteretic model was used to model kinematic and isotropic hardening.
6. The strongback braces, beams, and columns were modeled with force-based beam-column elements using Steel02 material parameters based on a Menegotto-Pinto hysteretic model with 0.3% strain hardening and the yield strength set to $R_y F_y$ of 55ksi for wide-flange members and 60ksi for HSS members, as recommended by Yang et. al. [35]. Co-rotational transformations were also provided for all members to capture large global displacements, like brace out-of-plane buckling.
7. Elastic braces were given an initial imperfection of L/1000. Smaller elements of L/20 were placed outside expected plastic hinge regions to ensure consistent strains for the calibrated fatigue parameters. Fatigue parameters [8] were calibrated from the experimental tests [36] and were similar to the calibrated fatigue parameters found by Uriz et al [8]. Three integration points were used for the braces and five integration points were used for the other members.
8. Two leaning columns were used at one bay length to either side of the frame to capture P-Δ effects. They were connected to the columns via rigid truss elements and pinned at the base. Each leaning column was given a moment of inertia and area that was the sum of the gravity columns associated with that braced bay.
9. Gravity was provided by downward point loads at the leaning columns. The gravity load was equal to the half of the gravity load per floor minus the gravity load acting on the columns of the lateral load-resisting frame. Additional point loads were added at the nodes of the strongback columns to represent the gravity load acting directly on the lateral-resisting frame. Horizontal and vertical lumped masses were provided at each column node of the strongback. This mass represented the mass of half of the floor in the horizontal direction and of the column line tributary area in the vertical direction. The mass was equally distributed between two nodes on a floor.
10. No slab or distributed load was provided for the beam to represent a cut-out slab that might be used for this type of system. Future models will take into account the effect of the slab.
11. The damping ratio was generally taken to be 3%. Rayleigh coefficients were calculated based on two periods, $\mathrm{T_i = 1.5T_1}$ and $\mathrm{T_j = T_3}$, where $\mathrm{T_1}$ is the fundamental period and $\mathrm{T_3}$ is the third mode period [37]. Note that this coefficient was taken as 2% based on $\mathrm{T_i = T_1}$ and $\mathrm{T_j = T_3}$ for the 50%/50yr hazard where there was likely to be only limited yielding.
12. Each frame was subjected to one horizontal and one vertical component of ground motion.

## 6. Numerical Results

While a variety of different parameters could be used to evaluate a building's behavior, peak story drift, peak absolute floor acceleration, and peak residual story drift were used in this study. Results are plotted in Fig. 9 for the fault normal direction and for each hazard level. The behavior of the strongback frame was slightly better for the fault parallel direction, and while those results will be discussed, for brevity those plots for the fault parallel direction will not be shown in this paper.

It can be seen from the plots that the story drift ratio is nearly uniform for all of the earthquake cases, indicating that the strongback was successful in imposing a nearly uniform deformed shape for all but seven of the earthquakes, one in the fault-parallel direction and six in the fault-normal direction. Not a single case of fracture in any of the members was observed for all 240 ground motions. Story drifts were generally less than 2.0% for the 10%/50 year hazard level and met general code requirements. Absolute accelerations can be seen to be generally uniform. This is because the BRBs in every floor are engaged and yield. Relative floor accelerations tended to increase with story height and reflected the linear story drift distribution.

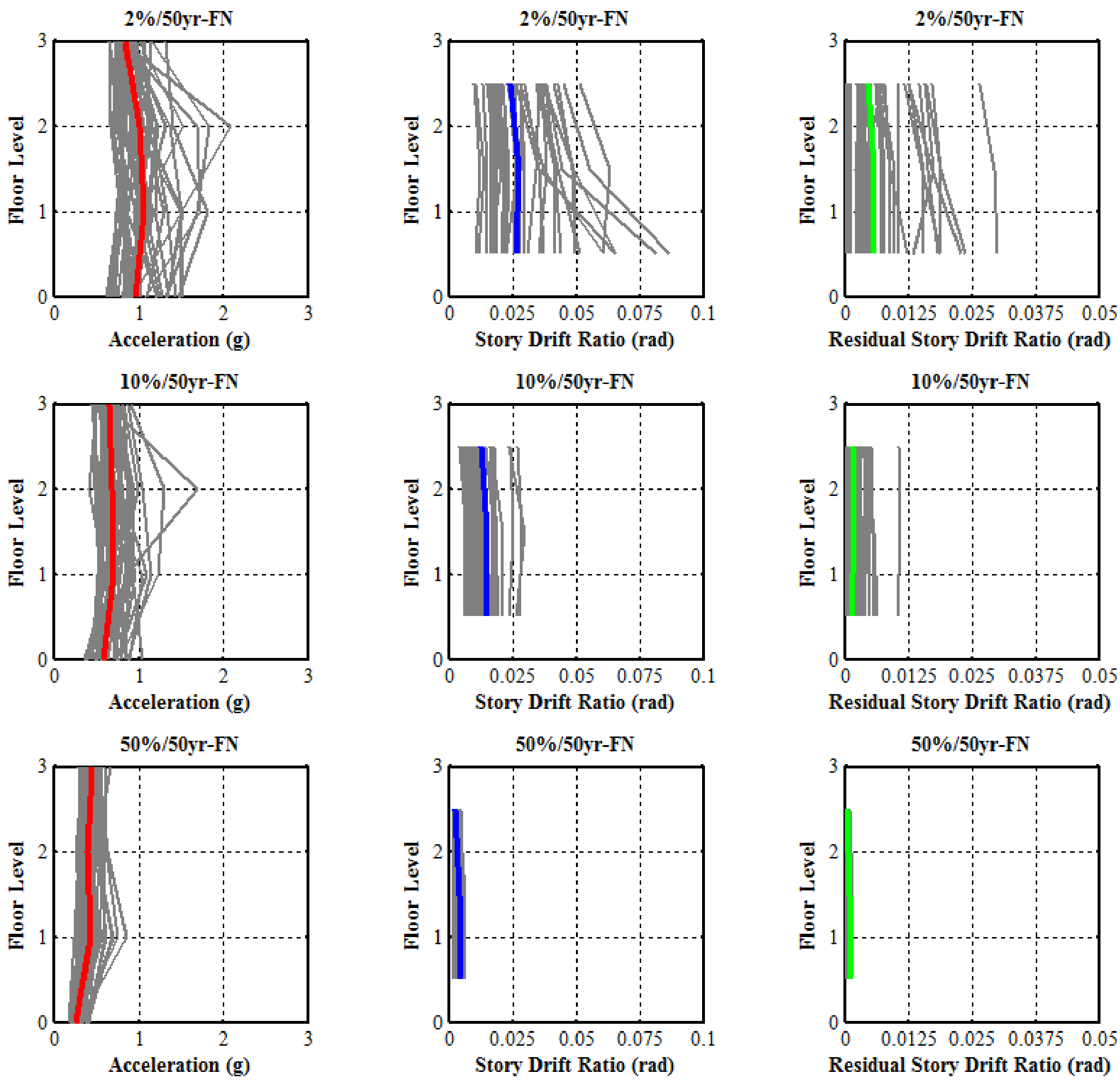



Fig. 9 – Engineering demand parameters and the median response:
peak absolute acceleration, story drift ratio, and residual story drift ratio.

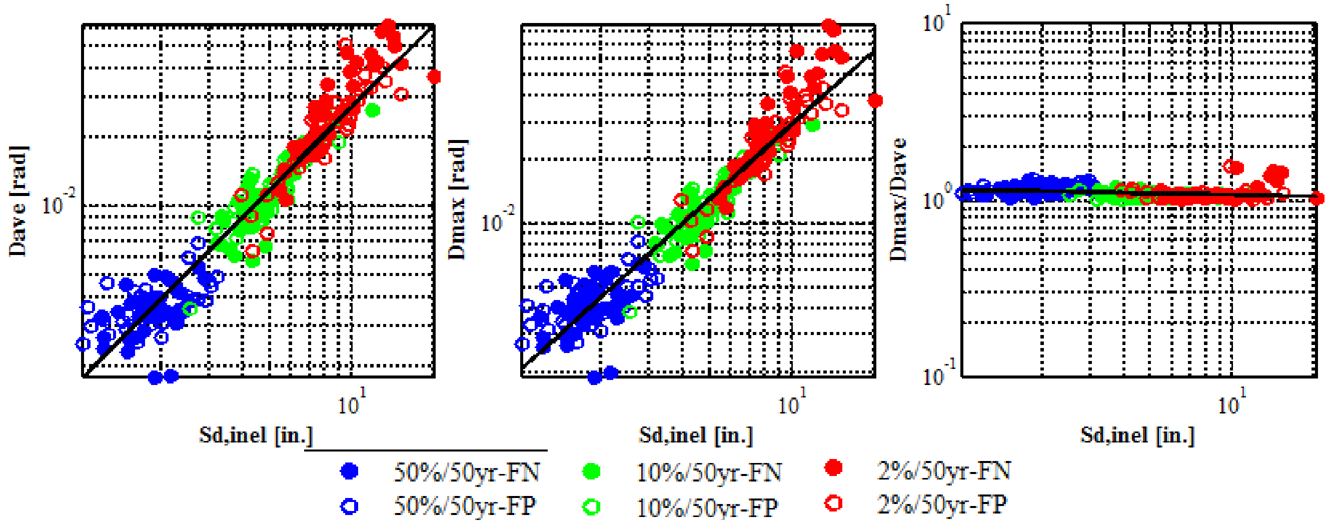



Fig. 10 – Relationship between the peak story drift ratio and $S_{d,inel}$.

Seven ground motions showed brace buckling in the strongback at the first story for the 2%/50 year hazard level, as shown by the nonuniform story drift plots in Fig. 9. These seven instances of inelastic behavior in the strongback corresponded to larger residual drifts than when the strongback remained relatively elastic.

The median residual drift was less than 1% for both the 50%/50 year and 10%/50 year case, but was significantly higher for the 2%/50 year cases, especially for the fault-normal direction. These results may indicate that re-centering systems may be desirable for the strongback system depending on the desired performance of the building.

Fig. 10 shows the log-log relation of the peak story drift ratio to $S_{d,inel}$, the inelastic spectral displacement for the record used in the analysis at the fundamental period of the model. In the plots $D_{max}$ and $D_{ave}$ represent the maximum story drift ratio over all three stories and the average (or roof) story drift ratio respectively. The ratio of $D_{max}/D_{ave}$ indicates the tendency of the system to form a weak story. From the plot of this ratio in Fig. 10, it can be seen that the strongback keeps this average at approximately 1.0 for all but seven of the ground motions, demonstrating no weak story formation in 233 of the 240 ground motions.

## 7. Conclusions

The SB system behaved well and as intended during the experimental test, mitigating a weak-story mechanism, even after the rupture of the BRB, the primary energy-dissipating mechanism. The beam appeared capable of participating as a secondary energy-dissipation mechanism through plastic hinging near the middle gusset connection. The weak-axis column was also capable of having large rotational demands, allowing the "strongback" half to reach larger lateral displacements. The system's hysteretic loop was full and stable, with no indication of degradation until after the 2% design roof drift ratio. The components of the "strongback" half exhibited only minor damage at the end of the loading protocol, and plastic hinge regions were well predicted by a simple kinematic diagram of the frame's failure mechanism; see Fig. 5.

A numerical study of a three story strongback frame with a shifted centerline was successful in reducing the large rotations at the beam-column connections and strains in the BRBs that were seen during the experimental test. Results from the numerical study indicated that a shifted geometry reduces axial strains in the BRBs and no instances of BRB rupture were observed for any of the ground motions. Of the 240 ground motions used in the numerical study, only seven indicated weak story behavior after the first story brace buckled in the strongback during the severest levels of ground shaking, indicating that the strongback can be successful at mitigating a weak story mechanism. However, further research is still being conducted to improve the strongback's performance. Currently guidelines are being developed to improve and refine the design of the strongback, including an examination of additional recentering capabilities added to the strongback's design. An economic evaluation of the strongback system in terms of both initial and repair costs is also underway.

## 8. Acknowledgements

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