17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

SEISMIC RETROFIT OF A STEEL BRACED FRAME BUILDING USING STRONGBACK SYSTEM

J. Lai[(1)]

[(1)] *Ph.D., PE, Design Engineer, Degenkolb Engineers, USA, jlai@degenkolb.com*

Abstract

Steel concentrically braced frames are prone to forming a story mechanism during strong earthquake if not designed properly. This kind of deformation concentration may intensify damage in one level or several levels, eventually having greater nonstructural or structural damage at these levels compared to buildings with more uniform distribution of deformation over the height. To reduce the tendency of forming soft-story in steel braced frame, an elastic spine (or strongback) is typically introduced to form a hybrid structural system. The purpose of the hybrid strongback system is to promote uniform deformation over the height of a structure. This paper presents a case study of seismic upgrade of a steel braced frame using strongback system (SBS). Two SBS seismic upgrade options are studied and a series of nonlinear response history analyses are performed to compare the global and local dynamic response of three systems. Results generally show that the SBS can effectively reduce the tendency of deformation concentration with a simple design strategy. Finally, simplified retrofit cost estimates are presented and compared between two retrofit options.

Keywords: concentrically braced frame; strongback system; seismic retrofit; nonlinear dynamic procedure

1. Introduction

It is well recognized for a long time that steel concentrically braced frame is an efficient lateral force resisting system. However, it is prone to form a soft-story mechanism during strong earthquake [1, 2, 3, 4, 5]. This concentration of deformations intensifies damage to braces at certain floor levels that causes greater structural and nonstructural damage, and premature failure of braces at these floor levels compared to buildings having uniform distribution of deformation over the height of the building. The concentration of deformations can amplify the global P-Δ effects, which will in turn increase lateral drifts in the softened stories. Also soft stories are more likely to result in larger residual displacements after earthquakes, which could be infeasible or expensive to repair.

Therefore, it is desirable to enhance the ability of steel concentric braced frames to prevent deformation and damage concentration in a few stories. If the system can mitigate soft or weak story behavior, the brace maximum deformation demands and maximum residual deformations might be reduced. To date, several approaches such as dual system [6, 7], zipper frame system [1, 8], rocking system [9, 10] and strongback system (SBS) [11, 12, 13, 14, 15] have been studied by various researchers worldwide to reduce deformation concentration and achieve smaller building residual drifts.

This paper focuses on the study of SBS seismic behavior and presents a case study of seismic retrofit of a steel braced frame building using SBS systems. The as-built building and two retrofit alternatives are examined. One near site record as well as a suite of ground motions (BSE-1E) considered representative of the building site are selected for nonlinear response history analyses (NLRHA). The NLRHA results are compared to assess the ability of the SBS system to minimize or eliminate soft story behavior and differences in dynamic behavior for the different brace types considered. Comparisons of the simple cost estimates for the materials used for each system, and the global demands are used to evaluate the SBS system and develop recommendations for retrofit design and future study.

2. Strongback System (SBS)

The concept of strongback system is originated and extended from zipper frames [1], tied eccentrically braces frames [11, 12] and elastic truss system [3, 13, 14, 15]. The system introduces vertical tie elements over the height of a braced bay and connects the locations where the diagonal braces intersect along the beam spans. As shown in Fig. 1, segments of the augmented braced bay are proportioned to provide a continuous and elastic vertical truss that prevents potential soft story mechanisms. The vertical truss (see Fig. 1) provides an elastic mast that imposes a nearly uniform lateral deformed shape over the height of the building. In general, there are three main components in the SBS system: (1) elastic vertical strongback truss; (2) appropriately detailed pin or fix connections at the base of SBS; and (3) braces and beams outside of the strongback mast are sized and detailed to yield, and thus either conventional braces or buckling-restrained braces (BRBs) can be used in conjunction with the SBS system.

Several possible brace configurations and SBS spines are shown in Fig. 2. With proper sizing of the SBS spines, the designer may have greater flexibility in locating and orienting the braces that yield. Note that the spines are not limited to vertical trusses, other essentially elastic systems such as steel or reinforced concrete shear walls, mega plate girders, and so on, could be used for the strongback spine. As shown in Fig. 2(d), the brace intersections at the floor beams can be shifted from the beam mid-span, which can facilitate in proportioning the load to various members in the SBS. In the case where the vertical elastic truss portion of the bay is narrower than half the bay width, making the inelastic elements longer so that they have greater length over which to yield. Reducing the inclination angles of the inelastic braces has the benefit that they can be smaller to resist the same lateral forces. In addition, for large lateral building drift, the beam length in the inelastic portion of the bay will be longer, reducing its shear and the plastic hinge rotation demands that might form at the ends of the beams.

In the case study present here, strongback bays with conventional bucking braces (SBS-01) and strongback bays with BRBs acting as energy dissipation devices (SBS-02) are considered (see Fig. 2(a) for example).

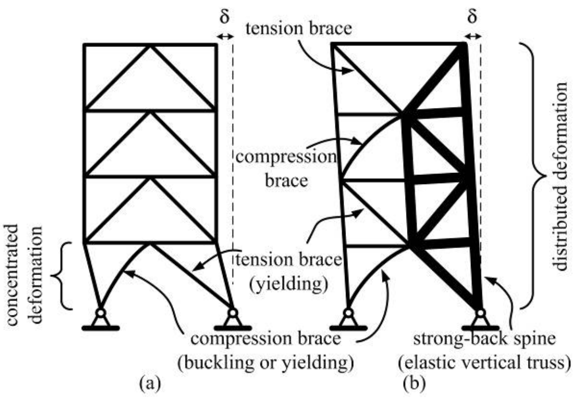


Fig. 1 – Comparison of CBF drifts: (a) conventional; (b) strongback system, extracted from [14]

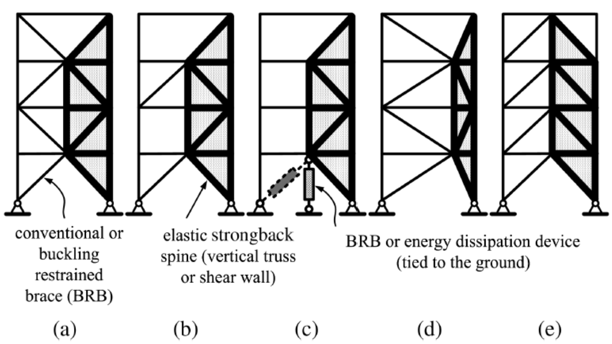


Fig. 2 – Different strongback system configurations, extracted from [14]

3. Building Descriptions, Retrofit Schemes and Modeling Approaches

3.1 As-built Building Description

The case study building is a four-story steel concentrically braced frame office building located at North Hollywood, California of the United State [16]. The building was built in 1986 and originally designed per

1980 Los Angles Building Code (LABC). The aerial view from the East side of the building is shown in Fig. 3 and the representative 2^{nd} floor framing plan is shown in Fig. 4. Typical floor diaphragms consist of 3-inch concrete fill over 3-inch metal deck spanning to wide flange floor beams. The gravity framing system consists of steel beams and columns with typical 36-ft (10,973 mm on Fig. 4) spacing in each direction. Floor heights are 21-ft (6,401 mm), 15-ft (4,572 mm), 15-ft (4,572 mm) and 16-ft (4,877 mm) for the first, second, third and fourth floors, respectively. The lateral force resisting system mainly consists of six steel braced bays in each principal direction (except two braced bays at building East side) as shown in Fig. 4 and most of the existing braced bays are configured around the perimeter of the building with inverted-V (Chevron) bracing configuration. All existing steel braces are hollow structural sections where HSS14 x 14 x 1/2 braces are used in the 1^{st} story, HSS 12 x 12 x 3/8 braces are used in both 2^{nd} and 3^{rd} story, and HSS 10 x 10 x 5/16 braces are used in the 4^{th} story. Foundation system consists of the combination of drilled piers and spread footings. The drilled piers are used below the steel braced bays to resist the system overturning. Previous site investigations indicated that the building site is underlain by medium dense to very dense alluvial cohesionless soils, which can be classified as Site Class C per ASCE 7-10 standard [17].

Fig. 3 – Aerial view from the East side of case study building (Google Earth)

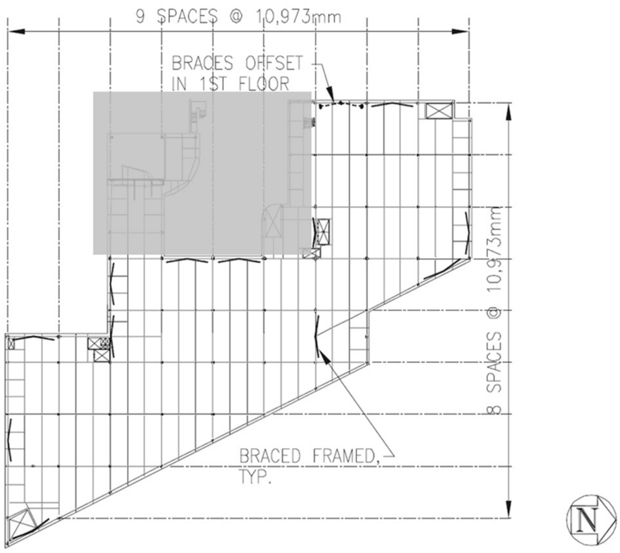


Fig. 4 – Case study building 2^{nd} floor framing plan (3^{rd}, 4^{th} and roof similar), extracted from [16]

3.2 Retrofit Schemes and Design Strategies

It should be noted that the case study building was retrofitted using zipper frames after the 1994 Northridge earthquake where structural and nonstructural damages were reported [16, 18]. At the time when building was retrofitted, the development of elastic spine concept (i.e. strongback system) was not that mature and the three-dimensional nonlinear dynamic analysis tools were not widely available in practice. Therefore, it is worthwhile to evaluate the building again using advanced nonlinear dynamic analysis tools, modern modeling approaches and the newly developed SBS systems that will improve the braced frame building global performance.

The current study focuses on the retrofit schemes using SBS configuration as illustrated in Fig. 2(a). The nonlinear components outside the elastic spines are either the conventional buckling braces or steel BRBs with conventional yield strength ($F_{y,\,brb}$ = 42 ksi = 289.6 MPa). A total of three nonlinear models are examined in this study. The as-built chevron (inverted-V) bracing configuration is used as the benchmark and is designated as model IV, while the two retrofitted building schemes are designated as model SBS-01 and model SBS-02, respectively. The selected split-X brace configuration will reduce the unbalanced loads in the braced bay beams and thus minimize the retrofit works in the existing floor beams.

For the retrofit design strategies, in addition to the basic design requirements stipulated in ASCE 7-10 standard [17] and the AISC Seismic Provisions [19], the members in the vertical elastic spine are designed to remain essentially elastic under the target design seismic forces. The design concept used here is based on the code-specified system over-strength factor (which equals to 2.0 in this case) and it is proved in previous study [14] that designing the elastic spine elements using this approach can achieve uniform deformation distribution in braced frame systems. Although the vertical spine is designed to remain elastic, it is expected that under severe ground shaking some members in the spine will be subjected to inelastic demands. The main goal behind using the simple design strategy is to design a system that achieves the goal of preventing deformation concentration in the system at little increased cost.

3.3 Modeling Approaches

Three-dimensional nonlinear models are developed using PERFORM-3D [20]. All framing members in the gravity system and the lateral force resisting system are modeled explicitly in PERFORM-3D program. Note that the shaded gray area in Fig. 4 does not have significant contribution to the building global stiffness and lateral strength, thus the framing members within this area are not modeled in the nonlinear model. Only the seismic masses from those members are considered. In the dynamic model, seismic masses are lumped to nodes based on the tributary areas. Floor diaphragms are assumed as rigid diaphragms. Rigid end zones are applied at member ends based on the actual member sizes in the models. Pinned connections are assumed at every brace end. A total of 2% damping ratio with combination of modal damping (1.75%) and Rayleigh damping (0.25%) is used for all three models. The P-Delta effects are considered in PERFORM-3D models. Soil-structural interactions are not considered in this study and the column bases are assumed as pin connections.

For the modeling of nonlinear elements, steel columns are modeled using frame member compound components with lumped plasticity PMM hinges at top and bottom of columns, steel beams are modeled using frame member compound components with lumped plasticity moment hinges (rotation type) at potential hinge locations, conventional bucking braces are modeled using inelastic bar elements and the BRBs are modeled using BRB compound components in PERFORM-3D. All nonlinear hinge modeling parameters and acceptance criteria are developed per ASCE 41-13 standard [21].

4. Damaged Observed in the Northridge Earthquake

Soon after the 1994 Northridge Earthquake, the case study building (as-built condition) was reported with non-structural and structural damages [16, 18]. Three key findings related to structural damages are listed below:

(1) The structural damages are concentrated mostly in the 2nd story braces and less damage reported in other stories.

(2) More structural or non-structural damages along the North-South direction than the East-West direction are observed in this building. It is consistent with other damaged buildings around the region investigated by engineers and it is consistent with the actual recorded ground motion intensities for each direction.

(3) Brace global buckling, brace local buckling, brace member fracturing and brace-to-gusset plate connection failures are the main failure modes observed in the damaged building.

Detail structural and non-structural damage reports after the Northridge earthquake for the building can be found in literatures [16, 18].

5. Evaluation Methods and Performance Objectives

5.1 Performance Objectives and Acceptance Criteria

The as-built building and the retrofitted buildings are evaluated in accordance with the ASCE 41-13 standard "Seismic Evaluation and Retrofit of Existing Buildings" [21]. In this case study, they are evaluated to achieve the Life Safety (LS) Structural Performance Level for the BSE-1E earthquake level as defined by ASCE 41-13 standard (i.e. limited performance objectives). Acceptance criteria for different structural components under different structural performance levels are also developed according to ASCE 41 standard. For simplicity, several seismic evaluation items such as foundations, diaphragms, deformation compatibility checks, detail connection design checks, retrofit design optimizations and the BSE-2E hazard level building responses are outside the scope of this study.

5.2 Target Spectrum and Ground Motions

The BSE-1E target spectrum for the case study building is generated using the online seismic design parameter calculation tool through USGS website. Seismic design coefficients considered are summarized in Table 1. Total 11 ground motion records are selected from the PEER Ground Motion Database (https://peer.berkeley.edu/peer-strong-ground-motion-databases) and scaled for NLRHA. Each ground motion is selected using the online ground motion database searching tool with predefined record acceptance criteria. Each record contains two horizontal components and one vertical component. Vertical components of ground motions are not used in this study. Ground motion records are scaled to match the BSE-1E target spectrum per ASCE 41-13. The scale factors of the ground motions are limited to be less than two. Each pair of ground motions is briefly summarized in Table 2. Scaled average spectral acceleration of selected ground motion records, BSE-1E target spectrum and the individual elastic spectrums are plotted in Fig. 5.

Table 1 – Seismic design parameters (ASCE 41-13)

Hazard Level	S_s	F_a	S_{XS}	S_1	F_v	S_{X1}	T_L	Site Class
BSE-1E	0.897	1.041	0.934	0.314	1.486	0.467	8	C

Table 2 – Selected ground motions for NLRHA (BSE-1E)

Ground Motion ID	Record Sequence Number	Station Name	Duration (sec.)	Δt (sec.)	Magnitude	Scale Factor
1E_01	5825	El Mayor-Cucapah EQ. - Cerro Prieto	100	0.005	7.2	0.75
1E_02	3748	Cape Mendocino EQ. - Ferndale FS	28.8	0.005	7.01	0.78
1E_03	185	Imperial Valley 1979 EQ. - Holtville	37.9	0.005	6.53	0.94
1E_04	179	Imperial Valley 1940 EQ. - Array #4	39.1	0.005	6.53	0.66
1E_05	6	Imperial Valley 1979 EQ. - Array #9	53.7	0.01	6.95	1.08
1E_06	6962	Darfield EQ. - ROLC	70.5	0.005	7.0	0.72
1E_07	1082	Northridge EQ. - Roscoe Blvd.	30.3	0.01	6.69	0.92
1E_08	802	Loma Prieta EQ. - Saratoga Aloha Ave.	40	0.005	6.93	0.82
1E_09	292	Irpinia EQ. - Sturno	39.3	0.0024	6.9	0.77
1E_10	266	Victoria EQ. - Chihuahua	27	0.01	6.33	1.63
1E_11	803	Loma Prieta EQ. - West Valley College	40	0.005	6.93	0.82

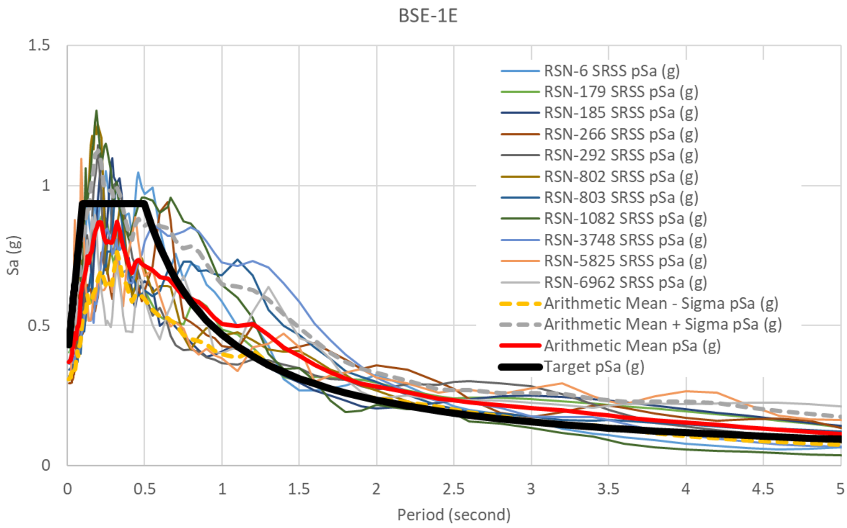


Fig. 5 – BSE-1E ground motion elastic spectrums and target spectrum (5% damping)

6. Analysis Results and Discussion

Several dynamic response quantities are examined and summarized in this section for the as-built building model and the retrofitted building models. Table 3 shows the three highest periods for each model. Note that in the SBS-02 (X-BRBF) scheme, the fundamental period increases due to less steel BRB core areas required to provide similar story shear capacity which is common in the design of BRBF because of the symmetric hysteretic behavior under tension and compression loads.

Table 3 – Modal analysis summary of three models

Model	IV (as-built)	SBS-01 (X-CBF)	SBS-02 (X-BRBF)
1st Mode	0.78 sec. (N-S)	0.73 sec. (N-S)	0.90 sec. (N-S)
2nd Mode	0.67 sec. (E-W)	0.64 sec. (E-W)	0.76 sec. (E-W)
3rd Mode	0.62 sec. (Torsional)	0.56 sec. (Torsional)	0.70 sec. (Torsional)

6.1 As-built Building

In addition to using the BSE-1E hazard level motions in NLRHA, the as-built building model (model IV) is analyzed with one near site ground motion that was recorded during the 1994 Northridge earthquake. All three components (two horizontal and one vertical) are input to the nonlinear model. Fig. 6 illustrates the location of the record station (RSN 1042) and the case study building site. The response spectrum of this record is plotted over the BSE-1E target spectrum in Fig. 7, and it shows that the record is approximately matching the 225-yr return period target spectrum.

Dynamic responses under this near site ground motion show that there are six HSS braces and one column in the building do not meet the acceptance criteria for LS performance level (see Fig. 8). It should be noted that most of the components do not meet the LS criteria are in the second story except one brace in the fourth story as shown in red in Fig. 8. This damage concentration is consistent with the actual observation after the Northridge earthquake for the case study building. At the end of the ground motion, some components even not satisfy the collapse prevention (CP) performance level acceptance criteria as shown in Fig. 9 in red (where usage ratio larger than one).

For the NLRHA results under BSE-1E earthquakes, the distribution of average component usage ratios (average over 11 nonlinear dynamic responses) is shown in Fig. 10. In general, the components that have average usage ratios larger than one (shown in red) are concentrated in the second story. Every HSS brace (total of 24 braces) and 3 steel columns in the second story, plus one HSS brace in the first story are required to retrofit to meet the LS performance goal under BSE-1E hazard level.

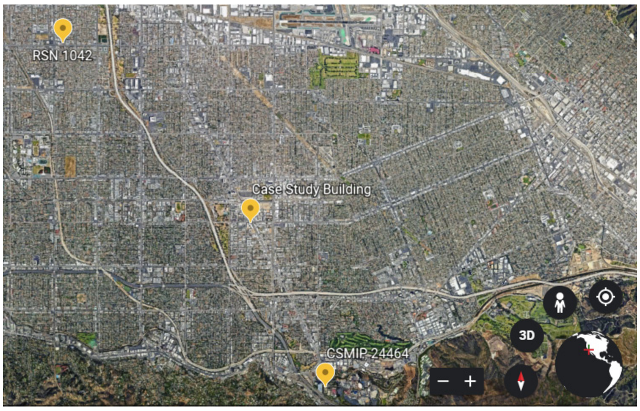


Fig. 6 – Location of record station (RSN 1042) and the case study building (Google Earth)

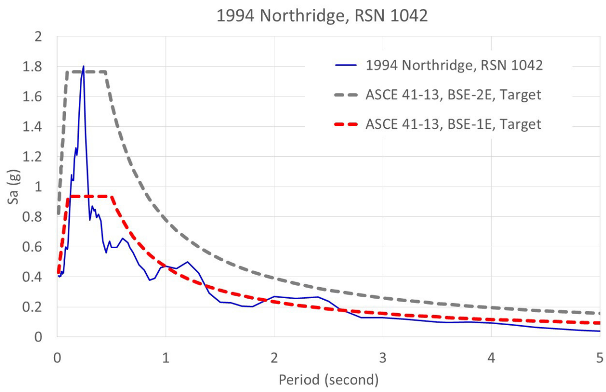


Fig. 7 – Response spectrum (RSN 1042) and ASCE 41-13 BSE-1E, BSE-2E target spectrums for the site

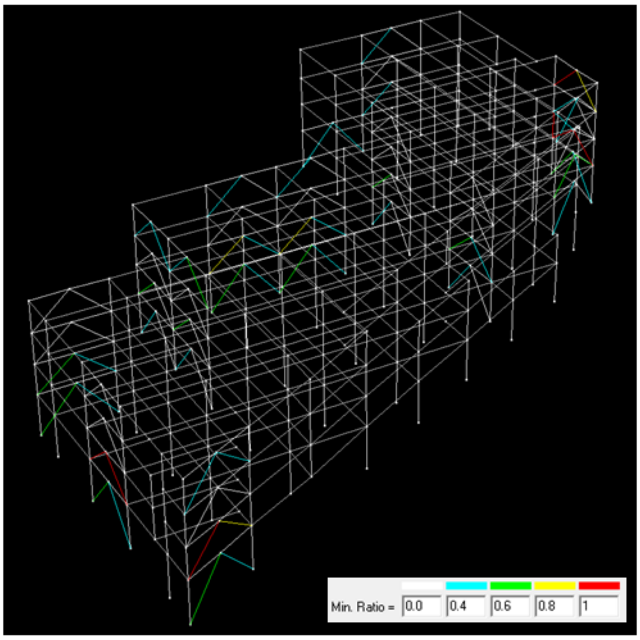


Fig. 8 – Distribution of LS performance level usage ratios under Northridge earthquake (RSN 1042)

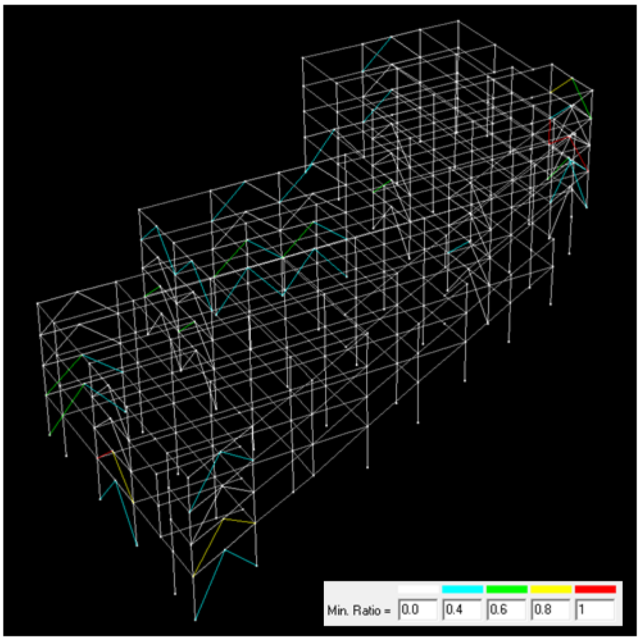


Fig. 9 – Distribution of CP performance level usage ratios under Northridge earthquake (RSN 1042)

6.2 Retrofit Scheme 1

This retrofit scheme simply uses the double story split-X brace configuration in one-half of the braced bay and the concept of elastic spine (or SBS) in the remaining half of the braced bay (model SBS-01, split-X CBF). Member sizes in the typical braced bay are shown in Fig. 11. Beam, column sizes and the HSS braces outside the elastic spine remain the same as the as-built building. Note that the width-to-thickness (b/t) ratios of the HSS braces are exceeding the AISC Seismic Provision [19] limit for the moderately ductile members. However, the main idea of this retrofit scheme is to show the advantage of using the SBS concept in existing braced frame building to avoid deformation concentration even if the braces outside the strongback spine are not meeting the b/t ratio requirement. Distribution of member usage ratios for LS performance level under BSE-1E earthquakes are shown in Fig. 12 and all members satisfy the LS acceptance criteria.

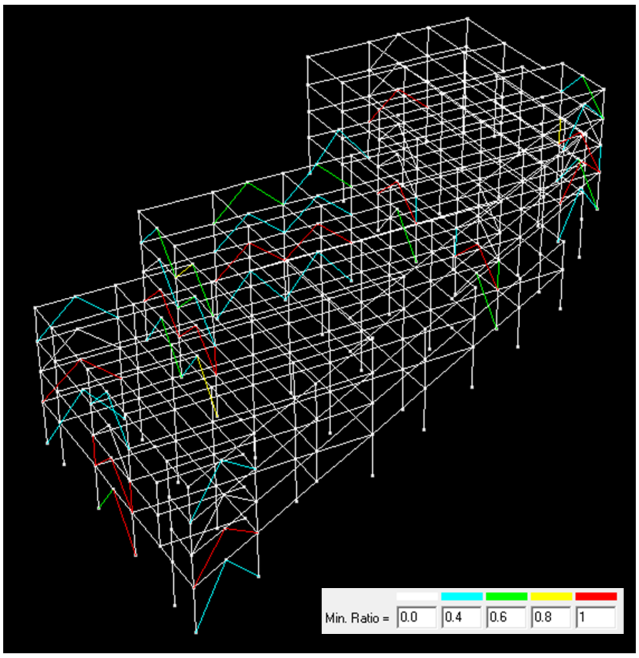


Fig. 10 – Distribution of LS performance level usage ratios under BSE-1E earthquakes (model IV)

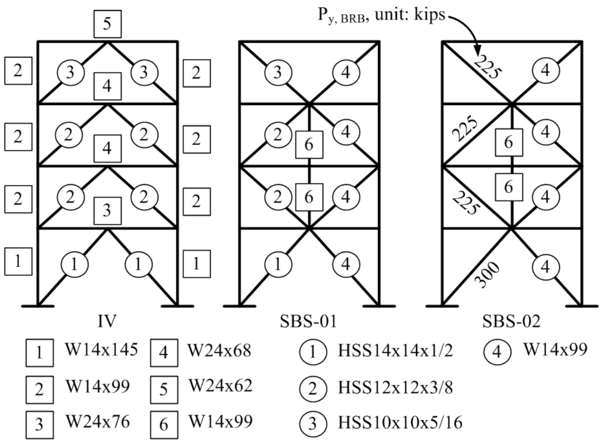


Fig. 11 – Typical braced bay member sizes for model IV (as-built), model SBS-01 (X-CBF) and model SBS-02 (X-BRBF)

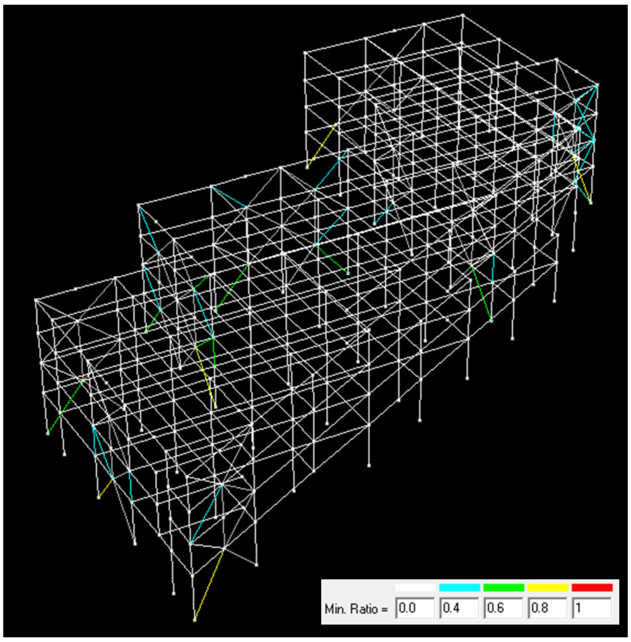


Fig. 12 – Distribution of LS performance level usage ratios under BSE-1E earthquakes (model SBS-01)

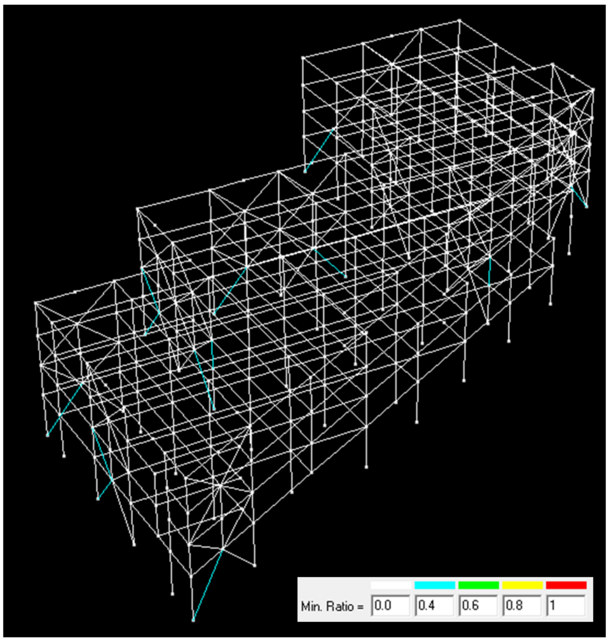


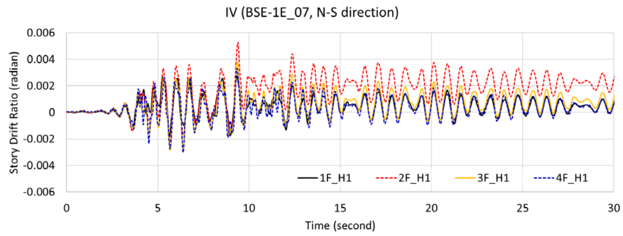
Fig. 13 – Distribution of LS performance level usage ratios under BSE-1E earthquakes (model SBS-02)

6.3 Retrofit Scheme 2

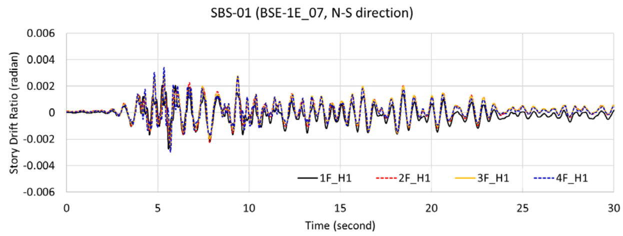
This scheme evolves from the previous retrofit idea by replacing the conventional buckling braces with BRBs but keep the elastic spine in the remaining half of the braced bay (model SBS-02, split-X BRBF). Member sizes in the typical braced bay are also shown in Fig. 11. Like the previous retrofit scheme, beam and column sizes remain the same as the as-built building. BRBs are considered as energy dissipation devices in the SBS system because of its symmetric hysteresis behavior under both tension and compression loads. Distribution of member usage ratios for LS performance level under BSE-1E earthquakes are shown in Fig. 13. Again, all members including BRBs satisfy the LS acceptance criteria.

6.4 Dynamic Responses under One BSE-1E Ground Motion

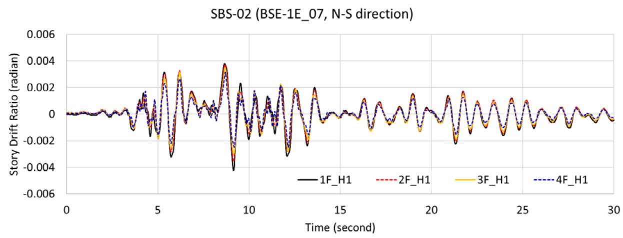
As shown in Fig. 14 and Fig. 15, both retrofit schemes effectively prevent the deformation concentration in certain floor level(s) and keep the story drift responses uniform over the height of the building. Only the story drift responses selected from one BSE-1E ground motion are shown herein. The as-built building clearly forms soft-second-story in both horizontal directions and also forms soft-ground-story in the East-West direction as shown in Fig. 14(a) and Fig. 15(a). Localized concentration of deformation is significantly reduced in model SBS-01 (see Figs. 14(b) and 15(b)), and it is further reduced in model SBS-02 (see Figs. 14(c) and 15(c)).



(a) Model IV

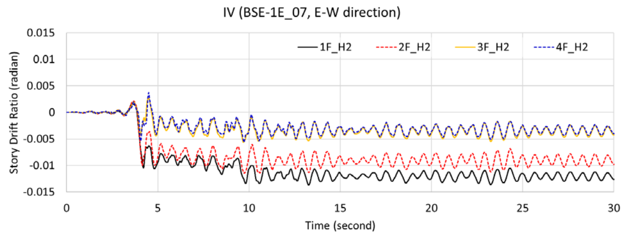


(b) Model SBS-01

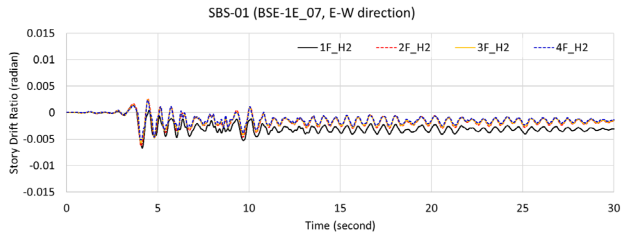


(c) Model SBS-02

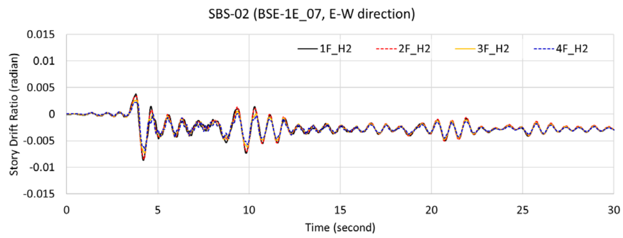
Fig. 14 – Story drift response histories under BSE-1 ground motion No. 7 for each model (N-S direction)



(a)Model IV



(b) Model SBS-01



(c) Model SBS-02

Fig. 15 – Story drift response histories under BSE-1 ground motion No. 7 for each model (E-W direction)

Most of W14 x 99 braces and tie-columns in the vertical spine remained elastic during the dynamic analyses, and all buckling braces or BRBs outside the spine are triggered to yield or buckle. All BRBs in SBS-02 model deformed into the nonlinear range and exhibited stable hysteresis loops with significant strain hardening observed. For the retrofitted building models, all residual story drift ratios are less than 0.3%, as shown in Fig. 15. Also, the as-built building residual story drift ratios are significantly larger than that in the retrofitted buildings.

6.5 Simple Cost Comparison

The steel weight of typical braced bay for two retrofitted schemes is estimated to examine the retrofit costs as a result of introducing the SBS vertical spine and BRBs into the braced frame systems. The fabrication costs per tonnage steel was assumed to be 3500 US$/ton, and the costs per BRB component for the mid-rise building are assumed to be 5000 US$/brace [14] including the miscellaneous connection costs. The equivalent connection tonnage is assumed as 15% of steel weight for typical steel members. Table 4 summarizes the weight and costs of each retrofit scheme. From the cost ratios shown in Table 4, the cost of using BRBs in the SBS is about 20% higher compare with that of using conventional buckling braces. It should be noted that the HSS brace b/t ratios in the SBS-01 model do not meet the AISC Seismic Provisions. In order to meet the b/t limit, heavier braces will be required, and this will further reduce the estimated cost difference between two retrofit schemes.

Table 4 – Estimated retrofit costs for two schemes

Retrofit Scheme	**HSS Brace Weight (ton)**	**Vertical Spine Weight (ton)**	**Number of BRBs**	**BRB Cost**	**Equivalent Connection Weight (ton)**	**Total Cost Without BRBs**	**Total Cost**	**Cost Ratio**
SBS-01	37	76.4	-	-	17	$ 456,400	$ 456,400	1.00
SBS-02	-	76.4	48	$ 240,000	11.5	$ 307,650	$ 547,650	1.20

7. Conclusions

The use of SBS concept in a braced frame building retrofit is proposed in this case study with the goal of preventing deformation concentration and satisfying the limited performance objectives. Seismic performance of the as-built building and its two retrofitted schemes are investigated for BSE-1 LS performance level. Based on the NLRHA results, the following conclusions can be drawn:

1. Both retrofit schemes using SBS concept can effectively prevent the soft-story mechanism in the building and all main structural members meet the acceptance criteria.
2. The simple design strategy for proportioning elastic vertical spine member sizes in SBS achieves the nearly uniform distribution of story drift over height.
3. Using BRBs in the SBS will further reduce the concentration of story drifts and will improve the deformation capacity of the entire system, however, larger residual deformations are observed.
4. The SBS-01 model results indicate that conventional braced frame structure can easily achieve uniform deformations over height by incorporating vertical elastic spines in the lateral force resisting system even if the buckling braces outside the spines do not meet the current code b/t limit.

8. Acknowledgements and Disclaimers

The author would like to thank to his doctoral study mentor, the late professor Stephen A. Mahin for initiating the idea of strongback system that makes this extended study possible. Any findings, opinions, conclusions or recommendations expressed in this paper are those of the author and do not necessarily reflect the views of Degenkolb Engineers.

9. References

[1] Khatib, I. F., Mahin, S. A., and Pister, K. S. (1988). “Seismic behavior of concentrically braced frames.” Rep. UCB/EERC-88/01, Earthquake Engineering Research Center, Univ. of California, Berkeley, CA.

[2] Rai, D. C., and Goel, S. C. (1997). “Seismic evaluation and upgrading of existing steel concentric braced structures.” Rep. UMCEE 97-03, Dept. of Civil and Environmental Engineering, Univ. of Michigan, Ann Arbor, MI.

[3] Tremblay, R. (2003). “Achieving a stable inelastic seismic response for multi-story concentrically braced steel frames.” AISC Eng. J., 40(2), 111–129.

[4] Hines, E. M., Appel, M. E., and Cheever, P. J. (2009). “Collapse performance of low-ductility chevron braced steel frames in moderate seismic regions.” AISC Eng. J., 46(3), 149–180.

[5] Lai, J.-W., Chen, C. H., and Mahin, S. A., (2010). “Experimental and analytical performance of concentrically braced steel frames.” Proc., Structures Congress, ASCE, Reston, VA, 2339–2350.

[6] Foutch, D. A., Goel, S. C., and Roeder, C. W. (1987). “Seismic testing of full-scale steel building—Part I.” J. Struct. Eng., 10.1061/(ASCE)0733 -9445(1987)113:11(2111), 2111–2129.

[7] Kiggins, S., and Uang, C.-M., (2006). “Reducing residual drift of buckling-restrained braced frames as a dual system.” Eng. Struct., 28(11), 1525–1532.

[8] Yang, C. S., Leon, R. T., and DesRoches, R., (2010). “Cyclic behavior of zipper-braced frames.” Earthquake Spectra, 26(2), 561–582.

[9] Clough, R. W., and Huckelbridge, A. A., (1977). “Preliminary experimental study of seismic uplift of a steel frame.” Rep. UCB/EERC-77/22, Earthquake Engineering Research Center, Univ. of California, Berkeley, CA.

[10] Kelly, J. M., and Tsztoo, D. F. (1977). “Earthquake simulation testing of a stepping frame with energy-absorbing devices.” Rep. UCB/EERC-77/17, Earthquake Engineering Research Center, Univ. of California, Berkeley, CA.

[11] Popov, E. P., Ricles, J. M., and Kasai, K., (1992). “Methodology for optimum EBF link design.” Proc., 10th World Conf. on Earthquake Engineering, CRC Press/Balkema, Rotterdam, Netherlands.

[12] Ghersi, A., Neri, F., and Rossi, P. P. (2000). “Seismic response of tied and trussed eccentrically braced frames.” Proc., Int. Workshop and Seminar on “behavior of steel structures in seismic areas”, CRC Press/Balkema, Rotterdam, Netherlands.

[13] Tremblay, R., and Merzouq, S., (2004). “Dual buckling restrained braced steel frames for enhanced seismic response.” Proc., Passive Control Symp., Tokyo Institute of Technology, Yokohama, Japan.

[14] Lai, J.W. and Mahin, S.A., (2014). “Strongback System: A Way to Reduce Damage Concentration in Steel-Braced Frames.” J. Struct. Eng., 141(9), 10.1061/(ASCE)ST.1943-541X.0001198.

[15] Simpson, B and Mahin, S.A., (2018). “Experimental and Numerical Investigation of Strongback Braced Frame System to Mitigate Weak Story Behavior.” J. Struct. Eng., 144(2), 10.1061/(ASCE) ST.1943-541X.0001960.

[16] Kelly, D., Bonneville, D. and Bartoletti, S., (2000). “1994 Northridge Earthquake: Damage to A Four-Story Steel Braces Frame Building and Its Subsequent Upgrade.” Proc., 12th World Conf. on Earthquake Engineering, New Zealand, paper No. 2589.

[17] ASCE. (2010). “Minimum design loads for buildings and other structures.” ASCE 7-10, Washington, DC.

[18] Bonneville, D., and Bartoletti, S., (1996). “Case Study 2.3: Concentric Braced Frame.” 1994 Northridge Earthquake Building Case Studies Project, Proposition 122: Product 3.2, SSC 94-06, Seismic Safety Commission State of California.

[19] American Institute of Steel Construction (AISC). (2010). “Seismic provisions for structural steel buildings.” AISC 341-10, Chicago.

[20] Computers and Structures, Inc. (2011). “PERFORM-3D Nonlinear Analysis and Performance Assessment for 3D Structures.” Version 5.0.0, Walnut Creek, CA.

[21] American Society of Civil Engineers, ASCE (2013). “Seismic Evaluation and Retrofit of Existing Buildings.” ASCE 41-13, Reston, VA.

