Experimental Investigation of a Combined System in Steel Braced Frames

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ABSTRACT

In general, the analysis and design of buildings must satisfy two criteria. First, under frequently occurring low to moderate earthquakes, the structure should have sufficient strength and stiffness to control deflection and to prevent any structural damage. Second, under rare, severe earthquakes, the structure must have sufficient ductility to prevent collapse. In this case, significant damage to the structure and nonstructural elements is acceptable. In this paper, the performance of an innovative, eccentric and knee bracing system called Eccentrically Knee Brace (EKB) is discussed and the behavior is investigated. A combination of eccentrically braced steel frames and knee braced steel frames has been assessed, and the concepts of the design of defined schemes are reviewed. As the structural fuse of the frame, the knee element will yield first during a moderate earthquake. In large earthquakes, both of them contribute in dissipating energy. Two half-scale EKBs were tested using the SAC loading protocol and an innovative loading protocol. The performance evaluation procedure includes laboratory and computer simulations. Cyclic loading tests were conducted to study the behavior of EKB.

Keywords:
Ductility; Eccentrically braced frames;
Knee braced frames;
Experimental study

1. Introduction

Concentrically braced frames (CBFs) are frequently used to provide lateral strength and stiffness to low- and mid-rise buildings to resist wind and earthquake forces, Figure (1b). Cyclic testing of CBFs shows that these braces buckle in compression and yield in tension. Plastic hinges occur after the brace has buckled (the buckle brace will straighten out, but will never fully return to the original position) and the stiffness and resistance of the frame decrease. Past earthquakes have demonstrated that this idealized behavior may not be realized if the braced frame and its connections are not properly designed. Eccentrically braced frames (EBFs) are one of the seismic load resisting systems in which the waste of energy (inelastic action) is performed through ductile links. In some of the common types of braced frames, one end of the link is attached to a column, and in these kinds of systems the correct behavior of this link is crucial to a safe performance of the EBF, Figure (1c). Therefore, the EBF design procedure prescribed in the 2010 AISC Seismic Provisions for Structural Steel Buildings [1] relies on an understanding of link behavior under severe cyclic loading. The current building code rules for EBFs in the AISC Seismic Provisions, including link design, link rotation limits, and link overstrength factors, were developed from rather extensive experimental studies conducted almost exclusively on wide-flange shapes of ASTM A36 or ASTM A992 steel.

However, structural steel shapes most commonly used in Iran today are produced according to the ST37 steel and Euro standard. The excellent seismic behaviour reported numerical and experimental studies on traditional eccentrically braced structures subjected to lateral static forces (e.g. see [2-6]). The remarkable dissipative capacity of EBFs, in particular, has been widely highlighted by researchers, albeit very often with reference to single-storey or multi-storey structures withstanding seismic equivalent static forces. Experimental investigation has gradually persuaded the scientific community of the structural effectiveness of EBFs, and hence, induced building codes to propose rather high values of the behaviour factor for the design of such structures. More recently, in contrast with this common belief, some papers have underlined that standard design procedures, may lead to concentration of large plastic deformations in links of single floors when applied to high-rise structures. In addition, the same studies have highlighted that the seismic response of eccentrically braced structures may be strongly dependent on the dynamic characteristics of the seismic input. An alternative system, which combines the advantages of the moment resisting frame, Figure (1a) and those of the concentric braced frames is the knee braced frames (KBFs), where one end of the brace is connected to a knee member (anchor) instead of the beam-column joint. In this system, the knee element acts as a "ductile fuse" to prevent collapse of the structure under extreme seismic excitations by dissipating energy through flexural and shear yielding, Figure (1d). A diagonal brace with at least one end connected to the knee element provides most of the elastic lateral stiffness [7].

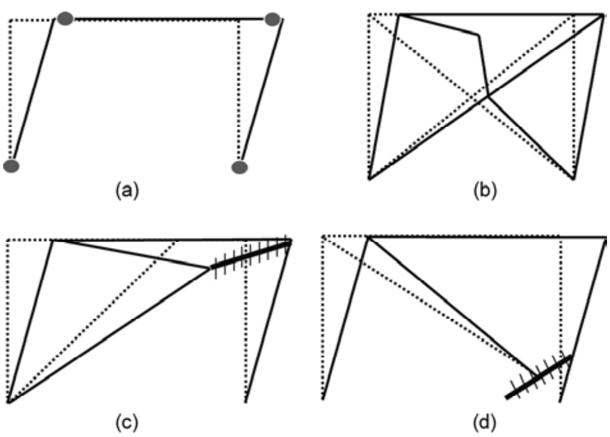


Figure 1. Description of energy dissipation mechanisms of framed structures subjected to lateral load: (a) steel moment resisting frame; (b) concentrically braced frame; (c) eccentrically braced frame; (d) knee braced frame.

As the nature and occurrence of earthquakes are random, it is necessary to consider different levels of earthquake intensity in designing earthquake resistant structures. To improve the seismic performance of the steel framed structures, further modification to enhance the structural performance is essential. In this paper, one of the most effective braced frame systems through which a high level of energy dissipation capacity may be attained is investigated. For this purpose, a modified structural form that adopts knee brace elements in the corner regions of the beams and columns, namely Eccentrically Knee Braced frame (EKB), as shown in Figure (2), is considered in this study. This study focused on the experimental evaluation of the seismic performance of EKB with an energy dissipation mechanism.

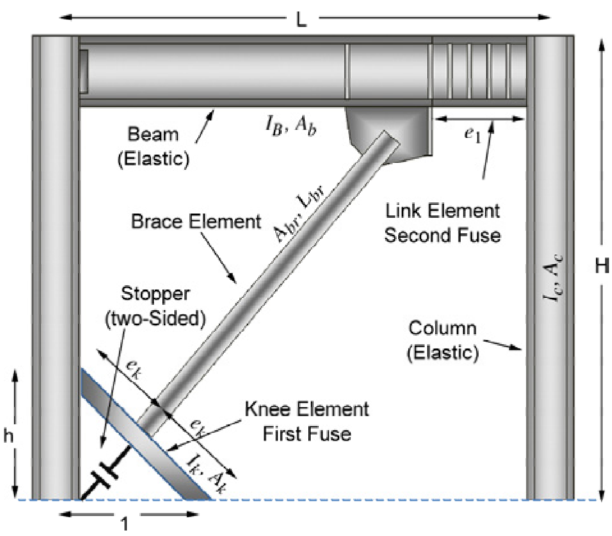


Figure 2. The proposed frame configuration.

2. Design Concept of Ductile EKB

The design of an EKB is based on a preselected yield mechanism that limits inelastic activities to ductile segments of the frame. For this structural system, seismic energy is dissipated by means of the yielding of the knee braces first (first fuse) and that of the link beam next (second fuse) with a selected yield mechanism. The knee and link beam acts as a fuse to prevent other elements in the frame being overstressed. In a SCBF, braces are designed and detailed as structural fuses. For an EKB, however, links and knee elements need to be properly designed and detailed to have adequate strength and ductility. The strength of the knee and link elements is selected to achieve the desired mechanism. The other members (columns, beam outside the link and brace) in the frame are designed to remain elastic under the largest forces generated by fully yielded and strain-hardened plastic hinges, except at the column bases where plastic hinges are required to complete the mechanism. For this purpose, column bases are designed to resist horizontal and vertical movements, but with a rotation (columns were hinged at their base). Consider an EKB configuration, as shown in Figure (2), where the knee with length e_k and link with length e_l deforms inelastically and resists the applied base shear, V_b, while the framing outside the link is designed to remain elastic. Figure (2) illustrates the basic type of an EKB in which at least on end of bracing member connects to a beam a short distance from a beam-column connection. This short section of grider between the brace end and column is known as the link beam. In FEMA 356 a wide range of structural performance requirements could be desired by individual building owners. Table C1-3 FEMA356, relates structural performance levels to the limiting damage states for common vertical elements of the lateral-force-resisting systems. For this configuration, the knee element designed for the 1% drift (a performance level between immediate occupancy and life safety) and link element designed for the 2% drift (collapse prevention performance) [8].

Prior to giving equations that are specific to EKB, a brief review of the kinematics of a common EKB configuration is provided as shown in Figure (3). In this configuration, the base shear capacity of the frame can be written in terms of the plastic link shear strength, V_p, as:

$$V_b = V_p \frac{L}{H} \tag{1}$$

where L is the frame width and H is the frame height. Before yielding of link, the drift angle of the frame, θ, can then be written in terms of the knee rotation angle, γ_k, as:

$$\theta = \gamma_k \frac{a \times b}{(a+b)h\cos\theta} \tag{2}$$

In the second step, the drift angle of the frame, θ, can then be written in terms of the link rotation angle, γ_l, as:

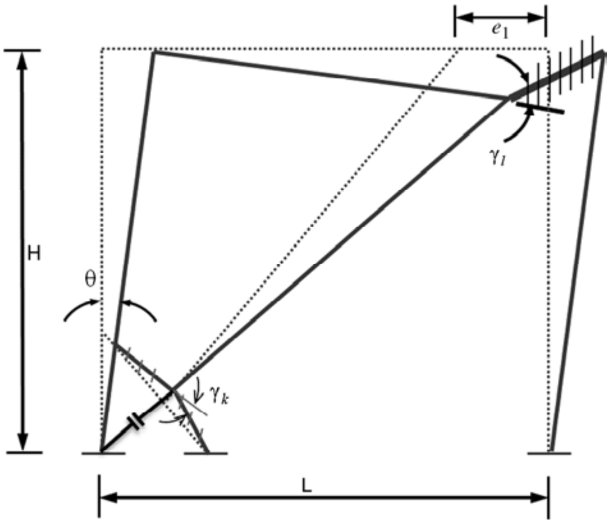


Figure 3. Typical deformed EKB frame.

$$\theta = \gamma_l \frac{e}{L} \tag{3}$$

where the deformations of the framing outside the link have been neglected, which is reasonable considering that the link may be subjected to large inelastic deformations while the surrounding framing remains essentially elastic. AISC 341-10 [1] defines three modes of response for links in EBFs using a normalized link length, ρ, which is defined in Eqs. (4) to (6): (1) for ρ less than 1.6, the response is governed by shear yielding (shear links), (2) for ρ greater than 2.6, the response is governed by flexural yielding (flexural links), and (3) for ρ between 1.6 and 2.6, the response is a function of the coexisting shearing force and moment (intermediate links).

$$\rho = \frac{e}{M_p / V_p} \tag{4}$$

$$V_p = 0.6F_y(d - 2t_f)t_w \tag{5}$$

$$M_p = F_y Z \tag{6}$$

In these equations, V_p is the plastic shear strength, M_p is the plastic moment strength, F_y is the yield stress for steel, e is the link length, d is the link depth, t_w is the web thickness, t_f is the flange thickness, and Z is plastic modulus of the cross-section. The design procedure is based on AISC Seismic Provisions (2010). ST37 steel is used for the beams, brace, knee and the columns. Column and brace sections are chosen from IPB series and from rectangular hollow sections with equal depth and width values, respectively. Beam and knee element are chosen from IPE series. The experimental studies have shown that the shear yielding of a link is the most ductile yielding mode. In order to ensure that the shear yielding dominates the inelastic behaviour, the link length should satisfy the condition of ρ less than 2, if the strain hardening is not taken into consideration. However, based on the experimental evidence of shear links, the ultimate moment value and shear force reaches nearly $1.2\,M_p$ and $1.5\,V_p$, respectively. Forcing the yielding to occur in, and to be confined to, ductile knee elements (first) and ductile link elements and is the primary goal of EKB design. Capacity-based design is utilized to realize this objective. In the capacity-based design, while the knee and link are sized for the load combinations specified by the code, the members outside the knee and link (columns and brace) should be designed to resist the forces generated by the fully yielded and strain hardened knee and link. The required shear strength of the knee element and the link element V_u, shall not exceed the designed shear strength of the link:

$$\phi V_n \geq V_u \quad , \quad \phi = 0.9 \tag{7}$$

where, V_u = Nominal shear strength of the knee or link, equal to the lesser of V_p or $2M_p/e$. The V_u in knee simultaneously with 1% drift and V_u in link simultaneously with 2% drift. The required strength of the diagonal braces of an EKB can be taken as the forces generated by the following values of the link shear:

$$Link\ shear = 1.25 R_y \times V_n \tag{8}$$

where R_y = Ratio of the expected yield strength to the minimum specified yield strength and 1.25 is the factor considering the strain hardening. The design procedure of columns is quite different. Columns of EKBs with only a few stories should have the axial design strength to sustain the $1.25 \times R_y$ times the nominal shear strength of all the links above the level of the column under consideration. This is based on the premise that all the links simultaneously reach their maximum strength. The elements should satisfy the code-based compactness conditions, too.

For I-shaped links, the AISC Seismic Provisions (AISC, 2002) limit the inelastic link rotation for shear links and flexural links to 0.08 rads and 0.02 rads, respectively, with linear interpolation based on normalized link length to be used for intermediate links. Web buckling of links in EBFs cause rapid strength and stiffness degradation, and this significantly impedes the energy dissipation capabilities of the system [1]. Web stiffeners can be used to delay web buckling beyond a certain rotation level. Kasai and Popov [4] derived the stiffener spacing formula for WF links that appears in the Provisions. Fully restrained boundary conditions for the web sides adjacent to the flanges were used in the Kasai and Popov derivation because of the presence of flange sections on both sides of the web and the high moment gradient. According to concept design of EBF, for design of knee element, it was assumed that shorter part of knee element would experiment inelastic deformations. The knee element was designed according to AISC Seismic provisions, similar with EBF systems.

3. Experimental Program

The laboratory simulations may be necessary in cases where the behavior of a structural system or its components is inadequately understood and therefore cannot be modeled confidently for computer simulation. By integrating the laboratory specimen behavior and computer models, the complete system performance can be simulated. The results of the performance evaluation are often presented in terms of the deformation and strength capacity of the structure. The proof-of-concept EKB was designed to be as large as possible considering the constraints of the available equipment in the Structural Engineering Research Center Laboratory at the International Institute of Earthquake Engineering and Seismology (IIEES). Quasi-static cyclic loading was chosen and the maximum force output of the actuator available (980 kN) was divided by 1.5 to account for the possibility of obtaining material with a higher-than-specified yield stress as well as strain hardening, making the design base shear 653 kN. The general test setup is shown in Figure (4). The overall test specimen dimensions were set to a height of 1600 mm and width, L, of 2250 mm. As shown, a hydraulic actuator applied horizontal

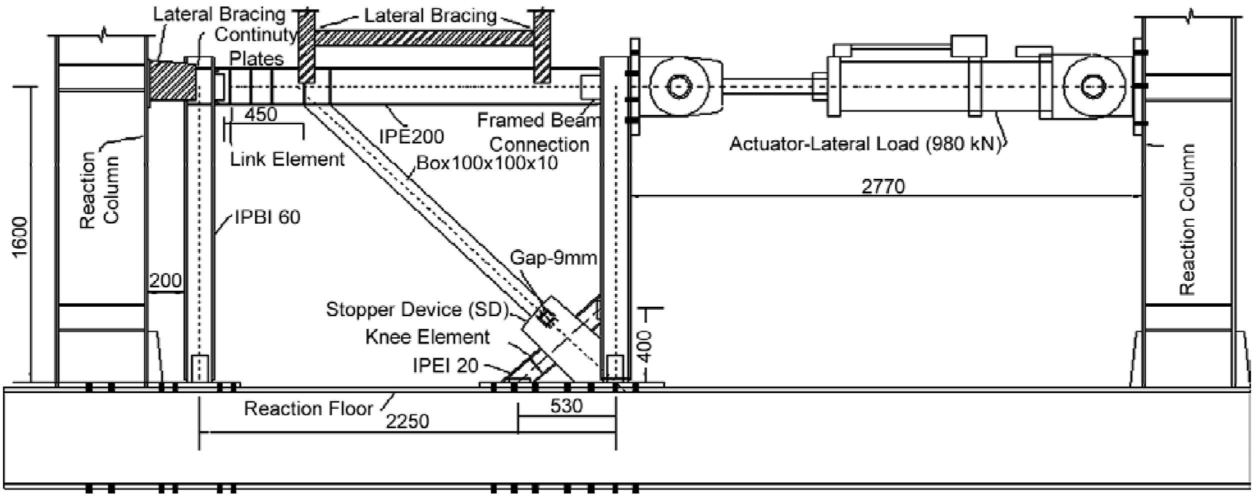


Figure 4. Proof-of-concept test setup with dimensions (elevation).

force to top of right column (a small variation in the displacement to each column is expected due to the axial deformation of the beam). The frame was hinged at the base of each column that were fastened to a foundation beam that attached to the strong floor of the laboratory and also to the reaction frame where the actuator was mounted. For safety and stability of the frame, the setup was globally braced for out-of-plane stability at three points on the beam by using the towers available in the laboratory, as shown in Figure (4); two lateral bracings were provided to the link element and another one was provided to the mid-point of the beam. The EKB system has a fundamental part called "Stopper Device" (SD), which can create two separate levels of energy dissipation by restricting displacement of knee element. The gap between knee element and SD is limited to 0.9 cm. This limitation has coincided with 1% drift. The configuration of SD is shown in Figure (5). Besides, Figure (6) shows a photograph of the EKB braced subassembly.

The knee and link element must be designed to satisfy several conditions simultaneously. It must have the desired shear strength, flexural strength, and link length, while meeting the limits for flange compactness (b_f / t_f) and web compactness (d / t_w) for I sections from the Provisions. Additionally, a frame drift-to-link rotation ratio must be selected so that adequate energy dissipation can be achieved prior to the link rotation reaching 0.08 rad. In this proof-of-concept test, it was decided to test a shear link, as these were deemed more likely to be used in practical applications (partly due to their larger rotation limits). Finally, the beam outside the link must be able to resist large axial forces and moments

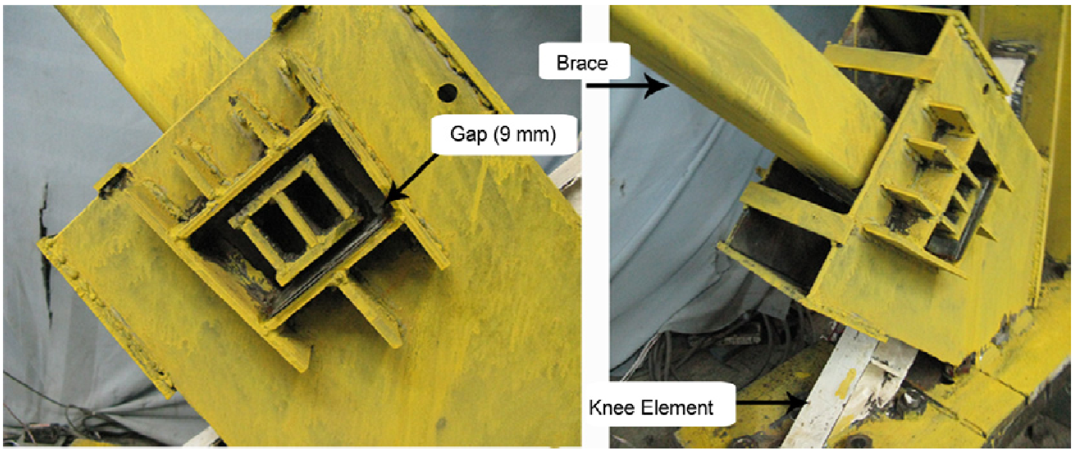


Figure 5. Detail of the SD (elevation and 3D).

Figure 6. Photograph of EKB frame subassembly.

acting simultaneously. All welding was performed according to the shielded metal arc welding (SMAW) process by using E7018 electrodes for link connection and E6013 electrodes for other connections. Welding and weld-access hole details for the link-to-column connection followed the recommendations of SMF beam-to-column connections outlined in AISC 341-10 [1]. More specifically, the link cross-section for the experimental test specimen was sized as follows:

- To ensure that the actuator would have the capacity to push the specimen well into the strain hardening range, and to account for material yield stresses that may be larger-than specified, the maximum applied force, V was set to 653 kN.
- From a nonlinear analysis, the required knee shear force, V_N, was found to be 71 kN for the 1% drift, h of 1600 mm, $L = 2250$ mm. Moreover, the required link shear force, V_L, for the 2% drift was found to be 170 kN.
- The required shear area, $A_s = (d - t_f)t_w$, was found to be 470 mm^2 from Eq. (2) and (3), rearranged as:

$$(d - 2t_f)t_w = \frac{V_p}{0.6\phi_v F_y} \tag{9}$$

Note that a resistance factor, ϕ_v, of 0.9 was considered, the knee and link plastic shear force V_p was taken as V_L and V_N from a nonlinear analysis, and the yield stress, F_y, was assumed to be 235 MPa.

- Next, the minimum knee and link length was determined to achieve a link rotation, 0.08 rad at a minimum drift. The minimum knee and link length was determined to be 656 mm and 450 mm, respectively.
- Next, the maximum shear link length, e*, was determined from Eq. (4). Here, ρ was taken as 1.6 and the plastic moment was multiplied by a resistance factor, ϕ_b, of 0.9. Additionally, the plastic shear was multiplied by the R_y for ST37 steel by 1.2, which is the ratio of expected to specified yield stress (based on Iranian code of practice for design of steel structures, Subject 10). The resulting conservative equation for the maximum shear link length was:

$$e^* = 1.2\frac{M_p}{V_p} \tag{10}$$

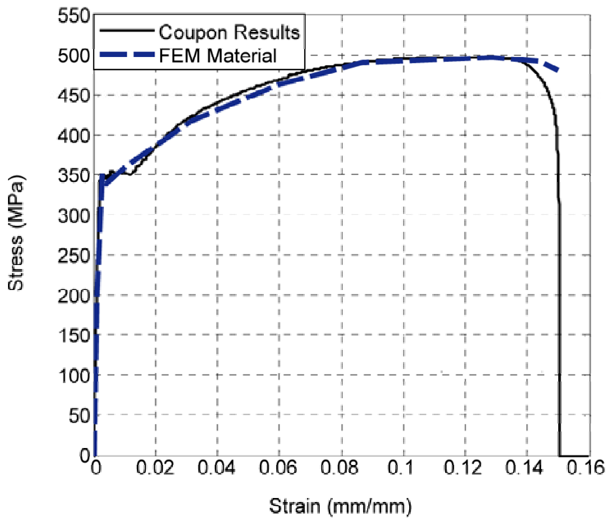
Assuming a yield stress of 235 MPa, for the minimum required shear area, and the limits of the Provisions for flange compactness, given by:

$$\frac{b}{t_f} \le 0.3\sqrt{\frac{E}{F_y}} \tag{11}$$

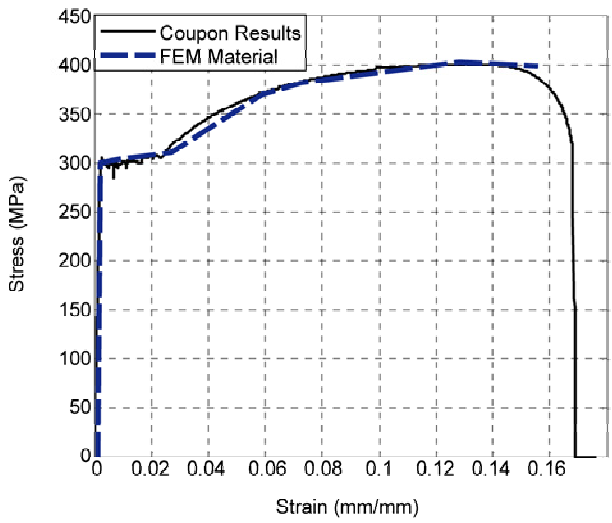
The following link cross-section dimensions and length were chosen: d = 200 mm, b = 100 mm, t_f = 8.5 mm, t_w = 5.6 mm (IPE200), and e_l = 450 mm.

Knee cross-section dimensions and length were chosen as well: $d = 120$mm, $b = 64$ mm, $t_f = 6.3$mm, $t_w = 4.4$ mm (IPE120), and $h = 400$mm and $l = 530$mm. Using these values, the plastic shear force and plastic moment were 158 kN, 52 kN.m and 74 kN, 14.3 kN.m for the link and the knee elements, respectively. Braces were HSS 100x100x10 and columns were IPB160 and the link-to-column designed to be moment resisting. The link and knee stiffener spacing and stiffener sizes were designed using the recommended of the Provisions. A stiffener spacing of 112 mm for the link and the knee was calculated, a minimum stiffener thickness of 10 mm and minimum stiffener width of 47 mm for link and 30 mm for knee were selected. Assuming ST37 steel with a yield stress of 235 MPa is used for the stiffeners, a 5 mm fillet weld on both sides of each stiffener and all-around the link-to-stiffener interface, was designed to resist the full yield strength of the stiffeners. Coupons for tension testing conforming to ASTM standards [9] were fabricated from web plate materials of link and knee element. Because the beam and column were both expected to remain linear elastic, coupon tests were conducted only for the link and knee material. Mean coupon test results are shown in Figure (7). Note that the yield stress for the web material of link, 345 MPa ($R_y = 1.46$) and for the web material of knee, 300 MPa are considerably higher than the 235 MPa specified ($R_y = 1.27$). Using the results of the coupon tests, the link plastic shear, V_p, and plastic moment, M_p, were determined to be 232 kN, and 76.2 kN.m for link, and 95 kN, and 18.2 kN.m for knee, respectively.

In the research reported herein, two proof-of-concept EKBs were tested to determine their cyclic behavior. First, the quasi-static loading protocol used here was developed based on the guidelines presented in SAC [10]. Figure (8) shows the loading history for proof-of-concept test. Verification of the yield force was carried out by checking the values of the principal strains from the rosettes on the web of the link and knee elements. Beyond yield, the subsequent cycles were applied in displacement control using the horizontal displacement recorded at the link beam level. The cyclic tests continued until a fracture was observed or the strength or stiffness of test specimens reduced significantly. Acceptance criteria for links are based on inelastic link rotation as defined in AISC (2010). Displacement of the actuator was used as the control parameter during testing. Table (1) lists the key characteristics of the test specimens. The test specimens were only different in loading protocol.



(a) Web Material of Link Element



(b) Web Material of Knee Element

Figure 7. Stress-strain curves.

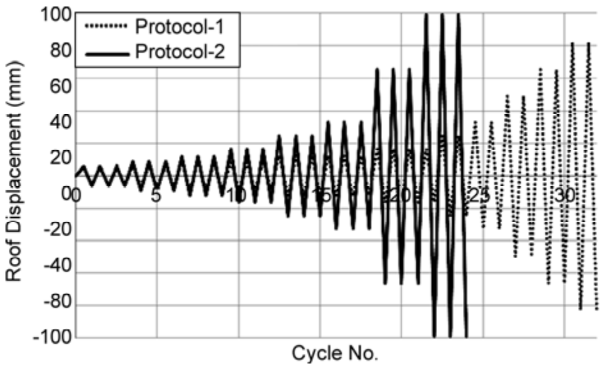


Figure 8. Loading history for proof-of-concept test.

Table 1. Summary of EKB specimens.

Specimen No	Link Section	Knee Section	Brace Section	Loading Protocols
SP-1	IPE200	IPE120	Box100x100x10	Protocol-1
SP-2	IPE200	IPE120	Box100x100x10	Protocol-2

4. Experimental Results

Expanded descriptions of specimens' hysteresis behavior and failure modes are presented here. The initial stiffness of the specimen was determined to be 32 kN/mm from the elastic cycles. After yielding knee element, frame stiffness is only provided by the link element. The yield drift was identified as 0.28% and corresponded to a base shear of 220 kN, while the maximum base shear and drift reached were 600 kN and 4%, respectively, this drift is an inappropriate seismic characteristic.

For both specimen, at the lateral force of 65 kN imposed to the top of the specimen, the principal strains on the web of the knee element, exceeded from steel yield level. However, in spite of this local yielding, overall behavior of test frames remained linear. The experimentally hysteretic response is shown in Figures (9) and (10). The applied load on the beam, (base shear), is plotted against the imposed displacement at the center of the beam. As shown in these figures, the great hysteretic performance of EKB at two levels can be noticed.

The link and knee deformed at the peak of the positive excursion of Cycle 19 for SP-2 is shown in Figures (11) and (12). No buckling of the webs or flanges was observed, indicating that

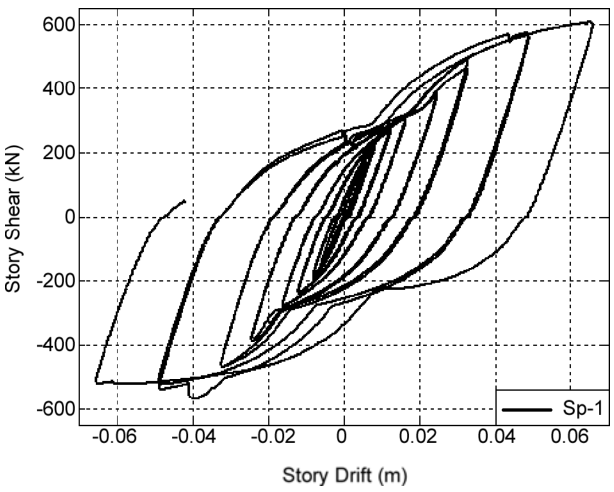


Figure 9. Hysteretic response of specimen SP-1.

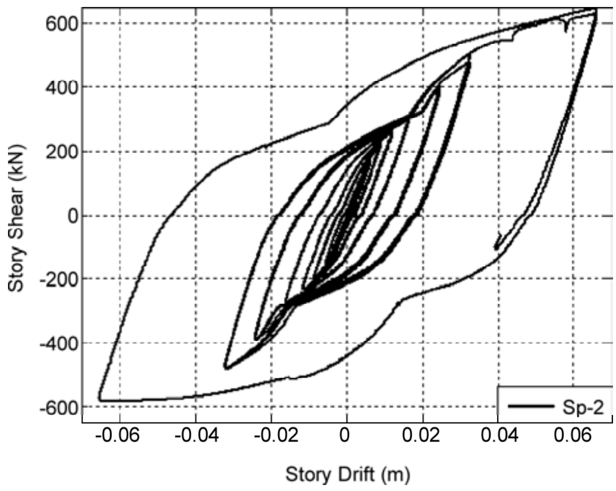


Figure 10. Hysteretic response of specimen SP-2.

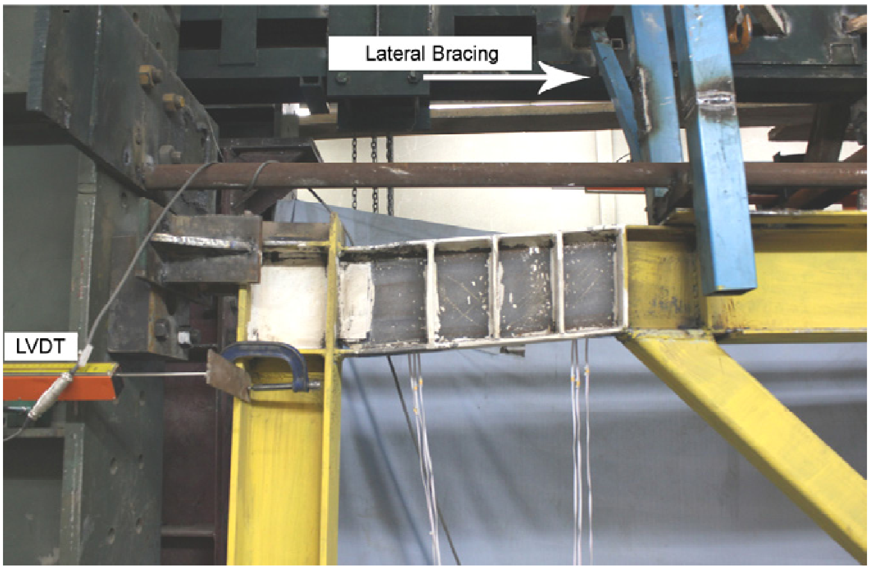


Figure 11. Deformed link during Cycle 19 for SP-2.

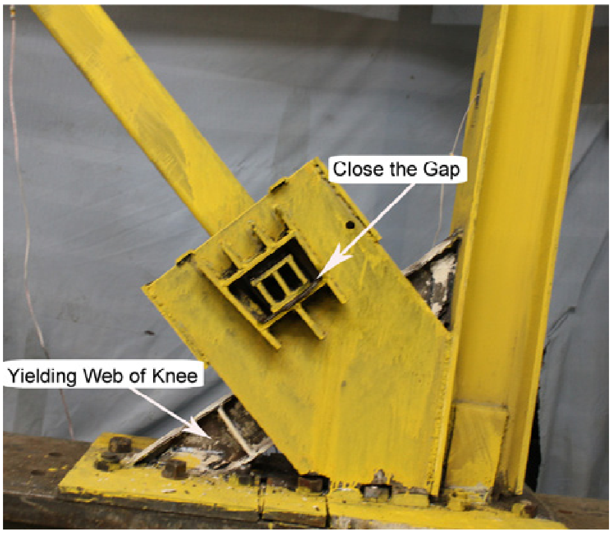


Figure 12. Deformed knee during Cycle 19 for SP-2.

the compactness ratios and stiffener spacing used for this proof-of-concept link (which met the AISC prescribed limits) were adequate to prevent local buckling prior to achieving large rotations. In addition, there was no evidence of lateral torsional buckling, and the out-of-plane moments for the beam outside-the-link.

At the lateral displacement of 5 mm, knee element experienced plastic deformations. The failure mode of the link and the knee was the yielding of the web. Besides, the failure mode of the brace to link connection was at the peak of the negative excursion of Cycle 19 for SP-2 as shown in Figure (13). The factors that most likely have contributed to this are the large plastic strain demands at that location, the high degree of constraint due to the presence of the stiffeners, and the welds used for the link-to-brace connection, as well as the heat-affected-zone (HAZ) brittleness near the connection

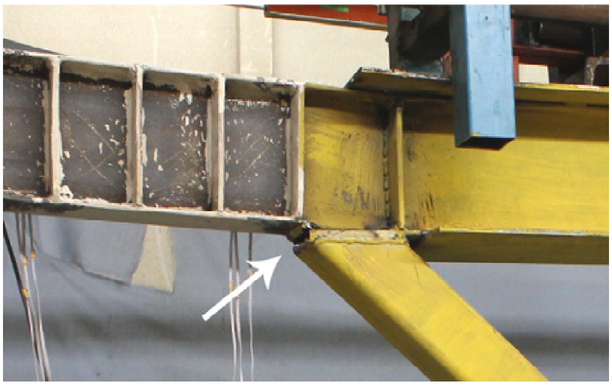


Figure 13. Brace to link connection fracture during Cycle 19.

weld. There was no evidence of crack initiation in the full penetration groove weld used to assemble the flanges of the link and knee. In Figure (14), a comparison in terms of cumulative dissipated energy of all specimens is provided. Cumulative dissipated energy and dissipated energy in each cycle is one of the most important characteristics affecting the seismic performance of the EKB system. In the specimens, with increasing drift, cumulative dissipated energy of the specimens increased. It can be found from Figures (9) and (10) that both SP1 and SP2 exhibited significant deformation capabilities.

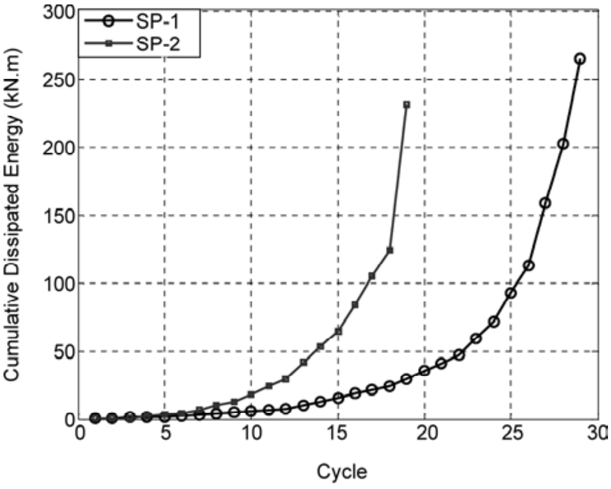


Figure 14. Cumulative dissipated energy of the specimens.

However, the ductility of the SP1 was less than that of the SP1. This characteristic justified the importance of loading protocol to the structural performance of steel EKB frames. Both of the hysteretic loops are unpinched and no significant deterioration in stiffness or strength is observed. Due to limitations in testing set-up and experimental equipment ductility was calculated at almost 4% in most tests. Therefore, ductility was considered only for this range. The yield (Δ_y) and maximum (Δ_u) displacements for SP1 were found to be 11 mm and 64 mm, respectively. This corresponds to a ductility factor (Δ_u/Δ_y) more than 5.8. Moreover, Δ_y and Δ_u displacements for SP2 were found to be 8 mm and 64 mm, respectively. This corresponds to a ductility factor (Δ_u/Δ_y) more than 8. Out-of-plane movements at the knee-brace connection, implying LTB (Lateral Torsional Buckling) of knee element, were reported very small. Therefore, no lateral bracing seems to be essentially required for knee-brace connection. Figure (15) shows the flaked whitewash on the knee

Figure 15. Yielding of the knee at the end of test.

of SP2, at the final cycle of lateral loading corresponding to top lateral force of 600 kN.

5. Conclusions

In this paper, a novel braced frame that is a combination of eccentrically braced steel frames and knee braced steel frames has been developed, tested, and modeled analytically. There are two objectives of the study reported herein. The first objective is to develop and implement the EKB for two seismic performances. The knee and link beam acts as a fuse to prevent other elements in the frame being overstressed. The second objective is to demonstrate the EKB performance using experimental and analytical simulation. To accomplish these objectives, two quasi-static tests were conducted and presented to study the force-deformation hysteresis behavior of an EKB braced subassembly. The results of the quasi-static tests were used to calibrate an analytical brace element capable of simulating general hysteresis behavior of steel braces. The results of the analytical simulation and experimental simulation tests demonstrate that the system has many redundancies, and the intended force redistribution in the EKB system can occur under two level design earthquake loading. A proof-of-concept experiment showed that it can achieve and exceed the maximum drift for the frame specified in the ASCE. Design equations and requirements have been proposed and, in a preliminary sense, verified by the successful testing of the proof-of-concept specimen. It was found that, at least for this specimen, its shear-moment interaction can be neglected, as both the plastic shear and plastic moment capacity of the cross-section, calculated using actual material properties, were exceeded due to strain hardening. Furthermore, the maximum link shear exceeded the plastic shear strength calculated using the ultimate stress of the web material, indicating that some shear was likely to be carried by the flange as well. Finally, a finite element model of the proof-of-concept link was developed using shell elements, and it showed reasonable agreement in terms of deformations and hysteretic behavior.

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