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Eccentrically Knee Bracing: Improvement in Seismic Design and Behavior of Steel Frames

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ABSTRACT

The use of passive control systems is widely considered as a reliable approach for controlling earthquake vibrations in steel structures. First, under frequently occurring low to moderate earthquakes, the structure should have sufficient strength and stiffness to control deflection and prevent any structural damage. Second, under rare and severe earthquakes, the structure must have sufficient ductility to prevent collapse. For this case, significant damage of the structure and non-structural elements is acceptable. In this paper, the performance and the lateral stiffness of a new eccentric and knee bracing system named Eccentrically Knee Brace (EKB) is investigated. The stiffness of eccentrically knee braced frames (EKBs) is difficult to calculate by hand because they are indeterminate and have significant shear, flexural and axial deformations in different members. EKB stiffness is important for design, because it is used to compute story drifts and the knee and link rotations, which have prescribed limits. This note presents an equation for the stiffness of an EKB story in terms of the design story shear, frame geometry, and beam depth. The equation is independent of specific member sizes, making it useful for determining appropriate geometry in design. The equation is developed numerically and verified with experimental data from code compliant. One application of the equations is the estimate of the beam depth required to ensure a link or knee will satisfy inelastic rotation limits.

Keywords:
Seismic design;
Stiffness; Steel frames;
Two levels of performance

1. Introduction

For some decades now eccentrically braced frames (EBFs) have been indicated as the distinctive elements of a structural typology suitable for satisfying the different design objectives of modern performance-based seismic engineering in medium or high-rise steel buildings [1]. They have often been proposed as a cheaper and more valid alternative to the most common moment resisting frames (MRFs) and concentrically braced frames (CBFs), as they incorporate the good qualities of the above-mentioned structures. Indeed, owing to the presence of bracings and links, EBFs are expected to incorporate characteristics of both high lateral stiffness and high energy dissipation capacity [2-6]. 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. An alternative system that combines the advantages of the moment resisting frame 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 [7]. A diagonal brace with at least one end connected to the knee element provides most of the elastic lateral stiffness. As the nature and occurrence of earthquakes are random, it is necessary to consider different levels of earthquake intensity in designing earthquake resistant structures [8]. 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), and this paper describes the seismic behavior of EKB, as shown in Figure (1), is considered in this study. The design of an EKB is based on creating a frame that will remain essentially elastic outside a well-defined link and knee. During moderate lateral loads, the knee element deform inelastically and during extreme loading it is anticipated that the link and the knee will deform inelastically with significant ductility and energy dissipation. As the nature and occurrence of earthquakes are random, it is necessary to consider different levels of earthquake intensity in the design of earthquake resistance structures. To improve the seismic performance of the steel framed structures, further modification to enhance the structural performance is essential [9]. 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 first yielding of the knee braces (first fuse) and second yielding of the link beam (second fuse) with a selected yield mechanism. The knee and link beam acts as a fuse to prevent other element in the frame being overstressed. 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, columns base are designed to resist horizontal and vertical movement, but with rotation (columns were hinged at their base). Where the knee with length ek and link with length el deforms inelastically and resists the applied base shear, Vb, while the framing outside the link is designed to remain elastic. 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 lateral-force-resisting systems. For this configuration, the knee element designed for 1% drift (a performance level between immediate occupancy and life safety) and link element designed for 2% drift (collapse prevention performance) [10]. In seismic design of common structural steel frames for resisting lateral forces, either moment-resisting frames or diagonal bracing is usually employed. The current trend on the one

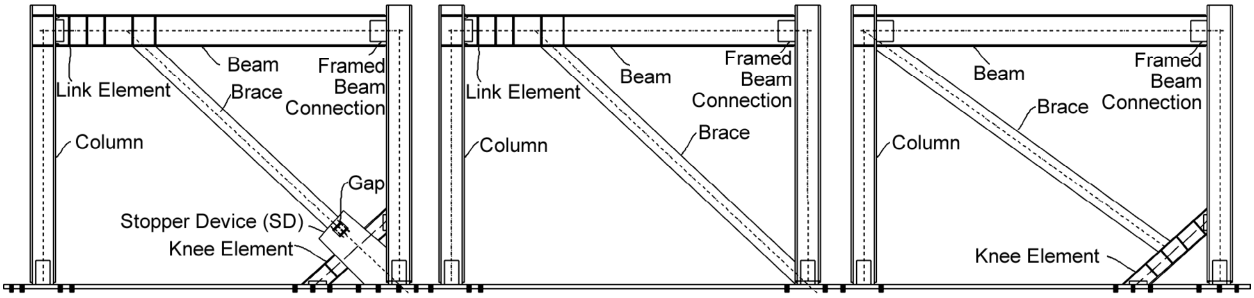

Figure 1. Braced frame geometry: (a) The geometry of the proposed system, eccentrically knee braced frame; (b) eccentrically braced frame; (c) knee braced frame.

hand of limiting the displacement of storey of a building for a small or moderate earthquake ground motion, and on the other hand of providing large frame ductility for a major shake, raises some objections to either scheme. The storey drift of a MRF depends primarily upon rotation of the beams causing rotation of the joints, flexure of the columns, and shear deformation of the panel zones [11]. In a correctly designed frame the deformation of a panel zone is kept small. If necessary, doubler plates are provided to minimize this effect [1]. The journey to the objective of the research in this research started from the authors' inquiry about an efficient way to reduce the response of structures subjected to lateral loading in two levels. For this structural system, seismic energy is dissipated by means of the yielding of the knee element first (first fuse) as well as that of the link beam next (second fuse) with a selected yield mechanism. The knee and the link beam act as fuse to prevent other frame elements from being overstressed. The current study is intended to modify the conventional EBF system by making use of a knee element connecting the end-points of the diagonal brace [9]. In the eccentrically knee braced frames (EKBs), braces are offset from beam-column connections or each other by a distance e and a knee element connecting to the end-points of the diagonal brace as shown in Figure (1a). The EKB system has a fundamental part, named "Stopper Device" (SD), which can create two separate level of energy dissipation by restricting displacement of knee element. The gap between knee element and SD is limited. This limitation has coincided with 1% drift. The geometry of EKBs makes them more complicated to analyze by hand (or spreadsheet) than eccentrically knee braced frame (Figure (1b)) or knee braced frame (Figure (1c)).

While all three frames of Figure (1) are indeterminate, and for them, stiffness cannot be estimated easy because the beam or knee elements experiences significant flexural and shear deformations. EKB pushover analyses suggest that there may be a way to estimate the frame stiffness of an EKB frame, based on the frame strength, geometry and independent of specific member sizes. Figure (2) shows data from pushover analyses of three EKBs with equal heights. All models are similar 1-story,

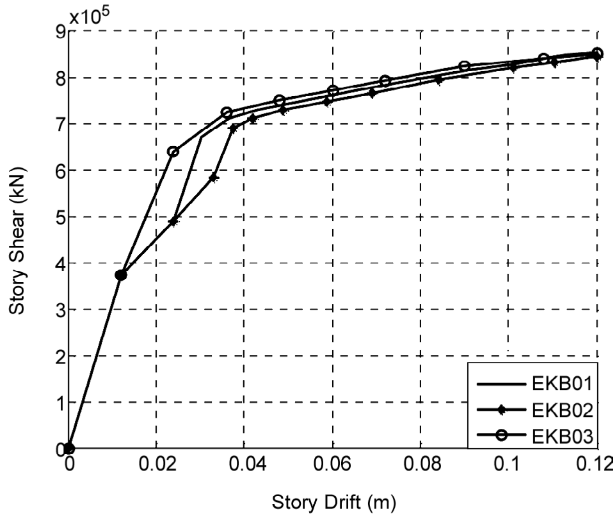

Figure 2. Pushover analysis of three EKBs.

1-bay frames with the height of 3 m and 5 m as bay width. To compare new proposed system (EKB) with one another, these EKBs were different length of gap. The maximum base shear capacity corresponding to ultimate roof displacement is obtained from these analyses. The drift at yield is similar for all the frames because brace sizes are proportioned relative to the strength of the knee and link. If the yield drift were known for a given geometry, the stiffness could be estimated by dividing the frame strength by the yield drift.

The elastic lateral stiffness of the seismic-force-resisting system affects induced deformation demands on critical components, the dynamic behavior and sensitivity to sideways stability (P-delta) effects. Stiffness is a function of the design earthquake intensity and imposed drift limits, system configuration, and relative stiffness of certain key components, such as foundations. To assess the accuracy of the modeling procedure, one of the models previously tested by B. Hosseini Hashemi and Alirezaei [9] was reproduced and the stiffness were compared with that of in previous paper (is shown in Figure 3). Since the basic modeling concept is only some minor changes, e.g. different utilized elements, material modeling techniques, imposing the load, etc., are made to the approach, using just one model to verify the results seems sufficient. Other characteristics of the model such as loading and boundary conditions were selected just as was used in the test.

The EKB system has a fundamental part called

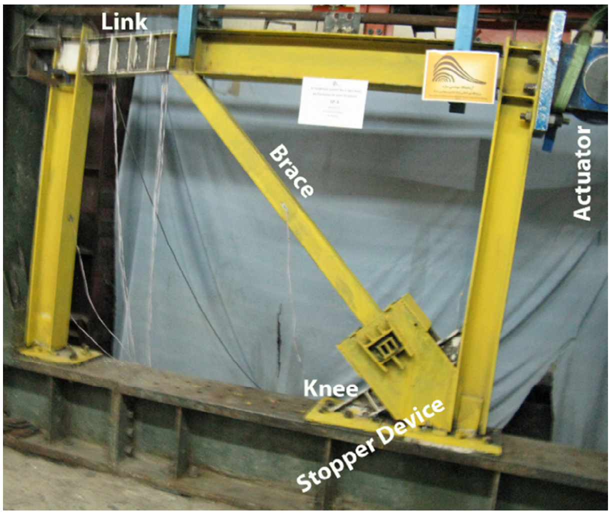

Figure 3. Experimental setup of EKB frame (at end of test) [9].

"Stopper Device" (SD), which can create two separate level of energy dissipation by restricting displacement of knee element. The configuration of SD has been shown in Figure (4).

Knowing the stiffness of an EKB prior to sizing members could eliminate much of the iteration inherent in EKB design. The frame stiffness is important because it is used to estimate link or knee rotations and story drifts, which have prescribed limits. In typical EKB design, a value of e and length of knee is assumed; member sizes are selected based on strength; and then, the story drifts and link rotations are checked. If drifts or rotations are unacceptable, e or length of knee and the member sizes are iterated until an acceptable solution is found. An expression for the stiffness of the EKB that is dependent only on geometry and strength could be used to pick a link length e and beam depth

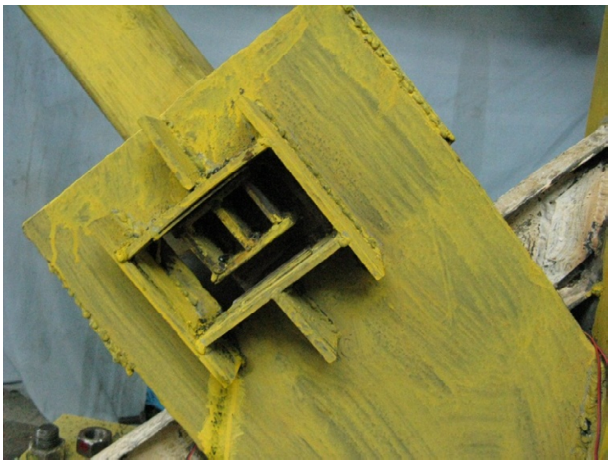

Figure 4. Photograph of stopper device.

that will satisfy deformation limits prior to member sizing. This would reduce iteration and focus the designer's attention on the importance of the EKB geometry itself. This research develops and validates the following expression for the stiffness *k* of an EKB story:

$$K = \frac{V}{\Delta_{da} + \Delta_{ba} + \Delta_{bv} + \Delta_{bf} + \Delta_{kv}} \tag{1}$$

where V_{design} = design story shear and other terms defined later. This equation will be shown to be reasonably accurate for estimating the stiffness of EKB stories, when shear-yielding links and knee are used. In the following this equation is developed theoretically and then validated using data from two experimental EKBs. The validation study indicates that Eq. (1) is sufficiently accurate to be useful in the early design of EKB stories with shear-yielding links and design story shears of at least 600 kN; these conditions are true for the majority of EKB stories. For multistory buildings, Eq. (1) can be used to compute the shear stiffness of individual stories, which can then be combined and also be manipulated to give the required beam depth to achieve a desired stiffness.

2. Development

The development of Eq. (1) is presented in three steps. First, the elastic displacement of an EKB story is expressed in terms of system geometry and member properties. Second, several approximations are used to reformulate the expressions to give the story displacement at yield in terms of frame geometry only. Finally, the story strength is divided by this yield displacement to obtain the expression for frame stiffness.

2.1. *Elastic Displacement in Terms of System Geometry and Member Properties*

The elastic displacement of an EKB story can be computed by summing the displacements caused by the deformations of individual components of the EKB [2]. Figure (5) illustrates the EKB dimensions and displacements resulting from brace axial deformation (Δ_{da}), beam axial deformation (Δ_{ba}), link shear deformation (Δ_{bv}), knee shear deformation (Δ_{kv}) and beam and link flexural deformations (Δ_{bf}).

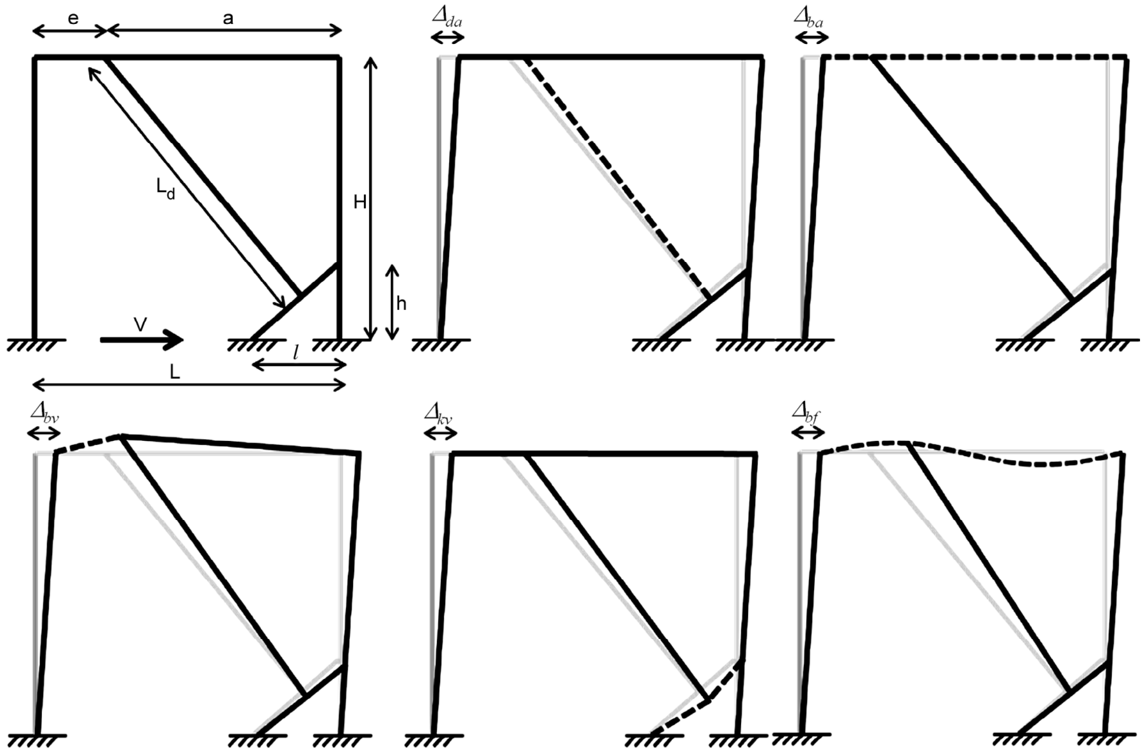

Figure 5. EKB dimensions and components of story drift.

Deflections associated with column axial deformations are negligible for this system. The deflections shown in Figure (5) are quantified in Eqs. (2) to (6), where V is the story shear, E and G are elastic moduli, A_d is area of diagonal brace, A_b is area of beam, A_{bv} is web area of the beam, A_{kv} is web area of the knee and other terms are defined in Figure (5).

$$\Delta_{da} = V\left(\frac{H^2 + a^2}{a^2}\right)\frac{L_d}{A_d E} \tag{2}$$

$$\Delta_{ba} = V\left(\frac{a}{EA_b}\right) \tag{3}$$

$$\Delta_{bv} = \frac{VH^2}{\left(L - \left(\frac{e}{2}\right)\right)GA_{bv}}\left(\frac{e}{L}\right) \tag{4}$$

$$\Delta_{kv} = \frac{Ve_k}{2GA_{kv}\cos^2(\theta)} \tag{5}$$

Above equations were developed as follows. The axial elongation of the diagonal brace is:

$$\delta_d = \frac{F_d L_d}{A_d E} \tag{7}$$

where F_d is the axial force of the brace. With the above definition and base on geometry, the lateral displacements resulting from brace axial deformation can be obtained as follows:

$$\delta_d = \Delta_{da}\left(\frac{a}{\sqrt{H^2 + a^2}}\right) = \frac{F_d L_d}{A_d E} \tag{8}$$

$$\Delta_{da} = \frac{F_d L_d}{A_d E} \times \left(\frac{\sqrt{H^2 + a^2}}{a}\right) = V\left(\frac{H^2 + a^2}{a^2}\right)\frac{L_d}{A_d E} \tag{9}$$

where story shear $V = F_d(a/\sqrt{H^2 + a^2})$. Eqs. (4) to (6) were developed as follows. The relationship between story displacement and link shear rotation, based on AISC341-10 is:

$$\Delta_{bf} \approx 0.8 \times \frac{Ve\tan^2(\theta)\begin{bmatrix} 4L^8 - 8L^7 a + 4L^6 a^2 - 12L^6 e^2 + 18L^5 ae^2 + 4L^5 e^3 - 6L^6 a^2 e^2 - 6L^4 ae^3 + 9L^4 e^4 + \\ 2L^3 a^2 e^3 - 9L^3 ae^4 - 6L^3 e^5 + 9L^2 a^3 e^3 + 9L^2 a^2 e^4 + 6L^2 ae^5 + L^2 e^6 - 6La^3 e^4 - \\ 6La^2 e^5 - Lae^6 + a^3 e^5 + a^2 e^6 \end{bmatrix}}{12EIL^6} \tag{6}$$

$$\Delta_{bv} = \gamma_v h\left(\frac{e}{L}\right) \tag{10}$$

The link shear rotation γ_v can be expressed in terms of the link shear V_L, link shear area A_{bv}, and the shear modulus G:

$$\gamma_v = \frac{V_L}{GA_{bv}} \tag{11}$$

Substituting Eq. (11) into Eq. (10) gives:

$$\Delta_{bv} = \frac{V_L}{GA_{bv}} \times H\left(\frac{e}{L}\right) \tag{12}$$

Based on Figure (6) with taking a moment about support of column gives:

$$V = \frac{V_L}{H} \times \left(L - \left(\frac{e}{2}\right)\right) \tag{13}$$

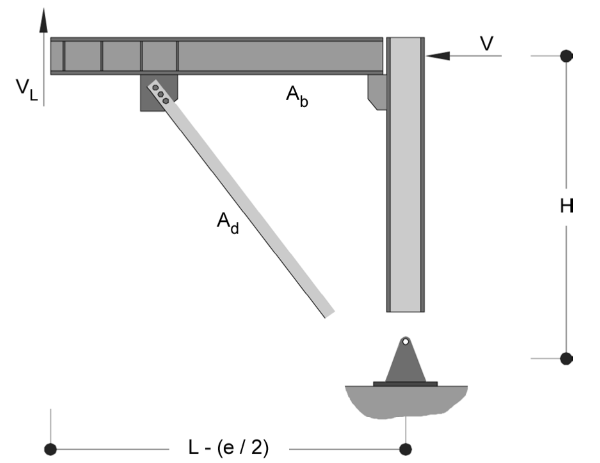

Figure 6. Shear component in link of EKB.

Substituting Eq. (13) into Eq. (12) gives:

$$\Delta_{bv} = \frac{VH^2}{\left(L - \left(\frac{e}{2}\right)\right) GA_{bv}} \left(\frac{e}{L}\right) \tag{14}$$

If P_v is vertical component of brace, we have:

$$P_v = V\tan(\theta) \tag{15}$$

Figure (7) illustrates the bending-moment diagrams under vertical component of brace and lateral unit force. In this figure, the moment at point of load and at fixed end of beam is:

$$M_1 = P_v L\left(\frac{3e^2 L}{2L^3}\right) - P_v(L-a) \tag{16}$$

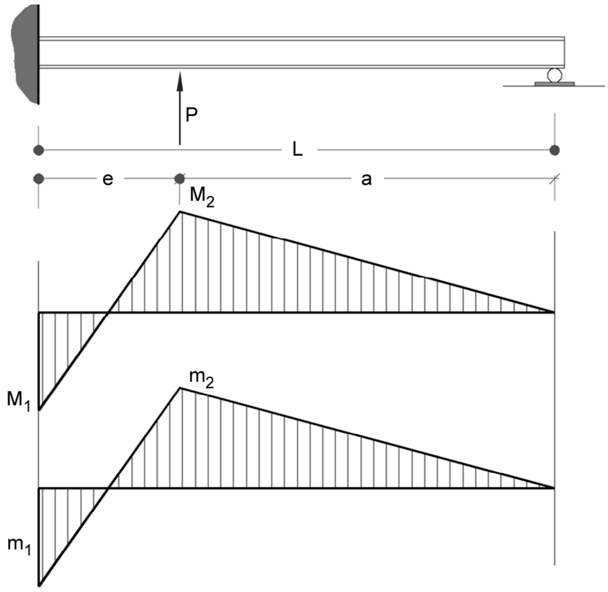

Figure 7. Bending-moment diagrams under vertical component of brace and lateral unit force.

$$M_2 = P_v L\left(\frac{3e^2 L - e^3}{3L^3}\right) \tag{17}$$

$$m_1 = \tan(\theta) L\left(\frac{3e^2 L}{2L^3}\right) - \tan(\theta)(L-a) \tag{18}$$

$$m_2 = a\left(\frac{3e^2 L - e^3}{2L^3}\right)\tan(\theta) \tag{19}$$

Using virtual work, the amount of displacement caused by the bending beam frame after simplification will be as Eq. (6). In most cases, braces are rigidly connected to the beam to relieve the beam of some of the link end moment, preventing yielding of the beam outside the link where axial loads tend to be high. Since the braces relieve the beam of some moment, the drift due to beam flexural deformations is reduced in proportion to the reduced moment. In typical designs, braces take between 10 to 30% of the link end moment. The story displacement caused by flexural deformation of the beam (Eq. 6) is assuming the brace is relieving 20% of the moment.

To determine the frame displacement caused by shear deformation knee element, the shear force in the knee element should be estimated. Diagonal brace axial force under lateral load V is:

$$P = V/\cos(\theta) \tag{20}$$

In the above equation, θ is angle of diagonal bracing. The knee shear force in the knee elements can be assumed half axial force of brace. The knee shear rotation γ_v can be expressed in terms of the knee shear V_k, knee shear area A_{kv}, and the shear modulus G:

$$\gamma_v = \frac{V_k}{GA_{kv}} = \frac{V}{2GA_{kv}\cos(\theta)} \tag{21}$$

Substituting Eq. (21) into Eq. (20), the Eq. (5) will be achieved.

3. Validation of EKB Frame Stiffness

Since several approximations were required to develop Eq. (1), its accuracy is suspect without validation. To check the equation, the stiffness of an EKB was determined from experimental result and then compared with the stiffness estimated by Eq. (1) (based on the geometry only). The EKB for the validation study represented typical geometries and strengths. To assess the accuracy of the stiffness, one of the models (SP1) that previously tested by Hosseini Hashemi and Alirezaei [12]. Strength and stiffness are the major characteristics of the lateral resistant systems in resisting the seismic loads [13]. In Table (1), stiffness, yield strength, ultimate strength, and ductility of the all specimens based on experimental hysteresis curves are presented. In this study, component stiffness or effective stiffness was calculated based on the secant stiffness at yield level forces [9].

Table 1. Comparison of the stiffness and shear strength of experimental models [12].

Specimen	Stiffness (K) (kN/m)	Yield Strength (F_y) (kN)	Ultimate Strength (F_u) (kN)	Ductility $\mu=\Delta_u/\Delta_y$
SP1	28400	420	600	5.8
SP2	28400	345	630	8

The overall test specimen dimensions were set to a height of 1600 mm and width, L, of 2250 mm. After yield of knee element, stiffness of frame is only provided by link element. Based on the experimental investigation, the initial stiffness of the specimen SP1 was determined to be 28400 kN/m from the elastic cycles. Besides, from analytical result base on Eq. (1) the elastic stiffness was determined to be 31208 kN/m.

3. Conclusions

In this research the lateral elastic stiffness of a new eccentric and knee bracing system named Eccentrically Knee Brace (EKB) is investigated. The validation study showed that Eq. (1) is close conformity with experimental results. The percent difference between Eq. (1) and the stiffness determined from experimental investigation is less than 10%. The tests reveal that the EKB can dissipate a large amount of energy without an appreciable loss of strength in two level of performance. The knee and link beam act as a fuse to prevent other element in the frame being overstressed. The second objective is to demonstrate the EKB performance using experimental and analytical simulation. 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. The study shows that if EKB is used in a frame, the seismic behavior of these frames such as the ductility of the specimen, reduced the input of the earthquake energy and had enough lateral displacement stiffness improves noticeably.

Acknowledgments

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