Cyclic Loading Test of Double K-Braced Reinforced Concrete Frame Subassemblies with Buckling Restrainted Braces

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Abstract: Although the unbalanced brace force at the mid-length of columns discourages the use of K- or double K-bracing in concentrically braced frames, this limitation needs re-evaluation when K- or double K-bracing is implemented in buckling restrained braced frames. Unlike ordinary steel braces, buckling restrained braces (BRBs) exhibit almost identical behavior in tension and compression, and to arrange BRBs in a double K configuration may provide an efficient solution of connecting BRBs to reinforced concrete members by exempting the steel-to-concrete interface from unfavorable tensile force. Three 1/2-scaled one-story one-span subassemblies were subjected to cyclic loading to examine the above idea. Two of the specimens were braced by BRBs in double K configuration, whilst the other was a bare frame for comparison. During the test, the BRBs performed as intended in both tension and compression. The RC frames in the braced specimens were only moderately damaged at 1% inter-story drift when the BRB cores sustained large plastic strain. The damage patterns of the braced RC frames were similar to that of the bare frame throughout the loading process up to 2% inter-story drift. No cracks were observed at the mid-length of the columns or the mid-span of the beams where the BRBs were connected.

Keywords: double K-braced frames; seismic damage, buckling restrained brace; steel-to-concrete connections; reinforced concrete frame; Menegotto-Pinto model

1 Introduction

Developed in late 1980s in Japan for seismic protection of steel structures \(^1\), \(^2\), buckling restrained braces (BRBs) have been increasingly used in reinforced concrete (RC) structures as well during the past decades over the world \(^3\)-\(^6\). Although RC moment-resisting frames exhibit higher lateral stiffness than steel frames do, the effectiveness of using BRBs to mitigate the seismic damage to RC frames has been demonstrated through both numerical analyses and experimental tests (e.g., \(^5\),\(^7\),\(^8\) among many others).

Nevertheless, there remain several challenges when combining steel braces with concrete frames. A significant challenge is to transfer the brace tension force to concrete members which are weak in tension. Conventional solutions include attaching or inserting steel braced frames instead of separate braces to RC frames by extensive anchor bolts. This practice is widely used in Japan for the seismic retrofit of existing RC frames and efforts have been made to find ways of reducing the number of anchor bolts to lower the cost and environmental impact \(^9\),\(^10\). For new constructions, an intuitive solution is to embed some steel parts in concrete members to receive the brace gusset connections \(^11\), in some cases even making the RC members into steel reinforced concrete (SRC) ones \(^12\). Recently, several innovative solutions were proposed to deal with the unfavorable tension force. Benavent-Climent \textit{et al.} \(^13\) proposed to use a pair of shear key plates to hold the corner gusset of a brace, which is not directly connected to concrete. In such a manner, the axial tension force of BRBs is converted to almost pure shear force before being transmitted to the concrete. Similar idea was partially applied by Ichikawa \textit{et al.} \(^14\) to separate the tension and shear resistance in the
corner gussets of BRBs in RC frames. Both methods can greatly simplify the local load transfer around corner gusset connections, thus making the connections more reliable and easier to design.

Another problem comes from the detrimental interaction between the gusset plate and the frame members, which is usually referred to as the ‘frame action’ (Figure 1). Kishiki et al. [15] investigated the stress concentration at the toes of corner gussets in steel braced frames because of the frame action. Such concentrated stress may lead to premature fracture of the gusset plate. Chou and Liu [16] demonstrated in their experimental test that the corner gusset plate could buckle at much lower story drift than expected when free-edge stiffeners were absent. Numerical analysis showed that the force in the gusset plate resulting from the frame action was on the same order as the brace axial force. Another side effect of the frame action is the considerable over-strength in braced frames [17], which may not be properly taken into account in the design. While specific design methods can be developed to address the effect of frame action in corner gussets [18], alternative corner gusset configurations were proposed to avoid frame action. Ishii et al. [19] anchored the gusset plates of BRBs to the side surfaces of RC beam ends by post-tensioned steel rods. Berman and Bruneau [20] proposed an unconstrained gusset connection for steel constructions, in which the gusset plate is separated with the column by an intended gap and bolted to the beam end. This idea was later applied to RC frames with BRBs by Khampanit et al.[21] and Qu et al. [22].

To address both the challenges of axial tension force and frame action, Qu et al. [23] proposed to install BRBs in a zigzag configuration in RC frames and eliminate the RC beams in the braced span so that the braced span forms a Warren truss. In the zigzag configuration, the BRB gusset plates are anchored on the side surfaces of beam-to-column joints by independent shear and tension resistant elements. The axial forces of two adjacent BRBs, which share the same gusset plate, are expected to counteract each other when the structure deforms laterally, so that the demands for the horizontal resistance of the connection can be greatly reduced or even eliminated. However, numerical analysis showed that the higher mode vibration of buildings would lead to significant tension force on the gusset-to-concrete interface [24].

The double-K bracing as shown in Figure 2 provides a promising solution to this problem. Similar to those in zigzag configurations, the BRBs in a double-K configuration share gusset plates. The two neighboring braces are always acting in opposite directions, that is, one in tension and the other in compression. This mechanism is not influenced by higher mode vibration because it takes place within a single story. Ideally, when BRBs of the same properties are used in a single span, there will be little unbalanced tension force on the gusset plate. As a result, much less studs or post-installed anchors would be required to transfer the brace axial force to the concrete. Some unbalanced compression force may rise from the higher compression strength of BRBs. AISC 341-10 [25] requires that the BRB compression strength should be no greater than 1.3 times its tension strength. The same requirement was included in a Chinese specification for BRBs which is to be published soon. This sets an upper limit for the unbalanced
compression force at the mid-length of the beams and columns. Such compression force may introduce friction to the connection interface, and thus increase the shear capacity of the connection.

In double-K bracing, the gusset connections at the mid-length of the beams and columns are free of the detrimental ‘frame action’, and are thus easier to proportion. In addition, the gusset connections are away from the potential ‘plastic hinges’ at the RC beam ends, and therefore the inelastic behavior (e.g., concrete cracking, rebar yielding) would not interact with the gusset connections in an unintended manner. For quick construction, the four BRBs and their gusset plates in a single span can be prefabricated as an integrated energy-dissipating unit ready to be shipped to the site for on-site installation.

Figure 2. Reinforced concrete frames braced by BRBs in double-K configuration.

The term ‘K-bracing’ has been used in the past to refer to what we call inverted V-bracing nowadays e.g. [26]. Nonetheless, AISC 341-02 [27] defines K-braced frame as “an OCBF (ordinary concentrically braced frame) in which a pair of diagonal braces located on one side of a column is connected to a single point within the clear column height”. In AISC 341-10 [25], this definition is evolved to “a braced-frame configuration in which braces connect to a column at a location with no out-of-plane support”. By both definition, the bracing configuration in Figure 2 falls into the category of K-bracing. To emphasize that the ‘K’ needs to appear in pair (that is, four BRBs in a single span) so that the interfacial tension force can be eliminated, the term ‘double-K’ is used in this paper.

For steel constructions, double K-bracing is prohibited by AISC 341-10 [25] for special concentrically braced frames. This is primarily because the braces intersect at mid-height of the columns and their unbalanced force may cause the columns to failure, thus triggering collapse of the building [28]. The same provisions also prohibit the use of double K-bracing in buckling restrained braced frame. However, as already mentioned, the unbalanced force of intersected BRBs should be small.

In the Chinese seismic code [29], there is only a small section regarding the design requirements for structures with energy dissipating devices, which does not include any restraint for the use of specific bracing configurations. However, in the section for steel structures in the same code, it is prohibited to use BRBs in a K-bracing configuration. Since no detailed explanation is provided for this restriction, it is deemed to have been strongly influenced by the AISC provisions.

In Japan, however, it always remains an option to connect braces within the clear span of columns. As a demonstration of the “damage tolerant structure” concept, buckling restrained braces were installed in the perimeter steel frames of a 40-story office building in Tokyo [30]. The brace configuration in a single span was similar to that in Figure 2 but the two adjacent BRBs did not intersect at the mid-length of the beams/columns. More recently, a minimal-disturbance arm damper was proposed in Japan for the seismic retrofit of existing frames. The damper connects the mid-span of beams with the upper part of column and thus imposes an additional shear force in the column within its clear span, whereas the magnitude of this
detrimental force on the column is limited by the strength of the bending plates in the damper [31].

To assess the seismic performance of double-K braced frame structures with BRBs, cyclic loading tests were performed on three subassembly specimens and the preliminary results were reported by [32]. More detailed discussions on the test results are summarized in this paper, which would provide substantial evidence to support the use of double-K bracing in RC frames with BRBs. It is also worth noting that a global plastic mechanism that can activate more energy dissipating devices is usually preferred for all types of passively controlled structures including the one with double-K configured BRBs as discussed herein. Additional design effort such as those introduced by [7] and [33] is needed in proportioning the primary structural components and the energy dissipating devices. This is beyond the scope of the current discussion but represents a future research interest.

2 Test Program

2.1 Specimens

Three 1/2-scaled RC frame subassemblies were subjected to cyclic loading with increasing lateral drift amplitudes to assess the seismic damage to RC frames and the cyclic performance of BRBs in double-K configuration. Two of the specimens were braced by BRBs, whereas the other was a bare frame counterpart for comparison (Table 1). To simulate the behavior of a middle story in a multi-story building, the RC part in each subassembly consists of two beams framing into two columns (Figure 3). The multi-story building from which the subassemblies were separated was assumed to be over 24 m high and locate on a site of Intensity 8 (i.e., the design level spectral acceleration in short period region \( S_{ad} = 0.45 \) g), so that the frames fell into the Seismic Design Category I per the Chinese seismic code [29]. The design of the RC parts conforms to the Chinese code for concrete structures [35], and a strong column-weak beam concept was applied in proportioning the beams and columns. In the Chinese seismic code, the sum of the flexural strength of columns framing into a joint is required to be no less than 1.2 to 1.7 times the sum of the design strengths of the beams framing into the same joint for bare moment-resisting frames, depending on the Seismic Design Category. While the required amplification factor is 1.7 for bare frames in Category I, it is allowed by the code to lower it by at most 0.2 if the structural is protected by additional energy dissipating devices. Therefore, an amplification factor of no less than 1.5 was adopted in proportioning the specimens of the current test. As a result, the beam flexural reinforcement ratio is about 1.1%, and the column overall longitudinal reinforcement ratio is 2.7%. Ductile detailing was provided for both the beams and columns. The intervals of the beam stirrups and column hoops were reduced by half in the regions close to the beam-to-column joints. C40 concrete (40 MPa nominal compressive strength of cubes) and HRB400 rebars (400 MPa nominal yield strength) were used for the RC frames. The measured average compressive strength of concrete cubes was 49.7 MPa. The measured yield and ultimate strength of the \( \#14 \) longitudinal rebars were 563 MPa and 673 MPa, respectively.

Table 1. Specimens

<table>
<thead>
<tr>
<th>ID</th>
<th>Specimen</th>
<th>Steel-to-concrete connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.1</td>
<td>Bare RC frame</td>
<td>-</td>
</tr>
<tr>
<td>No.2</td>
<td>Double-K braced RC frame with BRBs</td>
<td>Embedded shear studs</td>
</tr>
<tr>
<td>No.3</td>
<td>Double-K braced RC frame with BRBs</td>
<td>Post-installed chemical anchors</td>
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Four identical BRBs were installed in a double-K configuration in each braced specimen. The BRBs were manufactured by Lead Dynamic Engineering Co., Ltd., a joint-venture by Nippon Steel and another two Chinese local companies. The proportioning of the BRBs was controlled by (1) the brace-to-frame strength ratio and (2) the ultimate strain of the BRB steel cores. The steel core was made of 10 mm thick steel plate of Chinese Q235 steel (commonly-used structural steel of 235 MPa nominal yield strength). The nominal axial strength (including an assumed 1.5 times over-strength because of strain hardening) of each BRB is 200 kN so that the additional lateral strength provided by the BRBs is 338 kN, approximately two times that of the RC frame, which is 167 kN per the nominal material strengths.

While the distance between the intersection points of the BRB axes is 1595 mm, the length of the plastic segment of the BRB steel core is 500 mm (Figure 4). As a result, the strain in the plastic segment of the BRBs would be approximately 1.7% at 1/100 inter-story drift assuming that the elastic deformation of the remaining parts of the BRBs including the connections is negligibly small. It would become 3.4% at 1/50 inter-story drift. Previous tests on similar BRBs provided by the same company demonstrated stable hysteretic performance of the BRBs at such strain levels [23].

The equivalent axial stiffness of each brace between the two intersection points is approximately 144.7 kN/mm. The additional stiffness provided by the braces is thus 122.4 kN/mm, which is approximately three times that of the bare frame, which is 41.6 kN/mm. The high stiffness of the braces corresponds to a 1/927 inter-story drift ratio when the BRBs first yield (no strain hardening). Such a yield drift ratio is very small as compared to that of the bare RC frame, which is estimated to be 1/100 according to the empirical equation for yield story drift ratio, $IDR = 0.5\varepsilon_y L_B/h_B$, where, $\varepsilon_y$ is the yield strain of rebars, $L_B$ and $h_B$ are the span length and sectional height of beams, respectively (Priestley et al, 2007) [34].
The BRBs were bolted to the gusset plates, which were anchored to the RC beams or columns by two different methods for the two braced specimens. In one method proposed for new constructions, M19 shear studs (19mm in diameter) were welded to the base plate of the gusset plates, and were embedded in the RC components (Figure 5(a)). In the other method, 12 post-installed chemical anchors were used for the connection to simulate applications in seismic retrofit of existing buildings. In this case, the RC frame was first cast without the BRB gusset plates and cured for 28 days. Then the chemical anchors were installed in the 15 holes drilled in the concrete with epoxy resin, then fit into the tapered holes and welded to the base plate of gusset base (Figure 5(b)). Following the capacity design concept, the number of shear studs and post-installed anchors were determined by the resultant shear force of the adjusted BRB strength at the connection interfaces, neglecting the possible compression or tension force on the interface. The adjusted BRB strength takes into account 50% strain hardening for both BRBs connecting to the same gusset plate and another 30% over-strength for the BRB in compression. The design of the embedded studs conforms to the provisions on composite beams in the Chinese code for steel structures \[36\]. The specified yield strength of studs is 215 MPa, and a strain hardening factor of 1.67 is applied in calculating the stud ultimate strength. The chemical anchor group is designed conforming to the Chinese code for strengthening concrete structure \[37\]. The specified shear strength of the anchor steel is 290 MPa.

2.2 Test setup, loading and measurement

The specimens were mounted on two mechanical pins that were firmly fastened on the strong floor (Figure 6 and 7). Two horizontal actuators of 1000 kN capacity were employed to load the specimens. The upper actuator was displacement-controlled to impose the desired inter-story drift ratio (IDR) on the specimen, while the lower actuator was force-controlled to fully counteract the upper actuator so that the bottom mechanical pins were waived from excessive shear.

End plates were provided at the far ends of the beams and columns for connections to the loading jigs.
The longitudinal rebars were welded to the end plates. Four M16 high-strength bolts protruded from the outer surface of each end plate, ready to be fastened to the steel loading jigs.

Two spherical-headed oil jacks were employed to exert axial force through steel jigs on top of the columns. Roller cushions were installed between the oil jacks and the overhead reaction beam so that the jacks could move horizontally along with the top of the columns. The steel jigs fastened to the top of the columns were restrained by pantographs for out-of-plane stability. Before lateral loading, the oil jacks applied 482 kN axial compression force to each of the columns to simulate the gravity load of upper stories. This corresponds to 20% of the column’s nominal axial strength, and approximately 16% of its actual axial strength. To simulate the realistic construction process, the BRBs were not fully fastened until the dead load was imposed. During the test, the forces in the oil jacks were constantly adjusted to counteract the overturning moment resulting from the horizontal loading, so as to maintain constant axial forces in the lower halves of the columns to protect the mechanical pins under them.

The additional distributed loads along the beams (e.g., the dead load of slabs and the live loads) were neglected in the test because (1) the inclusion of these load would bring about additional complexity of the test setup without making significant difference to the global behavior of the specimens and (2) these loads would result in more complicated curvature distributions in the beams, which would make the analysis of the unbalance force of BRBs more difficult.

Figure 6. Test setup
Inter-story drift ratios of 1/1200, 1/550, 1/200, 1/100, 1/67 and 1/50 were selected as the target amplitudes for the loading protocol (Figure 7). Among these, IDR = 1/550 and 1/50 are the lateral drift limits for RC moment-resisting frames at their serviceability and ultimate limit states, respectively, per the Chinese seismic code \[29\]. IDR = 1/200 corresponds to the drift limit stipulated in the Building Standard Law of Japan for serviceability limit state \[38\]. It is also the drift limit in Eurocode 8 for damage limitation of RC frames with brittle nonstructural elements \[39\]. IDR = 1/100 is usually taken as an inelastic drift limit for passive-controlled structures under major earthquakes in Japan’s seismic design practice. The 1/100 and 1/200 drift limits were adopted in the loading protocol to explore the specimens’ behavior at more levels of deformation, although they are not stipulated in the Chinese codes. In particular, the 1/100 drift ratio is regarded as the performance target of the archetype structure under major earthquakes. Two cycles of static loading were performed at each amplitude.

Displacement sensors were used to monitor the deformation of the specimens. The locations of the sensors are shown in Figure 9. In particular, two sensors, namely D1 and D2, are used to monitor the story drift of the specimen. B1–B8 are for the axial deformation of the BRBs, R1–R4 are for the end rotations of the lower eastern BRB, and S1–S4 are for the slip of the four gusset plates of BRBs. In addition to the displacement sensors, strain gauges were attached to the outermost longitudinal rebars at three sections (i.e., both ends and mid-length) of each beam or column. The locations of the sections are also depicted in Figure 9.
3 Test results

3.1 Global responses

As shown in Figure 10, compared to the bare frame specimen (No.1), the two braced frame specimens exhibited stable and full hysteresis throughout the loading process up to 1/50 inter-story drift ratio. The BRBs started to yield at IDR = 1/604 and 1/560 in the braced frame with shear studs (No.2) and the one with chemical anchors (No.3), respectively. These drift ratios are much larger than the estimated yield drift ratio of 1/927. This is primarily because of the local deformation loss at the brace connections, which will be discussed later in Section 3.4.

On the other hand, the bare frame specimen did not exhibit significant yielding until IDR = 1/78 in the negative direction and 1/81 in the positive direction. Rather than the first yielding of rebars determined by specific strain readings, these yield drift ratios were identified on the skeleton curve as the points where the tangent stiffness was degraded significantly (Figure 10a). For the current test setup, the drift ratio includes both the deflection of the RC column and the rotation of the lower beam-to-column joints which is related to the chord rotation of the beams. In this specific case, the story drift associated with the beam chord rotation constitutes approximately 77% of the total story drift when the frame remained elastic.

For specimen No.3, one of the four BRBs was removed at the unloaded point before the last load cycle of IDR = 1/50. Then the loading was resumed to check the BRB-to-concrete connection performance when the force balance of BRBs was broken. The hysteresis of this last loading cycle is depicted by the dashed line in Figure 10. The only significant phenomenon was that the peak force dropped by 21% (from 821.4 kN to 650.2 kN) and 18% (from −895.4 kN to −693.8 kN) in the positive and negative loading directions, respectively. The damage to the concrete members or the steel-to-concrete connections were not exacerbated by the removal of the BRB.
The BRBs in a double-K configuration are shorter than in other configurations such as V- or inverted-V bracing. This leads to shorter plastic segments and consequently larger plastic strain in the core if the story drift is the same. The results in Figure 10 show that the commercially available BRBs used in the current test successfully exhibited stable hysteresis. The maximum average strains in the cores, which were taken as the measured axial deformation divided by the length of the BRBs’ plastic segments, were approximately 3.6% in specimen No.2 and 3.2% in No.3, where the BRB’s axial deformation was measured by displacement sensors B1~B8 as shown in Figure 6. Since these sensors spanned over the whole lengths of the BRBs including their elastic segments and the connections, the above strains are upper-bound values for the actual strains experienced by the plastic segments.

3.2 BRB hysteresis

For the statically undetermined system of the loading setup, the axial force of individual BRBs could hardly be directly measured during the test. To evaluate the force transmitted to the steel-to-concrete connections, the Menegotto-Pinto model is adopted to estimate the BRB axial force from the measured axial deformation, which takes the form of Equation (1) [40].

\[
\sigma^* = b\varepsilon^* + \left(1 - b\right)\varepsilon^* \left(1 + \varepsilon^* R\right) R
\]

where \(\varepsilon^*\) and \(\sigma^*\) are the normalized strain and stress that are calculated by Equation (2) and (3), respectively; \(b\) is the strain hardening ratio; \(R\) is a parameter that controls the shape of the transition curve between the two asymptotes of elastic loading and post-yield hardening branches.

\[
\varepsilon^* = \frac{\varepsilon - \varepsilon_r}{\varepsilon_0 - \varepsilon_r}
\]

\[
\sigma^* = \frac{\sigma - \sigma_r}{\sigma_0 - \sigma_r}
\]

where \(\varepsilon\) and \(\sigma\) are strain and stress; \(\varepsilon_0\) and \(\sigma_0\) are stress and strain at the intersection point of the two asymptotes; \(\varepsilon_r\) and \(\sigma_r\) are strain and stress at the previous strain reversal.

\(R\) is further evaluated by Equation (4) as below.

\[
R = R_0 \left(1 - \frac{c_{R1} \xi}{c_{R2} + \xi}\right)
\]

where \(R_0\), \(c_{R1}\) and \(c_{R2}\) are experimentally determined parameters; \(\xi\) is the normalized plastic strain as defined in Equation (5).
\[ \xi = \left| \frac{\varepsilon_p - \varepsilon_0}{\varepsilon_y} \right| \]  \hspace{1cm} (5)

where \( \varepsilon_p \) and \( \varepsilon_y \) are plastic strain and yield strain, respectively.

Filippou et al. \cite{41} proposed a stress shift to consider the isotropic hardening on the basis of the Menegotto-Pinto model. Before evaluating the intersection point \((\varepsilon_0, \sigma_0)\), the yield asymptotes is first moved parallel to its direction by a stress shift, \(\sigma_{st}\), which is calculated by Equation (6).

\[ \sigma_{st} = a_1 \left(\frac{\varepsilon_{max} - \varepsilon_{min}}{2a_2\varepsilon_y}\right)^{0.8} \]  \hspace{1cm} (6)

where \( \varepsilon_{max} \) and \( \varepsilon_{min} \) are the absolute maximum and minimum strains at strain reversal, respectively; \(a_1\) and \(a_2\) are experimentally determined parameters.

The above model is referred to as the Steel02 model (Giuffré-Menegotto-Pinto Model with Isotropic Strain Hardening) in OpenSees \cite{42}. To capture the characteristics of BRBs which exhibit slightly higher strength in compression than in tension, the model is modified to allow for different sets of strain hardening parameters, \(b, a_1\) and \(a_2\) for compression and tension.

The model parameters are calibrated against the hysteretic curve of a uniaxially loaded BRB that was the same as those used in the present subassembly tests (Figure 11). With the values of the parameters as listed in Table 2, the model gives satisfactory estimates of the BRB hysteresis. The deviation between the model and the test result is slightly larger in compression than in tension. This is because linear strain hardening with a larger slope in compression was used to approximately model the complicated behavior of compression over-strength of the BRB, which is dependent on the sectional expansion of the steel core and the friction between the core and the outer restraining elements.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calibrated value</th>
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</thead>
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<tr>
<td></td>
<td>In compression</td>
</tr>
<tr>
<td>(R_0)</td>
<td>26</td>
</tr>
<tr>
<td>(\sigma_{R1})</td>
<td>0.925</td>
</tr>
<tr>
<td>(\sigma_{R2})</td>
<td>0.15</td>
</tr>
<tr>
<td>(b)</td>
<td>0.02</td>
</tr>
<tr>
<td>(a_1)</td>
<td>0.035</td>
</tr>
<tr>
<td>(a_2)</td>
<td>1</td>
</tr>
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</table>

![Graph showing hysteresis curve](image-url)
The axial forces of the BRBs in the subassemblage specimens during the loading is calculated corresponding to the measured BRB axial deformations by the above calibrated model. Although these force results are considered accurate enough for the following discussions, it should be noted that their accuracy may be influenced by such factors as the scattering of the material properties and the BRB end rotations in the subassembly test which did not present in the uniaxial calibration test.

3.3 Unbalanced BRB force on connection interface and its influence

The force components on the four steel-to-concrete connection interfaces are evaluated through the equilibrium of the BRB axial force. Figure 12(a) depicts the hysteresis of the force component normal to the connection interface on the western column of specimen No.3, which is connected to the lower western (LW) and the upper western (UW) braces. During the second load cycle of $IDR = 1/100$, both the tension and compression forces are small as compared to the BRB’s strength. In particular, the maximum tension force at point C is less than 20 kN, almost negligible in the design of the shear connection (either of shear studs or chemical anchors). In Figure 12(b), it is clear that the compressive normal force in Figure 12(a) is primarily a result of the compression over-strength of the BRBs, whereas the tensile normal force takes the maximum value during the transition from elastic loading to yielding, which is related to the Bauschinger effect.
Similar phenomena can be observed for the second load cycle of IDR = 1/50 except that the magnitude of the normal tensile and compressive force were increased (Figure 11). Even though, the maximum tensile force is no greater than 30 kN, which is only approximately 8% of the design shear force of the BRB connections on the columns. It is worth noting that the drift limit of RC structures with supplemental energy dissipating devices such as BRBs are usually required to be less than IDR = 1/100 under major earthquakes (such as in the Japanese practice), the behavior at IDR=1/50 should be interpreted as beyond-design level performance.

The negligible influence of the small normal forces at the mid-length of the columns and beams is also demonstrated by the curvature distributions in the RC frames. The curvatures were captured by the strains of the longitudinal rebars at three cross sections of each beam and column. Because the strain readings of many rebars, especially those in the beams, became unreliable when the deformation approached IDR = 1/100, the results at the second load cycle at IDR = 1/200 are given in Figure 13. At such a drift level, the RC members remain essentially elastic and the curvatures are expected to be small. The curvature distributions in the RC frames of the three specimens are quite similar to one another. No obvious curvature is observed at the mid-length of the beams or columns in specimen No.2 and 3, where the BRBs are intersected.
The apparent damage to the RC frames in the three specimens corresponded well to their curvature distributions. At the peak drift of each load cycle, the cracks on the concrete surface were marked and recorded on a grid that coincides with the rebars and stirrups (Figure 14). The crack widths were read by crack scales on points where the crack crossed the grid lines (Figure 15). Regardless of the existence of the BRBs, the crack patterns (both the distribution and widths) of specimens No.2 and 3 were similar to that of the bare frame specimen (No.1). As intended, most damage was concentrated in the beam ends, where major cracks developed and slight concrete spalling was observed at the final stage of the loading. Take specimen No.2 for example. Cracks were first observed on the beams, and no cracks were found on the columns until the second load cycle of IDR = 1/100. At IDR = 1/100, a few cracks at the beam ends grew to wider than 0.2 mm. These cracks continued to open and some of them became wider than 1.0 mm at IDR = 1/50. Cracks on the columns were distributed at the beam-to-column joints and regions close to the joints. The column crack widths were considerably smaller than those on the beams.

Figure 13. Curvature distributions of RC frames at: (a) IDR = 1/200 and (b) IDR = −1/200 (unit: με/mm).

Figure 14. Damaged RC members with marked cracks of Specimen No.2 at end of 1/50 load cycle: (a) lower-east joint and (b) lower-west joint.
The above results show that the BRBs and their gusset connections in the double-K configuration would not deteriorate the seismic performance of the RC frame. The similar levels of damage should not be interpreted in a way that the BRBs bring no benefit to reduce the structural damage because such a similarity is observed at the same story drift. The maximum story drift of an RC frame equipped with BRBs would be much less than that of a bare frame because of the addition stiffness and energy dissipation provided by the braces. In such cases, the damage to the RC frame braced by BRBs is expected to be much less than that to a bare frame.

3.4 Deformation loss at BRB-to-concrete connections

It is usually preferred to have stiff brace connections so that the brace axial deformation is concentrated in the energy dissipating segments. In this sense, the effectiveness of brace connections can be evaluated in terms of the ‘deformation loss’ in the connection, which is calculated by subtracting the actual deformation of a BRB’s plastic core from its maximum possible deformation when assuming the rest of the brace and the connections are rigid. In the double-K configuration as shown in Figure 16 in which one-fourth of a single span is taken as a free body, the maximum possible deformation of a BRB, $\delta_{\text{max}}$, can be expressed as a function...
of the story drift ratio, IDR, the rotation at the mid-length of the beam, \( \theta_b \), and the column, \( \theta_c \) (Equation 7). By assuming that the RC frame remains elastic, \( R \), \( \theta_b \) and \( \theta_c \) can be calculated by Equation (8) ~ (10).

![Diagram showing the deformation of braces](image)

Figure 16. Maximum possible axial deformation of braces

\[
\delta_{lm} = \frac{RH}{2} \cos \beta + \frac{h_c}{2} \theta_c \sin \beta + \frac{h_b}{2} \theta_b \sin \beta
\]  

(7)

where \( H \) is the story height; \( h_c \) and \( h_b \) are the depths of the column and beam, respectively; \( \beta \) is the inclination angle of the brace.

\[
\Delta = \frac{RH}{2} = \frac{PH^2}{24} \left( \frac{H}{E I_c} + \frac{L}{E I_b} \right)
\]  

(8)

\[
\theta_c = \frac{PH}{4} \left( \frac{H}{2E I_c} + \frac{L}{3E I_b} \right)
\]  

(9)

\[
\theta_b = \frac{PHL}{24E I_b}
\]  

(10)

where \( P \) is the story shear force; \( L \) is the span length; \( E \) is the concrete Young’s modulus; \( I_b \) and \( I_c \) are the moment of inertia of the beam and column, respectively.

Substituting the specific geometric and material properties of the current specimens in Equation (7) ~ (10) gives a simple numerical relationship between the inter-story drift ratio, IDR, and the maximum possible brace deformation, \( \delta_{lm} = 0.58 \cdot (IDR) \cdot H \cos \beta \). The deformation loss ratio is defined in Equation (11) by \( \delta_{lm} \) and the measured BRB core deformation \( \delta_{BRB} \), and is depicted in Figure 17 for all the drift amplitudes. The values shown in the figure are the average of the four BRBs in each specimen (in case of No.3 during the second load cycle of 1/50 drift, it is the average ratio of the three remaining BRBs).

\[
\text{Deformation loss ratio} = \frac{\delta_{lm} - \delta_{BRB}}{\delta_{lm}}
\]  

(11)
At small drift amplitudes when the BRBs were essentially elastic, the deformation loss took more than 20% of the maximum possible brace deformation. This ratio decreased rapidly at larger drift amplitudes as the BRBs sustained significant plastic deformation. The deformation loss in specimen No.3 is larger than that of No.2 at all stages, indicating that the post-installed chemical anchors exhibited lower stiffness than the embedded studs.

Although the deformation loss may come from various sources, it is predominated by the slip of the gusset plate along the beam/column’s axis in double-K braced frames in which the tension force on the connection interface is very limited. Taking the gusset connection on the lower beam in specimen No.3 for example, Figure 18 shows that the deformation loss at a single connection (taken as half the deformation loss of a brace) and the contribution of gusset slip, \( \delta \cdot \cos \beta \), where \( \delta \) is the measured gusset slip, are very close to each other at various drift amplitudes.

The peak gusset slips in specimens No.2 and 3 were almost identical when the story drift ratio, IDR, was no greater than 1/100, although the hysteresis curves were somehow different (Figure 19(a)). During the remaining load cycles, the gusset slip in specimen No.2 remained almost the same as in previous load cycles while that in No.3 was significantly increased (Figure 19(b)), indicating softening of the chemical anchor connection.
For specimen No.3, one of the BRBs was removed before the second load cycle of IDR = 1/50. As a result, half of the shear force on the connected gusset plate was released and the gusset slips were thus decreased as shown by the dashed line in Figure 19(b). On the other hand, it broke the BRB force balance and subjected the gusset connection to significant tension force. The latter effect was so overwhelming that, despite the considerable decrease in gusset slips, the overall deformation loss of BRBs was increased by 43% from 16% in the first 1/50 load cycle to 23% in the second 1/50 cycle (Figure 17). In other words, the break of BRB force balance by removing one of the braces exaggerate the deformation loss in the connections, although it did not cause strength problem in the current test.

3.5 BRB end rotations

In addition to the aforementioned larger core strains in the BRBs in double-K configuration, which are usually shorter than in other commonly-seen configurations, another possible side effect is larger rotations at the ends of BRBs’ plastic cores, which may have detrimental effects on the performance of the BRBs [43]. In a double-K configuration, the BRB end rotation can be decomposed into components related to (a) lateral drift, $\Delta$, (b) beam gusset rotation, $\theta_b$ and (c) column gusset rotation, $\theta_c$, as depicted in Figure 20. The end rotations of a brace at its column-side end and beam-side end can thus be estimated by Equation (11) and (12), respectively, assuming that the BRB connections and segments outside the core are rigid.

\[
\theta_{EC} = \theta_{EC1} + \theta_{EC2} + \theta_{EC3} = -\arctan\left(\frac{\Delta \sin \beta}{L_{core} - \Delta \cos \beta}\right) - \frac{L_{conn}}{L_{core}}\theta_b + \left(1 + \frac{L_{conn}}{L_{core}}\right)\theta_c \tag{11}
\]
\[ \theta_{EB} = \theta_{EB1} + \theta_{EB2} + \theta_{EB3} = \arctan \frac{\Delta \sin \beta}{L_{core}} - \Delta \cos \beta + \left(1 + \frac{L_{conn}}{L_{core}}\right) \theta_c - \frac{L_{conn}}{L_{core}} \theta_c \] (12)

where \( L_{core} \) is the length of the BRB’s plastic core.

The estimates given by Equation (11) and (12) generally agree well with the test results measured by pairs of displacement sensors for the lower western BRB in each braced specimen (Figure 21). Linear regression of the test results shows that the column-side end rotations are only a small fraction of the inter-story drift ratio, \( IDR \), whereas the beam-side end rotations are slightly larger than \( IDR \). Such level of end rotations is only slightly larger than those reported in the past literature for commonly-used inverted-V bracing [44].

Figure 21. End rotations of lower western BRB at: (a) column-side end and (b) beam-side end.

4 Conclusions

Although currently prohibited by steel structure design codes for seismic applications, double-K bracing is a promising system for buckling restrained braced reinforced concrete frames. It can simplify the design of the BRB-to-concrete connection and help improve the seismic performance of these connections.

Three RC frame subassemblies were subjected to cyclic loading, two of which were braced by BRBs in double-K configuration and one was a bare frame for comparison. The Menegotto-Pinto model with isotropic hardening was modified to simulate the BRB hysteresis so that the forces on the gusset connections could be explicitly evaluated in the statically undetermined specimens. The test results confirmed the reliable seismic performance of double-K braced RC frames by providing the following findings.

(1) The RC frames sustained quite similar flexural deformation and damage patterns no matter if it is braced or not, given that the story drift is the same. The unbalanced force of BRBs on the frames had negligible effect on the behavior of the RC frames. Since the maximum story drift of a braced frame with BRBs is always much smaller than that of a bare frame under the same earthquake excitation, the benefit of using BRBs to reduce the seismic damage is guaranteed.

(2) The BRB-to-concrete connections were primarily subjected to shear force, which could be well
withstood by either embedded studs or post-installed chemical anchors. The tension force on the connection interface was negligibly small.

(3) The BRBs behaved well in the double-K configuration to develop full and stable hysteresis. In particular, the end rotations of the BRBs in the present test were similar in magnitude to those in commonly-used inverted-V bracing.

Although the BRBs in a double-K configuration are usually smaller than in other configurations and thus may sustain higher levels of plastic strain in the cores, it would not bring about difficulties for practical design because the currently available BRBs usually exhibit superior low-cycle fatigue performance.

Acknowledgements

The experimental investigation is sponsored by the National Natural Science Foundation of China (No.51308514 and No. 51478441). The financial support is highly appreciated.

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