Numerical Assessment of Seismic Performance of Continuously Buckling Restrained Braced RC Frames

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SUMMARY:
The seismic performance of continuously buckling restrained braced frame system for reinforced concrete buildings is investigated through nonlinear dynamic analysis. The proposed system is featured by its connection details for the buckling restrained braces, which separate the vertical and horizontal components of the force imposed by the braces to be resisted by essentially independent structural parts, thus make the connection behaviour easier to estimate and control. The seismic response of an example building of the continuously buckling restrained braced frame system is evaluated to demonstrate the effectiveness of the buckling restrained braced system in reducing the dynamic responses of the building and thus in reducing the damage to the concrete structure. The demand, especially the horizontal force demand, for the brace connection is analyzed to show the significant influence of higher modes in a yielding system. On the other hand, practical mechanical models are built for the BRB connection and is adopted in the nonlinear dynamic analysis of the whole structure to verify the insignificance of its influence on the global dynamic response of the building.

Keywords: buckling restrained braced frame, reinforced concrete, connection, nonlinear dynamic analysis

1. INTRODUCTION

The applications of buckling restrained braces (BRBs) in reinforced concrete (RC) structures are still quite limited, and are mostly in the retrofit of existing buildings. In recent years, efforts have been devoted to applying BRBs in new concrete constructions. Particularly, some methods of fastening BRBs to concrete components were proposed and tested (Ogawa et al, 2004; Gu et al, 2011 among others). All such studies reported good performance of BRBs in terms of dissipating energy and enhancing seismic performance of RC frames. On the other hand, however, searching for appropriate methods of applying BRBs in RC frame buildings remains an important issue for both retrofit and new constructions. The existence of a gusset plate for BRBs may subject the surrounding concrete members to very complicated, and sometimes unfavorable, load conditions. The damage of the surrounding concrete parts may, in turn, impair the performance, especially the stiffness, of the BRB connection. In addition, the BRBs, which carry significant axial force, are likely to change the force distribution among the surrounding frame members. For example, considerable tensile force may be transmitted by the BRBs to the reinforced concrete columns in the braced span, which may even exceed the gravity load and subject the columns to tension. As an attempt to address some of the above issues, a new braced system, namely the ‘continuously buckling restrained braced frame (BRBF)’ system, together with corresponding details of BRB connection, were proposed for RC buildings by the authors (Qu et al, 2011). It is featured by its special configuration of the braces (see Figure 2.1, for example), which allows for separation of tensile and shear action on the BRB connection. The two BRBs in neighboring stories share the same gusset plate, which is attached to the side surface of the beam-to-column joint. The beam in the braced span is eliminated to give way for the BRBs (see Figure 3.3(a)). In such a way, the horizontal components of the BRB force are expected to cancel each other out so that the tensile strength demand for the gusset plate-to-concrete connection can be minimized. At the same time, the impairment of the slenderness of the column because of the gusset plate is also minimized. This might reduce the risk of unfavorable shear failure of captive columns.

To verify the seismic performance of the newly proposed system, numerical analysis is conducted on an example 12-story continuously BRBF building. In particular, the global dynamic response of the system and the influence of the local behavior of BRB connection are of interest.

2. EXAMPLE STRUCTURE AND NUMERICAL MODEL

A 12-story, 3-span continuously buckling restrained braced RC plane frame structure (Fig. 2.1) is analysed in ABAQUS 6.8. Cross sectional properties of the reinforced concrete components of the structure is listed in Table 2.1. The total weight of the plane structure is 26509 kN and the axial force at the bottom of the middle and side columns of the structure due to gravity is 8380 kN and 5124 kN, about 19.3% and 11.8% of their respective nominal axial strength, $N_0=f_c'bh$, respectively. f_c' is the concrete compressive strength and b and h are the width and height of the cross section, respectively. The beams are modelled as T-section beams with wide flange representing the contribution of the cast-in-site floor slabs. Rigid zones are adopted for the RC beam-column joints. The width of the equivalent beam flange and the rigid zone is determined in accordance with the AIJ standard for design of concrete structures (AIJ, 2010).

Table 2.1. Cross sectional properties of reinforced concrete columns and beams

Floor	f_c'	Beam section			Column section		
	(MPa)	b (mm)	h (mm)	ρ_s (%)[1]	b (mm)	h (mm)	ρ_s (%)[2]
11,12	30	600	900	0.92	850	850	1.76
9,10	36	600	900	1.11	900	900	1.89
7,8	36	600	900	1.11	950	950	2.02
4,5,6	42	600	900	1.11	950	950	2.02
1,2,3	48	600	1000	1.00	950	950	2.38
Foundation	42	600	2500	LE[3]			

[1] $\rho_s = A_s/b/d$, where A_s is the area of longitudinal rebar in the tensile side; d is the effective section depth;
[2] $\rho_s = A_s/b/h$, where A_s is the area of longitudinal reinforcement in the whole section;
[3] Foundation beam is assumed linear elastic through the analysis.

As mentioned above, the beams in the braced span are eliminated to give way to the BRBs in a continuously BRBF system. However, elastic spring elements (dashed lines in Fig. 2.1) are used in the positions of these beams in the numerical model to tie up the left and right span of the frame, representing the axial stiffness of floor slabs and secondary beams out of the plane of the analyzed plane frame. These spring elements transmit only axial force along their axis.

The RC beams and columns are modelled by fiber-section beam elements with user-defined uniaxial hysteresis for the concrete and the reinforcement fiber. Shear failure is not modelled. BRBs are modelled by truss elements with elastic-perfectly plastic hysteresis. It is assumed that the yield strength of all the six BRBs in the model is identical. The yield strength of the BRB is such determined that the sum of the vertical components of the yield strength of all BRBs would not exceed the gravity compressive force at the bottom of the columns in the braced span. In other words, the columns would not be subjected to tensile force at the bottom all the BRBs are yielded and are imposing tensile force on it at the same time. As a result, 2100 kN yield strength is determined for the BRBs. The initial stiffness of the BRB elements is then determined as an equivalence of the two elastic portions at both ends and a plastic portion in the middle of a BRB with assumptions on the length and cross section area of the elastic portions. Mass proportional damping is assumed and 2% damping ratio is assigned for the 1st mode.

Three ground motions recorded in Japan are selected to assess the seismic performance of the above structure. Their basic properties are listed in Table 2.2, where D is the Joyner-Boore distance; T_S is the significant duration. In the analysis, all the records are normalized by the peak ground velocity (PGV) to Level II (i.e., PGV = 50 cm/s) intensity of the seismic design practice in Japan. Takatori-NS and JMA Kobe-NS records represent typical near-fault ground motions with significant medium- or long-period component. The period of the JMA Kobe-NS record corresponding to the peak spectral velocity almost coincides with the elastic fundamental period of the analysed structure.

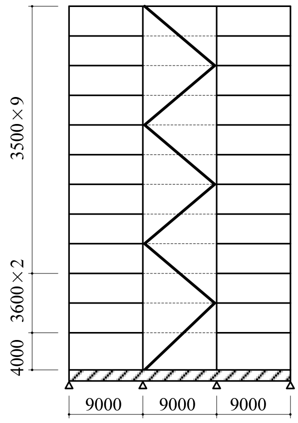

Figure 2.1. Elevation of the analyzed 12-story building

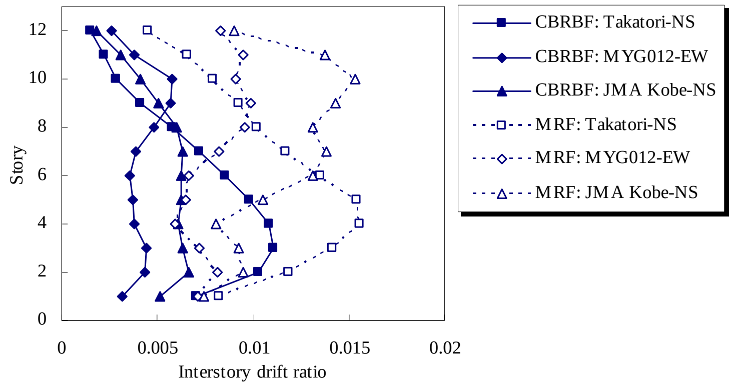

Figure 3.1. Maximum interstory drift of moment-resisting frame (MRF) and continuously buckling restrained braced frame

Table 2.2. Designation and basic properties of recorded ground motions used in this study

Designation	Owner	Earthquake	M_w	D (km)	T_S (s)
Takatori-NS	CUE	Kobe, 1995	6.9	1.46	11.3
JMA Kobe-NS	JMA	Kobe, 1995	6.9	0.94	8.4
MYG012-EW	NIED	Tohoku, 2011	9.0	168.18*	103.1

* Epicenter distance.

3. GLOBAL SEISMIC RESPONSE

By adopting the continuous bracing system, the maximum interstory drift of the example building is significantly reduced for all the three ground motion excitations, as is evident from Fig. 3.1. Regardless the variability of the characteristics of the input ground motions, more than 90% of the hysteretic energy dissipation is concentrated in the BRBs of the continuously braced frame, while on the other hand, the concrete beams and columns in the bare moment-resisting frame have to dissipate a considerable amount of energy by means of inelastic deformation, which at the same time indicates damage to these components (Fig. 3.2). The results show that the hysteretic energy dissipated by the concrete components of the bare moment-resisting frame building is 7.5, 26.6 and 4.6 times that dissipated by those of the continuously BRBF building for the ground motion of Takatori-NS, JMA Kobe-NS and MYG012-EW, respectively.

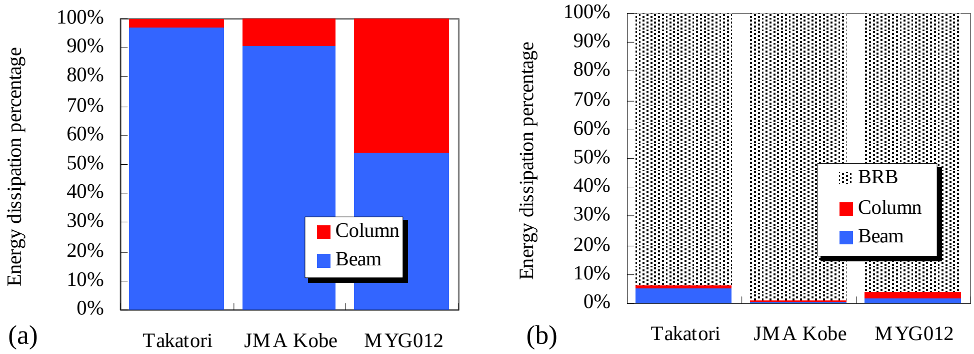

Figure 3.2. Hysteretic energy dissipation distributions among different components: (a) moment resisting frame (MRF) and (b) continuously buckling restrained braced frame

In the proposed BRB connection, the horizontal and vertical components of the force acting on a BRB connection are resisted by essentially independent members, that is, RC corbels for the vertical component and prestressing bolts for the horizontal one (see Fig. 3.3(a)). Ideally, the lower and upper BRBs connecting to the same gusset plate may yield at the same time and in opposite directions (that is, one in tension and the other in compression) so that the vertical components of their force may subject the gusset plate to very large vertical force, while the horizontal ones may counter-act each other and impose very small force on the anchor bolts.

As an example, Fig. 3.3 depicts the time history of the two components of the force acting on the BRB connection at the 6th floor level during the JMA Kobe-NS motion. As expected, the vertical force on a connection readily reaches its peak when the two BRBs yield in opposite directions (Fig. 3.3(b)). As a result, the strength demand for the corbels is easy to estimate, which is simply the sum of the vertical components of the BRB yield strength.

On the other hand, however, the maximum horizontal force on the connection may become comparable in peak magnitude with the yield strength of a single BRB (Fig. 3.3(c)). Such significant horizontal force should not have occurred if the structure vibrates in only its 1st mode. It can be well explained by the fact that the plasticity of the structure during the ground motion is effective in suppressing the force corresponding to the 1st mode vibration while it has much less influence on the force corresponding to higher modes of vibration. This has been observed and reported by previous researches, especially in estimating the strength demand for slender shear walls (Eibl and Keintzel, 1988; Rodriguez et al, 2002; Priestley et al, 2007; Panagiotou and Restrepo, 2009)

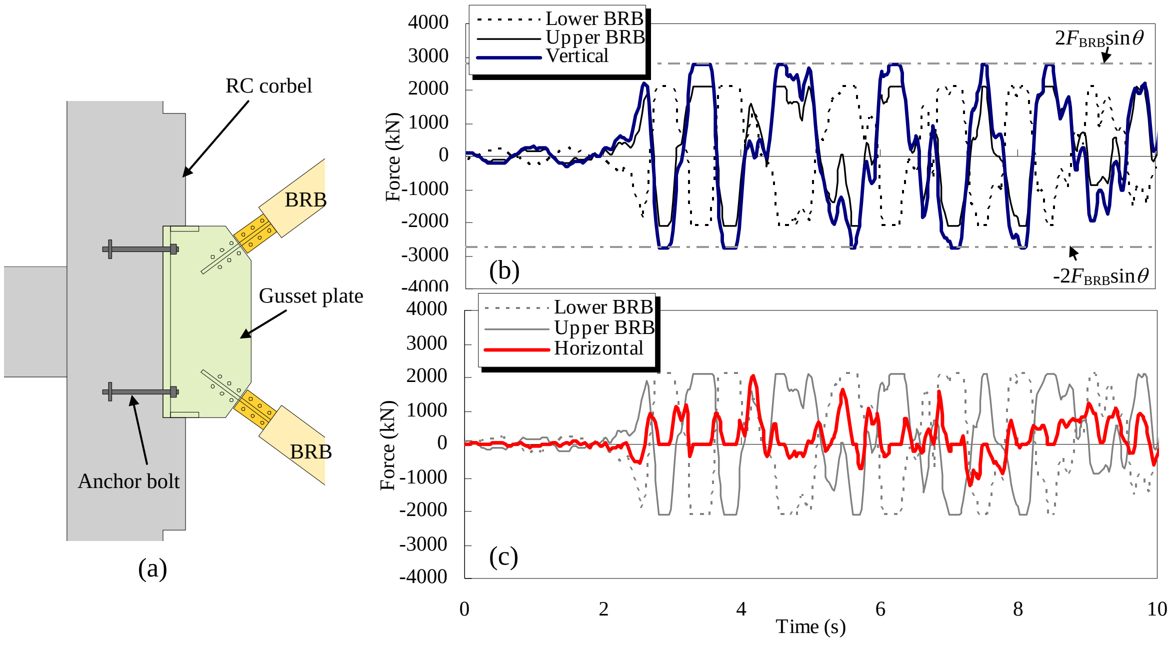

Figure 3.3. Force in the BRB connection at 6th floor under JMA Kobe-NS: (a) connection detail, (b) vertical component and (c) horizontal component.

The maximum horizontal force acting on the BRB connections along the height of the structure subjected to the three ground motions is depicted in Fig. 3.4 in terms of tensile force ratio, which is the ratio of the maximum horizontal force at a connection to the greater of the horizontal components of the yield strength of the two BRBs connected to it. Results obtained from linear-elastic and nonlinear models are compared. It is not difficult to understand that the horizontal force in nonlinear cases is much greater than that in linear elastic cases. In the following discussions, upper bound force (the bold dash line in Fig. 3.4, which is the sum of the horizontal components of the BRB yield strength) is used as a basis of proportioning the prestressing bolts.

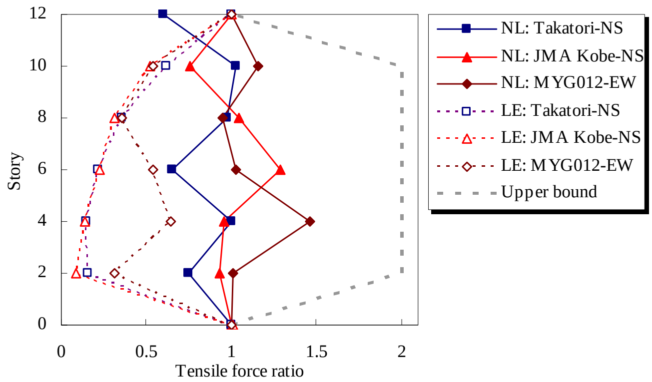

Figure 3.4. Maximum horizontal force in BRB connections

4. INFLUENCE OF NONLINEAR BEHAVIOR OF BRB CONNECTION

As a more realistic model for the BRB connections, user-defined dimensionless bi-axial spring elements, as illustrated in Fig. 4.1(a), are used to connect the BRB element and the rigid zone of the concrete joint. The two nodes of the spring element, Node 1 and Node 2, share the same coordinates, making the spring element zero-length. Each spring element consists of two uniaxial springs in orthogonal directions, one representing the corbels above and below the gusset plate (i.e., the corbel spring in Fig. 4.1(b)) and the other representing the group of prestressing bolts (i.e., the bolt spring in Fig. 4.1(c)).

Quadric ascending skeleton curve is assumed for the corbel spring, which is analogous to the stress-strain relationship of concrete in compression (see Fig. 4.1(b)). In the pair of corbels around a gusset plate, each corbel only carries one-way force. As a result, the resistance would not recover during reloading until the residual deformation of the previous loading cycle is exceeded.

The unbond anchor bolts embedded in the concrete joint are prestressed to increase the stiffness of the connection. As a result, the bolt exhibits different stiffness before and after the separation of the gusset plate and the underneath concrete surface. This behaviour is described by a trilinear skeleton curve as shown in Fig. 4.1(c), which features a separation and a yield point.

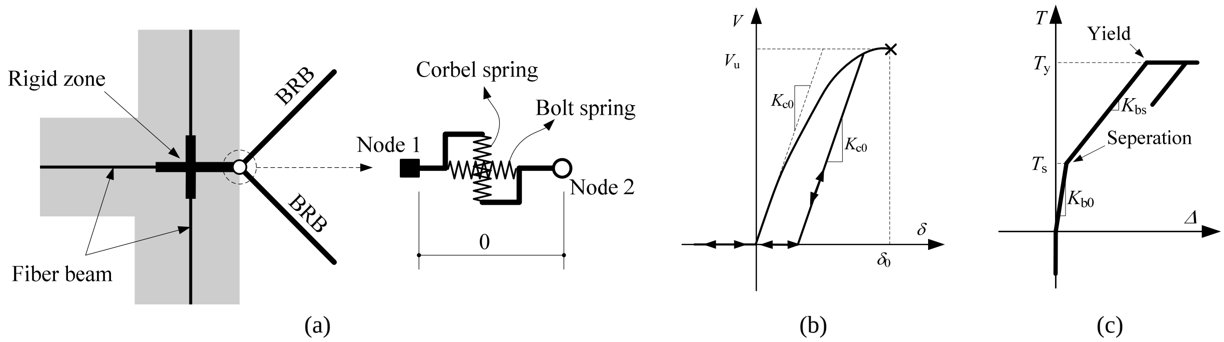

Figure 4.1. Modeling of BRB connections: (a) bi-axial spring element, (b) hysteresis of a single corbel and (c) force-displacement relationship of bolt spring

The required shear strength, V_{dem}, for the concrete corbels is readily determined as the sum of the vertical components of the two BRB yield strength. On the other hand, the tensile strength demand, T_{dem}, for the prestressed bolts, as suggested above, is safely taken as the sum of the horizontal components of the two BRB yield strength. Based on these strength demand, RC corbels with three sets of properties, referred to as Corbel A, B and C, and prestressed bolts with two sets of properties, referred to as, Bolt A and B, are assumed for the example building. Their skeleton curves are compared in Fig. 4.2. The difference between Corbel A and C is in their hysteresis. Corbel A adopts a plastic model with residual deformation as shown in Fig. 4.1(b) while Corbel C adopts a nonlinear elastic model. The properties of these corbel and bolt models are listed in Table 4.1. Different combinations of these models yield five analysis cases, which are also listed in Table 4.1.

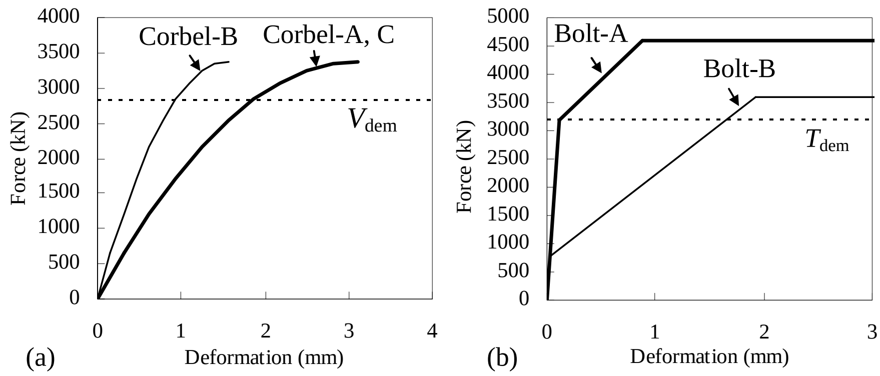

Figure 4.2. Models of springs in the BRB connection: (a) corbel models and (b) prestressing bolt models

Table 4.1. Analysis cases and connection properties

	Corbel	K_{c0} kN/mm	V_u kN	Nonlinearity	Bolt	K_{b0} kN/mm	K_{bs} kN/mm	T_s kN	T_y kN
1	Rigid	-	-	-	A	26197	1868	3178	4592
2	Rigid	-	-	-	B	24137	1481	794	3592
3	A	2167	3378	Plastic	A	26197	1868	3178	4592
4	B	4334	3378	Plastic	A	26197	1868	3178	4592
5	C	2167	3378	Elastic	A	26197	1868	3178	4592

The maximum connection deformation in the bolt spring and the corbel spring in Case 1 to Case 4 is depicted in Fig. 4.3. The separation of prestressed bolts substantially increases the maximum deformation in the bolts (Case 2 versus Case 1). The distribution of this deformation varies from record to record, indicating the dynamic nature of the tensile force in the BRB connections, which has already been examined above. For the concrete corbels, the maximum deformation is proportional to the stiffness and its distribution is almost independent of ground motion records. A bold gray line is also plotted in Fig. 4.3(b) demonstrating the estimated maximum corbel deformation at V_{dem} for Case 3. It matches well with the dynamic analysis results of the building in the medium- and lower-stories, where the BRBs are well yielded. The maximum corbel deformation in Case 5 is almost the same as that in Case 3 and is thus not depicted in Fig. 4.3.

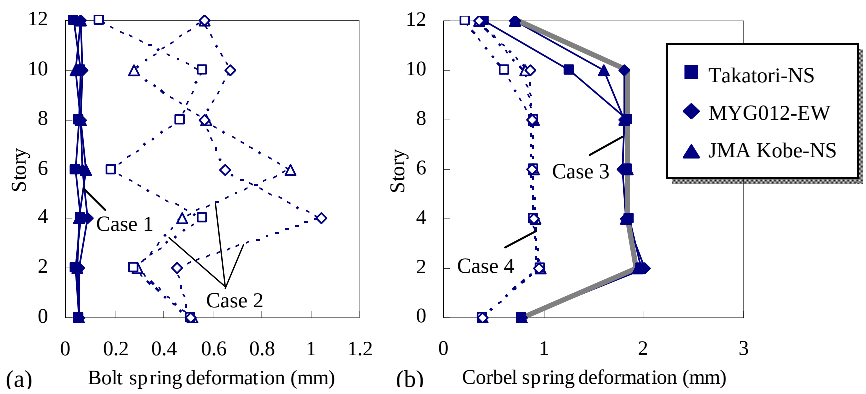

Figure 4.3. Maximum bolt spring deformations

The maximum inter-story drift ratios (IDR) of both the moment-resisting frame (MRF) and the continuously BRBFs with various BRB connection properties are compared in Fig. 4.4(a). Although the flexibility of the connection, especially that of the concrete corbel (e.g., in Case 3, 4 and 5), would somewhat increase the maximum IDR, this increase is generally insignificant as compared to the reduction of maximum IDR by adopting the bracing system. The absolute difference between the maximum IDR with flexible BRB connections (Case 1~5) and those with rigid ones is depicted in Fig. 4.4(b). The influence of the prestressing bolt seems negligible even if the gusset plate separates with the concrete surface (Case 2). Most of the increase in IDR may be attributed to the deformation of the concrete corbels. The difference in IDR is practically proportional to the stiffness of the corbel (Case 3 versus Case 4). The residual deformation also has an effect but not as significant as that of the stiffness (Case 3 versus Case 5). Similar observation can be made upon the energy dissipation loss ratio in BRB (Fig. 4.4(c)), which is defined herein as the ratio of the difference in energy dissipation of a BRB in continuously BRBFs with flexible and with rigid BRB connections to the total energy dissipation of the BRBs in continuously BRBF with rigid BRB connections. It is also observed that the maximum difference in IDR is generally less than the sum of the maximum corbel deformation at V_{dem} at both ends of a brace, which is indicated by the grey line in Fig. 4.4(b).

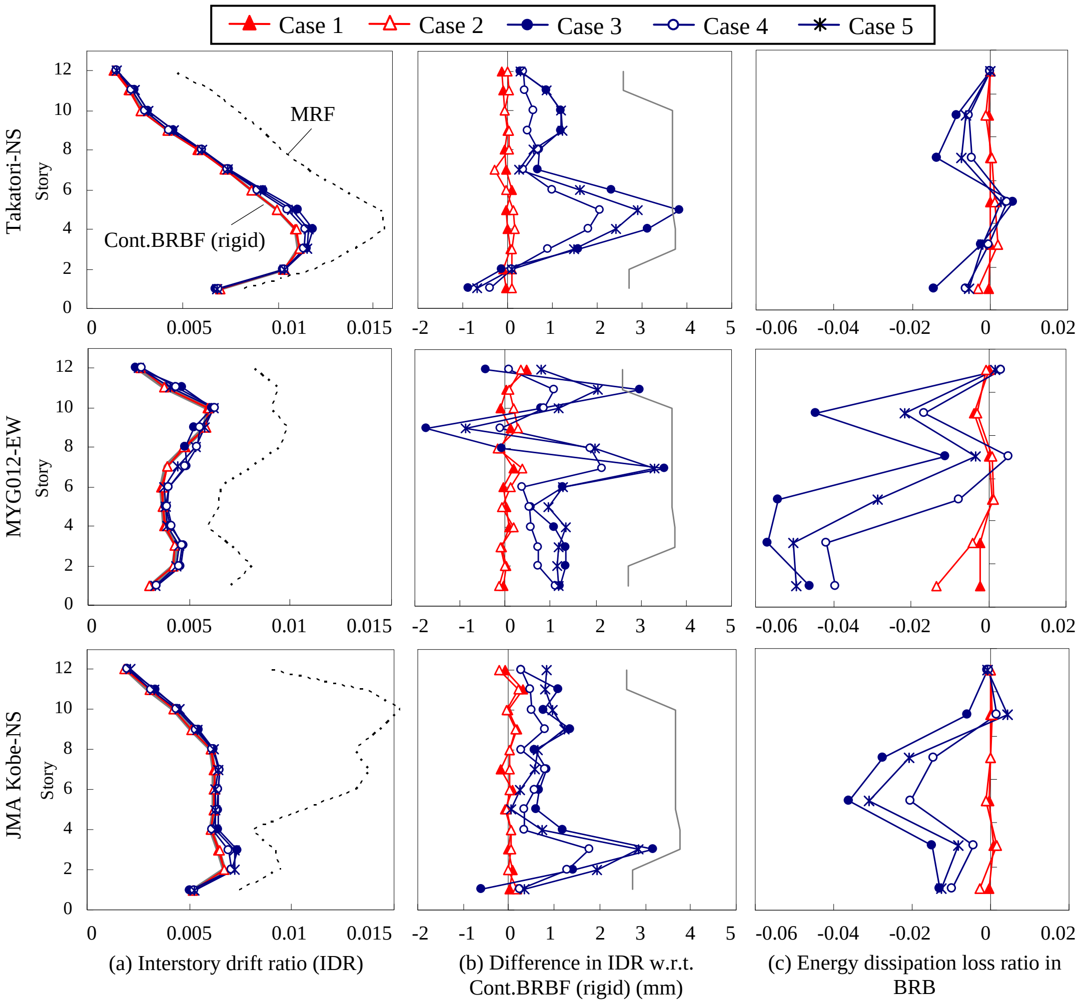

Figure 4.4. Deformation and energy dissipation results

5. CONCLUSIONS

Together with the details for fastening BRBs to concrete members, the continuously braced system investigated in this paper, , provides an alternative of applying BRBs to enhance the seismic performance of reinforced concrete structures. The effectiveness of the bracing system is demonstrated through the study of an example building, in which the BRBs concentrated more than 90% of the hysteretic energy dissipation and thus greatly reduce the damage to the concrete part, regardless of the different characteristics of the selected ground motions.

With the connection models, the influence of the nonlinear behavior of the connection to the global seismic response of the system is evaluated. The results show that the global seismic response of the building is insensitive to the flexibility of the prestressing bolt, primarily because the great force in the bolt connection rises from the higher-mode effect and generally does not coincide with the peak of the interstory deformation. On the other hand, the flexibility of the concrete corbel may lead to an increase in the interstory drift of the building. This effect should be taken into account in evaluating the seismic performance of the buckling restrained braced frames.

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