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SEISMIC RESPONSE OF HYBRID DUCTILE-ROCKING BUCKLING RESTRAINED BRACED FRAMES WITH CAST STEEL SUPPLEMENTAL ENERGY DISSIPATION ELEMENTS

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Abstract

This paper presents numerical results on the seismic response of the design of a hybrid ductile-rocking (HDR) seismic force-resisting system. The HDR system consists of a code-designed buckling-restrained braced (BRB) frame with specially designed column-foundation connections that permit limited base rocking. The intent of the system is to economically improve the performance of BRB frames by reducing large concentrations of drift and damage at a few stories and reducing excessive residual deformations. The HDR system maintains the highly desirable limit on forces and accelerations that are typical of the response of BRB frames, in contrast to controlled rocking (CR) structures, where frame members are designed to remain elastic but can be overloaded by higher mode demands. This paper outlines the basic mechanics of the proposed HDR system and then presents an example design of a 6-storey BRB frame located in Los Angeles. The system is detailed with a column-foundation connection that includes a lockup device that ensures that the codified lateral resistance of the BRB frame can be achieved after the system locks up at a predetermined base rotation. Supplemental energy dissipation elements comprised of ductile cast steel yielding connectors are also included at the base of the structure to provide hysteretic damping to the rocking system. Results indicate that a significant reduction in drift concentrations, residual deformations, and cyclic damage can be achieved with the HDR system when compared to conventional BRB frames, while still controlling storey shear forces and accelerations as effectively as conventional BRB frames with a fixed base. This suggests that the HDR system offers an economical method for enhancing the performance of BRB frame structures.

Keywords: buckling-restrained brace; rocking; residual; damage concentration; cast steel

1. Introduction

Buckling restrained braced (BRB) frames are popular systems in high seismic regions in part due to their large force reduction factor (R=8 [1]) and low post-yield stiffness, which allow for low capacity design forces and well-controlled storey accelerations [2,3]. The low post-yield stiffness, however, has a tendency to increase drift concentrations and cyclic damage at one or only a few stories over the height of the structure, as well as cause increased residual deformations [3]. This concentration of cyclic damage and residual deformations can substantially increase the cost of repairs or even lead to the building's condemnation following a major seismic event.

Controlled rocking (CR) structures have been developed as higher-performance systems that address the above-mentioned drawbacks by concentrating the inelastic response at the base of the structure through a flexurally activated rocking mechanism while the structure above the base is designed to remain elastic. This is achieved by allowing column uplift in conjunction with a restoring force from gravity loads and/or post-tensioning tendons and providing energy dissipation elements that are activated by the rocking motion at the base [4, 5, 6, 7]. While the beneficial intent of CR structures is to withstand an earthquake while sustaining minimal cyclic damage and permanent deformations, they are also associated with increased cost due to issues related to their dynamic response and practical implementation. CR structures do not completely limit the seismic shear force at the base, and the overturning moments along the height of the structure, and thus large member sizes can be required in order to elastically resist the forces associated with the higher mode response [6, 7]. Large higher mode effects in elastic rocking structures are illustrated in Fig. 1, which shows the results of nonlinear time-history analyses on a six-storey base-rocking frame that was designed with ductile BRBs, as compared to a similar frame with elastic braces (this study is described in further detail in Section 4.2). The frame with ductile braces achieved much lower and more controlled storey forces (Fig. 1a) and accelerations (Fig. 1b), as compared to the elastic rocking frame. These results demonstrate why distributed ductile braces are beneficial with regard to forces and accelerations even in frames that are designed as base-rocking systems. While [8, 9] developed and validated effective techniques for mitigating these higher modes effects in CR structures while still designing members to behave elastically, such as incorporating multiple rocking structures and/or self-centering braces in the first storey, these options are associated with increased design and detailing complexity and cost. Additionally, CR systems with such advanced higher mode mitigation strategies still require full validation before they can be incorporated into building codes and conveniently used.

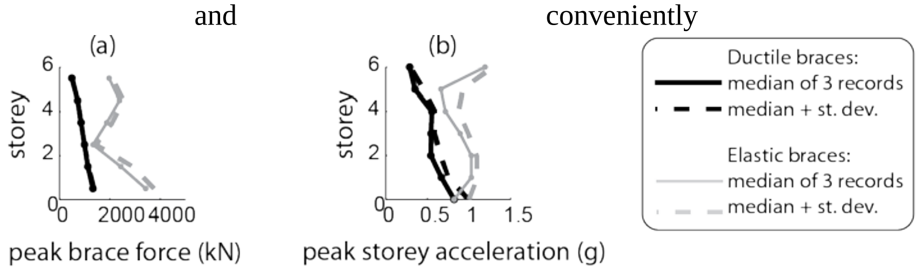

Figure 1: Response of ductile and elastic rocking frames to three records scaled to DBE (a) peak brace forces; (b) peak storey accelerations (analysis details are provided in Section 4.2)

This paper presents an example design of a hybrid ductile-rocking (HDR) system, which seeks to reduce the disadvantages of BRB frames by incorporating aspects of CR structures, while still being similar to code-compliant conventional systems with respect to economy in design and detailing. The HDR system also includes the beneficial force and acceleration control associated with BRB frames. The proposed HDR system consists of a code-designed BRB frame that features specially designed column-foundation connections that permit limited column uplift. After a predetermined base rotation, a lockup device limits the column uplift and thus ensures the code-prescribed system resistance of the BRB frame is achieved. Supplemental energy dissipation elements provide hysteretic damping and reduce the demand on the base rocking connection.

A column-foundation rocking detail is developed that includes an innovative cast steel hysteretic fuse that is a modified version of the yielding connection (YC) cast steel energy dissipating system for concentrically braced frames [10]. A numerical model of this frame is developed and calibrated based on previous tests of BRBs and YC devices. Results from nonlinear time-history analyses are presented that confirm the large higher mode demands on rocking elastic frames, and demonstrate the improved performance and reduced damage associated with the hybrid ductile-rocking BRB frame as compared to the fixed base frame.

2. Overview of HDR frame mechanics

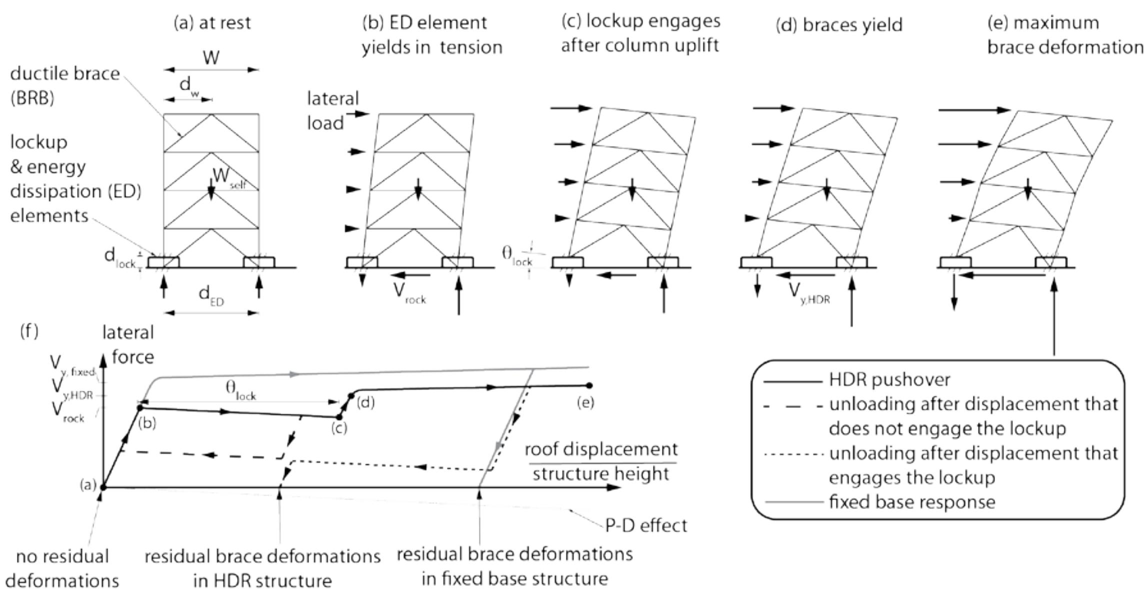

Figure 2: Overview of HDR mechanics: (a) schematic frame at rest; (b) energy dissipation element yields; (c) lockup engages after column uplift; (d) braces yield; (e) maximum brace deformation; (f) schematic force-deformation plot for HDR system

Fig. 2 gives a brief overview of the mechanics of the HDR system when lateral load is applied. Beginning at rest (Fig. 2a), the frame deforms similarly to a fixed base frame until the energy dissipation element on one side yields in tension and the base shear corresponds to $V=V_{rock}$ (Fig. 2b), after which the frame rocks on its foundation up to a predetermined amount, θ_{lock} (Fig. 2c). Subsequently, the lockup device limits the vertical deformation of the column and the braces eventually begin yielding when $V=V_{y,HDR}$ (Fig. 2d) and then proceed to deform inelastically (Fig. 2e). Fig. 2f shows a force-deformation plot of this behavior, as compared to the corresponding fixed base frame that yields at $V=V_{y,fixed}$. It can also be seen in Fig. 2f that the HDR frame sustains zero residual deformations when unloaded from a peak deformation that does not engage the lockup. If unloaded from a peak deformation that does engage the lockup, the HDR residual deformations are still less than those associated with the fixed base frame. It should be noted that under dynamic loading, higher mode effects can also cause brace yielding even if the lockup does not engage. A detailed description of HDR frame mechanics can be found in [11]. For an HDR system with a self-weight, W_{self}, located a distance d_w from the rocking toe, energy dissipation elements located on the side of each frame a distance d_{ED} apart (as shown in Fig. 2a), and an energy dissipation parameter, β (as defined for the rocking joint in [6, 11]), the energy dissipation element strength can be defined as (see [11]):

$$F_{ED} = \frac{\beta W_{self} d_W}{d_{ED}(2-\beta)} \quad (1)$$

3. Overview of building design

3.1 Design of 6-storey fixed base frame as basis for HDR system

The superstructure of the frame that is discussed in this paper corresponds to the 6-storey reference frame building design from [11]. Fig. 3 shows the floor plan, frame elements, and the gravity and seismic load data. The frame was designed for Los Angeles, CA, according to ASCE 7-10 [1] using a response spectrum analysis (RSA) procedure, and the provisions of AISC 360-10 [12] and AISC 341-10 [13] for the detailing of the members. Capacity design forces were determined assuming that the braces had a yield stress of 248 MPa, a strain hardening adjustment factor of 1.5, and a compression force adjustment factor of 1.1. The maximum inelastic drift calculated using the code-based design procedure was 1.1%, a value that was below the code-prescribed limit of 2.0%.

3.2 Selection of HDR base rocking parameters

This 6-storey frame was modified to include the hybrid ductile-rocking concept described above. Values of allowable rocking before lockup, θ_{lock}, and supplemental energy dissipation element strength, F_{ED}, were chosen based on the results of analyses on HDR frames presented in [11]. A θ_{lock} value of 1.0% was chosen. This value corresponds to 91.4 mm of allowable uplift, after which the lockup prevents further column uplift. A β value of 1.0 was chosen, and the corresponding energy dissipation element strength, F_{ED} was determined to be 1200 kN from Eq. 1. The parametric study in [11] also showed that for this 6-storey frame, a β value of 1.0 allowed for a large supplemental energy dissipation strength without resulting in a lateral rocking strength that was greater than the lateral strength of the fixed base BRB frame. As well, this is the maximum possible β value before residual base rotations are possible assuming an elasto-plastic energy dissipation element response. In reality, some residual base rotations are possible due to strain hardening and post-yield strengthening in the energy dissipation elements, but these are not expected to be large, as has been shown to be the case for self-centering systems with β values slightly larger than 1.0 [see 14].

The main requirements of the HDR column-foundation detail are to accommodate column uplift while resisting seismic base shear, resist foundation tension after lockup, accommodate the supplemental energy dissipation elements, and accommodate the brace connection. Fig. 4 shows an overview of the proposed detail that incorporated cast steel yielding devices as the energy dissipating element. Fig. 4a shows an isometric view of the completed detail and highlights the primary features of the concept. Fig. 4b shows the column assembly which includes two cast steel yielding devices that are bolted to the column web. Gusset plates with slotted holes are welded to the column flanges and allow for foundation tension to be resisted after a predetermined amount of uplift that is defined by the length of the slotted holes (in this case 91.4 mm) is achieved. One of the gusset plates also accommodates the BRB connection, which is detailed as a true pin connection. Finally, Fig. 4c shows the base plate assembly which includes the energy dissipation reaction collar and lockup reaction plates, as well as holes for foundation anchor rods. All welds were designed as complete joint penetration welds.

The energy dissipation elements were designed based on the case steel yielding connector (YC) concentric braced frame system [10]. The YC connector is a commercially available product line provided by Cast Connex, and features large ductility and a characteristic post-yield stiffening and strengthening effect that offers performance improvements for braced frames over conventional buckling restrained braces [15]. The cast YC connectors are relatively compact, and through a redesign that was carried out by Cast Connex, these connectors were adapted for use in the HDR system. Fig. 4d shows an isometric view of one of the modified units, and notes important features including the ductile triangular fingers, the elastic backing plate that connects the fingers to the column web, and holes for inserting bolts in the connection. The elastic backing plate was designed so that all the inelastic demand in the connection occurred in the triangular fingers. The thickness of the plate was determined by considering the combined axial and bending moments associated with the over strength yield force associated with each finger. Ten 1-inch bolts were chosen to fasten the energy dissipation elements to the column webs. The bolts were designed to be through-bolts across both energy dissipation units and the column webs. The bolts were checked for combined shear and tension as per [12].

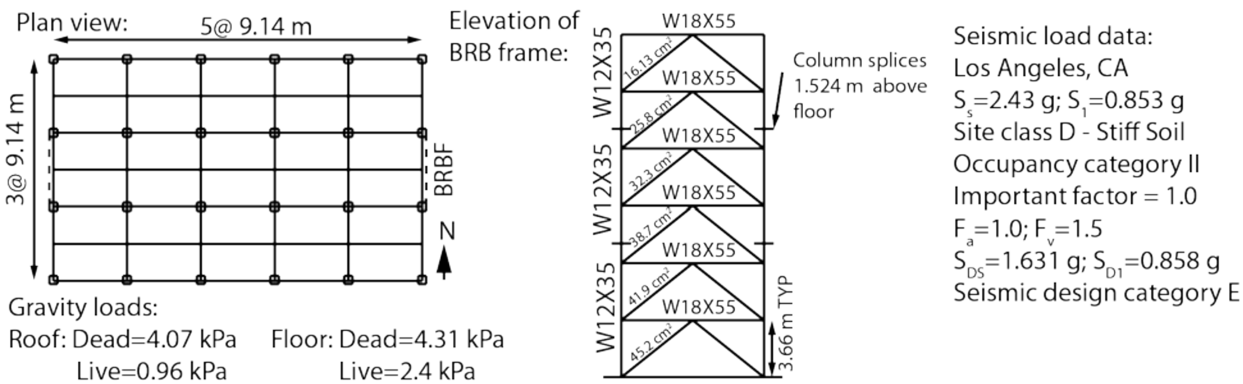

Figure 3: Overview of example building design

The HDR detail was capacity designed to resist shear, compression, and tension forces. Additionally, this detail included special consideration for the forces applied by the supplemental energy dissipation elements. The design seismic shear force for each column-foundation connection was determined to be 1850 kN using capacity design principles, considering full compressive and tensile overstrength in the BRBs. Earthquake loads governed the compression and tension forces, and were determined using the load combinations in [1]. The maximum tension force in each lockup, 4540 kN, was lessened by the nominal strength of the energy dissipation elements, since at full lockup the yielded energy dissipation elements pull down on the column. The design foundation compression was determined to be 8230 kN at each connection. The nominal energy dissipation strength was determined using the equations from [10] to be 1200 kN. This value was multiplied by an overstrength factor of 2.0, giving a value of 2400 kN, when determining maximum capacity design forces on the rest of the connection in order to conservatively take into consideration the overstrength due to strain hardening and geometric nonlinearity that is a feature of the cast YC connectors [10].

The gusset plates were designed to accommodate the vertical lockup forces and horizontal shear forces. One of the gusset plates includes a pin detail to connect to the BRB. The gusset plates were welded to the column flanges, but were not connected to the base plate in order to permit column uplift. The primary loads resisted by the gusset plate welds are the lockup tension after uplift and the maximum brace compression force. The gusset plate was checked so that there was no yielding under the combined effects of shear and bending. For the lockup, ten 1-1/8 inch A-490 bolts were chosen. The gusset plates were checked for bolt tear-out, bearing failure, and shear block failure as per [12]. The slotted holes were designed to carry the full base shear.

3.3 Column-foundation detail design

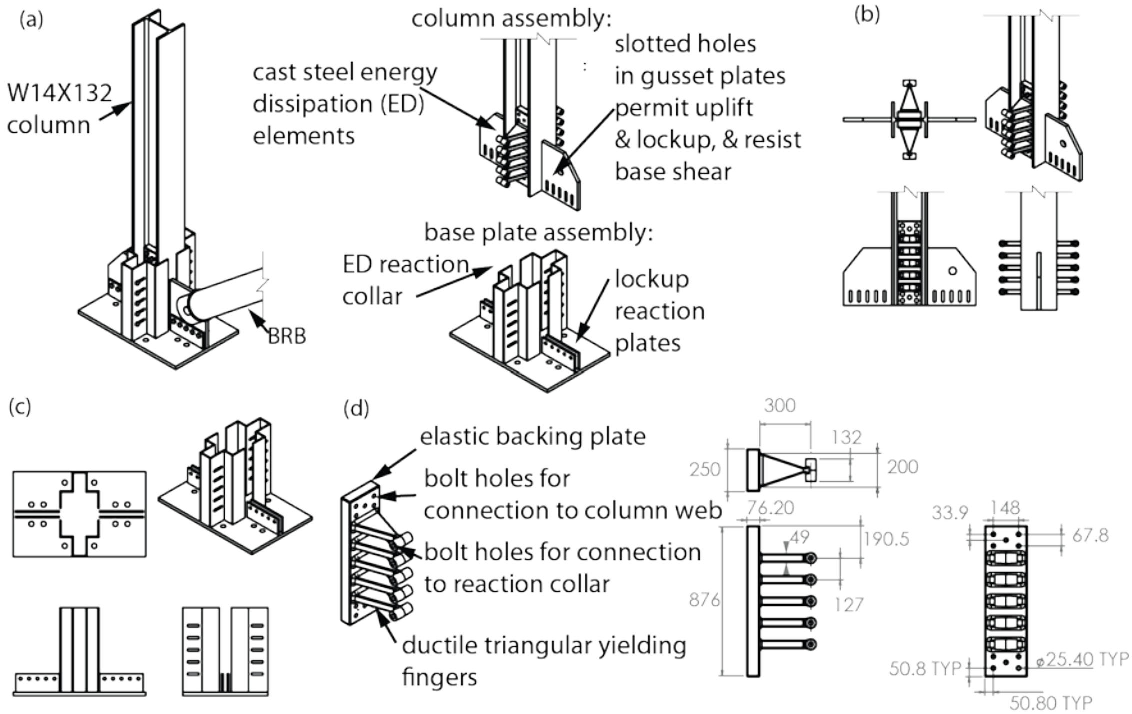

Figure 4: HDR column-foundation detail: (a) overview; (b) column assembly; (c) base plate assembly; (d) cast steel energy dissipation element

The base plate assembly consists of the energy dissipation reaction collar, the lockup reaction plates, and the base plate, as shown in Fig. 4c. The energy dissipation reaction collar was designed to connect the energy dissipation elements to the foundation. The collar consists of a welded plate assembly that is in turn welded to the base plate. Slotted holes allow for the fuses to react without causing excessive tension forces in the yielding fingers due to second order geometric effects [10]. Since the energy dissipation elements act eccentrically to the centroid of this built up section, each half of the reaction collar was checked to ensure that the section did not yield under combined axial forces and bending moments. A 5 mm gap was left between the column and the reaction collar in order to allow for some rotation of the column after uplift. It is noted that this collar could serve as a secondary base shear transfer if the gusset plate bolts were damaged during an earthquake.

The lockup reaction plates were designed to be welded to the base plate on each side of the gusset plate. 1-1/8 inch bolts were specified to fasten through all three plates and ensure that the seismic base shear is transferred perpendicular to the direction of the slotted holes in the gusset plates, and that the lockup tension is transferred vertically. Finally, the base plate was designed to transfer the tension, compression and shear force to the foundation using a series of 50.8 mm diameter anchor rods.

4. Numerical study comparing fixed base BRB frame to HDR frame with cast steel fuses

Two-dimensional numerical models (shown in Fig. 5a) of both the fixed base and HDR 6-storey seismic force-resisting systems (SFRS) were created using the open source earthquake engineering framework OpenSees [16]. The BRBs were modelled using corotational truss elements and the nonlinear Steel02 material. These elements were calibrated to physical tests from [17], as shown in Fig. 5b. This calibration is described in further detail in [11]. The steel beams and columns were modelled using elastic beam-column elements, with columns continuous between floors and full moment connections at column splices, and beams continuous between columns with pin connections at the columns. The P-Δ effect was modelled with rigid truss elements that were pinned at each floor and accounted for second order effects. The leaning column was loaded to represent the gravity loads for half of the tributary area of the entire building, not including the load associated with the columns in the SFRS. The entire dead load and 25% of the live load was applied to the leaning column. The seismic mass assigned to each storey considered the dead load of the structure. This mass was lumped at each learning column connection, except for the portion of the mass that is associated with the tributary area of the SFRS which was assigned to the beam-column connections of the SFRS in order to capture the effects related to the excitation of vertical mass during rocking. This vertical mass modelling choice was shown in [11] to represent an upper bound on the amplification of forces due to the dynamic response of vertical mass in HDR structures.

For the fixed base frame, the column-foundation connections were modelled as pinned. For the HDR model, the column-foundation connections were modelled by specifying two nodes at the same location at the base of the column. One node was fully fixed, while the other was free to uplift but restrained in the horizontal direction. The column and brace were connected to this uplifting node. Three zero-length elements were placed in parallel connecting these two nodes. The first element was an elastic-no-tension element with a compressive stiffness of 1000 kN/mm that simulated column compression on the footing, but allowed column uplift. The second element was assigned a stiffness of 1000 kN/mm in tension after a vertical displacement of 91.4 mm, in order to capture the behavior of the lockup. Finally, the energy dissipation element was modelled using the “cast fuse” material which is a hysteretic material similar to Steel02 that explicitly captures the post-yield strengthening and stiffening associated with the cast YC connectors [15], and was calibrated based on tests performed by Cast Connex at the University of Toronto in 2014 (Fig. 5c). The column-foundation connection modelling assumptions are shown in Fig. 5a.

3.3 Nonlinear time-history analysis

Ten records were selected and scaled in order to match the design base earthquake (DBE) spectrum for the fixed base design. The records were scaled using the two-step procedure described in reference [6]). A maximum considered earthquake (MCE) suite was created by scaling the DBE suite up by 50%. A third suite was created by scaling the DBE suite down 50% in order to represent a more frequent, service level earthquake event. Fig. 4d shows the acceleration spectra for the DBE suite. A list of individual records and scaling factors can be found in [11]. Inherent damping was modelled as 3% Rayleigh tangent stiffness damping in modes 1 and 2. A krylov-Newton algorithm was used with an analysis time step of 0.0005 seconds.

4.2 Preliminary study comparing ductile and elastic rocking structures

A preliminary study was performed that compared the HDR frame with ductile BRBs to a model that included elastic braces with the same stiffness. The purpose of presenting results for a rocking structure with elastic brace elements is to demonstrate how high the demands are in the frame superstructure even if a flexural rocking mechanism forms at the base at the base of the frame. The ductile and elastic HDR models were analyzed under the DBE suite of records. For this suite of records, five out of ten of the records caused the lockup to engage for the ductile model, and 7 out of ten cause the lockup to engage for the elastic model. In order to highlight superstructure demands from higher mode demands as opposed lockup engagement, results from only the three records that did not cause lockup engagement for both models are presented.

Fig. 1 shows the brace demands for the ductile and elastic frames for the three records that did not cause the lockup to engage. The median forces in the elastic braces were significantly larger than the ductile braces - as much as 4.0 times greater in the sixth storey, and 2.6 times greater in the first storey. As well, the storey accelerations were much greater in the elastic frame than in the ductile frame. The frame with ductile BRBs did not tend to magnify the peak ground acceleration (PGA), while the elastic frame tended to have storey accelerations that were greater than the PGA along the height of the frame. These results demonstrate the benefit of including ductile braces in rocking frames, and highlight the inability for rocking structures to effectively limit seismic forces unless higher mode mitigation techniques are implemented [8, 9].

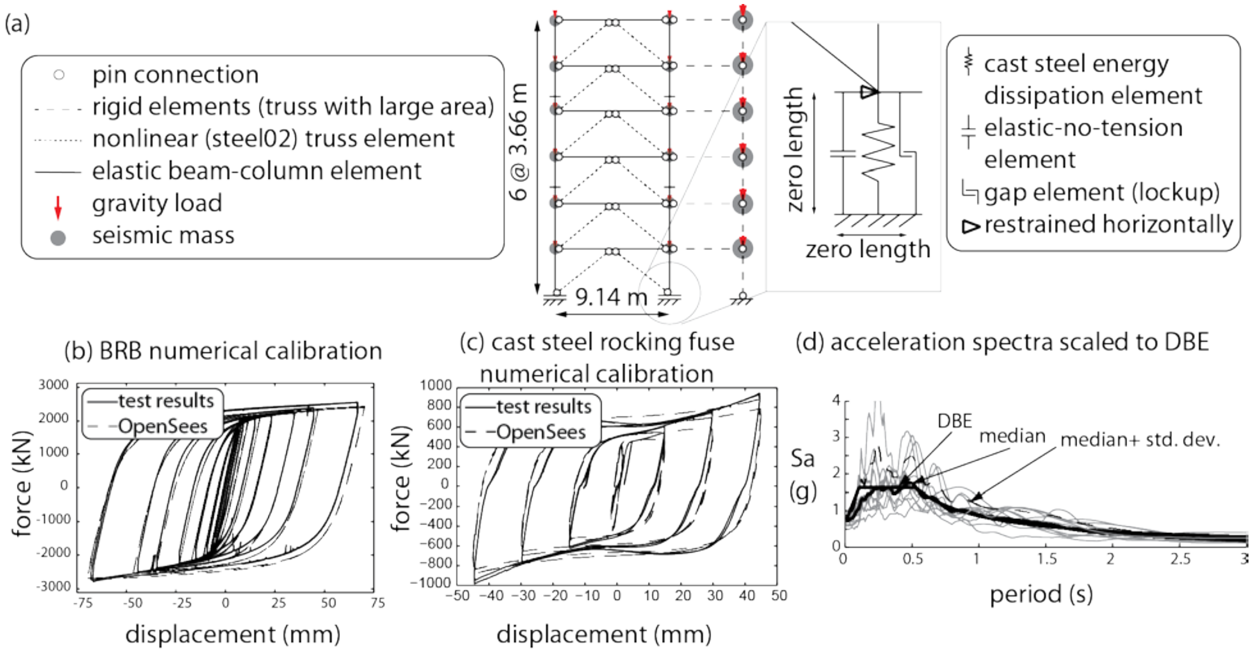

Figure 5: Numerical modelling and analysis: (a) modelling assumptions; (b) calibration of buckling-restrained brace elements to tests [17]; (c) calibration of cast steel supplemental energy dissipation elements to tests performed at the University of Toronto in 2014

4.2 Analysis results

Table 1 gives a summary of peak response values for the fixed base and HDR frames (with ductile braces) for all the record suites, and Fig. 6 shows important results from the nonlinear time-history analyses for the DBE suite of records. Fig. 6a shows the median peak interstorey drifts for the fixed and HDR frames. The interstorey drifts were calculated at each time-step as the interstorey lateral displacement divided by the floor height at each floor. For the fixed base frame there was a notable concentration of demand in the lower stories, with interstorey drifts exceeding the code-derived limit during design of 2.0%. The HDR frame, however, had lower peak interstorey drifts at the lower storeys, and features a much more uniform drift profile. Table 1 shows that the median peak interstorey drifts were less those of the HDR frame for all earthquake intensity levels.

Fig. 6b shows the peak drifts minus base rotation (DMR), which was calculated at each timestep by subtracting the base rotation from the total intestorey drift at each storey. This value reflects the contribution to total storey deformations from the frame itself, and is associated with the damage sustained by the BRBs in the frame. The DMR results in Table 1 further illustrate that for all intensity levels, the deformation demands on the braces in the HDR frame were significantly less than the demands on the fixed base frame.

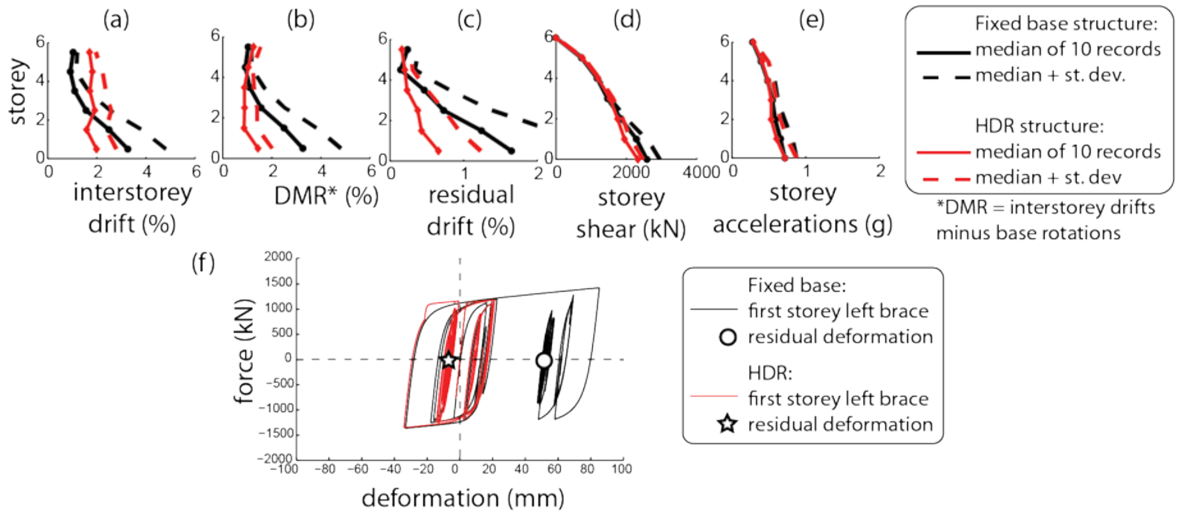

Figure 6: Results of nonlinear timehistory analysis for DBE suite: (a) interstorey drift; (b) interstorey drift minus base rotation (DMR); (c) residual drift; (d) storey shear; (e) storey accelerations; (f) first storey left brace hysteresis

Fig. 6c shows the residual drift profile under DBE. The fixed base frame sustained significant residual deformations, with drifts far in excess of the 0.5% limit that has been suggested by researchers as a possible limit beyond which building demolition might be required [3, 18]. The HDR frame, however, sustained significantly less residual deformation with the first storey median results only slightly exceeding the 0.5% limit. This result is highlighted in Fig. 6f, which shows the first storey left brace hysteresis for an example record scaled to DBE. The HDR frame did not experience the large peak and residual deformations associated with the fixed base frame. Table 1 shows that the residual deformations in the HDR frame were less than the fixed base frame for the 0.5 DBE and MCE suites as well. Notably, under the 0.5 DBE suite of records, while the fixed base frame median value exceeded the 0.5% threshold, the HDR frame sustained a very small peak residual interstorey drift (0.12%), which reflects the beneficial tendency for the HDR system to achieve very low damage under smaller intensity ground motions. It should be noted that 1 record engaged the lockup under 0.5 DBE, 5 records engaged the lockup under DBE, and 8 records engaged the lockup under MCE. Thus under the 0.5 DBE suite of records, aside from the single record that engaged the lockup, the small inelastic brace demands and residual deformations that were observed were induced by the higher mode response of the structure. While the residual deformation response for this HDR frame was very beneficial, it should be noted that there is the potential for increased residual deformations in HDR frames as the hybrid response may limit the amount of reverse yielding after large inelastic cycles. This phenomenon was identified and investigated in [11], and was shown to be most severe for structures associated with a large amount of base rotation demand, such as shorter period frames. Special care is recommended to ensure that HDR designs consider this phenomenon.

Fig. 6d and 6c gives the storey shear and storey acceleration response of the fixed and HDR frames, and reflect that the beneficial force and acceleration control associated with BRB frames is maintained for the HDR system. In fact, the HDR peak base shears (shown in Fig. 6d for DBE and given in Table 1) are consistently less than the fixed base frame's base shear, since the reduction in damage concentration in the lower stories limited the amount of nonlinear over strength experienced by the ductile braces.

Table 1 gives the peak and residual base rotation results. The peak base rotations were limited to the 1% value allowed by the lockup (peak values are slightly greater than 1% due to the elastic deformations of the lockup). The residual base rotations were near zero for the 0.5 DBE and DBE, and relatively small under MCE.

The cumulative damage experienced by the braces in the fixed and HDR frames is reflected by the total energy dissipated by the braces, given in Table 1. The energy dissipated by each brace was calculated by integrating the force-deformation response of each brace from each record. For all three suites of records, the HDR braces dissipated significantly less energy than the fixed base frame. Under MCE, the median dissipated energy for the HDR frame was 54% of the value obtained by the fixed base frame. This result suggests that if a conventionally designed BRB frame could withstand one MCE level event, making the HDR modifications would allow for the same frame to be able to withstand roughly two such events, thus eliminating the need of replacing the BRBs after an MCE even if the residual deformations are not excessive after the first record.

Table 1 also shows the peak column compression and foundation tension obtained in the analyses. Foundation tension values included the effects of the lockup and energy dissipation elements. The column compression and foundation tension values were similar between the HDR and fixed base frames, which is to be expected since in both frames the column forces and foundation tension were limited by the capacity of the ductile braces. The column force and foundation tension forces were less than those determined from capacity design. An increase in the median and median plus standard deviation values is observed under MCE records for both column compression and foundation tension. This result can be attributed to an excitation of the numerical vertical mass specified in the models, which was modelled as lumped at the beam-column connections. Reference [11] investigated a variety of different vertical mass modelling options, and showed that column forces in HDR frames were sensitive to these choices and that lumping the mass at the beam-column connections represents an upper bound result, since mass in a real structure is distributed primarily across a floor slab.

Table 1: Summary of results from nonlinear time-history analysis, median of ten results (median + standard deviation)

	0.5 DBE			**DBE**			**MCE**	
	Fixed	**HDR**		**Fixed**	**HDR**		**Fixed**	**HDR**
Peak drift (%)	1.5 (2.2)	1.1 (1.5)		3.3 (4.8)	2.0 (2.8)		4.4 (7.4)	3.9 (6.4)
DMR (%)*	-	0.72 (0.90)		-	1.4 (2.0)		-	3.1 (5.4)
residual drift (%)	0.54 (1.0)	0.12 (0.21)		0.17 (2.8)	0.66 (1.2)		2.7 (5.6)	1.4 (4.3)
peak base rotation (%)	-	0.64 (0.88)		-	0.91 (1.1)		-	1.0 (1.1)
residual base rotation (%)	-	0.00		-	0.010 (0.016)		-	0.18 (0.41)
peak storey acceleration (g)	0.52 (0.66)	0.51 (0.62)		0.96 (1.2)	0.86 (1.1)		1.41 (1.7)	1.5 (1.8)
energy dissipation by BRBs (kN-m)	705 (1170)	217 (317)		2380 (4210)	1043 (1580)		4560 (8350)	2470 (4100)
peak foundation tension (kN)	2060 (2120)	1446 (1795)		2350 (2650)	2290 (3300)		2830 (3360)	4060 (5430)
peak base shear (kN)	2110	1883		2510	2290		2920	2760

	(2270)	(1995)		(2850)	(2440)		(3500)	(3140)
peak column compression (kN)	3690 (3730)	3520 (3660)		3870 (4030)	4110 (4390)		4070 (4370)	4570 (5020)

4. Conclusions

This paper demonstrated how the performance of a conventionally designed buckling restrained braced frame can be greatly improved by redesigning the column-foundation connection to accommodate limited base rocking in conjunction with the distributed ductility of the BRB frame. The paper also shows that this proposed hybrid ductile-rocking system offers advantages with regard to storey forces and accelerations when compared to structures that are designed to rock at the base while the braces are designed to remain elastic. The example 6-storey HDR frame achieved significant improvements compared to a fixed base conventional BRB frame, as evidenced by the following conclusions from a study of nonlinear time-history analysis results:

- Residual deformations were reduced in the HDR frame compared to the conventional fixed base frame
- The peak deformation demands on the braces were more evenly distributed along the height of the frame, as opposed to the concentration of damage in the lower stories that were evident in the conventional BRB frame.
- The energy dissipated by the ductile braces was roughly halved suggesting that an HDR frame could withstand two large earthquakes if a fixed base frame could only withstand one.

It is proposed that the HDR system be considered effectively code-compliant with respect to the strength requirements of the code since it features a code-compliant BRB frame as its superstructure, and a lockup device that ensures, after a predetermined amount of allowable base rocking, the development of the full superstructure codified specified lateral resistance and ductility. With respect to global deformations of the structure, which are linked to the global stability of the system, the studies in this paper showed that the peak drifts in the HDR system were typically similar if not smaller when compared to the conventional fixed base frames. Based on these results, it could then be inferred that the HDR structures considered in this paper do therefore meet the intent of the current seismic design codes for fixed base structures with respect to global deformations of the system. It is recommended that future studies be performed to formally establish the collapse performance of the HDR system, and confirm that this performance is equal or better than a conventional fixed base BRBF.

It is also worth noting that the idea of allowing small amounts of column uplift is already recognized in modern building codes, even if rocking is not explicitly intended as an approved seismic system. In AISC 341-10 [13], column forces in braced frames can be limited by the “resistance of the foundation to overturning uplift”. Thus the HDR column-foundation detail could be considered a form of foundation rocking in a conventional SFRS that is already recognized by the building codes. In this regard, the HDR system is recommended for engineers who wish to design lower-damage systems but not be burdened by the complexity and expense associated with controlled rocking structures that have yet to be codified.

Further research is also recommended to confirm the ability of conventional slab details to accommodate the vertical deformations associated with rocking (maximum of 91.4 mm of column uplift in the example in this paper). If conventional slab details are used, the only extra detailing cost associated with the HDR system is the isolated column-foundation connections at the base of the SFRS, as opposed to other detailing complexities (eg. higher mode mitigation and post-tensioning tendons) that are associated with controlled rocking structures. Future studies should also consider the combined effect of lateral and vertical accelerations. The HDR system can be expanded to include other damage-based systems like moment-resisting frames, eccentrically braced frames, and conventional braced frames.

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