SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

FINITE ELEMENT INVESTIGATION OF MOMENT RESISTING FRAMES WITH A NEW SHEAR STRUCTURAL FUSE

M.T. NIKOUKALAM[1] and Kiarash M. DOLATSHAHI[2]

ABSTRACT

Special moment resisting frames (SMRFs) are of the most desirable lateral load resisting systems owing to their high ductility and energy dissipation under severe cyclic loading and also the architectural freedom they provide. However, beam depth and beam span-to-depth ratio are so significant in the inelastic behavior of beam-to-column connections in MRFs that design codes have limited their application to cases where the clear span-to-depth ratio of the beam is limited to a minimum value. This limit is to guarantee the plastic hinging at the beam ends with a sufficient length. This requirement limits the application of flexural beams mostly in framed-tube structures, which are one of the most desirable and efficient types of MRFs for tall buildings in a wide range of heights, consist of closely spaced columns in perimeter that are tied by deep beams. In such cases, shear in the beams seems to be a much better candidate for displacement-controlled component rather than the moment at the beam ends. To overcome the abovementioned limit, in this paper replaceable shear link beam in eccentrically braced frames (EBFs) is introduced as a new ductile structural fuse for MRFs free of bracing. Similar to EBFs, the link is so tuned that the seismic energy is dissipated by shear yielding in a small segment at the mid span of the beam with stable hysteresis behaviour. Two alternative details of a replaceable link beam applicable in design of new structures and also retrofit of existing ones are introduced. Moreover, the cyclic behavior of a single span-single story MRF equipped with the proposed shear fuse is investigated through finite element modelling and compared with the conventional one with flexural plastic hinges at reduced beam section of the beam. The results showed the capability of the shear link beam to work as a high capacity energy dissipation system with a stable hysteretic behavior. In general, the link beams yielding in shear is proved to be a great replace for flexural hinges in MRFs, offering a ductile structural fuse with the ability to be replaced after an earthquake.

INTRODUCTION

Moment resisting frames (MRFs) designed according to the latest seismic codes are so tuned that are expected to dissipate energy through formation of flexural hinges at the beam ends. In the recent two decades, especially after 1994 Northridge, California, earthquake, many attempts have been made to improve the seismic behavior of MRFs through improving their energy dissipation mechanism, mostly resulted to development of new beam-to-column connection concepts (AISC 2010a). In all these new concepts the main focus is on providing a more stable behavior for the flexural hinges at the specific segments of the beam that are designed and detailed to act as ductile fuses, dissipating energy through a stable hysteretic response while limiting the forces transmitted to other components in the structure.

[1] Research assistant, University of Tehran, Tehran, Iran, nikoukalam@ut.ac.ir
[2] Assistant professor, Sharif University of Technology, Tehran, Iran, dolatshahi@sharif.edu

The prequalified connections introduced in ANSI/AISC 358-10 (AISC 2010a), have been extensively tested and showed good ductility and energy dissipation under severe cyclic loading. However, to guarantee the flexural plastic hinging with a sufficient length, their application is limited to cases where the clear span-to-depth ratio of the beam is greater than a minimum value specified in codes (FEMA 2000; AISC 2010a). This requirement limits the application of flexural beams mostly in framed-tube structures that generally consist of closely spaced exterior columns and deep spandrel beams rigidly connected together. In practice, the framed tubular behavior is achieved by placing columns at 3.05 m to as much as 6.1 m apart, with spandrel depth varying from 0.90 to 1.52 m, that leads to span-to-depth ratio between 3.4 and 4 (Taranath 2011). This is much lower than seven, specified as the minimum value in AISC Prequalified Connections (AISC 2010a) for special moment resisting frames. Rather than this limitation, there are several drawbacks that apply to the suggested prequalified connections in ANSI/AISC 358-10 (AISC 2010a). In traditional ductile structural systems, since the ductile fuse is an integral part of the beam, strength and drift design of the structure are coupled. In these systems, when drift requirements control the design and member sizes are increased to meet drift limits, the capacity of the yielding fuse also increases. This in turn results to larger force demands on other parts of the structure, including columns, floor slabs, connections, and foundations, often resulting in overdesigned structures and increased overall costs. In addition, significant damage can result in the beam from repeated inelastic deformation and localized buckling during a design level earthquake. As the cumulative inelastic action of the structure is unknown, it is difficult to assess the extent of damage and the structure's ability to provide an adequate level of safety for any subsequent loading. Furthermore, repair of the beam is very difficult, disruptive, and costly (Shen et al. 2010).

To overcome the abovementioned limit and drawbacks, in this paper replaceable shear link concept is introduced as a new ductile fuse for MRFs. In this new concept, shear force in the beam is considered as the displacement-controlled component of the system and similar to eccentrically braced frames (EBFs), is so tuned that the seismic energy is dissipated by shear yielding in a small segment at the mid span of the beam. For this purpose, replaceable links with smaller shear capacity than the beam elements are introduced at the mid length of the beam. Two alternative details of a replaceable link beam applicable in design of new structures and also retrofit of existing ones are introduced. Moreover, the cyclic behavior of a single span-single story MRF with span-to-depth ratio of four and Reduced Beam Section (RBS) connection is studied through finite element models with two alternative structural fuses: (1) RBS connection; (2) shear link beam.

IDEA INCEPTION

Formation of flexural plastic hinges in the beams is the most common energy dissipating mechanism in ductile MRFs (Bruneau et al. 2011). However, beams with shorter span to-depth ratio will have a sharper moment gradient across the beam span, resulting in reduced length of the beam participating in plastic hinging and increased strains under inelastic rotation demands (AISC 2010a). Moreover, for a given beam cross section, due to the formation of plastic hinges, decreasing the beam length results in the increase of the shear force demand (V_p) calculated by:

$$V_p = \frac{2M_p}{L} \tag{1}$$

where M_p is the nominal plastic flexural strength of beam section and L is the length of the beam.

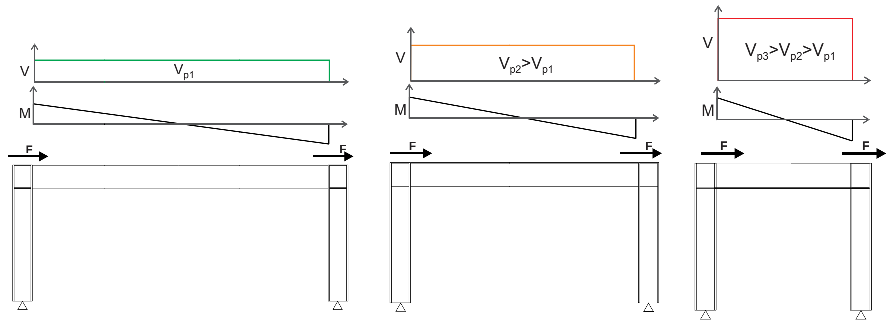


Figure 1. Variation of the shear demand by decreasing the beam span length in MRFs.

Based on the current design and detailing approaches in seismic codes, the shear in the beam is a force-controlled action and should be designed based on the capacity of the ductile fuse, which is the flexural hinge at beam ends. The required shear capacity is provided by increasing the beam web thickness which is followed by an increase in the plastic flexural strength of the beam (M_p). As a result, the capacity demand requirements for all other members, diaphragms, and foundations are also increased. Alternatively, in such cases shear in the beams seems to be a much better candidate for displacement-controlled component rather than the moment at the beam ends. Therefore, a yielding shear fuse can be placed at the beam mid-span to dissipate the input energy to the structure while limiting the capacity design forces on other parts of the structure (Figure 3). Figure 3 illustrates an MRF designed according to the replaceable shear link concept. The shaded portions of the structure represent elements spanning multiple stories that can be fabricated in the shop, shipped, and connected through the bolting of the nonlinear links on the site. Note that according to Figure 1, since the flexural demand due to the lateral load at the beam mid-span region is much lower than the beam ends, it is expected that employing the shear link with reduced section compared to other parts of the beam at this place will not greatly affect the lateral stiffness of the structure. Figure 2 shows a 34 story framed tube structure constructed with column-tree MRFs which supports the idea of placing shear link at the mid-span considering constructional issues.

Figure 2. 34-story framed tube structure with beam splices at the mid-span of the beams; Third Millennium Tower (TMTower), Tehran, Iran (Photos by M.T.Nikoukalam).

PROPOSED DETAILS

Two types of replaceable link configurations with alternate link-to beam connections are proposed to employ a replaceable shear link in MRFs. Figure 4(a) shows the first type which is a W-shape link welded to unstiffened end plates, which are bolted to the floor beam end plates. The second replaceable link type consists of two channel sections, back-to-back, connected to the web of the floor beam through an eccentrically loaded bolted or welded web connection, as shown in Figure 4(b). Both link types has been employed in MRFs as a replaceable flexural hinge and in EBFs as a replaceable link beam and tested by Shen et. al. (2010) and Mansour et. al. (2011). The second link type is also applicable in rehabilitation of existing buildings, such as the one shown in Figure 2, since it can be fabricated on-site with no or minimal change in the floor slab. In Figure 4(c) the application of a W-section link with bolted end-plate connection is presented in a frame with non-prismatic beams. In this practice which is very common in Iran, the beam section varies in the length according to the moment diagram, resulting to a more economical design.

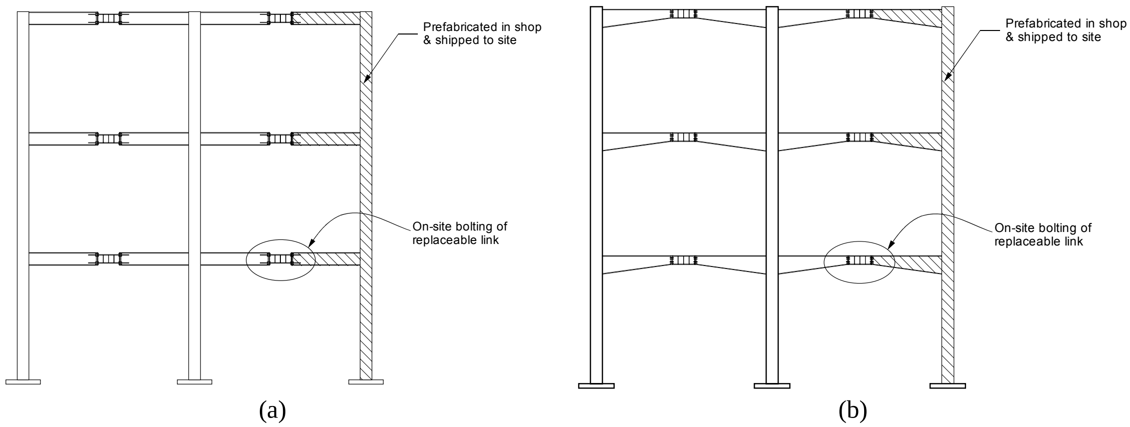


Figure 3. Replaceable shear link concept in MRFs: (a) Prismatic beam; (b) non-prismatic beam.

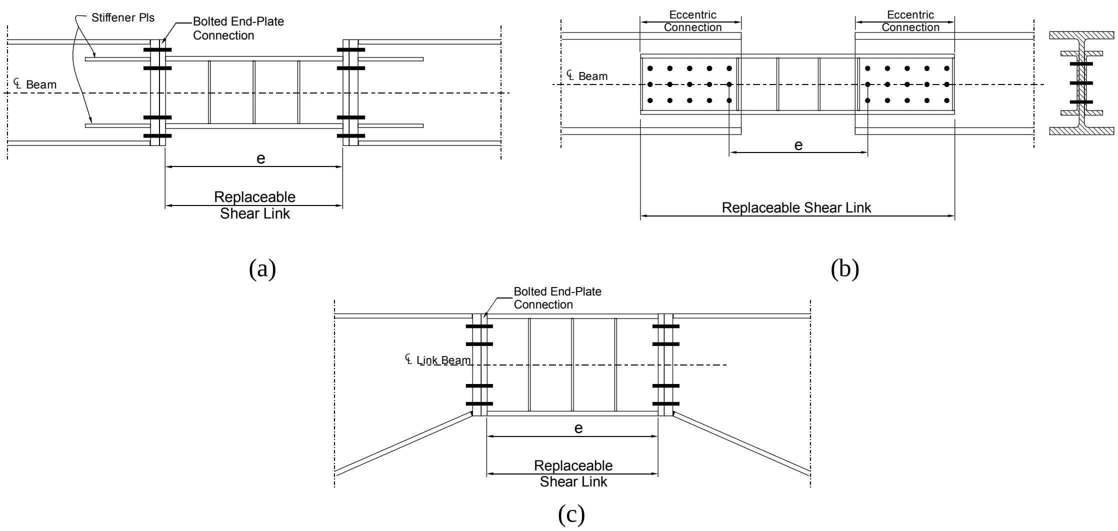


Figure 4. Replaceable link configurations: (a) end-plate connected; (b) web connected; (c) end-plate connected to non-prismatic beam.

DESIGN OF REPLACEABLE SHEAR LINKS

The link design procedure introduced in this study is based on the concept of replacing the traditional flexural hinges at the beam ends in a MRF by a shear fuse at the mid-span of the beam. The design shear strength of the link beam (V_L) is determined as below so that all inelastic deformations occur in the link beam rather than beam ends:

$$V_L \leq \varphi V_{pb} \tag{2}$$

Where V_{pb} = the shear force throughout the floor beam corresponding to formation of flexural hinges at beam ends, and φ= accounts for the increase in stresses due to strain hardening of the link yielding in shear which is assumed to be 1/1.5 in this study.

The required flexural strength of the link beam is defined from the bending moment diagram along the beam so that no flexural yielding occurs at link ends prior to the shear yielding in the link web. Defining the link length is the next step. The same concept and rules for defining the length of link beams in EBFs is adopted here, since the behavior of the link beam proposed in this study is almost the same. Based on AISC Seismic Provisions (AISC 2010b), links with a length less than $1.6M_{pL}/V_{pL}$ (V_{pL}= the link plastic shear resistance, and M_{pL}= the link plastic flexural resistance) are dominated by shear yielding, whereas those longer than $2.6M_{pL}/V_{pL}$ are dominated by flexural yielding. Shorter links that yield in shear are preferred because these have a more stable energy dissipation mechanism and a more predictable post yield behavior than the longer links that yield in flexure (Engelhardt and Popov 1989). The design and details of link stiffeners follow the same rules specified for link beams in EBFs in AISC Seismic Provisions (AISC 2010b).

To allow for the placement of the floor deck on top of the floor beam, the link-to-beam connections should be sized such that the end plates are flush with the floor beam section. Moreover, since the link section depth is less than the floor beam depth, the link is not connected to the floor slab, allowing for replacement of the link.

FINITE ELEMENT STUDY

The main purpose of this study is to evaluate the efficacy of the proposed shear fuse in MRFs with low span-to-depth ratio where flexural hinges are anticipated not to be effective as energy dissipaters. The finite element study involves modelling a single span-single story MRF with low span-to-depth ratio (L/d=4) which is much lower than the minimum of seven specified in AISC358-10 (AISC 2010a) for most prequalified moment connections in SMRFs. The frame is modelled with two alternatives as the energy dissipation mechanism: (1) formation of flexural hinges at beam ends with Reduced Beam Section (RBS); (2) shear yielding in the link beam at the mid span.

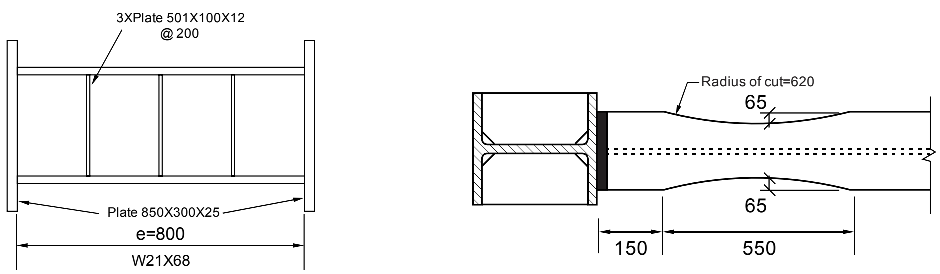
The general-purpose nonlinear finite element analysis (FEA) program ABAQUS (SIMULIA 2011) was used to develop 3-D nonlinear finite element models of frames. The standard W33X118 section was selected for the beam and W14X193 for the column section to ensure a weak beam-strong column configuration, which is required by the AISC Seismic Provisions (AISC 2010b). The link beam section was determined to be W21X68 from the design procedure presented before. The details of the link beam are presented in Figure 5(a). The column length was considered to be 3960 mm and the beam clear span 3344 mm (four times the beam depth). A992 steel (which has a nominal yield stress of 345 MPa) was used for the beams, column, link beam, doubler plates, stiffener, end plates, and continuity plates. The RBS design followed the limits in AISC Prequalified Connections (AISC 2010a) with details shown in Figure 5(b). The panel zone strength of the model was based on the required strength per AISC Seismic Provisions (AISC 2010b).

Models were capable of predicting strength degradation resulting from buckling of the flanges, web, and stiffeners. Strength degradation associated with material fracture or tearing was beyond the scope of this study. In order to improve computation time, the model was developed using reduced integration shell elements, indicated S4R in ABAQUS. A shell element was used to model the members in lieu of a solid element, since a shell element is more capable of properly capturing the effects of local buckling. Details of stiffeners, continuity plates, and end plates were not considered and the welds were not modelled explicitly. Mesh refinement study was conducted to determine the optimized level of refinement necessary to reach the accuracy in the connection region.

Von Mises yield surface and an associated flow rule was used to model the plasticity. The hardening model used in the analysis included combined nonlinear isotropic and kinematic strain hardening. Data from cyclic coupon testing conducted by Kaufmann et al. (2001), designated as Steel C, were used for calibrating the cyclic material properties for the analysis. The Steel C material was A572 grade 50 steel with yield strength of 372 MPa and an ultimate strength of 496 MPa under monotonic testing. The same cyclic material properties were used for all components of the models.

Simple support was defined for the bottom of the columns as boundary condition. Two identical displacement-controlled loadings were applied at top of the columns. Initial imperfections were included in the analysis, and were based on a proportion of the amplitude of the first two buckling modes of the model. The buckling modes were determined by a linear eigenvalue buckling analysis. Moreover, Geometric nonlinearity option in ABAQUS was utilized to account for large displacement effects so that local buckling could be captured and the post buckling behavior of the components could be simulated.

The loading protocol specified in Section K2 of the 2010 AISC seismic provisions (AISC 2010b) for qualifying cyclic tests of beam-to-column moment connections in special moment frames was used in all analysis.



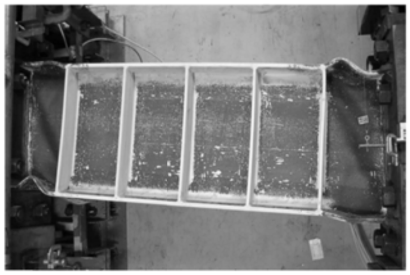
(a) (b)

Figure 5. Details of: (a) shear link beam; (b) RBS connection.

MODEL VERIFICATION

In order to verify the modelling approach discussed previously, finite element models were created of a link beam and an RBS connection tested in prior researches and the analysis results compared to the test results.

Specimen 9-RLP which was an intermediate link beam (2.6M_{pL}/V_{pL} < e <1.6M_{pL}/V_{pL}) constructed of A992 steel tested by Okazaki and Engelhardt (2007), was modelled based on the measured dimensions of the experimental specimen using the techniques described previously. Nodes on both link ends were restrained against all rotations. Loads were applied by imposing transverse displacements on the right end nodes. Left end nodes were permitted to translate horizontally, but were constrained to all have the same horizontal translation. Loading in this manner resulted in constant shear along the length of the link with equal end moments and no axial forces. The link loading protocol in Section K of the 2010 AISC Seismic Provisions was used for the analysis. Figure 6 compares the deformed geometry and the inelastic rotation versus shear hysteresis loops of specimen 9-RLP and the corresponding model. The model properly predicted distributed link web yielding and web and flange local buckling in the end panels resulted to strength degradation that occurred in the test specimen. Therefore, the simulation results are considered to be in good agreement with the experimental results.



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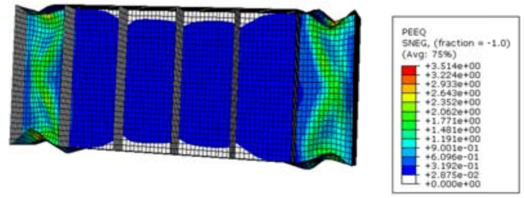

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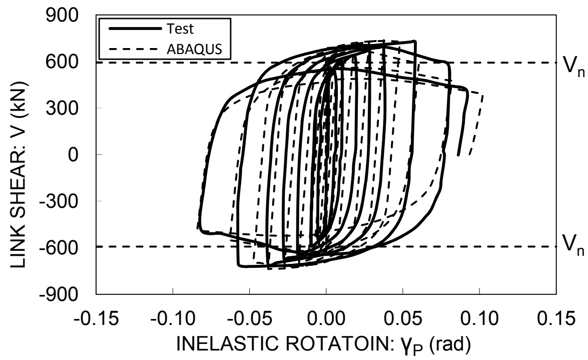
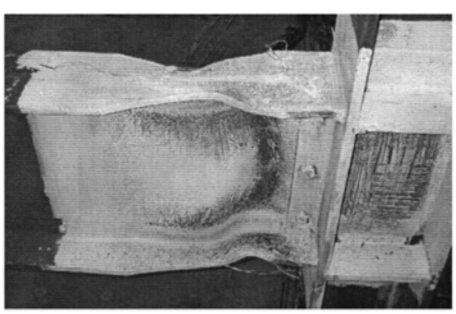


(c)

Figure 6.Link beam verification results: (a) Deformed geometry from experiment (Okazaki and Engelhardt 2007); (b) Deformed geometry from model; (c) comparison of shear versus inelastic rotation hysteresis

Specimen DB700-SW which was a bare RBS moment connection with strong panel zone and welded beam web connection type tested by Lee et. al. (2005), was modelled based on geometry and coupon material test results given in (Lee et al. 2005). The deformed geometry and story drift versus normalized moment at column face is shown and compared with the test results in Figure 7. the normalization was based on the nominal plastic moment of the unreduced beam section. Based on the results, the model properly predicted significant beam yielding, minor yielding in the panel zone, beam web and flange local buckling in the RBS, and strength degradation that occurred in the test specimen. The predicted response by the model is in good agreement with the experimental results.



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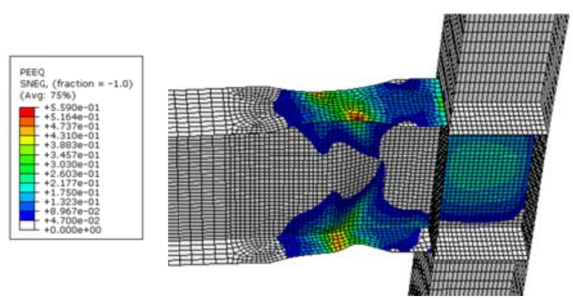

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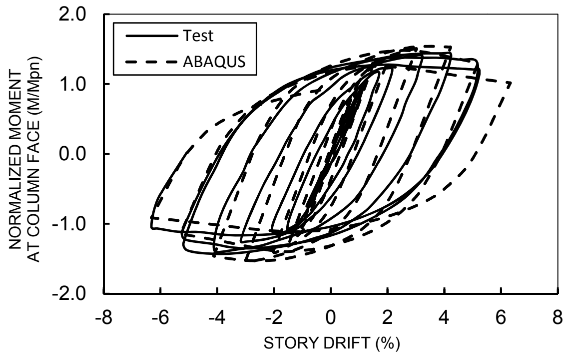
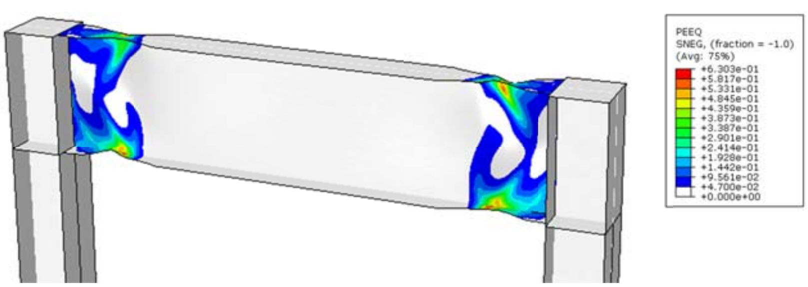


(c)

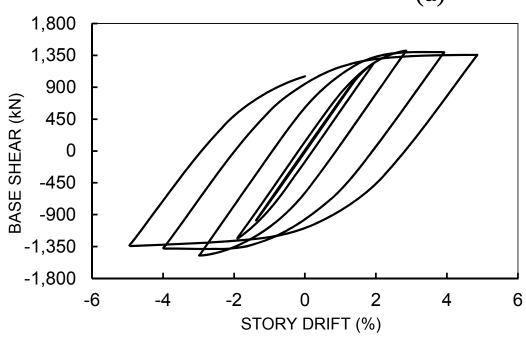
Figure 7. RBS connection verification results: (a) Deformed geometry from experiment (Lee et al. 2005); (b) Deformed geometry from model; (c) comparison of story drift versus normalized moment at column face hysteresis.

RESULTS AND DISCUSSION

Figure 8(a) shows the distribution of plastic equivalent strain (PEEQ) and the deformed shape of the MRF with RBS connection. Significant yielding in the beam flanges and web accompanied by local buckling in flange and web is evident in Figure 8(a). Acceptance criteria defined in AISC Seismic provisions for moment connections in special MRFs is that the required flexural strength of the connection, determined at the column face, must equal at least 80 percent of the nominal plastic moment of the connected beam at an interstory drift angle of 0.04 radians. Based on the acceptance criteria described in AISC, the flexural strength at the column face has remained above 80% of the nominal capacity (M_p) up to 5% story drift, as shown in Figure 8(c). Moreover, the plasticity has extended to the column face resulting to high strain level at the column face and at the reduced beam section. However, based on previous experimental and analytical studies on RBS connections (FEMA 2000), it is anticipated that in beams with span-to-depth ratio lower than seven, the plastic hinging cannot extend as it is occurred in Figure 8(a). This is mainly due to the high potential of fracture as a result of high plastic strains in beam flanges.



(a)



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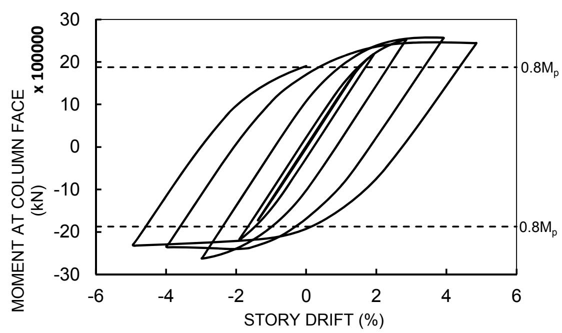

(c)

Figure 8. (a) Deformed shape and distribution of plastic equivalent strain at the end of 5% drift cycle; (b) story drift versus base shear; (c) story drift versus moment at the column face.

The distribution of plastic equivalent strain and the deformed shape of the same MRF equipped with a replaceable shear link is shown in Figure 9(a). In this case shear yielding and minor local buckling in the link beam panels is detected by the model. The base shear versus story drift and link shear versus total link rotation is shown in Figure 9(b) and (c), respectively. Acceptance criteria for link beams are based on inelastic link rotation as defined in AISC Seismic Provisions (AISC 2010b). The codes specifies that a shear yielding link (e < 1.6M_{pL}/V_{pL}) should be capable of developing an inelastic rotation of 0.08 rad. The inelastic rotation capacity of the link specimens is defined as the maximum level of inelastic rotation sustained for at least one full cycle of loading prior to the link shear strength dropping below the nominal link shear strength. Since no significant deterioration has occurred, the behavior of the link is acceptable based on the specified criteria in AISC Seismic Provisions for shear link beams.

Link overstrength is defined as the maximum shear force developed in the link (V_{max}) divided by the plastic shear strength of the link (V_{pL}). Based on the analysis results shown in Figure 9(c), the link overstrength is calculated as 1700/1130=1.53. Thus, the link overstrength factor of 1.5, as assumed in the design of the link beam per the current U.S. code provisions, appears to be reasonable here.

Since the shear link beam is placed at the mid-span of the beam which has minimal contribution in providing the frame lateral stiffness, it is expected that it would not significantly affect the stiffness. Comparing the elastic stiffness of the frame with shear link beam (18.5 kN/mm) with the frame with RBS connection (18.3 kN/mm) reveals the point that the new proposed shear fuse has provided a stable energy dissipation system without affecting the stiffness of the system. Per this characteristic, it is possible to decouple the strength and stiffness of the MRFs with this new structural shear fuse which can result to a more economical design of the forced controlled elements of the system such as column, floor diaphragm, and foundation.

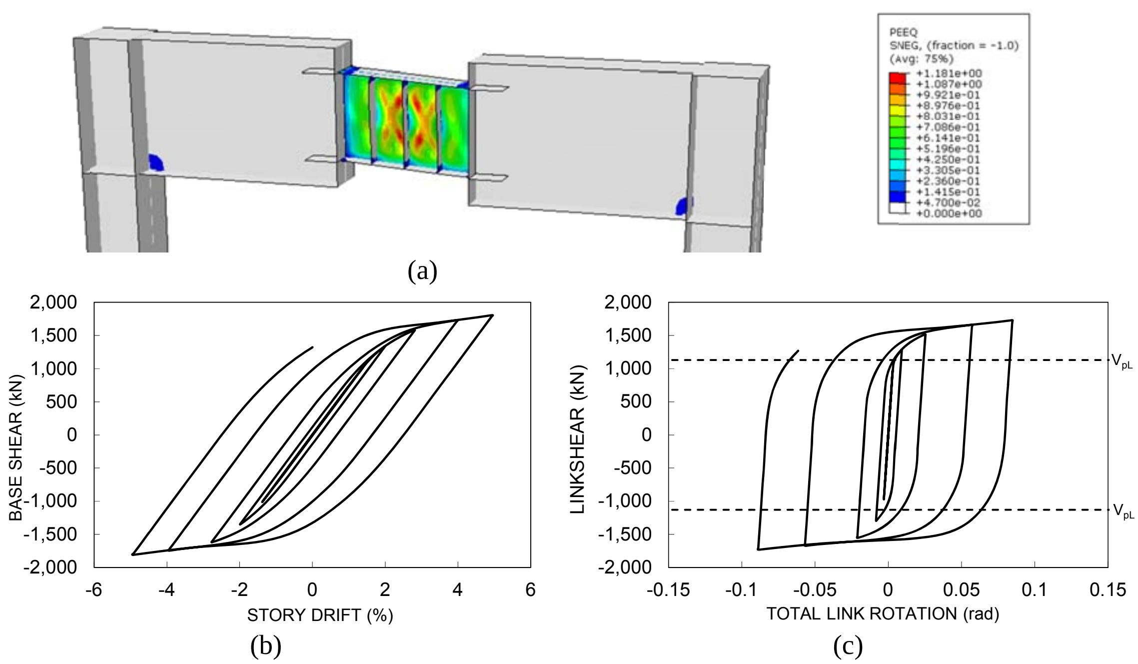

Figure 9. (a) Deformed shape and distribution of plastic equivalent strain at the end of 5% drift cycle; (b) story drift versus base shear; (c) link rotation versus link shear.

Figure 10 compares the total energy dissipated in both systems after 5% total story drift. As it is evident, the energy dissipation capacity of the shear link beam is comparable to RBS connections. It should be noted that as it was stated before, it is expected that the RBS connection in beams with low span-to-depth ratio could not provide such ductility capacity due to reduced length of the beam participating in plastic hinging and increased strains under inelastic rotation demands. Since the extended plastic hinge in the studied RBS connection is accompanied by high strains in the beam flange, it is not reasonable to be considered as real. Therefore, as the shear link beam has provided almost the same ductility capacity as the RBS connection, it proves to be a suitable alternative for not only the RBS, but also for other traditional moment connections.

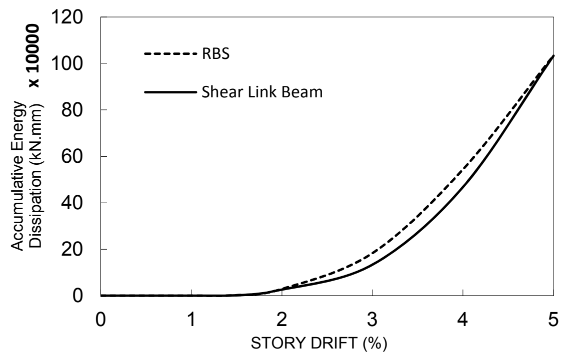

Figure 10. Comparison of energy dissipation capacity of RBS with shear link beam.

CONCLUSIONS

A new energy dissipation mechanism that works in shear has been proposed in this study for moment resisting frames (MRF) with low span-to-depth ratios. Based on current US codes, beams with span-to-depth ratio less than seven are not allowed to be used in special moment resisting frames since it is anticipated that they cannot provide the required ductility capacity. A new replaceable structural shear fuse, that is similar to shear link beams in eccentrically braced frames, is introduced and studied in this paper as a replacement for traditional flexural fuse in MRFs with low span-to-depth ratios. Two alternative configurations for the link beam have been introduced: (1) a W-shape link welded to unstiffened end plates, which are bolted to the floor beam end plates; (2) two channel sections, back-to-back, connected to the web of the floor. The cyclic behavior of an MRF with RBS connections with span-to-depth ratio of four has been studied through finite element models created in ABAQUS. Then the RBS connections are removed and the frame is equipped with the new proposed shear link beam designed based on the given procedure presented in this paper. The cyclic behavior of this new frame is studied and compared with the conventional frame with RBS connections. Based on the results, it can be concluded:

- The new proposed shear fuse can provide a high ductility capacity for MRFs with low span-to-depth ratios. Although the results show a high ductility capacity in RBS connection, but it should be noted that this cannot be considered as real since it is anticipated that fracture and tearing would occur due to high strains in the beam flange.
- The shear link beam can be employed in MRFs as a structural fuse with minimal effect on the stiffness of the system. Thus, it is possible to decouple the strength and stiffness of the MRFs with this new structural shear fuse which can result to a more economical design of the forced controlled elements of the system such as column, floor diaphragm, and foundation. Moreover, Using the replaceable link concept, the designer has greater flexibility to choose a section for the yielding link that best meets the required strength without automatically changing the floor beam section.

- Since all accumulative damages will be concentrated only within the link, damaged links can be quickly inspected and replaced following a major earthquake, significantly minimizing the downtime of the structure and extending its life span.

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