Numerical Validation on Noval Replaceable Reduced Beam Section Connections for Moment-Resisting Frames

Yasin Onuralp Özkılıç (✉ yoozkilic@gmail.com)

Necmettin Erbakan Üniversitesi <https://orcid.org/0000-0001-9354-4784>

Mehmet Bakır Bozkurt

Manisa Celal Bayar Üniversitesi: Manisa Celal Bayar Üniversitesi

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Abstract

After a severe seismic activation, the buildings designed based on seismic specifications are expected to have damages without any local and global collapse. The key parameters that influence re-occupancy of the damaged buildings are workmanship cost and time. In order to circumvent shortcomings of these crucial parameters, replaceable fuse members with high energy dissipation capacity have been developed up to date. Moment resisting frames (MRFs) are one of the most preferred lateral load resisting systems in terms of architectural versatility and high energy dissipation capacity. Even though many beam-to-column connection details have been investigated over the years, none of them has addressed assembly difficulties when replacing the damaged fuse members under the residual drift ratio occurred after an earthquake. Therefore, this study presents replaceable reduced beam sections with connection detail for MRFs under residual drift. The proposed detail includes end-plated mid-spliced reduced beam sections. Pursuant to this goal, the proposed detail has been investigated under cyclic loading protocol required by AISC 341-16 by using ABAQUS finite element program. Seismic response of the proposed connection detail was compared from PEEQ and Rupture Index standpoint. The obtained findings indicate that the proposed mid-spliced replaceable reduced beam section satisfies all requirements mandated by AISC 341-16.

1. Introduction

Lateral loads acting on the structures due to wind and earthquake are resisted by lateral load resisting systems. Moment resisting frames (MRFs) and concentrically braced frames (CBFs) are the most preferred systems developed up to date. As opposed to CBFs, MRFs composed of beams, columns and rigidly connected beam to column connections have high energy dissipation capacity thanks to plastic deformations which, in turn, result in plastic hinges on the ends of the beams of MRFs under seismic loads. Therefore, all structural members other than the ends of the beams of MRFs need to be designed to remain fully elastic. On the other hand, unexpected material hardening of the beams and lack of beam to column connections may lead to brittle failures. In order to avoid the undesired behavior of MRFs, reduced beam sections adjacent to beam-to-column connections are employed. Reduced beam section (RBS) concept was first mentioned in the Plumier studies (Plumier, 1997). More than 150 buildings where MRFs were utilized as lateral load resisting systems encountered brittle failures at the beam to column connections in 1994 Northridge and 1995 Kobe earthquakes (Shen et al., 2011). Two strategies which are strengthening beam-to-column connections and weakening beam sections arose in order to promote poor seismic behavior of MRFs after these EQs. RBSs that diminish or eliminate plastic deformation of the beam-to-column connections by producing plastic hinge in the reduced beam section domain present the most efficient connection facility from a seismic response standpoint. RBS area is produced by cutting both top and column flanges of the beam segment adjacent to beam to column connections. Tahamouli Roudsari et al. (Moradi Garoosi et al., 2018; Tahamouli Roudsari et al., 2020; Tahamouli Roudsari et al., 2018) improved the seismic performance of the RBS connections by adding stiffeners in different configurations. Morshedi et al. (Morshedi et al., 2017) developed double reduced beam section to improve

the behavior of the RBS. Swati ve Gaurang (Swati and Gaurang, 2014) investigated the moment connections to evaluate the existence of the RBS and stated that the RBS connections demonstrated better seismic performance compared to conventional moment connections under cyclic loading. The RBSs employing HEA European section were examined in the study conducted by Pachoumis et al. (Pachoumis et al., 2009). Sophianopoulos and Deri (Sophianopoulos and Deri, 2011) recommend using the RBSs with end-plated connections. Similarly, Sofias et al. (Sofias et al., 2014) examined the RBSs with end-plated connections and demonstrated that the tested specimens showed adequate seismic performance under cyclic loading. Moreover, Oh et al. (Oh et al., 2015) investigated seismic performance of the RBSs where one end of that was connected to the column by making use of weld and the other end was connected to beam with splice connection.

After a severe earthquake, the fuse members of the lateral load resisting systems are expected to be damaged but not fractured. Therefore, the buildings which have damaged structural members due to the seismic event need to be retrofitted as soon as possible to reoccupy. In order to recover the damaged structure where MRFs are employed, replaceability of the yielded segment of the existing beam members may be sufficient. However, torch cutting and site welding need to be applied to replace the damaged beam segment with the new ones. These operations bring about costly solutions, time-consuming and poor quality (Mahmoudi et al., 2019; Shen et al., 2011). It is crucial to develop new strategies to reduce the repair and retrofitting costs as well as reoccupy damaged buildings as soon as possible (Piluso et al., 2014). In order to overcome these problems, replaceable fuse members have been developed over the years.

The concept of replaceable members has been used especially in MRFs and eccentrically braced frames (EBFs). For EBFs, Stratan and Dubina (Stratan and Dubina, 2004) proposed the first replaceable links which can be easily replaced after an earthquake. Dubina and Stratan (Dubina et al., 2008) subsequently conducted an experimental program to investigate the seismic performance of the replaceable link in EBFs. Various replaceable links for EBFs have been developed by many researchers (Bozkurt et al., 2019; Bozkurt and Topkaya, 2017, 2018; Mansour et al., 2011; Ozkilog, 2021; Özkılıç and Topkaya, 2021a; Özkılıç et al., 2021). As for MRFs, the first replaceable connection detail where end-plated and web-connected connections were utilized was proposed by Balut and Gioncu (Bălut and Gioncu, 2018). After that, in order to investigate the seismic performance of the replaceable connection details proposed by Balut and Gioncu (Bălut and Gioncu, 2018) for MRFs, four full-scale experiments were conducted by Shen et al. (Shen et al., 2011). Experimental results indicated that all specimens tested demonstrated sufficient seismic performance. Nikoukalama and Dolatshahi (Nikoukalama and Dolatshahi, 2015) proposed replaceable link connections for MRFs. To evaluate the performance of the connections, a numerical investigation was carried out by using ABAQUS software under cyclic loading. The findings showed that the proposed connections exhibited much more ductility than the conventional connections. Mahmoudi et al. (Mahmoudi et al., 2019) subsequently conducted an experimental program to validate and evaluate the seismic performance of the replaceable link connections proposed by Nikoukalama and Dolatshahi (Nikoukalama and Dolatshahi, 2015). This experimental study proved that the proposed connection can be an alternative connection for MRFs. Replaceable beam to column connection detail utilizing pin

connections (Pongiglione et al., 2021), buckling-restrained steel plates and mechanic pin connections (Peng et al., 2020), perforated shear links (Mirghaderi et al., 2021) have been developed for MRFs. Richards (Richards, 2019) suggested a novel replaceable connection detail and conducted two experimental tests to examine seismic performance of the proposed detail. On the other hand, Kalehbasti and Dolatshahi (Rezazadeh Kalehbasti and Dolatshahi, 2018) developed two shear links including perforated and slit shear links for MRFs and investigated using numerical analyses. Dolatshahi et al. (Dolatshahi et al., 2018) proposed slitted replaceable links for MRFs. Garoosi et al. (Moradi Garoosi et al., 2018) carried out both numerical as well as experimental studies to investigate reduced beam connections as replaceable links. In these studies, the reduced beam section was connected to both the column and the beam by means of end-plated connection. Vajdian et al. (Vajdian et al., 2020) developed a replaceable drilled flange connection for beam-to-column connections. The numerical results showed that plastic deformations were observed in the replaceable part. Zareia et al. (Zareia et al., 2016) carried out a numerical study using ABAQUS software to investigate the behavior of the RBS connection in crooked conditions. It is concluded that the RBS connection is the best choice for beam-to-column connection in crooked conditions.

After a seismic event, the damaged reduced beam sections need to be replaced as soon as possible to recover the damaged buildings to initial ductility and strength capacity. In most cases, however, uneconomical methods are followed because of long and heavy beams. In order to avoid the aforementioned disadvantage, the replaceable RBSs have also been recommended by Özkılıç (Ozkilic, 2020). In the proposed detail, as shown in Fig. 1a, RBS is connected to a column with end plate whereas it is connected to a beam with splice connection. Therefore, thanks to the gap formed between the beams in splice detail, the RBS which is expected to yield under the earthquake load can be separated from the main beam with no need for torch cutting and hydraulic piston. However, the proposed detail does not allow the replacement of the RBS under the residual drift. The disadvantage of the proposed connection detail due to residual drift is aimed to be eliminated with the novel connection detail developed in the scope of this study. The proposed detail provides end-plated connection to connect RBS to column and side-plated connection to connect RBS to the main beam. The side plated connection detail was inspired by the detail used in the mid-splice replaceable link in EBF proposed by Özkılıç et al. (Fig. 1b) (Ozkilic, 2021).

Due to the side plated connection, the proposed RBS detail allows easy replaceability even under the certain amount of residual drift expected after an earthquake (Fig. 2). Theoretically, the bending moment at mid-point of the link where the side plated connection is employed is zero in EBFs. However, in the region where the side-plated connection proposed in this study, the bending moment is close to M_{pr} which is plastic moment capacity of the RBS. Therefore, response of the side-plated connection in MRFs is different than that in EBFs which may affect global performance of MRF where the proposed detail is utilized. In order to investigate seismic response of the replaceable RBS where side-plated connection is used, a numerical study was undertaken. In the proposed side-plated connection detail, slip critical and bearing typed bolted connections and welded connections were taken into account as prime variables. In

order to facilitate easy replaceability, bolted connections were applied as slotted and site drilled. All specimens were subjected to loading protocol mandated by AISC 341-16 (Aisc, 2016). Numerical studies indicated the potential of the proposed replaceable RBS used in MRFs.

2. Replaceable Reduced Beam Section Concept

The proposed connection detail employed to connect the RBS to the main beam presents a replaceable RBS for MRFs. In this detail, in order to provide bolted connection, end plate is welded to one end of the RBS whereas the channel section is welded to the other end of the RBS. Channel sections can be selected from hot-rolled sections or they can be produced as a built-up section. While the replaceable RBS is connected to the column with end-plated connection, it is connected to the main beam with side-plated connection. The gap provided between the main beam and replaceable RBS facilitates easy removal of the damaged RBS after an earthquake. In addition, due to the site drilled, site welded and slotted connection types recommended in the side-plated joint detail, it is possible to install the new RBS easily even under residual drifts. Since residual drifts do not exist in the new construction, bolted connection with standard holes and alternatively slotted holes can be utilized for the side plated connections. While the bolted connection with standard holes can be designed as bearing type and slip critical type, the bolted connection with slotted holes has to be designed as slip critical type to eliminate relative slip between the RBS and the main beam. After a seismic event, site drilled or site welded connection types need to be used for easy replacement of the damaged RBS where standard holes are used in the side-plated connection. On the other hand, as the slot lengths are designed according to the amount of residual drifts of the structure where slotted holes are used in the side-plated connection, the same slotted connection design is used when the damaged RBS is replaced. It is recommended that the slots, which provide easy replacement, should be formed vertically on the side plates and horizontally in the channel sections. However, the bidirectional slot used in the connection causes to reduce the strength of the side-plated connection which in turn increases the number of bolts required. Therefore, since the vertical slots of the side plates are not needed in the design of the new building, it is recommended to use only horizontal slots in the channel sections to increase the strength of the connection. However, even if the side plates are not damaged at all after an earthquake, in order to easily replacement of the damaged RBS under residual drift, the side plates without vertical slots should be replaced with the side plates with vertical slots.

Two different methods can be followed for the channel sections, as shown in Fig. 3. In the first option, the depth of the channel sections should be greater than the flange width of the main beam. Therefore, the bolts are easily installed in the main beams with narrow flange width. In this detail, an eccentricity which is indicated by " r " in Fig. 3b between the side plates and the beam flanges occurs. Yielding may occur in the web of the channel sections due to significant bending moment and shear forces resulted from this eccentricity. In order to circumvent this undesired yielding which in turn pinching behavior of the proposed connection, the stiffeners that connect the flanges of the channel sections are proposed to be used (Fig. 3b). In the second second option, as shown in Fig. 3c, the flanges of the channel sections are welded directly to the flanges of the main beam and the replaceable RBS. Therefore, the eccentricity and the

significant amount of the bending moment caused by this eccentricity in the first option are eliminated. However, this option can only be preferred in the beam sections with wide flanges for easy installation of the bolts used in the side-plated connection.

The limit values of the parameters of “ a ”, “ b ”, “ c ” in the proposed replaceable RBS are similar to the parameters given for RBS defined in AISC 358-16 (Ansi) (Eqn. (1), (2), and (3). Moreover, it is recommended the parameter of “ e ”, indicated in Fig. 3, to be at least 10 mm longer than the flange of the channel section in order to keep the RBS away from heat affected zone occurred during to welding process.

$$0.5b_{bf} \leq a \leq 0.75b_{bf} (1)$$

$$0.65d \leq b \leq 0.85d (2)$$

$$0.1b_{bf} \leq c \leq 0.25b_{bf} (3)$$

3. Side-plated Connection Design

The design of the side-plated connection employed to connect the replaceable RBS to the main beam is based on the probable maximum moment capacity of the RBS computed by using Equation (4). Therefore, unlike the similar connection type used in the mid-spliced end-plated replaceable link concept (Ozkilic, 2021), the connection aforementioned is subjected to a significant amount of bending moment. The probable maximum moment at the center of the RBS M_{pr} is calculated as follows:

$$M_{pr} = C_{pr} R_y F_y Z_{RBS} (4)$$

Where, C_{pr} is a coefficient and equal to $(F_y + F_u)/2F_y \leq 1.2$, R_y is the ratio of expected yield stress to specified minimum yield stress, F_y is the yield stress and Z_{RBS} is the plastic section modulus at the center of the reduced beam section and calculated as follows:

$$Z_{RBS} = Z_x - 2ct_{bf}(d - t_{bf}) (5)$$

Where, Z_x is the plastic section modulus about x-axis for full beam cross section

The bending moment, M_s acting on the connection is computed by using the geometrical relationship of the moment diagram depicted in Fig. 4.

The side-plated connection is also subjected to the shear force obtained by the required shear force at the center of the RBS which is calculated by Equation (6).

$$V_{RBS} = \frac{2M_{pr}}{L_h} + \frac{wL_h}{2} (6)$$

where, w is uniform beam gravity load that is calculated by considering the load combination of $1.2D+0.5L+0.2S$.

In summary, the required bending moment M_s and the required shear force V_s shown in the free body diagram of the proposed connection region (Fig. 5) are calculated by using Equation (7) and (8), respectively.

$$M_s = M_{pr} - V_{RBS} s_s \quad (7)$$

$$V_s = \frac{2M_s}{L_h} + \frac{w(L_h - s_s)}{2} \quad (8)$$

where s_s is the distance from the center of the side-plated connection to the center of the RBS.

In order to translate the bending moment and the shear force produced on the main beam to the column safely, the side plates, the channel sections and their connections need to be designed under M_s and V_s in accordance with the rules defined in AISC 360-16. The moment strength and shear strength of the side plates subjected to both M_s and V_s need to be calculated based on AISC 360-16 Chapter F11 and Chapter J4.2 respectively. If an eccentricity occurs between the side plates and the beam flanges, as shown in Fig. 3b (Option A), the channel sections need to be designed under shear forces and both in-plane and out-of-plane bending moments resulting from this eccentricity. On the other hand, if the flanges of the channel sections are welded directly to the beam flanges, as shown in Fig. 3c (no eccentricity, Option B), the channel sections only need to have a minimum flange thickness from plate bearing and tear-out strength standpoint. The bolted or welded joint detail used to connect the side plates to the channel sections need to be designed as per AISC 360-16 Chapter J. Single, double row or staggered bolt configurations are the alternatives of the bolt connections with bearing type and slip critical types. While standard holes are the only alternative of the connection where bearing type connection is employed, both standard holes and slotted holes may be used in the slip critical type connection. The design flowchart for the design of the bolted connection is depicted in Fig. 6.

On the other hand, the design of the end-plate connection can be performed according to AISC 358-16. In this study, the thick end-plate approach where end-plate behaves as elastic is considered. The step by step design guideline can be found in The Design Guide 04: Extended End-Plate Moment Connections Seismic and Wind Applications prepared by Murray and Sumner (Murray and Sumner, 2003). It should be kept in mind that the thickness of the end-plate, which affects the weight of the replaceable part, can be decreased using different approaches (Özkılıç, 2021a, b, c, e; Özkılıç and Topkaya, 2021b).

4. Numerical Study

The use of finite element programs has been increased with the development of computer technology. The parametric numerical analyses were conducted using the finite element tool, ABAQUS. Geometric and

material nonlinearities were taken into account. Different numerical models were generated to prove the concept of the proposed connection.

4.1. Finite Element Modeling Details

All members were modeled utilizing C3D8R elements, eight-node brick elements with reduced integration. Four layers of elements were applied for all members in order to accurately simulate buckling and bending behavior (Ozkilic, 2020). Finer mesh size was selected for the proposed connection while coarser mesh size was applied for the rest of the beam. The mesh configuration is shown in Fig. 7.

The column was modeled explicitly. A region of the column flange where the end-plated bolted to the column was implanted. All degrees of freedom were restrained at lower surface of the column face. The interactions between the side plate and bolts, channel section and side plate, channel section and bolts, end-plate and bolts were simulated by defining surface-to-surface interaction. Multi-point constraint (MPC) was defined to a reference point located at center of the beam end. The loading was applied through this point. Cyclic loading based on the loading protocol mandated by AISC exposed to the all beams. A lateral bracing was applied to the flanges of beams where 250 mm away from the connection. Tangential behavior with the coefficient of friction (μ) of 0.3 was considered. Welding was not implanted explicitly; instead, tie constrain was utilized to connect the welded members. The pretension force was applied utilizing “Bolt Load” option in ABAQUS.

Nonlinear isotropic and kinematic hardening material model was introduced based on the studies of Elkady and Lignos (Elkady and Lignos, 2015) and Özkılıç (Ozkilic, 2020). Nonlinear isotropic and kinematic hardening parameters were defined using Eqns. (9-10).

$$\alpha = \frac{C}{F_y}(\sigma - \alpha)\varepsilon_{pl} - \gamma\alpha\varepsilon_{pl} \quad (9)$$

$$\sigma = F_y + Q_\infty(1 - e^{-b\varepsilon_{pl}}) \quad (10)$$

where α is the backstress, C is the initial kinematic hardening modulus, F_y is yield stress of plate, σ is the equivalent yield stress at zero plastic strain, ε_{pl} is cumulative plastic strain, γ is the rate at which C decreases with cumulative plastic strain ε_{pl} , Q_∞ is the maximum change in the size of the yield surface and b is the rate at which the size of the yield surface changes as plastic deformation develops.

Fracture was not modeled explicitly, instead, response indices were utilized to assess the level of potential fracture. Rupture index (RI) is one of the response indices frequently preferred for this purpose. The location where large values of RI are accumulated indicates the location of the greatest potential fracture (Mao et al., 2001). Although RI is not a direct indication for fracture initiation, different configurations can be compared to evaluate which one has the highest potential of the fracture (Bozkurt et al., 2019; Ozkilic, 2020, 2021; Özkılıç, 2021d). Rupture index is indicated in Eq. (11) (El-Tawil et al., 1999).

$$RI = \frac{PEEQ}{\exp \left(-1.5 \frac{p}{q} \right)} \quad (11)$$

where $PEEQ$ is the plastic equivalent strain and calculated as ratio of effective plastic strain to yield strain. $PEEQ$ measures local plastic strain demand and higher value of $PEEQ$ indicates potential damage and vulnerability. p and q are hydrostatic pressure and von Mises stress, respectively. The calculation of $PEEQ$, p and q are given in Eqs. (12-14).

$$PEEQ = \sqrt{\frac{2}{3} \varepsilon_{ij}^p \varepsilon_{ij}^p} \quad (12)$$

$$p = -\frac{1}{3} \text{tr}(\sigma_{ij}) = -\frac{1}{3} \sigma_{ii} \quad (13)$$

$$q = \sqrt{\frac{3}{2} S_{ij} S_{ij}} \quad (14)$$

where ε_{ij}^p stand for the plastic strain in direction i and j , σ_{ij} is Cauchy stress and S_{ij} is deviatoric stress.

4.2. Verification

In order to verify the assumptions in the numerical section, an experiment on reduced beam section conducted by Özkılıç (Ozkilic, 2020) was simulated. The specimen was HEA240 section with a length of 1650 mm. The specimen had a of 152 mm, b of 166 mm, c of 45 mm and R of 94 mm. The specimen was laterally supported and connected to the column with the use of a bolted end-plated connection. Cyclic loading protocol mandated by AISC 341 exposed to the specimen. The specimen successfully completed 0.06 rad rotation and failed at 0.07 rad rotation due to excessive buckling of flanges. The hysteresis behavior of the experimental and numerical results are compared in Fig. 8. It is seen that the numerical model captured strength degradation due to buckling of the flanges. Moreover, the load difference between numerical results and experimental findings is less than 3%. Fig. 9 demonstrates the failure modes observed in the numerical model and experimented specimen. The numerical model successfully simulated the buckling behavior of the flanges.

4.3. Validation of Proposed Connection Concept

The numerical models were exposed to cyclic loading according to the AISC 341. Six cycles were repeated at 0.00375 rad, 0.005 rad, and 0.0075 rad, then four cycles were repeated at 0.01 rad, later two cycles were repeated at 0.015 rad and 0.02 rad. Following cycles were repeated two times with increments of 0.01 rad. A beam should complete 0.04 rad and sustain a moment at the column face above 0.8 times the moment capacity of the beam in order to satisfy AISC 341 criteria.

For this section, section of HEA240 was utilized. Reduced section had a of 120 mm, b of 150 mm and c of 60 mm. Lateral support was utilized after reduced section. Initially, reference beam without any proposed connection was analyzed. Here, the reference beam was utilized with and without end-plate connection and hysteresis behavior of them were compared in Fig. 10. On the other hand, Fig. 11 illustrates PEEQ and RI distributions of the reference beam. It is seen that these beams exhibited similar seismic behavior. These beams completed 0.08 rad rotation and failed at 0.09 rad rotation since the moment reduced 0.08 moment capacity.

In the scope of this study, bearing and slip-critical typed connections were utilized for the numerical study. For bearing typed connections, straight and staggered bolted connections were adopted for mid-spliced connections. Standard sized holes and vertical and/or horizontal slotted holes were investigated for slip-critical connections. Moreover, the model with slip-critical connection with residual drifts was studied as well. Hysteresis behaviors, PEEQ and RI distributions are compared in the following sections.

4.4. Response of the Bearing Typed Connections

4.4.1. Straight Bolt Configuration

Five different numerical models were investigated for the bearing typed connections with straight bolted connection, in which details are given in Fig. 12. Model A represents mid-spliced connection without stiffener. Stiffeners were included in Model B. On the other hand, channels were directly welded to the flanges of the beam for Model C. For Models D and E, the stiffeners were welded above the channel. Moreover, distance of “ e ” was kept as 30 mm for Model E in order to reduce the weight of the replaceable part. Fig. 13 compares the hysteresis behavior of these models. The results showed that Model A, which does not have stiffeners, exhibited poor seismic performance when compared to the others. The reason for the inadequate performance can be explained with RI and PEEQ distributions which are shown in Fig. 16. Unlike Models B and C, Model A produced higher RI and PEEQ values at the connection of beam-to-channel. The channels were yielded followed by buckling due to the moment transferred by the main beam and this led to the transition of the damage from RBS to channel web. This problem was mitigated by two solutions. In the first approach which represents Model B, stiffeners were employed by making use of welding between channel flanges and web in order to prevent web buckling of the channels. In the second one referring Model C, the channel sections were welded directly to the beam. These two alternative connection details exhibited similar behavior. However, lower RI values were observed at the connection of beam-to-channel for Model B, which in turn caused a reduction in RI distributions at RBS compared to Model C. Buckling of RBS was detected at Model C which led to strength degradation at the last cycles. The effect of the stiffener location on the seismic response of the RBS were examined in Model D. In this Model, the stiffeners were placed above the channel in order to provide an easy erection. However, shifting the stiffeners away from the beam flanges caused the shifting of the damage from RBS to the channel section which led to a reduction in the capacity. Model D and Model E are identical except that the length of the RBS in Model E was reduced in order to reduce the weight of the replaceable RBS. Since the proposed connection detail in Model E was subjected to more bending moment compared to

the detail in other Model D, more poor seismic performance were obtained which indicated as the required bending moment increases, the seismic performance reduces.

4.4.2. Staggered Bolt Configuration

In order to decrease the height of the proposed connection detail, a staggered bolt configuration may be employed. Four different numerical models were investigated for bearing typed connections with the staggered bolted connection. Model A represents mid-spliced connection without stiffener whereas Model B includes stiffeners between flanges and web of channels. On the other hand, channels were welded to the beam for Model C. For these three models, the height of the proposed connection is considered higher than the beam depth. In other words, one row bolts were included outer of the beam height due to increase the moment capacity of the connection. Model D is identical to Model C except that the bolts were kept within the beam height in order to come up with compact connection detail. The details of these models are depicted in Fig. 15.

Figure 16 compares the hysteresis behavior of models having bearing typed connections with staggered bolted configuration. RI and PEEQ distributions are illustrated in Fig. 17. Although RBS members in Model B and C were buckled after 0.06 rad rotation, they exhibited excellent seismic performance by completing 0.09 rad rotation, which is much greater than the minimum rotation of 0.04 rad defined in AISC 341, without any strength degradation. These models dissipated significantly higher energy than that of Models A and D. Due to the absence of the stiffeners in Model A, buckling of the channel was observed, which led to inadequate performance compared to Models B and C. The highest RI and PEEQ values were detected at the beam-channel connection. Adding stiffener or welding channel to the beam circumvent this problem. On the other hand, Model D performed a pinched behavior. Due to insufficient moment capacity, spliced connection detail exhibited in-plane rotation during the loading, which caused significant stiffness degradation.

4.5. Response of the Slip-Critical Typed Connections

For slip-critical typed connection, both standard-sized holes and slotted holes were investigated. Forty M24 bolts were utilized for mid-spliced connection. The details of the slip-critical typed connection detail with standard-sized holes are displayed in Fig. 18. Three different numerical models were studied. No stiffener was utilized in Model A while stiffeners were introduced to Model B. On the other hand, flanges of the channel section were directly welded to the beam in Model C.

Hysteresis behaviors of the numerical models are compared in Fig. 19 while PEEQ and RI distributions are shown in Fig. 20. Although Models B and C performed satisfactory seismic behavior, Model A showed poor performance due to buckling of the channel. A slight strength degradation due to buckling was observed at Models B and C but at the last cycle Model C exhibited significant strength degradation. No PEEQ and RI values were observed at the connection of channel-to-beam for Models B and C.

Three models were created to investigate the behavior of slip-critical with slotted holes connections providing easy replacement of damage RBS under residual drift. Models A and B represent the mid-

spliced connection with only vertical slots at side plates. Unlike Model A, for Model B, 0.5% residual drift was applied for the same beam at Model A. On the other hand, horizontal slots were included in channels for Model C. For these models, forty M27 bolts were utilized. The details of the models are depicted in Fig. 21.

Hysteresis behaviors of the numerical models with slotted holes are compared in Fig. 22 while PEEQ and RI distributions are shown in Fig. 23. All three models exhibited almost similar seismic behavior. After 0.06 rad rotation, strength degradation due to buckling of the reduced section was observed for all three models. All models successfully completed 0.09 rad rotation.

5. Discussion Of The Analysis Results

The energy dissipation capacities and moment capacities of the proposed numerical models were normalized by those of the reference beam without end-plate connection. Fig. 24 demonstrates the normalized energy dissipation and moment capacities. Based on the obtained findings, stiffener placed near the flanges of the beam contributes to increasing the energy dissipation and moment capacities of the RBS with the proposed detail. The absence of stiffener resulted in stress concentration at the beam to channel connections which in turn causes local buckling of webs of channel sections. As an alternative to stiffeners, flanges of channel sections may be directly welded flanges of beam members. This approach also decreases the weight of the replaceable part of the proposed connection detail. Even though slip-critical typed connection increases the cost of the proposed connection detail, the RBS with the slip-critical typed connection has a higher energy dissipation capacity than the RBSs with bearing typed connection. Vertical and horizontal slots may be utilized for easy replacement of the damage RBSs under residual drift.

6. Conclusions

In this study, a connection detail in order to provide easy replacement of damaged RBS under residual drift taken place after an earthquake was proposed. The potential of the proposed connection detail was validated by performing a series of numerical analyses. Usage and location of the stiffener, type and configuration of the bolted connection are considered as prime variables. According to analyses, it should be stated that not only stiffeners but also their location plays an important role in terms of energy dissipation and moment capacity. If the stiffeners are not used, it is recommended to weld the flanges of the channel sections directly to the flanges of the beams. Staggered bolt configuration may be preferred to be used in order to reduce the number of the bolts used in the proposed connection detail. Numerical analyses employing PEEQ and RI as indicators showed that the proposed connection detail, where either bearing or slip-critical typed connection was used, satisfied all requirements mandated by AISC 341-16.

Declarations

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Competing Interests

The authors confirm that there are no known conflicts of interest associated with this publication and there has been no financial support for this work that could have influenced its outcome.

Author Contributions

Yasin Onuralp Özkılıç: Software, Formal Analysis, Writing - original draft, Writing - review & editing, Conceptualization, Methodology.

Mehmet Bakır Bozkurt: Formal Analysis, Writing - original draft, Writing - review & editing, Conceptualization, Methodology.

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Figures

Figure 1

(a) The replaceable RBS with splice connection proposed by Özkılıç (Ozkilic, 2020) and (b) the mid-spliced end-plated replaceable link proposed by Özkılıç et al. (Ozkilic, 2021)

Figure 2

The proposed replaceable reduced beam section

Figure 3

Details of the replaceable reduced beam sections a) Technical drawing b) Option A c) Option B

Figure 4

Moment distribution of the beam section

Figure 5

Free-body diagram between center of side-plated connection and face of column

Figure 6

Flowchart for the design of bolted connections for side plates

Figure 7

Mesh configuration.

Figure 8

Comparison of hysteresis behavior

Figure 9

Comparison failure modes

Figure 10

Comparison of hysteresis behavior for reference beam

Figure 11

Figure 12

Details of bearing type connection with straight bolt configuration.

Figure 13

Comparison of hysteresis behavior for bearing typed connections with straight bolted connection

Figure 14

RI and PEEQ distributions for bearing typed connections with straight bolted connection

Figure 15

Details of bearing type connection with staggered bolt configuration.

Figure 16

Comparison of hysteresis behavior for bearing typed connections with staggered bolted configuration

Figure 17

RI and PEEQ distributions for bearing typed connections with staggered bolted connection

Figure 18

Details of Models having slip-critical typed connection with standard-sized holes.

Figure 19

Comparison of hysteresis behavior for slip-critical typed connections with standard sized holes

Figure 20

RI and PEEQ distributions of Models having slip-critical typed connection with standard sized holes.

Figure 21

Details of Models having slip-critical typed connection with slotted holes.

Figure 22

Comparison of hysteresis behavior for slip-critical typed connections with slotted holes

Figure 23

PEEQ and RI distributions of models having slip-critical typed connection with slotted holes.

Figure 24