Design approach to bolted end-plate vertical-slits RWS connection

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ABSTRACT

Extensive research has been carried out on steel moment frames to improve the cyclic performance of seismic resisting connections with reduced beam section (RBS). The RBS connections are conventionally known by the radial reduction of the beam flange. Where the contribution of the beam flange to the flexural resistance is greater than that of the beam web, some researchers have proposed reduced web section (RWS) connections, instead. The present study dedicates to the RWS connections with vertical-slits (VS), as a cost-effective alternative with multiple design parameters. This paper aims to obtain proper ranges for the geometric design parameters of the VS-RWS connection. In this order, two full-scale specimens of the bolted end-plate VS-RWS connection were experimentally tested under the SAC cyclic loading to evaluate the performance of connections, and then a parametric study was carried out using the verified numerical models. The parameters consist of the distance between the column face and the beginning of the reduced region, the length of the reduced region, as well as the depth and width of the vertical-slits. Based on the results, certain recommendations for the ranges of the geometric parameters of VS-RWS have been suggested. In order to obtain the story drift of the frame caused by the VS-RWS beam flexural deformation using the conjugate beam method, the original VS-RWS was replaced with an equivalent constant-cut reduced beam section (CC-RBS). At last, a simple design procedure for VS-RWS connections was provided according to AISC-358.

Keywords: Experimental behavior, Parametric study, Bolted connection, Reduced web section, Vertical-slits.

1. Introduction

Prior to 1994, the moment resisting frame system had been widely used in the construction industry in seismic zones. Due to the Northridge and Kobe earthquakes in 1994 and 1995, many traditional moment connections performed poorly in terms of providing the expected ductility, and the connections were subject to a brittle fracture. One of the ways to improve the performance of these connections is to weaken the beam section, based on the principle of the strong column-weak beam by reducing the beam flange or web to form a plastic hinge in the reduced region. This region, which acts as a fuse, creates some proper ductility to prevent the brittle damage to the connection. Since the web of the beam makes a small contribution to
its flexural rigidity, the weakening of the beam web instead of the flange can be a good option for reducing the beam cross section as long as it provides adequate shear capacity.

In some types of reduced web section (RWS) connections, the beam web is weakened by creating a rectangular, circular, or vertical elliptical opening ([1-5]). In these connections, a relatively large void is required to reduce the stresses over the beam-to-column CJP welding region, yet the large void causes the local buckling instability at the cutting edges of the beam web [1, 4]. In order to overcome this instability, some researchers have proposed and investigated the accordion/tubular-web RWS connections in which the flat web is replaced by a corrugated-web [6-9]. Nadi et al. [8] showed that the Holed Tubular-Web (HTW) which is a kind of accordion-web, decreases the fatigue phenomenon caused by the stress concentration in the connection of the tube and beam. Besides, Zahraei et al. [6] demonstrated that the tubular-web RWS provides a better condition than the accordion-web RWS in terms of lateral-torsional buckling stability of the beam and low-cycle fatigue. In any case, although this type of connection behaves well in terms of plastic hinge formation in the reduced region, its construction is relatively difficult, costly, and time-consuming.

In addition to local instability, another drawback of the large openings is the early failure of the beam section due to the significant effect of the shear-flexure interaction in the T sections remaining at the top and bottom of the large opening. Therefore, in order to reduce the depth of the web perforation, Davarpanah et al. [10, 11] introduced and studied the horizontal elliptical-shaped RWS connections. They suggested ranges for the purpose of choosing the geometric variables of the elliptical-cut and the reduced region to ensure that the formation of the plastic hinge is away from the column edge. Their research also illustrated that reducing the web instead of the flange reduces the lateral-torsional instability greatly.

Some researchers turned toward the RWS connections with a multi-hole web instead of a single large-opening web [12-14]. Maleki and Tabbakhha [12] studied the reduced beam section (RWS) connections with the slotted beam web named slotted-web-reduced-flange (SWRF) connection. Hedayat and Celikag [13] proposed RWS connections with two parallel horizontal voids in the beam web and presented a step-by-step design process for this purpose. In order to prevent the beam flange/web buckling, they had to add web stiffeners and two tubes in the center of voids. In a recent study, Nazaralizadeh et al. [14] have introduced a new type of the RWS connections with vertical-slits (VS) (Fig. 1) as an easy-to-build alternative that has multiple geometrical parameters for the control of the connection behavior. Their comparative study of the bolted end-plate VS-RWS connection with the conventional RBS
and with various cut-shaped RWS connections exhibits good ductility and flexural strength, as well as low-localized stresses in the end-plate welding line of the proposed connection.

This research aims to perform experimental tests, numerical validation, and parametric study to obtain the knowledge required to control the performance of VS-RWS connections and to find the optimum ranges for the geometrical parameters of the reduced region using a design approach. This paper is structured as three following parts. The first part is dedicated to the experimental tests and performances of two full-scale VS-RWS connection specimens with different configurations of the reduced region. The second part is devoted to the development of a finite element model (FEM) for the VS-RWS connection to simulate the cyclic response of various VS-RWS connections with different geometrical parameters of the reduced region. Based on the results, the optimum ranges for the RWS region parameters are recommended. In the third part, the original VS-RWS is replaced with an equivalent constant-cut depth RBS (CC-RBS) to obtain the story drift of the frame caused by the VS beam flexural deformation using the conjugate beam method. Last of all, for the design purpose, a simple design procedure for VS-RWS connections according to the optimum ranges is provided for the reduced region.

![Fig. 1. Configuration of VS-RWS connection](image)

### 2. Experimental process

#### 2.1. Test specimens

The beam and column profiles of the experimental test specimens were taken from the seventh floor of a 10-story library building in a high-seismic region. The structural building was analyzed and designed by ETABS \[15\] under the gravity and lateral loading. The first-story height was 4.5 m and the other stories were 3 m. The dead and live loads were 0.5 and 0.3 t/m² respectively. The lateral load resisting system was chosen as a special moment frame. Based on the design, the IPE270 and IPB200 profiles with nominal yield and ultimate stresses of 240 and 370 MPa were employed for the beam and column, respectively. The lengths of the specimen beam and column in experimental tests were considered as 1460 and 1075 mm,
respectively, according to the previous studies [16-18]. The connection details, including the end-plate, its bolts and holes, the web plates, the continuity plates, and the welds were designed according to the LRFD method. Two web plates were installed on either side of the column web, and the continuity plates were welded to the web plates and column flanges. Thus, the panel zone could establish the strong-column-weak-beam strategy [17].

In this study, two full-scale bolted end-plate RWS connections with vertical-slits named VS-RWS1 and VS-RWS2 were built and tested. The details of the test specimens have been provided in Fig. 2. In this figure, "a", "b", "c"", and "g" are the distance between the column face and the beginning of the reduced region, the length of the reduced region, half of the vertical-slit depth, and the total cutting size along the beam length for a single cut or multiple discontinuous cuts, i.e. b ≥ g, respectively. The reduced region parameters, "a" and "b", for the case of VS-RWS1, as well as "b" for the case of VS-RWS2 were selected based on the criteria required by FEMA-350 [19] for conventional RBS connections (0.5bf < a < 0.75bf, 0.65db < b < 0.85db). The assembly operation of the specimen components, including the welding, the drilling by CNC (Computer Numerical Control), and the bolts pre-tensioning were performed in the factory to ensure the quality of the test specimens.

![Fig. 2. Characteristics of the tested specimens](image-url)
The mechanical properties of materials were determined according to ASTM A370 [20] standard tensile tests [14] (Table 1). In many studies on connections [1, 18, 21-23] the cyclic loading is based on the SAC [24] protocol; hence, in the present study, this protocol has been used [14]. The quasi-static cyclic load has been applied to the free-end of the beam by the displacement control method at a rate of 0.02 [16].

| Coupon            | Elasticity modulus (GPa) | Yield stress (MPa) | Ultimate stress (MPa) |
|-------------------|--------------------------|--------------------|-----------------------|
| Beam web          | 172                      | 243                | 336                   |
| Beam flange       | 181                      | 277                | 405                   |
| Column web        | 192                      | 268                | 392                   |
| Column flange     | 194                      | 301                | 420                   |
| End-plate         | 199                      | 330                | 445                   |

2.2. Test setup

The experimental tests were conducted in the structural lab of Isfahan University of Technology. The test setup and specimens are shown in Fig. 3. In this setup, the column is as a horizontal member, the two ends of the column have been welded to the plates with dimensions of 300×300×15 mm³, and these plates are connected to the setup chassis with eight high-resistance bolts (M27,10.9) in such a manner that the desired support conditions are provided. To verify that the supports are sufficiently rigid, a displacement gauge with the 0.01 mm accuracy was placed on the back of the columns attached to the chassis. To apply the cyclic load to the beam end in a horizontal direction, a hydraulic jack with a load capacity of 300 kN and a displacement capacity of 90 mm was employed. To obtain the values of the beam end displacement due to the cyclic load, a linear potentiometer transducer (LPT) at a distance of 1070 mm from the column face was placed on the beam. The applied load and displacement were measured and recorded by a digital data logger.
3. Experimental results

3.1. Overall behavior

Table 2 describes the overall behavior of the experimental test specimens at different rotation angles. As can be seen, in the VS-RWS1 connection, the proper performance of the reduced region prevents any damage to the welding line of beam to end-plate, and no significant deformation can be observed at the location of the bolts. In the VS-RWS2 connection, although the reduced region puts an obstacle to the damage to the beam-to-column junction, the connection cannot provide the appropriate cyclic performance due to the early rupture and shear failure in the beam slits (Fig. 4).

Table 2. Overall behavior of experimental test specimens

| Rotation angle (rad) | VS-RWS1 | VS-RWS2 |
|---------------------|----------|----------|
| 0.03                | No significant deformation was observed. | Some of the slits were deformed and the yellow color layer around the slits was flaky. |
| 0.04                | A slight deformation was observed at the beam flange. | A local buckling occurred at the beam flange, small tears were observed at the strips between some of the slits, and therefore the flexural strength was reduced. |
| 0.05                | A slight deformation was seen around some slits. Also, a local buckling were occurred at the beam flange and web in the RWS region. | The buckling instability of the beam was increased, and some slit strips began to rupture. |
| 0.06                | The beam deformations were increased due to the local buckling, such that the out-of-plane deformation at the web around the upper and lower slits was clearly visible. The end-plate began a slight deformation. | The rupture in the slit strips was expanded toward the beam flanges. |
| 0.07                | Due to the large deformation of the beam, some slits were damaged. The rupture occurred in the upper and lower slits and extended obliquely toward the beam flange. | A full shear rupture occurred at the slit strips. |
3.2. Bilinear curves

In this paper, the ductility ratio and effective stiffness have been calculated according to FEMA440 [25, 26]. For this purpose, at first the backbone diagram was drawn using the envelope of the hysteresis curve. Then an equivalent bilinear curve was fitted to the backbone diagram (Fig. 5). Only the ascending segment of the hysteresis curve was used to obtain the backbone diagram and the decreasing portion was ignored ("m" is the slope of the "d" line and is positive). To equalize the two diagrams based on the concept of equivalent energy, the area underneath the bilinear diagram must be equal to that of the backbone diagram. This means that the area of the yellow and red hatched parts would be equal in Fig. 10. The point C in this figure is considered as a yield point. By using these diagrams, the yield flexural moment ($M_y$) and the yield rotation angle ($\theta_y$), as well as the ultimate flexural moment ($M_u$) and the ultimate rotation angle ($\theta_u$) of the tested connections are determined and indicated in Table 3.

Fig. 4. Deformation and PEEQ of the connections at different rotation angles; a) VS-RWS1; and b) VS-RWS2
According to the available data, the effective stiffness of the connections as the line slope of the elastic zone in the bilinear diagram and the ductility ratio for the two connections are presented in Table 3. This table shows that both the ductility ratio and the effective stiffness of the VS-RWS1 are higher than those of the VS-RWS2.

![Fig. 5. Schematic diagram for backbone and bilinear curves](image)

Table 3. Ductility ratio and effective stiffness of tested specimens

| Specimens | $\theta_y$ (%) | $M_y$ (kN.m) | $\theta_u$ (%) | $M_u$ (kN.m) | $\mu = \theta_u / \theta_y$ | $k_e = M_y / \theta_y$ (kN.m) |
|-----------|----------------|--------------|---------------|--------------|---------------------------|-----------------------------|
| VS-RWS1   | 1.47           | 89.7         | 4.98          | 142.3        | 3.39                      | 6102                        |
| VS-RWS2   | 0.98           | 56.5         | 2.97          | 104.2        | 3.03                      | 5765                        |

3.3. Lateral-torsional buckling of beam

The lateral-torsional buckling phenomenon in the connections has a great effect on their cyclic performance; thus, this section is devoted to the investigation of this phenomenon in the proposed connection. One of the disadvantages of the common RBS connections is the large lateral deformation in the beam [27, 28]. The experimental observations of the two tested connections showed that although the beam is weakened by vertical-slits, as can be seen in Fig. 6, at the end of the tests, the significant lateral-torsional buckling does not occur in the connections, which is one of the important advantages of connections of this type.
3.4. Stiffness degradation

Many structural steel components will exhibit some level of stiffness degradation when subjected to cyclic loading. To evaluate the stiffness degradation of connections under cyclic loading, the stiffness degradation coefficient ($S_j$) can be utilized as follows [22]:

$$S_j = \frac{\sum_{i=1}^{n} p_i^j}{\sum_{i=1}^{n} u_i^j}$$  \hspace{1cm} (1)

where $p_i^j$ is the maximum load applied at the beam tip in the $i^{\text{th}}$ cycle; $u_i^j$ is the maximum rotation of the $i^{\text{th}}$ cycle; and "n" is the number of loading cycles. The diagram of $S_j$ versus the rotation angle ($\theta$) at different rotations is plotted in Fig. 7. According to this figure, by increasing the rotation angle, the stiffness degradation coefficient on the positive and negative sides is decreased. In addition, the flexural stiffness of VS-RWS2 is lower than that of VS-RWS1. This originates from the local buckling and deformation of the VS-RWS2 beam.
4. Numerical analysis and verification

The bolted end-plate RWS connections with vertical-slits were simulated in ABAQUS [29]. To achieve the accurate numerical results, it was necessary to include the slipping phenomenon in the modeling of the bolted junctions by introducing the contact interaction between the surfaces of the components. Fig. 8 shows the interacting surfaces and the meshing details of the connection model. The details of FEM modeling, including the element type, the geometric and material characteristics, the analysis type, the bolt pre-tension force, the type of contacts and the mesh size, were in accordance with Ref. [14].

Fig. 9 compares the hysteresis curves of the numerical models with those of the experimental specimens. As can be seen, there is acceptable agreement between the results. As can be observed in this figure, in the hysteresis curve of the VS-RWS1 connection up to a rotation angle of 0.05 rad, the drop is not observed and the maximum bending moment value occurs at this rotation angle, which is equal to 142.3 kN. Since then, the dropping in the moment-rotation curve has begun and the bending moment has decreased by increasing the rotation angle; however, hysteresis loops have good stability. As a result of the formation of a plastic
hinge in the predicted region, the proper bending capacity and the energy dissipation, the connection showed acceptable seismic behavior. Since the drop in the flexural strength of the VS-RWS1 connection at 0.04 rad is less than 20% of the cross-section plastic moment, this connection has met the requirements of the special bending frames in AISC-358 [30]. According to Fig. 9, the flexural capacity of the VS-RWS2 connection is equal to 104.2 kN, which is about 37% less than that of the VS-RWS1 connection. The bending moment of VS-RWS2 connection at 0.04 rad is less than 80% of the cross-section plastic moment. Accordingly, it lacks the AISC-358 [30] criteria for being utilized in special moment frames.

Fig. 9. Comparison of experimental and numerical hysteresis curves; a) VS-RWS1; and b) VS-RWS2

The numerical and experimental deformation of the connections is depicted in Fig. 4 and Fig. 10. As can be seen, the numerical models have been able to give an appropriate estimate of the tested specimens deformation. Furthermore, the distribution of the equivalent plastic strain (PEEQ) at various rotation angles and the stress distribution of the von-Mises stress at the rotational angle of 0.07 rad are indicated in Fig. 4 and Fig. 10, respectively. Apparently, the location of the occurrence of rupture around the slits in the tested specimens corresponds to the location of the maximum strain or stress in the numerical models. The numerical results show that as expected, the plasticity is limited to the reduced region of the beam (Gray region in Fig. 10), thereby protecting the panel zone, and preventing the welding damage.

Fig. 10. Deformation of experimental and numerical models, as well as distribution of von-Mises stress
5. Parametric study

To achieve the appropriate ranges for the reduced region parameters, including "a", "b", "c''" and "g", various numerical analyses were conducted. Each parameter is introduced as a percentage of the beam depth ($d_b$). It should be mentioned that the design codes have not already recommended any criterion for these parameters of the RWS connections [2]. According to Refs. [13, 31], the plastic hinge should be formed at a distance approximately equal to the beam depth from the column face.

The numerical simulation comprises thirty models, which are categorized into five groups (Table 4). In Groups I, II and III, the parameters "a", "b", and "c''" are variable, respectively, and the other parameters are fixed constants like those of the VS-RWS connection. In Groups I, II and III, the variables "a", "b", and "c''" are assumed within the ranges of 25-65%$d_b$, 40-90%$d_b$ and 10-35%$d_b$, respectively (in the VS-RBS1, $a=37.5%d_b$, $b=80%d_b$ and $c'=16.3%d_b$). Since the quantity of "c''" affects the plastic section modulus, this parameter is expected to have a significant effect on cyclic behavior of the VS-RWS connection. In Group IV, the parameters "a", "b" and "c''" are similar to the VS-RWS1 specimen, and the parameter "g" is variable within a range of 10-100%$b$ (equal to 8-80%$d_b$). In order to consider the simultaneous effects of "g" and "c''", the Group V models were investigated. In this group, both the parameters "g" and "c''" are variable, and the parameters "a" and "b" are considered according to the VS-RWS1 specimen. To name the models, at first, the group number was used and then the variable parameter was introduced as a percentage of beam depth ($d_b$). For example, a model of the second group in which the parameter "b" is 70% of the beam depth is regarded as II ($b=70%d_b$). The Bolted ordinary rigid connection (ORC-B) is a non-reduced model as a reference; and the models in which "g" is equal to 100%$b$ have a rectangular opening. To examine the effect of the mentioned parameters, the hysteresis curve, the PEEQ, the rupture index, the end-plate deformation, and the dissipated energy of the connection models were studied and compared. Hence, in the design approach, the proper ranges for the parameters would be recommended.
Table 4. Numerical connection models

| Group | Name | a (mm) | b (mm) | c’ (mm) | g (mm) |
|-------|------|--------|--------|---------|--------|
| Original | ORC-B | - | - | - | - |
| (I) | I(a=25%d_b) | 67.5 | 216 | 43.9 | 86.4 |
| | I(a=35%d_b) | 94.5 | 216 | 43.9 | 86.4 |
| | I(a=45%d_b) | 121.5 | 216 | 43.9 | 86.4 |
| | I(a=55%d_b) | 148.5 | 216 | 43.9 | 86.4 |
| | I(a=65%d_b) | 175.5 | 216 | 43.9 | 86.4 |
| (II) | II(b=40%d_b) | 101.3 | 108 | 43.9 | 54 |
| | II(b=50%d_b) | 101.3 | 135 | 43.9 | 54 |
| | II(b=60%d_b) | 101.3 | 162 | 43.9 | 64.8 |
| | II(b=70%d_b) | 101.3 | 189 | 43.9 | 75.6 |
| | II(b=80%d_b) | 101.3 | 216 | 43.9 | 86.4 |
| | II(b=90%d_b) | 101.3 | 243 | 43.9 | 97.2 |
| (III) | III(c’=10%d_b) | 101.3 | 216 | 27 | 86.4 |
| | III(c’=15%d_b) | 101.3 | 216 | 40.5 | 86.4 |
| | III(c’=20%d_b) | 101.3 | 216 | 54 | 86.4 |
| | III(c’=25%d_b) | 101.3 | 216 | 67.5 | 86.4 |
| | III(c’=30%d_b) | 101.3 | 216 | 81 | 86.4 |
| | III(c’=35%d_b) | 101.3 | 216 | 94.5 | 86.4 |
| (IV) | IV(g=10%b=8%d_b) | 101.3 | 216 | 43.9 | 21.6 |
| | IV(g=50%b=40%d_b) | 101.3 | 216 | 43.9 | 64.8 |
| | IV(g=70%b=56%d_b) | 101.3 | 216 | 43.9 | 108 |
| | IV(g=90%b=72%d_b) | 101.3 | 216 | 43.9 | 151.2 |
| | IV(g=100%b=80%d_b) (Rectangular opening) | 101.3 | 216 | 43.9 | 194.4 |
| (V) | V(c’=10%d_b, g=8%d_b) | 101.3 | 216 | 27 | 21.6 |
| | V(c’=15%d_b, g=24%d_b) | 101.3 | 216 | 40.5 | 64.8 |
| | V(c’=20%d_b, g=40%d_b) | 101.3 | 216 | 54 | 108 |
| | V(c’=25%d_b, g=56%d_b) | 101.3 | 216 | 67.5 | 151.2 |
| | V(c’=30%d_b, g=72%d_b) | 101.3 | 216 | 81 | 194.4 |
| | V(c’=35%d_b, g=80%d_b) (Rectangular opening) | 101.3 | 216 | 94.5 | 216 |

5.1. Hysteresis curve

The moment-rotation curves of the models are drawn in Fig. 11. According to Fig. 11a,b, the connection models of Groups I and II are subject to strength degradation after the rotation angle of 0.05 rad due to the local buckling of the beam flange and web. Since the flexural strength of the connections in these groups at 0.04 rad is more than 80% of the plastic flexural capacity of the beam, these connections satisfy the seismic criteria of AISC-341 [32] standard for use in special moment frames.
In Group III, the connection III(c=35% d_b) does not have the conditions for use in special moment frames because its flexural strength at 0.04 rad drops below 80% of the beam plastic flexural capacity (Fig. 11c). Hence, it is recommended that to achieve the proper cyclic performance of VS-RWS connections, the parameter "c" does not exceed 30%d_b. By increasing the parameter "g" in Group IV, the strength degradation is intensified at a rotation angle of around 0.04 rad; however, all the connections in this group meet the AISC-341 [32] requirement for use in special moment frames (Fig. 11d). In Group V, the connection V(c’=30%d_b, g =72%d_b) does not provide the conditions of AISC-341 [32] for application in special moment frames (Fig. 11e). Therefore, by examining the models of Groups III, IV, and V, it is recommended that the values of the parameters "c'" and "g" be limited to 25%d_b and 40%b (0.32d_b), respectively.
Fig. 11. Hysteresis curves of (a) ORC-B and Group I, (b) Group II, (c) Group III, (d) Group IV, and (e) Group V.
5.2. Flexural capacity

Fig. 12 illustrates the diagram of the flexural capacity ratio for all the studied connections. The vertical axis is the ratio of the flexural capacity of the VS-RWS to that of the ORC-B, and the horizontal axis is the reduced region parameters as a percentage of the beam depth ($d_b$). According to the curves of Groups I and II in this figure, the parameters "a" and "b" have a slight effect on the flexural capacity of the connections. The significant sensitivity of the Group III connections to Parameter "c" indicates that the length of vertical-slits has a great influence on the flexural capacity of the connections in such a way that by increasing the VS length from 10%$d_b$ to 35%$d_b$ the flexural capacity is decreased by about 56%. The curve of the Group IV connections reveals that the parameter "g" does not have any significant effect on the flexural capacity. According to the Group V curves of Fig. 10, the flexural capacity of the connection $V(c'=10\%d_b, g=8\%d_b)$ with a small cutting area is very close to that of the ORC-B. Moreover, the connection $V(c'=35\%d_b, g=80\%d_b)$ with the largest VS length and cutting area has the flexural capacity, which is about 40% less than that of the connection $V(c'=10\%d_b, g=8\%d_b)$ because of the local buckling or low-cycle fatigue phenomena. As expected, the flexural capacity of the ORC-B with the largest cross-section is higher than that of the other connection models.

![Fig. 12. Flexural capacity ratios for connection models](image)

5.3. Equivalent plastic strain

The equivalent plastic strain (PEEQ) is an indicator that measures the local inelastic strain demand [33]. The PEEQ can be determined as follows [14]:

$$ PEEQ = \sqrt[2]{\frac{2}{3} \varepsilon_{ij} \varepsilon_{ij}} \tag{2} $$

where, $\varepsilon_{ij}$ is the plastic strain components.

To examine the strain concentration at various locations, the PEEQ distributions of the connection models are indicated in Fig. 13. Table 5 describes the plastic strains and
deformations of the connection models in the various groups. As can be seen, a significant strain concentration occurs at the slit tips. Due to the severe deformation and local buckling of the beam flange in the reduced region of the models II(b=40%d_b) and II(b =50%d_b), in Fig. 13b the lower and upper bounds for "b" are considered to be 60%d_b and 90%d_b, respectively (0.6d_b ≤ b ≤ 0.9d_b).

The PEEQ distribution and the buckling of beam flange near the column face in models III(c′=10%d_b), IV(c′ =10%b=8%d_b) and V(c′=10%, g=8%d_b) in Fig. 13c,d,e, indicate that a decrease in "c′" below 15%d_b, or a decrease in "g" below 20%b, increases the deformation of the beam flange at the column face. Therefore, the minimum values of "c′" and "g" are recommended 15%d_b and 20%b (0.16d_b), respectively. Thus, based on the results of the present and 5.1 sections, the ranges for parameters "c′" and "g" are presented as 15%d_b ≤ c′ ≤ 25%d_b and 20%b ≤ g ≤ 40%b.
Fig. 13. PEEQ at 0.07 rad; a) Group I, b) Group II, c) Group III, d) Group IV, and e) Group V.
Table 5. Descriptions for plastic strain and deformations of the connection models

| Fig. 13 | Group | Descriptions |
|---------|-------|--------------|
| a       | I     | For ORC-B model, the strain concentration occurs at the column face, while for VS-RWS models, the strains are concentrated in a region far from the column face. The torsional deformation of the beam is clearly observed in ORC-B model, though this phenomenon is negligible in the VS-RWS models. Increasing the "a" causes the plastic hinge to move away from the column face. |
| b       | II    | Increasing the "b" reduces the deformation of the flanges. In the models of this group, the plastic hinges are formed in the flanges in line with the bottom slit, and the "b" has no significant effect on the location of plastic hinge. |
| c       | III   | The variation of "c" has a considerable effect on plastic strain intensity. As the slit length increases, a large plastic deformation at the slit tips and a severe local buckling in the beam web and flange occur, leading to a reduction in the flexural capacity of the connections. In Models III(c' = 30%d_b) and III(c' = 35%d_b), the plastic deformations and the low-cycle fatigue may cause cracks and eventually a rupture at the slit tips. In the model III(c' = 10%d_b), the lateral-torsional deformation of the beam is observed. |
| d       | IV    | The parameter "g" has a slight effect on plastic strain intensity. The model IV(c' = 10%b = 8%d_b), unlike the other models, is subject to a severe torsional deformation in a region near the welding line of the beam to the end-plate. This occurrence might lead to brittle failure of the welds between beam and endplate, when the dynamic effects are considered. The beams of the connection models have similar local buckling mode shapes and the plasticity occurs away from the column face. The optimal value for parameter "g" is recommended about 30%b. |
| e       | V     | The simultaneous increase of two parameters "c" and "g", which means an increase in the cutting area, causes the plastic strains to move away from the column face. In the model with c' = 10%d_b, the beam torsional deformation is larger than that of the other models. |

Fig. 14 shows the comparison of the maximum PEEQ for all models at different rotation angles. Fig. 14a,b indicates that in Group I and II models, the parameters "a" and "b" have a slight effect on the maximum PEEQ value. According to Fig. 14c, the PEEQ is strongly increased by increasing the cutting depth, such that in models III(c' = 30%d_b) and III(c' = 35%d_b) the rupture in the slit strips due to the low-cycle fatigue or local buckling may lead to the connection failure. As can be seen in Fig. 14d,e for Groups IV and V, by increasing the cutting area, the maximum PEEQ increases. In addition, the maximum PEEQ is created at the tips of vertical-slits, while in the models with rectangular openings, it occurs in the corner of the rectangular opening with a smaller amount than that of the VS-RWS models. In the ORC-B connection, the maximum PEEQ occurs at the column face and is less than the VS-RWS connections.
The results of numerical study (sections 5.1-5.3) showed that the parameter having the greatest impact on the plastic strain distribution of VS-RWS connections is the "c’" parameter. Fig. 15 depicts the effect of the cutting depth (2c’) on the PEEQ at the column face for Groups III and V. According to this figure, in the models with a large rectangular opening, the maximum PEEQ and consequently the possibility of failure at the column face are lower than those of the VS-RWS models; therefore, creating a large opening in the beam web cannot be recommended. By examining the results obtained from the study of Groups III and V, the optimal value for the parameter "c’" is suggested to be 20%\(d_b\). Increasing the "c’" to more than

Fig. 14. Maximum PEEQ for connection models
the optimal value improves the connection behavior in terms of removing the strain concentration from the column face and the lateral-torsional stability, while it drastically reduces the bending capacity (Fig. 11).

Fig. 15. Effect of the cutting depth ($2c'$) on the maximum PEEQ at 0.07 rad

5.4. Rupture index

In this section, the rupture index (RI) is used as a criterion for assessing the fracture potential of various locations in a structure [17, 34, 35]. This index is obtained by Eq. (3) [14].

$$
Rupture\ Index = \alpha \frac{\varepsilon_y}{\varepsilon_f} = \frac{Peeq}{\exp(1.5 \frac{p}{q})}
$$

(3)

where $\varepsilon_y$ and $\varepsilon_f$ are the yield strain and fracture strain; $\alpha$ is a material constant; $p$ and $q$ are hydrostatic stress and equivalent stress (von-Mises stress), respectively.

The fracture of connections at the column face is an important failure mechanism that needs to be avoided. Also, the regions with the maximum PEEQ have the largest values of RI [35]. Hence, the maximum values of RI at the column face and at the locations of the maximum PEEQ are plotted in Fig. 16. According to Fig. 16a, the RI at the column face of all the VS-RWS connections is lower than that of the ORC-B connection. This means that the vertical-slits reduce the potential of connections for the weld failure. Among the present studied connections, the V($c'=35%d_b$, $g=80%d_b$) and III($c'=35%d_b$) have the lowest RIs at the welding line; however, Fig. 16b shows that the mentioned connections have a high potential for the low-cycle fracture around the slits.
The failure modes of the ORC-B connection constitute the local buckling of beam flange at the column face, the lateral-torsional buckling of the beam, and the fracture of the welding line, while the general failure modes of the VS-RWS connections are the large deformation and cracking of slit strips, as well as the local buckling or fracture of the reduced region.

To determine the effect of the reduced region parameters on the stresses of the VS-RWS connections, the ratio of the maximum von-Mises stress to the beam web yield stress ($F_y = 243$ MPa) is shown in Fig. 17a. According to this figure, in Groups III and V, increasing the "c'" parameter up to $25\%d_b$ leads to an increase in the stresses at the slit tips. Then by passing the "c'" parameter through the value of $25\%d_b$, in Group III the stresses remain almost constant, and in Group V, the stresses are decreased. Thus, the size of the cutting depth and the area of the reduced region greatly affect the maximum stress of the connections. In the ORC-B connection with no slit, the stress ratio has the lowest value. The diagram of the stress ratio at the location of the column face is drawn in Fig. 17b. This figure illustrates that in the ORC-B, as expected, the stress ratio at the column face is higher than that of the VS-RWS connections, and the yielding occurs in some parts of the beam welding line; hence, there is a possibility of failure at this location. Among the VS-RWS connections studied here, only in $V(c'=10\%d_b, g=8\%d_b)$ with the lowest cutting depth and area, the stress ratio is larger than one and the stress at the column face reaches to yielded stress. Fig. 17a indicates that in Group I, the model I(a=65\%d_b) has the highest stress ratio at the column face, whereas according on Fig. 13a, the plastic strains and deformations of the beam in all models of this group are almost similar; therefore, the range for the parameter "a" is recommended as $25\%d_b \leq a \leq 55\%d_b$. 

Fig. 16. Maximum value of RI; a) at the column face; and b) at the location of maximum PEEQ.
The maximum deformation in the beam occurs in two sections a-a and b-b sections for all models at the final loading step. In the a-a section, for all models, the maximum deformation of the end-plate is seen at the location from the beam flange to the end-plate weld, which has the highest stress. The thickness of the end-plate is 20 mm, and its yield and ultimate stresses are 330 and 445 MPa, respectively. The proper design of the end-plate and the weakening of the beam through creating vertical-slits have caused the end-plate deformations to be very small and have made the end-plate possess good out-of-plane bending stiffness. The combination of the end-plate deformation in two sections a-a, and b-b emphasizes the bi-directional bending for all the models. The asymmetric deformation of the end-plate along the section b-b for the models with no or low cutting area, such as the ORC-B, as well as IV(g=10%b=8%d_b) and V(c'=10%d_b, b=8%d_b) models can be attributed to the relatively strong torsional buckling of the beam in these connections. The approximate matching of the curves related to Groups I and II shows that the parameters "a" and "b" have a little effect on the end-plate deformation. Investigating the diagrams of Group III in Fig. 18 indicates that increasing the cutting depth (2c') reduces the out-of-plane deformation of the end-plate in both sections, a-a and b-b, by about 150% and 100%, respectively, which is a significant impact. In Group IV, the "g" parameter is not effective in the end-plate deformation at the a-a section, while this parameter affects the end-plate deformation in the b-b section. For this reason, in the model IV (g=10%b=8%d_b), where the cutting area is very small, the end-plate deformation in the b-b section is different from that of the other models in this group.

Fig. 17. The ratio of von-Mises to yield stress at the final loading step a) in the whole connection b) at the column face

5.5. End-plate deformation

Fig. 18 shows the diagrams of the end-plate deformation in both a-a and b-b sections for all models at the final loading step. In the a-a section, for all models, the maximum deformation of the end-plate is seen at the location from the beam flange to the end-plate weld, which has the highest stress. The thickness of the end-plate is 20 mm, and its yield and ultimate stresses are 330 and 445 MPa, respectively. The proper design of the end-plate and the weakening of the beam through creating vertical-slits have caused the end-plate deformations to be very small and has made the end-plate possess good out-of-plane bending stiffness. The combination of the end-plate deformation in two sections a-a, and b-b emphasizes the bi-directional bending for all the models. The asymmetric deformation of the end-plate along the section b-b for the models with no or low cutting area, such as the ORC-B, as well as IV(g=10%b=8%d_b) and V(c'=10%d_b, b=8%d_b) models can be attributed to the relatively strong torsional buckling of the beam in these connections. The approximate matching of the curves related to Groups I and II shows that the parameters "a" and "b" have a little effect on the end-plate deformation. Investigating the diagrams of Group III in Fig. 18 indicates that increasing the cutting depth (2c') reduces the out-of-plane deformation of the end-plate in both sections, a-a and b-b, by about 150% and 100%, respectively, which is a significant impact. In Group IV, the "g" parameter is not effective in the end-plate deformation at the a-a section, while this parameter affects the end-plate deformation in the b-b section. For this reason, in the model IV (g=10%b=8%d_b), where the cutting area is very small, the end-plate deformation in the b-b section is different from that of the other models in this group.
Diagrams of Group V in Fig. 18, which are a combination of Groups III and IV, confirm the effects of the two parameters "g" and "c'" on the out-of-plane deformation of the end-plate.

**Diagram 1**
- Section (a-a)
- Group I
- Section (b-b)
- ORC-B

**Diagram 2**
- Section (a-a)
- Group II
- Section (b-b)
- ORC-B

**Diagram 3**
- Section (a-a)
- Group III
- Section (b-b)
- ORC-B
5.6. Energy dissipation

The area enclosed by the hysteresis curves of the connections represents the energy dissipated due to cyclic loading. Fig. 19 illustrates the energy dissipated by the modeled connections.

As can be seen, the "a" and "b" parameters, in Groups I and II, have a different, yet insignificant effect on the energy dissipation. The highest and lowest energy dissipations are related to the models II(b=60%d_b) and I(a=55%d_b), respectively, with a roughly 6%
difference. In Group III, increasing the "c'" from 10%dₜ to 35%dₜ has reduced the dissipated energy by about 30%, indicating that the "c'" is the most effective parameter on energy dissipation. Since the hysteresis curves of Group IV models are almost similar, as expected, the amounts of energy dissipation of connections in this group will be very close, and the "g" parameter will have a very small effect on the dissipated energy. Among the models, V(c'=35%dₜ) with the largest slit depth and cutting area has the lowest energy dissipation and as mentioned in the previous sections, this connection has the lowest bending capacity.

6. Equivalent radial-cut and constant-cut for vertical-slits

In this paper, the equivalent radial-cut (RC) RBS and the equivalent constant-cut (CC) RBS of a VS-RWS, in terms of their flexural strain, are suggested via using the classical conjugate beam method in the closed form [36]. Fig. 20 illustrates the bending moment diagram, as well as the geometries of VS-RWS, RC-RBS, and CC-RBS. According to this figure, the flexural moment of the beam can be obtained as follows:

\[ M(x) = V \left( \frac{L_b}{2} - x \right) \]

where V is the shear force at the inflection point of the beam; and L_b is the beam length.
According to Fig. 21, in the VS-RWS beam, the shear force (V) of the T-section of the beam creates the secondary flexural moment (m) and the amount of this moment in the center of each slit is calculated from Eq. (5).

\[ m = \frac{V}{2} \frac{g}{4} = \frac{Vg}{16} \]  

(5)

Therefore, the stress at the flange of the beam cross-section in each vertical-slit is obtained by sum of the stresses caused by the M(x) bending moment and by the secondary flexural moment (m) as Eq. (6). In addition, Eq. (7) calculates the stress at the flange of the beam full-section (the non-reduced cross-section).

\[ \sigma(x) = \frac{M(x)}{S_{VS}} + \frac{m}{S_T} \]  

(6)

\[ \sigma(x) = \frac{M(x)}{S_b} \]  

(7)

where, \( S_{VS} \), \( S_T \) and \( S_b \) are the cross-section modules of the beam VS-section, the T-shape part of the VS-section, and the beam full-section, respectively (Fig. 21) which can be calculated by the following equations;

\[ S_{VS} = \frac{I_{VS}}{d_b} = \frac{1}{12} \left( b_f d_b^3 - (b_f - t_w)(d_b - 2t_f)^3 - t_w (2c')^3 \right) \]  

(8)

\[ S_T = \frac{I_T}{\bar{y}} = \frac{1}{12} \left( t_w (d_b/2 - t_f - c')^3 + b_f t_f^3 \right) \]  

(9)

\[ S_b = \frac{I_b}{d_b} = \frac{1}{12} \left( b_f d_b^3 - (b_f - t_w)(d_b - 2t_f)^3 \right) \]  

(10)

The flexural strain of the beam flange is then achieved in the VS-section and in the full-section by Eqs. (11) and (12), respectively.
By integrating the flexural strain over the length of the reduced region, the total elongation of the beam flange is obtained from Eq. (13).

\[
\Delta_{VS-RWS} = \int_a^{a+b} \varepsilon(x) \, dx = \frac{1}{E} \int_{\text{reduced-section}}^{} \left( \frac{M(x)}{S_{VS}} + \frac{m}{S_T} \right) \, dx + \frac{1}{E} \int_{\text{full-section}}^{} \frac{M(x)}{S_{bR}} \, dx
\]

(13)

Similarly, in the RC-RBS connection at the beam flange, the strain, and the elongation related to the reduced region is calculated as follows:

\[
\sigma(x) = \frac{M(x)}{S_{RC}(x)}
\]

(14)

\[
S_{RC}(x) = \frac{I_{RC}(x)}{d_b} = \frac{1}{12} \left( b_f(x) d_b^3 - (b_f(x) - t_w)(d_b - 2t_f)^3 \right)
\]

(15)

\[
\varepsilon(x) = \frac{\sigma(x)}{E} = \frac{M(x)}{S_{RC}(x)}
\]

(16)

\[
\Delta_{RC-RBS} = \int_a^{a+b} \varepsilon(x) \, dx = \frac{1}{E} \int \left( \frac{M(x)}{S_{RC}(x)} \right) \, dx
\]

(17)

where \( S_{RC}(x) \) is the cross-section modulus of the beam in the RC-section; and \( b_f(x) \) is the reduced flange width as a function of the distance from the column face \( x \) as Eq. (18). In this equation, the Taylor series expansion is used to simplify \( b_f(x) \).

\[
b_f(x) = (b_f - 2c_{eq} - 2R) - 2R \sqrt{1 - \left( \frac{x - a - \frac{b}{2}}{R} \right)^2} \cdot b_f - 2c_{eq} + \frac{1}{R} \left( x - a - \frac{b}{2} \right)^2 \cdot (a \leq x \leq a + b)
\]

(18)

In the same way, in the CC-RBS connection with a constant cutting depth \( c_{eq} \) at the beam flange, the strain and the elongation over the reduced region of the beam are achieved by Eqs. (19) to (22).
where $S_{CC}$ is the cross-section modules of the beam with the constant-cut. Since the elongation of the beam flange is directly related to the flexural rotation of the beam, the validity of the elongation criterion for three connections is explained as Eq. (23).

\[ \Delta_{CC-RBS} = \int_a^b \varepsilon(x) \, dx = \frac{1}{E} \int \left( \frac{M(x)}{S_{CC}} \right) \, dx \]

Inserting, Eqs. (13), (17) and (22) in to Eq. (23), and solving integrals, give the following equation.

\[ \frac{g b}{S_{VS}} + \frac{2(L_b - 2a - b)}{S_{T}} + \frac{b - g}{S_b} = \frac{2}{S_{RC}} \left( \frac{6R_d}{\sqrt{d_b - (d_b - 2t_f)^3}} \right) \tan^{-1} \left( \frac{2}{b} \sqrt{\frac{6R_d}{\sqrt{S_{RC} - (d_b - 2t_f)^3}}} \right) = \frac{b}{S_{CC}} \]

where $S_{RC}$ is the cross-section modules of the RC-RBS beam in the center of the radial-cut. In order to obtain $\Delta_{CC-RBS}$ in the CC-RBS connection, in addition to the presented method, it can be obtained by calculating the limit of $\Delta_{RC-RBS}$ when "$R$" (the radius of the cut is depicted in Fig. 20) approaches infinity using the Maclaurin series as follows:

\[ \lim_{R \to \infty} \Delta_{CC-RBS} = \Delta_{RC-RBS} = \Delta_{CC-RBS} \]

Inserting, Eqs.(13), (17) and (22) in to Eq.(23), and solving integrals, give the following equation.

In order to check the accuracy of the equivalent radial-cut and the equivalent constant-cut for the VS-RWS, a finite element analysis was conducted. In this numerical study, three connections (Table 6) were applied under the cyclic loading. The loading protocol, the beam and column profiles and the dimensions were considered to be the same as those in the tested specimens. All the equivalent parameters including "c" and "c_{eq}" implicitly satisfy Eq. (24).
Table 6. Geometric specifications of VS-RWS, RC-RBS and CC-RBS

| Specimens | a (mm) | b (mm) | VS  | RC  | CC  |
|-----------|-------|--------|-----|-----|-----|
| VS-RWS   | 101.3 | 216    | 86.4| 54  | -   |
| RC-RBS   | 101.3 | 216    | -   | -   | 27  |
| CC-RBS   | 101.3 | 216    | -   | -   | 19  |

The load-displacement curves of the beams in Fig. 22 indicate that the loading capacity is almost the same in all three connections, and the beam end deflections corresponding to the applied load of the VS-RWS connection differ from those of the RC-RBS and CC-RBS connections by about 0.7% and 0.9%, respectively, which is negligible. It is noteworthy that the authors have conducted several numerical analyses on the VS-RWS, RC-RBS and CC-RBS connections with various reduced region parameters that satisfy Eq. (24). The good agreement between the results confirmed the efficiency of the proposed criterion; however, they have not been presented here for the sake of brevity.

7. Story drift due to beam flexural deformation

In regular moment frames under lateral loading, the inflection points are usually near the mid-span of the beam and the mid-height of the column. The beam slope ($\theta_b$) and the story drift ($\delta_b$) in such frames can, therefore, be determined by analyzing a length of the beam between the middles of the two spans. Fig. 23 shows the deflected shape of the connection caused by only the beam flexural deformation.
In this paper, to attain the story drift of frame caused by the VS beam flexural deformation, at first the equivalent CC-RBS was obtained for the VS-RWS; then the story drift due to the constant-cut beam flexural deformation was calculated using the conjugate beam method. Based on this method, a distributed load obtained by dividing the value of the real beam bending moment by the EI was placed on the conjugate beam. The obtained distributed load was then converted to the equivalent concentrated load (P) and was placed in the load area center (\( \bar{x} \)). By writing equilibrium equations in the form of the free body diagram, the shear force on the conjugate beam was calculated, which is equal to the slope (\( \theta_b \)) of the real beam. The elastic load diagram of the equivalent beam with the constant-cut is illustrated in Fig. 24.

\[
\delta_b = H_c \theta_b
\]

According to Fig. 23, by multiplying \( \theta_b \) by the column height, the story drift of frame (\( \delta_b \)) caused by the beam flexural deformation is achieved by Eq. (26).

\[
\delta_b = \frac{H_c}{L_b + d_c} \left( L_b \int_0^{L_b/2} \frac{M(x)}{EI(x)} \, dx - 2 \int_0^{L_b/2} \frac{M(x) x}{EI(x)} \, dx \right)
\]  

By solving the integrals of Eq. (26) and simplifying it, the value of \( \delta_b \) is obtained as follows:
where $I_p$ and $t_p$ are the moment of inertia and thickness of the end-plate, respectively. The story drift ($\delta_b$) from Eq. (27) is obtained by the sum of three terms; the first, second and third terms are related to the end-plate, the reduced-section beam, and the full-section beam, respectively.

8. Proposed design procedure

In order to design the common RBS connections, a specified procedure and suitable criteria have been presented in Chapter 5 of AISC-358 [30]. Nevertheless, no suitable criteria have been provided for VS-RWS connections. In this section, according to the parametric study of the previous sections, as well as to the AISC-358 [30], a step-by-step procedure for the design of VS-RWS connections is proposed as follows:

Step 1. According to the following limits, consider the trial value for the location and length of the beam web reduction:

\[
0.25d_b \leq a \leq 0.55d_b
\]
\[
0.6d_b \leq b \leq 0.9d_b
\]  

Step 2. Choose the trial value for the cutting depth ($2c'$) based on the following range:

\[
0.15d_b \leq c' \leq 0.25d_b
\]

For the first attempt, it is recommended that you assume "$c''" and g to be equal to the optimal values of $0.2d_b$ and $0.3b$, respectively.

Step 3. By using Eq. (24), obtain $S_{eq}$ and "$c_{eq}" for the equivalent RC-RBS or for the equivalent CC-RBS. $S_{eq}$ is the reduced-section modulus ($S_{eq}=S_{RC}$ for RC-RBS; $S_{eq}=S_{CC}$ for CC-RBS).

Step 4. Compute the plastic reduced-section modulus of about x-axis ($Z_{eq}$, mm$^3$), and the probable maximum moment ($M_{px}$) at the equivalent RC-RBS or CC-RBS.

\[
Z_{eq} = t_f (b_f - 2c_{eq})(d_b - t_f) + \frac{t_w (d_b - 2t_f)^2}{4}
\]  

(31)
$M_{pr} = C_{pr} R_y F_y Z_{eq}$ \hspace{1cm} (32)

$C_{pr} = \frac{F_y + F_d}{2F_y} \leq 1.2$ \hspace{1cm} (33)

where based on the AISC-358 [30] seismic provisions, $C_{pr}$ and $R_y$ are the factors which account for the peak strength and the ratio of the expected yield stress to the specified minimum yield stress, respectively.

**Step 5.** Obtain the shear force ($V_{eq}$) in the center of the radial-cut for RC-RBS, or at the beginning of the constant-cut for CC-RBS at each end of the beam by using the free-body diagram of the beam (Fig. 25), including $M_{pr}$ and gravity loads ($V_{gravity}$) acting on the beam as Eq. (34).

$$V_{eq} = \frac{2M_{pr}}{L} + \frac{1}{2} V_{gravity}$$ \hspace{1cm} (34)

**Step 6.** Compute $M_f$, the probable maximum moment at the column face from a free-body diagram of the segment of the beam between the center of the radial-cut and the column face for RC-RBS (Fig. 25a and Eq.(35)) or between the beginning of the constant-cut and the column face for CC-RBS (Fig. 25b and Eq. (35)).

$$M_f = M_{pr} + V_{eq} S_h$$ \hspace{1cm} (35)

where $S_h=a+b/2$ for RC-RBS, and $S_h=a$ for CC-RBS.

**Step 7.** Obtain the plastic moment of the beam ($M_{pe}$) based on the expected yield stress as follows:

$$M_{pe} = R_y F_y (b_f t_f (d_b - t_f) + \frac{t_w (d_b - 2t_f)^2}{4})$$ \hspace{1cm} (36)

**Step 8.** Check the flexural strength of the beam at the column face through Eq. (37). In this equation, $\phi_d$ is the resistance factor for ductile limit states ($\phi_d =1$).

$$M_f \leq \phi_d M_{pe}$$ \hspace{1cm} (37)
Step 9. If Eq. (37) is not satisfied, modify the values of ”c”’, ”g”, ”a”, ”b”, and repeat steps 3 through 9.

Step 10. Determine the beam-required shear strength ($V_{eq}$) using Eq. (34), and then obtain the nominal shear strength as follows [37]:

$$ V_n = 0.6F_y A_w C_{v1} $$

(38)

where, $F_y$ and $A_w$ are the specified minimum yield stress of the type of steel being used (MPa) and the area of the beam web (mm$^2$), respectively. Check the shear strength of the beam through Eq. (39).

$$ V_{eq} \leq \phi_v V_n $$

(39)

In LRFD method, $C_{v1}$ and $\phi_v$ for the webs of the rolled I-shaped members with $h/t_w \leq 2.24(E/F_y)^{0.5}$ are equal to one, where $E$, $h$ and $t_w$ are the modulus of elasticity of steel (=200 GPa), the clear distance between flanges less the fillet at each flange (mm), and the web thickness, respectively.

Step 11. Establish preliminary values for the end-plate geometry, namely the horizontal and vertical distances between bolts (mm), thickness of end-plate (mm) etc, and bolt grade.

Step 12. Determine the required bolt diameter ($d_{b,req}$) using the following expressions:

$$ d_{b,req} = \sqrt{\frac{2M_t}{\pi \phi_n F_{nt} (h_0 + h_1)}} $$

(40)

where $F_{nt}$ is the nominal tensile strength of the bolt from the AISC-358 [30] specification (MPa); $h_0$ and $h_1$ are the distances from centerline of the beam compression flange to the centerline of the $i^{th}$ tension bolt row and the distance from centerline of compression flange to tension-side outer bolt row (mm), respectively. $\phi_n$ is equal to 0.90.

Step 13. Determine the required dimensions of the end-plate based on the section 6.8 of AISC-358 [30].
Step 14. Design the flange-to-end-plate and web-to-end-plate welds using the requirements of the section 6.7.6 of AISC-358 [30].

(a) 

(b) 

Fig. 25. Example calculation for the maximum moment of the beam at the column face and for the beam shear a) in the center of radial-cut and b) at the beginning of constant-cut

9. Conclusions

In this paper, experimental and numerical studies were carried out on two full-scale bolted reduced web section connections with vertical-slits (VS-RWS), to evaluate their cyclic performance. After verifying the validity of the numerical model, a parametric analysis was performed on thirty connection models to investigate the effect of the geometric parameters of the reduced region on VS-RWS connections. The studied parameters were composed of the distance between the column face and the beginning of the cutting (a), the length of the reduced region (b), the half of the vertical-slit depth (c'), and the total cutting size along the beam length (g) for a single cut or multiple discontinuous cuts. In this paper, the story drift of the frame caused by the flexural deformation of the VS beam was calculated using the conjugate beam method, and for the proper design of the VS-RWS connection, the optimal ranges for the parameters of the reduced region were recommended. The results of this study are summarized as follows:

- Investigation of the VS-RWS connections shows that the vertical-slits could cause the plastic hinge and stress/strain concentrations to move away from the weld line of the
end-plate and to transfer them around the slits. This will prevent a brittle fracture or damage at the column face.

- As compared with the ordinary rigid connection, the vertical-slits improve the lateral-torsional stability behavior of VS-RWS connections.
- According to the conducted experimental tests, the depth and area of the vertical-slits play an important role in the local instability and the beginning of slit-rupture, which leads to the strength degradation in the cyclic behavior.
- The depth of the vertical-slit is an effective parameter in the behavior of the VS-RWS connections, such that the increase of the slit depth from \( c' = 10 \% d_b \) to \( c' = 35 \% d_b \) can reduce the amounts of the flexural capacity and the energy dissipation by about 40% and 25%, respectively.
- For the proper design of VS-RWS connections, the ranges for the parameters "a", "b", "c'" and "g" are recommended as a percentage of the beam depth as follows:
\[
(0.25d_b \leq a \leq 0.55d_b), \ (0.6d_b \leq b \leq 0.9d_b), \ (0.15d_b \leq c' \leq 0.25d_b), \ \text{and} \ \ (0.2b \leq g \leq 0.4b).
\]
- Considering \( c' > 0.25d_b \) reduces the flexural capacity and energy dissipation, and \( c' < 0.15d_b \) causes the local or torsional buckling, the increase of stress/strain concentration at the column face, as well as the out-of-plane deformation of the end-plate.
- Based on the parametric study, the optimal values for "c'" and "g" parameters in this type of connection are about 20%\(d_b\) and 30%\(b\), respectively.
- Considering the proper geometric parameters of the reduced region, the VS-RWS connections are capable of being used in special moment frames based on AISC-358.
- This paper calculates the story drift of the VS-RWS frame caused by the flexural deformation of the beam by presenting an equivalent constant-cut or radial-cut RBS connection using the conjugate beam method. By applying the equivalent RBS connection, a step-by-step design procedure has been proposed for VS-RWS connections.

**Recommendations for further research**
The authors recommend further research on the bolted or welded VS-RWS connections with different beam profiles, or with the non-equal depth of vertical-slits for various parameters, such as the number of slits and the beam length.
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