# Study of Reduced Beam Section Connections with Web Opening

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**SUMMARY:**
Seismic behavior of beam to column connections can be improved by shifting the location of inelasticity away from the column's face. Such connections can be achieved by reducing flange area at a specific distance from the beam-column connection, called Reduced Beam Section (RBS) or by reducing web area by introducing a perforation into the web, called Reduced Web Section (RWS). Research on RWS connections shows that, if properly designed, they can satisfy the "strong column-weak beam" and "strong connection-weak component" criteria. Shear and moment interaction in RWS connections is in high importance for designing of these connections. A parametric study has been done on the effect of perforation size, perforation location and span length of these connections. In addition, an interaction formula for moment and shear is derived for design purposes. Furthermore, a step by step method for designing of these connections is prescribed.

*Keywords: Reduced Web Section, Plastic Hinge, Shear-moment interaction, Beam-column Connection*

## 1. INTRODUCTION

There were many brittle fractures in the beam-to-column connections of steel moment resisting frames (MRFs) subjected to the seismic action before their components yield in the Northridge earthquake in 1994 and the Kobe earthquake in 1995 (Miller. 1998). Some approaches have focused on improving the toughness of the connection or reducing the intensity of stress concentrations, while others have shifted the location of inelasticity away from the beam-column connection. In the latter case, the beam section can be intentionally reduced at a specific distance from the beam-column connection, and thus to induce plastic hinging within the reduced section of the beam away from the connection. In order to plastic hinges take place far enough from column's face, two main theories have been developed, reducing flange area of the beam, called Reduced Beam Section (RBS) and reduction in web area of the beam, called reduced web section (RWS). Through extensive experimental studies (Engelhardt et al. 1998; Chen. 2001; Jin & El-Tawil. 2005; Ricles et al. 2004; Itani, Cheng & Saiidi. 2004), it is confirmed that RBS connections can develop high inelastic deformations and attain acceptable plastic rotations. Studies of RWS steel frames still remained limited, which hinder acceptance of this type of connections. Based on the limited results of the analyses which are carried out by Kazemi & Hoseinzadeh Asl (2011), the frames with RWS connection can provide at least the same level of seismic improvement that the frames with RBS connection can. Lepage, Aschheim & Senescu (2004) used the reduced web section beams in the lateral load resisting system and the reduced zones of the beams were modeled by uncoupled rigid-plastic springs. Shanmugan, Lian & Thevendran (2002) used finite element modeling of plate girders containing circular and rectangular web openings to investigate the nonlinear behavior and ultimate capacity of these beams. It is important to note that reducing the flanges' section in the RBS connection can cause a reduction in frame stiffness and this may lead to at least 4 percent increase in drift (Lee & Chung. 2007). Since moment inertia of a beam is not affected much by area of beam's web, decrease in lateral stiffness of frames, including RWS connections is negligible. Experimental studies have shown that the seismic energy is dissipated by local deformation in the weakened area of beam due to the opening in the case of severe earthquake action, and the expected failure mode of a ductile frame, i.e., ‘strong column but weak beam’ and ‘strong connection but weak component’, is reached, which may result in an improvement of the aseismic behaviors of steel MRFs (Qingshan, Bo & Na. 2009). All the steel beams with web openings of various shapes and sizes behave similarly among each other in terms of deformed shapes under a wide range of applied moments and shear forces (Liu & Chung. 2003). Moreover, the failure modes are common among all beams, namely, shear failure, flexural failure and Vierendeel mechanism, depending on the loading and the support conditions of the beams and also the location of the web openings along the beam length. Furthermore, the load–deflection curves of steel beams with web openings of different shapes and sizes are also very similar to each other (Liu & Chung. 2003).

In this paper, a parametric study has been done on the effect of perforation size, perforation location and span length of RWS connections. In order to reduce the local buckling effect, a perimeter stiffener is considered around the perforation. In addition, an interaction formula for moment and shear is derived for design purposes. Furthermore, a step by step method for designing of these connections is prescribed.

## 2. FINITE ELEMENT MODEL

Eighteen models with the difference in perforation size, perforation location and span length were modeled. The configuration of the models is illustrated in Fig. 2.1. According to AISC seismic provisions for the special moment frames, the beam to column connection shall be capable of sustaining an inter-story drift angle of at least 0.04 radians (AISC. 2005). In order to achieve this purpose, the following three conditions must be satisfied at the target inter-story drift angle:

a) For satisfying ‘strong connection but weak component’, the connection components at the face of the column must be designed to remain elastic, in the fully yielded and strain-hardened condition that can be forced in the reduced section, under seismic loads and gravity loads.

b) In order to avoid any fracture in the reduced section, maximum plastic strain shall be less than ultimate strain of steel, which is assumed to be 0.2.

c) For the connections with strength degradation, the decrease in strength should not exceed 20 %.

Models which satisfy all mentioned conditions will be acceptable and can be used as an RWS connection.

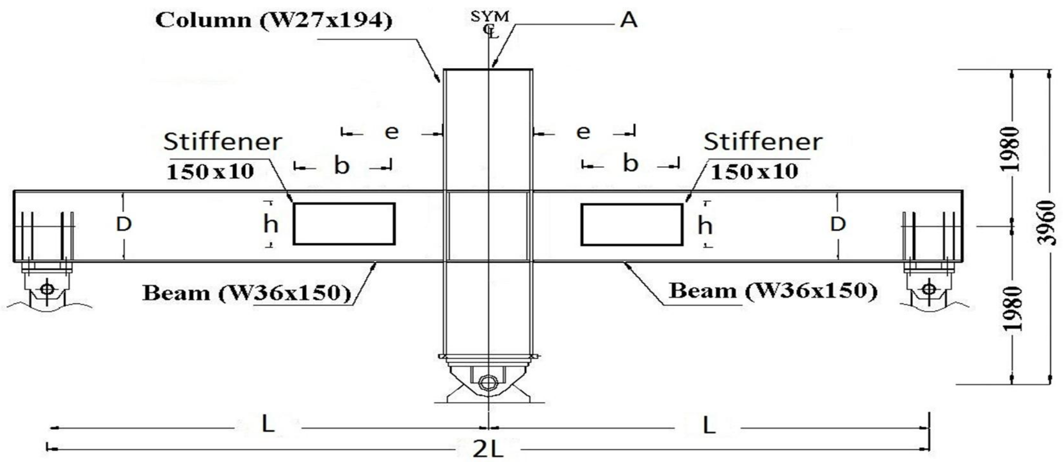


**Figure 2.1.** The Modeled RWS Connection (Dimensions in mm)

The parameters of models are listed in Table 2.1.

**Table 2.1.** Parameters of Models

| Models | h (mm) | b (mm) | b/h | Percentage of perforation in height (%) | e (mm) | e/D | 2L (mm) |
|---|---|---|---|---|---|---|---|
| BD285 | 780 | 1560 | 2 | 85.3 | 91.44 | 1 | 9000 |
| BD275 | 690 | 1380 | 2 | 75.5 | 91.44 | 1 | 9000 |
| BD265 | 600 | 1200 | 2 | 65.6 | 91.44 | 1 | 9000 |
| BD1.585 | 780 | 1170 | 1.5 | 85.3 | 91.44 | 1 | 9000 |
| BD1.575 | 690 | 1035 | 1.5 | 75.5 | 91.44 | 1 | 9000 |
| BD1.565 | 600 | 900 | 1.5 | 65.6 | 91.44 | 1 | 9000 |
| BD185 | 780 | 780 | 1 | 85.3 | 91.44 | 1 | 9000 |
| BD175 | 690 | 690 | 1 | 75.5 | 91.44 | 1 | 9000 |
| BD165 | 600 | 600 | 1 | 65.6 | 91.44 | 1 | 9000 |
| B0.75D265 | 600 | 1200 | 2 | 65.6 | 68.58 | 0.75 | 9000 |
| B0.75D1.585 | 780 | 1170 | 1.5 | 85.3 | 68.58 | 0.75 | 9000 |
| B0.75D1.575 | 690 | 1035 | 1.5 | 75.5 | 68.58 | 0.75 | 9000 |
| B0.75D1.565 | 600 | 900 | 1.5 | 65.6 | 68.58 | 0.75 | 9000 |
| B0.75D185 | 780 | 780 | 1 | 85.3 | 68.58 | 0.75 | 9000 |
| B0.75D175 | 690 | 690 | 1 | 75.5 | 68.58 | 0.75 | 9000 |
| B0.75D165 | 600 | 600 | 1 | 65.6 | 68.58 | 0.75 | 9000 |
| BD2853 | 780 | 1560 | 2 | 85.3 | 91.44 | 1 | 6000 |
| BD2856 | 780 | 1560 | 2 | 85.3 | 91.44 | 1 | 12000 |

## 3. ANALYSES RESULT

In order to check the previously mentioned conditions, static analyses were done on the models by using a general purpose finite element program. A step by step monotonic lateral displacement was applied at the top of the column at point A of Fig. 2.1 until failure occurred.
Table 3.1 lists the eight models, out of the eighteen listed in Table 2.1, which satisfied all the conditions. In the other models, the strain exceeded the yield strain at the face of the column. Fig. 3.1 shows the plastic strain via story drift of the selected elements of the model BD165, during the analysis. Story shear and corresponding story drift of accepted models are listed in Table 3.1. In addition, Fig. 3.2 shows a diagram of story shear versus story drift of accepted models.

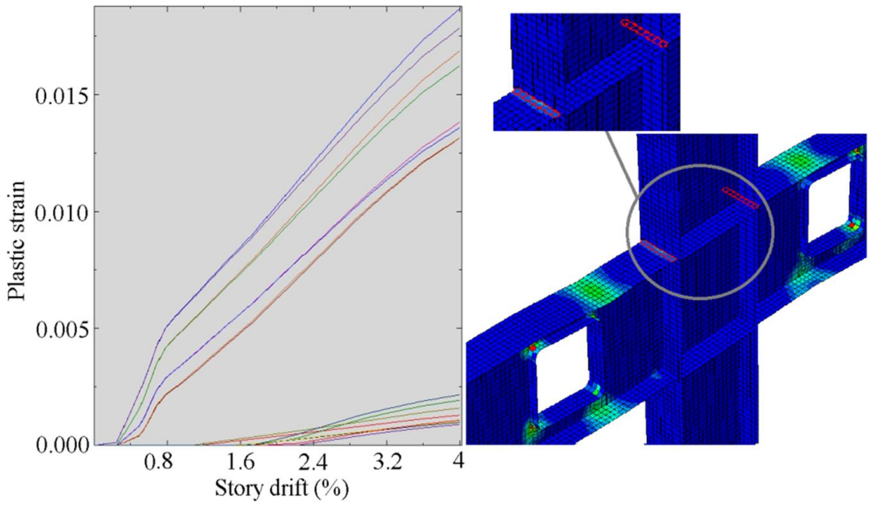


**Figure 3.1.** Progress of Plastic Strain of Selected Points vs. Story Drift

**Table 3.1.** Story Shear and Corresponding Story Drift of Accepted Models

| | Story shear (kN) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Story drift (rad) | BD185 | BD1.575 | BD1.585 | BD275 | BD285 | B0.75D185 | BD2856 | BD2853 |
| 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 0.0051 | 1359 | 1422 | 1192 | 1287 | 1103 | 1358 | 1174 | 1028 |
| 0.0101 | 1486 | 1567 | 1308 | 1414 | 1214 | 1483 | 1295 | 1129 |
| 0.0152 | 1530 | 1618 | 1338 | 1456 | 1243 | 1526 | 1326 | 1156 |
| 0.0202 | 1559 | 1651 | 1358 | 1482 | 1259 | 1554 | 1345 | 1169 |
| 0.0303 | 1586 | 1683 | 1374 | 1508 | 1268 | 1577 | 1359 | 1174 |
| 0.0404 | 1581 | 1688 | 1367 | 1508 | 1255 | 1570 | 1348 | 1155 |
| 0.0455 | 1572 | 1684 | 1359 | 1504 | 1244 | 1560 | 1338 | 1142 |
| 0.0505 | 1560 | 1676 | 1350 | 1498 | - | 1547 | 1326 | 1128 |
| 0.0657 | 1506 | 1637 | 1315 | 1471 | - | 1489 | 1283 | 1089 |
| 0.0808 | - | - | 1276 | 1432 | - | - | - | 1067 |
| 0.0909 | - | - | 1250 | - | - | - | - | 1051 |
| 0.1162 | - | - | - | - | - | - | - | 997.5 |
| 0.1263 | - | - | - | - | - | - | - | 972.7 |

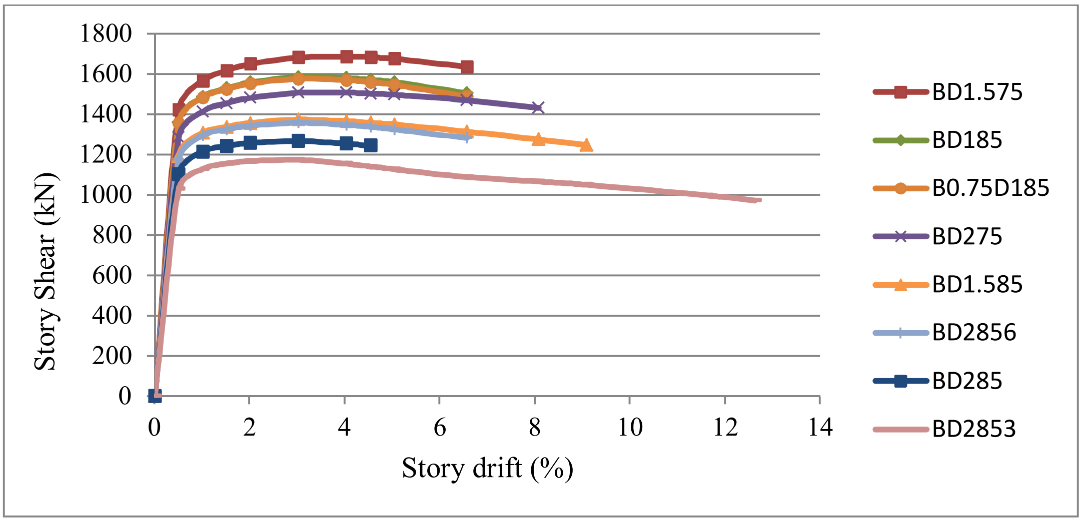


**Figure 3.2.** Diagram of Shear Force vs. Corresponding Displacement of Point A for Acceptable Models

The analyses results showed that strain of elements at the face of the column in models with square shape opening passed the yield limit, thus rectangular perforations behaved better than square one. In addition, increase of the width of the perforation up to twice the height of it, made significant strength degradation, so it is suggested that the ratio of width to height of the perforation be between one and two.

Comparison of the models with the same perforation's width, indicated that when the height of perforations are less than 65% of the height of the beams, the plastic strains developed at the face of the column, which is not desired. On the other hand, when the height of the perforations exceeded the 85% of the height of the beam, despite increasing flexibility of the models, the strength degradation at 4% story drift will be more than 20%. In order to avoid such problems, it is suggested that height of the perforations be chosen between 70% and 85% of the height of the beams.

As listed in table 2.1, the ratio of e/D for some models is assumed to be 0.75 and for others to be 1. The analyses results showed that when the ratio of e/D was chosen to be 0.75, inelastic behavior in the vicinity of the column happened, and none of those models were acceptable except for one. On this basis, it is suggested that the distance between the middle of the perforation and the face of the column, "e" in Fig. 2.1, be greater than the height of the beam.

## 4. INTERACTION FORMULA

Shear and moment relation in RWS connections is in high importance for design purposes. In order to derive an interaction formula for these connections, cantilever beams with the opening dimensions of accepted models had been studied. The lengths of the studied cantilevers were equal to 2e, and a shear and a moment force were applied at the end of the beam. By changing the magnitude of the forces, points on the yield curve of each model were derived and fitting a curve to the points concluded the interaction formula of the models as is shown in Fig. 4.1. Points in each diagram in Fig. 4.1 are the result of finite element analyses and continuous curves are fitted curves which their equations are written in each diagram.

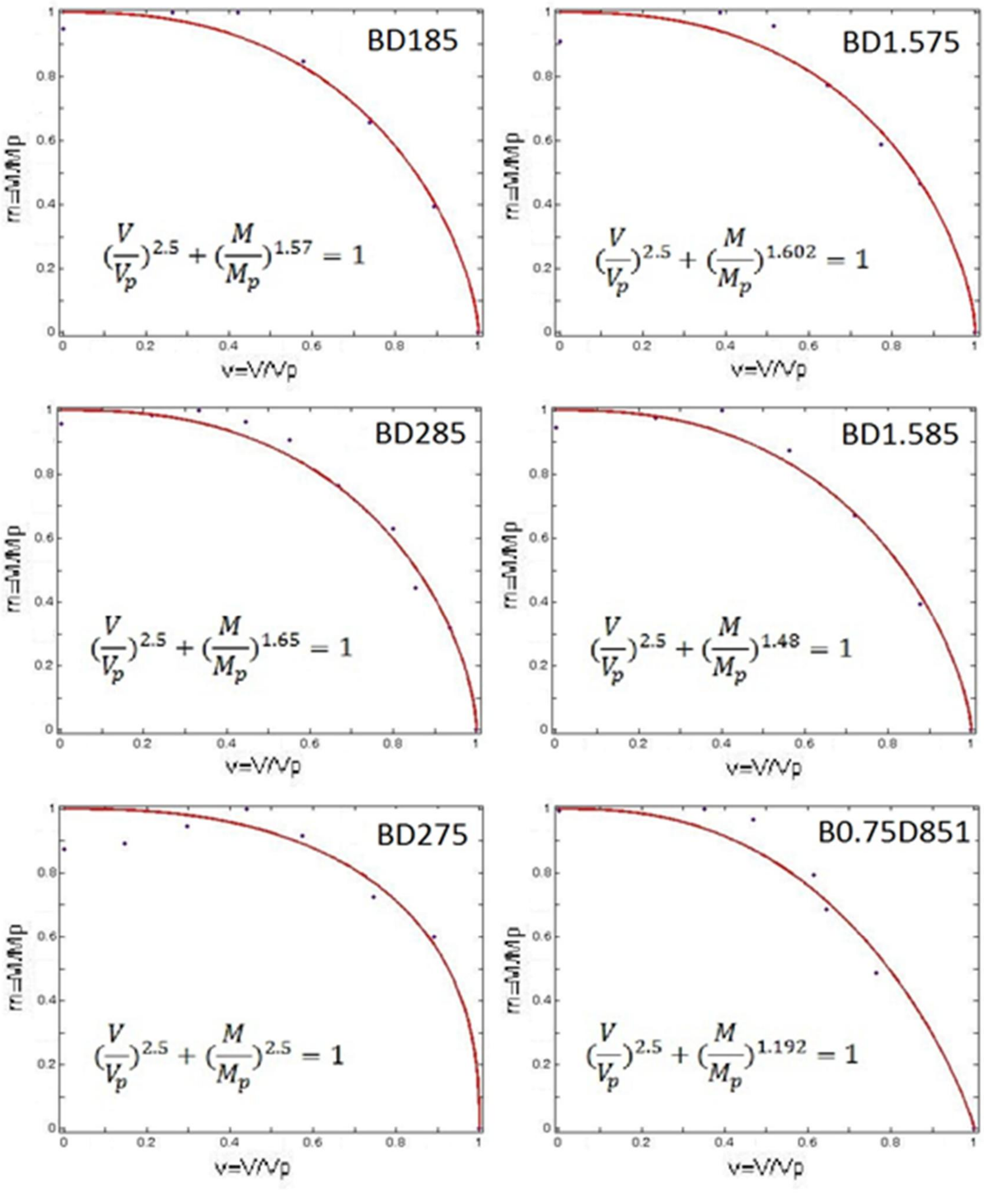


**Figure 4.1.** Interaction Curves and Equations for Each Model

It should be noted, as it is shown in Fig. 4.2, by changing the span length of the beam, the magnitude of moment changes, therefore, applying various magnitude of moments at the end of the beam considers different span lengths, so BD2853 and BD2856 models which were chosen for span length effect were eliminated from interaction formula calculation.

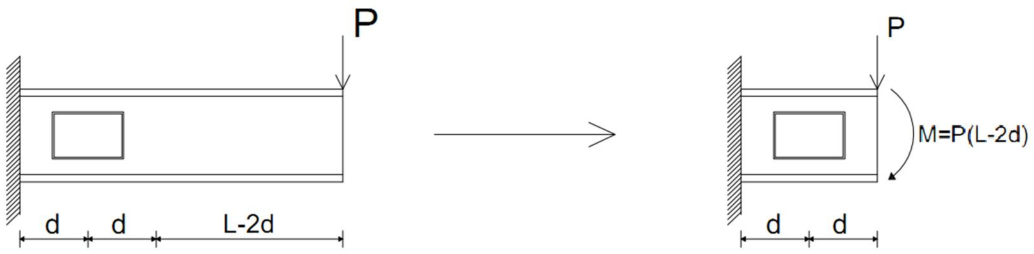


**Figure 4.2.** Effect of Span Length in the Magnitude of Moment Force

In order to derive an interaction formula that governs to RWS connections, all the interaction curves are drawn in a unit diagram as is shown in Fig. 4.3. The equation of the blue curve, which has the least closed area, can conservatively be considered as the governing interaction formula of the beam. According to the mentioned explanations, Eqn. 4.1 can be used for shear and moment relation.

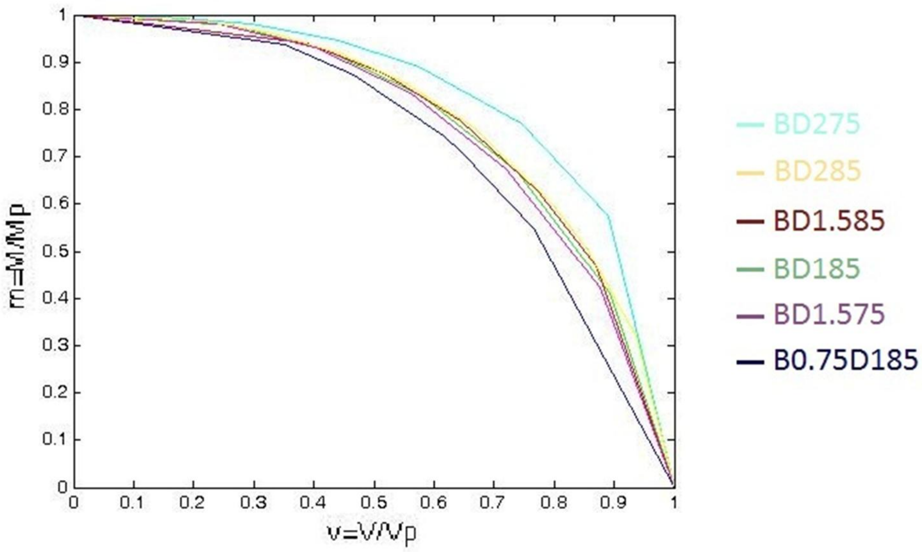


**Figure 4.3.** Interaction Curves of Models in a Unit Diagram

$$(\frac{V}{V_p})^{2.5} + (\frac{M}{M_p})^{1.192} = 1 \qquad (4.1)$$

Where V, M, $V_p$ and $M_p$ are applied shear force, moment force, plastic shear and plastic moment at the perforated section, respectively.

## 5. DESIGN PROCEDURE

The following steps describe the procedure for design of perforation dimensions, such that the connection components outside the opening remain elastic, in the fully yielded and strain-hardened condition that can be forced in the reduced section, under seismic loads and gravity loads:

1- Calculation of $V_{RWS}$ and $M_{RWS}$

Fig. 5.1 shows the pure shear at the center of reduced section. The maximum shear and moment at each T section will be equal to $\frac{V_{RWS}}{2}$ and $\frac{V_{RWS}b}{4}$, respectively. A shear-flexural interaction can be written for mentioned point as follows:

$$\left(\frac{\frac{V_{RWS}}{2}}{R_yC_{pr}(0.6F_y)A_w}\right)^2+\left(\frac{\frac{V_{RWS}b}{4}}{R_yC_{pr}F_yZ_{RWS-T}}\right)^2=1 \tag{5.1}$$

By simplifying the above equation, the $V_{RWS}$ can be easily calculated. In the above equation, $R_y$ is the ratio of the expected yield stress to the specified minimum yield stress and will be taken as 1.1, as suggested by AISC seismic provisions (2005). $C_{pr}$ is the factor to account for peak connection strength, including strain hardening and will be taken as 1.15 as prescribed in AISC prequalified connections (2005). $A_w$ is the total web area of single T section and $Z_{RWS-T}$ is the plastic section modulus of single T section of the reduced section including stiffener. b is the perforation length as illustrated in Fig. 2.1.

The maximum probable pure flexural strength at the center of the reduced web section, $M_{RWS}$, can be calculated as:

$$M_{RWS}=R_yC_{pr}Z_{RWS}F_y \tag{5.2}$$

Where $Z_{RWS}$ is the total plastic section modulus of the reduced section.

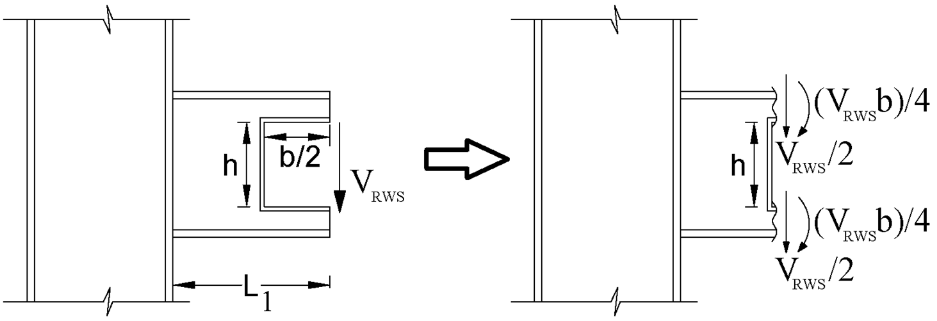


**Figure 5.1.** Pure Shear at the Center of Reduced Section

2- Calculation of $V_L$, $V_R$, $M_L$, and $M_R$

Due to shear force-flexural moment interaction in the reduced section, the expected moment at the reduced section will be less than $M_{RWS}$. The expected shear force and flexural moment at the left side ( $V_L$and $M_L$) and at the right side ( $V_R$ and $M_R$) of a beam are shown in Fig. 5.2.

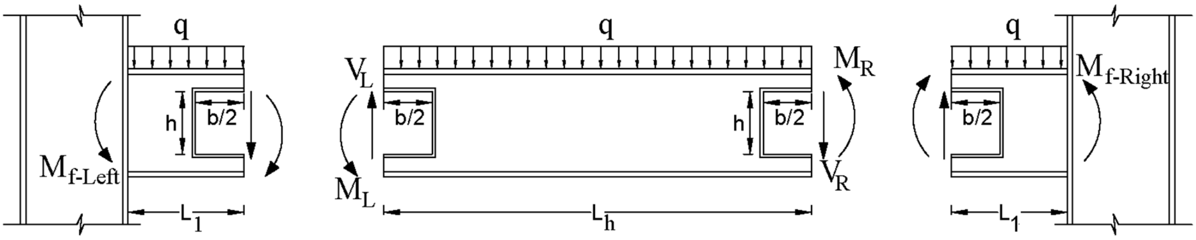


**Figure 5.2.** Structural Layout of Expected Forces of Beam

By solving the Eqns. 5.3 to 5.6, the values of $V_L$, $V_R$, $M_L$ and $M_R$can be obtained:

$$\left(\frac{V_L}{V_{RWS}}\right)^{2.5} + \left(\frac{M_L}{M_{RWS}}\right)^{1.192} = 1 \tag{5.3}$$
$$\left(\frac{V_R}{V_{RWS}}\right)^{2.5} + \left(\frac{M_R}{M_{RWS}}\right)^{1.192} = 1 \tag{5.4}$$
$$V_L = \frac{M_R+M_L}{L_h} + \frac{qL_h}{2} \tag{5.5}$$
$$V_R = \frac{M_R+M_L}{L_h} - \frac{qL_h}{2} \tag{5.6}$$

The first two above equations are interaction equations in the reduced section. The others are static equations for the middle part of the beam in the Fig. 5.2.

3- Calculation of $M_{f-Left}$ and $M_{f-Right}$

According to Fig. 5.2, the expected moment at the face of the column can be calculated by Eqns. 5.7 and 5.8:

$$M_{f-Left} = M_L + V_L L_1 + q\frac{L_1^2}{2} \tag{5.7}$$
$$M_{f-Right} = M_R + V_R L_1 - q\frac{L_1^2}{2} \tag{5.8}$$

4- The $M_{f-Left}$ and $M_{f-Right}$ of Eqn. 5.7 and Eqn. 5.8 must be less than the flexural strength of the ends of the girder, $\varphi M_n$:

$$M_{f-Left} \leq \varphi M_n \tag{5.9}$$
$$M_{f-Right} \leq \varphi M_n \tag{5.10}$$

**6. CONCLUSION**

Investigations were carried out on seismic behavior of steel MRFs with RWS connections, which includes a perforation at the beam's web, just away from the column's face. Eighteen finite element models were analyzed and a parametric study had been done on the effect of the perforation size, location and beam span length of RWS connections. In order to reduce the local buckling effect, a perimeter stiffener was considered around the opening.

Based on the analyses results, it is suggested that the height of the perforation be between 70% and 85% of the height of the beam to satisfy the seismic demands; Lower value for perforation height causes an institution of plastic strains in beam section at the column's face, which is not desired. On the other hand, greater value for perforation height causes significant strength degradation in reduced section.

Analyses results also showed that the connections have better seismic behavior when the width of the perforation is greater than the perforation's height and less than twice of it.

It is also concluded that the distance between the middle of the perforated section and the face of the column should be at least equal to height of the beam.

In connections with web opening, flexural strength is affected by shear force thus for achieving a proper design for RWS connections, an interaction formula between shear and moment is derived.
A step by step method for designing of these connections is also prescribed.

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