*Technical Note*

# Stability of Plate Girders in RBS Connections: A Numerical Approach

**A. Vasseghi[1*] and A. Tajoddini[2]**

1. Assistant Professor, Structural Engineering Research Center, International Institute of Earthquake Engineering and Seismology (IIEES), Tehran, I.R. Iran,
* Corresponding Author; email: vasseghi@iiees.ac.ir
2. Graduate Student, Islamic Azad University, Tehran, I.R. Iran

## ABSTRACT

*Steel moment frame connection with Reduced Beam Section (RBS) is one of several pre-qualified connections which have been proposed in FEMA 350 for use in moment resisting frame structures. Previous studies on behavior of RBS connections are limited to connections with rolled sections and design requirements have been developed for such sections. Large size rolled sections are not readily available in developing countries like Iran and steel frame structures are usually built using plate girders. In such structures the slenderness ratios of web and flanges could greatly influence the seismic performance of the RBS connection. In this paper the effect of slenderness ratios of web and flanges on the behavior of RBS connections is studied by nonlinear finite element analyses. The analyses simulate inelastic local buckling of the girder as ductility and energy dissipating capacity of the connection are directly influenced by such inelastic behavior. Twelve RBS connections with various web and flange slenderness ratios are analyzed to evaluate the effect of slenderness ratios on ductility of the connection. The results indicate that FEMA-350 requirements for maximum slenderness ratios of web and compression flange are too conservative. Connections in which the slenderness ratios of girder web and flanges exceeded the allowable limits by up to 30 percent have shown proper ductile behavior in the analyses.*

**Keywords:**
RBS Connection; Seismic; Compactness; Buckling

## 1. Introduction

The Northridge earthquake of January 17, 1994 is a significant event regarding the design of moment resisting steel frames and their connections. Following that earthquake and the widespread damages to steel moment frame connections, a number of pre-qualified connections have been proposed in *FEMA* 350 [1] for use in moment resisting frame structures. One such connection which have been studied extensively both experimentally and analytically is a connection with Reduced Beam Section (*RBS*). *RBS* connection which was first introduced by Plumier in 1990 [2], has been widely researched after the Northridge earthquake [3-13]. In this connection, section of the beam at a distance from face of column is reduced so that plastic hinge is formed in the reduced section at moments lower than those which induce the full inelastic demand on the connection. Figure (1) shows the most common reduced section geometries which include straight cut section, tapered cut section and radius cut section. Straight cut *RBS* connections have not shown proper performance in past tests. Stress concentration at the corner of the cut causes early fracture of the reduced section. Tapered cut *RBS* is theoretically rational because flange width along the length of reduced section varies with moment diagram and yielding along the length of reduced section is relatively uniform. However, inconsistent behavior including fractures at the corners has been observed in the previous tests. Radius cut *RBS*

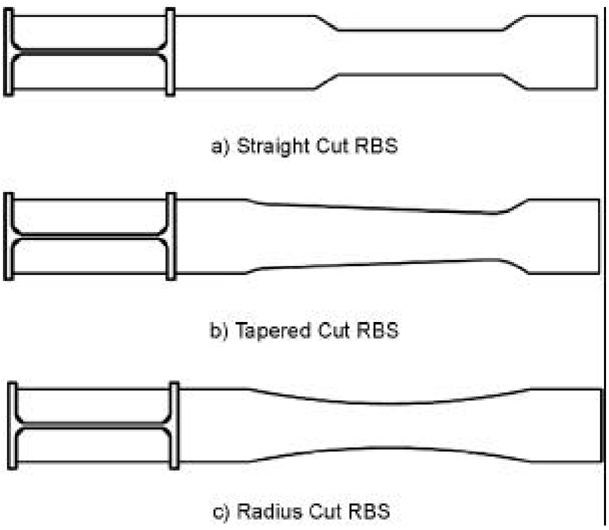


**Figure 1.** Different types of RBS.

connections with smooth transition from reduced section to full section are more commonly used in design. Performance of this type of connection in various tests has been very good. Test results indicate that radius cut *RBS* connections can attain significant plastic rotation capacity (more than 0.03 radian) without premature failure [14].

Previous studies on behavior of *RBS* connections are limited to connections with rolled sections and design requirements have been developed for such sections. Large size rolled sections are not readily available in developing countries like Iran and steel frame structures are usually built using plate girders. In such structures, the slenderness ratios of web and flanges could greatly influence the seismic performance of the *RBS* connection. In this paper, the effect slenderness ratios of web and flanges on the behavior of *RBS* connections is studied by nonlinear finite element analyses. The scope is limited to radius cut *RBS* connection.

## 2. Design Requirements

*FEMA* 350 guideline recommends the following formulas to define the length, *b*, the location, *a*, and depth, *c*, of flange reduction.

$$a \cong (0.5 - 0.75)b_f$$
$$b \cong (0.65 - 0.85)d_b$$
$$c \cong (0.2 - 0.25)b_f$$

Where $b_f$ and $d_b$ are flange width and beam height, respectively. Depth of flange reduction, *c*, should be determined within the allowable range, so that with formation of plastic hinge in the reduced section, moment at face of column is less than plastic moment capacity of full section.

In order to prevent premature local buckling of reduced section, the slenderness ratios of web and flanges is limited based on the following equations:

$$\frac{b_f}{2t_f} \le \frac{137}{\sqrt{F_{y-Mpa}}}$$

$$\frac{h_w}{t_w} \le \frac{1100}{\sqrt{F_{y-MPa}}}$$

The maximum slenderness ratio of flange $(b_f / 2t_f)$ is consistent with the seismic criteria in *AISC* specification [15]. However, it is recommended that this ratio be determined based on the flange width $(b_f)$ measured at the ends of the center 2/3 of the reduced section of beam as shown in Figure (2).

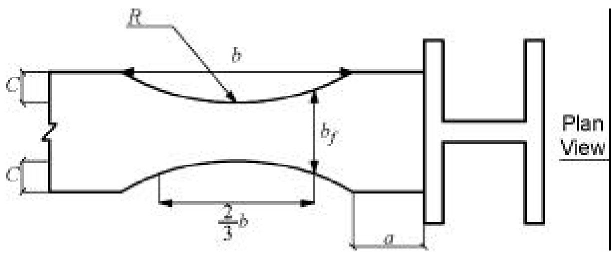


**Figure 2.** Geometric parameters of RBS connection.

The maximum allowable web slenderness ratio is less than the seismic requirement in *AISC* specification $(250/\sqrt{F_y})$. The allowable web slenderness ratio is based on statistical study of 55 full-scale tests [16] with web slenderness ratios less than $(418/\sqrt{F_y})$. Lack of test results beyond this limit is believed to be one of the reasons for specifying this limit for web slenderness ratio. The other reason may be that almost all the rolled sections in United-States satisfy this limit.

## 3. Analytical Study

Behavior of radius cut *RBS* connection is investigated by finite element analysis method using *ANSYS* software. Geometric nonlinearity due to local buckling and nonlinear material behavior are included in the analytical model. The main objective of this study is to evaluate the effect of web and flange slenderness on connection's behavior.

### 3.1. Finite Element Model

Figure (3) shows the analytical model of *RBS* connection which has the same dimensions as the specimen which was tested by Pantelides et al [6]. This model that simulates the connection of beam to an external column of a building consists of half of beam (3.65 *m long*) and two half columns in upper and lower stories (total length = 4.92*m*). Web and flanges of beam and column are modeled using nonlinear shell elements (shell 181). This element which has 4 nodes with 6 degrees of freedom on each node is suitable for nonlinear analysis with large displacements. The ends of columns are restrained against translations but they are free to rotate (hinged boundary condition). Loading are applied by assigning vertical displacement at the end of beam. Steel material was modeled bi-linearly with elastic modulus of 200000*MPa*, plastic modulus of 200*MPa* and yield strength of 355*MPa*. Plasticity theory with bi-linear kinematic hardening is used to model the material behavior. Strain hardening of steel material is not included in the analytical model.

In all analytical samples, the column's sections is *W*14x283 with following dimensions in *SI* unit:

Column web: 5.3*cm* x 40.9*cm*

Column flanges: 3.3*cm* x 42.4*cm*

Flange width and web height of girders are 26.6*cm* and 77*cm* respectively, and web and flange thicknesses are variable. Figure (4) displays the location and dimensions of the radius cut in the flanges.

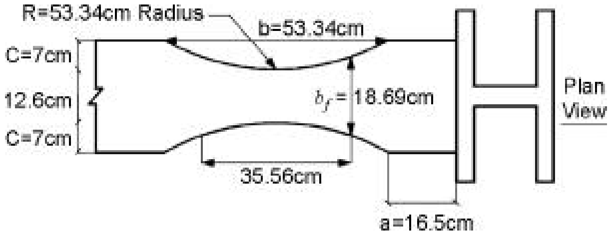


**Figure 4.** Dimensions of the radius cut.

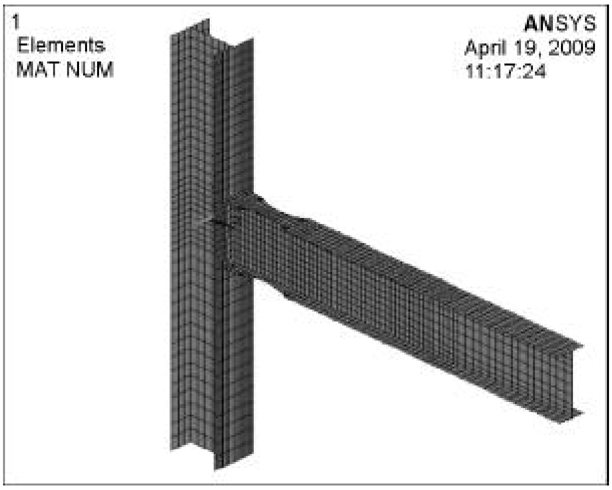


**Figure 3.** Finite element model of the RBS connection.

### 3.2. Effect of Slenderness Ratios on Connection Behavior

In order to evaluate the effect of slenderness ratios of girder web and flanges on behavior of *RBS* connections, 12 sections with different slenderness ratios are analyzed. According to *FEMA*350, the dimensions and slenderness ratios of web and flanges and maximum limit are listed in Table (1). In sections no. 1, 4 and 7, web and flange slenderness ratios are within *FEMA* 350 allowable limit. In sections no. 2, 3, 5, 6, 8 and 9, the flange slenderness ratios are within the allowable limit but web slenderness ratios exceed the allowable limit. In section

**Table 1.** Girder dimensions and slenderness ratios.

| Section No. | Web Height (cm) | Web Thickness (cm) | Flange Width (cm) | Flange Thickness (cm) | Web Slenderness | Flange Slenderness | FEMA350 Max | | Section Type |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Web Slenderness | Flange Slenderness | |
| 1 | 77 | 1.57 | 26.6 | 2.54 | 49 | 3.68 | 58.38 | 7.27 | Allowed |
| 2 | 77 | 1 | 26.6 | 2.5 | 77 | 3.74 | 58.38 | 7.27 | Not Allowed |
| 3 | 77 | 0.5 | 26.6 | 2.5 | 154 | 3.74 | 58.38 | 7.27 | Not Allowed |
| 4 | 77 | 1.5 | 26.6 | 2 | 51.3 | 4.67 | 58.38 | 7.27 | Allowed |
| 5 | 77 | 1 | 26.6 | 2 | 77 | 4.67 | 58.38 | 7.27 | Not Allowed |
| 6 | 77 | 0.5 | 26.6 | 2 | 154 | 4.67 | 58.38 | 7.27 | Not Allowed |
| 7 | 77 | 1.5 | 26.6 | 1.5 | 51.3 | 6.23 | 58.38 | 7.27 | Allowed |
| 8 | 77 | 1 | 26.6 | 1.5 | 77 | 6.23 | 58.38 | 7.27 | Not Allowed |
| 9 | 77 | 0.5 | 26.6 | 1.5 | 154 | 6.23 | 58.38 | 7.27 | Not Allowed |
| 10 | 77 | 1.5 | 26.6 | 1 | 51.3 | 9.34 | 58.38 | 7.27 | Not Allowed |
| 11 | 77 | 1 | 26.6 | 1 | 77 | 9.34 | 58.38 | 7.27 | Not Allowed |
| 12 | 77 | 0.5 | 26.6 | 1 | 154 | 9.34 | 58.38 | 7.27 | Not Allowed |

no. 10, the web slenderness ratio is within the limit but flange slenderness ratio exceeds the limit. Finally, in sections no. 11 and 12, both web and flange slenderness ratios are more than the allowable limits.

Behaviors of connections are investigated under monotonic loading condition where displacement at end of the girder is increased until formation of plastic hinge or local buckling of the reduced section. Results of these analyses are displayed as moment rotation curves in Figure (5a)-(5d). In these curves, moment at face of column is normalized with respect to plastic moment capacity of the full section and rotation is normalized with respect to rotation at first yield. This figure indicates that maximum moment capacities of sections with very large web slenderness ratio (154) are well below the plastic moment capacity. These sections failed due to local buckling of the reduced section. This figure also shows that all sections with web slenderness ratio of 51.3 and 77 have good ductility. The limiting web slenderness ratio based on *FEMA* 350 is 58.4 which is significantly less than 77. Considering the ductile behavior of sections with web slenderness ratio of 77 (32% more than allowable limit), it seems that *FEMA* 350 limitation on web slenderness is too conservative. Figure (5d) indicates that the behavior of sections no. 10 and 11, in which the flange slenderness ratio is 9.37 (28% more than allowable limit), is also very ductile. Therefore, based on the results of these analyses, the *FEMA* 350 limitation on flange slenderness is also very conservative.

Figure (5) also indicates that rotational ductility of sections with web slenderness ratio of 51.3 ($t_w$=1.5*cm*) and 77 ($t_w$=1.0*cm*) are larger than 6, but there are noticeable difference between their moment capacities. The normalized moment (moment at face of column divided by plastic moment of full section) is always higher for the less slender or thicker web. This is because for a given web height, web contribution to the moment capacity of the reduced section increases with web thickness. As a result, the moment at face of column increases with decreasing web slenderness ratio. Such increase is not appropriate when moment at face of column exceed the plastic moment capacity of the full section ($M/Mp$>1). For such cases (e.g. sections 4, 7, 10 and 11), deeper cuts would be required to further reduce the moment capacity of the reduced section.

Figure (6) displays the distribution of equivalent plastic strain in sections no. 4, 11 and 9. In section no. 4, where ratios of web and flange slenderness are within allowable limit, plastic hinge is formed in the reduced section. In section no. 11, plastic hinge is

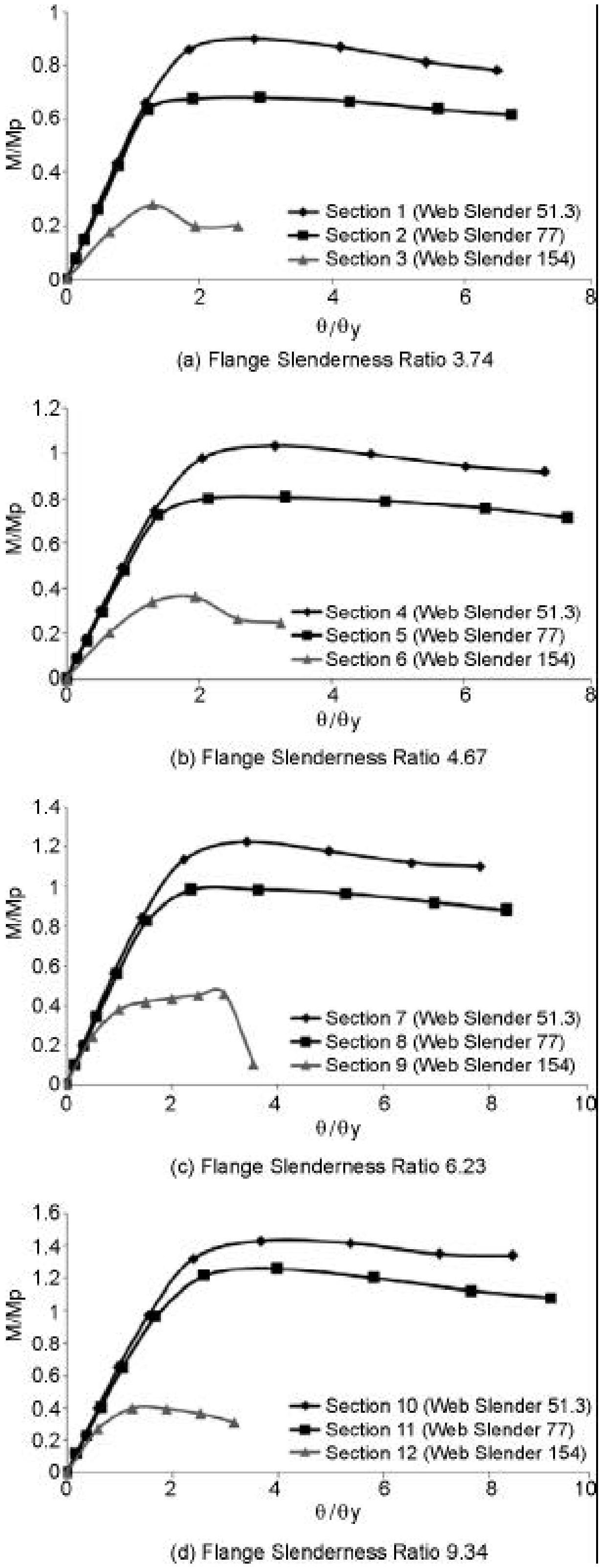


**Figure 5.** Moment rotation curves.

also formed in the reduced section. In this section, flange slenderness ratio (9.34) and web slenderness ratio (77) are 28% and 32% more than allowable limit, respectively. In section no. 9, where the web slenderness ratio (154) is considerably larger than the allowable limit, premature local buckling occurred before formation of plastic hinge in the reduced section.

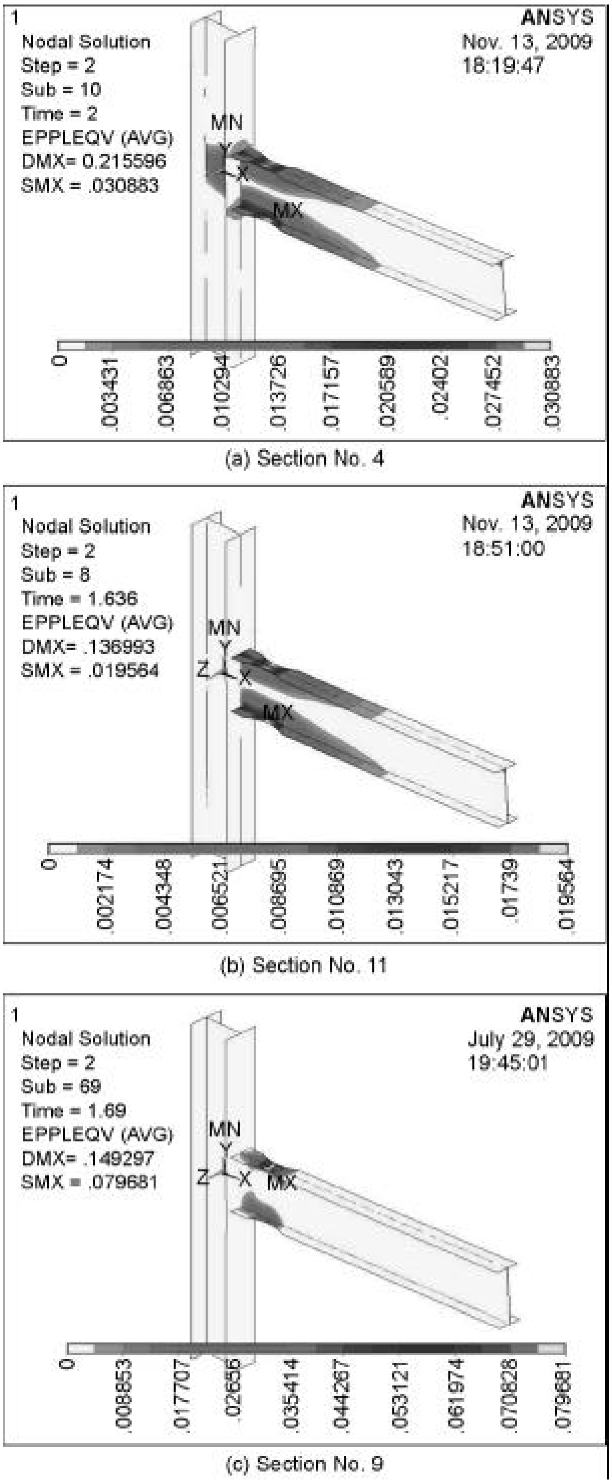


**Figure 6.** Distribution of equivalent plastic strain.

## 4. Conclusion

The behavior of radius cut *RBS* connections is studied using nonlinear finite element analysis method. The objective of this study is to evaluate the effect of beam web and flange slenderness ratio on the behavior of *RBS* connections. Results of this study indicate that *FEMA*350 limitation on slenderness ratios of web and flanges are very conservative. Connections in which the slenderness ratios of beam web and flanges exceeded the allowable limits by about 30 percent have shown proper ductile behavior in the analyses. However, it is necessary to conduct experimental study on sections with high slenderness ratios to verify the analytical results.

## References

1. FEMA 350 (2000). *Recommended Seismic Design Criteria for New Steel Moment Frame Buildings*, Federal Emergency Management Agency, Washington, D.C.

2. Plumier, A. (1990). “New Idea for Safe Structure in Seismic Zones”, *IABSE Symposium*, Brussels, Belgium.

3. Plumier, A. (1997). “The Dogbone: Back to the Future”, *Engineering Journal*, **35**(4), 61-67.

4. Popov, E.P., Yang, T.S., and Chang, S.P. (1998). “Design of Steel MRF Connections Before and After 1994 Northridge Earthquake”, *Engineering Structures*, **20**(12), Elsevier, London.

5. Popov, E.P., Balan, T.A., and Yang, T.S. (1998). “Post Northridge Earthquake Seismic Steel Moment connections”, *Earthquake Spectra*, **14**(4), 659-677.

6. Pantelides, C.P., Okahashi, Y., and Reaveley, L.D. (2004). “Experimental Investigation of Reduced Beam Section Moment Connections without Continuity Plates”, *Earthquake Spectra*, **20**(4), 1185-1209.

7. Engelhardt, M.D., Fry, G., Johns, S., Venti, M., and Holliday, S. (2000). “Behavior and Design of Radius-Cut, Reduced Beam Section Connections”, SAC Report 00/17, SAC Joint Venture.

8. Engelhardt, M.D., Winneberger, T., Zekany, A.J., and Potyraj, T.J. (1998). “Experimental Investigation of Dogbone Moment Connections”,

*Engineering Journal*, **35**(4), American Institute of Steel Construction, 1998.

9. Engelhardt, M.D., Fry, G.T., and Jones, S.L. (2002). "Experimental Evaluation of Cyclically Loaded Reduced Beam Section Moment Connection", *Journal of Structural Engineering*, *ASCE*, **128**(4), 441-451.

10. Chen, S.J., Yeh, C.H., and Chu, J.M. (1996). "Ductile Steel Beam-Column Connections for Seismic Resistance", *Journal of Structural Engineering*, *ASCE*, **122**(11), 1292-1299.

11. Iwankiw, R.N. and Carter, C.J. (1996). "The Dogbone: A New Idea to Chew on", Modern Steel Construction, *AISC*, **36**(4), 18-23, Chicago IL.

12. Moslehi Tabar, A. and Deylami, A. (2005). "Instability of Beams with Reduced Beam Section Moment Connections Emphasizing the Effect of Column Panel Zone Ductility", *Journal of Constructional Steel Research*, **61**(11), 1475-1491.

13. Deylami, A. and Moslehi Tabar, A. (2008). "Experimental Study on the Key Issues Affecting Cyclic Behavior of Reduced Beam Section Moment Connections", *The 14th World Conference on Earthquake Engineering*, Beijing, China.

14. FEMA-355D (2000). "State of the Art Report on Connection Performance", Prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

15. AISC (1997). "Seismic Provisions for Structural Steel Buildings", American Institute of Steel Construction, Chicago, Illinois.

16. Uang, C.M. and Fan, C.C. (2001). "Cyclic Stability Criteria for Steel Moment Connections with Reduced Beam Section", *Journal of Structural Engineering*, *ASCE*, **127**(9), 1021-1027.