Computational Engineering and Physical Modeling 4-3 (2021) 43-54

Contents lists available at **CEPM**

Computational Engineering and Physical Modeling

Journal homepage: www.jcepm.com

Optimization of Reduced Beam Sections (RBS) for Ductile Detailing of Seismic Joint Connections Using Finite Element Analysis (FEA)

J.P. Cimagala*

Department of Civil Engineering, College of Engineering and Architecture, University of the Cordilleras, Baguio City, Philippines

Corresponding author: *jeffcimagala@yahoo.com.ph*

https://doi.org/10.22115/CEPM.2021.273581.1153

ARTICLE INFO

Article history:
Received: 16 February 2021
Revised: 19 May 2021
Accepted: 09 August 2021

Keywords:
Special moment resisting frames (SMRF);
Plastic hinge;
Reduced beam section (RBS);
Ductile detailing.

ABSTRACT

Steel structures used as Special Moment Resisting Frames (SMRF) designed to resist lateral loads (due to wind and seismic) are expected to undergo large inelastic deformations, hence the ductility requirements are explicitly stated in almost all standards. In any given frame, inelastic deformations should occur in the horizontal elements (e.g. beams) in the form of plastic hinges. Most structural analysis can be performed assuming the beam-column joint (nodes) as a fixed (rigid) connection, however, this may mean that hinging may occur at the connection and thus possibly affect the column through the flange or web connection. In order to ensure a ductile system can be achieved, special detailing requirements are necessary. Among the available methods require the use of Reduced Beam Sections (RBS) adjacent to the beam-column connection to warrant the strong-column/weak-beam design philosophy. The main objective of this paper is to optimize the geometry of the RBS using Finite Element Analysis (FEA) in conjunction with the available standards e.g. BS EN 1998-3 and ANSI/AISC 358-16. While standard codes of practice provide the range of values that can be used in determining the geometry of the RBS, it would be beneficial for a designer to come up with basic rules-of-thumb that can be applied in actual design calculations.

How to cite this article: Cimagala, J. P. (2021). Optimization of Reduced Beam Sections (RBS) for Ductile Detailing of Seismic Joint Connections Using Finite Element Analysis (FEA). Computational Engineering and Physical Modeling, 4(3), 43–54. https://doi.org/10.22115/cepm.2021.273581.1153

1. Introduction

Structural steel elements (columns and beams) in Special Moment Resisting Frames (SMRF) are designed to resist the lateral loads derived from wind or seismic excitation. However, the overall strength of the frames can only be as strong as the connections used to join the elements at the nodes. Prior to the Northridge earthquake (1994), welded-flange-bolted-web moment resisting connections were typically used as shown in Fig. 1 [1]. However, in the aftermath of the earthquake, steel-framed buildings suffered severe damages in the connections of moment-resisting frames. The most commonly observed damage was in bottom flange welds that exhibited brittle connection failure as highlighted in the report by [2]. The occurrence of observed behavior of the connections prompted researchers to review the existing design methodologies at the time regarding ductility and investigate why the expectations were not met and how ductility can be enhanced [3]. The importance of designing and detailing a proper connection to achieve the required ductility has been aptly summarized by [4] considering the fundamental rule stated as follows: the resistance of a ductile mechanism in the connection or close to the connection must be lower than the resistance of any potential failure mechanism which is brittle or has low ductility. This will ensure yielding of the ductile zone prior to the failure of the neighboring elements.

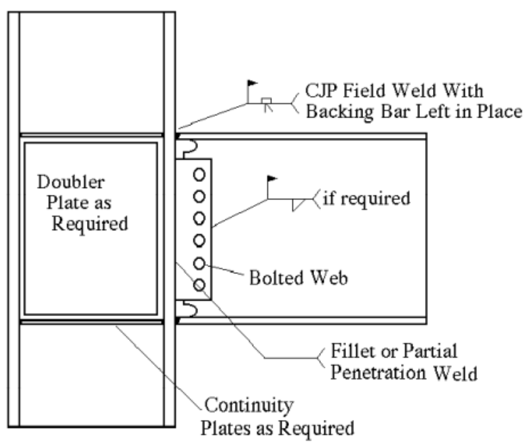

Fig. 1. Fully Restrained Weld-Flange-Bolted-Web Connection [1].

As a result of the studies conducted, numerous types of joint connections (beam to column) were developed to ensure suitability for such kind of loadings. One of them is the so called Reduced Beam Sections (RBS), a.k.a. dog-bone due to its familiar shape resembling a dog bone. The idea was developed by [5] to induce a specific weakened zone outside the connections but close to them wherein yielding may take place in a safe and ductile manner.

With the introduction of this mechanism, various researchers [6–10] adopted the concept and investigated further the effects of introducing such mechanism on the behavior of the joints and the moment-resisting frame when subjected to lateral loads.

In recent practice, the widely adopted geometric shape of the RBS is shown in Fig. 2 [11] relative to the beam edge. The following parameters are used to define the geometry. Table 1 indicates the differences in the proposed dimensions of the cut based on the two aforementioned standard codes of practice. It should be noted that the equation for the radius, r, of the cut is the same irrespective of the code.

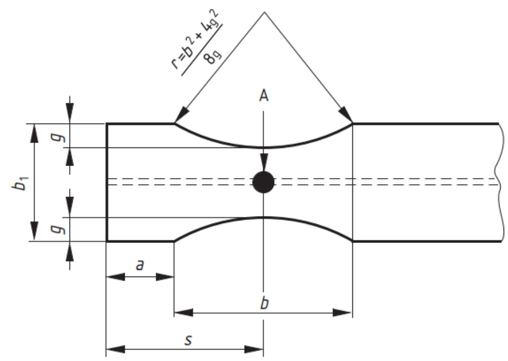

Fig. 2. Flange reduction geometry [11].

Where:

a = horizontal distance from face of column flange to start of an RBS cut, mm
b = length of RBS cut, mm
g = depth of cut at center of reduced beam section, mm
s = location of the intended plastic hinge (center of the RBS), mm
r = radius of curvature of the flange cuts (top and bottom), mm

Table 1
Comparison of the geometric variables.

Reference Standard	Dimensions				
	a (1)	b (2)	g (3)	s (4)	r (5)
ANSI/ AISC 358-16	$\geq 0.25b_f$ $\leq 0.75b_f$	$\geq 0.65d$ $\leq 0.85d$	$\geq 0.10b_f$ $\leq 0.25b_f$	$a + \frac{b}{2}$	$\frac{b^2 + 4g^2}{8g}$
BS EN 1998-3	$0.60b_f$	$0.75d$	$0.20b_f$ $\leq 0.25b_f$	$a + \frac{b}{2}$	$\frac{b^2 + 4g^2}{8g}$

Given that standards provide differing values; it is the objective of this paper to come up with a simpler rule of thumb to follow with the secondary goal of achieving a value engineered section. Using the defined range of values, a series of investigations shall be carried out using a Finite Element Analysis software to determine the RBS geometry that will produce the largest section capacity across the assumed plastic hinge location.

2. Code based design procedures

The intentional reduction of the section through the cut-outs on the flanges induces a weakened element adjacent to the connection. Prior researches on the use of RBS in seismic-resistant steel moment frames have shown that it is capable of providing ductile and reliable performance [12]. In the US standard [13], joints utilizing beams with an RBS is categorized as a pre-qualified seismic moment connection used for Special Moment Frame and Intermediate Moment Frame systems. Fig. 3 shows the typical sub-frame assembly incorporating the reduced beam sections (RBS) from the supporting columns.

Beams with RBS's on both ends maybe jointed into the supporting columns using bolted or welded connections. Previous research by Lee et al. [14] however, seem to indicate that specimens with a bolted web connection performed poorly due to premature brittle fracture of the beam flange at the weld access hole whereas welded-web RBS moment connections' panel zone could easily develop a plastic rotation of 0.01 rad. without distressing the beam flange groove welds. Consequently, this paper will utilize the findings on welded connections and shall be used for both beam flanges and web.

The following steps highlight the procedures in designing RBS's as per Eurocode [11] and ANSI/AISC [13].

Eurocode [11]	ANSI/AISC [13]
1. Compute the distance of the beginning of the RBS from the column face, a, and the length over which the flange will be reduced, b, as per Table 1. 2. Compute the distance of the intended plastic hinge section at the centre of the RBS, s, from the column face. 3. Determine the depth of the flange cut (g) on each side as per Table 1. 4. Compute the plastic modulus (Z_{RBS}) and the plastic moment ($M_{pl,Rd,RBS}$) of the plastic hinge section at the centre of the RBS. $Z_{RBS} = Z_x - 2gt_f(d - t_f)$ (6) $M_{pl,Rd,RBS} = Z_{RBS} f_{yb}$ (7) 5. Compute the shear force ($V_{pl,RBS}$) in the section of plastic hinge formation from equilibrium of the beam part (L') between the two intended plastic hinges. $V_{pl,RBS} = \frac{2M_{pl,Rd,RBS}}{L'} + \frac{wL'}{2}$ (8) 6. Compute the beam plastic moment away from the RBS, $M_{pl,Rd,b}$. $M_{pl,Rd,b} = Z_b f_{yb}$ (9) 7. Verify that $M_{pl,Rd,b}$ is greater than the bending moment that develops at the column face when a plastic hinge forms at the centre of the RBS. 8. Check the width-to-thickness ratios at the RBS to prevent local buckling. 9. Compute the radius, r, of the cuts in both top and bottom flanges over the length b of the RBS of the beam. 10. Check that the fabrication process ensures the adequate surface roughness (i.e. between 10 and 15 μm) for the finished cuts and that grind marks are not present.	1. Choose trial values for the beam sections, column sections and RBS dimensions a, b and g (Fig. 1) subject to the limits shown on Table 1. 2. Compute the plastic section modulus at the center of the reduced beam section: $Z_{RBS} = Z_x - 2gt_f(d - t_f)$ (6') 3. Compute the probable maximum moment, M_{pr}, at the center of the reduced beam section: $M_{pr} = C_{pr} R_y F_y Z_{RBS}$ (10) $C_{pr}C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.20$ (11) 4. Compute the shear force at the center of the reduced beam sections at each end of the beam. 5. Compute the probable maximum moment at the face of the column. $M_f = M_{pr} + V_{RBS}(s)$ (12) 6. Compute M_{pe}, the plastic moment of the beam based on the expected yield stress. $M_{pe} = R_y F_y Z_x$ (13) 7. Check the flexural strength of the beam at the face of the column. $M_f \leq \emptyset_d M_{pe}$ (14) 8. Determine the required shear strength, V_u, of beam and beam web-to-column connection. $V_u = \frac{2M_{pr}}{L_h} + V_{gravity}$ (15) 9. Design the beam web-to-column connection. 10. Check continuity plate requirements. 11. Check column-beam relationship limitations. 12. Check column panel zone.

Where:

Z_{RBS} = plastic section modulus at center of reduced beam section, mm^3
Z_x = plastic section modulus about x-axis, for full beam cross section, mm^3
t_f = thickness of beam flange, mm
C_{pr} = factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement and other connection conditions
R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y
R_y = 1.5 (ASTM A36); 1.3 (ASTM A1043); 1.1 (ASTM A572) from [15]
F_y = specified minimum yield stress of yielding element, MPa
f_{yb} = yield strength of the steel in the beam taken equal to the nominal value multiplied by the over-strength factor γ_{ov} = 1.25 for the steel of the beam, MPa from [7]
M_f = probable maximum moment at face of column, N-mm
s = distance from face of column to plastic hinge, mm
V_{RBS} = larger of the two values of shear force at center of the reduced beam section at each end of beam, N
L_h = distance between plastic hinge locations, mm
$V_{gravity}$ = beam shear force resulting from worst case gravity load combination, N
V_u = required shear strength of beam and beam web-to-column connection, N

Step 3 [13] may be simplified by using a factor of 1.15 that accounts for strain hardening according to [12]. Therefore, the probable maximum moment, M_{pr}, may be expressed in the form shown below.

$$M_{pr} = 1.15\, R_y\, F_y\, Z_{RBS} \tag{16}$$

Similarly, V_{RBS} can be calculated using equations (15) and (16).

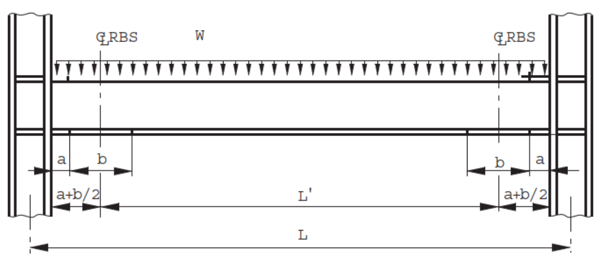

Fig. 3. Typical sub-frame assembly with reduced beam sections (RBS) from [11].

$$V_{RBS} = \frac{2M_{pr}}{L'} + \frac{wL'}{2} \tag{17}$$

$$V'_{RBS} = \frac{2M_{pr}}{L'} - \frac{wL'}{2} \tag{18}$$

Where:

V_{RBS} = shear force at the center of the RBS cut at the beam end (larger shear force), N
V'_{RBS} = shear force at the center of the RBS cut at the beam end (smaller shear force), N
L' = distance between centers of RBS cuts, m
w = uniformly distributed gravity load on beam, N/m

Lastly, when verifying the flexural strength of the beam at the face of the column according to equation (12), the ratio of M_f to M_{pe} (or $M_{cf,Ed}$ to $M_{pl,Rd,b}$) shall be in the range of 0.85 to 1.0. Where it falls outside this range, the values of a, b and g shall be adjusted accordingly.

3. FEA modeling and analysis procedures

Based on the steps enumerated above, it can be inferred that both codes have similar approach in designing RBS moment connections. Furthermore, determining the most suitable flange cut dimensions is an iterative process. Therefore, the objective of this study is to recommend a set of flange cut dimensions that are not only economical but also ensures the least section capacity reduction (or the highest M_f/M_{pe} ratio) as well as in compliance with the provisions of the relevant standard code of practice.

IDEA Statica FEM software has been chosen to analyze the capacity of the connections due to its capability to provide rotational stiffness utilizing the Component Based Finite Element Modeling Method (CBFEM) [16]. Fig. 4 below shows the typical 3D detail of the RBS connection.

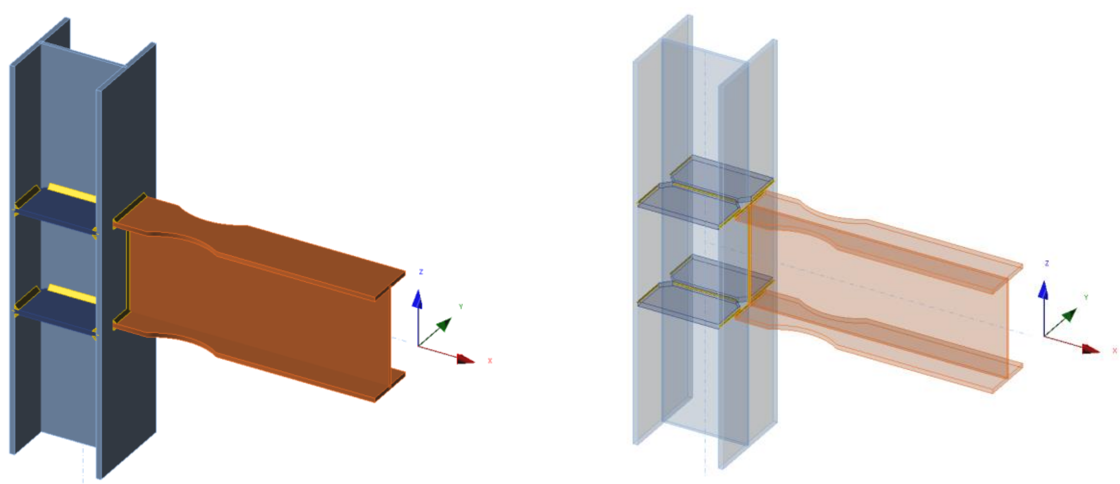

Fig. 4. Finite Element Model of the RBS Connection.

HEA200 will be used as the column supporting the beam whereas an IPE220 will be used for the main beam connected to the column flange through fillet welds of code-based minimum thickness (4.6mm on beam flanges, 3mm on beam web and 5mm on stiffeners). The theoretical beam length has been set to 6m for the purpose of calculating the stiffness. Column flange stiffeners are 10mm thick.

The analysis will be carried out in four stages using the range of values stipulated in ANSI/AISC [13]. The first stage shall be used to determine the most efficient flange cut dimensions i.e. highest ratio of applied moment to plastic moment capacity, of the RBS using the combinations of minimum and maximum values as shown on Table 2. The values of the parameters are as follows: a_{MIN} = 55mm; a_{MAX} = 82.5mm; b_{MIN} = 143mm; b_{MAX} = 187mm; g_{MIN} = 11mm and g_{MAX} = 27.5mm.

Table 2
Combinations of minimum and maximum RBS flange cut dimensions.

Model	a	b	g
1	MIN	MIN	MIN
2	MIN	MIN	MAX
3	MIN	MAX	MIN
4	MIN	MAX	MAX
5	MAX	MIN	MIN
6	MAX	MIN	MAX
7	MAX	MAX	MIN
8	MAX	MAX	MAX

The second stage shall be used to determine the effect of varying the length of the RBS, b. Since the objective is to minimize the reduction in capacity, the depth of the cut at the center of the RBS, g, shall also be minimized. Correspondingly, the distance from the end of the beam will also be minimized. Values of the flange cut dimensions are shown in Table 3.

Table 3
RBS flange cut dimensions with variable "b" values.

Model	a	b	g
9	MIN	0.65d	MIN
10	MIN	0.70d	MIN
11	MIN	0.75d	MIN
12	MIN	0.80d	MIN
13	MIN	0.85d	MIN

The third stage shall be used to determine the effect of varying horizontal distance from face of column flange to start of an RBS cut, a. In the same manner, the depth of the cut at the center of the RBS, g, shall also be minimized together with b. Values of the flange cut dimensions are shown in Table 4.

Table 4
RBS flange cut dimensions with variable "a" values.

Model	a	b	g
14	$0.50b_f$	MIN	MIN
15	$0.55b_f$	MIN	MIN
16	$0.60b_f$	MIN	MIN
17	$0.65b_f$	MIN	MIN
18	$0.70b_f$	MIN	MIN
19	$0.75b_f$	MIN	MIN

It should be noted that for an RBS moment connection, the protected zone is defined as portion of beam between the face of the column and the end of the reduced beam section cut farthest from the face of the column. In these areas, no attachment of lateral bracing shall be made to the beam and no penetrations will be made through the beam web at these locations. Fig. 5 below shows the location of the protected zone [17].

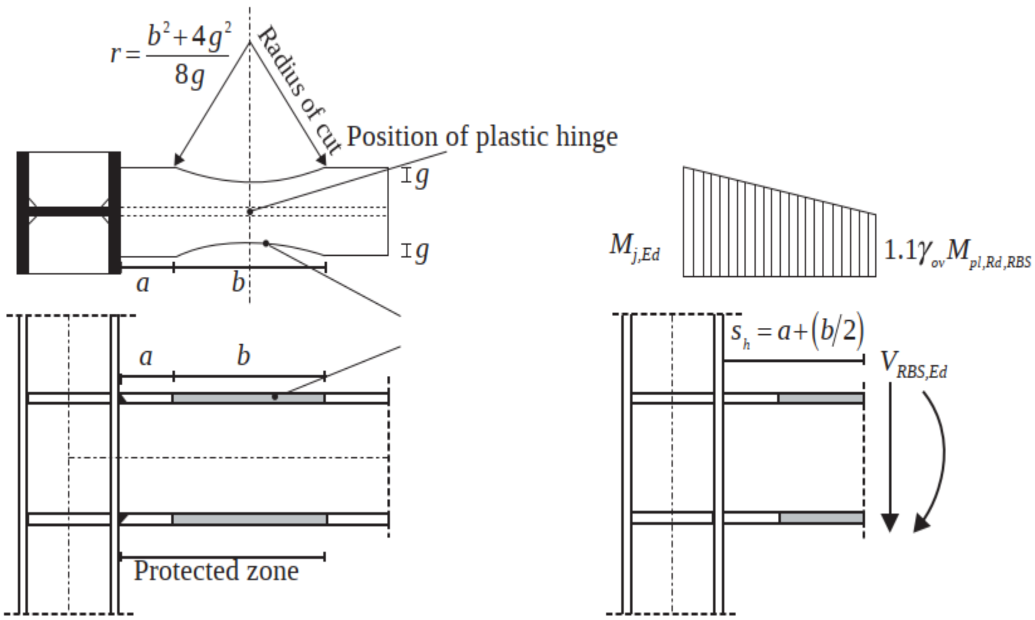

Fig. 5. Free body diagram of a welded RBS moment connection [16].

The fourth stage will be the modeling and analysis of the deemed most efficient flange cut dimensions. It should be emphasized that each model analysis will be carried out assuming the applied load is equal to the plastic moment capacity with no applied factors, that is, $M_{pl,RBS} = M_{cf,Ed} = Z_{RBS}F_y$ assuming the full capacity of the section at the center of the RBS will be utilized.

4. Results

Tables 5 to 8 show the results of the analysis for each model indicating the maximum bending moment resistance, $M_{j,Rd}$. The last column of each table provides the ratios of the applied load and the connection capacity. This also indicates how much can be utilized prior to failure. Note that the applied loads are equal to the theoretical values of the plastic moment at the center of the RBS section, thus, any increase in the loads may lead to subsequent connection or member failure.

Table 5
Results of stage 1 analysis.

	a	b	g	$M_{cf,Ed}$	$M_{j,Rd}$	%
Model	mm	mm	mm	kNm	kNm	$M_{pl,RBS}$ / $M_{j,Rd}$
1	*55*	*143*	*11*	*66.64*	*70.30*	*94.80%*
2	55	143	27.5	49.04	53.80	91.16%
3	55	187	11	66.64	69.90	95.34%
4	55	187	27.5	49.04	52.90	92.71%
5	82.5	143	11	66.64	69.80	95.48%
6	82.5	143	27.5	49.04	53.70	91.33%
7	82.5	187	11	66.64	69.80	95.48%
8	82.5	187	27.5	49.04	52.90	92.71%

Table 6
Results of stage 2 analysis.

	a	b	g	$M_{cf,Ed}$	$M_{j,Rd}$	%
Model	mm	mm	mm	kNm	kNm	$M_{pl,RBS}$ / $M_{j,Rd}$
9	55	143	11	66.64	70.30	94.80%
10	*55*	*154*	*11*	*66.64*	*70.40*	*94.66%*
11	55	165	11	66.64	70.30	94.80%
12	55	176	11	66.64	70.00	95.20%
13	55	187	11	66.64	69.80	95.48%

Table 7
Results of stage 3 analysis.

	a	b	g	$M_{cf,Ed}$	$M_{j,Rd}$	%
Model	mm	mm	mm	kNm	kNm	$M_{pl,RBS}$ / $M_{j,Rd}$
14	55	143	11	66.64	70.30	94.80%
15	60.5	143	11	66.64	70.90	93.99%
16	*66*	*143*	*11*	*66.64*	*71.00*	*93.86%*
17	71.5	143	11	66.64	68.70	97.00%
18	77	143	11	66.64	69.30	96.16%
19	82.5	143	11	66.64	69.80	95.48%

Table 8
Results of stage 4 analysis.

	a	b	g	$M_{cf,Ed}$	$M_{j,Rd}$	%
Model	mm	mm	mm	kNm	kNm	$M_{pl,RBS}$ / $M_{j,Rd}$
20	64	154	11	66.64	70.80	94.13%
21	66	165	22	54.91	59.70	91.97%

5. Discussions

Based on the results shown on Table 5, it can be inferred that RBS dimensions with the least depth of cut at the center of reduced beam sections provide higher bending moment capacities. This is also true if the center of the RBS section is nearer from the ends of the beam. The initial findings confirmed the previous assumption that minimal depth of cut means minimum reduction in section capacity.

Setting the values of “a” and “g” to the minimum, Table 6 shows that the maximum capacity can be achieved if the length of the RBS is varied from 0.70d to 0.75d. Fig. 6 indicates the variation of RBS moment capacity relative to the RBS length, b. It can be inferred that the maximum moment capacity can be derived when b = 154mm.

Furthermore, assuming the least values of “b” and “g”, Table 7 shows that the maximum capacity can be achieved if the horizontal distance from face of column flange to start of an RBS cut, a, is varied from $0.55b_f$ to $0.60b_f$. Fig. 7, on the other hand, indicates the variation of RBS moment capacity relative to the RBS distance from the face of the supporting column flange, a. From the figure, it can be inferred that the maximum moment capacity can be derived when a = 64mm.

Combining the results of the series of analysis, the following dimensions have been adopted: a = 64mm (0.582b_f), b = 154mm (0.70d), g = 11mm (0.10b_f), s = 141mm, r = 275mm.

Using the aforementioned values and calculating the capacity of the connection, Fig. 8 shows the stiffness diagram of the connection where the calculated value, $M_{j,Rd}$, has been determined equal to 70.80kNm.

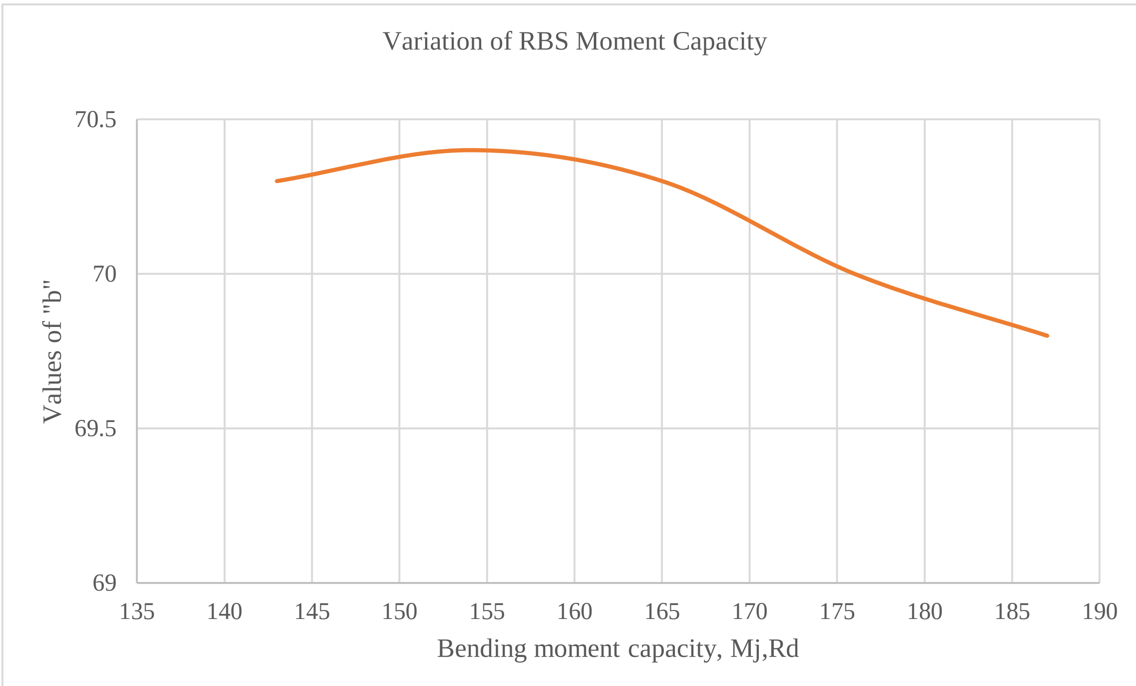

Fig. 6. Variation of RBS moment capacity vs RBS length, b.

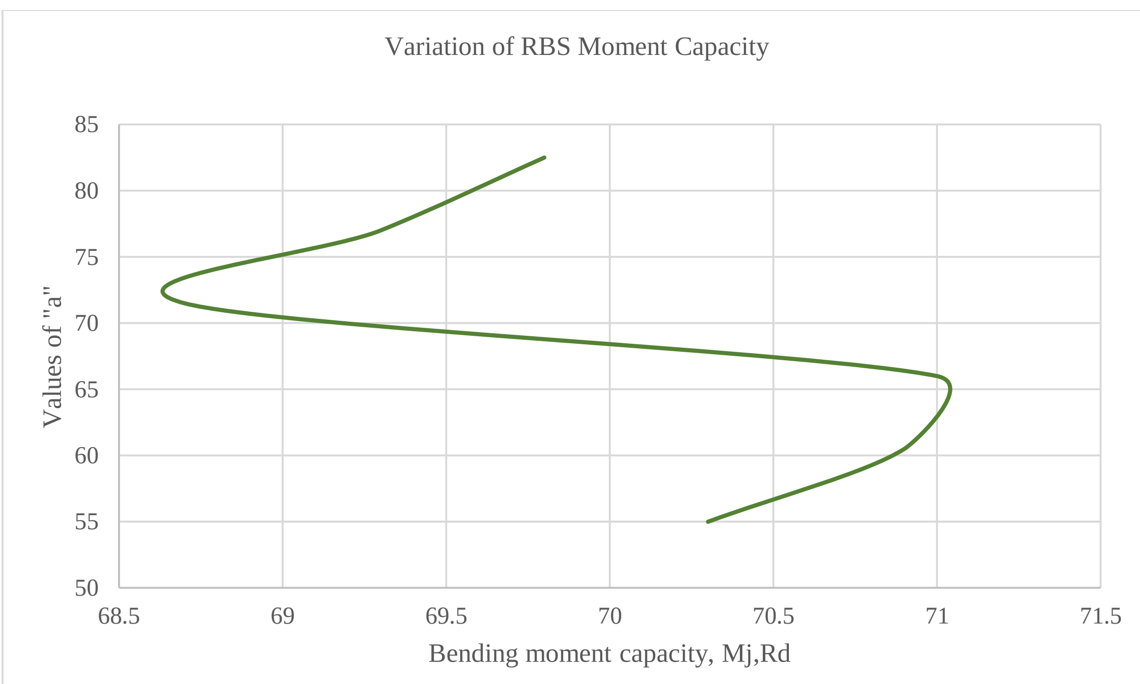

Fig. 7. Variation of RBS moment capacity vs RBS distance from face of column flange, a.

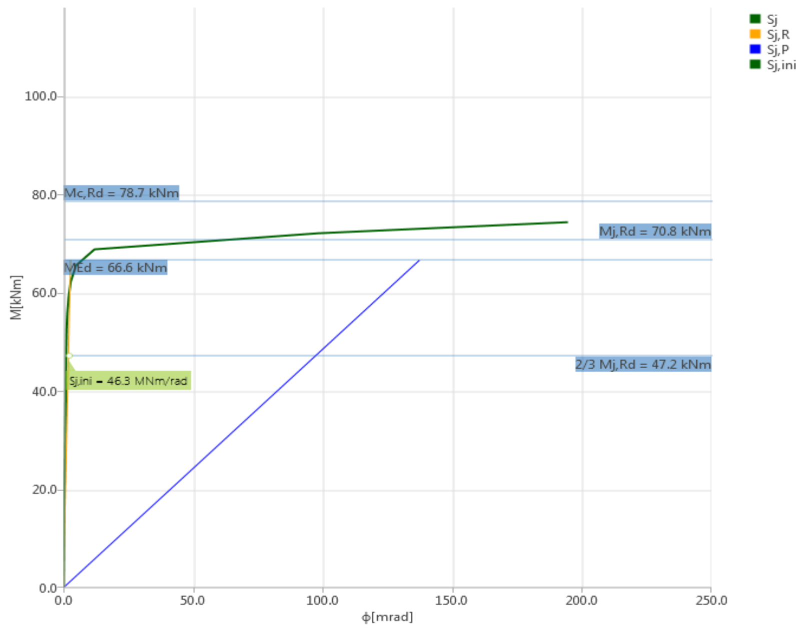

Fig. 8. Stiffness diagram of the adopted RBA flange cut dimensions.

The results of the fourth stage analysis are shown on Table 8. A model has been run based on the recommended flange cut dimensions of Eurocode [11]. The capacity of the connection utilizing the proposed flange cut dimensions yielded approximately 18% higher than the code recommended geometry.

6. Conclusion

Steel structures used as Special Moment Resisting Frames (SMRF) designed to resist lateral loads incorporate Reduced Beam Sections (RBS) in joints due to its efficiency and ease of use. While internationally accepted codes of standards in design of structures for earthquake resistance provide guidance on the process of designing such elements, most of them leave a room for the engineer to interpolate on the most efficient combinations of flange cut dimensions. Hence, codes like the BS EN 1998 and ANSI/AISC 358 give a range of values that are compatible with the ductility requirements. However, rules of thumb that can be used in the design are scarcely available.

Based on the study conducted, it is therefore recommended to use the following flange cut dimensions: $a = 0.582b_f$, $b = 0.70d$, $g = 0.10b_f$. Adopting the recommended geometric configuration of the RBS cut may eliminate the trial and error procedure (Step 1) and ensure a cost-effective design that is within the allowable limits for ductile moment-resisting frame. Lastly, it should be highlighted that use of these values are still subject to the compliance of other requirements not included in this study.

Funding

This research received no external funding.

Conflicts of interest

The author declares no conflict of interest.

References

[1] (FEMA) FEMA. State of the art report on connection performance 2000.

[2] Youssef NFG, Bonowitz D, Gross JL. A survey of steel moment-resisting frame buildings affected by the 1994 Northridge earthquake. US National Institute of Standards and Technology; 1995.

[3] Miller DK. Lessons learned from the Northridge earthquake. Eng Struct 1998;20:249–60. doi:10.1016/S0141-0296(97)00031-X.

[4] Plumier A. Behaviour of connections. J Constr Steel Res 1994;29:95–119. doi:10.1016/0143-974X(94)90058-2.

[5] Plumier A. New idea for safe structures in seismic zones. IABSE Symp. Struct. Incl. New Mater., 1990.

[6] Chambers JJ, Almudhafar S, Stenger F. Effect of Reduced Beam Section Frame Elements on Stiffness of Moment Frames. J Struct Eng 2003;129:383–93. doi:10.1061/(ASCE)0733-9445(2003)129:3(383).

[7] Zhang X, Ricles JM. Experimental Evaluation of Reduced Beam Section Connections to Deep Columns. J Struct Eng 2006;132:346–57. doi:10.1061/(ASCE)0733-9445(2006)132:3(346).

[8] Montuori R. The Influence of Gravity Loads on the Seismic Design of RBS Connections. Open Constr Build Technol J 2014;8:248–61. doi:10.2174/1874836801408010248.

[9] Jones SL, Fry GT, Engelhardt MD. Reduced beam section welded steel moment frames. 12th world Conf. Earthq. Eng. Auckland, New Zealand, Pap., vol. 1671, 2000.

[10] Sofias C, Tzourmakliotou D. Reduced Beam Section (RBS) Moment Connections-Analytical Investigation Using Finite Element Method. Civ Eng J 2018;4:1240. doi:10.28991/cej-0309170.

[11] British Standards Institution. Eurocode 8: Design of structures for earthquake resistance Part 3: Assessment and retrofitting of buildings. London: British Standards Institution. 11 January 2006 n.d.

[12] Lee C-H, Jeon S-W, Kim J-H, Uang C-M. Effects of Panel Zone Strength and Beam Web Connection Method on Seismic Performance of Reduced Beam Section Steel Moment Connections. J Struct Eng 2005;131:1854–65. doi:10.1061/(ASCE)0733-9445(2005)131:12(1854).

[13] American Institute of Steel Construction. ANSI/AISC 358-16. Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, including Supplement No. 1. Chicago, USA: American Institute of Steel Construction n.d.

[14] Lee CH, Jeon SW, Kim JH, Kim JH, Uang CM. Seismic performance of reduced beam section steel moment connections: effects of panel zone strength and beam web connection method. 13th World Conf. Earthq. Eng., 2004, p. 1–6.

[15] American Institute of Steel Construction. ANSI/AISC 341-16. Seismic Provisions for Structural Steel Buildings. Chicago, USA: American Institute of Steel Construction n.d.

[16] IDEA Statica. General theoretical background. Retrieved from https://www.ideastatica.com

[17] Landolfo R, Mazzolani FM, Dubina D, da Silva LS, d'Aniello M. Design of Steel Structures for Buildings in Seismic Areas: Eurocode 8: Design of Steel Structures in Seismic Areas. General Rules and Rules for Buildings, ECCS; 2017.