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Numerical and parametric study of welded I-beam to RHS column connections

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Abstract. This paper presents a numerical investigation of the moment resistance and initial stiffness of welded connections between I-beams and rectangular hollow section (RHS) columns with unequal widths under in-plane bending. A finite element (FE) model was developed and validated to examine the influence of key geometric parameters on the structural behavior. The parametric study analyzed variations in: RHS column thickness (t_0), I-beam width (b_1), and I-beam height (h_1), while evaluating three non-dimensional ratios: column slenderness ($2\gamma = b_0/t_0$), width ratio ($\beta = b_1/b_0$), and height-to-width ratio ($\eta = h_1/b_0$). It was observed that, for higher values of the widths of the I-beam and column thicknesses, higher joint resistance and stiffness results were found. The results indicated that the failure mode was column face plastification. In the connections analyzed, a comparison with current international design standards revealed that existing provisions conservatively predict moment resistance.

Keywords: Steel structures; steel hollow sections; numerical analysis; welded connections.

1 Introduction

The use of rectangular hollow section (RHS) columns has become increasingly popular in steel construction due to their high compressive strength, lightweight potential, and compatibility with composite concrete-filled systems. When combined with I-beams, these columns offer excellent performance in flexural applications, making them a common solution in building structures [1].

Welding I-beams directly to the flat face of RHS columns is a practical and efficient technique. It simplifies fabrication, eases on-site alignment, reduces maintenance demands, and ensures effective bending moment transfer. However, the structural performance of these joints, particularly their stiffness and failure mode, is strongly influenced by geometric and detailing parameters. Among the eight potential failure modes identified by Wardenier *et al*. [2], column face plastification is the most common, especially when the width ratio between the beam flange and the column face (β) is below 0.85, which is a frequent condition given the wide range of available cross-sections.

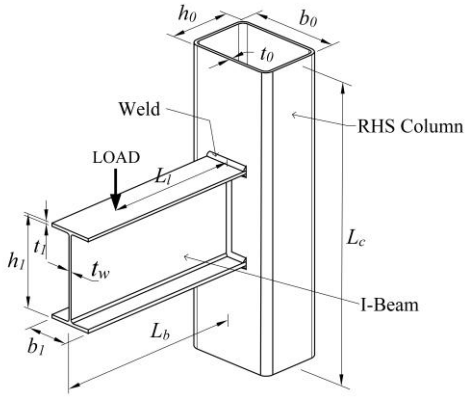
In recent years, extensive research, both experimental and numerical, has focused on RHS-to-I-beam welded connections [3–9]. Lu *et al.* [10] demonstrated that column face plastification dominates the failure mechanism when β is small, typically resulting in moment-rotation curves with no peak load. These findings supported the development of the deformation limit criteria, later adopted by the International Institute of Welding (IIW) [11]. Other studies, such as those by Serrano-López *et al.* [12], further emphasized the role of parameters like column thickness and weld positioning in the joint's behavior.

Despite these advancements, design standards such as NBR 16239 [13], ISO 14346 [14], and prEN 1993-1-8:2021[15] still provide limited guidance for these connections. Their validity ranges are often restricted to specific geometric and material conditions, which can limit the broader adoption of welded joints, particularly in regions like Brazil, where a wide variety of RHS and I-section profiles are used.

In this context, the present study develops a validated finite element (FE) model to evaluate in-plane bending in T-joint configurations, focusing on joints governed by column face plastification. A parametric study is then conducted by varying the column thickness (t_0), beam flange width (b_1), and beam height (h_1), and assessing the influence of the non-dimensional ratios: column slenderness ($2\gamma = b_0/t_0$), width ratio ($\beta = b_1/b_0$), and height-to-width ratio ($\eta = h_1/b_0$). Finally, the results are compared with predictions from international design standards to evaluate their accuracy and applicability.

2 Geometric properties, validity ranges and design provisions

Figure 1 shows the geometric parameters used to evaluate the connection. The width ratio (β) is the ratio between the beam flange and the column width, the slenderness (2γ) is the ratio between the column width and thickness, and η is the ratio between the beam height and the column width. These parameters follow current design standards and are used to identify the failure mode and verify the applicability of existing formulations.



$$\beta = \frac{b_1}{b_0} \qquad 2\gamma = \frac{b_0}{t_0} \qquad \eta = \frac{h_1}{b_0}$$

b_0 – column width;
h_0 – column height;
t_0 – column wall thickness;
b_1– flange width;
h_1 – I-beam height;
t_1 – flange thickness;
t_w – web thickness of the I-beam;
L_c – column length;
L_b – I-beam length;

Figure 1. Geometric parameters

In this study, failure was primarily governed by column face plastification, typically observed when the width ratio (β) is less than 0.85. The design for column face plastification in prEN 1993-1-8:2021 [15] uses a yield line model treating I-beam flanges as plates under axial force. It provides equations for flange resistance (Eq. 1) and in-plane moment resistance (Eq. 2), also adopted by ISO 14346 [14] for welded plate-to-RHS joints. Although ISO 14346 [14] does not explicitly cover I-beam to RHS connections, its formulation applies. These equations are valid for cross-sectional classes 1 and 2 per EN 1993-1-1 [16]. NBR 16239 [13] addresses similar connections but lacks specific equations for column face plastification. The ponderation factor (γ_{a1}) recommended is 1.10, but was not considered in this work. Each standard sets different geometric applicability limits, summarized in Table 1.

$$N_{1,Rd} = Q_f f_{y0} t_0^{\,2} \frac{2 + 2.8\beta}{\sqrt{1 - 0.9\beta}} \tag{1}$$

$$M_{1,Rd} = N_{1,Rd}(h_1 - t_1) \tag{2}$$

For this study, the chord stress factor Q_f is 1.0, and f_{y0} is the column yield strength.

Table 1: Validity ranges of the design equation standards for column face plastification failure

Criterion	NBR 16239 [16]	prEN 1993-1-8 [18]	ISO 14346 [17]
$\beta(b_1/b_0)$	$0.50 \le \beta \le 0.85$	$0.50 \le \beta \le 0.85$	$0.4 \le \beta \le 0.85$
$2\gamma(b_0/t_0)$	$2\gamma \le 30$	$2\gamma \le 30$	$2\gamma \le 40$
$\eta(h_1/b_0)$	$\eta \ge 2\sqrt{1-\beta}$	$\eta \ge 2\sqrt{1-\beta}$	-

3 Numerical analysis

3.1 Numerical model

The numerical analysis was conducted using the finite element method through the commercial software ANSYS 18.2 [17], widely employed in structural studies and analyses. The FE model was developed to accurately represent the geometric and material characteristics and test configuration, with validation results described in Oliveira *et al.* [18].

Two different element types were used in the numerical model. The SHELL 281 with an 8-noded shell element was chosen to represent the beams and columns, while the SOLID 185 with an 8-noded solid element was used for the welds; the FE model is illustrated in Figure 2. The radius of curvature of the rectangular tubular profile was set as 2 times the RHS thickness (t_0). The mesh was defined as a size of 10 mm to distribute elements evenly and reduce stress concentrations in the connection region.

The boundary conditions were established to simulate the experimental T-joint model, with column ends fixed (all displacements and rotations restricted). The load was applied by vertical displacement increments at 800 mm from the face of the beam, to induce a bending moment. The adopted solution method was the iterative Newton-Raphson method.

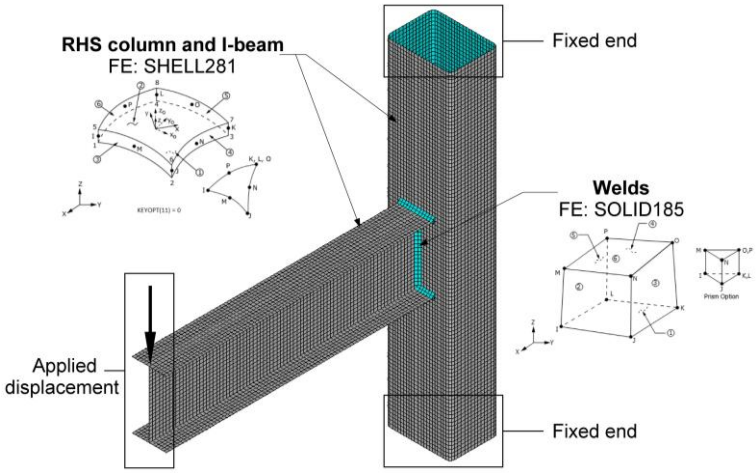


Figure 2. FE mesh and boundary conditions

3.2 Material Properties

Among the various options for representing the steel behavior in the connection components in EN 1993-1-5 [19], the bilinear diagram was chosen, with elasto-plastic behavior to represent the material non-linearity, as shown in Figure 3. Therefore, the values of yield stress (f_{y0}), Young's modulus (E), and strain-hardening slope (E_t), considered equal to E/100, are required. For the weld strength, a value of 500 MPa was used. The choices for these diagrams were based on studies by Oliveira *et al.* [18] and Barros *et al.* [4]. The nominal properties of the materials used in the parametric study are presented in Table 2.

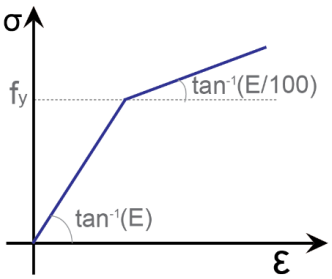


Figure 3. Stress *versus* strain steel diagram: Elastoplastic with hardening [19]

Table 2. Material properties.

Elasticity modulus (GPa)		Yield tension (MPa)		
E	Tangent E_t	Column f_{y0}	I-beam f_{y1}	Weld f_w
200	E/100	450	345	500

3.3 Parametric study

In the parametric study, 45 models were developed using an RHS column with a length of 1000 mm and chord cross sections with 250mm x 150mm. In all models, the analyses were performed by bending around the weak axis of the RHS column. Three different values of 2γ were analyzed: 25, 30, and 40, in which the column thicknesses were adjusted accordingly. Five different values of height (h_1) varying from 200.0 to 500. 0 mm with 75 mm increments were used to assess the effect of the ratio between I-beam height and RHS column width ($\eta=h_1/b_0$), with values varying from 0.8 to 2.0, with 0.3 increments.

The I-beam thickness adopted was equal to 8mm to avoid failure in the beam. The β values varied from 0.4 to 0.6 with 0.10 intervals. The flange width was defined according to the adopted β value. These dimensions were chosen to allow the predominance of the column face plastification, with the specific geometric properties presented in Figure 4.

The weld leg size was set as 1.5 times the lower thickness value between the connected members, as recommended by NBR 16239 [13].

RHS Column	I-Beam	
t_0 and 2γ	b_1 and β	h_1 and η
$\begin{Bmatrix}10.0\\8.33\\6.25\end{Bmatrix}=\begin{Bmatrix}25\\30\\40\end{Bmatrix}$	$\begin{Bmatrix}100\\125\\150\end{Bmatrix}=\begin{Bmatrix}0.4\\0.5\\0.6\end{Bmatrix}$	$\begin{Bmatrix}200 & 275 & 350\\ 425 & 500 & \end{Bmatrix}=\begin{Bmatrix}0.8 & 1.1 & 1.4\\ 1.7 & 2.0 & \end{Bmatrix}$

Figure 4. Variables of the parametric study.

3.4 Deformation limit criterion and initial stiffness

A deformation limit criterion was adopted to define the joint strength in the numerical analysis, considering both serviceability and ultimate limit states. This approach, proposed by Lu *et al.* [10], is appropriate for cases where no peak load is observed in the load–displacement or moment-rotation curve, as typically occurs when column face plastification governs the failure.

According to this criterion, the joint displacement is expressed as a percentage of the column width (b_0). A displacement of 1% of b_0 defines the serviceability limit, while 3% of b_0 defines the ultimate limit. For connections under bending moment, the rotation equivalent to this percentage of displacement of the column face.

The moment is calculated at both displacement levels. If the ratio between the moments at rotation referring to 3%b_0 and 1%b_0 is less than 1.5, the ultimate limit state (3%b_0) governs. Otherwise, the serviceability limit state controls, and the moment resistance is taken as the value at 1%b_0 multiplied by 1.5.

The joint rotation was determined by the horizontal displacements in the column towards the center lines of the upper and lower flanges of the beam. The initial rotational stiffness was determined using the secant method applied to the linear portion of the moment–rotation curve, as also adopted by Barros *et al.* [4]. These authors recommend using a reference rotation corresponding to 1/5 of 1% of the column width (b_0). In this study, the same reference value was adopted.

4 Results and discussion

4.1 Failure mode and von Mises stress distribution

In Figure 5, the von Mises stress distributions were shown for three different models, with width ratios (β) of 0.4, 0.5, and 0.6. High stress values was observed in the joint region of the RHS, particularly at the interface between the column face and the beam flanges. The column face plastification was identified in all numerical models. Moreover, as β increases, the stresses tend to spread more widely, with higher stress levels observed along the lateral walls, indicating a participation of the lateral wall in the connection behavior.

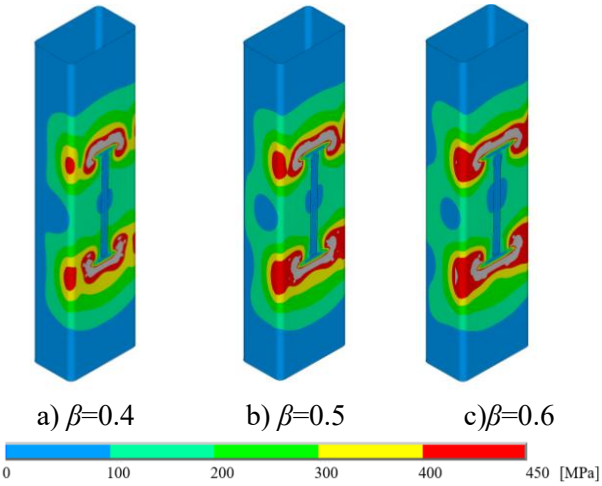


Figure 5. Stress distribution at serviceability moment of the model of 2γ=30,η=1.4:a) β=0.4,b)β=0.5 and c)β=0.6.

4.2 Influence of geometric parameters on connection resistance and stiffness

Figure 6 shows the moment-rotation curves obtained from FE models for different non-dimensional parameter values. Figure 6a) shows the effect of varying parameter β in the joint resistance, enabling the assessment of its influence on joint behavior. To evaluate the effect of the column slenderness on the joint resistance, the column thickness was varied, resulting in changes in the 2γ factor (Figure 6b).

The peak load is not observed. In the initial phase, the response is elastic, followed by a reduction in stiffness as yielding begins, with deformations progressively increasing due to membrane action effects.

Furthermore, it is observed that as the values of β and η increase and the 2γ decreases, the initial stiffness and moment resistance are high. The influence of β can be explained by the increase in width values (b_1), causing a decrease in the punching effect on the frontal face. Larger I-beam heights provide greater flexural stiffness, thereby enhancing the stiffness of the connection.

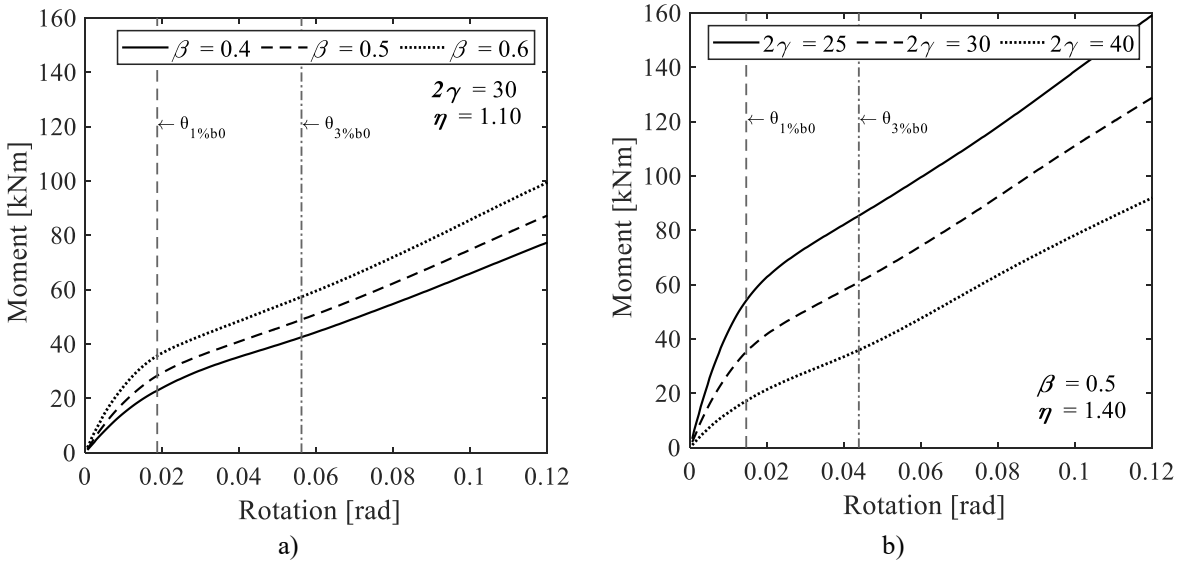
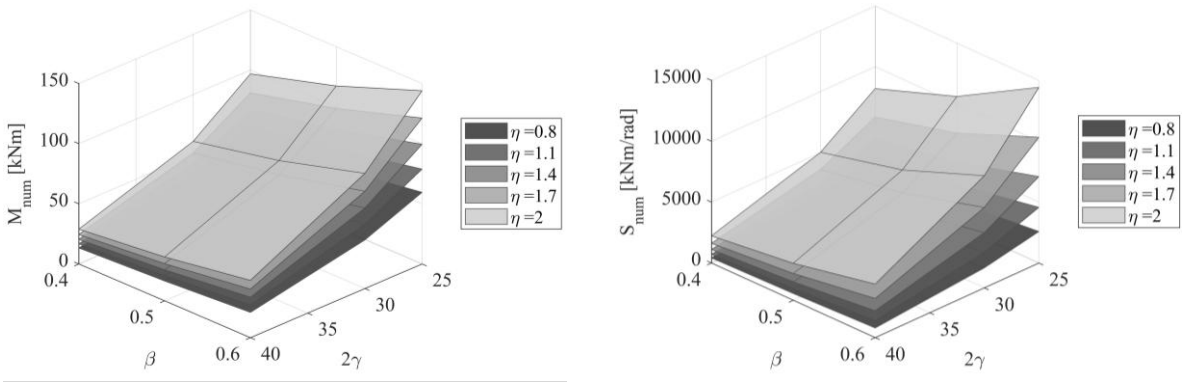


Figure 6. Moment-rotation connection behavior: a) effect of parameter β; b) effect of column slenderness (2γ).

Figure 7 shows the variation of numerical moment resistance (M_{num}) and initial stiffness as a function of parameters β and 2γ, for η values of 0.8, 1.1, 1.4, 1.7, and 2.0. Analysis of the surfaces indicates that β, 2γ, and η influence the joint moment resistance. Moment resistance and stiffness increased with higher β and η values, due to the reduction of punching effects on the column's frontal face from the larger beam flange width and the greater I-beam height. In all graphs, increasing 2γ and consequently the RHS column wall thickness (t_0) increased moment resistance and stiffness, as thicker column walls allowed more effective force transfer.



a) Moment resistance b) Initial stiffness

Figure 7. Influence of non-dimensional parameters: a) moment resistance; b) initial stiffness.

4.3 Results comparison

In Figure 8, the numerical joint strength values (M_{num}) are compared with the analytical predictions ($M_{1,Rd}$) for all models analyzed. The results indicate that the numerical strengths exceeded the analytical estimations, demonstrating a good correlation trend. On average, the ratio between numerical and analytical strengths was 1.09, with a coefficient of variation (CoV) of 0.100, indicating limited dispersion among the results. This outcome confirms that the analytical formulation provides a conservative estimation of the joint resistance, while still maintaining reasonable accuracy across the evaluated parameter range. For the studied welded T-joints, the current code equation provides reasonable predictions, yet deviations from numerical results occur within and beyond the validity limits. Nevertheless, further investigations are recommended to assess the influence of the non-dimensional parameters on the analytical predictions and to refine their applicability to a broader range of joint configurations.

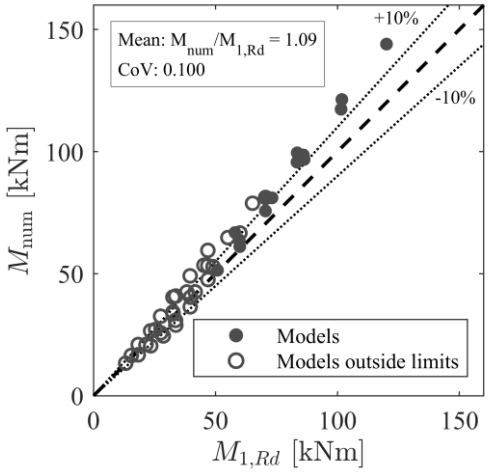


Figure 8. Comparison between numerical and design moment resistance for all configurations.

5 Conclusions

A study was conducted on welded T-joints between I-beams and RHS columns, considering variations in column wall thickness, and I-beam width and height. A numerical model was developed using the finite element method to analyze joint behavior, moment resistance, and initial stiffness.

Column face plastification was the dominant failure mode in all cases, as expected for joints with parameter $\beta \leq 0.85$. For lower 2γ values (greater wall thickness), and higher β and η values, the joints presented higher moment resistance and initial stiffness.

The numerical results were compared with an analytical equation. The mean numerical-to-analytical strength ratio was 1.09, with a coefficient of variation of 0.10, considering FE models both within and outside the current validity limits of the code provisions. The design of welded T-joints with I-beams and RHS columns, within the geometric parameters studied, can be performed using the current code equation; however, noticeable deviations from the numerical results were observed both within and outside the specified validity limits.

These findings confirm the general reliability of current design provisions but indicate the need for improved

analytical equations, especially for joints with higher slenderness or variations in the parameter η. Further studies should incorporate the influence of the non-dimensional parameters β, 2γ, and η to enhance model accuracy and support future updates of design codes and structural applications.

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