Evaluation of the Mechanical Properties of bolted connections with extended endplate by EN 1993-1-8:2005 and ABNT NBR 8800:2008

Avaliação das propriedades mecânicas de ligações parafusadas com chapa de extremidade estendida segundo EN 1993-1-8:2005 e ABNT NBR 8800:2008

Evaluación de las propiedades mecánicas de las uniones atornilladas con chapa de testa extendida según EN 1993-1-8:2005 y ABNT NBR 8800:2008

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Abstract
This paper concerns the study of the semi-rigid behavior of bolted beam-column connections with extended endplate and transverse stiffeners in the column web based on the Component Method presented in EN 1993-1-8:2005. The moment x rotation relationship of such connections with different endplate thicknesses are obtained according to the Brazilian and European standards, from where the related resistance values are determined. It is observed that all connection designs with the Brazilian standard have a lower resistance to bending moment if compared to the European standard designs, getting worse as the thickness of the endplate decreases. Finally, it is concluded that it may be incorrect not to associate the concept of semi-rigid behavior to connections with transversely stiffened endplates, since the behavior of such connections can be quite different from an ideally rigid connection assumption.
Keywords: Steel structure; Semi-rigid connection; Component method.

Resumo
Este artigo tem como objetivo o estudo do comportamento semirrígido de ligações viga-pilar com chapa de extremidade estendida e enrijecedores transversais considerando o Método dos Componentes, proposto no EN 1993-1-8:2005. Neste trabalho, a relação momento x rotação dessas ligações com diferentes espessuras de chapa são obtidas de acordo com as normas brasileira e europeia, a partir das quais os parâmetros de resistência são determinados. Observa-se que todos os projetos de ligação utilizando a norma brasileira apresentam menor resistência ao momento fletor se comparada aos projetos com a norma europeia, agravando-se à medida que a espessura da placa de extremidade diminui. Conclui-se que pode ser errôneo associar o conceito de ligações rígidas às ligações com chapas de extremidade enrijecidas transversalmente, uma vez que o comportamento desse tipo de ligação pode ser bem diferente do comportamento de uma ligação idealmente rígida.
Palavras-chave: Estrutura de aço; Ligação semirrígida; Método dos componentes.

Resumen
Este artículo se refiere al estudio del comportamiento semirrígido de uniones atornilladas viga-pilar con chapa de testa extendida y rigidizadores transversales en el alma del pilar según el método de los componentes presentado en EN 1993-1-8:2005. La relación momento x rotación de las uniones con diferentes espesores de chapa se obtiene de acuerdo con las normas brasileña y europea, a partir de las cuales se determinan los valores de resistencia relacionados. Se observa

que todos los proyectos de conexiones con la norma brasileña tienen una menor resistencia al momento flector en comparación con los proyectos de acuerdo con Eurocódigo 3, empeorando a medida que disminuye el espesor de la chapa de testa. Se concluye que puede ser erróneo no asociar el concepto de comportamiento semirrígido a uniones atornilladas viga-pilar con chapa de testa extendida y rigidizadores transversales, ya que el comportamiento de tales componentes estructurales puede ser bastante diferente de una suposición de conexiones perfectamente rígida.
Palabras clave: Estructura de acero; Unión semirrígida; Método de los componentes.

1. Introduction

Although considering the hypotheses of fully rigid or ideally pinned behavior of connections considerably simplifies the analysis and design of a structure, the validity of these hypotheses can be questioned in many cases. The reality is that, due to the impossibility in practice of designing ideal connections, these structural components present an intermediate response, which varies between these extreme cases, indicating a nonlinear behavior (Dave & Savaliya, 2010; Viana et al., 2019). Indeed, even connections designed as fully rigid may admit relative rotation between the connected elements, as well as the connections designed as ideally pinned may allow transmission of bending moments between the attached elements. Due to this, the development of new analysis methods with the aim of obtaining more realistic and less conservative estimates of the semi-rigid connection behavior has been the focus of several authors (Ataei et al., 2015; Kishi & Chen, 1990; Kong & Kim, 2017, 2018; Lee & Moon, 2002; Shi & Chen, 2017; Thai & Uy, 2016; Yee & Melchers, 1986; Zhou et al., 2018, 2019).

The behavior of the connections depends directly on the interaction between the elements and the parts that compose them: angles, plates, welds and bolts, as well as on the geometric characteristics of the connected profiles (Faella et al., 1999; Hortencio & Falcón, 2018; Pfeil & Pfeil, 2000). All these components contribute for the overall connection behavior, which can be represented by a moment-relative rotation curve M-θ_r. From the M-θ_r curve, three fundamental properties of a connection are defined: the initial rotational stiffness ($S_{j,ini}$), the design moment resistance ($M_{j,Rd}$) and the rotational capacity (θ_{cd}). An interesting feature of a steel connection is that it can have several rotational behaviors by simply changing the parameters of bending moment resistance and stiffness, such as: thickness of the endplate and the arrangement and diameter of the bolts. The incorporation of the connection behavior in the structural analysis requires a mathematical representation of the moment x relative rotation curve, which can be performed by means of analytical (Zhou et al., 2018, 2019), experimental (Tahir et al., 2018), mechanical (Yu et al., 2009) and numerical (Kong and Kim, 2018) models.

In Brazil, the standard for steel construction, ABNT NBR 8800:2008 (ABNT, 2008), does not have a specific regulation for the design of semi-rigid connections and recommends the application of Eurocode 3 (CEN, 2005), when there is no applicable Brazilian standard. The calculation methodology presented in ABNT NBR 8800:2008 suggests the verification of each of the main elements of a connection and does not present a systematazation of which checks must be carried out and in which way the efforts can be obtained for the connections. The scarcity of technical references when it comes to semi-rigid connections according to the Brazilian standard is also notorious. Most studies are restricted to determining the resistance of the connection components. On the other hand, the European standard presents a consistent methodology for obtaining the moment x relative rotation curve of the connection, known as the Component Method. Such a method consists of decomposing the connection into several components that make it up and then analyzing the strength, stiffness and deformation of each one separately. Finally, through a reduction procedure, the beam-column behavior can be obtained.

In this work, a study of the behavior of beam-column bolted connections with extended endplate and transverse stiffeners in the column web is presented. The Component Method, recommended in EN 1993-1-8:2005 (CEN, 2005), and the criteria of ABNT NBR 8800:2008 (ABNT, 2008), complemented, when necessary, by other standards and recommendations, are used in order to determine the moment x relative rotation curve, which is performed based on the parameters of stiffness, resistance and rotational capacity of such connections. Subsequently, the behavior of these connections is evaluated varying the thickness of the endplate.

2. Methodology

Regarding its nature, this research is classified as applied since it is motivated by the need to produce knowledge that enables a solution to a problem encountered in reality. In addition, this research has a predominantly quantitative approach, which works with quantifiable, measurable variables in order to obtain results related to the research objective (Gil, 2002). As this work aims to analyze a specific type of connection in order to obtain an extensive and detailed knowledge about the design process of semi-rigid beam-to-column joints, it can be concluded that it is a case study. As stated by Gil (2002), the case study aims to obtain an overview of the problem and identify possible factors that influence it or are influenced by it. Therefore, it is worth emphasizing here that it is not the objective of this work to generalize the results found.

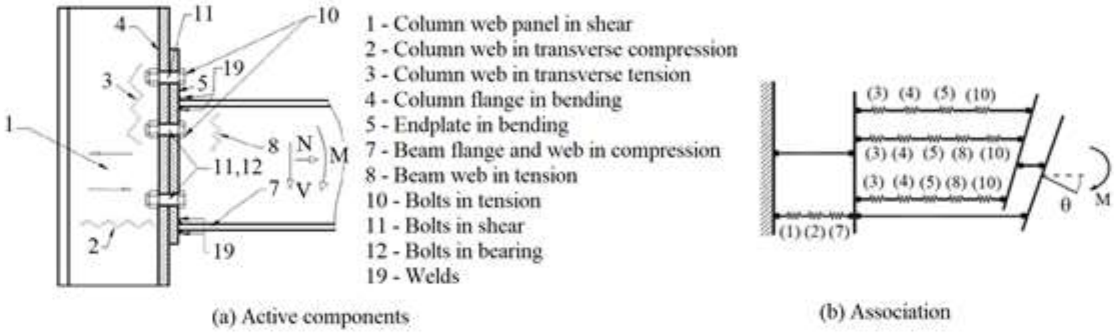
The first step in conducting this research was a bibliographic survey, which aimed to verify the material already published related to the topic. From this, the procedures for design and analysis of connections according to EN 1993-1-8:2005 and ABNT NBR 8800:2008 were detailed and summarized. The objective was to present methodologies that can be applied in the analysis and design of beam-to-column connections based on the component method.

From the presented procedures, the parameters of the moment x relative rotation curve of a bolted connection with extended endplate and transverse stiffeners in the column web were obtained. In this case, the thickness of the plate was 19 mm. Finally, a comparative analysis was carried out by varying the thickness of the connection's endplate. Therefore, the moment x relative rotation curves were built for the connection with endplate thicknesses of 9.53 mm, 12.70 mm and 15.88 mm in order to observe the influence of this variable on the connection's resistance parameters.

3. Procedures for Design and Analysis of Connections According to EN 1993-1-8:2005

The component method, Eurocode 3 (CEN, 2005), is a mechanical model that consists of identifying the active components of the connection (see Figure 1a), establishing force versus displacement relationships for each one of these components and, finally, performing the association of the components to obtain the bending moment x relative rotation curve of the connection, as shown in Figure 1b. In such a method the components are represented by translational springs, with linear or nonlinear behavior. Then, these components compose systems that are treated as structures to simulate the moment x relative rotation behavior of the connections. Figure 1a shows the most common components applicable to beam-column connections with an extended endplate. All equations for determining the resistance and stiffness of the basic components of an extended endplate connection with transverse stiffeners can be obtained in Eurocode 3 (CEN, 2005).

Figure 1 – Component method procedures.

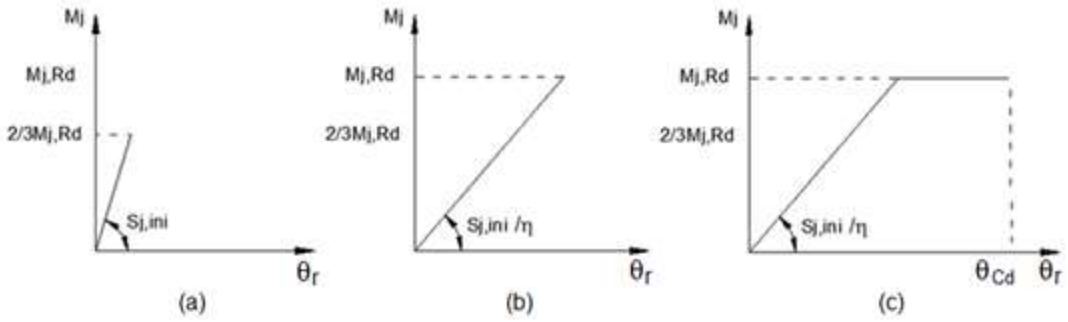


Source: Adapted from Oliveira (2011).

3.1 Representation of the moment-rotation curve

In general, the moment-rotation curve is not linear, however, a linear curve, or a succession of linear curves, can represent in a simplified way the M-θ_r relation. This representation can be obtained from the parameters $M_{j,Rd}$ and $S_{j,ini}$ of a connection, depending on the type of connection behavior that each structural analysis procedure requires. Eurocode 3 (CEN, 2005) allows, for an elastic global analysis, the modeling of the connections based on the adoption of a linear curve with the initial rotational stiffness ($S_{j,ini}$); nonetheless, the actual bending moment in connection ($M_{j,Ed}$) must not exceed two thirds of the design moment resistance ($M_{j,Rd}$) (See Figure 2a). For cases where $M_{j,Ed}$ is higher than this limit, Eurocode 3 (CEN, 2005) recommends that the linear curve be defined by the secant rotational stiffness of the joint, equal to $S_{j,ini} / \eta$, in which η is the stiffness modification coefficient, equal to 2 for beam-column connections with bolted extended endplate. In the case of an elastic-plastic global analysis, the connection can be modeled, in a simplified way, by a bilinear curve (See Figure 2c).

Figure 2 – Simplification of the M-θ_r response: (a) Elastic global analysis $M_{j,Ed} \leq 2/3\ M_{j,Rd}$; (b) Elastic global analysis $M_{j,Ed} > 2/3\ M_{j,Rd}$; (c) Elastic-plastic global analysis

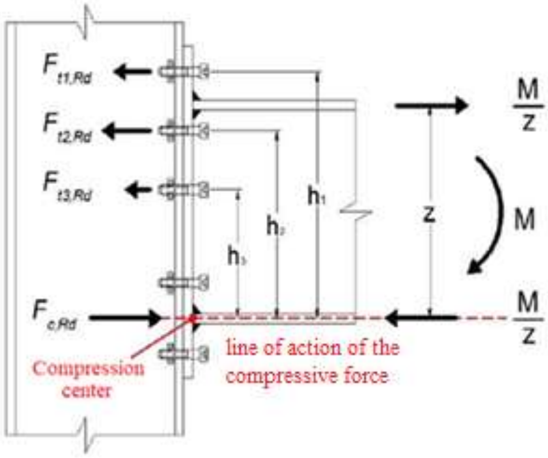


Source: Authors.

3.2 Design moment resistance

The design moment resistance ($M_{j,Rd}$) of a beam-to-column connection is evaluated by balancing the efforts of the three regions of the connection: tension, compression and shear regions. To obtain the potential resistance of the tensioned region of the joint with an extended endplate, the design resistance of each bolt-row must be evaluated. Figure 3 depicts the efforts acting on a connection with extended endplate, where h_r is the distance from the bolt-row r to the compression center, which is located at the centroid of the lower beam flange for the considered example. $F_{tr,Rd}$ is the effective design tension resistance for each bolt-row and $F_{c,Rd}$ is the design compressive resistance.

Figure 3 – Connection's efforts.



Source: Adapted from Oliveira (2015)

3.3 Initial Stiffness

The bending moment resistance can be obtained by the following expression:

$$M_{j,Rd} = \sum_r h_r F_{tr,Rd} \tag{1}$$

The rotational stiffness ($S_{j,ini}$) of a joint is constituted by the combination of stiffness parameters k_i, associated with the translational stiffness of the basic components involved in the connection. To determine the rotational stiffness ($S_{j,ini}$) of the beam-to-column joint with several rows of tensioned bolts, the subsequent steps must be followed:

1. Associate in series all the stiffnesses of components relative to each bolt-row r to obtain the effective stiffness coefficient (k_{eff}), according to:

$$k_{eff,r} = 1/\sum \frac{1}{k_{i,r}} \tag{2}$$

2. Associate in parallel the parameters of effective stiffness of each bolt-row r, considering that the rotation center of the connection is located at the centroid of the lower beam flange. Thus, the equivalent stiffness of the associated row in series is given by

$$k_{eq} = (\sum_r k_{eff,r}\ h_r)/Z_{eq} \tag{3}$$

where Z_{eq} is the equivalent lever arm obtained by

$$Z_{eq} = (\sum_r k_{eff,r}\ h_r^2)/(\sum_r k_{eff,r}\ h_r) \tag{4}$$

3. Determine the rotational stiffness by combining the stiffness parameters of the tension, compression and shear region, as follows:

$$S_j = E\, Z_{eq}^2 / \left[\mu \left(\frac{1}{k_1} + \frac{1}{k_2} + \frac{1}{k_{eq}}\right)\right] \tag{5}$$

in which k_1 and k_2 are the stiffness coefficients related to the column web panel in shear and the column web in compression, respectively; μ is the stiffness ratio, which depends on the applied moment ($M_{j,Ed}$) and the design moment resistance ($M_{j,Rd}$).

The stiffness ratio μ can be determined through the following equation:

$$\mu = \begin{cases} 1 \quad , & M_{j,Ed} \leq \frac{2}{3} M_{j,Rd} \\ \left(\frac{1{,}5\, M_{j,Ed}}{M_{j,Rd}} \right)^{\Psi} , & \frac{2}{3} M_{j,Rd} < M_{j,Ed} \leq M_{j,Rd} \end{cases} \tag{6}$$

where Ψ is a coefficient that depends on the connection type. For connections with bolted extended endplate Ψ is equal to 2.7.

Table 1 presents the expressions for calculating the resistance and stiffness of the basic components of a bolted connection with an extended endplate and transverse stiffeners located at the column web. The specific coefficients of each expression are defined in Eurocode 3:1-8 2005 (CEN, 2005).

The evaluation of the connection rotation capacity is fundamental for the structural design, since all connections require a minimum rotation capacity. According to ANSI / AISC 360-10 (Committee, 2010), in the absence of a more precise analysis, a rotation of $\theta = 0.03$ rad is adequate.

Table 1 – Resistance and stiffness of the basic components of an extended endplate connection with transverse stiffeners attached to the column web.

Case	Resistance	Stiffness
Column web panel in shear	$V_{wp,Rd} = A_{vc} \cdot \frac{0.9 f_{ywc}}{\gamma_{M0}\sqrt{3}}$	$k_1 = \infty$
Column web in compression	$F_{c,wc,Rd} = \frac{\omega k_{wc} b_{eff,c,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$	$k_2 = \infty$
Column web in tension	$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}}$	$k_3 = \frac{0.7 b_{eff,t,wc} t_{wc}}{d_c}$
Column flange in bending (Σ)	$\begin{cases} F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} \\ F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{T,Rd}}{m+n} \\ F_{T,3,Rd} = \sum F_{T,Rd} \end{cases}$	$k_4 = \frac{0.9 l_{eff} t_{fc}^3}{m^3}$
Endplate in bending (<lower)	$\begin{cases} F_{T,1,Rd} = \frac{4M_{pl,1,Rd}}{m} \\ F_{T,2,Rd} = \frac{2M_{pl,2,Rd} + n\sum F_{T,Rd}}{m+n} \\ F_{T,3,Rd} = \sum F_{T,Rd} \end{cases}$	$k_5 = \frac{0.9 l_{eff} t_p^3}{m^3}$
Beam flange and web in compression	$F_{c,fb,Rd} = \frac{M_{c,Rd}}{(h - t_{fb})}$	$k_7 = \infty$
Beam web in tension	$F_{t,wb,Rd} = \frac{b_{eff,t,wb} t_{wb} f_{y,wb}}{\gamma_{M0}}$	$k_8 = \infty$
Bolts in tension	$F_{t,Rd} \leq \frac{0.6 \pi d_w t f_u}{\gamma_{M2}}$	$k_{10} = \frac{1.6 A_s}{L_b}$

Source: Authors.

4. Procedures for Design and Analysis of Connections According to NBR 8800:2008

The procedures for determining the design moment resistance of the connection using the criteria of the Brazilian standard NBR 8800:2008 (ABNT, 2008) are described in detail below. When necessary, they are complemented by national and/or international standards and recommendations.

The design moment resistance ($M_{j,Rd}$) of the connection is evaluated by balancing the efforts of the regions of the beam-to-column joint subjected to tension, compression and shear. The initial stiffness of the connection is determined according to the procedures defined in the European standard (CEN, 2005).

As reported by ABNT NBR 8800:2008 (ABNT, 2008), the column's stiffeners must satisfy the following requirements:
a) The sum of the width of each stiffener with half the thickness of the column web cannot be lower than one third of the width of either the column flange or connection plate that receives the localized force;
b) The stiffener's thickness cannot be less than half the thickness of the column flange or the connecting plate that receives the localized force. Additionally, the stifferner's thickness cannot be less than its width divided by 15.

4.1 Column web in shear

The shear resistance of the column web due to forces transmitted by the beam flange is given by:

$$F_{Sd} \leq 0.4N_{pl} \rightarrow F_{Rd}=V_{Rd} \tag{7}$$

$$F_{Sd} > 0.4N_{pl} \rightarrow F_{Rd}=V_{Rd}(1.4 - F_{Sd} / F_{Sd}) \tag{8}$$

where V_{Rd} is the shear resistance of the panel obtained according to section 5.4.3 of the Brazilian standard and N_{pl} is the compressive yielding load of the column's cross section ($A_g f_y$).

4.2 Local bending of the column flange (web in tension)

The tension resistance of the column web is given by:

$$F_{Rd}=6,25t_f^2 f_y/\gamma_{a1} \tag{9}$$

where t_f is the column flange thickness, f_y is the steel yielding strength and γ_{a1} is the weighting coefficient of the steel yielding strength.

Verification is not necessary if the length of the force acting perpendicular to the column length is less than $0.15b_f$, where b_f is the width of the column flange. When the force acts at a distance from the column end less than 10 times the thickness of the column flange, the resistance force must be halved.

4.3 Local yielding of the column web under compression/tension

When the force is at a distance from the bar's end greater than the height of the cross section, the resistance force is given by:

$$F_{Rd}=1.10(5k+l_n)f_y t_w /\gamma_{a1} \tag{10}$$

When the force is at a distance from the bar's end less than or equal to the height of the cross section, the resistance force is given by:

$$F_{Rd}=1.10(2.5k+l_n)f_y t_w /\gamma_{a1} \tag{11}$$

where l_n is the length of the force acting in the longitudinal direction, t_w is the thickness of the column web, k is the thickness of the loaded beam flange added to: a) the leg length of the weld fillet that is parallel to the web, in the case of welded profiles; b) the root radius of the steel profile, in the case of laminated profiles.

4.4 Column web wrinkling under compression

When the compressive force is at a distance from the column end greater than or equal to half the height of the cross section, the resistance force is given by:

$$F_{Rd}=(0.66\ t_w^2/\gamma_{a1})\left[1+3(l_n/d)\left(t_w/t_f\right)^{1.5}\right]\sqrt{Ef_y t_f/t_w} \tag{12}$$

Otherwise, the resistance force is obtained by

$$l_n/d \leq 0.2 \rightarrow F_{Rd}=(0.33t_w^2/\gamma_{a1})\left[1+3(l_n/d)\left(t_w/t_f\right)^{1.5}\right]\sqrt{Ef_y t_f/t_w} \tag{13}$$

$$l_n/d > 0.2 \rightarrow F_{Rd}=(0.33t_w^2/\gamma_{a1})\left[1+(4l_n/d - 0,2)\left(t_w/t_f\right)^{1.5}\right]\sqrt{Ef_y t_f/t_w} \tag{14}$$

where *d* is the height of the column cross section and *E* is the modulus of elasticity of steel.

When the resistance force is greater than the design value of the internal force, there will be no need for transverse stiffeners in the column web.

4.5 Column web buckling under compression

The design resistance force of the column web under compression is given by

$$F_{Rd}=24t_w^3\sqrt{Ef_y}/d\gamma_{a1} \tag{15}$$

When the pair of concentrated forces is less than half the beam cross-section height, F_{Rd} must be reduced by half.

4.6 Lateral buckling of column web under compression (local compression of flange)

The column web under compression, caused by a localized force acting on the compressed flange, must be checked to the ultimate limit state (ULS) of lateral buckling, in case the relative lateral displacement between the loaded compressed flange and the tensioned flange is not prevented at the application force point. The value of F_{Rd} is computed as follows:

When the rotation of the loaded flange is prevented, for $(h/t_w)/(l/b_f) \leq 2.30$:

$$F_{Rd}=\frac{C_r t_w^3 t_f}{\gamma_{a1}h^2}\left[0.94+0.37\left(\frac{h/t_w}{l/b_f}\right)^3\right] \tag{16}$$

When the rotation of the loaded flange is not prevented, for $(h/t_w)/(l/b_f) \leq 1.70$:

$$F_{Rd}=\frac{C_r t_w^3 t_f}{\gamma_{a1}h^2}\left[0.37\left(\frac{h/t_w}{l/b_f}\right)^3\right] \tag{17}$$

where C_r is equal to $32E$, when $M_{Sd} < M_r$, and equal to $16E$ when $M_{Sd} \geq M_r$ in the force section; *l* is the longest length unlocked laterally, involving the acting section of the concentrated force considering the two flanges. M_{Sd} is the requesting bending moment and M_r is the bending moment corresponding to the beginning of the yielding.

If $(h/t_w)/(l/b_f)$ exceeds 2.30 or 1.70, when the rotation of the loaded flange is prevented or not, the ultimate limit state of lateral buckling of the web has no possibility of occurring.

4.7 Resistance of transverse stiffeners (column web in tension)

The resistance force of the transverse stiffeners necessary to resist localized forces that produce tension in the column web is given by the lower value between the yielding of the gross section (Eq. 18) and the rupture of the net section (Eq. 19):

$$F_{t,Rd}=A_g f_y/\gamma_{a1} \tag{18}$$

$$F_{t,Rd}=A_e f_u/\gamma_{a2} \tag{19}$$

4.8 Resistance of transverse stiffeners (column web in compression)

The resistance force of the transverse stiffeners necessary to resist localized forces that produce compression in the column web is given by

$$F_{c,Rd} = \chi Q A_g f_y / \gamma_{a1} \tag{20}$$

The cross section to be considered is formed by the stiffeners and a web strip, whose width is equal to $12t_w$, if the stiffeners are located at the ends, and equal to $25t_w$, if they are in an internal section. The buckling length must be taken equal to $0.75h$. The reduction factor associated with compressive strength (χ) and the total reduction factor associated with local buckling (Q) are determined according to section 5.3.3 and *appendix F* of the Brazilian standard, respectively.

4.9 Endplates under bending

The design stresses in the endplate due to the shear force ($V_{b,Sd}$) are considered to be low enough that they do not significantly interact with the effects of the bending moment ($M_{b,Sd}$) and the normal force ($N_{b,Sd}$), therefore:

$$V_{b,Sd} / 2ht_p \leq 0.2\ (0.6f_y/1.10) \tag{21}$$

$$V_{b,Sd} / n_L b_p t_p \leq 0.2\ (0.6f_y/1.10) \tag{22}$$

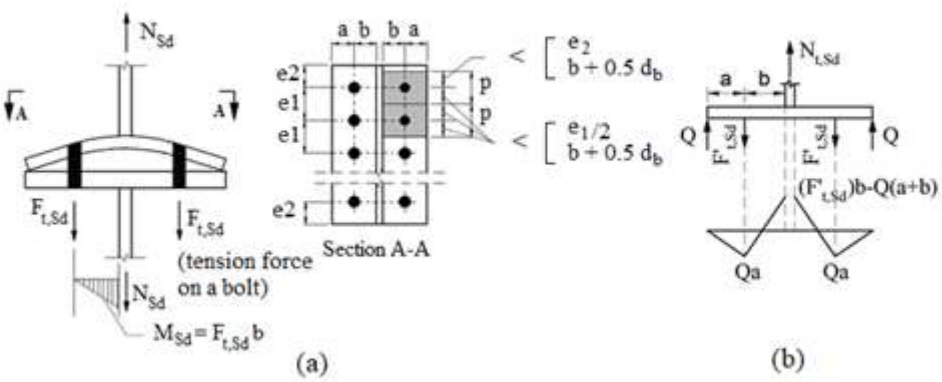
where h is the distance from center to center of the upper and lower beam flanges, t_p is the plate thickness, n_L is the number of horizontal bolt-rows and b_p is the plate width. The requesting normal force in beam ($N_{b,Sd}$) is also considered to be less than or equal to 5% of the normal resistance force ($N_{b,pl,Rd}$).

The design requesting moment acting on the endplate (M_{sd}) can be calculated without considering the lever effect, as shown in Figure 4a. In the absence of this effect, each bolt connecting the upper beam flange is subject to a force equal to:

$$F_{t,Sd} = N_{Sd}/n \tag{23}$$

where n is the number of bolts in the tension region.

Figure 4 – Lever effect.



Source: Adapted from ABNT (2008).

Thus, the requesting moment can be obtained by:

$$M_{Sd} = F_{t,Sd}\, b \tag{24}$$

The design moment resistance on the plate (M_{Rd}) can be taken equal to M_{pl} /1.10, using the tributary width (p) (see Figure 4a) for the calculation of M_{pl}:

$$M_{pl} = Zf_y = \frac{p{t_p}^2}{4} f_y \tag{25}$$

The tributary width is calculated as shown in Figure 4a. The value of p for external and internal bolts are given, respectively, by:

$$p = \left(< \begin{cases} \mathrm{e_2} \\ \mathrm{b} + 0{,}5\mathrm{d_b} \end{cases}\right) + \left(< \begin{cases} e_1/2 \\ \mathrm{b} + 0{,}5\mathrm{d_b} \end{cases}\right) \tag{26}$$

$$p = 2\left(< \begin{cases} e_1/2 \\ \mathrm{b} + 0{,}5\mathrm{d_b} \end{cases}\right) \tag{27}$$

where e_1 is the distance between hole centers and e_2 is the distance from the hole center to edge.

For connections with initial prestressed bolts, the lateral ends restriction of the connection plate causes the appearance of forces Q at these ends. These additional forces cause an increase in the tensile forces acting on the bolts as well as a change in the bending moments acting on the connection plate, due to the lever effect. The determination of the lever effect is complex and depends on several parameters, mainly the thickness of the plate and the geometry. If more rigorous analyzes are not carried out, ABNT NBR 8800:2008 (ABNT, 2008) allows, in a conservative way, a simplified procedure, in which it is assumed that the lever effect is properly considered if one of the possibilities below is met:

a) For the thicknesses of the connected parts, the plastic moment resistance, Zf_y, and the resistant tensile force for bolts design are reduced by 33%;

b) For the thicknesses of the connected plates, the elastic moment resistance, Wf_y, and the resistant tensile force for bolts design are reduced by 25%.

In a less conservative way, the lever effect force (Q) can be considered based on the limit situation, as shown in Figure 4b. In this case, the calculation of the requesting force on the bolt ($F_{t,Sd}$) is given by:

$$F'_{t,Sd} = N_{Sd} / n + Q = F_{t,Sd} + Q \tag{28}$$

where $F_{t,Sd}$ is the design requesting force on the bolt without the lever effect, and n is the total number of bolts. Considering equal the positive and negative bending moments (extreme situation), it is obtained:

$$Qa = F'_{t,Sd}b - Q(a+b) \tag{29}$$

$$F'_{t,Sd} = Q(2a+b) / b \tag{30}$$

Equating Eqs. (28) and (30), it arrives at:

$$Q = 0.5F_{t,Sd}(b/a) \tag{31}$$

Finally, with the condition already imposed that a is equal to or greater than b, it is led to:

$$Q_{máx} = F_{t,Sd} /2 \tag{32}$$

Therefore, the lever effect varies from 0 to 50% of the $F_{t,Sd}$ value calculated without this effect.

To assess intermediate situations between these two extremes and obtain a plate thickness lower than that, which would be necessary without taking into account the lever effect, the procedure presented by Queiroz and Vilela (2012) is adopted. In this procedure, the bending moment Qa must be equal to or less than δM_{Rd}, where δ is the relationship between the design resistance of the section with hole and M_{Rd}, determined as follows:

$$\delta = \left(< \begin{cases} 1.0 \\ \left[\dfrac{(p-d')(f_u/1.35)}{p(f_y/1.10)}\right] \end{cases}\right) \tag{33}$$

where $d' = d_b + 1/8''$; d_b is the diameter of the bolt and p the tributary width according to Eqs. (26) and (27). Making the bending moment close to the "T-stub" web equal to M_{Rd}, it is obtained:

$$F_{t,Sd}b - Qa = M_{Rd} \tag{34}$$

$$\alpha = (F_{t,Sd}b - M_{Rd})/\delta M_{Rd} \tag{35}$$

The design tensile force on the bolt is given by:

$$F_{t,Sd}+Q \tag{36}$$

$$Q=(F_{t,Sd}b - M_{Rd})/a \tag{37}$$

If $Q < 0$, make $Q = 0$. The use of stiffeners on the column at the level of the tensioned beam flange reduces the local stresses on the column flange, making the described procedures in favor of safety.

4.10 Beam web and flange under compression

The strength of the web and flange of the beam under compression is given by the same formulation presented in section 4.8 of this article.

4.11 Beam web and flange under tension

The resistant force of the beam web to resist the tensile force is given according to section 4.7 for the yielding of the gross section and rupture of the net section.

4.12 Resistance of bolts under tension

The resistance force of a bolt under tension is given by:

$$F_{t,Rd} = A_{be}f_{ub}/\gamma_{a2} \tag{38}$$

where A_{be} is the effective area equal to 75% of the gross area of the bolt cross-section, f_{ub} is the bolt rupture strength and γ_{a2} is the weighting coefficient of the resistance to rupture, equal to 1.35.

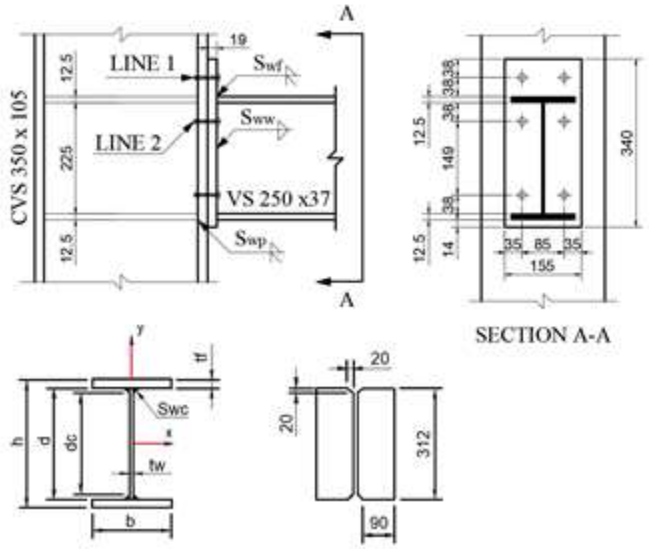
5. Comparative Analysis

This section aims to compare the moment x relative rotation curves of a connection with extended endplate, obtained by the mechanical model considering the European and Brazilian standards**,** and the one obtained by the experimental test executed by Ribeiro et al. (1998). Subsequently, the behavior of the connections is evaluated by varying the thickness of the endplate. The calculation memory of the values in tables can be checked in Mendes (2017).

5.1 Connection characteristics

The bolted connection with extended endplate is transversely stiffened on the upper and lower beam flange, as shown in Figure 5. The welded profiles used for the column and beam have yield strength equal to 250 MPa, modulus of elasticity of 200 GPa and consist of CVS profiles 350 x 105.0 and VS 250 x 37.0, respectively. The bolts are made of ASTM-A325 steel with a diameter of 19 mm. The endplate has a thickness of 19 mm and yield strength equal to 276 MPa. The thickness of the stiffener is 1.25 cm and the yield strength (f_y) is 25kN/cm².

Figure 5 – Geometric characteristics of the connection (mm).



Source: Authors (2021).

where: Sww is the weld leg between the beam web and the endplate, Swf is the weld leg between the beam flanges and the endplate, Swc is the weld leg of the column and Swp is the weld leg of the stiffener on the column. In this study, Sww = 5 mm, Swf = 5 mm, Swc = 6 mm and Swp = 6 mm.

5.2 Results of the connection

The results of the resistance force of the active components of the connection obtained according to the procedures of EN 1993-1-8:2005 and ABNT NBR 8800:2008 (ABNT, 2008) are presented in Tables 2 and 3, respectively. Table 4 presents the parameters for elaboration of the $M\text{-}\theta_r$ curve, when dealing with an elastic-plastic behavior, considering both standards. The values presented in parentheses are results obtained without the influence of the safety coefficients, which are γ_{Mo}, γ_{M1} and γ_{M2} for analysis according to Eurocode, and γ_{a1} and $\gamma_{a2,}$ for analysis according to Brazilian standard. The initial rotational stiffness employed in the analysis based on the Brazilian norm is the same as that determined by EN 1993-1-8:2005 (CEN, 2005).

Table 2 – Results of resistance force and stiffness of the connection components – EN 1993-1-8:2005.

	Active components	Resistance force (kN)	Stiffness (cm)
1	Column web in shear	549.44 (549.44)	∞
2	Column web in compression	1,004.77 (1,004.77)	∞
3	Column web in tension – line 1	439.34 (439.34)	0.48
3	Column web in tension – line 2	525.67 (525.67)	0.58
4	Column flange in bending – line 1	236.94 (236.94)	3.27
4	Column flange in bending – line 2	253.04 (283.50)	3.91
5	Endplate in bending – line 1	187.17 (220.56)	1.22
5	Endplate in bending – line 2	253.04 (302.53)	2.86
7	Beam flange and web in compression	490.18 (490.18)	∞
8	Beam web in tension	323.35 (323.35)	∞
10	Bolt in tension	126.52 (158.15)	0.90

Source: Authors.

Table 3 – Results of resistance force of the connection components – ABNT NBR 8800:2008.

	Active components	Resistance force (kN)
1	Column web in shear	656.25 (656.25)
2	Local bending of the column flange (web in tension)	512.78 (564.06)
3	Local yielding of the column web under tension	758.44 (834.28)
4	Local yielding of the column web under compression	715.63 (787.19)
5	Column web wrinkling under compression	789.67 (789.67)
6	Column web buckling under compression	871.62 (958.78)
7	Lateral buckling of column web under compression	6,340.07 (6,974.08)
8	Column flange under bending	403.20 (448.02)
9	Endplate under bending	362.08 (406.40)
10	Beam web and flange under compression	530.45 (583.50)
11	Beam web and flange under tension	391.34 (528.25)
12	Rupture of bolts under tension	519.80 (702.90)

Source: Authors.

From Tables 2 and 3, it is observed that the endplate bending is the component that limits the connection resistance, with failure mode 2, in which the endplate suffers plastification and the bolts break. Failure mode 1 occurs when there is only the plate plastification, and failure mode 3 occurs when there is only rupture of the bolts. The mentioned failure modes are presented in the studies of Zoetemeijer (1974).

Table 4 – M-θ_r curve data - elastic-plastic analysis.

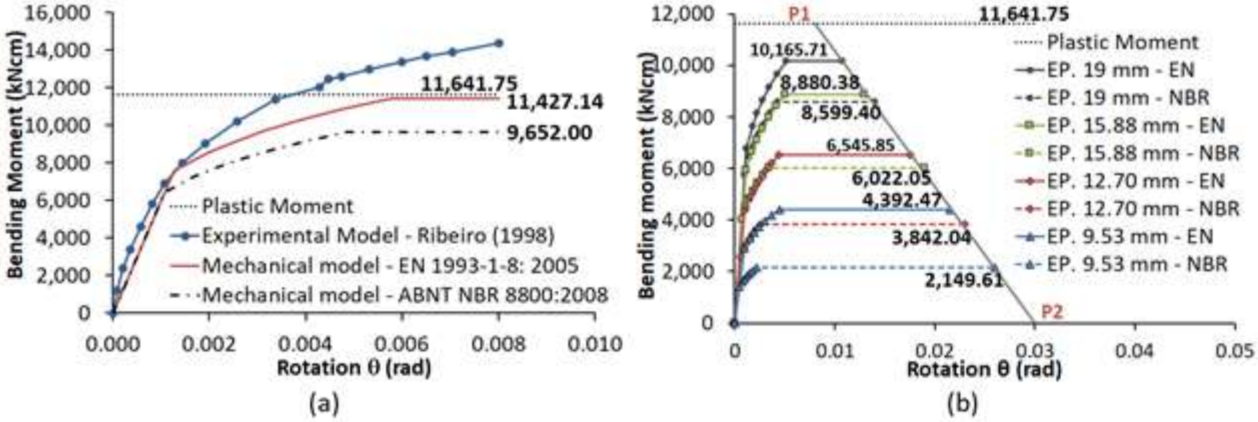
Bending Moment (kNcm)			Rotational Stiffness (kNcm/rad)		Rotation (rad)		
	EN	NBR 8800		EN/NBR 8800		EN	NBR 8800
$2/3M_{j,Rd}$	6,777.14 (7,618.09)	5,732.93 (6,434.67)	$S_{j,ini}$	5,943,425.60 (5,943,425.60)	θ	0.0011 (0.0013)	0.0010 (0.0011)
$M_{j,Rd}$	10,165.71 (11,427.14)	8,599.40 (9,652.00)	Sj	1,988,765.47 (1,988,765.40)	θ	0.0051 (0.0057)	0.0043 (0.0049)

Source: Authors.

5.3 Moment x rotation curves – Comparative analysis

Figure 6a shows the moment x rotation curves with parameters determined by the component method considering the procedures of European and Brazilian standards. The curve obtained by Ribeiro et al. (1998) is also depicted. For comparison purposes, the safety coefficients were not considered when calculating the parameters of the active components.

Figure 6 – Moment x relative rotation curve.



Source: Authors.

According to Figure 6a, it can be seen that the Brazilian standard design is more rigorous with regard to the bending moment resistance, with resistance about 15% lower than that of the European standard design. Furthermore, it is observed that the curve of the experimental model exceeded the plastic moment of the beam (M_{pl}) equal to 11,641.75 kNcm, which assumes that the yield strength of test steel profiles used by Ribeiro et al. (1998) exceed the theoretical value of 250 MPa adopted.

In the experimental model, the connection failed due to rupture of the bolts in the internal row. The plate also showed a deformation; however, it was the bolts that collapsed. According to the European standard mechanical model, the failure mode was mode 2, which is due to the interaction between the plate bending and rupture of the bolts. Nevertheless, this failure mode does not specify the contribution of the plate or bolt to the collapse of the connection. According to the Brazilian standard, the endplate bending was the component that limited the beam-to-column joint strength.

The previous study was extended using thicknesses of 9.53 mm; 12.70 mm, 15.88 mm and taking into account the design values (consideration of safety parameters). The M-θ_r curves were developed according to the normative criteria. Figure 6b shows the M-θ_r behavior of the connections by varying the thickness of the endplate (EP). To define the rotational capacity of the connections, a straight line with a slope defined by two points P1 = (0.008; 11641.75) and P2 = (0.03; 0) is considered in a simplified way. Point P1 is defined by the rotation equal to 0.008 rad, which was obtained by Ribeiro et al.'s test (1998) for endplate with thickness equal to 19 mm, and by the value of the beam's plastic moment equal to 11,641.75 kNcm. Point P2 is defined by the rotation capacity equal to 0.03 for a perfectly flexible connection (M = 0).

The parameters of stiffness, strength and rotation capacity are shown in Table 5. For both standards, the endplate bending is the component that limits the value of the bending moment resistance of the connections for all plate thicknesses. The values of rotational stiffness determined by ABNT NBR 8800:2008 follow the procedures of Eurocode 3.

Table 5 - Connection parameters.

Endplate thickness (mm)	Resistant bending moment $M_{j,Rd}$ (kNcm)		Initial rotational stiffness $S_{j,ini}$ (kNcm/rad)	Rotation capacity θ_{cd} (rad)	
	EN	NBR 8800	EN/NBR 8800	EN	NBR 8800
19.00	10,165.71	8,599.40	5,943,425.60	0.011	0.014
15.88	8,880.38	6,022.05	5,278,235.04	0.013	0.019
12.70	6,545.85	3,842.04	4,408,860.34	0.018	0.023
9.53	4,392.47	2,149.61	2,923,485.81	0.022	0.026

Source: Authors.

From Figure 6b and Table 5, it is observed that the bending moment resistance decreases as the thickness of the endplate becomes thinner. It is also noted that all connections designed with the Brazilian standard have a lower bending moment resistance compared to the European standard design, getting worse as the thickness of the plate decreases. This difference is due to the procedure adopted, which takes into account the lever effect in determining moments on the plate, as performed by Queiroz and Vilela (2012).

6. Conclusion

The objective of this work was to present a study of the behavior of beam-column connections made up of extended endplates with transverse stiffeners. A comparison analysis was made between the moment x relative rotation curves, obtained by the experimental model proposed by Ribeiro (Ribeiro et al., 1998) and the mechanical model according to EN 1993-1-8:2005 (CEN, 2005) and ABNT NBR 8800:2008 (ABNT, 2008). The curves obtained by the standards in question showed a good approximation with the experimental curve, mainly in the elastic branch; nonetheless, they are conservative in relation to the moment resistance.

ABNT NBR 8800:2008 (ABNT, 2008) allows a less rigorous analysis of connections for situations where there is a lever effect, which eliminates the determination of the Q force. The simplified design methodology of the endplate leads to plates with greater thicknesses. The most economical design results are found when the plate is considered to have a thin plate behavior, obtaining the strength of the connection subjected to the maximum lever effect. Therefore, a complement was added to the simplified procedure of the Brazilian standard, which was presented by Queiroz and Vilela (2012). This complementary procedure assesses the intermediate situation between the two extreme cases, i.e, the absence and the extreme situation of lever force, in determining moments on the plate.

Subsequently, an analysis was carried out by varying the thickness of the connection’s endplate. A connection with an extended endplate can show different moment resistance and rotation capacity simply by changing the plate thickness. The stiffness and resistance increase significantly with the increment of the endplate thickness, being the resistance the most influenced aspect.

The authors concluded that it may be incorrect not to associate the concept of semi-rigid behavior to connections with transversely stiffened endplates, since the behavior of such connections can be quite different from the ideally rigid connection assumption. Nevertheless, adequate knowledge of the connections’ behavior allows to adopt, in a conscious and safe way, the simplified analysis procedures with hypotheses of idealized behaviors.

This study was limited to the analysis of one type of beam-to-column connection. Therefore, for future research work, it is suggested the study of the behavior of other types of joints, such as connections with top and seat angles or with flush endplate, as well as its applications in steel frame structures.

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References

ABNT, N. B. R. (2008). 8800: Projeto de estruturas de aço e de estruturas mistas de aço e concreto de edifícios. *Associação Brasileira de Normas Técnicas*.

Ataei, A., Bradford, M. A., & Valipour, H. R. (2015). Moment-rotation model for blind-bolted flush end-plate connections in composite frame structures. *Journal of Structural Engineering*, *141*(9), 4014211.

CEN, E. N. (2005). 1-8 Eurocode 3: Design of Steel Structures–Part 1–8: Design of Joints. *European Committee for Standardization, Brussels*.

Committee, A. (2010). Specification for structural steel buildings (ANSI/AISC 360-10). *American Institute of Steel Construction, Chicago-Illinois*.

Dave, U., & Savaliya, G. (2010). Analysis and Design of Semi-Rigid Steel Frames. *Structures Congress*, 3240–3251.

Faella, C., Piluso, V., & Rizzano, G. (1999). *Structural steel semirigid connections: theory, design, and software* (Vol. 21). CRC press.

Gil, A. C. (2002). *Como elaborar projetos de pesquisa* (Vol. 4). Atlas.

Hortencio, R. da S., & Falcón, G. A. S. (2018). Optimal design of beam-column connections of plane steel frames using the component method. *Latin American Journal of Solids and Structures*, *15*(11).

Kishi, N., & Chen, W.-F. (1990). Moment-rotation relations of semirigid connections with angles. *Journal of Structural Engineering*, *116*(7), 1813–1834.

Kong, Z., & Kim, S.-E. (2017). Moment-rotation model of single-web angle connections. *International Journal of Mechanical Sciences*, *126*, 24–34.

Kong, Z., & Kim, S.-E. (2018). Numerical estimation for initial stiffness and ultimate moment of T-stub connections. *Journal of Constructional Steel Research*, *141*, 118–131.

Lee, S.-S., & Moon, T.-S. (2002). Moment–rotation model of semi-rigid connections with angles. *Engineering Structures*, *24*(2), 227–237.

Mendes, F. T. C. (2017). *Determinação das propriedades mecânicas das ligações viga-pilar com chapas de extremidade estendida visando à análise pelo método dos componentes*. Dissertação de Mestrado — Centro Federal de Educação Tecnológica de Minas Gerais.

Oliveira, C. de R. (2011). *Estudo do comportamento de uma ligação viga-pilar*. Dissertação de Mestrado — Instituto Politécnico de Viseu.

Oliveira, L. A. R. de. (2015). *Análise de pórticos de aço com ligações viga-pilar e de base de pilar semirrígidas a partir do método dos componentes*. Dissertação de Mestrado — Universidade Federal de Minas Gerais.

Pfeil, W., & Pfeil, M. (2000). *Estruturas de Aço: Dimensionamento Prático* . Grupo Gen-LTC.

Queiroz, G., & Vilela, P. M. L. (2012). *Ligações, regiões nodais e fadiga de estruturas de aço*.

Ribeiro, L. F. L., Gonçalves, R. M., & Castiglioni, C. A. (1998). Beam-to-column end plate connections-an experimental analysis. *Journal of Constructional Steel Research*, *1*(46), 264–266.

Shi, G., & Chen, X. (2017). Moment-rotation curves of ultra-large capacity end-plate joints based on component method. *Journal of Constructional Steel Research*, *128*, 451–461.

Tahir, M. M., Mohammadhosseini, H., Ngian, S. P., & Effendi, M. K. (2018). I-beam to square hollow column blind bolted moment connection: Experimental and numerical study. *Journal of Constructional Steel Research*, *148*, 383–398.

Thai, H.-T., & Uy, B. (2016). Rotational stiffness and moment resistance of bolted endplate joints with hollow or CFST columns. *Journal of Constructional Steel Research*, *126*, 139–152.

Viana, H. F., Salles, M. C. S. P., Silva, R. G. L., Lavall, A. C. C., & Costa, R. S. (2019). Nonlinear elastic transient analysis of steel frames with semi-rigid connections. *XL Ibero-Latin-American Congress on Computational Methods in Engineering*, 13. http://limacloud.duckdns.org:89/CILAMCE/5769.pdf

Yee, Y. L., & Melchers, R. E. (1986). Moment-rotation curves for bolted connections. *Journal of Structural Engineering*, *112*(3), 615–635.

Yu, H., Burgess, I. W., Davison, J. B., & Plank, R. J. (2009). Tying capacity of web cleat connections in fire, Part 2: Development of component-based model. *Engineering Structures*, *31*(3), 697–708.

Zhou, G., An, Y., Li, D., & Ou, J. (2019). Analytical model of moment-rotation relation for steel beam to CFST column connections with bidirectional bolts. *Engineering Structures*, *196*, 109374.

Zhou, G., An, Y., Wu, Z., Li, D., & Ou, J. (2018). Analytical Model for Initial Rotational Stiffness of Steel Beam to Concrete-Filled Steel Tube Column Connections with Bidirectional Bolts. *Journal of Structural Engineering*, *144*(11), 4018199.

Zoetemeijer, P. (1974). A design method for the tension side of statically loaded, bolted beam-to-column connections. *HERON, 20 (1), 1974*.