Performance of Semi-Rigid Steel Connections under Monotonic and Cyclic Loadings: A Review

Rasha K. AL-Fisalawi¹,², Laith Khalid AL-Hadithy² and Mustafa Kamal AL-Kamal²

¹Graduated student, Department of Civil Engineering, Al-Nahrain University, Baghdad Iraq.
²Structural Engineering, Faculty Member, Department of Civil Engineering, Al-Nahrain University, Baghdad, Iraq.
³rasha.faisaly@yahoo.com

Abstract: Since the turn of the century, numerous articles have been published with analysis of semi-rigid connections in steel structures. This paper offers a comprehensive survey of major recently published research work dealing with the behaviour of semi-rigid beam-column connections under various configurations of fasteners and welding lines under both monotonic and cyclic loads. The review has two main respects: the first is the moment versus curvature behaviours of semi-rigid steel connections, while the second involves finite element analysis of such connections under monotonic and cyclic loads. The main conclusions concerning the dynamic behaviours of semi-rigid steel connections emerge with regard to the vital influence of beam-column connections on the global seismic performance of steel frame structures. Developing semi-rigidity should thus be considered an effective way to achieve the required performance.

1. Introduction
An uncountable number of structures are now made of steel expressing the enormous possibilities that this material offers. Some of justifications for the choice of steel to build a structure or its elements, include its high strength to volume ratio, its reliability, and its ability to adapt to almost any architectural form, offering a wide range of possible applications, these are supported further by the availability of a large number of standardised parts.
Owing to their high ductility and energy dissipation abilities, semi-rigid steel connections have been favoured in recent moment-resisting steel frames exposed to gravitational monotonic loading alongside lateral or vertical cyclic excitations. Adequate design of members’ end-to-end connections is thus required to allow these steel structures to perform well in sustaining such loads. Yet the conventional analysis of steel framed structures supposes one of the two well-known idealised extremities: the rigid joint or pinned joint hypotheses. However, currently prevalent steel frame connections are most likely to display semi-rigid responses, contributing significantly to overall member stress distribution. In general, steel structures can be formed from any combinations of simple or composite pieces joined together in a design that adequately resists forces and moments together.
For this to occur, the projected joints must be capable of adequately transmitting forces and moments between members as required. The behaviour of a connection is thus usually quite complex and has a high degree of indeterminacy, which on many occasions makes rigorous analysis unfeasible or simply uneconomic. The expressions used for dimensioning are thus generally based on empirical considerations expanding on the results of tests that have been correctly carried out. Joining techniques can be classified as fixed (rivets and welds) and removable (bolts). Rivets have traditionally been the most widely used joining element in steel construction, although these have now practically ceased to be used in some countries because of the high labour costs of rivet joints, which required several additional operations such as the layout and drilling of holes. They also add more weight than welded joints due to requiring physical and rivets, which also make them bulkier than welding, affecting the joint appearance. The main advantages of bolted joints as compared to welded joints are that the execution of the joint can be carried out in unfavorable atmospheric conditions, they offer better behaviours against fatigue and brittle breakage, the costs related to the quality control are lower, and the lead times are shorter; however, the project costs derived from their calculation and representation are higher; the assembly of the parts is more demanding, with tighter tolerances; and the final appearance is less aesthetic than with welded joints.

Every joint represents, by nature, a discontinuity and, thus a potentially dangerous area. Most problems that arise in steel structures are usually due to poorly designed or poorly executed joints.

For the proper analysis of a structure, it is necessary to define the types of joints with which the connections between elements will be made. To achieve this, it is necessary to define the constraints to movement (relative rotational displacements) at ends of the meeting members, which allows determination of the values and types of forces transmitted between them. Application standards for the analysis and design of the joints only permit their resistance to be checked against the stresses that request them, without the inclusion of any criteria for stiffness. To ensure that the joints are rigid, if this is required, the nodes are solved for this by the systematically placing of stiffeners. However, this causes execution costs to increase substantially without any precise evaluation. More recent standards have made it possible to analyse the real behaviour of joints based on their moment-rotation curve, and thus to consider their effects both on the strength of the joint and on the overall behaviour of the structure. This type of analysis allows structural designers to decompose the unions in the different elements that compose a structure and evaluate, the deformability, rigidity and resistance of each one. This method, the Component Method (MC), was studied by several different researchers such as Zoetemeijer [1], Yee and Melchers [2], Jaspart [3] and Faella et al. [4].

Identifying an approximation to the real behaviour of the joints has a considerable impact on the cost of the materials used. Many joints designed as articulated in fact have some stiffness, while other joints may behave as rigid even where they do not include stiffeners. The study of both of these as semi-rigid joints would allow reduction in the dimensions of the beams, and therefore in the total cost of the structure. Any savings in labour costs also potentially have a significant influence on the overall cost of the structure, making it advantageous to unify and standardise the different types of joints, to take the advantages of serial manufacturing, as well as to facilitate workshop manufacture for assembly in the construction situation. The process of designing a joint also requires extensive experience from the structural designer.

The theory of mathematical optimisation applies numerical methods focusing on searching for the best design among a collection of alternatives, without having to explicitly evaluate all of the alternatives. The concept of optimisation is fundamental to design, as the classic function of the designer is to design new, better, more efficient and less expensive systems. In general, a problem of optimisation engineering design consists of examining design suggestions for certain loading
variables; by checking how these fulfilling a set of requirements, usually represented by algebraic equations or inequalities, the best current possible design can be selected.

2. Moment-rotation behaviours of semi-rigid connections

2.1 Mathematical Expressions
The moment-rotation behaviours of connections can be generated using mathematical models that estimate behaviour patterns without the need for experimentation. Advanced early models provided curve fitting of testing data based on regression analysis. In 1975, Frye and Morris [5], formulated an odd-powered empirical polynomial model that explicitly utilised rotation as a mechanism of moment along with other curve-fitting variables. Curves can thus be attached to the experimental observations for connections exposed to monotonic loading, which generally resulted in a relationship with M-Θ is expressed as:
\[
θ_d = C_1(KM)^1 + C_3(KM)^3 + C_5(KM)^5
\]
where \(θ_d\) is the angle of rotation of the connection in (rad); \(M\) is the moment applied in the connection (kN.m); \(K\) is a constant that depends on the geometric and mechanical properties of the union; and \(C_1, C_2\) and \(C_3\) are curve adjustment constants obtained by regression analysis.

Various studies by different researchers expanded the field of application of such models. Many types of connection were thus studied and some typical results for parameters \(K, C_1, C_2\) and \(C_3\) are shown in Table 1 and Figure 1.

| Types of Connections                  | Constants for Curves Fittings | Constants for Standardisations |
|--------------------------------------|-------------------------------|--------------------------------|
| Single-Web Connection                | \(C_1 = 42.8\times10^{-4}\)   | \(K = d - 2.4 \cdot t - 1.18 \cdot g \cdot 0.15\) |
|                                      | \(C_2 = 14.5\times10^{-10}\)  |                                |
|                                      | \(C_3 = 15.1\times10^{-17}\)  |                                |
| Double-Web Angle Connection          | \(C_1 = 36.6\times10^{-5}\)   | \(K = d - 2.4 \cdot t - 1.18 \cdot g \cdot 0.15\) |
|                                      | \(C_2 = 11.5\times10^{-7}\)   |                                |
|                                      | \(C_3 = 45.7\times10^{-9}\)   |                                |
| Top-and Seat-Angle Connection        | \(C_1 = 84.6\times10^{-5}\)   | \(K = d - 1.5 \cdot t - 0.5 \cdot l - 0.7 \cdot db - 1.1\) |
|                                      | \(C_2 = 10.1\times10^{-5}\)   |                                |
|                                      | \(C_3 = 12.4\times10^{-9}\)   |                                |
| Top-and Seat-Angle - with Double Web-Angle Connection | \(C_1 = 22.3\times10^{-6}\)   | \(K = d - 1.287 \cdot t - 1.128 \cdot t - 0.415 \cdot l - 0.694 \cdot (g \cdot db/2) 1.350\) |
|                                      | \(C_2 = 18.5\times10^{-9}\)   |                                |
|                                      | \(C_3 = 31.9\times10^{-13}\)  |                                |
| End-Plate Connection with Column Stiffener | \(C_1 = 17.9\times10^{-4}\)   | \(K = d - 2.4 \cdot t - 0.6\) |
|                                      | \(C_2 = 17.6\times10^{-5}\)   |                                |
|                                      | \(C_3 = 20.4\times10^{-5}\)   |                                |
| T-Stud Connection                    | \(C_1 = 21 \times 10^{-5}\)   | \(K = d - 1.5 \cdot t - 0.5 \cdot f - 1.1 \cdot l - 0.7\) |
|                                      | \(C_2 = 62 \times 10^{-7}\)   |                                |
|                                      | \(C_3 = -76 \times 10^{-10}\) |                                |
| Header-Plate Connection              | \(C_1 = 51 \times 10^{-6}\)   | \(K = t - 1.6 \cdot g \cdot 1.6 \cdot d - 2.3 \cdot w \cdot 0.5\) |
|                                      | \(C_2 = 62 \times 10^{-11}\)  |                                |
|                                      | \(C_3 = 24 \times 10^{-14}\)  |                                |
Richard and Abbott [6] in 1975, developed a power model with three-parameters to develop a description for the rotational behaviour of a moment connection subject to monotonic loading:

$$M_u = R_{ki} \theta_r \left[ 1 + \left( \frac{\theta_r}{\theta_0} \right)^n \right]$$  \hspace{1cm} (2)

where $R_{ki}$ is the initial connection stiffness; $n$ is the shape parameter; $\theta_0 = M_u / R_{ki}$ is the reference plastic rotation; and $M_u$ is the ultimate moment capacity. The resulting moment versus rotation curves for various values are presented in Figure 2, which also shows the eventual moment-rotation curves for several values.

Ang and Morris [7] in 1984, examined the moment-rotation behaviours of disparate connection types by exploring the use of a standardised Ramberg-Osgood model as in the following form:

$$\frac{\theta}{\theta_0} = \frac{(KM)}{(KM)_0} \left[ 1 + \left( \frac{(KM)}{(KM)_0} \right)^{n-1} \right]$$  \hspace{1cm} (3)

where $\theta_0$, $n$ and $(KM)_0$ are constants that rely on dimensional proportioning and connection type. In 1986, Kishi and Chen [8] explained the relationship between moment and rotation as:
$$M = M_0 + \sum_{j=1}^{n} C_j \left[ 1 - \exp\left( \frac{\theta}{2|\alpha|} \right) \right] + \sum_{k=1}^{n} D_k (|\theta| - |\theta_k|) H(|\theta| - |\theta_k|) \quad (4)$$

where $M_0$ is the initial moment value of connection attached to the curve; $D_k$ represents the constant parameter of the linear part for the curve; $\theta_k$ is the rotation at beginning of the linear part in the curve; and $H[\theta]$ is a function of the Heaviside step.

**Figure 2:** A typical three-parameter power model (Richard and Abbott [6]).

Attigbe and Morris [9] in 1991, compared experimental results with the models proposed by Richard Abbott and Ramberg Osgood, finding that the first model provided better adjustments. They also used multiple regression analysis to obtain prediction equations for the double angle connection in the web. In 1997, Sherbourne and Bahaari [10] obtained empirical expressions the $M$-$\theta$ curve of the connection with an extended endplate with four bolts in the tension and compression zones. As well as using finite element parameterisation and the experimental results obtained by other researchers, they also used the Richard Abbott model. In 2005, Fengfeng Zhou [11] used finite element models and experimental trials to write mathematical expressions obtained by regression evaluating the $M$-$\theta$ curve of an extended endplate connection, while Syahril Taufit [12] in 2013, conducted similar studies on non-extended end plate connections, finding that the use of column stiffeners increased moment capacity by up to 15%.

### 2.2 Analytical Model

Analytical models are paragons built on the theory of elastic structural analysis using equipoise, compatibility and material-constituent relationships to predict initial rigidity. Ultimate moment is also predicted based on plastic analysis. Kishi [13] in 1988, used a three-parameter power function to standardise the moment-rotation relationship of top and seat-angle with double web-angle connections. Two parameters, the initial stiffness and ultimate moment capacity of a connection were determined by an analytical procedure in their closed forms. The moment-rotation behaviours of connections were predicted based on the power function and then compared with experimental results to ensure obtaining good agreement. The researcher utilised both geometric and mechanical properties to forecast the tangent stiffness and the moment resistance of the top angle and the seat-angle connections in addition to double-web-angle connections.
Lee and Moon [14] in 2002, developed a semi-analytical model to describe the non-linear behaviours of the moment-rotation curve, which depend on two parameters γ and n, as seen in Eq.5a, which was derived in terms of initial stiffness (\( R_{ki} \)) and plastic stiffness (\( R_{kp} \)) by applying a regression analysis. They developed expressions for connections with double angles in the web (Eq. 5b and 5c) and for top and seat angle connection (Eq. 5d and 5e), however, this model also provides acceptable M-\( \theta_r \) curve adjustments for other types of connections.

\[
M = \gamma [\ln(10^3 n \theta_r + 1)]^n \quad (5a)
\]

\[
\gamma = 1.499E - 0.3R_{ki} + 1.449E - 0.3R_{kp} + 0.704 \quad (5b)
\]

\[
n = -3.594E - 0.5R_{ki} - 3.496E - 0.5R_{kp} + 1.170 \quad (5c)
\]

\[
\gamma = 9.689E - 0.4R_{ki} + 9.562E - 0.4R_{kp} + 3.850 \quad (5d)
\]

\[
n = 4.500E - 0.6R_k + 4.400E - 0.6R_{kp} + 0.601 \quad (5e)
\]

The parameter \( \gamma \) affects the initial stiffness and final moment, while \( n \) governs the shape of the curve. The units of \( R_{ki} \) and \( R_{kp} \) are kN.m/rad, while \( E \) is in N/m.m². Among other investigations, Primoz et al. [15] in 2009, proposed semi-analytical equations for connection with top and seat angles based on settings in finite element and document analysis carried out by Azizinamini [16] and Komuro et al. [17]. Their study showed that axial tension reduces elastic stiffness and moment capacity, and the resulting model has the ability to predict the connection behaviours taking both axial load and moment into account.

2.3. Mechanical Models

Mechanical models were also presented as a promising method; these are focused on the physical properties required to estimate the attitude of the connection. In mechanical models, springs with specified load deformation characteristics are used to distinguish the various ingredients of any connection constructed from a combination of deformable and stiff components. Wales and Rossow [18] in 1983, were the first that use this type of model, and their model simulated the action of the double angle connection in the web, based on imposing axial loads and bending moments. They found that the axial force had a significant influence on the behaviours of the connection, and subsequently Simões da Silva and Girão Coelho [19] in 2001, evaluated the response of the welded beam-column connection under axial load and bending moments, determining that the axial load decreased the moment capacity of the connection. In 2009, Lemonis and Ghents [20] presented a methodology to estimate the totality of the M-\( \theta_r \) curve of the end plate seat angle, top angle and end plate connections in addition to a set of two angles of the web, characterising the revealing behaviours through the a T-equivalent method; they used the component method to obtain the corresponding mechanical model, validating this by means of experimental tests and numerical models. Del Sabio et al. [21] in 2009, proposed a mechanical model to estimate endplate type connection behaviours subjected to axial loads and moment simultaneously, as shown in Figure 3, while In 2011 and 2012, Hu et al. [22, 23] achieved high precision predictions for the behaviours of the beam-column connections with T-equivalence under cyclic loads.
3. Finite element analysis of semi-rigid connections under cyclic and monotonic loads

Finite element models can be extremely economic and reliable calculation tools, though they require adequate modelling to predict the behaviours of the connection reliably, as many different factors interact in such cases such as non-linearity in the material, pre-tensioning in the bolts, interaction between bolts and plates, friction forces, and landslides among other imperfections. Bursi and Jaspart [24,25] in 1997, presented the results of a finite element analysis of extended end-plate connections within ABAQUS; provided an overview of current developments for estimating moment-rotation characteristics and attempted to establish a methodology for finite element analysis. Calibration development of a three-dimensional FE model using test data of an elementary T-stub connection was followed by performed initially bolt actions to examine isolated extended end-plate steel connections recreation of the based on the assemblage of beam elements. The efficiency and accuracy of the submitted FE models were highlighted by comparing the calculated and measured values in each stage. Abolmaali et al. [26] in 2005, presented experimentally obtained moment-rotation hysteresis behaviours for a family of semi-rigid connections, including the double web angle (fully bolted and bolted to column flange and welded to beam web), top and seat angle, flush endplate, and extended endplate connections. He thus pointed out that these connections might not only be proportioned to possess similar moment capabilities to those seen in commonly used rigid connections but also to possess more ductility, as it is also extremely important to pre-tension the bolts in order to make use of the connection ductility under cyclic loads. For a few selected connections, he also developed analytical moment-rotation hysteresis models with varying degrees of complexity. In 2002 Yang et al. [27] in 2000, evaluated the load-displacement interactions, moment-rotation curves, and stress dispensation of double angle connections using the finite element program ABAQUS. The axial tensile loads, shear loads, and a combination of these were imposed on all variable types of connections. Three-dimensional simulation of the angle segregated from the column and modelling of the force applied between both the bolt heads and the angles were then attempted, and the experimental test data were compared with the simulation results to support these findings. In 2002, Swanson et al. [28] used 2D and 3D models in FE to simulate connection behaviours using T-stubs, these models incorporated several contact interactions between the column flange and T-stub, between the beam flange and T-stub, between the bolt heads and T-stub, and between the shear bolts and bolt holes. Their study revealed that four tension bolts are not sufficient to fully develop a mechanism in the thicker T-stub flanges examined, as well as the conclusion that T-
stub flange deformation is sensitive to bolt pre-tension but that flange strength is not greatly affected by bolt pre-tension as shown in Figure 4.

![Finite element model using ANSYS](image1)

**Figure 4**: Finite element model using ANSYS (Swanson et al. [28]).

In 2002, Citipitioglu [29] developed a 3D finite element model for a top-seat with double web angle connection, including the contact between all connection components, friction, and slip, and a method for applying pre-tension in the bolts. The models were compared with the experimental results produce by Azizinamini and Radziminski [30], and the results of the analysis highlighted the effect of blot pre-tension on the behaviours of the connection, which could vary the ultimate moment-rotation by up to 25%. They thus developed a three-dimensional model, as shown in figure 5. Various numerical investigations showed that the proposed analytical approach was in a good agreement with the experimental results.

![3D finite element model for seat angles and top angles with connection of two webs](image2)

**Figure 5**: 3D finite element model for seat angles and top angles with connection of two webs (Citipitioglu, [29]).
According to Abolmaali et al. [31] in 2005, used a nonlinear analysis of 3-D finite element model (FEM) to simulate flush end plate connections by using 8-node isoperimetric elements for all component parts with one row of bolts below the tension flange as shown in Figure 6. FE analyses progression of the connection (M- θ) data for these were tested then curve fitted to generate a three-parameter power model by using Ramberg-Osgood [32], and Kishi and Chen [33] equations to obtain the criteria to define the former equations. Assumptions of model parameter equations were developed using regression equations as mechanisms for flush-end-plate connection geometric variables. It was thus shown that the M– θ plots could be predicted closely by both models, with the three-parameter power model being the more accurate model.

![Finite element model of a flush end-plate connection (Abolmaali et al. [31]).](image)

In 2005, Maggi et al. [34] utilized parametric analyses to study the behaviours of bolted extended end plate connections using finite element analysis. T-stub failure models were used for calculations of the flexural strength for the end plate, while the failure modes included the formation of yield-lines in the plate and bolt tension failure was well defined. Failure due to a combination of these mechanisms represented the levels of interaction between the end plate and bolts which can otherwise be difficult to predict accurately. The models are calibrated with experimental results, and nonlinearities of material, as well as geometric and large displacements were applied, as shown in Figure 7.
In 2006, Cabrero [35] developed finite element models in ABAQUS to simulate connections with extended endplates. The elements used 8-node and incompatible modes with rigid contact. The bolts were modelled with high strength and the results compared with experiments, giving excellent agreements. Pirmoz et al. [15] in 2009, studied the behaviours of top-seat angles with double web angles connections subjected to shear and moment requests, conducting a parametric study of the geometric and mechanical properties. They used a 3D model with solid elements and elements of contact, with pre-stressing of the bolts modelled in the first case load. The results were then compared with those obtained experimentally by other researchers, and good correlations were obtained.

He et al. [36,37], in 2010 constructed a 3-D nonlinear finite element model to evaluate the mechanical properties of steel frame beam-to-column joints with joined panels to examine their performance and cyclic behaviours with respect to strength, rigidity, and hysteretic responses. Monotonic loading was applied in one of the parametric studies, being imposed on the connections to investigate the influences of connection geometry, with the result that the resistance ratio was found to influence the failure modes, whereas the thickness of the joint panels had a larger effect on strength and rigidity under shear failure mode. Wang [38] in 2012, conducted tests on two full-scale specimens of angled steel, using a H-section member. The specimens were subjected to cyclic reversal loads on a steel moment-resisting force well within the inelastic range to ascertain the effect of design parameters such as column flange stiffener, pre-tensioning of bolts and the angle’s flange thickness on overall behaviours. Observations were thus made on the response of the connections and elements in terms of strength, stiffness and energy dissipation, also in 2017 Mehr and Ghobadi [39] proposed a methodology and retrofitting detail of welded flange plate (WFP) moment resisting connections as shown in figure 8. Existence of defected complete joint penetration (CJP) groove weld only at the top flange of the connection and the presence of box columns with different ductility were the main construction challenges for these connections. An experimentally validated analytical study was thus performed to evaluate the applicability of the design approach involving retrofitting connections with rib plates. The ductility of the column and the number of utilized rib plates were the main criteria considered, and the potential for fracture in the connection welds was investigated in cyclically loaded specimens. The experimental and analytical results proved the applicability of their proposed design method.

**Figure 7:** Overview of FEM representation and meshing of the bolted extended end plate connection (Maggi et al. [34])
Figure 8: Schematic detail of WFP connection with fillet weld or with defective CJP groove (Mehr and Ghobadi [39]).

In 2017, Kong and Kim [40] established the characteristics for top-seat angles with double web angles connections from collapse mechanisms developed by Kishi and Chen [41] where these observations assisted in the estimation of moment rotation attitude in this connection. These researchers also proposed a capacity of ultimate moment model which agreed well with various test data. A more precise model of the moment-rotation relationship was similarly proposed. In 2018, Chen, Jiang and Jia [42] carried out a finite element investigation of beam-column composite connection and adapter brackets. The latter included two end plates connected to the column and the beam by bolts and welding, respectively. As shown in Figure 9, the joint of bolted extended end plate was modelled in ABAQUS and verified on the basis of experimental results, they conducted a parametric study to assess the efficiency of performance of the innovated joint based on evaluating the rotational stiffness, moment resistance, and fracture mechanisms. Their main parameters were the thicknesses of flanges and end plates in addition to the size of the fasteners. They thus claimed that the performance of their joint was primarily affected by the three dimensional variables identified, and only slightly affected by the thickness of the second end plate.

Figure 9: Numerical and experimental failure modes: (a) MES1 and (b) MES2 (Chen, Jiang and Jia [42]).
El Kalash and Hantouche \[43,\] in 2019 investigated the behaviours of eight-bolt extended endplate connections with circular bolts configurations under monotonic and cyclic loadings. The results of a series of component experimental tests and finite element simulations were used to develop a strength model to predict the connection capacity and prying forces, and the effect of the column flange thickness, the endplate thickness, and the bolt diameter on the connection performance and the prying forces were investigated as Figure 10.

![Deformation of connection](image)

**Figure 10.** Deformation of connection: (a) After testing (b) from FE( El Kalash and Hantouche [43]).

In 2019, Hasan, Al-Deen and Ashraf \[44\] carried out an investigation on the performance of connections consisting of double web angles and top seats. They conducted experimental tests on full scale connections, and then they used the results to construct a finite element model. From their experimental results they verified the considerable moment resistance accompanying significant rotations that created substantial inelastic at varies portions within the joint, they also observed visible deformations in the top angle and the attached bolts connecting this to the column. Plastic deformation of the seat angles and those attached to the webs in addition to the holes of the bolts was dominant during loading. Figure 11 shows their finite element model, indicating that the numerical prediction is in good agreement with the experimental evidence.
Figure 1: Deformation of connection: (a) after testing (b) from FEM(Hasan, Al-Deen and Ashraf [44]).

4. Conclusions

Examining existing recent literature on the behaviours of semi-rigid connections allows the following conclusions to be drawn:

1. To evaluate the actual behaviour of a joint during analysis of steel and composite steel-concrete connection structures, it is importance to account for the semi-rigid performance of such connections, as this makes it possible to obtain significant savings in term of the total cost of the structure developing a better balance between the costs of labour and material (Cabrero [35]).

2. The cost of a typical connection in steel structure (ITEA, 2000 [45]) can be divided into material (20 to 40%) and labour (60 to 80%) costs. The whole cost of the connections can thus reach up to 40% of the total cost of the structure, making it important to consider the real behaviour of these connection in any analysis of the structure.

3. The use of simplified models (Frye and Morris [5]) has the main drawback that the strength and rotational stiffness of a connection thus modelled are unlikely to exactly correspond to those of the real connection.

4. Current standards make it possible to analyse, by means of the component method, the real behaviours of the connections based on a moment-rotation curve and to, thus, consider their effects on both the joint strength and the overall behaviours of the structure.

5. The behaviour of steel beam-column connections is a structural problem which is dominated by bending. At least three layers of elements are, thus, necessary to adequately reproduce the behaviour of such steel connections (Bursi and Jaspar [24]).
6. Beam-column connections with extended end plates offer semi-rigid connection without stiffeners with the highest rigidity and resistance. These are a simple connections to manufacture and to assemble, and they combines the advantages of welding in the workshop with bolting in situ. They are, thus, one of the most commonly used connections in metallic structures.

7. Experimental models are the most reliable for obtaining global information on the rotational behaviour of any connection. The main drawback of such models is their high cost, however, which causes them to be reserved for research and validation of results obtained by theoretical, analytical, mechanical, or numerical methods.

8. Finite element analysis is probably best suited for investigating the rotational response of connection, although full numerical simulation is expensive to perform. Nevertheless, it is cheaper and easier than testing experimental models, and the use of 3D elements allows the introduction of phenomena such as contact, plasticity, large deformations, and large displacements in both simple and more detailed forms.

9. Correct modelling of a bolted joint requires pre-loading of the bolts to simulate tightening. Several options are available for this, including beam elements connected by rigid elements subjected to compression stresses (Sherbourne and Bahairi, [10]), pre-stressing elements (Abolmaali et al., [26]), and thermal analysis with variations in temperature to introduce stresses equivalent to the effect of tightening (Pirmoz et al., [46]).

References

[1] Zoetemeijer, P. (1983). Summary of the research on bolted beam-to-column connections (period 1978-1983). P. Zoetemeijer, Report, (6-85).
[2] Yee, Y. L., & Melchers, R. E. (1986). Moment-rotation curves for bolted connections. Journal of Structural Engineering, 112(3), 615-635.
[3] Jaspart, J. P. (1991). Study of the semi-rigidity of beam-to-column joints and its influence on the resistance and stability of steel buildings. Liège University, PhD thesis.
[4] Faella, C. (2000). PilusoV., Rizzano G. Structural steel semi rigid connections: Theory, design and software
[5] Frye, M. J., & Morris, G. A. (1975). Analysis of flexibly connected steel frames. Canadian journal of civil engineering, 2(3), 280-291.
[6] Richard, R. M., & Abbott, B. J. (1975). Versatile elastic-plastic stress-strain formula. Journal of the Engineering Mechanics Division, 101(4), 511-515.
[7] Ang, K. M., & Morris, G. A. (1984). Analysis of three-dimensional frames with flexible beam-column connections. Canadian Journal of Civil Engineering, 11(2), 245-254.
[8] Kishi, N., & Chen, W. F. (1986). Steel Construction Data Bank Program. Structural Engineering Report No. CE-STR-86, 18.
[9] Attiogbe, E., & Morris, G. (1991). Moment–rotation functions for steel connections. *Journal of Structural Engineering, 117*(6), 1703-1718.

[10] Sherbourne, A. N., & Bahaari, M. R. (1997). Finite element prediction of end plate bolted connection behavior. I: Parametric study. *Journal of Structural Engineering, 123*(2), 157-164.

[11] Zhou, F. (2005). Model-based simulation of steel frames with endplate connections (Doctoral dissertation, University of Cincinnati).

[12] Taufik, S. (2013). Numerical Modelling of Semi-rigid Connection with High Strength Steel. *Study of Civil Engineering and Architecture (SCEA)*, 2(2).

[13] Kishi, N., Chen, W. F., Matsouka, K. G., & Nomachi, S. G. (1988). Moment-rotation relation of top and seat angle with double web angle connections, Connection in steel structures, behavior strength and design, Bjarhovde et al., eds.

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

[15] Pirmoz, A., Khoei, A. S., Mohammadrezapour, E., & Daryan, A. S. (2009). Moment–rotation behavior of bolted top–seat angle connections. *Journal of Constructional Steel Research, 65*(4), 973-984.

[16] Azizinamini, A. (1982). Monotonic response of semi-rigid steel beam to column connections (Doctoral dissertation, University of South Carolina).

[17] Komuro, M., Kishi, N., & Chen, W. F. (2004). Elasto-plastic FE analysis on moment-rotation relations of top-and seat-angle connections. *Connections in Steel Structures V, Amsterdam*, 111-120.

[18] Wales, M. W., & Rossow, E. C. (1983). Coupled moment-axial force behavior in bolted joints. *Journal of Structural Engineering, 109*(5), 1250-1266.

[19] Da Silva, L. S., & Coelho, A. G. (2001). An analytical evaluation of the response of steel joints under bending and axial force. *Computers & Structures, 79*(8), 873-881.

[20] Lemonis, M. E., & Gantes, C. J. (2009). Mechanical modeling of the nonlinear response of beam-to-column joints. *Journal of Constructional Steel Research, 65*(4), 879-890.

[21] Del Savio, A. A., Nethercot, D. A., Vellasco, P. C. G. S., Andrade, S. A. L., & Martha, L. F. (2009). Generalised component-based model for beam-to-column connections including axial versus moment interaction. *Journal of Constructional Steel Research, 65*(8-9), 1876-1895.

[22] Hu, J. W., Leon, R. T., & Park, T. (2011). Mechanical modeling of bolted T-stub connections under cyclic loads part I: stiffness modeling. *Journal of Constructional Steel Research, 67*(11), 1710-1718.

[23] Hu, J. W., Leon, R. T., & Park, T. (2012). Mechanical models for the analysis of bolted T-stub connections under cyclic loads. *Journal of Constructional Steel Research, 78*, 45-57.

[24] Bursi, O. S., & Jaspart, J. P. (1997a). Benchmarks for finite element modelling of bolted steel connections. *Journal of Constructional Steel Research, 43*(1-3), 17-42.

[25] Bursi, O. S., & Jaspart, J. P. (1997b). Calibration of a finite element model for isolated bolted end-plate steel connections. *Journal of Constructional Steel Research, 44*(3), 225-262.

[26] Abolmaali, A., Matthys, J. H., Farooqi, M., & Choi, Y. (2005). Development of moment–rotation model equations for flush end-plate connections. *Journal of Constructional Steel Research, 61*(12), 1595-1612.

[27] Yang, J. G., Murray, T. M., & Plaut, R. H. (2000). Three-dimensional finite element analysis of double angle connections under tension and shear. *Journal of Constructional Steel Research, 54*(2), 227-244.

[28] Swanson, J. A., Kokan, D. S., & Leon, R. T. (2002). Advanced finite element modeling of bolted T-stub connection components. *Journal of Constructional Steel Research, 58*(5-8), 1015-1031.

[29] Citipitioglu, A. M., Haj-Ali, R. M., & White, D. W. (2002). Refined 3D finite element modeling of partially-restrained connections including slip. *Journal of constructional steel research, 58*(5-8), 995-1013.

[30] Radziminski J Band y Azizinamini (1988). Prediction of Moment-Rotation Behavior of Semi-Rigid Beam-to-Column Connections. *Elsevier Applied Science Publishers*, London.

[31] Abolmaali, A., Matthys, J. H., Farooqi, M., & Choi, Y. (2005). Development of moment–rotation model equations for flush end-plate connections. *Journal of Constructional Steel Research, 61*(12), 1595-1612.
[32] Ramberg, W., & Osgood, W. R. (1943). Description of stress-strain curves by three parameters. Advisory Committee for Aeronautics, Technical report 902.

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

[34] Maggi, Y. I., Gonçalves, R. M., Leon, R. T., & Ribeiro, L. F. L. (2005). Parametric analysis of steel bolted end plate connections using finite element modeling. Journal of Constructional Steel Research, 61(5), 689-708.

[35] Cabrero-Ballarín, J. M. (2006). Nuevas propuestas para el diseño de pórticos y uniones semirrígidas de

[36] He, J., Yoda, T., Takaku, H., Liu, Y., Chen, A., & Iura, M. (2010). Experimental and numerical study on cyclic behaviour of steel beam-to-column joints. International Journal of Steel Structures, 10(2), 131-146.

[37] He, J., Yoda, T., Takaku, H., Liu, Y., Chen, A., & Iura, M. (2010). Experimental and numerical study on cyclic behaviour of steel beam-to-column joints. International Journal of Steel Structures, 10(2), 131-146.

[38] Wang, X. W., & Han, D. (2012). Experimental Research on Hysteric Behavior of Top-Seat Angles Beam-Column Connections. In Applied Mechanics and Materials (Vol. 166, pp. 110-113). Trans Tech Publications Ltd.

[39] Mehr, S. M. R. F., & Ghobadi, M. S. (2017). Seismic performance of retrofitted WFP connections joined to box columns using ribs. Journal of Constructional Steel Research, 137, 297-310.

[40] Kong, Z., & Kim, S. E. (2017). Moment-rotation behavior of top-and seat-angle connections with double web angles. Journal of Constructional Steel Research, 128, 428-439.

[41] Kishi N, Komuro M and Chen W F 2004 Four-parameter power model for m-θ r curves of end-plate connections Connections in Steel Structures V Amsterdam.

[42] Chen, S., Jiang, J., & Jia, L. (2018). Numerical study on the performance of beam-to-concrete-filled steel tube column joint with adapter-bracket. Advances in Structural Engineering, 21(10), 1542-1552.

[43] El Kalash, S. N., & Hantouche, E. G. (2019). Prying effect in unstiffened extended endplate connection with circular bolts configuration. Journal of Constructional Steel Research, 160, 402-410.

[44] Chen, S., Jiang, J., & Jia, L. (2018). Numerical study on the performance of beam-to-concrete-filled steel tube column joint with adapter-bracket. Advances in Structural Engineering, 21(10), 1542-1552.

[45] Hasan, M. J., Al-Deen, S., & Ashraf, M. (2019). Behaviour of top-seat double web angle connection produced from austenitic stainless steel. Journal of Constructional Steel Research, 155, 460-479.

[46] International Technology Education Association. "ITEA.(2000)." Standards for technological literacy: Content for the study of technology 19 (2002).

[47] Pirmoz, A., Daryan, A. S., Mazaheri, A., & Darbandi, H. E. (2008). Behavior of bolted angle connections subjected to combined shear force and moment. Journal of Constructional Steel Research, 64(4), 436-446.