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# Stability Consideration in Design of Steel Structures: A Review

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## Abstract

The adoption of steel in the construction industry will consistently grow due to rapid urbanisation and the demand of more structures and infrastructures. The main reasons of steel adaptation in construction industry are due to steel attributes that are flexible, sustainable, cost effective and a versatile material. The significant characteristics of steel provide the suitability for the construction of structures such as tall buildings and bridges all around the world. Along with the constant development of technology, the steel industry also aims to increase the sustainability of steel structure construction through constructing low carbon neutral and energy efficient building with steels. Moreover, steels are also considered as one of the most recycled materials in the world which allows the enhancement of the overall environmental performance of a structure's life cycle. With the increasing utilisation of steel in the design of structures, the stability consideration of the steel structures has become the most crucial concern during the structural designing phase. Stability of structures is vital for every building as the structure instability may lead to catastrophe such as structural collapse that may threatens the safety of occupants inside the building as well as the well -being of the community around the area.

## Keywords

**Steel, Stability, Beam, Frames, H-section**

## 1. Introduction

Steel is a popular material used in the field of civil engineering and construction industry. According to the World Steel Association, steels used in the construction industry for buildings and infrastructures accounts to 50% and more of the world steel demand (World Steel Association, 2020). Steel adaptation in construction industry is able to significantly reduce the concrete usage where it concurrently reduces the overall carbon dioxide emission as well as mitigate climate change through construction project (Nidheesh & Kumar, 2019). Hence, steels are recommended materials for the design of structures as it possesses great advantages in terms of its functionality as well as towards the environment.

In addition, stability of structure is also part of fundamental issues in solid and structural mechanics which are relatively important in ensuring the integrity of the structure. The stability theory plays a key role in various civil engineering fields and structures including tall structures, geotechnical structures, space frame structures and material sciences. According to the structural collapse history, it is observed that the collapse of structure is mainly caused by the misinterpretation or neglection of stability aspect during the process of structural design. Evident of collapsed structures due to instability includes the Tacoma Narrow Bridge collapsed in 1940 and the cause of instability of steel structure Matukituki Suspension Footbridge collapsed in 1977 (Arioli & Gazzola, 2017). These disastrous events have also raised interest of engineers and designers to take stability and structural fundamental concerns seriously in the future designs.

## 2. Stability of beam-columns

Instability of a steel structure are caused by several reasons which includes neglecting of stability design, human error during construction phase, degradation of steel due to fire and failures due to seismic event (Alpsten, 2017; Bravo-Haro et al., 2020; Xu et al., 2018). To avoid these issues as mentioned, the consideration of stability should be improved to ensure better structural integrity. Analysis of structural stability and integrity can be studied using the Euler's theory in accordance with the fundamental of solid and structural mechanics. Both linear and non-linear behaviour analysis should also be considered during the structural designing phase for better and more accurate design results. The structural element that will directly affect the stability of the structure consists of structural beam-columns, frames, and joints. In this paper, each structural element will be comprehensively discussed for the understanding of the consideration between the stability of structure with regards to the structural elements of beam-columns, frames, and joints.

1. A formulas of interpolation (Nidheesh & Kumar, 2019) concerning Eurocode 3, Part 1–1 clauses 6.2.9, which represents the cross-sectional resistance, and Eurocode 3, Part 1–1 clauses 6.3.3, which describes the member buckling resistance, for the moment load effect Mz,Ed =0.
2. The so-called general method of clause 6.3.4 c.

The member buckling resistance by its linear compound of two stability utilization ratio components can be determined by the interpolation formulas presented in Eurocode 3, Part 1–1 clause 6.3.3(Nidheesh & Kumar, 2019):

a) axial compression buckling resistance utilization ratio, and
b) Lateral–torsional buckling (LTB) utilization ratio for moment about the y-y axis multiplied by the interaction coefficients $kij$ for $ij=yy$ or zy that consider the nonlinear character and complexity of the behaviour of steel-beam column elements of actual structural systems

The general second-order relation produced the $kij$ coefficient and presented in (Nidheesh & Kumar, 2019) throughout two methods. Method 1 which is presented in Annex A and is considered the more accurate method, but it requires very complicated hand calculation. Whereas method 2 is presented in Annex B is considered for quick verification of the resistance. These coefficients, which are named the equivalent uniform bending moment factors, are dependent on many other parameters. The parameters are defined by a sensitivity analysis as the most critical factors that play an important role in affecting the accuracy of the design interpolation criteria. Boissonnade et al., 2004 and Greiner, 2006 have reported the degree of the verification accuracy of this kind of design methods.

The behaviour of the in-plane beam-columns is considered a crucial engineering practice especially in the design of the steel structures, such as planar portal and multi-storey frames. The behaviour of the beam-columns with ideal geometry was described by Chen and Atsuta, 2007. Moreover, Trahair et al., 2017 and Ziemian, 2010 stated the guidelines of the stability and design of imperfect segments. Columns and beam-columns are susceptible to flexural buckling (FB) especially, in the case of in-plane behaviour. Bjorhovde, 2010 was stated in his review study the development criteria of the column stability included in studies and design codes.

An experimental and numerical study of stocky beam-columns manufactured of hot-rolled steel I-sections under combined compression and bending moment was reported by Yun et al., 2018. Experimental and numerical studies of laterally restrained steel columns with variable web-tapered I cross-section were reported in Tankova et al., 2018, Cristutiu et al., 2012. The buckling behaviour of high-strength compression steel columns was experimentally investigated by Ban et al., 2018; Ban et al., 2012.

Goncalves and Camotim, 2004 conducted a study about the utilization of so-called Level 1 (Greiner, 2001) and Level 2 (Muzeau et al., 2002) beam-column interaction formulae to isolated members with arbitrary loading and end support conditions. They conducted a comparison between the analytical in-plane resistance of members and the values of finite element analyses under second-order plastic zone beside the application of initial bow imperfection and residual stress distributions. In their analysis, focus was directed to problems relating to the proper collection and decision of the equivalent moment factor used in formulas of analytical interaction.

The evaluation of the rules of safety stated in EN 1993-1-1 (Gardner and Nethercot, 2005) for flexural buckling of the columns manufactured by hot-rolled I-shaped cross-sections was illustrated by Silva et al., 2017. The assessment revealed that the imperfection factors for flexural buckling about the minor axis of steel-columns produced of S460 steel, which are described in Eurocode (Gardner and Nethercot, 2005), are unsuitable, and a new suggestion was made and recommended that the use of buckling curves were more proper and adequate. In the recent version of the EN 1993-1-1 (Gardner and Nethercot, 2005), flexural buckling curves for hot-rolled I-shaped cross-sections with height-to-width ratios $h/b_f > 1.2$ and flange thicknesses $t_f > 100$ mm are not available. Snijder et al., 2014 and Spoorenberg et al., 2014 were detailed these kinds of heavy I-shaped cross-sections which are formed by mild and high-strength steel beside the use of European buckling curves (Gardner and Nethercot, 2005). It was reported by Taras and Greiner, 2008 that the "torsional and flexural–torsional buckling phenomena of laterally restrained columns" could not be represented by the European buckling curves obtained from Gardner and Nethercot, 2005. It was formulated a new flexural–torsional buckling (FTB) curve for columns subjected to uniform compression, which is fully compatible with the background and methodology of the European column buckling curves, by Greiner and Taras, 2010.

According to EN 1993-1-1 (Gardner and Nethercot, 2005), the resistance of steel columns and beam-columns can be determined by second-order analysis considering equivalent initial bow imperfections. Checking the flexural buckling in accordance with (Gardner and Nethercot, 2005) using bow imperfections was produced by Lindner et al., 2016. Whereas it may be conservative to use the equivalent imperfections based on (Gardner and Nethercot, 2005) where this was reported by Lindner et al., 2016 and Jönsson et al., 2017. In (Chladný and Štujberová, 2013) it was proposed that by applying the Ayrton–Perry formulation, the structural analysis could be performed with equivalent column bow imperfections." It was performed by Chladny and Stujberova, 2013 the procedure by using the equivalent unusual global and local imperfection in the form of the elastic buckling mode. A design example of a planar steel multi-storey frame was considered. It was remarked that the shape of the imperfection, which is given by the higher mode, could be considered (Agüero et al., 2015). Papp, 2016 was stated a suggestion for the generalization of an overall imperfection method employing linear buckling analysis for beams, columns, and beam-columns. Lechner, 2006 reported the application of the Eurocode design methods in (Gardner and Nethercot, 2005) to determine the FB resistance of a steel planar portal frame. Effects of geometric imperfections on flexural buckling resistance of laterally braced columns can be found in (Dou & Pi, 2016).

In-plane resistance of structural steel elements can be verified with the use of both stiffness reduction (Kucukler et al., 2014, Kucukler et al., 2016) and direct strength (Taras, 2016) methods. Recently, Tankova et al., 2018 reported a novel general formulation that is based on stress utilization, with the buckling mode as the shape of the initial imperfection that can be used to check in-plane resistance of beam-columns.

The application of the general method approach proposed in (Gardner and Nethercot, 2005) relates to the out-of-plane buckling resistance. The background of this method was explained in detail by Bijlaard et al., 2009, Simoes da Silva et al., 2010, and in the ECCS design manual (Simoes da Silva et al., 2010).

Recently, research efforts for establishing design criteria for beam-column resistance have been focused on the Ayrton-Perry approach to stability problems. Tankova et al., 2017 presented a simplification of the exact solution yielded from the differential equilibrium state of crooked beam–column elements for their use in the codification of FTB resistance of beam-columns. The method proposed in (Tankova et al., 2017) is closely based on the theoretical derivation of the generalization of the Ayrton-Perry formula reported by Szalai and Papp, 2010 and referenced to LTB problems of beams and beam-columns. A complete closed-form universal Ayrton-Perry format-type solution for all types of buckling modes of steel beam-columns was derived by Szalai, 2017. Simple concepts of the Ayrton-Perry analytical formulation were developed in (Gizejowski et al., 2018) for FB of columns and in (Gizejowski et al., 2016) for FTB resistance of beam-columns. These concepts were further investigated by Gizejowski et al., 2019 and presented in two parts as a unified approach to predicting the resistance of beam-columns with regard to their in-plane and out-of-plane failure modes. As a result of Gizejowski et al., 2019 study, the in-plane interaction curve, expressed in dimensionless coordinates, that describes the beam-column in-plane flexural buckling resistance without considering lateral-torsional buckling effects, is obtained. The results of nonlinear finite element simulations are used for the verification of the developed analytical formulation. It was concluded that this proposal yields fewer conservative predictions than those based on the interaction relationships of clause 6.3.3 of Eurocode 3, Part 1–1.

# 3. Stability of frames

Frames in steel structures are commonly seen in tall buildings. Tall buildings with frames consist of various height and width that may significantly affect the stability of structures. Hence, tall buildings are the most vital structures that require the consideration of stability to avoid any structural issues in the near future. It is evident that the failure of most structures is mainly due to the frame instability with regards to the P-delta effect where the global or element of structure observes imperfection and deformations (Walport et al., 2018). P-delta analysis is crucial for structures significantly affected by wind loads, seismic events or even natural catastrophe. Structural frames should be rigid to withstand the lateral forces that will be acting on the structures to avoid structural instability. Instability of structures often occurs when the structure is close to failures which include swaying of structure frames or even buckling due to external loading.

P-delta effect is caused by the horizontal movement or loads where the second order overturning moment is generated which will lead to the deformation of structure. This deformation can be calculated by the total summation of "P" each vertical axial load multiply with the "Delta" lateral displacement of the structure to obtain the structure overturning moment. Tall buildings that possess great stiffness with regards to weight ratios which are seismically strong and well strengthen can neglect the P-Delta consideration as the changes of structural displacement is relatively low and less than 10% of the internal force in first order theory (Abhishek & Sumit, 2019). However, the imperfection of construction work and human error may not provide a perfect structure which might eventually affect the stability of structure. Hence, P-delta analysis is crucial in the design of tall steel buildings. To accurately obtain the collapse load of the structure, the P-delta effect should be also considered as the P-delta analysis provides the true moment rotation relationship of the structure over a period of time. Consideration of the relationship between the P-delta effect and the storey stability should also be studied to understand the overall structural stability. The stability assessment of steel frame structure as well as the needs of the P-delta effect consideration are also listed in the standards of steel design EN 1993-1-1 (Walport et al., 2018).

Moreover, the frame stability is also closely related to the yielding of material as well as the amount of plastic hinges formation on frames. These two factors play an important role in avoiding frame structure deterioration as well as the instability of structure. The yielding of material can be determined using tensile test for the purpose of understanding the material strength as well as the elastic modulus of the material. The stress-strain model of the material will also be generated through the tensile test technique for identifying the internal forces and its yielding point of the material prior to structural design (Gardner, 2018). In addition, the plastic hinge formation that will also significantly affect the stability of steel frame structure can be studied through the global sway behaviour model. The global sway behaviour model can be elucidated by plotting the graph of load factor against the lateral displacement (Fig. 1) swayed by the structure. Through the graph, it is observed that the increasing number of plastic hinges reduces the required loading condition of the elastic buckling load which also indicates the loss of stiffness in the steel structure frames.

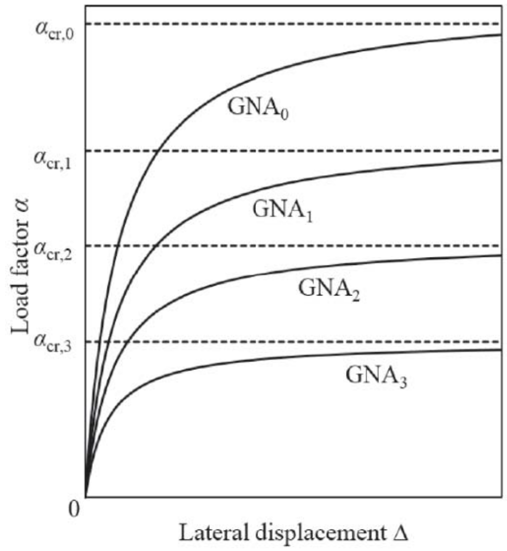


***Fig 1. Second order elastic frame response with plastic hinges (Walport et al., 2018).***

Meanwhile, as load acting on the frame structure starts deflecting, the load induces second order forces as well as moments which cause the response diverges from the linear graph. This response is also known as the second order elastic analysis as shown in the figure above. During this process, loads acting on the frame structure increases which eventually leads to the increase of frame internal forces as well as the moment until it reaches a turning point on the straight linear line where it indicates the formation of plastic hinge. Furthermore, to identify the stiffness degradation degree of structural frame more accurately and consistently, the consideration of modified elastic buckling load factor (αcr,mods) can be adopted. This consideration takes into account the stiffness reduction of the frame structure as well as its plasticity in accordance with the provided design load and its material. This technique implies the structure's first order plastic analysis where the initial stiffness, (K) relates to the secant stiffness (Ks) as shown in the figure below. The modified elastic buckling load factor (αcr,mods) can also be obtained by using the equation shown below. With this technique, the treatment of second order effect for both plastic and elastic analysis can be identified accurately as well as consistently. Hence, by determining the accurate load factor allows the engineers as well as designers to design the steel frame structure more efficient, stable, and able to withstand more lateral loading. The analysis of frame stability is fairly important and can be conducted using computational methods or advance structural software. By identifying the suitable structural material used as well as the ultimate elastic buckling load of structure for design may efficiently ensure the stability of structure and avoid any catastrophe such as failure and collapse of structure.

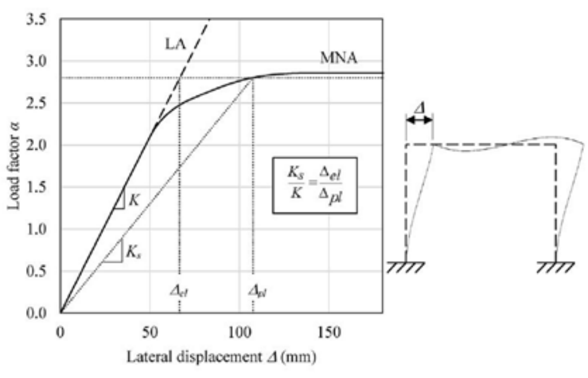


***Fig 2. Material nonlinearity results based on reduction of global sway stiffness (Gardner, 2019).***

$$\alpha cr, mods \ = 0.8\, KsK\ \alpha cr \qquad (1)$$

Subsequently, to enhance the stability of steel frame structure the adoption of bracing on structure should also be considered. The utilisation of bracing increases the structural strength which allows the structure ability to withstand more lateral load especially during seismic events. Moreover, the additional use of bracing can effectively reduce the horizontal storey drift as well as observes a significant increase in stiffness, energy dissipation and strength (Setyowulan et al., 2020). However, the drawback of eccentrically brace frame system is the connection between the ends of bracing as well as structural element should be rigid to ensure the bracing efficiency at its maximum capability. Therefore, the use of bracing should be adopted by tall steel frame structures with huge lateral drift for the structural stability enhancement.

### 3.1 Bracing systems

Bracing systems are typically currently three types, namely Moment resisting frames (MRFS), Concentric braced system (CBFS) and Eccentric braced frame (EBFS). For MRFS it is the frames with longitudinal beam and column assemblies, with beams strongly linked to the columns while CBFS is effective and economical lateral load-resistant devices in seismic regions worldwide to withstand seismic loads (Dawood, 2019). For EBFS it is to reduce the total material specifications and contribute to an intense seismic architecture feature that remains in moderate loads and ductile (Dawood, 2019).

The seismic reaction of steel frameworks is influenced primarily by the behaviour. Many experiments have been performed by various scholars who could not come up with any conclusions on which types of bracings should be used (Kumar, 2014). Comparative analyses by separate authors were obtained from the analysis of the joint by Jagadeesh, Anitha and Dhiman. The seismically reaction of the steel framework with concentrated bracing method has been tested by Jagadeesh, 2016. Two configurations were used: the irregular vertical (VIRM) and vertical irregular model with mega bracing (VIRM MB). The stainless-steel frame versions comprised of five bays with five stories with and without bracing. In addition, the analysis involved the use of ETABS (Jagadeesh, 2016). A variety of criteria were compared such as storey drift, storey displacements and base shear. Analysis of inelastic properties found that mega bracing was more successful than VIRM with no bracing frame to survive earthquakes, owing to decreased drifts, storey displaced erratic verticals were 77.64% and the highest base shear was 23.42% (Jagadeesh, 2016).

A distinction has been made between the seismic impact on knee braced steel frame and other forms of bracings by Anitha & Divya, 2015. The modelling was conducted with the assistance of finite element programme, ANSYS 14.5, under non-linear time history analysis and non-linear static analysis. For non-linear static studies, dynamic loading of 10kN was allocated and El-Centro earthquake data were assigned for the study of time history (Anitha & Divya, 2015). For the required seismic refitting process, knee bracing can be used. Double knee bracings demonstrated outstanding results during seismic activity in non-linear static study, as ultimate loads were very strong and lateral rigidity. The overall displacement observed during historical research was 90.5% more than without bracing and 50% greater than the eccentric bracing system (Anitha & Divya, 2015).

The response from braced and unbraced systems subject to seismic loads was evaluated by Dhiman et al., 2015. Different types of braces were used for dead and live loads such as cross brace, braced chevron, diagonal k-braced and dead load, X and Z seismic load respective (Dhiman et al., 2015). Structure displacement decreased as the bracing system was applied and lateral displacement was maximally diminished when the cross-bracing system had been used. The inclination and shear forces in columns were also diminished (Dhiman et al., 2015).

## 4. STABILITY OF JOINTS

Connections or joints shall be used for the transition to other parts of the structure or supports of the forces provided by a structural member. The braces and other members that provide the structural component with constraints are also used as connection. Eurocode 3 have different meaning for the connections and joints where connection consists of fasteners such as bolts, pins, rivets, or welds, fastener that is connected to the local member elements and might consist of external plates or cleats (Trahair et al., 2017). While joint consists of the region in which the members are linked, and involves the connection, along with the portions of the member or members required to facilitate the transition of action at the joint. The structure of a joint is normally selected to suit the type of motion (force or moment) that is being transmitted and to link the variety of member (tension or compression member, beam, or beam-column). The structure should be preferred to prevent unnecessary costs because it is typically time-consuming to plan, detail, construct and assemble a joint because it could cause increase in cost and impact. Such that, a heavier part is often best used instead of stiffeners because this decreases the amount of manufacturing processes needed (Trahair et al., 2017). A joint is first constructed by defining transfers of force from the member to the other part of the system via the components of the joint. Each part is then proportioned such that the force to be conveyed can be resisted adequately.

### 4.1 Stability Issues Between Joint and Steel Structure

Steel moment-resisting frames become a common seismic-resistant method because steel is a well-established with high strength to mass ratio with the ductile material. Brittle fracture of connection is not the cause of damage but associated with local buckling and yielding (Kumar, 2014). The development of plastic bends in beam-column joints and column bases is one of the characteristics of an inelastic action that resists the frame for steel moment (Kalyana Chakravarthy et al., 2018). The connections between cyclic loading and failure modes have been varied in several experimental works but show mainly ductile behaviour based on several mechanisms, characterised by bolt slip, yielding of steel, elongation of bolt holes, etc.

The use of bolted connections between the dissipative areas and the remaining structures to replace the damage incurred by dissipating elements on eccentrically braced frames will minimise lateral movement. An additional brace at the end of the RBS will necessarily decrease beam flanges and column twist transverse rotation, which can induce to cyclic degradation (Kalyana Chakravarthy et al., 2018).

The analysis of braced system subjected to join shows that the bracings minimise floor lateral movement, in columns, axial forces rise from unbraced to brace, less horizontal movement in cross-bracing than diagonal bracing, cross braced stories would have a higher peak than unbraced and diagonal braced frames, shear forces decline in the column from unbraced to braced. shear strength is stronger in diagonal braced columns than in cross-braced, the cross bracing is subject to more simple shear than diagonal bracing and the steel brace frame is with more base shear than unbraced frames (Coelho and Bijlaard, 2010). Moreover, the structure with higher heights or more stories would have baser shear under the same bracing system and loading than the smaller one and the bending moment decreases in column from unbraced to brace. The braced diagonal column is more twisting than the braced diagonal column.

When transmitting force by in-plan motion, plates are relatively strong and rigid but relatively weak and flexible when the force is transmitted by external bending. The angle cleat and the seat are therefore flexible and permit the attachment members to rotate comparatively, while flanges and web stiffeners are rigid and limit their relative rotation. The simplicity and comparative rigidity of the welded connections have also contributed to the failure of stiffening plates when it is not needed for force purposes. Thus, by welding the beam directly to the column flanges and by excluding the web stiffeners column, the rigid connection can be greatly simplified (Shin & Park, 2014). This neglect would, however, make the connection much more versatile because local column flange and web distortions are no longer preventable.

## 4.2 Beam-to-Column Connection

For the beam- to- column connection, the failure mode takes different forms because of the diversity within connection type configurations such as fracture around welds, fractures in highly tight fracture material, welding access holes fractures in the net section of bolts, bolt failure due to shearing and tensile, bolt bearing and bolt shear failures.

## 4.3 Column Splices

Column splice at the failure modes is similar to the connection of beam to column modes. In the case of column splices failure, the bending and tension resistance is not only reduced or eliminated, but also the shear force transfer column is reduced or removed.

## 4.4 Column Bases

Column bases mode of failure depends on the column-foundation connection. They involve anchorage or pulling off, fracturing in plates of the base or in connections between columns and base plate, and extreme local and lateral torsional buckling if the area above the base connection has inelastic deformations localised.

## 4.5 Connections in Braced Frame

A Braced Frame is a structural structure designed mainly to withstand the effects of wind and earthquake. Members of the braced frame act like a truss system where it is designed to function in stress and compression. The braced frames consist of steel components only. The benefit of the bracing system is that it is ideal for all kinds of systems, such as bridges, ships, grids, homes, and electric transmission towers, simple to install with lack of experience or know-how, and no difficulties in linking the connections if the bolted connections are used.

Bracing connections entail bolting of flat, angle, channel, I-section, and hollow section members to a gusset plate under which the column or other members are supported. The bracing component can operate either in equilibrium or in tension and compression alone and stabilise the main components by load distribution. The choice of a bracing configuration depends on a variety of considerations. This includes the proportions of height to width of the bay and the scale and position of the required open areas in the structure height. These requirements will substitute design parameters with structural optimisation (Kalyana Chakravarthy et al., 2018). The implementation of the e/L parameter leads to a widespread framing device definition.

## 4.6 Bolted Double Angle Cleats

Typical dual-angle bolted cleat links all along the column's main and minor axis. Any basic study of equilibrium can be used when constructing such a relation. The course of motion that the shear transition between the beam and the column is recommended for this publication suggests that the column is on its face (Soltani & Kerdal, 2011). The bolt group that attaches the cleats to the beam web must be optimised for the shear force of the end shear product and for the time generated by the column face to the eccentricities of the bolt group. For the applied shear alone, the connections linking the cleats of the bolt to the column face should be designed. The cleats to the column are never crucial in operation, and the arrangement is almost always guided by the bolt bearing to the network of the beam (Soltani & Kerdal, 2011). The rotary capability of this relation is primarily determined by the angles and slip distortion capacity between the attached components. The rotation of the links is mostly due to angle deformation, while the connection deformation is very minimal.

## 4.7 Fin plates

The implementation of the fin plate relation was a more modern invention that follows Australian and American standard. This form of link is used mainly for beam end reactions and is cost efficient in construction and easy to install (Kalyana Chakravarthy et al., 2018). It is necessary to identify the proper course of action for the shears when designing a fin plate connection. The shear operates on the column face, or the shear works alongside the middle of the bolt group and links the end plate to a beam web (Shin & Park, 2014). For this purpose, the vertical shear product and distance between the face of the column (or a beam web) and the middle of the bolt group should be tested at a minimum moment. The resulting moment is regulated in accordance with the vertical shear for each of the critical sections. The fin plate is welds to their maximum strength due to the unknown moment added to the fin plate (Coelho & Bijlaard, 2010). Fin plate connections derive them in-plane rotational capacity is determined by bolt distortion in shear, distortion of the bearing bolt holes, and the out-of-plane bending of the fin plate. Be aware of the possibility failing in lateral torsional due to buckle fin plates with long projection. In the construction protocols for fin plate connections, extra monitoring for this activity is used.

# 5. CONCLUSIONS

A three-dimensional frame structure may be analysed as the group of a number of independent two-dimensional frames, while individual members are usually considered as one-dimensional and the joints as points. It can be believed that the joints are frictionless hinges or that they are semi-rigid or rigid. In some cases, comparisons are made on an idealized model which approximates part, or all of the structure may substitute or complement the analysis. For beams and columns, structural steel members can be one-dimensional (whose lengths are far wider than their transverse dimensions) or two-dimensional as for frames (whose lengths and widths are much greater than their thicknesses). Thus, according to the mechanism by which they distribute the forces in the structure, structural members may be categorized as stress or compression members, beams, beam-columns, torsion members, or plates. A person member's real behaviour will depend on the powers working on it. Thus, before their material non-linearity becomes essential, tension members, laterally supported beams, and torsion members remain linear, until they reach the maximum plastic state. However, geometric non-linearity is demonstrated by compression members and laterally unsupported beams as they reach the buckling loads. Beam-columns are members that relay transverse and axial loads, such that both material and geometric non-linearity are reflected. The structural steel members can also be joined together in a range of ways at joints, and by using a variety of connectors. This includes pins, rivets, bolts, and welds. Beam-columns are structural members which combine the beam function of transmitting transverse forces or moments with the function of transmitting axial forces of the compression (or tension) component. In skeletal arrangements, structural frames are composed of one-dimensional members joined together that transfer the applied loads to the supports. Thus, the function of beam-column, joint and frame in column plays important rule for the stability of the structure and maintaining the stability of the material is important to provide strong and durable steel structure.

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