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Title

Parametric Study on the Beam-to-Column Connections with Top and Seat Angles

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Dedication

This thesis is dedicated to:

My great parents ,who never stop giving of themselves in countless ways

My dear wife

My beloved brothers and sisters who encourage and support me

All my family and My friends

HADJ SMAIL BOUKRAA

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LIST OF FIGURES

Figure I-1: Simple beam-to-column connections ……................................……………..…..…4

Figure I-2: Rigid beam-to-column connections (a) Short end plate (b) Extended end plate (c) Hunched …………...............................………………………………………...4

Figure I-3 : Semi-rigid beam-to-column connections.....................…………………………..…..5

Figure I-4 : Major-axis joint configuration ………........…..…………………….……………..6

Figure I -5 : Beam splice ……………..……6

Figure I-6 : Column splice ………..……7

Figure I-7 : Column base ……………………………….......................................…..…8

Figure I-8 : Bolted end-plate beam-to-column connection ……………….........................…...8

Figure I-9: beam-to-column connections with angles …..9

Figure I-10 : The Eurocode 3 classification by moment resistance…………………………..…10

Figure I-11 : Boundaries for stiffness classification of beam-to-column joints………..……10

Figure. I.12 Classification of connections based on rotation capacity................................12

Figure I-13 : Non-linear (M- φ) curve ……………..…13

Figure I-14: Possibilities for curve idealisation …………....………………………………......13

Figure II-1. Connection Parameters ………………………..........................……………...20

Figure II-2 : Components of end plate beam to column connection....................………...…23

Figure II-3: Different Components of connection ....…...24

Figure II-4: Spring model for a composite flush end-plate connection …........................……27

Figure II-5 : Typical top and seat angle connection………………………...........…………31

Figure II-6 : Deflected configuration at elastic condition ……………..................................31

Figure II-7 : The influence of the bolts location to initial stiffness ………..........………34

Figure II-8 : The influence of the angle thickness (ta) to initial stiffness …......................…..35

Figure II-9 : Initial stiffness for each beam section …..…37

Figure II-10 : Initial stiffness boundary for each beam section……….............................…38

Figure II-11 : Top and seat with double web angle ……………........................…………39

Figure II-12 : Top and seat angle connection ………………............................………….40

Figure II-13 : Double web angles connection …………………….............................……41

Figure II-14 : The M- θ behaviour ……………..…...……43

Figure III-1 Portal frame with one floor and a single span ……………...……………………45

Figure III-2 Autodesk Robot interface ……………………..……..............................…..…45

Figure III-3: Different section profiles 46

Figure III-4: Window to choose the dimensions of frame ………….........................……46

Figure III-5: 3D frame view ……………………………..….............................………..47

Figure III-6: Defining the load types…………………...…47

Figure III-7: Defining the vertical and the horizontal loads ……………..........…………...…47

Figure III-8: Verification of the stability of the frame ……………………..........………..…..48

Figure III-9 : Defining beam to column connections ………..............................…...…...48

Figure III-10 : Defining the geometrical parameters of the double flange connection …...….49

Figure III-11. : Flange connection view …...…49

Figure III-12 : Vertical and horizontal loadings...50

Figure III-13: Diagram of the bending moment without configuration of the connections 51

Figure III-14: Flange connection details …………...…......…52

Figure III-15. Diagram of the bending moment corresponding to (e0 = 40mm)…………….53

Figure III-16: Bending moment diagram corresponding to (e0 = 50mm)…...................….…53

Figure III-17: Diagram of the bending moment corresponding to (e0 = 60mm).................…54

Figure III-18. Diagram of the bending moment corresponding to (t_a = 8mm)……………......55

Figure III-19: Diagram of the bending moment corresponding to (t_a = 10mm)….................56

Figure III-20 Diagram of the bending moment corresponding to (t_a = 12mm) …………….56

List of Tables:

Tableau I-1: Type of joint model...7

Tableau I-2: Basic mechanical properties of structural bolts..13

Table II.1 : Mathematical formulations..14

Table II.2 : curve-fitting constants and standardization constants for Frye-Morris polynomial model (all size parameters are in centimeters)..18.

Table II.3 : Values of the coefficient ψ 23

Table II.4 : Common initial stiffness of semi-rigid connections 27

Table II.5 : Values of the initial stiffness corresponding to (e0) 30

Table II.6 : initial stiffness by changing the value of (ta) 32

Table II.7 : Geometrical parameters of the Beam 33

Table II.8: Different beam sections used in the analysis..33

Table II.9: different beam initial stiffness boundaries 34

Table II.10 initial stiffness of the connection by varying beam sections :............................34

Table II.11 : connection parameters...36

Table II.12 : Table of result for (TSDWA)..37

Table II.13 : (TSA) connection parameters 38

Table II.14 : Table of result for (TSA)...38

Table II.15 : (DWA) connection parameters 39

Table II.16 : Table of results for (DWA)...39

Table III. 1 . Values of the initial stiffness corresponding to (e0) :.......................................52

Table III. 2 . Values of the initial stiffness corresponding ta :...55

Notations and symbols

A_b : Area of the section of the metal beam.

b_c : Width of the metal column section .

b_b : Width of the metal beam section .

C_j , D_k : curve-fitting parameters

C_1, C_2, C_3 : curve fitting constants

d : the height of a column .

E : Young's modulus

e : Bolt pitch

h_1 : is the total of the height of beam and the thickness of the angle

K : standardization constant

L_b : the length of a beam

$_{Ie}$: Length of the angle

M_b : Bending moment

$M_{pl.Rd}$: plastic moment resistance of the connected beam

$M_{j,Rd}$: moment resistance

M_0: the initial connection moment

M_{u} : is the ultimate moment

R_{ki} : is the initial stiffness of the connections

$S_{j,ini}$: initial rotational stiffness

t_p : The end plate thickness

t_T : the thickness of Tee-stub

t_{wa} : Thickness of web-angle

(φ_{cd}) : the rotation capacity

α : a scaling factor

θ_k : the initial rotation

Summary

General Introduction...1

CHAPTER I : Generalities on Steel Structure Connections

I.1 INTRODUCTION...3

I.2. Types of Connections...3

I.2.1. Simple connections...3

I.2.2. Rigid connections..4

I.2.3. Semi rigid connections...5

I.3. Different types of connections in steel structure...6

I.3.1. Beam and column splices..6

I.3.2. Column base..7

I.3.3. Beam-to-column connection..8

I.4. Classification of Connections..9

I.4.1. Classification by strength...10

I.4.2. Classification by stiffness...10

I.4.3. Classification by rotation capacity..11

I.5. Idealisation..12

I.6. Assembly mode...14

I.6.1. Bolted connections..14

I.6.1.1. Unfinished bolts..14

I.6.1.2. High strength bolts..14

I.7. Conclusion...15

CHAPTER II : Evaluation of Initial Stiffness and Rotation Capacity

II-1. INTRODUCTION..16

II-2. Mathematical formulations (empirical models)..17

II-2-1. Exponential Model...18

II-2-2. Frye- Morris Polynomial Model...19

II-3. Component method...22

II-3-1. Basic components of an end plate joint..23

II-3-2. Rotational stiffness..25

II-3-3 Initial stiffness Sj,ini...27

II-3-4. Connections with more than one layer of components in tension........27

II-4 . Evaluation of the initial stiffness of semi rigid connections..............................30

II-5 . Parametric Study...31

II-5-1 .The initial stiffness of semi rigid connections......................................31

II-5-1-1 Influence of the bolt location eo..33

II-5-I-2. Influence of the Angle Thickness ta.......................................34

II-5-2 . Parametric study for Frye- Morris polynomial Model.........................39

II-6 . Conclusion...43

CHAPTER III : Parametric study on a flange beam-to-column connection

III-1 .Introducton 44

III-2 . Description of Autodesk Robot Structural Analysis Professional 44

III-3 . Analysis of semi-rigid connections (flange connection)................................44

III-3-1 . Description of the frame 44

III-3-2 . Design of the frame with Autodesk Robot Structural Analysis 45

III-3-3 . Analysis of a rigid structure without assembly configuration :(rigid by default)...50

III-3-4 . Analysis of the structure with connection configuration 51

III-3-4-1 . Influence of the bolt location e_o 52

III-3-4-2 . Influence of the angle thickness t_0 54

III-3-5 . Results and discussion...57

III-4 . Conclusion 57

General conclusions 58

General Introduction

General Introduction :

One of the most vital parts in steel structures concerning its strength is the connection between members. A steel structure can only perform its best in sustaining load when the connections between members are designed adequately.

Many researchers show that it is undesirable to design the frame as rigid or pinned connection, as it will lead to an uneconomical design. Experimental and analytical results show that there is a variation of internal force transfer from beam to column which depend upon the bending stiffness of the joint, thickness and location of bolts etc...

These parameters affect the behavior of the frame and decrease the negative end moment and increase mid-span moment on the beam. Hence, researchers have concluded that the semi-rigid connection should be taken into consideration for analyses and design of steel frames

The connections in steel structures are generally carried out by welding and / or bolting. Angles connections are widely used in steel structures . However, The structural behavior of this type of connection is extremely complex to analyze . This complexity is due to the variation of their geometrical and materials properties which leads to a difficult behavior to predict .

For the realization of our thesis , we divided the work into three chapters in addition to a general introduction and general conclusions :

First chapter :

In the first chapter generalities on connections in steel structures are presented. The classification of metal assemblies in terms of stiffness and strength, the moment-rotational relationship of rigid, semi-rigid and pinned assemblies, are discussed in this chapter. Then the characteristics of the assembly behavior , as well as the classification of steel connections are also discussed. In this classification, we present the connections according to rigidity, resistance and rotational capacity.

Second chapter :

Many researchers have developed mathematical formulation of connection behaviors taking into account different elements considered in the connections, some mathematical models used to describe the non-linear behavior of some kinds of connections found in practice are shown in this chapter, as well as a parametrical study for Frye- Morris polynomial Model and the evaluation of an approximate initial stiffness of a flange connection are presented in this section .

Third chapter :

In the last chapter we are interested in the modeling of steel frame with flange connection (top and seat angle), based on the European norm (Eurocode 3), applicable on steel structures which are composed of beam and column elements and interconnected by bolted connections, the effect of varying different geometrical parameters of a same flange assembly on the level of performance and on the frame responses is analysed in this section . This study is carried out by using Autodesk Robot calculation software.

CHAPTER I :

Generalities on Steel Structure Connections

I.1 INTRODUCTION:

Steel structures are constructed by properly connecting the available standard sections, so the connections are an important part of steel structure and are designed more conventionally than any individual members.

In this chapter generalities on connections in steel structures are presented. First, different types of connections in this type of construction are defined, and then the characteristics of the assembly behaviour , as well as the classification of steel connections are discussed. In this classification, we present the connections according to rigidity, resistance and rotational capacity.

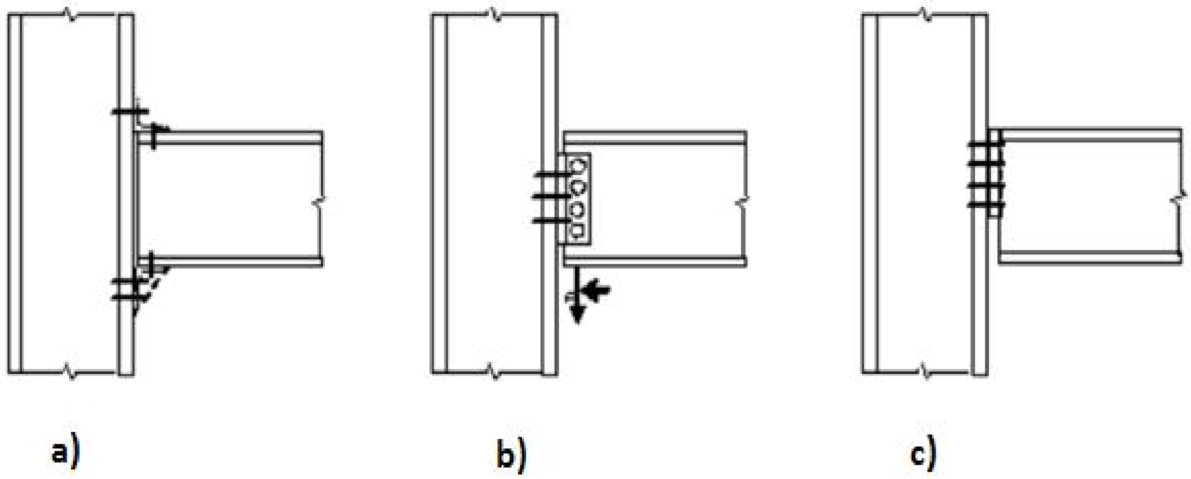
I.2. Types of Connections:

Connections can be classified by rigidity and are associated closely to the frame design methods. Various types of connections such as simple, rigid and semi-rigid connections are discussed below. [12]

I.2.1. Simple connections: [15]

Simple connections are assumed to transfer shear force only at some nominal eccentricity. Therefore such connections can be used only in non-sway frames where the lateral loads are resisted by some alternative arrangement such as bracings or shear walls.

Simple connections are typically used in frames up to about five storey in height, where strength rather than stiffness govern the design. Some typical details adopted for simple connections are shown in **Fig. I.1**



(a) Clip and seating angle (b) Web cleats (c) Curtailed end plate

Fig. I.1. Simple beam-to-column connections

I.2.2. Rigid connections:

Rigid connections transfer significant moments to the columns and are assumed to undergo negligible deformations.

Rigid connections are necessary in sway frames for stability and also contribute in resisting lateral loads. In high-rise and slender structures, stiffness requirements may need the use of rigid connections. Examples of rigid connections are shown in **Fig. I.2.**

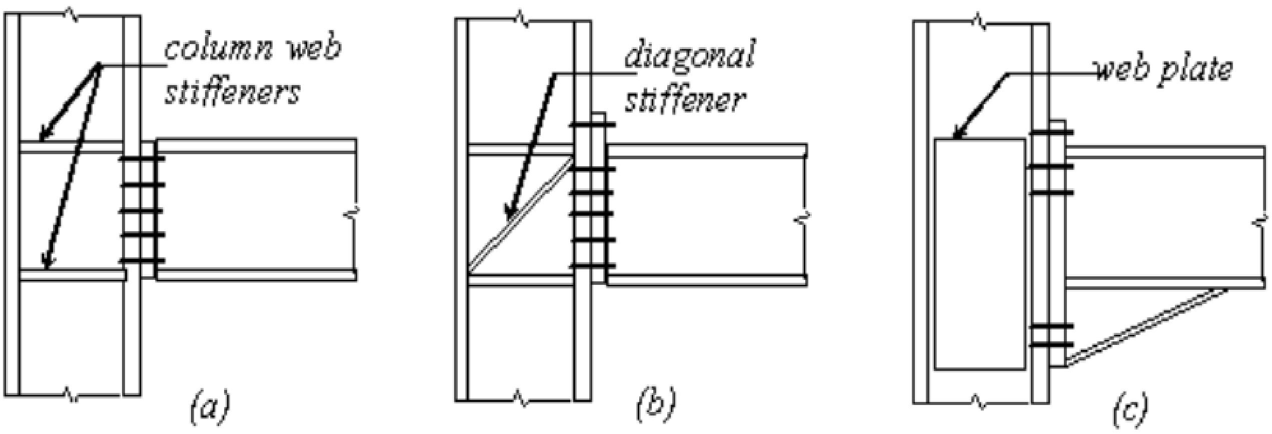


Fig. I.2. Rigid beam-to-column connections (a) Short end plate (b) Extended end plate (c) Hunched

I.2.3. Semi rigid connections :

Semi-rigid connections are those falling between simple and rigid connections. The fact that most simple connections do have some degree of rotational rigidity was recognised and efforts to utilise it led to the development of the semi-rigid connections. [15]

The use of semi-rigid connections makes the analysis somewhat difficult but leads to economy in member design. In order to analyze semi-rigid connection behaviour accurately, the moment rotation curves provide the means of realizing the capacity provided by the connection and the production of these curves have the utmost importance. For instance, top and seat angles and double web angles (TSADWA) connection types have higher moment capacity when compared with double web angles (DWA) connections. As a general rule of thumb, a stiffer connection usually has a higher moment capacity.

Examples of semi-rigid connections are shown in **Fig.1.3** .

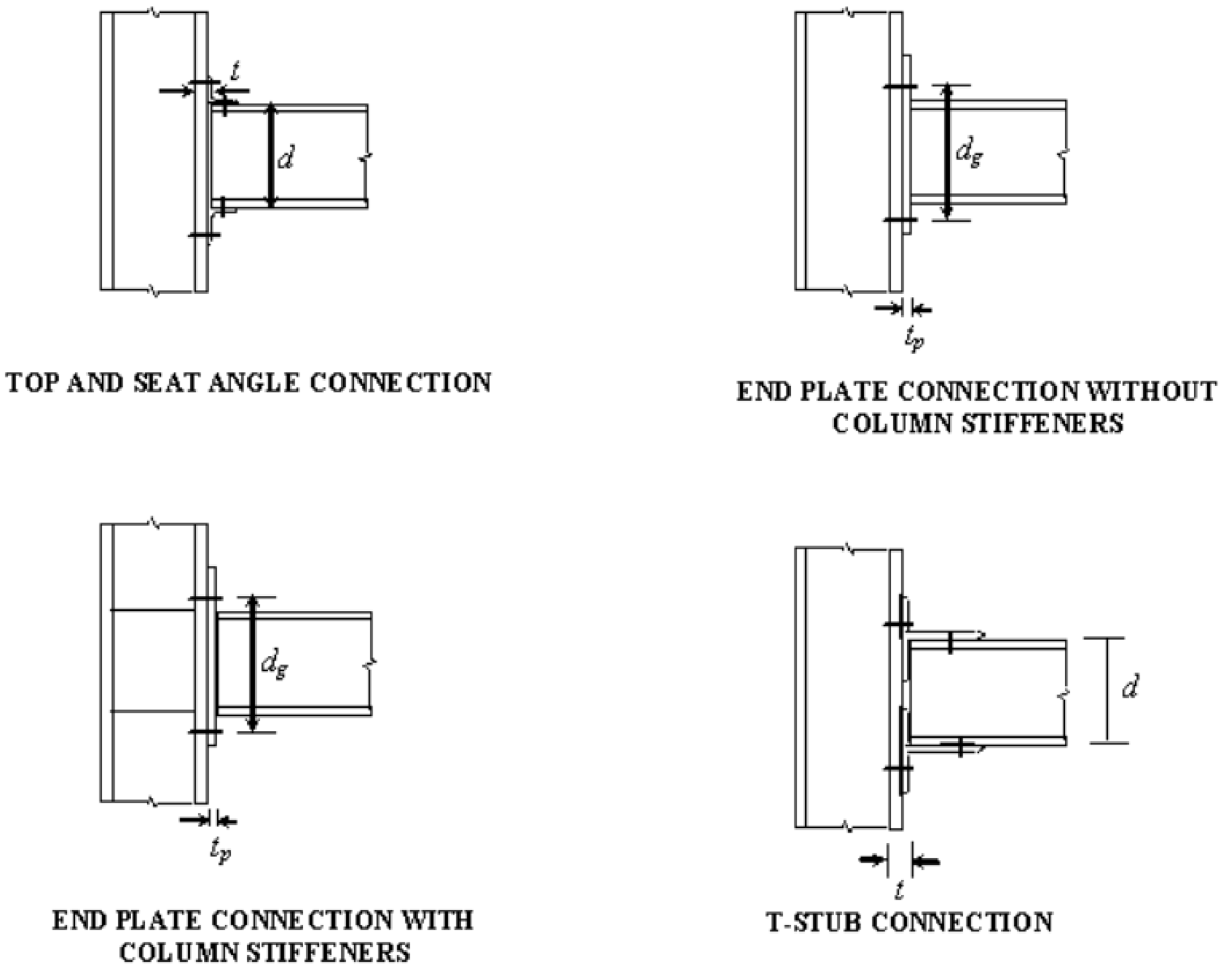


Fig. I.3. Semi-rigid beam-to-column connections

I.3. Different types of connections in steel structure:

Different connection types within a structure that can be encountred in practice are shown in figure I.4 below :

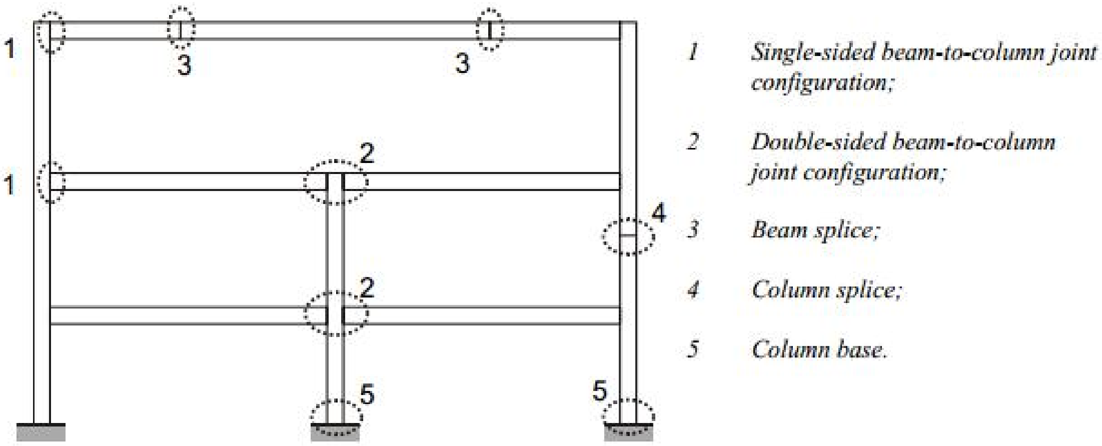


Fig. I.4. Major-axis joint configuration [15]

I.3.1. Beam and column splices :

It is often required to join structural members along their length due to the available length of sections being limited and also due to transportation and erection constraints. Such joints are called splices.

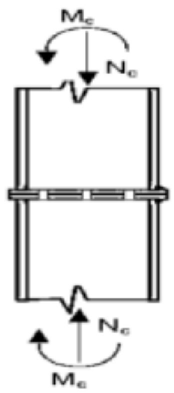
Splices have to be designed so as to transmit all the member forces and at the same time provide sufficient stiffness and ease in erection.

Splices are usually located away from critical sections, and the requirements of the code should be satisfied.

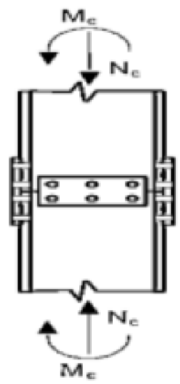
a) end plated b) Bolted cover plates

Fig. I.5 Beam splice

Figures I.5 and I.6 show beam and column connections respectively using end plates and splices.



a) end plated



b) Bolted cover plates

Fig. I.6 Column splice .

I.3.2. Column base : [6]

A column base consists of a column, a base plate and an anchoring assembly. In general they are designed with unstiffened base plates, but stiffened base plates may be used where the connection is required to transfer high bending moments.

The column base is usually supported by either a concrete slab or a sub-structure (e.g.a piled foundation).

Figure I.7 show a column base connected to a concrete foundation.

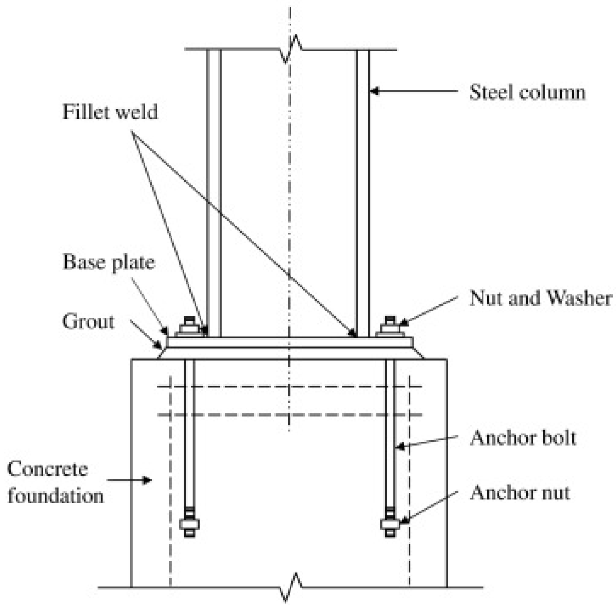
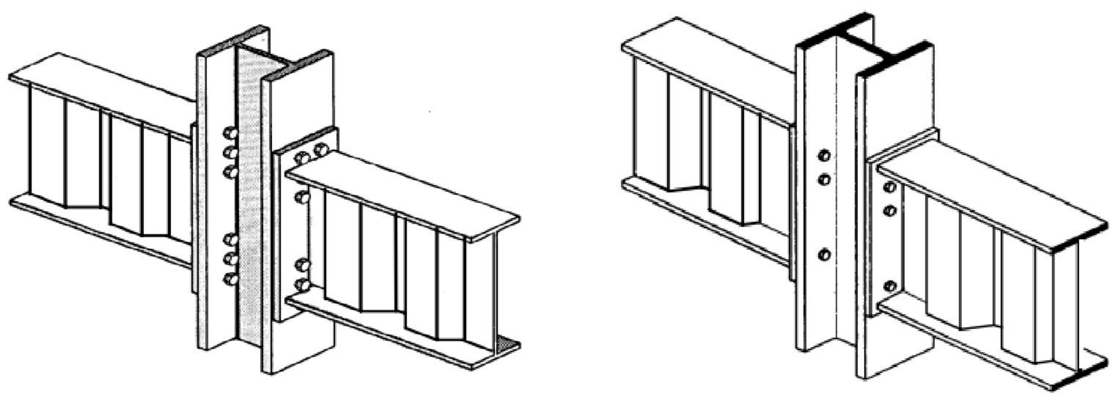


Fig. I.7 Column base

I.3.3. Beam-to-column connection :

Beam-to-column connections can be classified as rigid, semi-rigid or flexible depending on the amount of moment transfer taking place between the beam and the column: [3]. Beams connected to columns by means of end plates are described in figure I.8. Double web angles, top and seat angles and top and seat with double web angles (Figure I.9) can be used as beam to column connections.

a) End plate connections:

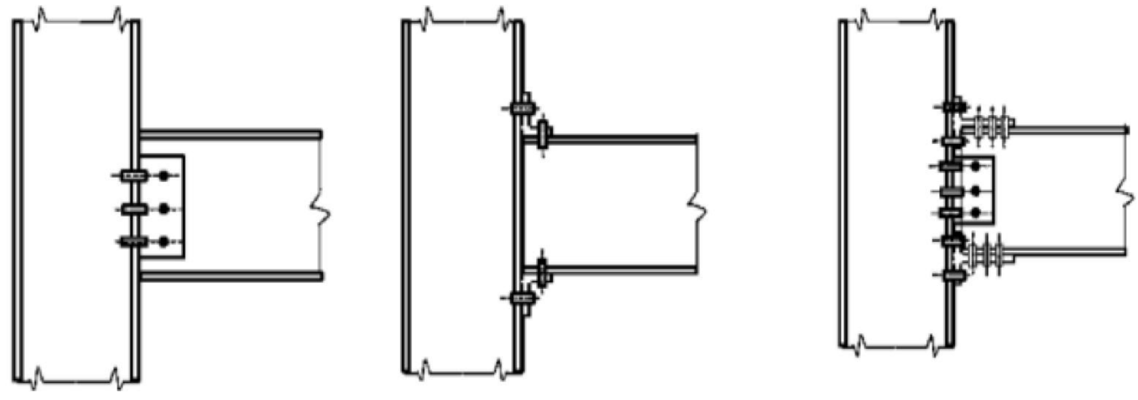


a) Extended end plate connection

b) Flush end plate connection

Fig. I.8 Bolted end-plate beam-to-column connection. [12]

b) Beam-to-column connections with angles :



a) double web angles b) top and seat angles c) top and seat with double web angles

Fig. I.9 beam-to-column connections with angles [10]

I.4. Classification of Connections:

In practice, it is very important to be able to recognise the category of connections whether it is rigid, pinned (flexible) or semi-rigid [12] , In EN 1993-1-8 this is provided by a classification system based on joint resistance and stiffness.

Table I.1 shows the types of joint model (representing behaviour), the classification of joints and the method of global analysis. [7]

Table I.1 : Type of joint model

Method of global analysis	Classification of joint		
Elastic	Nominally pinned	Rigid	Semi-rigid
Rigid-Plastic	Nominally pinned	Full-strength	Partial-strength
Elastic-Plastic	Nominally pinned	Rigid and full-strength	Semi-rigid and partial -strength Semi-rigid and full -strength Rigid and partial-strength
Type of joint model	Simple	Continuous	Semi-Continuous

I.4.1. Classification by strength :

A beam-to-column joint may be classified as full-strength, nominally pinned or partial strength. The Eurocode 3 classification of joints with respect to its design moment resistance is shown in (Fig. I.10). While the boundary between full-strength and partial-strength is well defined by the design plastic moment resistance of the connected beam ($M_{pl.Rd}$) , the boundary between partial-strength and pinned ($0.25M_{pl.Rd}$) is polemical. [9]

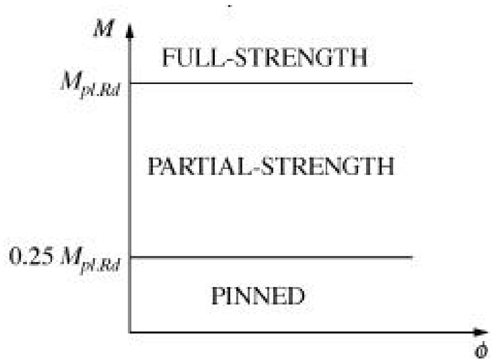


Fig. I.10 The Eurocode 3 classification by moment resistance.

I.4.2. Classification by stiffness:

A joint may be classified as rigid, nominally pinned or semi-rigid according to its rotational stiffness, by comparing its initial rotational stiffness ($S_{j,ini}$) with the classification boundaries given in Figure. I.11 :

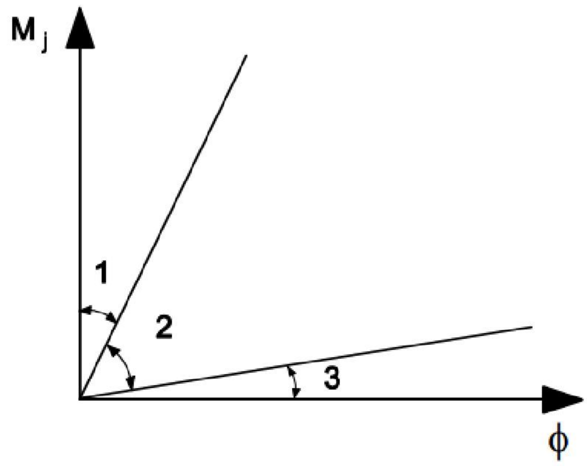


Fig. I.11 Boundaries for stiffness classification of beam-to-column joints

Zone 1:

$$\text{Rigid if } S_{j,ini} \geq \begin{cases} 25EI_b/L_b & \text{Unbraced Frames .} \\ 8\,EI_b/L_b & \text{Braced frames .} \end{cases}$$

Zone 2:

Semi-rigid: All joints in zone 2 should be classified as semi-rigid.

Zone 3:

Nominally pinned , if : $S_{j,ini} \leq 0.5\ EL_b/L_b$

I.4.3. Classification by rotation capacity : [5]

Three basic parameters describe the behaviour of connections: strength, stiffness and ductility. In moment resistant connections, the ductility is achieved by a sufficient rotation capacity. Although there do exist well-elaborated methods for determination of the initial stiffness and strength of beam-to-column joints, there are no generally accepted procedures for the determination of the rotation capacity. Indicatively it may be said that the relevant Eurocode, prEN 1993-1-8 devotes only one out of 75 pages to the determination of the rotation capacity.

The estimation of the rotation capacity is very important in many applications, as e.g. when partial-strength connections are used under seismic conditions or plastically designed frames. It is therefore evident that a simple method for the determination of the rotation capacity for everyday design applications is needed.

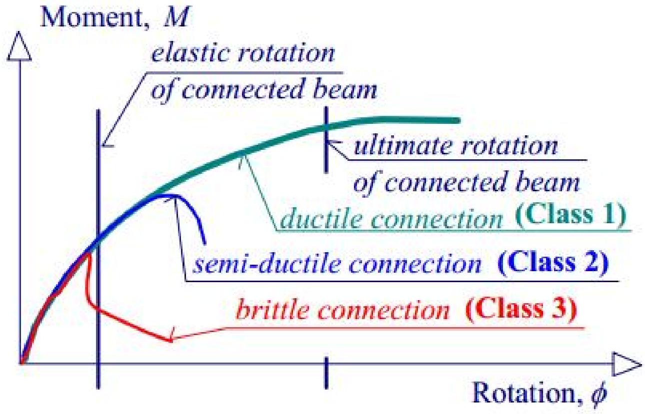


Figure. I.12 Classification of connections based on rotation capacity [6]

Figure I.12 shows different classes of connections based on their rotation capacities.

I.5. Idealisation: [7]

In EN 1993-1-8 , it is considered that full non-linear (M-φ) curve consists of three parts, as shown in **Fig. I.13** . Up to a level of **2/3** of the design moment resistance ($M_{j,Rd}$), the curve is assumed to be linear elastic. The corresponding stiffness is the so-called initial stiffness ($S_{j,ini}$) . Between 2/3 ($M_{j,Rd}$) and ($M_{j,Rd}$) the curve is non-linear. After the moment in the joint reaches ($M_{j,Rd}$), a yield plateau appears. The end of this (M-φ) curve indicates the rotation capacity (φ_{cd}) of the joint.

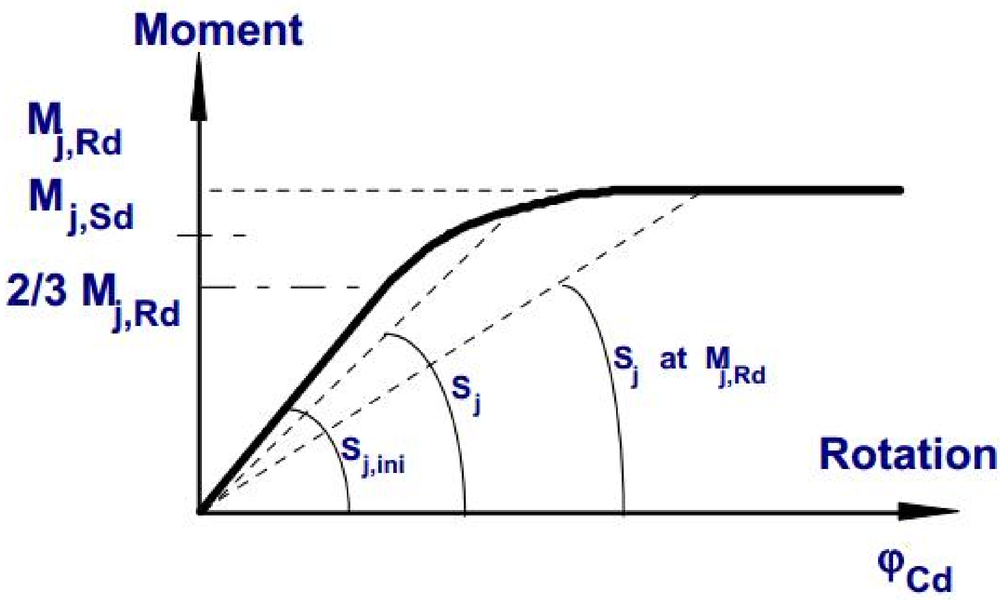


Fig. I.13 Non-linear (M- φ) curve

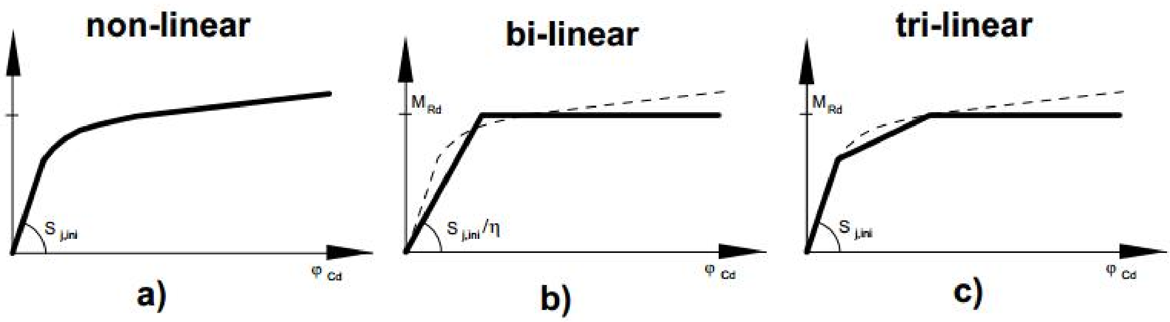


Fig. I.14 Possibilities for curve idealisation

The actual moment-rotation response of joints usually is described by means of a non-linear curve, see Figure. I.14 a. However, the use of such non–linear curves requires sophisticated frame analysis programs.

In order to enable a (more simple) linear calculation, e.g. an elastic global frame analysis , the nonlinear curve may be simplified by straight lines. As a conservative assumption each curve lying below the non-linear curve in general may be used in the frame analysis.

For example a tri-linear curve is shown in (**Fig. I.14** c). The most simple curve is a bi-linear curve (**Fig. I.14** b). The use of a bi-linear curve provides the most efficient solutions. [7]

I.6. Assembly mode :

Connections are used to transfer the forces from one member to another. Although both welded and bolted connections can be used in steel structures .

I.6.1. Bolted connections : [6]

Bolted connections are commonly used because of the ease of fabrication, erection and ability to accommodate minor site adjustments.

In the different types of bolted connections including cover plates, end plates and cleats, the bolts are used to mechanically fasten the steel elements.

The performance of a bolted connection is complicated and both the stress distribution in the connection and the forces in the bolts are dependent on the stiffness of the bolts, and the connecting steel elements (end plates, cleats, etc.).

I.6.1.1. Unfinished bolts : (4.6 and 5.6)

Unfinished bolts are also called ordinary, common, rough or black bolts. There are used for light structures (purlins , bracings, etc.) under static loads. They are not recommended for connections subjected to impact loads, vibrations and fatigue.

Bolts are forged from low carbon rolled steel circular rods, permitting large tolerances.

I.6.1.2. High strength bolts : (8.8 and 10.9) :

They are used for high strength (connections..) :

- Provide a rigid joint. There is no slip between the elements connected.

- Because of the clamping action, load is transmitted by friction only and the bolts are not subjected to shear and bearing.

- For some strength, lesser number of bolts are required as compared to rivets which brings overall economy. Table I.2 shows basic mechanical properties of structural bolts.

Table I.2 : Basic mechanical properties of structural bolts

type	Bolts grade	f_{yb} (MPa)	f_{ub} (MPa)
ORDINARY	4.6 5.6	240 300	400 500
HIGH STRENGTH BOLT	8.8 10.9	640 900	800 1000

I.7. Conclusion :

Structural steel isn't just for skyscrapers, large buildings and garages. There are multiple reasons why steel makes an attractive building option from start to finish, and one of the most vital parts in steel structures concerning its strength is the connection between members. Hence the advanced methods of calculation consider assembly as a separate element. A steel structure can only perform its best in sustaining load when the connections between members are designed adequately.

CHAPTER II :

Evaluation of Initial Stiffness and Rotation Capacity

II-1. INTRODUCTION

Many researchers show that it is undesirable to design the frame as rigid or pinned connection, as it will lead to an uneconomical design. Experimental and analytical results show that there is a variation of internal force transfer from beam to column which depend upon the bending stiffness of the joint, thickness and location of bolts etc...

These parameters affect the behaviour of the frame and decrease the negative end moment and increase mid-span moment on the beam. Hence, researchers have concluded that the semi-rigid connection should be taken into consideration for analyses and design of steel frames [14].

In this section, the influence of angle parameters in steel structure connections using (top and seat with double web angles, top and seat angles and double web angles), using Frye-Morris Polynomial Model, is studied. Classification of these connections by their rotation moment curve, and the evaluation of an approximate initial stiffness of one type of these connections (top and seat angle) is performed. .

II-2. Mathematical formulations (empirical models) : [12]

Many researchers have developed mathematical formulations of connection behaviours taking into account different elements considered in the connections. Table II.1 shows some mathematical models used to describe the non-linear behaviour of some kinds of connections found in practice.

Table II.1 : Mathematical formulations

Model	Proposed by	Year	Comments
1. Linear			
• Linear	• Bartho • Rathbun • Baker	1931, 1934, 1936 1936 1934	
• Bi-Linear	• Melchers & Kaur • Romstad & Subramaniam • Lui & Chen	1982 1970 1983	
• Piecewise Linear	• Razzaq	1983	
2. Polynomial	• Frye & Morris	1975	$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5$
3. Cubic B-Spline		1972, 1981, 1982	
4. Power	• Batho & Lash • Krishnamurthy • Colson & Louveau • Golberg & Richard • Richard & Abbott	1936 1979 1983 1963 1975 1987	$\theta_r = aM^b;\ a > 0, b > 1$ $\theta_r = \frac{\lvert M \rvert}{K_i} \frac{1}{1 - \lvert M / M_u \rvert^n}$
	• Kishi & Chen • Ang & Morris	1984	$\theta_r = \frac{M}{K_i\left[1 - (M / M_u)^n\right]^{1/n}}$ $\frac{\theta}{(\theta_r)_0} = \left\lvert \frac{KM}{(KM)_0} \right\rvert \left[1 + \left(\left\lvert \frac{KM}{(KM)_0} \right\rvert\right)^{n-1}\right]$
5. Exponential	• Chen & Lui • Kishi & Chen • Yee & Melchers	1986 1986 1986	$M = \sum_{j=1}^{m} C_j \left[1 - \exp\left(-\frac{\lvert\theta_r\rvert}{2ja}\right)\right] + M_0 + R_{kf}\lvert\theta_r\rvert$ $M = M_0 + \sum_{j=1}^{m} C_j \left[1 - \exp\left(-\frac{\lvert\theta_r\rvert}{2ja}\right)\right] + M_0 + \sum_{k=1}^{n} D_k(\theta_r - \theta_k) H[\theta_r - \theta_k]$ $H[\theta] = 1 \quad \theta \geq 0$ $H[\theta] = 0 \quad \theta < 0$ $M = M_p \left\lvert 1 - \exp\left\lvert -\frac{(K_i - K_p + C\theta)}{M_p} \right\rvert\right\rvert + K_p\theta$

II-2-1. Exponential Model : [13]

Lui and Chen (1986) used an exponential function to curve-fit the experimental M–θr data:

$$\boldsymbol{M = \sum_{j=1}^{m} C_j \left\{1 - e^{-\frac{|\theta_r|}{2j\alpha}}\right\} + M_0 + R_{kf}|\theta_r|} \qquad \text{(Eq. II-1)}$$

This model is a good representation of the monotonic nonlinear connection behaviour. However, if there are some sharp changes in slope in the M–θr curve, this model cannot adequately represent it (Wu, 1989). Kishi and Chen (1986) refined the Lui-Chen exponential model to accommodate any sharp changes in slope in the M-θr experimental data (modified exponential model). The Kishi-Chen model can be used instead of the M–θr experimental data. The modified exponential model is represented by a function of the following form:

$$\mathbf{M = M_0 + \sum_{j=1}^{m} C_j \left\{1 - e^{-\frac{|\theta_r|}{2j\alpha}}\right\} + \sum_{k=1}^{n} D_k \left(|\theta_r| - |\theta_k|\right) H \left[|\theta_r| - |\theta_k|\right]} \qquad \text{(Eq. II-2)}$$

where M_0 is the initial connection moment, α is a scaling factor for the purpose of numerical stability, C_j and D_k are curve-fitting parameters, θ_k is the initial rotation of the k-th linear component given from the experimental M–θr curve, and H[θ] is Heaviside's step function (unity for $\theta \geq 0$, zero for $\theta < 0$).

Using the linear interpolation technique for the original M–θr experimental data, the weight function for each piece of M–θr data is nearly equal. The constants C_j and D_k for the exponential and linear terms of the function are determined by linear regression analysis.

II-2-2. Frye- Morris Polynomial Model : [16]

The most popular model for structural analysis is the polynomial function proposed by Frye and Morris (1975). The Frye-Morris model was developed based on a procedure formulated by Sommer (1969). They used the method of least square to determine the constants of the polynomial.

This model has the general form shown in (Eq. II-3)

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5 \qquad \text{(Eq. II-3)}$$

where K is the standardization parameter dependent upon the connection type and geometry, and C1, C2, and C3 are curve fitting constants.

This model represents the M- θr behaviour reasonably well. The main disadvantage is that the nature of the polynomial is to peak and trough within a certain range. Then, the first derivative of this function, which indicated the tangent and connection stiffness, may become negative at some value of connection moment M. This is physically unacceptable. This negative stiffness makes structural analysis difficult if the analysis scheme with tangent connection stiffness is used.

The curve fitting constants C1, C2, and C3 and the standardization constant K for each connection type are summarized in Table II.2. Detail of connection parameters can be seen in figure II.1

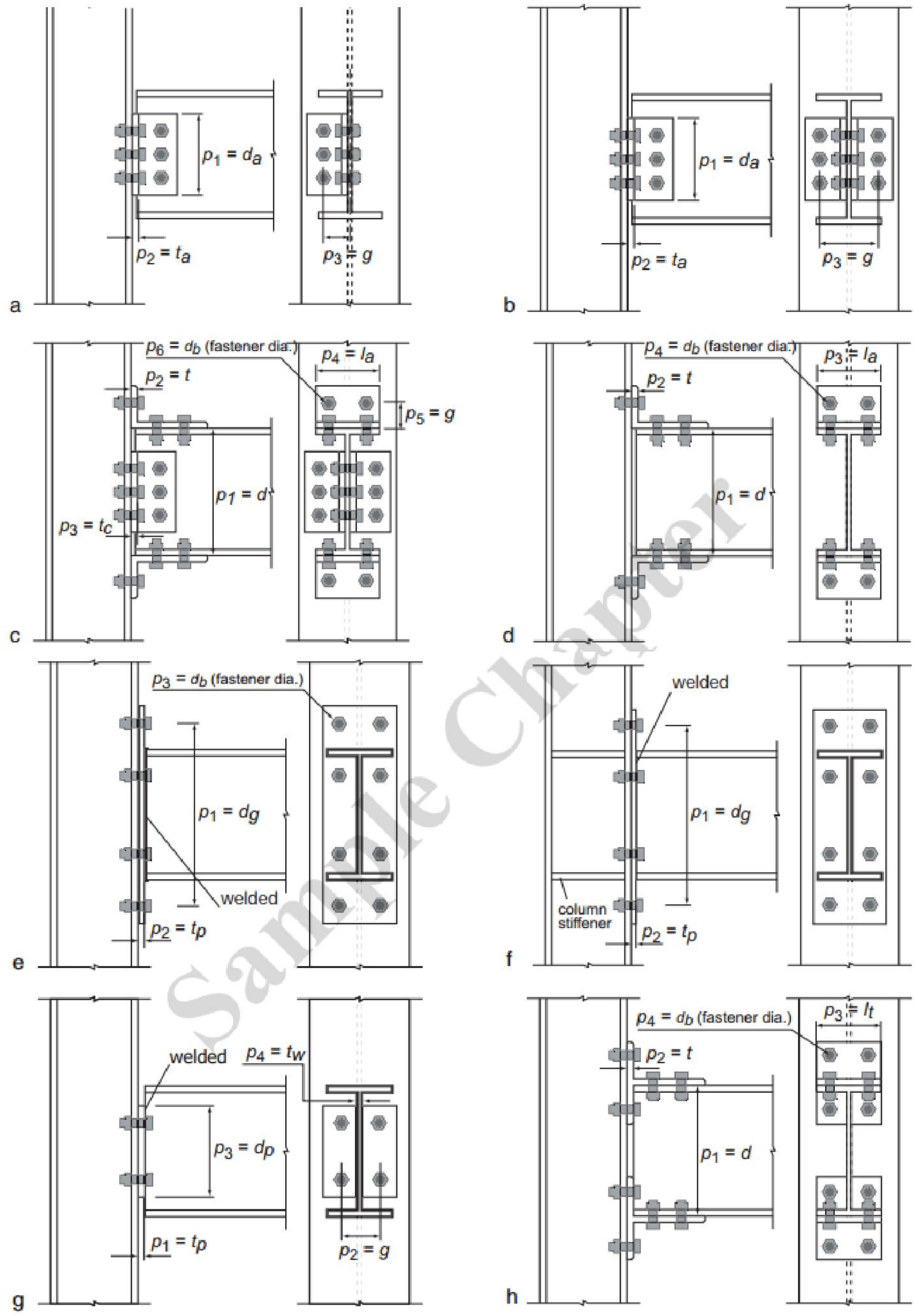


Fig. II.1 Connection Parameters [11]

Table II.2 : curve-fitting constants and standardization constants for Frye-Morris polynomial model (all size parameters are in centimeters) [13] .

Connections types	Curve-fitting constants	Standardization constants
a) Single web angle connection	C1 = 1.67×10^{0} C2 = 8.56×10^{-2} C3 = 1.35×10^{-3}	$K = d_a^{-2.4}\ t_a^{-1.81}\ g^{0.15}$
b) Double web angle connection	C1 = 1.43×10^{-1} C2 = 6.79×10^{1} C3 = 4.09×10^{5}	$K = d_a^{-2.4}\ t_a^{-1.81}\ g^{0.15}$
c) Top and seat angle with double web angle connection	C1 = 1.50×10^{-3} C2 = 5.60×10^{-3} C3 = 4.35×10^{-3}	$K = d^{-1.287}\ t^{-1.128}\ t_c^{-0.415}\ I_a^{-0.694}$ $(g - d_b/2)^{1.35}$
d) Top and seat angle connection	C1 = 2.59×10^{-1} C2 = 2.88×10^{3} C3 = 3.31×10^{4}	$K = d^{-1.5}\ t^{-0.5}\ I_a^{-0.7}\ d_b^{-1.1}$
e) End-plate connection without column stiffeners	C1 = 8.91×10^{-1} C2 = -1.2×10^{4} C3 = 1.75×10^{8}	$K = d_g^{-2.4}\ t_p^{-0.4}\ d_b^{-1.5}$
f) End-plate connection with column stiffeners	C1 = 2.60×10^{-1} C2 = 5.36×10^{2} C3 = 1.31×10^{7}	$K = d_g^{-2.4}\ t_p^{-0.6}$
g) Header plate connection	C1 = 6.14×10^{-3} C2 = 1.08×10^{-3} C3 = 6.05×10^{-3}	$K = t_p^{-1.6}\ g^{1.6}\ d_p^{-2.3}\ t_w^{-0.5}$
h) T-stub	C1 = 6.42×10^{-2} C2 = 1.77×10^{2} C3 = -2.03×10^{4}	$K = d^{-1.5}\ t^{-0.5}\ I_t^{-0.7}\ d_b^{-1.1}$

II-3. Component method [2]

The component method entails the use of relatively simple joint mechanical models, based on a set of rigid links and spring components. The component method can be used to determine the joint's resistance and initial stiffness . Its application requires the identification of active components, the evaluation of the force–deformation response of each component (which depends on mechanical and geometrical properties of the joint) and the subsequent assembly of the active components for the evaluation of the joint moment versus rotation response .

Nowadays, using the Eurocode 3 component method, it is possible to evaluate the rotational stiffness and moment capacity of semi-rigid joints when subject to pure bending. However, this component method is not yet able to calculate these properties when, in addition to the applied moment, an axial force is also present.

Eurocode 3 suggests that the axial load may be disregarded in the analysis when its value is less than 5% of the beam's axial plastic resistance, but provides no information for cases involving larger axial forces. Although the component method does not consider the axial force, its general principles could be used to cover this situation, since it is based on the use of a series of forces versus displacement relationships, which only depend on the component's axial force level, to characterise any individual component's behaviour.

The calculation procedures are based on the component method which requires three steps:

- Definition of the active components for the studied joint;
- Evaluation of the stiffness coefficients (k_i) and/or strength ($\mathbf{F}_{Rd,i}$) characteristics of each individual basic component;
- Assembly of the components to evaluate the stiffness (S_j) and/or resistance (M_e, M_{Rd}) characteristics of the whole joint.

II-3-1. Basic components of an end plate joint

The design moment-rotation characteristic of a joint depends on the properties of its basic components :

- Tension area :(column web, bolts and beam web in tension, column flange and end plate in bending) .
- compression area : (column web, beam web and flange) .
- shear area : (column web) .

The components of an end plate connection can be seen in figure II.2. Different components of connections covered by Eurocode 3 [7]are presented in figure II.3.

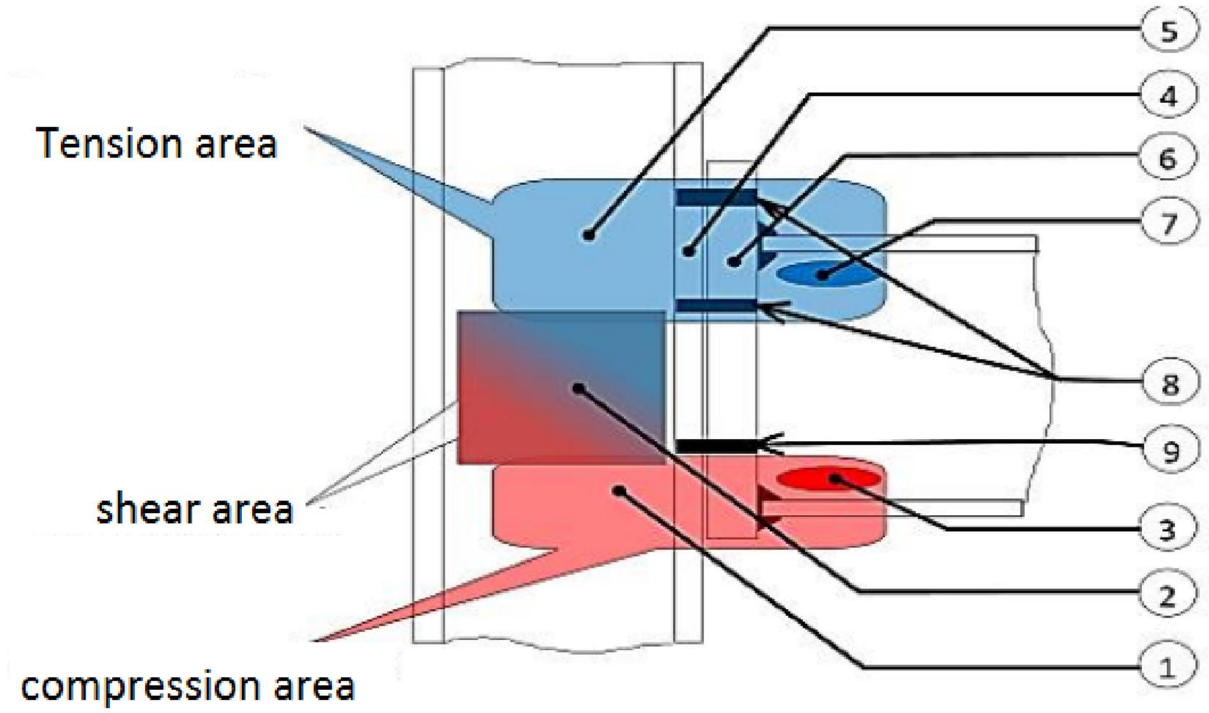


Fig. II.2 Components of end plate beam to column connection

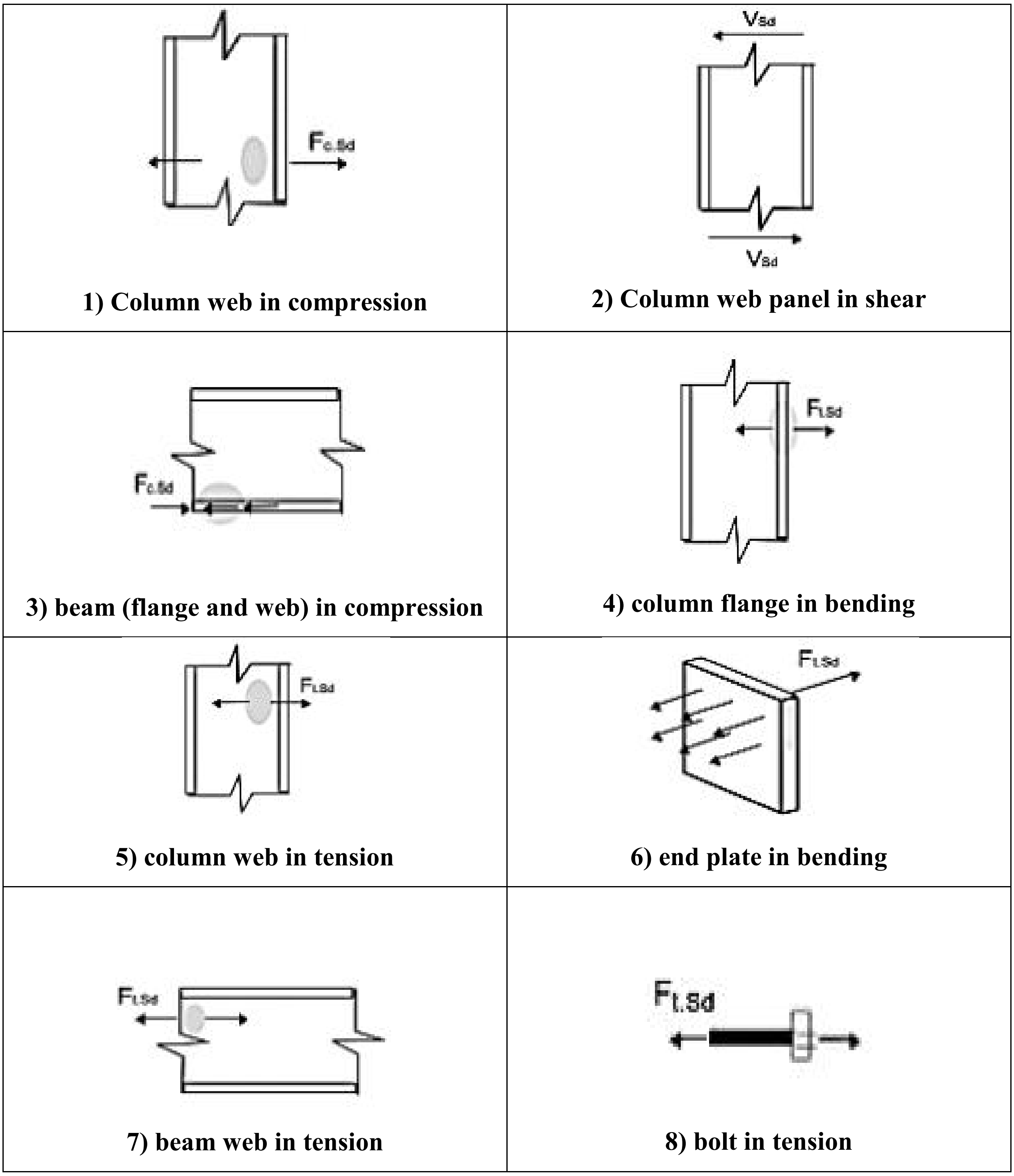


Fig. II.3 Different Components of connection [7]

II-3-2. Rotational stiffness:[8]

The rotational stiffness of a joint should be determined from the flexibilities of its basic components, which represented by its elastic stiffness coefficient ki . These elastic stiffness coefficients are of general application. The numbering of stiffness coefficients is consistent with that in prEN 1993-1-8. [7]

The elastic translational stiffness of a component i is obtained by multiplying ki with Ea**.** For connections with more than one layer of components in tension, the stiffness coefficients ki for the related basic components should be combined .

In a bolted connection with more than one bolt-row in tension, as a simplification, the contribution of any bolt-row may be neglected, provided that the contributions of all other bolt rows closer to the centre of compression are also neglected. The number of bolt–rows retained need not necessarily be the same for the determination of the moment resistance. Provided that the axial force N_{Sd} in the connected member does not exceed 10% of the resistance $N_{pl,Rd}$ of its cross-section, the rotational stiffness S_j of a joint, for a moment $M_{j,Sd}$ less than the moment resistance $M_{j,Rd}$ of the joint, may be obtained with sufficient accuracy from:

$$S_j = \frac{E_a \, . \, Z^2}{\mu \sum_i^n \frac{1}{K_i}} \qquad \text{(Eq. II-4)}$$

where:

Ea modulus of elasticity of steel

ki stiffness coefficient for basic joint component i .

z lever arm

μ stiffness ratio $S_{j,ini}$ / S_j , see below

$S_{j,ini}$ initial rotational stiffness of the joint, given by the expression above with μ=1,0.

The stiffness ratio μ should be determined from the following:

- If $M_{j,Sd} \leq \frac{2}{3} M_{j,Rd}$ $\mu = 1$

- If $\frac{2}{3} M_{j,Rd} < M_{j,Sd} \leq M_{j,Rd}$ $\mu = (1.5M_{j,Sd} / M_{j,Rd})^{\psi}$

In which the coefficient ψ is obtained from **Table II.3** below:

Table II.3 : Values of the coefficient ψ

Type of connection	Value of ψ
Contact plat	1.7
Bolted end-plate	2.7

II-3-3 Initial stiffness $S_{j,ini}$:

The initial rotational stiffness $S_{j,ini}$ is derived from the elastic translational stiffness of the joint components. The elastic behaviour of each component is represented by a spring. The force deformation relationship of this spring is given by:

$$F_i = E.K_i.W_i \qquad \text{(Eq. II-5)}$$

Fi the force in the spring i

E the modulus of elasticity of structural steel

ki the translational stiffness coefficient of the spring i

wi the deformation of spring i.

Separate rotational stiffness can be derived for the connection and the web panel in shear. In simplified joint modelling a value for the overall joints is all that is required. The derivation of this is now explained.

II-3-4. Connections with more than one layer of components in tension :

Figure. II.4 (a) shows the spring model adapted for a more complicated case where tensile forces arising from bending are carried not only by a layer of reinforcement but also by a bolt-row in tension within the steelwork connection. The reinforcement is assumed to behave like a bolt row in tension, but with different deformation characteristics. It is assumed that the deformations are proportional to the distance to the point of compression, but that the elastic forces in each row are dependent on the stiffness of the components.

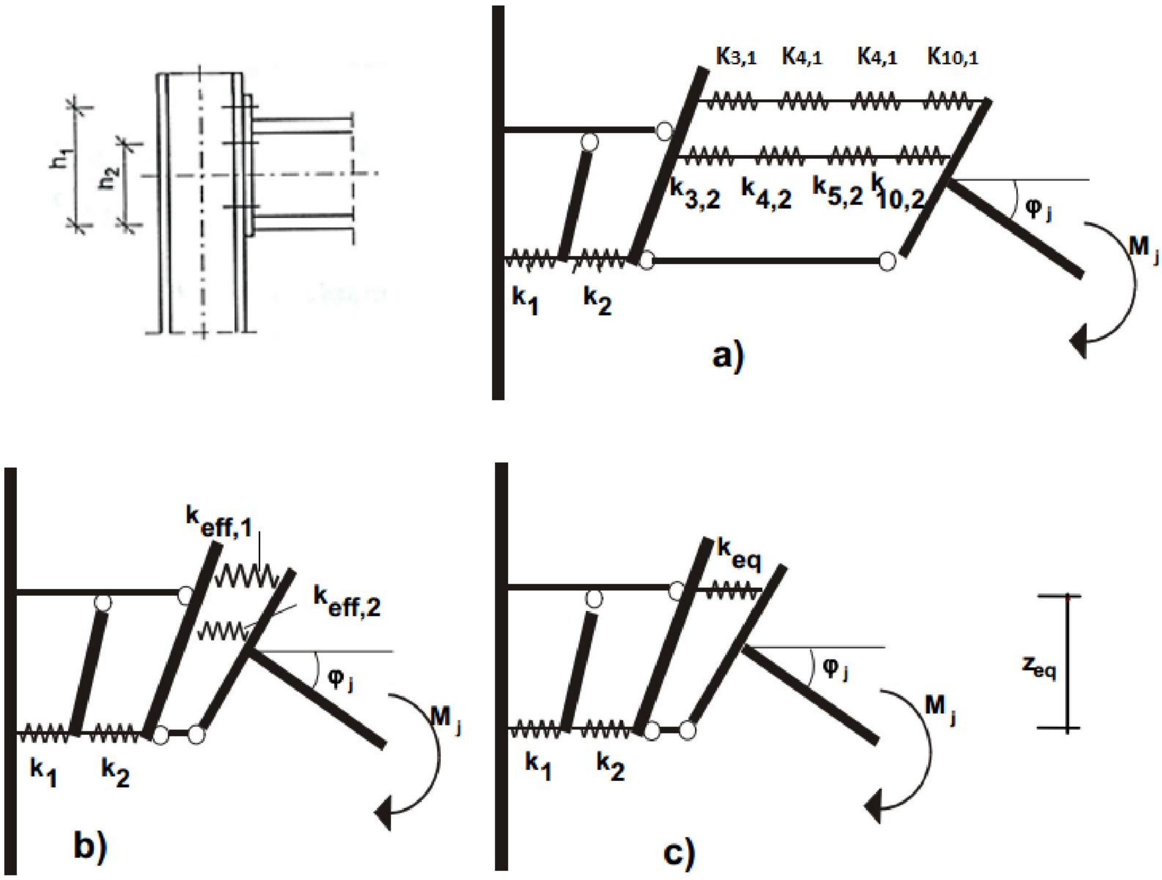


Fig. II.4 Spring model for a composite flush end-plate connection

Figure. II.4 - b shows how the deformations per bolt-row of the end-plate in bending, the bolts in tension, the column flange in bending and the column web in tension are added to form an effective spring per bolt-row, with an effective stiffness coefficient $K_{eff,r}$ (r is the index of the row number). Fig. II.4 (c) indicates how these effective springs at each level are replaced by an equivalent spring acting at a corresponding lever arm z. The stiffness coefficient of this equivalent spring is k_{eq} and this can be directly applied in the preceding equation for the stiffness $S_{j,ini}$.

The formulae given below to determine $k_{eff,r}$, z and k_{eq} can be derived from the sketches of Figure. II.4. The basis for these formulae is that the moment-rotation behaviour of each of the systems in Figure II.4 is equal. An additional condition is that the compressive force in the lower rigid bar is equal in each of these systems.

For composite connections with more than one layer of components in tension, the basic components related to all of these layers should be represented by a single equivalent stiffness coefficient k_{eq} determined from:

$$K_{eq} = \frac{\sum_r K_{eff.r}.h_r}{Z_{eq}} \qquad \text{(Eq. II-6)}$$

where:

h_r is the distance between layer r and the centre of compression.

$K_{eff,r}$ is the effective stiffness coefficient for layer r taking into account the stiffness ki for the basic components listed below as appropriate.

Z_{eq} is the equivalent lever arm

For a composite joint with more than a single layer of reinforcement considered effective in tension, the above provisions are applicable provided that the layers are represented by a single layer of equivalent cross-sectional area and equivalent distance to the centre of compression.

The effective stiffness coefficient $k_{eff,r}$ for bolt-row r should be determined from:

$$K_{eff,r} = \frac{1}{\sum_i \frac{1}{K_{i.r}}} \qquad \text{(Eq. II-7)}$$

where:

$k_{i,r}$ is the stiffness coefficient representing component i relative to bolt-row r.

The equivalent lever arm z_{eq} should be determined from:

$$Z_{eq} = \frac{\sum_r K_{eff,r} \cdot h_r^2}{\sum_r K_{eff,r} \cdot h_r} \quad \text{(Eq. II-8)}$$

Then the stiffness (ki) and the design resistance ($F_{Rd,i}$) of each component are evaluated from analytical models. The assembly is achieved as follows:

Initial stiffness:

$$S_{j,ini} = \frac{E_a \cdot Z^2}{\sum_{i=1}^{n} \frac{1}{K_i}} \quad \text{(Eq. II-9)}$$

where:

z relevant lever arm .

n number of relevant components .

Ea steel elastic modulus.

- Nominal stiffness:

$$S_j = \frac{S_{j,ini}}{1,5}$$ for beam-to-column joints with contact plates

$$S_j = \frac{S_{j,ini}}{2,0}$$ for beam-to-column joints with flush end -plates

- Plastic design moment resistance:

$$F_{Rd} = \min\left[F_{Rd,i}\right] \quad \text{(Eq. II-10)}$$

$$M_{Rd} = F_{Rd} \cdot Z \quad \text{(Eq. II-11)}$$

- Elastic design moment resistance:

$$M_e = \frac{2}{3} M_{Rd} \qquad \text{(Eq. II-12)}$$

II-4 . Evaluation of the initial stiffness of semi rigid connections [17]**:**

Wang Yan proposed a way to calculate the initial stiffness for semi rigid connections (extended end-plate bolted connections , top and seat angle connection , Flange and web angle connection , top-and-seat Tee-stub connection).

Table II.4 shows that the initial stiffness of semi-rigid connections is mainly related to the bending stiffness of the joints, thickness and location of bolts. Therefore the geometry of the joint can be adjusted in order to increase or decrease the initial stiffness of connections.

Table II.4 : Common initial stiffness of semi-rigid connections

Connection type	extended end-plate bolted connections	top and seat angle connection	top-and-seat Tee-stub connection	Flange and web angle connection
Initial stiffness ($S_{j,ini}$)	$\frac{192EI_p}{1+\frac{12.48t_p^2}{e^2}}\frac{h_0^2}{e^3}$	$\frac{3EI_a}{1+\frac{0.78t_a^2}{e_0^2}}\frac{h_1^2}{e_0^3}$	$\frac{192EI_T}{1+\frac{12.48t_T^2}{e^2}}\frac{h_0^2}{e^3}$	$\frac{3EI_a\ h_1^2}{e_0(e_0^2+0,78t_a^2)} + \frac{6EI_{wa}\ (h_t/2)^2}{e_3(e_3^2+0,78t_{wa}^2)}$

t_{wa} is thickness of web-angle; EI_T , EI_{wa} are the bending stiffness of tee-stub and web-angle respectively .

II-5 . Parametric Study :

II-5-1 .The initial stiffness of semi rigid connections :

The connection considered in our parametric study is a beam to column connection with top and seat angles as shown in figures II.5 and II.6.

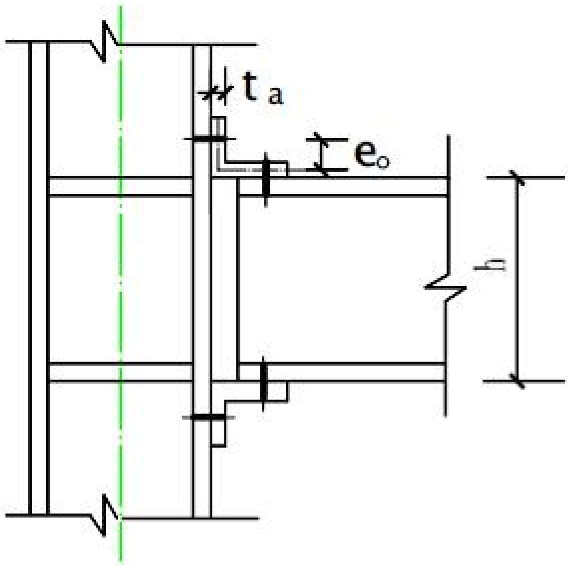


Fig. II.5 Typical top and seat angle connection

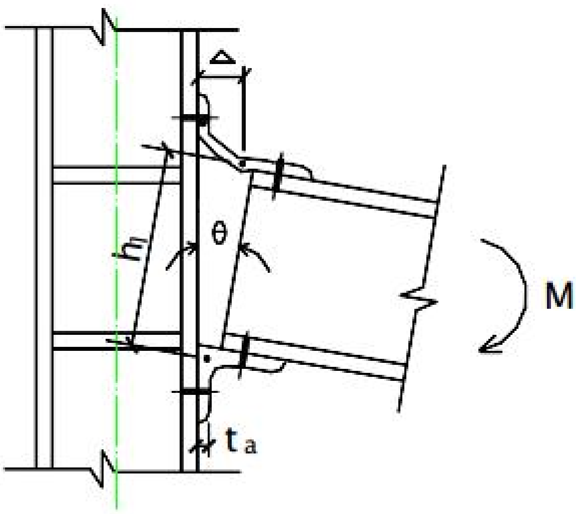


Fig. II.6 Deflected configuration at elastic condition

- **Connection details :**

Beam section : (IPE 220)

- Beam height (h) = 220 mm .
- Beam width (bf) = 110 mm .
- Moment of inertia of a beam $I_y = I_b = 204.990\ \text{cm}^4$

Angle details :

- The angle section (L80×80× 8) mm .
- Length of the angle (I_e) = 110 mm .
- Moment of inertia of angle : $\mathbf{I_a = \frac{(I_e \times t_a^3)}{12}}$.
- Bolts location (e_o), (varies).
- Young's modulus (E) = 2.10 X 10^5 N/mm² .

- **Boundaries for stiffness classification :**

- Rigid boundary $S_{j,ini(rg)} = 25\frac{(E.I_b)}{L_b}$
- Pinned boundary $S_{j,ini(pin)} = 0.5\frac{(E.I_b)}{L_b}$

Where :

L_b Beam length : $L_b = 5$ m ;

I_b Beam moment of inertia : I_b(IPE 220) = 204.990 cm^4

II-5-1-1 Influence of the bolt location eo :

- In this case all the parameters are fixed, we only change (eo), therefore:

$$I_a = \frac{(I_e \times t_a^3)}{12} = \frac{110 \times 8^3}{12} = 4693.33 \text{ mm}^4 \ ;$$

$$h_1 = h + t_a = 220 + 8 = 228 \text{ mm} \ ;$$

$$Sj,ini = \frac{3EI_a}{1+\frac{0,78t_a^2}{e_0^2}} \frac{h_1^2}{e_0^3} \ ;$$

$$Sj,ini(rig) = 25\frac{(E.I_b)}{L_b} = 25\frac{(21000\ .\ 204.990)}{500} = 215239.5 \text{ KN.cm}$$

$$Sj,ini(pin) = 0.5\frac{(E.I_b)}{L_b} = 0.5\frac{(21000\ .\ 204.990)}{500} = 4304.79 \text{ KN.cm}$$

Table II.5 : Values of the initial stiffness corresponding to (e_0)

E (KN/cm²)	Ia (mm^4)	h1 (mm)	e0 (mm)	ta (mm)	Sj,ini (KN.cm)
21000	4693.33	228	**40**	8	**232899,61**
21000	4693.33	228	**50**	8	**120557,74**
21000	4693.33	228	**60**	8	**70187,06**
21000	4693.33	228	**70**	8	**44360,40**

✓ **Results and discussion :**

- As can be seen from table II.5, When the value of **(e_0)** decreases, the initial stiffness of the connection increases , and tends to become a more rigid connection. However, that drives the internal forces (tension) to the bolts .
- Almost in all the cases the initial stiffness's found are in between the initial stiffness boundaries, therefore they are all considered as semi-rigid connections. Figure II.7 shows the influence of the variation of e_0 on the initial stiffness of a connection.

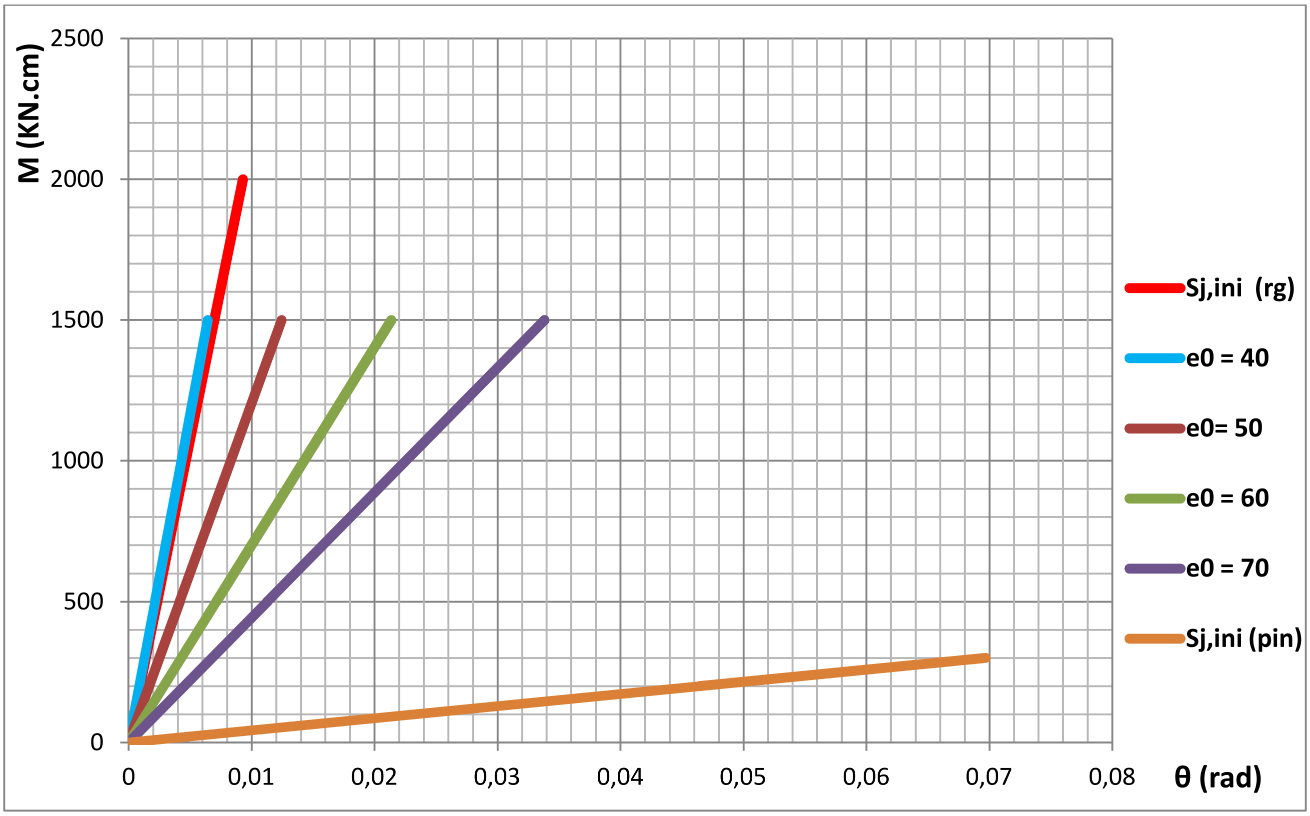


Fig. II.7 The influence of the bolts location to initial stiffness .

From table II.5 and figure II.7, it can be seen that if:

(e_0) > 40 mm the connection is classified as semi-rigid.

(e_0) ≤ 40 mm the connection is classified as rigid.

II-5-I-2. Influence of the Angle Thickness t_a :

In this case the value of t_a is varied where :

The angle section (CAE L80×80×ta) ;

$$I_a = \frac{(I_e \times t_a^3)}{12} = \frac{110 \times ta^3}{12} \quad ;$$

$$h_1 = h + t_a = 220 + t_a$$

$$S_{j,ini} = \frac{3EI_a}{1+\frac{0,78t_a^2}{e_0^2}} \frac{h_1^2}{e_0^3} \quad ;$$

$$S_{j,ini}(rig) = 25\frac{(E.I_b)}{L_b} = 25\frac{(21000\ .\ 204.990)}{500} = 215239.5\ KN.cm$$

$$S_{j,ini}(pin) = 0.5\frac{(E.I_b)}{L_b} = 0.5\frac{(21000\ .\ 204.990)}{500} = 4304.79\ KN.cm$$

Table II.6 : initial stiffness by changing the value of (t_a)

E (KN/cm²)	Ia (mm^4)	h1 (mm)	e0 (mm)	ta (mm)	$S_{j,ini}$ (KN.cm)
21000	4693.33	228	65	8	55315,94
21000	6682.50	229	65	9	79207,10
21000	9166.67	230	65	10	109225,23
21000	15840.00	232	65	12	190518,38

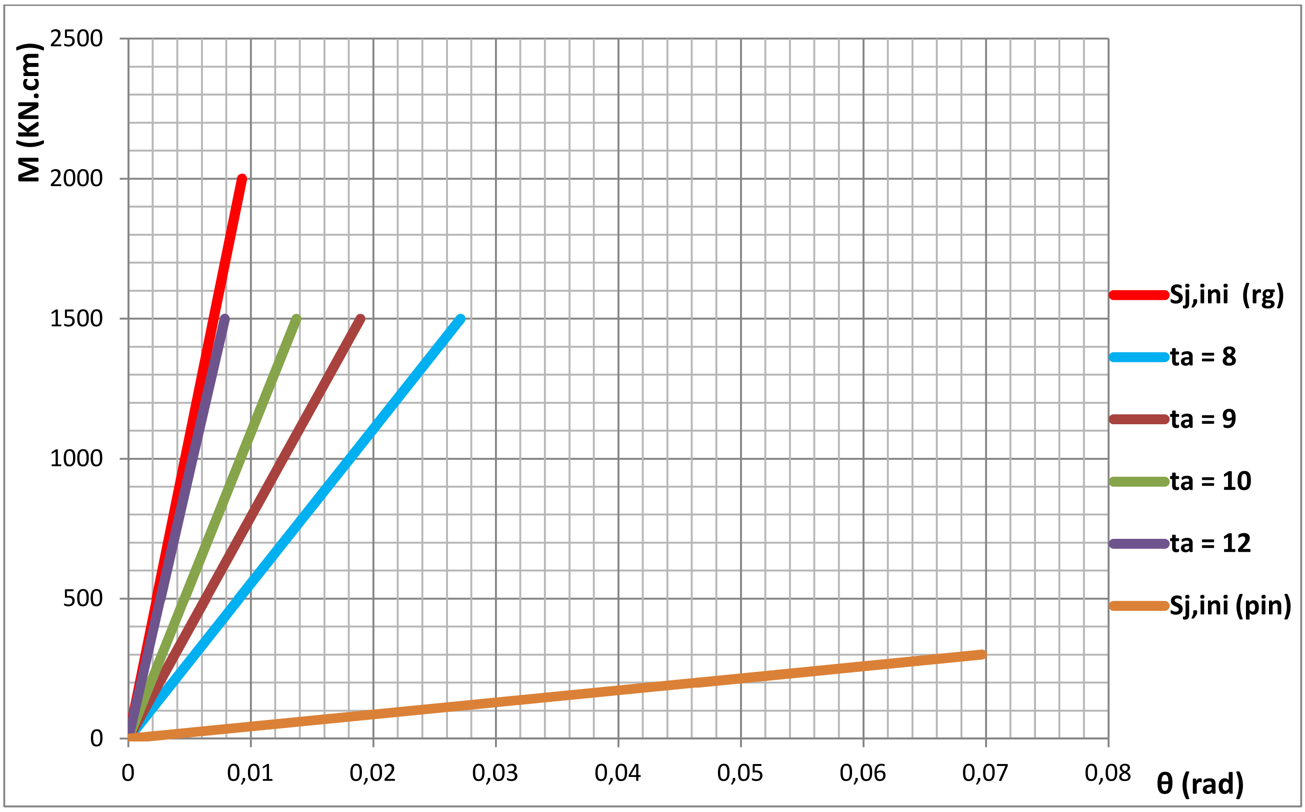


Fig. II.8 The influence of the angle thickness (ta) to initial stiffness

- It can be observed that when the value of the angle thickness **(t_a)** increases, the initial stiffness of the connection increases too. Therefore this tends to become stiffer i.e. more rigid.

- Table II. 6 and figureII.8 show that whatever the value of **(t_a = 8 to 12 mm),** This kind of connection is classified as semi-rigid connection .

c) Influence of beam height (h) :

- In this cas we change the beam section , and fixe the other parameters (bolt location **e_0** and angle thickness **t_0**) :

Table II.7 : Geometrical parameters of the Beam

Section	IPE 220	IPE 240	IPE 270	IPE 300
Height (mm)	220	240	270	300
Width (mm)	110	120	135	150
Moment of inertia -I_b- (cm^4)	204.990	283.990	420.020	603.990

Where :

Angle section (CAE L80×80×8) ;

t_a = 8 mm ;

$$I_a = \frac{(I_e \times t_a^3)}{12} = \frac{110 \times 8^3}{12} = 4693.33 \text{ mm}^4 ;$$

e_0 = 65 mm ;

$h_1 = h + t_a$; (table below)

Table II.8: Different beam sections used in the analysis

Section	IPE 220	IPE 240	IPE 270	IPE 300
h1 (mm)	228	248	278	308

Initial stiffness Boundaries :

$$S_{j,ini(rig)} = 25\frac{(E.I_b)}{L_b} \quad ; \quad S_{j,ini(pin)} = 0.5\frac{(E.I_b)}{L_b}$$

Table II.9: different beam initial stiffness boundaries

Section	IPE 220	IPE 240	IPE 270	IPE 300
Sj,ini(rig) (KN.cm)	215239.50	298189.50	441021.00	634189.50
Sj,ini(pin) (KN.cm)	4304.79	5963.79	8820.42	12683.79

Table II.10 initial stiffness of the connection by varying beam sections :

E (KN/cm²)	section	Ia (mm^4)	h1 (mm)	e0 (mm)	ta (mm)	Sj,ini (KN.cm)
21000	IPE 220	4693.33	228	65	8	55315,94
21000	IPE 240	4693.33	248	65	8	65446,13
21000	IPE 270	4693.33	278	65	8	82237,56
21000	IPE 300	4693.33	308	65	8	100944,35

- **Results and discussion :**

- As shown in figure II.9, when the section and the height of the beam increase , the initial stiffness of the connection increases too, and this connection tends to become more rigid .

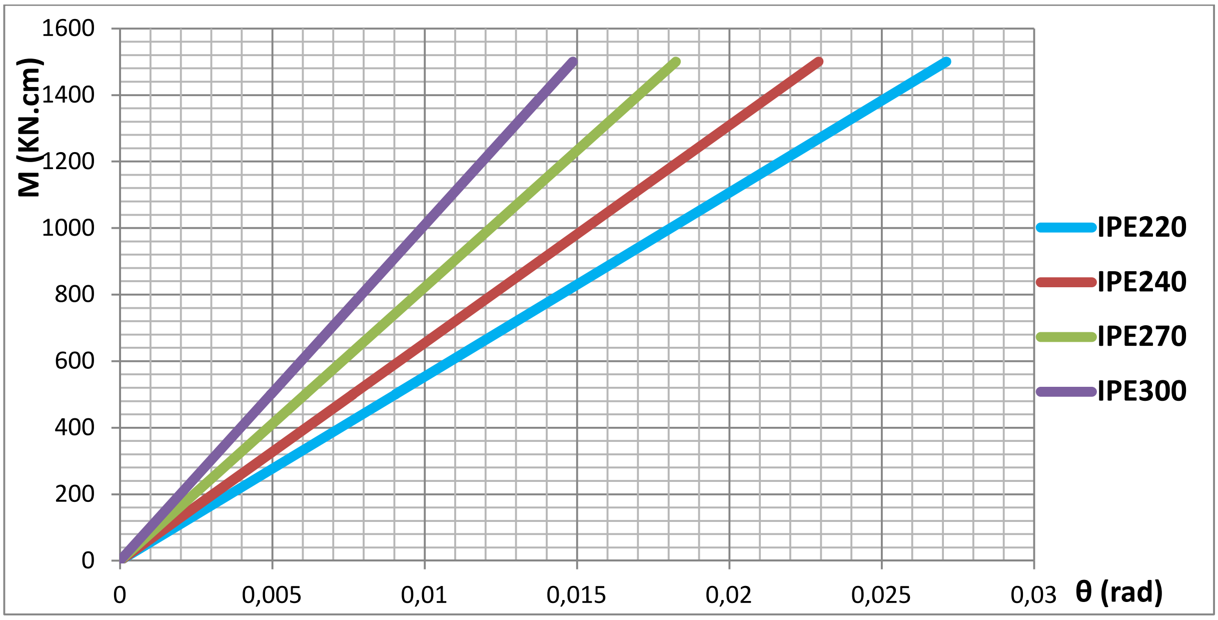


Fig. II.9 Initial stiffness for each beam section

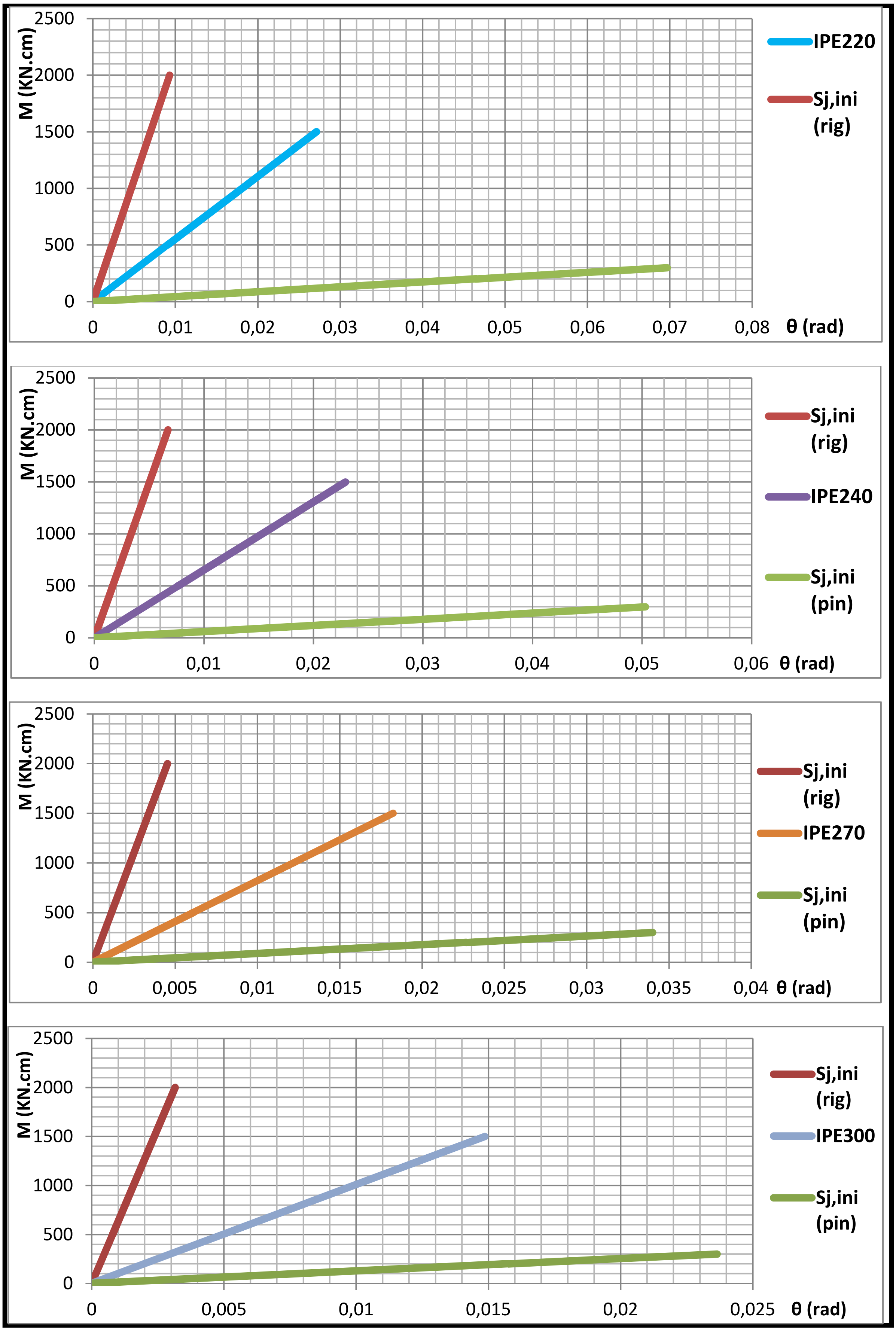


Fig. II.10 Initial stiffness boundary for each beam section

II-5-2 . Parametric study for Frye- Morris polynomial Model :

In this section the polynomial model is applied on several types of semi rigid connections (top and seat with double web angle connection , top and seat angle connection , double web angle connection) , and draw its (M-θ) curves in a same graph to compare in between .

- Connection elements :

Beam section : (IPE 220) ;

Angles section (CAE $80 \times 80 \times 8$) .

a) Top and seat with double web angle connection : (TSDWA)

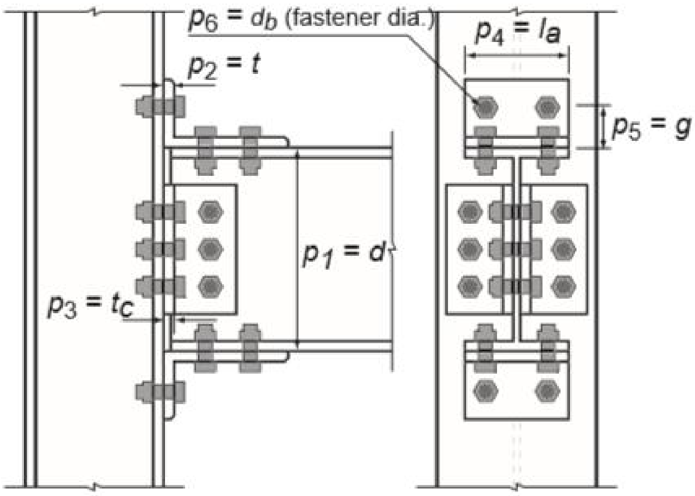


Fig. II.11 Top and seat with double web angle

Table II.11 : Connection parameters

d (cm)	t_a (cm)	t_c (cm)	la (cm)	g (cm)	d_b (cm)
22	0.8	0.8	11	6	1.6

where :

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5$$

and :

C1 = 1.50× 10^{-3} ; C2 = 5.60× 10^{-3} ; C3 = 4.35× 10^{-3}

$K = d^{-1.287}\ t^{-1.128}\ t_c^{-0.415}\ I_a^{-0.694}\ (g - d_b/2)^{1.35}$;

K = $22^{-1.287}\ 0.8^{-1.128}\ 0.8^{-0.415}\ 11^{-0.694}\ (6 - 1.6/2)^{1.35}$

K = 0.046314

Table II.12 : Table of result for (TSDWA)

M	M×K	C_1× (KM)	C_2 × $(KM)^3$	C_3 × $(KM)^5$	Θ
1	0.0463	6.947 E^{-05}	5.563 E^{-07}	9.269 E^{-10}	7.000 E^{-05}
2	0.0926	1.389 E^{-04}	4.451 E^{-06}	2.966 E^{-08}	1.430 E^{-04}
3	0.1389	2.084 E^{-04}	1.502 E^{-05}	2.252 E^{-07}	2.240 E^{-04}
4	0.1852	2.779 E^{-04}	3.560 E^{-05}	9.491 E^{-07}	3.140 E^{-04}
8	0.3705	5.558 E^{-04}	2.848 E^{-04}	3.037 E^{-05}	8.710 E^{-04}
10	0.4631	6.947 E^{-04}	5.563 E^{-04}	9.269 E^{-05}	1.340 E^{-03}

note : "M" takes an unitary value .

b) Top and seat angles connection : (TSA)

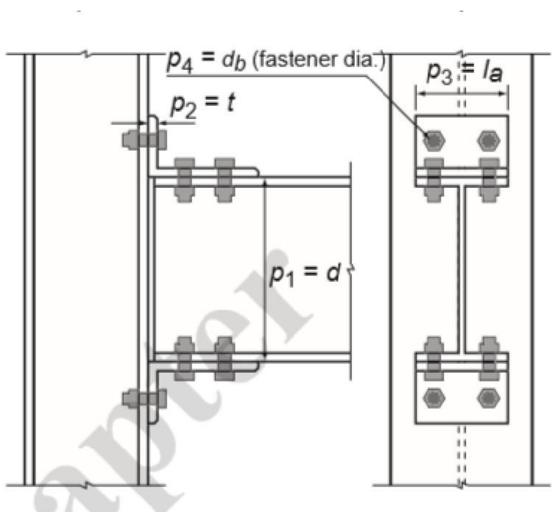


Fig. II.12 Top and seat angle connection

Table II.13 : (TSA) connection parameters

d (cm)	t (cm)	la (cm)	db (cm)
22	0.8	11	1.6

where :

$$\boldsymbol{\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5}$$

and :

C1 = 2.59×10^{-1} C2 = 2.88×10^{3} C3 = 3.31×10^{4}

$$K = d^{-1.5}\ t^{-0.5}\ I_a^{-0.7}\ d_b^{-1.1}$$

$K =$ 0,00120591

Table II.14 : Table of result for (TSA)

M	M×K	$C_1\times$ (KM)	$C_2\times (KM)^3$	$C_3\times (KM)^5$	Θ
1	0.0120 E^{-01}	3.1233 E^{-04}	5,0505E^{-06}	8,4410 E^{-11}	3,17 E^{-04}
2	0.0241 E^{-01}	6.2466 E^{-04}	4,0404E^{-05}	2,7011 E^{-09}	6,65 E^{-04}
3	0.0361 E^{-01}	9.3699 E^{-04}	1.3636 E^{-04}	2,0511 E^{-08}	1,07 E^{-03}
4	0.0482 E^{-01}	12.49320 E^{-04}	3.2323 E^{-04}	8,6436 E^{-08}	1,57 E^{-03}
8	0.0964 E^{-01}	24.98640 E^{-04}	2.58586 E^{-03}	2,7659 E^{-06}	5,09 E^{-03}
10	0.1205 E^{-01}	31.2330 E^{-04}	5.05051 E^{-03}	8,4410 E^{-06}	8,18 E^{-03}

c) Double web angles connection : (DWA)

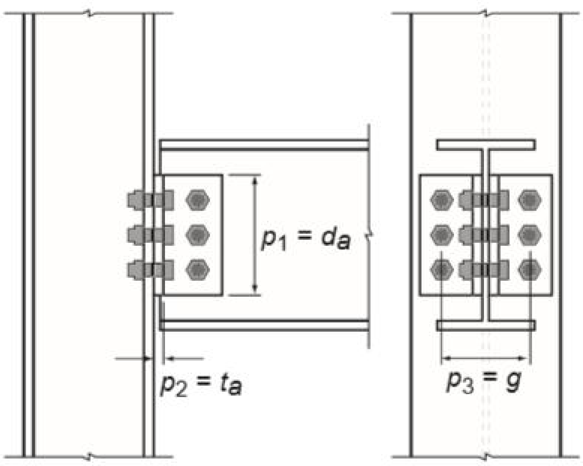


Fig. II.13 Double web angles connection

Table II.15 : (DWA) connection parameters

da (cm)	ta (cm)	g (cm)
11	0,8	11

where :

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5$$

and :

C1 = 1.43× 10^{-1} C2 = 6.79× 10^{1} C3 = 4.09× 10^{5}

$$K = d_a^{-2.4}\ t_a^{-1.81}\ g^{0.15}$$

$$K = 0.0067963$$

Table II.16 : Table of results for (DWA)

M	M×K	C_1× (KM)	C_2 × $(KM)^3$	C_3 × $(KM)^5$	Θ
1	0,0067	0.9718 E^{-03}	2,1315E^{-05}	0.5930 E^{-05}	9,99 E^{-04}
2	0,0135	1.9437 E^{-03}	1.7052 E^{-04}	0.1897 E^{-03}	2,30 E^{-03}
3	0,0203	2.9156 E^{-03}	5.7551 E^{-04}	0.1441 E^{-02}	4,93 E^{-03}
4	0,0271	3.8874 E^{-03}	13.6417 E^{-04}	0.6072 E^{-02}	1,13 E^{-02}
8	0,0543	7.7749 E^{-03}	10.9134 E^{-03}	0,1943	2,13 E^{-01}
10	0,0679	9.7187 E^{-03}	21.3152 E^{-03}	0,5930	6,24 E^{-01}

- From the tables of results **(Table II.12 ; Table II.14 ; Table II.16),** we draw different **(M- θ)** curves of each table in a same graph to compare in between :

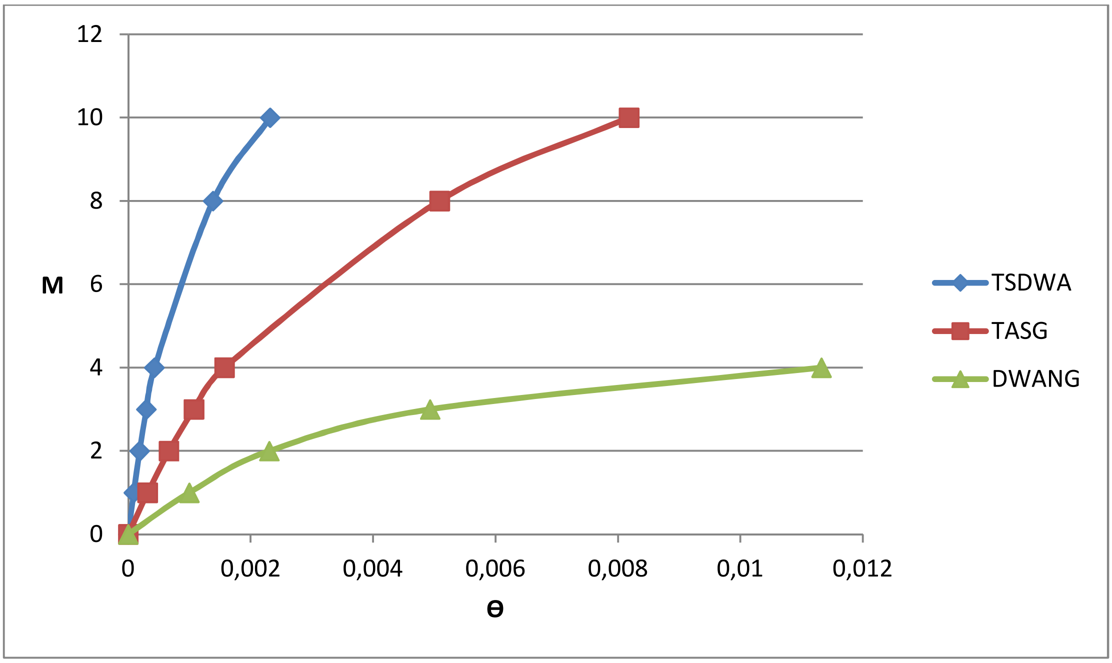


Fig. II.14 The M- θ behaviour

II-6 . Conclusion

- The Frye-Morris polynomial model and Evaluation of the initial stiffness are based on the assembly parameters that can be easily determined from the assembly configuration and their details, thus lending itself for practical uses.

CHAPTER III :

Parametric study on a flange beam-to-column connection

III-1 .Introducton :

In this chapter we are interested in the modeling of steel frame with flange connection (top and seat angle), based on the European norm (Eurocode 3), applicable on steel structures which are composed of beam and column elements and interconnected by bolted connections . This part consists on the analysis of the effect of varying different geometrical parameters of a same flange assembly on the level of performance and on the frame responses.The frame is subjected to the same type of loading. The sizing and the verifications of the structure and the connections studied were obtained using Autodesk Robot Structural Analysis Professional Software .

III-2 . Description of Autodesk Robot Structural Analysis Professional :

Autodesk Robot Structural Analysis Professional is a software of CAD / DAO for modeling, analyzing and dimensionning different types of structures (steel , concrete ,wood , ...) .

This software enables to create structures, design even more complex models with powerful finite element auto-meshing, nonlinear algorithms, verify the results obtained, size the specific elements of the structure, and as the last step managed by Robot, creates the documentation for the calculated structure and dimensions.

III-3 . Analysis of semi-rigid connections (flange connection):

III-3-1 . Description of the frame :

The structure that is the subject of our study is a (2D) portal frame with one floor and a single span . The span of the beam measured between the axis of the columns is 5 m, The height from the column base to the axis of the floor beam is 3 m. Permanent and live loads are applied, horizontal load at the node (1) of the structure F=25KN and vertical load Q=20 kN/m, see **Fig III.1** .

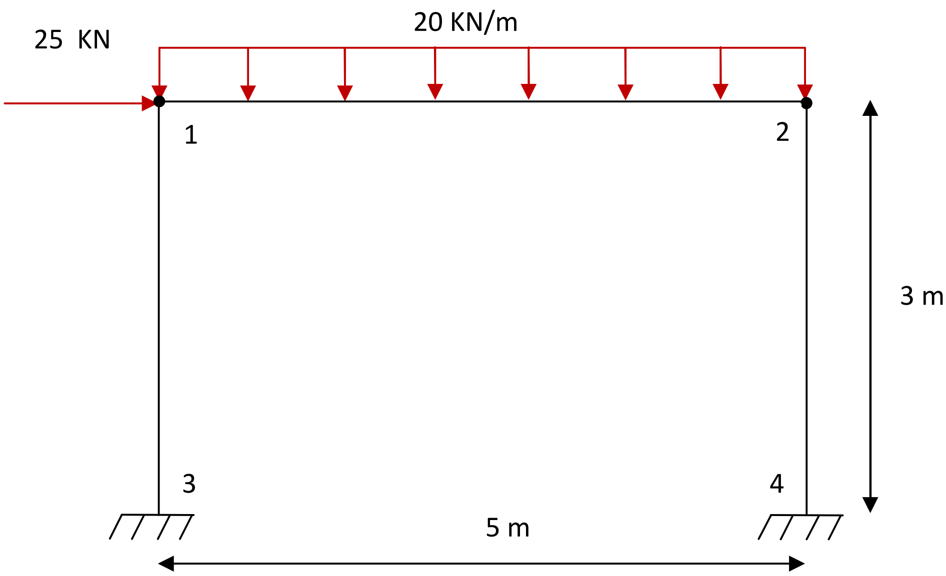


Fig III.1 Portal frame with one floor and a single span .

III-3-2 . Design of the frame with Autodesk Robot Structural Analysis :

- By clicking on the " **Frame 2D Design** " (FigureIII.2), the choice of the frame can be made.

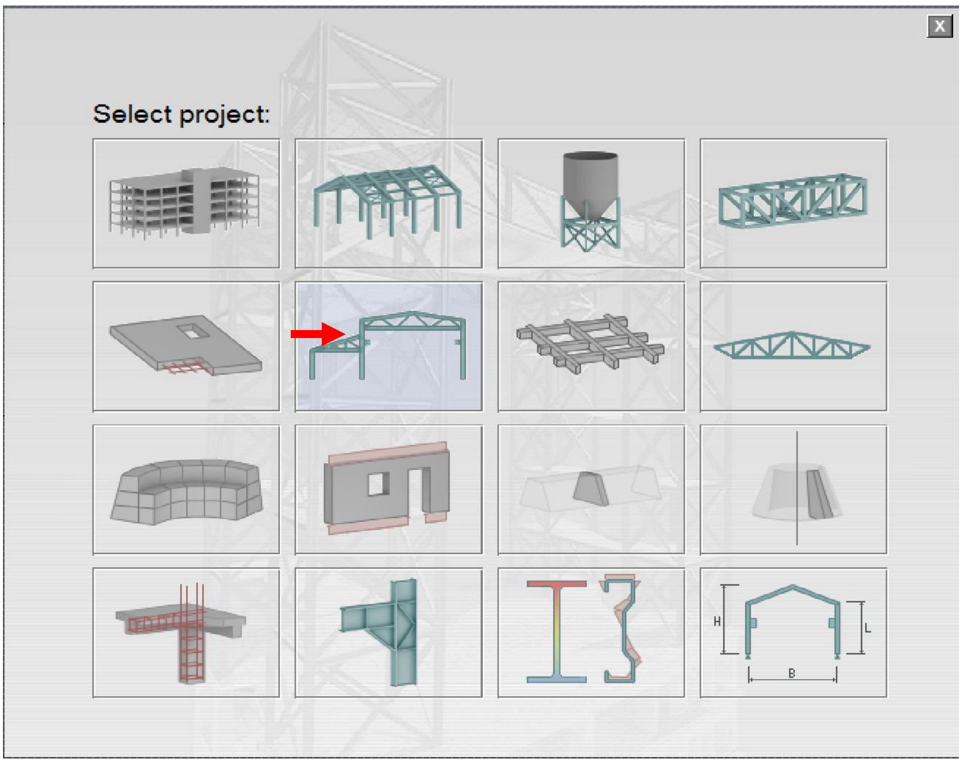


Fig III.2 Autodesk Robot interface

- The Choice of the sections and the drawing of the frame are shown in Figure III.3 and FigureIII.4 respectively :

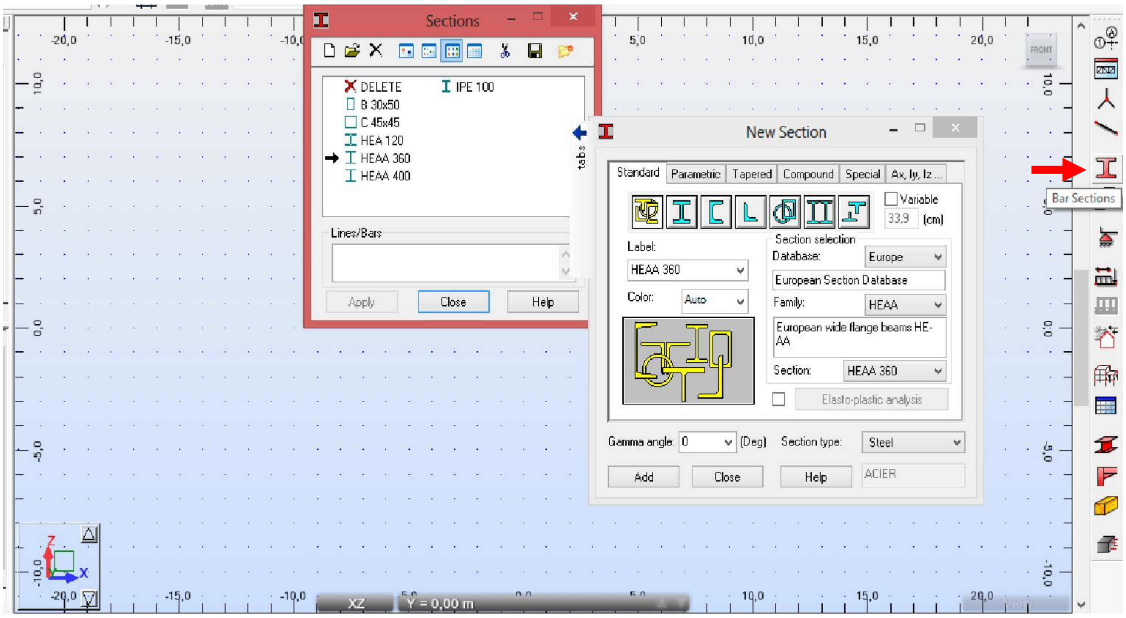


Fig III.3 Different section profiles

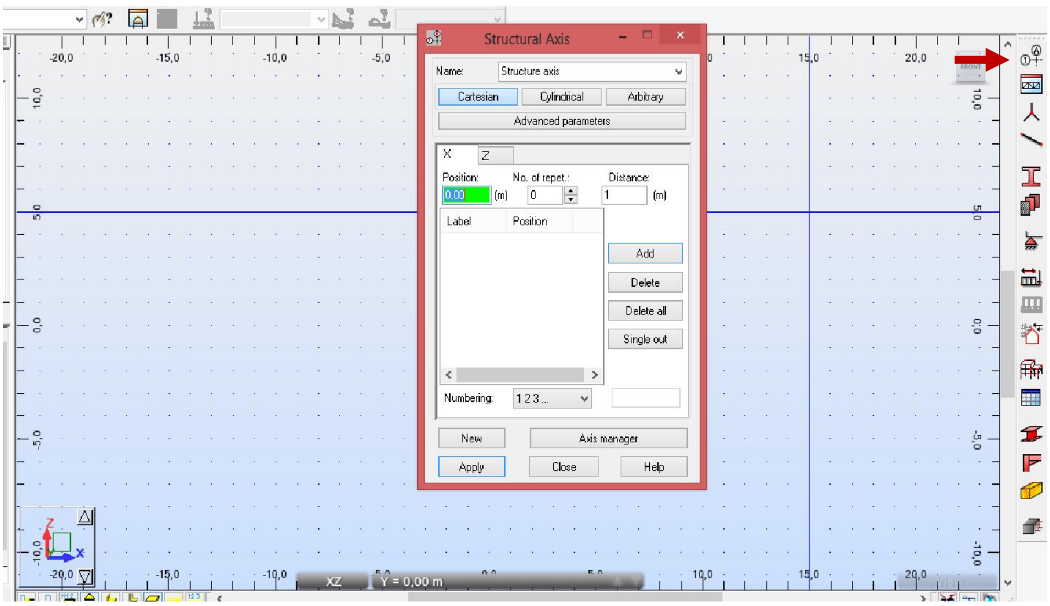


Fig III.4 Window to choose the dimensions of frame

- Figure III.5 shows the view of the 3D portal frame analysed :

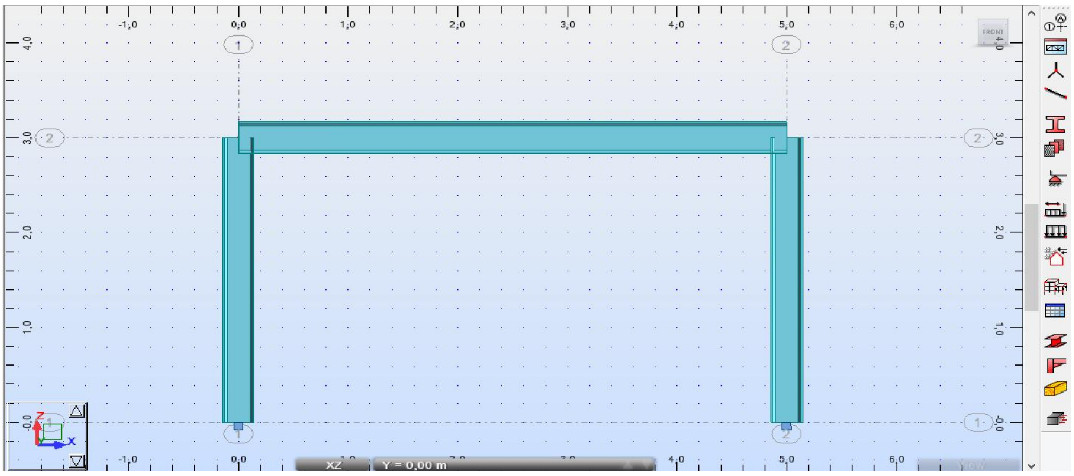


Fig III.5 3D frame view

- Permanent and live loads are choosen and presented in Figure III.6 :

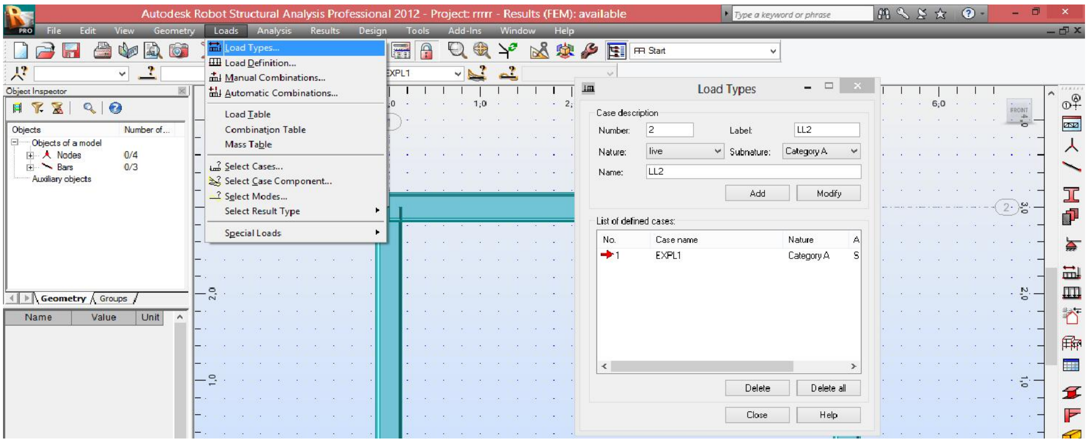


Fig III.6 Defining the load types

- Vertical and horizontal loads are applied by clicking on "**Load Definition**" as shown in Figure III.7:

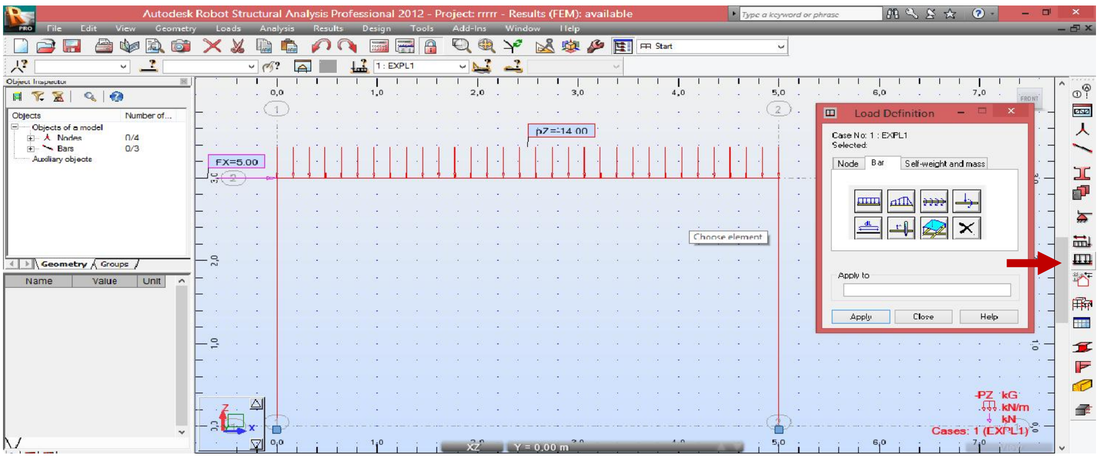


Fig III.7 Defining the vertical and the horizontal loads

- Checking the structure against the instability phenomenon is presented in Figure III.8 :

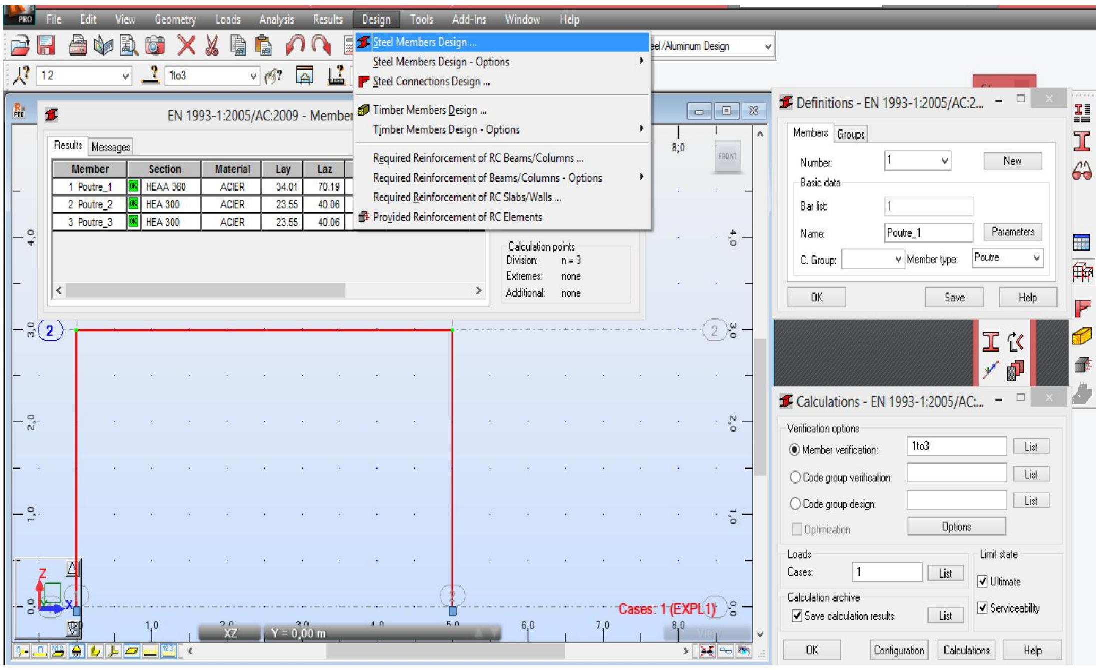


Fig III.8 Verification of the stability of the frame

- Figure III.9 shows how the connections are designed :

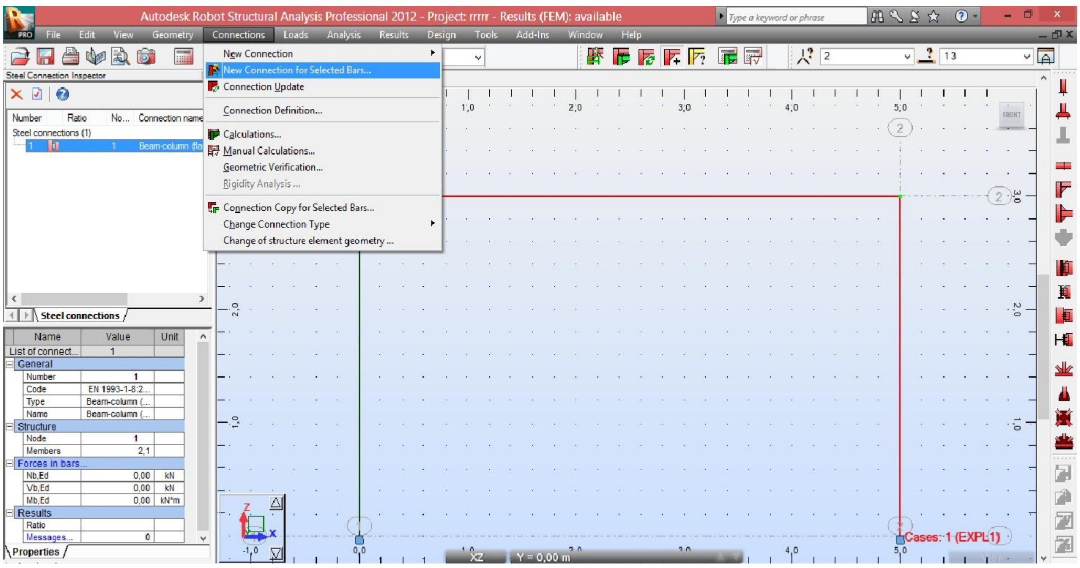


Fig III.9 Defining beam to column connections

- Determining the geometrical parameters of the connection is presented in Figure III.10

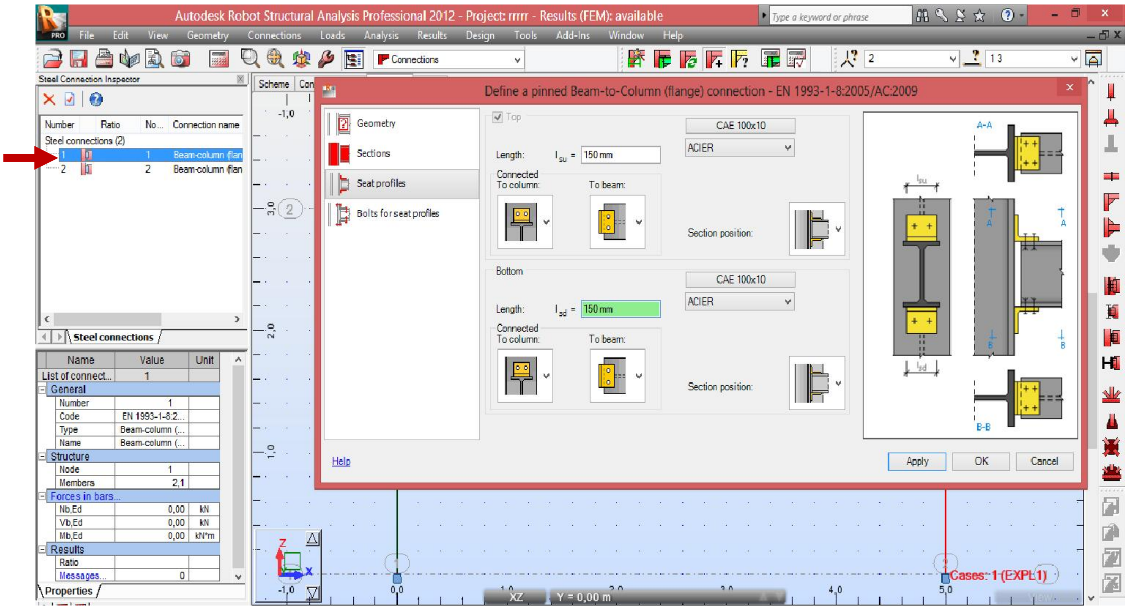


Fig III.10 Defining the geometrical parameters of the double flange connection

- A (3D) Double Flange connection view is shown in Figure III.11 :

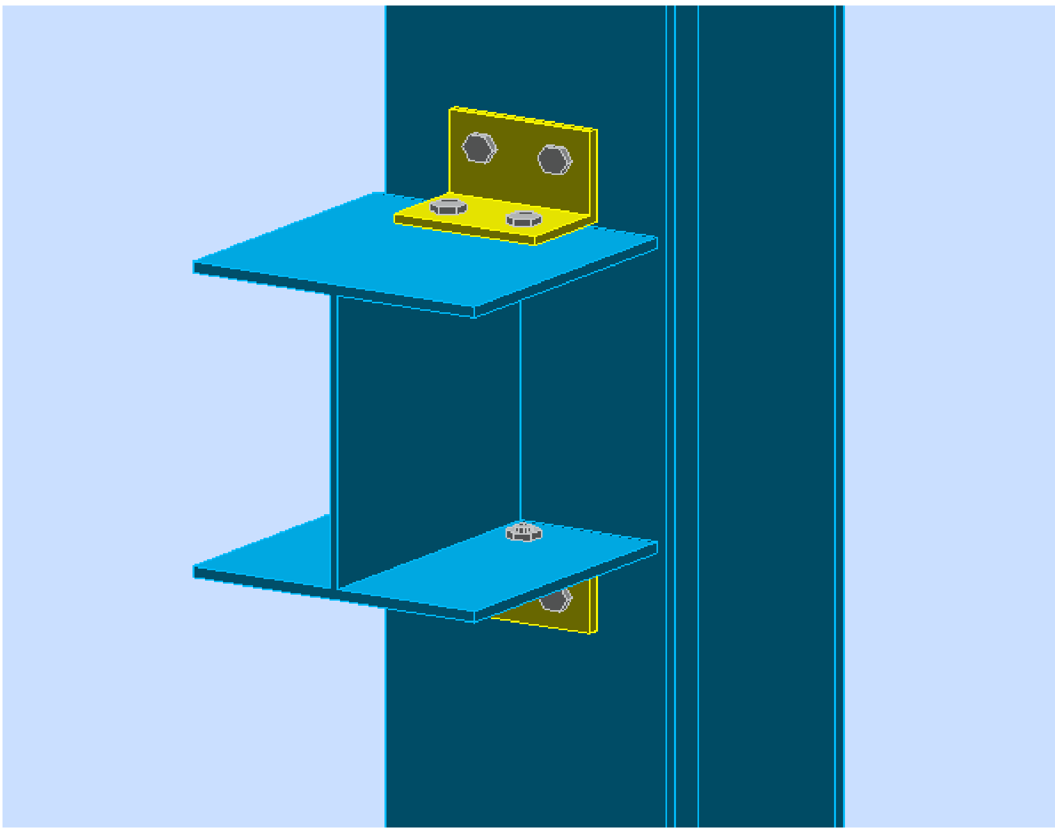


Fig III.11 Flange connection view

III-3-3 . Analysis of a rigid structure without assembly configuration :(rigid by default)

In this case the structure is subjected to a uniformly distributed vertical load on the beam and a horizontal concentrated load at the top left corner node (1) as shown in **Fig III-12.** In this case the beam to column connections are assumed to be fully rigid without any connection configuration. Figure III.12 extracted from our simulation using the Autodesk Robot Structural Analysis V.2012 software shows the application of the loads on the structure.

Table III-1 presents the optimal sections of the beam and column elements obtained after design and verification of the structure analyzed by Autodesk robot software.

Table III-1 : Optimal sections obtained after analysis .

Element number	Profile	Section area [cm^2]
Beam (1)	HEAA 360	106,61
Column (2 , 3)	HEAA 300	112,53

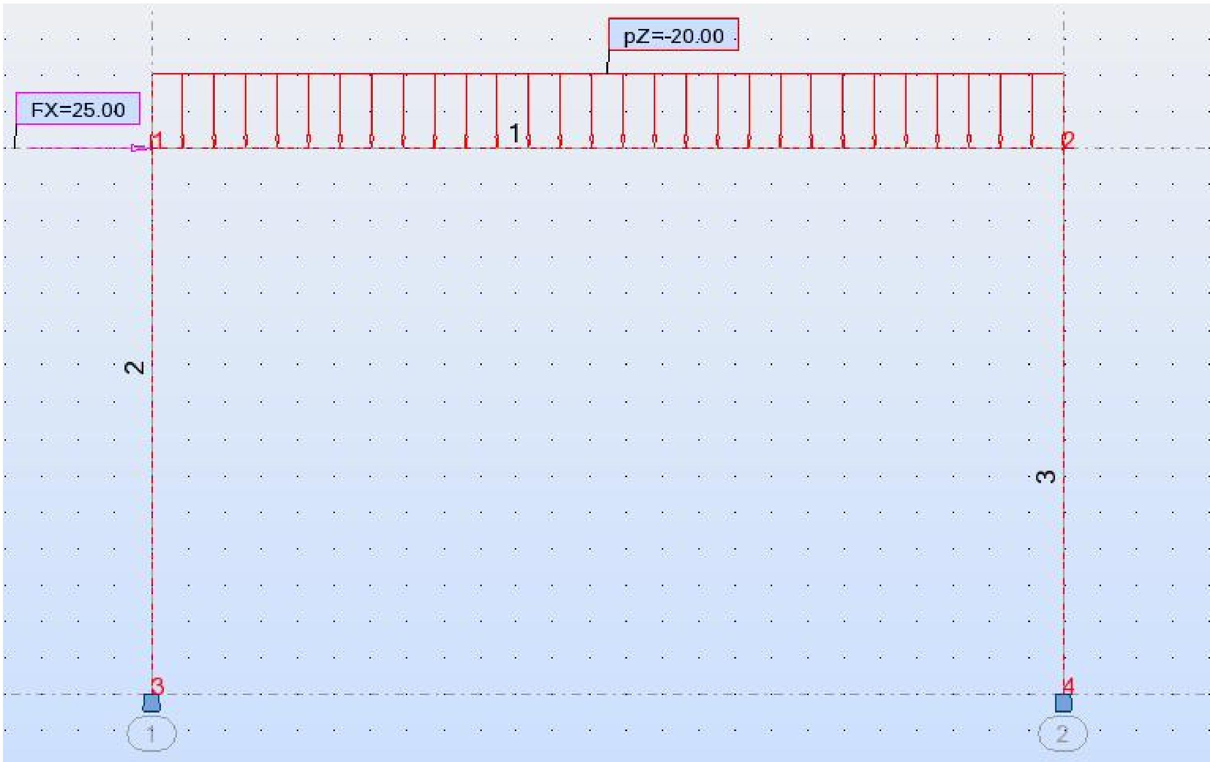


Fig III.12 Vertical and horizontal loadings

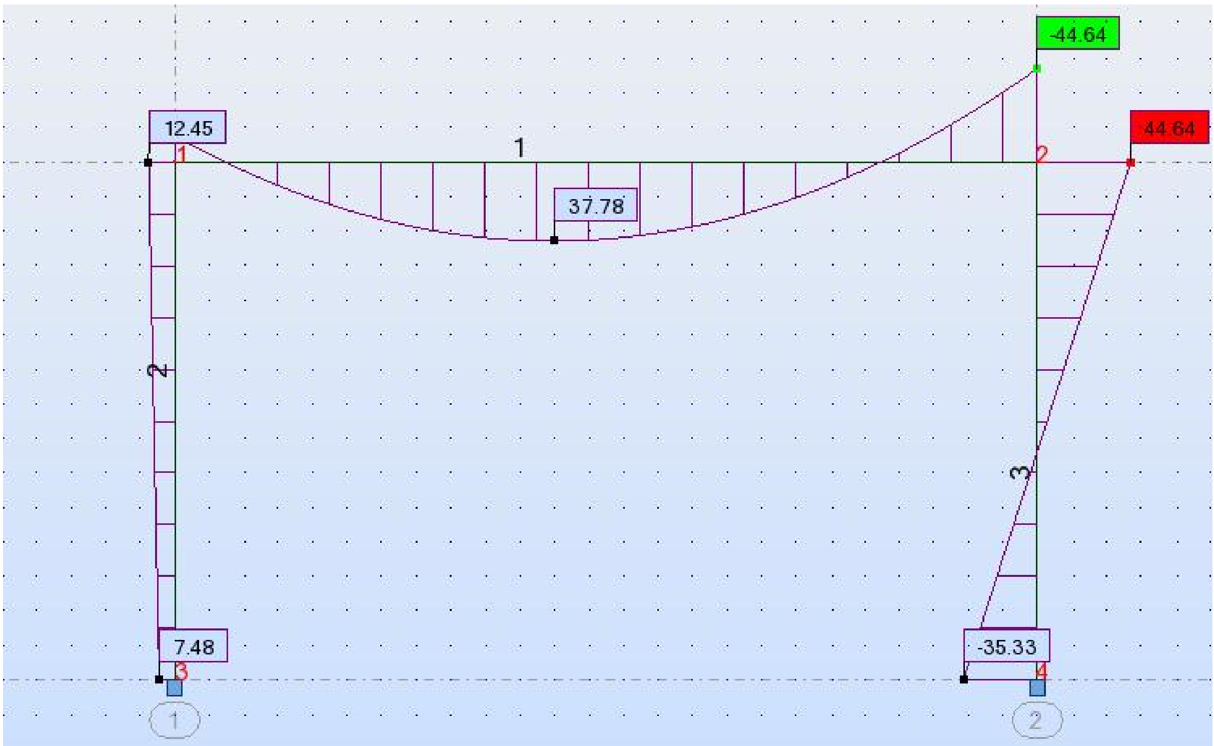


Fig III.13 Diagram of the bending moment(in KN.m) without configuration of the connections

- **Results:**

  - Figure III-13 shows the bending moments diagram as well as the maximum moments in the frame elements. It can be seen that the maximum moment is at the node 2 with a value of **- 44.64 kN.m.**

  - The bending moment in the mid-span is **37.78 (KN.m)** .

III-3-4 . Analysis of the structure with connection configuration :

- In the following analysis ,different geometrical parameters of top and seat angles connection are varied for both nodes 1 and 2 of the frame **(Fig III.12).** The purpose is to analyse the influence of these geometrical parameters on the global response of the frame and make a comparison with the results obtained with a fully rigid connections by default.

✓ **Connection details :**

- Beam section : (HEAA360)
  - Beam height (h) = 339 mm .
  - Beam width (bf) = 300 mm .
  - Moment of inertia of a beam

$$I_y = I_b = 23037{,}40 cm^4$$

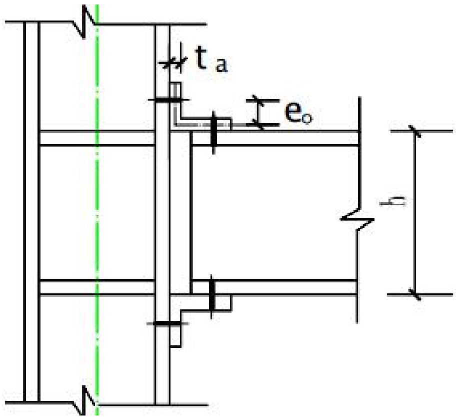


Fig III. 14 Flange connection details

Angle details :

- The angle section (CAE 100×100 × 10) mm
- Moment of inertia of angle : $I_a = \frac{(I_e \times t_a^3)}{12}$.
- Length of the angle (I_e) = 150 mm .
- Bolt location e_0 = 65 mm
- Young's modulus (E) = 2.10 X 10^5 N/mm² .

III-3-4-1 . Influence of the bolt location e_o :

In this study different values of the bolt location are considered in order to show the influence of this parameter on the frame response. Table III. 1 presents different values of the initial stiffness depending on the location of the bolts e_0. These values are introduced in the Autodesk Robot Structural Analysis to illustrate the bending moments of the frame.

Table III. 1 . Values of the initial stiffness corresponding to (e_0) :

e_0 (mm)	$S_{j,ini}$ (kN.m / deg)
40	14290,56730
50	7441,29461
60	4346,48756

- Figure III.14 shows the bending moment diagram for e_0 = 40mm

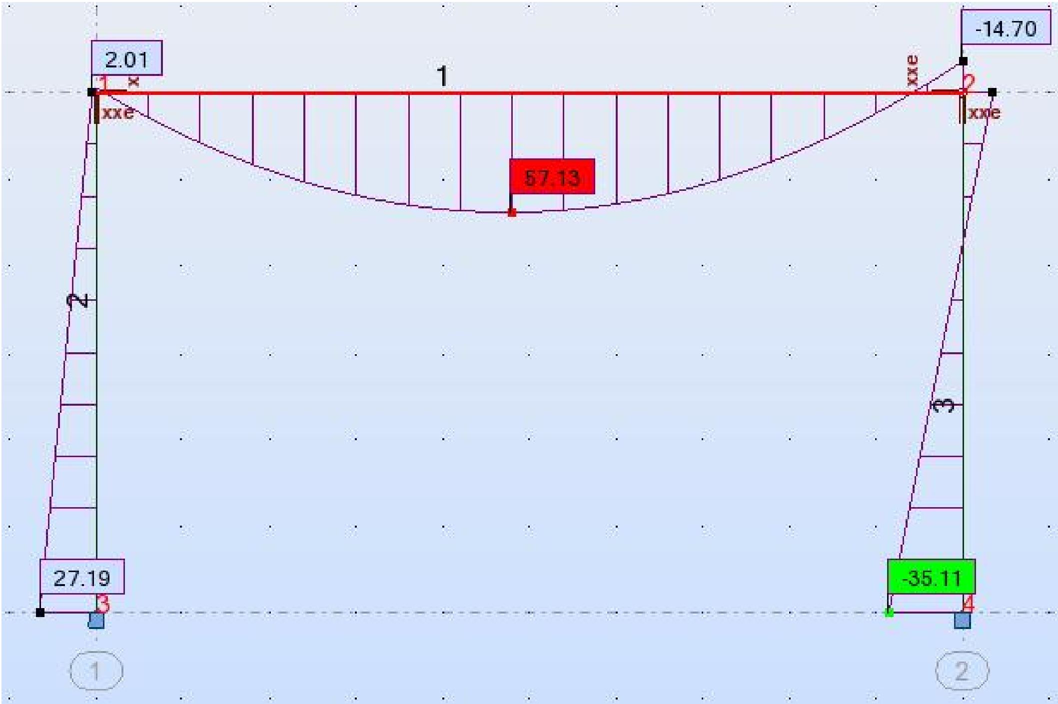


Fig III. 15 Diagram of the bending moment corresponding to (e_0 = 40mm)

- It can be observed from figure III.15 that the maximum bending moment on supports is at the column base at node 4 with a value of -35.11 kN.m, and that the maximum span bending moment is 57.13 kN.m on the beam n ° 1.
- The bending moment of Figure III.16 is concerned with a bolt location e_0 = 50mm :

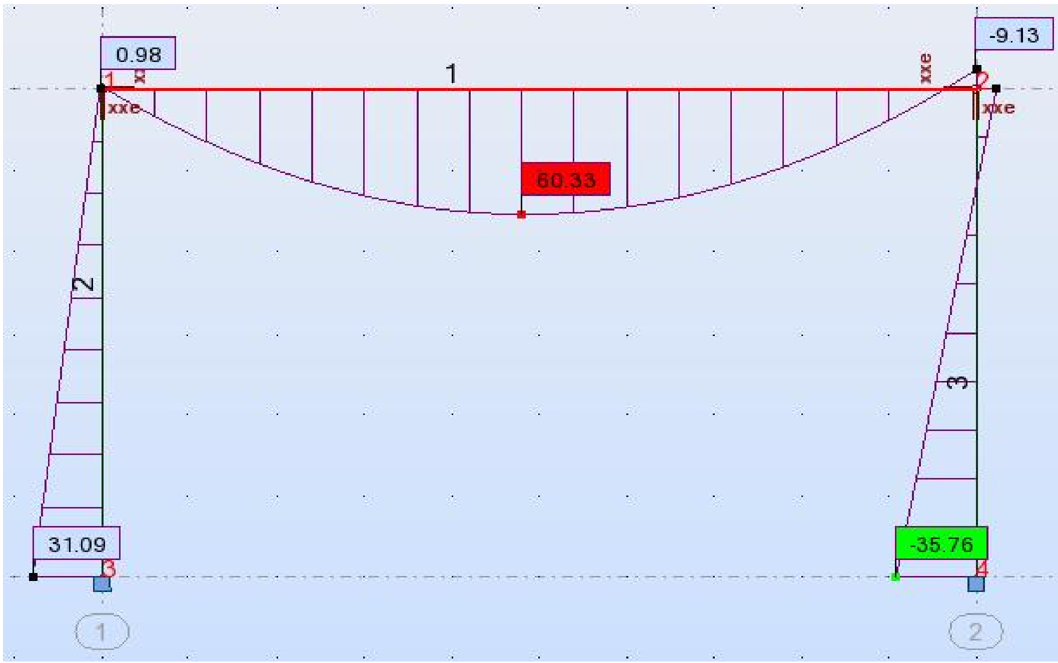


Fig III. 16 Bending moment diagram corresponding to (e_0 = 50mm)

- For the bolt location e_0 = 50mm, figure III.16 shows that the maximum moment on supports is at the column base node 4 and equals -35.76 kN.m. The maximum moment in span of the beam n ° 1 equals 60.33 kN.m.

- Bending moment diagram for e_0 = 60mm:

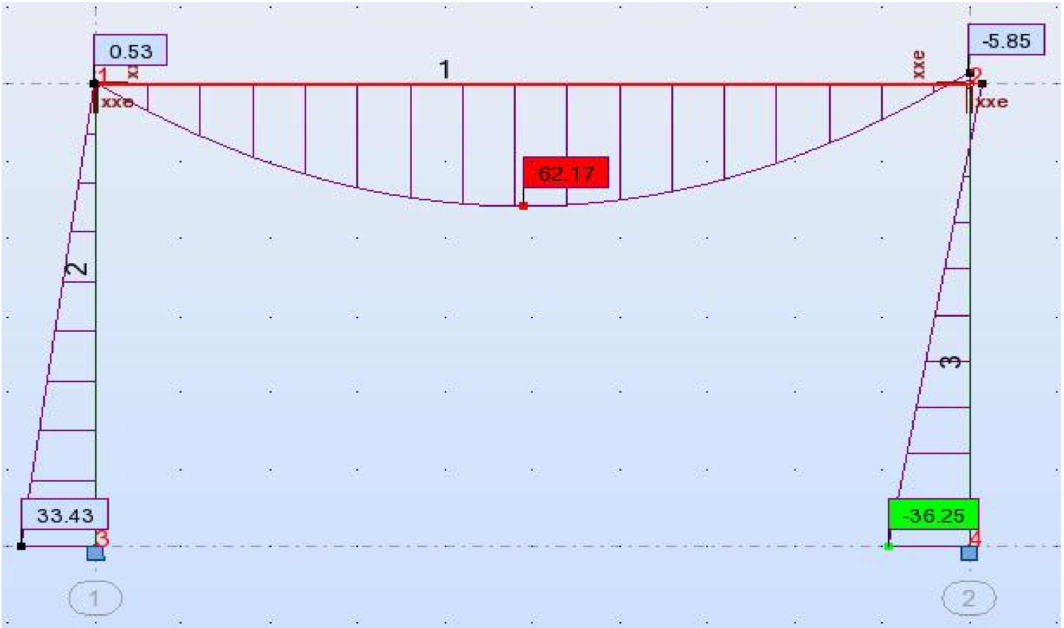


Fig III. 17 Diagram of the bending moment corresponding to (e_0 = 60mm)

- In case where e_0 = 60mm, figure III.17 shows that the maximum moment on supports is at the column base node 4 and has a value of -36.25 kN.m.
- , and that the maximum moment in span is 62.17 KN.m on the beam n ° 1.

III-3-4-2 . Influence of the angle thickness t_a :

The frame also responded when the angle thickness values varied. Table III. 2 shows different values of initial stiffness corresponding on the thickness of the angle t_a , these values are placed in the Autodesk Robot Structural Analysis to illustrate the bending moments of the frame .

Table III. 2 . Values of the initial stiffness corresponding t_a :

t_a (mm)	$S_{j,ini}$ (kN.m / deg)
8	1747,18191
10	3429,38871
12	5946,66827

- Figure III.18 shows the bending moments diagram for $t_a = 8$ mm :

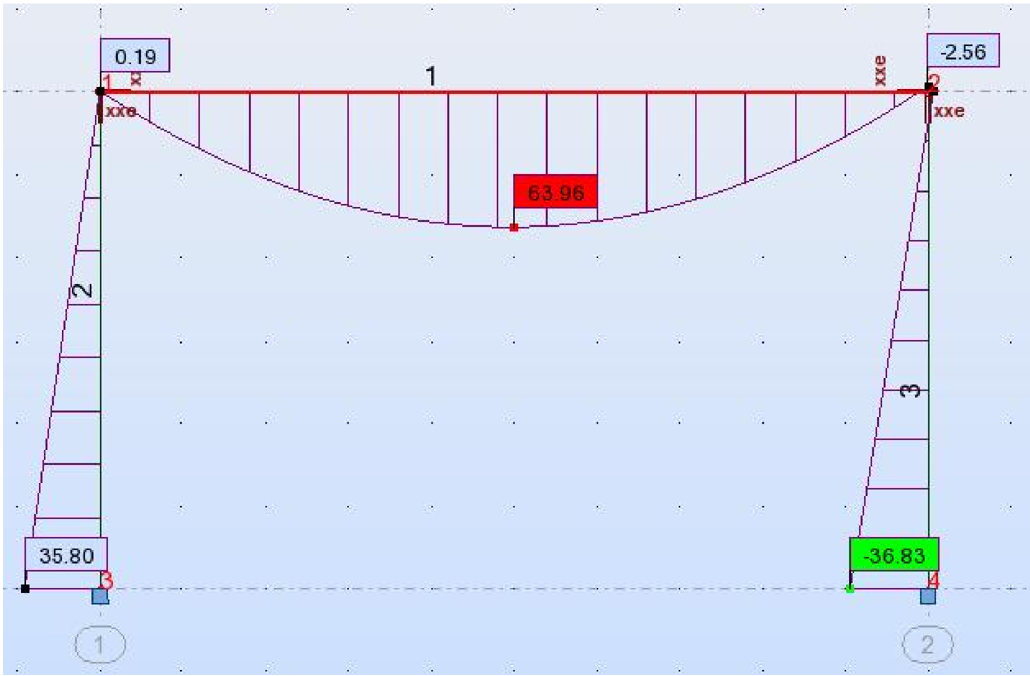


Fig III. 18 Diagram of the bending moment corresponding to (t_a = 8mm)

- It can be observed from figure III.18 that the maximum moment on supports is at the column base node 4 and it's -36.83 KN.m , and that the maximum moment in span is 63.96 KN.m on the beam n ° 1 .

- Figure III.19 is concerned with $t_a = 10mm$:

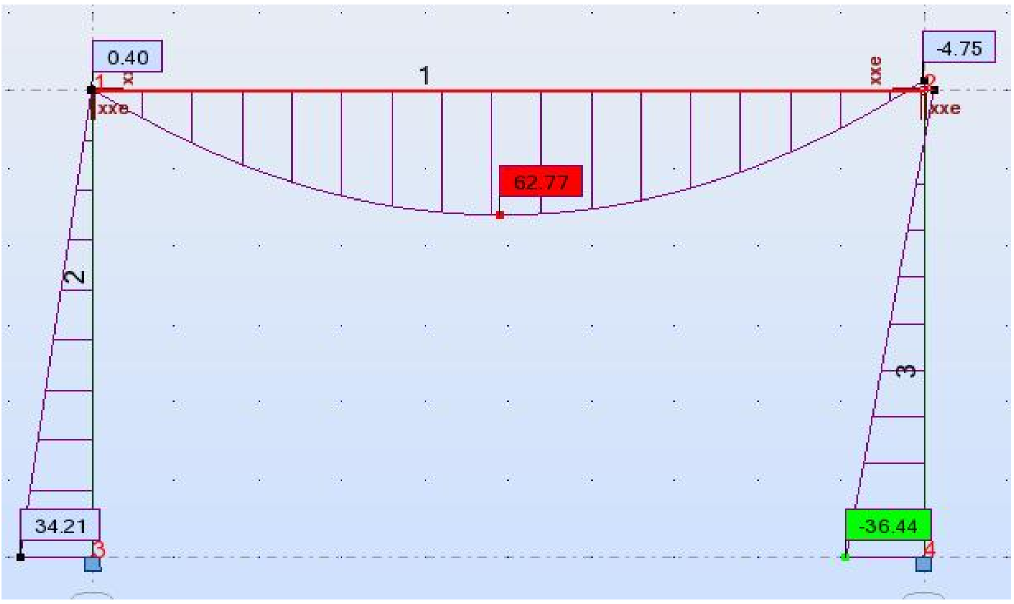


Fig III. 19 Diagram of the bending moment corresponding to (t_a = 10mm)

- For t_a = 10mm ,the Figure III.19 shows that the maximum moment in span is 62.77KN.m , and shows that the maximum moment on supports is at the column base node 4 and it's about -36.44KN.m

- Figure III.20 presents the bending moments for $t_a = 12mm$:

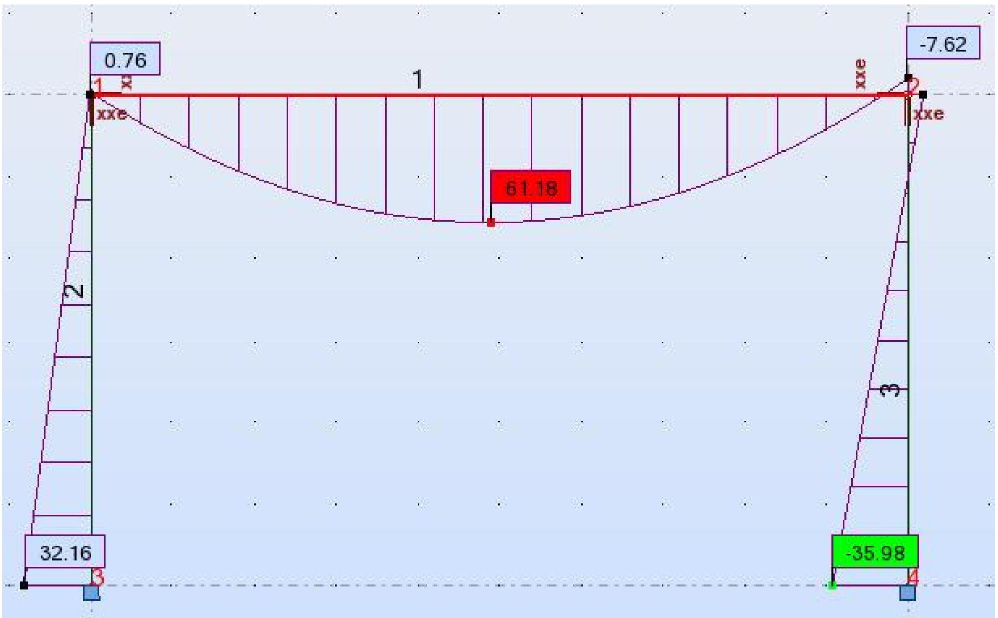


Fig III. 20 Diagram of the bending moment corresponding to (t_0 = 12mm)

- ✓ Figure III.20 shows that the maximum moment on supports for the angle thickness t_a=12mm is at the column base node 4 and it's -35.98 KN.m , and that the maximum moment in span is 61.18 KN.m on the beam n ° 1 .

III-3-5 . Results and discussion:

- As can be seen from figures III.15, III.16 and III.17, that as the value of **(e_0)** increases , the moment in the beam span increases as well . However, the moments at the nodes (1 and 2) decrease..
- It is observed from the figures III.18 , III.19 and III.20, that when the value of **(t_a)** increases, the moments at the beam ends increase .

III-4 . Conclusion :

In this chapter, we first introduced Autodesk Robot Structural Analysis Professional Software, which specializes in modeling and designing different structures and assemblies.

The software presents a completely integrated system intended to model, analyse, and optimise structures of particular types .

From the results obtained in this chapter, we can conclude the following:

- ❖ The location of the bolts e_0 and the angle thickness t_a have great influence on the initial rigidity of the connection and therefore on the bending moment diagrams of the portal frame.
- ❖ The moments at beam-to-column connections decrease as the location of bolts e_0 increases .
- ❖ The same moments at beam-to-column connections decrease when the thickness of the angles decreases . However the mid-span moments dincrease.
- ❖ Top and seat angle connection can be considered as a semi rigid connection and tends to become a pinned connection when the end beam moments take low values .

General Conclusions

General conclusions :

The main purpose of this thesis is to study the global behavior of steel structures with top and seat angle connections and to determine the main characteristics of these assemblies ($S_{j,ini}$).

For the dimensioning and verification of the different structures which are composed of beam and column, subjected to the same type of loading. the calculations required for sizing and stability verification. This difficulty and complexity increases the interest of an automated calculation by Autodesk robot software.

The main results obtained at the end of this work made it possible to conclude the following :

- ❖ A steel structure can only perform its best in sustaining load when the connections between members are designed adequately .
- ❖ The most popular model for structural analysis is the polynomial function proposed by Frye and Morris .
- ❖ Almost in all types of angles connections the initial stiffness's found are in between the initial stiffness boundaries, therefore they are all considered as semi-rigid connections .
- ❖ For the flange connection, when the value of the bolt location (e_0) decreases, the initial stiffness of the connection increases, and tends to become a more rigid connection. However, that drives the internal forces (tension) to the bolts .
- ❖ Also when the value of the angle thickness (ta) increases, the initial stiffness of the top and seat angles connection increases too. Therefore this tends to become stiffer i.e. more rigid.

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ملخص

يتم إجراء الروابط في الهياكل الفولاذية عمومًا عن طريق اللحام و / أو التثبيت. تستخدم روابط الزوايا على نطاق واسع في الهياكل الفولاذية ، ومع ذلك ، فإن السلوك الهيكلي لهذا النوع من الاتصال معقد التحليل للغاية ، يرجع هذا التعقيد إلى تباين خصائصها الهندسية و خصائصها المادية التي تؤدي إلى سلوك يصعب التنبؤ به .الهدف من هذا العمل هو تحليل الهيكل المعدني مع روابط الزاوية وتغيير المعلمات الهندسية لهذه الوصلات الهيكلية بهدف مقارنة مستوى أدائها وتأثيرها على الهيكل. بعد التحجيم والتحقق من الهيكل المتعلق بعدة أنواع مختلفة من روابط الزاوية و التي تخضع لنفس النوع من التحميل ، سيتم تحديد سلوك عزم الدوران لتكوينات مختلفة من روابط الزاوية التي سيتم استخدامها في الهيكل ، بواسطة طريقة نموذج متعدد الحدود . بالإضافة إلى تقييم تقريبي للصلابة الأولية لرابط زاوية استنادًا إلى المعيار الأوروبي " اوروكود 3 " .

كلمات مفتاحية : الصلابة الاولية ، العزم الدوراني ، الروابط ، عنصر الزاوية ، أوتوديسك روبوت

Résumé

Les assemblages dans les structures en acier sont, en règle générale, réalisés par soudure et/ou par boulonnage. L'assemblage par cornières est largement utilisé dans les structures en acier. Toutefois, le comportement structural de ce type d'assemblages est extrêmement complexe à analyser. Cette complexité est due à la variation de leurs propriétés géométriques et matérielles qui aboutit à un comportement difficile à prédire. L'objectif de ce travail est d'analyser une structure métallique avec des assemblages avec cornières de semelles et de faire varier les paramètres géométriques de ces assemblages , dans le but de comparer leur niveau de performance et leur influence sur les réponses de structures . Après le dimensionnement et la vérification de la structure, utilisant différents types d'assemblages soumises au même type de chargement , le comportement moment-rotation des différentes configurations d'assemblages à utiliser dans les structures sera déterminé par la méthode du modèle polynomial, ainsi que l'évaluation de rigidité initiale approximative d'un assemblage avec cornière d'âme et de semelles basée sur la norme européenne "Eurocode3".

Mots clés: Rigidité initiale , Moment-rotation , Assemblage , cornières - Autodesk Robot

Abstract

The connections in steel structures are generally carried out by welding and / or bolting. Angles connections are widely used in steel structures . However, The structural behavior of this type of connection is extremely complex to analyze . This complexity is due to the variation of their geometrical and materials properties which leads to a difficult behavior to predict .The objective of this work is to analyze a metal structure with top and seat angle connections and varying the geometrical parameters of these structure connections with the aim of comparing their level of performance and their influence on the frame responses. After sizing and verification of the structure using different types of connections subjected to the same type of loading, the moment-rotation behavior of the various configurations of assemblies to be used in the structures , will be determined by the method of polynomial model , as well as the evaluation of an approximate initial stiffness of a flange connection based on the European standard "Eurocode3".

Keywords : The initial stiffness - The moment-rotation - Connection - The angle - Autodesk Robot.