ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

FACULTY OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

A COMPUTER PROGRAM FOR ELASTIC ANALYSIS AND DESIGN OF STEEL SPACE FRAME STRUCTURES ACCORDING TO EBCS 3 1995

A thesis submitted to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Structures)

By

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Dedicated to

Temro Mastemar Association

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Abstract

An application program shall be developed for the elastic analysis of steel space frame structures and design of the structure members. Matrix method for the analysis of the structure and EBSC 3 1995 specifications for the design of members will be implemented. Subsequently a small application program which provides user interfaces in the input of data, analysis of the structure and design of members will be generated.

Chapter 1 INTRODUCTION

1.1. General

The primary function of any structure is to support and transfer externally applied loads to the reaction points while at the same time being subjected to some specified constraints and a known temperature distribution. In most civil engineering structures, the reaction points are those points on the structure which are attached to a rigid foundation.

The structural designer is, therefore, concerned with the analysis of known structural configurations which are subjected to known distributions of static or dynamic loads, displacements, and temperatures. Furthermore, the structural designer is also required to perform the most efficient design (optimum design) for the set of specified loads. Consequently, the ultimate objective in structural design should not be limited to the analysis of a given structural configuration but must be followed by the most efficient design for specified design criteria (*Reference No. 2, 4, 10, 11, 13, 14, 15 and 18*).

Analysis of structures predicts the behavior of a structure in its environment, including mainly the assessment of the force actions and deformations induced by the load system. It is required selection of configuration, member sizes and materials to be performed in the most efficient design of the structure (*Reference No. 1, 7, 8, 9, 12, 13 and 17*).

1.2. Objective of the study

Recent advances in structural technology have required greater accuracy and speed in the analysis of structural systems. The requirement of accuracy in analysis has been brought about by a need for demonstrating structural safety. Consequently, accurate methods of analysis had to be developed since the conventional methods, although perfectly satisfactory when used on simple structures, have been found inadequate when applied to complex structures.

The requirement of speed, on the other hand, is imposed by the need of having comprehensive information on the structure sufficiently early in the design cycle so that any structural modifications deemed necessary can be incorporated before the final design is decided upon.

Furthermore, in order to achieve the most efficient design a large number of different structural configurations may have to be analyzed rapidly before a particular configuration is selected for detailed study.

In efforts made to satisfy the need of accuracy and speed, a vast amount of data and enormous computational programs of work, in valuing large number of simultaneous equations are faced. Nowadays these can be handled more easily with digital computers. Practicing engineers combine the analysis and design of various structures using graphics with the help of computer programs. The accuracy and power that practicing engineers acquired to perform various design and analysis works using application softwares drive them to use it exceedingly in their day to day life.

In our country, many practicing engineers make use of available application softwares for analysis and design of both concrete and steel structures. These softwares are based on various national standards. These include European, German, British and American codes, among others.

The softwares that are currently used by our practicing engineers do not consider local conditions. This coerces these engineers to use other countries building code standards and thus they may not be able to incorporate local condition that should be included especially in the design of structures. The need to have application software which includes local conditions and building code standards become unquestionable.

Therefore, the aim of this study is to develop an application program for the elastic analysis of a steel space frame structures and design of members of the structure that takes national code provisions and utilizes EBCS 3 1995 (EADoSSF).

Chapter 2 ANALYSIS OF STRUCTURES

2.1. General

The structural analysis deals with the development of suitable arrangement of structural elements for the structures to support the external loads or the various critical combinations of the loads which are likely to act on the structure. The analysis also deals with the determination of internal forces in the various members (namely axial forces, bending moments and shear forces), state of stresses or critical combination of the stresses of the various points (which includes the nature, magnitude and direction of these stresses) and the external reactions due to the worst possible combination of the loads. The external reactions are transmitted to the foundation. The methods of structural analysis and the principles involved in them remain independent of the materials used for all types of structures, whether the structures are built of plastics, aluminum, timber, reinforced concrete or steel.

The structural analysis concept adopted in the study will be explained in this chapter.

2.2. Matrix method of structural analysis

As it is mentioned in chapter one the need for accuracy and speed in the analysis of structural systems is an issue due to advances in structural technology.

The method of analysis which meets the requirement mentioned above uses matrix algebra, which is ideally suited for automatic computation on high-speed digital computers. The scope and power of matrix methods have been brought out by the formulation of general matrix equations for the analysis of complex structures. In these methods the digital computer is used not only for the solution of simultaneous equation but also for the whole process of structural analysis from the initial input data to the final output, which represents stress and force distribution, and deflections.

Matrix methods are based on the concept of replacing the actual continuous structure by a mathematical model made up from structural elements of finite size having known elastic and inertial properties that can be expressed in matrix form (*Reference No. 1, 2 and 8*). The matrices representing these properties are considered as building blocks, which, when fitted together according to a set of rules derived from the theory of elasticity, provide static and dynamic properties of the actual structural system.

The flexibility method (also called force method) and the stiffness method (also called displacement method) use matrix formulation in the solution of the set of simultaneous algebraic equation. The stiffness method is more suitable for computer programming.

In the stiffness method, actions are expressed in terms of displacements. The unknown quantities are the joint displacements in the structure.

In matrix method of structural analysis, the expressions involving actions in terms of displacements are formulated for individual members of the structural system with the help of superposition. These expressions should be assembled for the whole structure from the contributions of individual members by a formal matrix multiplication procedure. This matrix multiplication procedure involves a large and sparse compatibility matrix-containing terms that are easy to evaluate correctly. Further more, neither the generation of this matrix nor the multiplication process for assembly would be suitable for computer programming. A better methodology incorporating drawn ideas from superposition and formal matrix multiplication and adding a few computer-oriented techniques is known as the direct stiffness method (*Reference No. 8*).

Direct stiffness method

In the analysis of space rigid-jointed frames, the following assumptions are made (*Reference No. 8*).

1. There are no restrictions on the location of joints, directions of members, or direction of loads.
2. The individual members may carry internal axial forces, torsional moments, bending moments in both principal directions of the cross-section, and shearing forces in both principal directions.
3. The members are assumed to have two axes of symmetry in the cross-section so that bending and torsion occur independently of one another.
4. Each member has a straight axis and uniform cross-section throughout its length.
5. All members behave in a linear elastic manner.
6. Changes of geometry are ignored due to sufficiently small deflection in the structure.

7. The members of the frame are assumed to be rigidly connected at the joints, and each joint that is not restrained is assumed to translate and rotate in a completely general manner in space.

Steps in direct stiffness method (*Reference No. 8*)

1. *Idealizing the actual structure*

   In this step, information such as number of members, number of joints and number of degrees of freedom will be determined. It is done through idealizing the actual structure in a mathematical model. Furthermore, the elastic properties of the material, the location of joints of the structure, the section properties of each member in the structure and the condition of restraint at the supports of the structure must be specified and identified

2. *Constructing stiffness matrix*

   The stiffness matrix for individual member will be formed by setting up the force deflection of each member. Individual member stiffness matrices summation results formation of joint stiffness matrix. It is related to all possible joint displacements, including a support displacement which is called the over-all joint stiffness matrix.

3. *Formation of combined load vector*

   The joint load vector and the equivalent load vector will be formed and added to form combined load vector.

4. *Computation of results*

   In the final phase of the analysis all of the joint displacements, reactions, and member end-actions are computed. The calculation of member end-actions proceeds member by member instead for the structure as a whole.

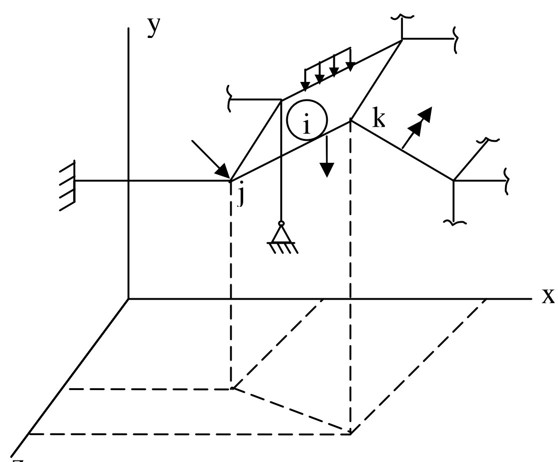

Fig. 1: Space frame

The above procedure is explained with the following example. Consider a typical space frame member i having a positive direction cosines as indicated in *Fig. 2(a)* with joints at its ends denoted by j and k.

In *Fig. 2(a)*, the x_m axis is taken along the axis of the member. The y_m and z_m axis are selected as the principal axes of the cross section at end j of the member. The complete 12 x 12 stiffness matrix $\mathbf{S_{Mi}}$ for the member axes is as shown below.

$$S_{Mi}=\frac{E}{L^3}\begin{array}{cc} & \begin{array}{cccccccccccc} 1 & 2 & 3 & 4 & 5 & 6 & 7 & 8 & 9 & 10 & 11 & 12 \end{array} \\ \begin{array}{c} 1\\2\\3\\4\\5\\6\\7\\8\\9\\10\\11\\12 \end{array} & \begin{bmatrix} A_xL^2 & 0 & 0 & 0 & 0 & 0 & -A_xL^2 & 0 & 0 & 0 & 0 & 0 \\ 0 & 12I_z & 0 & 0 & 0 & 6I_zL & 0 & -12I_z & 0 & 0 & 0 & 6I_zL \\ 0 & 0 & 12I_y & 0 & -6I_yL & 0 & 0 & 0 & 12I_y & 0 & -6I_yL & 0 \\ 0 & 0 & 0 & GI_xL^2 & 0 & 0 & 0 & 0 & 0 & -6I_xL^2 & 0 & 0 \\ 0 & 0 & 6I_yL & 0 & 4I_yL^2 & 0 & 0 & 0 & 6I_yL & 0 & 2I_yL^2 & 0 \\ 0 & 6I_zL & 0 & 0 & 0 & 4I_zL^2 & 0 & -6I_zL & 0 & 0 & 0 & 2I_zL^2 \\ -A_xL^2 & 0 & 0 & 0 & 0 & 0 & A_xL^2 & 0 & 0 & 0 & 0 & 0 \\ 0 & 12I_z & 0 & 0 & 0 & -6I_zL & 0 & 12I_z & 0 & 0 & 0 & -6I_zL \\ 0 & 0 & -12I_y & 0 & GI_yL & 0 & 0 & 0 & 12I_y & 0 & 6I_yL & 0 \\ 0 & 0 & 0 & -GI_xL^2 & 0 & 0 & 0 & 0 & 0 & GI_xL^2 & 0 & 0 \\ 0 & 0 & -6I_yL & 0 & 2I_yL^2 & 0 & 0 & 0 & 6I_yL & 0 & 4I_yL^2 & 0 \\ 0 & 6I_zL & 0 & 0 & 0 & 2I_zL^2 & 0 & -6I_zL & 0 & 0 & 0 & 4I_ZL^2 \end{bmatrix} \end{array} \quad (2.1)$$

This stiffness matrix must be transformed in to a matrix S_{MSi} in the directions of structure axes by means of a notation transformation matrix. The notation matrix $\mathbf{R_T}$ is given by:

$$\mathbf{R}_T=\begin{bmatrix} \mathbf{R} & \mathbf{0} & \mathbf{0} & \mathbf{0} \\ \mathbf{0} & \mathbf{R} & \mathbf{0} & \mathbf{0} \\ \mathbf{0} & \mathbf{0} & \mathbf{R} & \mathbf{0} \\ \mathbf{0} & \mathbf{0} & \mathbf{0} & \mathbf{R} \end{bmatrix} \quad (2.2a)$$

$$\text{where } \mathbf{R}=\begin{bmatrix} C_X & C_Y & C_Z \\ \dfrac{-C_XC_Y\cos\alpha - C_Z\sin\alpha}{C_{XZ}} & C_{XZ}\cos\alpha & \dfrac{-C_YC_Z\cos\alpha + C_X\sin\alpha}{C_{XZ}} \\ \dfrac{C_XC_Y\sin\alpha - C_Z\cos\alpha}{C_{XZ}} & -C_{XZ}\sin\alpha & \dfrac{C_YC_Z\sin\alpha + C_X\cos\alpha}{C_{XZ}} \end{bmatrix} \quad (2.2b)$$

$$C_X=\frac{x_k - x_j}{L} \quad (2.2c)$$

$$C_y=\frac{y_k - y_j}{L} \quad (2.2d)$$

$$C_z=\frac{z_k - z_j}{L} \quad (2.2e)$$

$$C_{xz} = \sqrt{C_x^2 + C_z^2} \qquad (2.2f)$$

$L = \sqrt{(x_k - x_j)^2 + (y_k - y_j)^2 + (z_k - z_j)^2}$ (2.2g) and x_k, y_k and z_k are member i coordinates of end k; x_j, y_j and z_j are member i coordinates of end j; L is the length of the member and α is an angle measured (in the positive sense which is counter clockwise direction) from that plane to one of the principal axes of the cross-section.

Once the member stiffness matrix $\mathbf{S_{Mi}}$ and the rotation matrix formed as mentioned above, the member stiffness matrix about the structure axis may be computed by the usual matrix multiplications:

$$\mathbf{S_{Ms}} = \mathbf{R_T}^T \mathbf{S_M} \mathbf{R_T} \qquad (2.3)$$

The framed structure members and joints are numbered in arbitrary sequence, but each member and each joint must have a number. The members are numbered consecutively 1 through m, and the joints are numbered consecutively 1 through n_j. The number of degrees of freedom n in a space frame may be determined from the number of joints n_j and the number of restraints n_r as

$$n = n_j - n_r \qquad (2.4)$$

For a member i in space frame having joint numbers j and k at its ends, the six possible displacements at a joint j are:

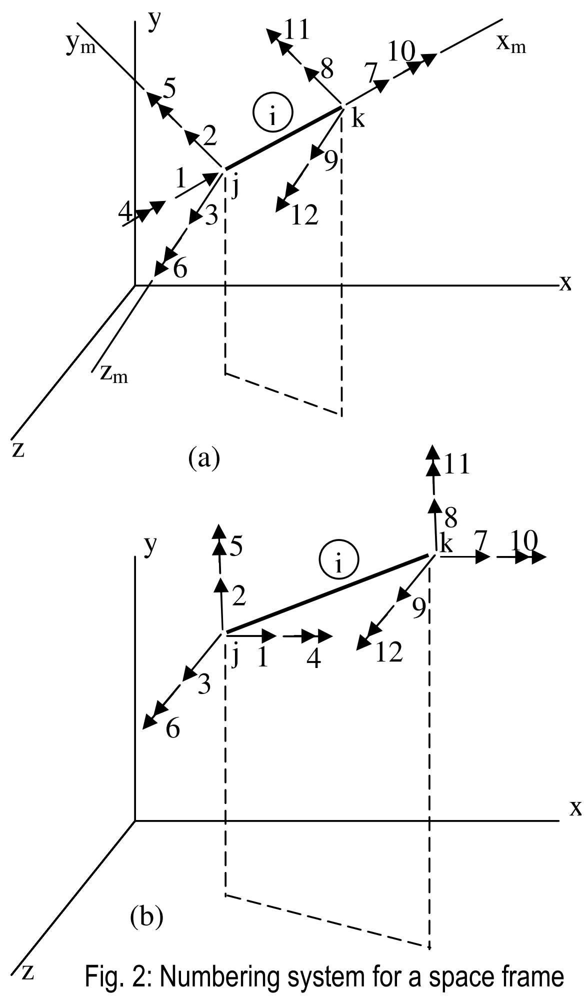

Fig. 2: Numbering system for a space frame

j1 = 6 x j – 5, is index for translation in the x-direction;

j2 = 6 x j – 4, is index for translation in the y-direction;

j3 = 6 x j – 3, is index for translation in the z-direction;

j4 = 6 x j – 2, is index for rotation in the x sense;

j5 = 6 x j – 1, is index for rotation in the y sense; and

j6 = 6 x j, is index for rotation in the z sense.

Like wise at a joint k

k1 = 6 x k – 5,	k4 = 6 x k – 2,
k2 = 6 x k – 4,	k5 = 6 x k – 1,
k3 = 6 x k – 3,	k6 = 6 x k.

In the construction of the joint stiffness matrix in an orderly fashion; first the 12 x 12 stiffness matrix $\mathbf{S_{MSi}}$, for structure axes is generated for i-th member in the frame as shown above. Member i contributes to the stiffness of joints j and k at the ands of the member. Therefore, appropriate elements from the matrix $\mathbf{S_{MSi}}$ for this member may be transferred to the over all joints stiffness matrix $\mathbf{S_J}$ through an organized handling of subscripts. For example, the first column in the matrix $\mathbf{S_{MSi}}$ contributes to the joint stiffness matrix $\mathbf{S_J}$ as follows.

$$\left.\begin{array}{ll}
(S_J)_{j1,j1} = \sum S_{Ms} + (S_{MS1,1})_i & (S_J)_{k1,j1} = (S_{MS7,1}) \\
(S_J)_{j2,j1} = \sum S_{Ms} + (S_{MS2,1})_i & (S_J)_{k2,j1} = (S_{MS8,1}) \\
(S_J)_{j3,j1} = \sum S_{Ms} + (S_{MS3,1})_i & (S_J)_{k3,j1} = (S_{MS9,1}) \\
(S_J)_{j4,j1} = \sum S_{Ms} + (S_{MS4,1})_i & (S_J)_{k4,j1} = (S_{MS10,1}) \\
(S_J)_{j5,j1} = \sum S_{Ms} + (S_{MS5,1})_i & (S_J)_{k5,j1} = (S_{MS11,1}) \\
(S_J)_{j6,j1} = \sum S_{Ms} + (S_{MS6,1})_i & (S_J)_{k6,j1} = (S_{MS12,1})
\end{array}\right\} \quad (2.5)$$

Eleven other sets of expressions similar to the above equation may be written to make a total of twelve sets of equations. The next step is forming the joint load vector $\mathbf{A_J}$ and transferring the equivalent load vector, $\mathbf{A_E}$ formed for member axis to structure axes. Following the calculation of $\mathbf{A_E}$, this vector is added to the vector $\mathbf{A_J}$ to form the combined load vector $\mathbf{A_C}$.

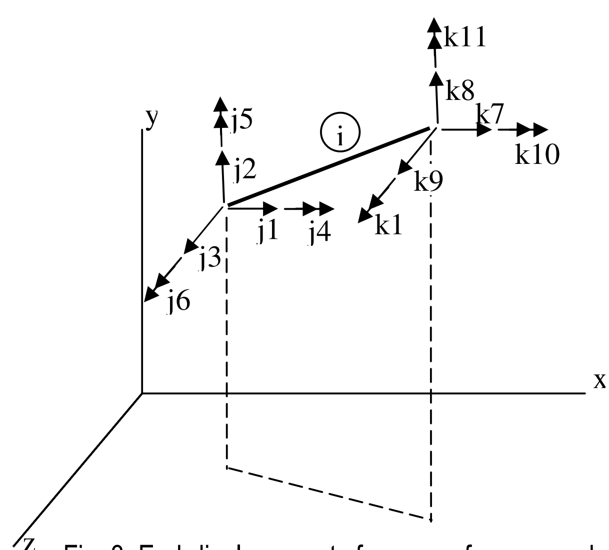

Fig. 3: End-displacements for space frame member

The relationship between end actions and end displacement for the space frame element i in a compact form is

$$\mathbf{A_J} = \mathbf{S_J D_J} \quad (2.6)$$

When it is expanded through partitioning the joint stiffness matrix $\mathbf{S_J}$ in to sub-matrices pertaining to the free joint displacements $\mathbf{D_F}$, the support displacements $\mathbf{D_R}$, and their corresponding actions, $\mathbf{A_F}$ and $\mathbf{A_R}$, it will have a form of

$$\begin{Bmatrix} \mathbf{A_F} \\ \mathbf{A_R} \end{Bmatrix} = \begin{bmatrix} \mathbf{S_{FF}} \mathbf{S_{FR}} \\ \mathbf{S_{RF}} \mathbf{S_{RR}} \end{bmatrix} \begin{Bmatrix} \mathbf{D_F} \\ \mathbf{D_R} \end{Bmatrix} \tag{2.7}$$

Then solve for free joint displacements $\mathbf{D_F}$ (translations and rotations in the x, y, and z directions), using

$$\mathbf{D_F} = \mathbf{S_{FF}}^{-1} \mathbf{A_{FC}} \tag{2.8}$$

where $\mathbf{A_{FC}}$ is a vector of combined joint loads corresponding to $\mathbf{D_F}$.

$$\mathbf{A_R} = -\mathbf{A_{RC}} + \mathbf{S_{RF}} \mathbf{D_F} \tag{2.9}$$

Equation (2.9) will be used to solve for support reactions. And finally the member end-actions for each member are computed by substituting the member stiffness matrix $\mathbf{S_{Mi}}$ for member axes and the appropriate form of the rotation transformation matrix $\mathbf{R_{Ti}}$ into the following equation:

$$\mathbf{A_{Mi}} = \mathbf{A_{MLi}} + \mathbf{S_{Mi}} \mathbf{R_{Ti}} \mathbf{D_{ji}} \tag{2.10}$$

Chapter 3 STRUCTURAL MEMBERS DESIGN

3.1. General

The structural design deals with the selection of proper material, proper sizes, proportions and shape of each member and its connecting details. The selection is such that it is economical and safe, satisfies all the stress requirements imposed by the most sever combination of the loads to which the structure is required to transmit or resist including its self-weight. The structural design in a limited sense also deals with the design of various parts or member of a structure.

Among the different methods of design, the limit state design method is adopted for this study.

This chapter describes the method used to design members and the application of this method for member design as follows.

A structure or part of a structure is considered unfit for use when it exceeds a particular state, called a *limit state*, beyond which it infringes one of the criteria governing its performance or use.

The limit states as per EBCS 3 1995 are categorized in to two states (*Reference No. 3*).

a) *The ultimate limit states*: These are associated with collapse or with other forms of structural failure which may endanger the safety of people. States prior to structural collapse which for simplicity are considered in place of the collapse itself are also treated as ultimate limit states. Loss of equilibrium of a part or the whole of the structure which is considered as a rigid body; and failure by excessive deformation, rupture or loss of stability of the structure or any part of it including supports and foundations are the considerations that must be included in this limit states.

b) *The serviceability limit states*: These correspond to states beyond which specified service requirements are no longer met. In this limit states consideration includes:

- *Deformations or deflections* which affect the appearance or effective use of the structure or cause damage to finishes of non-structural elements.

- *Vibration*, which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness.

All relevant limit state shall be considered in the design so as to ensure an adequate degree of safety and serviceability. The usual approach will be to design on the basis of the most critical limit state and then to check that the remaining limit states will not be reached.

A limit state is defined as a condition in which a structure or structural component becomes unsafe (that is, a violation of the strength limit state) or unsuitable for its intended function (that is, a violation of the serviceability limit state). In a limit state design, the structure or structural component is designed in accordance to its limits of usefulness, which may be strength-related or serviceability related.

In the limit state format as is adapted in the EBCS 3 1995, mathematically, this design concept is expressed as:

$$E_d \leq C_d \qquad (3.1)$$

where E_d is the design value of the particular effect of actions being considered and C_d is the design capacity of the effect of actions.

3.2. Design of steel beams

A beam, otherwise flexural member, is defined as a structural member subjected to transverse loads. The plane of transverse load is parallel to the plane of symmetry or the cross-section of the beam and it passes through the shear center, so that simple bending occurs. The transverse loads produce bending moments and shear forces in the beams at all the section of the beam.

According to the width thickness ratios of the component elements steel sections used for flexural members are classified into a member of classes depending on the standard specification implemented for design. Accordingly EBCS 3 1995 classifies flexural members into four classes which are:

Class 1 or Plastic sections: develop their plastic moment resistance with the rotation capacity required for plastic analysis and may only be used for plastic design.

Class 2 or Compact sections: develop their plastic moment resistance but with limited rotation capacity.

Class 3 or Semi-Compact sections: those which can reach their ‘yield’ moment resistance but local buckling prevents the development of the plastic moment resistance.

Class 4 or thin-walled sections: those which contain thin walled elements subject to compression due to moment or axial force. Local buckling may prevent the stress in a thin-walled section from reaching the design strength.

According to EBCS 3 1995 the classification of sections depends on the classification of flange and web elements, the classification also depends on whether the compression elements are in pure compression, pure bending or under the influence of combined axial force and bending.

Design criteria

For establishing the moment resistance of flexural members, the criteria that should be considered are:

i. Yielding of the cross section or its flexural strength;
ii. Local buckling (this applied only for class 4 sections);
iii. Lateral torsional buckling;
iv. Shear strength including shear buckling ;
v. Local strength at points of loading or reaction, i.e. criteria for concentrated loads and with respect to serviceability limits states.; and
vi. Deflection criterion.

Beams are to be designed in such a way that both the cross-sections resistance to applied loads be established and member capacity verified against possible buckling failures.

Some EBCS 3 1995 design criteria’s (*Reference No. 3*)

Cross-section resistance to pure bending

Members designed to resist a factored uni-axial bending M_{Sd}, calculated using appropriate load combinations, must satisfy the condition

$$M_{Sd} \leq M_{C,Rd} \qquad (3.2)$$

where $M_{C,Rd}$ is the design moment resistance of the cross-section in the absence of fasteners hole. It is taken as the smallest of:

a) Design plastic resistance moment of the gross section $M_{Pl,Rd}$

$$M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{Mo}} \text{ for class 1 or 2 cross-sections} \tag{3.3a}$$

b) Design elastic resistance moment of the gross section $M_{el,Rd}$

$$M_{el,Rd} = \frac{W_{el} f_y}{\gamma_{Mo}} \text{ for class 3 cross-sections and} \tag{3.3b}$$

c) Design local buckling resistance moment of the gross section, $M_{O,Rd}$

$$M_{o,Rd} = \frac{W_{eff} f_y}{\gamma_{M1}} \text{ for class 4 cross-sections} \tag{3.3c}$$

where W_{pl} is plastic section modulus, W_{el} is elastic section modulus, W_{eff} is effective section modulus, f_y yield strength of the section and γ_m is partial safety factor, where $\gamma_{mo} = \gamma_{m1} = 1.1$.

Resistance to plastic shear

The shear resistance is limited by the shear plastic resistance, $V_{Rd.}$

Members designed to resist a factored shear force, V_{Sd}, calculated using appropriate load combinations, must satisfy the condition:

$$V_{SR} \le V_{pl,Rd} \tag{3.4a}$$

where $V_{pl,Rd}$ is the plastic shear resistance to a cross-section given by:

$$V_{pl,Rd} = \frac{A_V \left(f_y / \sqrt{3}\right)}{\gamma_{Mo}} \tag{3.4b}$$

in which A_v is the shear area, normally given by $h \times t_w$ where h is the overall depth of the web and t_w is the web thickness.

Shear buckling resistance

According to EBCS 3 1995 of the simple post-critical method, the design shear buckling resistance, $V_{b,Rd}$, may be obtained from:

$$V_{ba,Rd} = \frac{d t_w \tau_{ba}}{\gamma_{M1}} \tag{3.5}$$

where τ_{ba} is the simple post-critical shear strength which is obtained from;

$$\left.\begin{aligned}\tau_{ba} &= \left(f_{yw}/\sqrt{3}\right) && \text{if } \bar{\lambda}_w \le 0.8\\ \tau_{ba} &= \left[1-0.625\left(\bar{\lambda}_w - 0.8\right)\right]\left(f_{yw}/\sqrt{3}\right) && \text{if } 0.8 < \bar{\lambda}_w < 1.2\\ \tau_{ba} &= \left(0.9/\bar{\lambda}_w\right)\left(f_{yw}/\sqrt{3}\right) && \text{if } \bar{\lambda}_w > 1.2,\end{aligned}\right\} \quad (3.6a)$$

in which $\bar{\lambda}_w$ is the web slenderness given by

$$\bar{\lambda}_w = \sqrt{\left[\left(f_{yw}/\sqrt{3}\right)/\tau_{Cr}\right]} = \frac{d/t_w}{37.4\varepsilon\sqrt{k_\tau}} \quad (3.6b)$$

τ_{cr} = the elastic critical shear strength

k_τ = 5.34 if transverse stiffness are arranged at supports only;

k_τ = 4 + 5.34 $(a/d)^2$ if transverse stiffness are arranged at and between supports and if a/d < 1; and

k_τ = 5.34 + 4 $(a/d)^2$ if transverse stiffness are arranged at and between supports and if a/d ≥ 1;

a is the distance between transverse stiffness.

Lateral torsional buckling of beams

If the member bending only takes place about the minor axis, or if beams laterally restrained throughout their length by adequate bracing or if the lateral slenderness parameter $\bar{\lambda}_{LT} < 0.3$, then the possibility of lateral torsional bucking may not be considered. Otherwise, the design bucking resistance moment of a laterally unrestrained beam shall be taken as:

$$M_{b,Rd} = \frac{\chi_{LT}\beta_w W_{pl,y} f_y}{\gamma_{M1}} \ge M_{y,Sd} \quad (3.7)$$

where $\beta_w = 1$ for class 1 or 2 cross-sections ;

$= \dfrac{W_{el,y}}{W_{pl,y}}$ for class 3 cross-sections;

$= \dfrac{W_{eff,y}}{W_{pl,y}}$ for class 4 cross-sections.

$W_{pl,y}$ is plastic modulus of cross-section about the major axis,

$W_{el,y}$ is elastic modulus of cross-section about the major axis,

$W_{eff,y}$ is elastic modulus of effective cross-section about the major axis.

χ_{LT} is a reduction factor accounting for lateral torsional buckling and given by:

$= 1$ for $\bar{\lambda}_{LT} \le 0.4$

$= \dfrac{1}{\phi_{LT} + \left[\phi_{LT}^2 - \bar{\lambda}_{LT}^2\right]^{0.5}}$ for $\bar{\lambda}_{LT} > 0.4$, but $\chi_{LT} \le 1$

$$\phi_{LT} = 0.5\left[1+\alpha_{LT}\left(\overline{\lambda}_{LT}-0.2\right)+\overline{\lambda}_{LT}^2\right] \text{ and } \overline{\lambda}_{LT} = \sqrt{\frac{\beta_w W_{pl,y} f_y}{M_{cr}}},$$

M_{cr} is elastic critical moment for lateral-torsional buckling which is given by the general formula for uniform cross-section beam symmetrical about the minor axis:

$$M_{cr} = C_1 \frac{\pi E I_Z}{(kL)^2}\left\{\sqrt{\left(\frac{k}{k_W}\right)^2 \frac{I_W}{I_Z} + \frac{(kL)^2 G I_z}{\pi^2 E I_Z} + \left[C_2 Z_g - C_3 Z_j\right]^2} - \left[C_2 Z_g - C_3 Z_j\right]\right\}$$

k refers to end rotation in plane,

k_w refers to end warping,

both values are 0.5 for full fixity;

1.0 for no fixity; and

0.7 for one end fixity and one end free.

C_1, C_2 and C_3 are factors depending on the loading and end restraint given in Tables 4.12 and 4.13 of EBCS 3 1995.

L is system length of the member between points of lateral restraint

$Z_g = Z_a - Z_s$

Z_a is the co-ordinate of the point of load application

Z_s is the coordinate of the shear center.

Resistance to bending and shear

When the design value of the shear force, V_{Sd} exceeds 50% of the design plastic shear resistance, $V_{pl,Rd}$, the design resistance moment of the cross-section should be reduced to $M_{v,Rd}$ (the reduced design plastic resistance moment allowing for the shear force).

- For cross-sections with equal flanges, bending about the major axis:

$$M_{v,Rd} = \left[W_{pl} - \frac{\rho A_v^{\,2}}{4t_w}\right]\frac{f_y}{\gamma_{Mo}} \text{ but } M_{v,Rd} \le M_{C,Rd} \qquad (3.8)$$

where $\rho = \left(2V_{Sd}/V_{pl,Rd} - 1\right)^2$

- For other cases $M_{v,Rd}$ should be taken as the design plastic resistance moment of the cross-section using a reduced strength $(1-\rho)f_y$ for the shear area, but not more then $M_{c,Rd}$ which is the appropriate value in cross-section resistance to pure bending. This applies to class 1, 2, 3 and 4 cross-sections.

Design steps for members subjected to bending are:

1. Maximum bending moment determination;
2. Required plastic modulus determination from $W_{pl} = M_{sd}\gamma_{Mo}/f_y$, M_{Sd} is the actual bending moment and f_y is the yield strength;
3. Checking whether the provided W_{pl} is grater than the required W_{pl};
4. Determination of moment resistance, considering the shear effect (if any);
5. Checking for shear resistance of the section;
6. Checking for lateral torsional buckling and then either provide intermediate lateral support for the compression flange or choose a larger profile (if any).

3.3. Design of steel columns

A column is defined as a structural member subjected to compressive force in a direction parallel to its longitudinal axis and/or bending moment.

Nearly all members in a structure are subjected to both bending moment and axial force-either tension or compression. When the magnitude of one or the other is relatively small, its effect is usually neglected and the member is designed either as a beam, or as an axially loaded column. For many situations neither effect can properly be neglected and the behavior under combined loading must be considered in design. A special class of such members that are subjected to both axial compression force and bending moment are called beam-columns. They represent the general load case of an element in a structural frame.

There are a number of factors that affect the performance of a member under combined axial force and bending moment. A number of categories of combined bending and axial load along with the likely mode of failure may be summarized as follows:

a. Axial tension and bending: failure usually by yielding.
b. Axial compression and bending about one axis: failure by instability in the plane of bending, with out twisting.
c. Axial compression and bending about the strong axis: failure by lateral-torsional buckling.
d. Axial compression and biaxial bending-torsionally stiff sections: failure by instability in one of the principal directions.

e. Axial compression and biaxial bending thin-walled open sections: failure by combined twisting and bending on these torsionally weak sections.
f. Axial compression, biaxial bending and torsion: failure by combined twisting and bending when plane of bending does not contain the shear center.

In addition to mode of moment application as noted above, the behavior of a beam-column also depends on its length on its lateral support conditions. In this later context, and with special reference to beam-columns, the behavior can be classified into the following five cases.

Case 1: A short column subjected to axial load and uni-axial bending about either axis or biaxial bending;

Failure generally occurs when the plastic capacity of the section is reached. Note limitations set in case (2) below.

Case 2: A slender column subjected to axial load and uni-axial bending about the major axis;

If the column is supported laterally against buckling about the minor axis out of the plane of bending, the column fails by buckling about the major axis. This is not a common case. At low axial loads or if the column is not very slender a plastic hinge forms at the end or point of maximum moment.

Case 3: A slender column subjected to axial load and uni-axial bending about the minor axis;

The column does not require lateral support and there is no buckling out of the plane of bending. The column fails by buckling about the minor axis. At very low axial loads it will reach the bending capacity for minor axis.

Case 4: A slender column subjected to axial load and uni-axial bending about the major axis;

This time the column has no lateral support. The column fails due to a combination of column buckling about the minor axis and lateral torsional buckling where the column section twists as well as deflecting in the major and minor planes.

Case 5: A slender column subjected to axial load and biaxial bending;

The column has no lateral support. The failure is the same as in case 4 above but minor axis buckling will have the greatest effect. This is the general loading case.

Some EBCS 3 1995 design criteria (*Reference No. 3*)

Resistance of cross-section to bending and axial force

Members designed to resist factored bending moments $M_{y,Sd}$ and $M_{z,Sd}$, calculated using appropriate load combinations, must satisfy:

a) When $V_{Sd} < 0.50\,V_{pl,Rd} = \dfrac{A_v\left(f_y/\sqrt{3}\right)}{\gamma_{Mo}}$

$$\frac{N_{Sd}}{N_{Rd}} + \frac{M_{y,Sd} + N_{Sd}e_{Ny}}{M_{c,y,Rd}} + \frac{M_{z,Sd} + N_{Sd}e_{Nz}}{M_{c,z,Rd}} \le 1.0 \qquad (3.9a)$$

where N_{sd}, $M_{y,Sd}$, $M_{z,Sd}$ are the design forces acting at the cross-section

N_{Rd} is design compressive resistance of the cross-section which is the smaller of:

- *Design plastic resistance*, $N_{pl,Rd}$ of the gross section for class 1, 2 or 3 given by $N_{pl,Rd} = \dfrac{Af_y}{\gamma_{Mo}}$ and (3.9b)
- *Design local buckling resistance* of the gross section of class 4 given by $N_{o,Rd} = \dfrac{A_{eff}f_y}{\gamma_{M1}}$ (3.9c)

$M_{c,y,Rd}$ and $M_{c,z,Rd}$ are the resistances to uni-axial moment respectively

e_{Ny}, e_{Nz} are the shifts in the neutral axis when the cross-section is subjected to uniform compression.

b) When $V_{Sd} > 0.50\,V_{pl,Rd} = \dfrac{A_v\left(f_y/\sqrt{3}\right)}{\gamma_{Mo}}$

The design resistance of the cross-section to the combination of moment and axial force should be calculated using a reduced yield strength $(1-\rho)f_y$ for the shear where $\rho = \left(2V_{Sd}/V_{pl,Rd} - 1\right)^2$.

Buckling resistance of compression member with moments

For members subjected to combined bending and axial compressive load, interaction effects should be considered between bending moments and axial compression load. One may use the following interaction criteria.

1. Axial compression and uni-axial major axis moment:

    i. To avoid buckling about the major axis:

$$\frac{N_{Sd}}{N_{b,y,Rd}} + \frac{1.5M_{y,Sd} + N_{Sd}e_{Ny}}{\beta_{w,y}W_{pl,y}f_y/\gamma_{M1}} \le 1.0 \tag{3.10a}$$

    ii. To avoid buckling about the minor axis (for members subjected to lateral torsional buckling):

$$\frac{N_{Sd}}{N_{b,x,Rd}} + \frac{M_{y,Sd} + N_{Sd}e_{Ny}}{M_{b,Rd}} \le 1.0 \tag{3.10b}$$

2. Axial compression and uni-axial minor axis moment:

    To avoid buckling about the minor axis:

$$\frac{N_{Sd}}{N_{b,x,Rd}} + \frac{1.5M_{y,Sd} + N_{Sd}e_{Ny}}{\beta_{w,x}W_{pl,x}f_y/\gamma_{M1}} \le 1.0 \tag{3.11}$$

3. Axial compression and biaxial moments.

    i. All members should satisfy:

$$\frac{N_{Sd}}{(N_{b,Rd})_{min}} + \frac{1.5M_{y,Sd} + N_{Sd}e_{Ny}}{\beta_{w,y}W_{pl,y}f_y/\gamma_{M1}} + \frac{1.5M_{x,Sd} + N_{Sd}e_{Nx}}{\beta_{w,x}W_{pl,x}f_y/\gamma_{M1}} \le 1.0 \tag{3.12a}$$

    ii. Members potentially subject to lateral torsional buckling should also satisfy;

$$\frac{N_{Sd}}{N_{b,x,Rd}} + \frac{M_{y,Sd} + N_{Sd}e_{Ny}}{M_{b,Rd}} + \frac{1.5M_{z,Sd} + N_{Sd}e_{Nz}}{\beta_{w,z}W_{pl,z}f_y/\gamma_{M1}} \le 1.0 \tag{3.12b}$$

Where N_{sd}, e_{Ny}, and e_{Nz} are defined previously

$M_{y,Sd}$ and $M_{z,Sd}$ are the maximum design moments, each considered separately, occurring in the member.

$N_{b,y,Rd}$ and $N_{bz,Rd}$ are the flexural buckling resistances for the major and minor axis respectively.

The design flexural buckling resistance, $N_{b,Rd}$ is determined as follows.

$$N_{b,Rd} = \frac{\chi\beta_A Af_y}{\gamma_{M1}} \tag{3.13}$$

where β_A = 1 for class 1, 2 or 3 cross-sections;

$\beta_A = A_{eff}/A$ for class 4 cross-sections

χ is a reduction factor accounting for buckling given by

$$\chi = \frac{1}{\phi + \left(\phi^2 - \bar{\lambda}^2\right)^{0.5}} \text{ but } \chi < 1.0 \text{ for } 0.2 < \bar{\lambda} \le 3.0.$$

For $\bar{\lambda} \le 0.2$, $\chi = 1.0$. In which $\phi = 0.5\left(1 + \alpha\left(\bar{\lambda} - 0.2 + \bar{\lambda}^2\right)\right)$

α is an imperfection factor

= 0.21 for buckling curve a

= 0.34 for buckling curve b

= 0.49 for buckling curve c and

= 0.76 for buckling curve d.

$$\bar{\lambda} = \sqrt{\frac{\beta_A A f_y}{N_{cr}}} = \frac{\lambda}{\lambda_1}\sqrt{\beta_A}$$

λ is the slenderness for the relevant buckling mode $= \frac{L_{eff}}{i}$, L_{eff} is effective length of member and i is radius of gyration of the gross cross-section.

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\varepsilon,\ \varepsilon = \sqrt{\frac{235}{f_y}} \ (f_y \text{ in MPa})$$

N_{cr} is the elastic critical force for the relevant buckling mode.

$\left(N_{b,Rd}\right)_{min}$ is the lesser of $N_{b,y,Rd}$ and $N_{b,z,Rd}$ (all buckling modes considered).

$\beta_{w,y}$ and $\beta_{w,z}$ are the values of β_w determined for major and minor axes respectively in which $\beta_w = 1$ for class 1 or 2 cross-sections;

$= W_{el}/W_{pl}$ for class 3 cross-sections;

$= W_{eff}/W_{pl}$ for class 4 cross-sections.

$W_{pl,y}$ and $W_{pl,z}$ are the plastic moduli for the major and minor axes respectively

$M_{b,Rd}$ is the lateral torsional buckling moment (see Section 3.2 of this material).

Biaxial bending resistance of cross-section

Check must be done for cross-sectional resistance as provided earlier as well as the general requirements for beam members.

The following interaction criteria are suggested for verification of buckling resistance for biaxial bending in the absence of axial compression:

$$\frac{M_{y,Sd}}{\beta_{w,y} W_{pl,y} f_y / \gamma_{M1}} + \frac{M_{z,Sd}}{\beta_{w,z} W_{pl,z} f_y / \gamma_{M1}} \leq 1.0 \qquad (3.14a)$$

and where lateral torsional buckling is a possible buckling mode:

$$\frac{M_{y,Sd}}{M_{b,Rd}} + \frac{M_{z,Sd}}{\beta_{w,z} W_{pl,z} f_y / \gamma_{M1}} \leq 1.0 \qquad (3.14b)$$

in which all quantities are defined before.

One may follow the design steps described below to design columns subjected to both axial load and bending moment.

1. Determine the axial loads and bending moments;
2. Determine the effective length of the column;
3. Check selected section for local buckling;
4. Check resistance of cross-section to compression, pure bending (with reduced resistant moment if any), to shear and combined action of M-V-N;
5. Check member stability against axial force and moment separately, i.e., buckling resistance of axially loaded member and lateral torsional buckling of the bending member;
6. Check interaction of compression members with moments.

Chapter 4 EADoSSF PROGRAM

4.1. General

The use of computer in structural design greatly increases speed of calculation and numerical accuracy. The ultimate target of this thesis was to develop an application program capable of performing elastic analysis of steel space frame structures and designing members of the structure according to EBCS 3 1995 specifications.

4.2. Input data

As the end user of the EADoSSF program starts to execute it initially requires a file name to write on the input and output data (*Fig. 4*). This is also necessary to start any new work irrespective of the previous work state. It is a must to specify a file to proceed to the next activity as the program writes on the file during data input, analysis of the structure under consideration and design of its members. The end user of EADoSSF can either give a new file name or accept the default file name.

Once a file name is specified input data forms automatically activate step by step from entering control data till entering load data.

Control data

The control data includes:

Structure number, which specifies the number of the structure that you consider. It can be 1 for the first structure, 2 for the second structure, etc.

The type of the structure, which is always 6, to specify the structure is space frame.

The number of loading systems indicates the number of sets of load data that accompany a given set of structural data (*Fig. 5*).

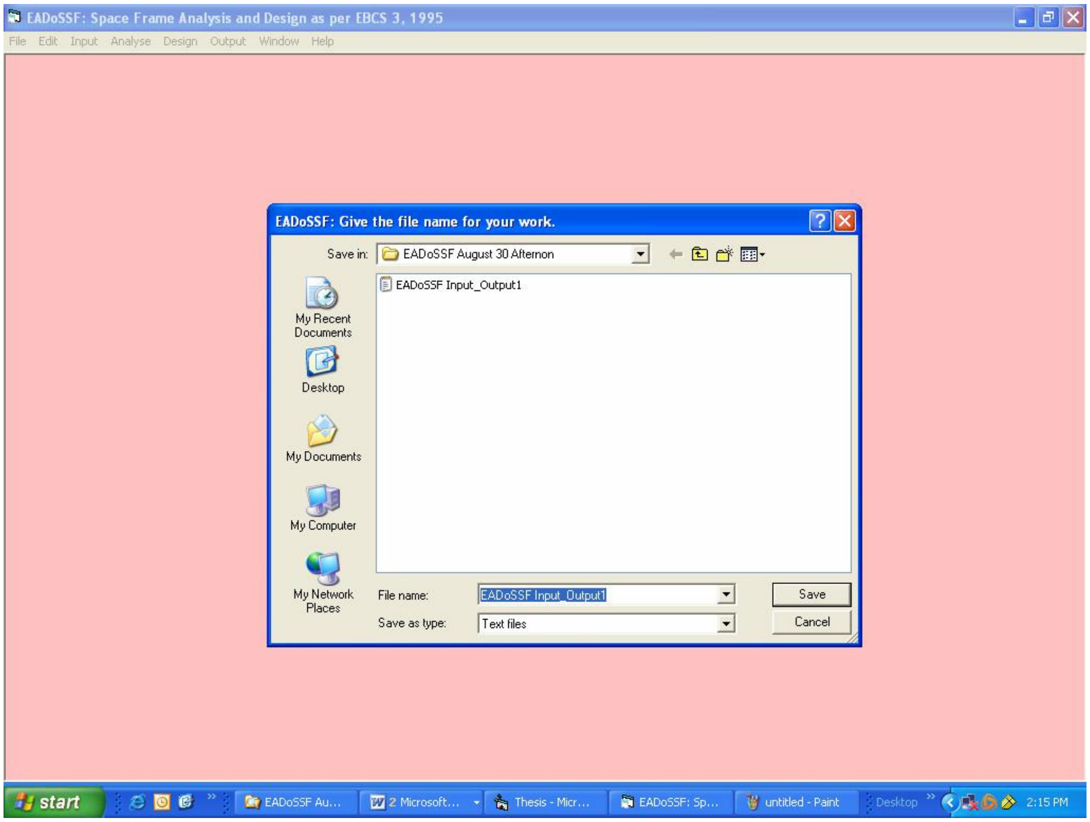

Fig. 4: File name input dialogue box

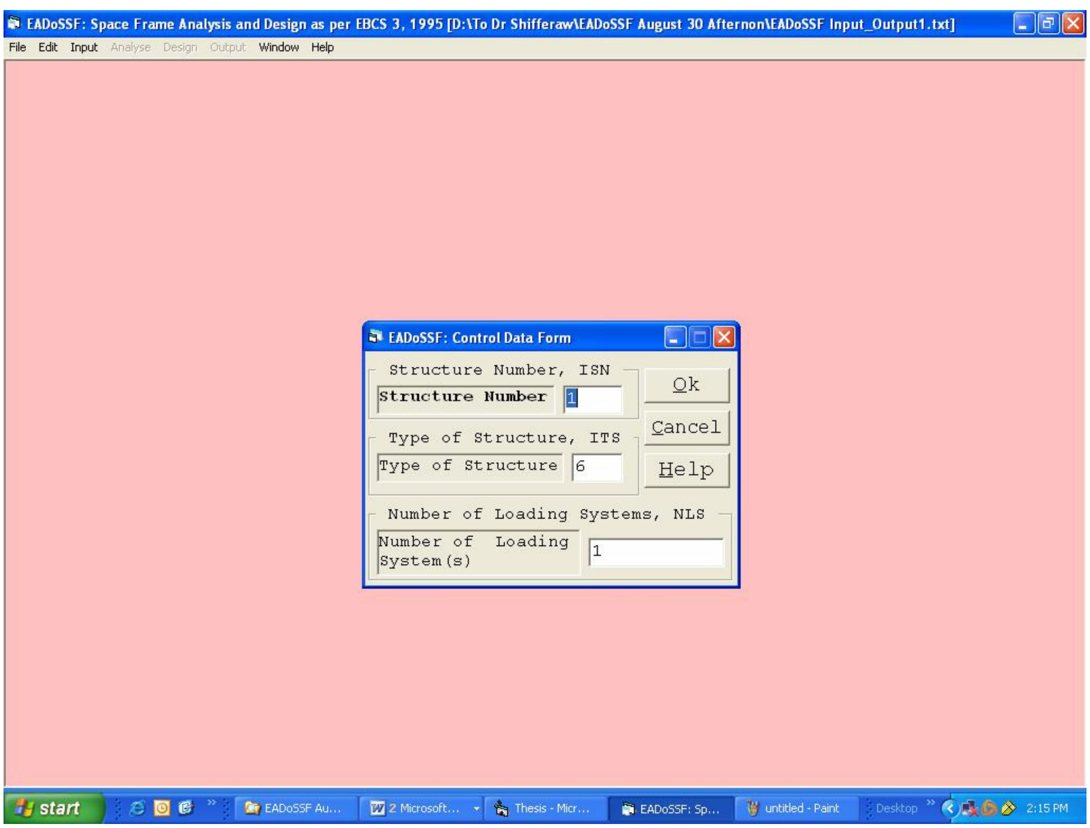

Fig. 5: Control data form

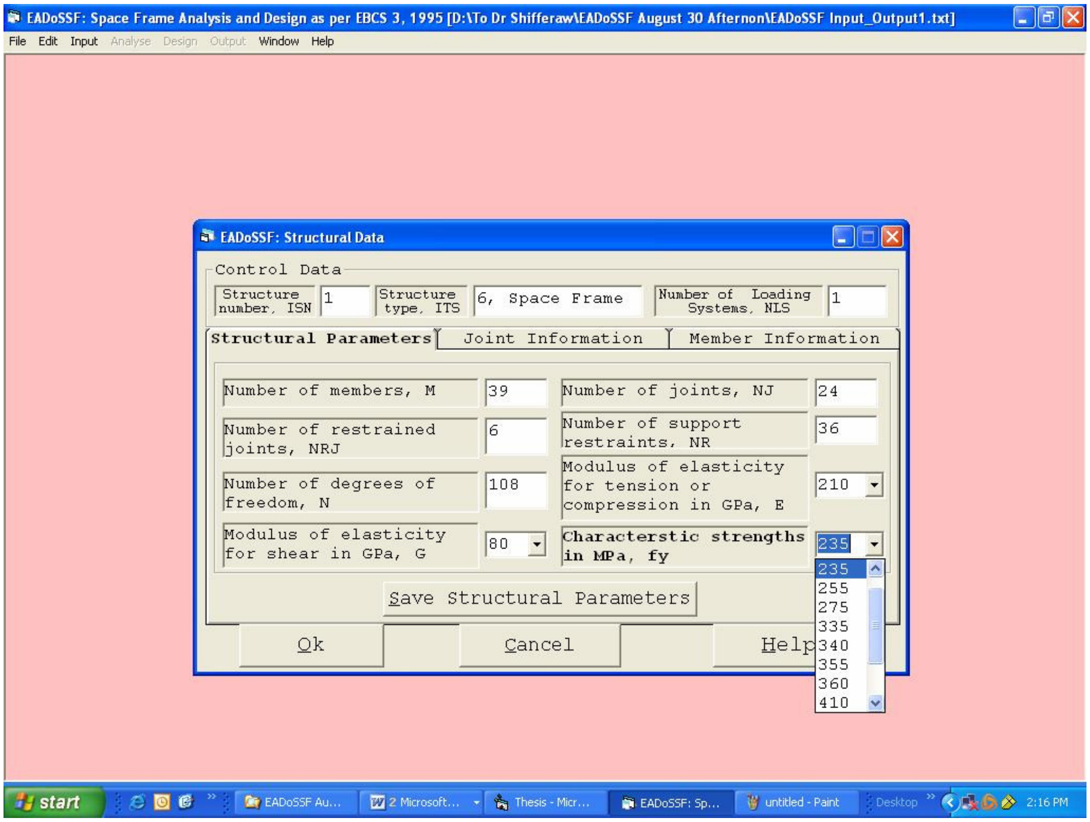

Fig. 6: Structural parameters tab in structural data form

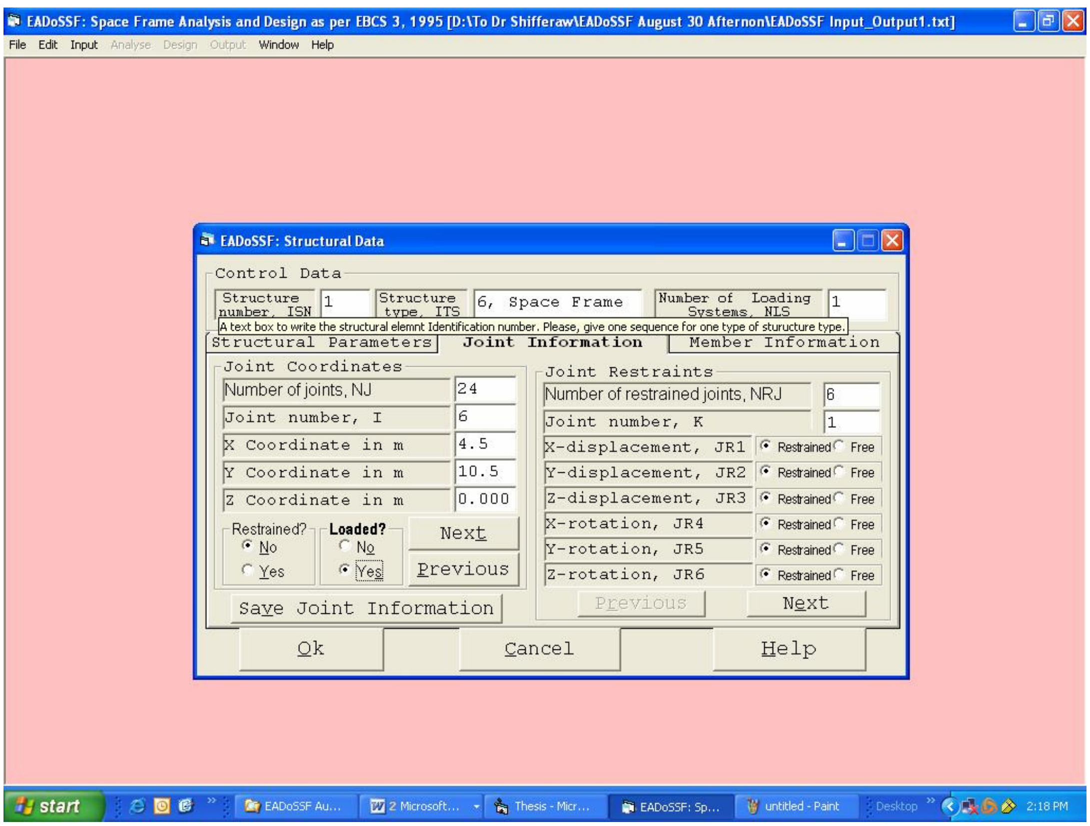

Fig. 7: Joint information tab in structural data form

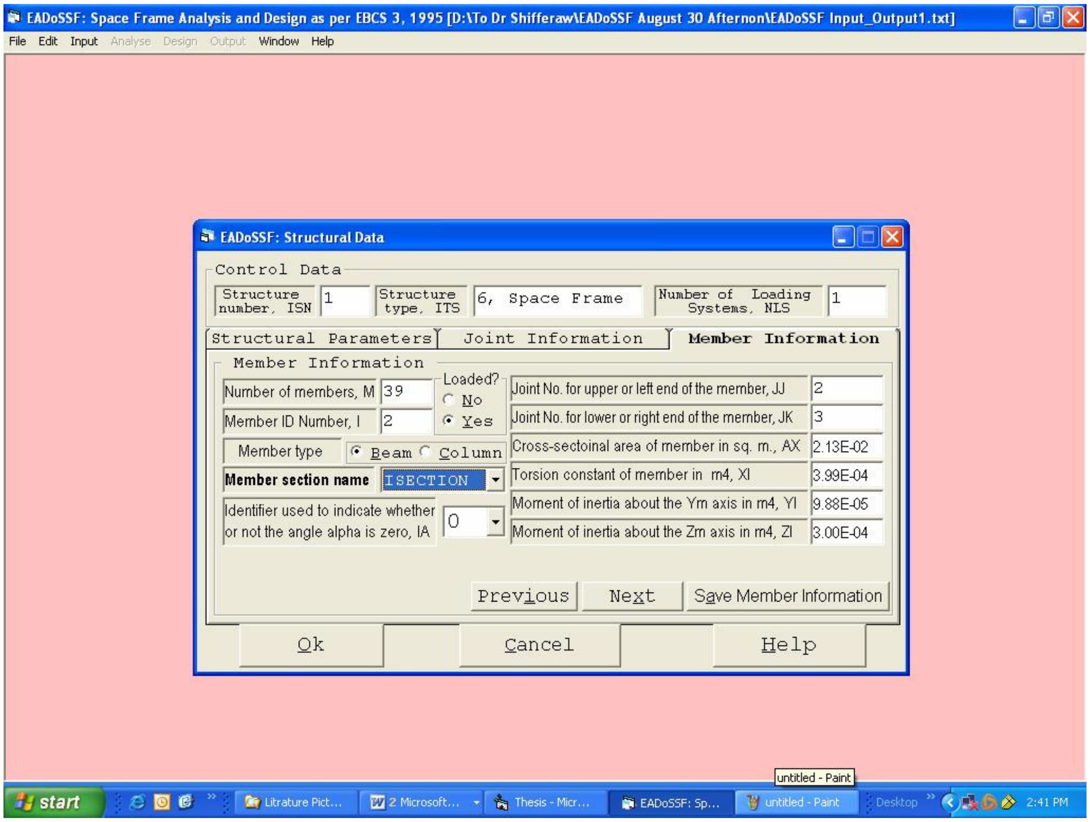

Fig. 8: Member information tab in structural data form with IA = 0

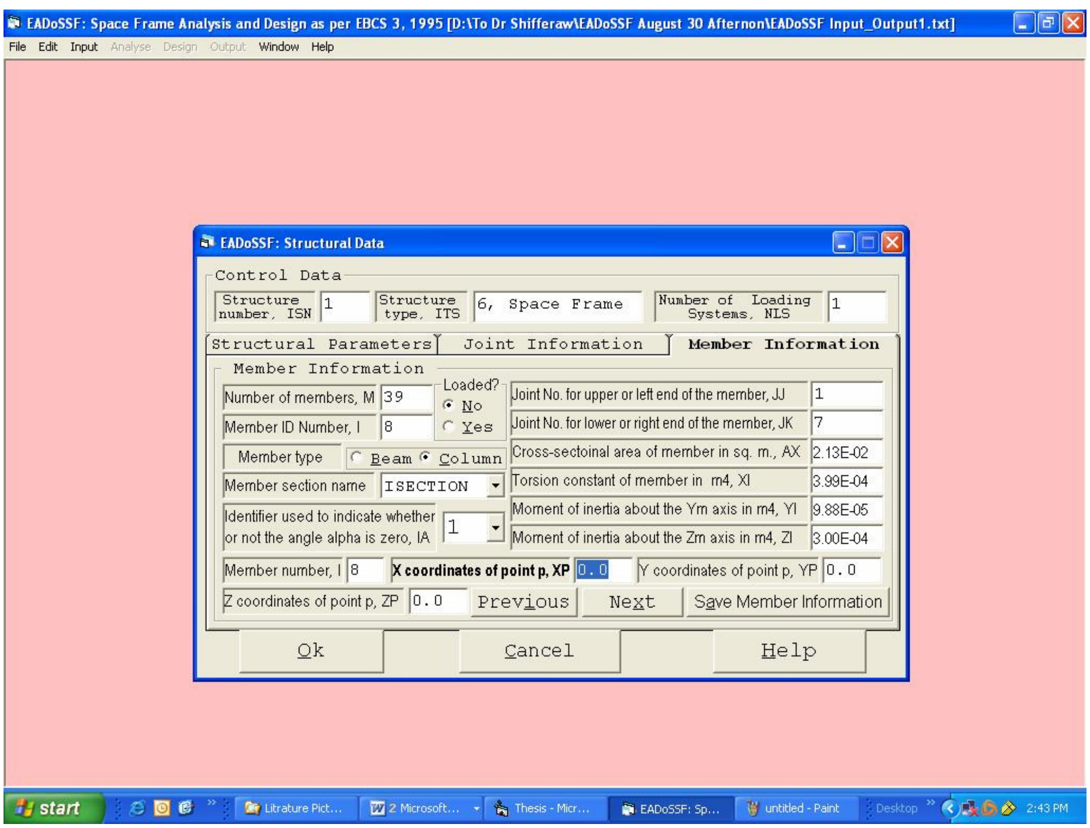

Fig. 9: Member information tab in structural data form with IA = 1

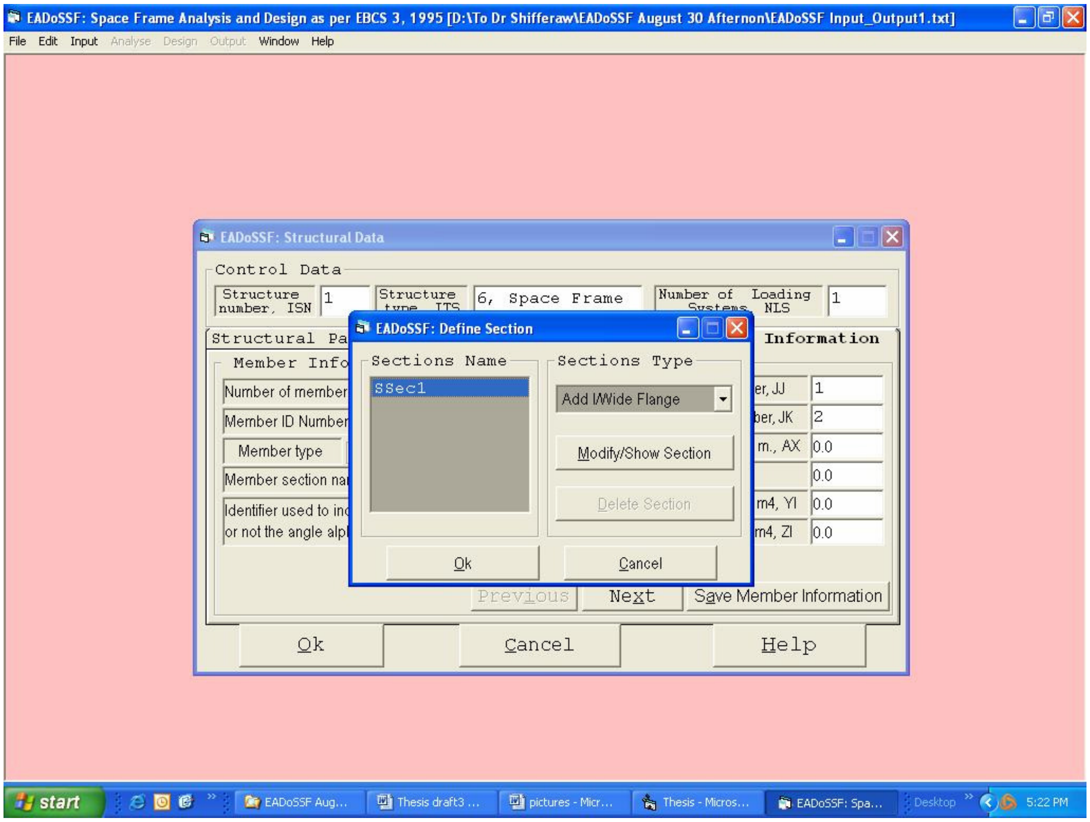

Fig. 10: A from used to define and/or modify section name and its shape

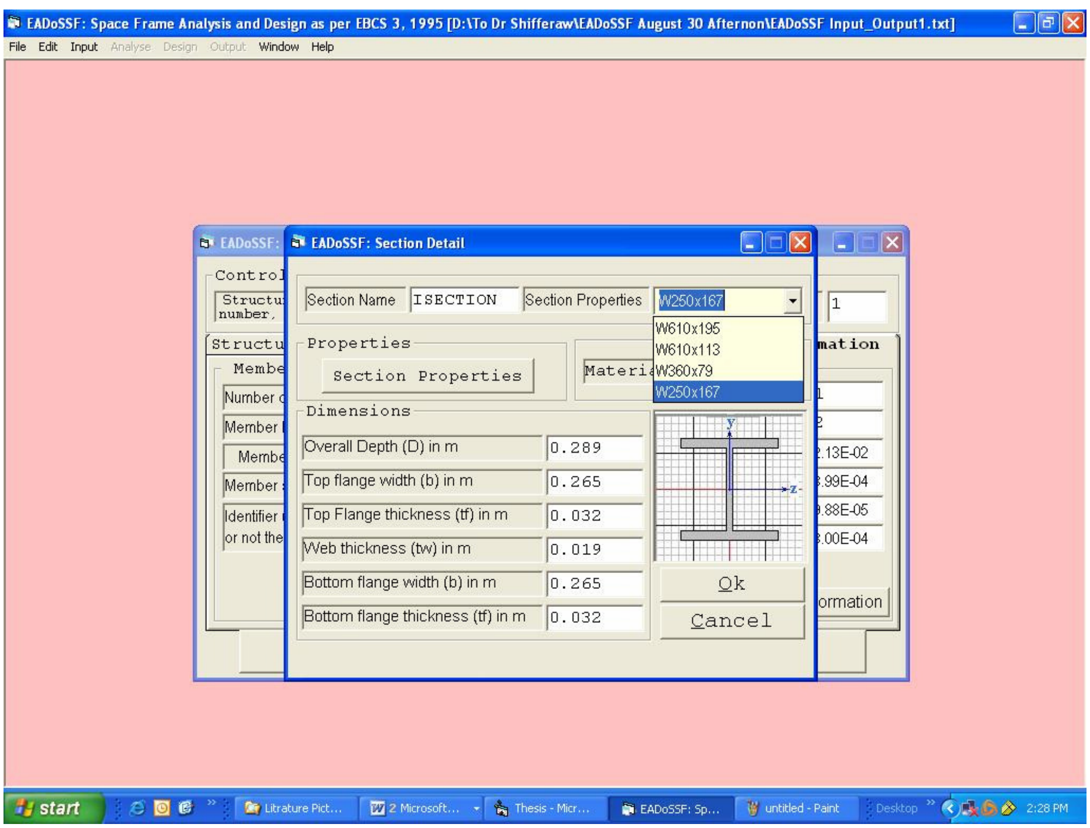

Fig. 11: Section detail form used to view the detail of a selected section (a)

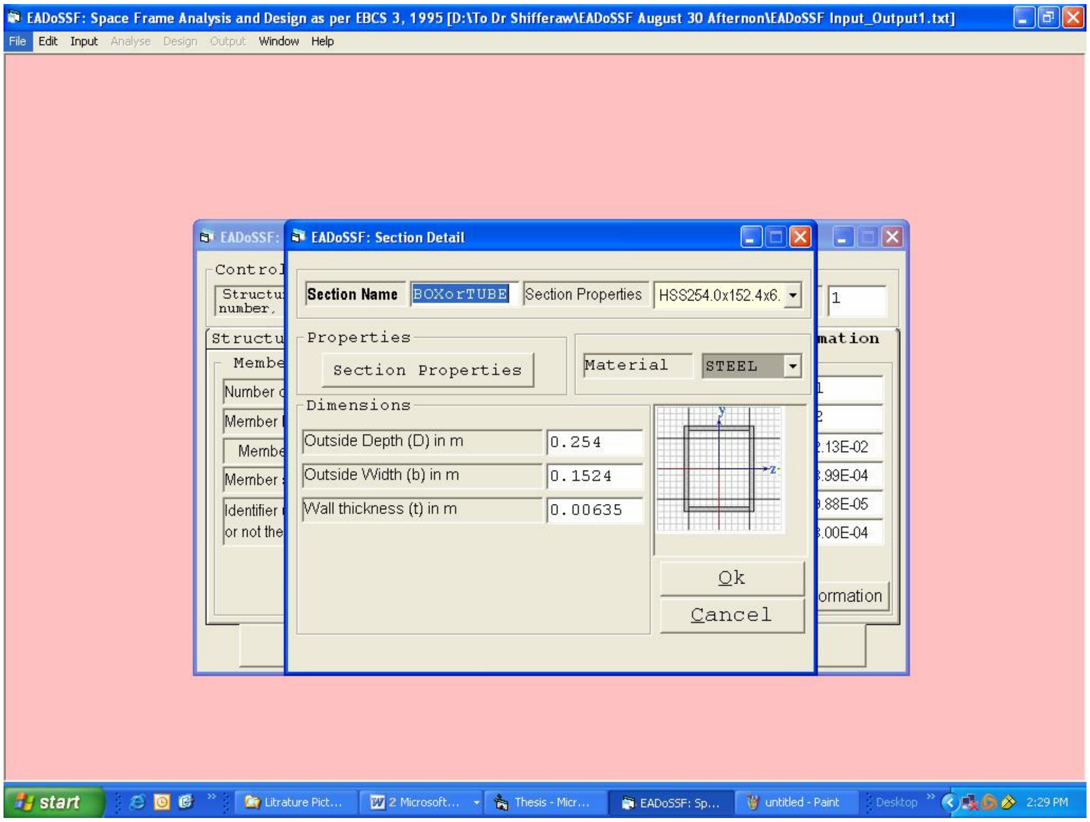

Fig. 12: Section detail form used to view the detail of a selected section (b)

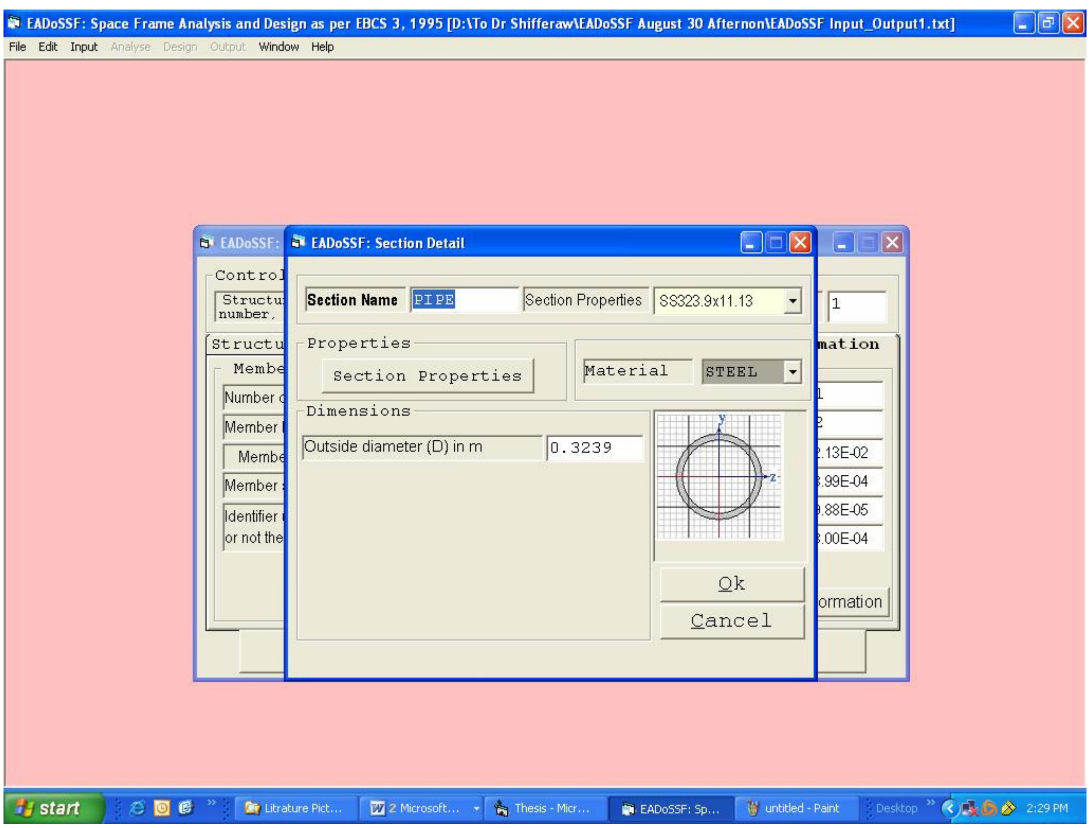

Fig. 13: Section detail form used to view the detail of a selected section (c)

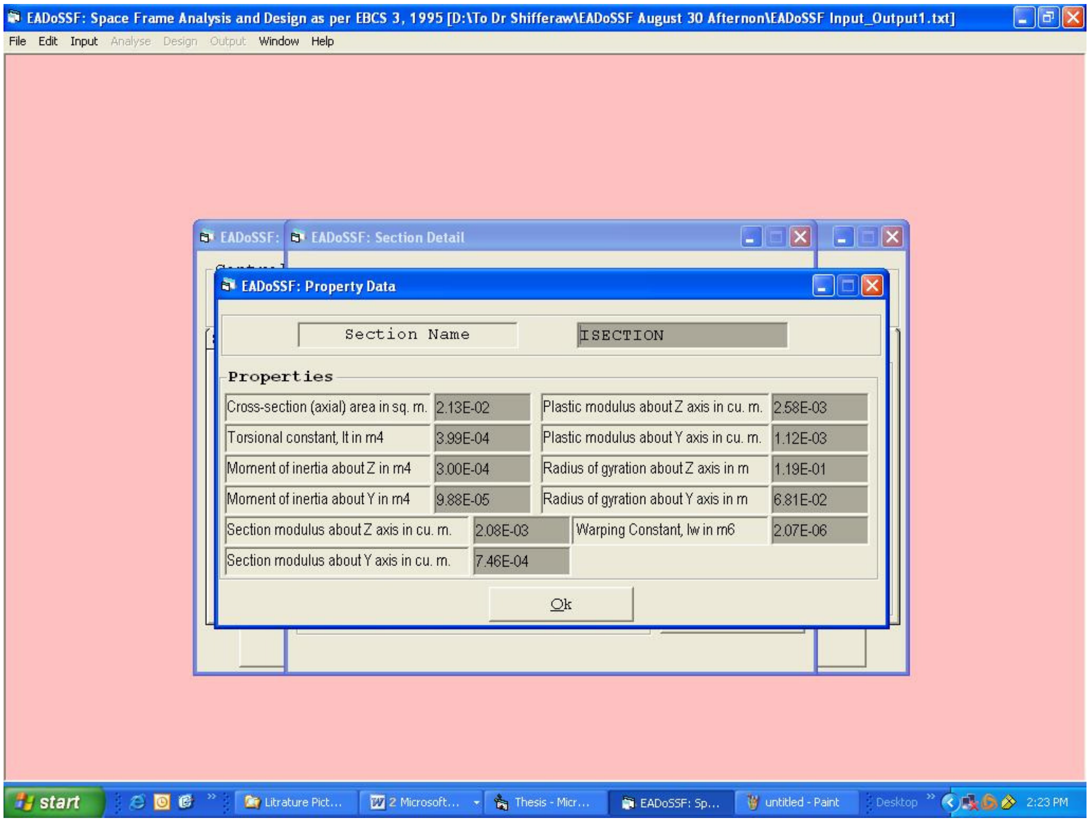

Fig. 14: Property data form

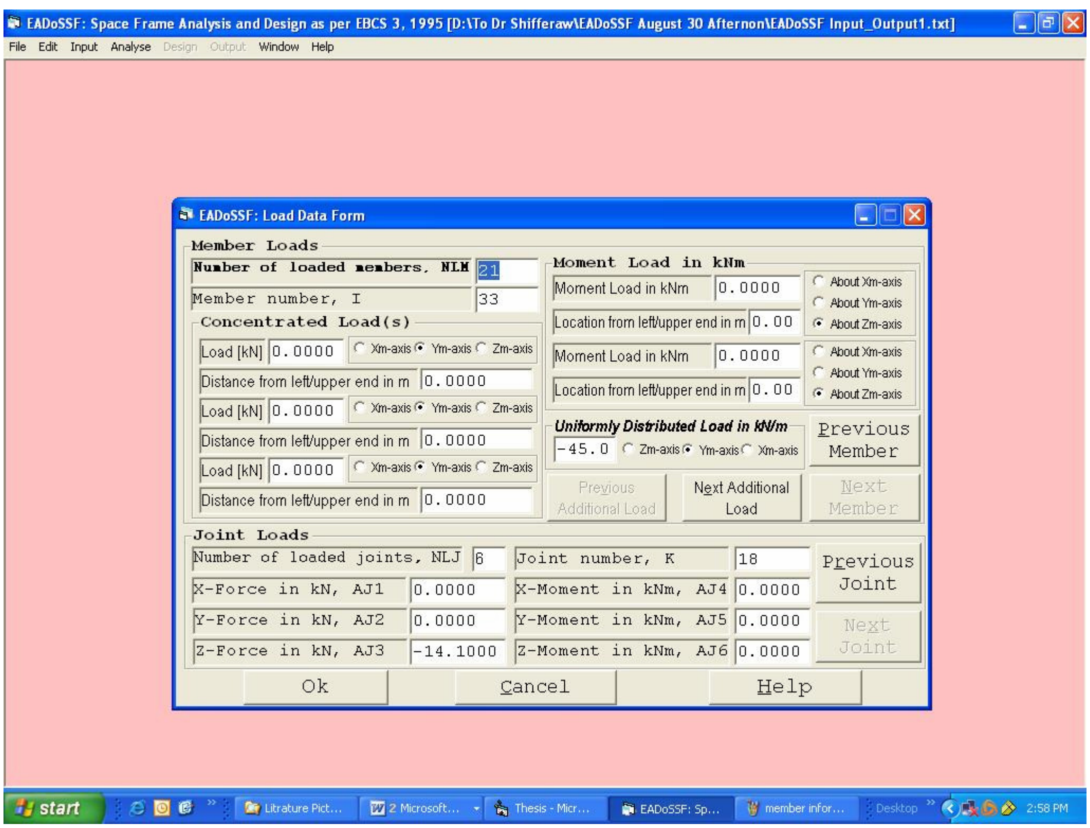

Fig. 15: A form used to input load data on members and joints of the structure

Structural data

This includes both member information and the structure information as a whole. Data for the structure information as a whole (*Fig. 6 and Fig. 7*) are:

- Number of members;
- number of joints;
- number of restraints obtained by count up the restraint in each joint in either some or all of displacements and rotations;
- number of restrained joints;
- the elastic modulus E;
- the shear modulus G;
- the yield strength;
- joint coordinates; and
- joint restraint lists.

The user is expected to define a global coordinate axis, its origin can be located any where but the orientation must be similar as it is shown in *Fig. 1* of chapter 2.

Member information includes:

- member number;
- the left or upper end and right or lower end joint number;
- the section type (beam or column);
- section name;
- section shape (I/wide flange, box or tube and circular pipe);
- their properties such as total depth, width, web thickness, etc.; and
- also an identifier 0 or 1 whether the member is oriented inclined to all the global coordinate axes.

The section properties are obtained from the database by selecting the appropriate trial section in the list on the section detail form. If the member is oriented inclined, indicated by 1, and then select a point p in the member and give the coordinates of this point (*Fig. 8 and 9; also refer Fig. 7*). p is an arbitrary point in the x_m-y_m plane, but not in the axis of the member (Fig. 3), of which the x, y and z coordinates are denoted as x_p, y_p and z_p. These coordinates are used to obtain the rotation matrix of member axis and the related expressions to the global axis.

Load data

Loads are mainly classified into two; member loads and joint loads (*Fig. 15*). The member loads include concentrated, uniformly distributed and moments (about member axes y_m, z_m, and x_m, which is torsion). EADoSSF allows entering 6 concentrated loads on a member (if any) with its location from the left or upper end of the member and the direction of the load along the member axis. It also allows entering one uniformly distributed load with direction in the member axis and 4 moments with their location from the left or upper end of the member. All member loads direction is in the sense of member axes. *Fig. 2(a)* shows orthogonal member-oriented axes with the origin always located at the left or upper end (end j). The x_m axis coincides with the centroidal axis of the member and is positive in the sense from the upper/left end (end j) to the lower/right end (end k). The y_m and z_m axes are principal axes for the member; that is, the x_m-y_m and x_m-z_m planes are principal planes of bending. *Right hand rule* is used to orient member axis in which the y_m axis coincides the thumb.

The joint loads, forces in each global coordinate axis and moments in the sense of each global coordinate axis can be entered for the respective joint if any (*Fig. 15*).

Once all the data are entered the analysis will be triggered by clicking the run command under the analysis menu. Then the design can be performed through clicking the start command under the design menu.

4.3. Output data

From the analysis of space frame structure, EADoSSF results joint deflections in the sense of global (structure) axes, both translation and rotation at each joints. Member end-actions at each end of a member, each axis forces and moments about each axis are also the outputs of EADoSSF in the sense of member axes. In addition, EADoSSF also computes and stores permanently support reactions both forces and moments in the sense of structure axes. It also shows axial and shear force diagrams and bending moment diagrams for each member of the structure as required (*Fig. 19 and Fig. 20*).

Adequacy of trial beam section for bending moment, shear force, axial force and any combination of these are the output of design for beam. In design of column the output of EADoSSF are computed values of the critical compressive stress resistance, bending resistance and buckling resistance moment capacity (in the major and minor axis, if any) and various stress ratios between the load and the resistance capacity. Besides, an insufficient capacity for any requirement of member design will be showed with red color on the 3D view of the structure after design (*Fig. 21*).

4.4. Program description

The EADoSSF program is described hereafter. EADoSSF is written with Visual Basic 6.0 (VB 6.0) programming language to run on IBM compatible personal computer. It is developed based on the requirement of EBCS 3, 1995. It has various forms to provide user interface in the input of data, in the analysis of the structure and design of members. The main form is known as multiple document interface (MDI) (*Fig. 16*), the one with the menu and the rest forms are child of the MDI. The forms are presented with their feature in the preceded articles and below.

The child forms are used to input data used for the analysis of the structure and design of members. Some of the forms are used to display the analysis and design result graphically (*Fig. 19 through Fig. 21*).

EADoSSF may be interpreted into three components: the input, the analysis and the design components. In addition to this, EADoSSF also provides features such as editing input data, displaying output results, etc.

i. Entering/Editing input data

Any end user is expected to prepare input data as per the requirement of EADoSSF. Initially the end user must *define the global coordinates* of the space frame structure. *Numbering joints* is the next step, 1 through the total number of joints, NJ consecutively. The *members are numbered* consecutively 1 through total number of members, M. The sequence of numbering is arbitrary, but each member and each joint must have a number. *Fig. 22* shows member numbers in a square box and joint numbers in a circle. Member dimensions, imposed load(s), joint restraint(s); and etc. are better to be prepared as shown in *Fig. 22*, which makes the application of EADoSSF for the analysis of the structure and/or design of members painless and straightforward.

When EADoSSF run initially or when new command triggered in the file menu, it asks a file name for working space in storing the input and output data permanently in simple text file format (*Fig. 4*). Following it, control data form will activate to allow users to input or edit control data (*Fig. 5*). This form and almost all data input form have their own default values to avoid possible error in the execution of EADoSSF. In the control data the end user is expected to change only number of loading system which is the value in the third text box. This text box allows the end user to analyze the structure for different possible loading systems. As the end user allows EADoSSF to store the input control data through clicking '`Ok`' command, the structural and member data form will activate (*Fig. 6*). This form allows the user to enter data like number of members, number of restraint (which is equal to the sum of restraints in restrained joints), number of joints, number of restrained joints, elastic modulus, shear modulus and yield strength. Once these data are inserted properly one can permanently store them by clicking the "`Save Structural Parameters`" command (*Fig. 6*). This shifts the form to the second tab, joint information tab in which joint coordinates, their loading and restrain condition (if any) are defined (*Fig. 7*). Clicking the "`Save Joint Information`" command permanently stores the joint information and proceeds to the third tab, member information tab. When the third tab got focus on the structural data form a new form will popup to define section name(s), shape(s) and section properties (*Fig. 10*). As the user tries to modify the section properties of a given section or tries to add additional shape the section detail form popup (*Fig. 11, 12 and 13*). This form enables the user to select the appropriate section from the list obtained from the database and populated in the dropdown list box of the section detail form automatically. It is possible to view details of the section properties by clicking the section properties command button, which results a view of property data form (*Fig. 14*). As the user clicks '`Ok`' command of the section detail forms, the defined or modified section name will be inserted in the "Sections Name" list box of the define section form (*Fig. 10*). While the section details saved at the back, the section names defined in the "Sections Name" list box of the define section form will be inserted to the dropdown list box of the member information tab in structural data form. Pressing the "`Save Member Information`" command permanently saves member information defined in the respective tab. '`Ok`' command of the structural data form allows EADoSSF to verify the need to store the input structural data and proceed the user to the next step.

Here one can see the 3D skeleton of the structure by clicking the 3D view under the output main menu. The next step is entering load data using the load data input/edit form (*Fig. 15*). The load data form allows the user to enter concentrated loads, uniformly distributed load, bending moments as a member load in the sense of member axes and joint loads in the sense of structure axes (*Fig. 15*). The load data will be stored permanently as the user clicks the 'ok' command. At this stage the input process completed.

ii. Analysis of the structure

The analysis of the space frame structure is performed as the user clicks the run command under the analysis menu during which the analysis sub-procedure started to execute. In the analysis, joint deflections, member end-actions and support reactions are computed and stored permanently. In addition, member loads are converted into equivalent joint load and summed up to the actual joint loads. As the analysis ends EADoSSF displays a message box that indicates the completion of the analysis if you enter appropriate data other wise EADoSSF displays a message that tells wrong data were enter and EADoSSF terminates it self properly. After the analysis, axial force diagram, shear force diagram and bending moment diagram can be viewed using a form used to select element force (*Fig. 19*) and a form used to display element force, bending moment diagram (*Fig. 20*) once the user clicks on element force/stresses menu under the output menu on the MDI form.

The analysis can be done for any number of members in a structure and for any number of loadings on a structure. It does not have any limitation in the degree of freedom. Besides, it can perform the analysis and/or design of members of any number of structures either on the same file or on different file.

iii. Design of members

The sub-procedures which design a beam and a column will be executed for the number of beams and columns of the structure, as the user clicks on the start command under the design menu. The sub-procedures will save their outputs permanently on the specified file. These procedures are written as per the requirements of Ethiopian Building Code Standards EBCS 3, 1995. As soon as the design task completed, EADoSSF displays a message box that indicates the completion of the design task. Under the output menu by clicking on the design result, one can view the output of the design whether there is any member insufficient to any of beam or column design criteria in EBCS 3, 1995. Insufficient section of a member is identified by the red color of the member itself on the 3D view of this command. If the user wants to know for which criteria the member(s) is/are insufficient, she/he has to see the input output file properly.

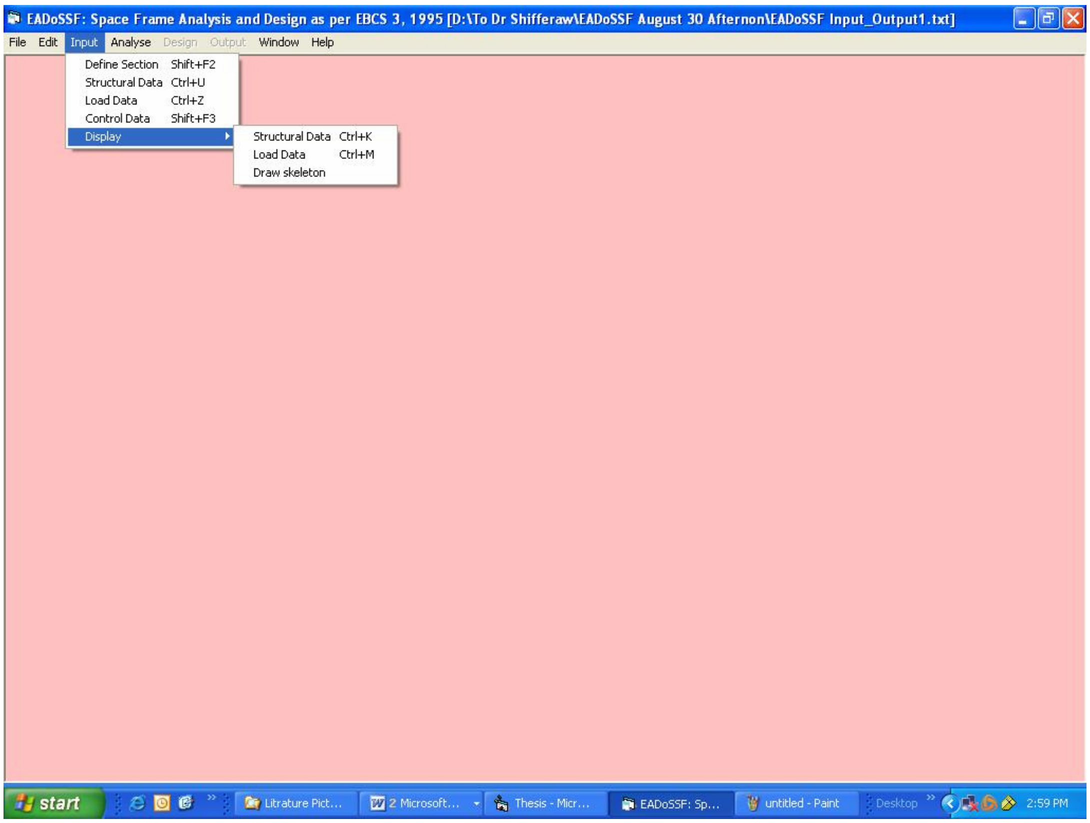

Fig. 16: Multiple document interface (the main form)

Fig. 17: A splash form displayed when the EADoSSF starts to run describing about the program

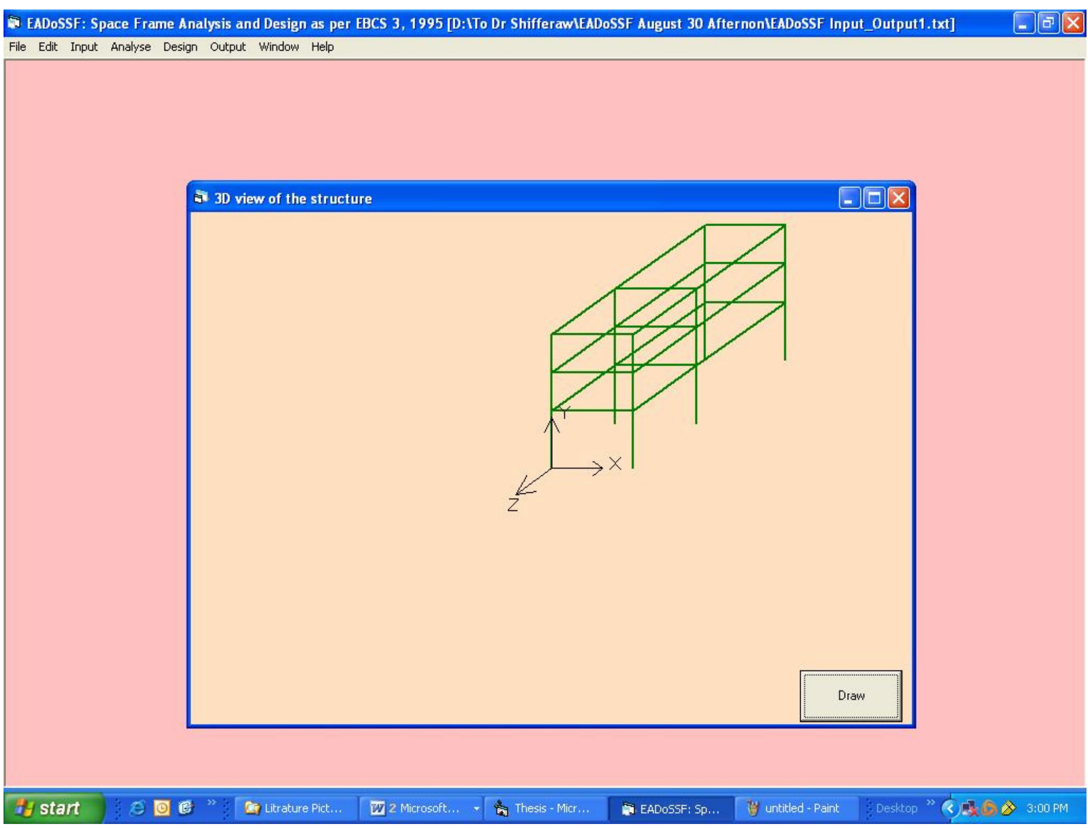

Fig. 18: A form used to display 3D view of the structure

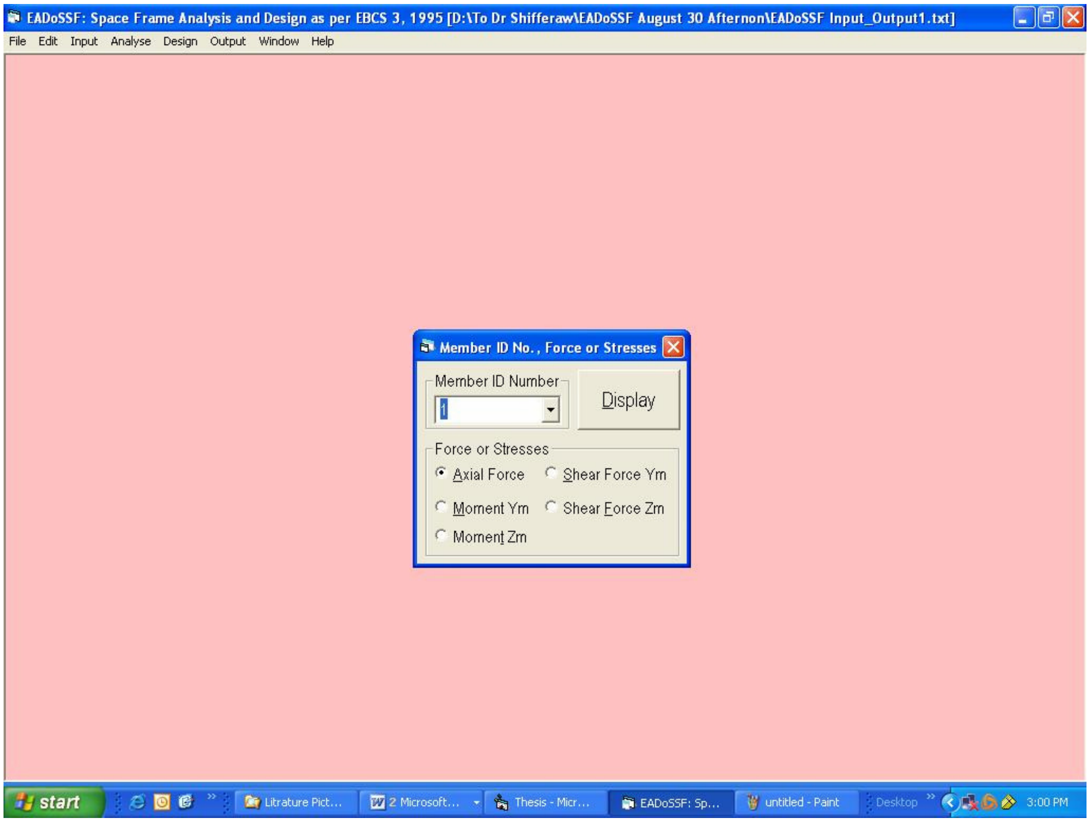

Fig. 19: A form used to select element force

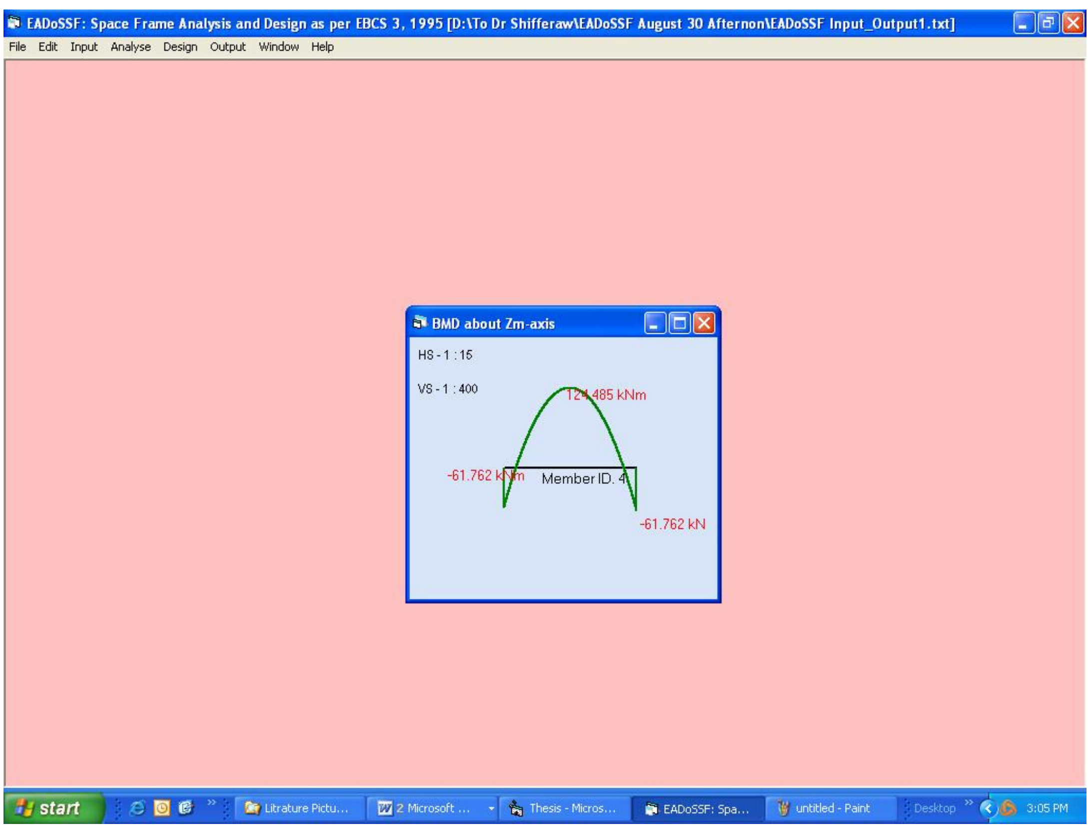

Fig. 20: A form used to display element force, bending moment diagram

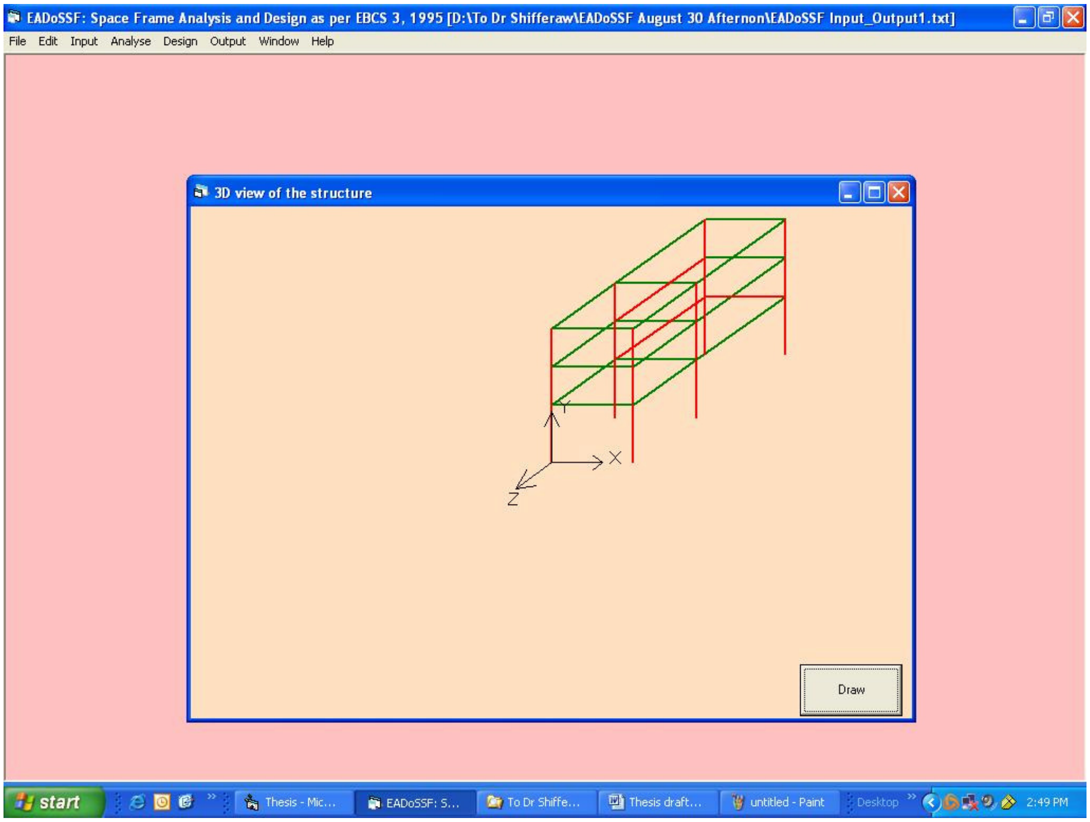

Fig. 21: Design output displayed graphically with red color for insufficient section of a member

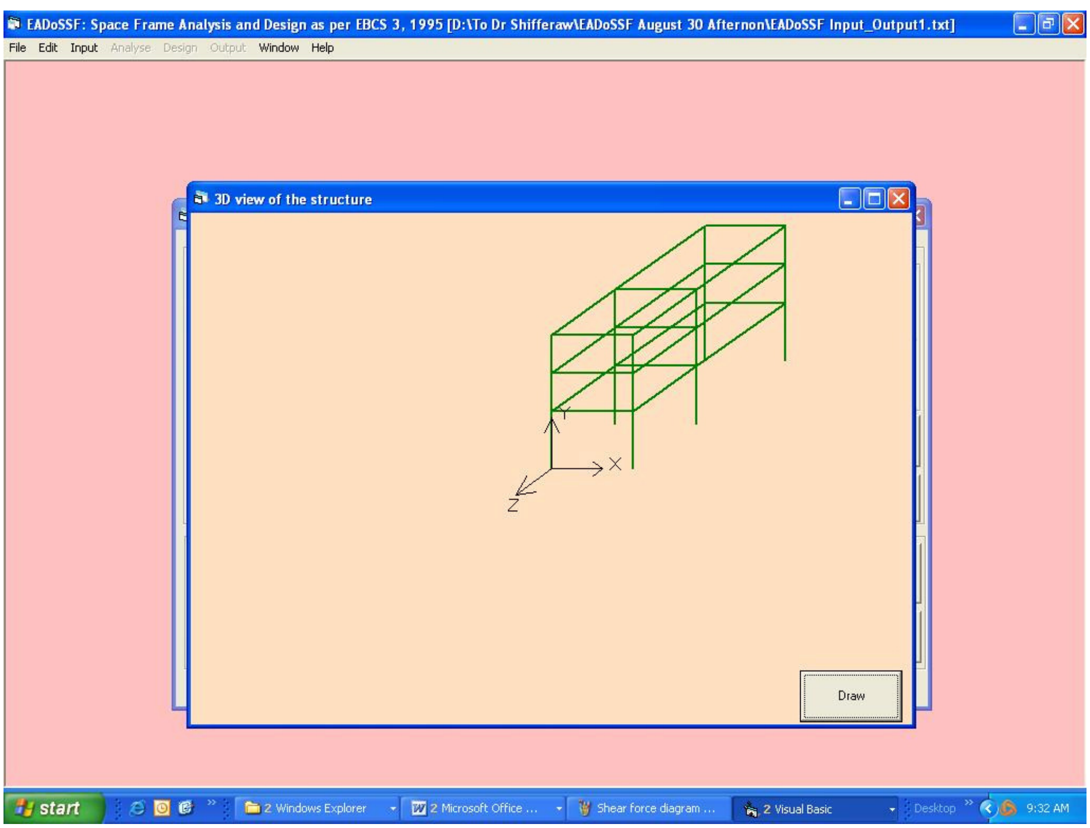

Fig. 22: Sample problem 3D view

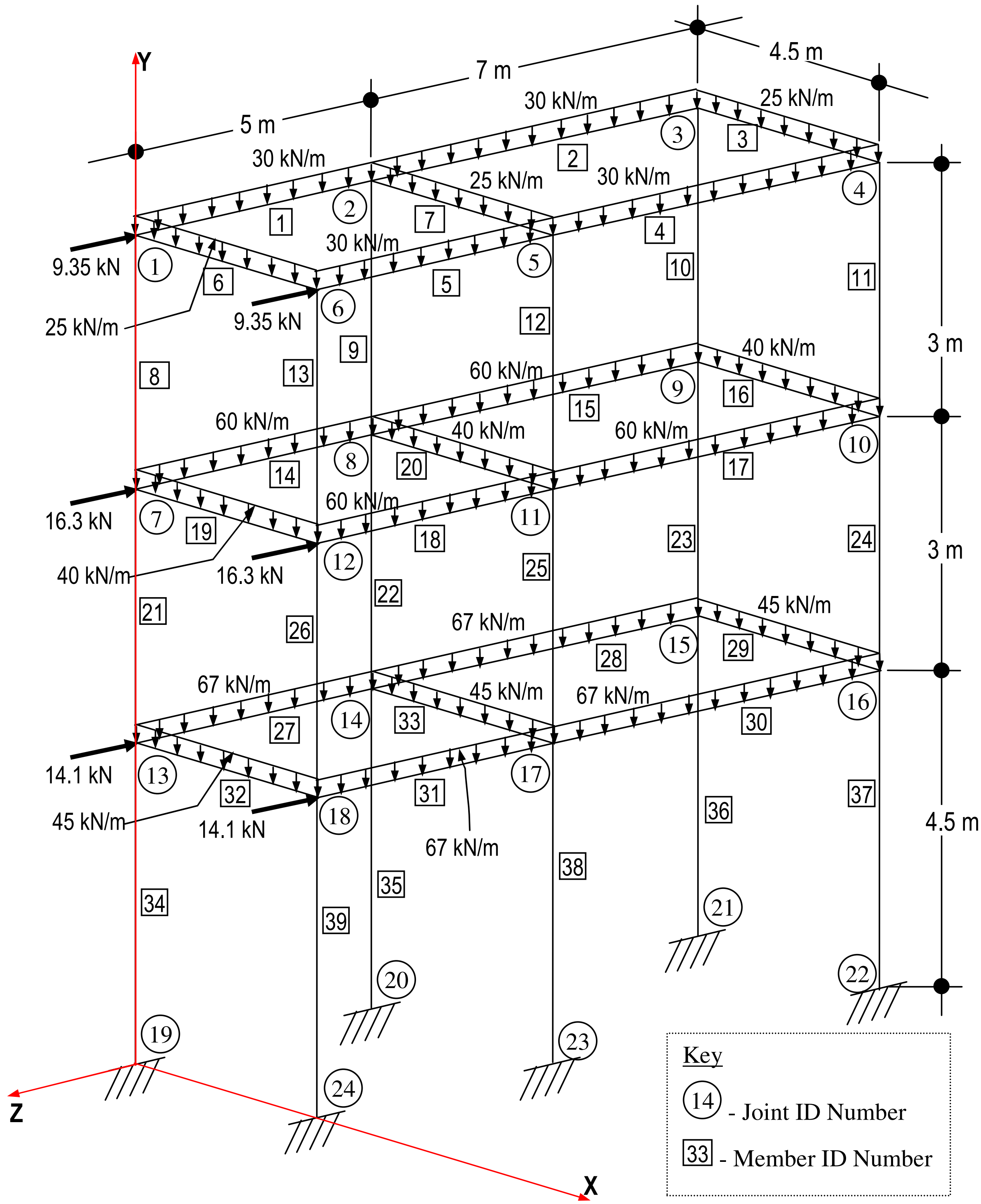

Fig. 23: A space frame for the sample problem

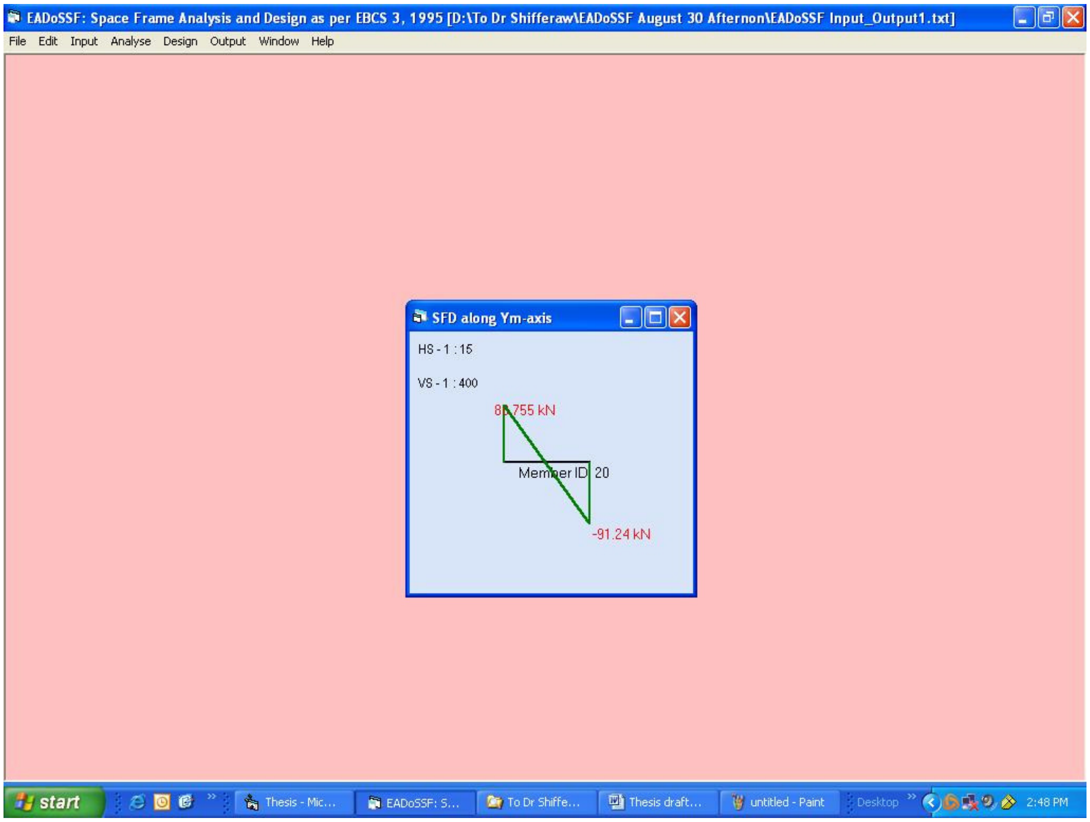

Fig. 24: Shear force diagram of the sample problem for member 20

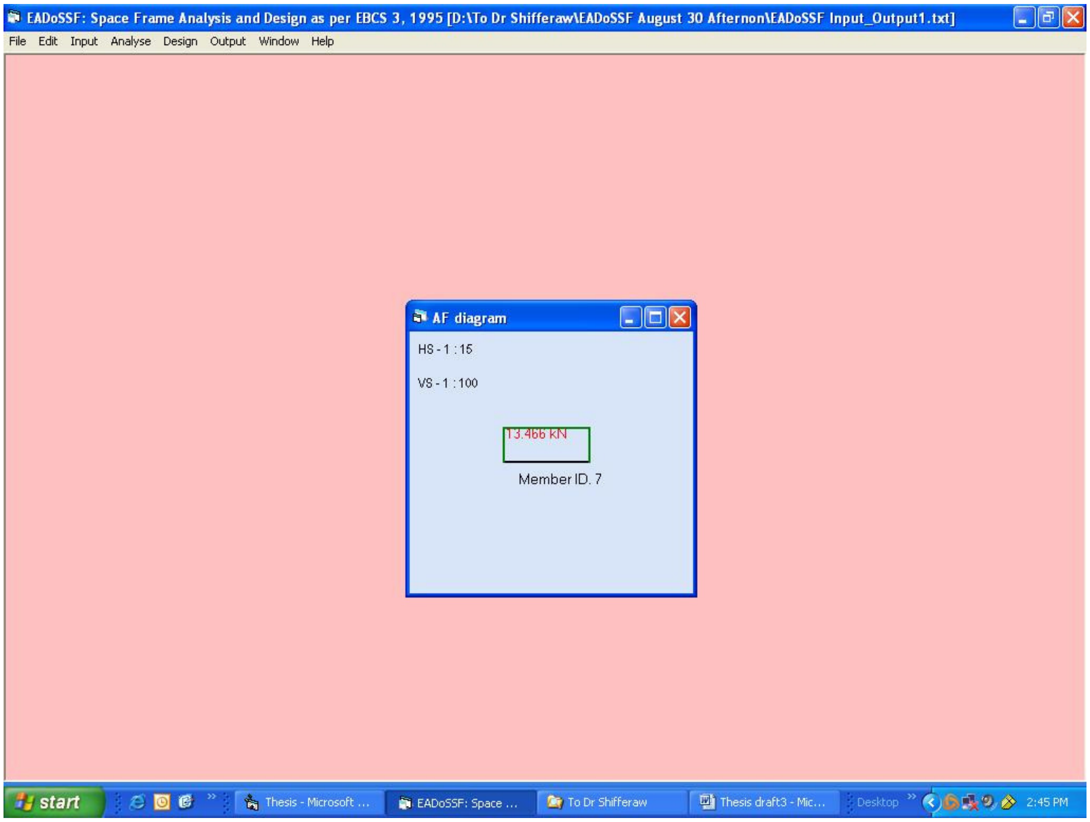

Fig. 25: Axial force diagram of the sample problem for member 7

4.5. Flow chart of EADoSSF

The flowchart of EADoSSF is presented in Annex 1. The flow chart describes mainly the steps followed in the use of EADoSSF to analyze a structure and design its member.

4.6. Sample problem

A structure shown in *Fig. 22* is solved with the EADoSSF and the input-output file is presented below with some of the graphical outputs in *Fig. 23, Fig. 24 and Fig. 25.*

EADoSSF: Elastic Analysis and Design of Steel Space Frame as per EBCS 3, 1995 9/12/2006 2:33:43 PM

----------------------------Input data--

STRUCTURE NUMBER = 1, SPACE FRAME

NUMBER OF LOADING SYSTEMS = 1

STRUCTURAL PARAMETERS

M	N	NJ	NR	NRJ	E in N/m2	G in N/m2
39	108	24	36	6	210000000000.0	80000000000.0

JOINT COORDINATES

JOINT	X in m	Y in m	Z in m
1	0.0000	10.5000	0.0000
2	0.0000	10.5000	-5.0000
3	0.0000	10.5000	-12.0000
4	4.5000	10.5000	-12.0000
5	4.5000	10.5000	-5.0000
6	4.5000	10.5000	0.0000
7	0.0000	7.5000	0.0000
8	0.0000	7.5000	-5.0000
9	0.0000	7.5000	-12.0000
10	4.5000	7.5000	-12.0000
11	4.5000	7.5000	-5.0000
12	4.5000	7.5000	0.0000
13	0.0000	4.5000	0.0000
14	0.0000	4.5000	-5.0000
15	0.0000	4.5000	-12.0000
16	4.5000	4.5000	-12.0000
17	4.5000	4.5000	-5.0000
18	4.5000	4.5000	0.0000
19	0.0000	0.0000	0.0000
20	0.0000	0.0000	-5.0000
21	0.0000	0.0000	-12.0000
22	4.5000	0.0000	-12.0000

23	4.5000	0.0000	-5.0000
24	4.5000	0.0000	0.0000

MEMBER INFORMATION

MEMBER	JJ	JK	AX in m2	XI in m4	YI in m4	ZI in m4	IA	Type	Shape	Section Name
1	1	2	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
2	2	3	0.0049	0.0001	0.0000	0.0000	0	Beam	Box/Tube	BOXorTUBE
3	3	4	0.0109	0.0003	0.0001	0.0001	0	Beam	Pipe	PIPE
4	5	4	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
5	6	5	0.0049	0.0001	0.0000	0.0000	0	Beam	Box/Tube	BOXorTUBE
6	1	6	0.0109	0.0003	0.0001	0.0001	0	Beam	Pipe	PIPE
7	2	5	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
8	1	7	0.0213	0.0004	0.0001	0.0003	0	Column	I/Wide Flange	ISECTION
9	2	8	0.0049	0.0001	0.0000	0.0000	0	Column	Box/Tube	BOXorTUBE
10	3	9	0.0109	0.0003	0.0001	0.0001	0	Column	Pipe	PIPE
11	4	10	0.0213	0.0004	0.0001	0.0003	0	Column	I/Wide Flange	ISECTION
12	5	11	0.0049	0.0001	0.0000	0.0000	0	Column	Box/Tube	BOXorTUBE
13	6	12	0.0109	0.0003	0.0001	0.0001	0	Column	Pipe	PIPE
14	7	8	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
15	8	9	0.0049	0.0001	0.0000	0.0000	0	Beam	Box/Tube	BOXorTUBE
16	9	10	0.0109	0.0003	0.0001	0.0001	0	Beam	Pipe	PIPE
17	11	10	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
18	12	11	0.0049	0.0001	0.0000	0.0000	0	Beam	Box/Tube	BOXorTUBE
19	7	12	0.0109	0.0003	0.0001	0.0001	0	Beam	Pipe	PIPE
20	8	11	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
21	7	13	0.0213	0.0004	0.0001	0.0003	0	Column	I/Wide Flange	ISECTION
22	8	14	0.0049	0.0001	0.0000	0.0000	0	Column	Box/Tube	BOXorTUBE
23	9	15	0.0109	0.0003	0.0001	0.0001	0	Column	Pipe	PIPE
24	10	16	0.0213	0.0004	0.0001	0.0003	0	Column	I/Wide Flange	ISECTION
25	11	17	0.0049	0.0001	0.0000	0.0000	0	Column	Box/Tube	BOXorTUBE
26	12	18	0.0109	0.0003	0.0001	0.0001	0	Column	Pipe	PIPE
27	13	14	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
28	14	15	0.0049	0.0001	0.0000	0.0000	0	Beam	Box/Tube	BOXorTUBE
29	15	16	0.0109	0.0003	0.0001	0.0001	0	Beam	Pipe	PIPE
30	17	16	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
31	18	17	0.0049	0.0001	0.0000	0.0000	0	Beam	Box/Tube	BOXorTUBE
32	13	18	0.0109	0.0003	0.0001	0.0001	0	Beam	Pipe	PIPE
33	14	17	0.0213	0.0004	0.0001	0.0003	0	Beam	I/Wide Flange	ISECTION
34	13	19	0.0213	0.0004	0.0001	0.0003	0	Column	I/Wide Flange	ISECTION
35	14	20	0.0049	0.0001	0.0000	0.0000	0	Column	Box/Tube	BOXorTUBE
36	15	21	0.0109	0.0003	0.0001	0.0001	0	Column	Pipe	PIPE
37	16	22	0.0213	0.0004	0.0001	0.0003	0	Column	I/Wide Flange	ISECTION
38	17	23	0.0049	0.0001	0.0000	0.0000	0	Column	Box/Tube	BOXorTUBE
39	18	24	0.0109	0.0003	0.0001	0.0001	0	Column	Pipe	PIPE

LOAD PARAMETERS

LOADING NUMBER (LN) is 0

NLJ	NLM
6	21

ACTIONS AT JOINTS

JOINT	X-Force [kN]	Y-Force [kN]	Z-Force [kN]	X-Moment [kNm]	Y-Moment [kNm]	Z-Moment [kNm]
1	0.0000	0.0000	-9.3500	0.0000	0.0000	0.0000
6	0.0000	0.0000	-9.3500	0.0000	0.0000	0.0000
7	0.0000	0.0000	-16.3000	0.0000	0.0000	0.0000
12	0.0000	0.0000	-16.3000	0.0000	0.0000	0.0000
13	0.0000	0.0000	-14.1000	0.0000	0.0000	0.0000
18	0.0000	0.0000	-14.1000	0.0000	0.0000	0.0000

ACTIONS ON MEMBER

MEMBER 1

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -30.0000 Load Direction Y

MEMBER 2

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -30.0000 Load Direction Y

MEMBER 3

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -25.0000 Load Direction Y

MEMBER 4

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000		
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000		
Moment About						

Uniformly Distributed Load in [kN/m] -30.0000 Load Direction Y

MEMBER 5

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -30.0000 Load Direction Y

MEMBER 6

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -25.0000 Load Direction Y

MEMBER 7

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -25.0000 Load Direction Y

MEMBER 14

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -60.0000 Load Direction Y

MEMBER 15

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -60.0000 Load Direction Y

MEMBER 16

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -40.0000 Load Direction Y

MEMBER 17

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -60.0000 Load Direction Y

MEMBER 18

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -60.0000 Load Direction Y

MEMBER 19

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000		
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000		
Moment About						

Uniformly Distributed Load in [kN/m] -40.0000 Load Direction Y

MEMBER 20

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						
Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000		
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000		
Moment About						

Uniformly Distributed Load in [kN/m] -40.0000 Load Direction Y

MEMBER 27

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						
Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000		
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000		
Moment About						

Uniformly Distributed Load in [kN/m] -67.0000 Load Direction Y

MEMBER 28

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						
Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000		
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000		
Moment About						

Uniformly Distributed Load in [kN/m] -67.0000 Load Direction Y

MEMBER 29

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						
Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000		
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000		
Moment About						

Uniformly Distributed Load in [kN/m] -45.0000 Load Direction Y

MEMBER 30

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -67.0000 Load Direction Y

MEMBER 31

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -67.0000 Load Direction Y

MEMBER 32

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -45.0000 Load Direction Y

MEMBER 33

Concentrated Load in [kN]	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Concentrated Load Direction						

Moment Load in [kNm]	0.0000	0.0000	0.0000	0.0000
Location from left/upper end in m	0.0000	0.0000	0.0000	0.0000
Moment About				

Uniformly Distributed Load in [kN/m] -45.0000 Load Direction Y

-------------------------- Analysis result ---------------------------------

JOINT DISPLACEMENTS

JOINT	X-Disp. [m]	Y-Disp. [m]	Z-Disp. [m]	X-Rotat. [rad]	Y-Rotat. [rad.]	Z-Rotat. [rad.]
1	4.77E-06	-9.09E-07	1.57E-05	3.77E-07	8.01E-07	-9.14E-07
2	7.21E-07	-8.19E-06	1.58E-05	-3.45E-06	8.40E-07	-1.28E-06
3	-4.83E-06	-2.39E-06	1.63E-05	6.05E-06	9.49E-07	-4.18E-07
4	-4.87E-06	-1.25E-06	1.19E-05	4.40E-06	8.14E-07	9.43E-07
5	7.21E-07	-7.76E-06	1.18E-05	-2.63E-06	8.29E-07	1.47E-06
6	4.72E-06	-1.92E-06	1.16E-05	1.15E-06	8.93E-07	4.55E-07
7	3.07E-06	-8.23E-07	6.43E-06	1.31E-06	6.18E-07	-7.71E-07
8	1.88E-07	-7.50E-06	6.44E-06	-4.99E-06	6.64E-07	-1.97E-06
9	-3.10E-06	-2.18E-06	6.54E-06	5.72E-06	7.47E-07	-8.55E-08
10	-3.11E-06	-1.14E-06	3.21E-06	4.97E-06	5.78E-07	7.82E-07
11	1.88E-07	-7.07E-06	3.16E-06	-4.62E-06	6.17E-07	2.16E-06
12	3.06E-06	-1.75E-06	3.01E-06	1.53E-06	7.40E-07	1.16E-07
13	1.37E-06	-5.87E-07	-1.34E-06	-3.77E-07	3.03E-07	-1.01E-06
14	-7.10E-09	-5.44E-06	-1.40E-06	-4.07E-06	3.31E-07	-2.29E-06
15	-1.41E-06	-1.57E-06	-1.71E-06	6.24E-06	3.67E-07	-7.48E-07
16	-1.38E-06	-8.23E-07	-3.03E-06	4.88E-06	2.63E-07	1.01E-06
17	-6.67E-09	-5.11E-06	-2.97E-06	-5.05E-06	2.93E-07	2.45E-06
18	1.41E-06	-1.26E-06	-2.77E-06	-1.91E-06	3.74E-07	7.70E-07
19	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
20	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
21	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
22	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
23	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
24	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00

MEMBRER END-ACTIONS

MEMBER	Joint	X-force [kN]	Y-force [kN]	Z-force [kN]	X-moment [kNm]	Y-moment [kNm]	Z-moment [kNm]
1	1	72.8055	72.6071	-0.1155	-2.3630	0.1274	104.7160
1	2	-72.8055	77.3929	0.1155	2.3630	0.4502	-116.6806
2	2	73.2350	105.0000	0.0000	0.9893	0.0000	122.5000
2	3	-73.2350	105.0000	0.0000	-0.9893	0.0000	-122.5000
3	3	23.1721	56.3657	1.0004	8.8371	-1.6205	36.0953
3	4	-23.1721	56.1344	-1.0004	-8.8371	-2.8811	-35.5749
4	5	50.8319	104.2590	-0.1155	-2.3933	0.4496	56.6806
4	4	-50.8319	105.7410	0.1155	2.3933	0.3591	-61.8673
5	6	51.2619	75.0000	0.0000	1.6192	0.0000	62.5000
5	5	-51.2619	75.0000	0.0000	-1.6192	0.0000	-62.5000
6	1	23.4559	56.1892	0.7355	-4.1221	-2.0826	35.6613
6	6	-23.4559	56.3109	-0.7355	4.1221	-1.2270	-35.9351
7	2	-0.1155	56.1033	0.4296	-5.8195	-0.9173	3.3523
7	5	0.1155	56.3967	-0.4296	5.8195	-1.0159	-4.0125
8	1	128.7963	-23.3403	62.7188	-1.9552	-100.5938	-38.0243
8	7	-128.7963	23.3403	-62.7188	1.9552	-87.5626	-31.9967
9	2	238.4959	0.0000	0.0000	-0.4671	0.0000	0.0000
9	8	-238.4959	0.0000	0.0000	0.4671	0.0000	0.0000
10	3	161.3657	-23.1718	-74.2349	-1.6205	113.6629	-37.0846
10	9	-161.3657	23.1718	74.2349	1.6205	109.0417	-32.4308
11	4	161.8754	23.0565	-49.8297	-2.5220	70.7044	37.9683

11	10	-161.8754	-23.0565	49.8297	2.5220	78.7848	31.2013
12	5	235.6557	0.0000	0.0000	-0.5664	0.0000	0.0000
12	11	-235.6557	0.0000	0.0000	0.5664	0.0000	0.0000
13	6	131.3108	23.4558	42.6466	-1.2270	-66.6221	37.5543
13	12	-131.3108	-23.4558	-42.6466	1.2270	-61.3177	32.8132
14	7	14.0811	134.7575	-0.6492	-7.6698	1.4276	166.1872
14	8	-14.0811	165.2425	0.6492	7.6698	1.8186	-242.3999
15	8	15.1754	210.0000	0.0000	2.1528	0.0000	245.0000
15	9	-15.1754	210.0000	0.0000	-2.1528	0.0000	-245.0000
16	9	4.2332	91.4661	0.9798	3.9985	-1.4161	66.7496
16	10	-4.2332	88.5339	-0.9798	-3.9985	-2.9930	-60.1520
17	11	30.8419	199.6367	-0.6493	-6.2974	2.3898	122.3999
17	10	-30.8419	220.3633	0.6493	6.2974	2.1550	-194.9428
18	12	31.9362	150.0000	0.0000	3.2697	0.0000	125.0000
18	11	-31.9362	150.0000	0.0000	-3.2697	0.0000	-125.0000
19	7	4.4072	88.4879	1.0058	-1.1786	-2.8305	59.9586
19	12	-4.4072	91.5121	-1.0058	1.1786	-1.6956	-66.7630
20	8	-0.6492	90.0568	1.0948	-2.6002	-2.2404	9.8226
20	11	0.6492	89.9432	-1.0948	2.6002	-2.6863	-9.5671
21	7	352.0417	-27.0986	59.4946	-3.3581	-77.4460	-35.6317
21	13	-352.0417	27.0986	-59.4946	3.3581	-101.0379	-45.6641
22	8	703.7952	0.0000	0.0000	-0.8889	0.0000	0.0000
22	14	-703.7952	0.0000	0.0000	0.8889	0.0000	0.0000
23	9	462.8315	-27.4047	-90.3899	-3.0366	131.9600	-36.4716
23	15	-462.8315	27.4047	90.3899	3.0366	139.2098	-45.7424
24	10	470.7729	26.6411	-79.6916	-3.3601	120.1565	35.2481
24	16	-470.7729	-26.6411	79.6916	3.3601	118.9182	44.6751
25	11	675.2360	0.0000	0.0000	-0.8630	0.0000	0.0000
25	17	-675.2360	0.0000	0.0000	0.8630	0.0000	0.0000
26	12	372.8222	27.8628	59.2887	-2.9226	-64.8609	37.2196
26	18	-372.8222	-27.8628	-59.2887	2.9226	-113.0051	46.3688
27	13	-47.0289	129.6905	-0.4227	-8.1906	0.9391	91.5421
27	14	47.0289	205.3095	0.4227	8.1906	1.1744	-280.5897
28	14	-46.5614	234.5000	0.0000	1.7620	0.0000	273.5833
28	15	46.5614	234.5000	0.0000	-1.7620	0.0000	-273.5833
29	15	-18.8443	100.7967	-0.2633	7.2321	1.0796	66.7305
29	16	18.8443	101.7033	0.2633	-7.2321	0.1053	-68.7706
30	17	-41.1736	223.7649	-0.4227	-6.5891	1.5710	146.5897
30	16	41.1736	245.2351	0.4227	6.5891	1.3879	-221.7357
31	18	-40.7063	167.5000	0.0000	2.6853	0.0000	139.5833
31	17	40.7063	167.5000	0.0000	-2.6853	0.0000	-139.5833
32	13	-19.1842	101.6196	-0.2646	8.1747	0.2636	68.4655
32	18	19.1842	100.8804	0.2646	-8.1747	0.9270	-66.8022
33	14	-0.4227	101.4007	0.4674	7.0064	-0.8741	9.9525
33	17	0.4227	101.0993	-0.4674	-7.0064	-1.2291	-9.2744
34	13	583.3519	-7.4917	-1.3698	-2.1554	1.3211	-30.9919
34	19	-583.3519	7.4917	1.3698	2.1554	4.8428	-2.7208
35	14	1245.0050	0.0000	0.0000	-0.5886	0.0000	0.0000
35	20	-1245.0050	0.0000	0.0000	0.5886	0.0000	0.0000
36	15	798.1281	-8.5603	-43.5652	-1.9570	127.1415	-22.7501
36	21	-798.1281	8.5603	43.5652	1.9570	68.9020	-15.7712
37	16	817.7113	7.3742	-38.7813	-1.8670	110.0497	30.6846
37	22	-817.7113	-7.3742	38.7813	1.8670	64.4663	2.4995
38	17	1167.6010	0.0000	0.0000	-0.5210	0.0000	0.0000
38	23	-1167.6010	0.0000	0.0000	0.5210	0.0000	0.0000

39	18	641.2027	8.6786	4.2177	-1.9956	-18.4036	23.1187
39	24	-641.2027	-8.6786	-4.2177	1.9956	-0.5762	15.9351

SUPPORT REACTIONS

JOINT	X-force [kN]	Y-force [kN]	Z-force [kN]	X-moment [kNm]	Y-moment [kNm]	Z-moment [kNm]
19	7.4917	583.3519	1.3698	4.8428	-2.1554	-2.7208
20	0.0000	1245.0050	0.0000	0.0000	-0.5886	0.0000
21	8.5603	798.1281	43.5652	68.9020	-1.9570	-15.7712
22	-7.3742	817.7113	38.7813	64.4663	-1.8670	2.4995
23	0.0000	1167.6010	0.0000	0.0000	-0.5210	0.0000
24	-8.6786	641.2027	-4.2177	-0.5762	-1.9956	15.9351

--------------------------- Design Output ----------------------------------

Member ID No. is 1, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.
Web is class 3.
Over all section is class 3.
MsdY = 0.4501 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 116.6032 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 60240.2687 KN/m2, and fyd = 213636.3590 kN/m2
Section is sufficient for combined effects including shear effect, if any!
VsdY = 77.3629 kN, and VplRdY = 610.3012 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.1155 kN, and VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
MbRd = 551.1818 kNm, and MsdY = 0.4501 kNm
Section is sufficient for lateral-torsional buckling!

Member ID No. is 2, Member shape is Box/Tube and it is a Beam identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,

Section is assumed to be class 2!
MsdY = 0 kNm, and McRdY = 78.0235 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 122.5 kNm, and McRdZ = 110.2300 kNm
Section is insufficient for moment about the major axis, Z!
MsdY = 0 kNm, and MNRdY = 87.2345 kNm
Section is sufficient for axial force and uni-axial bending including shear effect, if any!
VsdY = 105 kN, and VplRdY = 226.6428 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0 kN, and VplRdZ = 226.6428 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 3, Member shape is Pipe and it is a Beam identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,

Section is class 1.
MsdY = 2.8801 kNm, and McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 36.0953 kNm, and McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 23.1721 kN, NplRd = 2328636.3131 kN, MsdY = 2.8801 kNm, McRdZ = 234.9999 kNm, MsdZ = 36.0953 kNm and McRdY = 234.9999 kNm.
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.1658
Section is sufficient for combined effects including shear effect, if any!
VsdY = 56.3656 kN and VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!

VsdZ = 1.0003 kN and VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 4, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.
Web is class 3.
Over all section is class 3.
MsdY = 0.4496 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 124.4852 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 63004.4458 KN/m2, and fyd = 213636.3590 kN/m2
Section is sufficient for combined effects including shear effect, if any!
VsdY = 105.71095 kN, and VplRdY = 610.3012 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.1155 kN, and VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
MbRd = 522.4946 kNm, and MsdY = 0.4496 kNm
Section is sufficient for lateral-torsional buckling!

Member ID No. is 5, Member shape is Box/Tube and it is a Beam identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,

It = 0.0000624 m4,

Section is assumed to be class 2!
MsdY = 0 kNm, and McRdY = 65.2300 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 62.5 kNm, and McRdZ = 25.4621 kNm
Section is insufficient for moment about the major axis, Z!
MsdY = 0 kNm, and MNRdY = 34.5263 kNm
Section is sufficient for axial force and uni-axial bending including shear effect, if any!
VsdY = 75 kN, and VplRdY = 226.6427 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0 kN, and VplRdZ = 226.6427 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 6, Member shape is Pipe and it is a Beam identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,

Section is class 1.
MsdY = 2.0825 kNm, and McRdY = 234.999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 35.8788 kNm, and McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 23.4559 kN, NplRd = 2328636.3131 kN, MsdY = 2.0825 kNm, McRdZ = 234.9999 kNm, MsdZ = 35.878 kNm and McRdY = 234.9999 kNm.
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.1615
Section is sufficient for combined effects including shear effect, if any!
VsdY = 56.2858 kN and VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.7355 kN and VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 7, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,

rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.
Web is class 3.
Over all section is class 3.
MsdY = 1.0155 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 59.5992 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 30197.6472 KN/m2, and fyd = 213636.3590 kN/m2
Section is sufficient for combined effects including shear effect, if any!
VsdY = 56.3717 kN, and VplRdY = 610.3012 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.4296 kN, and VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
MbRd = 551.1818 kNm, and MsdY = 1.0155 kNm
Section is sufficient for lateral-torsional buckling!

Member ID No. is 8, Member shape is I/Wide Flange and it is a Column identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,
Leff is 3.0 m.

Flange is class 1.
Web is class 3.
Over all section is class 3.
Nsd = 128.7962 kN and NplRd = 4550.4544468261 kN
Section is sufficient to axial compression force!
MsdY = 100.5938 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!

MsdZ = 38.0242 kNm and McRdZ = 444.3636 kNm

Section is sufficient for moment about the major axis, Z!

SigmaXEd = 171485.3577 kN/m2, fyd = 213636.3590 kN/m2

Section is sufficient for combined effects including shear effect, if any!

VplRdY = 610.3012 kN, VsdY = 23.3403 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 62.7188 kN, VplRdZ = 610.3012 kN

Section is sufficient for shear resistance about the major axis!

Nsd = 128.7963 kN, NbRd = -493.9979 kN

Section is insufficient for the axial force when there is no bending!

MbRd = 551.1818 kNm, MsdY = 100.5938 kNm

Section is sufficient for lateral-torsional buckling!

((Nsd / (Xmin * Area * fy / GammaM1)) + (KY * MsdY / (Wpy * fy / GammaM1))+ (KZ * MsdZ / (Wpz * fy / GammaM1))) = 0.2312

Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 9, Member shape is Box/Tube and it is a Column identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,
Leff is 3.0 m.

Section is assumed to be class 2!

Nsd = 238.4959 kN, NplRd = 1046.8182 kN

Section is sufficient to axial compression force!

MsdY = 0 kNm, McRdY = 14.2356 kNm

Section is sufficient for moment about the minor axis, Y!

MsdZ = 0 kNm, McRdZ = 9.2374 kNm

Section is sufficient for moment about the major axis, Z!

VsdY = 0 kN VplRdY = 226.6428 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 0 kN VplRdZ = 226.6428 kN

Section is sufficient for shear resistance about the major axis!

Nsd = 238.4959 kN NbRd = -20.4232 kN

Section is insufficient for the axial force when there is no bending!

Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 10, Member shape is Pipe and it is a Column identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,
Leff is 3.0 m.

Section is class 1.
Nsd = 161.3657 kN NplRd = 2328.6363 kN
Section is sufficient to axial compression force!
MsdY = 113.6629 kNm McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 37.0846 kNm McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 161.36571 kN, NplRd = 2328.6363 kN, MsdY = 113.6629 kNm, McRdZ = 234.9999 kNm, MsdZ = 37.0848 kNm, McRdY = 234.9999 kNm
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.7108
Section is sufficient for combined effects including shear effect, if any!
VsdY = 23.1718 kN VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 74.2349 kN VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 161.3657 kN NbRd = -46.0168 kN
Section is insufficient for the axial force when there is no bending!
MbRd = 234.9999 kNm MsdY = 113.6629 kNm
Section is sufficient for lateral-torsional buckling!
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 11, Member shape is I/Wide Flange and it is a Column identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,
Leff is 3.0 m.

Flange is class 1.
Web is class 3.
Over all section is class 3.
Nsd = 161.8754 kN and NplRd = 4550.4544 kN
Section is sufficient to axial compression force!
MsdY = 78.7349 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 37.9683 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 141041.6518 kN/m2, fyd = 213636.3590 kN/m2
Section is sufficient for combined effects including shear effect, if any!
VplRdY = 610.3012 kN, VsdY = 23.0565 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 49.8297 kN, VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 161.8754 kN, NbRd = -493.9979 kN
Section is insufficient for the axial force when there is no bending!
MbRd = 551.1818 kNm, MsdY = 78.7349 kNm
Section is sufficient for lateral-torsional buckling!
((Nsd / (Xmin * Area * fy / GammaM1)) + (KY * MsdY / (Wpy * fy / GammaM1))+ (KZ * MsdZ / (Wpz * fy / GammaM1))) = -74.5526
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 12, Member shape is Box/Tube and it is a Column identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,
Leff is 3.0 m.

Section is assumed to be class 2!
Nsd = 235.6557 kN, NplRd = 1046.8182 kN
Section is sufficient to axial compression force!
MsdY = 0 kNm, McRdY = 11.4387 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 0 kNm, McRdZ = 22.1053 kNm
Section is sufficient for moment about the major axis, Z!
VsdY = 0 kN VplRdY = 226.6428 kN

Section is sufficient for shear resistance about the minor axis!
VsdZ = 0 kN VplRdZ = 226.6428 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 235.6557 kN NbRd = -20.4232 kN
Section is insufficient for the axial force when there is no bending!
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 13, Member shape is Pipe and it is a Column identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,
Leff is 3.0 m.

Section is class 1.
Nsd = 131.3108 kN NplRd = 2328.6363 kN
Section is sufficient to axial compression force!
MsdY = 66.6221 kNm McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 37.5543 kNm McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 131.3108 kN, NplRd = 2328.6363 kN, MsdY = 66.6221 kNm, McRdZ = 234.9999 kNm, MsdZ = 37.5543 kNm, McRdY = 234.9999 kNm
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.4997
Section is sufficient for combined effects including shear effect, if any!
VsdY = 23.4558 kN VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 42.6466 kN VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 131.3108 kN NbRd = -46.0168 kN
Section is insufficient for the axial force when there is no bending!
MbRd = 234.9999 kNm MsdY = 66.6221 kNm
Section is sufficient for lateral-torsional buckling!
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 14, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,

rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.
Web is class 3.
Over all section is class 3.
MsdY = 1.8179 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 242.2347 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 119996.2863 KN/m2, and fyd = 213636.3590 kN/m2
Section is sufficient for combined effects including shear effect, if any!
VsdY = 165.1825 kN, and VplRdY = 610.3012 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.6492 kN, and VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
MbRd = 551.1818 kNm, and MsdY = 1.8179 kNm
Section is sufficient for lateral-torsional buckling!

Member ID No. is 15, Member shape is Box/Tube and it is a Beam identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,

Section is assumed to be class 2!
MsdY = 0 kNm, and McRdY = 34.7258 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 245 kNm, and McRdZ = 114.7138 kNm
Section is insufficient for moment about the major axis, Z!
MsdY = 0 kNm, and MNRdY = 14.7183 kNm
Section is sufficient for axial force and uni-axial bending including shear effect, if any!
VsdY = 210 kN, and VplRdY = 226.6428 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0 kN, and VplRdZ = 226.6428 kN

Section is sufficient for shear resistance about the major axis!

Member ID No. is 16, Member shape is Pipe and it is a Beam identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,

Section is class 1.
MsdY = 2.9921 kNm, and McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 66.7496 kNm, and McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 4.2332 kN, NplRd = 2328636.3132 kN, MsdY = 2.9921 kNm, McRdZ = 234.9999 kNm, MsdZ = 66.7496 kNm and McRdY = 234.9999 kNm.
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.2968
Section is sufficient for combined effects including shear effect, if any!
VsdY = 91.4661 kN and VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.9798 kN and VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 17, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.
Web is class 3.
Over all section is class 3.
MsdY = 2.3898 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!

MsdZ = 209.7236 kNm and McRdZ = 444.3636 kNm

Section is sufficient for moment about the major axis, Z!

SigmaXEd = 105960.0184 KN/m2, and fyd = 213636.3590 kN/m2

Section is sufficient for combined effects including shear effect, if any!

VsdY = 220.3033 kN, and VplRdY = 610.3012 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 0.6493 kN, and VplRdZ = 610.3012 kN

Section is sufficient for shear resistance about the major axis!

MbRd = 523.2905 kNm, and MsdY = 2.3898 kNm

Section is sufficient for lateral-torsional buckling!

Member ID No. is 18, Member shape is Box/Tube and it is a Beam identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,

Section is assumed to be class 2!

MsdY = 0 kNm, and McRdY = 5.1247 kNm

Section is sufficient for moment about the minor axis, Y!

MsdZ = 125 kNm, and McRdZ = 113.1245 kNm

Section is insufficient for moment about the major axis, Z!

MsdY = 0 kNm, and MNRdY = 51.4381 kNm

Section is sufficient for axial force and uni-axial bending including shear effect, if any!

VsdY = 150 kN, and VplRdY = 226.6428 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 0 kN, and VplRdZ = 226.64278 kN

Section is sufficient for shear resistance about the major axis!

Member ID No. is 19, Member shape is Pipe and it is a Beam identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,

Section is class 1.

MsdY = 2.8305 kNm, and McRdY = 234.9999 kNm

Section is sufficient for moment about the minor axis, Y!

MsdZ = 66.6716 kNm, and McRdZ = 234.9999 kNm

Section is sufficient for moment about the major axis, Z!

Nsd = 4.4072 kN, NplRd = 2328636.3132 kN, MsdY = 2.8305 kNm, McRdZ = 234.9999 kNm, MsdZ = 66.671555 kNm and McRdY = 234.9999 kNm.

((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.2958

Section is sufficient for combined effects including shear effect, if any!

VsdY = 91.4721 kN and VplRdY = 855.8963 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 1.0058 kN and VplRdZ = 855.8963 kN

Section is sufficient for shear resistance about the major axis!

Member ID No. is 20, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.

Web is class 3.

Over all section is class 3.

MsdY = 2.6852 kNm and McRdY = 159.3727 kNm

Section is sufficient for moment about the minor axis, Y!

MsdZ = 91.5552 kNm and McRdZ = 444.3636 kNm

Section is sufficient for moment about the major axis, Z!

SigmaXEd = 48056.8606 KN/m2, and fyd = 213636.3590 kN/m2

Section is sufficient for combined effects including shear effect, if any!

VsdY = 90.0568 kN, and VplRdY = 610.3012 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 1.0948 kN, and VplRdZ = 610.3012 kN

Section is sufficient for shear resistance about the major axis!

MbRd = 551.1818 kNm, and MsdY = 2.6852 kNm

Section is sufficient for lateral-torsional buckling!

Member ID No. is 21, Member shape is I/Wide Flange and it is a Column identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,
Leff is 3.0 m.

Flange is class 1.
Web is class 3.
Over all section is class 3.
Nsd = 352.0417 kN and NplRd = 4550.4544 kN
Section is sufficient to axial compression force!
MsdY = 100.9784 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 45.6370 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 186195.5966 kN/m2, fyd = 213636.3590 kN/m2
Section is sufficient for combined effects including shear effect, if any!
VplRdY = 610.3012 kN, VsdY = 27.0986 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 59.4946 kN, VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 352.0417 kN, NbRd = -493.9979 kN
Section is insufficient for the axial force when there is no bending!
MbRd = 551.1818 kNm, MsdY = 100.9784 kNm
Section is sufficient for lateral-torsional buckling!
((Nsd / (Xmin * Area * fy / GammaM1)) + (KY * MsdY / (Wpy * fy / GammaM1)) + (KZ * MsdZ / (Wpz * fy / GammaM1))) = -201.6097
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 22, Member shape is Box/Tube and it is a Column identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,

Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,
Leff is 3.0 m.

Section is assumed to be class 2!
Nsd = 703.7952 kN, NplRd = 1046.8182 kN
Section is sufficient to axial compression force!
MsdY = 0 kNm, McRdY = 18.4297 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 0 kNm, McRdZ = 41.2726 kNm
Section is sufficient for moment about the major axis, Z!
VsdY = 0 kN VplRdY = 226.6428 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0 kN VplRdZ = 226.6428 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 703.7952 kN NbRd = -20.4232 kN
Section is insufficient for the axial force when there is no bending!
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 23, Member shape is Pipe and it is a Column identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,
Leff is 3.0 m.

Section is class 1.
Nsd = 462.8315 kN NplRd = 2328.6363 kN
Section is sufficient to axial compression force!
MsdY = 139.1194 kNm McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 45.71503 kNm McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 462.8315 kN, NplRd = 2328.6363 kN, MsdY = 139.1194 kNm, McRdZ = 234.9999 kNm, MsdZ = 45.7150 kNm, McRdY = 234.9999 kNm
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.9852
Section is sufficient for combined effects including shear effect, if any!
VsdY = 27.4047 kN VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!

VsdZ = 90.3899 kN VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 462.8315 kN NbRd = -46.0168 kN
Section is insufficient for the axial force when there is no bending!
MbRd = 234.9999 kNm MsdY = 139.1194 kNm
Section is sufficient for lateral-torsional buckling!
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 24, Member shape is I/Wide Flange and it is a Column identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,
Leff is 3.0 m.

Flange is class 1.
Web is class 3.
Over all section is class 3.
Nsd = 470.7729 kN and NplRd = 4550.4544 kN
Section is sufficient to axial compression force!
MsdY = 120.1565 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 44.6484 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 219342.6296 kNm, fyd = 213636.3590 KNm
Section is insufficient for combined effects including shear effect, if any!
VplRdY = 610.3012 kN, VsdY = 26.6411 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 79.6916 kN, VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 470.7729 kN, NbRd = -493.9979 kN
Section is insufficient for the axial force when there is no bending!
MbRd = 551.1818 kNm, MsdY = 120.1565 kNm
Section is sufficient for lateral-torsional buckling!
((Nsd / (Xmin * Area * fy / GammaM1)) + (KY * MsdY / (Wpy * fy / GammaM1))+ (KZ * MsdZ / (Wpz * fy / GammaM1))) = -331.3391
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 25, Member shape is Box/Tube and it is a Column identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,
Leff is 3.0 m.

Section is assumed to be class 2!
Nsd = 675.2359 kN, NplRd = 1046.8182 kN
Section is sufficient to axial compression force!
MsdY = 0 kNm, McRdY = 14.2543 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 0 kNm, McRdZ = 16.4234 kNm
Section is sufficient for moment about the major axis, Z!
VsdY = 0 kN VplRdY = 226.6428 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0 kN VplRdZ = 226.6428 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 675.2359 kN NbRd = -20.4232 kN
Section is insufficient for the axial force when there is no bending!
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 26, Member shape is Pipe and it is a Column identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,
Leff is 3.0 m.

Section is class 1.
Nsd = 372.8222 kN NplRd = 2328.6363 kN
Section is sufficient to axial compression force!
MsdY = 112.9458 kNm McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!

MsdZ = 46.3409 kNm McRdZ = 234.9999 kNm

Section is sufficient for moment about the major axis, Z!

Nsd = 372.8222 kN, NplRd = 2328.6363 kN, MsdY = 112.9458 kNm, McRdZ = 234.9999 kNm, MsdZ = 46.3409 kNm, McRdY = 234.9999 kNm

((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.8379

Section is sufficient for combined effects including shear effect, if any!

VsdY = 27.8628 kN VplRdY = 855.8963 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 59.2887 kN VplRdZ = 855.8963 kN

Section is sufficient for shear resistance about the major axis!

Nsd = 372.8222 kN NbRd = -46.0168 kN

Section is insufficient for the axial force when there is no bending!

MbRd = 234.9999 kNm MsdY = 112.9458 kNm

Section is sufficient for lateral-torsional buckling!

Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 27, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.

Web is class 3.

Over all section is class 3.

MsdY = 1.1739 kNm and McRdY = 159.3727 kNm

Section is sufficient for moment about the minor axis, Y!

MsdZ = 280.3844 kNm and McRdZ = 444.3636 kNm

Section is sufficient for moment about the major axis, Z!

SigmaXEd = 138976.7698 KN/m2, and fyd = 213636.3590 kN/m2

Section is sufficient for combined effects including shear effect, if any!

VsdY = 205.2425 kN, and VplRdY = 610.3012 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 0.4227 kN, and VplRdZ = 610.3012 kN

Section is sufficient for shear resistance about the major axis!

MbRd = 551.1818 kNm, and MsdY = 1.1739 kNm

Section is sufficient for lateral-torsional buckling!

Member ID No. is 28, Member shape is Box/Tube and it is a Beam identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,

Section is assumed to be class 2!
MsdY = 0 kNm, and McRdY = -33717.5378 kNm
Section is insufficient for moment about the minor axis, Y!
MsdZ = 273.5833 kNm, and McRdZ = 225.4573 kNm
Section is insufficient for moment about the major axis, Z!
MsdY = 0 kNm, and MNRdY = 20.1245 kNm
Section is sufficient for axial force and uni-axial bending including shear effect, if any!
VsdY = 234.5 kN, and VplRdY = 226.6428 kN
Section is insufficient for shear resistance about the minor axis!
VsdZ = 0 kN, and VplRdZ = 226.6428 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 29, Member shape is Pipe and it is a Beam identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,

Section is class 1.
MsdY = 1.0796 kNm, and McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 68.6689 kNm, and McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 18.8443 kN, NplRd = 2328636.3132 kN, MsdY = 1.0796 kNm, McRdZ = 234.9999 kNm, MsdZ = 68.6689 kNm and McRdY = 234.9999 kNm.
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.2968
Section is sufficient for combined effects including shear effect, if any!
VsdY = 101.6583 kN and VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!

VsdZ = 0.2633 kN and VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 30, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.
Web is class 3.
Over all section is class 3.
MsdY = 1.5710 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 227.0723 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 113603.9438 KN/m2, and fyd = 213636.3590 kN/m2
Section is sufficient for combined effects including shear effect, if any!
VsdY = 245.1681 kN, and VplRdY = 610.3012 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.4227 kN, and VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
MbRd = 523.1190 kNm, and MsdY = 1.5710 kNm
Section is sufficient for lateral-torsional buckling!

Member ID No. is 31, Member shape is Box/Tube and it is a Beam identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,

It = 0.0000624 m4,

Section is assumed to be class 2!
MsdY = 0 kNm, and McRdY = 22.4218 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 139.58332824707 kNm, and McRdZ = 102.5729 kNm
Section is insufficient for moment about the major axis, Z!
MsdY = 0 kNm, and MNRdY = 40.1729 kNm
Section is sufficient for axial force and uni-axial bending including shear effect, if any!
VsdY = 167.5 kN, and VplRdY = 226.6428 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0 kN, and VplRdZ = 226.6428 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 32, Member shape is Pipe and it is a Beam identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,

Section is class 1.
MsdY = 0.9267 kNm, and McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 68.4655 kNm, and McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 19.1842 kN, NplRd = 2328636.3132 kN, MsdY = 0.9267 kNm, McRdZ = 234.9999 kNm, MsdZ = 68.4655 kNm and McRdY = 234.9999 kNm.
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.2953
Section is sufficient for combined effects including shear effect, if any!
VsdY = 101.6196 kN and VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.2646 kN and VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!

Member ID No. is 33, Member shape is I/Wide Flange and it is a Beam identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,

Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Flange is class 1.
Web is class 3.
Over all section is class 3.
MsdY = 1.2286 kNm and McRdY = 159.3727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 104.2930 kNm and McRdZ = 444.3636 kNm
Section is sufficient for moment about the major axis, Z!
SigmaXEd = 52051.2813 KN/m2, and fyd = 213636.3590 kN/m2
Section is sufficient for combined effects including shear effect, if any!
VsdY = 101.4007 kN, and VplRdY = 610.3012 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 0.4674 kN, and VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
MbRd = 551.1818 kNm, and MsdY = 1.2286 kNm
Section is sufficient for lateral-torsional buckling!

Member ID No. is 34, Member shape is I/Wide Flange and it is a Column identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,
Leff is 4.5 m.

Flange is class 1.
Web is class 1.
Over all section is class 1.
Nsd = 583.3519 kN and NplRd = 4550.4544 kN
Section is sufficient to axial compression force!
MsdY = 4.8414 kNm and McRdY = 239.2727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 30.9919 kNm and McRdZ = 551.1818 kNm

Section is sufficient for moment about the major axis, Z!

Nsd = 583.3519 kN, NplRd = 4550.4544 kN, MsdY = 4.8414 kNm, McRdZ = 551.1818 KNm, MsdZ = 30.9919 kNm and McRdY = 239.2727 kNM

((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.2665

Section is sufficient for combined effects including shear effect, if any!

VplRdY = 610.3012 kN, VsdY = 7.4917 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 1.3698 kN, VplRdZ = 610.3012 kN

Section is sufficient for shear resistance about the major axis!

Nsd = 583.3519 kN, NbRd = -89.9227 kN

Section is insufficient for the axial force when there is no bending!

MbRd = 551.1818 kNm, MsdY = 4.8414 kNm

Section is sufficient for lateral-torsional buckling!

((Nsd / (Xmin * Area * fy / GammaM1)) + (KY * MsdY / (Wpy * fy / GammaM1))+ (KZ * MsdZ / (Wpz * fy / GammaM1))) = -126.2234

Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 35, Member shape is Box/Tube and it is a Column identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,
Leff is 4.5 m.

Section is assumed to be class 2!

Nsd = 1245.0052 kN, NplRd = 1046.8182 kN

Section is sufficient to axial compression force!

MsdY = 0 kNm, McRdY = 14.7249 kNm

Section is sufficient for moment about the minor axis, Y!

MsdZ = 0 kNm, McRdZ = 17.5781 kNm

Section is sufficient for moment about the major axis, Z!

VsdY = 0 kN VplRdY = 226.6428 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 0 kN VplRdZ = 226.6428 kN

Section is sufficient for shear resistance about the major axis!

Nsd = 1245.0052 kN NbRd = -127.2114 kN

Section is insufficient for the axial force when there is no bending!

Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 36, Member shape is Pipe and it is a Column identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,
Leff is 4.5 m.

Section is class 1.
Nsd = 798.1281 kN NplRd = 2328.6363 kN
Section is sufficient to axial compression force!
MsdY = 127.1415 kNm McRdY = 234.9999 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 22.7501 kNm McRdZ = 234.9999 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 798.1281 kN, NplRd = 2328.6363 kN, MsdY = 127.1415 kNm, McRdZ = 234.9999 kNm, MsdZ = 22.7501 kNm, McRdY = 234.9999 kNm
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.9806
Section is sufficient for combined effects including shear effect, if any!
VsdY = 8.5603 kN VplRdY = 855.8963 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 43.5652 kN VplRdZ = 855.8963 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 798.1281 kN NbRd = -55.9535 kN
Section is insufficient for the axial force when there is no bending!
MbRd = 234.9999 kNm MsdY = 127.1415 kNm
Section is sufficient for lateral-torsional buckling!
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 37, Member shape is I/Wide Flange and it is a Column identified by W250x167.

h = 0.289 m,
b = 0.265 m,
tf = 0.032 m,
tw = 0.019 m,
A = 0.0213 m2,
Iz = 0.0003 m4,
Wez = 0.00208 m3,
Wpz = 0.00258 m3,
rz = 0.119 m,
Iy = 0.0000988 m,
Wey = 0.000746 m3,
Wpy = 0.00112 m3,
ry = 0.0681 m,
It = 0.000399 m4,
Iw = 0.00000207 m6,

Leff is 4.5 m.

Flange is class 1.
Web is class 1.
Over all section is class 1.
Nsd = 817.7113 kN and NplRd = 4550.4544 kN
Section is sufficient to axial compression force!
MsdY = 110.0497 kNm and McRdY = 239.2727 kNm
Section is sufficient for moment about the minor axis, Y!
MsdZ = 30.6846 kNm and McRdZ = 551.1818 kNm
Section is sufficient for moment about the major axis, Z!
Nsd = 817.71130 kN, NplRd = 4550.4544 kN, MsdY = 110.0497 kNm, McRdZ = 551.1818 KNm, MsdZ = 30.6846 kNm and McRdY = 239.2727 kNM
((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.5076
Section is sufficient for combined effects including shear effect, if any!
VplRdY = 610.3012 kN, VsdY = 7.3742 kN
Section is sufficient for shear resistance about the minor axis!
VsdZ = 38.7813 kN, VplRdZ = 610.3012 kN
Section is sufficient for shear resistance about the major axis!
Nsd = 817.7113 kN, NbRd = -109.3403 kN
Section is insufficient for the axial force when there is no bending!
MbRd = 551.1818 kNm, MsdY = 110.0497 kNm
Section is sufficient for lateral-torsional buckling!
((Nsd / (Xmin * Area * fy / GammaM1)) + (KY * MsdY / (Wpy * fy / GammaM1))+ (KZ * MsdZ / (Wpz * fy / GammaM1))) = -655.6365
Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 38, Member shape is Box/Tube and it is a Column identified by HSS254.0x152.4x6.35.

h = 0.254 m,
b = 0.1524 m,
t = 0.00635 m,
A = 0.0049 m2,
Iz = 0.0000429 m4,
Wez = 0.000338 m3,
Wpz = 0. 000435 m3,
rz = 0.0936 m,
Iy = 0.0000195 m4,
Wey = 0.000256 m3,
Wpy = 0. 000142 m3,
ry = 0.0631 m,
It = 0.0000624 m4,
Leff is 4.5 m.

Section is assumed to be class 2!
Nsd = 1167.6008 kN, NplRd = 1046.8182 kN
Section is sufficient to axial compression force!
MsdY = 0 kNm, McRdY = 13.2452 kNm

Section is sufficient for moment about the minor axis, Y!

MsdZ = 0 kNm, McRdZ = 28.1927 kNm

Section is sufficient for moment about the major axis, Z!

VsdY = 0 kN VplRdY = 226.6428 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 0 kN VplRdZ = 226.6428 kN

Section is sufficient for shear resistance about the major axis!

Nsd = 1167.6008 kN NbRd = -127.2114 kN

Section is insufficient for the axial force when there is no bending!

Section is sufficient for combined axial force and bending moment resistance!

Member ID No. is 39, Member shape is Pipe and it is a Column identified by SS323.9x11.13.

D = 0.3239 m,
t = 0.01113 m,
A = 0.0109 m2,
Iz = 0.000134 m4,
Wez = 0.000827 m3,
Wpz = 0.0011 m3,
rz = 0.111 m,
It = 0.000268 m4,
Leff is 4.5 m.

Section is class 1.

Nsd = 641.2027 kN NplRd = 2328.6363 kN

Section is sufficient to axial compression force!

MsdY = 18.4036 kNm McRdY = 234.9999 kNm

Section is sufficient for moment about the minor axis, Y!

MsdZ = 23.1187 kNm McRdZ = 234.9999 kNm

Section is sufficient for moment about the major axis, Z!

Nsd = 641.2027 kN, NplRd = 2328.6363 kN, MsdY = 18.4036 kNm, McRdZ = 234.9999 kNm, MsdZ = 23.1187 kNm, McRdY = 234.9999 kNm

((Nsd / NplRd) + (MsdY / McRdZ) + (MsdZ / McRdY)) = 0.4520

Section is sufficient for combined effects including shear effect, if any!

VsdY = 8.6786 kN VplRdY = 855.8963 kN

Section is sufficient for shear resistance about the minor axis!

VsdZ = 4.2177 kN VplRdZ = 855.8963 kN

Section is sufficient for shear resistance about the major axis!

Nsd = 641.2027 kN NbRd = -55.953 kN

Section is insufficient for the axial force when there is no bending!

MbRd = 234.9999 kNm MsdY = 18.4036 kNm

Section is sufficient for lateral-torsional buckling!

Section is sufficient for combined axial force and bending moment resistance!

Chapter 5 CONCLUSIONS AND RECOMMENDATIONS

Because of advances in structural technology, greater accuracy and speed in the analysis of structural systems and a number of alternatives to be evaluated in the design of structures are required. As a result practicing engineers make use of available application softwares for analysis and design of structures. The softwares that are currently used by our practicing engineers do not consider local conditions. Thus these engineers may not be able to incorporate local condition that should be included.

In this study an effort has bean made to develop an application program for the analysis of steel space frame structures and design of members of the structure that takes national code provisions and utilizes EBCS 3 1995 specification . The program being unique in that it provides a user interface in each of tasks to be performed in the input of data, analyzing the structure and designing members with graphical presentation of results such as axial force, shear force and bending moment diagrams, etc.

Practicing engineers may be used EADoSSF at least for preliminary analysis and design of steel space frame structures.

Further more it can be used as a base to a complete application program software in the analysis and design of structures which accounts local conditions.

The author also believes that the study can be used as a teaching tool specifically in the study of matrix method of structural analysis and in some design courses.

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Annexes

Annex 1 Flowchart

Please refer a file named by “Thesis Flowchart.pdf” on the CD attached with this paper at the inside of back cover.

Annex 2 Source code

Please refer to a file named by “Thesis Source Code.pdf” on the CD attached with this paper at the inside of back cover.

DECLARATION

I, the undersigned, declare that this thesis is my work and all sources of materials used for the thesis have been duly acknowledged.

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Signature	___________________
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Date of submission	September 2006