CHAPTER ONE

INTODUCTION

1.1 Introduction

Analysis of steel framed structures includes more complexity when compared with RC framed structures The steel material behavior and its mechanical properties adds problems in analysis of steel structure to consider the effects of residual stresses , initial geometric imperfections and tendency of buckling, Also steel structures are capable to undergo large deformation before collapse. This will causes the second order effects. Steel possesses excellent ductility and post elastic strength. All these facts make the analysis of steel framed structure more complex especially when the design is required by limit state concept.

1.2 Research Problem

The research problem is the comparison of structural performance between the elastic and plastic design of unbraced steel frames.

1.3 Research objectives

The objectives of research were summarized as follows:

1. Study of the Linear and non-linear behavior of 2D steel frames
2. Review of the elastic and plastic analysis of unbraced 2D steel frames.
3. Programming of mechanism method in MATLAB code to obtain the critical load factors.
4. Applying the computer analysis programs SAP 2000 and MASTAN2, for first and second orders in elastic analysis of different unbraced steel 2D frames.
5. Comparison between the elastic and plastic design of structural elements of frames, which were programmed by excel spread sheet.

1.4 Research layout

Thesis is divided into five chapters. Chapter One contains the introduction, research problem, and research objectives. Chapter Two contains the previous studies, elastic and plastic Behavior of steel frames, types of steel frame, steel frame analysis and design methods. Chapter Three presents Frames Analysis by different programs using structural programs MASTAN2 and SAP 2000. The MATLAB computer implementation and its flow chart of the frames to calculate the critical load factor may also be included in this chapter. In Chapter Four, comparison of analysis Results were presented by MASTAN2 and SAP 2000. The elastic and plastic design of structural elements was explained in this chapter. Chapter Five presents the Conclusions and recommendations.

1.5 Research methodology

In this research, the first and second order analysis of frames were done using different computer programs such as MASTAN2 and SAP 2000 for elastic and plastic analysis methods. Loads calculations were done using excel spread sheet. The critical load factors for 2D frames were done using mechanism method, which programmed by MATLAB code. The elastic and plastic design of frames done using American Institute of Steel Construction (AISC) by preparing excel spread sheet.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Structural steel is an important construction material because its properties such as strength, stiffness, toughness, and ductility that is very desirable in modern constructions.

When constructing a larger building that needs a big open space the Steel frames are usually the choice because of the economical aspect and efficiency of a single-storey building. However, a problem that might occur is when designing for a cost effective solution the slenderness may be decreased, that in the end may contribute to instability of the entire structure.

2.2 Previous Studies

Nicholas s Trahair, Trends in The Analysis and Design of Steel Framed Structures, School of Civil Engineering The University of Sydney, Sydney NSW 2006 Australia, February 2012. The paper surveys trends in the analysis and design of steel framed structures with reference to design codes such as the US AISC Specification, the UK BS5950, the Australian AS4100, the European EC3, and the Hong Kong Code of Practice . A possible future solution to these problems is to allow the use of purpose-built computer programs, which can provide accurate predictions of member strength. Thus future design codes might only describe the characteristics of the structural analysis method and those of determining the design strengths of structural members which may be used. Such a code would have all the inaccuracies and shortcomings of approximating the member strengths removed and replaced by more accurate member strength computer programs. To some extent, this is already in place with the present practice of some codes which either allow or require design by elastic buckling analysis.

Shrikant S. Ingale*1, Dr. M. R. Shiyekar, Second Order Inelastic behavior of Steel Moment Restraing Frame , World Journal of Engineering Science, 08 Aug 2013, this paper review of different methods of analysis of steel frame is studied and attempt is made to establish relation between analysis results of first order elastic and second order inelastic analysis. In this dissertation work attempt is made to establish relation between analysis results of first order elastic and second order inelastic analysis, which may help structural designers to use codal provision of IS800:2007 in a more convenient manner.

2.3 Elastic and Plastic Behavior of Steel [1]

Most structural materials undergo an elastic state before a plastic state is reached. This applies to both material behavior of a cross section and the structure as a whole.

For a simply supported steel beam with a cross section symmetrical about a horizontal axis under an increasing load applied at mid-span, the general stress and strain variations in the cross section at mid-span from a fully elastic state to fracture are shown in Figure 2.1.

The beam is initially loaded producing an elastic stress $f_e$corresponding to an elastic strain ε_e for loading between points A and B. When the load is reached, the maximum stress in the top and bottom fibers of the cross section becomes yielded stress f_y, corresponds to a yield strain ε_y.As the load is increased further, the cross section undergoes a plasticity process in which the yielded area becomes larger and larger spreading inward toward the center of the cross section. This plasticity with a relatively constant yield stress f_y occurs between B and C, at which the stress corresponding to strains starts to increase again. From point C, the cross section enters into a strain-hardening stage until an ultimate stress (fu) at point D is reached. From point D, the stress starts to decrease with increasing strain until the material fractures at point E. The plasticity process is important for steel in plastic design as it ensures that the material has adequate ductility for the cross section to sustain loading beyond its elastic limit at point B.

For design purposes, it is prudent to ignore the extra strength provided by strain hardening, which becomes smaller in magnitude as the grade strength of steel becomes greater. Hence, for simplicity, steel is always idealized as an elastic-perfectly plastic material with a stress–strain relationship as shown in Figure 2.2. also In Figure 2.2, the stress–strain relationship for the elastic part AB is linear and its slope is equal to the modulus of elasticity.

The corresponding cross-section plasticity of a symmetric section was presented as shown in Figure 2.3

2.4 Moment-Curvature Relationship in an Elastic–Plastic Range [1]

A cross section under increasing bending moment undergoes three stages of transformation in its plasticity process. As shown in Figure 2.4, For $M < M_y$, the section behaves elastically, giving the straight line AB. At point B, the moment equals the yield moment My, and the M-ϕ relation is no longer linear. Finally, as the moment tends towards the fully plastic moment, the curvature ϕ tends to infinity they are elastic (AB), elastic–plastic (BD), and fully plastic (DE).

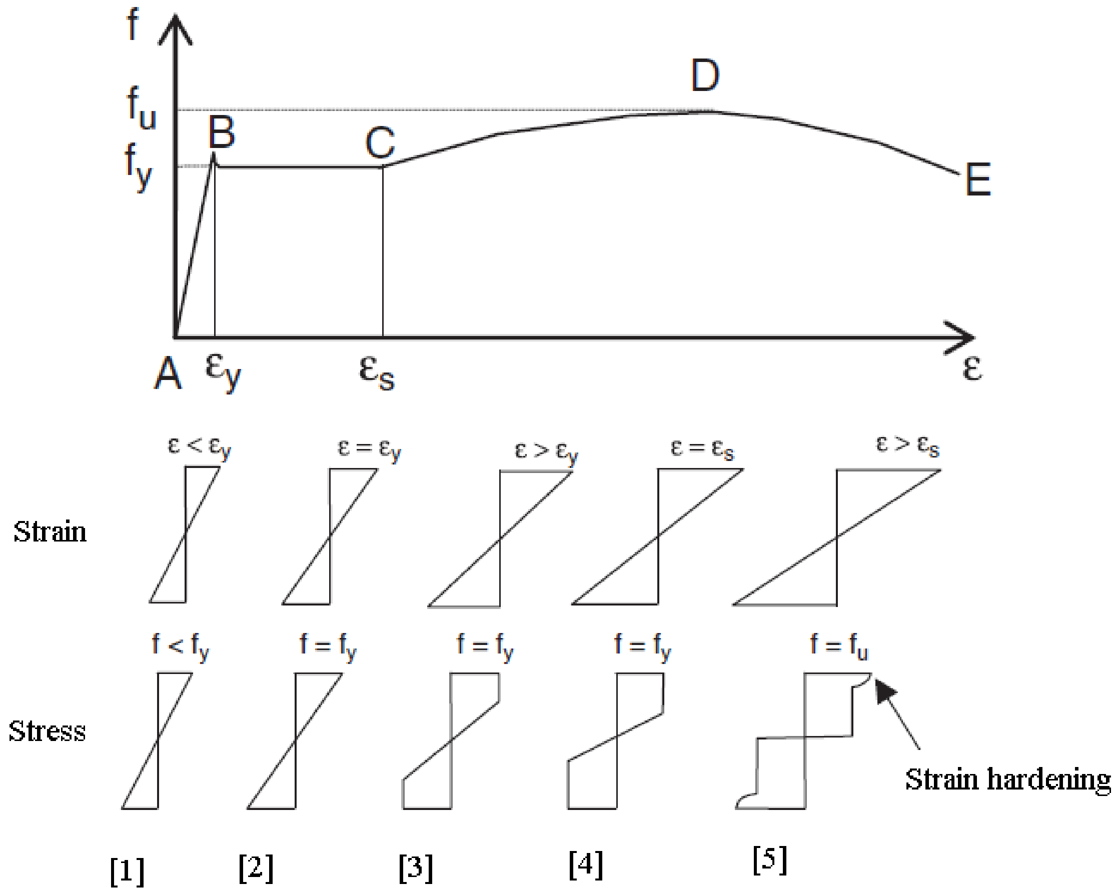


Figure 2.1.Stress–strain behavior of a cross section.

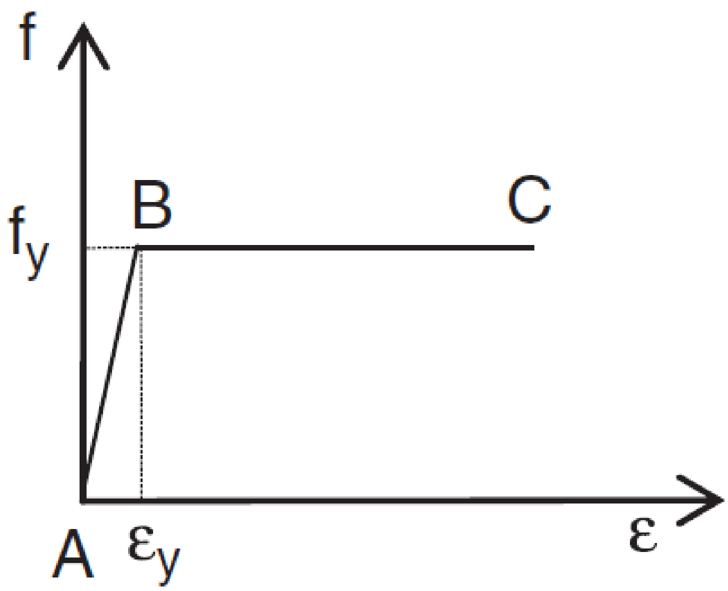


Figure2.2.Elastic perfectly plastic behavior for steel.

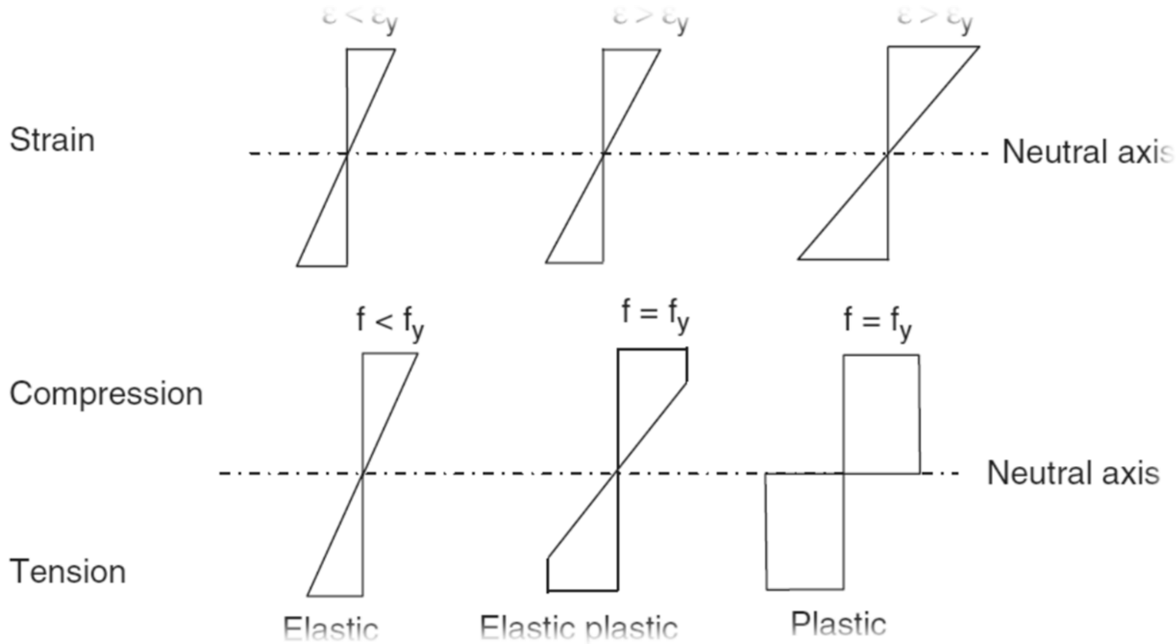


Figure2.3. Plastic of a cross section.

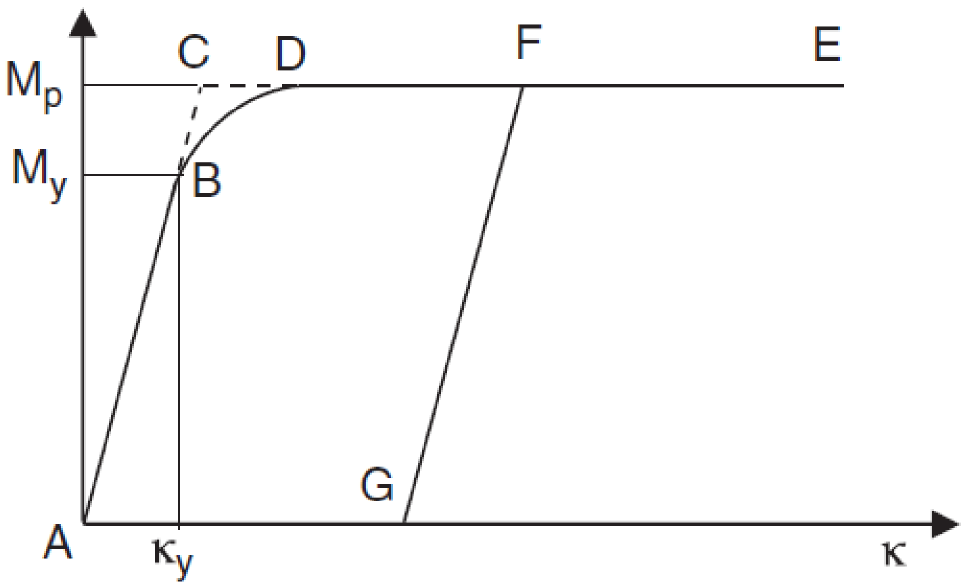


Figure 2.4.Moment–curvature relationship of a cross section.

The P-Δ and P-δ effects: Modern design provisions are based on the principle that the member forces are calculated by a second-order elastic analysis, where the equilibrium is satisfied on the deformed geometry of the structure. The effects of the loads acting on the deformed geometry of the structure are known as the second-order or the P-Delta effects

The P-Delta effects come from two sources: global lateral translation of the frame and the local deformation of members within the frame as shown in figure 2.5.

First-order analysis: In which equilibrium is formulated for the un deformed position of the structure, so that the moments caused by products of the loads and deflections are ignored.

Second-order analysis: In which equilibrium is formulated for the deformed position of the structure, so that the moments caused by the products of the loads and deflections are included.

Shape Factor: is a ratio of the maximum resisting moment of a cross-section to the yield moment $[\frac{M_p}{M_y}]$, the shape factor for rectangular section 1.5 and varies from about 1.10 to 1.20 for standard rolled-beam sections.

Safety Factor: is obtained by dividing the yield point of the steel by its working stress for compact laterally supported beam of A36 steel this safety factor is1.5 using the ASD Specification

In plastic design, a factor by which the working load is multiplied to determine the ultimate load, is called the load factor which is equal product of the safety factor is and shape factor. For Rectangular Section dead load and live loads only is multiplied by 1.70 and 1.30 for dead, live, and Wind or earthquake Loads combined

A plastic hinge is said to form in a structural member when the cross-section is fully yielded. If material strain hardening is not considered in the analysis, a fully yielded cross-section can undergo indefinite rotation at a constant restraining plastic moment Mp.

Most of the plastic analyses assume that plastic hinges are concentrated at zero length plasticity.

Maximum moment of resistance of a fully-yielded cross-section is called Plastic Moment (Mp)

Mechanism: A system of members than can move without an increase in load.

Redistribution of Moment is a process which results in the successive formation of plastic hinges until the ultimate load is reached, By it, the less highly stressed portions of a structure also may reach the (M_p)-value.

2.5 Type of steel frames [2]

Structural frames are composed of one-dimensional members connected together in skeletal arrangements, which transfer the applied loads to the supports. The behavior of a structural frame depends on its arrangement and loading, and on the type of connections used.

Steel frames have been widely used in single-storey, low-rise industrial buildings, power plants, ore mines, oil and gas offshore platforms and multi-storey, high-rise buildings as shown in Figure 2.6.

For building frame design, it is useful to define various frame systems in order to simplify analysis of models:

2.5.1. Rigid Frames [2]

A rigid frame derives its lateral stiffness mainly from the bending rigidity of frame members interconnected by rigid joints. The joints shall be designed in such a manner that they have adequate strength and stiffness and negligible deformation.

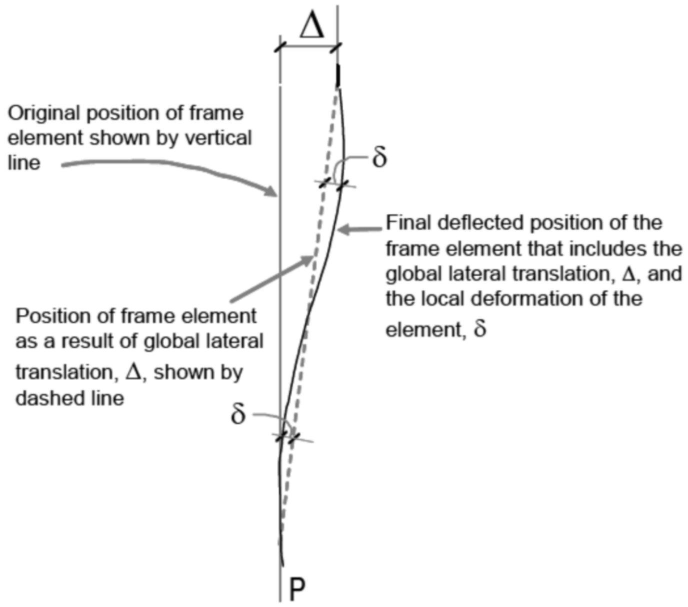


Figure 2.5.The P-Δ and P-δ effects.

The deformation must be small enough to have any significant influence on the distribution of internal forces and moments in the structure or on the overall frame deformation. A rigid unbraced frame should be capable of resisting lateral loads without relying on an additional bracing system for stability.

The frame, by itself, has to resist all the design forces, including gravity as well as lateral forces. At the same time, it should have adequate lateral stiffness against sideway when it is subjected to horizontal wind or earthquake loads. Even though the detailing of the rigid connections results in a less economic structure, rigid unbraced frame systems have the following benefits:

* Rigid connection is more ductile and therefore the structure performs better in load reversal situations or in earthquakes.

* From the architectural and functional points of view, it can be advantageous not to have any triangulated bracing systems or solid wall systems in the building.

2.5.2 Simple Frames (Pin-Connected Frames) [2]

A simple frame refers to a structural system in which the beams and columns are pinned connected and the system is incapable of resisting any lateral loads. The stability of the entire structure must be provided for by attaching the simple frame to some form of bracing system. The lateral loads are resisted by the bracing systems while the gravity loads are resisted by both the simple frame and the bracing system.

In most cases, the lateral load response of the bracing system is sufficiently small such that second order effects may be neglected for the design of the frames.

There are several reasons of adopting pinned connections in the design of steel multistory frames:

1 Pin-jointed frame is easier to fabricate and erect. For steel structures, it is more convenient to join the webs of the members without connecting the flanges.

2. Bolted connections are preferred over welded connections, which normally require weld inspection, weather protection and surface preparation.

3. It is easier to design and analyze a building structure that can be separated into system resisting vertical loads and system resisting horizontal loads. For example, if all the girders are simply supported between the columns, the sizing of the simply supported girders and the columns is a straight forward task.

4 It is more cost effective to reduce the horizontal drift by means of bracing systems added to the simple framing than to use unbraced frame systems with rigid connections.

(a) **(b)**

(c)

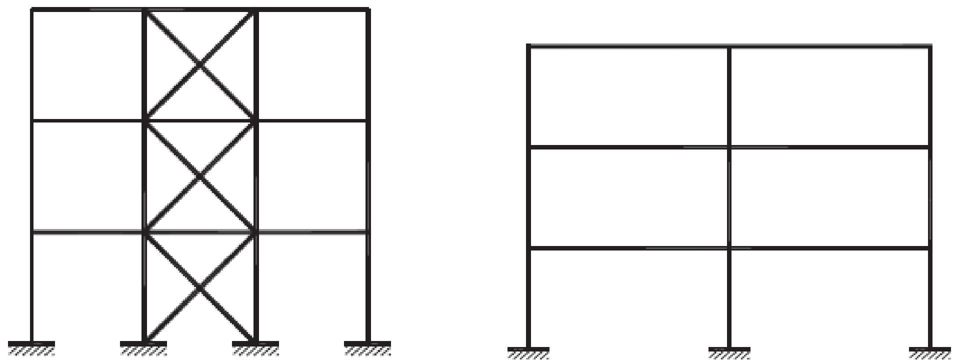
(d)

(e)

Figure 2.6.Types of Steel Frames: (a) low-rise industrial buildings; (b) power plants; (c) ore-mines; (d) offshore plant forms; and (e) multi-story high-rise buildings.

2.5.3. Bracing Systems [2]

The main function of a bracing system is to resist lateral forces. Building frame systems can be separated into vertical load-resistance and horizontal load-resistance systems. Bracing systems refer to structures that can provide lateral stability to the overall framework. Common bracing systems is trusses or shear walls. In steel structures, it is common to represent a bracing system by a triangulated truss because, unlike concrete structures where all the joints are naturally continuous, the most immediate way of making connections between steel members is to hinge one member to the other. As a result, common steel building structures are designed to have bracing systems in order to provide side sway resistance. Therefore, bracing can only be obtained by use of triangulated trusses .The efficiency of a building to resist lateral forces depends on the location and the types of the bracing systems employed see figure 2.7.



(a) **(b)**

Figure 2.7.Braced and Unbraced Frame: (a) Braced Frame and (b) Unbraced Frame.

2.5.4. Sway Frames and Non-Sway Frames [2]

Normally a frame with bracing is classified as non-sway, while an unbraced frame is classified as sway. The identification of sway frames and non-sway frames in a building is useful for evaluating safety of structures against instability. In the design of multi-story building frame, it is convenient to isolate the columns from the frame and treat the stability of columns and the stability of frames as independent problems. For a column in a braced frame, it is assumed that the columns are restricted at their ends from horizontal displacements and therefore are only subjected to end moments and axial loads as transferred from the frame. It is then assumed that the frame, possibly by means of a bracing system, satisfies global stability checks and that the global stability of the frame does not affect the column behavior. This gives the commonly assumed non-sway frame. The design of columns in non-sway frames follows the conventional beam-column capacity check approach, and the column effective length may be evaluated based on the column end restraint conditions. Another reason for defining “sway” and “non-sway frames” is the need to adopt conventional analysis in which all the internal forces are computed on the basis of the undeformed geometry of the structure. This assumption is valid if second-order effects are negligible. The design of sway frames has to consider the frame sub assemblage or the structure as a whole. Moreover, the presence of “inelasticity” in the columns will render some doubts on the use of the familiar concept of “elastic effective length” On the basis of the above considerations, a definition can be established for sway and non-sway frames as: A frame can be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes. The codes provide a procedure to distinguish between sway and non-sway frames.

For non-sway frame first-order analysis may always be used. For Sway frame second-order analysis shall be used.

2.6. Steel Frames Analysis Method [3]

In most methods of structural analysis, the distribution of forces and moments throughout the frame is determined by using the conditions of static equilibrium and of geometric compatibility between the members at the joints. The way in which this is done depends on whether a frame is statically determinate (in which case the complete distribution of forces and moments can be determined by statics alone), or is statically indeterminate (in which case the compatibility conditions for the deformed frame must also be used before the analysis can be completed).

A statically indeterminate frame can be analyzed approximately if a sufficient number of assumptions are made about its behavior to allow it to be treated as if determinate. One method of doing this is to guess the locations of points of zero bending moment and to assume there are frictionless hinges at a sufficient number of these locations that the member forces and moments can be determined by statics alone.

The accurate analysis of statically indeterminate frames is complicated by the interaction between members: the equilibrium and compatibility conditions and the constitutive relationships must all be used in determining the member forces and moments. There are a number of different types of analysis which might be made.

2.6.1 Elastic Analysis [11]

In the elastic range all steel elements and even the complete steel structure can be assumed to follow Hooke’s law and recover completely to their original state upon removal of load. For framed structures, linear (first-order) elastic theory is traditionally used for analysis. With the aid of computer program, second-order analysis taking account of deflections in the structure can be performed. The maximum elastic load capacity is determined when any point in any member section reaches. Elastic analysis based on the assumption that the stress-strain behavior of the material is linear. As a consequence, to perform a global elastic analysis, the stresses applied in any cross section of any member, must be lower than the yield strength of the material (f_y in steel structures).

2.6.1.1 First-order elastic analysis [3&4]

Which is also called simply elastic analysis, for some frames, it is common to use a first-order elastic analysis, which is based on linear elastic constitutive relationships.

In this case, deformations are assumed to be small so that the equation of equilibrium may be written with reference to un-deformed configuration of structure. Additionally, superposition is valid and any inelastic behavior of material is ignored. This approach is used in the development of common analysis tools of profession, such as slope deflection method, moment distribution method that is found in most computer software.

In the case of First- order elastic analysis, the deformations (and internal forces) are proportional to the applied loads, and as a consequence the principle of superposition of effects can be used to simplify the analysis.

2.6.1.2 Second-order Elastic Analysis [3]

When equilibrium is expressed with reference to deformed shape of member as well as structure, the resulting analysis is a second order elastic analysis. It is a geometrical nonlinear analysis. Analysis includes member deformation i.e. P-δ effect and also sway i.e. P-Δ effect.

Second order elastic analysis accounts for elastic stability effect but does not indicate limit strength. The load displacement history by this analysis may approach the critical buckling load obtained from the eigen value solution, which requires an iterative process, only practicable by using computer programs. In the case of simple 2D structures (such as 2D steel frames) there are simplified second order processes. Hence this type of analysis is more complex than the first order elastic analysis. There are two basic methods commonly used in performing computerized elastic second-order analysis of beam-columns and frames. These are called the stability function approach and the geometric stiffness approach.

2.6.2 Plastic analysis [11]

When a steel specimen is loaded beyond the elastic limit the stress remains constant while the strain increases.

Plastic analysis allows the plasticity of some cross-sections (in general forming plastic hinges) and consequent redistribution of forces for other sections (with less force). In this type of analysis the material is modeled by constitutive relationships non-linear: rigid-plastic; elastic-perfectly plastic (structural steel), elastic-plastic.

Plastic analysis is based on determining the minimum load that causes the structure to collapse. Collapse occurs when sufficient plastic hinges have formed to convert the structure to a mechanism.

For a beam or column subjected to increasing moment this behavior results in the formation of a plastic hinge where a section rotates at the plastic moment capacity.

2.6.2.1. Manual Plastic Analysis Methods [1]

Plastic analysis theorems satisfy the following conditions:

1. Equilibrium condition: At collapse, the bending moments must correspond to a state of equilibrium between the external loads and the internal actions.

2. Mechanism condition: At collapse there must be sufficient plastic hinges to create a partial or complete collapse mechanism.

3. Yield condition: At collapse the bending moments must everywhere be less than or equal to plastic moment (M_p)

Using these conditions, three fundamental theorems of plastic analysis can be stated

If the bending moments are in equilibrium with the external load and $M \leq M_p$ everywhere, the load is a lower bound (i.e. load is $\leq$ collapse load), this is called lower bound theorem.

For an assumed mechanism in which the virtual work done in the plastic hinges equals the virtual work done by the external loads, the load is an upper bound (i.e. load is $\geq$ collapse load). This is called upper bound theorem.

If a bending moment distribution can be found that satisfies the three conditions of equilibrium, mechanism and yield, then the corresponding load is the collapse load, it is called uniqueness theorem.

The statical method of analysis is based on the lower bound theorem.

The procedure is summarized as flows

1. Select redundant
2. Draw the moment diagram for determinate structure.
3. Draw moment diagram for structure loaded by redundant.
4. Sketch composite moment diagram in such a way that a mechanism is formed.
5. Compute value of ultimate load by solving equilibrium equation.
6. Check to see that $M \leq MP$.

Mechanism or Kinematic Method of Analysis This method of analysis is based directly on the upper bound theorem. The basic idea is to try all the likely collapse mechanisms and select the one which gives the lowest collapse load.

The procedure is summarized as follows:

1. Identify the likely plastic hinge locations (under point loads, at supports, at joints, at zero shear positions under distributed loads).

2. Select possible independent and composite mechanism.

3. Solve equilibrium equation (virtual displacement method) for lowest load.

4. Check to see that $M \leq MP$ at all sections.

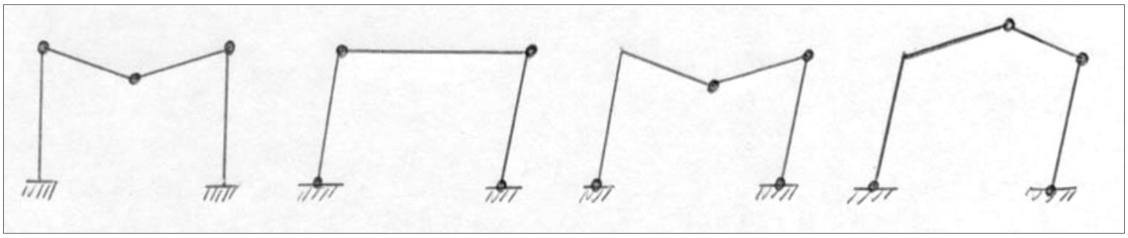


Figure 2.8.Type of Mechanism for Single-Story Frames.

The types of mechanism were summarized as follows:

1. Beam Mechanism: it results when there is a high proportion of vertical to lateral load.

2. Sway Mechanism is sometimes referred to as a panel mechanism and is caused by large lateral forces applied to the top of the frame.

3. Joint Mechanism.

4. Gable Mechanism: is a special case of the combination type and applies to gabled frames only.

The design procedure consists of determining the largest Mp for each of the three (or four) mechanism types. The largest of these then is the plastic moment for which the frame should be designed. The step-by-step procedure is as follows:

- Assume a specific collapse mechanism.

- Calculate the amount of internal virtual work which is defined as the sum of all the products of the plastic moments and their corresponding internal virtual angle changes. The internal virtual angle changes are computed by designating any one angle as θ and calculating all others in terms of θ and the geometry of the frame.

- Calculate the amount of external virtual work which is defined as the sum of all the external loads time the virtual distance through which they move at the collapse mechanism. This distance is calculated by recognizing that it is the product of the angle θ and the distance from the angle change to the load.
- Equate external to internal work. The angle θ cancels out and Mp can be solved for in terms of the loads and the dimensions of the frame.
- Sufficient collapse mechanisms are tried so that the designer is satisfied that one with the largest Mp has been found. This is done by drawing the plastic moment diagram for each trial mechanism to see that there are no moments larger than the plastic moment.

2.6.2.2. First Order plastic Analysis [4]

There are two main assumptions for first-order plastic analysis:

1. The structure is made of ductile material that can undergo large deformations beyond elastic limit without fracture or buckling.
2. The deflections of the structure under loading are small so that second-order effects can be ignored.

This analysis accounts for post elastic strength of member of the structure. Therefore it is also known as material non-linear analysis. In progressive loading and when elastic limit is crossed highly stressed section of the member yields completely and the section behaves a hinge known as plastic hinge. When it happens the particular section continues to resist plastic moment and undergoes large deformation. Progressive loading is continued till sufficient numbers of plastic hinges are developed and structure no longer resists any further additional load due to transformation of structure into a mechanism and hence it is said to be a plastic collapse. In this analysis, member deformations and sway effect of structure are not considered therefore the analysis does not reflect buckling and stability assessment.

2.6.2.3. Second Order Plastic Analysis [4]

The addition of effects of member deformation and drift effect of the structure in first order Plastic analysis calls the second order plastic analysis. This gives complete, realistic and accurate analysis but makes the process complex. This analysis includes both geometric and material non-linear ties and known as “advanced analysis” .This advanced analysis is further classified into following categories:

2.6.2.3.1. Elastic –Plastic hinge method [4] is simple, approximate and efficient for representing Plasticity in frames. In this method, zero length plastic hinges are assumed to form at the ends of members, whereas other portions are assumed to remain elastic. Thus, it accounts for Plasticity but disregards the spread of yielding and residual stress effects between the plastic hinges.

The elastic –plastic hinge method can be first-order or second-order plastic analysis. The first-order elastic –plastic hinge method, in which the non-linear geometric effects are neglected, predict the same ultimate load as conventional rigid-plastic analysis. In second-order elastic –plastic hinge analysis, the deformed structural geometry is considered for formulating the stiffness equation.

2.6.2.3.2. Plastic zone method [4] in this method the cross section is subdivided in to small sub-elements, the residual stresses are considered constant within each sub-element. The stress state at each sub-element can be traced clearly and hence the gradual spread of yielding can be predicted. The plastic zone method eliminates the need for separate member capacity check, hence this method accepted to provide exact solution.

In this method a frame member is discretized into finite elements, and the cross-section of each finite element is subdivided into many fibers. The deflection at each division along the members is obtained by numerical integration. A plastic-zone analysis eliminates the need for separate member capacity checks since second-order effects, the spread of plasticity, and residual stresses are accounted for directly. As a result, a plastic-zone solution is considered “exact.

2.6.2.3.3. Refined Plastic hinge method [4]

This approach is a refined version of elastic-plastic hinge approach. This method considers gradual stiffness degradation of plastic hinge section as well as gradual stiffness degradation of member between two plastic hinges. In this analysis, stability functions are used to predict second-order effects. The benefit of stability functions is that they make the analysis method practical by using only one element per beam-column. The refined plastic hinge analysis uses a two-surface yield model and an effective tangent modulus to account for stiffness degradation due to distributed plasticity in framed members. Column tangent modulus is used to represent the effective stiffness of the member when it is loaded with a high axial load. Thus, the refined plastic hinge model approximates the effect of distributed plasticity along the element length caused by initial imperfections and large bending and axial force actions.

2.7. Structural Loads [5]

The building structure must be designed to carry or resist the loads that are applied to it over its design-life. In this research the building structure will be subjected to loads that have been categorized as follows:

2.9.1. Dead Loads (D_L): are permanent loads acting on the structure. These include the self-weight of structural and non-structural components. They are usually gravity loads.

Live Loads (L_L): are non-permanent loads acting on the structure due to its use and occupancy. The magnitude and location of live loads changes frequently over the design life. Hence, they cannot be estimated with the same accuracy as dead loads.

Wind Loads (W_L): are in the form of pressure or suction on the exterior surfaces of the building. They cause horizontal lateral loads (forces) on the structure, which can be critical for tall buildings. Wind loads also cause uplift of light roof systems.

Design wind loads for buildings can be based on Simplified procedure, Analytical procedure, and Wind tunnel.

2.7.1. Analytical Procedure for Calculate Wind Force

A building or other structure whose design wind loads are determined in accordance with this section shall meet all of the following conditions:

1. The building or other structure is a regular-shaped building.

2. The building or other structure does not have response characteristics making it subject to a cross wind loading, vortex shedding, instability due to galloping or flutter; or does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

Design Procedure

Determine the basic wind speed V and wind directionality facto K_d.

1. Determine an importance factor I.

2. Determine an exposure category or exposure categories and velocity pressure exposure coefficient K_z.

3. Determine a topographic factor K_{zt}.

4. Determine a gust effect factor G.

5. Determine an enclosure classification.

6. Determine an internal pressure coefficient G Cp.

7. Determine an external pressure coefficients Cp or G Cp f or force coefficients Cf.
8. Determine a velocity pressure (q).
9. Determine a design wind load (p).

Table 2.1.Classification of Buildings and Other Structures for Wind

Description	Category
Storage facilities (low hazard to human life)	I
All others (not listed in category I, III, IV)	II
A subst. hazard to human life (schools)	III
Essential facilities (emergency shelters)	IV

Table 1.2.Importance Factor, I

Category	I
I	0.87
II	1.00
III	1.15
IV	1.15

Exposure Categories

- Exposure A. Large city centers with at least 50% of the buildings having a height in excess of 70 ft.
- Exposure B. urban and suburban areas or other terrain with numerous closely spaced obstructions having the size of single family dwellings or larger.
- Exposure C. Open terrain with scattered obstructions having heights generally less than 30 ft (flat open country and grass lands).

- Exposure D. Flat unobstructed areas exposed to wind flowing over large bodies of water.

Enclosure Classifications

- Open: each wall at least 80% open
- Partially Enclosed: the total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building (more than 4 SF)
- Enclosed

The equations may be carried out as follows:

$P_z = q_z$ GC (2.1)

$q_z = 0.00256 * K_z * K_{zt} * V^2 * I$ (2.2)

P_z = design pressure in psf.

q_z = velocity pressure in psf.

G = gust effect factor.

C = pressure or force coefficient.

0.00256 = constant for density of air and dimensions.

K_z = velocity pressure exposure coefficient.

K_{zt} = topographic factor.

V = basic wind speed in mph.

I = importance factor.

Velocity Pressure Exposure Coefficients K_z:

Velocity pressure exposure coefficient K_z is a function of height and exposure type.

GUST EFFECT FACTOR G:

Rigid Structure: Simplified Procedure

G = 0.80 for Exposures A and B

G = 0.85 for Exposures C and D

TOPOGRAPHIC FACTOR K_{zt}

$K_{zt} = (1 + K_1K_2K_3)^2$ (2.3)

Velocity pressure exposure coefficient.(K_z):

K_z = 2.01*(15/ Z_g) ^ (2/ α); High <15 (2.4a)

K_z = 2.01*(High/Z_g) ^ (2/ α)) High >15 (2.4b)

Terrain Exposure Constant, α

Exposure	B	C	D
α	7	9.5	11.5

Terrain Exposure Constant, Z_g

Exposure	B	C	D
Z_g	1200	900	700

CHAPTR THREE

ANALYSIS AND DESIGN METHODS OF 2D FRAMES

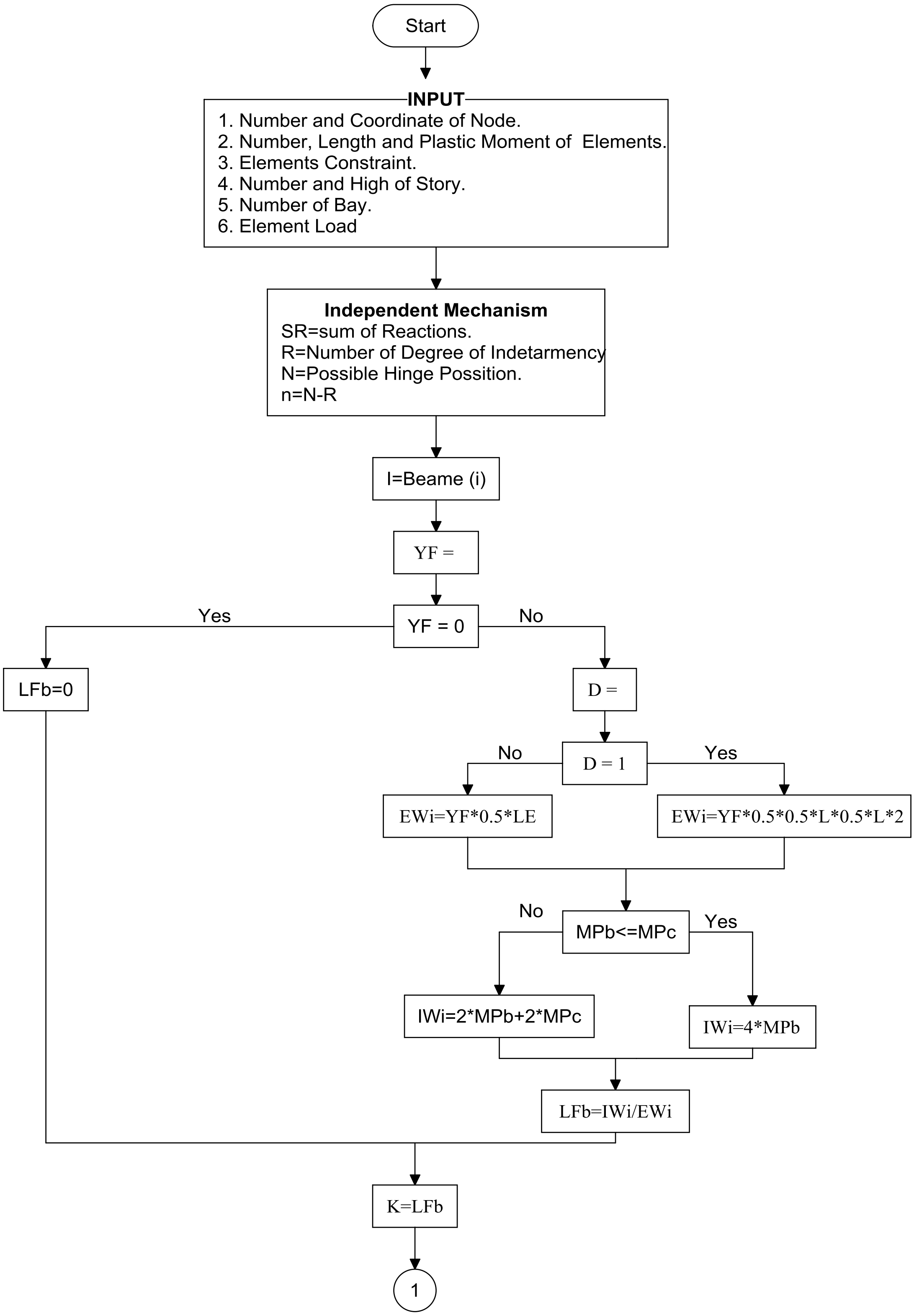
3.1. Introduction

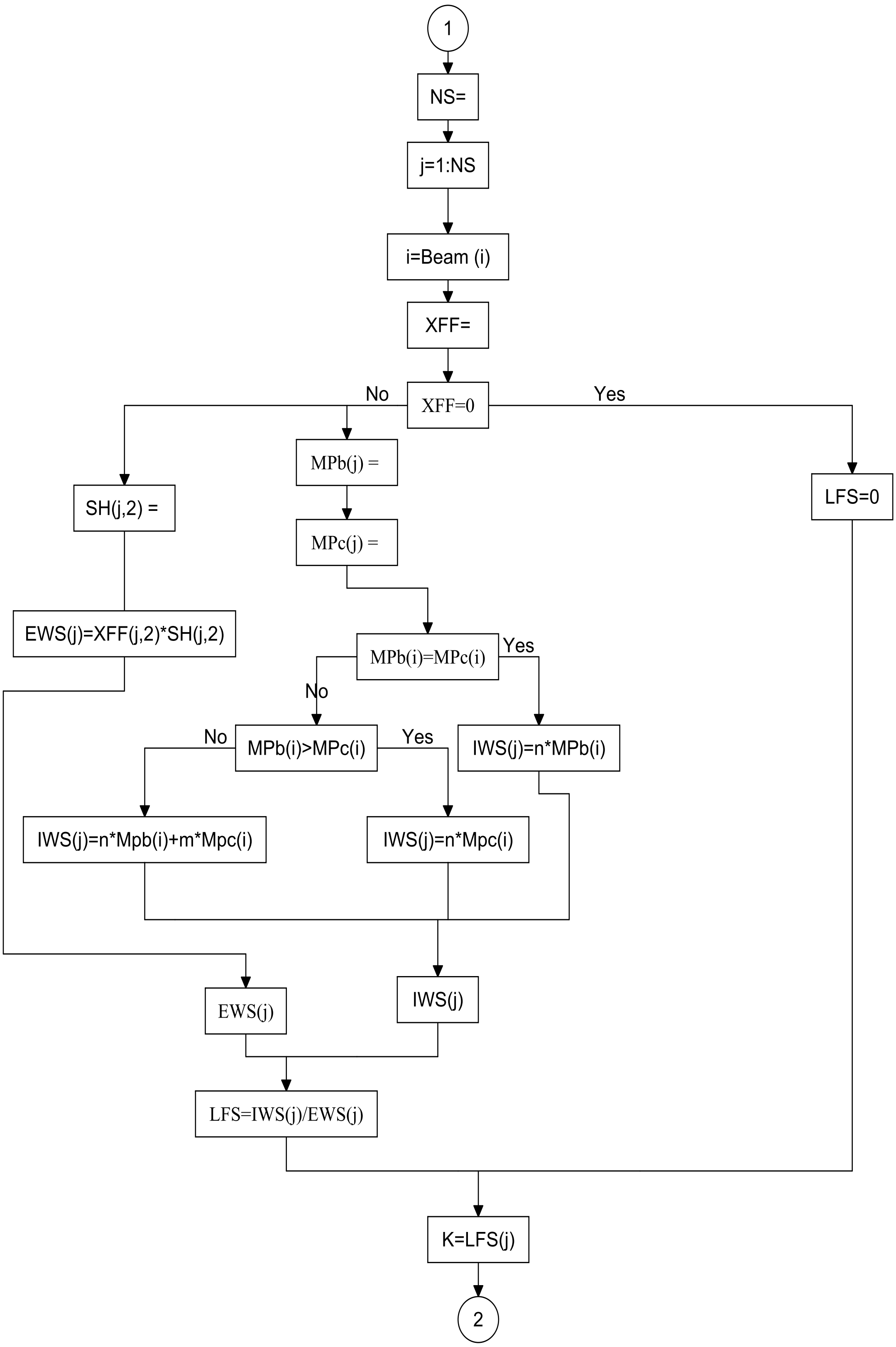
The first and second orders elastic and plastic analyses of unbraced plane frames were done by using different computer programs. MATLAB code was used for determining the critical load factors of different plane frames to obtain the collapse mechanism. Then the first and second orders elastic and plastic analyses were carried out using structural analysis programs MASTAN2 and SAP 2000 for different unbraced plane frames of a building. In this chapter, the flow charts of elastic and plastic design of frames structural elements (beams, columns, connections and base plates) were prepared to enhance the design process.

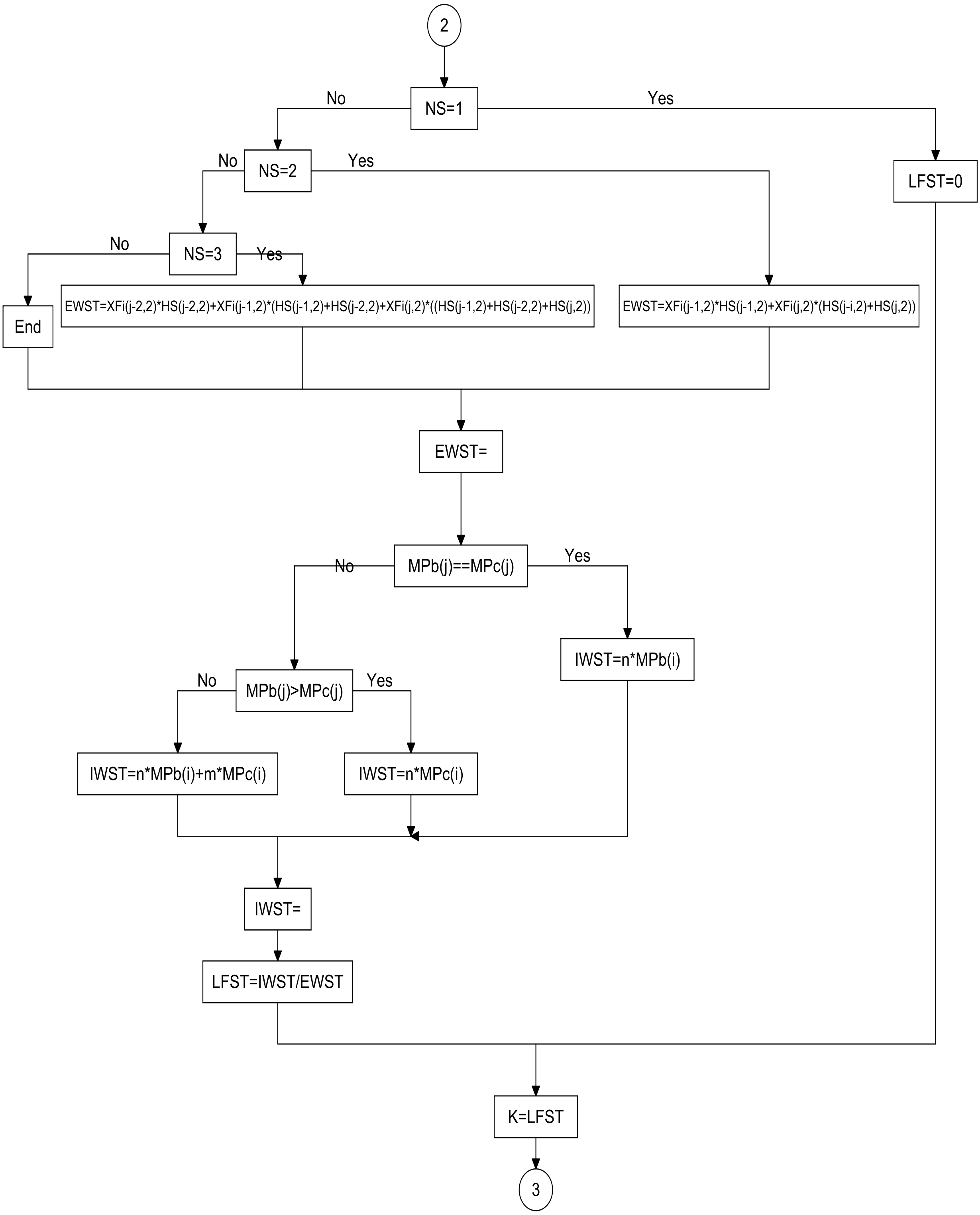
3.2. Plastic Analysis of Frames Using MATLAB

MATLAB means a laboratory matrices produced by the company, which is very important for every engineer as convenient tool to solve engineering problems because it has many functions that find frequent use in solving problems. In civil engineering there are a lot of complex problems and solving these problems manually is very difficult, but using MATLAB makes the solution more easily and by using the MATLAB is possible to draw the variables and functions and display the results.

The critical load factor of unbraced frame was obtained using the mechanism method that modeled by MATLAB. The data for determining the critical load factor was entered in excel spread sheet and exported to MATLAB code by function of Read Microsoft Excel spreadsheet file (xlsread).The number of possible plastic hinges of plane frames was modeled by MATLAB code depending on the number of Storeys and bays, vertical forces and constraints. The flow chart of plastic analysis of plane frame by MATLAB code was presented in Figure 3.1.


Start
INPUT
1. Number and Coordinate of Node.
2. Number, Length and Plastic Moment of Elements.
3. Elements Constraint.
4. Number and High of Story.
5. Number of Bay.
6. Element Load
Independent Mechanism
SR=sum of Reactions.
R=Number of Degree of Indetarmency
N=Possible Hinge Possition.
n=N-R
I=Beame (i)
YF =
YF = 0
Yes
No
LFb=0
D =
D = 1
No
Yes
EWi=YF*0.5*LE
EWi=YF*0.5*0.5*L*0.5*L*2
MPb<=MPc
No
Yes
IWi=2*MPb+2*MPc
IWi=4*MPb
LFb=IWi/EWi
K=LFb
1


1
NS=
j=1:NS
i=Beam (i)
XFF=
XFF=0
No
Yes
MPb(j) =
MPc(j) =
SH(j,2) =
LFS=0
EWS(j)=XFF(j,2)*SH(j,2)
MPb(i)=MPc(i)
Yes
No
No
MPb(i)>MPc(i)
Yes
IWS(j)=n*MPb(i)
IWS(j)=n*Mpb(i)+m*Mpc(i)
IWS(j)=n*Mpc(i)
EWS(j)
IWS(j)
LFS=IWS(j)/EWS(j)
K=LFS(j)
2


2
NS=1
No
Yes
NS=2
No
Yes
LFST=0
NS=3
No
Yes
End
EWST=XFi(j-2,2)*HS(j-2,2)+XFi(j-1,2)*(HS(j-1,2)+HS(j-2,2)+XFi(j,2)*((HS(j-1,2)+HS(j-2,2)+HS(j,2))
EWST=XFi(j-1,2)*HS(j-1,2)+XFi(j,2)*(HS(j-i,2)+HS(j,2))
EWST=
MPb(j)==MPc(j)
No
Yes
IWST=n*MPb(i)
MPb(j)>MPc(j)
No
Yes
IWST=n*MPb(i)+m*MPc(i)
IWST=n*MPc(i)
IWST=
LFST=IWST/EWST
K=LFST
3

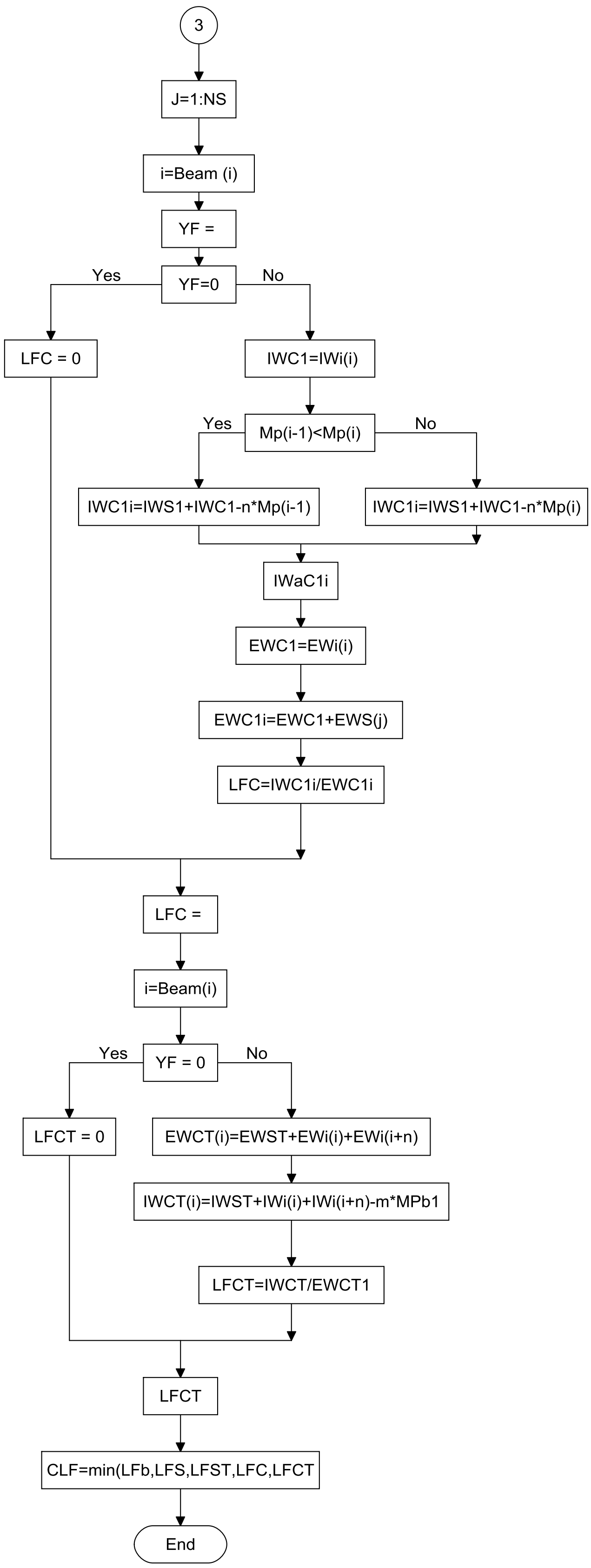


Figure 3.1.Flow Chart of Frame Plastic Analysis using MATLAB.

3.3. Elastic and Plastic Analysis of Frames Using MASTAN2

MASTAN2 is an interactive graphics program that provides preprocessing, analysis, and post-processing capabilities. Pre-processing options include definition of structural geometry, support conditions, applied loads, and element properties. The analysis routines provide the user the opportunity to perform first- or second-order elastic or inelastic analyses of two- or three-dimensional frames and trusses subjected to static loads. Post-processing capabilities include the interpretation of structural behavior through deformation and force diagrams, printed output, and facilities for plotting response curves. MASTAN2 is based on MATLAB, a premier software package for numeric computing and data analysis.

The program's linear and nonlinear analysis routines are based on the theoretical and numerical formulations presented in the text Matrix Structural Analysis, 2nd Edition, by McGuire, Gallagher, and Ziemian.

The analysis of frames using MASTAN2 was carried out by the following procedures:

1. Frame Definition

- From the Geometry menu select Define Frame.
- At the bottom menu bar add the number and width of bay, and story, and click on the apply button.

2. Section Properties

- Properties menu → Define Section →
- From the Properties menu select Attach Section.

3. Material Properties

- Properties menu → Define Material.
- From the Properties menu select Attach Material.

4. Support Conditions

- Conditions menu → Define Fixities.

5. Loads

Assign Frames Load in MASTAN2

For each Frame:

Distributed Load on Frames as follows:

Roof: W_y=-[Frame Roof (D_L)) +Frame Roof (L_L))]

Floor: W_y=-[Frame Floor (D_L)) +Frame Floor (L_L)]

Horizontal Load for Frames joint:

Roof: P_X=Frame Roof (W_w)

Floor: P_X=Frame Floor (W_w)

For Plastic analysis multiply the Frames load by 1.3

For Point Loads**:** Conditions menu → Define Forces.

For Uniform Loads: Conditions menu → Define Uniform Loads.

6. First Order Elastic Analysis

Analysis menu → 1st-Order Elastic → Planar Frame (x-y) → Apply.

7. Second Order Elastic Analysis

Analysis menu → 2nd-Order Elastic → Planar Frame (x-y).

8. First Order Plastic Analysis

Analysis menu → 1st-Order Inelastic → Planar Frame (x-y) → Apply.

9. Second Order Plastic Analysis

Analysis menu → 2nd-Order Inelastic → Planar Frame (x-y) →
Apply.

3.4. Analysis of Frames Using SAP 2000

SAP2000 is a general purpose finite element program, which performs the static or dynamic, linear or nonlinear analysis of structural systems. It is also a powerful design tool to design structures following different codes.

3.4.1 First Order Elastic analysis of 2D frame

The analysis steps were summarized as follows:

1. Definition of Units.

2. Draw the 2D grid by enters the horizontal and vertical displacements in two directions for menu:

File → New Model → 2D Frames.

3. Define Materials from the menu:

Define → Materials

4. Define Sections: form the Menu:

Define → Section Properties → Frame Sections.

5. Joints Restraints Definition:

Assign → Joint → Restraints

6. Members Connection: Press Select All and from menu:

Assign Joint Constraints.

7. Define Loads, Load Cases, and Load Combinations: from the Menu:

Define → Load Patterns, Load Cases, Load Combinations

3.4.2 Second Order Elastic Analysis

The second order elastic analysis is the same as first order but should be including the p-delta effects.

3.5. Elastic- Plastic Analysis of Frames [1]

The method for incremental elastic plastic analysis gives a complete load–deflection history of the structure until collapse. The Elastic –Plastic analysis may be first or second order. This method is based on the plastic hinge concept for fully plastic cross sections in a structure under increasing proportional loading. Proportional loading applies to a structure with loads multiplied by a common load factor. The method consists of a series of elastic analysis, each of which represents the formation of a plastic hinge in the structure. Results for each elastic analysis are transferred to a spreadsheet from which the location for the formation of a plastic hinge and the corresponding increment of loading in terms of the common load factor can be obtained.

The primary purpose for carrying out an elastic plastic analysis of a structure is to find its plastic collapse load. If a structure is subject to a set of applied loads, P_1, P_2 . . . , a common load multiplier λ is applied to all the loads. The collapse load of the structure is defined by λ_C at which the structure fails by plastic collapse.

To find λ_C an incremental elastic- plastic analysis can be performed on the basis that the member forces increase with λ. The procedures by using excel spared sheet as follows:

1. Create the model and run the analysis of frames.
2. The analysis results were exported excel spread sheet
3. Set up the table shown below in Excel spreadsheet, to perform the steps below.

Analysis Stage No: Critical load factor,CLF =

Element	**Node**	**M0**	**Mp**	**Mi**	**Mp-Mi**	**LF**	**CLF**	**M(i+1)**

4. Calculate the Plastic Moment for the plane frame sections as follows:

$$M_p = Z_p * F_y$$

Mp and M_O must have the same sign at any stage

5. For stage No.4, the initial moment [M_i] equal zero.
6. Calculate the residual plastic moment [M_p-M_i].
7. Calculate the load factor [LF] = [M_p-M_i]/ [M_o]

If M_o = zero skip this step.

8. Choose the smallest load factor[CLF] and calculate the cumulative bending moment using[CLF] for all members.

M_{i+1} = M_i + CLF * M_o

9. Calculate the residual plastic moment for all other section
10. Insert a hinge at the joint corresponding to section with CLF(end Releases

of M33 in SAP 2000)

11. Repeat (5) to (10) until structure collapses (when the critical load factor equal zero).

The final collapse load CLF is the summation of the load factors CLF from all stages of analysis.

Plastic theory concentrated only on the strength of the structure thus it make no attempt to assess deflections. Thus deflection considered to be very large when a displacement due to basic loads become larger than 100 items the cumulative displacement at any freedom.

The cumulative deflection is calculated by:

$\nabla_{i+1} = \nabla_o + \nabla_i * \text{CLF}$

The Cumulative Shear force is calculated by:

$V_{i+1} = V_0 + V_i * \text{CLF}$

The Cumulative Axial Force is calculated by:

$P_{i+1} = P_o + P_i * \text{CLF}$

The second order plastic analysis is the same as first order but should be including the p-delta effects.

3.5.1. Distributed Load in Elastic- Plastic Analysis Method [1]

For structures subjected to point loads, plastic hinges occur at sections either at joints or where the point loads act. For members subjected to distributed load, plastic hinges may occur within the length of the load along the member.

The usual way of dealing with distributed load in plastic analysis is to simulate the action of the distributed load by equivalent point loads. The member subjected to the distributed load is then discretized into shorter elements according to the number of equivalent point loads generated.

In this method the problem is determining an adequate number of equivalent point loads and the accuracy of results are not known.

A member of length L subjected to a distributed load linearly varying from w1 at one end to w2 at the other is shown in Figure 4.20.

The bending moment M_x at a distance x from end i is given by:

$$M_x = V_i * x - \left(\frac{w_1 * X^2}{2}\right) - \frac{(w_2 - w_1) * x^3}{6L} - M_i \qquad 3.2$$

Where:

V_i , M_i = the shear force and bending moment at end i of the member respectively, obtained at the completion of an elastic-plastic analysis.

When the shear force is zero, the bending moment M_x is a maximum

That is,

$$V_x = \frac{\delta M_x}{\delta x} = V_i - w_1 * x - \frac{(w_2 - w_1) * x^2}{2L} = 0 \qquad 3.3$$

To find the location of plastic hinge solve the equation (3.3) for x. The member can be discretized into two with the joint located at the point where maximum M_x occurs and the collapse load can be reevaluated. This procedure can be repeated until both the location of the plastic hinge and the collapse load of the structure converge with satisfactory accuracy.

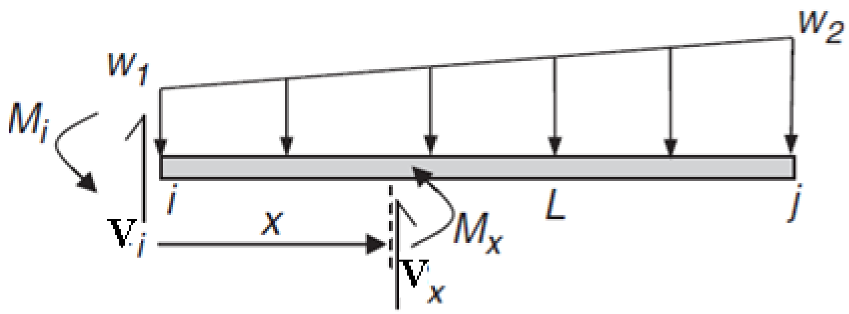


Figure 3.2.Member with a Linearly Varying Distributed Load.

3.6. Design of Steel Frames

Structural design should be performed to satisfy three criteria: strength, serviceability and economy. Strength pertains to the general integrity and safety of the structure under extreme load conditions. The structure is expected to withstand lateral overloads without severe distress and damage during its lifetime. Serviceability refers to the proper functioning of the structure as related to its appearance, maintainability, and durability under normal, or service load conditions. Deflection, vibration, permanent deformation, cracking, and corrosion are some design considerations associated with serviceability. Economy concerns the overall material and labor costs required for the design, fabrication, erection, and maintenance processes of the structure.

In AISC code there are three philosophies of design are in current use:

1. Allowable Stress Design (ASD).
2. Plastic Design (PD).
3. Load and Resistance Factor Design (LRFD).

3.6.1. Elastic Design of Structural Elements

Beams members subjected to flexural loads, i.e., shear force and bending moment only. The axial force in a beam member is negligible in elastic design.

Design of beam using (ASD) satisfies following requirements:

1. Maximum bending stress F_b must not exceed allowable stress.
2. Maximum shear stress F_v shall not exceed allowable shear stress.
3. Deflection should not exceed allowable limit.

The flow chart of beam elastic design is shown in Figure 3.4.

Column member subjected to compressive axial force only or subjected to compressive axial force and bending moment. The flow chart of elastic design is shown in Figure 3.5.

The flow chart of elastic design of column base plate is also shown in Figure 3.6

Finally, the design of bolt connections was presented in

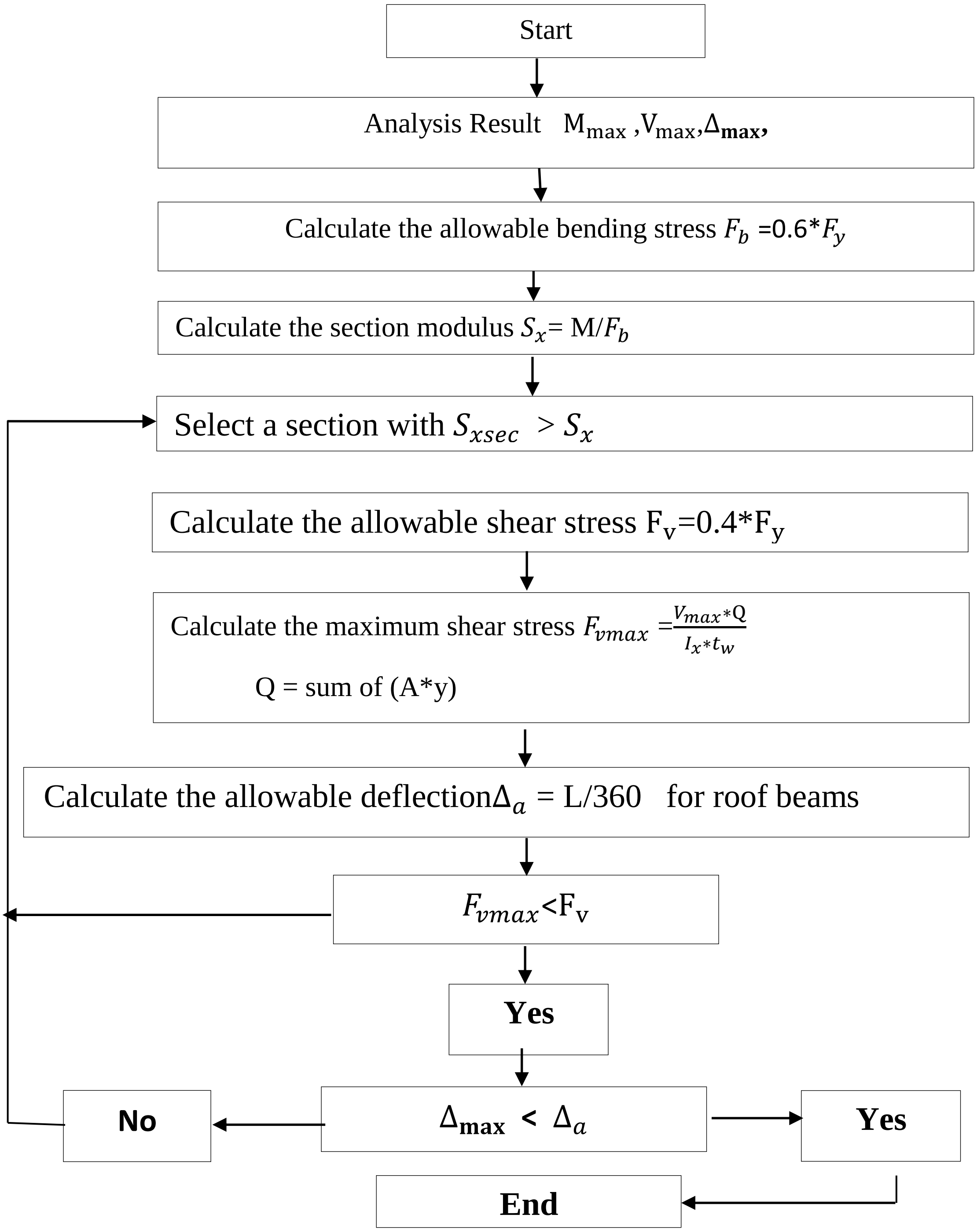


Figure3.3. Flow Chart of Elastic Design of Beam.

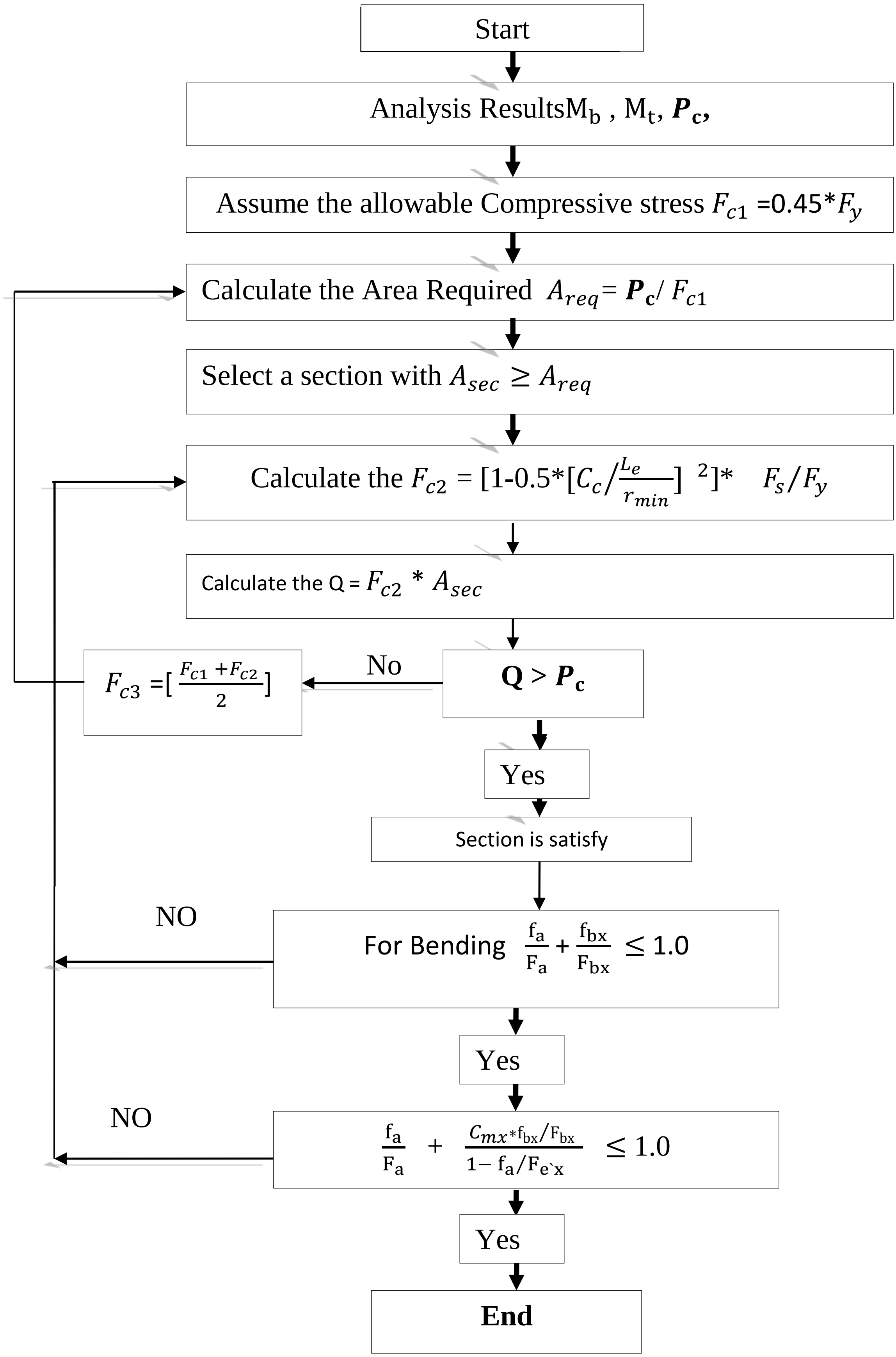


Figure3.4. Flow Chart of Elastic Design of Column.

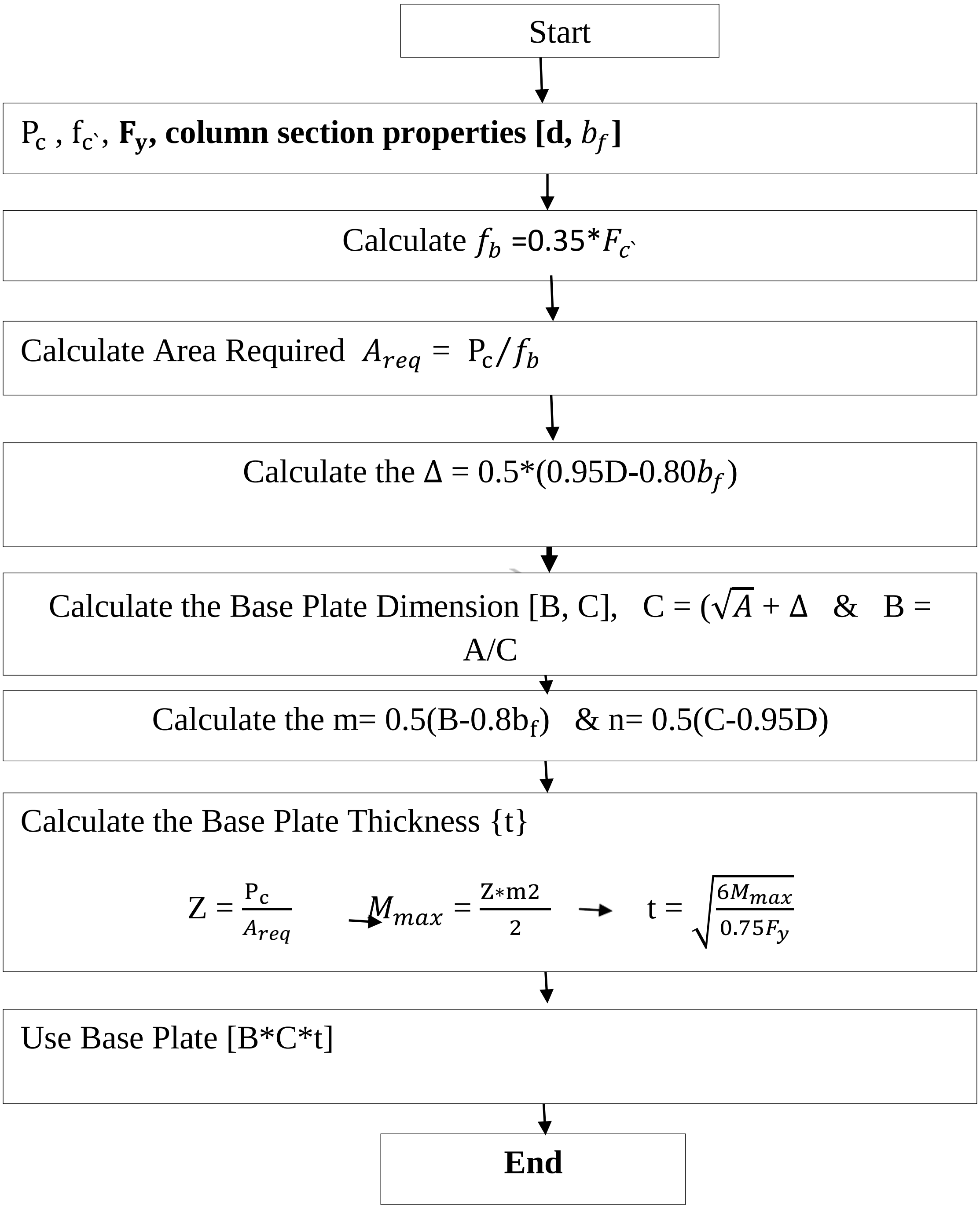


Figure 3.5. Flow Chart of Elastic Design of Base Plate.

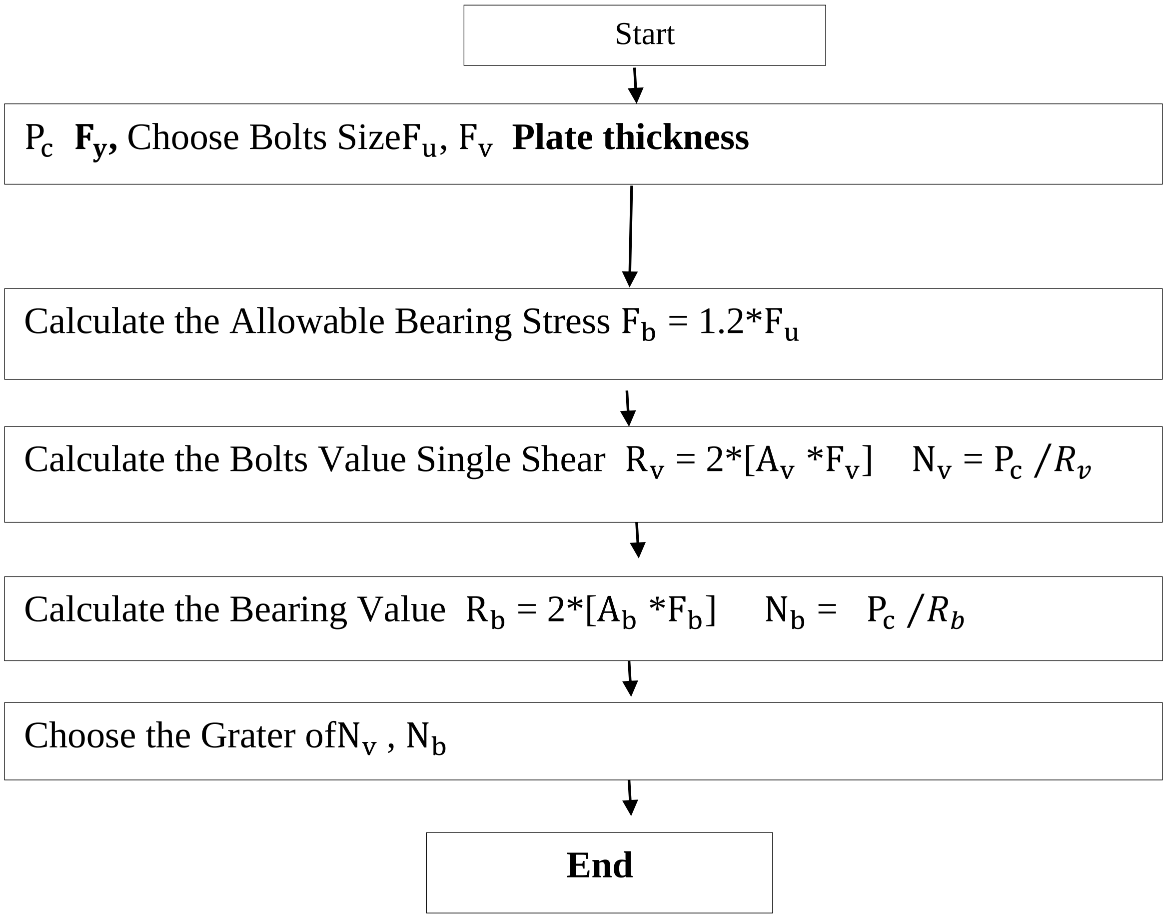


Figure 3.6. Flow Chart of Elastic Design of Column Splice Connection.

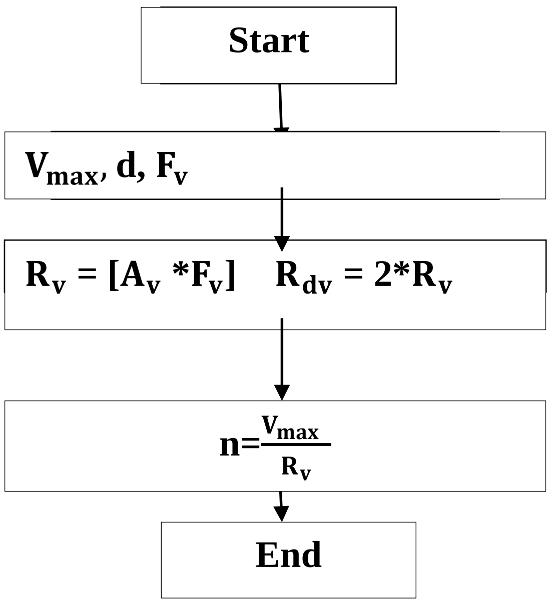


Figure 3.7.Flow Chart of elastic Design beam splice Connection.

Start

↓

M_{max} V_{max} **F_y**, F_u, F_v **Beam section properties [D,** t_w , t_f b_f **]** Bolts size

↓

Calculate the Allowable Bearing Stress $F_b = 1.2*F_u$

↓

Allowable tensile stress $F_t = 0.5*F_u$

↓

Calculate the Bolts Value

$R_v = [A_v *F_v]$ $R_{dv} = 2*[A_v *F_v]$ $N_v = P_c/R_v$

↓

Calculate the Bearing Value

$R_{pw} = 2*[d_w *d*F_b]$ $R_{pf} = 2*[t_f *d*F_b]$ $N_b = P_c/R_b$

↓

Select the Angle

The Gage =0.5[depth –D] Depth = $\frac{M_{max}}{2*R_t}$ Tension $R_t = A_b * F_t$

↓

Number of Bolts:

$N_f = T/R_v$ $N_w = V_{max}/R_v$ $T = M_{max}/depth$

↓

Web Member: $N_w * R_v \geq V_{max}$ Flange: $N_f * R_v \geq T$

↓

End

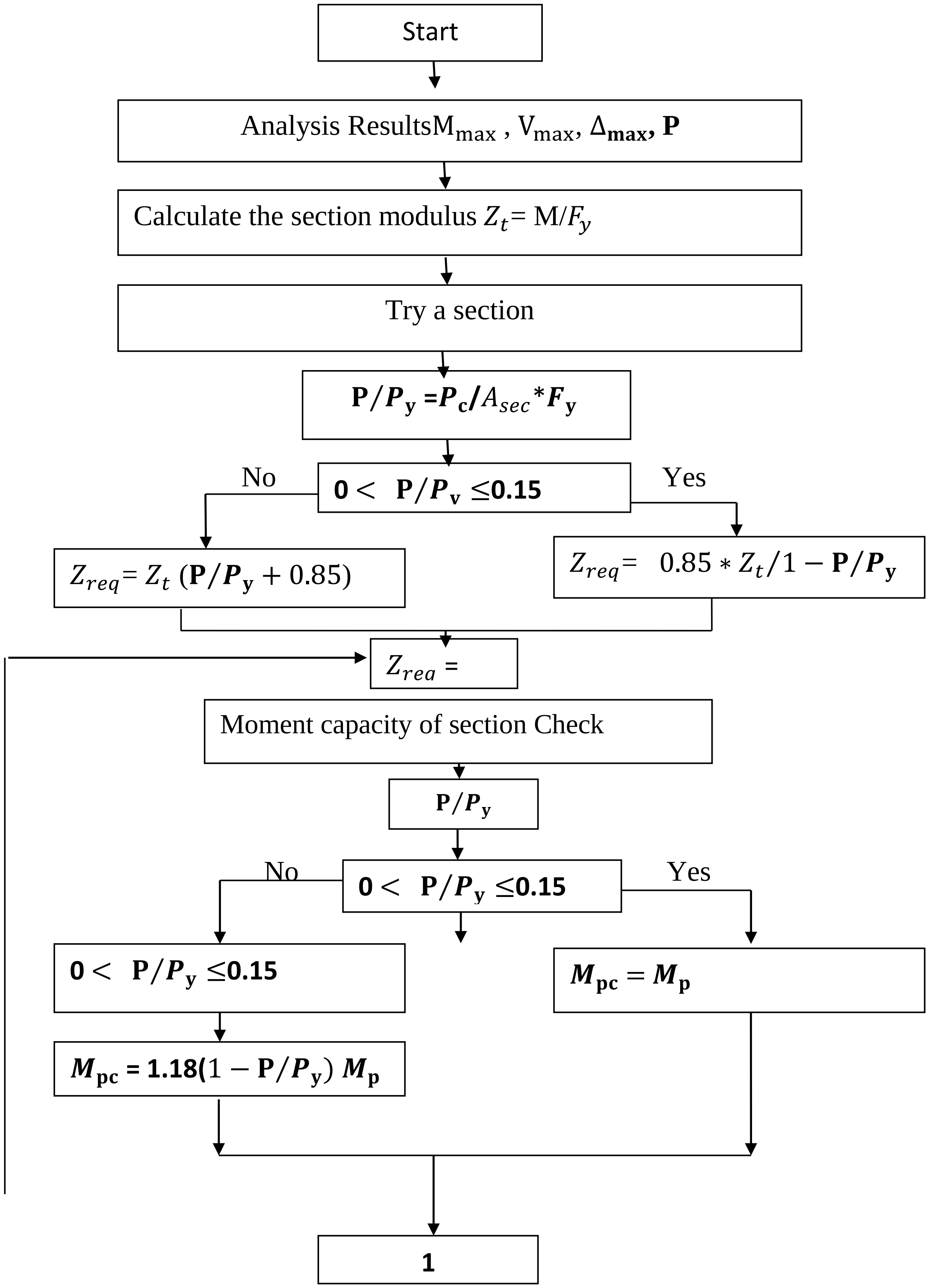
Figure 3.8. Flow Chart of Elastic Design of Beam Column Connection.

3.6.2. Plastic Design of Structural Elements

Plastic analysis and design is permitted only for steels with yield stress not exceeding 65 ksi. The reason for this is that steels with high yield stress lack the ductility required for inelastic rotation at hinge locations. Without adequate inelastic rotation, moment redistribution cannot take place.

In plastic design, the predominant limit state is the formation of plastic hinges. Failure occurs when sufficient plastic hinges have formed for a collapse mechanism to develop. To ensure that plastic hinges can form and can undergo large inelastic rotation, in the plastic design the sections must be compact.

The flow charts of beam and column plastic design were shown in Figures 3.10– 3.11. Figure 3.12 presents chart for calculate the M/M_p


Start
Analysis ResultsM_{max} , V_{max}, $\Delta_{\mathbf{max}}$, $\mathbf{P}$
Calculate the section modulus Z_t= M/F_y
Try a section
$\mathbf{P}/\boldsymbol{P}_{\mathbf{y}}$ =$\boldsymbol{P}_{\mathbf{c}}$/$A_{sec}$*$\boldsymbol{F}_{\mathbf{y}}$
No
$\mathbf{0} <\ \mathbf{P}/\boldsymbol{P}_{\mathbf{v}} \leq$0.15
Yes
Z_{req}= Z_t $(\mathbf{P}/\boldsymbol{P}_{\mathbf{y}} + 0.85)$
Z_{req}= $0.85 * Z_t/1 - \mathbf{P}/\boldsymbol{P}_{\mathbf{y}}$
Z_{rea} =
Moment capacity of section Check
$\mathbf{P}/\boldsymbol{P}_{\mathbf{y}}$
No
$\mathbf{0} <\ \mathbf{P}/\boldsymbol{P}_{\mathbf{y}} \leq$0.15
Yes
$\mathbf{0} <\ \mathbf{P}/\boldsymbol{P}_{\mathbf{y}} \leq$0.15
$\boldsymbol{M}_{\mathbf{pc}} = \boldsymbol{M}_{\mathbf{p}}$
$\boldsymbol{M}_{\mathbf{pc}}$ = 1.18$(1 - \mathbf{P}/\boldsymbol{P}_{\mathbf{y}})$ $\boldsymbol{M}_{\mathbf{p}}$
1

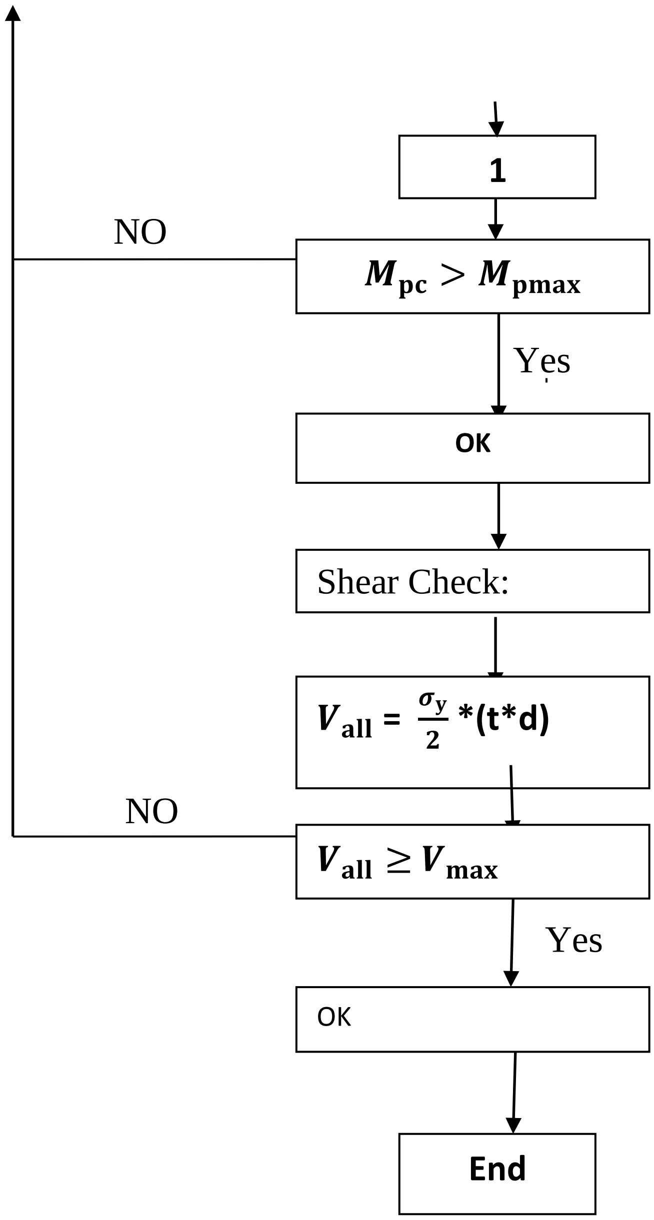
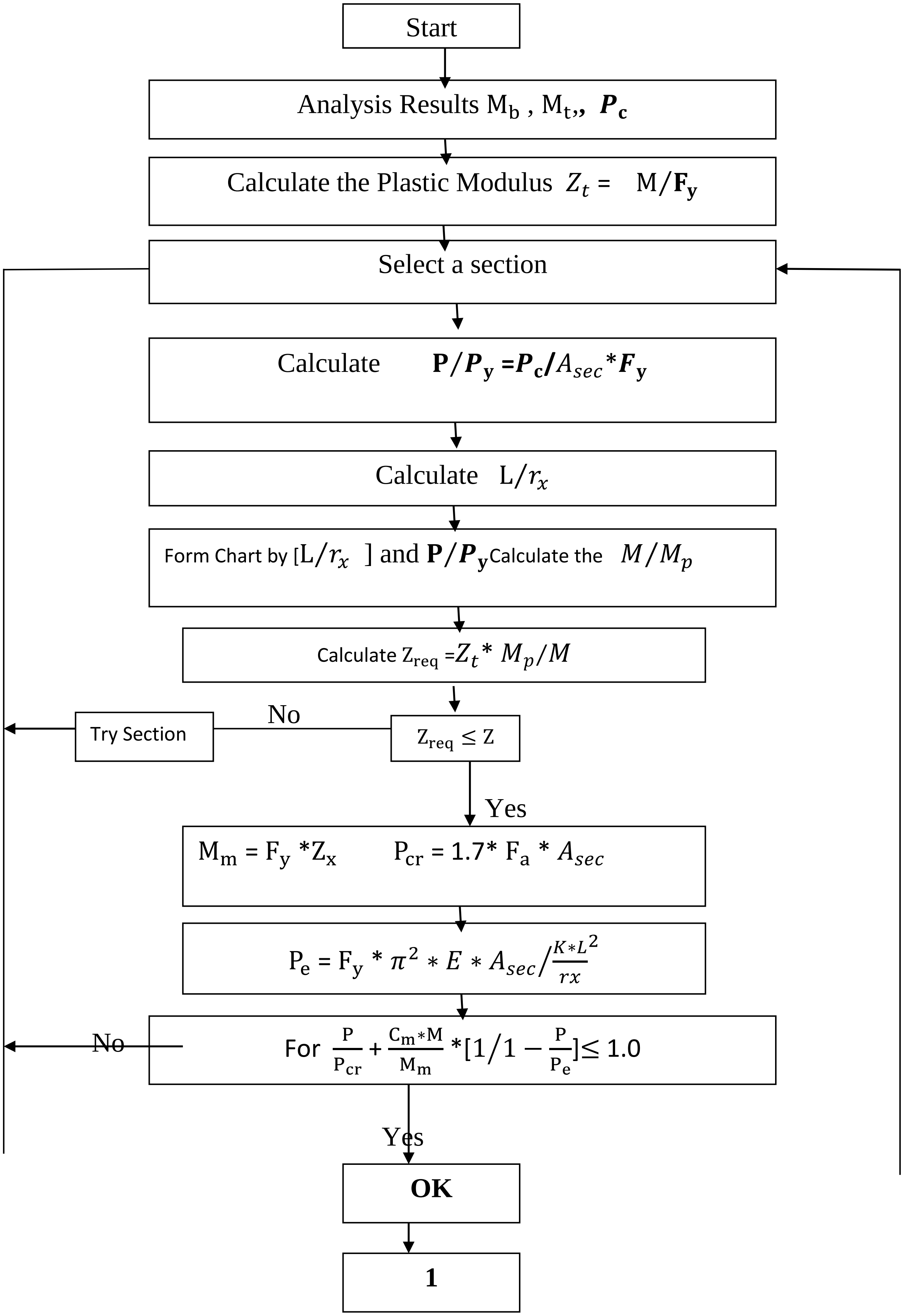


Figure 3.9.Flow Chart Plastic Design of Beam.


Start
Analysis Results M_b , M_t,, $\boldsymbol{P_c}$
Calculate the Plastic Modulus $Z_t = M/\mathbf{F_y}$
Select a section
Calculate $\mathbf{P}/\boldsymbol{P_y} = \boldsymbol{P_c}/A_{sec} * \boldsymbol{F_y}$
Calculate L/r_x
Form Chart by [L/r_x] and $\mathbf{P}/\boldsymbol{P_y}$ Calculate the M/M_p
Calculate $Z_{req} = Z_t * M_p/M$
No
Try Section
$Z_{req} \leq Z$
Yes
$M_m = F_y * Z_x$ $P_{cr} = 1.7 * F_a * A_{sec}$
$P_e = F_y * \pi^2 * E * A_{sec} / \frac{K*L}{rx}^2$
No
For $\frac{P}{P_{cr}} + \frac{C_m * M}{M_m} * [1/1 - \frac{P}{P_e}] \leq 1.0$
Yes
OK
1

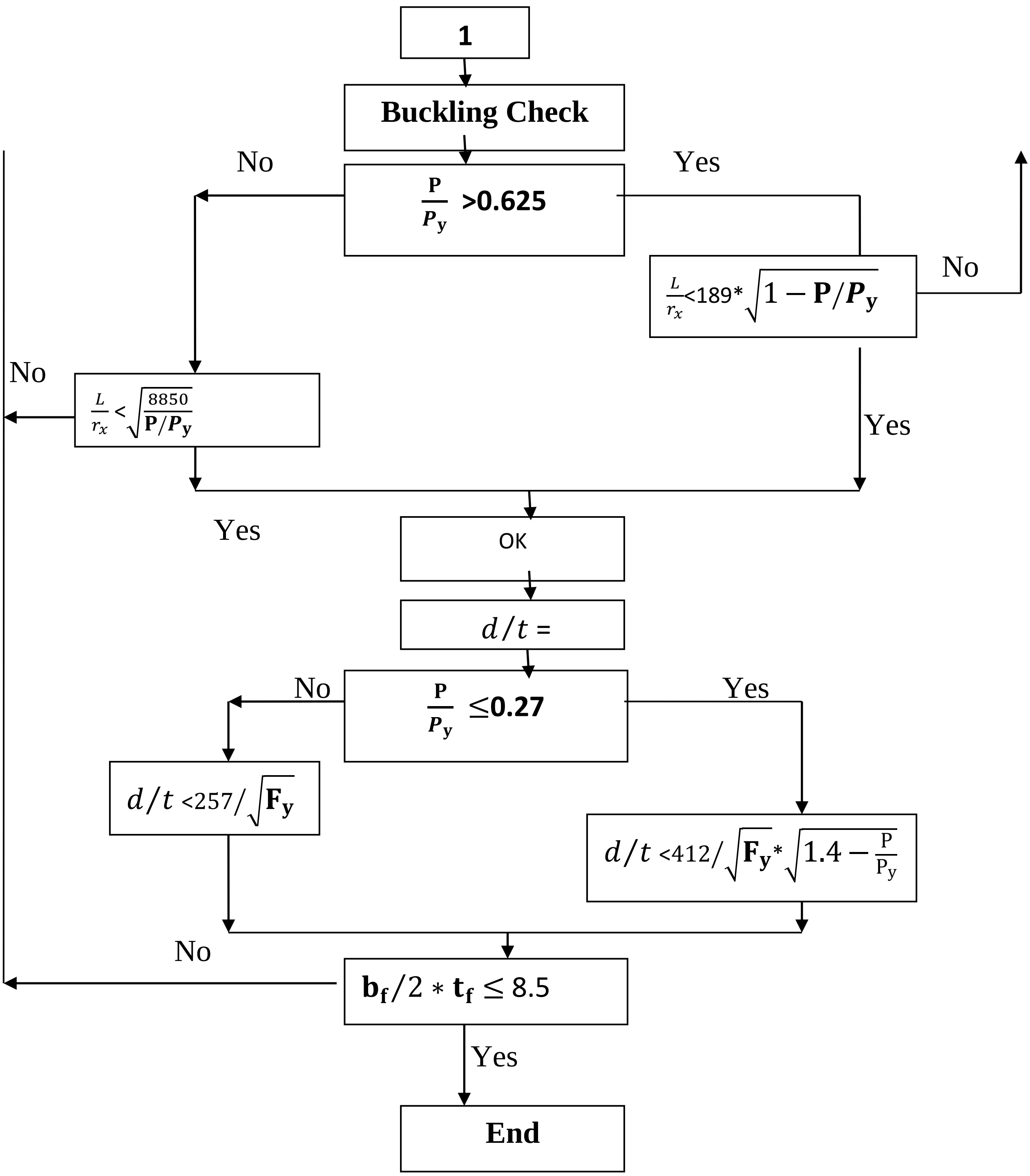


Figure 3.10.Flow Chart Plastic Design of Column.

CHAPTER FOUR

ANALYSIS AND DESIGN OF UNBRACED 2D FRAMES

4.1 Introduction

In this chapter, elastic and plastic analysis of unbraced frame under different loads (dead, live and wind) and their combinations using two structural analysis programs (MASTAN2 and SAP 2000).The critical load factors of unbraced frame were calculated using mechanism method. The mechanism method was programmed by MATLAB code. In this study, it was taken a plan of building which consists of three Storeys as shown in **Figure 4.1**.

In order to analyze the building, it was divided into four plane frames. The longitudinal frames were denoted by A-A-1 and A-A-2, the transverse frames by B-B-1 and B-B-2 as shown in **Figure 4.**2.

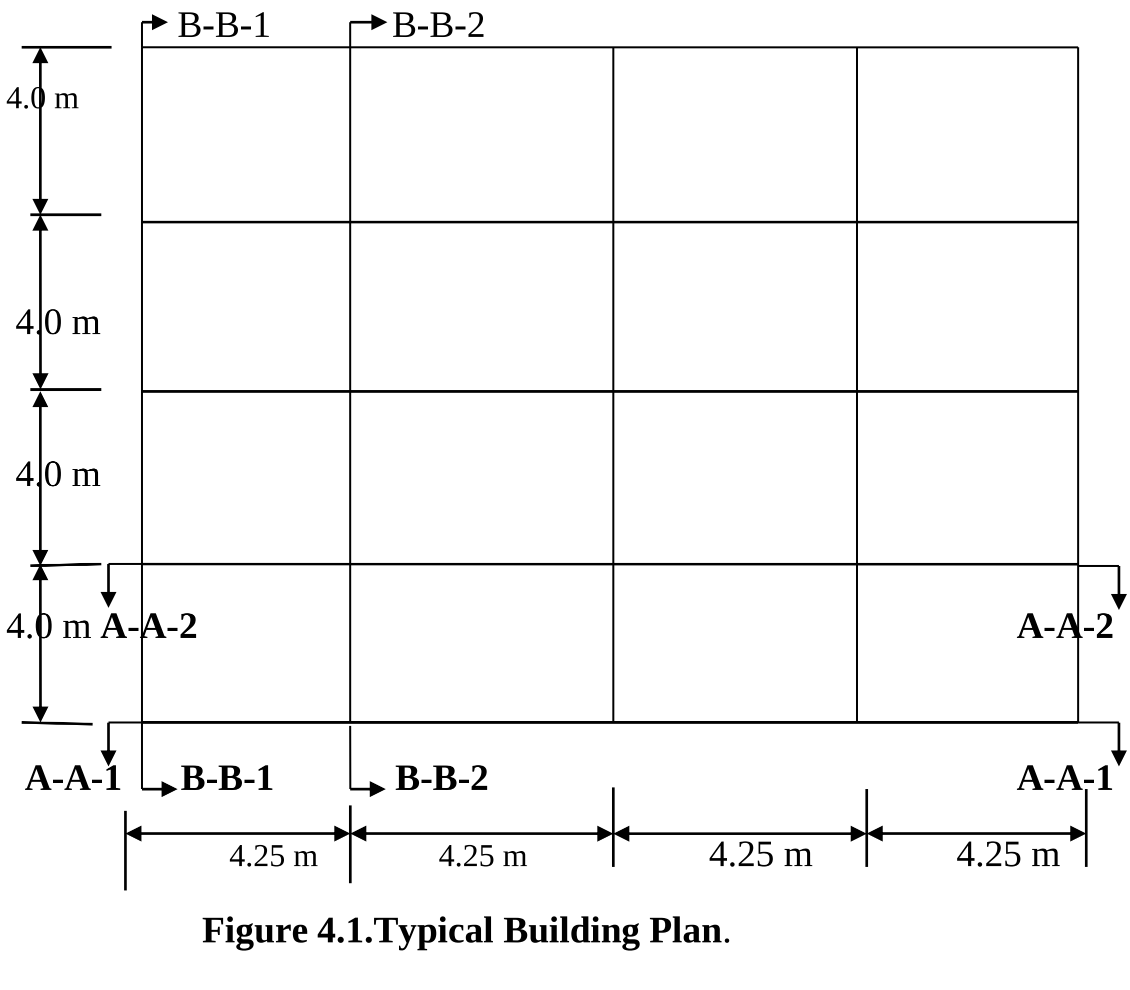


Figure 4.1.Typical Building Plan.

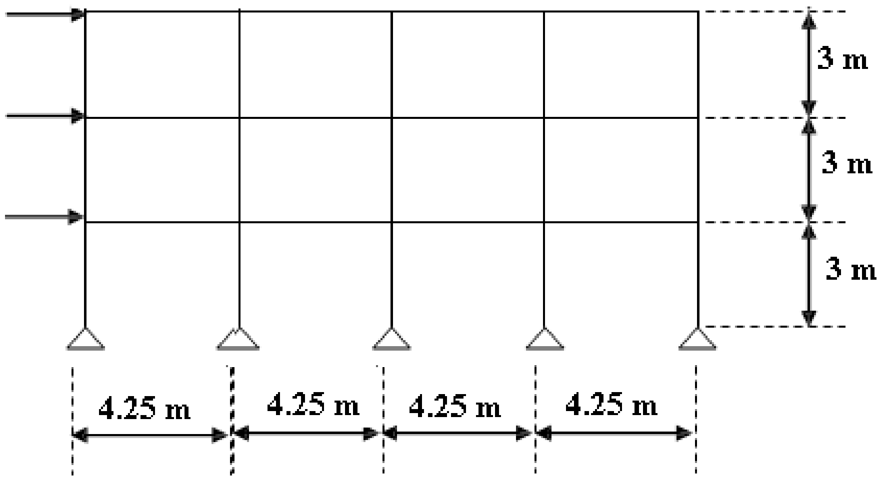


Figure 4.2a.Longitudinal Frame A-A.

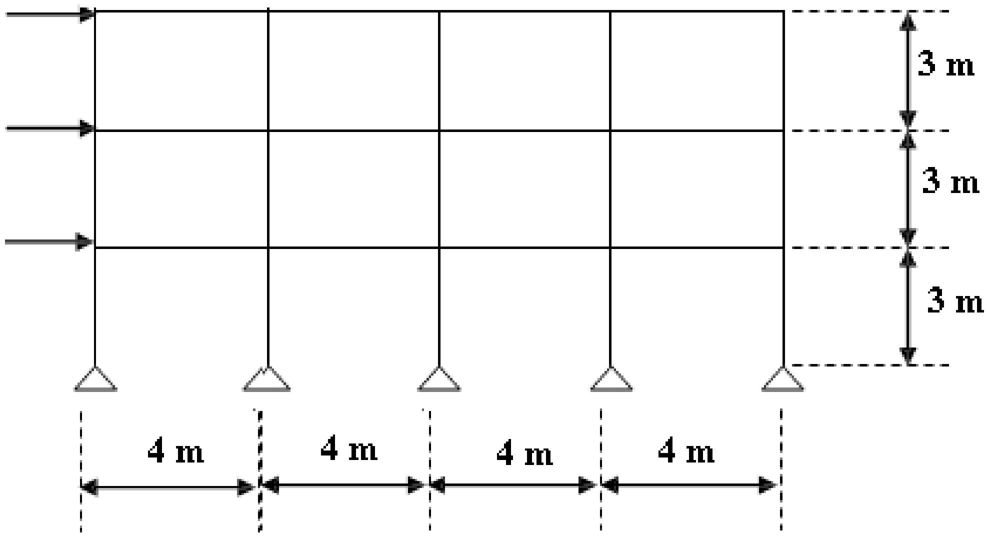


Figure 4.2b Transverse Frame B-B.

4.2. Loads calculation

The distribution of dead and live loads on the building slabs was presented in Figure 4.3. The loads applied to unbraced plane frames were carried out using excel spread sheet. Table 4.1 shows properties of materials and loads applied on the building. The loads applied to beams were calculated by excel spread sheets as shown in Tables 4.2a - 4.2b.

The wind loads applied on plane frames were calculated using analytical method according to ASCE-05 as shown in Tables 4.3a - 4.3b.

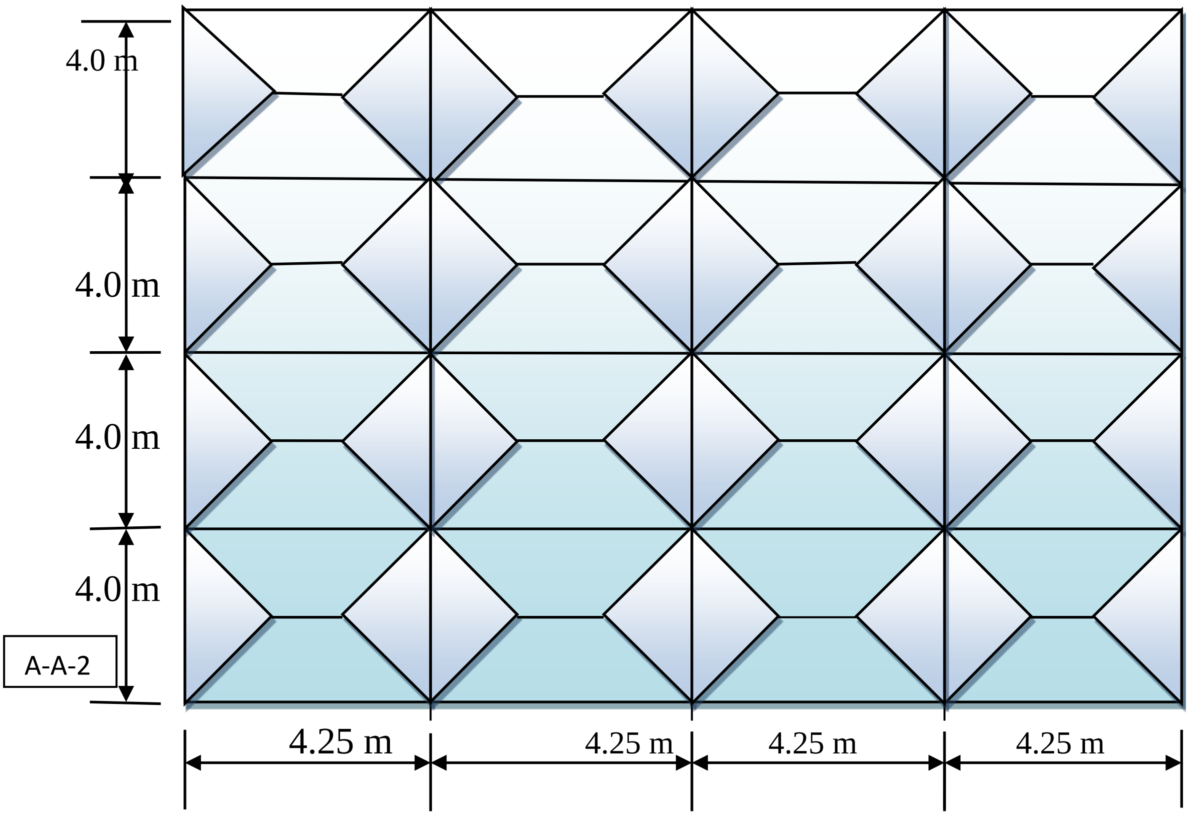


Figure 4.3.Distribution of slab Loads on Beams.

Table 4.1.Materials Properties and loads

Materials Properties	
Concrete Density (γ_{con})	24kN/m^3
Brick Density (γ_{brick})	18kN/m^3
Sections Properties	
Slab Thickness (h_{slab})	0.140m
Wall Thickness (b_{wall})	0.20m
Wall Height (h_{wall})	
Roof	**Floor**
1.50m	3.0m
Finishing Load	
Finishing Load	1.0kN/m^2
Live Load	
Roof	Floor
1.5kN/m^2	3.0kN/m^2

Table 4.2a.Dead Loads Calculate on Slab

Self-weight of slab		
Element	Equation	Value
Slab	$\gamma_{con}*h_{slab}$	3.36kN/m^2
roof walls	0.5*γ_{brick}*2($\frac{Lx+L}{Lx*L}$)* h_{wall}*b_{wall}	2.621kN/m^2
floor walls	0.5*γ_{brick}*2($\frac{Lx+L}{Lx*L}$)* h_{wall}*b_{wall}	5.24kN/m^2
Total Dead Load		
Roof	Self-weight of Wall+ Finishing Load+ Self-weight of Slab	6.981kN/m^2
Floor	Self-weight of Wall+ Finishing Load+ Self-weight of Slab	9.6kN/m^2

Table 4.2b.Dead and Live Loads Applied on plane Frames

Frame	**Load**	**Roof**(kN/m)	**Floor**(kN/m)
Frame A-A-1	Dead Load	19.68	27.06
	Live Load	4.23	8.46
Frame A-A-2	Dead Load	39.36	54.13
	Live Load	8.46	16.91
Frame B-B-1	Dead Load	18.61	25.60
	Live Load	4.00	8.00
Frame B-B-2	Dead Load	37.23	51.21
	Live Load	8.00	16

Table 4.3a.Wind Load Data

Exposure Category	B
GF	0.85
C	0.80
K_d	0.85
K_z	from Equation(2.4)
I	1.0; Exposure Category B
V	100mph
K_{zt}	1.0; assume a flat surface (Terrain)

Table 4.3b.Calculate of Wind Load on Frames

Risk Category	II	
basic wind speed	100.00	mph
Directionality Factor (Kd)	0.85	
Exposure Category	B	
importance factor	1.00	
Topographic Factor, K_{zt}	1.00	
Gust Effect Factor, G_f	0.85	
Terrain Exposure Constant, a	7.00	
Terrain Exposure Constant, z_g	1200.00	
topographic factor	1.00	
pressure or force coefficient,(C)	0.80	

Wind pressure				
Height, z(ft)	Kz	qz	P(Psf)	P(kN/m^2)
3	0.57	13.24	8.47	0.41
6	0.62	14.31	9.16	0.44
9	0.70	16.07	10.28	0.49

Frame Wind Load		
Frame	Floors (kN)	roof (kN)
A-A	6.29	3.15
B-B	5.92	2.96

For elastic analysis, it was taken five load cases as shown in Figure 4.4.

4.3. Load Combination for Elastic Analysis

1. 1.2DL+1.6LL

2. 1.2DL+0.8w

3. 1.2DL+1.LL+1.6*W

Table 4.4 show the Load Combination for elastic analysis

Table 4.4.Elastic Analysis Load Combination

Load combination 1			
Frame	Roof	Floor	
A-A-1	30.38	46.01	
A-A-2	60.76	92.02	
B-B-1	28.74	43.52	
B-B-2	57.48	87.05	
Load combination 2			
Frame	Roof (kN/m)	Floor(kN/m)	Wind (kN)
A-A-1	23.61	32.48	2.52
A-A-2	47.23	64.96	5.03
B-B-1	22.34	30.72	2.37
B-B-2	44.68	61.45	4.74
Load combination 3			
Frame	Roof	Floor	Wind
A-A-1	27.84	40.93	5.03
A-A-2	55.68	81.87	10.06
B-B-1	26.34	38.72	4.74
B-B-2	52.68	77.45	9.47

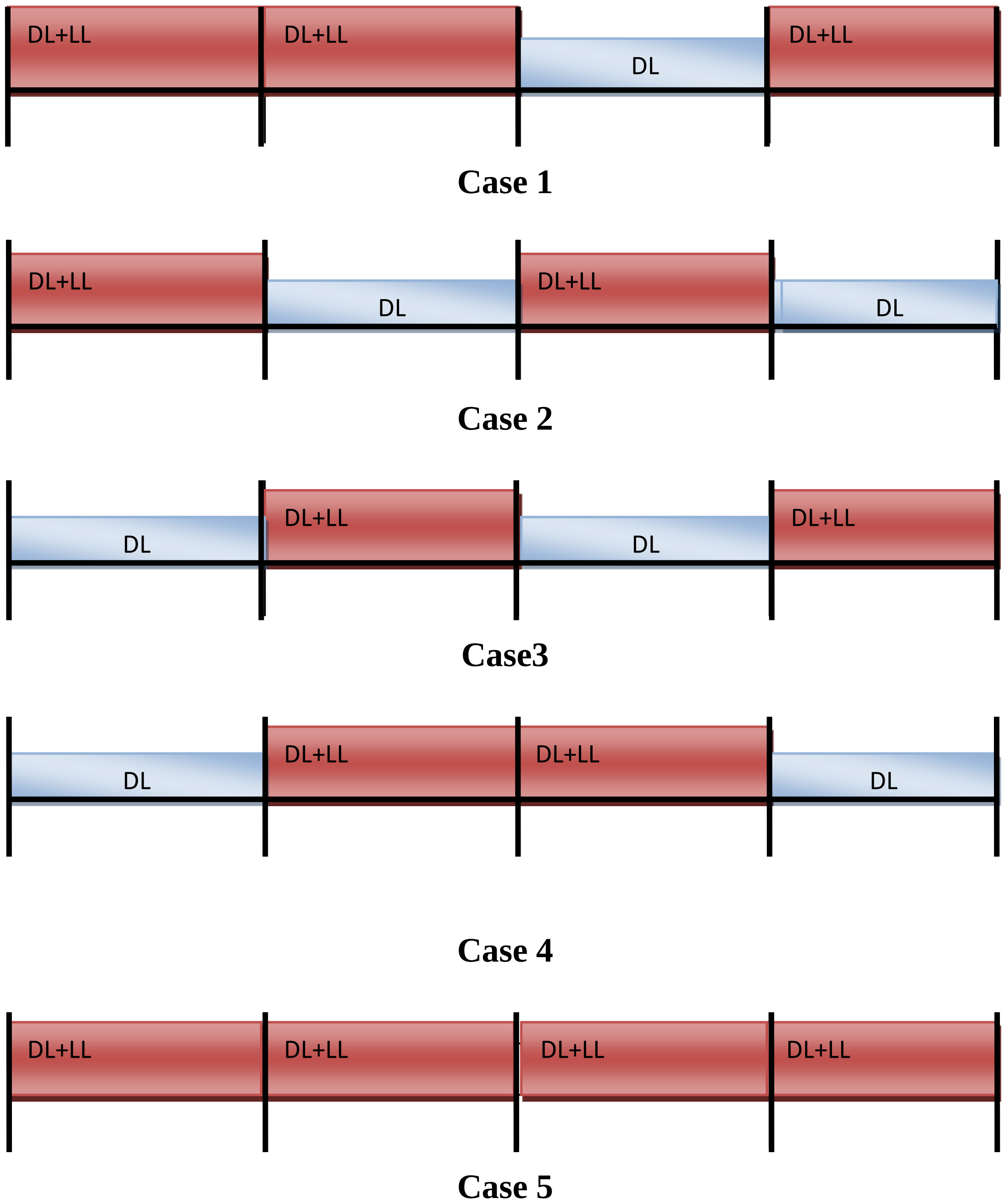


Figure 4.4.Load Cases Applied on the 2D Frames for Elastic Analysis

4.4. Results of Unbraced Frames Analysis

First and second orders elastic and plastic analyses of longitudinal and transverse plane frames were carried out using structural analysis programs MASTAN2 and SAP 2000. The elastic and plastic analysis results were presented in Tables (4.9 - 4.12) – (4.13 – 4.16) respectively.

Table 4.9.First Order Elastic Analysis of Beams

Frame		Program	M_{max} (kN.m)		V_{max} (kN)	Δ_{max} (mm)
			+ve	-ve		
Frame A-A-1	Roof	MASTAN2	24.05	47.18	66.04	4.19
		SAP 2000	23.63	47.42	66.3	3.36
		Difference	-1.75	0.51	0.39	-19.81
	Floor	MASTAN2	35.72	71.09	99.03	5.65
		SAP 2000	34.73	71.21	99.24	4.85
		Difference	-2.77	0.17	0.21	-14.16
Frame A-A-2	Roof	MASTAN2	47.89	93.94	131.5	8.17
		SAP 2000	46.99	94.32	131.87	6.68
		Difference	-1.88	0.40	0.28	-18.24
	Floor	MASTAN2	71.24	140.4	197.5	11.25
		SAP 2000	69.2	140.66	197.75	9.66
		Difference	-2.86	0.19	0.13	-14.13
Frame B-B-1	Roof	MASTAN2	20.21	39.46	58.79	3.17
		SAP 2000	20.34	39.6	58.91	3.95
		Difference	0.64	0.35	0.20	24.61
	Floor	MASTAN2	29.99	60.53	88.2	4.3
		SAP 2000	30.26	60.3	89.36	5.49
		Difference	0.90	-0.38	1.32	27.67
Frame B-B-2	Roof	MASTAN2	40.23	78.53	117	6.22
		SAP 2000	40.56	78.85	117.37	6.04
		Difference	0.82	0.41	0.32	-2.89
	Floor	MASTAN2	59.79	117.6	175.9	8.53
		SAP 2000	60.04	117.85	176.13	8.11
		Difference	0.42	0.21	0.13	-4.92

Table 4.10.Second Order Elastic Analysis of Beams

Frame		Program	M_{max} (kN.m)		V_{max} (kN)	Δ_{max} (mm)
			+ve	-ve		
Frame A-A-1	Roof	MASTAN2	24.1	47.2	66.06	4.11
		SAP 2000	23.63	47.42	66.3	3.36
		Difference	-1.95	0.47	0.36	-18.25
	Floor	MASTAN2	35.69	71.68	99.01	5.65
		SAP 2000	36.72	71.81	99.24	4.79
		Difference	2.89	0.18	0.23	-15.22
Frame A-A-2	Roof	MASTAN2	48.06	94.01	131.6	8.18
		SAP 2000	47.01	94.33	131.88	6.68
		Difference	-2.18	0.34	0.21	-18.34
	Floor	MASTAN2	71.09	140.03	197.4	11.24
		SAP 2000	69.21	140.66	197.75	9.66
		Difference	-2.64	0.45	0.18	-14.06
Frame B-B-1	Roof	MASTAN2	20.24	39.47	58.8	3.17
		SAP 2000	20.39	39.92	58.99	2.48
		Difference	0.74	1.14	0.32	-21.77
	Floor	MASTAN2	29.97	61.01	88.18	4.3
		SAP 2000	30.46	61.69	89.36	3.68
		Difference	1.63	1.11	1.34	-14.42
Frame B-B-2	Roof	MASTAN2	40.33	78.86	117.3	6.28
		SAP 2000	40.57	78.86	117.38	5.1
		Difference	0.60	0.00	0.07	-18.79
	Floor	MASTAN2	59.71	117.6	175.8	8.53
		SAP 2000	60.04	117.86	176.14	7.26
		Difference	0.55	0.22	0.19	-14.89

Table 4.11.First Order Elastic Analysis of Columns

`Column	Program	M_b kN.m	M_t kN.m	P_{max} kN
Ground Floor				
Column C1	MASTAN2	0	38.25	272.90
	SAP 2000	0	39.08	275.51
	Difference		2.17	0.96
Column C2	MASTAN2	0	63.73	531.40
	SAP 2000	0	65.62	533.75
	Difference		2.97	0.44
Column C3	MASTAN2	0	54.42	538.40
	SAP 2000	0	56.02	542.17
	Difference		2.94	0.70
Column C4	MASTAN2	0	27.46	1059.0
	SAP 2000	0	27.62	1064.88
	Difference		0.58	0.56
1st Floor				
Column C1	MASTAN2	40.00	42.82	170.80
	SAP 2000	38.78	39.89	172.47
	Difference	-3.05	-6.84	0.98
Column C2	MASTAN2	78.39	70.57	332.0
	SAP 2000	75.62	70.47	333.54
	Difference	-3.53	-0.14	0.46
Column C3	MASTAN2	65.92	59.86	336.60
	SAP 2000	64.44	59.79	339.17
	Difference	-2.25	-0.12	0.76
Column C4	MASTAN2	27.79	26.02	662.40
	SAP 2000	26.50	25.70	665.38
	Difference	-4.64	-1.23	0.45
2nd Floor				
Column C1	MASTAN2	34.13	43.47	68.55
	SAP 2000	34.03	43.14	69.42
	Difference	-0.29	-0.76	1.27

Column C2	MASTAN2	67.02	86.56	132.30
	SAP 2000	66.59	85.90	133.10
	Difference	-0.64	-0.76	0.60
Column C3	MASTAN2	56.37	72.60	135.3
	SAP 2000	55.97	71.96	136.61
	Difference	-0.71	-0.88	0.97
Column C4	MASTAN2	20.73	21.87	265.3
	SAP 2000	21.20	21.85	266.75
	Difference	-2.27	-0.09	0.55

Table 4.12.Second Order Elastic Analysis of Columns

`Column	**Program**	M_b **kN.m**	M_t **kN.m**	P_{max} **kN**
		Ground Floor		
Column C1	MASTAN2	0	38.88	272.80
	SAP 2000	0	40.29	275.50
	Difference		3.63	0.99
Column C2	MASTAN2	0	64.78	531.50
	SAP 2000	0	67.91	533.75
	Difference		4.83	0.42
Column C3	MASTAN2	0	55.71	538.40
	SAP 2000	0	57.84	542.17
	Difference		3.82	0.70
Column C4	MASTAN2	0	29.54	1059.0
	SAP 2000	0	30.30	1064.90
	Difference		2.57	0.56
		1st Floor		
Column C1	MASTAN2	40.53	42.95	170.80
	SAP 2000	39.22	40.46	172.52
	Difference	-3.23	-5.80	1.00
Column C2	MASTAN2	78.98	71.57	332.10
	SAP 2000	75.69	71.81	333.53
	Difference	-4.16	0.34	0.43

Column C3	MASTAN2	69.31	78.79	336.60
	SAP 2000	63.83	60.83	339.17
	Difference	-7.91	-22.79	0.76
Column C4	MASTAN2	27.72	27.46	662.40
	SAP 2000	26.55	27.36	665.39
	Difference	-4.22	-0.36	0.45
2nd Floor				
Column C1	MASTAN2	34.50	43.62	68.56
	SAP 2000	34.32	43.37	69.42
	Difference	-0.52	-0.57	1.25
Column C2	MASTAN2	67.45	86.83	132.40
	SAP 2000	66.61	85.92	133.10
	Difference	-1.25	-1.05	0.53
Column C3	MASTAN2	56.62	72.77	135.30
	SAP 2000	55.98	71.97	136.62
	Difference	-1.13	-1.10	0.98
Column C4	MASTAN2	20.58	21.87	265.30
	SAP 2000	21.18	22.09	266.75
	Difference	2.91	1.01	0.55

Table 4.13.First Order Plastic Analyses of Beams

Frame		Program	M_{max} (kN.m)		V_{max} (kN)	Δ_{max} (mm)	P_{max} kN
			+ve	-ve			
Frame A-A-1	**Roof**	MASTAN2	56.25	56.62	53.35	10.08	25.28
		SAP 2000	56.03	56.03	53	11.08	24.85
		Difference	-0.39	-1.04	-0.66	9.92	-1.70
	Floors	MASTAN2	82.87	82.87	78.23	13.96	14.59
		SAP 2000	83	83	78.26	14.42	13.9
		Difference	0.16	0.16	0.04	3.30	-4.73
Frame A-A-2	**Roof**	MASTAN2	55.06	55.04	51.93	9.91	24.75
		SAP 2000	57.76	57.76	54.3	11.4	24.13
		Difference	4.90	4.94	4.56	15.04	-2.51
	Floors	MASTAN2	82.89	82.89	78.1	19.74	17.2
		SAP 2000	83	83	78.14	16.76	15.2
		Difference	0.13	0.13	0.05	-15.10	-11.63
Frame B-B-1	**Roof**	MASTAN2	55.84	55.78	56.08	9.26	25.04
		SAP 2000	56.04	56.04	55.61	10.32	24.74
		Difference	0.36	0.47	-0.84	11.45	-1.20
	Floors	MASTAN2	82.87	82.87	83.12	11.7	14.24
		SAP 2000	83	83	83.14	13.19	13.7
		Difference	0.16	0.16	0.02	12.74	-3.79
Frame B-B-2	**Roof**	MASTAN2	59.18	58.74	59.09	9.8	25.25
		SAP 2000	57.91	57.91	57.61	10.65	24
		Difference	-2.15	-1.41	-2.50	8.67	-4.95
	Floors	MASTAN2	82.89	82.89	82.99	17.68	16.68
		SAP 2000	83	83	83.03	15.59	15.07
		Difference	0.13	0.13	0.05	-11.82	-9.65

Table 4.14.Second Order Plastic Analysis of Beams

Frame		Program	M_{max} (kN.m)		V_{max}	Δ_{max}	P_{max}
			+ve	-ve	(kN)	(mm)	kN
Frame A-A-1	**Roof**	MASTAN2	56.37	56.80	53.50	10.12	25.13
		SAP 2000	55.70	55.52	51.22	10.91	24.85
		Difference	-1.19	-2.25	-4.26	7.81	-1.11
	Floors	MASTAN2	82.86	82.89	78.15	15.47	13.52
		SAP 2000	82.90	82.90	77.48	14.41	13.43
		Difference	0.05	0.01	-0.86	-6.85	-0.67
Frame A-A-2	**Roof**	MASTAN2	54.90	54.89	51.79	9.90	24.38
		SAP 2000	55.66	55.66	49.54	11.14	23.60
		Difference	1.38	1.40	-4.34	12.53	-3.20
	Floors	MASTAN2	82.89	82.89	78.10	20.03	17.82
		SAP 2000	82.90	82.90	76.08	18.17	16.76
		Difference	0.01	0.01	-2.59	-9.29	-5.95
Frame B-B-1	**Roof**	MASTAN2	55.44	55.41	55.68	9.20	24.70
		SAP 2000	56.54	56.54	53.19	10.36	24.84
		Difference	1.98	2.04	-4.47	12.61	0.57
	Floors	MASTAN2	82.87	82.87	82.98	11.77	14.20
		SAP 2000	82.90	82.90	80.33	16.12	13.44
		Difference	0.04	0.04	-3.19	36.96	-5.35
Frame B-B-2	**Roof**	MASTAN2	59.36	59.00	59.33	9.83	25.28
		SAP 2000	55.96	55.96	52.77	10.29	23.26
		Difference	-5.73	-5.15	-11.06	4.68	-7.99
	Floors	MASTAN2	82.90	82.89	82.96	19.09	17.72
		SAP 2000	82.90	82.90	80.81	13.46	14.99
		Difference	0.00	0.01	-2.59	-29.49	-15.41

Table 4.15.First Order Plastic Analysis of Columns

`Column		M_b kN.m	M_t kN.m	P_{max} kN
Ground Floor				
Column C1	MASTAN2	0	29.93	194.70
	SAP 2000	0	29.75	193.91
	Difference	0	-0.61	-0.41
Column C2	MASTAN2	0	30.35	418.50
	SAP 2000	0	27.01	418.37
	Difference	0	-12.37	-0.03
Column C3	MASTAN2	0	30.04	397.80
	SAP 2000	0	26.96	395.31
	Difference	0	-11.42	-0.63
Column C4	MASTAN2	0	6.81	440.20
	SAP 2000	0	6.36	430.93
	Difference	0	-7.08	-2.15
1st Floor				
Column C1	MASTAN2	39.19	36.17	120.70
	SAP 2000	37.02	35.16	120.16
	Difference	-5.86	-2.87	-0.45
Column C2	MASTAN2	46.94	39.37	262.20
	SAP 2000	40.30	35.11	262.21
	Difference	-16.48	-12.13	0
Column C3	MASTAN2	45.96	38.35	249.50
	SAP 2000	40.10	34.98	247.78
	Difference	-14.61	-9.63	-0.69
Column C4	MASTAN2	2.68	5.12	276.20
	SAP 2000	3.20	4.49	269.56
	Difference	16.25	-14.03	-2.46
2nd Floor				
Column C1	MASTAN2	31.99	36.98	47.0
	SAP 2000	31.47	35.78	46.82
	Difference	-1.65	-3.35	-0.38

Column C2	MASTAN2	34.45	36.58	107.60
	SAP 2000	32.30	36.77	107.55
	Difference	-6.66	0.52	-0.05
Column C3	MASTAN2	34.13	38.10	102.40
	SAP 2000	32.10	36.42	101.64
	Difference	-6.32	-4.61	-0.75
Column C4	MASTAN2	1.71	4.16	113.4
	SAP 2000	2.44	4.02	110.48
	Difference	29.92	-3.48	-2.64

Table 4.16.Second Order Plastic Analysis of Columns

`Column		**M_b kN m**	**M_t kN m**	**P_{max} kN**
Ground Floor				
Column C1	MASTAN2	0	33.21	194.20
	SAP 2000	0	36.27	199.18
	Difference		9.21	2.56
Column C2	MASTAN2	0	32.37	414.20
	SAP 2000	0	41.48	418.42
	Difference		28.14	1.02
Column C3	MASTAN2	0	32.97	296.60
	SAP 2000	0	33.27	427.93
	Difference		0.91	44.28
Column C4	MASTAN2	0	9.27	439.6
	SAP 2000	0	20.89	413.41
	Difference	0	125.35	-5.96
1st Floor				
Column C1	MASTAN2	41.25	37.41	119.80
	SAP 2000	40.88	37.46	121.65
	Difference	-0.90	0.13	1.54
Column C2	MASTAN2	47.58	39.78	259.90
	SAP 2000	43.45	38.11	263.77
	Difference	-8.68	-4.20	1.49

Column C3	MASTAN2	48.05	39.89	249.50
	SAP 2000	38.37	34.66	247.43
	Difference	-20.15	-13.11	-0.83
Column C4	MASTAN2	2.29	6.16	266.20
	SAP 2000	3.10	8.88	259.34
	Difference	35.37	44.16	-2.58
2^{nd} Floor				
Column C1	MASTAN2	32.05	37.15	46.61
	SAP 2000	30.84	36.17	47.13
	Difference	-3.78	-2.64	1.12
Column C2	MASTAN2	34.02	36.52	106.70
	SAP 2000	30.29	36.16	108.27
	Difference	-10.96	-0.99	1.47
Column C3	MASTAN2	34.73	38.44	102.50
	SAP 2000	30.76	35.31	101.53
	Difference	-11.43	-8.14	-0.95
Column C4	MASTAN2	1.40	4.37	113.5
	SAP 2000	2.38	4.76	106.29
	Difference	70.00	8.92	-6.35

4.5. Excel Spread Sheet for MATLAB Code of Frame A-A-1

Tables (4.17 –4.23) presented the excel spread sheet for MATLAB code for determine the critical load factors for 2d unbraced frames.

Enter Number of Story	**3**
Enter Number of Bay	**4**

Enter coordinates, respect to node ID

Table 4.17.Coordinates of Nodes

NS=3 ; NB=4			
ID	X	Y	n
1	0	0	1
2	0	3000	2
3	0	6000	3
4	0	9000	4
5	4500	0	5
6	4500	3000	6
7	4500	6000	7
8	4500	9000	8
9	8750	0	9
10	8750	4250	10
11	8750	6000	11
12	8750	9000	12
13	13000	0	13
14	13000	3000	14
15	13000	6000	15
16	13000	9000	16
17	17250	0	17
18	17250	3000	18
19	17250	6000	19
20	17250	9000	20

D	**1**

D = 1 for distributed load and 2equal 2 for concentrated load

Enter properties of element, respect to ID

Table 4.18.Properties of 2D unbraced Frame

NS=3 : NB=4				
Element	Mp	Start	End	L(m)
1	82899	1	2	3000
2	82899	2	6	4500
3	82899	5	6	3000
4	82899	4	8	3000
5	82899	3	7	4500
6	82899	6	7	3000
7	82899	3	4	3000
8	82899	4	8	4500
9	82899	7	8	3000
10	82899	8	12	4250
11	82899	11	12	3000
12	82899	7	11	4250
13	82899	10	11	3000
14	82899	6	10	4250
15	82899	9	10	3000
16	82899	12	16	4250
17	82899	15	16	3000
18	82899	11	15	4250
19	82899	10	14	4250
20	82899	13	14	4250
21	82899	14	15	3000
22	82899	16	20	4250
23	82899	19	20	3000
24	82899	15	19	4250
25	82899	18	19	3000
26	82899	14	18	4250
27	82899	17	18	3000

Table 4.19.Number of constraints

<table>
<tr><td colspan="5">number of constraint nodes(number of support)</td><td>2</td><td>3</td><td>4</td></tr>
<tr><td>ID</td><td>X ?</td><td>Y ?</td><td>Z?</td><td>sum</td><td colspan="2">SR</td><td>n</td></tr>
<tr><td>1</td><td>1</td><td>1</td><td>0</td><td>2</td><td colspan="2">2</td><td>1</td></tr>
<tr><td>5</td><td>1</td><td>1</td><td>0</td><td>2</td><td colspan="2">4</td><td>2</td></tr>
<tr><td>9</td><td>1</td><td>1</td><td>0</td><td>2</td><td colspan="2">6</td><td>3</td></tr>
<tr><td>13</td><td>1</td><td>1</td><td>0</td><td>2</td><td colspan="2">8</td><td>4</td></tr>
<tr><td>17</td><td>1</td><td>1</td><td>0</td><td>2</td><td colspan="2">10</td><td>5</td></tr>
</table>

Table 4.20.Vertical Forces on frame A-A-1

NS=3 ; NB=4	
Element	Y-force
1	0
2	196.26
3	0
4	0
5	196.26
6	0
7	0
8	132.08
9	0
10	132.08
11	0
12	196.26
13	0
14	196.26
15	0
16	132.08
17	0
18	196.26
19	0
20	196.26
21	0
22	132.08
23	0
24	196.26
25	0
26	196.26
27	0

Table 4.21.Storys High of Frame A-A-1

Enter Story High		3000	3000	3000
Story	High			
1	3000			
2	3000			
3	3000			

Table 4.22.Horizontal Forces of Frame A-A-1

Enter Horizontal Force	3
Story	X-force
1	8.18
2	8.18
3	4.08

Table 4.23.Frames Properties for sway Mechanism

Story 1									
EN	1	2	3	4	5	6	7	8	9
Element	1	2	3	9	10	18	19	24	25
Mps	82899	82899	82899	82899	82899	82899	82899	82899	82899
MPb1	82899				MPc1	82899			
Mpb1					Mpc1				
NB=1									
EN	2				EN	1	3		
Mpb1	82899				Mpc1	82899	82899		
NB=2									
EN	2	9			EN	1	3	10	
Mpb1	82899	82899			Mpc1	82899	82899	82899	
NB=3									
EN	2	9	18		EN	1	3	10	19
Mpb1	82899	82899	82899		Mpb1	82899	82899	82899	82899
NB=4									

EN	2	9	18	24	EN	1	3	10	19	25
Mpb1	82899	82899	82899	82899		82899	82899	82899	82899	82899

%Story 2									
EN	1	2	3	4	5	6	7	8	9
Element	4	5	6	7	8	17	20	23	26
Mps	82899	82899	82899	82899	82899	82899	82899	82899	82899
MPb2	82899				MPc2	82899			
Mpb2					Mpc2				
NB=1									
EN	5				EN	4	6		

Mpb1	**82899**				**Mpc1**	**82899**	**82899**			
NB=2										
EN	**5**	**7**			**EN**	**4**	**6**	**8**		
Mpb1	**82899**	**82899**			**Mpc1**	**82899**	**82899**	**82899**		
NB=3										
EN	**5**	**7**	**17**		**EN**	**4**	**6**	**8**	**20**	
Mpb1	**82899**	**82899**	**82899**		**Mpb1**	**82899**	**82899**	**82899**	**82899**	
NB=4										
EN	**5**	**7**	**17**	**23**	**EN**	**4**	**6**	**8**	**20**	**26**
Mpb1	**82899**	**82899**	**82899**	**82899**	**Mpb1**	**82899**	**82899**	**82899**	**82899**	**82899**
%Story 3										
EN	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	
Element	**7**	**8**	**9**	**10**	**15**	**16**	**21**	**22**	**27**	
Mps	**82899**	**82899**	**82899**	**82899**	**82899**	**82899**	**82899**	**82899**	**82899**	
MPb3	**82899**				**MPc3**	**82899**				
Mpb3					**Mpc3**					
NB=1										
EN	**8**				**EN**	**7**	**9**			
Mpb1	**82899**				**Mpc1**	**82899**	**82899**			
NB=2										
EN	**8**	**10**			**EN**	**7**	**9**	**15**		
Mpb1	**82899**	**82899**			**Mpc1**	**82899**	**82899**	**82899**		
NB=3										
EN	**8**	**10**	**16**		**EN**	**7**	**9**	**15**	**21**	
Mpb1	**82899**	**82899**	**82899**		**Mpc1**	**82899**	**82899**	**82899**	**82899**	
NB=4										
EN	**8**	**10**	**16**	**22**	**EN**	**7**	**9**	**15**	**21**	**27**
Mpb1	**82899**	**82899**	**82899**	**82899**	**Mpc1**	**82899**	**82899**	**82899**	**82899**	**82899**

The critical load factor of longitudinal and transverse frames were determined by first and second order plastic analysis by MATLAB Code, MASTAN2, and SAP 2000 as shown in table 4.24-4.25.

Table 4.24 Critical Load Factors Using First Order Plastic Analysis

Frame	PROGRAM	γ_c	PROGRAM	γ_c
Frame A-A-1	MASTAN2	0.724	MASTAN2	0.724
	MATLAB	0.795	SAP 2000	0.718
	Difference	9.807	Difference	-0.829
Frame A-A-2	MASTAN2	0.382	MASTAN2	0.382
	MATLAB	0.398	SAP 2000	0.370
	Difference	4.188	Difference	-3.141
Frame B-B-1	MASTAN2	0.804	MASTAN2	0.804
	MATLAB	0.892	SAP 2000	0.802
	Difference	10.945	Difference	-0.249
`Frame B-B-2	MASTAN2	0.427	MASTAN2	0.427
	MATLAB	0.447	SAP 2000	0.415
	Difference	4.684	Difference	-2.810

Table 4.25 Critical Load Factors Using Second Order Plastic Analysis

Frame	PROGRAM	γ_c
Frame A-A-1	MASTAN2	0.724
	SAP 2000	0.709
	Difference	-2.072
Frame A-A-2	MASTAN2	0.380
	SAP 2000	0.360
	Difference	-5.263
Frame B-B-1	MASTAN2	0.796
	SAP 2000	0.807
	Difference	1.382
`Frame B-B-2	MASTAN2	0.427
	SAP 2000	0.399
	Difference	-6.557

4.6. Results Dissection

The elastic and plastic analyses of first and second order for longitudinal and transverse unbraced frames of building were carried out using structural analysis programsMASTAN2 and SAP 2000.The Program MASTAN2 is as taken as basic reference for comparison.

In order to determine the critical load factors of unbraced frames, mechanism method was programmed using MATLAB code for first and second order plastic analysis

The comparison of floor and roof beams results (moments, shear forces and deflections) for each frame for elastic and plastic analyses of first and second order was summarized as follows:

Longitudinal frame A-A-1

1. For elastic analysis by first order the differences of moments, shear forces and deflections for floor and roof beams were about (-1.75% - -2.77%), (0.21% - 0.39%), and (-14.16% - -19.81%) respectively. But for second order analysis the differences of moments, shear forces and deflections for floor and roof beams were about (-2.18% - -2.64%), (0.18% - 0.21%), and (-14.06% - -18.34) respectively.
2. For Plastic Analysis by first order the differences of moments, shear forces, deflections and axial forces for floor and roof beams about (-0.39% - 0.16), (0.04% - -0.66%), (3.3% - 9.9%), and (-1.70% - -4.73%) respectively. But for second order analysis the differences of moments, shear forces, deflections and axial forces for floor and roof beams were about (0.05% to -1.19%), (-0.86% - -4.26%), (-6.85% - 7.81), and (-0.67% - -1.11%) respectively.

Longitudinal frame A-A-2

1. For elastic analysis by first order the differences of moments, shear forces and deflections for floor and roof beams were about (-1.88% - -2.86%), (0.13% - 0.28%), and (-14.13% - -18.24%) respectively, but for second order analysis the differences of moments, shear forces and deflections for floor and roof beams were about (-1.95% - 2.89%), (0.23% - 0.36%), and (-15.22% - -18.25) respectively.
2. For plastic analysis by first order The differences of moments, shear forces, deflections and axial forces for floor and roof beams were about (0.13% - 0.94%), (0.05% - 4.56%), (15.04% - -15.10%), and (-2.51% - -1.63%) respectively, but for second order

analysis the differences of moments, shear forces, deflections and axial forces for floor and roof beams were about (0.01% - 1.38%), (-2.59% - -4.34%), (-9.29% - 12.53), and (-3.20% - -5.95%) respectively.

Transverse frame B-B-1

1. For elastic analysis by first order the differences of moments, shear forces and deflections for floor and roof beams were about (0.64% - 0.9%), (0.20% - 0132%), and (24.61% - 27.67%) respectively, but for second order analysis The differences of moments, shear forces and deflections for floor and roof beams were about (0.74% - 1.63%), (0.32% - 1.34%), and (-14.42% - -21.77) respectively.
2. For plastic analysis by first order The differences of moments, shear forces, deflections and axial forces for floor and roof beams were about (0.16% - 0.36%), (0.02% - -0.84%), (11.45% - 12.74%), and (-1.20% - -3.79%) respectively, but for second order analysis The differences of moments, shear forces, deflections and axial forces for floor and roof beams were about (0.04% - 1.98%), (-3.13% - -4.47%), (12.61% - 36.96), and (-14.16% - -19.81%) respectively

Transverse frame B-B-2

1. For elastic analysis by first order The differences of moments, shear forces and deflections for floor and roof beams were about (0.42% - 0.82%), (0.13% - 0.32%), and (-2.89% - -4.92%) respectively, but for second order analysis the differences of moments, shear forces and deflections for floor and roof beams were about (0.55% - 0.60%), (0.07% - 0.19%), and (-14.89% - -18.79) respectively.

2 For plastic analysis by first order the differences of moments, shear forces, deflections and axial forces for floor and roof

beams were about (0.13% - -2.15%), (0.05% - -2.50%), (8.67% - -11.82%), and (-4.95% - -9.65%) respectively, but for second order analysis The differences of moments, shear forces, deflections and axial forces for floor and roof beams were about (0% - -5.73%), (-2.59% - -11.06%), (4.68% - -29.49.79), and (-7.99% - -15.41%) respectively.

The comparison of columns results (top moments, button moments and axial forces) for elastic and plastic analyses of first and second order was summarized as follows

Column C1

- Ground Floor

1. For first order elastic analysis the differences of top moments, button moments and axial forces were (0%), 2.17%, and 0.44% respectively, but for second order elastic analysis The differences of top moments, button moments and axial forces were about 0%, 3.63, and 0.99% respectively.
2. For first order plastic analysis the differences of top moments, button moments and axial forces were (0%), (-2.17%), and (-0.96%) respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about 0%, 9.21, and 2.56% respectively.

- 1st Floor

1. For first order elastic analysis the differences of top moments, button moments and axial forces were -3.05%, -6.84%, and 0.98% respectively. But for second order elastic analysis, the differences of top moments, button moments and axial forces were about -3.23%, -5.80, and 1.00% respectively.

2. For first order plastic analysis the differences of top moments, button moments and axial forces for first floor Column C1 were -5.86%, 2.87%, and-0.45% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about -0.90%, 0.13, and 1.54% respectively.

- 2nd Floor

1. For first order elastic analysis the differences of top moments, button moments and axial forces were -0.29%, -0.76%, and 1.27% respectively. But for second order elastic analysis, the differences of top moments, button moments and axial forces were about-0.52 %, -0.57, and 1.25% respectively.
2. For first order plastic analysis the differences of top moments, button moments and axial forces were -1.65%, -3.35%, and -0.38% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about -3.78%, -2.64, and 1.12% respectively.

Column C2

- Ground Floor

1. For first order elastic analysis The differences of top moments, button moments and axial forces were 0%, 2.97%, and 0.44% respectively, but for second order elastic analysis The differences of top moments, button moments and axial forces were about 0%, 4.83, and 0.0.42% respectively.
2. For first order plastic analysis, the differences of top moments, button moments and axial forces were 0%, -0.61%, and -0.41% respectively. But for second order plastic analysis, the differences

of top moments, button moments and axial forces were about 0%, 28.14, and 1.02% respectively.

- 1st Floor

  1. For first order elastic analysis, the differences of top moments, button moments and axial forces were -3.53%, -.14%, and -0.46% respectively. But for second order elastic analysis, the differences of top moments, button moments and axial forces were about -4.16%, 0.34, and 0.43% respectively.
  2. For first order plastic analysis, the differences of top moments, button moments and axial forces were -16.48%, -12.13%, and 0% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about -.8.68%, -4.20, and 1.49% respectively.

- 2nd Floor

1. For first order elastic analysis, the differences of top moments, button moments and axial forces were -0.64%, -0.76%, and 0.60% respectively. But for second order elastic analysis, the differences of top moments, button moments and axial forces were about -1.25%, -1.05%, and 0.53% respectively.
2. For first order plastic analysis, the differences of top moments, button moments and axial forces were -6.66%, 0.52%, and -0.05% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about -10.96%, -.99, and 1.47% respectively.

Column C3

- Ground Floor

1. For first order elastic analysis the differences of top moments, button moments and axial forces were 0%, 2.94%, and 0.70%

respectively, but for second order elastic analysis the differences of top moments, button moments and axial forces were about 0%, 3.82, and 0.70% respectively.

2. For first order plastic analysis, the differences of top moments, button moments and axial forces were 0%, -12.37%, and -0.03% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about 0%, 0.91, and 44.28% respectively.

- 1st Floor
    1. For first order elastic analysis, the differences of top moments, button moments and axial forces were -2.25%, -.12%, and -0.76% respectively. But for second order elastic analysis, the differences of top moments, button moments and axial forces were about -7.91%, -22.79, and 0.76% respectively.
    2. For first order plastic analysis, the differences of top moments, button moments and axial forces were -14.61%, -9.63%, and -0.69% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about -20.15%, -13.11, and -0.83% respectively.
- 2nd Floor
    1. For first order elastic analysis, the differences of top moments, button moments and axial forces were -0.71%, -0.88%, and 0.97% respectively. But for second order elastic analysis, the differences of top moments, button moments and axial forces were about -1.13%, -1.10, and 0.98% respectively.
    2. For first order plastic analysis, the differences of top moments, button moments and axial forces were -6.32%, -

4.61%, and -0.75% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about -11.43%, -8.14, and -0.95% respectively.

Column C4

- Ground Floor

1. For first order elastic analysis the differences of top moments, button moments and axial forces were 0%, 0.58%, and 0.56% respectively, but for second order elastic analysis the differences of top moments, button moments and axial forces were about (0%), (-3.63), and (-0.99%) respectively.
2. For first order plastic analysis, the differences of top moments, button moments and axial forces were 0%, -7.08%, and -2.15% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about 0%, 125, and -5.96%) respectively.

- 1st Floor

1. For first order elastic analysis, the differences of top moments, button moments and axial forces C4 were -4.64%, -1.23%, and 0.45% respectively. But for second order elastic analysis, the differences of top moments, button moments and axial forces were about -4.22%, -.36, and 0.45% respectively.
2. For first order plastic analysis, the differences of top moments, button moments and axial forces 16.25%, -14.03%, and -2.46% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about 35.37%, 44.16, and -2.58% respectively.

- 2nd Floor

1. For first order elastic analysis, the differences of top moments, button moments and axial forces were -2.27%, -0.09%, and 0.55% respectively. But for second order elastic analysis, the differences of top moments, button moments and axial forces were about 2.91%, 1.01, and 0.55% respectively.
2. For first order plastic analysis, the differences of top moments, button moments and axial forces were 29.92%, -3.48%, and -2.64% respectively. But for second order plastic analysis, the differences of top moments, button moments and axial forces were about 70%, 8.92, and -6.35% respectively.

   Comparison of unbraced frames critical load factors for first order plastic analysis using MASTAN2 and MATLAB was summarized as follows

   For longitudinal frames A-A-1 and A-A-2 the difference of critical load factors was (9.807) and (4.188) respectively.

   The difference of critical load factor for transverse frames B-B1 and B-B-2 was (10.945) and (4.684) respectively.

   Comparison of unbraced frames critical load factors for first and second order plastic analysis using MASTAN2 and SAP 2000 was summarized as follows

   1. For first and second order plastic analysis, the difference of critical load factors for longitudinal frame A-A-1 was (-0.829) and (-2.072) respectively.
   2. For first and second order plastic analysis, the difference of critical load factors for longitudinal frame A-A-2 was (-3.141) and (-5.263) respectively.
   3. . For first and second order plastic analysis, the difference of critical load factors for transverse frame B-B-1 was (-0.249) and (1.382) respectively.

4. For first and second order plastic analysis, the difference of critical load factors for transverse frame B-B-2 was (-2.810) and (-6.557) respectively.

4.7. Elastic and Plastic Design of Frames Elements

The beams and columns design was carried out using AISC Specifications to resist the maximum internal forces that obtained from analysis of 2D unbraced frame. The equations used for design were formulated using excel spreadsheet and the elastic and plastic design results for frames were presented for all frames shown in Appendix B. Tables ((4.26) – (4.31)) present the tables design of 2D unbraced steel frames.

Table4.26.Beams Section for Elastic and Plastic Design

Beams Sections	Elastic design	Plastic design
Frame A-A-1 [Roof]	W8*24	W6*20
Frame A-A-1 [Floors]	W8*35	W8*21
Frame A-A-2 [Roof]	W8*48	W6*20
Frame A-A-2 [Floors]	W8*67	W8*21
Frame B-B-1 [Roof]	W8*21	W6*20
Frame B-B-1 [Floors]	W8*31	W8*21
Frame B-B-2 [Roof]	W8*40	W6*20
Frame B-B-2 [Floors]	W8*58	W8*21

Table 4.27.Columns Section for Elastic and Plastic Design

Columns Sections	Elastic Design	Plastic Design
Ground Floor		
Column C1	W8*31	W6*16
Column C2	W10*45	W6*20
Column C3	W8*28	W6*20
Column C4	W10*54	W5*16
First Floor		
Column C1	W10*112	W5*19
Column C2	W12*106	W6*20
Column C3	W10*39	W6*20
Column C4	W10*39	W4*13
Second Floor		
Column C1	W8*21	W5*16
Column C2	W10*30	W5*19
Column C3	W8*31	W6*16
Column C4	W6*25	W4*13

Table 4.28Elastic Design of Column Splice Connections

Column	Elastic Design
Column C1	Use 2 of A325 Bolts 30 mm
Column C2	Use 2 of A325 Bolts 30 mm
Column C3	Use 2 of A325 Bolts 30 mm
Column C4	Use 4 of A325 Bolts 30 mm

Table 4.29 Elastic Design of Beam Splice Connection

Beam Splice Connection	Elastic Design
Frame A-A-1 [Roof]	Use 2 of A325 Bolts 30 mm
Frame A-A-1 [Floors]	Use 2 of A325 Bolts 30 mm
Frame A-A-2 [Roof]	Use 2 of A325 Bolts 30 mm
Frame A-A-2 [Floors]	Use 2 of A325 Bolts 30 mm
Frame B-B-1 [Roof]	Use 2 of A325 Bolts 30 mm
Frame B-B-1 [Floors]	Use 2 of A325 Bolts 30 mm
Frame B-B-2 [Roof]	Use 2 of A325 Bolts 30 mm
Frame B-B-2 [Floors]	Use 2 of A325 Bolts 30 mm

Table 4.30 Elastic Design of Beam Column Connection

Beam Column Connection	Elastic Design
Frame A-A-1 [Roof]	Use L51x51x3.2 Angle Use 6 of A325 Bolts 30 mm for Flange Use 2 of A325 Bolts 30 mm for Web
Frame A-A-1 [Floors]	Use L25x25x3.2 Angle Use 6 of A325 Bolts 30 mm for Flange Use 2 of A325 Bolts 30 mm for Web
Frame A-A-2 [Roof]	Use L19x19x3.2 Angle Use 2 of A325 Bolts 30 mm for Flange Use 2 of A325 Bolts 30 mm for Web
Frame A-A-2 [Floors]	Use L51x51x3.2 Angle Use 6 of A325 Bolts 30 mm for Flange Use 6 of A325 Bolts 30 mm for Web
Frame B-B-1 [Roof]	Use L64x64x4.8 Angle Use 6 of A325 Bolts 30 mm for Flange Use 6 of A325 Bolts 30 mm for Web
Frame B-B-1 [Floors]	Use L32x32x3.2 Angle Use 6 of A325 Bolts 30 mm for Flange Use 6 of A325 Bolts 30 mm for Web
Frame B-B-2 [Roof]	Use L19x19x3.2 Angle Use 6 of A325 Bolts 30 mm for Flange Use 6 of A325 Bolts 30 mm for Web
Frame B-B-2 [Floors]	Use L32x32x3.2 Angle Use 6 of A325 Bolts 30 mm for Flange Use 6 of A325 Bolts 30 mm for Web

Table 4.31 Elastic Design of Base plate:

Column	Elastic Design
Column C1	Use Base Plate 160*190*5
Column C2	Use Base Plate 190*280*6
Column C3	Use Base Plate 210*270*20
Column C4	Use Base Plate 300*270*30

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

For elastic analysis, the deference of the moments, shear forces and deflections for floor and roof beams were about (-2.86 – 2.89), (0.07 – 1.34) and (-18.89 – 24.61) respectively. The deference of the button moments, top moments and axial forces for columns were about (-7.91 – 2.91), (-22.79 – 4.83) and (0.42 – 1.27) respectively. For plastic analysis the deference of the moments, shear forces, deflections and axial forces were about (-5.73 – 4.94), (-11.06 – 4.56), (-15.10 – 15.04) and (-15.41 – 0.57) respectively. The deference of the button moments, top moments and axial forces for columns were about (-20.15 – 16.25), (-14.03 – 28.14) and (-6.35 – 2.56) respectively. The elastic analysis gives the large results than plastic one. The Critical Load Factors were calculated using different Programs (MATLAB Code, MASTAN2, and SAP 2000), and the deference of the results that obtained by this programs was about 0.0 - 10.945%. The differences of first and second order analysis were small. The sections obtained by plastic design are smaller than sections in elastic one.

5.2 Recommendations and Suggestions for Future Work

Recommendations and suggestions for future Work were summarized as follows:

1. The use of Second order design of steel frames.
2. The use of advanced method of analysis of frames such as refined method and Plastic Zone Method witch give results effective solution and economy for steel frames.
3. Develop MATLAB code to draw the deformed shapes and internal forces diagram of steel frames.
4. Applying the advanced methods of analysis for 3D steel frames.

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Appendix A

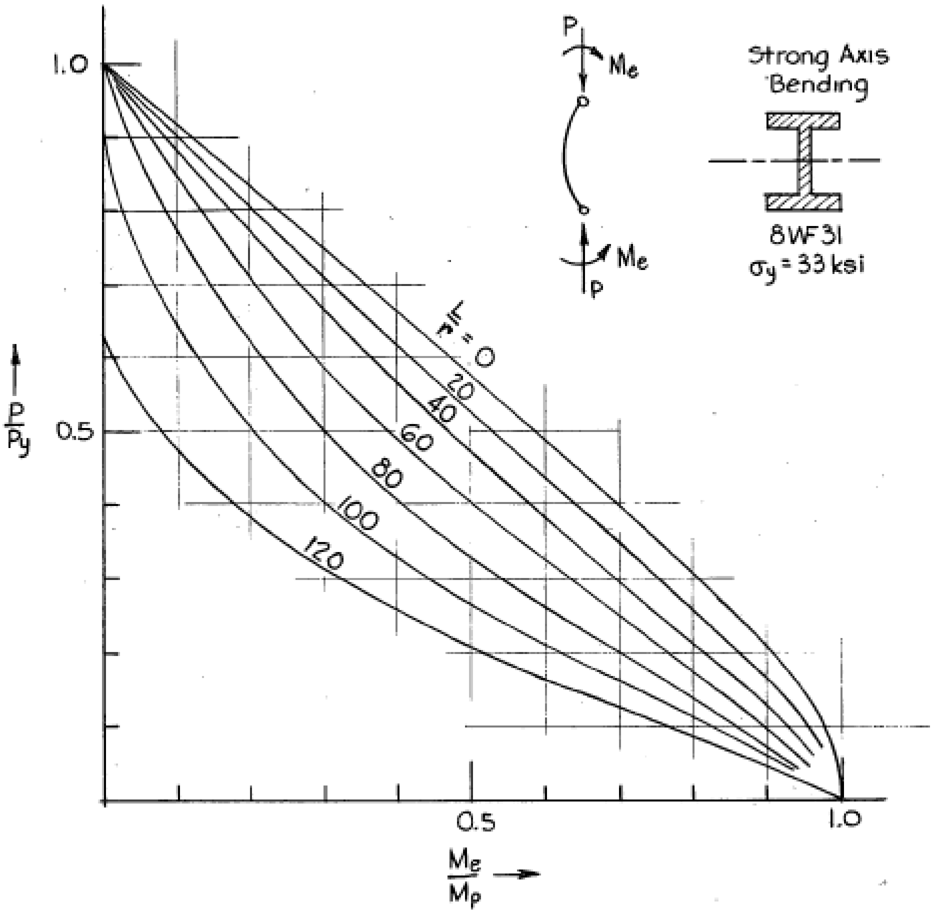


Figure A.1: Chart of Calculate M/Mp.

Appendix B
Elastic and Plastic Design of 2D Steel Frames

Table B.1. Elastic Design of roof Beam A-A-1

Maximum Bending Moment			47.42		kN m
Maximum Shear Force			66.30		kN m
Maximum Deflection			3.36		mm
Fb			165.00		N/mm2
Fv			100.00		N/mm2
Try Section					
Elastic Modulus [Sx]			287393.94		mm^3
Try Section			W8x24		Ok
d	201.42	mm	tw	6.22	mm
bf	165.10	mm	tf	10.16	mm
Ix	34422303.70	mm^4	Sx	342490.39	mm^3
SHEAR CHECK		Q	185925.66		mm^3
Fvmax		81.20	N/mm2		Section Is ok
Deflection Check					
Roof Beams		11.81	mm		Ok

Table B.2. Elastic Design of floors Beam A-A-1

Maximum Bending Moment			71.21		kN m
Maximum Shear Force			99.24		kN m
Maximum Deflection			4.85		mm
Fb		165.00			N/mm2
Fv		100.00			N/mm2
Try Section					
Elastic Modulus [Sx]			431575.76		mm^3
Try Section			W8x35		Ok
d	206.25	mm	tw	7.87	mm
bf	203.71	mm	tf	12.57	mm
Ix	52861337.00	mm^4	Sx	342490.39	mm^3
SHEAR CHECK		Q	280303.57		mm^3
Fvmax		66.83	N/mm2		Section Is ok
Deflection Check					
Roof Beams		11.81	mm		Ok

Table B.3. Elastic Design of roof Beam A-A-2

Maximum Bending Moment			94.32		kN m
Maximum Shear Force			131.87		kN m
Maximum Deflection			8.17		Mm
Fb			165.00		N/mm2
Fv			100.00		N/mm2
Try Section					
Elastic Modulus [Sx]			571636.36		mm^3
Try Section			W8x48		Ok
d	215.90	mm	tw	10.16	Mm
bf	205.99	mm	tf	17.40	Mm
Ix	76586504.00	mm^4	Sx	707922.72	mm^3
SHEAR CHECK		Q	397376.06		mm^3
Fvmax		67.16	N/mm2		Section Is ok
Deflection Check					
Roof Beams		11.81	mm		Ok

Table B.4. Elastic Design of floors Beam A-A-2

Maximum Bending Moment			140.66		kN m
Maximum Shear Force			197.75		kN m
Maximum Deflection			9.66		Mm
Fb			165.00		N/mm2
Fv			100.00		N/mm2
Try Section					
Elastic Modulus [Sx]			852484.85		mm^3
Try Section			W8x67		Ok
d	228.60	mm	tw	14.48	Mm
bf	210.31	mm	tf	23.75	Mm
Ix	113214832.00	mm^4	Sx	989780.84	mm^3
SHEAR CHECK		Q	570940.67		mm^3
Fvmax		68.88	N/mm2		Section Is ok
Deflection Check					
Roof Beams		11.81	mm		Ok

Table B.5. Elastic Design of roof Beam B-B-1

Maximum Bending Moment			39.60		kN m
Maximum Shear Force			58.91		kN m
Maximum Deflection			3.95		Mm
Fb			165.00		N/mm2
Fv			100.00		N/mm2
Try Section					
Elastic Modulus [Sx]			240000.00		mm^3
Try Section			W8x21		Ok
d	210.31	mm	tw	6.35	Mm
bf	133.86	mm	tf	10.16	Mm
Ix	31342194.30	mm^4	Sx	298245.22	mm^3
SHEAR CHECK		Q	164755.05		mm^3
Fvmax		48.77	N/mm2		Section Is ok
Deflection Check					
Roof Beams		11.11	mm		Ok

Table B.6. Elastic Design of floors Beam B-B-1

Maximum Bending Moment			60.30		kN m
Maximum Shear Force			89.36		kN m
Maximum Deflection			5.49		Mm
Fb			165.00		N/mm2
Fv			100.00		N/mm2
Try Section					
Elastic Modulus [Sx]			365454.55		mm^3
Try Section			W8x31		Ok
d	203.20	mm	tw	7.24	Mm
bf	203.20	mm	tf	11.05	Mm
Ix	45785410.00	mm^4	Sx	450645.25	mm^3
SHEAR CHECK		Q		245382.59	mm^3
Fvmax		66.16		N/mm2	Section Is ok
Deflection Check					
Roof Beams		11.11		mm	Ok

Table B.7. Elastic Design of roof Beam B-B-2

Maximum Bending Moment			78.85		kN m
Maximum Shear Force			117.37		kN m
Maximum Deflection			6.04		Mm
Fb			165.00		N/mm2
Fv			100.00		N/mm2
Try Section					
Elastic Modulus [Sx]			477878.79		mm^3
Try Section			W8x40		Ok
d	209.55	mm	tw	9.14	Mm
bf	204.98	mm	tf	14.22	Mm
Ix	60769726.00	mm^4	Sx	581742.05	mm^3
SHEAR CHECK		Q	322234.97		mm^3
Fvmax		68.06	N/mm2		Section Is ok
Deflection Check					
Roof Beams		11.11	mm		Ok

Table B.8. Elastic Design of floors Beam B-B-2

Maximum Bending Moment			117.85		kN m
Maximum Shear Force			176.13		kN m
Maximum Deflection			8.11		Mm
Fb			165.00		N/mm2
Fv			100.00		N/mm2
Try Section					
Elastic Modulus [Sx]			714242.42		mm^3
Try Section			W8x58		Ok
d	222.25	mm	tw	12.95	Mm
bf	208.79	mm	tf	20.57	Mm
Ix	94900668.00	mm^4	Sx	852129.20	mm^3
SHEAR CHECK		Q	486268.20		mm^3
Fvmax		39.56	N/mm2		Section Is ok
Deflection Check					
Roof Beams		11.11	mm		Ok

Table B.9. Elastic Design of Ground Floor Column C1

Mb	0.00				kN m
Mt	39.08				kN m
P(Max)	275.51				kN
Fc1	0.45*Fy		112.50		N/mm^2
Areq	P(Max)\Fc1		2448.98		mm^2
Choose Section:	W8x31				Ok
A	5883.86				mm2
Rx	88.14				Mm
Ry	51.31				Mm
Sx	450645.25				mm^3
Sy	152028.00				mm^3
Cc	3971.30	Le/R(min)	49.70		
Le/Rmin/Cc	0.01	Fc2	149.69		N.mm^2
Q	880.75	Fc3	131.09	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Fa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	46.82			N/mm^2
fbx	Mx/Sx	86.72			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.84	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				299.29
	0.84				OK

Table B.10. Elastic Design of Ground Floor Column C2

Mb	0.00				kN m
Mt	65.62				kN m
P(Max)	533.75				kN
Fc1	0.45*Fy		112.50		N/mm^2
Areq	P(Max)\Fc1		4744.44		mm^2
Choose Section:	W10x45				Ok
A	8580.63				mm2
Rx	109.73				Mm
Ry	51.05				Mm
Sx	804606.61				mm^3
Sy	218120.00				mm^3
Cc	3971.30	Le/R(min)	49.95		
Le/Rmin/Cc	0.01	Fc2	149.69		N.mm^2
Q	1284.42	Fc3	131.09	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Fa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	62.20			N/mm^2
fbx	Mx/Sx	81.56			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.91	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				296.34
	0.95				OK

Table B.11. Elastic Design of Ground Floor Column C3

Mb	0.00				kN m
Mt	56.02				kN m
P(Max)	542.17				kN
Fc1	0.45*Fy		112.5		N/mm^2
Areq	P(Max)\Fc1		497.96		mm^2
Choose Section:	W8x28				Ok
A	5316.1184				mm2
Rx	87.63				mm
Ry	41.148				mm
Sx	398206.53				mm^3
Sy	108732				mm^3
Cc	3971.30	Le/R(min)	61.97142		
Le/Rmin/Cc	0.02	Fc2	149.68		N.mm^2
The Allowable Load	795.73	Fc3	131.09	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Ffa	0.6*Fy	150			N/mm^2
fa	Pc/A	10.54			N/mm^2
fbx	Mx/Sx	140.68			N/mm^2
Fbx	0.66*Fy	165			N/mm^2
	0.92	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				192.4973
	0.84				OK

Table B.12. Elastic Design of Ground Floor Column C4

Mb	0.00				kN m
Mt	27.62				kN m
P(Max)	1064.88				kN
Fc1	0.45*Fy		112.50		N/mm^2
Areq	P(Max)\Fc1		9465.60		mm^2
Choose Section:	W10x54				Ok
A	10193.53				mm2
Rx	111.00				mm
Ry	65.02				mm
Sx	983226.00				mm^3
Sy	337840.00				mm^3
Cc	3971.30	Le/R(min)	39.22		
Le/Rmin/Cc	0.01	Fc2	149.69	N.mm^2	
Q	1525.90	Fc3	131.10	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Ffa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	104.47			N/mm^2
fbx	Mx/Sx	28.09			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.87	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				480.70
	0.70				OK

Table B.13. Elastic Design of First Floor Column C1

Mb	38.78				kN m
Mt	39.89				kN m
P(Max)	172.47				kN
Fc1	0.45*Fy	112.50			N/mm^2
Areq	P(Max)\Fc1	1533.07			mm^2
Choose Section:	W10x112				Ok
A	21225.76				mm2
Rx	118.36				mm
Ry	68.07				mm
Sx	2064774.60				mm^3
Sy	742920.00				mm^3
Cc	3971.30	Le/R(min)	30.85		
Le/Rmin/Cc	0.01	Fc2	149.70		N.mm^2
Q	3177.41	Fc3	131.10	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Ffa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	8.13			N/mm^2
fbx	Mx/Sx	19.32			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.17	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				776.79
	0.15				OK

Table B.14. Elastic Design of First Floor Column C2

Mb	75.62				kN m
Mt	70.47				kN m
P(Max)	333.54				kN
Fc1	0.45*Fy		112.50		N/mm^2
Areq	P(Max)\Fc1		2964.80		mm^2
Choose Section:	W12x106				Ok
A	20128.99				mm2
Rx	138.94				mm
Ry	78.99				mm
Sx	2376129.50				mm^3
Sy	808520.00				mm^3
Cc	3971.30	Le/R(min)			26.58
Le/Rmin/Cc	0.01	Fc2	149.70		N.mm^2
Q	3013.25	Fc3	131.10	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Ffa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	16.57			N/mm^2
fbx	Mx/Sx	140.37			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.96	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				1046.06
	0.11				OK

Table B.15. Elastic Design of First Floor Column C3

Mb	64.44				kN m
Mt	59.79				kN m
P(Max)	339.17				kN
Fc1	0.45*Fy		112.50		N/mm^2
Areq	P(Max)\Fc1		3014.84		mm^2
Choose Section:	W10x39				Ok
A	7419.34				mm2
Rx	108.46				mm
Ry	50.29				mm
Sx	689896.91				mm^3
Sy	185320.00				mm^3
Cc	3971.30	Le/R(min)	41.76		
Le/Rmin/Cc	0.01	Fc2	149.69		N.mm^2
Q	1110.62	Fc3	131.10	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Ffa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	45.71			N/mm^2
fbx	Mx/Sx	93.41			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.87	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				424.00
	0.84				OK

Table B.16. Elastic Design of First Floor Column C4

Mb	26.50				kN m
Mt	25.70				kN m
P(Max)	665.38				kN
Fc1	0.45*Fy		112.50		N/mm^2
Areq	P(Max)\Fc1		5914.49		mm^2
Choose Section:	W10x39				Ok
A	7419.34				mm2
Rx	108.46				mm
Ry	50.29				mm
Sx	689896.91				mm^3
Sy	185320.00				mm^3
Cc	3971.30	Le/R(min)	41.76		
Le/Rmin/Cc	0.01	Fc2	149.69		N.mm^2
Q	1110.62	Fc3	131.10	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Fa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	89.68			N/mm^2
fbx	Mx/Sx	38.41			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.99	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				424.00
	0.85				OK

Table B.17. Elastic Design of Second Floor Column C1

Mb	34.03				kN m
Mt	43.14				kN m
P(Max)	69.42				kN
Fc1	0.45*Fy		112.5		N/mm^2
Areq	P(Max)\Fc1		617.07		mm^2
Choose Section:	W8x21				Ok
A	3974.1856				mm2
Rx	88.646				mm
Ry	32.004				mm
Sx	298245.22				mm^3
Sy	60844				mm^3
Cc	3971.30	Le/R(min)	65.6168		
Le/Rmin/Cc	0.02	Fc2	149.68		N.mm^2
Q	594.86	Fc3	131.09	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Fa	0.6*Fy	150			N/mm^2
fa	Pc/A	17.46773			N/mm^2
fbx	Mx/Sx	144.6461			N/mm^2
Fbx	0.66*Fy	165			N/mm^2
	0.99	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				171.70
	0.12				OK

Table B.18. Elastic Design of Second Floor Column C2

Mb	66.59			kN m
Mt	85.90			kN m
P(Max)	133.10			kN
Fc1	0.45*Fy		112.5	N/mm^2
Areq	P(Max)\Fc1		1183.11	mm^2
Choose Section:	W10*30			Ok
A	5703.21			mm2
Rx	111.25			mm
Ry	34.80			mm
Sx	530942.04			mm^3
Sy	94300.00			mm^3
Cc	3971.30	Le/R(min)	60.35	
Le/Rmin/Cc	0.02	Fc2	149.68	N.mm^2
Q	853.68	Fc3	131.09 N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1			
Fa	0.6*Fy	150.00		N/mm^2
fa	Pc/A	23.34		N/mm^2
fbx	Mx/Sx	125.42		N/mm^2
Fbx	0.66*Fy	165.00		N/mm^2
	0.99	OK		
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1			
Cmx				0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}			202.99
	0.89			OK

Table B.19. Elastic Design of Second Floor Column C3

Mb	55.97				kN m
Mt	71.96				kN m
P(Max)	136.61				kN
Fc1	0.45*Fy		112.5		N/mm^2
Areq	P(Max)\Fc1		1214.31		mm^2
Choose Section:	W8*31				Ok
A	5883.86				mm2
Rx	88.14				mm
Ry	51.31				mm
Sx	450645.25				mm^3
Sy	152028.00				mm^3
Cc	3971.30	Le/R(min)	40.93		
Le/Rmin/Cc	0.01	Fc2	149.69		N.mm^2
Q	880.77	Fc3	131.10	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Fa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	23.22			N/mm^2
fbx	Mx/Sx	124.20			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.99	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				441.31
	0.83				OK

Table B.20. Elastic Design of Second Floor Column C4

Mb	21.20				kN m
Mt	21.85				kN m
P(Max)	266.75				kN
Fc1	0.45*Fy		112.5		N/mm^2
Areq	P(Max)\Fc1		2371.11		mm^2
Choose Section:	W6*25				Ok
A	4748.38		mm2		
Rx	68.58		Mm		
Ry	38.61		Mm		
Sx	275303.28		mm^3		
Sy	92004.00		mm^3		
Cc	3971.30	Le/R(min)	54.39		
Le/Rmin/Cc	0.01	Fc2	149.69	N.mm^2	
Q	710.77	Fc3	131.09	N.mm^2	Ok
Check 1	fa/Fa + fbx / Fbx ≤ 1				
Fa	0.6*Fy	150.00			N/mm^2
fa	Pc/A	56.18			N/mm^2
fbx	Mx/Sx	77.01			N/mm^2
Fbx	0.66*Fy	165.00			N/mm^2
	0.99	OK			
Check 2	fa/Fa+[Cmx*(fbx/Fbx)]/[1-fa/Fe`x] <= 1				
Cmx					0.85
Fe`x	(12/32)*[^2*E]/[KL/rx}				249.88
	0.89				OK

Table B.21. Elastic Design of Splice column C1 connection at 6 m

Top	W8x21	Bottom	W10x112
Plate Thickness	20.00	Mm	
Pc	172.47	kN	
A 325 bolts	30.00	Mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
Nv	Pc/Rv	0.84	bolts
Rb	Ab*Fp	292.32	kN
Nv	Pc/Rb	0.59	bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.22. Elastic Design of Splice column C2 connection at 6 m

Top	W10x30	Bottom	W12x106
Plate Thickness	20.00	Mm	
P	333.54	kN	
A 325 bolts	30.00	Mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
Nv	Pc/Rv	1.63	bolts
Rb	Ab*Fp	292.32	kN
Nv	Pc/Rb	1.14	bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.23. Elastic Design of Splice column C3 connection at 6 m

Top	W8x31	Bottom	W10x39
Plate Thickness	20.00	Mm	
Pc	339.17	kN	
A 325 bolts	30.00	Mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
Nv	Pc/Rv	1.66	bolts
Rb	Ab*Fp	292.32	kN
Nv	Pc/Rb	1.16	kN
Use 2 of A325 Bolts 30 mm Dia			

Table B.24. Elastic Design of Splice column C4 connection at 6 m

Top	W6x25	W10x33	
Bottom	W10x39		
Plate Thickness	20.00	Mm	
P	665.38	kN	
A 325 bolts	30.00	Mm	
Fu	406.00	N\mm^2	
Fv	144.795	N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
Nv	Pc/Rv	3.25	bolts
Rb	Ab*Fp	292.32	kN
Nv	Pc/Rb	2.27620416	kN
Use 4 of A325 Bolts 30 mm Dia			

Table B.25. Elastic Design of Splice Beam A-A-1connection [Roof]

Roof Beams Frame A-A-1			
Beam	W8x24		
A 325 bolts	30.00	mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Vmax	66.30	kN	
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
n	Vmax/Rv	0.65	Bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.26. Elastic Design of Splice Beam A-A-1connection [Floors]

Floors Beams A-A-1			
Beam	W8x35		
A 325 bolts	30.00	mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Vmax	99.24	kN	
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
n	Vmax/Rv	0.97	Bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.27. Elastic Design of Splice Beam A-A-2connection [Roof]

Roof Beams Frame A-A-2			
Beam	W8x48		
A 325 bolts	30.00	mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Vmax	131.87	kN	
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
n	Vmax/Rv	1.29	Bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.28. Elastic Design of Splice Beam A-A-2connection [Floors]

Floor Beams A-A-2			
Beam	W8x67		
A 325 bolts	30.00	mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Vmax	197.75	kN	
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
n	Vmax/Rv	1.93	Bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.29. Elastic Design of Splice Beam B-B-1connection [Roof]

Roof Beams Frame B-B-1			
Beam	W8x21		
A 325 bolts	30.00	mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Vmax	58.91	kN	
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
n	Vmax/Rv	0.58	Bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.30. Elastic Design of Splice Beam B-B-1connection [Floors]

Floor Beams B-B-1			
Beam	W8x31		
A 325 bolts	30.00	mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Vmax	89.36	kN	
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
n	Vmax/Rv	0.87	Bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.31. Elastic Design of Splice Beam B-B-2connection [Roof]

Roof Beams Frame B-B-2			
Beam	W8x40		
A 325 bolts	30.00	mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Vmax	117.37	kN	
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
n	Vmax/Rv	1.15	Bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.32. Elastic Design of Splice Beam B-B-2connection [Floors]

Floor Beams B-B-2			
Beam	W8x58		
A 325 bolts	30.00	mm	
Fu	406.00	N\mm^2	
Fv	144.80	N\mm^2	
Vmax	176.13	kN	
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
n	Vmax/Rv	1.72	Bolts
Use 2 of A325 Bolts 30 mm Dia			

Table B.33. Elastic Design of Beam Column Connection [Roof Beam A-A-1]

Beam	W8x24			
W8x35				
D	201.42		mm	
tw	6.22		mm	
Bf	165.10		mm	
tf	10.16		mm	
Fu	406.00		N/mm^2	
Fy	250.00		N/mm^2	
A 325 bolts	30.00		mm	
Fv	144.795		N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2	
Ft	303.38		N\mm^2	
Mmax	47.42		kN.m	
Vmax	66.30		kN	
Rv	Av*Fv	102.30	kN	
Rv	2*Av*Fv	204.60	kN	
Rb	Ab*Fp	90.96	kN	
Rb	Ab*Fp	148.50	kN	
Rt	Abolt*Ft	214.34	kN	
Depth	Mmax/2*Rt	110.62	mm	
Gage	0.5*[Depth-D]	45.40	mm	
use L51x51x3.2				
T	Mmax/Depth	428.68		kN
Flange	T/Rv	4.19		bolts
say	6	Bolts		
Web	Vmax/Rv	0.65		bolts
say	2	Bolts		
Check1:	web number of bolts*Rv>=Vmax			OK
Check 2:	flange number of bolts *Rv>=T			OK

Table B.34. Elastic Design of Beam Column Connection [Floors Beam A-A-1]

Beam	W8x35			
W8x35				
D	206.25		mm	
tw	7.87		mm	
Bf	203.71		mm	
tf	12.57		mm	
Fu	406.00		N/mm^2	
Fy	250.00		N/mm^2	
A 325 bolts	30.00		mm	
Fv	144.795		N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2	
Ft	303.38		N\mm^2	
Mmax	71.21		kN.m	
Vmax	99.24		kN	
Rv	Av*Fv	102.30	kN	
Rv	2*Av*Fv	204.60	kN	
Rb	Ab*Fp	115.09	kN	
Rb	Ab*Fp	183.77	kN	
Rt	Abolt*Ft	214.34	kN	
Depth	Mmax/2*Rt	166.12	mm	
Gage	0.5*[Depth-D]	20.07	mm	
use L25*25*3.2				
T	Mmax/Depth	428.68		kN
Flange	T/Rv	4.19		bolts
say	6	Bolts		
Web	Vmax/Rv	0.97		bolts
say	2	Bolts		
Check1:	web number of bolts*Rv>=Vmax			OK
Check 2:	flange number of bolts *Rv>=T			OK

Table B.35. Elastic Design of Beam Column Connection [Roof Beam A-A-2]

Beam	W8x48			
W8x48				
D	215.90		mm	
tw	10.16		mm	
Bf	205.99		mm	
tf	17.40		mm	
Fu	406.00		N/mm^2	
Fy	250.00		N/mm^2	
A 325 bolts	30.00		mm	
Fv	144.795		N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2	
Ft	303.38		N\mm^2	
Mmax	94.32		kN.m	
Vmax	131.87		kN	
Rv	Av*Fv	102.30	kN	
Rv	2*Av*Fv	204.60	kN	
Rb	Ab*Fp	30.80	kN	
Rb	Ab*Fp	254.30	kN	
Rt	Abolt*Ft	214.34	kN	
Depth	Mmax/2*Rt	220.03	mm	
Gage	0.5*[Depth-D]	2.06	mm	
useL19x19x3.2				
T	Mmax/Depth	428.68		kN
Flange	T/Rv	2.00		bolts
say	2	bolts		
Web	Vmax/Rv	1.29		bolts
say	2	bolts		
Check1:	web number of bolts*Rv>=Vmax			OK
Check 2:	flange number of bolts *Rv>=T			OK

Table B.36. Elastic Design of Beam Column Connection [Floors Beam A-A-2]

Floors Beam A-A-2				
Beam	W8x67			
W8x67				
D	228.60		mm	
Tw	14.48		mm	
Bf	210.31		mm	
Tf	23.75		mm	
Fu	406.00		N/mm^2	
Fy	250.00		N/mm^2	
A 325 bolts	30.00		mm	
Fv	144.795		N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2	
Ft	303.38		N\mm^2	
Mmax	140.66		kN.m	
Vmax	197.75		kN	
Rv	Av*Fv	102.30	kN	
Rv	2*Av*Fv	204.60	kN	
Rb	Ab*Fp	211.61	kN	
Rb	Ab*Fp	347.12	kN	
Rt	Abolt*Ft	214.34	kN	
Depth	Mmax/2*Rt	328.13	mm	
Gage	0.5*[Depth-D]	49.76	mm	
use L51x51x3.2				
T	Mmax/Depth	428.68		kN
Flange	T/Rv	4.19		bolts
say	6	bolts		
Web	Vmax/Rv	1.93		bolts
say	6	bolts		
Check1:	web number of bolts*Rv>=Vmax			OK
Check 2:	flange number of bolts *Rv>=T			OK

Table B.37. Elastic Design of Beam Column Connection [Roof Beam B-B-1]

Roof Beam B-B-1				
Beam	W8x21			
W8x21				
D	210.31		mm	
Tw	6.35		mm	
Bf	133.86		mm	
Tf	10.16		mm	
Fu	406.00		N/mm^2	
Fy	250.00		N/mm^2	
A 325 bolts	30.00		mm	
Fv	144.795		N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2	
Ft	303.38		N\mm^2	
Mmax	39.60		kN.m	
Vmax	58.91		kN	
Rv	Av*Fv	102.30	kN	
Rv	2*Av*Fv	204.60	kN	
Rb	Ab*Fp	92.81	kN	
Rb	Ab*Fp	148.50	kN	
Rt	Abolt*Ft	214.34	kN	
Depth	Mmax/2*Rt	92.38	mm	
Gage	0.5*[Depth-D]	58.97	mm	
use L64x64x4.8				
T	Mmax/Depth	428.68		kN
Flange	T/Rv	4.19		bolts
say	6	bolts		
Web	Vmax/Rv	0.58		bolts
say	6	bolts		
Check1:	web number of bolts*Rv>=Vmax			OK
Check 2:	flange number of bolts *Rv>=T			OK

Table B.38. Elastic Design of Beam Column Connection [Floors Beam B-B-1]

Beam	W8x31		
W8x31			
D	203.20		mm
Tw	7.24		mm
Bf	203.20		mm
Tf	11.05		mm
Fu	406.00		N/mm^2
Fy	250.00		N/mm^2
A 325 bolts	30.00		mm
Fv	144.795		N\mm^2
Fb	1.2*Fu	487.20	N\mm^2
Ft	303.38		N\mm^2
Mmax	60.30		kN.m
Vmax	89.36		kN
Rv	Av*Fv	102.30	kN
Rv	2*Av*Fv	204.60	kN
Rb	Ab*Fp	105.81	kN
Rb	Ab*Fp	161.49	kN
Rt	Abolt*Ft	214.34	kN
Depth	Mmax/2*Rt	140.67	mm
Gage	0.5*[Depth-D]	31.27	mm
use L32x32x3.2			
T	Mmax/Depth	428.68	kN
Flange	T/Rv	4.19	bolts
say	6	bolts	
Web	Vmax/Rv	0.87	bolts
say	6	bolts	
Check1:	web number of bolts*Rv>=Vmax		OK
Check 2:	flange number of bolts *Rv>=T		OK

Table B.39. Elastic Design of Beam Column Connection [Roof Beam B-B-2]

Beam	W8x40			
W8x40				
D	209.55		mm	
Tw	9.14		mm	
Bf	204.98		mm	
Tf	14.22		mm	
Fu	406.00		N/mm^2	
Fy	250.00		N/mm^2	
A 325 bolts	30.00		mm	
Fv	144.795		N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2	
Ft	303.38		N\mm^2	
Mmax	78.85		kN.m	
Vmax	117.37		kN	
Rv	Av*Fv	102.30	kN	
Rv	2*Av*Fv	204.60	kN	
Rb	Ab*Fp	133.65	kN	
Rb	Ab*Fp	207.90	kN	
Rt	Abolt*Ft	214.34	kN	
Depth	Mmax/2*Rt	183.94	mm	
Gage	0.5*[Depth-D]	12.81	mm	
useL19x19x3.2				
T	Mmax/Depth	428.68		kN
Flange	T/Rv	4.19		bolts
say	6	bolts		
Web	Vmax/Rv	1.15		bolts
say	6	bolts		
Check1:	web number of bolts*Rv>=Vmax			OK
Check 2:	flange number of bolts *Rv>=T			OK

Table B.40. Elastic Design of Beam Column Connection [Floors Beam B-B-2]

Beam	W8x58			
W8x58				
D	222.25		mm	
Tw	12.95		mm	
Bf	208.79		mm	
Tf	20.57		mm	
Fu	406.00		N/mm^2	
Fy	250.00		N/mm^2	
A 325 bolts	30.00		mm	
Fv	144.795		N\mm^2	
Fb	1.2*Fu	487.20	N\mm^2	
Ft	303.38		N\mm^2	
Mmax	117.85		kN.m	
Vmax	176.13		kN	
Rv	Av*Fv	102.30	kN	
Rv	2*Av*Fv	204.60	kN	
Rb	Ab*Fp	189.34	kN	
Rb	Ab*Fp	3051.65	kN	
Rt	Abolt*Ft	214.34	kN	
Depth	Mmax/2*Rt	274.92	mm	
Gage	0.5*[Depth-D]	26.33	mm	
use L32x32x3.2				
T	Mmax/Depth	428.68		kN
Flange	T/Rv	4.19		bolts
say	6	bolts		
Web	Vmax/Rv	1.72		bolts
say	6	bolts		
Check1:	web number of bolts*Rv>=Vmax			OK
Check 2:	flange number of bolts *Rv>=T			OK

Table B.41. Elastic Design of Base Plate [Column C1]

Column C1		W8x31	
Column:	W8x31		
d	203.2	mm	
bf	203.2	mm	
Fy	250	N/mm^2	
Pc	275.51	kN	
fc`	28	N/mm^2	
fb	0.35*fc`	9.80	N\mm^2
A	Pc/fb	28113.27	Mm^2
delta	0.5*(0.95D-0.80bf)	15.24	
C	(A)^0.5+delta	182.91	mm
B	A/C	153.70	mm
n	0.5(B-0.8bf)	4.43	mm
m	0.5(C-0.95D)	5.06	mm
Z	Pc/(B*C)	9.80	N\mm^2
Mmax	(Z*m2)/2	125.70	N mm
t	((6*Mmax)/(0.75*Fy))^0.5	2.01	mm
say	t	5	mm

Use Base Plate 160*190*5

Table B.42. Elastic Design of Base Plate [Column C2]

Column C2		W10x45	
Column:	W10x45		
d	256.54	mm	
bf	203.708	mm	
Fy	250	N/mm^2	
Pc	533.75	kN	
fc`	28	N/mm^2	
fb	0.35*fc`	9.80	N\mm^2
A	Pc/fb	54464.29	Mm^2
delta	0.5*(0.95D-0.80bf)	40.37	
C	(A)^0.5+delta	273.75	mm
B	A/C	198.96	mm
n	0.5(B-0.8bf)	18.00	mm
m	0.5(C-0.95D)	15.02	mm
Z	Pc/(B*C)	9.80	N\mm^2
Mmax	(Z*m2)/2	1105.16	N mm
t	((6*Mmax)/(0.75*Fy))^0.5	5.95	mm
say	t=	6	mm

Use Base Plate 190*280*6

Table B.43. Elastic Design of Base Plate [Column C3]

Column C3		W8x28	
Column:	W8x28		
d	204.724	mm	
bf	166.116	mm	
Fy	250	N/mm^2	
Pc	542.17	kN	
fc`	28	N/mm^2	
fb	0.35*fc`	9.80	N\mm^2
A	Pc/fb	55323.47	Mm^2
delta	0.5*(0.95D-0.80bf)	30.80	
C	(A)^0.5+delta	266.01	mm
B	A/C	207.98	mm
n	0.5(B-0.8bf)	37.54	mm
m	0.5(C-0.95D)	35.76	mm
Z	Pc/(B*C)	9.80	N\mm^2
Mmax	(Z*m2)/2	6265.86	N mm
t	((6*Mmax)/(0.75*Fy))^0.5	14.16	mm
say	t	15	mm

Use Base Plate 210*270*20

Table B.44. Elastic Design of Base Plate [Column C4]

Column C4		W10x54	
Column:	W10x45		
d	256.54	mm	
bf	203.708	mm	
Fy	250	N/mm^2	
Pc	1064.88	kN	
fc`	28	N/mm^2	
fb	0.35*fc`	9.80	N\mm^2
A	Pc/fb	108661.22	Mm^2
delta	0.5*(0.95D-0.80bf)	40.37	
C	(A)^0.5+delta	370.01	mm
B	A/C	293.67	mm
n	0.5(B-0.8bf)	65.35	mm
m	0.5(C-0.95D)	63.15	mm
Z	Pc/(B*C)	9.80	N\mm^2
Mmax	(Z*m2)/2	19540.30	N mm
t	((6*Mmax)/(0.75*Fy))^0.5	25.01	mm
say	t	26	mm

Use Base Plate 300*270*30

Table B.45. Plastic Design of Roof Beam A-A-1

Mmax		56.62	kN.m			
Vmax		53.35	kN			
Pmax		25.28	kN			
Max Deflec		10.08	mm			
Zt	M/Fy	226480.00	mm^3			
Try		W6x20				
Zt	246000.00	mm^3	Asec	3799.99	mm^2	
Bf	152.91	mm	d	157.48	mm	
T	6.60	mm	tf	9.27	mm	
P/Py	0.03	Zreq	198534.76		OK	
moment capacity of section Check					MpS >MpR	
Mpc		61.5	kN m		OK	
Shear Check		Vall	0.55*Fy*(t*d)		143.00	kN
Vall		>	Vmax		OK	
Buckling Check		d/t	23.85	66.11	OK	
bf/2tf		8.25	8.5		OK	

Table B.46. Plastic Design of floors Beam A-A-1

Mmax		82.87	kN.m			
Vmax		78.23	kN			
Pmax		14.59	kN			
Max Deflec		13.96	mm			
Zt	M/Fy	331480.00	mm^3			
Try		W8x21				
Zt	334560.00	mm^3	Asec	3974.19	mm^2	
bf	133.86	mm	d	210.31	mm	
t	6.35	mm	tf	10.16	mm	
P/Py	0.01	Zreq	285957.22		OK	
moment capacity of section Check					MpS >MpR	
Mpc		83.64	kN m		OK	
Shear Check		Vall	0.55*Fy*(t*d)		183.63	kN
Vall		>	Vmax		OK	
Buckling Check		d/t	33.12	67.25	OK	
bf/2tf		6.59	8.5		OK	

Table B.47. Plastic Design of Roof Beam A-A-2

Mmax		55.06	kN.m		
Vmax		51.93	kN		
Pmax		24.75	kN		
Max Deflec		9.91	mm		
Zt	M/Fy	220240.00	mm^3		
Try		W6x20			
Zt	246000.00	mm^3	Asec	3799.99	mm^2
bf	152.91	mm	d	157.48	mm
t	6.60	mm	tf	9.27	mm
P/Py	0.03	Zreq	192211.63		OK
moment capacity of section Check					MpS >MpR
Mpc		61.5	kN m		OK
Shear Check		Vall	0.55*Fy*(t*d)		143.00 kN
Vall		>	Vmax		OK
Buckling Check		d/t	23.85	42.83	OK
bf/2tf		8.25	8.5		OK

Table B.48. Plastic Design of floors Beam A-A-2

Mmax		82.89	kN.m		
Vmax		78.10	kN		
Pmax		17.20	kN		
Max Deflec		19.74	mm		
Zt	M/Fy	331560.00	mm^3		
Try		W8x21			
Zt	334560.00	mm^3	Asec	3974.19	mm^2
bf	133.86	mm	d	210.31	mm
t	6.35	mm	tf	10.16	mm
P/Py	0.0002	Zreq	284425.24		OK
moment capacity of section Check					MpS >MpR
Mpc		83.64	kN m		OK
Shear Check		Vall	0.55*Fy*(t*d)		192.76 kN
Vall		>	Vmax		OK
Buckling Check		d/t	33.12	68.65	OK
bf/2tf		6.59	8.5		OK

Table B.49. Plastic Design of Roof Beam B-B-1

Mmax		55.84	kN.m		
Vmax		56.08	kN		
Pmax		25.04	kN		
Max Deflec		9.26	mm		
Zt	M/Fy	223360.00	mm^3		
Try		W6x20			
Zt	246000.00	mm^3	Asec	3799.99	mm^2
bf	152.91	Mm	d	157.48	mm
t	6.60	Mm	tf	9.27	mm
P/Py	0.03	Zreq	194995.69		OK
moment capacity of section Check					MpS >MpR
Mpc		61.5	kN m		OK
Shear Check		Vall	0.55*Fy*(t*d)		143.00 kN
Vall		>	Vmax		OK
Buckling Check		d/t	23.85	66.11	OK
bf/2tf		8.25	8.5		OK

Table B.50. Plastic Design of Floors Beam B-B-1

Mmax		82.87	kN.m		
Vmax		83.12	kN		
Pmax		14.24	kN		
Max Deflec		11.70	mm		
Zt	M/Fy	331480.00	mm^3		
Try		W8x21			
Zt	334560.00	mm^3	Asec	3974.19	mm^2
bf	133.86	mm	d	210.31	mm
t	6.35	mm	tf	10.16	mm
P/Py	0.01	Zreq	288511.08		OK
moment capacity of section Check					MpS >MpR
Mpc		83.64	kN m		OK
Shear Check		Vall	0.55*Fy*(t*d)		192.76 kN
Vall		>	Vmax		OK
Buckling Check		d/t	33.12	67.29	OK
bf/2tf		6.59	8.5		OK

Table B.51. Plastic Design of Roof Beam B-B-2

Mmax		59.18	kN.m		
Vmax		59.09	kN		
Pmax		25.25	kN		
Max Deflec		9.80	mm		
Zt	M/Fy	236720.00	mm^3		
Try		W6x20			
Zt	246000.00	mm^3	Asec	3799.99	mm^2
bf	152.91	mm	d	157.48	mm
t	6.60	mm	tf	9.27	mm
P/Py	0.03	Zreq	198534.76		OK
moment capacity of section Check					MpS >MpR
Mpc		61.5	kN m		OK
Shear Check		Vall	0.55*Fy*(t*d)	143.00	kN
Vall		>	Vmax		OK
Buckling Check		d/t	23.85	66.11	OK
bf/2tf		8.25	8.5		OK

Table B.52. Plastic Design of Floors Beam B-B-2

Mmax		82.89	kN.m		
Vmax		82.99	kN		
Pmax		16.68	kN		
Max Deflec		17.68	mm		
Zt	M/Fy	331560.00	mm^3		
Try		W8x21			
Zt	334560.00	mm^3	Asec	3974.19	mm^2
bf	133.86	mm	d	210.31	mm
t	6.35	mm	tf	10.16	mm
P/Py	0.03	Zreq	289231.72		OK
moment capacity of section Check					MpS >MpR
Mpc		83.64	kN m		OK
Shear Check		Vall	0.55*Fy*(t*d)	192.76	kN
Vall		>	Vmax		OK
Buckling Check		d/t	33.12	67.05	OK
bf/2tf		6.59	8.5		OK

Table B.53. Plastic Design of Ground Floor Column C1

Mb			0.00	kN m		
Mt			29.75	kN m		
P(Max)			193.91	kN		
Zt		M/Fy	119000.00	mm^3		
Try Section			W6x16			191729.1
Shape			W6x16			Ok
d		159.51	mm^2	bf	102.36	mm
t w		6.60	mm	tf	10.29	mm
Asec		3058.06	mm	Zx	191729.07	mm^3
Zy		55552.27	mm^3	rx	66.04	mm
ry		24.56	mm	d/t	24.15	
P/Py		0.25	L/rx	45.43		
M/Mp		Form Chart	0.74	Zreq	160810.8	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	85.5	Cm		0.85
Fa	150.0	Pcr	779805	P/Pcr		0.0002
Mm		Fy*Zx		47.9		kN m
Pex	4043485.2	kN	0.53			OK
Check 2		P/Py+0.85M/Mp<=1		0.78		OK
Bucling Check		L/rx	45.43	187		OK
d/t			24.15	44.28		OK
bf/2*tf			4.98	8.50		OK

Table B.54. Plastic Design of Ground Floor Column C2

Mb			0.00	kN m		
Mt			27.01	kN m		
P(Max)			418.37	kN		
Zt		M/Fy	108040.0	mm^3		
Try Section			W6x20			191729.1
Shape			W6x20			Ok
d		157.48	mm^2	bf	152.91	mm
t w		6.60	mm	tf	9.27	mm
Asec		3799.99	mm	Zx	245806.50	mm^3
Zy		110121.31	mm^3	rx	67.56	mm
ry		38.10	mm	d/t	23.85	
P/Py		0.44	L/rx	44.40		
M/Mp		Form Chart	0.51	Zreq	211843.14	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	55.118	Cm		0.85
Fa	150.0	Pcr	968998	P/Pcr		0.0004
Mm		Fy*Zx		61.45		kN m
Pex	5259075.32	kN	0.37			OK
Check 2		P/Py+0.85M/Mp<=1		0.81		OK
Bucling Check		L/rx	45.40	142		OK
d/t			23.85	42.83		OK
bf/2*tf			8.25	8.50		OK

Table B.55. Plastic Design of Ground Floor Column C3

Mb			0.00	kN m		
Mt			26.96	kN m		
P(Max)			395.31	kN		
Zt		M/Fy	107840.0	mm^3		
Try Section			W6x20			245806.5
Shape			W6x20			Ok
d		157.48	mm^2	bf	102.36	mm
t w		6.60	mm	tf	10.29	mm
Asec		3799.99	mm	Zx	191729.07	mm^3
Zy		110121.31	mm^3	rx	66.04	mm
ry		38.10	mm	d/t	24.15	
P/Py		0.42	L/rx	44.40		
M/Mp		Form Chart	0.57	Zreq	189192.98	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	18.373	Cm		0.85
Fa	150.0	Pcr	968998	P/Pcr		0.0004
Mm		Fy*Zx		61.45		kN m
Pex	4043485.2	kN	0.37			OK
Check 2		P/Py+0.85M/Mp<=1		0.79		OK
Bucling Check		44.40	44.40	44.40		OK
d/t			23.85	23.85		OK
bf/2*tf			8.25	8.25		OK

Table B.56. Plastic Design of Ground Floor Column C4

Mb			0.00	kN m		
Mt			6.36	kN m		
P(Max)			430.93	kN		
Zt		M/Fy	25440.0	mm^3		
Try Section			W5x16			157807.8
Shape			W5x16			Ok
d		127.25	mm^2	bf	127.00	mm
t w		6.10	mm	tf	9.14	mm
Asec		3038.70	mm	Zx	157807.77	mm^3
Zy		75052.92	mm^3	rx	54.10	mm
ry		32.00	mm	d/t	20.88	
P/Py		0.57	L/rx	55.45		
M/Mp		Form Chart	0.34	Zreq	74823.5	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	65.617	Cm		0.85
Fa	150.0	Pcr	774869	P/Pcr		0.0006
Mm		Fy*Zx		39.45		kN m
Pex	5259075.32	kN	0.14			OK
Check 2		P/Py+0.85M/Mp<=1		0.70		OK
Bucling Check		55.45	55.45	125		OK
d/t			20.88	42.83		OK
bf/2*tf			6.94	8.5		OK

Table B.57. Plastic Design of First Floor Column C1

Mb			37.02	kN m		
Mt			35.16	kN m		
P(Max)			120.16	kN		
Zt		M/Fy	148080.0	mm^3		
Try Section			W5x19			190090.4
Shape			W5*19			Ok
d		130.81	mm^2	bf	127.76	mm
t w		6.86	mm	tf	10.92	mm
Asec		3587.09	mm	Zx	190090.36	mm^3
Zy		90620.66	mm^3	rx	55.12	mm
ry		32.51	mm	d/t	19.07	
P/Py		0.13	L/rx	54.43		
M/Mp		Form Chart	0.87	Zreq	170206.9	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	64.6	Cm		0.85
Fa	150.0	Pcr	914708	P/Pcr		0.0001
Mm		Fy*Zx		47.50		kN m
Pex	4871546.51	kN	0.66			OK
Check 2		P/Py+0.85M/Mp<=1		0.80		OK
Bucling Check		L/rx	54.43	257		OK
d/t			19.07	55.79		OK
bf/2*tf			5.85	8.50		OK

Table B.58. Plastic Design of First Floor Column C2

Mb			40.30	kN m		
Mt			35.11	kN m		
P(Max)			262.21	kN		
Zt		M/Fy	161200.00	mm^3		
Try Section			W6x20			245806.5
Shape			W6x20			Ok
d		157.48	mm^2	bf	152.91	mm
t w		6.60	mm	tf	9.27	mm
Asec		3799.99	mm	Zx	245806.50	mm^3
Zy		110121.31	mm^3	rx	67.56	mm
ry		38.10	mm	d/t	23.85	
P/Py		0.28	L/rx	44.40		
M/Mp		Form Chart	0.68	Zreq	237058.8	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	55.12	Cm		0.85
Fa	150.0	Pcr	968998	P/Pcr		0.0003
Mm		Fy*Zx		61.50		kN m
Pex	775445289.53	kN	0.56			OK
Check 2		P/Py+0.85M/Mp<=1		0.83		OK
Bucling Check		L/rx	44.40	179		OK
d/t			23.85	42.83		OK
bf/2*tf			8.25	8.50		OK

Table B.59. Plastic Design of First Floor Column C3

Mb			40.10	kN m		
Mt			34.98	kN m		
P(Max)			247.78	kN		
Zt		M/Fy	160400.0	mm^3		
Try Section			W6x20			191729.1
Shape			W6x20			Ok
d		157.48	mm^2	bf	152.91	mm
t w		6.60	mm	tf	9.27	mm
Asec		3799.99	mm	Zx	245806.50	mm^3
Zy		110121.31	mm^3	rx	67.56	mm
ry		67.56	mm	d/t	23.85	
P/Py		0.26	L/rx	44.40		
M/Mp		Form Chart	0.73	Zreq	219726	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	31.08	Cm		0.85
Fa	150.0	Pcr	968998	P/Pcr		0.0003
Mm		Fy*Zx		61.45		kN m
Pex	7754452.90	kN	0.82			OK
Check 2		P/Py+0.85M/Mp<=1		0.82		OK
Bucling Check		L/rx	44.40	184		OK
d/t			23.85	43.59		OK
bf/2*tf			8.25	8.5		OK

Table B.60. Plastic Design of First Floor Column C4

Mb			3.20	kN m		
Mt			4.49	kN m		
P(Max)			269.56	kN		
Zt		M/Fy	17960.00	mm^3		
Try Section			W4x13			102911.0
Shape			W4*13			Ok
d		105.66	mm^2	bf	103.12	mm
t w		7.11	mm	tf	8.76	mm
Asec		2470.96	mm	Zx	102910.99	mm^3
Zy		47850.33	mm^3	rx	43.69	mm
ry		43.69	mm	d/t	14.86	
P/Py		0.44	L/rx	68.67		
M/Mp		Form Chart	0.43	Zreq	41767.44	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	48.07	Cm		0.85
Fa	150.0	Pcr	630096	P/Pcr		0.0004
Mm		Fy*Zx		25.7		kN m
Pex	4043485.2	kN	0.15			OK
Check 2		P/Py+0.85M/Mp<=1		0.59		OK
Bucling Check		L/rx	68.67	142		OK
d/t			14.86	42.83		OK
bf/2*tf			5.88	8.50		OK

Table B.61. Plastic Design of Second Floor Column C1

Mb		31.47	kN m		
Mt		35.78	kN m		
P(Max)		46.82	kN		
Zt	143120.0	119000.00	mm^3		
Try Section		W5x16			157807.8
Shape		W5x16			Ok
d	127.25	mm^2	bf	127.00	mm
t w	6.10	mm	tf	9.14	mm
Asec	3038.70	mm	Zx	157807.8	mm^3
Zy	75052.92	mm^3	rx	54.10	mm
ry	32.00	mm	d/t	20.88	
P/Py	0.06	L/rx	55.45		
M/Mp	Form Chart	0.74	Zreq	152255.3	OK
Check 1	P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	65.6	Cm	0.85
Fa	150.0	Pcr	774869	P/Pcr	0.0002
Mm		Fy*Zx		39.45	kN m
Pex	3976057.97	kN	0.77		OK
Check 2	P/Py+0.85M/Mp<=1		0.83		OK
Bucling Check	L/rx	55.45	379		OK
d/t		20.88	62.74		OK
bf/2*tf		6.94	8.50		OK

Table B.62. Plastic Design of Second Floor Column C2

Mb			32.30	kN m		
Mt			36.77	kN m		
P(Max)			107.55	kN		
Zt		M/Fy	147080.0	mm^3		
Try Section			W5x19			190090.4
Shape			W5x19			Ok
d		130.81	mm^2	bf	127.76	mm
t w		6.86	mm	tf	10.92	mm
Asec		3587.09	mm	Zx	190090.36	mm^3
Zy		90620.66	mm^3	rx	55.12	mm
ry		32.51	mm	d/t	127.76	
P/Py		0.12	L/rx	54.43		
M/Mp		Form Chart	0.92	Zreq	159869.6	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	64.6	Cm		0.85
Fa	150.0	Pcr	914708	P/Pcr		0.0001
Mm		Fy*Zx		47.5		kN m
Pex	4871546.51	kN	0.66			OK
Check 2		P/Py+0.85M/Mp<=1		0.78		OK
Bucling Check		L/rx	54.43	272		OK
d/t			19.07	21.68		OK
bf/2*tf			5.85	8.50		OK

Table B.63. Plastic Design of Second Floor Column C3

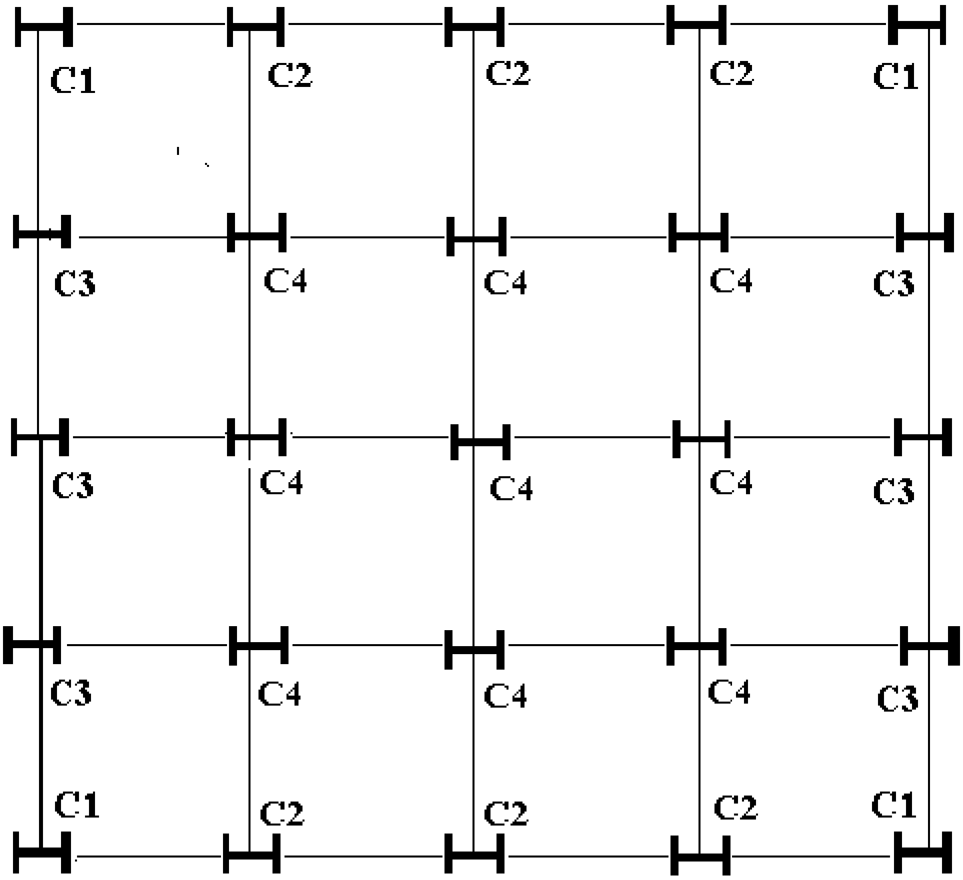
Mb			32.10	kN m		
Mt			36.42	kN m		
P(Max)			101.64	kN		
Zt		M/Fy	145680.0	mm^3		
Try Section			W6x16			191729.1
Shape			W6x16			Ok
d		159.51	mm^2	bf	102.36	mm
t w		6.60	mm	tf	10.29	mm
Asec		3058.06	mm	Zx	191729.07	mm^3
Zy		55552.27	mm^3	rx	66.04	mm
ry		24.56	mm	d/t	24.15	
P/Py		0.1	L/rx	45.43		
M/Mp		Form Chart	0.74	Zreq	160810.8	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	85.5	Cm		0.85
Fa	150.0	Pcr	779805	P/Pcr		0.0001
Mm		Fy*Zx		47.9		kN m
Pex	373800452	kN	0.65			OK
Check 2		P/Py+0.85M/Mp<=1		0.78		OK
Bucling Check		L/rx	45.43	258		OK
d/t			24.15	55.89		OK
bf/2*tf			4.98	8.50		OK

Table B.64. Plastic Design of Second Floor Column C4

Mb			2.4	kN m		
Mt			4.0	kN m		
P(Max)			110.5	kN		
Zt		M/Fy	16080.0	mm^3		
Try Section			W4x13			102911.0
Shape			W4x13			Ok
d		105.7	mm^2	bf	103.1	mm
t w		7.1	mm	tf	8.8	mm
Asec		2471.0	mm	Zx	102911.0	mm^3
Zy		47850.3	mm^3	rx	43.7	mm
ry		25.4	mm	d/t	14.9	
P/Py		0.18	L/rx	68.67		
M/Mp		Form Chart	0.67	Zreq	24000.0	OK
Check 1		P/Pcr+[Cm*M)/(Mm)*(1/(1-P/Pex) <= 1				
Cc	3971.3	KL/r	27.6		Cm	0.85
Fa	150.0	Pcr	630096		P/Pcr	0.0002
Mm		Fy*Zx			25.73	kN m
Pex	2108279.9	kN	0.13			OK
Check 2		P/Py+0.85M/Mp<=1			0.31	OK
Bucling Check		L/rx	68.67		222	OK
d/t			14.86		19.53	OK
bf/2*tf			5.88		8.50	OK

Appendix C

Distribution of Columns and Base Plates



FigureC.1: Distribution of Columns

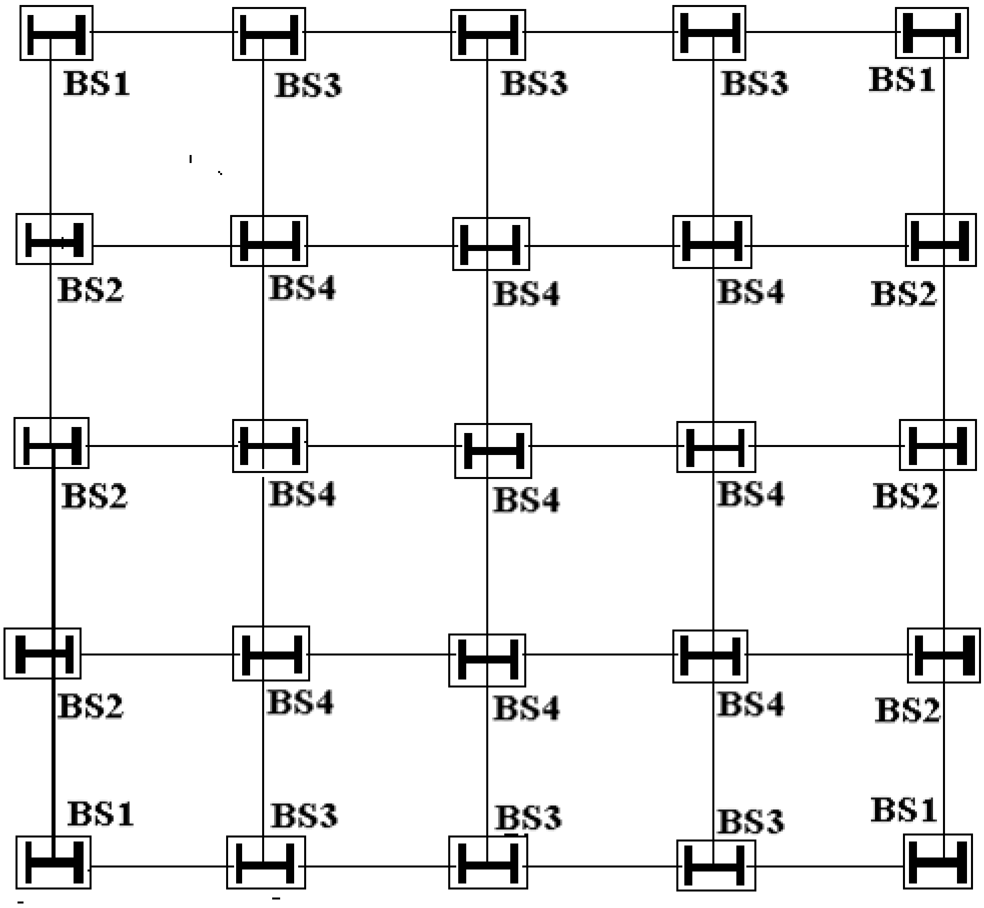
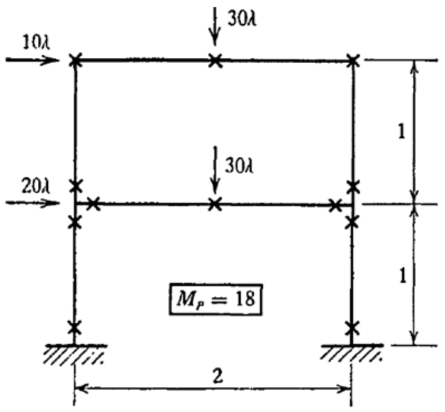


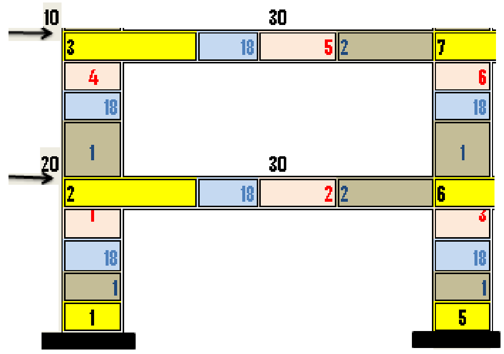
Figure C.2: Distribution of Base Plates

Appendix D

Validation of The MATLAB Code

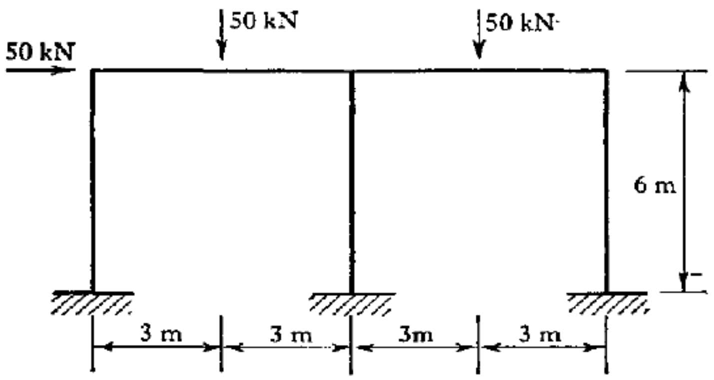


$\lambda = 1{\cdot}80$

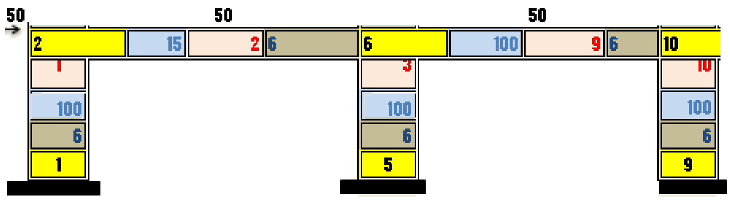


$\lambda = 1{\cdot}80$

Figure D.1: Example 4.4



$\lambda = 1.78$



$\lambda = 1.78$

Figure D.2: Example 4.4

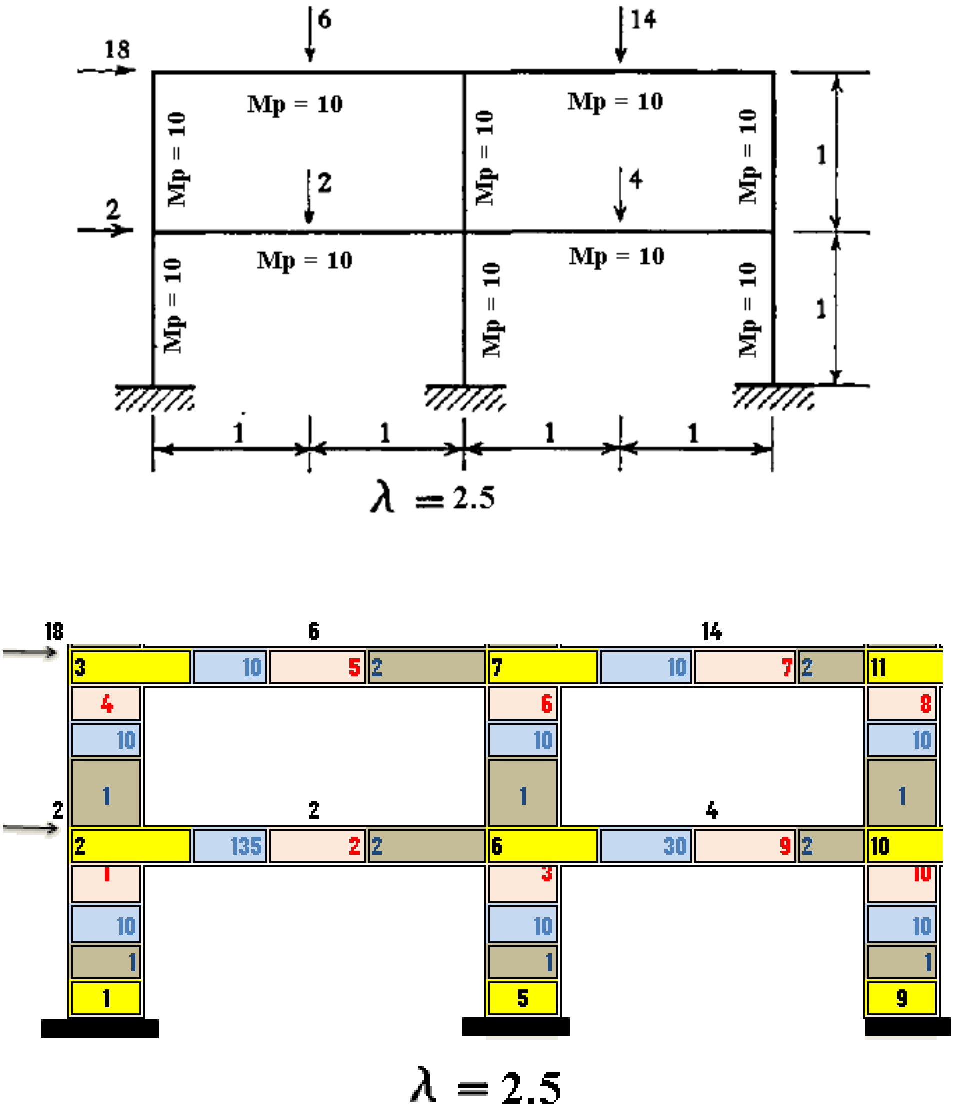
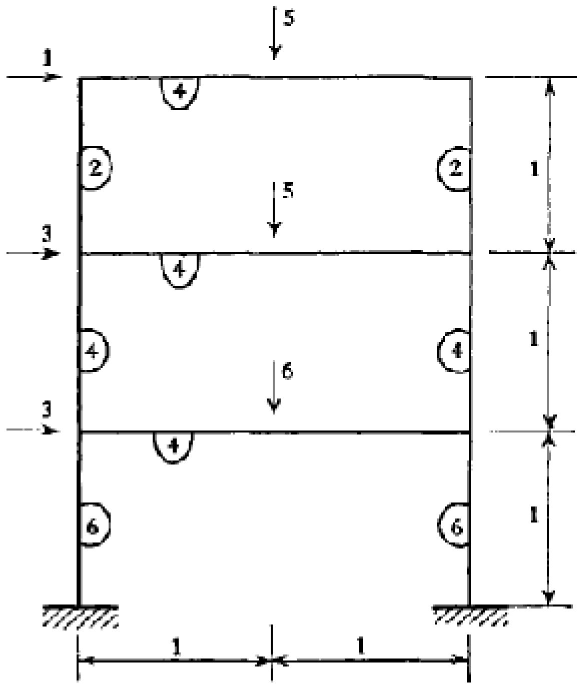
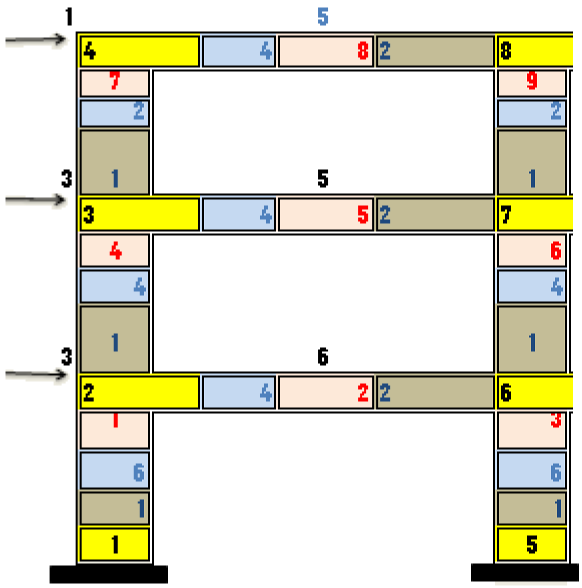


Figure D.3: Example 4.4



$\lambda = 2.0$



$\lambda = 2.0$

Figure D.4: Example 4.4