MAJOR PROJECT REPORT

ON

ANALYSIS OF THE STRENGTH OF STEEL MEMBERS ACCORDING TO IS 800-2007 AND VERIFY WITH ANSYS

Project Guide:-

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(Session 2016-2017)

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CERIFICATE

This is to certify that the project entitled **Analysis strength of steel member according to IS 800-2007 and verify with ANSYS** submitted by **Deepali Nigam, Krishan Dutt Yadav, Pragyanand and Mukesh Dalava** to the Department of Civil Engineering towards partial fulfillment of curriculum requirement for final year is a bonafide record of the work carried out by them under my supervision. They have carried out this project with utmost sincerity and hard work.

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Abstract

Analysis of the strength of steel members according to IS: 800-2007 and verify with ANSYS .When a beam is transversely loaded in such a manner that the resultant force passes through the longitudinal shear center axis, the beam only bends and no torsion will occur, When the resultant acts away from shear center axis then the beam will not only bent but also twist. Lateral torsional buckling may occur in an unrestrained beam. A beam is considered to be unrestrained when its compression flange is free to displace laterally and rotate. The assessment of buckling of steel members and beam-columns requires the computation of the elastic critical load, which strongly depends on both bending moment diagram and end support restriction. The elastic critical load is directly affected by the following factor: material properties, such as modulus of elasticity and shear modulus; geometric property of the cross-section, such as torsion constant, warping constant, and the moment of inertia about the minor axis; properties of the beam, such as length of the beam, lateral bending and warping restrictions at the supports. A structural member which is subjected to compressive forces which tend to decrease its length is called a compression member.

The aim of the thesis is to investigate the buckling behaviour of steel members like beams and columns. The thesis comprises a literature study of the behaviour of beams and columns. The aim of the thesis is to investigate the capacity of the different type of steel members, comparison with the ANSYS software results and get the percentage errors.

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SYMBOLS

h = depth of beam

P = applied force

b = width of the beam

Z_p = plastic section modulus

Z_e = elastic section modulus

Z_{ee} = effective elastic section modulus

M_y = yield moment

M_p =plastic moment

M_d = design bending strength of the section

V_d =design shear strength of the cross-section

β_b = $Z_{e,}/Z_p$

$Z_{p,}$,Z_e = plastic section modulus and elastic section modulus with respect

to extreme compression fiber respectively

f_y = yield stress of the material

γ_{m0} = partial safety factor

M_d = design bending strength

M_{dv} = the design bending strength under high shear

V = factored applied shear force

L_0 =length between points of zero moment (inflection) in the span

b_0 = width of the flange with outstand

b_i = width of the flange as an internal element.

G = shear modulus

E = young modulus

L = Length of beam

EI_z = Low flexural stiffness about the weak axis

GI_t = Low torsional stiffness

EI_w = Low warping stiffness

GC = centre of gravity,

SC = shear centre,

TC = torsion centre,

T = Torque applied

f_{bd} = design bending compressive stress

α_{LT} =the imperfection parameter

λ_{LT} =non-dimensional slenderness ratio,

M_{cr} = elastic critical moment

$f_{cr,b}$ = extreme fiber bending compressive stress corresponding to elastic lateral buckling moment

I_t, I_w= st-venant torsion constant and warping constant respectively

L_{LT} = Effective length

h_f =center to center distance between flanges

t_f = thickness of flange

I_y,r_y =moment of inertia and radius of gyration, respectively about the weaker axis

KL/r = effective slenderness ratio or ratio of effective length (KL) to approximate radius of gyration, r

χ = stress reduction factor for different buckling class, Slenderness ratio and yield stress

CHAPTER 1

INTRODUCTION

A structural member subjected to transverse loads (Transverse loads means loads perpendicular to its longitudinal axis) is called a beam. When provided in buildings to support the roofs, they are called the **joists**. A large beam supporting a number of joists is called a **girder**. Exterior beam at the floor level of buildings, which carry part of the floor load and that of the exterior wall are called the **spandrels**. Beam which carry the roof loads from masonry over the openings are called **lintels**. A horizontal beam spanning the wall columns of industrial buildings used to support wall coverings is called a **girt**. Structural members subjected to bending, when accompanied by large axial compressive loads are knows as **beam-columns**.

A beam may, in general, be subjected to either simple, unsymmetrical, or bi-axial bending.

1.1 Symmetrical bending

For simple bending to occur, the loading plane must coincide with one of the principal planes of doubly symmetric section (I-section) and for singly symmetric section (a channel section) it must be pass through the shear center.

- Bending moment about a principal axis.
- Neutral axis coincides with the principal axis.

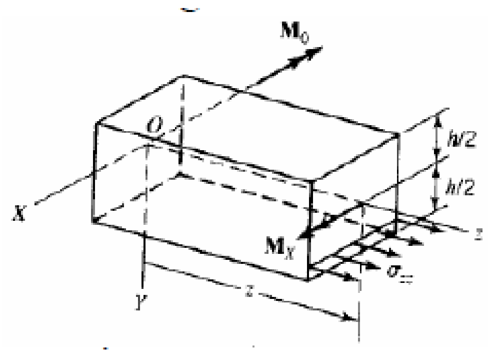


Fig 1 Symmetrical bending

1.2 Unsymmetrical bending

When the plane of the loading does not pass through the shear center the bending is called the unsymmetrical.

1.3 Shear center

- the shear center (for any transverse section of the beam) is the point of intersection of the bending axis and the plane of the transverse section.
- Shear center of a section can be defined as a point about which the applied force is balanced by the set of shear forces obtained by summing the shear stress over the section (for unsymmetrical section such as angle section and channel section, summation of shear stresses in each leg gives a set of forces which should be in equilibrium with the applied shear force).
  The shear center is also known as “center of twist.”
- in case of a beam having two axes of symmetry, the shear center coincides with the centroid.

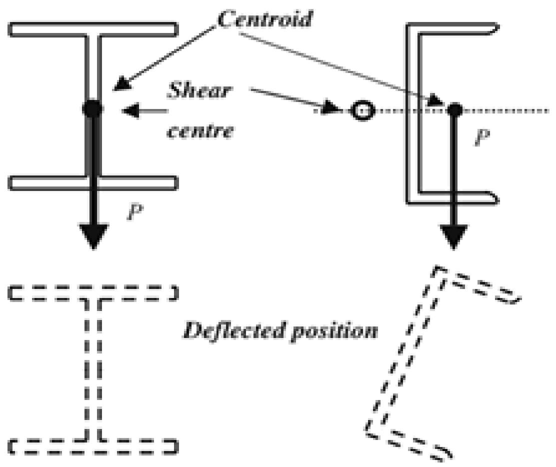


Fig 2 ***Deflection of beams loaded through the centroid***

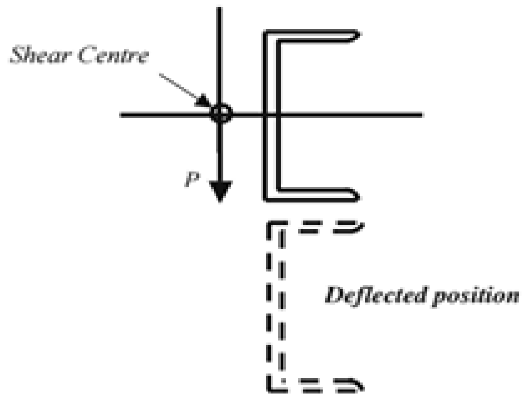


Fig 3 ***Deflection of channel beam loaded through the shear centre***

- In case of sections having one axis of symmetry, the shear center does not coincide with the centroid but lies on the axis of symmetry.

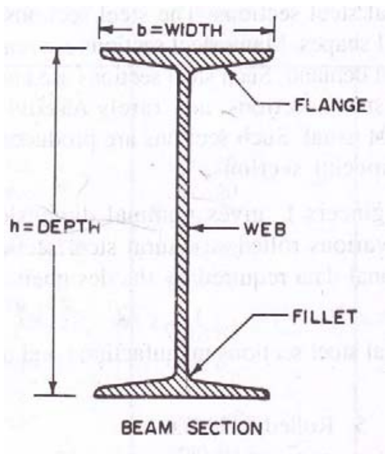
- When the load passes through the shear center then there will be only bending in the cross-section and no twisting.

1.4 Types of sections

a number of rolled steel sections are in use. These are basically classified as **open sections** and **closed sections**.

In general, the cross sections employed are open type in which most material is distributed in flanges and that too away from their centroid. This improves their resistance to in-plane bending. Moreover, open sections are very convenient for making connections with other members. Although, they possess high major axis bending strength, but are quite weak with regards to their minor axis bending strength and torsional resistance. Angle and t-sections are inherently weak in bending while channels can only be used for light loads.

A rolled steel I-section is generally preferred as a beam.



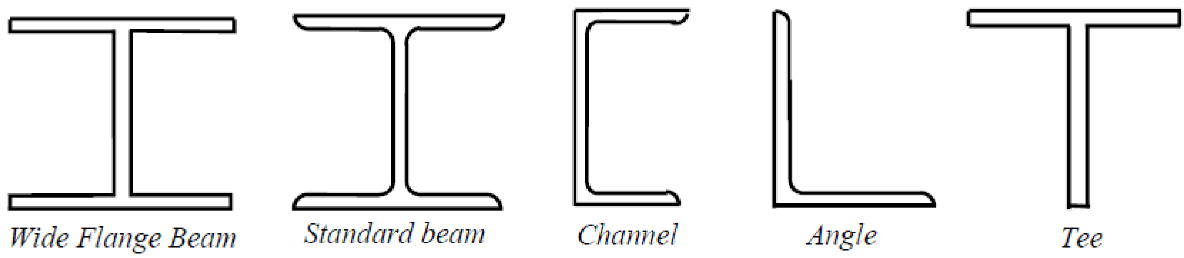


Fig 4 *Open sections*

Closed sections have high torsional stiffness, often as high as 100 times that of an open section, but are difficult to connect with other members and are rarely used. However, square/rectangular hollow sections prove to be most efficient when laterally unsupported length of the girders is quite large.

Fig 5 ***Closed sections***

Usually, the most efficient and economical shapes are I-shapes, either rolled or built-up.

An I-section with cover plates is provided when a large modulus of section is required. Generally, ISLB or ISMB sections are provided in such cases.

When heavy loads are encountered, as may be in the case of a bridge, the I-section is built-up with plates for welded constructions or with plates and angles for riveted/bolted constructions.

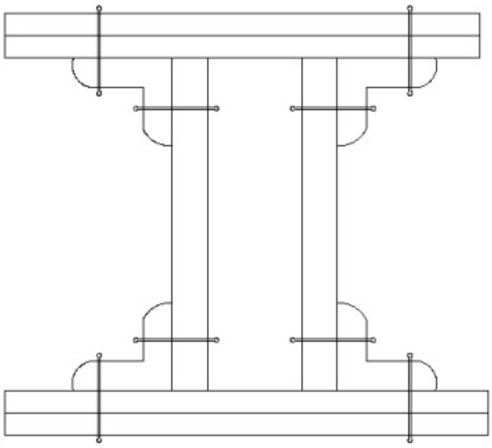


Fig 6 Built up section

When the beams are subjected to lateral loads at the compression flange level, an I-section, preferably ISWB section, along with a channel section is provided.

1.5 Classification of cross sections

In the limit state design of steel beams, it is necessary to classify sections as the moment capacity of the classified section depends on the different formulation.

There are four classes of sections

- The Plastic ($M_d = Z_p f_y$)

- The compact ($M_d = Z_p f_y$)
- The semi-compact ($Md = Z_e f_y$)
- The slender ($M_d = Z_{ee} f_y$)

Where Z_p is the plastic section modulus

Z_e is the elastic section modulus

Z_{ee} is the effective elastic section modulus

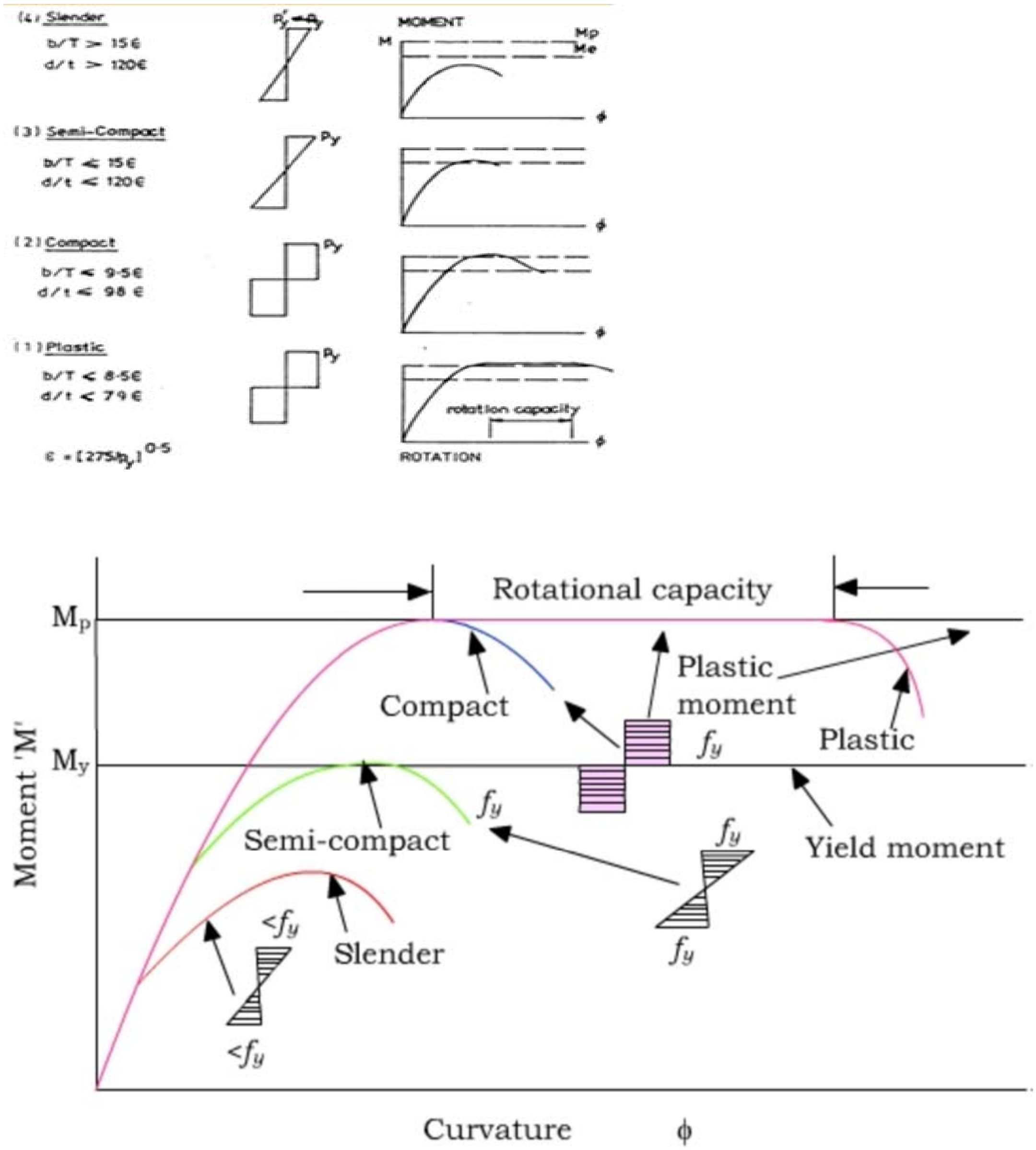


Fig 7 Classification of Cross section

1.6 Lateral stability of beams

Laterally stable steel beams can fail only by (a) Flexure (b) Shear or (c) Bearing, assuming the local buckling of slender components does not occur. These three conditions are the criteria for limit state design of steel beams. Steel beams would also become unserviceable due to excessive deflection and this is classified as a limit state of serviceability.

The factored design moment, M at any section, in a beam due to external actions shall satisfy

$M \leq M_d$

Where M_d = design bending strength of the section

1.6.1 Calculation of design bending strength of the section

1. When the factored shear force does not exceed 0.6 V_d, where V_d is the design shear strength of the cross-section, the design bending strength,

M_d shall be taken as:

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$

Where

β_b = 1 for plastic and compact sections

= Z_p / Z_e for semi compact sections

Z_p, Z_e = plastic section modulus and elastic section modulus with respect

to extreme compression fibre

f_y = yield stress of the material

γ_{m0} = partial safety factor

2. When the factored shear force V exceed 0.6 V_d , where V_d is the design shear strength of the cross-section, the design bending strength,

M_d shall be taken as:

$$M_d = M_{dv}$$

Where M_{dv} is the design bending strength under high shear calculated as follows

A) For plastic or compact section
$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e f_y / \gamma_{m0}$

Where

$$\beta = (2V/V_d - 1)^2$$

M_d = plastic design moment of the whole section disregarding high shear force effect considering web buckling effects

V = factored applied shear force

Vd = design shear strength

M_{fd} = plastic design strength of the area of the cross section excluding the shear area

B) For semi-compact section

$M_{dv} = Z_e f_y/\gamma_{m0}$

1.7 shear lag effect

The simple theory of bending is based on the assumption that plane sections remain plane after bending. But, the presence of shear strains causes the section to warp. Its effect in the flanges is to modify the bending stresses obtained by the simple theory, producing higher stresses near the junction of a web and lower stresses at points away from it.

This effect is called 'shear lag'. This effect is minimal in rolled sections, which have narrow and thick flanges and more pronounced in plate girders or box sections having wide thin flanges when they are subjected to high shear forces, especially in the vicinity of concentrated loads.

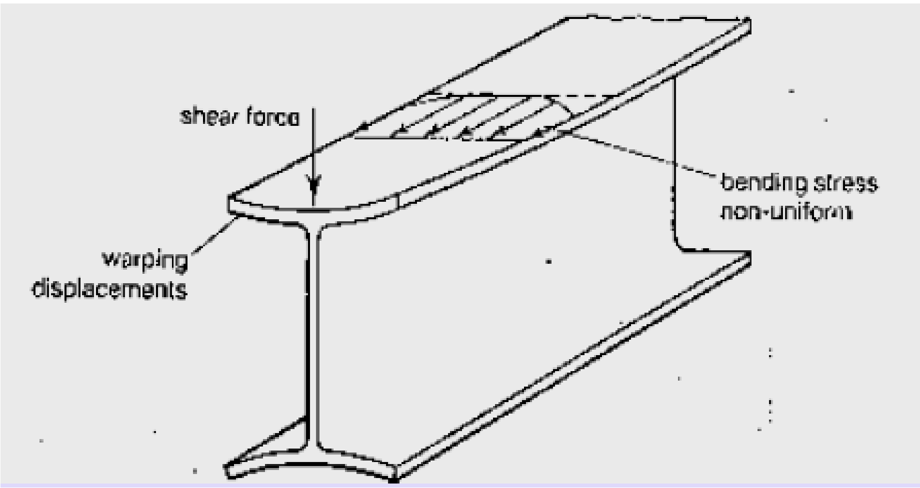


Fig 8 Shear leg effect

As per IS :800

Provisions for shear lag

The shear lag effects may be disregarded provided :

- For outstand elements (supported along one edge), $b_0 \leq L_0/20$
- For internal elements (suppoeted along two edges), $b_i \leq L_0/10$

Where

L_0=length between points of zero moment (inflection) in the span

b_0 = width of the flange with outstand

b_i = width of the flange as an internal element.

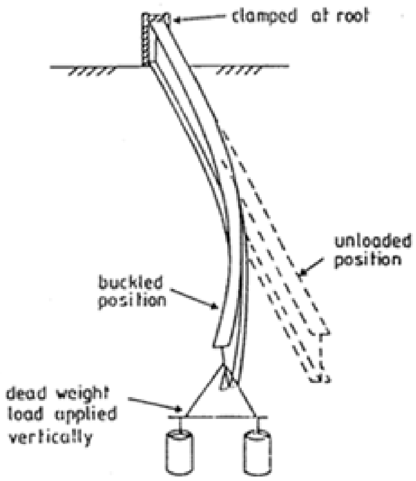


Fig 9 Lateral stability of beam

1.8 Stability

A theory called the classical stability theory is based on three main assumptions:

- Linear elastic material
- Ideal shape of structural elements
- Small displacements assumed for constitutive relations.

In the theory it is differentiated between three states of equilibrium; stable, indifferent and unstable, illustrated with cases A), B) and C) in respectively. In the stable state of equilibrium a structure goes back to its original state after a small disturbance.

When a structure is in an unstable equilibrium, only a small disturbance is needed to give rise to forces that take the structure further away from the original state. If a disturbance leads to a change in the structure that neither goes back nor increases after the disturbance ceases, it is referred to as an in different state of equilibrium.

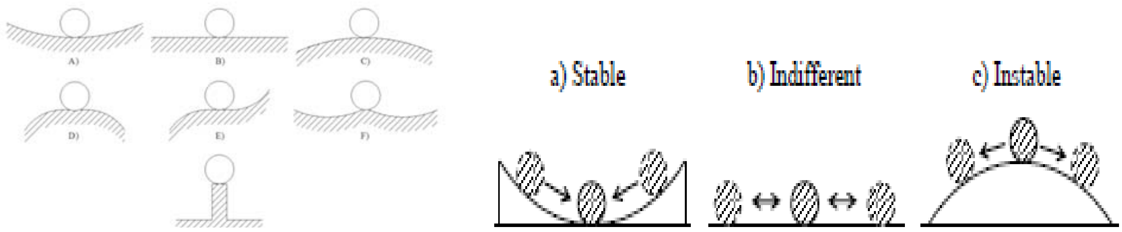


Fig 10 Illustration of different type of equilibrium stage

More types of equilibrium states can be thought of them are presented in D). shows a system that is stable as long as it is only affected by a very small disturbance, but as soon as the disturbance is big enough the system becomes unstable. The effect of disturbance can be dependent on the direction or the types of the disturbance in the same way as the magnitude, as E) represent. Equilibrium case F) illustrates a structure that, due to a small disturbance, becomes more stable after being transferred from its initial state. An equilibrium state that is stable but highly dependent on the effects of a small disturbance is shown in case G).

1.9 Behavior of ideal beam and real beam

An **ideal beam** is a beam that is free of imperfections, such as initial bow imperfection and residual stresses and made of linearly elastic material. When a bending moment about the major axis is applied on a ideal beam its behavior in lateral-torsional buckling can be described as follows. The beam only deforms vertically for loads up to a certain magnitude called the critical load. At that point the beam goes from a stable equilibrium position to an indifferent one and a sudden lateral deflection and a twist occur simultaneously.

A real beam always has some imperfections. The initial lateral bow of the beam results in an immediate increase in lateral deflection and a twist as the beam is loaded. When the load gets close to the critical load the lateral deflection and the twist increases more rapidly. But the theoretical critical load can never be reached.

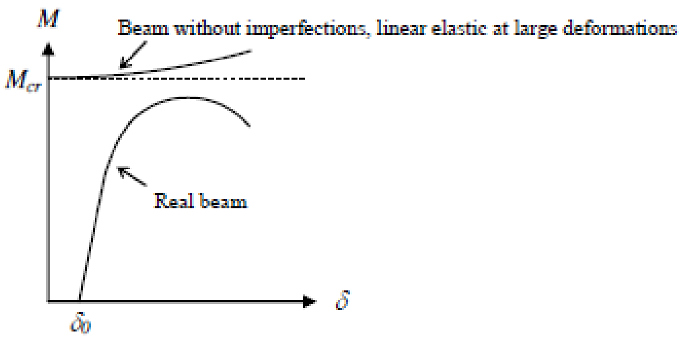


Fig 11 Ideal beam

Increasing moment plotted against lateral deflection. An ideal beam shows no deflection until the elastic critical moment is reached, where a large instantaneous deflection occurs. A real beam has imperfections and residual stresses, and deflects as the load is increased. The failure of the real beam is governed by its imperfections and non-linear response

Lateral-torsional buckling of real beam is affected by five factors

- Non-linear material properties (plastic response)
- Amplitude and shape of the initial lateral imperfections
- Residual stresses due to a shrinkage of the steel during manufacture
- Defects such as holes and asymmetry.

CHAPTER 2

BUCKLING

Buckling is an instability phenomenon caused by a bifurcation of equilibrium and is best described by considering different states of equilibrium. It implies that there is a load level where further loading can lead to more than one equilibrium state.

2.1 BUCKLING TYPES

2.1.1 Local buckling

Buckling may be governed by local buckling for beams with slender webs or flanges. It can be recognized by many small buckles along the web or the flanges. For local buckling of the web, the buckling length is approximately the width of the web, see fig. for a flange be buckling length can vary between 1 and 5 times the width of the flange.

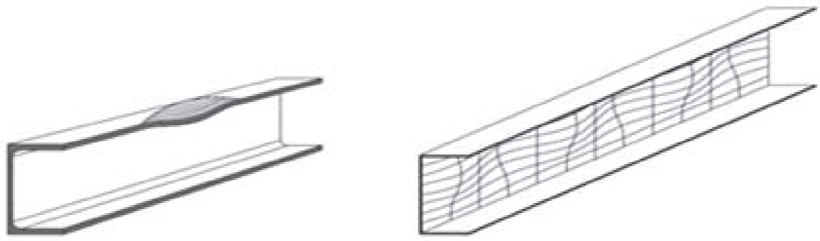


Fig 12 Local buckling

2.1.2 Distortional buckling

Distortional buckling may occur for comparatively short beams. This is a result of an interaction between two buckling modes; lateral-torsional buckling and local buckling, which both are commonly designed for separately.

When distortional buckling occurs, the web distorts and the flanges twist and deflects laterally.

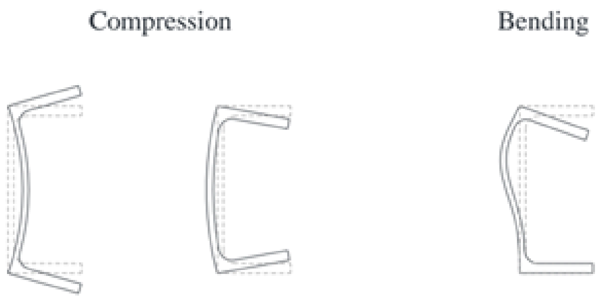


Fig 13 Distortion Buckling

2.1.3 Global buckling

Buckling that occurs and influences the beam globally is called global buckling. This type of buckling can be subdivided into two

1. Flexural buckling
2. Torsional buckling
3. Torsional-flexural buckling
4. Lateral-torsional buckling

2.1.3.1 Flexural buckling

Under axial loading or simultaneous axial and moment loading, a beam may buckle in one plane without twisting. This is called flexural buckling. If a member undergoes a pure axial load, it may buckle laterally and will take the shape of sinus wave.

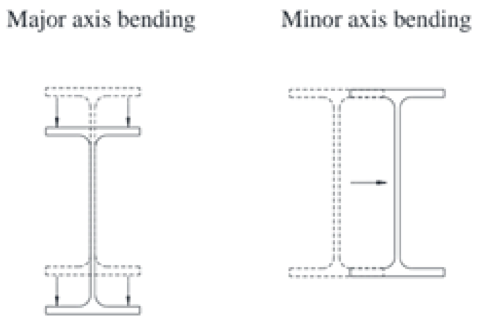


Fig 14 Flexural buckling

2.1.3.2 Torsional buckling

Members with a small torsional stiffness can buckle in that way the cross section twists when an axial load is applied, as shown in fig.

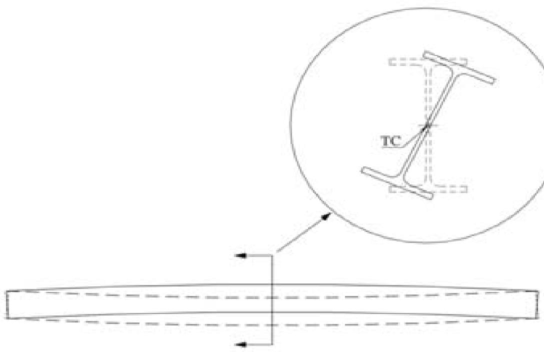


Fig 15 Torsional buckling

2.1.3.3 Torsional-flexural buckling

Torsional-flexural buckling may occur due to axially loaded beams with a monosymmetric I-section and a channel cross section. The beam will simultaneously twist and deflect laterally when it buckles.

2.1.3.4 Lateral-torsional buckling

When a pure moment acts on an I-beam about its major axis, one flange undergoes compression and the other in tension. If the lateral stiffness is insufficient the compression flange will buckle laterally before the plastic for class 1 and 2, elastic for class 3 and effective for class 4 moment resistance of the beam has been reached. The compression flange pushes the beam sideways which results in a lateral deflection. The tension flange pulls the beam towards its original location which results in twisting of the beam. This type of buckling is lateral-torsional buckling.

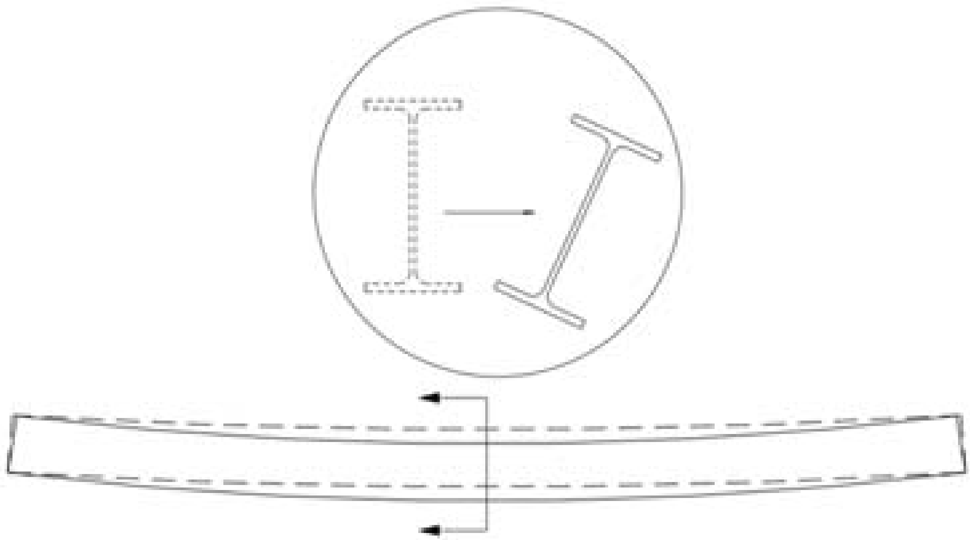
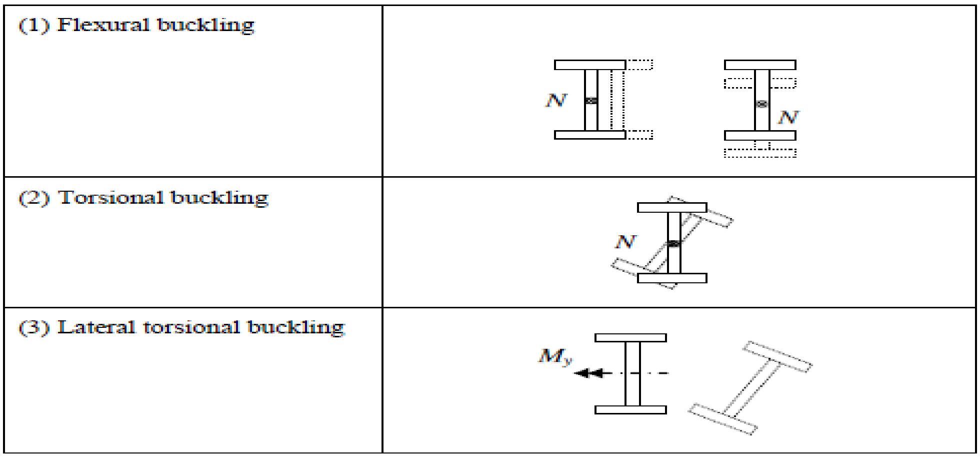


Fig 16 Lateral Torsional Buckling

When a member loaded in bending deflects laterally and twists at the same time, it is called lateral torsional buckling. The resistance to this buckling mode is governed by the flexural, torsional and warping stiffness.

Deformations due to flexural, torsional and lateral torsional buckling



2.2 Factors that affect lateral torsional buckling

Factors that affect lateral torsional buckling	
Material properties	Shear modulus (*G*) Youngs's modulus (*E*)
Cross-section properties	Torsion constant (I_t) Warping constant (I_w) Second moment of inertia about weak axis (I_z)
Geometric properties	Length of the beam (*L*)
Boundary conditions	Bending about major axis Bending about minor axis Warping
Load	Type of loading (distributed, concentrated etc.) Point of load application (top flange, in shear centre, bottom flange etc.)

Choosing a beam with the right properties, lateral torsional buckling can be avoided. The risk of lateral torsional buckling is high for beams with the below listed properties.

- Low flexural stiffness about the weak axis (EI_z)
- Low torsional stiffness (GI_t)
- Low warping stiffness (EI_w)
- High point of load application
- Long unrestrained spans (L)

2.2.1 Influence of the cross-section

Lateral torsional buckling is only possible in major axis bending. If the flexural stiffness is high enough about the weak axis or if the stiffness is equal about both axes, lateral torsional buckling will not occur.

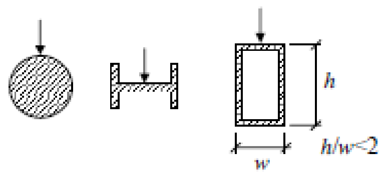


Fig 17 Different cross section influence

Sections with little or no risk of lateral torsional buckling

2.2.2 Influence of the point of load application

A low point of load application reduces the risk of lateral torsional buckling. Hence, a load placed on the bottom flange makes a beam considerably more stable than a load on the top flange. This is provided that the load does not contribute with any restraining effects.

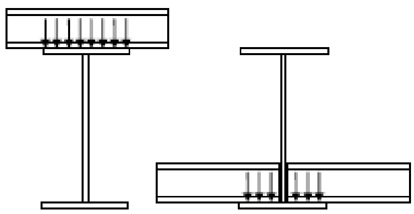


Fig 18 Influence of point load application

A beam loaded by a concentrated load from a secondary girder. If a low point of load application is used the load helps to stabilize the beam. A high point of load application contributes to twisting moments and makes the beam less stable. When a beam deflects due to lateral torsional buckling, loads above the center of twist will contribute with a twisting moment. Loads below the center of twist counteract rotations of the sections and stabilizes the beam. *A load, F, acting above the center of twist contributes to the rotation of the section with an additional moment. When placed under the center of twist, the load stabilizes the section*

2.2.3 Influence of lateral restraints

If the flange in compression is restrained from lateral bending, lateral torsional buckling can be fully prevented. The spacing of the restraints must be small enough so that buckling cannot occur between them.

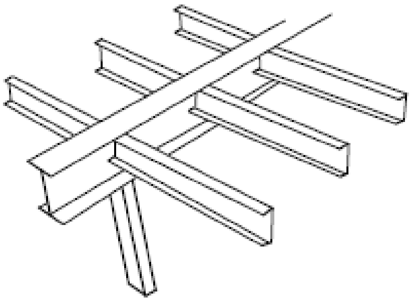


Fig 19 Influence of lateral restraints

Provided that the top flange is in compression, the beam can be restrained from lateral torsional buckling by secondary girders

2.2.4 Restrained flange in compression

Restraints can often be provided from secondary members in a construction. If a simply supported beam is loaded directly by an evenly distributed load from a slab, the beam can be fully prevented from lateral torsional buckling provided that the slab is part of a stable system. The upper flange of the beam is in compression in the whole span, and restrained at all points. A twist of such a beam will be counteracted by the slab.

Even if the slab is part of a stable system in the finished structure, other conditions may apply during the erection of the building. Hence, commonly the beam is attached to the slab by some kind of connection to assure that it is restrained also during assembly.

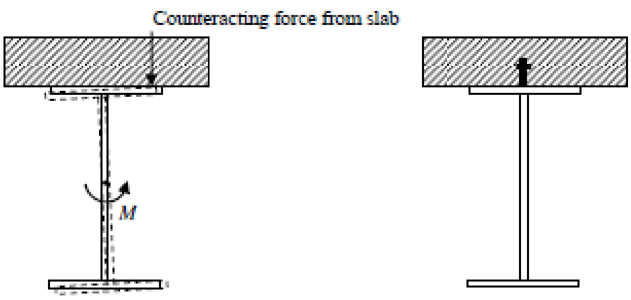


Fig 20 Restrained flange in compression

A beam subjected to a twisting moment is restrained on the compression flange by a floor slab. A counteracting moment is created from the reaction force from the slab, holding the beam in place (left). Often the beam is attached to the slab to make sure that system is stable (right).

2.2.5 Restrained flange in tension

If the beam instead is made continuous, the top flange will be in tension close to the mid-supports. Restraining the tension flange will also increase the stability of the beam, but in some cases this is not enough to prevent the compression flange from buckling. This type of buckling is called *distortional buckling* and can be prevented by restraining the compression flange, i.e. restraining the bottom-flange, close to the mid-supports.

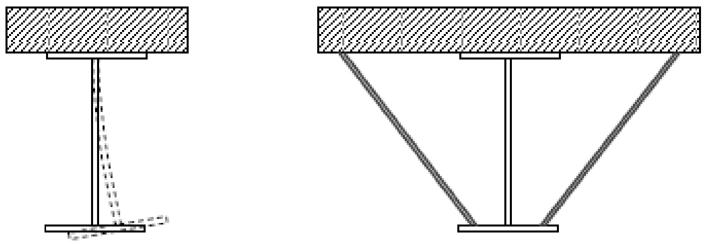
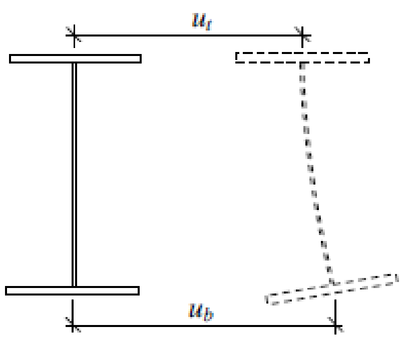


Fig 21 Restrained flange in tension

If a beam is restrained on the tension flange, the unrestrained compression flange may buckle laterally due to distortional buckling (to the left). This can be prevented by placing restraints onto the compression flange (to the right)

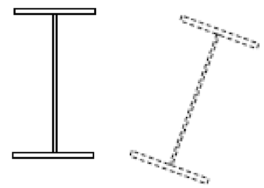
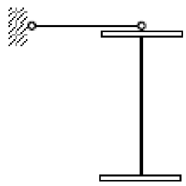
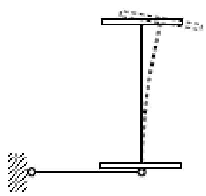
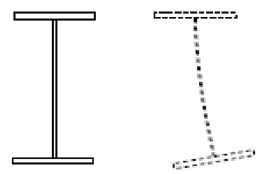
2.2.6 Unrestrained beams with slender webs and rigid flanges

Unrestrained beams sometimes buckles in a mix between lateral torsional buckling and distortional buckling. This buckling mode is possible for beams with slender flexible webs and rigid flanges, and is called *lateral distortional buckling.*



Lateral distortional buckling is possible for beams with slender webs and rigid flanges.

2.3 Buckling modes of restrained and unrestrained beams

No restraints - Free to buckle due to lateral torsional buckling	
Compression flange restrained - No lateral torsional buckling is possible	
Flange in tension restrained - Distortional buckling is possible	
No restraints - Lateral distortional buckling is possible for beams with slender flexible webs and rigid flanges	

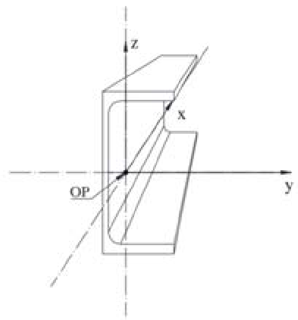
2.4 MODELLING PROPERTIES

2.4.1 Orientation of Coordinate system

The coordinate system used hereafter is a Cartesian coordinate system where the x-axis lies along the beam, the positive z-direction is vertical pointing upward and the y-direction is horizontal, perpendicular to the x-axis.

The origin noted as OP in fig. is located at the centroid.

For a vertically loaded beam the y-axis is considered its major axis and the z-axis its minor axis. Bending about the y-axis will consequently be called major axis bending and bending about the z-axis minor axis bending.



Coordinate system for a channel cross section with the origin noted as OP.

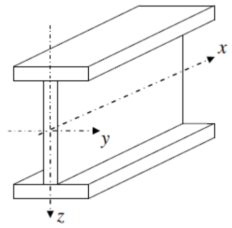
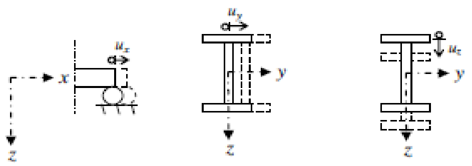
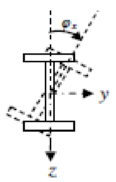
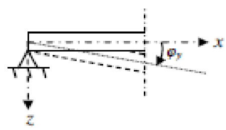
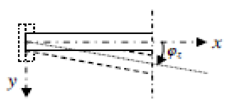
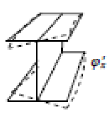


Fig 22 *Orientation of beam coordinates.*

2.4.2 Degree of freedom

A general beam element has two nodes, see in the fig each node has 6 degrees of freedom; translations in the x-,y- and z-direction and rotations about the x-,y- and z-axis. A special kind of a beam element, hereafter referred to as a warping beam element, has an additional warping degree of freedom, to take warping into account. Shell elements can vary in shape, have different number of nodes and integration points, for example. Each node in a shell element is considered to have the same 6 degree of freedom as a general beam element; 3 translation and 3 rotational.

Degrees of freedom for a beam element.

Translation in x, y and z	
Rotation about x-axis	
Rotation about y-axis (Major axis bending)	
Rotation about z-axis (Minor axis bending, lateral bending)	
Warping (see chapter 2.1.10)	

2.4.3 Center of gravity:

The center of gravity, GC, is the origin of the coordinate system. When looking at a cross section of a homogeneous beam, this point is where the weighted relative position of the mass sums to zero.

2.4.4 Centroid:

For beams with a cross section of a homogeneous material, the centroid coincides with the GC as the density of the material is not taken into account, only the geometry.

2.4.4 Torsion center

if a beam with an open section is subjected to torsion, every cross section will rotate around the torsion center, TC, if warping is prevented in any section. The TC coincides with the SC.

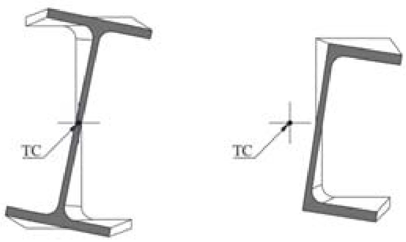


Fig 23 Torsion center

2.4.5 Point of load application

The point of load application, represents the point where the load acts on the cross section of a beam, the term Z_g is usually used to describe the coordinate of the point of load application with respect to SC in the z-direction. Another common term is Z_a, which is the coordinate of the poin of load application with respect to the GC in the z-direction. The location of the pont of load application in the y-direction can be described by the terms y_g and y_a respectively. y_g=0 represents a centric loading and $y_g \neq 0$ an eccentric loading. For beams with a cross section symmetric about the y-axis, Z_g and Z_a are equal. For beams with a cross section symmetric about the z-axis, such as I-beams, y_g and y_a are equal.

2.4.6 Support conditions

In order to have a system that is in equilibrium, a model of a loaded beam must include supports, i.e. some degrees of freedom have to be restrained. Looking at the ordinary single span beam, it is supported at both ends and regardless of the support type; the y- and z-translation restrained. The translation along the beam, in the x-direction, must be restrained at least at one node and most commonly the x-translation is fixed at one end but free at the other. To prevent the beam from rotating about its own longitudinal axis, the x-rotation should be restrained at both supports.

Consider the single spanned beam, with supports at both ends, modelled with the warping beam elements. The 3 of the nodes 7 degree of freedom that are left and available for altering are:

- Rotation about y-axis(k_y)
- Rotation about z-axis (k_z)
- Warping (k_w)

Two common single spanned beam models. Model A) is considered to have fixed supports, where rotations about the y-axis and the z-axis as well as warping are all fixed at both supports so that $k_y = k_z = k_w = 0.5$. Model B) is considered to have hinged supports, where rotations about the y-axis and the z-axis as well as warping are all free at both supports so that $k_y = k_z = k_w = 1.0$.

In general the value of an effective buckling length factor varies from 0.5 for full restraint to 1.0 for no restraint and takes the value 0.7 for a beam fixed at one end and free at the other.

2.4.6 Torsion

According to the saint-venant torsion theory the typical torsion stresses occur only when free warping can take place. That situation is not common in practice. Another restriction for st-venant torsion is that distributed moment loading is not allowed. When an infinitesimal beam element is subjected to torsion the element will twist by an angle *dφ.*

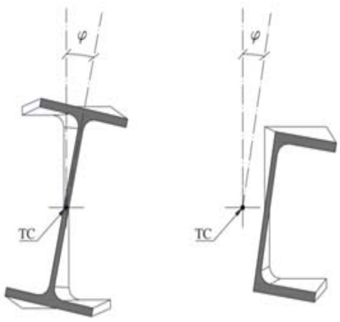
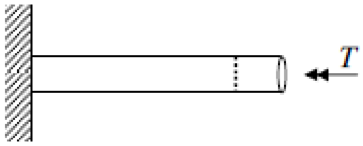


Fig 24 Twisting of I- shape and channel cross section with ϕ as the angle of twist

2.4.7 Warping

When torsion is applied to a beam, every section will rotate around its TC. For circular members every section rotates in plane without any out of plane deformation. For members with other cross sections, the sections rotate as in the case of members with circular cross sections, but in addition to that the cross section twist about the z- and /or the y-axis which results in warping. Elements in a circular-symmetric section subjected to a torque, will twist to a new location within the same plane. Hence, the twisted section will consist of the same material as before the torque was applied.

Circular-symmetric cantilever member subjected to a twisting moment. Any section in the member remains in-plane after the moment is applied



However, when non-circular-symmetric sections are subjected to torsion, sectional elements will not remain in their initial planes. The reason for this is that the twisting moment will cause parts of the member to bend, moving elements along the member and out of their initial planes. This type of deformation is called warping. A cantilever I-beam is subjected to a twisting moment resulting in warping. Observing the beam-end it is clear that the section has not remained in its initial plane, the material movement along the beam can be seen clearly. At the support, the flanges cannot move and thus warping is prevented.

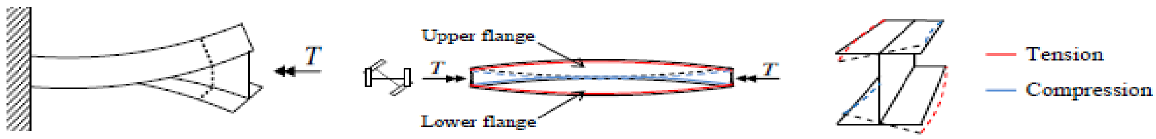


Fig 25 warping

Due to the warping, stresses are created in the flanges as shown in Figure
Tension and compression stresses are created along the flanges as a result of lateral bending. it can be distinguished between three different section
categories with regard to warping; *warping free sections* where no warping occurs, *semi warping free sections* where warping can be neglected in calculations, and *warping sections* where warping must be taken into account.

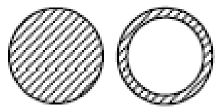
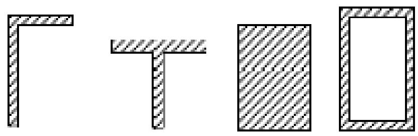
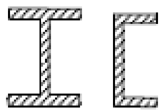
(1) Warping free sections	
(2) Semi warping free sections	
(3) Warping sections	

Fig 26 Classification of section shapes with regard to warping sensitivity

2.5 The factors influencing elastic critical moment Mcr according to analytical expressions are:

- The stiffness about the minor axis
- The torsional stiffness
- The warping stiffness
- The length of the beam
- The boundary conditions
- The type of the load
- The vertical position of the loading
- Material parameters
- Degree of symmetry about the major axis

CHAPTER 3
INTRODUCTION OF COMPRESSION MEMBER

A structural member which is subjected to compressive forces which tend to decrease its length is called a compression member. If the net end moments are zero, then the load is said to be acting concentrically to the member and the structure is said to be axially loaded. Compression members are usually given names: the vertical compression members in building frames are called columns, the inclined ones are called struts. The principal compression member in a crane is called a boom.

The strength of a column depends on the following parameter: -

- Material of the column or member.
- Cross-sectional configuration.
- Support conditions.
- Length of the column.
- Residual stresses.

3.1POSSIBLE FAILURE MODES OF A COMPRESSION MEMBER (AXIALLY LOADED)

3.1.1 LOCAL BUCKLING

Failure occurs by buckling or deflection of one or more parts of the member, for example: flange or web of an I-section. No overall deflection is observed in this kind of buckling.

3.1.2 SQUASHING

Squashing occurs in relatively small length columns. It occurs by yielding of a cross section of the column.

3.1.3 OVERALL FLEXURAL BUCKLING

In this mode, failure of the member occurs by excessive deflection in the plane of weaker principal axis.

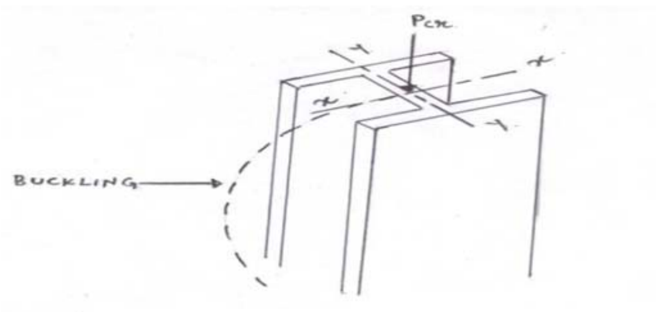
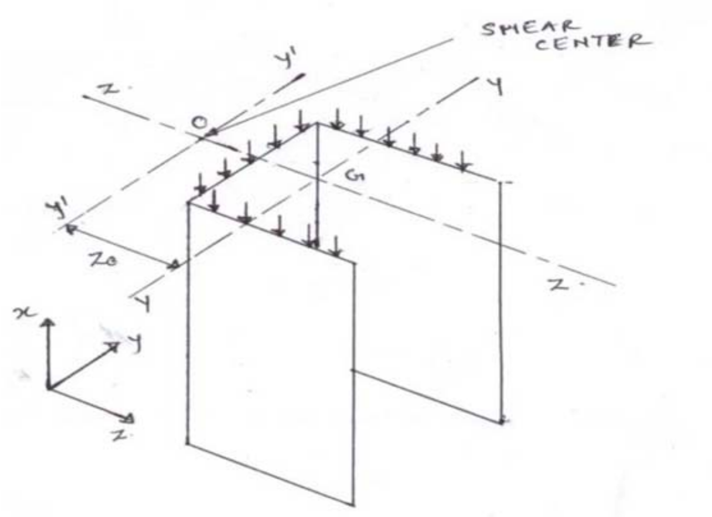


Fig 27 Buckling

In the above fig, x-x and y-y axis are shown. Ixx > Iyy , so the resistance about y-y axis is less as compared to x-x axis. Hence buckling will occur about y-y axis.

3.2 TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING

Torsional buckling failure occurs by twisting of the column about shear center in the longitudinal axis. A combination of flexural and torsional buckling is called flexural-torsional buckling.



3.3 CLASSIFICATION OF COLUMNS BASED ON THEIR LENGHTS AND THEIR BEHAVIOUR:

3.3.1 SHORT COLUMN

Short columns are very short a compression member. Slenderness ratio of such columns is very low. The failure of such columns occurs by yielding and hence stresses at failure are yield stresses. No buckling is observed in such columns.

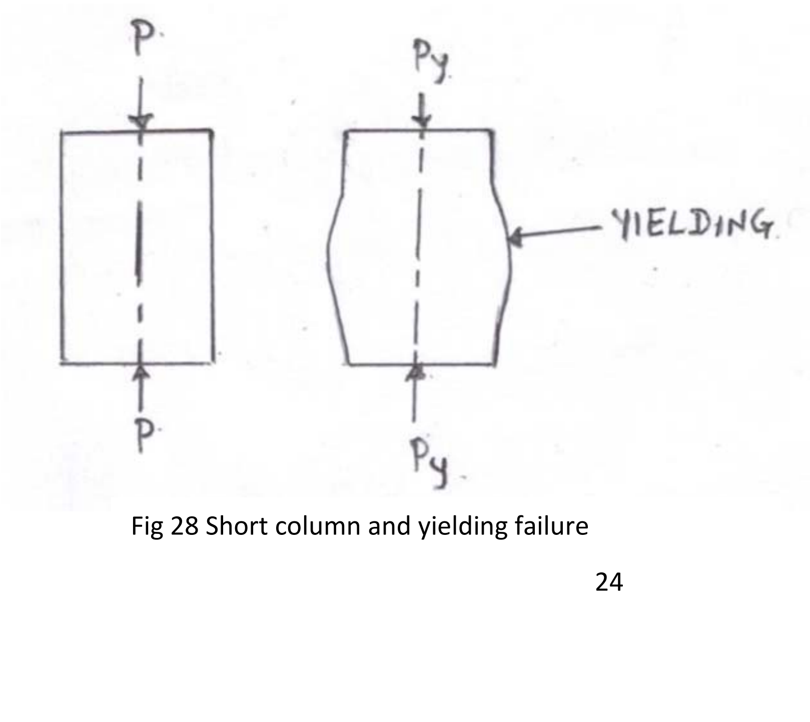


Fig 28 Short column and yielding failure

3.3.2 SLENDER OR LONG COMPRESSION MEMBERS

The strength of any compression member decreases with the increase in its length. Compression members with high slenderness ratio and which satisfy all the conditions of Euler's Formula for buckling are called slender or long compression members. These columns will fail by elastic buckling. The stresses induced during failure (buckling) are well below the yield limit and lie in the elastic zone. So, the failure occurs elastically.

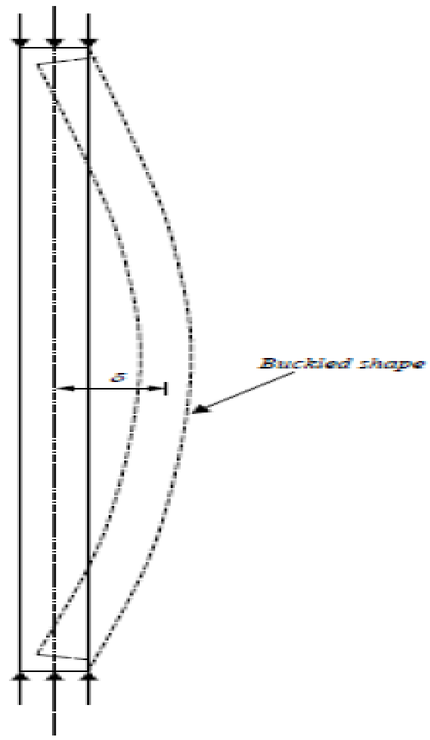


Fig 29 Long column fail by Buckling

.3.2 INTERMEDIATE COLUMNS

A column under an axial load has some fibers yielded some fibers in the elastic limit are known as intermediate columns. These compression members would fail both by yielding and buckling. The failure would fall under the 'inelastic' category. Hence, Euler's formula is not applicable for such columns.

3.4 SLENDER COMPRESSION MEMBERS (ELASTIC BUCKLING):

The buckling of slender compression member or a column was first described by Euler. He was the first one to give remarks about the strength of a column. Euler considered an ideal column with the following properties-

- Material of the member is perfectly isotropic and homogeneous.
- Column has no imperfections.
- Column is pinned at both the ends.
- Column is initially straight and the load are acting concentrically.

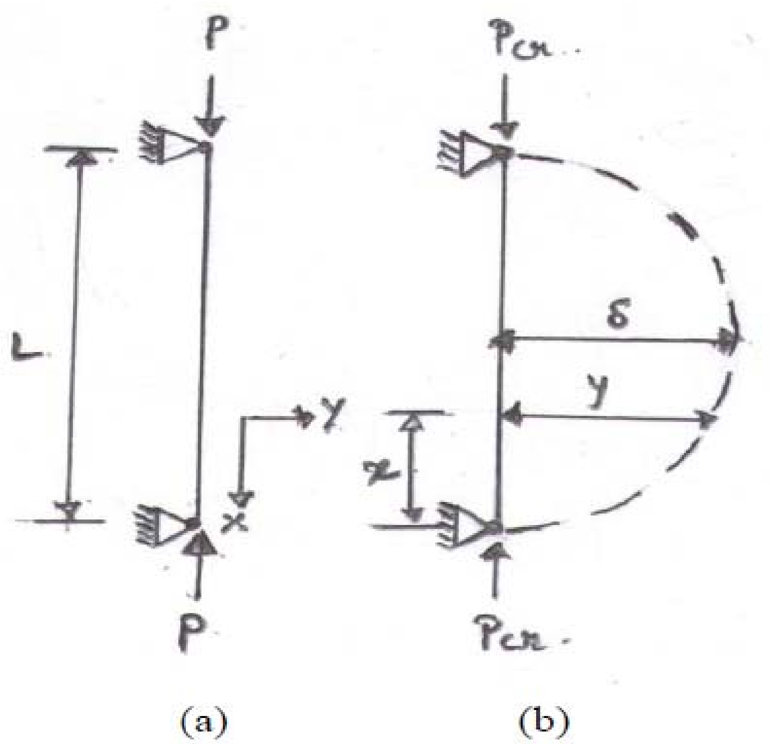


Fig 30 Slender compression member

3.5 STRENGTH OF COMPRESSION MEMBERS IN PRACTICE:

The highly idealized column cannot be achieved in actual practice. It was ignored because the test results did not agree with it. The column in actual practice tends to have initial crookedness, experience accidental eccentric loading, local or lateral buckling and may have residual stresses. Due to these imperfections, the deflection curve of a real column will differ from the curve of an idealized column. Three main factors which result in reduction of the strength of the column are:

3.5.1 EFFECT OF ECCENTRICITY OF APPLIED LOADING:

As discussed earlier, it is impossible to ensure that the load is applied at the exact centroid axis of the column. The above figure shows a load P applied at an eccentricity e to the centroid axis of the column which induces a bending moment (P×e) and causes lateral deflection of the column.

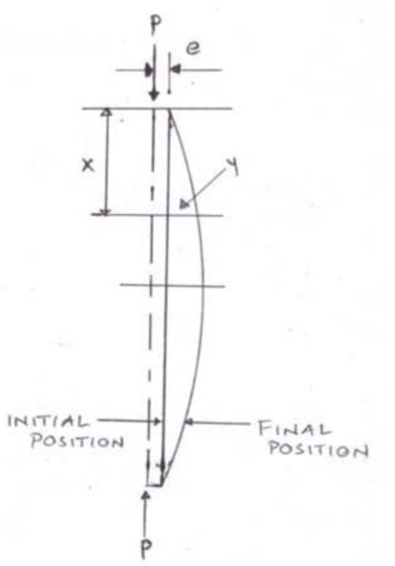


Fig 31 Effect of eccentricity of applied load

3.5.2 EFFECT OF RESIDUAL STRESSES:

As a consequence of differential cooling of different parts of the member while forming process, residual stresses are formed in it.

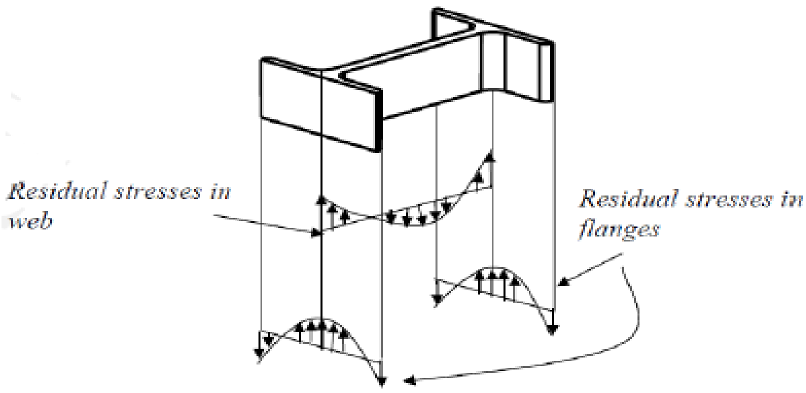


Fig 32 Effect of residual stresses

So, when compressive forces are applied to the compression member with residual stresses, they add upto the residual stresses and may affect the strength of the member by phenomena like premature yielding (for short or stocky columns) or spontaneous buckling (for slender columns).

3.5.3 EFFECTS OF IMPERFECTIONS TAKEN TOGETHER (MULTIPLE COLUMN CURVES) ACCORDING TO IS:800

Fig shows a non-dimensional form of a strength curve of a Euler column. Considering all the imperfection factors encountered in a real compression member, the Indian Code (13800) has adopted the multiple column curves, as shown in the fig below.

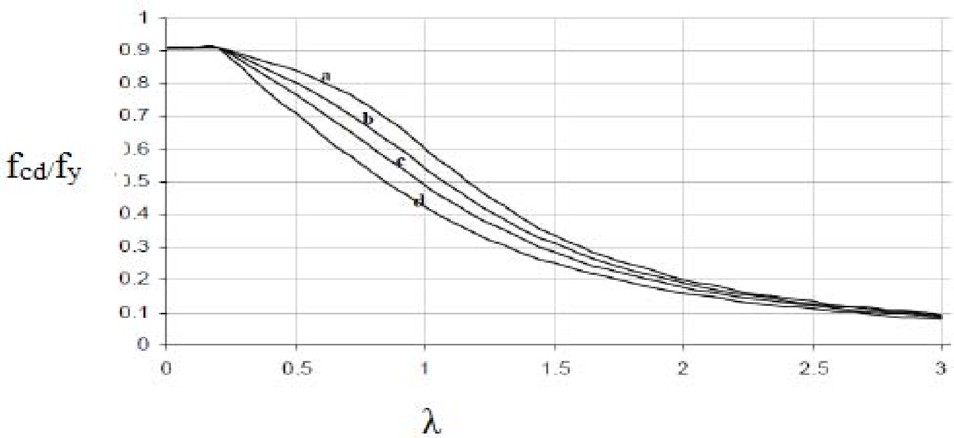


Fig 33 Strength curve of a Euler formula

3.6 The design compressive strength of compression member:

Common hot rolled and built-up steel members are used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a, b, c or d.

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

Buckling class of cross section

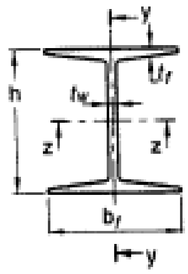
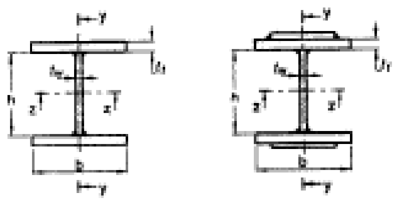
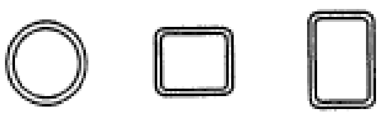
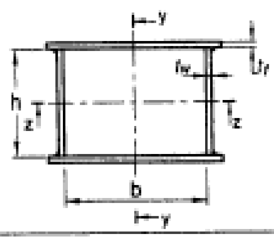
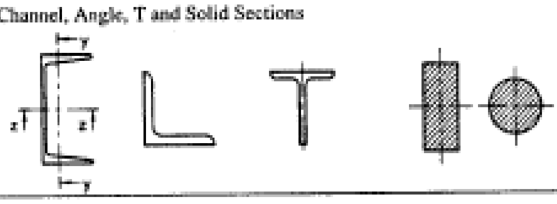
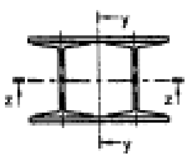
Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \le 40$ mm $40 \le$ mm $< t_f \le 100$ mm	z-z y-y z-z y-y	a b b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm $t_f > 100$ mm	z-z y-y z-z y-y	b c d d
Welded I-Section 	$t_f \le 40$ mm $t_f > 40$ mm	z-z y-y z-z y-y	b c c d
Hollow Section 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section 	Generally (except as below)	Any	b
	Thick welds and $h/t_f < 30$ $h/t_w < 30$	z-z y-y	c c
Channel, Angle, T and Solid Sections 		Any	c
Built-up Member 		Any	c

Fig 34 Buckling class of cross section

-the design compressive strength P_d, of a member is given by:

$P < P_d$

Where

$P_d = A_e f_{cd}$

Where

A_e =effective cross sectional area of the member.

f_{cd} =design compressive stress

-Design compressive stress, f_{cd} , of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{f_y/\gamma_{m0}}{[\phi+(\phi^2-\lambda^2)^{0.5}]} = \chi f_y/\gamma_{mo} \leq f_y/\gamma_{mo}$$

where

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$$

λ = non-dimensional effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}}$$

f_{cc}= Euler buckling stress

$$f_{cc} = \frac{\pi^2 E}{\left(KL/r\right)^2}$$

Where

KL/r = effective slenderness ratio or ratio of effective length (KL) to approximate radius of gyration, r

α = imperfection factor

χ = stress reduction factor for different buckling class, Slenderness ratio and yield stress

$$\chi = \frac{1}{[\phi+(\phi^2-\lambda^2)^{0.5}]}$$

$\gamma_{mo} =$ partial safety factor for material strength

3.7 Effective length of compression member

The effective length KL, is calculated from actual length L, of the member, considering the rotational and relative translation boundary conditions at the ends. The actual length shall be taken as the length from center-to-center of its intersections with the supporting members in the plane of the buckling deformation.

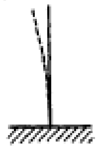
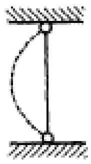
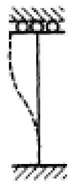
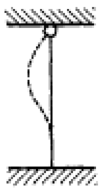
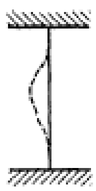
Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation (1)	Rotation (2)	Translation (3)	Rotation (4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0L
Free	Restrained	Free	Restrained		
Restrained	Free	Restrained	Free		1.0L
Restgained	Restrained	Free	Restrained		1.2L
Restrained	Restrained	Restrained	Free		0.8L
Restrained	Restrained	Restrained	Restrained		0.65L

Fig 35 Effective length of prismatic cross section

Chapter 4

4.1 For circular section

Problem: calculate the design compressive load for a circular section ISRO 80, 2 m high. The column is restrained in one end and other end is free for both translation and rotation. Use steel of grade Fe 410.

Solution:

Section Properties

Cross section	Circular
Diameter	D= 80 mm
Cross section Area	A =5026 mm^2
Moment of Inertia	I_{xx} = 2010619.298 mm^4 I_{yy} =2010619.298 mm^4
Radius of Gyration	r_{xx} =20 mm r_{yy}= 20 mm
Unsupported Length	L=2000 mm

End condition's

- At one end Translation and rotation both are restrained
- At other end translation and rotation both are free

Than K= 2.0

- Effective Length L_{eff}= 4000 mm

Material Properties

- Structural steel
- Yield Stress f_y = 250 MPa
- Modulus of elasticity E = $2*10^5$ MPa
- Imperfection factor α=0.49

Calculation of parameter

- Euler buckling stress

$$f_{cc} = \frac{\pi^2 E}{\left({KL}/{r}\right)^2} = 49.34\ MPa$$

- Non-dimensional effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = 2.2509$$

- Inclination of the tension field stress in web

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 3.356$$

- Stress reduction factor

$$\chi = \frac{1}{[\phi + (\phi^2 - \lambda^2)^{0.5}]} = 0.19$$

- Design compressive stress

$$f_{cd} = \frac{\chi f_y}{\gamma_{mo}} = 48.31\ MPa$$

- Design compressive strength

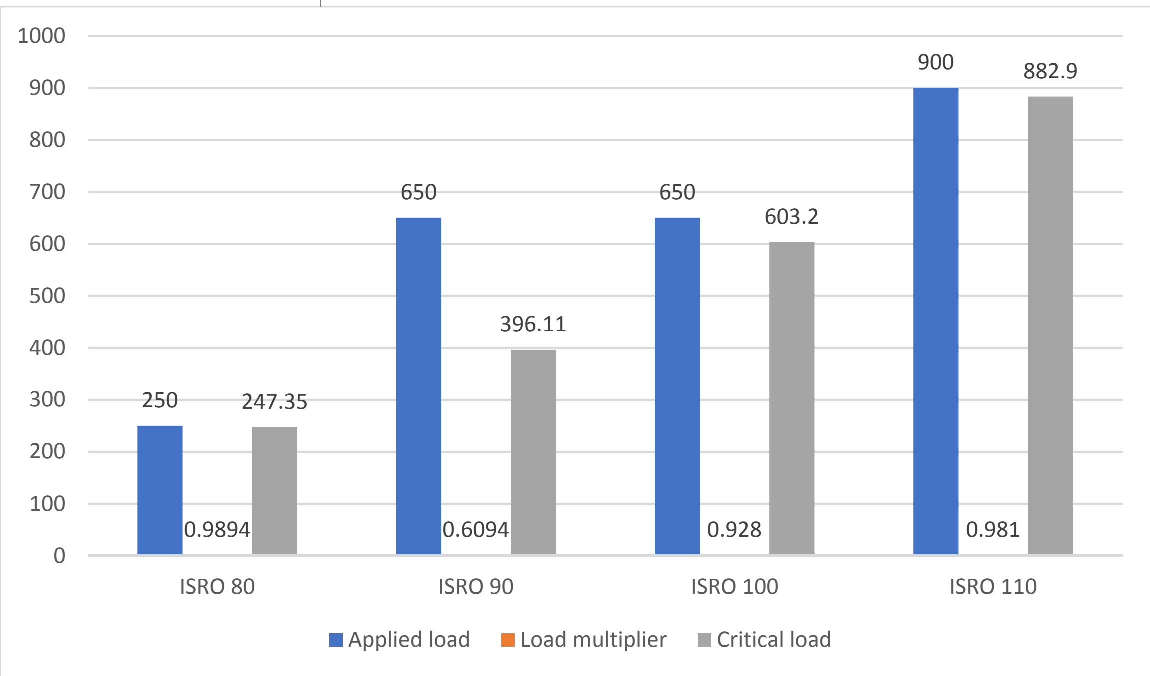
$$P_d = Af_{cd} = 242.79\ \ kN$$

Cross Section	Diameter mm	Area (A) mm²	Moment of Inertia (I) mm⁴	Radius of Gyration (r) mm	K	Unsupported length (L) mm	Effective length (KL) mm	Slenderness ratio (KL/r)
ISRO 80	80	5026	2010619.298	20	2	2000	4000	200
ISRO 90	90	6326	3220623.344	22.5	2	2000	4000	177.77
ISRO 100	100	7854	4908738.521	25	2	2000	4000	160
ISRO 110	110	9503	7186884.069	27.5	2	2000	4000	145.45

f_y MPa	E MPa	α	f_{cc} MPa	λ	ϕ	ϰ	f_{cd} MPa	P_d kN
250	$2*10^5$	0.49	49.34	2.25	3.356	0.19	48.31	242.79
250	$2*10^5$	0.49	62.46	2.0	2.941	0.24	60.44	382.174
250	$2*10^5$	0.49	77.106	1.8	2.512	0.29	74.36	584.07
250	$2*10^5$	0.49	93.304	1.63	2.190	0.35	89.019	845.952

BY ANSYS

CROSS SECTION	APPLIED LOAD (KN)	LOAD MULTIPLIER	CRITICAL LOAD (KN)
ISRO 80	250	0.9894	247.35
ISRO 90	650	0.6094	396.11
ISRO 100	650	0.9280	603.20
ISRO 110	900	0.9810	882.90



4.2 For Square section

Problem: calculate the design compressive load for a square section ISSQ 100, 2 m high. The column is restrained in one end and other end is free for both translation and rotation. Use steel of grade Fe 410.

Solution:

Section Properties

Cross section	square
Side	100 mm
Cross section Area	A =10000 mm^2
Moment of Inertia	I_{xx} = 8.33*10^6 mm^4
Radius of Gyration	r_{xx} =28.87 mm
Unsupported Length	L=2000 mm

End conditions

- At one end Translation and rotation both are restrained
- At other end translation and rotation both are free

Then K= 2.0

- Effective Length L_{eff}= 4000 mm

Material Properties

- Structural steel
- Yield Stress f_y = 250 MPa
- Modulus of elasticity E = 2*10^5 MPa
- Imperfection factor α=0.49

Calculation of parameter

- Euler buckling stress

$$f_{cc} = \frac{\pi^2 E}{\left({KL}/{r}\right)^2} = 102.82\ MPa$$

- Non-dimensional effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = 1.559$$

- Inclination of the tension field stress in web

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 2.048$$

- Stress reduction factor

$$\chi = \frac{1}{[\phi + (\phi^2 - \lambda^2)^{0.5}]} = 0.39$$

- Design compressive stress

$$f_{cd} = \frac{\chi f_y}{\gamma_{mo}} = 99.072\ MPa$$

- Design compressive strength

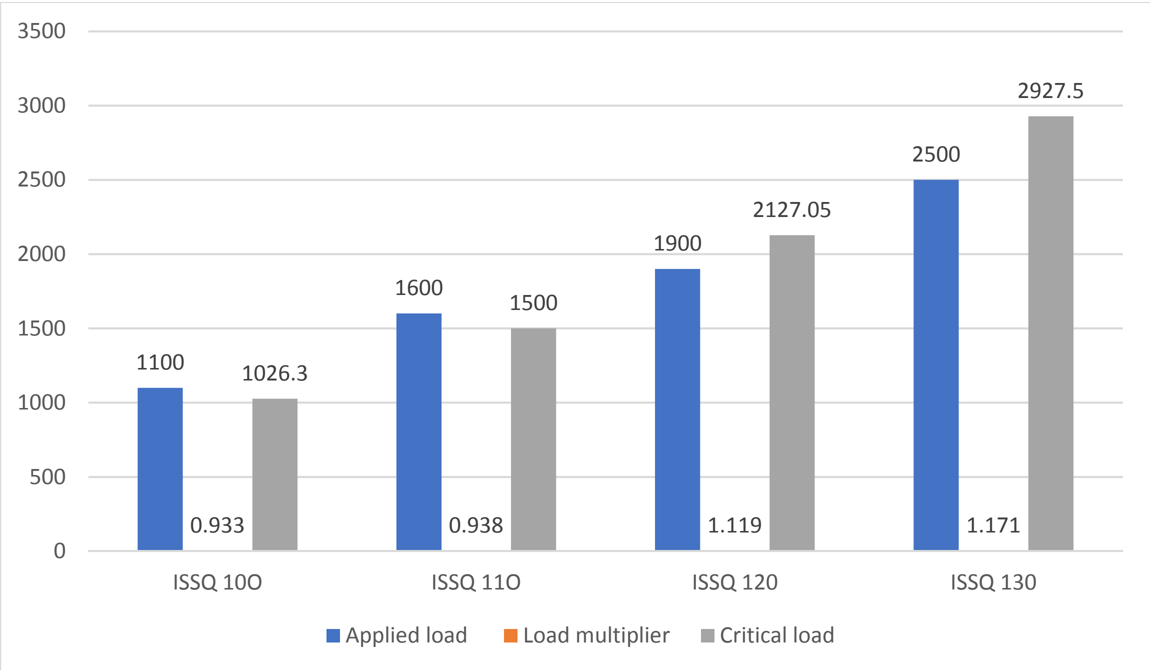
$$P_d = A f_{cd} = 990.72\ \ kN$$

Cross Section	Side mm	Area (A) mm^2	Moment of Inertia (I) mm^4	Radius of Gyration (r) mm	K	Unsupported length (L) mm	Effective length (KL) mm	Slenderness ratio (KL/r)
ISSQ 100	100	10000	$8.33*10^6$	28.87	2	2000	4000	138.56
ISSQ 110	110	12100	$12.20*10^6$	31.75	2	2000	4000	125.97
ISSQ 120	120	14400	$17.288*10^6$	34.64	2	2000	4000	115.47
ISSQ 130	130	16900	$23.80*10^6$	37.52	2	2000	4000	106.59

f_y MPa	E MPa	α	f_{cc} MPa	λ	ϕ	ϰ	f_{cd} MPa	P_d kN
250	$2*10^5$	0.49	102.82	1.559	2.048	0.39	99.072	990.72
250	$2*10^5$	0.49	124.39	1.418	1.804	0.47	118.70	1436.36
250	$2*10^5$	0.49	148.04	1.229	1.61	0.56	139.75	2012.43
250	$2*10^5$	0.49	173.73	1.19	1.463	0.63	159.7	2698.94

By ANSYS

CROSS SECTION	APPLIED LOAD (KN)	LOAD MULTIPLIER	CRITICAL LOAD (KN)
ISSQ 100	1100	0.933	1026.3
ISSQ 110	1600	0.938	1500
ISSQ 120	1900	1.119	2127.05
ISSQ 130	2500	1.171	2927.5



4.3 For I section

Problem: calculate the design compressive load for a I section ISHB 300 @577 N/m, 3 m high. The column is restrained in one end and other end is free for both translation and rotation. Use steel of grade Fe 410.

Solution:

Section Properties

Cross section	I section
Cross section Area	A =7485 mm^2
Moment of Inertia	I_{xx} = 124552000 mm^4 I_{yy} =21936000 mm^4
Radius of Gyration	r_{xx} =129.5 mm r_{yy}= 54.1 mm
Unsupported Length	L=3000 mm

Depth of section	300 mm
Width of flange	250 mm
Thickness of flange	10.6 mm
Thickness of web	7.6 mm

End condition's

- At one end Translation and rotation both are restrained
- At other end translation and rotation both are free

Than K= 2.0

- Effective Length L_{eff}= 6000 mm

Material Properties

- Structural steel
- Yield Stress f_y = 250 MPa
- Modulus of elasticity E = $2*10^5$ MPa
- Imperfection factor α=0.49

Calculation of parameter

- Euler buckling stress

$$f_{cc} = \frac{\pi^2 E}{\left({KL}/{r}\right)^2} = 160.47\ MPa$$

- Non-dimensional effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = 1.248$$

- Inclination of the tension field stress in web

$$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 1.5355$$

- Stress reduction factor

$$\chi = \frac{1}{[\phi+(\phi^2-\lambda^2)^{0.5}]} = 0.58$$

- Design compressive stress

$$f_{cd} = \frac{\chi f_y}{\gamma_{mo}} = 146.14\ MPa$$

- Design compressive strength

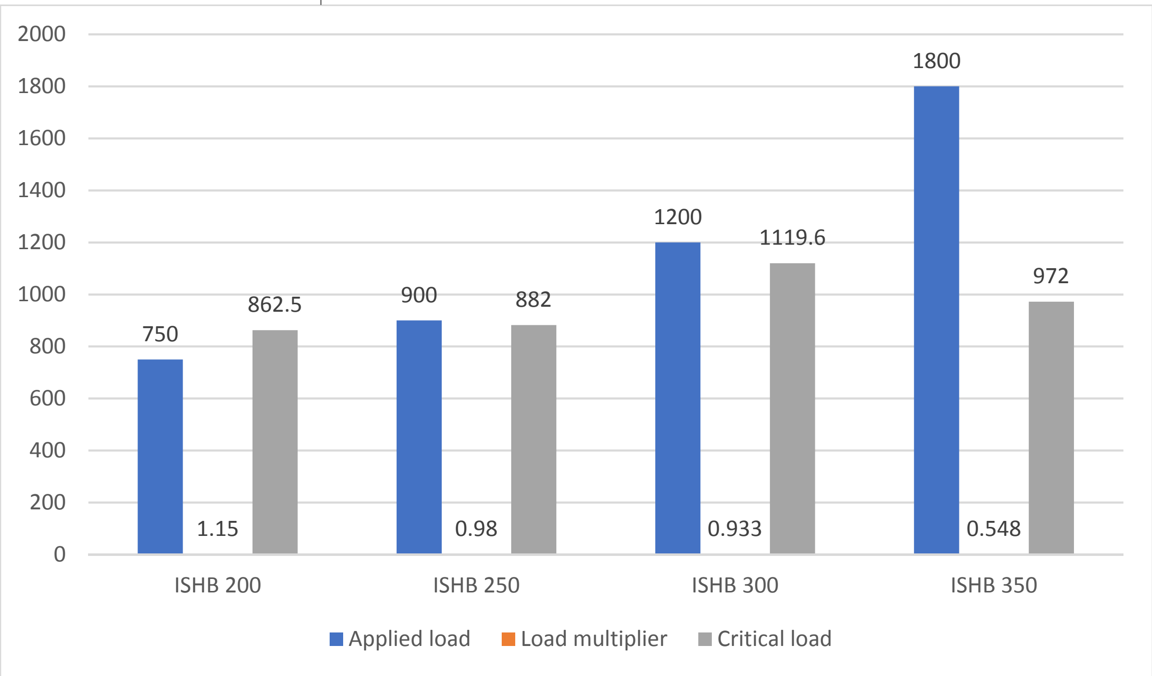
Cross Section	Area (A) mm^2	Moment of Inertia (I) mm^4	Radius of Gyration (r) mm	K	Unsupported length (L) mm	Effective length (KL) mm	Slenderness ratio (KL/r)
ISHB 200	4754	I_{XX}= 36084000 I_{YY}=9671000	R_{xx}=87.1 R_{yy}=45.1	2	2500	5000	110.86
ISHB 250	6496	I_{XX}=7736500 I_{YY}=19613000	R_{xx}=109.1 R_{yy}=54.9	2	2700	5400	198.3
ISHB 300	7485	I_{XX}=125452000 I_{YY}=21936000	R_{xx}=129.5 R_{yy}=54.1	2	3000	6000	110.91
ISHB 350	9221	I_{XX}=198028000 I_{YY}=25105000	R_{xx}=146.5 R_{yy}=52.2	2	3200	6400	122.60

$$P_d = Af_{cd} = 1093.86\ kN$$

f_y MPa	E MPa	α	f_{cc} MPa	λ	ϕ	ϰ	f_{cd} MPa	P_d kN
250	$2*10^5$	0.34	160.61	1.248	1.456	0.72	179.03	851.102
250	$2*10^5$	0.34	204.28	1.106	1.266	0.53	133.29	865.88
250	$2*10^5$	0.49	160.47	1.248	1.5355	0.58	146.14	1093.86
250	$2*10^5$	0.34	131.33	1.380	1.652	0.42	103.77	956.92

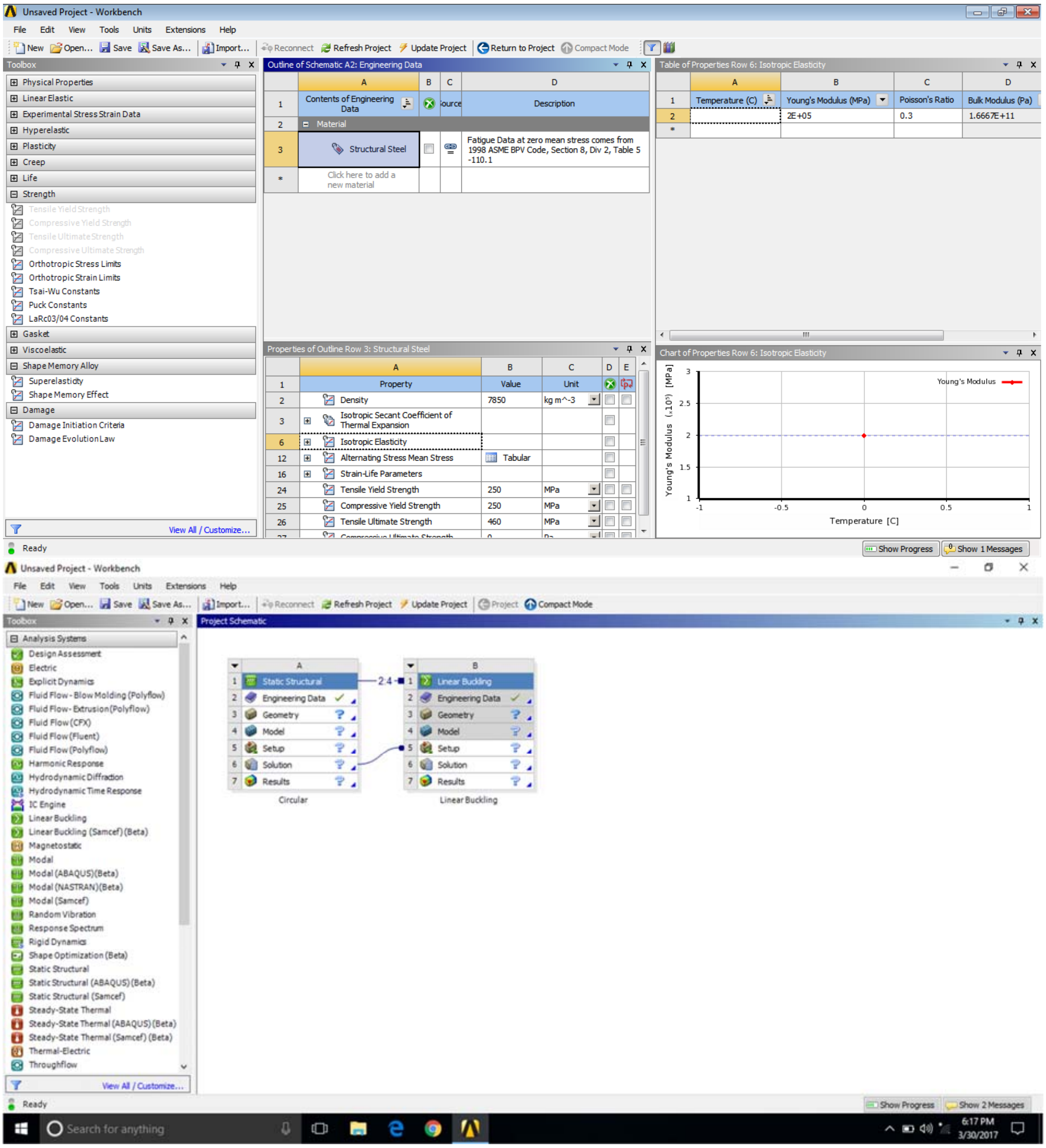
By ANSYS

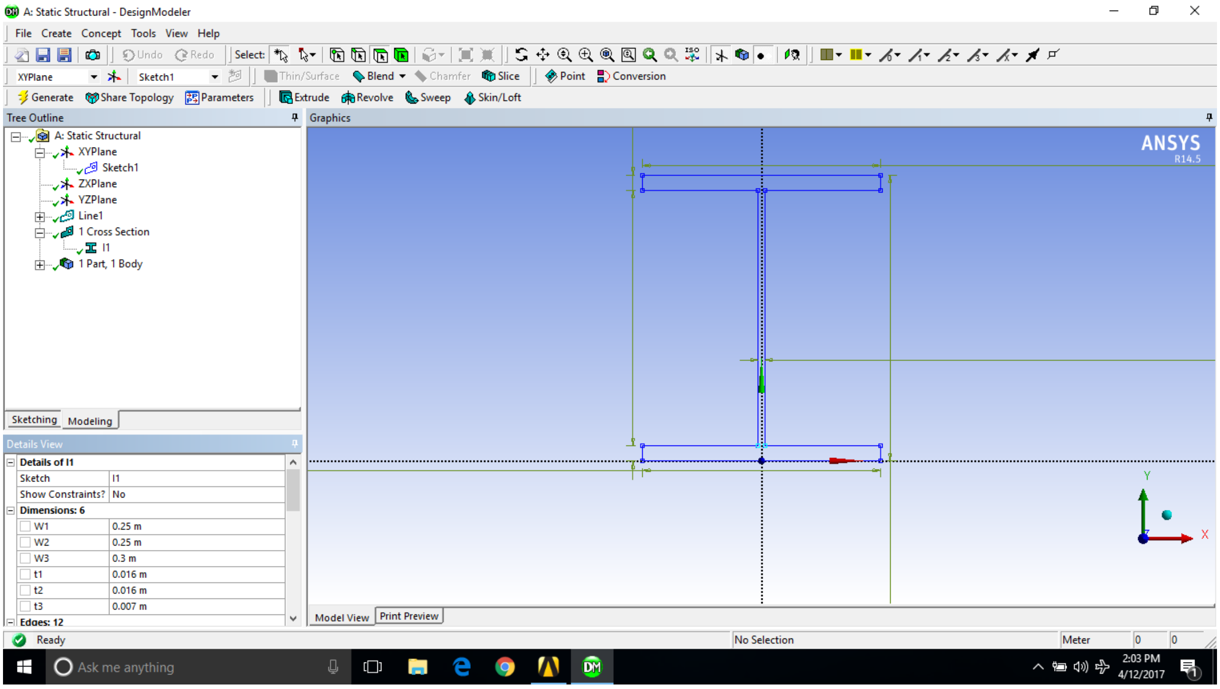
CROSS SECTION	APPLIED LOAD (KN)	LOAD MULTIPLIER	CRITICAL LOAD (KN)
ISHB 200	750	1.15	862.5
ISHB 250	900	0.98	882.0
ISHB 300	1200	0.933	1119.6
ISHB 350	1800	0.548	972.0

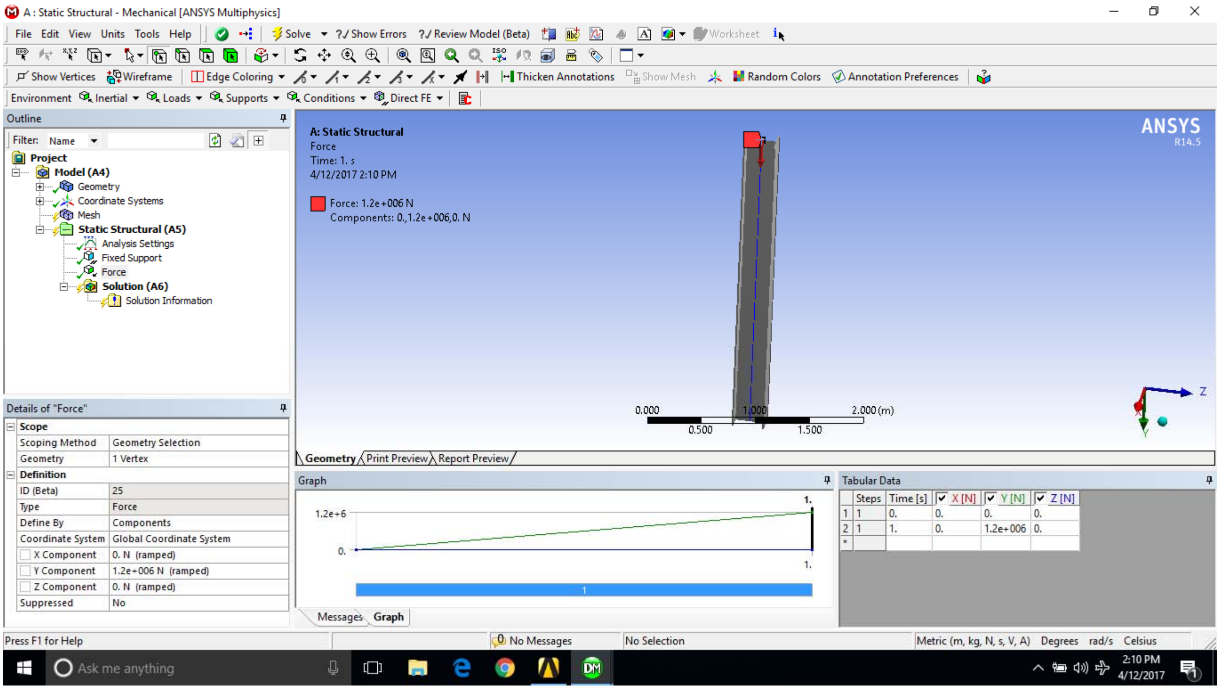


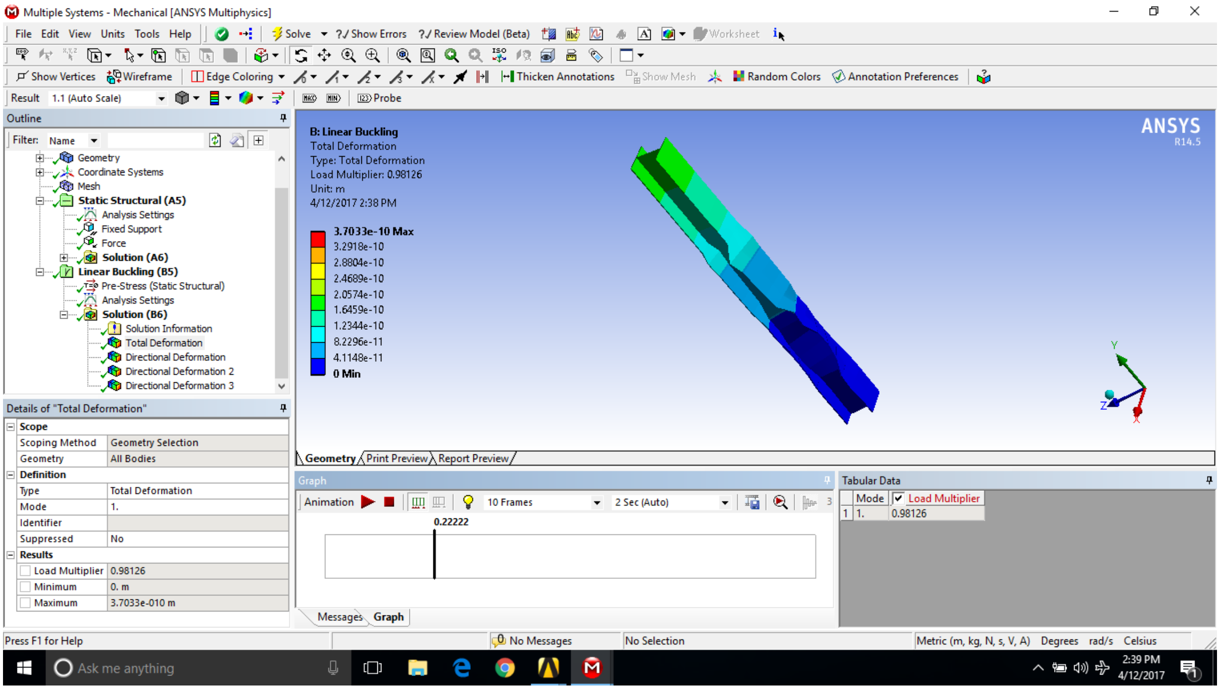
Chapter 5

ANSYS Steps



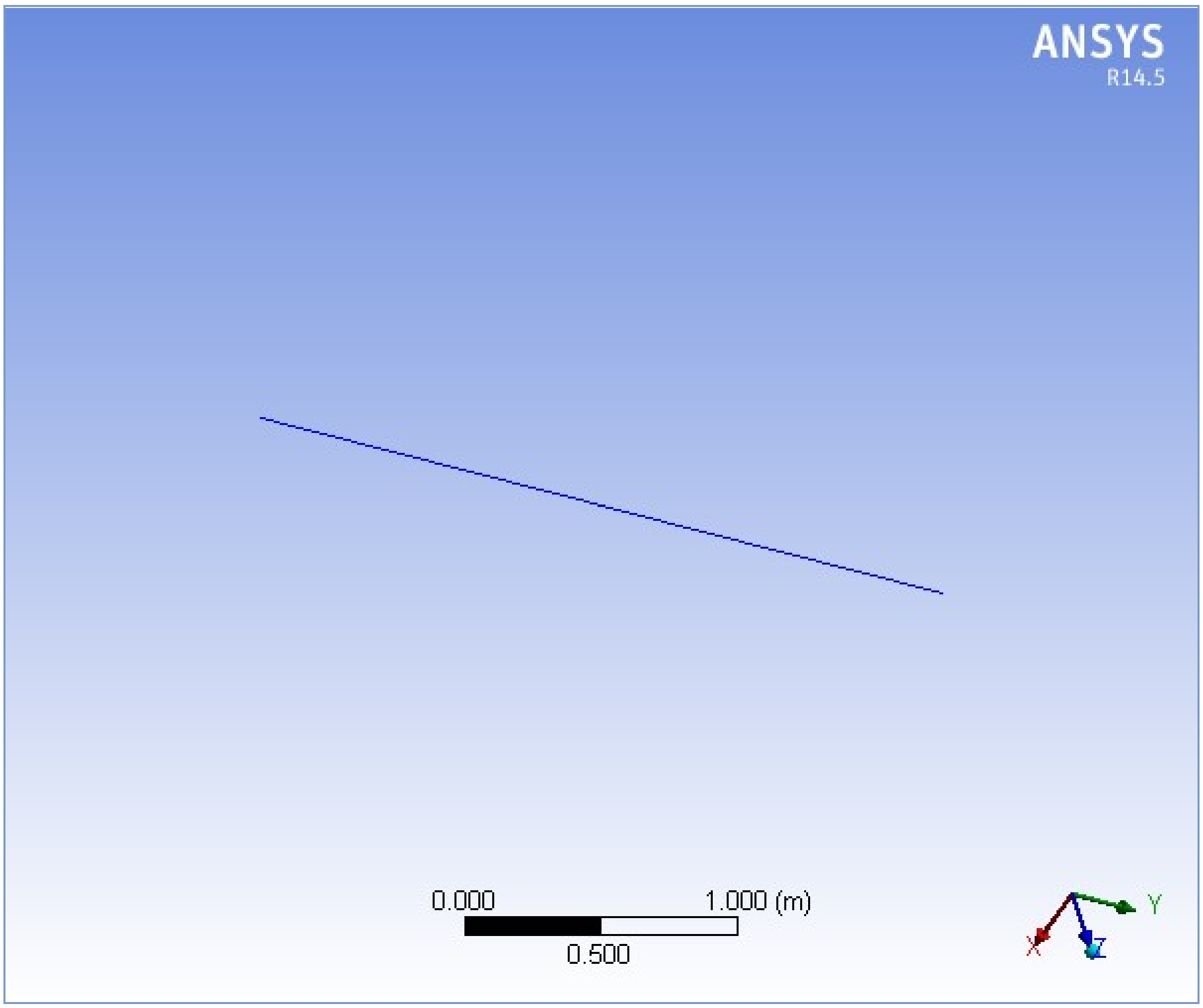






Project

First Saved	Wednesday, April 12, 2017
Last Saved	Wednesday, April 12, 2017
Product Version	14.5 Release
Save Project Before Solution	No
Save Project After Solution	No



Contents

Units

TABLE 1

Unit System	Metric (m, kg, N, s, V, A) Degrees rad/s Celsius
Angle	Degrees
Rotational Velocity	rad/s
Temperature	Celsius

Model (A4, B4)

Geometry

TABLE 2
Model (A4, B4) > Geometry

Object Name	*Geometry*
State	Fully Defined
Definition	
Source	C:\Users\pragyan\Desktop\i section_files\dp0\SYS\DM\SYS.agdb
Type	DesignModeler
Length Unit	Meters
Element Control	Program Controlled
Display Style	Body Color
Bounding Box	
Length X	0. m
Length Y	3. m

Length Z	0. m
Properties	
Volume	2.9628e-002 m³
Mass	232.58 kg
Scale Factor Value	1.
Statistics	
Bodies	1
Active Bodies	1
Nodes	43
Elements	21
Mesh Metric	None
Basic Geometry Options	
Parameters	Yes
Parameter Key	DS
Attributes	No
Named Selections	No
Material Properties	No
Advanced Geometry Options	
Use Associativity	Yes
Coordinate Systems	No
Reader Mode Saves Updated File	No
Use Instances	Yes
Smart CAD Update	No
Attach File Via Temp File	Yes
Temporary Directory	C:\Users\pragyan\AppData\Local\Temp
Analysis Type	3-D
Decompose Disjoint Geometry	Yes
Enclosure and Symmetry Processing	Yes

TABLE 3
Model (A4, B4) > Geometry > Parts

Object Name	*Line Body*
State	Meshed
Graphics Properties	
Visible	Yes
Glow	0
Shininess	1
Transparency	1
Specularity	1
Definition	
Suppressed	No
Coordinate System	Default Coordinate System
Reference Temperature	By Environment
Offset Mode	Refresh on Update
Offset Type	Centroid
Model Type	Beam
Material	
Assignment	Structural Steel
Nonlinear Effects	Yes
Thermal Strain Effects	Yes
Bounding Box	
Length X	0. m

Length Y	3. m
Length Z	0. m
Properties	
Volume	2.9628e-002 m³
Mass	232.58 kg
Length	3. m
Cross Section	I1
Cross Section Area	9.876e-003 m²
Cross Section IYY	1.7271e-004 m²·m²
Cross Section IZZ	4.1674e-005 m²·m²
Statistics	
Nodes	43
Elements	21
Mesh Metric	None

Coordinate Systems

TABLE 4
Model (A4, B4) > Coordinate Systems > Coordinate System

Object Name	*Global Coordinate System*
State	Fully Defined
Definition	
Type	Cartesian
Coordinate System ID	0.
Origin	
Origin X	0. m
Origin Y	0. m
Origin Z	0. m
Directional Vectors	
X Axis Data	[1. 0. 0.]
Y Axis Data	[0. 1. 0.]
Z Axis Data	[0. 0. 1.]

Mesh

TABLE 5
Model (A4, B4) > Mesh

Object Name	*Mesh*
State	Solved
Defaults	
Physics Preference	Mechanical
Relevance	0
Sizing	
Use Advanced Size Function	Off
Relevance Center	Coarse
Element Size	Default
Initial Size Seed	Active Assembly
Smoothing	Medium
Transition	Fast
Span Angle Center	Coarse
Minimum Edge Length	3.0 m
Inflation	

Use Automatic Inflation	None
Inflation Option	Smooth Transition
Transition Ratio	0.272
Maximum Layers	5
Growth Rate	1.2
Inflation Algorithm	Pre
View Advanced Options	No
Patch Conforming Options	
Triangle Surface Mesher	Program Controlled
Advanced	
Shape Checking	Standard Mechanical
Element Midside Nodes	Program Controlled
Number of Retries	Default (4)
Extra Retries For Assembly	Yes
Rigid Body Behavior	Dimensionally Reduced
Mesh Morphing	Disabled
Defeaturing	
Pinch Tolerance	Please Define
Generate Pinch on Refresh	No
Automatic Mesh Based Defeaturing	On
Defeaturing Tolerance	Default
Statistics	
Nodes	43
Elements	21
Mesh Metric	None

Static Structural (A5)

TABLE 6
Model (A4, B4) > Analysis

Object Name	*Static Structural (A5)*
State	Solved
Definition	
Physics Type	Structural
Analysis Type	Static Structural
Solver Target	Mechanical APDL
Options	
Environment Temperature	22. °C
Generate Input Only	No

TABLE 7
Model (A4, B4) > Static Structural (A5) > Analysis Settings

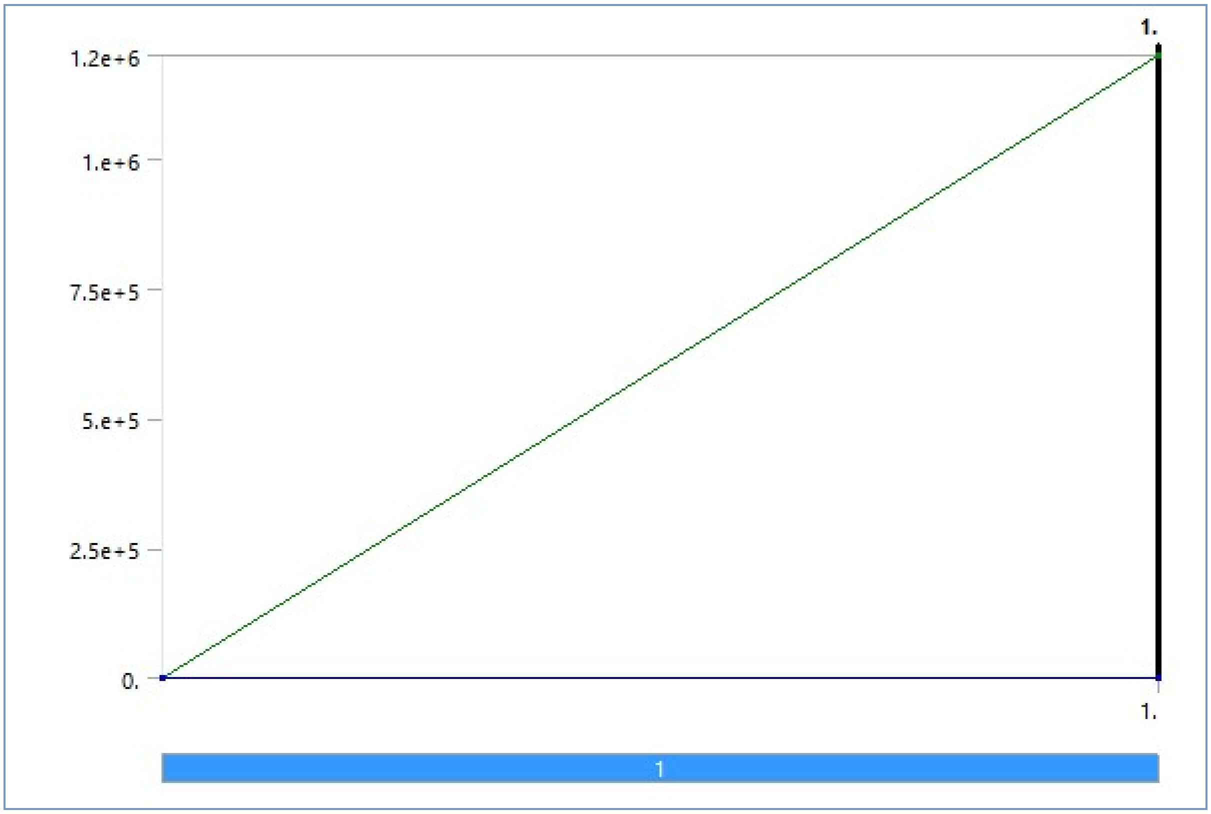
Object Name	*Analysis Settings*
State	Fully Defined
Restart Analysis	
Restart Type	Program Controlled
Status	Done
Step Controls	
Number Of Steps	1.
Current Step Number	1.
Step End Time	1. s
Auto Time Stepping	Program Controlled

Solver Controls	
Solver Type	Program Controlled
Weak Springs	Program Controlled
Large Deflection	Off
Inertia Relief	Off
Restart Controls	
Generate Restart Points	Program Controlled
Retain Files After Full Solve	Yes
Nonlinear Controls	
Force Convergence	Program Controlled
Moment Convergence	Program Controlled
Displacement Convergence	Program Controlled
Rotation Convergence	Program Controlled
Line Search	Program Controlled
Stabilization	Off
Output Controls	
Stress	Yes
Strain	Yes
Nodal Forces	No
Contact Miscellaneous	No
General Miscellaneous	No
Store Results At	All Time Points
Cache Results in Memory (Beta)	Never
Max Number of Result Sets	Program Controlled
Analysis Data Management	
Solver Files Directory	C:\Users\pragyan\Desktop\i section_files\dp0\SYS\MECH\
Future Analysis	Prestressed analysis
Scratch Solver Files Directory	
Save MAPDL db	No
Delete Unneeded Files	Yes
Nonlinear Solution	No
Solver Units	Active System
Solver Unit System	mks

TABLE 8
Model (A4, B4) > Static Structural (A5) > Loads

Object Name	*Fixed Support*	*Force*
State	Fully Defined	
Scope		
Scoping Method	Geometry Selection	
Geometry	1 Vertex	
Definition		
ID (Beta)	24	25
Type	Fixed Support	Force
Suppressed	No	
Define By		Components
Coordinate System		Global Coordinate System
X Component		0. N (ramped)
Y Component		1.2e+006 N (ramped)
Z Component		0. N (ramped)

FIGURE 1
Model (A4, B4) > Static Structural (A5) > Force



Solution (A6)

TABLE 9
Model (A4, B4) > Static Structural (A5) > Solution

Object Name	*Solution (A6)*
State	Solved
Adaptive Mesh Refinement	
Max Refinement Loops	1.
Refinement Depth	2.
Information	
Status	Done

TABLE 10
Model (A4, B4) > Static Structural (A5) > Solution (A6) > Solution Information

Object Name	*Solution Information*
State	Solved
Solution Information	
Solution Output	Solver Output
Newton-Raphson Residuals	0
Update Interval	2.5 s
Display Points	All
FE Connection Visibility	
Activate Visibility	Yes
Display	All FE Connectors
Draw Connections Attached To	All Nodes
Line Color	Connection Type
Visible on Results	No
Line Thickness	Single

Display Type	Lines

TABLE 11
Model (A4, B4) > Static Structural (A5) > Solution (A6) > Results

Object Name	*Directional Deformation*	*Directional Deformation 2*	*Directional Deformation 3*	*Total Deformation*
State	Solved			
Scope				
Scoping Method	Geometry Selection			
Geometry	All Bodies			
Definition				
Type	Directional Deformation			Total Deformation
Orientation	X Axis	Y Axis	Z Axis	
By	Time			
Display Time	Last			
Coordinate System	Global Coordinate System			
Calculate Time History	Yes			
Identifier				
Suppressed	No			
Results				
Minimum	0. m			
Maximum	0. m	1.8226e-003 m	0. m	1.8226e-003 m
Information				
Time	1. s			
Load Step	1			
Substep	1			
Iteration Number	1			

Linear Buckling (B5)

TABLE 12
Model (A4, B4) > Analysis

Object Name	*Linear Buckling (B5)*
State	Solved
Definition	
Physics Type	Structural
Analysis Type	Linear Buckling
Solver Target	Mechanical APDL
Options	
Generate Input Only	No

TABLE 13
Model (A4, B4) > Linear Buckling (B5) > Initial Condition

Object Name	*Pre-Stress (Static Structural)*
State	Fully Defined
Definition	
Pre-Stress Environment	Static Structural
Pre-Stress Define By	Program Controlled
Reported Loadstep	Last
Reported Substep	Last
Reported Time	End Time

Contact Status	Use True Status

TABLE 14
Model (A4, B4) > Linear Buckling (B5) > Analysis Settings

Object Name	*Analysis Settings*
State	Fully Defined
Options	
Max Modes to Find	1.
Output Controls	
Stress	No
Strain	No
General Miscellaneous	No
Cache Results in Memory (Beta)	Never
Analysis Data Management	
Solver Files Directory	C:\Users\pragyan\Desktop\i section_files\dp0\SYS-1\MECH\
Future Analysis	None
Scratch Solver Files Directory	
Save MAPDL db	No
Delete Unneeded Files	Yes
Solver Units	Active System
Solver Unit System	mks

Solution (B6)

TABLE 15
Model (A4, B4) > Linear Buckling (B5) > Solution

Object Name	*Solution (B6)*
State	Solved
Adaptive Mesh Refinement	
Max Refinement Loops	1.
Refinement Depth	2.
Information	
Status	Done

FIGURE 2
Model (A4, B4) > Linear Buckling (B5) > Solution (B6)

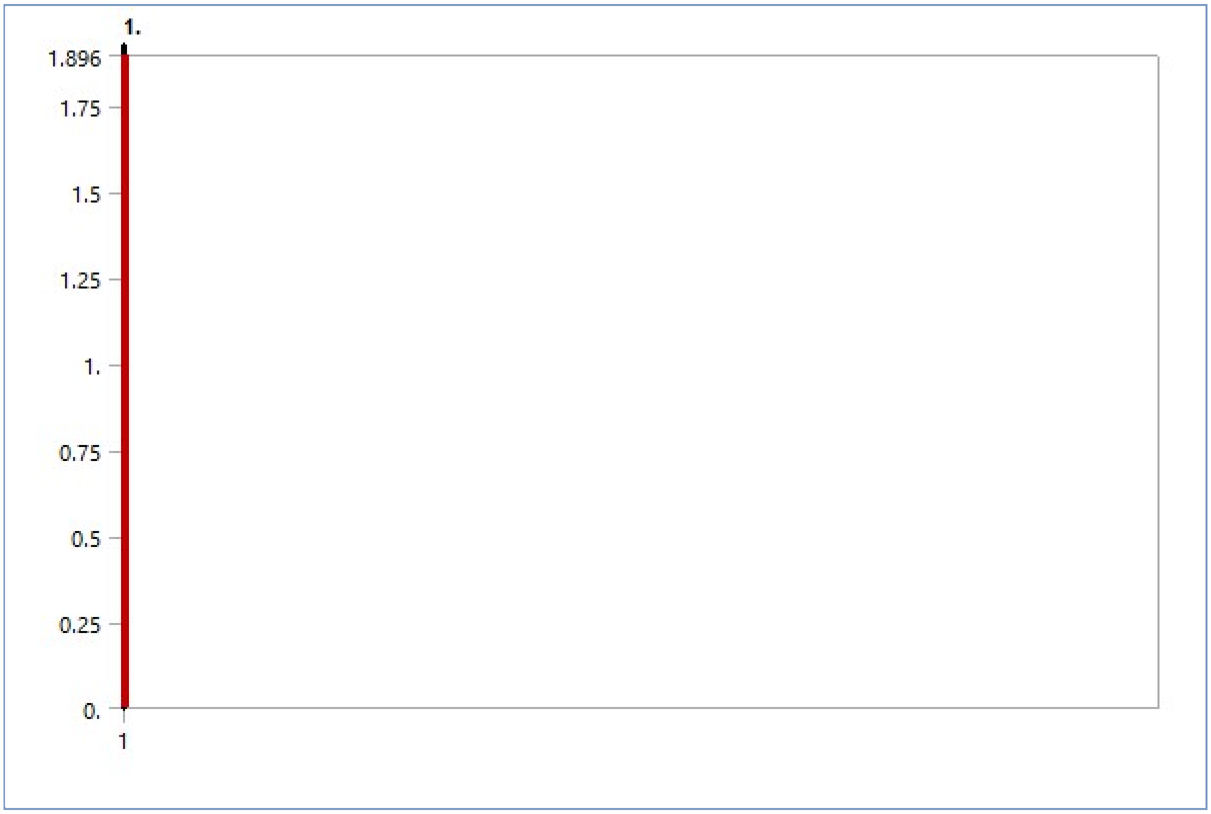


TABLE 16
Model (A4, B4) > Linear Buckling (B5) > Solution (B6)

Mode	Load Multiplier
1.	1.896

TABLE 17
Model (A4, B4) > Linear Buckling (B5) > Solution (B6) > Solution Information

Object Name	*Solution Information*
State	Solved
Solution Information	
Solution Output	Solver Output
Newton-Raphson Residuals	0
Update Interval	2.5 s
Display Points	All
FE Connection Visibility	
Activate Visibility	Yes
Display	All FE Connectors
Draw Connections Attached To	All Nodes
Line Color	Connection Type
Visible on Results	No
Line Thickness	Single
Display Type	Lines

TABLE 18
Model (A4, B4) > Linear Buckling (B5) > Solution (B6) > Results

Object Name	*Total Deformation*	*Directional Deformation*	*Directional Deformation 2*	*Directional Deformation 3*
State	Solved			
Scope				

Scoping Method	Geometry Selection			
Geometry	All Bodies			
Definition				
Type	Total Deformation	Directional Deformation		
Mode	1.			
Identifier				
Suppressed	No			
Orientation		X Axis	Y Axis	Z Axis
Coordinate System		Global Coordinate System		
Results				
Load Multiplier	1.896			
Minimum	0. m		-1.2604e-012 m	-3.5421e-009 m
Maximum	1. m		1.3834e-012 m	2.4209e-009 m

TABLE 19
Model (A4, B4) > Linear Buckling (B5) > Solution (B6) > Total Deformation

Mode	Load Multiplier
1.	1.896

TABLE 20
Model (A4, B4) > Linear Buckling (B5) > Solution (B6) > Directional Deformation

Mode	Load Multiplier
1.	1.896

TABLE 21
Model (A4, B4) > Linear Buckling (B5) > Solution (B6) > Directional Deformation 2

Mode	Load Multiplier
1.	1.896

TABLE 22
Model (A4, B4) > Linear Buckling (B5) > Solution (B6) > Directional Deformation 3

Mode	Load Multiplier
1.	1.896

Material Data

Structural Steel

TABLE 23
Structural Steel > Constants

Density	7850 kg m^-3
Coefficient of Thermal Expansion	1.2e-005 C^-1
Specific Heat	434 J kg^-1 C^-1
Thermal Conductivity	60.5 W m^-1 C^-1
Resistivity	1.7e-007 ohm m

TABLE 24
Structural Steel > Compressive Ultimate Strength

Compressive Ultimate Strength Pa
0

TABLE 25
Structural Steel > Compressive Yield Strength

Compressive Yield Strength Pa
2.5e+008

TABLE 26
Structural Steel > Tensile Yield Strength

Tensile Yield Strength Pa
2.5e+008

TABLE 27
Structural Steel > Tensile Ultimate Strength

Tensile Ultimate Strength Pa
4.6e+008

TABLE 28
Structural Steel > Isotropic Secant Coefficient of Thermal Expansion

Reference Temperature C
22

TABLE 29
Structural Steel > Alternating Stress Mean Stress

Alternating Stress Pa	Cycles	Mean Stress Pa
3.999e+009	10	0
2.827e+009	20	0
1.896e+009	50	0
1.413e+009	100	0
1.069e+009	200	0
4.41e+008	2000	0
2.62e+008	10000	0
2.14e+008	20000	0
1.38e+008	1.e+005	0
1.14e+008	2.e+005	0
8.62e+007	1.e+006	0

TABLE 30
Structural Steel > Strain-Life Parameters

Strength Coefficient Pa	Strength Exponent	Ductility Coefficient	Ductility Exponent	Cyclic Strength Coefficient Pa	Cyclic Strain Hardening Exponent
9.2e+008	-0.106	0.213	-0.47	1.e+009	0.2

TABLE 31
Structural Steel > Isotropic Elasticity

Temperature C	Young's Modulus Pa	Poisson's Ratio	Bulk Modulus Pa	Shear Modulus Pa
	2.e+011	0.3	1.6667e+011	7.6923e+010

TABLE 32
Structural Steel > Isotropic Relative Permeability

Relative Permeability
10000

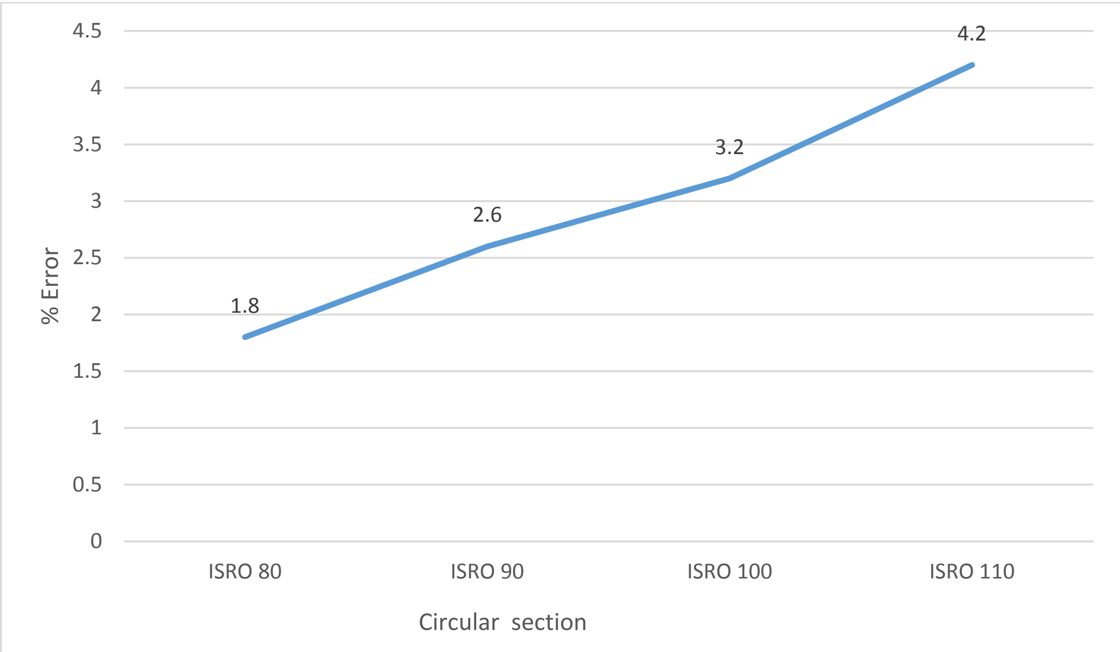
CHAPTER 6

Conclusion

% Error

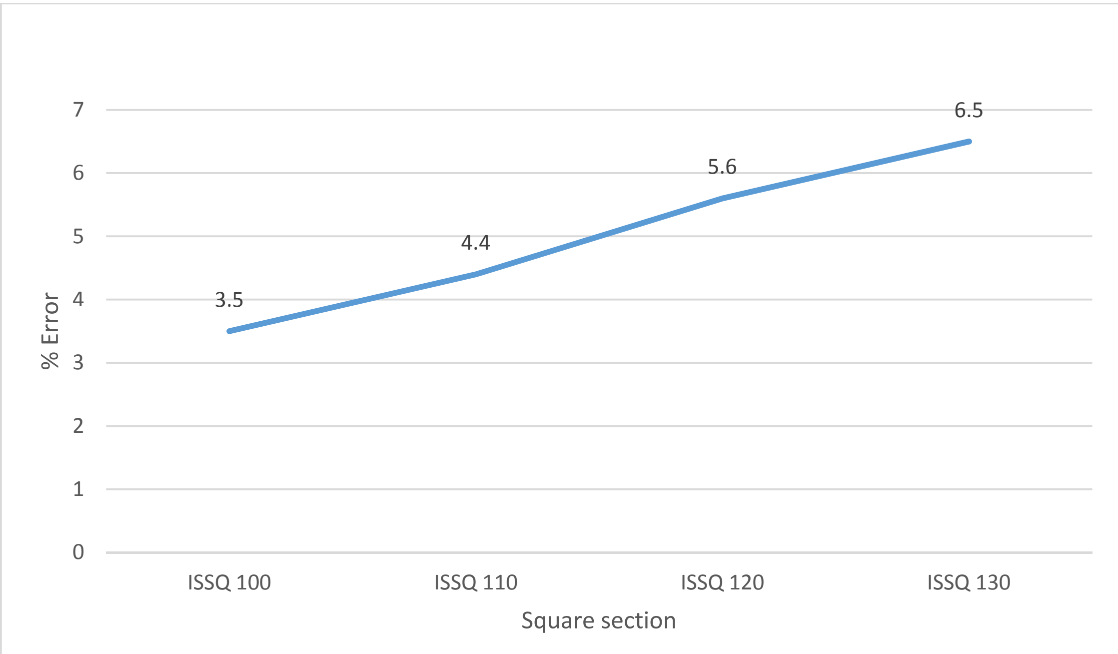
For circular section

CROSS SECTION	BY IS 800-2007	BY ANSYS	% ERROR
ISRO 80	242.79	247.35	1.8
ISRO 90	382.174	396.11	2.6
ISRO 100	584.07	603.20	3.2
ISRO 110	845.952	882.90	4.2



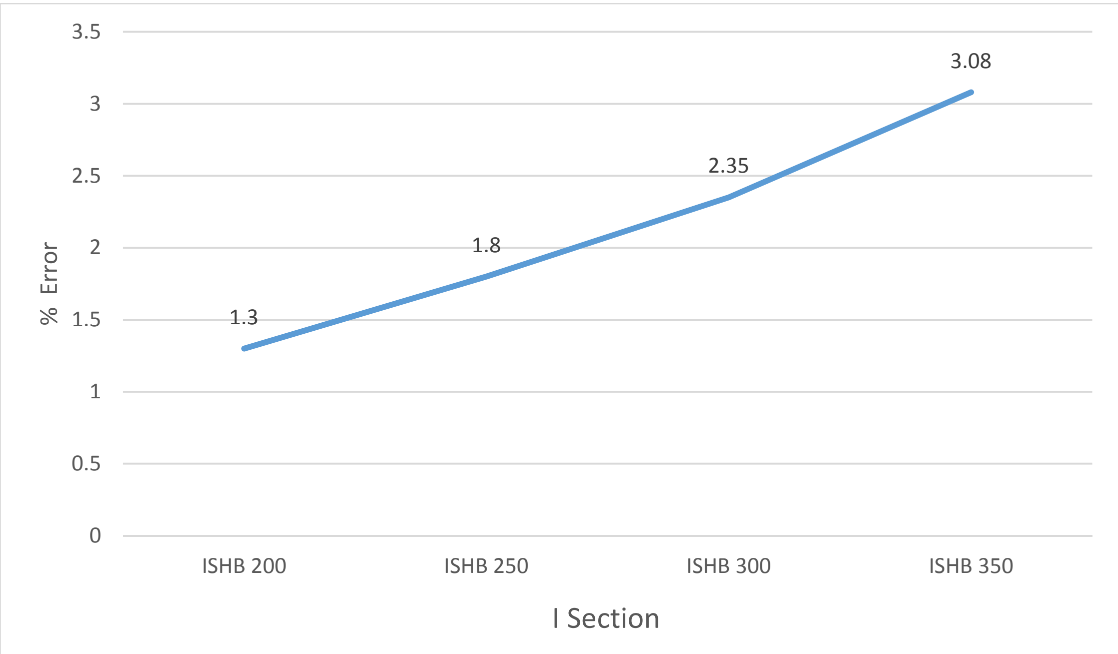
For Square section

CROSS SECTION	BY IS 800-2007	BY ANSYS	% ERROR
ISSQ 100	990.72	1026.3	3.5
ISSQ 110	1436.36	1500	4.4
ISSQ 120	2012.43	2127.05	5.6
ISSQ 130	2698.94	2927.5	6.5



For I section

CROSS SECTION	BY IS 800-2007	BY ANSYS	% ERROR
ISHB 200	851.102	862.5	1.3
ISHB 250	865.88	882.0	1.8
ISHB 300	1093.86	1119.6	2.35
ISHB 350	956.92	972.0	3.08



As per above results of some problems, we find out the critical load according to IS 800-2007 and ANSYS. The percentage error is generally lies within 2% to 3%. According to IS 800-2007 the valve of critical load is less then value calculated on ANSYS. As the geometry of the member increases the percentage error is increases.

Some of the points based on results are conclude below:-

1. The strength of compression members made of such sections depends on their slenderness ratio. Higher strengths can be obtained by reducing the slenderness ratio *i.e.* by increasing the moment of inertia of the cross-section.
2. If an open section such as I-section column subjected to uniform compression, it is pointed out the flanges which are outstands tend to buckle before the webs which are supported along all edges.
3. In closed sections such as the hollow rectangular section, both flanges and webs behave as internal elements and the local buckling of the flanges and webs depends on their respective width-thickness ratios. In this case also, local buckling occurs along the entire length of the member.
4. In the case of beams, the compression flange behaves as a plate element subjected to uniform compression and, depending on whether it is an outstand or an internal element, undergoes local buckling at the corresponding critical buckling stress. However, the web is partially under compression and partially under tension. Even the part in compression is not under uniform compression. therefore, the web buckles as a plate subjected to in-plane bending compression. Normally, the bending moment varies over the length of the beam and so local buckling may occur only in the region of maximum bending moment.
5. Local buckling has the effect of reducing the load carrying capacity of columns and beams due to the reduction in stiffness and strength of the locally buckled plate elements. Therefore, it is desirable to avoid local buckling before yielding of the member. Most of the hot rolled steel sections have enough wall thickness to eliminate local buckling before yielding.

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