COMPARATIVE ANALYSIS OF TRUSS MEMBERS ACCORDING TO DESIGN PARAMETER OF IS 800:2007 AND IS 800:1984

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by

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CERTIFICATE

This is to certify that the project entitled **“COMPARATIVE ANALYSIS OF TRUSS MEMBERS ACCORDING TO DESIGN PARAMETER OF IS 800:2007 AND IS 800:1984”** is a bonafide work of **Ms. Shaikh Tarannum Rasheed, Ms. Maniyar Shifa Dayim. A., Mr. Ansari Manauvar Ali K., Mr. Ansari Shahbaz Alam** submitted to the University of Mumbai in partial fulfilment of the requirement for the award of the degree of **Bachelor of Engineering** in **Civil Engineering** course conducted by university of Mumbai in Anjuman-I-Islam’s Kalsekar Technical Campus, Navi-Mumbai.

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DECLARATION

We declare that this written submission represents my ideas in our own words and where other ideas or words have been included, we have adequately cited and referenced the original sources. We also declare that, we have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea/data/fact/source in our submission. I understand that any violation of the above will be cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

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ABSTRACT

The following will evaluate the study conducted on structural components ie. Tension member and compression member in both limit state method as well as working state method. The two methods have been compared in this study by designing members in both tension as well as compression. there are different sections used in the study to get the difference. The sections are used is from the IS steel table. The assumptions in both the methods which made us study them to acquire a conclusion which will be beneficial for engineers like us to know the economy of the two and which one of them to bring in to practice, especially for steel structures. Since steel structures is a task in our country, this study will be a boon for future steel structures. The design of the components is done using excel sheet which will give a graphical result, which is then compared to check the economy of the two method. The use of excel sheet for the study is because of its excellent accuracy, time consumption, and the ease it gives for the parallel study of the two methods, which we do not get using any other electronic spreadsheet. The two codes are used for the designing purpose of the components which are IS 800:1984 and IS 800: 2007.

Keywords: compression member, tension member, IS 800:1984, IS 800:2007, WSM, LSM, spreadsheet.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF PARAMETERS

Sr No	Designation	Name of parameters
1.	A	Cross-sectional area
2.	A_g	Gross cross-sectional area
3.	A_{g0}	Gross area of the outstanding leg
4.	A_n	Net cross-sectional area
5.	A_1	Effective net cross-sectional area of the connected leg
6.	A_2	The gross cross-sectional area of the unconnected (outstanding) leg
7.	B	Projection of the base plate beyond the column
8.	D	Gross diameter of rivet
9.	f_u	Ultimate stress of the material;
10.	f_y	Design strength of the member
11.	G	Gauge length between the bolt holes
12.	I	Subscript for summation of all the inclined legs
13.	k_1,k_2,k_3	Constants depending upon the end condition
14.	L	Length of the member
15.	L_c	Effective length of the member
16.	T	Thickness of plate
17.	T_d	Design strength of the member
18.	N	Number of bolt holes in the critical section
19.	P	Staggered-pitch length between line of bolt holes
20.	R	Radius of gyration of section
21.	r_y	Radius of gyration about the minor axis
22.	d_h	Diameter of the bolt hole
23.	α	Imperfection factor
24.	γ_{m0}	Partial safety factor for failure in tension by Yielding
25.	γ_{m1}	Partial safety factor for failure at ultimate stress
26.	σ_{ac}	Permissible stress in axial compression in MPa
27.	σ_{at}	Stress in axial tension in mpa
28.	ε	Yield stress ratio $\sqrt{\frac{250}{f_y}}$
29.	λ	Non-dimensional effective slenderness ratio
30.	f_{cc}	Elastic critical stress in compression $= \frac{\Pi^2 E}{\Lambda^2}$
31.	f_y	Yield stress of steel in MPa

Chapter 1

Introduction

1.1 General

Steel, as a building material has been used extensively in various types of structures. Some of the examples of civil engineering works in the steel are industrial buildings, high raised building skeletons, transmission towers, railway bridges, overhead tanks, chimneys (stacks), towers, bunkers and silos, etc. Steel structures are composed of elements which are rolled to a basic cross section in a mill, and work to the desired size and form in a fabricating shop or site. Indian hot-rolled I-sections & Channel sections are found to be used commonly in India. Structural steel has several advantages over other competing materials (for example concrete and wood), such as high strength to weight ratio, high ductility, uniformity, and its ability to be fully recyclable. Ductility and toughness are very important when steel is subjected to earthquake loads or impact loads (Kulkarni, R. B. and Jirage, R. S.). It offers much better compressive and tensile strength than concrete

The design of steel structure comprises of the planning of different structures for specific purpose. The structure should serve the intended purpose during the design life, and this is accomplished by proper functional planning. With an appropriate degree of safety, they should withstand all the loads and deformations, during construction and use and have adequate resistance to certain expected accidental loads and fatigue. Structure should be stable and have alternate load paths to prevent disproportionate overall failure under accidental loading. (IS 800:2007).

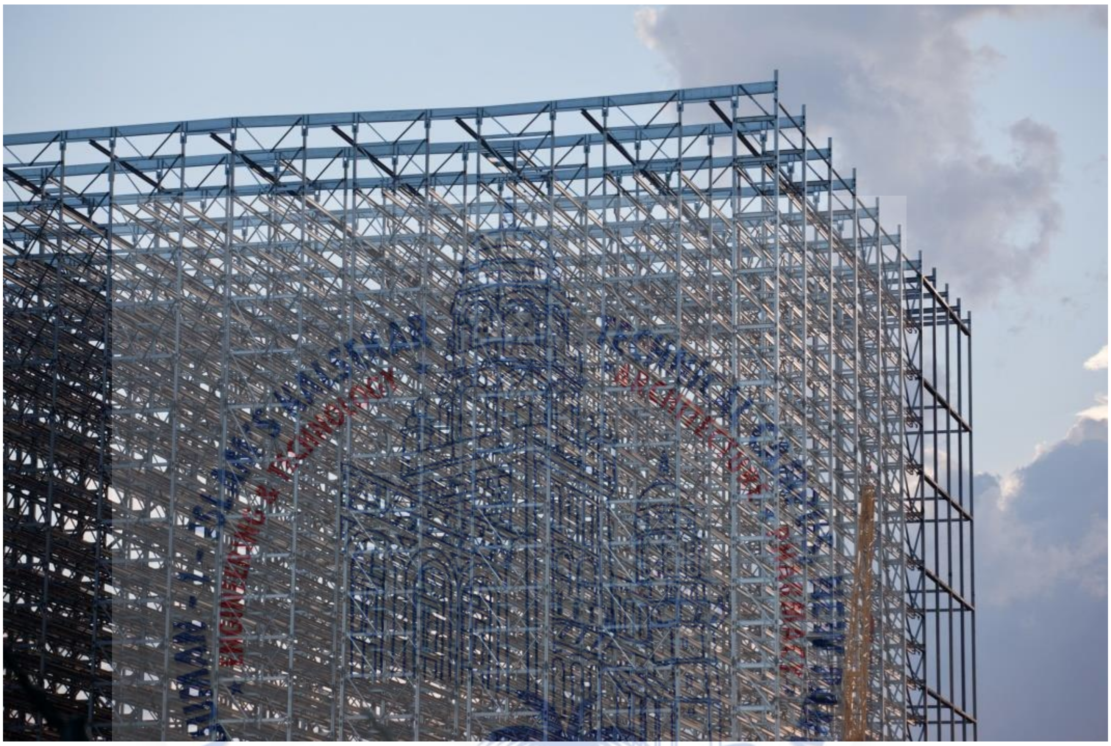


Figure 1.1: Steel Structure

Each and every element of a structure has its own property and own design constraints. Each component plays its own role in withstanding the applied load(s) and aids to the overall performance of the structure. Three major factors in steel structure design are stability, strength and ductility of individual members. Apart from these, connections play an important role in the overall performance of the structure. Even though the structural members are adequately designed, inadequate connections can result in failure of the structure. Because of the above reasons, design of steel structures and its elements has become an important aspect of structural engineering. A proper design considering these, together with collapse mechanism under strong seismic shaking results in good overall performance of the structure.

The design methodologies for the steel structures are working stress design method and limit state design method. The codes published by the Bureau of Indian Standard, for the design of steel structures are IS 800: 2007 and IS 800:1984. Steel design codes are consistently developing through the years to meet the needed execution of the structural components. The most recent edition of the Code of Practice for general construction in steel is IS 800:2007, which is based on Limit State Method of design. The design concept is totally changed in comparison to earlier IS 800:1984 which is based on elastic method. The provisions in this section are applicable to the steels commonly used in steel construction, namely, structural mild steel and high tensile structural steel.

1.2 Tension Members

A structural member subjected to two pulling (tensile) forces applied at its ends is called a tension member. The stress in such members is assumed to be uniformly distributed over the net section and hence members subjected only to axial tension are supposed to be the most efficient and economical. However, if some eccentricity exists either due to the member not being perfectly straight or due to eccentricity in connections, either bending stresses are considered in the design and specifications are provided to account for reduction in the net area. The strength of these members is influenced by several factors such as the length of connection, size and spacing of fasteners, net area of cross section, type of fabrication, connection eccentricity and shear lag at the end connection. Figure 1.2 shows a tension member under tension.

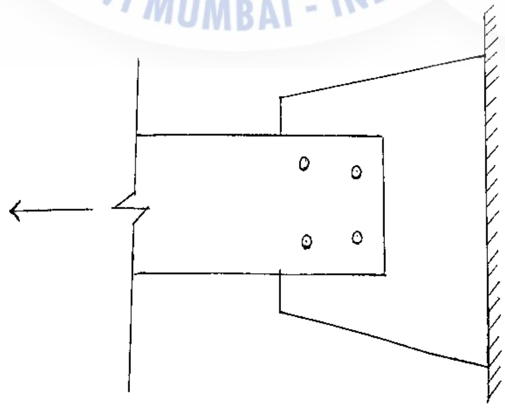


Figure 1.2: Member under tension

Tension members are generally found in bridge trusses, roof trusses, towers, bracing systems and as the rods. Tension members are also found in Howe truss as well as in

Pratt truss. Tension members with bolted end connections are frequently used in trusses and lateral bracing systems, and three limit states are normally considered in the design of the members. When angle is used as Tension Member then single bolt is used.

1.2.1 Modes of Failure of tension member:

Gross section yielding: A tension member can fail by reaching one of the following two limit states viz. excessive deformation or fracture. In general, a tension member is strong enough to withstand loads up to ultimate load without failure. A tension member deforms considerably in the longitudinal direction before fracture and becomes unserviceable. This can be explained as follows.

Although steel tension members can sustain loads up to the ultimate load without failure, the elongation of the members at this load would be nearly 10-15% of the original length and the structure supported by the member would become unserviceable. Hence, in the design of tension members, the yield load is usually taken as the limiting load.

When a tension member is subjected to tensile force although the net cross section yields first, the deformation within the length of connection will be smaller than deformation in the remainder of tension member. It is because the net section exists within a small length of the member, and the total elongation is the product of the length of the member and the strain. Most of the members will not have the reduced cross section, so attainment of yield stress on the gross area will result in larger total elongation. It is the larger deformation, not the first yield that is the limit state. This makes one of the limit states of strength corresponding to the yielding of gross cross section. To prevent excessive deformation, initiated by yielding, the load on the gross section must be small enough so that the stress on the gross section is less than the yield stress.

Net section yielding: Frequently plates under tension have bolt holes. The tensile stress in a plate at the cross section of a hole is not uniformly distributed in the Tension Member but exhibits stress concentration adjacent to the hole. The ratio of the maximum elastic stress adjacent to the hole to the average stress on the net cross section is referred to as the Stress Concentration Factor. This factor is in the range of 2 to 3, depending upon the ratio of the diameter of the hole to the width of the plate normal to the direction of stress.

At service loads, these holes cause stress concentration adjacent to bolt holes (P/A). The stress at these places can be as high as three times the average stress and about more than two times the average stress on net area at fillets of rolled shapes. Because of the inherent ductility of structural steel, this localized over stress is usually neglected. The ratio of maximum elastic stress (f_{max}) to average stress (f_{av}) is known as stress concentration factor. The stress concentration, however, may be minimized by providing a suitable joint and member details.

When a tension member with a hole is loaded statically, the fibres adjacent to the hole yield due to stress concentration. On increasing the load, the redistribution of stress takes place and additional stress is transferred to adjacent areas of cross section. Stress in the fibres adjacent to holes remain constant, f_y and the fibres away from the hole progressively reach yield stress f_y as shown in Fig.(b). It is the ductility of steel that permits the initially yielded zone to deform without fracture as stress on remainder of the cross section continues to increase until finally rupture of the member occurs. At this stage the entire net section reaches the ultimate stress f_u due to prevent failure of the tension member by net section rupture,

$$T < A_n f_u$$

$$T_{dn} = \frac{T}{\gamma_{m1}} = \frac{A_n f_u}{\gamma_{m1}}$$

$$T_{dn} = 0.9\frac{A_n f_u}{\gamma_{m1}}$$

Block shear failure: This type of failure is characterized by tearing out of a segment or block of material at the end of a member for certain connection configurations and in coped beams. The block shear failure occurs along a path involving tension on one plane and shear on a perpendicular plane. Figure 1.2 shows block shear failure of a tension member. When applied tensile load is increased, the fracture strength of the weaker plane is approached. However, this plane does not fail as it is restrained by the stronger plane. The load can still be increased until the fracture strength of the stronger plane is reached. During this time, the weaker plane is yielding. The total strength equals the fracture strength of the stronger plane plus the yield strength of weaker plane.

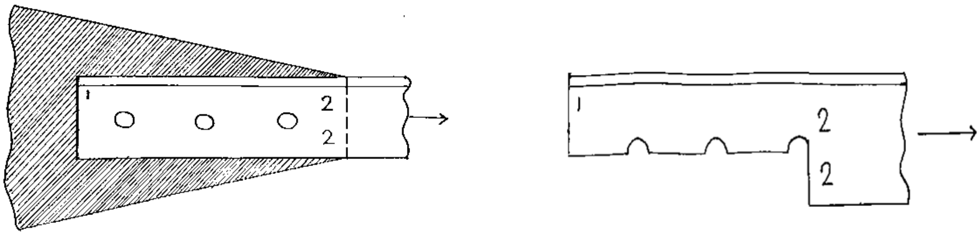


Figure 1.3: block shear failure of an angle.

A bolted connection of angle to gusset plate shows a welded connection for the same. Thus, for the tension member, when the surface fails by fracture of gross shear surface accompanied by yielding on the net tension surface, the shaded block will tend to tear out. One of the has a large shear area and a small tensile area. Thus, the primary resistance to a block shear failure is shearing and not tensile in this case. Therefore, it is assumed that when shear fracture occurs on this large shear resisting area, the small tensile area has yielded. The end block may also tear out by fracture of the tension surface and yielding of the shear surface. For the welded connection, the member has a large tensile area and a small shear area. For this case the primary resisting force against a block shear will be tensile and not shearing. Thus, a block shear failure cannot occur until the tensile area fractures; it is logical to assume that shear area has yielded. Therefore, for the block shear failure to occur, one of the surface fractures while the other yields. Because the limit state is fracture, the controlling equation for block shear will be the one that contains the larger fracture term. The block shear strength at an end connection may be expressed as follows.

For Plates

For shear yield and tension fracture

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + 0.9\frac{A_{tn} f_u}{\gamma_{m1}}$$

For tension yield and shear fracture

$$T_{db2} = \frac{A_{tg} f_y}{\gamma_{m0}} + 0.9\frac{A_{vn} f_u}{\sqrt{3}\gamma_{m1}}$$

where,

A_{vg} and A_{vn} = minimum gross and net areas in shear along the line of action of force, respectively.

A_{tg} and A_{tn} = minimum gross and net areas in tension from the hole to the toe of the angle o next last row of bolts in plates perpendicular to the line of force, respectively.

1.3 Compression Members

A compression member is a structural member which is straight and subjected to two equal and opposite forces applied at its ends. It can be also defined as a structural member which is subjected to compressive forces which tend to decrease its length. Different terms are used for a compression member depending upon its position in structures. Compression members are very important components for any building. All the kinds of loads such as dead load or live load are ultimately transferred to the columns (compression members) which in turn transfer it to the foundation. Thus, a column can be considered to be the main supporting unit for any kind of structure.

Strut is compression member used in the roof truss and bracing. They are of a small span and may be vertical or inclined. Column, stanchion or post is a vertical compression member supporting floors or girders in a building. These compression members are subjected to heavy loads. The principal rafter is a top chord member in roof truss and boom is the principal compression member in a crane. Stability plays an important role in the design of compression members. Ordinary structural analysis is based on the condition of stable equilibrium between internal and external forces, and a linear relationship is assumed to exist between stress and strain. However, when buckling is involved, it is necessary to investigate the potentially unstable equilibrium between the external and internal responses that are further complicated by the complex stress-strain relationship of the material extending from elastic to inelastic range. The term unstable used here pertains to a condition in which the slightest increment of deflection results in a further increase, which may lead to collapse of structure.

Columns are sometimes classified as long, short, or intermediate. The column whose lateral dimension is very small when compared to its length or height then it is called as a long column. The ratio of effective length to the least radius of gyration is greater than 45 and the load carrying capacity of a long column is less than a short column. Excessive compression of long columns may cause yielding or buckling. It can fail due to yielding if it is absolutely straight, has perfectly homogeneous material, concentric loads and no initial residual stresses. These are ideal conditions which may never exist in an actual structure. As compressive loading of a column is increased, it eventually causes some eccentricity. This in turn sets up some bending moment, causing the column to deflect or buckle slightly. This deflection increases the eccentricity and thus the bending moment. This may progress to where the bending moment is increasing at a rate greater than the increase in load, and the column soon fails by buckling. In general, long columns fail by elastic buckling, intermediate columns by inelastic buckling, and very short columns usually fail by crushing or yielding. At the point of failure, the stress in a long column will not exceed the proportional limit and it may be much lower than this limit for a very slender column. Failure of the intermediate column occurs after the extreme fibres have reached the yield point. A very short column is not really column as such but is considered to be a block without buckling. A short column under applied axial force is subjected to compressive strain. The column shortens in the direction of applied force. Figure 4.1 (a) shows a short column under compression and figure 4.1 (b) shows a long column under compression.

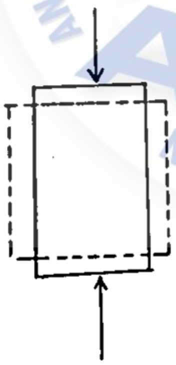


Figure 1.4 (a)
Short column under compression

Figure 1.4 (b)
Long column under compression

In any, some maximum compressive stress can be set as a limit of strength and an allowable working stress is chosen accordingly. Also, it is logical to apply large factor of safety to a long column and a smaller factor of safety to a shorter one. It should be taken note of that the terms long, intermediate and short columns are only relative. They are defined by the interpretation of their slenderness ratio.

1.3.1 Failure modes of compression member:

Following are the various types of failure of compression members under axial force:

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**.**

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

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

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

1.3.2 Theoretical Background

Effective Length (Equivalent Length): The length of the column which bends as if it is hinged (pin jointed) at both ends is called effective length. It is denoted by *l*. Effective length of a column depends upon its end conditions. The effective length is derived from the actual length L of the column. Effective length for different end conditions will be taken according to IS: 800-2007 **as follows:**

Buckling: Generally, buckling refers to an event whereby a compression member diverges from its linear elastic behaviour. A structure can be deemed unusable or can be considered to have and large deformation accompanied by change of member shape due to a very small increase in loading.

Table 1.1 Effective Lengths as per IS-800 2007

Boundary Conditions				Effective Length
At one end		At the other end		KL
Translation	Rotation	Translation	Rotation	
Restrained	Restrained	Free	Free	2.0L
free	restrained	restrained	free	2.0L
Restrained	free	restrained	free	1.0L
restrained	restrained	free	restrained	1.2L
Restrained	restrained	restrained	Free	0.8L
Restrained	restrained	restrained	restrained	0.65L

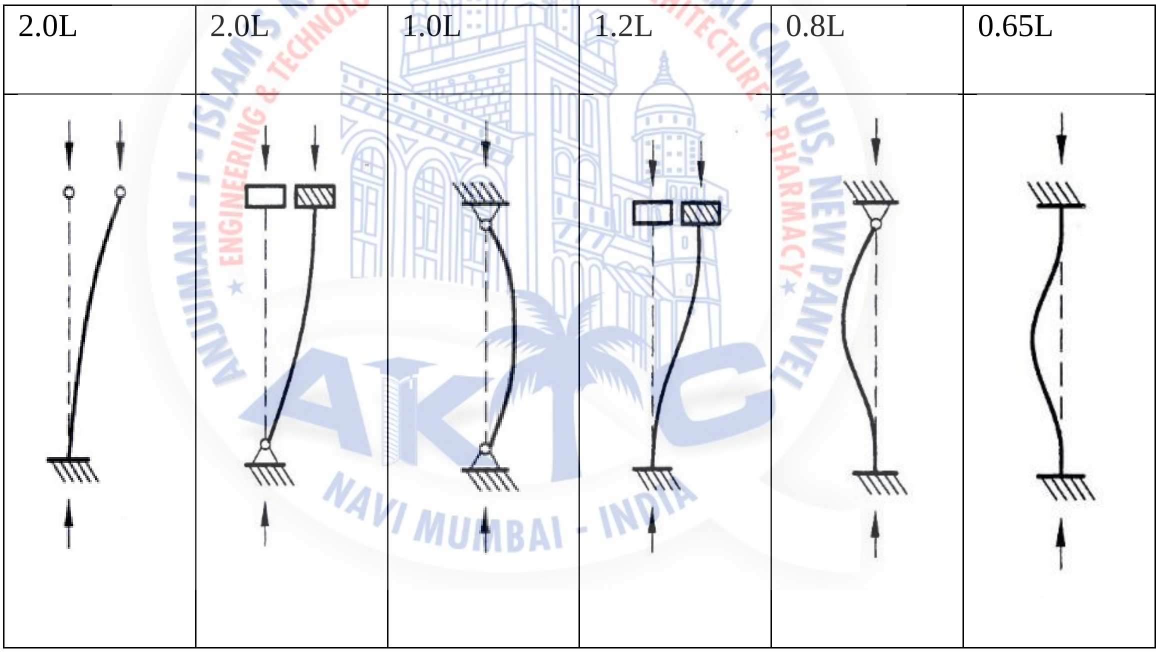


Figure 1.5: Effective Length for different end conditions

Slenderness Ratio: It is defined as the ratio between the effective length of compression member and its least radius of gyration.

Slenderness ratio = l / r

Where,

l = effective length of compression member

r = least radius of gyration of section of a member.

Radius of gyration is the property of a section. It is always worked out with reference to a certain axis by the expression:

$$r = \sqrt{I/A}$$

Where,

I = Moment of inertia of the section.

A = Area of the section

The slenderness ratio is very important factor deciding upon the x-sectional dimensions of compression members. When slenderness is too high, the x-section of member is small. It will show visible deflection under load. Thus, to avoid slim (thin and long) looking members, the slenderness ratio should be within specified limits depending upon nature of load. Also, if the slenderness ratio is too low, the x-section of member will be stocky. Table (4.3) shows maximum values of slenderness ratio.

Table 1.3: Maximum slenderness ratio (λ)

Sr. No.	Type of member	λ
1	A strut connected by single rivet at each end.	180
2	A member carrying compressive loads resulting from dead loads and imposed loads.	180
3	A member subjected to compressive forces resulting from wind/earthquake forces, provided the deformation of such members does not adversely affect the stress in any part of the structure.	250
4	Compression flange of a beam.	300
5	A member normally acting as a tie in a roof truss or a bracing system not considered effective subjected to possible reversal of stresses resulting from the action of wind or earthquake forces.	350

Chapter 2

Literature Review

2.1 General

As the present study deals with the comparative analysis of IS 800:2007 and IS 800:1984 for the design of compression and tension members, a detailed analysis is discussed, and the literature review has been conducted on various design procedures for the same according to B.I.S. codes of practices, i.e. IS 800:1984 and IS 800:2007. It also comprises Design and safety checks of compression and tension members, modes of failure at different end conditions, and modes of application of load. Most importantly the literature review also includes prior attempts and development of design charts and spreadsheet applications for design of tension and compression members.

2.2 Review of Literature

Munse and Chesson (1963) studied riveted and bolted joints and observed factors that reduce net section capacity. They conducted numerous experiments on various specimens and connection details. In addition to this they examined test results from extensive bolted and riveted tests. These tests included flat plates, single angles, double angles connected on the same side of the gusset plate, double angles connected on opposite sides of a gusset plate, built-up I-sections, single channel sections and double channel sections. Prior to these studies, tension member capacity was based solely on gross section yielding or net section rupture. The effects of many fabrication and driving factors have been evaluated.

Kulak, L. G. and Eric, Y. W. (1997) conducted test on 24 single and double angle tension members by using bolted end connections. They examined shear lag effects and presented accountings for shear lag in American and Canadian design specifications. Effect of Out-of-Plane Constraint, Angle Thickness, Angle Disposition, Connection Length were analyzed. Effect of connection length was determined either by connecting the long leg or by the short leg of the angle. It was observed that, in all cases where the long leg of the angle is connected, the net section efficiency is higher than for the corresponding cases where the short leg is connected. It was also observed that the net section efficiency is strongly affected by the connection length in certain ranges. The failure of these single and double angle specimens was preceded by a necking down of the net area between the leg edge and lead bolt hole. Rupturing of the specimen initiated at the leg edge and propagated through to the bolt hole and then on through the rest of the specimen.

Orbison, G. J. et.al (2002) performed test on three series of short tension member specimens to investigate the influence of varying connection eccentricity and length on the load capacity of the members. It was observed that the rupture load capacity of the net section was significantly reduced with moderate connection eccentricity, and a net section efficiency factor is developed and proposed as a replacement for the current shear lag factor in determining the effective net area of a tension member. Their test results indicate that the effects of both shear lag and connection eccentricity are largely functions of two geometric factors which are connection eccentricity and connection length. They observed that all specimens were failed by developing neck of the net section, initiating at the lead bolt hole. Sections with small eccentricities exhibited near-simultaneous rupture of the full net section except for a Specimen, which exhibited net section rupture from the bolt hole to the free edge of the web followed immediately by shear rupture of the bolt line. After comparing experimental net section efficiencies, obtained from the three-test series it was found that net section efficiency was found to increase with increasing connection length.

Topkaya (2004) discussed finite element parametric studies on block shear failure of steel tension members and conducted tests to develop simple block shear load capacity prediction equations that are based on Finite Element Analysis. Over a thousand nonlinear analyses were performed to identify the important parameters that influence block shear capacity. In addition, the effects of eccentric loading were investigated. Based on the parametric study block shear load capacity prediction equations were developed. The predictions of the developed equations were compared with the experimental findings and were found to provide estimates with acceptable accuracy

Goswami, R. et.al (2005) studied the sectional properties (strength and stability) considering the different code requirements for desired performance under strong seismic conditions. They found that Indian hot-rolled I-sections (tapered and parallel flanges) are inadequate for use in tall structures in high seismic regions.

Chavda, H. J. (2007) have developed excel spreadsheet to analyse and design tension members as per IS 800:2007. He has also prepared design aids to find out the capacity on angled tension member with single row of bolts connected to the gusset plate. the spreadsheet provide the designed values of design strength due to yielding of gross section (T_{dg}), design strength due to rupture of critical section (T_{dn}) , design strength due to block shear (T_{db}), design strength of the member(T_d), gauge distance (g), diameter of hole (d_h), pitch distance (p), edge distance (e) just by changing the value of yield stress of the material (f_y) and ultimate stress of the material (f_u), as per revised IS 800-2007 provision.

Bouchair et. al (2008) studied analysis of the behaviour of stainless steel bolted connections and conducted experiments on two types of bolted connections that were common in steel structures. They concerned cover plate connections and T- stubs, where the bolts were loaded in shear or in tension. The requirements for stainless steel connection design were essentially the same as for carbon steel. The study considered the case of austenitic stainless steel for which the conventional elastic limit was relatively low compared to the ultimate strength. In bearing, criteria on deformation limits must be considered for cover plate connections. In T-stubs, strain hardening of stainless steel exhibits a continuous increase of the applied load and can influence the failure mode. A Finite Element Model is developed and validated for the two types of connections. A more extensive parametric study should be carried out to develop a better understanding of the behaviour of stainless steel connections.

Kulkarni, R. B. and Jirage, R. S. (2011) observed that for the design of tension member for equal and unequal angles that the Limit State Method gives higher values than Working Stress Method. For equal angles, variation is from 16% to 47% for higher sections to smaller sections. While for Unequal angle long leg connected with single row bolted and double row bolted, the variation is from 12% to 31% for higher sections to smaller sections. For unequal angle short leg connected with single row bolted and double row bolted, it varies from 22% to 54% for higher sections to smaller sections. It was also found that the design of tension member using Angles by Limit state method is economical over the working stress method which values for 12% to 54%.

Comparative design of compression member using allowable stress design method as per IS: 800-1984 and limit state design method as per IS: 800-2007 was studied by **Krishnamoorthy, M. And Tensing, D. (2012).** It includes the comparison of Columns fixed at both ends, column fixed at one end and hinged at other, column pinned at both ends for columns of different lengths. On comparison of the strength of sections calculated using old and new code, they observed that the strength increases with increase in size of the sections to the maximum of 15%. The author concluded that the load carrying capacity of the compression members as per IS 800-2007 is controlled by 'stress reduction factor, inclination of tension field stress in web and effective slenderness ratio. Also, the slenderness ratio is inversely proportional to the stress reduction factor and the design compressive stress is directly proportional to 'stress reduction factor'. The percentage increase in load carrying capacity as per IS 800-1984 is marginally higher than IS 800-2007. The maximum increase was found to be a maximum of 5%.

Parikh, S. (2013) has given a brief description about the characteristics and behaviour of steel compression members as per IS 800:2007. The author has briefly studied and explained various failure modes of axially loaded compression members. The author has focused on the comparison between analytical design of the compression members and behaviour of column in actual practice. His study shows that the highly idealized column cannot be achieved in actual practices. 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, the deflection curve for a real column will differ from the curve of an idealized column.

Patil, S. S. and Pasnur, L. A. (2013) carried the detailed study of structural components as tension members and compression members by using Limit State Method and Working Stress Method. The comparative study of the same has been shown in the form of graphs. They concluded that the design of tension member using Angles by Limit state method (IS 800-2007) is economical over the working stress method (IS 800-1984) which values for 12% to 54%. The percentage increase in load carrying capacity of compression member as per IS 800-1984 is found to be slightly higher than IS 800-2007. The maximum increase was found to be a maximum of 15%. Which means that the section fails if designed by IS 800-2007. So here, the elastic method is economical than limit state method.

Jayaraman, A. et. al (2014) performed a study on behaviour of roof truss and purlins by comparing LSM and WSM to find the economical design. The study was carried out to determine the economical design for the fink type roof trusses, channel section purlins. The study involves examination of theoretical investigations of specimens in series. Overall two methods were designed and compared to evaluate the co-existing moments and shear forces at the critical cross-section with same configuration area by keeping all other parameters constant. Their studies reveal that the limit state method design has high bending strength, high load caring capacity, minimum deflection and minimum local buckling & distortional buckling compare to the working stress method. But working stress method is most economical compare to the limit state method design.

Comparative study of IS 800:2007 and IS 800: 1984 for the design of steel structure was carried out by **Gawatre, D. W. et. al (2015)** the author has also mentioned that there was no need for section classification in the design of beams using IS 800:1984. However, in the limit state design of steel beams, section classification becomes very essential as the moment capacities of each classified section takes different values. The distribution of stresses in cross-sections of members subjected to axial tensile forces is assumed to be uniform. However, some parameters like residual stresses and connection which result in a non-uniform distribution of stresses, but they don't affect the ultimate resistance of the member. To account for eccentric loading in case of angle connected by one leg due to shear lag effect etc., the reduction factor ß is introduced in IS 800:2007 and a coefficient k is introduced in IS 800:1984 with the area of outstanding leg which depends upon the type of connection with the gusset plate. Design tensile strength capacity of unequal section will be more. Authors concluded that lower section is required to carry same load as per IS 800:2007 than that of as per IS 800:1984, number of rivets required are as per IS 800:2007 is less than that of as per IS 800:1984, length of end connection required is also reduced as per IS 800:2007 than that of as per IS 800:1984.

P. V. Supreetha and Y. R. Suresh (2016) studied a series of problem for Single angle and double angle section bolted connection. The design strength was computed by keeping diameter of bolt same and shown with the help of graphs. Similarly, design strength was computed by keeping load and then number of bolts same for different sections. Similar set of problems were also carried out for welded connection.

Jain, A. K. and Shivanshi (2017) compared the analysis of Limit State Method and Working State Methods for the design of Roof truss. The comparison was carried out for shear force, bending moment deflection and displacement. The structure was designed considering wind load with fix supported conditions. After the comparison of various parameters like Shear force, bending moment, Reaction, Weight of the structure, they concluded that the load carrying capacity of limit state method is higher than the working stress method. Actual deflection and bending stress is same in both the method. The study also reveals that the limit state method design has high bending strength, high load carrying capacity, minimum deflection and minimum local buckling and distortional buckling compare to the working stress method.

Comparative analysis of is 800: 2007 and is 800:1984 for the design of equal angle tension member was carried out by **Chitte, C. J. (2017).** The results show that the load carrying capacity of section designed by IS: 800:2007 increases as compare to the design by IS 800:1984. The design of tension member using Angles by Limit state method is economical over the working stress method which values for 12% to 54%. In order to prevent excessive deformation of the members In Limit State Design, the yielding of the gross section must also be considered in addition to net section failure and block shear failure.

Chapter 3

Objective of the study

After studying above literature review it is found that very less research work has been done on comparative design of tension and compression members as well as IS 800:1984 and IS 800 2007. This venture is an attempt to set up a Microsoft Excel Spreadsheet for the comparative design of tension and compression member for IS 800:1984 and IS 800:2007 codes which can be utilized to check the design status of a desired section and to strike the balance between a safe section and an economic section along with precision and flexibility.

Our main objective of the project is to do a comparative study between LSM and WSM.

The objective is also to study tension and compression members in LSM and WSM.

To study the effect of change in length on tension and compression members.

To develop a spreadsheet for the design of tension and compression member by LSM and WSM

To determine and compare the load carrying capacity or the strength of tension by LSM and WSM over a range of thickness of unequal angle sections.

To determine and compare the load carrying capacity or the strength of compression member by LSM and WSM over a range of thickness of equal angle sections.

To compare strength/weight ratios of compression members by LSM and WSM.

To compare strength/weight ratios of Tension members by LSM and WSM.

To establish relation between strength and thickness of tension and compression members to select the suitable member for a loading condition.

Conclusion and recommendations.

Chapter 4

Methodology

4.1 General

The literature found have attempted finite element analysis, stipulations in various codes and specifications regarding the design of tension and compression members are primarily based on the experimental studies. Only a couple of previous studies had included comparison of the limit state method and working state method on practical basis. Based on the literature obtained we have designed examples in limit sate method as well as working state method, tension and compression each in both the states. We designed and analysed members in excel sheet in the following manner and method.

4.2 Ms Office Microsoft excel

Excel is a commercial spreadsheet application produced and distributed by Microsoft for Microsoft Windows and Mac OS X and is produced by Microsoft Corp. It has the same basic features as every spreadsheet, which use a collection of cells arranged into rows and columns to organize data manipulation. It features the ability to perform basic calculations, use graphing tools, create pivot tables and create macro programming language. They also display data as charts, histograms and line graphs. Excel also allow users to organize, format and calculate data with formulas using a spreadsheet system. This software is a part of the Microsoft office suite and is compatible with other applications in the office suite.

4.2.1 Importance of Spreadsheet as a Design Tool

Spreadsheet applications are easily available and among the most widely used computational tools in the world of engineering and science. Traditionally, engineers have used spreadsheets to solve complex analysis and design problems. Spreadsheets are relatively easy to learn and use, very much compatible, and is a low-cost software tool. It provides a powerful and user-friendly platform for the development of simple to fairly complex analysis and design problems and gives us flexibility to use our data unlike other software,

Excel permits users to section data so as to view various factors from different perspectives. Programmers are given an option to code directly using the Visual Basic Editor, including Windows for writing code, debugging and code module organization, which allow users to create variety of complex numerical methods.

4.3 Assumptions

1. The material used is assumed to be homogenous.
2. No imperfections were considered in these investigations.
3. No temperature effects are present.
4. Hot rolled angle sections are taken as tension and compression members.
5. Only equal angle sections are considered in the design of compression members.
6. Only unequal angle sections are considered in the design of tension members.
7. Considering four or more bolts along the length in the end condition. i.e. $\alpha = 0.8$
8. Compressive stress f_{cd} is assumed to be 180 MPa throughout the design.

9. The ideal column is assumed to be absolutely straight having no crookedness.

10. The modulus of elasticity is assumed to be constant in a built-up column.

11. Secondary stresses (which may be of the order of even 25%-40% of primary Stresses) are neglected.

4.4 Design Methodology of IS 800:1984

4.4.1 Tension member

Tension Member Subjected to Axial Load (WSM)

The following procedure may be followed in the design of an axially loaded tension member.

Step 1) The net area required (A_{net}) to carry the design load P is obtained by the equation,

$$P = \sigma_{at} A_{net}$$

Step 2) The net area calculated thus, is increased suitably (25%- 40%) to compute the gross sectional area.

From I.S. Handbook No. 1 suitable section/sections providing a cross-sectional area matching with the computed gross sectional area is selected.

Step 3) The number of rivets required to make the connection is calculated. These are arranged in a suitable pattern and the net area of the section provided is calculated. This should be more than the net area calculated in step (1).

Step 4) The slenderness ratio of the member is checked as per the I.S. specifications

4.4.2 Compression member

The procedure is thus of trial and error and is as follows,

Step 1) Average allowable compressive stress in the section is assumed.

(It should not be more than the upper limit for the column formula specified by the code).

Step 2) The cross-sectional area required to carry the load at the assumed allowable stress is computed as

$$A = \frac{P}{\text{Allowable compressive stress}}$$

Where,

A = tentative cross-sectional area required (in mm), and

P = load on column in Newtons.

Step 3) A section that provides the estimated required area is selected from IS hand Book No.1.

(The section is so chosen that the minimum radius of gyration of the section selected is as large as possible. The appropriate least radius of gyration for the section selected is recorded.)

Step 4) The effective length of the column is calculated based on the end conditions and slenderness ratio is computed by

$$\lambda = \frac{l}{r}$$

which should be less than permissible slenderness ratio.

Step 5) For this estimated value of slenderness, at the maximum allowable compressive stress, σ_{ac} is calculated. In case of struts this value may have to be reduced to 80% depending on the end conditions.

Step 6) The load carrying capacity of the member is composed by multiplying the maximum compressive stress thus obtained with the cross-sectional area provided. This value of the load carrying capacity of the member should be more than load to be supported by it.

4.5 Design Methodology by IS 800:2007

4.5.1 Tension member

Design of Tension Member Subjected to Axial Load

1) Gross section yielding,

$$T_{dg} = \frac{A_{g.f_y}}{\gamma_{m0}}$$

2) Net section fracture,

For plates and threaded rods,

$$T_{dn} = \frac{0.9A_n f_u}{\gamma_{m0}}$$

For angles, etc.

$$T_{dn} = \frac{\alpha f_u A_n}{\gamma_{m0}}$$

3) Once the trial shape is selected, the section is checked for slenderness ratio limit, gross section yielding, net section fracture, and block shear failure.

The step by step procedure for the design of tension member subjected to axial load is as follows,

Step 1) The net area required A_n to carry the factored load T is obtained by,

$$A_n = \frac{T}{0.9f_u/\gamma_{m0}}$$

Or by,

$$A_n = \frac{T}{\alpha f_u/\gamma_{m1}}$$

As appropriate.

Where,

T = factored design load,

f_u = ultimate strength of material,

A_n = net area of cross section,

α=0.6, 0.7, and 0.8 as appropriate and

γ_{m1}=1.25

Step 2) The net area calculated thus is increased suitably (10%-25%) to compute the tentative gross sectional area.

Step 3) The trial gross area is also determined from its yield strength by

$$A_g = \frac{T}{f_y/\gamma_{m0}}$$

Where,

f_y = Yield strength of the material and

$\gamma_{m0} = 1.1$

Step 4) From IS handbook no.1, suitable rolled section/built-up section providing a gross sectional area matching with the computed gross sectional area is selected.

Step 5) The no. of bolts (or weld) required to make the connection is calculated. These are arranged in a suitable pattern and the net area of the section provided is calculated. Thereafter the effective net area is determined if the section selected is not connected with all of its elements.

Step 6) The design strength T_d of the trial section is calculated. This will be minimum of the strength T_{dg} , T_{dn} and T_{db}. The design strength T_d should be more than the factored design load.

Step 7) The slenderness ratio of the member is checked as per IS specification.

4.5.2 Compression member

The design compressive strength P_d of a member is given by:

$P < P_d$

Where, $P = A_e \times f_{cd}$

Where, A_e = effective sectional area,

f_{cd} = design compressive stress,

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\emptyset + [\emptyset^2 - \lambda^2]^{0.5}} = \frac{\chi f_y}{\gamma_{m0}} \le \frac{f_y}{\gamma_{m0}}$$

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

λ= non-dimensional effective slenderness ratio

$$= \sqrt{f_y/f_{cc}} = \sqrt{f_y(KL/r)^2/\pi^2 E}$$

f_{cc} = Euler buckling stress = $\frac{\pi^2 E}{(KL/r)^2}$

where, KL/r = Effective slenderness ratio

α = imperfection factor given in (table 4.5)

χ = Stress reduction factor (table 8)

$$= \frac{1}{\emptyset + [\emptyset^2 - \lambda^2]^{0.5}}$$

γ_{m0} = Partial safety factor for material strength

Table 4.5: Imperfection factor

Buckling class	a	B	C	d
α	0.21	0.34	0.49.,	0.76

4.6 Sequence of Project Completion

This study was approached in the following sequence:

1. A literature survey was conducted which investigated the design of tension member and compression member using Limit state and working state methods.

2. Review of recommendations on tension and compression member from codes Bureau of Indian Standard-800 (2007), Bureau of Indian Standard-800 (1984), and hand book for structural steel.

3. Designing and making spreadsheet using the MS Office Excel.

4. Comparison of results obtained from LSM and WSM for tension and compression members.

5. Conclusion and recommendation on the structural and economical behaviour of tension and compression member.

6. Final submission.

Chapter 5

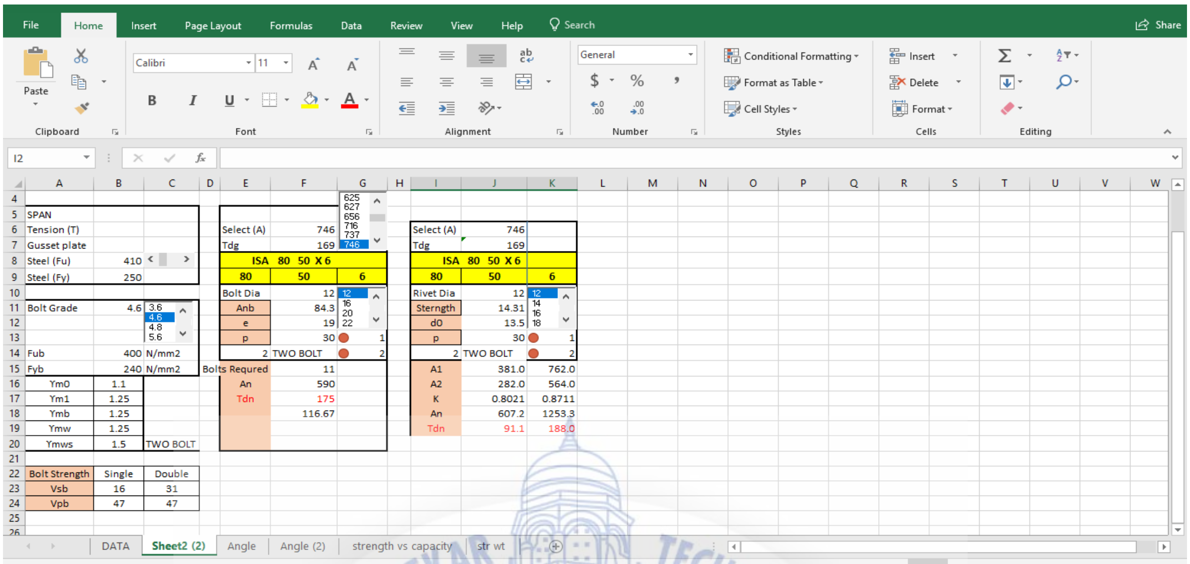
Results and Discussions

5.1 General

In this approach, the design procedure for different steel compression and tension member is completely done in an excel spreadsheet and the results of the differences between the working state method and limit state method are shown graphically.

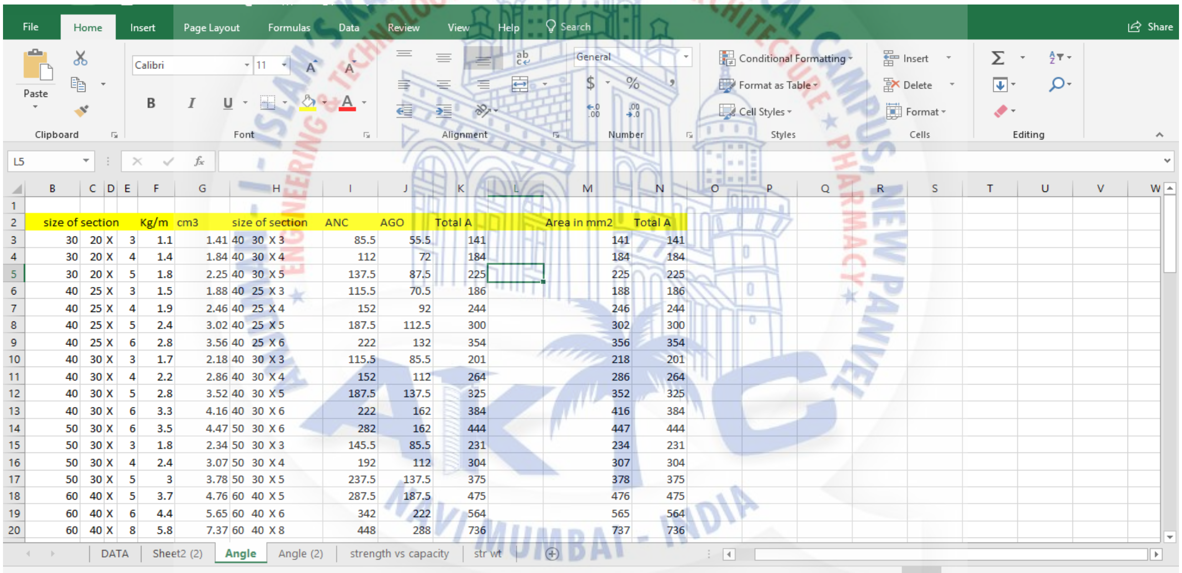
The graphs that are formed below are showing the results that are occurred while conducting this project. For tension members, we have compared thickness of unequal angle with capacity of tension members, also graphical study has done for the Strength Weight Ratio Vs Section. For compression member, the results were tested for member fixed at both ends and hinged at both ends, for a length of 0.5m, 1m, 1.5m, 2m. The testing was done for unequal angle section as tension member and equal angle sections as compression members only and no other section is considered.

Snapshots of Spreadsheets



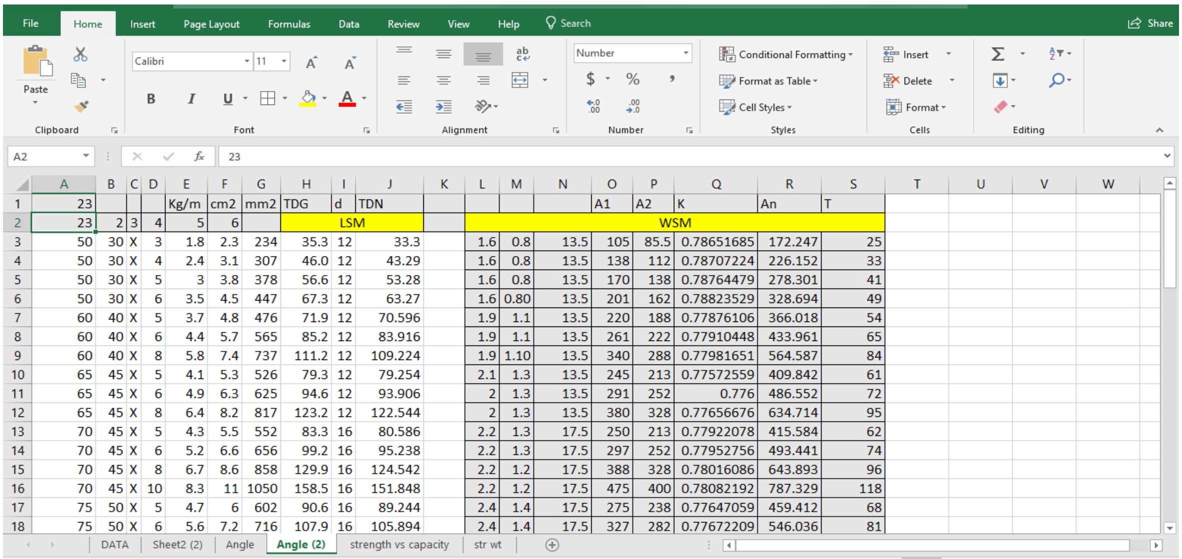
	A	B	C	D	E	F	G	H	I	J	K
5	SPAN										
6	Tension (T)				Select (A)	746	625 627 656 716 737 746		Select (A)	746	
7	Gusset plate				Tdg	169			Tdg	169	
8	Steel (Fu)	410			ISA 80 50 X 6				ISA 80 50 X 6		
9	Steel (Fy)	250			80	50	6		80	50	6
10					Bolt Dia	12	12 16 20 22		Rivet Dia	12	12 14 16 18
11	Bolt Grade	4.6	3.6 4.6 4.8 5.6		Anb	84.3			Sterngth	14.31	
12					e	19			d0	13.5	
13					p	30	1		p	30	1
14	Fub	400	N/mm2		2	TWO BOLT	2		2	TWO BOLT	2
15	Fyb	240	N/mm2		Bolts Requred	11			A1	381.0	762.0
16	Ym0	1.1			An	590			A2	282.0	564.0
17	Ym1	1.25			Tdn	175			K	0.8021	0.8711
18	Ymb	1.25				116.67			An	607.2	1253.3
19	Ymw	1.25							Tdn	91.1	188.0
20	Ymws	1.5	TWO BOLT								
22	Bolt Strength	Single	Double								
23	Vsb	16	31								
24	Vpb	47	47								

DATA | Sheet2 (2) | Angle | Angle (2) | strength vs capacity | str wt



	B	C	D	E	F	G	H	I	J	K	L	M	N
2	size of section				Kg/m	cm3	size of section	ANC	AGO	Total A		Area in mm2	Total A
3	30	20	X	3	1.1	1.41	40 30 X 3	85.5	55.5	141		141	141
4	30	20	X	4	1.4	1.84	40 30 X 4	112	72	184		184	184
5	30	20	X	5	1.8	2.25	40 30 X 5	137.5	87.5	225		225	225
6	40	25	X	3	1.5	1.88	40 25 X 3	115.5	70.5	186		188	186
7	40	25	X	4	1.9	2.46	40 25 X 4	152	92	244		246	244
8	40	25	X	5	2.4	3.02	40 25 X 5	187.5	112.5	300		302	300
9	40	25	X	6	2.8	3.56	40 25 X 6	222	132	354		356	354
10	40	30	X	3	1.7	2.18	40 30 X 3	115.5	85.5	201		218	201
11	40	30	X	4	2.2	2.86	40 30 X 4	152	112	264		286	264
12	40	30	X	5	2.8	3.52	40 30 X 5	187.5	137.5	325		352	325
13	40	30	X	6	3.3	4.16	40 30 X 6	222	162	384		416	384
14	50	30	X	6	3.5	4.47	50 30 X 6	282	162	444		447	444
15	50	30	X	3	1.8	2.34	50 30 X 3	145.5	85.5	231		234	231
16	50	30	X	4	2.4	3.07	50 30 X 4	192	112	304		307	304
17	50	30	X	5	3	3.78	50 30 X 5	237.5	137.5	375		378	375
18	60	40	X	5	3.7	4.76	60 40 X 5	287.5	187.5	475		476	475
19	60	40	X	6	4.4	5.65	60 40 X 6	342	222	564		565	564
20	60	40	X	8	5.8	7.37	60 40 X 8	448	288	736		737	736

DATA | Sheet2 (2) | Angle | Angle (2) | strength vs capacity | str wt



	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
1	23				Kg/m	cm2	mm2	TDG	d	TDN					A1	A2	K	An	T
2	23	2	3	4	5	6		LSM				WSM							
3	50	30	X	3	1.8	2.3	234	35.3	12	33.3		1.6	0.8	13.5	105	85.5	0.78651685	172.247	25
4	50	30	X	4	2.4	3.1	307	46.0	12	43.29		1.6	0.8	13.5	138	112	0.78707224	226.152	33
5	50	30	X	5	3	3.8	378	56.6	12	53.28		1.6	0.8	13.5	170	138	0.78764479	278.301	41
6	50	30	X	6	3.5	4.5	447	67.3	12	63.27		1.6	0.80	13.5	201	162	0.78823529	328.694	49
7	60	40	X	5	3.7	4.8	476	71.9	12	70.596		1.9	1.1	13.5	220	188	0.77876106	366.018	54
8	60	40	X	6	4.4	5.7	565	85.2	12	83.916		1.9	1.1	13.5	261	222	0.77910448	433.961	65
9	60	40	X	8	5.8	7.4	737	111.2	12	109.224		1.9	1.10	13.5	340	288	0.77981651	564.587	84
10	65	45	X	5	4.1	5.3	526	79.3	12	79.254		2.1	1.3	13.5	245	213	0.77572559	409.842	61
11	65	45	X	6	4.9	6.3	625	94.6	12	93.906		2	1.3	13.5	291	252	0.776	486.552	72
12	65	45	X	8	6.4	8.2	817	123.2	12	122.544		2	1.3	13.5	380	328	0.77656676	634.714	95
13	70	45	X	5	4.3	5.5	552	83.3	16	80.586		2.2	1.3	17.5	250	213	0.77922078	415.584	62
14	70	45	X	6	5.2	6.6	656	99.2	16	95.238		2.2	1.3	17.5	297	252	0.77952756	493.441	74
15	70	45	X	8	6.7	8.6	858	129.9	16	124.542		2.2	1.2	17.5	388	328	0.78016086	643.893	96
16	70	45	X	10	8.3	11	1050	158.5	16	151.848		2.2	1.2	17.5	475	400	0.78082192	787.329	118
17	75	50	X	5	4.7	6	602	90.6	16	89.244		2.4	1.4	17.5	275	238	0.77647059	459.412	68
18	75	50	X	6	5.6	7.2	716	107.9	16	105.894		2.4	1.4	17.5	327	282	0.77672209	546.036	81

DATA | Sheet2 (2) | Angle | Angle (2) | strength vs capacity | str wt

Snapshots of Spreadsheets for Compression Members

AD9

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T
1		Length Of STRUT				2														
2																				
3																			88.8126	
4													>=2	>=2	1	1	>=2	>=2	1	1
5													Fixed	Hinged	Fixed	Hinged	Fixed	Hinged	Fixed	Hinged
6	sr. no	size of section				kg/m	cm2	rxx=ryy	ruu	rvv	λvv	λφ	λe				φ			
7		1	2	3	4	5	6	10	11	12										
8	1	20	20	X	3	0.9	1.12	0.58	0.7	0.4	6.0858	0.07506	3.644	4.791	3.718	4.484	7.981	13.100	8.275	11.603
9	2	20	20	X	4	1.1	1.45	0.58	0.7	0.4	6.0858	0.05629	3.637	4.789	3.712	4.468	7.955	13.093	8.249	11.525
10	3	25	25	X	3	1.1	1.41	0.73	0.9	0.5	4.7909	0.09382	2.900	3.810	2.993	3.641	5.366	8.642	5.664	7.970
11	4	25	25	X	4	1.4	1.84	0.73	0.9	0.5	4.7909	0.07037	2.887	3.807	2.980	3.609	5.325	8.632	5.623	7.847
12	5	25	25	X	5	1.8	2.25	0.72	0.9	0.5	4.7909	0.05629	2.880	3.806	2.974	3.594	5.305	8.627	5.603	7.790
13	6	30	30	X	3	1.4	1.73	0.89	1.1	0.6	3.9504	0.11259	2.432	3.182	2.543	3.133	4.005	6.294	4.307	6.125
14	7	30	30	X	4	1.8	2.26	0.89	1.1	0.6	3.9504	0.08444	2.409	3.178	2.521	3.079	3.944	6.279	4.246	5.946

LSM | WSM | thick vs capacity 2 L0.5 | wt strength 2B L0.5 | thick vs capacity 1B L0.5 | wt str 1B L0 ...

AD9

	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB
1																	
2																	
3								88.8126									
4		>=2	>=2	1	1	>=2	>=2	1	1	>=2	>=2	1	1	>=2	>=2	1	1
5		Fixed	Hinged	Fixed	Hinged	Fixed	Hinged	Fixed	Hinged	Fixed	Hinged	Fixed	Hinged	Fixed	Hinged	Fixed	Hinged
6	λφ	λe				φ				Fcd				Pd			
7																	
8	0.07506	3.644	4.791	3.718	4.484	7.981	13.100	8.275	11.603	15.068	8.99	14.51	10.190	1.114	0.664	1.072	0.753
9	0.05629	3.637	4.789	3.712	4.468	7.955	13.093	8.249	11.525	15.121	8.99	14.55	10.261	1.447	0.860	1.393	0.982
10	0.09382	2.900	3.810	2.993	3.641	5.366	8.642	5.664	7.970	22.999	13.86	21.70	15.090	2.140	1.290	2.019	1.404
11	0.07037	2.887	3.807	2.980	3.609	5.325	8.632	5.623	7.847	23.194	13.88	21.87	15.341	2.817	1.685	2.656	1.863
12	0.05629	2.880	3.806	2.974	3.594	5.305	8.627	5.603	7.790	23.285	13.88	21.95	15.459	3.458	2.062	3.260	2.296
13	0.11259	2.432	3.182	2.543	3.133	4.005	6.294	4.307	6.125	31.626	19.38	29.20	19.956	3.611	2.213	3.334	2.279
14	0.08444	2.409	3.178	2.521	3.079	3.944	6.279	4.246	5.946	32.165	19.43	29.66	20.601	4.798	2.899	4.424	3.073

LSM | WSM | thick vs capacity 2 L0.5 | wt strength 2B L0.5 | thick vs capacity 1B L0.5 | wt str 1B L0 ...

F29 =C29*1000/(A29*9.81*0.5)

	A	B	C	D	E	F	G	H
1								
2								
3	kg/m	t	2B FIX(L)	2B HIN(L)	2R WSM	LSM	WSM	size of section
25	2.3	3	23.611	22.9222	40.22	2092.9	3564.89	50 50x3
26	3	4	36.8957	31.4053	52.90	2507.35	3594.7	50 50x4
27	3.8	5	49.564	39.525	65.30	2659.15	3503.52	50 50x5
28	4.5	6	61.5947	47.3663	77.44	2790.57	3508.23	50 50x6
29	4.1	5	53.4291	43.9517	73.22	2656.78	3640.79	55 55x5
30	4.9	6	67.1527	52.8841	86.97	2794.01	3618.66	55 55x6
31	6.4	8	92.9101	70	113.65	2959.67	3620.28	55 55x8
32	7.9	10	116.907	86.2611	139.21	3016.98	3592.61	55 55x10
33	4.5	5	56.8572	48.3068	81.08	2575.93	3673.57	60 60x5
34	5.4	6	72.2425	58.275	96.33	2727.47	3636.69	60 60x6
35	7	8	101.271	77.5223	126.18	2949.51	3674.97	60 60x8
36	8.6	10	128.362	95.858	154.91	3042.98	3672.3	60 60x10
37	4.9	5	59.98	52.6719	89.23	2495.58	3712.42	65 65x5
38	5.8	6	77.1701	63.8456	106.21	2712.58	3733.52	65 65x6
39	7.7	8	109.508	85.1981	139.18	2899.45	3685.02	65 65x8
40	9.4	10	139.772	105.642	171.12	3031.47	3711.37	65 65x10
41	5.3	5	62.7961	57.0496	97.66	2415.56	3756.55	70 70x5

LSM | WSM | thick vs capacity 2 L0.5 | wt strength 2B L0.5 | thick vs capacity 1B L0.5 | wt str 1B L0 ...

5.2 Tension Member

The comparison of LSM and WSM in excel sheet has given graphs which is explained below. The tension member alone is independent of the length factor that means it does not affected by length. Whereas when the member in a truss is seen, both tension and compression is seen in a single member. Hence slenderness ratio cannot be taken as less or equal to 180. For a member normally acting as a tie in truss, not considered effective subjected to possible reversal of stresses take slenderness ratio less than 350. Hence it is dependent on length factor. The result that we get here is that when there are two sections taking equal amount of load, the section to be taken under practice will be the one which will be least to slenderness ration 350. This will give more economy.

The graphs comprises of x- axis showing thickness of unequal angle and y- axis showing capacity. For unequal angles the connecting length is larger than the other. It is called the outstanding leg. It is denoted by L_0. First, sections are taken from the range 50×30×3 to 100×65×10 as shown in Figure (5.1), Figure (5.2) and Figure (5.3). It is observed that the gross area strength is greater than the net area strength which is also greater than the strength in that of working state method. This resulted that the limit state method gives more strength than working state method. Then sections are taken from 100×75×6 to 125×75×10 as shown in Figure (5.3) and Figure (5.4). The observation for these sections is that the strength in gross area and net area is approximately same. But the result obtained from this was the same as that of the previous group of sections, that limit state method gives more strength than that of working state method. Lastly sections ranging from 125×95×9 to 200×150×20 are taken as shown in Figure (5.4) and Figure (5.5). In this the result obtained is something which is not practically possible. It showed that the net area strength is greater than that of gross area method. Here also the strength given by limit state method is more than the strength given by working state method. After reading the result carefully it was seen that the net area strength was more than the gross area strength because $\propto$ taken is 0.8 for three bolts. The equation form IS code 800: 2007 was used for the comparison of the strength.

$$T_{dn} = \frac{\propto A_n f_u}{\gamma_{m1}}$$

Where, $\propto$ = 0.6 for one two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length.

E.g.

For section 50×30×3 the net area strength given by limit state method is 33.3kN. But for the same section the strength given by working state method is 25kN. After following the design steps, the required strength comes out to be 30kN. Here the section fails in working state method and the section of more thickness has to be acquired i.e. 50×30×4, as shown in Figure (5.6), this becomes uneconomical. But on the other hand, the same section is safe in limit state method compared to working state method. This proves the economy and the strength giving capacity of limit state method over working state method.

Later we have taken the average of the ratio of strength/ weight for the all sections in tension member. We observed that the average of ratio in limit state method is more than the working stress method. The range in LSM is 1990 to 1960, and in WSM the range is 1410 to 1620. The average that was found was 1962 for LSM and1497 for WSM.

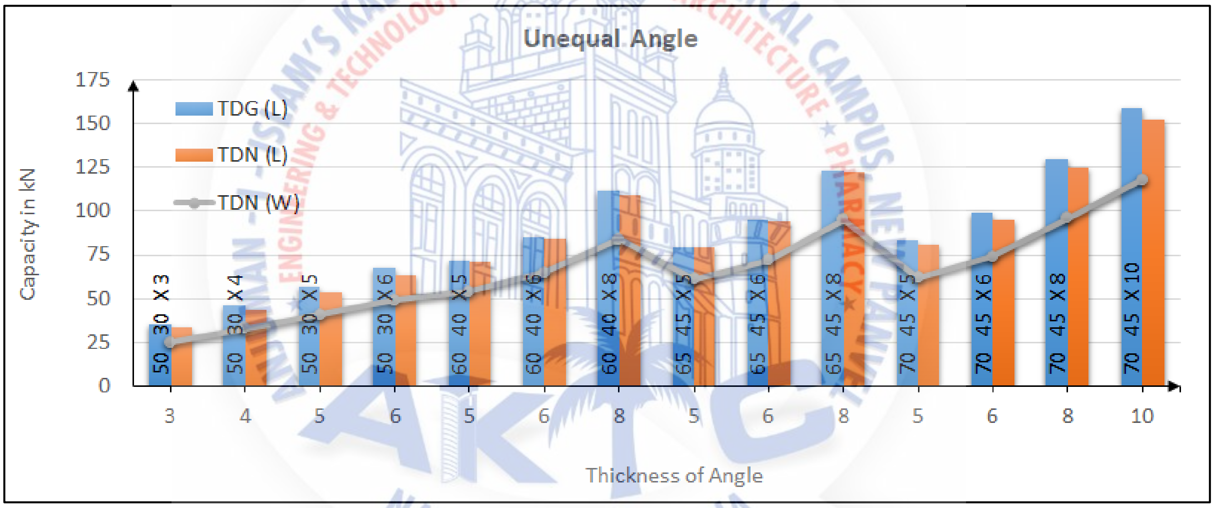


Figure. (5.1): Capacity v/s Thickness of Angle (50×50×3-70×45×10)

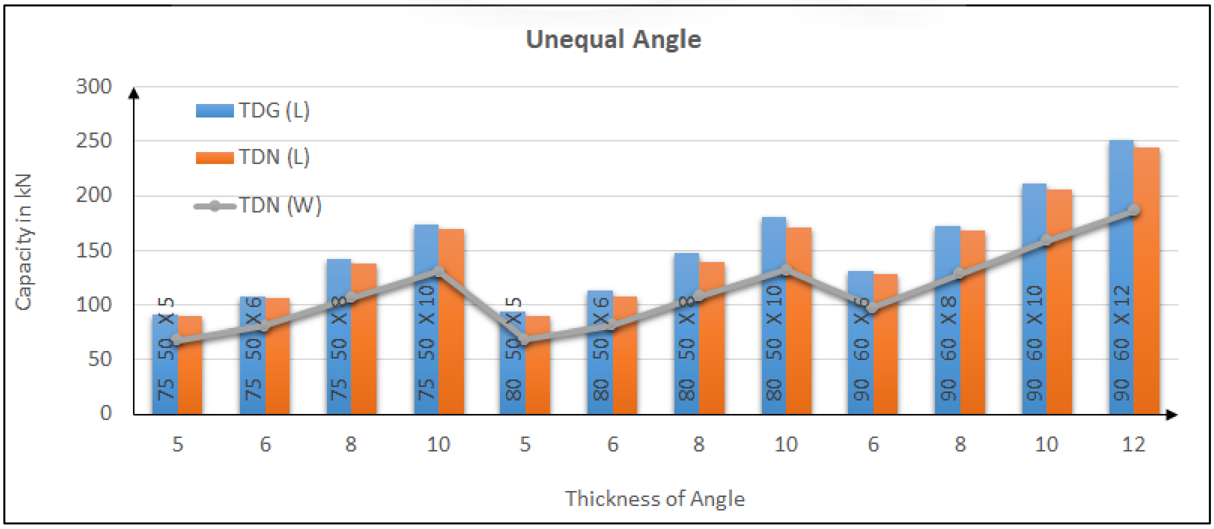


Figure (5.2): Capacity v/s Thickness of Angle (70×70×5-90×60×12)

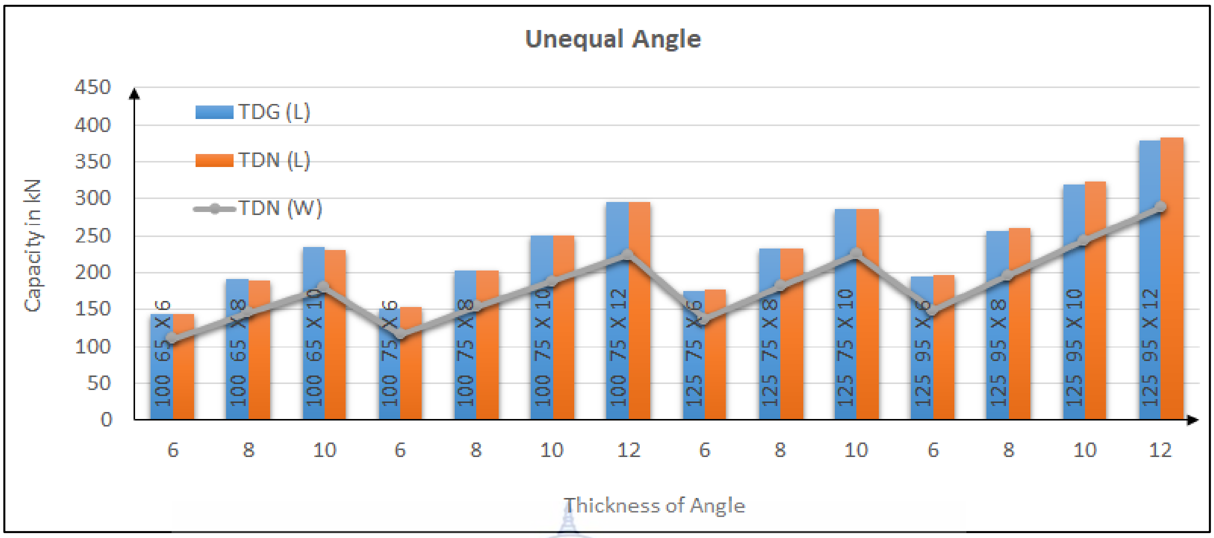


Figure (5.3): Capacity v/s Thickness of Angle (100×65×6-125×95×12)

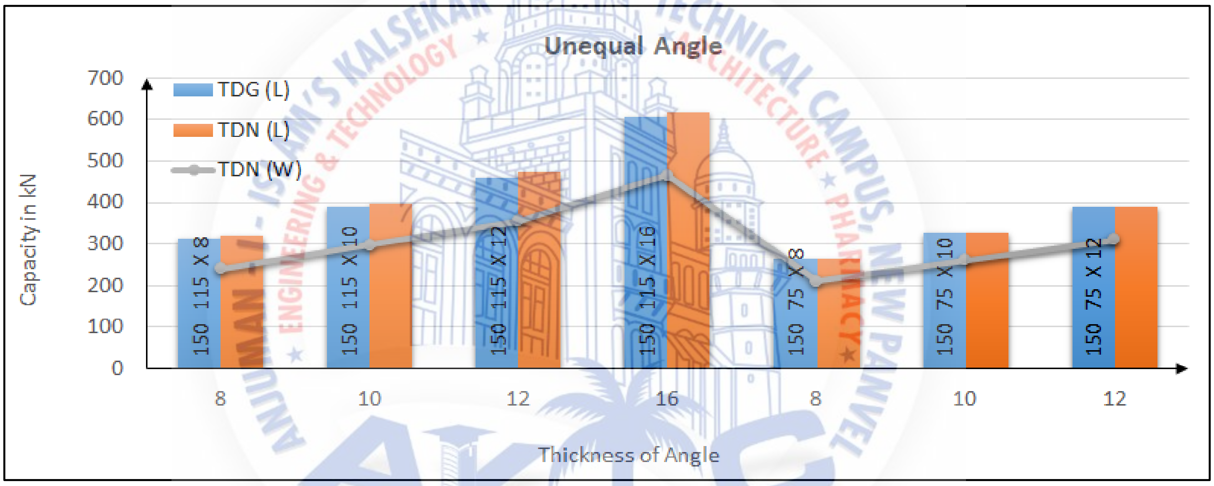


Figure (5.4) Capacity v/s Thickness of Angle (150×115×8-150×75×12)

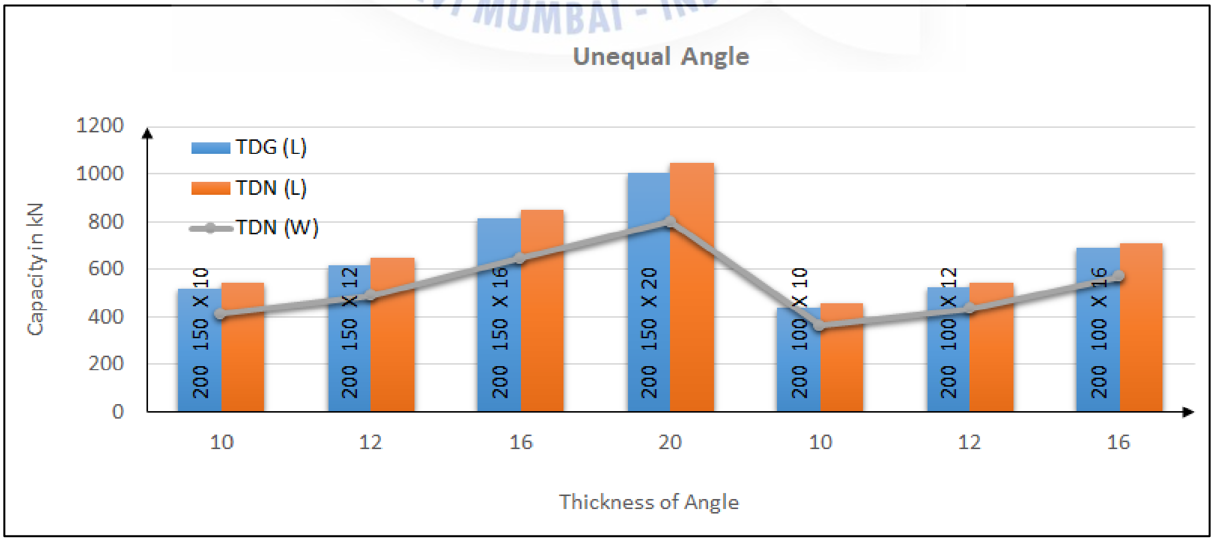


Figure (5.5) Capacity v/s Thickness of Angle (200×150×10-200×100×16)

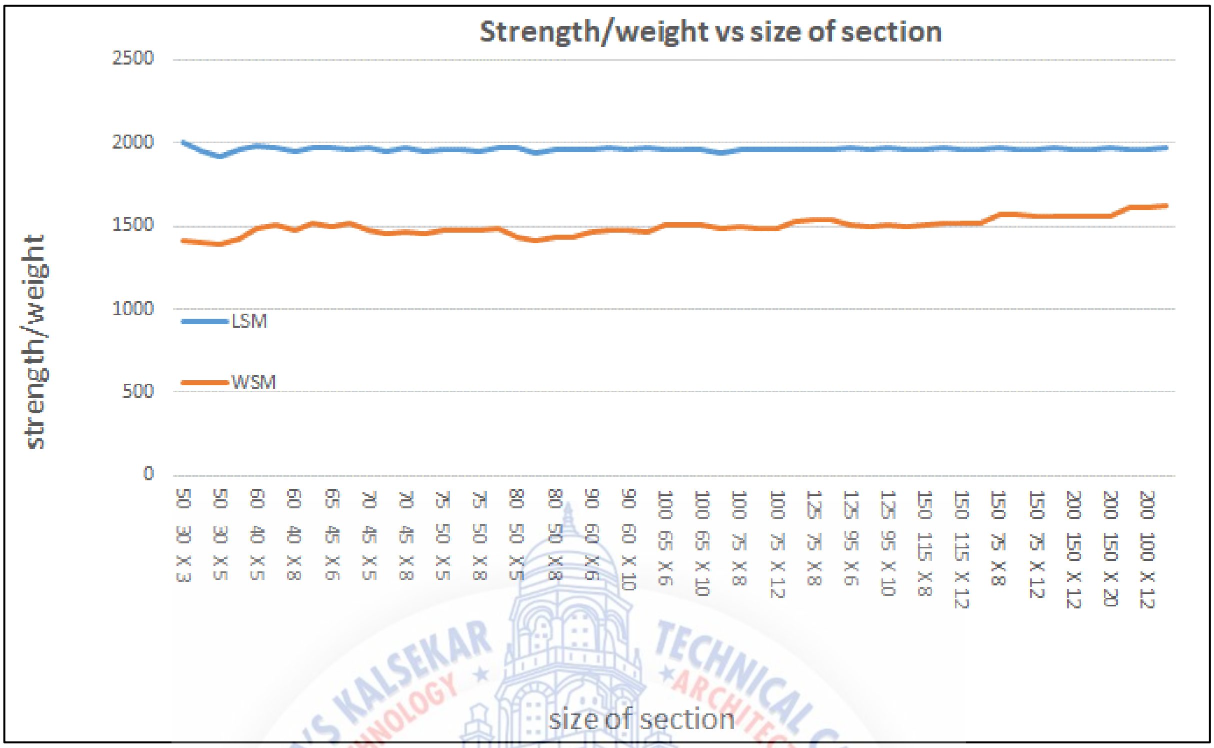


Figure (5.6): Strength/weight ratio v/s size of section (50×50×3-200×200×12)

5.3 Compression member

Since unequal angles cannot be used in compression members, equal angle section is used here. For compression members the range of length that we have taken varies from 0.5m to 2m. There are two types of graphs that we have one is thickness v/s capacity and the other is strength v/s weight. The graphs comprises of x- axis showing thickness and y- axis showing capacity. And similarly, for strength and weight.

1. The length taken is 0.5m.

We have compared 2 bolts with both ends fixed condition and 2 bolts with both ends hinged condition in limit state method to 2 rivets in working state method. The section ranges from 50×50×3 to 200×200×25 as shown in Figure (5.7), Figure (5.8), Figure (5.9) and Figure (5.10). Through the graph acquired the strength for 2 bolts with both ends fixed condition is greater than 2 bolts with both ends hinged condition in LSM and the strength acquired from 2 rivets in WSM is greater than that in LSM.

When 1 bolt with both ends fixed condition, and 1 bolt with both ends hinged condition for LSM and 1 rivet for WSM is assumed, the strength acquired in WSM is again greater than that in LSM. As shown in Figure (5.11), Figure (5.12), Figure (5.13) and Figure (5.14)

After observing the graphs, the result obtained was that, in both the cases the economy of WSM is greater than that of LSM. As shown in Figure (5.39), and Figure (5.40).

2. The length taken is 1m.

The same range of section is taken in this case also. Even the end conditions are taken same here. i.e. 2 bolts for both ends fixed and both ends hinged conditions in LSM and 2 rivet in WSM, we observed that the strength in LSM is less than that of in WSM. And the result obtained here is that WSM is more economical than that of LSM. As shown in Figure (5.15), Figure (5.16), Figure (5.17) and Figure (5.18).

When the condition of 1 bolt with both ends fixed and 1 bolt with both ends hinged conditions for LSM and 1 rivet for WSM is assumed for another range of section from 65×65×5 to 200×200×25, the result changes. The economy of WSM is seen to be greater than LSM. As shown in Figure (5.19), Figure (5.20), Figure (5.21) and Figure (5.22).

3. The length taken is 1.5m.

The section ranging from 50×50×30 to 65×65×10 has been studied here as shown in Figure (5.23). For the 2 bolts with both ends fixed condition and 2 bolts with both ends hinged conditions in LSM and 2 rivets in working stress method the graphs obtained resulted that the strength in LSM is greater than WSM. Here the LSM is more economical than WSM.

For section 70×70×5 and 70×70×6 as shown in Figure (5.24), 2 bolts with both ends fixed ends condition and 2 bolts with hinged ends condition in LSM and 2 rivets in working state method, the strength in WSM increases and in LSM decreases. And again, the economy of WSM is more than LSM.

For the section 75×70×8 and 70×70×10 and end conditions 2 bolt with both ends fixed condition, and 2 bolts with both ends hinged condition in LSM and 2 rivets in working state method the strength in LSM and WSM is approximately same. As shown in Figure (5.24).

For sections ranging from 75×75×5 to 200×200×25 as shown in Figure (5.24), Figure (5.25), and Figure (5.26), and end conditions as both ends fixed and both ends hinged in LSM for 2 bolts and taking 2 rivets in WSM, the strength in LSM is less than WSM. The result we get from here is that the economy of WSM is more than LSM.

Here the sections taken are from 50×50×3 to 80×80×12 as shown in Figure (5.27), and Figure (5.28). The 1 bolt with member fixed at both end condition gave more strength than 1 bolt with member hinged at both ends condition in LSM. Whereas, 1 rivet section in WSM gave less strength in comparison with LSM.

But for the section 90×90×6 to200×200×25 as shown in Figure (5.29), and Figure (5.30), when same end conditions were applied the result of strength in LSM obtained is same as previous, but in WSM the 1rivet condition gave more strength than that of LSM. Here the economy of WSM was seen to be more.

For the study of strength/weight v/s section sizes and with end conditions as both ends fixed and both ends hinged for 2 bolts in LSM, 2 rivets in WSM we observed that the strength/ weight ratio of the section increases with increase in size of the section in WSM, whereas in LSM it remains constant. This observation was seen for the sections ranging from 50×50×4 to 55×5×5, where the strength/ weight range in LSM was seen to be more than that in WSM. Though the section ranging from 60×60×5 to 200×200×5 showed the range of strength/ weight ratio to be more in LSM than WSM. the fact that WSM is giving more strength does not change. The range of the strength/weight ratio is as 450 – 980 in LSM and 480 – 1250 in WSM. When the end conditions of 1 bolt with both ends fixed condition and 1 bolt with both ends hinged condition in LSM and 1 rivet in WSM is applied the sections ranging from 50×50×30 to 70×70×8 showed that the range of strength/weight ratio in LSM is more than WSM except for the section 70×70×5. Later the sections ranging from 70×70×10 to 80×80×12 showed the range of strength/weight ratio in both LSM and WSM is approximately same. And then the sections from 90×90×60 to 200×200×25, they showed a different result. They showed that the range of strength/weight ratio in WSM is more than LSM. The range is as 370 – 710 in LSM and 300 – 970 in WSM. As shown in Figure (5.43), Figure (5.44).

4. The length taken is 2m.

For the end conditions 2 bolts with both ends fixed end condition and 2 bolts with both ends hinged condition in LSM, the strength of 2 bolts fixed was seen to more. And the strength in WSM for 1 rivet was seen to be less than the LSM. The sections taken here is 50×50×3 to 80×80×12 as shown in Figure (5.31) and Figure (5.32).

But when sections 90×90×6, 90×90×8 and 90×90×10 as shown in Figure (5.33), was compared it is seen that for the end condition as both ends fixed gave more strength than both ends hinged in LSM for 2 bolts. Whereas, 2 rivet condition in WSM gave more strength than that in LSM. Here the result obtained is that, for these three sections, they give more strength in WSM and that, this method is more economic.

Then we took section 100×100×6 as shown in Figure (5.33), here the end conditions were applied as previous for 2 bolts in LSM, 2 rivets in WSM, and it was observed that LSM gave more strength. And it resulted that LSM is more economic than WSM for this section.

For the same length we decided to take more section and observe them. We took the sections ranging from 100×100×6 to 200×200×25 as shown in Figure (5.33), and Figure (5.34). In LSM the end condition of both ends fixed gave or 2 bolts gave more strength than both ends hinged for 2 bolts. But in WSM the 2 rivets condition gave more strength than LSM. And it resulted that WSM is more economic than LSM for these sections.

When the above criteria’s are compared in the graph of strength/weight v/s size of section, the economy of WSM is to be seen more than LSM. The strength/weight ratio ranged from 260 to 710 in LSM, whereas the strength in WSM kept on increasing with increase in weight.

Now we compared the sections changing the number of bolt as 1. The end conditions taken was both ends fixed and both ends hinged in LSM, and 1 rivet in WSM. The sections are from 50×50×3 to 65×65×10 as shown in Figure (5.35). It showed that the 1 bolt with both ends fixed gave more strength than 1 bolt with both ends hinged. But in between it was seen that the section 65×65×5 and 65×65×6 acted differently. They resulted that the 1 bolt for both ends hinged gave more strength/weight ratio than 1 rivet in WSM. But the overall observation was the same, that LSM gave more strength than WSM.

For the section 70×70×10 as shown in Figure (5.34), it was seen that for both ends fixed with 1 bolt gave more strength than 1 bolt with both ends hinged. And 1 bolt with both ends hinged condition in LSM gave more strength than 1 rivet in WSM. But the strength/weight ratio for both ends hinged with 1 bolt was equal to the strength from 1 rivet. These were the fluctuations which were seen in between.

The sections ranging from 75×75×5 to 90×90×12 as shown in Figure (5.36), and Figure (5.37), for the above end conditions showed that the strength/weight ratio in 1 bolt with both ends fixed in LSM is more than 1 rivet in WSM, except the section 90×90×6 which showed that the strength is greater in WSM than LSM.

Then we took sections 100×100×6 and 100×100×8 as shown in Figure (5.37), which resulted that the economy in WSM is greater than LSM. Since the strength in WSM is more than LSM for the end conditions specified before. The 1 bolt with both ends fixed gave more strength than 1 bolt with both ends hinged and 1 rivet in WSM gave more strength than 1 bolt with both ends hinged, but overall it resulted that LSM gave more economy than WSM. The sections which showed this result was 100×100×10 and 100×100×12.

The sections 100×100×8, 100×100×10 and 100×100×12 also showed the economy as well as the strength of LSM is more than WSM, for the above-mentioned end conditions for both LSM and WSM.

For the section 110×110×15 it was seen that the 1 bolt with both ends fixed condition gave less strength/weight ratio than 1 rivet condition but in LSM the 1 bolt with both ends fixed condition gave more strength than 1 bolt with both ends hinged as before. Overall, here also the LSM economy was seen to be more than WSM. But on the other hand, for the same conditions the ranging from 130×130×8 to 200×200×25 showed the result that, WSM is more economic than LSM. Here the WSM gave more strength than LSM. When the sections are observed under strength/weight v/s size of section graphs the results are different. When the sections ranging from 50×50×30 to 70×70×8 are observed the range of strength in LSM is more than the range of strength in WSM. Except for the section 70×70×5 which do not show the same result. Then the sections ranging from 70×70×10 to 90×90×12 is observed the range of strength/weight ratio acquired was approximately same. Then the other set of groups of sections were observed from 100×100×6 to 200×200×25 which showed that the strength/weight ratio range in WSM is more or greater than range of strength/weight in LSM. The range of strength/weight ratio in LSM is 260 – 720, and in WSM it is 220 – 890. When the number of bolts were changed to 1 bolt for both ends fixed and both ends hinged condition in LSM, 1 rivet in WSM, the results were different. The section ranging from 50×50×3 to 90×90×12 showed that the range of strength/weight ratio in LSM is more than WSM, except for the section 90×90×6 which showed different result. The range of strength/weight from 110×110×8 to 100×100×12 was seen to be fluctuating and did not show any proper pattern. The sections from 110×110×6 to 200×200×25 showed that strength/weight ratio range in WSM is more than LSM, except for the section 110×110×15. As shown in Figure (5.45) and Figure (5.46).

5. In compression members for the length of 0.5m, the average value obtained in LSM and WSM are 2725.6 and 3765.8 respectively, these were the values for end conditions both ends fixed and both ends hinged for 2 bolts in LSM and 2 rivets in WSM. For the same end conditions with 1 bolt in LSM and 1 rivet in WSM the value obtained are 1998.273 and 3765.758 respectively. Now the length has been changed up to 2 m. For length 1 m, the average value obtained in LSM and WSM are 1248.08 and 1628.75 respectively, these were the values for end conditions fixed at both ends and hinged at both ends for 2 bolts in LSM and 2 rivets in WSM. For the same end conditions with 1 bolt in LSM and 1 rivet in WSM, the value obtained are 926.98 and 1210.74 respectively. For length 1.5 m, the average value obtained in LSM and WSM are 730.456 and 870.223 respectively, these were the values for 2 bolts with end conditions both ends fixed and both ends hinged in LSM and 2 rivets in WSM. For the same end conditions with 1 bolt in LSM and 1 rivet in WSM, the value obtained are 554.1042 and 613.2213 respectively. For length 2 m, the average value obtained in LSM and WSM are 470.4261 and 513.1973 respectively, these were the values for end conditions both ends fixed and both ends hinged taking 2 bolts in LSM and 2 rivets in WSM. For the same end conditions considering 1 bolt in LSM and 1 rivet in WSM the value obtained are 365.754 and 347.3487 respectively.

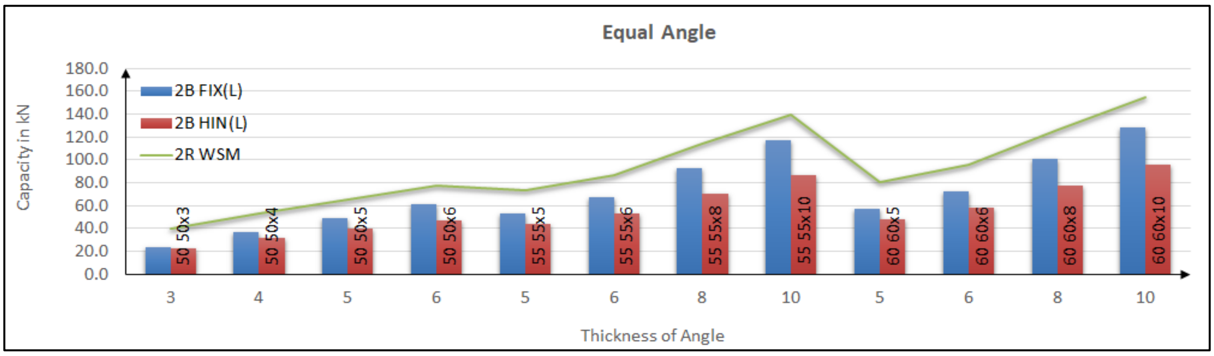


Figure (5.7): Capacity v/s Thickness for length=0.5m, No. of bolts=02, (50×50×3-60×60×10)

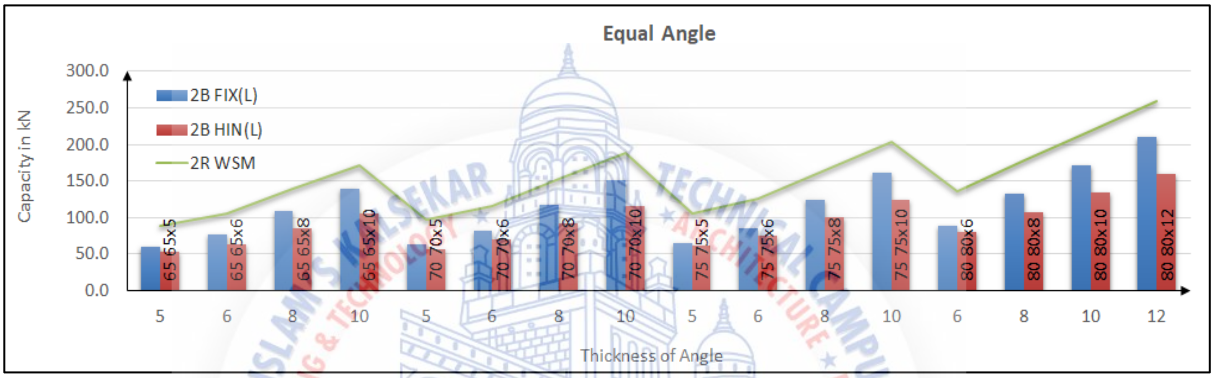


Figure (5.8): Capacity v/s Thickness for length=0.5m, No. of bolts=02, (65×65×5-80×80×12)

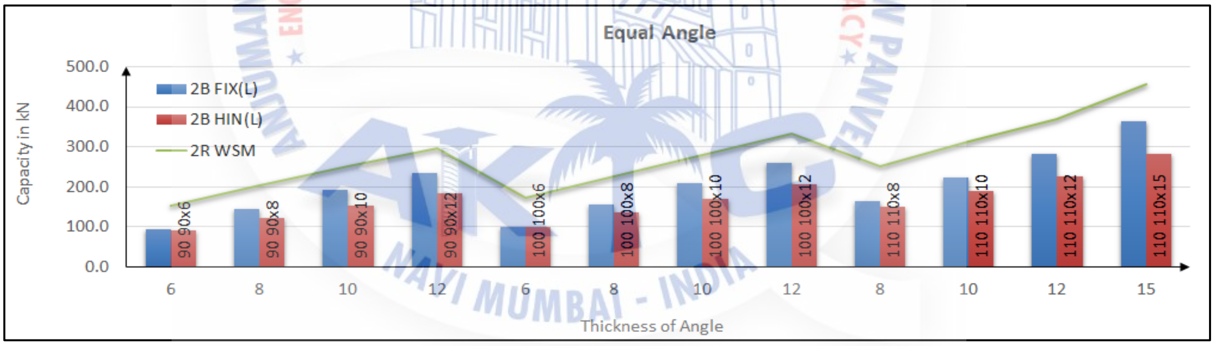


Figure (5.9): Capacity v/s Thickness for length=0.5m, No. of bolts=02, (90×90×6-110×110×15)

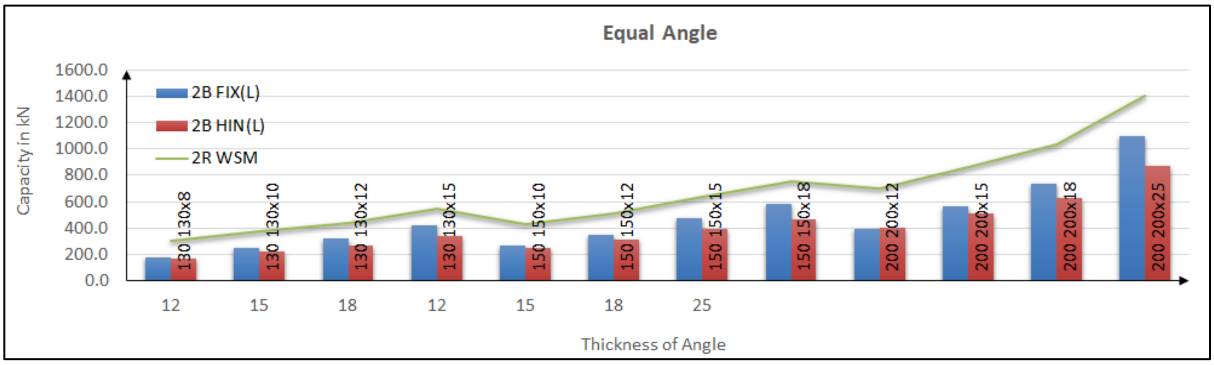


Figure (5.10): Capacity v/s Thickness for length=0.5m, No. of bolts=02, (130×130×8-200×200×25)

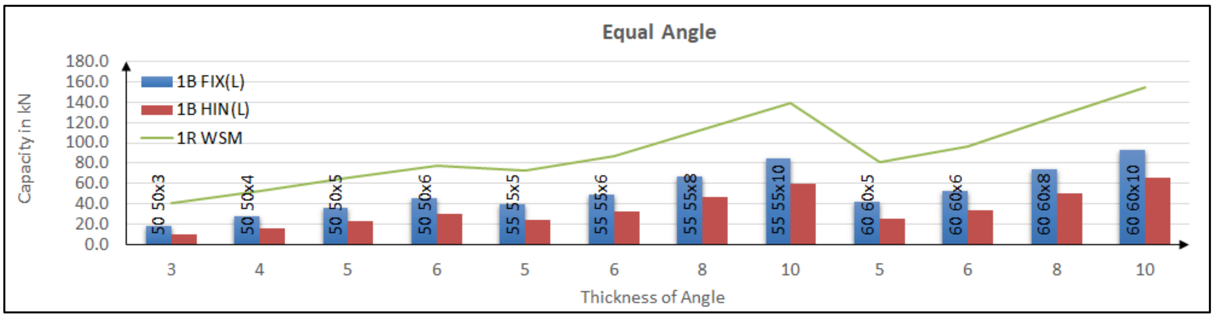


Figure (5.11): Capacity v/s Thickness for length=0.5m, No. of bolts=01, (50×50×3-60×60×10)

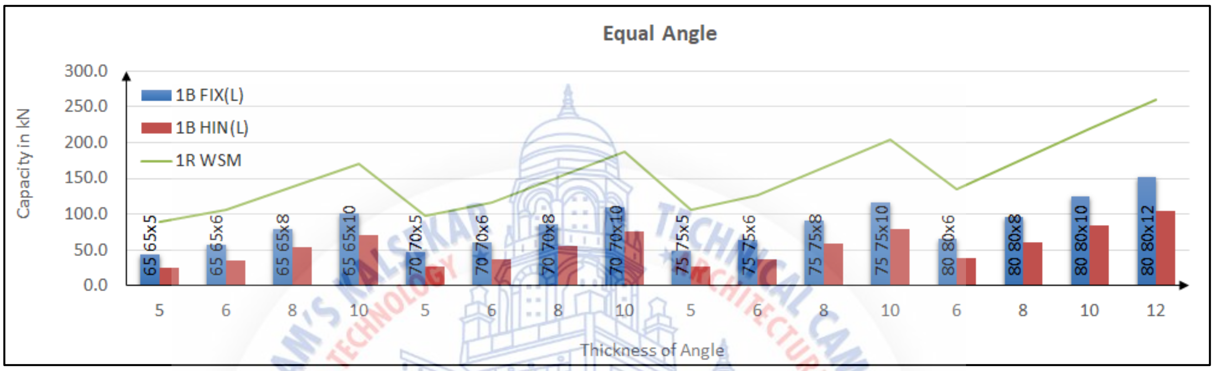


Figure (5.12): Capacity v/s Thickness for length=0.5m, No. of bolts=01, (65×65×5-80×80×12)

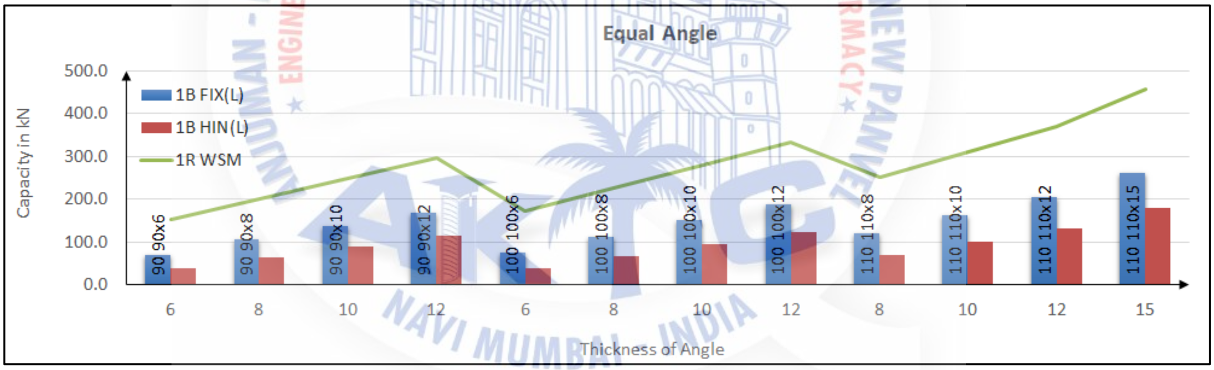


Figure (5.13): Capacity v/s Thickness for length=0.5m, No. of bolts=01, (90×90×6-110×110×15)

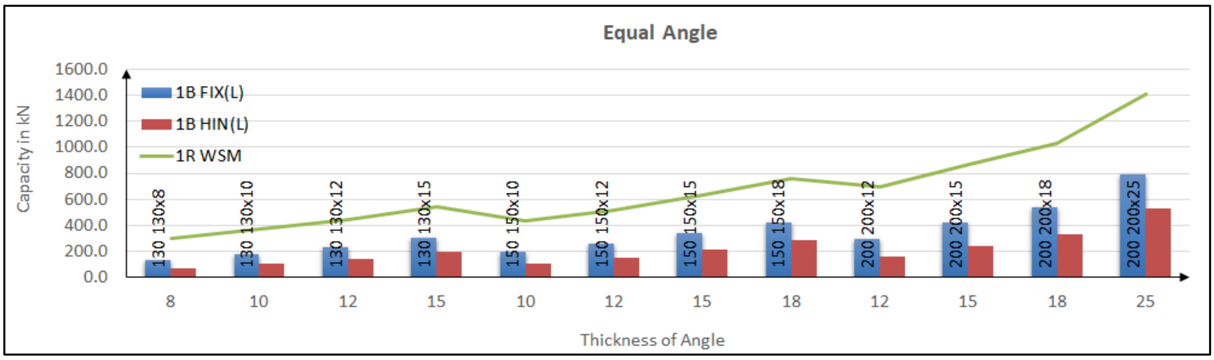


Figure (5.14): Capacity v/s Thickness for length=0.5m, No. of bolts=01, (130×130×-200×200×25)

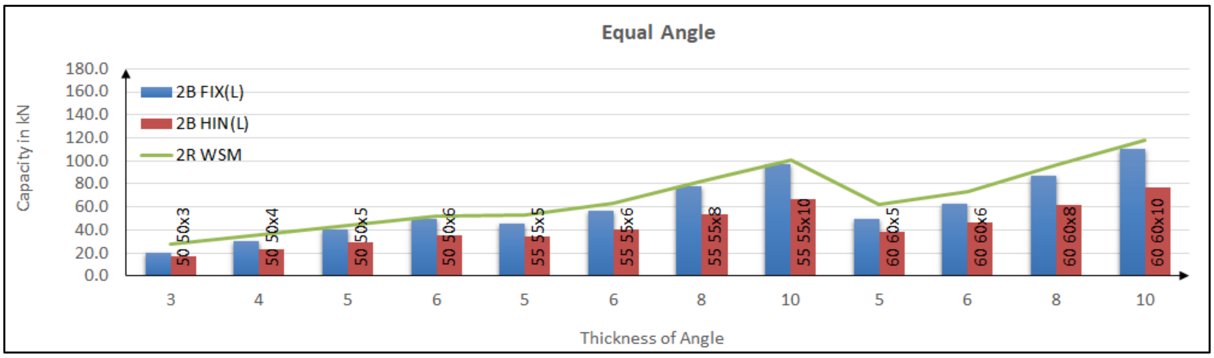


Figure (5.15): Capacity v/s Thickness for length=01m, No. of bolts=02, (50×50×3-60×60×10)

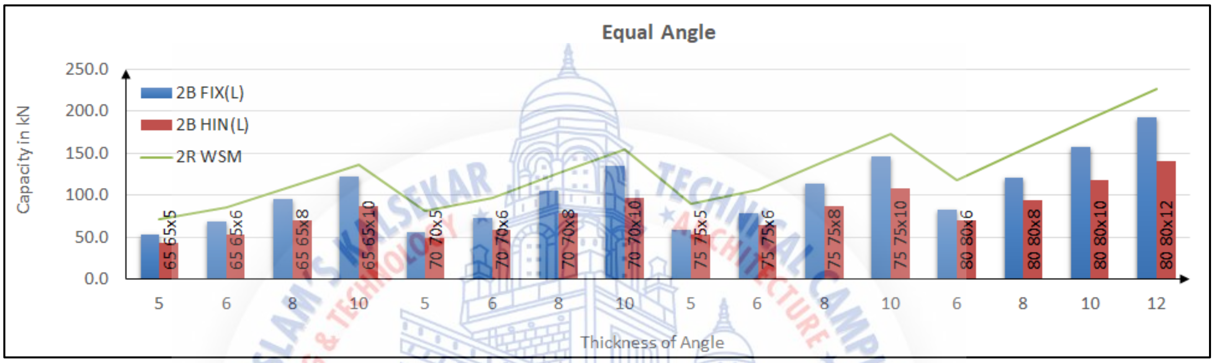


Figure (5.16): Capacity v/s Thickness for length=01m, No. of bolts=02, (65×65×5-80×80×12)

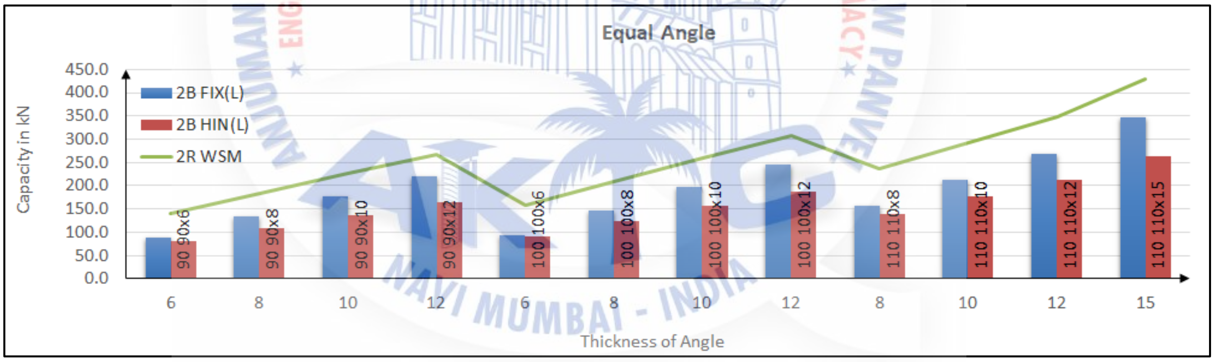


Figure (5.17): Capacity v/s Thickness for length=01m, No. of bolts=02, (90×90×6-110×110×15)

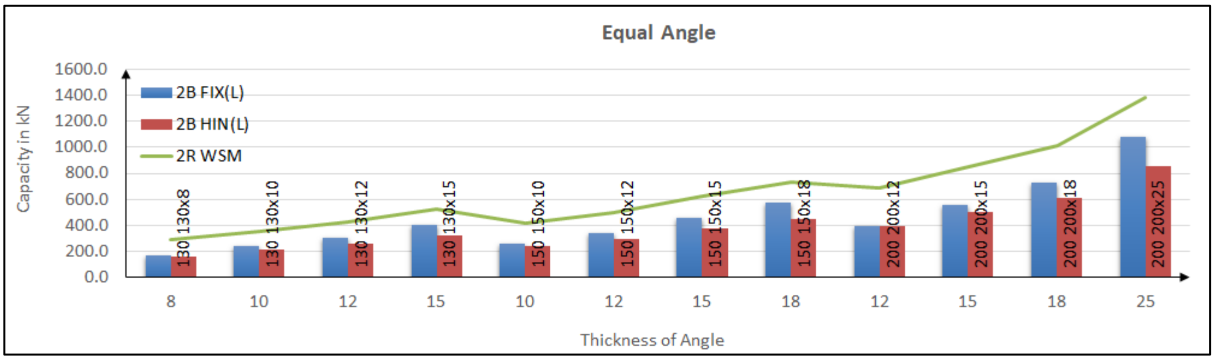


Figure (5.18): Capacity v/s Thickness for length=01m, No. of bolts=02, (50×50×3-60×60×10)

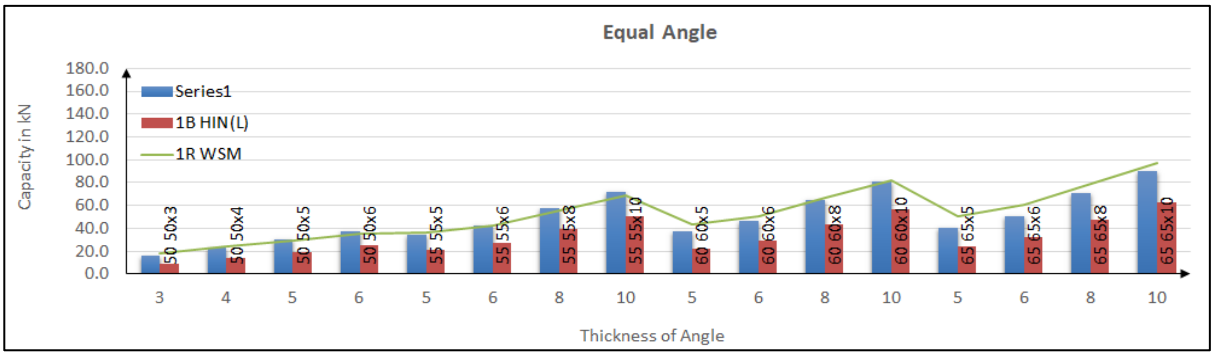


Figure (5.19): Capacity v/s Thickness for length=01m, No. of bolts=01, (50×50×3-60×60×10)

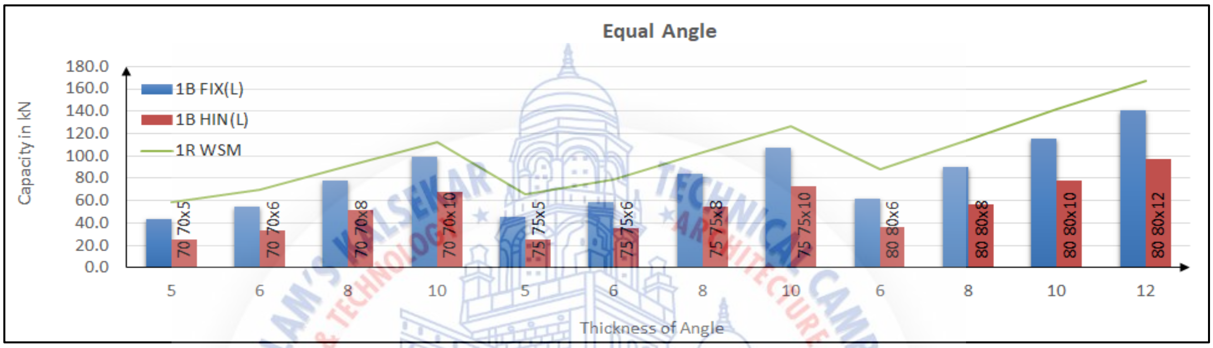


Figure (5.20): Capacity v/s Thickness for length=01m, No. of bolts=01, (70×70×5-80×80×12)

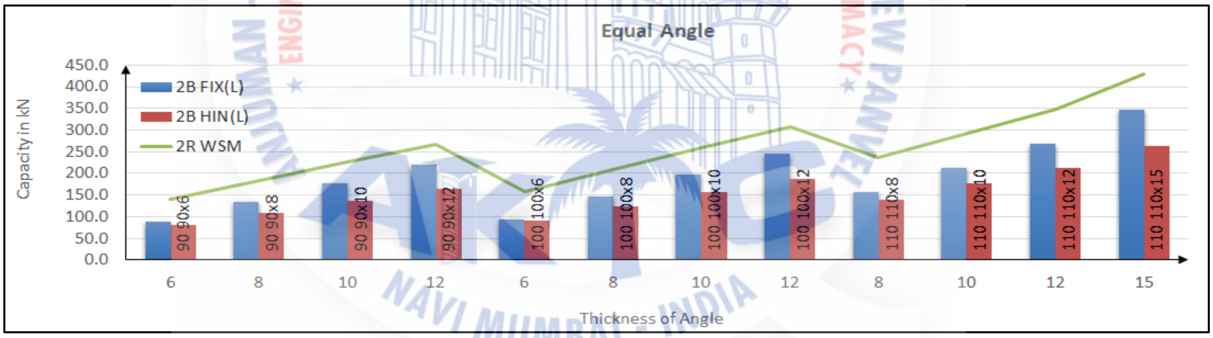


Figure (5.21): Capacity v/s Thickness for length=01m, No. of bolts=01, (90×90×6-110×110×15)

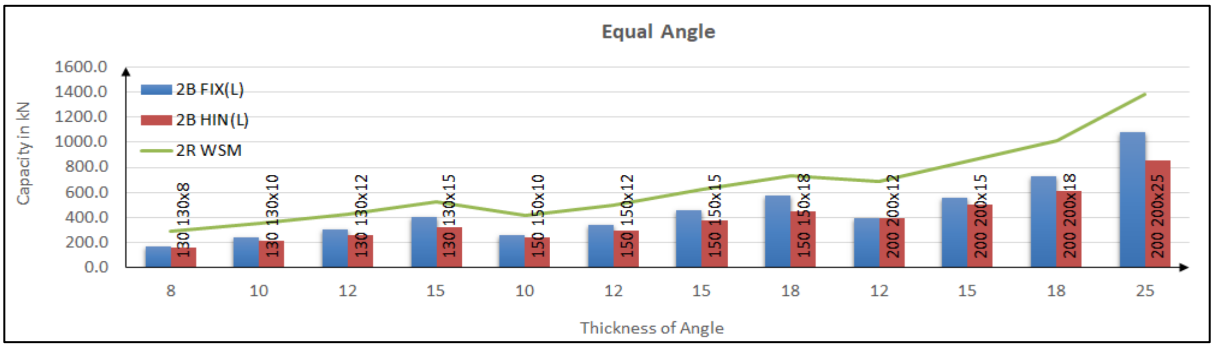


Figure (5.22): Capacity v/s Thickness for length=01m, No. of bolts=01, (130×130×8-200×200×25)

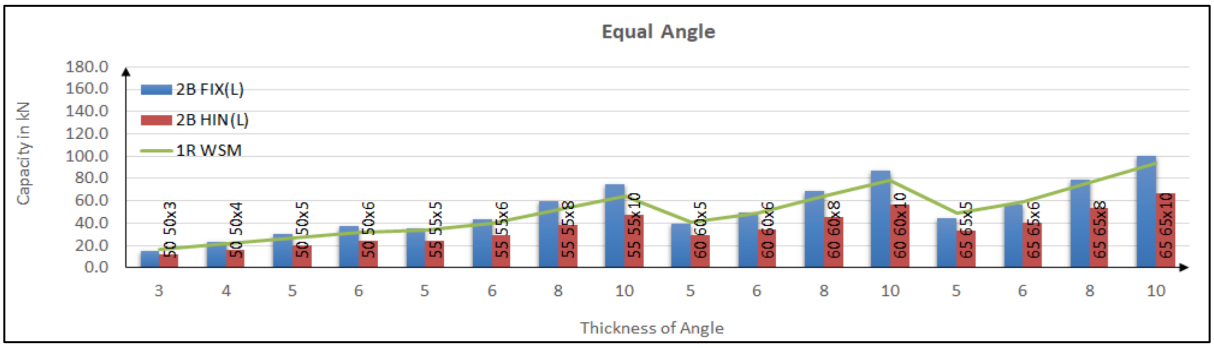


Figure (5.23): Capacity v/s Thickness for length=1.5m, No. of bolts=02, (50×50×3-65×65×10)

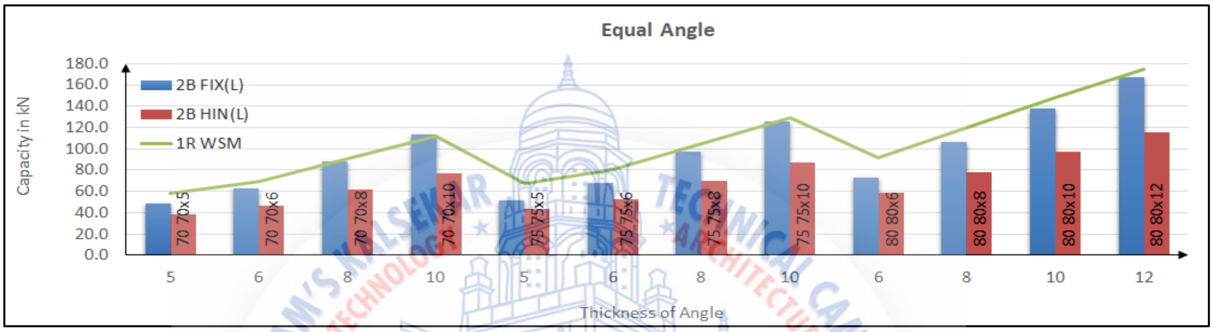


Figure (5.24): Capacity v/s Thickness for length=1.5m, No. of bolts=02, (70×70×5-80×80×12)

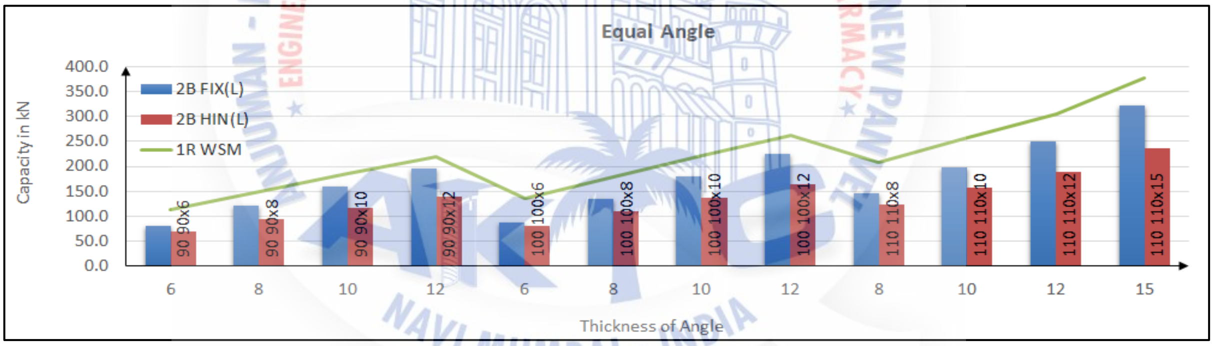


Figure (5.25): Capacity v/s Thickness for length=1.5m, No. of bolts=02, (90×90×6-110×110×15)

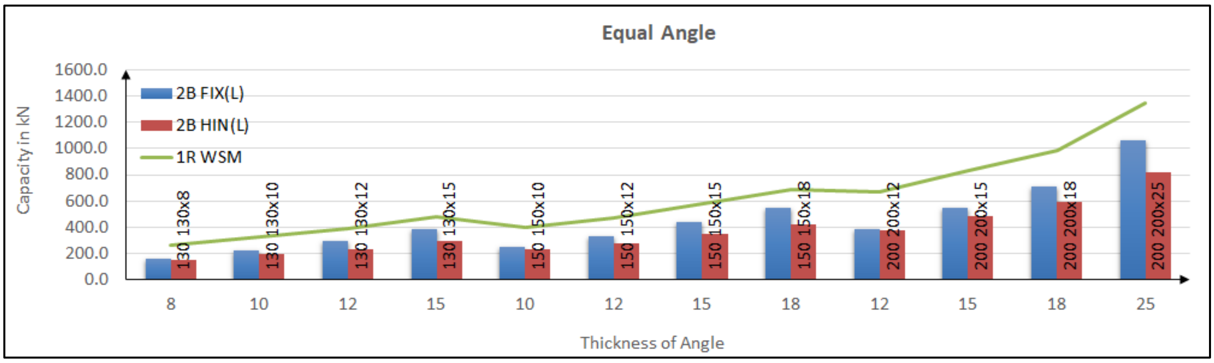


Figure (5.26): Capacity v/s Thickness for length=1.5m, No. of bolts=02, (130×130×8-200×200×25)

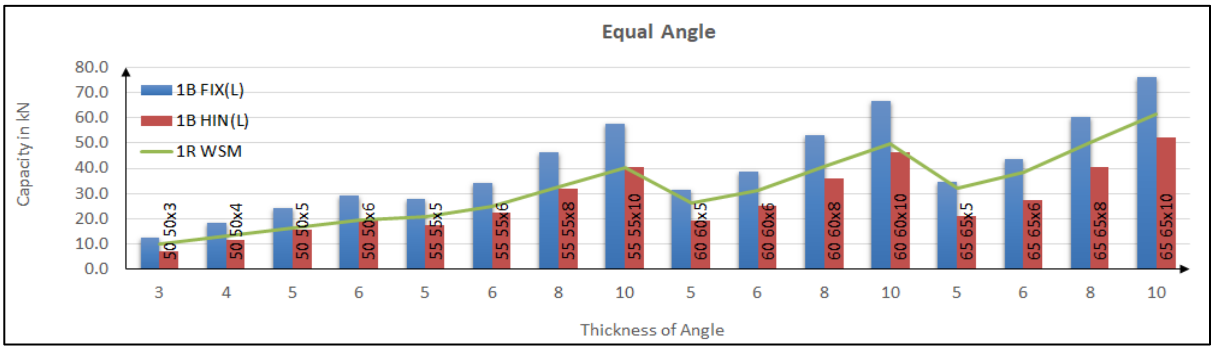


Figure (5.27): Capacity v/s Thickness for length=1.5m, No. of bolts=01, (50×50×3-65×65×10)

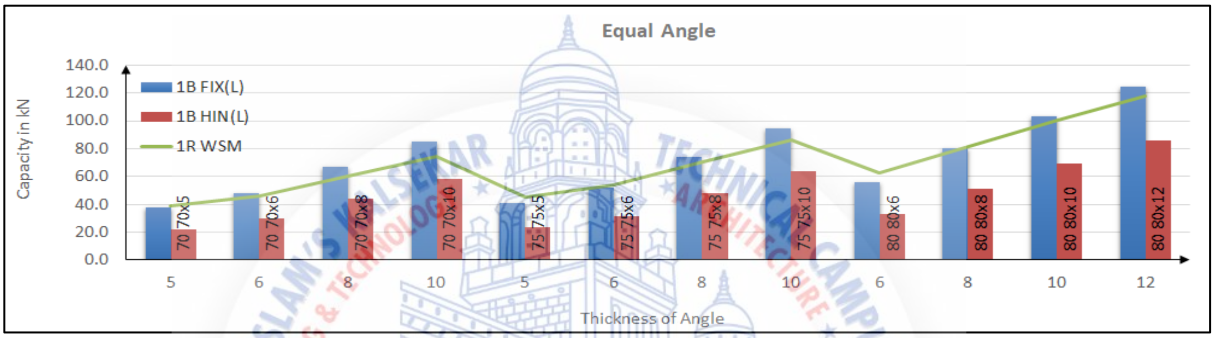


Figure (5.28): Capacity v/s Thickness for length=1.5m, No. of bolts=01, (70×70×5-80×80×12)

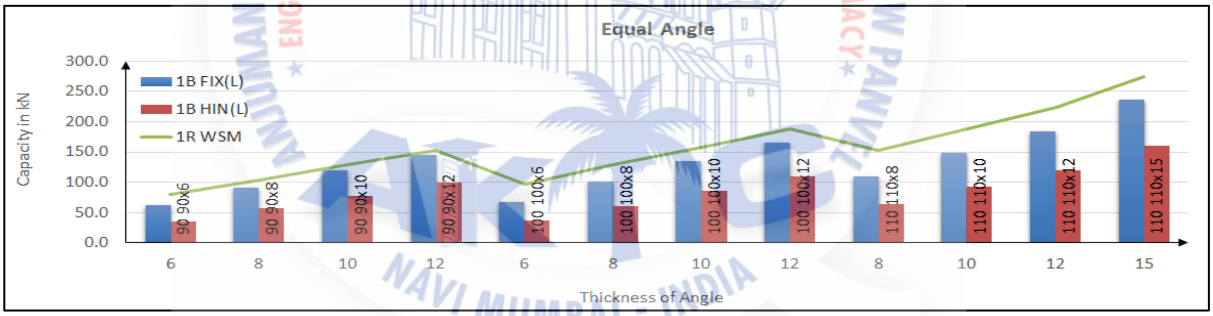


Figure (5.29): Capacity v/s Thickness for length=1.5m, No. of bolts=01, (90×90×6-110×110×15)

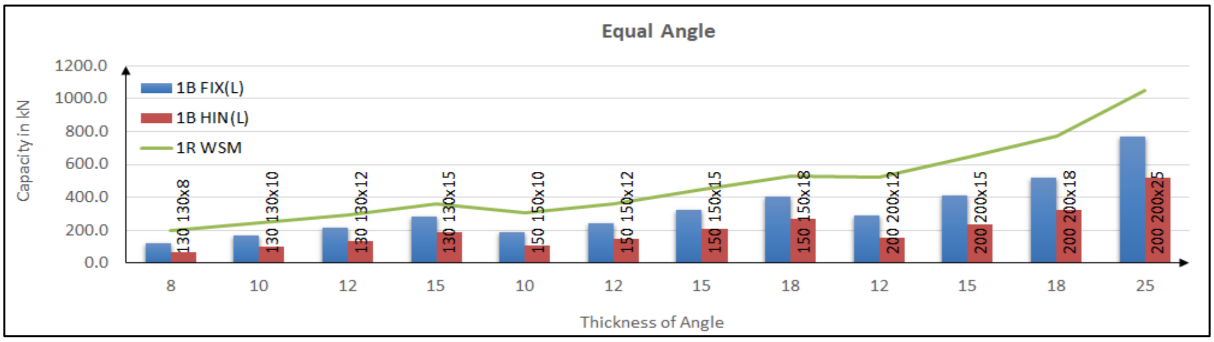


Figure (5.30): Capacity v/s Thickness for length=1.5m, No. of bolts=01, (130×130×8-200×200×25)

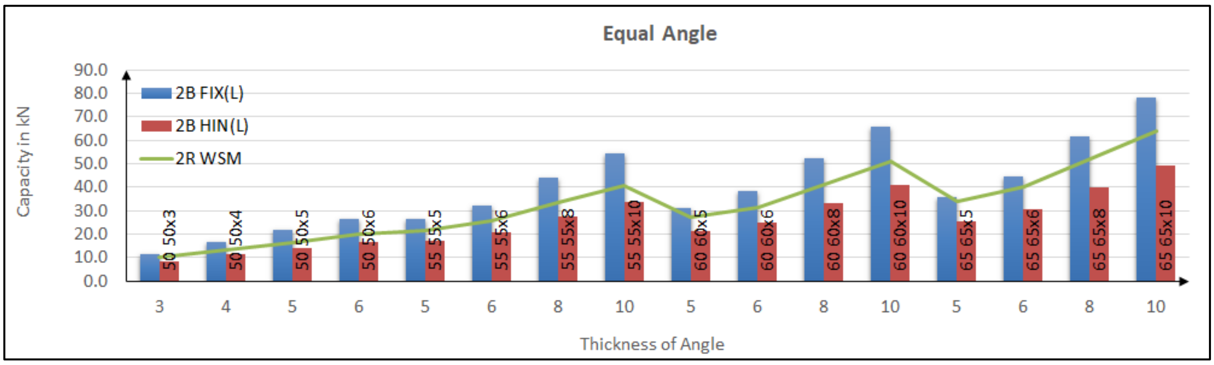


Figure (5.31): Capacity v/s Thickness for length=2.0m, No. of bolts=02, (50×50×3-65×65×10)

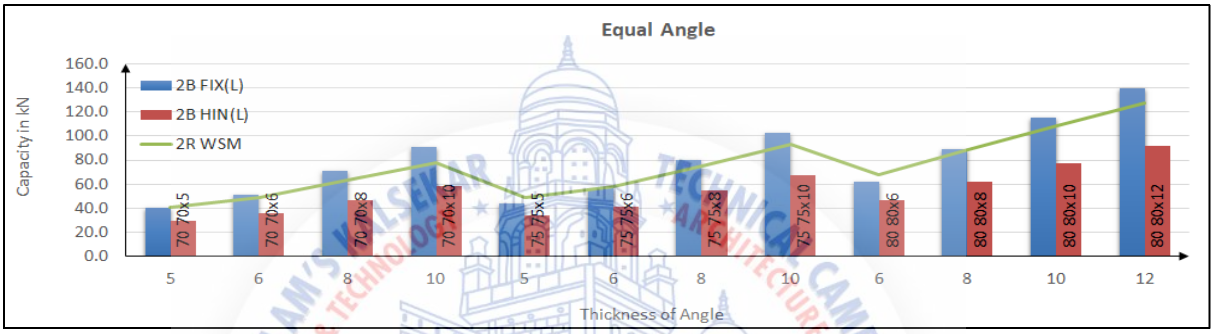


Figure (5.32): Capacity v/s Thickness for length=2.0m, No. of bolts=02, (70×70×5-80×80×12)

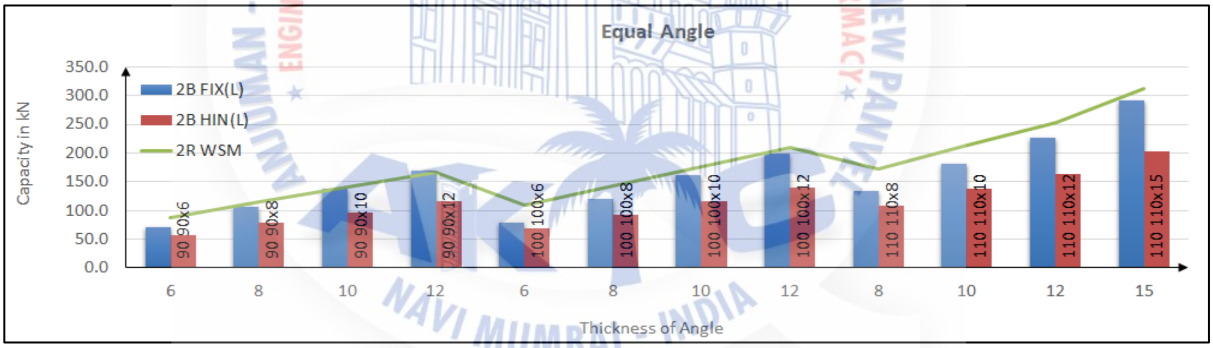


Figure (5.33): Capacity v/s Thickness for length=2.0m, No. of bolts=02, (90×90×6-110×110×15)

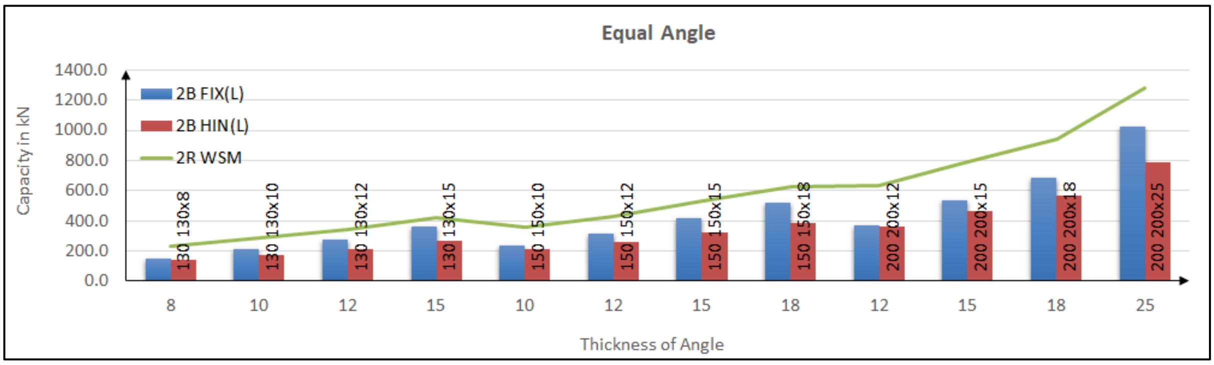


Figure (5.34): Capacity v/s Thickness for length=2.0m, No. of bolts=02, (130×130×8-200×200×25)

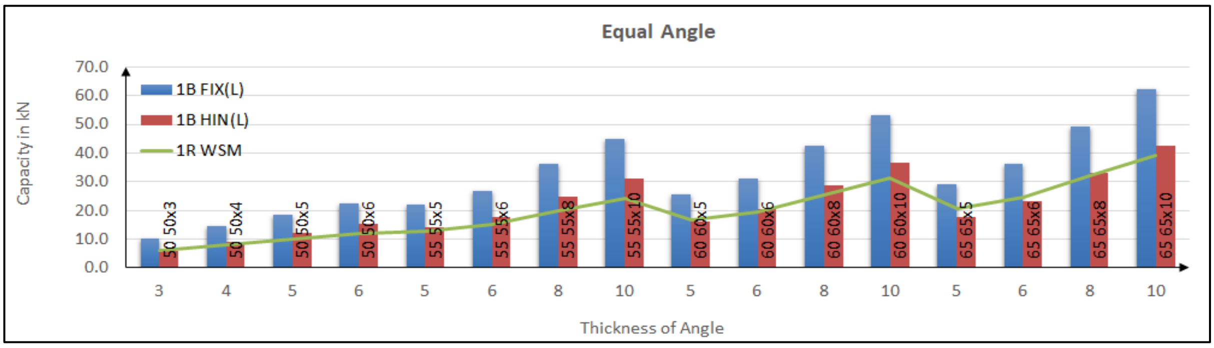


Figure (5.35): Capacity v/s Thickness for length=2.0m, No. of bolts=01, (50×50×3-65×65×10)

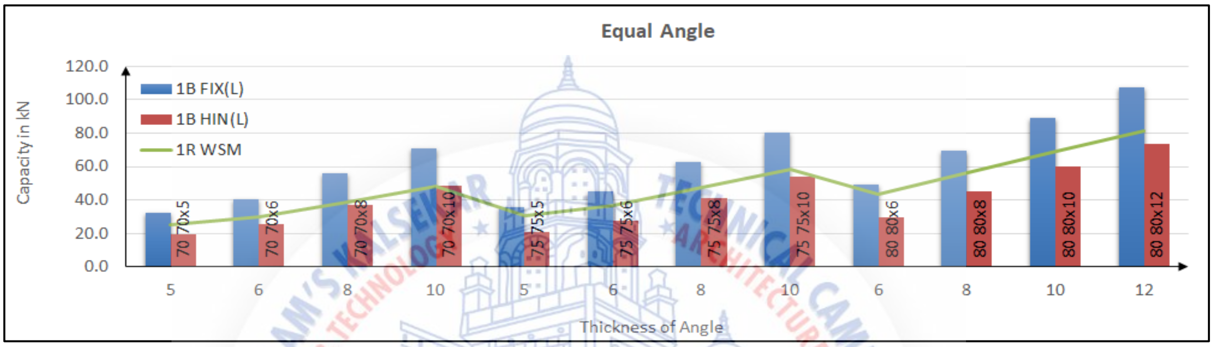


Figure (5.36): Capacity v/s Thickness for length=2.0m, No. of bolts=01, (70×70×5-80×80×12)

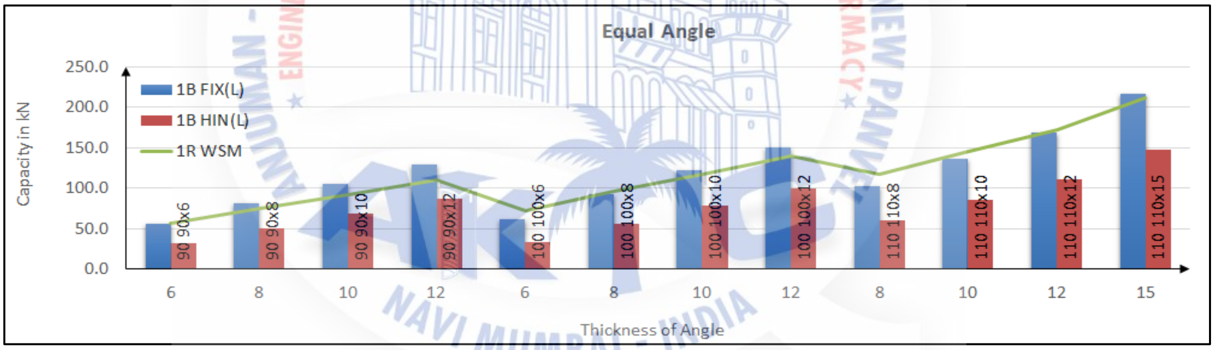


Figure (5.37): Capacity v/s Thickness for length=2.0m, No. of bolts=01, (90×90×6-110×110×15)

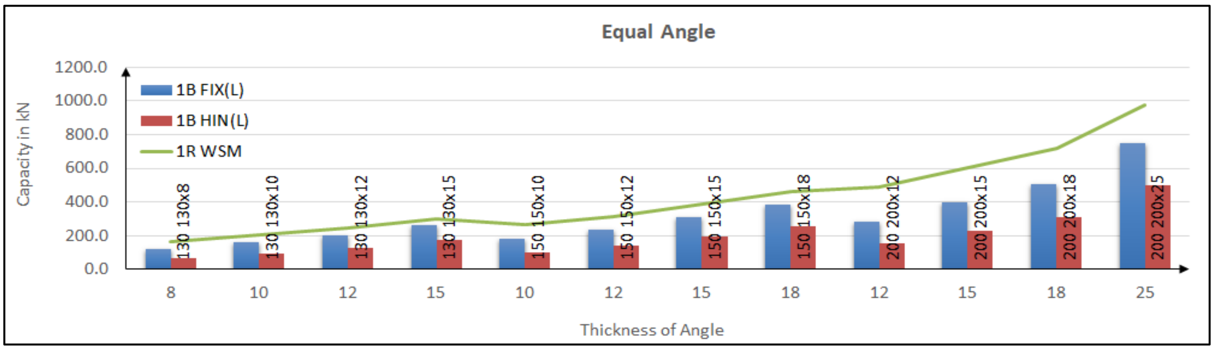


Figure (5.38): Capacity v/s Thickness for length=2.0m, No. of bolts=01, (130×130×8-200×200×25)

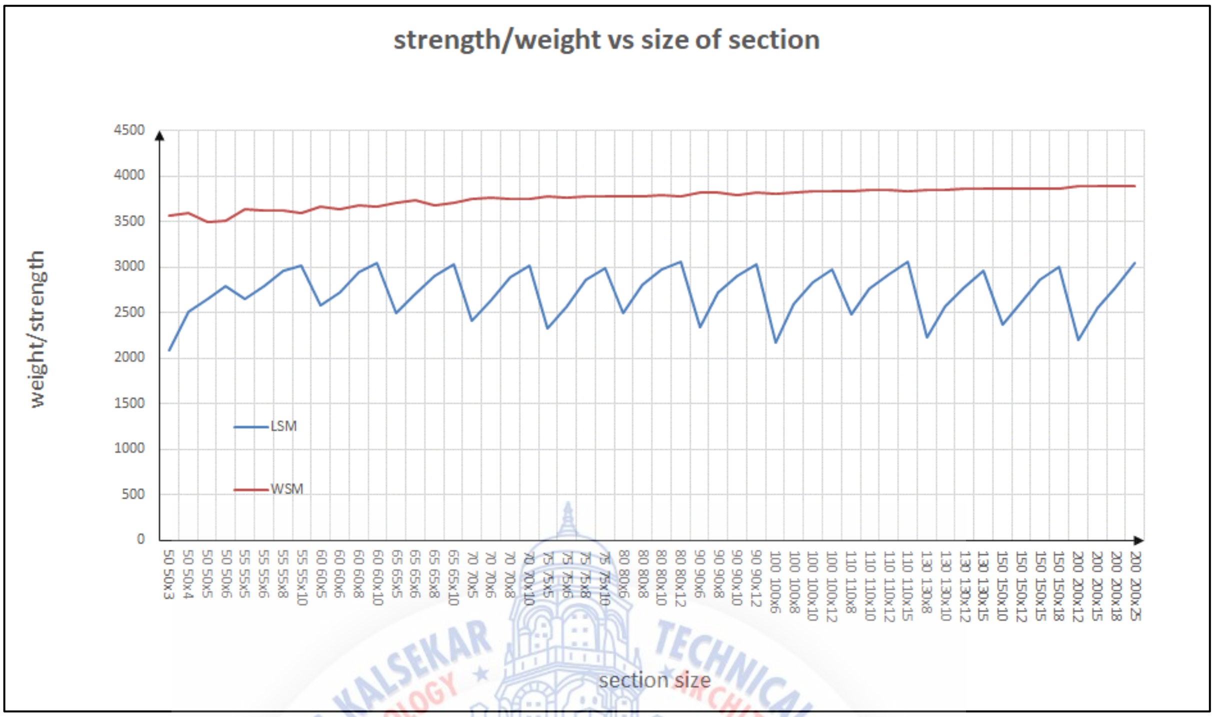


Figure (5.39): Strength/weight v/s section size for length= 0.5m and no. of bolt=2

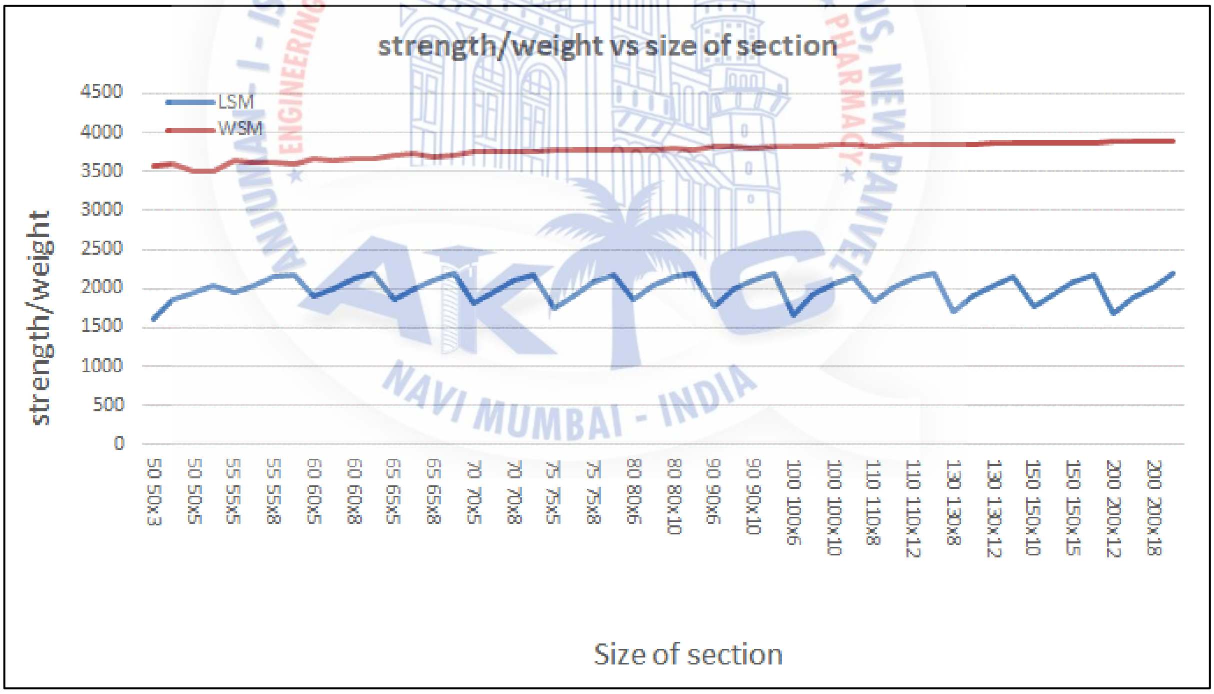


Figure (5.40): Strength/weight v/s section size for length= 0.5m and no. of bolt=01

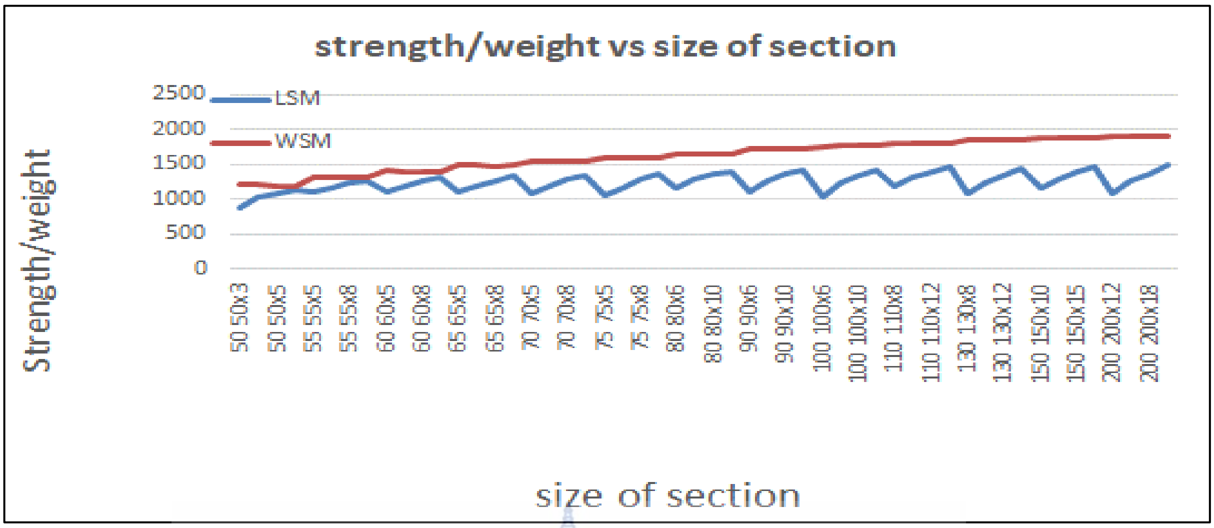


Figure (5.41): Strength/weight v/s section size for length= 1.0m and no. of bolt=2

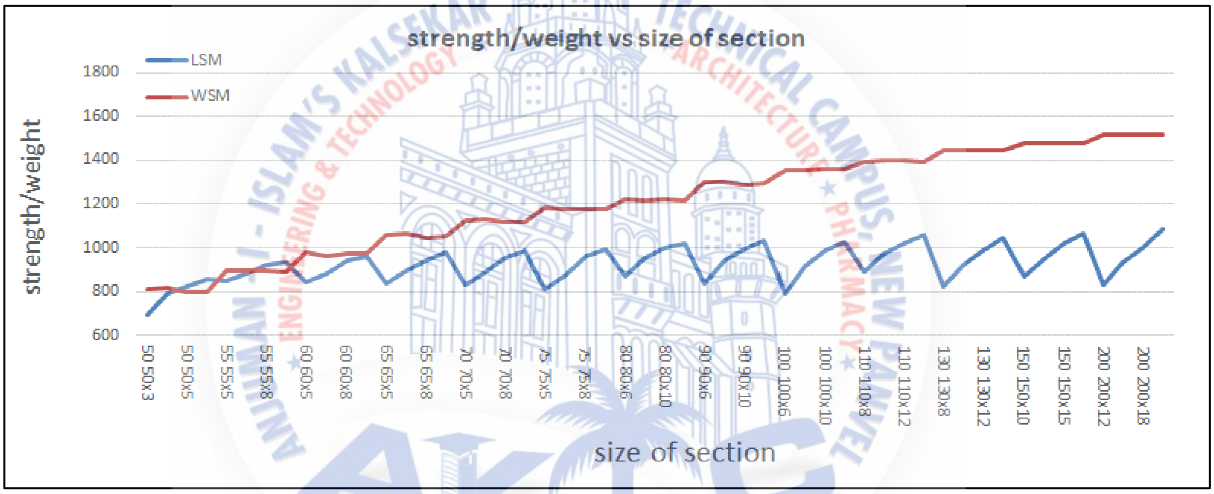


Figure (5.42): Strength/weight v/s section size for length= 1.0 m and no. of bolt=01

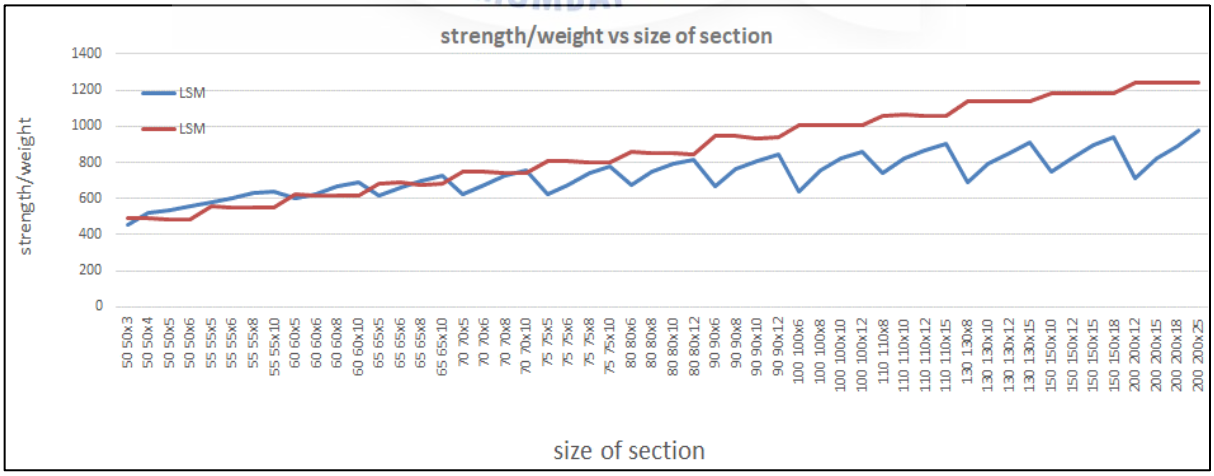


Figure (5.43) Strength/weight v/s section size for length= 1.5m and no. of bolt=2

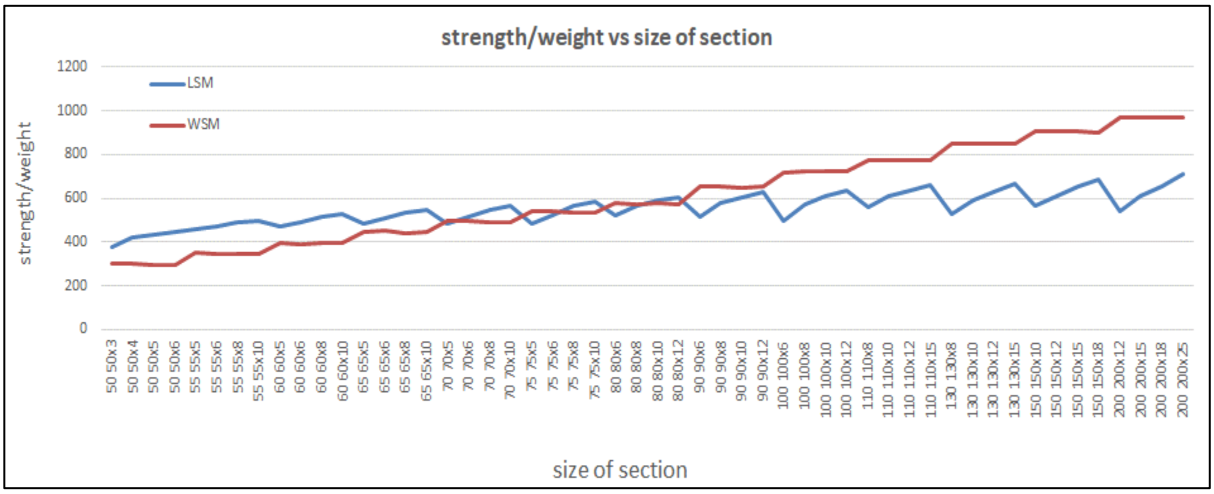


Figure (5.44): Strength/weight v/s section size for length= 1.5m and no. of bolt=01

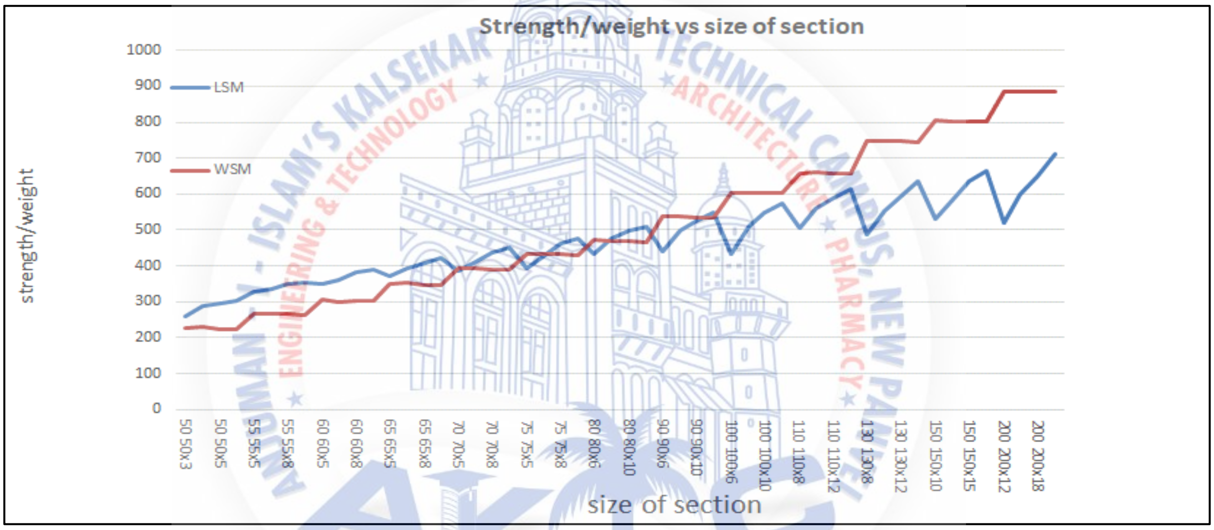


Figure (5.45): Strength/weight v/s section size for length= 2.0m and no. of bolt=2

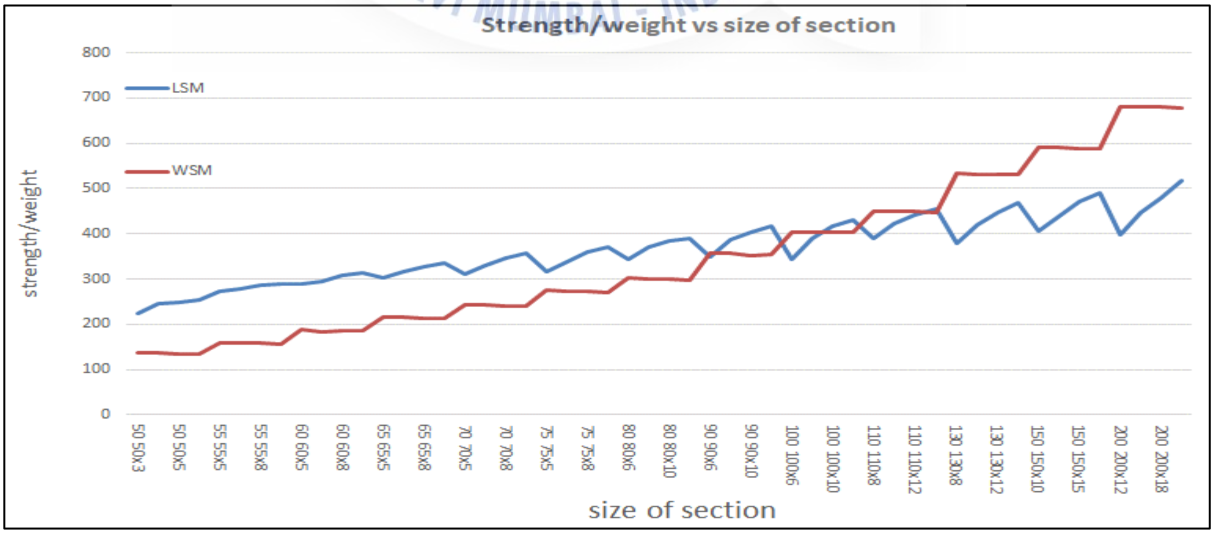


Figure (5.46): Strength/weight v/s section size for length= 2.0m and no. of bolt=01

Chapter 6

Conclusions

Tension Member:

Tension members are normally slender member, so while comparing its strength we need to consider its net strength, as it is lesser than gross strength. So, while comparing net strength in both the cases i.e. LSM and WSM, it's clearly seen that LSM is much more economical. Any designer that wants to design tension member should adopt LSM as it is giving lower and economical sections for the same loading.

Compression Member:

While dealing with compression members, we have taken 0.5m – 2m and from the results obtained, it shows that the strength is drastically changing. From length 0.5m – 1m strength of member in LSM is higher as compared to WSM. So here WSM is economical. When we increase length over 1m, we see that the trend of the graphs is slightly changing, and the capacity is approximately equal. So here we need to make judgmental decision. But for higher length over 1.5m the strength acquired in WSM is nearly 20% more than the strength we are getting in LSM.

So, for the economy one can go with WSM for designing compression member when the length is more than 1.5m. We are not taking the sections more than 2m because the sections are failing above 2m. This may be due to the formula we have used, which considers only 1 bolt while carrying tensile strength.

Future Scope of The Project:

In order to keep the present study within permissible limits, the scope has been restricted to design of tension member using unequal angles and design of compression member using unequal angles. For the projects to come the study can be done the vice-versa of the above.

The same study can be further expanded for channel section and I-section.

The increment done in the graphs in this project is 0.5m. In future the increment can be taken more.

We have considered here only 1 bolt in cross section, so the same study can further be extended by considering actual numbers of bolts.

We have approximately calculated the Net strength of angle sections as per clause 6.3.3. IS 800:2007, the same study can be further expanded by considering formula for β.

Further we can extend the same program in excel by adding calculation of welded connection.

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Appendix – A

Design Methodology of IS 800:1984

Tension member

2	size of section			Kg/m	cm3	size of section	ANC	AGO	Total A	Area in mm2	Total A
3	30	20 X	3	1.1	1.41	40 30 X 3	85.5	55.5	141	141	141
4	30	20 X	4	1.4	1.84	40 30 X 4	112	72	184	184	184
5	30	20 X	5	1.8	2.25	40 30 X 5	137.5	87.5	225	225	225
6	40	25 X	3	1.5	1.88	40 25 X 3	115.5	70.5	186	188	186
7	40	25 X	4	1.9	2.46	40 25 X 4	152	92	244	246	244
8	40	25 X	5	2.4	3.02	40 25 X 5	187.5	112.5	300	302	300
9	40	25 X	6	2.8	3.56	40 25 X 6	222	132	354	356	354
10	40	30 X	3	1.7	2.18	40 30 X 3	115.5	85.5	201	218	201
11	40	30 X	4	2.2	2.86	40 30 X 4	152	112	264	286	264
12	40	30 X	5	2.8	3.52	40 30 X 5	187.5	137.5	325	352	325
13	40	30 X	6	3.3	4.16	40 30 X 6	222	162	384	416	384
14	50	30 X	6	3.5	4.47	50 30 X 6	282	162	444	447	444
15	50	30 X	3	1.8	2.34	50 30 X 3	145.5	85.5	231	234	231
16	50	30 X	4	2.4	3.07	50 30 X 4	192	112	304	307	304
17	50	30 X	5	3	3.78	50 30 X 5	237.5	137.5	375	378	375
18	60	40 X	5	3.7	4.76	60 40 X 5	287.5	187.5	475	476	475

1	23				Kg/m	cm2	mm2	TDG		TDN				A1	A2	K	An	T
2	23	2	3	4	5	6												
3	50	30	X	3	1.8	2.34	234	35.3	12	33.3	1.59	0.83	13.5	105	85.5	0.786516854	172.24719	25
4	50	30	X	4	2.4	3.07	307	46.0	12	43.29	1.58	0.82	13.5	138	112	0.787072243	226.15209	33
5	50	30	X	5	3	3.78	378	56.6	12	53.28	1.57	0.81	13.5	170	137.5	0.787644788	278.30116	41
6	50	30	X	6	3.5	4.47	447	67.3	12	63.27	1.56	0.80	13.5	201	162	0.788235294	328.69412	49
7	60	40	X	5	3.7	4.76	476	71.9	12	70.596	1.89	1.12	13.5	220	187.5	0.778761062	366.0177	54
8	60	40	X	6	4.4	5.65	565	85.2	12	83.916	1.88	1.11	13.5	261	222	0.779104478	433.96119	65
9	60	40	X	8	5.8	7.37	737	111.2	12	109.224	1.86	1.10	13.5	340	288	0.779816514	564.58716	84
10	65	45	X	5	4.1	5.26	526	79.3	12	79.254	2.05	1.28	13.5	245	212.5	0.775725594	409.84169	61
11	65	45	X	6	4.9	6.25	625	94.6	12	93.906	2.04	1.27	13.5	291	252	0.776	486.552	72
12	65	45	X	8	6.4	8.17	817	123.2	12	122.544	2.02	1.25	13.5	380	328	0.776566757	634.7139	95
13	70	45	X	5	4.3	5.52	552	83.3	16	80.586	2.22	1.26	17.5	250	212.5	0.779220779	415.58442	62
14	70	45	X	6	5.2	6.56	656	99.2	16	95.238	2.21	1.25	17.5	297	252	0.779527559	493.44094	74
15	70	45	X	8	6.7	8.58	858	129.9	16	124.542	2.19	1.24	17.5	388	328	0.780160858	643.89276	96
16	70	45	X	10	8.3	10.5	1050	158.5	16	151.848	2.16	1.22	17.5	475	400	0.780821918	787.32877	118
17	75	50	X	5	4.7	6.02	602	90.6	16	89.244	2.38	1.42	17.5	275	237.5	0.776470588	459.41176	68
18	75	50	X	6	5.6	7.16	716	107.9	16	105.894	2.37	1.41	17.5	327	282	0.77672209	546.03563	81
19	75	50	X	8	7.4	9.38	938	141.9	16	138.528	2.85	1.40	17.5	428	368	0.777239709	714.02421	107
20	75	50	X	10	9	11.5	1150	173.8	16	169.164	2.33	1.38	17.5	525	450	0.777777778	875	131
21	80	50	X	5	4.9	6.27	627	94.6	20	89.91	2.55	1.40	21.5	280	237.5	0.779582367	465.15081	69
22	80	50	X	6	5.9	7.46	746	112.6	20	107.226	2.54	1.39	21.5	333	282	0.779859485	552.92037	82
23	80	50	X	8	7.7	9.78	978	147.9	20	139.86	2.52	1.37	21.5	436	368	0.780429594	723.19809	108
24	80	50	X	10	9.4	12	1200	181.2	20	171.162	2.49	1.36	21.5	535	450	0.781021898	886.45985	132
25	90	60	X	6	6.8	8.65	865	130.5	20	127.872	2.86	1.71	21.5	393	342	0.775147929	658.10059	98

Screenshots of Excel Sheets for Tension Members

Sample problem: design a tension member of section 50×30×3 mm

(a) Design for the tension member by working stress method

Let us provide 12mm diameter power driven rivets for the connections

Gross diameter of rivet, d = 12+1.5 = 13.5 mm

Area of connected leg, A_1 = $\left(50 - 13.5 - \frac{3}{2}\right) \times 3 = 105\ mm^2$

Area of outstanding leg, A_2 = $\left(30 - \frac{3}{2}\right) \times 3 = 85.5\ mm^2$

k = $\frac{3A_1}{3A_1 + A_2}$ = $\frac{3 \times 105}{3 \times 105 + 85.5}$ = 0.7865

Net area required = $A_1 + kA_2$ =105+ 0.7865 x 85.5

= 172.247 mm^2

Strength of the member = (150× $Net\ area\ required)/1000$

= $\frac{150 \times 172.247}{1000}$ = 25.837kN.

(b) Design for the tension member by limit state method

I.S.A 50 x 30 x 3 mm @ 1.8 kg/m

Area of the section = 234mm^2

gross Strength of the member = $\frac{A_g f_y}{\gamma_{m0}}$ = $\frac{234 \times 250}{1.1 \times 1000}$ = 53.18181 kN

Using factor of safety = $\frac{1}{1.5}$

gross Strength of the member = $\frac{1}{1.5} \times\ 53.181$ = 35.1kN

Net area of the section = Area of the section – no. of bolt in the cross section × (total no of bolt - 2) × thickness of the angle

Net area of the section = 234 – 1 × (12 - 2) × 3 =204 mm^2

Net Strength of the member = $\frac{0.9 A_g f_u}{\gamma_{m1}}$

= $\frac{0.8 \times 204 \times 410}{1.25 \times 1000}$

= 53.5296kN

Using factor of safety = $\frac{1}{1.5}$

Net Strength of the member = $\frac{1}{1.5} \times 53.5296 = 35.65$Kn

Design Methodology of IS 800:2007

Compression Member

1						kg/m	cm2	rxx=ryy	ruu	rvv	X	x1	x2	y1	y2	Fcd	PD						kg/m	cm2	rxx=ryy	ruu	rvv	X	x1	x2	y1	y2		
2		1	2	3	4	5	6	10	11	12.00	2								1	2	3	4	5	6	10	11	12.00	2						
3	1	20	20	X	3	0.9	1.12	0.58	0.73	0.37	459.459459	450.00	460	#N/A	#N/A	#N/A	#N/A	1	20	20	X	3	0.9	1.12	0.58	0.73	0.37	540.540541	540.00	550	#N/A	#N/A	#N/A	#N/A
4	2	20	20	X	4	1.1	1.45	0.58	0.72	0.37	459.459459	450.00	460	#N/A	#N/A	#N/A	#N/A	2	20	20	X	4	1.1	1.45	0.58	0.72	0.37	540.540541	540.00	550	#N/A	#N/A	#N/A	#N/A
5	3	25	25	X	3	1.1	1.41	0.73	0.93	0.47	361.702128	360.00	370	#N/A	#N/A	#N/A	#N/A	3	25	25	X	3	1.1	1.41	0.73	0.93	0.47	425.531915	420.00	430	#N/A	#N/A	#N/A	#N/A
6	4	25	25	X	4	1.4	1.84	0.73	0.91	0.47	361.702128	360.00	370	#N/A	#N/A	#N/A	#N/A	4	25	25	X	4	1.4	1.84	0.73	0.91	0.47	425.531915	420.00	430	#N/A	#N/A	#N/A	#N/A
7	5	25	25	X	5	1.8	2.25	0.72	0.91	0.47	361.702128	360.00	370	#N/A	#N/A	#N/A	#N/A	5	25	25	X	5	1.8	2.25	0.72	0.91	0.47	425.531915	420.00	430	#N/A	#N/A	#N/A	#N/A
8	6	30	30	X	3	1.4	1.73	0.89	1.13	0.57	298.245614	290.00	300	#N/A	#N/A	#N/A	#N/A	6	30	30	X	3	1.4	1.73	0.89	1.13	0.57	350.877193	350.00	360	#N/A	#N/A	#N/A	#N/A
9	7	30	30	X	4	1.8	2.26	0.89	1.12	0.57	298.245614	290.00	300	#N/A	#N/A	#N/A	#N/A	7	30	30	X	4	1.8	2.26	0.89	1.12	0.57	350.877193	350.00	360	#N/A	#N/A	#N/A	#N/A
10	8	30	30	X	5	2.2	2.77	0.88	1.11	0.57	298.245614	290.00	300	#N/A	#N/A	#N/A	#N/A	8	30	30	X	5	2.2	2.77	0.88	1.11	0.57	350.877193	350.00	360	#N/A	#N/A	#N/A	#N/A
11	9	35	35	X	3	1.6	2.03	1.05	1.33	0.67	253.731343	250.00	260	18	#N/A	#N/A	#N/A	9	35	35	X	3	1.6	2.03	1.05	1.33	0.67	298.507463	290.00	300	#N/A	#N/A	#N/A	#N/A
12	10	35	35	X	4	2.1	2.66	1.05	1.32	0.67	253.731343	250.00	260	18	#N/A	#N/A	#N/A	10	35	35	X	4	2.1	2.66	1.05	1.32	0.67	298.507463	290.00	300	#N/A	#N/A	#N/A	#N/A
13	11	35	35	X	5	2.6	3.27	1.04	1.31	0.67	253.731343	250.00	260	18	#N/A	#N/A	#N/A	11	35	35	X	5	2.6	3.27	1.04	1.31	0.67	298.507463	290.00	300	#N/A	#N/A	#N/A	#N/A
14	12	35	35	X	6	3	3.86	1.03	1.29	0.67	253.731343	250.00	260	18	#N/A	#N/A	#N/A	12	35	35	X	6	3	3.86	1.03	1.29	0.67	298.507463	290.00	300	#N/A	#N/A	#N/A	#N/A
15	13	40	40	X	3	1.8	2.34	1.21	1.54	0.77	220.779221	220.00	230	23	21	22.84	5.3	13	40	40	X	3	1.8	2.34	1.21	1.54	0.77	259.74026	250.00	260	18	#N/A	#N/A	#N/A
16	14	40	40	X	4	2.4	3.07	1.21	1.53	0.77	220.779221	220.00	230	23	21	22.84	7.0	14	40	40	X	4	2.4	3.07	1.21	1.53	0.77	259.74026	250.00	260	18	#N/A	#N/A	#N/A
17	15	40	40	X	5	3	3.78	1.2	1.51	0.77	220.779221	220.00	230	23	21	22.84	8.6	15	40	40	X	5	3	3.78	1.2	1.51	0.77	259.74026	250.00	260	18	#N/A	#N/A	#N/A
18	16	40	40	X	6	3.5	4.47	1.19	1.5	0.77	220.779221	220.00	230	23	21	22.84	10.2	16	40	40	X	6	3.5	4.47	1.19	1.5	0.77	259.74026	250.00	260	18	#N/A	#N/A	#N/A
19	17	45	45	X	3	2.1	2.64	1.38	1.74	0.87	195.402299	190.00	200	30	28	28.92	7.6	17	45	45	X	3	2.1	2.64	1.38	1.74	0.87	229.885057	220.00	230	23	21	21.02	4.4
20	18	45	45	X	4	2.7	3.47	1.37	1.73	0.87	195.402299	190.00	200	30	28	28.92	10.0	18	45	45	X	4	2.7	3.47	1.37	1.73	0.87	229.885057	220.00	230	23	21	21.02	5.8
21	19	45	45	X	5	3.4	4.28	1.36	1.72	0.87	195.402299	190.00	200	30	28	28.92	12.4	19	45	45	X	5	3.4	4.28	1.36	1.72	0.87	229.885057	220.00	230	23	21	21.02	7.2
22	20	45	45	X	6	4	5.07	1.35	1.7	0.87	195.402299	190.00	200	30	28	28.92	14.7	20	45	45	X	6	4	5.07	1.35	1.7	0.87	229.885057	220.00	230	23	21	21.02	8.5
23	21	50	50	X	3	2.3	2.95	1.53	1.94	0.97	175.257732	170.00	180	37	33	34.90	10.3	21	50	50	X	3	2.3	2.95	1.53	1.94	0.97	206.185567	200.00	210	28	25	26.14	6.2
24	22	50	50	X	4	3	3.88	1.53	1.93	0.97	175.257732	170.00	180	37	33	34.90	13.5	22	50	50	X	4	3	3.88	1.53	1.93	0.97	206.185567	200.00	210	28	25	26.14	8.1
25	23	50	50	X	5	3.8	4.79	1.52	1.92	0.97	175.257732	170.00	180	37	33	34.90	16.7	23	50	50	X	5	3.8	4.79	1.52	1.92	0.97	206.185567	200.00	210	28	25	26.14	10.0
26	24	50	50	X	6	4.5	5.68	1.51	1.9	0.97	175.257732	170.00	180	37	33	34.90	19.8	24	50	50	X	6	4.5	5.68	1.51	1.9	0.97	206.185567	200.00	210	28	25	26.14	11.9
27	25	55	55	X	5	4.1	5.27	1.67	2.11	1.06	160.377358	160.00	170	41	37	40.85	21.5	25	55	55	X	5	4.1	5.27	1.67	2.11	1.06	188.679245	180.00	190	33	30	30.40	12.8
28	26	55	55	X	6	4.9	6.26	1.66	2.1	1.06	160.377358	160.00	170	41	37	40.85	25.6	26	55	55	X	6	4.9	6.26	1.66	2.1	1.06	188.679245	180.00	190	33	30	30.40	15.2
29	27	55	55	X	8	6.4	8.18	1.64	2.07	1.06	160.377358	160.00	170	41	37	40.85	33.4	27	55	55	X	8	6.4	8.18	1.64	2.07	1.06	188.679245	180.00	190	33	30	30.40	19.9
30	28	55	55	X	10	7.9	10.02	1.62	2.03	1.06	160.377358	160.00	170	41	37	40.85	40.9	28	55	55	X	10	7.9	10.02	1.62	2.03	1.06	188.679245	180.00	190	33	30	30.40	24.4

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ	AK
1						kg/m	cm2	rxx=ryy	ruu	rvv	X	x1	x2	y1	y2	Fcd	PD							kg/m	cm2	rxx=ryy	ruu	rvv	X	x1	x2	y1	y2				
2		1	2	3	4	5	6	10	11	12.00	2									1	2	3	4	5	6	10	11	12.00	2								
3	1	20	20	X	3	0.9	1.12	0.58	0.73	0.37	459.459459	450.00	460	#N/A	#N/A	#N/A	#N/A		1	20	20	X	3	0.9	1.12	0.58	0.73	0.37	540.540541	540.00	550	#N/A	#N/A	#N/A	#N/A		
4	2	20	20	X	4	1.1	1.45	0.58	0.72	0.37	459.459459	450.00	460	#N/A	#N/A	#N/A	#N/A		2	20	20	X	4	1.1	1.45	0.58	0.72	0.37	540.540541	540.00	550	#N/A	#N/A	#N/A	#N/A		
5	3	25	25	X	3	1.1	1.41	0.73	0.93	0.47	361.702128	360.00	370	#N/A	#N/A	#N/A	#N/A		3	25	25	X	3	1.1	1.41	0.73	0.93	0.47	425.531915	420.00	430	#N/A	#N/A	#N/A	#N/A		
6	4	25	25	X	4	1.4	1.84	0.73	0.91	0.47	361.702128	360.00	370	#N/A	#N/A	#N/A	#N/A		4	25	25	X	4	1.4	1.84	0.73	0.91	0.47	425.531915	420.00	430	#N/A	#N/A	#N/A	#N/A		
7	5	25	25	X	5	1.8	2.25	0.72	0.91	0.47	361.702128	360.00	370	#N/A	#N/A	#N/A	#N/A		5	25	25	X	5	1.8	2.25	0.72	0.91	0.47	425.531915	420.00	430	#N/A	#N/A	#N/A	#N/A		
8	6	30	30	X	3	1.4	1.73	0.89	1.13	0.57	298.245614	290.00	300	#N/A	#N/A	#N/A	#N/A		6	30	30	X	3	1.4	1.73	0.89	1.13	0.57	350.877193	350.00	360	#N/A	#N/A	#N/A	#N/A		
9	7	30	30	X	4	1.8	2.26	0.89	1.12	0.57	298.245614	290.00	300	#N/A	#N/A	#N/A	#N/A		7	30	30	X	4	1.8	2.26	0.89	1.12	0.57	350.877193	350.00	360	#N/A	#N/A	#N/A	#N/A		
10	8	30	30	X	5	2.2	2.77	0.88	1.11	0.57	298.245614	290.00	300	#N/A	#N/A	#N/A	#N/A		8	30	30	X	5	2.2	2.77	0.88	1.11	0.57	350.877193	350.00	360	#N/A	#N/A	#N/A	#N/A		
11	9	35	35	X	3	1.6	2.03	1.05	1.33	0.67	253.731343	250.00	260	18	#N/A	#N/A	#N/A		9	35	35	X	3	1.6	2.03	1.05	1.33	0.67	298.507463	290.00	300	#N/A	#N/A	#N/A	#N/A		
12	10	35	35	X	4	2.1	2.66	1.05	1.32	0.67	253.731343	250.00	260	18	#N/A	#N/A	#N/A		10	35	35	X	4	2.1	2.66	1.05	1.32	0.67	298.507463	290.00	300	#N/A	#N/A	#N/A	#N/A		
13	11	35	35	X	5	2.6	3.27	1.04	1.31	0.67	253.731343	250.00	260	18	#N/A	#N/A	#N/A		11	35	35	X	5	2.6	3.27	1.04	1.31	0.67	298.507463	290.00	300	#N/A	#N/A	#N/A	#N/A		
14	12	35	35	X	6	3	3.86	1.03	1.29	0.67	253.731343	250.00	260	18	#N/A	#N/A	#N/A		12	35	35	X	6	3	3.86	1.03	1.29	0.67	298.507463	290.00	300	#N/A	#N/A	#N/A	#N/A		
15	13	40	40	X	3	1.8	2.34	1.21	1.54	0.77	220.779221	220.00	230	23	21	22.84	5.3		13	40	40	X	3	1.8	2.34	1.21	1.54	0.77	259.74026	250.00	260	18	#N/A	#N/A	#N/A		
16	14	40	40	X	4	2.4	3.07	1.21	1.53	0.77	220.779221	220.00	230	23	21	22.84	7.0		14	40	40	X	4	2.4	3.07	1.21	1.53	0.77	259.74026	250.00	260	18	#N/A	#N/A	#N/A		
17	15	40	40	X	5	3	3.78	1.2	1.51	0.77	220.779221	220.00	230	23	21	22.84	8.6		15	40	40	X	5	3	3.78	1.2	1.51	0.77	259.74026	250.00	260	18	#N/A	#N/A	#N/A		
18	16	40	40	X	6	3.5	4.47	1.19	1.5	0.77	220.779221	220.00	230	23	21	22.84	10.2		16	40	40	X	6	3.5	4.47	1.19	1.5	0.77	259.74026	250.00	260	18	#N/A	#N/A	#N/A		
19	17	45	45	X	3	2.1	2.64	1.38	1.74	0.87	195.402299	190.00	200	30	28	28.92	7.6		17	45	45	X	3	2.1	2.64	1.38	1.74	0.87	229.885057	220.00	230	23	21	21.02	4.4		
20	18	45	45	X	4	2.7	3.47	1.37	1.73	0.87	195.402299	190.00	200	30	28	28.92	10.0		18	45	45	X	4	2.7	3.47	1.37	1.73	0.87	229.885057	220.00	230	23	21	21.02	5.8		
21	19	45	45	X	5	3.4	4.28	1.36	1.72	0.87	195.402299	190.00	200	30	28	28.92	12.4		19	45	45	X	5	3.4	4.28	1.36	1.72	0.87	229.885057	220.00	230	23	21	21.02	7.2		
22	20	45	45	X	6	4	5.07	1.35	1.7	0.87	195.402299	190.00	200	30	28	28.92	14.7		20	45	45	X	6	4	5.07	1.35	1.7	0.87	229.885057	220.00	230	23	21	21.02	8.5		
23	21	50	50	X	3	2.3	2.95	1.53	1.94	0.97	175.257732	170.00	180	37	33	34.90	10.3		21	50	50	X	3	2.3	2.95	1.53	1.94	0.97	206.185567	200.00	210	28	25	26.14	6.2		
24	22	50	50	X	4	3	3.88	1.53	1.93	0.97	175.257732	170.00	180	37	33	34.90	13.5		22	50	50	X	4	3	3.88	1.53	1.93	0.97	206.185567	200.00	210	28	25	26.14	8.1		
25	23	50	50	X	5	3.8	4.79	1.52	1.92	0.97	175.257732	170.00	180	37	33	34.90	16.7		23	50	50	X	5	3.8	4.79	1.52	1.92	0.97	206.185567	200.00	210	28	25	26.14	10.0		
26	24	50	50	X	6	4.5	5.68	1.51	1.9	0.97	175.257732	170.00	180	37	33	34.90	19.8		24	50	50	X	6	4.5	5.68	1.51	1.9	0.97	206.185567	200.00	210	28	25	26.14	11.9		
27	25	55	55	X	5	4.1	5.27	1.67	2.11	1.06	160.377358	160.00	170	41	37	40.85	21.5		25	55	55	X	5	4.1	5.27	1.67	2.11	1.06	188.679245	180.00	190	33	30	30.40	12.8		
28	26	55	55	X	6	4.9	6.26	1.66	2.1	1.06	160.377358	160.00	170	41	37	40.85	25.6		26	55	55	X	6	4.9	6.26	1.66	2.1	1.06	188.679245	180.00	190	33	30	30.40	15.2		
29	27	55	55	X	8	6.4	8.18	1.64	2.07	1.06	160.377358	160.00	170	41	37	40.85	33.4		27	55	55	X	8	6.4	8.18	1.64	2.07	1.06	188.679245	180.00	190	33	30	30.40	19.9		
30	28	55	55	X	10	7.9	10.02	1.62	2.03	1.06	160.377358	160.00	170	41	37	40.85	40.9		28	55	55	X	10	7.9	10.02	1.62	2.03	1.06	188.679245	180.00	190	33	30	30.40	24.4		

Screenshots of Excel Sheet for Compression Members

Sample Problem: (Design of Compression Member)

(b) Design of compression member by limit state method

Design a section ISA 40×40×3@ 1.8 kg/m

Area of the section = 234 mm^2

$$r_{xx} = r_{yy} = 1.21$$

$$r_{uu} = 1.5$$

$$r_{vv} = 0.8$$

$$\lambda_{vv} = \frac{1.8 \times 1000}{88.82\ r_{vv} \times 10}$$

$$= 2.9243$$

$$\lambda_{\emptyset} = \frac{\text{angle leg1} + \text{angle leg2}}{88.82 \times 2 \times \text{thickness of the angle}}$$

$$= \frac{40+40}{88.82 \times 2 \times 3}$$

$$= 0.15012$$

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

$$f_{cc} = \textit{eulers buckling stress}$$

$$f_{cc} = \frac{\pi^2 E}{\frac{KL}{r}^2}$$

For,

one fixed denoted by 1 and one hinged denoted by 2,

$(\lambda_e) = \sqrt{k_1 + k_2\lambda_{vv}{}^2 + k_3\lambda_{\emptyset}{}^2}$

$(\lambda_{e1}) = \sqrt{0.75 + 0.35 \times 2.9243^2 + 20 \times 0.15012^2} = 2.048$

$(\lambda_{e2}) = \sqrt{1.25 + 0.5 \times 2.9243^2 + 60 \times 0.15012^2} = 2.623$

$\emptyset = 0.5\times \left(1 + \alpha(\lambda_e - 0.2) + \lambda_e{}^2\right)$

$\emptyset_1 = 0.5\times (1 + 0.49(2.048-0.2) + 2.048^2) = 3.0499$

$\emptyset_2 = 0.5\times (1 + 0.49(2.623 - 0.2) + 2.623^2) = 4.533$

$$f_{cd} = \frac{\frac{f_y}{\gamma_{m0}}}{\emptyset + (\emptyset^2 + \lambda^2)^{0.5}}$$

$$f_{cd} = \frac{\frac{f_y}{\gamma_{m0}}}{\emptyset + (\emptyset^2 + \lambda^2)^{0.5}}$$

$$= \frac{\frac{250}{1.1}}{3.0499+(3.0499^2+2.04^2)^{0.5}}$$

$$f_{cd1} = 33.82 \text{ N}/mm^2$$

$$f_{cd2} = \frac{\frac{250}{1.1}}{4.533 + (4.533^2 + 2.623^2)^{0.5}}$$

$$f_{cd2} = 27.617 \text{ N}/mm^2$$

Using factor of safety = $\frac{1}{1.5}$

$P_{d1} = A_e f_{cd}$ = 234×33.82

=7913.88N

=7.913kN × $\frac{1}{1.5}$

h= 5.222kN

$P_{d2} = A_e f_{cd}$ = 234× 27.617

=6462.378N

= 6.462kN × $\frac{1}{1.5}$

= 4.265kN

(b) Design of compression member by working stress method

Design a section ISA 40×40×3@ 1.8 kg/m

Area of the section = 234 mm^2

$r_{xx} = r_{yy} = 1.21$

$r_{uu} = 1.5$

$r_{vv} = 0.8$

Assuming the length of the member = 2 m

Minimum radius of gyration =$r_{xx} = r_{yy} = 12.1$ mm

Effective length of the member (l) = 0.80L

l = 0.85 × 1000 = 850 mm

$\frac{l}{r} = \frac{850}{7.7} = 110.39$

= 110.39 < 180 ………. Ok

From IS:800-1984, the allowable compressive for,

$\frac{l}{r}$ = 110.39 and $f_y = 250$ N/mm^2

f_{cd} for 110.39,

X_1 Y_1

110	72
120	64

$$f_{cd} = 71.69$$

Safe load taken by member (P_d),

$$P_d = \frac{A_e f_{cd}}{1000}$$

$$= \frac{234 \times 71.639}{1000}$$

$$= 16.77 \text{ kN.}$$

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