A
MAJOR PROJECT
ON

COMPARATIVE STUDY OF IS : 800 (2007) CODE & IS : 800 (1984) CODE (GENERAL CONSTRUCTION IN STEEL – CODE OF PRACTICE)

SUBMITTED IN THE PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE AWARD OF DEGREE OF

MASTER OF ENGINEERING
(STRUCTURAL ENGINEERING)

SUBMITTED BY:

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2009-2011

CERTIFICATE

It is certified that the work presented in this thesis entitled **"COMPARATIVE STUDY OF IS : 800 (2007) CODE & IS : 800 (1984) CODE (GENERAL CONSTRUCTION IN STEEL – CODE OF PRACTICE)"** by me, University Roll No. 9072 in partial fulfillment of the requirement for the award of the degree of Master of Engineering in Structural Engineering, Delhi Technological University (Formerly Delhi College of Engineering), Delhi, is an authentic record. The work has been carried out by me under the guidance and supervision of **Prof. Kongan Aryan** in the academic year 2010-2011.

This is to hereby certify that this work has not been submitted by me, for the award of any other degree in any other institute.

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Date: University Roll NO. 9072

This is to certify that the above statement made by ASHISH GOYAL bearing roll no. is correct to the best of my knowledge.

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ACKNOWLEDGEMENT

It give me immense pleasure to present this report entitled

"COMPARATIVE STUDY OF IS : 800 (2007) CODE & IS : 800 (1984) CODE (GENERAL CONSTRUCTION IN STEEL – CODE OF PRACTICE) ".I wish to acknowledge with deep sense of gratitude, my indebtedness to my guide **Prof. Kongan Aryan** for his valuable guidance. In spite of his busy schedule, hespared time, took keen interest, reviewed my work, good company, discussed at length, gave me constant encouragement and moral support to complete this dissertation.

I am also thankful to **Dr. Jashpreet Kaur**, member of British library for extending relevant facilities during this work, giving wise advice helping with various applications and so on.

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SECTION : - A – INTRODUCTION

A.1 GENERAL

A.1.1 Development in Steel Construction:-

During last two decades many changes had occurred in the science of Structural Engineering. Steel quality and construction methods are continually being improvement and these factors help in development of Rational Design Technique. Design in steel is reached a level of competence after 20 years of hard won experience. The designer is now much better supported and is able to be more accurate. Codes of practice have become more comprehensive. The advent of Limit State design concentrates the designer's mind on the most important aspect of a particular design.

Steel is a essential components of building and civil engineering structures .It is used in a wide range of application in the commercial, residential and industrial building sectors and in civil engineering infrastructures such as bridges, car-parks ,stadia, wind turbines and masts .The emphasis of this strategy is on building –sustainability within the broader civil engineering.

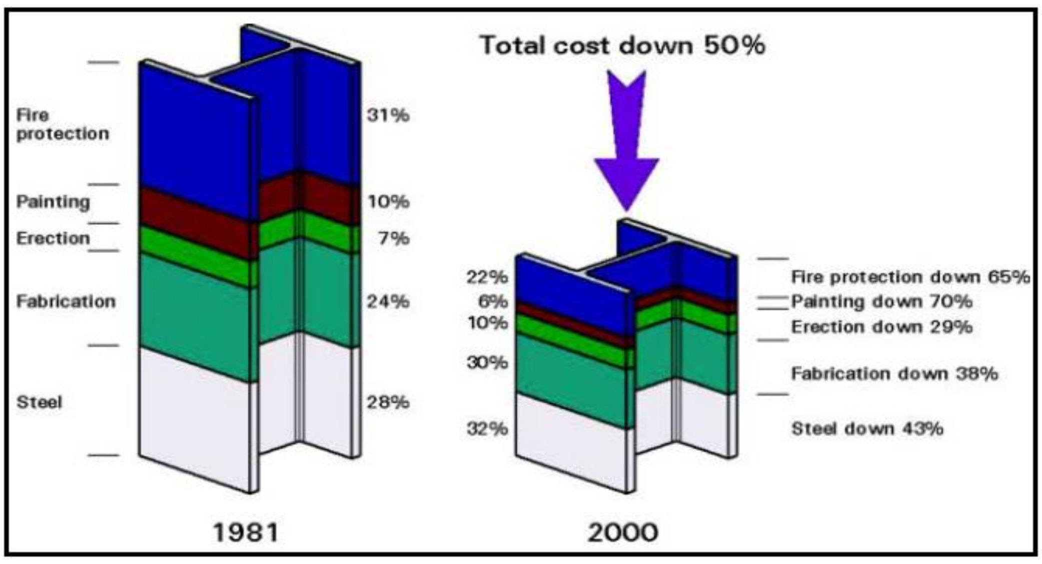
Steel construction has a great deal to offer sustainable development. It is important for us all that the sector flourishes in a way that the sector qualities of steel and steel construction to be fully realized and to contribute to broader construction industry. This in turn is of vital importance if we are to be achieve together our wider sustainable goals and ensure a better quality of life for everyone now and for future generation.

In modern constructions, the key issue is how the choice of materials can create a scope for reducing burdens. The sector recognizes that there is an onus on manufacturers and suppliers to develop system and methods for using their product that will allow design for reduced impacts. There is a further onus on specifiers to use these property.

The steel construction sector's long term commitment to greater efficiency and competitiveness has already delivered many of the actions required to achieve a sustainable future as defined by Government.

The plans for the future outlined in this strategy will, when adopted across the sector , promote the continued development of this successful and progressive industry and enable it to become a major assets in achieving the goal of sustainable construction.

There has been a concerted effort, particularly over the past 30 years, to improve the competitiveness of all parties in the sector. The productivity of steel manufacturing has been improved, new fabrication technologies have been introduced. The economic benefits of this collective effort is demonstrated by steel's healthy market share of 70% of single storey industrial structures.



Cost breakdown of multi-storey structural steel; 1981 and 2000
50% cost reduction in real terms since 1981

Fig:A.1.1

A review of progress made by the sector has demonstrated.

- Steel construction is efficient, competitive and makes a significant contribution to the national economy.
- Building can be rapidly constructed using steel based components that are efficiently manufactured at site and therefore are of high quality and with few defects.

- Steel framing and cladding systems provide the scope, in association with other materials, to design buildings with low overall environments impacts.
- Steel –based construction systems provide flexible spaces which have the potential to be easily modified and adopted so that the life of the building can be extended by accommodating changes in use, layout and size.

Arup Campus, Blythe Valley Park, West Midlands
'Buildings should be considered as a warehouse of its parts. Treating the individual components as being 'hired' rather than 'used' will encourage a philosophy of re-use.' (Image courtesy of Arup Associates)

Fig:-A.1.2

- At the end of useful life of buildings steel components can be dismantled relatively easily. Reclaimed steel products can be re-used or recycled without degradation of properties.

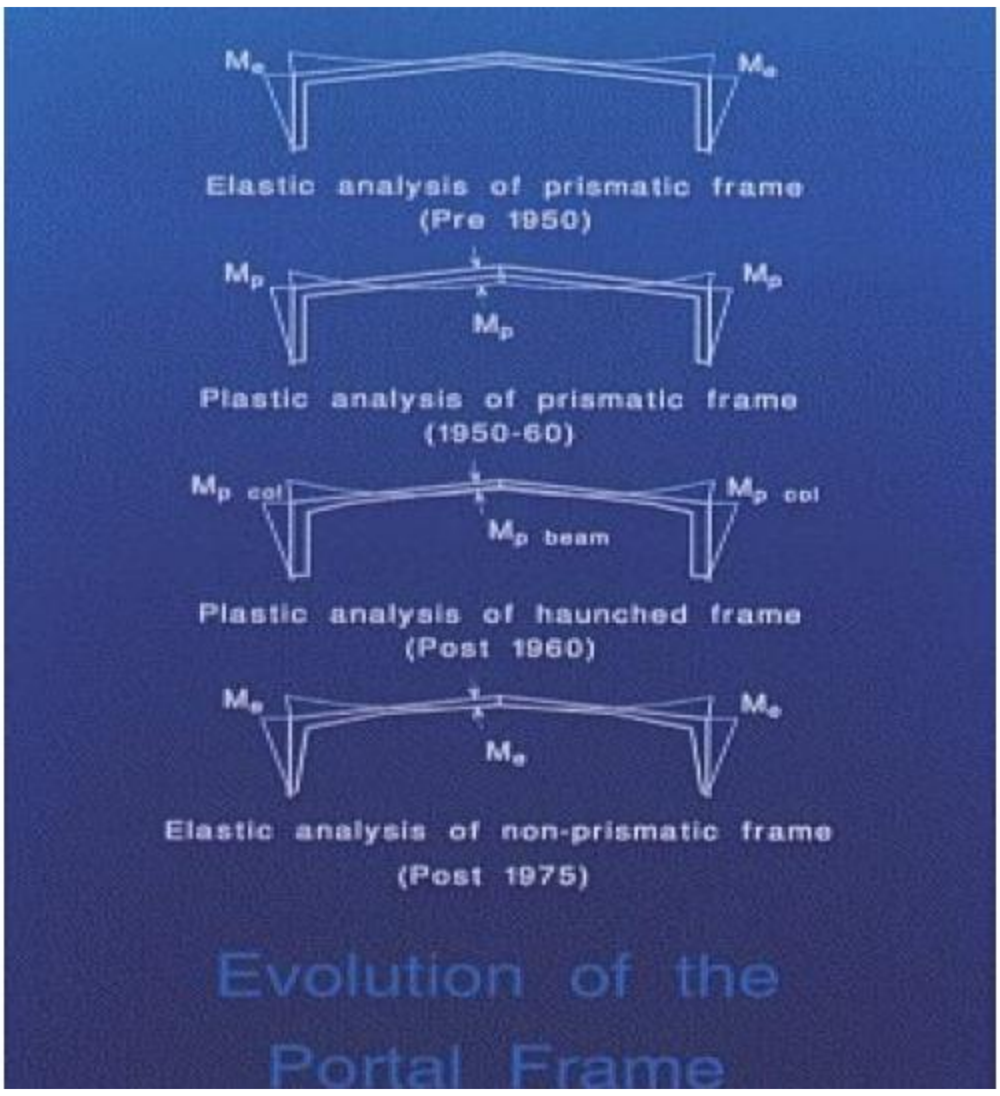


FIg A.1.3

A.1.2 Design Codes /Code of Practice:-

A design code should be a set of minimum requirements for any construction covering safety and serviceability. The safety involves life, health, fire and structural stability. The Code may be administered by a county, or state, or city or by a combination of the three.

Essential of an efficient code of practice for design of steel structure:-

It should be based upon Rational Design Theory.

The Code should be simple, understandable and easy to use.

It should be updated regularly to cater the development in the field of research and technology.

As per above discussion of design codes and its essentials we will overview our existing IS: 800 -1984 (Code of practice for use of steel in structures),IS:800 -2007 as well as Countries and their Design Formats

A.1.2.1 About IS: 800-1984 (Code of practice for use of steel in structures):-

IS 800 (Code of practice for use of steel in structures), which was prepared in 1984 and reaffirmed in 1991, is based on Allowable Stress Design procedure (ASD). The methodology of design of steel structures as per existing IS 800 has not been updated to cater to changes due to research and the state-of-the-art knowledge all over the world. Since the technical knowledge generated through research is generic in nature and can be applied across the world, it is essential to evolve Indian code provisions based on efficient design philosophies.

Considering that the current practice all over the world is based on Limit State Method (LSM) or Load and Resistance Factor design Method, it has been felt by experts that the IS 800 should modified to Limit State Method (LSM) while maintaining Allowable Stress Design (ASD) as a transition alternative, which will help the designers to understand both the design methods and utilize the most advantageous one, and only recently the Indian Standards Institution has taken up the job of revising IS 800 to the Limit state method design which is at present at an advanced stage, with a purpose of evolving a code which will be understandable, easy to use and based on good and widely practiced structural theory to deal properly with elastic instability, dynamic loads and fatigue.

A.1.2.2 Countries and their Design Formats

Almost all advanced countries are now taking advantage of efficient code stipulations, and current practice all over the world is based on either Limit State Method (LSM) or Load and Resistance Factor Design (LRFD).

Following table shows the various major countries and their Design Format

Table A.1.1:- Countries and their Design Formats

Countries	Design Formula(For steel Structure)
Australia , Canada , China , Europe ,Japan , UK	Limit State Design(LSM)
U.S.A.	Load and Resistance Factor Design (LRFD)
India a) IS 800 - 1984 b) IS 800 – 2007	 Allowable Stress Design(ASD) Limit State Design

IS: 800 :2007

The total Draft is prepared is based on the stipulations of International Standards as applicable and Teaching Resource for Structural Steel Design of INSDAG (a committee comprising experts from IIT, SERC)

Following International Standards are referred for IS: 800 -2007

AISC-1999

:-Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings, American Institute of Steel Construction, INC, Chicago, Illinois.

AS 4100-1998 :-Steel Structures (second edition), Standards Australia (Standards Association of Australia), Homebush, NSW 2140.

BS 5950-2000

:-Structural Use of Steelwork in Buildings: Part1Code of practice for design in simple and continuous construction: hot rolled sections, British Standards Institution, London.

CAN/CSA-S16.1-94

:-Limit States Design of Steel Structures, Canadian

Standards Association, Rexdale (Toronto), Ontario, Canada M9W 1R3.

A.1.3 Objectives of Dissertation (i.e. Comparative Study of IS: 800 -1984 and IS 800 -2007)

Through the comparison of IS: 800 -2007 and IS :800 - 1984 following objectives are to be achieved.

1) Becoming familiar with new design methodology i.e. “Limit State Design” for design of steel structure.

2) Learning as well as understanding the basis (why and how?) of various clauses concerned with different section (such as design of tension member, compression member, flexural member, member subjected to combined forces etc).

3) Comparing similarities as well as differences between both codes and also examining the efficient way of designing and if possible finding how best we can incorporate it in our code.

4) Searching limitations of both codes and if possible trying to overcome it through detailed study.

5) To document step-by step procedure for designing different types of structural elements, clearly highlighting different methodology adopted in two different codes so that it may be helpful for undergraduate student as well as practicing engineer.

6) To study economy achieved by designing through both code.

A.1.4 Scope of Present work

IS 800 : 2007 consists of total 17 section and 9 appendices where as IS 800:1984 Consists of 12 section and 7 appendices covering the specifications , standards and rule for design off steel structure. It is considered cover the basic and elementary section for in detail study purpose . The study is broadly divided in to following three parts

Part 1 :

- Basis of Design
- Tension Member
- Compression Member
- Member Subjected to combined forces
- Connections

This consists of studying the basis of clauses (for above mentioned sections) mentioned in both codes followed by illustrated examples by corresponding codes.

Part 2 : - A Detail Design Problem

- Analysis of Foot Over Bridge (FOB)
- Design of Foot Over Bridge By IS 800-1984 & IS 800 – 2007

  This will give the full design process by both code followed by comparison in terms of economy feasibility and safety.

SECTION A : - INTRODUCTION

SECTION A.2 : - ABOUT IS 800: 2007

Table A.2.1 :- Contents of IS 800 : 2007

Sections / Chapters	Name of Sections
Sections 1	General
Section 2	Materials
Section 3	General design requirements
Section 4	Method of Structural Analysis
Section 5	Limit State design
Section 6	Design of tension member
Section 7	Design of compression member
Section 8	Design of member subjected to bending
Section 9	Member subjected to combined forces
Section 10	Connection
Section 11	Working load design format
Section 12	Design and detailing for earthquake load
Section 13	Fatigue
Section 14	Design assisted by testing
Section 15	Durability
Section 16	Fire resistance
Section 17	Fabrication and erection

Table A.2.2:- Appendix of IS : 800 – 2007

Appendix	Name of Appendix
Appendix A	Chart showing highest maximum temperature
Appendix B	Chart showing lowest minimum temperature
Appendix C	Advanced method of analysis and design
Appendix D	Design against floor vibration
Appendix E	Method for determining effective length of columns in frame
Appendix F	Lateral torsional buckling
Appendix G	Connections
Appendix F	General recommendations for steelwork tenders and contract
Appendix I	Plastic properties of beams

A.2.2 Overview of IS: 800 - 2007 with respect to IS: 800 -1984

As far as for comparison purpose the IS: 800:2007 should be first broadly compared with respect to existing IS: 800-1984 so that we can appreciate the changes due to change in design methodology (i.e. from ASD to LSM)

Table A.2.3:- General comparison between IS: 800-1984 and IS:800 -2007

Points of comparison	IS 800 – 1984	IS 800 – 2007
Number of pages	137	206
Number of sections	12	17
Number of appendices	7	9
Number of symbols	79	327
Number of IS code referred	54	87
Number of terms defined	13	111

There were 12 (Twelve) sections in IS: 800-1984 whereas in IS: 800 (Draft Code) there are 17 (Seventeen) sections are included. In Draft Code newly introduced sections are 8 (Eight). From IS: 800-1984 (Old Code) the Section-10 “Design of Encased Member’’ has been removed, where the Section-12 “Steel Work Tenders and Contracts’’ is considered under Appendix- H of Draft Code. The Section-9 “Plastic Design’’ of IS: 800-1984 which was introductory in old code is removed and the Concept of Plastic Analysis is considered in Draft code. The newly introduced Sections in IS: 800-2006(Draft Code) are discussed in brief as below **:-**

Newly added Sections and Appendices

Sections:-

Following are 8 newly introduced sections:-

1) SECTION-4 METHODS OF STRCTURAL ANALYSIS

In this section; the methods of determining the action effects (i.e. Structural analysis) have been discussed. These methods are

a) Elastic analysis

b) Plastic analysis

c) Advanced analysis

The assumptions, requirements and application of each above method have been discussed in detail in this section. In addition to the above method of analysis, for the purpose of analysis and design the Classification of structural frames, Forms of constructions assumed for analysis are described.

2) SECTION -5 LIMIT STATE DESIGN

In this section; basis for limit state design, two limit state viz Limit state of strength and Limit state of serviceability are discussed

The actions (Load), classification of actions, design action, strength, design

strength, ultimate strength, and partial safety factors for loads (γ_f) and material strength (γ_m) are described in detail.

The Sections – 6, 7, 8, 9, 10 (considering Design of -Tension member, Compression member, Members subjected to bending, Member subjected to combined forces) deals with Limit State Design format.

3) SECTION-11 WORKING LOAD DESIGN FORMAT

This section deals with working load design format In old code design is based on working stress method which is modified and presented under Working Load Design Format in the Draft Code This section deals with design criteria for

a) Tension member

b) Compression member

c) Member subjected to bending

d) Member subjected to combined stresses

4) SECTION-12 DESIGN AND DETAILING FOR EARTHQUAKE LOADS

This section covers the requirements for designing and detailing of steel frames so as to give them adequate strength, stability and ductility to resist sever earthquake in all zones of IS:1893 without collapse.

In this section additional load combination for earthquake are mentioned. The design details of lateral load resisting systems (Such as Braced frame system, Moment frame system) from point of view of earthquake load combination are discussed.

5) SECTION-13 FATIGUE

This new section deals with design against fatigue. Terms like fatigue, fatigue strength, stress range, stress cycle counting, S-N curves are defined In this section different details of member and connection (such as non welded details, welded details in hollow and non hollow section, bolted connection details) are classified under different fatigue classes and design stress ranges corresponding to various number of cycles of loading are given for each fatigue class.

6) SECTION-14 DESIGN ASSISTED BY TESTING

This section is introduced not only to provide an alternative to calculation methods (if these methods are not adequate for design of a particular structure, its element or when design or construction is not entirely in accordance with section of given standard) but also necessary in special circumstances (such when the actual performance of an existing structure capacity is in question or when confirmation is required on the consistency of production of material components members or structures originally designed by calculation) In this section types of tests a) Acceptance test (such as NDT) b) Strength test c)Test to failure d)Check test along with Test conditions ,Test loading and Criteria for acceptance have been discussed.

7) SECTION-15 DURABILITY

This section deals with durability of steel structure, it discuss requirement for durability, environmental exposure condition (Table15.2), corrosion protection methods, surface protection methods. Table15.3 gives protection guide for steel work application in detail.

8) SECTION-16 FIRE RESISTANCE

This newly introduced section applies to steel building elements designed to exhibit a required fire resistance level as per given specification This section include definition of related terms, different fire exposure condition, fire resistance level, periods of structural adequacy as well as the variation of mechanical properties of steel with temperature (i.e. variation of yield stress fy and modulus of elasticity Es)

Appendices:-

Following are 3(three) newly introduced appendices:-

1) APPENDIX -C ANALYSIS AND DESIGN METHODS (ADVANCED STRUCTURAL

ANALYSIS AND DESIGN)

This appendix gives advanced structural analysis and design methods for a frame comprising members of compact section with full lateral restraints (i.e. laterally supported members) and Second Order Elastic and Design

2) APPENDIX- D DESIGN AGAINST FLOOR VIBRATION

This section applicable for design of floors with longer spans and of lighter section and less damping as these structure are more sensitive to vibrations under normal human activities.

The appendix gives the determination of floor frequency, peak acceleration and table for critical damping which required for dynamic analysis.

3) APPENDIX-G CONNECTIONS

In this appendix requirements for design of splice (Beam splice, Column splice) and beams to column connections as well as recommendations for their design are discussed.

SECTION B: - STUDY OF BOTH CODES

SECTION B.1 : - BASIS OF DESIGN

IS 800 -2007 adopts the Limit State Design Format for design of steel structures. The basics, requirement, advantages of LSM are discussed in this chapter.

B.1 Limit State Method:-

The Object of limit state design can be Paraphrased as "Achievement of an acceptable probability that a part or whole of structure will not become unfit for it's intended use during it 's life time owing to collapse, excessive deflection etc., under the actions of all loads and load effects.

The acceptable limits of safety and serviceability requirements before failure occurs are called as limit state.

B.1.1 Principles of Limit State Design:-

At its most basic level limit state design simply provides a framework within which explicit and separate consideration is given to a number of distinct performance requirements . It need not necessarily imply the automatic use of statistical and probabilistic concepts, partial safety factors etc. nor of plastic design, ultimate load design etc.

Rather it is a formal procedure which recognizes the inherent variability of loads, materials, construction practices, approximations made in design etc. and attempts to take these into account in such a way that the probability of the structure becoming unfit for use is suitably small. The concept of variability is important because the steel designer must accept that , his performing his design calculation he is using quantities which are not absolutely fixed or deterministic. Examples include values for loadings and the yield stress of steel although must less variable that the properties of some other structural materials, is known to exhibit a certain scatter. Account must be taken of these variations in order to ensure that the effects of loading do not exceed the resistance of the structure to collapse. This approach is represented schematically in figure which shows hypothetical frequency distribution curves for the effect of loads on a structural elements and its strength or resistance where two curves overlap, shaped area , the effect of loads is greater than the resistance of the element , and the element will fail shown by the shaded area, the effect of the loads is greater than the resistance of the element, and the element will fail.

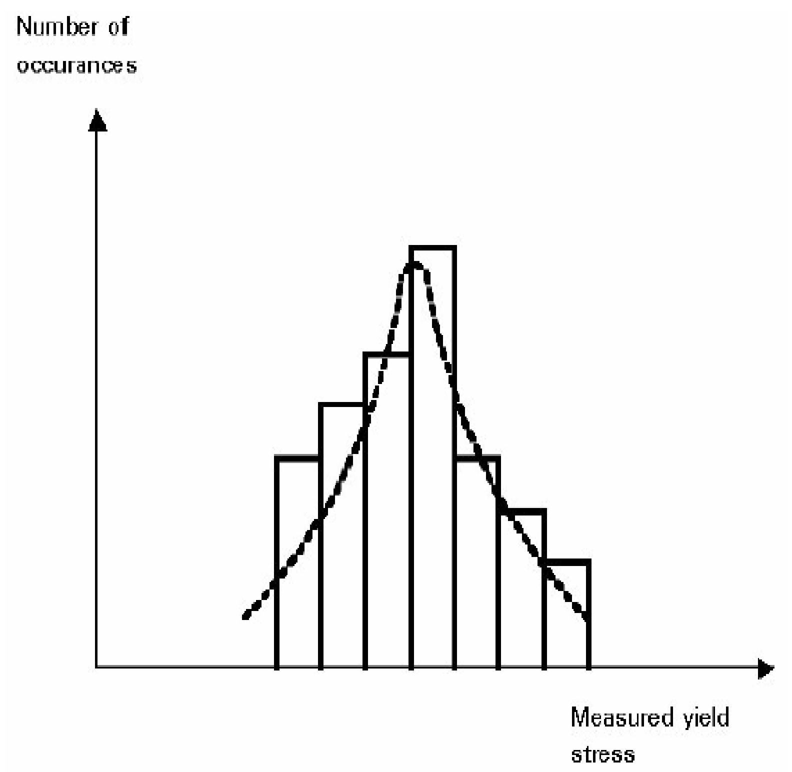


Fig B.1.1 :- Variability of yield stress

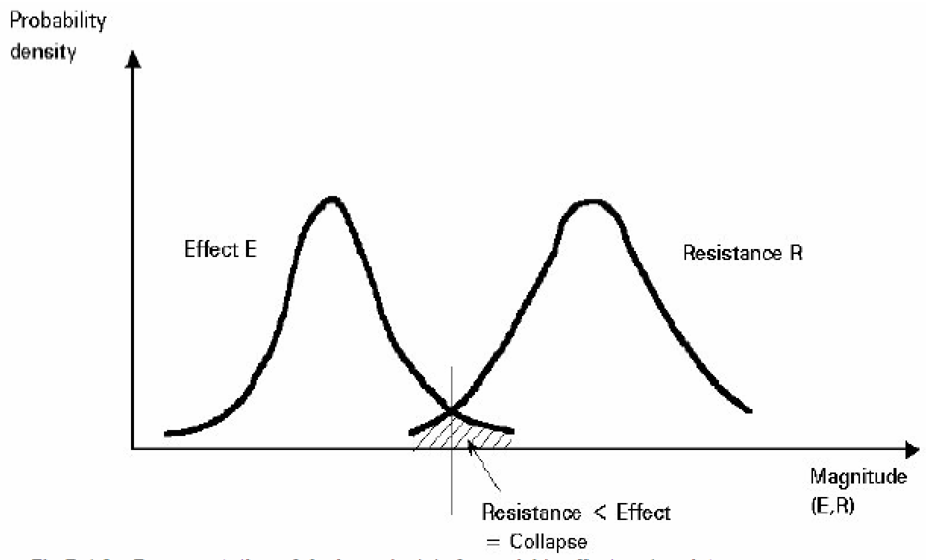


Fig B.1.2 :-Representation of design principle for variable effect and resistance

The procedure of Limit State design can therefore be summarized as follows:-

- Define relevant limit states at which the structural behavior is to be checked.
- For each limit state determine appropriate action to be considered.

Using appropriate structural models for design and taking account of the inevitable variability of parameters data, verify that none of the relevant limit state is exceeded.

Limit States are classified as

- Ultimate limit state
- Serviceability limit state

Table B.1.1 Limit States

Ultimate Limit States	Serviceability Limit States
Strength(yield & buckling)	Deflection
Stability against overturning and Sway	Vibration
Fracture due to sway	Fatigue
Brittle failure	Corrosion

Ultimate Limit States	Serviceability Limit States
Strength(yield & buckling)	Deflection
Stability against overturning and Sway	Vibration
Fracture due to sway	Fatigue
Brittle failure	Corrosion

The Ultimate Limit State include:-

- Loss of equilibrium of the structure as a whole or any of its parts or components.
- Loss of sway of the structure (including the effect of sway where appropriate and overturning or any of its parts including supports and foundation.)
- Failure of excessive deformation ,rupture of the structure or any of its parts or components.
- Fracture due to fatigue.

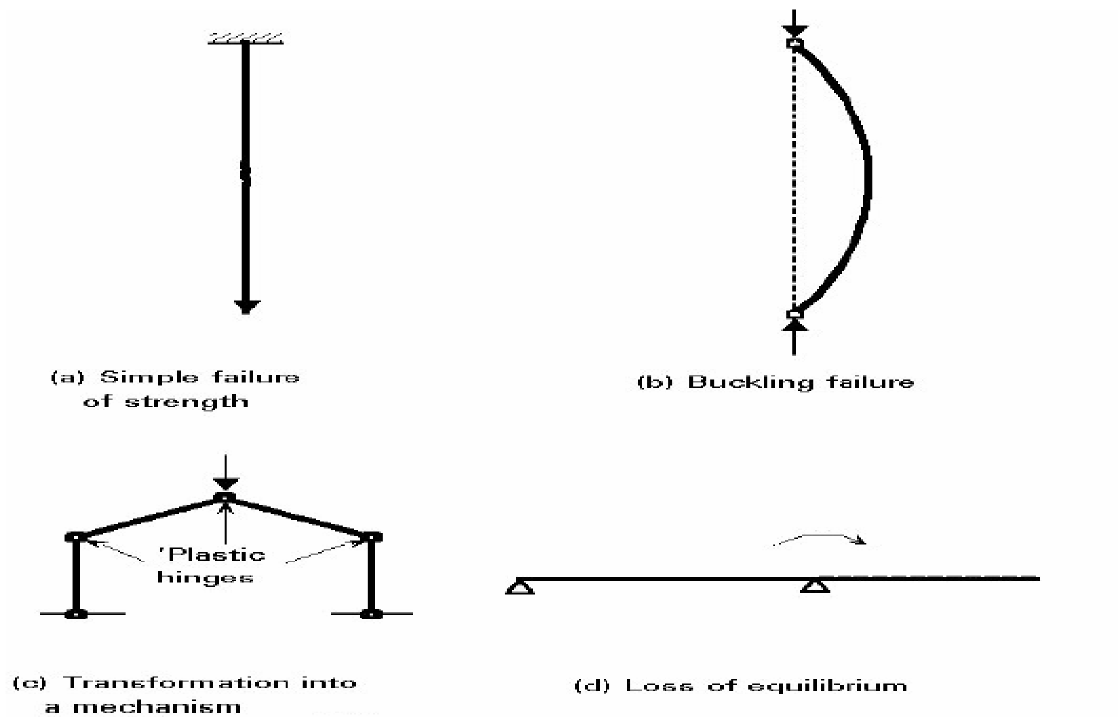


Fig B.1.3:- Ultimate faliure conditions

The Limit State of Serviceability include

- Deformation and deflection which may adversely affect the use of the structure or may cause improper
- Functioning of equipment or services or may cause damages to finishes and non-structural members.

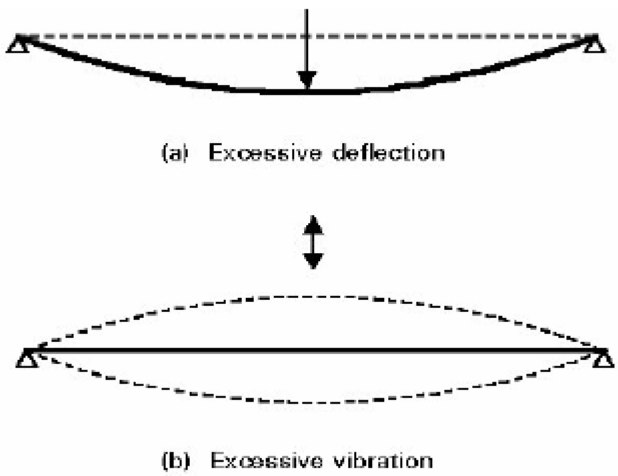


Fig B.1.4:- Serviceability faliure conditions

B.1.2 Limit State Design:-

For ensuring the design objectives , the design should be based on characteristics values for materials strengths and applied loads (actions) which take into account the probability of variation in the materials strengths and in the loads to be supported. The design values are derived from the characteristics values through the use of partial safety factors,the reliability of design is ensured by requiring that

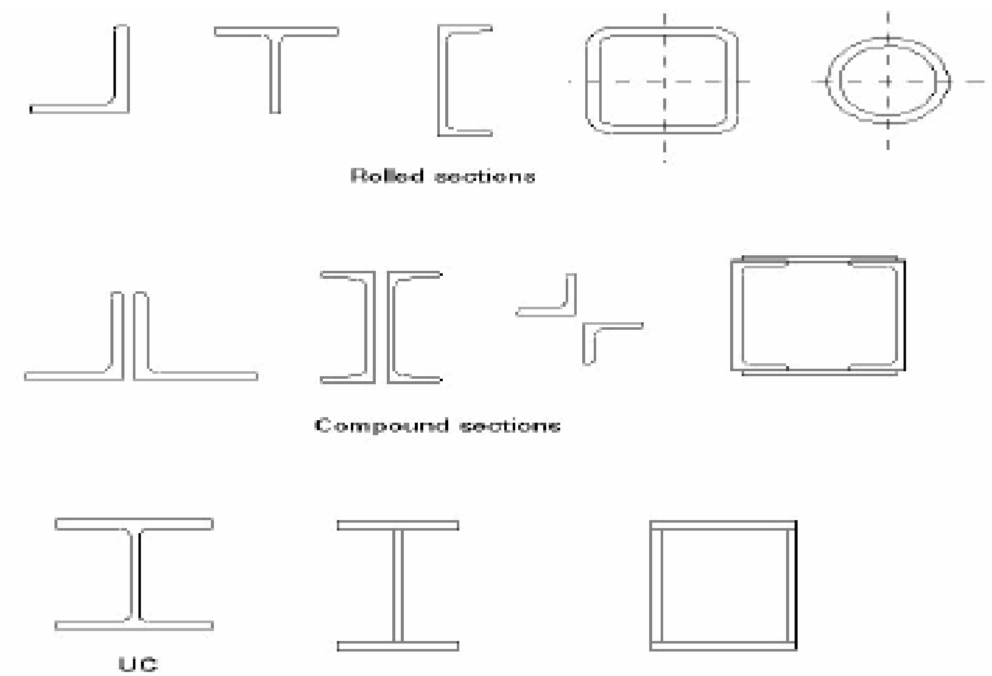
<u>Design Action ≤ Design Strength</u>

B.2.1 General

A structural member subjected to two pulling (tensile) forces applied at its ends is called a tension member. Steel tension member are probably the most common and efficient member. These are efficient because the entire cross- section is subjected to almost uniform stress.(in other words the whole cross-sectional area is utilized). 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. On the other hand , if some eccentricity exists either due to the member not being perfectly straight or due to some eccentricity in connections, either bending stresses are considered in the design or specifications are provided to account for reduction in the net area.

The strength of these members is influenced by several factors such as length of connection, size and spacing of fasteners, net area of cross section, and type of fabrication, connection eccentricity, and shear lag at the end connection.

B.2.2 Cross section of tension member (fig. B.2.1)



B.2.3 Behaviour of Tension Members :-

As per IS 800 - 2007

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads up to the ultimate load, at which stage they may fail by rupture at a critical section. However , if the gross area of the member yields over a major portion of its length before the rupture load is reached , the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted regions.

Modes of failure of tension member:-

Following are different modes of tension member

- Gross section yielding
- Net section rupture
- Block shear failure

Gross section yielding

Generally a tension member without bolt hole can resists loads up to the ultimate load without failure. But such a member will deform in longitudinal direction considerably (nearly 10% - 15% of its original length) before failure. At such a large deformation a structure will become unserviceable. Hence in limit state design, addition of gross section yielding in modes of failure must also be considered , so as to prevent excessive deformation of the member.

Net section rupture

When a tension member is connected using bolts , tension members have holes and hence reduced cross- section, being referred to as the net area. Holes in the member causes stress concentration.

Block shear failure

Block shear commonly refers to the tearing of block of material, and it presumes a combination of tension rupture and shears yield or a combination of shear rupture and tension yield. Block shear failure is usually associated with bolted details because a reduced area is present in that case, but in principle it can also be present in welded details.

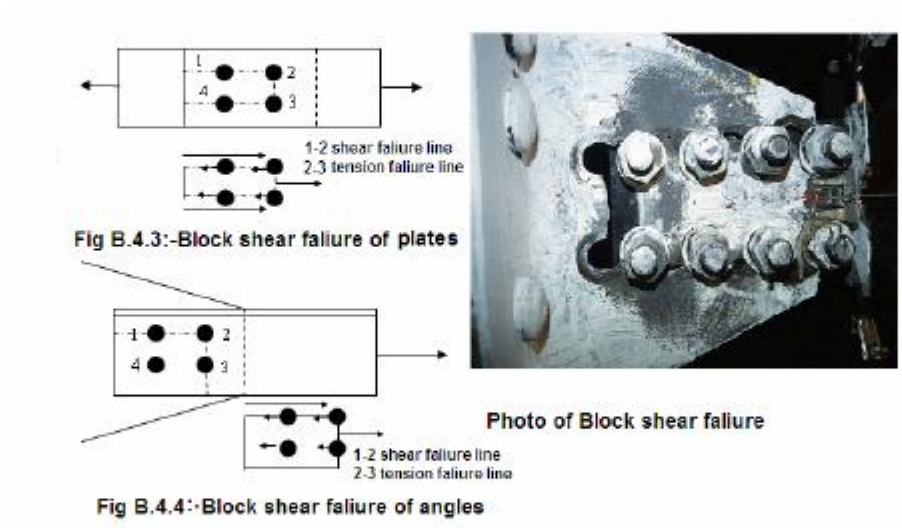


Fig B.4.3:-Block shear faliure of plates

Photo of Block shear faliure

Fig B.4.4:-Block shear faliure of angles

The influence of Residual stresses and connection (Effect of holes) :-

Residual stresses develop when the member is formed and are due to the production process . Their origin can be thermal , either developed during solidification of the steel or they can be mechanically induced when trying to produce counter-deflection or straightening the member. The induced stresses are self equilibrated and although they do not affect the ultimate resistance of the member they induce non – linearities in the strain stress behaviour as well as greater deformability.

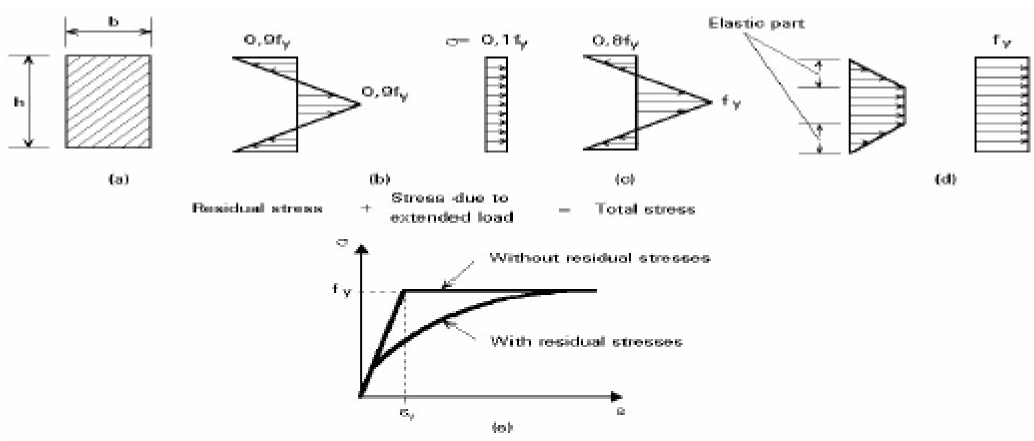
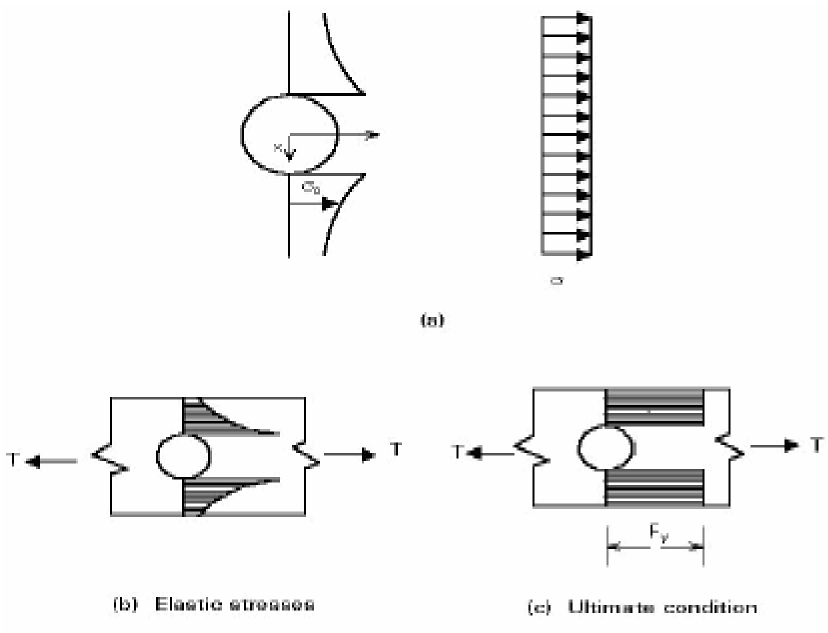


Fig B.2.5 Influence of Residual Stresses on the behaviour of cross-section

Connections (Effect of holes on tension capacity) :-

Connections are generally made either by bolting or welding. When several members have to be connected , additional plates must be used which introduce secondary effects due to the moments developed.The holes that are needed to fix the bolt significantly distort the ideal behaviour of the cross – section. Firstly , there is an area reduction and also a distortion in the stresses distribution that induces a non – uniformity in the strain (fig. 2.2.6)



(a)

(b) Elastic stresses (c) Ultimate condition

As per IS 800 – 1984

Structural members that are subjected to axial tensile force ,any cross – sectional configuration may be used , since the only determinant of strength is the net cross-sectional area.The net sectional area of a tension member is the gross sectional area of the member minus the sectional area of the maximum number of holes. Stresses in atension member are calculated on the basis of minimum net cross-sectional area available . The reason for considering the net section in the calculation of stresses is the failure of sections with holes.

The unit stress in a tension member is increased due to the presence of a hole even if the hole is occupied by a rivet. This is because the area of steel to which load is distributed is reduced and some concentration of stress occurs along the edge of the hole. But for static loading this increase in unit stress is neglected because at yielding , the effect of stress concentration is nullified and tension is therefore assumed to be uniformly distributed over the net section.

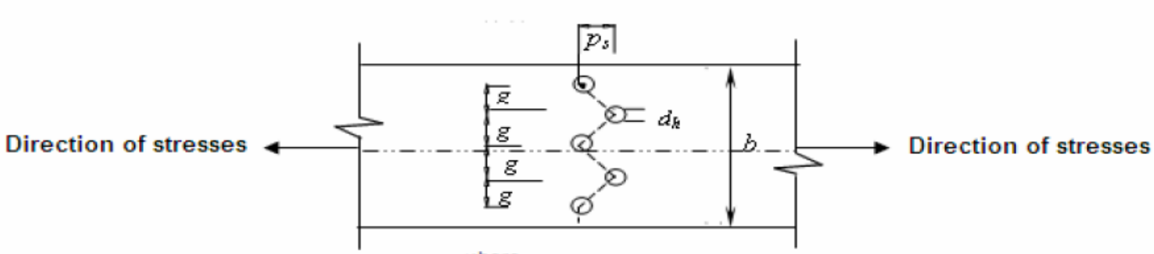
Although there are some parameter like residual stress and connection which result in a non-uniform distribution of stresses, it is generally assumed that the distribution of stresses in cross-sections of members subjected to axial tensile force is uniform.

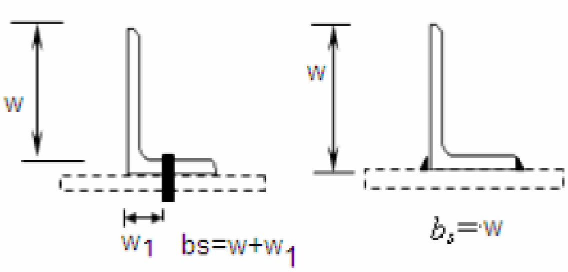
B.2.4 Codal provisions for design of tension member

IS 800 – 1984	IS 800 – 2007
The permissible stress in axial tension σ_{at} in MPa on the net effective area of the section shall not exceed $\sigma_{at} = 0.6\ f_y$ (where f_y = minimum yield stress of steel) (clause 4.1)	Factored design tension T in the member shall be :- $T < T_d$ (clause 6.1) Where T_d = Design tensile strength of the Member = least of T_{dg} , $T_{dn,}$ T_{db} T_{dg} = design strength due to yielding gross section T_{dn} = design strength due rupture of critic section T_{db} = design strength due to block shear

About Net area :-

According to both codes " the net area of a cross section or element section shall l taken as its gross area less appropriate deductions for all holes other opening.
Provided that the fastener holes are not staggered the total area to be deducted shall be the maximum sum of the sectional areas of the holes in any cross-secti perpendicular the member axis.



IS 800 – 1984	IS 800 – 2007
Net effective area $=A_{net} = A_1 + KA_2$ For angles and Tees (clause 4.2) With bolted and welded connection Provide a reduction coefficient to take Account of the unavoidable Eccentricities , stress concentrations etc. In case of single angle connected Through one leg $K = 3A_1/(3A_1 + A_2)$ A_1 = area of connected leg A_2 = area of outstanding leg In case of double angle connected same side of the gusset plate $K = 5A_1/(5A_1 + A_2)$	For angles (clause 6.3.3) With bolted and welded connection $T_{dn} = 0.9 \times f_u \times A_{nc}/\gamma_{m1} + \beta \times Ago \times fy/\gamma_{mo}$ o $= \alpha A_n \times f_u/\gamma_{m1}$ α = 0.6 for one or two rivets = 0.7 for three rivets = 0.8 for four or more rivets β = 1.38-0.076 x w/t x fy/fu x bs/L  w and b_s are shown in fig A_n = net area of the total cross section A_{nc} = net area of the connecting leg A_{go} = gross area of outstanding leg t = thickness of leg L = length of end connection

About angle connected by one leg

In many cases , angles are connected to gusset plate by welding or bolting only through one of the two legs. This type of connection results in eccentric loading, causing non-uniform distribution of stress over the cross-section. Further since the load is applied by connecting one leg of member there is a shear lag at end connection.

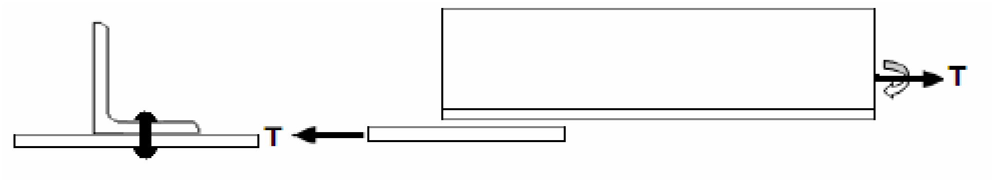


Fig B.2.7 Angle eccentrically loaded through gusset plate

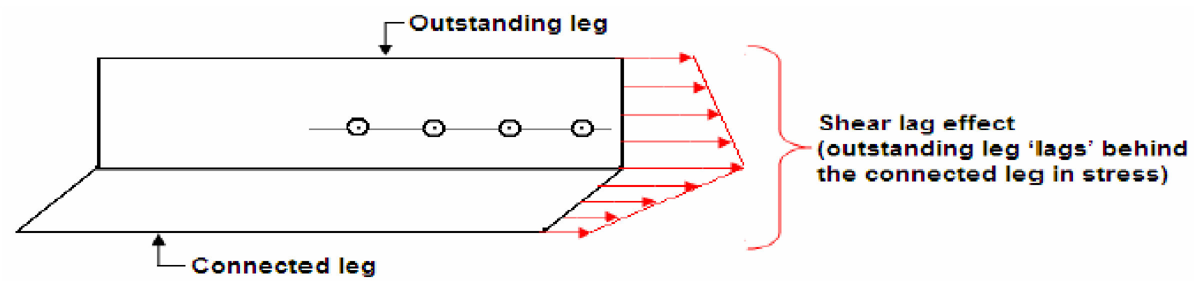


Fig B.2.8 Shear lag effect when angle is connected by one leg

Effect of shear leg:-

The force is transferred to a tension member (angles,channels,or T-section) by a gusset or the adjacent member connected to one leg either by bolting or by welding.The force thus transferred to one leg by the end connection as tensile stress over the entire cross-section by shear.Hence the tensile stress on the section from the first bolt up to the last bolt will not be uniform.The connected leg will have higher stresses even of the order of ultimate stress while outstanding leg stresses may be below the yield stress.Thus transfer of stress from connected leg to outstanding leg will be by shear and because one part Lags behind the other , this phenomenon is referred to as shear lag. However at the section away from the end connection, the stress distribution is more uniform.Hence shear lag effect reduces with increase in connection length.

Therefore to account for eccentric loading due to the shear lag effect the reduction factor β is introduced in IS 800 – 2007. If we calculate the design strength at net cross section by both codes , we can say that

- IS 800 – 2007 consider that connected leg of an angle is stressed up to ultimate stress f_u and outstanding leg is stressed up to yield stress f_y .The reduction factor β is applied to connected leg strength. The value of β increases with length of connection.

- In IS 800 – 1984 the reduction factor is applied to net area of outstanding leg to account of effect of unavoidable eccentricities due to shear lag.The value of reduction factor depends upon type of connection with the gusset. The connection should be designed so as to reduce the effect of bending to a minimum due to eccentricities.

B.2.5 Design process by both codes

Stiffness requirement : -

Although the stiffness is not required for the strength of a tension member since stability is of little concern. However they may be subjected to load reversals during transportation , shipping , erection etc. In order to provide adequate rigidity to prevent undesirable lateral movement or excessive vibrations , design specifications usually contain a limiting slenderness ratio for tension members.

Table B.2.1 Maximum Slenderness Ratios

S.NO.	Member	Max. Slenderness ratio
1.	A tension member in which a reversal of direct stress due to loads other than winds or seismic forces occurs.	180
2.	A member normally acting as a tie in a roof truss or a bracing system but subjected to possible reversal of stresses resulting from the action of the wind or earthquake forces	350
3.	Tension members (other than pre-tensioned member)	400

B.2.6 Worked Example of Tension member :-

The worked example include analysis and design of the tension member by IS 800 – 1984 and IS 800 – 2007

PROBLEM OF TENSION MEMBER BY IS: 800 – 1984

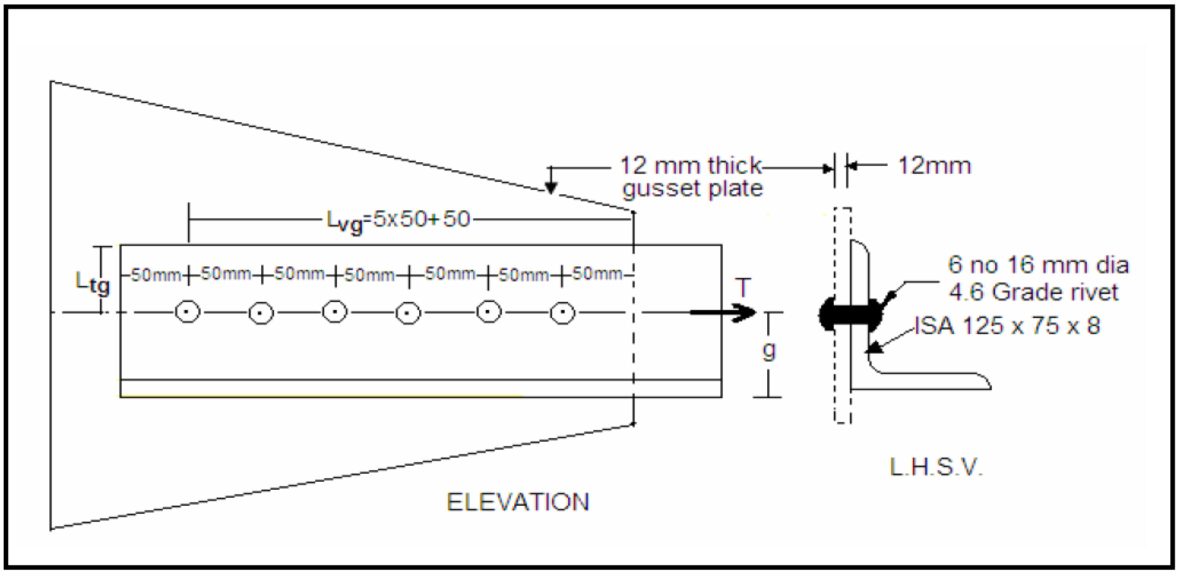
Analysis Problem:-

A Single unequal angle 125 mm x 75 mm x 8 mm is connected to 12 mm thick gusset plate at ends with 6 no. 16 mm diameter rivets to transfer tension as shown in fig. Determine the tensile Strength of the unequal angle section if

(a) Longer leg is connected to gusset plate

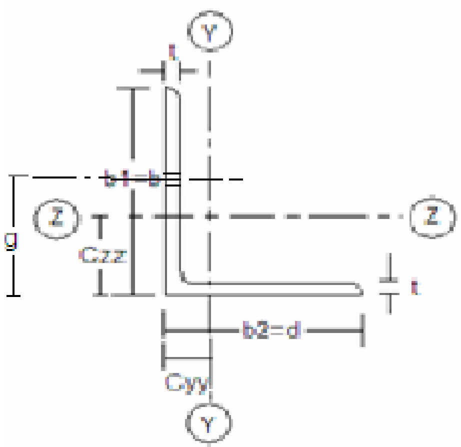
(b) Shorter leg is connected to gusset plate

The value of yield stress (f_y) = 250 MPa



(Fig B.2.9)

Analysis Steps	References

	
Sectional Properties	
$A = 1538\ mm^2$, $b = 125$ mm , $d = 75$ mm	IS Handbook NO. 1
$t = 8$ mm , $g = 75$ mm	Properties of the ang section
Nominal dia of rivet = 16 mm	
Effective dia of rivet (d) = 16+1.5 = 17.5mm	
When longer leg is connected to gusset plate	Clause 3.6.1.1
Area of connected leg (A_1) = (125-17.5-8/2)x8	
$= 828\ mm^2$	
Area of outstanding leg(A_2)=(75-8/2)x8	
$= 568\ mm^2$	
$K = 3A_1/(3A_1+A_2)$	
K = 3 x 828 / (3x828 + 568)	
= 0.814	Clause 4.2.1.1
$A_{net} = A_1 + KA_2$	

$= 828 + 0.814x568$ $= 1290.35\ mm^2$ Strength of member = $\sigma_{at} A_{net}$ $= 0.6x250x1290.35$ $= 193.55$ KN **When Shorter leg is connected to gusset plate** Area of connected leg (A_1) = (75-17.5-8/2)x8 $= 428\ mm^2$ Area of outstanding leg(A_2)=(125-8/2)x8 $= 968\ mm^2$ $K = 3A_1/(3A_1+A_2)$ K = 3 x 428 / (3x428 + 968) = 0.570 $A_{net} = A_1 + KA_2$ $= 428 + 0.570x968 = 979.76\ mm^2$ Strength of member = $\sigma_{at} A_{net}$ = 0.6x250x979.76 = 147 KN	

PROBLEM OF TENSION MEMBER BY IS: 800 – 2007

Analysis Problem:-

A Single unequal angle 125 mm x 75 mm x 8 mm is connected to 12 mm thick gusset plate at ends with 6 no. 16 mm diameter rivets to transfer tension as shown in fig. Determine the tensile Strength of the unequal angle section if

(a) Longer leg is connected to gusset plate

(b) Shorter leg is connected to gusset plate

The value of yield stress (f_y) = 250 MPa

(refer same figure as in problem discussed by IS 800-1984)

Analysis	References
Sectional Properties A = 1538 mm^2 , b = 125 mm , d = 75 mm t = 8 mm , g = 75 mm Nominal dia of rivet = 16 mm Effective dia of rivet (d_o) = 16+2.0 = 18.0 mm When longer leg is connected to gusset plate Net area of connecting leg A_{nc} = (125-18-8/2)x8 = 824 mm^2 Gross area of outstanding leg A_{go} = (75-8/2)x8	IS handbook no. 1 Clause 3.6.1

$= 568\ mm^2$	
Gross area of section	
$A_g = 1538\ mm^2$	
Design strength due to yielding of cross section	
$T_{dg} = A_g f_y/\gamma_{mo}$ = 1538 x 250 /1.1	Clause 6.2
= 349.5 KN	
Design strength due to rupture of critical Section	
$T_{dn} = 0.9 \times f_u \times A_{nc}/\gamma_{m1} + \beta \times Ago \times fy/\ \gamma_{mo}$ or	Clause 6.3.3
$= \alpha A_n \times f_u/\gamma_{m1}$	
α = 0.6 for one or two rivets	
= 0.7 for three rivets	
= 0.8 for four or more rivets	
For our case	
$\alpha = 0.8$	
fy = 250 MPa	
$\gamma mo = 1.10$	
$\gamma m1 = 1.25$	
So , Tdn = 0.8 x (824 + 568) x 410/1.25	
= 365.26 KN	
Design strength due to Block Shear Tdb	Clause 6.4.2
$Tdb = A_{vg} \times f_y/(\sqrt{3} \times \gamma_{m0}) + (A_{tn} \times f_u) / \gamma_{m1}$	

….or… $A_{vn} \times f_u /(\sqrt{3} \times \gamma_{m1}) + A_{tg} \times f_y/\gamma_{m0}$ Where Avg & Avn = Minimum gross and net area in shear Along a line of transmitted force respectively Atg & Atn = Minimum gross and net area in tension From hole to toe of an angle or next last row of bolts In plate Here Avg = Lvg x t Avg = (5 x 50 + 50) x 8 = 2400 mm^2 Avn =(5 x 50 + 50 – 5.5 x 18)x 8 = 1608 mm^2 Atg = Ltg x t Atg = 50 x 8 = 400 mm^2 Atn = (50 – 0.5 x 18) x 8 = 328 mm^2 Therefore Tdb = 2400 x 250 /(√3 x1.10) + 328 x 410/1.25 = 422.50 KN Or , Tdb = 1608 x 410 /(√3 x 1.25) + 400 x 250/1.10 = 395.42 KN Considering lower value for Tdb = 395.42 KN Design tensile strength of ISA 125 x 75 x 8 if longer l	

connected to gusset plate	
Td = Least of Tdg , Tdn , Tdb	
= 349.5 KN	Clause 6.1
When shorter leg is connected to gusset plate	
Net area of connecting leg	
A_{nc} = (75-18-8/2)x8	
= 424 mm^2	
Gross area of outstanding leg	
A_{go} = (125-8/2)x8	
= 968 mm^2	
Gross area of section	
A_g = 1538 mm^2	
Design strength due to yielding of cross section	
$T_{dg} = A_g f_y/\gamma_{mo}$ = 1538 x 250 /1.1	Clause 6.2
= 349.5 KN	
Design strength due to rupture of critical Section	
$T_{dn} = 0.9 \times f_u \times A_{nc}/\gamma_{m1} + \beta \times Ago \times fy/\ \gamma_{mo}$ or	
$= \alpha A_n \times f_u/\gamma_{m1}$	Clause 6.3.3
α = 0.6 for one or two rivets	
= 0.7 for three rivets	
= 0.8 for four or more rivets	

For our case $\alpha = 0.8$ $f_y = 250$ MPa $\gamma_{mo} = 1.10$ $\gamma_{m1} = 1.25$ So , $T_{dn} = 0.8 \times (424 + 968) \times 410/1.25$ $= 365.26$ KN Or, $\beta = 1.38-0.076 \times w/t \times f_y/f_u \times b_s/L$ In our case w = 125 -4 = 121 mm , w1 = 40 mm $b_s = 161$ mm , L = 250 mm , $f_y = 250$ MPa $f_u = 410$ MPa , $\gamma_{m0} = 1.10$, $\gamma_{m1} = 1.25$ then , $\beta = 0.93$ considering the value of $\beta = 0.93$ $T_{dn} = (0.9 \times 410 \times 424)/1.25 + 0.93 \times (968 \times 250)/1.10$ $T_{dn} = 329.77$ KN Hence take the lower value of $T_{dn} = 329.77$ KN Design strength due to Block Shear T_{db} $T_{db} = A_{vg} \times f_y/(\sqrt{3} \times \gamma_{m0}) + (A_{tn} \times f_u) / \gamma_{m1}$ ….or… $A_{vn} \times f_u /(\sqrt{3} \times \gamma_{m1}) + A_{tg} \times f_y/\gamma_{m0}$ Where A_{vg} & A_{vn} = Minimum gross and net area in shear	 Clause 6.4.2

Along a line of transmitted force respectively A_{tg} & A_{tn} = Minimum gross and net area in tension From hole to toe of an angle or next last row of bolts In plate Here $A_{vg} = L_{vg} \times t$ $A_{vg} = (5 \times 50 + 50) \times 8 = 2400\ mm^2$ $A_{vn} = (5 \times 50 + 50 – 5.5 \times 18) \times 8 = 1608\ mm^2$ $A_{tg} = L_{tg} \times t$ $A_{tg} = 35 \times 8 = 280\ mm^2$ $A_{tn} = (35 – 0.5 \times 18) \times 8 = 208\ mm^2$ Therefore $T_{db} = 2400 \times 250 /(\surd 3 \times 1.10) + 208 \times 410/1.25$ $= 383.14$ KN Or , $T_{db} = 1608 \times 410 /(\surd 3 \times 1.25) + 280 \times 250/1.10$ $= 368.14$ KN Considering lower value for T_{db} = 368.14 KN Design tensile strength of ISA 125 x 75 x 8 if longer leg connected to gusset plate T_d = Least of T_{dg} , T_{dn} , T_{db} = 329.77 KN Conclusion from problem	Clause 6.1

design tensile strength capacity of unequal section Will be more if longer leg is connected to gusset plate Than if shorter leg connected to gusset plate.	

PROBLEM OF TENSION MEMBER BY IS 800 – 1984

Design Problem

Design a tension member to carry the design axial tension of 375 KN with riveted connections (provided rivets in a single row). Use fy = 250 MPa

DESIGN STEPS	REFERENCES
1 Data P = 375 KN Rivetted connections	
2. Allowable tensile stress	Clause 4.1
$\sigma_{at} = 0.6$ fy $= 0.6 \times 250$ $= 150$ N/mm^2	
3. Net cross section area required $= 375 \times 10^3 / 150$ $= 2500$ mm^2	
Increase the net area about 40% (to Account for rivet hole) to find the gross	

Area. Gross sectional area required $= 1.4 \times 2500 = 3500\ mm^2$ 4. Trial Section Let us try a Single angle section(longer Leg connected to the gusset plate) Try ISA 150 x 115 x 15 mm Sectional area = $3752\ mm^2$ Provide 20 mm dia rivets. Gross dia d = 20 + 1.5 = 21.5 mm Area of connected legs A_1 $= (150 – 21.5 – 15/2) \times 15$ $= 1815\ mm^2$ Area of outstanding legs A_2 $= (115 – 15/2) \times 15$ $= 1612.50\ mm^2$ $K = 3A_1/(3A_1+A_2) = 0.77$ Net area provided = 1815 + 0.77 x1612.5 $= 3056.63\ mm^2$ Strength of the member =3056.63 x 150 = 458.50 KN > 375 KN	 IS Handbook no.1 Clause 4.2.1.1

Which is all right	

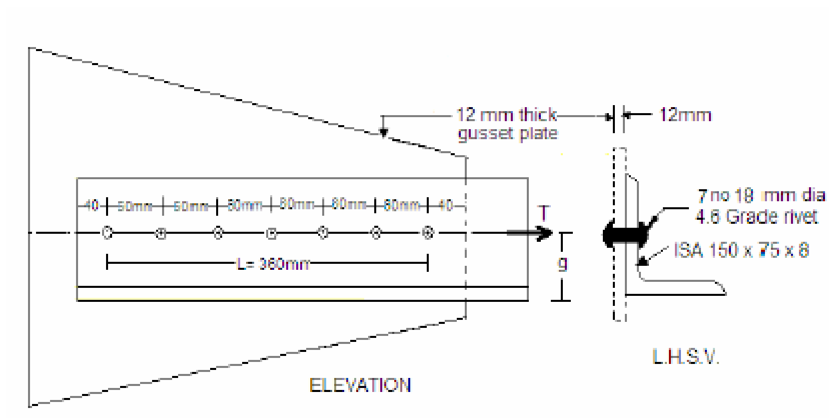
PROBLEM ON TENSION MEMBER BY IS 800 – 2007

Design Problem

Design a tension member to carry the design axial tension of 375 KN with riveted connections (provided rivets in a single row). Use fy = 250 MPa

Design Steps	References
1 Data P = 375 KN Rivetted connections 2. Trial Section $(A_g)_{req} = P \times \gamma_{mo} / f_y$ $= 375 \times 10^3 \times 1.10/250$ $= 1650\ mm^2$ Increase in by 5% so that $(A_g)_{req} = 1.05 \times 1650 = 1733\ mm^2$ Let us try 150 x 75 x 8 mm with longer leg is connected gusset plate. Sectional properties A = 1742 mm^2 , b= 150mm ,d=75mm	 IS Handbook no.1

t = 8 mm, g = 75 mm



3. Connection design

Diameter of rivet d = 6.03 x √t = 6.03 x √8 ≈ 18 mm

(To avoid failure of rivet in bearing)

Effective dia of rivet = 18 + 2 = 20 mm

Rivet value = Shear capacity of rivet in single shear =

V_{ns} = π/4 x 20^2 x 410/(√3 x1.25) = 59.50 KN

Therefore no. rivet required = 375/ 59.50 ≈ 7

Provide edge distance = 40 mm > 30 mm for 18 mm dia riv

Pitch (p):-

Clause 3.6.1

For tension member max. pitch = 16 x t or 200 m whichever is less , minimum pitch = 2.5 x d

Clause 10.2.3.2

Hence , provide p = 60 mm

Therefore length of end connection L = 360 mm

4. Tension capacity of section	Table 10.10
A_{nc} = Net area of connected leg = (150-20-8/2)x8 1008mm^2	Clause 10.2.2 Clause 10.2.1
A_{go} = Gross area of outstanding leg = (75-8/2)x8 = 568 mr	
A_g = Gross area of whole section = 17 mm^2	
A_n = Net area of total cross section = $A_{nc} + A_{go}$ = 15 mm^2	
Design strength due to yielding of gross section	
$T_{dg} = A_g f_y/\gamma_{mo}$ = 1742 x 250 / 1.1 = 395.90 kN	
Design strength due to Rupture of Critical Section	
$T_{dn} = 0.9 \times f_u \times A_{nc}/\gamma_{m1} + \beta \times Ago \times fy/\gamma_{mo}$ or	Clause 6.2
$= \alpha A_n \times f_u/\gamma_{m1}$	
α = 0.6 for one or two rivets	Clause 6.3.3
= 0.7 for three rivets	
= 0.8 for four or more rivets	
T_{dn} = 0.8 x 1576 x 410/1.25 = 413.54 kN	
Design strength due to Block Shear	
$Tdb = A_{vg} \times f_y/(\sqrt{3} \times \gamma_{m0}) + (A_{tn} \times f_u) / \gamma_{m1}$	
….or… $A_{vn} \times f_u /(\sqrt{3} \times \gamma_{m1}) + A_{tg} \times f_y/\gamma_{m0}$	
Here	Clause 6.4.2

A_{vg} = (6 x 60 + 40) x 8 = 3200 mm^2 A_{vn} = (6 x 60 + 40 – 6.5 x 20) x 8 = 2160 mm^2 A_{tg} = 75 x 8 = 600 mm^2 A_{tn} = (75- 0.5 x 20) x 8 = 520 mm^2 Therefore , T_{db} = 3200 x 250/(√3 x 1.10) +520 x410/1.25 = 590.45 kN Or, T_{db} = 2160 x 410/(√3 x 1.25) + 600 x 250/1.10 = 545.40 kN Considering lower value for T_{db} = 545.40 kN	
Design Tensile Strength of ISA 150 x 75 x 8 mm T_d = Least of T_{dg} ,T_{dn} , T_{db} = 395.90 kN > 375 kN (which is all right)	Clause 6.1

Design of Single angle member connected by single row of rivets by both codes

Points	IS 800 – 1984	IS 800 – 2007
Section for tensile force P = 375 kN	ISA 150 x 115 x 15 mm	ISA 150 x 75 x 8 mm
Length of end connection	13 nos 18 mm dia of Rivets p = 60 mm c/c Hence L = 720 mm	7 no. 18 mm dia of rivetsp p = 60 mm c/c Hence L = 360 mm
Failure mode	Failure along net cross Section at holes	Yielding of gross section

SECTION B : - STUDY OF BOTH CODES

Section B.3 Design of Compression Member

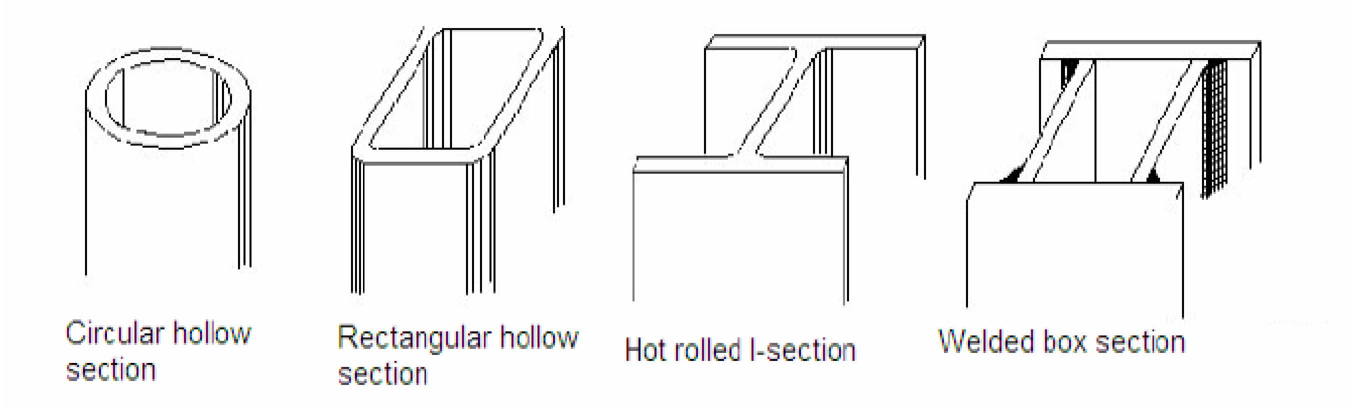
B.3.1 Introduction

The term 'Compression' member is generally used to describe structural components subjected to axial compression loads. Columns, top chord of trusses, diagonals and bracing members are all examples of compression members .Columns are usually thought of as straight compression members where lengths are considerably greater than their cross sectional dimensions.

B.3.2 Cross-sections of Compression members:-

For optimum performance compression members need to have a high radius of gyration in the direction where buckling can occur; circular hollow sections should, therefore, be most suitable in this respect as they maximize this parameter in all directions. The connections to these sections are, however, expensive and difficult to design. It is also possible to use square or rectangular hollow sections whose geometrical properties are good (the square hollow sections being the better); the connections are easier to design than those of the previous shape, but again rather expensive. Hot-rolled sections are, in fact, the most common cross-sections used for compression members. Most of them have large flanges designed to be suitable for compression loads. Their general square shape gives a relatively high transverse radius of gyration and the thickness of their flanges avoids the effect of local buckling. Welded box or welded I-sections are suitable if care is taken to avoid local flange buckling. They can be designed for the required load and are easy to connect to other members; it is also possible to reinforce these shapes with welded cover plates. Built-up columns are fabricated from various different elements; they consist of two or more main components, connected together at intervals to form a single compound member (Figure B.5.1).

Channel sections and angles are often used as the main components but it is also possible to use I - sections; they are laced or battened together with simple elements (bars or angles or smaller channel sections).



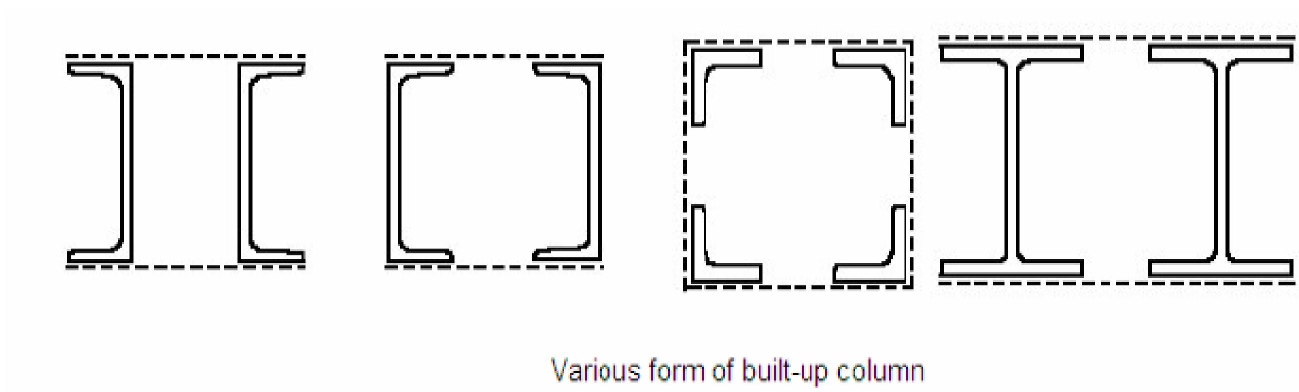


Fig B.3.1 Cross- section of compression member

It should be noted that :-

- In the design of compression member design the type of connection is important it defines the effective length to be taken into account in the evaluation of buckling. Circular section do not represent the optimum solution if the effective length is not the same in the two principle directions , in this case , non symmetrical shapes are preferable.

B.3.3 Behaviour of Compression member :-

Columns are sometimes classified as long , short and intermediate .

- Short compression member

  Short compression member are characterised by very low slenderness , are not effected by buckling and can be designed to the yield stress f_y.

  According to the IS 800 – 2007, if local buckling does not affect the compression resistance (as can be assumed for Plastic (class 1) , compact (class 2) , semi-compact(class 3) cross-sections), the mode of failure of such theoretically occurs when each fiber of the cross-section reaches f_y . It is to be noted that residual stresses aand geometrical imperfections are practically without influence on the ultimate strength of this kind of column and most experimental short columns fail above the yield stress because of strain hardening.

  According to IS 800 – 1984 , very short columns usually fail by crushing or yielding . A very short column is not really a column as such but is considered to be block without buckling.

  IS 800 – 2007 had adopted same multiple column curves (modified ECCS curves developed by European countries) . The ECCS curves considers that columns are stocky when their effective slenderness ratio $\bar{\lambda}$ is such that $\bar{\lambda} \leq 0.2$.

  IS 800 – 1984 had adopted some maximum compressive stress can be set as a limit of strength , and a allowable maximum working compressive stress is chosen accordingly .Also it is logical to apply a smaller factor of safety for the short compression members.

- Long compression member (High slenderness)

  For these compression member the Euler formula , predicts the strength of long compression member very well, where the axial buckling stress remain below the proportional limit . Such compression member buckles elastically.

- Intermediate length compression member (member slenderness)

  For intermediate length compression member , some fibres would have yielded and some fibre will still be elastic. These compression members will fail both by yielding and buckling and their behaviour is said to be inelastic.

The European buckling curves

IS 800 – 2007 uses multiple column curves (Modified ECCS buckling curves) which are based on perry-Robertson approach.The following terms are defined in accordance with these curves.

Non dimensional slenderness ratio ƛ is

$ƛ = \sqrt{(f_y/f_{cc})} = \sqrt{f_y(KL/r)^2/\pi^2E}$ for all cross – sections

where f_{cc} = Euler's critical stress and

(KL/r) = Slenderness of compression member

Basis of the ECCS Buckling Curves

From 1960 onwards, an international experimental programme was carried out by the ECCS to study the behaviour of standard columns. More than 1000 buckling tests, on various types of members (I, H, T, U, circular and square hollow sections) , with different values of slenderness (between 55 and 160) were studied. A probabilistic approach, using the experimental strength, associated with a theoretical analysis, showed that it was possible to draw some curves describing column strength as a function of the reference slenderness).

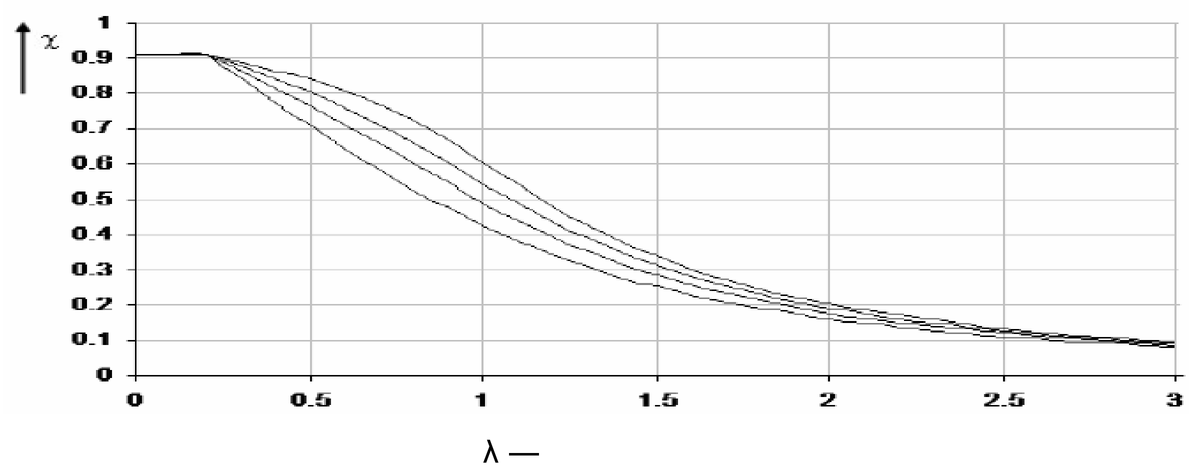


Fig:-5.3.2 ECCS curves

These give the value for the reduction factor X of the resistance of the column as a function of the reference slenderness for different kinds of cross-sections (referred to different values of the imperfection factor α).

The imperfection factor α depends on the shape of the column cross-section considered, the direction in which buckling can occur (y axis or z axis) and the fabrication process used on the compression member (hot-rolled, welded or cold- formed); values for α, which increase with the imperfections.

As stated earlier the imperfection factors take in to account the initial out-of- straightness, residual stresses, eccentricity of axial applied loads and strain-hardening.

Table B.3.1 Imperfection factor , α

Buckling Curve	Imperfections	Imperfection factor
a	Quasi perfect shapes	0.21
b	Shapes with medium imperfections	0.34
c	Shapes with a lot of imperfections	0.49
d	Shapes with maximum imperfections	0.76

B.3.4 Codal provisions for designing the Compression member

IS 800 – 1984	**IS 800 – 2007**
• The direct stress in compression on the cross- (clause 5.1.1) sectional area of an axially loaded compression member is limited to 0.6 f_y. Therefore for formula for permissible compressive stress derived from the Merchant Rankine formula is, $\sigma_{ac} = 0.6 \times f_{cc}\, f_y / \{(f_{cc})^n + (f_y)^n\}^{1/n}$ where σ_{ac} = permissible stress in axial compression f_y = yield stress of steel f_{cc} = elastic critical stress in compression $= \pi^2 E/ƛ^2$ ƛ = slenderness ratio	• The design compression strength of a member is given by (clause 7.1.2) $P_d = A_e\, f_{cd}$ where , A_e = Effective area in compression f_{cd} = design stress in compression $f_{cd} = x(f_y/\gamma_{m0}) \le (f_y/\gamma_{m0})$ x = stress reduction factor $x = 1/\{\varphi + (\varphi^2 - ƛ^2)\}^{0.5}$ in which $\varphi = 0.5\{1+\alpha(ƛ-0.2)+ƛ^2\}$ α = Imperfection factor ƛ = Non dimensional slenderness ratio $ƛ = \sqrt{(f_y/f_{cc})} = \sqrt{f_y(KL/r)^2/\pi^2 E}$ f_{cc} = Euler buckling stress

n = A factor assumed as 1.4	

n should be in the range 1.0 – 3.0 • The designer is supposed to select a section which provides a large radius of gyration without providing more area and in which the average compressive stress does not exceed the allowable compressive stress. Allowable compressive stress in the section is assumed. It should not be more than the upper limit for the column formula specified in the relevant code. For struts it may be 60 – 85 MPa For columns , 85 – 110 MPa • The cross sectional area required to carry the load at the assumed allowable stress is A = P / allowable compressive stress	KL/r = Effective slenderness ratio “ratio of effective length KL to appropriate radius of gyration r γ_{m0} = Partial safety factor for material strength • The designer is supposed to select a section which provides a large radius of gyration without providing more area and in which the design compressive strength just exceed the factored compressive load. Estimate of slenderness ratio or design compressive strength. If average column height is 3 to 5 m the slenderness ratio will generally fall between 40 and 60. (Use Table 8 and Table 9 of IS code) • The cross sectional area required to carry the factored load at the assumed compressive stress is computed. $A_g = P_u$ / assumed compressive stress

As per IS 800 – 2007 (Clause 7.5.1.2)

For a single angle section loaded concentrically , the flexural buckling strength strength of single angle loaded in compression through one of its leg may be determined using the equivalent slenderness ratio :-

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

where ,

- k_1 , k_2 and k_3 are constants depending upon the end condition as given in Table 12 of IS 800 – 2007
- $\lambda_{vv} = (l / r_{vv}) / \varepsilon \sqrt{(\pi^2E/250)}$
- $\lambda_\varphi = (b_1 + b_2)/ \{\varepsilon \sqrt{(\pi^2E/250)} \times 2t\}$
- ε = yield stress ratio $(250 /f_y)^{0.5}$

Table B.3.2 constants k_1 , k_2 and k_3

No of bolts at each end connection	Gusset/connecting member Fixity	k_1	k_2	k_3
≥ 2	Fixed hinged	0.20 0.70	0.35 0.60	20 5
1	Fixed hinged	0.75 1.25	0.35 0.50	20 60

Buckling curve to be used = ‘c’ curve

Imperfection factor α = 0.49 for curve c

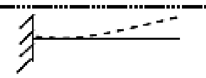
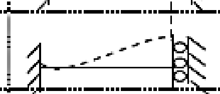
NOTE –

- If the design compressive strength of the column exceeds the factored load over the column by more than 5% the section needs a revision (being over safe and economical)
  (As per IS 800 – 2007)
- If the load carrying capacity of the column exceeds the load over the column by more than 5% , the section needs a revision. (As per IS 800 – 1984)

Effective Length KL (Buckling length l) of a Compression member:-

The effective length, KL, is calculated from the actual length, L, of the member, considering the rotational and relative translational boundary conditions at the ends. The actual length shall be taken as the length from center to center of its intersections with the supporting members in the plane of the buckling deformation, or in the case of a member with a free end, the free standing length from the center of the intersecting member at the supported end.

Table B.3.3 Buckling Length of Prismatic Compression member as per both codes

Boundary Conditions				Schematic representation	Effective Length
At one end		At the other end			
Translation	Rotation	Translation	Rotation		
Restrained	Restrained	Free	Free		2.0*L*
Free	Restrained	Restrained	Free		
Restrained	Free	Restrained	Free		1.0*L*
Restrained	Restrained	Free	Restrained		1.2*L*
Restrained	Restrained	Restrained	Free		0.8*L*
Restrained	Restrained	Restrained	Restrained		0.65 *L*

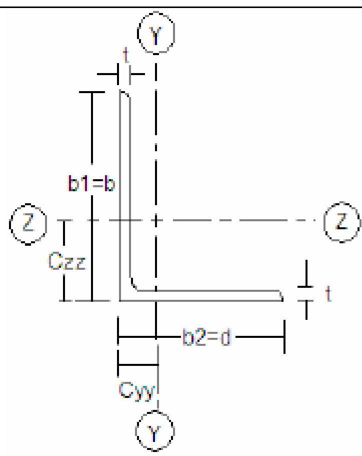
B.3.5 Worked examples for Compression member

The worked example include Analysis problem on single angle strut and design problem on build up column by IS 800 – 1984 and IS 800 – 2007

Single angle discontinuous strut by IS 800 – 1984

Analysis Problem

Calculate load carrying capacity of a single angle discontinuous strut of length 2.7 m connected by two or more rivets in line along the angle at each end (imparting partial restraints against in plane rotation) if ISA 100 x 65 x 8 is used.(use f_y = 250 MPa)

Analysis steps	References
 Data Unsupported length of strut = L = 2.7 m No. of rivets at each end = 2 or more When ISA 100 x 65 x 8 is used Sectional Properties A = 1257 mm^2 , b = 100 mm , d = 65 mm t = 8 mm r_{vv} = 13.9 mm The effective length = 0.85 x 2.7 = 2.30 mm l /r = 2300 / 13.9 = 165.46	IS Handbook no.1 clause 5.1.1 Table 5.1 of IS 800 – 1984

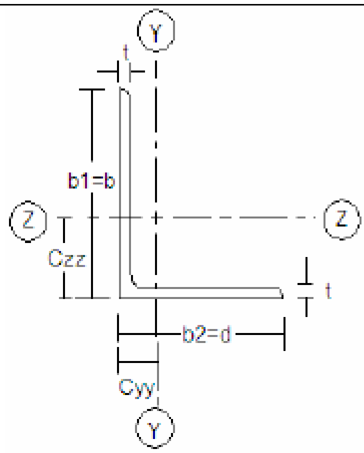
< 350 for this slenderness ratio , allowable working compressive stress (σ_{ac}) = 38.46 MPa Strength of the strut = 38.46 x 1257 = 48.34 kN	

Single angle discontinuous strut by IS 800 – 2007

Analysis Problem

Calculate load carrying capacity of a single angle discontinuous strut of lengh 2.7 m connected by two or more rivets in line along the angle at each end (imparting partial restraints against in plane rotation) if ISA 100 x 65 x 8 is used.(use f_y = 250 MPa)

Analysis Problem	References

 Data Unsupported length of strut = L = 2.7 m No. of rivets at each end = 2 or more	IS Handbook No.1
When ISA 100 x 65 x 8 is used	
Sectional Properties $A = 1257\ mm^2$, b = 100 mm , d = 65 mm t = 8 mm r_{vv} = 13.9 mm Equivalent slenderness ratio	
$\bar{\lambda}_e = \sqrt{(k_1 + k_2\bar{\lambda}_{vv}^2 + k_3\bar{\lambda}_{\varphi}^2)}$	clause 7.5.1.2
for our case, the fixity as partial so take the average value of fixed and hinged condition	
Here, $k_1 = 0.45$, $k_2 = 0.475$, $k_3 = 12.5$	Table 7.6 of IS 800 -2007
$\bar{\lambda}_{vv} = 2700/13.9 /\{1x\sqrt{(\pi^2 x2x10^5/250)}\} = 2.186$	
$\bar{\lambda}_{\varphi} = (100+65)/\{1x\sqrt{(\pi^2 x2x10^5/250)}x2x8\} = 0.116$	

Therefore, $\bar{\lambda}_e = \sqrt{(0.45+0.475 \times 2.186^2+12.5 \times 0.116^2)} = 1.7$	clause 7.1.2.1
$\varphi = 0.5[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2] = 2.313$	
stress reduction factor	
$X = 1/[\varphi + \{\varphi^2 - \bar{\lambda}^2\}^{0.5} = 0.257$	
Design compressive stress	
$f_{cd} = x\ f_y/\gamma_{mo} \leq f_y/\gamma_{mo}$	
$= 0.257 \times 250/1.10 = 58.41\ N/mm^2$	clause 7.1.2
Design compressive strength $= P_d = A_d \times f_{cd}$	
Section classification	Table 3.1 of IS 800 - 2007
$b/t = 100/8 = 12.5 < 15.7\varepsilon$	
(Hence not slender class)	
$d/t = 65/8 = 8.13 < 15.7\varepsilon$	
(Hence not slender class)	
$(b+d)/t = (100+65)/8 = 20.63 < 25\varepsilon$	
(Hence not slender class)	
Hence whole section is of non slender class and group cross sectional area is effective in compression therefore $A_g = A_e = 1257\ mm^2$	
capacity of ISA 100 x65 x8	
$P_d = 1257 \times 58.41 = 73.42kN$	
while designing the compression member avoid	

to design the slender cross section because whole cross sectional area will not be effective in compression .	

Analysis of single angle discontinuous strut by both codes

Points	IS 800 – 1984	IS 800 – 2007
Allowable/design compressive stress	σ_{ac} = 38.46 MPa (Directly calculated on the basis of slenderness ratio)	f_{cd} = 58.41MPa (Effective non dimensional slenderness ratio , stress reduction factor and partial safety factor are responsible to calculate.)

SECTION B :- STUDY OF BOTH CODES

Section B.4 :-Design of Member Subjected to Bending

B.4.1 Introduction :-

A structural member subjected to transverse loads (loads perpendicular to its longitudinal axis) is called a beam. It is obvious , of course , that a beam is combination of a tension member and a compression member.The concepts of tension members and compression members are combined in the treatment of a beam. A beam may , in general , be subjected to either simple , unsymmetrical or bi-axial bending. For simple bending to occur , the loading plane must coincide with one of the principle planes of doubly- symmetrical section (I section) and for singly symmetrical section (a channel section) it must pass through the shear centre. When the plane of loading does not pass through the shear centre the bending is called unsymmetrical. Beams are supposed to be most critical members in any structure. Their design should therefore not only be economical but also safe. The main considerations in the design of beams are the following : -

They should be proportioned for strength in bending keeping in view the lateral and local stability of the flange and the capacity of the selected shape to develop the necessary strength in shear and local bearing.

They should be proportioned for stiffness , keeping in mind their deflections and deformations under service conditions.

They should be proportioned for economy , paying attention to the size and grade of steel to yield the most economical design.

B.4.2 Classification of beams on the basis of lateral restraints provided:-

Depending upon the lateral restrained provided along the compression element (one of the flange) the beams can be categorized in to

Laterally supported beams (restrained beams)

Laterally unsupported beams (Unrestrained beams)

Here a general introduction of classification of beam is given

Laterally supported beams (Restrained beams)

Laterally supported beam is one which-

Unable to move laterally

Unaffected by 'out of plane' of buckling (lateral torsional instability)

Lateral torsional instability will not occur if any of the following conditions apply

The section is bent about its minor axis.

Full lateral restrained is provided , e.g. by positive attachment of the top flange of a simply supported beam to a concrete slab.

Closely spaced , discrete bracing is provided so that the weak axis slenderness (KL/r) of the beam is low.

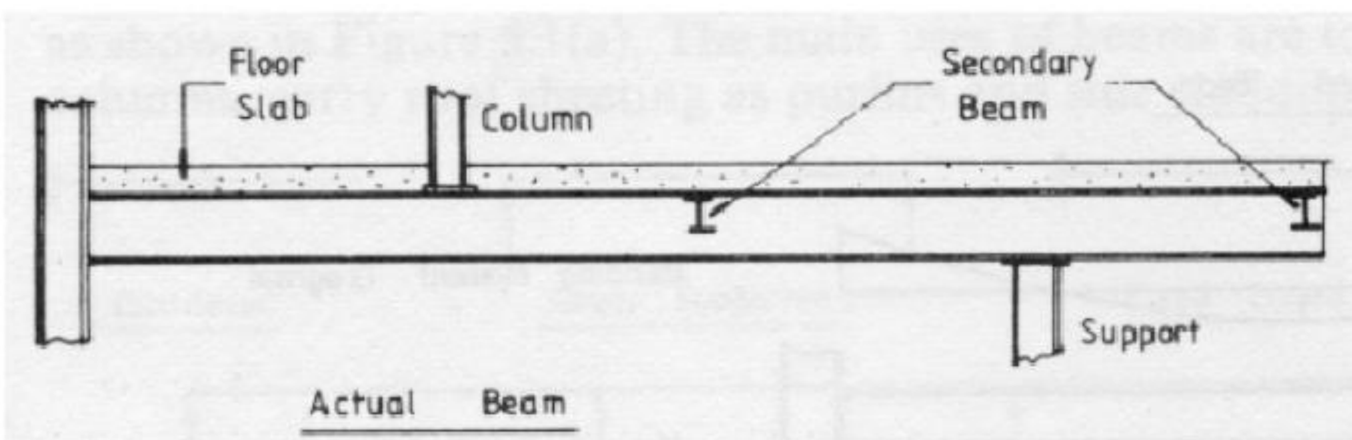


Fig B.4.1 Laterally Supported Beam

Laterally unsupported beams (Unrestrained beams)

The beam is considered laterally unsupported when

Compression flange of beam is not restrained laterally against the lateral buckling Bending take place in weaker direction.

For hot rolled beams and channel section which have very small moment of inertia about minor axis as compared to that about major axis , this make section relatively weak against torsion and bending about weaker axis . This bending usually accompanied by twisting. This phenomenon of bending in the weaker direction and twisting may be called as ‘Lateral Torsional Instability.

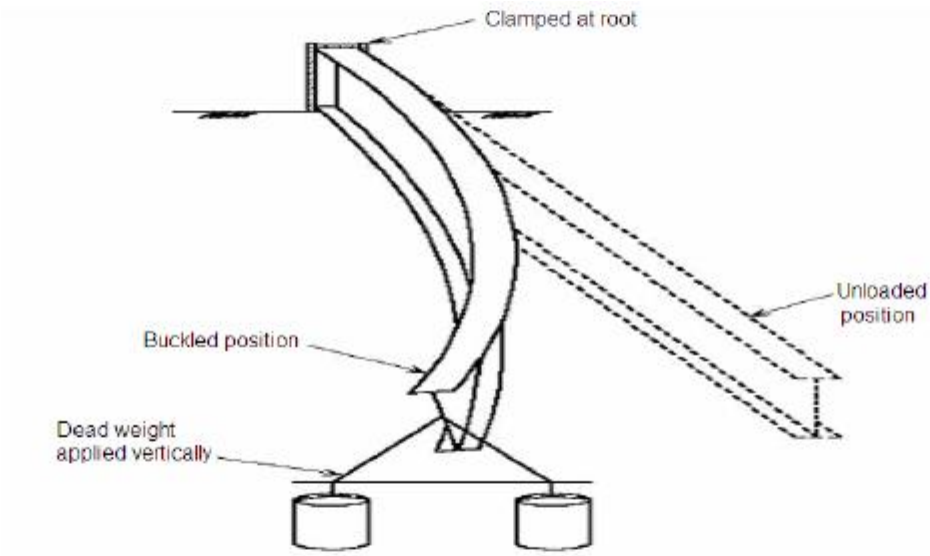


Fig B.4.2 Laterally Unsupported beam

B.4.3 Design theory for beams

IS 800 – 2007 uses Limit State Method design approach but IS 800 – 1984 uses Working Stress Method design approach . The clear benefits of LSM verses WSM are observed in the design of flexural member.

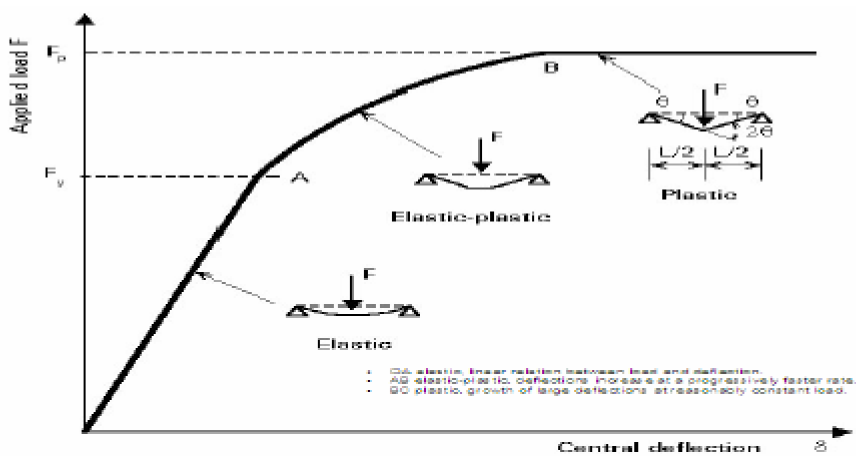
Limit state design of beams

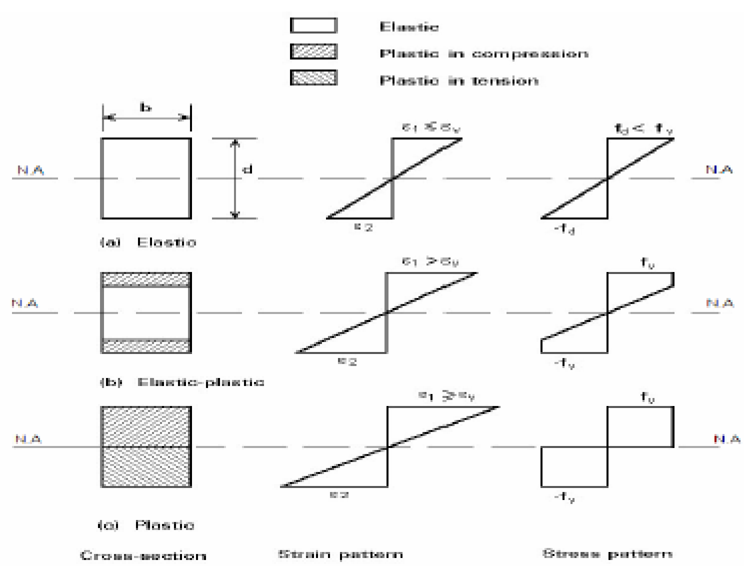
Limit state design of beams

When a restrained steel beam of "compact (Class 2)" proportions is subjected to loads producing vertical bending, its response will consist of a number of stages. Initially it will behave elastically, with vertical deflections being related linearly to the applied load. As the loading is increased, the most highly stressed regions will develop strains in excess of yield, resulting in a local loss of stiffness. For the beam as a whole, deflections will now start to increase rather more rapidly. Additional load will cause this process to continue until complete plasticity is reached at one cross- section. For a simply supported beam, this point will correspond to the maximum load that can be carried without strain hardening and will also be the point at which deflections become very large. On the other hand, for continuous structures, further increases of load are possible as redistribution of moments takes place.

B.4.4 Behaviour of steel beams in bending

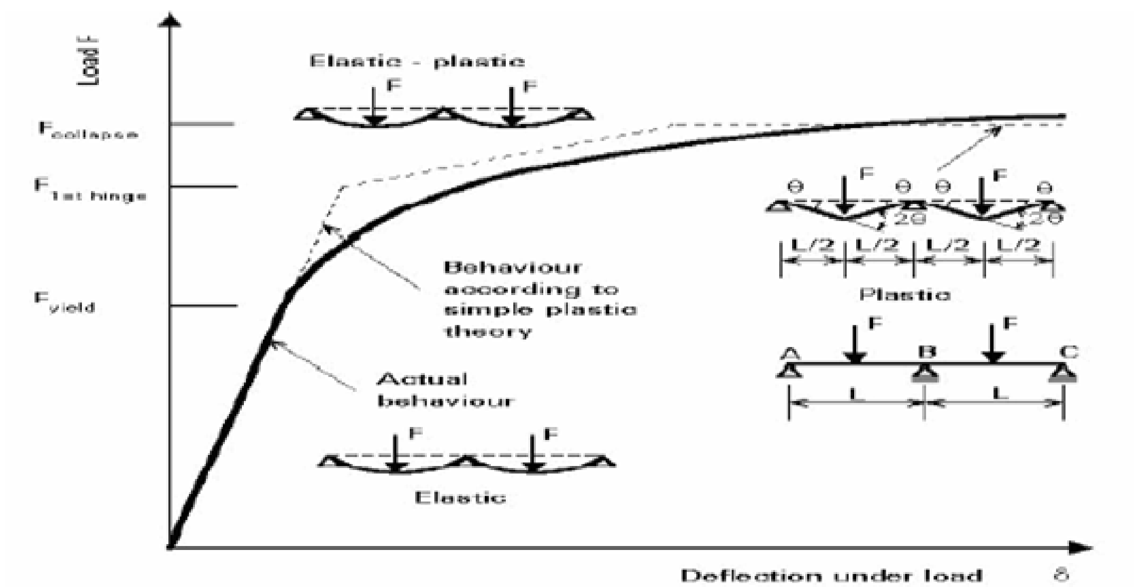
Statically Determinate beams (fig B.4.3)





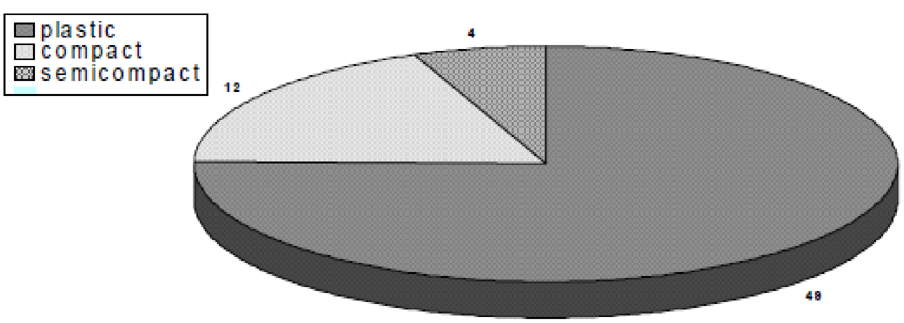
Statically Indeterminate Beams (fig B.4.4)

If the steel beam is continuous and Plastic class (Class 1), then the formation of the first plastic hinge at the point of maximum moment, previously obtained from an elastic analysis, will not mark the limit of its load-carrying resistance. it signifies a change in the way in which the beam responds to further loads. For the two-span beam of Figure B.5.5 the insertion of a real hinge at the central support (B) would cause each span to behave as if it were simply supported. Thus both would be capable of sustaining load, and would not collapse until this load caused a plastic hinge to form at mid-span. The formation of a plastic hinge at produces qualitatively similar behaviour. Thus continuous structures do not collapse until sufficient plastic hinges have formed to convert them into a mechanism. At collapse, the beam will appear as shown in Figure



B.4.5:- Concept of section classification

The critical local buckling stress of the constituent plate element of a beam, for a given material and boundary conditions is inversely proportional to its ‘breadth to thickness ratio’. Hence by suitably reducing the slenderness of the plate elements , its resistance to local buckling could be enhanced. Once the local buckling is prevented , the beam can develop its full flexural moment capacity or the limit state in flexure. Hence depending upon the slenderness of the constituent plate element of the beam , they are classified as slender , semi-compact,compact and plastic . This section classification is new to the Indian structural designers who are familiar with the code of practice for structural steel-work in India , the IS 800 – 1984 .Since IS 800 – 1984 is based on ‘Allowable Stress Method’ the extreme fibre stress in the beams is restricted to 0.66 f_y . In addition, the ‘I’ sections rolled in India are found to be at least semi-compact in which the section classification for Indian standard ‘I’ beams have been presented.In other words the flange outstands of the ‘I’ beams rolled in India are so proportioned that they attain yield stress before local buckling . Because of these two reasons, 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 .



PLASTIC	49
COMPACT	12
SEMICOMPACT	4

B.4.6 :-Codal provision for design of laterally supported and laterally unsupported beam

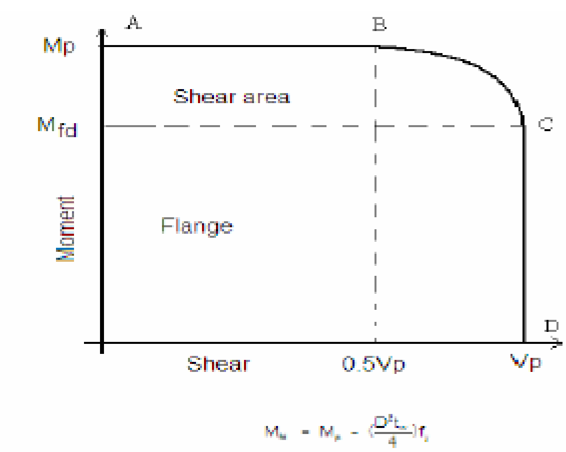
As per IS 800 – 1984	As per IS 800 – 2007
The calculated stress in a member subjected to bending shall not exceed any of the appropriate maximum permissible stresses. A) Laterally supported beam- The maximum bending stress in tension ($\sigma_{at,cal}$) or in compression ($\sigma_{bc\ ,cal}$) in extreme fibre calculated on the effective section of a beam shall not exceed the maximum permissible bending stress in tension (σ_{bt}) or in compression (σ_{bc}) obtained as follows nor the values specified in clause 6.2.2 , 6.2.3 , 6.2.5 and 6.2.6 of IS 800 -1984 as appropriate σ_{bt} or $\sigma_{bc} = 0.66\ f_y$ (clause 6.1.1 The average shear stress in a member calculated on the cross-section of the web shall not exceed $\tau_{va} = 0.4\ f_y$ clause The bearing stress in any part of the beam when calculated on the net area of contact shall not exceed the value of σ_p determined by the following formula $\sigma_p = 0.75\ f_y$	The factored design moment , M at any section, in a beam due to external actions shall satisfy $M \le M_d$ A) Laterally Supported Beam- clause 8.2.1 when $V \le 0.6\ V_d$ $M_d = \beta_b Z_p f_y / \gamma_{mo} \le 1.2\ Z_e f_y / \gamma_{mo}$ where β_b = 1.0 for plastic and compact section $\beta_b = Z_e/Z_p$ for semi-compact section Z_p = plastic section modulus Z_e = Elastic section modulus f_y = yield stress of material γ_{mo} = partial safety factor here IS 800 – 2007 doesn't specify the moment capacity of slender section. when $V > 0.6\ V_d$ $M_d = M_{dv}$ clause 9.2 where M_{dv} = design bending strength under high shear $= M_d - \beta(M_d - M_{fd})$ for plastic and compact section $= Z_e f_y / \gamma_{mo}$ for semi – compact section

	$\beta = [(2V/V_d)-1]^2$ V = Factored applied shear force.As governed by web yielding or web buckling. V_d = design shear strength as governed by web yielding or web buckling M_{fd} = plastic design strength of the area of the cross section excluding the shear area.

Codal provision for design of laterally supported beam (As per IS 800 – 2007)

- About effect of shear force on M_p (capacity of section under high shear)

If the design value of shear force is greater than 50% of the plastic design shear resistance in shear , a member subjected to co-existing bending and shear has to use its web to resists the shear force as well as to assists the flanges in resisting moment . Thus a cross – section subjected to co-existing bending and shear has to reduced moment resistance in presence of high shear. The interaction between moment and shear is shown.



- About holes in tension zone

A reduction in the moment carrying capacity on account of holes in tension flange is justifiable. The plastic moment capacity is computed using yield stress acting over the net area. For cases where ultimate capacity in bending is M_y. or less negligence of holes in tension flange gives no more change in factor of safety , stress concentration adjacent to holes provide carrying capacity to replace that lost by making holes. If the ultimate capacity is M_p . The holes will reduce the factor of safety because M_p will reduce almost in proportion to the area of holes.

- Shear lag effects (clause 8.2.1.5)

The shear lag effects in flanges may be disregarded provided:-

- For outstand elements (supported along one edge)
  $b_0 \le L_0 /20$
- For internal elements (supported along two edge)
  $b_f \le L_0 /10$
  where
  L_0 = Length between poins of zero moments.
  b_0 = Outstand width
  b_f = Internal element width

When these limits are exceeded , the effective width of flange for design strength may be calculated using specialist literature or conservatively taken as the value , satisfying the limit given above.

Codal provisions for design of laterally unsupported beam-

As per IS 800 – 1984	As per IS 800 – 2007
The maximum bending compressive stress on the extreme fibre , calculated on the effective section shall not exceed the maximum permissible bending stress σ_{bc} $= 0.66\ f_{cb} . f_y /[(f_{cb})_n + (f_y)_n]^{1/n}$	Effect of Lateral Torsional Buckling (LTB)on flexural strength need not be considered if $\bar{\lambda}_{LT} \le 0.4$ (clause 8.2.2) where, $\bar{\lambda}_{LT}$ = Non – dimensional slenderness ratio for lateral torsional ratio for lateral torsional

where ,

f_{cb} = elastic critical stress in bending

f_y = yield stress of steel

n = a factor assumed as 1.4

Elastic critical stress –

If an elastic flexural analysis is not carried out , the elastic critical stress f_{cb} for beams shall be calculated using the following formula

$f_{cb} = k_1 (X + k_2 Y) c_2/c_1$ where,

$X = Y \sqrt{[1+(1/20)(l\,T/r_y\,D)^2\,]}$ MPa

$Y = 26.5 \times 10^5 /(l\,/r_y)^2$ MPa

k_1 = a coefficient to allow for reduction in thickness or breadth of flanges between points of effective lateral restraints and depends on Ψ , the ratio of the total area of both flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment between such points of restraint .

k_2 = A coefficient to allow for the inequality of flanges ,and depends on ω , the ratio of the

buckling

Then

$M_d = \beta_b Z_p f_{bd}$

f_{bd} = design bending compressive stress, obtained as follow

$f_{bd} = X_{LT} \times f_y/\gamma_{mo}$

where ,

X_{LT} = stress reduction factor for LTB

$X_{LT} = 1/[\varphi_{LT} + \{ \varphi_{LT}^2 – \bar{\lambda}_{LT}^2\}^{0.5}\,] \le 1$

$\varphi_{LT} = 0.5[\,1+ \alpha_{LT} (\bar{\lambda}_{LT} – 0.2) + \bar{\lambda}_{LT}^2\,]$

α_{LT} = Imperfection factors

0.21 for rolled section

0.49 for welded section

$\bar{\lambda}_{LT} = \sqrt{[(\beta_b \times Z_p \times f_y) / M_{cr}]}$

where

M_{cr} = elastic critical moment

$M_{cr} = \sqrt{[\,\{ \pi^2 EI_y /(KL)^2\} \{GI_t + (\pi^2 EI_w/(KL)^2\}]}$

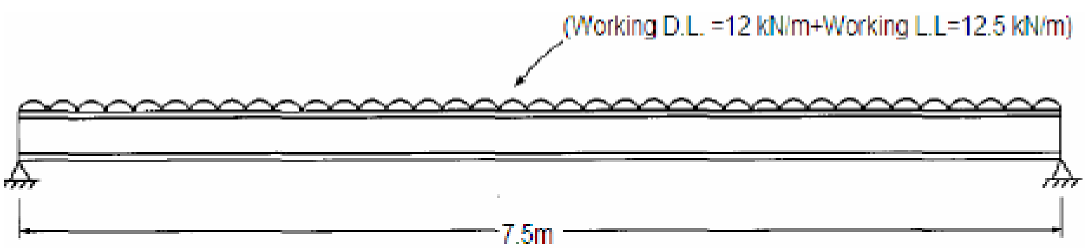
The following simplified conservative equation may be used in the case of prismatic members made of standard rolled I- sections and welded doubly symmetric sections , for calculating the

moment of inertia of the compression flanges , each calculated about its own axis parallel to the y-y axis of the girder, at the point of maximum bending moment. C_1 &C_2 = respectively the lesser and greater distances from the section neutral axis to the extreme fibres. I_y & I_x = moment of inertia of the whole section lying in the plane of bending and normal to the plane of bending respectively. • Values of X & Y are given in Table 6.5 of IS 800 -1984 for appropriate values of D/T and l/r_y • Values of K_1 and K_2 for beams are given in Table 6.3 and Table 6.4 respectively. Values of fcb shall be increased by 20 % when T/t is not greater than 2.0 and d1/t is not greater than 1344/√fy. Guidance for calculating elastic buckling force may be found in the references listed in Appendix E of IS 800-1984	elastic lateral buckling moment, $M_{cr} = \beta_{LT}\pi^2 EI_{yy}h/2(kl)^2 \times \sqrt{[1+ 1/20 \times (KL/r_{yy})^2/(h/t_f)^2]}$ I_t = torsional constant I_w = warping constant I_y = moment of inertia about the weaker axis r_y = radius of gyration of the section about the weak axis KL = effective laterally unsupported length of the member h = depth of the section t_f = thickness of the flange β_{LT} = 1.2 for plastic and compact sections with t_w/t_f =2 β_{LT} = 1.0 for semi compact sections or sections with $t_f/t_w > 2$ Method of calculating Mcr for different beam sections , considering loading , and a support condition as well as for non-prismatic member is given in Appendix F of IS 800-

	2007

Design problem on beams by IS 800 – 1984

The secondary beam of a floor beam system is simply supported at both ends. The beam carries working dead load = 12 kN/m and working live load = 12.5 kN/m from slab.The compression flange of beam is fully embedded in R.C.C. slab.Design the floor beam and apply usual checks.Take f_y = 250 MPa



DESIGN STEPS	REFERENCES
Loading on beam dead load from slab = 12 kN/m	

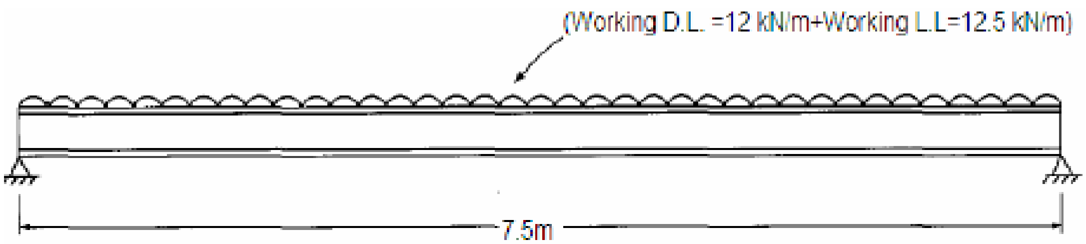
Self wt. of beam = 1 kN/m --- Total dead load = 13 kN /m --- Total Live load = 12.5 kN/m Total load(DL+LL) = 13 + 12.5 = 25.5 kN/m Maximum bending moment $M = wl^2 / 8$ $= 25.5 \times 7.5^2 /8$ $= 179.30$ kN-m maximum shear force = $25.5 \times 7.5 /2$ $V = 95.63$ kN Modulus of section required = M /σ_{bc} $= 179.30 \times 10^6 / 165$ $= 1087\ cm^3$ Trial section	

Let us try 450 @ 0.710kN/m	IS Handbook No. 1
sectional properties	
A = 9227 mm2	
h = 450 mm	
B = 150 mm	
b = 75 mm	
t_w = 9.4 mm	
t_f = 17.4 mm	
h_2 = 35.40 mm	
$I_{xx} = 30390.8 \times 10^4 \text{ mm}^4$	
$I_{yy} = 834.0 \times 10^4 \text{ mm}^4$	
$Z_{xx} = 1350.7 \times 10^3 \text{ mm}^3$	
Check for shear	
τ_{va}= V / (h x t_w)	
95.63 x 103 / (450 x 9.4)	clause 6.4.2
= 22.61 N /mm^2	
< 0.4 fy (100 N /mm^2 for steel	

fy = 250 n/mm2 which is safe Check for defection $\delta_{allowable}$ = L /325 = 7.5 x 10^3 / 325 = 23.08 mm δ_{cal} = (5/384)(wl^4/EI) = 5 x 25.5 x (7.5 x 10^3)^4 /(384 x 2 x 105 x 30390 .8 x 104) = 17.28 mm < 23.08 mm which is safe Check for crippling Let 75 mm bearing length = 25.5 x 10^3 /[(75 + 35.40 x √3) x 9.4] = 19.90 N /mm^2 < 0.75 f_y	clause 6.3

Design problem on beams by IS 800 – 2007

The secondary beam of a floor beam system is simply supported at both ends. The beam carries working dead load = 12 kN/m and working live load = 12.5 kN/m from slab.The compression flange of beam is fully embedded in R.C.C. slab.Design the floor beam and apply usual checks.Take f_y = 250 MPa

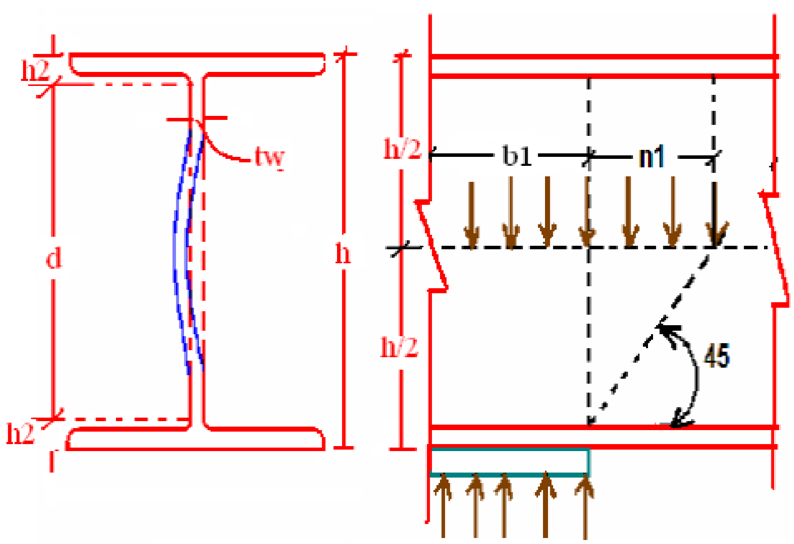


DESIGN STEPS	REFERENCES
Loading on beam dead load from slab = 12 kN/m Self wt. of beam = 1 kN/m --- Total Permanent action G_k = 13 kN /m --- Total Variable action Q_k = 12.5 kN/m	

Partial Safety Factors for Permanent action ($\gamma_{G,sup}$) = 1.5 for variable action (γ_Q) = 1.5 $F_d = \gamma_{G,sup} G_k + \gamma_Q . Q_k$ = 1.5 x 13 + 1.5 x 12.5 = 38.25 kN /m Design Moment $M_{sd} = F_d \times L^2 /8$ = 38.25 x 7.5^2 /8 = 269 kN –m Design Shear force $V_{sd} = F_d \times L /2$ = 38.25 x 7.5 / 2 = 143.45 kN Trial section Let us try ISMB 450 @ 0.710 kN/m Sectional properties A = 9227 mm2 h = 450 mm B = 150 mm b = 75 mm t_w = 9.4 mm	 Table 5.1 of IS 800 :2007 Clause 5.3.3

$t_f = 17.4$ mm $h_2 = 35.40$ mm $I_{xx} = 30390.8 \times 10^4\ mm^4$ $I_{yy} = 834.0 \times 10^4\ mm^4$ $Z_{xx} = 1350.7 \times 10^3\ mm^3$ section classification $\sqrt{(250 / f_y)} = 1$ Flange :- $b / t_f = 75 / 17.4 = 4.31 < 9.4\ \varepsilon$ Hence flange is plastic Web : - $d / t_w = 379.2 / 9.4 = 40.34 < 83.9\ \varepsilon$ where, d = distance between fillet ($h - 2 h_2$) = $450 - 2 \times 35.40 = 379.2$ mm Hence the web is plastic Shear capacity Design shear strength of cross section $V_d = V_p$ as $(d/t_w) < 67\ \varepsilon$ V_p = plastic shear resistance $= A_v f_y/(\sqrt{3} \times \gamma_{mo})$	Table 3.1 of IS 800:2007

$= 450 \times 9.4 \times 250 /(\sqrt{3} \times 1.1)$ $= 555 \times 10^3$ N As V (design shear force) < 0.6 Vd . the effect of shear force on plastic moment capacity Mp . Moment capacity Since the section is plastic and there is not effect of shear force on plastic moment capacity Therefore , design bending strength of cross section $M_d = \beta_b \times Z_p \times (f_y / \gamma_{mo})$ $= 1 \times 1553 .36 \times 10^3 \times (250 / 1.10)$ = 348.5 kN m > 269 kN m (M = design moment) Hence safe Checks Deflection checks :- deflections are to be checked for most adverse but realistic combination of service load and their arrangements by elastic analysis using load factor 1. Now , $\delta_{max} = 5 \times w \times L^4 /(384 \times E \times I_{xx})$ here w = 1.0 x 13 + 1.0 x 12.5 25.5 kN So ,	Clause 8.2.1.2 Clause 8.2.1

δ_{max} = 17.28 mm < L/300 …..(not susceptible to cracking) < L/350…..(susceptible to cracking) Hence safe Web buckling check :- The buckling resistance of the web is given by P_w = width of stiff bearing plate n_1 = dispersion of load through the web @ 45^0 to the level of half the depth of cross section f_{cd} = design calculated strength as per 7.1.2.1  Fig :- Web Buckling Let 75 mm stiff bearing plate	Clause 5.6.1 and Table 5.3 of IS 800 - 2007

therefore , b1 = 75 mm n1 = 225 mm To find fcd = X x (fy γmo) here , X = stress reduction factor $X = 1 /[\varphi + (\varphi^2 – \bar{\lambda}^2)^{0.5}]$ $\varphi = 0.5[1+ \alpha (\bar{\lambda} – 0.2) + \bar{\lambda}^2]$ α = imperfection factor $\bar{\lambda}$ = non dimensional effective slenderness ratio KL = 0.7 x d = 0.7 x 379.2 = 265.44 mm $r_{zz} = t_w / \sqrt{12}$ = 2.71 mm therefore , $\bar{\lambda}$ = 1.102 for solid web section we have to use buckling curve c irrespective of axis of bending Therefore , α = 0.49 therefore, φ = 1.558	 Clause 8.7.3.1

So , $X = 2.66$

then ,

$f_{cd} = 2.66 \times 250 / 1.10$

$= 85.46$ kN / mm^2

Hence , the buckling resistance of the web

$P_w = [(75 + 225) \times 9.4] \times 85.46$

$= 241$ kN > 143 kN

Hence safe

Web Bearing or Web Crippling checks : -

The crippling resistance of web is given by

$F_w = (b_1 + n_2) \times t_w \times f_{yw} / \gamma_{mo}$

where,

b_1 = stiff bearing length

n_2 = length obtained by dispersion through the flange to web junction at a slope of 1:25 to the plane of flange

f_{yw} = yield stress of web

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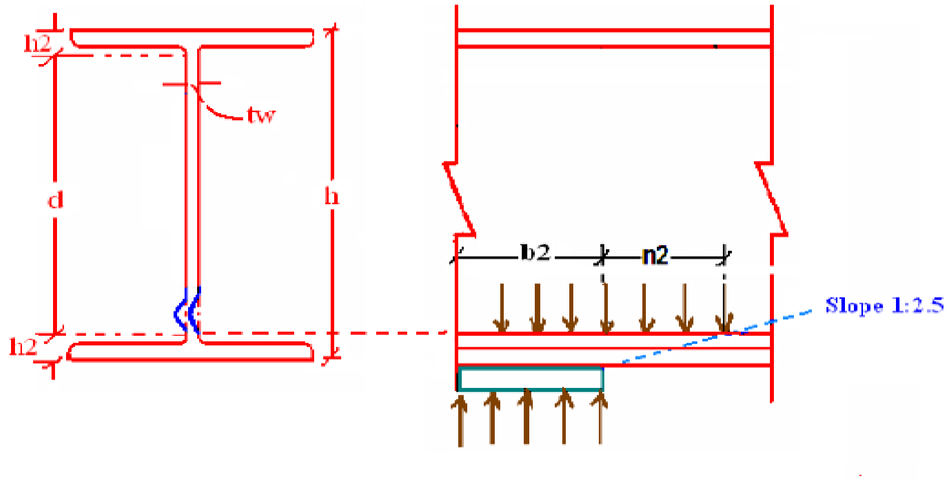

Fig:- Web bearing or Web crippling

Let 75 mm stiff bearing length

$b1 = 75$ mm

$n_2 = 2.5 \times h2$

$= 2.5 \times 35.40$

$= 88.5$ mm

$tw = 9.4$ mm

$f_{yw} = 250$ n/ mm^2

$\gamma_{mo} = 1.1$

Therefore web crippling resistance Fw = $(75 + 88.5) \times 9.4 \times 250 / 1.1 = 350$ kN > 143.45 kN (safe)
</td>
<td>Clause 8.7.4</td>
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SECTION B : - STUDY OF BOTH CODES

Section B.5 :- Design of Member Subjected to Combine Forces / Combined stresses

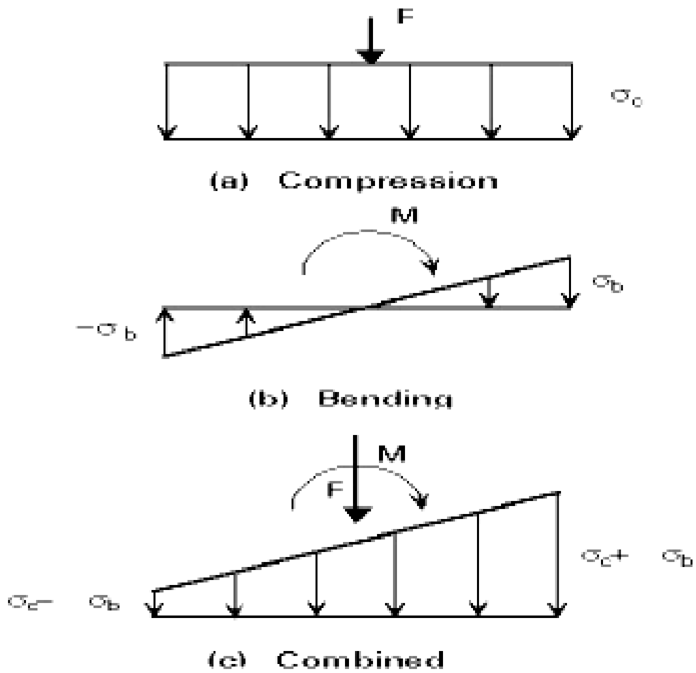
B.5.1 General

Depending upon the extreme actions over the members in structural framing system , the combined forces or stresses may be broadly classified as

- Combined Axial compression and bending
- Combined Axial tension and bending
- Combined bending and shear

B.5.2 :- Cross – Sectional behaviour for combined axial compression and bending

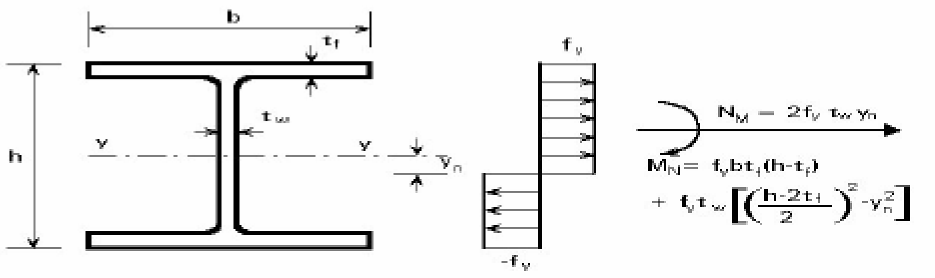
Given fig.(B.5.1) shows a point somewhere along the length of an I – section column where the applied compression and moment about the z – axis produce the uniform and varying stress distributions.



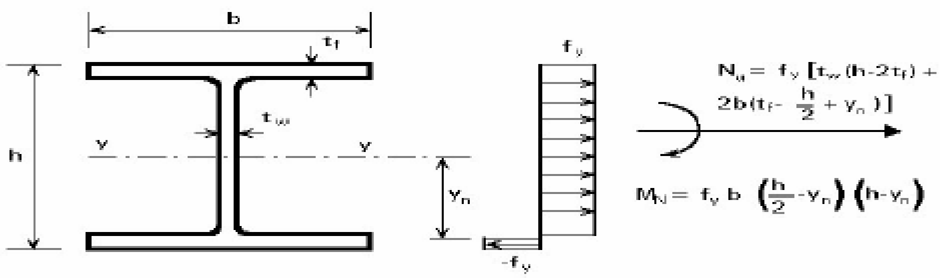
B.5.3 :- Elastic behaviour of cross – section in compression and bending:-

Note : - Beam columns are defined as members to combined bending and compression . In principle , all members in frame structures are actually beam – columns.

If full plasticity is allowed to occur , then the failure condition for combined axial load (compression) and bending will be as shown in fig.B.5.2



(a) $y_n < (h-2t_f)/2$



(a) $y_n > (h-2t_f)/2$

Full plastic under axial compression and moment

Where,

N_M = Reduced plastic resistance of gross section allowing moment.

M_N = Reduced plastic resistance moment of cross – section

Now if full plasticity is allowed to occur , then the failure condition will be

For $y_n \leq (h - 2t_f)/2$ neutral axis lies in web

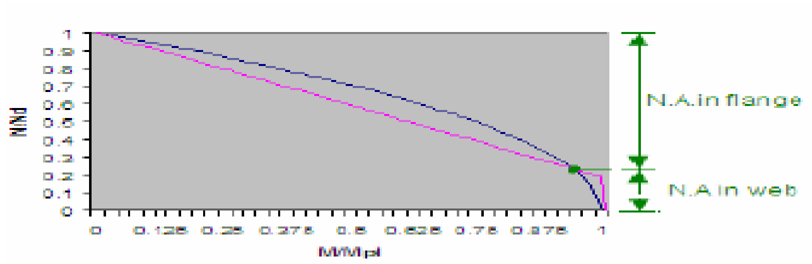
$N_M = 2f_y t_w y_n$

$M_N = f_y\ b\ t_f\ (h - t_f) + f_y[\ \{ (h-2t_f)/2\}^2 - y_n^2$

For yn > (h-2tf) /2 Neutral axis lies in flanges

$N_M = f_y\ [\ t_w\ (\ h - 2\ t_f\ ) + 2\ b\ [\ t_f - h/2 + y_n]$

$M_N = f_y\ b\ [\ h/2 - y_n]\ [\ h - y_n]$



Interaction curve between moment and axial force.

Theoretical curve —

As per IS 800 :2007 —

The design interation curve is lower bound when plastic neutral axie lies flange (when the applied moment is smaller a compare to applied axial force)

B.5.4 :- Codal provisions for design of member subjected Combined axial compression and bending:-

As per IS 800 – 1984	As per IS 800 – 2007
Combined Axial compression and bending Member subjected to axial compression and bending shall be proportioned to satisfy the following requirements : - $\sigma_{ac\,,\,cal} / \sigma_{ac} + C_{mx} \,.\, \sigma_{bcx\ cal} / [\ \{1- \sigma_{ac,cal} / 0.6 f_{ccx}\} \sigma_{bcx}]$ $+ C_{my} .\sigma_{bcy\ ,cal} / [\{1 - \sigma_{ac,cal} / 0.6 f_{ccy}\} \sigma_{bcy}]$	Combined axial force and bending moment (clause 5.4.8) 1) Section strength 1.a) For plastic and compact section (clause 9.3.1.1) Following relationship should be satisfy $[M_y/M_{ndy}]^{\alpha}{}_1 + [M_z / M_{ndz}]^{\alpha}{}_2 \le 1.0$ or conservatively $N/N_d + M_y / M_{dy} + M_z / M_{dz} \le 1.0$

≤ 1.0 However if the ratio $\sigma_{ac\,,\,cal}$ / σ_{ac} is less than 0.15 , the following expression may be used in lieu of the above $\sigma_{ac\,,\,cal} / \sigma_{ac} + \sigma_{bcx\,cal} / \sigma_{bcx} + \sigma_{bcy\,,cal} / \sigma_{bcy} \leq 1.0$ The value of σ_{bcx} and σ_{bcy} to be used in the above formula shall each of lesser of the values of the maximum permissible stresses σ_{bc} given in section 6 if IS 800 -1984 Combined axial tension and bending – A member subjected to both axial tension and bending shall be proportioned so that the following condition is satisfied : $\sigma_{at,cal} / 0.60\, f_y + \sigma_{btx,cal} / 0.66\, f_y + \sigma_{bty,cal} / 0.66\, f_y \leq 1.0$ $\sigma_{ac\,,\,cal}$ = calculated average axial compressive stress $\sigma_{at,cal}$ = calculated average axial tensile stress $\sigma_{bc,cal}$ = calculated bending compressive stress in extreme fibre $\sigma_{bt,cal}$ = calculated bending tensile stress in extreme fibre. σ_{ac} = permissible axial compressive stress In the member subjected to axial compressive load only σ_{at} = permissible axial tensile stress in the member subjected to axial tensile load only	where , M_y, M_z = Factored applied moments about the minor and major axis of the cross – section ,respectively M_{ndy} , M_{ndz} = design reduced flexural strength under combined axial force and the respective uni –axial moments acting alone. N = factored applied axial force (Tension or comp.) N_d = Design strength in tension or comp. M_{dy} , M_{dz} = design strength under corresponding moment acting alone. α_1 ,α_2 = constants (as per table 9.1 of IS 800 :2007) n = N / N_d 1.b) For Semi – compact section (clause 9.3.1.3) In the absence of high shear force semi-compact section design I satisfactory under combined axial force and bending , if the maximum longitudinal stress under combined axial force and bending , fx satisfying the following criteria $f_x \leq f_y/\gamma_{mo}$ for the cross section without holes above criteria reduced to $N/N_d + M_y / M_{dy} + M_z / M_{dz} \leq 1.0$ 1.c) For slender section , No guideline given by IS 800 – 2007 2) Overall member strength

σ_{bc} = permissible bending compressive stress in extreme fibre σ_{bt} = permissible bending tensile stress in extreme fibre Cm = 0.85 (For members in frames where sideway is not prevented and if sideway is prevented in the plane of loading and subjected to transverse loading between their supports , ends are restrained against rotation) Cm = 1.0 (For members whose ends are unrestrained against rotation) Bending and shear Irrespective of any increase in the permissible stress,the equivalent stress $\sigma_{e,cal}$,due to co-existent bending (tension and compression) and shear stresses obtained from the formula given if clause 7.1.4.1 shall not exceed the value σ_e = 0.9 f_y where , maximum permissible equivalent stress The equivalent stress $\sigma_{e,cal}$ is obtained from the following formula : $\sigma_{e,cal} = \sqrt{[\sigma_{bt}^2{}_{,cal} + 3\tau_{vm}^2{}_{,cal}]}$ or	a) Bending and axial tension (clause 9.3.2.1) Member shall be checked for lateral torsional buckling under reduced moment M_{eff} due to tension and bending $M_{eff} = [M - \Psi T Z_{ec}/A] \leq M_d$ where , M , T = Factored applied moment and tension,respectively A =area of cross – section Zec = Elastic section modulus of the section with respect to extreme compression fibre Ψ = 0.8 if T and M vary independently = 1.0 otherwise b) Bending and axial compression (clause 9.3.2.2) Members subjected to combined axial compression and bi axial bending shall satisfy the following interaction relationship $P/P_d + K_y M_y / M_{dy} + K_z M_z / M_{dz} \leq 1.0$ K_y , Kz = Moment amplification factor about minor and major axis respectively P = Factored applied axial compression M_y , M_z = Maximum factored applied bending moments about y and z axis of the member P_d , M_{dy} ,M_{dz} = Design strength under axial compression , bending about y and z axis respectively , as governed by overall buckling Design bending strength about major axis and minor axis About major axis Mdz= Md

$= \sqrt{[\sigma_{bc}^{2}{}_{,cal} + 3\tau_{vm}^{2}{}_{,cal}]}$	M_d = design flexural strength about z axis given by section 8.2.1 of IS 800 – 2007 when lateral torsional buckling is prevented and by section 8.2.2 of IS 800 -2007 where lateral torsional buckling governs about minor axis $M_{dy} = M_d$ M_d = Design flexural strength about y axis calculated using plastic section modulus for plastic and compact sections and elastic section modulus for semi – compact sections.

Comments : -IS 800 – 2007 mention one interaction formula and it covers Plastic Compact and semi compact class and both failure mode (i.e. flexural buckling and lateral torsional buckling).About slender class IS 800 – 2007 doesn't give any guidelines because slender (class 4) section shall be avoided as far as possible in design of member subjected to axial compression and bending . As such design of compression member and flexural member is based on section classification.

SECTION B : - STUDY OF BOTH CODES

Section B.6 : - Connections

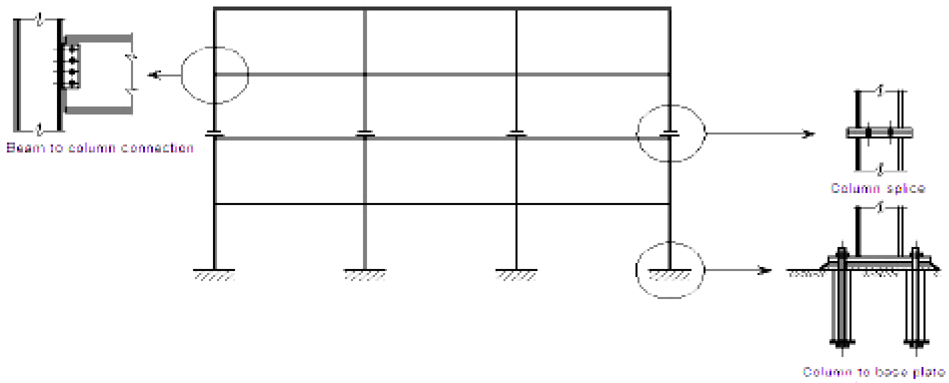
B.6.1 : - Introduction

The various elements of a steel structure like tension member, compression member and flexural member are connected by fasteners .Different types of structural elements ,each of which has to be properly attached to the neighbouring parts of the structure. This will involve the use of several forms of connection. The main classes of connection are :-

Where a change of direction occurs , e.g. beam – to- column connections , beam –to – beam connections and connections between different members in trusses.

To ensure manageable sizes of steelwork for transportation and erection e.g. columns are normally spliced every two or three storeys.

Where a change of component occurs , including connection of the steelwork to other parts of the building , e.g. column bases , connections to concrete cores and connections with walls , floors and roofs.



(Fig B.6.1)

B.6.2 :-Classification of connection

Connection may be classified :

- By rigidity

By component used in connection

Classified by rigidity

- Simple connection
- Rigid connection
- Semi- rigid connection

Simple connection (shear connection)

A simple connection ix designed in such a manner that the significant moment (which might be adversely affect the member of structure) will not develop. Simple connections are assumed to transfer only shear at some nominal eccentricity and typically used in frames up to about five stories in hight , where strength rather than stiffness govern the design.

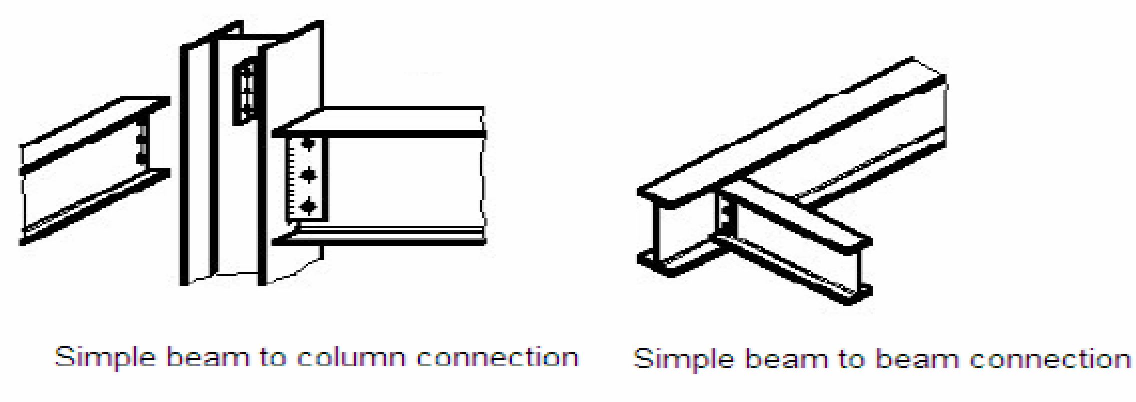


Fig B.6.2

Rigid connections (moment connections)

Rigid connections are capable of transmitting the forces and moments . A rigid connection shall be designed that its deformation has no significant influence neither on the distribution of internal forces and moments in the srtecture. Nor on its overall deformation. These are necessary in sway frames for stability and also contribute in resisting loads.

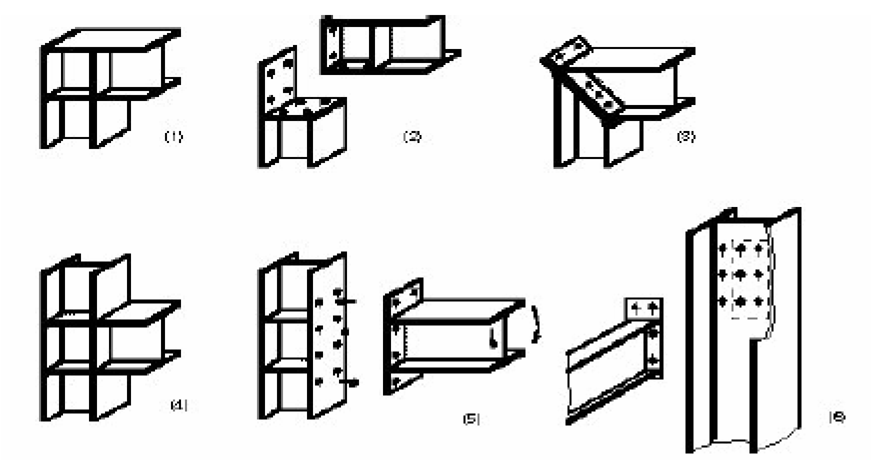


Fig B.6.3

Semi- rigid connection

A connection which does not meet the criteria for a rigid connection or a simple pinned connection as discussed above shall be classified as a semi-rigid connection. In actual practice simple connections do have some degree of rotational rigidity the developments in the semi-rigid connections. Similarly rigid connections do experience some degree of joint deformation and this can be utilized to reduce the joint design moments.

Classified by components used for connection

- Riveted
- Bolted
- Welded

Based on above classification the following combination can be achieved

- Riveted or bolted shear connection
- Riveted or bolted moment connection
- Welded shear connection
- Welded moment connection

B.6.3 :-DESIGN OF BEAM COLUMN

DATA

A column in a building 4m in height bottom end fixed , top end hinged.

Reaction load due to beam is 500 kN at an eccentricity of 100 mm from major axis of section.

DESIGN

Column is subjected to axial compression of 5×10^5 N with bending moment of 50×10^6 Nmm.

Taking design compressive stress for axial loading as 80 Mpa.

A_e reqd = $500 \times 10^3 / 80 = 6250$ mm^2

To account for additional stresses developed due to bending compression.

Try ISHB 300 @ 0.58 kN/m

A_g = 7485 sq.mm, r_{xx} = 129.5 mm, r_{yy} = 54.1 mm

f_y = 250 Mpa

Classification of section

b/t_f = 125 / 10.6 = 11.79 > 10.5 (limit for compact section)

Flange is semi compact

h_1/t_w = 249.8 / 7.6 = 32.86 < 84

Web is plastic

Therefore overall section is semi compact.

a) Section strength as governed by material failure *(clause 9.3.1)*

Axial stress = N/A_e = 500 X 10^3 / 7485

= 66.80 N/mm^2

Bending stress M_z/Z_e = 50 X 10^6 / 836.3 X 10^3

= 59.78 N/mm^2

As the section is semi compact use clause 9.3.1.3 *(p. no. 71)*

Due to bending moment at top, horizontal shear developed 'V' is

18.75 kN = 18750 N

Shear strength of section $V_d = ((f_y / \sqrt{3}) \cdot h \cdot t_w) / 1.10$

= 299 kN

As $V/V_d = 18750 / 299 \times 10^3 = 0.062 < 0.6$

Reduction in moment capacity need not be done.

As per clause 9.3.1.3 (p. no. 71)

Total longitudinal compressive stress

$f_x = 66.80 + 59.78$

$= 126.58 < f_y/\gamma_{mo} = 227.27$…… OK

Alternately

$N = 500$ kN

$N_d = A_g \cdot f_y / \gamma_{mo} = 7485 \times 250 / 1.1 = 1701.136$ kN

$M_z = 50 \times 10^6$ Nmm = 50 kNm

$M_{dz} = Z_e \cdot f_y / \gamma_{mo} = 836.3 \times 10^3 \times 250 /1.10$

$= 190.068$ kN

Hence, $(500 / 1701.136) + (50 / 190.068)$

$= 0.557 < 1$ ……. OK

b) Member strength as governed by buckling failure clause 9.3.2 (p. no. 71)

In the absence of M_y, equations are reduced to

Where, $P = 500 \times 10^3$ N

$M_z = 50 \times 10^6$ Nmm

$M_{dz} = \beta_b \cdot Z_p \cdot f_{bd}$

$\beta_b = Z_e / Z_p$ as section is semicompact

Therefore $M_{dz} = Z_e f_{bd}$

$f_{bd} = \chi_{LT} f_y / \gamma_{mo}$

χ_{LT} = bending stress reduction factor to account

torsional buckling.

$\alpha_{LT} = 0.21$ for rolled section

$f_{cr,b}$ depends on following factors

$k_L / r_{yy} = 0.8 \text{ X } 4000 / 54.1 = 59.15$

$h / t_f = 300/10.6 = 28.30$

Using table 14, (p. no. 57)

$f_{cr,b} = 691.71 \text{ N/mm}^2$

$= 0.060 < 0.4$

As per clause 8.2.2 (p. no. 54)

Resistance to lateral buckling need not be checked and member may be treated as laterally supported.

$M_{dz}=Z_e . f_y / \gamma_{mo} = 190$ kNm

Evaluation of P_{dy} buckling load @ yy axis

Referring table 10 (p. no. 44)

$h/b_f=300/250 = 1.2$

buckling @ yy axis is by class ‘c’

$t_f = 10.6 \text{ mm} < 100\text{mm}$

buckling @ zz axis is by class ‘b’

l_y / r_y = 3200/54.1 = 59.15

For f_y = 250 and using Table 9 (c), (p. no. 42)

F_{cdy} = 169.275 N/mm^2

$P_{dy} = A_g . f_{cdy}$

= 1267.02 kN

Evaluation of P_{dz} buckling @ zz axis

l_z / r_z = 3200 / 129.5 = 24.71

For f_y = 250 and using *Table 9 (b), (p. no. 41)*

f_{cdz} = 220.76 N/mm^2

Therefore $p_{dz} = A_g . f_{cdz}$

= 1652.38 kN

$K_z = 1 + (\lambda_z - 0.2)n_z$

Where,

l_z / r_z = 24.71, h/t_f = 300 / 10.6 = 28.30

From table 14 (p. no. 57)

$f_{cr,z}$ = 4040 N/mm^2

Ratio of actual applied load to axial strength,

n_z = 500 / 1625.38 = 0.30

n_y = 500 / 1267.02 = 0.39

$\lambda_z = \sqrt{}$ 250/4040 = 0.246

$K_z = 1 + (\lambda_z - 0.2)\, n_z = 1.0138 < 1 + 0.8\, n_z$

$< 1.24 \ldots.$ OK

ψ = ratio of minimum to maximum BM

$\psi = -25 / 50 = -1 / 2$

$C_{mz} = 0.6 + 0.4 \times (\psi) = 0.4$

$= 0.844$

$< 1 \ldots\ldots.$ OK

$< 1 \ldots\ldots.$ OK

Hence select ISHB 300 @ 0.58 kN/m as a section for eccentrically loaded column.

Design of Beam Column
Working Stress Method
IS : 800 – 1984

Checking section ISHB 300 @ 0.58 kN/m

$A = 7485$ sq mm

$\sigma_{ac,cal} = P/A = 66.80\ N/mm^2$

slenderness ratio $= L / r_{yy} = 59.15$

for $f_y = 250$ Mpa, $\sigma_{ac} = 121.15 N/mm^2$

from table 5.1 (p. no. 39)

β=ratio of smaller to larger moment = 0.5

Therefore, $C_{mx} = 0.6 – 0.4 \times 0.5 = 0.4 \geq 0.4$ OK

$\sigma_{bcx,cal.} = 50000 / 836.3 = 59.78$ N/mm^2

f_{cc} = elastic critical stress in compression

$= \pi^2 E / \lambda^2 = 563.6$ N/mm^2

σ_{bcx} = Permissible bending stress in compression.

As column is laterally unsupported

following ratios are evaluated.

D/T = 28.30, L / r_{yy} = 59.15

As T / L = 10.6 / 7.6 < 2

for f_y = 250 using table 6.1 B (p. no. 58)

$\sigma_{bcx} = 150$ N/mm^2

< 1 ….. OK

Hence requirement of section for a column under

eccentric load is same as ISHB 300 @ 0.58 kN/m

Thus reserve strength in a section by LSM is more than WSM.

LSM

- Interaction bet^n axial & uniaxial bending is considered taking buckling due to axial loading about both axes of c/s

- C_{mx} = 0.4
- Combined interaction is considered for buckling @ both axes of cross section.
- Interaction values are

  @ yy axis… 0.612

  @ zz axis… 0.406

WSM

- Interaction is countered only by taking buckling due to axial load @ weaker axis with bending @ major axis.
- C_{mx} = 0.4
- Combined interaction is considered for buckling @ yy axis only.
- Interaction value is

  @ yy axis… 0.7486

SECTION C : - PROJECT PROBLEM

Section C.1 : - Problem Data and Analysis

The project problem consists of analysis of a foot over bridge by STAAD – PRO and then design by IS 800 – 2007 & IS 800 – 1984. This design example covers design of all basic structural members (axial , flexural and combined).

Data of railway foot over bridge :-

Clear span = 34 m

Walk way width = 2.444 m

Clear height above top of rail = 6.5 m

Type of truss configuration = N type

Depth of truss girder = 2.3 m

Lateral restrain for compression chord = X bracing

Assumption : - (For design by both codes)

1) Dead load and live loads IS 875 part I & II
2) Earthquake loading IS 1893 – 2002
3) Wind loading IS 875 part III
4) Grade of steel (f_y) 250 MPa
5) Cross sections used Indian standard hot rolled sections
6) While designing care is taken that strength of system is governed by member and not by the connections.Therefore connection details are not covered.

Loading calculation as per IS : 875 (part I to V)

1) Dead load :-
   a) RCC slab 110 mm thick.
      Contributory width of floor beams = 2.266 m

Density of concrete = 25 kN/m^3

UDL on beams (602 to 617) = 0.11 x 25 x 2.266 = 6.3 kN/m

UDL on beams (601 & 618) = 3.2 MTon /m Say

B) vertical cladding@ 15 kg/m^2

Height of cladding = 2.3 m

UDL on beams (18 to 32, 80 to 94 , 305 ,307, 601 ,618) = 2.3 x 0.015

= 0.0345 MTon /m

b) Staircase load

Total rise to climb = Height b/w FGL and bottom of girder + depth of bottom girder + depth of bottom chord (ssumed as ISMC 300) + Thickness of R.C.C. slab

= 6.5 + 0.3 + 0.11 = 6.9 m

Rise say = 200 mm

No of risers = 6.9/0.2 = 34.5 say 35

Hence no. of trends = 35 -1 = 34

Say tread of 300 mm , hence total going = 0.3 x 34 = 10.2 m

Let there be central landing of 1.2 m

Hence total plan length of staircase ,provide central column = 10.2 + 1.2 = 11.4 m

Provide central column ,

Hence contributory span on bridge structure , = 11.4/4 = 2.85

Staircase width = 2.3 m

Hence plan area for one column DL @ 0.2 $MTon/m^2$ = 3.2 x 0.2 = 0.64 MTon

Joint load applied (17 65 112 117) = 0.64 MTon

2) Live Load @ 0.5 Ton/m^2

Contributory width of floor beams = 2.26 m

UDL on beams (602 to 617) = 0.5 x 2.266 = 11.4 kN/m = 1.14 MTon/m

UDL on beams (601 & 618) = 0.57 MTon /m say

Staircase LL @ 0.5 MTon/m^2 = 3.2 x 0.5 = 1.6 MTon

Wind Load (IS 875 – III)

Design wind pressure (N/m^2) is given by

$P_z = 0.V_z^2$ where V_z is design wind speed in (m/s) $V_z = V_b\ k_1k_2k_3$

Here,

$V_{\mathbf{b}}$ = Basic wind speed in m/s for given region .

= 44 m/s

$K_{1\ =}$ probability factor (risk coefficient) (risk coefficient) (see 5.3.1)

= 1.0 (Table 1 all general building and structure)

K_2 = terrain , height and structure size factor (see 5.3.2)

= 0.98 (Terrain category 2 & class B i.e. max dimension is greater than 20 m)

K_3 = topography factor (see 5.3.3)

= 1.0

V_z = 44 x 1 x 0.98 x 1 = 43.12 m/s

P_z = 0.6 x 43.12/1000 = 0.1 MTon/m^2

Min. value of P_z is = 0.15 MTon/m^2

Considering side cladding of ht . = 2.3 m

Uniform wind load applied on structure (top and bottom chord)

= 2.3/2 x 0.15 = 0.17 MTon/m

Joints loads to be applied = 0.17 x 2.27 = 0.38 MTon

(joints 1 to 14 , 110 ,111 ,34 to 46 ,104 ,114 and 127)

At end joints = 0.38 x 0.5 = 0.19 MTon

(joints 64 , 105 , 116 and 128)

UDL applied on column = 0.15 x 2.27/2 = 0.17 MTon/m

(members in Z direction only ,not considered in X direction as projection area in X is less)

3) Earthquake loading (IS 1893 – 2002)

Z = 0.16 (Zone factor for Delhi)

I = 1.0 (Importance factor)

R = 5 (eccentricity braced frame)

T = 0.085 x $H^{0.75}$ (time period)

(H is the height of the structure)

= 0.085 x $8.8^{0.075}$ = 0.43 sec

Soil type assumed as soft hence Sa/g = 2.5 , Damping C = 1.4 for steel structures

Hence A_n = C xZ/2 x I/R x Sa/g = 1.4 x 0.16/2 x 1/5 x 2.5 = 0.056

Dead load mass = 54.5 MTon (from static check)

50% live load mass = 60.5/2 = 30.25 MTon (from static check)

Hence minimum base shear = 0.056 x (54.5 +30.25) = 4.75 MTon

Appiled to two joints 116 & 117 only = 2.375 MTon

(Applied in x direction only , not considered in Z direction as wind is governing)

Analysis:-

(Analysis is done using staad-pro-2006)

```
STAAD SPACE
START JOB INFORMATION
```

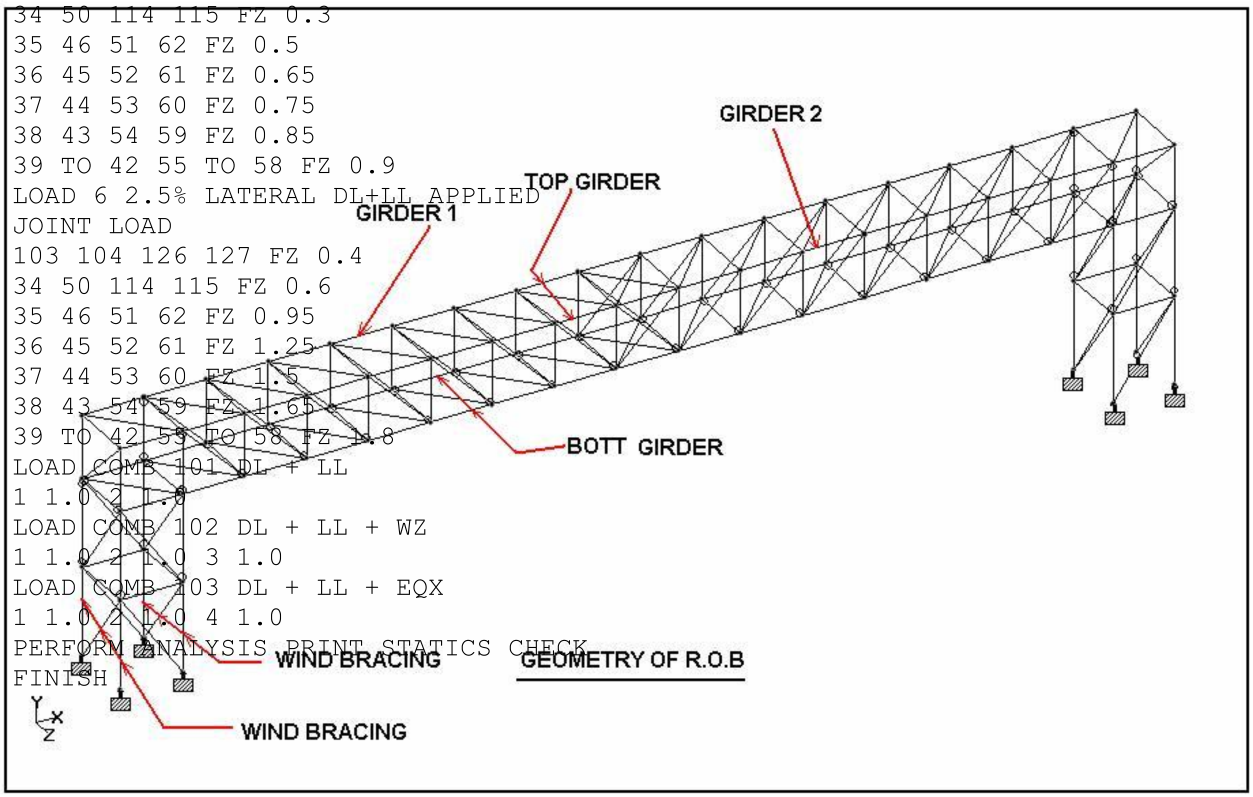
```
JOB NAME FOB
JOB CLIENT VEERMATAT JIJABAI INSTITUTE OF TECHNOLOGY
JOB NO 01
JOB PART ILLUSTRATIVE DESIGN OF ROB
ENGINEER NAME S.B.Kharmale
CHECKER NAME B.A.Naik
ENGINEER DATE 20-Jan-07
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER MTON
JOINT COORDINATES
1 0 0 0; 2 2.266 0 0; 3 4.532 0 0; 4 6.798 0 0; 5 9.064 0 0; 6 11.33 0 0;
7 13.596 0 0; 8 15.862 0 0; 9 18.128 0 0; 10 20.394 0 0; 11 22.66 0 0;
12 24.926 0 0; 13 27.192 0 0; 14 29.458 0 0; 17 0 0 2.44; 18 2.266 0 2.44;
19 4.532 0 2.44; 20 6.798 0 2.44; 21 9.064 0 2.44; 22 11.33 0 2.44;
23 13.596 0 2.44; 24 15.862 0 2.44; 25 18.128 0 2.44; 26 20.394 0 2.44;
27 22.66 0 2.44; 28 24.926 0 2.44; 29 27.192 0 2.44; 30 29.458 0 2.44;
34 2.266 2.3 0; 35 4.532 2.3 0; 36 6.798 2.3 0; 37 9.064 2.3 0; 38 11.33
2.3 0;
39 13.596 2.3 0; 40 15.862 2.3 0; 41 18.128 2.3 0; 42 20.394 2.3 0;
43 22.66 2.3 0; 44 24.926 2.3 0; 45 27.192 2.3 0; 46 29.458 2.3 0;

50 2.266 2.3 2.44; 51 4.532 2.3 2.44; 52 6.798 2.3 2.44; 53 9.064 2.3
2.44;
54 11.33 2.3 2.44; 55 13.596 2.3 2.44; 56 15.862 2.3 2.44; 57 18.128 2.3
2.44;
58 20.394 2.3 2.44; 59 22.66 2.3 2.44; 60 24.926 2.3 2.44; 61 27.192 2.3
2.44;
62 29.458 2.3 2.44; 64 -2.266 0 0; 65 -2.266 0 2.44; 66 0 -6.5 0;
67 0 -6.5 2.44; 68 -2.266 -6.5 0; 69 -2.266 -6.5 2.44; 82 -2.266 -3.25
2.44;
83 0 -3.25 2.44; 84 0 -3.25 0; 85 -2.266 -3.25 0; 103 0 2.3 2.44; 104 0
2.3 0;
105 -2.266 2.3 0; 106 -2.266 2.3 2.44; 110 33.99 0 0; 111 31.724 0 0;
112 33.99 0 2.44; 113 31.724 0 2.44; 114 31.724 2.3 0; 115 31.724 2.3
2.44;
116 36.256 0 0; 117 36.256 0 2.44; 118 33.99 -6.5 0; 119 33.99 -6.5 2.44;
120 36.256 -6.5 0; 121 36.256 -6.5 2.44; 122 36.256 -3.25 2.44;
```

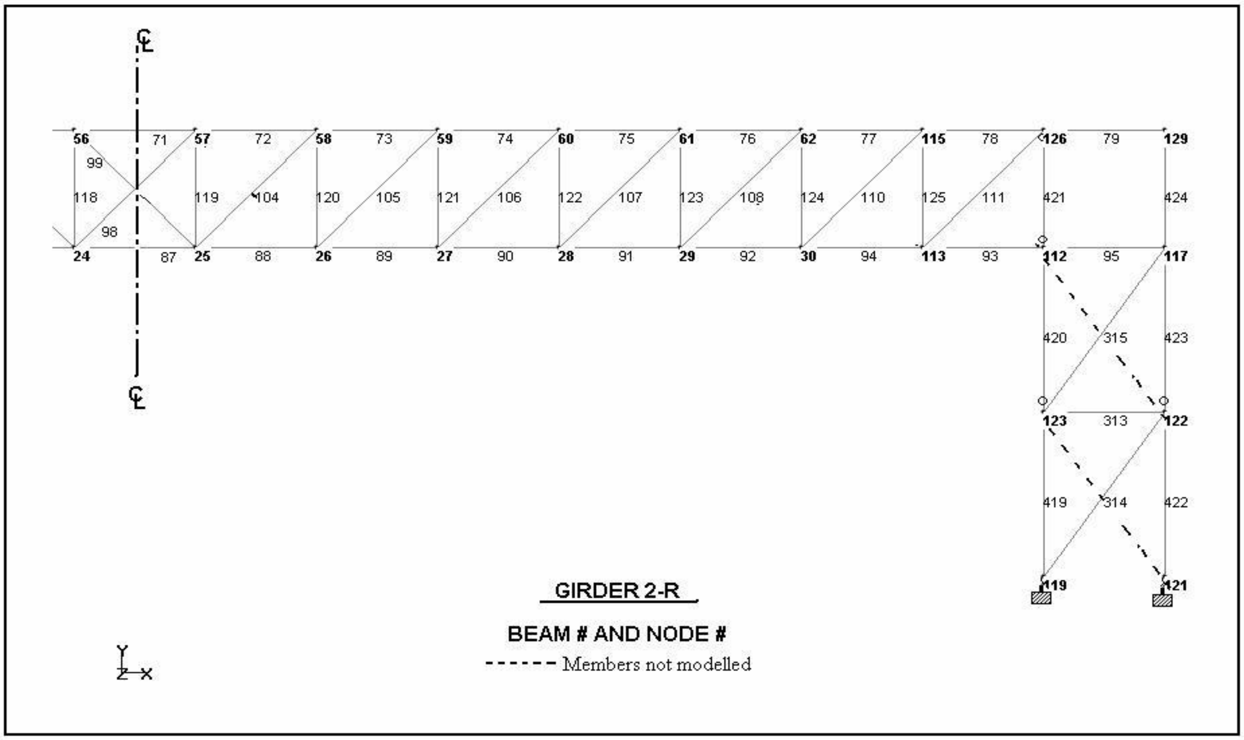
```
123 33.99 -3.25 2.44; 124 33.99 -3.25 0; 125 36.256 -3.25 0;
126 33.99 2.3 2.44; 127 33.99 2.3 0; 128 36.256 2.3 0; 129 36.256 2.3
2.44;
MEMBER INCIDENCES
1 104 105; 2 34 104; 3 34 35; 4 35 36; 5 36 37; 6 37 38; 7 38 39; 8 39 40;
9 40 41; 10 41 42; 11 42 43; 12 43 44; 13 44 45; 14 45 46; 15 114 46;
16 114 127; 17 127 128; 18 1 2; 19 2 3; 20 3 4; 21 4 5; 22 5 6; 23 6 7; 24
7 8;
25 8 9; 26 9 10; 27 10 11; 28 11 12; 29 12 13; 30 13 14; 31 111 14; 32 110
111;
33 2 34; 34 3 35; 35 4 36; 36 5 37; 37 6 38; 38 7 39; 39 8 40; 40 9 41;
41 10 42; 42 11 43; 43 12 44; 44 13 45; 45 14 46; 46 111 114; 47 104 2;
48 34 3; 49 35 4; 50 36 5; 51 37 6; 52 38 7; 53 39 8; 54 8 41; 55 40 9;
56 42 9; 57 43 10; 58 44 11; 59 45 12; 60 46 13; 61 114 14; 62 127 111;
63 103 106; 64 50 103; 65 50 51; 66 51 52; 67 52 53; 68 53 54; 69 54 55;
70 55 56; 71 56 57; 72 57 58; 73 58 59; 74 59 60; 75 60 61; 76 61 62;
77 115 62; 78 115 126; 79 126 129; 80 17 18; 81 18 19; 82 19 20; 83 20 21;
84 21 22; 85 22 23; 86 23 24; 87 24 25; 88 25 26; 89 26 27; 90 27 28; 91
28 29;
92 29 30; 93 112 113; 94 113 30; 95 117 112; 96 50 19; 97 24 57; 98 56 25;
99 51 20; 100 52 21; 101 53 22; 102 54 23; 103 55 24; 104 25 58; 105 26
59;
106 27 60; 107 28 61; 108 29 62; 109 103 18; 110 115 30; 111 126 113;
112 18 50; 113 19 51; 114 20 52; 115 21 53; 116 22 54; 117 23 55; 118 24
56;
119 25 57; 120 26 58; 121 27 59; 122 28 60; 123 29 61; 124 30 62; 125 113
115;
301 64 84; 302 85 66; 303 116 124; 304 125 118; 305 64 1; 306 84 85;
307 116 110; 308 124 125; 309 65 17; 310 82 83; 311 82 67; 312 65 83;
313 122 123; 314 122 119; 315 117 123; 316 83 84; 317 85 82; 318 82 68;
319 17 84; 320 65 85; 321 66 83; 322 64 106; 323 123 124; 324 125 122;
325 122 120; 326 112 124; 327 117 125; 328 118 123; 329 116 129; 401 68
85;
402 85 64; 403 64 105; 404 66 84; 405 84 1; 406 1 104; 407 118 124;
408 124 110; 409 110 127; 410 120 125; 411 125 116; 412 116 128; 413 69


82;
414 82 65; 415 65 106; 416 67 83; 417 83 17; 418 17 103; 419 119 123;
420 123 112; 421 112 126; 422 121 122; 423 122 117; 424 117 129; 501 105
106;
502 103 104; 503 50 34; 504 51 35; 505 52 36; 506 53 37; 507 54 38; 508 55
39;
509 56 40; 510 57 41; 511 58 42; 512 59 43; 513 60 44; 514 61 45; 515 62
46;
516 115 114; 517 126 127; 518 128 129; 519 105 103; 520 104 50; 521 34 51;
522 35 52; 523 36 53; 524 37 54; 525 38 55; 526 39 56; 527 56 41; 528 40
57;
529 57 42; 530 58 43; 531 59 44; 532 60 45; 533 61 46; 534 114 62; 535 127
115;
536 128 126; 601 65 64; 602 17 1; 603 18 2; 604 19 3; 605 20 4; 606 21 5;
607 22 6; 608 23 7; 609 24 8; 610 25 9; 611 26 10; 612 27 11; 613 28 12;
614 29 13; 615 30 14; 616 113 111; 617 112 110; 618 117 116; 619 1 18;
620 2 19; 621 3 20; 622 4 21; 623 5 22; 624 6 23; 625 7 24; 626 8 25; 627
24 9;
628 25 10; 629 26 11; 630 27 12; 631 28 13; 632 29 14; 633 64 17; 634 111
30;
```

```
635 110 113; 636 116 112; 637 69 85; 638 67 84; 639 68 84; 640 69 83;
641 82 17; 642 85 1; 643 82 64; 644 83 1; 645 65 105; 646 121 123; 647 120
124;
648 119 124; 649 121 125; 650 122 112; 651 125 110; 652 122 116; 653 123
110;
654 117 128;
START GROUP DEFINITION
MEMBER
_MAIN_GIRDER 2 TO 16 18 TO 62 64 TO 78 80 TO 94 96 TO 125 404 TO 409 416
TO 421
_FLR_BMS 601 TO 618
_TPGRDER 1 TO 17 63 TO 79 501 TO 536
_BPGRDER 18 TO 32 80 TO 95 305 307 309 601 TO 636
_W_Z_BRAC 316 TO 329 401 TO 405 407 408 410 TO 417 419 420 422 TO 424 501
-518 601 602 617 618
_BU_PR_BR 501 TO 536
_EQX_BRAC 95 303 304 307 308 313 TO 315 407 408 410 411 419 420 422 423
END GROUP DEFINITION
DEFINE MATERIAL START
ISOTROPIC STEEL
E 2e+007
POISSON 0.3
DENSITY 7.85
END DEFINE MATERIAL
CONSTANTS
MATERIAL STEEL ALL
MEMBER PROPERTY INDIAN
619 TO 636 TABLE SD ISA50X50X6 SP 0.008
95 305 307 309 601 618 TABLE ST ISMB150
1 TO 32 63 TO 94 TABLE D ISMC300 SP 0.008
*1 TO 32 63 TO 94 TABLE SD ISA100X100X10 SP 0.01
401 TO 403 410 TO 415 422 TO 424 TABLE ST ISMB300


301 TO 304 306 308 310 TO 329 637 TO 654 TABLE SD ISA50X50X6 SP 0.008
38 TO 41 50 TO 59 97 98 100 TO 107 117 TO 120 TABLE SD ISA50X50X6 SP 0.008
602 TO 617 TABLE TB ISMB150 WP 0.065 TH 0.008
509 510 524 TO 531 TABLE SD ISA50X50X6 SP 0.008
37 42 116 121 TABLE SD ISA60X60X6 SP 0.008
47 TO 49 60 TO 62 96 99 108 TO 111 TABLE SD ISA65X65X8 SP 0.008
35 36 43 44 114 115 122 123 TABLE SD ISA65X65X8 SP 0.008
33 34 45 46 112 113 124 125 TABLE SD ISA75X75X8 SP 0.008
523 532 TABLE SD ISA60X60X6 SP 0.008
519 TO 522 533 TO 536 TABLE SD ISA65X65X8 SP 0.008
508 511 TABLE SD ISA60X60X6 SP 0.008
504 TO 507 512 TO 515 TABLE SD ISA75X75X8 SP 0.008
501 TO 503 516 TO 518 TABLE SD ISA100X100X8 SP 0.008
404 TO 409 416 TO 421 TABLE ST ISMB500
*78 92 122 TABLE TB ISMB300 WP 0.17 TH 0.025
SUPPORTS
66 TO 69 118 TO 121 FIXED
*66 TO 69 118 TO 121 PINNED
MEMBER RELEASE
601 TO 618 START MZ
601 TO 618 END MZ
MEMBER TRUSS
```

```
2 TO 16 18 TO 62 64 TO 78 80 TO 94 96 TO 125
MEMBER TRUSS
306 308 310 313
MEMBER TRUSS
316 317 323 324
MEMBER TRUSS
18 TO 32 80 TO 95 305 307 309 619 TO 636
MEMBER TRUSS
1 TO 17 63 TO 79 501 TO 536
MEMBER TENSION
301 TO 304 311 312 314 315 639 TO 642 646 647 650 651
MEMBER TENSION
318 TO 322 325 TO 329 637 638 643 TO 645 648 649 652 TO 654
LOAD 1 DEAD LOAD (DL)
SELFWEIGHT Y -1.05
*5 % EXTRA WEIGHT ADDED TO TAKE CARE OF CONNECTIONS
MEMBER LOAD
602 TO 617 UNI GY -0.63
601 618 UNI GY -0.32
*THICKNESS OF SLAB 125MM, WEIGHT = 2.5*0.11 = 0.275 T/SQM
*CW = 2.266 M, HENCE UDL = 0.275*2.266 = 0.63 T/M SAY
*CLADDING AT 15 KG/SQM W = 0.015*2.3 = 0.0345
18 TO 32 80 TO 94 305 307 601 618 UNI GY -0.0345
*STAIRCASE LOAD
*Total rise to climb = 6.5 + 0.3+0.11 = 6.9 m
*Rise say = 200 mm
*No of Risers = 6.9/0.2 = 34.5 say 35
*Hence No of treads = 35-1 = 34
*Say Tread of 300 mm, hence Total going = 0.3x34 = 10.2 m
*Let there be central landing of 1.2m
*Hence Total Plan length of staircase = 10.2+1.2=11.4m
*Provide central column,
*Hence contributory span on Bridge structure = 11.4/4 =2.85
*Staircase width = 2.3 m
*Hence plan area for one column = 2.85x2.3/2 = 3.2 sqm
*DL @ 0.2 T/sqm = 3.2 x 0.2 = 0.64 MTon
*LL @ 0.5 T/Sqm = 3.2 x 0.5 = 1.6 MTon
JOINT LOAD
17 65 112 117 FY -0.64
LOAD 2 LIVE LOAD (LL)
MEMBER LOAD
602 TO 617 UNI GY -1

601 618 UNI GY -0.57
*STAIRCASELOAD
JOINT LOAD
17 65 112 117 FY -1.6
*INTENSITY CONSIDERED = 0.5 T/SQM
*CW = 2.266 M, HENCE UDL = 0.5*2.266 = 1.14 T/M SAY
LOAD 3 WIND LOAD IN Z DIR
JOINT LOAD
1 TO 14 34 TO 46 104 110 111 114 127 FZ 0.38
64 105 116 128 FZ 0.19
MEMBER LOAD
401 402 404 405 407 408 410 411 UNI GZ 0.17
*PLEASE NOTE IN X DIRECTION WIND WILL NOT GOVERN AS PROJECTED AREA IS LESS
*HENCE WIND LOAD IS NOT APPLIED AND GOVERNING LOAD CASE IS EQX WHICH IS
```

```
CONSIDERED
LOAD 4 EQ X
*Z=0.16(ZONE FACTOR IS MUMBAI)
*I=1.0(IMPORTANCE FACTOR)
*R=5(ECCENTRIC BRACED FRAME)
*TIME PERIOD T= 0.085 * H^0.75 =0.085 *8.8^0.75 = 0.43 SEC
*SOIL TYPE ASSUMED AS SOFT HENCE SA/G = 2.5
*DAMPING C = 1.4 FOR STEEL STRUCTURES
*HENCE Ah = (0.16/2)*2.5*(1/5)*1.4=0.056
*DL MASS = 54.5 MTON (APPROX)
*50%LL MASS = 60.5/2 =30.25 MTON
*HENCE MIN BASE SHEAR = 0.056(54.5+30.25)=4.75 MTON SAY
JOINT LOAD
116 117 FX 2.375
*NOTE THIS LOAD CASE IS USED ONLY TO DESIGN OF TOP PLAN BRACING
LOAD 5 2.5% LATERAL DL APPLIED
JOINT LOAD
103 104 126 127 FZ 0.2
34 50 114 115 FZ 0.3
35 46 51 62 FZ 0.5
36 45 52 61 FZ 0.65
37 44 53 60 FZ 0.75
38 43 54 59 FZ 0.85
39 TO 42 55 TO 58 FZ 0.9
LOAD 6 2.5% LATERAL DL+LL APPLIED
JOINT LOAD
103 104 126 127 FZ 0.4
34 50 114 115 FZ 0.6
35 46 51 62 FZ 0.95
36 45 52 61 FZ 1.25
37 44 53 60 FZ 1.5
38 43 54 59 FZ 1.65
39 TO 42 55 TO 58 FZ 1.8
LOAD COMB 101 DL + LL
1 1.0 2 1.0
LOAD COMB 102 DL + LL + WZ
1 1.0 2 1.0 3 1.0
LOAD COMB 103 DL + LL + EQX
1 1.0 2 1.0 4 1.0
PERFORM ANALYSIS PRINT STATICS CHECK
FINISH
```

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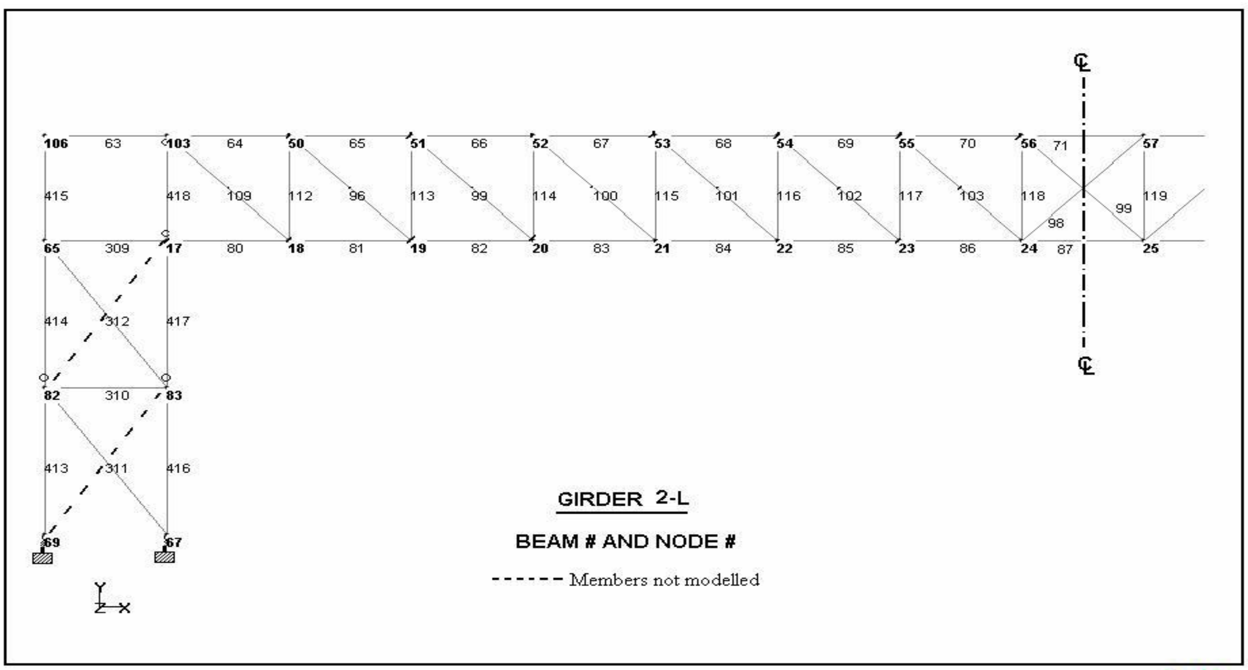
CL
GIRDER 2-R
BEAM # AND NODE #
Members not modelled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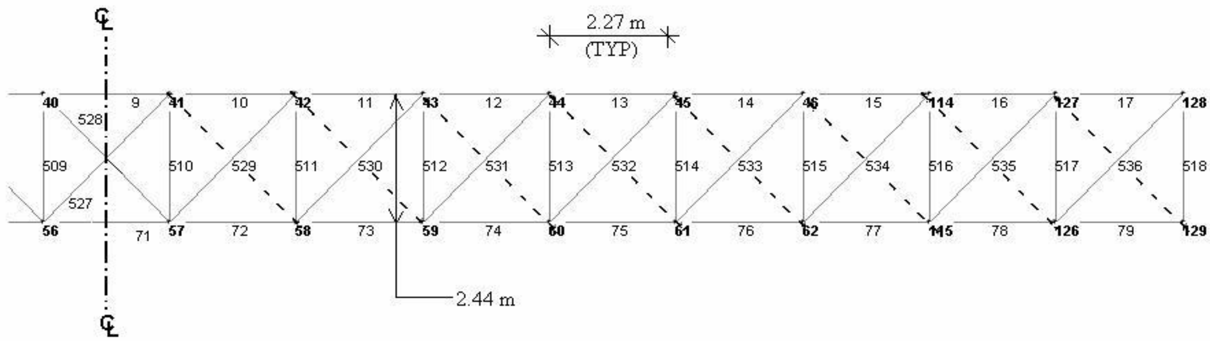
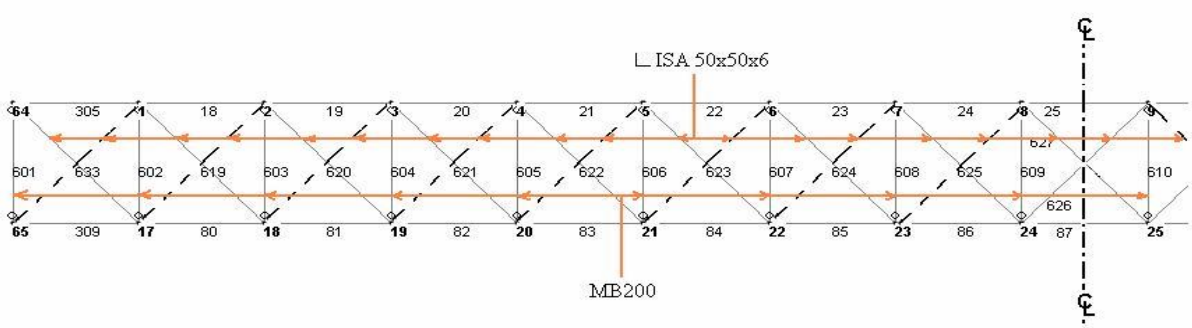
CL
GIRDER 2-L
BEAM # AND NODE #
Members not modelled



TOP GIRDER - R

BEAM # AND NODE #

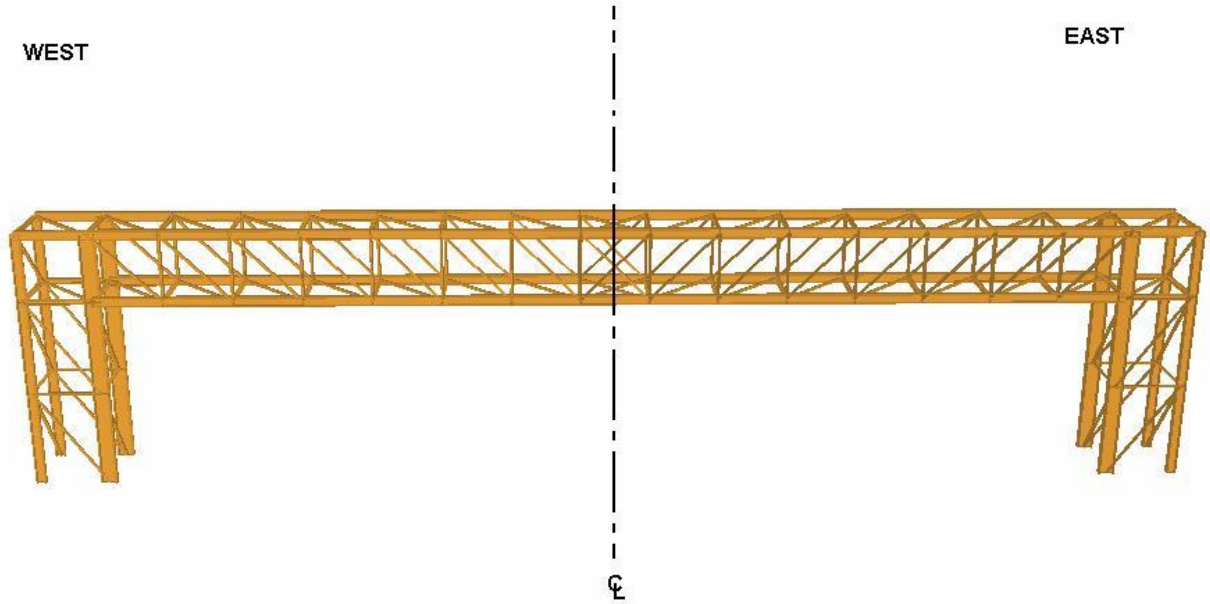
------ Members not modelled



BOTT GIRDER - L

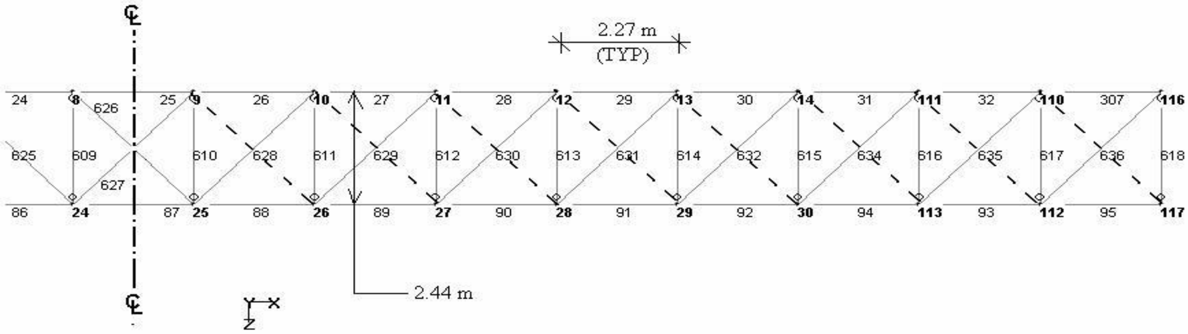
BEAM # AND NODE #

------ Members not modelled



3D VIEW - 02

3D VIEW - 03



BOTT GIRDER - R

BEAM # AND NODE #

------ Members not modelled

3D VIEW -01

Fig C.1.2:- Modeling of FOB in STAAD PRO 2006

Section C.2:- Design by both codes (Table C.2.1) :-

s.no.	staad member	Nature	Sections as per IS :800(1984)	Sections as per IS :800 (2007)	Capacity KN or KN - m							
					IS :800 (1984)			IS :800 (2007)				
					P_d	A(cm^2)	IR	P_d	P_u	A(cm^2)	IR	
1.	21 to 29 83 to 91	T	2ISA 110x110x15	2ISA 100x100x12	648	61.62	0.70	972	1026.8	45.2	0.95	
2.	18to20 ,30 to 32 ,80 to 82 and 92 to 94 ,47 to 50,59 to 62	T	2ISA 70x70x10	2ISA 60X60X8	268	26.04	0.68	401	407	17.92	0.99	
3.	96,99,100, 107 to 111	T	2ISA 70x70x10	2ISA 60x60x8	270	26.04	0.69	405	407	17.92	1.00	
4.	51 to 58 101 to 106 98 and 99 302 and 301	T	ISA 75x75x10	ISA 70x70x6	145	14.02	0.69	173	183	8.06	0.96	
5.	303 ,304and 311,312,314	T	ISA 50x50x5	ISA 55x55x5	48	4.79	0.67	57	119.78	5.27	0.48	

	315											
6.	633,619 to 632 and 634 to 636	T	ISA 50x50x5	ISA 50x50x6	47	4.79	0.67	56	129.1	5.68	0.43	
7.	519 to 523 and 532 to 536	T	2ISA 70x70x10	2ISA 60x60x8	272	26.04	0.70	408	407	17.92	1.00	
8.	524 to 531											
9.	318 to 320 325 to 328	T	ISA 110x110x8	ISA 65x65x8	180	17.02	0.71	215	221	9.76	0.97	
10.	33 to 37 and 42 to 45 ,112 to 116 and 121 to 125	T	ISA 65x65x8	ISA 60x60x8	100	9.76	0.69	120	203.5	8.96	0.59	
		C	2ISA 70x70x6	2ISA 70X70X8	168	16.12	0.70	252	264.5	21.20	0.95	
11.	38 to 41 and 117 to 120 , 316,317,323and 324	C	ISA 45x45x5	ISA 70x70x8	44.1	4.28	0.69	52.9	74	10.59	0.71	
12.	306,310,313 and 308 ,305,95,309 ,307	C	ISA 30x30x5	ISA 60x60x6	25	2.77	0.60	30	38.85	7.44	0.77	
13.	5 to 13 and 67 to 75	C	2ISA 130x130x15	2ISA 110x110x15	691.3	73.62	0.63	1037	1090.6	51.7	0.95	
14.	1 to 4 and 63 to 66 ,14 to 17 76 to 79	C	2ISA 90x90x12	2ISA 90x90x12	424	40.38	0.70	635	636	40.38	1.00	
15.	501 to 505 514 to 518	C	2ISA 90x90x6	2ISA 70x70x10	200	20.94	0.64	299	302	26.04	0.99	
16.	506 to 513	C	ISA 75x75x8	ISA 65x65x8	120	11.38	0.70	180	206	9.76	0.87	
17.	401 to 403 and 413 to 415, 410 to 412,	flexure	ISMB 350	ISMB 200	122	66.71	0.95	183	204	32.33	0.98	

	422 to 424												
18.	**601 to 618**	**flexure**	**ISMB 175**	**ISMB 175**	**17.5**	**24.02**	**0.70**	**21**	**28.53**	**24.02**	**0.74**		
19.	**404 to 409 & 416 to 421**	**flexure**	**ISMB 600**	**ISMB 500**	**395**	**156.24**	**0.78**	**474**	**1442.6**	**110.74**	**0.78**		

CONCLUSIONS

The main objective of this comparative study of IS: 800 (2007) & IS :800 (1984) is to study the Limit State Method for design of steel structure and then compare the design methodology for basic structural element by both codes Following conclusions are drawn and summarized from the Section B “Study of both codes” and Section C “Project problem” of this dissertation.

From Section B “Study of both codes”

Basis of design

The design methodology by IS :800 (2007) is based on Limit State and IS 800(1984) is based on Working/Allowable Stress Method . Even though IS: 800 (2007) doesn’t disregard the allowable/working stress design format completely but in the section 11 of IS: 800 (2007) it has been proposed that wherever it is not possible or feasible one can adopt the working stress design format.

Section classification

IS 800 – 1984 is based on ‘Allowable Stress Method’ the extreme fibre stress in the beams is restricted to 0.66 f_y . In addition, the ‘I’ sections rolled in India are found to be at least semi-compact in which the section classification for Indian standard ‘I’ beams have been presented. In other words the flange outstands of the ‘I’ beams rolled in India are so proportioned that they attain yield stress before local buckling . Because of these two reasons, 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 .

IS :800 (2007) classify the cross section based on limiting width to thickness ratio of individual plate element to avoid the local buckling and remains silent on this matter about slender class. (As such some of ISA section falls in slender class.)

Design of Tension member

It is generally assumed that the distribution of stresses in cross-sections of members subjected to axial tensile forces is 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 β 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.

In LSD ,in addition to net section failure and block shear failure , yielding of the gross section must also be considered so as to prevent excessive deformation of the member.

Design of Compression member :-

IS 800 (2007) uses multiple column curves (modified ECCS buckling curves) which are based on Party – Robertson approach. The factors, non-dimensional slenderness ratio, imperfection factor and the stress reduction factor are responsible for evaluating the strength of single angle strut. However, as per IS 800 (1984) only slenderness ratio is responsible for deciding allowable stress for section.

Design of flexural member :-

In IS 800 (1984) the local buckling is avoided by specifying b/t limits . Hence we don't consider local buckling explicitly . However, In IS 800 (2007) , the local buckling would be the first aspect as far as the beam design is concern (by using section classification).

The section designed as per LSD is having more reserve capacity for BM and SF as compared to WSM beam designed by LSM is more economical. Plastic design strength of flexural member by IS 800 (2007) governs by flexural-torsional buckling mode of failure so this code may prove economical .

Design of Beam Column :-

As per LSM , Interaction both axial & uniaxial bending is considered taking buckling due to axial loading about both axis of cross-section . As per WSM Interaction is countered only by taking buckling due to axial load @ weaker axis with bending @ major axis.
Combined interaction is considered for buckling @ both axis of cross-section. However as per WSM combined interaction is considered for buckling @ yy axis only . So reserve strength in a section by LSM is more than WSM.

From Section B "Project Problem"

In Project problem the interaction ratios (IR = Action / Strength) are designed by both codes .As per IS 800 (2007) the IR's are found more closer than the IR's are found from IS 800 (1984).From load combination sections are designed by both codes (Table C.1.1) and it is found that in most of the cases sections designed by LSM is more economical than the sections designed by WSM.

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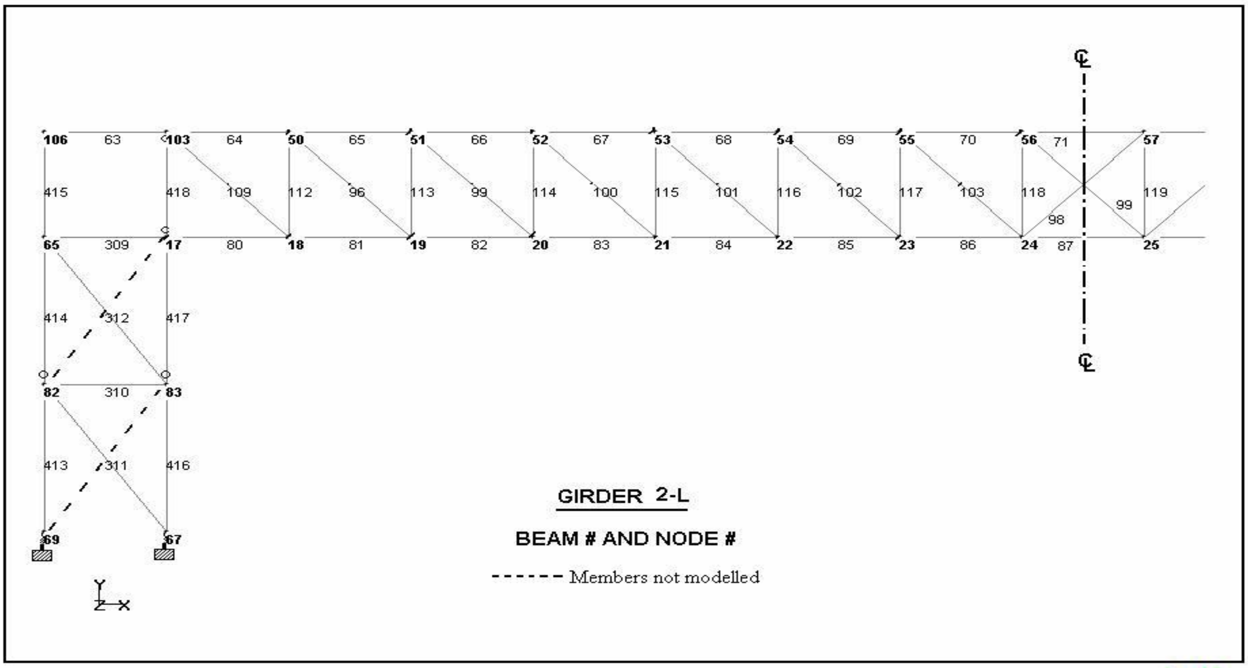
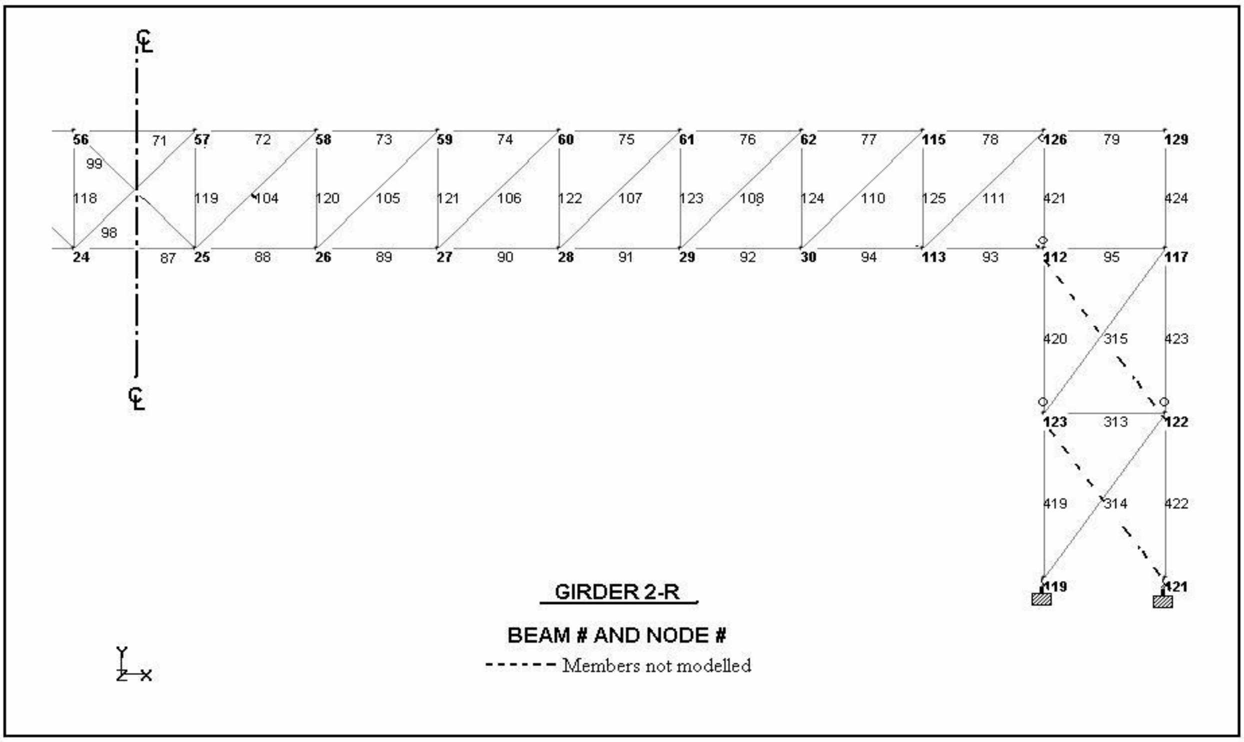
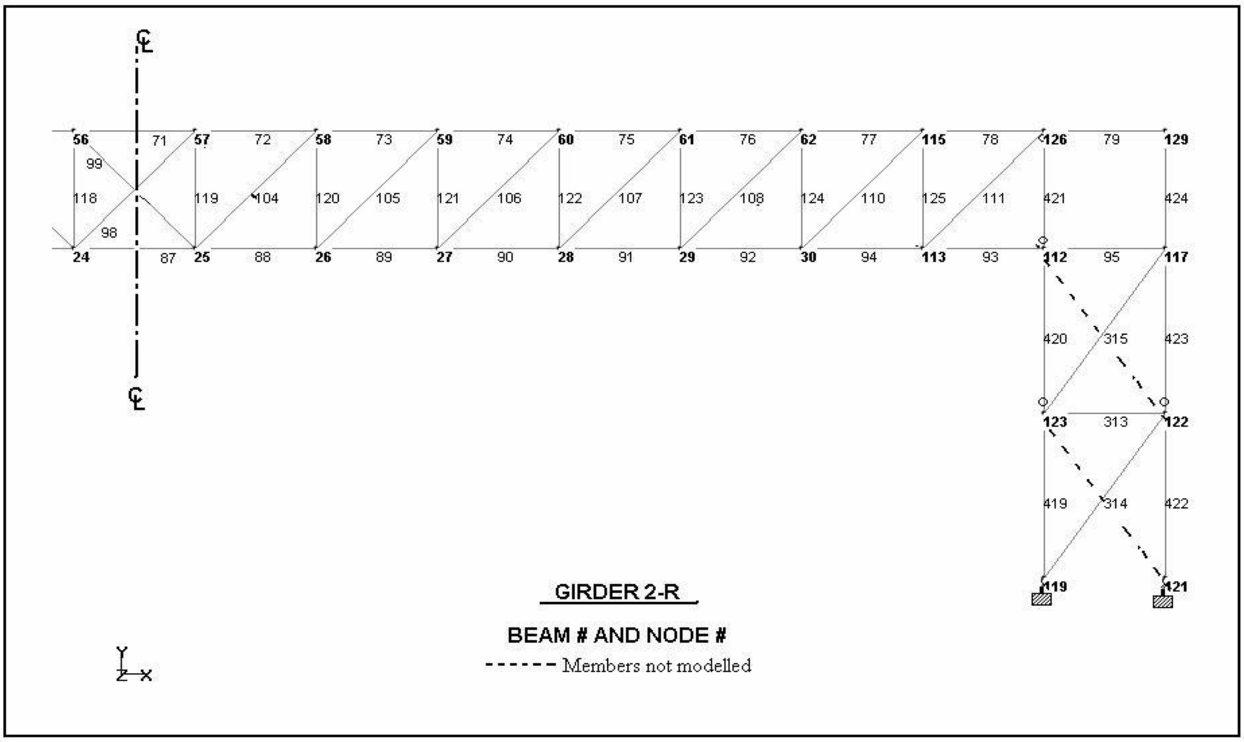


GIRDER 2-L
BEAM # AND NODE #
Members not modelled



56 71 57 72 58 73 59 74 60 75 61 76 62 77 115 78 126 79 129
99
118 119 104 120 105 121 106 122 107 123 108 124 110 125 111 421 424
98
24 87 25 88 26 89 27 90 28 91 29 92 30 94 113 93 112 95 117
420 315 423
123 313 122
419 314 422
119 121
GIRDER 2-R
BEAM # AND NODE #
Members not modelled
Y
Z X



56 71 57 72 58 73 59 74 60 75 61 76 62 77 115 78 126 79 129
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118 119 104 120 105 121 106 122 107 123 108 124 110 125 111 421 424
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24 87 25 88 26 89 27 90 28 91 29 92 30 94 113 93 112 95 117
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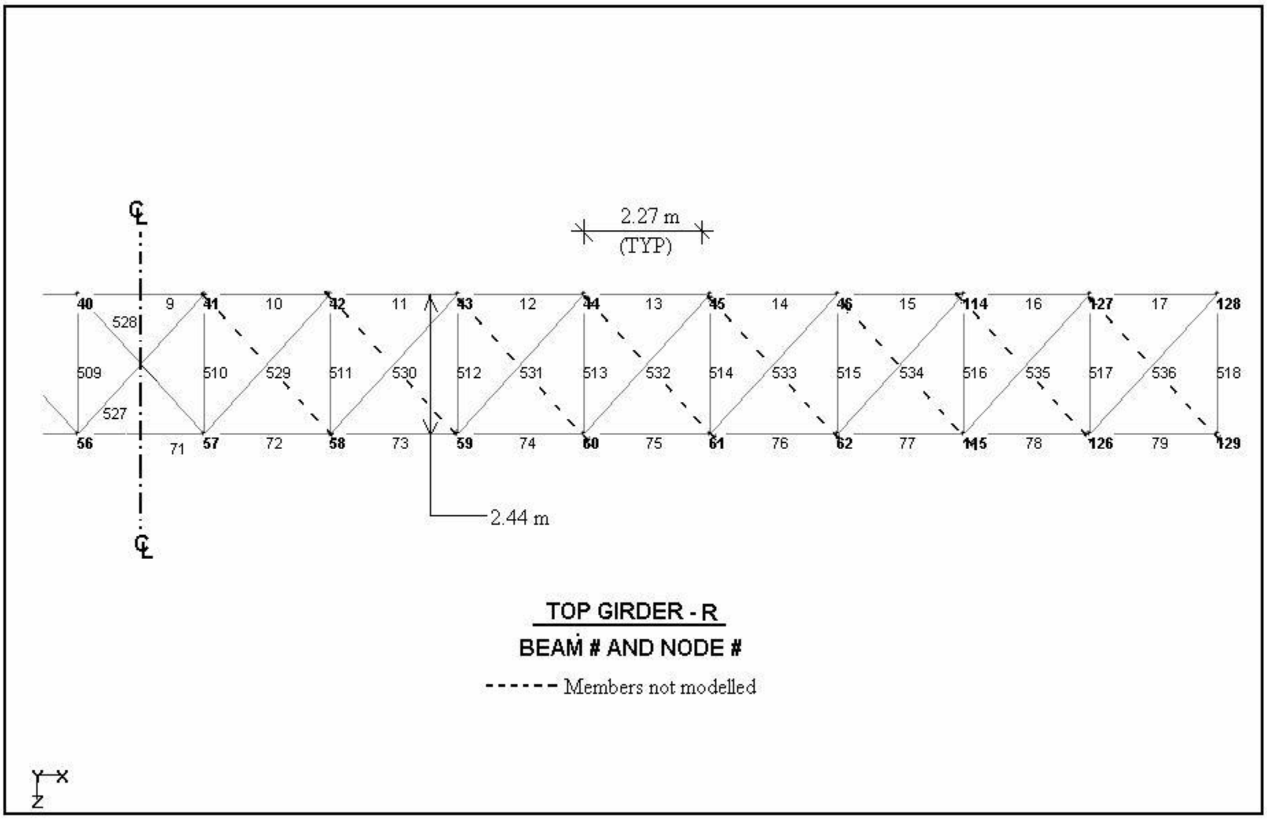
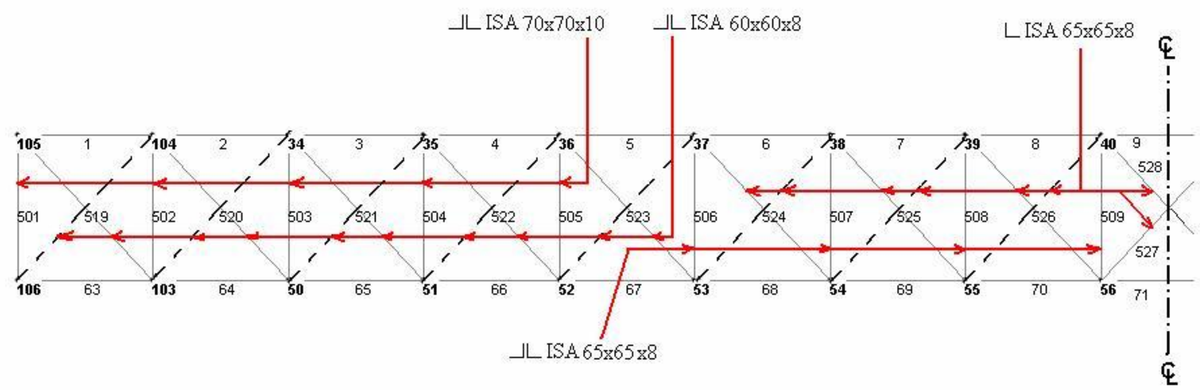
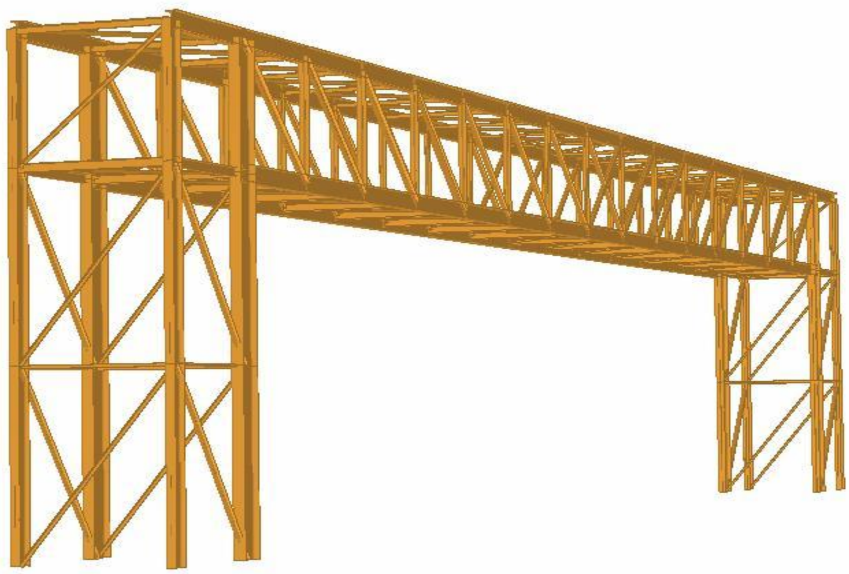
123 313 122
419 314 422
119 121
GIRDER 2-R
BEAM # AND NODE #
Members not modelled
Y
Z X


2.27 m
(TYP)
2.44 m
TOP GIRDER - R
BEAM # AND NODE #
Members not modelled

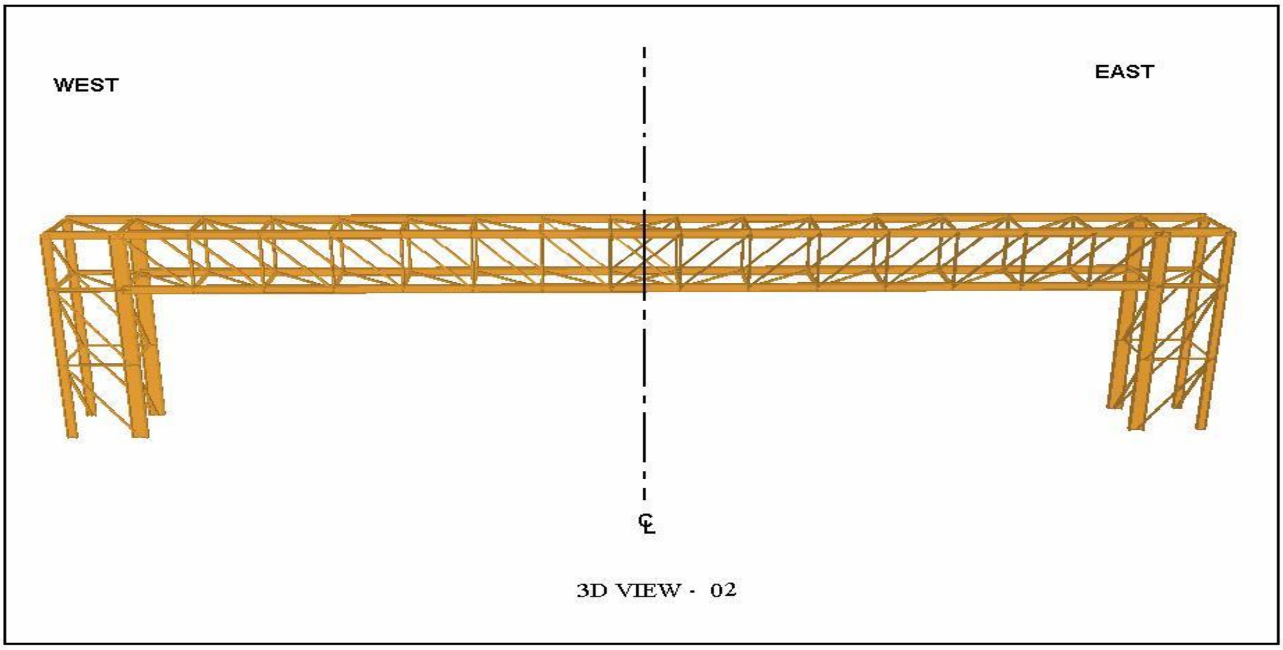


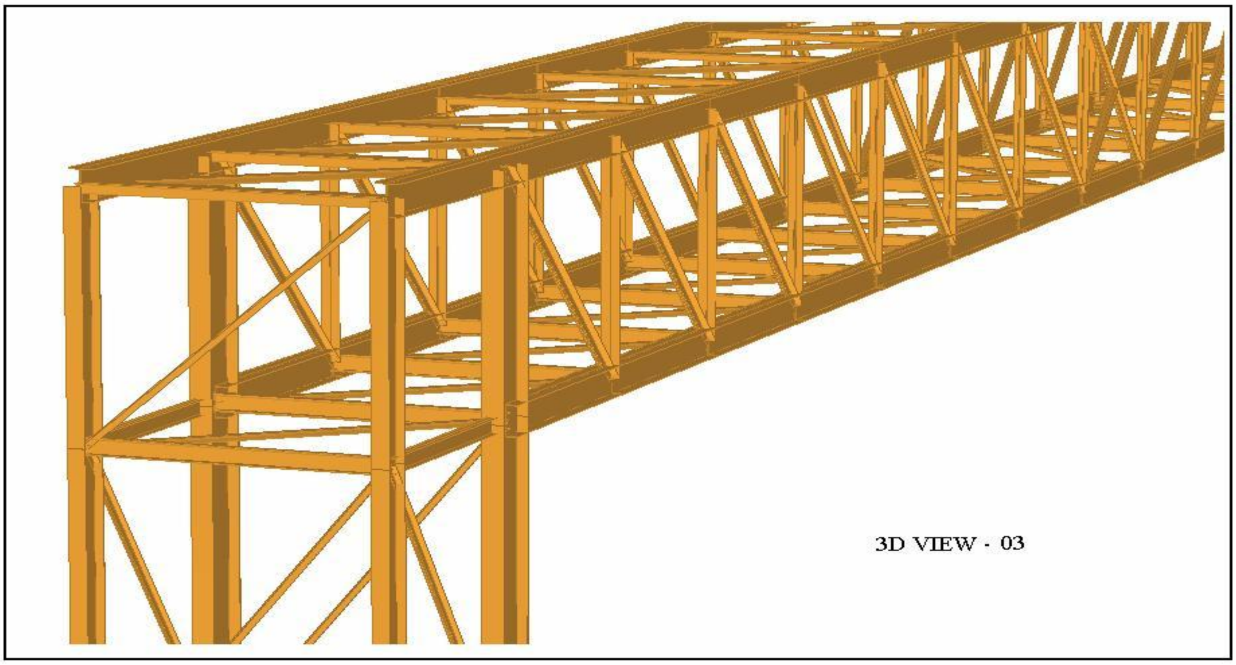
TOP GIRDER - L

BEAM # AND NODE #



3D VIEW -01


WEST
EAST
℄
3D VIEW - 02


3D VIEW - 03