Design of Flexural Members (IS 800:2007)
&
Comparison with Other Codes and Design Aids

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Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

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Guide
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CERTIFICATE

This is to certify that the Major Project entitled "Design of Flexural Members (IS 800:2007) & Comparison with Other Codes and Design Aids" submitted by Mr. Hasmukh D. Panchani (07MCL020), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Limit state design method has been developed in the early 1970. Till then steel structure design has been carried out by the Working Stress Method (WSM).This new design approach adopted by many countries has been proved to be technologically sound and resulted in significant economy in the completed structures. In India because of globalization in engineering practice, practicing engineers required to adopt new method of design. Realizing the advantages of Limit State Method (LSM), the Bureau of Indian Standards, IIT Madras and Institute for Steel Design and Growth (INSDAG) Kolkata, together have introduced a draft of IS:800 (LSM Version).

Limit State Method is a totally new method in India. Mainly designers are familiar with the philosophy of LSM as it is being used in the design of concrete structures but LSM in the design of steel structures considers completely different design considerations. Therefore attempt is made to illustrate some problems using LSM, which helps to understand the new method of design and design aids is prepared which will be proved helpful to structural engineers and will lead to quick implementation of the IS 800:2007.

Few problems of simply supported laterally restrained and unrestrained beam have been illustrated by preparing a design table of load carrying capacity for different span. In design of gantry girder, spread sheets and design table are prepared for different load capacity with varying span for single crane and double crane.

Plate girders are built-up beams comprising of plate sections for web and flanges when welded connections are used. Present study includes theory behind pre buckling and post buckling behaviour of slender web when it is transversely stiffened and use of different types of stiffeners. Example of plate girder is illustrated.

Comparison of IS 800:2007 with other codes like AISC 360-05, BS-5950, and Eurocode-3 is also included.

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ABBREVITION NOTATION AND NOMENCLATURE

A_{gf} Gross area of the flange in tension

A_{nf} Net area of the flange in tension

A_q Area of the stiffener in contact with the flange

a Spacing of transverse stiffeners

b_1 Stiff bearing length

b_f, t_f Width and thickness of the relevant flange, respectively

b_s Outstand width of the stiffener (in mm)

c Extreme fibre distance form neutral axis

c_s Spacing of transverse stiffeners

d_2 Twice the clear distance from the compression flange angles, plates or tongue plates to the neutral axis

d_w Depth of web

E Modulus of elasticity

F_{qd} Design resistance of intermediate web stiffener corresponding to buckling about axis parallel to the web

F_x External load or reaction at the stiffener

F_{xd} Design resistance of a load carrying stiffener

f_1 Extreme fibre stress

f_{bd} Design bending compressive stress

f_f Means longitudinal stress in the smaller flange due to moment and/or axial force

f_u Ultimate stress of material

F_v Shear force in a member

f_y Yield stress

f_{yf} Yield stress of the flange

f_{yw} Yield stress of the web

f_{yq} Yield stress of the stiffener

I_t Torsional constant

I_s Second moment of inertia

I_w Warping constant

L_{LT} Effective length for lateral torsional buckling

M Factored design moment

Mc Moment of resistance

M_{cr} Elastic critical moment

$M_{c,Rd}$ Design bending strength of section

M_{bx} Buckling moment resistance

M_{cx} Moment resistance at major axis

M_{d} Design bending strength of section

M_{p} Plastic moment

M_{pf} Plastic moment capacity of the smaller flange, about its own equal area axis perpendicular to the plane of the web

M_{pw} Plastic moment capacity of the web, about its own equal area axis perpendicular to the plane of the web

M_{q} Moment on the stiffener due to eccentrically applied load

M_{tf} Moment in end panel

M_{yq} Yield moment capacity of the stiffener based on its elastic modulus about an axis parallel to the web

n_{1} Dispersion of the load through the web at 45°, to the level of half the depth of the cross section

p_{b} Bending strength (lateral-torsional buckling)

P_{v} Shear capacity

P_{y} Design strength of steel

p_{yf} Design strength of the flange

p_{yw} Design strength of the web

R_{tf} Resultant longitudinal shear in end panel

r_{y} Radius of gyration

S_{x} Plastic modulus about the major axis

S_{y} Plastic modulus about the minor axis

s_{c}, s_{t} Anchorage lengths of tension field along the compression and Tension flange respectively

T_{cf} Maximum thickness of compression flange of the span under consideration

t_q, t_s Thickness of the stiffener

t_w Thickness of web

V , V_{sd} Factored design shear force

V_d Design shear strength of section

V_f flange-dependent shear buckling resistance

V_n Nominal shear strength

$V_{pl,Rd}$ Shear resistance

v Slenderness factor for a beam

W_{el} Elastic sectional modulus

$W_{pl,y}$ Plastic modulus about the major axis

w_{tf} The width of the tension field

Z_c Compression modulus of a section in bending

Z_e Elastic sectional modulus

Z_p Plastic sectional modulus

α_{LT} Imperfection parameter

χ_{LT} Bending stress reduction factor to account for lateral torsional buckling

ϕ Inclination of the tension field

γ_{m0} Partial safety factor

γ_{m1} Partial safety factor for ultimate stress

γ_{fft} Partial safety factors for load

γ_{mft} Partial safety factors for strength

ε Yield stress ratio

ε_y Yield strain

λ_{LT} Non-dimensional slenderness ratio

λ_w Non-dimensional web slenderness ratio for shear buckling stress

μ Poisson's ratio

$\tau_{cr,e}$ Critical shear stress

ρ Radius of curvature

1. INTRODUCTION

1.1 BACKGROUND

Limit state Design, an improved design philosophy was developed in the late *1970*'s and has been extensively incorporated in design standards and codes formulated in all the developed countries. Although there are many variations between practices adopted in different countries the basic idea is broadly similar. The probability of operating conditions not reaching failure condition forms the basis of Limit States Design adopted in all countries.

Table 1.1 Design formats or codes for various countries

Sr.No.	Countries	Design Formats or Codes
[A]	i. Australia ii. Canada iii. China iv. Europe v. U.K. vi. Japan	Limit State Method (LSM)
[B]	USA	Load and Resistance Factor Design (LRFD)
[C]	India	Allowable Stress Design (ASD) , Now LSM

Due to globalization, engineering practice has not remained limited for a particular area therefore practicing engineers is facing problems with existing code. Realizing these difficulties the Bureau of Indian Standards, New Delhi requested faculty members of Civil Engineering, Indian Institute of Technology Madras to help and prepare draft for the revision of IS:800. This work was carried out in a project mode with the financial support from the Institute for Steel Development and Growth (INSDAG), Calcutta.

In India research and development in steel has done up to certain extent, so IS 800:2007 is based on the international experience. This new code is a improvement over the previous code (IS: 800-1984), with new provisions on partial safety factor based limit state method of design, design against fatigue, design for fire load, design for durability, design by testing etc.

It includes parameters like fatigue, ultimate strength, member end connections, restraints etc. having greater influence on the design considerations which makes IS 800:2007 more complicated and time consuming for new users.

Hence providing design aids to facilitate steel design using IS 800:2007 would prove to be very useful and will lead to quick implementation of the IS 800:2007.

1.2 DESIGN PHILOSOPHY

The earlier design philosophy is the working stress method of design (WSM). It is based on linear elastic theory and still existing in India, USA and some other countries, although it has now been replaced by modern limit states design philosophy. The working stress method of design (WSM) was developed in the 1950. It was based on the ultimate strength of steel at ultimate loads. This method was introduced as an alternative to WSM in IS 800:1962.

Probabilistic concepts of design were developed in 1960. The philosophy was based on the theory that the various uncertainties in design could be handled more rationally in the mathematical framework of probability theory. The risk involved in design was quantified in terms of a probability of failure which is known as reliability based method. It was not accepted in professional practice, mainly because it appeared to be complicated.

The probabilistic 'reliability method' approach was simplified and reduced to a deterministic format involving partial safety factors rather than probability of failure. The philosophy of the limit state method was introduced in the British Code CP 110(1972) (now BS 8110), and the Indian concrete code IS 456:1978. Limit states design was first adopted for steel structures in the Canadian code in 1974, which was followed by the British codes BS 5950 and BS 5400. In USA, the American Institute of Steel Construction introduced the LSM in the form of load resistant factor design (LRFD) in 1993.

1.3 SCOPE OF WORK

- Literature review
- Comparison of different codal provisions for flexure member (including plate girder and gantry girder) and general design consideration for Indian, British and American standards.

- To study of different aspects of flexure member like bending, shear, web buckling, web crippling, deflection.
- Detailed design procedure for flexure members (Beam, Gantry girder, Plate girder) as per IS 800:2007.
- Calculating load carrying capacity for different sizes of sections with different span and simply supported condition.
- Parametric study of WB section compared to MB section for moment of resistance and weight per unit length for different span.
- Preparation of design aids for flexure member (Beam, Gantry girder, Plate girder).

1.4 ORGANIZATION OF MAJOR PROJECT

The contents of major project are divided in various chapters as follows;

Chapter 1 presents the introduction of major project work which includes background information of limit state design, necessary to revise IS 800:1984. It includes design philosophy and scope of work. The need of design aids for IS 800:2007 is also explained in this chapter.

Literature review is discussed in Chapter 2. In this chapter literature review related to design of laterally restrained and unrestrained beam, design of gantry girder, comparison of different code with IS 800:1984, additional sectional property for Indian standard tapered flange sections are included.

Chapter 3 deals with design of laterally supported and unsupported beam. It includes theory related to behavior of steel beam, web buckling, web crippling and lateral stability of beam. Illustration of laterally supported and unsupported beam is given. Design table and comparison chart of load carrying capacity of sections having same depth is included. It also includes comparison table of IS 800:2007 with other codes like AISC 360-05, BS-5950 and EUROCODE-3.

Chapter 4 gives brief introduction of components of crane system. The various considerations due to the moving loads have been discussed. A brief discussion on fatigue effects and various factors that may affect the choice of girders have been included. It also includes comparison table of IS 800:2007 with other codes like BS-5950 and EUROCODE-3.

Design of plate girder is discussed in Chapter 5, includes shear resistance of slender web when it is transversely stiffened. Shear buckling design by simple post-critical method and tension field method and the brief introduction of various types of stiffener. Illustration of plate girder in the form of design steps and code comparison for design of plate girder is also included.

Finally chapter 6 deals with summary of the work done, conclusions and future scope of work.

2. LITERATURE REVIEW

2.1 GENERAL

Literature survey has been carried out for steel design by different design philosophies with different configurations to know about the difference in existing code and new code which is based on limit state design .The prime importance in the review was given to understand the different design concepts used in steel design.

2.2 LITERATURE REVIEW

Various literatures have been studied for the steel design and brief review of which has been discussed below.

Bandyopadhyay et al. [1] discussed in his paper an over view of IS:800 and compared with other countries 's codal provision for steel structures, introduction of the necessary of revision of existing IS:800 and brief introduction of the working stress method and limit state method, compared codal provision of IS:800 (1984), IS:11384 (1985), IS:807 with BS:5950, some area where guide lines are not available in IS code like usage of castellated section, guidelines for provision of splices, method of measurement for span, vibration ect.

Seetharaman et al. [2] presented a paper on comparison of wide flanged section and the existing rolled section which gives a brief introduction of working stress design, plastic design and limit state design method, in India mainly ISMB sections are manufactured. These sections are not economical for compression members due to high slenderness ratio. They discussed the advantage of wider flange section compared to the conventional beam and also discussed compression member design as per draft IS:800 and provision of effective length and eccentricity for column. Detailed comparative study carried out for compressive strength of wide flange section and conventional flange section by varying length and yield strength of section, the weight per meter of both sections differ by more than ±5% to ±10%.

Bandyopadhyay and Guha [3] presented a paper on structural member design based on draft IS:800. It shows procedure for the design of flexure member as per draft code and comparison with existing code and they show that design of tension member, flexure member are economical by draft code and design of compression member is economical by existing code.

Mary and Shoba [4] presented a paper on LRFD (Load Resisting Factored Method) format for the design of steel structures for Indian conditions. He has solved steel frame example and compared with working stress method and shows percentage reduction in weight of beam and column by LRFD with same loading.

Nataraja [5] discussed behaviour of laterally supported and laterally unsupported gantry girder and the design of doubly symmetric I-gantry girder with draft code and existing code.

Mary [6] has presented a paper on Analysis of semi rigid frame as per draft IS:800. It gives an outline about the new limit state concept proposed in the draft version of IS:800.

Gupta [7] has published a paper on Additional sectional properties of Indian standard parallel flange section where compared conventional tapered flange section and parallel flange section. It shows parallel flange sections are more efficient than the conventional tapered flange section in terms of strength, workability and economy and it is suggested that the additional properties such as the depth between root fillets, local buckling ratio for web, warping constant and torsional constant be provided in IS:12778-2004.

Rao [8] discussed in his paper Economy is the hallmark of parallel flange sections that hot rolled sections having parallel flange are available up to 700 mm depth in India and the use of parallel flange sections results in overall saving in material and cost of steel structure. ISMB section can also be more efficient, almost similar to parallel flange section, by making necessary changes in their dimension.

Gupta [9] has presented a paper on Additional sectional properties of Indian standard tapered flange sections. It shows the additional sectional properties like depth between root fillets, local buckling ratio for web, warping constant and torsional constant for Indian standard medium weight beam and channel section with tapered flanges.

N.Subramanian [10] has described the procedure involved in designing structural components like tension member, compression member, member subjected to flexure like gantry girder and plate girder. Typical problems have been solved using limit state design method as per IS: 800-2007.

3. DESIGN OF BEAM

3.1 INTRODUCTION

Beams are structural members frequently used to carry loads that are transverse to their longitudinal axis. They transfer loads primarily by bending and shear. For a beam (loaded predominantly by flexure) two essential requirements must be met to develop its full moment capacity:

1. The element of the beam should not buckle locally.
2. The beam as whole should not buckle laterally.

To satisfy the first condition the cross section of the flange and the web chosen must be plastic or compact (*Ref.3.3*) and for second condition to avoid lateral buckling, restraints are provided to the beam in the plane of compression flange which is called as laterally restrained beams. In steel structure mainly beams are carrying deck floor which provides lateral support to the beam. In the absences of any such restraint designer has to provide adequate lateral support to the compression flange of beam.

3.2 BEHAVIOUR OF STEEL BEAMS

Laterally stable beam can fail only by (a) flexure (b) shear or (c) bearing with assumption that local buckling of slender components do not occur. These three conditions are the criteria for limit state of collapse for steel beams. Steel beams may also become unserviceable due to excessive deflection and this is considered as the limit state of serviceability.

3.2.1 Steel Beams in Flexure

If a flexural member is progressively loaded, it deflects and the curvature of such bending varies along its length. Initially the beam is elastic throughout its length. Let us consider a small portion of the beam at a point A as shown in Fig.3.1 (a) where the curvature is ρ. If we consider a small segment of the beam at A as shown in Fig.3.1 (b), then the variation of the strain across the depth of the member could be found out geometrically as

$$\varepsilon = \frac{z}{\rho} \qquad \dots (3.1)$$

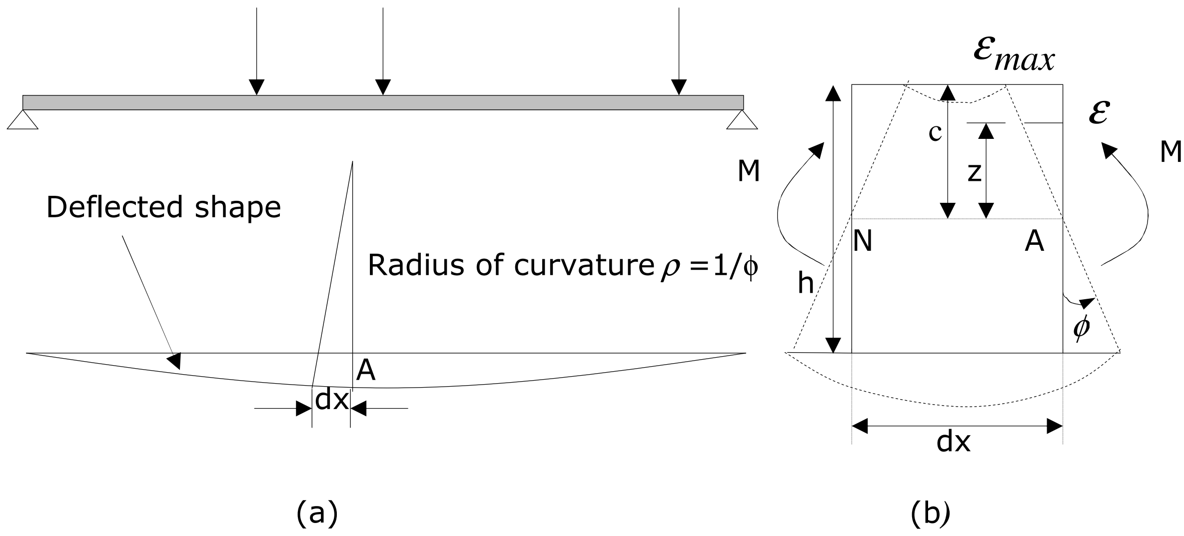


Fig.3.1 Bending Curvature

From Equation (3.1), the strain at any fiber is proportional to its distance z from the neutral axis. This is obtained from the assumption that plane sections which are normal to the longitudinal axis before bending, remains plane and normal even after bending. For each strain 'ε' one can get corresponding stress 'f' from the idealized stress-strain curve for steel shown in Fig.3.2

3.2.2 Elastic Flexural Behaviour

Consider the point (1) in Fig.3.2 in which the strain ε_1 is less than ε_y, at this stress is directly proportional to the strain. It can be shown that the moment of resistance is given by

$$M_1 = \frac{f_1 I}{c} \qquad \text{... (3.2)}$$

The term Z = I/c is the elastic section modulus which is a geometric property of the section. Hence Equation (3.2) can be written as

$$M_1 = f_1 Z \qquad \text{... (3.3)}$$

3.2.3 Yielding and Plastic Behaviour

Consider the point (2) in Fig.3.2 in which the extreme fibre strain reached yield value ε_y and also the stress $f_2 = f_y$. The stress and strain are proportional up to this point as the extreme fibre is within the elastic range. The moment corresponding to this point is the first yield moment which causes the extreme fibres to yield

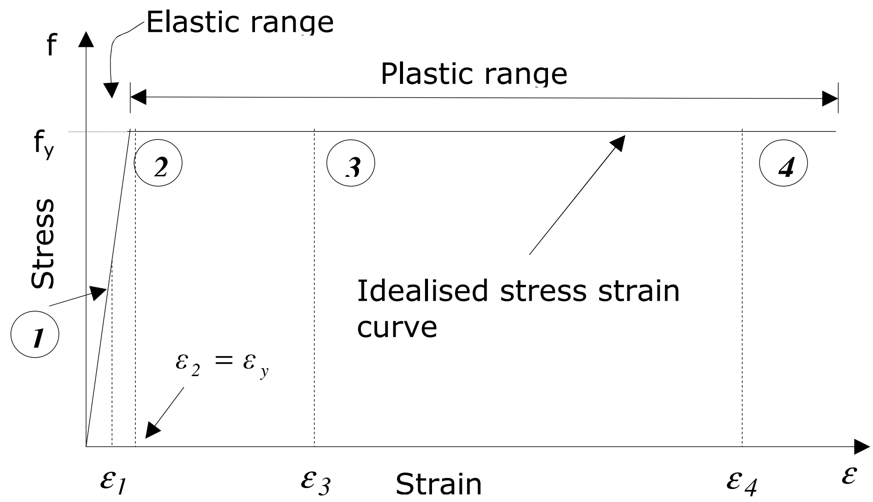


Fig.3.2 Idealized elasto- plastic stress- strain curve for steel

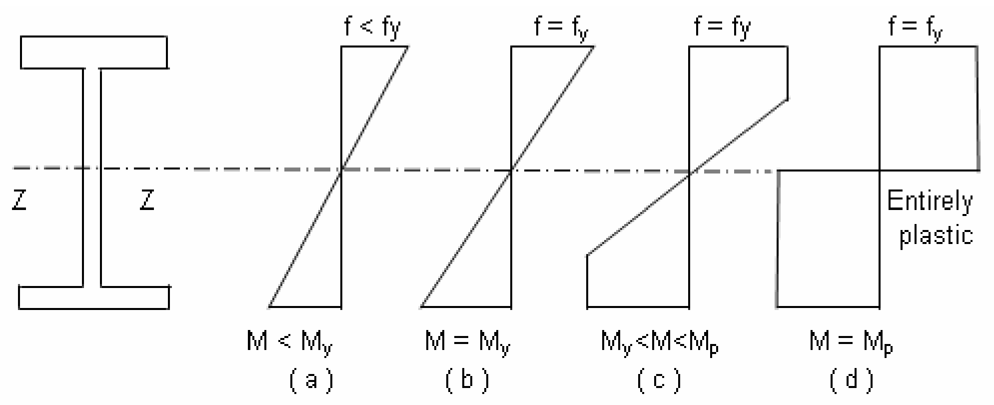


Fig.3.3 stress distribution at different stages of loading

$$M_2 = M_y = f_y Z \qquad \text{... (3.4)}$$

Fig.3.3 shows that the extreme fibres after attaining yield stress do not take any more stresses. When loading is further increased the outermost fibre strain ε_{max} near mid span of the beam (i.e. point of maximum bending moment) would attain a value say, $\varepsilon_3 > \varepsilon_y$ and this is identified as point (3) in Fig. 3.2 and moment at this point is $M_3 < M_p$. At this stage the strain is in the plastic stage, and extreme fibre stress still equals yield stress f_y. It also shows that the stresses have been redistributed to the inner fibres towards the neutral axis and these fibres gradually attain a stress equal to f_y. This is shown in Fig.3.3 and cross-section become plastic, moment at this stage is $M_4 = M_p$.

3.3 SECTION CLASSIFICATION

The plate element of a cross section may buckle locally due to compressive stresses. The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross-section subjected to compression due to axial force. When a thin walled beam subjected to bending as shown in Fig.3.3, extreme fibers in the maximum moment region reach the yield stress as shown in Fig.3.3 (a). As further loading, more and more fibers reach the yield stress and the stress distribution is as shown in Fig.3.3 (c). The bending moment that causes the whole cross section to reach yield stress as shown in Fig.3.3 (d). It is known as the plastic moment of the cross section M_p.
The cross section is incapable of resisting any additional moment, but may maintain the plastic moment which is acting as plastic hinge for some more amount of rotation. If the section is slender, it may fail by local buckling even before reaching the yield stress. Four classes of section have been identified

a. Plastic (class 1): Plastic cross-sections are those which can develop their full plastic moment M_p and allow sufficient rotation at or above this moment so that redistribution of bending moments can take place in the structure until complete failure mechanism is formed. ($b/t_f < 9.4\varepsilon$ and $d/t_w < 84\varepsilon$)
b. Compact (class 2): Compact cross-sections are those which can develop their full-plastic moment M_p but where the local buckling prevents the required rotation at this moment to take place. ($b/t_f < 10.5\varepsilon$ and $d/t_w < 105\varepsilon$)
c. Semi-compact (class 3): Semi-compact cross-sections are those in which the stress in the extreme fibers should be limited to yield stress because local buckling would prevent the development of the full-plastic moment M_p. Such sections can develop only yield moment M_y .($b/t_f < 15.7\varepsilon$ and $d/t_w < 126\varepsilon$)
d. Slender (class 4): Slender cross-sections are those in which yield in the extreme fibers cannot be attained because of premature local buckling in the elastic range.

3.4 DESIGN STRENGTH OF LATREALLY SUPPORTED BEAMS

For laterally supported beams the factored design moment, M at any section should satisfy

$$M < M_d \quad \text{... (3.5)}$$

Where, M_d = Design bending strength of section

This relationship is obtained with the assumption that the beam web is stocky. When the flanges are plastic, compact, or semi-compact and the web is slender i.e. $d/t_w > 67\varepsilon$, the design bending strength may be calculated using one of the following method.

- The flanges resist the bending moment and the axial force acting on the section and web resist only the shear.
- The whole section resists the bending moment and the axial force and therefore the web has to be designed for combined shear and its share of normal stresses.

Shear force does not have any influence on the bending moment for values of shear up to $0.6V_d$, the design bending strength M_d

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \le 1.2 Z_e f_y / \gamma_{m0} \quad \text{... (3.6)}$$

Where,

β_b = 1.0 for plastic and compact sections

$\beta_b = Z_e / Z_p$ For semi-compact section

Z_p = plastic section modulus

Z_e = elastic section modulus

f_y = yield stress of material

γ_{m0} = partial safety factor

The additional check $M_d < 1.2 Z_e f_y / \gamma_{m0}$ is provided to prevent the onset of plasticity under unfactored dead, imposed and wind loads. For most of the I-beams and channels in IS 808, Z_p / Z_e is less than 1.2 and hence the plastic moment capacity governs the design. For section where $Z_p / Z_e > 1.2$ then the constant 1.2 may be replaced by the ratio of factored load/ unfactored load.

If the design value of the shear force is greater than 60% of the plastic design resistance in shear, a member subjected to co-existing bending and shear has to use its web to resist the shear force as well as to assist flanges in resisting moment. Therefore, when cross section subjected to bending and high shear has a reduced moment resistance in the presence of high shear. The interaction diagram between moment and shear is shown in Fig.3.4.

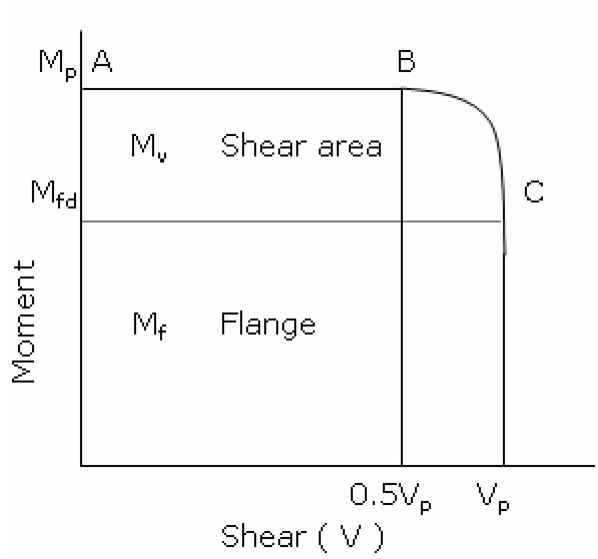


Fig.3.4 Interaction between moment and shear

When the design force V exceeds 0.6V_d, where V_d is the design shear strength of the cross section, the design bending strength M_d will be taken as

$$M_d = M_{dv} \quad \text{... (3.7)}$$

Where,

M_{dv} = design bending strength under high shear

For plastic and semi-compact section, M_{dv} calculated as follows:

(a) Plastic or Compact section

In Fig.3.4, as the shear force V is increased from zero to 0.5V_p, there will be no reduction in plastic moment M_p

$$V_p = A_v f_v \quad \text{... (3.8)}$$

Where,

V_p = shear strength of web

A_v = $D t_w$ (rolled section)

= $d t_w$ (built up section)

f_v = 0.6 f_y

f_v is slightly greater than the true von mises value of $f_y / \sqrt{3}$.When the full capacity in shear V_p is reached, the shear area is assumed to be completely ineffective in resisting the moment and hence the reduced plastic moment M_{dv} becomes M_{fd}.

$M_{fd} = M_p - \left(D^2 t_w / 4\right) f_y$ (rolled section) ... (3.9)

$M_{fd} = M_p - \left(d^2 t_w / 4\right) f_y$ (built up section) ... (3.10)

Between V = 0.5V_p andV_p, M_{dv} is assumed to reduce parabolically according to the curve BC given Fig.3.4.

$M_{dv} = M_p - \beta \left(M_p - M_{fd}\right) \leq 1.2 Z_e f_y / \gamma_{m0}$ ... (3.11)

Where, $\beta = \left(2V / V_p - 1\right)^2$

M_p = design plastic moment of section excluding high shear force effect

V_p = Design shear strength as governed by web yielding

= 0.6 $D t_w f_y$

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

γ_{m0} = partial safety factor

(b) Semi-Compact section

$M_{dv} = Z_e f_y / \gamma_{m0}$ ... (3.12)

3.5 DESIGN STRENGTH OF LATREALLY UNSUPPORTED BEAMS

Beam having bending about major axis and when its compression flange is not restrained against lateral buckling, may fail by lateral torsional buckling before attaining its bending strength. The effect of lateral torsional buckling on flexural strength need not be considered when $\lambda_{LT} \leq$0.4. The design bending strength of laterally unsupported beam is given by

$M_d = \beta\ Z_p f_{bd}$ ... (3.13)

λ_{LT} = non-dimensional slenderness ratio

The design bending compressive stress is given by

$f_{bd} = \chi_{LT}\ f_y / \gamma_{m0}$ ... (3.14)

where,

χ_{LT} = bending stress reduction factor to account for lateral torsional buckling

$$\chi_{LT} = \frac{1}{\left[\phi_{LT} + \left(\phi_{LT}^{2} - \lambda_{LT}^{2}\right)^{0.5}\right]} \le 1.0 \quad \text{... (3.15)}$$

where,

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}^{2}\right]$$

The value of imperfection factor α_{LT} for lateral torsional buckling of beams is given by

α_{LT} = 0.21 for rolled steel section

α_{LT} = 0.49 for welded steel section

γ_{m0} = partial safety factor for material = 1.10

The non-dimensional slenderness ratio λ_{LT} is given by

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \le \sqrt{1.2 Z_e f_y / M_{cr}} \quad \text{... (3.16)}$$

$$= \sqrt{\frac{f_y}{f_{cr,b}}}$$

where,

M_{cr} = elastic critical moment

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

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 E I_y}{(L_{LT})^2}\right)\left[G I_t + \frac{\pi^2 E I_w}{(L_{LT})^2}\right]\right\}} \quad \text{... (3.17)}$$

where,

I_t = torsional constant

$= \sum b_i t^3{}_i / 3$

I_w = warping constant

$= I_y h^2 / 4$

I_y = moment of inertia about the weak axis

L_{LT} = effective laterally unsupported length of the member

Equation (3.14) and (3.17) has been adopted from Euro code 3.

3.6 SHEAR STRENGTH OF BEAMS

Shear force generally exists with bending moments, the maximum shear stress in a beam is to be compared with the shear yield stress. Shear force may control in cases where the beams are short and carry heavy concentrated loads. The pattern of shear stress distribution in I-section is shown in Fig.3.5.

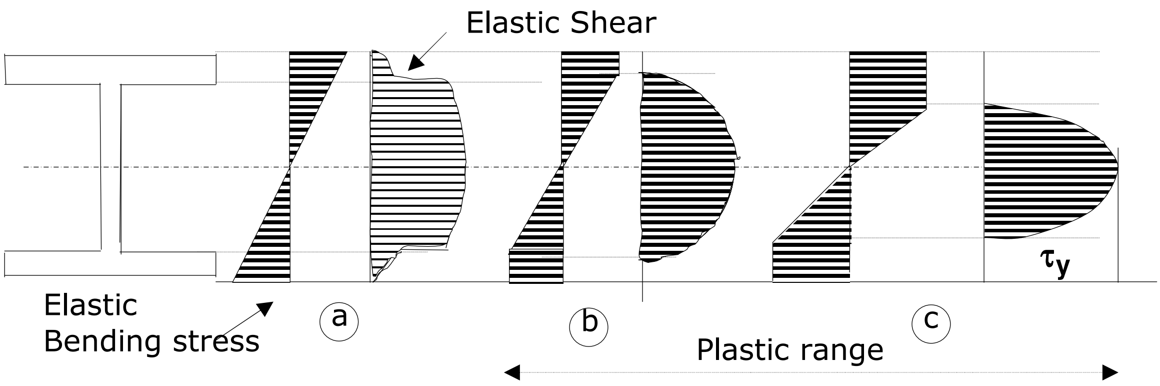


Fig.3.5 combined bending and shear in beams

Fig.3.5. shows that a significant proportion of shear force carried by the web and in elastic region the shear stress distribution over the web area is nearly uniform. Hence, the average shear stress for most commonly adopted sections (such as I, channel, T etc.) which is given by

$$\tau_{av} = V / t_w d_w \qquad \text{... (3.18)}$$

where,

t_w = thickness of the web

d_w = depth of web

The nominal shear yielding strength of webs is based on the Von Mises yield criterion, which states that for an unreinforced wed of a beam whose width to thickness ratio is comparatively small the shear strength may taken as

$$\tau_y = f_y / \sqrt{3} \; = \; 0.58 f_y \qquad \text{... (3.19)}$$

where, f_y = yield stress

The factored design shear force V in a beam due to external actions should satisfy

$$V \le V_d \qquad \text{... (3.20)}$$

where, V_d = design strength

$$V_d = V_n / \gamma_{m0} \qquad \text{... (3.21)}$$

The nominal shear strength of a cross section V_n may be governed by plastic shear resistance or the strength of the web governed by shear buckling. The nominal plastic shear resistance under pure shear is given by $V_n = V_p$

$$V_p = A_v f_{yw} / \sqrt{3} \qquad \text{... (3.22)}$$

where,

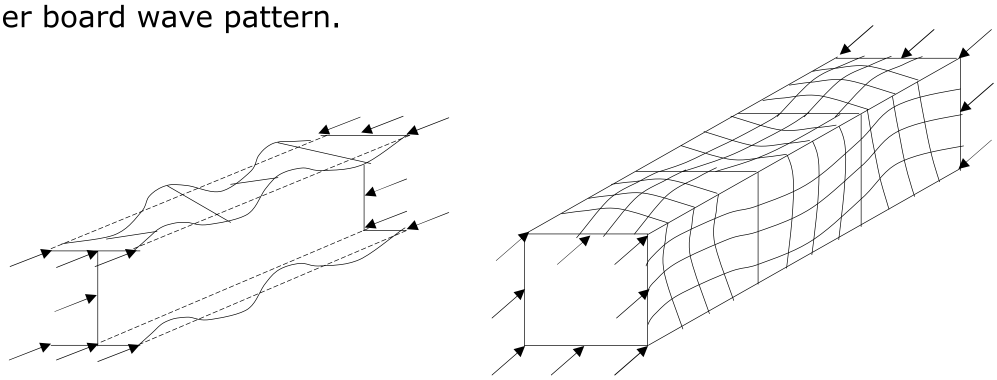
A_v = shear area

f_{yw} = yield strength of the web

3.7 LOCAL BUCKLING

Sections normally used in steel structure are closed and opened section. These sections made up of combination of individual plate elements connected together to form the required shape. The strength of compression member depends on the slenderness ratio, sections having less slenderness ratio will have higher strength. When the plate element of section is subjected to compression or shear it is going to buckle before the overall beam or column failure by lateral buckling or yielding. This phenomenon is called local buckling.

Consider an I section column which is subjected to uniform compression, as shown in Fig.3.6 (a) .Flanges are buckling before the web which are supported along two sides in the form of waves. In Fig.3.6 (b) closed section shown where local buckling of both web plates and flange plate take place in the form of chequer board wave pattern.



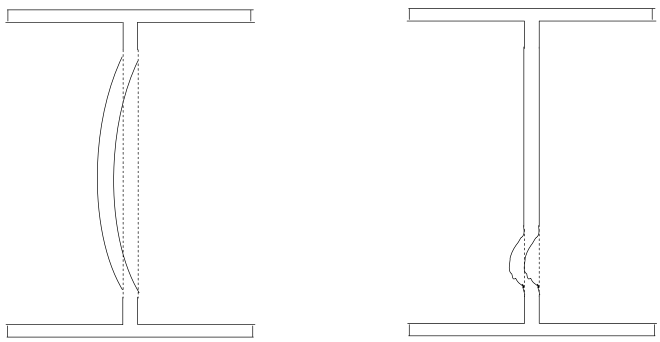
(a) (b)

Fig.3.6 local buckling of Compression Members

In the case of beams, web portion behave as plate element subjected compression so it undergoes local buckling at the corresponding critical buckling stress. Local buckling will occur where the beam is having the maximum bending moment.

3.8 WEB BUCKLING AND WEB CRIPPLING

The application of heavy concentrated loads produces a region of high compressive stresses in the web either at the support or under the load. This may cause either the web to buckle as shown in the Fig.3.7 (a) or the web to cripple as shown in Fig.3.7 (b). In the case of web buckling, the web may be considered as a strut restrained by the beam flanges. Such *idealised struts* should be considered at the points of application of concentrated load or reactions at the supports as shown in Fig.3.8 and Fig.3.9.



(a). Web buckling (b). Web crippling

Fig.3.7 local buckling of the web

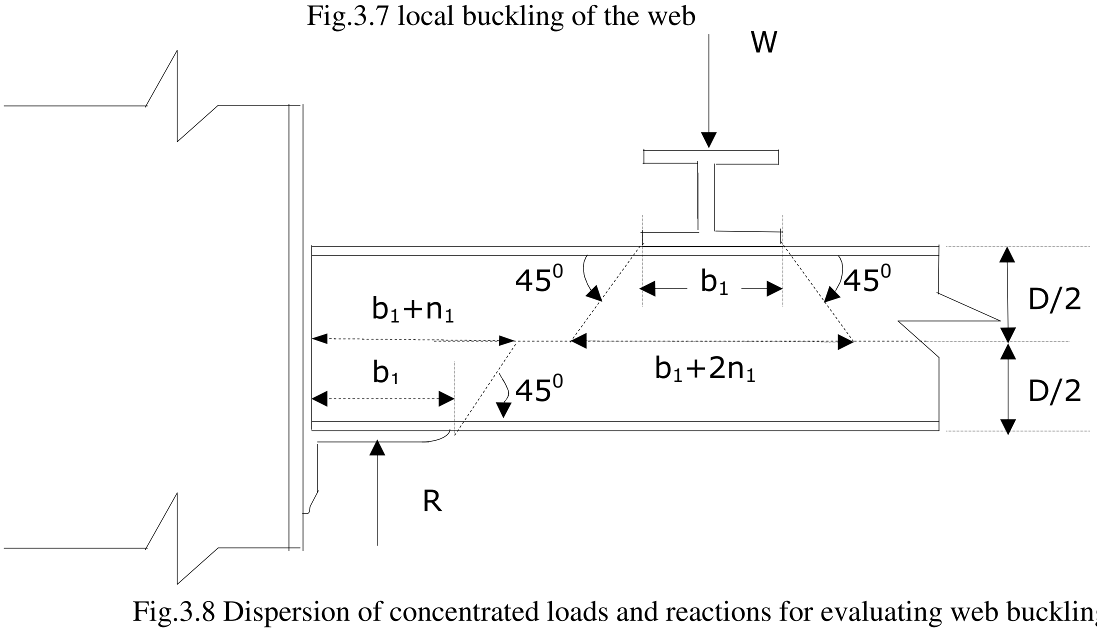


Fig.3.8 Dispersion of concentrated loads and reactions for evaluating web buckling

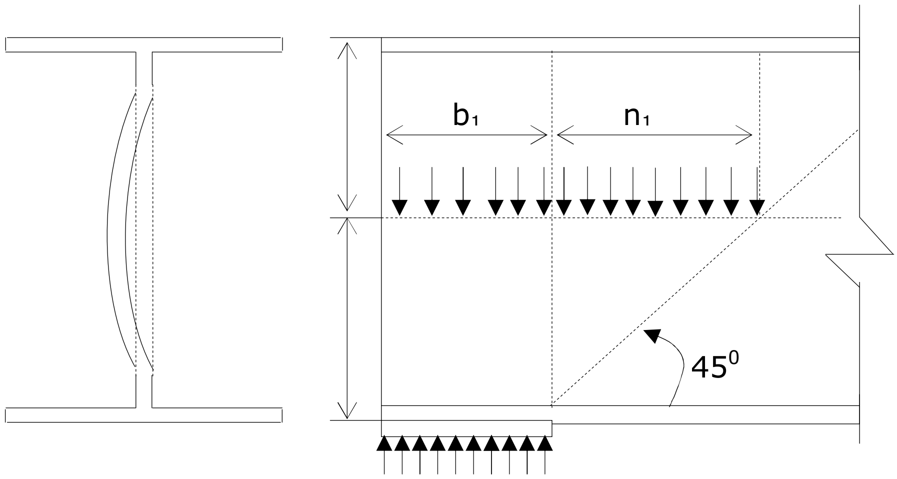


Fig.3.9 Effective width for web buckling

The load is spread out over a finite length of the web as shown in Fig.3.8. This is known as the *dispersion length* and it is very complex in theoretical. Hence empirical formulae based on experiments are used. One such assumption is that the dispersion length is taken as $(b_1 + n_1)$ and the web buckling strength at the support is given by

$$F_{wb} = (b_1 + n_1)\, t\, f_c \qquad \text{… (3.23)}$$

Where,

b_1 = stiff bearing length

n_1 = dispersion length as shown in Fig.3.9

t = thickness of web

f_c = allowable compressive stress corresponding to the assumed web Strut

3.9 LATERAL STABILITY OF BEAM

It is well known that slender members under compression are prone to instability. When slender structural elements are loaded in their strong planes, they have a tendency to fail by buckling in their weaker planes. So it's necessary to explain the phenomenon of lateral buckling in more details.

Consider a simply supported and laterally unsupported beam of short-span subjected to incremental transverse load at its mid section as shown in Fig.3.10 (a).

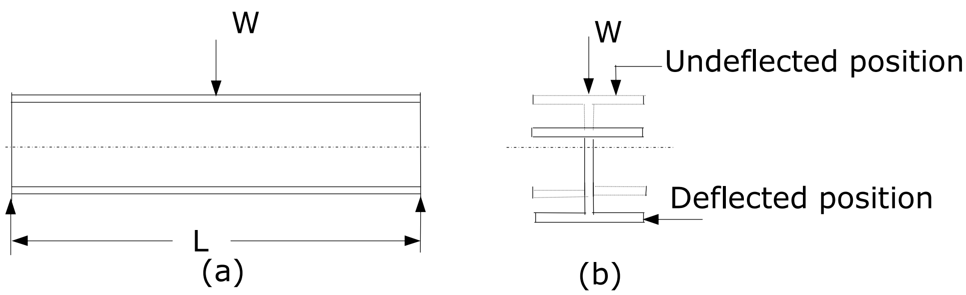


Fig.3.10 short span beam

It will deflect downward in the direction of loading but when a long span beam subjected to incremental transverse load, it will deflect downwards and when the load exceeds a particular value, it will tilt sideways due to instability of the compression flange and rotate about the longitudinal axis as shown in fig.3.11.

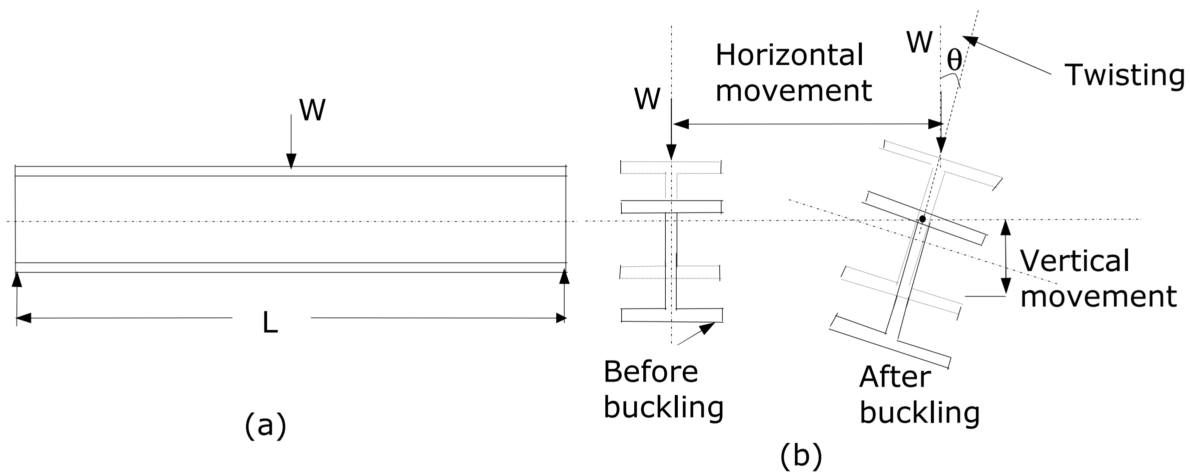


Fig.3.11 Long span beam

3.9.1 Lateral Torsional Buckling of Symmetric Sections

In lateral torsional buckling, beam buckles about weak axis even though it is loaded in the strong plane. When I beam bends about its strong axis up to critical load at which it buckles laterally as shown in Fig.3.12.

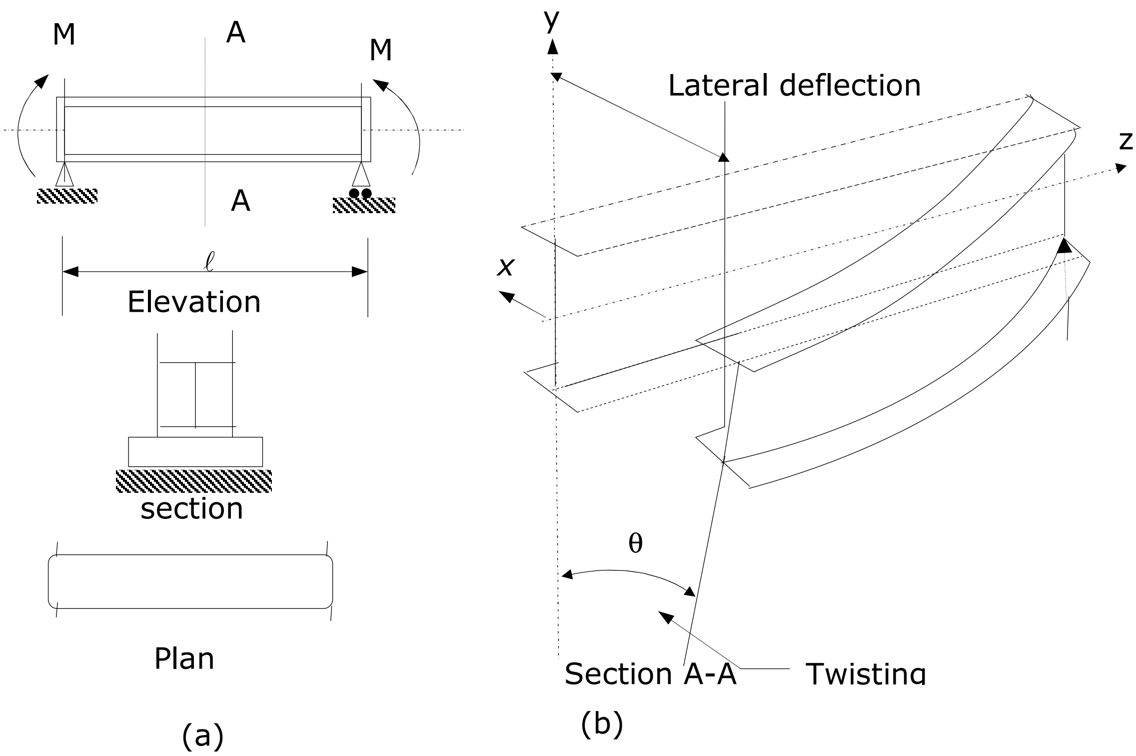


Fig.3.12 Lateral torsional buckling of beams

The differential equation for the angle of twist for the beam shown in Fig.3.12

$$EI_w \frac{d^4\phi}{dx^4} - GI_t \frac{d^2\phi}{dx^2} - \frac{M^2}{EI_y}\phi = 0 \qquad \text{... (3.24)}$$

The solution of this differential equation is given by

$$M_{cr} = (\pi / L)\sqrt{EI_y GI_t + \left(\frac{\pi E}{L}\right)^2 I_w I_y} \qquad \text{... (3.25)}$$

where,

EI_y = minor axis flexural rigidity

GI_t = torsional rigidity

EI_w = warping rigidity

Equation (3.25) is the elastic lateral torsional buckling strength of an I shaped section under the action of constant moment in the plane of the web over the laterally unbraced length L.

Equation (3.25) may be rewritten as for simply supported, prismatic member with symmetric cross section, the elastic lateral buckling moment

$$M_{cr} = \frac{\pi}{L}(EI_y GI_t)^{0.5}\left[1+\frac{\pi^2 EI_w}{L^2 GI_t}\right]^{0.5} \qquad \text{... (3.26)}$$

It is also written as

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(L_{LT})^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(L_{LT})^2}\right]\right\}} \qquad \text{... (3.27)}$$

Where,

L_{LT} = effective laterally unsupported length of the member

The magnitude of the first term in Equation (3.27) relates the capacity or the shape to resist St Venant torsion and second term is a measure of the contribution of warping to the torsion resistance of the beam. In general, for rolled section second term should be negligible as compared to first term because for short deep girder having larger value of EI_w and for long shallow girders having low warping stiffness $I_w \approx 0$, so Equation (3.26) reduces to

$$M_{cr} = \frac{\pi}{L}(EI_y GI_t)^{0.5} \qquad \text{... (3.28)}$$

3.10 DIFFERENT CODE COMPARESION FOR STEEL BEAM DESIGN

Sr.No.	Topics	IS 800:2007	BS 5950-1:2000	AISC 360-05	EUROCODE-3
1.	Design Philosophy	Limit State Method	Limit State Method	Load Resistant Factored Method (LRFD)	Limit State Method
2.	Load Factor	Live load = 1.5 Dead Load = 1.5	Live load = 1.6 Dead Load = 1.4	Live load = 1.2 Dead Load = 1.6	Live load = 1.5 Dead Load =1.35
3.	Yield stress	d or t < 20 $f_y = 250$ N/mm^2	T < 16 S275 $p_y = 275$ N/mm^2	K36 $f_y = 248.2$N/mm^2	$t_f < 40$ $f_y = 275$ N/mm^2
4.	Classification of section Class 1 (Plastic)	$b/t_f < 9.4\varepsilon$ $d/t_w < 84\varepsilon$	$b/t_f < 9\varepsilon$ $d/t_w < 80\varepsilon$	-	$b/t_f < 10\varepsilon$ $d/t_w < 72\varepsilon$
5.	Class 2 (Compact)	$b/t_f < 10.5\varepsilon$ $d/t_w < 105\varepsilon$	$b/t_f < 10\varepsilon$ $d/t_w < 100\varepsilon$	$b/t_f < 0.38\sqrt{E/f_y}$ $h/t_w < 3.76\sqrt{E/f_y}$	$b/t_f < 11\varepsilon$ $d/t_w < 83\varepsilon$
6.	Class 3 (Semi compact)	$b/t_f < 15.7\varepsilon$ $d/t_w < 126\varepsilon$	$b/t_f < 15\varepsilon$ $d/t_w < 120\varepsilon$	$b/t_f < 1.0\sqrt{E/f_y}$ $h/t_w < 5.7\sqrt{E/f_y}$	$b/t_f < 15\varepsilon$ $d/t_w < 124\varepsilon$
7.	Shear Buckling	$d/t_w < 67\varepsilon$	$d/t_w < 70\varepsilon$	$d/t_w < 69\varepsilon$	$d/t_w < 69\varepsilon$
8.	Shear Capacity	$V < 0.6V_d$	$F_v < P_v$	$V < V_n$	$V_{sd} < V_{pl,Rd}$

		$V_{pl,Rd} = \frac{f_y\ h\ t}{\gamma_{m0}\sqrt{3}}$	$P_v = 0.6\,t\,D$	$V_n = 0.6F_{yw}\,A_w\,c_v$	$V_{pl,Rd} = \frac{f_y\ A_v}{\gamma_{m0}\sqrt{3}}$
9.	Moment Capacity For Class 1 (Plastic) & Class 2 (Compact)	$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}\sqrt{3}}$ $\leq \frac{1.2 Z_e f_y}{\gamma_{m0}}$	$M_x < M_{cx}$ $M_{cx} = P_y Z_x$ $M_{cx} < 1.2 P_y Z_x$	$M_u = \phi Z\,f_y$ $\phi = 0.9$	$M_{c,Rd} = W_{pl} f_y / \gamma_{m0}$ $\gamma_{m0} = 1.05$
	Class 3 (Semi compact)	$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}\sqrt{3}}$ $\beta_b = \frac{Z_e}{Z_p}$	$M_{cx} = P_y Z_x$	-	$M_{c,Rd} = W_{el} f_y / \gamma_{m0}$ $\gamma_{m0} = 1.05$
10.	Deflection	L / 300	L / 360	L / 360	L / 350

3.11 ILLUSTRATED DESIGN

3.11.1 Design of Simply Supported Laterally Restrained Beam

Design Data

Total UDL on Beam = 163 kN/m

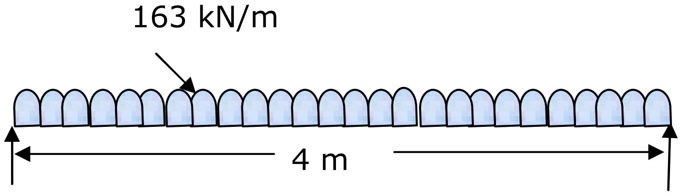
Span = 4 m

f_y = 250 N/mm^2

Load Factor = 1.5

Resistance governed by yielding γ_{m0} = 1.1

$\mathrm{E} =$ 200000 N/mm^2



Step-1 Calculation of Maximum Bending moment and Shear Force

Factored Load = 244.5 kN/m

Bending moment = 489.0 kN.m

Shear force = 489.0 kN

Step-2 Calculation of required plastic section modulus

$$Z_p = \frac{M\,\gamma_{m0}}{f_y} = 2152000 \text{ mm}^3$$

Step-3 Selection of suitable section

Choose a trial section of ISWB 600(1) @ 1.337 kN/m

Section properties

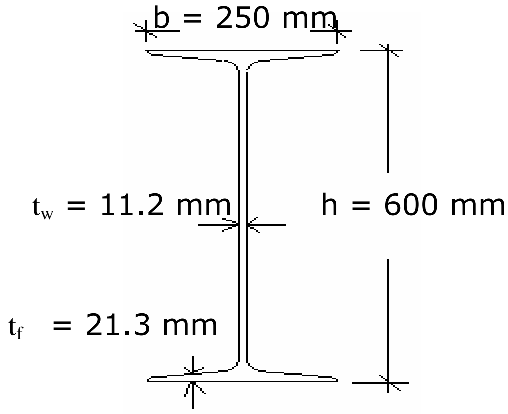
r_y = 52.5 mm　　Radius at root R = 17 mm

r_x = 249.7 mm　　I_{zz} = 1.062E+09 mm^4

$d_w = h - 2(t_f + R)$ = 523.4 mm　　I_{yy} = 47025000 mm^4

Elastic section modulus Z_e = 3540000 mm^3

Elastic section modulus Z_p = 3986700 mm^3



Effective length L_{LT} = 4000 mm

Section Classification: $\varepsilon = \sqrt{\dfrac{250}{f_y}}$ = 1

b/t_f = 5.87 < 9.4ε and d/t_w = 46.73 < 84ε

The section is classified as a Plastic. (Refer Table 2 of IS 800:2007)

Step-4 Adequacy of the section including self weight of the beam

Self weight of the section (factored) = 2.01 kN/m

Maximum bending moment = 493.01 kN.m

Plastic section modulus Z_p = 2169250 cm^3 < 3986700 mm^3, Safe

Step-5 Design strength of the section (*Ref. Cl.8.4*)

Check for Shear Buckling, $d/t_w < 67\varepsilon$

d/t_w=46.73 < 67, $\varepsilon = \sqrt{\dfrac{250}{f_y}}$ = 1 , Safe for shear buckling

Factored shear force (including self weight) V = 493.01 kN

$V_d = \dfrac{f_y h t_w}{\gamma_{m0}\sqrt{3}}$ = 881.8 kN

Design shear force 0.6V_d = 529.1 kN, Hence Safe

Step-6 Check for design capacity of the section (*Ref. Cl.8.2.1.2*)

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \le 1.2 Z_e f_y / \gamma_{m0}$$

β_b = 1.0 for plastic and compact sections

M_d = 906.5 kN.m ≤ 965.5 kN.m

Design strength 906.5 kN.m > 493.01 kN.m required strength, Safe

Step-7 Check for deflection (*Ref. Table 6*)

$\delta = \dfrac{5\,w\,l^4}{384\,EI}$ = 2.6 mm

Allowable maximum deflection = L / 300 = 13.33 mm, O.K.

Step-8 Check for Web buckling

Assume stiff bearing length b_1 = 100 mm

Depth of web t_w = 11.2 mm

$A_b = (b_1 + n_1)\, t_w$

$n_1 = D/2$ = 300 mm

A_b = 4480.0 mm^4

I_{eff} web = $b_1 t^3{}_w /12$ = 11707.73 mm^4

A_{eff} web = $b_1 t_w$ = 1120.0 mm^2

$r = \sqrt{\dfrac{I_{eff}\,web}{A_{eff}\,web}}$ = 3.23 mm

Effective length coefficient = 0.65

Effective length of web d_{eff} = 340.21 mm

$I = d_{eff} / r$ = 105.2 mm

f_{cd} = 110.7 N/mm^2

Buckling resistance = $f_{cd} A_b$ = 495.87 kN > 493.01 kN, Safe

Step-9 Check for web bearing

$F_w = (b_1 + n_2)\, t_w f_y / \gamma_{m0}$

n_2 = 2.5 (17+21.3) = 95.75 mm

F_w = 498.27 kN > 493.01 kN , Safe

3.11.2 Design of Simply Supported Laterally Unrestrained Beam

Design Data

Total UDL on Beam = 126 kN/m

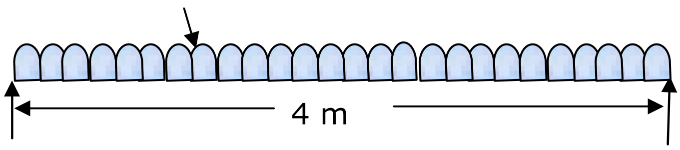
Span = 4 m

f_y = 250 N/mm^2

Load Factor = 1.5

Resistance governed by yielding γ_{m0} = 1.1

E = 200000 N/mm^2



Step-1 Calculation of Maximum Bending moment and Shear Force

Factored Load = 189.0 kN/m

Bending moment = 378.0 kN.m

Shear force = 378.0 kN

Step-2 Selection of intial section

Assume L_{LT}/r_y = 125 and $h/t_f = 30$

α_{LT} = 0.21 for rolled section

$$f_{cr,b} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1+\frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5} = 189.3 \text{ N/mm}^2$$

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{cr,b}}} = 1.147$$

$$\phi_{LT} = 0.5[1+\alpha_{LT}(\lambda_{LT}-0.2)+\lambda_{LT}^{\ 2}] = 1.26$$

$$\chi_{LT} = \frac{1}{\left[\phi_{LT}+\left(\phi_{LT}^{\ 2}-\lambda_{LT}^{\ 2}\right)^{0.5}\right]} \le 1.0 = 0.564$$

$f_{bd} = \chi_{LT}\, f_y / \gamma_{m0}$ = 128.21 N/mm^2

Therefore required $Z_p = M/\beta_b f_{bd}$ = 2948.28 cm^3

Step-3 Selection of suitable section

Choose a trial section of ISWB 600(1) @ 1.337 kN/m

Section properties

r_y = 52.5 mm

Radius at root R = 17 mm

r_x = 249.7 mm

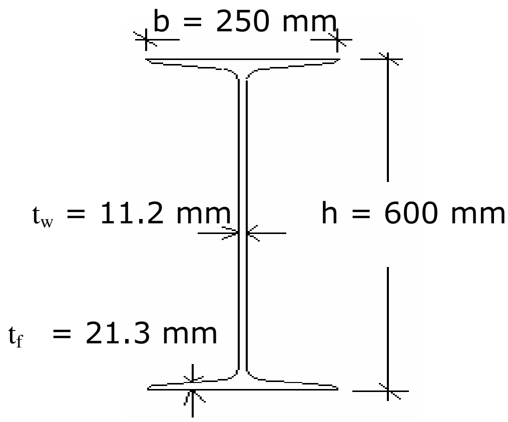
I_{zz} = 1.062E+09 mm^4

$d_w = h - 2(t_f + R)$ = 523.4 mm

I_{yy} = 47025000 mm^4

Elastic section modulus Z_e = 3540000 mm^3

Elastic section modulus Z_p = 3986700 mm^3



Effective length L_{LT} = 4000 mm

Section Classification: $\varepsilon = \sqrt{\dfrac{250}{f_y}}$ = 1

b/t_f = 5.87 < 9.4ε and d/t_w = 46.73 < 84ε

The section is classified as a Plastic. (Refer Table 2 of IS 800:2007)

Step-4 Calculation of lateral-torsional buckling moment (*Ref. cl.8.2*)

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(L_{LT})^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(L_{LT})^2}\right]\right\}}$$

$G = E/2(1+\mu)$ = 76923.08 N/mm^2

$I_t = \sum b_i t^3{}_i/3$ = 1.88E+06 mm^3, $I_w = (1-\beta_f)\beta_f I_y h^2{}_f$

$\beta_f = I_{fc}/(I_{fc} + I_{ft})$ = 0.5

I_w = 3.937E+12 mm^6

M_{cr} = 1912.48 kN.m

Check as per the approximate equation given in the code

$$M_{cr} = \frac{\pi^2 EI_y h_f}{2(L_{LT})^2}\left[1 + \frac{1}{20}\left[\frac{L_{LT}/r_y}{h_f/t_f}\right]^2\right]^{0.5}$$

M_{cr} = 2034.0 kN.m

$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \leq \sqrt{1.2 Z_e f_y / M_{cr}}$

= 0.72 ≤ 0.75, O.K.

$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$ = 0.82

$\chi_{LT} = \frac{1}{\left[\phi_{LT} + \left(\phi_{LT}^2 - \lambda_{LT}^2\right)^{0.5}\right]} \leq 1.0$ = 0.837 < 1, O.K.

$f_{bd} = \chi_{LT}\ f_y / \gamma_{m0}$ = 190.28 N/mm^2

β_b = 1.0 for plastic and compact sections

$M_d = \beta\ Z_p f_{bd}$ = 758.58 kN.m < M = 378.0 kN.m, Safe

Step-5 Adequacy of the section including self weight of the beam

Self weight of the section (factored) = 2.01 kN/m

Maximum bending moment = 382.01 kN.m

Design strength 758.58 > 382.01 kN.m required strength, Safe

Step-6 Design strength of the section (*Ref. cl.8.4*)

Check for Shear Buckling, $d / t_w < 67\varepsilon$

$d / t_w = 46.73 < 67,\ \varepsilon = \sqrt{\frac{250}{f_y}}$ = 1 , Safe for shear buckling

Factored shear force (including self weight) V = 382.01 kN

$V_d = \frac{f_y h t_w}{\gamma_{m0}\sqrt{3}}$ = 881.8 kN

Design shear force 0.6V_d = 529.1 kN, Safe

Step-7 Check for deflection (*Ref. Table 6*)

$\delta = \frac{5\, w\, l^4}{384\, EI}$ = 2.0 mm

Allowable maximum deflection = L / 300 = 13.33 mm, O.K.

Step-8 Check for Web buckling

$A_b = (b_1 + n_1)\ t_w$

$b_1 = (b_f - t_w)/2$ = 119.4 mm $\quad n_1 = D/2$ = 300 mm

A_b = 4697.3 mm^4

I_{eff} web = $b_1 t^3{}_w / 12$ = 13979.0 mm^4

A_{eff} web = $b_1 t_w$ = 1337.28 mm^2

$r = \sqrt{\dfrac{I_{eff} web}{A_{eff} web}}$ = 3.23 mm

Effective length coefficient = 0.65

Effective length of web d_{eff} = 340.21 mm

$I = d_{eff} / r$ = 105.2 mm

f_{cd} = 110.7 N/mm^2

Buckling resistance = $f_{cd} A_b$ = 383.0 kN > 382.01, Safe

Step-9 Check for web bearing

$F_w = (b_1 + n_2)\, t_w f_y / \gamma_{m0}$

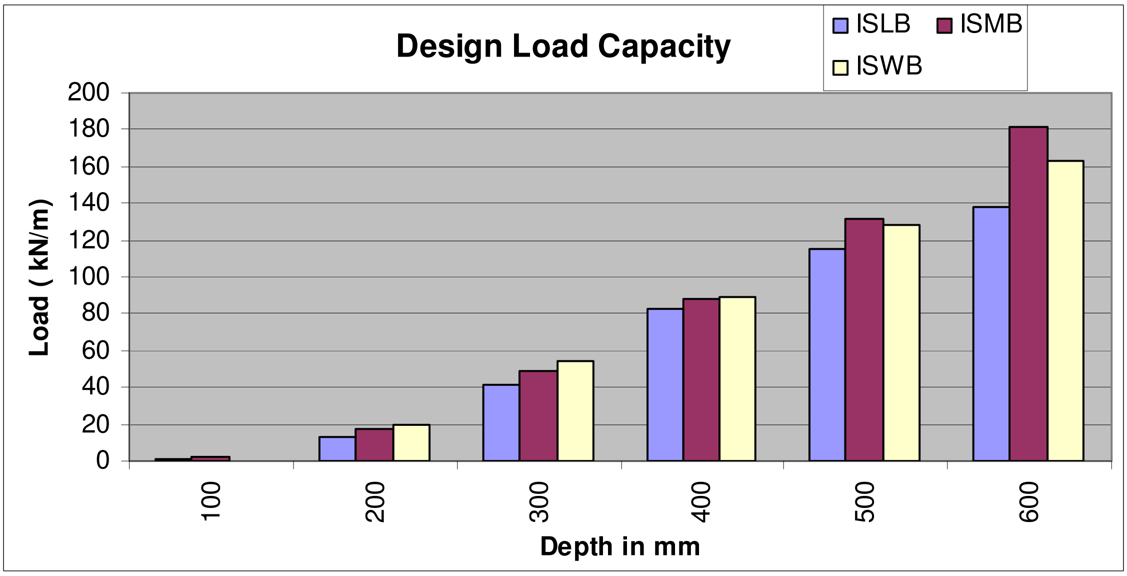
n_2 = 2.5 (17+21.3) = 95.75 mm

F_w = 547.65 kN > 382.01, Safe

3.12 DESIGN TABLE

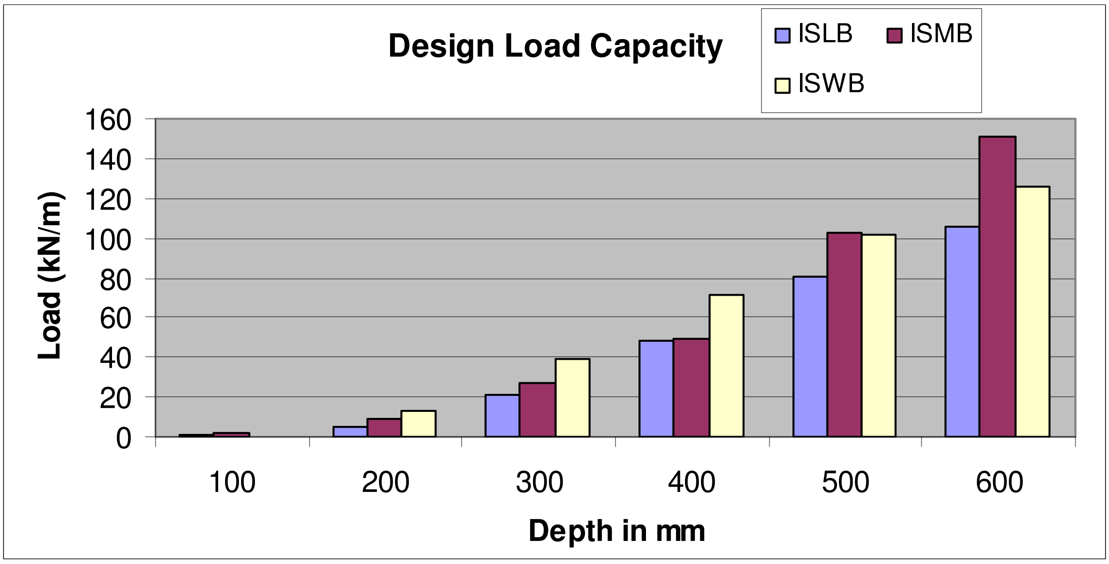
3.12.1 Safe UDL Carrying Capacity of Simply Supported Laterally Restrained Beam

section	W kN/m						
	4	6	8	10	12	14	16
ISLB 100	1.32	0.39	0.16				
ISLB 200	13.5	4	1.6	0.8	0.5		
ISLB 300	41.5	17.3	7.3	3.7	2.1	1.3	0.9
ISLB 400	82.5	36	19	9.5	5.5	3.5	2
ISLB 500	115	58	32	19	11	7	4
ISLB 600	138	92	52	32	21	13.5	9
ISMB 100	2	0.6	0.25				
ISMB 200	17.5	5	2	1	0.65	0.4	0.25
ISMB 300	48.5	20	8.5	4	2.5	1.5	1
ISMB 400	88	38	20	10	6	3	2
ISMB 500	132	68	38	23	13	8	5
ISMB 600	182	116	65	41	27	17	11
ISWB 200	20	6	2.5	1			
ISWB 300	54	23	9	5	2.5	1.5	1
ISWB 400	89	42	23	11	6	4	2.5
ISWB 500	128	78	43	26	15	9	6
ISWB 600(1)	163	108	74	46	31	19	13
ISWB 600(2)	180	120	80	51	34	21	14



3.12.2 Safe UDL Carrying Capacity of Simply Supported Laterally Unrestrained Beam

section	W kN/m						
	4	6	8	10	12	14	16
ISLB 100	0.85	0.2					
ISLB 200	5.5	1.5	0.5				
ISLB 300	21	5.5	2	0.8			
ISLB 400	48	12	4.5	2	0.9		
ISLB 500	81	21	8	3.5	1.5		
ISLB 600	106	40	14	6.5	3	1.5	
ISMB 100	2	0.5					
ISMB 200	9	2.5	1	0.3			
ISMB 300	27	7.5	3	1	0.5		
ISMB 400	49	13	5	2	1		
ISMB 500	103	29	11	5	2.5	1	
ISMB 600	151	60	23	11	5.5	3	1.5
ISWB 200	13	3.5	1	0.6			
ISWB 300	39	11	4	1.5	0.9		
ISWB 400	71	21	8	3.5	1.5	0.8	
ISWB 500	102	46	18	8	4	2	
ISWB 600(1)	126	83	35	17	9	5	2.5
ISWB 600(2)	145	96	41	20	10	6	3.5



4. DESIGN OF GANTRY GIRDERS

4.1 INTRODUCTION

Gantry girders are provided in industrial buildings to support overhead cranes for the transportation and lifting of heavy load. These cranes may be manually (hand) operated overhead traveling cranes (MOT) or electrically operated overhead traveling (EOT) cranes. A typical arrangement of gantry girder is shown in Fig.4.1.

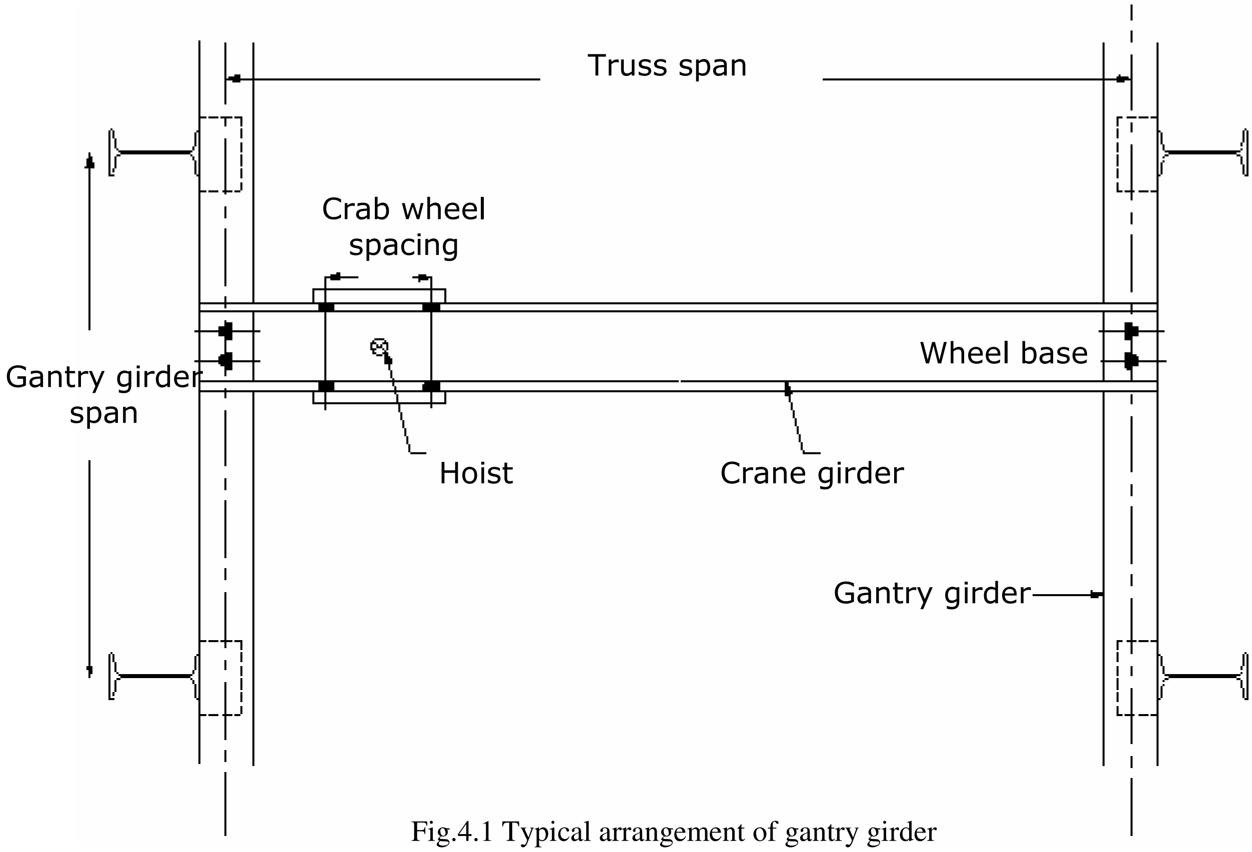


Fig.4.1 Typical arrangement of gantry girder

4.2 COMPONENTS OF CRANE SYSTEM

The following are the main components of crane system

a. Hoist
b. Crane crab or trolley
c. Crane girder
d. Gantry Girder

The loads are lifted using a hook and moved in longitudinal and transverse direction anywhere in the building through the movement of a crab car or trolley on the crane bridge and the crane wheels on the crane rails. The rails on either side of the bridge rest on gantry girders. The crane rails are fastened to gantry girder to avoid creeping and crowding of rails as shown in Fig.4.2.

Fig.4.2 Clamping of rails with bolts

The gantry girder is supported by brackets attached to the main columns of the building as shown in Fig.4.3 or by stepped columns.

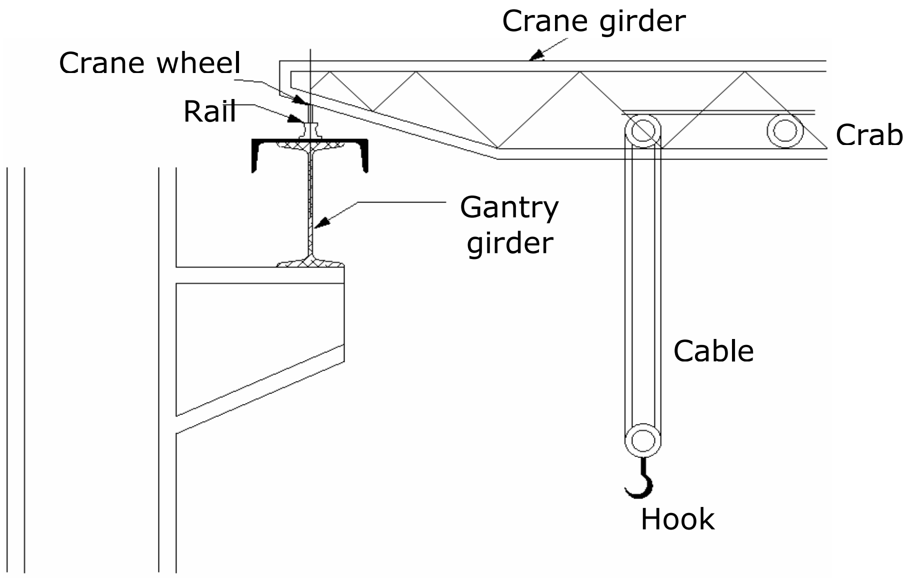
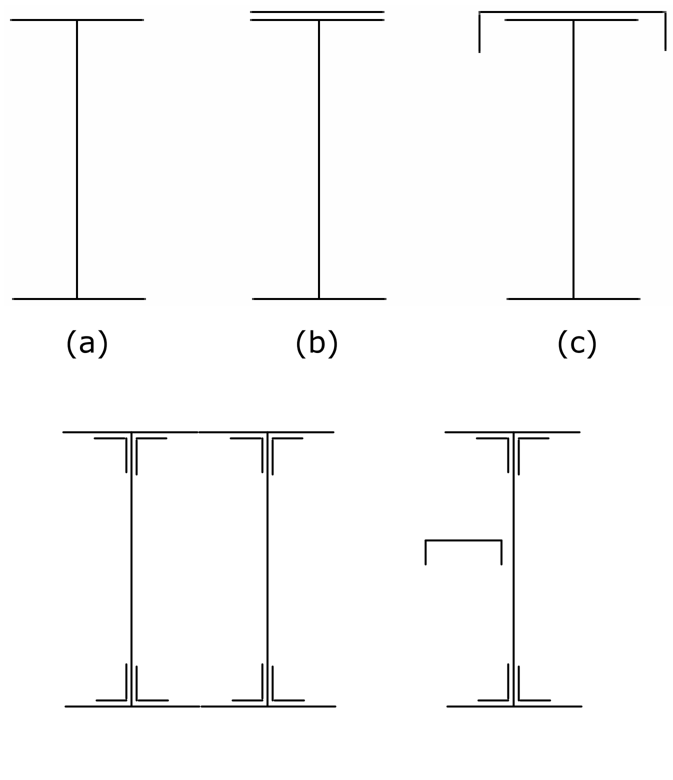


Fig.4.3 Details of gantry girder and crane girder attachment

Stops are provided at the end to prevent the crab from going over the girder. Some clearance is provided between the centre line of the gantry girder and the crane to avoid damage due to cross travel.

Gantry girders can be of many types depending on the span and crane capacity. In case of the compression flanges is laterally supported by either a catwalk or by additional member then the allowable stress in bending compression is same as that in tension. If the compression flange is laterally unsupported then the bending stress in compression flange decreases significantly. Thus the critical moment will be significantly less than its plastic capacity.

For small to medium spans, rolled beams with or without plates are used as shown in Fig.4.4 (a) and (b). For large spans plate girders, box girders and monosymmetric shapes consisting of I and Channel sections are used as shown in Fig.4.4 (c). Placing of channel or plate on the top of I section which will increase moment of inertia of compression flange about minor axis which is helping to resist lateral forces. Built up section shown in Fig.4.4 (d) & (e) which is used for large span or heavy cranes



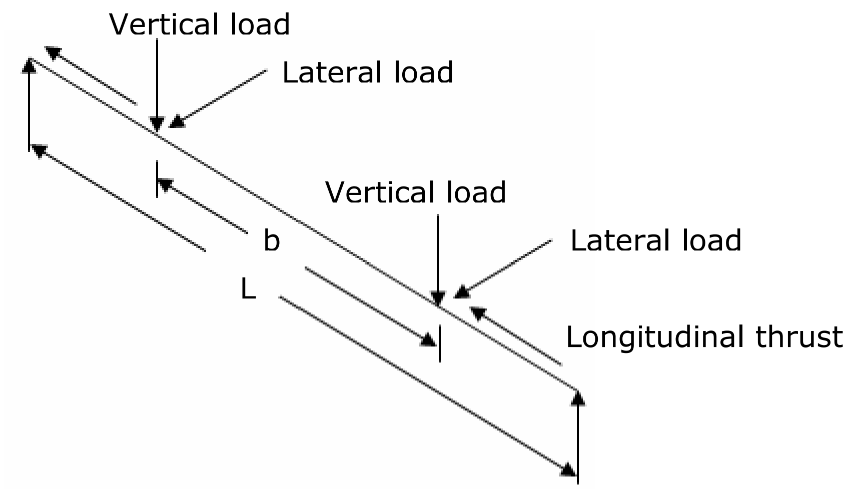
(a) (b) (c)

(d) (e)

Fig.4.4 Form of gantry girder

4.3 FORCES

Gantry girder undergoes bending due to the following forces:



a. Vertical reaction from the loads on the crane girder

b. Longitudinal forces due to starting or stopping of the crane

c. Lateral force due to starting or stopping of the crab

d. Impact effect

e. Fatigue effect

4.3.1 Vertical Forces

Vertical forces acting on the gantry girder are the vertical reaction from the crane girder and self weight of the gantry girder. The maximum wheel load is due to the weight of the crane girder, the crab and the crane capacity and occurs when the crab is nearest to the gantry girder. The effect of impact has to be included.

4.3.2 Horizontal Forces

Horizontal forces are of two types:

a. Longitudinal Forces are those which act parallel to the gantry girder.
b. Lateral Forces are those which act in a direction perpendicular to the gantry girder.

a. Longitudinal Forces

These are caused due to the starting/stopping or acceleration/deceleration of the crane. These produce thrust along the longitudinal direction of the gantry girder. These are transferred at the rail level. Therefore, the gantry girders are subjected to moments due to these forces.

b. Lateral Forces

These are caused due to the starting/stopping or acceleration deceleration of the crab. These produce thrust normal to the gantry girder. These produce bending moment in the girder in a horizontal plane.

4.3.3 Fatigue Effects

Gantry Girders are subjected to fatigue effects due to moving loads. In IS 800:2007 special provision for fatigue assessment is given in section 13, if member or connection having stress range $f \le 27/\gamma_{mft}$ or if the actual number of stress cycles N_{sc}, satisfies

$$N_{sc} < 5 \times 10^6 \left(\frac{27/\gamma_{mft}}{\gamma_{fft} f} \right)^3 \qquad \text{... (4.1)}$$

Where,

γ_{mft} , γ_{fft} = partial safety factors for strength and load, respectively

f = actual fatigue stress range for the detail.

then fatigue strength check is not required. For heavy-duty cranes, the gantry girders are to be checked for fatigue loads (also refer IS 1024 and IS 807).

4.4 PRILIMINARY CHOICE OF SECTION

The preliminary choice of the gantry girder depends upon the crane capacity, span of gantry girder, etc.

Table 4.1 Preliminary Choice for Different Conditions

Sr.no.	Choice	Condition
1.	I-section	MOT cranes
2.	I-sections with plates/channels	spans up to 8 m and 50 kN cranes
3.	Plate girders	spans from 6 to 10 m
4.	Plate girder with channels, angles etc.	spans more than 10 m
5.	Box girders with angles	Span more than 12 m

4.5 DESIGN OF GANTRY GIRDER

Step 1: Calculate the Maximum Wheel load

This occurs when the crab is closest to the gantry girder. The maximum load will be half the reaction as there two wheels to the crab. Add the impact load.

Step 2: Calculate the Maximum Bending Moment Due to Vertical Forces

This is due to wheel load, impact and dead load. The dead load bending moment $\left(M_1 = wl^2/8\right)$ is calculated by assuming the dead load which is to be checked later on. The bending moment (M_2) due to wheel load is absolutely maximum, when the resultant and the load under which the bending moment is maximum, are equidistant from the mid-span. (Fig.4.5)

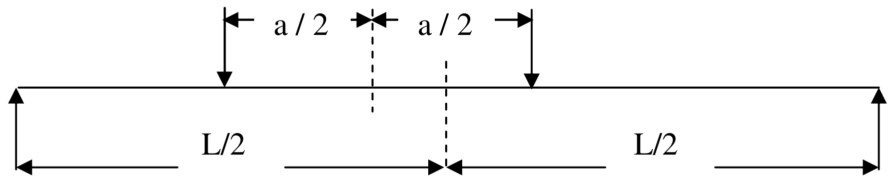


Fig.4.5 Maximum Bending moment (Live Load)

Step 3: Calculate the Maximum Shear Force

It is due to wheel load, impact load and dead load. The shear force due to wheel load is maximum, when one of the wheel load is at the support. (Fig.4.6)

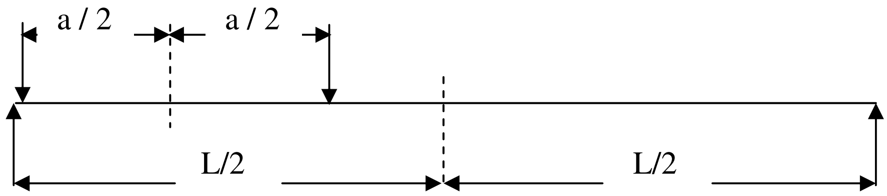


Fig.4.6 Maximum Shear Force

Step 4: Assume preliminary size of the girder

For depth = L/12 and width = L/30

Step 5: Calculate sectional properties

Step 6: Calculation of plastic modulus

Considering plastic neutral axis divide the section into two equal areas, calculate plastic modulus.

Step 7: When lateral support is provided to compression flange of the section, moment capacity of whole section is checked (clause 8.2.1.2)

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2 Z_e f_y / \gamma_{m0} \qquad \text{... (4.2)}$$

The top flange should be checked for bending in both axis using the interaction equation

$$\frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} < 1 \qquad \text{... (4.3)}$$

Step 8: If the compression flange is not supported, buckling resistance to be checked in the same way as in step 6 but replacing f_y with design bending compressive stress f_{bd} (clause 8.2.2)

Step 9: Check for web buckling and web crippling

Step 10: Check for deflection

4.6 COMPARISON OF DESIGN OF GANTRY GIRDER AS PER IS 800:2007 AND IS 800:1984

Sr. No.	IS 800:1984 (WSM)	IS 800:2007 (LSM)
1.	Calculate the Maximum Wheel load	Calculate the Maximum Wheel load
2.	Calculate the Maximum Bending Moment Due to Vertical Forces (M_z)	Calculate the Maximum Bending Moment Due to Vertical Forces (M_z) Apply multiplication factor 1.5 for live load and dead load
3.	Calculate the Maximum Shear Force	Calculate the Maximum Shear Force
4.	$Z_{req} = \frac{M_x}{\sigma_{bc}}$, $\sigma_{bc} = 0.66 f_y$	Approximate $I_{ZZ} = \frac{15.6W(L-c)}{LE}[2L^2 + 2L_c - c^2]$ $Z_p = 1.4 \times M / f_y$
5.	Assume section	Classify the section (plastic, compact, semi-compact)
6.	Calculate I_{xx} ,I_{yy} and Z_1, Z_2 $\sigma_{bc,cal} = \frac{M_x}{Z_1}$ $\sigma_{bt,cal} = \frac{M_x}{Z_2}$ $\sigma_{bt,cal} < 1.1 \times 0.66 f_y$	Calculate I_z, Z_z, Calculation of plastic sectional modulus (Z_{pz}, Z_{py})

7.	Lateral Forces = 10% of weight of crab and weight lifted $\sigma_{bcy,cal} = \dfrac{M_y}{I_{yy} of\ compression.flange} \times \dfrac{b}{2}$ Calculate $\sigma_{bc,permissible}$ from Table 6.1 $f_{cb} = k_1(X + k_2 Y)\dfrac{c_2}{c_1}$	Check for local moment capacity $M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2 Z_e f_y / \gamma_{m0}$ Combined local capacity check $\dfrac{M_z}{M_{dz}} + \dfrac{M_y}{M_{dy}} < 1$
8.	Longitudinal Forces = 5% of static wheel load Check the stresses	Check for buckling resistance $M_d = \beta\ Z_p f_{bd}$ $M_{cr} = C_1 \dfrac{\pi^2 E I_y h_f}{2(L_{LT})^2}\left[1 + \dfrac{1}{20}\left[\dfrac{L_{LT}/r_y}{h_f/t_f}\right]^2\right]^{0.5}$ $\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \leq \sqrt{1.2 Z_e f_y / M_{cr}}$ $\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}{}^2\right]$ $\chi_{LT} = \dfrac{1}{\left[\phi_{LT} + \left(\phi_{LT}{}^2 - \lambda_{LT}{}^2\right)^{0.5}\right]} \leq 1.0$ $f_{bd} = \chi_{LT}\ f_y / \gamma_{m0}$
9.	$\dfrac{\sigma_{(bcx,cal)}}{1.1 \times \sigma_{(bcx,per)}} + \dfrac{\sigma_{(bcy,cal)}}{1.1 \times \sigma_{(bcy,per)}} \leq 1$	$\dfrac{M_z}{M_{dz}} + \dfrac{M_y}{M_{dy}} < 1$
10.	Check for shear $\tau_{va} < \tau_{va,per}$ $\tau_{va,per} = 1.1 \times 0.4 f_y$	$V \leq 0.6 V_d$ $V_d = A_v f_{yw} / (\sqrt{3} \times \gamma_{m0})$
11.	Check for deflection	Check for deflection Same like IS 800:1984
12.	Weld design $q = VA\,\overline{y} / I_Z$ strength of weld = $0.7 \times s \times l \times 108$	Weld design $q = VA\,\overline{y} / I_Z$ strength of weld = $0.7 s \left(\dfrac{f_u}{\sqrt{3}\gamma_{mw}}\right) l$

4.7 DIFFERENT CODE COMPARESION FOR DESIGN OF GANTRY GIRDER (LATERALLY UNRESTAINED)

Sr.No.	Topics	IS 800:2007	BS 5950-1:2000	EUROCODE-3
1.	Transverse Force	$W_{H1} = \dfrac{0.1\left(W_{cb} + W_{cap}\right)}{No.\ of\ \ wheels}$	$W_{H1} = \dfrac{0.1\left(W_{cb} + W_{cap}\right)}{No.\ of\ \ wheels}$	-
2.	Longitudinal Force	$W_{H2} = 0.05\,W_{wheel}$	$W_{H2} = 0.05\,W_{wheel}$	
3.	Moment Capacity	$M_d = \beta\ Z_p f_{bd}$ $f_{bd} = \chi_{LT} f_y / \gamma_{m0}$ $\chi_{LT} = \dfrac{1}{\left[\phi_{LT} + \left(\phi_{LT}{}^2 - \lambda_{LT}{}^2\right)^{0.5}\right]} \le 1.0$ $\phi_{LT} = 0.5[1 + \alpha_{LT}\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}{}^2]$ $\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}}$ $\le \sqrt{1.2 Z_e f_y / M_{cr}}$ $M_{cr} = \dfrac{\pi^2 E I_y h_f}{2(L_{LT})^2}\left[1 + \dfrac{1}{20}\left[\dfrac{L_{LT}/r_y}{h_f/t_f}\right]^2\right]^{0.5}$	$M_x \le M_{cx}$ $M_x \le \dfrac{M_b}{m_{LT}}$ $M_b = P_b S_x$ $\lambda_{LT} = uv\lambda\sqrt{\beta_W}$ Using P_y and λ_{LT} Get P_b from table 16	$M_{b,Rd} = \chi_{LT\,\mathrm{mod},} W_{pl,y} f_y / \gamma_{M1}$ $\chi_{LT\,\mathrm{mod},} = \chi_{LT} / f$ $f = 1 - 0.5\left(1 - k_c\right)\left[1 - 2\left(\bar{\lambda}_{LT} - 0.8\right)^2\right]$ $\phi_{LT} = 0.5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}\right) + \beta\,\lambda^{-2}{}_{LT}\right]$ $\lambda_{LT} = \sqrt{\dfrac{W_{pl,y} f_y}{M_{cr}}}$, $\chi_{LT} = \dfrac{1}{\phi_{LT} + \sqrt{\phi^2{}_{LT} - \beta\,\lambda^{-2}{}_{LT}}} \le 1$ $M_{cr} = C_1 \dfrac{\pi^2 E I_z}{(KL)^2}\left\{\sqrt{\left(\dfrac{K}{K_w}\right)^2 \dfrac{I_w}{I_z} + \dfrac{(KL)^2 G I_t}{\pi^2 E I_z} + \left(C_2 z_g\right)^2} - C_2 z_g\right.$
4.	Deflection	MOT cranes = L/500 EOT cranes < 500 kN = L/750 EOT cranes > 500 kN = L/1000	Vertical deflection = L/600 Horizontal deflection = L/500	-

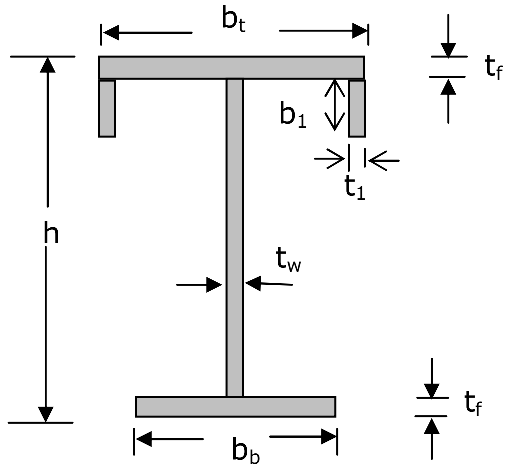
4.8 DESIGN AIDS

To prepare design aids spread sheets have been prepared for design of gantry girder considering single crane having capacity of 100kN, 200kN and 250kN taking span of gantry girder 6m, 7.5m and 10m and crane girder span of 16m and 18m. For double crane same capacity as mentioned as above with difference in span of gantry girder as 9m, 10.5m and 12m is considered.

100 kN capacity (single crane)			
Span	Gantry Girder Span		
16 m	6 m	7.5 m	9 m
	ISMB 450, ISLC 200	ISMB 500, ISLC 250	ISMB 600, ISLC 250
Total wt.	0.912 kN/m	1.127 kN/m	1.477 kN/m
18 m	ISMB 450, ISLC 225	ISMB 500, ISMC 250	ISMB 600, ISLC 250
Total wt.	0.946 kN/m	1.151 kN/m	1.477 kN/m

200 kN capacity (single crane)			
Span	Gantry Girder Span		
16 m	6 m	7.5 m	9 m
	ISMB 550, ISLC 250	ISMB 600, ISLC 350	Section 1
Total wt.	1.292 kN/m	1.583 kN/m	2.17 kN/m
18 m	ISMB 550, ISLC 250	ISMB 500, ISMC 350	Section 2
Total wt.	1.292 kN/m	1.583 kN/m	2.17 kN/m

250 kN capacity (single crane)			
Span	Gantry Girder Span		
16 m	6 m	7.5 m	9 m
	Section 3	Section 4	Section 5
Total wt.	1.46 kN/m	1.99 kN/m	2.48 kN/m
18 m	Section 6	Section 4	Section 5
Total wt.	1.47 kN/m	1.99 kN/m	2.48 kN/m



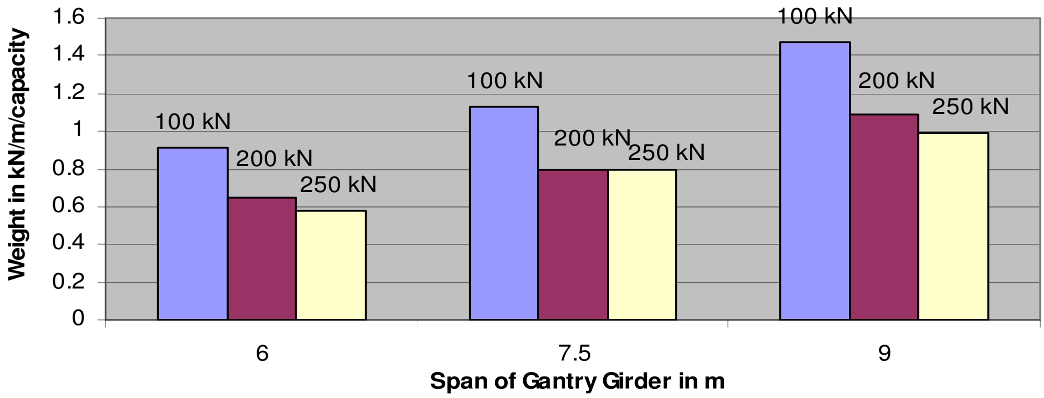
100 kN capacity (double crane)			
Span	Gantry Girder Span		
16 m	9 m	10.5 m	12 m
	Section 7	Section 8	Section 9
Total wt.	2.02 kN/m	2.41 kN/m	2.77 kN/m
18 m	Section 10	Section 8	Section 11
Total wt.	2.03 kN/m	2.41 kN/m	2.79 kN/m

200 kN capacity (double crane)			
Span	Gantry Girder Span		
16 m	9 m	10.5 m	12 m
	Section 12	Section 13	Section 14
Total wt.	2.72 kN/m	3.01 kN/m	3.64 kN/m
18 m	Section 15	Section 13	Section 16
Total wt.	2.73 kN/m	3.01 kN/m	3.66 kN/m

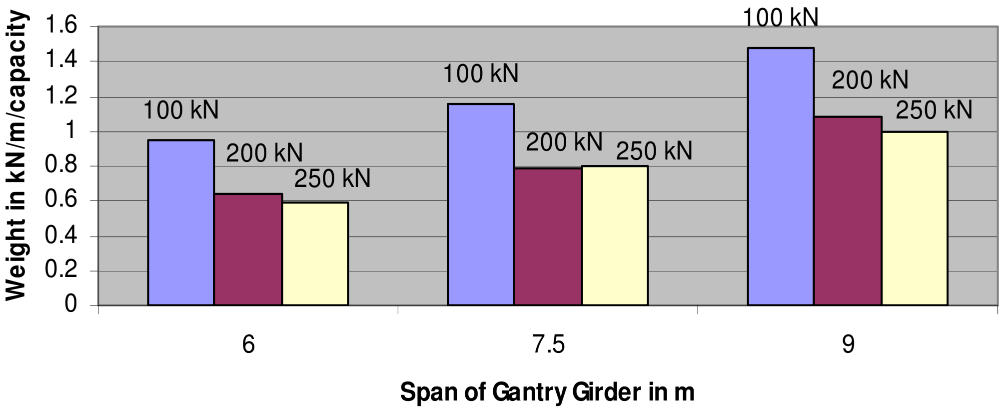
250 kN capacity (double crane)			
Span	Gantry Girder Span		
16 m	9 m	10.5 m	12 m
	Section 17	Section 18	Section 19
Total wt.	2.97 kN/m	3.47 kN/m	4.15 kN/m
18 m	Section 17	Section 18	Section 19
Total wt.	2.97 kN/m	3.47 kN/m	4.15 kN/m

Table 4.2 List of Sectional Properties

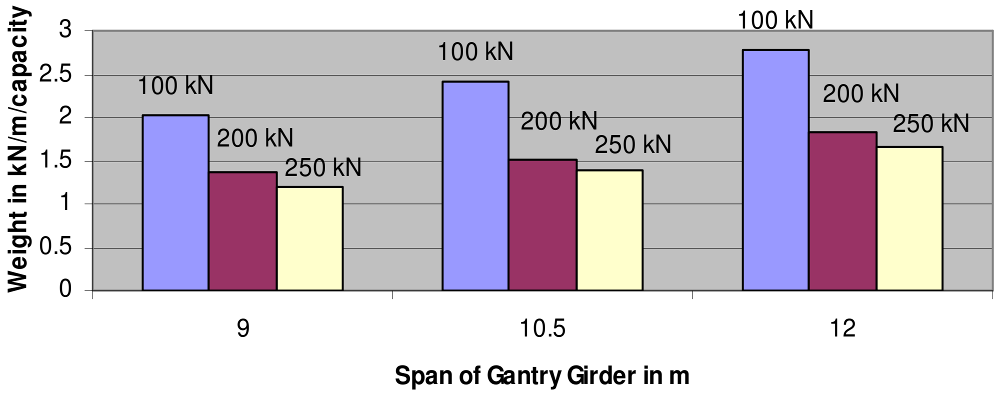
Section No.	w/m (kN/m)	A (mm^2)	h (mm)	b_t (mm)	b_b (mm)	t_f (mm)	t_w (mm)	b_1 (mm)	t_1 (mm)
1.	2.17	28160	800	320	280	20	16	100	20
2.	2.17	28160	800	320	280	20	16	100	20
3.	1.46	18896	700	280	250	16	12	75	16
4.	1.99	25784	770	300	280	18	16	100	18
5.	2.48	32120	880	350	300	20	18	100	20
6.	1.47	19056	700	280	260	16	12	75	16
7.	2.02	26104	790	300	280	18	16	100	18
8.	2.41	31196	880	330	280	22	16	100	22
9.	2.77	35952	930	380	330	24	16	100	24
10.	2.03	26284	790	310	280	18	16	100	18
11.	2.79	36112	940	380	330	24	16	100	24
12.	2.72	35268	940	350	320	22	18	100	22
13.	3.01	38992	1000	420	370	24	16	100	24
14.	3.64	47244	1160	450	400	26	18	100	26
15.	2.73	35448	950	350	320	22	18	100	22
16.	3.66	47424	1170	450	400	26	18	100	26
17.	2.97	38468	1020	400	350	22	18	100	22
18.	3.47	44976	1160	440	400	24	18	100	24
19.	4.15	53860	1250	500	450	26	20	100	26



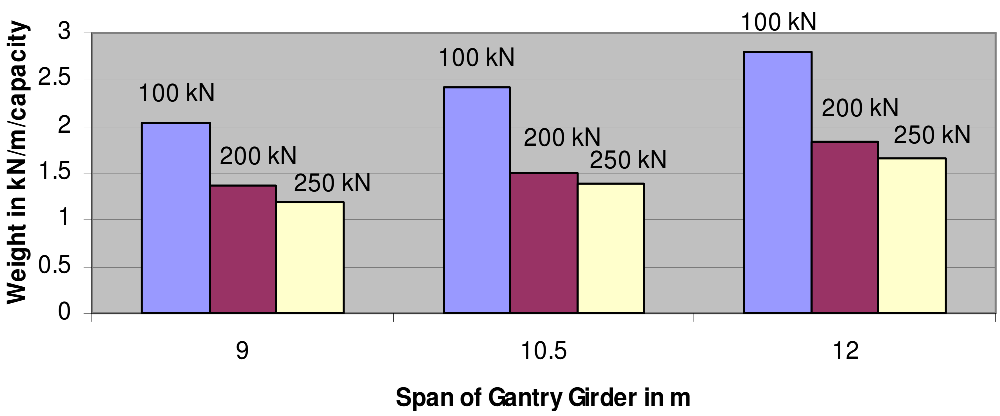
Single Crane (For 16 m)



Single Crane (For 18 m)



Double Crane (For 16 m)



Double Crane (For 18 m)

5. DESIGN OF PLATE GIRDER

5.1 GENERAL

Modern plate girders are normally fabricated by welding together two flanges and a web plate, as shown in Fig.5.1. Such girders are capable of carrying greater loads over longer spans than is generally possible using standard rolled sections or compounds girders. Plate girders are typically used as long span floor girders in buildings, as bridge girders and as crane girders in industrial structures.
For efficient design it is usual to choose a relatively deep girder, thus minimizing the required area of flanges for a given applied moment and a deep web whose area will be minimized by reducing its thickness to the minimum required to carry the applied shear. Such a web may be quite slender (i.e. a high d/t_w ratio) and may be prone to local buckling and shear buckling. Such buckling problems have to be given careful consideration in plate girder design. One way of improving the load carrying resistance of a slender plate is to employ stiffeners, the selection of appropriate forms of stiffening is an important aspect of plate girder design.

5.2 SHEAR RESISTANCE OF TRANSVERSELY STIFFENED PLATE GIRDER

The shear resistance of a plate girder depends upon the depth to thickness ratio of the web and the spacing of the intermediate stiffener provided. Webs of plate girders are stiffened transversely as shown in Fig.5.1

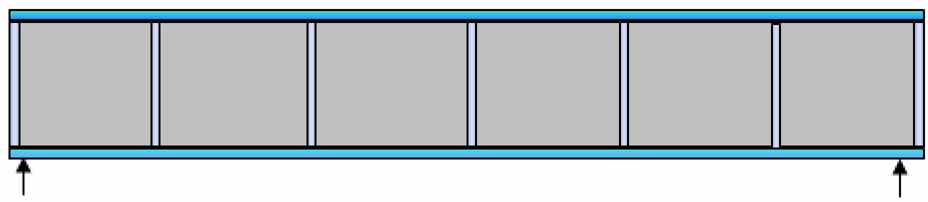


Fig.5.1 Transversely stiffened plate girder

The shear capacity of the web has two components:

1. Strength before the onset of buckling
2. Strength after post buckling

As shear load is increased on a stiffened web panel, it will go to buckle. This load does not indicate the maximum shear capacity of the web. The load can be still increased and the web panel continues to carry further load relying on the tension field action. Part of the buckled web takes the load in tension. This tension member action is across the web panel in an inclined direction to the web panel diagonal as shown in Fig.5.2

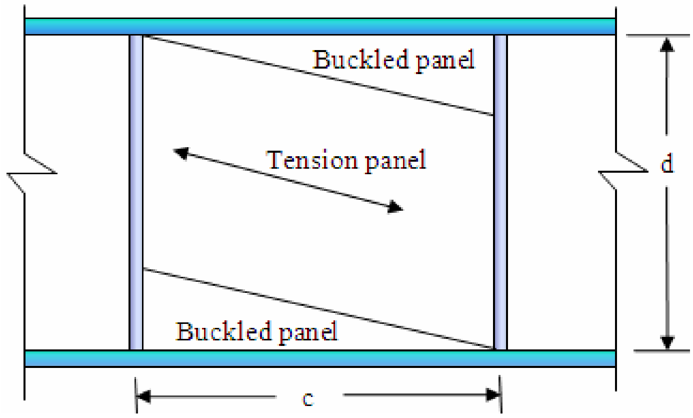


Fig.5.2 Tension Field Action in Panel

At this stage, the girder acts like an N-truss with the compression forces carried by the flanges and the intermediate stiffeners and buckled web resisting the tension. This additional reserve strength is termed as tension field action. If no intermediate stiffeners are present or their spacing is large, it is not possible for tension field to take place and the shear capacity is restricted to the strength before buckling.

5.2.1 Pre-buckling behaviour (stage 1)

When a web plate is subjected to shear, it can be visualize the structural behaviour by considering the effect of complementary shear stresses generating shear stresses diagonal tension and diagonal compression as shown in Fig.5.3

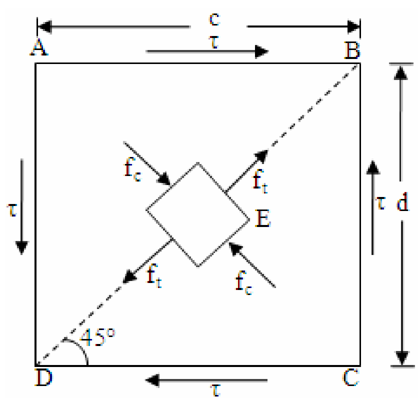


Fig.5.3 Unbuckled shear panel

Consider an element E in equilibrium inside a square web plate subject to a shear stress (τ). The element is subjected to principal compression along the direction AC and tension along the direction of BD. As the applied loading is gradually increased, τ increase and the plate will buckle along the direction of the compressive stress. The plate will lose its capacity to any further increase in compressive stress, the corresponding shear stress in plate is the *critical shear stress* ($\tau_{cr,e}$).

$$\tau_{cr,e} = k_v \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t_w}{d}\right)^2 \qquad \text{... (5.1)}$$

Where,

μ = Poisson's ratio

k_v = 5.35 when transverse stiffeners are provided only at supports

= $4.0 + 5.35/(c/d)^2$ for $c/d < 1$

= $5.35 + 4.0/(c/d)^2$ for $c/d \geq 1$

c, d = spacing of transverse stiffeners and depth of web, respectively

5.2.2 Post buckled behaviour (stage 2)

As we increase loading the compression diagonal (AC) is unable to resist any more loading beyond the one corresponding to the elastic critical stress at that time new load carrying mechanism is developed along the principal tensile direction which is known as tension field action. The tension field is constituted by the portion of the plate in the principal tensile direction and anchored at the boundaries along the top and bottom flanges and the stiffener member on either side of web as shown in Fig.5.4.

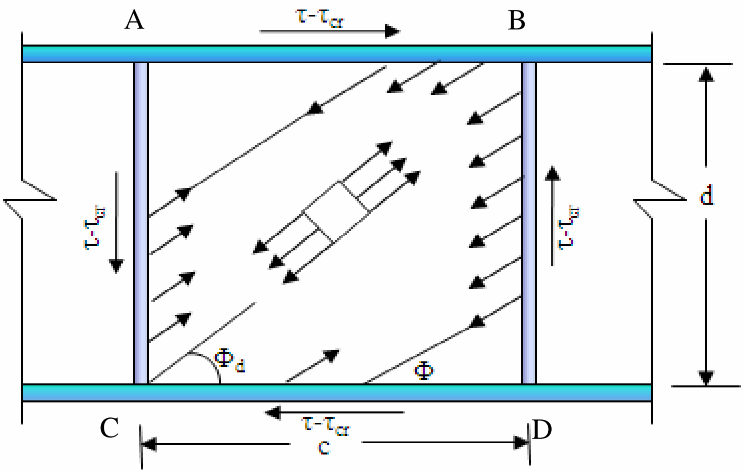


Fig.5.4 Post-buckled behaviour

At this stage, total stress in web portion is composed of the applied critical shear stress $\tau_{cr,e}$ and the post-buckled membrane tensile stress f_t due to tension field action. The state of stress at this stage is shown in Fig.5.5.

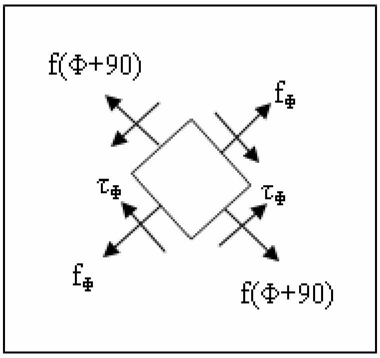


Fig.5.5 State of stress in the web in the web in the post-buckled stage

Resolving these stresses in the direction along and perpendicular to the inclination, ϕ.

$$f_{\phi} = \tau_{cr,e} \sin 2\phi + f_t \qquad \text{... (5.2)}$$

$$f_{(\phi+90)} = -\tau_{cr,e} \sin 2\phi \qquad \text{... (5.3)}$$

$$\tau_{\phi} = -\tau_{cr,e} \cos 2\phi \qquad \text{... (5.4)}$$

5.3 SHEAR BUCKLING DESIGN

In design of plate girder resistance to shear buckling shall be verified as specified, when $d/t_w > 67\varepsilon\sqrt{\frac{K_v}{5.35}}$, $d/t_w > 67\varepsilon$ for a web with stiffeners and without stiffeners, respectively. The nominal shear strength V_n of web with or without intermediate stiffeners as governed by buckling may be evaluated using one of the following methods:

1. Simple post-critical method
2. Tension field method

5.3.1 Simple post-critical method

It is based on shear buckling strength and used for webs of I-section girders, with or without intermediate transverse stiffeners and transverse stiffeners at the supports. The nominal shear strength is given by

$$V_n = V_{cr} \qquad \text{... (5.5a)}$$

where,

V_{cr} = shear force corresponding to web buckling

$V_{cr} = A_v \tau_b$ ... (5.5b)

where,

τ_b = shear stress corresponding to web buckling, determined as follows:

1) when $\lambda_w \leq 0.8$

$\tau_b = f_{yw} / \sqrt{3}$ ... (5.5c)

2) when $0.8 < \lambda_w < 1.2$

$\tau_b = [1 - 0.8(\lambda_w - 0.8)] f_{yw} / \sqrt{3}$ ... (5.5d)

3) when $\lambda_w \geq 1.2$

$\tau_b = f_{yw} / \left(\sqrt{3} \lambda^2{}_w\right)$ ... (5.5e)

where,

λ_w = non-dimensional web slenderness ratio for shear buckling stress

= $\sqrt{f_{yw} / \left(\sqrt{3} \tau_{cr,e}\right)}$ ... (5.6)

5.3.2 Tension field method

It is based on post-shear buckling strength and used for webs of I-section girders, with or without intermediate transverse stiffeners, in addition to the transverse stiffeners at the supports. It is also necessary that $c / d \geq 1$.The nominal shear resistance is given by

$V_n = V_{tf}$ ... (5.7a)

where,

$V_{tf} = \left[A_v \tau_b + 0.9 w_{tf} t_w f_v \sin\phi\right] \leq V_p$ ... (5.7b)

where,

$f_v = [f^2{}_{yw} - 3\tau^2{}_b + \Psi^2]^{0.5} - \Psi$ ... (5.7c)

$\Psi = 1.5 \tau_b \sin 2\phi$ ... (5.7d)

ϕ = inclination of the tension field

$\phi = \tan^{-1}(d / c)$ ... (5.7e)

w_{tf} = the width of the tension field

$w_{tf} = d \cos\phi - (c - s_c - s_t) \sin\phi$ ... (5.7f)

f_{yw} = yield stress of the web

s_c, s_t = anchorage lengths of tension field along the compression and Tension flange respectively

$$s = 2/\sin\phi\left[M_{fr}/(f_y t_w)\right]^{0.5} \le c \qquad \text{... (5.7g)}$$

where,

$$M_{fr} = 0.25 b_f t_f^{\ 2} f_{yf}\left[1 - \left\{N_f/(b_f t_f f_{yf}/\gamma_{m0}\right\}^2\right] \qquad \text{... (5.8)}$$

where,

b_f, t_f = width and thickness of the relevant flange, respectively

f_{yf} = yield stress of the flange

5.4 DESIGN OF END PANELS

In simple design it may be assumed that the capacity of the end panel is restricted to $\tau_{cr,e}$ so that no tension field develops in it. In addition end stiffener should be capable of resisting shear force R_{tf}, moment M_{tf} and the reaction plus a compressive force due to the moment.

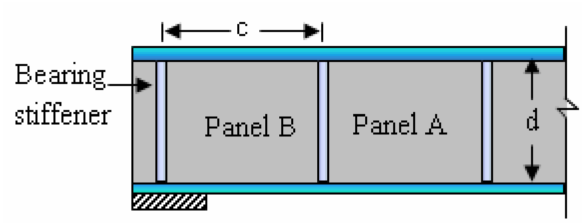


Fig.5.6 End panel design without tension field action

5.4.1 End panel design with tension field action

In design of end panel, provided with an end post consisting of a single or double stiffener which should play the roles of a bearing stiffener and end post.

In case of single stiffener, the top of the end post should be rigidly connected to the flange using full strength welds. The end post should be capable of resisting the reaction plus moment from the anchor forces equal to $2/3M_{tf}$

$$R_{tf} = \frac{H_q}{2} \text{ and } M_{tf} = \frac{H_q d}{10} \qquad \text{... (5.9a)}$$

where,

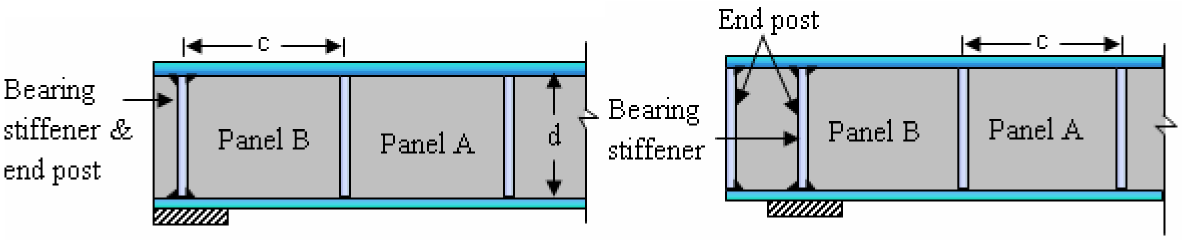
$$H_q = 1.25 dt \frac{f_y}{\sqrt{3}}\left(1 - \frac{V_{cr}}{P_v}\right)^{0.5} \qquad \text{... (5.9b)}$$

$$V_p = dtf_{yw}/\sqrt{3} \qquad \text{... (5.9c)}$$

If $V < V_{tf}$ then the values of H_q may be reduced by the ratio

$$\left(\frac{V - V_{cr}}{V_{tf} - V_{cr}} \right) \quad \text{... (5.9d)}$$

In case of double stiffener, the end post should be checked as a beam spanning between the flanges of the girder and capable of resisting a shear force R_{tf} and moment M_{tf} .



(a). Single stiffener (b). Double stiffener

Fig.5.7 End panel design with tension field action

5.5 STIFFENERS

For economical design it is usual to choose a relatively deep girder and such a girder having web quite slender. When the webs are inadequate to carry load, made strong and stable by the provision of a wide variety of stiffeners. Stiffeners are provided to transfer transverse concentrated compressive force on the flange into the web and are essential for desired performance of web panels. The stiffeners are classified as follows:

1. Intermediate transverse stiffener
2. Load carrying stiffener
3. Bearing stiffener
4. Torsion stiffener
5. Longitudinal Stiffener
6. Diagonal Stiffener
7. Tension Stiffener

5.5.1 Intermediate transverse stiffener

It is provided to improve the buckling strength of a slender web due to shear. In design of intermediate stiffener, it is necessary to satisfy following provision:

1. The outstand of the stiffener from the face of the web is restricted to a value of $20t_q\varepsilon$. If the outstand value is between $14t_q\varepsilon$ and $20t_q\varepsilon$, a core

value of $14t_q\varepsilon$ is chosen as the outstand value. t_q is thickness of the stiffener.

2. The effective length for intermediate transverse stiffener used in calculating the buckling resistance, F_{qd} should be taken as 0.7 times length, L of the stiffener.

3. The stiff bearing length b_1 is determined by considering the dispersion of the load at 45˙ through a solid material and a steel bearing element such as the bearing or flange plates as shown in Fig.5.8

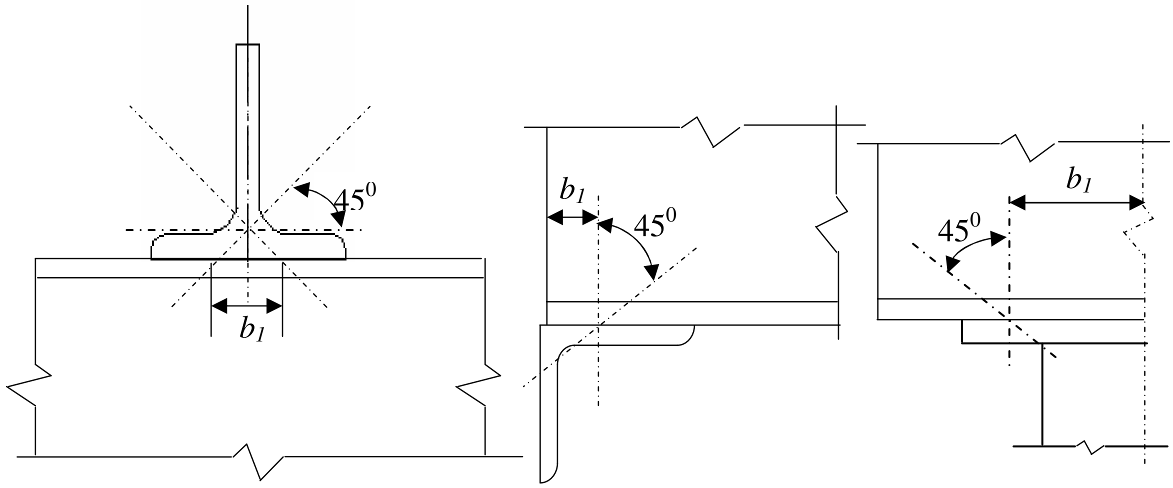


Fig.5.8 Stiff Bearing Length, b_1

4. The minimum requirement for intermediate stiffener, when they are subjected to external loads or moments is considered in terms of a second moment area I_s about the centre line of the web.

if $c/d < \sqrt{2}$, $I_s = 1.5(d/c)^2 dt_w{}^3$ … (5.10a)

if $c/d \geq \sqrt{2}$, $I_s = 0.75dt_w{}^3$ … (5.10b)

5. Buckling check on intermediate stiffeners
   - Stiffeners are not subjected to loads or moments should be checked for a stiffener force:

     $F_q = V - V_{cr}/\gamma_{m0} \leq F_{qd}$ … (5.11)

     where,

     V = factored shear force adjacent to the stiffener

V_{cr} = shear buckling resistance of the web panel designed without using tension field action

F_{qd} = Design resistance of intermediate stiffener

- If it is subjected:

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \le 1 \qquad \text{... (5.12)}$$

where,

F_{x} = external load or reaction at the stiffener

F_{xd} = design resistance of a load carrying stiffener corresponding to buckling about axis parallel to the web

M_q = moment on the stiffener due to eccentrically applied load

M_{yq}= yield moment capacity of the stiffener based on its elastic modulus about an axis parallel to the web

6. In connection of intermediate stiffener and web, shear between stiffener and the web should not be less than:

$$t_w{}^2 / (8b_s) \qquad \text{... (5.13)}$$

where,

t_w = web thickness (in mm)

b_s = outstand width of the stiffener (in mm) F_{cdw}

5.5.2 Load carrying stiffener

It is provided to prevent local buckling of the web due to concentrated loading or when reaction exceeds the buckling resistance F_{cdw} of the web alone. The buckling resistance of the web alone may be calculated by using the effective length and the section area of the stiffener. The area of cross-section is taken as $(b_1+n_1)\, t_w$.
where,

b_1 = width of stiff bearing on the flange (see Fig.5.8)

n_1 = dispersion of the load through the web at 45°, to the level of half the depth of the cross section (see Fig.5.9)

Load carrying web stiffener should also be of sufficient size so that the bearing strength of the stiffener, $F_{psd.}$

$$F_{psd} = A_q f_{yq} / (0.8\gamma_{m0}) \ge F_x \qquad \text{... (5.14)}$$

where,

F_x = external load or reaction

A_q = area of the stiffener in contact with the flange

f_{yq} = yield stress of the stiffener

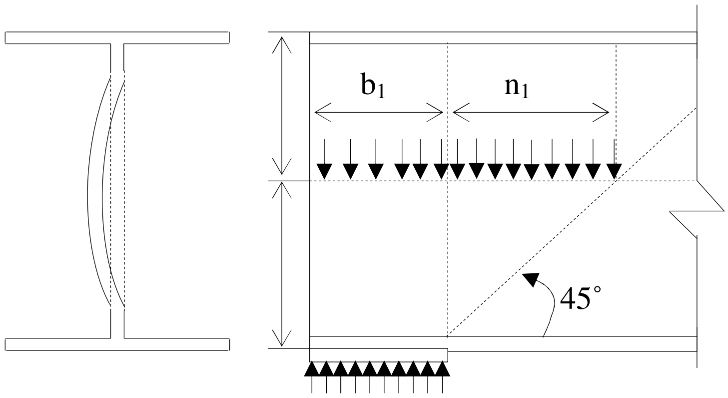


Fig.5.9 Effective width for web buckling

5.5.3 Bearing stiffener

Bearing stiffeners should be provided for webs where forces applied through a flange by loads or reactions exceed the local capacity of the web at its connection to the flange, F_w is calculated as follows:

$$F_w = (b_1 + n_2) t_w f_{yw} / \gamma_{m0} \quad \text{... (5.15)}$$

where,

b_1= stiff bearing length (see Fig.5.8)

n_2= length obtained by dispersion through the flange to the web junction at a slope of 1:2.5 to the plane of the flange (see Fig.5.10)

t_w = thickness of the web

f_{yw} = yield stress of the web

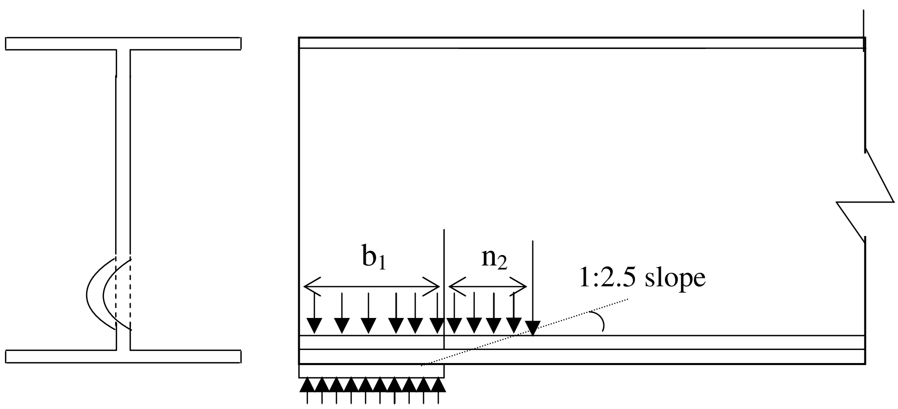


Fig.5.10 Effective width of web bearing

5.5.4 Torsion stiffener

In design of plate girder, bearing stiffener is provided that should be meeting the following criteria, to provide torsional restraint to beams and girders at supports.

I. The local capacity of the web is exceeded as calculated in eq.(5.15)
II. The second moment of area of the stiffener section about the centerline of the web, I_s.

$$I_s \geq 0.34\alpha_s D^3 T_{cf} \qquad \text{... (5.16)}$$

where,

$\alpha_s = 0.006$ for $L_{LT}/r_y \leq 50,$

$= 0.3/(L_{LT}/r_y)$ for $50 < L_{LT}/r_y = 100,$

$= 30/(L_{LT}/r_y)^2$ for $L_{LT}/r_y > 100$

D = overall depth of beam at support

T_{cf} = maximum thickness of compression flange of the span under consideration

L_{LT} = laterally unsupported effective length of the compression flange of the beam

r_y = radius of gyration about minor axis

5.5.5 Longitudinal Stiffener

For greater economy and efficiency in the design of plate girders, slender webs are often reinforced both longitudinally and transversely. The longitudinal stiffeners are most efficient when it is placed in the compression zones at about *d/5* from the compression edge of the girder. The main function of the longitudinal stiffeners is to increase the buckling resistance of web. In design of stiffener

$$I_s \geq 4ct_w^3 \qquad \text{... (5.17)}$$

where,

c = actual distance between the vertical stiffeners

In case of second stiffener provided at neutral axis

$$I_s \geq d_2 t_w^3$$

where,

d_2= twice the clear distance from the compression flange angles, plates or tongue plates to the neutral axis

5.5.6 Diagonal Stiffener

Diagonal stiffener should be designed to carry the portion of the applied shear and bearing that exceeds the capacity of the web. If the web and the stiffener have different design strength, the smaller value should be used for both.

5.5.7 Tension Stiffener

Tension stiffeners should be designed to carry the portion of the applied load or reaction less capacity of the web. Where the web and the stiffener have different design strength, the smaller value should be used for both.

5.6 CURTAILMENT OF FLANGE PLATES

For a plate girder subjected to external loading, the maximum bending moment occurs at one section usually, e.g. when the plate girder is simply supported at the ends, and subjected to the uniformly distributed load, then, maximum bending moment occurs at the centre. Since the values of bending moment decreases towards the end, the flange area designed to resist the maximum bending moment is not required at other sections. Therefore the flange plates may be curtailed at a distance from the centre of span where the plate is no longer required as the bending moment decreases towards the ends. It gives economy as regards to the material and cost. At least one flange plate should run for the entire length of the girder.

In section classification, flange is classified as plastic at centre and due to curtailment classified as semi compact or slender at support when there is only one plate.

5.7 DIFFERENT CODE COMPARESION FOR DESIGN OF PLATE GIRDER

Sr. No.	Topics	IS 800:2007	BS 5950-1:2000	AISC 360-05
1.	Sizing of girder	$D = 1/12\, to\, 1/18\, L$ $t_f = 0.3d$ Minimum $t_w = 6\, to\, 8\, mm$	$D = 1/8\, to\, 1/15\, L$ $t_f = 16\ to\ 40mm$ $t_w = D/150$	$D = 1/10\, to\, 1/12\, L$ Minimum $t_w = 6\, to\, 8\, mm$
2.	Serviceability	$t_w > d/200\varepsilon$ $t_w > d/345\varepsilon_f^{\ 2}$	$t_w > d/250$ $t_w > \dfrac{d}{250} \times \dfrac{p_{yf}}{345}$	-
3.	Shear buckling design	Tension field method $V_n = \left[A_v \tau_b + 0.9 w_{tf} t_w f_v \sin\phi\right] \le V_p$ $f_v = [f^2_{yw} - 3\tau^2_b + \Psi^2]^{0.5} - \Psi$ $\Psi = 1.5\tau_b \sin 2\phi$ $\phi = \tan^{-1}(d/c)$ $w_{tf} = d\cos\phi - (c - s_c - s_t)\sin\phi$ $s = 2/\sin\phi\left[M_{fr}/(f_y t_w)\right]^{0.5} \le c$ $M_{fr} = 0.25 b_f t_f^{\ 2} f_{yf} \left[1 - \{N_f/(b_f t_f f_{yf}/\gamma_{m0}\}^2\right]$	More exact method Shear buckling resistance V_b if the flanges of the panel are fully stressed $f_f = p_{yf}, V_b = V_w = dtq_w$ if $f_f < p_{yf}$ $V_b = V_w + V_f$ but $V_b < 0.6 p_y A_v$ $V_f = \dfrac{P_v(d/a)\left[1 - (f_f/p_{yf})^2\right]}{1 + 0.15(M_{pw} + M_{pf})}$	Shear strength with tension field action $i).\ h/t_w \le 1.10\sqrt{k_v E/F_y}$ $V_n = 0.6 F_y A_w$ $ii).\ h/t_w > 1.10\sqrt{k_v E/F_y}$ $V_n = 0.6 F_y A_w \left(C_v + \dfrac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right)$ $k_v = 5 + 5/(a/h)^2$ $k_v = 5\ when\ a/h > 3$ or $a/h > 260/(h/t_w)^2$ if $h/t_w \le 1.10\sqrt{k_v E/F_y}$, $C_v = 1$ if $1.10\sqrt{k_v E/F_y} \le h/t_w \le 1.37\sqrt{k_v E/F_y}$ $C_v = 1.10\sqrt{k_v E/F_y}\, /(h/t_w)$ if $h/t_w \ge 1.37\sqrt{k_v E/F_y}$ $C_v = 1.51 E k_v /((h/t_w)^2 \times F_y)$

4.	Check at End panel	Anchor force $R_{tf} = \frac{H_q}{2} \quad M_{tf} = \frac{H_q d}{10}$ $H_q = 1.25dt\frac{f_y}{\sqrt{3}}\left(1-\frac{V_{cr}}{P_v}\right)^{0.5}$ If $V < V_{tf}$ $H_q = 1.25dt\frac{f_y}{\sqrt{3}}\left(\frac{V-V_{cr}}{V_{tf}-V_{cr}}\right)\left(1-\frac{V_{cr}}{P_v}\right)^{0.5}$	Anchor force $H_q = 0.5dtp_y\left(1-\frac{V_{cr}}{P_v}\right)^{0.5}$ If $F_v < V_w$ $H_q = 0.5dtp_y\left(\frac{F_v-V_{cr}}{V_w-V_{cr}}\right)\left(1-\frac{V_{cr}}{P_v}\right)^{0.5}$ Compressive Force at end panel $F_{tf} = 0.15H_q(d/a_e)$	-
5.	Outstand of Stiffener	Outstand of stiffener $< 20t_q\varepsilon$ If it is $14t_q\varepsilon$ to $20t_q\varepsilon$ Design based on effective cross section with an outstand $14t_q\varepsilon$ Effective length of web = $20t_w$	Outstand of stiffener $< 19\varepsilon t_s$ If it is $13\varepsilon t_s$ to $19\varepsilon t_s$ Design based on effective cross section with an outstand $13\varepsilon t_s$ Effective length of web = $15t_w$	Outstand of stiffener at web end $12\varepsilon t_s$ and under concentrated load $25\varepsilon t_s$ Effective length of web $= 20t_w$
6.	Minimum stiffener	$c/d < \sqrt{2} \quad I_s = 1.5(d/c)^2 dt_w{}^3$ $c/d \geq \sqrt{2} \quad I_s = 0.75dt_w{}^3$	$a/d < \sqrt{2} \quad I_s = 1.5(d/a)^2 dt_{\min}{}^3$ $a/d \geq \sqrt{2} \quad I_s = 0.75dt_{\min}{}^3$	$I_s = at^3{}_w j$, $j = 2.5/(a/h)^2 - 2 \geq 0.5$

5.8 DESIGN OF PLATE GIRDER

Design Data

Total factored UDL on Beam = 55 kN/m

Factored point load p_1 = 270 kN

Factored point load p_2 = 270 kN

Distance of p_1 from left support = 7 m

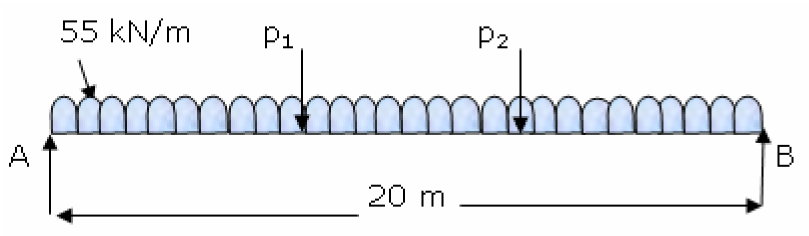
Distance of p_2 from left support = 13 m

Span of Beam = 20 m

Load Factor = 1.5

Resistance governed by yielding $\gamma_{m0} = 1.1$

E = 200000 N/mm^2



Step-1 Calculation of Maximum Bending moment and Shear Force

Bending moment = 4640 kN.m

Shear force = 820 kN

Step-2 Approximate Sizing of the section

Optimum depth of the plate girder = $(Mk / f_{yf})^{0.33}$ = 1404.8 mm

Assume depth of girder d = 1400 mm

Optimum value of thickness of web = $(M / f_{yf} k^2)^{0.33}$ = 8.25 mm

Assume thickness of web t_w = 8 mm

$A_f = M_z \times \gamma_{m0} / (f_y \times D)$ = 15190.5 mm^2

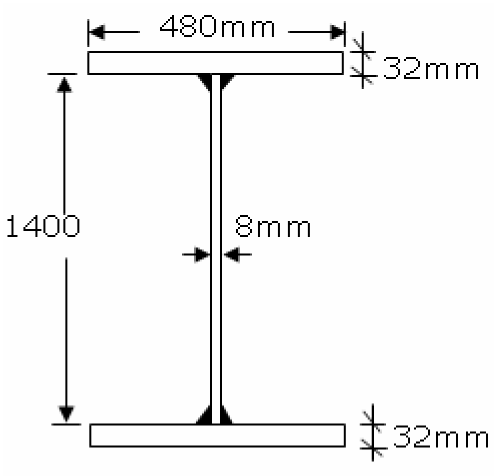
Approximate thickness of flange = $\sqrt{A_f / (2 * 8.4)}$ = 30.07 mm

Approximate width of flange = 505.18 mm

Assume thickness of flange t_f = 32 mm

Assume width of flange = 480 mm

The section is classified as a Semi-compact. (Ref. Table 2 of IS 800:2007)



Step-3 Check for moment capacity (*Ref. Cl.8.2.1.2)*

Considering flange only resist the bending moment

$$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2 Z_e f_y / \gamma_{m0}$$

M_d = 4694.91 kN.m ≤ 5633.89 kN.m

Design strength 4694.91 kN.m > 4640 kN.m required strength, $\therefore$Safe

Step-4 Check for shear resistance of web (*Ref. Cl.8.4)*

Check for resistance to Shear Buckling, $d / t_w < 67\varepsilon$

$d / t_w = 175 > 68.34,\ \varepsilon = \sqrt{\frac{250}{f_y}}$ = 1.02

Simple post critical method

$\tau_{cr,e} = k_v \frac{\pi^2 E}{12(1-\mu^2)} \left(\frac{t_w}{d}\right)^2$ = 31.58 N/mm^2, λ_w = 2.095, τ_b = 31.58 N/mm^2

$V_{cr} = A_v \tau_b$ = 353.7 kN < 820 kN

$\therefore$Intermediate stiffeners are required

Assume spacing of intermediate stiffener = 2000 mm

Step-5 Check for shear capacity of the end panel

$V_p = dtf_{yw} / \sqrt{3}$ = 1551.92 kN

$H_q = 1.25 V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{1/2}$ = 1609.76 kN

$R_{tf} = \frac{H_q}{2}$ = 804.88 kN < $V_n = \frac{A_v f_{yw}}{\sqrt{3} \times \gamma_{m0}}$ = 1410.83 kN

$\therefore$The end panel is safe to carry the shear due to anchoring force.

Step-6 Check for moment capacity of the end panel

$M_{tf} = \frac{H_q d}{10}$ = 225.37 kN.m, $M_q = \frac{I}{y} \frac{f_{yw}}{\gamma_{m0}}$ = 1163.64 kN.m

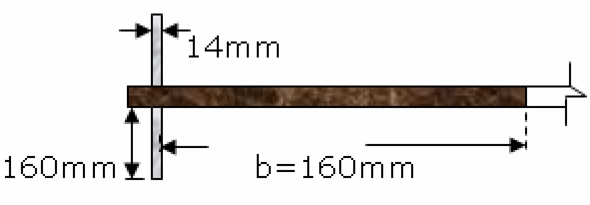
$\therefore$The end panel is safe to carry the bending moment due to anchoring force.

Step-7 Design of stiffener at support

Force due to M_{tf} = M_{tf} / c = 112.685 kN

Total compression force F_c = 820 + 112.685 = 932.685 kN

Assume size of stiffener b = 160 mm < $20 t_q \varepsilon$, t = 14 mm



I_{zz} = 4.12E+07 mm4, *A*= 95.87 mm, λ = 10.23, A = 4480 mm^2

f_{cd} = 217.954 N/mm^2 (from table 9(c))

Buckling resistance = 976.43 kN < 932.685 kN, $\therefore$Safe

Check stiffener as load bearing stiffener:

$F_w = (b_1 + n_2) t_w f_{yw} / \gamma_{m0}$ = 139.64 kN

Bearing stiffener is designed for = F_c - F_w = 932.685 - 139.64 = 793.05 kN

Bearing capacity of stiffener = 977.46 kN

At location of concentrated loads capacity of the web = 279.28 kN > 270 kN

Step-8 Design of stiffener at 2 m from support

$c / d \geq \sqrt{2}$ so minimum second moment of area

$I_s = 0.75 d t_w{}^3$ = 5.27E+05 mm^4

size of stiffener b = 50 mm < $20 t_q \varepsilon$, t = 8 mm, I_s = 8.39E+05 mm^4, $\therefore$Safe

Buckling check (*Ref. Cl.8.7.1.2*)

$$F_q = \left[\frac{V - V_{cr}}{\gamma_{m0}}\right] = 226.72 \text{ kN}$$

As per clause 8.7.1.5, an effective length of web equal to $20t_w$ on each side of the centre line of stiffener can be considered along with the stiffener.

A = 3360 mm^2, I_z = 8.53E+05 mm^4, λ = 61.49, f_{cd} = 160.765 N/mm^2

Buckling resistance = 540.18 kN > 226.72 kN, F_x = 270 kN

$$\left[\frac{F_q - F_x}{F_{qd}}\right] + \left[\frac{F_x}{F_{xd}}\right] + \left[\frac{M_q}{M_{yq}}\right] \le 1 = 0.42 < 1, \therefore \text{Safe}$$

Step-9 Design of Weld at web flange junction

$q_w = VA\,\bar{y}/2I_Z$ = 262.21 N/mm

Strength of weld = $0.7s\left(\frac{f_u}{\sqrt{3}\gamma_{mw}}\right)$, minimum size of weld = 2 mm

Step-10 Design of Weld for intermediate stiffener

q_1 = $t_w^{\,3}/5b_s$ = 0.08 kN/mm, available length of weld L= 1370 mm

q_2 = $V - V_{cr}/L$ = 0.25 kN/mm, q_w = 0.33 kN/mm

Minimum size of weld = 2 mm

Step-11 Design of web splices

Decide spacing of web splice

First splice at 6 m from support, Second splice at 10 m from support

B.M. at section a_1 = 1965 kN.m, S.F. at section a_1 = 245 kN

I_{zz} = 1.76E+10 mm^4 ,I_w = 1.83E+09 mm^4

Moment carrying by web M_w = 204.46 kN.m

Size of web splices(splice provided on both side of web)

Assume size of splice d = 1350 mm , t = 8 mm, b = 120 mm

$\bar{x}$ = 110.95 mm

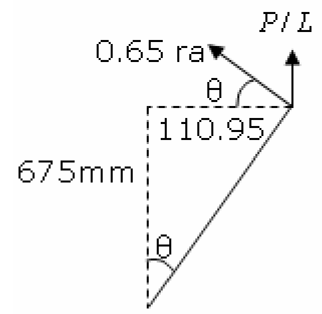
Resistance offered by the weld per mm length against translation

P/L = 154.09 N/mm

Resistance of the weld per mm length against rotation

S = Kr, I_{zz} = 3.14E+08 mm^3 , I_{yy} = 1.02E+06 mm^3

$K = \dfrac{M}{I_{zz} + I_{yy}} = 0.65$, $S = 0.65ra$



Total vertical component per mm length of weld

$V = p/L + S_a sin\theta$ = 226.21 N/mm

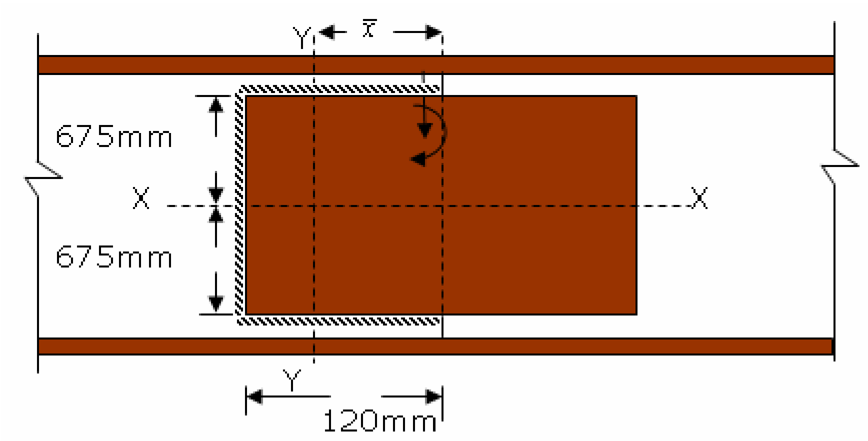
$H = S_a cos\theta$ = 438.75 N/mm

Resistance per mm length $R = \sqrt{V^2 + H^2}$ = 493.64

Assume size of weld = 6mm

Maximum shear stress intensity in the weld be

= 117.54 N/mm^2 < 189 N/mm^2 , $\therefore$Safe



B.M. at section a_2 = 2320 kN.m

S.F. at section a_2 = 0 kN

Provide plate size of d = 1350 mm , t = 8 mm, b = 140mm

Step-12 Design of flange splices

Decide spacing of web splice

First splice at 3 m from support, Second splice at 7m from support

B.M. at section a_1 = 2212.5 kN.m

To find out tensile and compression force carried by flanges

$\bar{y}_1 = 716$ mm, $\bar{y}_2 = 716$ mm, I_{zz} = 1.76E+10 mm^4

Tensile force = $\dfrac{M}{I_z} \times \bar{y}_1 \times A_{fb}$ = 1384.04 kN

Compression force = $\dfrac{M}{I_z} \times \bar{y}_1 \times A_{ft}$ = 1384.04 kN

Single-V groove weld, throat thickness = $5/8t_f$ = 20 mm

Effective length of weld = 480 mm

Strength of weld = $L_w t_e f_y / \gamma_{mw}$ = 1843.2 kN > 1384.04 kN , $\therefore$Safe

B.M. at section $a_2 =$ 4392.5 kN.m

Tensile force = 2747.76 kN

Compression force = 2747.76 kN

Double-V groove weld, throat thickness = 32 mm

Strength of weld = $L_w t_e f_y / \gamma_{mw}$ = 2949.12 kN > 2747.76 kN, $\therefore$Safe

5.9 Comparison of weight of section with varying load.

In present study, span of 20 m girder with two point load consider for study the variation in weight of section by decreasing and increasing the load.

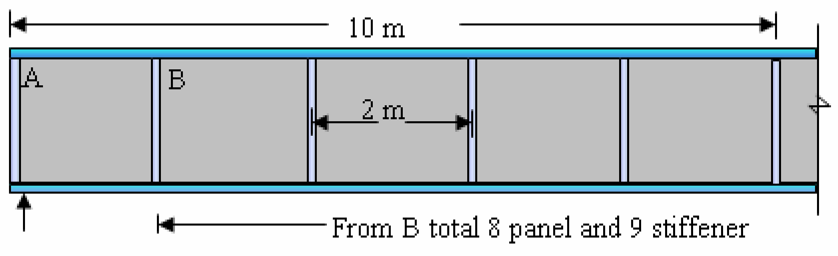


Fig.5.11 Stiffener arrangement in Plate Girder

Table 5.1 Comparison between weights of section with varying load

Multiply factor	Section (Span constant (20 m))					stiffener at A*		stiffener at B**		Web splice***			No. of stiffener A* from B**		stiffener (mm^3)		splice(mm^3)	kN/m^3
	d	t_w	b	t_f	mm^3	b	t	b	t	d_s	t_s	b_s	A	B	A	B		
0.2	900	6	250	16	268000000	80	6	0	0	850	8	60	4	0	3E+06		1632000	1.0577
										850	8	120					1632000	
0.4	1100	6	340	20	404000000	100	8	30	6	1050	8	80	2	9	4E+06	4E+06	2688000	1.6021
										1050	8	140					2352000	
0.6	1200	6	380	25	524000000	110	10	40	6	1150	8	100	2	9	5E+06	5E+06	3680000	2.0832
										1150	8	160					2944000	
0.8	1250	8	430	28	681600000	120	12	40	8	1200	8	160	2	9	7E+06	7E+06	6144000	2.7196
										1200	8	220					4224000	
1	1400	8	450	32	800000000	130	14	50	8	1350	8	160	2	9	1E+07	1E+07	6912000	3.2013
										1350	8	200					4320000	
1.2	1400	8	480	32	838400000	160	14	50	8	1350	8	240	2	9	1E+07	1E+07	10368000	3.3394
										1350	8	280					6048000	
1.4	1500	8	440	40	944000000	190	14	50	8	1450	8	220	2	9	2E+07	1E+07	10208000	3.7991
										1450	8	250					5800000	
1.6	1500	10	490	40	1.084E+09	210	14	60	8	1450	10	160	2	9	2E+07	1E+07	9280000	4.3516
										1450	10	220					6380000	
1.8	1600	10	520	40	1.152E+09	210	16	70	8	1550	10	160	2	9	2E+07	2E+07	9920000	4.6446
										1550	10	220					6820000	
2	1800	12	510	40	1.248E+09	240	16	90	8	1750	12	120	2	9	3E+07	2E+07	10080000	5.0723
										1750	12	200					8400000	

* stiffener at support, ** stiffener at 2 m from support, *** web splice at 6 m and 10m from support

6. SUMMARY AND CONCLUSIONS

6.1 SUMMARY

Necessity of revision of existing code which is done by Department of Civil Engineering, Indian Institute of Technology Madras and Institute for Steel Development and Growth (INSDAG), Calcutta is discussed in the introduction. The behaviour of steel beam in flexure, shear strength of beam, local buckling, lateral stability of beam and section classification is discussed in design of beam. Simply supported laterally restrained and unrestrained beam is illustrated using design steps. Flexural capacity of selected Indian rolled I section is calculated and charts for design are drawn. A comparison of load carrying capacity of section with same depth and same span is made which is presented in the form of bar charts for laterally restrained and unrestrained beams. In design of gantry girder, brief introduction of component of crane system and forces acting on gantry girder are discussed and the Fatigue effect is also considered. A comparison of design of gantry girder as per IS 800:2007 and IS 800:1984 is made. To prepare design aids spread sheets have been prepared for design of gantry girder, table is prepared for different load capacity with varying span for single crane and double crane. Based on capacity of sections bar charts are prepared for single crane and double crane. The ultimate behaviour of plate girder in detail when their webs are transversely stiffened is discussed in design of plate girder. Shear buckling design by simple post-critical method and tension field method is explained in detail. Intermediate transverse stiffener, load carrying stiffener, bearing stiffener, torsion stiffener, longitudinal stiffener, diagonal stiffener and tension stiffener are also explained. For design of plate girder, spread sheet included shear buckling design by simple post-critical method and tension field method. Design of flexural member comparison was made with codes like AISC 360-05, BS-5950, and Eurocode-3.

6.2 CONCLUSIONS

- Selected rolled sections are plastic which leads to economy in design using LSM.
- For same section and span, the load carrying capacity of laterally unrestrained beam decreases 50 to 60% compared to laterally restrained beam and as span increases this percentage also increases.

- The bar chart reveals that, load carrying capacity of ISWB increases as depth increases up to 500 mm depth. At 500 mm depth ISMB and ISWB both have the same capacity and beyond this stage ISMB have more load carrying capacity compared to ISWB.

- In design of gantry girder, bar chart shows that weight of section per m for lower capacity of crane is more as compare to higher capacity of crane. This is applicable for 16 m and 18 m crane girders of single crane and double crane of gantry girder span 6 to 12 m.

- In code comparison, it is revealed that in IS 800:2007 , many clauses are similar to BS 5950:1 and Eurocode-3

6.3 FUTURE SCOPE OF WORK

The present study can be extended to include following:

1. Design of castellated beam and its design aids.
2. Design of latticed beam.
3. Design of gantry girder for higher load, with built up section of varying depth.
4. Design of plate girder with corrugated webs.
5. Design of Box girder.
6. Parametric study for shear buckling design by simple post-critical method and tension field method in design of plate girder.

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4. S. Arul Mary, A. Shoba, "LRFD (Load Resisting Factored Method) format for the design of steel structures for Indian conditions", *Insdag's Steel in Construction,* Volume.5, No.2, July (2004), pp.41-52.

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10. N.Subramanian, *Design of Steel Structures*, Oxford University Press, New Delhi, 2008.

11. IS 800:2007, "General Construction in Steel-code of practice", Bureau of Indian Standards, New Delhi, 2007.

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13. BS 5950-1:2000, "Structural use of steelwork in building-1", British Standard, UK, 2003.

14. Taylor, Baddoo, Morrow, C. Gibbons, "steelwork Design Guide to Eurocode-3: part 1.1", The Steel Construction Institute, Ascot, 1999.

APPENDIX A

DESIGN OF GANTRY GIRDER

Design Data

Crane Capacity	= 100 kN
Self-weight of the crane girder excluding trolley	= 140 kN
Self-weight of trolley, hook etc.	= 35 kN
Minimum hook approach L_1	= 1.2 m
Distance between wheel centers c	= 3.5 m
Span of crane between the rails L_c	= 16 m
Span of gantry girder	= 6 m

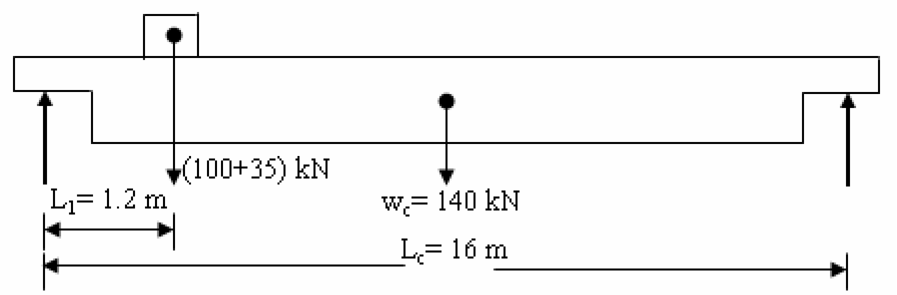
Yield stress of steel = 250 N/mm^2

E = 200000 N/mm^2

Step-1 Load and bending moment calculations

(1.1) Load

(i) Vertical load



Maximum static wheel load due to the weight of the crane = 35 kN

Weight of trolley and crane capacity w_t = 100+35 = 135 kN

Maximum static wheel load due to crane load = $[w_t(L_c - L_1)]/2L_c$ = 62.44 kN

Total load due to the weight of the crane and the crane load = 97.44 kN

Add for impact @ 25% = 97.44 x 1.25 = 121.8 kN

Factored wheel load W_c = 182.7 kN.

(ii) Lateral surge load

Lateral load (per wheel) = 10% (hook+crab load)/4 = 3.375 kN

Factored lateral load = 5.07 kN

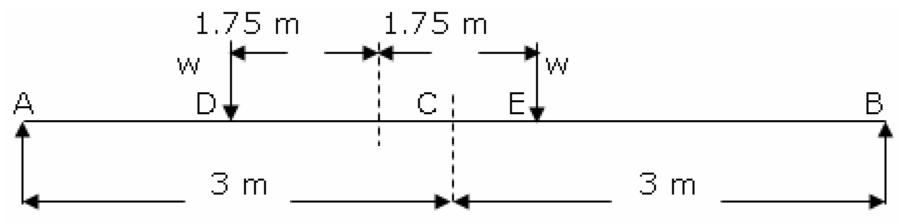
(iii) Longitudinal braking load

Horizontal force along rails = 5% of wheel load = 6.09 kN

Factored longitudinal load = 9.14 kN

(1.2) Maximum bending moment

(i) Vertical maximum bending moment (excluding self weight)



$M_E = 2W_c(L/2 - c/4)^2/L$ = 275.01 kN.m

M_c = 274.05 kN.m, $\therefore M$ = 275.01 kN.m

Assume, self weight of gantry girder = 1.7 kN/m

Self weight of rail = 0.3 kN/m

Total factored dead load = 3 kN/m

Bending at centre moment due to D.L. = 13.50 kN.m

(ii) Horizontal bending moment

M_{Ey} = 7.64 kN.m, M_{cy} = 7.61 kN.m, $\therefore M_y$ = 7.64 kN.m

(iii) Bending moment due to drag

Assume the rail height = 0.15 m

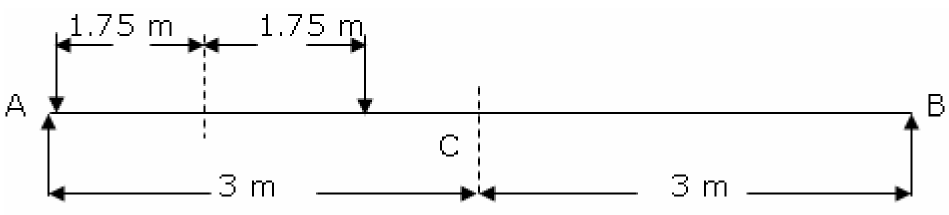
Reaction due to drag force = 9.14 X (0.3+0.15)/6 = 0.69 kN

$M_E = R(L/2 - c/4)/L$ = 1.46 kN.m

Total design bending moment M_z = 275.01+13.5+1.46 = 289.97 kN.m

(1.3) Shear force

(i) Vertical shear force



Reaction at A = $W_c(2-c/L)$ = 258.83 kN

Shear force due to dead load = $wl/2$ = 9.0 kN

Maximum ultimate shear force V_z = 258.83+9 = 267.83 kN

(ii) Shear force due to surge load

V_y = 5.07 (2 - 3.5/6) = 7.19 kN

Reaction due to drag force = 0.69 kN

Maximum ultimate reaction = 268.52 kN

Step-2 Preliminary selection of the girder

Choose the I, using deflection limit $L/750$

$$I = \frac{15.6W\,(L-c)}{LE}[2L^2 + 2Lc - c^2]$$

$= 4.03\text{E+08 mm}^4$

Required Z_p = $1.4M/f_y$ = 1.62E+06 mm^3

(2.1) Section properties

ISMB 450				ISLC 200			
w	=	0.7102	kN/m	w	=	0.202	kN/m
A_B	=	9227	mm^2	A_{ch}	=	2622	mm^2
h_b	=	450	mm	h_c	=	200	mm
b	=	150	mm	b	=	75	mm
t_f	=	17.4	mm	t_f	=	10.8	mm
t_w	=	9.4	mm	t_w	=	5.5	mm
I_{zz}	=	30391	cm^4	I_{zz}	=	1726	cm^4
I_{yy}	=	834	cm^4	I_{yy}	=	146.9	cm^4
R	=	15	mm	C_y	=	23.5	mm

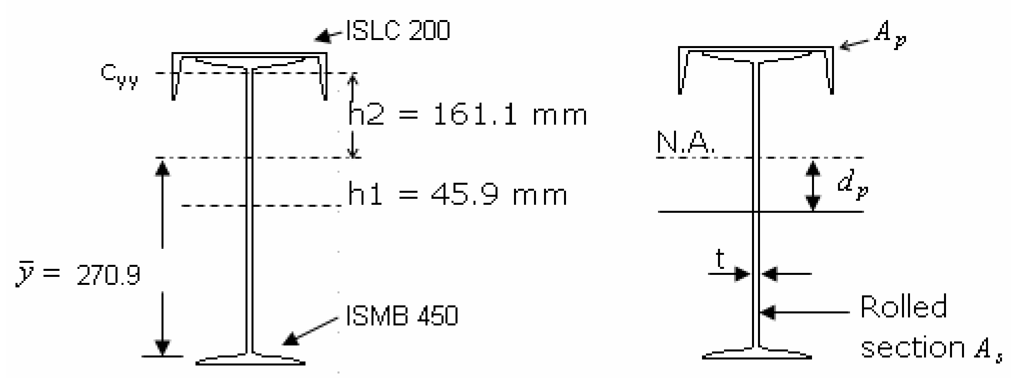
(i) Elastic properties of the combined section

Total area $A = A_B + A_{ch}$ = 9227+2622

= 11849 mm^2

The distance of N.A. of the built-up section

from the extreme fibre of tension flange, $\bar{y} = 270.9$ mm



$h_1 = \bar{y} - h_B/2 = 45.9$ mm

$h_2 = (h_B + t_{ch}) - \bar{y} - C_y = 161.1$ mm

$I_z = I_{ZB} + A_B h_1^2 + (I_y)_{ch} + A_{ch} \times h_2^2 = 3.93E+08\ mm^4$

z_z = 1.45E+06 mm^3, combined I_{yy} = 2.56E+07 mm^4

I_y for tension flange about the y-y axis I_{tf} = 4.89E+06 mm^4

For compression flange about the y-y axis I_{cf} = 2.21E+07 mm^4

z_y (for top flange alone) = 221487.5 mm^3

(2.2) Calculation of plastic modulus

The plastic N.A. divides the area into two equal areas = 5924.5 mm^2

$d_p = A_{ch}/2t$ = 2622/(2 x 9.4) = 139.47 mm

Depth of equal area axis from bottom flange

(450/2)+139.47 = 364.47 mm

Depth of equal area axis from top

(450+5.5) - 364.47 = 91.03 mm

Ignoring fillets, the plastic section modulus below the equal area axis

$\sum A\,\bar{y}$ = 1.49E+06 mm^3

Above the equal area axis = $\sum A\,\bar{y}$ = 3.99E+05 mm^3

z_{pz} = 1.89E+06 mm^3

For the top flange only z_{py} = 2.95E+05 mm^3

Step-3 Check for moment capacity

Section Classification ε = $\sqrt{\frac{250}{f_y}} = 1$

b / t_f of the flange of the I-beam = 4.05, Plastic

b / t_f of the flange of the channel-beam = 6.44, Plastic

d / t_w of the web of the of the I-beam = 44.18, Plastic

The section is classified as a Plastic

(3.1) Local moment capacity

$M_d = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2 Z_e f_y / \gamma_{m0}$, β_b = 1

$\beta_b Z_p f_y / \gamma_{m0}$ = 430.48 kN.m < $1.2 Z_e f_y / \gamma_{m0}$ = 395.52 kN.m

$\therefore$ Design moment $M_{d\,z}$ = 395.52 kN.m

Minor axis design moment

$\beta_b Z_p f_y / \gamma_{m0}$ = 67.02 kN.m < $1.2 Z_e f_y / \gamma_{m0}$ = 60.41 kN.m

$\therefore$ Design moment $M_{d\,y}$ = 60.41 kN.m

(3.2) Combined load capacity check

289.97/395.52 + 7.64/60.41 = 0.86, $\therefore$ safe

Step-4 Check for buckling resistance

$M_d = \beta_b Z_p f_{bd}$, β_b = 1

$$M_{cr} = C_1 \frac{\pi^2 E I_y h_f}{2(L_{LT})^2} \left[1 + \frac{1}{20} \left[\frac{L_{LT} / r_y}{h_f / t_f} \right]^2 \right]^{0.5}$$

h = 455.5 mm, L_{LT} = 6000 mm, t_f = 22.9 mm

r_y = 46.48 mm C_1 = 1.132

M_{cr} = 637.65 kN.m

Non-dimensional slenderness ratio

$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \leq \sqrt{1.2 Z_e f_y / M_{cr}}$

$\sqrt{\beta_b Z_p f_y / M_{cr}}$ = 0.87, $\sqrt{1.2 Z_e f_y / M_{cr}}$ = 0.83, λ_{LT} = 0.83

ϕ_{LTZ} = $0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$ = 0.91

χ_{LTZ} = $\frac{1}{[\phi_{LTZ} + (\phi^2_{LTZ} - \lambda^2_{LTZ})^{0.5}]} \leq 1$ = 0.778 < 1

f_{bd} = $\chi_{LT} f_y / \gamma_{m0}$ = 176.85 N/mm^2

$M_{d\,z}$ = $\beta_b Z_p f_{bd}$ = 334.98 kN.m

Design strength 334.98 > 289.97 kN.m $\therefore$ safe

Thus the beam is satisfactory under vertical loading .

Now, it is necessary to check it under biaxial bending.

(4.1) Check for biaxial bending

289.97/334.98 + 7.64/60.41 = 1, $\therefore$ safe

Step-5 Check for shear capacity

Check for Shear Buckling $d/t_w < 67\varepsilon$ = 44.17 < 67ε, $\therefore$ safe

$V_d = \dfrac{f_y h t_w}{\gamma_{m0}\sqrt{3}}$ = 555.05 kN

$0.6V_d$ = 333.03 kN > 267.83 kN, $\therefore$ safe

(5.1) Weld design

$\bar{y} = h_2$ = 161.1 mm, A = 2622 mm^2

$q = VA\bar{y}/I_Z = 287.97$ N/mm

This shear is taken by the welds.

Strength of weld = $0.7s\left(\dfrac{f_u}{\sqrt{3}\gamma_{mw}}\right)$

$\left(\dfrac{f_u}{\sqrt{3}\gamma_{mw}}\right)$ = 189 N/mm^2

Hence use a minimum weld of = 3 mm

Connecting the channel to the top flange of the I-beam.

$V_{dy} = \dfrac{f_y A_v}{\gamma_{m0}\sqrt{3}}$

A_v = (150 x 17.4 + 200 x 5.5) = 3710 mm^2

$0.6V_{dy}$ = 292.1 kN > 7.19, $\therefore$ safe

Step-6 Web buckling

Assume dispersion length b_1 = 150 mm

n_1 = 450/2 + 2*5.5 = 236 mm

Web slenderness ratio $\lambda = 2.5d/t = 102.447$

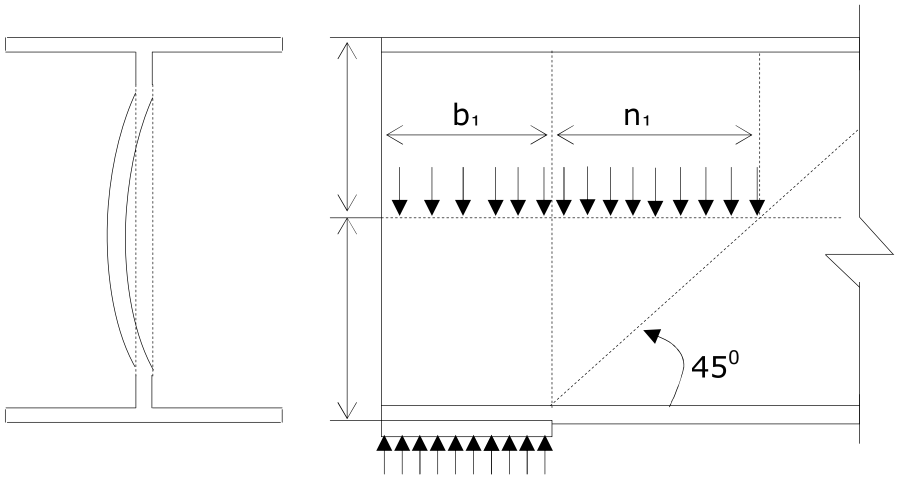
f_{cd} = 114.57 N/mm^2

Buckling resistance = $(b_1+n_1)\,t\,f_{cd}$ = 415.73 kN

Maximum wheel load = 182.7 kN, $\therefore$ safe

Step-7 Web bearing

Load dispersion at support with 1:2.5 dispersion



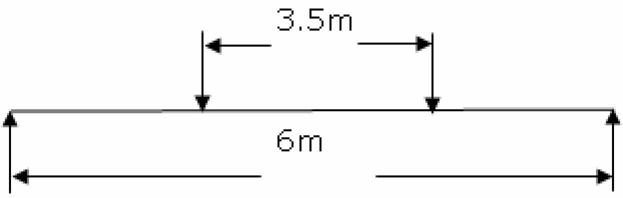
Minimum stiff bearing = $R_x/(tf_{yw}/1.1)-n_2$

n_2 = (17.4 + 15) x 2.5 = 81 mm, R_x = 268.52 kN, b_1 = 44.69 mm

Web bearing at support requires a minimum stiff bearing of 44.69 mm

Step-8 Check for deflection at working load

Serviceability vertical wheel load excluding impact = 97.44 kN



Deflection at mid-span = $WL^3[(3a/4L)-(a^3/L^3)]/(6EI)$

$a = (L-c)/2 = 1250$

(i) Vertical

Δ = 6.57 mm

Allowable maximum deflection = $L/750$ = 8 mm, $\therefore$ safe

(ii) Lateral

Only the compound top flange will be assumed to resist the applied lateral load as in the bending check

Δ = 4.04 mm

Allowable maximum deflection = 8 mm, $\therefore$ safe

Step-9 Fatigue strength (*Ref. section 13*)

The cranes operates = 200 days/year

= 8 hrs/day

Maximum trips of crane in 1 hour at maximum load level =3 per hrs

Design life of the building = 50 years

Category classification f_{fn} = 118

γ_{mft} = 1.15

Number of stress cycles N_{sc} = 240000 cycles < 5.0E+06

$f_f = f_{fn}\sqrt[3]{5*10^6/N_{sc}}$ = 324.69 N/mm^2

Design fatigue strength =324.69 /1.15 = 282.34 N/mm^2

Calculation of actual stress range:

f_{min} = 0, $f_{max} = M_z/Z_z$ = 199.95 N/mm^2

f = 199.95 N/mm^2 < 282.34 N/mm^2, $\therefore$ safe

Shear stress at support τ_f = 68.8 N/mm^2 > 27/1.15

$\therefore$ Fatigue assessment is required

Design fatigue strength in shear

$\tau_f = \tau_{fn}\sqrt[5]{5*10^6/N_{sc}}$ = 216.59/1.15 = 188.34 N/mm^2 >68.8 N/mm^2, $\therefore$ safe

APPENDIX B

LIST OF USEFUL WEBSITES

- www.steel-insdag.com
- www.steelstructure.com
- www.sefindia.com
- www.aisc.com
- www.asce.com
- www.steel.org.au
- www.childs-ceng.demon.co.uk
- www.designaids.com
- www.civil.usyd.edu.au

APPENDIX C

LIST OF PAPERS PUBLISHED

1. Hasmukh D. Panchani and G.N.Patel, "Design of Flexural Members (IS 800:2007) & Comparison with Other Codes and Design Aids", *National Conference on Advances and Innovation in Civil Engineering (AICE'09)*, Tamilnadu, 25^{th} March, 2009.

LIST OF PAPERS COMMUNICATED

1. Hasmukh D. Panchani and G.N.Patel, "Design of Flexural Members (IS 800:2007) & Comparison with Other Codes and Design Aids", *National Level Technical Festival (ENGINEER'09)*, Karnataka, 12-15 February, 2009. (Paper Selected)

2. Hasmukh D. Panchani, "Design of Gantry girder (IS 800:2007) & Comparison with Other Codes and Design Aids", *National Level Technical Festival (SAMANNVAY'09)*, Ahmedabad, 18-20 February, 2009. (Paper Presented)

3. Hasmukh D. Panchani and G.N.Patel, "Design of Gantry girder (IS 800:2007) & Comparison with Other Codes and Design Aids", *National Conference on Current Challenges in Civil Engineering (CCCE'09)*, Tamilnadu, 8^{th} May, 2009. (Abstract accepted)

4. Hasmukh D. Panchani and G.N.Patel, "Design of Plate girder-Impact of Shear Buckling (IS 800:2007)", *International Conference on Advances in Concrete, Structural and Geotechnical Engineering*, Bits Pilani, Rajasthan, 25-27 October, 2009. (Abstract accepted)