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HANDBOOK
FOR
STRUCTURAL ENGINEERS

2. STEEL BEAMS AND PLATE GIRDERS

BUREAU OF INDIAN STANDARDS

Price Rs 250.00

July 1962

STRUCTURAL ENGINEERS' HANDBOOK
No. 2

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BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

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SECTION V SPECIAL PROBLEMS IN BEAM AND GIRDER

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FOREWORD

This handbook, which has been processed by the Structural Engineering Sectional Committee, SMDC 7, the composition of which is given in Appendix C, has been approved for publication by the Structural and Metals Division Council of ISI.

Steel, which is a very important basic raw material for industrialization, had been receiving considerable attention from the Planning Commission even from the very early stages of the country's First Five Year Plan period. The Planning Commission not only envisaged an increase in production capacity in the country, but also considered the question of even greater importance, namely, the taking of urgent measures for the conservation of available resources. Its expert committees came to the conclusion that a good proportion of the steel consumed by the structural steel industry in India could be saved if higher efficiency procedures were adopted in the production and use of steel. The Planning Commission, therefore, recommended to the Government of India that the Indian Standards Institution should take up a Steel Economy Project and prepare a series of Indian Standard Specifications and Codes of Practice in the field of steel production and utilization.

Over six years of continuous study in India and abroad, and the deliberations at numerous sittings of committees, panels and study groups, have resulted in the formulation of a number of Indian Standards in the field of steel production, design and use, a list of which is included in Appendix B.

The basic Indian Standards on hot rolled structural steel sections are:

IS: 808-1957 SPECIFICATION FOR ROLLED STEEL BEAM, CHANNEL AND ANGLE SECTIONS

IS: 811-1961 SPECIFICATION FOR COLD FORMED LIGHT GAUGE STRUCTURAL STEEL SECTIONS

IS: 1161-1958 SPECIFICATION FOR STEEL TUBES FOR STRUCTURAL PURPOSES

IS: 1173-1957 SPECIFICATION FOR ROLLED STEEL SECTIONS, TEE BARS

IS: 1252-1958 SPECIFICATION FOR ROLLED STEEL SECTIONS, BULB ANGLES

IS: 1730-1961 DIMENSIONS FOR STEEL PLATE, SHEET AND STRIP FOR STRUCTURAL AND GENERAL ENGINEERING PURPOSES (Under print)

IS: 1731-1961 DIMENSIONS FOR STEEL FLATS FOR STRUCTURAL AND GENERAL ENGINEERING PURPOSES

IS: 1732-1961 DIMENSIONS FOR ROUND AND SQUARE STEEL BARS FOR STRUCTURAL AND GENERAL ENGINEERING PURPOSES

The design and fabrication of steel structures is covered by the following basic Indian Standards:

IS: 800-1956 CODE OF PRACTICE FOR USE OF STRUCTURAL STEEL IN GENERAL BUILDING CONSTRUCTION (Under revision)

IS: 801-1958 CODE OF PRACTICE FOR USE OF COLD FORMED LIGHT GAUGE STEEL STRUCTURAL MEMBERS IN GENERAL BUILDING CONSTRUCTION

IS: 806-1957 CODE OF PRACTICE FOR USE OF STEEL TUBES IN GENERAL BUILDING CONSTRUCTION

IS: 816-1956 CODE OF PRACTICE FOR USE OF METAL ARC WELDING FOR GENERAL CONSTRUCTION IN MILD STEEL

IS: 819-1957 CODE OF PRACTICE FOR RESISTANCE SPOT WELDING FOR LIGHT ASSEMBLIES IN MILD STEEL

IS: 823- CODE OF PROCEDURE FOR METAL ARC WELDING OF MILD STEEL (Under preparation)

IS: 1024- CODE OF PRACTICE FOR WELDING OF STRUCTURES SUBJECT TO DYNAMIC LOADING (Under preparation)

IS: 1261-1959 CODE OF PRACTICE FOR SEAM WELDING IN MILD STEEL

IS: 1323-1959 CODE OF PRACTICE FOR OXY-ACETYLENE WELDING FOR STRUCTURAL WORK IN MILD STEEL

In order to reduce the work involved in design computations, and to facilitate the use of the various Indian Standard Codes of Practice mentioned above, ISI undertook the preparation of a number of design handbooks. This handbook, which is the second in the series, relates to steel beams and plate girders. The first one on structural steel sections was published in March 1959. The third handbook which will cover steel columns and struts is under print. Other handbooks proposed to be published in the series in due course are expected to cover the following subjects:

1) Application of plastic theory in design of steel structures
2) Designing and detailing welded joints and connections
3) Design of rigid frame structures in steel
4) Economy of steel through choice of fabrication methods
5) Functions of good design in steel economy
6) High strength bolting in steel structures

7) Large span shed type buildings in steel
8) Light-weight open web steel joist construction
9) Multi-storey steel framed structures for offices and residences
10) Roof trusses in steel
11) Single-storey industrial and mill type buildings in steel
12) Steel transmission towers
13) Steelwork in cranes and hoists
14) Structural use of light gauge sections
15) Structural use of tubular sections

Metric system has been adopted in India and all quantities, dimensions and design examples have been given in this system.

This handbook is not intended to replace text books on the subject. With this object in view, theoretical treatment has been kept to the minimum needed. Special effort has been made to introduce only modern and practical methods of analysis and design that will result in economy in utilization of steel.

The information contained in this handbook may be broadly summarized as follows:

a) Explanation of the pertinent formulæ,
b) Design examples in a format similar to that used in a design office,
c) Commentary on the design examples, and
d) Tables of important design data.

In accordance with the main objectives, those types of beams and girder designs that lead to the greatest weight saving in steel have been emphasized as far as possible.

The calculations shown in the design examples have all been worked out using the ordinary slide rules. The metric sizes of rivets and plates incorporated in the design examples are likely to be the standard metric sizes which would be produced in this country. Indian Standards for these products are under preparation.

This handbook is based on, and requires reference to, the following publications issued by ISI:

IS: 226-1958 SPECIFICATION FOR STRUCTURAL STEEL (*Second Revision*)

IS: 800-1956 CODE OF PRACTICE FOR USE OF STRUCTURAL STEEL IN GENERAL BUILDING CONSTRUCTION (Under revision)

IS: 808-1957 SPECIFICATION FOR ROLLED STEEL BEAM, CHANNEL AND ANGLE SECTIONS

IS: 816-1956 CODE OF PRACTICE FOR USE OF METAL ARC WELDING FOR GENERAL CONSTRUCTION IN MILD STEEL

ISI HANDBOOK FOR STRUCTURAL ENGINEERS: 1. STRUCTURAL STEEL SECTIONS

In the preparation of this handbook, the technical committee has derived valuable assistance from Dr. Bruce G. Johnston, Professor of Structural Engineering, University of Michigan, Ann Arbor. Dr. Bruce G. Johnston prepared the preliminary draft of this handbook. This assistance was made available to ISI through Messrs. Ramseyer & Miller, Inc., Iron & Steel Industry Consultants, New York, by the Technical Co-operation Mission to India of the Government of USA under their Technical Assistance Programme.

The tabular material in Appendix A, a few photographs and quotations in sections VI and VII have been provided through the courtesy of the American Institute of Steel Construction, New York. An extract from the article by Mr. Henry J. Stetina as published in the Proceedings of the 1955 Conference of the Building Research Institute of Washington, D.C., has been quoted through the courtesy of the Building Research Institute of Washington, D.C.

No handbook of this type can be made complete for all times to come at the very first attempt. As designers and engineers begin to use it, they will be able to suggest modifications and additions for improving its utility. They are requested to send such valuable suggestions to ISI which will be received with appreciation and gratitude.

SYMBOLS

Symbols used in this handbook shall have the meaning assigned to them as indicated below:

A = Values obtained from Table XXI of IS: 800-1956 or Table III of this handbook

A_C = Area of the cover plate

A_f = Area of flange

A_w = Area of web

$\bar{A}_F$ = Clear area of flange of an I-Section after deducting an area for the portion of web assumed as extending up to the top of the flange

B = Values obtained from Table XXI of IS: 800-1956 or Table III of this handbook; Length of stiff portion of the bearing *plus* half the depth of the beam *plus* the thickness of flange plates (if any) at the bearing

$B_1, B_2, \ldots B_n$ = Various beams (*see* sketch on p. 32)

b = Width of flange

C = Permissible stress in the compression flange of the section with curtailed flanges or unequal flanges

c = Spacing (*see* p. 64)

C_{yy} = Distance of centre of gravity from the extreme fibre of the vertical leg of an angle or channel section

D = Overall depth (*see* sketch on p. 38)

d = Deflection, depth of beam or diameter of rivets

d_x = Depth at any section distant x from a reference point

$\frac{dy}{dx}$ = Slope [first differential of y (depth) with respect to x (the distance along the beam from a reference point)]

$\frac{d^2y}{dx^2}$ = Moment (second differential of y with respect to x)

$\frac{d^3y}{dx^3}$ = Shear (third differential of y with respect to x)

$\frac{d^4y}{dx^4}$ = Load (fourth differential of y with respect to x)

E = Young's modulus in tension or compression

e = Distance to either the extreme top or bottom of the beam from the neutral axis

f, f_b = Normal stress due to bending

f_c = Direct stress considered in perforated web beams

f_s = Shear stress

f_{sb} = Bending stress due to shear

f_v = Shear per linear cm (in welds)

F_b = Allowable bending stress in bearing plate

F_c = Allowable stress in direct compression

F_s = Allowable shear stress

F_{so} = Bending stress due to shear in a perforated web section

G = Shear modulus

g = Rivet gauge

h_2 = Distance from the root of vertical leg of fillet to top of flange

h_s = Splice plate height

h_w = Web height

h, $\bar{h}$ = Distance between centres of gravity of flanges; Economical web depth of a plate girder

I = Moment of inertia of the cross-section

I_{xy} = Product of inertia of the cross-section

K = A parameter used in the formula of economical web depth of a plate girder (*see* Eq 8); Torsional constant

k_1 = Coefficient of effective thickness of flange (*see* **E-2.1.1** of IS: 800-1956)

k_2 = Constant obtained from Table XX of IS: 800-1956

L = Span of beam; Angle section

l = Effective length of beam

l_{xx} = Effective length with respect to X-X axis

M = Bending Moment

M_a = Bending moment at centre of the beam due to reactions of other beams resting on it

M_t = Total maximum bending moment

M_w = Bending moment at centre of the beam due to beam weight only

M_c = Moment capacity of beam

M_T = Torsional moment

m, n = Assumed cantilever lengths in a perforated web section; Span ratios in continuous beams

N = The ratio of area of both flanges at the point of minimum bending moment to the area of both flanges at the point of maximum bending moment (*see* **E-2.1.1** of IS: 800-1956)

P_1 = Intensity of load distributed through the web and flange

P_2 = Bearing pressure

p = Pitch of rivets; Number of perforated panels

Q = Static moment about the centroidal axis of the portion of cross-sectional area beyond the location at which the stress is being determined

Q_{ba} = End reaction in a beam of simply supported span AB, at B

Q_{bc} = End reaction in a beam of simply supported span BC, at B

Q_B = $Q_{ba} + Q_{bc}$

q = Intensity of loading

R = Radius of curvature; Rivet strength; Reaction

r = Radius of gyration; Stress in rivet

r_m = Stress in the most stressed rivet caused by moment

r_v = Stress in the most stressed rivet, caused by shear force

S = Spacing of beams; Shear carrying capacity of beam; Spacing between intermittent welds

t = Thickness

t_e = Effective thickness of flange (*see* **E-2.1.1** of IS: 800-1956)

t_f = Flange thickness

t_w = Web thickness

uu, vv = The principal axes in the case of unsymmetrical sections

V = Total shear resultant on cross-section

W = Total load on a beam

w = Load intensity (*see* p. 43); Weld strength value; Width of a box section

x, y = Co-ordination of rivet centres from centre of gravity of the rivet group

X_c = Distance of centre of gravity from centre of web on X-X axis

X_s = Distance of shear centre from centre of web on X-X axis

y = Distance from neutral axis; Deflection

$\bar{y}$ = Distance of centre of gravity of the component section from the centre of gravity of the combined section

y' = Distance of centre of gravity of the component section from a reference point

Z = I/e = Section modulus

ϵ = Normal strain due to bending

ϕ = The change in slope $\left(\frac{d\theta}{dx}\right)$ per unit length of beam at any particular point

δ = Deflection

θ = Angle of twist per unit length

$d\theta$ = Rate of change of slope

℄ = Centre line

@ = At

$>$ = Greater than

$<$ = Less than

$\ngtr$ = Not greater than

$\nless$ = Not less than

$\leqslant$ = Less than or equal to

$\geqslant$ = Greater than or equal to

$\simeq$ = Approximately equal to

$\therefore$ = Therefore

ABBREVIATIONS

Some important abbreviations used in this handbook are listed below:

Units

Area in square centimetres	cm^2
Capacity of weld in kilogram per centimetre	kg/cm
Length in centimetres	cm
Length in metres	m
Length in millimetres	mm
Linear density in kilograms per metre per square centimetre	$kg/m/cm^2$
Load in kilograms	kg
Load in kilograms per metre	kg/m
Load in kilograms per square centimetre	kg/cm^2
Load in tonnes	t
Load in tonnes per metre	t/m
Moment in centimetre-kilograms	cm·kg
Moment in centimetre tonnes	cm·t
Moment in metre tonnes	m·t
Moment of inertia expressed in centimetre to the power of four	cm^4
Section modulus expressed in cubic centimetres	cm^3

Other Abbreviations

Alright	OK
Angle section	L
Bending moment	BM
Centre of gravity	CG
Centre to centre	c/c
Channel section	C
Dead load	DL

Equation	Eq
Indian Standard Angle Section conforming to and as designated in IS: 808-1957	ISA
Indian Standard Beam Sections conforming to and as designated in IS: 808-1957	ISLB, ISMB, etc
Indian Standard Channel Sections conforming to and as designated in IS: 808-1957	ISLC, ISMC, etc
Live load	LL
Neutral axis	NA
Number	No.
Shear force	SF
Single shear	SS
Wide flange beam	WB

SECTION I

GENERAL

1. INTRODUCTION

1.1 A beam or girder may be defined as a structural member, usually straight, that has the primary function of carrying transverse loads from specified points in space to specified points of support. An arch also carries transverse loads from points in space to points of support, but the normal stress in the cross-section through the interior of the arch is primarily compression. A suspension bridge also carries loads from points in space to points of support but the normal stress in the cross-section through the suspension rope is primarily tension. In the case of the suspended span and the arch span (considering only vertical loads) the supporting reactions are inclined with respect to the vertical, hence, depend on a lateral component of force that shall be provided by the foundation. In the case of the beam under transverse loads, the reactive forces at the supports are in the same direction as the applied loads and the normal ' bending ' stress on the beam cross-section varies linearly from a maximum compression to a maximum tension.

1.2 By far the greatest number of beams are designed to act in ' simple bending ' and the design of rolled sections for simple bending is covered in Section II. Plate girder design for simple bending is treated in Section III. Whenever feasible, for greatest economy in design, beam sections should be chosen, braced (if necessary), and oriented with respect to the specified loads so that the assumptions of simple bending are justified.

1.3 Simple bending is that type of bending in which the loads and the support reactions are in one and the same plane and the longitudinal axis of the beam remains in that same plane as the beam deflects. It is assumed that the cross-section of the beam does not twist during deflection. If simple bending is to be insured when an I-beam is loaded in the plane of its web, the compression flange either shall be supported laterally or the permissible stress (in some cases) shall be reduced to prevent the possibility of lateral buckling. But simple bending occurs naturally, without lateral support, in such cases as are shown in cross-sections given in Fig. 1.

In Fig. 1, it will be noted that in each case the plane of the loads coincides with an axis of symmetry of the cross-section. It is important to recognize the conditions under which simple bending will occur and the precautions that shall be observed in design of details and supports for other cases where simple bending is not natural although it may be forced or insured by special means. For example, the channel, used as a beam

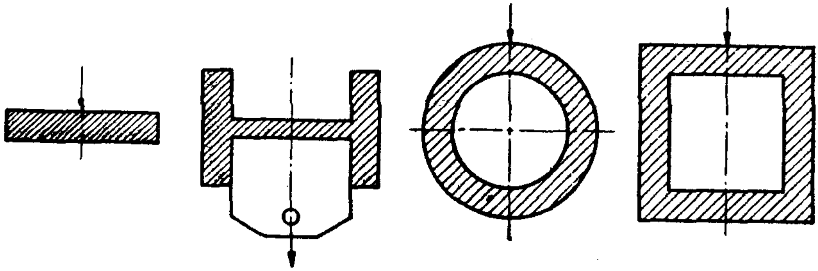
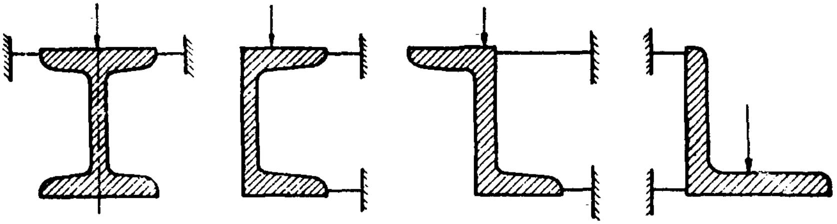


FIG. 1 CROSS-SECTIONAL SHAPES AND LOADING PLANES NATURALLY CONDUCIVE TO SIMPLE BENDING

with loads applied in the plane of its major principal axis, will twist and so also the common angle. Such complications in simple bending are treated separately in Section V. In spite of possible complications, simple bending is most often encountered in actual design because the widely used I-section steel beam shown in Fig. 2A requires but very little lateral support to insure against the possibility of lateral buckling.



2A 2B 2C 2D

FIG. 2 SUPPORT REQUIREMENTS TO PROVIDE SIMPLE BENDING

Simple bending may also be induced in the channel, loaded as shown in Fig. 2B, if restraint against twist and lateral buckling is provided along the member (*see* **30.1**). If an angle is loaded as shown in Fig. 2D, provision along the angle shall be made not only to prevent twist but to prevent lateral deflection out of the plane of the loads (*see* **29.1**). Where the lateral support is needed for stability alone, as in the case shown in Fig. 2A, there is no calculable stress in the lateral supports. In cases shown at Fig. 2B and 2C, however, there is a definitely calculable stress in the restraining members, thus a more clearly defined design problem exists.

1.4 The primary function of the beam is to carry transverse loads and the ability of the beam to perform its function is judged primarily by the adequacy of the beam cross-section at every point, along the axis to resist the maximum shear and moment that may occur at that section.

In the design of a beam under complex loading conditions the *shear diagram* and *bending moment diagram* are usually plotted (*see* Fig. 3). It is assumed that the reader is familiar with the determination of such diagrams. Reference may also be made herein to Illustrative Design Examples 1 and 2 and to Section IV.

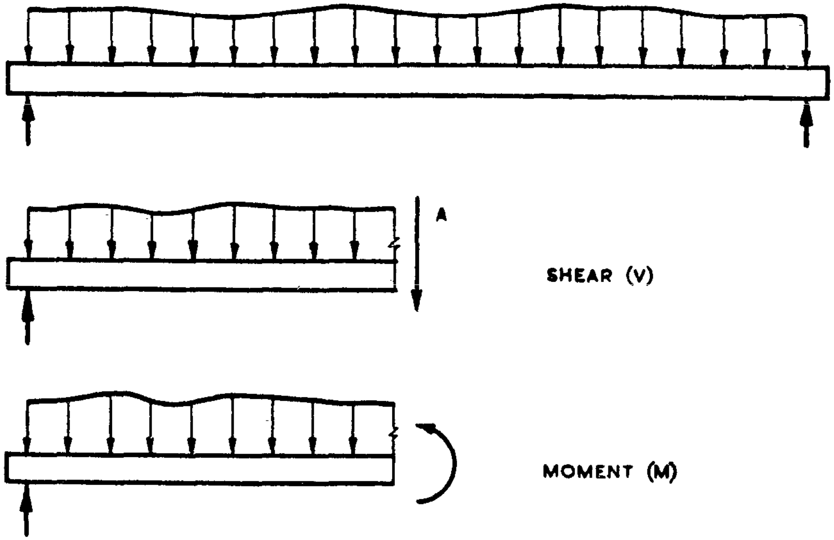


FIG. 3 POSITIVE LOAD, SHEAR AND MOMENT IN BEAMS

1.5 The design of a beam is considered adequate for bending moment and shear if the maximum normal stress due to bending and maximum shear stress due to shear are kept within specified limits that insure a factor of safety with respect to yielding. In simple beam theory, the normal strain parallel to the longitudinal axis of the beam is assumed proportional to the distance from the neutral axis of bending — an exact hypothesis (circular bending) in the absence of shear stress and a close approximation for most practical cases even when shear exists. As shown in texts on strength of materials; the normal stress is given by:

$$f_b = E\epsilon = E\phi y \qquad \ldots\ldots\ldots\ldots\ldots \text{(1)}$$

where

f_b = normal stress due to bending,

E = elastic modulus in tension or compression,

ϵ = normal strain due to bending,

ϕ = the change in slope ($d\theta/dx$) per unit length of beam at any particular point, and

y = distance from neutral axis (axis of zero normal stress).

In most texts, in place of ϕ, $1/R$ is written, where R is the radius of curvature. No one is able to see a radius of curvature in nature but, in observing a very flexible beam under load, such as the swaying branch of a tree, one may actually observe deflection, changes in slope and even curvature. Thus, it is intrinsically better to write the equation in terms of ϕ rather than $1/R$.

It is occasionally necessary to calculate the deflections of a beam and a knowledge of ϕ all along the beam leads first to a calculation of beam slope at any point — thence (as will be demonstrated in Section IV) to a calculation of deflections.

As shown in Fig. 4, ϕ is the angle between tangents to the axis of the beam at points one unit in length apart, hence, it represents *the change in slope per unit length.*

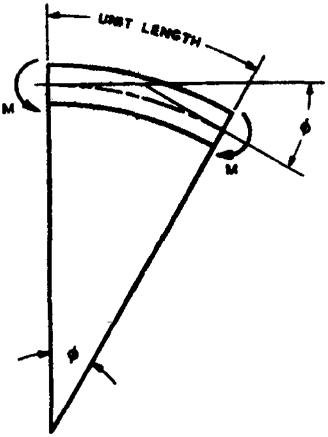


FIG. 4 UNIT LENGTH OF BENT BEAM

To obtain the familiar equation for normal stress due to bending, ϕ in Eq 1 shall be related to the bending moment, M. Below the yield point of steel (in the elastic range) there is a linear relationship between bending moment and beam curvature (a special case of Hooke's Law) which may be expressed as follows:

> The amount that a beam bends is proportional to the bending moment. The constant of proportionality, as derived in text books on strength of materials, is EI, the bending stiffness of the beam, and
>
> $$M = EI\phi \quad \dots\dots\dots\dots\dots \quad (2)$$
>
> where
>
> M = bending moment, and
> I = moment of inertia of the cross-section.
>
> In the limit, if variable, the angle change rate per unit length may be expressed more precisely in the language of differential calculus by introducing:
>
> $$\phi = \frac{d}{dx}\left(\frac{dy}{dx}\right) = \frac{d^2y}{dx^2} = -\frac{M}{EI} = -\frac{f_b}{Ey} \quad \dots\dots\dots \quad (3)$$
>
> The interrelationship between deflection, slope, moment, shear, and load may be summarized conveniently by functions of 'x', taken as the distance (to the right) along the beam:
>
> y = deflection (assumed positive downward),
> $\frac{dy}{dx}$ = slope (positive when y increases with increasing x),
> $\frac{d^2y}{dx^2} = +\frac{M}{EI}$ (moment assumed positive when normal stress is compression at top of beam) (*see* Fig. 3). . . . (4)
> $\frac{d^3y}{dx^3} = -\frac{V}{EI}$ (shear positive as shown in Fig. 3), and
> $\frac{d^4y}{dx^4} = +\frac{q}{EI}$ (load positive when down).

For various typical end conditions and load distributions, the solution of Eq 4 to provide equations for deflections, location and magnitude of maximum deflections, etc, reference may be made to any Structural Handbook.

In conventional, or elastic design, the adequacy of a beam to carry bending moment and shear is determined by limiting the maximum normal stress due to bending, and the *average* shear stress (assuming the web to take all of the shear) to the prescribed ' allowable stresses ' that provide a margin of safety with respect to the elastic limit or yield point strengths of the material.

The familiar equation for normal stress due to bending is obtained by combining Eq 1 and 2:

$$f_b = \frac{My}{I} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

If 'e' is designated as the distance y to either the extreme top or bottom of the beam, the maximum normal stress due to bending is:

$$f_b = \frac{Me}{I} = \frac{M}{Z} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

$Z = \frac{I}{e}$ and is termed the 'section modulus'.

The shear stress at any point of the cross-section is given by:

$$f_s = \frac{VQ}{It} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

where

f_s = shear stress,

V = total shear resultant on cross-section,

Q = static moment about the centroidal axis of the portion of cross-sectional area beyond the location at which the stress is being determined,

I = moment of inertia of the section about the centroidal axis, and

t = thickness of web.

IS: 800-1956 limits the maximum normal stress on a steel beam cross-section to 1 575 kg/cm^2 (*see* **9.2**) and the average shear stress (when web buckling is not a factor) is limited to 945 kg/cm^2 (*see* **9.3.2**). The average shear stress is calculated by dividing the resultant shear force (V) on the cross-section by the gross cross-section of the web, defined for rolled I-beams and channels (*see* **20.6.2.1** and **20.6.2.2** of IS: 800-1956) as 'the depth of the beam multiplied by the web thickness' and in the case of plate girders 'the depth of the web multiplied by its thickness'.

Table I (*see* p. 169) gives a convenient order for economical selection of the section moduli and shear capacity for the IS Rolled I-beams and channels.

Although not of direct use in design it is desirable to recognize that the normal stress as given by Eq 5 and the shear stress as given by Eq 7 are simply *components of the resultant stress that, in general, acts at an angle to the plane of the cross-section.* At the top and bottom of the beam the resultant stress and the normal stress become equivalent, since the shear stress is zero, and at the neutral axis of the beam, where the normal stress is zero, the resultant stress is the shear stress.

2. DESIGN PROCEDURE AND CODE OF PRACTICE

2.0 The foregoing discussion of simple beam theory presents merely a sketch of some of the more important facts. For a complete development of the theory of simple bending, reference should be made to reference books on strength of materials by such authors as Morley, Timoshenko, or others. Attention so far has been given primarily to bending moment and shear. Beams of normal proportions are usually selected on the basis of bending moment and a routine check made as to their shear capacity. Only in the case of very short beams, or beams in which high concentrations of load near one or both ends, will the shear control the design. In addition to shear and bending moment, however, there are a number of secondary factors that need to be checked in any beam design. These will be discussed very briefly in this 'Section' with complete reference in IS: 800-1956 and actual design details in succeeding sections.

2.1 In some cases, deflection limitations may affect the beam design. A beam that experiences large deflections is a flexible beam and is undesirable in locations where the loads are primarily due to human occupancy, especially in the case of public meeting places. Large deflections may result in noticeable vibratory movement producing uncomfortable sensations on the part of the occupants and in some cases loading toward cracking of plastered ceilings if these exist. The question as to what actual deflection will cause plaster cracking or whether the deflection itself is a primary cause is a debatable one but the usual specification limitations, no doubt, have their place even though they are not usually mandatory.

In addition to a check on deflections, safety against the local crushing or buckling of the web of a rolled beam should be checked at the ends and at points of concentrated load. In some cases stiffeners may have to be introduced.

2.2 When the available rolled beam sections become inadequate to carry the load, there are a number of alternatives leading to sections of greater bending moment capacity. One may go directly to a welded or riveted plate girder or, alternatively, flange plates may be welded or riveted to the flanges of available rolled sections. Another possibility is the use of a split-section formed of two T-sections with a web plate welded in between. This will provide a deeper beam section and will require somewhat less welding than a completely built-up plate girder. Other possibilities that should be considered are the use of continuous beams instead of simple beams, or use of plastic design, where applicable. The use of open web beams, tapered beams, or composite beams, offer other modifications of design to provide greater bending strength with the utilization of existing Indian rolled shapes. These alternatives to conventional design are discussed in Sections VI, VII and VIII.

The use of continuous beams should be considered in roof construction, for crane runway girders, or for other types of construction in which it is convenient to run the beam continuously over columns or other points of support.

2.3 The possibility of using plastic design becomes especially important when one goes to continuous beam or frame construction. IS: 800-1956 takes some account of the increased plastic reserve strength in bending beyond the yield point in the fact that 1 575 kg/cm^2 is permitted for rolled sections whereas the stress in plate girders with little plastic reserve is limited to 1 500 kg/cm^2. However, in continuous construction, reserve strength is available from another and greater source — redistribution of bending moment as ' plastic hinges ' develop. The plastic design method may be used advantageously limiting conditions. Reference should be made to the ISI Handbook for Structural Engineers on Application of Plastic Theory in the Design of Steel Structures (under preparation) for a more complete discussion of this design procedure. Plastic design should probably not be used when repeated loads are an important factor leading to the possibility of fatigue failure. Special attention also may be given in plastic design to modifications in the usual specification requirements for outstanding plate elements under compressive stress since local buckling should not only be avoided in the elastic range but prevented in the plastic range up to the inception of strain hardening. However, serious consideration should be given to plastic design of continuous beams and rigid frames of one- or two-storey height when fatigue is not a problem and only a few maximum loads are expected.

In the design of beams subject to severe repeated load stressing, the beam near maximum permissible limits with many expected repetitions, such as in the design of a crane runway support girder, stresses should be reduced to prevent possibility of fatigue failure.

Crane runway support beams and beams in similar situations are also subject to impact which sets up elastic vibrations and thereby increases the stresses. These additional stresses are taken care of by the use of impact factors and the crane runway support girder serves as a design illustration in Design Example 10.

SECTION II

DESIGN OF ROLLED BEAMS

3. GENERAL

3.1 Generally the following are the essential steps required in the selection of symmetrical I-shaped rolled steel beams:

a) Selection for bending moment and shear,
b) Lateral support requirements,
c) Design of beams without lateral support,
d) Deflection requirements,
e) Shear stress in beams,
f) Web crippling and buckling, and
g) End connections and bearing plate design.

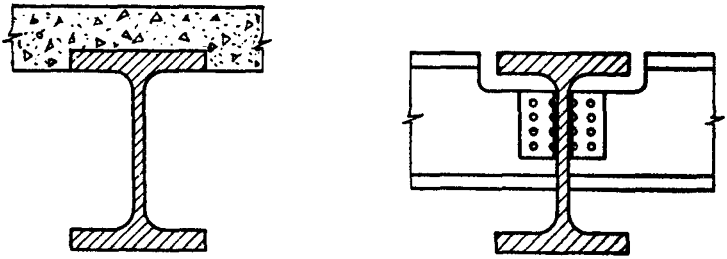
A general discussion of these steps is given in **4** to **10** and is followe by Design Example 1 (*see* **11**) in which the designs of beams for a specified floor framing plan are presented.

4. SELECTION FOR BENDING MOMENT AND SHEAR

4.1 As pointed out in Section I, the primary function of a beam is to carry load. The moment and shear capacity at every point along a beam shall, therefore, be greater than the actual moments and shears caused by the load. It is assumed the reader is familiar with the calculation of moment and shear diagrams as covered later in Section IV, and with general theory of simple bending as previously discussed in Section I. To facilitate the actual selection of a beam after the maximum moments and shears have been determined, Table I has been prepared listing all of the IS Rolled I-beams and channels in the order of their moment capacity. The index of moment capacity is the section modulus ' Z ' which is given in col 1 of Table I (*see* p. 169). Thus, as will be demonstrated in Design Example 1 after the required section modulus is determined, one may immediately select from the table the beam that will have the smallest weight per metre for the moment capacity needed by following the steps indicated in the note under the table. Except in the case of very short beams or beams carrying heavy loads near their ends, moment rather than shear will govern the design. However, it is convenient to list in the same beam selection table the maximum shear value of each beam in tonnes (*see* col 4 of Table I). Thus, after selecting the beam for moment one may immediately check its shear capacity. The standard designation of the rolled beam is given in col 2 of the table and its weight in kilograms per metre in col 3.

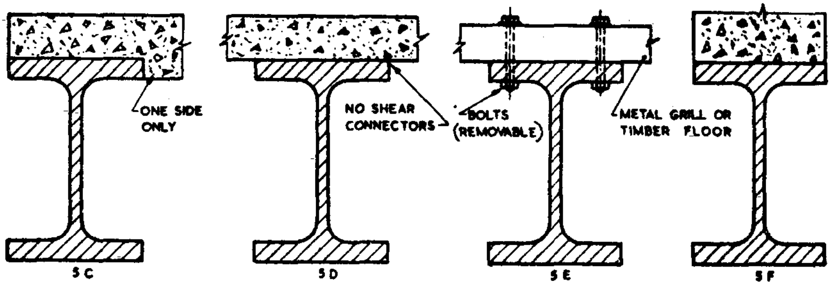
5. LATERAL SUPPORT REQUIREMENTS

5.1 The great majority of beams are designed as 'laterally supported' in which case no reduction in allowable stress due to bending is required to safeguard against lateral-torsional buckling. Any beam encased with concrete which is in turn contiguous with at least one adjacent slab may be considered as fully supported laterally. Other conditions of lateral support may be more questionable and some of these are indicated in Fig. 5. Full lateral support should be credited if a concrete slab encases the top flange so that the bottom surface of the concrete slab is flush with the bottom of the top flange of the beam. If other beams frame at frequent intervals into the beam in question, as indicated in Fig. 5B, lateral support is provided at each point but the main beam should still be checked between the two supports.



5A CONTINUOUS 5B LOCAL

ADEQUATE LATERAL SUPPORT



5C 5D 5E 5F

INADEQUATE LATERAL SUPPORT

FIG. 5 LATERAL SUPPORT REQUIREMENTS

5.2 No lateral support should be credited if the concrete slab holds the top flange of the beam from only one side as in Fig. 5C, or simply rests on the top as in Fig. 5D without any shear connectors or bond other than the surface between the two materials. Temperature change and deflection due to bending will destroy the bond leaving the beam with only friction to depend upon for top lateral support. Similarly, if plank or bar grating is attached to the top flange by means of bolts as in Fig. 5E, the support might be temporarily adequate if bolts were firmly fastened and the opposite ends of plank or grating securely attached to some other support. However, owing to the temporary nature of the connections, full dependence should not be given as there is always a possibility that the bolts might be omitted or removed. In this case, the design should be made as if lacking lateral support. The matter of designing beams without lateral support is covered in **6**.

6. DESIGN OF BEAMS WITHOUT LATERAL SUPPORT

6.1 When special conditions require that a beam be loaded in the plane of the web, without continuous or intermittent lateral support at sufficiently frequent intervals, the beam will ultimately fail by buckling with lateral and torsional deflections. In order to provide adequate safety against such buckling, the allowable stress is reduced in certain cases. The reduction in allowable stress increases with increasing l/b ratio and d/t_f ratio where:

l = unsupported length of beam,
b = width of flange,
d = depth of beam, and
t_f = flange thickness.

6.2 Permissible stresses are tabulated for various ratios in Table II of IS: 800-1956. The formula on which these values are based is given in Appendix E of IS: 800-1956 and tabulated values apply only to rolled beams of constant cross-section and of symmetrical I-shape. The formulæ may be applied to channels with over-safe results. For beams with variable flange shape, unequal flanges, etc, reference should be made to Appendix E of IS: 800-1956.

6.3 In certain parts of the tables in IS: 800-1956, it is difficult to interpolate properly. To overcome this deficiency elaborate tables showing permissible stresses for closer intervals of l/b (or l/r_y) and d/t_f (or d/t_e) (*see* Tables II and III on p. 172 and 174) are given in this handbook. Examples of the application of Table II are given in Design Example 1. Example of the application of Table III is given in Design Example 10.

7. DEFLECTION REQUIREMENTS

7.1 Recommended deflection limitations for beams and plate girders are given in **20.4** of IS: 800-1956. When rigid elements are attached to beams or girders, the specification calls for a maximum deflection of not more than 1/325 of the span. However, this may be exceeded in cases where no damage due to deflection is possible.

7.2 If a structure is subject to vibration or shock impulses, it may be desirable to maintain reasonable deflection limits such as will produce a stiff structure less apt to vibrate and shake appreciably. For example, excessive deflection in crane runway support girders will lead to uneven up and down motion of the crane as it proceeds down the building. Impact stresses in such case would be increased.

7.3 Possibility of excessive deflection will arise when a rather long span carries a very light load for which a relatively small beam size is required. Such a situation might exist, for example, in a foot bridge. The matter of deflections is very largely left up to the judgement of the engineer.

7.4 Very long beams subject to large deflections, such as the open joist type, are usually cambered so that unsightly sag will not be noticeable when the beams are fully loaded.

8. SHEAR STRESS IN BEAMS

8.1 The subject of shear stress has been discussed in Section I. It is to be noted that in the case of rolled beams and channels the design shear is to be figured as the average shear obtained by dividing the total shear by the total area of the web computed as (d) (t_w). In more complex beam problems such as those with cross-section unsymmetrical about the X-X axis, the more exact expression for the calculation of shear stress or shear per running metre should be used. The more exact expression should also be used in calculating horizontal transfer of shear by means of rivets or welds. The design example will illustrate these calculations.

9. WEB CRIPPLING AND BUCKLING

9.1 When a beam is supported by bearing pads or when it carries concentrated loads, such as columns, it shall be checked for safety against web crippling and web buckling. If the beam web alone is adequate, bearing stiffeners need not be added. Web crippling is a local failure which consists of crushing and local plastic buckling of the web immediately adjacent to a concentration of load. The load is assumed to spread or 'disperse' at an angle of 30° (*see* **20.5.4** of IS: 800-1956) as it goes through the flange and on out to the flat of the web at the line of tangency to the flange fillets.

The bearing stress of 1 890 kg/cm^2 that is allowed may result in minor localized plastic flow but provides a safe and reasonable basis for checking the design of this detail. In addition to the possibility of local crushing or crippling there is also the problem of general buckling of the web plate above a support or below a localized load. The web is assumed to act as a column with reduced length. A beam that is safe with respect to web crippling will usually be safe as well with respect to this type of web buckling.

These and other details of the design will be demonstrated in Design Example 1.

10. END CONNECTIONS AND BEARING PLATE DESIGN

10.1 If the end of the beam is supported directly on masonry without bearing plates, the local bending strength of the beam flanges should be checked to make sure that they may transfer the load from the local region under the web to the outer parts of the flange. The flanges of the beam act as small cantilevers to carry the permissible allowable load transmitted by the masonry without excessive stress. With stress thus limited they will be rigid enough to distribute the load to the masonry. If the flange were overstressed in bending, the load would be concentrated immediately below the web and local crushing of the masonry with possible subsequent cracks would result. The connection of the end of a beam to a column or girder may be either by means of web angles or top and bottom angles or by a combination of both. When a web angle connection frames to a beam or column web with beams entering from both sides and utilizing common rivets or bolts, it is desirable to add erection seats since it is difficult to hold both beams in place while rivets or bolts are being fitted. In general the engineer should carefully visualize just how the beam will be put in place during erection and make sure that a proper choice as to field or shop rivets is made so that erection will be facilitated. In the case of welded connections, there shall be provided a simple bolted erection plate or angle to hold the beam in place while field connections are being welded.

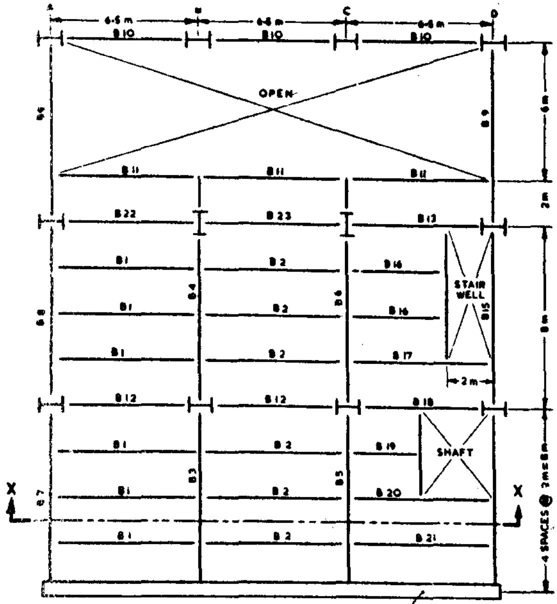
11. DESIGN EXAMPLE OF FLOOR BEAM FRAMING

11.1 The illustrative design example of floor beam framing showing the design of rolled beams is worked out in the following 17 sheets (*see* Design Example 1).

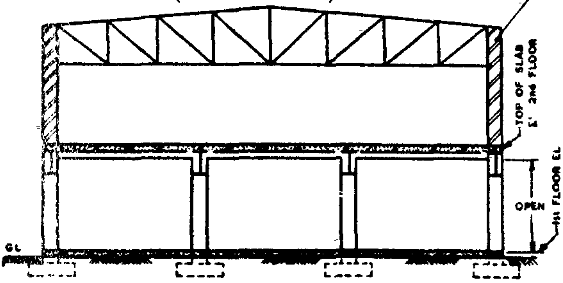
Design Example 1 — Floor Beam Framing

This sheet shows the framing plan for the beams and columns supporting a mill building loft and illustrates most of the typical situations that might be encountered. The design calculations of the beams are shown on the subsequent sheets.

Design Example I	**1* of 17**
Framing Plan and Section	



PLAN (SECOND FLOOR FRAMING)



SECTION XX

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*1 of 17 means that this Design Example has 17 sheets in all, of which this is the first sheet.

Design Example 1

Design of Beams B$_1$ & B$_2$

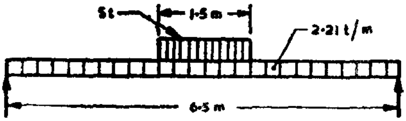
2 of 17

Initially, this sheet presents loading requirements for both dead and live loads. In addition to the distributed live load of 735 kg/m², the design is to include consideration of a 5-tonne 'roving' load that may be placed over any 1·5 × 1·5 m area. (This will permit the installation of a heavy piece of machinery on the floor but will rule out putting two such pieces of equipment in close proximity.) The first beams that shall be designed are those that do not receive reactive loads from other beams framing into them. Since floor dead load and the distributed live load both contribute uniform load per lineal metre to beams B_1 or B_2, the bending moment due to this load is calculated separately. The roving distributed load of 5 tonnes is first placed at the centre of the beam for maximum moment. The concrete slab encloses both sides of the top flange thereby providing adequate lateral support and the full permissible stress of 1 575 kg/cm² is permitted. The required section modulus is then determined and by reference to Table I (see p. 169) it is immediately seen that ISLB 450 beam would be satisfactory. The maximum shear is checked by moving the roving load to the end of the beam.

Sketch on Sheet 1 shows a plan and cross-sectional elevation of an industrial building. The end floor will carry a 12-cm RCC slab with 2·5 cm wearing surface added and will be designed for a live load of 735 kg/m² plus a 5-t load that may be placed over a 1·5 × 1·5 m area in any location. Exterior wall beams will support 670 kg per lineal metre in addition to any floor load they receive. The stairways are to be designed for 735 kg/m².

Use IS: 800-1956

BEAM B_1 or B_2



DL of slab (including wearing surface) = 370 kg/m²

Live load = 735 kg/m²

1 105 kg/m²

Load per metre length of beam being spaced at 2 m apart = $\frac{1\,105 \times 2}{1\,000} = 2{\cdot}21$ t/m

Assume beam weight = 75 kg/m = 0·075 t/m

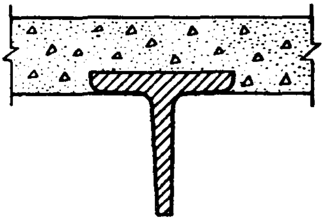
$w = 2{\cdot}285$ t/m

$$\text{BM @ } ⅊ = \frac{wl^2}{8} = \frac{2{\cdot}285 \times 6{\cdot}5 \times 6{\cdot}5}{8} = 12{\cdot}06 \text{ m·t}$$

or 1 206 cm·t

$$\text{Due to roving load, BM @ } ⅊ = \frac{5 \times 6{\cdot}5}{2 \times 2} - \frac{2{\cdot}5 \times 0{\cdot}75}{2}$$

$$= 7{\cdot}188 \text{ m·t } \textit{or } 718{\cdot}8 \text{ cm·t}$$

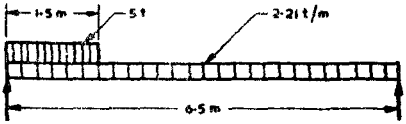


Provide lateral support

Total *Max* bending moment, 1 206 + 719 = 1 925 cm·t

$$\text{Required } Z = \frac{M}{fb} = \frac{1\,925\,000}{1\,575} = 1\,222 \text{ cm}^3$$

Referring to Table I: Choose ISLB 450, 65·3 kg — 1 223·8 — Z



Check shear value

$$V_{\max} = \frac{2{\cdot}285 \times 6{\cdot}5}{2} - \frac{5 \times 5{\cdot}75}{6{\cdot}5}$$

$$= 11{\cdot}85 < 36{\cdot}6 \text{ t} \ldots\ldots \text{OK.}$$

Design Example 1	**3**
Beam B_3 **Preliminary Design**	**of** **17**

Beam B_3 may now be designed since the reactions it receives as loads B_1 and B_2 or similar beams have now been determined. These reactions are introduced, however, without the roving load since this will be moved directly on the beam B_3 in its design. The required section modulus of 4 432 cm³ turns out to be higher than any IS rolled sections. One may either use an imported beam of greater depth or add top and bottom cover plates to strengthen ISWB 600, 145·1 kg section. This latter course is adopted and the estimate of dead weight of the beam itself is revised. It is noted that when the section is deepened by virtue of the welded plates, the contribution of the ISWB 600 to the total Z value is reduced and the reduction is estimated by multiplying by the ratio of depths before and after welding the cover plates. Final design check will be by moment of inertia procedure.

Combined B_1 and B_2 reactions (without roving load) $= 2{\cdot}285 \times \frac{6{\cdot}5}{2} \times 2$

$= 14{\cdot}9$ t

Assume beam weight $= 140$ kg/m

Max BM is @ ℄

M_w (due to beam weight) $= \frac{0{\cdot}14 \times (8)^2}{8} = 1{\cdot}1$ m·t

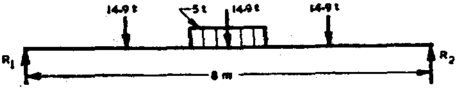
$$M_R = \frac{44{\cdot}7}{2}(4) - 14{\cdot}9(2) = 59{\cdot}6 \text{ m·t}$$

BM (roving load) $= 2{\cdot}5 \times 4 - 2{\cdot}5 \times \frac{0{\cdot}75}{2} = 9{\cdot}1$ m·t

M_t (total) $= 69{\cdot}8$ m·t

Z required $= \frac{6\,980\,000}{1\,575} = 4\,432$ cm³

No rolled section is available with this section modulus, so an ISWB 600, 145·1 kg section with welded cover plates (top and bottom) will be adopted.



Assume new weight of beam with cover plate $= 200$ kg/m

$$M_w = \frac{0{\cdot}2\,(8)^2}{8} = 1{\cdot}6 \text{ m·t}$$

$$M_t = 1{\cdot}6 + 59{\cdot}6 + 9{\cdot}1 = 70{\cdot}3 \text{ m·t}$$

$$Z = \frac{7\,030\,000}{1\,575} = 4\,460 \text{ cm}^3$$

Assuming that 1 of 2-cm thick cover plates will be required, the approximate Z contributed by ISWB 600, 145·1 kg will be:

$$3\,854 \times \frac{60}{62{\cdot}4} = 3\,700 \text{ cm}^3$$

∴ Z required in plates $= 4\,460 - 3\,700 = 760$ cm³

Approximate area required in one-flange plate $= \frac{760}{2 \times 31}$

$= 12{\cdot}3$ cm²

Design Example I | **4 of 17**

Beam B_3
Cover Plate Length

In order to determine the length of cover plate that is required, in view of the roving load, it is now necessary to draw to scale the envelope of various moment diagrams that are possible with the roving load in different positions along the span. The envelope of bending moment diagrams, plotted at the bottom of this sheet, indicates that the theoretical length of the cover plates will be about 2·26 m but in order to develop the plate at its ends it is customary to add a little more at each end making the total length approximately 2·8 m.

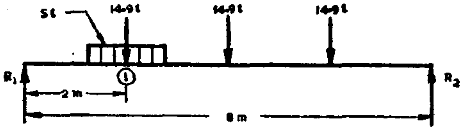
- -

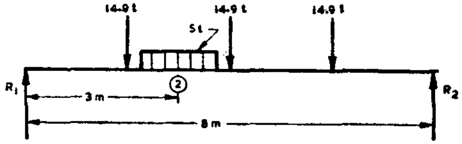
Length of Cover Plate

A quick method which is accurate enough for practical purposes is to draw the diagram of 'maximum moments' and scale the points which the ISWB 600 may take in moment.

Trial loadings for maximum moments with roving loads at $\frac{1}{4}$, $\frac{3}{8}$ and Centre Points of beam

The maximum moment at a section is when the roving load is at the Section.





Assume dead load $= 200$ kg/m

$M_w = 0{\cdot}2(4)(2) - 0{\cdot}2(2)(1) = 1{\cdot}2$ m·t

For B_1, B_2 reactions and roving load

$$R_1 = 22{\cdot}35 + \frac{5\times 6}{8} = 26{\cdot}10 \text{ t}$$

BM at $\frac{1}{4}$ Point due to B_1, B_2 and roving load

$= 26{\cdot}1(2) - 2{\cdot}5(0{\cdot}75) \quad = 51{\cdot}26$ m·t

Total *Max* BM at $\frac{1}{4}$ Point $M_{t_1} \quad = \overline{52{\cdot}46 \text{ m·t}}$

$R_1 = 22{\cdot}35 + 5 \times \frac{5}{8} = 25{\cdot}47$ t

$$\text{BM (at } \tfrac{3}{8} \text{ point)} = 25{\cdot}47(3) - 2{\cdot}5\left(\frac{0{\cdot}75}{2}\right) - 14{\cdot}9 \times 1$$
$$= 60{\cdot}57 \text{ m·t}$$

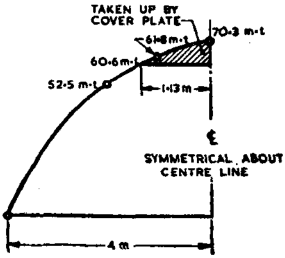
M_{t2}(at $\frac{3}{8}$ point) $= M_w + 60{\cdot}57 = 1{\cdot}2 + 60{\cdot}57 = 61{\cdot}8$ m·t

Total *Max* BM at Centre Point,
M_{t3} (*see* Sheet 3) $= 70{\cdot}3$ m·t

$$\text{ISWB 600 moment capacity} = \frac{3\,854 \times 1\,575}{100 \times 1\,000} = 60{\cdot}6 \text{ m·t}$$

Theoretical length required $= 2(1{\cdot}13) \quad = 2{\cdot}26$ m·t

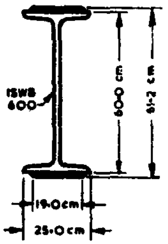
For making allowance for the customary extra length required on either side, adopt 2·8 m.



ENVELOPE OF THE BENDING MOMENT DIAGRAM SHOWING THEORETICAL LENGTH OF COVER PLATE REQUIRED

Design Example I	5 of 17
Beam B_5 **Welding Cover Plates**	

Flange plates 19·0×0·6 cm are tried and the moment of inertia calculated. The welds attaching the cover plate to the beam shall now be determined and since the weld requirement is a function of maximum shear, the roving load is put in a position that will produce the maximum shear near the end of the cover plate. The horizontal shear to be transferred is determined by Eq 7 (see p. 24) [multiplying both sides by t to obtain the total shear transfer (f_s) per linear centimetre].



Approximate area required in each cover plate (*see* Sheet 3) = 12·3 cm²

Try 2 plates, 19·0 × 0·6 cm

Area of one plate = 11·4 cm²

I of cover plate = 2(11·4) (30·3)² = 21 000 cm⁴

I of ISWB 600 = 115 627 cm⁴

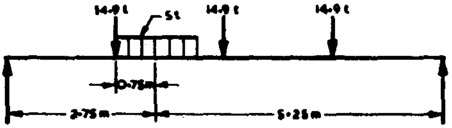
Total = 136 627 cm⁴

$$Z_t = \frac{136\ 627}{30{\cdot}6} = 4\ 470 \text{ cm}^3 > 4\ 460 \text{ required} \ldots\ldots \text{OK.}$$

Determine V_{max} @ location 1. (This is approximate calculation and is considered OK for practical purposes.)

$$V_{max} = 22{\cdot}35 + 5 \times \frac{5{\cdot}25}{8}$$

$$= 25{\cdot}63 \text{ t}$$



Approximate total weight of section 145+18= 163 kg/m

Total V_{max}= 25·6+4(0·163) = 26·25 t

Actual horizontal shear per linear centimetre:

$$\frac{VA_cY}{I} = \frac{26{\cdot}25 \times 11{\cdot}4 \times 30{\cdot}3}{136\ 627} = 0{\cdot}066\ 5 \text{ t/cm}$$

Use 6·0-mm fillet weld intermittent (*see* **6.2.2.1** & Table I of IS: 816-1956).

Working load/cm length = 6(0·7) (1 025)

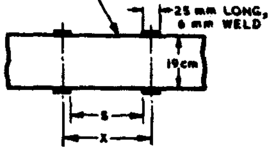
= 430 kg/cm (referring to **6.2.3**, Table II, **7.1** and Table III of IS: 816-1956)

Minimum length of weld = 4×6·0 = 24 mm (*see* **6.2.4.1** of IS: 816-1956)

Use 2·5 cm length

Working strength of 6·0-mm weld 25 mm long, 2 sides = 2×2·5×430

= 2 150 kg *or* 2·15 t



c/c Spacing = X

$0{\cdot}066\ 5 \quad X = 2{\cdot}15$

$$X = \frac{2{\cdot}15}{0{\cdot}066\ 5} = 32{\cdot}4 \text{ cm}$$

$$S = 32{\cdot}4 - 2{\cdot}5 = 29{\cdot}9 \text{ cm}$$

Since it is uneconomical of the welder's time as well as being less efficient to start and stop new welds, the length of weld in each intermittent section should be as long as possible in keeping with the requirement of the space thickness ratio. Thus, in the light of all of these factors, 2·5 cm long 6-mm fillet welds spaced 18·5 cm centre to centre are chosen. At the end of the plate, a continuous weld for a 16-cm length of plate is used so as to fully develop the cover plate at the point where it begins to be needed.

Required S by **6.2.6.2** of IS:816-1956

So that $S/t \leqslant 16$

$S = 16\ (0{\cdot}6) = 9{\cdot}6$ cm $< 29{\cdot}9$ cm

Hence adopt 9·5 cm clear spacing.

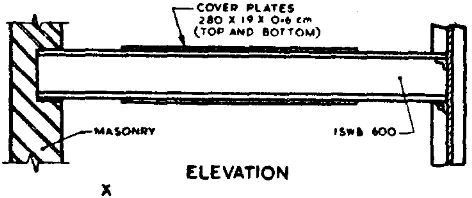
Weld strength at ends of cover plate to develop strength of plate (*see* **20.5.1** of IS: 800-1956):

Plate strength $= 0{\cdot}6 \times 19 \times 1{\cdot}575$

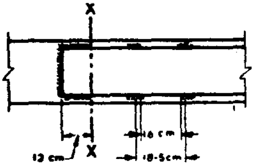
$= 17{\cdot}96$ t

$$L = \frac{17{\cdot}96}{0{\cdot}43} = 41{\cdot}75 \text{ cm}$$

$$\frac{41{\cdot}75 - 19^{*}}{2} = 11{\cdot}37 \text{ cm, } \textit{or say } 12 \text{ cm}$$



ELEVATION



PARTIAL PLAN

SECTION XX

It may be observed that with 6·0-mm intermittent fillet weld at 2·5-cm length, though the required spacing is 29·9 cm clear, the minimum Code (IS: 816-1956) requirement of 9·6 cm corresponding to 0·6 cm thickness of cover plate has to be adopted. Thus, there is still some waste in weld. Also some special precautions are to be taken while welding (*see* **6.2.5** of IS: 816-1956). These may be overcome by redesigning the cover plate thickness.

Choose 2 plates of 12·5×1·0 cm.

Area of 1 plate $= 12{\cdot}5$ cm^2

I of cover plate $= 2\ (12{\cdot}5)\ (30{\cdot}5)^2 = 23\,256$ cm^4

I of ISWB 600 $= 115\,627$ cm^4

$138\,883$ cm^4

$$Z = \frac{138\,883}{30{\cdot}8} = 4\,480 \text{ cm}^3 > 4\,460 \text{ cm}^3 \text{ required.}$$

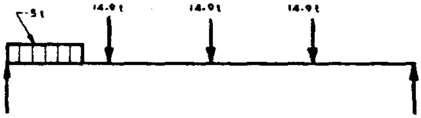
Using 6·0-mm weld at 25 mm length, the required spacing as worked out already is 29·9 cm clear. Code requirement $= 16$ t $= 16\ (1{\cdot}0)$ t $= 16{\cdot}0$ t

Use a clear spacing of 16 cm.

At ends, $\frac{41{\cdot}75 - 12{\cdot}5}{2} = 14{\cdot}62$ *or* 15 cm length to be welded

Check shear values.

Loading for $V_{\max}$ is as shown in the sketch



$$R_1 = V_{\max} = 22{\cdot}35 + 5\left(\frac{7{\cdot}25}{8}\right) + 0{\cdot}163\ (4)$$

$= 27{\cdot}53 < 66{\cdot}9$ t, shear value of the Section ISWB 600 . . . . . OK.

---

*Width of cover plate.

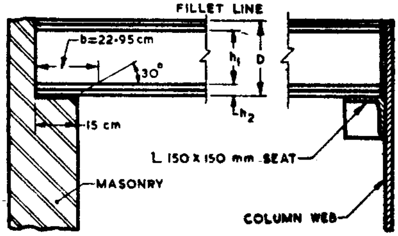
Design Example 1

Beam B_3 Bearing

7 of 17

The beam is assumed to rest on a bearing plate set in the masonry wall with an assumed length of bearing equal to 15 cm. Similarly, at the opposite end, the beam rests on the 15 cm outstanding leg of a seat angle. At either end the check as to web crushing or crippling is similar and the sketch shows how the load is assumed to be dispersed upwards from the bearing plate through a distance equal to the flange thickness plus the fillet radius for a distance h_2= 4·6 cm. Thus, the total effective length of web resisting local crippling is found to be 22·9 cm. The bearing stress is less than that permitted by the specification so we now turn to a check on the web buckling. The specification stipulates that no bearing stiffener is needed at points of local support provided the buckling requirements are met. The beam is found to be amply strong with respect to buckling to resist the maximum end reaction of 27·53 tonnes without any bearing stiffeners.

Check web crushing (crippling)



Angle of load dispersion = 30° (*see* **20.5.4** of IS: 800-1956)

Referring to Table I of ISI Handbook for Structural Engineers: 1. Structural Steel Sections

h_1 = depth of intersections of web to flange fillets = 50·79 cm

h_2 = 4·6 cm

b (*see* sketch above) = 15+4·6 cot 30° = 22·95 cm

$V_{\max}$= 27·53 (*see* Sheet 6); Web thickness, t_w = 11·8 mm

$$\text{Bearing stress} = \frac{27{\cdot}53 \times 1\ 000}{(22{\cdot}95)\ (1{\cdot}18)} = 1\ 015\ \text{kg/cm}^2 < 1\ 890\ \text{kg/cm}^2$$

(*see* **9.4** of IS: 800-1956)

Check for buckling

Allowable reactions with no stiffener = $F_c tB$ (*see* **20.7.2.1** of IS: 800-1956)

$$l/r = \frac{50{\cdot}79}{1{\cdot}18}\sqrt{3} = 64{\cdot}5; \quad F_c = 1\ 068\ \text{kg/cm}^2$$

Assuming 13 cm as stiff length of bearing for 15·0 cm seat angle:

$$B = 13 + \frac{60}{2} = 43\ \text{cm}; \ \text{Allowable}\ R = 1\ 068\ (1{\cdot}18)\ (43)$$

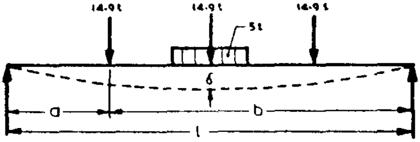
$$= 54 > 27{\cdot}53 \ldots\ldots \text{OK.}$$

Buckling strength at masonry support is safe, the stiff length of bearing being 15 cm > 13 cm . . . . . OK.

Design Example 1 | **8 of 17**

Beam B_3 — Deflection & Design of Bearing Plate

On the assumption that the designer wishes to comply with the optional specification requirement that the allowable deflection be less than 1/325 of the span length, this deflection is now calculated and is found to be well within the requirement. Then the bearing plate at the masonry supported end is designed. For the 15-cm length of bearing used, a 34-cm width is required. It would be more economical to use a more nearly square plate, requiring a smaller thickness, but it has been assumed that the available bearing length is limited to 15 cm. There is no question of failure but it is desired to provide a bearing plate stiff enough to spread the load to the masonry and prevent local cracks.



Loading sketch for maximum deflection is shown here.

Assuming 5 t load as a concentrated load, the loading may be considered as 14·9 t, 19·9 t and 14·9 t.

Due to central load $\delta_c = \dfrac{Pl^3}{48\,EI}$

Due to the two quarter point loads $\delta_l = \dfrac{Pa}{24\,EI}\,(3l^2 - 4a^2)$

$E = 205 \times 10^4$ kg/cm² (corresponding to 13 000 tons/in.²)

By Method of Superposition:

$$\delta = \frac{19{\cdot}9 \times (800)^3 \times 1\,000}{48\ (2\,050\,000)\ 137\,953} - \frac{14{\cdot}9 \times (200) \times 1\,000}{24\ (2\,050\,000)\ 137\,953.}(3 \times 800^2 - 4 \times 200^2)$$

$= 1{\cdot}52$ cm

Limiting deflection = 1/325 span (*see* **20.4.1** of IS: 800-1956)

$$= \frac{800}{325} = 2{\cdot}46 > 1{\cdot}52 \ldots\ldots \text{OK.}$$

Beam bearing plate

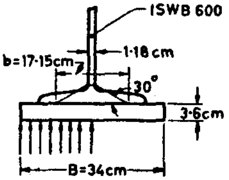
Assume allowable masonry bearing stress $= 55$ kg/cm²

Area required $= \dfrac{27{\cdot}53}{0{\cdot}055} = 500$ cm²

$B = \dfrac{500}{15} = 33{\cdot}3$ cm, *or say* 34 cm

$b = 1{\cdot}18 + 2\ (4{\cdot}6 \cot 30°)$ (based on 30° dispersal of web load through flange plus fillet)

$= 17{\cdot}15$ cm



$P_2 = \dfrac{27{\cdot}53 \times 1\,000}{34 \times 15} = 54$ kg/cm²

$P_1 = \dfrac{27{\cdot}53 \times 1\,000}{17{\cdot}1 \times 15} = 107{\cdot}1$ kg/cm²

$M@℄ = 17\ (54)\ (8{\cdot}5) - \dfrac{17{\cdot}1}{2}\ (107)\left(\dfrac{17{\cdot}1}{4}\right) = 3\,929$ cm·kg (taking a 1-cm strip)

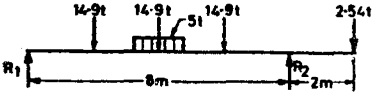
$M/I = f/y$ and f permissible 1 890 kg/cm² (*see* **9.2.3** of IS: 800-1956)

$1\,890 = \dfrac{3\,929\ t/2}{t^3/12}$, *or* $t = 3{\cdot}53$ cm

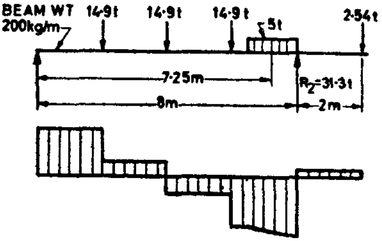
Use $15 \times 34 \times 3{\cdot}6$ cm bearing plate.

Design Example I	9 of 17
Beam B_4	

Beam B_4 is assumed to cantilever over column line 2 and this column is turned 90° so that the beam web and column web will be directly in line, thus providing the simplest arrangement for stiffeners if required at this support. It should be noted that in introducing the reaction of beam B_{11} the live load is entirely omitted as the maximum positive bending moment would be for this condition. At the interior column, reaction point, designated as R_2, the framed ends of beam B_{23} provide a partial stiffener and it will be assumed that local web buckling will be prevented. The bearing plate between the column and the beam will not be designed as reference may be made to ISI Handbook for Structural Engineers 3: Steel Columns and Struts for the design of similar bearing plates at a column base.



(It may be noted that the beam loads at the supports are not shown as these will not affect the BM and SF diagrams.)



Assume B_{11}, weight $= 20$ kg/m

Dead load of slab $= 370$ kg/m² (*see* Sheet 2)

B_{11} reaction (from two sides) $= \frac{2(2)\ (6 \cdot 5)}{4} 370 + 20 \times 6 \cdot 5$ kg

$= 2 \cdot 54$ t

Assume B_4 weight as 200 kg/m

$R_1 = \frac{200 \times 3 \times 10}{8} = 750$ kg

$R_2 = 1\ 250$ kg

BM at midway between supports $\left.\right\} = 0 \cdot 750 \times 400 - 0 \cdot 2 \times 4 \times 200 = 140$ cm·t

R_1 due to other loads $= \frac{3(14 \cdot 9)(4) + 5(4) - 2 \cdot 54(2)}{8} = 25 \cdot 49$ t

M_t due to other loads $= 25 \cdot 49(4) - 14 \cdot 9(2) - 2 \cdot 5(0 \cdot 375) = 7\ 122$ cm·t

M_t due to B_4 weight $= 140$

$Z = \frac{7\ 262}{1 \cdot 575} =$ *say* 4 610 cm³

Use ISWB 600, 133·7 kg with cover plate. Design of cover plate will not be shown here. It may be noted that in the design of beam B_3, the area required in the cover plates was small, as ISWB 600, 145·1 kg was adopted, and resulted in uneconomical welding details. Hence for beam B_4, ISWB 600, 133·7 kg with cover plate is recommended. This will also result in a lesser overall weight of beam B_4 than when ISWB 600, 145·1 kg with cover plates is used. Beam B_3 also could be designed with ISWB 600, 133·7 kg with cover plates.

Check shear value.

The loading sketch for maximum shear at B_{23} is as shown above.

$V_{max} = R_2 - 2 \cdot 54 = \frac{1 \cdot 25 + (3 \times 19 \cdot 9 \times 4 + 5 \times 7 \cdot 25) - 2 \cdot 42(2) - 2 \cdot 54}{8}$

$= 28 \cdot 8 < 63 \cdot 5$ t (Shear capacity of ISWB 600, 133·3 kg)

Check web crushing

It may be noted that for maximum shear in B_4, the reaction of beam B_{11} should include live load which was omitted while determining the maximum possible moment on B_4. The maximum shear with this correction is 38·4 tonnes (*see* Sheet 10) which is still less than the shear capacity of the beam ISWB 600 designed . . . . . OK.

In checking the local web crippling at R_2 the full live load is introduced into the reaction of B_{11} as this will produce the maximum R_2 reaction. The bearing stress in the web is considerably less than the permissible value.

Design Example I | **10 of 17**

Beam B_4 — Bearing Stress in Web

Assuming ISHB 300 as shown in the sketch at the bottom, for the column:

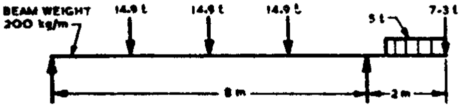
Load dispersion = 30° (*see* **20.5.4** of IS: 800-1956)

h_2 = 2·51 cm (*see* Table 1 of ISI Handbook for Structural Engineers: 1. Structural Steel Sections)

b = 30+2(2·5 cot 30°) = 38·7 cm

Reactions on this column will include B_{23} reactions, as they frame into B_4 at this support.

The two B_{13} reactions approximately = 14·9 t (same as B_1)



For *Max* R_2:

The reaction from B_{11} should include live load also.

Due to dead load = 2·5 t (*see* Sheet 9)

$$\text{Due to live load} = \frac{2(2\times6\cdot5)}{4}\ 0\cdot735^{*} = 4\cdot8\ \text{t}$$

$$\text{Total} = \overline{7\cdot3\ \text{t}}$$

R_2 = [3(14·9)(4)+5(9·25)+0·2(10)5+7·2(10)]÷8
= 38·4 t

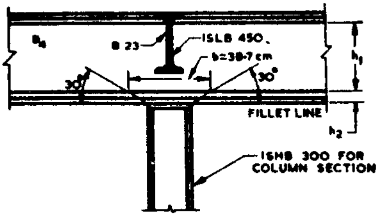
Total R = 38·4+14·9 = 53·3 t

$$\text{Bearing stress} = \frac{53\cdot3\times1\ 000}{38\cdot7\times1\cdot12}$$

= 1 542 kg/cm² < 1 890 kg/cm² permissible . . . . . OK.

Buckling load not to be checked due to stiffening effect of B_{23} connections.

Check section for moment at cantilever.



Assuming no cover plate at cantilever support:

w = 0·134 t/m (weight of ISWB 600, 133·7 kg)

M = 7·3(2)+0·134(2) 1+5(1·25) = 2 152 cm·t

$$Z = \frac{2\ 152\times1\ 000}{1\ 575} = 1\ 370\ \text{cm}^3 < 3\ 540\ \text{cm}^3 \ldots\ldots \text{OK.}$$

---

*Live load.

Design Example I

Beam B_9

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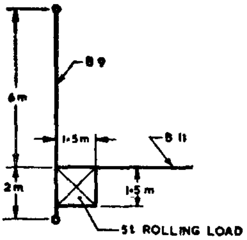
The design of beam B_9 introduces the problem of the lack of lateral support. For 6 m, this beam supports the exterior wall but has no effective lateral bracing because it is adjacent to the open area at the end of the building. Beam B_{11} provides effective lateral bracing at a point. Thus, while the bending moments are calculated on the basis of the full 8 m length an effective length of 6 m is used to determine the permissible stress.



The 5-t roving load is first distributed to B_{11} and B_{13}.

B_{11} share is $\dfrac{5\times1{\cdot}25}{2}$

The effect of this $\dfrac{5\times1{\cdot}25}{2}$ on B_9 is $\dfrac{5\times1{\cdot}25}{2}\times\dfrac{5{\cdot}75}{6}$

B_{11} reaction on B_9

$$=\frac{7{\cdot}5}{2}+5\left(\frac{1{\cdot}25}{2}\right)\left(\frac{5{\cdot}75}{6{\cdot}5}\right)=6{\cdot}5\text{ t}$$

Exterior partition load = 670 kg/m

Assume beam weight = 75 kg/m

Uniform load = 745 kg/m

$R_1 = V_1 = 6{\cdot}5\ (\tfrac{6}{8}) + 0{\cdot}745\times4$

$= 4{\cdot}87+2{\cdot}980 = 7{\cdot}85$ t

$M_{\text{max}}=7{\cdot}85(2)-0{\cdot}745(2)\ (1)=14{\cdot}2$ m·t

Effective l = 6 m (unsupported span against lateral bending)

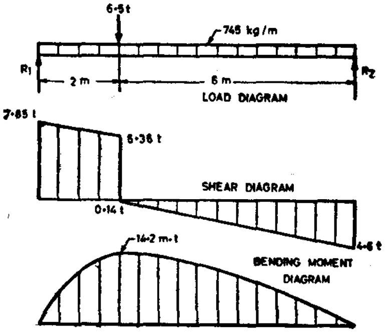
Try ISWB 400,

$Z = 1\,171{\cdot}3$ cm³; $S = 32{\cdot}5$ t

$l/b = \dfrac{600}{20} = 30;\ t_f = 13{\cdot}0$

$b = 200$ wt $= 66{\cdot}7$ kg/m

$d/t_f = \dfrac{400}{13{\cdot}0} = 31$



$F_b = 1\,142$ kg/cm² (*see* Table II on p. 172)

Required $Z = \dfrac{1\,420\,000}{1\,142} = 1\,240$ cm³ $> 1\,171{\cdot}3$ provided by ISWB 400. Hence another trial.

Try ISMB 450, 72·4 kg (The next higher section from Table I on p. 169)

$l/b = \dfrac{600}{15{\cdot}0} = 40 \qquad d/t_f = \dfrac{450}{17{\cdot}4} = 26 \qquad F_b = 889$ kg/m²

Required $Z = \dfrac{1\,420\,000}{889} = 1\,600 > 1\,350{\cdot}7$ — No Good.

Try ISLB 500, 75·0 kg; $Z_x = 1\,543{\cdot}8 \qquad V = 43{\cdot}47$ wt $= 75{\cdot}0$ kg/m

$t_f = 14{\cdot}1$ mm $b = 180 \qquad l/b = \dfrac{600}{18} = 33{\cdot}3 \qquad d/t_f = \dfrac{500}{14{\cdot}1} = 35{\cdot}7$

$F_b = 927$ kg/cm² (*see* Table II on p. 172)

Required $Z = \dfrac{1\,420\,000}{927} = 1\,532$ cm³ $< 1\,543$ ..... OK.

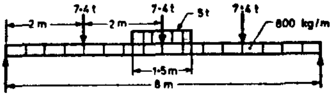
Beam weight assumed previously is OK.

Design Example I | **12 of 17**

Beams B_7, B_8 & B_{16}

Border beams B_7 and B_8 are now designed. These carry end reactions from beams B_1 amounting to half of the similar loads on beams B_3 and B_4. These beams also carry the exterior wall and they are not assumed as completely supported laterally, since the floor slab encases only one side of the flange. However, on the basis of the unsupported length of 2 m, no stress reduction is found necessary.

Beams B_{14}, B_{16} and B_{17} will now be designed in sequence. Although these do not introduce selection problems, they are included to illustrate the calculation of loads and reactions on interrelated beams. The design of beam B_{16} is routine.



Assume beam weight = 130 kg/m

Exterior partition load as in Sheet 11 = 670 kg/m

Total uniform load w = 800 kg/m

$$M_w = \frac{800 \times 8^2 \times 100}{8} = 640\,000 \text{ cm·kg}$$

BM (due to B_1 reactions) = 11·1 (4) − 7·4 (2) = 2 960 000 cm·kg

BM (due to 5 ft roving load) = 2·5 (4) − 2·5 (0·375) = 906 300 cm·kg

BM (total) = 4 506 300 cm·kg

Effective length = 2 m

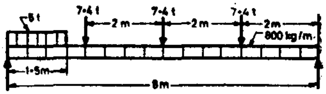
It is likely F_b = 1 575 kg/cm² as the beam is supported laterally at fairly close intervals.

$$\text{Required } Z = \frac{4\,506\,300}{1\,575} = 2\,860 \text{ cm}^3$$

From Table I on p. 169, choose ISMB 600, 122·6 kg

$Z = 3\,060{\cdot}4 \qquad b = 210 \qquad t_f = 20{\cdot}8$ mm

$$l/b = \frac{200}{21} = 9{\cdot}5 \qquad d/t_f = \frac{600}{20{\cdot}8} = 29$$

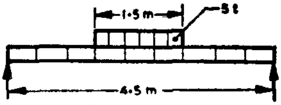


From Table II (*see* p. 172) F_b = 1 575 kg/cm² as assumed. Hence OK.

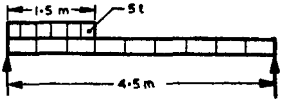
Check shear value

$$V = 11{\cdot}1 + 0{\cdot}800 \times 4 + 5\left(\frac{7{\cdot}25}{8}\right)$$

$$= 19 \text{ t} < 68{\cdot}0 \text{ t} \ldots\ldots \text{OK.}$$



POSITION OF LOAD FOR MAX MOMENT



POSITION OF LOAD FOR MAX SHEAR

Beam B_{16}

Assume beam weight = 60 kg/m

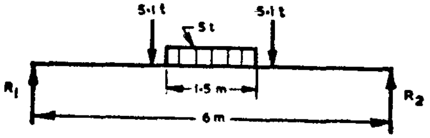
w = (Dead load + Live load, *see* Sheet 2) = 2 210 kg/m

Total = 2·27 t/m

(Routine design) Use ISLB 325, 43·1 kg.

Design Example I	13 of 17
Beams B_{14} & B_{17}	

Since beam B_{14} is adjacent to the stair well and carries a floor slab on one side only, it is assumed that the unsupported length is 2 m. Beam B_{17} is of interest because of the loading. It is assumed that a stairway starts at this level and runs to the ground floor below. Thus, at an assumed dead load plus live load of 1 200 kg/m² one-half of the total supported stair load is assumed to rest 2 m from the end of beam B_{17}. Although the shear and moment diagram for this beam might well have been drawn, it is quicker to calculate the bending moment at the centre line and at the point of load concentration. One of these two will be very close to the maximum and will be a satisfactory basis for design.



Beam B_{14}

B_{16} reaction (due to dead load and live load) = 2·25 (2·27)

= 5·11 t

Use ISLB 450, 65·3 kg.

Beam B_{17}

Assume beam weight = 90 kg/m

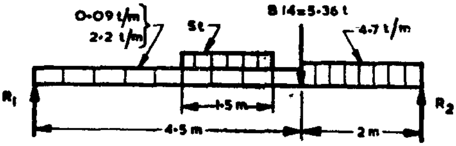
Stairs: DL+LL = 1 200 kg/m²

1 200 × 6/2 (*see* plan on Sheet 1) = 3 600 kg/m

½ panel load (*see* Sheet 1) 1 (1 105) = 1 105 kg/m

Total = 4 705 kg/m

or 4·7 t/m



Case 1 @ ℄:

$$R_1 = 2\cdot2\,(4\cdot5)\left(\frac{4\cdot25}{6\cdot5}\right) + 2\cdot5 + 5\cdot36\left(\frac{2\cdot0}{6\cdot5}\right) + 4\cdot7\,(2\cdot0)\left(\frac{1\cdot0}{6\cdot5}\right) + 0\cdot09\left(\frac{6\cdot5}{2}\right)$$

$$= 12\cdot4 \text{ t}$$

$$\text{BM} = 12\cdot4\,(3\cdot25) - (2\cdot2 + 0\cdot09)\left(\frac{6\cdot5}{2}\right)\left(\frac{6\cdot5}{4}\right) - 2\cdot5\,(0\cdot37)$$

$$= 27\cdot0 \text{ m}\cdot\text{t}$$

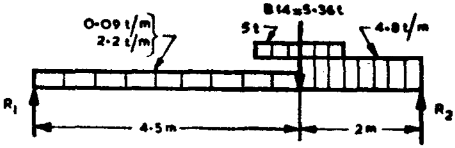
Case 2 @ *B_{14} reaction point*:

$$R_1 = 2\cdot2\,(4\cdot5)\left(\frac{4\cdot25}{6\cdot5}\right) + (5+5\cdot36)\frac{2}{6\cdot5} + 0\cdot09\left(\frac{6\cdot5}{2}\right) + \frac{4\cdot8 \times 2\times1}{6\cdot5}$$

$$= 11\cdot4 \text{ t}$$

$$\text{BM} = 11\cdot4\,(4\cdot5) - 2\cdot2 + 0\cdot09\,(4\cdot5)\,(2\cdot25) - 2\cdot5\,(0\cdot375)$$

$$= 27\cdot2 \text{ m}\cdot\text{t}$$



The beam selected for B_{17} is checked for an unsupported length of 2 m adjacent to the stairway.

Design Example 1

Beam B_{17}

14 of 17

Assume $F_b = 1\,575$ kg/cm² as the unsupported length is 2 m in the stair well,

$$\text{Required } Z = \frac{2\,720\,000}{1\,575} = 1\,733 \text{ cm}^3$$

Try ISLB 550,

$$Z_t = 1\,933{\cdot}2 \text{ cm}^3$$

$$w = 86{\cdot}3 \text{ kg/m}$$

$$b = 190 \text{ mm}$$

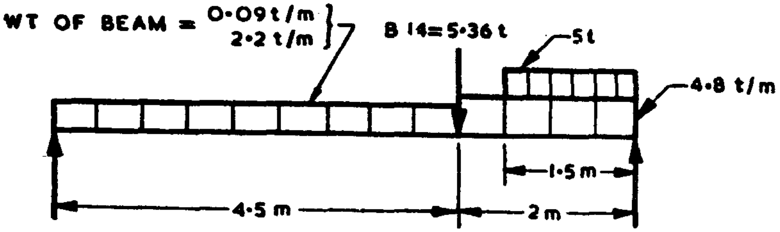
$$t_f = 15{\cdot}0 \text{ mm}$$

$$l/b = \frac{200}{19} = 11$$

$$d/t_f = \frac{500}{15{\cdot}0} = 33$$

$F_b = 1\,575$ kg/cm² as assumed . . . . . OK.

Check shear as before



Design Example I

Beams B_3 & B_4 Connections with Stiffened Seat

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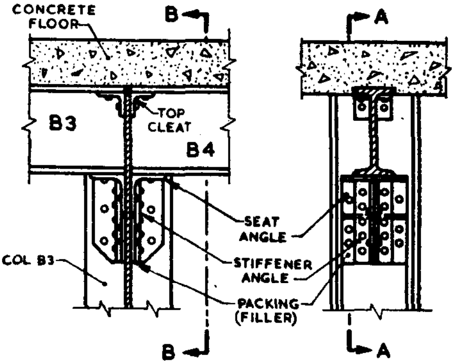
Typical riveted connections will now be designed for the floor framing plan of this example. Generally speaking, welded connections offer somewhat greater weight saving in steel than do riveted but since this will be covered in ISI Handbook for Structural Engineers on Designing and Detailing Welded Joints and Connections (under preparation), the connections in this example will be riveted. This design sheet shows the proposed arrangement for a stiffened seat with an inverted top cleat making the top of the column flush with the top of the beams near the bottom of the concrete floor slab. Permissible rivet stresses are computed on this sheet.

DESIGN OF CONNECTIONS



SECTION AA SECTION BB

Rivets: 22·0 mm (or $\frac{7}{8}$ in. dia)

Gross area of rivet $= \frac{(23{\cdot}0)^2}{4\times100} = 4{\cdot}2$ cm²

Rivet values (*see* **10.1** and Table IV in IS: 800-1956)

Shop (power driven) single shear = 4·2 (1 025)

= 4 300 kg

Double shear = 2 (4 300) = 8 600 kg

Permissible rivet bearing stress = 2 360 kg/cm²

Field (power driven) single shear = 4·2 × (945)

= 3 969 kg

Double shear = 2 (3 969) = 7 938 kg

Permissible rivet bearing stress = 2 125 kg/cm²

Design Example 1

Beams B_3 & B_4
Design of Stiffened Seat

16 of 17

Beams B_3 and B_4 carry their load into the column web by rivets common to both stiffened seat connections. Thus, these rivets have to transmit approximately twice the end reaction of either beam B_3 or B_4 and they are so calculated. The stiffened seat connection as shown in Sheet 15 consists of a horizontal cleat with two vertical cleats acting as stiffener. It is necessary to introduce packing or filler plates equal in thickness to that of the horizontal seat angles.

Maximum reaction of either B_3 or B_4 = 27·5 t (*see* Sheet 6)
Combined reaction transmitted to column = 2 (22·3) + 5 = *say* 50 t

Investigate first the column web in bearing, as this appears to be the most critical condition due to thin column web.

Note — The reaction due to B_{12} beams are not included here as these will be connected to column flanges directly and not to web of column section through B_3 or B_4.

Column is not designed in this example, so assume a column web, t_w = 1·0 cm

$$\text{No. of rivets to column web} = \frac{50}{(2{\cdot}3)\,(1{\cdot}0)\,(2{\cdot}360)} = 9{\cdot}5 = \textit{say}\ 10$$

Minimum thickness, t, required for any member of connection (angles, packing, etc) is found as follows:

t (2·3) (2·125) = 3·97 (field rivet bearing against single shear)

Note — This is in accordance with good practice.

$$t = \frac{3{\cdot}97}{(2{\cdot}3)\,(2{\cdot}125)} = 0{\cdot}8\ \text{cm}$$

$$\text{For shop rivet, } t = \frac{4\,300}{(2{\cdot}3)\,(2{\cdot}360)} = 0{\cdot}79\ \text{cm, use } 0{\cdot}8\ \text{cm for all cases.}$$

Additional rivets for packing as required by **24.6.1** of IS: 800-1956.

$$\frac{8}{1{\cdot}6}\,(2) = 10 \text{ percent, extra rivets required } \frac{10\,(10)}{100} = 1 \text{ rivet}$$

But, for the sake of symmetry, use 6 rivets in the extension of the packing as required in **24.6.1** of IS : 800-1956. We have already assumed 15·0 × 15·0 cm seat angle in Sheet 7. For adequate stiffening of the seat angle, you require at least 125 × 75 mm angle stiffener. The minimum thickness being 8 mm as calculated above, use 2 of 125 × 75 × 8·0 mm angle sections on each side of the column web. Effective length of outstanding leg = 125−10 = *say* 115 cm. Bearing capacity of 2 legs (outstanding) = 2(11·5) (0·8) (1·890) = 34·78 > 27·5 t.

Check rivet capacity of stiffener leg = 10(4·3) = 43 > 27·5 t ..... OK.

As cleat angles on top are only for lateral restraint, use a reasonable size, say 2 of ISA 10075, 8·0 mm. Dimensions of packing are determined by minimum pitch of rivets and edge distance requirement (*see* **25.2.1** and **25.4** of IS: 800-1956).

Design Example I

Alternative Connections with Web Cleat Angle

17 of 17

An alternative type of framed connection for the same location is designed. This is the web cleat angle connection as shown. Since both beams have common rivets framing to the column web it is not possible to erect them individually because erection bolts shall be placed through both beam connections and the column web at the same time. A seat angle for erection purposes only is added for this connection. This type of connection is actually more suitable when the beam frames into a column flange as is the case for beams B_{12} and B_{18}. For flange framing, the bearing is in single shear for both the web cleats and the column flange. An erection seat may be used if desired but it is not absolutely essential since each beam may be bolted in place temporarily while being held by the erecting equipment.

Type: Framed connections — Apply to same joint just for illustrating the design factor involved.

B_3 and B_4 ISWB 600, 133·7 kg t_w = 11·2 mm

Gross rivet diameter = 2·3 cm

Angles to web of beam:

No. of rivets (double shear at beam web) $= \frac{27{\cdot}5}{8{\cdot}6}$ = *say* 4

No. of rivets for bearing against connections angle using 1·2 cm thick angles $= \frac{27{\cdot}5}{2\,(1{\cdot}22)\,(2{\cdot}360)\,(2{\cdot}3)}$

= 2·1 = *say* 3

No. of rivets of bearing against beam web $= \frac{27{\cdot}5}{(1{\cdot}12)\,(2{\cdot}360)\,(2{\cdot}3)}$

= 4·5 = *say* 5

Angles to web of columns:

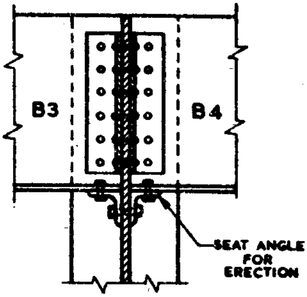
No. of rivets — single shear at column web $= \frac{27{\cdot}5}{3{\cdot}969}$

= 7·1 = *say* 8

No. of rivets — bearing on column web

assuming t_w = 1·0 cm, $\frac{50{\cdot}2}{(2{\cdot}3)\,(1{\cdot}0)\,(2{\cdot}125)}$ = 10·3, use 12.

This condition determines the number of rivets required.



SECTION III

DESIGN OF PLATE GIRDERS

12. GENERAL

12.1 When the required section modulus for a beam exceeds that available in any standard rolled section, one of the choices available to the designer is to build up a beam section by riveting or welding plate and/or angle segments to form a 'plate' girder. Plate girders are especially adapted to short spans and heavy loads. Two design examples, one for welded plate girder and another for riveted plate girder, are given in Design Examples 2 and 3. In order to facilitate comparison of the two types of plate girders, these are designed for carrying the same loads.

12.2 In the plate girder, the engineer is able to choose flange material and web material in the proper proportion to resist bending moment and shear respectively and he may vary the thickness of flange and web along the girder as the bending moments and shear vary. The design of plate girders may be tackled under the following steps:

a) Preliminary selection of web plate for economical depth;
b) Trial flange selection for maximum moment;
c) Check weight estimate;
d) Check design by moment of inertia method;
e) Determine flange thickness reduction points;
f) Transfer of shear stress from web to flange;
g) Design of bearing stiffeners;
h) Design of intermediate stiffeners;
j) Design of splices; and
k) Design of connections to columns, framing beams, and/or supports.

13. PRELIMINARY SELECTION OF WEB PLATE FOR ECONOMICAL DEPTH

13.1 According to IS : 800-1956 the web depth-thickness ratio may be as high as 200 in a girder without longitudinal stiffeners. However, if the depth thickness ratio is kept to 180 or less, the intermediate stiffeners may be placed considerably farther apart in the plate girder.

13.2 The economical web depth may be approximately determined by use of a formula of the type given in the following equation:

$$h = K\sqrt[3]{M/f_b} \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

13.3 Under the radical, M is the maximum bending moment and f_b is the maximum stress permitted which according to IS: 800-1956 would be 1 500 kg/cm^2 for a laterally supported girder. K is a parameter that may vary from 5 to more than 6 depending on the particular conditions. Actually, a considerable variation in h will not change the overall weight of the girder a great deal since the greater flange material required in a girder of lesser depth is offset by the lesser moment of material in the web. *Vawter and Clark propose K values of 5 for welded girders and 4·5 for riveted girders with stiffeners.

13.4 To obtain the minimum weight of steel in plate girder design, several different depths should be used in a variety of preliminary designs to determine the trend of weight with respect to variations in plate girder depth. Of course, the depth of a plate girder may infringe on head room or other clearance requirements and thus be limited by considerations other than minimum weight.

13.5 If longitudinal stiffeners are used, the web depth-thickness ratio may be increased above 200, but in short span plate girders used in building construction use of longitudinal stiffeners introduces considerable complexity in the framing and should be avoided unless a very clear-cut weight saving is established. If a very long span girder of 30 metres or more is required, then the possibility of economy through use of longitudinal stiffeners should be investigated. Such stiffeners are commonly used in continuous span highway bridge girders.

14. TRIAL FLANGE SELECTION FOR MAXIMUM MOMENT

14.1 After selecting the web depth, the preliminary selection of flange area is made on the basis of the common assumption that one-sixth of the area of the web in a welded girder or one-eighth of the web area in a riveted girder represents an equivalent flange area added by the web. The approximate moment capacity of the girder may then be given as follows:

$$M = f_b h \left(A_f + \frac{A_w}{6}\right) \text{(welded)} \quad \ldots\ldots\ldots \quad (9)$$

$$M = f_b h \left(A_f + \frac{A_w}{8}\right) \text{(riveted)} \quad \ldots\ldots\ldots \quad (10)$$

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*VAWTER, J. AND CLARK, J. G. Elementary Theory and Design of Flexural Members. New York. John Wiley & Sons, Inc., 1950.

14.2 In Eq 10, h is the estimated depth centre to centre of flanges. The web area may be determined by using the estimate of economical web depth together with the maximum permissible web depth-thickness ratio. In Eq 10, f_b should be the estimated average allowable flange stress obtained by multiplying the maximum allowable by h/d. The required area of flange material then may be determined directly as will be illustrated in Design Examples 2 and 3.

15. CHECKING OF WEIGHT ESTIMATES

15.1 After the web and flange areas have been approximately determined, the more accurate design weight estimate of the girder should be made. This may be arrived at within close enough design limits by estimating the weight of flange plates (angles, if used), and web plates and adding the following percentages for weights of stiffeners and other details:

a) Welded girders ...	30 percent of web weight
b) Riveted girders with crimped stiffeners ...	50 percent of web weight
c) Riveted girders with filler plates under all stiffeners ...	70 percent of web weight

16. DESIGN BY MOMENT OF INERTIA METHOD

16.1 After the weight estimate check, more accurate moment and shear diagrams may be drawn and the web plate thicknesses revised if necessary. Then the gross moment of inertia of the plate girders should be calculated at all critical sections for bending moment. Calculated bending stresses then should be multiplied by the ratio of gross to net area of flange as specified in **20.1** of IS : 800-1956.

17. DETERMINATION OF FLANGE THICKNESS REDUCTION POINTS

17.1 As illustrated in the design examples, the cut off points for extra cover plates in riveted plate girders or locations where plate thickness, width, or both should be reduced in welded plate girders are determined by drawing horizontal lines indicative of the various capacities of the plate girder at reduced sections. The cut off points are determined as at the intersections between these horizontal lines of moment capacity and the actual moment diagram or envelope of possible moment diagrams for variations in applied loading. As provided by IS : 800-1956, in riveted girders, cover plates should extend beyond their theoretical cut off points by sufficient length to develop one-half of the strength of the cover plate so extended and enough rivets should be added at the end to develop the entire strength of the cover plate. In the case of welded girders where a single flange plate is used, the point where the reduction in flange area is made should be at least 30 cm beyond the theoretical point in the case of girders under primarily static loading. If girders are under large fluctuations of repeated stress leading to possible fatigue failure, the changes in flange area should be made at locations where the unit stress is at less than three-quarters of the maximum allowable and preferably lower.

18. TRANSFER OF SHEAR STRESS FROM WEB TO FLANGE

18.1 From Eq 7 on p. 24, the shear transfer per linear centimetre is determined as $f_s t$ and the rivets or welds are supplied so as to provide the average shear value that is required. Thus, if s or p is the spacing between intermittent welds or rivets and W or R is the weld value of a single intermittent weld or rivet value respectively, the spacing is found by Eq 11 and Eq 12.

$$s = \frac{WI}{VQ} \text{ welded girder} \quad \ldots\ldots\ldots\ldots \quad (11)$$

$$p = \frac{RI}{VQ} \text{ riveted girder} \quad \ldots\ldots\ldots\ldots \quad (12)$$

In welded girders the smallest weld size is the most economical one and continuous welds are preferable to intermittent welds. Here, reference should be made to the discussion on intermittent welds in Sheet 6 of Design Example 1 where cover plates were applied to rolled wide flange sections.

19. DESIGN OF BEARING STIFFENERS

19.1 The function of the bearing stiffener is to transmit concentrations of load so as to avoid local bending failure of the flange and local crippling or buckling of the web. When a column applies load to a girder, either from above or as a reaction support at the underside, bearing stiffeners should be supplied in pairs so that they line up approximately with the flanges of the column. Thus, local bending of the plate girder flanges and resulting requirement for a thick bearing pad is automatically avoided. When the end of a plate girder is supported by a bearing pad and masonry wall, a single pair of bearing stiffeners may be sufficient but the bearing plate shall be thick enough to distribute the local bending loads without causing excessive bending stress in the flanges.

Initially the selection of stiffeners is usually made on the basis of local permissible contact bearing pressure of 1 890 kg/cm^2 at the points of bearing contact between the outstanding parts of the bearing stiffeners and the flanges. The bearing stiffeners may either be cut locally to clear the angle fillets in the riveted girder or welds in the welded girder.

Welds or rivets shall be supplied to transfer the total load from the bearing stiffeners into the web. The bearing stiffeners together with the web plate shall be designed as a column with an equivalent reduced slenderness ratio. In the case of riveted bearing stiffeners, filler plates shall be used.

20. DESIGN OF INTERMEDIATE STIFFENERS

20.1 The primary purpose of the intermediate stiffener is to prevent the web plate from buckling under a complex and variable stress situation resulting from combined shear and bending moment. Obviously compression stress predominates in the upper part of the girder. By breaking the web plate up into small panels supported along the lines of the stiffeners, the resistance of the plate to buckling under the complex stress pattern is measurably increased and the code design rules provide a conservative design basis. Intermediate stiffeners have a secondary function not generally recognized in that, if fitted against the flanges at top and bottom, they maintain the original 90° angle between flange and web. Some designers do not require the intermediate stiffeners to be against the flanges and this is probably unnecessary if the girder is adequately braced laterally along the compression flange. The stiffeners may perform their function with regard to web buckling without being fitted. However, if the girder is laterally unsupported or is subject to torsion due to any cause, there will be a tendency of the flanges to deflect laterally and independently of the web. This will cause local bending stress at the juncture between web and flange and will also reduce effectively the torsional resistance of the plate girder, which is important both with respect to lateral buckling and combined bending and torsion. Therefore, it would be good practice in the case of laterally unsupported girders to make all intermediate stiffeners fitted by adequately tack welding against both compression and tension flanges.

21. DESIGN OF SPLICES

21.1 Long simple span plate girders or continuous plate girders with segments too long to ship or handle conveniently during erection shall be spliced. Preferably, splices should be located away from points of maximum bending moment. The problem is much more complex in the riveted girder than in the welded girder where simple butt welds are fully effective in essentially providing a continuous plate for either web or flange. In the riveted plate girders spliced plate material shall be added on both sides of web and flange at splice points. Splice design procedure will be illustrated in Design Example 2.

22. DESIGN OF END CONNECTIONS

22.1 The design of end connections is essentially the same as for rolled beams. Web plates or angles may be used in welded and riveted girders respectively and stiffened seats may also be used. If the connection is to a column web by means of web angles or plates, an erection seat should always be provided to support the girder while the main connection is being made.

Deep plate girders framing to stiff columns are preferably supported by stiffened seats and flexible top angles (for lateral support). This avoids a tendency for the top of the girder to tear away from the column connection. End bearing stiffeners will be required in this case.

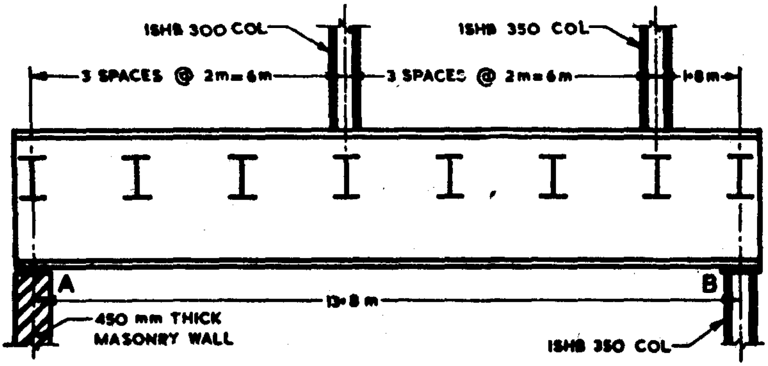
23. DESIGN EXAMPLE OF WELDED PLATE GIRDER

23.1 Design of a welded plate girder is illustrated in the following 15 sheets (*see* Design Example 2).

Design Example 2 — Welded Plate Girder

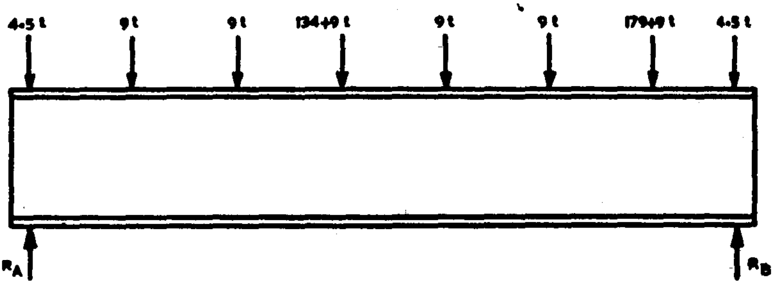
The sketch shows the general layout of loads and supports as required by the architectural features of the building. The columns are indicated as bearing directly on the top flange of the girder and the top surface shall be left smooth to provide uniform bearing support. Floor beams frame into the girder at 2 m centre to centre.

Design Example 2	**1 of 15**
Design Problem Cited	



A welded plate girder of 13·8 m span is supported by a concrete wall at *A* and by a ISHB 350 column section at *B*. The girder supports columns, at two points and floor beams that frame at 2 m c/c, except at the extreme right where the offset column and floor beam are at the same location at 1·8 m from the right end.

The loads introduced by the floor beams and columns are as shown below:



Design Example 2

Bending Moment & Shear Forces

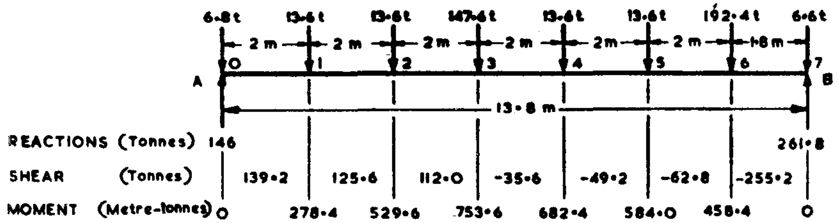
2 of 15

The uniform load on the girder together with a very preliminary estimate of its dead weight, based on experience, is transformed to equivalent concentrated loads at the points of floor beam framing. Reactions and moments are computed numerically. By Eq 8 (see p. 50), the economical depth of the plate girder is estimated at 185 cm. Of course, questions of over-all building height and clearance may have an important effect and the greater economy of a deeper plate girder may be offset by additional column and other material that might be required in other parts of the building owing to the increased over-all height.

The girder carries a uniform load of 1·4 t/m and its dead weight is estimated to be 0·9 t/m. Therefore, total additional load = 2·3 t/m.

Transforming the uniform load into equivalent concentrated loads at the locations of the floor beams:

$$\left.\begin{array}{l} 2 \times 2{\cdot}3 = 4{\cdot}6 \text{ t} \\ 1{\cdot}8 \times 2{\cdot}3 = 4{\cdot}1 \text{ t} \end{array}\right\}$$ These are suitably added to the given concentrated loads shown in the bottom sketch of Sheet 1, and shear and moment values are determined as shown in the following sketch.



$$\text{Over-all depth} = 5\left(\sqrt[3]{\frac{M}{f}}\right)$$

$$= 5 \times \left(\sqrt[3]{\frac{753{\cdot}6 \times 1\,000 \times 100}{1\,500^{*}}}\right)$$

$$= 185 \text{ cm}$$

Assuming flange thickness = 5 cm

Web depth = 175 cm

---

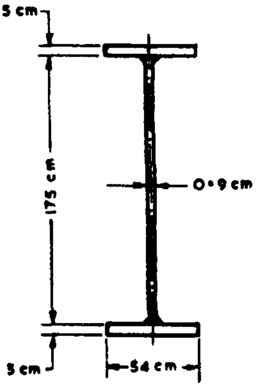
*As length between effective lateral supports is only 2 m, it is assumed throughout the calculations in this Design Example that F_b = 1 500 kg/cm^2.

Having selected the web plate, the trial selection for maximum flange area required is made as previously discussed in **14.1**.

Design Example 2	3 of 15
Design of Girder	

Minimum thickness of web plate to avoid use of horizontal stiffeners (*see* **20.6.1** and Table XI in IS: 800-1956) $= \frac{175}{200} = 0{\cdot}875$ cm

TRIAL FLANGE SECTION



Try the section shown in the sketch.

One-sixth of the web area may be counted as part of the flange area.

Assumed flange stress $= \frac{180 \times 1\,500}{185}$

$= 1\,460$ kg/cm

Moment capacity $= 1\,460\left(A_f + \frac{A_w}{6}\right) \times 180$

$$\therefore\ 1\,460\left(A_f + \frac{175 \times 0{\cdot}9}{6}\right)180 = 753{\cdot}6 \times 100 \times 1\,000$$

$$A_f = 260{\cdot}5 \text{ cm}^2$$

Try 54 cm × 5 cm flange plate.

Check by moment of inertia procedure

$$I_{\text{web}} = \frac{0{\cdot}9 \times 175^3}{12} = 402\,000 \text{ cm}^4$$

$$I_{\text{flange}} = 2 \times 270 \times 90^2 = \underline{4\,374\,000 \text{ cm}^4}$$

$$4\,776\,000 \text{ cm}^4$$

The moment of inertia of the flanges about their own centroid is neglected.

Bending stress, $f_b = \frac{753{\cdot}6 \times 10^5}{4\,776\,000} \times 92{\cdot}5 = 1\,460 < 1\,500$ kg/cm ..... OK.

The web plate as chosen on the basis of maximum depth thickness ratio of 200 is satisfactory throughout most of the girder but at the right end between the column and the support a thicker web plate is required as shown in the calculations on this sheet. It would be uneconomical of material to carry a heavy web throughout the whole length and a web splice, therefore, is introduced at the left of the 35-cm column load. Since the variation in dead weight has little effect on the maximum bending moment, no change is made in these calculations. Dead weight becomes increasingly important with increasing span of any given type structure. Thus, as the span increases, greater and greater care shall be taken in the proper estimate of dead weight. In order to save weight on flange material, the moments of inertia of three different tentative sections are now calculated embodying lesser thicknesses of flange plate than required at the location of maximum moment. As one assumed thinner and thinner flange thicknesses, care shall be taken to stay within the limit of the width-thickness ratio which shall be less than 16.

Selection of Web Plates at Ends

Left end *Max* shear $= 139\cdot2$ t; *Min* area required $= \dfrac{139\cdot2 \times 1\,000}{945} = 148$ cm^2

$\therefore$ The area $175 \times 0\cdot9 = 157\cdot5$ cm^2, that is provided is OK.

Right end *Max* shear $= 255\cdot2$ t; *Min* area required $= \dfrac{255\cdot2 \times 1\,000}{945} = 270$ cm^2

$\therefore$ Use web plate of $\dfrac{270}{175} = 1\cdot54 =$ *say* 1·8 cm thickness

Use $175 \times 1\cdot8$ cm plate.

The trial web selection permits the design of the bearing stiffeners that are required at reactions and concentrated load points [*see* **20.7.2.1** (b) of IS: 800-1956]

Check dead weight estimate

Web area $= 157\cdot5$ cm^2
Flange area $= 540\cdot0$ cm^2
Stiffeners and other details (40 percent of web) $= 63\cdot0$ cm^2
$760\cdot5$ cm^2

Weight per metre $= 760\cdot5 \times 0\cdot785 = 598$ kg < 900 kg ..... OK.
(Overly on safe side but variation has little effect on maximum BM)

Flange end section: Try $\dfrac{54}{2 \times 16} = 1\cdot7$, use minimum 2-cm thick plate.

$I_{\text{web}} = 402\,000$ cm^4

I_{flange} with plate 54×5, $2 \times 270 \times 90^2 = 4\,374\,000$ $I = 4\,776\,000$ cm^4
,, with plate $54 \times 3\cdot6$, $2 \times 182\cdot4 \times 89\cdot3^2 = 2\,908\,000$ $I = 3\,310\,000$ cm^4
,, with plate 54×2, $2 \times 108 \times 88\cdot5^2 = 1\,695\,000$ $I = 2\,097\,000$ cm^4

Moment capacity in metre-tonnes for:

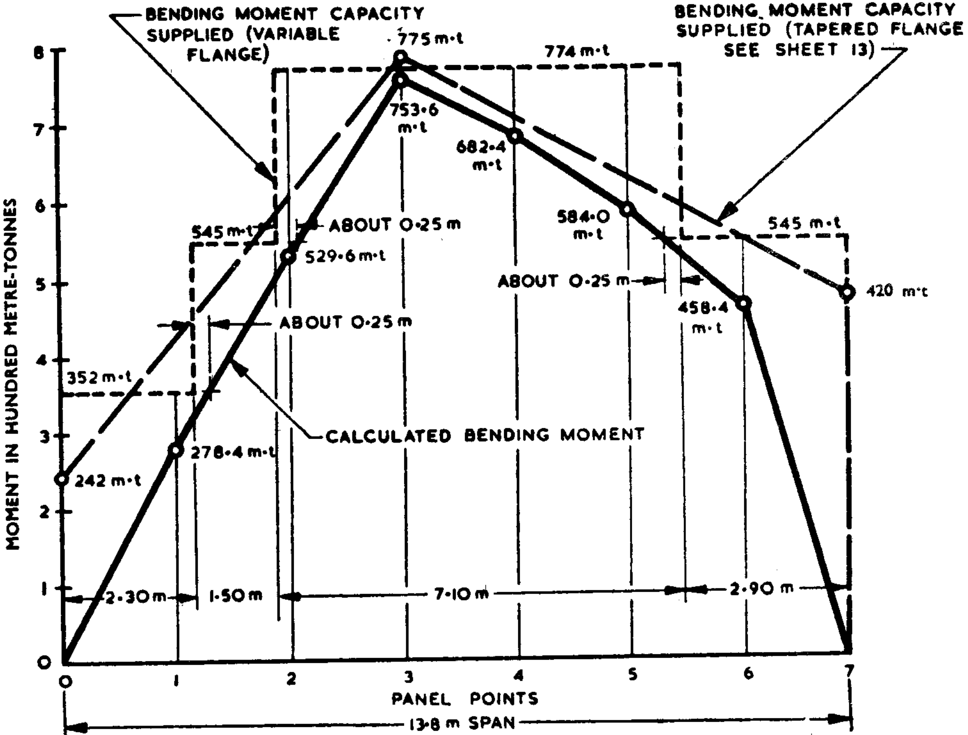
Plates 54×5, $M = \dfrac{4\,776\,000 \times 1\,500}{92\cdot5 \times 10^6} = 774$ m·t

Plates $54 \times 3\cdot6$, $M = \dfrac{3\,310\,000 \times 1\,500}{91\cdot1 \times 10^6} = 545$ m·t

Plates 54×2, $M = \dfrac{2\,097\,000 \times 1\,500}{89\cdot5 \times 10^6} = 352$ m·t

Design Example 2	5
Design of Girder	of 15

A sketch showing the Bending Moment diagram is drawn on this sheet. Horizontal lines denoting the bending moment capacities of the different flange sections, calculated on the previous sheet are drawn to determine the points of cut off in the different flange sections.



Bearing stiffeners are now designed. Under the columns, these are placed in duplicate pairs directly below the column flanges and 2 cm is deducted in the calculations from the length of the outstanding leg for cropping to provide weld clearance. The design of all-welded details in this example is governed by IS: 816-1956 as well as by IS: 800-1956. Thus, the use of intermittent welds is necessary because a smaller continuous weld, though adequate, would violate requirement of being too small a weld for the plate thickness in question.

Bearing Stiffeners

If four stiffeners are used (giving metal to metal transfer through the flange), it might be desirable to use 20-cm stiffeners.

Minimum thickness $= \frac{20}{16} = 1{\cdot}25$ cm (*see* **18.5.1.1** of IS: 800-1956)

Allow 2·0 cm for web clearance and avoidance of triaxial stress.

Try thickness 1·3 cm.

Bearing capacity $= 4\,(20-2)\;1{\cdot}3\times1\,890^{*} = 177$ t

This represents minimum capacity desirable for the 134 t and 179 t column loads but is inadequate for the right reaction:

Thickness required for right reaction $= t = \frac{261{\cdot}8\times1\,000}{4\,(18)\times1\,890} = 1{\cdot}95$, *say* 2·0 cm

Use 20 × 2 cm plates.

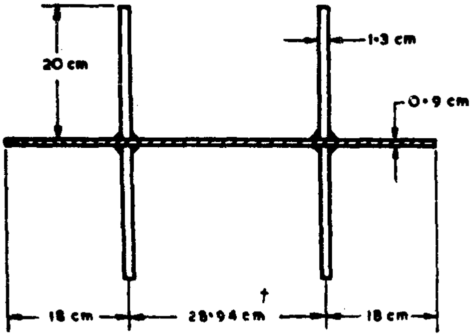
Check: Bearing stiffeners for strut action

Actual shear value V for left column load = 134 t

Area of stiffeners for web $= 4\times1{\cdot}3\times20 = 104\ \text{cm}^2$

$0{\cdot}9\times64{\cdot}94^{\dagger} = 58{\cdot}4\ \text{cm}^2$

$162{\cdot}4\ \text{cm}^2$



This column section being supported has been assumed ISHB 300 (*see* Sheet 1). The distance between the bearing stiffeners should be such that they are against the flanges of the column section. Hence, 28·94 cm c/c.

Moment of inertia $= 2\times1\,3\left(\frac{40{\cdot}9}{12}\right)^{3}$

$= 14\,820\ \text{cm}^4$

$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{14\,820}{162{\cdot}4}}$

$= 9{\cdot}56$ cm

$l/r = \frac{0{\cdot}7\times175}{9{\cdot}56} = 12{\cdot}8$ (*see* **20.7.2.2** of IS: 800-1956)

From **9.1.2** and Table I in IS: 800-1956, permissible stress = 1 230 kg/cm²

∴ Load capacity = 1 230 (162·4) = 200 t > 134·0 t

Thus as a strut the stiffener is OK.

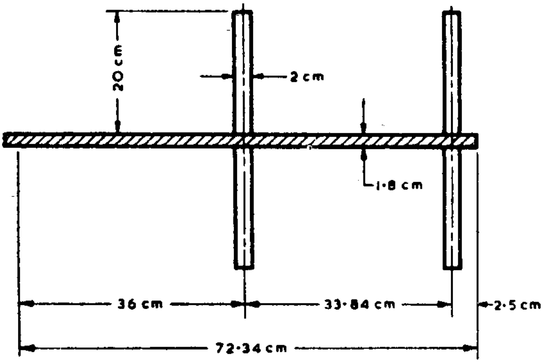
---

* *See* **9.4** of IS: 800-1956.

† c/c distance of flange of ISHB 300 column section being supported.

Right Support Bearing Stiffeners

No need to check strut action at right load point since web plate there is greater than at left and allowable stress is not much different.



At right support

Area of web $= 1{\cdot}8 \times 72{\cdot}3 = 130$ cm²

Stiffener area $= 4\ (20)\ (2) = 160$ cm²

$$290 \text{ cm}^2$$

Section Properties

$$\text{Moment of inertia, } I_{xx} = \frac{2\times2\times(41{\cdot}8)^3}{12} = 24\,300 \text{ cm}^4$$

$$r = \sqrt{\frac{24\,300}{290}} = 9{\cdot}15 \text{ cm}$$

$$l/r = \frac{0{\cdot}7\times175}{9{\cdot}15} = 13{\cdot}4$$

$$F_c = 1\,228 \text{ kg/cm}^2$$

Capacity of strut = 1 228 × 290 = 356 t > 261·8 t (*see* R_B on Sheet 2) ..... OK.

Welds for stiffeners (*see* IS: 816-1956)

Strength of weld required $= \dfrac{261{\cdot}8}{8\times175} = 187$ kg/cm

For 2·0-cm plate *Min* size of fillet weld = 6·0 mm (*see* **6.2.2** of IS: 816-1956)

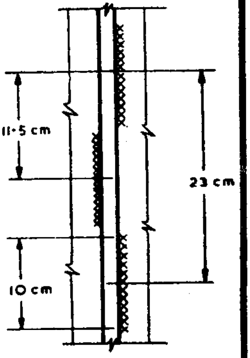
Capacity of weld = 0·7 (0·6) (1 025) = 430 kg/cm (*see* **6.2.3** and **7.1** of IS: 816-1956)

Use intermittent welds, 10 cm long.

Spacing c/c welds required $= \dfrac{10\times430}{187} = 23$ cm

Permissible *Max* clear spacing = 16 × 1·8 = 28·8 cm (*see* **6.2.6.2** of IS: 816-1956)

Use 10 × 0·6 cm @ 23 cm c/c (staggered).



Design Example 2

Left Support Concrete Wall Bearing Plate & Bearing Stiffeners

8 of 15

A bearing support is assumed on a 45-cm concrete wall at the left end to illustrate the design problems that are involved. A single pair of bearing stiffeners is used to centralize the load over the wall. The bearing plate will supply the necessary bending stiffness to distribute the load from the bearing stiffener to the masonry support. The permissible stress for local bearing on masonry determines the overall area requirement for the bearing plate.

Bearing Plate Design

At left support, concrete wall 45 cm thick, a single pair of stiffeners is desired to localize load near centre of wall and minimize welding.

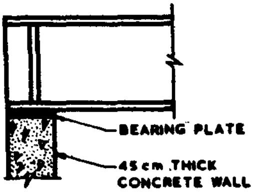
Local bearing stress on concrete assumed is 40 kg/cm^2.

$$\text{Length of bearing plate} = \frac{146 \times 1\,000}{40 \times 45} = 81{\cdot}2 \text{ cm}$$

Use 45 × 82 cm bearing plate.

$$\text{Actual masonry bearing pressure} = \frac{146\,000}{82 \times 45} = 39{\cdot}6 \text{ kg/cm}^2 \; \ldots\ldots \text{OK.}$$

Bending stress in bearing plate,
allowable F_b = 1 890 kg/cm^2 (*see* **9.2.3** of IS: 800-1956)



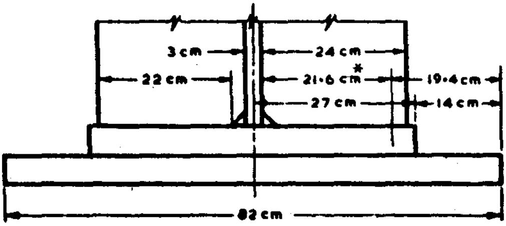
$$\text{Moment} = \frac{19{\cdot}4^2}{2} \times 45 \times 39{\cdot}6$$

$$= 320\,000 \text{ cm·kg}$$

$$Z = \frac{45t^2}{6} = 7{\cdot}5t^2$$

$$t = \sqrt{\frac{320\,000}{1\,890 \times 7{\cdot}5}} = 4{\cdot}75 \text{ cm}$$

Use 82 × 45 × 5 cm bearing plate.



Bearing Stiffener

Try 24-cm plate with cropped corner to clear weld leaving 22 cm contact length:

$$\text{Thickness} = \frac{146}{2 \times 22 \times 1{\cdot}89} = 1{\cdot}75 \text{ cm}$$

Use 24 × 1·8 cm stiffener plate.

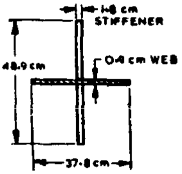
*0·8 × 27 = 21·6 (assumed) $\frac{b}{t} = \frac{24}{1{\cdot}8} = 13{\cdot}3 < 16$ ….. OK (*see* **18.4.2** and Table VII of IS: 800-1956).

---

*It is assumed that (column load) reaction is uniformly distributed on the rectangle, 0·95 d × 0·80 b where d and b are the dimensions of the rectangular bearing plate.

Design Example 2	9 of 15
Intermediate Stiffeners	

The intermediate stiffeners are put in at maximum permissible spacing and their adequacy according to the specification is then checked. It is to be noted that the depth-thickness ratio of the web exceeds 180 and that according to **20.7.1** *of IS: 800-1956 the maximum spacing is, therefore, 180 times the web thickness. Had the depth-thickness ratio been kept less than 180, the maximum spacing could have been increased to 1·5 times the depth of the girder. This is a rather important point and it is possible that a smaller depth girder might have been slightly more economical of steel. Some studies were made as to the possibility of using a still thinner web with longitudinal stiffeners but no appreciable weight-saving would have resulted. Interference between the horizontal stiffener and the local floor beam framing connections would have increased the cost of fabrication.*



$$\text{Area} = 37{\cdot}8 \times 0{\cdot}9 + 48 \times 1{\cdot}8 = 120{\cdot}42 \text{ cm}^2$$

$$\text{Stress} = \frac{146 \times 1\,000}{120{\cdot}42} = 1\,215 \text{ kg/cm}^2 < 1\,228 \text{ .....OK.}$$

(Obtained with earlier stiffener design, *see* Sheet 7)

Intermediate Stiffeners

	LEFT END AND CENTRE	RIGHT END
Web thickness	0·9	1·8
d/t	$\frac{175}{0{\cdot}9} = 194{\cdot}4$	$\frac{175}{1{\cdot}8} = 97{\cdot}2$

Stiffeners in region 1-6 (left end and centre)

175×0·9 cm plate from left end to right column

The required shear stress to be carried in the web is:

$$\frac{139{\cdot}2 \times 1\,000}{175 \times 0{\cdot}9} = 884 \text{ kg/cm}^2, \qquad \frac{d}{t} = 194{\cdot}4$$

According to Table IIIA of IS: 800-1956 or Table IV of this Handbook (*see* p. 182), (for the other dimension of the panel) the spacing should be 0·53 *d* (roughly).

∴ Use a spacing of 0·53×175 = 93 cm *or* 90 cm, *say* in the region 0-1.

In the panel 1-2, shear = 125·6 t; Shear stress in web $= \frac{126{\cdot}6 \times 1\,000}{175 \times 0{\cdot}9}$

$= 793$ kg/cm²

which allows a spacing of 0·80 *d* (*see* Table III on p. 174) or 140 cm

NOTE — The required spacing is 90 cm in the region from left end to location 1, and 140 cm from location 1 to location 2. Based on these figures, the actual spacing required may be adjusted suitably to give symmetrical spacing and good appearance. It should also be seen that the floor beam connections are not interfered in any way. In the region 2-6, the maximum permissible spacing of 162 cm may be used.

Design Example 2 | **10 of 15**

Intermediate Stiffeners

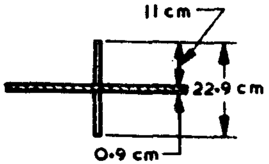
The required moment of inertia of the intermediate stiffeners is determined. In the present design, almost any reasonable selection of stiffener will satisfy the moment of inertia requirement. Although some designers will prefer to use intermediate stiffeners in pairs, this is not absolutely necessary provided adequate support is supplied and the stiffeners are not depended upon to maintain the shape of the cross-section. Some saving in steel weight could be effected here by using stiffeners on one side only but they are called for on both sides in this design as representing best design practice.

Moment of inertia of stiffeners about centre of web should be

$$1{\cdot}5\,\frac{d^3t^3}{c^2}\quad(\textit{see}\ \textbf{20.7.1.1}\ \text{of IS: 800-1956})$$

$$= 1{\cdot}5 \times \frac{(175)^3 \times (0{\cdot}9)^3}{(90)^2}$$

$$= 725\ \text{cm}^4$$



It is a good practice to use the outstanding leg of the stiffener not less than 5 cm plus 1/30 the depth of beam and its thickness 1/16 the width of the outstanding leg.

$$\therefore\ \text{Stiffener width} = 5 + \frac{175}{30}$$

$$= 11\ \text{cm}$$

$$\text{Thickness} = \frac{11}{16}$$

$$= 0{\cdot}7\ \text{cm}$$

$$\text{For } 11 \times 0{\cdot}8\ \text{cm},\ I_{xx} = \frac{0{\cdot}8\ (22{\cdot}9)^3}{12}$$

$$= 800\ \text{cm}^4 > 725\ \ldots\ldots\ \text{OK.}$$

Stiffeners in the region 6-7

$$d/t = \frac{175}{1{\cdot}8} = 97{\cdot}2$$

$$\text{Spacing } c = 180 - 33{\cdot}84 = 146{\cdot}16\ \text{cm}$$

$$c/d = \frac{146{\cdot}16}{175} = 0{\cdot}935$$

$$F_s = 945\ \text{kg/cm}^2\ \ldots\ldots\ \text{OK.}$$

Bearing stiffeners eliminate need of intermediate stiffeners in this region.

Design Example 2

Welding of Web & Flange Plates

11 of 15

The longitudinal transfer of shear stress between web plate and flange plate in the various panels along the girder is tabulated and requisite fillet welds are chosen. If the minimum size fillet weld permits a continuous weld without excessive loss in economy, such continuous welds are desirable, and should be used in any case at the ends of the girder as specified in IS: 800-1956. Reference should be made to **18** *(see p. 52).*

Welds

From Eq 7:

$$f_s = \frac{VQ}{It}, \qquad f_s t = \frac{VQ}{I}$$

$$\text{Thus shear per linear cm} = tf_s = f_v = \frac{VQ}{I} \times 1\,000 \text{ kg/cm}$$

PANEL	V MAXIMUM SHEAR tonnes	*Q cm^3	I cm^4	f_v kg/cm^2	2 FILLET WELDS
0-1	139·2	9 550	2 097 000	634	†6 mm continuous $(2 \times 430{\cdot}5 = 861$ kg/cm)
1-2	125·6	17 385	3 310 000	656	†9·5 mm × 23 mm @ 32 cm c/c $(922{\cdot}5 \times \frac{23}{32} = 663$ kg/cm)
2-5	112·0	24 000	4 776 000	569	†9·5 mm × 22 cm @ 35 cm c/c $(922{\cdot}5 \times \frac{22}{35} = 580$ kg/cm)
5-6	62·8	17 385	3 310 000	328	†9·5 mm × 10 cm @ 29 cm c/c $(922{\cdot}5 \times \frac{10}{29} = 350$ kg/cm)
6-7	255·2	17 385	3 310 000	1 334	†9·5 mm continuous $(2 \times 681{\cdot}5 = 1\,363$ kg/cm)

NOTE — In joining 3·6 cm and 5 cm thick plates, 9·5-mm fillet weld is the minimum permissible according to IS: 816-1956. In 1-2, 2-5 and 5-6, the web plate thickness is 0·9 cm. The strength of web plate in shear = 0·9 × 945 kg/cm. Since this is localized, a maximum of 1 025 (*see* **9.3.1** of IS: 800-1956) may be assumed to give 0·9 × 1 025 = 922·5 kg/cm length of plate. Compared to this, the strength of 9·5-mm weld lines on both sides of web plate = 2 × 0·7 × 0·95 × 1 025 = 1 363 kg/cm. Hence, the value 922·5 kg/cm controls.

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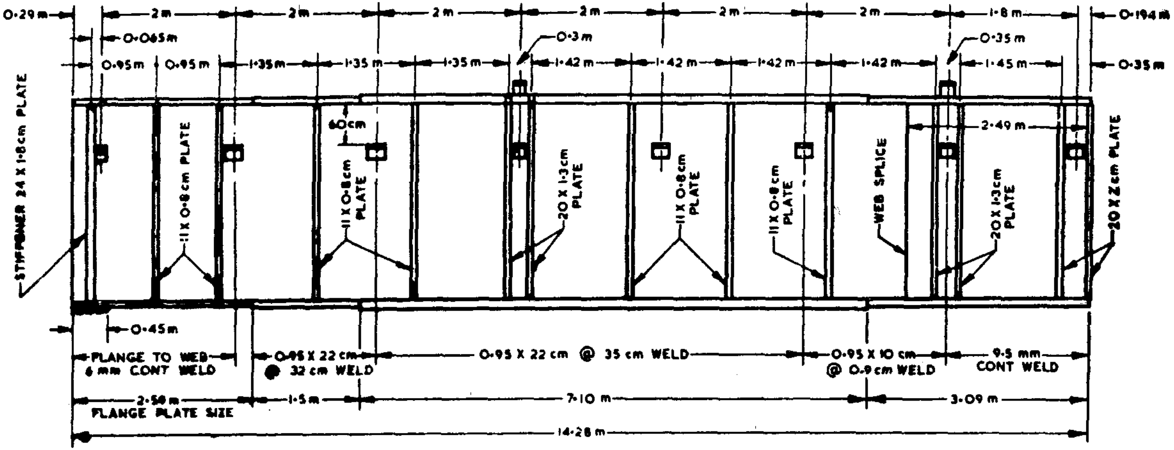
*Q = moment of flange area about neutral axis.

†*See* **7.1** of IS: 816-1956.

Design Example 2

Detailed Working Drawing

12 of 15



Design Example 2

Alternative Tapered Flange Design

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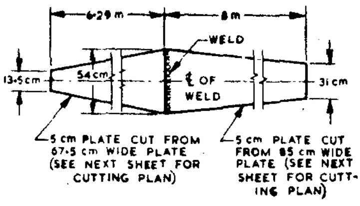
An alternative tapered flange design is shown in the sketch. Each of the two flange segments may be flame cut on the skew with little loss of metal as shown in the sketch. The tapered flange will save an appreciable amount of steel because of the closer fit of the moment capacity along the girder to the actual bending moment. The moment capacity is plotted as the line in dashes on Sheet 5. The introduction of the taper involves both advantages and disadvantages which are tabulated as follows:

Advantages

1) *Saving in steel.*
2) *Elimination of two transverse butt welds in each flange.*
3) *Reduction of stress concentration in the flange.*

Disadvantages

1) *Flanges shall be made in pairs by a longitudinal flame cut (in 67·5 cm width and 85 cm width plates for the above flange section).*
2) *In addition to the cost of flame cutting (which, however, should be more than offset by the saving in reduced welding) the flange plates may warp when they are split longitudinally owing to the existence of residual stresses. This would either require straightening afterward or annealing prior to flame cutting.*
3) *The fitting up of the girder would be more difficult because of the taper.*
4) *In supporting the girder at point of bearing or at concentrated load points, the local details may be a bit more complicated because of the taper.*



Design Example 2 | **14 of 15**

Details of Tapered Flange Design

The moment capacity of this 13·5 cm width flange (at left end) is:

$$M = \frac{fI}{y} = \frac{1\,500 \times {}^{*}1\,495\,500}{92{\cdot}5 \times 100 \times 1\,000} = 242 \text{ m·t}$$

At location 3:

$$M = \frac{1\,500 \times 4\,776\,000}{92{\cdot}5 \times 100 \times 1\,000} = 775 \text{ m·t} > 753{\cdot}6 \text{ (\textit{see} Sheet 2) } \ldots\ldots \text{OK.}$$

At location 4, a capacity of 682·4 m·t is required.

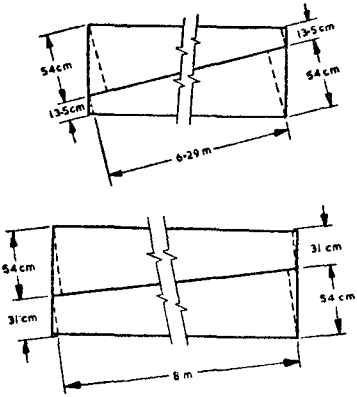
The strength at 0·3 m beyond location 4 will be kept at 682·4 m·t.

$$\therefore \text{ Required } I \text{ of section} = \frac{682{\cdot}4 \times 92{\cdot}5 \times 1\,000 \times 100}{1\,500} = 4\,200\,000 \text{ cm}^4$$

$$\text{Required } b = \frac{4\,200\,000 - 402\,000}{2 \times 5\ (88{\cdot}75)^2} = 48{\cdot}2 \text{ cm}$$

In 2 m length the reduction in width $= 54{\cdot}0 - 48{\cdot}2 = 5{\cdot}8$ cm

In 7·8 m length the reduction in width $= \frac{7}{2} \times 7{\cdot}8 = 27{\cdot}3$ cm $=$ *say* 27 cm



CUTTING PLAN

The available width at the end if this rate of reduction in width of plate is adopted as 54 − 27 = 27 cm > 0·25 (54) .... OK (required in IS: 800-1956, *see* Note under Table XIX of Appendix E).

The moment capacity for this flange width (at right end):

$$\frac{[27 \times 2 \times 5 \times (90)^2 + (402\,000)]\ 1\,500}{92{\cdot}5 \times 100 \times 1\,000} = 420 \text{ m·t}$$

With these values of the moment capacity, the capacity diagram is drawn in Sheet 5. It may be seen that it is adequate. Width at right end to be adopted which is at a distance of 7·99 *m from section* 3 = 13·5 cm.

Quantity of two tapered flanges (rect plate required):

$$5{\cdot}0\ (67{\cdot}5 \times 6{\cdot}29 \times 100 + 85 \times 8 \times 100) = 550\,000 \text{ cm}^3$$

The variable thickness flange required

$$= (2 \times 259{\cdot}0 \times 2 + 2 \times 150 \times 3{\cdot}6 + 710 \times 2 \times 5 + 3{\cdot}09 \times 3{\cdot}6 \times 2)54$$

$$= 617\,490 \text{ cm}^3$$

Thus the tapered beam is lighter by 530 159 kg (66 990 cm³) of flange plate per girder.

$^{*}I_{\text{web}}$ $= 402\,000$

$I_{\text{flange}} = 13{\cdot}5 \times 5 \times 5 \times 2 \times 90^2 = 1\,093\,500$

Total $= 1\,495\,500 \text{ cm}^4$

Weight take off of actual design with breakdown of various components: *Stiffeners and other details were found to be 45 percent of the weight of the web plate, comparing with the figure of 40 percent (based on the 175 × 0·9 cm web plate only) that was assumed.*

Design Example 2

Comparison of Weights of Welded & Tapered Plate Girders

15 of 15

Weights Take Off

1) *Welded plate girder (with flanges of variable thickness)*

Flanges	2–54×2 @ 0·79 kg/m/cm^2 for 2·59 m	=	430
	2–54×3·6 @ 0·79 kg/m/cm^2 for 1·5 m	=	460
	2–54×5 @ 0·79 kg/m/cm^2 for 7·10 m	=	3 030
	2–54×3·6 @ 0·79 kg/m/cm^2 for 3·09 m	=	950
			4 870 kg
Web plates	1–175×0·9 @ 0·79 kg/m/cm^2 for 11·79 m	=	1 460
	1–175×1·0 @ 0·79 kg/m/cm^2 for 2·49 m	=	620
			2 080 kg
Stiffeners	2–24×1·8 @ 0·79 for 1·75 m	=	119·5
	2×2–20×2 @ 0·79 for 1·75 m	=	221·0
	2×4–20×1·3 @ 0·79 for 1·75 m	=	288·0
	2×7–11×0·8 @ 0·79 for 1·75 m	=	170·5
			799·0 kg

$$\frac{\text{Stiffeners}}{\text{Web}} = \frac{799}{2\,080} = 0{\cdot}384 \text{ or } 38{\cdot}4^{*} \text{ percent}$$

2) *Welded plate girder (with tapered flanges)*

Flanges:

From Sheet 14, 550 000 (cm^3) × 0·007 9 (kg/cm^3) = 4 350 kg

3) *Total welded girder*

Variable thickness flanges: 7 749 kg

Tapered flanges : 7 229 kg

---

*Estimate of 40 percent was based on 175×0·9 cm web plate; on the same basis the proportion of stiffener actually required is $\frac{799}{1\,770} = 0{\cdot}45$ *or* 45 percent.

24. RIVETED PLATE GIRDER

24.1 The riveted plate girder provides a beam that has behind it many decades of successful design experience. Because of the following factors, the riveted girder will use more steel than its welded counterpart. It shall be designed with reduced stress allowed on the tension side on account of net section. Stiffeners shall be angles instead of plates, and filler plates shall be used under bearing stiffeners.

24.2 The following are weights of the welded girder as designed in Design Example 2 both with conventional variable thickness cover plates and with tapered width flange plates along with riveted plate girder design:

Welded girder, conventional flanges:	7 749 kg
Welded girder, tapered width flanges:	7 229 kg
Riveted girder (Design Example 3):	*9 602 kg

25. DESIGN EXAMPLE OF RIVETED PLATE GIRDER

25.1 The riveted plate girder is designed with exactly the same load conditions and general dimensions as the welded plate girder in Design Example 2, in the following 14 sheets (*see* Design Example 3).

---

*Could be reduced to 8 525 kg by crimping intermediate stiffeners.

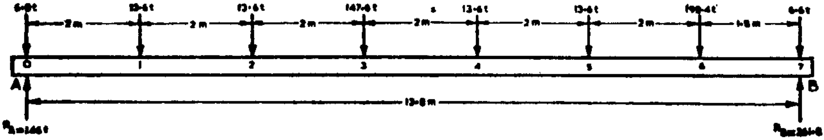
Design Example 3 — Riveted Plate Girder

Design Example 3 | **1 of 14**
Design of Girder

Loading, shear, and moment estimates are repeated from welded girder Design Example 2. Although the riveted girder uses more steel than its welded counterpart, the estimated allowance for girder dead weight was somewhat too high in the welded girder design and the same weight estimate should be satisfactory for the riveted girder design. The over-all depth by Eq 8 (see p. 50) involves a k factor of 5·5 instead of 5·0 as used for the welded girder. This is as recommended by Vawter and Clark. The riveted girder should be somewhat deeper than a welded girder for maximum economy because the flange angles bring the distance between the centroids of the flanges somewhat nearer together than in the welded girder. In addition, the web plate is supported at the edges of the flange angles and the over-all depth of the girder may, therefore, be relatively greater without exceeding the allowable web depth-thickness ratio. It will be noted at the bottom of Sheet 1 that 1/8 of the area of the web has been included in the flange area.*

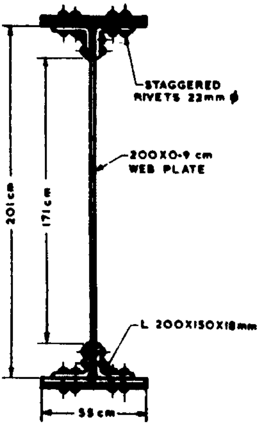
Riveted Plate Girder

Loading: (Same as Design Example 2)



The moments and shears from Sheet 2 (Design Example 2)

$$\text{Over-all depth } d = 5{\cdot}5\sqrt[3]{\frac{M}{f}} = 5{\cdot}5\sqrt[3]{\frac{753{\cdot}6}{1\,500^{\dagger}}} \times 100 \times 1\,000$$
$$= 36{\cdot}8 \times 5{\cdot}5 = 202 \text{ cm}$$



Clear distance between flanges (assuming 200-cm web depth and 0·5 cm clearance gap and 150 × 200 mm flange angle)

$= 200 + (2 \times 0{\cdot}5) - 2 \times 15 = 171$ cm (*see* sketch)

$$\text{Minimum web thickness} = \frac{171}{200} = 0{\cdot}85 \text{ cm}$$

Use web plate 0·9 × 200 cm.

If over-all depth is assumed to be 210 and effective depth 200 cm

$$\text{Average flange stress} = \frac{1\,500 \times 200}{210} = 1\,429 \text{ kg/cm}^2$$

$$\text{Area required} = \frac{753{\cdot}6 \times 100 \times 1\,000}{200 \times 1\,429} = 264 \text{ cm}^2$$

Trial section: (1/8) web 200 × 0·9 cm = 22·5 cm²
2 angles 150 × 200 × 18 = 59·76 × 2 = 119·5 cm²
Cover plates 55 × 1·25 cm = 68·8 cm²
Cover plates 55 × 1·4 cm = 77·0 cm²
Total area = 287·8 cm²

---

*VAWTER, J. AND CLARK, J. G. Elementary Theory and Design of Flexural Members, New York. John Wiley and Sons Inc., 1950.

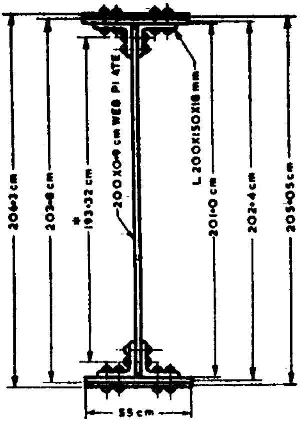
†See footnote on p. 56.

Design Example 3	2 of 14
Design of Girder	

The table at the top of the sheet provides a computation of gross moments of inertia for the plate girder without cover plates, with one cover plate, and with two cover plates. The moments of inertia without cover plates and with one cover plate will be needed both for determining cut-off points of the flange cover plates, and, in addition, to determine rivet pitch. Gross moment of inertia should also be used in deflection calculation. The static moments of the various flange areas both with and without angles are tabulated as these are needed later to determine rivet pitch.

Calculate gross and net flange areas and gross moment of inertia 0, 1 and 2 cover plates:

Part	Dimensions (cm)	Gross Area of Flange	Deduction	Net Area (Tensile Flange)	Inertia Multiplier	Gross I_{xx}
Web	200 × 0·9	80	—	—	$\frac{(200)^2 \times 1}{12}$	600 000
Web between angles	14·5 × 0·9 × 2	26·1	1 × 2·3 × 0·9	24·03	—	—
Angles	20·0 × 15·0 × 1·8	119·52	**(4 × 2·3 × 1·8)	102·94	†18 700	2 220 000
Angles	—	—	—	—	4 × 1 136·9	4 545
Flange with zero cover plate	—	119·52 ‡(+26)	§2 × 2·3 × 1·8	111·20 (+24)	—	2 824 454
1 cover plate	55 × 1·40	77	2 × 2·3 × 1·4	70·6	20 500	1 578 500
Flange with 1 cover plate	—	196·52 (+26)	—	173·54 (+24)	, —	4 403 045
Second plate	55 × 1·25	68·75	2 × 2·3 × 1·25	63·0	21 000	1 444 000
Flange with 2 cover plates	—	265·27 (+26)	—	236·54 (+24)	—	5 847 045



Moment capacity, M_o (*see* **20.1** in IS: 800-1956)

$$M_c = \frac{f_b\, I}{y}\left(\frac{A_F \text{ net}}{A_{Gr}}\right)$$

Flange with 2 cover plates

$$= M_o = \frac{1\,500 \times 5\,847\,045 \times 2 \times 260{\cdot}54}{291{\cdot}27 \times 206{\cdot}3 \times 100 \times 1\,000}$$

$$= 760 \text{ m·t}$$

One cover plate

$$= M_c = \frac{1\,500 \times 4\,403\,045 \times 2 \times 197{\cdot}54}{222{\cdot}52 \times 203{\cdot}8 \times 100 \times 1\,000}$$

$$= 575 \text{ m·t}$$

No cover plate

$$= M_c = \frac{1\,500 \times 2\,824\,545 \times 2 \times 135{\cdot}2}{145{\cdot}52 \times 201 \times 100 \times 1\,000}$$

$$= 395 \text{ m·t}$$

---

*201 − 2(3·84) (C_{yy} of ISA 200 150, 18 mm) = 193·32.

**The net area calculation is based on 4 rivets, taking that they are staggered.

† $\frac{d^2}{2} = \frac{(193{\cdot}32)^2}{2}$

‡ +26 is towards the web plate between the flange angles 14·5 × 0·9.

§Only 2 rivet holes considered as where there is no flange plate there will not be any rivet in the horizontal legs of angles.

The bending moment diagram is drawn (data on Sheet 2) and the theoretical cut-off points of cover plates are determined at the intersections of the horizontal bending moment capacity lines with the actual bending moment curve.

Design Example 3

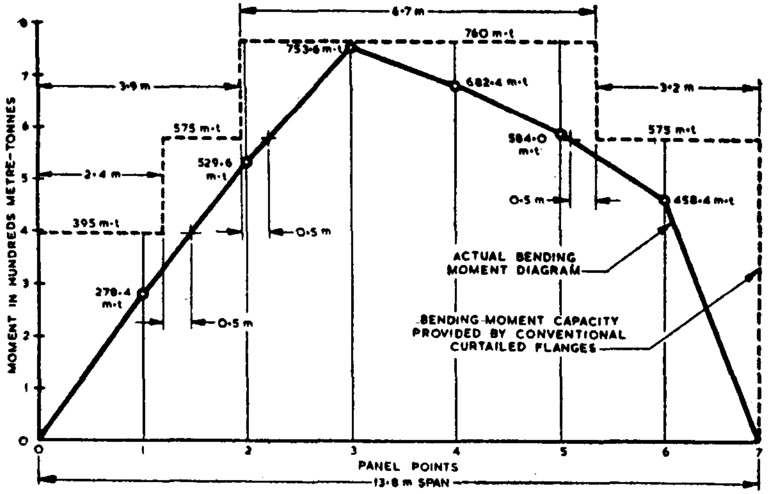
Design of Girder

3 of 14

Statical Moments of Flanges 'Q'

	Gross Area	Arm	Statical Moment Q
Angles	119·52	193·32/2	11 560 cm^3
1·4-cm plate	77	202·4/2	7 800 cm^3
Beam with one cover plate	—	—	19 360 cm^3
1·25-cm plate	68·75	205·05/2	7 050 cm^3
Beam with two cover plates	—	—	26 410 cm^3
2 plates no angles	—	—	14 850 cm^3

Bending Moment Capacity provided by conventional curtailed flanges (*see* Sheet 2).



Design Example 3	**4 of 14**
Flange to Web Rivets	

The dead weight estimate is checked and it may be pointed that 80 percent of the web weight has been added for stiffeners and other details. This is 10 percent more than the suggested weight allowance in **15.1** *(c) but the additional 10 percent is justified because of the two bearing stiffeners for columns in the interior part of the girder. At the bottom of this Sheet are tabulated the rivet pitch calculations based on previously determined values of shear (V), gross moment of inertia (I), rivet value (R), and static moment (Q) of the areas to which the stress is being transferred. The rivet values for the web to angle transfer are all based on web bearing for a single rivet, and the rivet values for angle to cover plate are based on two rivets in single shear as these rivets will be used in pairs.*

Web plates at right end:

Maximum shear $= 255 \cdot 2$ t

Area required $= \dfrac{255 \cdot 2 \times 1\,000}{945} = 270 \text{ cm}^2$

Use web plate $200 \times 1 \cdot 4$ cm $= 280 \text{ cm}^2$

Check dead weight estimate:

Web area (0·9-cm thick)	= 180
Flange area	= 530 (based on 2 cover plates)
Stiffeners and other details (80 percent of web)	= 144
	854 cm²

Weight per metre $= 854 \times 0 \cdot 79 = 675$ kg/m < 900 kg/m assumed ..... OK.

Flange to web rivets (power driven shop rivets)

Rivet values (R_v), allowable loads for 22 mm rivets

Bearing on 0·9-cm web	$= 2\,360 \times 0 \cdot 9 \times 2 \cdot 3$	= 4 885
Bearing on 1·4-cm web	$= 2\,360 \times 1 \cdot 4 \times 2 \cdot 3$	= 7 600
Bearing on 2 Ls	$= 2\,360 \times 1 \cdot 8 \times 2 \times 2 \cdot 3$	=19 550
Double shear for web rivets or shear of two rivets for flange rivets	$= 1\,025 \times 2 \times \dfrac{\pi(2 \cdot 3)^2}{4}$	= 8 500

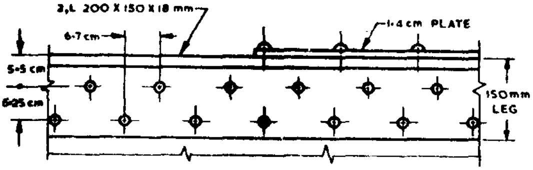
∴ Bearing on web will control.

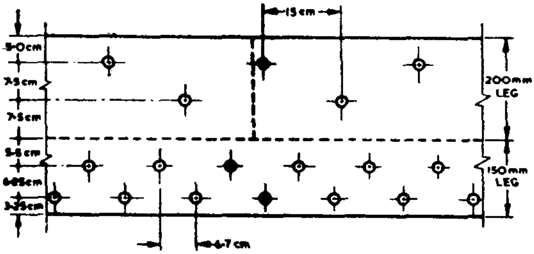
Rivet pitch $P = \dfrac{R\,I}{V\,Q}$ (Eq 12 on p. 52)

PANEL	V (tonnes)	I_{gross}	WEB TO ANGLES			ANGLES TO FLANGE PLATES		
			R	Q	P	R	Q	P
0-1	139·2	2 824 545	4·885	11 560	8·5	—	—	—
1-2	125·6	4 403 045	4·885	19 360	8·7	8·5	7 800	38
2-3	112·0	5 847 045	4·885	26 410	9·5	8·5	14 850	30
3-4	35·6	5 847 045	4·885	26 410	30·3	8·5	14 850	94
4-5	49·2	5 847 045	4·885	26 410	21·9	8·5	14 850	68
5-6	62·8	4 403 045	—	19 360	17·7 27·1	8·5	7 800	76·5
6-7	255·2	4 403 045	7·600	19 360	6·7	8·5	7 800	18·8

Design Example 3	5 of 14
Flange to Web Rivets	

Sketches indicate rivet arrangements near the ends of a cover plate and in an unfolded development of the angle. Minimum pitch is determined to maintain the net section that was used as a basis for the flange area requirement. Specification clauses for other rivet pitch requirements are referred to with the recommended pitch tabulated at the bottom of Sheet 5. It is permissible to use a greater pitch in the top plate than in the vertical legs of the angles.





According to **19.2.2** of IS : 800-1956, for staggered pitch it adds an area of $P^2/4g$ for every gauge space.

In calculating the net sectional area (*see* Sheet 2), we have deducted for one rivet hole for each staggered line of rivets. Therefore, minimum pitch allowable for this connection is that which adds one rivet hole diameter = 23 mm.

$\therefore$ For a 7·5-cm line spacing in 150-mm leg of angles

$$P^2/4g = \frac{P^2}{4\times 7{\cdot}5} = 2{\cdot}3$$

$$P = 8{\cdot}3 \text{ cm}$$

For a, *say* 6·25-cm line spacing

$$P = \sqrt{2{\cdot}3\times 4\times 6{\cdot}25}$$
$$= 7{\cdot}57 \text{ cm}$$

Straight line pitch for staggered rivets on same line in angles — Maximum 24 t or 300 mm (*see* **25.2.2.3** of IS: 800-1956).

TRIAL PITCH PANEL (1)	ANGLES STAGGERED PITCH, cm (2)	TOP PLATE STAGGERED PITCH*, cm (3)	REMARK (4)
0-1	8·5	No plate	
1-2	8·5	15	At ends of cover plates reduce to 6·0 cm pitch.
2-3	9·5	15	
3-4	15·0	15	
4-5	15·0	15	
5-6	15·0	15	
6-7	6·7†	15‡	

---

*For convenience of riveting, shop may prefer same pitch as in 150-mm leg (as in col 2 above).

†With 6·7-cm staggered pitch, net area in panel 6-7 is given by deducting from the figure to be obtained in Table in Sheet 2 for net area of flange with 1 cover plate (173·32+24).

A value corresponding to $P^2/4g = \frac{6{\cdot}7^2}{4(6{\cdot}25)} = 1{\cdot}79$ cm

$\therefore$ Revised net area assuming that same pitch of 6·7 cm for flange and web $= 197{\cdot}32 - 1{\cdot}79\ (1{\cdot}8)2 - 1{\cdot}79\ (1{\cdot}4)2 = 185{\cdot}85\ \text{cm}^2$

$$M_R = \frac{1\,500\times 4\,403\,045\times 2\times 185{\cdot}85}{223{\cdot}52\times 202{\cdot}8\times 100\times 1\,000} = 549 \text{ m}{\cdot}\text{t}$$

Actual Moment in 6-7 is 458·4. Hence, $\frac{80{\cdot}6}{458{\cdot}4} \times 100 = 17{\cdot}6$ percent more .... OK.

‡For panel 6-7 according to Sheet 4, a line pitch of 18·8 cm is required. If we have a staggered pitch of 9·4 cm it is alright. But for shop convenience same pitch as in 0-1, 1-2, etc, for the 150 mm leg is recommended.

On this sheet, the arrangement of rivets at the cover plate cut-off points is indicated and various code stipulations are referred to and explained.	**Design Example 3** **Rivets at Cover Plate Cut-off Points**	**6 of 14**

Allowable pitch per line of rivets (*see* **25.2.2.3** of IS: 800-1956) = 24 × 1·25 = 30 cm

∴ Use a staggered pitch of 15 cm.

Adopt same pitch for both compression and tension flanges for economy in fabrication.

Actual cut-off points

For 1·4-cm plate: (*see* **20.5.1** of IS: 800-1956)

The net area of the plate with the above pitch = 70·5 cm^2 (*see* Sheet 2)

Strength of the plate = 70·5 × 1 500 = 106·00 t

Shear value of rivet controls and is equal to 8·5/2 = 4·25 t (*see* Sheet 4)

No. of rivets required to develop the strength of plate = $\frac{106}{4{\cdot}25}$ = *say* 26 rivets

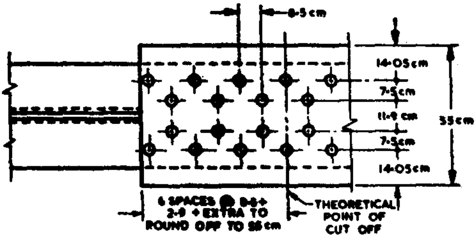
Half the number of these rivets (13) are required in the extension of cover plate beyond the theoretical point of cut off.

The minimum staggered pitch being that which adds one rivet hole diameter = 8·3 cm (*see* Sheet 5).

Adopting 8·5-cm pitch as required in the angles to web connections (*see* Sheet 5) and with two staggered lines of rivets, one on each side of web, the extension length required to carry 6 rivets is about 6 spaces plus the minimum edge distance = 29 mm (*see* Table XII in **25.4** of IS: 800-1956).

The edge distance of cover plate in the transverse direction = $\frac{55-(11{\cdot}9+15)}{2}$

= 14·05 cm



As this is a single plate at this point of cut-off and the plate girder is not in an exposed condition, the maximum edge distance permissible is 16 *t* = 16 × 1·4 = 22·4 > 14·05 . . . . . OK (*see* **18.4.1** and **25.4.1.1** of IS: 800-1956).

For 1·25-cm plate, the net area as before = 63 cm^2 (*see* Sheet 2)

Its strength = 63 × 1 500 = 94·5 t

A 22-mm diameter rivet has a shear value of 4·25 t (*see* Sheet 4).

No. of rivets required = $\frac{94{\cdot}5}{4{\cdot}25}$ = 22 rivets = *say* 26 rivets

Same extension and spacing of rivets as for the 1·4-cm thick cover plate may be adopted at both ends of cut-off of this cover plate.

Check the edge distance.

The plate girder not being in an exposed situation, the minimum permissible edge distance is 12 *t* or 15 cm whichever is small, 12 *t* = 12 × 1·25 = 15 cm > 14·0 cm adopted . . . . . OK (*see* **25.4.1** of IS: 800-1956).

Design Example 3 | **7 of 14**

Bearing Stiffeners Under Right Column Load

These sheets all concern the design of bearing stiffeners at supports and under column load locations. As in the case of the welded girder, the bearing stiffeners at the columns are put in double pairs so as to pick up the column flange load and eliminate bending stress in the cover plates and top angles. At the left end, where bearing is on a masonry wall, four angles are also used but these are turned together instead of apart so as to localize the load over the centre of the masonry wall. The design of bearing stiffeners is fully explained on the design sheets.

Bearing Stiffeners Under Right Column (Load 179 t*)*

Allowable bearing stress $= 1\,890$ kg/cm^2

Required contact area $= \dfrac{179 \times 1\,000}{1\,890} = 95$ cm^2

Allowing 1·5 cm for the root fillet of flange angle, if 200×100 angles are to be used, total thickness required $= \dfrac{95}{20{\cdot}0 - 1{\cdot}5} = \dfrac{95}{18{\cdot}5}$

$= 5{\cdot}15$ cm [*see* **20.7.2.2**(b) of IS: 800-1956]

Adopt 4 ISA 200 100, 15 mm which gives an area of contact:

$4(20{\cdot}0 - 1{\cdot}5)\ 1{\cdot}5 = 111$ cm^2 > 95 cm^2 required . . . . . OK.

With 22-mm rivets, R for bearing on web $= 4{\cdot}885$ t and controls (*see* Sheet 4)

No. of rivets for filler plates

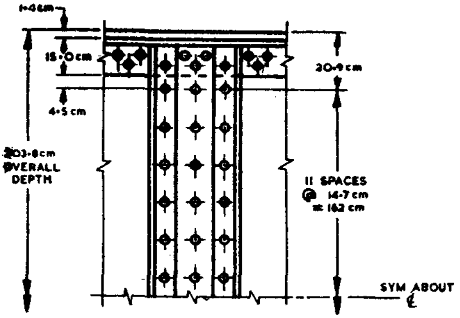
$= \dfrac{179}{4{\cdot}885} = 36{\cdot}64$, *or say* 36

For angles double shear controls and No. of rivets on Angles

$= \dfrac{179}{8{\cdot}5} = 21{\cdot}06$, *or say* 22

Being under compression, maximum pitch $= 16 \times 1{\cdot}5 = 240$ (*see* **25.2.2.1** of IS: 800-1956)

Use 2 rows of 12 rivets in angles and one row of the same number in the filler as shown in the sketch on the left.



Check for strut action:

Effective length of web $= 0{\cdot}9 \times 20 = 18{\cdot}0$ cm [*see* **20.7.2.2**(a) of IS: 800-1956]

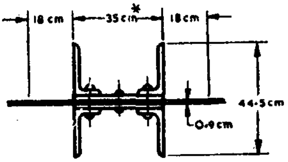
$I = 2 \times 1{\cdot}5 \times (44{\cdot}5)^3 \times 1/12 = 22\,000$ cm^4 (*see* sketch below)

I due to other parts is very small.

Area $= 4 \times 42{\cdot}78 = 171{\cdot}12$ cm^2

Web $= 0{\cdot}9 \times 71 = 63{\cdot}9$ cm^2

$235{\cdot}02$ cm^2



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*The column section supported by the girder at this section is ISHB 350. Hence, the bearing stiffeners are also kept at 35 cm apart to be against the flange of the column section.

Design Example 3

Bearing Stiffeners at Right (Steel Column) Support

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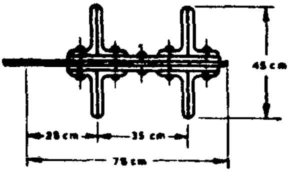
$$r = \sqrt{\frac{22\,000}{235}} = 9{\cdot}68$$

$$l/r = \frac{0{\cdot}7 \times 200}{9{\cdot}68} = 14{\cdot}5$$

Safe axial compression stress = 1 228 kg/cm² (*see* Table I of IS: 800-1956)

$$\therefore \text{ Load capacity } = \frac{235{\cdot}02 \times 1\,228}{1\,000} = 289 \text{ t} > 179 \text{ t } \ldots\ldots \text{ OK.}$$

Right Support Bearing Stiffeners



Place stiffener angles as shown with tight filler in between. Allowihg 1·5 for fillet of flange angles and using 4 angles 200×100:

$$\text{Thickness required} = \frac{261{\cdot}8 \times 1\,000}{4 \times 18{\cdot}5 \times 1\,890} = 1{\cdot}87 \text{ cm}$$

This thickness is not available in IS Rolled Angles.
Hence, use 8 Angles as shown.

$$\text{Thickness required} = \frac{261{\cdot}8 \times 1\,000}{8 \times 18{\cdot}5 \times 1\,890} = 9{\cdot}05 \text{ cm}$$

Use ISA 200 100, 10 mm sections as shown.
No. of rivets needed for filler, where bearing on 1·4-cm web is controlling, is $\frac{261{\cdot}8}{7{\cdot}60} = 34{\cdot}4$ rivets (*see* Sheet 4), and for angles where double shear controls, $\frac{261{\cdot}8}{8{\cdot}5} = 30{\cdot}8$ rivets.

Use 4 rows of 9 rivets on the angles which provides 36 > 34·4, and > 30·8 is OK. But according to **25.2.2.1** of IS: 800-1956 as the maximum pitch exceeds 16 t = 16×1·0 = 16 cm. Use 11 rivets on each row and one row of the same number in the filler plate. Alternatively, smaller size rivets may also be designed.

Check strut action

Effectivelength of web = 1·4×20 = 28·0 cm on either side, if available
Effectivelength = 78 cm (*see* sketch)

$$I = \frac{4 \times 10 \times (45)^2}{12} = 30\,050 \text{ cm}^4$$ (even neglecting I of other parts)

Area = 29·03×8 = 232·24 cm² (angles)

Web = 78×1·4 = $\underline{109{\cdot}0 \text{ cm}^2}$ [Note that filler plates have not been considered (*see* **20.7.2.2** of IS: 800-1956)]

341·24 cm²

$$r = \sqrt{\frac{30\,050}{341{\cdot}24}} = 9{\cdot}4 \text{ cm}$$

$$l/r = \frac{0{\cdot}7 \times 200}{9{\cdot}4} = 14{\cdot}9$$

F_c = 1 228 kg/cm²

Load capacity = 341·24×1 228 = 418 > 261·8 ..... OK (Shear at right support, *see* Sheet 1).

Design Example 3

Bearing Stiffeners at Left (Concrete Wall) Support

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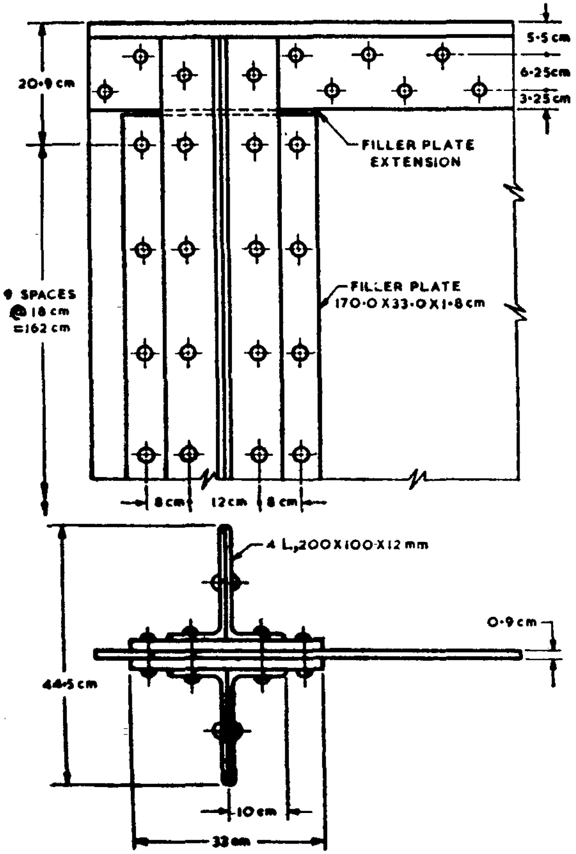
Using 4 Angles, 200×100 mm:

$$\text{Thickness required} = \frac{146 \times 1\,000}{4(20 - 1{\cdot}35) \times 1\,890} = 10{\cdot}4$$

Use 4 ISA 200 100, 12 mm sections.



Number of rivets:

$$\text{For filler} = \frac{146}{4{\cdot}885} = 29{\cdot}6$$

$$\text{For angles} = \frac{146}{8{\cdot}5} = 17{\cdot}2$$

Use 4 rows of 10 rivets as shown with a pitch of 18 cm c/c. This would provide 40 rivets for filler plates and 20 rivets for the stiffener.

Check strut action:

Effective length of web

$= 2(20 \times 0{\cdot}9)$

$= 36$ cm

$$I = \frac{2 \times 1{\cdot}2 \times (44{\cdot}5)^3}{12} = 17\,600 \text{ cm}^4$$

$$A = 4 \times 34{\cdot}59 + 36 \times 0{\cdot}9 = 170{\cdot}76 \text{ cm}^2$$

$$r = \sqrt{\frac{17\,600}{170{\cdot}76}} = 10{\cdot}15 \text{ cm}$$

$$l/r = \frac{0{\cdot}7 \times 200}{10{\cdot}15} = 13{\cdot}8$$

$F_c = 1\,228$ kg/cm^2

Load capacity = 170·76×1 228 = 209 > 146* ..... OK.

Bearing stiffeners under left column (load = 134 t)

Use 4 ISA 200 100, 12-mm sections.

Design is similar to that at right column load.

---

*Left support shear, *see* Sheet 1.

Design Example 3

Web Splice

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At the intersection of the 1·4-cm and 0·9-cm webs a splice shall be introduced to transfer the total bending moment and shear allotted to the web at this section. The splices are placed 0·5 m to the left of panel point 6 where the heavy column load causes a great increase in shear to the right end and creates the need for the 1·4-cm web plate. The splice plates are effective in moment in proportion to the square of their depth and, therefore, the total required cross-sectional area is determined by multiplying the web area by the square of the ratio of web height to splice plate height. Since the rivets are assumed to be stressed in proportion to the distance from the neutral axis, the maximum allowable horizontal component of rivet stress is reduced accordingly.

For an exact fit of the splice plate, 0·25-cm fillers would be needed on each side of the 0·9-cm web to make it match with the 1·4-cm web to which it is spliced. The value in bearing on the 1·4-cm plate is considerably greater than for the 0·9-cm plate and, therefore, only two rows of rivets are required on the right side of the splice. The rivet pattern as shown for the left side is arrived at after several rough trials.

Web Splice

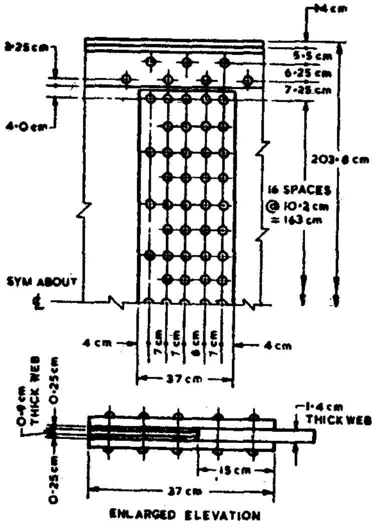
Splice to the left of the bearing stiffeners under the right column (0·5 m from panel point 6)

Shear at spliced section = 62·8 t; Moment at spliced section = 489·8 m·t

R 22 mm diameter rivet on 0·9-cm plate web = 4·885 (*see* Sheet 4)

" " " " on 1·4-cm web = 7·600

Total area of splice plates required $= A_w \left(\frac{h_w}{h_s}\right)^2 = 200 \times 0{\cdot}9 \left(\frac{200}{170}\right)^2 = 249 \text{ cm}^2$



(The height of splice plate being 200 − 2 × 15 = 170)

Total thickness required $= \frac{249}{170} = 1{\cdot}46$ cm

Use 2 plates (preferred dimensions for plates: 170 × 0·8 cm)

Maximum allowable value for horizontal stress in the splice plate rivets

$= 4{\cdot}885 \times \frac{\text{(distance between extreme outside lines of rivets on splice plates)}}{\text{(distance between extreme lines of rivets on flange angles to web)}}$

$= 4{\cdot}885 \times \frac{171 - {}^*(4{\cdot}0)2}{201 - (5{\cdot}5)2} = 4{\cdot}28$ t

Assume the adjacent pattern of rivets.

Width of splice plates

Edge distance = 3·8 cm for 22-mm rivets, adopt 4 cm on either side; pitch = 3*d* = 3 × 2·3 = 6·9 *or say* 7 cm.

---

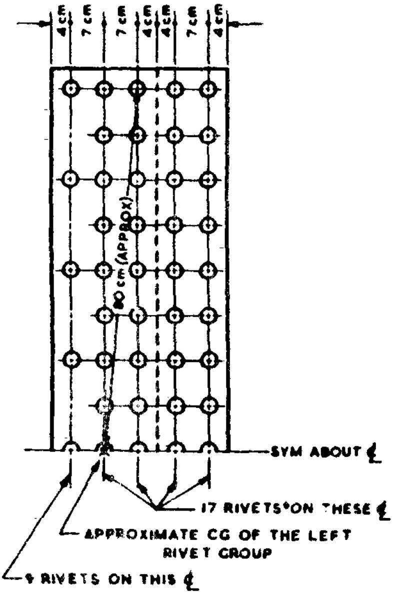
*Assuming the top and bottom edges of splice plates are sheared edges, for 22-mm rivets, **24.4** of IS: 800-1956 requires minimum edge distance of 38 cm and for the assumed pattern of rivets this distance has changed to 4·0 cm (*see* sketch).

The rivet pattern for the left side of the splice is shown. This pattern was arrived at after several rough trials.

Pitch at splice point should be slightly more as the minimum edge distance required for the web plates being spliced should also be satisfied. Taking these edges as sheared, Table XII of IS: 800-1956 requires 3·8 or say 4·0 cm minimum edge distance for 22 rivets. Hence, adopt 8-cm pitch in the middle portion shown in the sketch.

(*See* Elevation in sketch on Sheet 10)

Thus, the total width of splice plate = 37 cm



$\Sigma y^2 =$

$2 \times (10{\cdot}2)^2 = 208$

$3 \times (20{\cdot}4)^2 = 1\,248$

$2 \times (30{\cdot}6)^2 = 1\,872$

$3 \times (40{\cdot}8)^2 = 4\,994$

$2 \times (51)^2 = 5\,200$

$3 \times (61{\cdot}2)^2 = 11\,250$

$2 \times (71{\cdot}4)^2 = 10\,200$

$3 \times (81{\cdot}5)^2 = \underline{19\,950}$

$54\,922 \text{ cm}^2$

$y^2 = 2 \times 54\,922 = 109\,844$

$x^2 = {}^*26 \times (7)^2 = \underline{1\,274}$

$111\,118$

---

*The centre of gravity of the rivet group on the 0·9-cm thick web side may be approximately taken to be at the middle of the three rivets on the centre line (*see* sketch). Total number of rivets on left extreme line = 9. Total number of rivets on right extreme line of the group = 17. Total rivets = 26. The rivets on the middle line have no moment, the distance between centre of gravity and the rivet being zero.

Design Example 3

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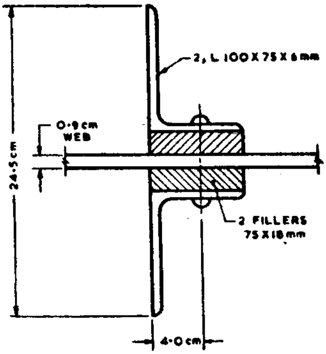
Web Splice Completed, Intermediate Stiffeners

One-eighth of the web area may be assumed as equivalent flange area in a riveted girder. Both the maximum horizontal component of rivet stress and the total resultant stress of the most stressed rivet are found to be within satisfactory limits. The intermediate stiffeners required 1·8-cm filler plates which add considerably to the weight of the girder. This use of filler plate is optional for intermediate stiffeners and depends on whether or not the fabricating shop has crimping equipment that will permit their elimination.

Moment of web to be resisted by splice $= \dfrac{A_w/8}{A_w/8+A_F} \times$ Moment at splice point



(A_F is net area of flange)

$$= \frac{200\times 0{\cdot}9^*}{8\times(200\times 0{\cdot}9^*/8)+173{\cdot}54)}\times 489{\cdot}8 = 56{\cdot}2 \text{ m·t}$$

$$r_m = \frac{56{\cdot}2\times 80\times 100}{111\,118}$$

$$= 4{\cdot}05 \text{ t} < 4{\cdot}22 \text{ t } (\textit{see} \text{ Sheet 10}) \ldots\ldots \text{OK.}$$

$$r_v = \frac{62{\cdot}8\dagger}{(17+17+9) \text{ rivets}} = \frac{62{\cdot}8}{43} = 1{\cdot}46 \text{ t}$$

$$r = \sqrt{r_m^2+r_v^2} = \sqrt{(4{\cdot}2)^2+(1{\cdot}46)^2}$$

$$= 4{\cdot}35 \text{ t} < 4{\cdot}885 \text{ t (rivet value)} \ldots\ldots \text{OK.}$$

Use 3 rows of rivets, left side of splice spaced at 10 cm c/c vertically as shown in Sheet 10.
On right side, the rivet value, $R = 7{\cdot}60$

Number of rivets required $= \dfrac{4{\cdot}885}{7{\cdot}60} \times 43 = 27{\cdot}8$

∴ use of 34 rivets as shown at Sheet 10 is OK.
Packing is less than 6 mm and no excess rivets required ..... OK (*see* **24.6.10** of IS: 800-1956)

Intermediate Stiffeners

In region 0-6: Thickness of web $= 0{\cdot}9\ \dfrac{177{\cdot}5\ddagger}{85} = 2{\cdot}09\ \dfrac{177{\cdot}5}{200}$

$= 0{\cdot}888$ cm (*see* **20.6.1** of IS: 800-1956)

∴ vertical intermediate stiffeners are required. Use pairs of angles 100 × 75 × 6 mm.

$I_{xx} = 0{\cdot}6\times(24{\cdot}5)^3/12 = 735 \text{ cm}^4$

Required $I_{\min} = 1{\cdot}5\ d^3t^3/C^2$ (*see* **20.7.1.2** of IS: 800-1956)

Assuming $c = 150$ cm, $I_{\min} = \dfrac{1{\cdot}5\ (171)^3\ (0{\cdot}9)^3}{(150)^2} = 243 \text{ cm}^3 < 735$ available ... OK.

Max spacing allowable $= 180\times 0{\cdot}9 = 162$ (*see* **20.7.1.1** of IS: 800-1956)

$d/t = 171/0{\cdot}9 = 190,\ c/d = 150/171 = 0{\cdot}88$

From Table III of IS: 800-1956, $f_s = 775 \text{ kg/cm}^2$

$V = 775\times 200\times 0{\cdot}9 = 140 \text{ t} > 139{\cdot}2$ ..... OK (shear in panel 0-1)

Assumed spacing is OK.

NOTE — If shop has crimping equipment, fillers may be omitted and the stiffeners may be crimped.

---

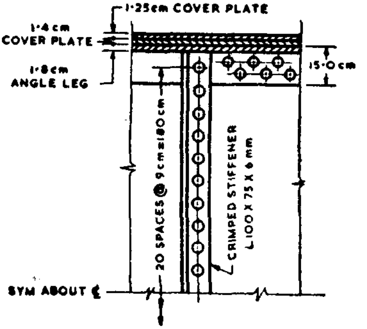
*It may be noted that 0·9-cm thick web plate has been considered for the calculations here, and not the 1·4-cm web. This is because 1/8 of the area of the 0·9-cm web was the basis in arriving at the girder sections (*see* Sheet 1).

†Shear in panel 6-7.

‡The height of web between closest rows of rivets = 201 − 2(5·5 + 6·25) = 177·5 cm

After further calculations in continuation of the last sheet, the weights take off of the riveted plate girder is given on this sheet and the estimated percentages used for details are approximately confirmed.

Design Example 3	**13 of 14**
Weights Take Off	



In region 6-7:

$c = 180 - 35 = 145$ cm $< 180 \times 1{\cdot}4$

$d/t = \frac{171}{1{\cdot}4} = 122, \quad c/d = \frac{145}{171} = 0{\cdot}85$

From Table III on page 174,

$f_s = 938$ kg/cm^2

$V = 938 \times 200 \times 1{\cdot}4 = 262{\cdot}6$ t $> 255{\cdot}2$ t

..... OK (Shear in panel 6-7)

∴ Bearing stiffeners eliminate need for intermediate stiffeners in this region.

Max spacing of rivets in intermediate stiffener allowed $= 16\,t = 16 \times 0{\cdot}6 = 9{\cdot}6$ cm

∴ Use 9 cm.

***Weights** (in* kg) *take off* (based on Sheet 14).

Flanges

2 Plates — 55 × 1·40 @ 0·785 kg/m/cm² for 11·725 m = 1 420
2 Plates — 55 × 1·25 @ 0·785 kg/m/cm² for 6·7 m = 730
4 Angles — 200 × 150 × 18 @ 46·9 kg/m for 14·35 m = 2 690
4 840

Web

1 Plate — 200 × 0·9 @ 0·785 kg/m/cm² for 11·725 m = 1 670
1 Plate — 200 × 1·4 @ 0·785 kg/m/cm² for 2·62 m = 570
2 240

Stiffeners

Intermediate stiffeners — 6 × 2 — 100 × 75 × 6 mm *L*s @ 8 kg/m 2·01 m = 193
Left support 4 of 200 × 100 × 12 *L*s at 27·2 kg/m for 2·01 m = 220
Right support 8 of 200 × 100 × 10 m at 22·8 kg/m for 2·01 m = 366
Right column 4 *L*s of 200 × 100 × 15 @ 39·4 kg/m for 2·01 m = 316
Left column 4 *L*s of 200 × 100 × 12 @ 27·2 kg/m for 2·01 m = 220
1 315

Fillers for Stiffeners

Intermediate Pls — 6 × 2 of 100 × 1·8 for 1·71 m at 0·785 kg/m/cm² = 292
Left support Pls — 2 of 36 × 1·8 for 1·71 m at 0·785 kg/m/cm² = 175
Right support Pls — 2 of 55 × 1·8 for 1·71 m at 0·785 kg/m/cm² = 268
Right column } — 2 × 2 of 35 × 1·8 for 1·71 m at 0·785 kg/m/cm² = 342
Left column }
1 077

Splice Plate

2 of 37 × 1·6 for 1·71 m at 0·785 kg/m/cm² = 160

(Stiffeners and fillers 1 315 + 1 077 = 2 392 kg for 2 240 kg of web or 107 percent of the web assumed on Sheet 4 as 80 percent of 200 × 0·9 cm web. Actual is about 110 percent.)

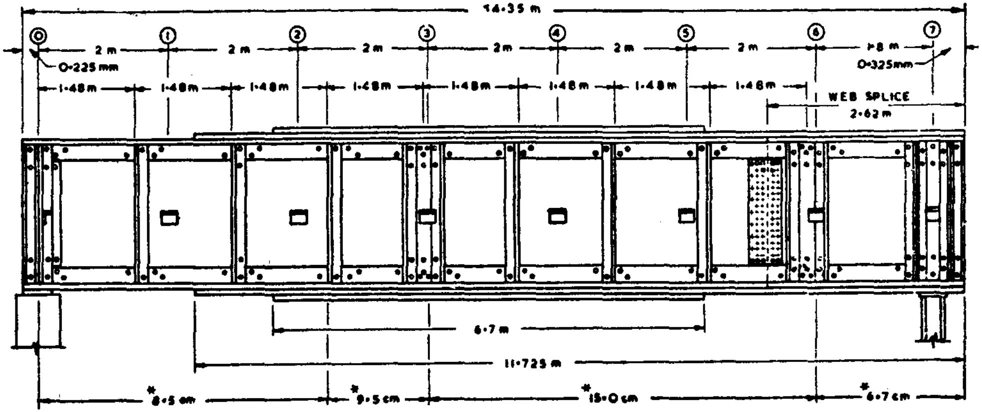
[No revision of loading is necessary on this account (*see* Sheet 4).]

The side view of the plate girder is shown with more than an adequate amount of detail arrangement so as to provide the draftsman sufficient information to make the final detail drawing.

Design Example 3

14 of 14

Detailed Drawing of Riveted Plate Girder



*Staggered pitch of angles to web rivets.

NOTE — For angles to plates, rivets space at 15-cm staggered (except at ends where use 8·5-cm pitch). Use 22-mm rivets (power-driven) countersunk at bearing areas.

SECTION IV

NUMERICAL ANALYSIS OF BENDING MOMENTS AND DEFLECTIONS IN BEAMS

26. GENERAL

26.1 Although shears, bending moments and deflections may be found for all of the usual cases by means of the tabulated equations in Appendix B, attention will here be given to more generally applicable methods for finding shears, moments, and deflections as required (for deflection) when the moment of inertia is variable and/or the loading conditions are complex.

A number of examples of calculations of shear and moment diagrams has already been given in the illustrated Design Examples 2 and 3. However, the subject will be included in this item as a preliminary to a numerical method previously developed by Newmark*. The Newmark procedure has special advantages for complex cases and will be extended later to the study of deflections in beams and to columns in Handbook for Design of Columns in Steel.

To the reader not already familiar with the Newmark procedure, it will at first seem unnecessarily complex when used in the computation of ordinary simple beam shear and bending moment diagrams. Such a conclusion would be correct if this were the only application. The great value of the procedure lies in its adoptability to many other problems, most of which it is not possible to cover in this manual, but which include column buckling, beam vibration, etc.

27. NEWMARK'S NUMERICAL PROCEDURE

27.1 In Newmark's numerical procedure, the beam is divided up into a number of equal length segments and the shear and moment are calculated at the segment juncture points. The use of equal length segments is essential to the full utility of the procedure. A distributed load is replaced by a series of equivalent concentrated loads acting at the juncture points between successive segments.

For example, if the length of each segment is λ, Fig. 6 and the following will illustrate simple cases relating distributed load to equivalent concentrated load.

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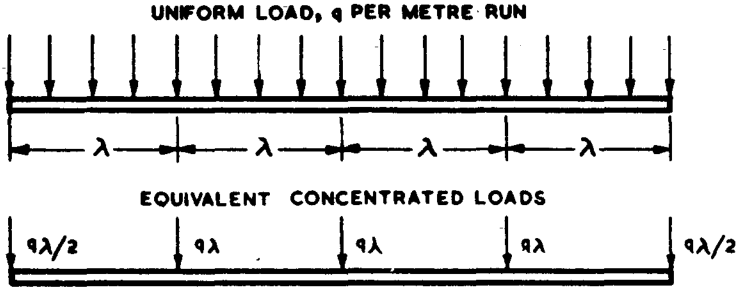
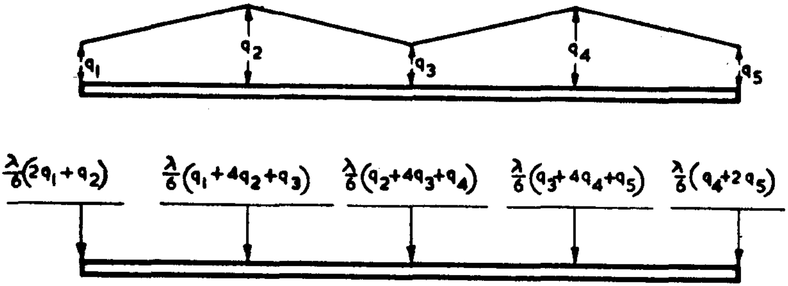
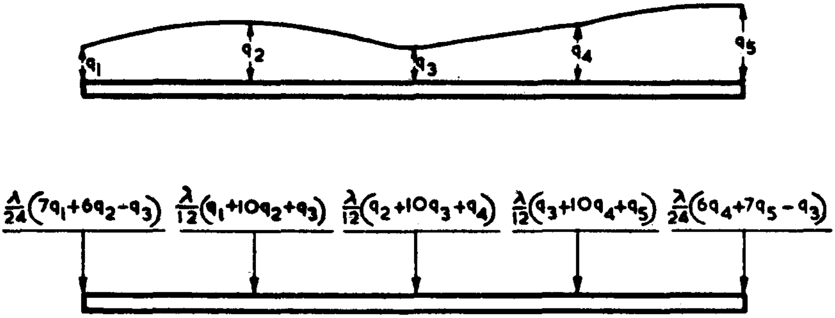
*Newmark, N. M. Numerical Procedure for Computing Deflections, Moments, and Buckling Loads. *Trans ASCE*, Vol 108, p. 1161-1234 (1943).



CASE (A) UNIFORM LOAD, q PER METRE



CASE (B) LINEAR VARIATION IN EACH SEGMENT



CASE (C) PARABOLIC VARIATION IN EACH SEGMENT

FIG. 6 EQUIVALENT CONCENTRATED LOADS REPLACING A DISTRIBUTED LOAD

The equivalent concentrated loads are the same as the panel point loads that would be caused by the reactions to a series of simple beams, each having a length λ. In Case (c) of Fig. 6, the load distribution is assumed to be a second degree parabola for two successive segments. At location (1), the segments run from 1 to 3, the same as at location (2).

In a more general way, the equivalent concentrated loads may be written in terms of any 3 successive locations. Let Q_{ba} be the end reaction of simple span *A-B* at *B*, Q_{bc} the end reaction of simple span *B-C* at *B*, then the equivalent concentrated (panel point) load at *B*, denoted as Q_b, is the sum of $Q_{ba} + Q_{bc}$. Thus, summarizing, for any 3 successive locations:

Case 1 (*see* Fig. 7): q_a, q_b, and q_c represent q with linear variation assumed over each length subdivision λ.

$$Q_{ab} = \frac{-\lambda}{12}(4q_a + 2q_b)\ Q_{ba} = \frac{-\lambda}{12}(4q_b + 2q_a) \quad \dots\dots (13)$$

$$Q_{bc} = \frac{-\lambda}{12}(4q_b + 2q_c) \quad \dots\dots (14)$$

$$Q_b = Q_{ba} + Q_{bc} = \frac{-\lambda}{12}(2q_a + 8q_b + 2q_c) \quad \dots\dots (15)$$

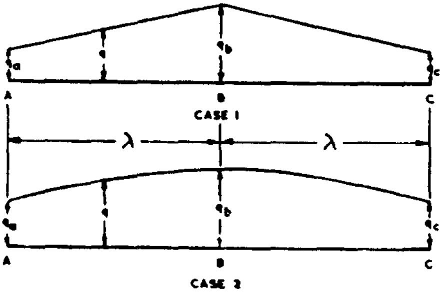


Fig. 7 Diagram for Calculation of Formulæ for Equivalent Concentrated Loads

Case 2 (*see* Fig. 7): q_b and q_c represent q with parabolic variation assumed over length 2λ between *A* and *C*.

$$Q_{ab} = \frac{-\lambda}{12}(3{\cdot}5q_a + 3q_b - 0{\cdot}5q_c) \quad \dots\dots\dots (16)$$

$$Q_{ba} = \frac{-\lambda}{12}(1{\cdot}5q_a + 5q_b - 0{\cdot}5q_c) \quad \dots\dots\dots (17)$$

$$Q_{bc} = \frac{-\lambda}{12}\,(1{\cdot}5q_c + 5q_b - 0{\cdot}5q_a) \qquad \ldots\ldots (18)$$

$$Q_b = Q_{ba} + Q_{bc} = \frac{-\lambda}{12}\,(q_a + 10q_b + q_c) \qquad \ldots\ldots (19)$$

In Case 1 applications, if there is a sudden discontinuity in q at any point B, Q_b may be calculated by using average values of q_b. Sudden discontinuities in Case 2 applications should be handled by calculating Q_{ba} and Q_{bc} separately, then adding to get Q_b. The same procedure should be used in cases where there is a transition between Case 1 and Case 2 at point B.

27.2 The equivalent load system is statically equivalent to the distributed load system and the following example will illustrate the calculation of shear and moment diagrams by the usual procedure as well as by the use of equivalent concentrated loads. It is again emphasized that the usual procedures are quite adequate in the calculation of shear and moment diagrams and the numerical method is introduced at this point as a matter of expediency in preparation for more useful applications later on.

27.3 A simple problem will be treated initially and it will be seen that the numerical procedure appears cumbersome and of no advantage in such a simple case. The value of the procedure lies in the fact that it handles very complex and special problems of beam deflections with a facility and accuracy that cannot be matched by other methods. The examples will be treated initially by conventional procedures.

Example 1:

Determine the shear and bending moment in a simply supported span of 8 metres under a uniform load of 6 tonnes per metre and a concentrated load of 10 tonnes at 2·5 metres from the left support.

Solution A — Shear Area Method (*see* Fig. 8)

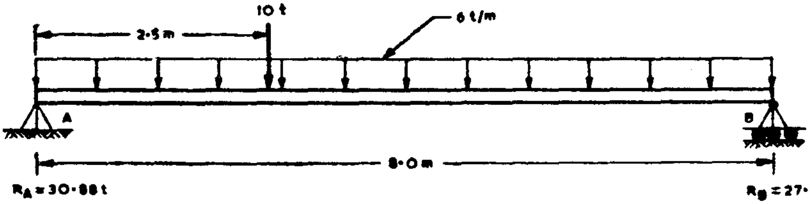


FIG. 8 LOAD DIAGRAM

$$R_A = \frac{6 \times 8}{2} + 10 \times \frac{5{\cdot}5}{8{\cdot}0} = 30{\cdot}88\ \text{t} \qquad \ldots\ldots\ldots\ldots (20)$$

$$R = 58 - 30{\cdot}88 = 27{\cdot}12\ \text{t} \qquad \ldots\ldots\ldots\ldots (21)$$

The shear diagram (*see* Fig. 9) is now determined. Immediately to the right of the left reaction, the shear is equal in magnitude to the reaction. For each segment of beam traversed to the right, the shear changes by the area under the load intensity diagram.

Thus, $$(V_X - V_A) = \int_A^X q\,dx \qquad \ldots\ldots\ldots\ldots (22)$$

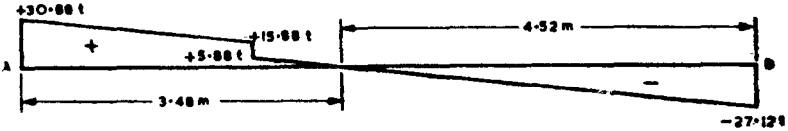


FIG. 9 SHEAR DIAGRAM

By starting at the left end (A) and plotting the shear diagram from A to B, a check is obtained on the calculation since the shear at the right end is equal in magnitude to the right end reaction R_B which was previously calculated by independent computation.

For each segment of beam traversed to the right, the moment changes by the area of the shear diagram:

$$(M_X - M_A) = \int_A^X V\,dx \qquad \ldots\ldots\ldots\ldots (23)$$

The moment is zero at A (hinged end support) (*see* Fig. 10) and by calculating the moments at various points, progressively, from A to B, an automatic check is obtained at B where the moment is again 0.

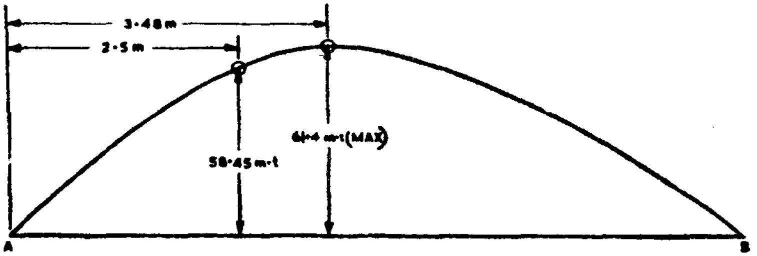


FIG. 10 BENDING MOMENT DIAGRAM

The location of maximum moment is easily determined as the point of zero shear, 3·48 m from the left end of the beam.

Solution B — Newmark's Numerical Procedure (*see* Fig. 11)

Newmark's numerical procedure will be illustrated (*see* Fig. 11) by dividing the beam into 4 segments, each 2 m in length, with segment or panel points labelled 1, 2, and 3. The reaction $R_A = 30{\cdot}88$ t is calculated as before, and the equivalent concentrated loads are calculated separately for the uniform load of 6 t/m and the localized load of 10 tonnes. For the uniform load (*see* Fig. 6), each concentrated load is $2 \times 6 = 12$ tonnes. The additional load at panel point (1) is the end reaction caused by 10 tonnes acting 10·5 m from the end of the first 2 m simple span, which is equal to 7·5 tonnes. Thus, at panel point (1), the total concentrated load is $12 + 7{\cdot}5 = 19{\cdot}5$ tonnes. Similarly, at panel point (2), the concentrated load is $12 + 2{\cdot}5 = 14{\cdot}5$ tonnes.

Since all of the loads have been replaced by a series of uniformly spaced concentrated loads, the shear is constant within each segment having length $\lambda = 2$ m. The concentrated loads are listed in line (*a*) in Fig. 11, and are given a minus sign. (In the application of the numerical procedure, upward loads are positive since they would cause an increase in positive shear in proceeding from left to right.)

Shear is calculated in line (*c*) in Fig. 11. Starting with the end reaction $R_A = 30{\cdot}88$ tonnes, the shear in the first panel is $30{\cdot}88 - 6 = 24{\cdot}88$ t. The shear in successive panels is formed by successive additions of the negative concentrated loads.

Since the shear is constant in each segment, the change in moment between successive panel points equals $V\lambda$. Thus, at panel point (1), the moment (initially zero) is $24{\cdot}88 \times 2 = 49{\cdot}76$ tonnes. However, a repetition of the simple additive process so convenient in getting the shear is desirable for moments, so 2 m in this case is introduced as a multiplier in the column so captioned at the right. The use of a common multiplier is made possible by reason of the uniform panel length λ used throughout the span. Now the moments are successive simple additions of the shears within successive panel segments. Finally, in line (*e*), the actual moments at the quarter and midpoints are obtained by multiplying each number in line (*d*) by 2.

Example 2:

The following example presents a more complex loading case and includes a cantilever extension of the simple beam to the right of the support at *B*. Reactions, shear, and moment diagrams are presented without detailed computations, followed by the numerical

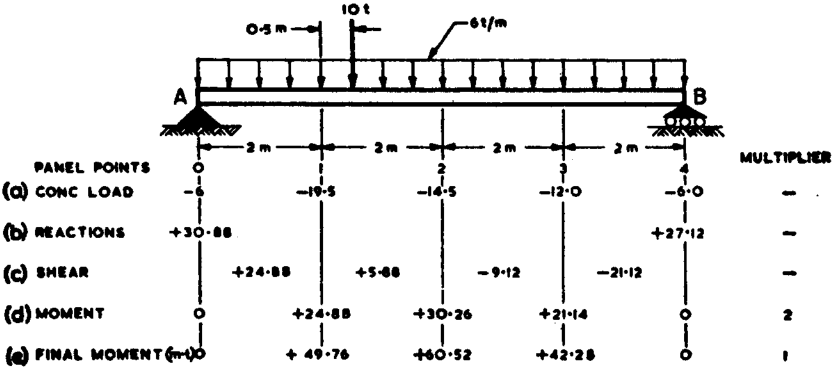


FIG. 11 SHEAR AND MOMENTS.BY NEWMARK'S NUMERICAL PROCEDURE

analysis on the basis of panel length $\lambda = 1{\cdot}25$ m. In this example, the multiplier λ is introduced in line (*a*) in Fig. 12 to simplify the computation of the concentrated loads and to keep the numbers in successive lines from getting too large.

In the foregoing example, it was necessary to ***divide*** the actual concentrated loads of 15 and 20 tonnes and the reaction at R_B by 1·25 to compensate for the use of the multiplier. Thus at *B*, the concentrated load (reaction) is:

$$+\frac{59{\cdot}66}{1{\cdot}25} = +\,47{\cdot}73 \text{ t} \qquad \ldots\ldots\ldots\ldots\ldots \quad (24)$$

Since the multiplier λ has already been used in lines (*a*) and (*b*) in Fig. 12, the computation in line (*d*) includes the multiplier $\lambda 2 = 1{\cdot}56$. The use of the numerical procedure in this particular case permits a simple routine calculation of the moment at 1·25 m intervals, thus permitting an accurate graphical plot of the complete moment diagram. In this problem, the computation work in comparison with usual procedure has been reduced.

A third illustrative example will illustrate the formulæ (*see* Fig. 6) for the replacement of a variable distributed load.

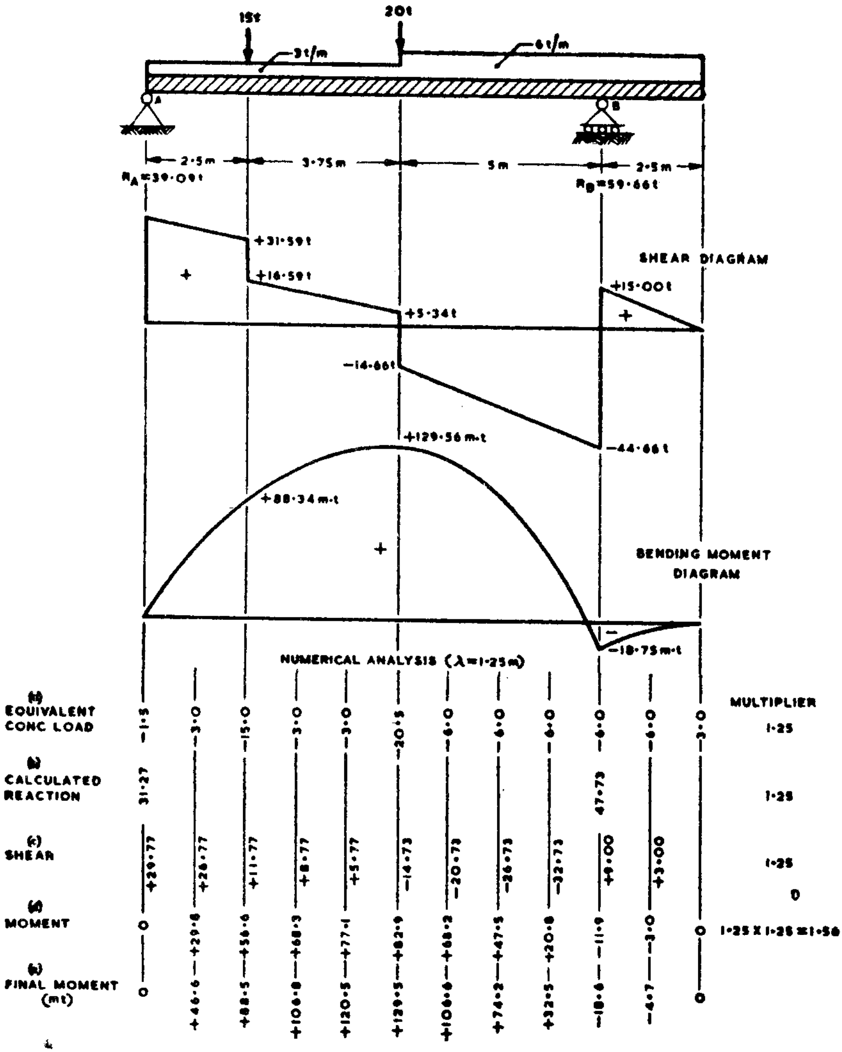
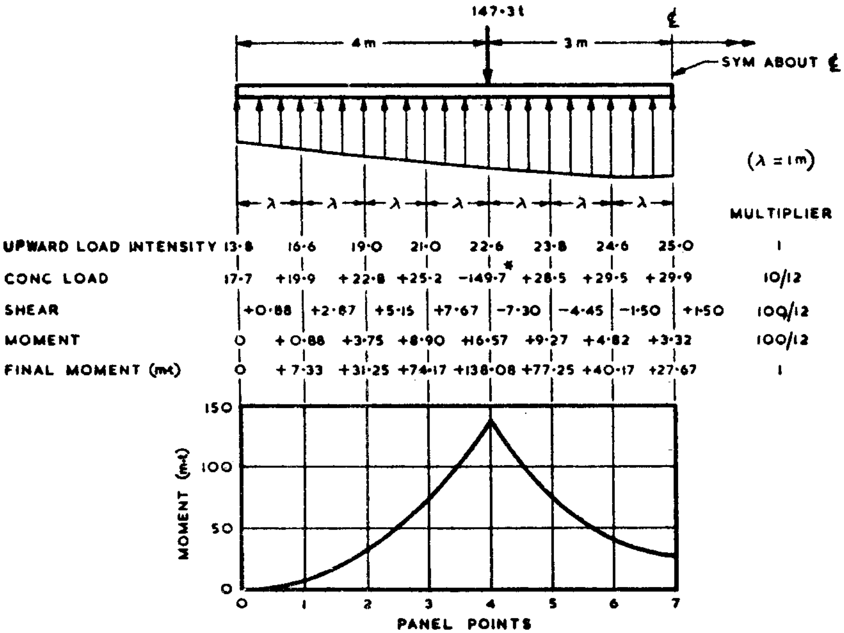


FIG. 12 ILLUSTRATION OF LOAD, SHEAR AND BENDING MOMENT

Example 3:

Assume that the upward pressure of soil on a one-metre wide strip of footing varies as shown in Fig. 13.



									MULTIPLIER
UPWARD LOAD INTENSITY	13·8	16·6	19·0	21·0	22·6	23·8	24·6	25·0	1
CONC LOAD	17·7	+19·9	+22·8	+25·2	−149·7*	+28·5	+29·5	+29·9	10/12
SHEAR	+0·88	+2·87	+5·15	+7·67	−7·30	−4·45	−1·50	+1·50	100/12
MOMENT	0	+0·88	+3·75	+8·90	+16·57	+9·27	+4·82	+3·32	100/12
FINAL MOMENT (mt)	0	+7·33	+31·25	+74·17	+138·08	+77·25	+40·17	+27·67	1

$$^{*}-147{\cdot}4\left(\frac{12}{10}\right) + 27{\cdot}08 = 149{\cdot}7$$

FIG. 13 ILLUSTRATION OF BENDING MOMENTS FOR THE REPLACEMENT OF A VARIABLE DISTRIBUTED LOAD

27.4 We now turn from the matter of determining shear and bending moment to the related problem of deflection calculation.

In finding the deflection of a beam under relatively complex load and support conditions, a most direct and accurate procedure, with a simple routine of calculation and self-checking characteristics, is a combination of 'Vestergaard's Conjugate Beam Method and Newmark's Numerical Procedure.

Consider the possibility that in a beam *AB* of length *L* there is a *local* or concentrated angle change ϕ_0 at a point distant '*a*' from the left end, as shown in Fig. 14.

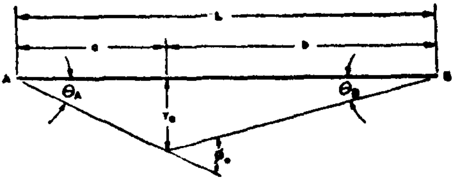


FIG. 14 CALCULATION OF DEFLECTION BY COMBINATION OF WESTERGAARD'S CONJUGATE BEAM METHOD AND NEWMARK'S PROCEDURE

The deflection $Y_a = \theta_A \cdot a = \theta_B \cdot b$ .... (25)

Thus $\theta_B = \theta_A \cdot \frac{a}{b}$ .... (26)

and, since $\phi_O = \theta_A + \theta_B$

$\therefore\ \theta_A = \phi_O \frac{b}{L}$ and $\theta_B = \phi_O \frac{a}{L}$ .... (27)

Finally, the deflection

$Y_a = \theta_A \cdot a = \phi_O \frac{ab}{L}$ .... (28)

Now, in place of the geometrical configuration of Fig. 14 consider the result of thinking of the concentrated angle change ϕ_O as a *load* acting on a fictitious or ' conjugate ' beam having the same length L as the beam in Fig. 14, with resulting shear and moment diagrams as shown in Fig. 15.

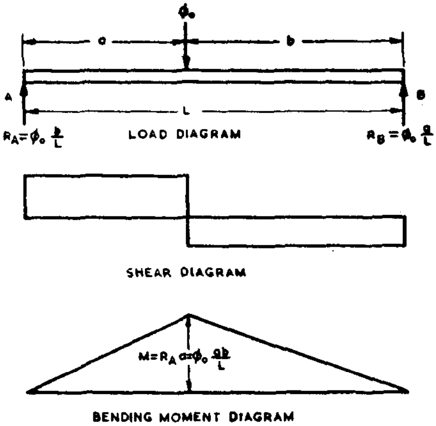


FIG. 15 LOAD, SHEAR AND BENDING MOMENT DIAGRAM FOR CONJUGATE BEAM

It is seen that the shear in the conjugate beam equals the slope in the real beam and the bending moment in the conjugate beam equals the deflection in the real beam.

The conjugate beam idea will now be used in the development of the underlying ideas in Newmark's Numerical Procedure as applied to the determination of beam deflections.

First, consider any arbitrary *segment* of a beam, λ in length, and plot the diagram of M/EI (*see* Fig. 16), thinking of it as a distributed load applied to a conjugate beam of length λ.

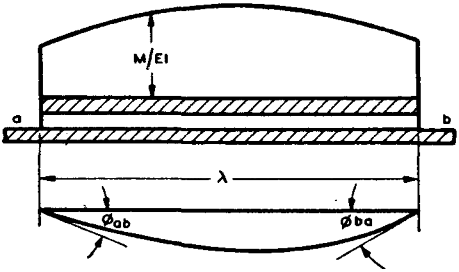


FIG. 16 LOAD OF THE CONJUGATE BEAM

The end slope ϕ_{ab} is the reaction at '*a*' in the conjugate beam and is the integration over *ab* of reaction caused by each incremental change in slope $d\theta = Mdx/EI$ acting as a load on the conjugate beam. If two successive segments, *ab* and *bc*, as shown in Fig. 17, are joined together to form a single smooth and continuous curve with a common tangent at *b*, the total change between the successive ' chords ' *AB* and *BC* is equal to the total panel point reaction at *b* of the distributed M/EI diagram acting as a load on the two beam segments *ab* and *bc*. Thus Eq 13 to 19 to determine panel point equivalent concentrations of load may also be applied to determine local ' concentrated angle changes ' by means of which the deflected curve of a beam may be replaced by a series of chord-like segments as shown in Fig. 18. The deflection at each juncture point is exactly equal to the deflection of the actual beam at that particular point.

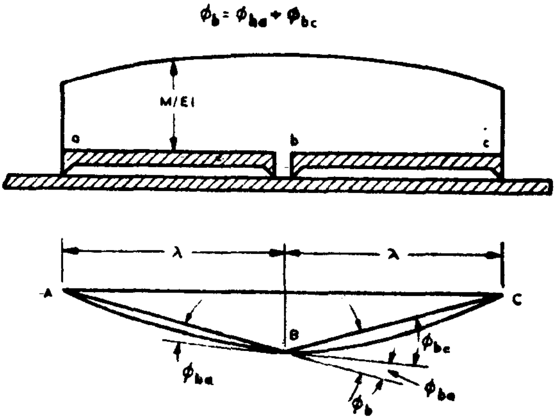


FIG. 17 CONJUGATE BEAM SEGMENTS

The conjugate beam relationships establish that the procedure previously demonstrated for determining shears and moments may be used to determine slopes and deflections. A geometrical demonstration of the procedure may be helpful. The average end slope θ_{01} may be determined as the average shear in panel 0-1 caused by the concentrated angle changes ϕ_1, ϕ_2 and ϕ_3 acting as loads on the conjugate beam. The shear in panel 1-2 is less than that in panel 0-1 by ϕ_1 — it is seen in Fig. 18 that θ_{12} is less than θ_{01} by ϕ_1. The deflection y_1 is equal to $\theta_{01}\lambda$, $y_2 = y_1 + \theta_{12}\lambda$ and so on across the beam — the same quantities are moments in the conjugate beam. As an introductory example, consider the case of the simple beam under uniform load.

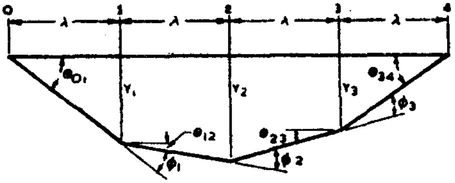


FIG. 18 SLOPES AND DEFLECTIONS OF CONJUGATE BEAM

Example 4:

Determine deflections in a uniformly loaded beam (*see* Fig. 19).

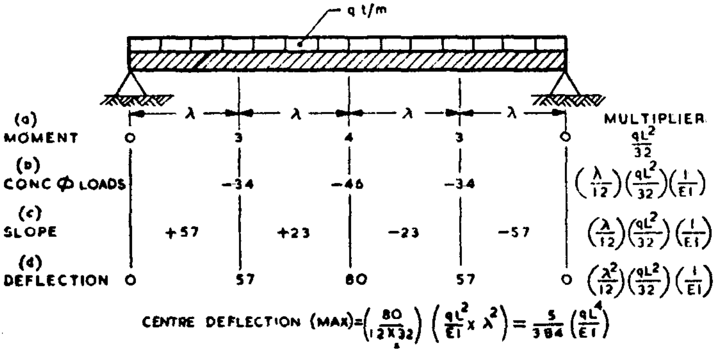


FIG. 19 DEFLECTION IN A UNIFORMLY LOADED BEAM

In line (*a*) of Fig. 19, the centre moment is $qL^2/8$ but $qL^2/32$ is introduced as a multiplier to provide convenient whole numbers for the arithmetic procedure that follows. The concentrated ϕ loads by Eq 16 to 19 are ‘exact’ since the moment diagram is a parabola. The loads are negative since downward load causes a negative change in shear when proceeding from left to right. The multiplier $\lambda/12$ out of Eq 16 to 19 is coupled with the multiplier in line (*a*) of Fig. 19. In line (*c*) of Fig. 19, it is not necessary to compute the slope in the first panel by determining the end shear. Because of symmetry, one knows that the slopes on either side of the centre line (shears in the conjugate beam) are equal in magnitude and opposite in sign. Thus, one may start with the concentrated angle change of 46 units at the centre and work to the right and left so as to provide symmetry in the slopes. It is noted, finally, that the deflections are exact at each panel point.

In cases where the loads are not uniform, the moment diagram no longer is a second degree curve in x and the numerical procedure does not always give an exact answer. But it still provides a close approximation. Consider the beam under a triangular load distribution and use the numerical procedure to determine shear, moment, slope, and deflection. In this example, the answers are exact, because of the continuous linear load variation.

Example 5:

Determine deflections in a beam under triangular loading (*see* Fig. 20).

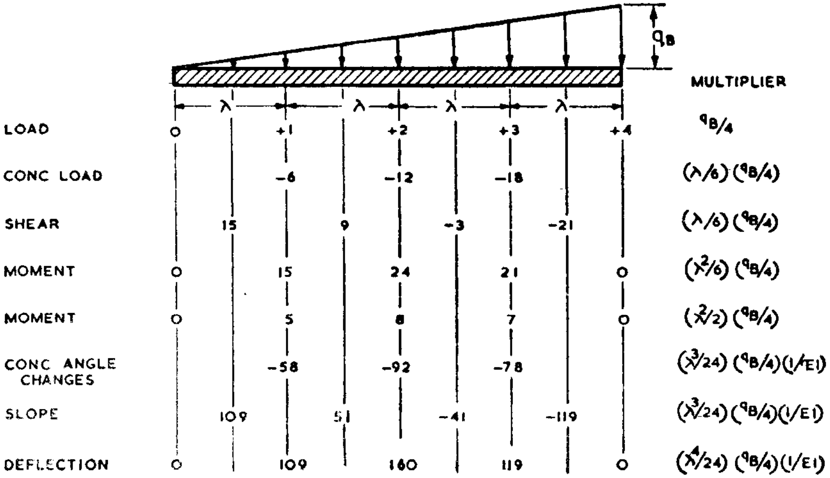


FIG. 20 DEFLECTION IN A BEAM UNDER TRIANGULAR LOADING

The foregoing examples have illustrated the numerical procedure of analysis but the real advantages in practical application are for those cases where the moment of inertia is variable and the load conditions more complex. Reference may be made to Design Example 14 where the deflection of a beam of variable moment of inertia is calculated by the same numerical procedure.

SECTION V

SPECIAL PROBLEMS IN BEAM AND GIRDER

28. GENERAL

28.1 This item covers certain special problems in beam design that are important when they do occasionally require special attention. These problems include biaxial bending of symmetrical sections, bending of unsymmetrical sections, with and without lateral constraint, bending of the channel section, and finally combined bending and torsion — when to avoid it and how to design for it if one is obliged to do so. Bending in these cases shall not be termed ' simple ' since it may involve deflections in a different plane from that in which the beam is loaded. A general discussion of the more typical problems will be illustrated by Design Examples 4 to 10, together with commentary, concerning the more typical cases that may be encountered in practice.

29. BIAXIAL BENDING

29.1 Whenever loads are applied on a beam at an angle other than 90° with respect to one of the principal axes, the beam is under ' biaxial ' bending. The load may be resolved into components in the direction of the two principal axes and the bending of the beam may be regarded as the superposition of the two bending components. This procedure for handling biaxial bending is particularly suited to the case in which there is at least one axis of symmetry since in this case the orientation of the principal axes is known by inspection and the distances from these axes to points in the beam cross-section that should be checked for stress are also immediately apparent. However, when the section itself has no axis of symmetry, such as in the case of an angle with unequal legs, the alternative procedure utilizing the general formulas for bending referenced to arbitrary X and Y axes in the plane of the cross-section is generally preferred. Nevertheless, if the orientation of the principal axes of the unsymmetrical section is determined, the first procedure may be used. Alternatively, unsymmetrical sections subjected to biaxial bending may be analysed graphically with convenience, by drawing the circle of inertia. Design Example 5 illustrates this method.

30. BIAXIAL BENDING OF A SECTION WITH AN AXIS OF SYMMETRY

30.1 The biaxial bending of a WB or I-section will be used as an example. The same procedure is applicable to the channel provided it is loaded through the shear centre (*see* **37**). The shear centre is an imaginary line parallel with the longitudinal beam axis through which the applied loads shall pass if twist is to be avoided.

The possible planes of application of the load are shown in Fig. 21. If the load is brought in at A in the direction shown by the arrow head, the member is loaded by twisting moment as well as in biaxial bending. Combined bending and twist will be discussed in **39**. If, however, the load is applied at B (coincident beam axis and shear centre) it may be resolved into components parallel with the X and Y axes and, since these are principal axes, the normal stress is simply the super-position of the effects of bending about the X and Y axes. In a simple span, the loading as shown in Fig. 21 would cause tension at C and compression at D. These locations would, therefore, govern what permissible stress should be used in the design. There is no explicit problem of lateral buckling involved, but if the horizontal component of bending approaches zero, the permissible stress should be governed by the reduced stress F_b as given in Table IIA, **9.2.2.2** of IS : 800-1956 (*see* Table II on p. 172 of this Handbook for an extension of the Table in IS : 800-1956). However, it is also obvious that if all of the load were applied horizontally in the X-X plane, the full allowable stress of 1 575 kg/cm^2 should govern as there is no tendency for lateral buckling. As a conservative basis for design, therefore, it is recommended that a simple interaction formula be used in such a case.

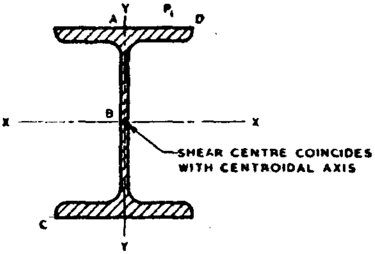


FIG. 21 BIAXIAL BENDING OF SECTION WITH TWO AXES OF SYMMETRY

$$\left(\frac{f_b}{F_b}\right)_{xx} + \left(\frac{f_b}{F_b}\right)_{yy} \ngtr 1 \qquad \ldots\ldots\ldots \quad (29)$$

In Eq 29, f_b represents the computed stress due to the two components of bending moment and F_b is the corresponding permissible stress if that component alone were acting. In the case of rolled wide flange or I-beams, the permissible stress F_b for bending about the Y-Y axis would always be 1 575 kg/cm^2. Members with relatively wide flanges, such as might be used for struts, will be most economical for biaxial bending.

The foregoing formula Eq 29 may be considered as an extension of the formula in **9.5** of IS: 800-1956 for combined bending and axial stress. In this case, the axial stress is zero. Sub-clause **9.5.1** in that standard states:

> 'When bending occurs about both axes of the member, f_b shall be taken as the sum of the two calculated fibre stresses.'

This provides no gradual interaction for the case under consideration here but when the allowable fibre stress about each axis is the same, the proposed interaction formula for biaxial bending reduces to that stated in **9.5** of IS: 800-1956.

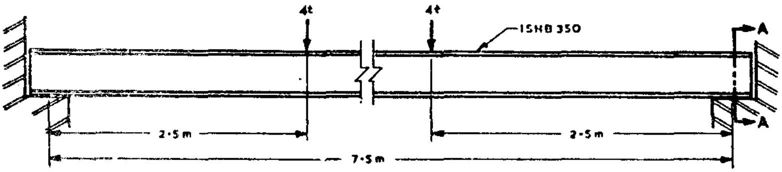
31. DESIGN EXAMPLE OF BIAXIAL LOADED BEAM

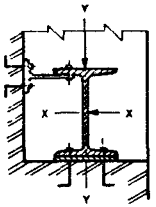
31.1 The illustrative design example of biaxial loaded beam is shown in the following two sheets (*see* Design Example 4).

Design Example 4 — Biaxial Loaded Beam

Design Example 4 | **1 of 2**
Biaxial Loaded Beam

The procedure for designing a beam with one or more axes of symmetry under biaxial bending is illustrated. Only that phase of design peculiar to biaxial bending is treated in this illustrative example and the deflections, if desired, could also be found as the superposition of deflections in the x and y directions as caused by the loads separately applied. It is to be noted that the section loaded as shown in Fig. 19 will not deflect in the plane of the loads.





ENLARGED SECTION AA

An I-shaped beam spans 7·5-m beam between supports. At each end it is adequately supported vertically and against twist as shown in the figure. In addition to its own weight, the beam carries 4-tonne loads at the third points and a horizontal load of 150 kg/m.

Estimate weight of beam @ 75 kg/m

Bending about X-X axis

$$\text{Dead load } M_{xx} = \frac{75 \times 7{\cdot}5^2}{8} = 527 \text{ m·kg}$$

$$\text{Live load } M_{xx} = 4 \times 2{\cdot}5 \times 1\,000 = \underline{10\,000 \text{ m·kg}}$$

$$10\,527 \text{ m·kg}$$

Bending about Y-Y axis:

$$M_{yy} = \frac{150 \times (7{\cdot}5)^2}{8} = 1\,055 \text{ m·kg}$$

As a preliminary guide, determine required Z_{xx}, Z_{yy} for M_{xx}, M_{yy} acting separately, estimating $f_{b(xx)} = 1\,200$ kg/cm².

$$Z_{xx} = \frac{10\,527 \times 100}{1\,200} = 877{\cdot}25 \text{ cm}^3, \quad Z_{yy} = \frac{1\,055 \times 100}{1\,575} = 67 \text{ cm}^3$$

Try ISHB 350, 67·4 kg. $Z_{xx} = 1\,094{\cdot}8$ cm³; $Z_{yy} = 196{\cdot}1$ cm³

$$\frac{l_{xx}}{b} = \frac{7{\cdot}5 \times 100}{25{\cdot}0} = 30, \quad d/t_f = \frac{350}{11{\cdot}6} = 30, \quad F_{b(xx)} = 1\,160 \text{ kg/cm}^2$$

Design Example 4

Biaxial Loaded Beam

2 of 2

Section check by Interaction Formula

$$f_{b(xx)} = \frac{10\,527 \times 100}{1\,094{\cdot}8} = 963 \text{ kg/cm}^2$$

$$f_{b(yy)} = \frac{1\,055}{196{\cdot}1} \times 100 = 537 \text{ kg/cm}^2$$

$$\frac{963}{1\,160} + \frac{537}{1\,575} = 1{\cdot}2 > 1 \text{ — No Good.}$$

Try ISHB 400, 77·4 kg

$$Z_{xx} = 1\,404{\cdot}2 \text{ cm}^3$$

$$Z_{yy} = 218{\cdot}3 \text{ cm}^3$$

$$l/b = \frac{7{\cdot}5}{25{\cdot}0} \times 100 = 30$$

$$d/t_f = \frac{400}{12{\cdot}7} = 31{\cdot}5$$

$$F_{xx} = 1\,123 \text{ kg/cm}^2$$

$$F_{byy} = 1\,575 \text{ kg/cm}^2$$

$$f_{bxx} = \frac{10\,527 \times 100}{1\,404{\cdot}2} = 750 \text{ kg/cm}^2$$

$$f_{byy} = \frac{1\,055}{218{\cdot}3} \times 100 = 483$$

$$\therefore \frac{750}{1\,123} + \frac{483}{1\,575} = 0{\cdot}975 < 1 \text{ ..... OK.}$$

Then check web shear, bearing plate, web crushing, web buckling, etc, for vertical loads in the same manner as for usual beam design.

32. LATERALLY CONSTRAINED BENDING OF SECTIONS WITH NO AXIS OF SYMMETRY

32.1 If an asymmetrical section, such as a Z-bar or unequal legged angle (*see* Fig. 22) is loaded in a plane parallel to one of the surfaces, it will, if unconstrained, bend in some other plane. However, if laterally supported by the roof or by other means, the bar may be constrained to bend in the same plane as the load. When so constrained, the stress due to applied loads may be calculated as in simple bending. For example, if deflections are permitted only in the Y-Y plane and the loads are in the same direction, the stress due to bending about the X-axis will be given by the usual beam equation:

$$f_b = \frac{M_x y}{I_x} \qquad \ldots\ldots\ldots\ldots\ldots (30)$$

if the beam is constrained in the x direction at the same points at which it is loaded in the y direction, the lateral constraining forces in the x direction will be related to the applied forces in the y direction by the ratio of I_{xy}/I_x where I_{xy} is the product of inertia. Hence, with lateral constraint:

$$M_y = \frac{M_x I_{xy}}{I_x} \qquad \ldots\ldots\ldots\ldots\ldots (31)$$

The product of inertia is not to be confused with the polar moment of inertia and is given by the following equation:

$$I_{xy} = \int_A xy dA \qquad \ldots\ldots\ldots\ldots\ldots (32)$$

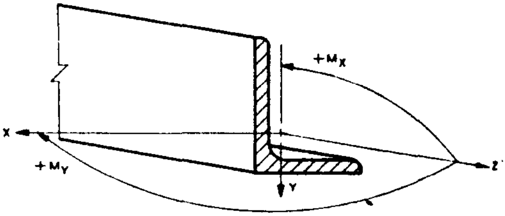


FIG. 22 ASYMMETRICAL SECTION UNDER BIAXIAL BENDING

33. DESIGN EXAMPLE OF ANGLE SECTION BEAM LOADED IN THE PLANE OF WEB

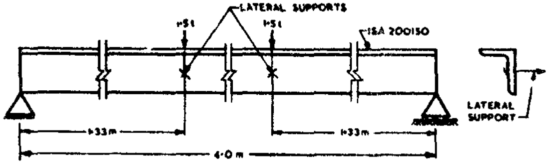
33.1 The illustrative design example of angle section beam loaded in the plane of web is shown in the following two sheets (*see* Design Example 5).

Design Example 5 — Angle Section Beam Loaded in the Plane of Web

Design Example 5	**1**
Angle Section Beam	**of 2**

Angle section beam with third point loading and 4-m span is designed under the assumption that lateral support bars are introduced along with the loads, as in the sketch. These bars force bending to be in the same plane as the loads. In this case, the design is routine and the required amount of lateral support is determined by Eq 31 on p. 104. Particular note should be made of the fact that the lateral force is approximately 41 percent of the vertical applied load and the lateral support bars shall have to be designed for this lateral force and, in addition, be of requisite rigidity.

By way of comparison, at the end of the calculations the stress is determined for the same load but with the lateral support removed. Eq 33 on p. 107 applies in this case and the maximum compression stress is found to be increased by a factor of 1·22. In the case of Z-bars, the factor will be as high as 2·5 thus going beyond the yield point of structural grade steel. It is obvious that the use of Z and angle bars loaded in the plane of the web is not particularly economical in view of the lateral support requirements. Z and angle bars are sometimes used as purlins on sloping roofs where, if properly oriented, the plane of the loads may be near the principal axis of the section and efficient use of the material thus realized.



Span = 4·0 m;

Load = 1·5 t at each 1/8 point

Lateral support is provided at each load point

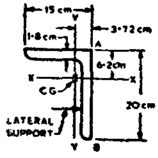
Estimate DL $= 30$ kg/m;

LL bending moment $= 1{\cdot}5\times1{\cdot}33\times1\,000 = 1\,995$ m·kg

DL bending moment $= \dfrac{30\times4^2}{8} = \underline{60}$ m·kg

$2\,055$ m·kg

Required $Z_x = \dfrac{2\,055\times100}{1\,500} = 137$ cm^3



ENLARGED VIEW

Design Example 5

Angle Section Beam

2 of 2

Use ISA 200 150, 15·0 mm angle.

$I_x = 2\,005{\cdot}6$ cm^4 $\quad I_y = 969{\cdot}9$ cm^4 $\quad Z_x = 145{\cdot}4$ cm^3

Lateral support requirement to provide for lateral bending moment:

$$(M_y) = \frac{M_x I_{xy}}{I_x} \quad (\textit{see} \text{ Eq } 31), \quad I_{xy} = -818{\cdot}5 \text{ cm}^4$$

$$\text{Lateral force} = \frac{-818{\cdot}5 \times 1{\cdot}5}{2\,005{\cdot}6}$$ = 0·612 t at each load point (neglecting the effect of dead load) (about 41 percent of the applied load)

What would be normal stress due to biaxial bending if no lateral support be provided ?

$$f = \frac{M_x}{I_x I_y - I_{xy}^2} (I_y y - I_{xy} x) \qquad (\text{check point } A;\ y = 6{\cdot}2 \text{ cm},\ x = 3{\cdot}72 \text{ cm})$$

$$\text{At } A, f = \frac{2\,055 \times 100}{2\,005{\cdot}6 \times 969{\cdot}9 - (818{\cdot}5)^2} \left[(-6{\cdot}2)\ (969{\cdot}9) - (-818{\cdot}5)\ (-3{\cdot}72) \right]$$

$$= -1\,460 \text{ kg/cm}^2 \text{ compression}$$

At B, $y = +13{\cdot}8$ cm
$x = -2{\cdot}22$ cm

$$\text{At } B, f = \frac{2\,055 \times 100}{2\,005{\cdot}6 \times 969{\cdot}9 - (818{\cdot}5)^2} (+13{\cdot}8 \times 969{\cdot}9) - (-818{\cdot}5) \times (-2{\cdot}22)$$

$$= \frac{2\,055 \times 100}{1\,275\,000} \times 11\,550 = 1\,860 \text{ kg/cm}^2 > 1\,500 \text{ kg/cm}^2$$

(permissible tensile stress)

Thus, if no lateral support be provided, the allowable load gets reduced in the proportion of 1 860 to 1 500 kg/cm^2 (1·24: 1).

NOTE — As the load assumed here is vertical, there is no M_y component. If the load were inclined, the components M_x and M_y should be determined and the stress calculated by Eq 33 and Eq 34 (*see* p. 107) and added to give the total fibre stress at either of the points, *A* or *B*.

34. UNCONSTRAINED BENDING OF SECTIONS WITH NO AXIS OF SYMMETRY

34.1 The most common example in ordinary use of the unsymmetrical section with no axis of symmetry is the rolled angle with unequal legs. Taking X and Y axes positive in the directions as shown in Fig. 22 and oriented parallel with the sides of the angle, the positive sense of bending moment components M_x and M_y are shown as chosen in the same figure. In sections of this type, it is convenient to resolve the bending moments due to any applied loads into components about the X and Y axes and calculate the stress due to bending by the following equations:

Bending about X-axis:

$$f_b = \frac{M_x}{I_x I_y - I^2_{xy}} (I_y y - I_{xy} x) \qquad \ldots\ldots\ldots\ldots (33)$$

Bending about Y-axis:

$$f_b = \frac{M_y}{I_x I_y - I^2_{xy}} (I_{xy} y - I_x x) \qquad \ldots\ldots\ldots\ldots (34)$$

If deflections are desired, it is preferable to determine the principal axes of inertia and study the bending problem by the same procedure followed in Design Example 4 for biaxial bending of a doubly symmetrical section.

The derivation of the foregoing Eq 33 and Eq 34 may be found in any advanced book on strength of materials, such as Timoshenko's 'Strength of Materials', 3rd ed, Part I, p. 230-231 published by D. Van Nostrand Company, Inc., New York. As examples of unsymmetrical bending, both constrained and unconstrained, Design Examples 5 and 6 concerning angle section are presented.

35. DESIGN EXAMPLE OF ANGLE BEAM LOADED PARALLEL TO ONE SIDE

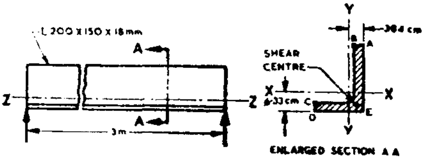
35.1 The illustrative design example of angle beam loaded parallel to one side is shown in the following sheet (*see* Design Example 6).

Design Example 6 — Angle Beam Loaded Parallel to One Side

Design Example 6	**1**
Angle Beam	**of** **1**

This is another example of a single angle, used as a beam and loaded in a plane parallel to one of its sides. No lateral support is provided. To get a preliminary trial design, the required section modulus for an angle with lateral support but with the allowable stress multiplied by a factor of 0·7 is determined. The properties of the angle having this modulus are tabulated and the product of inertia is determined. There are five possible locations for maximum stress as noted by letters A to E. However, if one visualizes the approximate direction of the principal axes of inertia it is obvious that both components of bending if resolved in the direction of the principal axes would produce compression at B and tension at E. These stresses are calculated by Eq 33 (see p. 107) and the maximum stresses are found to be within permissible limits. If there is any question about the location of maximum stress in the mind of the designer, it should also be checked at A, C and D.

For the angle to bend without twist, the resultant of the loads should go through the shear centre which is at the intersection of the middle planes of the two legs.



Single Angle as Beam Design

A single angle beam has a simply supported span of 3 metres with loads given below:

Live load = 1·50 t/m

Dead load = 0·20 t/m (including beam weight)

W = 1·70 t/m

Angle free to deflect laterally will be stressed more than if held. As an approximation, assume unsupported angle has capacity of 0·7 × supported angle

$$M_x = \frac{1{\cdot}70 \times 3^2}{8} = 1{\cdot}91 \text{ m·t, permissible stress} = 1\,500 \text{ kg/cm}^2.$$

$$\text{Required } Z_x = \frac{1{\cdot}91 \times 1\,000 \times 100}{1\,500 \times 0{\cdot}7} = 182 \text{ cm}^3$$ Try heaviest ISA 200 150, 18 mm section.

46·9 kg/m with, $Z_x = 172{\cdot}5 \text{ cm}^3$, $Z_y = 101{\cdot}9 \text{ cm}^3$, $I_{xy} = 958{\cdot}1 \text{ cm}^4$, $I_x = 2\,359 \text{ cm}^4$, $I_y = 1\,136{\cdot}9 \text{ cm}^4$

$$f = \frac{M_x}{I_x I_y - I_{xy}^2}(I_{yy} - I_{xy}x)$$ Maximum compressive stress is at B ($x = +2{\cdot}34$, $y = -13{\cdot}67$)

$$(f_b)B = \frac{191\,000}{1\,760\,000}(-1\,136{\cdot}9 \times 13{\cdot}67 + 958{\cdot}1 \times 2{\cdot}34)$$

$= 1\,440$ kg/cm² .....OK as permissible bending stress for $l/r_y = 69 \left(\frac{300}{4{\cdot}36}\right)$ $= 1\,500$ kg/cm²

Maximum tensile stress at E ($x = -3{\cdot}84$ cm, $y = 6{\cdot}33$ cm)

$$f_b(E) = \frac{191\,000}{1\,760\,000}(1\,136{\cdot}9 \times 6{\cdot}33 + 958{\cdot}1 \times 3{\cdot}84)$$

$= 1\,180 \text{ kg/cm}^2 < 1\,500 \text{ kg/cm}^2$ ..... OK [*see* **9.2.1**(b) of IS: 800-1956]

NOTE — As the load assumed here is vertical, there is no M_y component. If the load were inclined, the components M_x and M_y should be determined and the stress calculated by Eq 33 and 34 and added to give the total fibre stress at either of the points, A or B.

36. DESIGN EXAMPLE OF ANGLE BEAM DESIGN BY DRAWING CIRCLE OF INERTIA

36.1 The Design Example 6 of angle beam is now designed by drawing circle of inertia in the following two sheets (*see* Design Example 7).

Design Example 7 — Angle Beam Design by Drawing Circle of Inertia

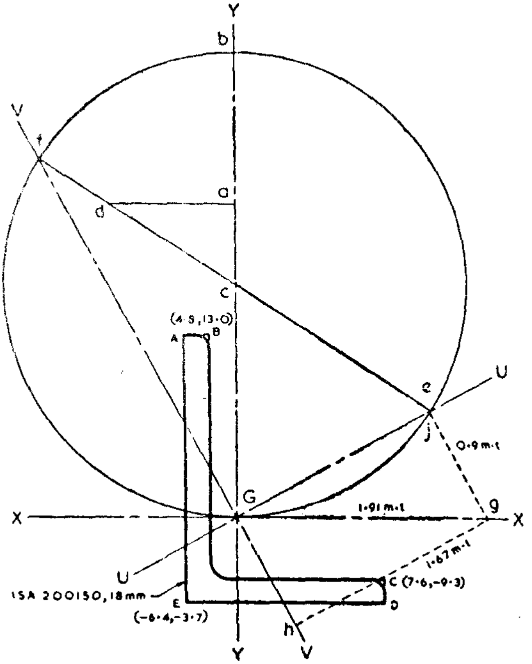
Design Example 7

Angle Beam Design by Circle of Inertia Method

1 of 2

In this example, the same problem as given in Design Example 5 will be solved graphically by drawing the circle of inertia. The method is as follows and the construction is given in the sketch.

The X-X and Y-Y axes are drawn from the centroid G of the angle section. Starting from G on the Y-Y axis, a and b are marked such that $Ga = I_{xx}$ *and* $ab = I_{yy}$ *to any convenient scale. With Gb as diameter and c as the centre, the circle of inertia is drawn. From 'a' towards left (as the product of inertia is negative in this case), ad is drawn parallel to X-X axis such that* $ad = I_{xy}$ *(product of inertia). c and d are joined and produced both ways to meet the circle at e and f. Now Ge and Gf determine the principle axes U-U and V-V of the angle section. With reference to X-X and Y-Y axes, g is plotted such that its co-ordinates represent bending moments* M_{xx} *and* M_{yy} *given. In this case, as* $M_{yy} = 0$*, g lies on the X-X axis. If gh and gj are drawn perpendicular to the principal axis, these represent respectively the components* M_{uu} *and* M_{vv}*. From inspection of the orientation of the angle section and the principal axis, it is obvious that maximum fibre stress should be checked at points B and E. If there is any question about the location of maximum stress in the mind of the designer, it should also be checked at A, C and D. The co-ordinates of these points with reference to the principal axes are measured. The fibre stress at the points could be determined as given below.*



Design Example 7

Angle Beam Design by Circle of Inertia Method

2 of 2

Co-ordinates of C with reference to

X-X and Y-Y axes $(+3{\cdot}84,\ +6{\cdot}33)$

$Ga = I_{xx} = 2\,359{\cdot}4\ \text{cm}^4$

$ab = I_{yy} = 1\,136{\cdot}9\ \text{cm}^4$

$ad = I_{xy} = -958{\cdot}1\ \text{cm}^4$

$ed = I_{uu} = 2\,880\ \text{cm}^4$

$df = I_{vv} = 616\ \text{cm}^4$

$Gg = M_{xx} = 1{\cdot}91\ \text{m·t}$ (*see* p. 108)

$gj = M_{uu} = 0{\cdot}9\ \text{m·t}$

$gh = M_{vv} = 1{\cdot}68\ \text{m·t}$

Co-ordinates of points B and E with reference to U-U and V-V axes are:

$B(+48,\ +130)$

$E(-65,\ -37)$

$$f = \frac{M_{uu}}{I_{uu}} \times v + \frac{M_{vv}}{I_{vv}} \times u$$

$$\therefore\ f_B = \frac{1{\cdot}68}{2\,880} \times \frac{13 \times 100 \times 1\,000}{1} + \frac{0{\cdot}9}{616} \times \frac{4{\cdot}8 \times 100 \times 1\,000}{1}$$

$$= +1\,450\ \text{kg/cm}^2 \quad (\text{compression})$$

$$f_E = \frac{-1{\cdot}68 \times 3{\cdot}7 \times 100 \times 1\,000}{2\,880} - \frac{0{\cdot}9 \times 4{\cdot}8 \times 100 \times 1\,000}{616}$$

$$= -1\,170\ \text{kg/cm}^2 \quad (\text{tension})$$

The fibre stress is in all cases less than 1 500 kg/cm^2 ..... OK.

It may be observed that the values f_B and f_E are very close to those as determined by the method given in Design Example 6.

37. BENDING OF CHANNELS WITHOUT TWIST

37.1 When a channel section is used as a beam with loads applied parallel with the web, it will bend without twist only if loaded and supported through its shear centre in which case the resultant shear stress in each flange acts horizontally and in opposite directions. The resultant shear stress in the web is simply the total shear at the cross-section due to the externally applied loads. The web shear and the couple produced by the flange shears may be replaced by a single resultant shear force, the location of which determines the shear centre. The location of the shear centre in a channel section may be calculated, in reference to the dimensions shown in Fig. 23 by the following Eq 35 for sloping and parallel flange sides, respectively:

$$X_s = \frac{X_c h^2}{4r_x^2} \quad (\textit{see}\ \text{Fig. 23 for symbols}) \quad \ldots\ldots\ldots\ldots (35)$$

It is usually impracticable to load a channel through its shear centre although it has been done in some special cases where the peculiar properties of the channel section have actually been utilized to advantage. A channel may be constrained against twisting at the load points and supported at its ends approximately at the shear centre location, as shown in the Design Example 8. The channel may then be treated as if it were being supported and loaded along its shear centre with restraining moments supplied at framing beam connections so as to apply the load resultants effectively at the shear centre.

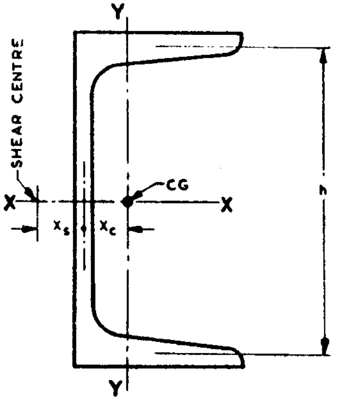


FIG. 23 SHEAR CENTRE OF CHANNEL SECTION

38. DESIGN EXAMPLE OF SINGLE CHANNEL AS BEAM

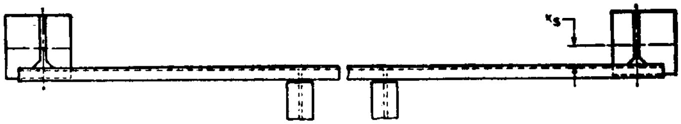
38.1 The illustrative design example of single channel as beam is shown in the following two sheets (*see* Design Example 8).

Design Example 8 — Single Channel as Beam

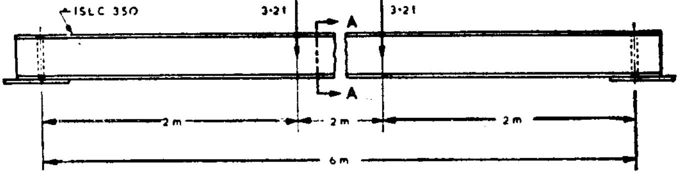
Design Example 8	1 of 2
Channel Beam	

The plan and elevation views show that the channel is loaded at its third points with an effective span of 6 m. A stiffener at each support point centralizes the bearing pad reaction under the shear centre. Section A-A shows the manner in which the beams are to be welded to the web of the channel. The beam weight is estimated and the moments due to dead and applied loads along with required section modulus are calculated in the usual manner. In the case of a single channel, the specification permits only 1 500 kg/cm² as the permissible stress. A channel with the required section modulus is selected and the allowable stress is checked for the 2-m unsupported length at the centre. The shear centre is located by Eq 35 and found to be 3·09 cm from the centre plane of the web or 2·72 cm from the back of the web. The ends of the framing beams, assumed to be ISLB 300 sections, are checked for connection and coped section strength at a-a and b-b. The weld along a-a is checked to make sure of its capacity to transit the moment that will be developed by the tendency of the channel to twist. The stress along line b-b, at end of coping, is also checked for eccentricity of 15·96 cm.

For a discussion of shear centre in more unusual shapes, reference may be made to Timoshenko's 'Strength of Materials', 3rd ed, Part I, p. 240, published by D. Van Nostrand Company, Inc., New York.



PLAN



ELEVATION

Single Channel as Beam

A channel beam is designed for loading as shown. Clear span = 6 m

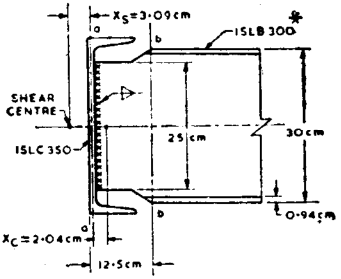
Estimate beam weight = 45 kg/m.

Moment due to conc load = $3{\cdot}2 \times 2 \times 1\,000 = 6\,400$ m·kg.

Moment due to dead load = $\frac{45 \times 6^2}{8}$ = 202·5 m·kg

= 6 602·5 m·kg

Required Z = $\frac{6\,602{\cdot}5 \times 100}{1\,500} = 4\,402$ cm³



ENLARGED SECTION AA

Design Example 8	2 of 2
Channel Beam	

Use ISLC 350, 38·8 kg

$Z_{xx} = 532{\cdot}1$ cm^3

$C_{yy} = 2{\cdot}41$ cm

$r_{xx} = 13{\cdot}72$ cm

$b = 100$ mm

$t_f = 12{\cdot}5$ mm

$h = d - t_f = 33{\cdot}75$ cm

$t_w = 7{\cdot}4$ cm

$X_c = C_{yy} - t_w/2 = 2{\cdot}04$ cm

Check allowable stress for 2 m unsupported length.

$$l/b = \frac{200}{10} = 20$$

By reference to IS: 800-1956 conservatively applicable to single channels, allowable stress is 1 500 kg/cm^2. Beam selection is, therefore, OK. Locate shear centre.

$$X_s = \frac{X_c h^2}{4 r_x^2} = \frac{2{\cdot}04 \times 33{\cdot}75}{4 \times 13{\cdot}72^2} = 3{\cdot}09 \text{ cm}$$

$3{\cdot}09 - 0{\cdot}37 = 2{\cdot}72$ cm from back of web, thus locating the effective location of supports and loads for cases where rotation is not permitted.

Design connecting weld between ISLB 300 web and channel web for an eccentricity of $2{\cdot}72 + 0{\cdot}74 = 3{\cdot}46$ cm [*see* ISI Handbook for Structural Engineers on Welded Connections for Design (under preparation)].

Check stress at end of coped section ($b-b$ in sketch) for eccentricity of $12{\cdot}5 + 2{\cdot}72 = 15{\cdot}22$ cm.

$$M = 15{\cdot}22 \times 3{\cdot}2 = 4{\cdot}87 \text{ cm·t}$$

Section modulus of coped web 25 × 67 cm

$$Z = \frac{0{\cdot}67 \times 25^2}{6} = 70 \text{ cm}^3$$

$$f = \frac{48{\cdot}7 \times 1\,000}{70} = 696 \text{ kg/cm}^2 < 1\,500 \text{ kg/cm}^2$$

Check end shear:

$\frac{3{\cdot}2 \times 1\,000}{25 \times 0{\cdot}67} \times \frac{3}{2} = 286$ kg/cm^2 $<$ 1 025 kg/cm^2 (*see* **9.3.1** of IS: 800-1956) . . . OK.

If web of channel is welded to its supports at each end, weld should be designed for an eccentricity of $3{\cdot}09 + \frac{0{\cdot}74}{2} = 3{\cdot}46$ cm.

NOTE — Foregoing design procedure is applicable only when members framing into channel together with end connections have moment capacity and resistance to rotation to effectively load channel in the plane of its shear centre.

If not so restrained, channel will twist, and shear stress due to torsion shall be included.

39. COMBINED BENDING AND TORSION

39.1 The structural engineer concerned with bridges and buildings occasionally finds torsion combined with bending; rarely is torsion without bending a design problem. If an ' open ' section, such as a wide flange shape or channel is used in a location where it is loaded in torsion as well as in bending, the torsion should be minimized as much as possible, preferably eliminated entirely by provision of suitable restraints or other means. Open sections are notoriously weak in torsion. *For minimum use of steel, if the engineer has to design for appreciable torsion, he should use a box or closed section.*

Although a very brief treatment of the torsion problem in open sections will be presented here, reference may be made to a pamphlet on ' Torsional Stresses in Structural Beams (Booklet S-57) ' available through the Bethlehem Steel Corporation, New York. This supplies information for a variety of cases of torsion combined with bending when wide flange or I-beam sections are used.

The torsional properties of an open section may be built up from those of the narrow rectangle. Torsional moment is related to sectional properties by Eq 36:

$$M_T = KG\theta \qquad \ldots\ldots\ldots\ldots\ldots (36)$$

In the foregoing equation, K is the torsion constant which, in the case of the circular section, becomes the polar moment of inertia. G is the shearing modulus of elasticity which for steel is 0·385 times the tensile modulus. θ is the angle of twist per unit length.

The torsional constant for the rectangle shown in Fig. 24A is:

$$K = \frac{bt^3}{3} - 0{\cdot}21\, t^4 \qquad \ldots\ldots\ldots\ldots (37)$$

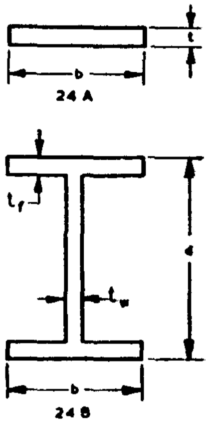
In general, for open shapes:

$$K = \Sigma\ (K \text{ of rectangular components}) \qquad \ldots\ldots\ldots\ldots (38)$$

As a typical example of an open section, K for the wide flange shape with parallel sides is obtained by summing the torsion constants of its rectangular component parts as follows:

Wide Flange Shape Fig. 24B

$$K = \frac{2bt_f^3}{3} + \frac{(d - 2t_f)}{3}\, t_w^3 - 0{\cdot}42\, t_f^4 - 0{\cdot}21\, t_w^4 \qquad \ldots\ldots\ldots (39)$$



24 A

24 B

FIG. 24 TORSION OF OPEN THIN SECTIONS

The stress due to torsion is given for either the wide flange or rectangular shape by Eq 41:

$$f_{t_{\max}} = \frac{M_T t_{\max}}{K} \qquad \ldots\ldots\ldots\ldots (40)$$

When loads cause a combination of torsion and bending, in a wide flange beam, the torsion is non-uniform. Longitudinal normal stresses due to localized torsional flange bending stresses are developed which add to the normal stress due to bending calculated in the usual manner.

When the objective of saving steel is paramount, one should avoid exposing open sections to torsion. Attention will, therefore, be turned to the torsional properties of the box section with a subsequent comparison to the open section.

The torsional moment resisted by a simple rectangular box section as shown in Fig. 25 is as follows:

$$M_T = 2whf_T t_w \qquad \ldots\ldots\ldots\ldots (41)$$

where f_T = Allowable torsional shear stress. The associated torsional constant for the same section is:

$$K = \frac{4w^2h^2}{\Sigma \frac{s}{t}} \qquad \ldots\ldots\ldots\ldots (42)$$

s is any distance along the periphery along which t is constant.

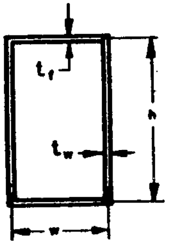


FIG. 25 BOX SECTION IN TORSION

As a demonstration of the superiority of the box section, Fig. 26 shows exactly the same area of steel used for a hypothetical wide flange shape in *A* and a box section having exactly the same bending strength area in *B*. The torsion constant is calculated by Eq 39 for the wide flange shape as follows:

$$K=2/3\times25{\cdot}0(2{\cdot}5)^3+\tfrac{1}{3}(45)(2{\cdot}0)^3-0{\cdot}42\times2{\cdot}5^4-0{\cdot}21\times2{\cdot}0^4=360{\cdot}2\text{ cm}^4$$

For the allowable shear stress of 945 kg/cm², Eq 40 provides a means of estimating the torsional moment capacity which is found as follows:

$$M_T=\frac{f_T K}{t_{\max}}=\frac{945\times360{\cdot}2}{100\times2{\cdot}5\times1\,000}=1{\cdot}36\text{ m}{\cdot}\text{t}$$

For the box section using Eq 41, the capacity is found to be:

$$M_T=2whf_Tt_w=\frac{2\times24\times47{\cdot}5\times945\times100}{100\times1\,000}=22{\cdot}5\text{ m}{\cdot}\text{t}$$

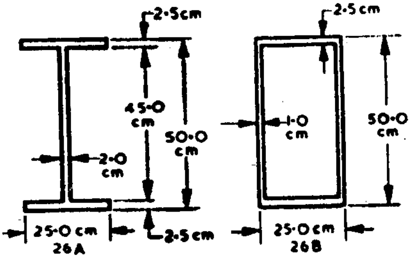


FIG. 26 COMPARISON OF TORSIONAL STRENGTH OF TWO SECTIONS OF EQUAL AREA AND EQUAL BENDING STRENGTH

Thus, exactly the same amount of steel provides $\frac{22{\cdot}5}{1{\cdot}38} = 16{\cdot}3$ times as much torsional strength in the box as in the open section. The torsion constant K for the box section is obtained from Eq 42 as:

$$K = \frac{4 \times 25^2 \times 47{\cdot}5^2}{2\left(\frac{50}{1} + \frac{23}{2{\cdot}5}\right)} = 47\ 700 \text{ cm}^4$$

Thus, for equal area, the box is found to be $\frac{47\ 700}{360{\cdot}2} = 132$ times more rigid than the wide flange shape.

If the wide flange shape were loaded to approximately 1/16 the torsional moment of the box so as to make each have approximately the same torsional shear of 945 kg/cm^2, the wide flange shape would twist 8 times as much as the box of the same length.

The foregoing comparison applies only to uniform torsion and not to combined bending and torsion. In combination with bending, similar relations would apply but the differences between the two behaviours would be reduced slightly.

The longitudinal stresses developed in the flanges in the case of uniform torsion of wide flange shapes introduce a complex and serious design limitation that is relatively absent in box section of proportions required in heavy steel structures. The following design example will demonstrate the actual design procedure that might be followed if and when the torsion problem could not be avoided in a steel structure.

Finally, an important attribute of the box beam, because of its great torsional rigidity, is the fact that it may be used within reasonable limits with no stress reduction when laterally unsupported.

40. DESIGN EXAMPLE OF BOX GIRDER FOR COMBINED BENDING AND TORSION

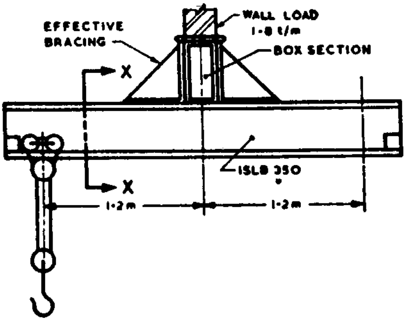
40.1 The illustrative design example of design of box girder for combined bending and torsion is shown in the following two sheets (*see* Design Example 9).

It is noteworthy that even greater torsional loads might be carried than those assumed in the design example with no apparent penalty to the bending strength of the box girder. This illustration amply demonstrates the importance of using a box section when torsion has to be included with bending in a design problem.

Design Example 9 — Box Girder for Combined Bending and Torsion

As shown in the sketch, the box girder is of 5-m span and carries a wall load of 1·8 tonnes per metre. At the centre, there is suspended a monorail I-beam hoist that cantilevers out 1·2 ml to either side of the box beam. The monorail hoist beam is first designed as a simple cantilever beam so as to obtain the dead load that it adds at the centre of the box beam. The bending moment in the box beam is now calculated and the required section modulus for bending alone determined on the basis of an allowable stress of 1 575 kg/cm^2. If the box were built up entirely of plates welded along 4 longitudinal lines, the allowable stress should probably be 1 500 kg/cm^2 but it is planned to use two channel sections to form the box and in this case a liberal interpretation of the code would indicate an allowable bending stress of 1 575 kg/cm^2.

Design Example 9	**1 of 2**
Box Section for Combined Bending & Torsion	



Hoist load = 5 t

Weight of hoist = 0·6 t

Bending moment in I-beam for monorail hoist (assume effective length = 1 m in consideration of bracing provided)

5×1×100 = 500 cm·t

Impact 100 percent = 500 cm·t

1 000 cm·t

DL 0·6×1×100 = 60 cm·t

1 060 cm·t

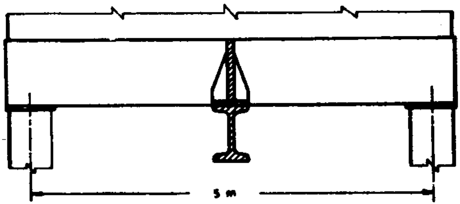
$$\text{Required Z} = \frac{1\,060 \times 1\,000}{1\,575}$$

= 674 cm^3

Use ISLB 350.

Z = 751·9 cm^3 for the hoist.

For the main girder, choose box beam for best torsional capacity. Make initial selection for bending alone with subsequent check on torsional stresses.



SECTION XX

Bending moment

$$\text{Masonry wall 1·8 t/m} = \frac{1·8 \times 5^2}{8} \times 100 = 562·5 \text{ cm·t}$$

Box beam, estimated weight

$$0·12 \text{ t/m} = \frac{0·12 \times 5^2}{8} \times 100 = 37·50 \text{ cm·t}$$

Hoist weight + load + impact

$$= 11 \text{ t, } \textit{say} = \frac{11 \times 5 \times 100}{4} = 1\,375 \text{ cm·t}$$

Total = 1 975 cm·t

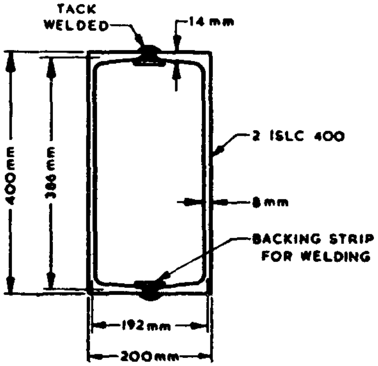
$$\text{Required Z} = \frac{1\,975 \times 1\,000}{1\,575} = 1\,255 \text{ cm}^3$$

Design Example 9 | **2 of 2**

Box Section for Combined Bending & Torsion

Two ISLC 400 channels supply a total section modulus of 1 399 cm^3 and should be satisfactory when welded up continuously to form a box as shown in section on this sheet. Plate diaphragms should be welded at each end to maintain the shape of the box and distribute both vertical and torsional loads to the reactions. The box should be made air-tight to prevent corrosion. Note that prior to welding the edges of the channels, they should receive a flame bevel cut and small steel back up strips should be tacked on the inside so that a full penetration butt weld can be made from the outside only. Since it will be impossible to weld internal diaphragms in this box, such as would be necessary on a large box section made up of 4 plate segments, there should be external stiffener plates provided as shown in the sketch on Sheet 1 adjacent to the location where the I-beam is suspended. Diaphragms did little or nothing directly to the torsional strength or rigidity aside from this incidental contribution. The shear stress in the box girder due to bending alone is now determined. This will be additive in one web to the shear stress due to torsion and will be subtractive in the other web. Thus, from the allowable shear stress of 945 kg/cm^2 there is subtracted the shear stress caused by bending. The remainder is the permissible shear stress available for torsional capacity which is calculated by Eq 41. The applied torsional moment which is distributed one-half to each end is found to be amply less than the torsional capacity. The shear stress in the web due to torsion is determined and added to that caused by bending with a resulting total of 718 kg/cm^2. If the total shear stress were closed to 945 kg/cm^2, it would be desirable to check for combined normal stress due to bending and shear near the top of the section according to ***12.2.3*** *of IS: 800-1956.*

There is no need to check the shear stress in the flange since this is considerably thicker than the web. It is to be noted that in a closed box section the maximum shear stress due to torsion is at the thinnest portion whereas in an open section the maximum shear is in the thickest portion.



Try two ISLC 400 welded toe to toe.

$Z = 699\cdot5 \times 2 = 1\,399$ cm^3

Torsion capacity:

Average shear stress due to simple bending $V = 5\cdot5 + 1\cdot92 \times 2\cdot5 = 10\cdot3$ t (including impact)

$$f_s = \frac{10\cdot3 \times 1\,000}{2 \times 40 \times 0\cdot8} = 161 \text{ kg/cm}^2$$

Available shear for torsion $= 945 - 161 = 784$ kg/cm^2

Torsion capacity:

$$M_T = 2 \times 19\cdot2 \times 38\cdot6 \times \frac{784}{1\,000} \times 0\cdot8$$

$= 930$ cm·t (*see* Eq 42)

Torsional moments applied:

$$\frac{11 \times 1\cdot2 \times 100}{2} = 660 \text{ cm·t}$$

< 930 cm·t . . . . . OK.

Maximum shear stress in web due to bending $= 161$

$$\text{Due to torsion} = \frac{660}{930} \times 784 = \underline{557}$$

718 kg/cm^2

41. DESIGN OF CRANE RUNWAY SUPPORT GIRDERS

41.1 The repeated loading and unloading of crane runway girders produces a very important maintenance problem. There is considerable literature on the subject based on mill building experience. If it is possible to minimize the differential building settlements, crane runway girders may well be continuous since the flexing of the rail over the support is reduced and the crane itself will have a smoother motion. It is outside the scope of this handbook to go into all of the maintenance problems and special devices that have been suggested for their reduction but reference may be made to ISI Handbook for Structural Engineers on Steel Work in Cranes and Hoists (under preparation) and to pages 194 to 208 of 'Planning Industrial Structures' by Dunham, a previously cited reference.

The design example presented herein is essentially one of biaxial bending under a moving load and is, therefore, similar to Design Example 4. However, since lateral load is introduced at the top of the beam because of inertial forces resulting from acceleration of the trolley and lifted load transverse to the direction of the crane runway, torsion is developed in the crane girder. To compensate and minimize the problem of combined bending and torsion, a channel is very often riveted to the top flange of an I-beam. The bottom flange of the beam is omitted as far as transverse load calculations are concerned. We then have a situation where a channel is loaded reasonably near its shear centre. The transverse loads are relatively small and the error involved in this simplification is not serious. The design example to be presented herein represents this type of solution to the crane runway girder problem and the design will be for a simple beam. The WB plus channel section would not be suitable if a continuous beam were used because of the change in sign of bending moment over the support. It is recommended that if continuous crane runway girders are employed, the possible use of the box girder construction be investigated. For the design of a box girder under combined bending and twist, reference may be made to the previous Design Example 9.

42. DESIGN EXAMPLE OF CRANE RUNWAY SUPPORT GIRDER

42.1 The illustrative design example of crane runway support girder is shown in the following five sheets (*see* Design Example 10).

Design Example 10 — Crane Runway Support Girder

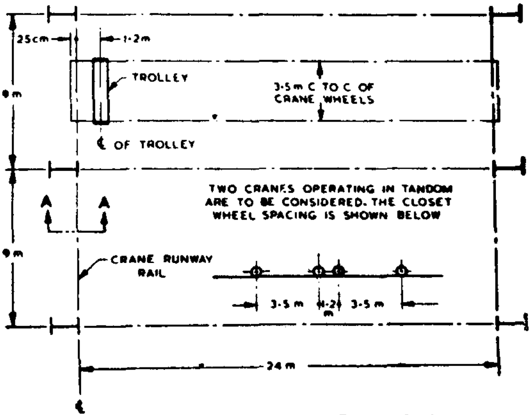
Design Example 10	1 of 5
Problem Cited	

It is assumed that in a long mill building with 24 m span cranes of 10 tonnes capacity, the runway girders will be supported as shown in Section A-A at the bottom of the sheet and a 30-kg/m rail will be bolted to the top flange by means of rail clips. It is assumed that there are two cranes and the design shall be based on their joint operation with close wheel spacing as shown in the sketch. A plan view is also provided with bends shown at 9 m centre to centre.

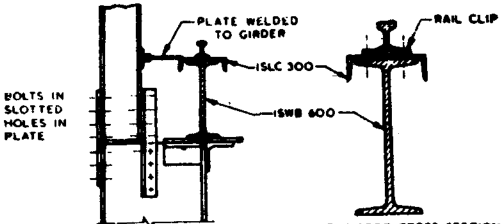
Crane Runway Support Girder Design

The problem is to design the crane runway girder to be used in an industrial building. The sketch shows a partial plan of the crane runway together with other pertinent information. The following are design conditions and requirements:

1) The crane girder is a simple beam to be seated on each end, and
2) The girder section is to be built up from a wide flange shape and a channel using 20-mm rivets.



PARTIAL PLAN OF CRANE BAY



SECTION AA

CRANE GIRDER CROSS-SECTION

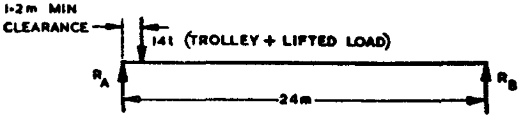
Design computations for maximum bending moment and shear are self-explanatory. It is to be noted that the loads for maximum moment are positioned as shown in the sketch here. The trolley is assumed to be as close as possible to one support of the 24-m span crane for maximum load on the crane runway girder.

Design Example 10

Maximum Bending Moment & Shear Calculations

2 of 5

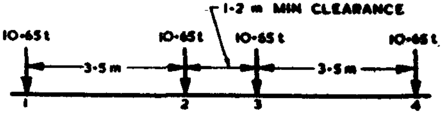
Crane capacity = 10 t Weight of crane including trolley = 20 t
Weight of trolley alone = 4 t
Determine maximum bending moment. Total crane girder weight = 16 t



POSITION OF CRANE TROLLEY FOR MAXIMUM CRANE RUNWAY GIRDER LOAD

ΣM about B and solving for R_A: $R_A = \frac{1}{24}\left(14\times 22{\cdot}8+16\times\frac{24{\cdot}0}{2}\right) = 21{\cdot}3$ t

This load distributed to two wheels on each of 2 cranes is shown as:

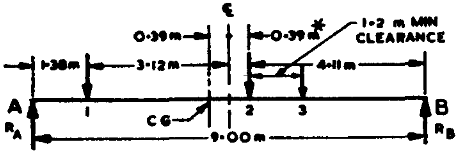


For maximum moment in 9-m span, 3 wheels (31·95 t) will be on span. Place centre of gravity and one wheel equidistant from centre line.

To locate centre of gravity—ΣM about (1) of (1), (2) and (3) should be divided by the total of the three wheel loads to give:

$$\frac{10{\cdot}65\times 3{\cdot}5+10{\cdot}65\times(3{\cdot}5+1{\cdot}2)}{3\times 10{\cdot}65} = 2{\cdot}73 \text{ m from (1)}$$

$$\therefore \text{ Distance of centre of gravity from (2) is } \frac{3{\cdot}5-2{\cdot}73}{2} = 0{\cdot}39 \text{ m*}$$



$$R_A = \frac{3\times 10{\cdot}65\times 4{\cdot}89}{9} = 17{\cdot}4 \text{ t}$$

At (2), $M_{\max} = 17{\cdot}4\times 4{\cdot}89-10{\cdot}65\times 3{\cdot}5$	=	47·7 m·t
Plus 25 percent impact	=	11·9 m·t
Live load moment	=	59·6 m·t

---

*See sketch.

Design Example 10

Preliminary Design

3 of 5

In this sheet, after calculating the bending moment due to horizontal load, an approximation of the required section modulus is obtained by dividing the bending moment due to vertical load by the allowable stress in the tension flange. The section modulus for tensile stress will be changed but little by the addition of the channel to the top flange since the distance from the neutral axis will be increased while the moment of inertia is likewise increased. No deduction is made for rivets in the top flange but full deduction for the drilled holes for the 20-mm rail clamp bolts shall be made in determining the stress in the compression flange. The compressive stress in the top flange due to vertical loads is determined and the compressive stress for lateral loads is calculated with the assumption that all lateral loads are taken by the channel and top flange of the wide flange shape.

DL Rail 30 kg/m = 0·03 t/m
Assume beam weight @ 200 kg/m = 0·20 t/m
0·23 t/m

DL Moment $= \dfrac{0{\cdot}23 \times (9)^2}{8} = 2{\cdot}33$ m·t, Total vertical moment = 61·93 m·t

Horizontal load = 10 percent of 14 t (*see* **6.2** of IS: 875-1957) = 1·4 t

Per wheel $= \dfrac{1{\cdot}4}{4}$ Horizontal moment $= \dfrac{0{\cdot}35}{10{\cdot}65} \times 47{\cdot}7 \times 100$

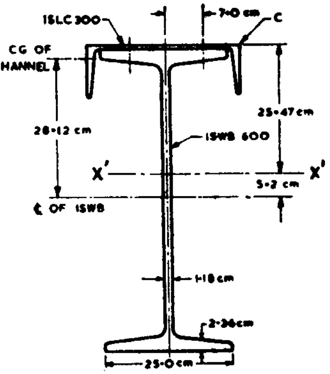
$= 0{\cdot}35$ t $= 157$ cm·t (by proportion)

Approximate required Z (due to vertical bending moment only)

$$= \frac{61{\cdot}93 \times 1\,000 \times 100}{1\,575}$$

$$= 3\,930 \text{ cm}^3$$

Try ISWB 600, 145·1 kg and ISLC 300, 33·1 kg



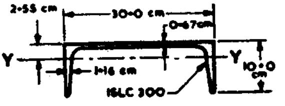
	AREA, A	y	A_y	$\bar{y}$	$A(\bar{y})^2$	I_x
ISWB	184·86	0	—	5·2*	4 990	115 626·6
ISLC	42·11	28·12	1 180	22·92*	22 100	346·0
	226·97		1 180		27 090	115 972·6

$I_{x'x'}$ (gross) $= 27\,090 + 115\,972{\cdot}6$
$= 143\,062{\cdot}6$

I_{xx} (net) $143\,062{\cdot}6 = 2 \times 2{\cdot}1 \times 3{\cdot}03 \times (23{\cdot}95)^2$ (for 20-mm rivets)
$= 135\,800$ cm^4

$$Z_{xx} = \frac{I_{\text{net}}}{c} = \frac{135\,812{\cdot}6}{25{\cdot}47} = 5\,320 \text{ cm}^3$$

$$f_{bxx} = \frac{61{\cdot}93 \times 100 \times 1\,000}{5\,320} = 1\,162 \text{ kg/cm}^2$$



For top flange and channel only:

Gross $I_{yy} = \dfrac{5\,298{\cdot}3}{2} + 6\,047{\cdot}9 = 8\,697{\cdot}1$ cm^4

Net $I_{yy} = 8\,697{\cdot}1 - 2 \times 2{\cdot}1 \times 3{\cdot}0 \times 7^2$
$= 8\,080$ cm^4

---

*$\bar{y}_{\text{WB}} = \dfrac{1\,180}{226{\cdot}97} = 5{\cdot}2$ cm, $\bar{y}_{\text{C}} = 30{\cdot}67 - 5{\cdot}2 - 2{\cdot}55 = 22{\cdot}92$ cm.

Design Example 10	**4 of 5**
Check of Preliminary Design	

Since section is unsymmetrical about the X-X axis, the permissible stress under vertical loads shall be determined by use of **E-3.1** *of Appendix E in IS: 800-1956 or Table III in this handbook. It is found, however, that the allowable stress by the tables in Appendix E of IS: 800-1956 is above the maximum permissible stress of 1 500 kg/cm², and the latter, therefore, governs. The problem then is simply one of combining the two stresses. Had the allowable stresses been different, the interaction formula procedure suggested previously in Design Example 4 would have been recommended. It is now necessary to check the tensile stress due to vertical bending alone and this is found to be 1 520 kg/cm². The allowable stress for a rolled section of 1 575 kg/cm² is assumed to govern on the tension side, so the selection is satisfactory.*

$$Z_{yy} = \frac{8\,080}{15} \simeq 540 \text{ cm}^3$$

$$f_{byy} = \frac{157 \times 1\,000}{540} = 290{\cdot}8 \textit{ or say } 300 \text{ kg/cm}^2$$

To determine final F_b,

use Appendix E of IS: 800-1956 or Table III of this handbook, which should not exceed 1 500 kg/cm² nor $C = A + k_2 B$

M for the purpose of Table XX of IS: 800-1956 $= \dfrac{8\,697{\cdot}1}{11\,346{\cdot}2} = 0{\cdot}766$

$k_2 = 0{\cdot}266$ (*see* Table XX of IS: 800-1956)

r_{yy} for the whole section $= \sqrt{\dfrac{11\,346{\cdot}2}{42{\cdot}11 + 184{\cdot}86}} = 7{\cdot}05$ cm

$l/r = \dfrac{900}{7{\cdot}05} = 128$, Flange area $= 42{\cdot}11 + 25 \times 2{\cdot}36 = 101{\cdot}11$ cm²

$t_e = \dfrac{101{\cdot}11}{30} = 3{\cdot}37$ cm (for $N = 1$, $k_1 = 1$), $d/t_e = \dfrac{60{\cdot}67}{3{\cdot}37} = 18$

Using Table XXI of IS: 800-1956 or more conveniently Table IV,
for $l/r = 128$, $d/t_e = 18$, $C = A + k_2 B > 1\,575$ kg/cm²
∴ 1 500 kg/cm² controls design (*see* **9.2** of IS: 800-1956)

At point C, **9.5.1** of IS: 800-1956 provides $\sum \dfrac{f_b}{F_b} \nless 1$

$$\therefore \quad \frac{1\,162 + 300}{1\,500} = \frac{1\,462}{1\,500} = 0{\cdot}97 < 1 \text{ ..... OK.}$$

Check tensile stress (based on gross I, vertical load only)

$$Z_{xx} \text{ (tension)} = \frac{143\,062{\cdot}6}{35{\cdot}2} = 4\,070 \text{ cm}^3$$

f_b (tension) $= \dfrac{61{\cdot}93 \times 10^5}{4\,070} = 1\,520 < 1\,575$ kg/cm² (Allowable stress for rolled section ..... OK)

Check weight assumed.

ISWB	=	145·1 kg/m
Channel	=	33·1
Rail	=	30·0
Fittings	=	7·5
		215·7 kg/m

215·7 kg/m = 0·22 t/m approximately equal to 0·23 t/m assumed in the design ..... OK.

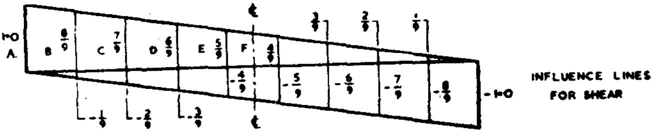
Design Example 10 | **5 of 5**

Rivets

In order to determine the rivet pitch, the maximum shear at various points along the girder shall be calculated. It is convenient to determine the maximum shear at metre intervals by use of the influence lines as shown on this Sheet. The summation of influence line co-efficients for maximum shear at the successive points between the centre line and one end are tabulated and since maximum spacing conditions are apt to govern rather than stress requirements, the 16-mm rivets are tried out in the design. 20-mm rivets could be used if preferred. It turns out, because of the light loads, which were purposely selected, that the rivet stress requirements do not govern at any point along the girder. The controlling factors are the code requirements for a maximum spacing at the ends of six times the rivet diameter and a maximum in line pitch not to exceed a pitch thickness ratio greater than 16. Since further aspects of this design duplicate procedures that were previously presented, they are omitted here, but final note is emphasized that the holes should all be drilled after assembly to minimize the possibility of fatigue cracks developing under repeated load.

Determine rivet pitch

Maximum shear required calculate at one-metre intervals and at ℄



Influence coefficients for maximum shear at:

A: $(9+5{\cdot}5+4{\cdot}3+0{\cdot}8)/9 = 19{\cdot}6/9$

B: $(8+4{\cdot}5+3{\cdot}3)/9 = 15{\cdot}8/9$

C: $(7+3{\cdot}5+2{\cdot}3)/9 = 12{\cdot}8/9$

D: $(6+2{\cdot}5+1{\cdot}3)/9 = 9{\cdot}8/9$

E: $(5+1{\cdot}5+0{\cdot}3)/9 = 6{\cdot}8/9$

F: $(4{\cdot}5+1{\cdot}0)/9 = 5{\cdot}5/9$

(*Power driven shop*) *rivets:*

Shear 2-16 mm rivets SS $= 2 \times 17^2 \times \frac{\pi}{4} \times 1\,025 = 4{\cdot}64$ t (*see* Table IV of IS: 800-1956)

Bearing $= 2 \times 17 \times 0{\cdot}67 \times 2\,360 = 5{\cdot}38$ t

Single shear controls, Rivet pitch $p = \frac{RI}{VQ}$, $Q = 42{\cdot}11 \times 22{\cdot}92 = 964$ cm^3 (gross)

$$\frac{RI}{Q} = \frac{4{\cdot}64 \times 143\,062{\cdot}6}{964} = 690$$

Location	Shear			Pitch
	DL	LL	DL+LL	
A	1·04	29*	30·04	20·0 cm or more (*see* **25.2.2.1** of IS: 800-1956). But other considerations as seen further do not permit pitch to be as high as 20 cm.
B	0·81	23·4	24·21	
C	0·58	18·9	19·48	
D	0·35	14·5	14·85	
E	0·12	10·1	10·22	
F	0·0	8·1	8·1	

Shear stress in web near compression flange $Q = 964 + 25 \times 2{\cdot}36 \times 23{\cdot}62$

$= 2\,359$ cm^3

$$V' = \frac{VQ}{It_w}$$

$$= \frac{30{\cdot}04 \times 2\,359 \times 1\,000}{143\,062{\cdot}6 \times 1{\cdot}18}$$

$= 419$ kg/cm^2

Try $6 \times 2{\cdot}0 = 12$-mm pitch (for 20 mm diameter rivet *see* **25.2.2.4** of IS: 800-1956). Maximum in line pitch governed by $P/t_{\text{web}} = 16$ (for intermediate length) $P = 16 \times 0{\cdot}67$† $= 10{\cdot}7$ cm < 12 cm. Hence use 10-cm pitch throughout the length 21-mm holes for rivets to be drilled after assembly by clamps.

For bearing, local crippling, etc, *see* Design Example 1.

---

*Live load = 10·65 t+25 percent impact = 13·31 t; 13·31 × 19·6/9 = 29 t.

†Web thickness of ISLC 300 = 67 mm.

SECTION VI

PERFORATED AND OPEN WEB BEAMS

43. OPEN WEB JOISTS AND BEAMS

43.1 Open Web Joists — There has been phenomenal growth of the use of open web bar joists in overseas countries. These joists, as discussed in ISI Handbook for Structural Engineers on Functions of Good Design in Steel Economy (under preparation) are not so much designed as they are developed and tested by various individual companies with the aim, however, of complying with certain standardized overall dimensions and load capacities for standard span lengths.

With changes in the figure numbers, the following is quoted verbatim from an article by Henry J. Stetina* regarding the detailed application of open web joists in building constructions:

> ' Why are open web joists popular? The primary consideration is usually that of costs. This floor system supported on a steel frame and fireproofed according to modern practices is regarded by many engineers and architects to be the most economical construction of all structural systems.
>
> Many factors contribute to this economy. Joists have been developed to a high degree of standardization and are produced by many manufacturers. The cross-sections of the popular types are shown in Fig. 27. All possess the common characteristic of being interchangeable for any given depth and span. That is so because all joists conform to a standard loading table. They vary in depth from eight to sixteen inches (20 to 40 cm) and are identified by a standard nomenclature.
>
> Joists are quickly installed, braced and welded, as shown in Fig. 28. A cover, such as a metal lath, illustrated in Fig. 29, is then connected to the top flanges, and the lath serves as a form. Besides lath, two other products are in common use for this purpose: a paper backed wire mesh and a light gauge corrugated steel sheet. In the case of lath, it is customary to supply some additional reinforcement in the form of wire mesh as shown in Fig. 30. All of these concrete forms are sufficiently sturdy to support workmen and light construction loads (*see* Fig. 31).

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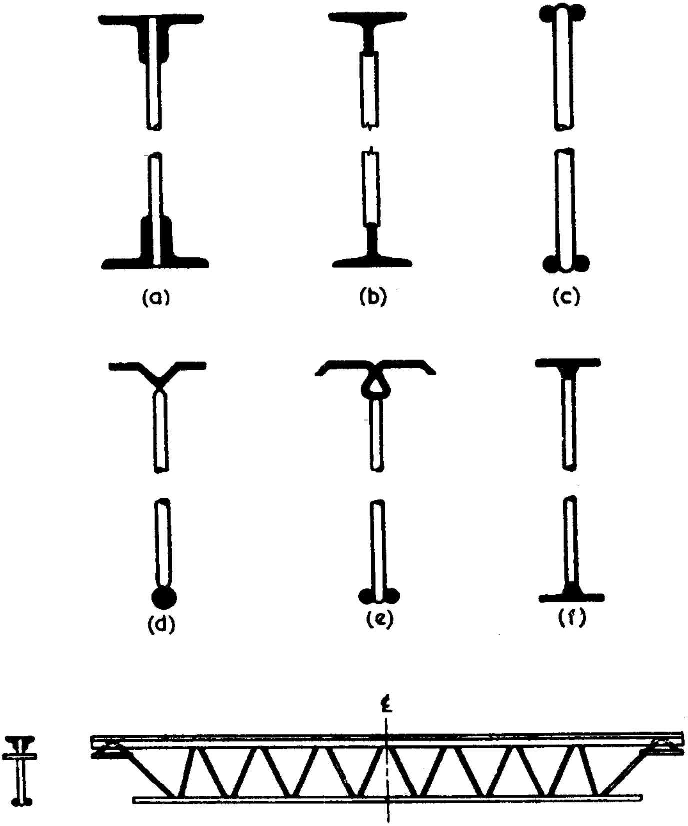
*NAS-NRC No. 441. BUILDING RESEARCH INSTITUTE CONFERENCE PROCEEDINGS ON ' FLOORS AND CEILINGS '. National Academy of Sciences, 2101 Constitution Ave., Washington 25, DC.



(g) DETAILS OF OPEN-WEB STEEL JOIST

FIG. 27 POPULAR TYPES OF OPEN WEB JOISTS

Ribbed metal lath or gypsum lath is fastened to the bottom flange and plaster applied. In the case of a double ceiling this contact ceiling may serve only as fire protection, therefore, the plaster finish coat is omitted. In the case of single ceilings it serves both as fire protection and finish. Fire resistance is readily obtained; two, three or four hours depending on thickness and composition of the plaster and the base material.

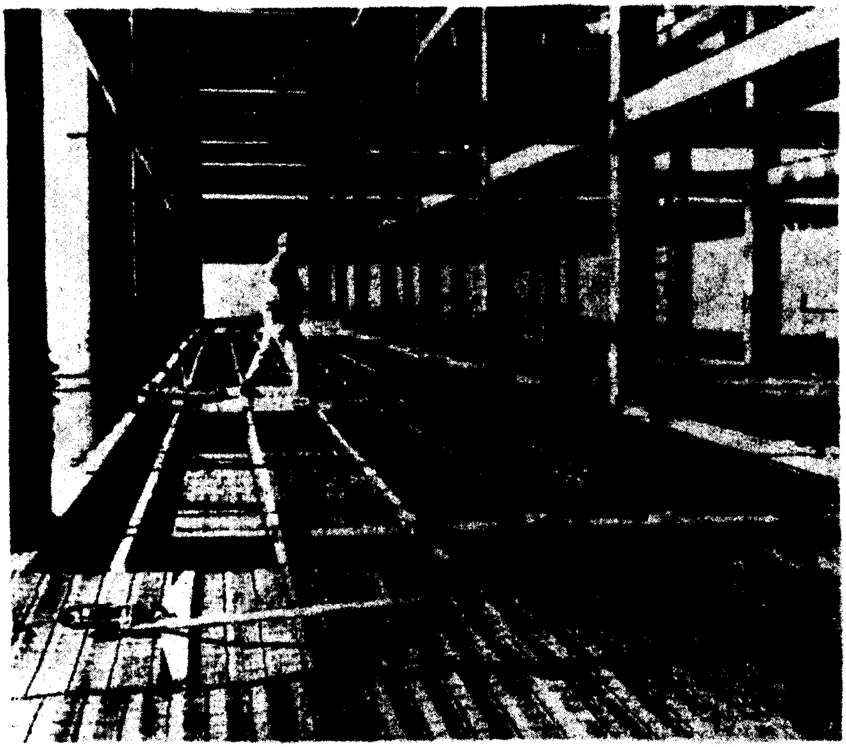


FIG. 28 OPEN WEB FLOOR JOIST INSTALLED AND BRACED

(Photograph by courtesy of American Institute of Steel Construction)

Still another advantage that some builders stress is that the open webs speed up the work of the following trades. Electrical work and pipes may be more readily installed.'

Figure 32 shows a pleasing use in school building construction of open web joists of the type indicated in cross-section in Fig. 27(a). It will be noted that these are made of four angles with a zig-zag bend reinforcing bar welded between.

FIG. 29 METAL LATH COVER FOR PERMANENT FORMS

(Photograph by courtesy of American Institute of Steel Construction)

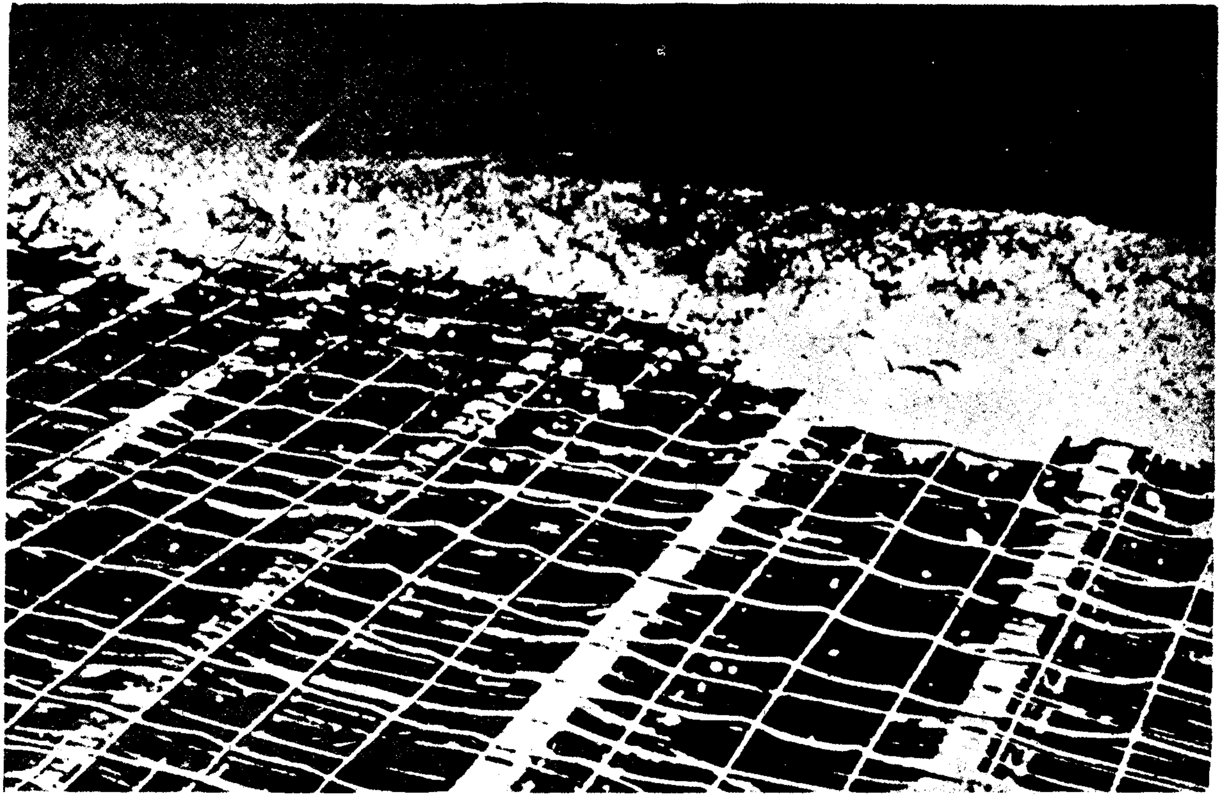


FIG. 30 ADDITIONAL WIRE MESH REINFORCEMENT
(Photograph by courtesy of American Institute of Steel Construction)

FIG. 31 PLACING CONCRETE FLOOR SLAB ABOVE OPEN WEB JOISTS

(Photograph by courtesy of American Institute of Steel Construction)

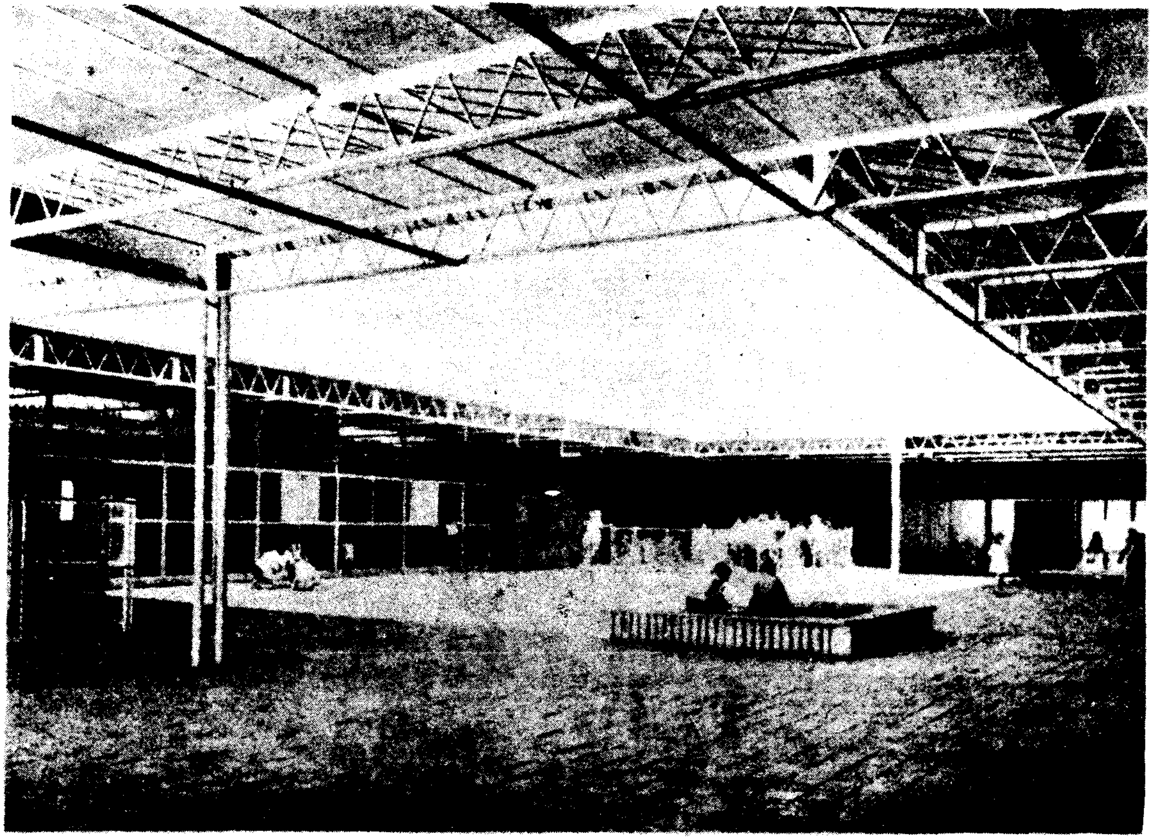


FIG. 32 OPEN WEB JOIST ROOF PURLINS FOR ELEMENTARY SCHOOL
(Photograph by courtesy of American Institute of Steel Construction)

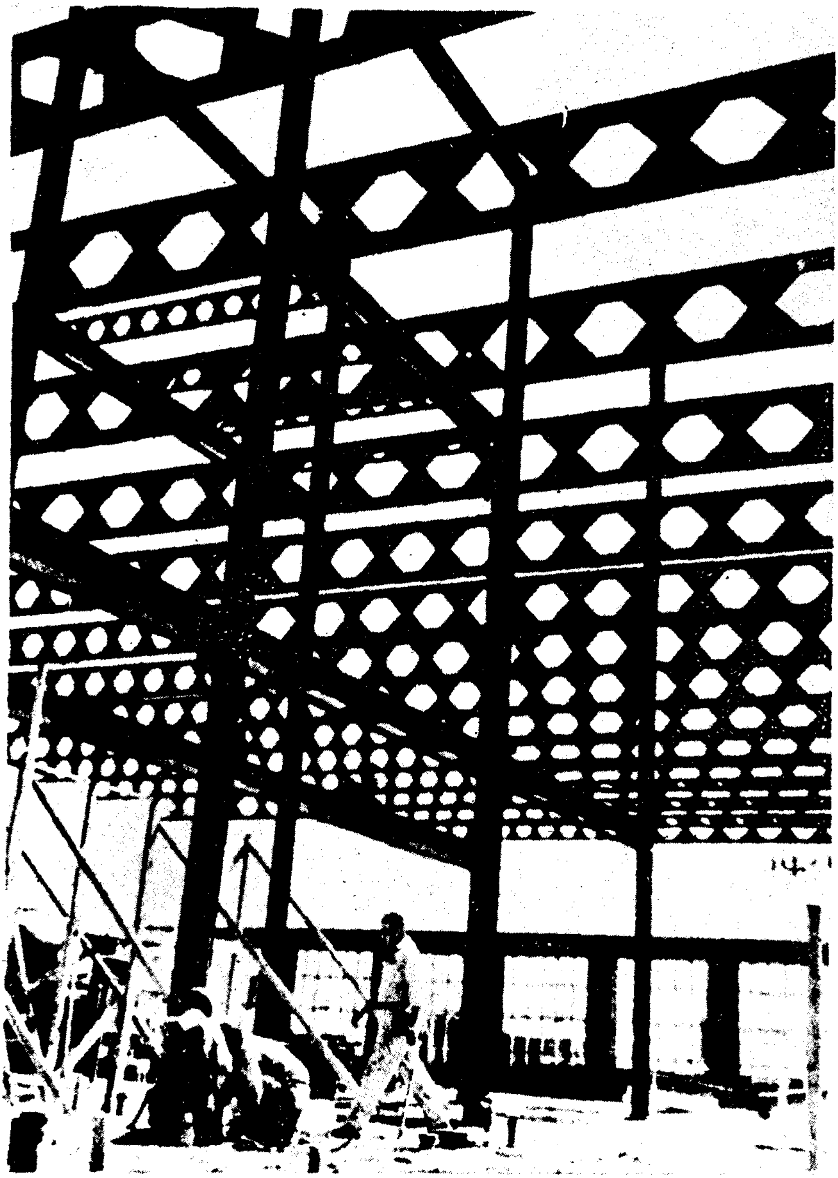


FIG. 33 PERFORATED WEB BEAM CONSTRUCTION
(Photograph by courtesy of American Institute of Steel Construction)

44. DESIGN OF BEAMS WITH PERFORATED WEBS

44.1 The beam with perforated web is another development which is similar to that of the open web joist. It is, in fact, an open web joist in itself but is made up out of a single rolled wide flange or I-beam shape by flame cutting the web along a zig-zag line and rewelding the two cut portions to give a beam that normally is 50 percent deeper than the original section from which it was obtained. An example of this type of construction in practice is shown in Fig. 33. A perforated web beam is suited to the same type of light uniform load and long span application as the standardized types of open web joists. The perforated web beam has not become nearly as popular in the United States as the open web joist. However, recent tests by Toprac, *et al** of a number of perforated web beams substantiate the procedures that will herein be presented as Design Example 11. In comparison with the open web joist, the perforated web beam requires somewhat more welding and would appear to be more wasteful of steel. However, weight comparisons for similar capacities in a number of spans seem to indicate little to choose from between the two as far as total steel requirement is concerned.

45. DESIGN EXAMPLE OF PERFORATED WEB BEAM

45.1 The illustrative design example of perforated web beam is shown in the following seven sheets (*see* Design Example 11).

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*ALTIFILLISCH, M., COOKE, B.R. AND TOPRAC, A.A. An Investigation of Open Web Expanded Beams. *Welding Journal Research Supplement.* Vol 22, No. 2, p. 77-88 (February 1957).

Design Example 11 — Perforated Web Beam

An open web roof beam with perforated web is designed for a clear span of 18 metres. The estimated weight of roofing, ceiling and purlins is 75 kg/m² and the live load is 150 kg/m. The spacing of the roof beams with perforated webs will be determined by capacity. By way of comparison, Design Example in the next section will be illustrative of the application of a tapered depth beam to the same load conditions. The cutting plan is shown on Sheet 1. Good practice maintains the depth of the stem of the tee section at the minimum beam cross-section to no less than one-quarter of the original beam depth as shown in this layout. When this is done, the over-all perforated web beam depth will be 50 percent greater than the original rolled section from which it was formed. Local bending stress will be checked in the tee section; hence the need to calculate sectional properties.

Design Example 11	**1 of 7**
Problem Cited	

Perforated Web Beam Design

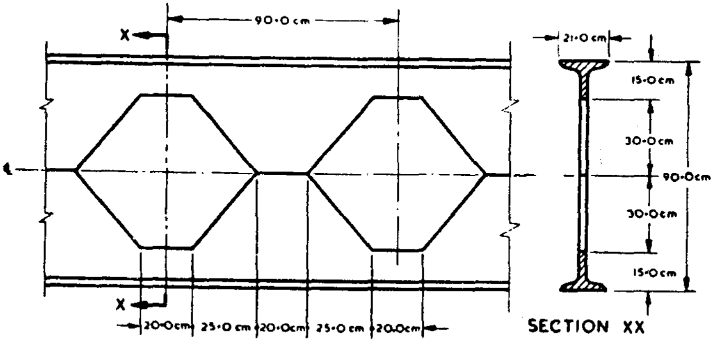
Design an open web roof beam with perforated web for clear span of 18 m.

Load assumptions:

Dead load including weight of roofing sheets. ceiling, and purlins = 75 kg/m²
Live load = 170 kg/m²
245 kg/m²

Spacing of perforated web roof beams will be determined by capacity.

Use ISMB 600, 122·6 kg



Calculate property of tee at open throat:

Area of flange $21{\cdot}0 \times 2{\cdot}08 = 43{\cdot}7\ \text{cm}^2$

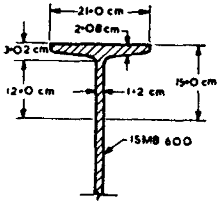
Web $12{\cdot}92 \times 1{\cdot}2 = 15{\cdot}5\ \text{cm}^2$

$59{\cdot}2\ \text{cm}^2$

Position of NA, by taking moments above top line

$$y = \frac{43{\cdot}7 \times 1{\cdot}04 + 15{\cdot}5 \times 8{\cdot}54}{59{\cdot}2}$$

$$= 3{\cdot}0\ \text{cm}$$



Design Example II

2 of 7

Preliminary Selection of Spacing of Beams

At the centre, the moment is maximum and the local bending stress in the tee section will be very small. It is assumed that the moment capacity at the centre is the product of the total tensile or compressive resultant and the distance between the tee centroids. The resultant stress in the tee is determined at an average stress of 1 500 kg/cm². The capacity in terms of total load on a simple beam is then determined and from this is subtracted the dead weight of the beam itself. On the basis of net capacity, the spacing of perforated web beams is chosen at 7 metres. The actual moment capacity is now calculated on the basis of the assumed spacing and the average shear stress in the end open section is found to be 458 kg/cm². The horizontal shear stress is now checked at location 1 by static equilibrium of the free body diagram shown at the centre of Sheet 3. The local combined stress in the tee section due to direct force plus bending is now investigated. The combination at location B in the sketch at the bottom of Sheet 3 is critical.

Calculate I of tee about NA:

$$\left.\begin{aligned} \frac{43\cdot7\times(2\cdot02)^2}{12} &= 16 \\ 43\cdot7\times(2\cdot96)^2 &= 182 \end{aligned}\right\} \text{Flange}$$

$$\left.\begin{aligned} \frac{15\cdot5\times(12\cdot92)^2}{12} &= 216 \\ 15\cdot5\times(5\cdot5)^2 &= \underline{463} \end{aligned}\right\} \text{Web}$$

$$877 \text{ cm}^4$$

Moment capacity based on average stress of 1 500 kg/cm². (Purlins welded to top flange plus cross bracing in plane of roof are provided for lateral support.)

$$M = 59\cdot2\times84\times\frac{1\ 500}{1\ 000} = 7\ 450 \text{ cm·t}$$

(84 cm = distance between NA of top and bottom tees)

Span = 18 m

If total distributed load for simple beam is W, then

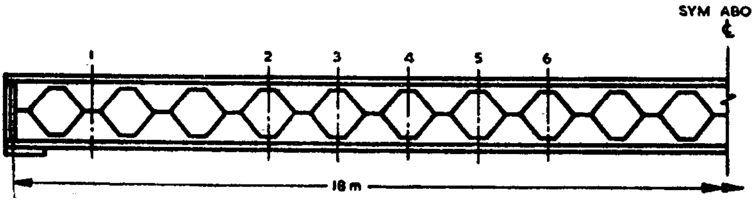
$$7\ 450 = \frac{W\times18\times100}{8}; \quad W = \frac{7\ 450\times8}{1\ 800} = 33\cdot2 \text{ t}$$

$$\text{Less weight of beam} = \frac{18\times122\cdot6}{1\ 000} = 2\cdot2 \text{ t; Net capacity} = 33\cdot2-2\cdot2 = 31\cdot0 \text{ t}$$

Let S = span c/c perforated web beams

$$\text{Load} = \frac{{}^*245\times S\times18}{1\ 000} = 31\cdot0 \text{ t} \quad \therefore \quad S = \frac{31\cdot0\times1\ 000}{245\times18} = \textit{say } 7 \text{ m}$$

Choose a spacing of 7 m c/c.



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*Load, *see* Sheet 1.

Design Example II | **3 of 7**

Check of Combined Stresses

Load per metre $= 245 \times 7 = 1\,715$

Beam weight $= \underline{122{\cdot}6}$

$1\,837{\cdot}6$ kg

$$\text{Maximum end shear} = \frac{1\,837{\cdot}6 \times 18}{2 \times 1\,000} = 16{\cdot}5 \text{ t}$$

$$\text{Maximum BM at centre} = \frac{16{\cdot}5 \times 18}{4} \times 100 = 7\,425 \text{ cm·t}$$ (less than 7 450 cm·t capacity, Sheet 2) . . . . . OK.

Average sheer at ends:

$$f_s = \frac{16{\cdot}5 \times 1\,000}{1{\cdot}2 \times 30} = 458 \text{ kg/cm}^2 < 945 \text{ kg/cm}^2 \text{ . . . . . OK.}$$

Check horizontal shear at 1 (Refer to sketch on Sheet 2)

Total shear at $A = 16{\cdot}5 \times 0{\cdot}9 = 14{\cdot}85$ t (*see* sketch here)

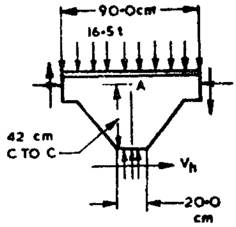
Assume compression normal force resultants pass through A and take moments about A (centroid)

$$42\, V_h = \frac{14{\cdot}85}{2} \times 90$$

$$\therefore \quad V_h = 15{\cdot}95 \text{ t}$$

$$\text{Shear stress } f_s = \frac{15{\cdot}95 \times 1\,000}{1{\cdot}2 \times 20{\cdot}0}$$

$$= 666 \text{ kg/cm}^2 < 945 \text{ kg/cm}^2 \text{ . . . . OK.}$$



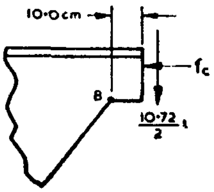
Maximum combined local bending and direct stress in tee segment near 1/4 point (Try locations 2, 3, 4 and 5 shown in sketch on Sheet 2)

Location 2:

Shear $= 16{\cdot}5 \times 0{\cdot}65 = 10{\cdot}72$ t

Moment $= 7\,425\,(1 - 0{\cdot}65^2)$ (Moment diagram being a parabola, moment gets reduced as the square of the distance from centre.)

$= 4\,300$ cm·t



(Halved because of two flanges at top and bottom.)

$$\text{Direct stress } f_c = \frac{4\,300}{59{\cdot}2 \times 83{\cdot}84}$$ (area of flange = 59·2 cm²)

$$= 867 \text{ kg/cm}^2$$ (distance between top and bottom centre of gravity = 83·84)

Design Example II

Check of Combined Stresses

4 of 7

The sample computation of local bending stress due to shear is given at the top of this sheet and the combined stress at each location is tabulated. The surprising uniformity results from the fact that as the average direct stress in the tee increases towards the centre of the span, the local bending stress due to shear decreases. The combined stress has a maximum value of 1 711 kg/cm². Although not covered by the code, it seems reasonable to permit the maximum combined stress to be 1 575 kg/cm² because of its localized nature. The spacing is revised to 6·3 metres to bring the stress down to 1 575 kg/cm².

Although d/t_w is less than 85 in the solid portion, the open nature of the web indicates the desirability of adding an end stiffener. A tee section split by flame cutting a wide flange shape is welded in position as shown at the bottom of Sheet 5.

Bending stress at B due to shear,

$$f_{sb} = \frac{10{\cdot}72 \times 10{\cdot}0 \times 12}{2 \times 877^{*}} \times 1\,000$$

$$= 730 \text{ kg/cm}^2$$

Combined stress at B $= f_c + f_{sb} = 867 + 730 = 1\,597$ kg/cm²

At 3, Shear $= 16{\cdot}5 \times 0{\cdot}55 = 9{\cdot}08$ t

Moment $= 7\,425\,(1 - 0{\cdot}55^2) = 5\,180$ cm·t

Direct stress at 3, f_c $= \dfrac{5\,180 \times 1\,000}{59{\cdot}2 \times 83{\cdot}84} = 1\,044$ kg/cm²

Bending stress due to shear at B_3

$$f_{sb} = \frac{9{\cdot}08 \times 10{\cdot}0 \times 12}{2 \times 877} \times 1\,000$$

$$= 617 \text{ kg/cm}^2$$

Combined shear at B $= f_c + f_{sb} = 1\,044 + 617 = 1\,661$ kg/cm²

At 4, Shear $= 16{\cdot}5 \times 0{\cdot}45 = 7{\cdot}43$ t

Moment $= 7\,425\,(1 - 0{\cdot}45^2) = 5\,921$ cm·t

Direct stress at 4, f_c $= \dfrac{5\,921 \times 1\,000}{59{\cdot}2 \times 83{\cdot}84} = 1\,198$ kg/cm²

Bending stress at B due to shear

$$f_{sb} = \frac{7{\cdot}43 \times 10{\cdot}0 \times 12}{2 \times 877} \times 1\,000$$

$$= 505 \text{ kg/cm}^2$$

Combined stress at 4 $= f_c + f_{sb} = 1\,198 + 505 = 1\,703$ kg/cm²

At 5, Shear $= 16{\cdot}5 \times 0{\cdot}35 = 5{\cdot}77$

Moment $= 7\,425\,(1 - 0{\cdot}35^2) = 6\,515$ cm·t

Direct stress at 5, f_c $= \dfrac{6\,515 \times 1\,000}{59{\cdot}2 \times 83{\cdot}84} = 1\,318$ kg/cm²

Bending stress at B_5

$$f_{sb} = \frac{5{\cdot}77 \times 10{\cdot}0 \times 12}{2 \times 877} \times 1\,000$$

$$= 393 \text{ kg/cm}^2$$

Combined stress at 5 $= f_c + f_{sb} = 1\,318 + 393 = 1\,711$ kg/cm²

---

*Moment of Inertia I, *see* Sheet 2.

Design Example 11 | **5 of 7**

Revision of Spacing of Beams due to *Max* Combined Stress Exceeding Permissible Limit

At 6, Shear $= 16{\cdot}5 \times 0{\cdot}25$

$= 4{\cdot}125$ t

Moment $= 7\,425\ (1-0{\cdot}25^2)$

$= 6\,975$ cm·t

Direct stress at 6, $f_c = \dfrac{6\,975 \times 1\,000}{59{\cdot}2 \times 83{\cdot}84}$

$= 1\,404$ kg/cm²

Bending stress at B_6 due to shear

$$f_{sb} = \frac{4{\cdot}125 \times 10 \times 12 \times 1\,000}{2 \times 877}$$

$= 281{\cdot}0$ kg/cm²

Combined stress $= f_c + f_{sb} = 1\,404 + 281 = 1\,685$ kg/cm²

Thus maximum combined stress is at section 5 = 1 711 kg/cm²

Hence the capacity has to be correspondingly reduced.

Reduced moment capacity $= \dfrac{1\,575 \times 7\,425}{1\,711}$

$= 6\,850$ cm·t

Load capacity $= \dfrac{6\,850 \times 8}{1\,800}$

$= 30{\cdot}40$ t

Less weight of beam $= 2{\cdot}24$ t

$28{\cdot}16$ t

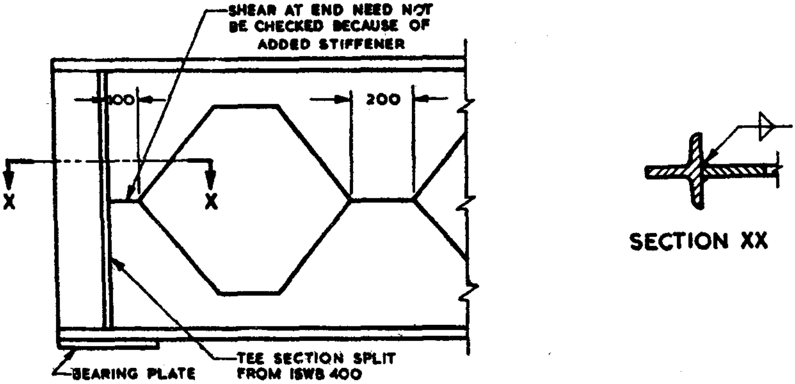
Revised spacing is $S = \dfrac{28{\cdot}16 \times 1\,000}{245 \times 18}$

$= 6{\cdot}37$ m

Choose 6·3-m spacing.

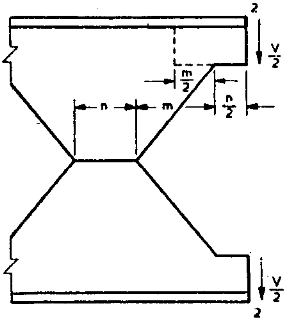
End support detail

Provide end stiffeners over supports even though $d/t < 85$ as perforated web beams are weak in web bending about longitudinal axis.



Design Example II	6 of 7
Deflections	

Previously cited tests (see **44.1** *) have shown that, for this type of perforated web beams of the proportions used in this design, the bending deflection of the beam may be approximated on the basis of the average moment of inertia of the solid and perforated sections. The deflection as a simple beam is found to be 5·2 cm.*



Since tests and more accurate studies of the local deflection show that the local effect is a relatively small one, it is possible to estimate roughly the magnitude by calculating the local bending deflection due to shear as follows:

The deflection of a tapered cantilever, length $m+\frac{n}{2}$ *may be approximated by a constant (minimum section) cantilever (shown by dashes in the sketch) of length* $\frac{m+n}{2}$

$$\text{Deflection of cantilever} = \frac{\left(\frac{V}{2}\right)\left(\frac{m+n}{2}\right)^{3}}{3EI_{T}} \quad \dots\dots\dots\dots\dots\dots (43)$$

In each perforation panel, deflection will be doubled, hence, per panel

$$\delta_v = \frac{V(m+n)^3}{24\ EI_T} \quad \dots\dots\dots\dots\dots\dots (44)$$

Let p = *number of perforation panels in a half span.*

$$\text{Total } \delta_v = \frac{V_{\text{avg}}\ p\ (m+n)^3}{24\ EI_T} \quad \dots\dots\dots\dots\dots\dots (45)$$

Using Eq 45, the deflection due to localized tee section bending is found to be 0·16 cm. The total of centre deflection is now estimated at 5·36 cm and it would probably be desirable to give a perforated web beam of this type a camber of at least 3 cm to avoid unsightly sag.

Check deflection:

Base beam bending deflection on average I of solid and open sections above the centre line, plus estimate of local bending deflection.

Perforated section:

$$I = 2\times 59{\cdot}2\times 42^2 = 208\,000$$
$$+2\times 877 \text{ (see Sheet 2)} = \underline{1\,754}$$
$$209\,754 \text{ cm}^4$$

Solid section:

I of perforated Beam $= 209\,754$

$+\dfrac{1{\cdot}2\times60^3}{12} = 21\,600$

$231\,354$ cm^4

Average $I = 220\,554$

Deflection due to general bending $= \dfrac{5}{384}\times\dfrac{{}^{*}30{\cdot}4\times1\,800^3\times1\,000}{2\,050\,000\times220\,554}$

$= 52$ cm

V at ends $= \dfrac{30{\cdot}4}{2}$ t

V at centre $= 0$

$\therefore\ V_{avg} = \left[\dfrac{30{\cdot}4}{2}+0\right]\dfrac{1}{2}$

$= 7{\cdot}6$ t

$p = 10\,(m+n) = 450$ mm

$E = 2\,050\,000$ kg/cm^2

$I_t = 877$ cm^4

Deflection due to localized tee section bending from Eq 43

$$\therefore\ \delta_v = \frac{7{\cdot}6\times10(45)^3\times1\,000}{24\times2\,050\,000\times877}$$

$= 0{\cdot}16$ cm

Total deflection at centre $= 5{\cdot}36$ cm

Camber 3·0 cm (Dead load+part of Live load).

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[*] Load capacity, *see* Sheet 5.

SECTION VII

TAPERED BEAMS

46. INTRODUCTION

46.1 A recent pamphlet entitled '*Welded Tapered Girder*' distributed in 1956 by the American Institute of Steel Construction has focussed attention on the growing use of roof girders of tapered depth. The introduction to this pamphlet, reproduced below with change in figure number, describes this type of construction and its advantages in a way that cannot be improved upon:

> 'In recent years tapered girders fabricated by welding plates together have become increasingly popular in the framing of roofs over comparatively large areas where it is desirable to either minimize the number of interior columns or to eliminate them altogether, dependent upon the width of the building. The two halves of the web are produced from wide plates, with little or no waste of material, by making one longitudinal diagonal cut. These halves are then rotated and spliced to give the maximum depth at mid-span. When camber is required, it may be obtained very simply by skewing the two halves slightly between their abutting edges before making the splice.
>
> Roof loads being relatively light, tapered girders may generally be fabricated from plates the thickness of which is limited only by availability and the maximum web depth-thickness provision of the AISC Specification.
>
> When the girders are used with the sloping flange up, their taper in both directions from the ridge provides the slope that may be required for drainage. Furthermore, by varying the end depth of successive girders the deck may be canted to drain toward roof boxes in the valleys between adjacent gabled spans and at flanking parapet walls, thereby eliminating the necessity for crickets.
>
> For flat roofs the girders are inverted, the tapered flange being down. Some other roof designs frequently call for a gable ridge in the centre span of three spans across the width of the building. In such a case inverted girders are used in the outside spans thereby continuing the same slope of decking to the walls.
>
> There are also additional advantages. Economy is realized in diminished overall height of exterior walls as a result of the reduced depth of web at the ends of the girders. Also when used as the principal carrying members for ordinary joisted roof construction above, and a fire retardant ceiling below, tapered girders provide the tight draft stops required by many building codes as a means of subdividing the attic space. One system of tapered beam construction is illustrated in Fig. 34.'

FIG. 34 TAPERED ROOF CONSTRUCTION
(Photograph by courtesy of American **Institute** of Steel **Construction**)

47. DESIGN EXAMPLE OF TAPERED BEAM

47.1 Tapered beams may also be built up by flame cutting on the skew the web of a rolled beam, then reversing the two segments and rewelding together with a single straight weld. Design Example 12 to be presented herein takes the same beam that was made into a perforated web section as Design Example 11 and develops a beam of identical span and load application as was used for the perforated web beam. The result indicates that the perforated web beam is slightly more economical than the tapered beam for this particular design. In addition, the perforated web beam deflects somewhat less than the tapered beam but both may be given a camber to eliminate any unsightliness due to deflection. When used as a roof member, the tapered beam has some advantage over the perforated beam in that a natural pitch is provided with a resulting pleasing appearance as well as simple drainage characteristics. Of course, if a flat roof is desired, the tapered beam may simply be inverted.

47.2 An illustrative design example of a tapered beam is shown in the following three sheets (*see* Design Example 12).

Design Example 12 — Tapered Beam

The moment is determined at the quarter point for an assumed uniform load. The fact that the web is tapered will cause negligible error in this assumption. Since this particular tapered girder is formed by reversing two skew-cut segments of a rolled beam for each half-beam span the section moduli and properties of the original beam section will be unchanged at the quarter point where the depth remains as originally rolled before flame-cutting and rewelding. Thus it is convenient to estimate the capacity of the tapered beam on the assumption that the maximum stress will be at the quarter point. For uniform load, the use of the tapered beam obviously will increase the capacity in comparison with a straight beam section by 33 percent. The bending moment at the quarter point is three-quarters of that at the centre where the moment would control in a straight beam. The net capacity of the tapered beam is found to be 26·17 t as compared with 28·16 t for the perforated web beam Design Example under the same load conditions and span length. The difference in amount of steel between the two special beams is about 10 percent but the cutting pattern and welding operation are more complex for the perforated web beam than for the tapered beam.

Design Example 12	**1 of 3**
Determining Load Carrying Capacity	

The procedure used in this design example to find the proper taper that will give a maximum stress at the quarter point is to determine the proper depth for the same maximum stress one metre closer to the reaction point than at the quarter point. Thus is determined that point at which the derivative of stress as a function of beam depth is equal to zero and thus the stress is (in this case) a maximum.

Tapered Beam Design

Light tapered roof girders formed by longitudinal skew-cut of ISMB 600 is shown as an alternative to perforated web beam formed from same section (*see* Design Example 11).

Make critical section at 1/4 point (18/4) = 4·5 m where tapered beam has original depth d.

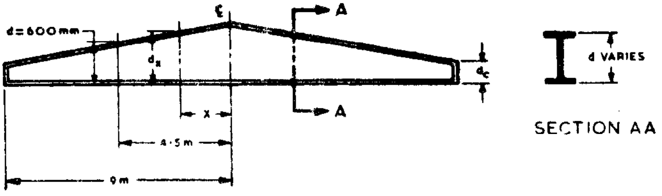
$$\text{At } 1/4 \text{ point } M = \frac{3}{4}\,\frac{WL}{8} = \frac{3\times 1\,800}{32}\times W \text{ cm·t}$$

$$Z_{xx} = 3\,060{\cdot}4 \text{ cm}^3 \ (\textit{see} \text{ IS: 808-1957})$$

$$\frac{M}{Z} = f \quad \text{Hence,} \quad \frac{3\times 1\,800}{32\times 3\,060{\cdot}4}\times W\times 1\,000 = 1\,575 \text{ kg/cm}^2$$

$$\therefore \quad W = \frac{1\,575\times 32\times 3\,060{\cdot}4}{3\times 1\,800\times 1\,000} = 28{\cdot}6 \text{ t}$$

Deduct dead weight 122·6 × 18 = 2·21
(10 percent added for stiffeners) = 0·22
2·43 t
Net load 26·17 t



Design Example 12

Determining Taper

2 of 3

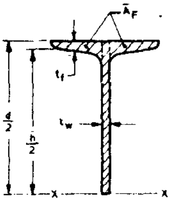
Most of Sheet 2 provides self-explanatory details whereby the section modulus of the tapered beam is found as a function of beam depth. Then by determining the depth one metre from the quarter point to make the stress equal to that of at the quarter point, the depth of beam and tapered slope are determined.

To determine proper taper, assume same stress at a section 5·5 m from ℄ and determine depth (one metre from quarter point).

$$M_c\ (@\ ℄) = \frac{28{\cdot}6 \times 18 \times 100}{8} = 6\,435 \text{ cm·t}$$

$$M\ @\ 5{\cdot}5 \text{ m} = M_c\left[1-\left(\frac{5{\cdot}5}{9{\cdot}0}\right)^2\right] = 6\,435 \times 0{\cdot}626 = 4\,035 \text{ cm·t}$$

To estimate Z as a function of depth for tapered section for ISMB 600, 122·6 kg, determine an equivalent flange area A_F as shown.



$$\bar{A}_F = \frac{A - dt_w}{2}$$

$b = 21{\cdot}0$ cm

$d = 60{\cdot}0$ cm

$t_w = 1{\cdot}2$ cm

$t_f = 2{\cdot}08$ cm

$I_{xx} = 91\,813 \text{ cm}^4$

$Z_{xx} = 3\,060{\cdot}4 \text{ cm}^3$

Assume $\bar{h} = 60 - 2{\cdot}08 = 57{\cdot}92$ cm

$$I_{xx} = \frac{1}{12}\,t_w d^3 + \frac{\bar{A}_F(\bar{h})^2}{2}$$

$$91\,813 = \frac{1}{12}\,1{\cdot}2 \times 60^3 + \bar{A}_F\,\frac{57{\cdot}92^2}{2}$$

$$\bar{A}_F = 41{\cdot}9 \text{ cm}^2$$

Area of section $= 41{\cdot}9 \times 2 + 1{\cdot}2 \times 60 = 155{\cdot}8 \text{ cm}^2$

The area from structural tables $= 156{\cdot}21 \text{ cm}^2$

But use 155·8 cm² only as equivalent (or effective) area.

For vertical depth

$$Z = \frac{2I}{d_x} = \frac{t_w d_x^2}{6} + \frac{\bar{A}_F(h)^2}{d_x}, \text{ where } h = d_x - 2{\cdot}08$$

$$\therefore\ Z = 0{\cdot}2d_x^2 + \frac{41{\cdot}9\ (d_x - 2{\cdot}08)^2}{d_x}$$

$$= 0{\cdot}2\ d_x^2 + 41{\cdot}9\left(d_x - 4{\cdot}16 + \frac{4{\cdot}32}{d_x}\right)$$

Required Z at 5·5 from ℄

to have a stress of 1 575 kg/cm² $= \dfrac{4\,035 \times 1\,000}{1\,575} = 2\,565 \text{ cm}^3$

Reduce to a quadratic equation by approximating $\dfrac{4{\cdot}32}{d_x} = 0{\cdot}07$

or $0{\cdot}2\ d_x^2 + 41{\cdot}9\ d_x - 171{\cdot}4 = 2\,565$

or $0{\cdot}2d_x^2 + 41{\cdot}9d_x - 2\,736 = 0$, $d_x = 52{\cdot}5$ cm

Taper slope $= 60 - 52{\cdot}5 = 7{\cdot}5$ in 1·0 m

d_c @ ℄ $= 60 + 7{\cdot}5 \times 4{\cdot}5 = 93{\cdot}75$ cm

d_c @ ends $= 60 - 7{\cdot}5 \times 4{\cdot}5 = 26{\cdot}25$ cm

Design Example 12 | **3 of 3**

Final Check of Stresses

As a final check on the stress due to bending, three points are chosen near the quarter point and direct calculation made of the stress at these points.

*A calculation of the deflection is made using Newmark's numerical procedure. This is very accurate for a problem of this type and especially useful because of the variable moments of inertia. For the details of this procedure, reference should be made to Section IV. The formulæ for equivalent concentrated angle change are given on p. 86 and it should be noted that at the centre line of the tapered beam there is a sharp discontinuity in the M/EI diagram. The statement made in the last two sentences of the last paragraph of **27.1** (see p. 88) applies to the calculation of the concentrated angle change at the centre line. Simpler examples of the procedure have been given previously on p. 97 and 98.*

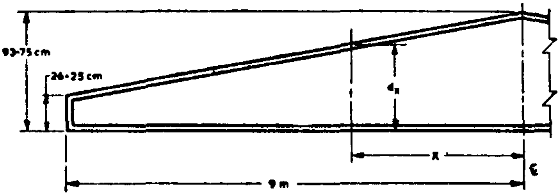
End shear capacity:

$V = 26{\cdot}25\ (1{\cdot}2)\ 945 = 29\,800$ kg or $29{\cdot}8$ t $> \frac{28{\cdot}6}{2}$ maximum end shear (*see* p. 146) ..... OK.

Check stress due to bending at 3 points near 1/4 span.

$$d = 93{\cdot}75 - \frac{67{\cdot}5}{9}x = 93{\cdot}75 - 7{\cdot}5x \text{ (x is in metres)};\ M = M_c\left(\frac{1-x^2}{9^2}\right)$$

$$Z = 0{\cdot}2\,d^2 = 41{\cdot}9\,d - 171{\cdot}6 \text{ (see Sheet 2)}$$



x	d	d^2	$0{\cdot}2\,d^2$	$41{\cdot}9\,d$	Z	x^2	$\frac{x^2}{81}$	$\frac{M}{M_c}$	$\frac{f_b}{M_c}$
4	63·75	4 060	813	2 670	3 310	16	0·198	0·802	$2{\cdot}423\times10^{-4}$
4·5	60·0	3 600	720	2 514	3 062	20·25	0·249	0·751	$2{\cdot}452\times10^{-4}$
5	56·25	3 170	634	2 360	2 822	25·0	0·308	0·692	$2{\cdot}451\times10^{-4}$
6	48·75	2 380	476	2 045	2 349	36	0·444	0·556	$2{\cdot}367\times10^{-4}$

This checks maximum stress due to bending at 1/4 point. Determine deflection by Newmark Method.

								MULTIPLIER
Location in span	0		1/6		1/3		℄	—
Moment	0		5/9		8/9		1	6 435
I	—		0·222		0·527		1	258 500 cm⁴
M/EI	0		2·505		1·685		1	6 435/2 585 E
Conc ϕ	—		−26·74		−20·36		−13·37	6 435/12 × 258 500 E (parabolic variations)
Slope		53·79		27·05		6·69		do
Deflection	0		5·38		8·09		8·76	264 350/12 × 258 500 E

$$\text{Centre deflection} = \frac{8{\cdot}76\times3^2\times64\,350\times100\times100\times1\,000}{12\times2\,050\,000\times258\,500} = 7{\cdot}95\text{ cm}$$

48. DESIGN EXAMPLE OF TAPERED GIRDER

48.1 The tapered beam is particularly suited to uniform loads applied to long spans but may be adapted to concentrated loads somewhat more effectively than the perforated web beam. Design Example 13 is for a tapered girder built up from two flange plates and two skew-cut web plates flame cut from each half span from a single rectangular plate. Design Example 14 compares the design of a three-span continuous beam for the same load conditions that are used in Design Example 13. The total weight of 3 simple spans using tapered beams is found to be about 10 percent less than that of the three-span continuous beam. However, no generalization should be drawn from one comparison.

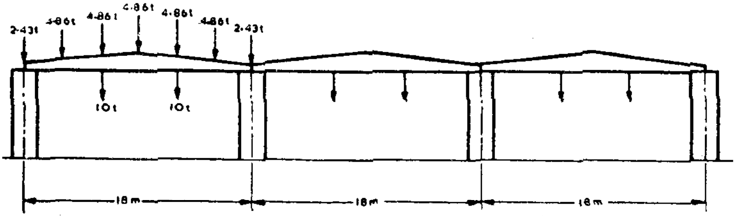
It is recognized that the ordinary beam theory does not apply precisely to the tapered beam and the stress in the sloping flange will be slightly greater than calculated. However, the tapered slopes are very small, and the stress raising coefficient due to slope may be neglected with small error. For additional information and other design examples utilizing the tapered beam the reader should obtain the previously cited reference from the American Institute of Steel Construction, 101 Park Avenue, New York City.

48.2 The illustrative design example of tapered girder is shown in the following three sheets (*see* Design Example 13).

Design Example 13 — Tapered Girder

Design Example 13 | **1 of 3**
Preliminary Design of Tapered Girder

The spacing between the bents is assumed at the outset to be 8 m centre to centre. It is obvious that the critical stress is at the third point because of the concentrated hoist loads. This eliminates the need of using the procedure of Design Example 12 to determine the point of maximum stress. The bending moment at the centre is not very much greater under maximum load than at the third point and a beam with a very small taper could very well be used. The preliminary selection of web is on the basis that the maximum d/t_w at the centre shall be less than 180 thus permitting the widest possible spacing of stiffeners.



Bents are 8 m c/c

Loading conditions:

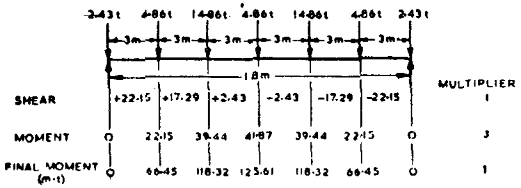
Monorail hoist load (including impact) = 10 t

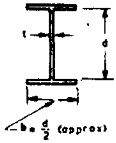
Roof live load	= 150 kg/m²
Roof load including purlins (3 m c/c concentrated)	= 30 kg/m²
	180 kg/m²

Loads on roof $= \frac{180 \times 3 \times 8}{1\ 000}$ = 4·32 t

Allowance for dead weight of girder (at purlin reactions) = 0·54 t

Total dead load = 4·86 t





For minimum use of stiffeners, assume d/t = 180 at ℄.

Section is critical at 1/3 point, where d/t is about 150

$$d = \sqrt[3]{\frac{1{\cdot}5M\ (d/t)}{f}} = \sqrt[3]{\frac{118{\cdot}32 \times 100 \times 1\ 000 \times 1{\cdot}5 \times 150}{1\ 500}} = 122 \text{ cm}$$

NOTE — Taking that web depth $= d = 2b$, and $t_w = t_f = t$

I of 2 flanges $= \frac{td^3}{4}$, I of web $= \frac{td^3}{12}$, Total $I = \frac{td^3}{3}$

$Z = I/d/2 = \frac{2td^3}{3d} = \frac{td^2}{1{\cdot}5}$ Hence, $d = \sqrt[3]{\frac{1{\cdot}5\ M\ d/t}{f}}$ (by $\frac{M}{Z} = f$)

Design Example 13	**2 of 3**
Finalizing Taper of Girder	

In this sheet, the required web depth at the end is found to be 29·3 cm. Since this is rather small it is arbitrarily increased to 40 cm, but for a 120-cm depth as assumed at the third point the d/t_w at the centre line would now be 200. To bring this down to a d/t_w of 180 at the centre it would be necessary to have an end depth of 80 cm. This, however, is judged to be rather wasteful of material and a 65-cm end depth is finally chosen to make the third point depth slightly less than initially assumed. In this design, with a sharp break in the moment curve at the third point, there is great flexibility in the choice of centre and end depths because the third point will be critical for maximum stress over a wide range of variation in taper.

In a preliminary trial design, the web was made one centimetre thick with a 160-cm depth at the centre and a 25-cm depth at the end. It was found that this would require more steel than the design now being recorded. Furthermore, the small depth near the end would lead to excessive deflection. This will be better understood if one studies the deflection calculation for Design Example 12 where the greater concentrations of curvature are fairly near the ends. At the bottom of this sheet, the stress is checked at the sixth point and centre line, and it is believed that the final choice of taper and depth has provided a well-balanced design as stiff and as economical of steel as one might desire.

Web thickness $t = \frac{120}{150} = 0\cdot8$ cm; For $V = 22\cdot15$ t @ end

F_s = 945 kg/cm² [*see* stiffened web as in Table IIIA in **9.3.2**(b) of IS: 800-1956]

Try 8-mm web. d (at end) $= \frac{22\cdot15}{8\times945} = 29\cdot3$ cm

Try 40×8 at end to 160×8 at ℄. $\frac{d}{t}$ at ℄ $= \frac{160}{0\cdot8} = 200$

Reduce d/t value by changing d to 140 at ℄, $d/t = 175$

At end, $d = 140-3\ (140-120) = 80$ cm

Reduce d at end to 65 cm to provide more taper (keeping web depth at ℄ same 140 cm).

Select flange at 1/3 point as probable critical location; approximate formula is:

$$Z = A_f d + \frac{d^2t}{6} \qquad \text{Required } Z = \frac{118\cdot32\times1\,000\times100}{1\,500} = 7\,888 \text{ cm}^3$$

Web segments will be cut from 65+140 = 205 cm, 205×0·8 cm plate

At 1/3 point, $d = 65+2/3\ (140-65) = 115$ cm

$$\therefore\ Z = 7\,888 = A_F\,115 + \frac{115^2\times0\cdot8}{6};\ \therefore\ A_F = \frac{6\,128}{115} = 53\cdot3 \text{ cm}^2 \text{ (area of one flange)}$$

Use 40×1·4 cm flange: $A_F = 56$ cm²

Check d/t of outstanding leg: $\frac{20}{1\cdot4} = 14\cdot3 < 16$ ..... OK.

Check stresses at 4 locations:

	END	1/6 l	1/3 l	CENTRE
d	65	90	115	140
Z (approx)	—	6 116	8 200	10 430
f_b	0	1 087	1 443	1 205

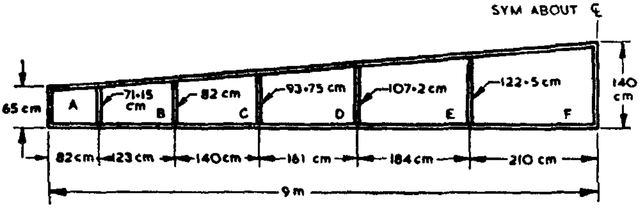
Since $l/b = \frac{300}{40} = 7\cdot5$, allowable $f_b = 1\,500$ kg/cm² (*see* **9.2.2.2** of IS: 800-1956)

Design Example 13

Intermediate Stiffeners

3 of 3

Vertical intermediate stiffeners are put in starting from the centre line at the maximum permitted spacing of 1·5 times the depth. The depth is conservatively taken as the greatest in each panel. Near the bottom of the sheet, the permissible web shear stresses by Table IV of this handbook worked out on the basis of Table III in IS: 800-1956 (based on average depth in each panel) are compared with the maximum shear stress in each panel.



Vertical stiffeners @ 1·5 *d* (*see* **20.7.1.1** of IS: 800-1956)

$1{\cdot}5 \times 140 = 210$ cm

$1{\cdot}5 \times 122{\cdot}5 = 184$ cm

$1{\cdot}5 \times 107{\cdot}2 = 161$ cm

$1{\cdot}5 \times 93{\cdot}75 = 140$ cm

$1{\cdot}5 \times 82 = 123$ cm

$1{\cdot}5 \times 71{\cdot}75 = 107$ (or the remaining distance of 82 cm)

Check shear on basis of conservative assumption that d = average depth of web segment between stiffeners and shear stress is maximum in the particular segment.

Check actual f_s with allowable F_s of Table IV (*see* p. 182)

$$\text{Segment } A = f_s \text{ (average)} = \frac{22{\cdot}15 \times 1\,000}{65 \times 0{\cdot}8} = 430 \text{ kg/cm}^2$$

$$d/t \text{ (average)} = \frac{71{\cdot}75 + 65}{2 \times 0{\cdot}8} = 86$$

Repeat procedure in other segments, and tabulate as follows:

SEGMENT	f_s (AVERAGE)	d/t (AVERAGE)	F_s (ALLOWABLE)*
A, *B*	430	86	945
C, *D*	265	99	920

(No need to check further panels as shear is reducing and allowable F_s *Min* from Table IV is 715 kg/cm²)

NOTE — Stiffener size computations, weld design and other details are omitted. Reference should be made to Design Example 2.

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*See Table IIIA of IS: 800-1956 or Table IV of this handbook.

SECTION VIII

COMPOSITE BEAM CONSTRUCTION

49. GENERAL

49.1 In composite construction, a reinforced concrete slab is integrally supported by, and attached to steel I-beams. Channel or other type shear connectors are welded to the top flange of the steel beams to bring the concrete slab and the beams into integral action. If a concrete slab is placed on the top of a steel beam without such shear connectors, there will be a certain amount of bond initially but this is apt to be destroyed during the course of time. The shear connectors that unite the slab and the beam in composite construction serve a two-fold purpose (1) tying the slab and beams together and (2) transferring between the slab and the beams the horizontal shear that is developed by composite action. The strength of the composite slab is considerably more than the sum of the strengths of the two components acting separately.

50. DESIGN EXAMPLE PRINCIPLES

50.1 In IS : 800-1956 the subject of composite beams is discussed in **20.8.2.2.** Reference is made in this to IS : 456-1957 Code of Practice for Plain and Reinforced Concrete for General Building Construction (*Revised*). Thus, the details of design are more a matter of reinforced concrete construction than steel construction but the steel designer should be aware of the economic possibilities that may be obtained.

50.2 The use of any particular type of shear connector in composite construction should be based on comprehensive tests in structural laboratories. The local distribution of stress around a connector is highly complex and the approach to be made is more one of ultimate strength than elastic analysis. Tests have been made at the University of Illinois on the channel type shear connector. Another type of shear connector in common use is a single reinforcing bar bent in the form of a spiral spring which is welded at contact points along the entire length of the top flange.

50.3 A section through a composite beam and concrete is shown in Fig. 35A and the assumed distribution of bearing of stress in the concrete that is the basis for the Illinois* recommendations of design in Fig. 35B. The use of

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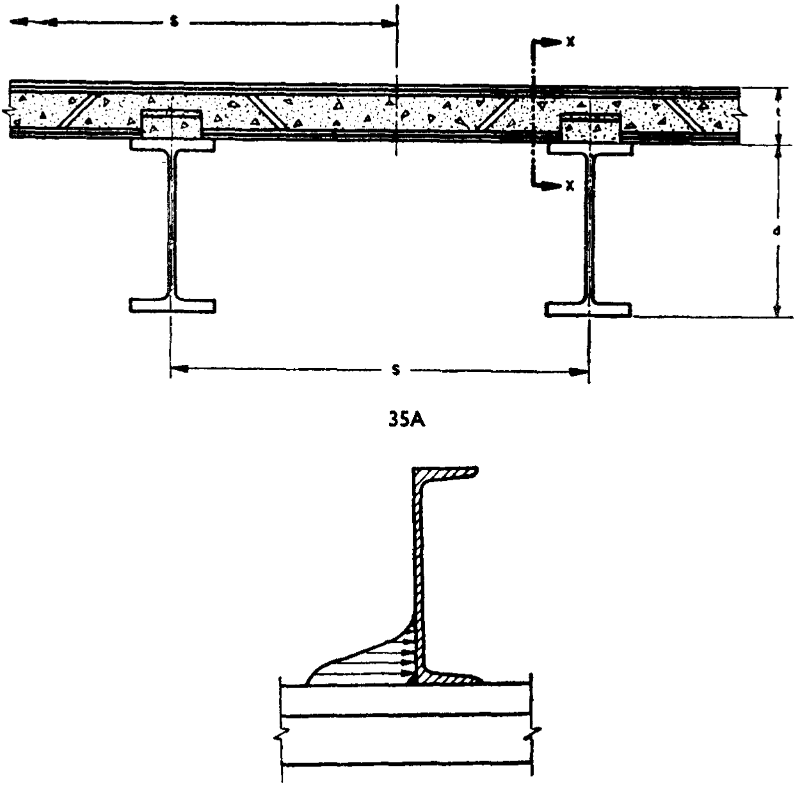
*VIEST, I. N. AND SIESS, C. T. Design of Channel Shear Connectors for Composite I-Beam Bridges. *Public Roads*, Vol 28, No. 1 (April 1954).



35A

ENLARGED SECTION XX

35B

Fig. 35 Composite Beam with Channel Shear Connections

composite concrete slab and steel beam construction has been especially common for the short span highway bridge where the concrete slab serves as the road slab but also acts integrally to form the compression flange of the steel beam into a single composite tee beam unit.

50.4 The top flange of the channel shear connector serves to hold the beam and slab together and most of the shear is transferred by pressure near the base of the channel as shown in Fig. 35B. The spacing of the channel shear connectors follows exactly the same principles used herein for spacing of rivets and intermittent welds in built-up steel girders. The special design problems that pertain to composite beams, as mentioned previously, are those in the realm of reinforced concrete design.

SECTION IX

CONTINUOUS BEAM DESIGN

51. INTRODUCTION

51.1 The use of continuity in beam construction involves similar problems to those encountered in the design of continuous or 'rigid' frames as covered in ISI Handbook for Structural Engineers on Rigid Frame Structures (under preparation). When loads are primarily static, consideration should be given to the possible application of plastic design as treated in the ISI Handbook for Structural Engineers on Plastic Theory and Its Application in the Design of Steel Structures (under preparation). The economy in continuous construction is open to some question but has found considerable favour in multi-span highway bridges. Use of all-welded construction lends itself especially to the use of continuity much in the same way as does reinforced concrete. Continuous beam construction will usually be somewhat more rigid than simple beam construction and has the advantage of supplying more inherent reserve of strength should a local failure result. The analysis is much more complex and design skill and time much greater than needed for simple beam design. If the structure has to adjust to a considerable differential settlement of support, simple beam construction has the advantage of being able to adjust to such settlement without causing stresses in the members.

For a more complete discussion of continuity in beam and frame design, reference should be made to ISI Handbook for Structural Engineers on Plastic Theory and Its Application in the Design of Steel Structures (under preparation) and attention here will primarily be given to Design Example 14 which will be for the same load and span conditions previously considered in Design Example 13. Thus, a comparison is afforded between the amount of steel required in a continuous beam design and a simple tapered beam design. It is found that the continuous beam requires 10 percent more than the three simple spans. No general conclusion should be drawn but it is obvious that the use of continuity in design does not automatically assure greater economy than simple beam construction.

52. DESIGN EXAMPLE OF CONTINUOUS BEAM

52.1 The illustrative design of continuous beam is shown in the following twelve sheets (*see* Design Example 14).

Design Example 14 — Continuous Beam

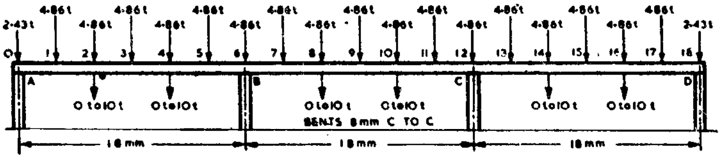
Design Example 14 | **1 of 12**

Bending Moment Due to Uniform Load

In the design of a continuous beam, some preliminary estimate shall be made as to the distribution of bending moment. The preliminary analysis serves as a basis for the selection of trial member sizes and locations for change in section by addition of cover plates or change in flange thickness. For the preliminary basis, a constant moment of inertia may be assumed and the tables of influence coefficients for reactions and moments in Tables V and VI (see p. 187 and 188) are especially convenient for preliminary bending moments. Three span beams have been covered and procedure explained in Appendix A. In view of the 'on' or 'off' nature of the monorail hoist loads which form an appreciable part of the total load, method of plastic design is not recommended because of the possibility of fatigue failure. The variation in loading also requires that various moment diagrams be drawn and the design prepared for the over-all range of maximum positive or negative moments as determined at any point in the span.

Continuous Beam Design

Alternative to Design Example 13 (Tapered Beam Frame)



Try constant I as basis for preliminary estimate of bending moment.

Monorail hoist load **10 t** (or none) at **1/3** points. (Because of alteration of moment due to hoist loads, plastic design is not recommended.)

Roof pitch provided by variable purlin position

Roof live load = **150 kg/m²**

Roof dead load (purlins plus corrugated sheet) = **30 kg/m²**

180 kg/m²

As bents are at 8 m c/c (taking roof load as uniformly distributed)

180 × 8 = **1 440 kg/m** width

Allowance for dead weight of girder at **180 kg/m** = **180 kg/m**

1 620 kg/m

Use Table VI of Appendix A

Plot moment due to uniform load (dead load + live load)

(Note — L in tables equals overall span here = 54 m)

Referring to Appendix A, $WL^2 = 1{\cdot}62 \times 54^2 = 4\,723{\cdot}92$, $m = n = 1/3$

MOMENT AT DISTANCE FROM LEFT, m (1)	MOMENT M_s DUE TO UNIFORM LOAD (All Spans Loaded), m·t (2)
*3·6	+ 0·006 7 × 4 723·9 = + 31·7
7·2	+ 0·008 9 × 4 723·9 = + 42·1
10·8	+ 0·006 7 × 4 723·9 = + 31·7
14·4	+ 0·000 0 × 4 723·9 = 0
18·0	− 0·000 7 × 4 723·9 = − 52·5
22·5	− 0·000 7 × 4 723·9 = − 3·32
27·0	+ 0·002 8 × 4 723·9 = + 13·25

---

*These distances are chosen because the coefficients given in Appendix A (*see* p. 184) correspond to these only.

Design Example 14

Max Positive & Negative Bending Moments

2 of 12

Moments are calculated for the following hoist load locations:

1) Centre and end span: Maximum negative moment at support.

2) Centre span only: Maximum positive moment in centre span.

3) Both end spans: Maximum positive moment in end span.

Maximum Negative Moment at Support B Due to Hoist Loads

Load spans AB and BC:

Obtain influence coefficients from Appendix A. Interpolate to get coefficients for M_5 (M_B in this design) with loads at 1/3 point.

LOAD LOCATION (Distance from Left End)	INTERPOLATED INFLUENCE COEFFICIENTS
6 m	−0·026 m·t
12 m	−0·031 m·t
24 m	−0·024 m·t
30 m	−0·018 m·t
	$-0{\cdot}099 \times \underset{P}{10} \times \underset{L}{54} = -54$ m·t

Maximum M_5 caused by hoist load *plus* live load *plus* dead load

$$= -52{\cdot}5 - 54 = -106{\cdot}5 \text{ m·t}$$

Maximum Positive Moment in Centre Span:

Load centre span BC only

$M_{\max}$ (approx) for hoist load $= 2 \times 10 \times 2/3 \times {}^{*}0{\cdot}058 \times 54 = 41{\cdot}2$ m·t

$\therefore$ Maximum positive M (Centre span) $= {}^{\dagger}13{\cdot}25 + 41{\cdot}2 = 54{\cdot}45$ m·t

End Span Moments

To calculate hoist load moment in end span with both end spans loaded, use influence coefficients for R_o and R'_o (R_A) in this design in Table V in Appendix A.

DISTANCE FROM LEFT END (R_0) AND RIGHT END (R'_0)	INTERPOLATED INFLUENCE COEFFICIENTS	
	For R_o	For R'_o
6	0·59	0·019
12	0·24	0·023

Maximum reaction $= {}^{\ddagger}0{\cdot}872 \times 10 = 8{\cdot}72$

	HOIST LOAD MOMENT	LL+DL MOMENT	COMBINED MOMENT
$3{\cdot}6 \times 8{\cdot}72$	31·4	31·7	63·1
$7{\cdot}2 \times 8{\cdot}72 - 1{\cdot}2 \times 10$	50·8	42·1	92·9
$10{\cdot}8 \times 8{\cdot}72 - 4{\cdot}8 \times 10$	46·2	31·7	77·9
$14{\cdot}4 \times 8{\cdot}72 - -8{\cdot}4 \times 10 - 2{\cdot}4 \times 10$	17·6	0	17·6

---

[*] *See* Sheet 1.

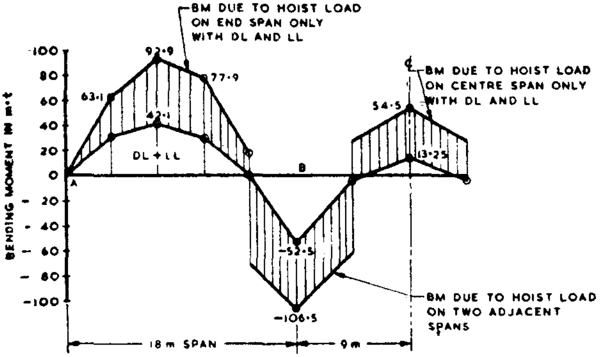
[†] Refer to Table V of Appendix A.

[‡] Total $R_0 + R'_0$ which gives R_A in this design for hoist loads in both the end spans. It has been possible to calculate R_A like this because of symmetry in load positions.

Design Example 14	**3 of 12**
Preliminary Design of Continuous Beam	

The moments computed for the beam of uniform cross-section are plotted for various load conditions on this sheet and the maximum required section moduli are determined on the assumption that it may be possible to use a rolled beam. This is not necessarily the most economical or lowest weight choice as a built-up plate girder with a greater depth-to-span ratio and less web material would require less weight of steel. However, it was desired to present a design example in which a rolled beam with welded cover plates was used in continuous construction. On the basis of the maximum positive moment in the centre span (where a section modulus of 3 460 cm³ is indicated) a rolled beam with a section modulus of 3 540 cm³ is chosen. When the continuous beam is stiffened up at the supports and in the end spans there will be a redistribution of moment and the positive moments in the centres of all spans will in general decrease slightly while the negative moments over the supports will tend to increase. Thus, the analysis on the basis of constant moment of inertia will over-estimate the section modulus requirements for positive moment in the centre span. This should be anticipated in the preliminary design.

Preliminary Estimate Range of Bending Moment



Maximum Required Z

(Lateral support of purlin connections is assumed.)

End span (positive): $\dfrac{93 \times 100 \times 100}{1\,575} = 5\,905 \text{ cm}^3$

Interior support (negative): $\dfrac{106{\cdot}5 \times 100 \times 1\,000}{1\,575} = 6\,760 \text{ cm}^3$

Centre span (positive): $\dfrac{54{\cdot}45 \times 100 \times 1\,000}{1\,575} = 3\,460 \text{ cm}^3$

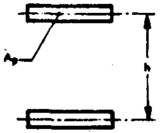
If a rolled beam with flange plate reinforcement is used as a basic section, actual moment at centre span will be less than the estimated value.

An ISWB 600, 133·7 kg (Z = 3 540 cm³) is chosen as a basic section.

Approximate Z of 2 plates:

$$I = 2A_p \left(\frac{h}{2}\right)^2 = \frac{A_p h^2}{2}$$

$$Z = \frac{2I}{h} = A_p h$$



Design Example 14

Preliminary Selection of Flange Plates

4 of 12

The additional section modulus requirement for the end span and at the interior support is determined and flange plates are shown at the bottom of this sheet where a check is also made by the moment of inertia method. The flange plates provide an excess section modulus at the interior support for negative moment and somewhat less than the preliminary estimate for positive moment in the end span. The cover plates are extended 0·5 m beyond their theoretical cut off points at which they will be assumed as fully active in the more accurate analysis to follow. In view of the possibility of fatigue failure under intermittent load, it is wise to be very conservative as to allowable stresses at cut off points of cover plates in welded plate girders. At these locations, concentrated stress is developed and it is here that fatigue cracks will occur if they develop anywhere. Also shown on this sheet are the field splice locations on the basis that a 14-m length represents the maximum desirable length for convenient in transport. In erection the 9-m segments over the interior columns could be placed first with temporary shoring underneath to provide stability. Then the interior and exterior 12-m segments may be introduced and field welds made at locations shown.

Flange Plates Trial Selection

At end span: Z to be added = *5 800 − 3 540 = 2 260, $h = 600$ mm

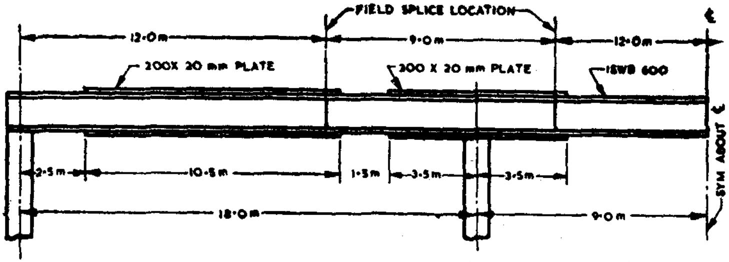
$$A_p = \frac{2\,260}{60} = 37{\cdot}7 \text{ cm}^2 \qquad \text{Try } 200 \times 20 \text{ mm plate.} \quad A = 40 \text{ cm}^2$$

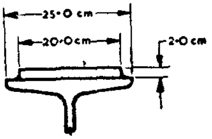
At support:

$$A_p = \frac{6\,760 - 3\,540}{60} = \frac{3\,220}{60} = 53{\cdot}7 \text{ cm}^2 \text{ (excess probably required).}$$

Try 300 × 20 mm plate. $A = 60$ cm² (excess probably required).

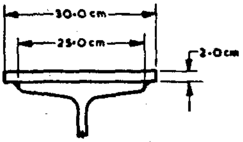
TRIAL LOCATION OF FLANGE PLATES





$$
\begin{aligned}
I_{WB} &= 106\,198{\cdot}5 \text{ cm}^4 \\
\text{Plates} = \frac{2 \times 40 \times (62)^2}{4} &= \underline{76\,800{\cdot}0 \text{ cm}^4} \\
&\ 182\,998{\cdot}5 \text{ cm}^4 \\
Z = \frac{182\,998{\cdot}5}{32} &= 5\,770 \text{ cm}^3
\end{aligned}
$$

$$
\begin{aligned}
I_{WB} &= 106\,198{\cdot}5 \text{ cm}^4 \\
\text{Plates} = \frac{2 \times 60 \times (62)^2}{4} &= \underline{115\,320 \quad \text{cm}^4} \\
&\ 221\,518{\cdot}5 \text{ cm}^4 \\
Z = \frac{221\,518}{32} &= 6\,922 \text{ cm}^3
\end{aligned}
$$



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*Slightly less than the estimated figure of 5 905 cm³ is taken.

Design Example 14	5 of 12
Method of Determining Redundant Reactions	

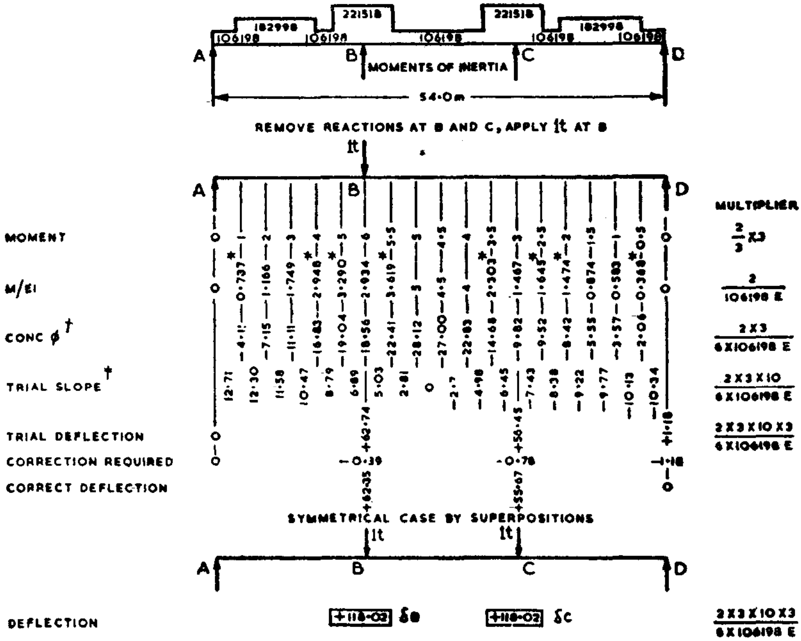
Most designers would probably prefer to use the generalized moment distribution procedure in making the final analysis of the structure with variable moment of inertia. However, this would require the availability of tables of distribution and carry-over factors. In this handbook, the general procedure of determining redundant reactions will be used for illustration. The principle is extremely simple and may be stated simply as follows:

> *Reactions at B and C (see sketch on Sheet 6) in a three-span beam are removed and the deflection at B is then thought of as the sum of the effects of the downward load and the two redundants. This is equated to zero. Under symmetrical load conditions, the reactions at B and C will be identical and a direct solution for redundant reaction R_b follows. For unsymmetrical load conditions, two simultaneous equations are obtained with the unknown R_b and R_c. The Newmark numerical method is used in the deflection calculations. In Sheet 6, the redundant reactions are shown removed. The deflections as calculated at B and C are for unit loads at those two points respectively. Thus, δ CB represents the deflection at C due to a unit load at B. By the law of reciprocal deflections, this is equal to δ BC. In these examples of the Newmark method, the original procedure suggested by him has been used in that the conjugate beam idea suggested in Section IV is not used. Instead, trial slopes are worked out consistent with the various concentrated angle changes along the beam. Then, in general, trial deflection is determined that will not come out to zero at the end of the span. In Sheet 6, for example, the trial deflection is $-1{\cdot}18$ at point D. The trial deflected beam position is rotated into correct position and a linear correction is introduced at points B, C and D. If the deflections were needed at intermediate points, they could likewise be determined but we are interested here in the deflection only at support points B and C.*

All the calculations described in the commentary above are worked out with diagrams in Sheets 6, 7, 8 and 9.

Design Example 14 Method of Determining Redundant Reactions	6 of 12

TO CHECK MOMENTS, REACTIONS AND SHEARS, USE METHODS OF DEFLECTIONS, UTILIZING NEWMARK'S NUMERICAL PROCEDURE FOR CALCULATIONS



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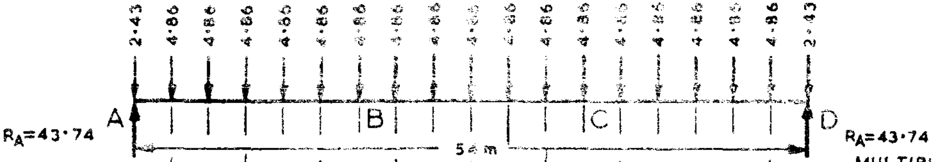
*These values are based on average $\frac{M}{EI}$ in adjacent segments.

†These values are worked out on the assumption that $\frac{M}{EI}$ varies linearly in each segment [*see* case (B) p. 86].

‡Assuming maximum deflection at the centre, the figures for trial slope are worked out at different points along the beam. Corrections are made in the further steps and the correct deflections at the support points are determined.

DEAD LOAD + SNOW LOAD



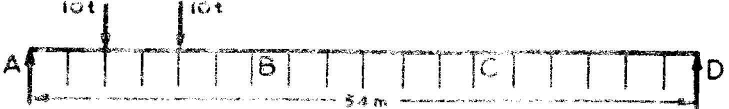
		MULTIPLIER
SHEAR	+17 +15 +13 +11 +9 +7 +5 +3 +1 −1 −3 −5 −7 −9 −11 −13 −15 −17	2·43
MOMENT	0 17 32 45 56 65 72 77 80 81 80 77 72 65 56 45 32 17 0	2·43 X 3
M/EI	0 1·253* 1·87 2·62 4·18* 4·27* 3·52 5·07* 8·0 8·1 8·0 5·07* 3·52 4·27* 4·14* 2·62 1·87 1·253* 0	$\frac{2\cdot43 \times 3 \times 10}{106198\ E}$
CONC φ†	0 −6·88 −11·35 −16·49 −23·45 −24·74 −23·42 −31·80 −45·17 −48·40 −45·17 −31·80 −23·42 24·74 23·45 16·49 11·35 6·88 0	$\frac{2\cdot43 \times 3 \times 10 \times 3}{6 \times 106198\ E}$
SLOPE	20·75 20·06 18·92 17·27 14·93 12·46 10·12 6·94 2·42 2·42 6·94 10·12 12·46 14·93 17·27 18·92 20·06 20·75	$\frac{2\cdot43 \times 9 \times 100}{6 \times 106198\ E}$
DEFLECTION δ	10·44 12·39 10·44	$\frac{2\cdot43 \times 9 \times 3 \times 1000}{6 \times 106198\ E}$

---

*These values are based on average $\frac{M}{EI}$ in adjacent segments.

†These values are worked out on the assumption that $\frac{M}{EI}$ varies linearly in each segment.

HOIST LOAD ON END SPAN ONLY



																				MULTIPLIER
SHEAR	+5	+5	+2	+2	−1	−1	−1	−1	−1	−1	−1	−1	−1	−1	−1	−1	−1	−1		10/3
MOMENT	0	5	10	12	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0	$\frac{10 \times 3}{3}$
M/EI	0	3·69*	5·83	6·70	10·50*	8·55*	5·87	7·34*	10	9	8	4·67*	2·94	3·23*	2·95*	1·75	1·17	0·74*	0	$\frac{10 \times 3}{3 \times 106198\,E}$
CONC φ†	0	−20·59	−33·71	−43·13	−57·25	−50·57	−39·27	−44·83	−56·24	−54·00	−45·61	−29·38	−19·66	−19·05	−16·84	−11·12	−7·17	−4·13	0	$\frac{10 \times 3 \times 3}{6 \times 3 \times 106198\,E}$
TRIAL SLOPE	15·46	13·40	10·03	5·72	0	−5·06	−8·99	−13·47	−19·09	−24·49	−29·05	−31·49	−33·46	−35·36	−37·04	−38·15	−38·87	−39·28		$\frac{10 \times 9 \times 10}{6 \times 3 \times 106198\,E}$

TRIAL DEFLECTION 0 + 39·55 −87·03 −309·19 — multiplier $\frac{10 \times 10 \times 9 \times 3}{6 \times 3 \times 106198\,E}$

CORRECTION REQD +103·06 +206·12 +309·19 — multiplier $\frac{10 \times 10 \times 9 \times 3}{6 \times 3 \times 106198\,E}$

CORRECT DEFLECTION 0 $\boxed{+142 \cdot 61}\ \delta B'$ $\boxed{+119 \cdot 09}\ \delta C'$ 0 — multiplier $\frac{10 \times 10 \times 9 \times 3}{6 \times 3 \times 106198\,E}$

---

*These values are based on average $\frac{M}{EI}$ in adjacent segments.

†These values are worked out on the assumption that $\frac{M}{EI}$ varies linearly in each segment.

Design Example 14

Method of Determining Redundant Reactions

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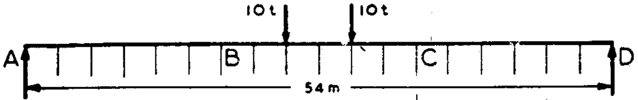
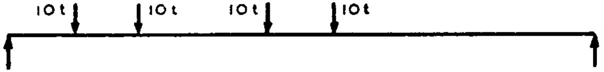
HOIST LOAD ON CENTRE SPAN ONLY



		MULTIPLIER
SHEAR	+1 +1 +1 +1 +1 +1 +1 +1 0 0 −1 −1 −1 −1 −1 −1 −1 −1	10
MOMENT	0 1 2 3 4 5 6 7 8 8 8 7 6 5 4 3 2 1 0	10 X 3
M/EI	0 0·737* 1·166 1·749 2·948* 3·29* 2·934 4·6* 8 8 8 4·6* 2·934 3·29* 2·948* 1·749 1·166 0·737* 0	$\frac{10 \times 3}{106198\ E}$
CONC φ†	0 −4·11 −7·15 −11·11 −16·83 −19·04 −19·63 −29·33 −44·60 −48·0 −44·60 −29·33 −19·63 −19·04 −16·83 −11·11 −7·15 −4·11 0	$\frac{10 \times 3 \times 3}{6 \times 106198\ E}$
SLOPE	+17·56 +17·15 +16·44 +15·33 +13·65 +11·75 +9·79 +6·86 +2·40 +2·40 +6·86 +9·79 +11·75 +13·65 +15·33 +16·44 +17·15 +17·56	$\frac{10 \times 9 \times 10}{6 \times 106198\ E}$
DEFLECTION	+91·88 δ′B = δ′C	$\frac{10 \times 9 \times 10 \times 3}{6 \times 106198\ E}$

HOIST LOAD ON CENTRE SPAN AB BY SUPERPOSITION



DEFLECTION	+139·42 δ′B +131·58 δ′C	$\frac{10 \times 9 \times 30}{6 \times 106198\ E}$

---

*These values are based on average $\frac{M}{EI}$ in adjacent segments.

†These values are worked out on the assumption that $\frac{M}{EI}$ varies linearly in each segment.

Design Example 14 | 10 of 12

Determining Redundant Reactions

For dead load and live load, both redundants are considered as simultaneously applied and the redundant reactions at B and C are found directly by equating the deflection due to these reactions to that due to dead load and live load. Then the reactions at A and D are found by over-all static equilibrium.

When the hoist load is on one end span, two simultaneous equations are obtained which sum the deflections due to various effects (applied loads, R_B and R_C) and equate the sum to zero. The term representing the downward deflection is put on the right side. After solution of the simultaneous equations, R_A and R_D are found by static equilibrium as before.

After the redundant reactions are determined, the shears and bending moments may be calculated by statics.

Calculate reactions $\left(\frac{300}{106\,198\,E}\text{ factors out of all equations}\right)$

Upward reactions are considered negative.

Dead load+Live load (symmetrical about ℄)

By equating the deflection at B:

$$\frac{118{\cdot}02\,R_B}{10} = -2{\cdot}43\times 15\times 10{\cdot}44;\ R_B = R_C = \frac{-2{\cdot}43\times 15\times 10{\cdot}44\times 10}{118{\cdot}02} = -32{\cdot}27\text{ t}$$

$$R_A = R_D = -2{\cdot}43\times 18+32{\cdot}27 = -11{\cdot}47\text{ t}$$

Hoist load on end span only

$$\frac{62{\cdot}35}{10}\,R_B+\frac{55{\cdot}57}{10}\,R_C = -\frac{142{\cdot}61\times 9}{18}\text{ (by equating deflections at } B\text{)}$$

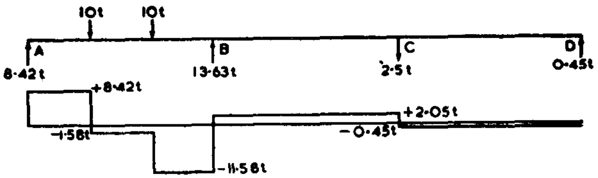
$$\frac{55{\cdot}67}{10}\,R_B+\frac{62{\cdot}35}{10}\,R_C = -\frac{119{\cdot}09\times 1}{2}\text{ (by equating deflections at } C\text{)}$$

$$6{\cdot}235\,R_B+5{\cdot}567\,R_C = -71{\cdot}305;\ 5{\cdot}567\,R_B+6{\cdot}235\,R_C = -59{\cdot}545$$

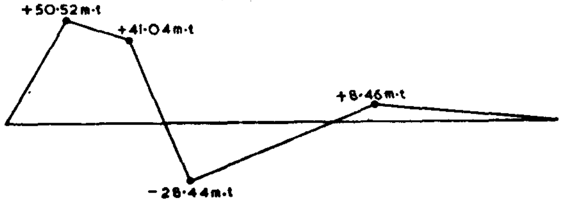
Solving $R_B = -13{\cdot}63$ t, $R_C = +2{\cdot}5$ t

$$R_A = -\left(\frac{5}{6}\times 20-\frac{2}{3}\times 13{\cdot}63+\frac{1}{3}\,2{\cdot}5\right) = -8{\cdot}42\text{ t (by taking moments about } D\text{)}$$

$$R_D = -(22{\cdot}5-8{\cdot}42-13{\cdot}63) = -0{\cdot}45\text{ t}$$



SHEAR DIAGRAM



MOMENT DIAGRAM

The final load condition analysis is made at the top of the sheet and at the bottom, the three basic load conditions for maximum shears and moments in various locations are summarized and the moments for the load combinations are tabulated.

Design Example 14

Bending Moment for Various Load Combinations

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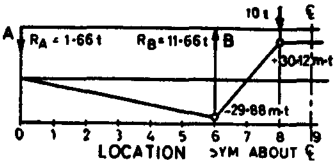
Hoist Load on Centre Span Only (Symmetrical about ℄)

$$\frac{118{\cdot}02}{10}\,R_B = -91{\cdot}88\times1{\cdot}5;$$

$$R_B = -\frac{91{\cdot}88\times1{\cdot}5}{11{\cdot}802} = -11{\cdot}66\text{ t}$$

Hence $R_A = 11{\cdot}66-10 = +1{\cdot}66$ t



BENDING MOMENTS FOR VARIOUS LOAD COMBINATIONS

Location	Dead Load + Live Load on All Spans		Hoist Loads on Spans *A-B*		Hoist Loads on Spans *C-D*		Hoist Loads on Spans *B-C*		Moment for Load Combinations		
	V	*M*	*V*	*M*	*V*	*M*	*V*	*M*	(3)+(5)+(7)	(3)+(9)	(3)+(5)+(9)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0		0		0		0		0	0		
	+9·04		+8·42		−0·45		−1·66				
1		+27·12		+25·26		+0		−4·98	+52·38		
	+4·18		+8·42		−0·45		−1·66				
2		+39·66		+50·52		+1·71		−9·96	+91·89		
	−0·68		−1·58		−0·45		−1·66				
3		+37·62		+45·78		+3·06		−14·94	+86·46		
	−5·54		−1·58		−0·45		−1·66				
4		+21·00		+41·04		+4·41		−19·92	+66·45		
	−10·40		−11·58		−0·45		−1·66				
5		−10·20		+6·30		+5·76		−24·90	+1·86	−35·10	−28·80
	−15·26		−11·58		−0·45		−1·66				
6		−55·98		−28·44		+7·11		−29·88	−77·31	−85·86	−114·30
	+12·15		+2·05		+2·05		+10·00				
7		−19·53		−22·29		+8·46		+0·12	−33·36	−19·41	−41·70
	+7·29		+2·05		+2·05		+10·00				
8		+4·34		−16·14		+2·31		+30·12	−9·49	+34·46	+18·32
	+2·43		+2·05		+2·05		+0				
9		+11·63		−9·99		−3·84		+30·12	−2·20	41·75	31·76
			+2·05		+2·05						
10				−3·84		−9·99					
			+2·05		+2·05						
11				+2·31		−16·14					
			+2·05		+2·05						
12				+8·46		−22·29					
			−0·45		−11·58						
13				+7·11		−28·44					
			−0·45		−11·58						
14				+5·76		+6·30					
			−0·45		−1·58						
15				+4·41		+41·04					
			−0·45		−1·58						
16				+3·06		+45·78					
			−0·45		+8·42						
17				+1·71		+50·52					
			−0·45		+8·42						
18				+0		+25·26					

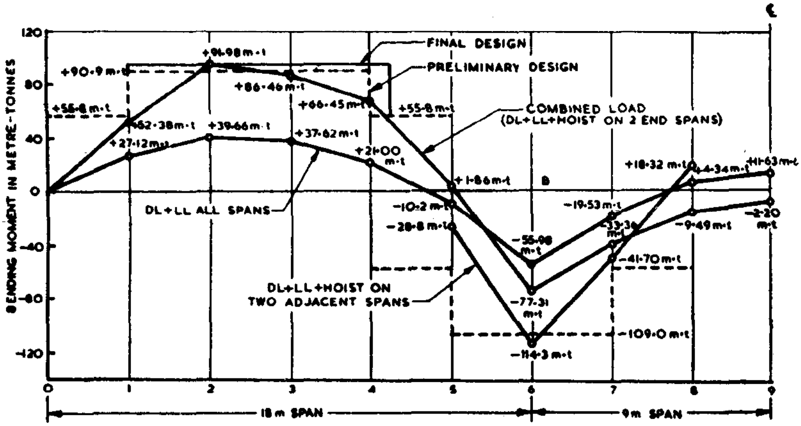
Design Example 14

Final Check of Preliminary Design of Beam

12 of 12

The moments tabulated on Sheet 11 are plotted on this Sheet along with the moment capacity for the cover plates originally selected. It is noted that the capacity over the interior support falls a little short but this should be acceptable since the actual moment due to the distribution of load at the support point will be considerably less than indicated.

The positive moment in the end span is more important and the flange plates are increased 25 mm in width and 0·5 m in length which provides adequate moment capacity.



Negative overstress at *B* not serious because of rounding off moments over support.

Now required Z for positive moment at end spans $= \dfrac{91{\cdot}9 \times 100 \times 1\,000}{1\,575}$

$= 5\,835\ \text{cm}^3$

Change-over plates to 210 × 20 mm and extend 0·5 m more towards the centre span.

$$A = 42\ \text{cm}^2$$

$$I \text{ of ISWB 600} = 106\,198{\cdot}5\ \text{cm}^4$$

$$I \text{ of plates} = \frac{2 \times 42 \times (62)^2}{4} = 80\,724\ \text{cm}^4 \quad Z = \frac{186\,922{\cdot}5}{32} = 5\,841\ \text{cm}^3$$

$$\text{Total } I = \overline{186\,922{\cdot}5\ \text{cm}^4}$$

No new analysis needed; change of *I* in end spans has little effect on distribution of moment.

Shear capacity of beam is more than ample. For other details of design, *see* Design Example 1.

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TABLE I SELECTION OF BEAMS AND CHANNELS USED AS FLEXURAL MEMBERS BASED ON SECTION MODULI

(*Clause* 4.1)

MODULUS OF SECTION (Z_{xx})	DESIGNATION	WEIGHT PER METRE (W)	SHEAR CARRYING CAPACITY (S)
(1)	(2)	(3)	(4)
cm^3		kg	kg × 10^3
3 854·2	***ISWB 600**	145·1	66·9
3 540·0	***ISWB 600**	133·7	63·5
3 060·4	***ISMB 600**	122·6	68·0
2 723·9	***ISWB 550**	112·5	54·6
2 428·9	***ISLB 600**	99·5	59·5
2 359·8	**ISMB 550**	103·7	58·2
2 091·6	***ISWB 500**	95·2	46·8
1 933·2	***ISLB 550**	86·3	51·5
1 808·7	**ISMB 500**	86·9	48·2
1 558·1	***ISWB 450**	79·4	39·1
1 543·2	***ISLB 500**	75·0	43·5
1 350·7	***ISMB 450**	72·4	40·0
1 223·8	***ISLB 450**	65·3	36·8
1 171·3	**ISWB 400**	66·7	32·5
1 022·9	***ISMB 400**	61·6	33·6
965·3	***ISLB 400**	56·9	30·2
887·0	**ISWB 350**	56·9	26·5
778·9	***ISMB 350**	52·4	26·8
754·1	***ISMC 400**	49·4	32·5
751·9	**ISLB 350**	49·5	24·5
699·5	***ISLC 400**	45·7	30·2
654·8	**ISWB 300**	48·1	21·0
607·7	***ISLB 325**	43·1	21·5
573·6	**ISMB 300**	44·2	21·3
571·9	***ISMC 350**	42·1	26·8
532·1	***ISLC 350**	38·8	24·5
488·9	***ISLB 300**	37·7	19·0
475·4	**ISWB 250**	40·9	15·8
424·2	***ISMC 300**	35·8	21·5
410·5	**ISMB 250**	37·3	16·3

(*Contd*)

TABLE I SELECTION OF BEAMS AND CHANNELS USED AS FLEXURAL MEMBERS BASED ON SECTION MODULI — *Contd*

MODULUS OF SECTION (Z_{xx})	DESIGNATION	WEIGHT PER METRE (W)	SHEAR CARRYING CAPACITY (S)
(1)	(2)	(3)	(4)
cm^3		kg	kg × 10^3
403·2	***ISLC 300**	33·1	19·0
392·4	***ISLB 275**	33·0	16·6
348·5	**ISWB 225**	33·9	13·6
305·9	***ISMB 225**	31·2	13·8
305·3	***ISMC 250**	30·4	16·8
297·4	***ISLB 250**	27·9	14·4
295·0	**ISLC 250**	28·0	14·4
262·5	**ISWB 200**	28·8	11·5
239·5	***ISMC 225**	25·9	13·6
226·5	***ISLC 225**	24·0	12·3
226·5	**ISMB 200**	25·4	10·8
222·4	***ISLB 225**	23·5	12·3
181·9	***ISMC 200**	22·1	11·5
172·6	***ISLC 200**	20·6	10·4
172·5	**ISWB 175**	22·1	9·6
169·7	***ISLB 200**	19·8	10·2
145·4	***ISMB 175**	19·3	9·1
139·8	***ISMC 175**	19·1	9·4
131·3	***ISLC 175**	17·6	8·4
125·3	***ISLB 175**	16·7	8·4
116·3	***ISJB 225**	12·8	7·9
116·1	**ISJC 200**	13·9	7·7
111·9	**ISWB 150**	17·0	7·7
103·9	***ISMC 150**	16·4	7·7
96·9	**ISLC 150**	14·9	6·8
93·0	**ISLC 150**	14·4	6·8
91·8	**ISLB 150**	14·2	6·8
82·3	***ISJC 175**	11·2	6·0
78·1	***ISJB 200**	9·9	6·4

(*Contd*)

TABLE I SELECTION OF BEAMS AND CHANNELS USED AS FLEXURAL MEMBERS BASED ON SECTION MODULI —*Contd*

MODULUS OF SECTION (Z_{xx})	DESIGNATION	WEIGHT PER METRE (W)	SHEAR CARRYING CAPACITY (S)
(1)	(2)	(3)	(4)
cm^3		kg	$kg \times 10^3$
71·8	**ISMB 125**	13·0	5·2
66·6	**ISMC 125**	12·7	5·9
65·1	**ISLB 125**	11·9	5·2
62·8	**ISJC 150**	9·9	5·1
57·1	**ISLC 125**	10·7	5·2
54·8	***ISJB 175**	8·1	5·3
51·5	**ISMB 100**	11·5	3·8
43·2	***ISJC 125**	7·9	3·5
42·9	***ISJB 150**	7·1	4·3
37·3	**ISMC 100**	9·2	4·4
33·6	**ISLB 100**	8·0	3·8
32·9	**ISLC 100**	7·9	3·8
24·8	***ISJC 100**	5·8	2·8
20·3	**ISMC 75**	6·8	3·1
19·4	**ISLB 75**	6·1	2·6
17·6	***ISLC 75**	5·7	2·6

NOTE — For using this table, proceed as follows:

a) Locate the required modulus of section (Z_{xx}) in col 1. Where the exact Z_{xx} is not available, select the immediate next higher value of Z_{xx}.

b) The section opposite this value in col 2 and all sections above it satisfy the requirements with regard to Z_{xx}.

c) If the section opposite this value bears an asterisk, it is the highest beam in the series to serve the requirement. Otherwise, proceed higher up and choose the first section bearing the asterisk(*).

d) If conditions require that the section shall not exceed a certain depth, proceed up the column until the required depth is reached. Check up to see that no lighter beam of the same depth appears higher up.

e) Check up the selected section for web capacity in shear. In cases of eccentric loading or any other special conditions of loading, exercise necessary check.

f) It is assumed in this table that compression flanges of the section have adequate lateral support.

TABLE II PERMISSIBLE BENDING STRESS IN COMPRESSION ON UNIFORM CROSS-SECTION

(Metric Units,

(*Clause*

l/b	d/t_f $=10$	d/t_f $=15$	For Every Value of d/t_f Increased by 1, Subtract	d/t_f $=20$	For Every Value of d/t_f Increased by 1, Subtract	d/t_f $=25$	For Every Value of d/t_f Increased by 1, Subtract	d/t_f $=30$	For Every Value of d/t_f Increased by 1, Subtract
20	1 575	1 575	—	1 575	—	1 575	—	1 575	—
21	1 575	1 575	—	1 575	—	1 575	—	1 575	—
22	1 575	1 575	—	1 575	—	1 575	—	1 575	—
23	1 575	1 575	—	1 575	—	1 575	—	1 575	—
24	1 575	1 575	—	1 575	—	1 575	0·4	1 573	20·7
25	1 575	1 575	—	1 575	—	1 575	13·4	1 508	21·7
26	1 575	1 575	—	1 575	—	1 575	25·2	1 449	22·5
27	1 575	1 575	—	1 575	7·6	1 537	31·9	1 378	21·4
28	1 575	1 575	—	1 575	22·6	1 462	31·7	1 303	20·1
29	1 575	1 575	—	1 575	38·6	1 382	29·7	1 233	20·4
30	1 575	1 575	9·6	1 527	46·2	1 296	31·9	1 151	19·0
31	1 575	1 575	30·2	1 424	43·8	1 206	27·9	1 066	17·5
32	1 575	1 575	31·6	1 417	44·2	1 196	27·4	1 059	19·7
33	1 575	1 575	39·4	1 378	43·5	1 160	27·4	1 022	18·9
34	1 575	1 575	48·0	1 335	42·7	1 121	27·4	984	18·0
35	1 575	1 575	57·4	1 288	41·6	1 080	26·0	950	18·2
36	1 575	1 575	67·4	1 238	40·5	1 036	25·7	907	17·3
37	1 575	1 547	69·2	1 201	39·6	1 003	25·6	875	16·5
38	1 575	1 495	66·6	1 161	38·7	968	24·4	846	16·8
39	1 575	1 466	66·0	1 136	38·1	945	24·4	823	17·0
40	1 575	1 410	64·0	1 090	36·9	906	23·9	786	16·1
41	1 575	1 370	62·0	1 060	36·2	879	22·9	764	16·3
42	1 575	1 355	61·8	1 046	35·9	866	23·1	751	15·8
43	1 575	1 313	60·3	1 011	35·0	836	22·7	723	15·1
44	1 575	1 289	59·0	994	34·6	821	22·0	711	15·4
45	1 575	1 242	57·3	955	33·4	788	21·6	680	14·5
46	1 575	1 217	56·5	934	32·9	770	21·4	663	14·7
47	1 575	1 187	54·9	912	32·2	751	20·6	648	14·8
48	1 575	1 158	53·9	888	31·6	730	20·4	628	14·2
49	1 575	1 154	54·1	884	31·5	726	20·6	623	13·9
50	1 575	1 118	52·2	857	30·7	704	19·7	605	13·9

NOTE 1 — The permissible stress for $l/b = 46$, $d/t_f = 27$ may be read as $769·9 - 2 \times 21·4 = 727·1$
The permissible stress for $l/b = 46$, $d/t_f = 28$ may be read as $769·9 - 3 \times 21·4 = 705·7$
The permissible stress for $l/b = 46$, $d/t_f = 29$ may be read as $769·9 - 4 \times 21·4 = 684·3$

THE EXTREME FIBRES OF BEAMS WITH EQUAL FLANGES AND (STEEL CONFORMING TO IS : 226-1958)

kg/cm²)

6.3)

d/t_f =35	For Every Value of d/t_f Increased by 1, Subtract	d/t_f =40	For Every Value of d/t_f Increased by 1, Subtract	d/t_f =45	For Every Value of d/t_f Increased by 1, Subtract	d/t_f =50	For Every Value of d/t_f Increased by 10, Subtract	d/t_f =100	l/b
1 575	—	1 575	—	1 575	—	1 575	—	1 575	20
1 575	—	1 575	—	1 575	—	1 575	2·4	1 558	21
1 575	—	1 575	—	1 575	1·2	1 569	28·4	1 427	22
1 575	7·8	1 536	12·1	1 475	7·9	1 435	27·9	1 295	23
1 469	15·6	1 391	11·2	1 335	7·3	1 298	27·0	1 163	24
1 399	15·0	1 324	9·2	1 278	8·6	1 235	27·4	1 098	25
1 336	14·4	1 264	10·3	1 212	8·2	1 171	27·6	1 033	26
1 271	13·8	1 202	11·3	1 145	7·9	1 106	27·5	968	27
1 203	14·5	1 130	10·7	1 076	7·5	1 039	27·3	902	28
1 131	14·9	1 056	10·1	1 006	7·0	970	26·8	836	29
1 056	13·9	986	9·5	939	7·7	900	26·0	769	30
978	12·9	914	9·9	864	7·2	828	25·0	702	31
961	12·6	897	9·7	848	7·1	813	25·6	684	32
928	13·3	862	9·3	815	6·8	780	25·7	652	33
894	12·7	830	9·0	785	7·6	747	25·5	619	34
858	12·2	797	9·6	749	7·2	713	25·3	586	35
821	12·5	758	9·1	712	6·9	678	24·9	553	36
792	13·0	727	8·7	684	6·6	650	24·8	527	37
762	12·4	700	9·2	654	6·3	623	24·5	500	38
738	11·1	682	8·9	638	6·9	603	24·4	481	39
706	11·3	649	8·5	606	6·6	574	23·9	454	40
683	11·6	625	8·1	584	6·3	553	23·7	434	41
672	11·3	615	8·7	572	6·2	540	23·8	422	42
647	10·9	593	8·3	552	6·6	518	23·4	402	43
634	11·2	578	8·1	538	6·3	506	23·5	389	44
608	11·3	551	7·6	513	6·1	482	22·9	368	45
589	10·3	538	8·0	498	5·9	468	22·7	355	46
574	9·9	524	8·3	482	5·7	454	22·5	342	47
557	10·1	507	7·5	469	6·0	439	22·2	328	48
553	10·5	501	7·3	464	6·0	434	22·3	322	49
535	10·1	485	7·6	447	5·7	419	22·0	309	50

Note 2 — It may be observed that there is a little difference between the values given in this table and those given in Table IIA of IS: 800-1956. The values given in this table have been obtained directly from the formula given in **E-1.1** of Appendix E in IS: 800-1956 and may be taken as more accurate.

TABLE III VALUES

(See Clause

(Clause

l/r_y	d/t_e = 10	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 15	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 20	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 25	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 30	For Every Value of d/t_e Increased by 1, Subtract
						A				
80	—	—	—	—	—	—	—	—	2 865	20·20
81	—	—	—	—	—	—	—	—	2 813	20·20
82	—	—	—	—	—	—	—	—	2 760	20·20
83	—	—	—	—	—	—	—	—	2 708	20·20
84	—	—	—	—	—	—	—	—	2 656	20·20
85	—	—	—	—	—	—	—	—	2 604	20·20
86	—	—	—	—	—	—	—	—	2 551	20·20
87	—	—	—	—	—	—	—	—	2 499	20·20
88	—	—	—	—	—	—	—	—	2 447	20·20
89	—	—	—	—	—	—	—	—	2 394	20·20
90	—	—	—	—	—	—	2 498	31·20	2 342	20·20
91	—	—	—	—	—	—	2 459	31·10	2 304	20·10
92	—	—	—	—	—	—	2 420	31·00	2 265	20·00
93	—	—	—	—	—	—	2 382	30·90	2 227	19·90
94	—	—	—	—	—	—	2 343	30·80	2 189	19·80
95	—	—	—	—	—	—	2 304	30·70	2 150	19·70
96	—	—	—	—	—	—	2 265	30·60	2 112	19·60
97	—	—	—	—	—	—	2 226	30·50	2 074	19·50
98	—	—	—	—	—	—	2 188	30·40	2 036	19·40
99	—	—	—	—	—	—	2 149	30·30	1 997	19·30
100	—	—	—	—	2 362	50·40	2 110	30·20	1 959	19·20
101	—	—	—	—	2 332	50·10	2 082	30·04	1 931	19·14
102	—	—	—	—	2 302	49·80	2 053	29·88	1 904	19·08
103	—	—	—	—	2 272	49·50	2 024	29·72	1 876	19·02
104	—	—	—	—	2 242	49·20	1 996	29·56	1 848	18·96
105	—	—	—	—	2 212	48·90	1 968	29·40	1 820	18·90
106	—	—	—	—	2 182	48·60	1 939	29·24	1 793	18·84
107	—	—	—	—	2 152	48·30	1 910	29·08	1 765	18·78
108	—	—	—	—	2 122	48·00	1 882	28·92	1 737	18·72
109	—	—	—	—	2 092	47·70	1 854	28·76	1 710	18·66

NOTE — The maximum permissible stress of steel to IS: 226-1958 should not exceed 1 500 kg/cm².

TABLES

OF '*A*' AND '*B*'

E-2.1.3 of IS: 800-1956)

6.3)

d/t_e = 35	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 40	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 45	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 50	For Every Value of d/t_e Increased by 10, Subtract	d/t_e = 100	d/t_e = ∞	l/r_y
A									*B*	
2 764	13·80	2 695	9·60	2 647	6·20	2 616	23·40	2 499	2 462	80
2 712	13·78	2 643	9·52	2 595	6·26	2 564	23·32	2 447	2 410	81
2 659	13·76	2 591	9·44	2 543	6·32	2 512	23·24	2 396	2 359	82
2 607	13·74	2 538	9·36	2 492	6·38	2 460	23·16	2 344	2 307	83
2 555	13·72	2 486	9·28	2 440	6·44	2 408	23·08	2 292	2 255	84
2 502	13·70	2 434	9·20	2 388	6·50	2 356	23·00	2 240	2 204	85
2 450	13·68	2 382	9·12	2 336	6·56	2 303	22·92	2 189	2 152	86
2 398	13·66	2 330	9·04	2 284	6·62	2 251	22·84	2 137	2 100	87
2 345	13·64	2 277	8·96	2 233	6·68	2 199	22·76	2 085	2 048	88
2 293	13·62	2 225	8·88	2 181	6·74	2 147	22·68	2 034	1 997	89
2 241	13·60	2 173	8·80	2 129	6·80	2 095	22·60	1 982	1 945	90
2 204	13·44	2 136	8·86	2 092	6·76	2 058	22·54	1 945	1 908	91
2 165	13·28	2 099	8·92	2 054	6·72	2 021	22·48	1 908	1 871	92
2 128	13·12	2 062	8·98	2 017	6·68	1 984	22·42	1 872	1 834	93
2 090	12·96	2 025	9·04	1 980	6·64	1 947	22·36	1 835	1 797	94
2 052	12·80	1 988	9·10	1 942	6·60	1 910	22·30	1 798	1 760	95
2 014	12·64	1 951	9·16	1 905	6·56	1 872	22·24	1 761	1 723	96
1 976	12·48	1 914	9·22	1 868	6·52	1 835	22·18	1 724	1 686	97
1 939	12·32	1 877	9·28	1 831	6·48	1 798	22·12	1 688	1 649	98
1 901	12·16	1 840	9·34	1 793	6·44	1 761	22·06	1 651	1 612	99
1 863	12·00	1 803	9·40	1 756	6·40	1 724	22·00	1 614	1 575	100
1 836	12·02	1 776	9·38	1 729	6·42	1 696	22·00	1 586	1 548	101
1 808	12·04	1 748	9·36	1 701	6·44	1 669	22·00	1 559	1 520	102
1 781	12·06	1 720	9·34	1 674	6·46	1 642	22·00	1 532	1 493	103
1 753	12·08	1 693	9·32	1 646	6·48	1 614	22·00	1 504	1 465	104
1 726	12·10	1 666	9·30	1 619	6·50	1 586	22·00	1 476	1 438	105
1 699	12·12	1 638	9·28	1 592	6·52	1 559	22·00	1 449	1 411	106
1 671	12·14	1 610	9·26	1 564	6·54	1 532	22·00	1 422	1 383	107
1 644	12·16	1 583	9·24	1 537	6·56	1 504	22·00	1 394	1 356	108
1 616	12·18	1 556	9·22	1 509	6·58	1 476	22·00	1 366	1 328	109

(*Contd*)

TABLE III VALUES

(See Clause

(Clause

l/r_y	$d/t_e = 10$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 15$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 20$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 25$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 30$	For Every Value of d/t_e Increased by 1, Subtract
					A					
110	—	—	2 496	86·80	2 062	47·40	1 825	28·60	1 682	18·60
111	—	—	2 470	86·34	2 039	47·16	1 803	28·48	1 661	18·58
112	—	—	2 445	85·88	2 016	46·92	1 781	28·36	1 639	18·56
113	—	—	2 420	85·42	1 992	46·68	1 759	28·24	1 618	18·54
114	—	—	2 394	84·96	1 969	46·44	1 737	28·12	1 596	18·52
115	—	—	2 368	84·50	1 946	46·20	1 715	28·00	1 575	18·50
116	—	—	2 343	84·04	1 923	45·96	1 693	27·88	1 554	18·48
117	—	—	2 318	83·58	1 900	45·72	1 671	27·76	1 532	18·46
118	—	—	2 292	83·12	1 876	45·48	1 649	27·64	1 511	18·44
119	—	—	2 266	82·66	1 853	45·24	1 627	27·52	1 489	18·42
120	3 131	178·00	2 241	82·20	1 830	45·00	1 605	27·40	1 468	18·40
121	3 104	176·86	2 220	81·76	1 811	44·80	1 587	27·26	1 451	18·30
122	3 078	175·72	2 199	81·32	1 793	44·60	1 570	27·12	1 434	18·20
123	3 051	174·58	2 178	80·88	1 774	44·40	1 552	26·98	1 417	18·10
124	3 025	173·44	2 157	80·44	1 755	44·20	1 534	26·84	1 400	18·00
125	2 998	172·30	2 136	80·00	1 736	44·00	1 516	26·70	1 383	17·90
126	2 971	171·16	2 116	79·56	1 718	43·80	1 499	26·56	1 366	17·80
127	2 945	170·02	2 095	79·12	1 699	43·60	1 481	26·42	1 349	17·70
128	2 918	168·88	2 074	78·68	1 680	43·40	1 463	26·28	1 332	17·60
129	2 892	167·74	2 053	78·24	1 662	43·20	1 446	26·14	1 315	17·50
130	2 865	166·60	2 032	77·80	1 643	43·00	1 428	26·00	1 298	17·40
131	2 843	165·60	2 015	77·36	1 628	42·82	1 414	25·88	1 284	17·34
132	2 820	164·60	1 997	76·92	1 613	42·64	1 399	25·76	1 271	17·28
133	2 798	163·60	1 980	76·48	1 597	42·46	1 385	25·64	1 256	17·22
134	2 775	162·60	1 962	76·04	1 582	42·28	1 371	25·52	1 243	17·16
135	2 753	161·60	1 945	75·60	1 567	42·10	1 356	25·40	1 230	17·10
136	2 731	160·60	1 928	75·16	1 552	41·92	1 342	25·28	1 216	17·04
137	2 708	159·60	1 910	74·72	1 537	41·74	1 328	25·16	1 202	16·98
138	2 686	158·60	1 893	74·28	1 521	41·56	1 314	25·04	1 188	16·92
139	2 663	157·60	1 875	73·84	1 506	41·38	1 299	24·92	1 175	16·86

Note — The maximum permissible stress of steel to IS: 226-1958 should not exceed 1 500 kg/cm^2.

TABLES

OF '*A*' AND '*B*'— *Contd*

E-2.1.3 of IS : 800-1956)

6.3)

$d/t_e = 35$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 40$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 45$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 50$	For Every Value of d/t_e Increased by 10, Subtract	$d/t_e = 100$	$d/t_e = \infty$	l/r_y
A									*B*	
1 589	12·20	1 528	9·20	1 482	6·60	1 449	22·00	1 339	1 301	110
1 568	12·16	1 507	9·18	1 461	6·56	1 428	21·98	1 318	1 280	111
1 546	12·12	1 486	9·16	1 440	6·52	1 407	21·96	1 298	1 259	112
1 525	12·08	1 465	9·14	1 419	6·48	1 387	21·94	1 277	1 239	113
1 504	12·04	1 444	9·12	1 398	6·44	1 366	21·92	1 256	1 218	114
1 482	12·00	1 422	9·10	1 377	6·40	1 345	21·90	1 236	1 197	115
1 461	11·96	1 401	9·08	1 356	6·36	1 324	21·88	1 215	1 176	116
1 440	11·92	1 380	9·06	1 335	6·32	1 303	21·86	1 194	1 155	117
1 419	11·88	1 359	9·04	1 314	6·28	1 283	21·84	1 173	1 135	118
1 397	11·84	1 338	9·02	1 293	6·24	1 262	21·82	1 153	1 114	119
1 376	11·80	1 317	9·00	1 272	6·20	1 241	21·80	1 132	1 093	120
1 360	11·72	1 301	9·04	1 256	6·22	1 225	21·70	1 116	1 077	121
1 343	11·64	1 285	9·08	1 239	6·24	1 208	21·60	1 100	1 061	122
1 326	11·56	1 269	9·12	1 223	6·26	1 192	21·50	1 084	1 045	123
1 310	11·48	1 253	9·16	1 207	6·28	1 175	21·40	1 068	1 029	124
1 294	11·40	1 236	9·20	1 190	6·30	1 159	21·30	1 052	1 012	125
1 277	11·32	1 220	9·24	1 174	6·32	1 143	21·20	1 037	996	126
1 260	11·24	1 204	9·28	1 158	6·34	1 126	21·10	1 021	980	127
1 244	11·16	1 188	9·32	1 142	6·36	1 110	21·00	1 005	964	128
1 228	11·08	1 172	9·36	1 125	6·38	1 093	20·90	989	948	129
1 211	11·00	1 156	9·40	1 109	6·40	1 077	20·80	973	932	130
1 198	11·06	1 142	9·28	1 096	6·36	1 064	20·82	960	919	131
1 184	11·12	1 129	9·16	1 083	6·32	1 051	20·84	947	906	132
1 171	11·18	1 115	9·04	1 070	6·28	1 038	20·86	934	893	133
1 157	11·24	1 101	8·92	1 057	6·24	1 025	20·88	921	880	134
1 144	11·30	1 088	8·80	1 044	6·20	1 012	20·90	908	868	135
1 131	11·36	1 074	8·68	1 030	6·16	1 000	20·92	895	855	136
1 117	11·42	1 060	8·56	1 017	6·12	987	20·94	882	842	137
1 104	11·48	1 046	8·44	1 004	6·08	974	20·96	869	829	138
1 090	11·54	1 033	8·32	991	6·04	961	20·98	856	816	139

(Contd)

TABLE III VALUES

(*See Clause*

(*Clause*

l/r_y	d/t_e = 10	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 15	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 20	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 25	For Every Value of d/t_e Increased by 1, Subtract	d/t_e = 30	For Every Value of d/t_e Increased by 1, Subtract
					A					
140	2 641	156·60	1 858	73·40	1 491	41·20	1 285	24·80	1 161	16·80
141	2 622	155·70	1 844	73·02	1 478	40·94	1 274	24·76	1 150	16·72
142	2 603	154·80	1 829	72·64	1 466	40·68	1 262	24·72	1 139	16·64
143	2 584	153·90	1 814	72·26	1 453	40·42	1 251	24·68	1 128	16·56
144	2 565	153·00	1 800	71·88	1 441	40·16	1 240	24·64	1 117	16·48
145	2 546	152·10	1 786	71·50	1 428	39·90	1 228	24·60	1 106	16·40
146	2 527	151·20	1 771	71·12	1 415	39·64	1 217	24·56	1 094	16·32
147	2 508	150·30	1 756	70·74	1 403	39·38	1 206	24·52	1 083	16·24
148	2 489	149·40	1 742	70·36	1 390	39·12	1 195	24·48	1 072	16·16
149	2 470	148·50	1 728	69·98	1 378	38·86	1 183	24·44	1 061	16·08
150	2 451	147·60	1 713	69·60	1 365	38·60	1 172	24·40	1 050	16·00
151	2 434	146·72	1 701	69·26	1 354	38·46	1 162	24·28	1 041	15·98
152	2 418	145·84	1 689	68·92	1 344	38·32	1 152	24·16	1 032	15·96
153	2 401	144·96	1 676	68·58	1 334	38·18	1 143	24·04	1 022	15·94
154	2 385	144·08	1 664	68·24	1 323	38·04	1 133	23·92	1 013	15·92
155	2 368	143·20	1 652	67·90	1 312	37·90	1 123	23·80	1 004	15·90
156	2 351	142·32	1 640	67·56	1 302	37·76	1 113	23·68	995	15·88
157	2 335	141·44	1 628	67·22	1 292	37·62	1 103	23·56	986	15·86
158	2 318	140·56	1 615	66·88	1 281	37·48	1 094	23·44	976	15·84
159	2 302	139·68	1 603	66·54	1 270	37·34	1 084	23·32	967	15·82
160	2 285	138·80	1 591	66·20	1 26C	37·20	1 074	23·20	958	15·80
161	2 271	138·08	1 580	65·88	1 251	37·04	1 066	23·12	950	15·74
162	2 256	137·36	1 570	65·56	1 241	36·88	1 057	23·04	942	15·68
163	2 242	136·64	1 559	65·24	1 233	36·72	1 049	22·96	934	15·62
164	2 228	135·92	1 548	64·92	1 224	36·56	1 041	22·88	926	15·56
165	2 214	135·20	1 538	64·60	1 214	36·40	1 032	22·80	918	15·50
166	2 199	134·48	1 527	64·28	1 205	36·24	1 024	22·72	911	15·44
167	2 185	133·76	1 516	63·96	1 196	36·08	1 016	22·64	903	15·38
168	2 171	133·04	1 505	63·64	1 187	35·92	1 008	22·56	895	15·32
169	2 156	132·32	1 495	63·32	1 178	35·76	999	22·48	887	15·26

NOTE — The maximum permissible stress of steel to IS: 226-1958 should not exceed 1 500 kg/cm².

OF ‘*A*’ AND ‘*B*’ — *Contd*

E-2.1.3 of IS: 800-1956)

6.3)

d/t_e = 35	FOR EVERY VALUE OF d/t_e INCREASED BY 1, SUBTRACT	d/t_e =40	FOR EVERY VALUE OF d/t_e INCREASED BY 1, SUBTRACT	d/t_e = 45	FOR EVERY VALUE OF d/t_e INCREASED BY 1, SUBTRACT	d/t_e = 50	FOR EVERY VALUE OF d/t_e INCREASED BY 10, SUBTRACT	d/t_e = 100	d/t_e = ∞	l/r_y
A									*B*	
1 077	11·60	1 019	8·20	978	6·00	948	21·00	843	803	140
1 066	11·58	1 008	8·22	967	5·96	938	20·98	833	793	141
1 056	11·56	998	8·24	957	5·92	927	20·96	822	782	142
1 045	11·54	987	8·26	946	5·88	916	20·94	812	772	143
1 034	11·52	977	8·28	935	5·84	906	20·92	801	761	144
1 024	11·50	966	8·30	924	5·80	896	20·90	791	751	145
1 013	11·48	955	8·32	914	5·76	885	20·88	781	741	146
1 002	11·46	945	8·34	903	5·72	874	20·86	770	730	147
991	11·44	934	8·36	892	5·68	864	20·84	760	720	148
981	11·42	924	8·38	882	5·64	854	20·82	749	709	149
970	11·40	913	8·40	871	5·60	843	20·80	739	699	150
961	11·34	904	8·34	862	5·64	834	20·74	731	691	151
952	11·28	895	8·28	854	5·68	826	20·68	722	682	152
943	11·22	887	8·22	846	5·72	817	20·62	714	674	153
934	11·16	878	8·16	837	5·76	808	20·56	705	666	154
924	11·10	869	8·10	828	5·80	800	20·50	697	658	155
915	11·04	860	8·04	820	5·84	791	20·44	689	649	156
906	10·98	851	7·98	812	5·88	782	20·38	680	641	157
897	10·92	843	7·92	803	5·92	773	20·32	672	633	158
888	10·86	834	7·86	794	5·96	765	20·26	663	624	159
879	10·80	825	7·80	786	6·00	756	20·20	655	616	160
871	10·76	818	7·78	779	5·96	749	20·22	648	609	161
864	10·72	810	7·76	771	5·92	742	20·24	641	602	162
856	10·68	803	7·74	764	5·88	735	20·26	633	595	163
849	10·64	795	7·72	757	5·84	728	20·28	626	588	164
841	10·60	788	7·70	750	5·80	720	20·30	619	581	165
833	10·56	781	7·68	742	5·76	713	20·32	612	574	166
826	10·52	773	7·66	735	5·72	706	20·34	605	567	167
818	10·48	766	7·64	728	5·68	699	20·36	597	560	168
811	10·44	758	7·62	720	5·64	692	20·38	590	553	169

(*Contd*)

TABLE III VALUES

(*See Clause*
(*Clause*

l/r_y	$d/t_e = 10$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 15$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 20$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 25$	For Every Value of d/t_e Increased by 1, Subtract	$d/t_e = 30$	For Every Value of d/t_e Increased by 1, Subtract
						A				
170	2 142	131·60	1 484	63·00	1 169	35·60	991	22·40	879	15·20
171	2 129	130·92	1 475	62·72	1 161	35·44	984	22·32	872	15·14
172	2 117	130·24	1 466	62·44	1 153	35·28	977	22·24	866	15·08
173	2 104	129·56	1 456	62·16	1 146	35·12	970	22·16	859	15·02
174	2 092	128·88	1 447	61·88	1 138	34·96	963	22·08	853	14·96
175	2 079	128·20	1 438	61·60	1 130	34·80	956	22·00	846	14·90
176	2 066	127·52	1 429	61·32	1 122	34·64	949	21·92	839	14·84
177	2 054	126·84	1 420	61·04	1 114	34·48	942	21·84	833	14·78
178	2 041	126·16	1 410	60·76	1 107	34·32	935	21·76	826	14·72
179	2 029	125·48	1 401	60·48	1 099	34·16	928	21·68	820	14·66
180	2 016	124·80	1 392	60·20	1 091	34·00	921	21·60	813	14·60
181	2 005	124·20	1 384	59·90	1 084	33·94	915	21·34	808	14·68
182	1 994	123·60	1 376	59·60	1 078	33·88	908	21·08	803	14·76
183	1 982	123·00	1 367	59·30	1 071	33·82	902	20·82	798	14·84
184	1 971	122·40	1 359	59·00	1 064	33·76	895	20·56	793	14·92
185	1 960	121·80	1 351	58·70	1 058	33·70	889	20·30	788	15·00
186	1 949	121·20	1 343	58·40	1 051	33·64	883	20·04	782	15·08
187	1 938	120·60	1 335	58·10	1 044	33·58	876	19·78	777	15·16
188	1 926	120·00	1 326	57·80	1 037	33·52	870	19.52	772	15·24
189	1 915	119·40	1 318	57·50	1 031	33·46	863	19·26	767	15·32
190	1 904	118·80	1 310	57·20	1 024	33·40	857	19·00	762	15·40
191	1 894	118·36	1 302	56·86	1 018	33·22	852	19·14	756	15·18
192	1 884	117·92	1 295	56·52	1 012	33·04	847	19·28	750	14·96
193	1 874	117·48	1 287	56·18	1 006	32·86	842	19·42	745	14·74
194	1 864	117·04	1 279	55·84	1 000	32·68	837	19·56	739	14·52
195	1 854	116·60	1 272	55·50	994	32·50	832	19·70	733	14·30
196	1 845	116·16	1 264	55·16	988	32·32	826	19·84	727	14·08
197	1 835	115·72	1 256	54·82	982	32·14	821	19·98	721	13·86
198	1 825	115·28	1 248	54·48	976	31·96	816	20·12	716	13.64
199	1 815	114·84	1 241	54·14	970	31·78	811	20·26	710	13·42
200	1 805	114·40	1 233	53·80	964	31·60	806	20·40	704	13·20

Note — The maximum permissible stress of steel to IS: 226-1958 should not exceed 1 500 kg/cm².

OF '*A*' AND '*B*' — *Contd*

E-2.1.3 of IS: 800-1956)
6.3)

d/t_e = 35	FOR EVERY VALUE OF d/t_e INCREASED BY 1, SUBTRACT	d/t_e =40	FOR EVERY VALUE OF d/t_e INCREASED BY 1, SUBTRACT	d/t_e = 45	FOR EVERY VALUE OF d/t_e INCREASED BY 1, SUBTRACT	d/t_e = 50	FOR EVERY VALUE OF d/t_c INCREASED BY 10, SUBTRACT	d/t_e = 100	d/t_e =∞	l/r_y
A									*B*	
803	10·40	751	7·60	713	5·60	685	20·40	583	546	170
797	10·36	745	7·60	707	5·64	679	20·34	577	540	171
790	10·32	739	7·60	701	5·68	672	20·28	571	534	172
784	10·28	733	7·60	695	5·72	666	20·22	565	528	173
778	10·24	727	7·60	689	5·76	660	20·16	559	522	174
772	10·20	720	7·60	682	5·80	654	20·10	553	516	175
765	10·16	714	7·60	676	5·84	647	20·04	547	511	176
759	10·12	708	7.60	670	5·88	641	19·98	541	505	177
753	10·08	702	7·60	654	5·92	635	19·92	535	499	178
746	10·04	696	7·60	658	5·96	628	19·86	529	493	179
740	10·00	690	7·60	652	6·00	622	19·80	523	487	180
734	10·00	684	7·54	647	5·96	617	19·78	518	482	181
729	10·00	679	7·48	642	5·92	612	19·76	513	477	182
724	10·00	674	7·42	636	5·88	607	19·74	508	472	183
718	10·00	668	7·36	631	5·84	602	19·72	503	467	184
712	10·00	662	7·30	626	5·80	597	19·70	498	462	185
707	10·00	657	7·24	621	5·76	592	19·68	494	456	186
702	10·00	652	7·18	616	5·72	587	19·66	489	451	187
696	10·00	646	7·12	610	5·68	582	19·64	484	446	188
690	10·00	640	7·06	605	5·64	577	19·62	479	441	189
685	10·00	635	7·00	600	5·60	572	19·60	474	436	190
680	9·94	631	7·04	595	5·56	568	19·56	470	432	191
676	9·88	626	7·08	591	5·52	563	19·52	466	428	192
671	9·82	622	7·12	586	5·48	559	19·48	461	423	193
666	9·76	617	7·16	582	5·44	554	19·44	457	419	194
662	9·70	613	7·20	577	5·40	550	19·40	453	415	195
657	9·64	609	7·24	572	5·36	546	19·36	449	411	196
652	9·58	604	7·28	568	5·32	541	19·32	445	407	197
647	9·52	600	7·32	563	5·28	537	19·28	440	402	198
643	9·46	595	7·36	559	5·24	532	19·24	436	398	199
638	9·40	591	7·40	554	5·20	528	19·20	432	394	200

TABLE IV PERMISSIBLE AVERAGE SHEAR STRESS IN WEBS FOR STEEL CONFORMING TO IS: 226-1958

[*See Clause* 9.3.2(b) of IS: 800-1956]

[Average Shear Stress (kg/cm^2) for Different Distances Between Vertical Stiffeners]

(*Design Example* 2, *Sheet* 9)

d/*t*	0·33*d*	0·40*d*	0·50*d*	0·60*d*	0·70*d*	0·80*d*	0·90*d*	1·00*d*	1·10*d*	1·20*d*	1·30*d*	1·40*d*	1·50*d*	*d*/*t*
90 and less	945	945	945	945	945	945	945	945	945	945	945	945	945	90 and less
92	945	945	945	945	945	945	945	945	945	945	943	942	940	92
94	945	945	945	945	945	945	945	945	945	945	942	938	935	94
96	945	945	945	945	945	945	945	945	945	945	940	935	930	96
98	945	945	945	945	945	945	945	945	945	945	939	931	925	98
100	945	945	945	945	945	945	945	945	945	945	937	928	920	100
102	945	945	945	945	945	945	945	945	943	940	931	922	914	102
104	945	945	945	945	945	945	945	945	941	935	925	916	907	104
106	945	945	945	945	945	945	945	945	938	930	919	910	901	106
108	945	945	945	945	945	945	945	945	936	925	913	904	894	108
110	945	945	945	945	945	945	945	945	934	920	907	898	888	110
112	945	945	945	945	945	945	944	941	929	914	901	892	882	112
114	945	945	945	945	945	945	943	937	923	908	896	886	876	114
116	945	945	945	945	945	945	941	934	918	903	890	880	870	116
118	945	945	945	945	945	945	940	930	912	897	885	874	864	118
120	945	945	945	945	945	945	939	926	907	891	879	868	858	120
122	945	945	945	945	945	942	934	921	902	886	873	862	852	122
124	945	945	945	945	945	939	929	916	896	881	867	856	846	124
126	945	945	945	945	945	937	923	911	891	875	861	850	839	126
128	945	945	945	945	945	934	918	906	885	870	855	844	833	128
130	945	945	945	945	945	931	913	901	880	865	849	838	827	130
132	945	945	945	945	942	926	908	896	875	859	843	832	821	132
134	945	945	945	945	939	921	904	891	870	853	838	825	814	134
136	945	945	945	945	937	917	899	886	864	848	832	819	808	136
138	945	945	945	945	934	912	894	881	859	842	827	812	801	138
140	945	945	945	945	931	907	890	876	854	836	821	806	795	140
142	945	945	945	944	927	903	885	871	849	830	815	800	789	142
144	945	945	945	943	923	898	880	866	843	825	809	794	783	144
146	945	945	945	942	918	894	875	860	838	819	803	788	777	146
148	945	945	945	941	914	889	870	855	832	814	797	782	771	148
150	945	945	945	940	910	885	865	850	827	808	791	776	765	150

152	945	945	945	936	908	880	860	845	822	802	785	770	759	152
154	945	945	945	932	905	875	855	840	816	796	779	764	753	154
156	945	945	945	928	903	871	851	835	811	791	774	758	746	156
158	945	945	945	924	900	866	846	830	805	785	768	752	740	158
160	945	945	945	921	888	861	841	825	800	780	762	746	734	160
162	945	945	945	917	884	857	836	820	795	774	756	741	728	162
164	945	945	944	913	880	852	831	815	789	768	750	735	721	164
166	945	945	944	910	876	848	827	810	784	763	744	730	715	166
168	945	945	943	906	872	843	822	805	778	757	738	724	708	168
170	945	945	943	902	868	839	817	800	773	751	732	719	702	170
172	945	945	940	898	864	834	812	795	768	746	726	713	696	172
174	945	945	936	894	859	830	807	790	762	740	721	706	690	174
176	945	945	933	890	855	825	802	785	757	735	715	700	684	176
178	945	945	929	886	850	821	797	780	751	729	710	693	678	178
180	945	945	926	882	846	816	792	775	746	724	704	687	672	180
182	945	945	923	878	842	812	787	—	—	—	—	—	—	182
184	945	945	919	874	838	807	783	—	—	—	—	—	—	184
186	945	945	916	871	833	803	778	—	—	—	—	—	—	186
188	945	945	912	867	829	798	774	—	—	—	—	—	—	188
190	945	945	909	863	825	794	769	—	—	—	—	—	—	190
192	945	945	906	859	821	789	764	—	—	—	—	—	—	192
194	945	945	903	855	816	784	759	—	—	—	—	—	—	194
196	945	945	899	852	812	780	755	—	—	—	—	—	—	196
198	945	945	896	848	807	775	750	—	—	—	—	—	—	198
200	945	945	893	844	803	770	745	—	—	—	—	—	—	200
202	945	943	890	840	799	766	—	—	—	—	—	—	—	202
204	945	941	886	836	795	761	—	—	—	—	—	—	—	204
206	945	938	883	833	791	757	—	—	—	—	—	—	—	206
208	945	936	879	829	787	752	—	—	—	—	—	—	—	208
210	945	934	876	825	783	748	—	—	—	—	—	—	—	210
212	945	931	872	821	779	743	—	—	—	—	—	—	—	212
214	945	928	869	817	774	738	—	—	—	—	—	—	—	214
216	945	926	865	814	770	734	—	—	—	—	—	—	—	216
218	945	923	862	810	765	729	—	—	—	—	—	—	—	218
220	945	920	858	806	761	724	—	—	—	—	—	—	—	220
222	945	917	855	802	757	720	—	—	—	—	—	—	—	222
224	945	915	852	798	753	715	—	—	—	—	—	—	—	224
226	945	912	849	794	748	—	—	—	—	—	—	—	—	226
228	945	910	846	790	744	—	—	—	—	—	—	—	—	228
230	945	907	843	786	740	—	—	—	—	—	—	—	—	230
232	945	904	839	782	736	—	—	—	—	—	—	—	—	232
234	945	901	836	778	731	—	—	—	—	—	—	—	—	234
236	945	899	832	775	727	—	—	—	—	—	—	—	—	236
238	945	896	829	771	722	—	—	—	—	—	—	—	—	238
240	945	893	825	767	718	—	—	—	—	—	—	—	—	240

APPENDIX A

(*Design Example* 14, *Sheet* 1)

CONTINUOUS SPAN COEFFICIENTS

A-1. GENERAL

A-1.1 Continuous spans are frequently used to reduce the maximum moments, in both bridge and building construction; for beams and girders framing to columns in tier buildings they are seldom economical, despite the saving in main material, on account of the added cost of necessary details at the supports.

A-1.2 The methods of calculation of shears and moments in continuous beams proceed from the fundamental, namely the 'Theorem of Three Moments'.

A-1.3 The design of continuous spans may be safely entrusted only to designers with an adequate grasp of the underlying theory and of the behaviour of such structures ; to these, however, it is an advantage to have available such short-cuts as may lighten the tedious arithmetical work.

A-1.4 To this end, two tables of coefficients have been presented for the three-span continuous beams. In these tables, the two end-spans are equal, and again the length of each bears a variety of ratios to the total length.

A-1.5 The following general considerations apply to the use of these tables:

a) The span-ratios chosen are intended to embrace those that frequently occur in practice. The intervals between span-ratios tabulated are close enough so that straight line interpolation for other ratios (vertical interpolation) will not introduce too great errors.

b) Theoretically, the tabulated coefficients for a particular function under investigation are to be used as ordinates to a series of points, through which the 'influence line' for the function is to be drawn in as a smooth curve. The number of such ordinates provided, enables such a curve to be faired in with sufficient accuracy for most purposes.

c) The actual drawing of influence lines may in many cases be avoided by a reasoned use of the tabulated information. For instance, for many short spans the maximum negative and maximum positive moment, directly obtainable from coefficients in the tables, will suffice to determine the size of the required beam.

d) Both spans in two-span beams, and both end spans of three-span beams, are divided into fifths because the maximum positive moments from single loads occur very close to the two-fifths points from the end supports. The central span of three-span beams is divided into fourths because this maximum occurs at mid-span.

e) The spacing of a specified group of concentrated loads is apt to be such that with one load placed at one of the fifth or quarter points tabulated, other loads fall between such points. Exact coefficients for such loads do not result from straight-line interpolation (horizontal interpolation) between the tabulated coefficients to right and to left, because the influence line between those points is a curve. Only in regions of sharp curvature, however, the error is important, and a mental correction to the straight-line interpolation, taking into account the direction of curvature of the influence line, is feasible.

f) All shear and moment coefficients have been expressed in terms of '*L*', the total length of the two (or three) spans. This is done in order that if, as is frequently the case, the total length is fixed and the intermediate span lengths are subject to the designer's discretion, comparison of the various functions for various layouts may be made on a common and constant basis.

A-2. THREE-SPAN TABLES

A-2.1 Table V gives the four reactions due to a unit load placed successively at each of fifteen points. Since the end spans are equal, two of these reactions are in reverse to the other two.

For moving groups of two or more loads, it will usually be desirable to plot the influence lines for all the shears and moments required in the design. The influence line ordinates for the maximum negative moment are tabulated (M_5). Maximum positive moment will occur at an undetermined point, but this point will lie not far from the point where a single load produces maximum moment; the position of this point is tabulated [*see* $+M$(MAX)]. Influence lines may be drawn, from the reaction tables, for this point and for other points close by, and these will envelop the influence line for absolute maximum positive moment in the span.

For longer spans, where changes of section will need to be made, the influence ordinates for moment (and sometimes for shear) may be calculated (from the reaction tables) at each of the fifth points. From these the maximum moment at each fifth point may be found and plotted to scale, and a moment curve faired through the eleven points thus established. This will provide the information for a detailed design for bending stress.

A-2.2 Table VI has been given to simplify the calculation of the shears and moments usually required in the case of uniform load per lineal foot.

Load covering one end span (M_1) produces positive moment throughout that span (except quite close to the intermediate support) and in the other end span to and including its intermediate support; and produces negative moment throughout the centre span (except quite close to the far intermediate support). Load covering the centre span (M_2) produces positive moment throughout that span (except quite close to the intermediate supports) and produces negative moment throughout both end spans to and including the intermediate supports.

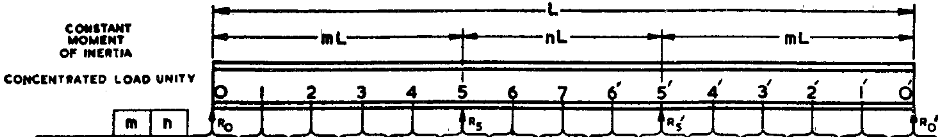
Therefore, load covering all three spans (M_3) does not produce the maximum moment at any point, but the coefficients as tabulated will often be required for the case of dead load.

For uniform live loading, the numerically greatest moment will occur at some points with one span, at some with two adjacent spans, and at some with two end spans, loaded. The coefficients for these moments are tabulated as *Max M*. Inspection will show what combinations of M_1, M_2 and M_1 reversed, produce them. The same is true of Rev M, the greatest moment of opposite sign to *Max M*.

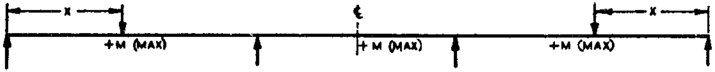
TABLE V THREE-SPAN SYMMETRICAL CONTINUOUS BEAMS

(Coefficients for Concentrated Loads)

(*Clause* A-2.1)



		m	n	0	1	2	3	4	5	6	7	6′	5′	4′	3′	2′	1′	0′
COEFFICIENT × UNIT LOAD P	R_0	0·25	0·50	1·0	0·764	0·537	0·328	0·146	0	−0·188	−0·188	−0·094	0	0·018	0·024	0·021	0·012	0
		0·30	0·40	1·0	0·755	0·522	0·310	0·133	0	−0·108	−0·111	−0·058	0	0·019	0·026	0·022	0·013	0
		1/3	1/3	1·0	0·749	0·510	0·298	0·123	0	−0·072	−0·075	−0·041	0	0·019	0·026	0·022	0·013	0
		3/8	1/4	1·0	0·740	0·495	0·280	0·110	0	−0·039	−0·042	−0·023	0	0·018	0·024	0·021	0·012	0
		0·40	0·20	1·0	0·734	0·485	0·268	0·101	0	−0·025	−0·027	−0·015	0	0·017	0·022	0·019	0·011	0
	R_5	0·25	0·50	0	0·260	0·505	0·720	0·890	1·0	0·984	0·688	0·297	0	−0·054	−0·072	−0·063	−0·036	0
		0·30	0·40	0	0·288	0·554	0·776	0·932	1·0	0·896	0·611	0·271	0	−0·084	−0·112	−0·098	−0·056	0
		1/3	1/3	0	0·315	0·602	0·830	0·973	1·0	0·853	0·575	0·259	0	−0·115	−0·154	−0·134	−0·077	0
		3/8	1/4	0	0·368	0·694	0·936	1·052	1·0	0·812	0·542	0·250	0	−0·180	−0·240	−0·210	−0·120	0
		0·40	0·20	0	0·419	0·784	1·039	1·129	1·0	0·794	0·527	0·247	0	−0·247	−0·329	−0·288	−0·165	0
	R_5'	REVERSE THE TABULATED COEFFICIENTS FOR R_5, FROM LEFT TO RIGHT																
	R_0'	REVERSE THE TABULATED COEFFICIENTS FOR R_0, FROM LEFT TO RIGHT																
COEFFICIENT × PL	M_5	0·25	0·50	0	−0·009	−0·016	−0·018	−0·014	0	−0·047	−0·047	−0·024	0	0·005	0·006	0·005	0·003	0
		0·30	0·40	0	−0·013	−0·024	−0·027	−0·020	0	−0·032	−0·033	−0·017	0	0·006	0·008	0·007	0·004	0
		1/3	1/3	0	−0·017	−0·030	−0·034	−0·026	0	−0·024	−0·025	−0·014	0	0·006	0·009	0·008	0·004	0
		3/8	1/4	0	−0·023	−0·039	−0·045	−0·034	0	−0·015	−0·016	−0·009	0	0·007	0·009	0·008	0·005	0
		0·40	0·20	0	−0·026	−0·046	−0·053	−0·040	0	−0·010	−0·011	−0·006	0	0·007	0·009	0·008	0·004	0
	M_5'	REVERSE THE TABULATED COEFFICIENTS FOR M_5, FROM LEFT TO RIGHT																



	m	n			
+M (MAX)	0·25	0·50	0·054 AT X = 0·449 (0·25) L	0·078	0·054 AT X = 0·449 (0·25) L
	0·30	0·40	0·063 AT X = 0·437 (0·30) L	0·067	0·063 AT X = 0·437 (0·30) L
	1/3	1/3	0·068 AT X = 0·428 (1/3) L	0·058	0·068 AT X = 0·428 (1/3) L
	3/8	1/4	0·074 AT X = 0·415 (3/8) L	0·047	0·074 AT X = 0·415 (3/8) L
	0·40	0·20	0·078 AT X = 0·407 (0·40) L	0·039	0·078 AT X = 0·407 (0·40) L

R_0, R_5, R_5' and R_0' are the reactions at supports 0, 5, 5′ and 0′ respectively, for a concentrated load of unity applied at the point indicated at the head of each column of coefficients.

From these reactions it is possible to construct the influence lines for maximum shear or maximum moment at any section.

M_5 is the moment at the intermediate support R_5, due to unit load placed at the point indicated. It is negative except when the load is placed on the farther end span. The tabulated moment coefficients constitute ordinates to the influence line for moment at 5. For M_5' they are reversed from left to right. Maximum moment at either interior support will occur with the farther end span unloaded. The total moment at 5 or 5′, due to two or more concentrated loads is the algebraic sum of the coefficients tabulated above for the points at which the several loads are placed. Usually it is greatest when they are placed in the longer of the two spans adjacent to the support.

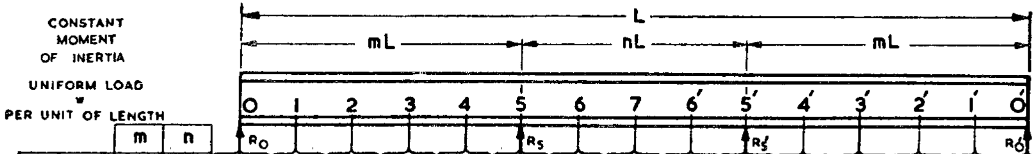
$+M$ (MAX) defines the load position for maximum positive moment, and gives the moment coefficient, in each span respectively. The information accurately locates the peak of the influence line for maximum positive moment due to a single load.

Coefficient for span-ratios m, n, not given, may be approximated by direct interpolation between the two nearest values tabulated.

TABLE VI THREE-SPAN SYMMETRICAL CONTINUOUS BEAMS

(Coefficients for Uniformly Distributed Loads)

(*Clause* A-2.2)



		m	n	0	1	2	3	4	5	6	7	6'	5'	4'	3'	2'	1'	0'
COEF × wL	MAX SHEAR	0·25	0·50	0·1172				-0·2992	0·2578		(0·4570 MAX R_5							
		0·30	0·40	0·1375				-0·1971	0·2169		(0·4140							
		1/3	1/3	0·1500				-0·2056	0·1944		(0·4000							
		3/8	1/4	0·1641				-0·2237	0·1777		(0·4014							
		0·40	0·20	0·1714				-0·2379	0·1800		(0·4179							

NOTE—PREFIX 0·0 TO ALL TABULATED MOMENTS; THUS, 044 SIGNIFIES 0·0044

		m	n	0	1	2	3	4	5	6	7	6'	5'	4'	3'	2'	1'	0'
COEFFICIENTS × 0·0 wL^2	M_1	0·25	0·50	0	044	063	057	027	- 029	- 020	- 010	000	010	008	006	004	002	0
		0·30	0·40	0	062	087	077	030	- 053	- 036	- 019	- 002	015	012	009	006	003	0
		1/3	1/3	0	074	104	089	030	- 074	- 051	- 028	- 005	019	015	011	007	004	0
		3/8	1/4	0	091	125	103	025	- 110	- 077	- 044	- 011	022	018	013	009	004	0
		0·40	0·20	0	101	137	110	018	- 137	- 097	- 057	- 017	023	018	014	009	005	0
	M_2	0·25	0·50	0	- 031	- 063	- 094	- 125	- 156	078	156	078	- 156	- 125	- 094	- 063	- 031	0
		0·30	0·40	0	- 018	- 036	- 053	- 071	- 089	061	111	061	- 089	- 071	- 053	- 036	- 018	0
		1/3	1/3	0	- 011	- 022	- 033	- 044	- 056	049	083	049	- 056	- 044	- 033	- 022	- 011	0
		3/8	1/4	0	- 005	- 010	- 016	- 021	- 026	033	052	033	- 026	- 021	- 016	- 010	- 005	0
		0·40	0·20	0	- 003	- 006	- 009	- 011	- 014	023	036	023	- 014	- 011	- 009	- 006	- 003	0
	M_3	0·25	0·50	0	015	005	- 030	- 091	- 176	059	137	069	- 176	- 091	- 030	005	015	0
		0·30	0·40	0	047	057	032	- 029	- 126	024	074	024	- 126	- 029	032	057	047	0
		1/3	1/3	0	067	089	067	000	- 111	- 007	028	- 007	- 111	000	067	089	067	0
		3/8	1/4	0	090	123	100	021	- 114	- 055	- 036	- 055	- 114	021	100	123	090	0
		0·40	0·20	0	102	141	115	025	- 129	- 091	- 079	- 091	- 129	025	115	144	102	0
	MAX M	0·25	0·50	0	046	067	- 094	- 125	- 186	078	156	078	- 186	- 125	- 094	067	046	0
		0·30	0·40	0	065	093	086	- 071	- 141	061	111	061	- 141	- 071	086	093	065	0
		1/3	1/3	0	078	111	100	044	- 130	- 056	083	- 056	- 130	044	100	111	078	0
		3/8	1/4	0	095	134	116	042	- 136	- 088	- 088	- 088	- 136	042	116	134	095	0
		0·40	0·20	0	105	146	123	037	- 151	- 114	- 114	- 114	- 151	037	123	146	105	0
	REV M	0·25	0·50	0	- 031	- 063	063	034	010	- 020	- 020	- 020	010	034	063	- 063	- 031	0
		0·30	0·40	0	- 018	- 036	- 053	042	015	- 038	- 038	- 038	015	042	- 053	- 036	- 018	0
		1/3	1/3	0	- 011	- 022	- 033	- 044	019	049	- 056	049	019	- 044	- 033	- 022	- 011	0
		3/8	1/4	0	- 005	- 010	- 016	- 021	022	033	052	033	022	- 021	- 016	- 010	- 005	0
		0·40	0·20	0	- 003	- 006	- 009	- 011	023	023	036	028	023	- 011	- 009	- 006	- 003	0

Maximum shear (× wL) is the maximum shear on the indicated side of the support, due to uniform load of w per lin m in the most effective position for shear.

M_1 and M_2 (× wL^2) are the moments at the indicated points due to uniform load w covering respectively the left and the centre span. (Moments from load covering the right hand span are the reverse from left to right of M_1 and are not tabulated.)

M_3 = moment at the indicated point due to load covering all spans; which is not a condition for maximum.

Max M = maximum possible moment of either sign at the indicated point, and is due to uniform load covering one complete span or two complete spans. The maximum possible positive moment occurs close to, and is negligibly greater than, that shown at Points 2 and 2'.

Rev M = maximum moment of reverse sign to *Max M*.

APPENDIX B

(*See* Foreword)

INDIAN STANDARDS ON PRODUCTION, DESIGN AND USE OF STEEL IN STRUCTURES

ISI has so far published the following Indian Standards in the field of production, design and utilization of steel and welding:

IS: 800-1956 Code of Practice for Use of Structural Steel in General Building Construction

IS: 801-1958 Code of Practice for Use of Cold Formed Light Gauge Steel Structural Members in General Building Construction

IS: 804-1958 Specification for Rectangular Pressed Steel Tanks

IS: 806-1957 Code of Practice for Use of Steel Tubes in General Building Construction

IS: 808-1957 Specification for Rolled Steel Beam, Channel and Angle Sections

IS: 812-1957 Glossary of Terms Relating to Welding and Cutting of Metals

IS: 813-1961 Scheme of Symbols for Welding (***Amended***)

IS: 814-1957 Specification for Covered Electrodes for Metal Arc Welding of Mild Steel

IS: 815-1956 Classification and Coding of Covered Electrodes for Metal Arc Welding of Mild Steel and Low Alloy High-Tensile Steels

IS: 816-1956 Code of Practice for Use of Metal Arc Welding for General Construction in Mild Steel

IS: 817-1957 Code of Practice for Training and Testing of Metal Arc Welders

IS: 818-1957 Code of Practice for Safety and Health Requirements in Electric and Gas Welding and Cutting Operations

IS: 819-1957 Code of Practice for Resistance Spot Welding for Light Assemblies in Mild Steel

IS: 1173-1957 Specification for Rolled Steel Sections, Tee Bars

IS: 1179-1957 Specification for Equipment for Eye and Face Protection During Welding

IS: 1181-1957 Qualifying Tests for Metal Arc Welders (Engaged in Welding Structures Other Than Pipes)

IS: 1182-1957 General Recommendations for Radiographic Examination of Fusion Welded Joints

IS: 1252-1958 Specification for Rolled Steel Sections, Bulb Angles

IS: 1261-1959 Code of Practice for Seam Welding in Mild Steel

IS: 1278-1958 Specification for Filler Rods and Wires for Gas Welding

IS: 1323-1959 Code of Practice for Oxy-Acetylene Welding for Structural Work in Mild Steel

IS: 1395-1959 Specification for ½-Percent Molybdenum Steel Covered Electrodes for Metal Arc Welding

IS: 1442-1959 Specification for Covered Electrodes for the Metal Arc Welding of High Tensile Structural Steel

APPENDIX C

(*See* Foreword)

COMPOSITION OF STRUCTURAL ENGINEERING SECTIONAL COMMITTEE, SMDC 7

The ISI Structural Engineering Sectional Committee, SMDC 7, which was responsible for processing this Handbook, consists of the following:

Chairman	
Director Standards (Civil)	Railway Board (Ministry of Railways)
Members	
Shri P. Balakrishnan	Public Works Department, Madras
Shri D. I. Paul (*Alternate*)	
Shri B. N. Bannerjee	Bridge & Roof Co. (India) Ltd., Calcutta
Shri Raghudas Baul	Public Works Department, Calcutta
Col G. Benjamin	Engineer-in-Chief's Branch, Army Headquarters
Shri R. S. Mehandru (*Alternate*)	
Shri J. G. Bodhe	K. R. Irani & Co., Bombay
Shri D. S. Desai	Institution of Engineers (India), Calcutta
Mr. F. J. Fonseca	Richardson & Cruddas Ltd., Bombay
Mr. W. Fernandes (*Alternate*)	
Joint Director Standards (B & S)	Railway Board (Ministry of Railways)
Deputy Director Standards (B & S) (*Alternate*)	
Shri S. C. Kapur	Central Public Works Department, New Delhi
Shri C. P. Malik	National Buildings Organization (Ministry of Works, Housing & Supply)
Shri Shri Krishna (*Alternate*)	
Shri L. R. Marwadi	Hindustan Construction Co. Ltd., Bombay
Shri P. S. Mehta	New Standard Engineering Co. Ltd., Bombay
Shri B. N. Mozumdar	Inspection Wing, Directorate General of Supplies & Disposals (Ministry of Works, Housing & Supply)
Shri P. L. Das (*Alternate*)	
Shri Y. K. Murthy	Central Water & Power Commission (Water Wing), New Delhi
Shri M. P. Nagarsheth	Ministry of Transport & Communications (Roads Wing)
Shri C. M. Shahani	Braithwaite, Burn & Jessop Construction Co. Ltd., Calcutta
Shri Sarup Singh	Committee on Plan Project, Planning Commission, New Delhi
Shri T. S. Vedagiri (*Alternate*)	
Shri D. S. Thakur	Bombay Municipal Corporation, Bombay
Shri A. R. Vainganjkar (*Alternate*)	
Maj R. P. E. Vazifdar	Bombay Port Trust, Bombay
Shri V. Venugopalan	Central Water & Power Commission (Power Wing), New Delhi
Shri S. S. Murthy (*Alternate*)	
Dr. Lal C. Verman (*Ex-officio*)	Director, Indian Standards Institution
Shri B. S. Krishnamachar (*Alternate*)	Deputy Director (S & M), Indian Standards Institution
Secretary	
Shri H. N. Krishnamurthy	Assistant Director (S & M), Indian Standards Institution