

Steel structures design - I

Syllabus

- 1-Introduction
- 2-Limit states for steel design
- 3-Working and factored loads
- 4-Design methods for buildings
- 5-Material properties and specifications
- 6-Design of beams
 - General
 - Laterally supported beams.
 - Laterally unsupported beams.
 - Design for flexure
 - Design for shear
 - Web buckling
 - Beams subjected to biaxial bending
 - Compound beams
- 7-Design of gantry girders (crane beams)
 - Moment capacity
 - Shear capacity
- 8- Connections
 - a- Bolted connections
 - Direct shear joints
 - Bolts in direct shear and torsion
 - Bolts in direct shear and tension
 - b - Welded connections
 - Design of fillet welds
 - Design of butt welds

References

- 1-Structural steelwork design to limit state theory. By D. Lam
- 2- BS 5950: part-1 :2000
- 3- Steelwork design guide to BS 5950-1-2000

Steel structures :

Steel frame building consist of a skeletal framework which carries all the load to which the building is subjected.

The aim of structural design should be to provide, with due regard to economy, a structure capable of fulfilling its intended function and sustaining the specified loads for its intended life.

Structural elements

- 1- Beams and girders : members carrying lateral loads in bending and shear
- 2- Ties : members carrying axial loads in tension
- 3- Struts , columns or stanchions : members carrying axial loads in compression . They are often subjected to bending as well as compression.
- 4- Trusses and lattice girders : framed members carrying lateral loads, and they are composed of struts and ties.
- 5- Purlins - beam members carrying roof sheeting
- 6- sheeting rails : beam members supporting wall cladding

7- Bracing - diagonal struts and ties used with other structural elements to resist wind loads.
(see p.3, Lam)

Steps of structural design : For a given structure the steps of design are :

- 1- estimation of loading
- 2- analysis of the structure to determine axial loads, shears and moments at critical points in all members.
- 3- design of the elements and connections
- 4- production of arrangement and detail drawings.

Design theories

Steel design may be based on three design theories

- 1) elastic design
- 2) plastic design
- 3) limit state design

In the elastic design method, the structures are analysed by elastic theory and sections are sized so that the permissible stresses are

not exceeded.

- * Plastic theory developed to take account of behaviour past the yield point and is based on finding the load that causes the structure to collapse. Then the working is the collapse load divided by a load factor.
- * limit state design has been developed to take account of all conditions that can make the structure become unfit for use. The design is based on the actual behaviour of materials and structures in use.

Limit states are the states beyond which the structure becomes unfit for its intended use.

Limit states for steel design

The limit states for which steelwork is to be designed as specified by BS 5950 are:

1- Ultimate limit states

include the following:

- a- Strength (including general yielding, rupture, buckling and transformation into mechanism).
- b- Stability against overturning and sway
- c- fracture due to fatigue
- d- brittle fracture.

When the ultimate limit states are exceeded, the whole structure or part of it collapses.

2- Serviceability limit states

consist of the following:

- a- deflection
- b- vibration
- c- repairable damage due to fatigue
- d- corrosion and durability.

The serviceability limit states, when exceeded, make the structure or part of it unfit for normal use but do not indicate that collapse has occurred.

All relevant limit states should be considered, but it is appropriate to design on the basis of strength and stability at ultimate loading and then check that deflection is not excessive under serviceability loading.

Working and factored loads

a - Working loads

The working loads (also known as the specified, characteristic or nominal loads) are the actual loads the structure is designed to carry.

The main loads on buildings are:

- 1 - dead loads : These are the weight of the structure's elements
- 2 - imposed loads : loads caused by people, furniture, equipment, etc. on the floors of building. The values of the floor loads used depend on the use of the building. Imposed loads are given in BS 6399 Part 1
- 3 - Wind loads : depend on the location and building size. Wind loads are given in BS 6399 Part 2
- 4 - Dynamic loads : these are caused mainly by cranes. Dynamic loads from cranes are given in BS 6399 Part 1.

b. Factored loads for the ultimate limit states

According to BS 5950, factored loads are used in design calculation for strength and stability.

Factored load = Working load \times relevant partial load factor, γ_f

The partial load factors are given in Table 2 BS 5950

<u>loading</u>	<u>load factor γ_f</u>
dead load	1.4
dead load restraining uplift or overturning	1.0
dead + imposed + wind	1.2
imposed	1.6
wind	1.4

Serviceability limit state deflection

Deflection is the main serviceability limit state that must be considered in design. The serviceability loads are the unfactored specified values.

The structure is considered to be elastic and the most adverse combination of loads is assumed.

Deflection limits are given in Table 8 of BS 5950

a) Vertical deflection of beams

due unfactored imposed loads

- cantilever length / 180
- beams carrying plaster span / 360
- all other beams span / 200

b) Horizontal deflection of columns due to unfactored

imposed and wind loads

- top of column in single-storey buildings height / 300
- in each storey of a building with more than one storey, storey height / 300

c) Crane gantry girders

- - -
- - -

Design methods for building

According to BS 5950, the design of buildings must be carried out in accordance with one of the following methods

1) Simple design

It is assumed that no moment is transferred from one connected member to another.

The distribution of forces may be determined assuming that members intersecting at a joint are pin connected, the structure should be laterally supported to provide sway stability.

2. Continuous design

It is assumed that joints are rigid and transfer moment between members

3. Semi-continuous design

True semi-continuous design is more complex than simple or continuous design because the real joint response is more realistically represented. The joints are assumed to have some degree of strength and stiffness.

Materials

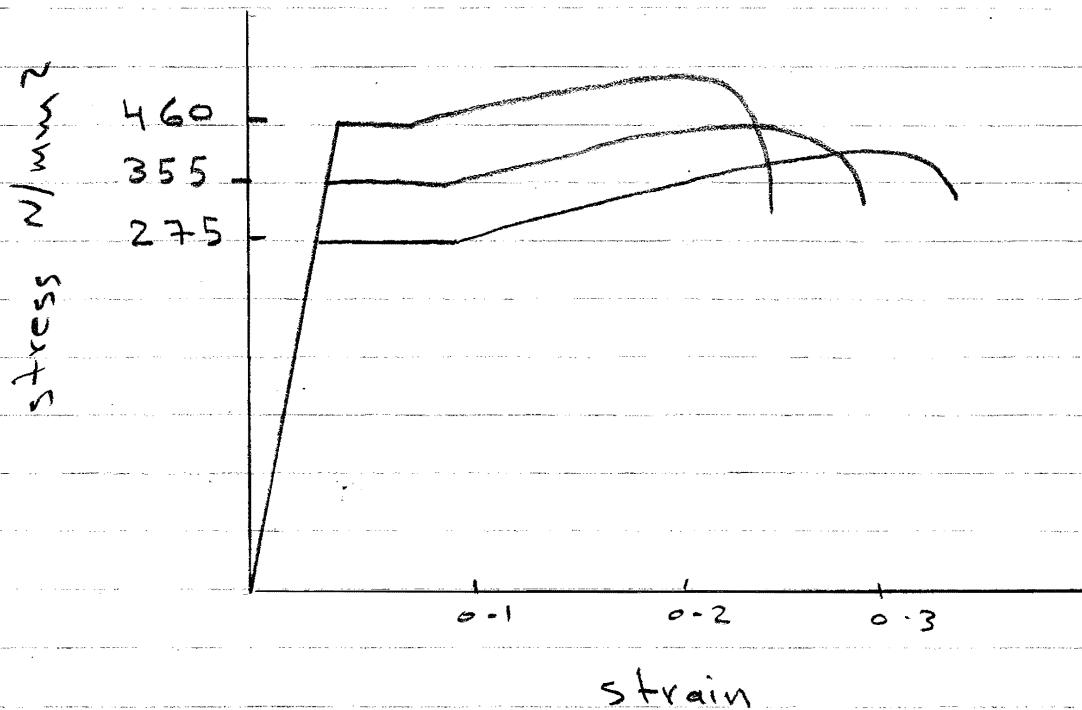
Steel as a structural material exhibits a number of useful properties such as

- linear elastic up to initial yield.
- deformations are directly proportional to applied load and are fully recovered on removal of the loading.
- the yield strength in tension is approximately equal to the crushing strength in compression.
- isotropic behaviour
- long-term deformation (creep) at normal temperatures is not a problem

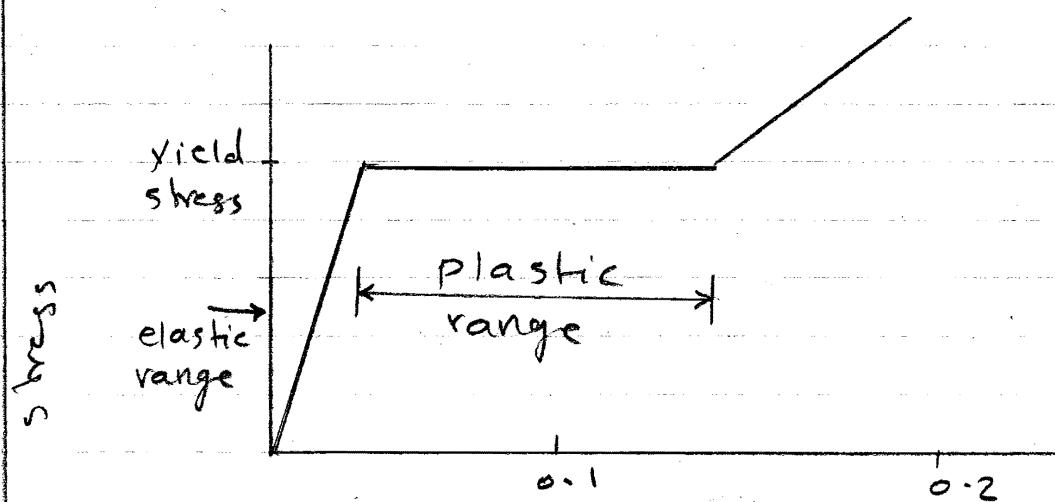
Steel is composed of about 98 % of iron with the main alloying elements carbon, silicon, and manganese. Copper and chromium are added to produce the weather-resistant steels that do not require corrosion protection

Structural steel is basically produced in three strength grades S275, S355 and S460. The important design properties are strength, ductility, impact resistance, and weldability.

stress - strain curve



Stress - strain curve for structural steel



Stress - strain curve for plastic design

Design strength

The design strengths of the three grades of steel are given in Table 9 of BS 5950-1.

The design strength may be taken as

$$P_y = 1.0 \gamma_s \text{ but not greater than } U_s / 1.2$$

P_y : design strength

γ_s : minimum yield strength

U_s : minimum ultimate tensile strength

steel grade	thickness (mm) less or equal to	P_y (N/mm ²)
S275	16	275
	40	265
	63	255
S355	16	355
	40	345
	63	335
S460	16	460
	40	440
	63	430

other properties

- modulus of elasticity = $205 \times 10^3 \text{ N/mm}^2$
- shear modulus $G = E/2(1+\nu)$
- Poisson's ratio $\nu = 0.30$
- Coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6}$ per $^\circ\text{C}$

steel sections

a- Rolled and formed sections

see

- 1- Universal beams : very efficient sections for resisting bending moment about major axis
- 2- Universal columns : resist axial load with high radius of gyration about the minor axis to prevent buckling in that plane
- 3- Channels : used for beams, bracing members, truss members and in compound
- 4- Equal and unequal angles : used for bracing members, truss members and for purlins.

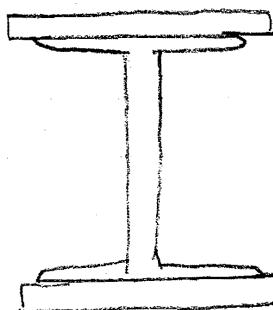
5- other sections : T , hollow sections, m



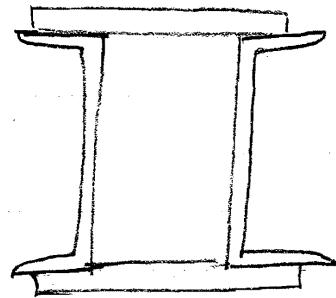
b. Compound sections

can be formed by the following means:

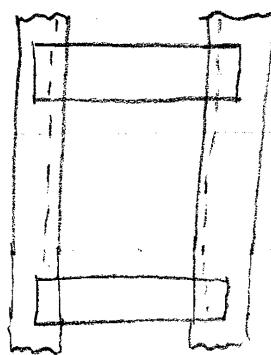
- 1- strengthening a rolled section with a cover plate
- 2- combining two rolled sections
- 3- connecting two members together to form a strong combined member



(1)



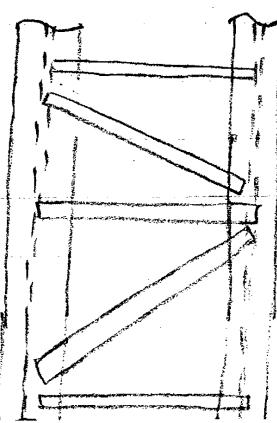
(2)



(3)

battened member

(3)
laced
member



c - built-up sections

are made by welding plates together to form I, H, or box members. These members are used where heavy loads have to be carried or where long spans may be required.

Section properties

The section properties are listed in tables of dimensions and cross section properties.

These properties include dimensions, centroid location, cross-sectional area, moment of inertia, radius of gyration, moduli of section, ...

Plastic modulus (S) of a section is equal to the algebraic sum of the first moments of area about the equal area axis.

$$\text{Elastic modulus, } Z = \frac{I}{y}, \quad y = \frac{D}{2} \text{ for I-section}$$

$$\text{Radius of gyration, } r = \sqrt{I/A}$$

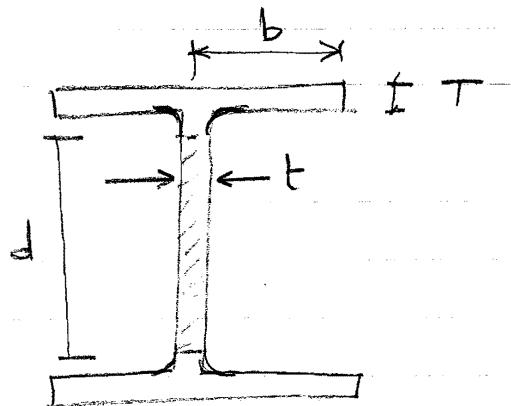
Design of Beams

The design of beams to satisfy the requirements of BS 5950-1 includes the consideration of:

- section classification
- shear capacity
- moment capacity
- deflection
- web buckling
- web bearing

Section classification

The classification is based on the aspect ratio of the elements of cross-section



b/t ratio for outstand of compression flange

d/t ratio for the web

see tables 11 and 12 of BS 5950

$$\text{in which } \epsilon = (275/\rho_y)^{0.5}$$

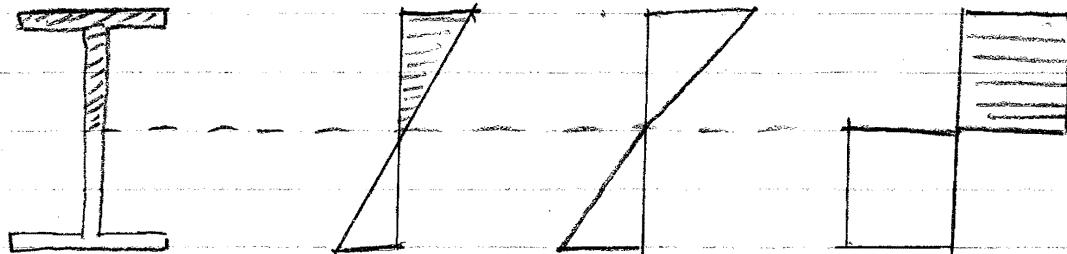
class 1, Plastic cross section : This can develop a plastic hinge with sufficient rotation capacity to permit redistribution of moments in the structure.

Class 2 Compact cross section : This can develop the plastic moment capacity but local buckling prevents rotation at constant moment.

class 3 semi-compact cross-section : The stress in the extreme fibres should be limited to the yield stress because local buckling prevents development of plastic moment capacity

class 4 Slender cross-section : Premature buckling occurs before yield is reached

compression $\sigma < P_y$ $\sigma = P_y$ $\sigma = P_y$



tension

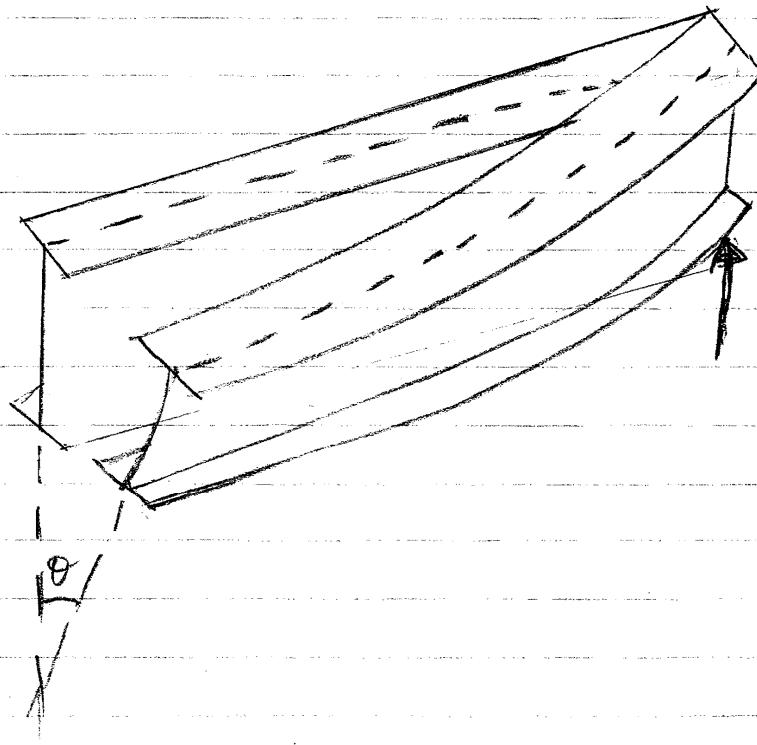
slender semi-compact plastic and
compact

The majority of hot rolled I and H sections are classified as plastic and are therefore suitable for plastic design

(from Handbook P33)

Lateral-torsional buckling

If a beam section is subjected to vertical loading that can move laterally with the beam, the imperfections of the beam mean it will tend to distort as shown.



One half of a simply supported beam

Fully restrained beams

Lateral torsional buckling is inhibited by the provision of lateral restraints to the compression flange.

The continuous restraint should be designed to resist a force of 2.5% of the max. force in the compression flange, this restraining force may be assumed to be uniformly distributed along the compression flange.

Shear capacity

The web of the section carries the applied shear force.

The shear capacity of a beam is defined in the code as

$$P_v = 0.6 P_y A_v$$

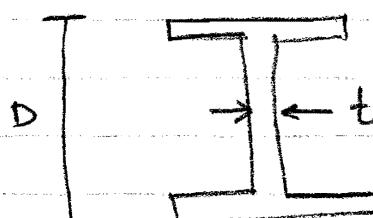
where

P_y = design strength

A_v = shear area

for I, H, E section, $A_v = D t$

for the other sections see BS 5950 (clause 4.2.3)
The applied shear F_v should be less than P_v



Moment capacity (M_c)

The moment capacity of the cross-section is affected by the value of the applied shear force (F_v)

a. low shear $F_v < 0.6 P_v$

- for class 1 plastic or class 2 compact sections

$$M_c = p_y \cdot S$$

- for class 3 semi-compact sections

$$M_c = p_y Z \text{ or alternatively } M_c = p_y S_{eff}$$

- for class 4 slender cross-sections

$$M_c = p_y Z_{eff}$$

where

S = the plastic modulus

S_{eff} = the effective plastic modulus
see 3-5-6

Z = the section modulus

Z_{eff} = the effective section modulus, see 3-6-2

b- High shear $F_v > 0.6 P_v$

- for class 1 plastic or class 2 compact sections

$$M_c = P_y (S - \rho S_v)$$

- for class 3 semi-compact sections,

$$M_c = P_y \left(Z - \frac{\rho S_v}{1.5} \right) \text{ or alternatively}$$

$$M_c = P_y (S_{eff} - \rho S_v)$$

- for class 4 slender sections

$$M_c = P_y \left(Z_{eff} - \frac{\rho S_v}{1.5} \right)$$

where

S_v = the plastic modulus of the shear area A_v

$$\rho = \left[2 \left(\frac{F_v}{P_v} \right) - 1 \right]^2$$

Note

For both low and high shear cases

M_c should be limited to $1.5 P_y Z$ generally and to $1.2 P_y Z$ for simply

supported beam or a cantilever

Span of a beam : distance between points of effective support. In general unless the supports are wide columns, the span can be considered as to center-to-center of the actual supports or columns.

Shape factor (γ) : The shape factor of a section is defined as:

$$\gamma = \frac{\text{Plastic modulus}}{\text{Elastic modulus}} = \frac{S_{xx}}{Z_{xx}}$$

The value of γ for most I-sections ≈ 1.15

Ex

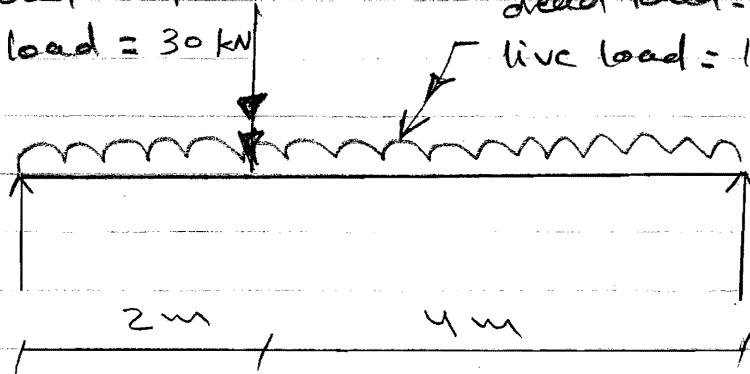
A simply supported laterally restrained beam carries the working loads shown. Select a suitable section considering section classification, shear, and bending. Assume dead load is inclusive of self-weight. Use grade S275 steel.

$$\text{dead load} = 10 \text{ kN}$$

$$\text{live load} = 30 \text{ kN}$$

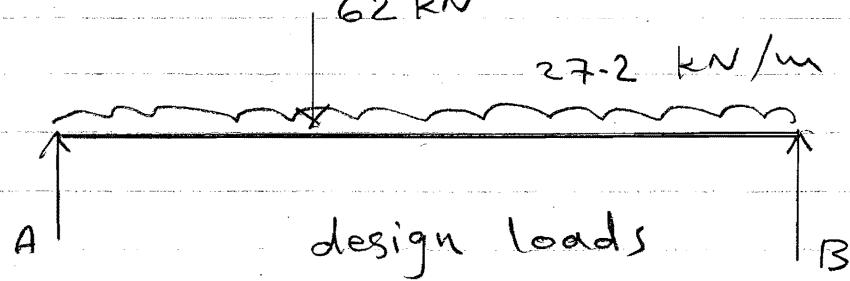
$$\text{dead load} = 8 \text{ kN/m}$$

$$\text{live load} = 10 \text{ kN/m}$$



$$62 \text{ kN}$$

$$27.2 \text{ kN/m}$$



$$122.9$$

$$68.5$$

$$6.5$$

$$2.24$$

$$102.3$$

S-F.D (kN)

Design point load = $(1.4 \times 10) + (1.6 \times 30) = 62 \text{ kN}$
 Design UDL = $(1.4 \times 8) + (1.6 \times 10) = 27.2 \text{ kN/m}$

$$R_A = 122.9 \text{ kN}$$

$$R_B = 102.3 \text{ kN}$$

position of zero shear = 2.24 m from A
 max. bending moment occurs at position
 of zero shear

$$M_{\max} = 192.2 \text{ kNm}$$

Assume low shear

use S275 steel

Assume thickness < 16 mm

$$\therefore f_y = 275 \text{ N/mm}^2$$

$$\text{plastic modulus } S_{xx} = \frac{M}{f_y} = \frac{192.2 \times 10^6}{275} \\ = 698.9 \times 10^3 \text{ mm}^3$$

A trial beam size can be selected from section tables

Try 305 x 165 x 46 UB

$$D = 306.6 \text{ mm}, d = 265.2 \text{ mm}, B = 165.7 \text{ mm}$$

$$T = 11.8 \text{ mm}, t = 6.7 \text{ mm}, b/T = 7.02$$

$$d/t = 39.6 \text{ mm}, S_{xx} = 720 \times 10^3 \text{ mm}^3$$

$$Z_{xx} = 646 \times 10^3 \text{ mm}^3$$

i. section classification

$$\epsilon = \left(\frac{275}{P_y} \right)^{1/2} = 1.0$$

Flange : outstand element of compression
flange rolled section

$$b/t = 7.02 < 9.0 \epsilon \therefore \text{Flange is plastic}$$

Web : bending only with neutral axis at mid-depth

$$d/t = 39.6 < 80 \epsilon \therefore \text{web is plastic}$$

\therefore section is plastic.

ii. shear

$$F_v = 122.9 \text{ kN}$$

$$P_v = 0.6 P_y A_v = 0.6 P_y t D$$

$$= 0.6 \times 275 \times 6.7 \times 306.6 \times 10^{-3} = 338.9 \text{ kN}$$

$$\therefore P_v > F_v$$

the section is adequate in shear

iii) Bending

Check the assumed low shear

$$0.6 P_v = 0.6 \times 338.9 = 203.3 \text{ kN} > 122.9 \text{ kN}$$

\therefore low shear

$$M_c = p_y S \leq 1.2 p_y Z \quad \text{simply supported beam}$$

$$p_y S = 275 \times 720 \times 10^3 = 198 \times 10^6 \text{ N-mm}$$

$$1.2 p_y Z = 1.2 \times 275 \times 646 \times 10^3 = 213.18 \times 10^6 \text{ N-mm}$$

$$M_c = 198 \text{ kNm} > M = 192.2 \text{ kNm}$$

section is adequate in bending

Beams without full lateral restraint

When lateral-torsional buckling is possible, the resistance of the beam to bending will be reduced by its tendency to buckle.

The following should be satisfied

$$M_x \leq M_{cx} \text{ and } M_x \leq M_b / m_{LT}$$

where

- M_x : the applied max. major axis moment
- M_{cx} : major axis moment capacity (Cl. 4.2.5)
- M_b : the buckling resistance moment
- m_{LT} : equivalent uniform moment factor
(Cl. 4.3.6.6)

The value of M_b depends on determination of a bending strength P_b (generally less than material design strength P_y). Values of P_b may be obtained from Table 16 of BS 5950 depending on the value of the equivalent slenderness λ_{LT} .

Equivalent slenderness λ_{LT}

$$\lambda_{LT} = u \sqrt{\lambda} \sqrt{\beta_w}$$

The main parameter in this expression is λ

$$\lambda = L_E/r_y$$

L_E = effective length

r_y = radius of gyration

λ = slenderness factor may be determined from table 19.

For equal flange beams λ may safely be taken as 1.0 (handbook ref)

u = buckling parameter, may be found from section tables. For rolled

I and H sections u may safely be taken as 0.9

β_w = ratio equal to 1.0 for plastic and compact section. For semi-compact

$$\beta_w = Z_x/S_x \text{ or } S_{x\text{eff}}/S_x$$

Note: β_w may always be taken as 1.0

Effective length

L_E determined from table 13 for beams and Table 14 for cantilevers.

In table 13 L_{LT} is the length of segment over which lateral-buckling can occur, i.e., the distance between points of restraint. Two loading conditions are identified, normal and

destabilising. Destabilising refers to a situation where the loading is applied to the top flange of the beam that is free to move laterally with the load. In the normal loading, the load is applied to the web or the bottom flange.

Equivalent uniform moment factor, m_{LT}

The values for P_b , based on λ_{LT} , have been derived assuming that the beam is under uniform moment throughout. In general, beams are subjected to varying bending moment along their length, which is less severe condition. Therefore the factor m_{LT} can be used for this case. The value of m_{LT} depends on the shape of the bending moment diagram. Its value is less than or equal to unity, (Table 18). m_{LT} may conservatively be taken as unity.

For destabilizing loads m_{LT} must always be taken as 1.0 (cl. 4.3-6.6)

Design procedure

1. Determine the max. design moment M_x from the bending moment diagram for the beam under factored load.
2. Find L_E from Table 13 or 14
3. From section tables find r_y and evaluate λ as L_E/r_y
4. Evaluate λ_{LT} as $4\sqrt{\lambda}\sqrt{B_w}$
5. Determine P_b from Table 16
6. Compute the buckling resistance moment, M_b as:

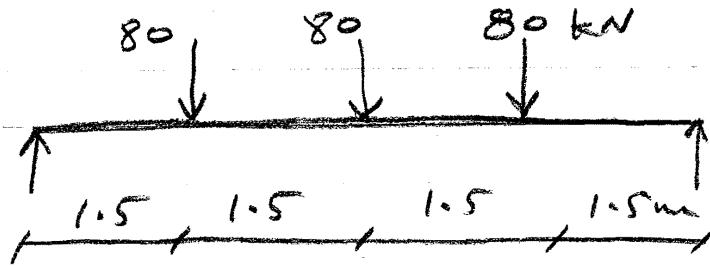
$$M_b = P_b S_x \text{ for plastic or compact sections}$$

$$M_b = P_b S_{x,eff} \text{ or } P_b Z_x \text{ for semi-compact sec.}$$

7. Ensure that $M_x \leq M_b/m_{LT}$
8. check $M_x \leq M_{ex}$ (If $m_{LT} = 1.0$, this check is unnecessary.)

Ex (Handbook)

A beam is required to span 6 m and is to carry three point loads at the quarter point, 1.5 m apart. Each factored load is 80 kN. The three loads are applied to the top flange of the beam and they are free to move laterally. The compression flange is unrestrained over the entire span. At one end the compression flange has partial torsional resistance. At the other end both flanges are not restrained to any degree against rotation on plan. Select a suitable UB section in grade S275 steel.

Solution

Max. BM due to point loads = 240 kN.m
assume self weight is 150 kg/m

$$\text{Max. BM due to self wt} = 1.4 \times 150 \times 9.81 \times 10^3 \times \frac{6^2}{8}$$

$$= 10 \text{ kN.m}$$

$$\text{Max design moment} = 240 + 10 = 250 \text{ kNm}$$

Effective length L_E

refer to Table 13, the loading is destabilising. The restraint at one end of the beam corresponds to (row 6) whilst the restraint at the other end corresponds to (row 7). Take an average value and assume that the beam depth is about 600 mm. Therefore

$$L_E = 1.3 L_{LT} + 2D = 1.3 \times 6 + 2 \times 0.6 = 9 \text{ m}$$

Select a trial section

assume $P_b = 70 \text{ N/mm}^2$

$$S_x \text{ required: } \frac{250 \times 10^6}{70 \times 10^3} = 3572 \text{ cm}^3$$

Try a 610 x 229 x 140 UB

from section tables, $S_x = 4140 \text{ cm}^3$, $T = 22.1 \text{ mm}$, $r_y = 5.03 \text{ cm}$

Check $M_x < M_{cx}$

$$M_{cx} = P_y S_x = 265 \times 4140 \times 10^3 = 1097 \text{ kNm}$$

compute M_b for trial section OK

$$T = 22.1 \text{ mm}, \text{ then } P_y = 265 \text{ N/mm}^2 \quad (\text{Table 9})$$

$$\lambda = LE/r_y = 9000/50 \cdot 3 = 179$$

$$\lambda_{LT} = uv\lambda (\beta_w)^{0.5}$$

Assume section is plastic or compact
(this can be checked from Table 11)

$$\text{Therefore } \beta_w = 1.0$$

Take $u = 0.9$, (its actual value from
section tables = 0.875)

Take $v = 1.0$. (using $x = 30.6$ from
section tables, $\lambda/x = 5.85$, Table 19
would give $v = 0.78$)

$$\text{then } \lambda_{LT} = 0.9 \times 1 \times 179 \times 1.0 = 161$$

From Table 16 $P_b = 58 \text{ N/mm}^2$

$$M_b = P_b S_x = 58 \times 4140 \times 10^3 = 240 \text{ kNm}$$

the required buckling resistance is 250 kNm
and the section fails this check
choose larger section

Note: using actual values of $u = 0.87$,

and $v = 0.78$ will result in $\lambda_{LT} = 121$,

and $P_b = 93 \text{ N/mm}^2$ and hence $M_b = 385$
kNm

$$\frac{M_b}{M_{LT}} = \frac{385}{1.0} > 250 \text{ kNm}$$

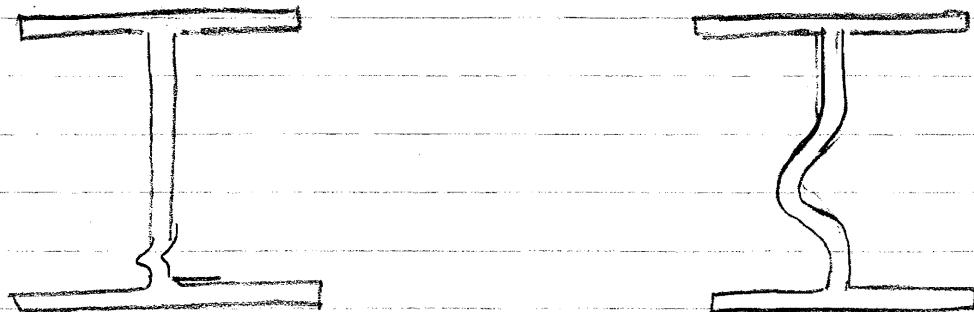
OK

$M_{LT} = 1$ destabilizing loading

Web bearing and web buckling

At locations of heavy concentrated loads, such as support reactions or where columns are supported on a beam flange, there are two other modes of failure which may occur,

- i) web bearing
- ii) web buckling



Bearing failure

Buckling failure

Stiff bearing length

Both checks require the identification of a stiff bearing length b_1 . This is the dimension parallel to the longitudinal axis of the beam, through which the load is applied to the outer face of the flange. Where load is transferred through a solid plate, it is the dimension of that plate.

Where load is transferred through an I or H section, b_1 is given by

$$b_1 = t + 1.6r + 2T$$

t = web thickness

r = root radius

T = flange thickness

} relating to the

beam applying the load

Web bearing capacity

The web bearing capacity P_{bw} is given by

$$P_{bw} = (b_1 + nk)t p_{yw}$$

where

b_1 = stiff bearing length

$n = 5$ (except at the end of a member)

$= 2 + (0.6 b_e/k) \leq 5$ at the end

of a member

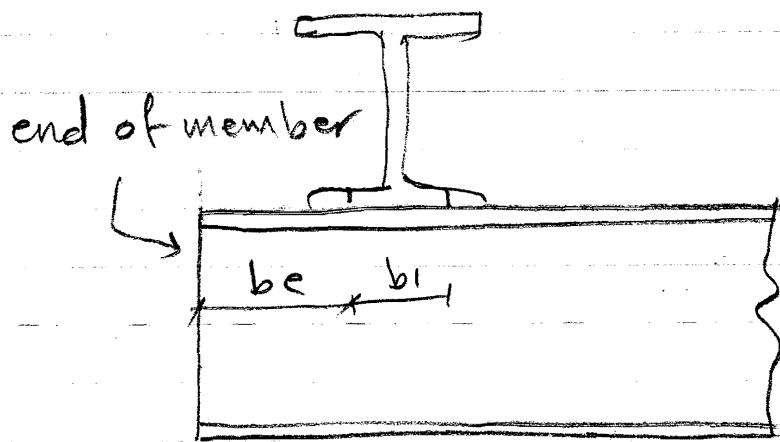
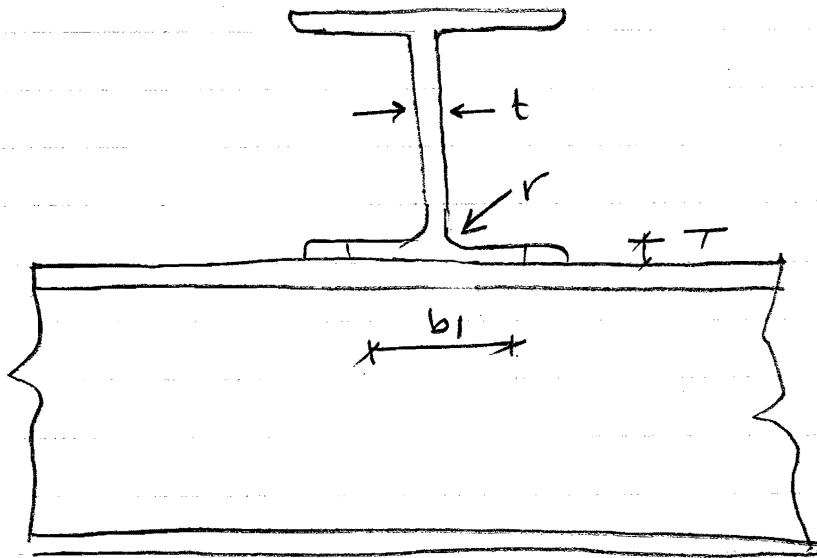
b_e = the distance to the end of the member from the nearest edge of the stiff bearing

$k = T + r$ for a rolled I or H sec.

T = flange thickness

r = root radius

t = web thickness



(all the above relating to the beam being designed)

If the value of the local compressive force exceeds P_{bw} , then web bearing stiffeners are required.

Web buckling capacity (P_x)

$$P_x = \frac{25 \epsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$$

where

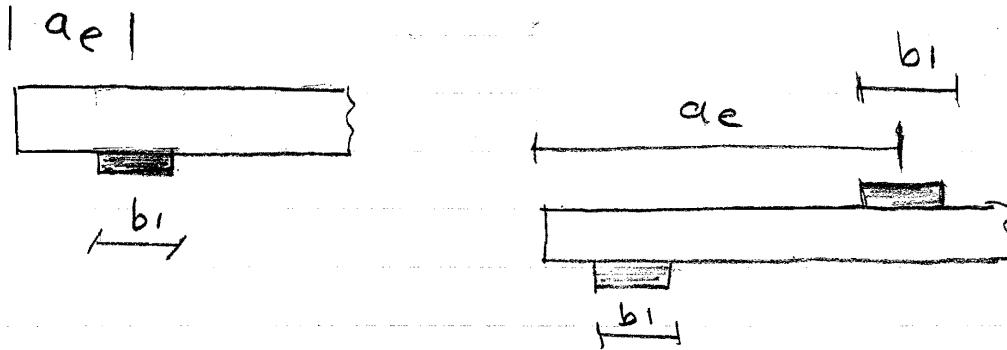
$$\epsilon = (275/\rho_y)^{0.5}$$

d = depth of the beam web

P_{bw} = Web bearing capacity

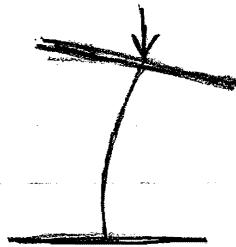
If the distance a_e from the load or reaction to the nearer end of the member is less than $0.7d$, the buckling resistance P_x is given by

$$P_x = \frac{a_e + 0.7d}{1.4d} \frac{25 \epsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$$

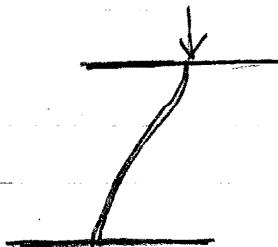


The above equations for (P_x) are for the case when the flange through which the load or reaction is applied is effectively restrained against both:

- a - rotation relative to the web
- b - lateral movement relative to the flange



(a)



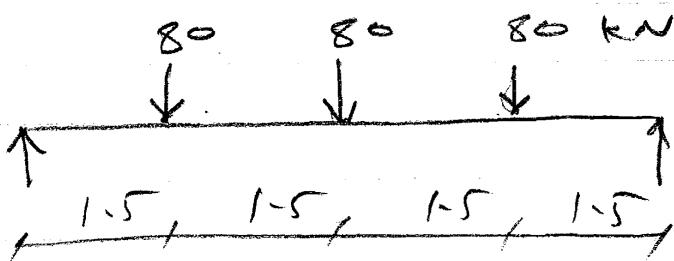
(b)

Flanges restrained against (a) and (b) is the situation most common in practice if not then another reduction in (P_x) is required.

(see cl. 4.5.3.1)

Ex:

Check the beam of the previous example for unrestrained beam at the loaded position and at the supports.



The load is transmitted to the beam via $457 \times 152 \times 67$ UB that sits on the top flange.

solution

a- Under the central point load

i- web bearing

from section tables for the above UB , $t = 9.0\text{mm}$, $T = 15\text{mm}$, $r = 10.2\text{mm}$

stiff bearing length b_1

$$b_1 = t + 1.6r + 2T = 9 + 1.6 \times 10.2 + 2 \times 15 \\ = 55.3 \text{ mm}$$

local capacity for web bearing

$$P_{bw} = (b_1 + n_k) t P_y w$$

$n = 5$ as this is not near to beam end

For the beam to be checked
(UB 610 x 229 x 140)

$$t = 13.1 \text{ mm}, T = 22.1 \text{ mm}, r = 12.7 \text{ mm}$$

$$P_{bw} = 265 \text{ N/mm}^2$$

$$k = T + r = 22.1 + 12.7 = 34.8 \text{ mm} \text{ (rolled section)}$$

$$P_{bw} = (55.3 + 5 \times 34.8) \times 13.1 \times 265 \times 10^{-3}$$

$$= 796 \text{ kN} > 80 \text{ kN} \quad \underline{\underline{\text{OK}}}$$

i- Web buckling check

The load is not applied near the beam end, hence $a_0 \gg 0.7d$
therefore

$$P_x = \left[\frac{25 \epsilon t}{\sqrt{(b_1 + nk) d}} \right] P_{bw}$$

$$\epsilon = 1.02, d = 547.6 \text{ mm}$$

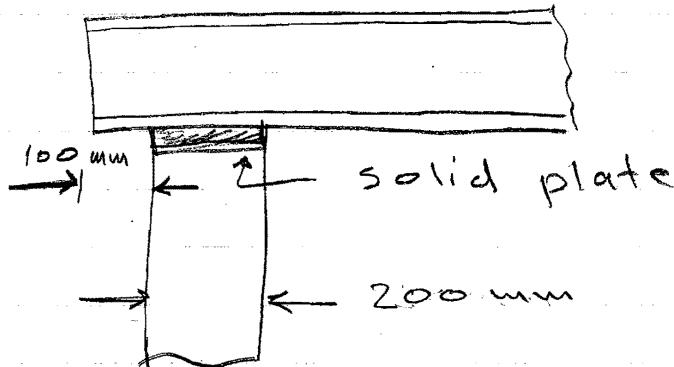
$$P_x = \left[\frac{25 \times 1.02 \times 13.1}{\sqrt{(55.3 + 5 \times 34.8) \times 547.6}} \right] \times 796$$

$$= 750 \text{ kN} > 80 \text{ kN} \therefore \underline{\underline{\text{OK}}}$$

b) AT supports

support reaction

$$R = 120 \text{ kN}$$



detail at end support

i. web bearing

$$P_{bw} = (b_1 + nk) t \cdot p_y w$$

$$n = 2 + (0.6 b_e / k) = 2 + (0.6 \times 100 / 34.8) \\ = 3.72$$

$$b_1 = 200 \text{ mm}$$

$$P_{bw} = (200 + 3.72 \times 34.8) \times 13.1 \times 265 \times 10^{-3} \\ = 1140 \text{ kN} > 120 \text{ kN} \quad \underline{\text{OK}}$$

Web buckling

$$a_o = 200 \text{ mm} < 0.7 d$$

$$\frac{a_o + 0.7d}{1.4d} = \frac{200 + 0.7 \times 547.6}{1.4 \times 547.6} = 0.76$$

$$P_x = 0.76 \left[\frac{25 \in t}{\sqrt{(b_1 + nk)d}} \right] P_{bw}$$

$$= 0.76 \left[\frac{25 \times 1.02 \times 13.1}{\sqrt{(200 + 3.72 \times 34.8) \times 547.6}} \right] \times$$

$\times 1140$

$$= 681 \text{ kN} > 120 \text{ kN} \quad \underline{\underline{\text{OK}}}$$

Biaxial bending

(1) fully restrained beams

The following relationship should be satisfied:

$$\left(\frac{M_x}{M_{cx}} \right)^{Z_1} + \left(\frac{M_y}{M_{cy}} \right)^{Z_2} \leq 1$$

where

M_x = factored moment about $X-X$ axis

M_y = " " " " $Y-Y$ axis

M_{cx} = moment capacity about $X-X$ axis

M_{cy} = " " " " $Y-Y$ axis

$Z_1 = 2$ for I and H sections and 1 for other open sections

$Z_2 = 1$ for all open sections

conservatively we can use $Z_1 = Z_2 = 1$

(2) Unrestrained beams

Lateral torsional buckling affects the moment capacity with respect to the major axis only of I-section beams.

The following interaction expressions must be satisfied:

i) Cross-section capacity check at point of maximum combined moments:

$$\frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

ii) member buckling at the centre of beam

$$\frac{m_x M_x}{P_y Z_x} + \frac{m_y M_y}{P_y Z_y} \leq 1$$

$$\frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{P_y Z_y} \leq 1$$

m_x, m_y, m_{LT} are the equivalent uniform moment factors

m_{LT} from Table 18

m_x, m_y from Table 26

M_b = buckling resistance moment

M_{LT} = max. major axis moment in the segment length L governing M_L

see Example 4.9.3 p 54 (Law)

Compound beams

A compound beam consisting of two equal flange plates welded to a universal beam

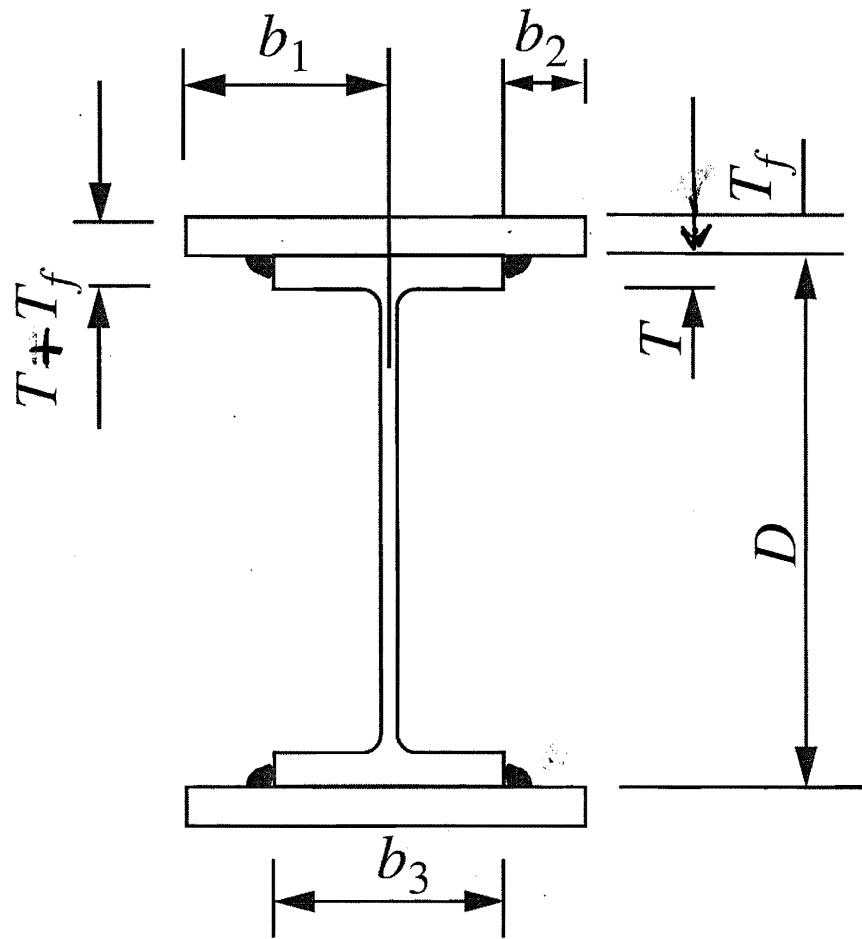
see Fig →

section classification

Table II of BS 5950-1 is used
The compound beam is treated as a section built up by welding.

The following checks are required

- 1) b_1/T
- 2) b_2/T_f
- 3) b_3/T_f
- 4) The universal beam flange and web must also be checked.



Compound beam

Ex (Lam P 58)

A compound beam is to carry a uniformly distributed dead load of 400 kN (total) and an imposed load of 600 kN (total). The beam is simply supported and has a span of 11m. Allow 30 kN for the weight of the beam. The overall depth must not exceed 700 mm. Full lateral support is provided for the compression flange. Use grade S275 steel.

Design the beam section for bending and check deflection.

Solution

$$\begin{aligned}\text{Total factored load} &= 1.4(400 + 36) + \\ &\quad 1.6 \times 600 = 1562 \text{ kN} \\ &= 142 \text{ kN/m}\end{aligned}$$

$$\begin{aligned}\text{Maximum bending moment} &= 142 \times 11^2 / 8 \\ &= 2147.8 \text{ kNm}\end{aligned}$$

Assume that the flanges of the universal beam are thicker than 16mm

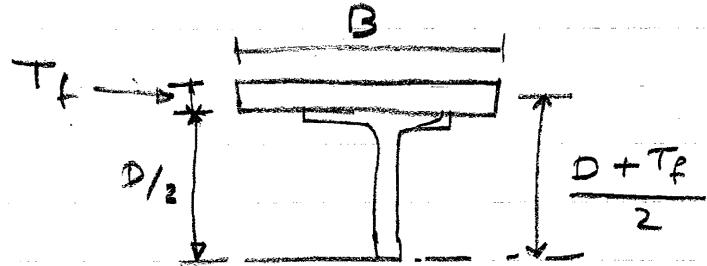
$$P_y = 265 \text{ N/mm}^2 \text{ (from Table 9)}$$

$$\begin{aligned}\text{Plastic modulus required } S_x &= \frac{M}{P_y} \\ &= \left(\frac{2147.8 \times 10^6}{265} \right) \times 10^{-3} = 8104.9 \\ &\quad \text{cm}^3\end{aligned}$$

Try $610 \times 229 \times 140$, where $S_x = 4146 \text{ cm}^3$

The additional plastic modulus required:
 $= 8104.9 - 4146 = 3958.9 \text{ cm}^3$

$$D = 617 \text{ mm}$$



$$S_{ax} = S_x - S_{UB} = 2B T_f \left(\frac{D + T_f}{2} \right)$$

$$\text{let } B = 300 \text{ mm}$$

$$3958.9 = 2 \times 300 \times T_f \left(\frac{617 + T_f}{2} \right) \times 10^{-3}$$

$$T_f^2 + 617T_f - 13196 = 0$$

$$T_f = 20.69 \text{ mm}$$

provide plates $300 \text{ mm} \times 25 \text{ mm}$

$$\text{Total depth} = 617 + 2 \times 25 = 667 < 700 \text{ mm}$$

$\therefore \underline{\text{ok}}$

Check the beam dimensions for local buckling

$$\epsilon = \left(\frac{275}{265} \right)^{0.5} = 1.02$$

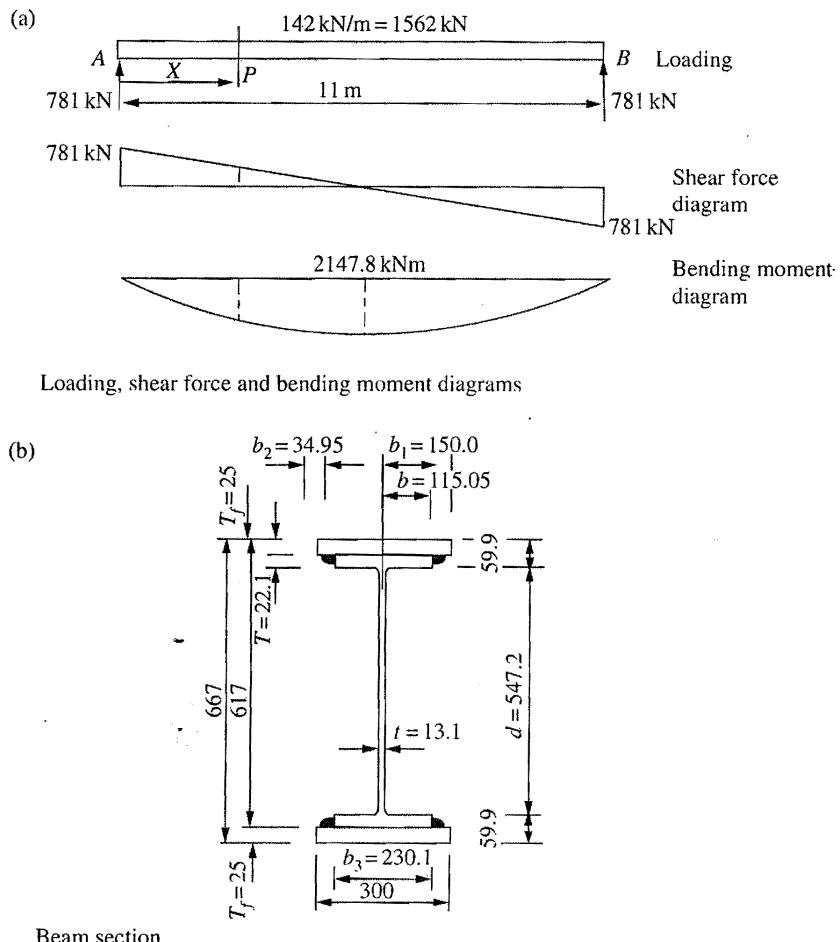


Figure 4.26 Compound beam

The loading, shear force and bending moment diagrams are shown in Figure 4.26(a).

Assume that the flanges of the universal beam are thicker than 16 mm:

$p_y = 265 \text{ N/mm}^2$ (from Table 9, BS 5950)

Plastic modulus required, $S_x = 2147.8 \times 10^3 / 265 = 8104.9 \text{ cm}^3$.
 Try 610×229 UB 140, where $S_x = 4146 \text{ cm}^3$.

The beam section is shown in Figure 4.26(b).

The additional plastic modulus required:

$$= 2 \times 300 \times T_f(617 + T_f)/(2 \times 10^3),$$

where the flange plate thickness T_f is to be determined for a width of 300 mm. This reduces to:

$$T_f^2 + 617T_f - 13196 = 0.$$

a - Universal beam (Table 11)

$$\text{Flange : } b/T = 115.1 / 22.1 = 5.21 < 9 \times 1.02 = 9.18$$

$$\text{Web : } d/t = 547.2 / 13.1 = 41.7 < 80 \times 1.2 = 81.6$$

b - compound beam (welded section)

$$\text{Flange : } b_1/T = 150 / 22.1 = 6.79 < 1.02 \times 8 = 8.1$$

$$b_2/T_f = 34.95 / 25 = 1.40 < 8.1$$

$$b_3/T_f = 230.1 / 25 = 9.2 < 28 \times 1.02 = 28.56$$

\therefore The section meets the requirements for a plastic section.

deflection

$$I_x \text{ per UB} = 111844 \text{ cm}^3$$

The moment of inertia for the compound beam is

$$I_x = 111844 + 2 \times \left(\frac{30 \times 2.5^2}{12} \right) + 2 \times (30 \times 2.5 \times 32.1^2) \\ = 266483 \text{ cm}^4$$

$$S = \frac{5WL^3}{384EI}$$

deflection due to the unfactored imposed load is

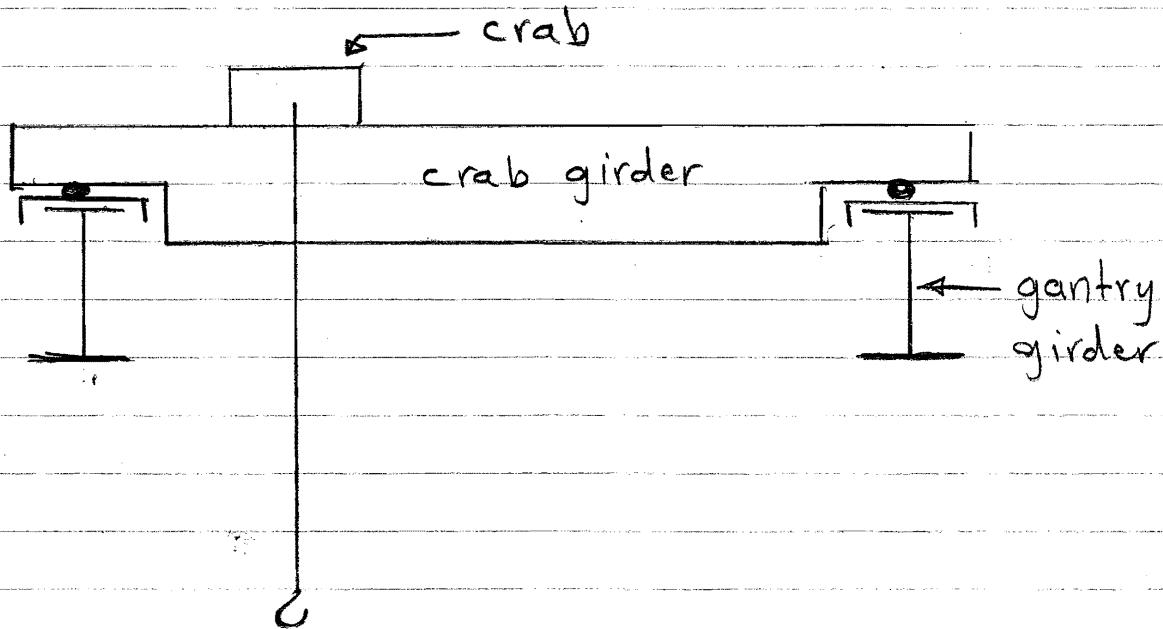
$$S = \frac{5 \times 600 \times 10^3 \times 11000^3}{384 \times 205 \times 10^3 \times 266483 \times 10^4}$$

$$= 19.03 \text{ mm}$$

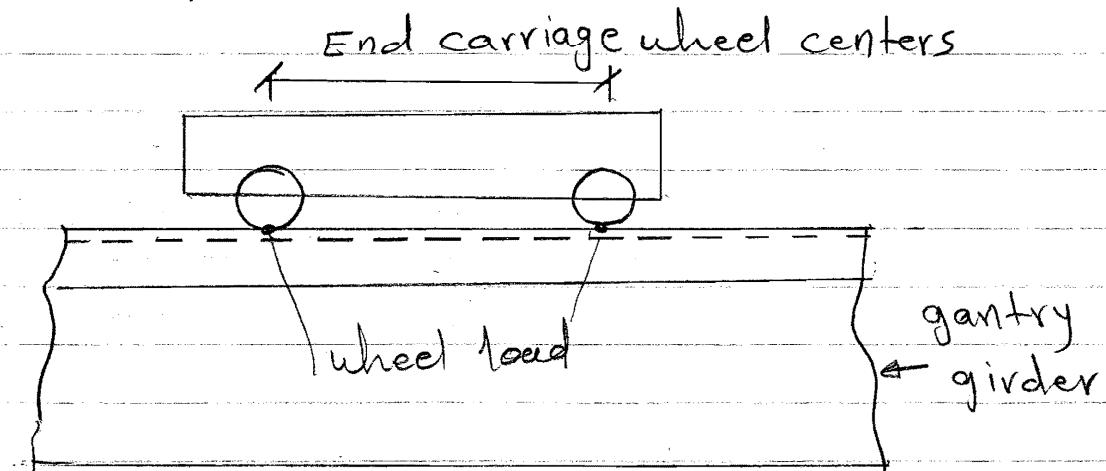
$$\text{allowable defl.} = \frac{L}{360} = \frac{11000}{360} = 30.5 \text{ mm}$$

$\therefore S < \text{allowable deflection} \quad \therefore \text{ok}$

Design of Gantry girder (Crane beam)



min. hook approach



Loads

Crane beam as subjected to:

1- Vertical loads

- self weight
- weight of the crane
- hook load
- impact

2- Horizontal loads

- For electric overhead cranes
10% (crab load + load lifted)
- For hand operated cranes
5% (crab load + load lifted)

Note

The following allowance shall be deemed to cover all forces set up by vibration, shock, impact, etc.:

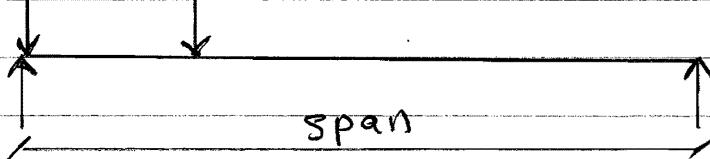
For loads acting vertically static wheel loads shall be increased by

- For electric overhead cranes 25%
- For hand operated cranes 10%

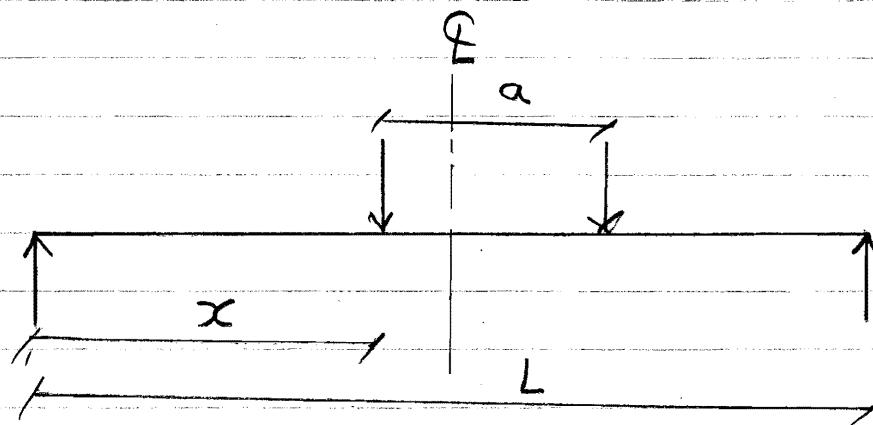
Maximum shear and moment

- 1- The max. shear occurs when one wheel load is nearly over a support

wheel loads



- 2- The maximum moment occurs when the centre of gravity of the wheel loads and one load are placed equidistance about the centre of the girder. The max. moment occurs under the wheel load nearest the centre of the girder



$$\text{position of max. BM, } x = \frac{L}{2} - \frac{a}{4}$$

Crane beam design

1) Buckling resistance moment for x-x axis

- locate the equal area axis by trial and error, then find the positions of the centroids of the tension and compression areas. If Z is the lever arm between these centroids, the plastic modulus is given by

$$S_x = A Z / 2$$

where A is the total area of cross-section

- calculate $\lambda_{LT} = u \sqrt{\lambda} \sqrt{\beta_w}$
- find p_b from Table 17 for welded sections
- $M_b = S_x \cdot p_b$ this must exceed the

factored moment for the vertical loads only including impact with load factor 1.6

2) Moment capacity for the y-y axis

The horizontal bending moment is assumed to be taken by the channel and top flange of the universal beam

$$M_{cy} = Z_y p_y$$

3) Biaxial bending check

$$\frac{M_x}{p_y Z_x} + \frac{M_y}{p_y Z_y} \leq 1$$

$$\frac{M_x}{M_b} + \frac{M_y}{p_y Z_y} \leq 1$$

4) Shear capacity

The vertical shear capacity is checked as for a normal beam.

The horizontal shear load is small and is usually not checked.

5) deflection

The deflection limitations are given in table 8 of BS 5950

Ex

Design a simply supported beam to carry an electric overhead crane. The design data are as follows:

crane capacity = 100 kN

span between crane rails = 20 m

weight of crane = 90 kN

weight of crab = 20 kN

minimum hook approach = 1.1 m

End carriage wheel centres = 2.5 m

Span of crane beam = 5.5 m

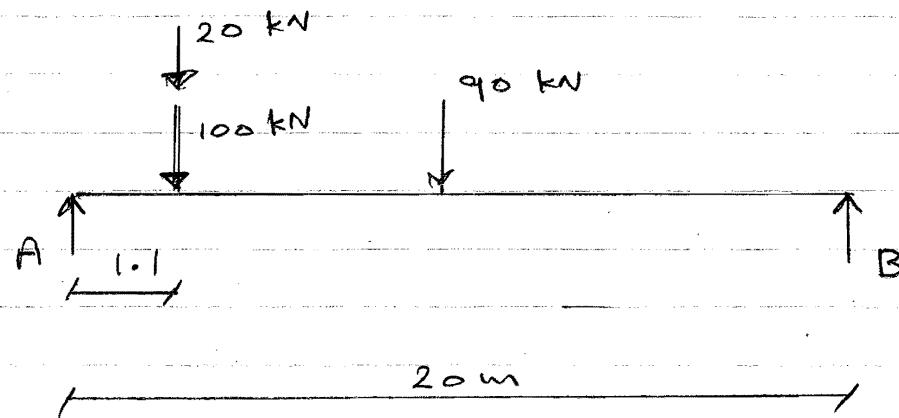
Self weight of crane beam = 8 kN

Assume $\gamma_s = 1$, $\gamma_d = 1$, $B_w = 1$

Use grade S275 steel

solution

1) wheel loads



$$R_A = \frac{120 \times 18.9 + 90 \times 10}{20} = 158.4 \text{ kN}$$

$$\text{vertical wheel load at A} = \frac{RA}{2} = \frac{158.4}{2} \\ = 79.2 \text{ kN}$$

the vertical wheel load including impact =

$$= 79.2 + 0.25 \times 79.2 = 99 \text{ kN}$$

$$\text{horizontal wheel load} = \frac{1}{4} \times 0.1 \times (\text{crab load} + \text{load lifted})$$

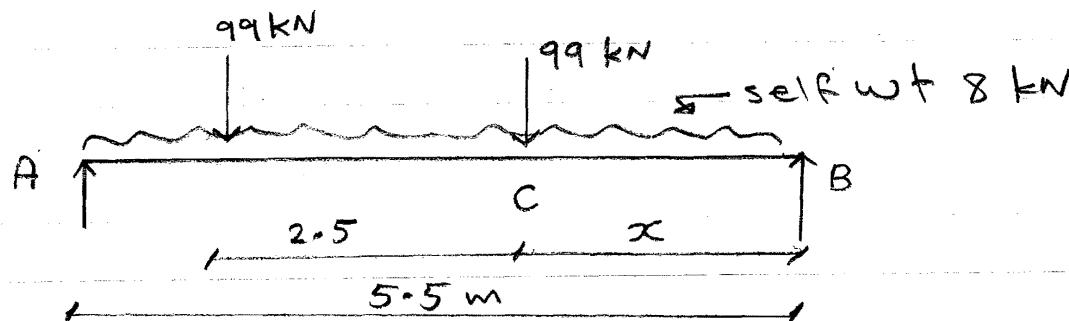
$$= \frac{1}{4} \times 0.1 \times (20 + 100) = 3 \text{ kN}$$

Note: $\frac{1}{4}$ is used because there are 4 wheels

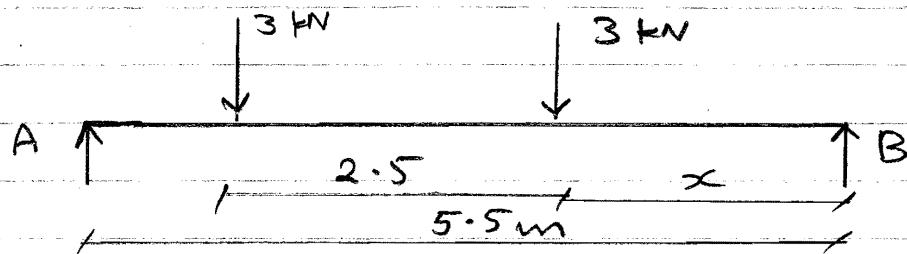
load factors (Table 2 of BS 5950)

- dead load (self weight) $\Rightarrow \gamma_f = 1.4$
- vertical and horizontal crane loads considered separately, $\gamma_f = 1.6$
- vertical and horizontal loads acting together $\gamma_f = 1.4$

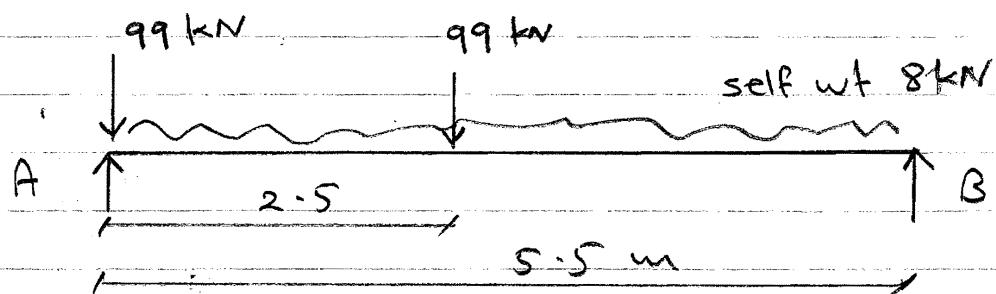
Maximum moment and shear



vertical loads - maximum moment



horizontal loads - maximum moment



$x = L - \frac{a}{4} = 5.5/2 - 2.5/4 = 2.125 \text{ m}$

The max. vertical moments due to dead load and crane loads are calculated separately

- Dead load

$$R_B = 4 \text{ kN}$$

$$M_c = 4 \times 2.125 - \frac{8}{5.5} \times 2.125 \times \frac{2.125}{2} = 5.22 \text{ kN.m}$$

- Crane load, including impact

$$R_B = \frac{99 \times 0.875 + 99 \times 3.375}{5.5} = 76.5 \text{ kN}$$

$$M_c = 76.5 \times 2.125 = 162.6 \text{ kN.m}$$

- crane load cases, vertical and horizontal

$$F_c = 1.4 \times 162.6 + 1.6 \times 5.22 = 267.5 \text{ kN}$$

- The max. horizontal moments

$$R_B = \frac{(3 \times 0.875 + 3 \times 3.375)}{5.5} = 2.32 \text{ kN}$$

$$M_C = 2.32 \times 2.125 = 4.93 \text{ kNm}$$

- The max. vertical shear

due to dead load, $R_A = 4 \text{ kN}$

due to crane loads including impact

$$R_A = 99 + 99 \times 3/5.5 = 153 \text{ kN}$$

The load factors are introduced to calculate the design moments and shear for the various load combinations:

1) Vertical crane loads with impact and no horizontal crane load, Max. moment M_C

$$M_C = (1.4 \times 5.22) + (1.6 \times 162.6) = 267.5 \text{ kNm}$$

max. shear

$$F_A = (1.4 \times 4) + (1.6 \times 153) = 250.4 \text{ kN}$$

2) \rightarrow

Crane load cases, vertical and horizontal

max. vertical load =

$(1.4 \times 162.6) + (1.6 \times 5.22)$

2) Vertical crane loads with impact and horizontal crane loads acting together
 max. vertical moment =

$$M_c = (1.4 \times 5.22) + (1.4 \times 162.6) = 234.95 \text{ kNm}$$

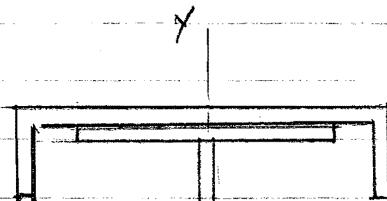
max. horizontal moment =

$$M_c = 1.4 \times 4.93 = 6.9 \text{ kNm}$$

Buckling resistance moment for the x-x axis

Try 457x191x74 UB + 254x76 channel

for the crane beam



$$r_y = 62 \text{ mm}$$

$$S_x = 2086.8 \text{ cm}^3$$

For the top section

$$Z_y = 331 \text{ cm}^3$$

$$I_y = 4202 \text{ cm}^3$$

is hot rolled steel, it is good.

top flange thickness = 22.6 mm total

To find ρ_b

$$L_E = \text{span} = 5500 \text{ mm}$$

$$\lambda = \frac{L_E}{r_y} = \frac{5500}{62} = 88.7$$

assume $u = 1.0$ and $\omega = 1.0$, $\beta_w = 1.0$

$$\lambda L_T = u \omega \lambda \sqrt{\beta_w} = 1 \times 1 \times 88.7 \times 1 = 88.7$$

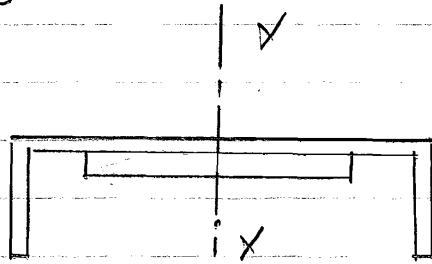
From Table 17 for welded section and for
 $\rho_y = 265 \text{ N/mm}^2$ (Top flange thickness 22.6 mm
 total)

$$\rho_b = 132 \text{ N/mm}^2$$

$$M_b = \rho_b S_x = 132 \times 2086.8 \times 10^3 \times 10^{-6} = 275.5 \text{ kNm}$$

* Moment capacity for the top section
 for the y-y axis

$$Z_y = 331 \text{ cm}^3$$



$$I_y = 4202$$

section resisting horizontal moment

$$M_{cx} = \rho_y \cdot Z_y = 265 \times 331 \times 10^3 = 87.7 \text{ kNm}$$

Check beam in bending

1) Vertical moment, no horizontal moment

$$M_x = 267.5 \text{ kNm} < 275.5 \text{ kNm} \quad \underline{\text{OK}}$$

2) Vertical moment with impact + horizontal moment

$$\frac{M_x}{M_b} + \frac{M_y}{p_y z_y} = \frac{234.9}{275.5} + \frac{6.9}{87.7} =$$

$$= 0.93 < 1.0 \quad \underline{\text{OK}}$$

\therefore The crane beam is satisfactory in bending.

Shear capacity

$$P_c = 0.6 D t p_y$$

$$= 0.6 \times 457 \times 9 \times 275 = 678.6 \text{ kN}$$

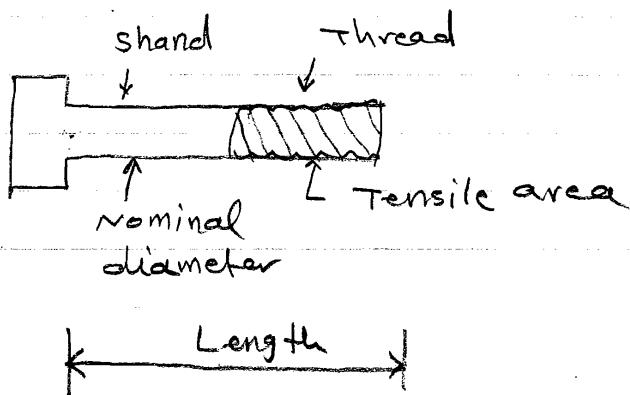
Max. factored shear = 250.4 kN

is OK

Connections

Bolted connections (Non-preloaded)

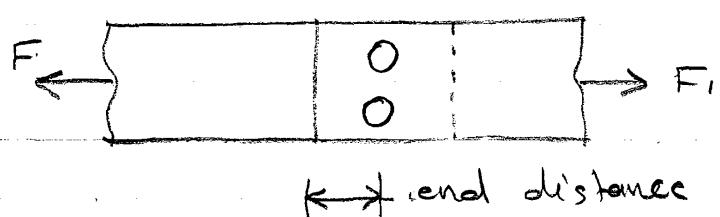
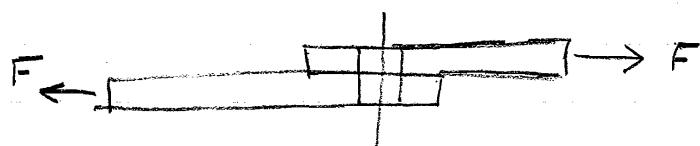
According to BS 4190, the bolts are in three common strength grades 4.6, 8.8 and 10.9. The mechanical properties are shown on P. 284 (Lam).



see typical connection P 285 (Lam)

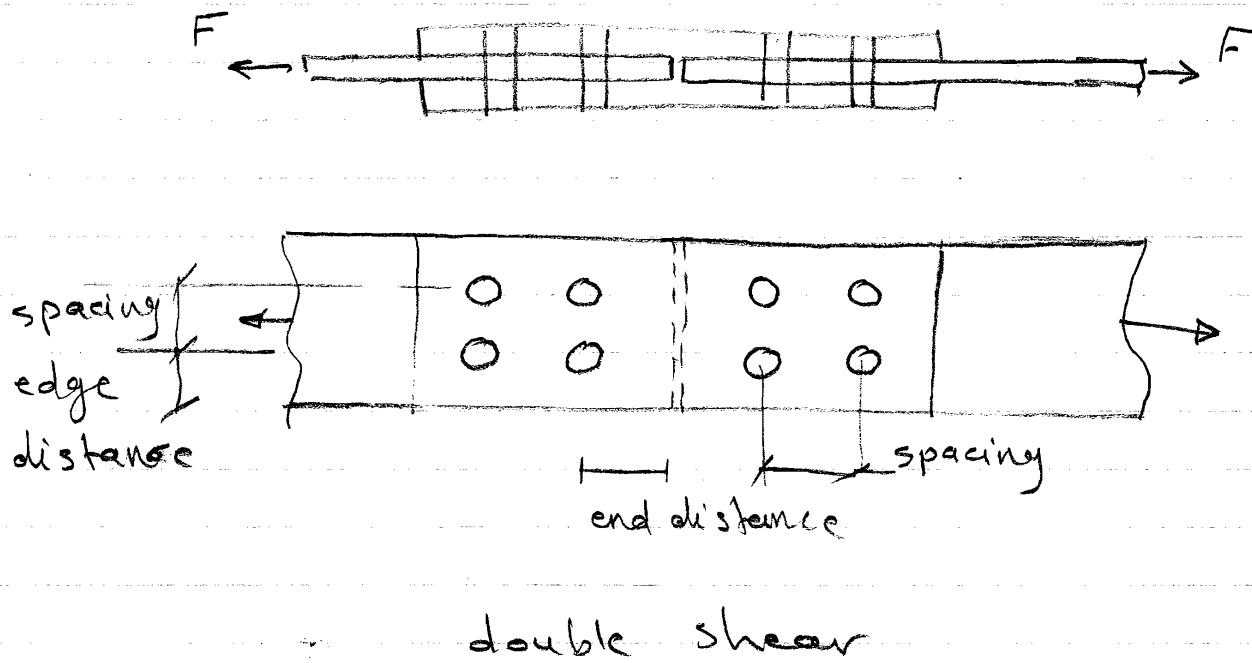
Direct shear joints

Bolts may be arranged in single or double shear



single shear

↔ end distance



BS 5950 provisions (clause 6.2)

- 1) the minimum spacing is 2.5 times the bolt diameter
- 2) the maximum spacing in unstiffened plates in the direction of stress is $14t$ where t is the thickness of the thinner plate connected
- 3) the minimum edge and end distance is given in Table 29
- 4) the maximum edge distance is $11t\epsilon$ where $\epsilon = (275/\rho_y)^{0.5}$
- 5) The standard dimensions of holes are specified in Table 33

Failure modes

A shear joint can fail in the following four ways

- 1) by shear on the bolt shank.
- 2) by bearing on the member or bolt.
- 3) by shear at the end of the member
- 4) by tension in the member.

(see Fig. 10.4 Lam)

These failure modes can be prevented by taking the following measures:

- a) For modes 1 and 2 provide sufficient bolts of suitable diameter
- b) provide sufficient end distance for mode 3
- c) For mode 4, design tension members for effective area.

Shear capacity

$$P_s = p_s A_s$$

where

p_s is the shear strength obtained from Table 30

A_s is the shear area (A or A_f)

A is the bolt shank area based on the nominal diameter. A is used when the shear plane occurs in the non-threaded portion

A_t is the nominal tensile stress area of the bolt (see Table 10.2 Lam)

A_f is used when the shear plane occurs in the threaded portion of the bolt

For a more conservative design, the tensile stress area A_f may be used throughout.

Bearing capacity

The bearing capacity of a bolt on any connected part should be taken as the lesser of the bearing capacity P_{bb} of the bolt and the bearing capacity P_{bs} of the part

i - bearing capacity of bolt P_{bb}

$$P_{bb} = d t_p p_{bb}$$

where

d = nominal diameter of the bolt

t_p = the thickness of the connected part

p_{bb} = bearing strength of the bolt, Table 31

ii - bearing capacity of connected part

$$P_{bs} = k_{bs} d t_p p_{bs} \leq 0.5 k_{bs} e t_p p_{bs}$$

where

e = the end distance

p_{bs} = bearing strength of the connected part, Table 32

k_{bs} = coefficient depending on the type of hole

$k_{bs} = 1.0$ for standard clearance hole

= 0.7 for oversize, short or long slotted hole

= 0.5 for kidney shaped slotted hole

Block Shear clause 6.2.4

Notes :-

Tension capacity of bolts

cl. 6.3.4.2

The tensile force per bolt should not exceed the nominal tension capacity P_{nom} of the bolt

$$P_t = P_{nom} = 0.8 \text{ pt } A_t$$

where

A_t is the tensile stress area

pt is the tension strength of the bolt
from Table 34

Eccentric connections

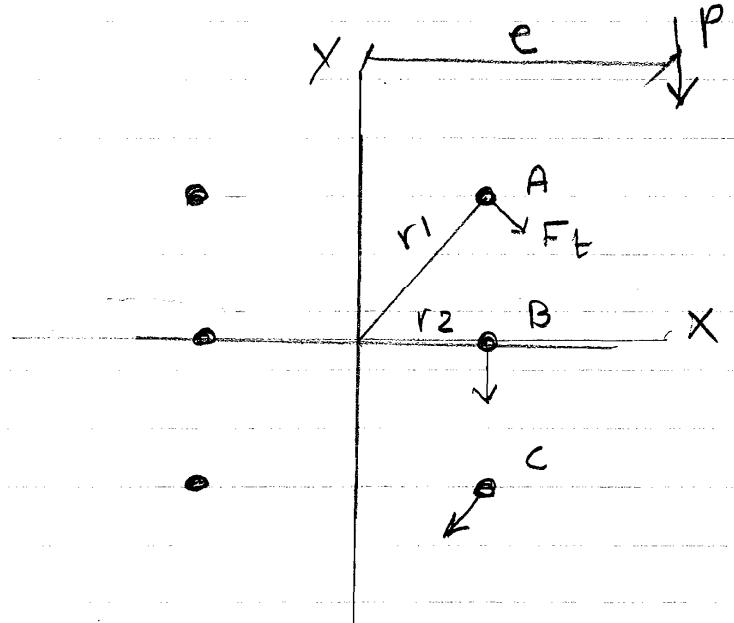
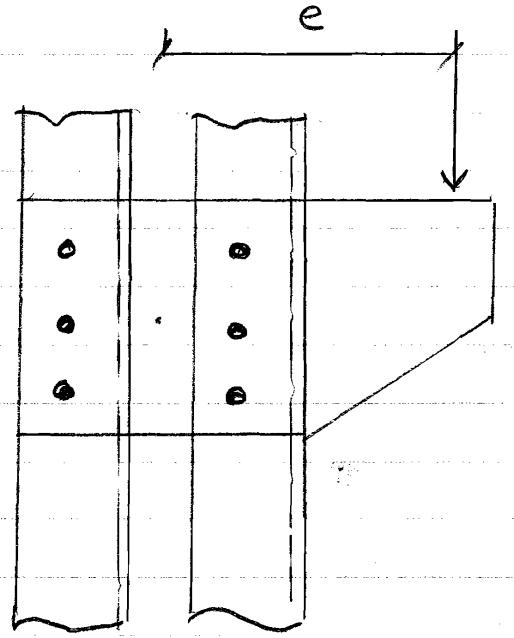
There are two types of eccentrically loaded connections:

- 1) Bolt group in direct shear and torsion
- 2) Bolt group in direct shear and tension

Bolts in direct shear and torsion

The moment is applied in the plane of the connection and the bolt group rotates about its centre of gravity. A linear variation of loading due to moment is assumed, with the bolt farthest from the centre of gravity of the group

carrying the greatest load.



Let the coordinates of each bolt be (x_1, y_1) & (x_2, y_2) etc.

The load F_t due to moment on the maximum loaded bolt A ($r = r_1$) is given by

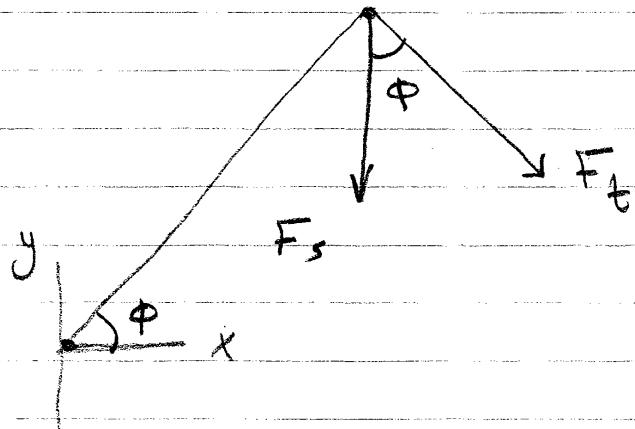
$$F_t = \frac{P \cdot e \cdot r_1}{\sum x^2 + \sum y^2}$$

The force on a bolt r_i from the centre of rotation is

$$F_i = F_t \frac{r_i}{r_1}$$

The direct shear is divided equally between the bolts, therefore the load F_s due to direct shear is given by

$$F_s = \frac{P}{\text{No. of bolts}}$$

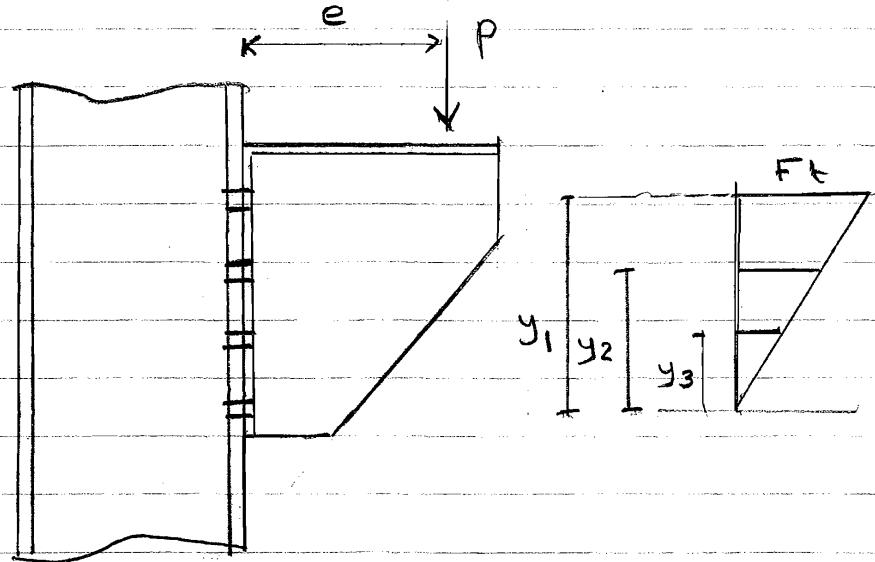


Resultant load on bolt A

$$F_R = \left[F_s^2 + F_t^2 + 2F_s F_t \cos \phi \right]^{0.5}$$

The size of bolt required can then be determined from the maximum load on the bolt.

Bolts in direct shear and tension



In the bracket-type connection shown above, the bolts are in combined shear and tension.

BS 5950 requires that

1. The factored applied shear F_s must not exceed the shear capacity P_s
2. The bearing capacity check must be satisfied.
3. The factored applied tension F_t must not exceed the tension capacity P_t
4. In addition to the above, the following relationship must be satisfied.

$$\frac{F_s}{P_s} + \frac{F_t}{P_t} \leq 1.4$$

In order to find the tension force that acts on each bolt of the bracket connection which is subjected to a factored load P with eccentricity(e), the centre of rotation is assumed to be at the bottom bolt in the group. The loads vary linearly as shown with the maximum load F_t in the top bolt.

$$F_i = F_t \cdot y_i / y_1$$

The moment of resistance of the bolt group is

$$\begin{aligned} M_R &= 2 \left[F_t y_1 + F_t \frac{y_2^2}{y_1} + \dots \right] \\ &= 2 \frac{F_t}{y_1} (y_1^2 + y_2^2 + \dots) \\ &= 2 \frac{F_t}{y_1} \sum y^2 \\ &= P \cdot e \end{aligned}$$

The maximum bolt tension is

$$F_t = P \cdot e \cdot y_1 / 2 \sum y^2$$

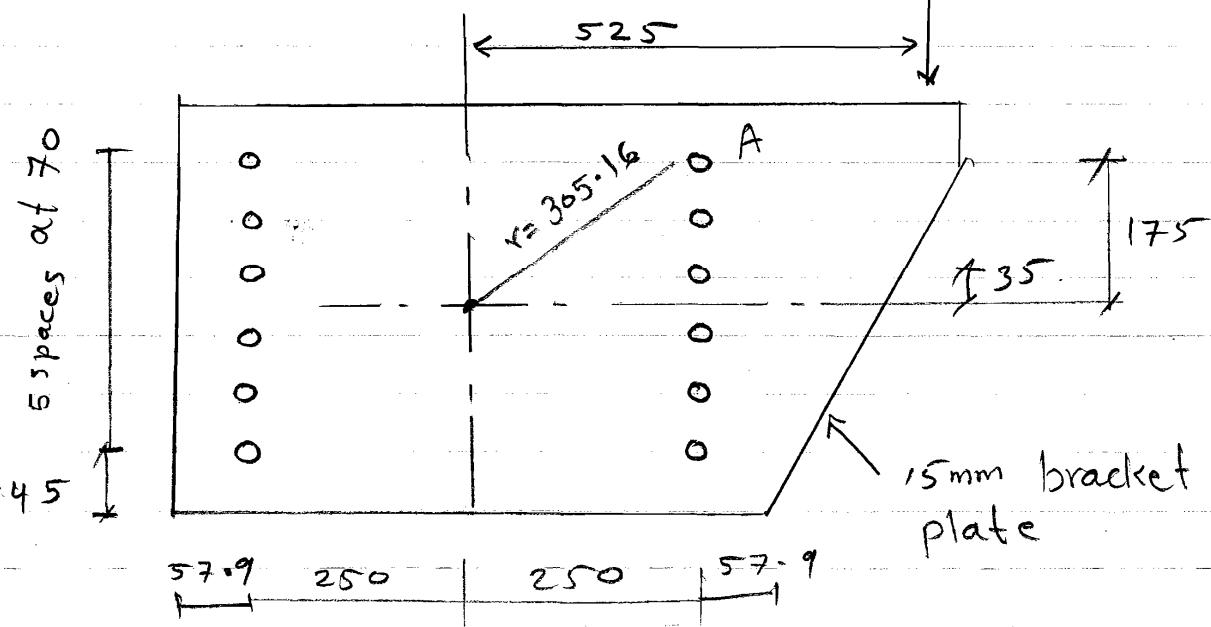
The vertical shear per bolt

$$F_s = P / \text{No. of bolts}$$

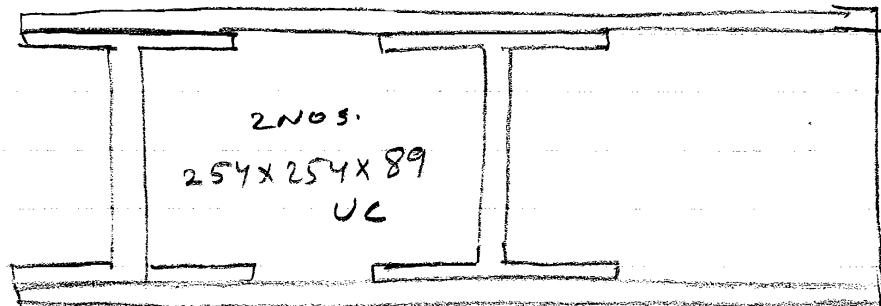
Ex

check that the bracket shown in the figure below is adequate. use grade 4.6 bolts and S275 steel

on each plate { dead load = 60 kN
imposed load = 80 kN



All bolts are 24 mm diameter



$$\text{Factored load} = (1.4 \times 60) + (1.6 \times 80) \\ = 212 \text{ kN}$$

$$M = 212 \times 525 \times 10^{-3} = 111.3 \text{ kN.m}$$

$$\sum x^2 = 12 \times 250^2 = 750 \times 10^3$$

$$\sum y^2 = 4(35^2 + 105^2 + 175^2) = 171.5 \times 10^3$$

$$\sum x^2 + \sum y^2 = 921.5 \times 10^3$$

$$\cos \phi = 250 / 305.16 = 0.819$$

bolt A is the bolt with the maximum load

$$\text{Load due to moment} = \frac{P \cdot e \cdot r}{\sum x^2 + \sum y^2} \\ = \frac{111.3 \times 305.16}{921.5 \times 10^3} \\ = 36.85 \text{ kN}$$

$$\text{Load due to direct shear} = 212 / 12 \\ = 17.67 \text{ kN}$$

$$\text{Resultant load} = [17.67^2 + 36.85^2 + (2 \times 17.67 \times 36.85 \times 0.819)]^{0.5} \\ = 52.31 \text{ kN}$$

bolt single shear strength = $p_s A_s$

$$p_s = 160 \text{ N/mm}^2 \quad (\text{Table 30 BS})$$

$$A_s = A_t = 353 \text{ mm}^2 \text{ assuming failure through}$$

the thread and using Table 10.2 Law

$$P_s = 160 \times 353 = 56.5 \text{ kN}$$

Universal column flange thickness = 17.3 mm
side plate thickness = 15 mm

Bearing capacity of the bolt = $d \times t_p \times p_{bb}$

$$\begin{aligned} P_{bb} &= 24 \times 15 \times 460 \times 10^{-3} \\ &= 165.6 \text{ kN} \end{aligned}$$

Bearing capacity of the plate = $k_{bs} d t_p p_{bs}$

$k_{bs} = 1.0$ assuming standard clearance hole

$p_{bb} = 460 \text{ N/mm}^2$ (Table 32)

$$\begin{aligned} P_{bs} &= 1.0 \times 24 \times 15 \times 460 \times 10^{-3} \\ &= 165.6 \text{ kN} \end{aligned}$$

i. The strength of the joint is controlled by the single shear value of the bolt

$P_s >$ Resultant load on the critical bolt

i. The joint is satisfactory.

check end and edge distances.

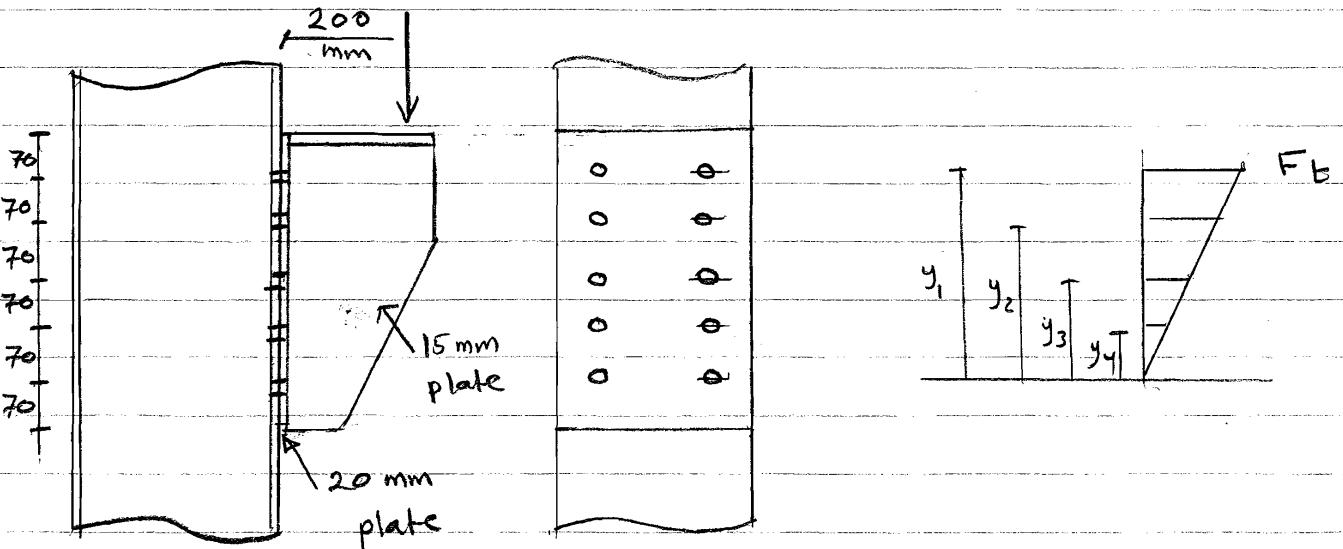
Ex

Determine the diameter of the ordinary bolts required for the bracket shown.

Use grade 4.6 bolts, and standard clearance holes.

$$DL = 80 \text{ kN}$$

Imposed load = 75 kN



$$\text{Factored load} = 1.4 \times 80 + 1.6 \times 75 = 232 \text{ kN}$$

Try 20 mm diameter bolts

$$\text{tension capacity } P_t = 0.8 p_f A_t$$

$$p_f = 240 \text{ N/mm}^2 \quad \text{Table 34}$$

$$A_t = 245 \text{ N/mm}^2 \quad \text{Table (10.2) (Law)}$$

$$\therefore P_t = 0.8 \times 240 \times 245 = 47.0 \text{ kN}$$

bolt single shear strength = $p_s A_s$

$$p_s = 160 \text{ N/mm}^2 \quad (\text{Table 30})$$

$A_s = A_t = 245 \text{ N/mm}^2$ assuming failure through the thread

$$\therefore P_s = 160 \times 245 = 39.2 \text{ kN}$$

$$F_s = 232 / 10 = 23.2 \text{ kN}$$

$$F_t = P_e y_1 / 2 \equiv y^2$$

$$\sum y^2 = 280^2 + 210^2 + 140^2 + 70^2 = 147000 \text{ mm}^2$$

$$F_t = \frac{232 \times 10^3 \times 200 \times 280}{2 \times 147000} \\ = 44.2 \text{ kN}$$

$$\frac{F_s}{P_s} + \frac{F_t}{P_t} = \frac{23.2}{39.2} + \frac{44.2}{47.0} = 1.53 > 1.4$$

Not OK

Try 22 mm bolts

$$P_t = 58.2 \text{ kN}$$

$$P_s = 48.5 \text{ kN}$$

$$\frac{23.2}{48.5} + \frac{44.2}{58.2} = 1.23 < 1.4 \therefore \text{OK}$$

Check bearing

$$\begin{aligned} \text{Bearing capacity of bolt} &= d \times t_p \times p_{bb} \\ &= 22 \times 20 \times 460 \\ &= 202 \text{ kN} > F_s = 23.2 \text{ kN} \end{aligned}$$

$$\begin{aligned} \text{Bearing capacity of plate} &= k_{bs} \cdot d \cdot t_p \cdot p_{bs} \\ &= 1 \times 22 \times 20 \times 460 \\ &= 202 \text{ kN} > F_s = 23.2 \text{ kN} \end{aligned}$$

OK

Welded connections

Welding is the process of joining metal parts by fusing them and filling in with molten from the electrode.

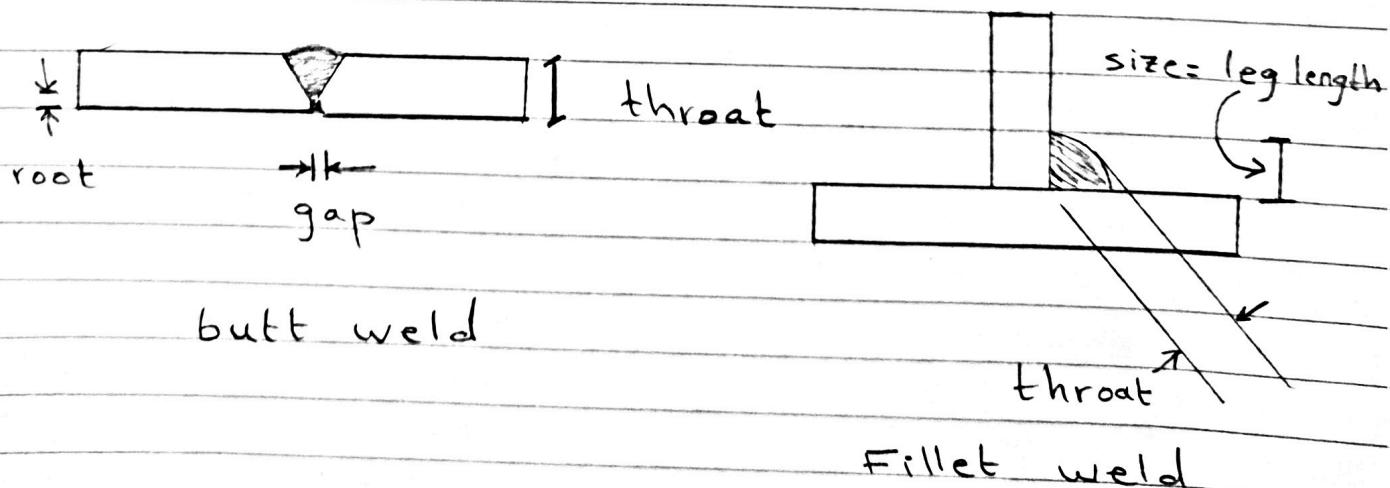
Welding produces neat, strong and more efficient joints than are possible with bolting. However, it should be carried out under close supervision, and this is possible in the fabrication shop.

Site joints are usually bolted. Though site welding can be done it is costly, and defects are more likely to occur.

Types of welds

There are two main types of welds,

a - butt weld , b - fillet weld

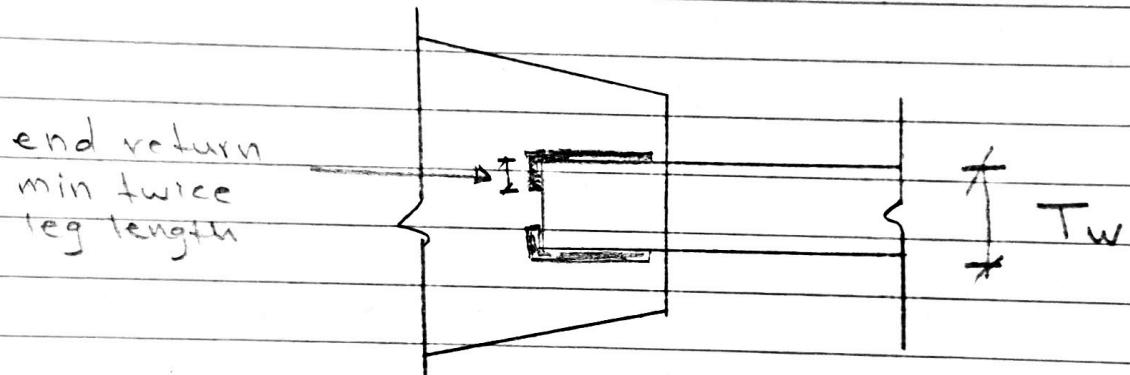


see Fig. 10.18 (Lam) for other weld types

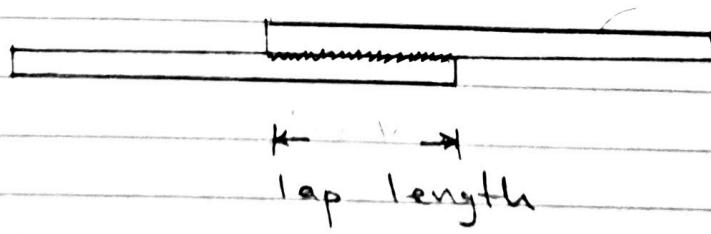
Design of fillet welds

Provisions regarding fillet welds are presented in clause 6.7.2 of BS 5950. Some of these are:

- 1- End returns for fillet welds around corners should be at least twice the leg length

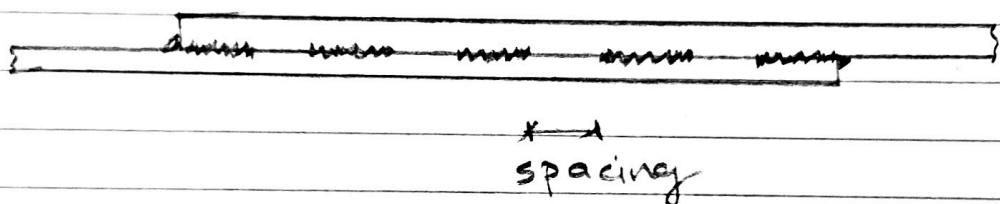


- 2- In lap joints the lap length should not be less than four times the thickness of the thinner plate.



- 3- Where the end of an element is connected only by longitudinal fillet welds the length of each weld should not be less than the transverse spacing (T_w)

4. Intermittent welds should not be used under fatigue conditions. The spacing between intermittent welds should not exceed 300 mm or $16t$ for parts in compression or $24t$ for parts in tension, where t is the thickness of the thinner plate



* BS 5950-1 gives two methods for checking fillet welds:

- a- The simple method (clause 6.8.7.2)
- b- The directional method (clause 6.8.7.3)

The second method recognizes the fact that the transverse capacity of the fillet weld is greater than the longitudinal shear capacity of the weld.

Design strength of fillet welds p_w

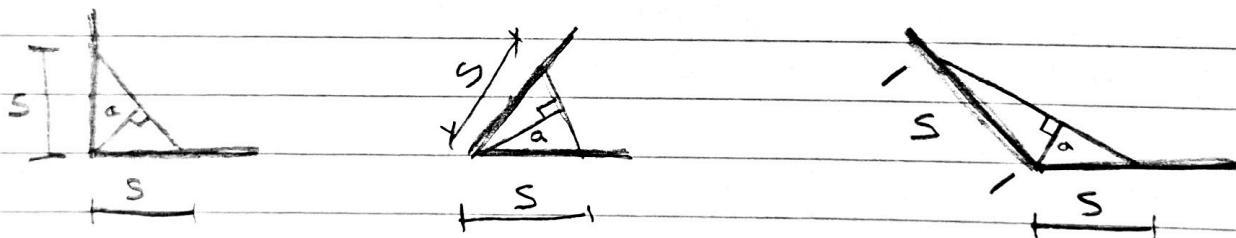
The design strength p_w of a fillet weld should be determined from Table 37, corresponding to the electrode classification and the steel grade

The simple method

In the simple method, the strength of a fillet weld is calculated using the throat thickness

$$\text{Strength of weld} = \text{throat thickness} \times p_w \\ = a \cdot p_w$$

The throat thickness can be found as follows :



$$a = 0.75S$$

$$a = 0.75$$

$$a = a \quad (\text{to be calculated})$$

where

a = throat thickness (the perpendicular distance from the root of the weld to the straight line joining the fusion faces that just lies within the cross-section of the weld)

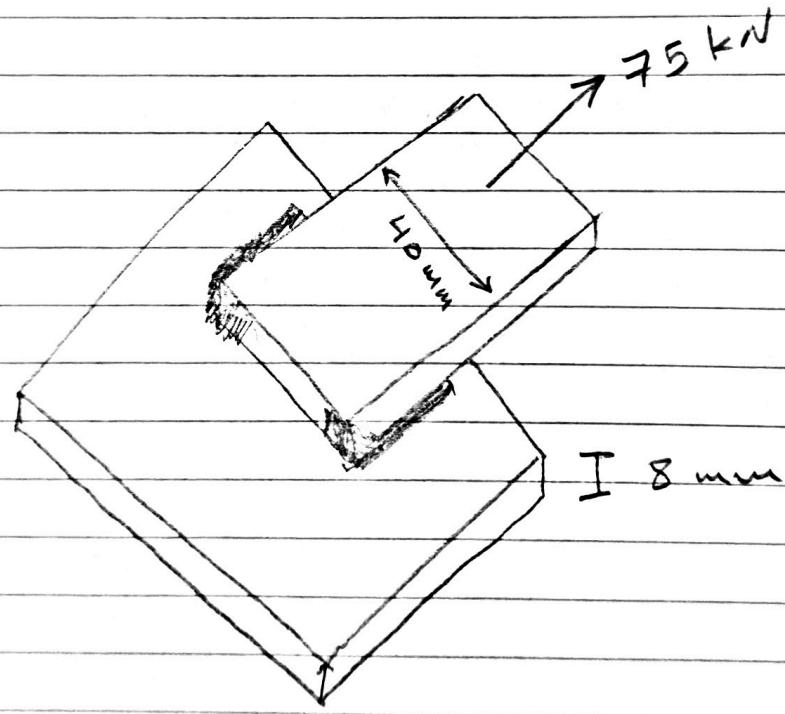
S = leg length (fillet welds are usually specified by the leg length)

Note : The effective length of the weld should be taken as equal to the overall length less one leg length (S) where the weld

does not return around a corner. The effective length of weld should be at least 40 mm but not less than 4S.

Example

For the welded plates shown below, design a suitable 6mm fillet weld to transmit a 75 kN axial force. Use grade S275 steel with grade 35 electrodes



Solution

Design strength of fillet weld $p_w = 220 \text{ N/mm}^2$

(Table 37)

strength of fillet weld/mm length = $p_w \times a$

$$a = 0.7S = 0.7 \times 6 = 4.2 \text{ mm}$$

$$\text{strength / mm} = 220 \times 4.2 \times 10^{-3} = 0.924 \text{ kN/mm}$$

$$\text{effective length of weld required} = 75 / 0.924 \\ = 81 \text{ mm}$$

$$\text{weld length on each side} = 81 / 2 \approx 41 \text{ mm}$$

overall length of weld on each side = $41 + 5$
 $= 41 + 6 = 47 \text{ mm}$
 and should be returned each corner at least
 twice leg length, i.e. 12 mm
 $T_w = 40 \text{ mm}$
 $L = 47 \text{ mm} > T_w \quad \underline{\underline{\text{OK}}}$
 $> 4 \times \text{min. thickness of plate} = 4 \times 8 = 32 \text{ mm}$

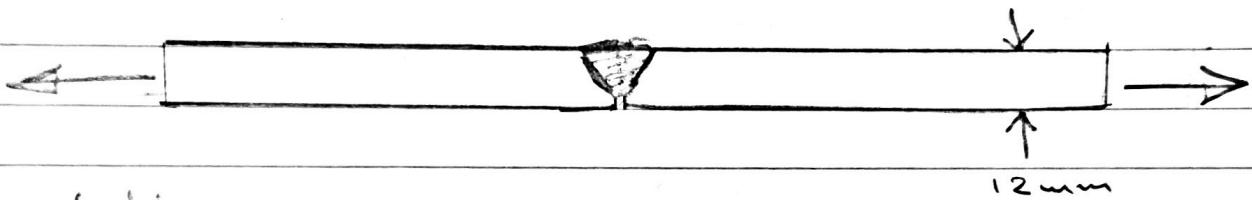
Design of butt welds (clause 6.9 BS)

The design strength of butt welds should be taken as equal to that of the parent metal, provided that suitable electrodes are used.

The throat thickness of a partial penetration butt weld should be taken as equal to the minimum depth of penetration see clause 6.9.2

Example

Calculate the capacity of two 12 mm thick by 100 mm wide tie members which are welded with a single V butt weld. Assume the penetration of the weld is the full thickness of the plate. use s275 steel

Solution

$$\text{throat thickness} = 12 \text{ mm}$$

Design strength $p_w = 275 \text{ N/mm}^2$ (py of the plate)

$$\text{throat area} = 12 \times 100 = 1200 \text{ mm}^2$$

$$\text{capacity} = 275 \times 1200 \times 10^{-3} = 330 \text{ kN}$$