Steel Structures Design

Syllabus

1- Specifications and properties
2- Loads
3- Design methods
4- Design of beams
 - laterally supported beams
 - laterally unsupported beams
5- plate girders
6- Tension members
7- Built up tension member
8- compression members
9- Built up columns.
10- connections

References

1- structural steelwork design to limit state theory. by D. Lam

2- BS 5950 : part 1

3- steelwork design guide to BS 5950 : part 1

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, funiture, equipment, etc. on the floors of building. The values of 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 x relevant partial load factor, γ_f

The partial load factors are given in Table 2 BS 5950

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

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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 that one storey	storey height/300

c) Crane gantry girders

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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.

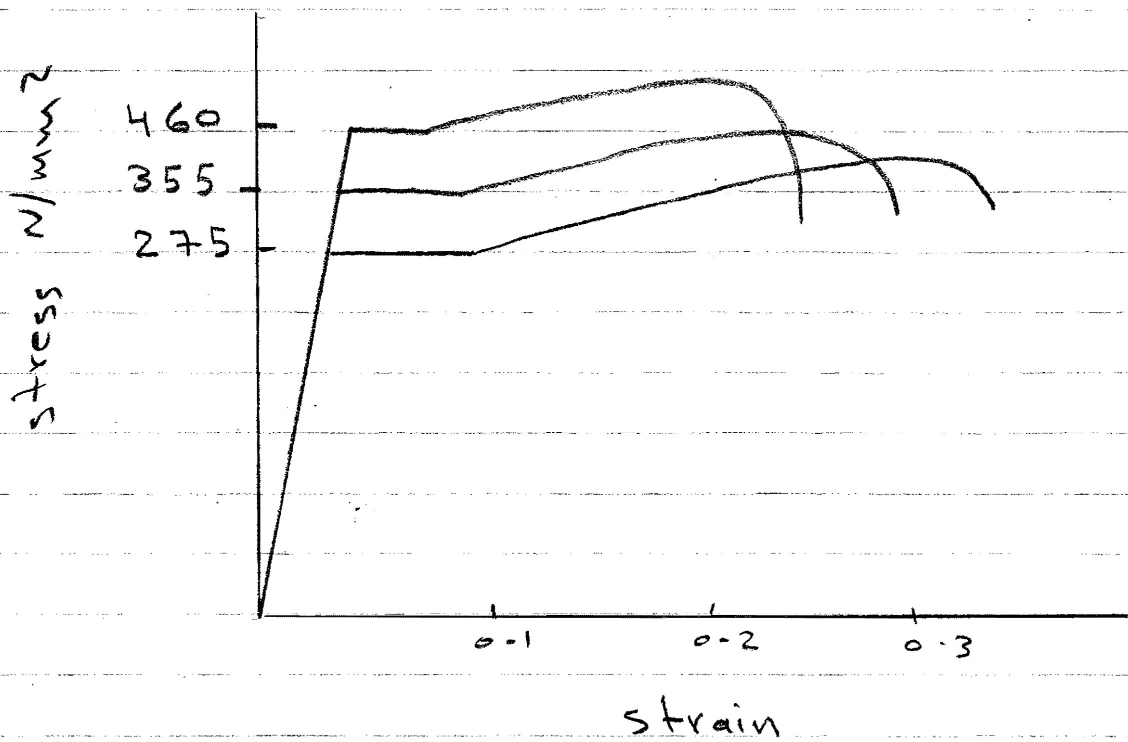
Materials

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

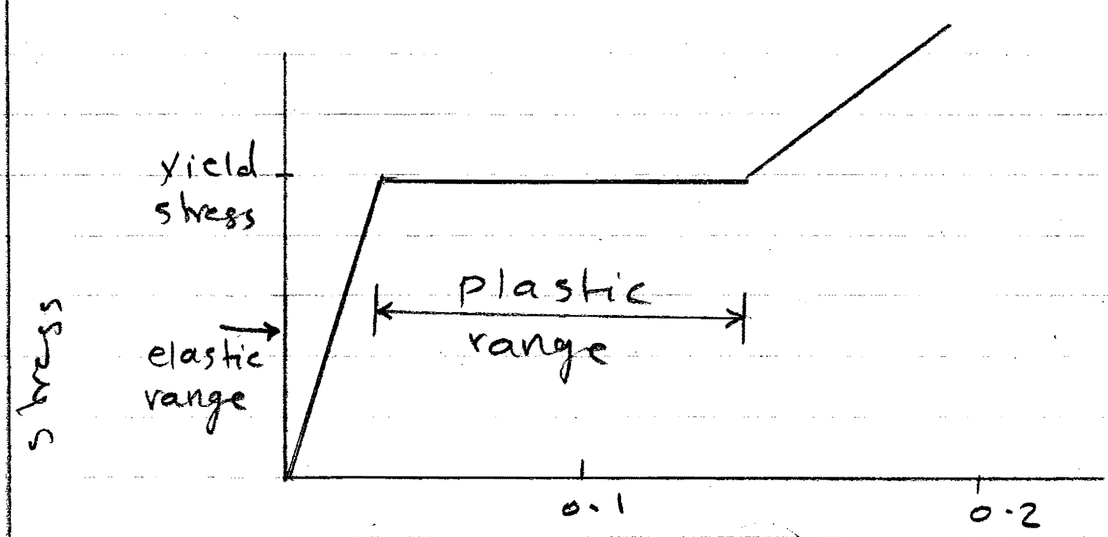
Steel is composed of about 98% of iron with the main alloying elements carbon, silicon, and manganese. Copper and chromium are added to produce the weather-resistant steels that do not require corrosion protection

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

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

P_y : design strength

Y_s = minimum yield strength

U_s = minimum ultimate tensile strength

steel grade	thickness (mm) less or equal to	P_y (N/mm²)
S275	16	275
	40	265
	63	255
S355	16	355
	40	345
	63	335
S460	16	460
	40	440
	63	430

other properties

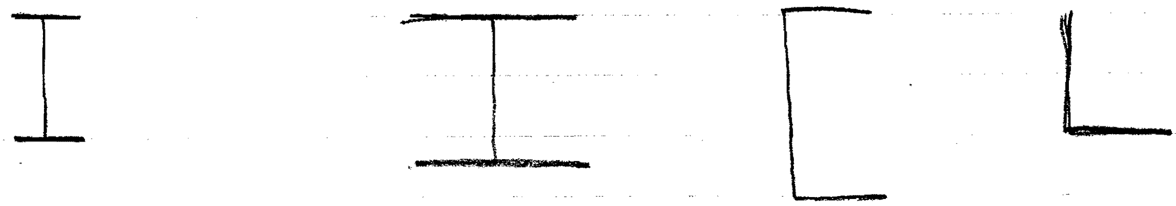
- modulus of elasticity $= 205 \times 10^3\ N/mm^2$
- shear modulus $G = E/2(1+\nu)$
- Poisson's ratio $\nu = 0.30$
- Coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6}$ per $^\circ C$

steel sections

a- Rolled and formed sections
see

1- Universal beams : very effient 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.

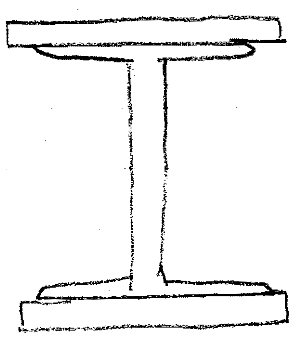
5- other sections : T , hollow sections, m



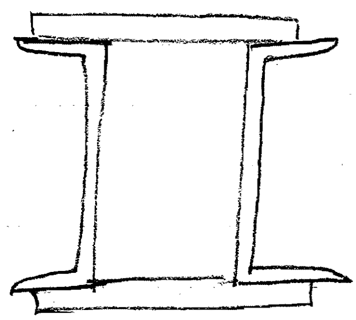
6- Compound sections

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)



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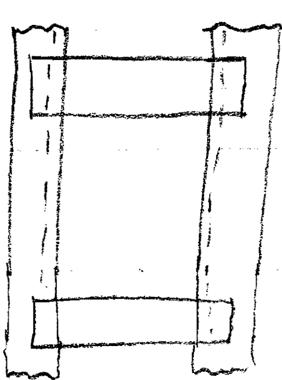
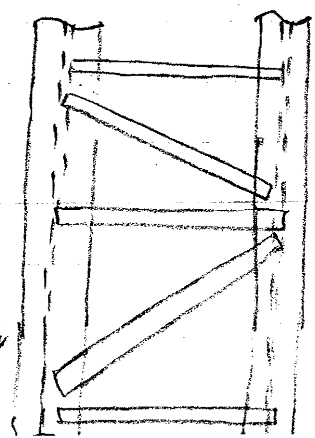


(3)
battened member



(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.

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

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

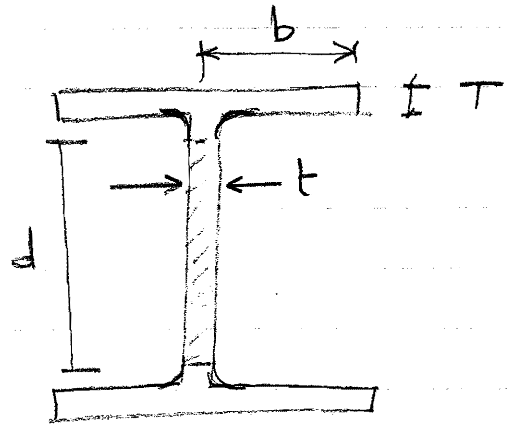
Design of Beams

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

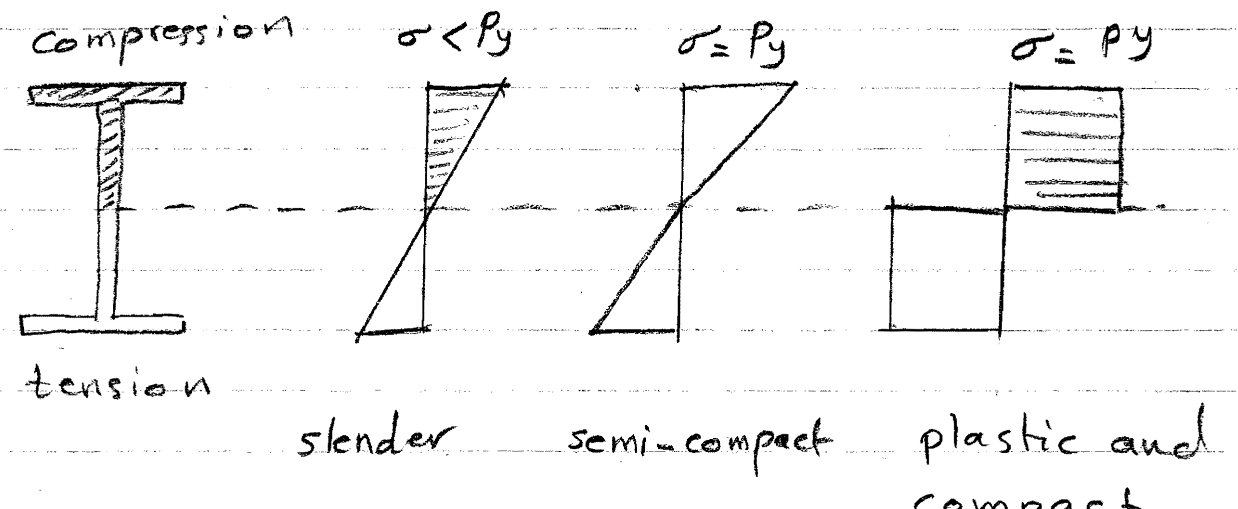
in which $\epsilon = (275/p_y)^{0.5}$

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

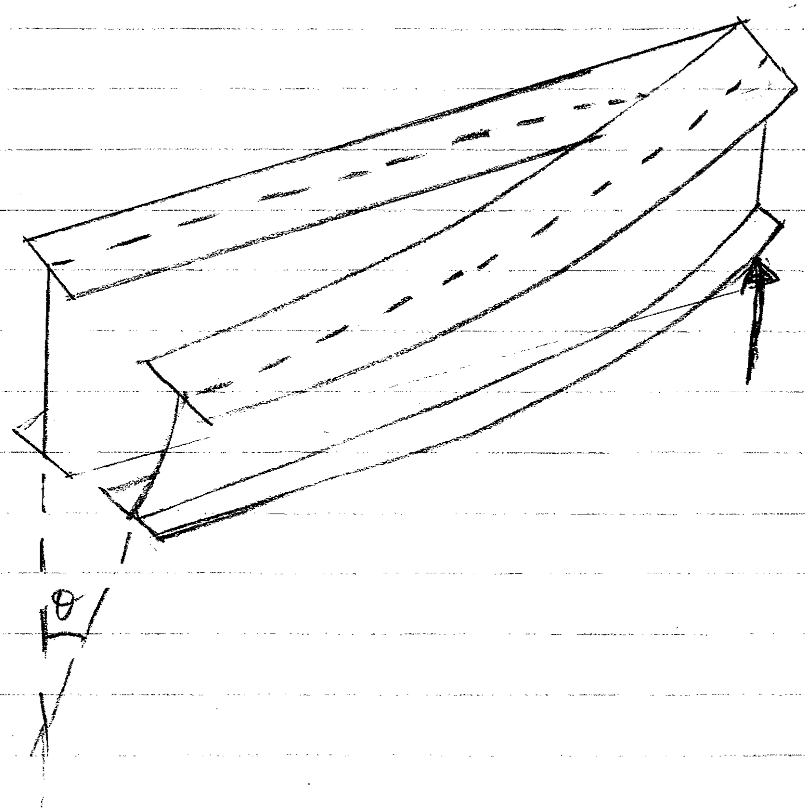


The majority of hot rolled I and H sections are classified as plastic and are therefore suitable for plastic design

(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

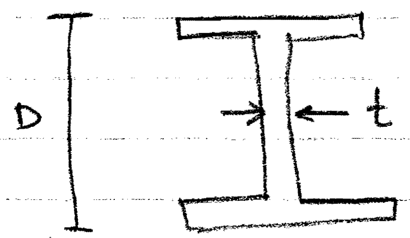
Lateral torsional buckling is inhibited by the provision of lateral restraints to the compression flange.
The continuous restraint should be designed to resist a force of 2.5% of the max. force in the compression flange. This restraining force may be assumed to be uniformly distributed along the compression 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, [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 section

$$M_c = p_y\, S$$

- for class 3 semi-compact sections

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

- 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

$$M_c = p_y \left(Z - \frac{\rho S_v}{1.5} \right)$$ 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.

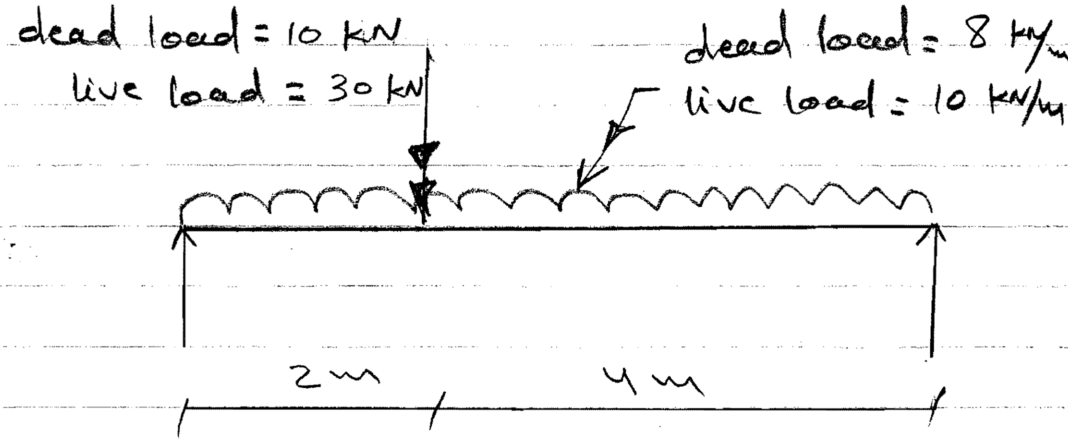
Shap factor (ν): The shape factor of a section is defined as:

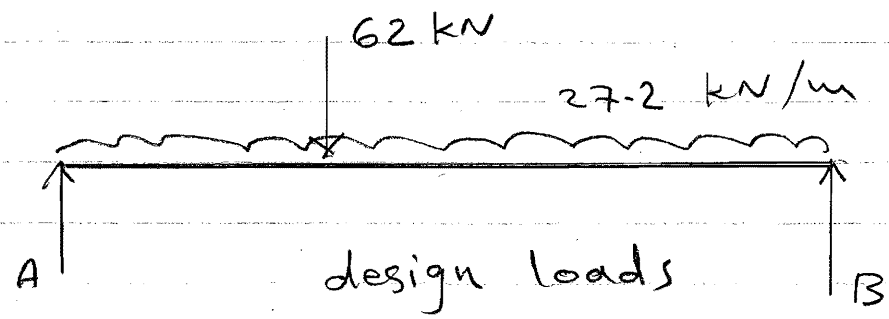
$$\nu = \frac{\text{Plastic modulus}}{\text{elastic modulus}} = \frac{S_{xx}}{Z_{xx}}$$

The value of ν for most I-sections $\simeq 1.15$

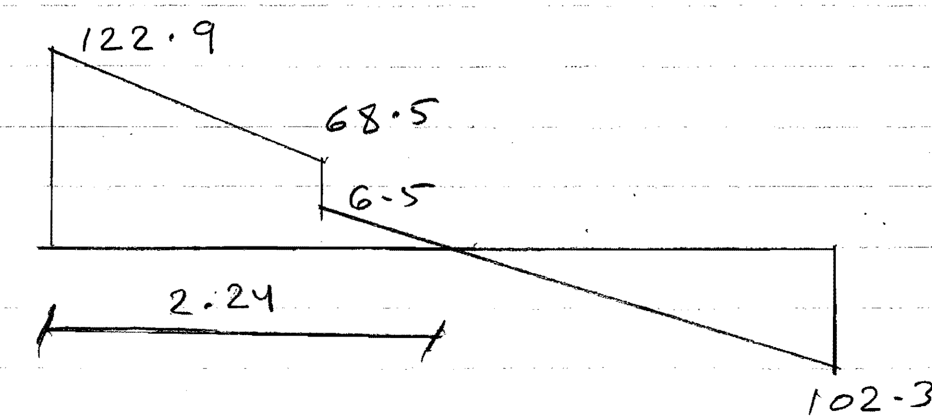
Ex

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





design loads



S.F.D (kN)

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

$R_A = 122.9$ kN
$R_B = 102.3$ kN

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

$M_{max} = 192.2$ kNm

Assume low shear
Use S275 steel
Assume thickness < 16 mm
$\therefore p_y = 275$ N/mm2

plastic modulus $S_{xx} = \dfrac{M}{p_y} = \dfrac{192.2 \times 10^6}{275}$
$= 698.9 \times 10^3$ mm^3

A trial beam size can be selected from section tables

Try 305 x 165 x 46 UB
$D = 306.6$ mm, $d = 265.2$ mm, $B = 165.7$ mm
$T = 11.8$ mm, $t = 6.7$ mm $b/T = 7.02$
$d/t = 39.6$ mm, $S_{xx} = 720 \times 10^3$ mm^3
$Z_{xx} = 646 \times 10^3$ mm^3

i. section classification

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

Flange : outstand element of compression flange rolled section

$b/T = 7.02 < 9.0\,\epsilon$ ∴ Flange is plastic

Web : bending only with neutral axis at mid-depth

$d/t = 39.6 < 80\,\epsilon$ ∴ web is plastic

∴ section is plastic .

ii. shear

$F_v = 122.9$ kN

$P_v = 0.6\,p_y\,A_v = 0.6\,p_y\,tD$

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

∴ $P_v > F_v$

the section is adequate in shear

iii) Bending

check the assumed low shear

$0.6\,P_v = 0.6 \times 338.9 = 203.3$ kN > 122.9 kN

∴ low shear

$M_c = P_y S \leq 1.2\, P_y Z$ simply supported beam

$P_y S = 275 \times 720 \times 10^3 = 198 \times 10^6$ N.mm

$1.2 P_y Z = 1.2 \times 275 \times 646 \times 10^3 = 213.18 \times 10^6$ N.mm

$M_c = 198$ kN.m $> M = 192.2$ kN.m

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

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

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 selenderness λ_{LT}.

Equivalent slenderness λ_{LT}

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

The main parameter in this expression is λ

$$\lambda = L_E / r_y$$

L_E = effective length

r_y = radius of gyration

v = slenderness factor may be determined from table 19. For equal flange beams v 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 $\beta_w = Z_x / S_x$ or $S_{x\,eff} / S_x$

Note: β_w may always be taken as 1.0

Effective length

Is 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. It's 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 $u v \lambda \sqrt{\beta_w}$
5- Determine p_b from Table 16
6- Compute the buckling resistance moment M_b as:

$M_b = p_b S_x$ for plastic or compact sections

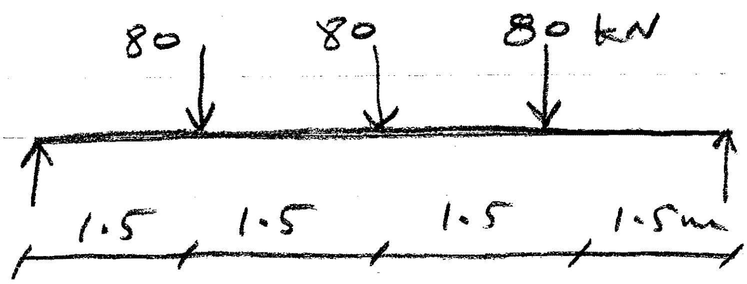
$M_b = p_b S_{x\,eff}$ or $p_b Z_x$ for semi-compact sec.

7- Ensure that $M_x \leqslant M_b / m_{LT}$

8- check $M_x \leqslant M_{cx}$ (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

Max. B.M due to self wt = $1.4 \times 150 \times 9.81 \times 10^{-3} \times 6^2/8$

$\simeq 10$ kN.m

Max design moment = 240 + 10 = 250 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

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

select a trial section

assume $p_b = 70$ N/mm^2

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

Try a 610 x 229 x 140 UB

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

check $M_x < M_{cx}$

$M_{cx} = p_y S_x = 265 \times 4140 \times 10^3 = 1097$ kN.m

ok

compute M_b for trial section

$T = 22.1$ mm, then $p_y = 265$ N/mm^2 (Table 9)

$\lambda = LE/r_y = 9000/50.3 = 179$

$\lambda_{LT} = u v \lambda (\beta w)^{0.5}$

Assume section is plastic or compact (this can be checked from Table 11)
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 $v = 0.78$)

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

From Table 16 $p_b = 58$ N/mm^2

$M_b = p_b S_x = 58 \times 4140 \times 10^3 = 240$ 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 results in $\lambda_{LT} = 121$, and $p_b = 93$ N/mm^2 and hence $M_b = 385$ kNm

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

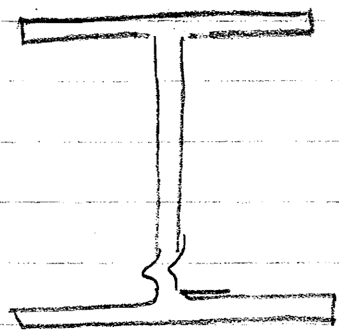
ok

$m_{LT} = 1$ destabilizing loading

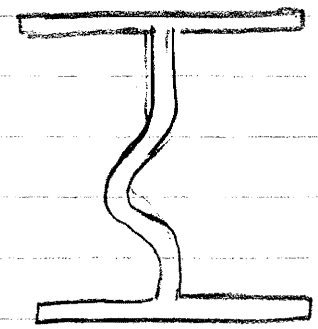
Web bearing and web buckling

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 = flang thickness

relating to the beam applying the load

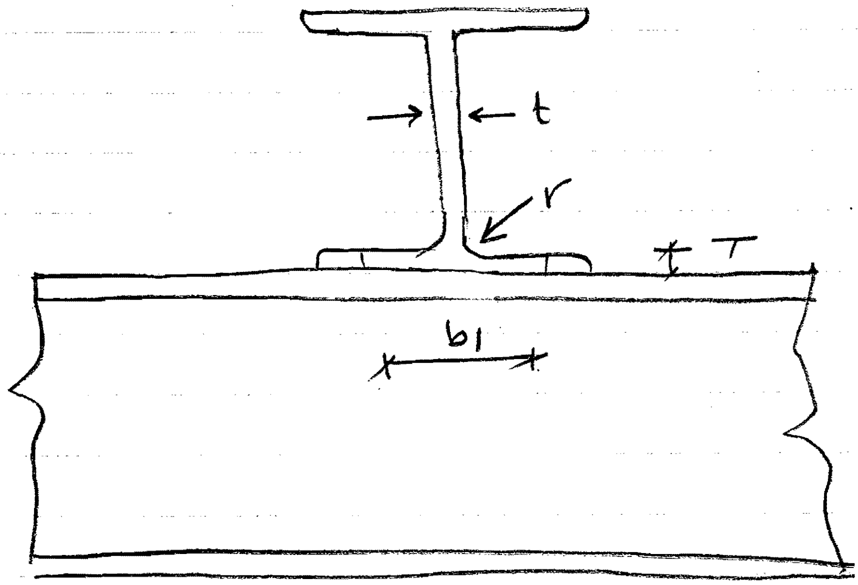
Web bearing capacity

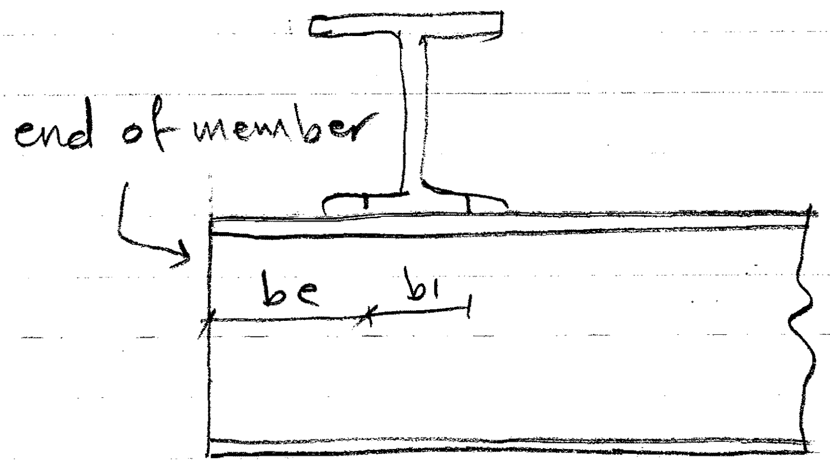
The web bearing capacity P_{bw} is given by

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

where

b_1 = stiff bearing length
n = 5 (except at the end of a member)
= $2 + (0.6\,b_e/k) \leqslant 5$ 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


t
r
T
b1


end of member
be
b1

(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

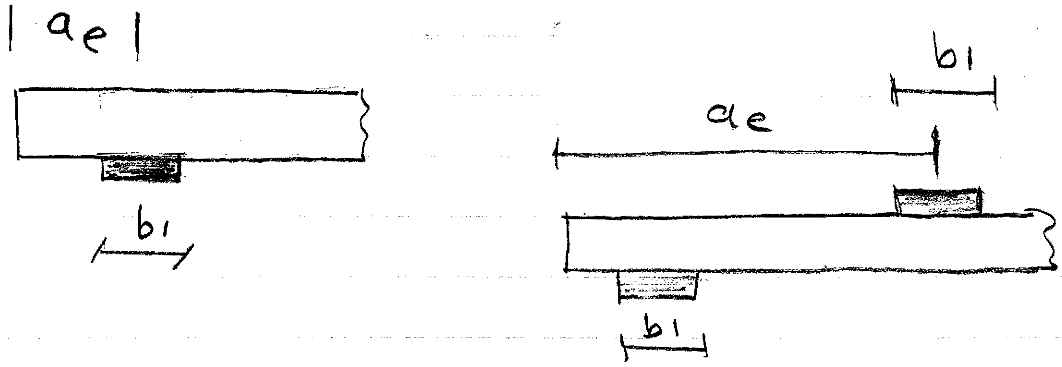
$\epsilon = (275/p_y)^{0.5}$

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 $0.7 d$, the buckling resistance P_x is given by

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



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

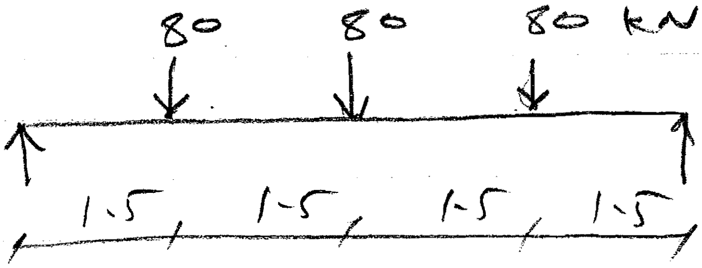
(a) (b)

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 previour example for unrestrained beam at the loaded position and at the supports



The load is transmitted to the beam via 457 x 152 x 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, $t = 9.0\,mm$, $T = 15\,mm$, $r = 10.2\,mm$

stiff bearing length b_1

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

local capacity for web bearing

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

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

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

$t = 13.1\,mm$, $T = 22.1\,mm$, $r = 12.7\,mm$
$P_{yw} = 265\ N/mm^2$

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

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

$= 796\ kN > 80\ kN$ ok

ii- Web buckling check

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

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

$\epsilon = 1.02$, $d = 547.6\ mm$

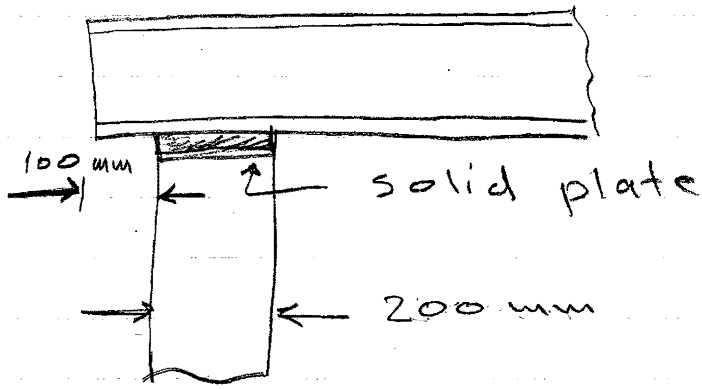
$$P_x = \left[\frac{25 \times 1.02 \times 13.1}{\sqrt{(55.3 + 5 \times 34.8) \times 547.6}} \right] \times 796$$

$= 750\ kN > 80\ kN \therefore$ ok

b. AT supports

support reaction

$R = 120$ kN



detail at end support

i. web bearing

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

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

$b_1 = 200$ mm

$P_{bw} = (200 + 3.72 \times 34.8) \times 13.1 \times 265 \times 10^{-3}$

$= 1140$ kN > 120 kN ok

Web buckling

$a_o = 200$ mm $< 0.7\, d$

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

$$P_x = 0\cdot76 \left[\frac{25\,\epsilon t}{\sqrt{(b_1 + nk)d}} \right] P_{bw}$$

$$= 0\cdot76 \left[\frac{25 \times 1\cdot02 \times 13\cdot1}{\sqrt{(200 + 3\cdot72 \times 34\cdot8) \times 547\cdot6}} \right] \times$$

$$\times 1140$$

$$= 681 \text{ kN} > 120 \text{ kN} \qquad \therefore \underline{\underline{ok}}$$

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

Conservetively 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:

$$\frac{Mx}{Mcx} + \frac{My}{Mcy} \leqslant 1$$

ii) member buckling at the centre of beam

$$\frac{m_x Mx}{py Zx} + \frac{my My}{py Zy} \leqslant 1$$

$$\frac{m_{LT} M_{LT}}{M_b} + \frac{my\ My}{py Zy} \leqslant 1$$

mx, my, m_{LT} are the equivalent uniform moment factors

m_{LT} from Table 18
mx, my from Table 26

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

see Example 4.9.3 P 54 (Lam)

Compound beams

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

see Fig →

section classification

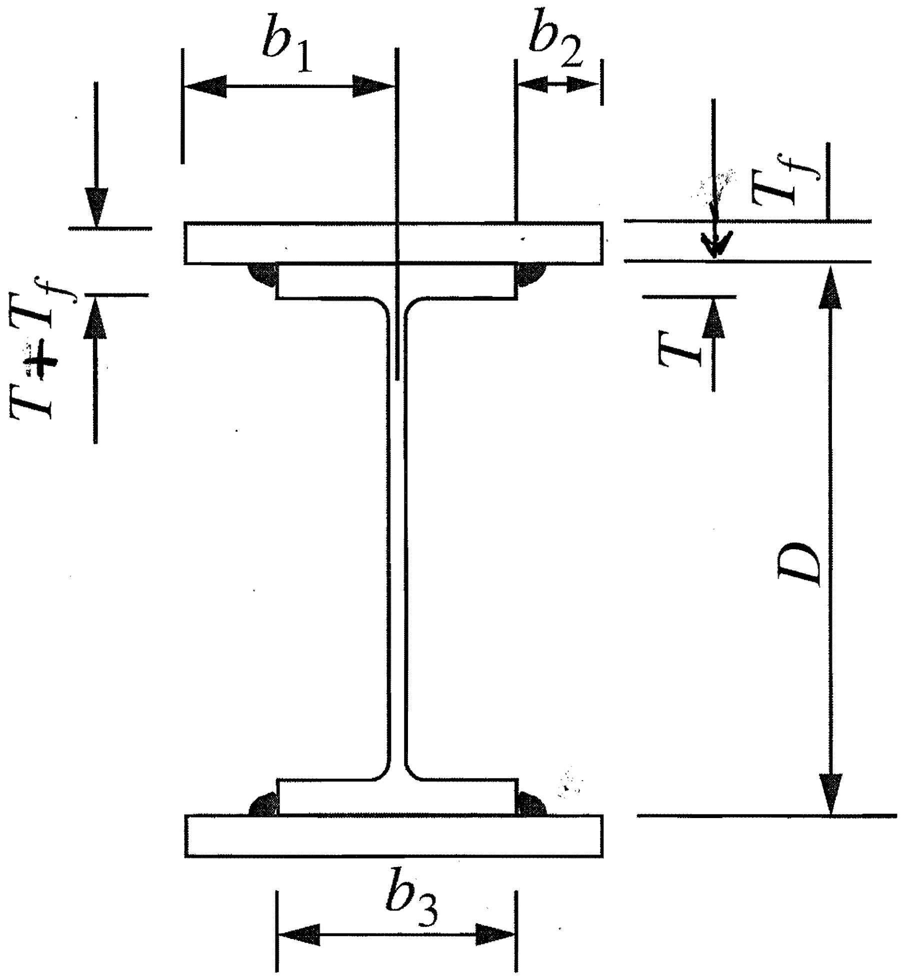
Table 11 of BS 5950-1 is used The compound beam is treated as a section built up by welding. The following checks are required

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 11 m. 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

Total factored load $= 1.4(400+30) + 1.6 \times 600 = 1562$ kN
$= 142$ kN/m

Maximum bending moment $= 142 \times 11^2 / 8$
$= 2147.8$ kN.m

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

$p_y = 265$ N/mm² (from Table 9)

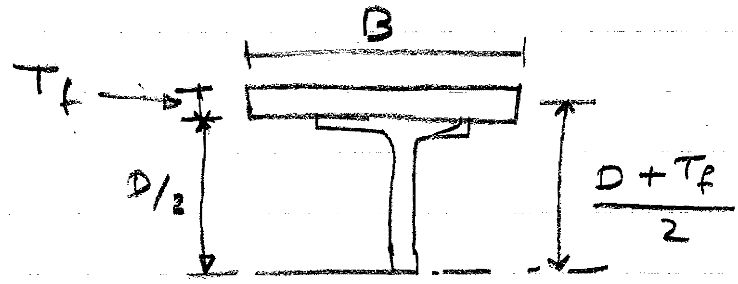
plastic modulus required $S_x = \dfrac{M}{p_y}$

$$= \left(\frac{2147.8 \times 10^6}{265}\right) \times 10^{-3} = 8104.9 \text{ cm}^3$$

Try 610 x 229 x 140, where $S_x = 4146\ cm^3$

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

$D = 617\ mm$



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

let $B = 300\ mm$

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

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

$T_f = 20.69\ mm$

provide plates $300\ mm \times 25\ mm$

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

$\therefore$ ok

Check the beam dimensions for local buckling

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

46'

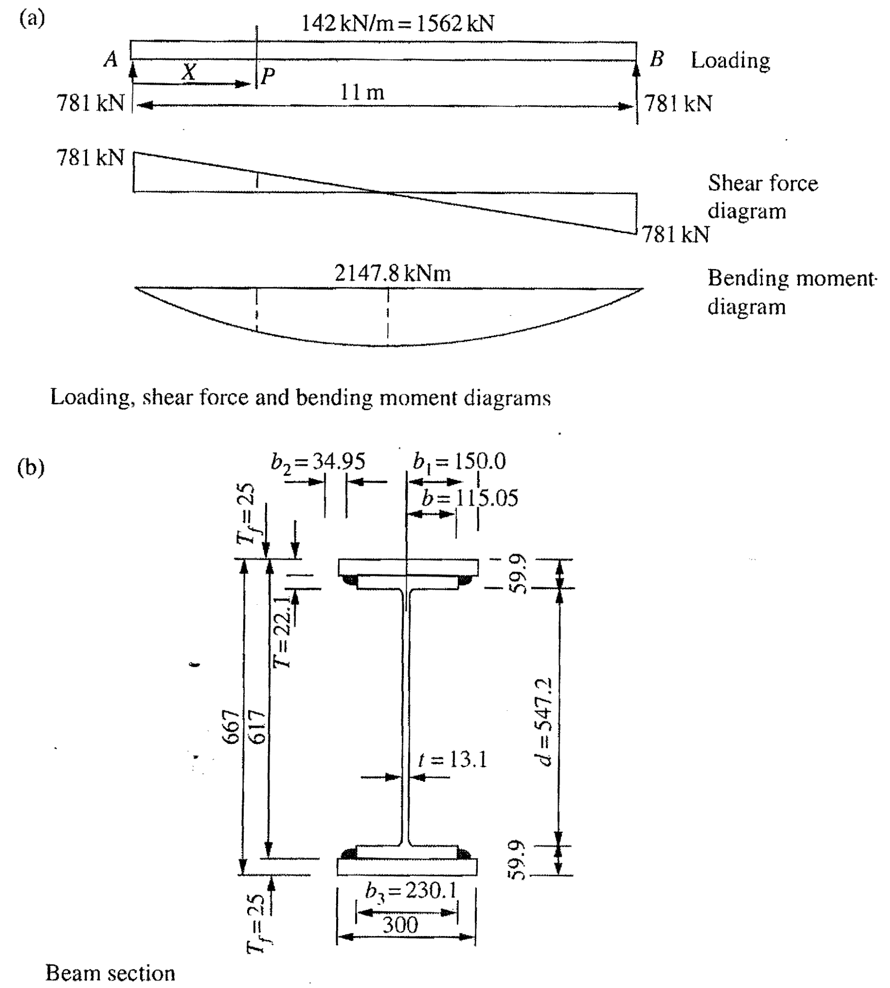


Figure 4.26 Compound beam

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:

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

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:

$$= 8104.9 - 4146 = 3958.9\,\text{cm}^3$$
$$= 2 \times 300 \times T_f(617 + T_f)/(2 \times 10^3),$$

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)

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

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

b- compound beam (welded section)

Flangè : $b_1/T = 150/22.1 = 6.79 < 1.02 \times 8 = 8.1$

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

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

∴ The section meets the requirements for a plastic section.

deflection

I_x for UB = 111844 cm^3

The moment of inertia for the compound beam is

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

$$= 266483 \ cm^4$$

$$\delta = \frac{5 W L^3}{384 E I}$$

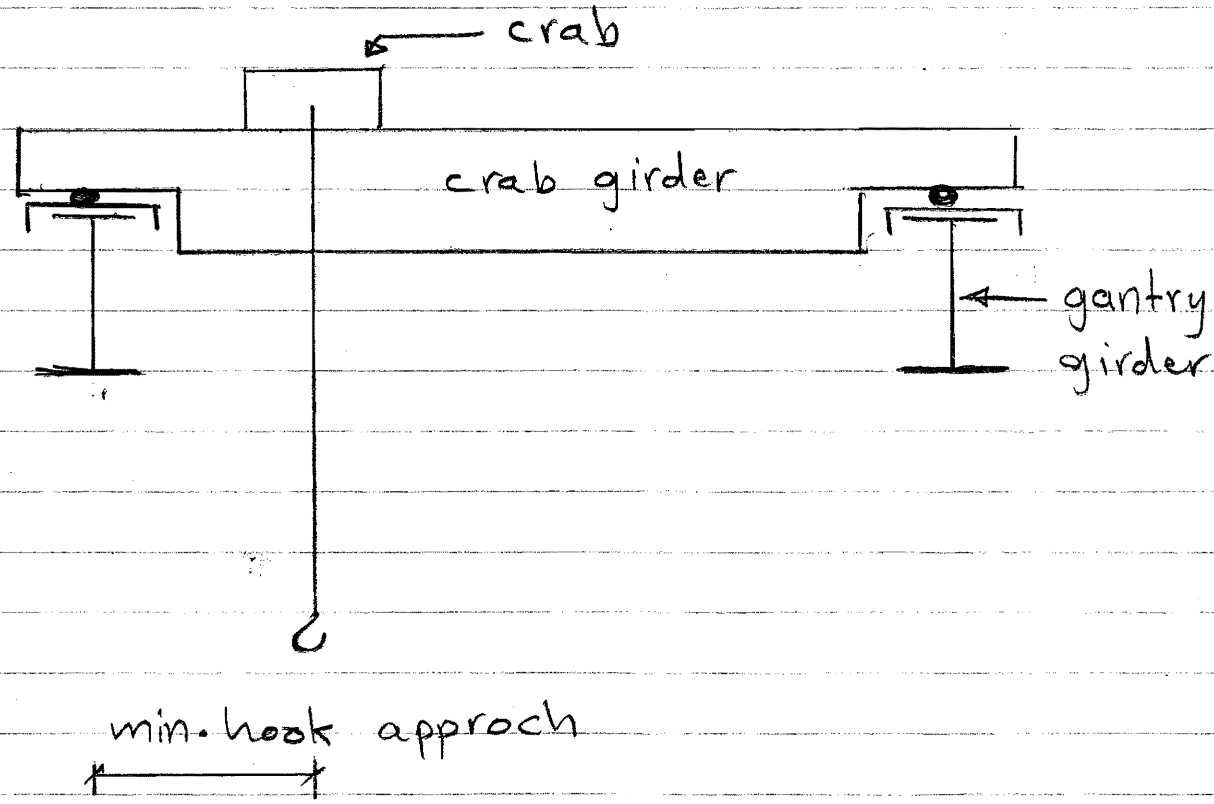
deflection due to the unfactored imposed load is

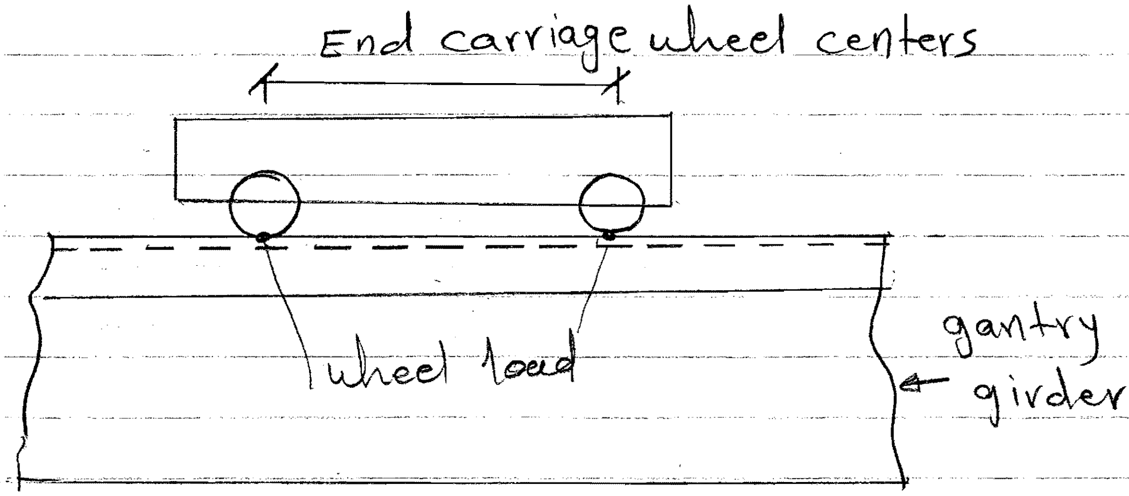
$$\delta = \frac{5 \times 600 \times 10^3 \times 11000^3}{384 \times 205 \times 10^3 \times 266483 \times 10^4}$$
$$= 19.03 \text{ mm}$$

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

$\therefore \delta <$ allowable deflection $\therefore$ ok

Design of Gantry girder (Crane beam)





Loads

Crane beam as subjected to:

1- Vertical loads
- self weight
- weight of the crane
- hook load
- impact

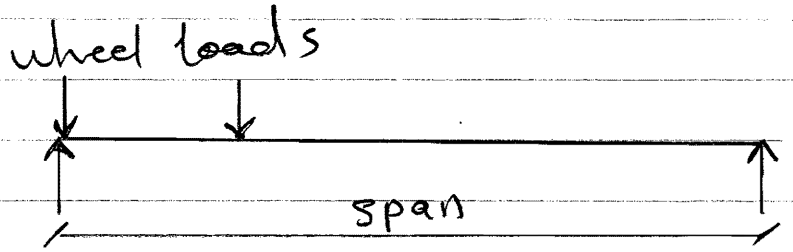
2- Horizontal loads
- For electric overhead cranes
  10 % (crab load + load lifted)
- For hand operated cranes
  5% (crab load + load lifted)

Note

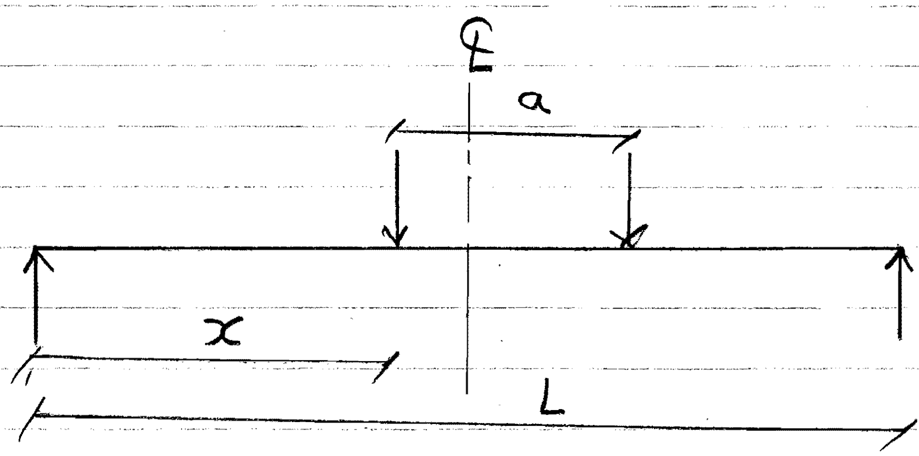
The following allowance shall be deemed to cover all forces set up by vibration, shock, impact, ect. :
For loads acting vertically static wheel loads shall be increased by
- For electric overhead cranes 25 %
- For hand operated cranes 10 %

Maximum shear and moment

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



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 nearst the centre of the girder



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 $\lambda_{LT} = u \nu \lambda \sqrt{\beta_w}$
- Find p_b from Table 17 for welded sections
- $M_b = S_x . p_b$ 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

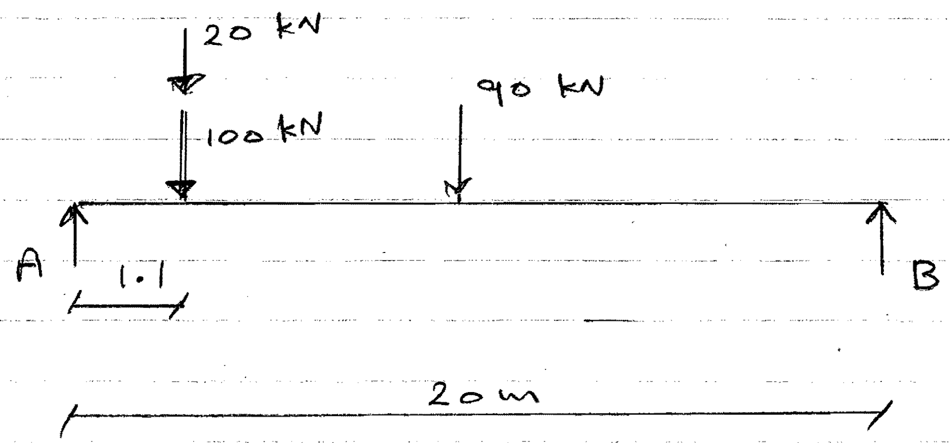
Ex

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
spane of crane beam = 5.5 m
self weight of crane beam = 8 kN
Assume $\gamma = 1$, $\gamma' = 1$, $\beta_w = 1$
Use grade S275 steel

solution

1) wheel loads



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

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

$= 79.2$ kN

the vertical wheel load including impact =

$= 79.2 + 0.25 \times 79.2 = 99$ kN

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

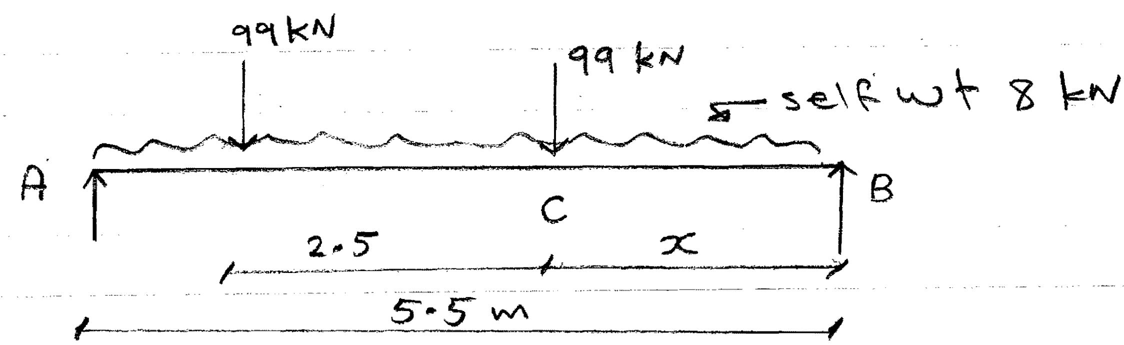
$= \frac{1}{4} \times 0.1 \times (20 + 100) = 3$ kN

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

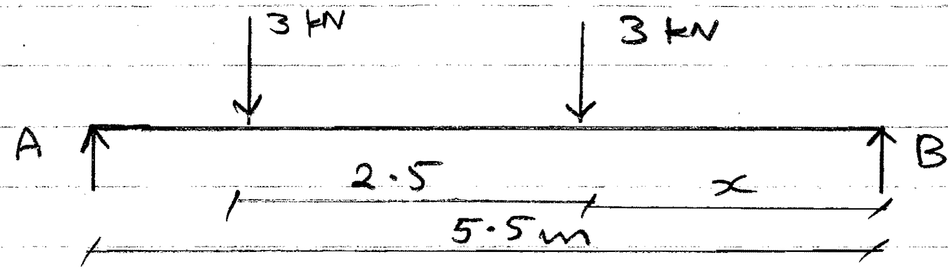
load factors (Table 2 of BS 5950)

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

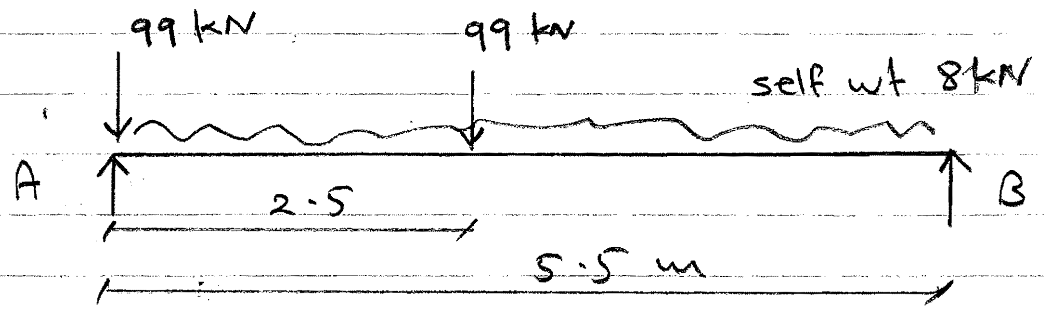
Maximum moment and shear



vertical loads - maximum moment



horizontal loads - maximum moment



load's causing max. vertical shear

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

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

- Dead load

$R_B = 4\ kN$

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

- Crane load, including impact

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

$M_c = 76.5 \times 2.125 = 162.6\ kN.m$

- The max. horizontal moments

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

$$M_c = 2.32 \times 2.125 = 4.93 \text{ kN.m}$$

- The max. vertical shear
  due to dead load, $R_A = 4$ 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

$$M_c = (1.4 \times 5.22) + (1.6 \times 162.6) = 267.5 \text{ kN.m}$$

max. shear

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

2)

2) vertical crane loads with impact and horizontal crane loads acting together

max. vertical moment =

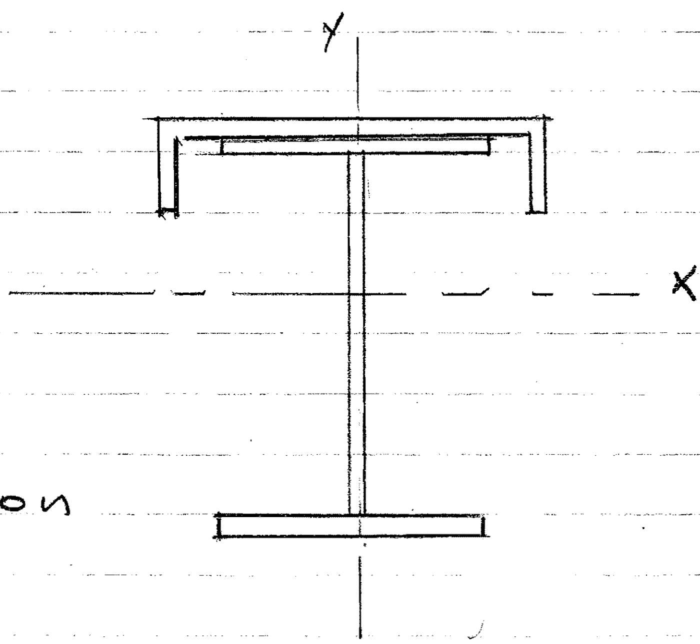
$M_c = (1.4 \times 5.22) + (1.4 \times 162.6) = 234.95$ kN.m

max. horizontal moment =

$M_c = 1.4 \times 4.93 = 6.9$ kN.m

Buckling resistance moment for the x-x axis

Try 457×191×74 UB + 254×76 Channel



for the crane beam

$r_y = 62$ mm

$S_x = 2086.8$ cm^3

For the top section

$Z_y = 331$ cm^3

$I_y = 4202$ cm^3

جميع هذه المعلومات تعطى في إنشاءات

top flange thickness = 22.6 mm total

To find pb

L_E = span = 5500 mm

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

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

$$\lambda_{LT} = u \nu \lambda \sqrt{\beta_w} = 1 \times 1 \times 88.7 \times 1 = 88.7$$

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

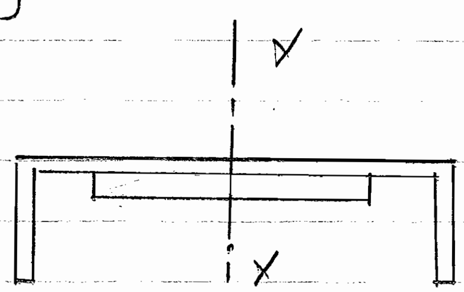
$p_b = 132\ N/mm^2$

$$M_b = p_b\, S_x = 132 \times 2086.8 \times 10^3 \times 10^{-6} = 275.5\ kN{\cdot}m$$

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

$Z_y = 331\ cm^3$

$I_y = 4202$



section resisting horizontal moment

$$M_{cx} = p_y \cdot Z_y = 265 \times 331 \times 10^3 = 87.7\ kN{\cdot}m$$

Check beam in bending

1) Vertical moment, no horizontal moment
$M_x = 267.5$ kN.m < 275.5 kNm ok

2) Vertical moment with impact + horizontal moment

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

$$= 0.93 < 1.0 \qquad \therefore \underline{ok}$$

∴ The crane beam is satisfactory in bending.

Shear capacity

$P_c = 0.6\ Dt\ p_y$
$= 0.6 \times 457 \times 9 \times 275 = 678.6$ kN

Max. factored shear = 250.4 kN

∴ ok