MADE EASY

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Leading Institute for ESE, GATE & PSUs

Detailed Solutions

ESE-2026 Mains Test Series

Civil Engineering Test No : 7

Section A : Design of Steel Structure + Hydrology

1. (a) (i) Solution:

Plane section remain plane and normal to the axis of bending at all stages. Thus it is assumed that strains are proportional to the distance from the neutral axis.

(b) The stress-strain relationship is idealized to consist of two straight lines as shown in figure.

(c) The stress in any fibre can be found from its strain according to the stress-section curve idealized as above without reference to other fibres. The shearing strains are neglected.

(d). The deformations are assumed to be small, so the slope of the beam at any point may be assumed to be equal to its tangent.

(e) Steel is ductile, able to deform plastically without fracture.

(f) The properties of steel in compression are assumed to be the same in tension.

(g) The influence of axial and shearing force are neglected.

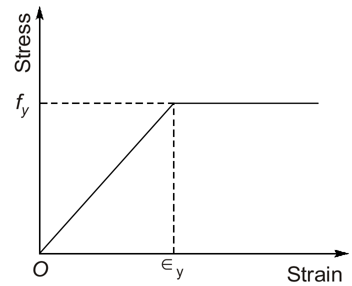
(h) Strain energy stored due to elastic bending is ignored.

(i) The connections provide full continuity so that plastic moment can be transmitted through them.

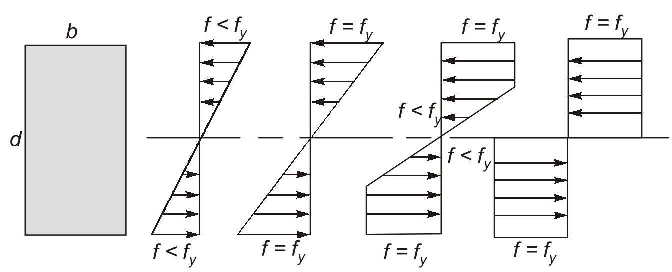
(j) Members are initially straight and prismatic and instability does not develop before plastic action develops.

(k) There is an axis of symmetry in the cross-section.

(e) Cross-section must be symmetrical w.r.t. the plane of loading (to avoid affect of twisting).



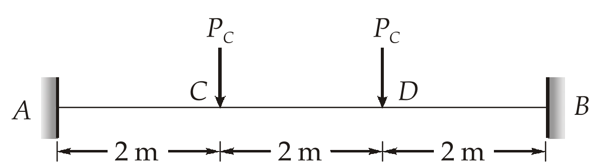
Idealised Stress-Strain diagram



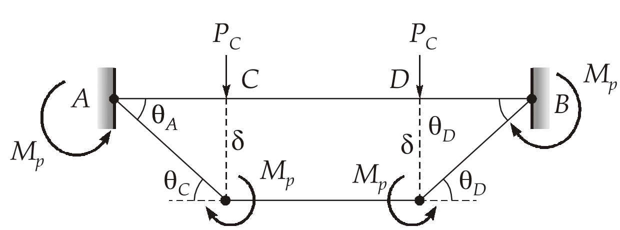
Plastic moment capacity of a beam section

1. (a) (ii) Solution:

Given data



Due to symmetry of the beam and loading, plastic hinge will get develop at both the fixed ends and under both the concentrated loads simultaneously. The mechanism is as shown below. Assume the central portion between the loads moves downward uniformly by a displacement δ.



Internal work done (W_i) is the sum of moments at all hinges multiplied by their rotations:

$$W_e = W_i = M_P(\theta_A + \theta_C + \theta_D + \theta_B)$$

as
$$\theta_A = \theta_B = \theta_C = \theta_D = \frac{\delta}{2}$$

$\therefore \quad P_c \times \delta + P_c \times \delta = M_p(\theta_A + \theta_C + \theta_D + \theta_B)$

$\Rightarrow \quad P_c \times \delta + P_c \times \delta = M_p\left(\frac{\delta}{2}+\frac{\delta}{2}+\frac{\delta}{2}+\frac{\delta}{2}\right)$

$\Rightarrow \quad 2P_c\delta = 2\, M_p\delta$

$\Rightarrow \quad P_c = M_P$

1. (b) Solution:

Given data

Factored load, $P = 1850$ kN

Grade of steel, Fe 410, $f_y = 250$ MPa

Grade of concrete, M25, $f_{ck} = 25$ MPa

Ends are machined for bearing

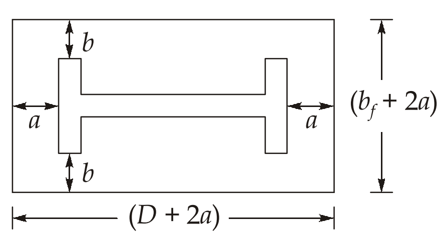
Required area of slab base

The permissible bearing pressure on concrete,

$$\sigma_c = 0.45 \times f_{ck}$$

$$\because \quad A = \frac{P}{0.45\times f_{ck}} = \frac{1850\times10^3}{0.45\times25}$$

$$\Rightarrow \quad A = 164444.444 \text{ mm}^2$$



Dimensions of the base plate

Let the equal overhang on all sides be a,

$$\because \quad A = (D + 2a) \times (b_f + 2a)$$

$$\Rightarrow \quad 164444.444 = (350 + 2a)(250 + 2a)$$

$$\Rightarrow \quad 4a^2 + 1200\, a - 76944.444 = 0$$

$$a = 54.294 \text{ mm} \approx 60 \text{ mm (provided)}$$

$$L = D + 2a = 350 + 2 \times 60 = 470 \text{ mm}$$

$$B = b_f + 2a = 250 + 2 \times 60 = 370 \text{ mm}$$

Actual bearing pressure

$$w = \frac{1850 \times 10^3}{470 \times 370}$$

$$w = 10.638 \text{ MPa} < 0.45 f_{ck}$$

Thickness of base plate

For equal overhangs ($a = b$),

$$t_s = \sqrt{\frac{2.5 \times w \times (a^2 - 0.3a^2) \times \gamma_{m0}}{f_y}}$$

$$\Rightarrow \quad t_s = \sqrt{\frac{2.5 \times 10.638 \times (60^2 - 0.3 \times 60^2) \times 1.1}{250}}$$

$$\Rightarrow \quad t_s = 17.172 \text{ mm} > t_f \text{ (Ok)}$$

Provide a 20 mm thick plate.

1. (c) Solution:

Given data:

Horton's equation: $f = f_c + (f_0 - f_c)e^{-kt} = 30 + 50e^{-2.2t}$ mm/hr

Final infiltration capacity: $f_c = 30$ mm/hr

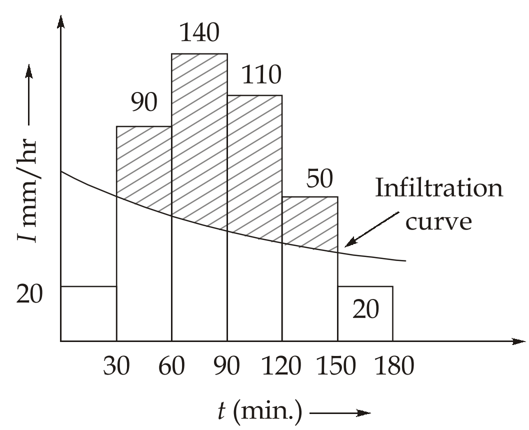
Initial infiltration capacity: $f_o = 80$ mm/hr

The incremental rainfall and rainfall intensity are derived from the mass curve:

Time (min)	Accumulated Rainfall (mm)	Incremental Rainfall (mm)	Rainfall Intensity i(mm/hr)
0–30	10	10	20
30–60	55	45	90
60–90	125	70	140
90–120	180	55	110
120–150	205	25	50
150–180	215	10	20

Using the Horton's equation, the infiltration capacity at each time is:

Time (min)	Time (hr)	Infiltration Capacity *f* (mm/hr)
0	0	80
30	0.5	46.64
60	1.0	35.54
90	1.5	31.84
120	2.0	30.61
150	2.5	30.20
180	3.0	30.06



Comparing rainfall intensity i with infiltration capacity f:

- 0–30 min: $i = 20 < f \rightarrow$ no runoff
- 30–150 min: $i > f \rightarrow$ runoff occurs
- 150–180 min: $i = 20 < f \rightarrow$ no runoff

The effective rainfall P_e is calculated for the period from $t = 0.5$ hr to $t = 2.5$ hr:

Total rainfall during this period = 215 – 20 = 195 mm

Infiltration loss F from 0.5 to 2.5 hr:

$$F = \int_{0.5}^{2.5} \left(30 + 50e^{-2.2t}\right) dt$$

$$\Rightarrow \quad F = \left[30t - 22.727e^{-2.2t}\right]_{0.5}^{2.5}$$

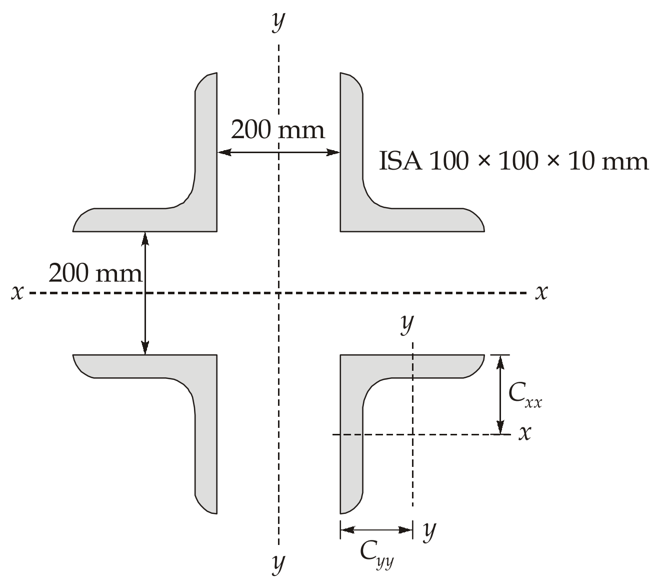
$$\Rightarrow \quad F = 67.472 \text{ mm}$$

Effective rainfall:

$$P_e = 195 - 67.472$$

$$\Rightarrow \quad P_e = 127.528 \text{ mm}$$

1. (d) Solution:



Total cross-sectional area

$$A_{total} = 4 \times 1903 = 7612 \text{ mm}^2$$

Distance from centroidal axis of built-up section to centroid of each angle

$$d = \frac{200}{2} + 28.4 = 128.4 \text{ mm}$$

Moment of inertia of built-up section about centroidal axis using parallel axis theorem

$$I_{XX} = 4[I_{XX} + Ad^2]$$

$$\Rightarrow \quad I_{XX} = 4[1.77 \times 10^6 + 1903\,(128.4)^2]$$

$$\Rightarrow \quad I_{XX} = 132.576 \times 10^6 \text{ mm}^4$$

Radius of gyration

$$r = \sqrt{\frac{I_{XX}}{A_{total}}} = \sqrt{\frac{132.576 \times 10^6}{7612}}$$

$$\Rightarrow \quad r = 131.972 \text{ mm}$$

Slenderness ratio

$$\lambda_{actual} = \frac{L_{eff}}{r} = \frac{6000}{131.972} = 45.464$$

As per IS 800:2007 Cl. 7.6.1.5, effective slenderness ratio for laced column

$$\lambda_{effective} = 1.05 \times 45.464 = 47.737$$

Design compressive stress

From given table

For $\lambda = 40, f_{cd} = 198$ MPa

For $\lambda = 50, f_{cd} = 183$ MPa

Interpolating for $\lambda = 47.737$,

$$f_{cd} = 198 - \frac{(198-183)}{(50-40)}(47.737-40)$$

$$\Rightarrow \quad f_{cd} = 186.394 \text{ MPa}$$

Safe factored axial load

$$P_d = A_{\text{total}} \times f_{cd}$$

$$\Rightarrow \quad P_d = 7612 \times 186.394$$

$$\Rightarrow \quad P_d = 1418831.128 \text{ N}$$

$$\Rightarrow \quad P_d = 1418.831 \text{ kN}$$

The safe factored axial compressive load for the laced built-up column is 1418.834 kN.

1. (e) Solution:

Factors Affecting Evaporation Losses

The following factors influence the rate of evaporation from a water surface:

- **Area of the water surface:** Evaporation is directly proportional to the surface area. A larger exposed area leads to higher evaporation losses.
- **Depth of water:** The depth of a water body significantly impacts evaporation. Greater depths typically reduce evaporation during summer and increase it during winter.
- **Humidity:** Higher atmospheric humidity results in lower evaporation rates. This occurs because the moisture gradient between the zone of higher moisture (water surface) and lower moisture (air) is reduced.
- **Wind velocity:** Increased wind velocity or turbulence removes the saturated layer of air near the water surface, facilitating quicker diffusion and dispersion of vapor, thus increasing evaporation.
- **Temperature:** Higher temperatures increase saturation vapor pressure, which in turn increases evaporation. Consequently, evaporation is greater in summer or hot climates compared to winter or cold climates.

- **Atmospheric pressure:** According to Dalton's law, higher atmospheric pressure leads to lower evaporation. While pressure is lower at high altitudes, the accompanying decrease in temperature often counteracts the expected increase in evaporation.
- **Quality of water:** Dissolved salts in water reduce the saturated vapor pressure, which consequently lowers the evaporation rate. For example, evaporation decreases by approximately 1% for every 1% increase in salinity. Additionally, turbidity can affect heat transfer within the water body, influencing evaporation.

Evaporation losses from water bodies can be reduced using the following methods:

1. **Reduction of surface area:** Evaporation is directly proportional to exposed surface area. Hence, designing deeper reservoirs instead of wide and shallow ones helps reduce losses. Narrow and deep storage minimizes the contact area with the atmosphere.
2. **Mechanical covers:** A physical barrier can be provided over the water surface to restrict evaporation. These include:
   a. Permanent roofs over tanks
   b. Temporary covers
   c. Floating covers such as rafts or lightweight sheets

   These methods are effective for small water bodies like tanks and ponds but are not economical for large reservoirs.
3. **Use of chemical films:** Thin monomolecular films are formed on the water surface using chemicals like cetyl alcohol and stearyl alcohol. These films:
   a. Act as evaporation inhibitors by resisting water molecule escape
   b. Are flexible and self-healing in nature
   c. Allow oxygen and carbon dioxide exchange, thus maintaining water quality
   d. Are colourless, odourless, and non-toxic

   However, they require periodic replenishment due to wind action, oxidation, and disturbance by birds or insects.

   Overall, chemical films are the most practical method, capable of reducing evaporation losses by about 20-60%, especially in small reservoirs.

2. (a) (i) Solution:

(a) Standard Project Flood (SPF)

The Standard Project Flood is the flood that would result from the most severe combination of meteorological and hydrological conditions that are considered "reasonably characteristic" of the region. It is derived using the Standard Project Storm (SPS), which is the heaviest rainstorm observed in the region of the basin. It represents a high-magnitude flood, but unlike the PMF, it excludes extremely rare or "catastrophic" combinations of circumstances. It is typically used for designing projects where the failure might cause significant damage but not a total catastrophe, such as intermediate dams or major canal headworks.

(b) Probable Maximum Flood (PMF)

The Probable Maximum Flood is the theoretical maximum flood that is physically possible in a particular basin. It is calculated based on the Probable Maximum Precipitation (PMP), assuming the most critical combination of atmospheric moisture, wind speed, and antecedent moisture conditions. The return period of a PMF is extremely large (often considered thousands of years), representing a near-zero risk of exceedance. This is the design standard for "High Hazard" structures, such as large dams, where failure would lead to a massive loss of life and property.

(c) Design Flood

The Design Flood is the specific flood magnitude adopted for the design of a particular hydraulic structure. It is chosen based on the importance of the structure and the economic consequences of its failure. The design flood may be equal to the PMF, the SPF, or a flood with a specific return period Engineering codes provide guidelines on whether to use *PMF* or *SPF* as the Design Flood based on the dam's storage capacity and hydraulic head.

2. (a) (ii) Solution:

Given data

Number of years of record: $N = 16$

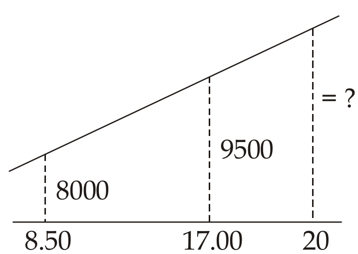
Required return period: $T = 20$ years

The recorded flood discharges are arranged in descending order and assigned rank m.

The probability of exceedance is calculated using the Weibull formula: $P = \dfrac{m}{N+1}$ and the corresponding return period is $T = \dfrac{1}{P}$.

Rank (m)	Q (m³/s)	$P = \frac{m}{N+1}$	T (years)
1	9500	0.059	17.00
2	8000	0.118	8.50
3	7900	0.176	5.67
4	6800	0.235	4.25
5	6200	0.294	3.40
6	5400	0.353	2.83
7	5100	0.412	2.43
8	4700	0.471	2.13
9	4000	0.529	1.89
10	3800	0.588	1.70
11	3500	0.647	1.55
12	3200	0.706	1.42
13	3100	0.765	1.31
14	2800	0.824	1.21
15	2500	0.882	1.13
16	2200	0.941	1.06

The return period of 20 years lies outside the recorded range, so extrapolation is done using the highest two ranks:



$$Q_{20} = Q_1 + \frac{Q_1 - Q_2}{T_1 - T_2} \times (T - T_1)$$

$$\Rightarrow \quad Q_{20} = 9500 + \frac{9500 - 8000}{17.00 - 8.50} \times (20 - 17.00) = 9500 + \frac{1500}{8.5} \times 3$$

$$\Rightarrow \quad Q_{20} = 9500 + 529.4118 = 10029.412 \text{ m}^3/\text{s}$$

The discharge of 6000 m³/s lies between rank 5 (6200 m³/s, P = 0.294) and rank 6 (5400 m³/s, P = 0.353). Using linear interpolation:

$$P = P_6 - \frac{P_6 - P_5}{Q_6 - Q_5} \times (6000 - 5400)$$

$$\Rightarrow \quad P = 0.353 - \frac{0.353 - 0.294}{6200 - 5400} \times (6000 - 5400)$$

$$\Rightarrow \quad P = 0.311$$

2. (b) Solution:

Given data

Angle section *ISA* 80 × 50 × 8

Gusset plate thickness = 10 mm

Factored tensile load, P = 220 kN

Transverse weld length = 80 mm

Ultimate stress, f_u = 410 MPa

Yield stress, f_y = 250 MPa

Partial safety factor for shop weld, $\gamma_{mw} = 1.25$

Size of weld

Minimum size of weld for 10 mm plate = 3 mm

Maximum size of weld at square edge = 8 – 1.5 = 6.5 mm

Maximum size of weld at rounded toe = $\frac{3}{4}\times 8$ = 6 mm

Adopt weld size, s = 5 mm

$$\text{Design strength of weld} = \frac{f_u}{\sqrt{3}\gamma_{mw}}\times 0.7s\times l_{eff}$$

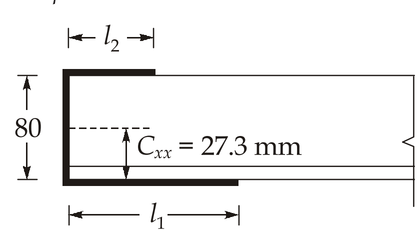
Equating weld strength to factored load,

$$\Rightarrow \quad 220\times 10^3 = \frac{0.7\times 5\times l_{eff}\times 410}{\sqrt{3}\times 1.25}$$

$$\Rightarrow \quad l_{eff} = 331.926 \text{ mm}$$

Total required weld length including end returns

$$l_{req} = 331.926 + (2\times 5) = 341.926 \text{ mm}$$



Let longitudinal weld lengths be l_1 and l_2.

$$l_1 + l_2 + 80 = 341.926 \text{ mm}$$

$$\Rightarrow \quad l_1 + l_2 = 261.926 \text{ mm} \qquad \text{...(i)}$$

Distribution of longitudinal welds

Taking moment equilibrium about centroid C_{yy},

$$l_1 \times 27.3 - l_2 \times (80 - 27.3) - 80 \times \left(\frac{80}{2} - 27.3\right) = 0$$

$$\Rightarrow \quad 27.3\, l_1 - 52.7 l_2 - 1016 = 0$$

On solving equation (i) and (ii)

$$l_2 = 76.682 \text{ mm}$$
$$l_1 = 185.244 \text{ mm}$$

Provide

$$l_1 = 186 \text{ mm}, l_2 = 77 \text{ mm}$$

Check for block shear strength

Block shear strength is the minimum of

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$T_{db1} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}$$

Areas for 10 mm gusset plate

$$A_{vg} = A_{vn} = (186 + 186) \times 10 = 3720 \text{ mm}^2$$
$$A_{tg} = A_{tn} = 80 \times 10 = 800 \text{ mm}^2$$

First value

$$T_{db1} = \frac{3720 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 800 \times 410}{1.25}$$

$$T_{db1} = 724.283 \text{ kN}$$

Second value

$$T_{db2} = \frac{0.9 \times 3720 \times 410}{\sqrt{3} \times 1.25} + \frac{800 \times 250}{1.1}$$

$$T_{db2} = 815.831 \text{ kN}$$

Design block shear strength

$$T_{db} = 724.283 \text{ kN}$$

Since 724.283 kN > 220 kN, the welded joint is safe against block shear.

2. (c) Solution:

Given data

Effective span, $L = 6.0$ m

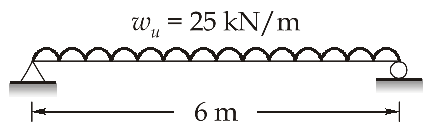
Factored UDL, $w_u = 25$ kN/m

Yield stress, $f_y = 250$ N/mm^2

Bearing length, $b_1 = 100$ mm

Modulus of elasticity, $E = 2 \times 10^5$ N/mm^2

Partial safety factor for material, $\gamma_{m0} = 1.1$



Maximum factored bending moment,

$$M_u = \frac{w_u L^2}{8} = \frac{25 \times 6^2}{8} = 112.5 \text{ kNm}$$

Maximum factored shear force,

$$V_u = \frac{w_u L}{2} = \frac{25 \times 6}{2} = 75 \text{ kN}$$

Required plastic section modulus,

$$Z_{p\ req} = \frac{M_u \times \gamma_{m0}}{f_y} = \frac{112.5 \times 10^6 \times 1.1}{250} = 495000 \text{ mm}^3 = 495 \times 10^3 \text{ mm}^3$$

Adopt ISMB 300 @ 44.2 kg/m

Since

$$Z_{p,\ provided} = 652 \times 10^3 \text{ mm}^3 > (Z_p)_{req} = 495 \times 10^3 \text{ mm}^3$$

The section is suitable.

Section classification

$$\varepsilon = \sqrt{\frac{250}{250}} = 1$$

$$\frac{b}{t_f} = \frac{140/2}{12.4} = 5.645 < 9.4\,\varepsilon$$

$$\frac{d}{t_w} = \frac{300 - 2(12.4 + 14)}{7.5} = 32.96 < 84\,\varepsilon$$

The section is plastic.

Design bending strength,

$$M_d = \frac{Z_p f_y}{\gamma_{m0}} = \frac{652 \times 10^3 \times 250}{1.1} = 148.182 \text{ kNm}$$

$$M_d = 148.182 \text{ kNm} > M_u = 112.5 \text{ kNm}$$

Hence, safe in bending.

Design shear strength,

$$V_d = \frac{f_y}{\sqrt{3}\gamma_{m0}} \times (h \times t_w) = \frac{250}{\sqrt{3} \times 1.1} \times (300 \times 7.5) = 295.236 \text{ kN}$$

$$0.6 V_d = 0.6 \times 295.236 = 177.142 \text{ kN}$$

Since

$$V_u = 75 \text{ kN} < 0.6\, V_d$$

it is a low shear case and safe.

Deflection check

Service load,

$$w = \frac{25}{1.5} = 16.667 \text{ kN/m}$$

Deflection,

$$\delta = \frac{5wL^4}{384EI} = \frac{5 \times 16.667 \times 6000^4}{384 \times 2 \times 10^5 \times 8604 \times 10^4} = 16.34 \text{ mm}$$

Permissible deflection,

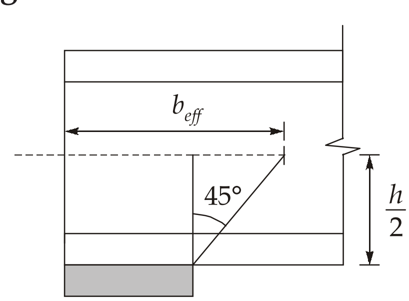
$$\frac{L}{300} = \frac{6000}{300} = 20 \text{ mm}$$

Since

$$16.34 \text{ mm} < 20 \text{ mm}$$

hence safe in deflection.

Safety against Web buckling



$b_1 = 100$ mm

Effective width,

$$b_{eff} = b_1 + n_1 = 100 + \frac{300}{2} = 250 \text{ mm}$$

Web buckling strength,

$$F_{wb} = b_{eff} \times t_w \times f_{cd} = 250 \times 7.5 \times 132 = 247500 \text{ N} = 247.5 \text{ kN}$$

$$F_{wb} = 247.5 \text{ kN} > V_u = 75 \text{ kN}$$

Hence, safe in web buckling.

Safety against web crippling

$$n_2 = 2.5(t_f + r_1) = 2.5(12.4 + 14) = 66 \text{ mm}$$

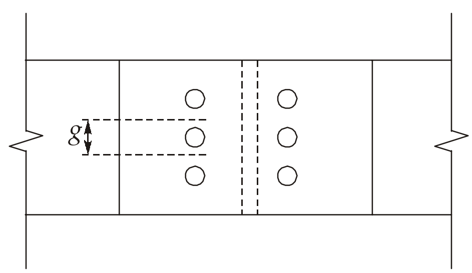
Crippling strength,

$$F_w = \frac{(b_1 + n_2)t_w f_{yw}}{\gamma_{mo}} = \frac{(100+66)\times 7.5 \times 250}{1.10} = 282.95 \text{ kN}$$

$$F_w = 282.95 \text{ kN} > V_u = 75 \text{ kN}$$

Hence, safe in web crippling.

3. (a) Solution:



Given data

Thickness of main plates,	t	$= 12$ mm
Thickness of cover plates,	t_c	$= 8$ mm
Bolt diameter,	d	$= 24$ mm
Bolt hole diameter,	d_0	$= 24 + 2 = 26$ mm
Bolt grade 4.6,	f_{ub}	$= 400$ MPa
Plate grade: Fe 410,	f_u	$= 410$ MPa, $f_y = 250$ MPa
Guage,	g	$= 80$ mm
End distance,	e	$= 1.5 \times d_0 = 1.5 \times 26 = 39$ mm

Cover plate thickness on both sides, $t_c = 8 + 8 = 16$ mm

Design Strength of Bolt

1. Shear Strength (V_{dsb})

Since it is a double cover butt joint, there are two shear planes (n = 2). As per the condition, $n_n = 1$ and $n_s = 1$.

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times 24^2 = 352.863 \text{ mm}^2$$

$$A_{sb} = \frac{\pi}{4} \times 24^2 = 452.389 \text{ mm}^2$$

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} \times (n_n A_{nb} + n_s A_{sb})$$

$$\Rightarrow \quad V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times (1 \times 352.858 + 1 \times 452.389) \times 10^{-3}$$

$$\Rightarrow \quad V_{dsb} = 148.772 \text{ kN}$$

2. Bearing Strength (V_{dpb})

Sum of cover plate thicknesses = 8 + 8 = 16 mm. Main plate thickness = 12 mm. Use minimum t = 12 mm.

$$k_b = \min\left(\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1\right)$$

$$k_b = \min\left(\frac{39}{3 \times 26}, \frac{2.5 \times 24}{3 \times 26} - 0.25, \frac{400}{410}, 1\right)$$

$$k_b = \min(0.5, 0.519, 0.976, 1) = 0.5$$

$$\Rightarrow \quad V_{dpb} = \frac{2.5 \times k_b \times d \times t \times f_u}{\gamma_{mb}}$$

$$\Rightarrow \quad V_{dpb} = \frac{2.5 \times 0.5 \times 24 \times 12 \times 410}{1.25} \times 10^{-3}$$

$$V_{dpb} = 118.08 \text{ kN}$$

Design Strength of Plate

1. Tearing Strength of Main Plate (T_{dn})

$$T_{dn} = \frac{0.9 \times f_u \times A_n}{\gamma_{m1}}$$

$$\Rightarrow \quad T_{dn} = \frac{0.9 \times 410 \times (80 - 26) \times 12}{1.25} \times 10^{-3}$$

$$\Rightarrow \quad T_{dn} = 191.29 \text{ kN}$$

2. Gross Yielding Strength of Plate (T_{dg})

$$T_{dg} = \frac{f_y \times A_g}{\gamma_{m0}}$$

$$\Rightarrow \quad T_{dg} = \frac{250 \times 80 \times 12}{1.1} \times 10^{-3}$$

$$\Rightarrow \quad T_{dg} = 218.182 \text{ kN}$$

Efficiency of Joint

Strength of joint per pitch length is the minimum of V_{dsb}, V_{dpb}, and T_{dn}.

Strength of joint = 118.08 kN

$$\text{Efficiency } (\eta) = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$$

$$\Rightarrow \quad \eta = \frac{118.08}{218.182} \times 100 = 54.12\%$$

3. (b) (i) Solution:

In structural steel design, choosing between welded and bolted connections involves balancing factors such as speed, cost, and long-term durability. Below are the advantages and disadvantages of welded connections compared to bolted connections:

Advantages of Welded Connections

- **Higher Efficiency:** Welded joints can achieve 100% efficiency, meaning the joint can be as strong as the parent metal itself.
- **Material Savings:** They eliminate the need for gusset plates, connecting angles, or splice plates, which significantly reduces the overall weight of the steel structure.
- **Aesthetic and Compact:** Welding produces a much cleaner and smoother appearance than bolts, making it ideal for architecturally exposed steelwork.
- **Air and Water Tightness:** Since the metal is fused together, welded joints are inherently leak-proof, which is essential for liquid-retaining structures like water tanks and pressure vessels.
- **No Strength Reduction:** Unlike bolted connections, welding does not require drilling holes, meaning the gross cross-sectional area of the member is fully utilized for load resistance.

Disadvantages of Welded Connections

- **Skilled Labor Requirement:** Welding requires highly trained and certified personnel, whereas bolting can often be performed by less specialized labour.

- **Inspection Difficulty:** Detecting internal defects in a weld is more complex and expensive, often requiring nondestructive testing (NDT) such as ultrasonic or radiographic testing.
- **Rigidity and Brittle Failure:** Welded joints are very stiff; if not designed correctly, they can be prone to brittle failure, especially under cyclic or fatigue loading.
- **Residual Stresses:** The intense heat generated during the welding process can cause significant thermal expansion and contraction, leading to residual stresses and potential warping of the members.
- **Environmental Sensitivity:** Quality welding is difficult to perform in poor weather conditions (such as high wind or rain) without proper shielding and site preparation.

3. (b) (ii) Solution:

Given Data

Total Runoff, $R = 7$ cm

Time interval, $\Delta t = 40$ minutes

Total storm duration, $t_r = 200$ minutes

Total Rainfall (P)

$$P = \Sigma(I \times \Delta t)$$

$$\Rightarrow \quad P = (2.5 + 6.0 + 11.0 + 3.5 + 5.0) \times (40/60)$$

$$\Rightarrow \quad P = 18.667 \text{ cm}$$

W-Index:

$$W_{\text{index}} = \frac{P - R}{t_r} = \frac{18.667 - 7}{(200/60)}$$

$$W_{\text{index}} = 3.500 \text{ cm/hr}$$

Calculation of ϕ_{index}

Assume all intervals contribute to runoff, then $W_{\text{index}} = \phi_{\text{index}}$

$$\phi = 3.5 \text{ cm/hr}$$

Verification of assumption

Compare intensities with $\phi = 3.5$ cm/hr:

$$i_1 = 2.5 < 3.5$$

$$i_2 = 6.0 > 3.5$$

$$i_3 = 11.0 > 3.5$$

$$i_4 = 3.5 = 3.5$$

$$i_5 = 5.0 > 3.5$$

Recalculate using contributing intervals i_2, i_3, i_5:

$$\phi_{index} = \frac{(6+11+5)\times\frac{40}{60}-7}{\left(\frac{3\times 40}{60}\right)}$$

$\Rightarrow$ $$\phi_{index} = 3.833 \text{ cm/hr}$$

3. (c) Solution:

Degree of static indeterminacy, $D_s = 4 - 3 = 1$

Number of independent mechanism, n = Possible locations of plastic hinge – D_s

$\therefore$ 2-Beam mechanism

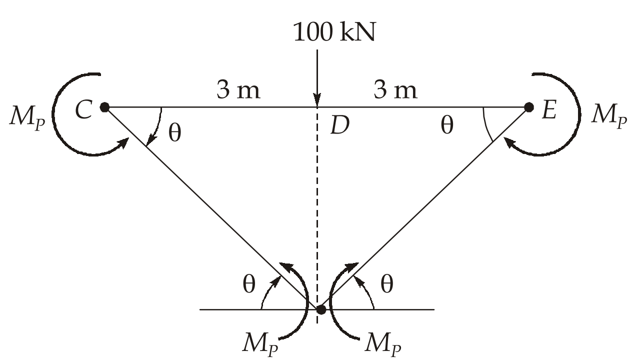
1-Sway mechanism

Beam mechanism in span CDE:

Hinges form at C, D and E.

At C, hinge forms in beam since $M_p < 2\,M_p$.

At E, hinge forms in beam since $M_p < 1.5\,M_p$.



Let rotation be θ.

Vertical displacement at D,

$$\Delta = 3\theta$$

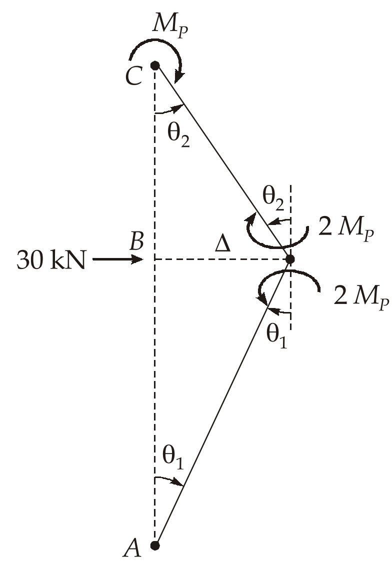
Internal work, $$W_i = M_p\theta + M_p(2\theta) + M_p\theta = 4M_p\theta$$

External work, $$W_e = 100 \times 3\theta = 300\theta$$

Equating internal and external work,

$$4M_p\theta = 300\theta$$

$$\Rightarrow \quad M_p = 75 \text{ kNm}$$



Beam Mechanism in span *ABC*

Equating, Internal work = External work

$$M_p(\theta_2) + 2M_p(\theta_1) + 2M_p(\theta_2) = 30 \times \Delta$$

Also, $\Delta = 6\theta_1 = 3\theta_2$

$$3M_p\theta_2 + 2M_p\theta_1 = 30 \times 6\theta_1$$

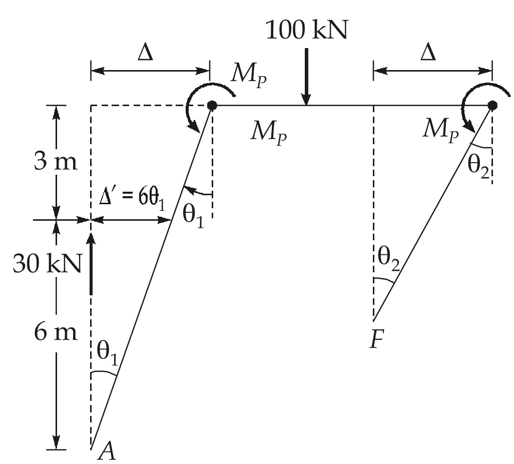
$$\Rightarrow \quad \theta_2 = 2\theta_1$$

Now, $3M_p \times 2\theta_1 + 2M_p\theta_1 = 180\,\theta_1$

$$\Rightarrow \quad M_p = \frac{180}{8} = 22.5 \text{ kN-m}$$

Sway mechanism

Hinges form at *C* and *E*.



Since lateral displacement is same for both columns,

$$\Rightarrow \quad 9\theta_1 = 6\theta_2$$

$$\Rightarrow \quad \theta_2 = 1.5\theta_1$$

Internal work, $W_i = M_p\theta_1 + M_p\theta_2$

$$W_i = M_p\theta_1 + M_p(1.5\theta_1) = 2.5M_p\theta_1$$

External work, $W_e = 30 \times \Delta' = 30 \times 6\theta_1 = 180\theta_1$

Equating internal and external work,

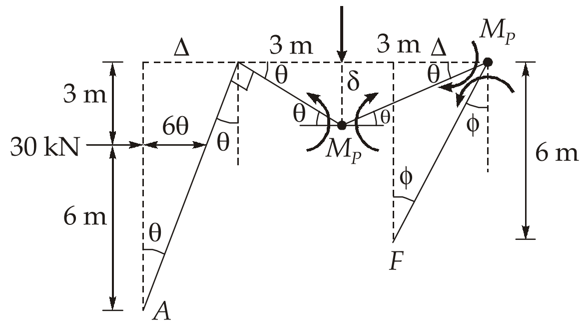
$$2.5\,M_p\theta_1 = 180\theta_1$$

$$\Rightarrow \quad M_p = 72 \text{ kNm}$$

Combined mechanism

Hinges form at D and E.

Let $\alpha = \theta_1, \phi = \theta_2 = 1.5\theta_1.$



$$\Delta = 9\theta = 6\phi$$

$\Rightarrow$ $\phi = 1.5\theta$

Internal work,

$$W_i = M_p(2\theta) + M_p\theta + M_p\phi$$

$\Rightarrow$ $W_i = 4.5\,M_p\theta$

External work,

$$W_e = 30(6\theta) + 100(3\theta) = 480\theta$$

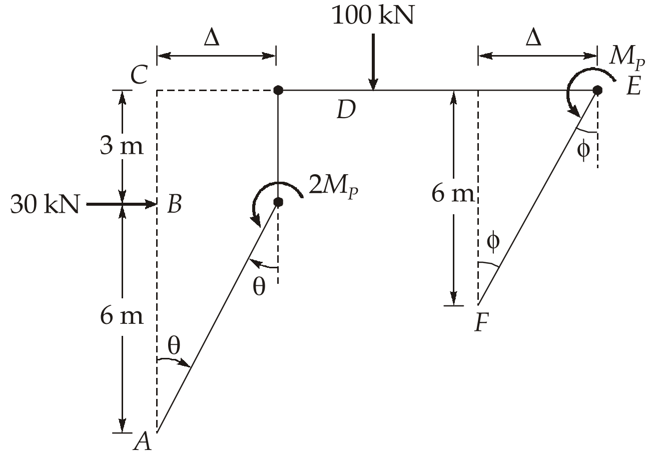
Equating internal and external work,

$$4.5M_p\theta = 480\theta$$

$\Rightarrow$ $M_p = 106.67$ KN-m

Combined mechanism

Hinges form at B and E



$\because$ $\Delta = 6\theta = 6\phi$

$\Rightarrow$ $\theta = \phi$

Internal work, $W_i = 2\,M_p(\theta) + M_p(\phi)$

$W_i = 3\,M_p\theta$

External work, $W_e = 30 \times 6\theta = 180\theta$

$\because$ $W_i = W_e$

$\Rightarrow$ $3M_p\theta = 180\,\theta$

$M_p = 60$ kN-m

$M_p = \max^m\{75, 22.5, 72, 106.67, 60\}$

$M_p = 106.67$ kN-m

4. (a) Solution:

Given data

Size of weld, $s = 10$ mm

Vertical weld length, $d = 400$ mm

Horizontal weld length, $b = 200$ mm

Eccentricity from column face, $e_{face} = 300$ mm

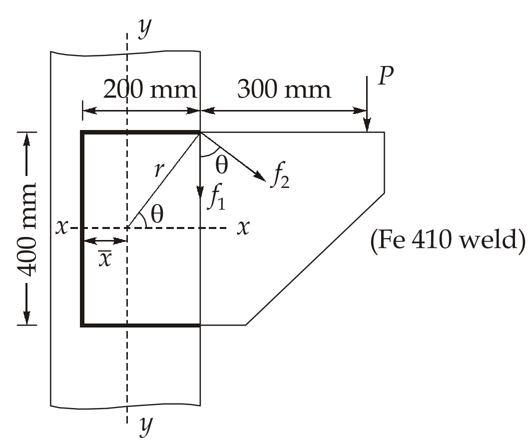
Ultimate stress, $f_u = 410$ N/mm^2

Partial safety factor for shop weld, $\gamma_{mw} = 1.25$

Effective throat thickness, $t_t = 0.7s = 0.7 \times 10 = 7$ mm

Total weld length, $L = 400 + 2 \times 200 = 800$ mm

Total weld area, $A = 800 \times 7 = 5600$ mm^2



Centroid from vertical weld,

$$\bar{x} = \frac{400 \times 7 \times 0 + 2 \times (200 \times 7) \times 100}{5600} = 50 \text{ mm}$$

Total eccentricity from centroid,

$$e = (200 - 50) + 300 = 450 \text{ mm}$$

Moment of inertia about x-axis,

$$I_{xx} = \frac{7\times 400^3}{12} + 2\left[\frac{200\times 7^3}{12} + 200\times 7(200)^2\right]$$

$$\Rightarrow \quad I_{xx} = 149.345 \times 10^6 \text{ mm}^4$$

Moment of inertia about y-axis,

$$I_{yy} = \frac{400\times 7^3}{12} + 400\times 7\times 50^2 + 2\left[\frac{7\times 200^3}{12} + 200\times 7\times 50^2\right]$$

$$\Rightarrow \quad I_{yy} = 23.345 \times 10^6 \text{ mm}^4$$

Polar moment,

$$J = I_{xx} + I_{yy} = 172.69 \times 10^6 \text{ mm}^4$$

Distance to critical point,

$$r = \sqrt{(200-50)^2 + 200^2} = \sqrt{150^2 + 200^2} = 250 \text{ mm}$$

$$\cos\theta = \frac{150}{250} = 0.6$$

Direct shear stress,

$$f_1 = \frac{P}{A} = \frac{P}{5600} \text{ (N/mm}^2\text{)}$$

Shear stress due to moment,

$$f_2 = \frac{Per}{J} = \frac{P\times 450\times 250}{172.69\times 10^6} = \frac{P}{1535.02} \text{ (N/mm}^2\text{)}$$

Resultant stress,

$$f_r = \sqrt{f_1^2 + f_2^2 + 2 f_1 f_2 \cos\theta}$$

$$\Rightarrow \quad f_r = \sqrt{\left(\frac{P}{5600}\right)^2 + \left(\frac{P}{1535.02}\right)^2 + 2\times\frac{P}{5600}\times\frac{P}{1535.02}\times 0.6}$$

$$\Rightarrow \quad f_r = 7.71934\times 10^{-4} P \text{ (N/mm}^2\text{)}$$

Design shear strength of weld,

$$f_{wd} = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3}\times 1.25} = 189.37 \text{ N/mm}^2$$

For safety, $7.71934 \times 10^{-4} \times P = 189.37$

$\Rightarrow \quad P = 245329 \text{ N} = 245.34 \text{ kN}$

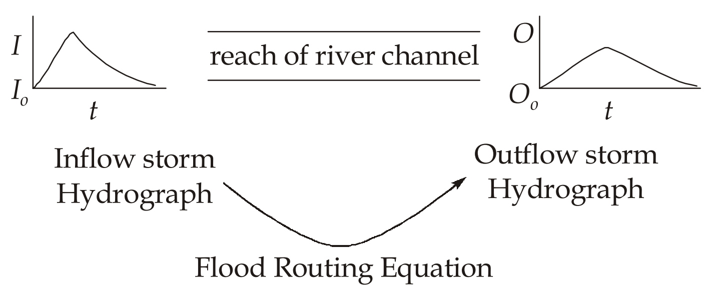
Maximum safe load that can be resisted,

$$P_{\text{safe}} = \frac{245.34}{1.5}$$

$$\Rightarrow \quad P_{\text{safe}} = 163.56 \text{ kN}$$

4. (b) (i) Solution:

In hydrology, **Routing** is a mathematical procedure used to predict the changing magnitude, speed, and shape of a water wave (hydrograph) as it moves through a river reach or a reservoir. It essentially determines the outflow hydrograph from a known inflow hydrograph, accounting for the effects of storage and resistance within the system.



Types of Routing

Hydrologic routing is broadly classified into two categories based on the physical principles used to describe the flow:

1. **Hydrologic Routing (Lumped Routing)**

   This method utilizes the **Equation of Continuity** and a simplified functional relationship between storage and discharge. It treats the entire reach as a single unit (lumped) and does not explicitly account for the momentum of the water.

   - Reservoir Routing: Used for routing through lakes or man-made reservoirs where the storage is primarily a function of the outflow $S = f(Q)$.
   - Channel Routing: Used for river reaches where storage depends on both inflow and outflow $S = f(I, Q)$, such as the Muskingum Method.

2. **Hydraulic Routing (Distributed Routing)**

   This method is more complex and accurate, as it uses both the Equation of Continuity and the Equation of Momentum (the Saint-Venant equations). It calculates flow as a function of both space and time (x, t) throughout the reach.

Applications of Flood Routing

Flood routing is a critical tool in water resource engineering and disaster management. Its primary applications include:

I. Flood Forecasting: Predicting the arrival time and peak magnitude of a flood wave at downstream locations to provide early warnings to communities.

II. Reservoir Design: Determining the required storage capacity of a dam to safely "attenuate" (reduce) a design flood peak, ensuring the structure's safety. MWL (max water level) of a dam is fixed using reservoir routing.

III. Protection Works: Designing levees, floodwalls, and bypass channels by estimating the maximum water surface elevations expected during a flood.

IV. Study of Channel Modifications: Assessing how changes to a river, such as dredging or lining, will impact the speed and height of flood waves downstream.

V. Environmental Impact: Evaluating how dam operations affect downstream flow regimes, which is vital for maintaining aquatic habitats.

4. (b) (ii) Solution:

Routing period, $\Delta t = 6$ h

Storage time constant, $K = 12$ h

Weighting factor, $x = 0.2$

Initial inflow, $I_0 = 10\ m^3/s$

Initial outflow, $Q_0 = 10\ m^3/s$

The routing constants are determined using

$$C_0 = \frac{-Kx + 0.5\Delta t}{K(1-x) + 0.5\Delta t}$$

$$C_1 = \frac{Kx + 0.5\Delta t}{K(1-x) + 0.5\Delta t}$$

$$C_2 = \frac{K(1-x) - 0.5\Delta t}{K(1-x) + 0.5\Delta t}$$

Substituting the given values,

$$C_0 = \frac{-12\times 0.2 + 0.5\times 6}{12\times(1-0.2) + 0.5\times 6} = 0.048$$

$$C_1 = \frac{12\times 0.2 + 0.5\times 6}{12\times(1-0.2) + 0.5\times 6} = 0.429$$

$$C_2 = \frac{12\times(1-0.2)-0.5\times6}{12\times(1-0.2)+0.5\times6} = 0.523$$

Sum check

$$0.048 + 0.429 + 0.523 = 1$$

For the first internal, 0 to 6 hr

$$I_1 = 10 \text{ m}^3/\text{sec}$$
$$I_2 = 20 \text{ m}^3/\text{sec}$$
$$Q_1 = 10 \text{ m}^3/\text{sec}$$

The routing equation used is

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1$$

Time (h)	I (m^3/s)	C_0I_2	C_1I_1	C_2Q_1	Q(m^3/s)
0	10	–	–	–	10
6	20	0.96	4.29	5.23	10.48
12	50	2.4	8.58	5.48	16.46
18	60	2.88	21.45	8.609	32.939
24	55	2.64	25.74	17.227	45.607
30	45	2.16	23.595	23.852	49.607
36	35	1.68	19.305	25.944	46.929
42	25	1.2	15.015	24.544	40.759
48	15	0.72	10.725	21.317	32.762

Attenuation in the peak = 60 – 49.607 = 10.393 m^3/s

Leg = 30 – 18 = 12 hours

4. (c) Solution:

Given data

Factored load, P = 450 KN

Gusset plate thickness, t_{gp} = 12 mm

Angle section = 2ISA 100 × 75 × 8 mm

Thickness of angle, t = 8 mm

Bolt diameter, d = 20 mm

Bolt hole diameter, d_0 = 22 mm

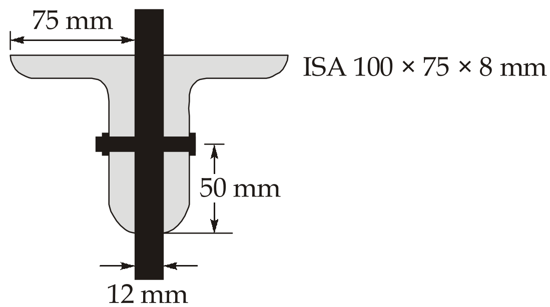
Bolt grade 4.6: f_{ub} = 400 MPa, f_{yb} = 240 MPa

Steel grade Fe410: $f_u = 410$ MPa, $f_y = 250$ MPa

Pitch, $p = 60$ mm

End distance, $e = 40$ mm

Partial safety factors: $\gamma_{mb} = 1.25$, $\gamma_{m0} = 1.1$, $\gamma_{m1} = 1.25$



Strength of bolt in double shear

Since one shear plane passes through threads and one through shank,

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$$

Area of shank,

$$A_{sb} = \frac{\pi}{4}d^2 = \frac{\pi}{4}(20)^2 = 314.16 \text{ mm}^2$$

Net area at threads,

$$A_{nb} = 0.78\, A_{sb} = 0.78 \times 314.16 = 245.04 \text{ mm}^2$$

Now, $$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25}(245.04 + 314.16)$$

$\Rightarrow$ $$V_{dsb} = \frac{400}{2.165} \times 559.2 = 103311 \text{ N}$$

$\Rightarrow$ $$V_{dsb} = 103.31 \text{ KN}$$

Strength of bolt in bearing

$$k_b = \min\left(\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1\right)$$

$$k_b = \min\left(\frac{40}{66}, \frac{60}{66} - 0.25, \frac{400}{410}, 1\right)$$

$$k_b = \min(0.606, 0.659, 0.976, 1) = 0.606$$

Thickness governing bearing,

$$t = \min(8+8, 12) = 12 \text{ mm}$$

Now,
$$V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$$

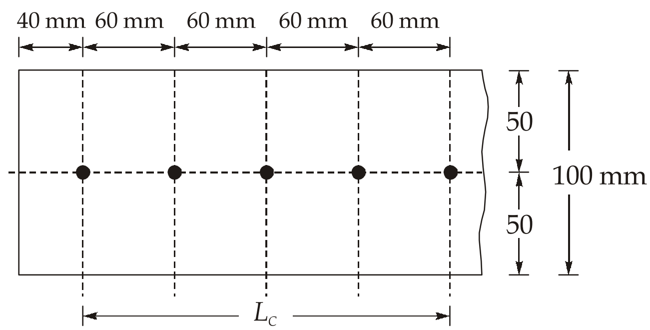
$$\Rightarrow \quad V_{dpb} = \frac{2.5 \times 0.606 \times 20 \times 12 \times 410}{1.25}$$

$$\Rightarrow \quad V_{dpb} = 119260.8 \text{ N} = 119.26 \text{ KN}$$

Bolt value,
$$V_{db} = \min(103.31, 119.26) = 103.31 \text{ KN}$$

Number of bolts required,
$$n = \frac{450}{103.31} = 4.35$$

Provide 5 bolts as shown below.



Check for strength of tension member

Gross area of double angles,
$$A_g = 2[(100-4) \times 8 + (75-4) \times 8]$$
$$A_g = 2(768 + 568) = 2672 \text{ mm}^2$$

Strength against yielding,
$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2672 \times 250}{1.1}$$
$$T_{dg} = 607272.7 \text{ N} = 607.27 \text{ KN}$$

Strength against rupture

Net area of connected legs,
$$A_{nc} = 2(100 - 22 - 4) \times 8$$
$$A_{nc} = 1184 \text{ mm}^2$$

Gross area of outstanding legs,

$$A_{go} = 2(75 - 4) \times 8 = 1136 \text{ mm}^2$$

Length of connection,

$$L_c = 4 \times 60 = 240 \text{ mm}$$

$$\{b_s = 50 + 75 - 8 = 117 \text{ mm}, w = 75 \text{ mm}\}$$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \frac{f_u}{f_y} \times \frac{\gamma_{mo}}{\gamma_{m1}} \ge 0.7$$

Shear lag factor,

$$\beta = 1.4 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{117}{240}\right) = 1.188 \le 1.44 \ge 0.7 \text{ Ok}$$

Here,

$$\frac{f_u}{f_y} \times \frac{\gamma_{mo}}{\gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44$$

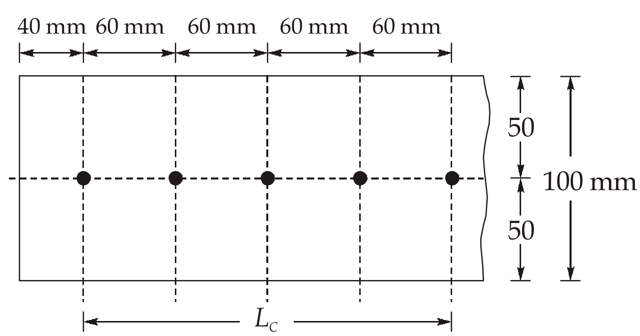
Now,

$$T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

$$T_{dn} = \frac{0.9 \times 410 \times 1184}{1.25} + \frac{1.188 \times 1136 \times 250}{1.1}$$

$$T_{dn} = 656236.8 \text{ N} = 656.24 \text{ kN}$$

Strength against block shear



$$A_{vg} = (40 + 4 \times 60) \times 8 \times 2 = 4480 \text{ mm}^2$$

$$A_{vn} = (280 - 4.5 \times 22) \times 8 \times 2 = 2896 \text{ mm}^2$$

$$A_{tg} = 50 \times 8 \times 2 = 800 \text{ mm}^2$$

$$A_{tn} = (50 - 0.5 \times 22) \times 8 \times 2 = 624 \text{ mm}^2$$

Now,

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} = \frac{4480 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 624 \times 410}{1.25}$$

$$= 772.052 \text{ kN}$$

$$T_{db2} = \frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg}f_y}{\gamma_{m0}} = \frac{0.9 \times 2896 \times 410}{\sqrt{3} \times 1.25} + \frac{800 \times 250}{1.1}$$

$$= 675.394 \text{ kN}$$

$$T_{db} = \min(772.052, 675.394) = 675.394 \text{ KN}$$

Since, the minimum design strength of the member is 607.27 KN which is greater than 450 KN, the section 2ISA 100 × 75 × 8 mm with 5 numbers of 20 mm diameter bolts is safe.

Section B : Structural Analysis-1+CPM PERT-1, Flow of fluids, hydraulic machines and hydro power-2

5. (a) (i) Solution:

Comparison of Structural Analysis Methods

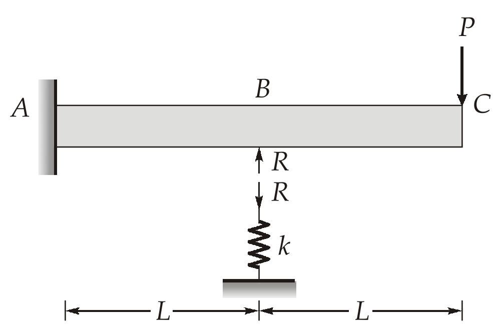
Feature	Flexibility Method (Force Method)	Stiffness Method (Displacement Method)
Primary Unknowns	Redundant forces and moments.	Joint displacements (translations and rotations).
Governing Equations	Compatibility Equations (ensuring the structure remains continuous).	Equilibrium Equations (ensuring forces at joints balance out).
Indeterminacy Used	Static Indeterminacy (D_s)	Kinematic Indeterminacy (D_k)
Base Structure	Converts the indeterminate structure into a statically determinate primary structure by removing redundants.	Converts the structure into a kinematically determinate structure by fixing all joints.
Matrix Type	Flexibility Matrix [f].	Stiffness Matrix [k]
Matrix Relationship	The flexibility matrix is the inverse of the stiffness matrix: $[f] = [k]^{-1}$.	The stiffness matrix is the inverse of the flexibility matrix: $[k] = [f]^{-1}$.
Computer Application	Difficult to automate because the choice of redundants is not unique.	Highly suited for computer programming and is the basis of Finite Element Analysis (*FEA*).
Efficiency Choice	Best used when $D_s < D_k$.	Best used when $D_k < D_s$.

5. (a) (ii) Solution:

Feature	Framed Structures	Truss Structures
Member Connections	Members are connected by rigid joints (welded or bolted to resist rotation).	Members are connected by pinned or hinged joints (assumed frictionless).
Force Transfer	Members transfer loads through bending moments, shear forces, and axial forces.	Members transfer loads exclusively through axial forces (tension or compression).
Load Application	Loads can be applied anywhere along the length of the members.	Loads must be applied only at the joints (nodes).
Deformation	Members undergo bending and curvature under external loading.	Members undergo only linear elongation or shortening.
Internal Moments	Support significant internal bending moments at joints and mid-spans.	Ideal trusses have zero bending moment throughout the members.
Examples	Multi-story buildings, rigid portals, and continuous beams.	Roof trusses, bridge trusses, and transmission towers.

5. (b) Solution:

The redundant force is the spring reaction R.



Compatibility condition:

Downward deflection of the beam at midpoint (δ_B) equals compression of the spring (Δ_s).

Deflection of beam at midpoint.

Deflection at B due to load P at distance $x = L$

$$\delta_P = \frac{PL^2}{3EI} + \frac{(PL)L^2}{2EI}$$

$$\Rightarrow \quad \delta_P = \frac{5PL^3}{6EI} \ (\downarrow)$$

Upward deflection at B due to redundant reaction R,

$$\delta_R = \frac{RL^3}{3EI} \ (\uparrow)$$

Net downward deflection at midpoint,

$$\delta_B = \delta_p - \delta_R$$

$$\Rightarrow \quad \delta_B = \frac{5PL^3}{6EI} - \frac{RL^3}{3EI} \; (\downarrow)$$

Compression of spring using Hooke's law,

$$\Delta_s = \frac{R}{k}$$

From compatibility,

$$\delta_B = \Delta_s$$

$$\frac{5PL^3}{6EI} - \frac{RL^3}{3EI} = \frac{R}{k}$$

$$\Rightarrow \quad \frac{5PL^3}{6EI} = R\left(\frac{1}{k} + \frac{L^3}{3EI}\right)$$

$$\Rightarrow \quad \frac{5PL^3}{6EI} = R\left(\frac{3EI + kL^3}{3kEI}\right)$$

$$\Rightarrow \quad R = \left(\frac{5PL^3}{6EI}\right)\left(\frac{3kEI}{3EI + kL^3}\right)$$

$$\Rightarrow \quad R = \frac{5PkL^3}{2(3EI + kL^3)}$$

5. (c) Solution:

Net Positive Suction Head (NPSH)

NPSH represents the net head available at the suction orifice of a pump or the inlet of a turbine to prevent the liquid from vaporizing. It is defined as the difference between the total stagnation pressure head at the suction point and the vapor pressure head of the liquid at the operating temperature. Maintaining a sufficient NPSH is vital to ensure that the local pressure within the machine remains above the vapor pressure, thus preventing the formation of cavitation bubbles.

For a centrifugal pump, the NPSH can be expressed as:

$$NPSH = \left(\frac{P_s}{\rho g} + \frac{V_s^2}{2g}\right) - \frac{P_v}{\rho g}$$

Where:

P_s = Static pressure at the suction flange

V_s = Velocity of the fluid at the suction flange

P_v = Vapor pressure of the liquid

ρ = Density of the liquid

g = Acceleration due to gravity

In practical applications, the Available NPSH ($NPSH_A$) provided by the system must always be greater than the Required NPSH ($NPSH_R$) specified by the manufacturer for the machine to operate without cavitation.

Thoma's Cavitation Factor (σ)

Thoma's cavitation factor is a dimensionless number used to characterize the cavitation behavior of hydraulic turbines and pumps. It relates the net positive suction head to the total head under which the machine is operating. It serves as a scale to predict the onset of cavitation for geometrically similar machines operating under different conditions.

The mathematical expression for Thoma's cavitation factor is:

$$\sigma = \frac{NPSH}{H}$$

Where, H represents the total net head for a turbine or the manometric head for a pump.

For a reaction turbine, where the runner is placed at a vertical height H above the tailrace level, the factor is often written as:

$$\sigma = \frac{H_{atm} - H_s - H_v}{H}$$

Where:

H_{atm} = Atmospheric pressure head

H_s = Suction head (height of runner above tailrace)

H_v = Vapor pressure head

Critical Cavitation Factor (σ_c)

Every hydraulic machine has a critical cavitation factor (σ_c). If the operating cavitation factor σ falls below this critical value, cavitation will commence, leading to a drop in efficiency and potential mechanical damage. Therefore, for safe operation, the condition $\sigma > \sigma_c$ must always be satisfied.

5. (d) Solution:

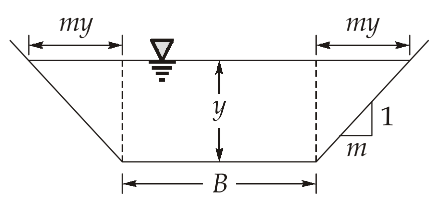
Given data

Bottom width, $b = 3$ m

Side slope, $m = 1.5$

Manning's coefficient, $n = 0.015$

Critical depth, $y_c = 1.2$ m



(i) Critical Flow Rate (Q_c):

At critical flow conditions, the following relationship must be satisfied:

$$\frac{Q^2T}{gA^3} = 1$$

Area of flow (A):

$$A = (b + my_c) \times y_c$$

$$\Rightarrow \quad A = (3 + 1.5 \times 1.2) \times 1.2$$

$$\Rightarrow \quad A = 5.76 \text{ m}^2$$

Top width (T):

$$T = b + 2my_c$$

$$\Rightarrow \quad T = 3 + 2 \times 1.5 \times 1.2$$

$$\Rightarrow \quad T = 6.6 \text{ m}$$

Now, Critical Discharge (Q_c),

$$\Rightarrow \quad \frac{Q_c^2 \times 6.6}{9.81 \times (5.76)^3} = 1$$

$$Q = 16.854 \text{ m}^3/\text{s}$$

(ii) Critical Slope (S_c)

The critical slope is determined using Manning's equation:

$$Q = \frac{1}{n} AR^{2/3}S^{1/2}$$

Wetted Perimeter (P)

$$P = b + 2y_c\sqrt{1+m^2}$$

$\Rightarrow$ $$P = 3 + 2 \times 1.2\sqrt{1+1.5^2}$$

$\Rightarrow$ $$P = 7.327 \text{ m}$$

Hydraulic Radius (R)

$$R = \frac{A}{P}$$

$\Rightarrow$ $$R = \frac{5.76}{7.327}$$

$\Rightarrow$ $$R = 0.786 \text{ m}$$

Now, Critical Slope (S):

$\Rightarrow$ $$16.854 = \frac{1}{0.015} \times 5.76 \times (0.786)^{2/3} \times S^{1/2}$$

$\Rightarrow$ $$S = 0.00266$$

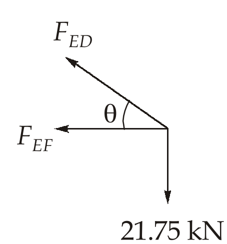
$\Rightarrow$ $$S \approx \frac{1}{375.94} = (1 \text{ in } 375.94)$$

5. **(e) Solution:**

Member forces due to applied load (*F* forces system):

The internal forces in the members due to the external load $P = 21.75$ kN are determined using the method of joints.

FBD of joint *E*:



Here, $\tan\theta = \frac{3}{12} = \frac{1}{4}$, $\quad \sin\theta = \frac{1}{\sqrt{17}}$, $\quad \cos\theta = \frac{4}{\sqrt{17}}$

$\Sigma F_y = 0 \quad \Rightarrow \quad F_{ED}\sin\theta = 21.75$

$$F_{ED} = 89.68 \text{ kN (tensile)}$$

$\Sigma F_x = 0 \quad \Rightarrow \quad F_{ED}\cos\theta + E_{EF} = 0$

$$F_{EF} = -87 \text{ kN} = 87 \text{ kN (Compressive)}$$

$$F_{ED} = F_{DC} = F_{CB} = 89.68 \text{ kN (Tension)}$$

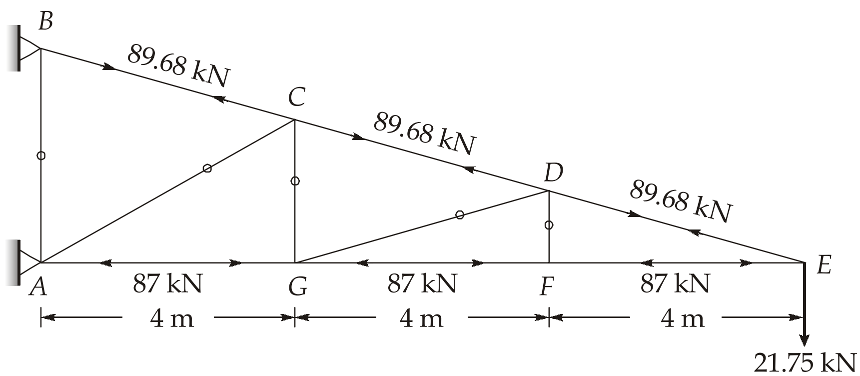
The horizontal components are balanced by the bottom chord members.

$$F_{EF} = F_{FG} = F_{GA} = -87 \text{ kN (Compression)}$$

From truss analysis, the following are zero-force members:

$$F_{AB} = F_{GC} = F_{FD} = F_{AC} = F_{DG} = 0 \text{ kN}$$

The final member forces are shown below:



Member forces due to unit vertical load at E (k forces system)

A unit load of 1 kN is applied vertically downward at joint E. The corresponding internal forces will get divided by 21.75 and are shown in table.

Vertical deflection at joint E (δ_E)

Using the unit load method, $\delta_E = \sum\left(\frac{FlL}{AE}\right)$

Member	F (kN)	k (kN/kN)	L (m)	FkL (kN-m)
ED	89.68	4.123	4.123	1524.48
DC	89.68	4.123	4.123	1524.48
CB	89.68	4.123	4.123	1524.48
EF	–87	–4	4	1392
FG	–87	–4	4	1392
GA	–87	–4	4	1392
Total				**8749.44**

$$\delta_E = \frac{8749.44}{1890000}$$

$$\delta_E = 4.629 \times 10^{-3} \text{ m} = 4.629 \text{ mm } (\downarrow)$$

6. (a) Solution:

Given data

Width of channel, $B = 1.5$ m

Pre-jump depth, $y_1 = 10$ cm $= 0.1$ m

post-jump depth, $y_2 = 45$ cm $= 0.45$ m

Friction force, $F_f = 45$ N

Case 1: Frictionless channel

In a horizontal, frictionless rectangular channel, the momentum per unit weight remains constant across the jump. The discharge per unit width is given by

$$\frac{q^2}{g} = \frac{y_1 y_2 (y_1 + y_2)}{2}$$

$$\Rightarrow \quad q = \sqrt{\frac{g y_1 y_2 (y_1 + y_2)}{2}}$$

$$\Rightarrow \quad q = \sqrt{\frac{9.81 \times 0.1 \times 0.45 \times (0.1 + 0.45)}{2}}$$

$$\Rightarrow \quad q = 0.348 \text{ m}^2/\text{s}$$

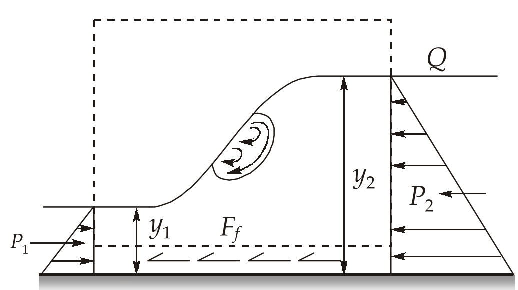
Total discharge,

$$Q = q \times B$$

$$Q = 0.348 \times 1.5$$

$$Q = 0.523 \text{ m}^3/\text{s}$$

Case 2: Considering friction



Including friction, the momentum equation becomes..

$$P_1 - P_2 - F_f = \dot{M}_2 - \dot{M}_1$$

$$\frac{1}{2}\gamma B y_1^2 - \frac{1}{2}\gamma B y_2^2 - F_f = \rho Q (V_2 - V_1)$$

Where, $\gamma = \rho g$ and $V = \frac{Q}{By}$. Substituting velocity terms,

$$\Rightarrow \quad \frac{1}{2}\rho g B\left(y_1^2 - y_2^2\right) - F_f = \rho Q\left(\frac{Q}{By_2} - \frac{Q}{By_1}\right)$$

$$\Rightarrow \quad \frac{1}{2}\rho g B\left(y_1^2 - y_2^2\right) - F_f = \frac{\rho Q^2}{B}\left(\frac{y_1 - y_2}{y_1 y_2}\right)$$

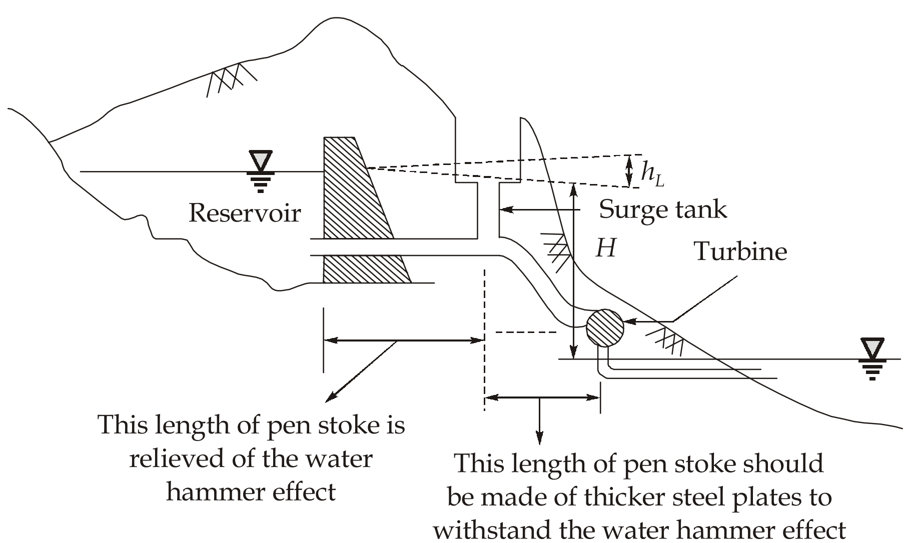
Substituting the given values,

$$\Rightarrow \quad \frac{1}{2}\times 1000\times 9.81\times 1.5\times\left(0.1^2 - 0.45^2\right) - 45 = \frac{1000Q^2}{1.5}\left(\frac{0.1-0.45}{0.1\times 0.45}\right)$$

$$\Rightarrow \quad Q = 0.531 \text{ m}^3/\text{s}$$

6. **(b) (i) Solution:**

A surge tank is a vertical chamber provided on conduits near the upstream end of the penstock in order to counter the water hammer phenomenon.



Advantages of Surge Tanks

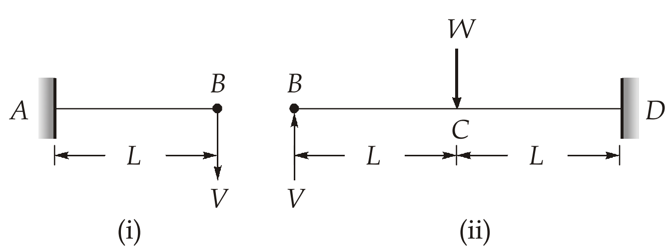
1. **Reduction of Water Hammer Pressure:** When the turbine gates are closed rapidly due to a decrease in load, the kinetic energy of the moving water is converted into high-pressure waves known as water hammer. The surge tank provides an additional surface area that allows the water to rise, effectively absorbing these pressure transients and preventing penstock rupture.
2. **Improved Governing and Stability:** By acting as a temporary storage or supply source, the surge tank helps the turbine governor respond more effectively to load

changes. It shortens the distance that water must travel from a free surface to the turbine, which reduces the time lag in regulating the flow.

3. **Regulation of Flow:** The surge tank serves as a buffer. During a sudden increase in load, the tank provides the extra water required immediately by the turbine before the water in the long headrace tunnel can accelerate to the new required velocity.
4. **Economical Penstock Design:** Because the surge tank absorbs the highest pressure surges, the downstream penstock can be designed for lower maximum pressures. This allows for thinner pipe walls, significantly reducing the material cost of the penstock.

6. (b) (ii) Solution:

The structure is statically indeterminate to one degree. The vertical reaction at hinge *B*, denoted by *V*, is taken as the redundant. The compatibility condition is that the deflection of point *B* from segment *AB* must be equal to the deflection of point *B* from segment *BD*.



Deflection of *B* in part *AB*.

Segment *AB* acts as a cantilever fixed at *A* with downward force *V* at free end *B*.

$$\delta_{AB} = \frac{VL^3}{3EI}(\downarrow)$$

Deflection of *B* in part *BD*.

Segment *BD* is fixed at *D* and subjected to load *W* at mid-point *C* and upward reaction *V* at *B*.

Deflection at *B* due to load *W*,

$$\delta_{BW} = \frac{WL^3}{3EI} + \frac{WL^2}{2EI} \times L = \frac{5WL^3}{6EI}(\downarrow)$$

Upward deflection at *B* due to redundant *V*,

$$\delta_{BV} = \frac{V(2L)^3}{3EI} = \frac{8VL^3}{3EI}(\uparrow)$$

Net deflection at *B* from segment *BD*,

$$\delta_{BD} = \frac{5WL^3}{6EI} - \frac{8VL^3}{3EI}(\downarrow)$$

From compatibility,

$$\delta_{AB} = \delta_{BD}$$

$$\Rightarrow \quad \frac{VL^3}{3EI} = \frac{5WL^3}{6EI} - \frac{8VL^3}{3EI}$$

$$\Rightarrow \quad 2V = 5W - 16V$$

$$\Rightarrow \quad 18V = 5W$$

$$\Rightarrow \quad V = \frac{5W}{18}$$

Substituting $W = 10$ kN, $V = \dfrac{5 \times 10}{18} = 2.778$ kN

Reactions at support A, $V_A = V = 2.778$ kN $(\uparrow)$

$$M_A = V \times L = 2.778 \times 4 = 11.112 \text{ kNm} (\circlearrowleft)$$

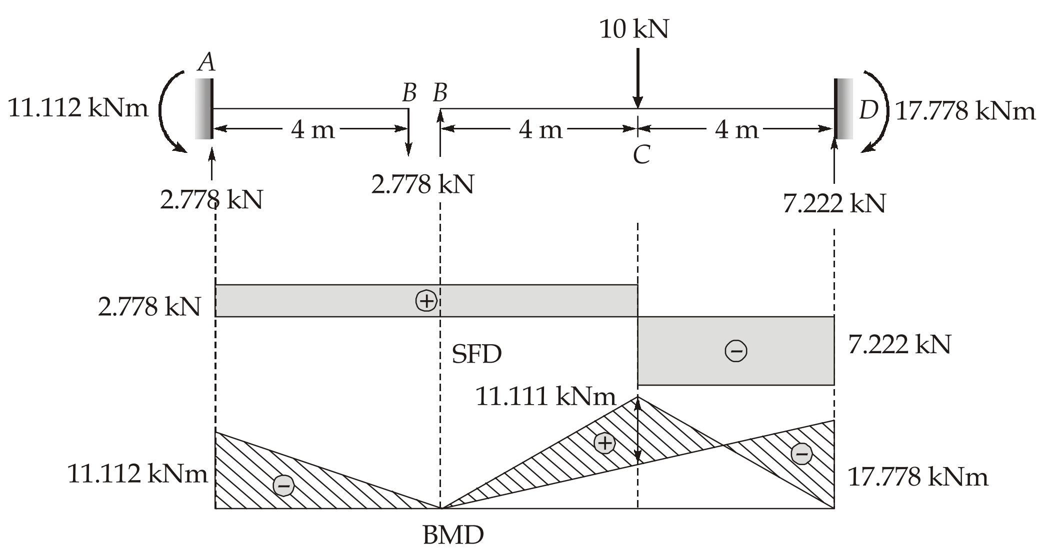
Reactions at support D.

$$V_D = W - V = 10 - 2.778 = 7.222 \text{ kN} (\uparrow)$$

$$M_D = W \times 4 - V \times 8 = 17.778 \text{ kNm} (\circlearrowright)$$

Moment at point C.

$$M_C = V \times L = 2.778 \times 4 = 11.111 \text{ kNm}$$



6. (c) Solution:

Given data

Fabrication error in AB (settlement of B relative to C), Δ = 10 mm = 0.01 m

Flexural rigidity, $EI = 100{,}000 \text{ kNm}^2$

Fixed End Moments

There are no external transverse loads on the members. The over-length member AB causes a vertical displacement Δ at joint B, which acts as a settlement for member BC.

$$M_{FAB} = M_{FBA} = 0$$

$$M_{FBC} = M_{FCB} = \frac{6EI\Delta}{L^2}$$

$$M_{FBC} = \frac{6\times100000\times0.01}{8^2} = 93.75 \text{ kN – m}$$

$$M_{FCB} = 93.75 \text{ kN – m}$$

Slope Deflection Equations

Using the standard slope deflection formula

$$M_{ij} = M_{Fij} + \frac{2EI}{L}\left(2\theta_i + \theta_j - \frac{3\Delta}{L}\right)$$

$$M_{AB} = \frac{2\times0.5\times100000}{4}\left(2\theta_A + \theta_B\right) = 50{,}000\theta_A + 25{,}000\theta_B$$

$$M_{BA} = \frac{2\times0.5\times100000}{4}\left(2\theta_B + \theta_A\right) = 25{,}000\theta_A + 50{,}000\theta_B$$

$$M_{BC} = 93.75 + \frac{2\times100000}{8}(2\theta_B + 0) = 50{,}000\theta_B + 93.75$$

$$M_{CB} = 93.75 + \frac{2\times100000}{8}(\theta_B + 0) = 25{,}000\theta_B + 93.75$$

Joint Equilibrium Equations

At Joint A (Pinned support):

$$M_{AB} = 0$$

$$\Rightarrow \quad 50{,}000\,\theta_A + 25{,}000\,\theta_B = 0$$

$$\Rightarrow \quad \theta_A = -0.5\,\theta_B$$

At Joint B:

$$M_{BA} + M_{BC} = 0$$

$$\Rightarrow \quad (25{,}000\theta_A + 50{,}000\theta_B) + (50{,}000\theta_B + 93.75) = 0$$

$$25{,}000\,(-0.5\theta_B) + 100{,}000\,\theta_B = -93.75 \qquad \because \quad \theta_A = -0.5\theta_B$$

$$87{,}500\,\theta_B = -93.75$$

$$\theta_B = -0.0010714 \text{ radian}$$

$$\theta_A = 0.0005357 \text{ radian}$$

Final Moments and Reactions

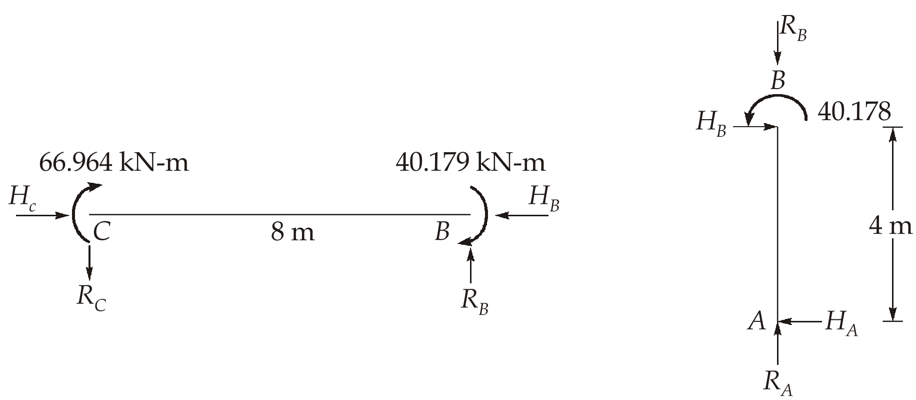
Substituting rotations into the slope deflection equations:

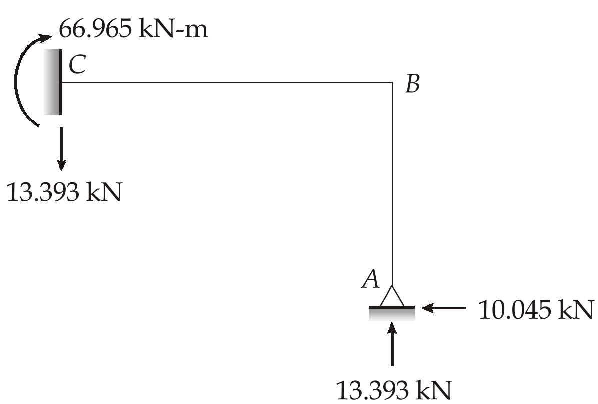
$$M_{AB} = 0$$

$$M_{BA} = -40.178 \text{ kN – m}$$

$$M_{BC} = 40.178 \text{ kN – m}$$

$$M_{CB} = 66.965 \text{ kN – m}$$

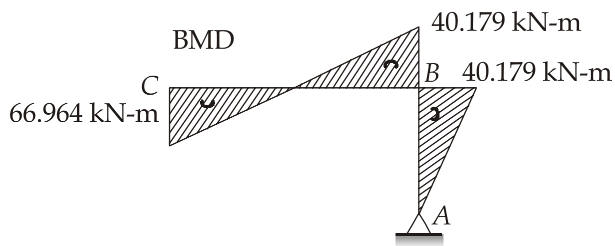




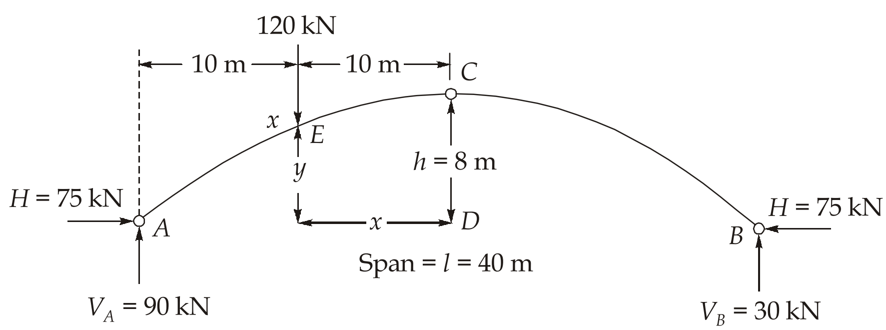
$$\text{Horizontal reaction at } A,\ H_A = \frac{M_{BA}}{4} = 10.045 \text{ kN } (\leftarrow)$$

$$\text{Vertical reaction at } B,\ R_B = \frac{M_{BC} + M_{CB}}{8} = 13.393 \text{ kN } (\uparrow)$$

Vertical reaction at C, R_C = 13.393 kN (↓)



7. (a) Solution:



Taking moments about A,

$$V_b \times 40 = 120 \times 10$$

$$\therefore \quad V_b = 30 \text{ kN} \uparrow$$

$$\therefore \quad V_a = 120 - 30 = 90 \text{ kN} \uparrow$$

Taking moments about C of the forces on the right side of C,

$$H \times 8 = 30 \times 20$$

$$H = 75 \text{ kN}$$

Let R be the radius of the arch

$$\therefore \quad 8(2R - 8) = 20 \times 20$$

$$\therefore \quad R = 29 \text{ m}$$

Let D be the middle point of AB.

The equation to the circular arch with D as origin is, $y = \sqrt{R^2 - x^2} - \sqrt{R^2 - \left(\frac{L}{2}\right)^2}$

$$y = \sqrt{29^2 - x^2} - \sqrt{29^2 - 20^2}$$

$$y = \sqrt{841 - x^2} - 21$$

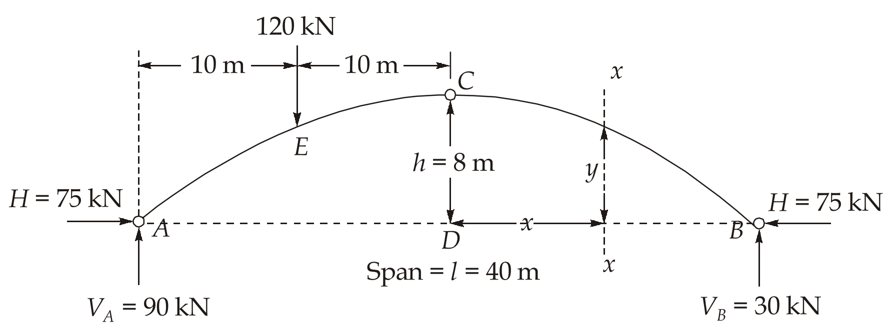
Maximum positive bending moment

The maximum positive bending moment occurs under the load, i.e.,

At E, $$y_E = \sqrt{841 - 10^2} - 21 = 6.221 \text{ m}$$

$$M_{max(+)} = 90 \times 10 - 75 \times 6.221 = +433.425 \text{ kNm}$$

For maximum negative bending moment



Taking moment about x-x section (Right side)

$$M_{xx} = 30 \times (20 - x) - 75y$$

$$\Rightarrow \quad M_{xx} = 30(20 - x) - 75\left(\sqrt{841 - x^2} - 21\right)$$

For maximum bending moment

$$\frac{dM_{xx}}{dx} = 0$$

$$\Rightarrow \quad -30 - 75 \times \frac{(-2x)}{2\sqrt{841 - x^2}} = 0$$

$$\Rightarrow \quad 2\sqrt{841 - x^2} = 5x$$

Squaring both sides

$$4(840 - x^2) = 25x^2$$

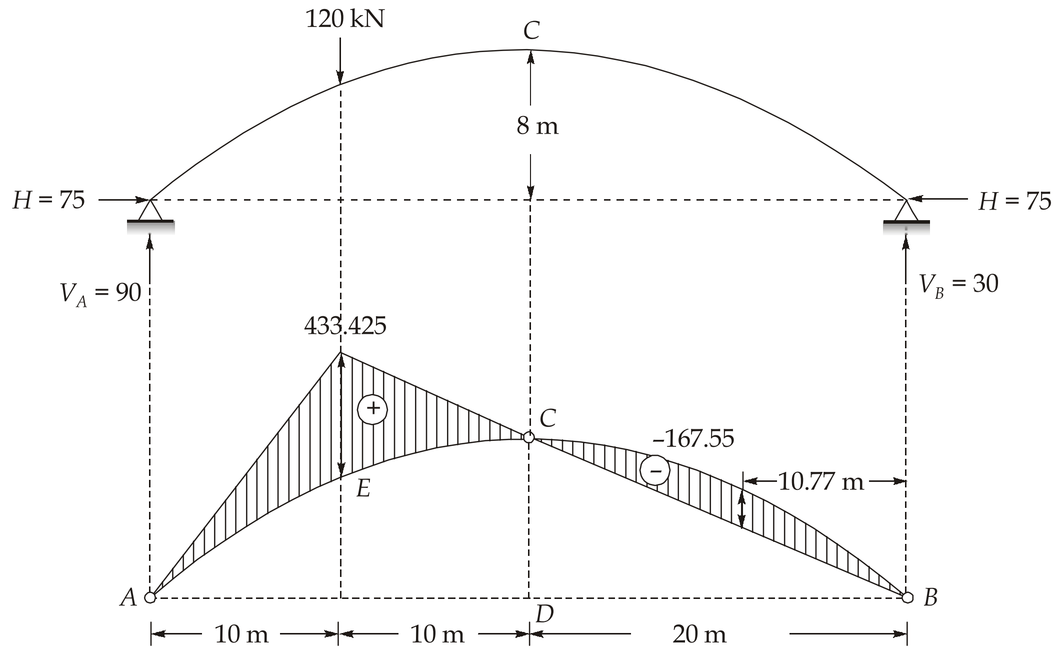
$$4 \times 841 = 29x^2$$

$$x = 10.77 \text{ m}$$

Maximum negative bending moment,

$$M_{\max\,(-ve)} = 30(20 - 10.77) - 75\left(\sqrt{841 - 10.77^2} - 21\right)$$

$$= -167.55 \text{ kNm}$$



BMD
(Values in kN-m)

7. (b) (i) Solution:

Work Breakdown Structure (WBS) is a systematic and hierarchical decomposition of a project into smaller, manageable components. It breaks the total project work into progressively smaller elements called work packages. Each lower level represents a more detailed definition of the project work. The purpose of WBS is to organize and define the total scope of the project so that planning, scheduling, budgeting, and control can be carried out effectively.

In a Work Breakdown Structure, the entire project is placed at the top level. This is then divided into major deliverables or phases. Each deliverable is further subdivided into smaller tasks until manageable work packages are obtained. A work package is the smallest unit in WBS and can be scheduled, cost-estimated, monitored, and assigned to a specific individual or team.

WBS follows the 100 percent rule, which means that it includes 100 percent of the total project work and excludes any work that is not part of the project scope. It is usually represented in a tree structure or tabular format.

Merits of Work Breakdown Structure

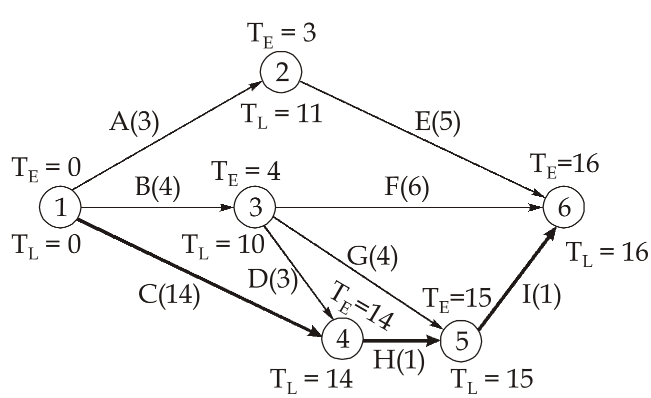
1. WBS clearly defines the scope of the project and avoids ambiguity.
2. It helps in systematic planning and scheduling of project activities.
3. It improves cost estimation and budgeting accuracy.
4. It facilitates assignment of responsibilities to individuals or departments.
5. It helps in effective monitoring and control of project progress.
6. It improves coordination and communication among project participants.
7. It reduces the chances of overlooking important activities.

Demerits of Work Breakdown Structure

1. Preparation of WBS may be time-consuming for large and complex projects.
2. It requires experienced personnel for proper structuring.
3. If not prepared carefully, some activities may be omitted or duplicated.
4. Frequent project changes require continuous updating of the WBS.
5. It focuses on deliverables and may not clearly show activity sequencing unless combined with network techniques.

7. (b) (ii) Solution:

Network diagram for the given project is



Values in parentheses shows the duration of the activities.

(1)→(4)→(5)→(6) or C – H – I is the critical path.

Based on the forward and backward pass calculations, the detailed schedule is presented below. Accuracy is maintained as per the standard integer values provided in the data.

Activity	Duration	EST	EFT	LST	LFT	TS	HS	TF	FF	IF
A	3	0	3	8	11	0	8	8	0	0
B	4	0	4	6	10	0	6	6	0	0
C	14	0	14	0	14	0	0	0	0	0
D	3	4	7	11	14	6	0	7	7	1
E	5	3	8	11	16	8	0	8	8	0
F	6	4	10	10	16	6	0	6	6	0
G	4	4	8	11	15	6	0	7	7	1
H	1	14	15	14	15	0	0	0	0	0
I	1	15	16	15	16	0	0	0	0	0

Here, TS = Tail slack, HS = Head slack, TF = Total float, FF = Free float,
IF = Independent float

FF = TF – HS

IF = FF – TS

7. (c) Solution:

Joint	Member	Member Stiffness	Joint Stiffness	Distribution factor
B	BA	$\frac{4(2EI)}{4}$	$3EI$	$\frac{2}{3}$
	BC	$\frac{3EI}{3}$		$\frac{1}{3}$

Fixed end moment

$$\bar{M}_{AB} = -\frac{16\times4}{8} = -8 \text{ kNm}$$

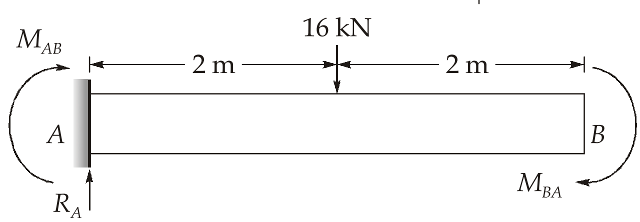
$$\bar{M}_{BA} = \frac{16\times4}{8} = +8 \text{ kNm}$$

$$\bar{M}_{BC} = -\frac{wl^2}{12} = -\frac{12\times3^2}{12} = -9 \text{ kNm}$$

$$\bar{M}_{CB} = +9 \text{ kNm}$$

Non-sway analysis

	A	B	B	C
DF		2/3	1/3	
FEM (kNm)	–8	+8	–9	+9
				–9
			–4.5	
	–8	+8	–13.5	0
		+3.67	+1.83	
	+1.835			0
Final moment (kNm)	–6.17	11.67	–11.67	0

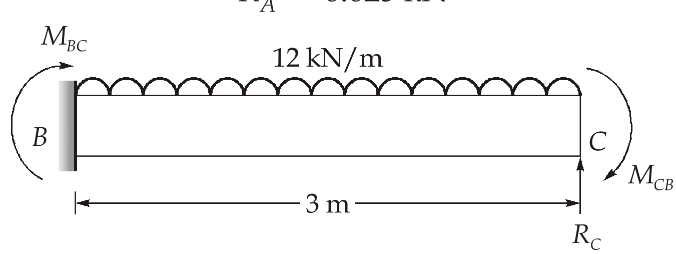


$$\Sigma M_B = 0$$

$$\Rightarrow \quad R_A \times 4 + M_{BA} + M_{AB} - 16 \times 2 = 0$$

$$\Rightarrow \quad R_A = \frac{-[M_{AB} + M_{BA} - 32]}{4}$$

$$\Rightarrow \quad R_A = 6.625 \text{ kN}$$



$$\Sigma M_B = 0$$

$$\Rightarrow \quad -R_C \times 3 + M_{CB} + M_{BC} + 12 \times 3 \times 1.5 = 0$$

$$\Rightarrow \quad R_C = \left(\frac{M_{BC} + 54}{3}\right) = 14.11 \text{ kN}$$

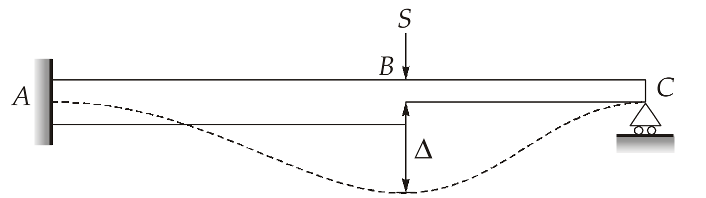
$$\Sigma F_y = R_A + R_C - 16 - 12 \times 3$$

$$= 6.625 + 14.11 - 16 - 12 \times 3$$

$$= -31.265 \text{ kN}$$

$\therefore$ Sway force, $S = 31.265 \text{ kN } (\downarrow)$

Sway analysis



Due to sway of Δ, the fixed end moments are,

$$\bar{M}_{AB} = \bar{M}_{BA} = -\frac{6E(2I)\Delta}{4^2} = -\frac{6EI\Delta}{8} = -\frac{3}{4}EI\Delta$$

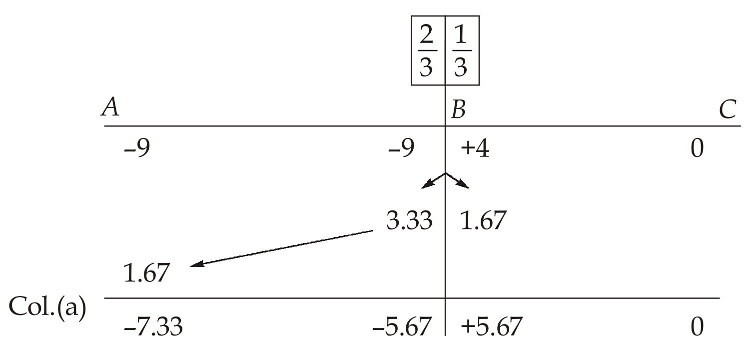
$$\bar{M}_{BC} = \frac{3EI\Delta}{3^2} = \frac{EI\Delta}{3}$$

$$\bar{M}_{CB} = 0$$

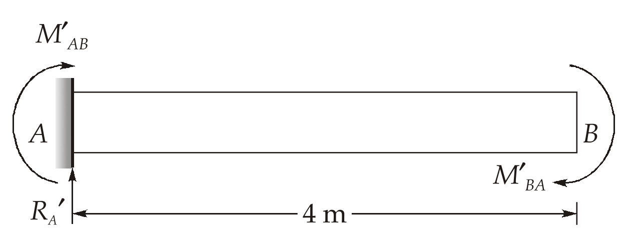
Ratio of fixed end moments,

$$\bar{M}_{AB} : \bar{M}_{BA} : \bar{M}_{BC} : \bar{M}_{CB} = -\frac{3}{4} : -\frac{3}{4} : \frac{1}{3} : 0$$

$$= -9 : -9 : +4 : 0$$



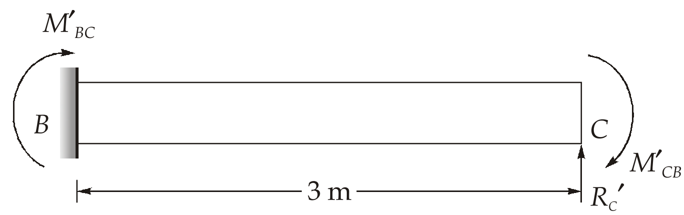
For S' $$R_A' + R_C' = S'$$



$$\Sigma M_B = 0$$

$$\Rightarrow \quad R_A' \times 4 + M'_{AB} + M'_{AB} = 0$$

$\Rightarrow$
$$R_A' = \left[\frac{M'_{AB} + M'_{BA}}{4}\right] = 3.25 \text{ kN}$$



$$\Sigma M_B = 0$$

$$R_C' = \frac{M'_{BC}}{3} = \frac{5.67}{3} = 1.89 \text{ kN}$$

$$S' = R_A' + R_C'$$
$$= 3.25 + 1.89$$
$$= 5.14 \text{ kN}$$

$\therefore$
$$\frac{S}{S'} = \frac{31.265}{5.14} = 6.083$$

Moments:

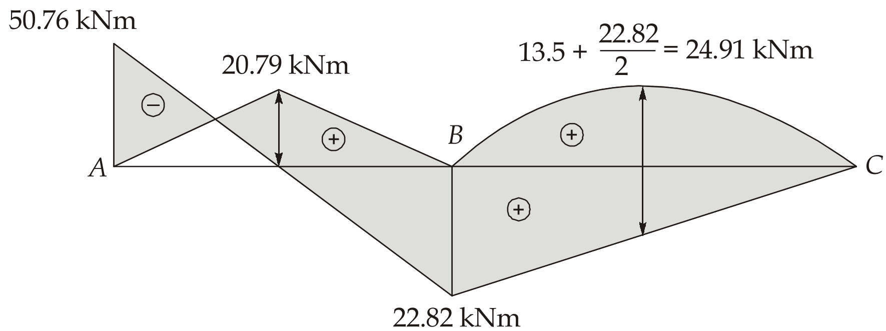
	AB	*BA*	*BC*	*CB*
Non-sway moments	–6.17	11.67	–11.67	0
Col.(a) $\times \left[\frac{S}{S'}\right]$	–44.59	–34.49	+34.49	0
Final moments	–50.79	–22.82	+22.82	0

Free Moments:

$$\text{Span } AB = \frac{WL}{4} = \frac{16 \times 4}{4} = 16 \text{ kNm}$$

$$\text{Span } BC = \frac{wL^2}{8} = \frac{12 \times 3^2}{8} = 13.5 \text{ kNm}$$

BMD



8. (a) Solution:

Fixed End Moments (FEMs):

$$M_{FAB} = -\frac{wL^2}{12} = -\frac{10 \times 8^2}{12} = -53.333 \text{ kNm}$$

$$M_{FBA} = -\frac{wL^2}{12} = -\frac{10 \times 8^2}{12} = 53.333 \text{ kNm}$$

$$M_{FBC} = -\frac{wL}{8} = -\frac{30 \times 4}{8} = -15 \text{ kNm}$$

$$M_{FCB} = \frac{wL}{8} = \frac{30 \times 4}{8} = 15 \text{ kNm}$$

$$M_{CD} = -20 \times 2 = -40 \text{ kNm}$$

Slope Deflection Equations:

Support A is fixed, so $\theta_A = 0$.

$$M_{AB} = M_{FAB} + \frac{2E(2I)}{8}(2\theta_A + \theta_B) = -53.333 + 0.5EI\theta_B$$

$$M_{BA} = M_{FBA} + \frac{2E(2I)}{8}(2\theta_B + \theta_A) = 53.333 + EI\theta_B$$

$$M_{BC} = M_{FBC} + \frac{2EI}{4}(2\theta_B + \theta_C) = -15 + EI\theta_B + 0.5EI\theta_C$$

$$M_{CB} = M_{FCB} + \frac{2EI}{4}(2\theta_C + \theta_B) = 15 + EI\theta_C + 0.5EI\theta_B$$

Joint Equilibrium Equations:

Joint B: $M_{BA} + M_{BC} = 0$

$(53.333 + EI\theta_B) + (-15 + EI\theta_B + 0.5EI\theta_C) = 0$

$$2EI\theta_B + 0.5EI\theta_C = -38.333 \quad \text{...(i)}$$

Joint C: $M_{CB} + M_{CD} = 0$

$$(15 + EI\theta_C + 0.5EI\theta_B) - 40 = 0$$

$$0.5EI\theta_B + EI\theta_C = 25 \qquad \text{...(ii)}$$

On Solving equation (i) and (ii)

$$EI\theta_B = -29.047$$

$$EI\theta_C = 39.524$$

Final Moments:

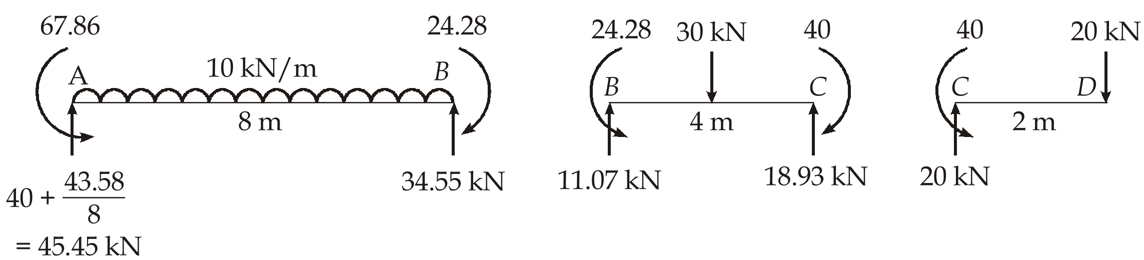
$$M_{AB} = -53.333 + 0.5(-29.047) = -67.86 \text{ kNm}$$

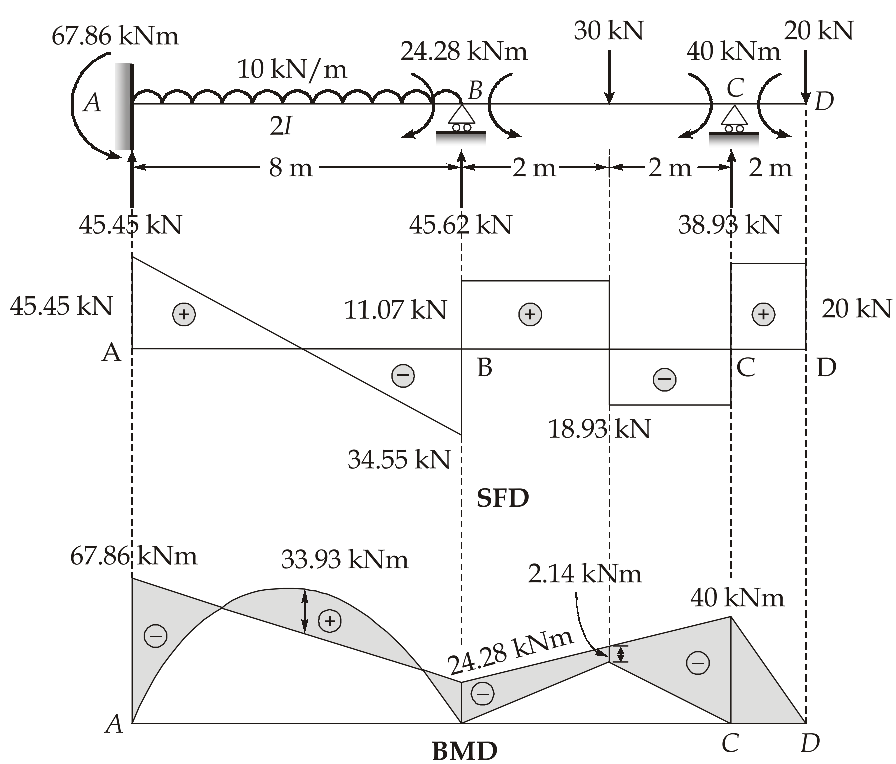
$$M_{BA} = 53.333 + (-29.047) = 24.28 \text{ kNm}$$

$$M_{BC} = -15 + (-29.047) + 0.5(39.524) = -24.28 \text{ kNm}$$

$$M_{CB} = 15 + 39.524 + 0.5(-29.047) = 40 \text{ kNm}$$

Support reactions:





8. (b) (i) Solution:

To classify the slope as mild or steep, the normal depth y_n is compared with the critical depth y_c.

Mild slope: $y_n > y_c$ which implies $S_0 < S_c$

Steep slope: $y_n < y_c$ which implies $S_0 > S_c$

The limiting condition occurs when

$$y_n = y_c$$

At this condition,

$$S_0 = S_c$$

Critical depth for a rectangular channel is obtained from the condition of critical flow,

$$\frac{q^2}{g} = y^3_c$$

$$\Rightarrow \qquad y_c = \left(\frac{q^2}{g}\right)^{1/3}$$

For a wide rectangular channel, hydraulic radius $R \approx y$.

Manning's equation for discharge per unit width is

$$Q = A \times \frac{1}{n} R^{2/3} S_o^{1/2}$$

$$\because \qquad Q = q \times B$$

$$\therefore \qquad q = \frac{1}{n} y_n^{5/3} S_0^{1/2}$$

At the critical condition, substitute $y_n = y_c$ and $S_o = S_c$,

$$q = \frac{1}{n} y_c^{5/3} S_c^{1/2}$$

Substitute the expression for y_c,

$$q = \frac{1}{n}\left[\left(\frac{q^2}{g}\right)^{1/3}\right]^{5/3} S_c^{1/2}$$

$$\Rightarrow \qquad q = \frac{1}{n} \frac{q^{10/9}}{g^{5/9}} S_c^{1/2}$$

$$\Rightarrow \qquad S_c^{1/2} = \frac{nqg^{5/9}}{q^{10/9}}$$

$$\Rightarrow \qquad S_c^{1/2} = \frac{ng^{5/9}}{q^{1/9}}$$

Squaring both sides,

$$\Rightarrow \qquad S_c = \frac{n^2 g^{10/9}}{q^{2/9}}$$

Hence, if $S_o < \dfrac{n^2 g^{10/9}}{q^{2/9}}$

the slope is mild and the flow is subcritical ($y_n > y_c$).

If $S_0 > \dfrac{n^2 g^{10/9}}{q^{2/9}}$

The slope is steep and the flow is supercritical ($y_n < y_c$).

8. **(b) (ii) Solution:**

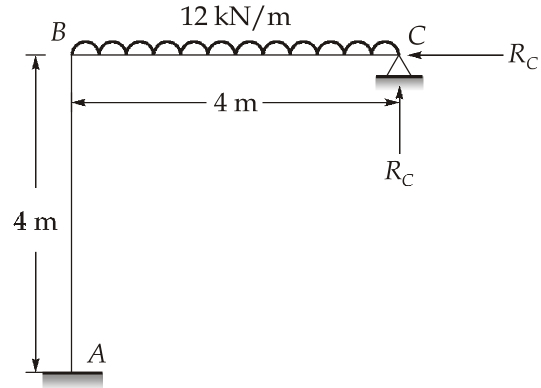
1. **Lump Sum Contract:** A Lump Sum Contract is a type of contract in which the contractor agrees to complete the entire work for a fixed total price. The scope, specifications, and drawings are clearly defined before execution. The contractor bears the risk of quantity variations and cost overruns. Payments are generally made based on milestones or stages of completion. This type of contract is suitable when the project scope is well defined. It provides cost certainty to the employer. However, changes in scope may lead to claims or disputes. It encourages efficient planning and cost control by the contractor. Quality must strictly follow contract specifications. It is widely used in building construction projects.
2. **Item Rate Contract:** In an Item Rate Contract, rates are quoted for individual items of work listed in the Bill of Quantities (BOQ). The total contract value depends on actual quantities executed. It is suitable where exact quantities cannot be determined in advance. Payments are made based on measured quantities at agreed rates. The risk of quantity variation is borne mainly by the employer. It provides flexibility during execution. Detailed measurement and record keeping are essential. It is commonly used in government works. Disputes may arise due to variation in quantities. It ensures transparency in rate comparison.
3. **Percentage Rate Contract:** A Percentage Rate Contract involves quoting a percentage above or below the estimated cost prepared by the department. The estimated schedule of rates forms the basis of payment. The contractor agrees to execute work

at the quoted percentage variation. It simplifies the tendering process. It is widely used in public works departments. Payments are made according to actual quantities executed. It reduces the effort required in preparing item-wise rates. However, accuracy of departmental estimates is crucial. It provides quick comparison of bids. It is suitable for routine and repetitive works.

4. **Cost Plus Contract:** In a Cost-Plus Contract, the contractor is paid the actual cost of work plus an agreed profit margin. The profit may be a fixed percentage or fixed fee. It is suitable when scope is uncertain or urgent work is required. The employer bears most of the financial risk. Proper documentation of expenses is essential. It ensures quality since cost cutting is not the primary concern. However, it may reduce incentive for cost control. Variations are easier to accommodate. It is often used in research or emergency projects. Transparency and auditing are very important.
5. **Turnkey Contract:** A Turnkey Contract requires the contractor to complete the entire project and hand it over in ready-to-use condition. The contractor is responsible for design, execution, and commissioning. The employer's involvement during execution is minimal. It ensures single-point responsibility. Time and cost are generally fixed. It reduces coordination issues between different agencies. The contractor bears significant risk. It is commonly used in industrial and infrastructure projects. Quality and performance standards must be clearly defined. The project is delivered as a complete functional facility.
6. **EPC (Engineering, Procurement and Construction) Contract:** An EPC Contract is a comprehensive contract covering engineering, procurement, and construction. The contractor is responsible for design, material supply, construction, and commissioning. It provides single-point accountability. The project is usually executed on a fixed price and time basis. Risks related to cost and delay are largely transferred to the contractor. It is widely used in power plants, highways, and large infrastructure projects. Strict performance guarantees are included. It ensures integrated project delivery. The employer monitors progress but has limited design involvement. It promotes efficiency and timely completion.

8. (c) Solution:

Given data



Height of member $AB = 4$ m

Length of member $BC = 4$ m

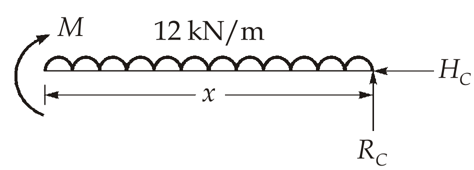
Uniformly distributed load, $w = 12$ kN/m

Flexural rigidity, EI = constant

Moment equations for each segment are written to apply the conditions

$$\frac{\partial U}{\partial H_C} = 0 \text{ and } \frac{\partial U}{\partial R_C} = 0$$

For member CB, taking origin at C and x measured toward B,



$$M = R_C x - \frac{12x^2}{2}$$

$$\frac{\partial M}{\partial H_C} = 0; \frac{\partial M}{\partial R_C} = x$$

For member BA, taking origin at B and y measured toward A,

$$M = M_B + H_C y$$

$$\frac{\partial M}{\partial H_C} = y; \frac{\partial M}{\partial R_C} = 4$$

Moment at joint B,

$$M_B = R_C \times 4 - \frac{12 \times 4^2}{2}$$

$$M_B = 4\,R_C - 96$$

Now, $\quad M = (4\,R_C - 96) + H_c y$

Applying Castigliano's theorem

For $\dfrac{\partial U}{\partial H_C} = 0,$

$$\frac{\partial}{\partial H_C}\int_0^4 \frac{\left[(4R_C - 96) + H_C y\right]^2 dy}{2EI} = 0$$

$$\Rightarrow \quad \int_0^4 \frac{\left(H_C y - (96 - 4R_C)\right)y}{EI}dy = 0$$

$$\Rightarrow \quad \frac{64}{3}H_C - \frac{96 \times 4^2}{2} + \frac{4R_C \times 4^2}{2} = 0$$

$$\Rightarrow \quad \frac{64}{3}H_C - 768 + 32\,R_C = 0$$

$$\Rightarrow \quad 64\,H_C + 96\,R_C - 2304 = 0 \qquad \text{...(i)}$$

For $\dfrac{\partial U}{\partial R_C} = 0,$

$$\int_0^4 \frac{\left(R_C x - 6x^2\right)x}{EI}dx + \int_0^4 \frac{\left(H_C y - 96 + 4R_C\right)4}{EI}dy = 0$$

$$\Rightarrow \quad \left[\frac{R_C x^3}{3} - \frac{6x^4}{4}\right]_0^4 + \left[4\left(\frac{H_C y^2}{2} - 96y + 4R_C y\right)\right]_0^4 = 0$$

$$\Rightarrow \quad \left(\frac{64R_C}{3} - 384\right) + (32H_C - 1536 + 64\,R_C) = 0$$

$$\Rightarrow \quad 32\,H_C + \frac{256R_C}{3} - 1920 = 0$$

$$\Rightarrow \quad 96\,H_C + 256\,R_C - 5760 = 0 \qquad \text{...(ii)}$$

On solving equation (i) and (ii)

$$R_C = 20.571 \text{ kN } (\uparrow) \text{ and } H_C = 5.143 \text{ kN } (\leftarrow)$$

Maximum bending moment in span BC occurs where shear force is zero.

$$V = R_C - 12x$$

$$\Rightarrow \quad 20.571 - 12x = 0$$

$$x = 1.714 \text{ m}$$

$$(M_{max})_{BC} = 20.571 \times 1.714 - \frac{12 \times 1.714^2}{2}$$

$$(M_{max})_{BC} = 17.632 \text{ kNm}$$

Moment at joint B, $$M_B = 20.571 \times 4 - \frac{12 \times 4^2}{2}$$

$\Rightarrow$ $$M_B = -13.716 \text{ kNm}$$

Moment at support A, $$M_A = H_C \times 4 - 13.716$$

$\Rightarrow$ $$M_A = 6.856 \text{ kNm}$$