RMF3

3D3 2023 Crib

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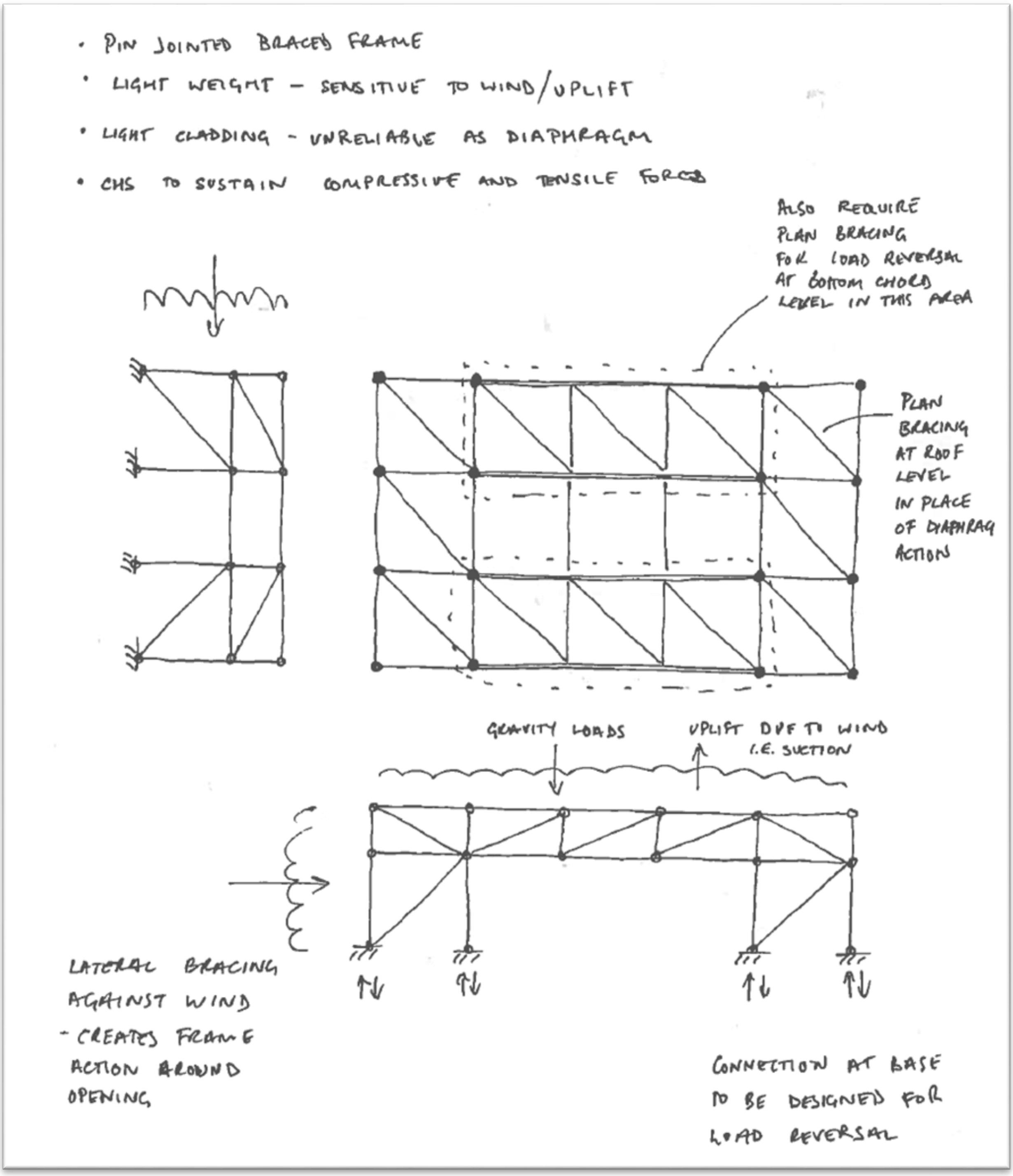
1)



- PIN JOINTED BRACED FRAME
- LIGHT WEIGHT – SENSITIVE TO WIND/UPLIFT
- LIGHT CLADDING – UNRELIABLE AS DIAPHRAGM
- CHS TO SUSTAIN COMPRESSIVE AND TENSILE FORCES

ALSO REQUIRE PLAN BRACING FOR LOAD REVERSAL AT BOTTOM CHORD LEVEL IN THIS AREA

PLAN BRACING AT ROOF LEVEL IN PLACE OF DIAPHRAG ACTIONS

LATERAL BRACING AGAINST WIND - CREATES FRAME ACTION AROUND OPENING

CONNECTION AT BASE TO BE DESIGNED FOR LOAD REVERSAL

Many possible solutions. Only one is shown.

For overall stability it is necessary to provide shear resistance in three orthogonal planes. Sensibly achieved here through diagonal CHS tension/compression bracing. Important to recognise that lightweight cladding unlikely to provide reliable diaphragm/plate action so plan bracing required.

Lightweight structure so wind load may lead to uplift/reversal. Important then to provide plan bracing to restrain compression chord of trusses at bottom chord level in addition to that at top chord level.

Gravity loads carried by cladding spanning to truss members and then nodes, then spanned by truss action to vertical struts to ground. Lateral loads against North/South elevations spanned to braced end walls by bracing and roof members acting as plan truss. Lateral loads against East/West elevations spanned to braced frame around opening. Upwards wind loads (i.e. suction) resisted by gravity load resisting system in reverse. Note the need to design base connections for overall uplift.

b)

i) PVB has negligible shear strength in medium term, design as non-composite.

From data book: $f_{gd} = \frac{k_{mod}k_A f_{gk}}{\gamma_{mA}} + \frac{f_{rk}}{\gamma_{mV}} = \frac{0.43\times1\times45}{1.8} + \frac{90}{1.2} = 85.75\ MPa$

$$\sigma = \frac{M}{z}$$

$$z = \frac{bd^2}{6}$$

Hence, for non-composite:

$$\frac{d}{2} = \sqrt{\frac{6\frac{M}{2}}{b\sigma}}$$

ULS design load: $w_{ED} = 3.5\times1.5 + 0.5\times1.35 = 5.925\ kN\ m^{-2}$

Simply supported so: $M_{Ed} = \frac{5.925\times1.5^2}{8} = 1.666\ kNm$

Hence: $d = 2\sqrt{\frac{6\times\frac{1.666\times10^6}{2}}{1000\times85,75}} = 15.3\ mm$

So 2 layers of 8 mm glass.

c)

Show checks for bearing and buckling.

$$f_d = \frac{f_k}{\gamma_m} = \frac{2}{3} = 0.67\ MPa$$

Bearing

Simply supported both sides so consider half thickness of wall and triangular distribution of load.

$$\sigma_{average} = \frac{\sigma_{max}}{2} = \frac{0.67}{2} = 0.33\ MPa$$

$V = 5.925 \times \frac{1.5}{2} = 4.44\ kN$

$\frac{4.44 \times 10^3}{40 \times 1000} = 0.11\ MPa$

$$0.11 < 0.33\ so\ ok$$

<u>Buckling</u>

$\frac{h}{t} = \frac{1200}{80} = 15$

Assuming uniform load distribution, i.e. equally loaded spans: $e\ =\ 0.05t$

From design chart, beta = 0.85

$$\beta f_d bt = 0.85 \times 0.67 \times 1000 \times 80 = 44.8\ kN$$

$P = 2V = 8.88\ kN$

$8.88 < 44.8\ so\ ok\ in\ buckling$

Assuming permanent and transient load on span 1 and permanent load only on span 2:

$V_1 = 5.925 \times \frac{1.5}{2} = 4.44\ kN$ which for triangular stress distribution occurs at $e = \frac{2}{3}\frac{t}{2}$

$V_2 = 0.675 \times \frac{1.5}{2} = 0.51\ kN$ which also occurs at $e = -\frac{2}{3}\frac{t}{2}$

$M_{net} = 4.44 \times \frac{2}{3}\frac{t}{2} - 0.51 \times \frac{2}{3}\frac{t}{2} = 2620t\ n = N\ mm$

$e = \frac{2620\frac{t}{2}}{4444 + 510} = 0.53\frac{t}{2} = 0.27t\ mm$

From design chart, beta = 0.48

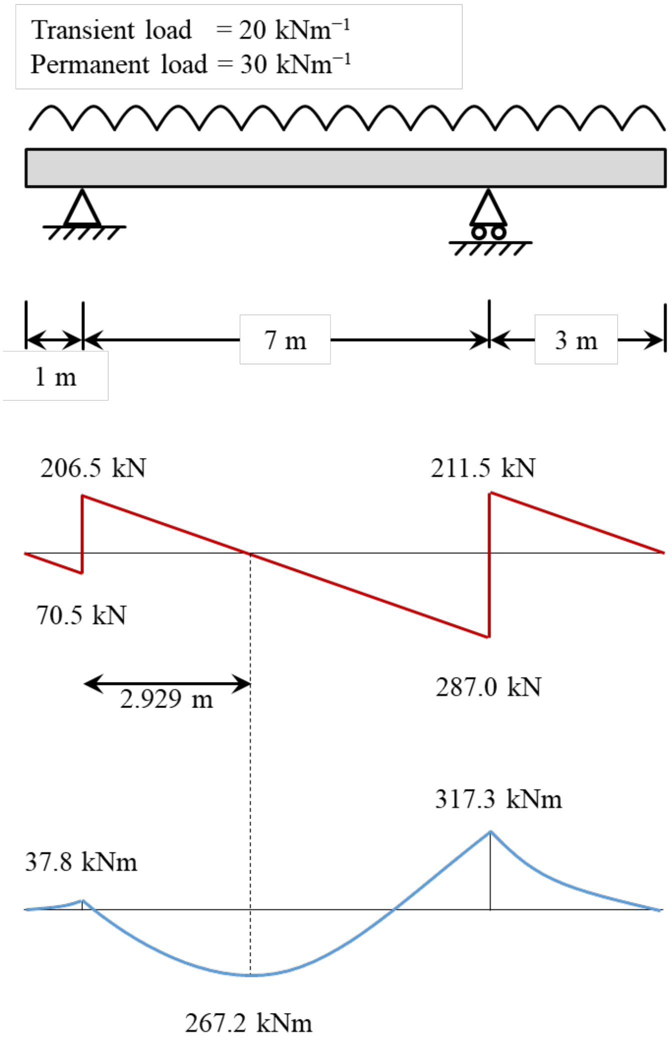
$$\beta f_d bt = 0.48 \times 0.67 \times 1000 \times 80 =\ 25.7\ kN$$

So also adequate for one-sided loading.

2)

a)

For uniform load distribution, i.e. no pattern loading.



$$w_{Ed} = 20 \times 1.5 + 30 \times 1.35 = 70.5\ kNm^{-1}$$

Taking moments about A

$$v_B = \frac{70.5 \times 11 \times \left(\frac{11}{2} - 1\right)}{7} = 498.5\ kN$$

$$v_A = 70.5 \times 11 - v_B = 277\ kN$$

$$M_A = 70.5 \times 1 \times 0.5 = 37.75\ kNm$$

$$M_B = 70.5 \times 3 \times 1.5 = 317.25\ kNm$$

$$M_{sag} = (277 \times 2.929) - \left(70.5 \times 3.929 \times \frac{3.929}{2}\right) = 267.2\ kNm$$

b)

span/depth ratios, highly stressed (i.e. beam):

simply supported: 7/14=0.5 m

cantilever: 3/6 = 0.5m

so d = 500 mm

for fire breadth >200 mm

for shear b = 287

so take b= 290 mm

c) Hogging over B is critical for flexure.

From b) moment at B is: $M_{Ed} = 317.25\ kNm$

Check whether compression reinforcement required: $M_u = \frac{0.225 f_{cu} b d^2}{\gamma_c} = \frac{0.225 \times 50 \times 290 \times 500^2}{1.5} = 543.75\ kNm$

$M_u < M_{Ed}$ so singly reinforced design is adequate.

Assuming flexural lever arm $z = 0.8d$

$$A_{s,req} = \frac{M_{ed}\gamma_s}{f_{yk} z} = \frac{317 \times 10^6 \times 1.15}{500 \times 0.8 \times 500} = 1823\ mm^2$$

Say 4H25 bars: $A_{s,prov} = 1963\ mm^2$

Check fit for one layer, considering bar diameter, bar spacing and axis distance:

$$4 \times 25 + 3 \times 25 + 2(65 - 12.5) = 280\ mm$$

b=290 mm, so bars fit in one layer at d=500

Check under-reinforced:

$$x = d\frac{\gamma_c A_s f_y}{\gamma_s 0.6 f_{cu} b d} = 147\ mm$$

$\frac{(d-x)\varepsilon_c}{x} = \frac{(500-147)0.0035}{147} = 0.008$ so steel has yielded d)

Critical location for shear is LHS of B, $V_{Ed} = 287\ kN$

Check whether shear reinforcement required:

$k = 1 + \sqrt{200/d} = 1.63 \leq 2.0$

$\rho_l = \dfrac{1963}{290 \times 500} = 0.014 \leq 0.02$

$V_{Rd,c} = \left[\dfrac{0.18}{\gamma_c} k \sqrt[3]{(100\rho_l f_{ck})}\right] b_w d = 108.5\ kN$

$V_{Rd,c} = \left(0.035\sqrt{k^3}\sqrt{f_{ck}}\right) b_w d = 66.8\ kN$

$V_{Rd,c} < V_{Ed}$ so shear reinforcement design required

Check concrete strut capacity at shallowest permissible strut angle 21.8 degrees:

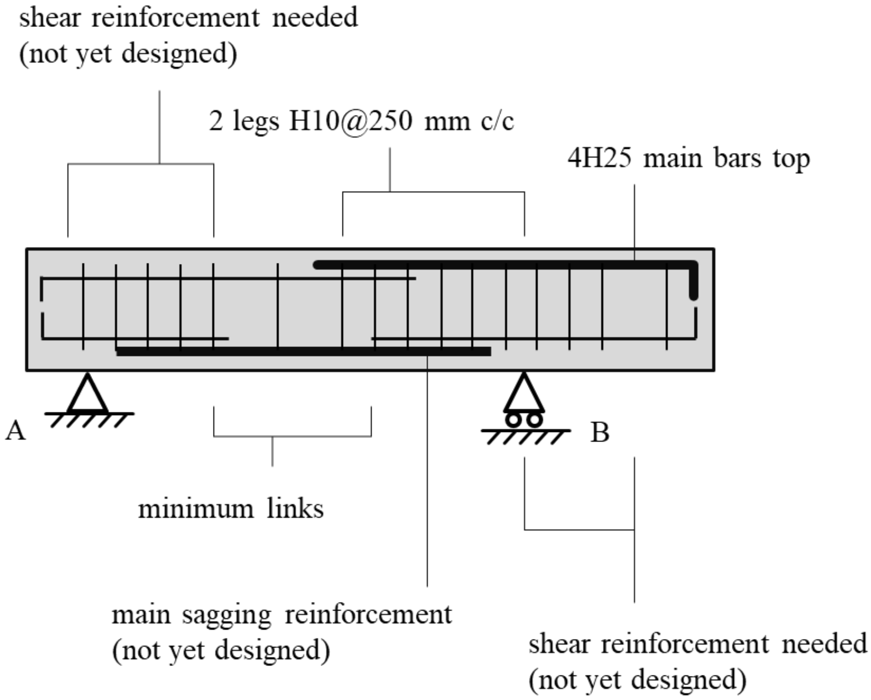
$f_{c,max} = 0.6\left(1 - \dfrac{f_{ck}}{250}\right)\dfrac{f_{cu}}{\gamma_c} = 16.8\ MPa$

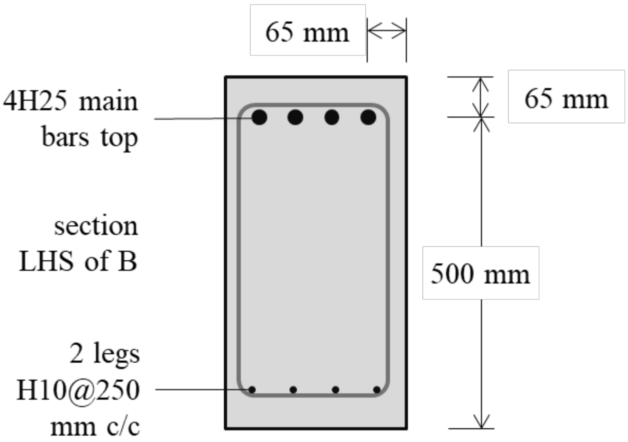
$V_{Rd,max} = f_{c,max}\frac{b_w 0.9d}{\cot\theta + \tan\theta} = 756\ kN$ so strut crushing not a problem

Taking 2-leg 10 mm links $A_{sw} = \ 157\ mm^2$

$s = \frac{A_{sw} f_y 0.9d \cot\theta}{V_{Rd,s}\gamma_s} = 268\ mm$ so use 10 mm closed links at 250 mm c/c spacing.

e)





f) Only uniform full load considered – no pattern loading. Adverse load case for sagging is no transient load on either cantilever.

Roughly … ULS is 43% transient, so sagging moment due to permanent $= 0.57 \times 267.2 = 152.7\ kNm$

For simply supported with transient over inner span only $\frac{20\times1.5\times7^2}{8} = 183.75\ kNm$

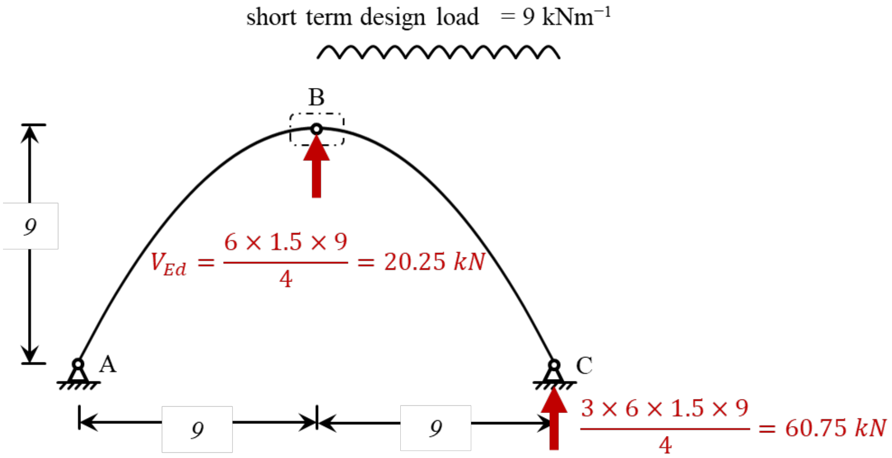
So sagging moment $\approx 336\ kN$ or at least 25% worse than for the uniform load case.

3)

a)

i)

by IA equilibrium…



ii)

Important to recognise that the eccentricity of centre of action of dowel group leads to a moment.

Assume resulting moment resisted by push-pull as in Prof Lawrence’s related examples paper question. In this case push-pull applies to rows 1 and 3 only, while shear carried across all dowels equally.

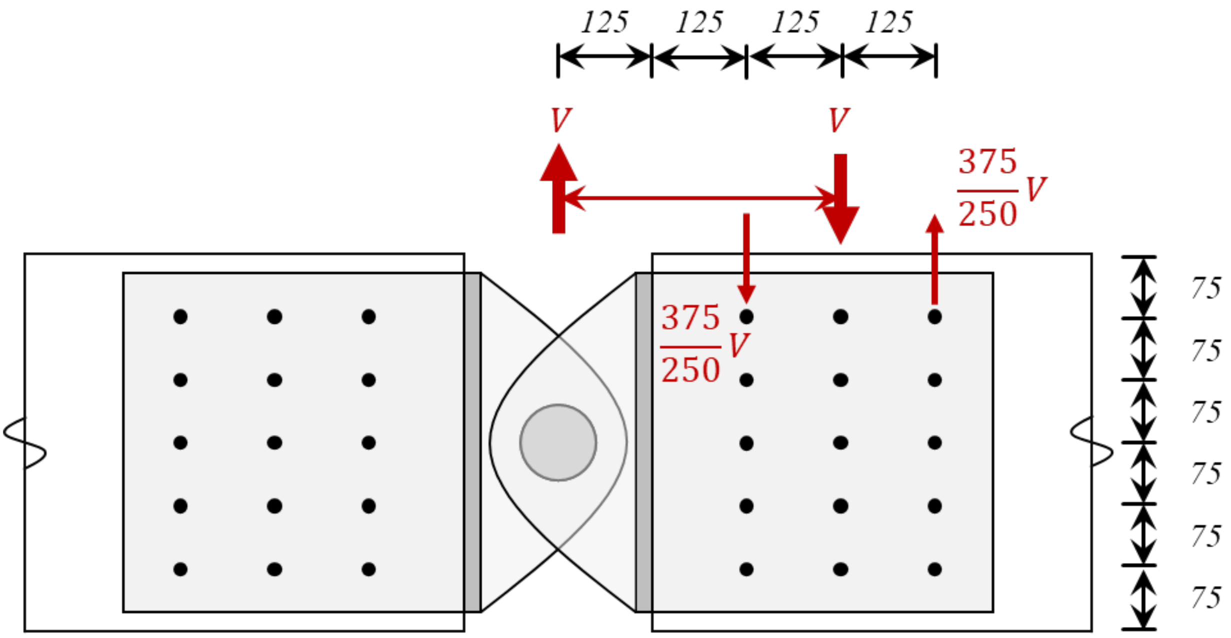
Resistance of dowel $F_{Rd} = \frac{characteric\ strength\ perpendicular}{\gamma_m} \times k_{mod} = \frac{12}{1.3} \times 0.9 = 8.3\ kN$

and

$$F_{Ed} = \frac{V_{Ed}}{15} + \frac{375}{250}\frac{V_{Ed}}{5} = 0.37V_{Ed} = 7.5\ kN$$

So $F_{Ed} < F_{Rd}$ so ok

RMF3



$$M_{Ed} = 0.375V = 0.375 \times 20.25 = 7.59\ kNm$$

(b)

$$M_{Ed} = \frac{wL^2}{8} = \frac{3 \times 9^2}{8} = 30.4\ kNm$$

$$\delta = \left(1 + k_{def}\right)\frac{5wL^4}{384EI}$$

$$I = \frac{bd^2}{12}$$

$$\delta = \left(1 + k_{def}\right)\frac{5wL^4}{384E\frac{bd^3}{12}}$$

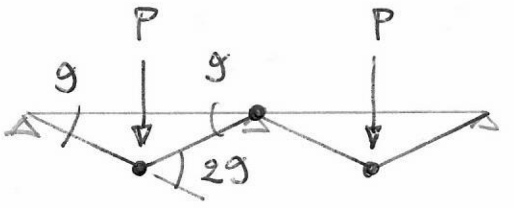
$$d = \sqrt[3]{\left(1 + k_{def}\right)\frac{60wL^4}{384Eb\delta}} = \sqrt[3]{1.6 \times \frac{60 \times 3 \times 9000^4}{384 \times 11000 \times 150 \times 30}} = \sqrt[3]{\frac{1.89 \times 10^{18}}{1.90 \times 10^{10}}} = 463\ mm$$

$$\sigma_{Ed} = \frac{M_{Ed}y}{I} = \frac{30.4 \times 10^6 \times \left(\frac{463}{2}\right)}{\frac{150 \times 463^3}{12}} = 5.7\ Nmm^{-2}$$

$$f_{m,d} = \frac{k_{mod}k_h k_{ls} k_{crit} f_{m,k}}{\gamma_m} = \frac{0.6 \times 1 \times 1 \times 1 \times 24}{1.3} = 11\ Nmm^2$$

so depth of 463 mm adequate in permanent condition.

(a)



$$P\left(\theta \frac{L}{2}\right) = 3\theta M_p \quad \Rightarrow \quad P = \frac{6M_p}{L}$$

$$M_p = W_{pl} \cdot f_y / \gamma_{M0} = (7.24)(10^5)(420) = 304 \text{ kNm}$$

$$\Rightarrow P_d = \frac{(6)(304)}{6} = 304 \text{ kN}$$

(b)

$b_f = 141.8$ mm
$t_f = 8.6$ mm
$r = 10$ mm
$h = 398$ mm
$t_w = 6.4$ mm

Flange

$$\frac{c}{t} = \frac{(b_f - t_w - 2r)/2}{t_f} = \frac{141.8 - 6.4 - 20}{(2)(8.6)} = 6.71$$

$\varepsilon = 0.75$

$\frac{c}{t} < 9\varepsilon = 6.75 \rightarrow$ Class ①

Web

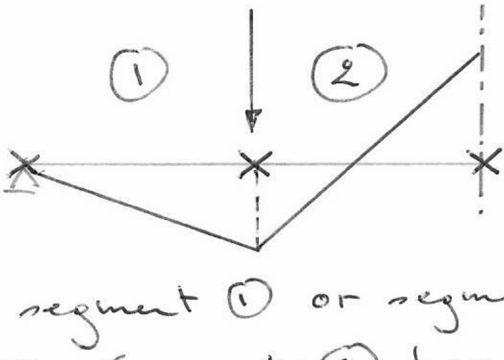
$$\frac{c}{t} = \frac{h - 2t_f - 2r}{t_w} = \frac{398 - 17.2 - 20}{6.4} = 56.4$$

$\frac{c}{t} > 72\varepsilon = 54$

$\frac{c}{t} < 83\varepsilon = 62 \rightarrow$ Class ② $\rightarrow$ Cross-section is Class ②

A Class ① cross-section is required for plastic design → the analysis method used in (a) is invalid.

(c)



Either segment ① or segment ② can be critical for LTB. Segment ② has the larger moment, but benefits from double curvature.

→ both need checking

$I_{zz} = (4.10)(10^6)$ mm^4

$J = (1.07)(10^5)$ mm^4

$C_w = I_{zz} d^2/4 = (1.55)(10^{11})$ mm^4

$d = h - t_f = 389.4$ mm

$L_{cr} = 3000$ mm

$$M_{cr} = C_1 \frac{\pi^2}{L_{cr}^2} E I_{zz} \sqrt{\frac{C_w}{I_{zz}} + \frac{L_{cr}^2}{\pi^2}\frac{GJ}{EI_{zz}}} = C_1(205) \text{ kNm}$$

①

$C_1 = 1.88$ (data sheets)

$M_{cr} = 385$ kNm

$\lambda = \sqrt{\frac{M_p}{M_{cr}}} = 0.89$

$\phi = 1.01$

$\chi = 0.67$

Curve ⓑ
Rolled
$h/b > 2$
$\alpha = 0.34$

$M_{b,Rd} = 204$ kNm

$\frac{5}{32} PL = 204$ kNm

$\Rightarrow P = 217$ kN

②

$C_1 \approx 2.73$ $(\psi = -\frac{5}{6})$

$M_{cr} = 559$ kNm

$\lambda = 0.74$

$\phi = 0.86$

$\chi = 0.76$

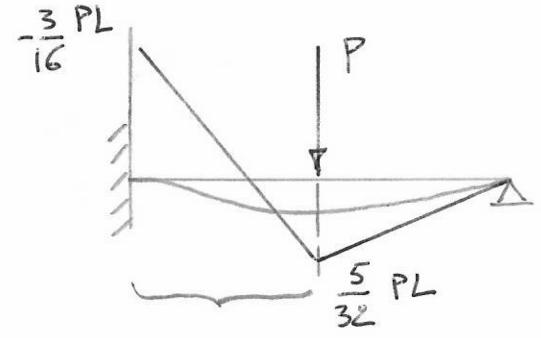
$M_{b,Rd} = 231$ kNm

$\frac{3PL}{16} = 231$ kNm

$\Rightarrow P = 205$ kN

critical

ⓓ



$$M(x) = \frac{\frac{11}{32}PL}{L/2} x - \frac{3}{16}PL = \frac{11}{16}Px - \frac{3PL}{16}$$

$$EI\,\alpha(x) = \frac{11}{32}Px^2 - \frac{3PL}{16}x + \cancel{C_1} \rightarrow \alpha = 0 \text{ for } x = 0$$

$$EI\,v(x) = \frac{11}{96}Px^3 - \frac{3}{32}PLx^2 + \cancel{C_2} \rightarrow v = 0 \text{ for } x = 0$$

$$\rightarrow EI\,v(x = 3m) = 1.97\,(10^9)\,P$$

$$P = \frac{275}{1.5}\text{ kN} = 183 \text{ kN}$$

$$\rightarrow v = \frac{(1.97)(10^9)(183)(10^3)}{(210\,000)(12510)(10^4)} = 14.4 \text{ mm}$$

$$L/300 = 20 \text{ mm} \rightarrow \underline{\text{Ok}}$$

Assessor's comments

Question 1

Most students attempted this question and performed adequately.

a) Load paths – the most common weaknesses were assuming diaphragm action from the lightweight cladding panels without justification and/or not considering the possibility of uplift due to wind and hence load-reversal (particularly as this means that the bottom chord of the trusses are laterally unrestrained). Some students parroted a standard description of load paths for a steel frame/concrete slab building that had limited relevance to the question.

b) i) Basic laminated glass design – many students did not recognize that the PVB interlayer would have negligible shear stiffness under anything other than a very short term load duration and that the glazing could thus be straightforwardly designed here as non-composite. Those who carried out an unnecessarily complicated design based on databook formulae for equivalent laminated sections were not marked harshly, but they inevitably lost valuable time.

ii) Masonry design – Relatively straightforward wall design for many students, but some inconsistency regarding assumed stress distributions.

Question 2

All candidates attempted this question and did so with reasonable success.

a) Shear and moment – most candidates could calculate and plot these for this statically determinate beam, but some did so much more efficiently than others.

b) Initial sizing – surprisingly few complete / reasoned choices. Inappropriate values of breadth or depth then sometimes led to more difficult design subsequently for bending and/or shear.

c) Flexure - A number of candidates, having correctly calculated a greater hogging than sagging moment, then inexplicably proceeded to design the beam for the sagging moment. Some candidates opted to solve a quadratic to obtain the flexural reinforcement which is slow and led to many calculation errors – those who estimated the flexural lever arm as 0.8d and then later checked the neutral axis depth had a much quicker and easier time.

d) Sketches of varying quality presented but most along the right lines.

e) Some correctly recognized that the load case presented (with transient load over the full length of the beam) would not be the worst case for sagging, most did not.

Question 3

Very few candidates attempted this question with one doing well and three doing poorly (having only attempted part b). It is unclear why the question should have proved so unpopular, as neither part of the question is tricky or laborious, and the first half is very similar to an examples paper question.

a) i) IA statics – The one student who attempted this part achieved full marks.

ii) Simplified multi-dowel connection check – Closely reflects an examples paper question. The one student who attempted this part achieved full marks.

b) Design simply supported purlin for long-term bending and deflection – no candidate properly included the creep factor in the deflection calculation, but otherwise done well by two of the four candidates who attempted this part.

Question 4

Most candidates attempted this question and did somewhat poorly. For most candidates this was their last question and poor time management may have played a part.

a) Fully plastic capacity check – many students did not recognise the need to perform a (IB plasticity) upper bound analysis. Many that did perform an upper bound analysis did not recognize the mid-span hinge rotation as 2θ rather than θ.

b) Cross section classification – most students performed this check correctly, or with only minor errors, but almost none recognized that the upper bound plastic analysis carried out in a) requires a Class 1 section.

c) Lateral torsional buckling check - poorly executed by most. Calculations often presented rather poorly, making them difficult to follow. Almost no recognition that the two segments of the beam are subject to different moment distributions.

d) Deflection check - A plethora of attempts to use databook cases to estimate deflections – some were logical, many were not. Many students did not seem to recognize that beam deflection is a serviceability consideration and should thus be checked with unfactored loads.