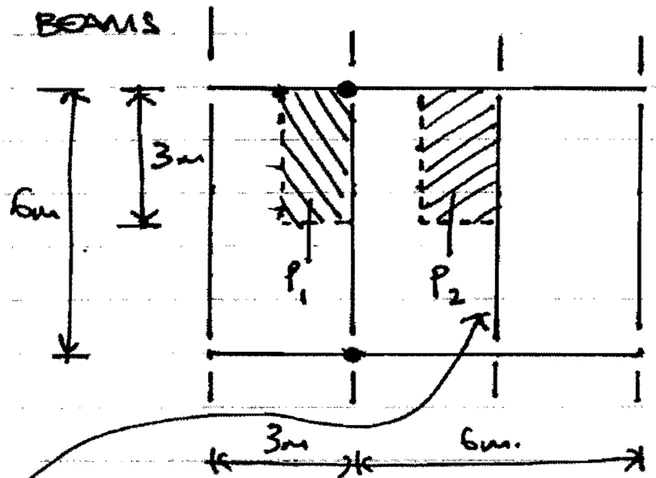
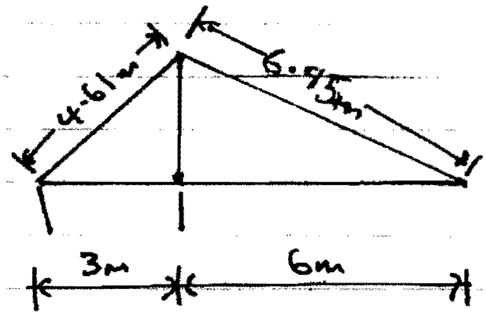
1a) VERTICAL LOADS ARISING FROM SELF-WEIGHT OF METAL CLADDING AND SNOW LOADS ARE RESISTED BY ONE-WAY FLEXURE OF METAL DECK.

THIS IS TRANSMITTED AS A UDL TO THE SECONDARY BEAMS





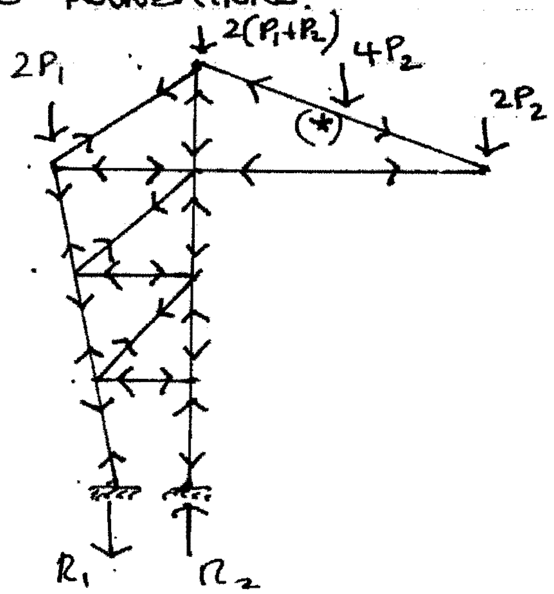
LOAD ON SECONDARY BEAM = $4P_2$

$$\therefore P_1 = q \times \frac{6}{2} \times \frac{4.61}{2}$$

$$P_2 = q \times \frac{6}{2} \times \frac{6.95}{4}$$

WHERE q = VERTICAL DESIGN LOAD

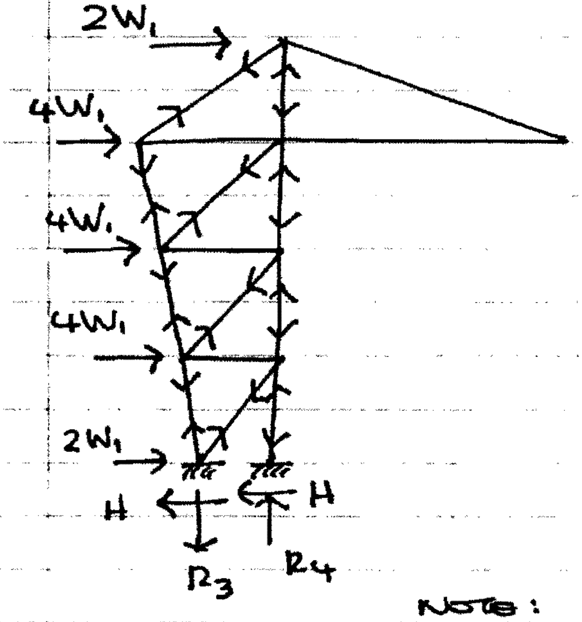
THE SECONDARY BEAMS ARE IN TURN SUPPORTED BY THE PRIMARY BEAMS (SKETCH BELOW). THESE POINT LOADS ARE TRANSFERRED AS TENSION / COMPRESSION IN THE TRIANGULATED FRAME AND SAFELY TO THE FOUNDATIONS.



(*) THIS MEMBER IN THE PRIMARY FRAME IS SUBJECTED TO FLEXURE IN ADDITION TO TENSION. ALL OTHER MEMBERS IN PRIMARY FRAME ARE STRUTS OR TIES.

HORIZONTAL LOADS IMPOSED BY WIND PRESSURE ARE RESISTED BY FLEXURE OF METAL CLADDING. THIS IS TRANSMITTED AS A UDL ON SECONDARY BEAMS (SIMILAR TO VERTICAL LOADS) WHERE:

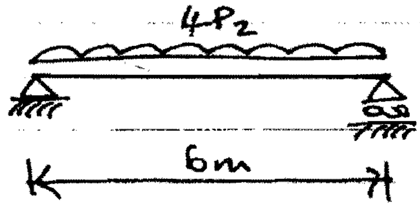
$$W_1 = w \times \frac{3.5}{2} \times \frac{6}{2} \quad (w = \text{WIND PRESSURE})$$



THESE ARE TRANSMITTED AS POINT LOADS ON THE PRIMARY FRAME (SKETCH LEFT). THESE POINT LOADS ARE SUBSEQUENTLY TRANSFERRED AS TENSION / COMPRESSION IN THE TRIANGULATED FRAME AND SAFELY TO THE FOUNDATIONS.

NOTE: STABILITY / LOAD PATH FOR TRANSVERSE WIND LOADS NOT DISCUSSED HERE.

1bi) SECONDARY BEAM A :



LOADS:

$$P_2 \text{ AT ULS} = \left[(1.2 \times 0.15\,kN/m) + (1.2 \times 0.75\,kN/m)\right] \times \frac{6}{2} \times \frac{6.9}{4}$$

$$= 5.63\ kN$$

$\therefore 4P_2$ AT ULS $= 22.52$ kN (EXCLUDES SELF. WT. OF BEAM)

$4P_2$ AT SLS $= 18.77$ kN

BENDING CAPACITY (ULS)

$$M_{APP} = (P_2)\,l/8 = 22.52 \times 6/8 = 16.89\ kNm$$

TRY UB 203×102×23 ($Z_p = 234\ cm^3$; $M_p = 83.1$ kNm)

THEORETICAL ELASTIC CRITICAL MOMENT:

$$M_c = \frac{\pi}{L}\left[EI_{yy}\left(GJ + \frac{\pi^2}{L^2}EC_w\right)\right]^{0.5}$$

$$C_w = \frac{D^2 I_{yy}}{4} = \frac{0.1939^2 \times 164\times10^{-8}}{4} = 1.54\times10^{-8}$$

$$\therefore M_c = \frac{\pi}{6}\left\{210\times10^9 \times 164\times10^{-8}\left[\left(81\times10^9\times7.02\times10^{-8}\right)+\frac{\pi^2}{36}\left(210\times10^9\times1.5\times10^{-8}\right)\right]\right\}^{0.5}$$

$$= 24.9\ \text{kNm}$$

$$\lambda_T = \sqrt{\frac{M_p}{M_c}} = \sqrt{\frac{83.1}{24.9}} = 1.82$$

$\therefore \chi_{LT} \simeq 0.25$ (FROM LTB CHART IN DATASHEET)

$\therefore M_{cr} = 0.25 \times 83.1$ kNm $= 20.76$ kNm > 16.89 kNm

NOTE: ADDITIONAL MOMENT AT ULS DUE TO SELF WT. $= (23\times9.81)\times 6^2/8 \times 1.2$

$= 1.2$ kNm

20.76 kNm $> (16.89 + 1.2)$ kNm

$\therefore$ UB 203×102×23 is SATISFACTORY

DEFLECTION (SLS)

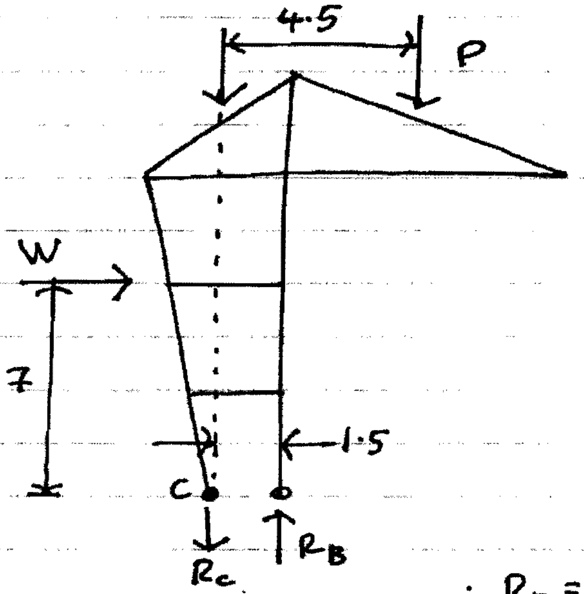
$$\delta_{MAX} = 6000/200 = 30\text{mm}$$

$$\delta = \frac{5(4P_2)\ell^3}{384\,EI} = \frac{5\times18.77\times10^3\times6000^3}{384\times210\times10^3\times2105\times10^4}$$

$$= 11.94\ \text{mm} < 30\ \text{mm}.$$

$\therefore$ UB 203×102×23 is SATISFACTORY

1bii) <u>COLUMN B LOADS :</u>



$\circlearrowleft^{C}$: $R_B = \frac{1}{1.5}(7W + 4.5P)$

WHERE: $P = q \times 6 \times 6.95$

$W = w \times 6 \times 14$

q = VERTICAL LOAD INTENSITY

w = WIND LOAD INTENSITY

$\therefore$ AT ULS: $q = 1.2 \times (0.15 + 0.75)$

$= 1.08 \text{ kN/m}^2$

$w = 1.2 \times 2$

$= 2.4 \text{ kN/m}^2$

$\therefore R_B = \frac{1}{1.5}(7 \times 201.6 + 4.5 \times 45) = \underline{1076 \text{ kN}}$

TRY UC 203×203×46 (PLASTIC SQUASH LOAD = 5870×355

= 2084 kN)

$\lambda = \frac{350}{5.13} = 68.2$; $\lambda_1 = \pi\sqrt{\frac{210 \times 10^3}{355}} = 76.4$

$\therefore \bar{\lambda} = 68.2/76.4 = 0.89$; $r/y = 5.13/(203.2/2) = 0.505$

$\therefore$ USE CURVE B

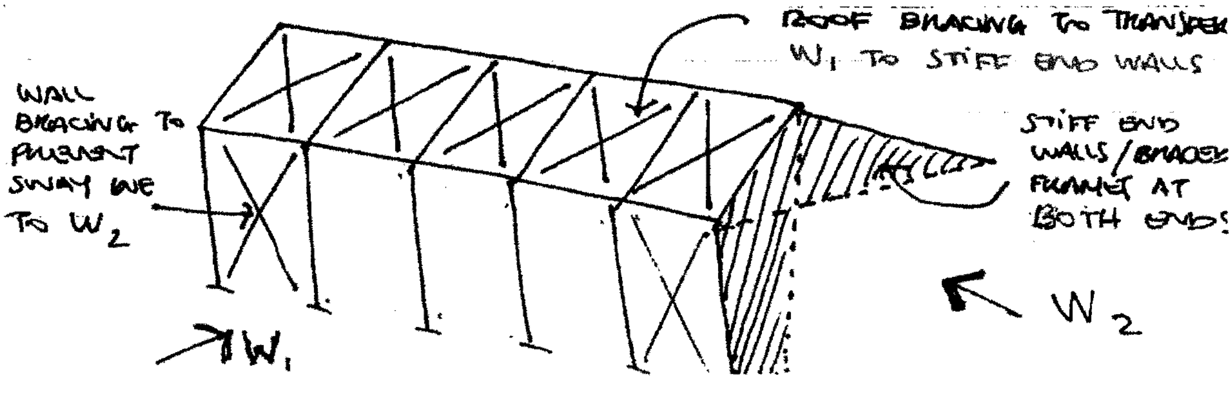
$\chi \doteq 0.66$ (FROM BUCKLING CHART IN DATASHEET)

$\therefore R_{ULT} = 0.66 \times 2084 \text{ kN}$

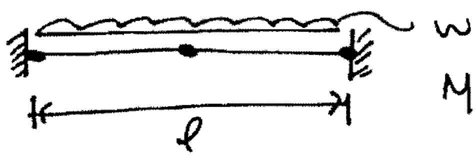
$= 1375 \text{ kN} > 1076 \text{ kN}$

$\therefore$ <u>UC 203 × 203 × 46 IS SATISFACTORY.</u>

1c)



2 a) FIRST ESTIMATE $\Rightarrow$ ASSUME FIXED ENDED BEAM



$M_p = wl^2/16$

WHERE l IS THE CLEAR SPAN BETWEEN HAUNCHES

$= 2(16-3) = 26\text{m}$

$\therefore M_p = 9 \times 26^2/16 = 380.3\ \text{kNm}$

DESIGN YIELD STRESS $= \sigma_y/\gamma_m = \dfrac{275}{1.05} = 262\ \text{MPa}$

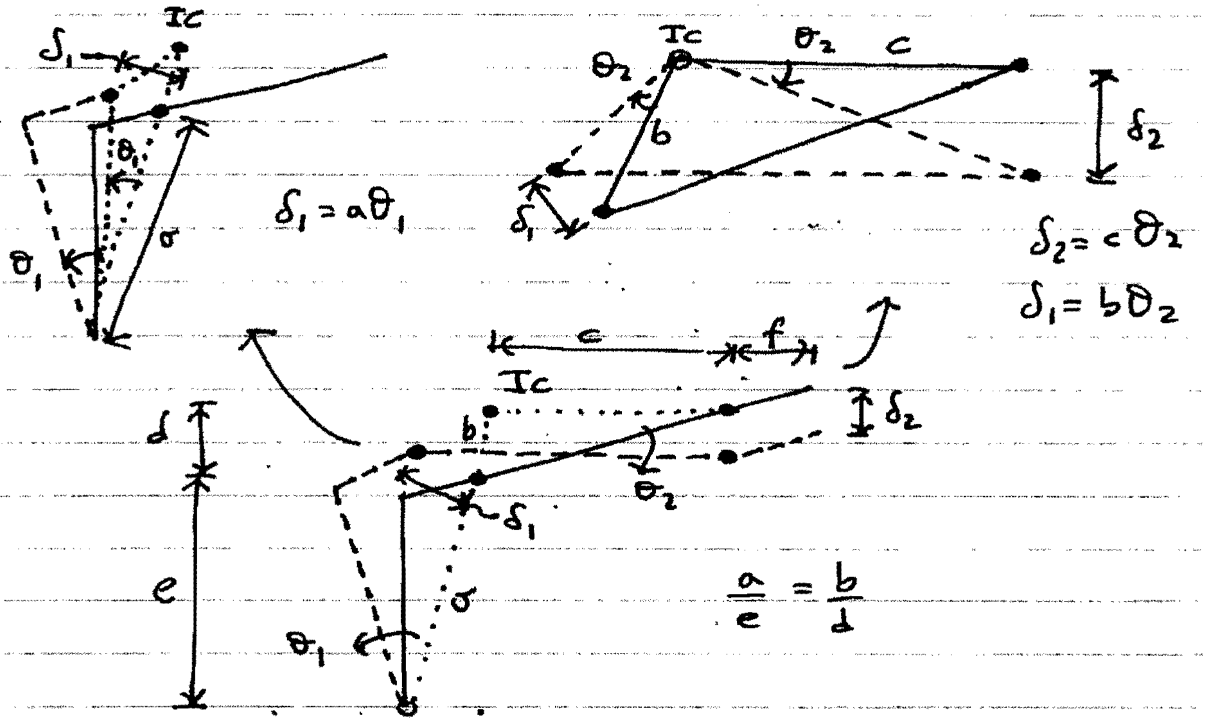
$\therefore Z_p = 380.3 \times 10^3\ \text{Nm} / 262\ \text{MPa} = 1452\ \text{cm}^3$

TRY $\underline{\text{UB } 457 \times 152 \times 67 \ (Z_p = 1453\ \text{cm}^3;\ M_p = 380.6\ \text{kNm})}$

$\therefore$ SELF-WT. $= 67 \times 9.81 \times 1.4/\cos 10° = 0.93\ \text{kN/m}$

$\therefore$ REVISED LOAD $= 9 + 0.93 = \underline{9.93\ \text{kN/m}}$

b)



$\dfrac{a}{e} = \dfrac{b}{d}$

$\therefore \theta_1 = \dfrac{b\theta_2}{a} = d\delta_2/ec \quad ; \quad \theta_2 = \delta_2/c$

EXTERNAL WORK DONE = WORK DISSIPATED IN HINGES

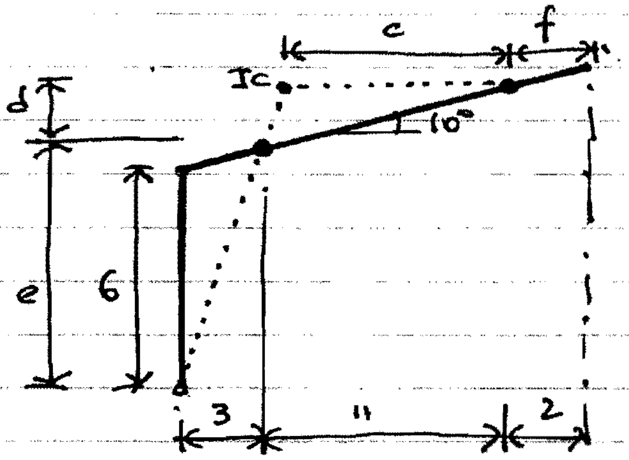
$$\underbrace{2}_{\text{2 HALVES}} \times \underbrace{\left(w f \delta_2 + w c \delta_2/2 \right)}_{} = \underbrace{2}_{\text{2 HALVES.}} M_p \left[(\theta_1 + \theta_2) + \theta_2 \right]$$

IGNORES −VE WORK DONE BY RISING OF RAFTER OVER HAUNCH REGION.

$$\therefore \; w \delta_2 \left(f + \frac{c}{2} \right) = M_p \left(\underbrace{\frac{\delta_2 d}{ce}}_{\theta_1} + \underbrace{\frac{2\delta_2}{c}}_{2\theta_2} \right)$$

$$M_p = \frac{w\left(f + \frac{c}{2}\right)}{\left(\frac{d}{ce} + \frac{2}{c}\right)} = \frac{\frac{wc}{2}\left(\frac{2f}{c} + 1\right)}{\frac{2}{c}\left(\frac{d}{2e} + 1\right)} = \frac{wc^2}{4}\left[\frac{1 + 2f/c}{1 + d/2e}\right]$$

2 c)



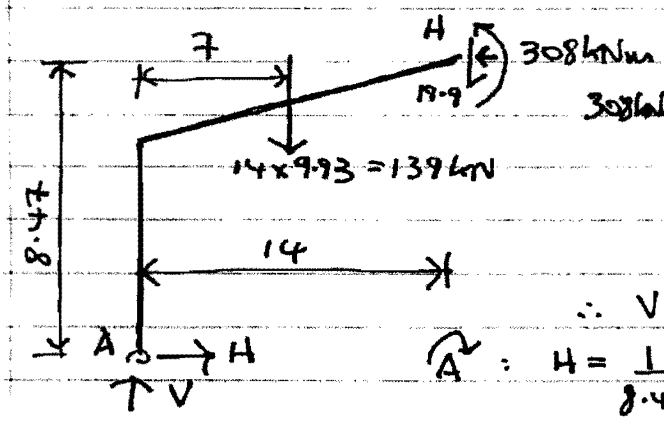
$d = 11 \tan 10° = 1.94\text{m}$

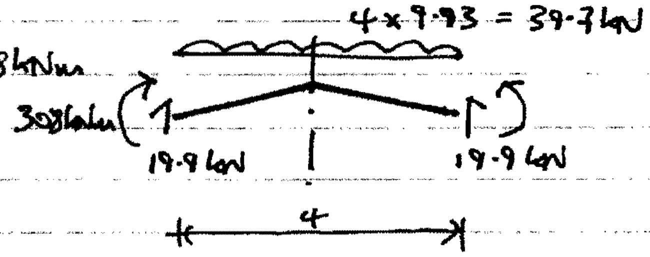
$e = 6 + 3\tan 10° = 6.53\text{m}$

$g = \frac{3}{6.53}(6.53 + 1.94) = 3.89\text{m}$

$c = 16 - (3.89 + 2) = 10.11\text{m}$

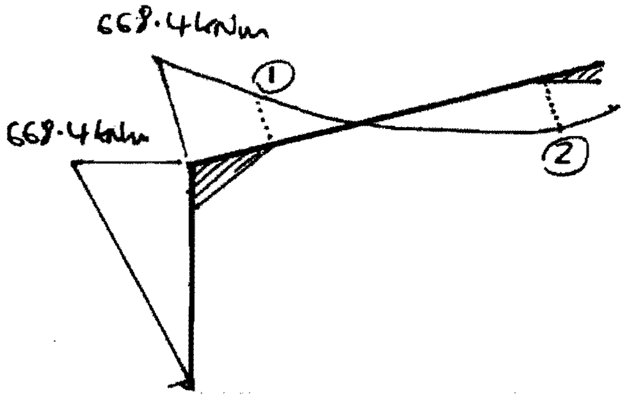
$$\therefore \; M_p = \frac{9.93 \times 10.11^2}{4}\left[\frac{1 + 4/10.11}{1 + 1.94/13.06}\right] = 308.3\,\text{kNm}$$





$\therefore \; V = 139\text{kN} + 19.9\text{kN} = 159\text{kN}$

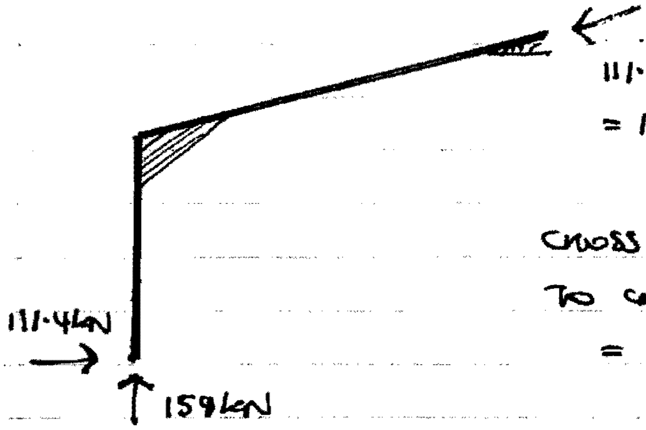
$\curvearrowright A: \; H = \frac{1}{8.47}\left[(19.9 \times 14) + (139 \times 7) - 308\right] = 111.4\text{kN}$



① = $668.4 - \left[(9.93 \times 3 \times 1.5) + 308\right]$

$= \underline{\underline{315.7\,\text{kNm}}}$

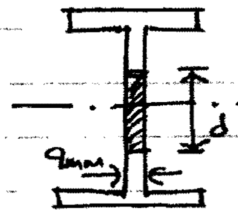
② $= \underline{\underline{308\,\text{kNm}}}$



$111.4 \cos 10^\circ + 159 \sin 10^\circ$
$= 137.3\,\text{kN}$

CROSS SECTIONAL AREA REQUIRED TO CARRY AXIAL FORCE

$$= \frac{137.3 \times 10^3}{262\,\text{N/mm}^2} = 524\,\text{mm}^2$$



UB 457×152×67

$\therefore\ d = 524/9 = 58\,\text{mm}$

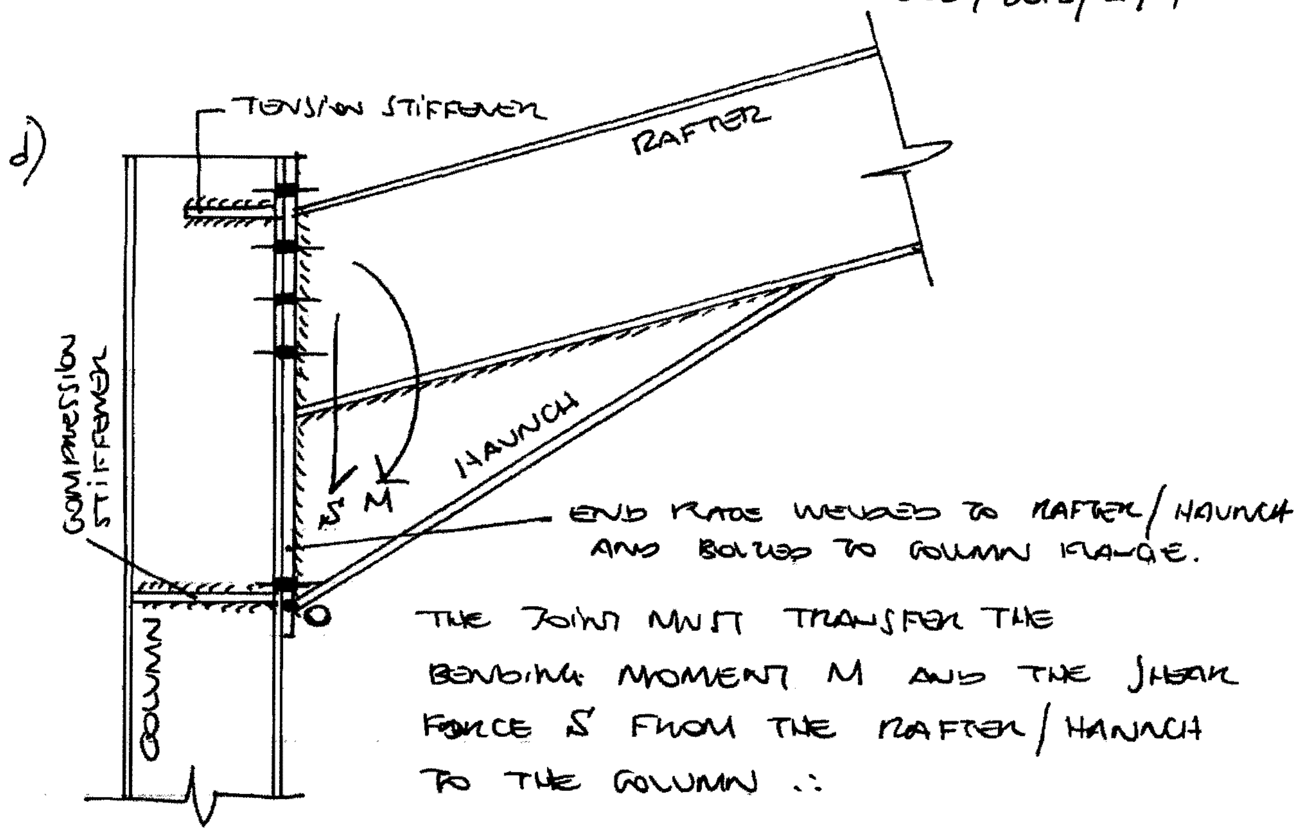
REDUCED $M_p = M_p - \sigma_d t d^2/4$

$= 380.6\,\text{kNm} - \left(262 \times 9 \times 58^2/4\right) \times 10^{-6}$

$= 378.6\,\text{kNm} > 315.7$

$\therefore$ <u>UB 457×152×67 is SATISFACTORY.</u>

d)



THE JOINT MUST TRANSFER THE BENDING MOMENT M AND THE SHEAR FORCE S FROM THE RAFTER/HAUNCH TO THE COLUMN ∴

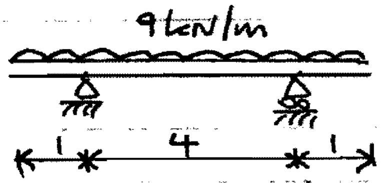
<u>DESIGN CHECK FOR M (RAFTER/HAUNCH PIVOTS ABOUT O):</u>

1. CHECK WELD IN TENSION BETWEEN RAFTER AND END-PLATE.
2. CHECK TENSION IN BOLTS, PARTICULARLY 1^{ST} AND 2^{ND} ROW FROM TOP.
3. CHECK PRYING/BENDING OF END-PLATE AND COLUMN FLANGE IN VICINITY OF TOP FLANGE OF RAFTER.
4. CHECK TENSILE CAPACITY OF COLUMN WEB IN VICINITY OF TOP FLANGE OF RAFTER (INTRODUCE TENSION STIFFENER IF REQUIRED).
5. CHECK BUCKLING OF COLUMN WEB IN VICINITY OF BOTTOM OF HAUNCH (INTRODUCE COMPRESSION STIFFENER IF REQUIRED).

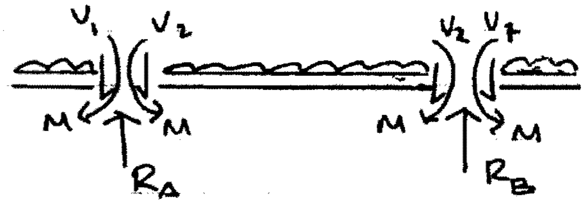
<u>DESIGN CHECK FOR S:</u>

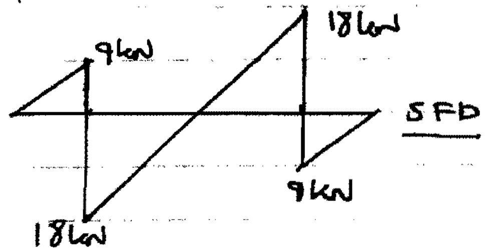
1. CHECK SHEAR CAPACITY OF WELD.
2. CHECK SHEAR CAPACITY OF BOLTS (NOTE: BOLTS WILL BE SUBJECTED TO COMBINED SHEAR AND TENSION).
3. CHECK BEARING CAPACITY OF END-PLATE AND COLUMN FLANGE AT BOLTS.

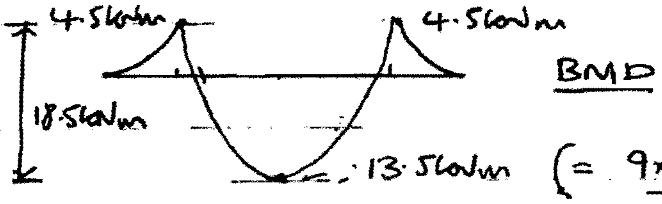
3a)



$\equiv$

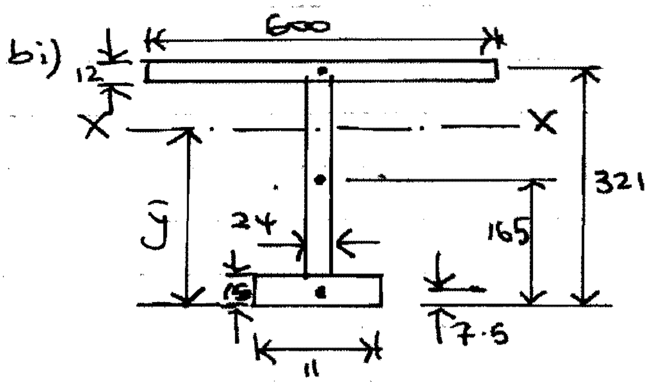






$\left(= \frac{9 \times 4^2}{8} - 4.5\right)$

bi)



$$\bar{y} = \Sigma Ay / \ldots$$

$$= \frac{\left[(600 \times 12 \times 321) + (300 \times 24 \times 165) + (15 \times 75 \times 7.5)\right]}{\left[(600 \times 12) + (300 \times 24) + (75 \times 15)\right]}$$

$\therefore \quad \bar{y} = 226\text{ mm}$

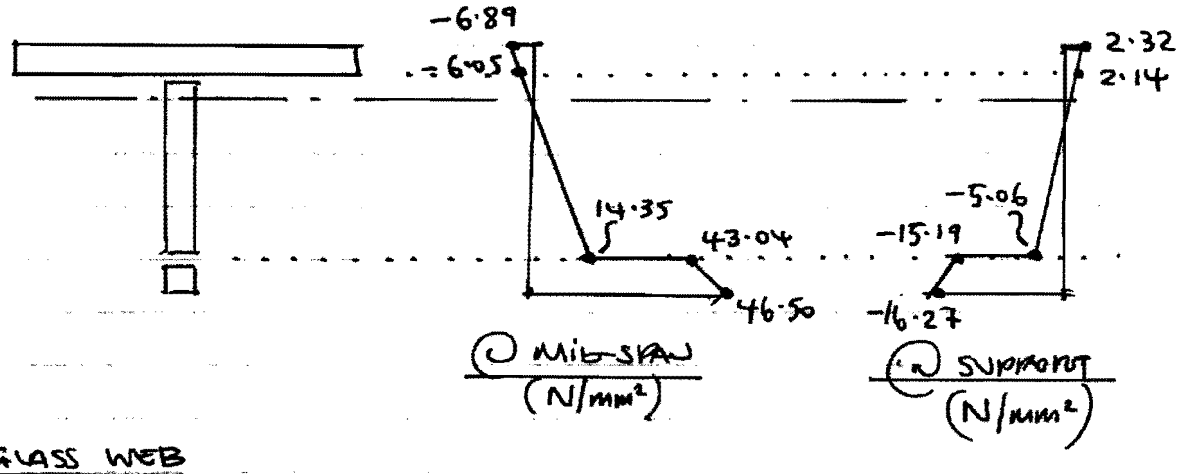
$$25 \cdot \frac{E_s}{E_g} = 25 \times \frac{210}{70} = 75\text{mm}$$

$$I_{xx} = \left[\left(\frac{600 \times 12^3}{12}\right) + (600 \times 12 \times 95^2)\right] + \left[\left(\frac{24 \times 300^3}{12}\right) + (24 \times 300 \times 61^2)\right]$$
$$+ \left[\left(\frac{75 \times 15^3}{12}\right) + (75 \times 15 \times 218.5^2)\right]$$

$$\therefore I_{xx} = 199.6 \times 10^6\ \text{mm}^4$$

AT MID-SPAN $\sigma(y) = My/I = 13.5 \times 10^6\, y / 199.6 \times 10^6 = 0.068y$

AT SUPPORT $\sigma(y) = My/I = 4.5 \times 10^6\, y / 199.6 \times 10^6 = 0.023y$

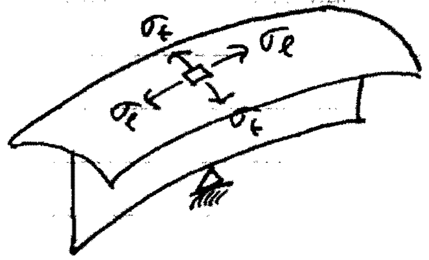


bii) GLASS WEB

MAX. LONGITUDINAL TENSILE STRESS = $14 \cdot 35\ N/mm^2$

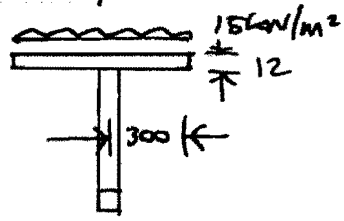
$\therefore$ USE FULLY TOUGHENED GLASS

GLASS FLANGE



$\sigma_l = 2 \cdot 32\ N/mm^2$

$\sigma_t \rightarrow$



$\therefore M = wl^2/2 = (15 \times 0 \cdot 3^2)/2 = 0 \cdot 675\,kNm$

$\therefore \sigma_t = \frac{M}{Z} = \frac{0 \cdot 675 \times 10^6 \times 6}{1000 \times 12^2} = 28 \cdot 2\ N/mm^2$

$\therefore \sigma_{resultant} = \sqrt{2 \cdot 32^2 + 28 \cdot 2^2} = 28 \cdot 3\ N/mm^2$

$\therefore$ USE FULLY TOUGHENED GLASS

STEEL BLOCK

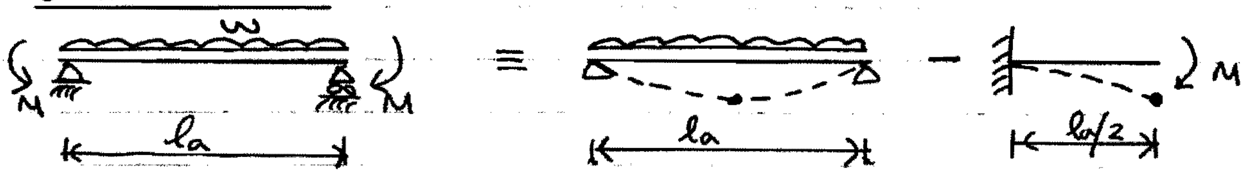
$\sigma = 46 \cdot 10\ N/mm^2$ $\therefore$ USE S275 STEEL

biii) TRANSVERSE SHEAR FLOW $q = SA_c\bar{y}/I \Rightarrow \tau = SA_c\bar{y}/It$

$\therefore t = \frac{18 \times 10^3 N \times 12 \times 600 \times 95}{199 \cdot 6 \times 10^6 \times 5\ N/mm^2} = 12 \cdot 3\ mm$ (GLASS - GLASS)

$$t = \frac{18\times10^3\,N \times (15\times75)\times(226-7.5)}{199.6\times10^6 \times 5\,N/mm^2} = \underline{\underline{4.4\,mm}}\ \text{(STEEL-GLASS)}$$

b iv) δ AT MID-SPAN:



$$= \frac{5wl_a^4}{384EI} - \frac{w}{2}\cdot\frac{l_a^2}{8EI}$$

$$= \frac{wl_a^2}{EI}\left(\frac{5l_a^2-24}{384}\right)$$

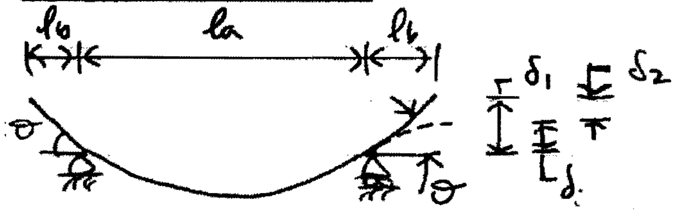
$$= \frac{wl_a^2}{384EI}\left(5l_a^2-24\right)$$

$$\therefore \delta_{MID\,SPAN} = \frac{9\times10^3\times4^2(5\times4^2-24)}{1.6\times384\times70\times10^9\times199\times10^{-6}}$$

γ_f (SLS)

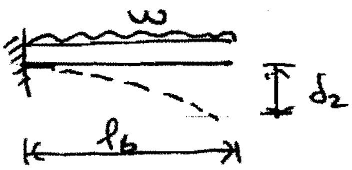
$$= \underline{\underline{0.94\,mm}} = SPAN/4255$$

δ AT FREE END:



$\therefore \delta = \delta_1 - \delta_2$

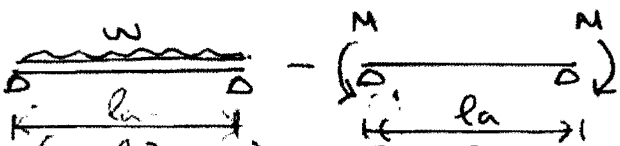
δ_2 :



$$\delta_2 = \frac{w\,l_b^4}{8EI}$$

$\delta = l_b\theta$

WHERE $\theta = \theta_1 - \theta_2$



$$\therefore \theta = \left(wl_a^3/24EI\right) - \left(\frac{wl_b^2}{2}\cdot\frac{l_a}{2}\cdot\frac{1}{EI}\right)$$

$$\therefore \delta_1 = l_b\left(\frac{w l_a^3}{24EI} - \frac{w l_a l_b^2}{4EI}\right)$$

$$\Rightarrow \delta = \frac{w l_b^4}{8EI} + \frac{w l_b^3 l_a}{4EI} - \frac{w l_a^3 l_b}{24EI}$$

$$= \frac{w l_b}{24EI}\left[3 l_b^2 (l_b + 2 l_a) - l_a^3\right]$$

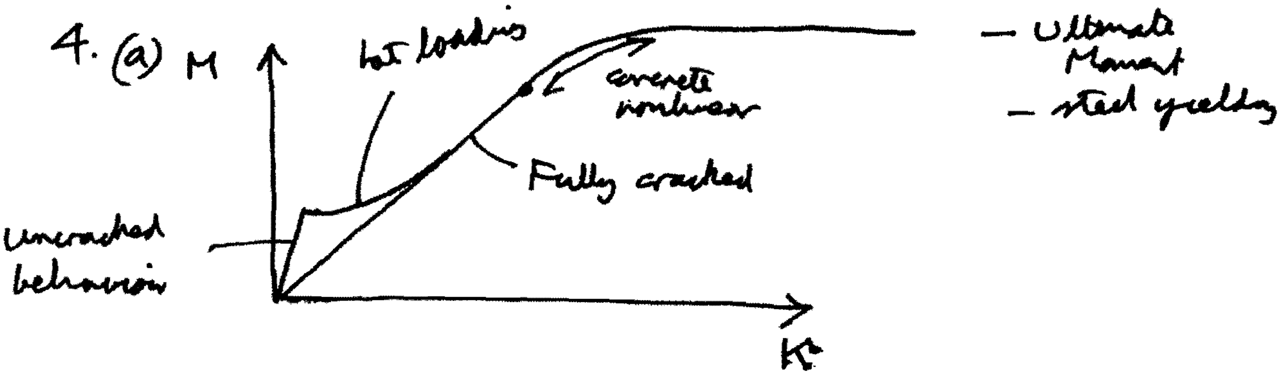
$$\therefore \delta = \frac{9 \times 10^3 \times 1}{1.6 \times 24 \times 70 \times 10^9 \times 199.6 \times 10^6}\left[3(1+8) - 4^3\right]$$

γ_f (SLS) ~ 1.6

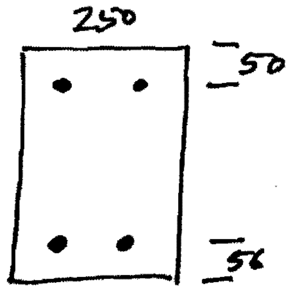
$$\underline{\underline{= -0.62\,\text{mm (UPLIFT)}}} = \text{SPAN}/6451$$

c)
- THICKNESS OF STEEL AND GLASS CAN BE REDUCED FURTHER, THEREBY INCREASING WORKING STRESSES UP TO DESIGN STRENGTHS WITHOUT BREACHING SERVICEABILITY LIMITS.
- MONOLITHIC GLASS FOR FLANGE IS NOT ADVISABLE AS IT PROVIDES NO REDUNDANCY ON FAILURE. CONSIDER REPLACING WITH LAMINATED GLASS.
- CONSIDER LONG TERM LOADING ON ADHESIVES
- CONSIDER SHEAR DEFORMATION OF ADHESIVE THAT COULD RESULT IN PARTIALLY COMPOSITE T-BEAM RATHER THAN FULLY COMPOSITE T-BEAM.
- CONSIDER DIFFERENT COEFFICIENTS OF THERMAL EXPANSION IN STEEL AND GLASS LEADING TO STRESSES IN THE ADHESIVE. THIS CAN BE CRITICAL IN LONG BEAMS.
- THE FREE END OF THE WEB IS IN COMPRESSION OVER THE SUPPORT. CHECK RESISTANCE TO LATERAL TORSIONAL BUCKLING OR PROVIDE ADEQUATE RESTRAINT.

4. (a)



(b)



$A_s' = \frac{\pi 20^2}{4} \cdot 2 = 628 \text{ mm}^2$

$A_s = \frac{\pi 32^2}{4} \cdot 2 = 1608 \text{ mm}^2$

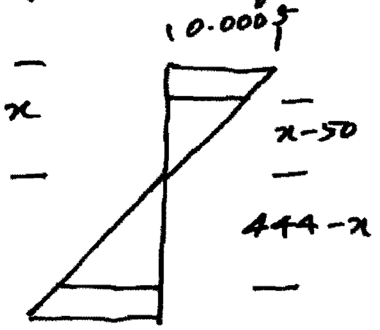
(i) 1st cracking when tensile stress at bottom = 3 MPa

$\frac{\sigma}{y} = \frac{M}{I} \Rightarrow \quad M = \frac{3}{250} \cdot \frac{500^3 \cdot 250}{12} \quad (\text{N, mm})$

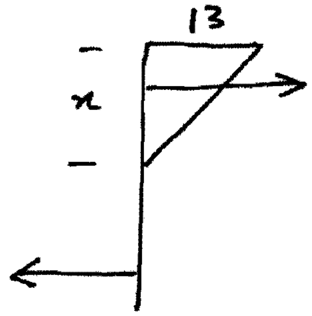
$\underline{M = 31.25 \text{ kNm}}$

[Curvature $\frac{\sigma}{E_c} \cdot \frac{1}{250} = \frac{3}{26 \cdot 10^3} \cdot \frac{1}{250} = 0.46 \cdot 10^{-6} \text{ mm}^{-1}$
not asked for]

(ii) Strain diagram



Stress (MPa)



Stress in top steel $= \frac{(x-50)}{x} \cdot \underbrace{0.0005 \cdot E_s}_{100}$ $\quad (E_s = 200 \cdot 10^3)$ (comp)

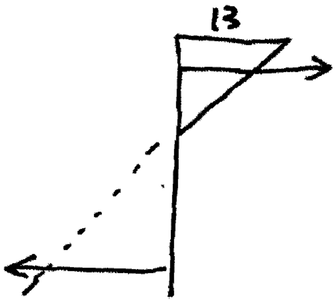
Stress in bottom steel $= \frac{(444-x)}{x} \cdot 100$ (tension)

Calculate x by satisfying longitudinal equilibrium

$$\underbrace{13 \cdot 250 \cdot \frac{x}{2}}_{\text{concrete}} + \underbrace{100 \cdot 628 \cdot \frac{(x-50)}{x}}_{\text{top steel}} = \underbrace{\frac{(444-x)}{x} \cdot 1608 \cdot 100}_{\text{bottom steel}}$$

Quadratic in x $\Rightarrow$ $x = 152.5$ mm (reasonable)

Can now calculate all strains $\Rightarrow$ stresses



$\varepsilon = 0.000335 \Rightarrow \sigma = 67$ MPa (comp)

$\varepsilon = 0.0009586 \Rightarrow \sigma = 191$ MPa (tens)

Take moments about top fibre (or any other defined point)

$$191 \cdot 1608 \cdot 444 - 67 \cdot 50 \cdot 628 - \frac{13 \cdot 250 \cdot 152.2}{2} \cdot \frac{152.2}{3}$$

N, mm

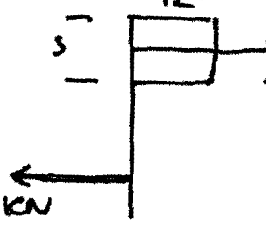
$$= \underline{109.2 \text{ kNm}}$$

$\left(\text{Curvature} = \frac{13}{26 \cdot 10^3} \cdot \frac{1}{102.2} = 4.89 \cdot 10^{-6} \text{ mm}^{-1}\right)$

(iii) Ultimate Moment Capacity assuming all steel yields

Stress in concrete $\frac{30 \cdot 0.6}{1.5} = 12$ MPa

Stress in steel yielding $= \frac{460}{1.1} = 418$ MPa



$\therefore 12 \cdot 250 \cdot s = (672 - 262.5) \cdot 10^3$

$\Rightarrow s = 136.5$ mm.

Take moments about the top

$$M = 672 \times 444 - 262.5 \,.\, 50 - \frac{136.5 \,.\, \overset{3}{\cancel{12}} \,.\, \cancel{250}}{\cancel{2} \,.\, \underset{4}{\cancel{1000}}} \,.\, \frac{136.5}{\cancel{2}}$$

(kN, mm)

$$= \underline{\underline{271.3 \text{ kNm}}}$$

$$\text{Curvature} = \frac{0.0035}{136.5} = 25.6 \,.\, 10^{-6} \text{ mm}^{-1}$$

Strain in top steel = $25.6 \,.\, (136.5 - 50) = 0.0022$

Stress in top steel = 442.9 N/mm^2

(not yielding)

Examiner's comments to be attached to the crib.

A disappointing paper with raw marks in all questions <50%. The candidates were clearly under time pressure but this does not explain the lack of logical thought in the bits they did do.

Qu 1. Steel design for a grandstand

The main problem here was that most of the candidates only checked for one of the three conditions (strength, stiffness and buckling), although the first two were specifically asked for and they were given a strong hint about the need to think about the buckling. Quite a large number could not determine the load in the column at the base, either leaving out the wind load, or leaving out the snow load, or simply being unable to apply equilibrium properly. The last part, where they were asked (implicitly) to find an alternative load path for the wind load was very poorly done - very few seemed able to think in three dimensions.

Qu 2. Steel Portal Frame

The most popular question. The methods adopted by the candidates were largely correct, but there were several errors and omissions in the calculations. The first part asked the candidates to estimate the size of the rafter by assuming a fixed-ended condition. Most candidates successfully derived (or recalled) that for a fixed ended beam subjected to UDL, $M_p = wl^2/16$, but several candidates used the full span between the columns rather than the clear span between the haunches. Part *b* of the question asked the candidates to derive the plastic moment at the plastic hinges of a portal frame. Most students set this up correctly by equating the external work done to the work dissipated in the hinges, but several candidates failed to notice that the total rotation in the hinge closest to the haunch was ($\theta_1 + \theta_2$) rather than θ_1. The free body diagrams required to solve Part *c* of the question were generally correct, but the most common error was an inconsistent sign convention leading to an incorrect direction of the bending moment at the apex haunch. Despite these errors, most candidates were able to sketch a sensible bending moment diagram. The final part of the question asked the candidates to consider the design of the column-rafter haunch. There were 'easy' marks to be gained in this final part, but few candidates attempted it. Those that did secured most of the marks allocated to this part.

Qu 3. Composite Glass/Steel beam

Very disappointing question. It could easily have been set to the 1A students as a revision exercise, since it is essentially the application of 1A principles to a new material, but it is almost as though they had put that knowledge away, never to be used again. They were unable to distinguish between loads per unit length and loads per unit area, and a significant number tried to use the density of the steel and the glass to transform the section when determining the second moment of area, rather than Young's modulus. Some simply ignored the distinction between two materials. Even if they got it right, they seemed incapable of sketching the expected stress distribution over the section. The crib expected them to take account of transverse bending of the flange when determining the stresses over the support but no one attempted that and it wasn't penalised given the overall difficulty. Two candidates included it in their answers to part (c) - "what else would you check?" Very few of them made any serious attempt at calculating the deflections.

Qu 4. Moment curvature relationship of a reinforced concrete beam

Quite a few candidates did not appear to know what a moment-curvature relationship was, and several did not distinguish between the uncracked and cracked behaviour. The biggest problem was that almost nobody used axial equilibrium in the beam for determining where the neutral axis was located, which was essential for determining the cracked-elastic and ultimate load stress states, and wrote various degrees of nonsense depending on where they thought the neutral axis might be. Several used the design equations to determine the ultimate moment capacity, for which they were given some credit. The overall impression from this question was that they were trying to remember "the formula for ..." rather than applying fundamental principles.

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May 2013.