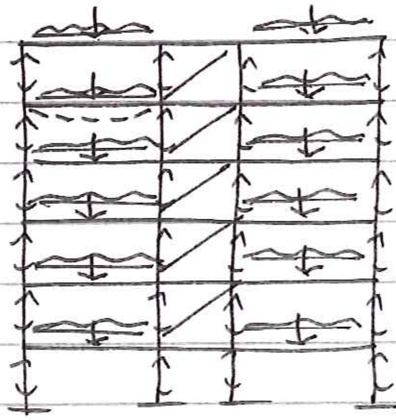
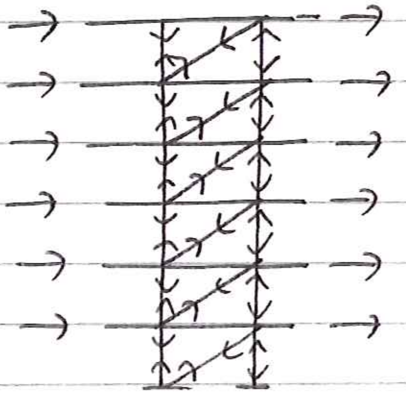
1(a) <u>VERTICAL LOADS</u> FROM THE SLAB SELF-WEIGHT AND IMPOSED LOADS ON SLABS ARE TRANSMITTED IN FLEXURE IN THE SLABS, SUPPORTED BY THE SECONDARY BEAMS. THE SECONDARY BEAMS ARE IN TURN SUPPORTED BY THE PRIMARY BEAMS / COLUMNS AND THE LOADS ARE TRANSMITTED AT EACH FLOOR DOWN TO THE FOUNDATIONS.

<u>HORIZONTAL LOADS</u> ARISING FROM WIND LOADS ARE TRANSFERRED BY FLEXURE OF THE CLADDING INTO THE FLOOR PLATES. THE FLOOR ACTS AS A STIFF DIAPHRAGM TO TRANSMIT THE LOADS TO THE BRACED CORE. THE CORE TRANSMITS THE LOADS TO THE FOUNDATIONS BY ACTING AS A VERTICAL CANTILEVER.

(4)

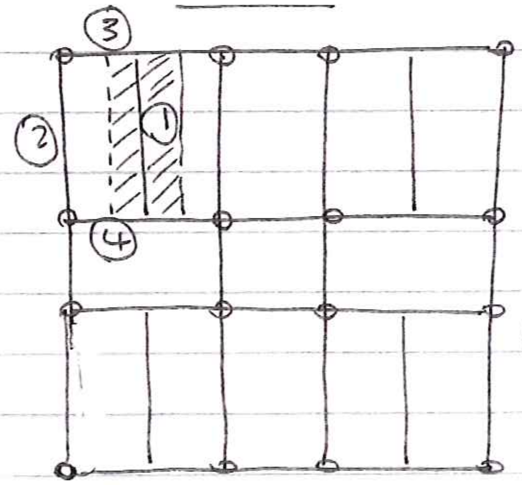


<u>VERTICAL</u>



<u>HORIZONTAL</u>

(b)



(2)

DESIGN LOAD AT (ULS):

$(0.375 \times 1.4) + (3 \times 1.6) = 5.325 \text{ kN/m}$

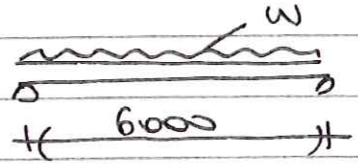
DESIGN LOAD AT SLS:

$(0.375 \times 1) + (3 \times 1) = 3.375 \text{ kN/m}$

STRENGTH CHECK: $Z_p > M/\sigma_y$

DEFLECTION CHECK: $\frac{l}{200} > \delta_{max}$ → $5wl^4/384EI$ (UDL) → $Wl^3/48EI$ (POINT LOAD)

BEAM TYPE (1)



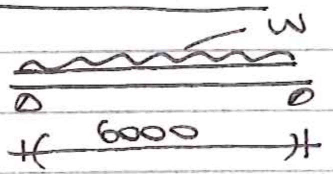
$w\ (ULS) = 5.325 \times 3 = 15.975$ kN/m
$w\ (SLS) = 3.375 \times 3 = 10.125$ kN/m

$M_{MAX} = 15.975 \times 6^2/8 = 71.89$ kNm
$Z_p > 71.89 \times 10^6/355 = 202.5\text{cm}^3$
∴ TRY UB 203 x 102 x 23

(2) CHECK $\delta = \dfrac{5 \times 10.125 \times 6000^4}{384 \times 210 \times 10^3 \times 2105 \times 10^4} = 38.6$ mm $> l/200$

∴ INCREASE TO: **UB 254 x 102 x 22**

BEAM TYPE (2)



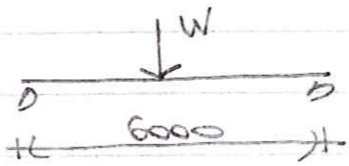
$w\ (ULS) = 7.99$ kN/m
$w\ (SLS) = 5.06$ kN/m

$M_{MAX} = 7.99 \times 6^2/8 = 35.9$ kNm
$Z_p > 35.9 \times 10^6 \times 10^6/355 = 101.25\text{cm}^3$
∴ TRY UB 152 x 89 x 16

CHECK $\delta = \dfrac{5 \times 5.06 \times 6000^4}{384 \times 210 \times 10^3 \times 834 \times 10^4} = 48$ mm $> l/200$

(2) ∴ INCREASE SIZE TO : UB 178 x 102 x 19
(REALISTICALLY TO: **UB254 x 102 x 22** DUE TO CLADDING SELF WEIGHT).

BEAM TYPE (3)



$W\ (ULS) = 5.325 \times 3 \times 6/2 = 47.925$ kN
$W\ (SLS) = 3.375 \times 3 \times 6/2 = 30.375$ kN

$M_{MAX} = 47.925 \times 6/4 = 71.89$ kNm

$Z_p > 71.89 \times 10^6 / 355 = 202.5 \text{ cm}^3$

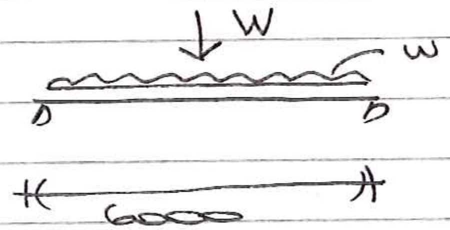
$\therefore$ TRY UB 203x102x23

CHECK $\delta = \dfrac{30.375 \times 10^3 \times 6000^3}{48 \times 210 \times 10^3 \times 2105 \times 10^4} = 31 \text{ mm} > \ell/200$

(2) $\therefore$ INCREASE SIZE TO: **UB 254x102x22**

(SAME SIZE AS SECONDARY TO FACILITATE CONNECTION).

BEAM TYPE (4)



= TYPE (2) + TYPE (3)

$\therefore Z_p > 202.5 + 101.25 = 303.75 \text{ cm}^3$

TRY: UB 254x102x25

CHECK $\delta = \dfrac{30375 \times 6000^3}{48 \times 210 \times 10^3 \times 3415 \times 10^4} + \dfrac{5 \times 10.125 \times 6000^4}{384 \times 210 \times 10^3 \times 3415 \times 10^4}$

$= 42.9 \text{ mm} > \ell/200$

(2)

$\therefore$ **USE UB 254x146x37**

(c) $(1.4 DL + 1.6 LL)$:

LOAD ON COLUMN $= 5.325 \text{ kN/m}^2 \times \text{AREA} \times 6$ FLOORS

$= 5.325 \left[(3+1.5)^2 - (1.5)^2\right] \times 6$

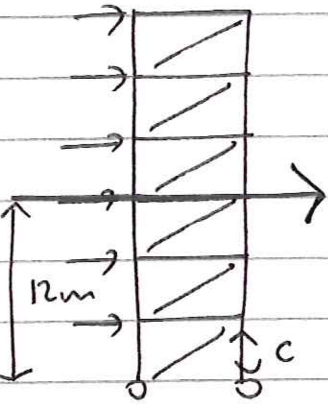
$= 575.1 \text{ kN}$

$(1.2 DL + 1.2 LL + 1.2 WL)$:

(2) $\left[(3+1.5)^2 - (1.5)^2\right] \cdot \left[(1.2 \times 0.375) + (1.2 \times 3)\right] \cdot 6$

$= 437.4 \text{ kN}$

P.T.O.



$$F = 1.2 \times \left(2.25\ kPa \times \frac{15}{2}\right)(6 \times 4) = 486\ kN$$

$$\therefore\ C = \frac{486 \times 4 \times \cancel{8}}{\cancel{8}} = 1944\ kN$$

(2)

∴ TOTAL COMPRESSION IN COLUMN = 1944 + 437.4

= 2381.4 kN

TRY UC 254×254×89

$\lambda = 4000 / 65.5 = 61.07$

$\lambda_0 = \pi \sqrt{210 \times 10^3 / 355} = 76.4$

$\therefore\ \bar{\lambda} = 61.07 / 76.4 = 0.80$

$\chi \approx 0.70$ (FROM CHART)

(2)

$\therefore\ P_{max} = 355 \times 113 \times 10^2 \times 0.7$

$= 2808\ kN > 2381\ kN$ ∴ OK.

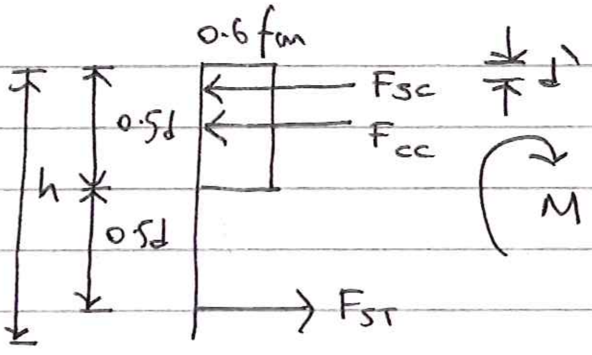
Q1. Examiner's Comment:

This was a popular question that first asked candidates to describe the vertical and horizontal load paths in the multi-storey steel frame building provided in the exam script. Most candidates were able to describe the complete load paths for vertical and horizontal loads and some provided good quality sketches to illustrate their answers. A few candidates lost marks as they failed to note (or sketch) the importance of the braced core in transferring the horizontal loads to ground level.

The second and third part of the question asked students to calculate suitable sizes for typical floor beams and a core column at ground floor level, respectively. Most candidates followed the correct procedure in these calculations, but there were several minor calculation errors along the way. Most candidates failed to realise that there were four different loading / span cases for the floor beams and some incorrectly assumed that the floor slabs were two-way spanning rather than one-way spanning, as specified in the question and shown in the diagram. Many understood that the beams were restrained against lateral torsional buckling. The column design was well executed, although several candidates had to truncate their calculation for lack of time.

Unsurprisingly students that followed a neat and methodological layout incurred the least errors and gained the most marks. Complete and correct solutions were few and far between, but there was a very good generic grasp of what to do.

2a)



$$F_{SC} = A'_s f_y / \gamma_s$$

$$F_{CC} = \frac{bd}{2} \cdot \frac{0.6 f_{cu}}{\gamma_c}$$

$$F_{ST} = A_s f_y / \gamma_s$$

MOMENTS ABOUT F_{ST}:

$$M = \frac{bd}{2} \cdot \frac{0.6 f_{cu}}{\gamma_c}(0.75d) + \frac{A'_s f_y}{\gamma_s}(d - d')$$

$$= \frac{0.225 f_{cu} b d^2}{\gamma_c} + \frac{A'_s f_y}{\gamma_s}(d - d')$$

(2)

$$\therefore A'_s = \frac{M - \left(0.225 f_{cu} b d^2 / \gamma_c\right)}{f_y (d - d') / \gamma_s}$$

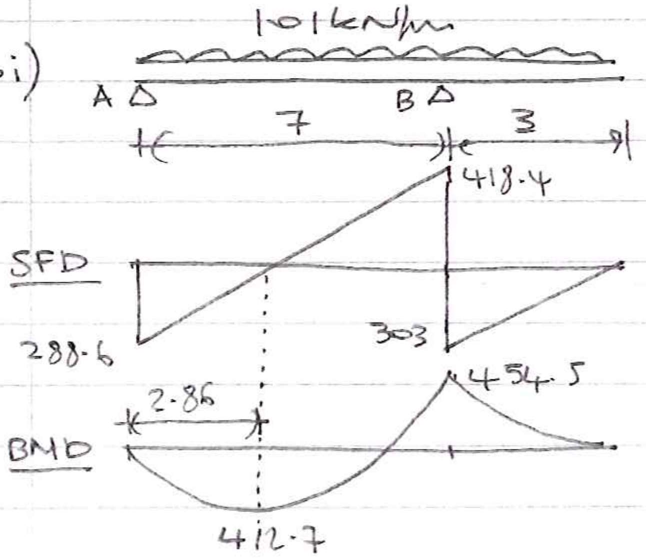
LONGITUDINAL EQUILIBRIUM:

$$\frac{A_s f_y}{\gamma_s} = \frac{0.6 f_{cu}}{\gamma_c} \cdot \frac{bd}{2} + \frac{A'_s f_y}{\gamma_s}$$

(2)

$$\therefore A_s = A'_s + \frac{0.3 f_{cu} b d / \gamma_c}{f_y / \gamma_s}$$

bi)



(3)

(3)

DESIGN LOAD = $(60 \times 1.6) + (0.3 \times 0.5 \times 24 \times 1.4)$

= 101 kN/m

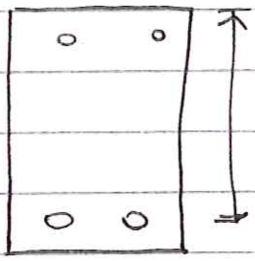
$R_B = 101 \times 10 \times 5/7 = 721.4$ kN

$R_A = 101 \times 10 \times 2/7 = 288.6$ kN

$101 \times 10 = 1010$ kN ✓

CONSIDER CRITICAL SECTION AT SUPPORT B:

bii) $M_u = 0.225 f_{cu} bd^2/\gamma_c = \dfrac{0.225 \times 50 \times 300 \times 439.5^2}{1.5}$



$d' = 40 + 12.5 + 8 = 60.5$

$d = 500 - 40 - 8 - 12.5 = 439.5$ mm

$= 434$ kNm < 454.5 kNm

∴ COMPRESSION STEEL REQUIRED.

$A'_s = \dfrac{(454.5 - 434) \times 10^6}{460 \times (439.5 - 60.5)/1.15} = 135 \text{ mm}^2$

∴ PROVIDE 2T16 (402 mm^2)

$A_s = \dfrac{402 + 0.3 \times 50 \times 300 \times 439.5/1.5}{460/1.15} = 2197.5 \text{ mm}$

(2) ∴ PROVIDE 5T25 (2455 mm^2)

SHEAR REINFORCEMENT:

ASSUME $\cot\theta = 2.5$

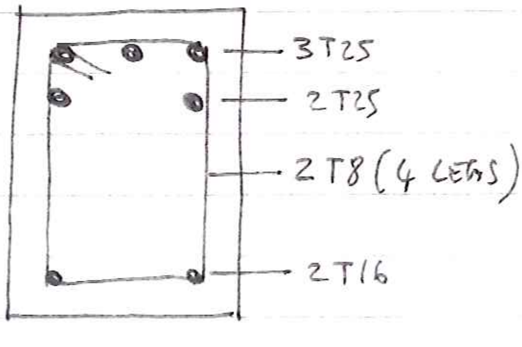
$\therefore A_{sv} = V_{Rdc} s \gamma_s / f_y\, 0.9 d \cot\theta$

$= \dfrac{418.4 \times 10^3 \text{ N} \times 175 \times 1.15}{460(0.9 \times 439.5) 2.5} = 185 \text{ mm}^2$

(2) ∴ PROVIDE 2 STIRRUPS (4 LEGS) OF T8 (201 mm^2)

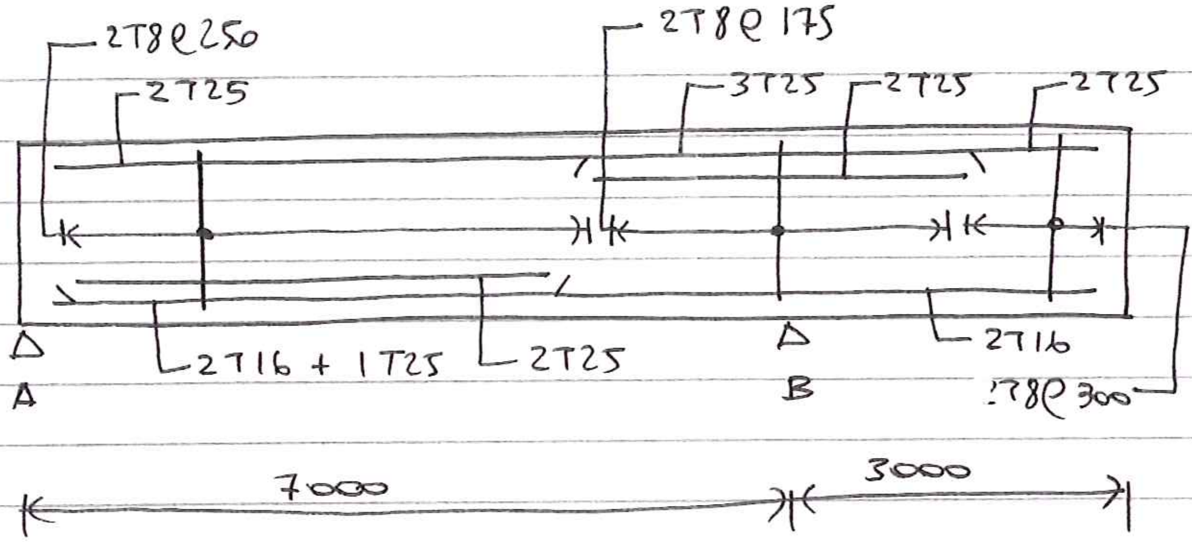
∴ AT CRITICAL SECTION:

(2)



biii)

(4)



Q2. Examiner's Comment:

This was another popular question and involved the derivation of steel reinforcement equations for doubly reinforced concrete beam followed by the design of reinforced concrete beam subjected to a uniformly distributed load.

Nearly all candidates were able to derive the expressions for steel reinforcement required in part (a) by correctly using equilibrium of movements and equilibrium of longitudinal forces.

Most candidates were able to determine the salient values for the shear forces and the bending moments in part b(i), but there were a few relatively minor errors in plotting the respective shear force and bending moment diagrams.

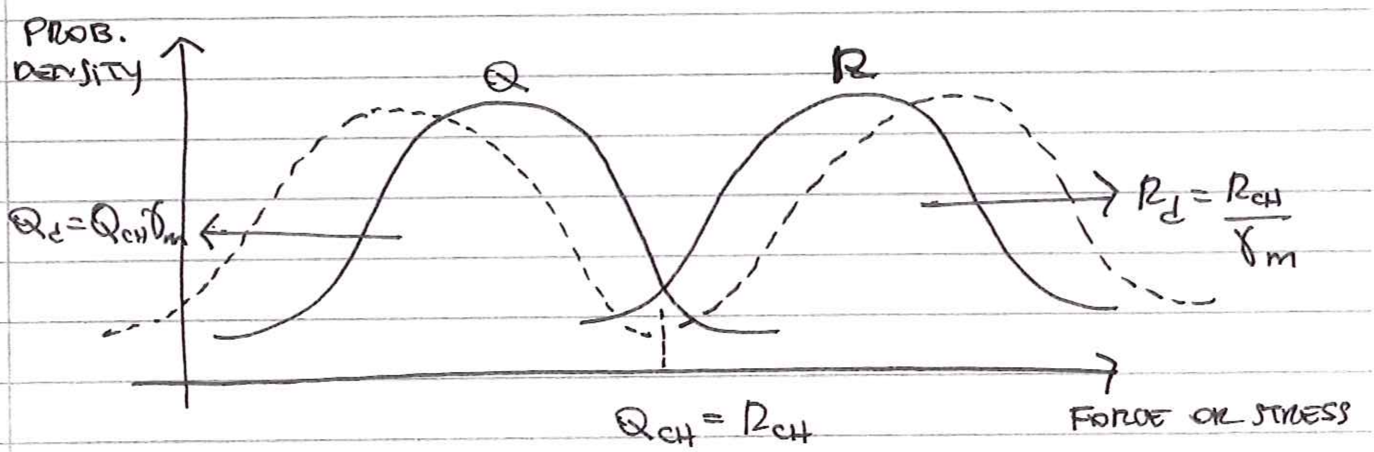
The largest number of errors was in determining whether the reinforced concrete beam required compression steel in part b(ii). These errors seemed to stem either from errors in calculation the effective depth, largely by ignoring the diameter of the stirrups or by incorrectly selecting the cross section at maximum hogging moment (between A and B) rather than the more critical section at support B. The calculations for the determining the longitudinal and shear reinforcement that followed were however generally correct. The sketches for the reinforcement layout required in part b(iii) were good, but only a small number of candidates attempted them.

In general there appeared to be a very good grasp of the general principles, but it appeared that the candidates lacked the practise / exam technique that would have enabled them to complete the question more speedily.

3a) • LIMIT STATE ⇒ LIMIT OF ACCEPTABILITY

• SERVICEABILITY LIMIT STATE → EXCESSIVE DEFORMATIONS, CRACKING, VIBRATIONS ETC.

• ULTIMATE LIMIT STATE ⇒ FAILURE / COLLAPSE.



(2)

Q : LOADS ; Q_{CH} : CHARACTERISTIC LOADS

R : RESISTANCES ; R_{CH} : CHARACTERISTIC RESISTANCES

STRUCTURAL STEEL TYPICALLY FITS A RELATIVELY NARROW (LOW-SCATTER) NORMAL DISTRIBUTION FUNCTION, WHEREAS GLASS FITS A 2-PARAMETER WEIBULL DISTRIBUTION. IT THEREFORE REQUIRES A LARGER γ_m IN ORDER TO REDUCE FAILURES FROM 1/20 TO AN ACCEPTABLE PROBABILITY OF FAILURE (TYPICALLY 1/1000).

(2)

b) i) $P_f = 1 - \exp\left[-kA(\sigma_f - f_{ru})^m\right] \therefore \sigma_f = \left[\frac{-\ln(1-P_f)}{kA}\right]^{1/m} + f_{ru}$

$$\therefore \sigma_f = \left[\frac{-\ln(1-0.001)}{7\times10^{-53}\times 1}\right]^{1/6.5} \times 10^{-6} + f_{ru}$$

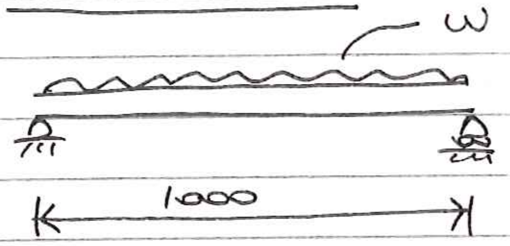
$$\sigma_f = 36.5\ \text{MPa} + f_{ru}.$$

$$\therefore \sigma_f = k_{mod}(36.5) + f_{ru}.$$

	SHORT	MEDIUM	LONG
ANNEALED GLASS	36.5	18.25	10.95
FULLY TOUGHENED GLASS	126.5	108.25	100.95

(4)

b ii) CONSIDER TREAD:



w { L.L. = $4\,kN/m^2 \times 0.3 = 1.2\,kN/m$
D.L. = $25\,kN/m^3 \times 0.01 \times 0.3 = 0.075\,kN/m$ }

DESIGN LOADS: $w_{LT} = 0.075 \times 1.4 = 0.105\,kN/m$

$w_{ST} = 0.075 \times 1.4 + 1.2 \times 1.6 = 2.025\,kN/m$

BENDING MOMENTS: $M_{MAX\,LT} = 0.105 \times 1^2/8 = \underline{\underline{0.013\,kNm}}$

$M_{MAX\,ST} = 2.025 \times 1^2/8 = \underline{\underline{0.253\,kNm}}$

$\sigma = M/Z = 6M/bd^2$

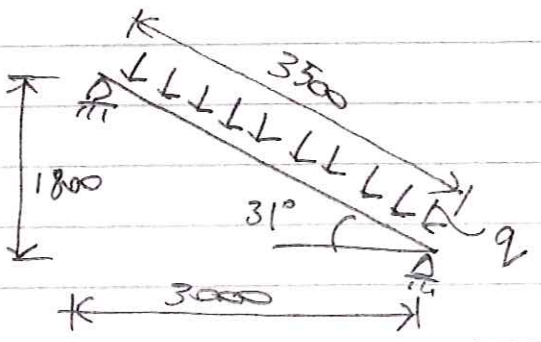
$\therefore \sigma_{LT} > \frac{6M_{LT}}{d^2 b} = \frac{6 \times 0.013 \times 10^6}{10^2 \times 300} = 2.6\,MPa$

$\sigma_{ST} > \frac{6M_{ST}}{d^2 b} = \frac{6 \times 0.253 \times 10^6}{10^2 \times 300} = 50.6\,MPa$

(3)

$\therefore$ <u>FULLY TOUGHENED GLASS REQUIRED</u>

CONSIDER STRINGER:



STRINGER SELF WT = $0.01 \times 1.2 \times 25 \times 1.4$
$= 0.42\,kN/m$

DESIGN LOADS: $q_{LT} = \left(\frac{0.105\,kN/m}{2} + 0.42\,kN/m\right)\cos 31° = 0.4\,kN/m$

$q_{ST} = \left(\frac{2.025\,kN/m}{2} + 0.42\,kN/m\right)\cos 31° = 1.23\,kN/m$

BENDING MOMENTS: $M_{MAX\,LT} = 0.4 \times 3.5^2/8 = \underline{\underline{0.6125\ kNm}}$

$M_{MAX\,ST} = 1.23 \times 3.5^2/8 = \underline{\underline{1.88\ kNm}}$

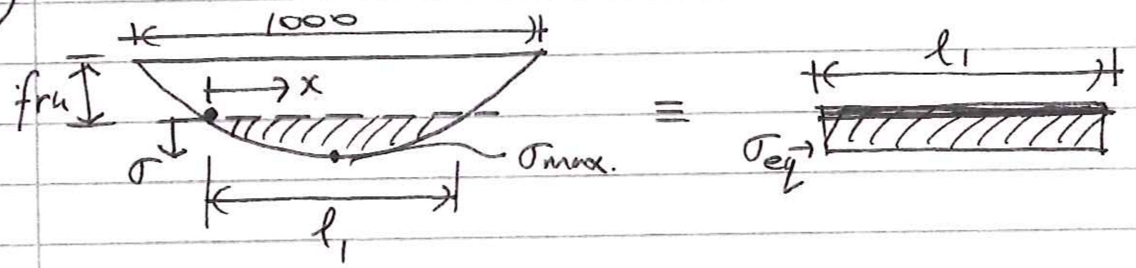
$$\sigma_{LT} > \frac{6M_{MAX\,LT}}{bd^2} = \frac{6 \times 0.6125 \times 10^6}{10 \times (1200 \cos 31)^2} = 0.347\ MPa$$

$$\sigma_{ST} > \frac{6M_{MAX\,ST}}{bd^2} = \frac{6 \times 1.88 \times 10^6}{10 \times (1200 \cos 31)^2} = 1.066\ MPa$$

(3)

∴ ANNEALED GLASS IS SUFFICIENT.

b iii) CONSIDER TREAD AS EXAMPLE:



SINCE σ VARIES ALONG SPAN OF TREAD NOT ALL FLAWS WILL BE SUBJECTED TO σ_{max} AND IT IS POSSIBLE TO CALCULATE AN EQUIVALENT CONSTANT STRESS σ_{eq}:

(3)

$$\sigma_{eq} = \frac{1}{l_1} \int_0^{l_1} \sigma(l)\, dx$$

RE-DESIGNED FOR $(\sigma_{eq} + f_{ru})$ WHERE $(\sigma_{eq} + f_{ru}) < \sigma_m$

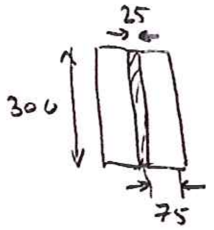
b iv)
- CHECK DEFLECTION AT SLS PARTICULARLY OF TREADS
- CHECK VIBRATIONS (NATURAL FREQUENCY)
- CHECK FOR LATERAL LOADS ON STRINGERS
- REPLACE MONOLITHIC GLASS WITH LAMINATED GLASS FOR ROBUSTNESS.
- CHECK LATERAL TORSIONAL BUCKLING OF STRINGERS.
- CONSIDER WHETHER GLASS CHARACTERISTICS REMAIN FROM EDGE STRENGTH

(3)

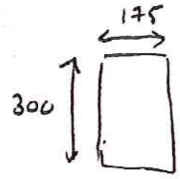
Q3. Examiner's Comment:

This was not a popular question. The candidates were first asked to describe the limit state approach and why the material safety factor for steel and glass differ. The candidates that attempted the question answered this in clear and comprehensive manner. Most candidates were also able to calculate some of the glass strengths required in part b(i). Most marks were lost part b(ii), as most candidates were unable to simplify the staircase into a secondary simply supported beam (the thread) and a primary simply (the stringer) for the purposes of structural analysis. Some candidates identified ways of improving their calculation (part b(iii) and showed a very good understanding of what other design checks would be required in part b(iv).
With the exception of a couple of very good attempts (that in fact showed that this was a relatively simple question), most of the attempts were rushed and incomplete giving the impression that this was a question of last resort for several of those that attempted it.

4)



flitch



timber

flitch steel ; $\sigma_y = 245\ MPa$, $E = 210\ GPa$, $\gamma_s = 1.15$

C24 timber ; $f_{m,k} = 24\ MPa$, $E_{0,mean} = 11\ GPa$, $k_h = 1$

$k_{ls} = 1$, $k_{crit} = 1$, $k_{mod} = 0.7$, $\gamma_m = 1.3$

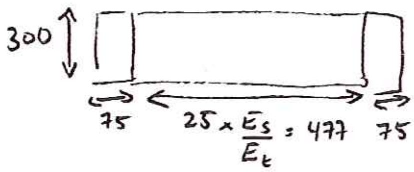
(class 3, short-term)

$$f_{m,d} = k_{mod} \times k_h \times k_{crit} \times k_{ls} f_{m,k} / \gamma_m$$

$$= (0.7 \times 1 \times 1 \times 1 \times 24) / 1.3 = 12.9\ MPa$$

a) <u>flitch beam</u>

- EI_{xx} - transform to timber

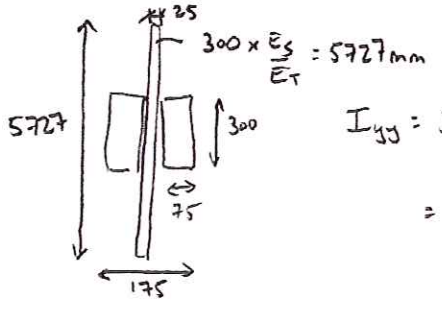


$$I_{xx} = \frac{bd^3}{12} = \frac{(477 + 2 \times 75)(300)^3}{12}$$

$$= 1.411 \times 10^9\ mm^4$$

$$EI_{xx} = 11000 \times 1.411 \times 10^9 = \underline{15.5 \times 10^{12}\ N.mm^2}$$

- EI_{yy} - transform to timber



$$I_{yy} = \frac{300 \times 175^3}{12} + \frac{(5727 - 300)\ 25^3}{12}$$

$$= 141 \times 10^6\ mm^4$$

(3)

$$EI_{yy} = 11000 \times 141 \times 10^6 = \underline{1.55 \times 10^{12}\ N.mm^2}$$

<u>solid timber</u>

(2)

- $EI_{xx} = 11000 \times 175 \times 300^3 / 12 = 4.33 \times 10^{12}\ N.mm^2$
- $EI_{yy} = 11000 \times 300 \times 175^3 / 12 = \underline{1.47 \times 10^{12}\ N.mm^2}$

(1)

$$\frac{EI_{xx}\ \text{flitch}}{EI_{xx}\ \text{timber}} = 3.5 \qquad \frac{EI_{yy}\ \text{flitch}}{EI_{yy}\ \text{timber}} = 1.05$$

↑ flitch much stiffer ↑ much the same

b) check if steel yields before timber fails

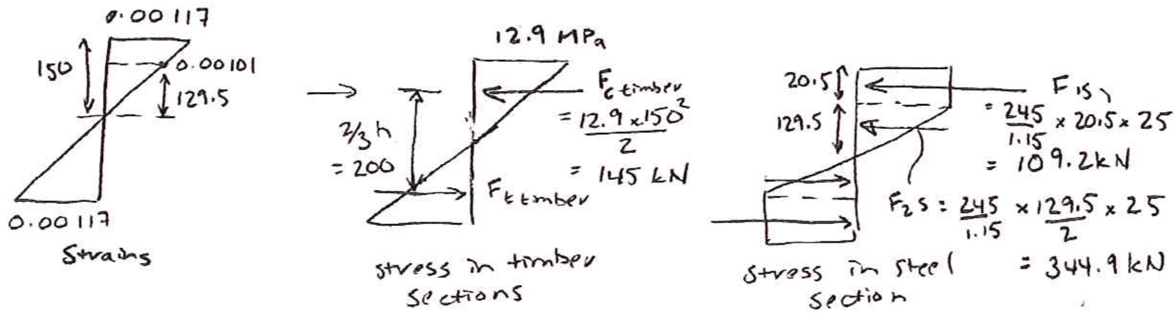
(2)

ε_u timber : $\frac{f_{m,d}}{E} = \frac{12.9}{11000} = 0.00117$

ε_y steel : $\frac{\sigma_y}{E} = \frac{245}{1.15 \times 210000} = 0.00101$ → yields first) } with safety factors

(without material safety factors
ε_u timber = $16.8 / 11000 = 0.00153$
ε_y steel = $245 / 210\,000 = 0.00117$ → yields first)

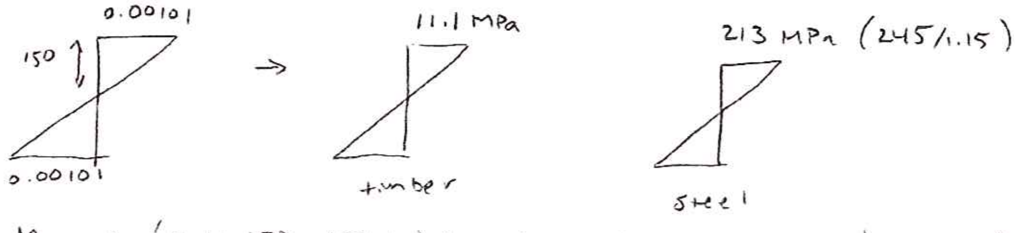
flitch - assume composite action - point at which timber fails (steel already started to yield)



(3)

$$M_{TOT} = 145 \times (200) + 109.2\,(150 - 20.5/2) + 344.9\,(2/3 \times 129.5) \times 2$$
$$= 103.8 \times 10^3\ \text{kN.mm}$$
$$= 103.8\ \text{kN.m}$$

OR since ε_u timber ≈ ε_y steel, consider capacity when steel first yields



$$M_{TOT} : (11.1 \times 150 \times 150/2)(200) + (213 \times 25 \times 150/2)(200)$$
$$= 104.9\ \text{kN.m}$$ (almost the same)

Solid timber

$M_{max} = \frac{f_{m,d} I_{xx}}{y} = \frac{12.9 \times 175 \times 300^3/12}{150} = 33.9 \text{ kN.m}$

(1)

∴ flitch is stronger but would need to check if assumption of perfect bond holds

c) lateral torsional buckling → bookwork

(2) from CTM notes: section on lateral torsional buckling (designing in ductile metal: steel) includes description of lateral-torsional buckling

$M_{crit} = \frac{\pi}{L_{ef}} \sqrt{E I_{yy} G J}$

(4)

much the same for flitch & solid timber

steel will contribute to higher GJ for flitch but probably not a large difference?

As the flitch beam has a higher moment capacity lateral torsional buckling may well dictate. The likelihood of lateral torsional buckling is reduced by reducing the distance between restraints and/or restraining the compression flange. In timber guidance is given on max height to breadth ratios in order to avoid lateral stability problems

(2)

Q4. Examiner's Comment:

The question asked candidates to consider a solid timber beam and a timber-steel composite (flitch) beam. There were a surprisingly large number of errors in calculating the bending stiffness of the composite beam required in part a. There were also several laborious calculations to determine whether the steel would yield before the timber failed (part b). This could be easily determined by considering compatibility of strains. Reassuringly there was very good understanding of underlying principles of lateral torsional buckling (part c) and whether the flitch beam would be effective in preventing this.