(1) (a) Many possible solutions for bracing arrangement. Any sensible arrangement is acceptable – most likely braced end bays or braced core. Most important that arrangement provides resistance in two orthogonal directions at every level. Symmetry is beneficial to avoid torsional effects. State assumption that concrete decking allows lateral loads to transfer through the floorplate by diaphragm action or else provide some other plan bracing solution.

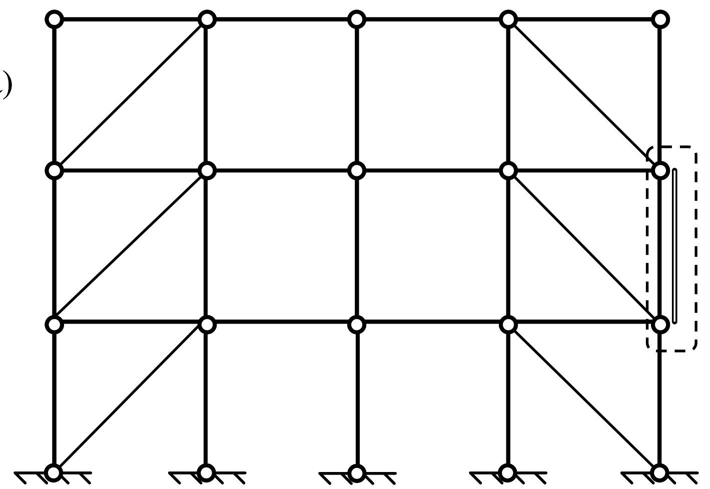
Just one example is shown…

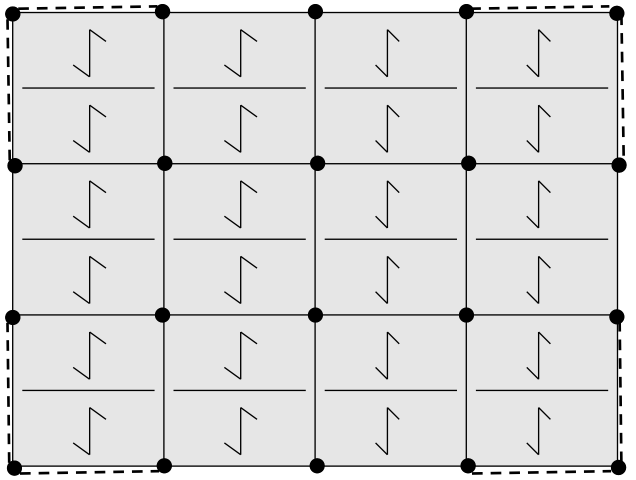
Vertical loading: Concrete decking spans one-way onto steel secondary beams and then to primary beams. Primary beams transfer vertical load to columns and then to foundations.

Horizontal loading: Principally wind load. Wind load is transferred from the cladding to the slab edges and then by diaphragm action of the floor slabs to the bracing. Bracing is tension only to allow slender members to be used efficiently without impeding views. Downside is that only one set of bracing on each side acts at a time.

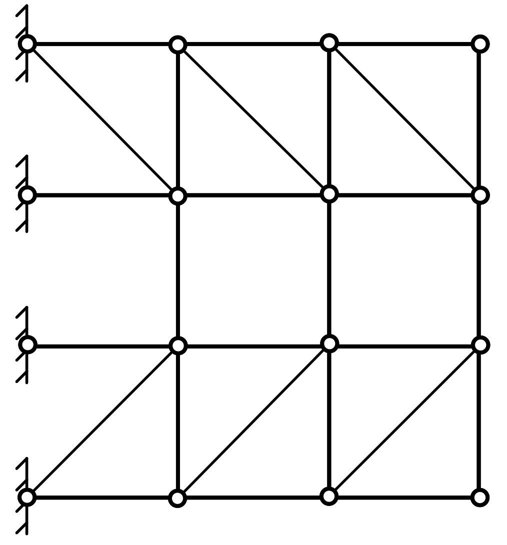
North elevation

(a)





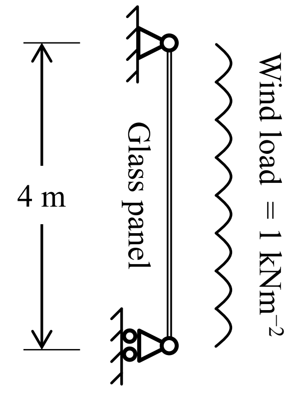
1st floor plan view



East elevation

Slab edge perimeter glazing support arrangement

(b)



(1) (b) (i)

w = 1 x 1.5 =1.5 kN/m^2
M_{Ed} = wL^2/8 = 1.5 x 4^2 / 8 = 3 kNm

From data book: $f_{gd} = f_{gk}$ x k_{mod} x k_A / g_{mA} + f_{rk} / g_{mV}
So from question: f_{gd} = 45 x 0.74 x 1 / 1.8 + 90 / 1.2 = 93.5 N/mm^2

For metre strip:
b = 1000 mm

Max bending stress = M_{Ed} / Z_e
Z_e = bh^2/6

Hence: h = sqrt((6 x 3 x 10^6)/(93.5 x 1000) = 13.9 mm

(ii)
Combining the two data book equations for equivalent glass thickness:

$$h_{eq,\sigma} = \sqrt{\frac{(1-\varpi)\sum_i {h_i}^3 + \varpi\left(\sum_i h_i\right)^3}{\left(h_i + 2\varpi h_{m,i}\right)}}$$

If there are two plies and hi=h1 =h2 as given in the question, and heq,sigma = 13.9 mm from (b), this rearranges to give:

$$h_i = \sqrt{\frac{(1+\varpi) h^{eq,\sigma^2}}{2+6\varpi}} = 8.5\ mm$$

So for equivalence, the total thickness required from the 2-ply = 17.0 mm

Most students who attempted this question performed reasonably well, with 16/27 attempting the question .
a) Load paths – the most common weakness being to omit mention of diaphragm action of the floor plate.
b)
i) Basic flexural glass design – a number of candidates made silly errors in calculating the bending stress.
ii) Laminated glass design – required more thought, skipped by some candidates but often done well by those that attempted it.

(2)
(a)
(i)
from databook, simply supported slab span / 20, so d = 245 mm is acceptable.

Since cover for fire governs, cnom = 35 mm, say maximum bar diameter for slab of 20 mm.

h = 245 + 20/2 +35 = 290 mm

(ii)
Selfweight = 0.29 x 1 x 25 = 7.25 kN/m^2, not negligible.
w = 2.5 x 1.5 + 7.25 x 1.35 = 13.5 kN/m^2

Per metre width of slab:
M_{Ed} = wL^2/8 = 13.5 x 4.898^2 / 8 = 40.6 kNm/m
V_{Ed} = wL/2 = 33.1 kN/m

f_{cd} = f_{cu} / 1.5 = 33.3 N/mm^2
f_{yd} = 500 / 1.15 = 435 N/mm^2

From databook: for singly reinforced, limiting M_u = 0.225 x f_{cd} x b x d^2 = 449.7 kNm/m

M_u>>M_{Ed} so singly reinforced is more than adequate.

$A_{s,req}$ can be determined by solving quadratic, but quicker to simply estimate z = 0.8 x d:
$A_{s,req}$ = M_{Ed} x g_s / (f_{cd} x 0.8 x d) = 40.6 x 10^6 x 1.15 / (33.3 x 0.8 x 245) = 476 mm^2/m

Say 12 mm diameter bars at 200mm c/c … A_s = 565 mm^2/m

Check x: x = A_s x f_{yd} / (0.6 x f_{cd} x b) = 565 x 435 / (0.6 x 33.3 x 1000) = 12.3 mm

So M_{Rd} = A_s x f_{yd} x (d – (x/2)) = 565 x 435 x (245-(12.3/2) = 58.7 kNm/m

iii)
From databook:
k = min(1 + sqrt(200 / d),2) = min(1 + sqrt (200 / 245),2) = 1.9

$Rho_{,l}$ = min(A_s/(bd),0.02) = min (565/(1000 x 245), 0.02) = 0.002

$v_{Rd,c1}$ = (0.018/gc) x k x (100 x $rho_{,l}$ x f_{ck})^(1/3)
= (0.018/1.5) x 1.9 x (100 x 0.002 x 40)^(1/3) = 0.05 N/mm^2/m

$v_{Rd,c2}$ = 0.035 x k^(3/2) x f_{ck}^(1/2) = 0.035 x 1.9^(3/2) x 40^(1/2) = 0.58 N/mm^2/m

$V_{Rd,c}$ = max(vRd,c1;vRd,c2) x b x d = 0.58 x 1000 x 245 = 142.4 kN/m

$V_{Rd,c}$ >>V_{Ed} so no additional shear reinforcement required for the slab.

(2)
(b)

Load on wall per metre is wL/2 where L is the full slab = 13.5 x 5 / 2 = 33.8 kN/m

Slab is simply supported so assume triangular stress distribution over thickness of wall

$f_d = f_k / g_m$ = 5 / 3 = 1.67 N/mm^2/m

Noting triangular stress distribution… for bearing:

<u>Pb = (f_d / 2) x t_{ef} = (1.67/2) x 102.5 = 85.4 kN/m</u>
So ok for bearing

For buckling:
e_x=t/6 = 102.5/6 = 17.1 mm

Assume pin-ended hef = 2400

Can either use datasheet graph noting h_{ef}/t_{ef} = 23 to find beta directly…

or recall formula for e_a = 102.5 x ((1/h_{ef}) x (h_{ef}/t_{ef})^2-0.015) = 21.9 mm
e_t = 0.6 x e_x + e_a = 32.1 mm
e_m = max(e_x,e_t) = 21.9 mm
beta = 1.1 x (1 - (2 x e_m)/t_{ef} = 0.41
<u>P_b = beta x f_d x t_{ef} = 0.41 x 1.67 x 102.5 = 70.1 kN/m</u>
So ok for buckling

Both bearing and buckling capacities are greater than load from slab, so walls are adequate.

Almost all candidates (26/27) attempted this question and did so with reasonable success.
a)
i) Preliminary sizing – most candidates identified the need to check cover requirements but failed to recognize the need to check span/depth ratios in order to ensure that a sensible beam depth was chosen.
ii) Reinforced concrete slab in flexure – some very inefficient approaches were taken to this rather straightforward question. Many 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.
iii) Reinforced concrete slab in shear – most candidates identified the correct equations to use, although a surprising number drew erroneous conclusions from the results.
b) Masonry wall design – most candidates did reasonably well, but many failed to recognize that the slab spanning on one side only meant that the eccentricity of the load on the top of the wall needed to be considered for both buckling and bearing checks.

(3)
(a)
(i)
Loads are design loads i.e. already factored
W = 10 kN/m
P = 30 kN
L = 8 m
Max moment occurs at midspan with point load at mid span: M_{Ed} = wL^2/8 + pL/4 = 140 kNm

From question:
Service class 1, long term loading…. data book gives K_{mod} = 0.7
k_h = 1
Fully restrained so k_{crit} = 1
k_{ls} = 1
C16 so from databook $f_{m,k}$ = 16
g_m= 1.3
From databook: $f_{m,d}$ = k_{mod} x k_h x k_{crit} x k_{ls} x f_{mk} / g_m = 8.62 N/mm^2

y = d/2 = 300 mm
I = bd^3/12 = 4.5 x 10^9 mm^4
Sigma bending = M_{Ed}y/I = 140 x 10^6 x 300 / (4.5 x 10^9) = 9.33 N/mm^2

<u>9.33 < 8.62 N/mm^2, so not adequate for bending</u>

(ii)
Max shear occurs when when point load is 0.6 m from support V_{Ed} = 10 x 8 /2 + (8-0.6) x 30 / 8 = 67.75 kN

Parabolic stress distribution with tau,max = 1.5 x $tau_{,ave}$
$tau_{,Ed}$ = 1.5 x 67.75 x 10^3 / (250 x 600) = 0.68 N/mm^2

From databook f_{vd} = k_{mod} x k_{ls} x f_{vk} / g_m = 0.7 x 1 x 1.8 / 1.3 = 0.97 N/mm^2

<u>0.97 > 0.68 N/mm^2, so adequate for shear</u>

(3)
(b)
(i)
Looks tricky but the approach is identical to a current example paper question…

Conservatively using V_{Ed} from (ii) = 67.75 kN

$$\tau(y) = \frac{SA(y)}{Ib}$$

Assuming elastic (parabolic) distribution of shear stresses, the tension to be transferred equals the integral of the shear stresses below the crack plane at the notch.

Take y +ve downward from neutral axis (at h/2)

$$T = b\int_{h/4}^{h/2} \frac{SA(y)}{Ib} dy$$

$$= \frac{S}{I}\int_{h/4}^{h/2} A(y)dy$$

$$A(y) = \left[\left(\frac{h}{2} - y\right)b\right]\left[y + \frac{\left(\frac{h}{2} - y\right)}{2}\right]$$

$$= b\left[\frac{h}{2} - y\right]\left[\frac{y}{2} + \frac{h}{4}\right]$$

$$= b\left[\frac{h^2}{8} - \frac{y^2}{2}\right]$$

$$\int_{h/4}^{h/2} A(y)dy = b\left[\frac{h^2}{8}y - \frac{1}{6}y^3\right]_{h/4}^{h/2}$$

$$= b\left[\frac{h^2}{8}\left(\frac{h}{2}\right) - \frac{1}{6}\left(\frac{h^3}{2}\right)\right] - b\left[\frac{h^2}{8}\left(\frac{h}{4}\right) - \frac{1}{6}\left(\frac{h^3}{4}\right)\right]$$

$$= bh^3\left[\frac{1}{16} - \frac{1}{48} - \frac{1}{32} + \frac{1}{384}\right]$$

$$= 703\ x\ 10^6 mm\text{^}3$$

$$T = \frac{S}{I}\int_{h/4}^{h/2} A(y)dy = \frac{67.75\ \times 10^3}{4.5 \times 10^9} 703 \times 10^6 = 10.6\ kN$$

(b)
(ii)

Practical solutions include long threaded screws, steel bolts, steel plates, etc. Any sensible means of transferring the force across the crack plane are acceptable. Note that appropriate anchorage above and below the crack plane is required.

Most students who attempted this question did poorly, with 15/27 attempting the question. For some candidates this was their last question attempted and poor time management clearly played a part.
a)
i) Timber beam in flexure - mostly ok, although some students failed to recognize that the the description of the beam as fully laterally restrained meant that k_{crit} could be taken as 1 without calculation.
ii) Timber beam in shear – mostly ok, although a surprising number of candidates failed to recognize that a parabolic elastic shear stress distribution means that the peak stress is 1.5 times the mean.
b) Splitting force at notch – not seriously attempted by most candidates; only done properly by one. At first sight the question is not easy, however it is very, very similar to a current example paper problem! Hence all candidates should have been able to make a good start to the question (thus obtaining a healthy proportion of the marks), even if time pressure meant that it would be challenging to finish for some.

i]

1] Class?

flange : $c/t = \frac{(b_f - t_w)/2}{t_f} = \frac{(300-10)}{2(40)} = 3.625$

$< 9\varepsilon = 9(0.81) = 7.3 \Rightarrow$ Class 1 flange

web : $c/t = \frac{h_w}{t_w} = \frac{800}{10} = 80 < 124\varepsilon = 100 \Rightarrow$ Class 3 web

$\rightarrow$ Class (3)

$$\rightarrow M_{c,Rd} = \frac{W_{el} \cdot f_y}{\gamma_{M0}}$$

small

$$I_{yy} = t_w \frac{(h_w)^3}{12} + 2 b_f \frac{(t_f)^3}{12} + 2 b_f t_f \left(\frac{h_w}{2} + \frac{t_f}{2}\right)^2 = 4.66\,(10^9)\ \text{mm}^4$$

$$W_{el} = \frac{I_{yy}}{h_w/2 + t_f} = 1.06\,(10^7)\ \text{mm}^3$$

$$M_{c,Rd} = M_{el} = 3763\ \text{kNm}$$

2] LTB

$$I_t = J = \sum \frac{bt^3}{3} = \frac{2 b_f t_f^3}{3} + \frac{h_w t_w^3}{3} = 1.31\,(10^7)\ \text{mm}^4$$

$$I_w = C_w = \frac{t_f}{24}(h_w + t_f)^2 b_f^3 = 3.175\,(10^{13})\ \text{mm}^6$$

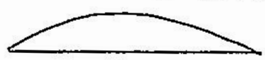
$$I_{zz} = 2\, t_f \frac{(b_f)^3}{12} = 1.80\,(10^8)\ \text{mm}^4$$

$$L_{cr} = 16\,000\ \text{mm}$$

$$M_{cr,0} = \frac{\pi^2 E I_{zz}}{L_{cr}^2} \sqrt{\frac{I_w}{I_{zz}} + \frac{L_{cr}^2}{\pi^2} \frac{G I_t}{E I_{zz}}} = 1383\ \text{kNm}$$

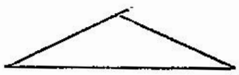
$$M_{cr} = C_1 M_{cr,0}$$

UDL (S.W.)



$C_1 = 1.13$

point load



$C_1 = 1.365$

Conservatively take $C_1 = 1.13$

($C_1 = 1.0$ is also acceptable)

$M_{cr} = (1.13)(1383) = 1563$ kNm

$\bar{\lambda}_{LT} = \sqrt{\dfrac{M_{el}}{M_{cr}}} = 1.55$ buckling curve : $\underline{d}$ ($\alpha = 0.76$)

$\phi_{LT} = 0.5 \left[1 + \alpha(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^2 \right] = 2.22$

$\chi_{LT} = \dfrac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}} = 0.263$

$M_{b,Rd} = \chi_{LT} \cdot M_{el} / \gamma_{M1} = \underline{990 \text{ kNm}}$

3) Load

$A = h_w \cdot t_w + 2 b_f \cdot t_f = 32000 \text{ mm}^2$

$S.W. = (32000)(10^{-6})(78.6) = 2.512$ kN/m

$M_{Ed} = \underbrace{(1.35)(2.512)\dfrac{(16)^2}{8}}_{S.W.} + \underbrace{(1.5)\dfrac{(7.5)}{2}\dfrac{(16)}{4}}_{trolley} + \underbrace{(1.5)\dfrac{P}{2}\dfrac{(16)}{4}}_{load} = 990$ kNm

$\Rightarrow P = 286$ kN

Most candidates (23/27) attempted this question and did poorly. For many candidates this was their last question and poor time management clearly played a part.

a) Steel beam in flexure – most candidates made a start on the question but did not appear to have practiced basic steel beam design and skipped important steps or got lost along the way. Scaffolding the question by subdividing into further sub-sections might have helped some candidates perform better.

b) Buckling comparison – this question required a bit of thought and, while done well by a few, was misunderstood by many.

c) Stiffener checks – like a) this was a relatively straightforward question for those who had practiced, but most candidates were rushing by this point and, having made a start, seemingly ran out of time.

ii) $I_{yy} = t_f \dfrac{(b_f)^3}{12} = 9(10^7)\ \text{mm}^4$

$P_{Euler} = \pi^2 \dfrac{E I_{yy}}{L^2} = 729\ \text{kN} \qquad (L = 16\ \text{m})$

$A = t_f \cdot b_f = 12000\ \text{mm}^2$

$P_y = A \cdot f_y = 4260\ \text{kN}$

$\lambda = \sqrt{\dfrac{P_y}{P_{Euler}}} = 2.42$ buckling curve $\underline{c}$ $\alpha = 0.49$

$\phi = 3.97 \rightarrow \chi = 0.14 \rightarrow \sigma_{cr} = \chi \cdot f_y = 49.9\ \text{MPa}$

$\Rightarrow M_{b,Rd} = \sigma_{cr} \cdot W_{el} = 529\ \text{kNm}$ (53% of the capacity found in (i))

- This value neglects the torsional resistance of the girder and the stabilizing effect of the tension zone.
- This value neglects the variation in bending moment along the length of the girder.

iii) $V_{Ed} = \underbrace{(1.35)(2.512)\dfrac{(16)}{2}}_{\text{S.W.}} + \underbrace{\dfrac{(P + 7.5)}{2}(1.5)}_{\text{trolley + load}}$ (Design shear force)

$= 247\ \text{kN}$

$\dfrac{h_w}{t_w} = 80 > 72\varepsilon \rightarrow$ Shear buckling needs to be checked

$k = 5.34 \rightarrow \tau_{cr} = k \dfrac{\pi^2 E}{12(1-\nu^2)} \left(\dfrac{t_w}{h_w}\right)^2 = 158\ \text{MPa}$

$\lambda_w = 0.76 \sqrt{\dfrac{f_y}{\tau_{cr}}} = 1.138$

$\chi_w = \dfrac{0.83}{\lambda_w} = 0.729$

$V_{b,Rd} = \chi_w \dfrac{h_w \cdot t_w \cdot f_y/\sqrt{3}}{\gamma_{M0}} = 1196\ \text{kN} \gg V_{Ed}$

No stiffening required.