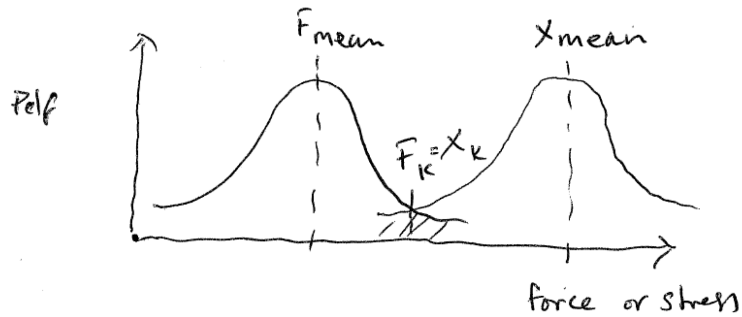
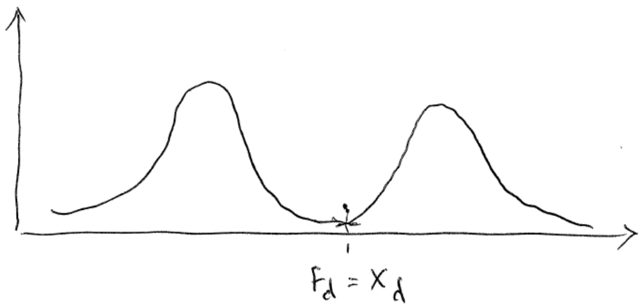
1

(a)	Using a limit state design approach to provide a suitably safe (or reliable) structural design means ensuring that the probability that the effect of an action F on a structure is greater than the value of resistance of that structure X, is acceptably small. This is achieved by ensuring that the design value of the action F_d is not greater than the design resistance X_d, i.e., $\mathbf{F_d \leq X_d}$.

Actions and resistance display considerable variability, therefore the use of mean values of actions and resistances for ultimate limite state design would often be unconservative. In order to overcome this difficulty, tail values of actions and loads are used instead of mean values to characterize the distribution. These characteristic values of actions F_k and resistances X_k are often, but not always, the 5th percentile values.



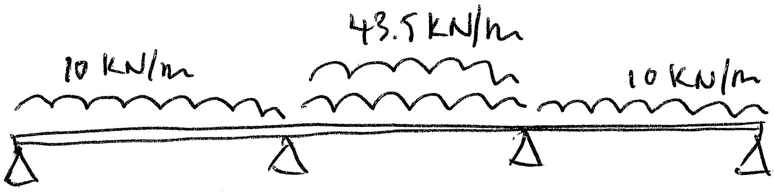
However, societal expectations of reliability of structures are very, very high, meaning that failure probabilities must be very, very low. The probability of failure shown as the overlap between the distributions when $F_k = X_k$ is thus unacceptable. Partial safety factors are thus used to further increase the values of actions γ_f and decrease the values of resistances γ_m such that the overlap (probability of failure) is suitably low (but non-zero!). These resulting values are the design values of actions and resistances for use in limit state calculations.



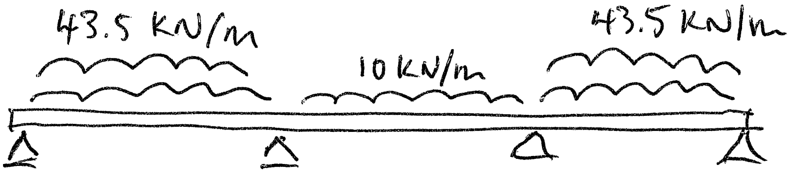
i.e. $F_{mean} < F_k \times \gamma_f = \mathbf{F_d \leq X_d} = 1/\gamma_m \times X_k < X_{mean}$

b)

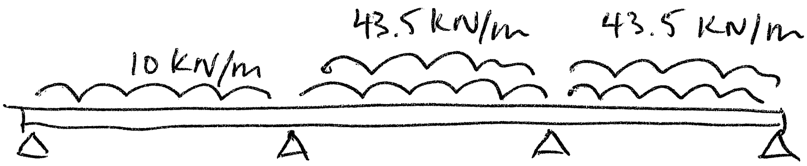
i)


43.5 kN/m
10 kN/m
10 kN/m

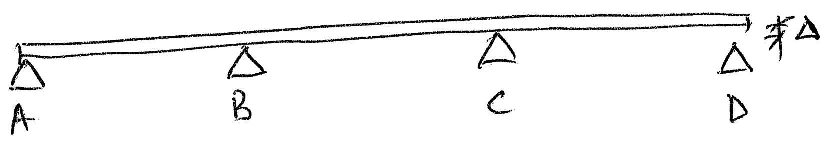
ii)

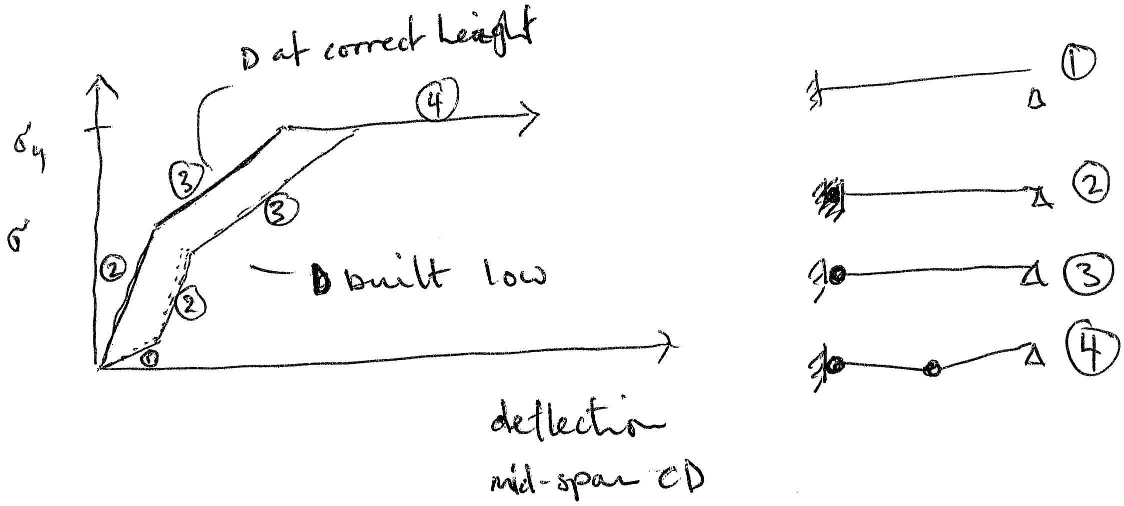

43.5 kN/m
43.5 kN/m
10 kN/m

iii)


43.5 kN/m
43.5 kN/m
10 kN/m

c)





The strength of the beam is unaffected by the support being built too low.

The beam is ductile meaning that failure/yield is brought about by the formation of two plastic ~~three~~ hinges in the span CD, regardless of the vertical position of the support D.

The mid span deflection is greater prior to yield for the case of D built too low because the initial deflections in the out of fit case are those of a cantilever 1, and the formation of the 1st plastic hinge ~~occurs~~ occurs earlier.

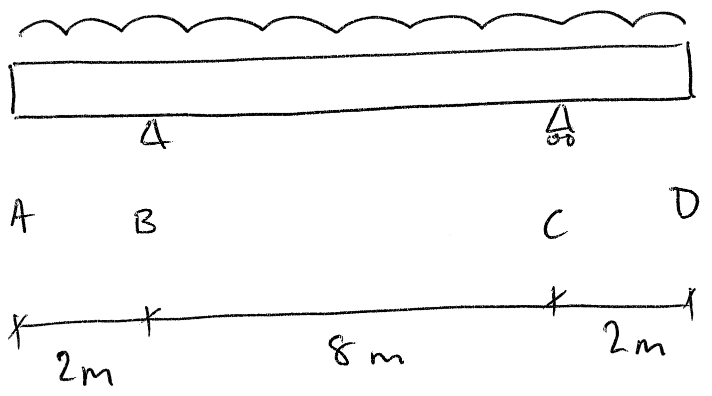
2

a)

DL = 3 kN/m
LL = 24 kN/m

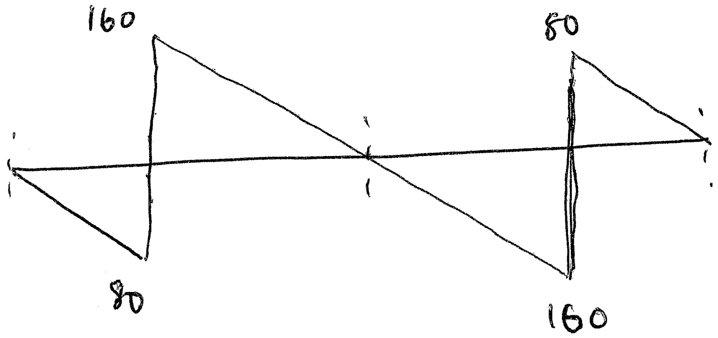
ULS = 1.35 × 3 + 1.5 × 24

= 40 kN/m

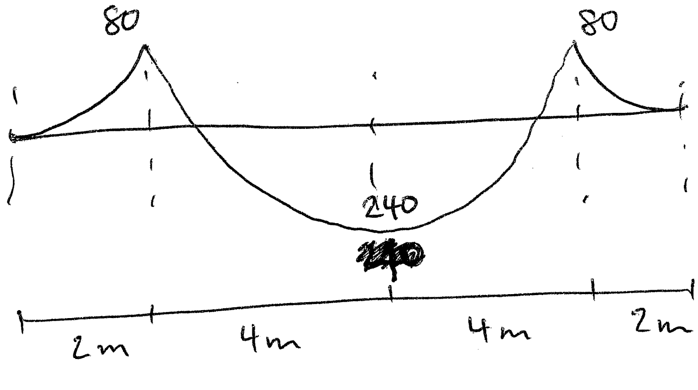


$$R_B = R_C = \frac{40 \times 12}{2} = 240 \text{ kN}$$

Shear
kN



Moment
kNm



Max shear = 160 kN at right of B and left of C

Max hogging = 80 kNm over B and C

Max sagging = 240 kNm at midspan BC

b)

$f_{ck} = 32$ Mpa

$f_{cu} = 40$ MPa

$f_{yk} = 500$ MPa

$\gamma_c = 1.5$

$\gamma_s = 1.15$

$c_{nom} = 45$mm

$b_w = 300$ mm

$h = 600$ mm

for max $\phi = 25$ mm

assume $d = 600 - 45 - \frac{25}{2}$

$= 542$mm

assume $z = 0.8d = 433$ mm

check $M_{Ed} < 0.225 f_{cu} b d^2 / \gamma_c$

240 kNm $\leq$ 529 kNm

so singly reinforced section is adequate.

for sagging mid BC:

$$A_s = \frac{M_{Ed}\,\gamma_s}{f_{yk}\, z} = \frac{240 \times 10^6 \times 1.15}{500 \times 433} = 1275 \text{ mm}^2$$

$\boxed{\text{3no. 25mm } \phi \text{ bars} = 1471 \text{ mm}^2}$

for hogging over supports:

$$A_s = \frac{80 \times 10^6 \times 1.15}{500 \times 433} = 425 \text{mm}^2$$

$\boxed{\text{3 no. 16mm } \phi \text{ bars} = 603 \text{ mm}^2}$

c)

Max shear = 160 kN , note falls in hogging zone

Check whether reinforcement is needed...

$$V_{Rd,c} = \frac{0.18}{\gamma_c} k\,(100 \rho_l f_{ck})^{1/3} b_w d \geq 0.035\, k^{3/2} f_{ck}^{1/2} b_w d$$

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

$$= 1 + \sqrt{200/542} = 1.61$$

$$\rho_l = 603 / (300 \times 542) = 0.0037$$

$$V_{Rd,c} = \frac{0.18}{1.5} \times 1.61 \times (100 \times 0.0037 \times 32)^{1/3} \times 300 \times 542 = 71.6\ \text{kN}$$

$$\geq 0.035 \times 1.61^{3/2} \times 32^{1/2} \times 300 \times 542 = 65.8\ \text{kN}$$

so shear reinforcement required.

Take $\theta = 45^\circ$, $s = 400 \approx 0.75d$

$$A_{sw} = \frac{160 \times 10^3 \times 400 \times 1.15}{500 \times 0.9 \times 542 \times 1} = 302\ \text{mm}^2$$

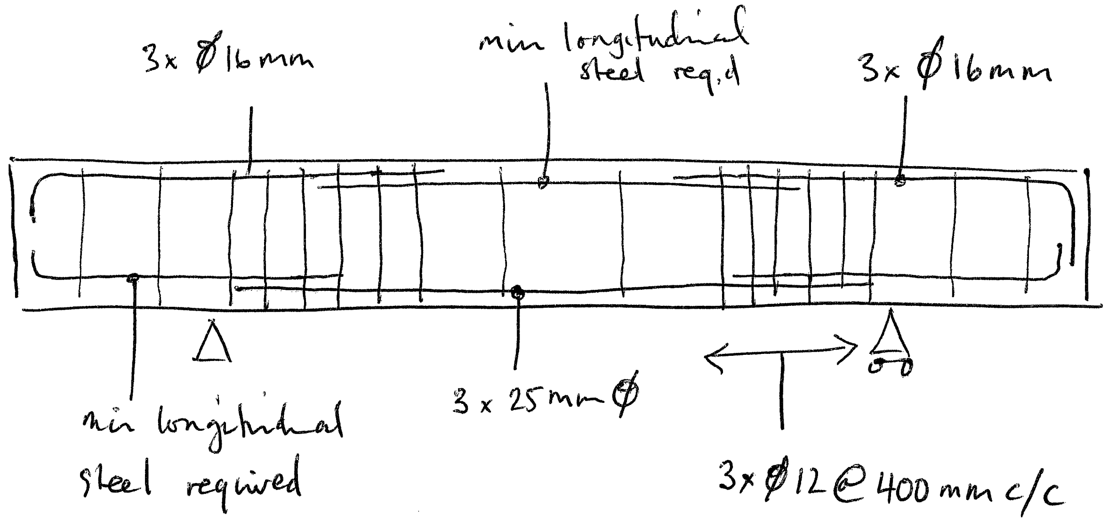
$$\boxed{\text{3 legs 12mm } \emptyset \text{ bar} = 339\ \text{mm}^2}$$

check $f_{c,max} = 0.6 \times (1 - 32/250)\ \frac{32}{1.5} = 11.16$ MPa

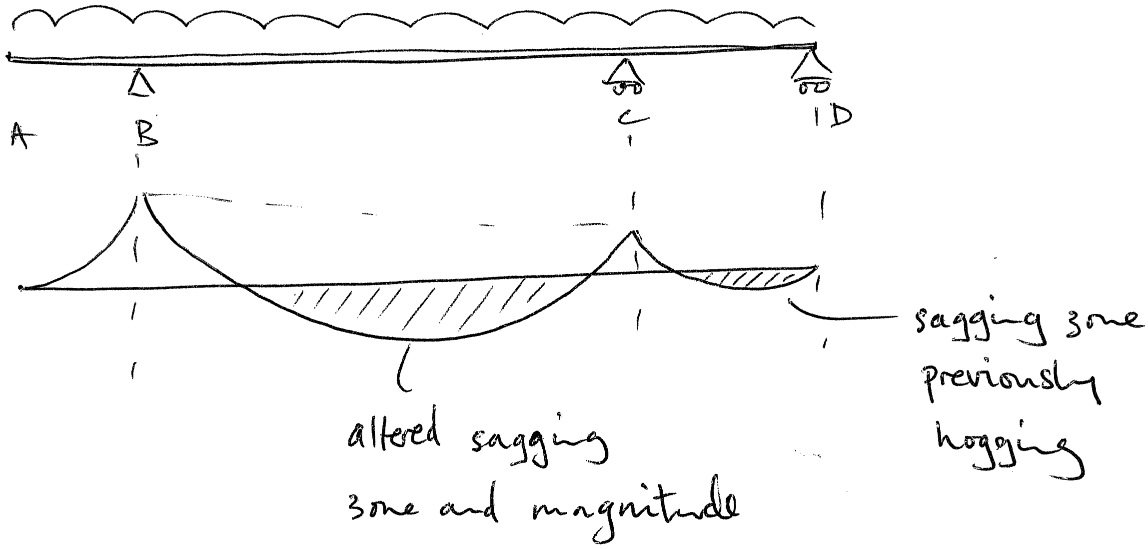
$$V_{Rd,max} = \frac{11.16 \times 300 \times 0.9 \times 542}{2} = 816.7\ \text{kN}$$

so ok.

d)


3x Ø16mm
min longitudinal steel req.d
3x Ø16mm
min longitudinal steel required
3 x 25mm Ø
3x Ø12 @ 400 mm c/c

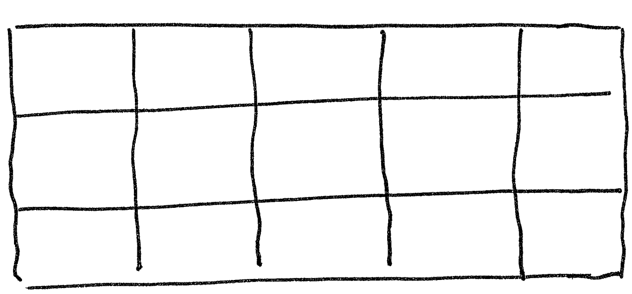
e)

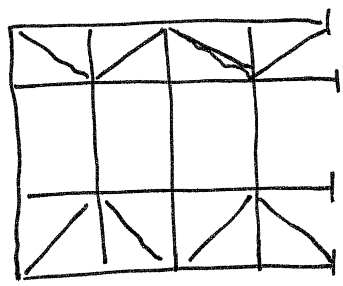


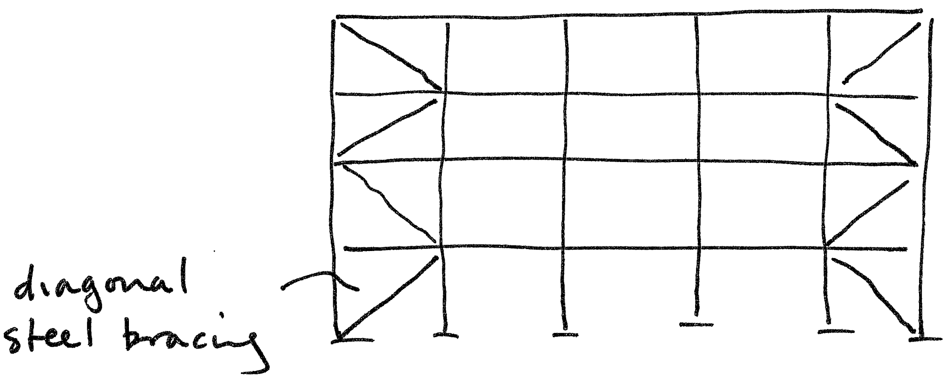
Providing an additional support at D will relieve hogging moments at ~~B~~ C but introduce at least two possible problems. First, the reduced hogging moment at C will tend to increase the sagging moments in span BC beyond those designed initially. Second, CD becomes a propped cantilever introducing sagging moments in a zone that was previously hogging. This means that the design tension reinforcement is on the wrong face! The adequacy of any longitudinal reinforcement in the bottom of span CD would thus need to be checked.

3 a) i)

<u>Steel.</u>

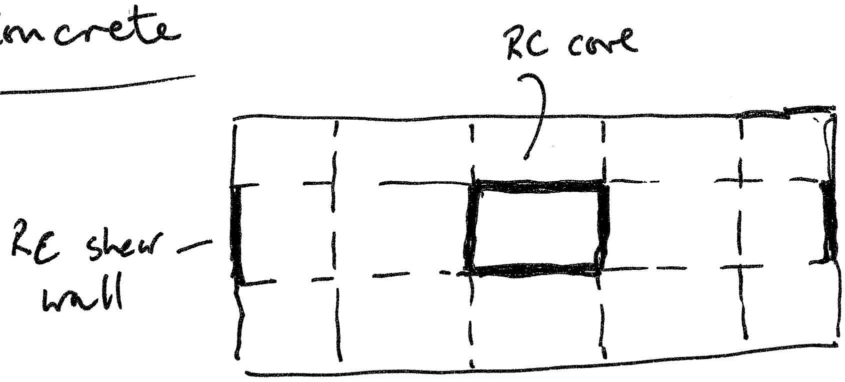


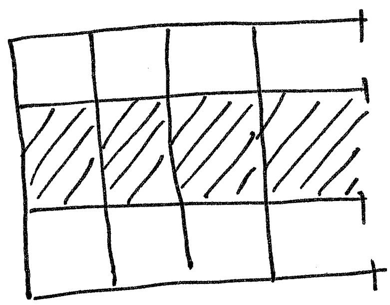


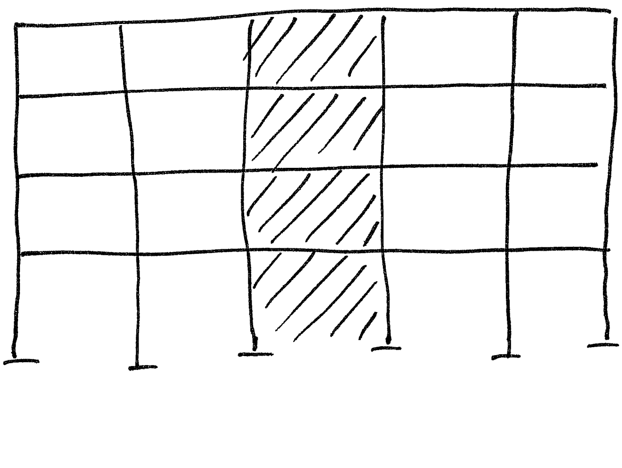


- All connections are pinned
- Concrete slabs at all levels provide diaphragm action

<u>Concrete</u>







- All connections designed as pinned initially
- Concrete flat slab at all levels provides diaphragm action.

ii) Comments to reflect designs, e.g.:
- symmetry
- moment frame or braced
- continuity or not of beams
- use of diaphragm action
- etc.

b) Answers should consider:
- embodied vs operational impacts
- carbon emissions versus sequestration
- resource availability
- life cycle
- end-of-life / reuse
- foundation reduction
- deforestation vs afforestation
- responsible sourcing

etc.

c) Answers should include:

Steel softens and thus requires protection
- intumescent
- boards, etc.

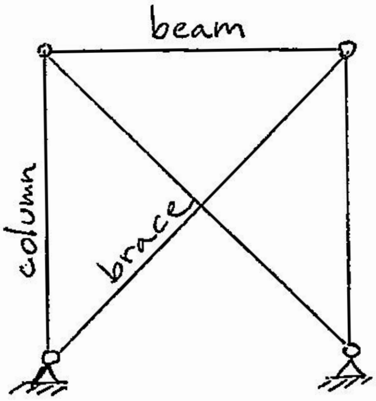
Concrete can be given resistance through provision of cover, but vulnerable to spalling

c) cont...

timber is combustible - different approaches exist

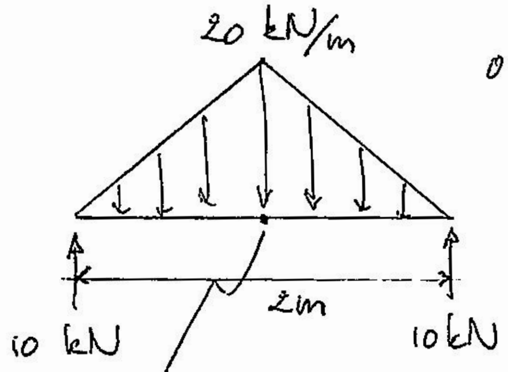
Primarily: · reduced section methods (charring)
· encapsulation

→ Sophisticated answers would include discussion of design for burnout and the challenge of coupled behavior for combustible materials.

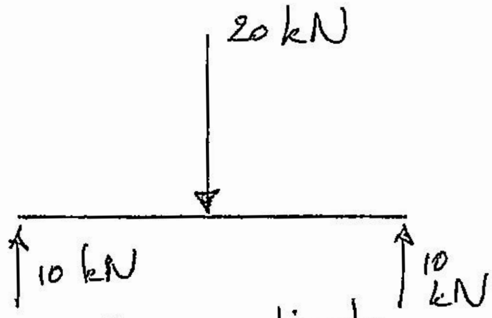


(1) Beam

Load distribution:



or:



$$M = (10)(1) - \frac{(20)}{2}\frac{1}{3} = 6.7 \text{ kNm}$$

Conservatively

$$M = \frac{(20)(2)}{4} = 10 \text{ kNm}$$

$$M_{Ed} = (6.7)\underbrace{(1.5)}_{\text{safety factor LL}} = 10 \text{ kNm}$$

$W_{pl} \cdot f_y > 10$ kNm $\Rightarrow W_{pl} > 28.2\,(10^3)$ mm^3

Choose : SHS $80 \times 80 \times 3.6$ mass : 8.53 kg/m

$\rightarrow M_{sw} = (0.05)(1.35) = 0.06$ kNm

$\hookrightarrow W_{pl} = 31.0\,(10^3)$ mm^3

$M_{c,Rd} = W_{pl} \cdot f_y / \gamma_{M0} = 31\,(10^3)(355)/1.0 = 11$ kNm $> M_{Ed} + M_{sw} = 10.06$ kNm

Ok

Class?

$\frac{c}{t} = 19.2 < 72\varepsilon = 58 \Rightarrow$ Class (1)

(databook) (datasheet)

$\rightarrow$ Using W_{pl} is justified

LTB? Not an issue for SHS

Shear $V_{Ed} = (10)(1.5) = 15$ kN

$$V_{pl,Rd} = \frac{f_y}{\sqrt{3}} A_v / \gamma_{M0} = \frac{(355)}{\sqrt{3}} \underset{\text{2 webs}}{(2)} \underbrace{[80 - 2(3.6)]}_{h_w} \underset{t_w}{(3.6)} = 107 \text{ kN} \gg V_{Ed}$$

$V_{Ed} < \frac{V_{pl,Rd}}{2} \rightarrow$ No reduction for V-M interaction

$\frac{c}{t} < 72\varepsilon$ (see above) $\rightarrow$ no shear buckling

2 Column

$$N_{Ed} = (20\text{ kN})(1.5) + \underbrace{(2+2+2\sqrt{2})(0.0835)(1.35)}_{\text{self-weight}}$$

$$= 30\text{ kN} + 0.7\text{ kN}$$

$$= 30.7\text{ kN}$$

Class ? $\frac{c}{t} = 19.2 < 33\varepsilon = 26.7 \rightarrow$ Class 1

$$N_{Euler} = \pi^2 \frac{EI}{L^2} = \pi^2 \frac{(200\,000)(105)(10^4)}{(2000)^2} = 518\text{ kN}$$

$$\bar{\lambda} = \sqrt{\frac{A.f_y}{N_{Euler}}} = \sqrt{\frac{(10.9)(10^2)(355)}{(518)(10^3)}} = 0.86$$

buckling curve a : $\alpha = 0.21$

$$\phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] = 0.94$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} = 0.765$$

$$N_{b,Rd} = \chi A f_y / \gamma_{M1} = 296\text{ kN} > N_{Ed} \quad \text{Ok}$$

3 Brace

$$F_{lat,Ed} = (0.02)(80\text{ kN})(1.5)/\underset{\text{2 frames}}{2} = 1.2\text{ kN}$$

$$N_{brace} = (\sqrt{2})(1.2) = 1.7\text{ kN}$$

Gross section yielding : $A f_y / \gamma_{M0} = 387\text{ kN} \gg 1.7\text{ kN}$

Net section fracture: $0.9 A_{net} \cdot \frac{f_u}{\gamma_{M2}} = 0.9 \frac{A f_u}{\gamma_{M2}} = 481\text{ kN} \gg 1.7\text{ kN}$

Ok