PART IIB 2010 4D7 - Concrete + Masonry Structures DR JM LEES
Q1 (a) 2009 / 2010 DR C MIDDLETON

CRIB

(i) Flooding

Causes & Damage – constriction to river flow caused by piers across the river results in an accelerated flow regime. This "rush" of water can cause scouring of the river bed + supporting material under + around the foundations of the piers + abutments. In effect this gouging out of the foundations undermines the bridge & can lead to collapse e.g. Cumbria floods in Dec 2009.

Damage may not be evident at first, nor immediately visible due to the torrent of flood water or depth of water obscuring sight of the. The only evidence maybe settling or dropping of one support as the support weakens. This might then be visible in the form on cracks on the deck or in the piers. Often looking along the line of the parapets shows that some settling has occurred.
The high lateral forces may also result in some transverse (sideways) movement of the bridge, in particular if the flood was sufficient to over-top the deck + fully submerge the bridge. The deck slab might be pushed away laterally or just shifted somewhat on its bearings.

Long term durability Inundation by water should not lead to any long term durability problems. Provided the bridge does not move + foundations are not undermined it should not need refurbishment.

Remedial Measures
- Place Large protective stones around piers.
- Gabions around piers
- Pile pier foundations down to bedrock + pin (bolt) foundation to it.
- If deck slabs move on bearings, tie them all together + also to the abutments.
- Might use open form of parapet rather than solid parapet to reduce lateral forces.
- Flood mitigation measures elsewhere.
- Increase opening of bridge (add spans, remove piers + replace with longer span deck)

NDT hydrographic survey of river bed.

(II) Fire

Causes + damage - 2 types of damage. (I) the actual impact damage of the truck hitting the pier + (II) the damage to the material integrity of the concrete + reinforcing steel exposed to the high temperatures. The actual severity of the damage will depend on (I) max temp reached

(II) duration of exposure

(III) cement type, W/C ratio, cement content, aggregate type, cover.

Visible changes occur in concrete at certain temperatures (see lecture notes).

Long term durability

- concrete strength ↓ progressively with time if $T > 400°C$.
- May get delamination cracks, eventually spalling off + exposing r/f to corrosion.
- r/f strength + ductility may be altered.

Remedial measures

Such accidents are extremely rare although several are recorded in the literature (Belgium, California).

The cost of protecting concrete bridge soffits to protect from fire would be prohibitive. Very little could really be done other than perhaps barriers around piers to reduce risk of trucks crashing into them.

Also need to ensure impact on piers doesn't damage bearing or move deck slab.

Impact damage can usually be repaired relatively easily using patch repair techniques.

NDT Petrographic analyses of concrete

R/f samples cut out + tested for strength + ductility.

(3)

ANSWER TO EXAM 2009-10 Q1 (a)(ii)

Fire damage: (refer to lecture notes Handout 4, Section 6.5, p70)

6.5 Fire Damage to Concrete Structures

Severity of damage is dependent on the maximum temperature reached and the duration of exposure of concrete to that maximum temperature. Depends on cement type, W/C ratio, cement content, aggregate type and depth of cover to reinforcement. Visible changes occur to concrete at certain temperatures which also can be used to estimate, within certain limits, the temperature reached by the structural members. At around 300°C to 400°C there is a noticeable colour change to pink or pale red. This pink colour may disappear if the temperature reached was very high but in such a case, the concrete would be friable. This colour change can sometimes fade with time. Investigators at fires observe the melting points of other materials at the scene to estimate the temperature reached.

Figure 6.13 Facia of fire damaged bridge

6.5.1 Effect of High Temperature on Concrete

As the temperature rises, the uncombined (surplus) water in the concrete will be driven off from the surface layers and some shrinkage cracking will occur. Up to about 100°C there will be no significant loss of chemically bound water even with a prolonged exposure to this temperature. As the temperature rises above 100°C there is a gradual loss of chemically bound water from the calcium silicate hydrates. The actual loss depends on time and temperature. With this loss of chemically combined water there is a drop in the strength of the concrete corresponding to the amount of water lost. However once the concrete cools there will be no further reduction in strength.

If the concrete reaches grater than 400°C the calcium silicates start to decompose into quicklime and silica. This is an irreversible process and leads to a progressive loss of strength with time.

When the concrete cools, the quicklime (CaO) will absorb moisture, converting to slaked lime ($CaOH$). This will result in disintegration of the heat affected areas of concrete. The fire will introduce high temperature gradients in the structural members and, as a result, the hot surface layers tend to separate and spall from the cooler interior ones.

With major fires it is usually found that some of the reinforcement in slab structures will have buckled and requires replacing. Samples can be taken of the reinforcement to check the yield strength and ductility. Damage to reinforcement in slabs is usually found to be much greater than in beams, probably due to the greater depth of cover normally found in beams.

(iii) Earthquake.

<u>Causes + damage</u>

Earthquakes can impose large lateral + vertical accelerations on a bridge deck.

- result in high compressive forces in columns which can lead to high bursting stresses. If inadequate links, confinement reo, columns can literally explode.
- also large lateral loads can induce shear failures in columns + piers if inadequately designed.
- can get deck slabs moving longitudinally + dropping of bearings or punching through abutment walls.

Long term durability – sections may yield + locally crush. resulting in ↓ strength + ↑ susceptibility to ingress of deterioration inducing chemicals.
May get substantial cracking which while not critical at ULS may reduce durability.

<u>Remedial measures</u>

Columns can be retrofitted to provide confinement (California). Similarly shear r/f can be added (can be very expensive)

Structural elements can be tied together to avoid progressive collapse.

Some structures have sacrificial elements designed to crush or fail + absorb energy whilst preserving rest of structure.

Could retrofit dampers / energy absorbing bearings – expensive.

Best defence is going to be good detailing of original design.

<u>NDT</u> – ~~Ult~~ Ultrasound for internal cracking.
– checking of steel in critical regions to ensure no rupture.

(v) Spalling

Causes • damage - corrosion of bottom r/f. Corrosive products are expansive - increases tensile stresses until concrete cover sections spall off.

- Need to identify fundamental cause
  it maybe - leaking from surface due to cracks
  - splashing of water from underneath
  - nearby joint failure allowing water to seep along soffit.
  - poor cover or original construction.

Long term durability - likely to progressively deteriorate unless source of problem/water is removed.
- this will result in ↑ rate of deterioration.

Remedial - Fix source of water / deleterious material.
- Remove any contaminated concrete
- Remove + replace badly corroded bars.
- could coat bars but risk problems at join to old concrete.
- Cathodic Protection (expensive)
If chlorides or carbonation either remove or chemically correct.
- Typically use patch repairs - possibly special epoxy mix.

NDT all corrosion related ones possible.
i.e. covermeter, chlorides, phenolphthalein, resistivity, half cell, thermographic.

Q1(b)

Structural failure is very rare, typical Pf's ~ 10^{-6}.
These rare events are assumed to be modelled mathematically by the shape of the tails of the probability distribution curves. The likelihood or probability of such events occurring is determined from the statistical properties of these tails. The validity of this approach can be questioned since there is often little or no data to support the mathetical models in these extreme regions at say, 5 or 6 standard deviations from the mean corresponding to probabilities of failure ~ 10^{-7} or less. An example is that of r/f steel. We specify a characteristic strength of 460MPa & typically design for ~ 400MPa yet actual r/f bar tends to have a characteristic strength of ~ 480 - 490 MPa & 460MPa seems to be a proof load value below which no bars are found. If the actual data was modelled in a reliability analysis it would indicate that a normally proportioned r.c. member would be almost impossible to fail under, for example, flexural loading.
It is important to remember that most recorded structural failures result from influences/load effects/causes that are not usually considered in conventional analysis. e.g. most failures of bridges occur due to scour around abutments or impact from ships.

Comment on partial factors for
γ_m - material strength, geometric variability
γ_{f2} - loading
γ_{f3} - uncertainty in model.

Supposedly derived by calibration & probabilistic study but in practise derived by matching to results from existing codes.

Assessor's comment:

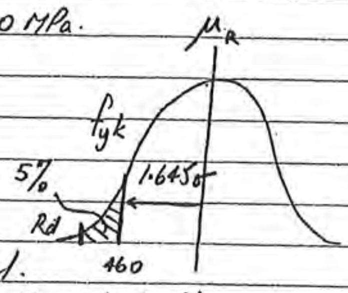
Candidates did well when discussing a specific phenomenon and associated remedial measures and test techniques e.g. steel corrosion but did less well when they had to think more laterally about a number of interconnected implications due to an extreme event such as an earthquake.

2010. Q2 (a) bookwork

Resistance (strength), R.

(b) (i) $f_{yk} = 460$ MPa. (5% level) $R_d = f_{yd} = \frac{f_{yk}}{\gamma_{ms}} = \frac{460}{1.15} = 400$ MPa.

$CoV = \frac{\sigma}{\mu} = 10\%$.



Find mean value: $z = 1.645$ at 5% level. (from table in Maths data book)

$$f_{yk} + 1.645\sigma = \mu_R \quad \text{where} \quad \frac{\sigma}{\mu_R} = 0.10.$$

$$\therefore 460 + 1.645(0.1\mu_R) = \mu_R.$$

$$\mu_R(1 - 0.1645) = 460$$

$$\therefore \mu_R = \frac{460}{0.84} = 550.57 \simeq 551 \text{ MPa}.$$

$$\therefore \sigma_R = 0.1\mu_R = 55.1 \text{ MPa}. \simeq 55 \text{ MPa}$$

Find probability that bar strength is less than $f_{yd} = 400$ MPa.

$$z = \frac{\mu_R - x_R}{\sigma_R} = \frac{551 - 400}{55} = 2.75$$

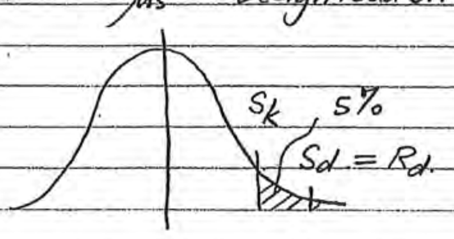
From maths data book for $z = 2.75$ $A = 0.9970$.

$$\Rightarrow P(x_R \leq 400\text{MPa}) = 1 - 0.9970 = 0.003 \quad (3 \times 10^{-3})$$

(ii) Design value of strength of bars = 400 MPa.

$\phi_{bar} = 16$mm $A_{bar} = \frac{\pi}{4} \times 16^2 = 201.1 \text{mm}^2$

Design load on bars, $S_d = f_{yd} \times A_{bar} = 201.1 \times 400 \times 10^{-3} = 80.4$ k



Load Normal with $\sigma_s = 10$ kN.

$S_d = \gamma_f . S_k$ Assume $\gamma_f = 1.4$ for DL.

$$\therefore S_k = \frac{S_d}{\gamma_f} = \frac{80.4}{1.4} = 57.4 \text{ kN}$$

$$\therefore \mu_s = S_k - 1.645 \times \sigma_s = 57.4 - 1.645 \times 10$$
$$= 40.95 = 41 \text{ kN}$$

Reliability $\beta = \frac{\mu_R - \mu_S}{\sigma_{R-S}}$

$\mu_R = 551 \times \underbrace{201.1}_{A_{bar}} \times 10^{-3} = 110.8$ kN

$\sigma_R = 55 \times 201.1 \times 10^{-3} = 11.1$ kN.

$$= \frac{110.8 - 41}{\sqrt{11.1^2 + 10^2}} = \underline{4.67}.$$

$\therefore$ In statistics 'databook' for $u = 4.67$ $\phi(u) = 0.9^5 8494$

$$P_f = 1 - \phi(u) = 1 - 0.9^5 8494 = \underline{1.51 \times 10^{-6}}$$

(iii) Modified strength

$\mu_R' = 500$ MPa $(100.6$ kN$)$ $\quad f_{yk} = 500 - 1.645 \times 23 = 462$ MPa.

$\sigma_R' = 23$ MPa $(4.63$ kN$)$

> f_{yk} specified of 460 MPa
> so bars OK

Bars supplied comply with specification.

Modified loads $\quad$ 20% ↑ in μ_S + σ_S over part (ii)

$\therefore \mu_S' = 1.2 \times 41 = 49.2$ kN

$\sigma_S' = 1.2 \times 10 = 12$ kN.

$$\therefore \beta = \frac{\mu_R' - \mu_S'}{\sqrt{\sigma_R'^2 + \sigma_S'^2}} = \frac{100.6 - 49.2}{\sqrt{4.63^2 + 12^2}} = \frac{51.4}{12.9} = \underline{4.0}$$

$\therefore$ From statistical data book for $\beta = 4$ $\quad$ i.e. $u = 4$ $\phi(u) = .9^4 6833$

$$\therefore P_f = 1 - \phi(u) = 1 - 0.9^4 6833 = \underline{3.2 \times 10^{-5}}$$

(iv) Target $\beta_t = 3.5$ $\quad \sigma_S = 10$ kN $\quad \sigma_R = 11.1$ kN

$\mu_R = 110.8$ kN

$$\therefore \beta_t = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} = \frac{110.8 - \mu_S}{\sqrt{11.1^2 + 10^2}} = \frac{110.8 - \mu_S}{14.94} = 3.5$$

$$\therefore \mu_S = 110.8 - 3.5 \times 14.94 = \underline{58.5 \text{ kN}}$$

∴ Characteristic load $S_k = \mu_s + 1.645 \times \sigma_s = 58.5 + 1.645 \times 10 = 74.95$

i.e. $S_k = 75$ kN

Design load $S_d = \gamma_f \times S_k = 1.4 \times 75 = 105$ kN

since γ_f for dead load is taken as 1.4.

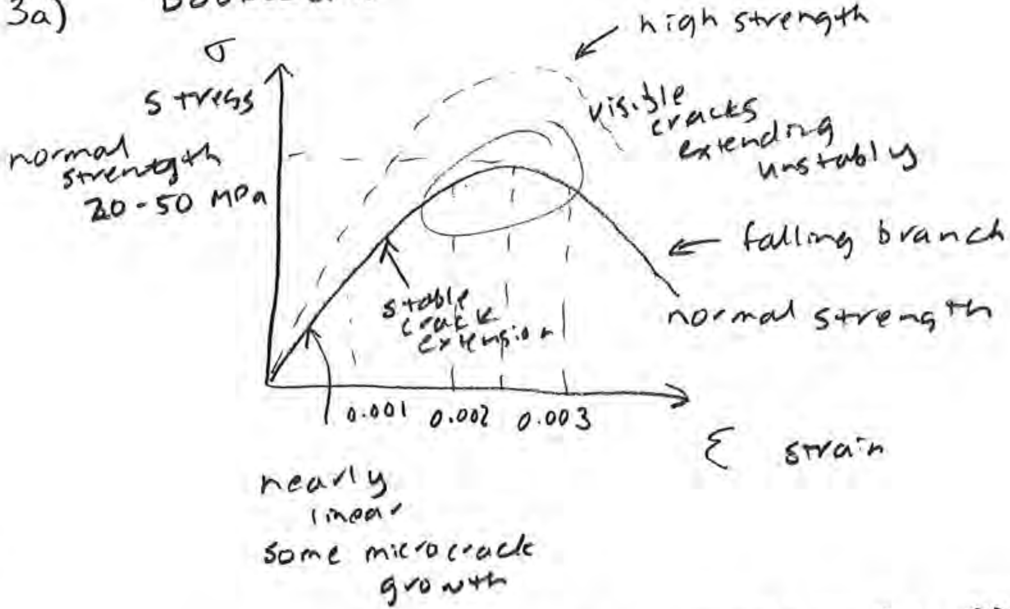
$P_f(\beta = 3.5) = 1 - 0.9^3 7674$
$= 2.3 \times 10^{-4}$

from tables $u = 3.5$
$\phi(u) = \cdot 9^3 7674$

Assessor's comment:

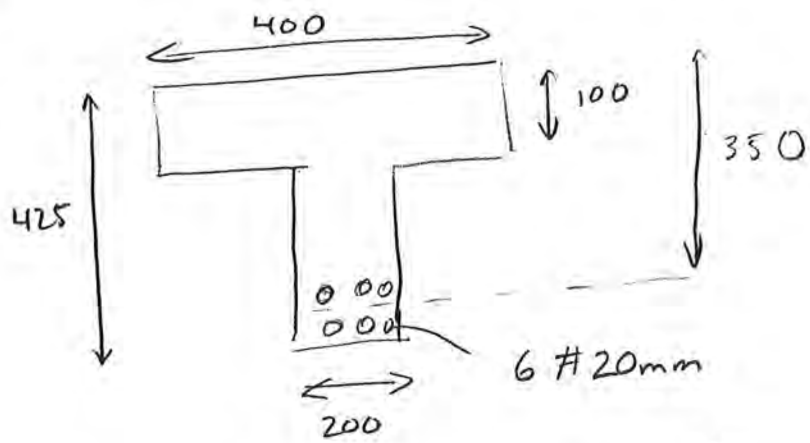
A popular question answered by all but one student. Candidates displayed a good understanding of the reliability calculations required and did very well.

3a) bookwork



· high strength concrete tends to be stiffer and more brittle

b)



$E_s = 210$ GPa , $f_{yd} = 430$ MPa
$E_c = 30$ GPa , $f_{cd} = 30$ MPa

i) $A_s = 6 \times \pi \frac{20^2}{4} = 1885\,mm^2$

$F_s = A_s f_{yd} = 1885 \times 430 = 810531\,N$

· assume n.a. is in flange

$\rightarrow$ max $F_{cf} = 0.6 f_{cd} \times 100 \times 400$
$= 0.6 \times 30 \times 100 \times 400$
$= 720000\,N < 810531\,N$

$\therefore$ n.a is in web

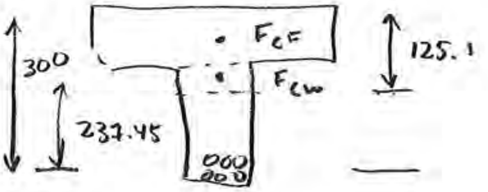
3 b) i)

$F_s = F_{cF} + F_{cw}$

$F_{cw} = F_s - F_{cF} = 810531 - 720000 = 90531$

$F_{cw} = 200 \times 0.6 f_{cd}\, x = 90531$

$\therefore\ x = 25.1\,mm$



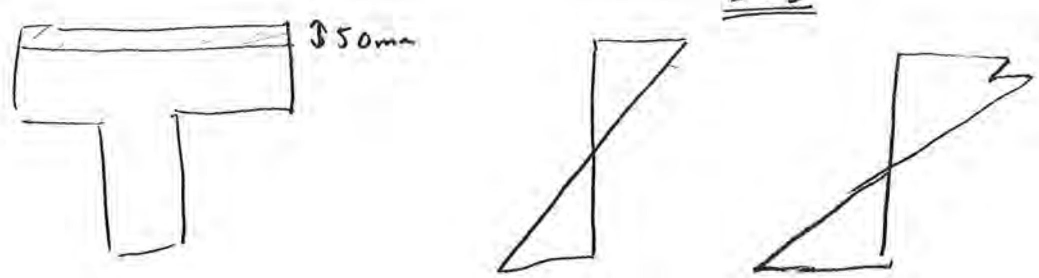
Take moments about centroid of steel

$F_{cF} \cdot 300 + F_{cw} \times 237.43 = M_u$

$M_u = 720000 \times 300 + 90531 \times 237.43$
$= 237.5\ kNm$

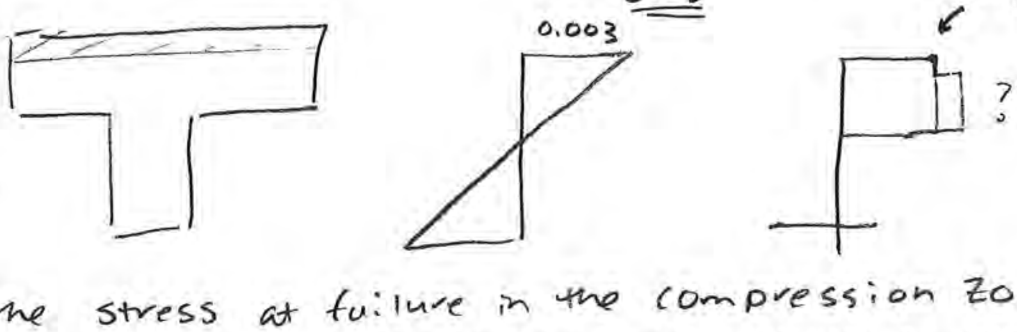
ii)

SLS



if concrete is weaker E is lower → slightly higher curvature but not a big diff

ULS



assuming rectangular stress block

the stress at failure in the compression zone is complex - from true stress-strain curves would expect:

ii) continued

so then there is the question of whether the rectangular stress block assumptions hold

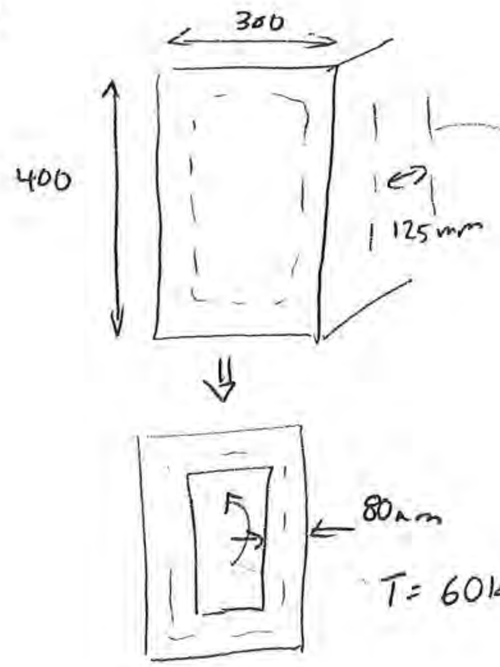
iii) · bookwork (see notes for further details)

e.g. - plastic shrinkage
- poor compaction
- poor curing
- segregation

Assessor's comment:

A surprising minority of students appeared to sketch something other than the axial compressive stress-strain response of concrete e.g. the behaviour of an under-reinforced concrete beam. The ultimate moment calculation was either done very well or poorly and this trend was also apparent in the stress and strain diagrams required for part b(iii).

4a)



10mm ϕ - $A_{sw} = \frac{\pi 10^2}{4} = 78.5 \text{ mm}^2$

$f_{yd} = 430 \text{ MPa}$

$A_e = (300-80)(400-80)$
$= 70400 \text{ mm}^2$

$u = (300-80) \times 2 + (400-80) \times 2 = 1080 \text{ mm}$

i)

$$q = \frac{T}{2A_e} = \frac{60\,000\,000}{2 \times 70400} = 426.14 \text{ N/mm}$$

$$q = f_{yd} \sqrt{\frac{A_w \cdot \Sigma A_l}{s \cdot u}}$$

$$\Sigma A_l = \left(\frac{q}{f_{yd}}\right)^2 \cdot \frac{s \cdot u}{A_w} = \left(\frac{426.14}{430}\right)^2 \cdot \frac{125 \cdot 1080}{78.5}$$

$$= 1689 \text{ mm}^2$$

Q4 a) ii)

$$\tan^2\theta = \frac{A_w u}{s \cdot \Sigma A_L} = \frac{78.5 \times 1080}{125 \times 1689} = 0.402$$

$\tan\theta = 0.633$ $\quad \therefore \theta = 32.36°$

$q = \sigma_c t \sin\theta \cos\theta$

$$\sigma_c = \frac{q}{t \sin\theta \cos\theta} = \frac{426.14}{80 \times \sin 32.36 \times \cos 32.36}$$

$= 11.78$ MPa

$\sigma_c < \underset{0.5}{\nu} f_{cd}$ $\quad \therefore f_{cd} = \frac{11.78}{0.5} = 23.6$ MPa

iii) if f_{yds} (stirrups) $\neq$ f_{ydL} (long) need to reflect the two values in the relevant expressions by considering derivation from first principles

$$q = \sqrt{\frac{A_w f_{yds} \cdot \Sigma A_L f_{ydL}}{s \cdot u}} \Rightarrow \Sigma A_L = 1453\ mm^2$$

$q = \frac{T}{2A_e}$ (unchanged) $= 426.14$ N/mm

$$\tan^2\theta = \frac{A_w f_{yds} \cdot u}{s \cdot \Sigma A_L \cdot f_{ydL}} \Rightarrow$$ if $\Sigma A_L = 1453\ mm^2$ used $\theta = 32.36$

(concrete strut angle will not change since total force $\Sigma A_L f_{ydL}$ req'd in longitudinal steel is as before)

$q = \sigma_c t \sin\theta \cos\theta$ $\quad \sigma_c = 11.78$ MPa

$\sigma_c < \nu f_{cd}$

$f_{cd} = 23.6$ MPa (as before)

4b) • bookwork (see notes for further details)

- is the building layout suitable for use of load bearing masonry? e.g obstructed areas services
- stability / robustness
- dimensional changes - how to accommodate
- thermal / fire / acoustic

• the mortar is subjected to triaxial compressive stress

- need to consider brick / block + mortar as a unit

Assessor's comment:

Although this was the least popular question, it was still answered by just over half the candidates. There were many good solutions to the torsion question which was pleasing. The responses to the masonry design question were patchy.