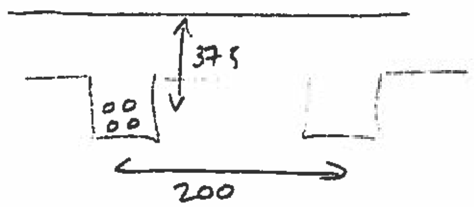
2015-16 4D7 CONCRETE STRUCTURES

Q1.

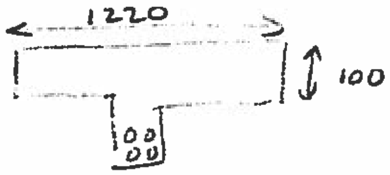


$f_{yd} = 400$
$f_{cd} = 30\,MPa$

(a)(i) find b_{eff} $\quad b_{eff} = b_w + l_0/5 \le b_{actual}$

$b_{eff} = 220 + 5000/5 = 1220\,mm$

equivalent T-beam



assume n.a is in flange + steel yields

$$0.6 f_{cd} \cdot x \times 1220 = 4 \times \frac{\pi 32^2}{4} \times 400$$

$$x = \frac{\pi 32^2 \times 400}{0.6 \times 30 \times 1220} = 58.6\,mm$$

$$M_u = A_s f_{yd} \times (375 - 58.6/2) = 444.8\,kN.m$$

(ii) $1 \le \cot\theta \le 2.5 \qquad f_{cmax} = 0.5 \times 30 = 15\,MPa$

max concrete contribution $\cot\theta = 1$

$$V_{Rd,max} = f_{cmax}\,(b_w 0.9d)/(\cot\theta + \tan\theta)$$

$$= 15 \times 220 \times 0.9 \times 375 / 2 = 556.9\,kN$$

$$V_{Rd,s} = A_{sw} f_{yd}\,(0.9d)(\cot\theta)/s$$

$$\frac{A_{sw}}{s} = \frac{556.9 \times 1000}{400 \times 0.9 \times 375 \times 1} = 4.125$$

$s = 100 \Rightarrow A_{sw} = 412.5\,mm^2 \qquad A_s/leg = 206.3$

$\phi 16 = A_{s\,prov}\ 201\,mm^2 \qquad$ O.K

Q1 (iii)

$A = 220 \times 350 + 2000 \times 100 = 277006\ mm^2 = 0.277\ m^2$

$W = \rho A = 24 kN/m^3 \times 0.277 = 6.65 kN/m$

Flexural failure

M_{max} mid-span $= \frac{W_{max} L^2}{8}$; $W_{max} = 1.4DL + 1.6LL$

$\therefore\ W_{maxF} = \frac{444.8 \times 8}{5^2} = 142.3 kN/m = 1.4DL + 1.6LL$

$LL = \frac{142.3 - 1.4 \times 6.64}{1.6} = 83.15 kN/m$

as a uniform pressure $\frac{83.15}{2} = 41.6 kN/m^2$
very high

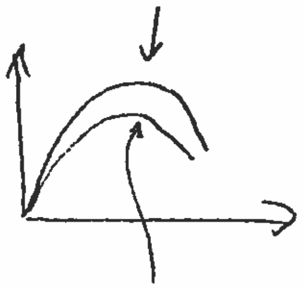
Shear failure

$V_{max} = \frac{W_{max} L}{2}$

$W_{maxS} = \frac{556.9 \times 2}{5} = 222.8 kN/m > W_{maxF}$ $\therefore$ doesn't control

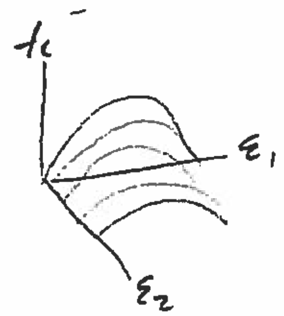
Q1 bookwork

(b)(i) For a cylinder in compression under uniaxial load



But the presence of tension reduces the ability of the concrete strut to sustain load.

In the MCFT the concrete material model reflects a reduction based on emperical values obtained from tests on panels under different bi-axial strain combinations. This led to relationships for compressive stress and strain as a function of ε_2 in the perpendicular direction



(ii) In a $V_c + V_s$ approach the concrete contribution calculated assuming no transverse reinforcement is add to a steel contribution from transverse steel stirrups typically using a 45° truss analogy, This suggests that the concrete capacity is the same with or without stirrups which is debatable

Q2.

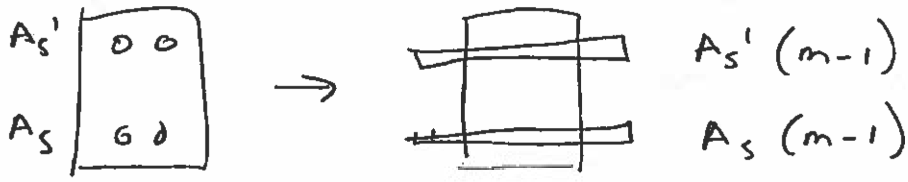
40mm to centre of steel

$E_s = 210\,GPa$

$E_c = 30\,GPa$

a) $M_{ce} = 22.5\,kN.m$

cross-section transformed to concrete



Q2.

a) cont.

$$A_s' = A_s$$

$$I = \frac{bd^3}{12} + A_s(m-1)y^2 \times 2 \qquad m = \frac{210}{30} = 7$$

$$= \frac{200 \times 400^3}{12} + A_s(6) \times (200-40)^2 \times 2$$

$$= 1.067 \times 10^9 + A_s \times 307200$$

at first cracking $\quad \sigma = \frac{My}{I} = f_{ct}$

$$3 \times (1.067 \times 10^9 + A_s \times 307200) = \underset{\text{N.mm}}{22.5 \times 10^6} \cdot 200$$

$$307200 A_s = 1.5 \times 10^9 - 1.067 \times 10^9$$

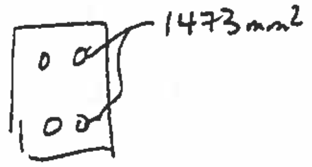
$$= 433.33 \times 10^6$$

$$\therefore A_s = 1411 \text{ mm}^2$$

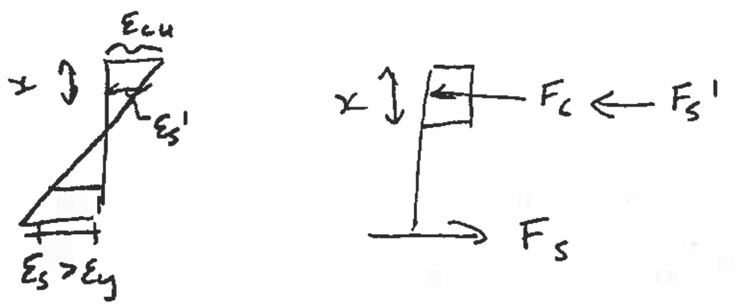
try $3\phi25 \Rightarrow A_s = 1473 \text{ mm}^2 = A_s'$ OK

(ii)

$0.6 f_{cd}$, $f_{cd} = 20$ MPa, $f_{yd} = 460$ MPa



At ULS - n.a. likely to be between two layers of steel but top layer of steel will not have yielded if there is no net axial force.



Q2.

(b) strain compatibility

$$\frac{\varepsilon_{cu}}{x} = \frac{\varepsilon_s'}{(x-40)}$$

$$\varepsilon_{cu} x - 40\varepsilon_{cu} = \varepsilon_s' x$$

longitudinal equilibrium at ULS

$$\underset{F_c}{0.6 f_{cd} \cdot b \cdot x} = \underset{F_s}{A_s f_{yd}} + \underset{F_s'}{A_s' \varepsilon_s' E_s / \gamma_s}$$

$$0.6 f_{cd} \cdot b \cdot x = A_s f_{yd} - A_s' \frac{(\varepsilon_{cu} x - 40\varepsilon_{cu})}{x} \cdot \frac{E_s}{\gamma_s}$$

$$0.6 f_{cd}\, b x^2 - A_s f_{yd}\, x + A_s' \varepsilon_{cu} x \frac{E_s}{\gamma_s} - A_s' 40 \frac{\varepsilon_{cu} E_s}{\gamma_s} = 0$$

$$0.6 \times 30 \times 200\, x^2 - 1473 \times 400\, x + 1473 \times \frac{0.0035 x \times 210000}{1.15}$$

$$- \frac{1473 \times 40 \times 0.0035 \times 210000}{1.15} = 0$$

$$3600 x^2 + (-589200 + 941439.1)x - 37657565 = 0$$

$$x = \frac{-352239 \pm \sqrt{352239^2 - 4(3600)(-37657565)}}{2 \times 3600}$$ 9.0379 e

15 min

$$= \frac{-352239 \pm 816297}{7200}$$

$$= 64.45\text{mm}$$

$$M_u = A_s f_{yd}(400-80) + 0.6 f_{cd}\, b\, x\,(40 - 64.45/2)$$
$$= 190 \times 10^6 \text{ kNm}$$

$$\varepsilon_s' = \frac{0.0035\,(24.45)}{64.45} = 0.01$$

check

F_c $0.6 \times 200 \times 30 \times 64.45 = 232029$ N

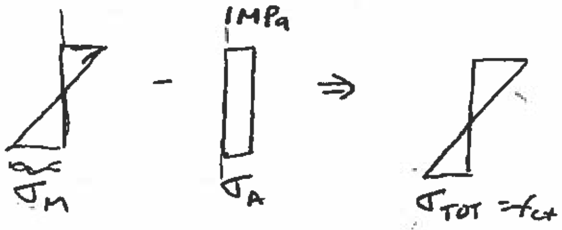
F_s' $1473 \times \cancel{0.0013}^{\,273} \times 210000 / 1.15 = 357171$

F_s $1473 \times 400 = 589200$

Q.2(b)(c) A longitudinal axial force of 80kN

$$\sigma_A = \frac{80000}{200 \times 400} = 1\text{MPa}$$

$$\sigma_{TOT} = \underbrace{\frac{My}{I}}_{\sigma_M} - \sigma_A$$



$$f_{ct} + \sigma_A = \sigma_M$$
$$4 = \frac{My}{I}$$

$\therefore$ A_s Required

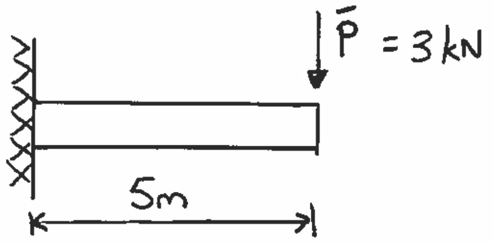
$$4\left(1.067 \times 10^9 + A_s \times 307200\right) = 22.5 \times 10^6 \times 200$$

$$307200 A_s = 1.125 \times 10^9 - 1.067 \times 10^9$$
$$= 58 \times 10^6$$

$\therefore A_s = 188 \text{ mm}^2$ much less

2015-16 407 CONCRETE STRUCTURES cm

Q3



$\gamma_m = 1.3$

$\gamma_f = 1.2$

$CoV = 0.1$

Resistance (R) Mean resistance at support $R = \mu_R = 30$ kNm

$CoV = 10\%$ Standard deviation $\sigma_R = 0.1\mu_R = 3$ kNm

Load effect (S) Mean load $\bar{P} = 3$ kN

At the support: Mean load effect $S = \mu_S = \bar{P} \times L = 3 \times 5 = 15$ kNm.

$COV = 10\%$ Standard deviation $\sigma_S = 0.1\mu_S = 1.5$ kNm

(a)(i) LOAD EFFECT (S) Mean value. $\mu_S = 15$ kNm.

Characteristic value $S_k = \mu_S + 1.645\,\sigma_S = 15 + 1.645 \times 1.5$
$= 17.5$ kNm

Design load effect $S_d = \gamma_f \times S_k = 1.2 \times 17.5 = 21.0$

RESISTANCE (flexural strength) (R) Mean value $\mu_R = 30$ kNm

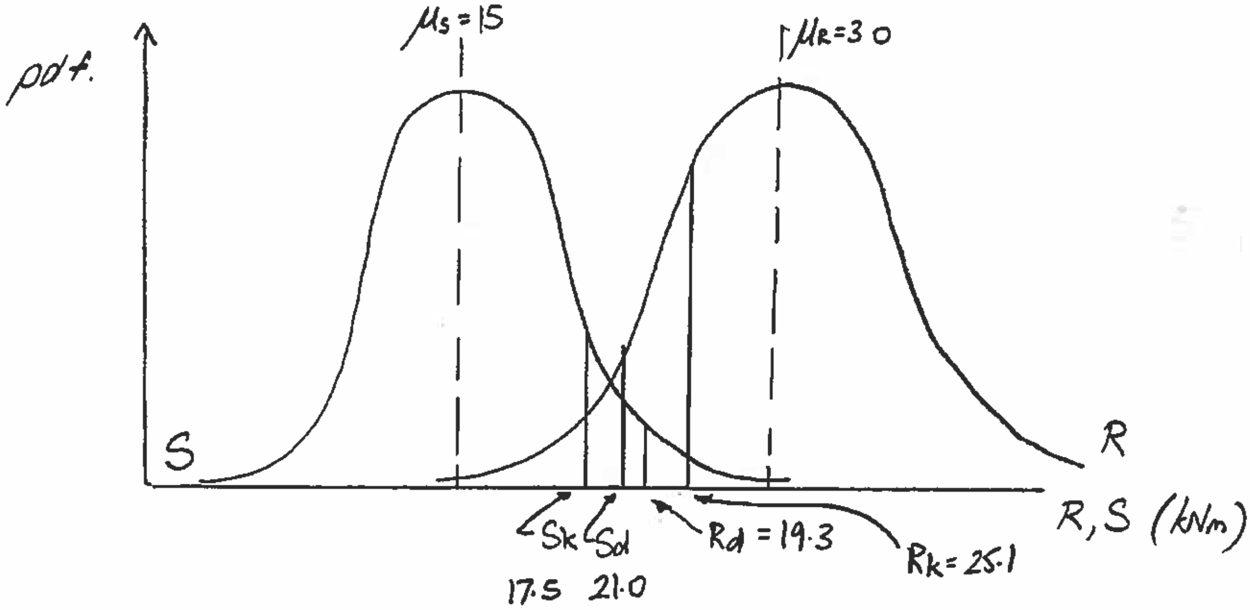
Characteristic value $R_k = \mu_R - 1.645 \times \sigma_R = 30 - 1.645 \times 3$
$= 25.1$ kNm

Design strength $R_d = \dfrac{R_k}{\gamma_m} = \dfrac{25.1}{1.3} = 19.3$ kNm

(ii) $\beta = \dfrac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} = \dfrac{30 - 15}{\sqrt{3^2 + 1.5^2}} = 4.47.$

$\phi(4.47) = 0.9^5 6089$
$\phi(-4.47) = 1 - 0.9^5 6089 = 3.9 \times 10^{-6}$

Prob. failure $= 3.9 \times 10^{-6}$



(iii) $R_d < S_d$ $\Rightarrow$ Design load > Design strength

$\Rightarrow$ UNSAFE Fails code safety criteria

DESPITE $\beta = 4.47$.

Reliability index is dependent on variety of parameters, in particular the coefficient of variation which gives an indication of the spread or variability in the parameters. It is possible to have a relatively high β index even though the code requirement for design load being less than design strength is not met.

(b) $\beta = 3.8$

$$\beta = \frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}} = 3.8$$

$$\therefore \quad \frac{30 - \bar{S}}{\sqrt{3^2 + \left(\frac{\bar{S}}{10}\right)^2}} = 3.8$$ Gives a quadratic in $\bar{S}$.

$$(30 - \bar{S})^2 = 3.8^2 \left(9 + \left(\frac{\bar{S}}{10}\right)^2\right)$$

$$900 - 60\bar{S} + \bar{S}^2 = 14.44\left(9 + \frac{\bar{S}^2}{100}\right)$$

$$0.856\,\bar{S}^2 - 60\bar{S} + 770 = 0$$

Q3 (b) cont.

$$\bar{s}^2 - 70\bar{s} + 900 = 0$$

$$S = \frac{+70 \pm \sqrt{70^2 - 4.1.900}}{2}$$

$$= \frac{70 \pm \sqrt{(4900 - 3600)}}{2}$$

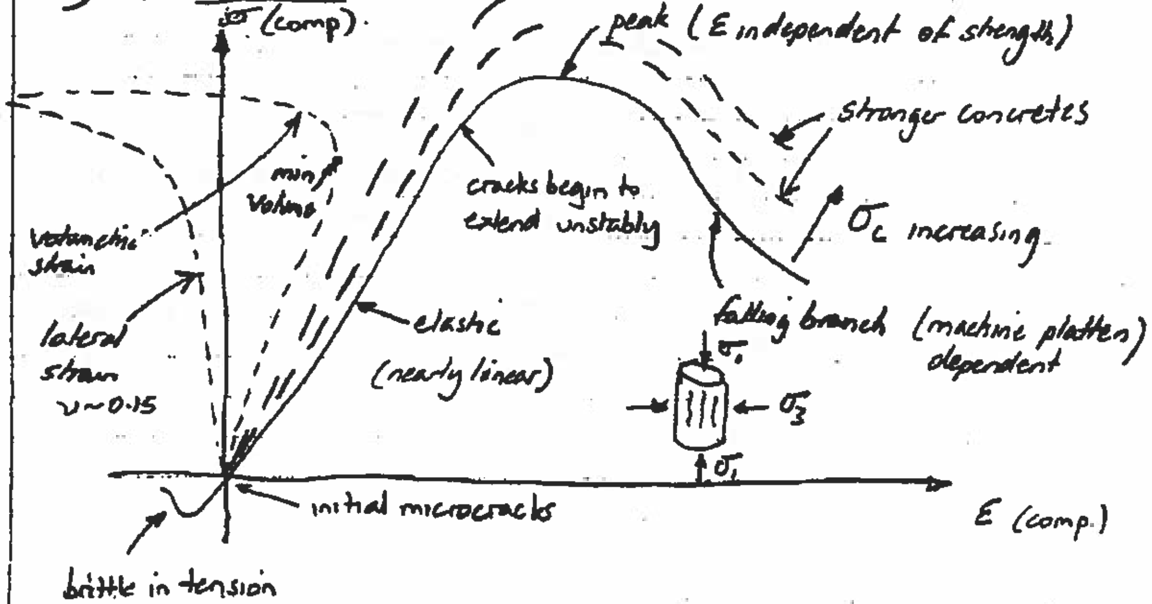
$$= \frac{70 \pm \sqrt{1300}}{2}$$

$$= 53 \quad \text{or} \quad 17.$$

17 is appropriate value here.

Point load $= \frac{17}{5} = 3.4$ kN

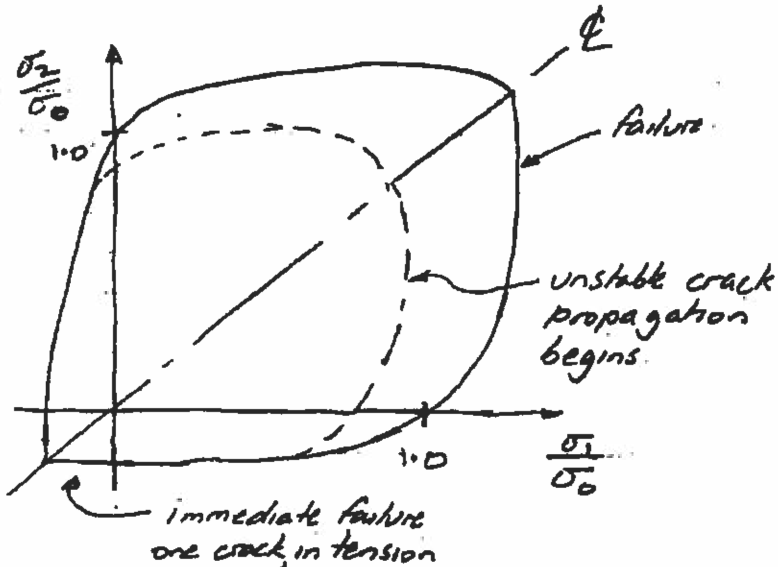
Q3 (c) Uniaxial



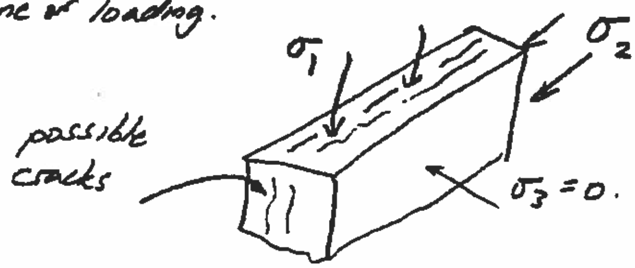
Increasing lateral stress (σ_3) has the effect of increasing ductility (shows as a rise in falling branch of σ–ε graphs)

Biaxial

σ_0 = uniaxial compressive strength



Magnitude of biaxial strength not much higher than uniaxial (15 to 20%) because the specimens can crack in the plane of stress + bulk out of the plane of loading.



Q3 (c) cont.

Triaxial   Crushing is inhibited in compression ∴ very strong

Also more ductile $\sigma \sim \varepsilon$ curves.
Can provide confinement in columns, members using links wrapped around members.

Examples where triaxial properties are exploited:
(1) Lateral binding of columns e.g. in earthquake regions

(2) concrete hinges in columns.

Q4 (a)

(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 Dec2009.

Damage may not be evident at first, nor immediately visible due to the torrent of flood water, or [illegible] observing 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 of 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.

Q4(a)

(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) U.

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 prevent 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 analysis of concrete

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

Q4(a)

(iii) Earthquake.

Causes / damage

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.

Remedial measures

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.

NOT - ultrasound for internal cracking.
- checking of steel in critical regions to ensure no rupture.

Q4(a)

(i) 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, phenolphalein, resistivity, half cell, thermographic.

Paper 4D7 Concrete and Masonry Structures

Solutions

4(b) Bookwork.

Candidates to provide examples of 2 failures.

Ronan Point tower block collapse, where a relatively minor gas explosion led to progressive collapse of parts of a tower block caused by inadequate tying-together of wall and floor precast units. It led directly to requirements to prevent collapses of structures that were out of proportion to the original failure (disproportionate collapse). Changing the factors of safety on the codes would not have made much difference.

Ynys-y-Gwas Bridge, which failed with no load applied to it due to corrosion of prestressing tendons. There were several aspects of the design that allowed the corrosion to take place. Transverse joints between precast elements were filled with mortar, and the deck had no continuous slab. In addition, the ducting for the tendons provided no barrier to water. One notable fact was that the oxygen supply was limited so the steel corroded to a product that did not stain the concrete, so it was not possible to see it from external inspection. Changing factors of safety in codes would have made no difference to the likelihood of failure, although having more or bigger tendons might have delayed it. The principal cause of failure was poor detailing.

Ferrybridge Cooling Towers, several of which collapsed under high, but not excessive winds. The designers had used a wind speed lower than the BS, and had not made any allowance for gusts, or for disturbances to the flow caused by the grouping of the towers. There was also inconsistent application of load factors (factoring the resulting stress, which was the difference of two components, rather than applying factors in the worst sense to the individual load elements).

Concorde Overpass Bridge in Montreal which failed when a brittle shear failure propagated from a half joint through a cantilever. There were inadequacies in the original design (especially in the absence of shear steel), the construction quality management, and in the poor inspection and maintenance regime. Higher safety factors would have made a difference provided shear steel had been included.

Various other failures were discussed in lectures. (Buildings in Chinese Earthquakes, Tasman Bridge Collapse. Tacoma Narrows Bridge is not allowed as it was not a concrete structure.)

4D7 Concrete Structures Examination 2016

Examiners Comments

Q1. The candidates knew that the variable angle truss method requires cot(θ) to be between 1 and 2.5, and that the best answer was at one end of that range, but it seemed to be a pretty random process which one they chose, and no one justified their answer. When asked for the maximum allowable load most ignored the partial load factor. The discussion sections in (b) were done badly if they were done at all; very few could give a succinct idea of what the modified compression field theory was or why it was important. The best answer said "*I don't know what MCFT is about but it involves lots of complicated equations using equilibrium, compatibility and material properties*", which could be a universal answer to all structures questions but got zero marks.

Q2. The first part was a simple elastic transformed section question that could have been done (probably better) by the 1A students. Many didn't take account of symmetry. Part (b) showed a fundamental lack of knowledge of basic mechanics; they needed to work out where the neutral axis was and many took first moments of area (i.e. assuming linear elasticity) and others assumed that the bottom steel was on the point of yielding. Both assumptions lead to the conclusion that the two layers of steel are yielding and the good ones picked up that this was nonsense because it meant that the concrete was doing nothing. Part (c) - if it was done at all, was done reasonably well qualitatively, but there were a lot of numerical errors for what should have been a simple calculation.

Q3. Reliability analysis of a structure and properties of confined concrete. The first part was done correctly by almost everyone, with only a few numerical errors; only 2 candidates made fundamental mistakes. The second part was largely a reproduction of their notes which most could do in more or less detail, although it wasn't always clear that this was accompanied by understanding.

Q4. Discussion of different sorts of damage that might affect a concrete bridge. Almost none of the answers reflected the specific details of the structures that had been given. The sections about earthquake and spalling damage were not done well and marks were lost when answers were incoherent or illegible.

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11th May 2016