Solution to 4D7

Q1(a) Fire Damage to Concrete Structures (section 6.5 in notes)

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.

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 greater 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.

* Also, in some circumstances, depending on moisture condition, pressure set up in the pores in driving moisture out of the system can also cause surface spalling, progressively – this can be alleviated by polymer fibres.

Q1(b) High Strength Concrete (HSC) / High Performance Concretes (HPC) (section 6.6 in notes)

High strength or high performance concretes are those which have compressive strengths typically in the range of around 50 – 120 MPa. These are now quite widely used in construction, in particular for columns in high-rise buildings. Such strengths have been obtained by reducing the water-cement ratio, which has been made possible by the use of high doses of superplasticizer and, to a lesser extent, through the use of silica fume. In addition, compared with normal strength concrete, the cement content is usually slightly increased and the maximum size of the coarse aggregate is usually smaller.

Reactive Powder Concrete (RPC) (section 6.6.1 in notes)

Information based on: http://www.new-technologies.org/ECT/Civil/reactive.htm

More recently, developments in concrete technology have lead to a new family of Portland cement based concretes that are essentially made of powders. Depending on the composition and the heat treatment to which they are subjected, *reactive powder concretes* (RPC) can achieve strengths of up to 200 MPa when cured in hot water (90°C) for three days. It is claimed that concretes have been produced in the laboratory with compressive strengths of up to 800 MPa when cured with dry heat at 400°C. The high strengths are achieved by:

- Improving the homogeneity of the mix by eliminating coarse aggregates.
- Improving the packing of the granular mix by carefully selecting the grain size of the different powders.
- Using the pozzolanic properties of highly refined silica fume and optimisation of the Portland cement chemistry to produce the highest strength hydrates.
- Pressing the concrete before and after setting so that the entrapped air can be eliminated as well as most of the chemical contraction accompanying hydration reactions
- Improving the microstructure after setting through a heat treatment that changes the nature of the hydrates formed
- Improving the ductility of the materials through the use of steel fibres.

Potential advantages of RPC

- RPC is claimed to be a better alternative to High Performance Concrete. It is also claimed to have the potential to structurally compete with steel.
- Its superior strength combined with higher shear capacity results in significant dead load reduction and limitless structural member shape.
- With its ductile tension failure mechanism, RPC can be used to resist all but direct primary tensile stresses. This eliminates the need for supplemental shear and other auxiliary reinforcing steel.
- Its low and non-interconnected porosity diminishes mass transfer making penetration of liquid/gas or radioactive elements nearly non-existent. Cesium diffusion is non-existent and Tritium diffusion is 45 times lower than conventional containment materials.

Application

An application of RPC can be seen in the Pedestrian Bridge in the city of Sherbrooke, Quebec, Canada. RPC has also been used for isolation and containment of nuclear waste of several projects in Europe.

Disadvantages

In a typical RPC mixture design, the least costly components of conventional concrete have been basically eliminated or replaced by more expensive elements. The fine sand used in RPC becomes equivalent to the course aggregate of conventional concrete, the Portland cement fills the role of the fine aggregate and the silica fume that of the cement. The mineral component optimization alone results in a substantial increase in cost over and above that of conventional concrete (5 to 10 times higher than High Performance Concrete.)

Fibre Reinforced Concrete (section 6.7 in notes)

Fibre reinforced concrete has been used widely in recent years. Small fibres of steel or various synthetic materials are added to the mix and distributed throughout the concrete. This provides additional tensile strength to the concrete.

Advantages – can be used to replace temperature steel near slab surface – it is not as susceptible to corrosion of reinforcing bars which leads to spalling and delamination. Close spacing of fibres results in very fine crack pattern compared to conventional mesh and bar arrangements spaces at typically 100mm.

1 (contd).

Disadvantages – limited tensile capacity - still need primary steel in locations of high tensile forces as FRC is mainly used for temperature and shrinkage crack control. Can be difficult to get uniform distribution of fibres throughout mix. Can be costly.

Examples of Use in Practice – It has been employed to reduce the extent of cracking and, in particular, to reduce shrinkage cracking e.g. insitu overlays on concrete bridge decks, concrete floor slabs to reduce surface cracking.

(c) (i) mean 529 s.d 42 MPa

$$f_{char} = 529 - 1.645 \times 42 = \underline{460 \text{ MPa}}$$

$$f_{design} = \frac{460}{1.15} = \underline{400 \text{ MPa}}$$

probability. $z = \dfrac{529-400}{42} = 3.07$, from Tables, probability $\underline{0.0010}$

(ii) mean 65 kN s.d. 20

$$Q_{char} = 65 + 1.645 \times 20 = \underline{97.9 \text{ kN}}$$

$$Q_{des} = 1.35 \times 97.9 = \underline{132.2 \text{ kN}} \text{ applied load}$$

~~Characteristic~~ Design strength (in load terms) is $400 \times \pi \times 20^2/4$
$= \underline{125.7 \text{ kN}}$

So strength < load for design purposes $\therefore$ Not Safe

(iii) Resistance has mean $529 \pi \dfrac{20^2}{4} = 166.2$ kN, s.d. 13.2 kN

$\therefore$ (Resistance − Load) has mean 101.2 kN s.d.
$\sqrt{20^2 + 13.2^2} = 24$ kN.

$\therefore$ β value is $\dfrac{101.2}{24} = \underline{\underline{4.22}}$

(i.e. resistance < load is 4.22 s.d's away from mean)

probability, from Tables, is 1.2×10^{-5}

sensitivity factor : resistance $\dfrac{13.2}{24} = \underline{0.55}$ load $\dfrac{20}{24} = \underline{0.83}$

(iv) Eurocode – well established, but no sensitivity analysis of what is important. But reliability index depends on assumption about normal distribution – not known at tail – and only an indication of merit. Best to say bar UNSAFE.

2. **Q 2(a) Fundamental requirements of limit state design in EC2:** [EC2 *Clause 2.1*]

(Section 1.3 in Lecture notes)

Note: In the Eurocodes a prefix P before a clause number denotes a *principal*, whereas application rules are indented.

Fit for purpose

P(1) A structure shall be designed and constructed in such a way that:
- with acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life and its cost,
  and
- with appropriate degrees of reliability, it will sustain all actions and influences likely to occur during execution and use and have adequate durability in relation to maintenance costs.

Disproportionate collapse

P(2) A structure shall also be designed in such a way that it will not be damaged by events like explosions, impact or consequences of human errors to an extent disproportionate to the original cause.

(3) The potential damage shall be limited or avoided by appropriate choice of one or more of the following:
- avoiding, eliminating or reducing the hazards which the structure is to sustain
- selecting a structural form which has low sensitivity to the hazards considered
- selecting a structural form and design that can survive adequately the accidental removal of an individual element
- tying the structure together.

P(4) The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, design, construction and use as relevant to the particular project.

The primary goal of the design is to produce a safe and robust structure that will fulfil the function for which it is designed over the full design life at the lowest feasible cost i.e. it is *fit for the purpose* for which it was intended. In theory a structure designed to comply with the requirements of the Eurocodes should be fit for purpose however there is always considerable flexibility for the designer to select the structural form, dimensions and materials and hence scope to produce an economic and successful design. In particular attention should be given to ensuring that a realistic evaluation of potential loadcases and hazards is undertaken at the start of a design and that not only the strength requirements at the ultimate limit state are met, but also the various serviceability limit states such as deflection. cracking, vibration and durability, since in practice these tend to determine the useful life and economic viability of a structure. To achieve this goal particular attention needs to be paid to reinforcement detailing to ensure proper distribution of tensile stresses and cracking in the structure, mix design (cement content and water cement ratio) and reinforcement cover, compaction and curing during construction to ensure a durable, impermeable material. The choice of materials used is also critical. Reinforcement corrosion is probably the single most common cause of failure in concrete structures so careful consideration should be given to the protection systems employed and possible allowance for repair or replacement in the future. For example in severe environments corrosion prevention measures such as galvanised or even stainless steel may be employed or else connections built in for the use of cathodic protection at some later time. Several studies have shown that a substantial increase in live load capacity can be achieved for

2.(cont'd)

only a marginal increase in initial cost thus conservative projections of future load demands can be a very cost-effective tool when designing structures to ensure they remain fit for purpose over their long design lives.

The requirement that a structure should not be susceptible to *disproportionate collapse* was included in codes in response to disasters such as Ronan Point Tower Block collapse in London in 1968 in which a small gas explosion in a flat lead to the progressive collapse of a significant proportion of the building. In particular it emphasised that a structure must have continuity between elements and also be able to sustain extensive damage without total collapse. The Oklahoma bombing was an example of a major collapse from a blast loading that should not have lead to such a catastrophic outcome. Similarly earthquake damage could be mitigated significantly if designs had sufficient attention to fundamental detailing to provide ductility, connectivity and continuity in extreme overload scenarios. Buildings supported on columns should be designed so that the loss of any single column should not result in total collapse of the building. Similarly it is good practice to design beam and column structures such that the beams would fail before the columns to avoid catastrophic collapse. Reinforcement detailing is again the key to both ductility and robustness with appropriate attention to confinement of primary steel bars and the structural concrete enclosed being an essential part of this process. Redundancy in the number of critical load paths is another key element used in designs that are robust to extreme events.

(b) Option 1 : stainless steel

Cost + travel delays $40 + 5 \times 20 = 140$k every 20 years

Discount to present value at 6% — assume work starts just after start of 21 st accounting period, so discount for 20 years, 40 etc

$$\therefore \quad WLC = 140 + \frac{140}{1.06^{20}} + \frac{140}{1.06^{40}} = £197.3\text{ k.}$$

Maintenance : continuous discounting $1.06 = \exp(r_c)$ $\therefore \; r_c = 0.05827$

$$A_0 = \int_0^{60} \frac{M.dt}{\exp(r_c t)}$$

$$= M\left[\frac{\exp(-r_c t)}{-r_c}\right]_0^{60} = \frac{M}{r_c}\left[1-\exp(-60 r_c)\right]$$

$= £83.2$ k for $M = 5$k/year

$\therefore$ WLC of option 1 is £280.5 k.

(with traffic delay costs : £139.6 k)

Option 2 : Install 70k, traffic delay 40k total 110k.

$$\therefore \text{WLC} = 110\left[1 + \frac{1}{1.06^{15}} + \frac{1}{1.06^{20}} + \frac{1}{1.06^{45}}\right]$$

$$= £183.04\text{ k}$$

Maintenance is now $\frac{3}{5} \times 83.2 = 49.92$

$\therefore$ WLC for option 2 is $\underline{£232.96\text{ k}}$

so option 2 is preferred on economic grounds.

Without traffic delay costs, option 2 is £116.48 k — so still preferable.

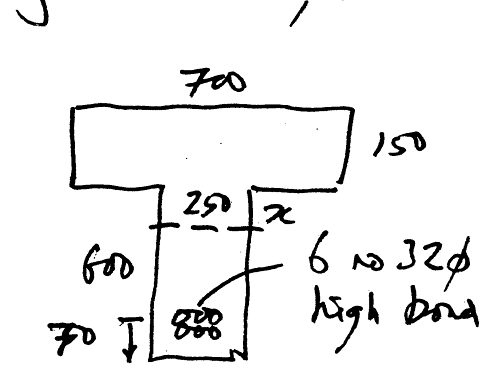
For traffic delay during annual maintenance, would need to work out period of closure etc — and add to maintenance rate per annum. Maybe done at night? Unlikely to make much difference.

3. (a) Limit crack widths, to improve durability (keep deleterious substances out of the concrete, thus limiting corrosion, freeze/thaw problems etc) and to retain public confidence (appearance etc). Limit deflections because of headroom problems below, appearance (public don't like sag) and to limit damage to brittle partitions etc (glass) supported by the structure in question. Also limit sideways deflection in tall buildings, which increase moments in columns by the P-Δ effect. Engineers avoid by following rules (on span/depth, bar sizes and spacings) which ensure that a structure properly designed at ULS will be satisfactory at SLS.

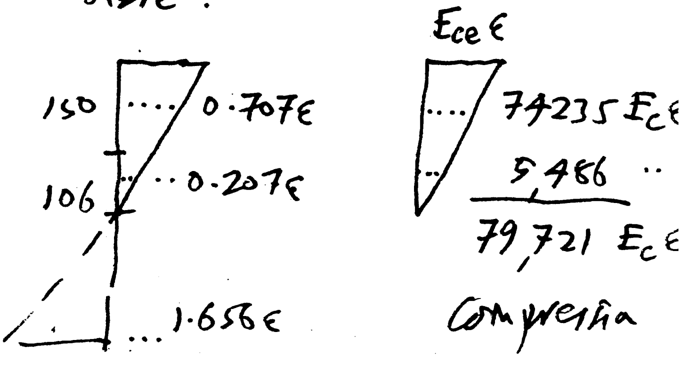
Bond properties influence crack widths, because bond stress is needed to build up tensile stress in concrete again either side of a crack. If bond is weak, new crack must occur further away — and if cracks already too close, no new one can form in between.

(b)

(i)



Given $x = 106$ mm. Check axial force for strain ε in top fibre:



$A_s = 6\frac{\pi}{4} \times 32^2 = 4825\ mm^2$

Tensile force in steel is $10 \times 4824 \text{ mm}^2 \, E_{ce} \times 1.656\,\epsilon$

$$= 79{,}885 \; E_c \, \epsilon$$

so compression and tension approx. balance

[Alternative, solve quadratic for x found by setting first moment of effective section about level x to zero.]

$$I_{cr} = \frac{700 \times 150^3}{12} + 700 \times 150 \times 181^2 + \frac{250 \times 106^3}{3}$$
$$+ 48240 \times 424^2 = \underline{1.241 \times 10^{10} \text{ mm}^4}$$

(ii) moment at midspan : $A = 0.7 \times 0.15 + 0.6 \times 0.25 = 0.25 \text{ m}^2$

$$\therefore \; w_d = 6.375 \text{ kN/m}$$

$$M = \frac{6.375 \times 12^2}{8} + \frac{200 \times 12}{8} = 415 \text{ kNm}$$

$$\therefore \; \sigma_s = \frac{415 \times 10^6}{1.241 \times 10^{10}} \times 424 \times 10 = 141.7 \text{ N/mm}^2$$

$$\therefore \; \underline{\epsilon_{sm} = 691 \times 10^{-6}}$$

$$s_{rm} = 50 + 0.25 \times 0.8 \times 0.5 \times 32 \left\{ \frac{250 \times 494}{4825} \right\}$$
$$= 132 \text{ mm}$$

$$\therefore \; w_k = 1.7 \times 132 \times 691 \times 10^{-6}$$
$$= \underline{0.155 \text{ mm}} \quad \text{say } \underline{0.16 \text{ mm}}$$

(< 0.3 mm, so probably OK).

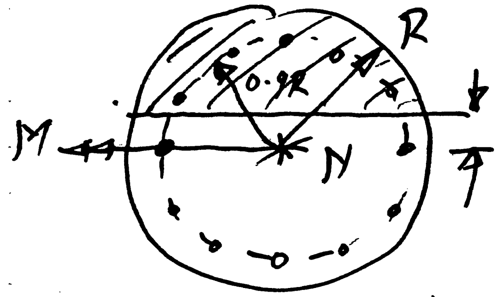
(iii) $\Delta = \frac{5}{48} \kappa L^2$ where κ is midspan curvature.

$$\kappa = \frac{691 \times 10^{-6}}{0.424} = 0.00163 \text{ m}^{-1}$$

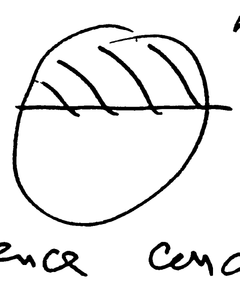
$$\Delta = \frac{5}{48} \times 0.00163 \times 12^2 \text{ m} = \underline{\underline{24.5 \text{ mm}}}$$

An overestimate, (1) because of tension stiffening (use interpolation formula) at midspan, (2) not whole length of beam will be cracked to this extent.

4. (a). This question is essentially bookwork, with some variants. With a lot of bars, the precise direction of the axis of M will not matter — can be selected arbitrarily. Only way forward will be to select a series of neutral-axis positions, e.g. x, calculate N and M for those positions, and plot on $N/f_{cd}R^2$ against $M/f_{cd}R^3$, say.



For each x



need area A of concrete in compression, and centroid position. Hence concrete contribution to N and M

$N_c = 0.6 f_{cd} A$ etc. Then for each steel bar need to use strain diagram

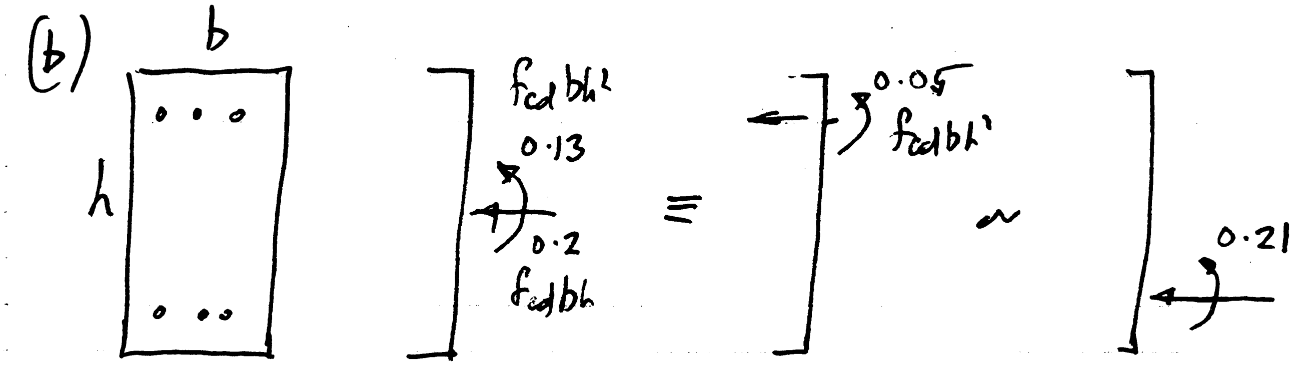
to read off strain in bar, hence ratio $\varepsilon/\varepsilon_y$ and stress in bar, $f_{yd}\,\varepsilon/\varepsilon_y$, hence contribution to N_s and M_s. Add up all concrete and steel contributions.

$n a f_{yd}/\pi R^2 f_{cd}$ is ratio of max. force in steel to that in concrete — measure of proportion of bars — will determine boundary under/over reinforced, etc.

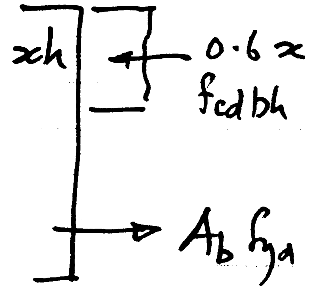
$\varepsilon_y/\varepsilon_{cm}$ will determine which bars have yielded at failure, which have not. For ratio small, all yield.

Prestress — look up σ-ε steel curve with prestrain relative to adjacent concrete added on.

(b)



(i) no top steel :



$0.6x\left(0.9-\frac{x}{2}\right)=0.2$

$\therefore 0.54x-0.3x^2=0.2$

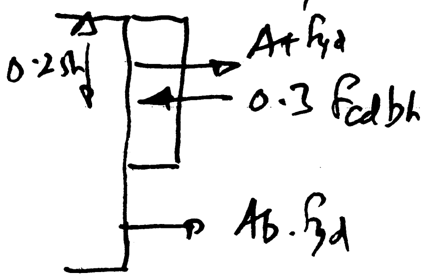
$x^2-1.8x+0.7=0$

$\therefore x = 0.568$ ✓

$\therefore \underline{A_b f_{yd} = 0.141 f_{cd} bh}$

$\frac{\varepsilon_y}{\varepsilon_{cm}} \leqslant \frac{0.332}{0.568} = \underline{0.58}$

(ii) top and bottom, but n.a. half way



$A_b . f_{yd} . 0.8h - 0.3 f_{cd} bh . 0.15h$

$= 0.05 f_{cd} bh^2$

$\therefore A_b f_{yd} = 0.11875 f_{cd} bh$

and $A_t . f_{yd} = -0.01875$ ..

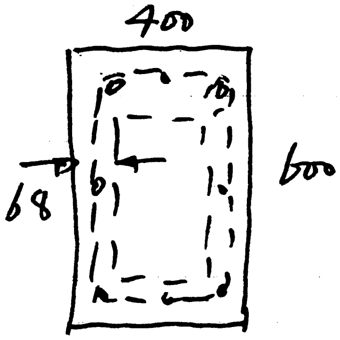
$\therefore$ required area is $\underline{0.1375 f_{cd} bh}$ smaller

$\frac{\varepsilon_y}{\varepsilon_{cm}} \leqslant \frac{0.4}{0.5} = \underline{0.8}$

To check claim further, must check with n.a. either side of centre to see whether true minimum. or argue from diagram, as in an Examples paper question. In practice, calcs. not likely, since moments likely to come on to columns in either sense, equal steel both faces — use interaction diagrams.

5 (a) If the torque is not necessary for equilibrium with the applied loads at the ULS, it may be neglected in design — an application of the lower bound theorem of plasticity. Near ULS the concrete surface will be in shear due to any torque, and so carry combined tension and compression at 90°. Concrete will crack and lose all tensile strength — so tension has to be carried by steel. Convenient to do this by stirrups (balancing lateral component of diagonal force in concrete compression struts) and longitudinal bars (at corners at least, to take stirrup forces round corner etc).

(i)



8 mm bars : $A_w = 50.26 \text{ mm}^2$

Spacing $s = 100$ mm.

25 mm : $\Sigma A_l = 3927 \text{ mm}^2$

perimeter $u = 2(332 + 532)$

$= 1728$ mm

from Sheet $\therefore \; q = 250 \left\{ \frac{50.26}{100} \times \frac{3927}{1728} \right\}^{1/2}$

$= 267$ N/mm

ult. torque $T = 2 q A_e$ where A_e is 332×532 m

$\therefore$ $\underline{T = 94.4 \text{ kNm}}$

Taking force cover puts stirrups at centre of wall carrying concrete compression members — no awkward couples out of balance etc.

(ii) $$\frac{A_w . f_{yd}}{s} = \sigma_c t \sin^2\theta$$

$$\frac{\Sigma A_l . f_{yd}}{u} = \sigma_c . t . \cos^2\theta$$

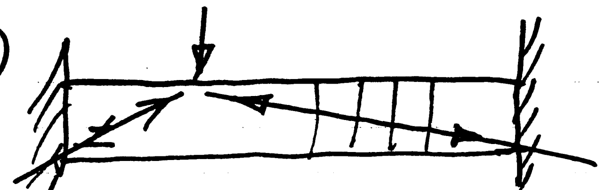
$$\therefore \tan^2\theta = \frac{A_w/s}{\Sigma A_l/u} = \frac{50.26}{100} \times \frac{1728}{3927}$$

$\therefore$ $\underline{\theta = 25.2^\circ}$ (rather low).

then $\sigma_c t = \frac{50.26}{100} \times \frac{230}{\sin^2(25.2)} = 693.8$ N/mm

$t = 68$ mm $\therefore$ $\sigma_c = 10.2$ MPa

if $\nu = 0.6$, required $f_{cd} = \frac{10.2}{0.6} = \underline{17}$ MPa

(b) 

For a flat arch, a line of thrust within the masonry can always be found, as shown above. So on the usual plastic theory for masonry — no sliding between blocks, infinite compressive strength but no tensile (joints) — the lower bound theorem says that any load can be carried. ~~Explain~~ The load would in fact be limited by actual strength of stone, movement of abutments allowing arch to sag etc.

Concrete becomes similar to masonry (compressive strength high relative to tensile): flat arches can occur, giving extra load — but very vulnerable to deflection $\therefore$ unstable, and to abutment movement.