

6 Concrete

C1

For high-rise buildings a structural core is most commonly chosen to provide stability.

C2

C40/50 indicates the compressive strength (40 N/mm² is the cylindrical strength; 50 N/mm² is the cubic strength). In calculations the cylindrical strength is most commonly used, which was given as the characteristic strength. The material factor for concrete is $\gamma_M = 1.5$, resulting in a design compressive strength of $f_{cd} = \frac{f_{ck}}{\gamma_M} = \frac{40}{1.5} = 26.6$ N/mm². It cannot be the tensile strength for two main reasons: 1) Tensile strength is considered zero for concrete so a graph for structural calculations does not make sense. 2) Tensile strength must be much lower than the compressive strength, for C40/50 the mean tensile strength is $f_{ctm} = 3.51$ N/mm².

C3

The slenderness ratios are used in the serviceability limit state (to limit deflections); they give an early indication of the floor thickness. Consider two floors of equal span, but differing boundary conditions. Qualitatively it can be said that support condition II results in greater deflections than support condition I; as clamping the ends will reduce midspan deflections. Therefore, support condition II will require a beam of greater depth (higher) to increase its stiffness and thereby reduce its deflections. This gives for support condition I a ratio of $l_{eff}/d = 35$ and for condition II a ratio of $l_{eff}/d = 25$ (without the need to remember the exact ratios; assume a unit depth of $d = 1$, allowing for condition I a greater span $l_{eff} = 35 > 25$). Important: Note that the answers given in the exam are incorrectly inversed.

C4

Important: Ignore the scrabbling on the paper and use the original dimensions of $l_1 = 12$ m and $l_2 = 1.5$ m. First, the bending moment diagram must be constructed to find the location with the highest stresses. Start with the equilibrium equations:

$$\begin{aligned}\sum M_A = 0 : \quad & 40 \cdot \frac{l_1}{2} + 20 \cdot (l_1 + l_2) - R_{VB} \cdot l_1 = 0 \\ \sum V = 0 : \quad & 40 + 20 - R_{VA} - R_{VB} = 0\end{aligned}$$

This gives:

$$R_{VB} = 42.5 \text{ kN}$$

$$R_{VA} = 17.5 \text{ kN}$$

The bending moment diagram has two peaks at the locations of the external loads, which can be calculated as follows:

$$\begin{aligned}M_{d, \text{left}} &= R_{VA} \cdot \frac{l_1}{2} = 105 \text{ kNm} \\ M_{d, \text{right}} &= 20 \cdot l_2 = 30 \text{ kNm}\end{aligned}$$

It can be seen that the maximum bending moment is $M_{Ed} = 105$ kNm. The approximate method is used, giving:

$$z = 0.9 \cdot d = 0.9 \cdot 550 = 495$$

From this and the steel design yield strength the minimum reinforcement can be calculated:

$$A_s \geq \frac{M_{Ed}}{z \cdot f_{yd}} = \frac{105}{495 \cdot 435} = 488 \text{ mm}^2$$

C5

The maximum shear can be found at midspan of l_1 (check this yourself with a shear diagram). This gives:

$$V_{Ed} = |R_{VA} - 40| = 22.5 \text{ kN}$$

The minimum shear can be calculated as follows:

$$k = 1 + \sqrt{\frac{200}{d}} = 1.603$$

$$v_{\min} = 0.035 \cdot k^{\frac{3}{2}} \cdot \sqrt{f_{ck}} = 0.035 \cdot 1.603^{\frac{3}{2}} \sqrt{30} = 0.389$$

$$V_{Rd,c} = v_{\min} \cdot b \cdot h = 0.389 \cdot 300 \cdot 600 = 64 \text{ kN}$$

7 Steel

S6

The elasticity of the steel grades is equal. Both the yield and ultimate tensile strength are larger for S355 compared to lower strength steels such as S235.

S7

Flange:

$$c = \frac{1}{2}(b - 2r - t_w)$$

$$c = \frac{1}{2}(300 - 2 \cdot 27 - 8.5) = 118.75$$

$$\frac{c}{t_f} = \frac{118.75}{14} = 8.48$$

This corresponds to a class 1 flange.

Web:

$$c = h - 2r - 2t_f$$

$$c = 290 - 2 \cdot 27 - 2 \cdot 14 = 208$$

$$\frac{c}{t_w} = \frac{208}{8.5} = 24.47$$

The beam is loaded by a uniformly distributed load, which results in bending of the beam. Considering this, the beam has a class 1 web.

As both flange and web are of class 1, the beam is of class 1.

S8

For a class 3 beam, plastic behavior cannot be achieved without (local) buckling. Therefore, the resistance must be determined using the elastic properties. It is assumed the beam is oriented such that it is loaded about its strong axis (y-y); as this is the normal orientation for a beam and it does not make sense to load it about its weak axis. This results in the following moment resistance:

$$M_{el,Rd} = \frac{W_{el,y} f_y}{\gamma_{M0}} = \frac{1260 \cdot 10^3 \cdot 235}{1.0} = 296 \text{ kNm}$$

The applied maximum bending moment (in the ULS; SLS is for deformations not strength) can be found as follows:

$$q_{Ed} = \gamma_{sw} q_{sw} + \gamma_v q_v = 1.2 \cdot 10 + 1.5 \cdot 25 = 49.5 \text{ kN/m}$$

$$M_{\max,Ed} = \frac{q_{Ed} L^2}{8} = \frac{49.5 \cdot 7.2^2}{8} = 320.76 \text{ kNm}$$

This gives a unity check of:

$$UC_b = \frac{M_{\max,Ed}}{M_{el,Rd}} = \frac{320.76}{296} = 1.08$$

This means that the moment resistance is not sufficient, as the applied load results in a higher bending moment than what the beam can resist.

S9

Buckling in-plane of the facade means that the beam buckles about its weak (z-z) axis. The table can now be used to find the correct buckling curve using the geometry:

$$\frac{h}{b} = \frac{200}{100} = 0.5 \leq 1.2$$

The flange properties are:

$$t_f = 8.5 \leq 100 \text{ mm}$$

For S235 and buckling about z-z, this results in **buckling curve c (0.49)**.

Important: Be aware the the mechanical schemes may be confusing. We are talking about buckling in-plane of the facade. From the right sectional view, it can be seen that this concerns buckling the y-direction (about the z-z axis). Next, look at the mechanical scheme on the left, which represents the buckling behavior in-plane of the facade. This is confirmed by the initial text, which tells us that buckling out-of-plane is fully restrained whereas in-plane buckling is only restrained at the ends and in the middle. However, when looking at the coordinate system, it shows that buckling is in the z-direction (about y-y), which is not the buckling behavior that is drawn and described. It may be assumed that the coordinate system on the left is incorrect, and should have its z-direction replaced by the y-direction.

S10

First, the Euler buckling load must be calculated. Buckling in-plane is considered so the second moment of area about z-z must be used. Additionally, L_{cr} can be derived from the boundary conditions. In this case, as it is restrained in the middle, the buckling length is $L_{cr} = \frac{1}{2}L = \frac{1}{2}5000 = 2500 \text{ mm}$. This results in:

$$N_{cr} = \frac{\pi^2 EI_z}{L_{cr}^2} = \frac{\pi^2 \cdot 210000 \cdot 142.4 \cdot 10^4}{2500^2} = 472225.04 \text{ N}$$

From this the following sequences of equations must be calculated:

$$\begin{aligned} \bar{\lambda} &= \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{2848 \cdot 235}{472225.04}} = 1.19 \\ \Phi &= 0.5 \left(1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right) = 0.5(1 + 0.34(1.19 - 0.2) + 1.19^2) = 1.38 \\ \chi &= \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1.38 + \sqrt{1.38^2 - 1.12^2}} = 0.48 \\ N_{b,Rd} &= \frac{\chi Af_y}{\gamma_{M1}} = \frac{0.48 \cdot 2848 \cdot 235}{1.0} = 323.47 \text{ kN} \end{aligned}$$

8 Aluminum

A11

In contrast to steel, yielding is not as clear-cut for aluminum. A method to roughly find the yielding moment is to find the stress where 0.2 strain is achieved after unloading. This is after the fully elastic branch (proportionality limit). This 0.2% strain limit $f_{0.2}$ is used in structural calculations.

A12

Strength very much depends on the used steel grade/alloys, but steel and aluminum (for normal applications) can show comparable strengths. Stability of a material depends very much on the cross-sectional shape, so nothing can be said about this (although it much easier to make all kinds of sections with an aluminum extrusion process; for example to include stiffeners; but this is not necessarily a material property). Aluminum is in fact much better resistant to corrosion due to its oxide layer. **A big downside of aluminum is its fire resistance, as aluminum has a low melting point.**

A13

Cross sections are often limited in height (compared to other building products); therefore it is

not expected that it is cut in three for transport reasons. Cutting a section does not improve the strength; it can even make it worse. Three parts instead of one does not make construction easier, there are more parts to be handled. **However, it is very likely that the die can only be of limited size. Therefore, it was necessary to extrude the section in three parts. Another reason for smaller dies can be that they may be reused. For example, the top and bottom extrusions are the same for all sections; but the middle section can vary in height to construct different sections with only a limited set of dies.**

9 Timber

T14

First, the load must be transformed from characteristic to design load:

$$F_{Ed} = Q_k \cdot \gamma_q = 11.8 \cdot 1.35 = 15.93 \text{ kN}$$

Next, the maximum bending moment at midspan can be calculated as:

$$M_{Ed} = \frac{1}{4} F_{Ed} \cdot \frac{L}{2} = \frac{1}{4} \cdot 15.93 \cdot \frac{2.5}{2} = 4.978 \text{ kNm}$$

Using this, the maximum bending stress is calculated as:

$$W = \frac{1}{6} \cdot w \cdot h^2 = \frac{1}{6} \cdot 59 \cdot 171 = 2.875 \cdot 10^5 \text{ mm}^3$$

$$\sigma_{m,d} = \frac{M_{Ed}}{W} = \frac{4.978 \cdot 10^6}{2.875 \cdot 10^5} = 17.3 \text{ N/mm}^2$$

For C24, sawn timber, and short indoors load duration, the following parameters can be read from the design tables (and k_h , which can be used for bending, is equal to 1.0 for this beam height):

$$\gamma_M = 1.30$$

$$k_{mod} = 0.90$$

$$f_{m,k} = 24$$

$$f_h = \left(\frac{150}{170} \right)^{0.2} = 0.974 \text{ (thus 1.0, as } \geq 1.0)$$

This gives the following strength:

$$f_{m,d} = \frac{f_{m,k}}{\gamma_M} \cdot k_{mod} = \frac{24}{1.30} \cdot 0.90 = 16.6 \text{ N/mm}^2$$

$$UC = \frac{\sigma_{m,d}}{f_{m,d}} = \frac{17.3}{16.6} = 1.04 \not\leq 1.0 \text{ not ok}$$

T15

Top timber-timber connection

There are two shear planes, with in total two bolts. This gives a resistance of:

$$F_{V,Rd} = 2 \cdot 2 \cdot F_{V,u,d} = 2 \cdot 2 \cdot 4.9 = 19.6 \text{ kN}$$

This gives a unity check of:

$$UC = \frac{11.8 \cdot 1.35}{19.6} = 0.81$$

Bottom timber-steel connection

There is one shear plane, with in total three bolts. This gives a resistance of:

$$F_{V,Rd} = 1 \cdot 3 \cdot F_{V,u,d} = 1 \cdot 3 \cdot 6.6 = 19.8 \text{ kN}$$

This gives a unity check of:

$$UC = \frac{11.8 \cdot 1.35}{19.8} = 0.81$$

Both unity checks are roughly $UC \approx 0.8$, which is ok.

T16

Both certification institutions have as a main goal to ensure sustainable forest management. They differ in how their certification processes work; but both have this as a goal.

T17

Option B is the best option for the following reasons:

1. The diagonals provide the shortest load path to the supports (hinges).
2. The diagonals are loaded in compression (and not tension), which is the stronger timber orientation.
3. The connections are loaded in compression, and not in tension, providing a more rigid connection.

T18

Wood is first and foremost strongest parallel to the grain (growing direction). This way the natural fibers are loaded; and not the lignin bonds between them (which is significantly weaker). Second, timber is stronger in compression than in tension. This is mostly due to how the fibers are compressed when they are loaded by compression. They start to interlock and resist further deformation.

This gives the following strengths in ascending order:

1. Tension perpendicular
2. Compression perpendicular
3. Tension parallel
4. Compression parallel

10 Masonry

M19

Pointing mortar is mostly used for aesthetic reasons as a finishing. It does protect the mortar slightly, but does not make it waterproof or structurally stronger.

Pinnacles (or buttresses) are often heavy to direct the normal forces in the flying buttresses downwards (and keep the line of thrust inside the arches/vaults/walls). These elements are important structural elements in churches.

M20

Shear

For shear half of the wall is considered, as half is supported by the floor and half by the ground floor. This gives the following load:

$$\begin{aligned} w_d \cdot \frac{h}{2} &\leq \mu \cdot F_v \\ w_d \cdot \frac{h}{2} &\leq \mu \cdot \frac{q_d \cdot L}{2} \\ w_d \cdot \frac{2}{2} &\leq 0.333 \cdot \frac{5 \cdot 6}{2} \\ w_d &\leq 5.0 \text{ kN/m} \end{aligned}$$

Overturing

For overturning, schematize the wall using two blocks. It is a block with the total vertical load

$F_v = 30/2 = 15$ kN and the distributed load over the height $h/2$. Moment equilibrium about the turning point gives:

$$\begin{aligned}\sum M = 0 : \quad w_d \cdot \frac{h}{2} \cdot \left(\frac{h}{2} \cdot \frac{1}{2} \right) - F_v \cdot t &= 0 \\ w_d &\leq \frac{F_v \cdot t}{\frac{h}{2} \cdot \left(\frac{h}{2} \cdot \frac{1}{2} \right)} \\ w_d &\leq \frac{\frac{5 \cdot 6}{2} \cdot 0.2}{\frac{2}{2} \cdot \left(\frac{2}{2} \cdot \frac{1}{2} \right)} = 6 \text{ kN/m}\end{aligned}$$

Therefore, shear is governing and a maximum wind load of $w_d = 5.0$ kN/m is acceptable.

M21

Lower bound

The lower bound can be found at the inner side of the arch, moment equilibrium about the point where the external load touches gives:

$$R_{H,\min} = \frac{200 \text{ kN} \cdot 1 \text{ m}}{2.75 \text{ m}} = 73 \text{ kN}$$

Upper bound

The upper bound is more difficult to find, as the height where the line of thrust is cannot be simply found. Therefore, both max. bounds are rewritten to a corresponding height. This is done as follows:

$$\begin{aligned}\sum M_{\text{load}} = 0 : \quad R_{H,\max} &= \frac{200 \text{ kN} \cdot 2 \text{ m}}{h} \\ 145 \text{ kN} &= \frac{200 \text{ kN} \cdot 2 \text{ m}}{h} \quad (h = 2.76) \\ 200 \text{ kN} &= \frac{200 \text{ kN} \cdot 2 \text{ m}}{h} \quad (h = 2)\end{aligned}$$

For the upper bound, the line of thrust touches the inner radius of the arch. Therefore, the height must be lower. For this reason, the bounds are 73-200 kN. (One can also assume that the line of thrust for the maximum case is roughly 1m lower, which is the radius of the arch. This comes close to this range as well (229 kN).)

M22

The following properties are given or can be extracted from the given tables:

$$\begin{aligned}f_k &= 1.7 \cdot 6 = 10.2 \text{ N/mm}^2 \\ f_b &= 40 \text{ N/mm}^2 \\ K &= 0.6 \\ \alpha &= 0.65 \\ \beta &= 0.25\end{aligned}$$

The following equation must be rewritten to find the compressive strength of the mortar:

$$\begin{aligned}f_k &= K \cdot f_b^\alpha \cdot f_m^\beta \\ f_m &= \sqrt[\beta]{\frac{f_k}{K \cdot f_b^\alpha}} = \sqrt[0.25]{\frac{10.2}{0.6 \cdot 40^{0.65}}} = 5.7 \text{ N/mm}^2 \quad (\text{minimal})\end{aligned}$$

M23

Top

The reduction at the top is:

$$\begin{aligned}
e_{uk} &= e_{u0} = 40 \text{ mm } (\geq 0.05 \cdot 200 = 10 \text{ mm}) \\
\frac{e_{uk}}{t} &= \frac{40}{200} = 0.2 \\
\Phi_u &= 0.6 \text{ for } h_{ef}/t_{ef} = 0
\end{aligned}$$

Top

The reduction at the bottom is:

$$\begin{aligned}
e_{lk} &= e_{l0} = 10 \text{ mm } (\geq 0.05 \cdot 200 = 10 \text{ mm}) \\
\frac{e_{lk}}{t} &= \frac{10}{200} = 0.05 \\
\Phi_l &= 0.9 \text{ for } h_{ef}/t_{ef} = 0
\end{aligned}$$

Mid

The reduction at the middle is:

$$\begin{aligned}
e_{m0} &= (e_{u0} + e_{l0})/2 = (40 + 10)/2 = 25 \text{ mm} \\
e_{mk} &= e_{m0} + h/450 = 28.44 \text{ mm } (\geq 0.05 \cdot 200 = 10 \text{ mm}) \\
\frac{e_{mk}}{t} &= \frac{28.44}{200} = 0.14 \\
\Phi_l &= 0.65 \text{ for } h_{ef}/t_{ef} = 10
\end{aligned}$$

The smallest reduction factor is normative, so this is $\Phi_u = 0.6$.