CTB3335

Concrete structures 2

Juni 2011
April 2012
Juni 2012

Tentamenbundel Civiele Techniek
Het Gezelschap "Practische Studie"

EEN REPRODUCERENDE LEERSTIJL
IS SCHADELIJK VOOR DE
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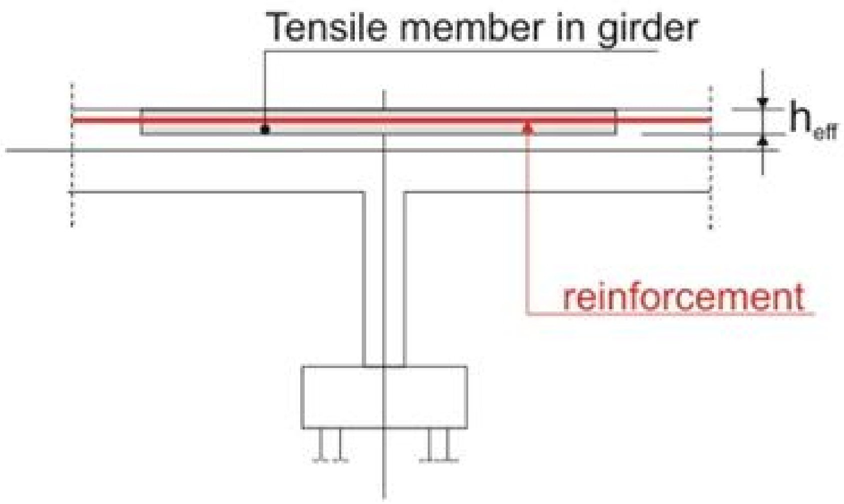
Please keep in mind that this exam of June 22nd 2012 was a computer exam of 2 hours. In the written examination as from 2012 also open question will be asked.

Question 1 Qrack Width

To determine the crack width in the support region of a continuous girder, a part of the structure is modeled as a reinforced tensile member, see the figure. The height of the tensile member is $h_{c,eff}$. The width of the tensile member is equal to the width of the girder (b). The dimensions of the girder are: b * h = 400 mm * 800 mm.



Average axial tensile strength concrete	: $f_{ctm} = 2.2$ N/mm^2
Diameter main reinforcement	: $\Phi = 32$ mm
Maximum crack width	: $w_{max} = 0.15$ mm
The required reinforcement is equal to the provided reinforcement	: $A_{s,required} = A_{s,provided} = 4400$ mm^2
Height tensile member	: $h_{c,eff} = 137.5$ mm.
Long term elastic modulus of concrete	: $E_{cm\,(\infty)} = 12000$ N/mm^2
Reinforcing steel	: B500B $f_{yd} = 435$ N/mm^2
Elastic modulus of steel	: $E_s = 200000$ N/mm^2

A. Calculate the strain of the tensile member (ε_{ct}), expressed in ‰, in case the concrete starts to crack for the first time.

B. Calculate the stress in the reinforcement (σ_{so}) as the concrete starts to crack, expressed in N/mm^2.

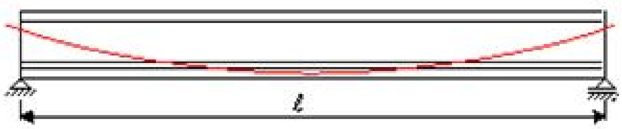
C. Calculate the tensile force (Ntcrack), expressed in kN, as the concrete starts to crack.

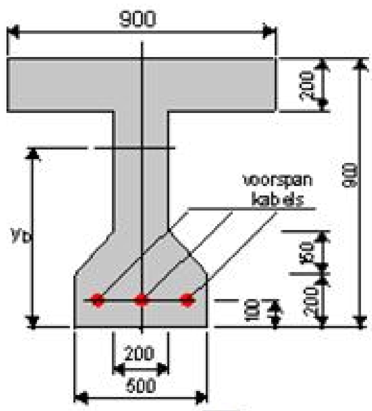
D. Calculate the stress (σ_{sr}) in the reinforcement in the crack, **just after** the first crack has occurred.

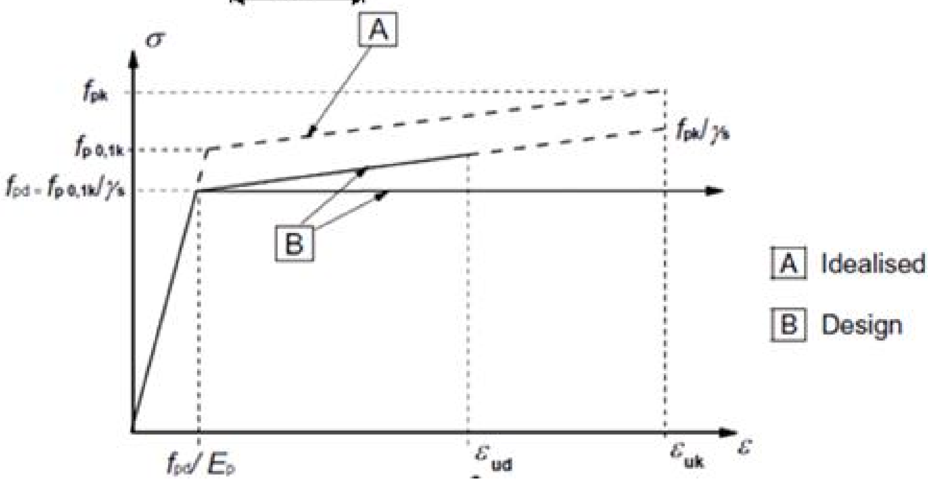
Question 2 Prestressed Tgirder

A prestressed girder with a span (l) of 21 meter, is prestressed using 3 prestressing tendons.
In the middle cross-section the prestressing tendons are 100 mm from the bottom fiber of the girder.
The center of gravity of the anchorages coincides with the center of gravity of the cross-section (e_o = 0). The girder is fully prestressed.
In the middle cross-section the following can be assumed:

- Friction loss and time dependent losses are 20% in total
- Elastic losses are compensated for.
- Variable loading (q) = 10.5 kN/m^1.







A_c = 0.403 m^2	y_b = 0.508 m	I_c = 0.037 m^4	
Strength class of concrete	C35/45	f_{cd} =23.33 N/mm^2	
Stress - strain diagram concrete	α = 0.75	β = 0.389	
Density of concrete	25 kN/m^3		
Prestressing steel Class A	Y1860S7 f_{pk} =1860 / 1.1 = 1691 N/mm^2		
	σ_{pm0} = 1395 N/mm^2		
	E_p = 195000 N/mm^2		
Partial load factors:	γ_g = 1.2	γ_q = 1.5	γ_p = 1.0

A. Calculate the working prestressing force (P_{mt}), expressed in kN, which is required to have no tensile stresses in the bottom fiber of the girder.

B. Calculate the initial prestressing force (P_{m0}), expressed in kN.

C. Calculate the required cross-sectional area of the prestressing steel (A_p), expressed in mm^2.

D. Calculate the value of the design bending moment (M_{Ed}), expressed in kNm.

E. Calculate the value of the bending moment capacity (M_{Rd}), expressed in kNm.
To answer this question, it can be assumed that the tensile stress in the prestressing steel in ULS is 1607 N/mm^2

F. To answer question 5, the height of the concrete compression zone x_u had to be calculated. Now calculate the strain of the prestressing steel (ε_{ud}) in ULS that follows from the calculated x_u, expressed in ‰.

G. Calculate the actual stress in the prestressing steel in the ULS , expressed in N/mm^2.

Use the stress - strain diagram of the prestressing steel with the following parameters:
f_{pd} = 1522 N/mm^2
f_{pk} / γ_s = 1691 N/mm^2
E_p = 195000 N/mm^2
ε_{uk} = 35 ‰

H. Calculated is:
M_{Ed} = (D) kN, and
M_{Rd} = (E) kN.
Does the critical cross-section fulfill the requirement with regard to safety?

Yes or No

Question X Multiple choice questions

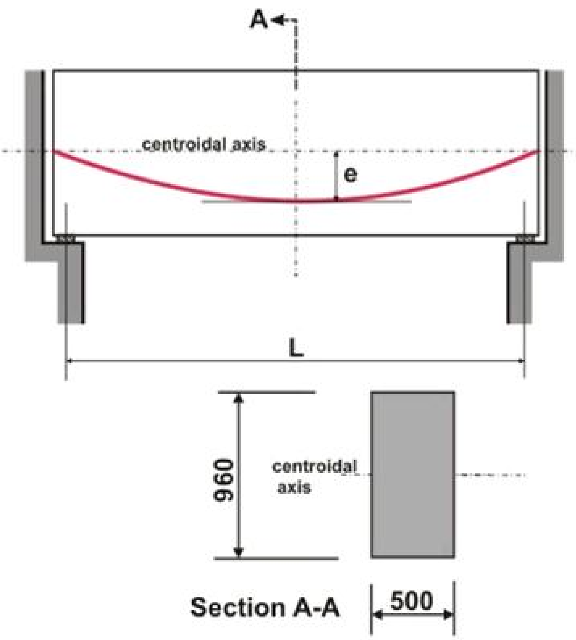
The multiple choice questions belonging to this exam are not included in this answer form.

Question 3 Eccentricity

A prestressed girder with a span of 19 meter has 1 curved tendon. The centroidal axis of the anchorages coincides with the centroidal axis of the girder ($e_O = 0$).
Dimensions of the girder: 500 mm * 960 mm (width * height)
The profile of the prestressing is presented in the drawing.
The magnitude of the initial prestressing force $P_{m0} = 2600$ kN.

Time-dependent losses and friction losses are 20% in total.
Elastic losses and losses caused by wedge set are compensated for.
Density of concrete is 25 kN/m^3.

Remark for all the questions
Requirement:
The girder has to be fully prestressed. No tensile stresses are allowed to occur in any fiber of the cross-section.



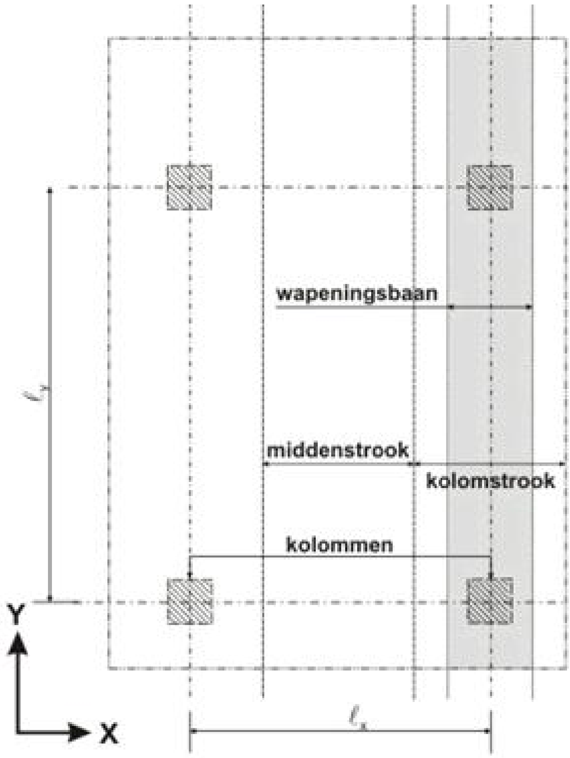
A. Calculate, in cross-section A-A, the maximum eccentricity (e_{max}), expressed in mm, at time = 0.
   Remark: A variable load isn't present at time t = 0.

B. Calculate, in cross-section A-A, the minimum eccentricity (e_{min}), expressed in mm, at time $t = \infty$.
   At time $t = \infty$ also a variable load is present
   The variable load is: 5 kN/m^1.

C. Calculate in the cross-section at the support, the maximum eccentricity (e_{max}), expressed in mm, at time = 0.
   A variable load isn't present at time t = 0.

Question 4 Flat slab floor - punch

A flat slab floor is supported by columns. The slab has several spans. Consider a panel continuous at all edges, see the figure. The slab is part of a braced structure (non-sway frame).



Spans	$l_x = 7$ m
	$l_y = 9.8$ m
Depth of the slab	h = 361.129 mm
Dimensions columns	320 * 320 mm
Strength class C35/45	$f_{ck} = 35$ N/mm^2
	$f_{cd} = 35 / 1.5 = 23.33$ N/mm^2
Density of concrete	$\rho = 25$ kN/m^3
Concrete cover	c = 25 mm
Diameter bars in x - and y - direction	$\Phi = 20$ mm
Internal lever arm	z = 0.9 * d
Reinforcing steel B500B	$f_{yd} = {}^{500}/_{1.15} = 435$ N/mm^2
Variable load	$q_{var} = 6{,}00$ kN/m^2
Partial load factors	$\gamma_G = 1.2$ $\gamma_Q = 1.5$

A. Calculate the design value of the punching shear load (V_{Ed}), which each column has to resist, expressed in kN.
   For this question it holds:
   The panel considered is continuous at all four edges. This implies that the column has to resists loads from the 4 adjacent spans (panels).
   The connection between the slab and the column can be regarded as being hinged.

B. Calculate the design value of the punchingshear stress, expressed in N/mm^2.
   For an ideal midfield column it is prescribed that factor $\beta = 1$.

A

$\sigma = E \cdot \varepsilon$

$\varepsilon = \frac{\sigma}{E}$

} first crack when f_{ctm} is reached.

$$\varepsilon = \frac{f_{ctm}}{E_{cm(\infty)}} = \frac{2.2}{12000} = 0.000183.$$

$$\varepsilon \text{ in ‰} = 0.183 \text{ ‰}$$

B

$\sigma_s = E_s \cdot \varepsilon$

$$\sigma_s = 200.000 \cdot 0.000183 = 36.67 \ N/mm^2$$

C Just before the crack, the Normal force $N_{t,crack}$ is the force in the concrete tensile member (concrete part A_c) + in the reinforcement A_s

$$N_{tcrack} = f_{ctm} \cdot (h_{c,eff} \cdot b) + \sigma_s \cdot A_s = 282.33 \ kN$$

D Just after the crack, the concrete part (over A_c) (from the previous question) is not contributing anymore. So the force present in the tensile member (in the cracked cross-section) should now be taken up entirely by the reinforcement.

$$\sigma_{sr} = \frac{N_{tcrack} \cdot 1000}{A_s} = 64.17 \ N/mm^2$$

A $t = \infty$ no stresses bottom fiber SLS

$q_{Eg} = A_c \cdot \rho = 0.403 \cdot 25 = 10.075$

$q_Q = 10.5$ kN/m

$q_{total} = 20.575$ kN/m

$$\frac{-P_{mt}}{A_c} - \frac{P_{mt} \cdot e}{W_b} + \frac{\frac{1}{8} q_{total} \cdot l^2}{W_b} = 0$$

$A_c = 0.403$

$W_b = \frac{I_c}{y_b} = \frac{0.037}{0.508} = 0.0728$

$e = 0.408$

$$\frac{-P_{mt}}{0.403} - \frac{P_{mt} \cdot 0.408}{0.0728} + \frac{\frac{1}{8} \cdot 20.575 \cdot 21^2}{0.0728} = 0.$$

$P_{mt} = 1926.5$ kN

B $P_{mt} = P_{mo} \cdot 0{,}8.$

$P_{mo} = \frac{P_{mt}}{0.8} = \frac{1926.5}{0.8} = 2408.1$ kN

C $\sigma_{pmo} = 1395$ N/mm²

$\sigma_{pmo} \cdot A_p = P_{mo}.$

$A_p = \frac{P_{mo}}{\sigma_{pmo}} = \frac{2408.1 \cdot 10^3}{1395} = 1726.$ mm².

D) calculate M_{Ed} in ULS

loads $q_{Eg} = 10.075$
$q_Q = 10.5$ (question 2A)

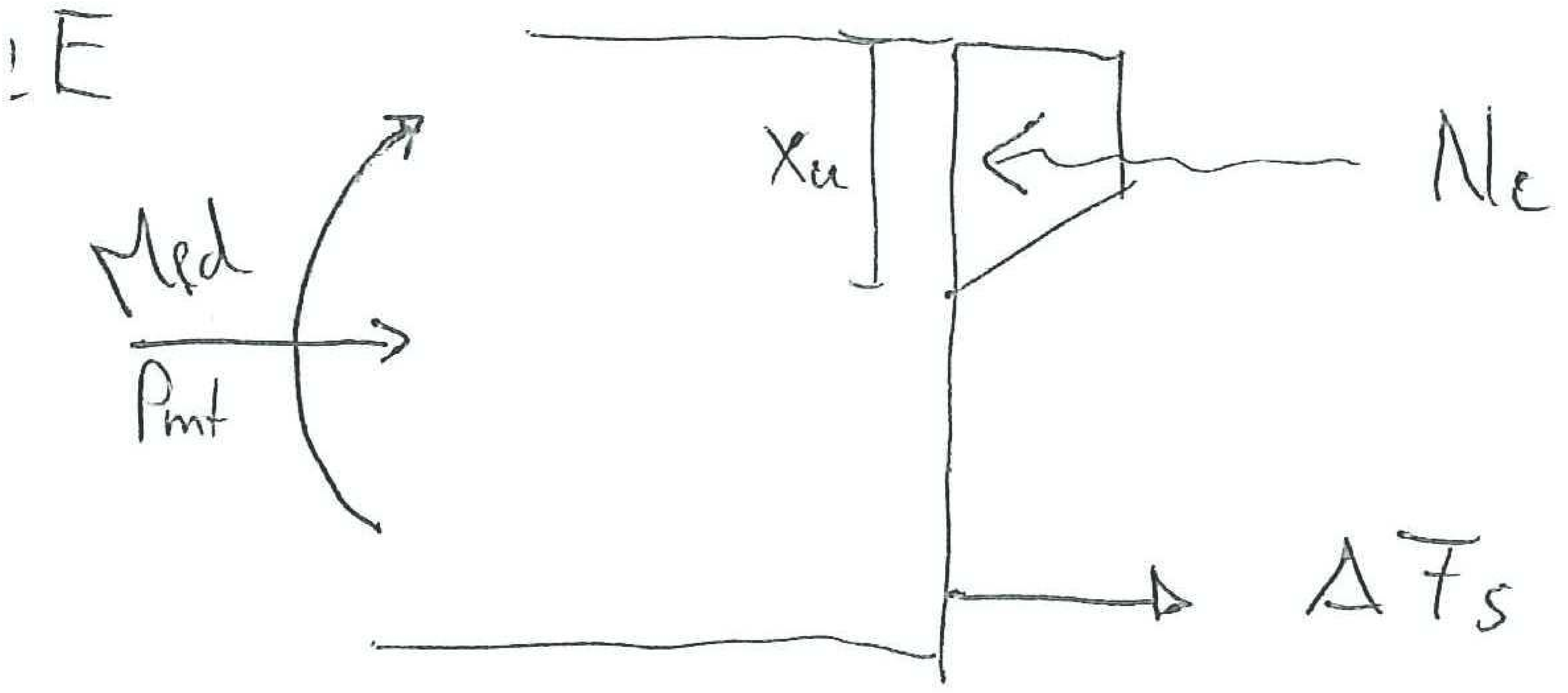
$$q_{Ed} = \underbrace{1.2 \cdot 10.075 + 1.5 \cdot 10.5}_{M_{external\ load}} + \underbrace{1.0 \cdot q_p}_{M_p}$$

$M_{external\ load} = \frac{1}{8} q l^2 = \frac{1}{8} \cdot 27.84 \cdot 21^2 = 1534.68$ kNm

$M_p = e P_{mt} = 0.408 \cdot 1926.5 = 786.02$ kNm

$M_{Ed} = 1534.68 - 786.02 = 748.66$

E



$\Delta F_s = (1607 - 1395 \cdot 0{,}8) \cdot A_s = 847.6$ kN

$P_{mt} = 1926.5$

$N_c = x_u \cdot \alpha \cdot b \cdot f_{cd} = x_u \cdot 0{,}75 \cdot 900 \cdot 23{,}33$

$\Sigma N = 0 \quad P_{mt} + \Delta F_s - N_c = 0.$

$1926.5^{(kN)} + 847.6^{(kN)} = x_u \cdot 0{,}75 \cdot 900 \cdot 23.33^{(N/mm^2)}$

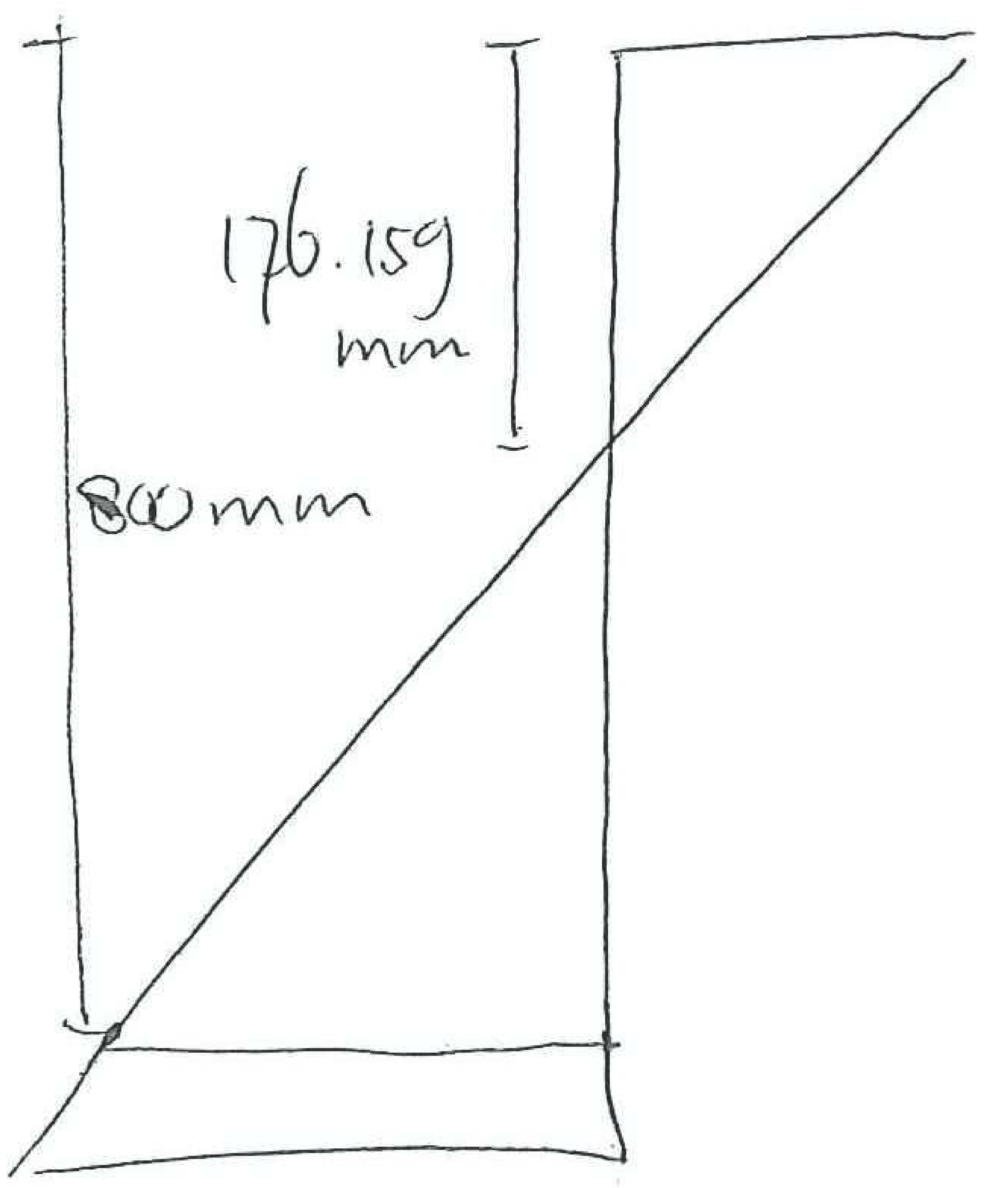
$x_u = 176.159$ mm

$\Sigma M = 0. \quad -M_{Ed} + \Delta F_s (y_b - 0.1) + N_c \cdot (0{,}9 - y_b - \beta x_u)^{(m)}$

$M_{Ed} = 847.6 \cdot 0{,}408 + \frac{176.159 \cdot 900 \cdot 0{,}75 \cdot 23.33}{1000} \cdot (0.9 - 0.508 - 0.389 \cdot 0.176)$

$M_{Ed} = 1243.171$

2 F ε



strain in steel ($\Delta\varepsilon$)

$$3.5‰ \left(\frac{800 - 176.159}{176.159} \right) = 12.39 ‰$$

strain in steel from prestressing

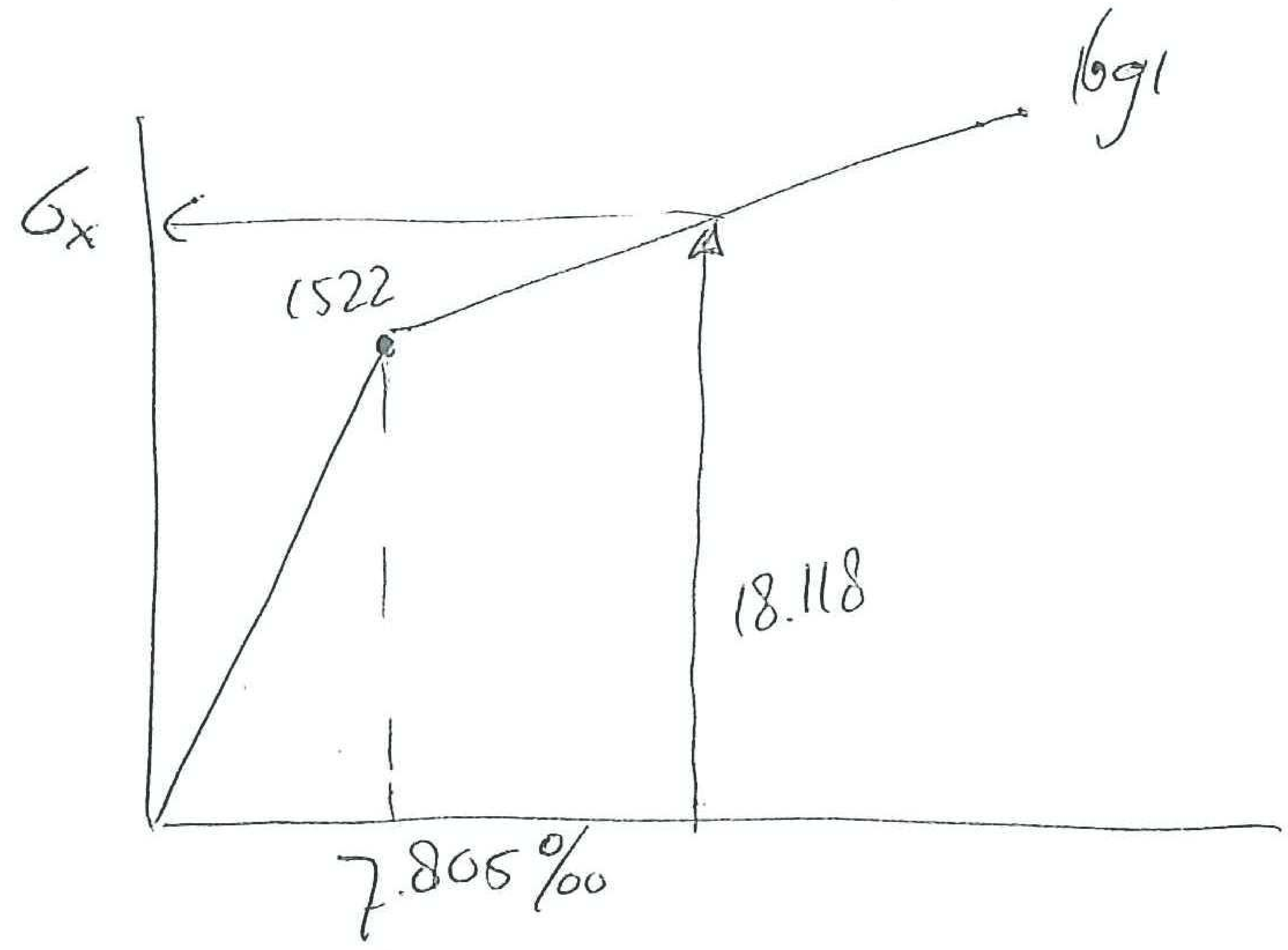
$$\sigma = E_s \cdot \varepsilon$$

$$\varepsilon = \frac{\sigma}{E_s} = \frac{P_{mt}/A_p}{E_s} = 0.005723.$$

$$\varepsilon = 5.723 ‰$$

$$\varepsilon_{total} = 12.39 ‰ + 5.723 ‰ = 18.118 ‰$$

σ



$$\frac{\sigma}{E} = \varepsilon = \frac{1522}{195000}$$

$$\sigma_x = 1522 + \frac{(1691 - 1522) \cdot (18.118 - 7.805)}{35 - 7.805} = 1586.1 \ N/mm$$

2 H a) yes

3A $t=0$

no stresses top fibre (SLS)

$$q_{eg} = (500 \cdot 960) \cdot 25 = 12 \text{ kN/m}$$

$$M = \frac{1}{8} q l^2 = \frac{1}{8} \cdot 19^2 \cdot 12 = 541.5 \text{ kNm}$$

$$-\frac{P_{mo}}{A_c} + \frac{P_{mo} \cdot e}{W_t} - \frac{M}{W_t} = \sigma_{top,0} = 0$$

$$-\frac{2600}{0.5 \cdot 0.96} + \frac{2600 \cdot e}{\frac{1}{6} \cdot 0.5 \cdot (0.96)^2} - \frac{541.5}{\frac{1}{6} \cdot 0.5 \cdot (0.96)^2} = 0$$

$$e = 368 \text{ mm}$$

B $t = \infty$

no stresses bottom fibre (SLS)

$$q_Q = 5 \text{ kN/m} \qquad M_Q = \frac{1}{8} \cdot 5 \cdot 19^2 = 225.6 \text{ kNm}$$

$$-\frac{P_{mt}}{A_c} + \frac{M_{eg} + M_Q}{W_b} - \frac{P_{mt} \cdot e}{W_b} = 0$$

$$P_{mt} = P_{mo} \cdot 0.8 = 2600 \cdot 0.8 = 2080$$

$$-\frac{2080}{0.48} + \frac{767.1}{0.0768} - \frac{2080 \cdot e}{0.0768} = 0 \qquad e = 208.8 \text{ mm}$$

C $t=0$ cross section at the <u>support</u>

$$-\frac{P_{mo}}{A_c} + \frac{P_{mo} \cdot e_s}{W_t} = 0$$

$$-\frac{2600}{0.48} + \frac{2600 \cdot e_s}{0.0768} = 0 \qquad e = 160 \text{ mm}$$

$q_Q = 6\ kN/m^2$

$q_{Eg} = h_{floor} \cdot \rho$

$q_{Eg} = 0{,}361129 \cdot 25 = 9.028$

$q_{Ed} = 1.5\, q_Q + 1.2\, q_{Eg} = 1.5 \cdot 6 + 1.2 \cdot 9.028 = 19.834\ kN/m^2.$

$V_{Ed} = l_x \cdot l_y \cdot q_{Ed} = 7 \cdot 9.8 \cdot 19.834 = 1360.59\ kN$

B $v_{Ed} = \dfrac{V_{Ed} \cdot \beta}{u \cdot d_{eff}}$

$d_x = h_{floor} - c - \frac{1}{2}\phi_x = 361.129 - 25 - 10 = 326.129$

$d_y = h_{floor} - c - \phi_x - \frac{1}{2}\phi_y = 361.129 - 25 - 20 - 10 = 306.129$

$d_{eff} = \dfrac{d_x + d_y}{2} = 316.129$

$u = 4 \cdot t_{column} + 2 \cdot \pi \cdot 2 d_{eff} =$

$4 \cdot 320 + 2 \cdot \pi \cdot 2 \cdot 316.129 = 5252.594\ mm$

$v_{Ed} = \dfrac{1360.59 \cdot 10^3 \cdot 1}{5252.594 \cdot 316.129} = 0.819\ N/mm^2$

TU Delft
Faculty of Civil Engineering and Geosciences

Write down on each examination sheet:
CT3150 or CIE 3150
Student number:
Name:

Examination CT3150/CIE3150 Concrete Structures 2

2012 April 12th, 9:00 - 12:00 h.

S.A.A.M. Fennis

General instructions:

- This exam consists of 4 questions, an equation form and an answer sheet for question 2.
- Write your name, student number and course code (CT3150 or CIE3150) on each answer sheet in the upper right corner.
- Place only one question on each sheet (front and back).
- Use only the paper provided. Ask for additional sheets, if required.
- **Show all work to arrive at your answer**: Include calculations and sketches.
- When reviewing the work, also neatness and readability are taken into account.
- Per question a time indication is presented to guide you through the exam.
- It is allowed to use a (graphical) calculator and 1 A4 (one side) with your own notes.
- The use of mobile phones (or any other devices than specified above) is not allowed during the exam. Please remove them from your desk.
- Please keep your student card available during the exam.

Best of Luck!

Question 1 Slabs		30 min
Question 2 Prestress (design)		60 min
Question 3 Tunnels		25 min
Question 4 Prestress (losses)		65 min
	Total	**180 min**

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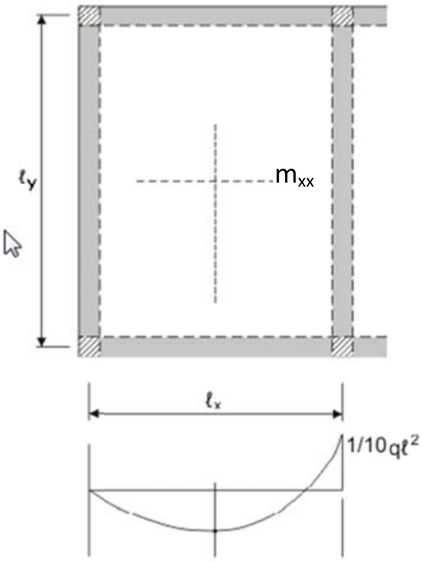
Question 1 Slabs (30 min)

One panel of a slab supported by stiff beams is regarded, see the drawing. The slab is loaded by a uniform load q_{Qk}. The slab is part of a braced structure.



Spans	: l_x = 7 m
	: l_y = 9 m
Total depth of the slab	: h = 235 mm
Concrete strength class	: C28/35, f_{cd}=28/1.5=18.67 N/mm^2
Density concrete	: ρ = 25 kN/m^3
Concrete cover	: c = 25 mm
Diameter reinforcement in x - direction	
	: ϕ = 20 mm, d_x = 200 mm
Diameter reinforcement in y - direction	
	: ϕ = 12 mm, d_y = 184 mm
Internal lever arm	: z = (0.9 * d)
Reinforcement class B500B	: f_{yd} = 500 / 1.15 = 435 N/mm^2
Variable load	: q_{Qk} = 16.0 kN/m^2 (archives room)
Partial load factors	: γ_G = 1.2 : γ_Q = 1.5

Figure 1.1 Overview of the slab and moment distribution m_{xx}.

1a) Describe in two sentences how the equilibrium method can be used to determine the percentage of the total load that is carried in x-direction and y-direction

1b) Write down the two equations which are required to derive the part of the load that is carried in x-direction $k_x = \frac{5l_y^4}{2l_x^4 + 5l_y^4}$ for the panel which is freely supported at three sides and continuous at one side. (The equations do not have to be solved.)

1c) Calculate the percentage of the total load that is carried in the x-direction, expressed in %.

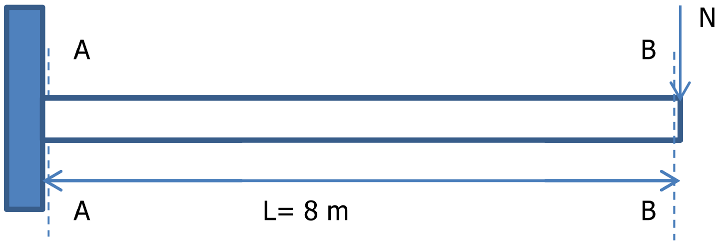
1d) Calculate the design value of the positive bending moment (span, sagging) in the x - direction (m_{Edx}), expressed in kNm/m.

Note: the negative moment at the support is $\frac{1}{10}ql^2$ (See also figure 1.1).

1e) Calculate the amount of reinforcement (A_s) required to resist the positive bending moment in the x- direction, expressed in mm^2 / m^1.

Question 2 Prestress (design) (60 min)

A girder with a span of 8 meter, is fully fixed on one side and loaded with a variable point load of N=50 kN at the free end. The girder is post-tensioned by one tendon.



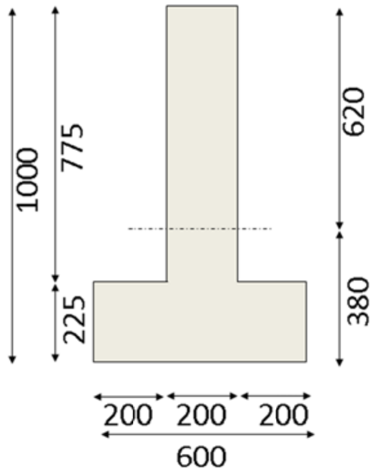


Figure 2.1 Overview of the girder and concrete cross section B.

Parameters:

Density concrete : ρ = 25 kN/m^3
Variable load (point load, see drawing), : N = 50 kN
Dimensions girder : see figure 2.1
Distance from centroidal axis to bottom fibre : 0.38 m
Distance from centroidal axis to top fibre : 0.62 m
Cross-sectional area girder : A_c = 0.29 m^2
Moment of inertia : I_c = 0.0264 m^4
Section modulus (top fibre) : W_t = 0.0426 m^3
Section modulus (bottom fibre) : W_b = 0.0695 m^3
Strength class of concrete : C50/60
Strength class of prestressing steel : Y1860S7
Initial tensile stress at anchor : σ_{pmo} = 0,75 * 1860 = 1395 N/mm^2
Elastic modulus prestressing steel : 195000 N/mm^2
Minimum distance centre of tendon to top or bottom fibre: 60 mm
Anchor size : 180x180 mm
Reinforcement class B500B : $f_{yd} = {}^{500} / {}_{1.15}$ = 435 N/mm^2
Partial load factors : γ_G = 1.2 : γ_Q = 1.5

2a) Choose a tendon profile to fully prestress the girder as efficient as possible. Draw the profile on the answer sheet and explain in maximum 4 sentences why this is the best profile.

The girder is prestressed by a force of 1000 kN in longitudinal direction. For aesthetic reasons it is chosen to position the prestress anchor centered at 380 mm from the bottom of the girder.
2b) Draw the strut-and-tie model for the introduction of the prestressing force on the answer sheet. Assume a constant stress distribution at the edge of the disturbed area (St. Venant).

To control the crack width in the disturbed area, a steel stress of 300 N/mm^2 is prescribed.
2c) Calculate the reinforcement required for the introduction of the prestressing force in mm^2. Include the answer in a practical reinforcement design and draw it on the answer sheet.

2d) Show that the unity check for shear capacity is below 1.0 in cross-section A. Assume flexural-shear failure.

Question 3 Tunnels (25 min)

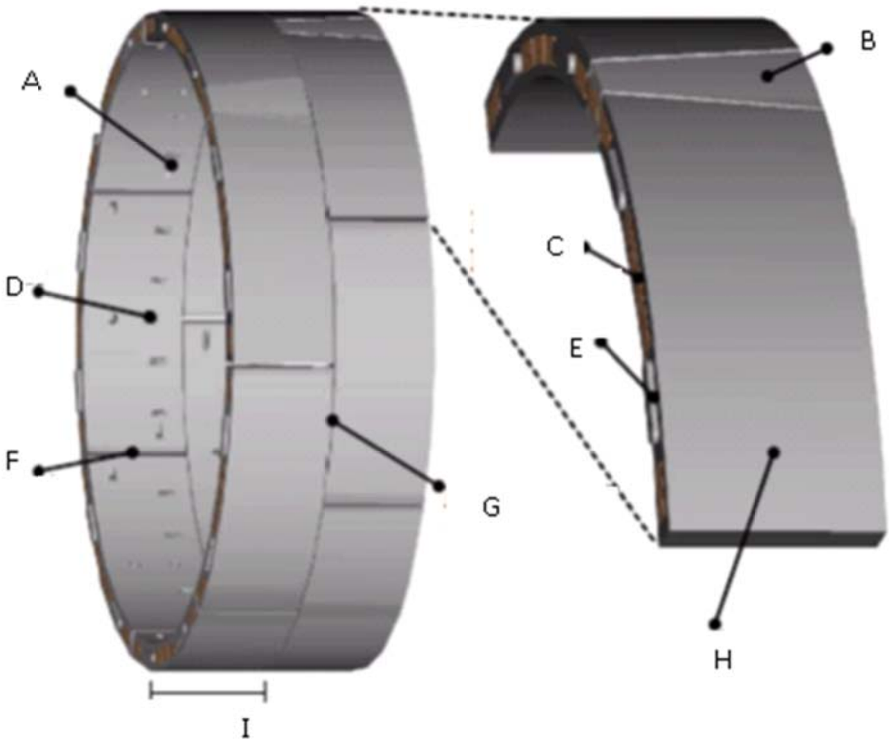


Figure 3.1 Tunnel lining.

3a) Fill in the correct definitions for the given elements A-I in the tunnel lining of figure 3.1. Choose from: Bolt pocket, Dowel, Handle hole, Lateral joint, Longitudinal joint, Key segment, Packing material, Ring, Segment.

3b) Describe (sketch) the ring lay out of the tunnel lining in figure 3.1 and explain in maximum two sentences the main advantage or disadvantage of this lay out compared to other possible layout(s).

3c) Multiple choice question: write down A, B, C or D.
What are the main functions of grout?
A: Sealing the tunnel against seepages and embedding the tunnel.
B: Prevention of surface damages and avoiding governing loading on the tunnel.
C: Prevention of soil movement and embedding the tunnel.
D: Protection against aggressive soil chemicals and avoiding tangential loading.

3d) Multiple choice question: write down A, B, C or D.
Ground supports the lining. In what case is the support the most effective?
A: At uniform compression.
B: At ring ovalisation.
C: When the ring concrete Young's modulus is infinite.
D: When the grout is just injected.

3e) To move the TBM forward, jacks are used with the centroidal axes of their shoes spaced at 1600 mm. Each jack shoe (size 150*800) exerts a force of F_{Ed} = 4000 kN on the concrete segments (strength class C30/37) with segmental thickness of 400 mm and a size of 1500 mm (ring width) * 3200 mm (segment length). Verify whether the shoe size is sufficient with regard to the ultimate compressive stress.

Question 4 Prestress (losses) (65 min)

A box girder bridge with a span of 30 meter (see figure) is fully prestressed by 6 draped tendons. The tendon profiles are shown in the figure. The fictitious tendon is positioned 200 mm from the bottom at midspan B, and 400 mm from the top at cross section A. The prestressing is applied with post-tensioned steel, with bond (see equation form, page 8-9) and anchored at the end supports A and C.

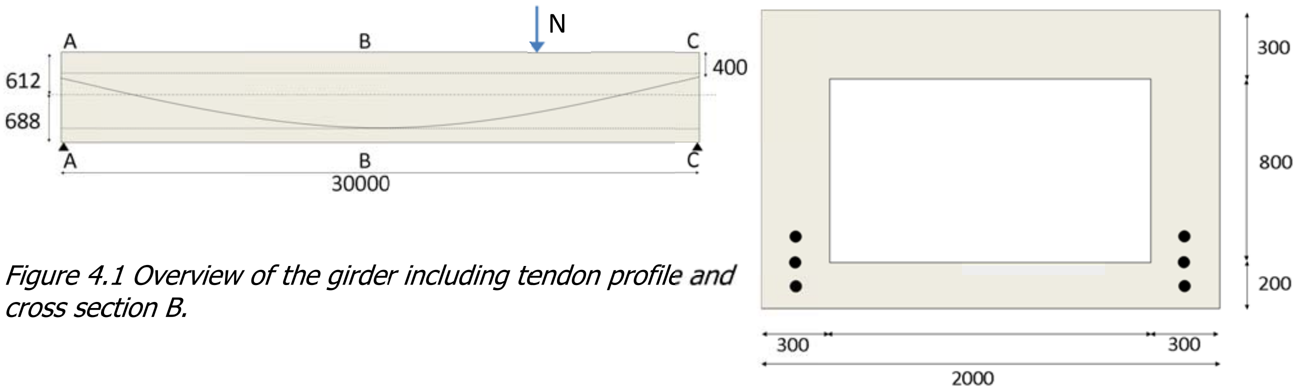


Figure 4.1 Overview of the girder including tendon profile and cross section B.

Parameters:

Density concrete	: ρ = 25 kN/m^3
Variable load (point load, see drawing),	: N = 400 kN (at 7.5 m from support C)
Dimensions girder	: see figure
Distance from centroidal axis to bottom fibre	: 0.688 m
Distance from centroidal axis to top fibre	: 0.612 m
Cross-sectional area girder	: A_c = 1.48 m^2
Moment of inertia	: I_c = 0.3015 m^4
Section modulus (top fibre)	: W_t = 0.493 m^3
Section modulus (bottom fibre)	: W_b = 0.438 m^3
Strength class of concrete	: C45/55
Youngs modulus concrete	: 36000 N/mm^2
Strength class of prestressing steel	: Y1860S7
Initial tensile stress at anchor	: σ_{pmo} = 0.75 * 1860 = 1395 N/mm^2
Elastic modulus prestressing steel	: 195000 N/mm^2

Minimum distance centre of tendon to top or bottom fibre: 100 mm

Partial load factors	: γ_G = 1.2 : γ_Q = 1.5
Creep coefficient	: φ = 2.4
Deformation caused by shrinkage	: ε_{cs} = 0.25 ‰

4a) At t=∞, the variable load of 400 kN should not lead to tensile stresses in cross-section B. Calculate the minimum value of the working prestress force $P_{m\infty}$ required for full prestressing at midspan B, in kN.

4b) Calculate and draw the profile of the prestressing stress, in N/mm^2, in the draped tendons at t=0 across the full beam length taking into account friction losses. Presume that the tendons are prestressed at end A only. Do not take into account wedge slip. Present the results at characteristic points along the tendon profile. *Use general data on page 7 and 8.*

4c) Calculate the time dependent prestress losses due to shrinkage, creep and relaxation in N/mm^2 at t=∞ in cross-section B. The elastic losses in the prestressing steel, caused by the average concrete compressive stress (at the tendon) of -3.5 N/mm^2 are compensated for.
Use general data on page 7 and 8.

4d) The designer has applied A_p= 4800 mm^2. Check for cross section B whether the girder is fully prestressed.

4e) Check whether M_{Rd} > M_{Ed} at midspan B. Assume that the prestressing steel reaches its design value $f_{pd} = f_{p0,1k} / \gamma_s$ = 1522 N/mm^2

General Data

Material properties:

Concrete

Design compressive strength

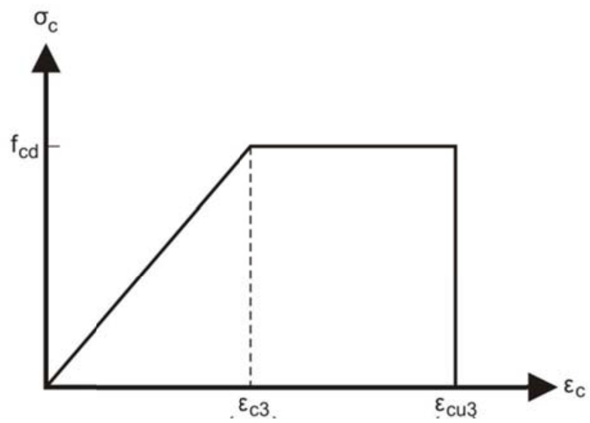
$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}$ with $\alpha_{cc} = 1{,}0$; $\gamma_c = 1{,}5$

Design tensile strength:

$f_{ctd} = \frac{\alpha_{ct} f_{ctk,0,05}}{\gamma_c}$ with $\alpha_{ct} = 1{,}0$; $\gamma_c = 1{,}5$

Concrete strength class C45/55:
compression:
$f_{ck} = 45$ N/mm^2
tension:
$f_{ctk,0,05} = 2{,}7$ N/mm^2

Stress-strain diagram for concrete in compression:



$\varepsilon_{cu3} = 3{,}5$ ‰
$\varepsilon_{c3} = 1{,}75$ ‰

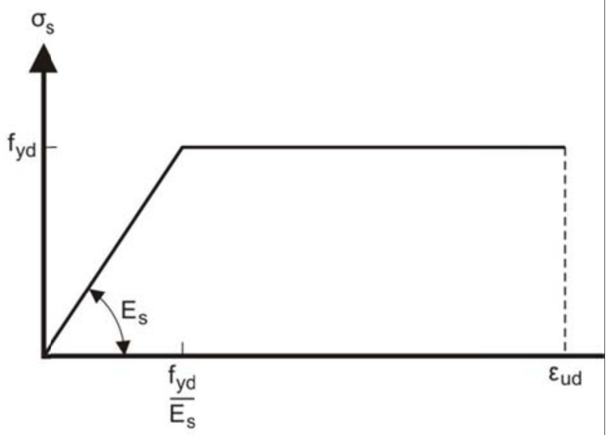
For a rectangular cross-section:

sectional area factor $\alpha = 0{,}75$ ($A = \alpha\, b\, x_u$)
distance factor $\beta = 0{,}39$ ($y = \beta\, x_u$)

Reinforcing steel: B500B:

$f_{yd} = 500 / 1{,}15 = 435$ N/mm^2
bond factor: $\xi_s = 1{,}0$.
$E_s = 200\,000$ N/mm^2

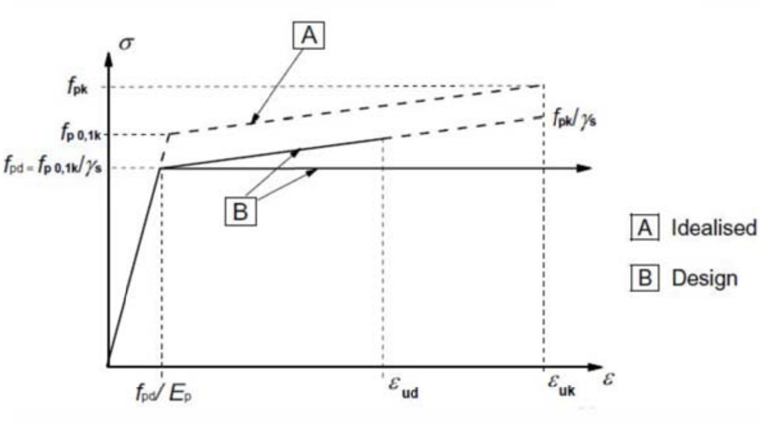
Stress-strain diagram of reinforcing steel



$\varepsilon_{ud} = 45$ ‰

Prestressing steel:

Stress-strain diagram for prestressing steel



Y1860S7:

$f_{pu} = f_{pk} / \gamma_s =$
$= 1860/1{,}1 = 1691$ N/mm^2

$f_{pd} = f_{p0.1k} / \gamma_s =$
$= 1674/1{,}1 = 1522$ N/mm^2

$\varepsilon_{ud} = \varepsilon_{uk} = 35$ ‰

bond factor: $\xi_1 = 0{,}5$

$E_p = 195\,000$ N/mm^2

Mechanical properties of prestressing steel

strength class	type	tensile strength		failure strain	0,1% fractile	permitted tensile stress during stressing	permitted tensile stress initial stress	kink in σ-ε diagram (ULS)	modulus of elasticity
		f_{pk}	f_{pk}/γ_s	ε_{uk}	$f_{p0,1k}$	$\sigma_{p,max}$	σ_{pm0}	f_{pd}	E_p
		MPa	MPa	‰	MPa	MPa	MPa	MPa	GPa
Y1030H	bar	1030	936	35	927	773	773	843	205 or 170
Y1670C	wire	1670	1518	35	1503	1336	1253	1366	205
Y1770C	wire	1770	1609	35	1593	1416	1328	1448	205
Y1860S7	strand	1860	1691	35	1674	1488	1395	1522	195

Prestressing force including frictional loss:

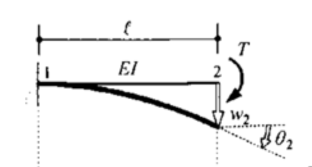
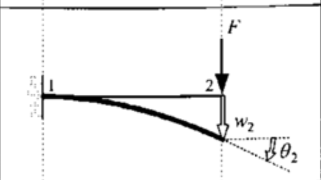
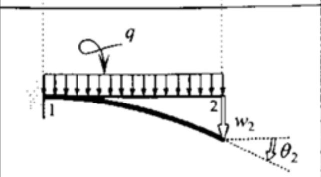
$$P_m(x) = P_m(x=0) \cdot e^{-\mu(\theta + k\,x)}$$

friction coefficient $\mu = 0{,}3$
Wobble-factor $k = 0{,}008$ rad/m

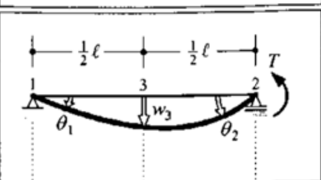
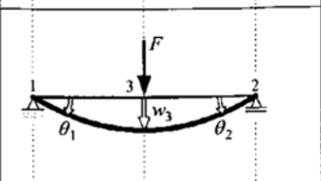
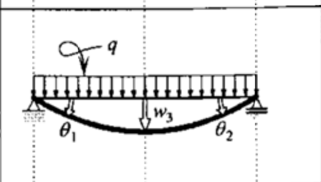
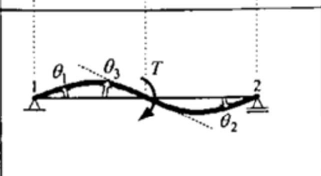
tensile

count

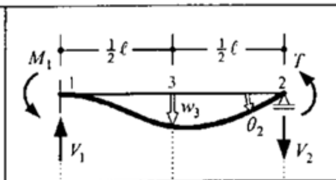
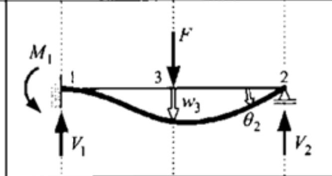
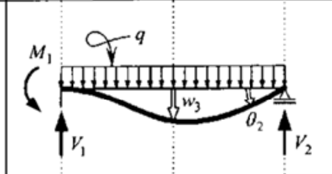
vergeet-mij-nietjes

(1)		$\theta_2 = \dfrac{T\ell}{EI};\quad w_2 = \dfrac{T\ell^2}{2EI}$
(2)		$\theta_2 = \dfrac{F\ell^2}{2EI};\quad w_2 = \dfrac{F\ell^3}{3EI}$
(3)		$\theta_2 = \dfrac{q\ell^3}{6EI};\quad w_2 = \dfrac{q\ell^4}{8EI}$

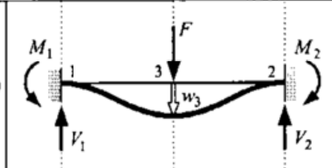
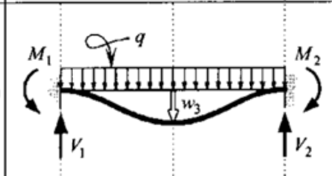
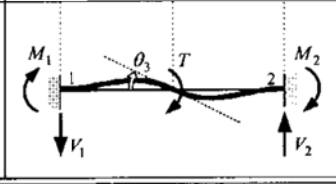
vrij opgelegde ligger (statisch bepaald)

(4)		$\theta_1 = \dfrac{1}{6}\dfrac{T\ell}{EI};\quad \theta_2 = \dfrac{1}{3}\dfrac{T\ell}{EI};\quad w_3 = \dfrac{1}{16}\dfrac{T\ell^2}{EI}$
(5)		$\theta_1 = \theta_2 = \dfrac{1}{16}\dfrac{F\ell^2}{EI};\quad w_3 = \dfrac{1}{48}\dfrac{F\ell^3}{EI}$
(6)		$\theta_1 = \theta_2 = \dfrac{1}{24}\dfrac{q\ell^3}{EI};\quad w_3 = \dfrac{5}{384}\dfrac{q\ell^4}{EI}$
(a)		$\theta_1 = \theta_2 = \dfrac{1}{24}\dfrac{T\ell}{EI};\quad \theta_3 = \dfrac{1}{12}\dfrac{T\ell}{EI};\quad w_3 = 0$

statisch onbepaalde ligger (enkelzijdig ingeklemd)

(7)		$\theta_2 = \dfrac{1}{4}\dfrac{T\ell}{EI};\quad w_3 = \dfrac{1}{32}\dfrac{T\ell^2}{EI}$ $M_1 = \dfrac{1}{2}T;\quad V_1 = V_2 = \dfrac{3}{2}\dfrac{T}{\ell}$
(8)		$\theta_2 = \dfrac{1}{32}\dfrac{F\ell^2}{EI};\quad w_3 = \dfrac{7}{768}\dfrac{F\ell^3}{EI}$ $M_1 = \dfrac{3}{16}F\ell;\quad V_1 = \dfrac{11}{16}F;\quad V_2 = \dfrac{5}{16}F$
(9)		$\theta_2 = \dfrac{1}{48}\dfrac{q\ell^3}{EI};\quad w_3 = \dfrac{1}{192}\dfrac{q\ell^4}{EI}$ $M_1 = \dfrac{1}{8}q\ell^2;\quad V_1 = \dfrac{5}{8}q\ell;\quad V_2 = \dfrac{3}{8}q\ell$

statisch onbepaalde ligger (tweezijdig ingeklemd)

(10)		$w_3 = \dfrac{1}{192}\dfrac{F\ell^3}{EI}$ $M_1 = M_2 = \dfrac{1}{8}F\ell;\quad V_1 = V_2 = \dfrac{1}{2}F$
(11)		$w_3 = \dfrac{1}{384}\dfrac{q\ell^4}{EI}$ $M_1 = M_2 = \dfrac{1}{12}q\ell^2;\quad V_1 = V_2 = \dfrac{1}{2}q\ell$
(b)		$\theta_3 = \dfrac{1}{16}\dfrac{T\ell}{EI};\quad w_3 = 0$ $M_1 = M_2 = \dfrac{1}{4}T;\quad V_1 = V_2 = \dfrac{3}{2}\dfrac{T}{\ell}$

Enkele formules voor prismatische liggers met buigstijfheid EI.
T, F en q zijn belastingen door resp. een koppel, kracht en gelijkmatig verdeelde belasting.
M_i en V_i zijn het buigend moment en de dwarskracht op einddoorsnede i van de ligger ten gevolge van de oplegreacties.

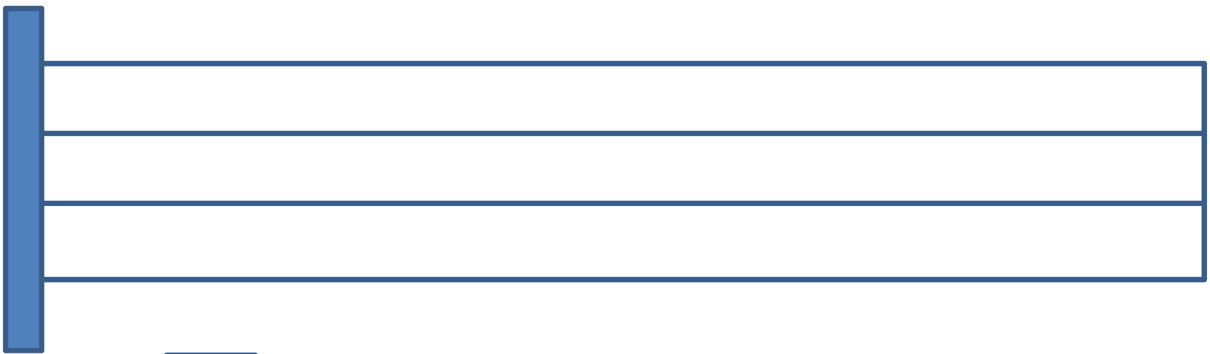
(empty)

Answer sheet: **Name:**

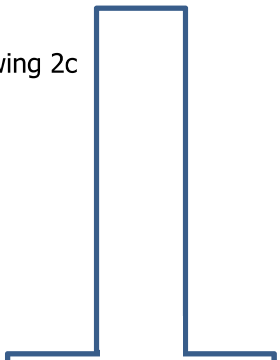
Drawing 2a side view **Studentnumber:**

Drawing 2b side view

Drawing 2b top view



Drawing 2c



Cross-section B and side view (free end)

(empty)

Tentamen CT3150 2012-04-12.

1a Take a strip of a certain width (1m) in x direction and y-direction in the middle of the panel.

Use 'forget-me-nots' (vergeet-me-nietjes) to distribute the load in such a way that $\delta_x = \delta_y$.
(The strips are assumed to deform independently.)

1b $q_t = q_x + q_y$ and

$\delta_x = \delta_y$.

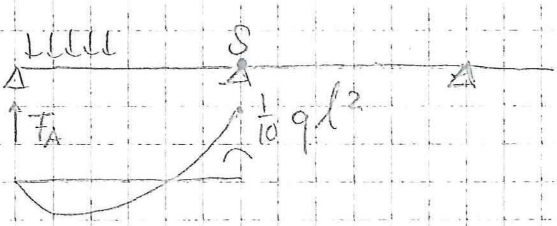
$$\frac{2}{384}\frac{q_x l_x^4}{EI} = \frac{5}{384}\frac{q_y l_y^4}{EI}$$

1c

$$k_x = \frac{5 l_y^4}{2 l_x^4 + 5 l_y^4} = 0{,}872$$

87,2 % in x direction

1d



$q_d = 16 \cdot 1{,}5 + (0{,}235 \cdot 1 \cdot 25) \cdot 1{,}2 =$
$q_d = 31{,}05$ kN/m²

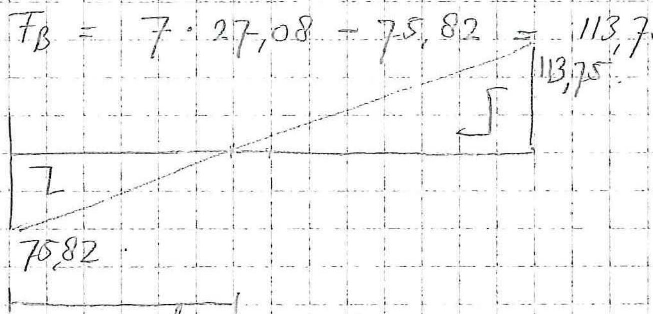
$q_x = 0{,}872 \cdot 31{,}05 = 27{,}08$ kN/m (per meter strip)

$\Sigma M_{S=0}$ (left part) $-F_A \cdot 7 + q \cdot 7 \cdot 3{,}5 - \frac{1}{10} q (7)^2 = 0$.

$$F_A = \frac{3{,}5 \cdot 7 \cdot (27{,}08) - \frac{1}{10} \cdot 7^2 \cdot (27{,}08)}{7}$$

$F_A = 75{,}82$ $(F_A = 0{,}4\, q_x \cdot l)$

$F_B = 7 \cdot 27{,}08 - 75{,}82 = 113{,}75$ $(F_B = 0{,}6\, q_x \cdot l)$



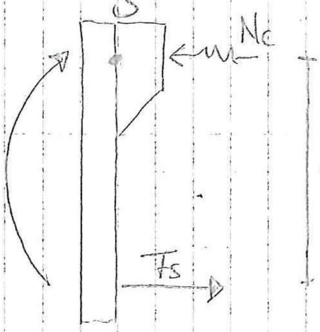
$= 2{,}8$ m

M_{max} $(V=0)$ $\Sigma M = 0$ for left part, at $x = 2{,}8$ m

$-75{,}82 \cdot 2{,}8 + 27{,}08 \cdot 2{,}8 \cdot \frac{1}{2} \cdot 2{,}8 + M_{max} = 0$

$M_{max} = 106{,}14$ kNm/m (strip)

1e



$z = 0{,}9d = 0{,}9 \cdot 200 = 180 \text{ mm}$

$M_{Ed} = 106{,}14 \text{ kNm/m}$

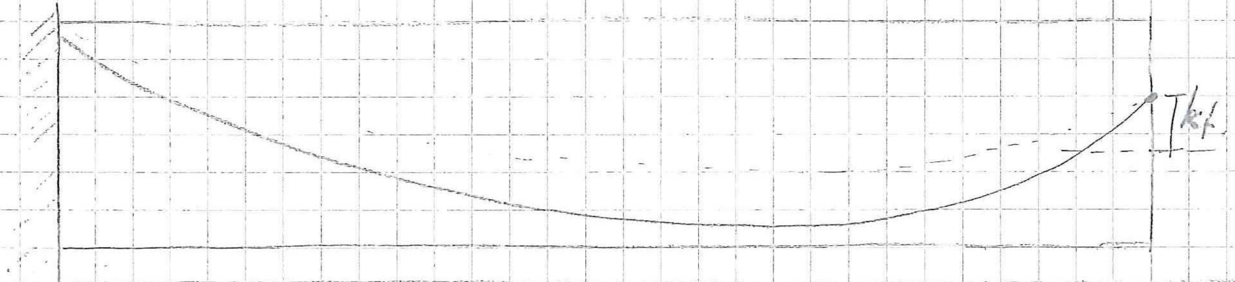
$\Sigma M_S = 0$

$-M_{Ed} + A_s \cdot f_{yd} \cdot z = 0$

$M_{Ed} = A_s \cdot f_{yd} \cdot z$

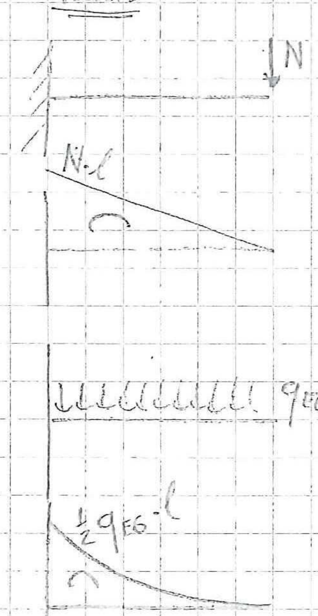
$A_s = \frac{M_{Ed}}{f_{yd} \cdot z} = \frac{106{,}14 \cdot 10^6}{435 \cdot 180} = 1355 \text{ mm}^2/\text{m strip}.$

2a
- at free end of beam in kern area.
- at the top on the fixed side (tension due to loads is in the top fibre)
- curvature upward to balance loads.

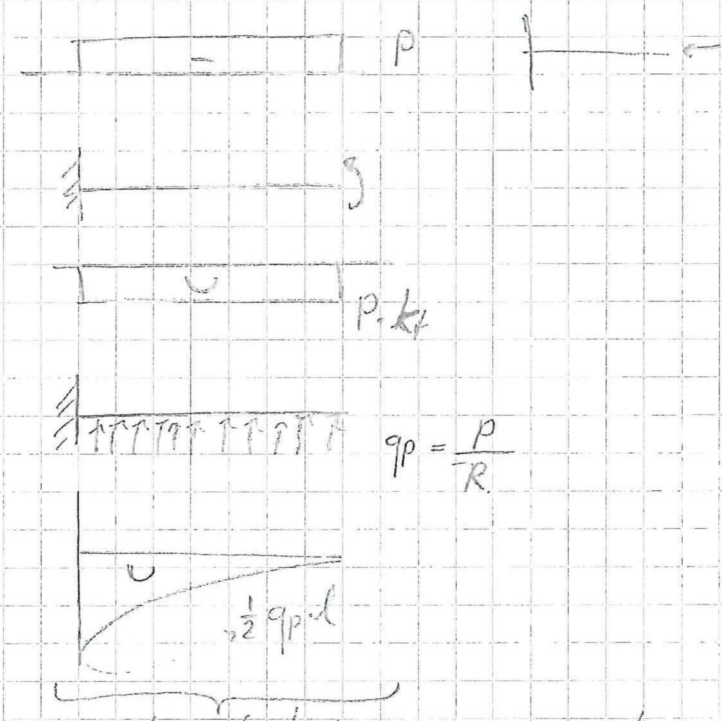


additional information (was not required for answer on exam.

loads.

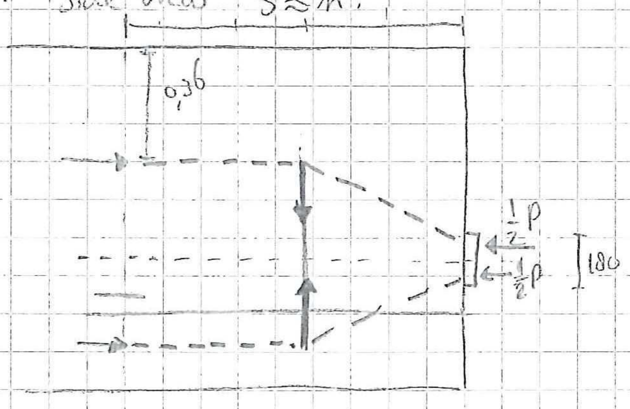


prestressing is preloading.



leads to highest compression at section in top fibre, so lowest P possible (most efficient)

2b. side view $s \approx h$.



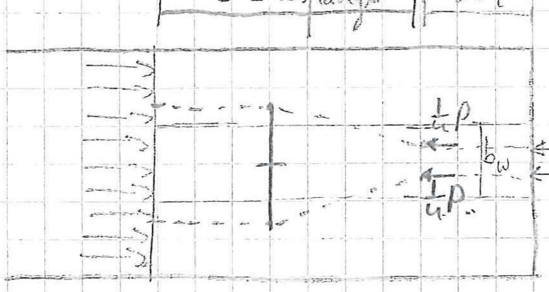
stresses constant distribution.
upper 50% of stresses balances with $\frac{1}{2} P_{(t_0)}$

$\frac{A_c}{2}$ = 0,145 m^2 : b_w = 200 mm
height is 0,725.

force is halfway 0,36

bottom part centre of gravity 122 mm from bottom.

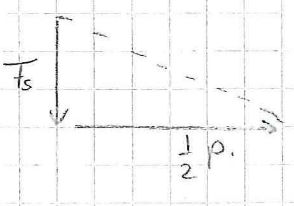
2b. Top view.



→ before ½P is in flange

force distributes horizontally over the flange

2C web.



$h = 21 - 0{,}36 - \frac{1}{4}\,2/80 = 0{,}215$. m.

$\frac{1}{2} s = \frac{1}{2} h = 500$ mm.

$F_s = \frac{215 \cdot \frac{1}{2}P}{500} = \frac{215 \cdot 500.000}{500} = 215.000$ N.

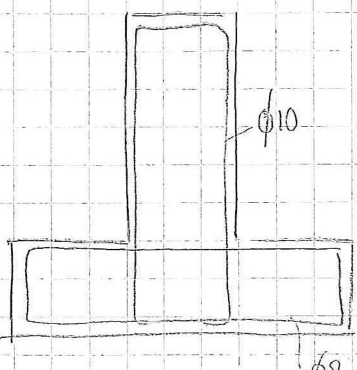
$A_s = \frac{F_s}{\sigma_s} = \frac{215000}{300} = 717 \text{ mm}^2$.

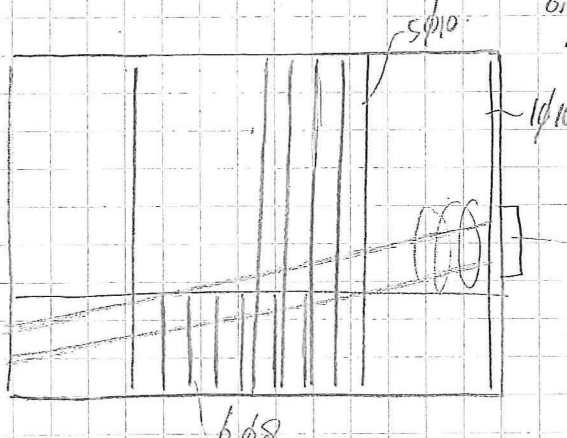
top flange. $F_s = \frac{\left(\frac{1}{4} b_{fl} - \frac{1}{4} b_w\right)}{\frac{1}{2} b_{fl}} \cdot \frac{1}{4} P = 83000$ N.

$A_s = 277 \text{ mm}^2$.

e.g bars ϕ 10. 78 mm^2 web → 10 bars.
5 double leg stirrups.

flange ϕ 10. 4 bars
2 double leg stirrups. } for spreading of forces rebars should be distributed.
better choose ϕ 8. (6 bars).
or add some rebars ϕ10 as practical reinforcement





2d. $V_{Ed} = 50.000 \cdot \underset{\gamma_Q}{1{,}5} + (0{,}29 \cdot 25) \cdot 8_{(m)} \cdot 1000 \cdot \underset{\gamma_G}{1{,}2} = 144600$

$v_{Ed} = \frac{V_{Ed}}{b \cdot d} = \frac{144600}{200 \cdot (1000-60)} = 0{,}77 \ N/mm^2$

$V_{Rd,c} = v_{min} + k_1 \sigma_{cp}$ flexural-shear failure.

$v_{min} = 0{,}035 \ k^{3/2} \sqrt{f_{ck}}$

$k = 1{,}46.$

$f_{ck} = 50 \ N/mm^2.$

$v_{min} = 0{,}437 \ N/mm^2$

$\sigma_{cp} = \frac{1000\,000}{0{,}29 \cdot 10^6}$

$k_1 = 0{,}15$

$v_{Rdc} = 0{,}437 + 0{,}15 \cdot \quad = 0{,}95 \ N/mm^2$

$v_{Ed} < v_{Rdc}$ $0{,}77 < 0{,}94 \ N/mm^2$

or unity check:

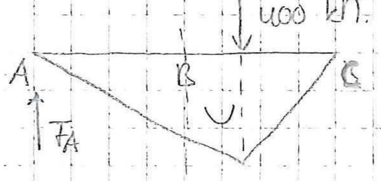
$\frac{v_{Ed}}{v_{Rdc}} = \frac{0{,}77}{0{,}95} = 0{,}81 < 1$

2 a) A Bolt Pocket

B Key Segment

[redacted]

4.a variable load (SLS).



$\Sigma M_C = 0$

$+400 \cdot 7{,}5 - F_A \cdot 30 = 0$

$F_A = 100$

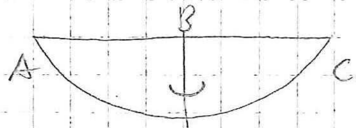
$F_C = 300$

$\Sigma M_B = 0$

$-F_A \cdot 15 + M_B = 0$

$M_B = 1500$ kNm.

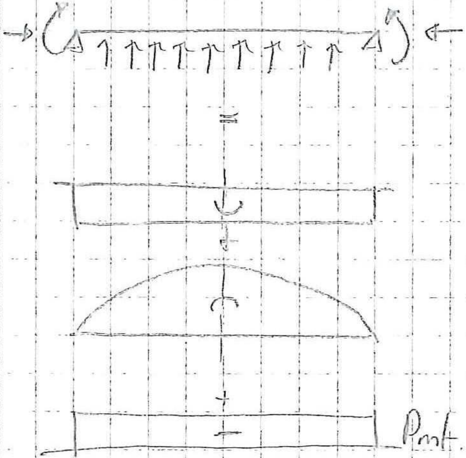
self weight



$\frac{1}{8} q l^2 = \frac{1}{8}(1{,}48 \cdot 25) \cdot 30^2$

$M_{B,(SW)} = 4162{,}5$ kNm.

prestressing = preloading



$P_{mt} \cdot 0{,}212$

$M_B(P) = \frac{1}{8} q_P l^2$ $\qquad q_P = \frac{8 P_{mt} \cdot f}{l^2}$

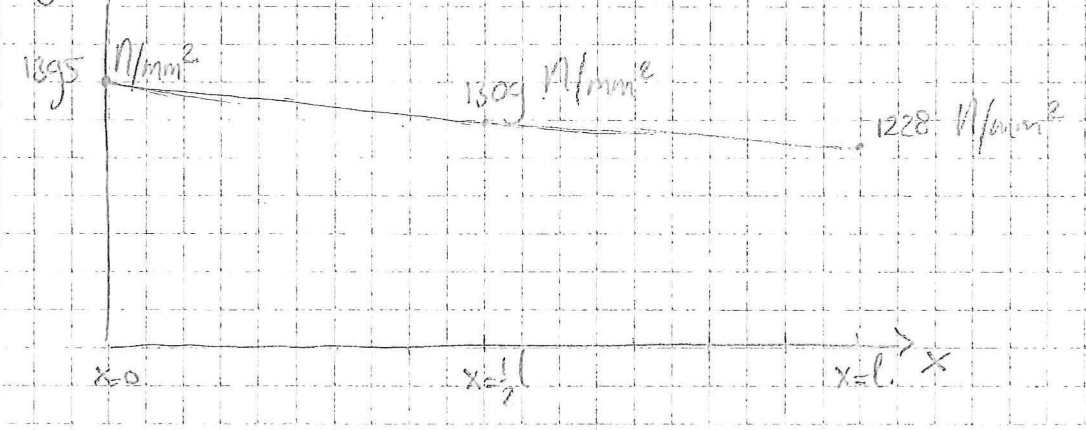
$M_{B}(P) = P_{mt} \cdot f$ $\qquad f = 0{,}7$ m $= 700$ mm.

$$-\frac{P_{mt}}{A_c} + \frac{P_{mt} \cdot 0{,}212}{W_b} - \frac{P_{mt} \cdot 0{,}7}{W_b} + \frac{4162{,}5}{W_b} + \frac{1500}{W_b} \leq 0$$

$$-P_{mt}\left(\frac{1}{1{,}48} + \frac{0{,}488}{0{,}438}\right) \leq -\frac{4162{,}5}{0{,}438} - \frac{1500}{0{,}438}$$

$P_{mt} \geq 7223.1$ kN

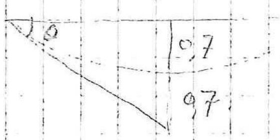
4b



$\sigma_{pm}(x) = \sigma_{pm}(x=0) \cdot e^{-\mu(\theta + kx)}$

$\mu = 0,3$
$k = 0,008$ rad/m.

$\tan\theta = \frac{2 \cdot 0,7}{15}$ $\theta = 0,093$ at $x = \frac{1}{2}l$.



$x = \frac{1}{2}l$ $e^{-\mu(\theta + k \cdot 15)} = 0,938$ $\sigma = 1309\ N/mm^2$
$x = l$ $e^{-\mu(2\theta + k \cdot 30)} = 0,880$ $\sigma = 1228\ N/mm^2$

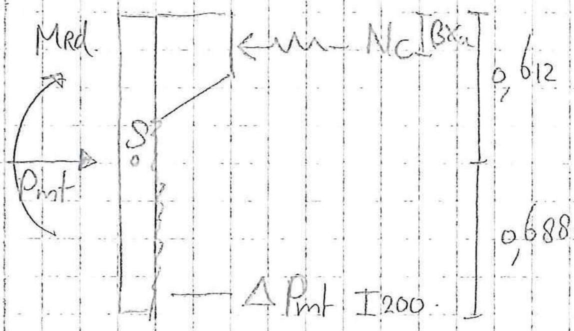
4c) Shrinkage:
$\varepsilon_{cs} = 0,25$ ‰
$\Delta\sigma_p = E \cdot \varepsilon_{cs} = 195000 \cdot 0,25 \cdot 10^{-3} = 48,75\ N/mm^2$

Creep:
$\varphi = 2,4$ so creep deformations are $2,4 \cdot$ elastic deformation
elastic deformation:
$\varepsilon_c = \frac{\sigma}{E_c} = \frac{-3,5}{36000} = 0,097$ ‰.

$\Delta\sigma_p = E \cdot 2,4 \cdot 0,097 \cdot 10^{-3} = 45,5\ N/mm^2$
Relaxation.
$\Delta\sigma_{pr} = \sigma_{pi} \cdot 0,66 \cdot \rho_{1000} \cdot e^{(9,1\mu)} \left(\frac{500.000}{1000}\right)^{0,75(1-\mu)} \cdot 10^{-5}$
$\mu = \frac{\sigma_{pi}}{f_{pk}} = \frac{1309}{1860} = 0,7$ (mean stress at $t=0$)
$\Delta\sigma_{pr} = 51,9\ N/mm^2$
$\Delta\sigma_{pr}(sh+cr) = 41,5\ N/mm^2$

Total time dependent losses: $48,75 + 45,5 + 41,5 = 135,8\ N/mm^2$
4d) $t = \infty$ In B stress is $1309 - 135,8 = 1173\ N/mm^2$
Applied $4800\ mm^2$
Prestress force applied $\overset{N/mm^2}{1173} \cdot \overset{mm^2}{4800} = 5632\ kN$.
Required (Q 4a) 7223,1 kN
the girder is not fully prestressed

4e



$\Sigma F_h = 0$

$P_{mt} + \Delta P_{mt} - N_c = 0$

$5632 \cdot 10^3 + (1522 - 117{,}8) \cdot 4800 - 30 \cdot 0{,}75 \cdot 2000 \cdot X_u = 0$

$x_u = 162$ mm

$\Sigma M_S = 0$

$\Delta P_{mt} \cdot (0{,}688 - 0{,}2) + 30 \cdot 0{,}75 \cdot 2000 \cdot 162 \cdot \left(0{,}612 - 0{,}39 \cdot \frac{162}{1000}\right) - M_{Rd} = 0$

$1675{,}2\ kN\ (0{,}488) + 7290\ kN \cdot (0{,}549) = M_{Rd}$

$M_{Rd} = 4818{,}4$ kNm

$M_{Ed} = 1{,}2 \cdot \frac{1}{8} \cdot (1{,}48 \cdot 25) \cdot 30^2 + 1{,}5 \cdot 1500 - 1{,}0 \cdot 5632 \cdot 0{,}7$
$+ 1{,}0 \cdot 5632 \cdot 0{,}212$

$M_{Ed} = 4496{,}6$ kNm

$M_{Ed} = 4496{,}6$ kNm $<$ $M_{Rd} = 4818{,}4$ kNm

sufficient

Faculty of Civil Engineering and Geosciences

Write down on each examination sheet:
CT3150 or CIE 3150
Student number:
Name:

Examination CT3150/CIE3150 Concrete Structures 2

2012 June 27th, 9:00 - 12:00 h.

S.A.A.M. Fennis

General instructions:

- This exam consists of 5 questions and an equation form.
- Write your name, student number and course code (CT3150 or CIE3150) on each answer sheet in the upper right corner.
- Place only one question on each sheet (front and back).
- Use only the paper provided. Ask for additional sheets, if required.
- **Show all work to arrive at your answer**: Include calculations and sketches.
- When reviewing the work, also neatness and readability are taken into account.
- Per question a time indication is presented to guide you through the exam.
- It is allowed to use a (graphical) calculator and 1 A4 (one side) with your own notes.
- The use of mobile phones (or any other devices than specified above) is not allowed during the exam. Please remove them from your desk.
- Please keep your student card available during the exam.

Best of Luck!

Question 1 Slabs		40 min
Question 2 Crack width		30 min
Question 3 Tunnels		30 min
Question 4 Prestress (design)		40 min
Question 5 Prestress (capacity)		40 min
	Total	**180 min**

(empty)

Question 1 Slabs (40 min)

A flat slab floor consisting of two panels supported by six columns is regarded, see figure 1.1. Reinforcement strips in the slab (gray areas) act as beams. The slab is loaded by a uniform load q_{Qk}. The slab is part of a braced structure.

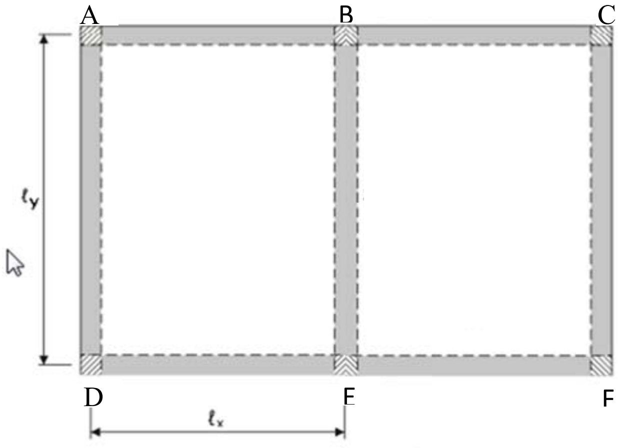


Figure 1.1 Overview of the slab

Spans	: l_x = 9 m
	: l_y = 10 m
Total depth of the slab	: h = 420 mm
	d_x = 365 mm
	d_y = 385 mm
Internal lever arm	: z = (0.9 * d)
Reinforcement ratio at column B:	ρ_{lx} = 1.1%
	: ρ_{ly} = 1.3%
Dimensions column	: 300*300 mm
Concrete strength class	: C45/55 *(see also page 8)*
Density concrete	: ρ = 25 kN/m^3
Reinforcement class B500B	: f_{yd} = 500 / 1.15 = 435 N/mm^2
Variable load	: q_{Qk} = 4.0 kN/m^2
Partial load factors	: γ_G = 1.2 : γ_Q = 1.5

1a) Support reactions can be calculated by subdividing the slab surface into different parts, depending on the support type (rigid line supports or rigid line support with additional rotational fixity). Draw on scale how the parts of the load are transferred to the reinforcement strips in a top view of the slab. Add the mechanical schematization of reinforcement strips ABC and BE including the loads and the supports.

1b) Calculate the design force carried by column strip BE to column B

1c) Explain the difference between a braced structure (non-sway frame) and unbraced structure (sway frame) and the influence of those systems on the design load for column B.

1d) Assume a design load of 1100 kN on column B. Perform the punching shear check of the slab near column B. Use Equation 1.1 to determine β, where u_1* is the reduced basic control perimeter for an edge column, see figure 1.2. (The punching reinforcement does not have to calculated.)

$$\beta = \frac{u_1}{u_1^*} \qquad \text{E1.1}$$

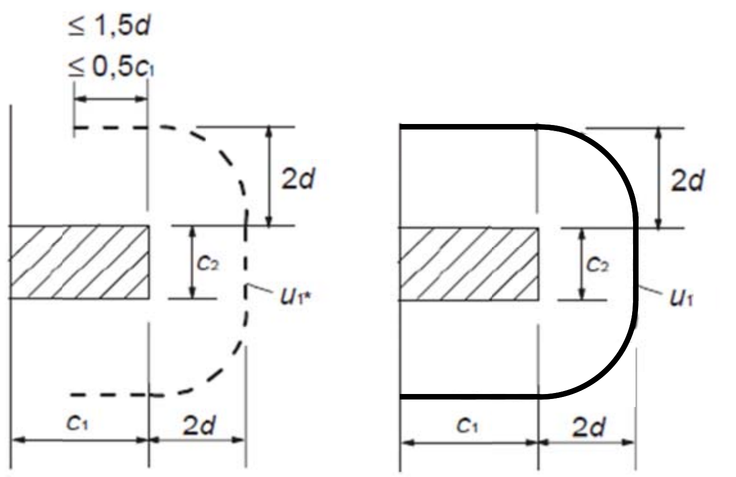


Figure 1.2 Reduced basic control diameter u_1* *for an edge column.*

Question 2 Crack width control (30 min)

Crack formation in a reinforced tensile bar.

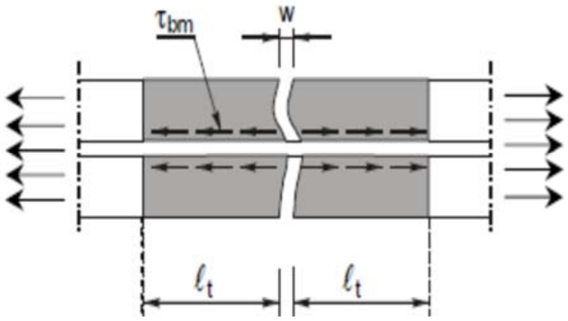


Figure 2.1 Overview of the reinforced tensile bar and location of cross sections A and B.

The force transferred in cross section B is $N_{tot} = E_{cm}A_c\left(1+\alpha_e\rho\right)\varepsilon_c$, with $\alpha_e = \frac{E_s}{E_{cm}}$ and $\rho = \frac{A_s}{A_c}$. In cross section A the concrete in the reinforced tensile bar is cracked.

2a) Derive from the equilibrium of forces working on a piece of concrete between cross section A and cross section B that the bond stresses τ_{bm} are transferred over a length: $l_t = \frac{A_c f_{ctm}}{\tau_{bm}\sum\pi\phi}$. Include short comments for each step of the derivation.

Parameters Beam:

Cross section	b = 240 mm
	h = 500 mm
Cover	c = 20 mm
Stirrups	ϕ = 10 mm
Reinforcement: 3 bars	ϕ = 20 mm
Mean tensile strength	f_{ctm} = 2.2 N/mm^2
Bond stress	$\tau_{bm} = 2.0f_{ctm}$

2b) Calculate the transfer length l_t for the specified beam subjected to bending in the SLS (height of compressive zone = 175 mm). The effective height $h_{c,ef}$ is the lesser of 2.5(h-d), (h-x)/3.

2c) Present the theoretical values of the minimum and maximum crack spacing in the stabilized cracking stage. Give a short explanation (2 sentences) combined with a sketch to explain these values. Give one reason why actual crack spacing could deviate from the theoretically determined spacing.

Question 3 Tunnels (30 min)

3a) Fill in the correct answers for the given descriptions of parts of the Tunnel Boring Machine (TBM). Choose from: Conveyor, Cutter head, Erector, Gantry, Jack, Rubber tail seal, Shield.

1. Is used to lift a segment in the TBM and place this segment in its final position
2. Pushes the TBM forwards and is used for steering
3. Conical steel cylinder that protects the TBM until the lining is placed
4. Is digging the soil by rotation
5. Moves elements from the back of the TBM to the front
6. Helps to remove soil from the pressure room
7. Prevents soil, grout and/or water from gaining access from the sides

3b) Describe (sketch) two different methods to create curves in the alignment of a tunnel.

When the segments are being placed in their final position the TBM exerts forces on the concrete segments as shown in Figure 3.1. The next questions regard the introduction of forces into the concrete.

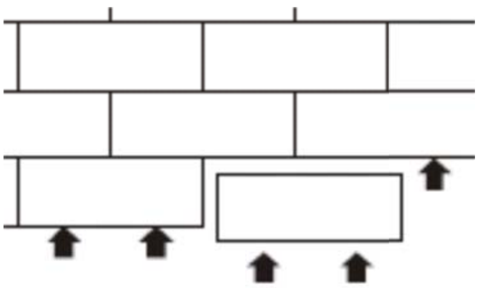


Figure 3.1 Top view of tunnel segments.

3c) Multiple choice question: write down A, B, C or D.

Statement 1: The required amount of splitting reinforcement depends on the magnitude of the force
Statement 2: The required amount of splitting reinforcement does not depend on the location of the forces.

A: Statement 1 and 2 are true
B: Statement 1 is true and 2 is false
C: Statement 1 is false and 2 is true
D: Statement 1 and 2 are false

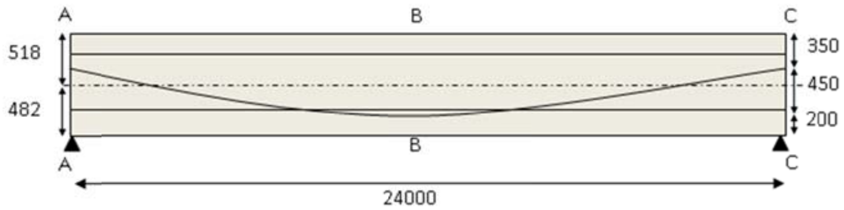
3d) To move the TBM forward, forces are introduced to the segments every 1600 mm on an area of 400*400 mm. On each concrete segment (size: 1600*3200 mm) two forces of each F_{Ed}= 4000 kN are exerted, see figure 3.1.

Draw the strut-and-tie model for the introduction of the forces in a top view of one tunnel segment. Assume a constant stress distribution at the opposite edge of the segment.

3e) Calculate the splitting reinforcement required for the introduction of the forces.

Question 4 Prestress (design) (40 min)

An I-girder of a bridge with a span of 24 meter (see figure 4.1) is fully prestressed by three draped tendons. The tendon profiles are shown in the figure. The fictitious tendon is positioned 200 mm from the bottom at midspan B, and 350 mm from the top at cross sections A and C. The prestressing is applied with post-tensioned steel and anchored at the end supports A and C.



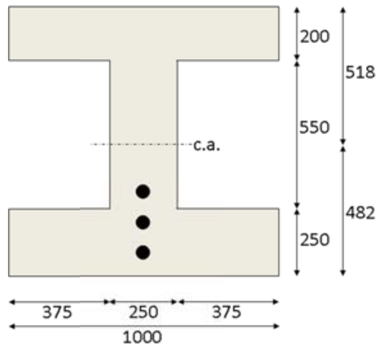


Figure 4.1 Overview of the girder including tendon profile and cross section B.

Parameters:

Density concrete	: ρ = 25 kN/m^3
Variable load	: q_{Qk} = 11 kN/m
Dimensions girder	: see figure
Distance from centroidal axis to bottom fibre	: 0.482 m
Distance from centroidal axis to top fibre	: 0.518 m
Cross-sectional area girder	: A_c = 0.5875 m^2
Moment of inertia	: I_c = 0.00725 m^4
Section modulus (top fibre)	: W_t = 0.140 m^3
Section modulus (bottom fibre)	: W_b = 0.150 m^3
Strength class of concrete	: C45/55 *(see also page 8 and 9)*
Youngs modulus concrete	: 36000 N/mm^2
Strength class of prestressing steel	: Y1860S7 *(see also page 8 and 9)*
Cross-section of one strand	: 140 mm^2
Initial tensile stress at anchor	: σ_{pmo} = 0.75 * 1860 = 1395 N/mm^2
Elastic modulus prestressing steel	: 195000 N/mm^2
Partial load factors	: γ_G = 1.2 : γ_Q = 1.5

4a) Calculate the minimum and maximum value of the initial prestress force P_{m0} [kN] for cross section B to fully prestress the girder. Governing situations to be calculated are: 1) no tensile stresses at t=∞ and 2) maximum concrete compressive stress at t=0. Assume total losses of 20%.

4b) How many strands do you advice to be applied in each tendon unit (available tendon units: 6-3, 6-4, 6-7 or 6-12). Explain why and what it means for your prestressing force and/or stresses at t=0.

4c) Assume that three tendon units are applied with each ten strands. Calculate the average loss per tendon due to elastic deformation ΔP_{el} [kN] by using Equation 4.1.

$$\Delta P_{el} = \frac{n-1}{2} P_m \frac{E_p A_p}{E_c A_c} \qquad \text{E4.1}$$

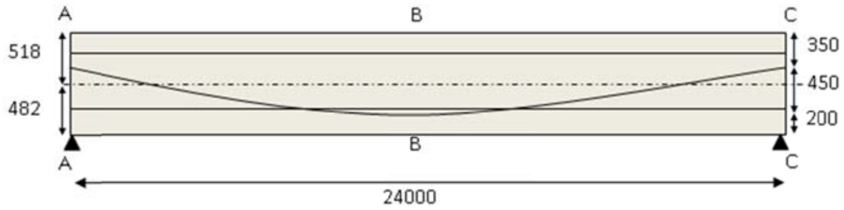
P_m is the prestressing force per tendon

A_p is the cross sectional area per tendon

4d) Make a table of the elastic losses of each tendon. How can these losses be compensated?

Question 5 Prestress (capacity) (40 min)

The I-girder of **question 4**, with a span of 24 meter (see figure 5.1) is fully prestressed by three draped tendons. The tendon profiles are shown in the figure. The fictitious tendon is positioned 200 mm from the bottom at midspan B, and 350 mm from the top at cross sections A and C. The prestressing is applied with post-tensioned steel and anchored at the end supports A and C.



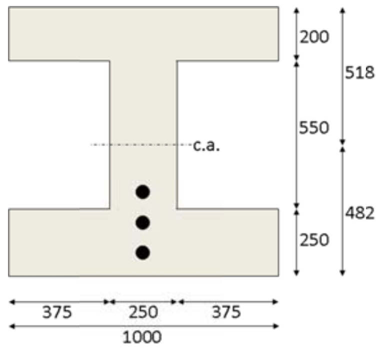


Figure 5.1 Overview of the girder including tendon profile and cross section B.

Parameters:

Density concrete	: ρ = 25 kN/m^3
Variable load	: q_{Qk} = 11 kN/m
Dimensions girder	: see figure
Distance from centroidal axis to bottom fibre	: 0.482 m
Distance from centroidal axis to top fibre	: 0.518 m
Cross-sectional area girder	: A_c = 0.5875 m^2
Moment of inertia	: I_c = 0.00725 m^4
Section modulus (top fibre)	: W_t = 0.140 m^3
Section modulus (bottom fibre)	: W_b = 0.150 m^3
Strength class of concrete	: C45/55 *(see also page 8 and 9)*
Youngs modulus concrete	: 36000 N/mm^2
Strength class of prestressing steel	: Y1860S7 *(see also page 8 and 9)*
Cross-section of one strand	: 140 mm^2
Initial tensile stress at anchor	: σ_{pmo} = 0.75 * 1860 = 1395 N/mm^2
Elastic modulus prestressing steel	: 195000 N/mm^2
Partial load factors	: γ_G = 1.2 : γ_Q = 1.5

Three tendon units are applied with each ten strands (A_p=4200 mm^2). At t=∞ the prestressing force $P_{m,\infty}$ is assumed to be 4700 kN and the concrete compressive zone x_u is assumed to be 330 mm high for a rectangular stress strain relationship (λ=0.8 and η=1.0).

5a) Draw the strain distribution over the height of cross section B in combination with the equilibrium between external and internal forces in cross section B. Use the rectangular stress strain relationship and mark important points/values.

5b) Determine from the equilibrium the stress in the prestressing steel at ultimate bending moment capacity in cross section B.

5c) Calculate the total strain in the prestressing steel by using the cross sectional strain distribution for a concrete compressive zone of 330 mm high.

5d) Compare the answers from question 5b and 5c in the stress strain diagram for prestressing steel Y1860S7 (drawing) and explain why the calculation is correct/incorrect.

5e) Check whether $M_{Rd} > M_{Ed}$ at midspan B.

General Data

Material properties:

Concrete

Design compressive strength

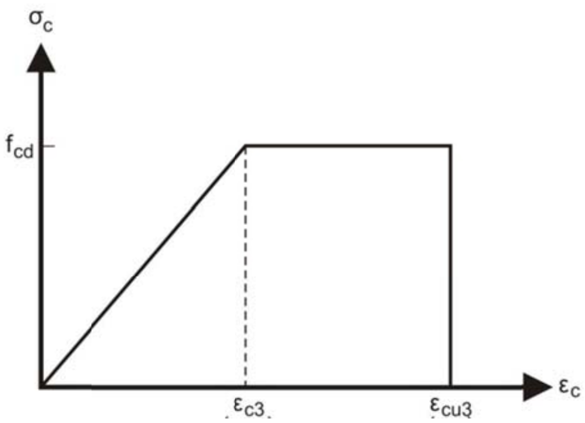
$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}$$ with $\alpha_{cc} = 1,0$; $\gamma_c = 1,5$

Design tensile strength:

$$f_{ctd} = \frac{\alpha_{ct} f_{ctk,0,05}}{\gamma_c}$$ with $\alpha_{ct} = 1,0$; $\gamma_c = 1,5$

Concrete strength class C45/55:
compression:
$f_{ck} = 45$ N/mm^2
tension:
$f_{ctk,0,05} = 2,7$ N/mm^2

Bi-linear stress-strain diagram for concrete in compression:



$\varepsilon_{cu3} = 3,5$ ‰
$\varepsilon_{c3} = 1,75$ ‰

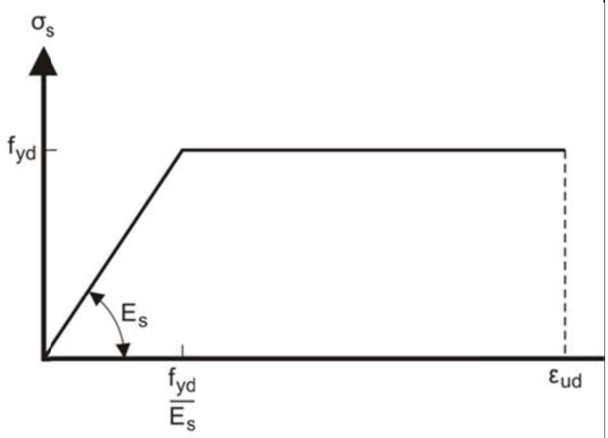
For a rectangular cross-section:

sectional area factor $\alpha = 0,75$ ($A = \alpha\, b\, x_u$)
distance factor $\beta = 0,39$ ($y = \beta\, x_u$)

Reinforcing steel: B500B:

$f_{yd} = 500 / 1,15 = 435$ N/mm^2
bond factor: ξ_s = **1**,0.
$E_s = 200\ 000$ N/mm^2

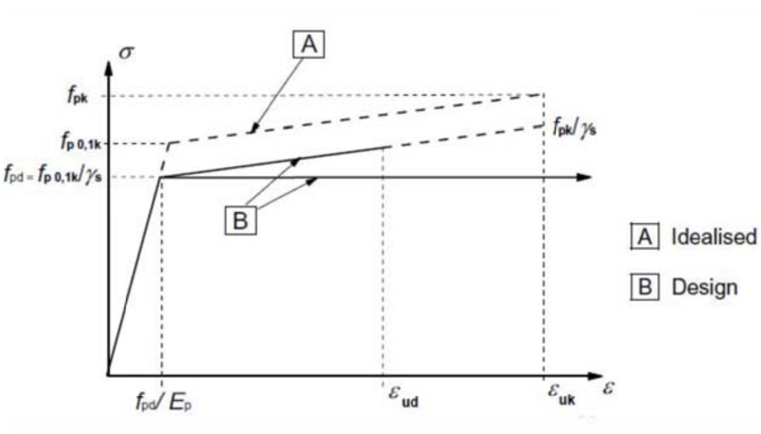
Stress-strain diagram of reinforcing steel



$\varepsilon_{ud} = 45$ ‰

Prestressing steel:

Stress-strain diagram for prestressing steel



Y1860S7:

$f_{pu} = f_{pk} / \gamma_s =$
$= 1860/1{,}1 = 1691$ N/mm²

$f_{pd} = f_{p0,1k} / \gamma_s =$
$= 1674/1{,}1 = 1522$ N/mm²

$\varepsilon_{ud} = \varepsilon_{uk} = 35$ ‰

bond factor: $\xi_1 = 0{,}5$

$E_p = 195\,000$ N/mm²

Mechanical properties of prestressing steel

strength class	type	tensile strength		failure strain	0,1% fractile	permitted tensile stress		kink in σ-ε diagram (ULS)	modulus of elasticity
						during stressing	initial stress		
		f_{pk}	f_{pk}/γ_s	ε_{uk}	$f_{p0,1k}$	$\sigma_{p,max}$	σ_{pm0}	f_{pd}	E_p
		MPa	MPa	‰	MPa	MPa	MPa	MPa	GPa
Y1030H	bar	1030	936	35	927	773	773	843	205 or 170
Y1670C	wire	1670	1518	35	1503	1336	1253	1366	205
Y1770C	wire	1770	1609	35	1593	1416	1328	1448	205
Y1860S7	strand	1860	1691	35	1674	1488	1395	1522	195

Prestressing force including frictional loss:

$$P_m(x) = P_m(x=0) \cdot e^{-\mu(\theta + k\,x)}$$

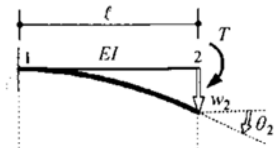
friction coefficient $\mu = 0{,}3$
Wobble-factor $k = 0{,}008$ rad/m

tensile

count

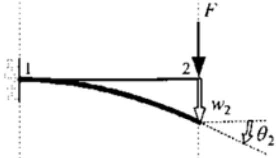
vergeet-mij-nietjes

(1)



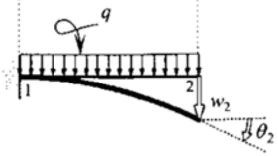
$\theta_2 = \frac{T\ell}{EI}$; $w_2 = \frac{T\ell^2}{2EI}$

(2)



$\theta_2 = \frac{F\ell^2}{2EI}$; $w_2 = \frac{F\ell^3}{3EI}$

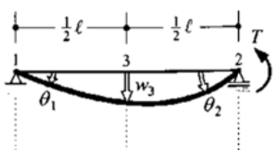
(3)



$\theta_2 = \frac{q\ell^3}{6EI}$; $w_2 = \frac{q\ell^4}{8EI}$

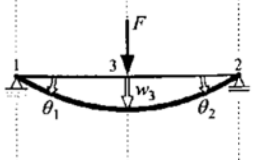
vrij opgelegde ligger (statisch bepaald)

(4)



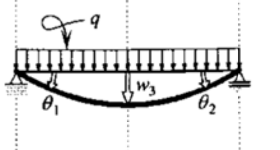
$\theta_1 = \frac{1}{6}\frac{T\ell}{EI}$; $\theta_2 = \frac{1}{3}\frac{T\ell}{EI}$; $w_3 = \frac{1}{16}\frac{T\ell^2}{EI}$

(5)



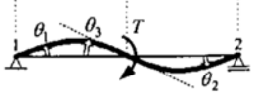
$\theta_1 = \theta_2 = \frac{1}{16}\frac{F\ell^2}{EI}$; $w_3 = \frac{1}{48}\frac{F\ell^3}{EI}$

(6)



$\theta_1 = \theta_2 = \frac{1}{24}\frac{q\ell^3}{EI}$; $w_3 = \frac{5}{384}\frac{q\ell^4}{EI}$

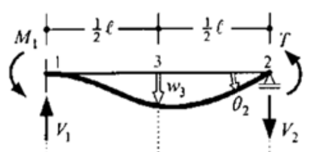
(a)



$\theta_1 = \theta_2 = \frac{1}{24}\frac{T\ell}{EI}$; $\theta_3 = \frac{1}{12}\frac{T\ell}{EI}$; $w_3 = 0$

statisch onbepaalde ligger (enkelzijdig ingeklemd)

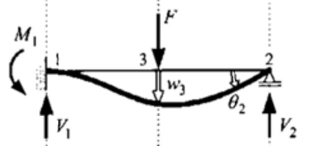
(7)



$\theta_2 = \frac{1}{4}\frac{T\ell}{EI}$; $w_3 = \frac{1}{32}\frac{T\ell^2}{EI}$

$M_1 = \frac{1}{2}T$; $V_1 = V_2 = \frac{3}{2}\frac{T}{\ell}$

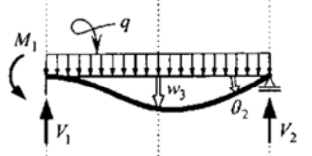
(8)



$\theta_2 = \frac{1}{32}\frac{F\ell^2}{EI}$; $w_3 = \frac{7}{768}\frac{F\ell^3}{EI}$

$M_1 = \frac{3}{16}F\ell$; $V_1 = \frac{11}{16}F$; $V_2 = \frac{5}{16}F$

(9)

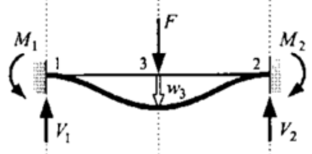


$\theta_2 = \frac{1}{48}\frac{q\ell^3}{EI}$; $w_3 = \frac{1}{192}\frac{q\ell^4}{EI}$

$M_1 = \frac{1}{8}q\ell^2$; $V_1 = \frac{5}{8}q\ell$; $V_2 = \frac{3}{8}q\ell$

statisch onbepaalde ligger (tweezijdig ingeklemd)

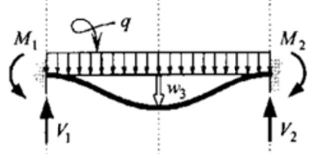
(10)



$w_3 = \frac{1}{192}\frac{F\ell^3}{EI}$

$M_1 = M_2 = \frac{1}{8}F\ell$; $V_1 = V_2 = \frac{1}{2}F$

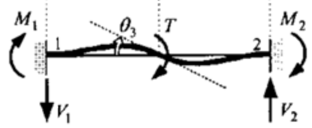
(11)



$w_3 = \frac{1}{384}\frac{q\ell^4}{EI}$

$M_1 = M_2 = \frac{1}{12}q\ell^2$; $V_1 = V_2 = \frac{1}{2}q\ell$

(b)



$\theta_3 = \frac{1}{16}\frac{T\ell}{EI}$; $w_3 = 0$

$M_1 = M_2 = \frac{1}{4}T$; $V_1 = V_2 = \frac{3}{2}\frac{T}{\ell}$

Enkele formules voor prismatische liggers met buigstijfheid EI.
T, F en q zijn belastingen door resp. een koppel, kracht en gelijkmatig verdeelde belasting.
M_i en V_i zijn het buigend moment en de dwarskracht op einddoorsnede i van de ligger ten gevolge van de oplegreacties.

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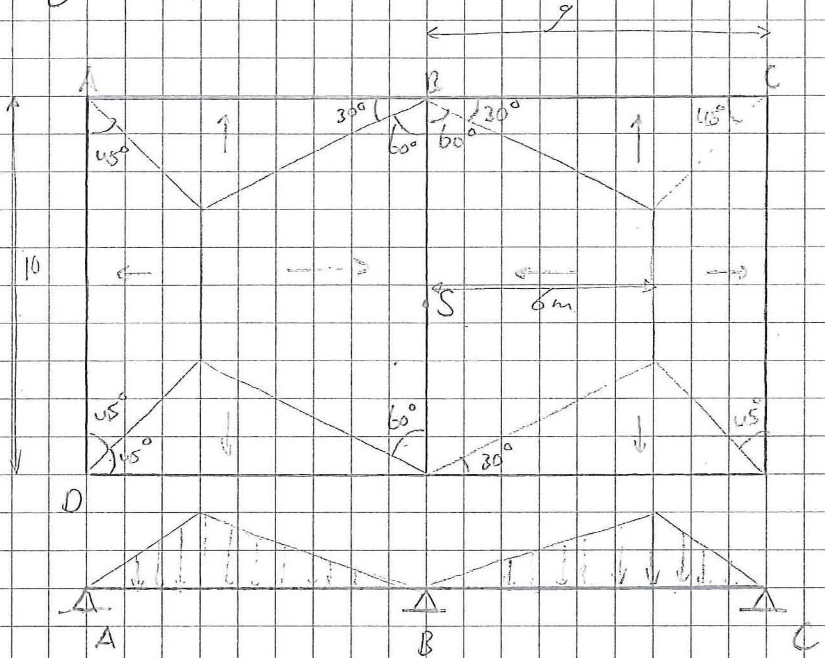
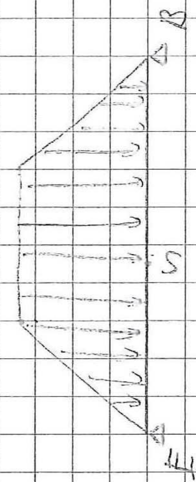
Exam 2012 June 27th

1a



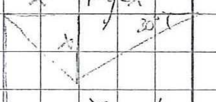


1b $h = 420$ mm

Self weight $0{,}42 \cdot 25 = 10{,}5$ kN/m²

Variable load 4 kN/m²

Design load: $10{,}5 \cdot 1{,}2 + 4 \cdot 1{,}5 = 18{,}6$ kN/m²



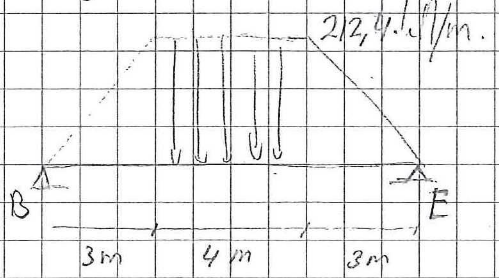
$\frac{x}{9-x} = \tan 30°$

$x = 3{,}29$

$y = 5{,}71$

For the middle part of the 'beam' (denoted S) this works over 2·5.71 meter width of the floor

Design load 'beam': $18{,}6 \cdot 11{,}42 = 212{,}4$ kN/m (maximum S)



Support reaction B:

$\frac{212{,}4 \cdot 7\text{ m}}{2} = 743{,}4$ kN.

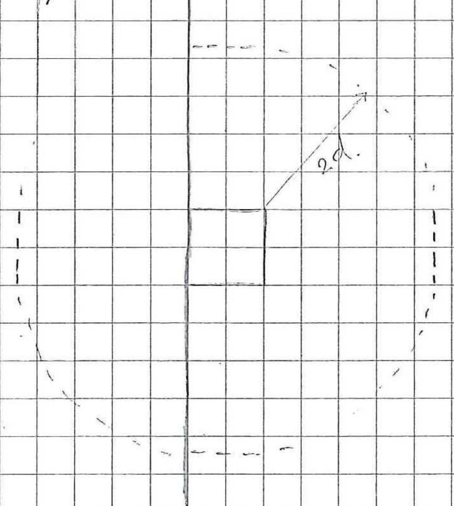
(Note that this is part of the column force since ABC is also carrying load to column B)

1c A braced structure is a structure where the lateral stability (horizontal forces) is provided by stability elements such as shear walls or

An unbraced structure is a structure in which the stability is provided by frame action between the slabs and the columns. This causes moments to occur in the connection between column and slab, wherefor it should be taken into account in the punching shear control. In Eurocode this is done increasing the β factor

1d 300 x 300 mm columns

$h = 420$ mm
$d_x = 365$ mm
$d_y = 385$ mm
$d_{eff} = 375$ mm.



$u_1 = 3 \cdot c + \pi \cdot (2d)$

$= 3 \cdot 300 + \pi \cdot 2 \cdot 375$

$u_1 = 3256,2$

$u_1^* \Rightarrow$ side distance $1,5\, d = 562,5$ mm
$0,5\, c = 150$ mm governing

$u_1^* = 2 \cdot (0,5c) + c + \pi (2d)$

$= 300 + 300 + \pi \cdot 2 \cdot 375$

$u_1^* = 2956.2$

$\beta = \frac{3256.2}{2956.2} = 1,10$

$v_{Ed} = \beta \cdot \frac{V_{Ed}}{u_1 \cdot d} = 1.10 \frac{1100 \cdot 10^3}{3256.2 \cdot 375} = 0,99$ N/mm²

$$v_{Rd,c} = 0{,}12\ k\ (100\ \rho_l\ f_{ck})^{1/3} + k_1\ \sigma_{cp}$$

(only when the floor is under compression or tension)

$$v_{Rd,c} \geq v_{min}$$

$$v_{min} = 0{,}035\ k^{3/2} \sqrt{f_{ck}}$$

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0$$

$$= 1 + \sqrt{\frac{200}{375}} = 1{,}73$$

$$\rho_l = \sqrt{\rho_{lx} \cdot \rho_{ly}}$$

$$= \sqrt{0{,}011 \cdot 0{,}013} = 0{,}012$$

$$v_{min} = 0{,}035 \cdot (1{,}73)^{3/2} \cdot \sqrt{45} = 0{,}53\ N/mm^2$$

$$v_{Rd,c} = 0{,}12 \cdot 1{,}73 \cdot (100 \cdot 0{,}012 \cdot 45)^{1/3} = 0{,}78\ N/mm^2$$

$$v_{Rd,c} = 0{,}78\ N/mm^2 < v_{Ed} = 0{,}99\ N/mm^2$$

punching shear reinforcement (or other solution) required.

2a) cross section A $N_{tot} = E_s \varepsilon_s A_s + E_c \varepsilon_c A_c$

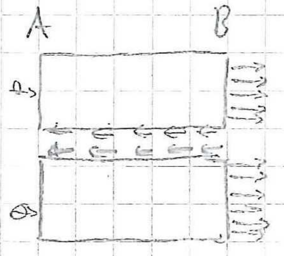
concrete cracks

$N_c = 0.$

cross section B $N_{tot} = E_s \varepsilon_s A_s + E_c \varepsilon_c A_c$

concrete la max tensile stress.

$N_c = E_c \varepsilon_c A_c = f_{ctm} A_c$



$\Sigma F_h = 0.$ (forces acting on concrete.)

$f_{ctm} \cdot A_c = \tau_{bm} \cdot l_t \cdot$ surface area steel

$f_{ctm} A_c = \tau_{bm} l_t \cdot \pi \phi$

$l_t = \dfrac{f_{ctm} \cdot A_c}{\tau_{bm} \cdot \pi \cdot \phi}$

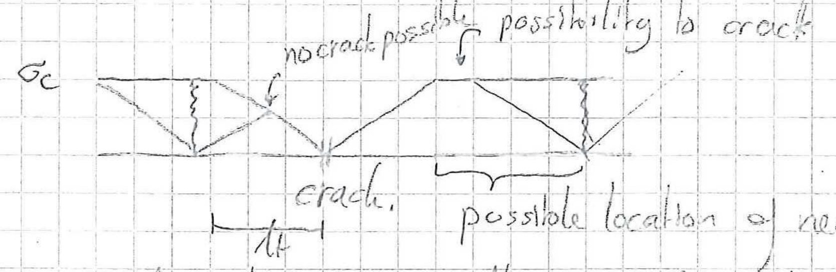
2b) $h_{cef(1)} = 2{,}5 \cdot (500 - (500 - 20 - 10 - \frac{1}{2} \cdot 20)) = 100$ mm ← governing.

$h_{cef(2)} = (500 - 175)/3 = 108$ mm

$h_{cef(3)} = 500/2 = 250$ mm.

$l_t = \dfrac{2{,}2 \cdot 100 \cdot 240}{2{,}2 \cdot 2{,}0 \cdot \pi \cdot \phi \cdot 3} = 63{,}7$ mm.

) theoretical crack distance in between lt and 2 lt.



If crack is over 2lt away from the previous crack, - a crack can still occur inbetween.

If crack is less than 2lt away (but at least lt) no more cracks will occur in between due to lower concrete stress

min: lt = 63,7 mm

max: 2lt = 127 mm

one reason could be imperfections in concrete (grindneed)

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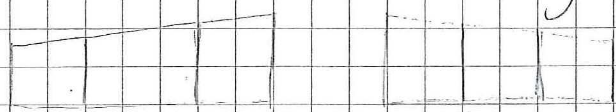
3a
1 Erector.
2 Jack.
3 Shield.
4 Cutterhead
5 Gantry
6 Conveyor
7 Rubber tail seal

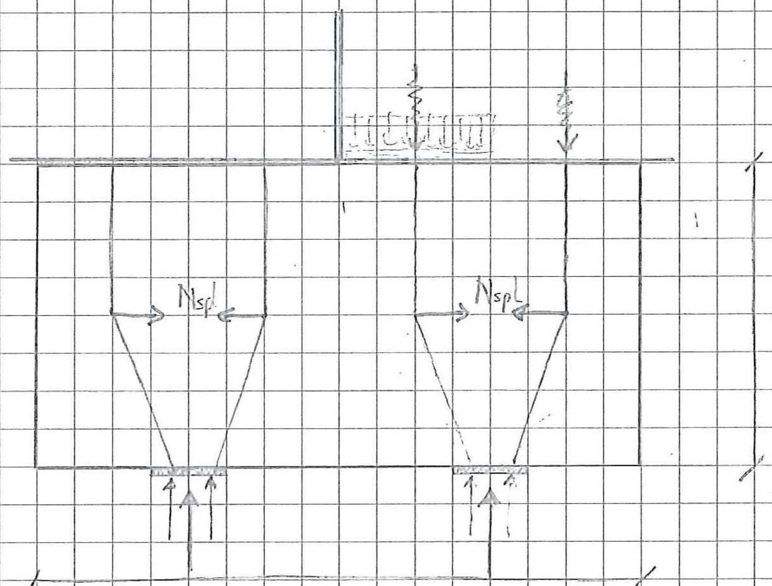
3b By making rings wedge shaped, the lining of the tunnel can make curves. One method is by using left and right rings to turn to the left or right. (the keysegment can be placed in the desired location)



Another method is by using uni rings, which are rotated/orientated in such way to the previous ring that a curve can be made.

3c B

3d



3e

$$N_{spl} = \frac{1}{4} F_p \left(1 - \frac{h_1}{h}\right) = \frac{1}{4} \cdot 4000 \cdot \left(1 - \frac{400}{1600}\right) = 750 \text{ kN}.$$

$$A_{s,spl} = \frac{N_{spl}}{f_s} = \frac{750\,000}{435} = 1724 \text{ mm}^2$$ for instance: 9 ⌀16

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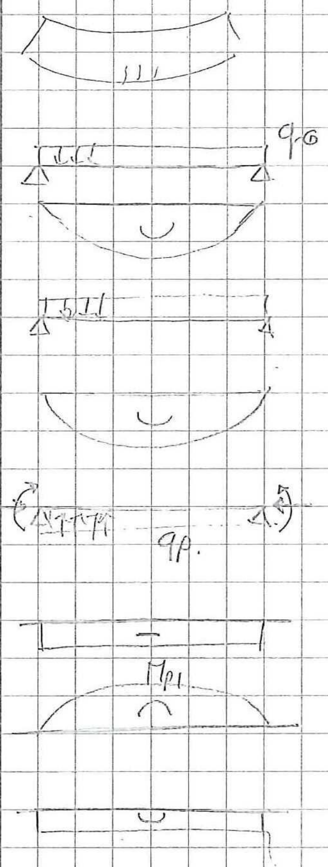
4a $t = \infty$ — check bottom fibre for tensile stresses.



Self weight

$$M_G = \frac{1}{8}(0{,}5875 \cdot 25) \cdot 24^2 = 1057{,}5 \text{ kNm}$$

Variable load

$$M_Q = \frac{1}{8} \cdot 11 \cdot 24^2 = 792 \text{ kNm}$$

due to curvature of tendons

$$N = P_{mt}$$

$$q_p = \frac{8 P_{mt} \cdot 0{,}450}{24^2} \qquad M_{p1} = \frac{1}{8} q_p \cdot 24^2$$

$$M_{p1} = \frac{1}{8} \cdot \frac{8 P_{mt} \cdot 0{,}450}{24^2} \cdot 24^2$$

$$M_{p1} = P_{mt} \cdot 0{,}45$$

$$M_{p2} = P_{mt}(0{,}518 - 0{,}35) = P_{mt} \cdot 0{,}168$$

due to moments at support A, C (prestressing ... a)

$\sigma_b \leq 0$

$$\frac{-P_{mt}}{A_c} + \frac{M_G + M_Q}{W_b} - \frac{M_{p1}}{W_b} + \frac{M_{p2}}{W_b} \leq 0$$

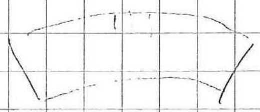
$$\frac{-P_{mt}}{0{,}5875} + \frac{1849{,}5}{0{,}15} - \frac{P_{mt} \cdot 0{,}45}{0{,}15} + \frac{P_{mt} \cdot 0{,}168}{0{,}15} \leq 0$$

$$P_{mt}\left(\frac{-1}{0{,}5875} - \frac{0{,}282}{0{,}15}\right) \leq -\frac{1849{,}5}{0{,}15}$$

$$P_{mt} \geq 3442 \text{ kN}$$

$$P_{m0} \geq 4303 \text{ kN}$$

t = 0.



maximum compressive stress is at bottom.

check bottom fibre:

$$-\frac{P_{m0}}{A_c} + \frac{M_G}{W_b} + \frac{M_{p2}}{W_b} - \frac{M_{p1}}{W_b} \geq -0{,}6 f_{ck} = -27\ N/mm^2$$

$27000\ kN/m^2$

$$-\frac{P_{m0}}{0{,}5875} + \frac{1057{,}5}{0{,}15} + \frac{P_{m0}\cdot 0{,}168}{0{,}15} - \frac{P_{m0}\cdot 0{,}45}{0{,}15} \geq -27000$$

$$P_{m0}\left(\frac{-1}{0{,}5875} + -\frac{0{,}282}{0{,}15}\right) \geq -\frac{1057{,}5}{0{,}15} - 27000$$

$$P_{m0} = 9505.5\ kN.$$

4b one strand $140\ mm^2$

Initial stress (max) $1395\ N/mm^2$.

$$4303 < P_{m0} < 9506$$

at least $\frac{4303.000}{1395} = 3084.3\ mm^2$ required.

at least $\frac{3084.3}{140} = 22$ strands required.

three tendons available so for instance choose

3x 6-12 and apply in each tendon unit 8 strands.

3x 8 strands is 24 strands $\geq$ 22 strands required.

if losses in the end would be bigger than 20% also some space is left to add strands to the tendons.

3x tendon unit 6-12 with each 8 strands => 24 strands.

$24 \cdot 140\ mm^2 \cdot 1395\ N/mm^2 = 4687\ kN.$

$4303 < 4687 < 9506$

4c 3 x 10 strands are applied (given)

$10 \cdot 140 \cdot 1395 = 1953$ kN per tendon.

$A_p = 10 \cdot 140 = 1400 \text{ mm}^2$ per tendon.

$$\Delta P_{el} = \frac{3-1}{2} \cdot 1953 \cdot 1000 \cdot \frac{195000 \cdot 1400}{36000 \cdot 587500}$$

$\Delta P_{el} = 25209$ N.

25,2 kN. per tendon.

4d. Average is 25,2 kN

Three tendons are applied

First tendon loss =

Second tendon. loss = 25,2 kN (when third is stressed)

Third tendon loss = 0 kN. (stressed last, so no loss)

Average 25,2 kN

First Tendon - loss is 50,4 kN

(Losses can be compensated by tensioning the first tendons again after stressing the last tendon. (Netherlands))

Or losses can be overstressed.

$(1488 - 1395) \cdot 1400 \text{ mm}^2 = 130$ kN per tendon

50,4 kN can be overstressed.

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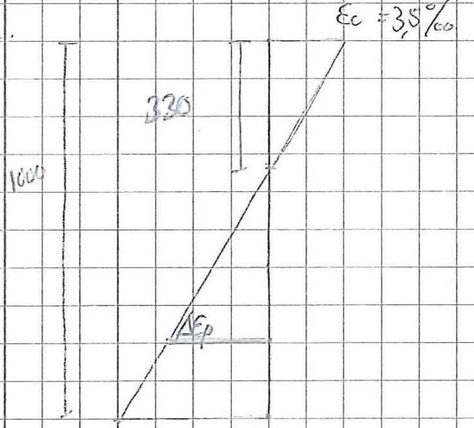
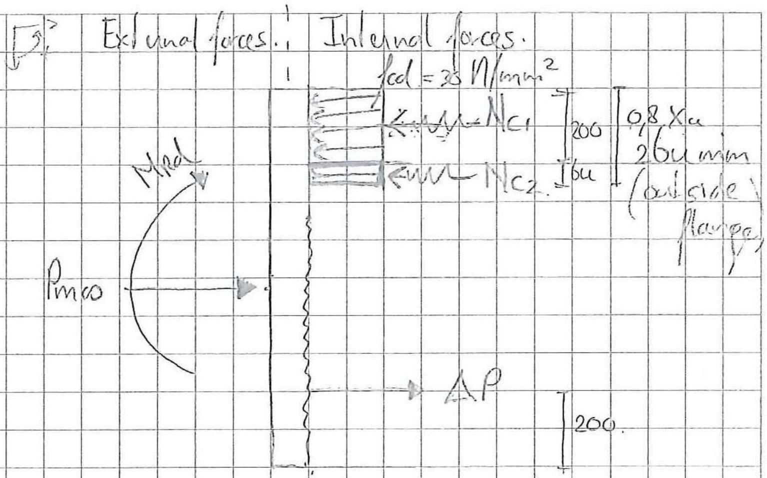
5a.





5b. Equilibrium of horizontal forces.

$\Sigma F_h = 0. \quad \Delta P + P_{m\infty} + -N_{c1} + -N_{c2} = 0.$

$P_{m\infty} = 4700\ kN.$

$N_{c1} = 30 \cdot 200 \cdot 1000 = 6000\ kN.$ (h · b)

$N_{c2} = 30 \cdot 64 \cdot 250 = 480\ kN.$

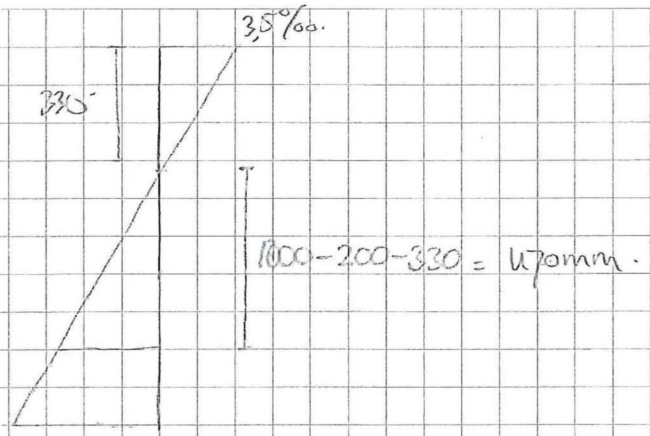
$\Delta P = 6000 + 480 - 4700 = 1780\ kN.$

$\Delta\sigma_p = \frac{1780{,}000}{4200} = 424\ N/mm^2.$

initial stress from the assumed prestressing force $P_{m\infty}$ is 4700 kN. $\frac{4700{,}000}{4200} = 1119\ N/mm^2$

$\sigma_p = 1119 + 424 = 1543\ N/mm^2.$

5c.



330	3,5 ‰
470	4,98 ‰

$\Delta\varepsilon_p$ = 4,98 ‰

initial strain in the prestressing steel → calculation from initial stress.

initial stress is 1119 N/mm² (Q5b) ≤ 1522 N/mm²

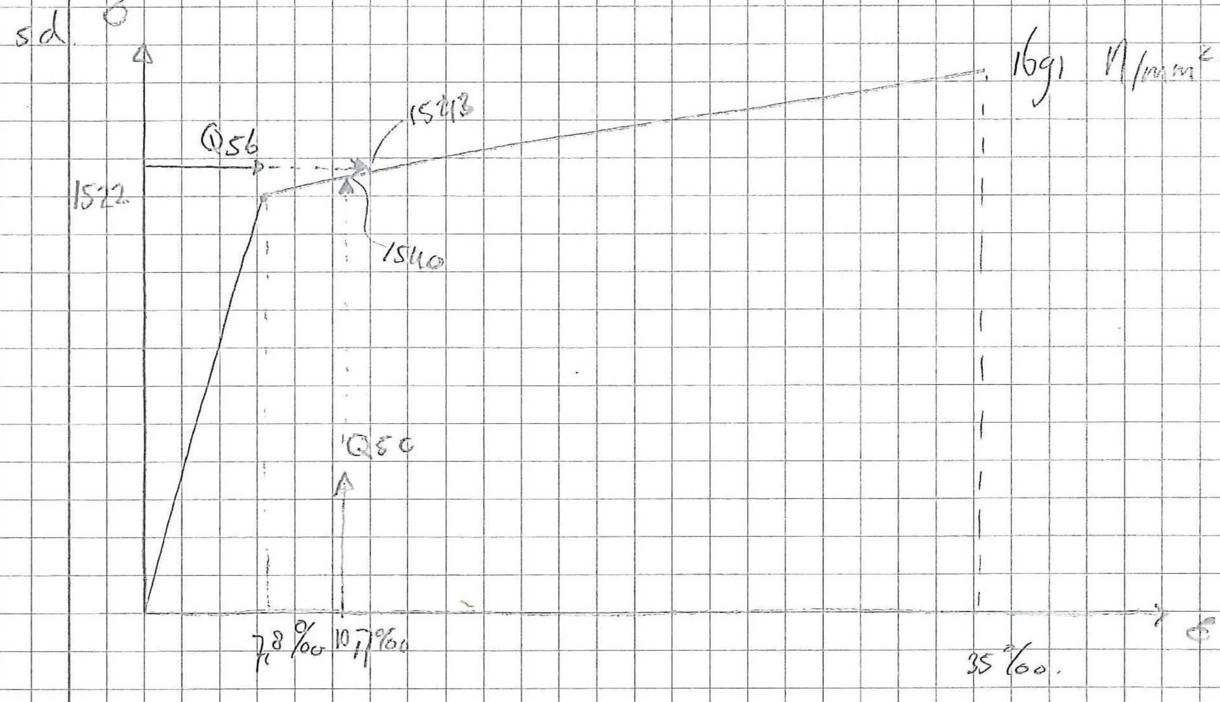
so behaviour is still linear elastic $\sigma = E \cdot \varepsilon$.

$$\varepsilon = \frac{\sigma}{E} = \frac{1119 \text{ N/mm}^2}{195.000 \text{ N/mm}^2} = 0{,}005739$$

5,74 ‰

Total strain 5,74‰ + 4,98‰ = 10,7 ‰.

5d.



Q 5c strain is 10,7 ‰.

stress is calculated from the stress strain diagram of the steel: $1522 + \frac{(10,7 - 7,8)}{(35 - 7,8)} \cdot (1691 - 1522) =$

$\sigma_p(5c) = 1540\ N/mm^2$

Q 5b stress is $1543\ N/mm^2$

The values for the stresses are very close to each other. This means the calculation is correct and the assumed compressive zone X_u is correct.

If the values were not close to each other iterative calculation procedure can be applied to find a more precise value for the height of the concrete compressive zone X_u.

5e.

Med from equilibrium around S (top fibre)

$-M_{Rd} - N_{c1} \cdot 0,100 - N_{c2} \cdot (0,2 + \frac{0,064}{2}) + P_{m\infty} \cdot 0,518 + \Delta P \cdot 0,800 =$

$-M_{Rd} - 6000 \cdot 0,1 - 480 \cdot 0,232 + 4700 \cdot 0,518 + 1780 \cdot 0,8 = 0$

$M_{Rd} = 3147\ kNm.$

$M_{Ed} = 1,2 \cdot M_G + 1,5 \cdot M_Q - 1,0\,(M_p)$ (see Q 4a).

$= 1,2 \cdot 1057,5 + 1,5 \cdot 792 - 1,0 \cdot (4700 \cdot 0,282) = 1132\ kNm$

$M_{Rd} > M_{Ed}.$

$3147 > 1132$ sufficient!