TECHNICAL STUDIES & COMPUTER APPLICATIONS

T[redacted]
[redacted]

CALCULATION REPORT

REFERENCE **202202_18-1/A.A&P**
DATE FEBRUARY 2022

CLIENT A[redacted]
[redacted]

PROJECT
ZOETERMEER ROPE COURSE

"VERSION"	"A"	"18.02.2022"

[redacted]

Introduction

The aim of this report is to verify the safety of the 'Zoetermeer" Rope Course with respect to the strength and stability of the structure.

The basic structure of a Rope Course consists of numberous vertical steel columns connected together with steel cables and/or steel beams. Some columns are secured with guy cables. All fixings to the ground (guy cables and columns) are securely anchored to the environment.

In general, two kinds of horizontal cables are envisaged. The first is called a "foot cable", which is loaded in normal conditions by the participant(s) weight(s) and auxiliaries. The second is called a "safety cable", which is to be a back-up or safety device.

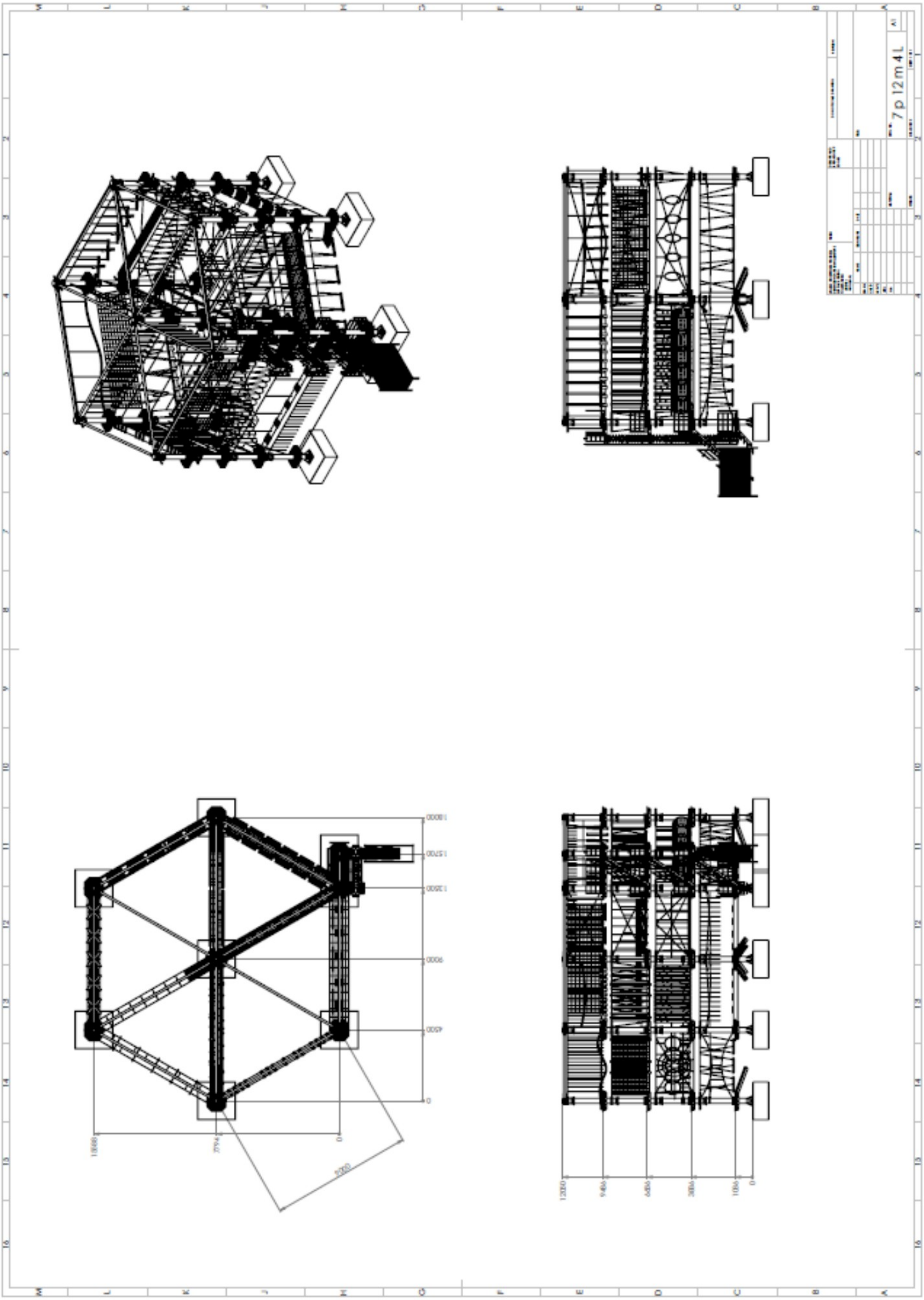
A minimum safety factor of 3 (with respect to the breaking strenght) is required for the safety cables according to **EN 15567-1 (2015) Sports and recreational facilities - Ropes Courses - Part 1: Construction and safety requirements** at a vertical load of 6 kN exerted at the safety cable. This safety margin is also applied for the ziplines at a vertical load of 3 kN.

The structure is generally an hyperstatic system. A system is hyperstatic when the set of equations expressing the equilibrium of forces and moments is insufficient to assess the external and internal forces and moments. The set has to be complemented by deformation expressions for the cables and poles. In addition, the system of equations is then closed (i.e. all forces and moments are unambiguously determined) by the boundary conditions, as e.g. the cable preloads or initial cable lengths or sags at zero load.

As a result, the safety factor depends on the cable's sag. Thus, the requirement of a minimum value of the safety factor results in a minimum sag of the cable. The bigger the cable's sag, the lower the cable's tension; for practical reasons the cable's sag is limited. In this report, the cable's sag at zero load is supposed to be 5% for foot and safety cables. This rather small sag of the safety cables results in considerable horizontal forces at the columns, when loaded by 6 kN.

The assessment is based on the drawing provided by A&P Prlocects:
Zoetermeer_15-11-2021 met hindernissen.STEP

The attractions are suspended between the columns at 4 different levels. The structure is charged with 1 participant per attraction. The weight of a participant is limited to 120 kg.


7p 12m 4L
A1
18000
15700
13500
9000
4500
0
15588
7794
0
9000
12000
9486
6486
3486
1086
0

1. Material and columns properties

1.0. Steel

Acier S235 $E_s := 210\text{GPa}$ $\rho_s := 7800\,\frac{\text{kg}}{\text{m}^3}$ $\sigma_P := 240\text{MPa}$

1.1. Columns - round tubes (10 mm thickness)

Outer diameter $\boxed{\phi_{10o} := 323.9\text{mm}}$

Thickness $\boxed{t_{10} := 10\text{mm}}$

Inner diameter $\phi_{10i} := \phi_{10o} - 2\cdot t_{10} = 303.9\cdot\text{mm}$

Section $\Omega_{10} := \frac{\pi}{4}\cdot\left(\phi_{10o}{}^2 - \phi_{10i}{}^2\right) = 98.61\cdot\text{cm}^2$

Linear weight $\lambda_{10} := \rho_s\cdot\Omega_{10} = 76.92\,\frac{\text{kg}}{\text{m}}$

Area moment of inertia $I_{10z} := \frac{\pi}{64}\cdot\left(\phi_{10o}{}^4 - \phi_{10i}{}^4\right) = 12158.3\cdot\text{cm}^4$ $I_{10y} := I_{10z}$

Bending resistance $W_{10z} := \frac{2\cdot I_{10z}}{\phi_{10o}} = 750.7\cdot\text{cm}^3$ $W_{10y} := W_{10z}$

Shear strength $A_{10z} := \Omega_{10} = 98.61\cdot\text{cm}^2$ $A_{10y} := A_{10z}$

Moment of inertia for torsion $I_{10t} := \frac{\pi}{32}\cdot\left(\phi_{10o}{}^4 - \phi_{10i}{}^4\right) = 24316.7\cdot\text{cm}^4$

Torsion resistance $W_{10t} := \frac{2\cdot I_{10t}}{\phi_{10o}} = 1501.5\cdot\text{cm}^3$

Length $L_{10} := 12030\text{mm}$

Weight $M_{10} := \lambda_{10}\cdot L_{10} = 925.3\,\text{kg}$

Buckling

Safety margin in plastic region $S_{KP} := 1.7$ Lastfall H = 1.7 ; HZ = 1.5 1.7

Safety margin in elastic region $S_{KE} := 3$

Load case Gev := "A"

Case A : ingeklemd ; vrij $L_k = 2\cdot L$

Case B : scharnier ; geleid $L_k = L$

Case C : ingeklemd ; geleid $L_k = 0.7L$

Case D : ingeklemd ; geleid ingeklemd $L_k = 0.5L$

Slenderness $\lambda = 217$

Critical buckling stress - allowed plastic stress $\sigma_k = 44 \cdot \text{MPa}$ $\sigma_{zul} = 141 \cdot \text{MPa}$

Allowed buckling stress $\sigma_{kt} = 15 \cdot \text{MPa}$ $V_K = 3.00$ $\omega = 9.59$

Allowed axial force $\boxed{F_{10kt} = 145.1 \cdot \text{kN}}$

1.2. Top beam - round tubes 133x4

Outer diameter $\phi_{lo} := 133\text{mm}$

Thickness $t_l := 4\text{mm}$

Inner diameter $\phi_{li} := \phi_{lo} - 2 \cdot t_l = 125 \cdot \text{mm}$

Section $\Omega_l := \frac{\pi}{4} \cdot \left(\phi_{lo}^{\ 2} - \phi_{li}^{\ 2}\right) = 16.21 \cdot \text{cm}^2$

Linear weight $\lambda_l := \rho_s \cdot \Omega_l = 12.64 \frac{\text{kg}}{\text{m}}$

Area moment of inertia $I_{lz} := \frac{\pi}{64} \cdot \left(\phi_{lo}^{\ 4} - \phi_{li}^{\ 4}\right) = 337.5 \cdot \text{cm}^4$ $I_{ly} := I_{lz}$

Bending resistance $W_{lz} := \frac{2 \cdot I_{lz}}{\phi_{lo}} = 50.8 \cdot \text{cm}^3$ $W_{ly} := W_{lz}$

Shear strength $A_{lz} := \Omega_l = 16.21 \cdot \text{cm}^2$ $A_{ly} := A_{lz}$

Moment of inertia for torsion $I_{lt} := \frac{\pi}{32} \cdot \left(\phi_{lo}^{\ 4} - \phi_{li}^{\ 4}\right) = 675.1 \cdot \text{cm}^4$

Torsion resistance $W_{lt} := \frac{2 \cdot I_{lt}}{\phi_{lo}} = 101.5 \cdot \text{cm}^3$

Length $L_l := 9000\text{mm}$

Weight $M_l := \lambda_l \cdot L_l = 113.8\,\text{kg}$

Buckling

Safety margin in plastic region $S_{KP} := 1.7$ Lastfall H = 1.7 ; HZ = 1.5

Safety margin in elastic region $S_{KE} := 3$

Load case Gev := "B"

Case A : ingeklemd ; vrij $L_k = 2 \cdot L$

Case B : scharnier ; geleid $L_k = L$

Case C : ingeklemd ; geleid $L_k = 0.7L$

Case D : ingeklemd ; geleid ingeklemd $L_k = 0.5L$

Slenderness $\lambda = 197$

Critical buckling stress - allowed plastic stress $\sigma_k = 53 \cdot \text{MPa}$ $\sigma_{zul} = 141 \cdot \text{MPa}$

Allowed buckling stress $\sigma_{kt} = 18 \cdot \text{MPa}$ $V_K = 3.00$ $\omega = 7.95$

Allowed axial force $\boxed{F_{lt} = 28.8 \cdot \text{kN}}$

Deflection caused by own weight - simply supported both ends

$$q := \rho_S \cdot \Omega_l = 12.64 \, \frac{\text{kg}}{\text{m}} \qquad f_m := \frac{5 \cdot q \cdot g \cdot L_l^4}{384 \cdot E_S \cdot I_{lz}} = 14.9 \cdot \text{mm} \qquad \frac{f_m}{L_l} = \frac{1}{602}$$

OK

1.3. Foot and safety cables (attractions) - guy cables

characteristic construction of an attraction: 1 or 2 cables 6x36+IWRC + wooden elements

length attraction $L_h := 9\text{m}$

wire strength $R_o := 1960 \, \frac{\text{N}}{\text{mm}^2}$

diameter $\phi_v := 12\text{mm}$

metallic section $\Omega_m := 66.2\text{mm}^2$

linear weight $q_v := 0.589 \, \frac{\text{kg}}{\text{m}}$

breaking strength $F_{bv} := 100\text{kN}$

apparent E-modulus $E_v := 123\text{GPa}$

Elasticity $AE_v := \Omega_m \cdot E_v = 8.143 \cdot \text{MN}$

weight attraction $\lambda_h := 2 \cdot q_v + 500 \, \frac{\text{kg}}{\text{m}^3} \cdot 0.60\text{m} \cdot 0.05\text{m} \cdot 0.33 = 6.1 \, \frac{\text{kg}}{\text{m}}$

2. Load caused by attractions

2.1. Load on the foot cables

▶

▶ hidden routines - definition of several functions

▶

the load is transfered over $n_k := 1$ kabel(s)

additional linear load, divided over n_k cable(s) $\Delta q := \frac{1}{n_k}\cdot\lambda_h - q_v = 5.539\,\frac{kg}{m}$

The initial length of teh cables depends on the cable's sag at zero load: $f_{ref} := 5\%$:

$P_{ref} := (0)\cdot kN$ @ $p_{ref} := \left(\frac{L}{2}\right)$

maximum number of participants $n := 1$

weight for 1 participant $F_v := 120kgf$

▶ hidden calculation DL

Initial cable length $L_0 = L + \Delta L$ avec $\Delta L = 58.1\cdot mm$ et $L = 9\ m$

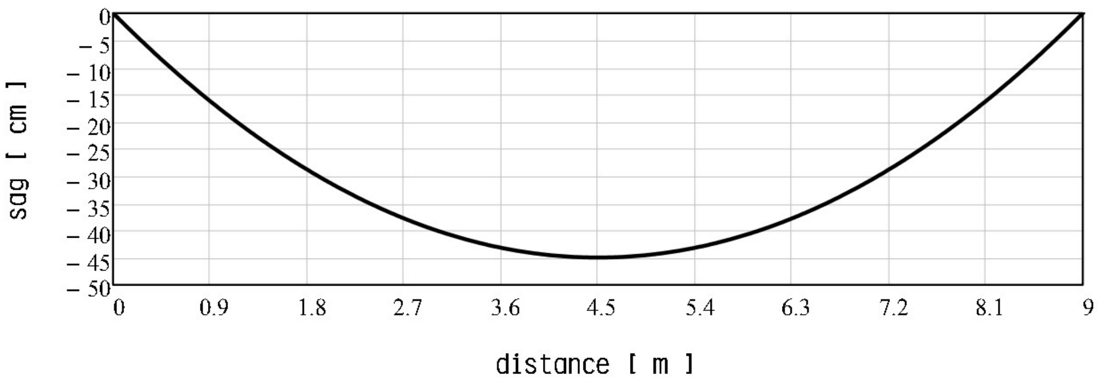
Load Case 1: Unloaded (only own weight of the ttraction)

$P := (0kgf)$ $p := \left(\frac{L}{2}\right)$

$CABLES := ADCABLE(\phi, L, \Delta L, P, p, \Delta q, h, alias, CABLES)$

$H_{k0} := CABLES_{6,2}\cdot kN = 1.352\cdot kN$

$V_{k0} := CABLES_{7,2}\cdot kN = 0.27\cdot kN$



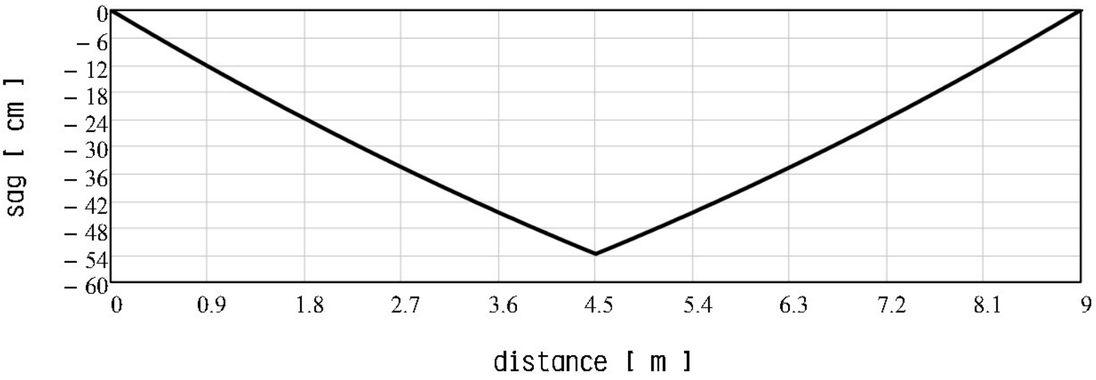
Load Case 2: load in the middle of the attraction by 1 participant

$$P := \frac{1}{n_k}\cdot\left(F_v\right) = (120)\cdot\text{kgf} \qquad p := \left(\frac{L}{2}\right)$$

$$\text{CABLES} := \mathit{ADCABLE}(\phi, L, \Delta L, P, p, \Delta q, h, \text{alias}, \text{CABLES})$$

$$H_{k1} := \text{CABLES}_{6,3}\cdot\text{kN} = 6.05\cdot\text{kN}$$

$$V_{k1} := \text{CABLES}_{7,3}\cdot\text{kN} = 0.859\cdot\text{kN}$$



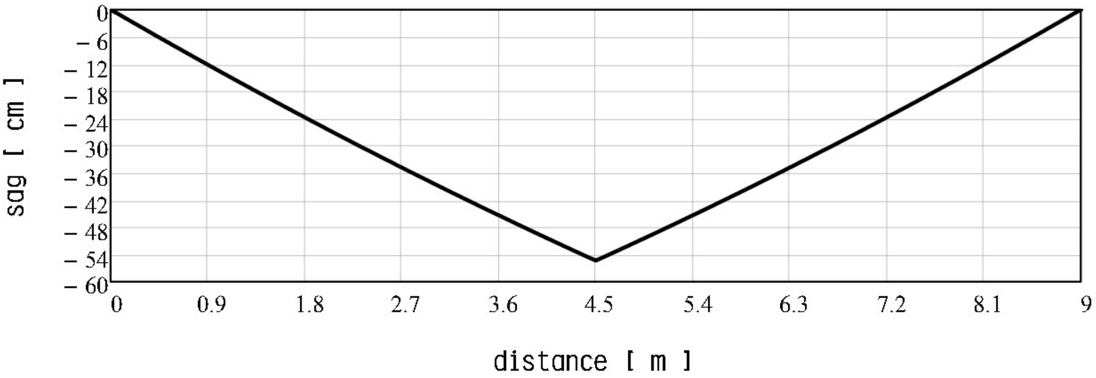
Load Case 3: maximum load in the middle of the attraction (1 participant + 1 guard)

$$P := \frac{1}{n_k}\cdot(200\text{kgf}) \qquad p := \left(\frac{L}{2}\right)$$

$$\text{CABLES} := \mathit{ADCABLE}(\phi, L, \Delta L, P, p, \Delta q, h, \text{alias}, \text{CABLES})$$

$$H_{kp} := \text{CABLES}_{6,4}\cdot\text{kN} = 9.065\cdot\text{kN}$$

$$V_{kp} := \text{CABLES}_{7,4}\cdot\text{kN} = 1.251\cdot\text{kN}$$



CABLES =

	0	1	2	3	4
0	""	""	"_1"	"_2"	"_3"
1	""	""	"AB_1"	"AB_2"	"AB_3"
2	"Ø"	"mm"	12.000	12.000	12.000
3	"h"	"m"	6.000	6.000	6.000
4	"q"	"kg/m"	6.128	6.128	6.128
5	"P @ p"	"kN @ m"	[1, 2]	[1, 2]	[1, 2]
6	"H"	"kN"	1.352	6.050	9.065
7	"VL"	"kN"	0.270	0.859	1.251
8	"VR"	"kN"	0.270	0.859	1.251
9	"TL"	"kN"	1.379	6.111	9.151
10	"TR"	"kN"	1.379	6.111	9.151
11	"f"	"mm"	450.000	538.000	554.000
12	"f/L"	"%"	4.999	5.980	6.155
13	"Lo"	"m"	9.058	9.058	9.058
14	"SF"	"-"	72.503	16.365	10.928

$WARNINGS(\mathrm{CABLES}, 3) =$

	0	1
0	"No warnings :"	"The safety factor of all"
1	"cables is greater than"	3

$\mathrm{CABLES} := \mathrm{CABLES}_0$

2.2. Load on the safety cables

▶

▶ hidden routines - definition of several functions

▶

the load is divided over $n_k := 1$ kabel(s)

additional linear load, divided over n_k kabels $\Delta q := 0$

de initial length of the cable depends on its sag at zero load: $f_{ref} := 5\%$:

$P_{ref} := (0)\cdot kN$ @ $p_{ref} := \left(\frac{L}{2}\right)$

Load on the cable $F_v := 6kN$

▶ hidden calculation DL

Initial cable length $L_0 = L + \Delta L$ avec $\Delta L = 59.5\cdot mm$ et $L = 9\,m$

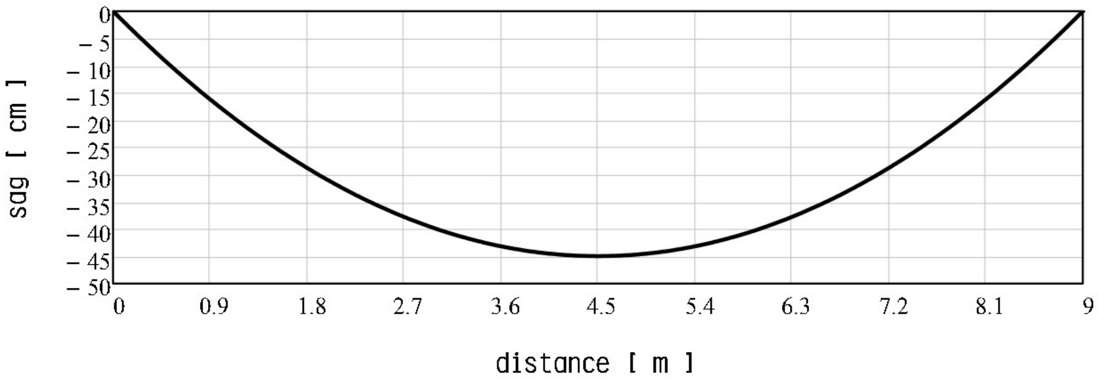
Load Case 1: Unloaded (only own cable weight)

$P := (0kgf)$ $p := \left(\frac{L}{2}\right)$

$\text{CABLES} := \mathit{ADCABLE}(\phi, L, \Delta L, P, p, \Delta q, h, \text{alias}, \text{CABLES})$

$H_{v0} := \text{CABLES}_{6,2}\cdot kN = 0.13\cdot kN$

$V_{v0} := \text{CABLES}_{7,2}\cdot kN = 0.026\cdot kN$



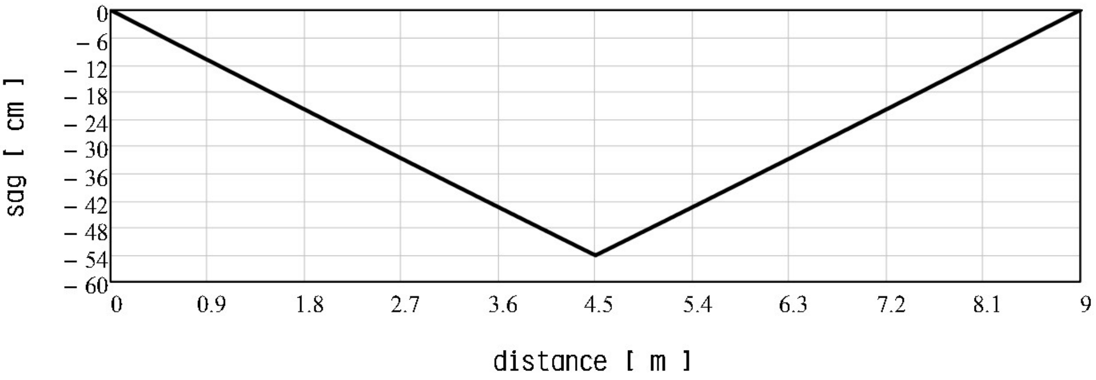
Load Case 2: loaded by 1 participant

$P := \frac{1}{n_k} \cdot (120\text{kgf}) \qquad p := \left(\frac{L}{2}\right)$

$\text{CABLES} := \mathit{ADCABLE}(\phi, L, \Delta L, P, p, \Delta q, h, \text{alias}, \text{CABLES})$

$H_{v1} := \text{CABLES}_{6,3} \cdot \text{kN} = 4.992 \cdot \text{kN}$

$V_{v1} := \text{CABLES}_{7,3} \cdot \text{kN} = 0.614 \cdot \text{kN}$



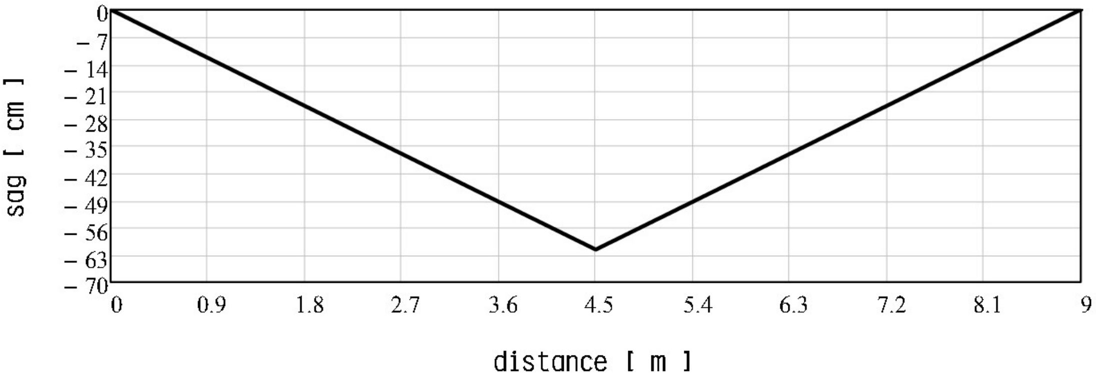
Load Case 3: maximum load

$P := \frac{1}{n_k} \cdot \left(F_v\right) = (6) \cdot \text{kN} \qquad p := \left(\frac{L}{2}\right)$

$\text{CABLES} := \mathit{ADCABLE}(\phi, L, \Delta L, P, p, \Delta q, h, \text{alias}, \text{CABLES})$

$H_{vp} := \text{CABLES}_{6,4} \cdot \text{kN} = 21.978 \cdot \text{kN}$

$V_{vp} := \text{CABLES}_{7,4} \cdot \text{kN} = 3.026 \cdot \text{kN}$



CABLES =

	0	1	2	3	4
0	""	""	"_1"	"_2"	"_3"
1	""	""	"AB_1"	"AB_2"	"AB_3"
2	"Ø"	"mm"	12.000	12.000	12.000
3	"h"	"m"	6.000	6.000	6.000
4	"q"	"kg/m"	0.589	0.589	0.589
5	"P @ p"	"kN @ m"	[1, 2]	[1, 2]	[1, 2]
6	"H"	"kN"	0.130	4.992	21.978
7	"VL"	"kN"	0.026	0.614	3.026
8	"VR"	"kN"	0.026	0.614	3.026
9	"TL"	"kN"	0.133	5.029	22.185
10	"TR"	"kN"	0.133	5.029	22.185
11	"f"	"mm"	450.000	542.000	617.000
12	"f/L"	"%"	5.000	6.024	6.855
13	"Lo"	"m"	9.059	9.059	9.059
14	"SF"	"-"	754.513	19.883	4.508

WARNINGS(CABLES, 3) =

	0	1
0	"No warnings :"	"The safety factor of all"
1	"cables is greater than"	3

CABLES := CABLES_0

▶ def load cases

3. Columns - stresses and deformations

3.1. Introduction

If the columns are elasticly supported at the bottom of the columns, the *deformations* (angular displacement Θ and lateral displacement d) *at the top of the fundation* caused by a bending moment $\mathcal{M}$ and a lateral force $\mathcal{T}$, can be expressed as follows:

$$\boxed{\Theta(\mathcal{M},\mathcal{T}) := \Theta_{m}\cdot\mathcal{M} + \Theta_{t}\cdot\mathcal{T}} \qquad \Theta_{m} = 5.3\times10^{-3}\cdot\frac{\text{deg}}{\text{kN}\cdot\text{m}} \qquad \Theta_{t} = 6.3\times10^{-3}\cdot\frac{\text{deg}}{\text{kN}}$$

$$\boxed{d(\mathcal{M},\mathcal{T}) := d_{m}\cdot\mathcal{M} + d_{t}\cdot\mathcal{T}} \qquad d_{m} = 0.125\cdot\frac{\text{mm}}{\text{kN}\cdot\text{m}} \qquad d_{t} = 0.405\cdot\frac{\text{mm}}{\text{kN}}$$

See e-mail [redacted] dd. 8.2.2022. "Standaard geboorde betonpalen type Atlas met nodige bewapening".

In the following analytic calculations, it is assumed that the displacement at the top of a column is negligible since the top is interconnected with the tops of two other columns by top **beams.**

The peak and full load on a column occurs in the following combined situations:

1/ Peak load (accidental) :
- level 1 : full load (120 kgf on the foot cable)
- level 2 : a fall of a particpant (6kN on the safety cable)
- level 3 : rescue (200 kgf on the foot cable)
- level 4 : full load (120 kgf on the foot cable)

2/ Nominal load :
- level 1 : full load (120 kgf on the foot cable)
- level 2 : full load (120 kgf on the safety cable)
- level 3 : full load (120 kgf on the foot cable)
- level 4 : full load (120 kgf on the foot cable)

With horizontal components:

$$F_{zero} = \begin{pmatrix} 0.13 \\ 0.13 \\ 0.13 \\ 0.13 \\ 1.352 \\ 1.352 \\ 1.352 \\ 1.352 \end{pmatrix}\cdot\text{kN} \qquad F_{nom} = \begin{pmatrix} 0.13 \\ 0.13 \\ 4.992 \\ 0.13 \\ 6.05 \\ 6.05 \\ 1.352 \\ 6.05 \end{pmatrix}\cdot\text{kN} \qquad F_{peak} = \begin{pmatrix} 0.13 \\ 0.13 \\ 21.978 \\ 0.13 \\ 6.05 \\ 9.065 \\ 1.352 \\ 6.05 \end{pmatrix}\cdot\text{kN}$$

And vertical components:

$$V_{zero} = \begin{pmatrix} 0.026 \\ 0.026 \\ 0.026 \\ 0.026 \\ 0.27 \\ 0.27 \\ 0.27 \\ 0.27 \end{pmatrix} \cdot kN \qquad V_{nom} = \begin{pmatrix} 0.026 \\ 0.026 \\ 0.614 \\ 0.026 \\ 0.859 \\ 0.859 \\ 0.27 \\ 0.859 \end{pmatrix} \cdot kN \qquad V_{peak} = \begin{pmatrix} 0.026 \\ 0.026 \\ 3.026 \\ 0.026 \\ 0.859 \\ 1.251 \\ 0.27 \\ 0.859 \end{pmatrix} \cdot kN$$

3.2. Outer Columns - loaded with the attractions (levels 1, 2, 3 en 4)

$I := I_{10z} \qquad W := W_{10z} \qquad L := 11.9m$

This column is stiffly connected to the environment by the top beams dia 133 x 4

3.2.1. Zero load (only own weight of the attractions)

Horizontal forces

$$F := F_{zero} = \begin{pmatrix} 0.13 \\ 0.13 \\ 0.13 \\ 0.13 \\ 1.352 \\ 1.352 \\ 1.352 \\ 1.352 \end{pmatrix} \cdot kN \qquad h := \begin{pmatrix} 11.3 \\ 8.5 \\ 5.7 \\ 2.9 \\ 9.3 \\ 6.5 \\ 3.7 \\ 0.9 \end{pmatrix} m$$

Bending moment @ bottom $\quad M_b := \sum_{i=0}^{rows(h)-1} \left(F_i \cdot h_i\right) = 3.61 \cdot kN \cdot m$

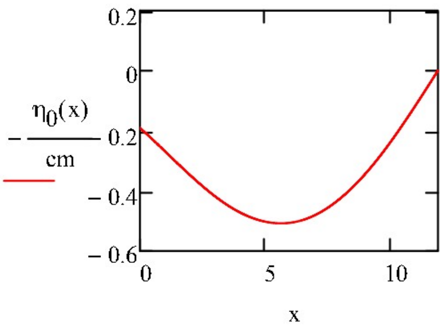
Bending stress @ bottom $\quad \sigma_b := \frac{M_b}{W} = 5 \cdot MPa$

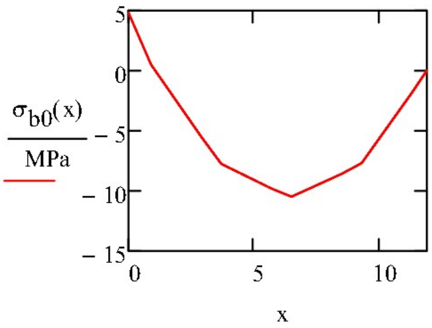
Maximum bending stress $\quad \boxed{\sigma_{b.max} = 10.5 \cdot MPa}$

Displacement @ top $\quad \eta_0(L) = -0.0 \cdot mm \qquad H_{top} = -2.324 \cdot kN$

Maximum displacement $\quad \eta_{max} = 0.51 \cdot cm \quad @ \quad x_{max} = 5.633\ m$

$$\frac{\eta_{max}}{L} = \frac{1}{2340}$$





3.2.2. Peak load - Direction 1

Horizontal forces

$$F := F_{peak} = \begin{pmatrix} 0.13 \\ 0.13 \\ 21.978 \\ 0.13 \\ 6.05 \\ 9.065 \\ 1.352 \\ 6.05 \end{pmatrix} \cdot kN \qquad h := \begin{pmatrix} 11.3 \\ 8.5 \\ 5.7 \\ 2.9 \\ 9.3 \\ 6.5 \\ 3.7 \\ 0.9 \end{pmatrix} m$$

▶

Bending moment @ bottom $$M_b := \sum_{i=0}^{rows(h)-1} \left(F_i \cdot h_i\right) = 36.18 \cdot kN \cdot m$$

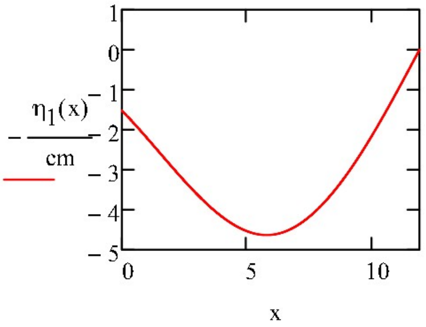
Bending stress @ bottom $$\sigma_b := \frac{M_b}{W} = 48 \cdot MPa$$

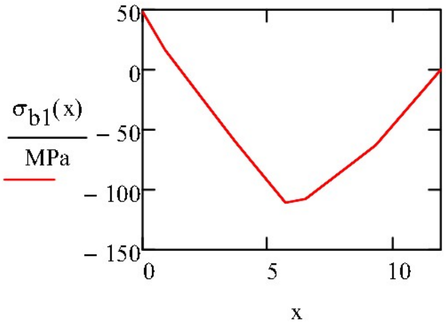
Maximum bending stress $$\boxed{\sigma_{b.max} = 110.9 \cdot MPa}$$

Displacement @ top $\eta_1(L) = -0.0 \cdot mm$ $H_{top} = -18.293 \cdot kN$

Maximum displacement $\eta_{max} = 4.64 \cdot cm$ @ $x_{max} = 5.787\ m$

$$\frac{\eta_{max}}{L} = \frac{1}{256}$$





3.2.3. Nominal load - Direction 2 and 3

Horizontal forces

$$F := F_{nom} = \begin{pmatrix} 0.13 \\ 0.13 \\ 4.992 \\ 0.13 \\ 6.05 \\ 6.05 \\ 1.352 \\ 6.05 \end{pmatrix} \cdot kN \qquad h := \begin{pmatrix} 11.3 \\ 8.5 \\ 5.7 \\ 2.9 \\ 9.3 \\ 6.5 \\ 3.7 \\ 0.9 \end{pmatrix} m$$

Bending moment @ bottom $M_b := \sum_{i=0}^{rows(h)-1} \left(F_i \cdot h_i\right) = 15.87 \cdot kN \cdot m$

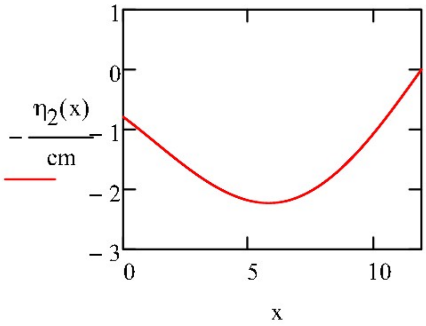
Bending stress @ bottom $\sigma_b := \frac{M_b}{W} = 21 \cdot MPa$

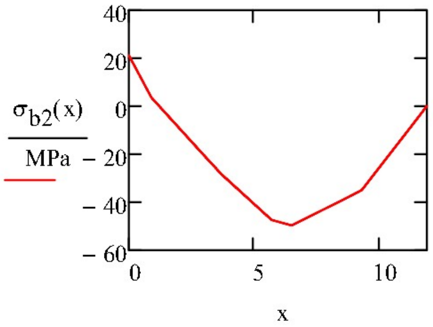
Maximum bending stress $\boxed{\sigma_{b.max} = 49.7 \cdot MPa}$

Displacement @ top $\eta_2(L) = -0.0 \cdot mm$ $H_{top} = -10.216 \cdot kN$

Maximum displacement $\eta_{max} = 2.23 \cdot cm$ @ $x_{max} = 5.804\ m$

$$\frac{\eta_{max}}{L} = \frac{1}{533}$$





3.2.4. Superposition of the directions 1 , 2 and 3

ZERO LOAD

$\theta_2 := -60deg$ $\theta_3 := 60deg$ $f_1 := 1$ $f_2 := 1$ $f_3 := 1$

$$M(x) := M_0(x) \cdot \sqrt{\left(f_1 + f_2 \cdot \cos\left(\theta_2\right) + f_3 \cdot \cos\left(\theta_3\right)\right)^2 + \left(f_2 \cdot \sin\left(\theta_2\right) + f_3 \cdot \sin\left(\theta_3\right)\right)^2}$$

$$T(x) := T_0(x)\cdot\sqrt{\left(f_1 + f_2\cdot\cos(\theta_2) + f_3\cdot\cos(\theta_3)\right)^2 + \left(f_2\cdot\sin(\theta_2) + f_3\cdot\sin(\theta_3)\right)^2}$$

$$\sigma_b(x) := \frac{M(x)}{W}$$

$$\eta(x) := \eta_0(x)\cdot\sqrt{\left(f_1 + f_2\cdot\cos(\theta_2) + f_3\cdot\cos(\theta_3)\right)^2 + \left(f_2\cdot\sin(\theta_2) + f_3\cdot\sin(\theta_3)\right)^2}$$

Bending moment @ bottom $M(0) = 7.23\cdot kN\cdot m$

components $M_0(0)\cdot\left(f_1 + f_2\cdot\cos(\theta_2) + f_3\cdot\cos(\theta_3)\right) = 7.23\cdot kN\cdot m$

$M_0(0)\cdot\left(f_2\cdot\sin(\theta_2) + f_3\cdot\sin(\theta_3)\right) = 0\cdot kN\cdot m$

Lateral force @ bottom $T(0) = 7.208\cdot kN$

components $T_0(0)\cdot\left(f_1 + f_2\cdot\cos(\theta_2) + f_3\cdot\cos(\theta_3)\right) = 7.208\cdot kN$

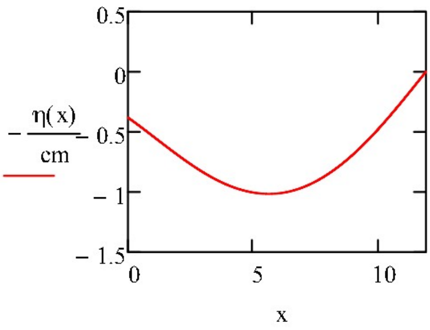
$T_0(0)\cdot\left(f_2\cdot\sin(\theta_2) + f_3\cdot\sin(\theta_3)\right) = 0\cdot kN$

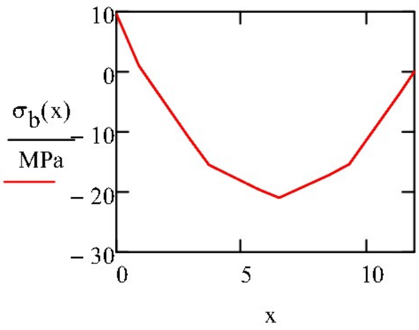
Bending stress @ bottom $\sigma_b(0) = 9.6\cdot MPa$

Maximum bending stress $\boxed{\sigma_{b.max} = 21\cdot MPa}$

Displacement @ top & bottom $\eta(L) = -0.0\cdot mm$ $\eta(0) = 3.8\cdot mm$

Maximum displacement $\eta_{max} = 1.02\cdot cm$ @ $x_{max} = 5.633\ m$ $\frac{\eta_{max}}{L} = \frac{1}{1170}$





PEAK LOAD

$\theta_2 := -60deg$ $\theta_3 := 60deg$ $f_1 := 1$ $f_2 := 1$ $f_3 := 1$ $M_3 = M_2$

$$M(x) := \sqrt{\left(f_1\cdot M_1(x) + f_2\cdot M_2(x)\cdot\cos(\theta_2) + f_3\cdot M_2(x)\cdot\cos(\theta_3)\right)^2 + \left(f_2\cdot M_2(x)\cdot\sin(\theta_2) + f_3\cdot M_2(x)\cdot\sin(\theta_3)\right)^2}$$

$$T(x) := \sqrt{\left(f_1\cdot T_1(x) + f_2\cdot T_2(x)\cdot\cos(\theta_2) + f_3\cdot T_2(x)\cdot\cos(\theta_3)\right)^2 + \left(f_2\cdot T_2(x)\cdot\sin(\theta_2) + f_3\cdot T_2(x)\cdot\sin(\theta_3)\right)^2}$$

$$\sigma_b(x) := \frac{M(x)}{W}$$

$$\eta(x) := \sqrt{\left(f_1 \cdot \eta_1(x) + f_2 \cdot \eta_2(x) \cdot \cos(\theta_2) + f_3 \cdot \eta_2(x) \cdot \cos(\theta_3)\right)^2 + \left(f_2 \cdot \eta_2(x) \cdot \sin(\theta_2) + f_3 \cdot \eta_2(x) \cdot \sin(\theta_3)\right)^2}$$

Bending moment @ bottom $\boxed{M(0) = 52.04 \cdot kN \cdot m}$

components $M_I = 52.04 \cdot kN \cdot m$

$M_{II} = 0 \cdot kN \cdot m$

Lateral force @ bottom $\boxed{T(0) = 41.26 \cdot kN}$

components $T_I = 41.26 \cdot kN$

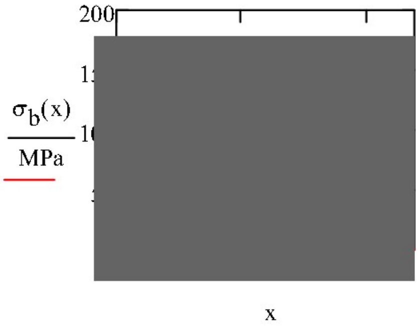
$T_{II} = 0 \cdot kN$

Bending stress @ bottom $\sigma_b(0) = 69.3 \cdot MPa$

Maximum bending stress $\boxed{\sigma_{b.max} = 158.4 \cdot MPa}$

Displacement @ top & bottom $\eta(L) = 0.0 \cdot mm$ $\eta(0) = 23.2 \cdot mm$

Maximum displacement $\boxed{\eta_{max} = 6.87 \cdot cm}$ @ $x_{max} = 5.792\ m$ $\frac{\eta_{max}}{L} = \frac{1}{173}$



NOMINAL LOAD

$\theta_2 := -60deg$ $\theta_3 := 60deg$ $f_1 := 1$ $f_2 := 1$ $f_3 := 1$ $M_1 = M_3 = M_2$

$M_1(x) := M_2(x)$ $T_1(x) := T_2(x)$ $\eta_1(x) := \eta_2(x)$

$$M(x) := \sqrt{\left(f_1 \cdot M_1(x) + f_2 \cdot M_2(x) \cdot \cos(\theta_2) + f_3 \cdot M_2(x) \cdot \cos(\theta_3)\right)^2 + \left(f_2 \cdot M_2(x) \cdot \sin(\theta_2) + f_3 \cdot M_2(x) \cdot \sin(\theta_3)\right)^2}$$

$$T(x) := \sqrt{\left(f_1 \cdot T_1(x) + f_2 \cdot T_2(x) \cdot \cos(\theta_2) + f_3 \cdot T_2(x) \cdot \cos(\theta_3)\right)^2 + \left(f_2 \cdot T_2(x) \cdot \sin(\theta_2) + f_3 \cdot T_2(x) \cdot \sin(\theta_3)\right)^2}$$

$$\sigma_b(x) := \frac{M(x)}{W}$$

$$\eta(x) := \sqrt{\left(f_1\cdot\eta_1(x) + f_2\cdot\eta_2(x)\cdot\cos(\theta_2) + f_3\cdot\eta_2(x)\cdot\cos(\theta_3)\right)^2 + \left(f_2\cdot\eta_2(x)\cdot\sin(\theta_2) + f_3\cdot\eta_2(x)\cdot\sin(\theta_3)\right)^2}$$

Bending moment @ bottom $\boxed{M(0) = 31.73\cdot kN\cdot m}$

components $M_I = 31.73\cdot kN\cdot m$

$M_{II} = 0\cdot kN\cdot m$

Lateral force @ bottom $\boxed{T(0) = 29.34\cdot kN}$

components $T_I = 29.34\cdot kN$

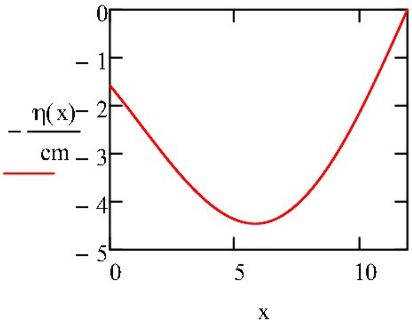
$T_{II} = 0\cdot kN$

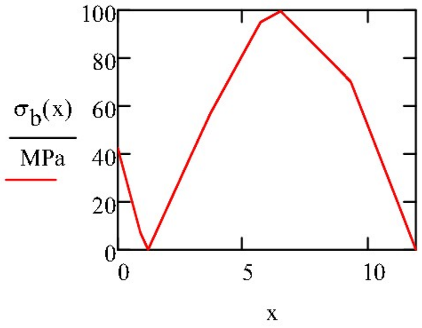
Bending stress @ bottom $\sigma_b(0) = 42.3\cdot MPa$

Maximum bending stress $\boxed{\sigma_{b.max} = 99.5\cdot MPa}$

Displacement @ top & bottom $\eta(L) = 0.0\cdot mm$ $\eta(0) = 15.8\cdot mm$

Maximum displacement $\boxed{\eta_{max} = 4.46\cdot cm}$ @ $x_{max} = 5.804\,m$ $\frac{\eta_{max}}{L} = \frac{1}{266}$





3.2.5. Vertical loads on the outer columns - buckling

Own weight columns $M_c := M_{10} = 925\,kg$

Platforms $M_{pl} := 4\cdot(185 + 75)kg = 1040\,kg$

Participants $M_p := 4\cdot 80kg = 320\,kg$

PEAK LOAD

Attractions $V_b := f_1\cdot V_{peak} + f_2\cdot V_{nom} + f_3\cdot V_{nom}$

$$V_{fs} := \sum_{i=0}^{rows(V_b)-1} V_{b_i} = 13.421\cdot kN$$

Total $V_{tot} := g\cdot(M_c + M_{pl} + M_p) + V_{fs} = 35.83\cdot kN$ < $F_{10kt} = 145.1\cdot kN$ OK

NOMINAL LOAD

Attractions $V_b := f_1 \cdot V_{nom} + f_2 \cdot V_{nom} + f_3 \cdot V_{nom}$

$$V_{fs} := \sum_{i=0}^{rows(V_b)-1} V_{b_i} = 10.617 \cdot kN$$

Total $V_{tot} := g \cdot (M_c + M_{pl} + M_p) + V_{fs} = 33.03 \cdot kN \quad < \quad F_{10kt} = 145.1 \cdot kN$ OK

3.3. Central Column - loaded with the attractions (levels 1, 2, 3 en 4)

$I := I_{10z} \qquad W := W_{10z} \qquad L := 11.9m$

This column is stiffly connected to the environment by the top beams dia 133 x 4

3.3.1. Zero load (only own weight of the attractions)

Horizontal forces

$$F := F_{zero} = \begin{pmatrix} 0.13 \\ 0.13 \\ 0.13 \\ 0.13 \\ 1.352 \\ 1.352 \\ 1.352 \\ 1.352 \end{pmatrix} \cdot kN \qquad h := \begin{pmatrix} 11.3 \\ 8.5 \\ 5.7 \\ 2.9 \\ 9.3 \\ 6.5 \\ 3.7 \\ 0.9 \end{pmatrix} m$$

Bending moment @ bottom $M_b := \sum_{i=0}^{rows(h)-1} (F_i \cdot h_i) = 3.61 \cdot kN \cdot m$

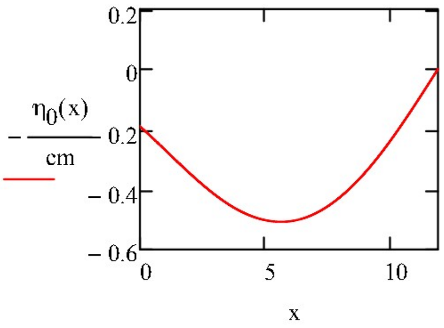
Bending stress @ bottom $\sigma_b := \frac{M_b}{W} = 5 \cdot MPa$

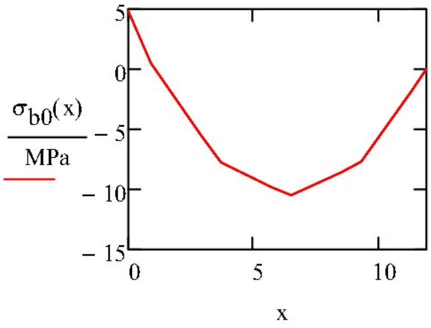
Maximum bending stress $\boxed{\sigma_{b.max} = 10.5 \cdot MPa}$

Displacement @ top $\eta_0(L) = -0.0 \cdot mm \qquad H_{top} = -2.324 \cdot kN$

Maximum displacement $\eta_{max} = 0.51 \cdot cm$ @ $x_{max} = 5.633\ m$

$$\frac{\eta_{max}}{L} = \frac{1}{2340}$$





3.3.2. Peak load - Direction 1

Horizontal forces

$$F := F_{peak} = \begin{pmatrix} 0.13 \\ 0.13 \\ 21.978 \\ 0.13 \\ 6.05 \\ 9.065 \\ 1.352 \\ 6.05 \end{pmatrix} \cdot kN \qquad h := \begin{pmatrix} 11.3 \\ 8.5 \\ 5.7 \\ 2.9 \\ 9.3 \\ 6.5 \\ 3.7 \\ 0.9 \end{pmatrix} m$$

Bending moment @ bottom $M_b := \sum_{i=0}^{rows(h)-1} \left(F_i \cdot h_i\right) = 36.18 \cdot kN \cdot m$

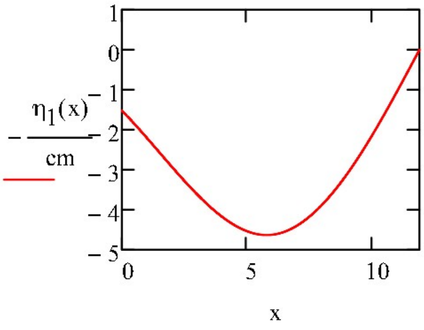
Bending stress @ bottom $\sigma_b := \frac{M_b}{W} = 48 \cdot MPa$

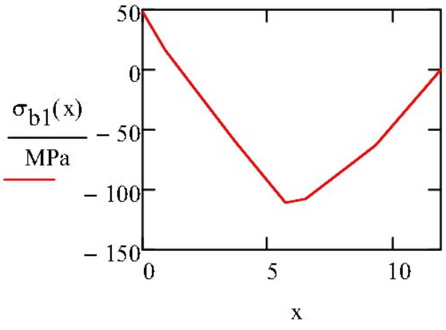
Maximum bending stress $\boxed{\sigma_{b.max} = 110.9 \cdot MPa}$

Displacement @ top $\eta_1(L) = -0.0 \cdot mm$ $H_{top} = -18.293 \cdot kN$

Maximum displacement $\eta_{max} = 4.64 \cdot cm$ @ $x_{max} = 5.787\ m$

$$\frac{\eta_{max}}{L} = \frac{1}{256}$$





3.3.3. Nominal load - Direction 2 and 3

Horizontal forces

$$F := F_{nom} = \begin{pmatrix} 0.13 \\ 0.13 \\ 4.992 \\ 0.13 \\ 6.05 \\ 6.05 \\ 1.352 \\ 6.05 \end{pmatrix} \cdot kN \qquad h := \begin{pmatrix} 11.3 \\ 8.5 \\ 5.7 \\ 2.9 \\ 9.3 \\ 6.5 \\ 3.7 \\ 0.9 \end{pmatrix} m$$

Bending moment @ bottom $M_b := \sum_{i=0}^{rows(h)-1} \left(F_i \cdot h_i\right) = 15.87 \cdot kN \cdot m$

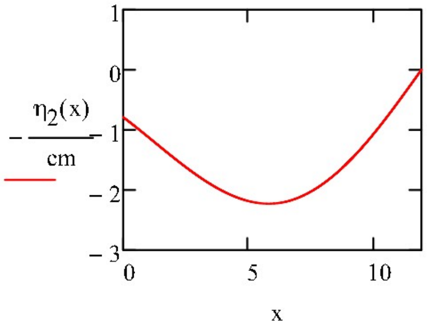
Bending stress @ bottom $\sigma_b := \frac{M_b}{W} = 21 \cdot MPa$

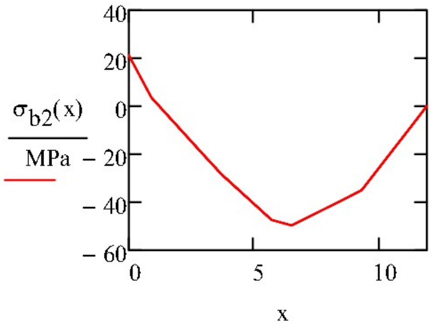
Maximum bending stress $\boxed{\sigma_{b.max} = 49.7 \cdot MPa}$

Displacement @ top $\eta_2(L) = -0.0 \cdot mm$ $H_{top} = -10.216 \cdot kN$

Maximum displacement $\eta_{max} = 2.23 \cdot cm$ @ $x_{max} = 5.804\,m$

$$\frac{\eta_{max}}{L} = \frac{1}{533}$$





3.3.4. Zero load - Directions 4, 5 and 6

Horizontal forces

$$F := F_{zero} = \begin{pmatrix} 0.13 \\ 0.13 \\ 0.13 \\ 0.13 \\ 1.352 \\ 1.352 \\ 1.352 \\ 1.352 \end{pmatrix} \cdot kN \qquad h := \begin{pmatrix} 11.3 \\ 8.5 \\ 5.7 \\ 2.9 \\ 9.3 \\ 6.5 \\ 3.7 \\ 0.9 \end{pmatrix} m$$

Bending moment @ bottom $M_b := \sum_{i=0}^{rows(h)-1} \left(F_i \cdot h_i\right) = 3.61 \cdot kN \cdot m$

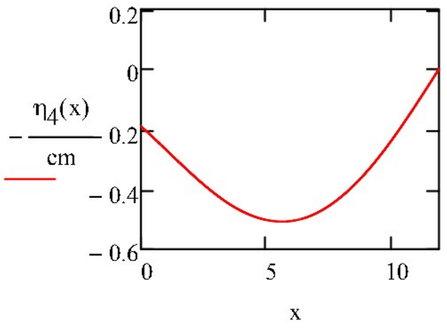
Bending stress @ bottom $\sigma_b := \frac{M_b}{W} = 5 \cdot MPa$

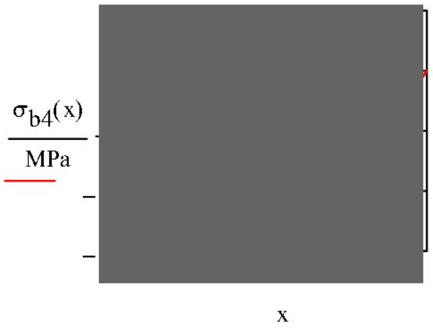
Maximum bending stress $\boxed{\sigma_{b.max} = 10.5 \cdot MPa}$

Displacement @ top $\eta_4(L) = -0.0 \cdot mm$ $\quad H_{top} = -2.324 \cdot kN$

Maximum displacement $\eta_{max} = 0.51 \cdot cm$ @ $x_{max} = 5.633\ m$

$$\frac{\eta_{max}}{L} = \frac{1}{2340}$$





3.3.5. Superposition of the six directions

$\theta_2 := -60deg \quad \theta_3 := 60deg \qquad f_1 := 1 \quad f_2 := 1 \quad f_3 := 1 \qquad M_3 = M_2$

$\theta_4 := -120deg \quad \theta_5 := 120deg \quad \theta_6 := 180deg \qquad f_4 := 1 \quad f_5 := 1 \quad f_6 := 1 \qquad M_4 = M_5 = M_6$

PEAK LOAD

Bending moment @ bottom $\boxed{M(0) = 44.81 \cdot kN \cdot m}$

components $M_I = 44.81 \cdot kN \cdot m$

$M_{II} = 0 \cdot kN \cdot m$

Lateral force @ bottom $\boxed{T(0) = 34.05 \cdot kN}$

components $T_I = 34.05 \cdot kN$

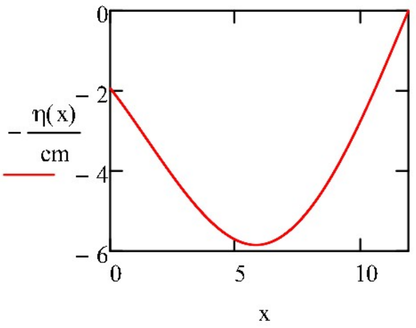
$T_{II} = 0 \cdot kN$

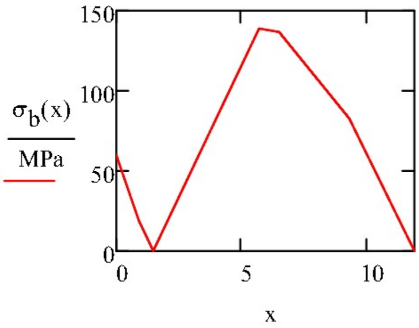
Bending stress @ bottom $\sigma_b(0) = 59.7 \cdot MPa$

Maximum bending stress $\boxed{\sigma_{b.max} = 138.8 \cdot MPa}$

Displacement @ top $\eta(L) = 0.0 \cdot mm$

Maximum displacement $\boxed{\eta_{max} = 5.85 \cdot cm}$ @ $x_{max} = 5.815\ m$ $\frac{\eta_{max}}{L} = \frac{1}{203}$





NOMINAL LOAD

Bending moment @ bottom $\boxed{M(0) = 24.5 \cdot kN \cdot m}$

components $M_I = 24.5 \cdot kN \cdot m$

$M_{II} = 0 \cdot kN \cdot m$

Lateral force @ bottom $\boxed{T(0) = 22.13 \cdot kN}$

components $T_I = 22.13 \cdot kN$

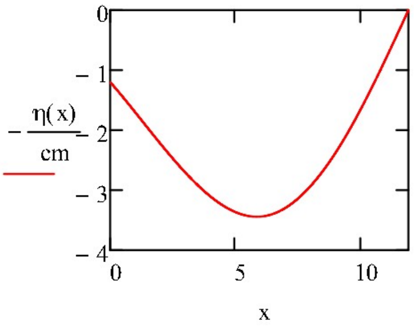
$T_{II} = 0 \cdot kN$

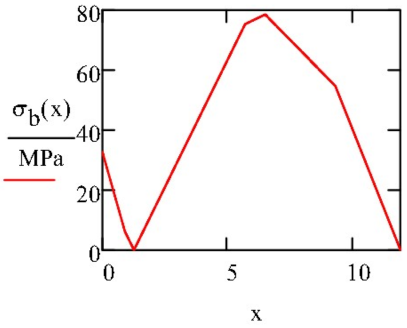
Bending stress @ bottom $\sigma_b(0) = 32.6\cdot\text{MPa}$

Maximum bending stress $\boxed{\sigma_{b.max} = 78.5\cdot\text{MPa}}$

Displacement @ top $\eta(L) = 0.0\cdot\text{mm}$

Maximum displacement $\boxed{\eta_{max} = 3.45\cdot\text{cm}}$ @ $x_{max} = 5.849\ \text{m}$ $\frac{\eta_{max}}{L} = \frac{1}{345}$





3.3.6. Vertical loads on the central column - buckling

Own weight columns $M_c := M_{10} = 925\,\text{kg}$

Platforms $M_{pl} := 4\cdot(185 + 75)\text{kg} = 1040\,\text{kg}$

Participants $M_p := 4\cdot 80\text{kg} = 320\,\text{kg}$

PEAK LOAD

Attractions $V_b := f_1\cdot V_{peak} + f_2\cdot V_{nom} + f_3\cdot V_{nom} + \left(f_4 + f_5 + f_6\right)\cdot V_{zero}$

$$V_{fs} := \sum_{i=0}^{\text{rows}(V_b)-1} V_{b_i} = 16.973\cdot\text{kN}$$

Total $V_{tot} := g\cdot\left(M_c + M_{pl} + M_p\right) + V_{fs} = 39.38\cdot\text{kN} < F_{10kt} = 145.1\cdot\text{kN}$ OK

NOMINAL LOAD

Attractions $V_b := f_1\cdot V_{nom} + f_2\cdot V_{nom} + f_3\cdot V_{nom} + \left(f_4 + f_5 + f_6\right)\cdot V_{zero}$

$$V_{fs} := \sum_{i=0}^{\text{rows}(V_b)-1} V_{b_i} = 14.169\cdot\text{kN}$$

Total $V_{tot} := g\cdot\left(M_c + M_{pl} + M_p\right) + V_{fs} = 36.58\cdot\text{kN} < F_{10kt} = 145.1\cdot\text{kN}$ OK

4. Top beams

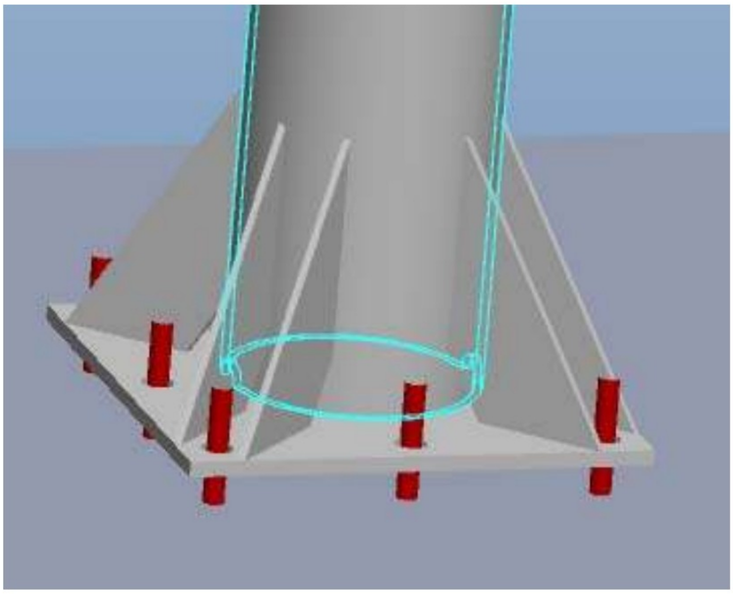
The maximum axial load on a top beam is $F_{a.max} = \max(H_{top})$

$F_{a.max} := 14.206kN$ < $F_{lt} = 28.79 \cdot kN$ OK

5. Connections

5.1. Columns on the concrete foundation

5.1.1. Threaded rods



Basis plate 600 mm x 600 mm x 20 mm
vertical reinforcement ribs, 10 mm thickness 360 mm height
3+2+3 threaded rods (M27 quality **8.8)**

Maxmum load occurs at outer columns. Since the rods are not configured in a circle, maximum load on the rods depends on the direction θ. Maximum moment, lateral force and vertical reaction on fundation are respectively:

$M_{max} = 52.043 \cdot kN \cdot m$ $T_{max} = 41.26 \cdot kN$ $V_{max} = 35.833 \cdot kN$ $\theta := 60deg$

The values with respect to the principal axes are then:

Horizontal $T_x := T_{max} \cdot \cos(\theta) = 20.63 \cdot kN$

$T_y := T_{max} \cdot \sin(\theta) = 35.732 \cdot kN$

Bending $M_x := M_{max} \cdot \cos(\theta) = 26.022 \cdot kN \cdot m$

$M_y := M_{max} \cdot \sin(\theta) = 45.071 \cdot kN \cdot m$

Threaded rods M27

Section	$A_S := 459\text{mm}^2$
0.2% Elasticity limit	$\sigma_S := 640\text{MPa}$
Allowed axial force	$F_{toeg} := A_S \cdot \frac{\sigma_S}{1.25} = 235 \cdot \text{kN}$
Side of plate	$h := 600\text{mm}$
Distance rods	$CC := 250\text{mm}$

The maximum traction in a threaded rod is assessed as follows:

$$h_0 := \frac{h - 2 \cdot CC}{2} = 50 \cdot \text{mm} \qquad l_0 := h_0 - \frac{h}{4} = -100 \cdot \text{mm}$$

$$h_1 := \frac{h}{2} = 300 \cdot \text{mm} \qquad l_1 := h_1 - \frac{h}{4} = 150 \cdot \text{mm}$$

$$h_2 := h - h_0 = 550 \cdot \text{mm} \qquad l_2 := h_2 - \frac{h}{4} = 400 \cdot \text{mm}$$

$$F_{max} := M_x \cdot \frac{l_2}{\left(2 \cdot l_1{}^2 + 3 \cdot l_2{}^2\right)} + M_y \cdot \frac{l_2}{\left(2 \cdot l_1{}^2 + 3 \cdot l_2{}^2\right)} = 54.2 \cdot \text{kN} \quad < \quad F_{toeg} = 235 \cdot \text{kN} \quad \text{OK}$$

Shear stress round the rods hole in the base plate

$$\phi_f := 40\text{mm} \qquad t := 20\text{mm}$$

--> $\tau := \frac{F_{max}}{\pi \cdot \phi_f \cdot t} = 21.55 \cdot \text{MPa}$

Surface pressure on the concrete (l'Eurocode 3 "Joints dans les constructions métalliques")

$$f_{jd} := \frac{2}{3} \cdot 1.5 \cdot 0.85 \cdot \frac{20\text{MPa}}{1.5} = 11.333 \cdot \text{MPa}$$

$$F_d := \frac{V_{max}}{h^2} = 0.1 \cdot \frac{\text{N}}{\text{mm}^2} \quad < \quad f_{jd} = 11.333 \cdot \frac{\text{N}}{\text{mm}^2} \quad \text{OK}$$

▶

Thickness plate is 20 mm - <u>reinforced</u> by upright ribs 360 mm (280 mm eff.) x 10 mm

Total bending reistance W and bending stress σ_b :

$$z := \frac{2 \cdot 280\text{mm} \cdot 10\text{mm} \cdot 240\text{mm} + 20\text{mm} \cdot 300\text{mm} \cdot 10\text{mm}}{2 \cdot 10\text{mm} \cdot 280\text{mm} + 300\text{mm} \cdot 20\text{mm}} = 121 \cdot \text{mm}$$

$$I := 2\cdot\left[\frac{10\mathrm{mm}\cdot(280\mathrm{mm})^3}{12} + (240\mathrm{mm} - z)^2\cdot 10\mathrm{mm}\cdot 240\mathrm{mm}\right] + \frac{300\mathrm{mm}\,(20\mathrm{mm})^3}{12} + (z - 10\mathrm{mm})^2\cdot 300\mathrm{mm}\cdot 20\mathrm{mm}$$

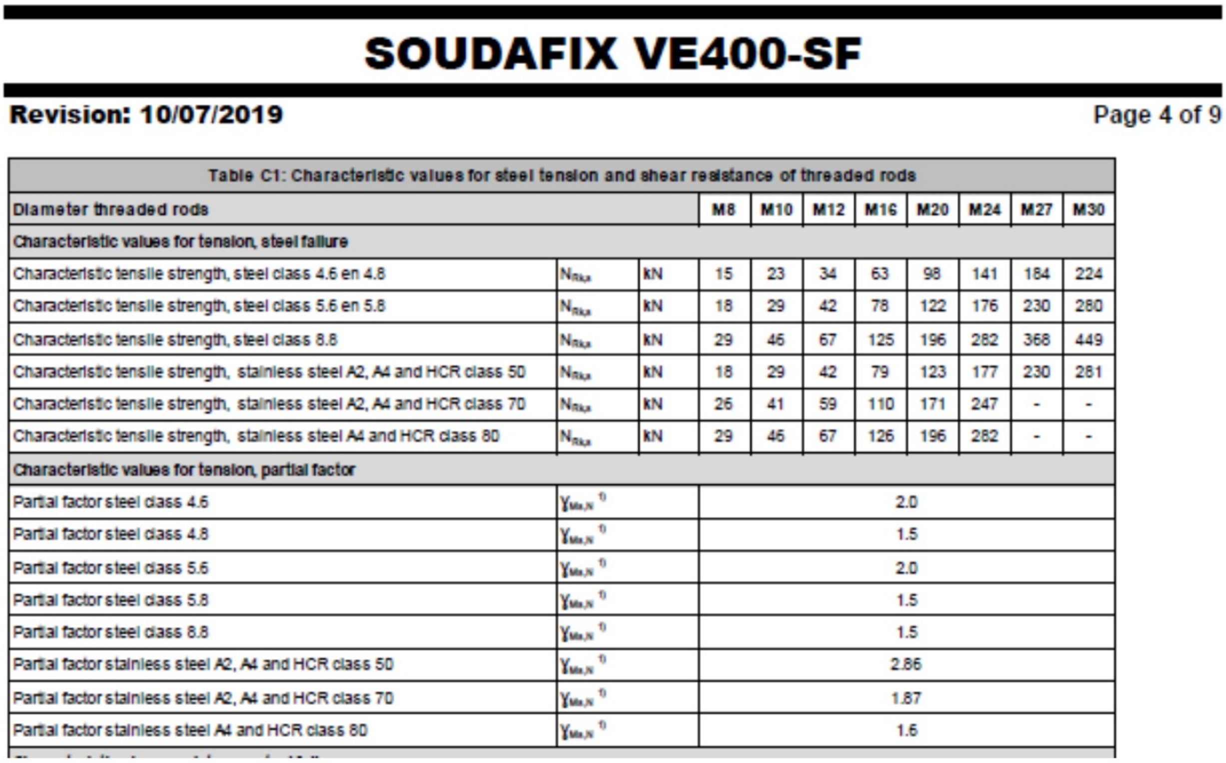
$$W := \min\left[\frac{I}{(380\mathrm{mm} - z)}, \frac{I}{z}\right] = 690\cdot\mathrm{cm}^3$$

$$\sigma_{b1} := \frac{F_{max}\cdot\left[\frac{\sqrt{2}}{2}\cdot(2\cdot CC) - \frac{\phi_{8o}}{2}\right]}{W} = 15\cdot\mathrm{MPa} \qquad \text{OK}$$

$$\sigma_{b2} := \frac{3\cdot F_{max}\cdot\left(CC - \frac{\phi_{8o}}{2}\right)}{W} = 20.7\cdot\mathrm{MPa} \qquad \text{OK}$$

5.1.2. Anchoring resin SOUDAFIX VE400-SF (only in case the threaded rods are anchored with Soudafix)

The anchoring resin SOUDAFIX VE400-SF: Technical data Sheet Rev. 10/07/2019 Tables C1 and C2 gives the characteristic values and partial factors for concrete C20/25.

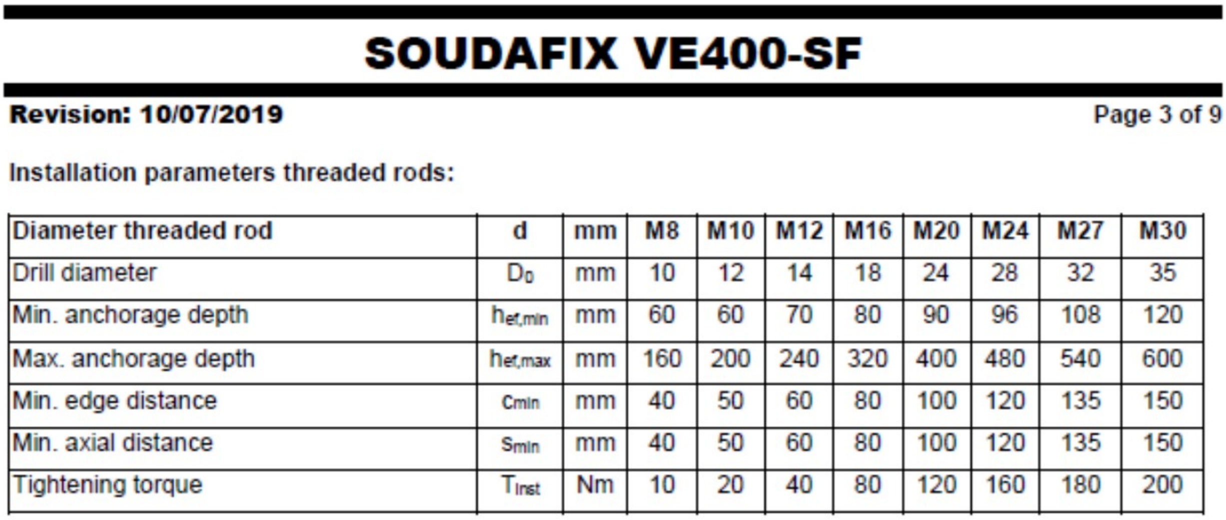


SOUDAFIX VE400-SF

Revision: 10/07/2019 Page 4 of 9

Table C1: Characteristic values for steel tension and shear resistance of threaded rods										
Diameter threaded rods			**M8**	**M10**	**M12**	**M16**	**M20**	**M24**	**M27**	**M30**
Characteristic values for tension, steel failure										
Characteristic tensile strength, steel class 4.6 en 4.8	$N_{Rk,s}$	kN	15	23	34	63	98	141	184	224
Characteristic tensile strength, steel class 5.6 en 5.8	$N_{Rk,s}$	kN	18	29	42	78	122	176	230	280
Characteristic tensile strength, steel class 8.8	$N_{Rk,s}$	kN	29	46	67	125	196	282	368	449
Characteristic tensile strength, stainless steel A2, A4 and HCR class 50	$N_{Rk,s}$	kN	18	29	42	79	123	177	230	281
Characteristic tensile strength, stainless steel A2, A4 and HCR class 70	$N_{Rk,s}$	kN	26	41	59	110	171	247	-	-
Characteristic tensile strength, stainless steel A4 and HCR class 80	$N_{Rk,s}$	kN	29	46	67	126	196	282	-	-
Characteristic values for tension, partial factor										
Partial factor steel class 4.6	$\gamma_{Ms,N}$ 1)		2.0							
Partial factor steel class 4.8	$\gamma_{Ms,N}$ 1)		1.5							
Partial factor steel class 5.6	$\gamma_{Ms,N}$ 1)		2.0							
Partial factor steel class 5.8	$\gamma_{Ms,N}$ 1)		1.5							
Partial factor steel class 8.8	$\gamma_{Ms,N}$ 1)		1.5							
Partial factor stainless steel A2, A4 and HCR class 50	$\gamma_{Ms,N}$ 1)		2.86							
Partial factor stainless steel A2, A4 and HCR class 70	$\gamma_{Ms,N}$ 1)		1.87							
Partial factor stainless steel A4 and HCR class 80	$\gamma_{Ms,N}$ 1)		1.6							

$$F_{allow} := 368\mathrm{kN} \qquad \gamma := 1.5 \qquad F_{max} = 54\cdot\mathrm{kN} \quad < \quad \frac{F_{allow}}{\gamma} = 245\cdot\mathrm{kN} \qquad \text{OK}$$



SOUDAFIX VE400-SF

Revision: 10/07/2019 Page 3 of 9

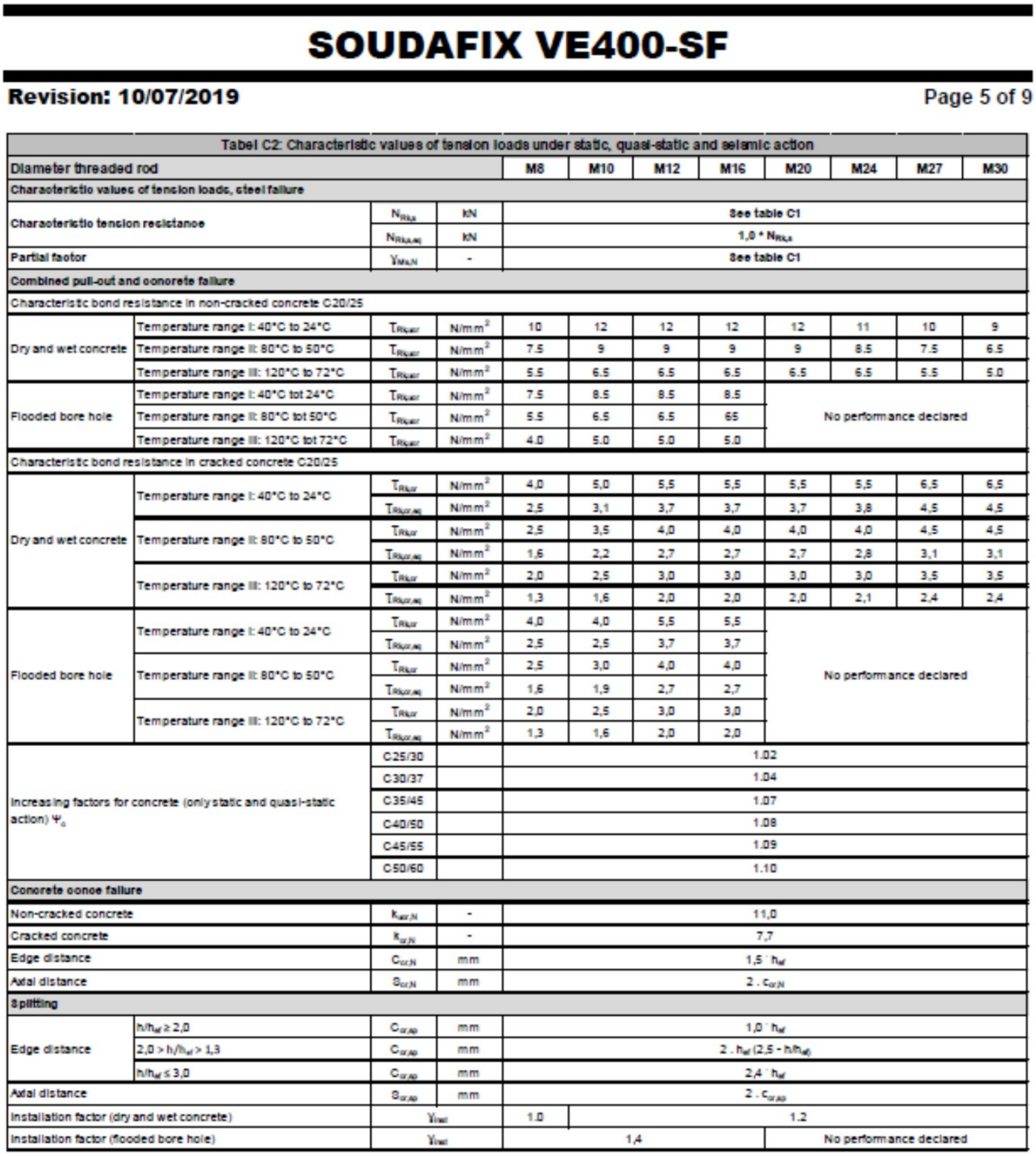
Installation parameters threaded rods:

Diameter threaded rod	**d**	**mm**	**M8**	**M10**	**M12**	**M16**	**M20**	**M24**	**M27**	**M30**
Drill diameter	D_0	mm	10	12	14	18	24	28	32	35
Min. anchorage depth	$h_{ef,min}$	mm	60	60	70	80	90	96	108	120
Max. anchorage depth	$h_{ef,max}$	mm	160	200	240	320	400	480	540	600
Min. edge distance	c_{min}	mm	40	50	60	80	100	120	135	150
Min. axial distance	s_{min}	mm	40	50	60	80	100	120	135	150
Tightening torque	T_{inst}	Nm	10	20	40	80	120	160	180	200

Drill diameter $\phi_f := 32\text{mm}$

Min. anchorage dept $h_{min} := 108\text{mm}$

Max. anchorage depth $h_{max} := 540\text{mm}$



SOUDAFIX VE400-SF

Revision: 10/07/2019 Page 5 of 9

Tabel C2: Characteristic values of tension loads under static, quasi-static and seismic action											
Diameter threaded rod				M8	M10	M12	M16	M20	M24	M27	M30
Characteristic values of tension loads, steel failure											
Characteristic tension resistance		$N_{Rk,s}$	kN	See table C1							
		$N_{Rk,s,eq}$	kN	1,0 * $N_{Rk,s}$							
Partial factor		$\gamma_{Ms,N}$	-	See table C1							
Combined pull-out and concrete failure											
Characteristic bond resistance in non-cracked concrete C20/25											
Dry and wet concrete	Temperature range I: 40°C to 24°C	$\tau_{Rk,ucr}$	N/mm²	10	12	12	12	12	11	10	9
	Temperature range II: 80°C to 50°C	$\tau_{Rk,ucr}$	N/mm²	7.5	9	9	9	9	8.5	7.5	6.5
	Temperature range III: 120°C to 72°C	$\tau_{Rk,ucr}$	N/mm²	5.5	6.5	6.5	6.5	6.5	6.5	5.5	5.0
Flooded bore hole	Temperature range I: 40°C tot 24°C	$\tau_{Rk,ucr}$	N/mm²	7.5	8.5	8.5	8.5	No performance declared			
	Temperature range II: 80°C tot 50°C	$\tau_{Rk,ucr}$	N/mm²	5.5	6.5	6.5	65				
	Temperature range III: 120°C tot 72°C	$\tau_{Rk,ucr}$	N/mm²	4.0	5.0	5.0	5.0				
Characteristic bond resistance in cracked concrete C20/25											
Dry and wet concrete	Temperature range I: 40°C to 24°C	$\tau_{Rk,cr}$	N/mm²	4,0	5,0	5,5	5,5	5,5	5,5	6,5	6,5
		$\tau_{Rk,cr,eq}$	N/mm²	2,5	3,1	3,7	3,7	3,7	3,8	4,5	4,5
	Temperature range II: 80°C to 50°C	$\tau_{Rk,cr}$	N/mm²	2,5	3,5	4,0	4,0	4,0	4,0	4,5	4,5
		$\tau_{Rk,cr,eq}$	N/mm²	1,6	2,2	2,7	2,7	2,7	2,8	3,1	3,1
	Temperature range III: 120°C to 72°C	$\tau_{Rk,cr}$	N/mm²	2,0	2,5	3,0	3,0	3,0	3,0	3,5	3,5
		$\tau_{Rk,cr,eq}$	N/mm²	1,3	1,6	2,0	2,0	2,0	2,1	2,4	2,4
Flooded bore hole	Temperature range I: 40°C to 24°C	$\tau_{Rk,cr}$	N/mm²	4,0	4,0	5,5	5,5	No performance declared			
		$\tau_{Rk,cr,eq}$	N/mm²	2,5	2,5	3,7	3,7				
	Temperature range II: 80°C to 50°C	$\tau_{Rk,cr}$	N/mm²	2,5	3,0	4,0	4,0				
		$\tau_{Rk,cr,eq}$	N/mm²	1,6	1,9	2,7	2,7				
	Temperature range III: 120°C to 72°C	$\tau_{Rk,cr}$	N/mm²	2,0	2,5	3,0	3,0				
		$\tau_{Rk,cr,eq}$	N/mm²	1,3	1,6	2,0	2,0				
Increasing factors for concrete (only static and quasi-static action) Ψ_c		C25/30		1.02							
		C30/37		1.04							
		C35/45		1.07							
		C40/50		1.08							
		C45/55		1.09							
		C50/60		1.10							
Concrete cone failure											
Non-cracked concrete		$k_{ucr,N}$	-	11,0							
Cracked concrete		$k_{cr,N}$	-	7,7							
Edge distance		$c_{cr,N}$	mm	1,5 · h_{ef}							
Axial distance		$s_{cr,N}$	mm	2 . $c_{cr,N}$							
Splitting											
Edge distance	$h/h_{ef} \geq 2,0$	$c_{cr,sp}$	mm	1,0 · h_{ef}							
	$2,0 > h/h_{ef} > 1,3$	$c_{cr,sp}$	mm	2 . h_{ef} (2,5 - h/h_{ef})							
	$h/h_{ef} \leq 3,0$	$c_{cr,sp}$	mm	2,4 · h_{ef}							
Axial distance		$s_{cr,sp}$	mm	2 . $c_{cr,sp}$							
Installation factor (dry and wet concrete)		γ_{inst}		1.0	1.2						
Installation factor (flooded bore hole)		γ_{inst}		1,4				No performance declared			

Minimum required drill depth $h_{ef} := 330mm$

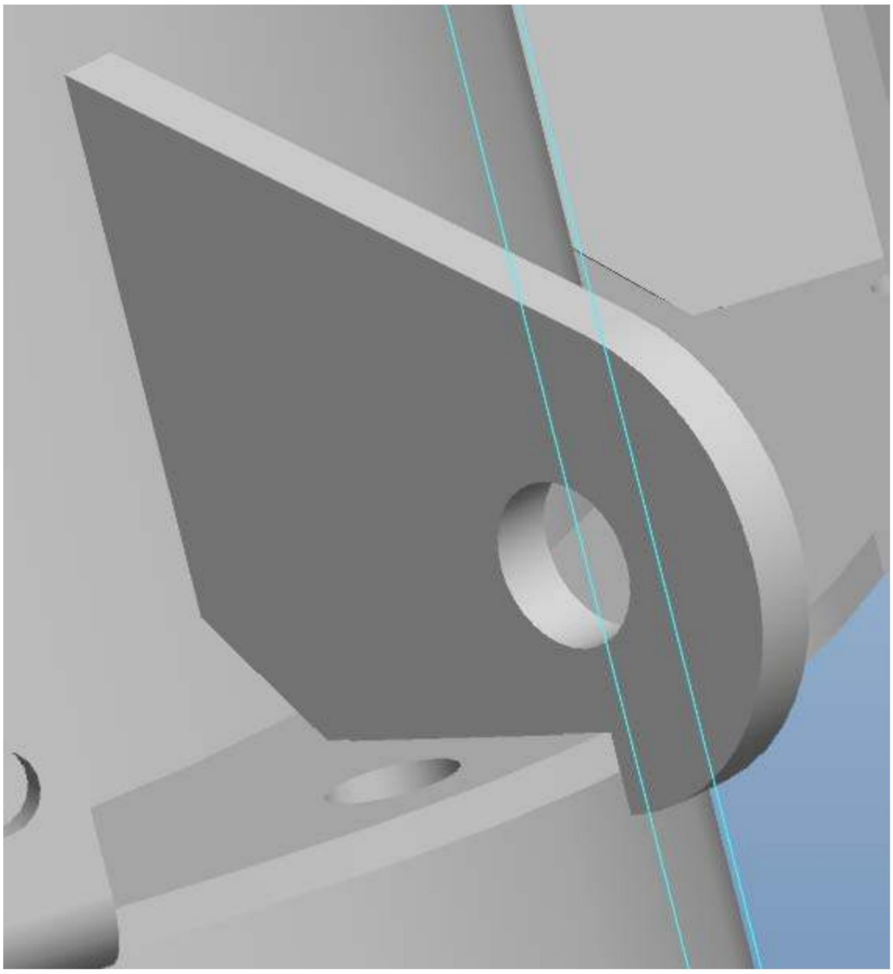
partial factor $\gamma := 1.8$ *We don't take 1.5 but the value mentioned in the version 2014 which was 1.8*

characteristic bound resistance for C20/25 $\tau_k := 10\frac{N}{mm^2}$

for C25/30 --> $\psi := 1.02$

--> $\tau := \frac{F_{max}}{\pi \cdot \phi_f \cdot h_{ef}} = 1.63 \cdot MPa$ < $\tau_d := \psi \cdot \frac{\tau_k}{\gamma} = 5.67 \cdot MPa$ OK

5.2. Attachment eye - safety (and/or foot) cables



hole diameter $\boxed{\phi_i := 20mm}$

outer diameter $\boxed{\phi_o := 60mm}$

thickness $t := 8mm$

Allowed tension stress $\sigma_{Tt} := 160MPa$ (steel 235)

angle between force F working direction and horizontal plane θ

axial component $T = F \cdot \cos(\theta)$

Shear comp0nent $D = F \cdot \sin(\theta)$

Stress in section through the hole

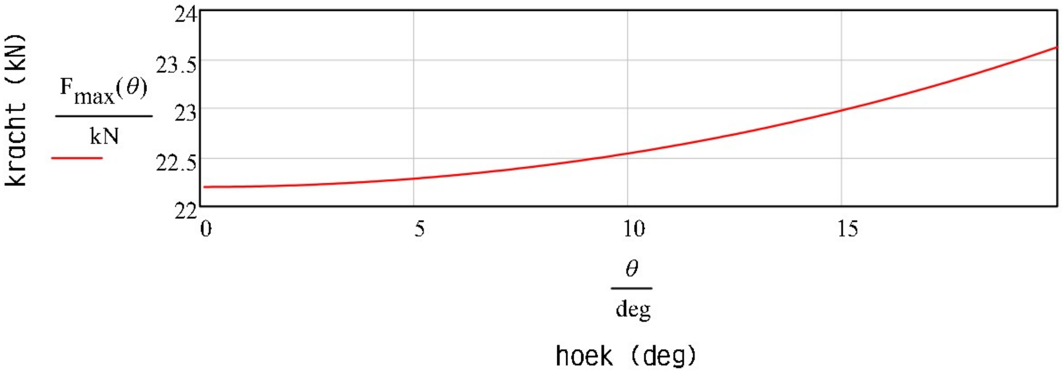
Roarks Formulas for stress and strain, Table 17.1, Case 7

stress concentration factor $K_t := 3.00 - 3.13 \cdot \left(\frac{\phi_i}{\phi_o}\right) + 3.66 \cdot \left(\frac{\phi_i}{\phi_o}\right)^2 - 1.53 \cdot \left(\frac{\phi_i}{\phi_o}\right)^3 = 2.307$

max. tension stress $$\sigma_{max} = K_t \cdot \frac{T}{t \cdot (\phi_o - \phi_i)}$$

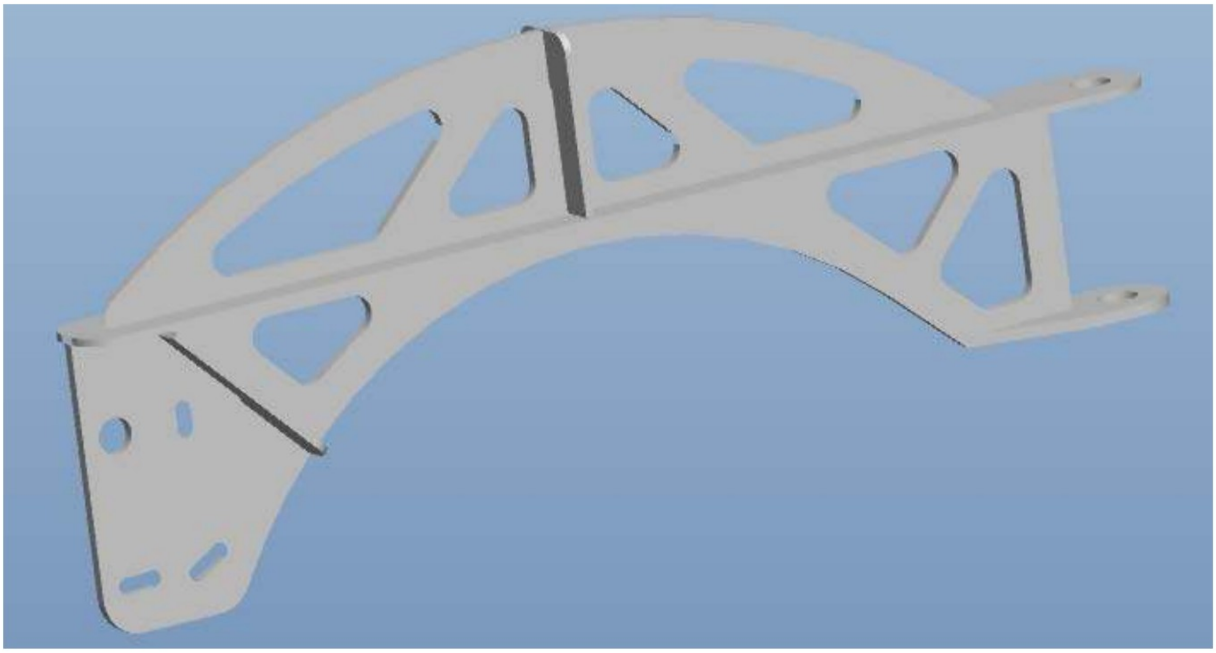
shear stress $$\tau = \frac{D}{t \cdot (H - \phi)}$$

maximum allowed force $$F_{max}(\theta) := \min\left[\frac{\sigma_{Tt} \cdot t \cdot (\phi_o - \phi_i)}{\cos(\theta) \cdot K_t}, \frac{\sigma_{Tt}}{1.7} \cdot \frac{t \cdot (\phi_o - \phi_i)}{\sin(\theta)}\right]$$



OK

5.3. Connection arc



Load on the arc caused by the safety cables:

horizontal force $H := 21.967\text{kN}$

vertical force $V := 3.023\text{kN}$

point d' accroche par rapport au fond $h_a := 30\text{mm}$

Dimensions of the arc

thickness $t := 6\text{mm}$ $\boxed{h := 111\text{mm}}$

horizontal reinforcement $t_v := 4\text{mm}$ $b_v := 50\text{mm}$

length $L_b := 560\text{mm}$

distance pointe d'accroche $d_a := 128\text{mm}$

1. The weakest section is in the middle of the arc

section $\Omega := t \cdot h = 6.66 \cdot \text{cm}^2$ $z := \frac{h}{2} = 55.5 \cdot \text{mm}$

$$\Omega_v := t_v \cdot b_v = 2 \cdot \text{cm}^2 \qquad z_v := 93\text{mm} + \frac{t_v}{2} = 95 \cdot \text{mm}$$

$$\Omega_t := \Omega + \Omega_v = 8.66 \cdot \text{cm}^2 \qquad z_t := \frac{\Omega \cdot z + \Omega_v \cdot z_v}{\Omega_t} = 64.622 \cdot \text{mm}$$

$$I_t := \frac{t \cdot h^3}{12} + \left(z_t - z\right)^2 \cdot \Omega + \frac{b_v \cdot t_v^{\,3}}{12} + \left(z_t - z_v\right)^2 \cdot \Omega_v = 92.407 \cdot \text{cm}^4$$

$$W_t := \min\left(\frac{I_t}{z}, \frac{I_t}{h - z}\right) = 16.65 \cdot \text{cm}^3$$

bending moment

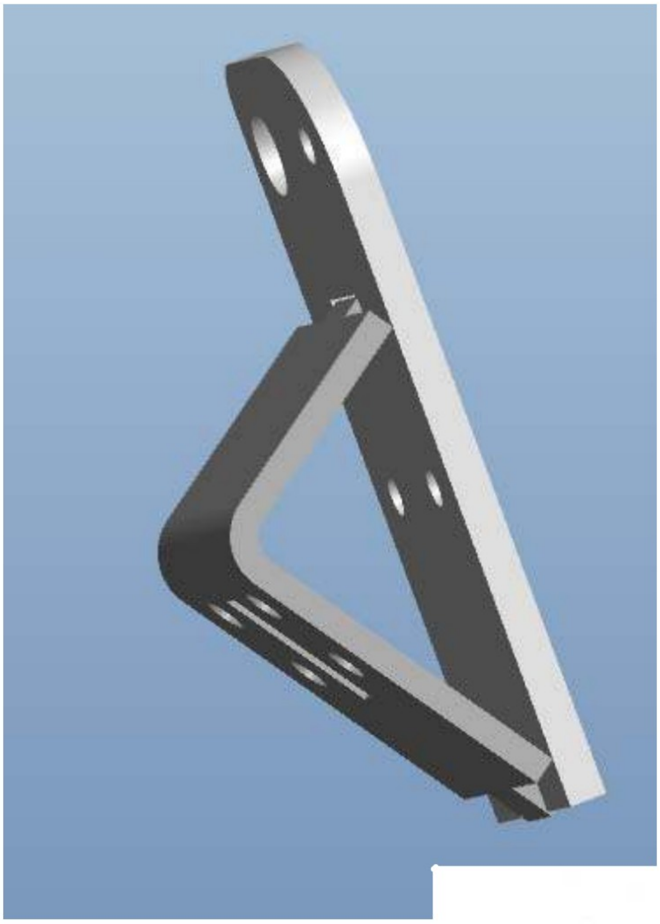
$$M := H \cdot \left[h_a + d_a + \left(z_v - z_t\right)\right] - V \cdot \frac{L_b}{2} = 3.292 \cdot \text{kN} \cdot \text{m}$$

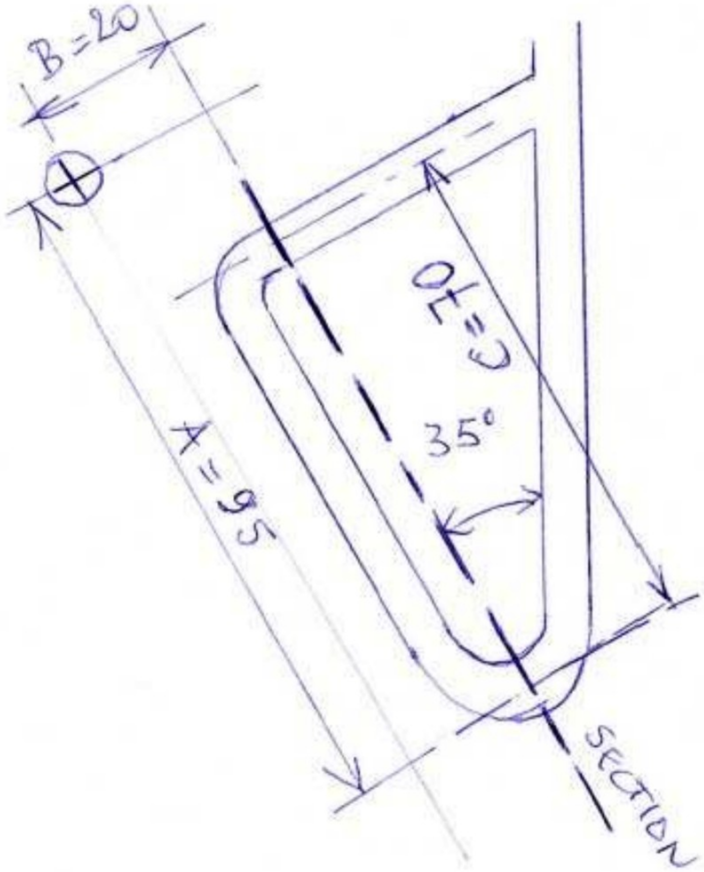
bending stress

$$\sigma_b := \frac{M}{W_t} = 198 \cdot \text{MPa}$$

this stress is relatively high, but less than the admitted value $\sigma_P = 240 \cdot \text{MPa}$, since this peak value load rarely occurs (fall of a participant).

2. Bended plate 6 mm thick





thickness bended plate:

$t := 6mm$

width bended plate:

$b := 75mm$

The horzontal force $H = 21.967\cdot kN$ and vertical force $V = 3.023\cdot kN$ causes following forces and moments in the section:

$\theta := 35deg$ $A := 95mm$ $B := 20mm$ $C := 70mm$

$H = 21.967\cdot kN$ $V = 3.023\cdot kN$

shear in X	H	bending round Z	$H\cdot B$
shear in Z	$V\cdot\cos(\theta)$	torsion round Y	$H\cdot\left(A - \frac{C}{2}\right)$
tension in Y	$V\cdot\sin(\theta)$	bending round X	$V\cdot\cos(\theta)\cdot B + V\cdot\sin(\theta)\cdot\left(A - \frac{C}{2}\right)$

this set of actions induces the next stresses in upper and lower sections :

upper section:

$$\tau_{xu} := \frac{H}{2\cdot t\cdot b} + \frac{H\cdot\left(A - \frac{C}{2}\right)}{2\cdot\left(\frac{C - t}{2}\right)\cdot t\cdot b} = 70\cdot MPa$$

$$\tau_{zu} := \frac{V\cdot\cos(\theta)}{2\cdot t\cdot b} = 3\cdot MPa$$

$$\sigma_{ul} := \frac{H\cdot B}{2\cdot\frac{t\cdot b^2}{6}} + \frac{V\cdot\sin(\theta)}{2\cdot t\cdot b} + \left[\frac{V\cdot\cos(\theta)\cdot B + V\cdot\sin(\theta)\cdot\left(A - \frac{C}{2}\right)}{2\cdot\left(\frac{C - t}{2}\right)\cdot t\cdot b}\right] = 46\cdot MPa$$

or

$$\sigma_{ur} := \frac{H\cdot B}{2\cdot\frac{t\cdot b^2}{6}} - \frac{V\cdot\sin(\theta)}{2\cdot t\cdot b} + \frac{V\cdot\cos(\theta)\cdot B + V\cdot\sin(\theta)\cdot\left(A - \frac{C}{2}\right)}{2\cdot\left(\frac{C - t}{2}\right)\cdot t\cdot b} = 42\cdot MPa$$

lower section

$$\tau_{xl} := \frac{H}{2\cdot t\cdot b} - \frac{H\cdot\left(A - \frac{C}{2}\right)}{2\cdot\left(\frac{C - t}{2}\right)\cdot t\cdot b} = -21\cdot MPa$$

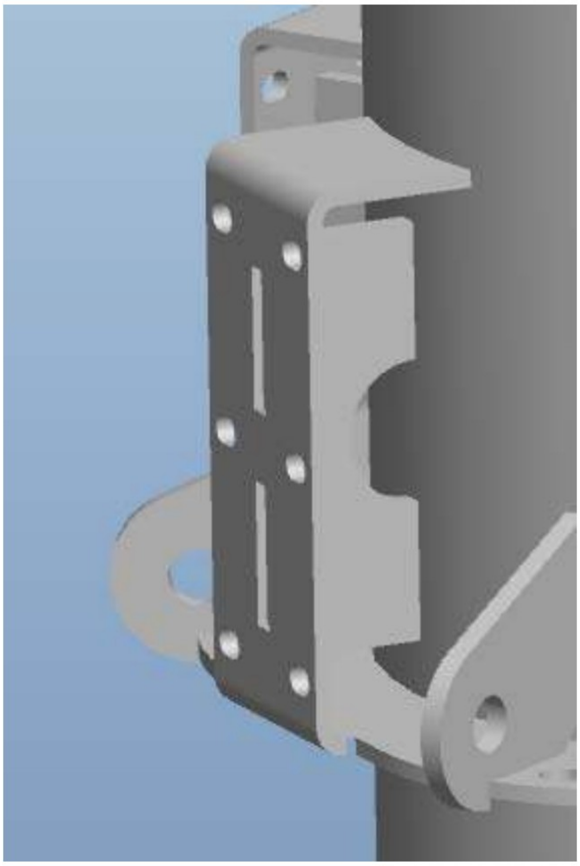
$$\tau_{zl} := \frac{V\cdot\cos(\theta)}{2\cdot t\cdot b} = 3\cdot MPa$$

$$\sigma_{ll} := \frac{H\cdot B}{2\cdot\frac{t\cdot b^2}{6}} + \frac{V\cdot\sin(\theta)}{2\cdot t\cdot b} - \frac{V\cdot\cos(\theta)\cdot B + V\cdot\sin(\theta)\cdot\left(A - \frac{C}{2}\right)}{2\cdot\left(\frac{C - t}{2}\right)\cdot t\cdot b} = 36\cdot MPa$$

or

$$\sigma_{lr} := \frac{H\cdot B}{2\cdot\frac{t\cdot b^2}{6}} - \frac{V\cdot\sin(\theta)}{2\cdot t\cdot b} - \frac{V\cdot\cos(\theta)\cdot B + V\cdot\sin(\theta)\cdot\left(A - \frac{C}{2}\right)}{2\cdot\left(\frac{C - t}{2}\right)\cdot t\cdot b} = 32\cdot MPa$$

5.4. Joint of the arc on the column



dimensions U-plate

thickness $t := 6\text{mm}$

width $b := 80\text{mm}$

distance to column

$c := 55\text{mm}$

distance between the horizontal plates

$d := 193\text{mm}$

forces in the safety cable

$H = 21.967\cdot\text{kN}$

$V = 3.023\cdot\text{kN}$

Stresses les tensions si le câble est prolongée au profil U

traction force in upper part

$$T_o := \frac{H\cdot(d - 20\text{mm})}{d} - \frac{V\cdot\left(L_b + c\right)}{d} = 10.058\cdot\text{kN}$$

$$\sigma_{to} := \frac{T_o}{b\cdot t} = 20.95\cdot\text{MPa}$$

traction in lower part

$$T_b := \left[H - \frac{H\cdot(d - 20\text{mm})}{d}\right] + \frac{V\cdot\left(L_b + c\right)}{d} = 11.91\cdot\text{kN}$$

$$\sigma_{tb} := \frac{T_b}{b\cdot t} = 24.81\cdot\text{MPa}$$

shear stress $\tau_o := \dfrac{V}{2\cdot b\cdot t} = 3.149\cdot\text{MPa}$

6. Conclusion and advice

When calculating the forces that are exerted by the **attractions** on the columns, the cables are supposed to be preloaded in such a way that at zero load (only own weight of the attractions), the cables have at least following sags:
- foot cables: 5%
- safety cables: 5%

A maximum of 1 participant (120 kg) is allowed on each attraction. Safety cables are loaded at most by a vertical load of 6 kN as a result of a falling participant. The tension in a safety cables is always less then a third of its breaking strength.

Since the columns are supported by drill piles which inherently exhibit flexibility, the reaction forces and moments at the top of these drill piles (foundation) are limited at a level such that there is no need to guy the columns by guy cables.

With the above mentioned conditions, the columns (all have outer diameter 323.9 mm) must have minimal thicknesses of 10 mm.