NEW EUROCODE2 PROJECT

DETAILED CALCULATION REPORT

Think Concrete, Go Precast

Milan, 27/06/2024

Update 2.1

DLC Consulting, the President

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Client: **BIBM**

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1 Premise

The present report contains the description of the results obtained within the EC2 project sponsored by BIBM (Federation of the European Precast Concrete Industry) following the call "Evolution of design provisions for precast concrete - Call for external technical expertise". DLC Consulting s.r.l. was selected for this activity.

Eurocode 2 – "EN 1992-1-1 (2004) Design of concrete structures - Part 1-1: General rules and rules for buildings" is the current version of the technical rules for the design of reinforced and prestressed concrete structures; EN 1992-1-2 includes the fire design for the same concrete structures. These standards are presently under revision (CEN Formal Vote stage) and, if approved, will replace the 2004 versions in the years to come (at latest in 2028) and become the new version.

The work was developed following, in addition to the current Eurocode standards under validity, the current drafts of the new documents, namely:

- FprEN1992-1-1:2022
- FprEN1992-1-2:2022

The drafts of the new versions, supplied by BIBM, include major changes compared to the current, and the interpretation of the impact of these modifications over the design of precast elements is difficult due to their large quantity, to be analysed as a whole rather than as single contributions. Within this project, selected structural elements representative of the European precast concrete industry for commercial/residential buildings were designed in detail following both current and draft new versions of the Eurocode 2 documents, considering an integrated structural design.

It is to be reminded that the activity concerns structural design, which is not an exact science. Many variables contribute to the achievement of the required performances, and what presented in this report is the result of reasonable and recurrent choices made by a team of experts in the field of design and research of precast concrete structures. Nevertheless, different arrangements could have fulfilled the same performance requests, so it is assumed that a certain subjectivity of the results is to be expected.

This report contains a detailed description of the structural calculations and results of the structural design of the selected members. Excerpts from the original calculation spreadsheets are given for the best clarity in the design procedure adopted. All deviations encountered during the design process are listed in the document.

Moreover, shop drawings made with 3D software are provided in order to transfer more efficiently to the reader the structural arrangement of the members, including detailed views of their assembled reinforcing cages.

The environmental impact in terms of the recent trends of decarbonisation, material efficiency and circular economy is evaluated on the basis of the detailed quantities of materials employed in the analysed elements and of Environmental Product Declarations (EPDs) deemed to be representative of the European production.

Brief notes concerning how to read the plotted instructions of the software Mathcad15:

:= definition of a function or a value

= calculation or recall of a value

= is a Boolean condition (imposition of equality) necessary to set solver instructions

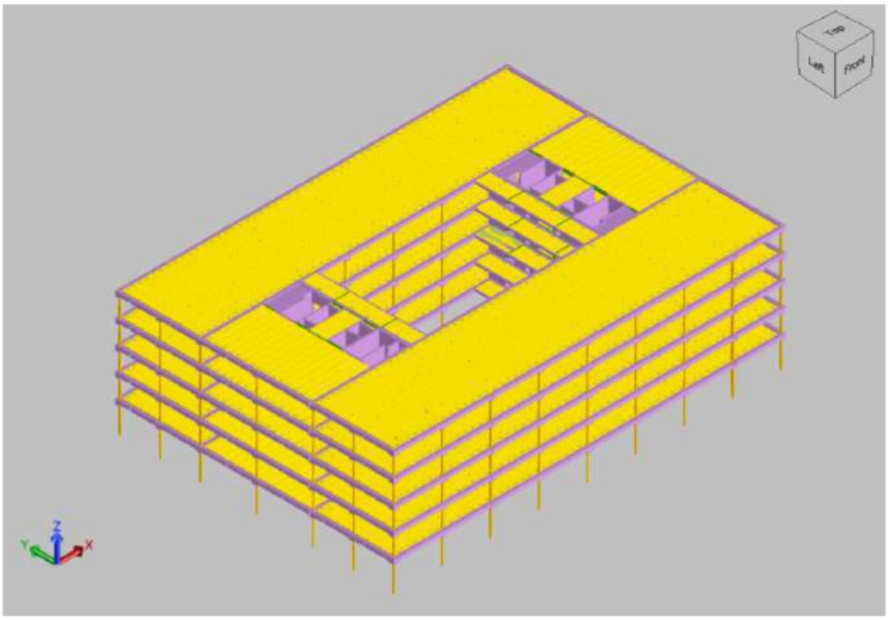
Extension strain and tension stress/load have positive sign “ + ”; shrinkage strain and compression stress/load have negative sign “ - ”.

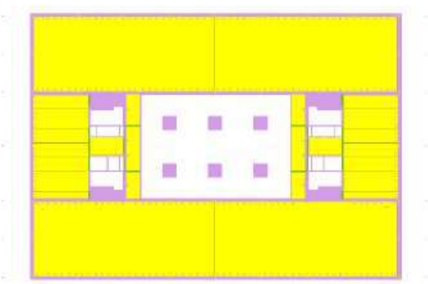
Operations are always done top to bottom (no external routines are employed) -> all members of an equation need to be defined prior to the writing of the equation itself.

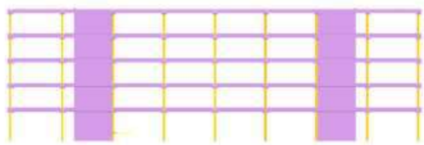
2 Case study building and numerical modelling

The case study building was assigned by the team of experts of BIBM. It consists of a 5-storey building above the ground only, with rectangular plan having a central rectangular court and two distribution cores (stairs, lifts, and MEP system main distribution) along the shorter court sides. The total covered area is about 15000 m^2.

Pictures of the provided drawings are collected in the following:





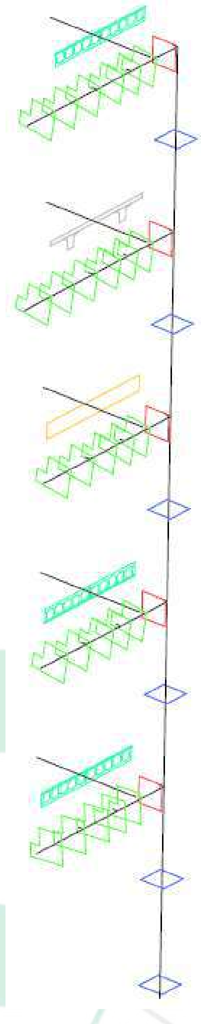


From the structural point of view, the building can be classified as a frame system braced by wall cores. The floors are assumed to be made with different technologies:

- 1st floor: adjacent precast hollowcore members
- 2nd floor: adjacent precast hollowcore members
- 3rd floor: precast lattice girders completed with additional reinforcement and cast-in-situ concrete pouring
- 4th floor: adjacent precast TT members

- 5th floor: adjacent hollowcore members

A sketch of the structural model showing the different typologies of floor is presented in the next figure:

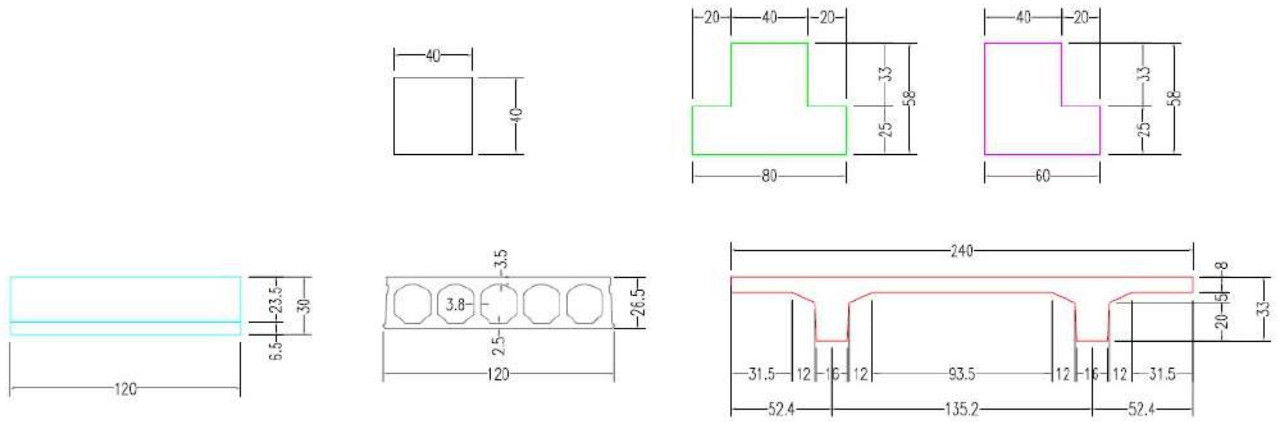


The floor elements are supported by L-shaped and inverted-T beams in the edge and centre of the slab, respectively. Following the assumed structural scheme, it is logical to assume that beams are protruding from the soffit, and floor members are supported over the beam corbels.

A numerical model was developed with the finite element software Straus7 (Strand7), release 2.4.6., with the aim to find out the actions on the different elements. Indeed, being all horizontal members simply supported (floor elements over beam corbels, and beam elements over column corbels), the model is not necessary for the determination of the action in the horizontal members, but rather on the vertical columns, which form part of the lateral load resisting system of the building. It is specified that the diaphragmatic action actually insists on the horizontal elements, since a reinforced concrete topping is not present at any storey, and therefore the horizontal loads are conveyed to the bracing cores through the horizontal members and their connections. These connections were included in the structural model as simplified elastic springs, with assigned stiffness from experimental tests of typical dowel or angle connections. The results showed that the out-of-plane bending and shear acting on the horizontal members, as well as the actions on the connections, are so low that they can be considered negligible (horizontal load from wind only is taken into account, neglecting seismic action).

Nevertheless, the numerical model was used not only to derive the combination of actions on the members, but also to compute the proper restraint coefficient for the effective shear span used for the checks on the column elements including 2nd order effects.

A preliminary proportioning, carried out with the aim to identify the structural own dead weight to be inserted in the numerical model, gave as a result the cross-sections shown in the following:



To be noted that the above cross-sections were confirmed after detailed analysis, apart from inner lightening pipes that were discretely installed in the column elements to save concrete volume, and from the depth and width of the partially precast lattice slab element, which were unified with respect to the depth of the TT element to 330 mm and 2400 mm, respectively.

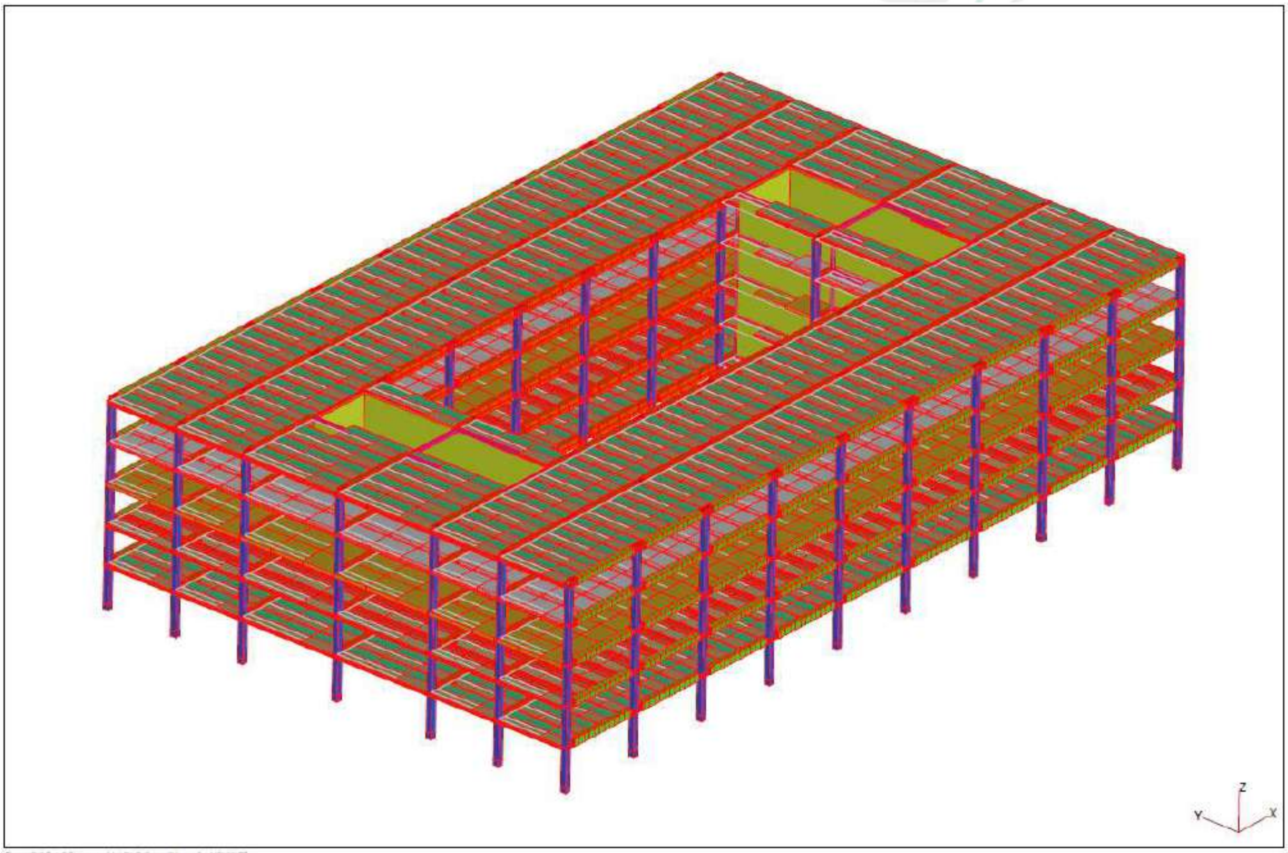
The structural materials considered are the following:

- Concrete C45/55 for all precast concrete cast (except columns)
- Concrete C80/95 for precast columns
- Concrete C25/30 for all cast-in-situ concrete*
- Steel B500C for reinforcing bars with diameter equal or larger than Φ16 mm
- Steel B400A for reinforcing bars with diameter smaller than Φ16 mm
- Steel Y1860 for prestressing tendons**

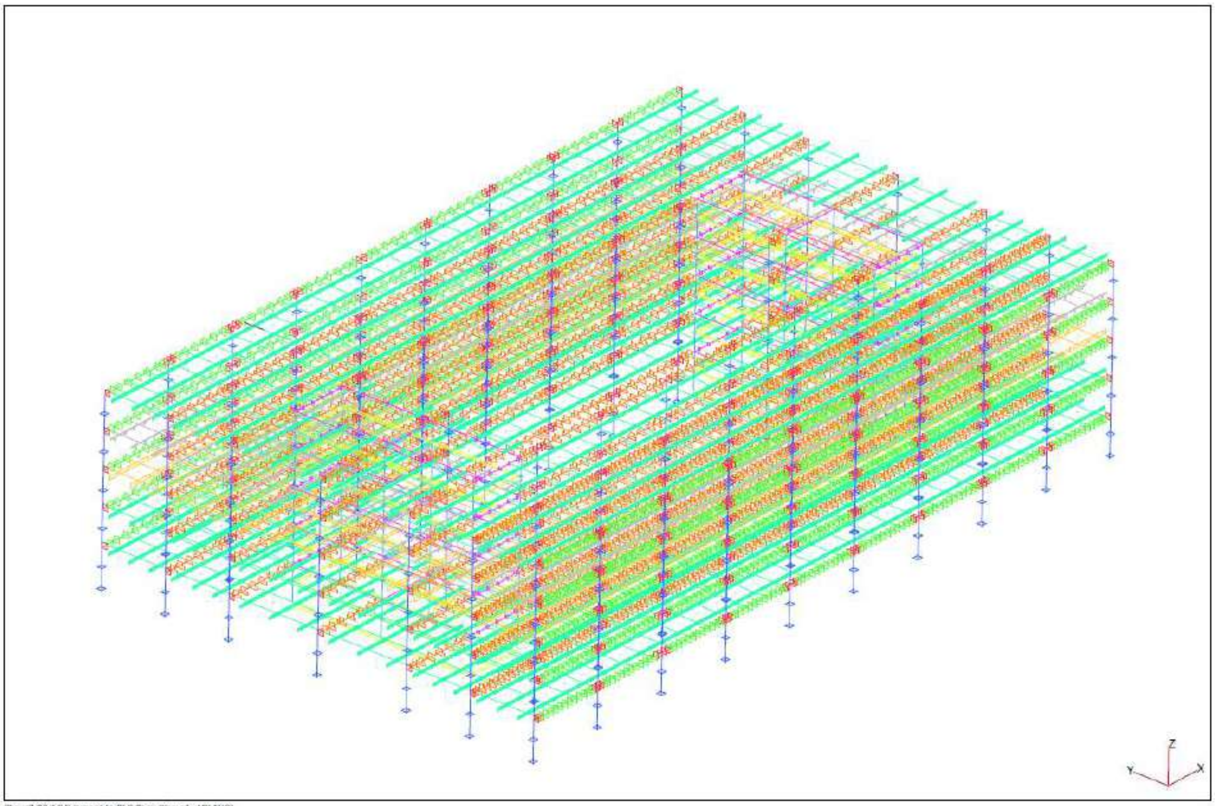
* completion cast-in-situ concrete is cast below the precast foundation footing, over the lattice girder plank, and inside selected core ends in the hollowcore elements.

** only 7w 0.5' tendons are employed, having a cross-sectional area of 93 mm^2.

Images of the numerical model are shown in the following. All structural members are modelled with elastic beam elements, including the cores, which are simplistically modelled as squat beam elements, also since they are not object of detailed calculation. All columns are assumed to be perfectly clamped to the ground. Stiff horizontal cantilever beam elements simulate the physical presence of the beam-supporting corbels. All horizontal members are perfectly hinged along the vertical displacement direction. In particular, a rotation restraint release was applied to all beams and floor elements. The presence of a well-distanced couple of support floor-to-beam connections is included in the model in terms of a saddle made at each end of the floor element and connected to the beam element nodes through rigid links. Zero-length elastic spring elements ("connection" elements in the software) were applied inside the saddle structure to model the presence of the connections, with stiffness values considered following experimental tests on the most typical connection typologies, including dowel and external angles.



Straus7 R2.4.6 [Licenced to:DLC Consulting srl - MILANO]
Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
10 agosto 2023 6:05 pm



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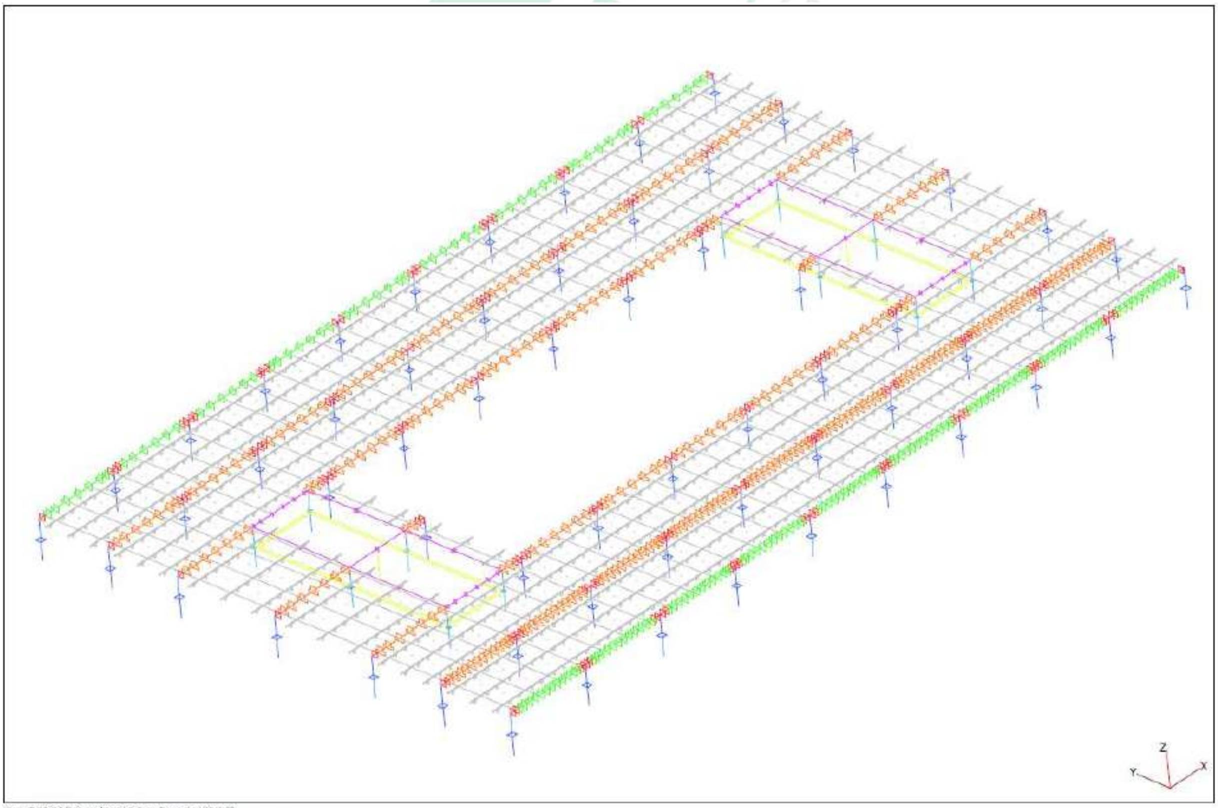

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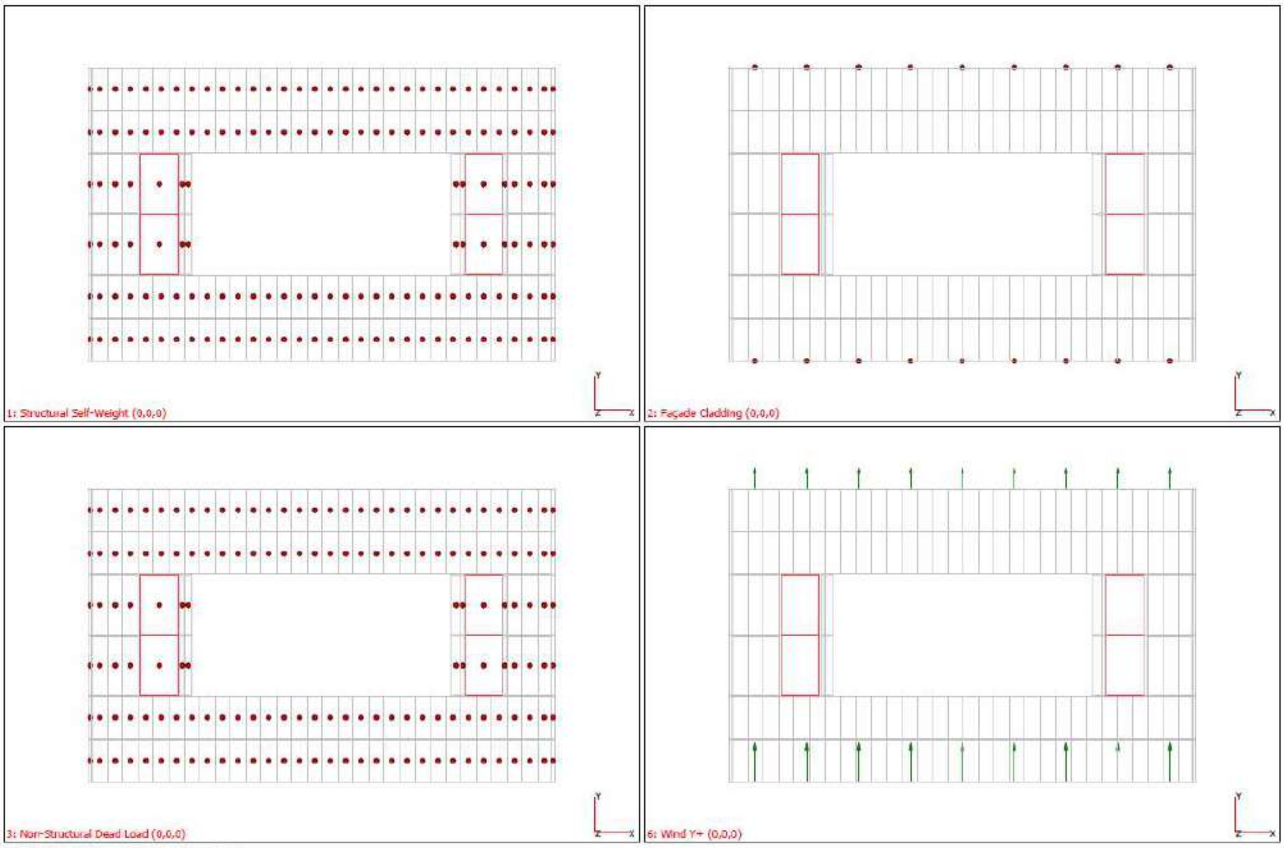
Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
10 agosto 2023 6:12 pm

The loads are introduced with different strategies:

- The structural dead selfweight of the modelled elements is introduced by explicitly attributing the density of reinforced concrete (2500 kg/m^3) to the beam elements;
- The non-structural dead loads are introduced with plate load patch elements to which a mass of 200 kg/m^2 is introduced to account for both technical layers and distributed lightweight partition walls;
- The mass of cladding walls is taken into account in the model in the form of distributed mass applied to the edge beam elements (350 kg/m);
- Live loads are assigned to the plate load patch elements (300 kg/m^2) in alternative positions, in order to model possible unbalanced live load distributions;
- Horizontal wind is applied as vertical plate load patch elements to which a distributed load of 50 kg/m^2 is applied;
- Fire load is taken into account considering an exposure of 60 minutes to nominal standard ISO 834 fire curve.

The considered environmental class is XC1, except for foundations, where it is XC2.



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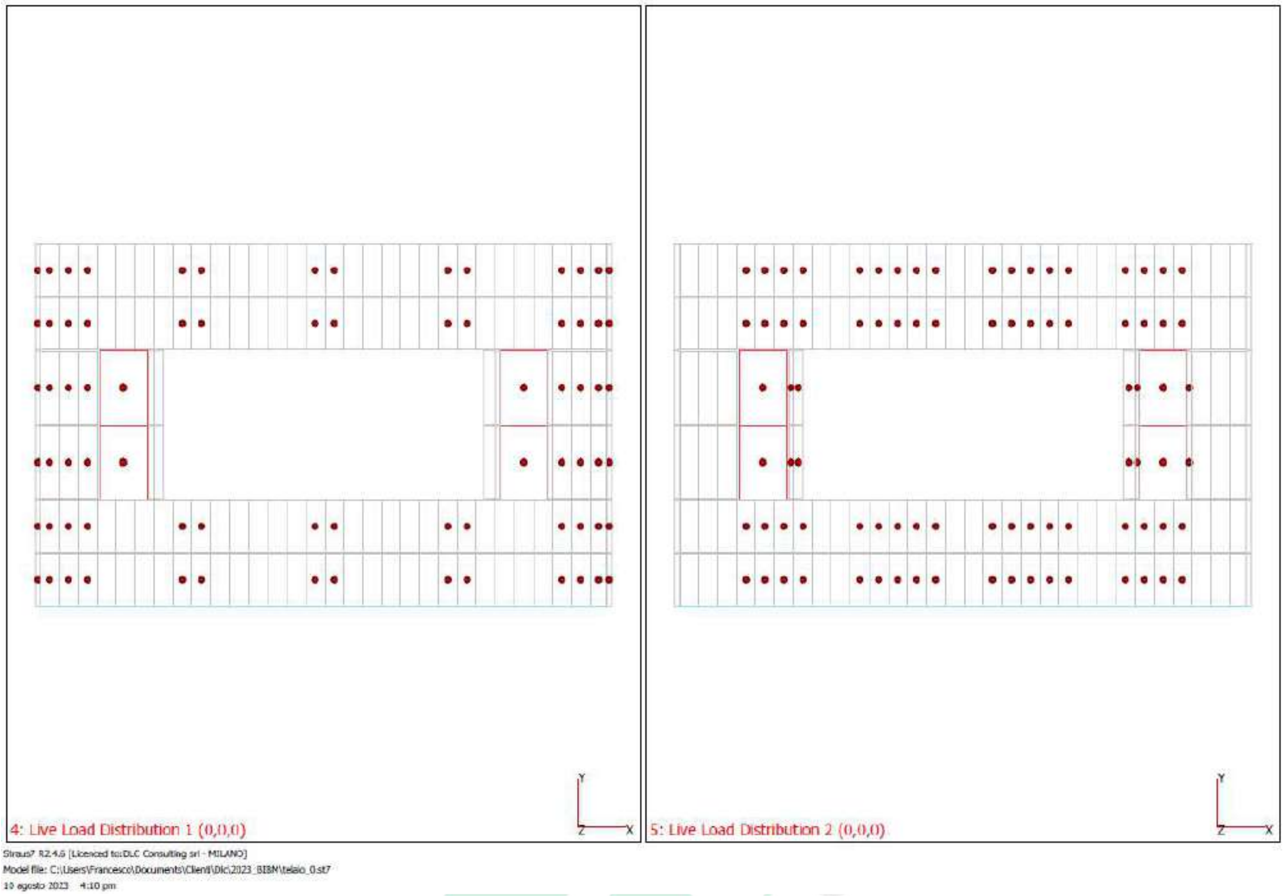
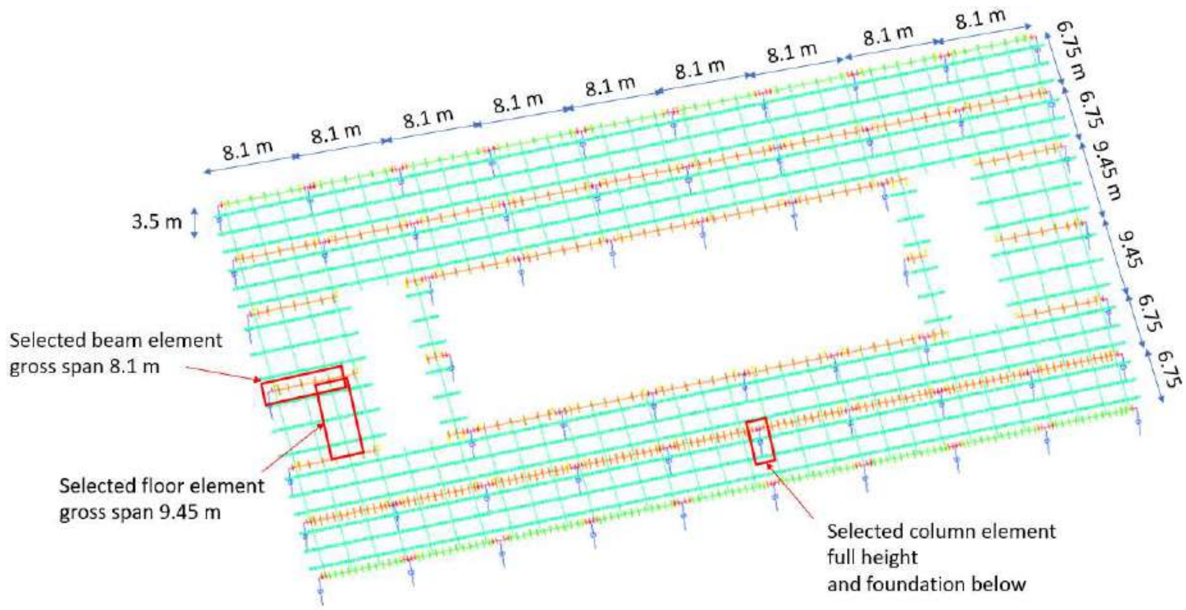


Among the modelled elements, according to the aim of the projects, only selected main elements were designed in detail. These elements were selected according to the most stressed (floor member with longer span, beam with larger influence area, central column and its foundation).

They are the following seven:

- Precast TT floor element
- Precast hollowcore floor element with end partial concrete filling
- Partially precast lattice girder floor element
- Precast prestressed central inverted-T beam
- Precast non-prestressed central inverted T-beam
- Precast central column
- Partially precast foundation footing

The selected elements are indicated in the following figure:



The following table contains the bill of materials modelled.

The total modelled mass is about 8500 tons.

Model: telaio_0

Bill of materials

Selected groups:
Model
Model\fondazione
Model\1 solaio
Model\1 vano scala
Model\2 solaio
Model\2 vano scala
Model\3 solaio
Model\3 vano scala
Model\4 solaio
Model\4 vano scala
Model\5 solaio
Model\5 vano scala

Included mass:
Structural Mass

	Mass kg	Volume m^3	Length m	Area m^2	Count	Material	Type	Section
Grand total:	8482556,521	3441,023	9459,500	15075,000				
Beam properties:								
1: Column	675000,000	270,000	1080,000		360		Beam	Solid Rectangle
2: Bracket	227500,000	91,000	364,000		845		Beam	Solid Rectangle
3: L-Shaped Beam	512550,000	205,020	603,000		835		Beam	Angle
4: Stair Core	1098000,000	439,200	36,000		12		Beam	Hollow Rectangle
5: Fictitious Column	0,000	13,125	210,000		60		Beam	Solid Rectangle
6: Fictitious Beam	0,000	34,875	558,000		160		Beam	Solid Rectangle
7: Inverted-T Beam	1171950,000	468,780	1202,000		1425		Beam	T-Section
8: TT Slab Element	720879,013	288,352	997,200		432		Beam	User Section
9: Slab-to-Beam Connection	0,000	0,000	288,500		2885		Connection	
10: Slab-to-Slab Connection	0,000	0,000	132,000		1320		Connection	
11: Hollow Core Slab Element	2281717,508	912,687	2991,600		1296		Beam	User Section
12: Solid Slab	1794960,000	717,984	997,200		432		Beam	Solid Rectangle
Total	8482556,521	3441,023	9459,500		10062			
Plate properties:								
1: solaio	0,000	0,000		11309,500	770		Load Patch	
2: vano scala	0,000	0,000		1134,000	20		Load Patch	

Straus7 R2.4.6 [Licenced to:DLC Consulting srl - MILANO]
Model file: C:\Users\Francesco\Documents\Clienti\DLC\2023_BIBM\telaio_0.st7
10 agosto 2023 6:01 pm

3: facciata	0,000	0,000	2551,500	90	Load Patch
Total	0,000	0,000	15075,000	880	

Centre of mass

	Mass kg	CM(X) m	CM(Y) m	CM(Z) m
fondazione	49250,000	36,450	22,950	-0,250
Model	0,000			
1 vano scala	213500,000	36,450	22,950	1,750
1 solaio	1274222,503	36,532	22,950	3,320
2 vano scala	213500,000	36,450	22,950	5,250
2 solaio	1274222,503	36,532	22,950	6,820
3 vano scala	213500,000	36,450	22,950	8,750
3 solaio	2308610,000	36,556	22,950	10,401
4 vano scala	213500,000	36,450	22,950	12,250
4 solaio	1234529,013	36,530	22,950	13,814
5 vano scala	213500,000	36,450	22,950	15,750
5 solaio	1274222,503	36,532	22,950	17,320
Total:	8482556,521	36,527	22,950	10,066

Local inertia

	Ixx $kg.m^2$	Iyy $kg.m^2$	Izz $kg.m^2$	Ixy $kg.m^2$	Iyz $kg.m^2$	Izx $kg.m^2$
fondazione	5379679,167	29832071,667	35209698,750	0,000	0,000	0,000
Model	0,000	0,000	0,000	0,000	0,000	0,000
1 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
1 solaio	321419141,907	663742114,212	984172186,669	0,000	0,000	18721,122
2 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
2 solaio	321419141,907	663742114,212	984172186,669	0,000	0,000	18721,122
3 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
3 solaio	560801460,266	1231153689,448	1790928978,707	0,000	0,000	24386,009
4 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
4 solaio	312232487,617	641967637,804	953213718,396	0,000	0,000	18314,607
5 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
5 solaio	321419141,907	663742114,212	984172186,669	0,000	0,000	18721,122
Total:	2033070242,017	4770587121,626	6417877146,685	0,000	0,000	265620,332

Straus7 R2.4.6 [Licenced to:DLC Consulting srl - MILANO]
Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
10 agosto 2023 6:01 pm

3: facciata	0,000	0,000	2551,500	90	Load Patch
Total	0,000	0,000	15075,000	880	

Centre of mass

	Mass kg	CM(X) m	CM(Y) m	CM(Z) m
fondazione	49250,000	36,450	22,950	-0,250
Model	0,000			
1 vano scala	213500,000	36,450	22,950	1,750
1 solaio	1274222,503	36,532	22,950	3,320
2 vano scala	213500,000	36,450	22,950	5,250
2 solaio	1274222,503	36,532	22,950	6,820
3 vano scala	213500,000	36,450	22,950	8,750
3 solaio	2308610,000	36,556	22,950	10,401
4 vano scala	213500,000	36,450	22,950	12,250
4 solaio	1234529,013	36,530	22,950	13,814
5 vano scala	213500,000	36,450	22,950	15,750
5 solaio	1274222,503	36,532	22,950	17,320
Total:	8482556,521	36,527	22,950	10,066

Local inertia

	Ixx $kg.m^2$	Iyy $kg.m^2$	Izz $kg.m^2$	Ixy $kg.m^2$	Iyz $kg.m^2$	Izx $kg.m^2$
fondazione	5379679,167	29832071,667	35209698,750	0,000	0,000	0,000
Model	0,000	0,000	0,000	0,000	0,000	0,000
1 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
1 solaio	321419141,907	663742114,212	984172186,669	0,000	0,000	18721,122
2 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
2 solaio	321419141,907	663742114,212	984172186,669	0,000	0,000	18721,122
3 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
3 solaio	560801460,266	1231153689,448	1790928978,707	0,000	0,000	24386,009
4 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
4 solaio	312232487,617	641967637,804	953213718,396	0,000	0,000	18314,607
5 vano scala	217947,917	137417851,667	137199903,750	0,000	0,000	0,000
5 solaio	321419141,907	663742114,212	984172186,669	0,000	0,000	18721,122
Total:	2033070242,017	4770587121,626	6417877146,685	0,000	0,000	265620,332

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Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
10 agosto 2023 6:01 pm

2.1 Linear elastic analysis

Linear elastic and buckling analyses are carried out.

The linear elastic analysis outcome form is shown in the following.

The load combinations employed are also shown in the following, together with the resulting diagrams of the main actions on the members:

- Bending moment and shear distribution on floor elements along the vertical direction;
- Bending moment and shear distribution on beam elements along the vertical direction;
- Axial forces in the columns.

```
*Solution commenced on 10/08/2023 at 18:12:52

Straus7 [2.4.6][Solver Build: 24141222] (32-Bit)

ANALYSIS TYPE               : LINEAR STATIC

COMPUTER NAME               : DESKTOP-ELEQNV4
USER LOGON NAME             : Francesco
CPU                         : Intel(R) Core(TM)2 Duo CPU E8400 @ 3.00GHz
USABLE PHYSICAL MEMORY      : 3.9 GB
USABLE VIRTUAL MEMORY       : 3.0 GB

MODEL FILE                  : "C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7"
RESULT FILE                 : "C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.LSA"
SCRATCH PATH                : "C:\Users\Francesco\Straus7\Tmp\"

TOTALS
Nodes                       : 11826
Beams                       : 10062
Plates                      :   880
Bricks                      :     0
Links                       :  5650

SOLVER UNITS
Length                      : m
Mass                        : kg
Force                       : N
Stress                      : Pa

FREEDOM CASE                : "Freedom Case 1"

LOAD CASES                  : "Structural Self-Weight"
                            : "Façade Cladding"
                            : "Non-Structural Dead Load"
                            : "Live Load Distribution 1"
                            : "Live Load Distribution 2"
                            : "Wind Y+"

STORAGE SCHEME              : Skyline
SORTING METHOD              : Tree [ 1]
SOLUTION TYPE               : Direct

NUMBER OF EQUATIONS         : 36957
MAXIMUM BANDWIDTH           : 2316
AVERAGE BANDWIDTH           : 530
[K] MATRIX SIZE             : 149.3 MB
MINIMUM RAM NEEDED          : 12.7 MB
FREE SCRATCH SPACE          : 51.7 GB

*NOTE[  4]:Link forces are added to node reaction calculations.
```

SUMMATION OF APPLIED LOADS

	Case	FX	FY	FZ	MX	MY	MZ	Name
Beams	1	7.10543E-10	1.70530E-11	-8.76606E+07	1.90084E-10	2.33589E+04	-1.19883E-10	"Structural Self-Weight"
	2	0.00000E+00	-4.38405E-11	-2.29635E+06	0.00000E+00	9.04237E-11	-5.03455E-11	"Façade Cladding"
	3	-2.84848E-25	-2.68589E-28	-2.50470E+07	1.78488E-10	7.90842E+02	-1.51461E-27	"Non-Structural Dead Load"
	4	2.98379E-25	-4.02883E-28	-1.81967E+07	1.87356E-10	7.90842E+02	-4.23082E-26	"Live Load Distribution 1"
	5	5.09516E-25	2.85845E-38	-2.05079E+07	1.72577E-10	7.90842E+02	-4.74579E-27	"Live Load Distribution 2"
	6	0.00000E+00	8.44481E+05	-1.13332E-11	0.00000E+00	-1.67213E-11	8.79652E-12	"Wind Y+"
Plates	1	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	"Structural Self-Weight"
	2	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	"Façade Cladding"
	3	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	"Non-Structural Dead Load"
	4	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	"Live Load Distribution 1"
	5	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	"Live Load Distribution 2"
	6	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	0.00000E+00	"Wind Y+"
Total	1	7.10543E-10	1.70530E-11	-8.76606E+07	1.90084E-10	2.33589E+04	-1.19883E-10	"Structural Self-Weight"
	2	0.00000E+00	-4.38405E-11	-2.29635E+06	0.00000E+00	9.04237E-11	-5.03455E-11	"Façade Cladding"
	3	-2.84848E-25	-2.68589E-28	-2.50470E+07	1.78488E-10	7.90842E+02	-1.51461E-27	"Non-Structural Dead Load"
	4	2.98379E-25	-4.02883E-28	-1.81967E+07	1.87356E-10	7.90842E+02	-4.23082E-26	"Live Load Distribution 1"
	5	5.09516E-25	2.85845E-38	-2.05079E+07	1.72577E-10	7.90842E+02	-4.74579E-27	"Live Load Distribution 2"
	6	0.00000E+00	8.44481E+05	-1.13332E-11	0.00000E+00	-1.67213E-11	8.79652E-12	"Wind Y+"
Vector	1	5.10703E-10	1.77351E-11	-8.74143E+07	-1.26947E+08	8.91601E+06	-7.82165E-11	"Structural Self-Weight"
	2	0.00000E+00	-4.40963E-11	-2.29635E+06	-1.77636E-15	9.10063E-11	-5.01217E-11	"Façade Cladding"
	3	-2.84848E-25	-2.68589E-28	-2.50470E+07	-1.80984E+07	-9.53227E+06	-1.27637E-27	"Non-Structural Dead Load"
	4	2.98379E-25	-4.02883E-28	-1.81966E+07	-1.81350E+07	-4.07409E+06	-4.19508E-26	"Live Load Distribution 1"
	5	5.09516E-25	2.85845E-38	-2.05078E+07	-1.80617E+07	-7.34865E+06	-4.74579E-27	"Live Load Distribution 2"
	6	0.00000E+00	8.44481E+05	-1.12905E-11	-2.22045E-15	-1.68008E-11	8.70415E-12	"Wind Y+"

SUMMATION OF MOMENTS OF APPLIED LOADS ABOUT THE ORIGIN [Load Vector]

Case	MXo	MYo	MZo	Name
1	-2.00616E+09	3.19280E+09	-1.37621E-08	"Structural Self-Weight"
2	-5.27012E+07	8.37020E+07	-1.92191E-09	"Façade Cladding"
3	-5.74829E+08	9.15934E+08	4.82346E-24	"Non-Structural Dead Load"
4	-4.17613E+08	6.58324E+08	-7.09252E-24	"Live Load Distribution 1"
5	-4.70655E+08	7.56912E+08	-1.34701E-23	"Live Load Distribution 2"
6	-8.21023E+06	1.78975E-10	3.07813E+07	"Wind Y+"

```
Reducing 36957 Equations (Using 149.4 MB RAM)...

MAXIMUM PIVOT               : 4.117582E+13 (Node 313 RX)
MINIMUM PIVOT               : 1.695052E+06 (Node 8546 RY)

Results for 6 Load Cases...
```

MAXIMUM DISPLACEMENT MAGNITUDES

Case	DX	DY	DZ	RX	RY	RZ	Name
1	1.76989E-04	2.80762E-04	2.08808E-02	3.41873E-03	1.08125E-02	3.23150E-04	"Structural Self-Weight"
2	8.77353E-06	2.56037E-05	4.28411E-04	6.40702E-05	1.41158E-04	3.65924E-05	"Façade Cladding"
3	1.10044E-04	1.94689E-04	1.41531E-02	2.15870E-03	7.38509E-03	1.12731E-04	"Non-Structural Dead Load"
4	1.72601E-04	1.56595E-04	1.15287E-02	3.09150E-03	5.63090E-03	1.06342E-04	"Live Load Distribution 1"
5	2.33561E-04	3.25956E-04	2.34744E-02	3.49501E-03	1.21738E-02	1.87157E-04	"Live Load Distribution 2"
6	4.29286E-05	5.77322E-04	3.41040E-05	8.86413E-05	1.14259E-05	6.37154E-05	"Wind Y+"

DIRECT SUMMATION OF NODE REACTION FORCES

Case	FX	FY	FZ	MX	MY	MZ	Name
1	-7.47605E-10	-4.75211E-11	8.76606E+07	1.28541E+03	7.94127E+05	-1.44469E+03	"Structural Self-Weight"
2	1.72804E-11	2.56080E-11	2.29635E+06	1.02535E+02	7.79160E+02	-2.11896E+02	"Façade Cladding"
3	-2.12367E-10	-5.45697E-12	2.50470E+07	4.10216E+02	1.00199E+05	-1.16444E+02	"Non-Structural Dead Load"
4	-4.36557E-11	-7.33280E-12	1.81966E+07	6.84232E+04	3.63422E+05	-8.21542E+04	"Live Load Distribution 1"
5	-4.71161E-11	1.54898E-12	2.05078E+07	-6.77966E+04	-2.25984E+05	8.18751E+04	"Live Load Distribution 2"
6	-1.78773E-11	-8.44481E+05	1.40403E-11	8.61686E+06	2.77706E+03	-8.28805E+03	"Wind Y+"

TOTAL CPU TIME : 20.859 Seconds (0:00:21)

*Solution completed on 10/08/2023 at 18:13:13
*Solution time: 22 Seconds

*SUMMARY OF MESSAGES
*Number of Notes : 1
*Number of Warnings : 0
*Number of Errors : 0

Linear Static Load Case Combinations

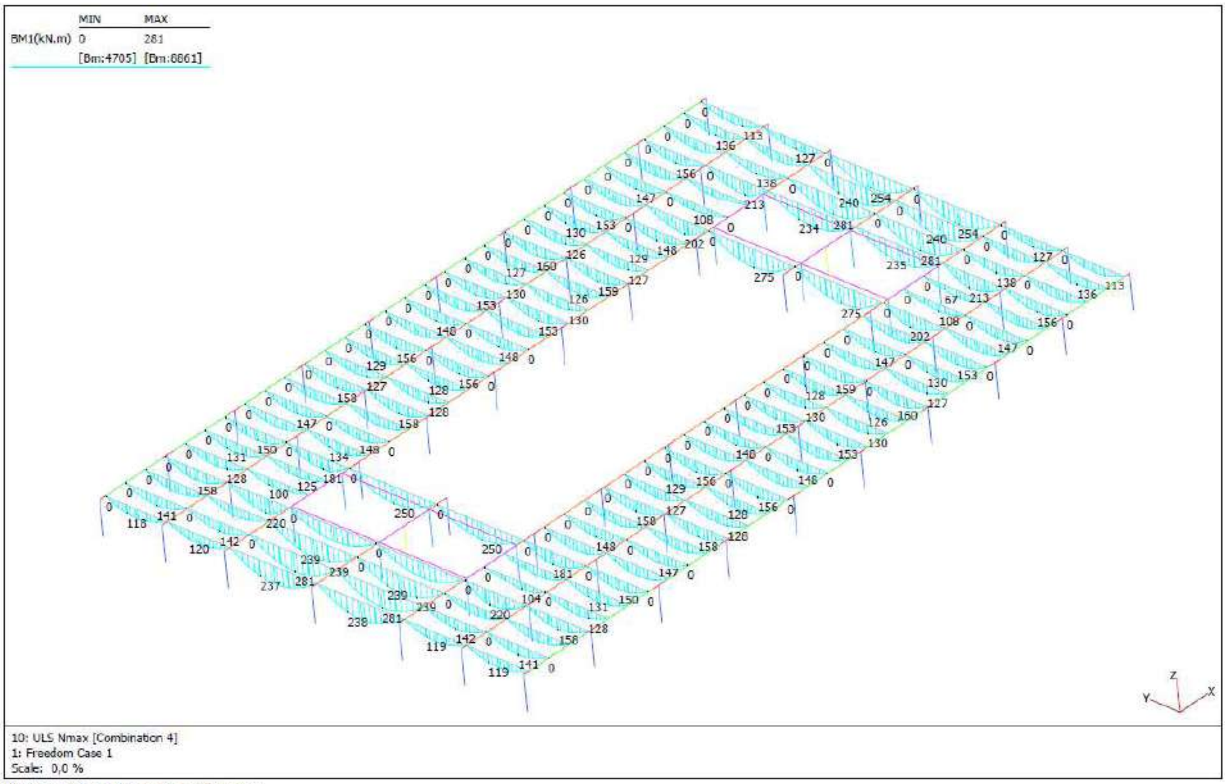
CASES	1 SLS QP
1: Structural Self-Weight [Freedom Case 1]	1,0
2: Façade Cladding [Freedom Case 1]	1,0
3: Non-Structural Dead Load [Freedom Case ...	1,0
4: Live Load Distribution 1 [Freedom Case 1]	0,3
5: Live Load Distribution 2 [Freedom Case 1]	0,3
6: Wind Y+ [Freedom Case 1]	0,0

CASES	2 SLS Rare Nmax
1: Structural Self-Weight [Freedom Case 1]	1,0
2: Façade Cladding [Freedom Case 1]	1,0
3: Non-Structural Dead Load [Freedom Case ...	1,0
4: Live Load Distribution 1 [Freedom Case 1]	1,0
5: Live Load Distribution 2 [Freedom Case 1]	1,0
6: Wind Y+ [Freedom Case 1]	0,6

CASES	3 SLS Rare Mmax
1: Structural Self-Weight [Freedom Case 1]	1,0
2: Façade Cladding [Freedom Case 1]	1,0
3: Non-Structural Dead Load [Freedom Case ...	1,0
4: Live Load Distribution 1 [Freedom Case 1]	1,0
5: Live Load Distribution 2 [Freedom Case 1]	0,0
6: Wind Y+ [Freedom Case 1]	0,6

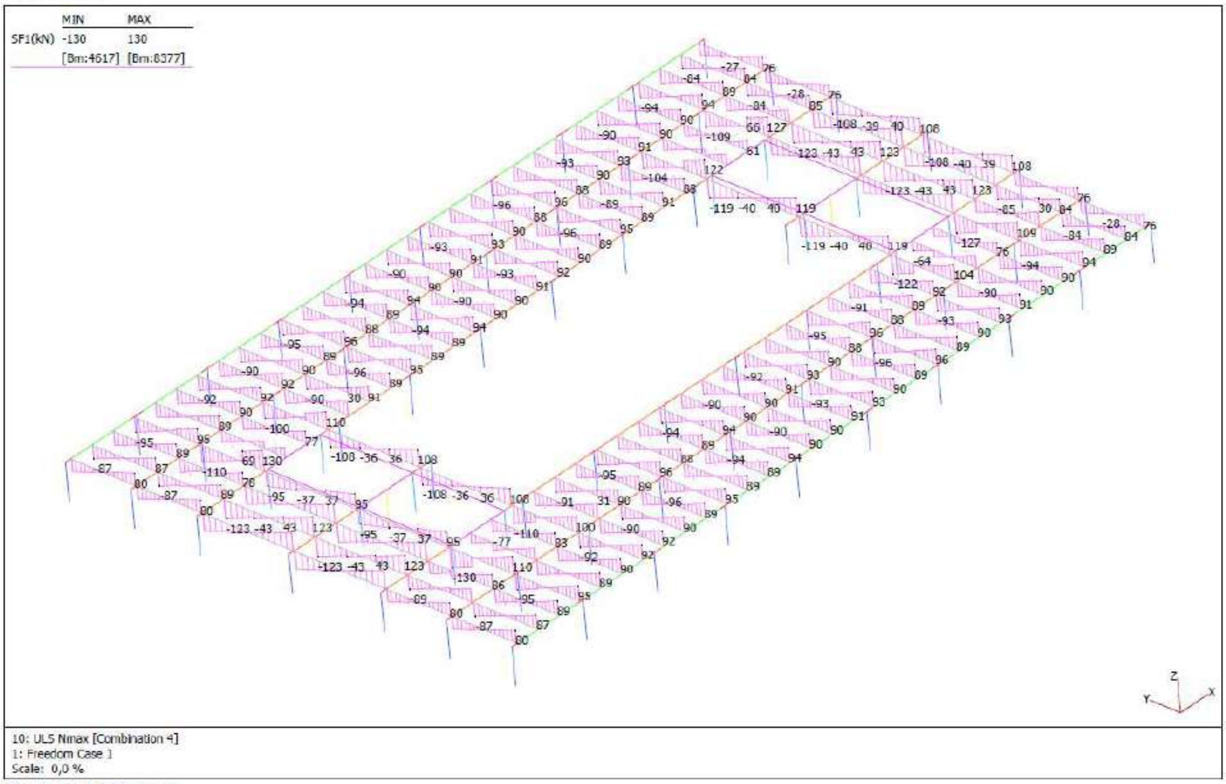
CASES	4 ULS Nmax
1: Structural Self-Weight [Freedom Case 1]	1,35
2: Façade Cladding [Freedom Case 1]	1,35
3: Non-Structural Dead Load [Freedom Case ...	1,5
4: Live Load Distribution 1 [Freedom Case 1]	1,5
5: Live Load Distribution 2 [Freedom Case 1]	1,5
6: Wind Y+ [Freedom Case 1]	0,9

CASES	5 ULS Mmax
1: Structural Self-Weight [Freedom Case 1]	1,35
2: Façade Cladding [Freedom Case 1]	1,35
3: Non-Structural Dead Load [Freedom Case ...	1,5
4: Live Load Distribution 1 [Freedom Case 1]	0,0
5: Live Load Distribution 2 [Freedom Case 1]	1,5
6: Wind Y+ [Freedom Case 1]	0,9



Bending Moment on Slab Elements

Straus7 R2.4.6 [Licenced to:DLC Consulting srl - MILANO]
Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
10 agosto 2023 6:17 pm



Shear on Slab Elements

Straus7 R2.4.6 [Licenced to:DLC Consulting srl - MILANO]
Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
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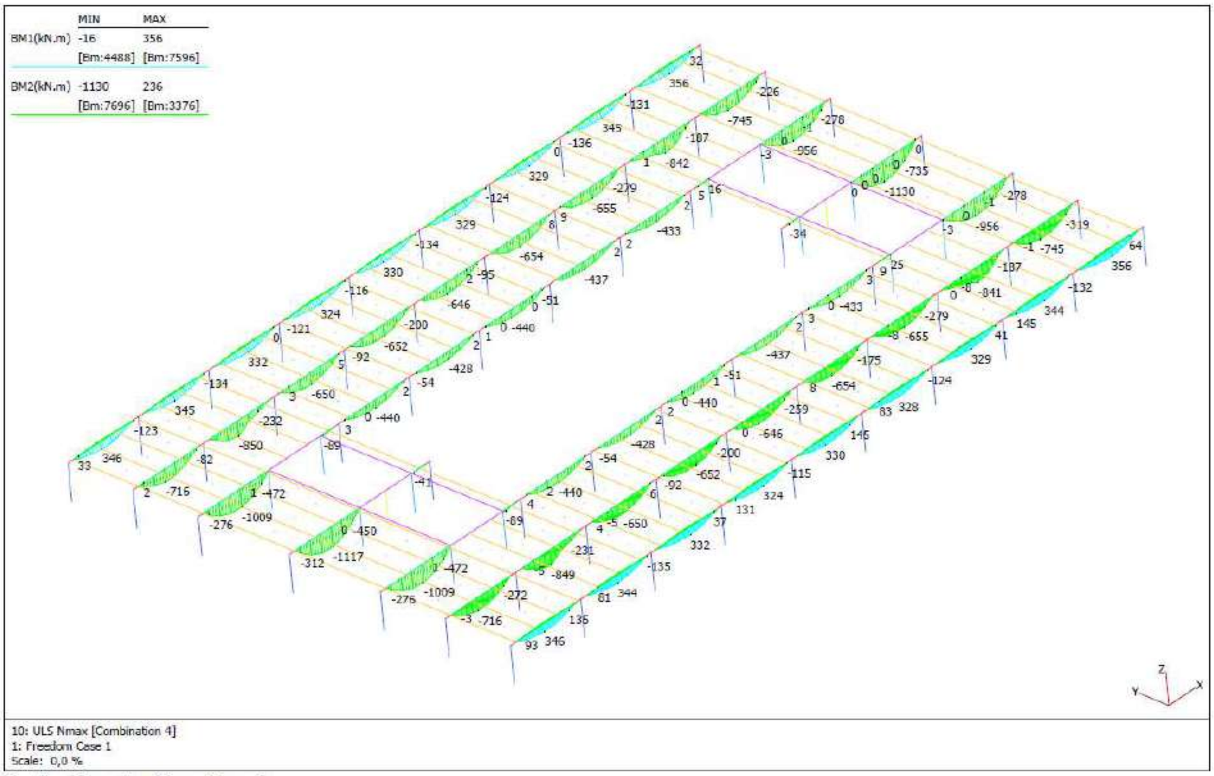
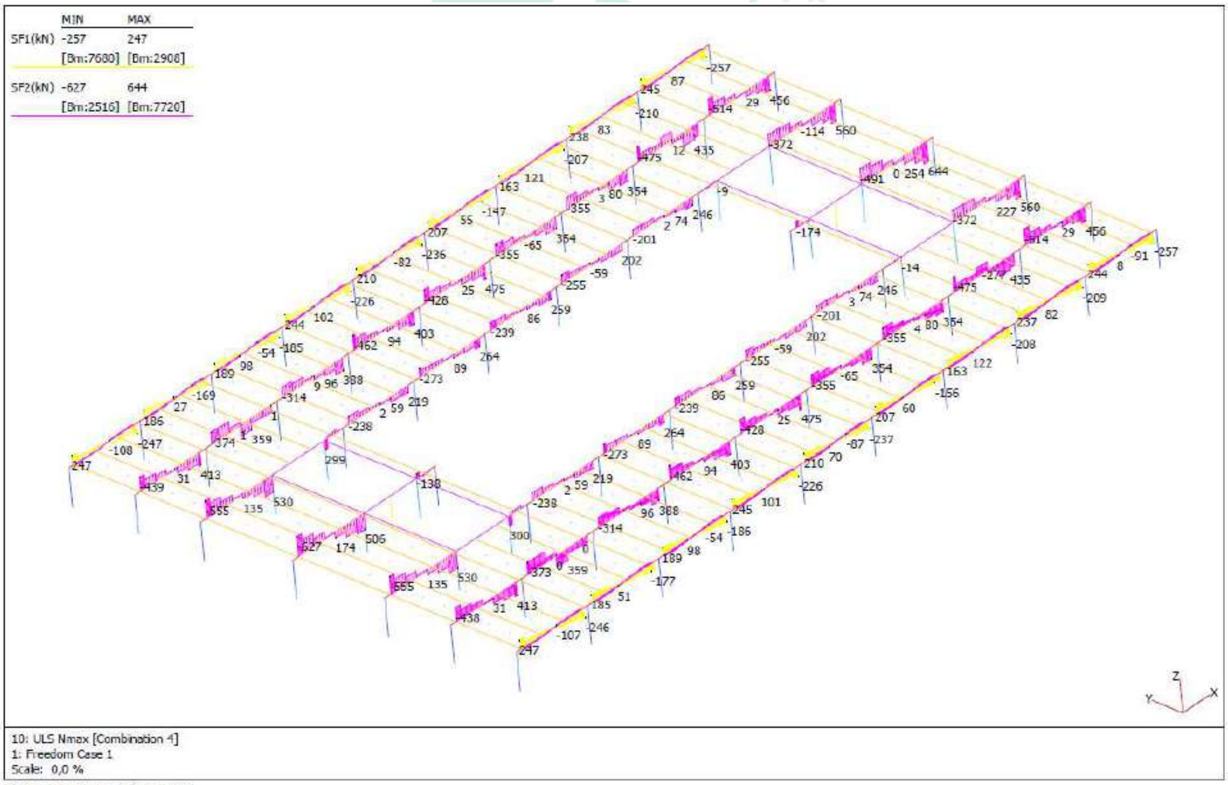


Bending Moment on Beam Elements

Straus7 R2.4.6 [Licenced to:DLC Consulting srl - MILANO]
Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
10 agosto 2023 6:20 pm



Shear on Beam Elements

Straus7 R2.4.6 [Licenced to:DLC Consulting srl - MILANO]
Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
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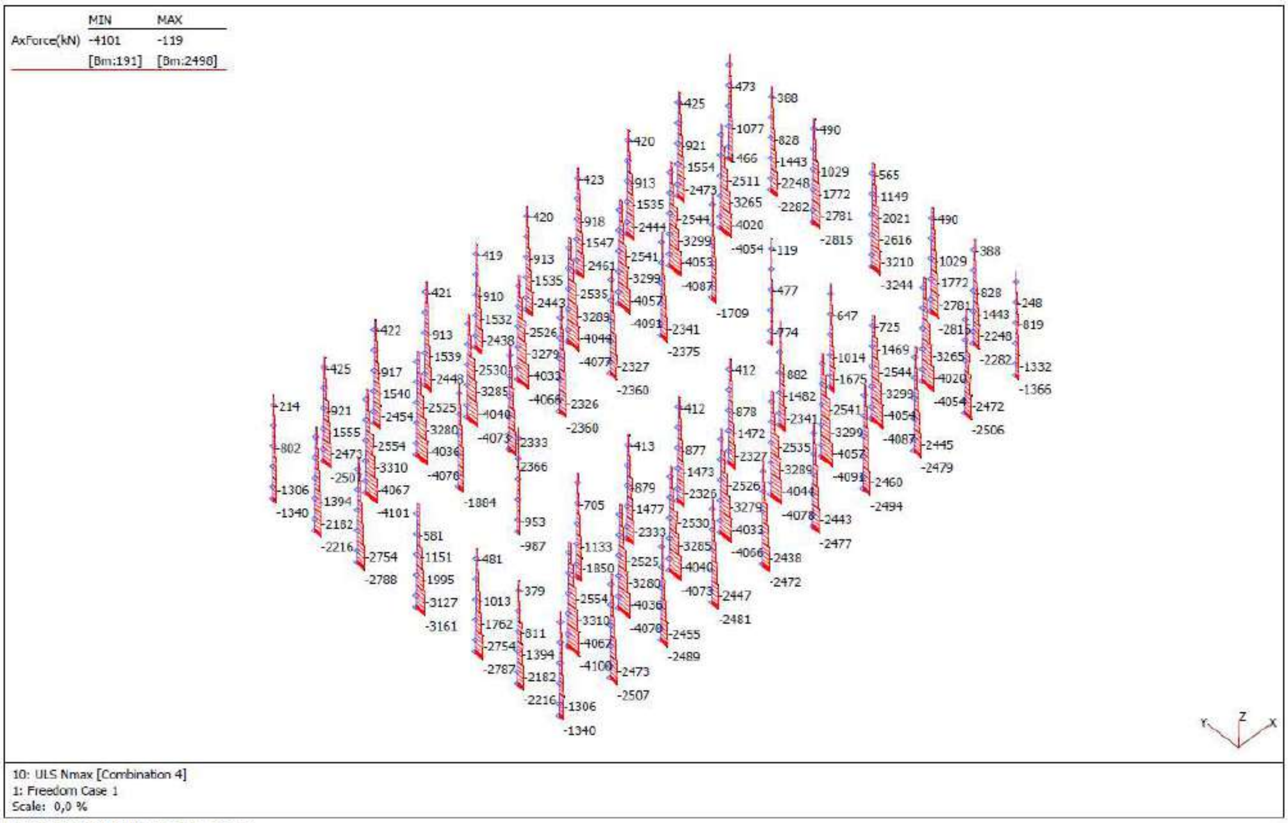


Axial Load on Column Elements

Straus7 R2.4.6 [Licensed to:DLC Consulting srl - MILANO]
Model file: C:\Users\Francesco\Documents\Clienti\Dlc\2023_BIBM\telaio_0.st7
10 agosto 2023 6:25 pm

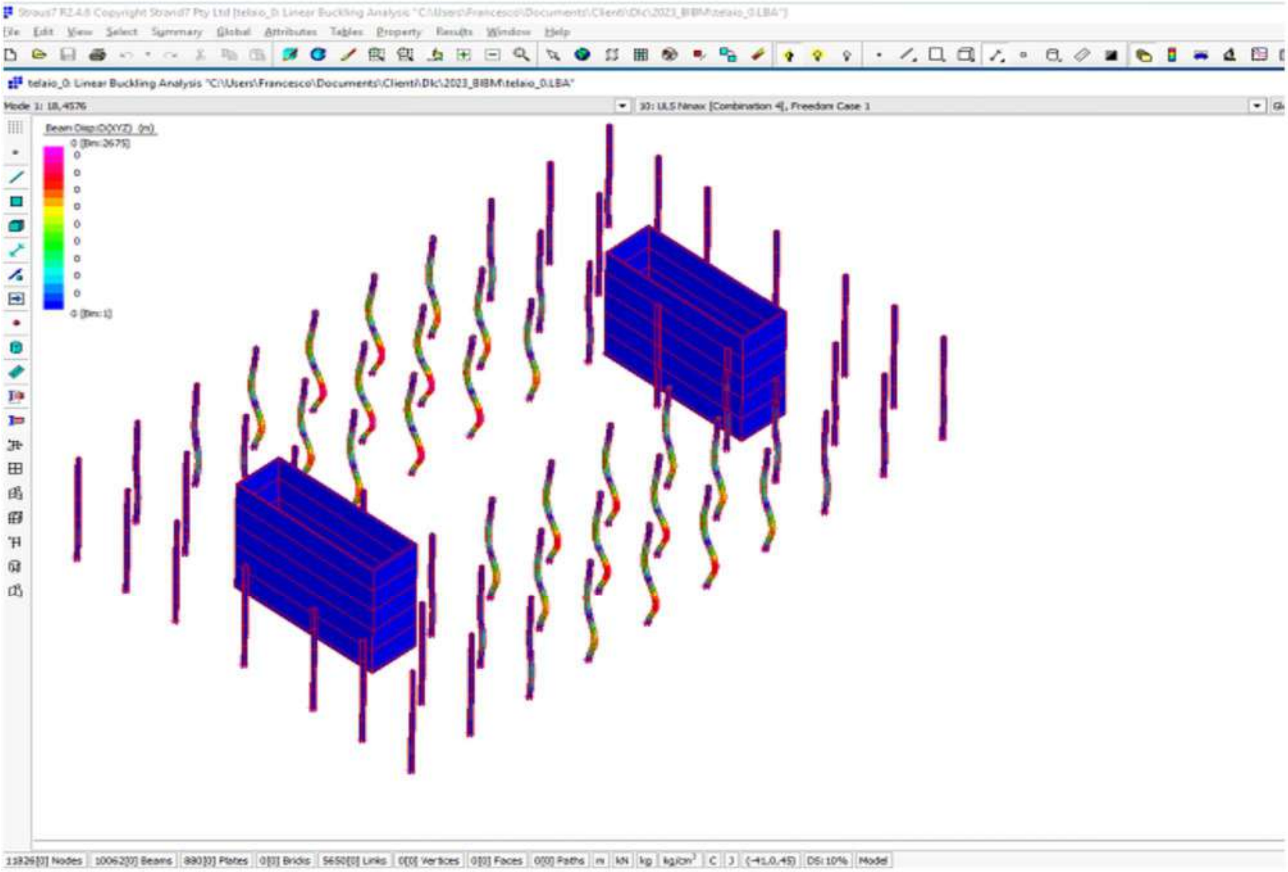
2.2 Buckling analysis

The results of the linear buckling analysis are instrumental to describe a proper shear span length of the column element considered as a whole. This information is crucial for the application of the model column (curvature-based) calculation method as included in both EN1992-1-1:2004 and FprEN1992-1-1:2022.

The first buckling mode is associated – as expected – to the buckling of the central part of the long sides of the building, which is the most distanced from the bracing cores.

The first buckling shape, shown in the following image, is associated with the column deformation, accompanied by the deformation of the diaphragm.

The load multiplier associated with the elastic buckling main mode is equal to 18.5. This high value, although referred to an unproper elastic buckling of reinforced concrete members, suggests that the influence of 2nd order effects in the considered structural arrangement is limited.



3 Material constitutive laws

This chapter describes the material constitutive laws of different classes and grades of concrete and steel materials employed in the project.

In particular, three types of concrete were used: C25/30 (cast-in-situ); C45/55 (precast); C80/95 (column HPC).

Moreover, three grades of steel were employed: prestressing steel (Y1860), mild reinforcing steel B500C for bar diameter equal or larger than Φ16, and mild reinforcement steel B500A for bar diameter lower than Φ16.

Concerning the material partial safety coefficients, for both standards, reduced coefficients ($\gamma_{c,red}$ = 1.40 and $\gamma_{s,red}$ = $\gamma_{p,red}$ = 1.10) were adopted in the analysis when precast concrete production is envisaged. Materials involving cast-in-situ concrete production and in-situ caging were dealt with standard partial safety coefficients (γ_c = 1.50 and γ_s = γ_p = 1.15).

Specific comments about deviations from EN1992-1-1:2004 to FprEN1992-1-1:2022 are collected in the dedicated chapter at the end of the document.

3.1 Concrete constitutive law following EN1992-1-1:2004

The general calculation procedure is shown for concrete class C80/95, but after it the application to C45/55 and C25/30 is also shown.

Concrete (§3.1)

$Rck := -95$ MPa cubic characteristic compressive resistance

$fck := -80$ MPa cylindric characteristic compressive resistance

$fcm := fck - 8 = -88$ MPa mean cylindric compressive resistance

$\gamma c := 1.5$ material safety coefficient for concrete

$\gamma cpcred := 1.4$ due to fulfillment of the conditions in A.2.1

$\alpha cc := 1$ $\gamma c := \gamma cpcred = 1.4$

Cylindrical design strength:

$fcd := fck \cdot \frac{\alpha cc}{\gamma cpcred}$ $fcd = -57.143$ MPa design cylindric compressive resistance

$$\varepsilon c1 := \max\left[-0.0028, -0.001 \cdot 0.7 (-fcm)^{\frac{1}{3}}\right] = -2.8 \times 10^{-3}$$

$$\varepsilon cu1 := \mathrm{if}\left[fck < -50, -0.001 \cdot \left[2.8 + 27 \cdot \left(\frac{98 + fcm}{100}\right)^4\right], -0.0035\right] = -2.803 \times 10^{-3}$$

$$\varepsilon c2 := \mathrm{if}\left[fck < -50, -0.001 \cdot \left[2.0 + 0.085 \cdot (-fck - 50)^{0.53}\right], -0.002\right] = -2.516 \times 10^{-3}$$ strain defining constitutive law

$$\varepsilon cu2 := \mathrm{if}\left[fck < -50, -0.001 \cdot \left[2.6 + 35 \cdot \left[\frac{(fck + 90)}{100}\right]^4\right], -0.0035\right] = -2.603 \times 10^{-3}$$

$$\varepsilon c3 := \mathrm{if}\left[fck < -50, -0.001 \cdot \left[1.75 + 0.55 \cdot \frac{(-fck - 50)}{40}\right], -0.00175\right] = -2.163 \times 10^{-3}$$

$$\varepsilon cu3 := \varepsilon cu2 = -2.603 \times 10^{-3}$$

$Ecm := 22000 \cdot \left(\frac{-fcm}{10}\right)^{0.3} = 4.224 \times 10^4$ MPa $Ec := Ecm$ Young (elastic) modulus

$$n := \mathrm{if}\left[fck < -50, 1.4 + 23.4 \cdot \left[\frac{(-fck - 90)}{100}\right]^4, 1.4\right] = 1.402$$

CONSTITUTIVE LAW OF CONCRETE IN COMPRESSION

Non linear EN1992-1-1:2004

$$k := 1.05 \cdot Ecm \cdot \frac{\varepsilon c1}{fcm} = 1.411$$

$$\sigma c\_nlsa(\varepsilon) := if\left[\varepsilon > \varepsilon cu1, fcm \cdot \left[\frac{k \cdot \left(\frac{\varepsilon}{\varepsilon c1}\right) - \left(\frac{\varepsilon}{\varepsilon c1}\right)^2}{1 + (k-2) \cdot \left(\frac{\varepsilon}{\varepsilon c1}\right)}\right], if[(\varepsilon > 0), 0, 0]\right]$$

Parabola-rectangle

$$\sigma c\_pr(\varepsilon) := if\left[\varepsilon > \varepsilon c2, fcd \cdot \left[1 - \left(1 - \frac{\varepsilon}{\varepsilon c2}\right)^2\right], if[(\varepsilon > \varepsilon cu2), fcd, 0]\right]$$

Triangle-rectangle

$$\sigma c\_tr(\varepsilon) := if\left[\varepsilon > \varepsilon c3, \frac{fcd}{\varepsilon c3} \cdot \varepsilon, if[(\varepsilon > \varepsilon cu3), fcd, 0]\right]$$

Stress block

$$\lambda := if\left[fck < -45, 0.8 - \frac{(-fck - 50)}{400}, 0.8\right] = 0.725$$

$$\eta := if\left[fck < -45, 1 - \frac{(-fck - 50)}{200}, 1\right] = 0.85$$

$$\sigma c\_sb(\varepsilon) := if[\varepsilon > (1 - \lambda) \cdot \varepsilon c3, 0, if[(\varepsilon > \varepsilon cu3), \eta \cdot fcd, 0]]$$

CONSTITUTIVE LAW OF CONCRETE IN TENSION

Linear elastic

$$fctm := \text{if}\left[fck < -50, 2.12\cdot\ln\left(1-\frac{fcm}{10}\right), 0.3\cdot(-fck)^{\frac{2}{3}}\right] = 4.839$$ MPa mean tension strength

$$\varepsilon ct := \frac{fctm}{Ecm} \qquad \varepsilon ct = 1.145\times 10^{-4}$$

$$\sigma ct(\varepsilon) := \text{if}(0 \le \varepsilon < \varepsilon ct, Ecm\cdot\varepsilon, 0)$$

$$fctk := 0.7\cdot fctm = 3.387$$ MPa characteristic tension strength

$$\alpha ct := 1$$

$$fctd := \alpha ct\cdot\frac{fctk}{\gamma c} = 2.419$$ MPa design tension strength

$$\varepsilon ct\_pr := \frac{fctd}{Ecm}$$

$$\sigma ct\_pr(\varepsilon) := \text{if}(0 \le \varepsilon < \varepsilon ct\_pr, Ecm\cdot\varepsilon, 0)$$

Linear elastic with softening

$$\sigma c\_traz\_soft(\varepsilon) := fctm\cdot\frac{1}{1+\left(\dfrac{\dfrac{\varepsilon}{\varepsilon ct}-1}{\dfrac{\varepsilon ct + 0.00005}{\varepsilon ct}-1}\right)^2}$$

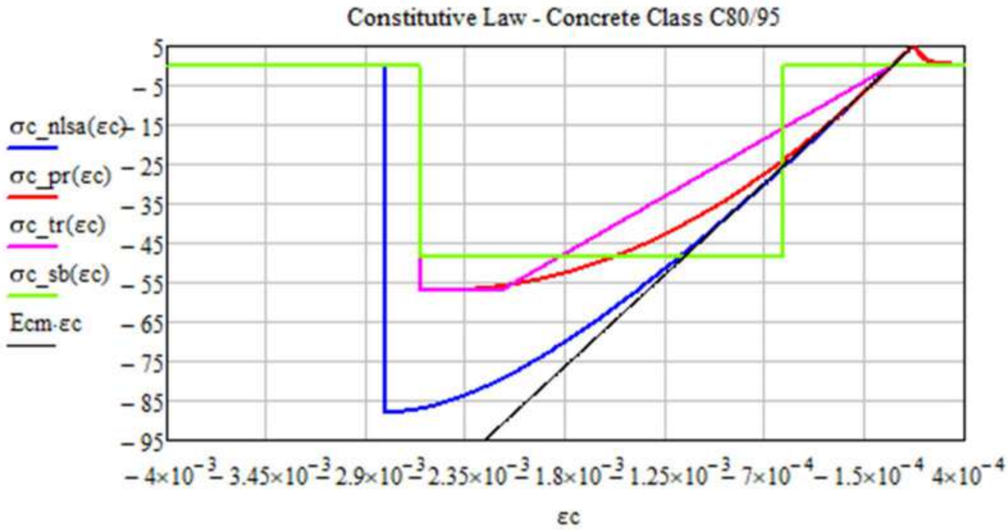
NOTE: post-peak formulation taken from Model Code needed for convergence easiness

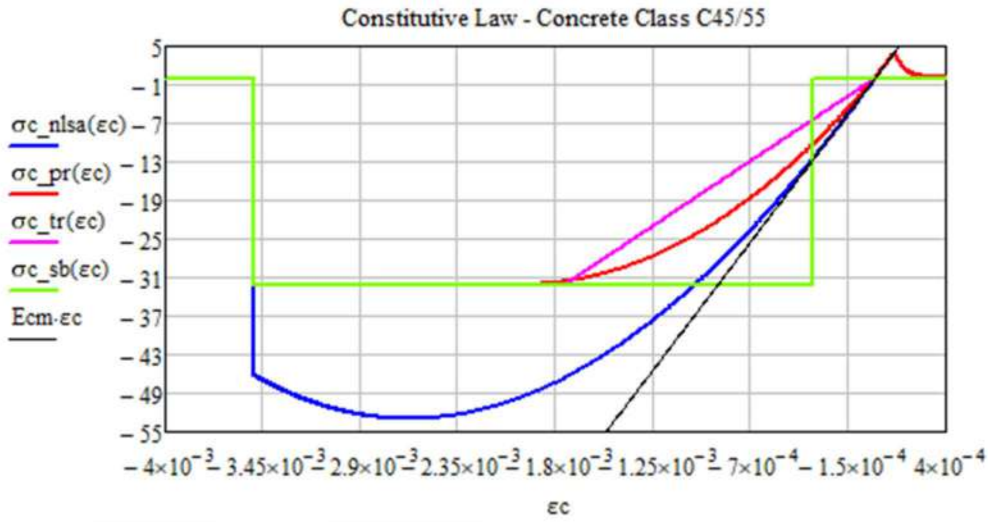
$$\sigma cts(\varepsilon) := \text{if}(0 \le \varepsilon < \varepsilon ct, Ecm\cdot\varepsilon, \text{if}(\varepsilon > \varepsilon ct, \sigma c\_traz\_soft(\varepsilon), 0))$$

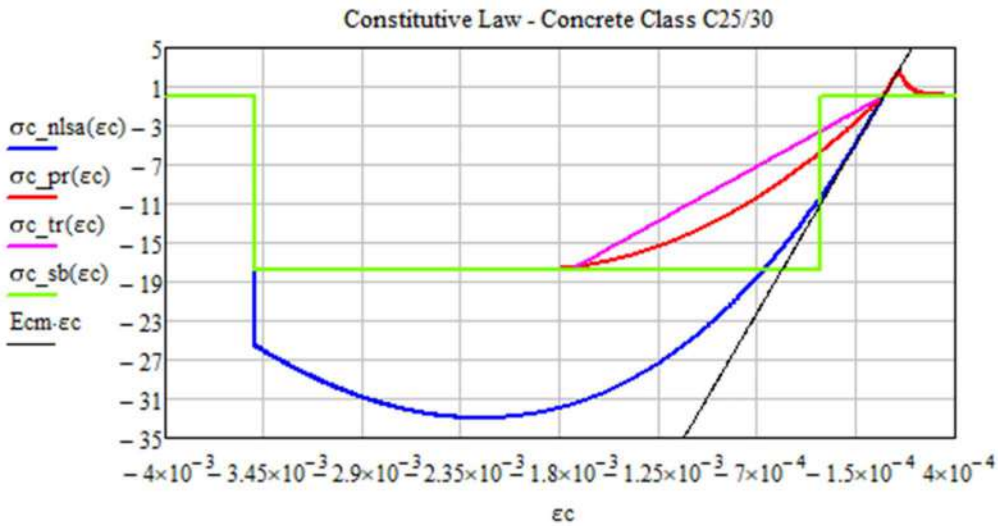
$$\sigma c\_nlsa(\varepsilon) := \text{if}(\varepsilon < 0, \sigma c\_nlsa(\varepsilon), \sigma cts(\varepsilon))$$

$$\sigma c\_pr(\varepsilon) := \text{if}(\varepsilon < 0, \sigma c\_pr(\varepsilon), \sigma cts(\varepsilon))$$

$$\sigma c\_tr(\varepsilon) := \text{if}(\varepsilon < 0, \sigma c\_tr(\varepsilon), 0)$$


Constitutive Law - Concrete Class C80/95
σc_nlsa(εc)
σc_pr(εc)
σc_tr(εc)
σc_sb(εc)
Ecm·εc
εc


Constitutive Law - Concrete Class C45/55
σc_nlsa(εc)
σc_pr(εc)
σc_tr(εc)
σc_sb(εc)
Ecm·εc
εc


Constitutive Law - Concrete Class C25/30
σc_nlsa(εc)
σc_pr(εc)
σc_tr(εc)
σc_sb(εc)
Ecm·εc
εc

3.2 Concrete constitutive law following FprEN1992-1-1:2022

The general calculation procedure is shown for concrete class C80/95, but at the end also the application to C45/55 and C25/30 is given.

Concrete (§5.1.6 + §8.1.2)

$Rck := -95$ MPa cubic characteristic compressive resistance

$fck := -80$ MPa cylindric characteristic compressive resistance

$fcm := fck - 8 = -88$ MPa mean cylindric compressive resistance

$\gamma c := 1.5$ material safety coefficient for concrete

$\gamma c := 1.4$ due to fulfillment of case (a) in table A.1 (NDP) and AVCP 2+

$ktc := 1$

$$\eta cc := \left(\frac{40}{-fck}\right)^{\frac{1}{3}} = 0.794$$

Cylindrical design strength:

$$fcd := \eta cc \cdot fck \cdot \frac{ktc}{\gamma c} \qquad fcd = -45.354$$ MPa design cylindric compressive resistance

$$\varepsilon c1 := \max\left[-0.0028, -0.001 \cdot 0.7 (-fcm)^{\frac{1}{3}}\right] = -2.8 \times 10^{-3}$$

$$\varepsilon cu1 := \max\left[-0.001 \cdot \left[2.8 + 14 \cdot \left(1 + \frac{fcm}{108}\right)^{4}\right], -0.0035\right] = -2.816 \times 10^{-3}$$

strain defining constitutive law

$\varepsilon c2 := -0.002$

$\varepsilon cu := -0.0035$

$$Ecm := 9500 \cdot (-fcm)^{\frac{1}{3}} = 4.226 \times 10^{4}$$ MPa Young (elastic) modulus

CONSTITUTIVE LAW OF CONCRETE IN COMPRESSION

Non linear EN1992-1-1:2004

$$k := 1.05 \cdot Ecm \cdot \frac{\varepsilon c1}{fcm} = 1.412$$

$$\sigma c\_nlsa(\varepsilon) := \mathrm{if}\left[\varepsilon > \varepsilon cu1, fcm \cdot \left[\frac{k \cdot \left(\frac{\varepsilon}{\varepsilon c1}\right) - \left(\frac{\varepsilon}{\varepsilon c1}\right)^{2}}{1 + (k-2) \cdot \left(\frac{\varepsilon}{\varepsilon c1}\right)}\right], \mathrm{if}[(\varepsilon > 0), 0, 0]\right]$$

Parabola-rectangle

$$\sigma c\_pr(\varepsilon) := \text{if}\left[\varepsilon > \varepsilon c2, fcd\cdot\left[1-\left(1-\frac{\varepsilon}{\varepsilon c2}\right)^2\right], \text{if}[(\varepsilon > \varepsilon cu), fcd, 0]\right]$$

Stress block

$$\sigma c\_sb(\varepsilon) := \text{if}[\varepsilon > (1-0.8)\cdot\varepsilon c2, 0, \text{if}[(\varepsilon > \varepsilon cu), fcd, 0]]$$

CONSTITUTIVE LAW OF CONCRETE IN TENSION

Linear elastic

$$fctm := \text{if}\left[fck < -50, 1.1\cdot(-fck)^{\frac{1}{3}}, 0.3\cdot(-fck)^{\frac{2}{3}}\right] = 4.74$$ MPa mean tension strength

$$\varepsilon ct := \frac{fctm}{Ecm} \qquad \varepsilon ct = 1.122\times10^{-4}$$

$$\sigma ct(\varepsilon) := \text{if}(0 \le \varepsilon < \varepsilon ct, Ecm\cdot\varepsilon, 0)$$

$$fctk := 0.7\cdot fctm$$

characteristic tension strength

$$ktt := 0.8$$

$$fctd := ktt\cdot\frac{fctk}{\gamma c} = 1.896$$ MPa design tension strength

$$\varepsilon ct\_pr := \frac{fctd}{Ecm} = 4.487\times10^{-5}$$

$$\sigma ct\_pr(\varepsilon) := \text{if}(0 \le \varepsilon < \varepsilon ct\_pr, Ecm\cdot\varepsilon, 0)$$

Linear elastic with softening

$$\sigma c\_traz\_soft(\varepsilon) := fctm\cdot\frac{1}{1+\left(\dfrac{\dfrac{\varepsilon}{\varepsilon ct}-1}{\dfrac{\varepsilon ct+0.00005}{\varepsilon ct}-1}\right)^2}$$

NOTE: post-peak formulation taken from Model Code needed for convergence easiness

$$\sigma cts(\varepsilon) := \text{if}(0 \le \varepsilon < \varepsilon ct, Ecm\cdot\varepsilon, \text{if}(\varepsilon > \varepsilon ct, \sigma c\_traz\_soft(\varepsilon), 0))$$

$$\sigma c\_nlsa(\varepsilon) := \text{if}(\varepsilon < 0, \sigma c\_nlsa(\varepsilon), \sigma cts(\varepsilon))$$

$$\sigma c\_pr(\varepsilon) := \text{if}(\varepsilon < 0, \sigma c\_pr(\varepsilon), \sigma cts(\varepsilon))$$

$$\sigma c\_sb(\varepsilon) := \text{if}(\varepsilon < 0, \sigma c\_sb(\varepsilon), 0)$$

EFFECT OF CONFINEMENT REINFORCEMENT - EXAMPLE OF APPLICATION FOR STANDARD COLUMNS

$Dlower := 16$ mm

$$ddg := \mathrm{if}\left[-fck > 60, 16 + Dlower\cdot\left(\frac{60}{-fck}\right)^2, 16 + Dlower\right] = 25$$

$$ddg := \min(ddg, 40) = 25$$

$$As\_conf := \pi\cdot\frac{10^2}{4}$$

$s := 200$ mm

$$fywd := \frac{500}{1.1} = 454.545$$

$$bcs := 400 - 50$$

$$\sigma c2d := 2\cdot\frac{As\_conf}{s\cdot bcs}\cdot fywd = 1.02$$

$$\Delta fcd := \mathrm{if}\left[\sigma c2d > 0.6\cdot -fcd, 3.5\cdot\sigma c2d^{\frac{3}{4}}\cdot(-fcd)^{\frac{1}{4}}, 4\cdot\sigma c2d\right] = 4.08 \qquad \frac{\Delta fcd}{-fcd} = 0.09$$

$$kconf\_b := \frac{1}{3}\cdot\left(\frac{bcs}{400}\right)^2 = 0.255$$

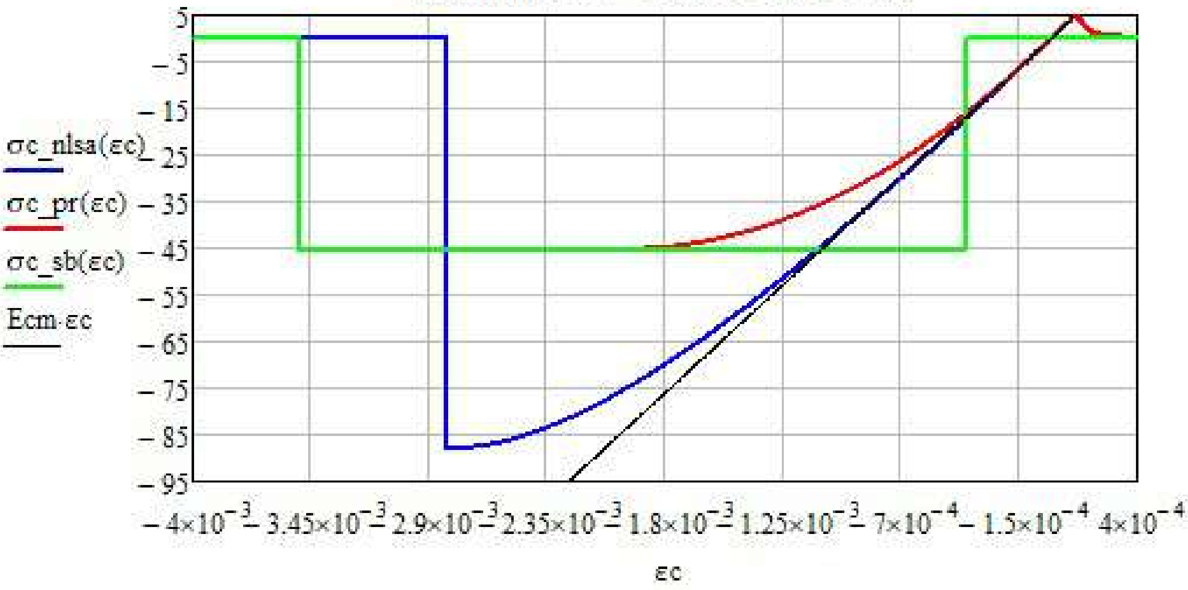
$$kconf\_s := \left(1 - \frac{s}{2\cdot bcs}\right)^2 = 0.51 \qquad 1 - \frac{s}{2\cdot bcs} = 0.714$$

$$fcd\_c := fcd - kconf\_b\cdot kconf\_s\cdot\Delta fcd = -45.886 \qquad \frac{fcd\_c}{fcd} = 1.012$$

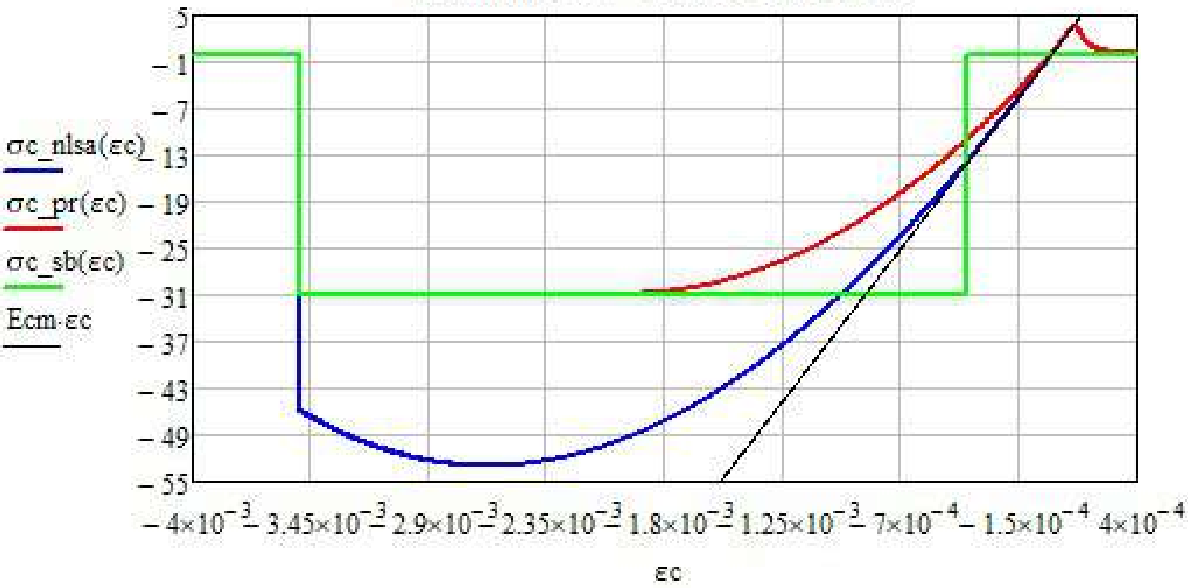
$$\varepsilon c2\_c := \varepsilon c2\cdot\left(1 - 5\cdot\frac{\Delta fcd}{fcd}\right) = -2.9\times 10^{-3} \qquad \frac{\varepsilon c2\_c}{\varepsilon c2} = 1.45$$

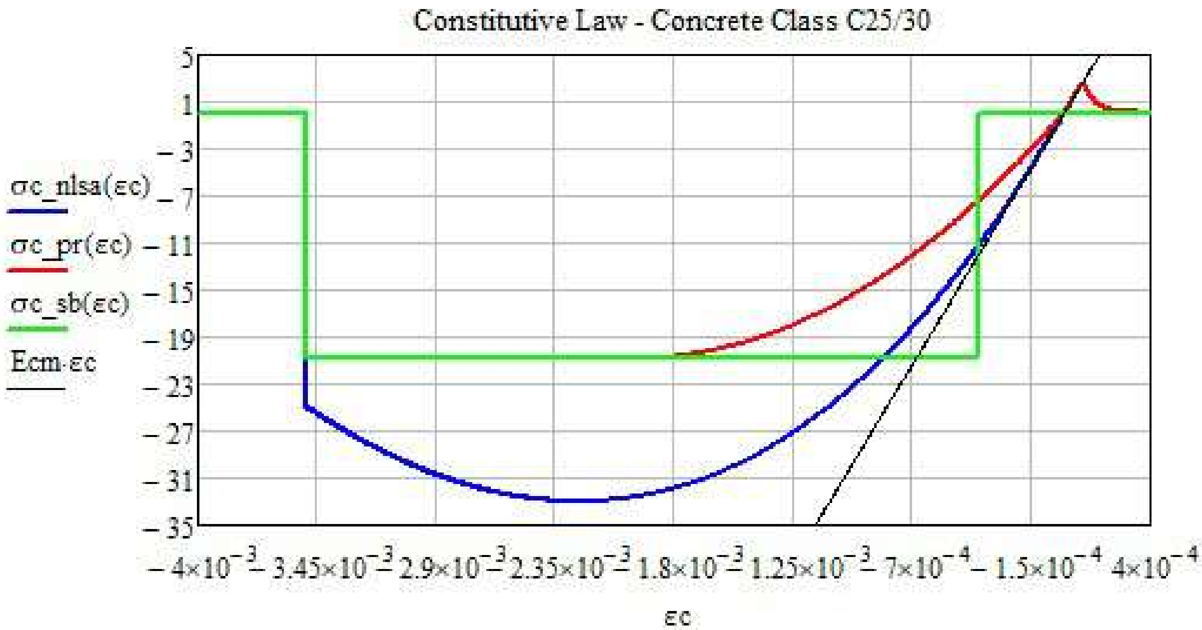
$$\varepsilon cu\_c := \varepsilon cu + 0.2\cdot\frac{\sigma c2d}{fcd} = -7.998\times 10^{-3} \qquad \frac{\varepsilon cu\_c}{\varepsilon cu} = 2.285$$

Constitutive Law - Concrete Class C80/95



Constitutive Law - Concrete Class C45/55





3.3 Steel constitutive law following EN1992-1-1:2004

The general calculation procedure is shown for steel grade B500C, but at the end also the application to mild steel B500A and prestressing steel Y1860 is given.

Rebar steel B500C (3.2)

Mild reinforcement bar steel

$\gamma s := 1.15$ Initial safety coefficient of mild steel

$\gamma spcred := 1.1$ $\gamma s := \gamma spcred = 1.1$ reduced safety coefficient of steel

$fsk := 500$ MPa characteristic axial yield strength of mild steel

$fsd := \dfrac{fsk}{\gamma spcred} = 454.545$ MPa design axial yield strength of mild steel

$fsuk := fsk \cdot 1.15 = 575$ MPa characteristic axial ultimate strength of mild steel

$fsud := \dfrac{fsuk}{\gamma spcred} = 522.727$ MPa design axial ultimate strength of mild steel

$Es := 200000$ MPa Young (elastic) modulus

$\varepsilon s\_y := \dfrac{fsd}{Es}$ $\varepsilon s\_y = 2.27273 \times 10^{-3}$ yield strain

$\varepsilon uk := 0.075$ strain at stress peak (conventional)

$\varepsilon ud := 0.9 \cdot \varepsilon uk = 0.068$ design strain at stress peak

Elastic-hardening

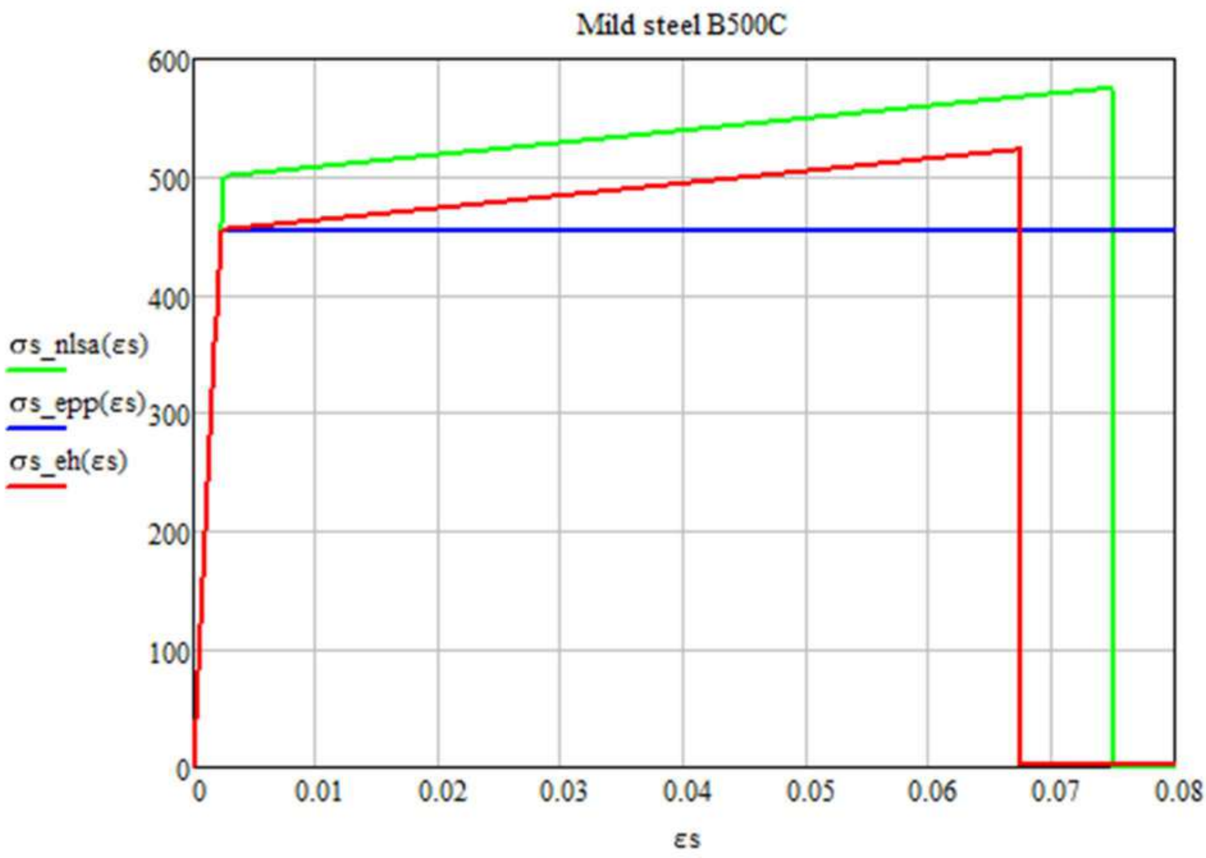
$$\sigma s\_nlsa(\varepsilon) := \mathrm{if}\left[\varepsilon \geq \frac{fsk}{Es} \wedge \varepsilon < \varepsilon uk, fsk + \frac{fsuk - fsk}{\varepsilon uk - \frac{fsk}{Es}} \cdot \left(\varepsilon - \frac{fsk}{Es}\right), \mathrm{if}\left(\varepsilon < \frac{fsk}{Es}, Es \cdot \varepsilon, 0\right)\right]$$

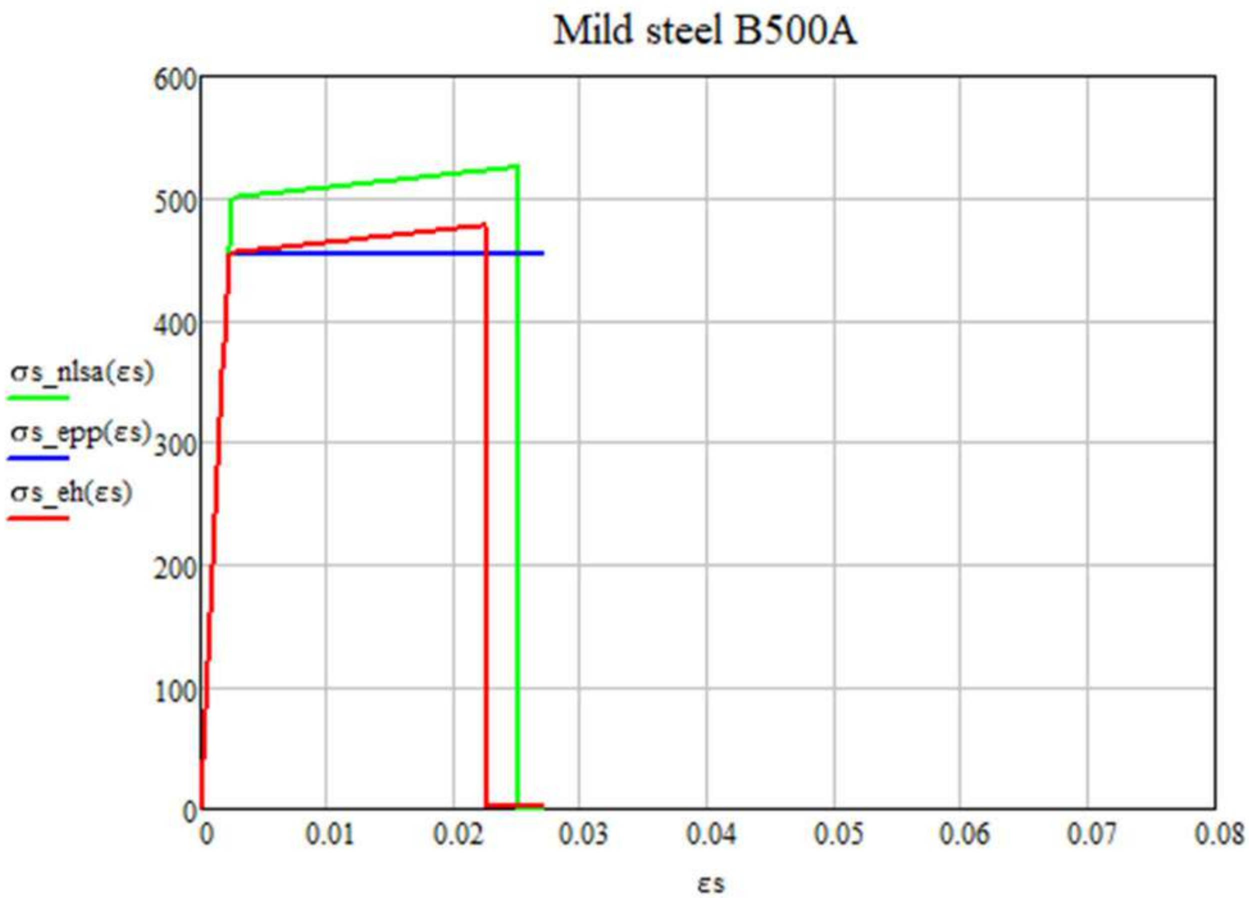
Elastic-perfect-plastic

$$\sigma s\_epp(\varepsilon) := \mathrm{if}(\varepsilon > \varepsilon s\_y, fsd, \mathrm{if}(\varepsilon < \varepsilon s\_y, Es \cdot \varepsilon, 0))$$

Elastic-hardening

$$\sigma s\_eh(\varepsilon) := \mathrm{if}\left[\varepsilon > \varepsilon s\_y \wedge \varepsilon < \varepsilon ud, fsd + \frac{fsud - fsd}{\varepsilon ud - \varepsilon s\_y} \cdot (\varepsilon - \varepsilon s\_y), \mathrm{if}(\varepsilon < \varepsilon s\_y, Es \cdot \varepsilon, 1)\right]$$


Mild steel B500C
600
500
400
300
200
100
0
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
σs_nlsa(εs)
σs_epp(εs)
σs_eh(εs)
εs


Mild steel B500A
600
500
400
300
200
100
0
σs_nlsa(εs)
σs_epp(εs)
σs_eh(εs)
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
εs

Prestressing steel Y1860 (§3.3)

$\gamma p := 1.15$ — Material safety coefficient for mild steel

$\gamma p := 1.1$ — due to fulfillment of the conditions in A.2.1

$Ep := 195000$ MPa — Young (elastic) modulus

$fptk := 1860$ MPa — Characteristic ultimate strength of prestressing steel in tension/compression

$fptd := fptk \cdot \frac{1}{\gamma p}$ — Design ultimate strength of prestressing steel in tension/compression

$\varepsilon pud := 0.02$ — design ultimate strain

$\varepsilon puk := \frac{\varepsilon pud}{0.9} = 0.022$ — characteristic ultimate strain

$fp01k := 0.9 \cdot fptk$ — Characteristic strength of prestressing steel in tension/compression at 0.1% of residual strain

$fp01d := \frac{fp01k}{\gamma p} = 1.522 \times 10^{3}$ MPa — Design strength of prestressing steel in tension/compression at 0.1% of residual strain

$\varepsilon py := \frac{fp01d}{Ep} = 7.804 \times 10^{-3}$ — design equivalent yield strain

Non-linear structural analysis

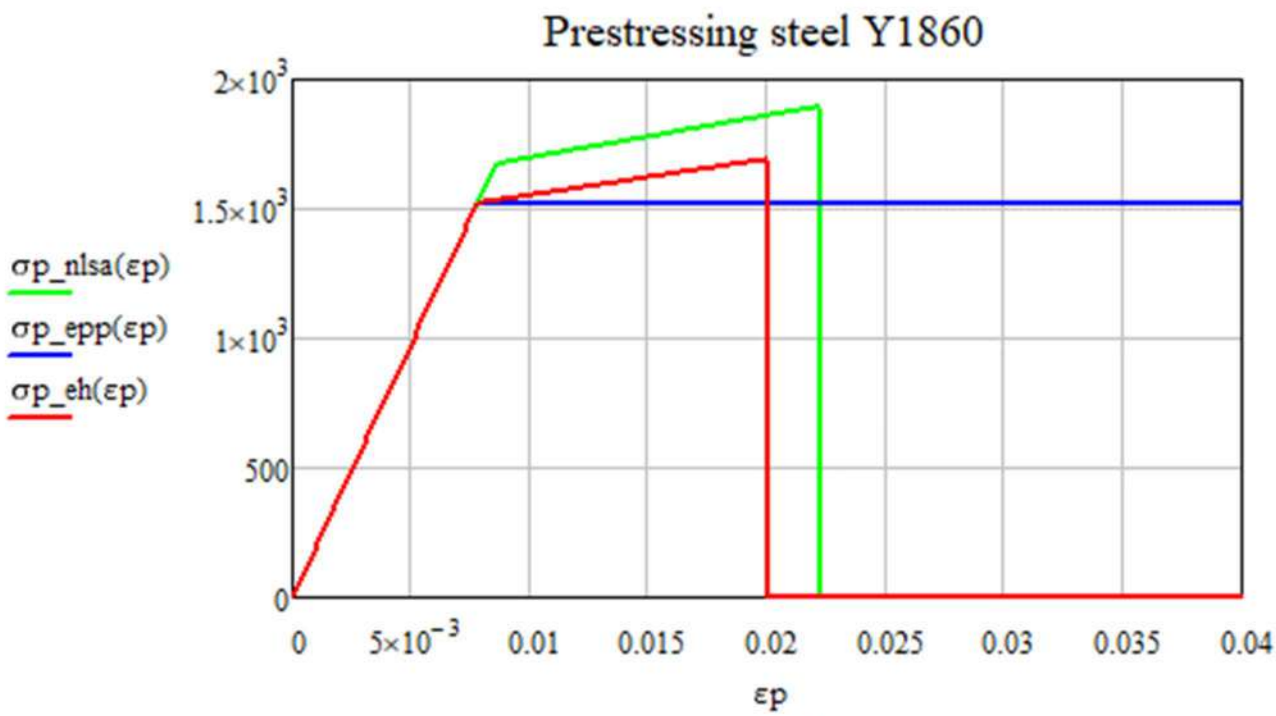
$$\sigma p\_nlsa(\varepsilon) := \mathrm{if}\left[\varepsilon > \frac{fp01k}{Ep} \wedge \varepsilon < \varepsilon puk, fp01k + \frac{fptk - fp01k}{\varepsilon pud - \frac{fp01k}{Ep}} \cdot \left(\varepsilon - \frac{fp01k}{Ep}\right), \mathrm{if}\left(\varepsilon < \frac{fp01k}{Ep}, Ep \cdot \varepsilon, 0\right)\right]$$

Elastic-perfect-plastic

$$\sigma p\_epp(\varepsilon) := \mathrm{if}(\varepsilon > \varepsilon py, fp01d, \mathrm{if}(\varepsilon < \varepsilon py, Ep \cdot \varepsilon, 0))$$

Elastic-hardening

$$\sigma p\_eh(\varepsilon) := \mathrm{if}\left[\varepsilon > \varepsilon py \wedge \varepsilon < \varepsilon pud, fp01d + \frac{fptd - fp01d}{\varepsilon pud - \varepsilon py} \cdot (\varepsilon - \varepsilon py), \mathrm{if}(\varepsilon < \varepsilon py, Ep \cdot \varepsilon, 0)\right]$$



3.4 Steel constitutive law following FprEN1992-1-1:2022

The general calculation procedure is shown for prestressing steel grade Y1860, but at the end also the application to mild steel B500C and mild steel B500A (equal to the previous standard) is given.

Prestressing steel Y1860

$\gamma p := 1.15$ Material safety coefficient for mild steel

$\gamma p := 1.1$ due to fulfillment of case (a) in table A.1 (NDP) and AVCP 2+

$Ep := 195000$ MPa Elastic modulus

$fptk := 1860$ MPa Characteristic ultimate strength of prestressing steel in tension/compression

$fptd := fptk \cdot \frac{1}{\gamma p} = 1.691 \times 10^{3}$ MPa Design ultimate strength of prestressing steel in tension/compression

$\varepsilon puk := 0.035$ design ultimate strain

$\varepsilon pud := 0.9 \cdot \varepsilon puk = 0.032$ characteristic ultimate strain

$fp01k := 1640$ MPa Characteristic strength of prestressing steel in tension/compression at 0.1% of residual strain

$fp01d := \frac{fp01k}{\gamma p} = 1.491 \times 10^{3}$ MPa Design strength of prestressing steel in tension/compression at 0.1% of residual strain

$\varepsilon py := \frac{fp01d}{Ep} = 7.646 \times 10^{-3}$ design equivalent yield strain

Non-linear structural analysis

$$\sigma p\_nlsa(\varepsilon) := if\left[\varepsilon > \frac{fp01k}{Ep} \wedge \varepsilon < \varepsilon puk, fp01k + \frac{fptk - fp01k}{\varepsilon pud - \frac{fp01k}{Ep}} \cdot \left(\varepsilon - \frac{fp01k}{Ep}\right), if\left(\varepsilon < \frac{fp01k}{Ep}, Ep \cdot \varepsilon, 0\right)\right]$$

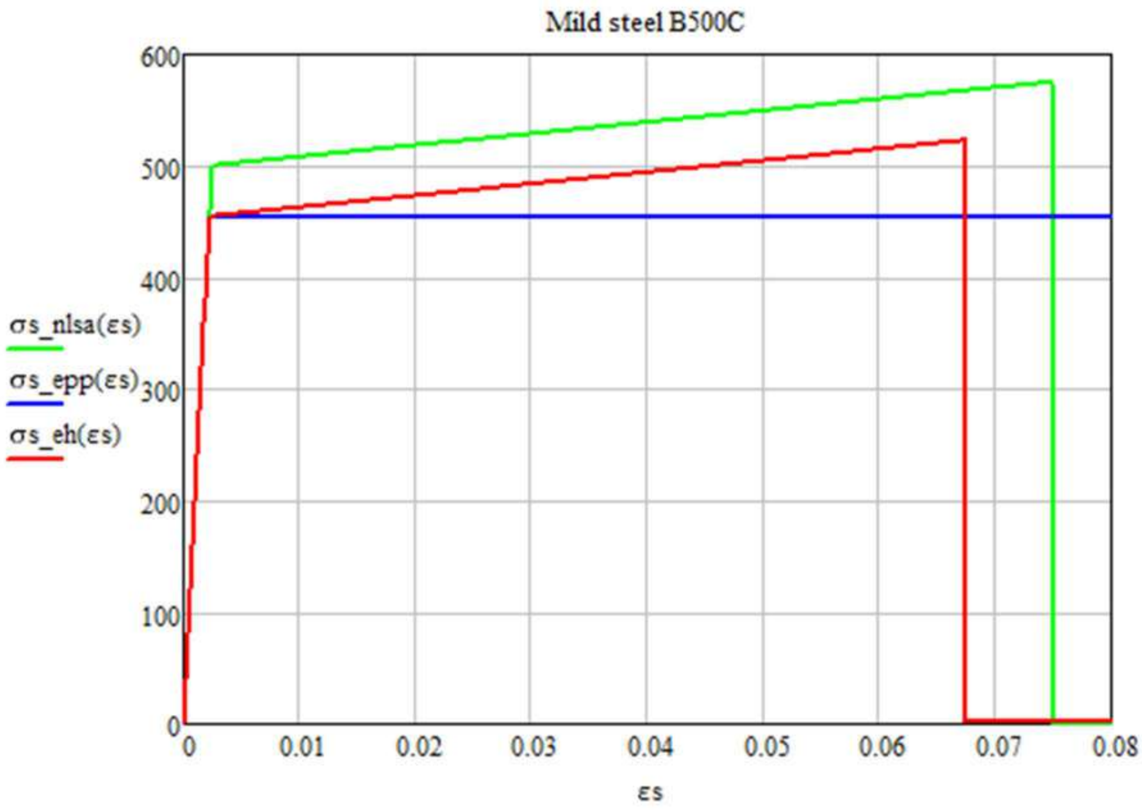
Elastic-perfect-plastic

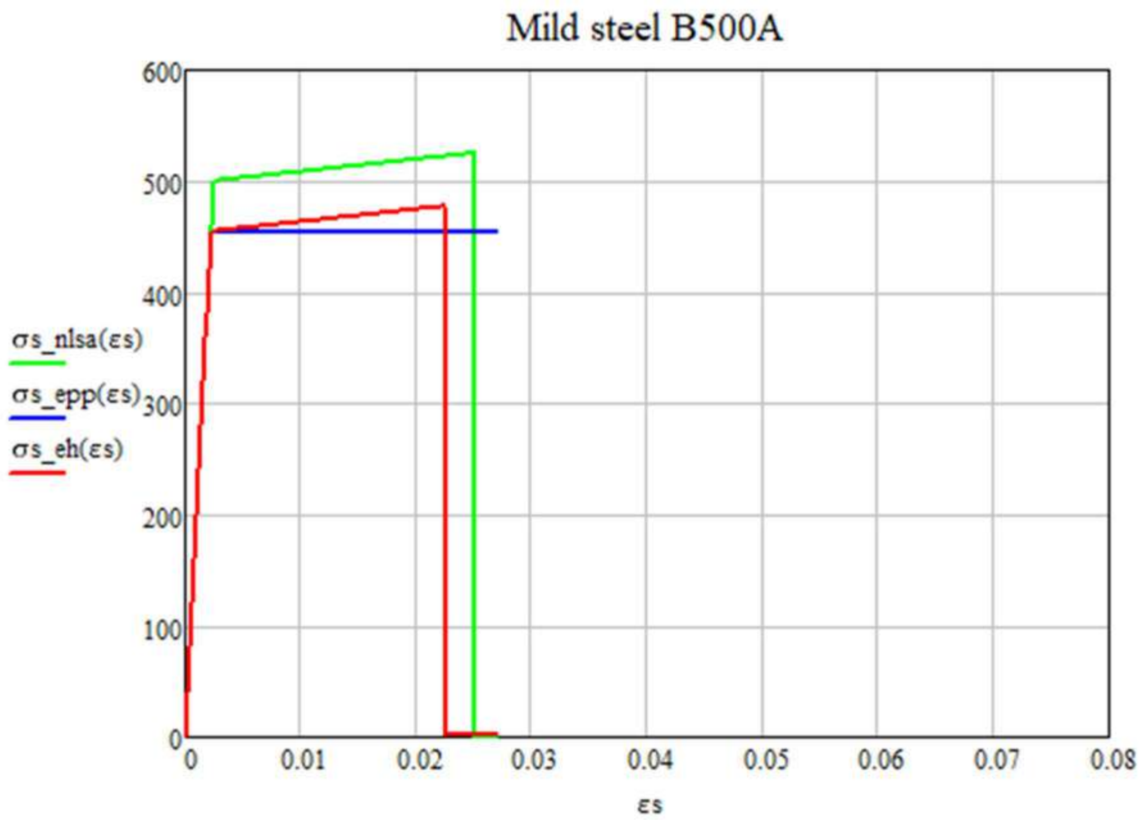
$$\sigma p\_epp(\varepsilon) := if(\varepsilon > \varepsilon py, fp01d, if(\varepsilon < \varepsilon py, Ep \cdot \varepsilon, 0))$$

Elastic-hardening

$$\sigma p\_eh(\varepsilon) := if\left[\varepsilon > \varepsilon py \wedge \varepsilon < \varepsilon pud, fp01d + \frac{fptd - fp01d}{\varepsilon pud - \varepsilon py} \cdot (\varepsilon - \varepsilon py), if(\varepsilon < \varepsilon py, Ep \cdot \varepsilon, 0)\right]$$

$\varepsilon p := 0, 0.0001 .. \varepsilon pud \cdot 2$


Mild steel B500C
σs_nlsa(εs)
σs_epp(εs)
σs_eh(εs)
εs
600
500
400
300
200
100
0
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08


Mild steel B500A
σs_nlsa(εs)
σs_epp(εs)
σs_eh(εs)
εs
600
500
400
300
200
100
0
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08

Prestressing steel Y1860

$\gamma p := 1.15$ Material safety coefficient for mild steel

$\gamma p := 1.1$ due to fulfillment of case (a) in table A.1 (NDP) and AVCP 2+

$Ep := 195000$ MPa Elastic modulus

$fptk := 1860$ MPa Characteristic ultimate strength of prestressing steel in tension/compression

$fptd := fptk \cdot \frac{1}{\gamma p} = 1.691 \times 10^{3}$ MPa Design ultimate strength of prestressing steel in tension/compression

$\varepsilon puk := 0.035$ design ultimate strain

$\varepsilon pud := 0.9 \cdot \varepsilon puk = 0.032$ characteristic ultimate strain

$fp01k := 1640$ MPa Characteristic strength of prestressing steel in tension/compression at 0.1% of residual strain

$fp01d := \frac{fp01k}{\gamma p} = 1.491 \times 10^{3}$ MPa Design strength of prestressing steel in tension/compression at 0.1% of residual strain

$\varepsilon py := \frac{fp01d}{Ep} = 7.646 \times 10^{-3}$ design equivalent yield strain

Non-linear structural analysis

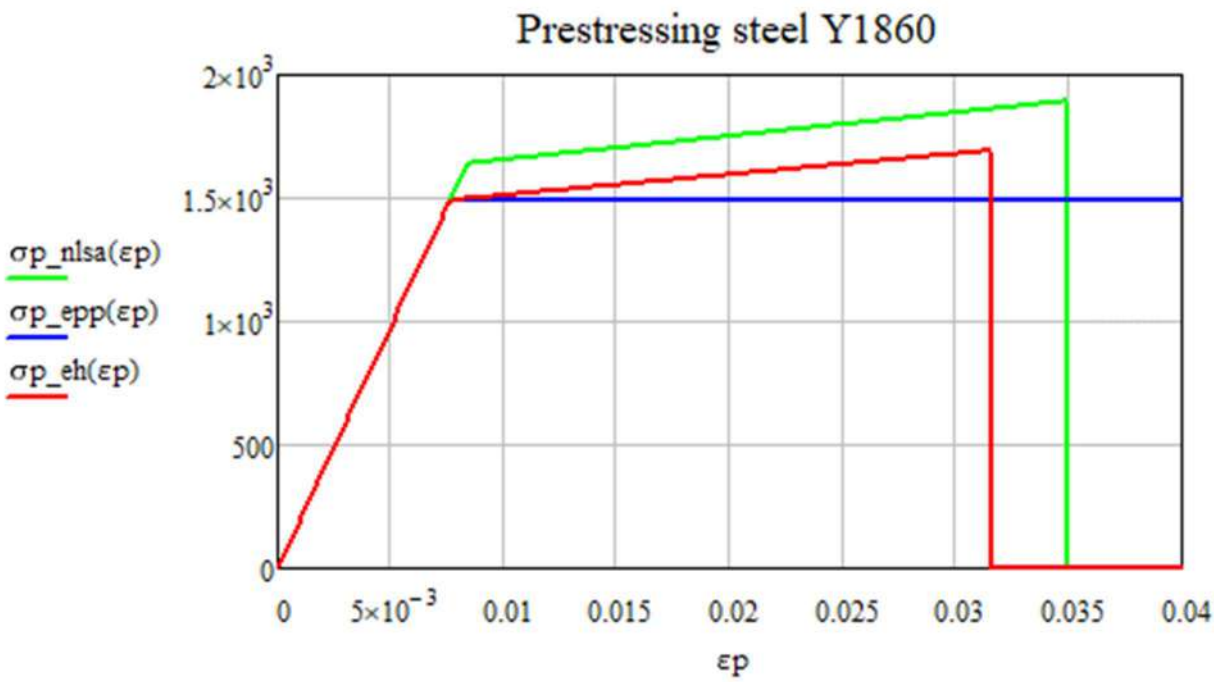
$$\sigma p\_nlsa(\varepsilon) := if\left[\varepsilon > \frac{fp01k}{Ep} \wedge \varepsilon < \varepsilon puk, fp01k + \frac{fptk - fp01k}{\varepsilon pud - \frac{fp01k}{Ep}} \cdot \left(\varepsilon - \frac{fp01k}{Ep}\right), if\left(\varepsilon < \frac{fp01k}{Ep}, Ep \cdot \varepsilon, 0\right)\right]$$

Elastic-perfect-plastic

$$\sigma p\_epp(\varepsilon) := if(\varepsilon > \varepsilon py, fp01d, if(\varepsilon < \varepsilon py, Ep \cdot \varepsilon, 0))$$

Elastic-hardening

$$\sigma p\_eh(\varepsilon) := if\left[\varepsilon > \varepsilon py \wedge \varepsilon < \varepsilon pud, fp01d + \frac{fptd - fp01d}{\varepsilon pud - \varepsilon py} \cdot (\varepsilon - \varepsilon py), if(\varepsilon < \varepsilon py, Ep \cdot \varepsilon, 0)\right]$$



3.5 Concrete time-dependent behaviour following EN1992-1-1:2004

CONCRETE STRENGTH DEVELOPMENT THROUGH TIME

$s := 0.2$ for class 52.5R cement

$$\beta cc(t) := e^{s \cdot \left[1 - \left(\frac{28}{t}\right)^{\frac{1}{2}}\right]}$$

$$Ecj(t) := Ecm \cdot \beta cc(t)^{0.3}$$

$$fcmj(t) := \beta cc(t) \cdot fcm$$

$$fckj(t) := fcmj(t) - 8$$

$$fctmj(t) := \text{if}\left(t < 28, \beta cc(t) \cdot fctm, \beta cc(t)^{\frac{2}{3}} \cdot fctm\right)$$

$$fctdj(t) := 0.7 \cdot \frac{\alpha cc \cdot fctmj(t)}{\gamma c}$$

3.6 Concrete time-dependent behaviour following FprEN1992-1-1:2022

CONCRETE STRENGTH DEVELOPMENT THROUGH TIME

$tref := 28$

$sc := if(-fck \le 35, 0.3, if(-fck \ge 60, 0.1, 0.2)) = 0.2$ for class CR cement

$$\beta cc(t) := if\left[t < tref, e^{sc\cdot\left(1-\sqrt{\frac{tref}{t}}\right)\cdot\sqrt{\frac{28}{tref}}}, 1\right]$$

$$Ecj(t) := Ecm\cdot\beta cc(t)^{\frac{1}{3}}$$

$$fcmj(t) := \beta cc(t)\cdot fcm$$

$$fctmj(t) := \beta cc(t)^{0.6}\cdot fctm$$

4 Minimum concrete cover

This chapter describes the minimum concrete cover (clear cover – from out of the bar diameter to the concrete edge) calculated according to the two standards. As described in the comment chapter, the procedure employed is the same for the two standards.

4.1 Exposure class XC1

MINIMUM CONCRETE COVER

initial structural class S4

nominal strand diameter

12.7 mm 0.5'

15.24 mm 0.6'

$exp\_class\_XC := 1$

$cmin\_b\_s := 10$ max diametre of rebar

$cmin\_b\_p := 2.5 \cdot 15.24 = 38.1$ associated to 0.6'

$cmin\_dur\_s := if(exp\_class\_XC = 1, 10, if(exp\_class\_XC = 4, 25, if(exp\_class\_XC = 0, 10, 20)))$ S3

$cmin\_dur\_p := if(exp\_class\_XC = 1, 20, if(exp\_class\_XC = 4, 35, if(exp\_class\_XC = 0, 10, 30)))$ S3

$\Delta cdur\_\gamma := 0$

$\Delta cdur\_st := 0$

$\Delta cdur\_add := 5$

$\Delta c\_dev := 5$ for precast members with production control

$cmin\_s := max(cmin\_b\_s, cmin\_dur\_s + \Delta cdur\_\gamma - \Delta cdur\_st - \Delta cdur\_add, 10) = 10$ mm

$cmin\_p := max(cmin\_b\_p, cmin\_dur\_p + \Delta cdur\_\gamma - \Delta cdur\_st - \Delta cdur\_add, 10) = 38.1$ mm

$cnom\_s := cmin\_s + \Delta c\_dev = 15$ mm

$cnom\_p := cmin\_p + \Delta c\_dev = 43.1$ mm

$cnom\_p + \frac{15.24}{2} = 50.72$

4.2 Exposure class XC2

MINIMUM CONCRETE COVER

initial structural class S4

nominal strand diameter

12.7	mm	0.5'
15.24	mm	0.6'

$exp\_class\_XC := 2$

$cmin\_b\_s := 10$ max diametre of rebar

$cmin\_b\_p := 2.5 \cdot 15.24 = 38.1$ associated to 0.6'

$cmin\_dur\_s := if(exp\_class\_XC = 1, 10, if(exp\_class\_XC = 4, 25, if(exp\_class\_XC = 0, 10, 20)))$ S3

$cmin\_dur\_p := if(exp\_class\_XC = 1, 20, if(exp\_class\_XC = 4, 35, if(exp\_class\_XC = 0, 10, 30)))$ S3

$\Delta cdur\_\gamma := 0$

$\Delta cdur\_st := 0$

$\Delta cdur\_add := 5$

$\Delta c\_dev := 5$ for precast members with production control

$cmin\_s := max(cmin\_b\_s, cmin\_dur\_s + \Delta cdur\_\gamma - \Delta cdur\_st - \Delta cdur\_add, 10) = 15$ mm

$cmin\_p := max(cmin\_b\_p, cmin\_dur\_p + \Delta cdur\_\gamma - \Delta cdur\_st - \Delta cdur\_add, 10) = 38.1$ mm

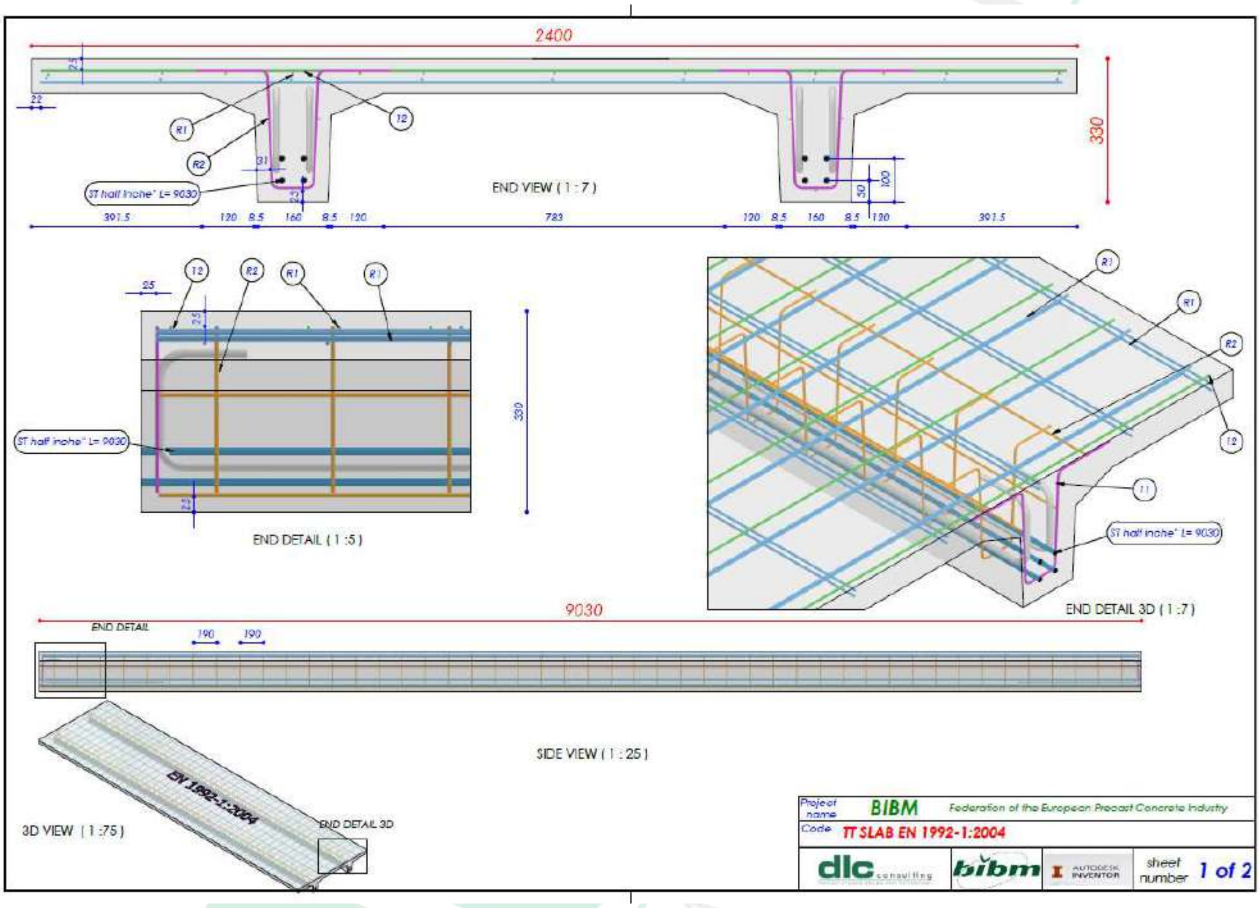
$cnom\_s := cmin\_s + \Delta c\_dev = 20$ mm

$cnom\_p := cmin\_p + \Delta c\_dev = 43.1$ mm

$$cnom\_p + \frac{15.24}{2} = 50.72$$

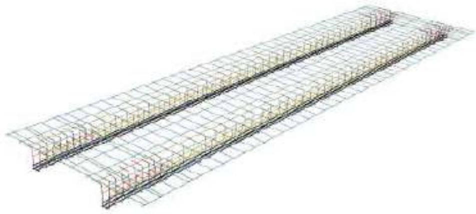
5 TT element – EN1992-1:2004

5.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_	Ø_longitudinal	pattern_T	Ø_transverse	pattern_L
	11	4	208	832	6 mm				
	12	24	522	12528	6 mm				
	21	8	2026	16208	16 mm				
Total mass rebars [kg]				29,57		Incidence kg/m³	11,33		
	R1	2	41610	83220		6 mm	200 mm	6 mm	300 mm
	R2	2	18683	37366		6 mm	200 mm	6 mm	190 mm
Total mass welded-wire-meshes [kg]				120,59		Incidence kg/m³	46,20		
	ST half inche" L=9030	8	6599	52792	12,7 mm				
Total mass strands [kg]				52,792		Incidence kg/m³	20,23		
	Total mass of steel [kg]			202,95		Total concrete volume [m³]	2,61		

CODE	DESCRIPTION	VOLUME
TT001 CONCRETE	TT SLAB 001 CONCRETE	2,611 m^3



Project name: BIBM — Federation of the European Precast Concrete Industry

Code: TT SLAB EN 1992-1:2004

dlc consulting | bibm | AUTODESK INVENTOR | sheet number 2 of 2

5.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$$y\_tr := (0 \;\; 79.99 \;\; 80 \;\; 130 \;\; 330)^T$$

$Htot := max(y\_tr)$ maximum depth

Width of corresponding chord:

$$b\_tr := (2400 \;\; 2400 \;\; 1566 \;\; 354 \;\; 320)^T$$

$r\_circ := 0$ radius of central void pipe

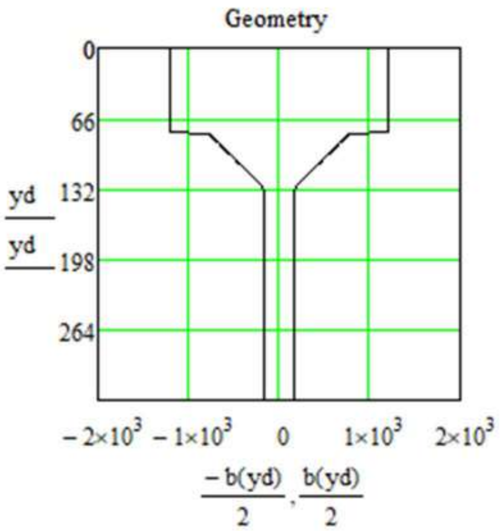
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$$b\_lin(y) := linterp(y\_tr, b\_tr, y)$$

$$b\_circ(y) := linterp(y\_tr, b\_tr, y) - x\_circ(y)$$

$$b(y) := if\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$$yd := 0..Htot$$



condensed 1D geometry plot

$u := 2400 + 1800 + 320 + 300\cdot4 = 5.72\times10^3$ mm exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (25 \quad 300)^T$$

Area of reinforcement at each depth

$$As := (12 \cdot A(6) \quad 2 \cdot A(6))^T$$

$$js := \text{rows}(As) \qquad js = 2$$

$$dsmax := \max(ds) \qquad dsmax = 300$$

$$As\_tot := \sum_{j=1}^{js} As_j = 395.841$$

Prestressing reinforcement

nominal strand diameter

12.7	mm	0.5'
15.24	mm	0.6'

Area of a single strand:

$Ap0 := 93$ $\phi p := 12.7$ mm nominal strand diameter

Depth of prestressing strands from upper chord:

$dp := (180\ \ 230\ \ 280)^T$

Area of strands at each depth:

$Ap := (0 \cdot Ap0\ \ 4 \cdot Ap0\ \ 4 \cdot Ap0)^T$

$\sigma p0 := 1400$ MPa

$\sigma prec := (0.4 \cdot \sigma p0\ \ 1 \cdot \sigma p0\ \ \sigma p0)^T$ initial prestressing

$losses := 0 \cdot (1\ \ 1\ \ 1)^T$ in percentual **% (losses are introduced later)**

$jp := rows(Ap)$ $jp = 3$

$k := 1 .. jp$

$$\sigma o_k := \sigma prec_k \cdot \left[\frac{(100 - losses_k)}{100}\right] \qquad \sigma o = \begin{pmatrix} 560 \\ 1.4 \times 10^3 \\ 1.4 \times 10^3 \end{pmatrix}$$

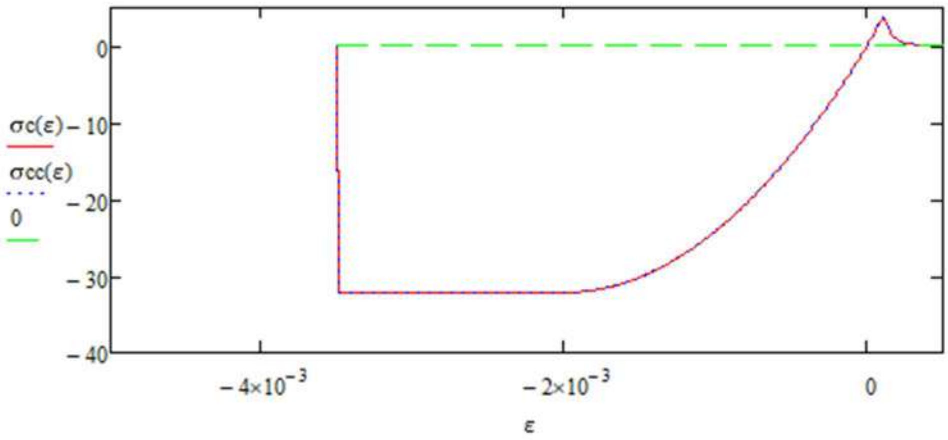
$$Ap\_tot := \sum_{k=1}^{jp} Ap_k \qquad Ap\_tot = 744$$

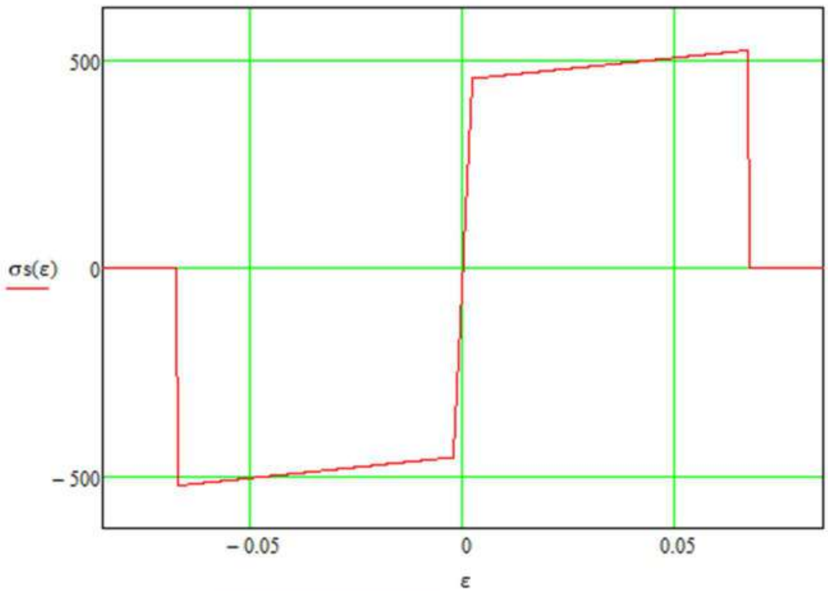
$ypmax := max(dp)$ $ypmax = 280$

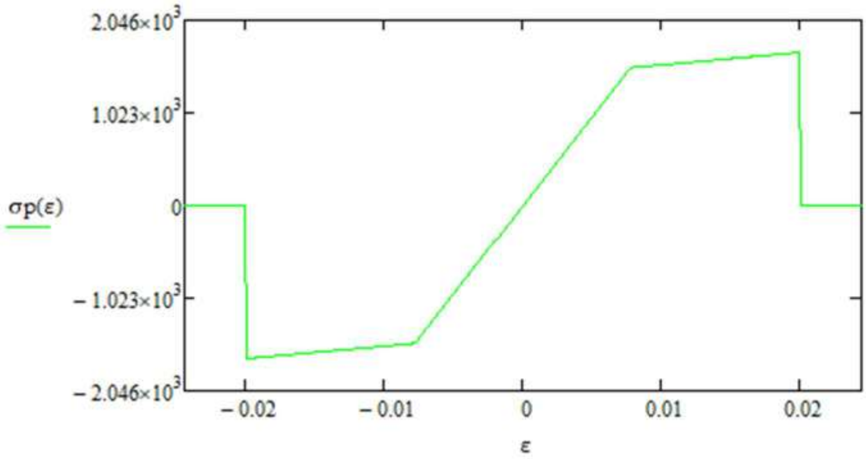
$$Np\_tot := \sum_{k=1}^{jp} \left(\left(Ap_k \cdot \sigma o_k\right)\right) \qquad Np\_tot = 1.042 \times 10^6$$ total prestressing initial force

$$Yp := \frac{\sum_{k=1}^{jp} \left(dp_k \cdot Ap_k \cdot \sigma o_k\right)}{\sum_{k=1}^{jp} \left(Ap_k \cdot \sigma o_k\right)} = 255$$ mm centre of gravity of prestressing

5.3 Material constitutive laws employed in the calculation







5.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 3.074 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 1.288 \times 10^{-3}$$ geometric ratio for longitudinal mild reinforcement

$$\rho p := \frac{Ap\_tot}{Ac} = 2.42 \times 10^{-3}$$ geometric ratio for longitudinal prestressing tendons

$$\rho tot := \frac{As\_tot + Ap\_tot}{Ac} = 3.708 \times 10^{-3}$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y)\cdot y\,dy \qquad Syc = 2.785 \times 10^7$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 90.619$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y)\cdot(y - yG)^2\,dy \qquad Ixo\_cls = 2.109 \times 10^9$$

Global area of all prestressing reinforcement

$$Area\_tr := \left|\begin{array}{l} s \leftarrow 0 \\ \text{for } x \in 1..jp \\ \quad s \leftarrow Ap_x + s \end{array}\right. \qquad Area\_tr = 744$$

First moment of the area referred to prestressing reinforcement only

$$Sxp := \sum_{i=1}^{jp} \left(Ap_i \cdot dp_i\right) \qquad Sxp = 1.897 \times 10^5$$

Centre of gravity of prestressing

$$Yp := \frac{Sxp}{Area\_tr} \qquad Yp = 255$$

Idealisation coefficients (elastic)

$$np := \frac{Ep}{Ecm} \qquad np = 5.374$$

$$ns := \frac{Es}{Ecm} \qquad ns = 5.512$$

Area of ideal cross-section

$$Aid := Ac + (np - 1)\cdot\sum_{j=1}^{jp} Ap_j + (ns - 1)\cdot\sum_{j=1}^{js} As_j \qquad Aid = 3.124\times10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac\cdot yG + (np - 1)\cdot(Area\_tr\cdot Yp) + (ns - 1)\cdot\sum_{j=1}^{js}\left(As_j\cdot ds_j\right) \qquad Sxid = 2.88\times10^7$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 92.181$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y)\cdot(y - Yid)^2\,dy - \sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right] - \sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the prestressing reinforcement area

$$Ixoidprec := np\cdot\sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right]$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns\cdot\sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidprec + Ixoidlenta \qquad Ixo\_id = 2.216\times10^9 \quad \text{mm^4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.051$$

5.5 Loads

LOADS

$interaxis := 2400$ mm

$g1 := Ac \cdot 0.000025 = 7.685$ kN/m dead load from self-weight

$g2 := 2 \cdot \frac{interaxis}{1000} = 4.8$ kN/m nonstructural dead load

$q := 3 \cdot \frac{interaxis}{1000} = 7.2$ kN/m live load

$L := 8850$ mm calculation length (span between supports)

$\psi 2 := 0.3$ non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from live load

5.6 Prestressing transfer and time-dependent behaviour

TRANSFER OF PRESTRESS (§8.10.2.2)

$\alpha 1 := 1$ gradual release of prestressing

$\alpha 2 := 0.19$ for 7-wire strands

$\sigma pm0 := \sigma p0 = 1.4 \times 10^3$ MPa initial prestressing

$\eta p1 := 3.2$ for 7-wire strands

$\eta 1 := 1$ in favourable position

$fbpt := \eta p1 \cdot \eta 1 \cdot fctdj(2) = 3.51$ MPa equivalent constant bond stress at prestress realease following §(8.15)

$lpt := \frac{\alpha 1 \cdot \alpha 2 \cdot \sigma pm0}{fbpt} \cdot \phi p = 962.587$ mm basic value of the transmission length following §(8.16)

$lpt1 := 0.8 lpt = 770.069$ mm lower-bound transfer length following §(8.17)

$lpt2 := 1.2 \cdot lpt = 1.155 \times 10^3$ mm upper-bound transfer length following §(8.18)

Prestress losses

$hn := 2 \cdot \frac{Ac}{u} = 107.477$ mm

$\varepsilon cs := \frac{0.65}{1000} = 6.5 \times 10^{-4}$ shrinkage strain assumed as a result of laboratory tests on the specific concrete mix employed

$\rho 1000 := 0.025$ for class 2 (low-relaxation) tendons following §3.3.2(5)

$k\rho := 0.16$

$t := 50 \cdot 365 = 1.825 \times 10^{4}$ days Life span

$$\sigma cpQP2(x) := \frac{-Np\_tot}{Aid} + \frac{[Mq\_SLSg1(x) - Np\_tot \cdot (Yp - Yid)] \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP2\left(\frac{L}{2}\right) = -10.265 \text{ MPa}$$

stress in quasi-permanent load combination at 2 days
(conventional equivalent time for prestressing release)

$$\sigma cpQP23(x) := \frac{Mq\_SLSg2(x) \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP23\left(\frac{L}{2}\right) = 3.452 \text{ MPa}$$

stress in quasi-permanent load combination at 23 days
(conventional time for assemblage of the structure on site)

$$\sigma cpQP91(x) := \frac{Mq\_SLSq(x) \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP91\left(\frac{L}{2}\right) = 1.553 \text{ MPa}$$

stress in quasi-permanent load combination at 91 days
(conventional time for enter in use of the structure)

$$\Delta\sigma pr(x,t) := \left[\sigma p0 + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x))\right] \cdot \rho 1000 \cdot \left(\frac{24 \cdot t}{1000}\right)^{k\rho}$$

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$h0 := 2 \cdot \frac{Ac}{u} = 107.477$ mm notional size of the member

$RH := 50$ % relative humidity

$t0\_T(t0) := t0$

$\alpha := 1$ for cement class R

$$t0\_mod(t0) := \max\left[t0\_T(t0) \cdot \left(\frac{9}{2 + t0\_T(t0)^{1.2}} + 1\right)^{\alpha}, 0.5\right]$$

time modification due to type of cement §B.9

$t0\_mod(2) = 6.189$

$$\alpha c1 := \left(\frac{35}{-fcm}\right)^{0.7} = 0.748$$

$$\alpha c2 := \left(\frac{35}{-fcm}\right)^{0.2} = 0.92$$

$$\alpha c3 := \left(\frac{35}{-fcm}\right)^{0.5} = 0.813$$

$$\beta h := \text{if}\left[-fcm > 35, \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250 \cdot \alpha c3, 1500 \cdot \alpha c3\right], \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250, 1500\right]\right] = 364.39$$

$$\beta t0(t0) := \frac{1}{0.1 + t0\_mod(t0)^{0.2}}$$

$$\beta c(t, t0) := \left(\frac{t - t0\_mod(t0)}{\beta h + t - t0\_mod(t0)}\right)^{0.3}$$

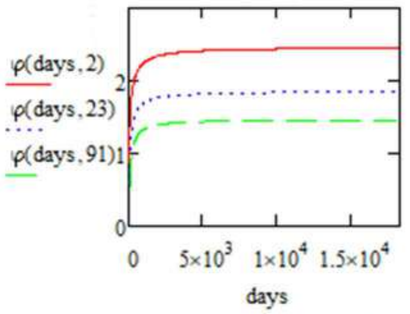
$$\beta fcm := \frac{16.8}{\sqrt{-fcm}} = 2.308$$

$$\varphi RH := \text{if}\left[-fcm > 35, \left(1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}} \cdot \alpha c1\right) \cdot \alpha c2, 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}}\right] = 1.644$$

$\varphi 0(t0) := \varphi RH \cdot \beta fcm \cdot \beta t0(t0)$

$\varphi(t, t0) := \varphi 0(t0) \cdot \beta c(t, t0)$

$\varphi(t, 2) = 2.45$ $\varphi(t, 91) = 1.46$



TIME-DEPENDENT LOSSES OF PRESTRESS (§5.10.6)

$$\Delta\sigma p\_csr(x,t) := \frac{-\varepsilon cs \cdot Ep - 0.8 \cdot \Delta\sigma pr(x,t) + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) \cdot \varphi(t,2) + \sigma cpQP23(x) \cdot \varphi(t,23) + \sigma cpQP91(x) \cdot \varphi(t,91))}{1 + \frac{Ep}{Ecm} \cdot \frac{Ap\_tot}{Ac} \cdot \left[1 + \frac{Ac}{Ixoidcls} \cdot (Yp - Yid)^2\right] \cdot \left(1 + 0.8 \cdot \frac{\varphi(t,2) \cdot \sigma cpQP2(x) + \varphi(t,23) \cdot \sigma cpQP23(x) + \varphi(t,91) \cdot \sigma cpQP91(x)}{\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)}\right)}$$

prestress losses following §(5.46)

NOTE: a weighed creep coefficient was considered accounting for the 3 load phases previously introduced

$$\sigma pm(x,t) := \sigma p0 - \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)) + \Delta\sigma p\_csr(x,t)$$ prestress considering immediate and delayed losses

$$\frac{\sigma pm\left(\frac{L}{2}, 365 \cdot 50\right)}{\sigma p0} = 0.852$$ expected residual prestress ratio after 50 years of life with respect to initial

$$\varepsilon pm := \frac{\sigma pm\left(\frac{L}{2}, 365 \cdot 50\right)}{\sigma p0} \cdot \varepsilon p0$$ expected residual strain after 50 years of life with respect to initial

$$\sigma pm\left(\frac{L}{2}, 365 \cdot 50\right) \cdot Ap\_tot = 8.876 \times 10^5$$ N residual prestress force after 50 years of life

$$Np\_tot = 1.042 \times 10^6$$ N initial prestress force

5.7 Non-linear moment-curvature diagram

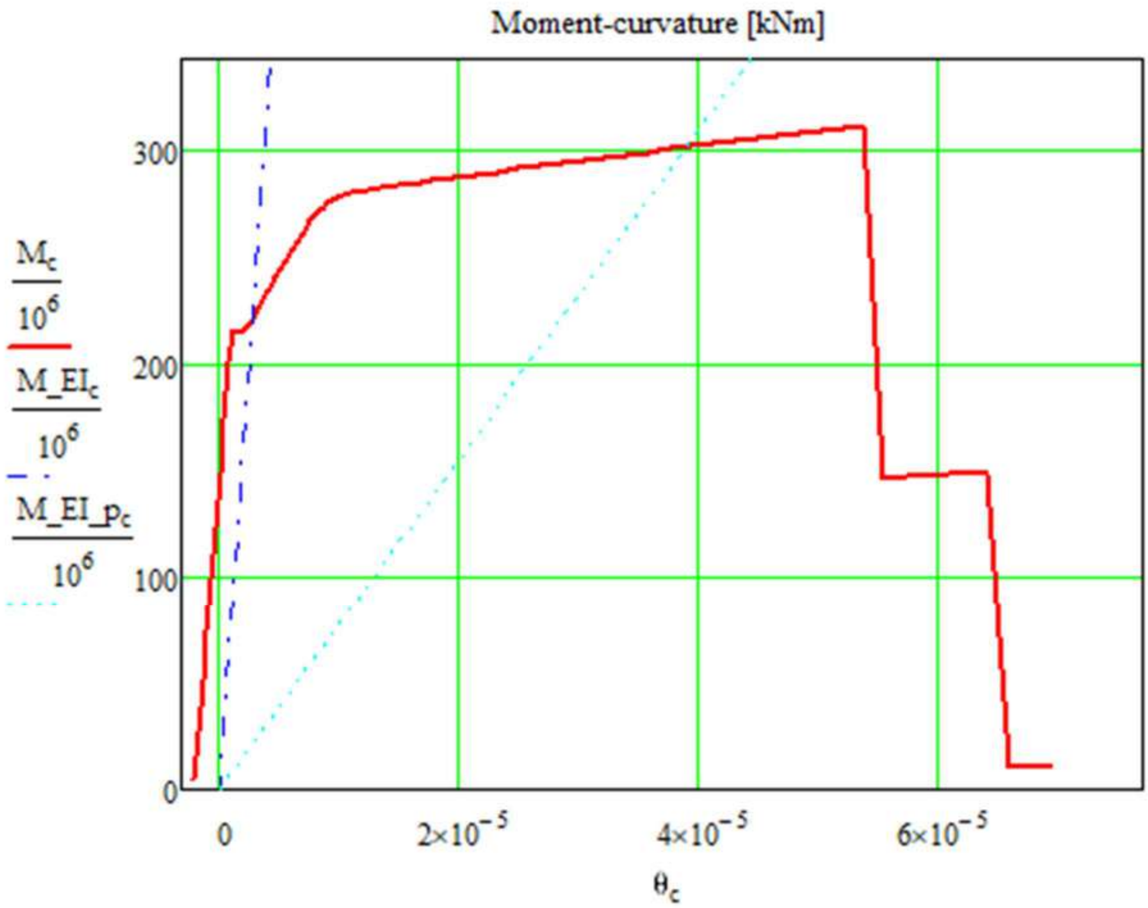
Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

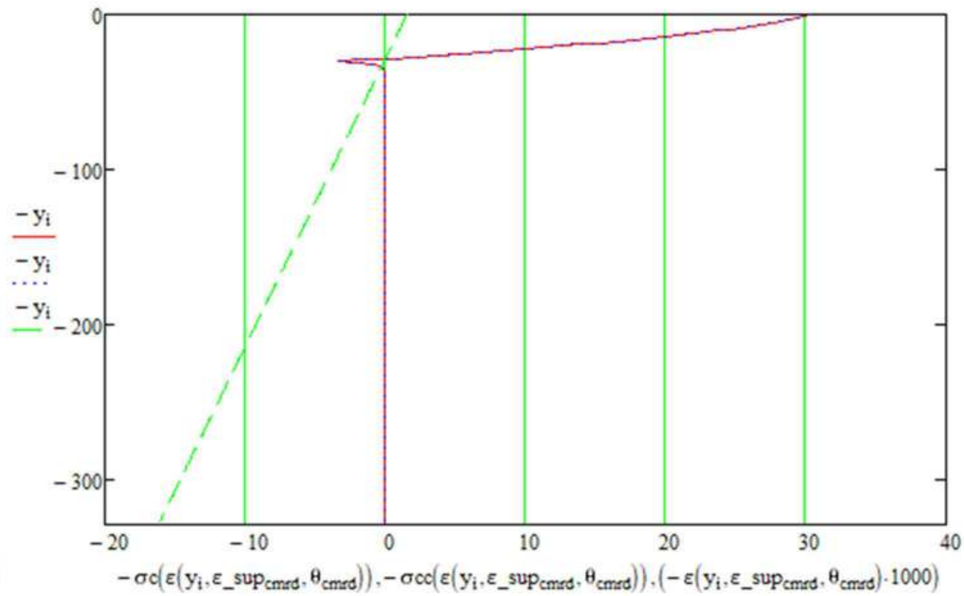
$$N(\varepsilon\_sup, \theta) := \sum_{i=1}^{Htot} \left(\sigma c\left(\varepsilon\left(y_i, \varepsilon\_sup, \theta\right)\right) \cdot b\left(y_i\right) \cdot \Delta y\right) + \sum_{j=1}^{jp} \left(\sigma p\left(\varepsilon\left(dp_j, \varepsilon\_sup, \theta\right) + \varepsilon pm_j\right) \cdot Ap_j\right) + \sum_{j=1}^{js} \left(\sigma s\left(\varepsilon\left(ds_j, \varepsilon\_sup, \theta\right)\right) \cdot As_j\right)$$

$$M(\varepsilon\_sup, \theta) := \sum_{i=1}^{Htot} \left[\sigma c\left(\varepsilon\left(y_i, \varepsilon\_sup, \theta\right)\right) \cdot b\left(y_i\right) \cdot \Delta y \cdot \left(y_i - yG\right)\right] + \sum_{j=1}^{jp} \left[\sigma p\left(\varepsilon\left(dp_j, \varepsilon\_sup, \theta\right) + \varepsilon pm_j\right) \cdot Ap_j \cdot \left(dp_j - yG\right)\right] + \sum_{j=1}^{js} \left[\sigma s\left(\varepsilon\left(ds_j, \varepsilon\_sup, \theta\right)\right) \cdot As_j \cdot \left(ds_j - yG\right)\right]$$

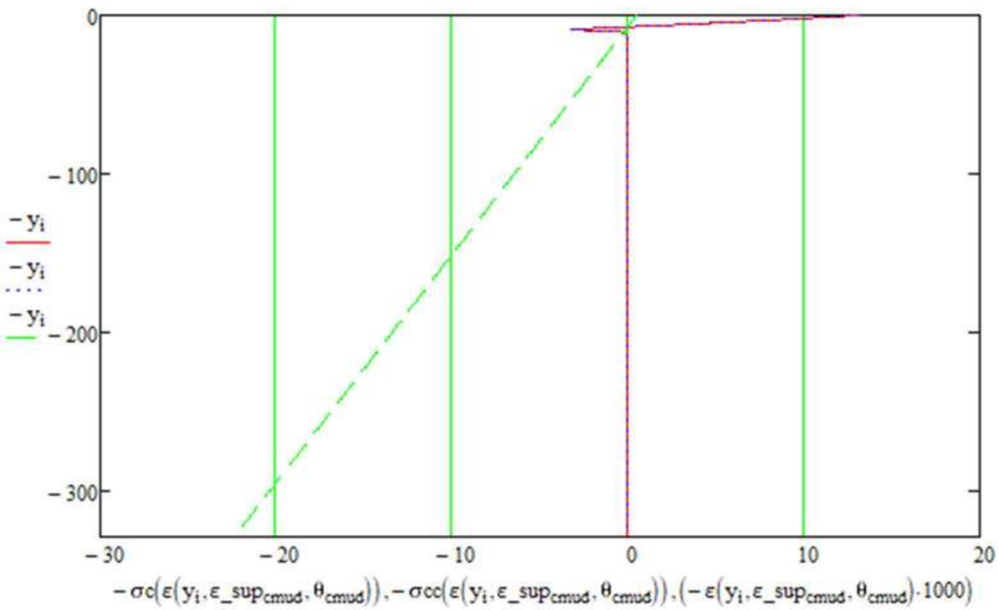
Design external axial load

$$NS := -0$$





Condition at resisting (peak) moment (stress and strain)



Condition at final computed step (stress and strain)

5.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformly distributed load q

$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformly distributed load q

$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformly distributed load q

$lpt = \blacksquare$

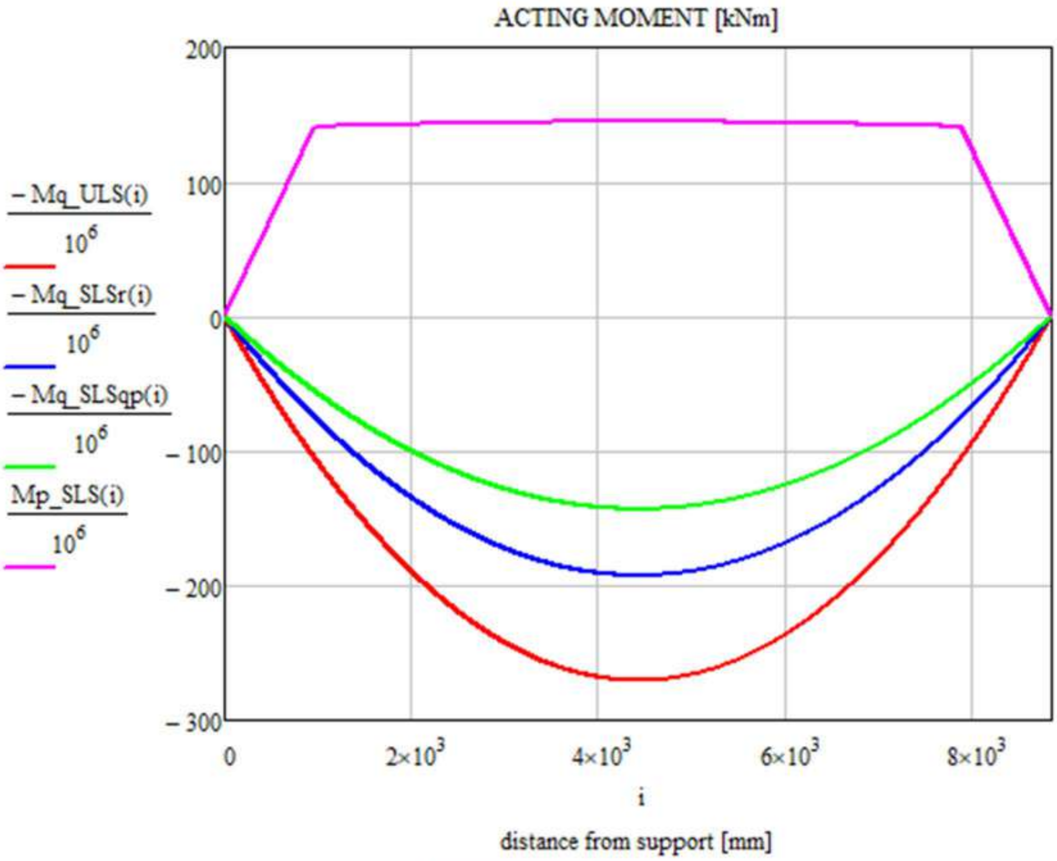
$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformly distributed load q

$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformly distributed load q

$Mp\_SLS(x) := \text{if}\left[x < lpt, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{x}{lpt}, \text{if}\left[x > L - lpt, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{-x + L}{lpt}, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid)\right]\right]$

contribution of prestressing equivalent load in SLS (without modification factors)

$i := 0..I$



5.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$v\_inf\_p(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(1 + \varphi(365\cdot50,23))$

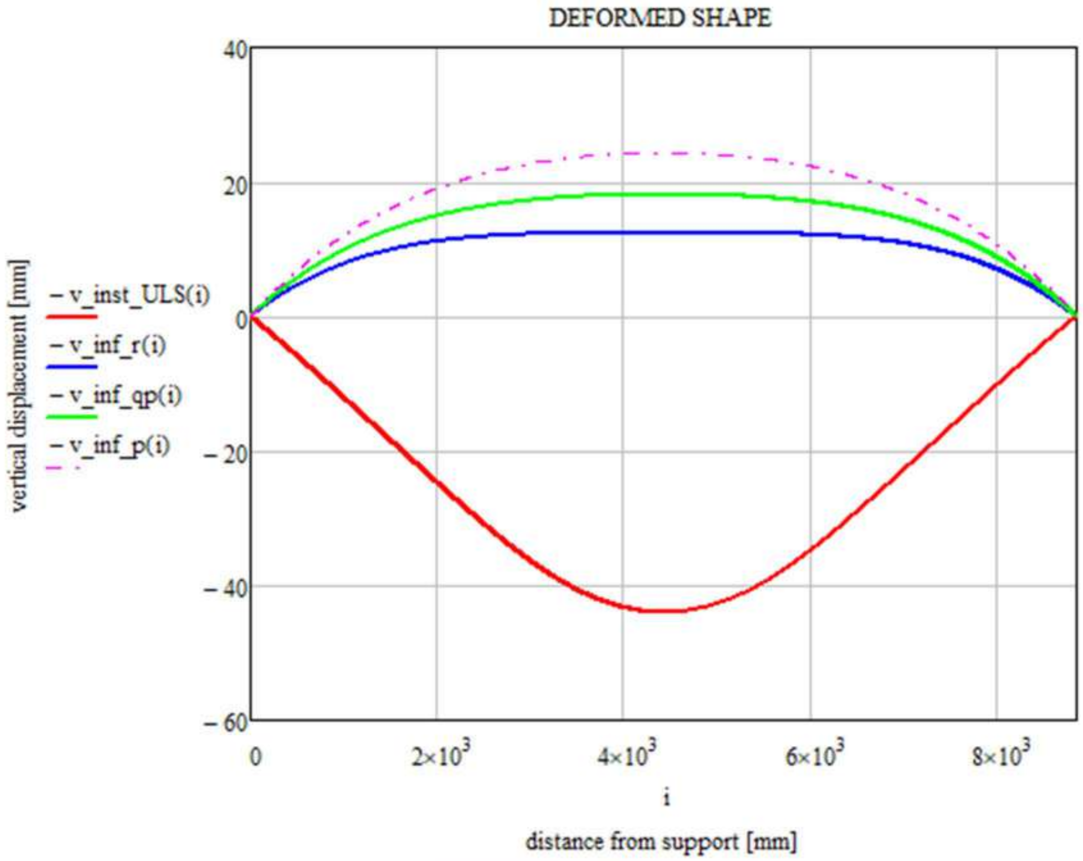
deflection profile at 50 years including creep for permanent load combination

$v\_inf\_qp(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot50,23) - \varphi(365\cdot50,91)) + v\_SLSqp(x)\cdot(1 + \varphi(365\cdot50,91))$

deflection profile at 50 years including creep for quasi permanent load combination

$v\_inf\_r(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot50,23) - \varphi(365\cdot50,91)) + v\_SLSqp(x)\cdot\varphi(365\cdot50,91) + v\_SLSr(x)$

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§7.4.3)

$v\_inf\_r\left(\frac{L}{2}\right) = -12.391$ < $\frac{L}{250} = 35.4$ CHECK maximum deflection

values calculated from differential equations above

$v\_inf\_p\left(\frac{L}{2}\right) = -24.348$ > $\frac{-L}{250} = -35.4$ CHECK maximum camber

$$cnom\_p := cmin\_p + \Delta c\_dev = 36.75 \quad \text{mm} \qquad cnom\_p + \frac{\phi p}{2} = 43.1$$

SLS STRESS CONTROL (§7.2)

$k1 := 0.6$ $\qquad rsup := 1.05$

$k2 := 0.45$ $\qquad$ prestressing modification coefficients

$k3 := 0.8$ $\qquad rinf := 0.95$

$k4 := 1$

$k5 := 0.75$

$$\sigma cpg1\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for selfweight loads only

$$\sigma cpg1\_top(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id}$$

elastic stress of top concrete chord for selfweight loads only

$$\sigma cpg1\_tops(x) := \frac{Es}{Ecm} \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(ds_1 - Yid\right)}{Ixo\_id} \right]$$

elastic stress of top series of mild steel for selfweight loads only

$$\sigma cpf\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSf(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for frequent load combination

$$\sigma cpr\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for rare load combination

$$\sigma cpr\_top(x) := \frac{-Np\_tot \cdot rinf}{Aid} + \frac{[Mq\_SLSr(x) - rinf \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id}$$

elastic stress of top concrete chord for rare load combination

$$\sigma cpr\_p(x) := \sigma pm(x,t) \cdot rsup + 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(dp_{jp} - Yid\right)}{Ixo\_id} \right]$$

creep stress of bottom prestressing steel for rare load combination

$$\sigma cpr\_s(x) := 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(ds_{js} - Yid\right)}{Ixo\_id} \right]$$

creep stress of bottom mild steel for rare load combination

$\sigma cpg1\_bot(lpt1) = -20.042$ > $k1\cdot(fcmj(2) + 8) = -13.578$ CHECK

> $k2\cdot fck = -20.25$ MPa not compulsory in environment XC

$\sigma cpg1\_top(lpt1) = 2.911$ < $fctmj(2) = 2.193$ MPa CHECK

if not the element is assumed to be cracked after transfer of prestressing

$\sigma cpg1\_tops(lpt1) = 6.461$ < $k3\cdot fsk = 400$ MPa

$\sigma cpf\_bot\left(\frac{L}{2}\right) = -5.711$ < $fctm = 3.795$ MPa CHECK

$\sigma cpr\_bot\left(\frac{L}{2}\right) = -1.929$ < $fctm = 3.795$ MPa

$\sigma cpr\_bot(lpt1) = -16.036$ > $k1\cdot fck = -27$ MPa CHECK

> $0.4\cdot fcm = -21.2$ MPa

$\sigma cpr\_top\left(\frac{L}{2}\right) = -4.482$ > $k1\cdot fck = -27$ CHECK

> $0.4\cdot fcm = -21.2$

$\sigma cpr\_p\left(\frac{L}{2}\right) = 1.219 \times 10^3$ < $k5\cdot fptk = 1.395 \times 10^3$ CHECK

$\sigma cpr\_s\left(\frac{L}{2}\right) = -31.91$ < $k3\cdot fsk = 400$ CHECK

SLS CRACK CONTROL (§7.3)

$c\_act := Htot - ds_{js} - 10 = 20$

$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1$

$wlim\_cal := 0.2$ mm

$w\_freq := 0$ < $wlim\_cal = 0.2$ CHECK

5.10 ULS checks

ULS BENDING-AXIAL CONTROL (§6.1)

$$Mrd = 311.571 \qquad > \qquad \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6} = 270.743 \qquad \text{CHECK}$$

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§6.2)

$$Vq\_ULS(x) := \left|(g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2} - x\right)\right|$$ shear distribution at Ultimate Limit State (ULS)

$d := Yp = 255$ mm — effective depth

$VEd := Vq\_ULS(d) = 1.153 \times 10^5$ N — maximum shear at effective depth from support

$bw := 320$ mm — web width

$z := 0.9\cdot d = 229.5$ mm — conventional resultant lever arm

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT (§6.2.2)

$$\rho l := \min\left(0.02, \frac{Ap\_tot + \sum_{j=1}^{js} As_j}{bw\cdot d}\right) = 0.014$$

reinforcement ratio

NOTE: the reinforcement ratio is assumed constant due to the introduction of additional support reinforcement which compensates the progressive anchorage of strands

$$\sigma cp(x) := \sigma pm(x,t)\cdot\frac{Ap\_tot}{Ac}$$ MPa — axial stress induced by prestressing

$\sigma cp(lpt2) = 2.821$ MPa after full transfer

$$kv := \min\left(1 + \sqrt{\frac{200}{d}}, 2\right) = 1.784$$

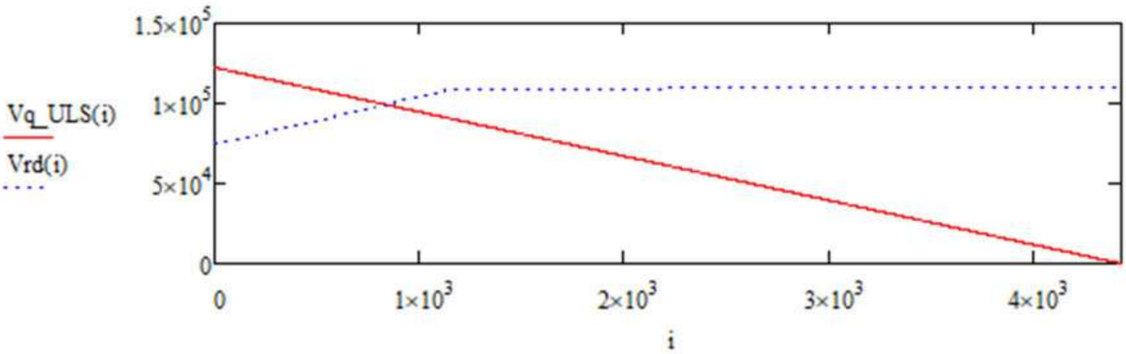
$k1v := 0.15$

$$Crdc := \frac{0.18}{\gamma cpcred} = 0.129$$

$$vmin := 0.035\cdot k^{\frac{3}{2}}\cdot(-fck)^{\frac{1}{2}} = 0.61$$ §6.3N

$$bw\cdot d\cdot\left[Crdc\cdot kv\cdot(100\cdot\rho l\cdot -fck)^{\frac{1}{3}} + k1v\cdot\sigma cp(lpt2)\right] =$$
$$(vmin + k1v\cdot\sigma cp(lpt2))\cdot bw\cdot d = 8.431 \times 10^4$$

$$VRdc(x) := \max\left[\left[Crdc\cdot kv\cdot(100\cdot\rho l\cdot -fck)^{\frac{1}{3}} + k1v\cdot\sigma cp(x)\right]\cdot bw\cdot d, (vmin + k1v\cdot\sigma cp(x))\cdot bw\cdot d\right]$$ §6.2.a+§6.2.b AFTER PRESTRESSING



MEMBERS PROVIDED WITH SHEAR REINFORCEMENT (§6.2.3)

$fywd := fsd = 454.545$ MPa — design yield stress of transverse reinforcement

$Asw := 4 \cdot \frac{6^2 \cdot \pi}{4} = 113.097$ mm2 — area of transverse reinforcement (pseudo-vertical stirrups)

$\theta v := atan\left(\frac{1}{2.5}\right) = 0.381$ rad — angle of inclination of concrete compressed strut

$s1 := 190$ mm — spacing of transverse reinforcement (constant throughout the member)

$Vrds := \frac{Asw}{s1} \cdot z \cdot fywd \cdot cot(\theta v) = 1.552 \times 10^5$ N > $Vq\_ULS(0) = 1.224 \times 10^5$ CHECK — shear resistance on steel side (§6.8)

$\nu 1 := 0.6 \cdot \left(1 - \frac{-fck}{250}\right) = 0.492$ §6.10

$\frac{Vq\_ULS(d)}{Vrds} = 0.743$

$\alpha cw(x) := if\left[\sigma cp(x) < 0.25 \cdot -fcd, 1 + \frac{\sigma cp(x)}{-fcd}, if\left[\sigma cp(x) > 0.5 \cdot -fcd, 2.5 \cdot \left(1 - \frac{\sigma cp(x)}{-fcd}\right), 1.25\right]\right]$ §6.11

$Vrdmax(x) := \alpha cw(x) \cdot bw \cdot z \cdot \nu 1 \cdot \frac{-fcd}{cot(\theta v) + tan(\theta v)}$ shear resistance on concrete side (§6.9)

$Vrdmax(0) = 4.35 \times 10^5$ N > $Vq\_ULS(0) = 1.224 \times 10^5$ CHECK

MOMENT DIAGRAM ACCOUNTING DUE TO SHEAR RESISTING MECHANISM (§9.2.1.3)

$\eta p2 := 1.2$

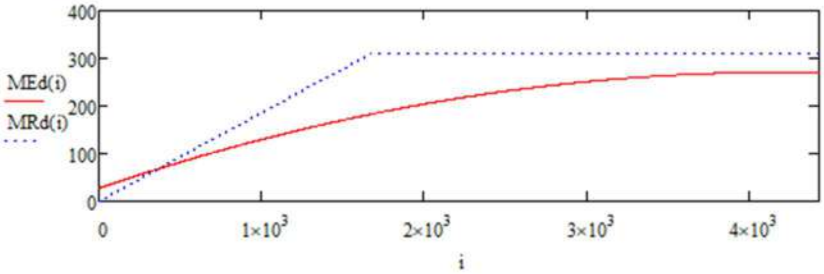
$fbpd := \eta p2 \cdot \eta 1 \cdot fctd = 2.277$ MPa given fck < 60 MPa

$$lbpd := lpt2 + \frac{\alpha 2 \cdot \left(fptd - \sigma pm\left(\frac{L}{2}, t\right)\right)}{fbpd} \cdot \phi p = 1.683 \times 10^{3}$$

$$MRd(x) := if\left[x < lpt2, Mrd \cdot \frac{\sigma pm(lpt2, 365 \cdot 50)}{fptd} \cdot \frac{x}{lpt2}, if\left[x > lbpd, Mrd, Mrd \cdot \frac{\sigma pm(lpt2, 365 \cdot 50)}{fptd} + \frac{(x - lpt2)}{(lbpd - lpt2)} \cdot \left(Mrd - Mrd \cdot \frac{\sigma pm(lpt2, 365 \cdot 50)}{fptd}\right)\right]\right]$$

$$al := z \cdot \left(\frac{\cot(\theta v)}{2}\right) = 286.875$$

$$MEd(x) := if\left(x > \frac{L}{2} - al, \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^{6}}, \frac{Mq\_ULS(x + round(z))}{10^{6}}\right)$$



MINIMUM REINFORCEMENT

$bt := b(Htot) = 320$ mm §9.1N §9.2.1.1(3)

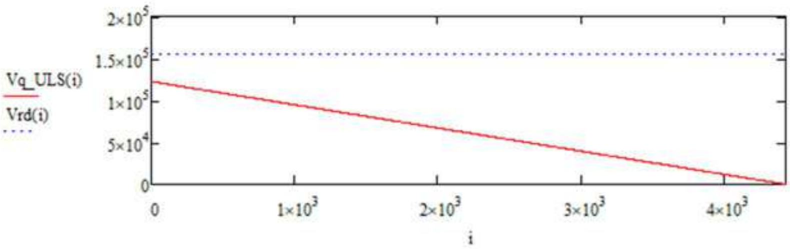
$Asmin := max\left(0.26 \cdot \frac{fctm}{fsk} \cdot bt \cdot d, 0.0013 \cdot bt \cdot d\right) = 161.048$ mm2 > $\rho l \cdot Ac = 4.294 \times 10^{3}$ mm2 CHECK < $0.04 \cdot Ac = 1.23 \times 10^{4}$ CHECK

for longitudinal reinforcement

$s2 := 190$ mm < $d \cdot 0.75 = 191.25$ mm CHECK for shear reinforcement §9.6N

$\rho w\_min := \frac{Asw}{s2 \cdot bw} = 1.86 \times 10^{-3}$ > $0.08 \cdot \frac{\sqrt{-fck}}{fsk} = 1.073 \times 10^{-3}$ CHECK §9.5N

$Vrd(x) := max(Vrds, Vrd(x))$



$s1 = 190$

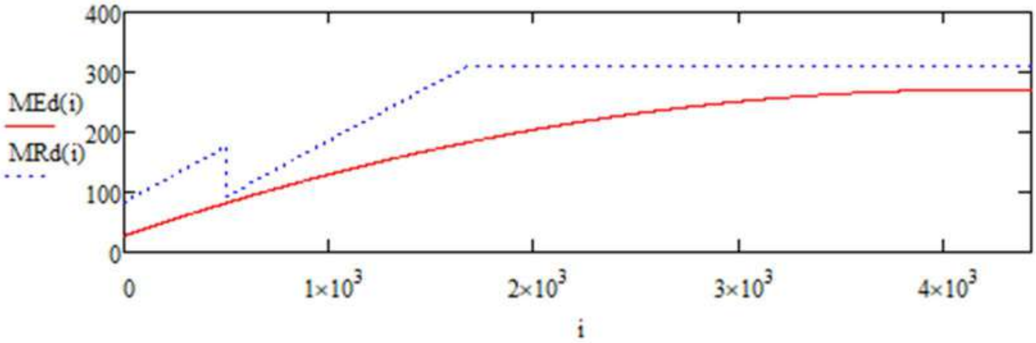
$s2 = 190$

CHECK OF SUPPORT MILD REBARS (§9.2.1.4(1))

$0.25 \cdot Mrd = 77.893$ Nmm $<$ $\left(4 \cdot \pi \cdot \frac{16^2}{4}\right) \cdot fsd \cdot 0.9 \cdot \frac{d}{10^6} = 83.898$ CHECK

area of support mild steel

$$MRd(x) := \text{if}\left[x < 500, 0.9 \cdot d \cdot \frac{fywd}{10^6} \cdot \left(4 \cdot \pi \cdot \frac{16^2}{4}\right) + MRd(x), MRd(x)\right]$$



ANCHORAGE (§8.4)

$\eta 1 = 1$

$\eta 2 := 1$

$fbd := 2.25 \cdot \eta 1 \cdot \eta 2 \cdot fctd = 4.27$ MPa §8.2

$lbrqd(\phi) := \frac{\phi}{4} \cdot \frac{fsd}{fbd}$ §8.3

$\alpha 1b := 1$

$\alpha 2b := 1$

$\alpha 3b := 1$

$\alpha 4b := 1$

$\alpha 5b := 1$

$lbd(\phi) := \alpha 1b \cdot \alpha 2b \cdot \alpha 3b \cdot \alpha 4b \cdot \alpha 5b \cdot lbrqd(\phi)$ §8.4

2.155

SHEAR AT THE WEB-FLANGE INTERFACE (§6.2.4)

$\eta 1 = 1$

$\eta 2 := 1$

$hf := 354$

$\Delta x := \frac{L}{4} = 2.212 \times 10^{3}$ $\quad$ $Mq\_ULS\left(\frac{L}{4}\right) = 2.031 \times 10^{8}$ $\quad$ $M_9 = 2.001 \times 10^{8}$

$$\Delta Fd := \sum_{i=1}^{80} \left(\sigma c\left(\varepsilon\left(y_i, \varepsilon\_sup_9, \theta_9\right)\right) \cdot 2400 \cdot \Delta y\right) + \sum_{j=1}^{1} \left(\sigma s\left(\varepsilon\left(ds_j, \varepsilon\_sup_9, \theta_9\right)\right) \cdot As_j\right) = -8.002 \times 10^{5}$$

$\theta f := 0.462$

$vEd := \frac{-\Delta Fd}{hf \cdot \Delta x} = 1.022$ $\quad < \quad$ $\nu 1 \cdot -fcd \cdot \sin(\theta f) \cdot \cos(\theta f) = 6.31$ $\quad$ CHECK $\quad$ compressed strut

$< \quad 0.4 \cdot fctd = 0.759$ $\quad$ CHECK

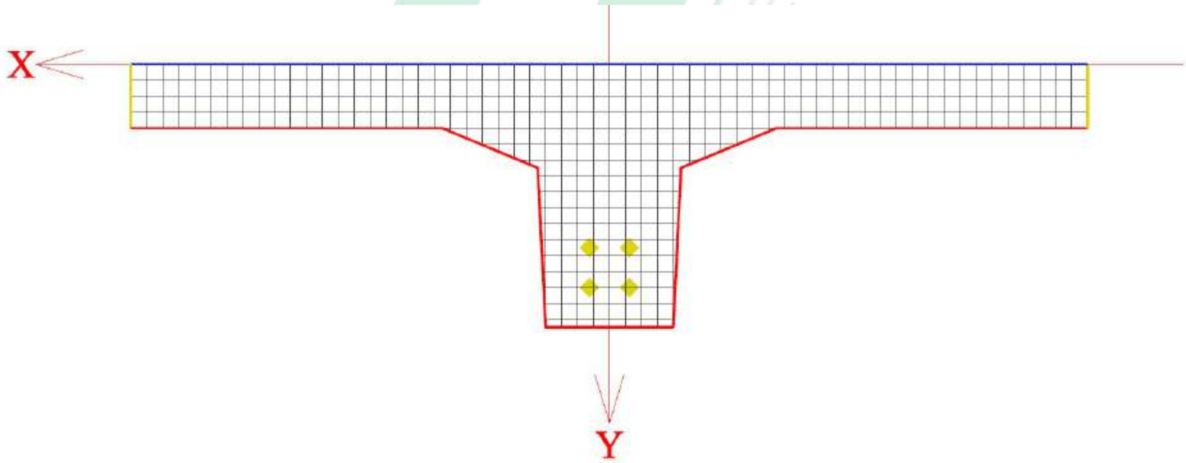
if not reinforcement is required

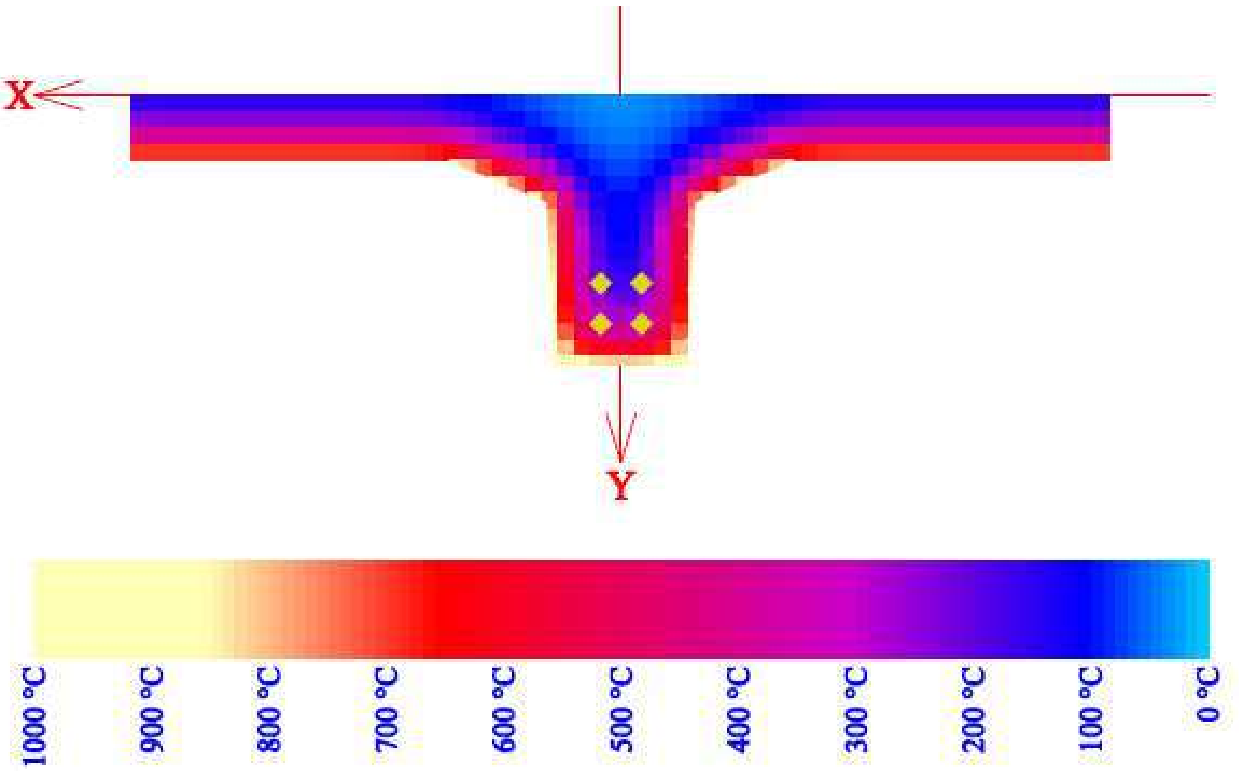
$Asf := 2 \cdot A(6) + A(6) = 84.823$ mm2 $\quad$ flange transverse reinforcement

$sf := 200$ mm $\quad$ spacing of additional flange reinforcement

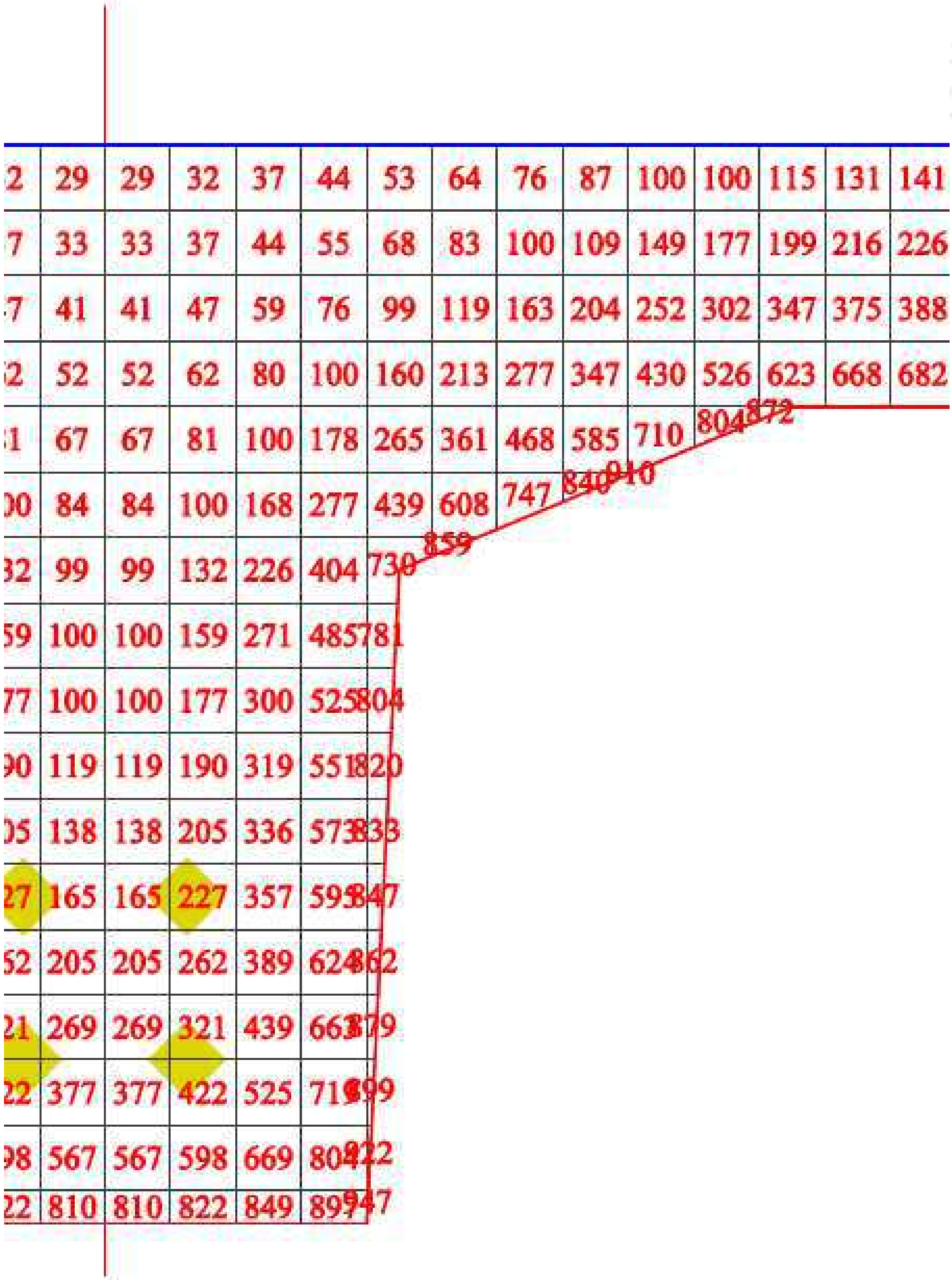
$Asf \cdot \frac{fsd}{sf} = 192.78$ mm2/mm $\quad > \quad$ $vEd \cdot \frac{hf}{\cot(\theta f)} = 180.087$ $\quad$ CHECK

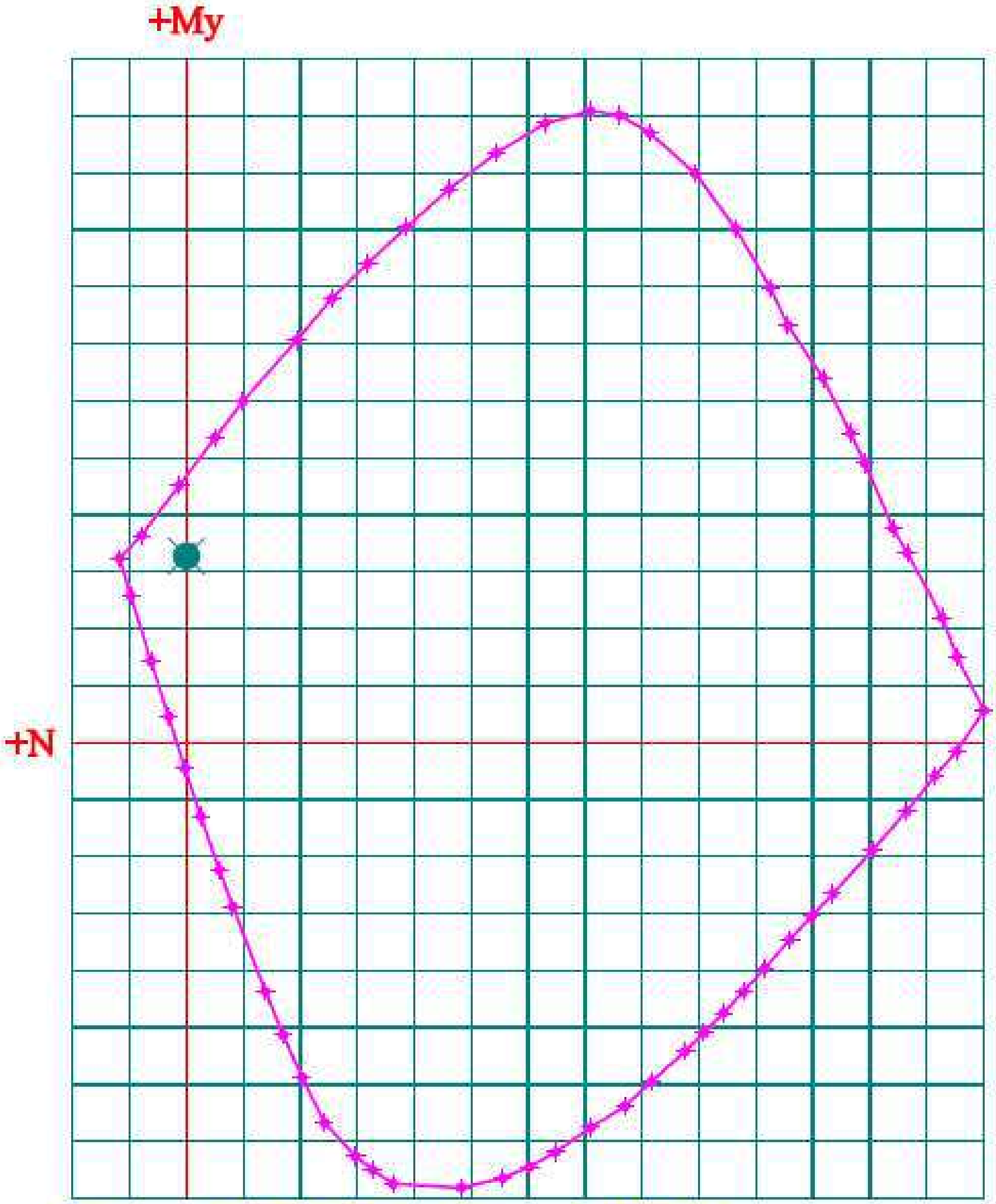
5.11 Fire checks





T(°C)


29 29 32 37 44 53 64 76 87 100 100 115 131 141
33 33 37 44 55 68 83 100 109 149 177 199 216 226
41 41 47 59 76 99 119 163 204 252 302 347 375 388
52 52 62 80 100 160 213 277 347 430 526 623 668 682
67 67 81 100 178 265 361 468 585 710 804 872
84 84 100 168 277 439 608 747 840 910
99 99 132 226 404 730 859
100 100 159 271 485 781
100 100 177 300 525 804
119 119 190 319 551 820
138 138 205 336 573 833
165 165 227 357 595 847
205 205 262 389 624 862
269 269 321 439 663 879
377 377 422 525 719 899
567 567 598 669 804 922
810 810 822 849 897 947

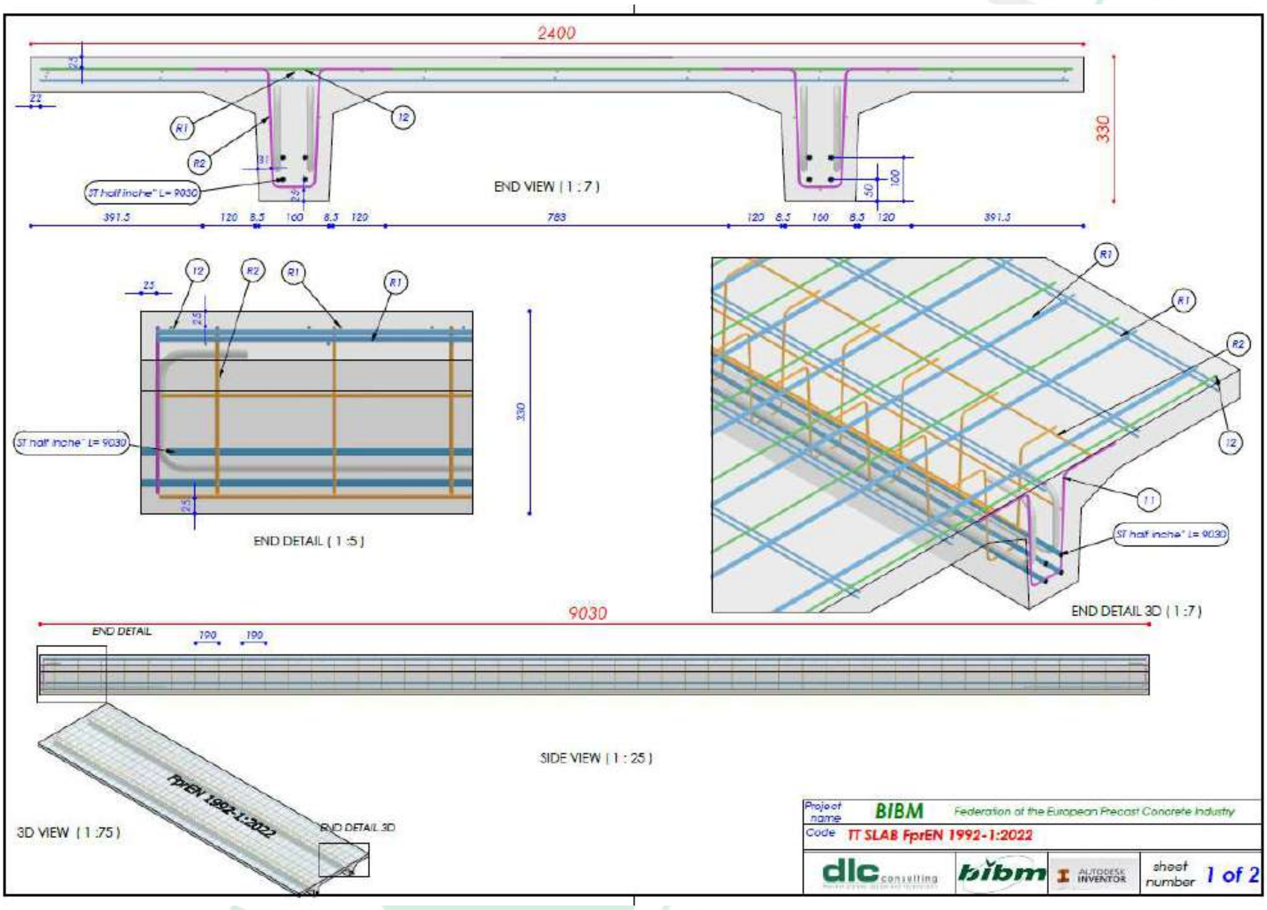


N: 1 sp = 380.00 kN;   M: 1 sp = 22.00 kN·m

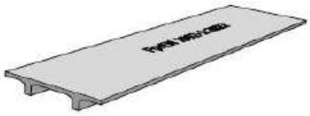
Nz = 0.00 kN
My+ = 103.02 kN·m
My- = -11.79 kN·m

6 TT element – FprEN1992-1:2022

6.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_	Ø_longitudinal	pattern_T	Ø_transverse	pattern_L
	11	4	208	832	6 mm				
	12	24	522	12528	6 mm				
	21	8	2026	16208	16 mm				
	Total mass rebars [kg]			29,57		Incidence kg/m³	11,33		
	R1	2	41610	83220		6 mm	200 mm	6 mm	300 mm
	R2	2	18683	37366		6 mm	200 mm	6 mm	190 mm
Total mass welded-wire-meshes [kg]				120,59		Incidence kg/m³	46,20		
	ST half inche' L= 9030	8	6599	52792	12,7 mm				
	Total mass strands [kg]			52,792		Incidence kg/m³	20,23		
	Total mass of steel [kg]			202,95		Total concrete volume [m	2,61		



Project name	BIBM	Federation of the European Precast Concrete Industry
Code	TT SLAB FprEN 1992-1:2022	
dlc consulting	bibm AUTODESK INVENTOR	sheet number 2 of 2

6.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0 \ \ 79.99 \ \ 80 \ \ 130 \ \ 330)^T$

$Htot := \max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := (2400 \ \ 2400 \ \ 1566 \ \ 354 \ \ 320)^T$

$r\_circ := 0$ radius of central void pipe

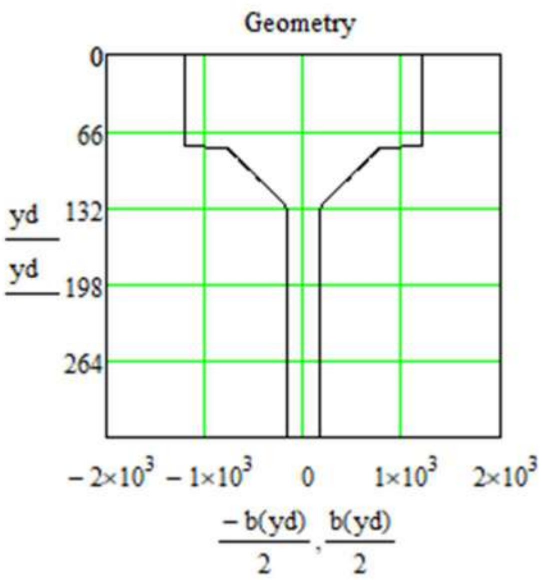
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$b\_lin(y) := \mathrm{linterp}(y\_tr, b\_tr, y)$

$b\_circ(y) := \mathrm{linterp}(y\_tr, b\_tr, y) - x\_circ(y)$

$$b(y) := \mathrm{if}\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$yd := 0..Htot$



condensed 1D geometry plot

$u := 2400 + 1800 + 320 + 300 \cdot 4 = 5.72 \times 10^3$ mm exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (25 \;\; 300)^T$$

Area of reinforcement at each depth

$$As := (12 \cdot A(6) \;\; 2 \cdot A(6))^T$$

$$js := rows(As) \qquad js = 2$$

$$dsmax := max(ds) \qquad dsmax = 300$$

$$As\_tot := \sum_{j=1}^{js} As_j = 395.841$$

Prestressing reinforcement

nominal strand diameter

12.7	mm	0.5'
15.24	mm	0.6'

Area of a single strand:

$Ap0 := 93$ $\qquad \phi p := 12.7$ mm $\qquad$ nominal strand diameter

Depth of prestressing strands from upper chord:

$dp := (180 \ \ 230 \ \ 280)^T$

Area of strands at each depth:

$Ap := (0 \cdot Ap0 \ \ 4 \cdot Ap0 \ \ 4 \cdot Ap0)^T$

$\sigma p0 := 1400$ MPa

$\sigma prec := (0.4 \cdot \sigma p0 \ \ \sigma p0 \ \ \sigma p0)^T$ $\qquad$ initial prestressing

$perdite := 0 \cdot (1 \ \ 1 \ \ 1)^T$ $\qquad$ in percentual **% (losses are introduced later)**

$jp := rows(Ap)$ $\qquad jp = 3$

$k := 1..jp$

$$\sigma o_k := \sigma prec_k \cdot \left[\frac{(100 - perdite_k)}{100}\right] \qquad \sigma o = \begin{pmatrix} 560 \\ 1.4 \times 10^3 \\ 1.4 \times 10^3 \end{pmatrix}$$

$$Ap\_tot := \sum_{k=1}^{jp} Ap_k \qquad Ap\_tot = 744$$

$ypmax := max(dp)$ $\qquad ypmax = 280$

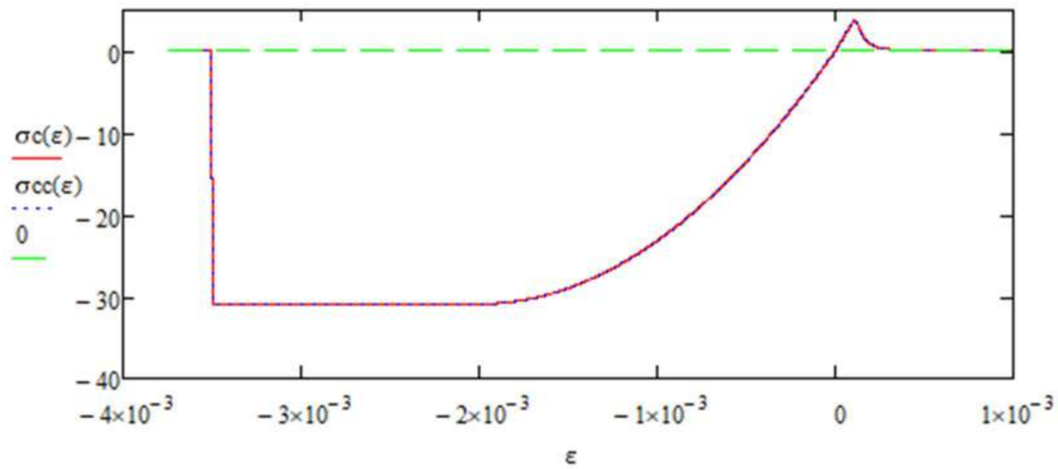
$$Np\_tot := \sum_{k=1}^{jp} \left(\left(Ap_k \cdot \sigma o_k\right)\right) \qquad Np\_tot = 1.042 \times 10^6 \text{ N}$$

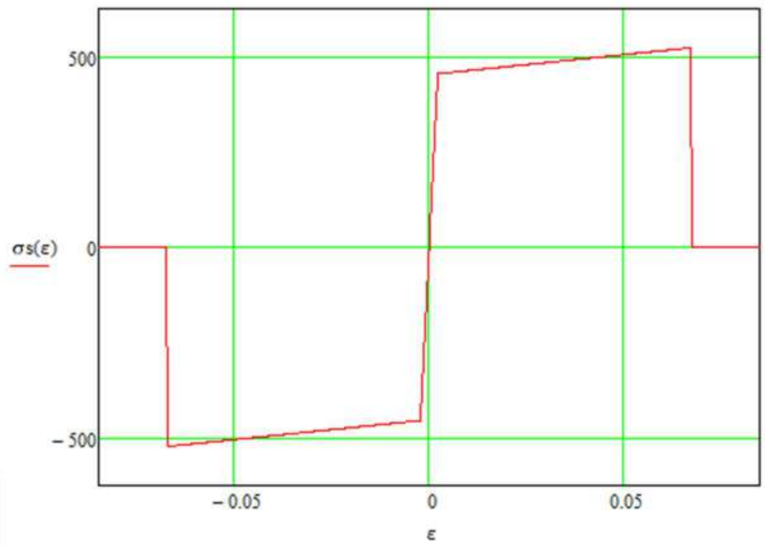
total prestressing initial force

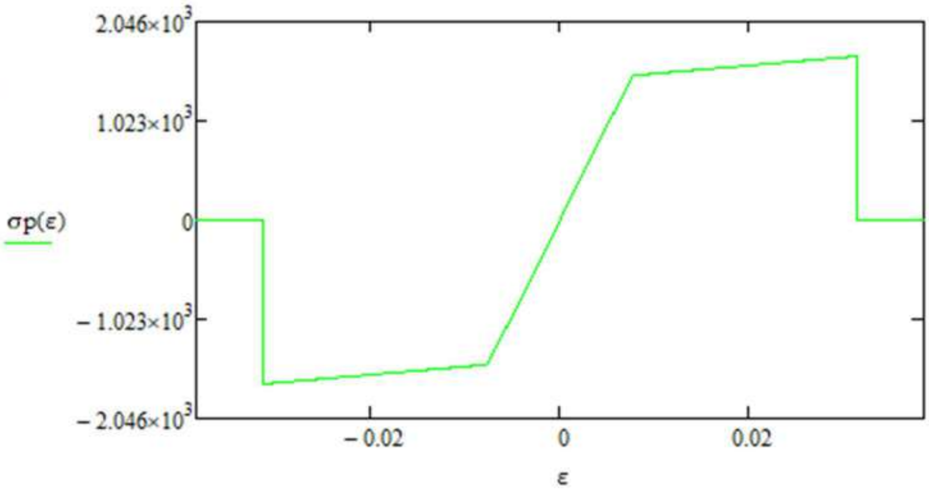
$$Yp := \frac{\sum_{k=1}^{jp} \left(dp_k \cdot Ap_k \cdot \sigma o_k\right)}{\sum_{k=1}^{jp} \left(Ap_k \cdot \sigma o_k\right)} = 255 \text{ mm}$$

centre of gravity of prestressing

6.3 Material constitutive laws employed in the calculation







6.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 3.074 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 1.288 \times 10^{-3}$$ geometric ratio for longitudinal mild reinforcement

$$\rho p := \frac{Ap\_tot}{Ac} = 2.42 \times 10^{-3}$$ geometric ratio for longitudinal prestressing tendons

$$\rho tot := \frac{As\_tot + Ap\_tot}{Ac} = 3.708 \times 10^{-3}$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y) \cdot y\,dy \qquad Syc = 2.785 \times 10^7$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 90.619$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y) \cdot (y - yG)^2\,dy \qquad Ixo\_cls = 2.109 \times 10^9$$

Global area of all prestressing reinforcement

$$Area\_tr := \left| \begin{array}{l} s \leftarrow 0 \\ \text{for } x \in 1..jp \\ \quad s \leftarrow Ap_x + s \end{array} \right. \qquad Area\_tr = 744$$

First moment of the area referred to prestressing reinforcement only

$$Sxp := \sum_{i=1}^{jp} \left(Ap_i \cdot dp_i\right) \qquad Sxp = 1.897 \times 10^5$$

Centre of gravity of prestressing

$$Yp := \frac{Sxp}{Area\_tr} \qquad Yp = 255$$

Idealisation coefficients (elastic)

$$np := \frac{Ep}{Ecm} \qquad np = 5.465$$

$$ns := \frac{Es}{Ecm} \qquad ns = 5.605$$

Area of ideal cross-section

$$Aid := Ac + (np - 1)\cdot\sum_{j=1}^{jp} Ap_j + (ns - 1)\cdot\sum_{j=1}^{js} As_j \qquad Aid = 3.125 \times 10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac\cdot yG + (np - 1)\cdot(Area\_tr\cdot Yp) + (ns - 1)\cdot\sum_{j=1}^{js}\left(As_j\cdot ds_j\right) \qquad Sxid = 2.882 \times 10^7$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 92.213$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y)\cdot(y - Yid)^2\,dy - \sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right] - \sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the prestressing reinforcement area

$$Ixoidprec := np\cdot\sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right]$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns\cdot\sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidprec + Ixoidlenta \qquad Ixo\_id = 2.219 \times 10^9 \ \text{mm^4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.052$$

6.5 Loads

LOADS

$interaxis := 2400$ mm

$g1 := Ac \cdot 0.000025 = 7.685$ kN/m — dead load from self-weight

$g2 := 2 \cdot \frac{interaxis}{1000} = 4.8$ kN/m — nonstructural dead load

$q := 3 \cdot \frac{interaxis}{1000} = 7.2$ kN/m — live load

$L := 8850$ mm — calculation length (span between supports)

$\psi 2 := 0.3$ — non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ — non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ — SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ — SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ — SLS bending moment distribution from live load

6.6 Prestressing transfer and time-dependent behaviour

TRANSFER OF PRESTRESS (§13.5.3)

$\alpha 1 := 1$ — gradual release of prestressing

$\alpha 2 := 0.26$ — for 7-wire strands

$\sigma pm0 := \sigma p0 = 1.4 \times 10^3$ MPa

$\eta 1 := 1$ — in favourable position

$lpt := \frac{\gamma c}{1.5} \cdot \frac{\alpha 1 \cdot \alpha 2 \cdot \sigma pm0}{\eta 1 \cdot \sqrt{(-fcmj(2) - 8)}} \cdot \phi p = 906.996$ mm — basic value of the transmission length following §(13.4)

$lpt1 := 0.8 lpt = 725.597$ mm — lower-bound transfer length following §(13.6)

$lpt2 := 1.2 \cdot lpt = 1.088 \times 10^3$ mm — upper-bound transfer length following §(13.7)

Prestress losses

$$hn := 2 \cdot \frac{Ac}{u} = 107.477 \quad \text{mm}$$

$$A := 0.79 + \frac{(hn - 200)}{(500 - 200)} \cdot (0.75 - 0.79) = 0.802$$

$$\varepsilon cs := \frac{0.65}{1000} = 6.5 \times 10^{-4}$$ shrinkage strain assumed as a result of laboratory tests on the specific concrete mix employed

$\rho 1000 := 0.025$ for class 2 (low-relaxation) tendons

$k\rho := 0.16$

$t := 50 \cdot 365 = 1.825 \times 10^{4}$ days Life span

$$\sigma cpQP2(x) := \frac{-Np\_tot}{Aid} + \frac{[Mq\_SLSg1(x) - Np\_tot \cdot (Yp - Yid)] \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP2\left(\frac{L}{2}\right) = -10.254$$

stress in quasi-permanent load combination at 2 days
(conventional equivalent time for prestressing release)

$$\sigma cpQP23(x) := \frac{Mq\_SLSg2(x) \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP23\left(\frac{L}{2}\right) = 3.448$$

stress in quasi-permanent load combination at 23 days
(conventional time for assemblage of the structure on site)

$$\sigma cpQP91(x) := \frac{Mq\_SLSq(x) \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP91\left(\frac{L}{2}\right) = 1.552$$

stress in quasi-permanent load combination at 91 days
(conventional time for enter in use of the structure)

$$\Delta \sigma pr(x,t) := \left[\sigma p0 + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x))\right] \cdot \rho 1000 \cdot \left(\frac{24 \cdot t}{1000}\right)^{k\rho}$$

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$$RH := 50$$

$$t0\_adj(t0) := t0$$

$$\beta bc\_fcm := \frac{1.8}{(-fcm)^{0.7}} = 0.112$$

$$\beta bc\_t\_t0(t,t0) := \ln\left[\left(\frac{30}{t0\_adj(t0)} + 0.035\right)^2 \cdot (t - t0) + 1\right]$$

$$\beta dc\_fcm := \frac{412}{(-fcm)^{1.4}} = 1.588$$

$$\beta dc\_RH := \frac{1 - \frac{RH}{100}}{\sqrt[3]{0.1 \cdot \frac{hn}{100}}} = 1.052$$

$$\beta dc\_t0(t0) := \frac{1}{0.1 + t0\_adj(t0)^{0.2}}$$

$$\gamma(t0) := \frac{1}{2.3 + \frac{3.5}{\sqrt{t0\_adj(t0)}}}$$

$$\alpha cm := \left(\frac{35}{-fcm}\right)^{0.5} = 0.813$$

$$\beta h := \min(1.5 \cdot hn + 250 \cdot \alpha cm, 1500 \cdot \alpha cm) = 364.374$$

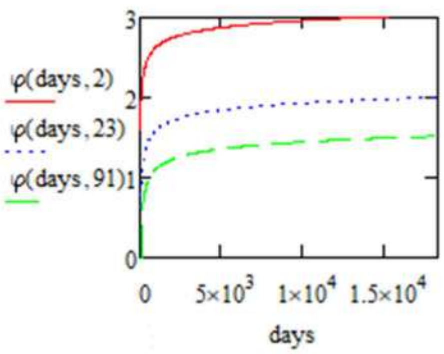
$$\beta dc\_t\_t0(t,t0) := \left[\frac{(t - t0)}{\beta h + (t - t0)}\right]^{\gamma(t0)}$$

$$\varphi dc(t,t0) := \beta dc\_fcm \cdot \beta dc\_RH \cdot \beta dc\_t0(t0) \cdot \beta dc\_t\_t0(t,t0)$$

$$\varphi bc(t,t0) := \beta bc\_fcm \cdot \beta bc\_t\_t0(t,t0)$$

$$\varphi(t,t0) := \varphi bc(t,t0) + \varphi dc(t,t0)$$

$$\varphi(t,2) = 3.034$$



TIME-DEPENDENT LOSSES OF PRESTRESS (§7.6.4)

$$\Delta\sigma p\_csr(x,t) := \frac{-\varepsilon cs \cdot Ep - 0.8\cdot\Delta\sigma pr(x,t) + \frac{Ep}{Ecm}\cdot\left(\sigma cpQP2(x)\cdot\varphi(t,2) + \sigma cpQP23(x)\cdot\varphi(t,23) + \sigma cpQP91(x)\cdot\varphi(t,91)\right)}{1 + \frac{Ep}{Ecm}\cdot\frac{Ap\_tot}{Ac}\cdot\left[1 + \frac{Ac}{Ixoidcls}\cdot(Yp - Yid)^2\right]\cdot\left(1 + 0.8\cdot\frac{\varphi(t,2)\cdot\sigma cpQP2(x) + \varphi(t,23)\cdot\sigma cpQP23(x) + \varphi(t,91)\cdot\sigma cpQP91(x)}{\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)}\right)}$$

prestress losses following §(7.35)

NOTE: a weighed creep coefficient was considered accounting for the 3 load phases previously introduced

$$\sigma pm(x,t) := \sigma p0 - \frac{Ep}{Ecm}\cdot\left(\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)\right) + \Delta\sigma p\_csr(x,t)$$ prestress considering immediate and delayed losses

$$\frac{\sigma pm\left(\frac{L}{2}, 365\cdot 50\right)}{\sigma p0} = 0.843$$ expected residual prestress ratio after 50 years of life with respect to initial

$$\varepsilon pm := \frac{\sigma pm\left(\frac{L}{2}, 365\cdot 50\right)}{\sigma p0}\cdot\varepsilon p0$$ expected residual strain after 50 years of life with respect to initial

$$\sigma pm\left(\frac{L}{2}, 365\cdot 50\right)\cdot Ap\_tot = 8.778\times 10^5$$ N residual prestress force after 50 years of life

$Np\_tot = ■$ N initial prestress force

6.7 Non-linear moment-curvature diagram

Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

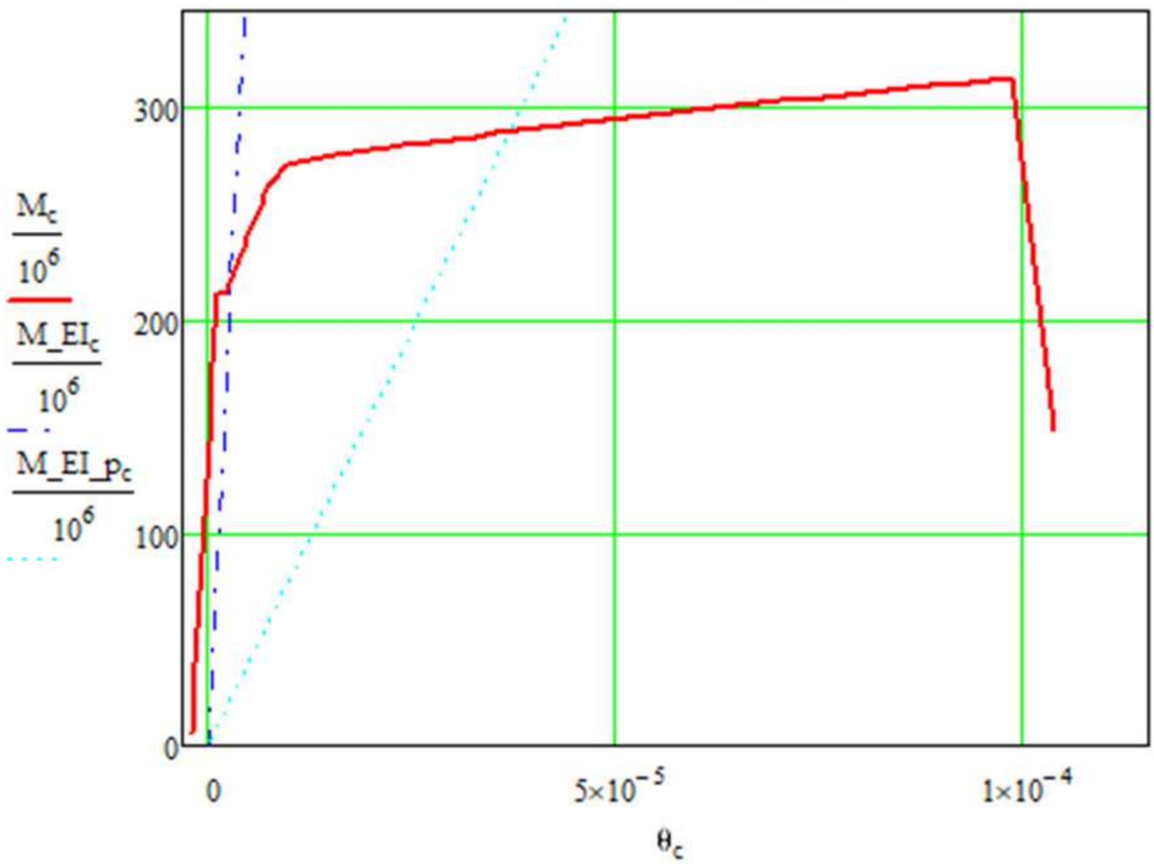
$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot}\left(\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\right) + \sum_{j=1}^{jp}\left(\sigma p\left(\varepsilon\left(dp_j,\varepsilon\_sup,\theta\right) + \varepsilon pm_j\right)\cdot Ap_j\right) + \sum_{j=1}^{js}\left(\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\right)$$

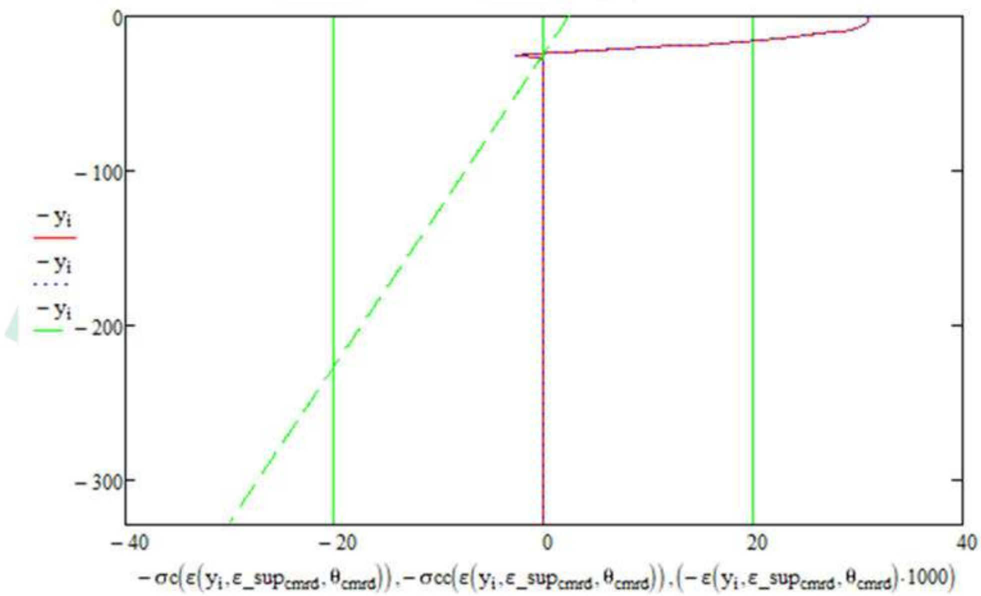
$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot}\left[\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\cdot\left(y_i - yG\right)\right] + \sum_{j=1}^{jp}\left[\sigma p\left(\varepsilon\left(dp_j,\varepsilon\_sup,\theta\right) + \varepsilon pm_j\right)\cdot Ap_j\cdot\left(dp_j - yG\right)\right] + \sum_{j=1}^{js}\left[\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\cdot\left(ds_j - yG\right)\right]$$

Design external axial load

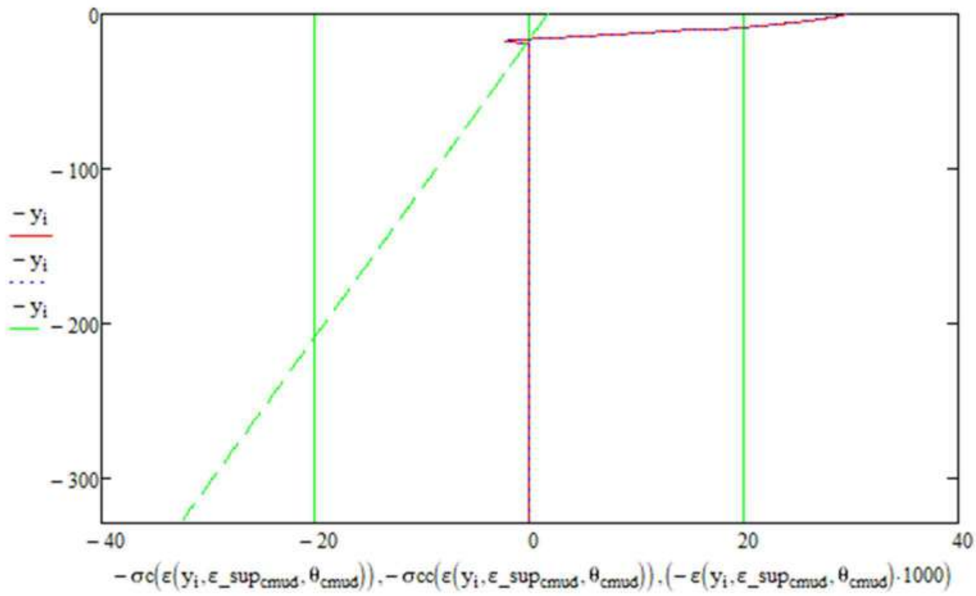
$NS := -0$

Moment-curvature [kNm]





Condition at resisting (peak) moment (stress and strain)



Condition at final computed step
(stress and strain)

6.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformly distributed load q

$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformly distributed load q

$lpt = 906.996$

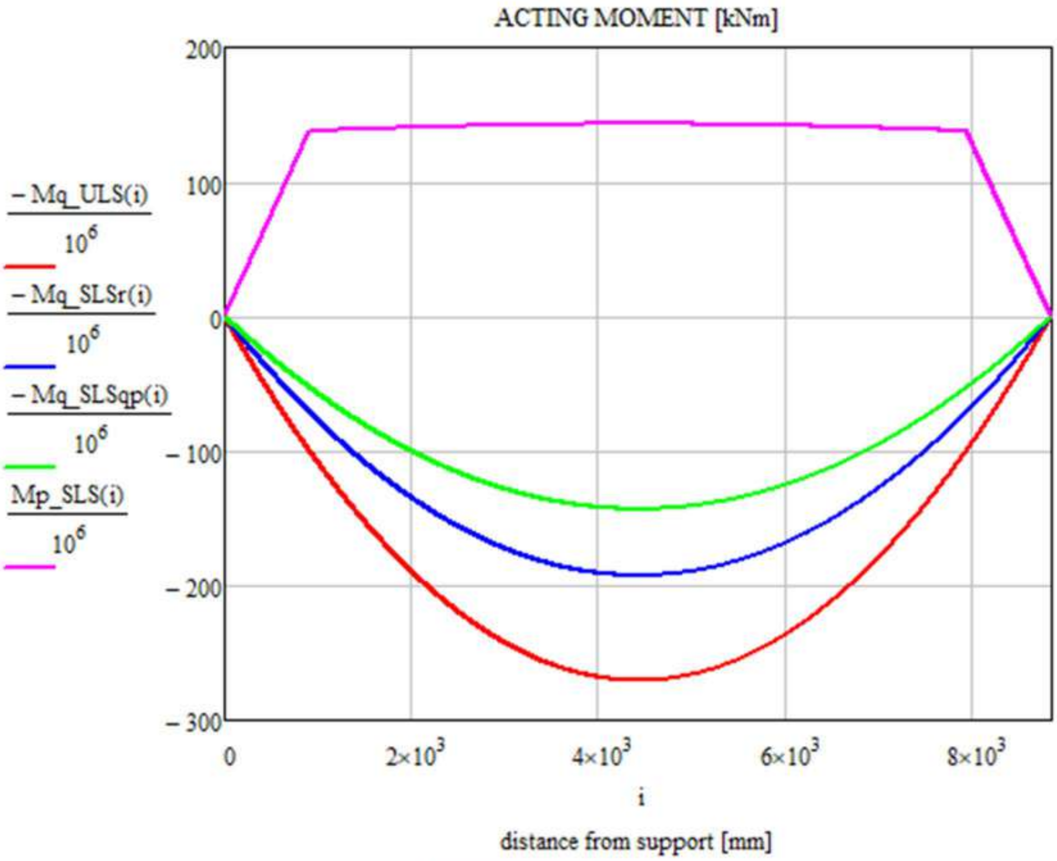
$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformly distributed load q

$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformly distributed load q

$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformly distributed load q

$$Mp\_SLS(x) := \mathrm{if}\left[x < lpt, \sigma pm(x,t) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{x}{lpt}, \mathrm{if}\left[x > L - lpt, \sigma pm(x,t) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{-x + L}{lpt}, \sigma pm(x,t) \cdot Ap\_tot \cdot (Yp - Yid)\right]\right]$$

contribution of prestressing equivalent load in SLS (without modification factors)



6.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$$v\_inf\_p(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2)-\varphi(t,23)) + v\_SLSg2(x)\cdot(1+\varphi(t,23))}{1.05}$$

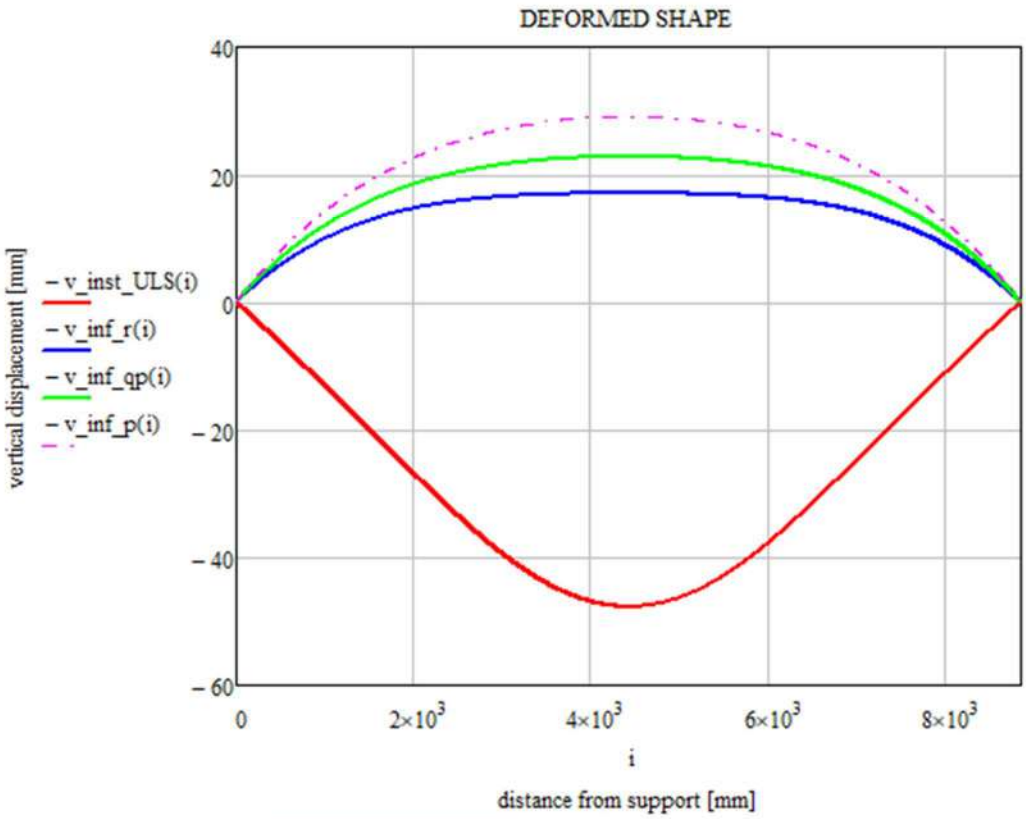
deflection profile at 50 years including creep for permanent load combination

$$v\_inf\_qp(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2)-\varphi(t,23)) + v\_SLSg2(x)\cdot(\varphi(t,23)-\varphi(t,91)) + v\_SLSqp(x)\cdot(1+\varphi(t,91))}{1.05}$$

deflection profile at 50 years including creep for quasi permanent load combination

$$v\_inf\_r(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2)-\varphi(t,23)) + v\_SLSg2(x)\cdot(\varphi(t,23)-\varphi(t,91)) + v\_SLSqp(x)\cdot\varphi(t,91) + v\_SLSr(x)}{1.05}$$

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§9.3.4)

$v\_inf\_r\left(\frac{L}{2}\right) = -17.037$ < $\frac{L}{250} = 35.4$ CHECK   maximum deflection

values calculated from differential equations above

$v\_inf\_p\left(\frac{L}{2}\right) = -29.1$ > $\frac{-L}{250} = -35.4$ CHECK   maximum camber

SLS STRESS CONTROL (§9.2.1)

$k1 := 0.6$ $\quad$ $rsup := 1.05$

$k2 := 0.45$ $\quad$ prestressing modification coefficients

$k3 := 0.8$ $\quad$ $rinf := 0.95$ $\quad$ $Np\_tot = 1.042 \times 10^{6}$

$k4 := 1$

$k5 := 0.8$ $\quad$ 0.75 in EN1992-1-1:2002

NOTE: the denomination of the allowable stress coefficients following k factors was kept similar to that of EN1992-1-1:2002

$$\sigma cpg1\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpg1\_bot(lpt1) = \cdot$$

elastic stress of bottom concrete chord for selfweight loads only

$$\sigma cpg1\_top(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id} \qquad \sigma cpg1\_top(lpt1) =$$

elastic stress of top concrete chord for selfweight loads only

$$\sigma cpg1\_tops(x) := \frac{Es}{Ecm} \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(ds_{1} - Yid\right)}{Ixo\_id} \right] \qquad \sigma cpg1\_tops(lpt1) =$$

elastic stress of top series of mild steel for selfweight loads only

$$\sigma cpf\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSf(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpf\_bot\left(\frac{L}{2}\right) = -5.$$

elastic stress of bottom concrete chord for frequent load combination

$$\sigma cpr\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpr\_bot\left(\frac{L}{2}\right) = -1.$$

elastic stress of bottom concrete chord for rare load combination $\qquad \sigma cpr\_bot(lpt1) = -1$

$$\sigma cpr\_top(x) := \frac{-Np\_tot \cdot rinf}{Aid} + \frac{[Mq\_SLSr(x) - rinf \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id} \qquad \sigma cpr\_top\left(\frac{L}{2}\right) = -4.$$

elastic stress of top concrete chord for rare load combination

$$\sigma cpr\_p(x) := \sigma pm(x,t) \cdot rsup + 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(dp_{jp} - Yid\right)}{Ixo\_id} \right] \qquad \sigma cpr\_p\left(\frac{L}{2}\right) = 1.205$$

creep stress of bottom prestressing steel for rare load combination

$$\sigma cpr\_s(x) := 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(ds_{js} - Yid\right)}{Ixo\_id} \right] \qquad \sigma cpr\_s\left(\frac{L}{2}\right) = -31.8$$

creep stress of bottom mild steel for rare load combination

EN1992-1-1:2002

$\sigma cpg1\_bot(lpt1) = -20.153$ > $k1 \cdot \beta cc(2)^{\frac{2}{3}} \cdot fck = -18.733$ CHECK

> $k2 \cdot fck = -20.25$ not compulsory in environment XC

$\sigma cpg1\_top(lpt1) = 2.959$ < $fctmj(2) = 2.731$

$\sigma cpg1\_tops(lpt1) = 6.77$ < $k3 \cdot fsk = 400$

$\sigma cpf\_bot\left(\frac{L}{2}\right) = -5.703$ < $fctm = 3.795$ CHECK

$\sigma cpr\_bot\left(\frac{L}{2}\right) = -1.926$ < $fctm = 3.795$

$\sigma cpr\_bot(lpt1) = -16.362$ > $k1 \cdot fck = -27$ CHECK

> $0.4 \cdot fcm = -21.2$

$\sigma cpr\_top\left(\frac{L}{2}\right) = -4.481$ > $k1 \cdot fck = -27$ CHECK

> $0.4 \cdot fcm = -21.2$

$\sigma cpr\_p\left(\frac{L}{2}\right) = 1.205 \times 10^3$ < $k5 \cdot fptk = 1.488 \times 10^3$ CHECK

$\sigma cpr\_s\left(\frac{L}{2}\right) = -31.868$ < $k3 \cdot fsk = 400$ CHECK

SLS CRACK CONTROL (§9.2.3)

$c\_act := Htot - ds_{js} - 10 = 20$

$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1$

$wlim\_cal := 0.2 \cdot ksurf = 0.2$ mm

$w\_freq := 0$ < $wlim\_cal = 0.2$ CHECK

6.10 ULS checks

ULS BENDING-AXIAL CONTROL (§8.1)

$$Mrd = 313.6 \quad kNm \qquad > \qquad \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6} = 270.743 \qquad \text{CHECK}$$

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§8.2)

$Vq\_ULS(x) := \left|(g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2} - x\right)\right|$ shear action distribution at Ultimate Limit State (ULS)

$d := Yp = 255$ mm effective depth of cross-section

$VEd := Vq\_ULS(d) = 1.153 \times 10^5$ N design shear action at control section at distance d from support

$\gamma v := 1.3$ safety factor for initial shear check

$bw := 320$ mm design web width

$z := 0.9\cdot d = 229.5$ conventional lever arm of internal stress resultants

$\tau Ed := \frac{VEd}{bw\cdot z} = 1.57$ MPa equivalent mean acting shear stress on control cross-section

$Dlower := 16$ mm maximum aggregate diameter following assumed mix design

$ddg := \min\left[\text{if}\left[-fck > 60, 16 + Dlower\cdot\left(\frac{60}{-fck}\right)^2, 16 + Dlower\right], 40\right] = 32$ size parameter

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT (§8.2.2)

$$\tau Rdc\_min(x) := \frac{11}{\gamma v}\cdot\sqrt{\frac{-fck}{(fptd - \sigma pm(x,t))}\cdot\frac{ddg}{d}}$$ §(8.20)

$\tau Rdc\_min(d) = 0.841$ MPa not checked with τEd -> detailed evaluation is mandatory following §8.2.1

$$\rho l(x) := if\left(x < lpt2, \frac{Ap\_tot}{bw\cdot d}\cdot\frac{x}{lpt2}, if\left(x > L - lpt2, \frac{Ap\_tot}{bw\cdot d}\cdot\frac{-x+L}{lpt2}, \frac{Ap\_tot}{bw\cdot d}\right)\right)$$ longitudinal geometric reinforcement ratio §(8.28)

$ep := Yp - Yid = 162.787$ mm eccentricity of prestressing

$$acs\_0(x) := max\left(\frac{Mq\_ULS(x)}{Vq\_ULS(x)}, d\right)$$ §(8.30) accounting for comments in §8.2.2(5)

$$k1(x) := min\left[\frac{0.5}{acs\_0(x)}\cdot\left(ep + \frac{d}{3}\right)\cdot\frac{Ac}{bw\cdot z}, 0.18\cdot\frac{Ac}{bw\cdot z}\right]$$ §(8.34)

$$av\_0(x) := \sqrt{\frac{acs\_0(x)}{4}\cdot d}$$ §(8.29) accounting for comments in §8.2.2(5)

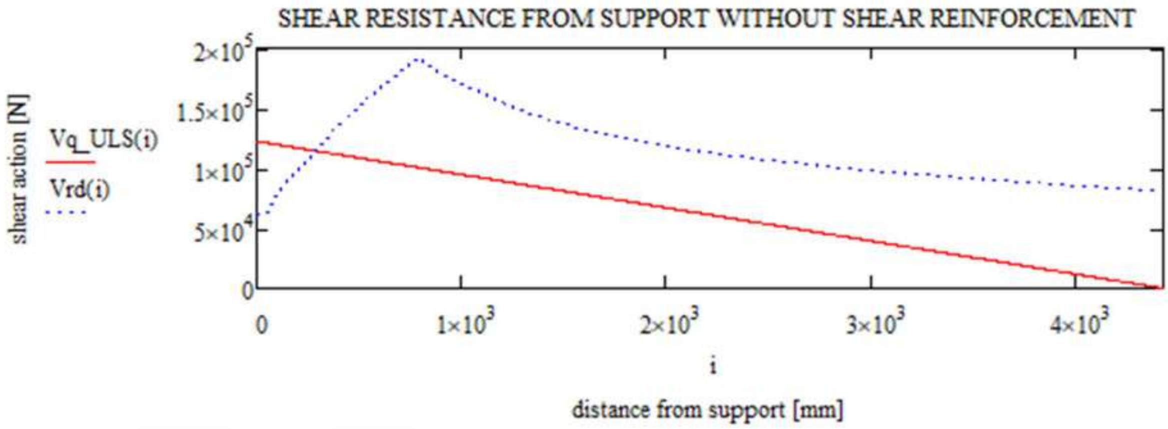
$$\tau Rdc\_0(x) := \frac{0.66}{\gamma v}\cdot\left(100\cdot\rho l(x)\cdot -fck\cdot\frac{ddg}{av\_0(d)}\right)^{\frac{1}{3}}$$ §(8.33)

$$\tau Rdcmax(x) := min\left[2.15\cdot\tau Rdc\_0(x)\cdot\left(\frac{acs\_0(x)}{d}\right)^{\frac{1}{6}}, 2.7\cdot\tau Rdc\_0(x)\right]$$ §(8.35)

$$\sigma cp(x) := \sigma pm(x,t)\cdot\frac{Ap\_tot}{Ac}$$ §(8.33)

$$\tau Rdc(x) := max(min(\tau Rdc\_0(x) + k1(x)\cdot\sigma cp(x), \tau Rdcmax(x)), \tau Rdc\_min(x))$$ §(8.32)

$Vrd(x) := bw\cdot z\cdot\tau Rdc(x)$



MEMBERS PROVIDED WITH SHEAR REINFORCEMENT (§8.2.3)

$\theta v := \operatorname{atan}\left(\frac{1}{2}\right) = 0.464$ rad angle of inclination of concrete compressed strut

$\nu := 0.5$ §8.2.3(6) NOTE: steel grade B500A is used

$\sigma cd := \tau Ed \cdot (\cot(\theta v) + \tan(\theta v)) = 3.926$ MPa < $\nu \cdot -fcd = 15.453$ MPa CHECK §(8.44)

$fywd := fsd = 454.545$ MPa design yield stress of shear reinforcement steel

$Asw := 4 \cdot \frac{6^2 \cdot \pi}{4} = 113.097$ mm2 area of transverse shear reinforcement

$s1 := 190$ mm spacing of transverse reinforcement (constant throughout the member)

$\tau Rd\_sy := \frac{Asw}{bw \cdot s1} \cdot fywd \cdot \cot(\theta v) = 1.691$ MPa > $\tau Ed = 1.57$ MPa CHECK §(8.42)

MOMENT DIAGRAM ACCOUNTING DUE TO SHEAR RESISTING MECHANISM (§12.3.2)

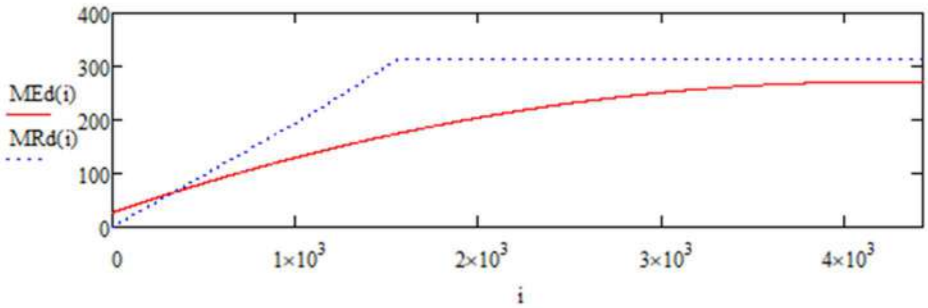
$\alpha 3 := 1$ fatigue check not required

$$lbpd := lpt2 + \frac{\gamma c}{1.5} \cdot \frac{2 \cdot \alpha 2 \cdot \alpha 3 \cdot \left(fptd - \sigma pm\left(\frac{L}{2}, t\right)\right)}{\eta 1 \cdot \sqrt{(-fck)}} \cdot \phi p = 1.558 \times 10^3 \quad \text{mm} \quad §13.5.2$$

$$MRd(x) := \text{if}\left[x < lpt2, Mrd \cdot \frac{\sigma pm(lpt2,t)}{fptd} \cdot \frac{x}{lpt2}, \text{if}\left[x > lbpd, Mrd, Mrd \cdot \frac{\sigma pm(lpt2,t)}{fptd} + \frac{(x - lpt2)}{(lbpd - lpt2)} \cdot \left(Mrd - Mrd \cdot \frac{\sigma pm(lpt2,t)}{fptd}\right)\right]\right]$$

$$al := z \cdot \left(\frac{\cot(\theta v)}{2}\right) = 229.5$$

$$MEd(x) := \text{if}\left(x > \frac{L}{2} - al, \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6}, \frac{Mq\_ULS(x + \text{round}(z))}{10^6}\right)$$



MINIMUM REINFORCEMENT (§12.2)

$kh := if[0.8 - 0.6\cdot(min(bw, Htot) - 0.3) < 0.5, 0.5, if[0.8 - 0.6\cdot(min(bw, Htot) - 0.3) > 0.8, 0.8, 0.8 - 0.6\cdot(min(bw, Htot) - 0.3)]] = 0.5$ §9.2.2(2)

$fct\_eff := fctm$

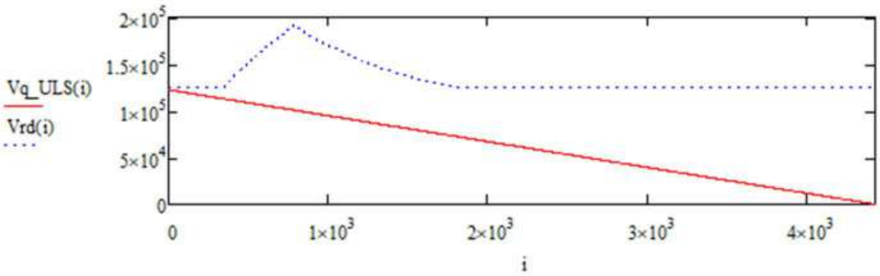
$As\_min\_w1 := 0.2\cdot kh\cdot fct\_eff\cdot\frac{Ac}{fsk} = 233.331$ mm2 $Ap\_tot + As\_tot - As_1 = 800.549$ mm2 CHECK §(9.2)

$rsup\cdot Np\_tot\cdot\frac{(Yp - Yid)}{10^6} + \left(fctm + \frac{Np\_tot\cdot rsup}{Aid}\right)\cdot\frac{Ixo\_id}{(Htot - Yid)\cdot 10^6} = 246.102$ < $Mrd = 313.6$ kNm CHECK §(12.1)

$s2 := 190$ < $0.75\cdot d = 191.25$ CHECK §12.1

$\rho w\_min := \frac{Asw}{s2\cdot bw} = 1.86\times 10^{-3}$ > $0.08\cdot\frac{\sqrt{-fck}}{fsk} = 1.073\times 10^{-3}$ CHECK §(12.4)

$Vrd(x) := max(\tau Rd\_sy\cdot bw\cdot z, Vrd(x))$



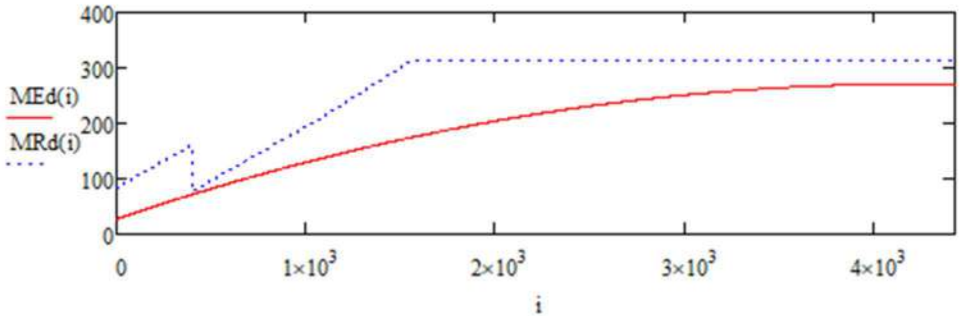
CHECK OF SUPPORT MILD REBARS

$MEd(0) = 27.414$ Nmm

$0.25 \cdot MEd\left(\frac{L}{2}\right) = 67.686$ Nmm §12.1(4) $< \left(4 \cdot \pi \cdot \frac{16^2}{4}\right) \cdot fsd \cdot 0.9 \cdot \frac{d}{10^6} = 83.898$ CHECK

area of support mild steel

$$MRd(x) := if\left[x < 400, 0.9 \cdot d \cdot \frac{fywd}{10^6} \cdot \left(4 \cdot \pi \cdot \frac{16^2}{4}\right) + MRd(x), MRd(x)\right]$$



ANCHORAGE (§11.4)

$klb := 50$

$kcp := 1$ for good bond conditions

$n\sigma := \frac{3}{2}$

$cs := 50$

$cx := 75$

$cy := 40$

$cd(\phi) := min(0.5 \cdot cs, cx, cy, 3.75 \cdot \phi)$ $cd(12) = 25$

$$lbd(\phi) := max\left[klb \cdot kcp \cdot \phi \cdot \left(\frac{fsd}{435}\right)^{n\sigma} \cdot \left(\frac{25}{-fck}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1.5 \cdot \phi}{cd(\phi)}\right)^{\frac{1}{2}}, 10 \cdot \phi\right]$$

$lbd(16) = 579.319$ $\frac{lbd(12)}{12} = 28.489$

length of straight part for 90° bent bars

$lb90(\phi) := max(70, lbd(\phi) - 15 \cdot \phi, 10 \cdot \phi)$

$lb90(12) = 161.872$ $lb90(16) = 339.319$

length of straight part for 135° bent bars (stirrups)

$lb135(\phi) := max(50, lbd(\phi) - 15 \cdot \phi, 5 \cdot \phi)$

$lb135(12) = 161.872$ $lb135(8) = 50$

SHEAR AT THE WEB-FLANGE INTERFACE (§8.2.5)

$hf := 354$

$\Delta x := \frac{L}{4} = 2.212 \times 10^3$ $\quad Mq\_ULS\left(\frac{L}{4}\right) = 2.031 \times 10^8$ $\quad M_7 = 2.118 \times 10^8$

$$\Delta Fd := \sum_{i=1}^{80} \left(\sigma c\left(\varepsilon\left(y_i, \varepsilon\_sup_7, \theta_7\right)\right) \cdot 2400 \cdot \Delta y\right) + \sum_{j=1}^{1} \left(\sigma s\left(\varepsilon\left(ds_j, \varepsilon\_sup_7, \theta_7\right)\right) \cdot As_j\right) = -8.64 \times 10^5$$

$\theta f := 0.462$

$$\tau Edfl\_we := \frac{-\Delta Fd}{hf \cdot \Delta x} = 1.103$$

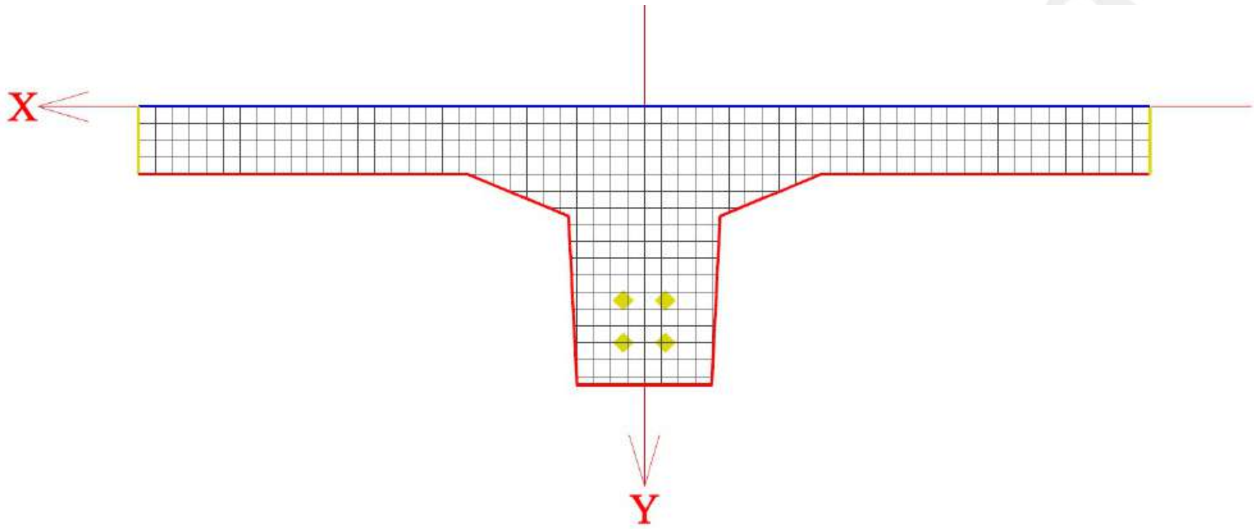
$\sigma cdfl\_we := \tau Edfl\_we \cdot (\cot(\theta f) + \tan(\theta f)) = 2.765$ $\quad < \quad$ $\nu \cdot -fcd = 15.453$ CHECK compressed strut

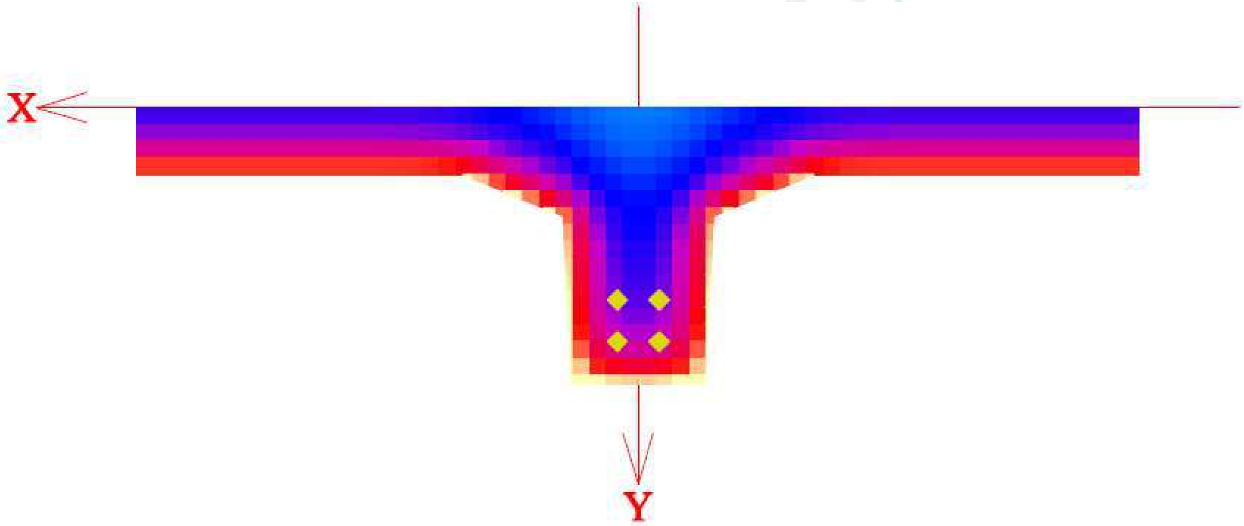
$Asf := 2 \cdot \pi \cdot \frac{6^2}{4} + \pi \cdot \frac{6^2}{4} = 84.823$ mm^2 transverse horizontal reinforcement

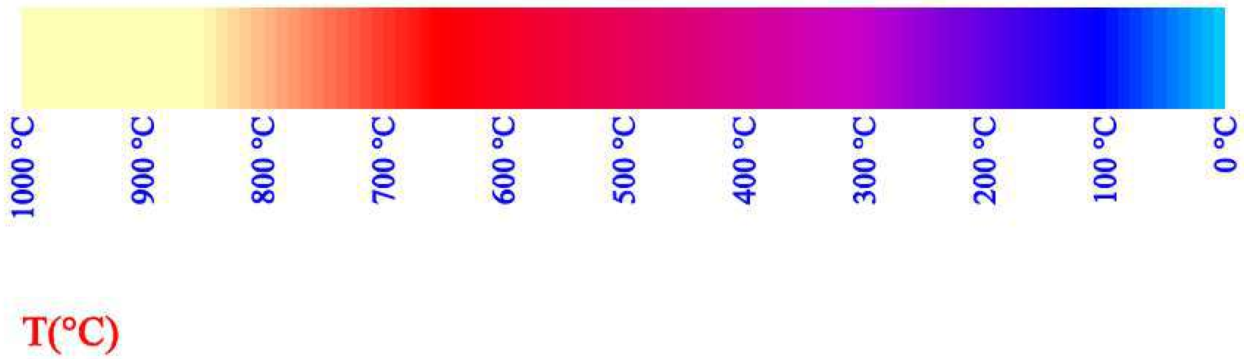
$sf := 200$ mm spacing of transverse horizontal reinforcement

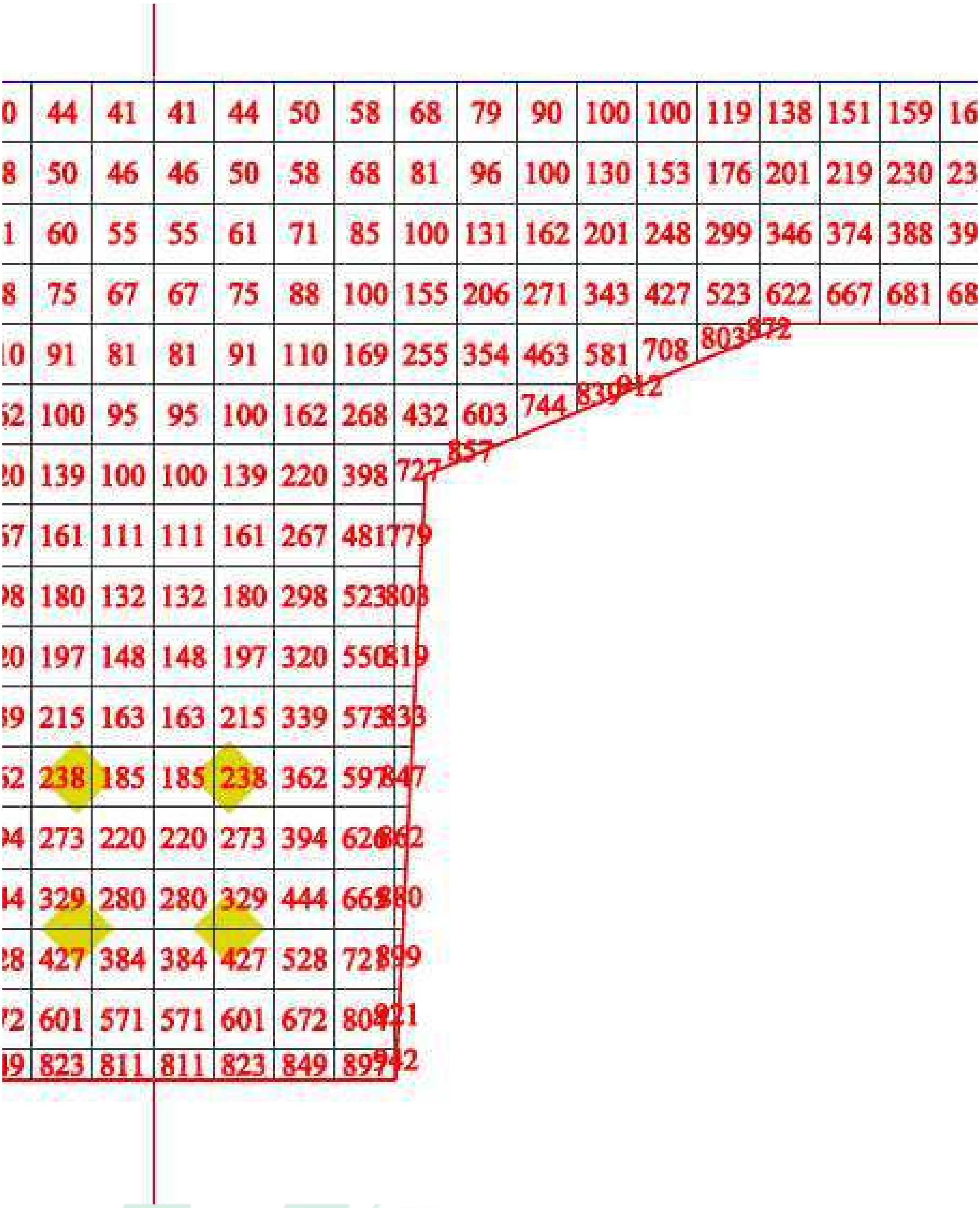
$Asf \cdot \frac{fsd}{sf} = 192.78$ $\quad > \quad$ $\frac{2.031}{2.118} \cdot \tau Edfl\_we \cdot \frac{hf}{\cot(\theta f)} = 186.474$ CHECK

6.11 Fire checks








0 44 41 41 44 50 58 68 79 90 100 100 119 138 151 159 16
8 50 46 46 50 58 68 81 96 100 130 153 176 201 219 230 23
1 60 55 55 61 71 85 100 131 162 201 248 299 346 374 388 39
8 75 67 67 75 88 100 155 206 271 343 427 523 622 667 681 68
10 91 81 81 91 110 169 255 354 463 581 708 803 872
52 100 95 95 100 162 268 432 603 744 830 912
20 139 100 100 139 220 398 727 857
57 161 111 111 161 267 481 779
98 180 132 132 180 298 523 803
20 197 148 148 197 320 550 819
39 215 163 163 215 339 573 833
52 238 185 185 238 362 597 847
94 273 220 220 273 394 626 862
14 329 280 280 329 444 665 880
28 427 384 384 427 528 721 899
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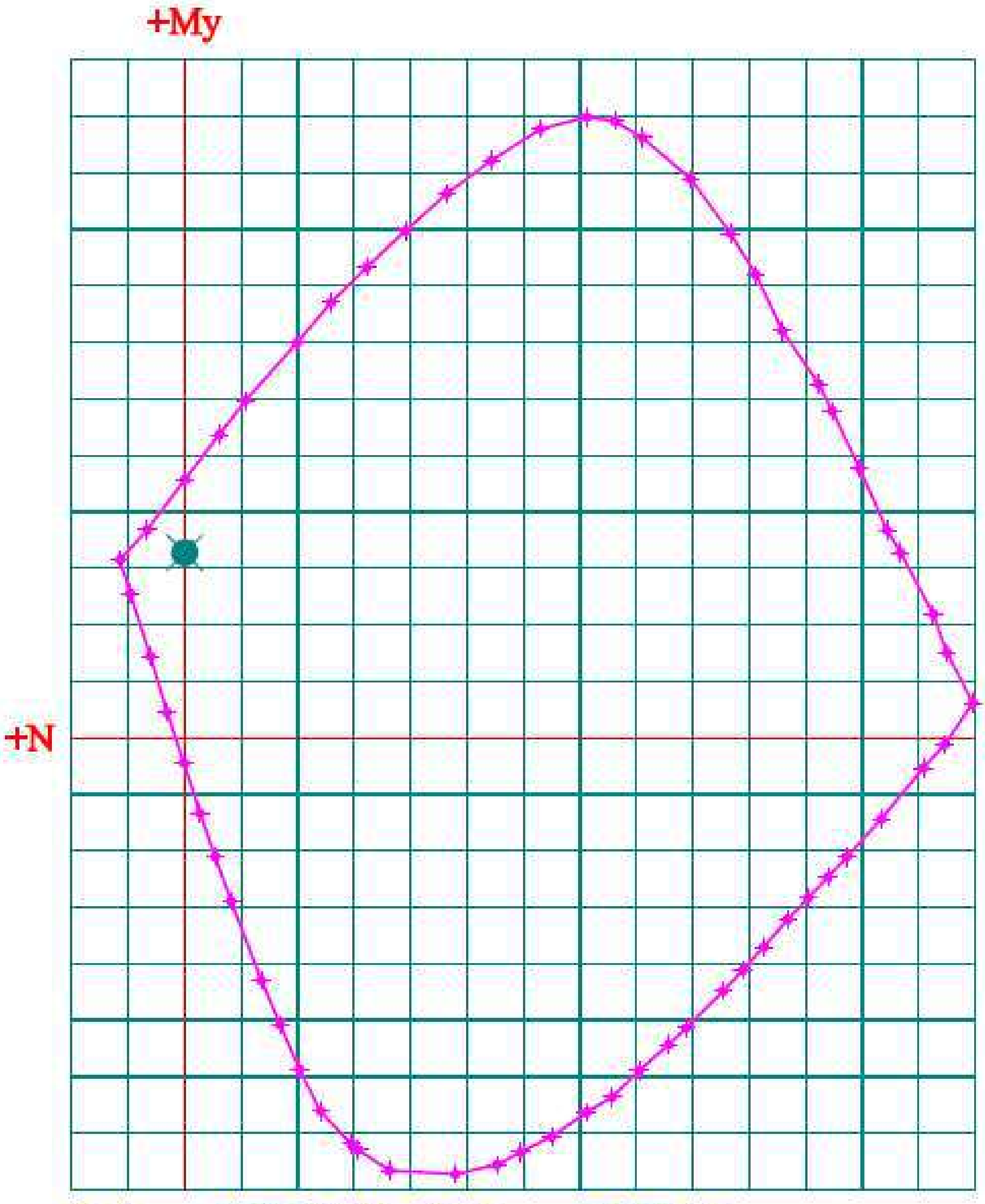
19 823 811 811 823 849 897 942

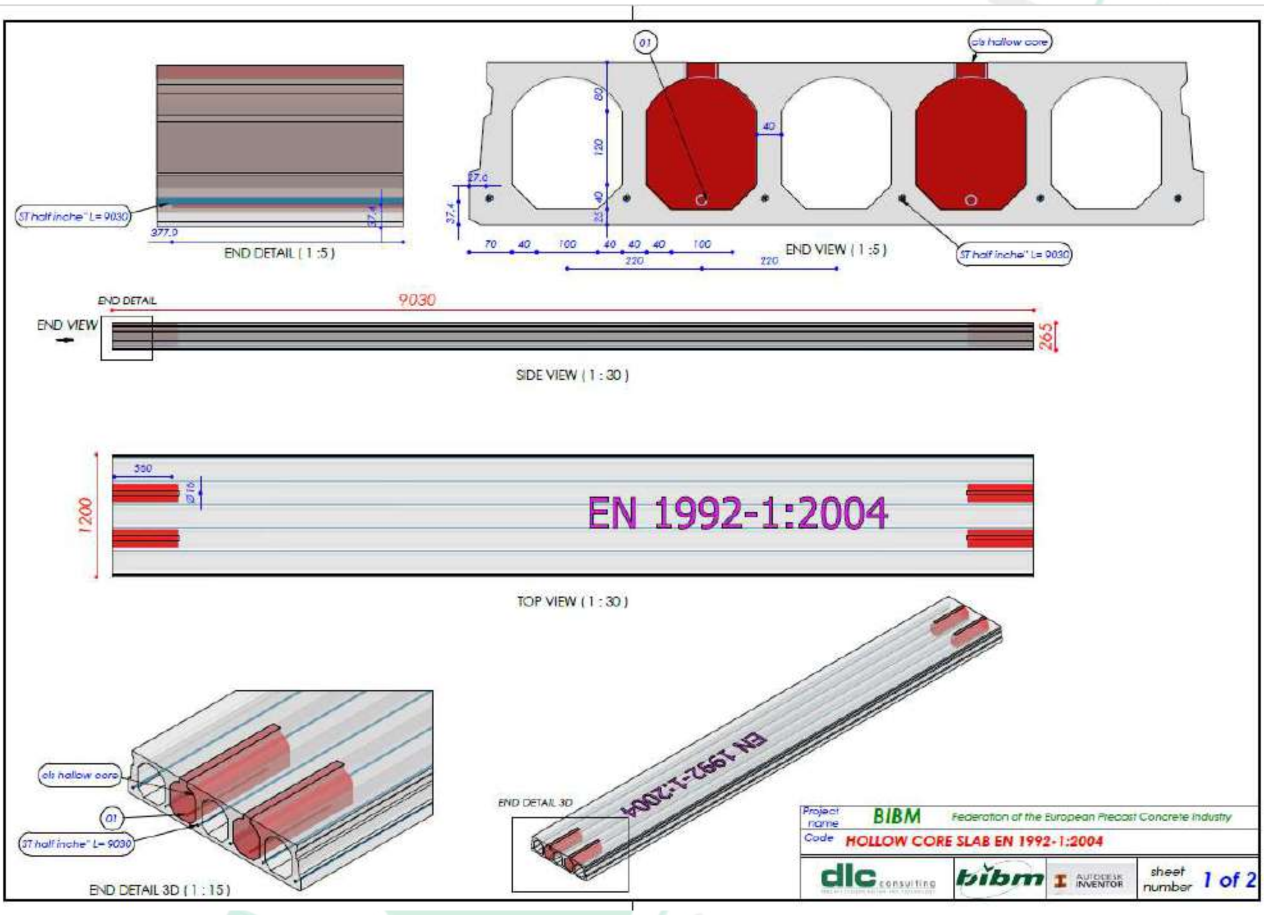


N: 1 sp = 380.00 kN;   M: 1 sp = 22.00 kN·m

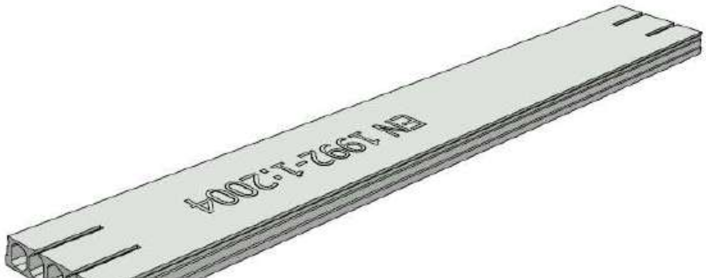
Nz =   0.00 kN
My+ =   100.34 kN·m
My- =   -10.53 kN·m

7 Hollowcore element –EN1992-1:2004

7.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_		
	01	4	884	3536	16 mm		
	Total mass rebars [kg]			3,54	Incidence kg/m²	1,65	
	ST half inche'' L= 9030	6	6599	39594	12.7 mm		
	Total mass strands [kg]			39,594	Incidence kg/m²	18,47	
	Total mass of steel [kg]			43,13		Concrete volume [m³]	1.224
						Cast in situ [m³]	0.92
						Total concrete [m³]	2.144



Project name	BIBM Federation of the European Precast Concrete Industry
Code	HOLLOW CORE SLAB EN 1992-1:2004
sheet number	2 of 2

7.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0 \;\; 42.5 \;\; 106.1 \;\; 106.11 \;\; 189.41 \;\; 240 \;\; 240.1 \;\; 265)^T$

$Htot := \max(y\_tr)$ maximum depth

Width of corresponding chord:

$b\_tr := 4\cdot(582 \;\; 578 \;\; 398 \;\; 112.8 \;\; 124.3 \;\; 378 \;\; 598 \;\; 600)^T$

$r\_circ := 126$ radius of central void pipe

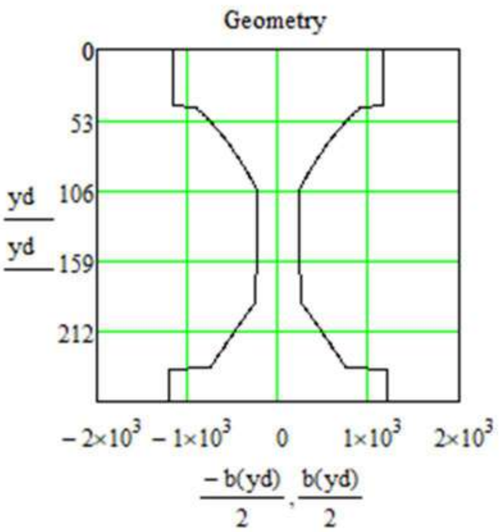
$x\_circ(y) := 2\sqrt{r\_circ^2 - (y - 160)^2}$

$b\_lin(y) := \mathrm{linterp}(y\_tr, b\_tr, y)$

$b\_circ(y) := \mathrm{linterp}(y\_tr, b\_tr, y) - 5\cdot x\_circ(y)$

$b(y) := \mathrm{if}(y \le 106.1 \wedge y \ge 42.5, b\_circ(y), b\_lin(y))$

$yd := 0..Htot$ $b(0) = 2.328 \times 10^3$



condensed 1D geometry plot

$u := 2400\cdot 2 + Htot\cdot 2 + 10\cdot 160\cdot\pi = 1.036 \times 10^4$ mm exposed perimeter

Prestressing reinforcement

nominal strand diameter

12.7	mm	0.5'
15.24	mm	0.6'

Area of a single strand:

$Ap0 := 93$ $\quad \phi p := 12.7$ mm nominal strand diameter

Depth of prestressing strands from upper chord:

$dp := (180\ \ 230\ \ 220)^T$

Area of strands at each depth:

$Ap := (0 \cdot Ap0\ \ 0 \cdot Ap0\ \ 12 \cdot Ap0)^T$

$0.75 \cdot 1860 = 1.395 \times 10^3$ $\quad 0.9 \cdot 0.9 \cdot 1860 = 1.507 \times 10^3$

$\sigma p0 := 1400$ MPa

$\sigma prec := (0.4 \cdot \sigma p0\ \ 1 \cdot \sigma p0\ \ \sigma p0)^T$ initial prestressing

$perdite := 0 \cdot (1\ \ 1\ \ 1)^T$ in percentual **% (losses are introduced later)**

$jp := rows(Ap)$ $\quad jp = 3$

$k := 1..jp$

$$\sigma o_k := \sigma prec_k \cdot \left[\frac{(100 - perdite_k)}{100}\right] \qquad \sigma o = \begin{pmatrix} 560 \\ 1.4 \times 10^3 \\ 1.4 \times 10^3 \end{pmatrix}$$

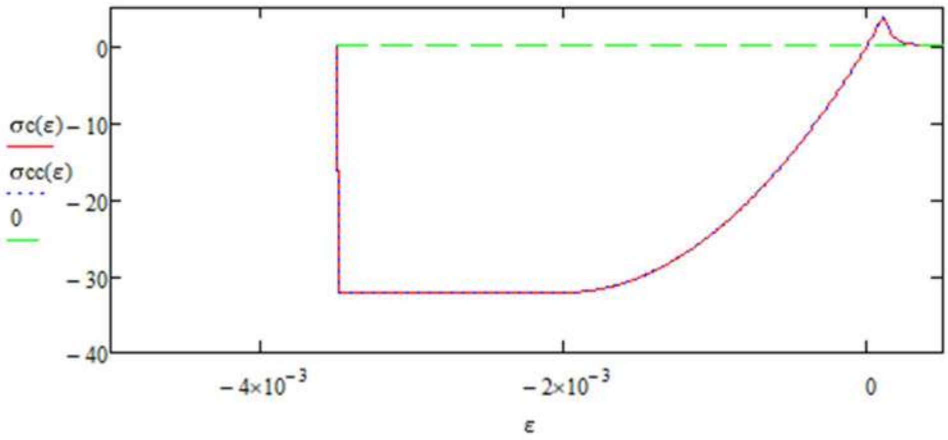
$$Ap\_tot := \sum_{k=1}^{jp} Ap_k \qquad Ap\_tot = 1.116 \times 10^3$$

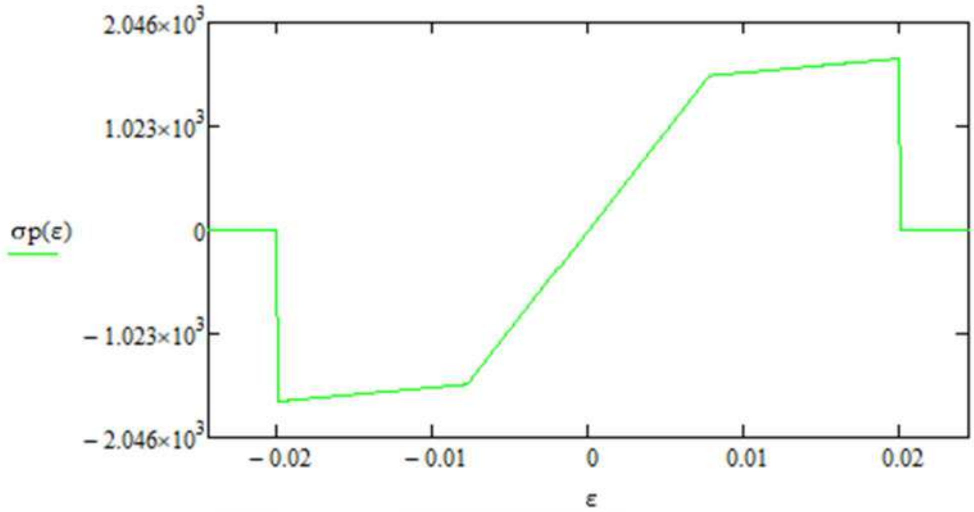
$ypmax := max(dp)$ $\quad ypmax = 230$

$$Np\_tot := \sum_{k=1}^{jp} \left(\left(Ap_k \cdot \sigma o_k\right)\right) \qquad Np\_tot = 1.562 \times 10^6 \text{ N}$$ total prestressing initial force

$$Yp := \frac{\sum_{k=1}^{jp} \left(dp_k \cdot Ap_k \cdot \sigma o_k\right)}{\sum_{k=1}^{jp} \left(Ap_k \cdot \sigma o_k\right)} = 220 \text{ mm}$$ centre of gravity of prestressing

7.3 Material constitutive laws employed in the calculation





7.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 3.163 \times 10^5$$

$$\rho p := \frac{Ap\_tot}{Ac} = 3.529 \times 10^{-3}$$ geometric ratio for longitudinal prestressing tendons

$$\rho tot := \frac{Ap\_tot}{Ac} = 3.529 \times 10^{-3}$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y)\cdot y\,dy \qquad Syc = 3.878 \times 10^7$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 122.619$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y)\cdot(y - yG)^2\,dy \qquad Ixo\_cls = 2.799 \times 10^9$$

Global area of all prestressing reinforcement

$$Area\_tr := \left| \begin{array}{l} s \leftarrow 0 \\ \text{for } x \in 1..jp \\ \quad s \leftarrow Ap_x + s \end{array} \right. \qquad Area\_tr = 1.116 \times 10^3$$

First moment of the area referred to prestressing reinforcement only

$$Sxp := \sum_{i=1}^{jp} \left(Ap_i \cdot dp_i\right) \qquad Sxp = 2.455 \times 10^5$$

Centre of gravity of prestressing

$$Yp := \frac{Sxp}{Area\_tr} \qquad Yp = 220$$

Idealisation coefficients (elastic)

$$np := \frac{Ep}{Ecm} \qquad np = 5.374$$

$$ns := \frac{Es}{Ecm} \qquad ns = 5.512$$

Area of ideal cross-section

$$Aid := Ac + (np - 1) \cdot \sum_{j=1}^{jp} Ap_j \qquad Aid = 3.212 \times 10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac \cdot yG + (np - 1) \cdot (Area\_tr \cdot Yp) \qquad Sxid = 3.985 \times 10^7$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 124.099$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y) \cdot (y - Yid)^2 \, dy - \sum_{i=1}^{jp} \left[Ap_i \cdot \left(dp_i - Yid \right)^2 \right]$$

Second moment of the prestressing reinforcement area

$$Ixoidprec := np \cdot \sum_{i=1}^{jp} \left[Ap_i \cdot \left(dp_i - Yid \right)^2 \right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidprec \qquad Ixo\_id = 2.844 \times 10^9 \text{ mm^4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.016$$

7.5 Loads

LOADS

$interaxis := 2400$ mm

$g1 := Ac \cdot 0.000025 = 7.907$ kN/m dead load from self-weight

$g2 := 2 \cdot \frac{interaxis}{1000} = 4.8$ kN/m nonstructural dead load

$q := 3 \cdot \frac{interaxis}{1000} = 7.2$ kN/m live load

$L := 8850$ mm calculation length (span between supports)

$\psi 2 := 0.3$ non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ SLS bending moment distribution from live load

7.6 Prestressing transfer and time-dependent behaviour

TRANSFER OF PRESTRESS (§8.10.2.2)

$\alpha 1 := 1$ gradual release of prestressing

$\alpha 2 := 0.19$ for 7-wire strands

$\sigma pm0 := \sigma p0 = 1.4 \times 10^3$ MPa initial prestressing

$\eta p1 := 3.2$ for 7-wire strands

$\eta 1 := 1$ in favourable position

$fbpt := \eta p1 \cdot \eta 1 \cdot fctdj(2) = 3.51$ MPa equivalent constant bond stress at prestress realease following §(8.15)

$lpt := \frac{\alpha 1 \cdot \alpha 2 \cdot \sigma pm0}{fbpt} \cdot \phi p = 962.587$ mm basic value of the transmission length following §(8.16)

$lpt1 := 0.8 lpt = 770.069$ mm lower-bound transfer length following §(8.17)

$lpt2 := 1.2 \cdot lpt = 1.155 \times 10^3$ mm upper-bound transfer length following §(8.18)

Prestress losses

$hn := 2 \cdot \frac{Ac}{u} = 61.076$ mm

$\varepsilon cs := \frac{0.65}{1000} = 6.5 \times 10^{-4}$ shrinkage strain assumed as a result of laboratory tests on the specific concrete mix employed

$\rho 1000 := 0.025$ for class 2 (low-relaxation) tendons following §3.3.2(5)

$k\rho := 0.16$

$t := 50 \cdot 365 = 1.825 \times 10^{4}$ days Life span

$\sigma cpQP2(x) := \frac{-Np\_tot}{Aid} + \frac{[Mq\_SLSg1(x) - Np\_tot \cdot (Yp - Yid)] \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP2\left(\frac{L}{2}\right) = -7.307$ MPa

stress in quasi-permanent load combination at 2 days
(conventional equivalent time for prestressing release)

$\sigma cpQP23(x) := \frac{Mq\_SLSg2(x) \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP23\left(\frac{L}{2}\right) = 1.585$ MPa

stress in quasi-permanent load combination at 23 days
(conventional time for assemblage of the structure on site)

$\sigma cpQP91(x) := \frac{Mq\_SLSq(x) \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP91\left(\frac{L}{2}\right) = 0.713$ MPa

stress in quasi-permanent load combination at 91 days
(conventional time for enter in use of the structure)

$$\Delta\sigma pr(x,t) := \left[\sigma p0 + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x))\right] \cdot \rho 1000 \cdot \left(\frac{24 \cdot t}{1000}\right)^{k\rho}$$

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$h0 := 2 \cdot \frac{Ac}{u} = 61.076$ mm notional size of the member

$RH := 50$ % relative humidity

$t0\_T(t0) := t0$

$\alpha := 1$ for cement class R

$$t0\_mod(t0) := \max\left[t0\_T(t0) \cdot \left(\frac{9}{2 + t0\_T(t0)^{1.2}} + 1\right)^{\alpha}, 0.5\right] \qquad t0\_mod(2) = 6.189$$

$$\alpha c1 := \left(\frac{35}{-fcm}\right)^{0.7} = 0.748$$

$$\alpha c2 := \left(\frac{35}{-fcm}\right)^{0.2} = 0.92$$

$$\alpha c3 := \left(\frac{35}{-fcm}\right)^{0.5} = 0.813$$

$$\beta h := if\left[-fcm > 35, \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250 \cdot \alpha c3, 1500 \cdot \alpha c3\right], \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250, 1500\right]\right] = 294.783$$

$$\beta t0(t0) := \frac{1}{0.1 + t0\_mod(t0)^{0.2}}$$

$$\beta c(t, t0) := \left(\frac{t - t0\_mod(t0)}{\beta h + t - t0\_mod(t0)}\right)^{0.3}$$

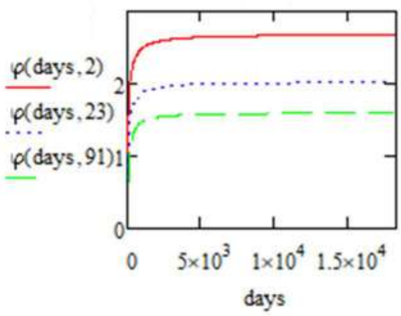
$$\beta fcm := \frac{16.8}{\sqrt{-fcm}} = 2.308$$

$$\varphi RH := if\left[-fcm > 35, \left(1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}} \cdot \alpha c1\right) \cdot \alpha c2, 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}}\right] = 1.794$$

$\varphi 0(t0) := \varphi RH \cdot \beta fcm \cdot \beta t0(t0)$

$\varphi(t, t0) := \varphi 0(t0) \cdot \beta c(t, t0)$

$\varphi(t, 2) = 2.676$ $\qquad \varphi(t, 91) = 1.595$



TIME-DEPENDENT LOSSES OF PRESTRESS (§5.10.6)

$$\Delta\sigma p\_csr(x,t) := \frac{-\varepsilon cs\cdot Ep - 0.8\cdot\Delta\sigma pr(x,t) + \frac{Ep}{Ecm}\cdot(\sigma cpQP2(x)\cdot\varphi(t,2) + \sigma cpQP23(x)\cdot\varphi(t,23) + \sigma cpQP91(x)\cdot\varphi(t,91))}{1 + \frac{Ep}{Ecm}\cdot\frac{Ap\_tot}{Ac}\cdot\left[1 + \frac{Ac}{Ixoidcls}\cdot(Yp - Yid)^2\right]\cdot\left(1 + 0.8\cdot\frac{\varphi(t,2)\cdot\sigma cpQP2(x) + \varphi(t,23)\cdot\sigma cpQP23(x) + \varphi(t,91)\cdot\sigma cpQP91(x)}{\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)}\right)}$$

prestress losses following §(5.46)

NOTE: a weighed creep coefficient was considered accounting for the 3 load phases previously introduced

$$\sigma pm(x,t) := \sigma p0 - \frac{Ep}{Ecm}\cdot(\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)) + \Delta\sigma p\_csr(x,t)$$ prestress considering immediate and delayed losses

$$\frac{\sigma pm\left(\frac{L}{2},365\cdot 50\right)}{\sigma p0} = 0.842$$ expected residual prestress ratio after 50 years of life with respect to initial

$$\varepsilon pm := \frac{\sigma pm\left(\frac{L}{2},365\cdot 50\right)}{\sigma p0}\cdot\varepsilon p0$$ expected residual strain after 50 years of life with respect to initial

$$Np\_tot = 1.562\times 10^6$$

$$\sigma pm\left(\frac{L}{2},365\cdot 50\right)\cdot Ap\_tot = 1.315\times 10^6$$ residual prestress force after 50 years of life

7.7 Non-linear moment-curvature diagram

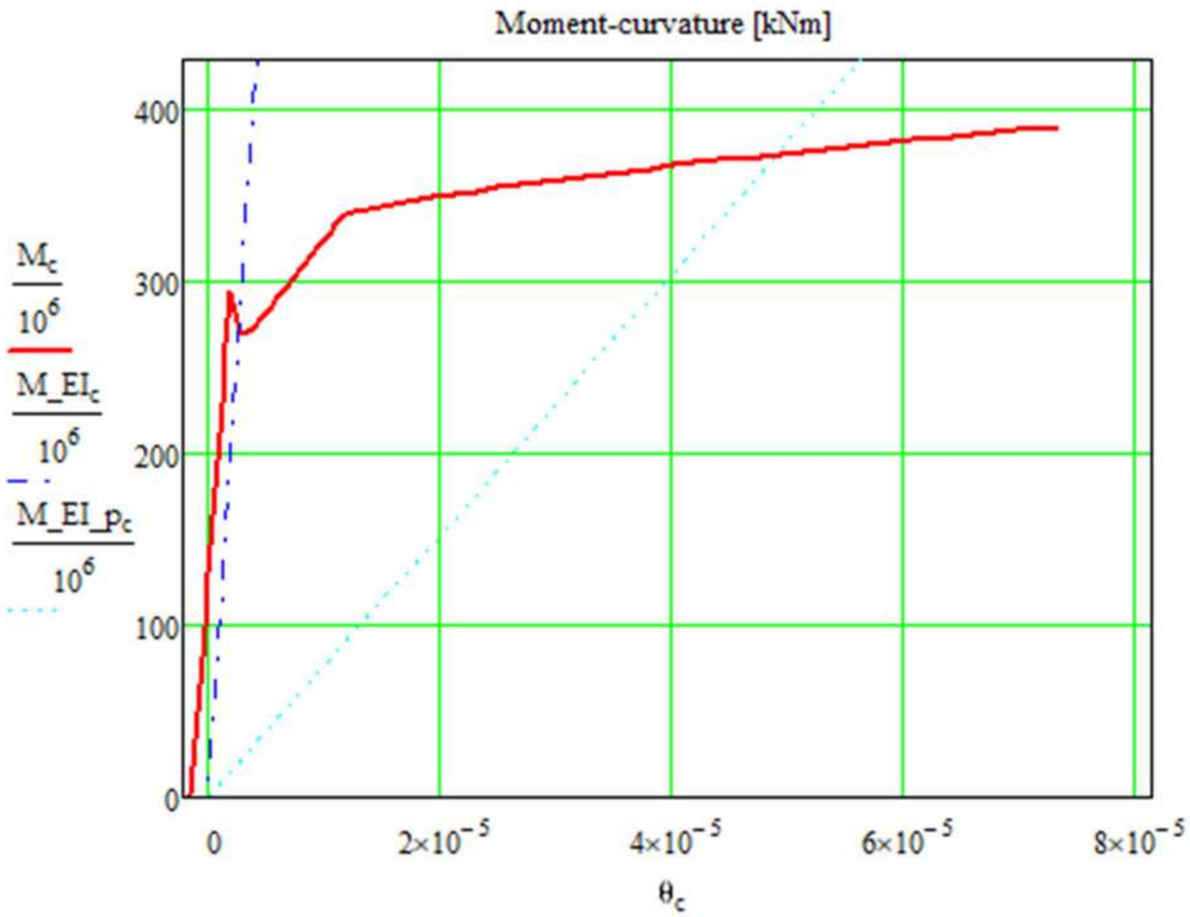
Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

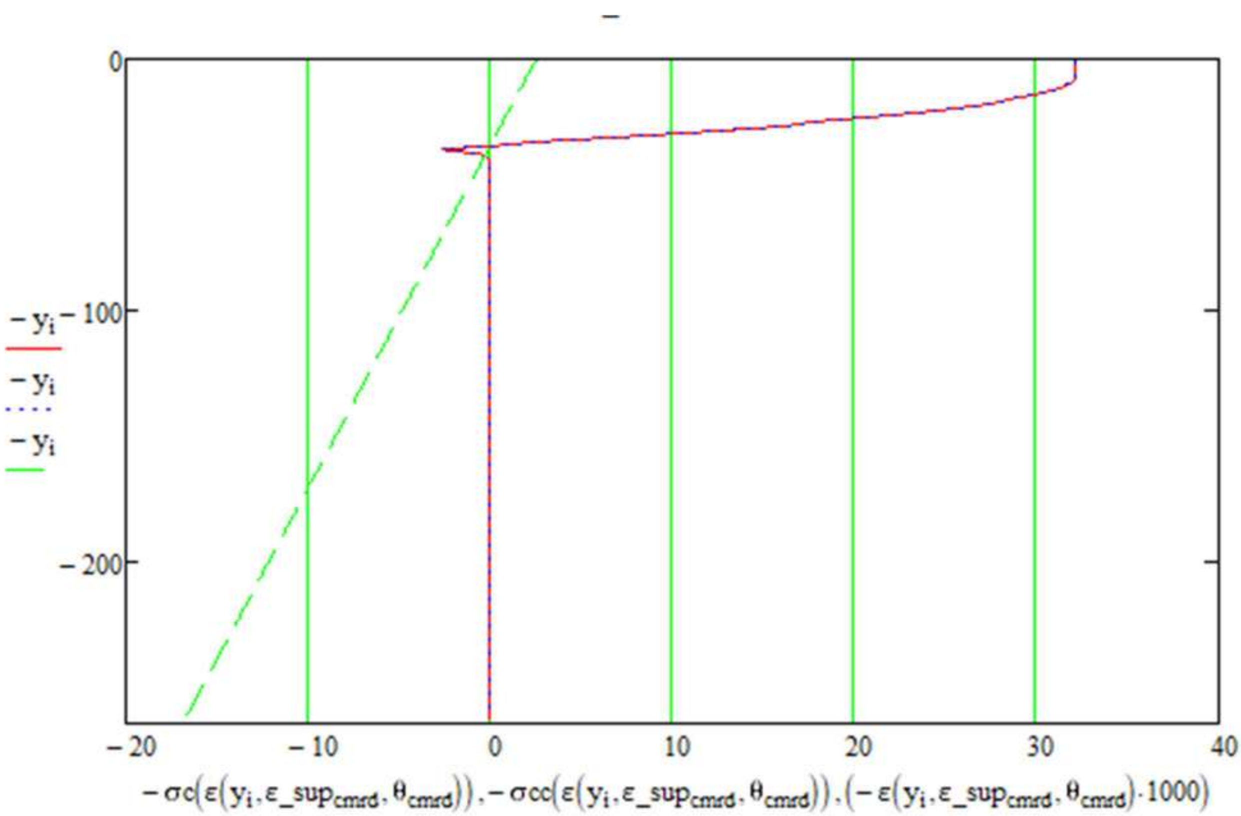
$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot}\left(\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\right) + \sum_{j=1}^{jp}\left(\sigma p\left(\varepsilon\left(dp_j,\varepsilon\_sup,\theta\right) + \varepsilon pm_j\right)\cdot Ap_j\right)$$

$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot}\left[\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\cdot\left(y_i - yG\right)\right] + \sum_{j=1}^{jp}\left[\sigma p\left(\varepsilon\left(dp_j,\varepsilon\_sup,\theta\right) + \varepsilon pm_j\right)\cdot Ap_j\cdot\left(dp_j - yG\right)\right]$$

Design external axial load

$$NS := -0$$


Moment-curvature [kNm]
$\frac{M_c}{10^6}$
$\frac{M\_EI_c}{10^6}$
$\frac{M\_EI\_p_c}{10^6}$
0
100
200
300
400
0
2×10^{-5}
4×10^{-5}
6×10^{-5}
8×10^{-5}
θ_c



Condition at resisting (peak) moment (stress and strain)

7.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformally distributed load q

$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformally distributed load q

$lpt = 962.587$

$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformally distributed load q

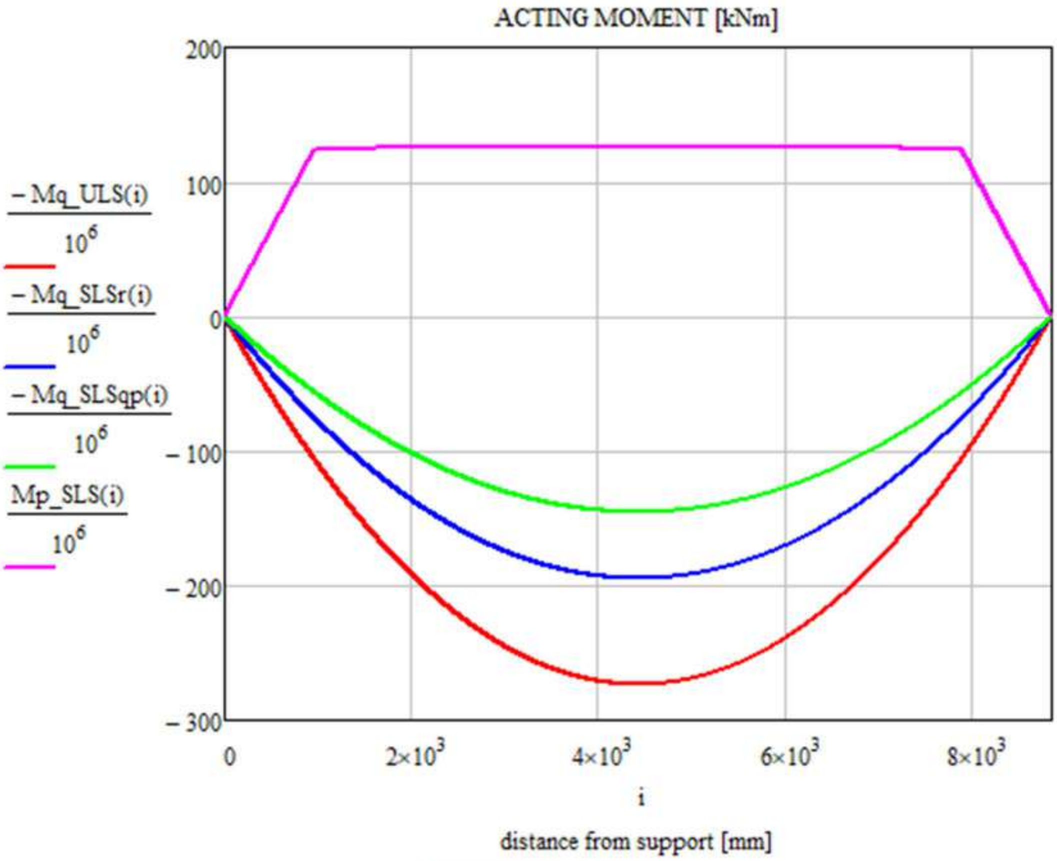
$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformally distributed load q

$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformally distributed load q

$$Mp\_SLS(x) := if\left[x < lpt, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{x}{lpt}, if\left[x > L - lpt, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{-x + L}{lpt}, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid)\right]\right]$$

contribution of prestressing equivalent load in SLS (without modification factors)

$i := 0 .. L$



7.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$v\_inf\_p(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(1 + \varphi(365\cdot50,23))$

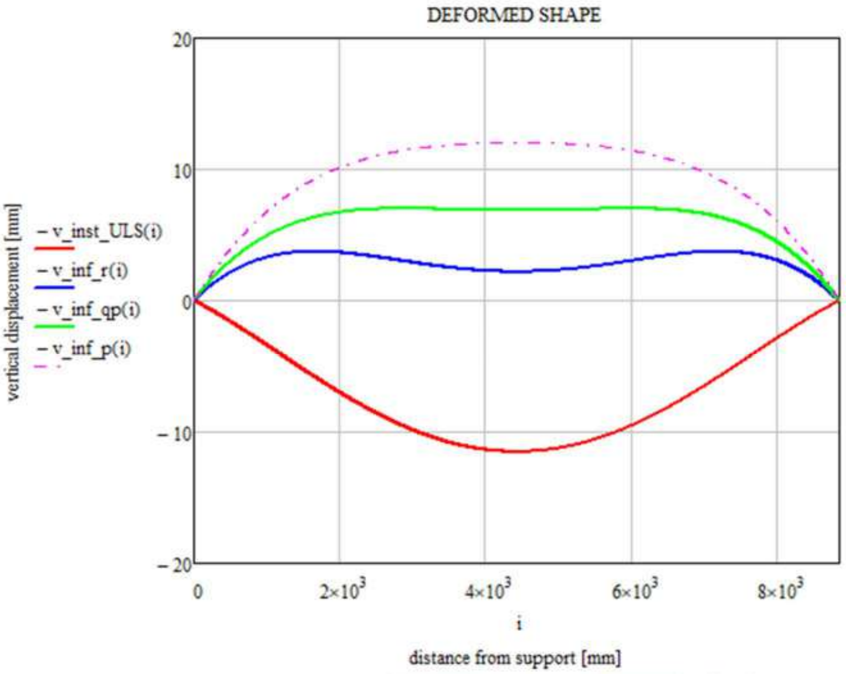
deflection profile at 50 years including creep for permanent load combination

$v\_inf\_qp(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot50,23) - \varphi(365\cdot50,91)) + v\_SLSqp(x)\cdot(1 + \varphi(365\cdot50,91))$

deflection profile at 50 years including creep for quasi permanent load combination

$v\_inf\_r(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot50,23) - \varphi(365\cdot50,91)) + v\_SLSqp(x)\cdot\varphi(365\cdot50,91) + v\_SLSr(x)$

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§7.4.3)

$v\_inf\_r\left(\frac{L}{2}\right) = -2.185$	<	$\frac{L}{250} = 35.4$	CHECK	maximum deflection
$v\_inf\_p\left(\frac{L}{2}\right) = -12.06$	>	$\frac{-L}{250} = -35.4$	CHECK	maximum camber

$cnom\_p := cmin\_p + \Delta c\_dev = 36.75$ mm $\quad cnom\_p + \frac{\phi p}{2} = 43.1$

SLS STRESS CONTROL (§7.2)

$k1 := 0.6 \qquad rsup := 1.05$

$k2 := 0.45$ prestressing modification coefficients

$k3 := 0.8 \qquad rinf := 0.95$

$k4 := 1$

$k5 := 0.75$

$$\sigma cpg1\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for selfweight loads only

$$\sigma cpg1\_top(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id}$$

elastic stress of top concrete chord for selfweight loads only

$$\sigma cpf\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSf(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for frequent load combination

$$\sigma cpr\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for rare load combination

$$\sigma cpr\_top(x) := \frac{-Np\_tot \cdot rinf}{Aid} + \frac{[Mq\_SLSr(x) - rinf \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id}$$

elastic stress of top concrete chord for rare load combination

$$\sigma cpr\_p(x) := \sigma pm(x,t) \cdot rsup + 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(dp_{jp} - Yid\right)}{Ixo\_id} \right]$$

creep stress of bottom prestressing steel for rare load combination

$\sigma cpg1\_bot(lpt1) = -11.684$ $\quad > \quad$ $k1 \cdot (fcmj(2) + 8) = -13.578$ CHECK

$\quad > \quad$ $k2 \cdot fck = -20.25$ not compulsory in environment XC

$\sigma cpg1\_top(lpt1) = 0.683$ $\quad < \quad$ $fctmj(2) = 2.193$ CHECK

if not the element is assumed to be cracked after transfer of prestressing

$\sigma cpf\_bot\left(\frac{L}{2}\right) = -4.993$ $\quad < \quad$ $fctm = 3.795$ CHECK

$\sigma cpr\_bot\left(\frac{L}{2}\right) = -3.247$ $\quad < \quad$ $fctm = 3.795$

$\sigma cpr\_top\left(\frac{L}{2}\right) = -6.915$ $\quad > \quad$ $k1 \cdot fck = -27$ CHECK

$\quad > \quad$ $0.4 \cdot fcm = -21.2$

$\sigma cpr\_p\left(\frac{L}{2}\right) = 1.18 \times 10^3$ $\quad < \quad$ $k5 \cdot fptk = 1.395 \times 10^3$ CHECK

$\sigma cpg1\_bot(lpt1) = -11.684$ $\quad > \quad$ $k1 \cdot (fcmj(2) + 8) = -13.578$ CHECK

$\quad > \quad$ $k2 \cdot fck = -20.25$ not compulsory in environment XC

$\sigma cpg1\_top(lpt1) = 0.683$ $\quad < \quad$ $fctmj(2) = 2.193$ CHECK

if not the element is assumed to be cracked after transfer of prestressing

$\sigma cpf\_bot\left(\frac{L}{2}\right) = -4.993$ $\quad < \quad$ $fctm = 3.795$ CHECK

$\sigma cpr\_bot\left(\frac{L}{2}\right) = -3.247$ $\quad < \quad$ $fctm = 3.795$

$\sigma cpr\_top\left(\frac{L}{2}\right) = -6.915$ $\quad > \quad$ $k1 \cdot fck = -27$ CHECK

$\quad > \quad$ $0.4 \cdot fcm = -21.2$

$\sigma cpr\_p\left(\frac{L}{2}\right) = 1.18 \times 10^3$ $\quad < \quad$ $k5 \cdot fptk = 1.395 \times 10^3$ CHECK

SLS CRACK CONTROL (§7.3)

$$c\_act := Htot - dp_{jp} - 10 = 35$$

$$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1.5$$

$wlim\_cal := 0.2$ mm

$w\_freq := 0$ < $wlim\_cal = 0.2$ CHECK

7.10 ULS checks

ULS BENDING-AXIAL CONTROL (§6.1)

$Mrd = 387.63$ > $\dfrac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6} = 273.68$ CHECK

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§6.2)

$Vq\_ULS(x) := \left|(g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2} - x\right)\right|$ shear distribution at Ultimate Limit State (ULS)

$d := Yp = 220$ mm effective depth

$VEd := Vq\_ULS(d) = 1.175 \times 10^{5}$ N maximum shear at effective depth from support

$bw := 400$ mm web width

$z := 0.9\cdot d = 198$ mm conventional resultant lever arm

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT (§6.2.2)

$\rho l(x) := \min\left(0.02, \mathrm{if}\left(x < lpt2, \frac{Ap\_tot}{bw\cdot d}\cdot\frac{x}{lpt2}, \mathrm{if}\left(x > L - lpt2, \frac{Ap\_tot}{bw\cdot d}\cdot\frac{-x+L}{lpt2}, \frac{Ap\_tot}{bw\cdot d}\right)\right)\right)$ reinforcement ratio

$\sigma cp(x) := \sigma pm(x,t)\cdot\frac{Ap\_tot}{Ac}$ MPa axial load induced by prestressing

$\sigma cp(lpt2) = 4.102$ MPa after full transfer

$kv := \min\left(1 + \sqrt{\frac{200}{d}}, 2\right) = 1.909$

$k1v := 0.15$

$Crdc := \frac{0.18}{\gamma cpcred} = 0.129$

$vmin := 0.035\cdot k^{\frac{3}{2}}\cdot(-fck)^{\frac{1}{2}} = 0.61$ §6.3N

$bw\cdot d\cdot\left[Crdc\cdot kv\cdot(\right.$

$(vmin + k1v\cdot\sigma c$

$VRdc(x) := \max\left[\left[Crdc\cdot kv\cdot(100\cdot\rho l(x)\cdot -fck)^{\frac{1}{3}} + k1v\cdot\sigma cp(x)\right]\cdot bw\cdot d, (vmin + k1v\cdot\sigma cp(x))\cdot bw\cdot d\right]$ §6.2.a+§6.2.b

(valid for hollowcores only if additional support mild reinforcement is employed)

$$VRdc\_HC(x) := \frac{Ixo\_id \cdot bw}{Sxid} \cdot \sqrt{fctd^2 + \min\left(\frac{x}{lpt2}, 1\right) \cdot \sigma cp(x) \cdot fctd}$$

$VRdc\_HC(1000) = 9.175 \times 10^4$ N for hollow core members (§6.4)

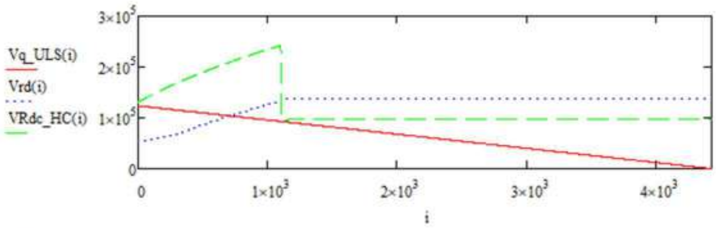
$bw\_add := 186 \cdot 4$ mm web width considering that 2 holes per end in a single HC member are filled with C30_37 concrete

$fctd\_C30\_37 := 1.448$ MPa design tensile strength of class C30_37 concrete

$$VRdc\_HC\_mod(x) := VRdc\_HC(x) + \frac{Ixo\_id \cdot (bw\_add)}{Sxid} \cdot \sqrt{fctd\_C30\_37^2 + \min\left(\frac{x}{lpt2}, 1\right) \cdot \sigma cp(x) \cdot fctd\_C30\_37}$$

modified shear resistance formula accounting for the hole filling

$VRdc\_HC(x) := if(x < 1100, VRdc\_HC\_mod(x), VRdc\_HC(x))$ actual shear resistance distribution assuming the filling stops at 1.1 m



$$\max\left[Crdc \cdot kv \cdot (100 \cdot \rho l(d) \cdot -fck)^{\frac{1}{3}}, vmin\right] + \left[\frac{VRdc(1000)}{bw \cdot d} - \max\left[Crdc \cdot kv \cdot (1\right.\right.$$

NOTE: the calculation above does account for the prescriptions included in EN1992-1-1:2004 only, neglecting those contained in the product standard EN1168:2005+A3:2011

MOMENT DIAGRAM ACCOUNTING FOR PRESTRESSING TRANSFER

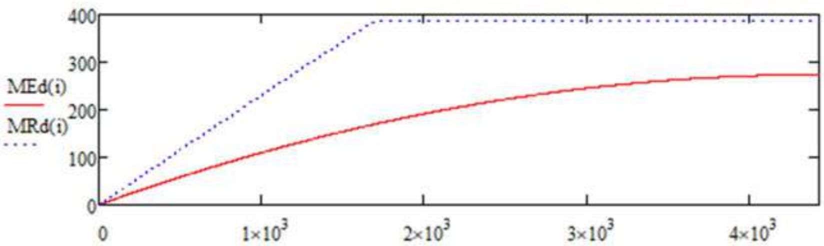
$\eta p2 := 1.2$

$fbpd := \eta p2 \cdot \eta 1 \cdot fctd = 2.277$ MPa given fck < 60 MPa

$$lbpd := lpt2 + \frac{\alpha 2 \cdot \left(fptd - \sigma pm\left(\frac{L}{2}, t\right)\right)}{fbpd} \cdot \phi p = 1.698 \times 10^3 \quad \text{mm}$$

$$MRd(x) := if\left[x < lpt2, Mrd \cdot \frac{\sigma pm(lpt2, 365 \cdot 50)}{fptd} \cdot \frac{x}{lpt2}, if\left[x > lbpd, Mrd, Mrd \cdot \frac{\sigma pm(lpt2, 365 \cdot 50)}{fptd} + \frac{(x - lpt2)}{(lbpd - lpt2)} \cdot \left(Mrd - Mrd \cdot \frac{\sigma pm(lpt2, 365 \cdot 50)}{fptd}\right)\right]\right]$$

$$MEd(x) := \frac{Mq\_ULS(x)}{10^6}$$



NOTE: no bending moment diagram translation was introduced, since the member is designed to not crack in shear, and not following the typical resistance mechanism for beam members not provided with transverse reinforcement

MINIMUM REINFORCEMENT

$bt := b(Htot) = 2.4 \times 10^3$

§9.1N

§9.2.1.1(3)

$Asmin := max\left(0.26 \cdot \frac{fctm}{fsk} \cdot bt \cdot d, 0.0013 \cdot bt \cdot d\right) = 1.042 \times 10^3$ mm2 < $\rho l\left(\frac{L}{2}\right) \cdot Ac = 4.011 \times 10^3$ mm2 CHECK < $0.04 \cdot Ac = 1.265 \times 10^4$ CHECK

CHECK OF SUPPORT MILD REBARS (§9.2.1.4(1))

$0.25 \cdot Mrd = 96.907$ kNm < $\left(4 \cdot \pi \cdot \frac{16^2}{4}\right) \cdot fsd \cdot 0.9 \cdot \frac{550}{10^6} = 180.956$ CHECK

area of support mild steel

ANCHORAGE (§8.4)

$\eta 1 = 1$

$\eta 2 := 1$

$Vq\_ULS(0) = 1.237 \times 10^5$

$fbd := 2.25 \cdot \eta 1 \cdot \eta 2 \cdot fctd\_C30\_37 = 3.258$

$lbrqd(\phi) := \frac{\phi}{4} \cdot \frac{fsd}{fbd}$

$\alpha 1b := 1$

$\alpha 2b := 1$

$\alpha 3b := 1$

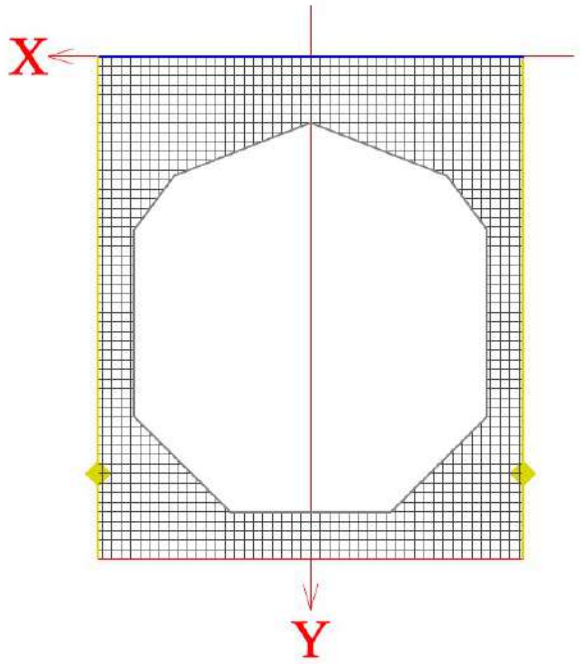
$\alpha 4b := 1$

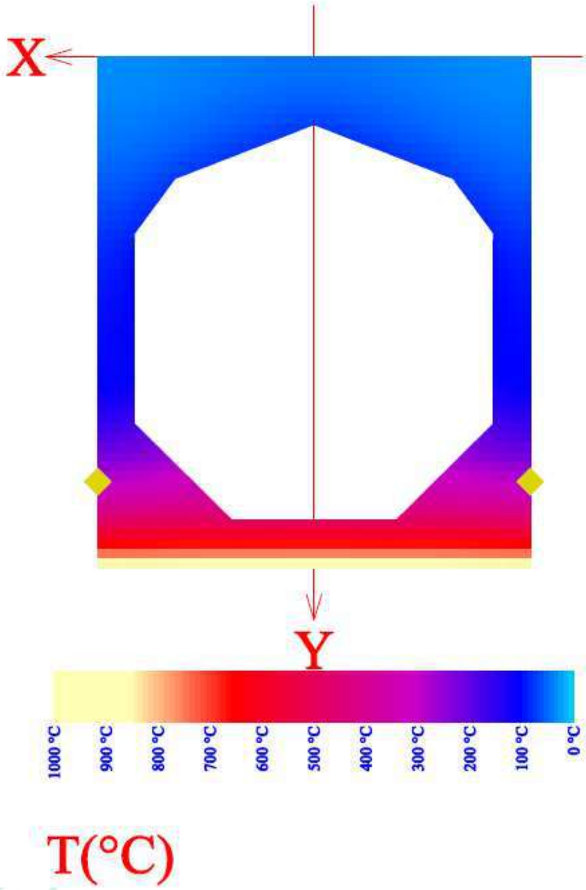
$\alpha 5b := 1$

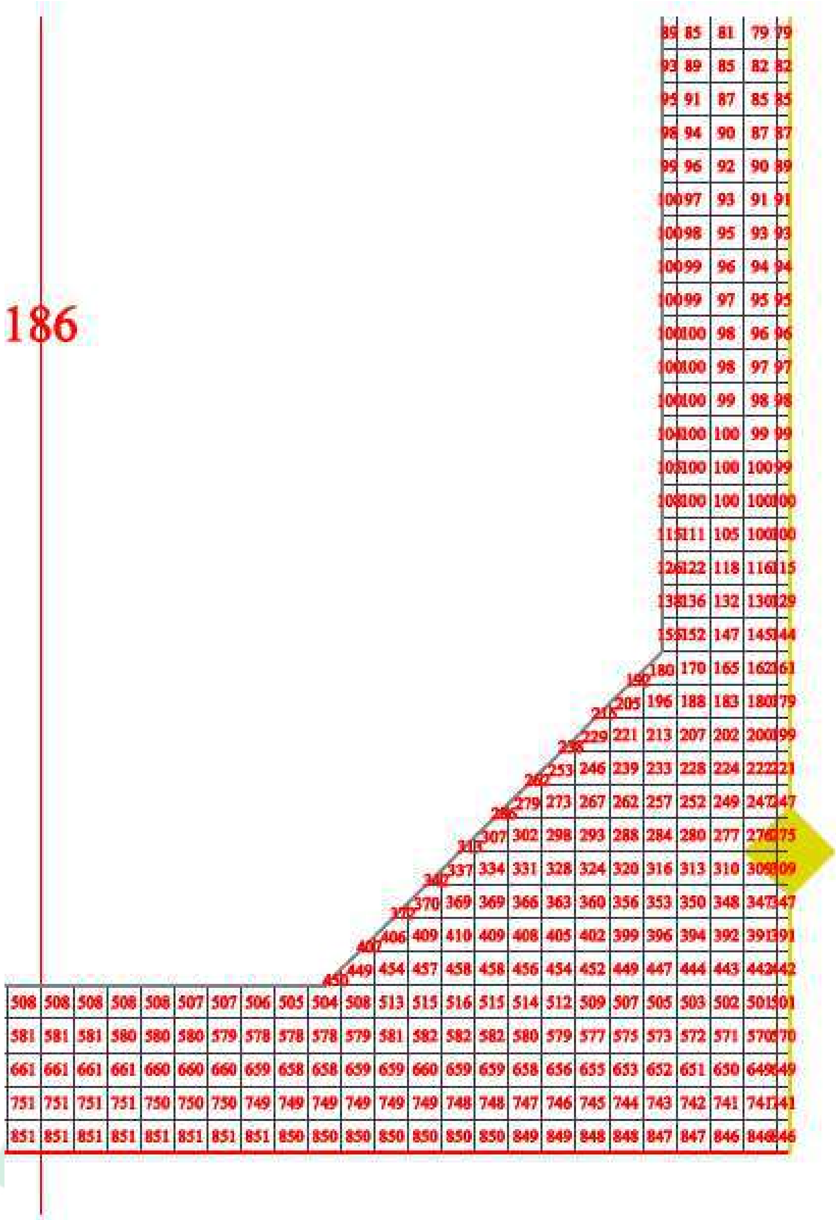
$lbd(\phi) := \alpha 1b \cdot \alpha 2b \cdot \alpha 3b \cdot \alpha 4b \cdot \alpha 5b \cdot lbrqd(\phi)$

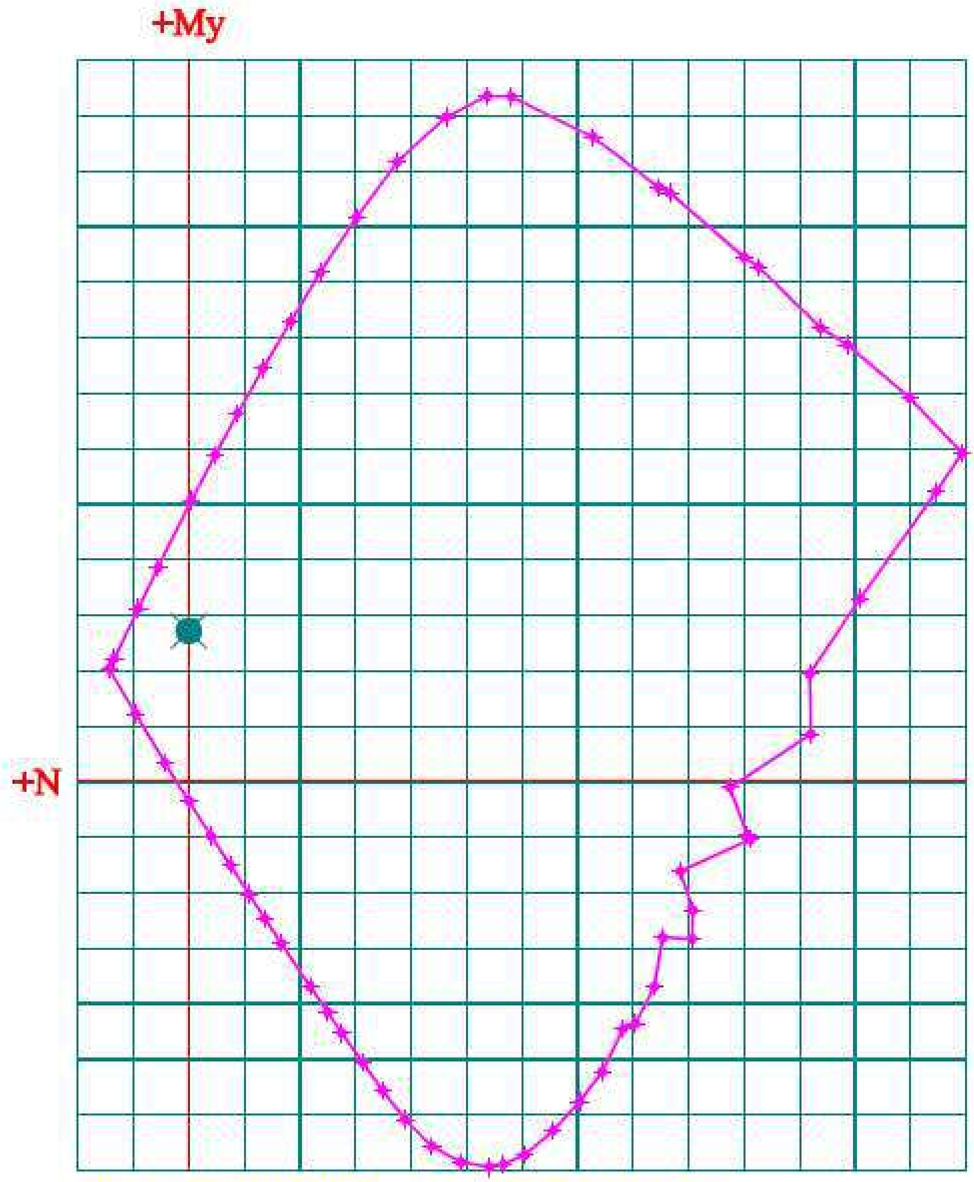
$lbd(16) = 558.067$ $\frac{lbd(12)}{12} = 34.879$

7.11 Fire checks






186

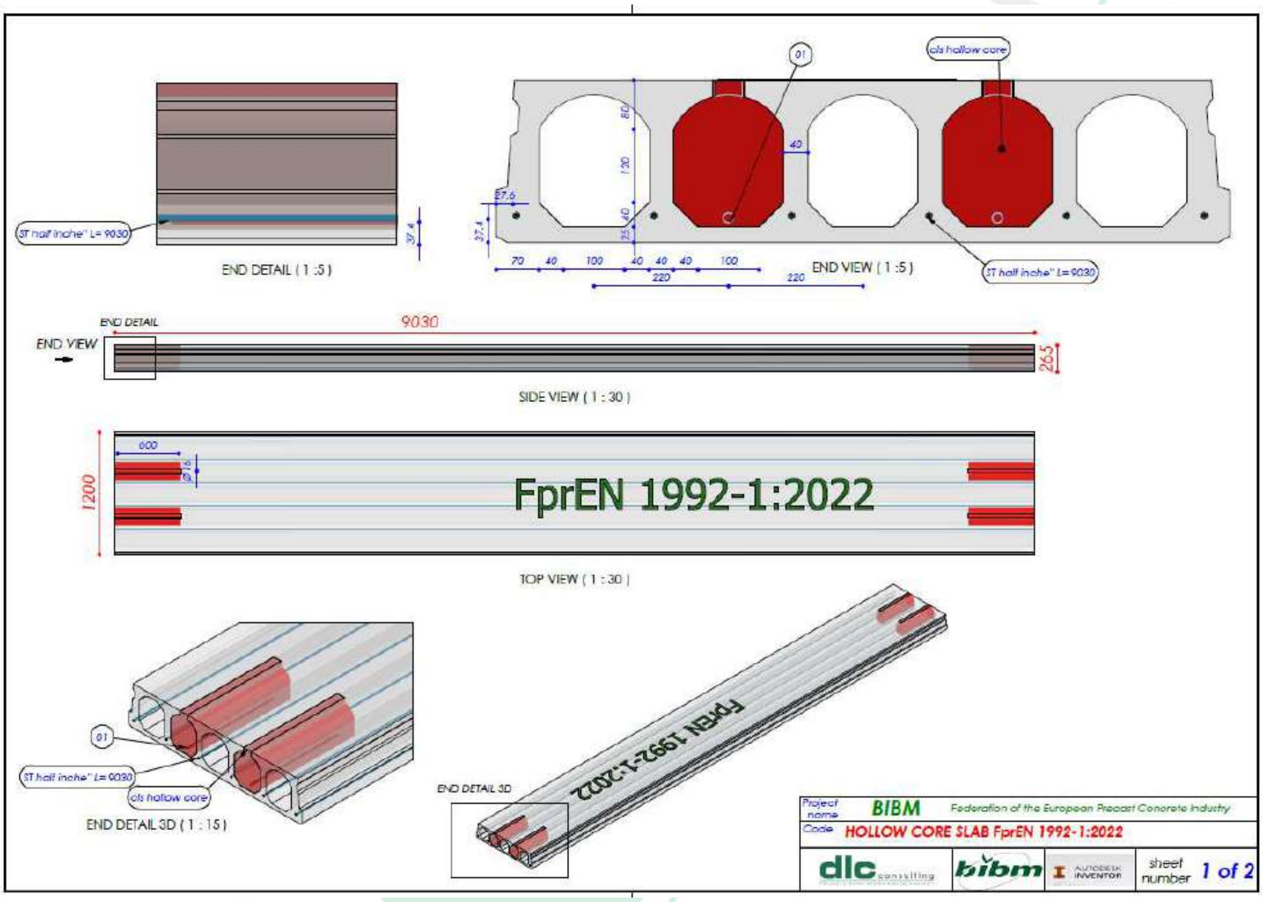


N: 1 sp = 81.00 kN;   M: 1 sp = 5.00 kN·m

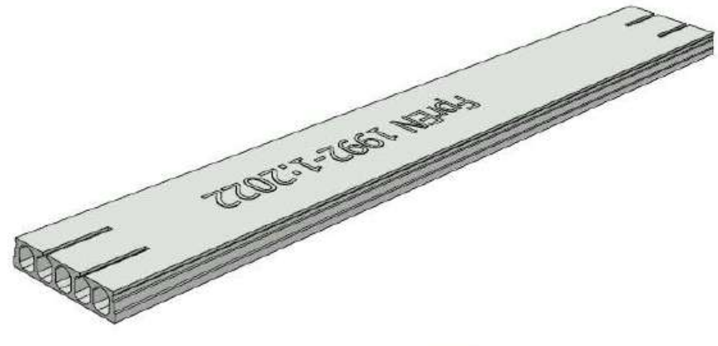
Nz = 0.00 kN
My+ = 24.86 kN·m
My- = -1.69 kN·m

8 Hollowcore element – FprEN1992-1:2022

8.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_		
	01	4	947	3788	16 mm		
	Total mass rebars [kg]			3,79	Incidence kg/m³	1,77	
	ST half inche" L= 9030	6	6599	39594	12.7 mm		
	Total mass strands [kg]			39,594	Incidence kg/m³	18,47	
	Total mass of steel [kg]			43,38		Concrete volume [m³]	1,224
						Cast in situ [m³]	0,92
						Total concrete [m³]	2.144



Project name	BIBM	Federation of the European Precast Concrete Industry
Code	HOLLOW CORE SLAB FprEN 1992-1:2022	
dlc consulting / bibm / AUTODESK INVENTOR	sheet number	2 of 2

8.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0\ \ 42.5\ \ 106.1\ \ 106.11\ \ 189.41\ \ 240\ \ 240.1\ \ 265)^T$

$Htot := max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := 4\cdot(582\ \ 578\ \ 398\ \ 112.8\ \ 124.3\ \ 378\ \ 598\ \ 600)^T$

$r\_circ := 126$ radius of central void pipe

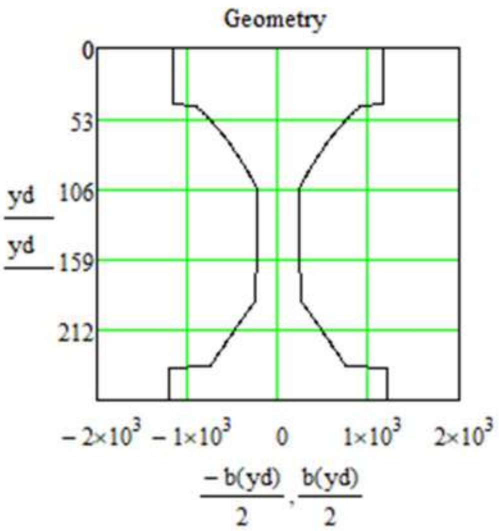
$x\_circ(y) := 2\sqrt{r\_circ^2 - (y - 160)^2}$

$b\_lin(y) := linterp(y\_tr, b\_tr, y)$

$b\_circ(y) := linterp(y\_tr, b\_tr, y) - 5\cdot x\_circ(y)$

$b(y) := if(y \le 106.1 \wedge y \ge 42.5, b\_circ(y), b\_lin(y))$

$yd := 0..Htot$ $b(0) = 2.328 \times 10^3$



condensed 1D geometry plot

$u := 2400\cdot2 + Htot\cdot2 + 10\cdot160\cdot\pi = 1.036 \times 10^4$ mm exposed perimeter

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0\ \ 42.5\ \ 106.1\ \ 106.11\ \ 189.41\ \ 240\ \ 240.1\ \ 265)^T$

$Htot := \max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := 4\cdot(582\ \ 578\ \ 398\ \ 112.8\ \ 124.3\ \ 378\ \ 598\ \ 600)^T$

$r\_circ := 126$ radius of central void pipe

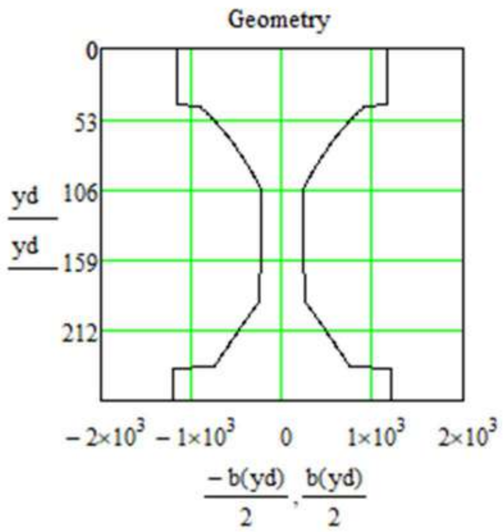
$$x\_circ(y) := 2\sqrt{r\_circ^2 - (y - 160)^2}$$

$b\_lin(y) := linterp(y\_tr, b\_tr, y)$

$b\_circ(y) := linterp(y\_tr, b\_tr, y) - 5\cdot x\_circ(y)$

$b(y) := if(y \le 106.1 \wedge y \ge 42.5, b\_circ(y), b\_lin(y))$

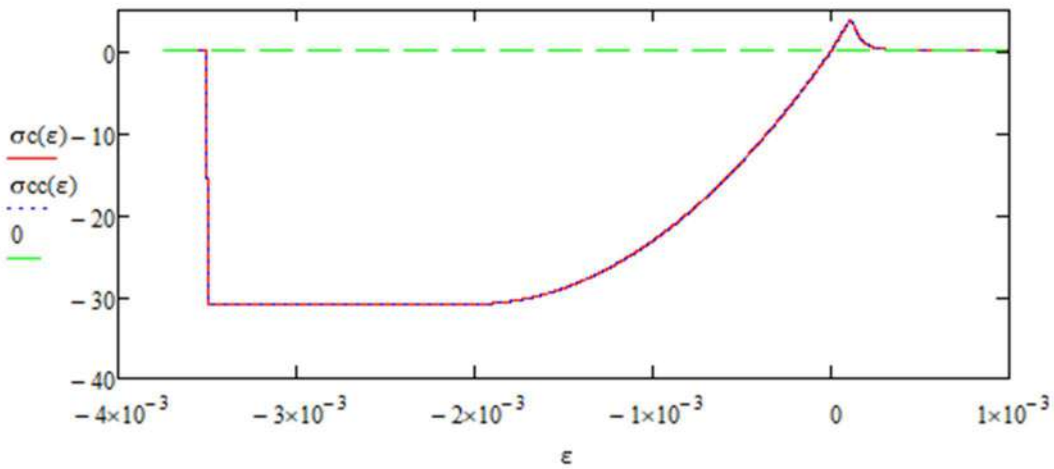
$yd := 0..Htot$ $b(0) = 2.328 \times 10^3$

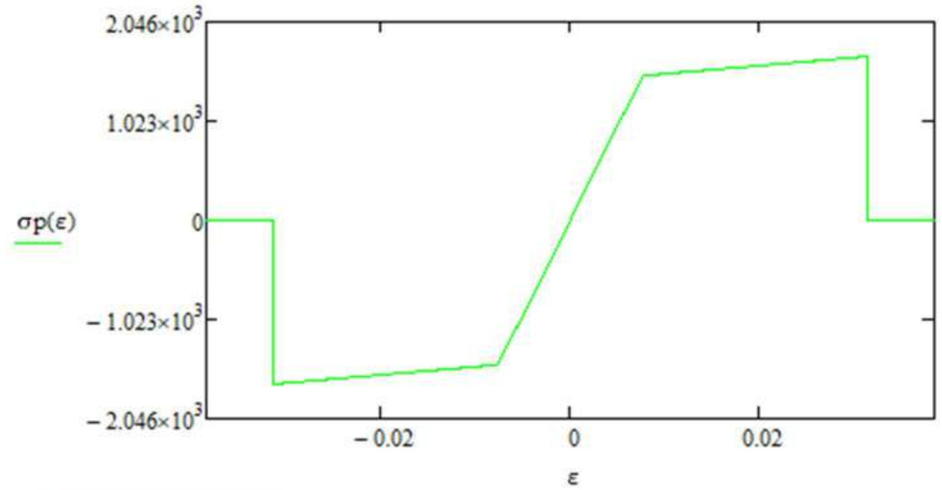


condensed 1D geometry plot

$u := 2400\cdot2 + Htot\cdot2 + 10\cdot160\cdot\pi = 1.036 \times 10^4$ mm exposed perimeter

8.3 Material constitutive laws employed in the calculation





8.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 3.163 \times 10^5$$

$$\rho p := \frac{Ap\_tot}{Ac} = 3.529 \times 10^{-3}$$ geometric ratio for longitudinal prestressing tendons

$$\rho tot := \frac{Ap\_tot}{Ac} = 3.529 \times 10^{-3}$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y)\cdot y\,dy \qquad Syc = 3.878 \times 10^7$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 122.619$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y)\cdot(y - yG)^2\,dy \qquad Ixo\_cls = 2.799 \times 10^9$$

Global area of all prestressing reinforcement

$$Area\_tr := \left| \begin{array}{l} s \leftarrow 0 \\ \text{for } x \in 1..jp \\ \quad s \leftarrow Ap_x + s \end{array} \right. \qquad Area\_tr = 1.116 \times 10^3$$

First moment of the area referred to prestressing reinforcement only

$$Sxp := \sum_{i=1}^{jp} \left(Ap_i \cdot dp_i\right) \qquad Sxp = 2.455 \times 10^5$$

Centre of gravity of prestressing

$$Yp := \frac{Sxp}{Area\_tr} \qquad Yp = 220$$

Idealisation coefficients (elastic)

$$np := \frac{Ep}{Ecm} \qquad np = 5.465$$

$$ns := \frac{Es}{Ecm} \qquad ns = 5.605$$

Area of ideal cross-section

$$Aid := Ac + (np - 1)\cdot\sum_{j=1}^{jp} Ap_j \qquad Aid = 3.213\times10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac\cdot yG + (np - 1)\cdot(Area\_tr\cdot Yp) \qquad Sxid = 3.988\times10^7$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 124.129$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y)\cdot(y - Yid)^2\,dy - \sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right]$$

Second moment of the prestressing reinforcement area

$$Ixoidprec := np\cdot\sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidprec \qquad Ixo\_id = 2.845\times10^9 \ \text{mm\^{}4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.017$$

8.5 Loads

8.6 Prestressing transfer and time-dependent behaviour

TRANSFER OF PRESTRESS (§13.5.3)

$\alpha 1 := 1$ gradual release of prestressing

$\alpha 2 := 0.26$ for 7-wire strands

$\sigma pm0 := \sigma p0 = 1.4 \times 10^{3}$

$\eta 1 := 1$ in favourable position

$lpt := \frac{\gamma c}{1.5} \cdot \frac{\alpha 1 \cdot \alpha 2 \cdot \sigma pm0}{\eta 1 \cdot \sqrt{(-fcmj(2) - 8)}} \cdot \phi p = 906.996$ mm basic value of the transmission length following §(13.4)

$lpt1 := 0.8lpt = 725.597$ mm lower-bound transfer length following §(13.6)

$lpt2 := 1.2 \cdot lpt = 1.088 \times 10^{3}$ mm upper-bound transfer length following §(13.7)

Prestress losses

$hn := 2 \cdot \frac{Ac}{u} = 61.076$

$A := 0.79 + \frac{(hn - 200)}{(500 - 200)} \cdot (0.75 - 0.79) = 0.809$

$\varepsilon cs := \frac{0.65}{1000} = 6.5 \times 10^{-4}$ shrinkage strain assumed as a result of laboratory tests on the specific concrete mix employed

$\rho 1000 := 0.025$ for class 2 (low-relaxation) tendons

$kp := 0.16$

$t := 50 \cdot 365 = 1.825 \times 10^{4}$ days Life span

$\sigma cpQP2(x) := \frac{-Np\_tot}{Aid} + \frac{[Mq\_SLSg1(x) - Np\_tot \cdot (Yp - Yid)] \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP2\left(\frac{L}{2}\right) = -7.302$

stress in quasi-permanent load combination at 2 days
(conventional equivalent time for prestressing release)

$\sigma cpQP23(x) := \frac{Mq\_SLSg2(x) \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP23\left(\frac{L}{2}\right) = 1.584$

stress in quasi-permanent load combination at 23 days
(conventional time for assemblage of the structure on site)

$\sigma cpQP91(x) := \frac{Mq\_SLSq(x) \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP91\left(\frac{L}{2}\right) = 0.713$

stress in quasi-permanent load combination at 91 days
(conventional time for enter in use of the structure)

$$\Delta\sigma pr(x,t) := \left[\sigma p0 + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x))\right] \cdot \rho 1000 \cdot \left(\frac{24 \cdot t}{1000}\right)^{kp}$$

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$$RH := 50$$

$$t0\_adj(t0) := t0$$

$$\beta bc\_fcm := \frac{1.8}{(-fcm)^{0.7}} = 0.112 \qquad \beta bc\_t\_t0(t,t0) := \ln\left[\left(\frac{30}{t0\_adj(t0)} + 0.035\right)^2 \cdot (t - t0) + 1\right]$$

$$\beta dc\_fcm := \frac{412}{(-fcm)^{1.4}} = 1.588$$

$$\beta dc\_RH := \frac{1 - \frac{RH}{100}}{\sqrt[3]{0.1 \cdot \frac{hn}{100}}} = 1.27$$

$$\beta dc\_t0(t0) := \frac{1}{0.1 + t0\_adj(t0)^{0.2}}$$

$$\gamma(t0) := \frac{1}{2.3 + \frac{3.5}{\sqrt{t0\_adj(t0)}}}$$

$$\alpha cm := \left(\frac{35}{-fcm}\right)^{0.5} = 0.813$$

$$\beta h := \min(1.5 \cdot hn + 250 \cdot \alpha cm, 1500 \cdot \alpha cm) = 294.774$$

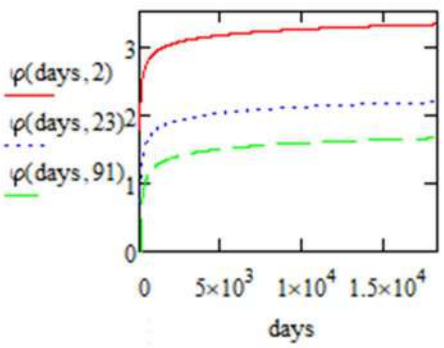
$$\beta dc\_t\_t0(t,t0) := \left[\frac{(t - t0)}{\beta h + (t - t0)}\right]^{\gamma(t0)}$$

$$\varphi dc(t,t0) := \beta dc\_fcm \cdot \beta dc\_RH \cdot \beta dc\_t0(t0) \cdot \beta dc\_t\_t0(t,t0)$$

$$\varphi bc(t,t0) := \beta bc\_fcm \cdot \beta bc\_t\_t0(t,t0)$$

$$\varphi(t,t0) := \varphi bc(t,t0) + \varphi dc(t,t0)$$

$$\varphi(t,2) = 3.312 \qquad \varphi(t,91) = 1.652$$



TIME-DEPENDENT LOSSES OF PRESTRESS (§7.6.4)

$$\Delta\sigma p\_csr(x,t) := \frac{-\varepsilon cs \cdot Ep - 0.8 \cdot \Delta\sigma pr(x,t) + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) \cdot \varphi(t,2) + \sigma cpQP23(x) \cdot \varphi(t,23) + \sigma cpQP91(x) \cdot \varphi(t,91))}{1 + \frac{Ep}{Ecm} \cdot \frac{Ap\_tot}{Ac} \cdot \left[1 + \frac{Ac}{Ixoidcls} \cdot (Yp - Yid)^2\right] \cdot \left(1 + 0.8 \cdot \frac{\varphi(t,2) \cdot \sigma cpQP2(624) + \varphi(t,23) \cdot \sigma cpQP23(624) + \varphi(t,91) \cdot \sigma cpQP91(624)}{\sigma cpQP2(624) + \sigma cpQP23(624) + \sigma cpQP91(624)}\right)}$$

prestress losses following §(7.35)

NOTE: a weighed creep coefficient was considered accounting for the 3 load phases previously introduced

$$\sigma pm(x,t) := \sigma p0 - \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)) + \Delta\sigma p\_csr(x,t)$$ prestress considering immediate and delayed losses

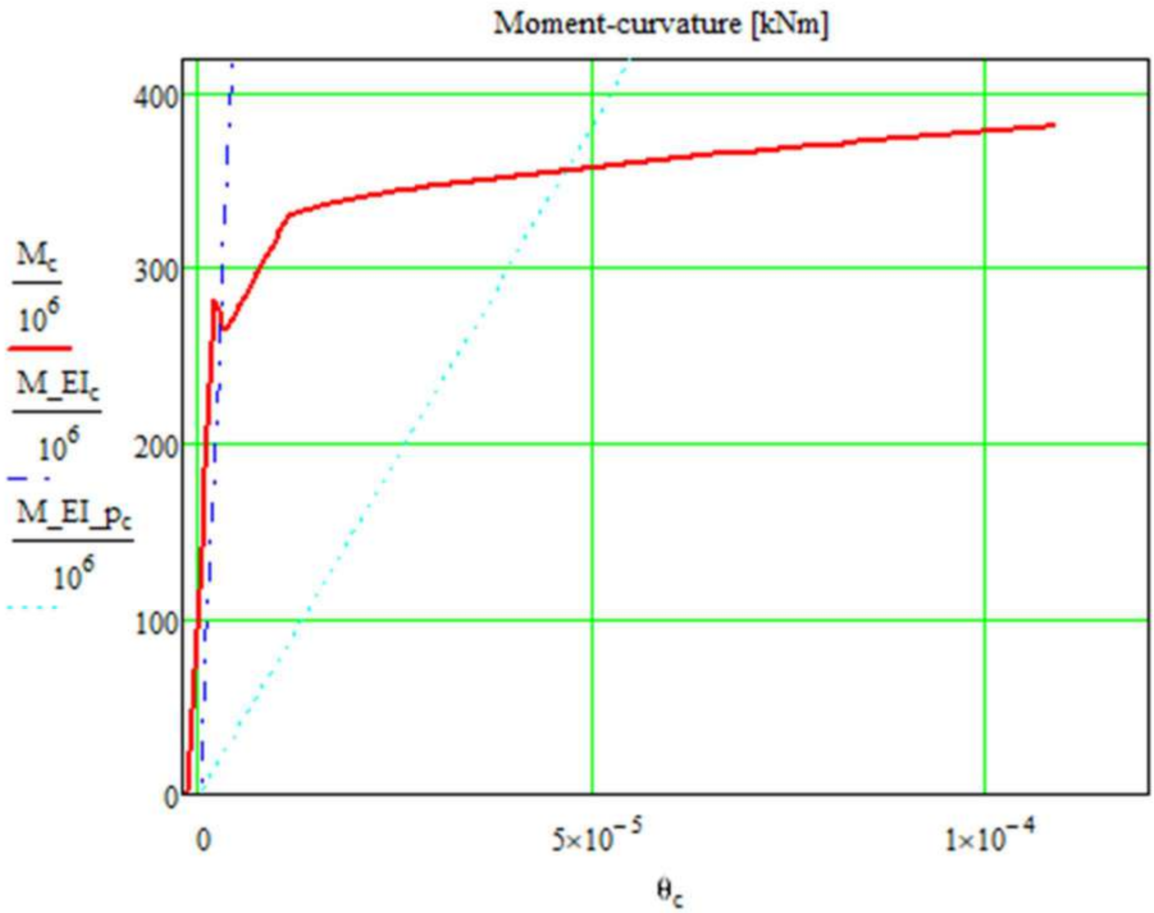
$$\frac{\sigma pm\left(\frac{L}{2},t\right)}{\sigma p0} = 0.829$$ expected residual prestress ratio after 50 years of life with respect to initial

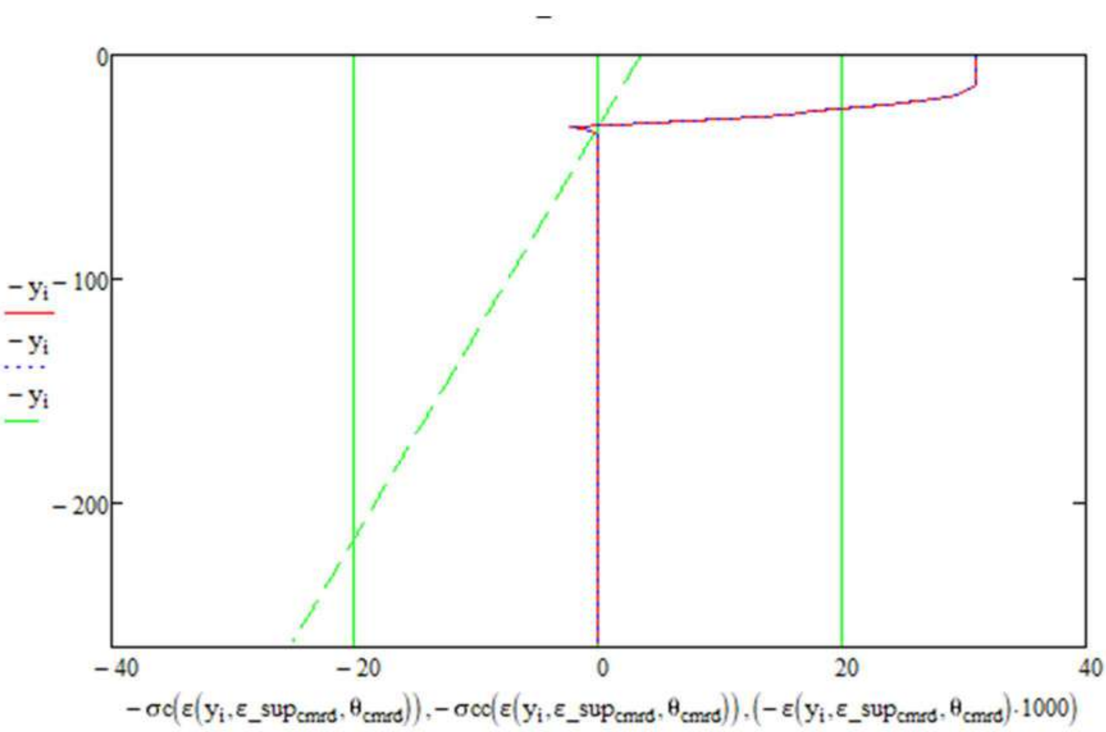
$$\varepsilon pm := \frac{\sigma pm\left(\frac{L}{2},t\right)}{\sigma p0} \cdot \varepsilon p0$$ expected residual strain after 50 years of life with respect to initial

$$\sigma pm\left(\frac{L}{2},t\right) \cdot Ap\_tot = 1.295 \times 10^6$$ N residual prestress force after 50 years of life

$Np\_tot = 1.562 \times 10^6$ N initial prestress force

8.7 Non-linear moment-curvature diagram





Condition at resisting (peak) moment
(stress and strain)

8.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$$
moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformly distributed load q

$$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$$
moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformly distributed load q

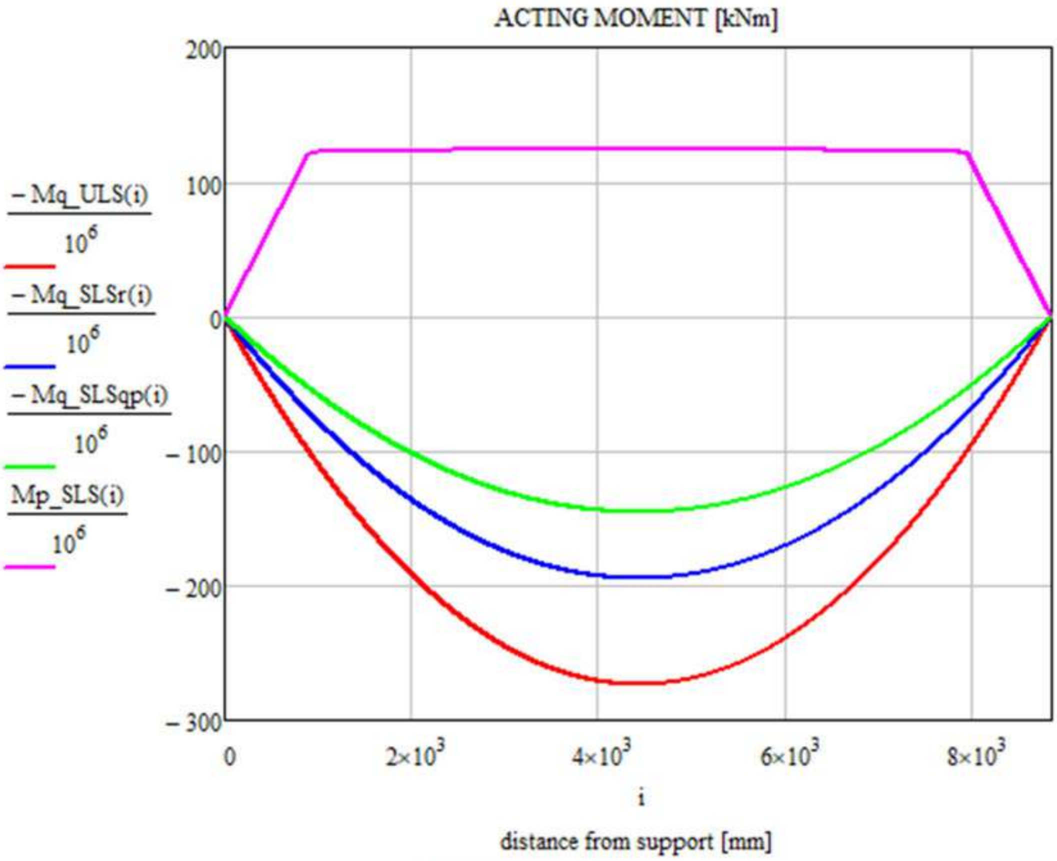
$$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$$
moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformly distributed load q

lpt = ▪

$$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$$
moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformly distributed load q

$$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$$
moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformly distributed load q

$$Mp\_SLS(x) := \text{if}\left[x \le lpt, \sigma pm(x,t) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{x}{lpt}, \text{if}\left[x \ge L - lpt, \sigma pm(x,t) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{-x + L}{lpt}, \sigma pm(x,t) \cdot Ap\_tot \cdot (Yp - Yid)\right]\right]$$
contribution of prestressing equivalent load in SLS (without modification factors)



8.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$$v\_inf\_p(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(1 + \varphi(t,23))}{1.05}$$

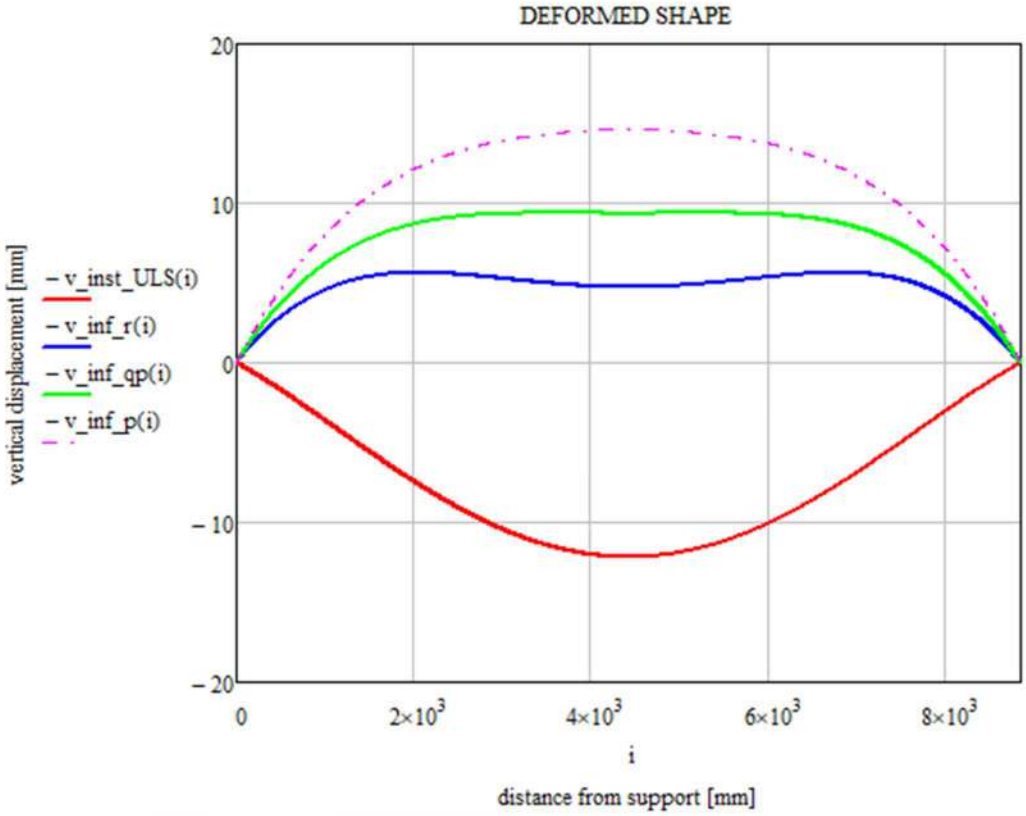
deflection profile at 50 years including creep for permanent load combination

$$v\_inf\_qp(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(\varphi(t,23) - \varphi(t,91)) + v\_SLSqp(x)\cdot(1 + \varphi(t,91))}{1.05}$$

deflection profile at 50 years including creep for quasi permanent load combination

$$v\_inf\_r(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(\varphi(t,23) - \varphi(t,91)) + v\_SLSqp(x)\cdot\varphi(t,91) + v\_SLSr(x)}{1.05}$$

deflection profile at 50 years including creep for rare load combination



SLS STRESS CONTROL (§9.2.1)

$k1 := 0.6$ $\quad$ $rsup := 1.05$

$k2 := 0.45$ $\quad$ prestressing modification coefficients

$k3 := 0.8$ $\quad$ $rinf := 0.95$ $\quad$ $Np\_tot = 1.562 \times 10^{6}$

$k4 := 1$

$k5 := 0.8$ $\quad$ 0.75 in EN1992-1-1:2002

NOTE: the denomination of the allowable stress coefficients following k factors was kept similar to that of EN1992-1-1:2002

$$\sigma cpg1\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpg1\_bot(lpt1) = -1$$

elastic stress of bottom concrete chord for selfweight loads only

$$\sigma cpg1\_top(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id} \qquad \sigma cpg1\_top(lpt1) = 0.$$

elastic stress of top concrete chord for selfweight loads only

$$\sigma cpf\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSf(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpf\_bot\left(\frac{L}{2}\right) = -4.9$$

elastic stress of bottom concrete chord for frequent load combination

$$\sigma cpr\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpr\_bot\left(\frac{L}{2}\right) = -3.2$$

elastic stress of bottom concrete chord for rare load combination

$$\sigma cpr\_top(x) := \frac{-Np\_tot \cdot rinf}{Aid} + \frac{[Mq\_SLSr(x) - rinf \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id} \qquad \sigma cpr\_top\left(\frac{L}{2}\right) = -6.9$$

elastic stress of top concrete chord for rare load combination

$$\sigma cpr\_p(x) := \sigma pm(x,t) \cdot rsup + 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(dp_{jp} - Yid\right)}{Ixo\_id}\right] \qquad \sigma cpr\_p\left(\frac{L}{2}\right) = 1.161 \times$$

creep stress of bottom prestressing steel for rare load combination

$\sigma cpg1\_bot(lpt1) = -11.74$ $\quad > \quad$ $k1 \cdot \beta cc(2)^{\frac{2}{3}} \cdot fck = -18.733$ CHECK

$> \quad k2 \cdot fck = -20.25$

$\sigma cpg1\_top(lpt1) = 0.739$ $\quad < \quad$ $fctmj(2) = 2.731$

$\sigma cpf\_bot\left(\frac{L}{2}\right) = -4.989$ $\quad < \quad$ $fctm = 3.795$ CHECK

$\sigma cpr\_bot\left(\frac{L}{2}\right) = -3.244$ $\quad < \quad$ $fctm = 3.795$

$\sigma cpr\_top\left(\frac{L}{2}\right) = -6.915$ $\quad > \quad$ $k1 \cdot fck = -27$ CHECK

$> \quad 0.4 \cdot fcm = -21.2$

$\sigma cpr\_p\left(\frac{L}{2}\right) = 1.161 \times 10^{3}$ $\quad < \quad$ $k5 \cdot fptk = 1.488 \times 10^{3}$ CHECK

SLS CRACK CONTROL (§9.2.3)

$c\_act := Htot - dp_{jp} - 10 = 35$

$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1.5$

$wlim\_cal := 0.2 \cdot ksurf = 0.3$ mm

$w\_freq := 0$ < $wlim\_cal = 0.3$ CHECK

8.10 ULS checks

ULS BENDING-AXIAL CONTROL (§8.1)

$Mrd = 380.537$ > $\frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6} = 273.68$ CHECK

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§8.2)

$Vq\_ULS(x) := \left|(g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} - x\right)\right|$ shear action distribution at Ultimate Limit State (ULS)

$d := Yp = 220$ mm effective depth of cross-section

$VEd := Vq\_ULS(d) = 1.175 \times 10^5$ N design shear action at control section at distance d from support

$\gamma v := 1.3$ safety factor for initial shear check

$bw := 400$ mm design web width

$z := 0.9 \cdot d = 198$ conventional lever arm of internal stress resultants

$\tau Ed := \frac{VEd}{bw \cdot z} = 1.484$ MPa equivalent mean acting shear stress on control cross-section

$Dlower := 16$ mm maximum aggregate diameter following assumed mix design

$ddg := \min\left[\mathrm{if}\left[-fck > 60, 16 + Dlower \cdot \left(\frac{60}{-fck}\right)^2, 16 + Dlower\right], 40\right] = 32$ size parameter

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT (§8.2.2)

NOTE: not proper for hollowcore members unless provided by additional longitudinal end reinforcement

$$\tau Rdc\_min(x) := \frac{11}{\gamma v}\cdot\sqrt{\frac{-fck}{(fptd - \sigma pm(x,t))}\cdot\frac{ddg}{d}}$$ §(8.20)

$\tau Rdc\_min(d) = 0.913$ MPa not checked with τEd -> detailed evaluation is mandatory following §8.2.1

$$\rho l(x) := if\left(x < lpt2, \frac{Ap\_tot}{bw\cdot d}\cdot\frac{x}{lpt2}, if\left(x > L - lpt2, \frac{Ap\_tot}{bw\cdot d}\cdot\frac{-x+L}{lpt2}, \frac{Ap\_tot}{bw\cdot d}\right)\right)$$ longitudinal geometric reinforcement ratio §(8.28)

$ep := Yp - Yid = 95.871$ mm eccentricity of prestressing

$$acs\_0(x) := max\left(\frac{Mq\_ULS(x)}{Vq\_ULS(x)}, d\right)$$ §(8.30) accounting for comments in §8.2.2(5)

$$k1(x) := min\left[\frac{0.5}{acs\_0(x)}\cdot\left(ep + \frac{d}{3}\right)\cdot\frac{Ac}{bw\cdot z}, 0.18\cdot\frac{Ac}{bw\cdot z}\right]$$ §(8.34)

$$av\_0(x) := \sqrt{\frac{acs\_0(x)}{4}\cdot d}$$ §(8.29) accounting for comments in §8.2.2(5)

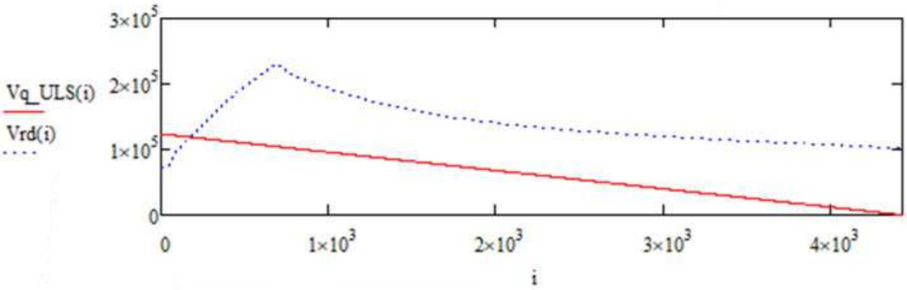
$$\tau Rdc\_0(x) := \frac{0.66}{\gamma v}\cdot\left(100\cdot\rho l(x)\cdot -fck\cdot\frac{ddg}{av\_0(d)}\right)^{\frac{1}{3}}$$ §(8.33)

$$\tau Rdcmax(x) := min\left[2.15\cdot\tau Rdc\_0(x)\cdot\left(\frac{acs\_0(x)}{d}\right)^{\frac{1}{6}}, 2.7\cdot\tau Rdc\_0(x)\right]$$ §(8.35)

$$\sigma cp(x) := \sigma pm(x,t)\cdot\frac{Ap\_tot}{Ac}$$ §(8.33)

$$\tau Rdc(x) := max(min(\tau Rdc\_0(x) + k1(x)\cdot\sigma cp(x), \tau Rdcmax(x)), \tau Rdc\_min(x))$$ §(8.32)

$Vrd(x) := bw\cdot z\cdot\tau Rdc(x)$



$acs(x) := acs\_0(x)$ av(:

$kvp(x) := min\left(1 + \frac{\sigma pm(x,t)\cdot}{Vq\_UI}\right.$

$Vrd\_2(x) := max\left[bw\cdot z\cdot\left[\frac{0.66}{\gamma v}\cdot\right.\right.$

SHEAR RESISTANCE OF PRECAST MEMBERS WITHOUT SHEAR REINFORCEMENT (§13.5.5)

$$\tau Ed\_HC(x) := Vq\_ULS(x)\cdot\frac{Sxid}{bw\cdot Ixo\_id} \qquad \tau Ed\_HC(0) = 4.334 \quad \text{MPa}$$

$$\sigma 1Ed(x) := \frac{\sigma pm(0,t)\cdot\frac{Ap\_tot}{Ac}}{2} + \sqrt{\left(\frac{\sigma pm(0,t)\cdot\frac{Ap\_tot}{Ac}}{2}\right)^2 + \tau Ed\_HC(x)^2} \qquad \sigma 1Ed(1000) = 6.359 \quad \text{MPa} < \quad fctd = 1.518 \quad \text{MPa} \quad \text{CHECK}$$

$$\sigma cpx(x) := \sigma pm(x,t)\cdot\frac{Ap\_tot}{Ac} \qquad \sigma cpx(0) = 3.97$$

hollow core members only

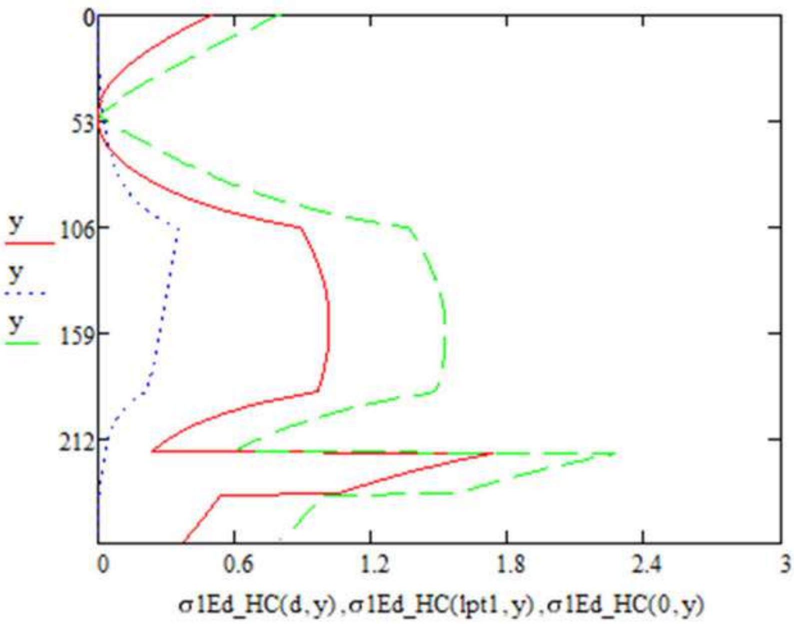
$$\sigma xEd(x,y) := \text{if}\left[x < lpt2, \frac{-Np\_tot\cdot rsup\cdot\frac{x}{lpt2}}{Aid} + \frac{\left[Mq\_ULS(x) - rsup\cdot Np\_tot\cdot\frac{x}{lpt2}\cdot(Yp - Yid)\right]\cdot(y - Yid)}{Ixo\_id}, \frac{-Np\_tot\cdot rsup}{Aid} + \frac{[Mq\_ULS(x) - rsup\cdot Np\_tot\cdot(Yp - Yid)]\cdot(y - Yid)}{Ixo\_id}\right]$$

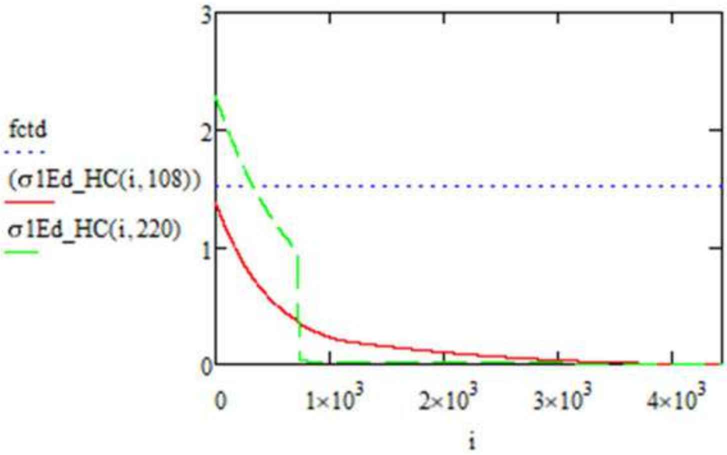
$$\tau cp(x,y) := \text{if}\left[x < lpt1, \text{if}\left[y < Yp, \frac{1}{b(y)}\cdot\left[\frac{\int_0^y b(y)\,dy}{Ac} - \frac{\int_0^y b(y)\cdot(yG - y)\,dy\cdot(yG - Yp)}{Ixo\_cls} + -1\right]\cdot rsup\cdot\sigma pm\left(\frac{L}{2},t\right)\cdot\frac{Ap\_tot}{lpt1}, \frac{1}{b(y)}\cdot\left[\frac{\int_0^y b(y)\,dy}{Ac} - \frac{\int_0^y b(y)\cdot(yG - y)\,dy\cdot(yG - Yp)}{Ixo\_cls}\right]\cdot rsup\cdot\sigma pm\left(\frac{L}{2},t\right)\cdot\frac{Ap\_tot}{lpt1}\right], 0\right]$$

from EN116800:2005+A3:2011

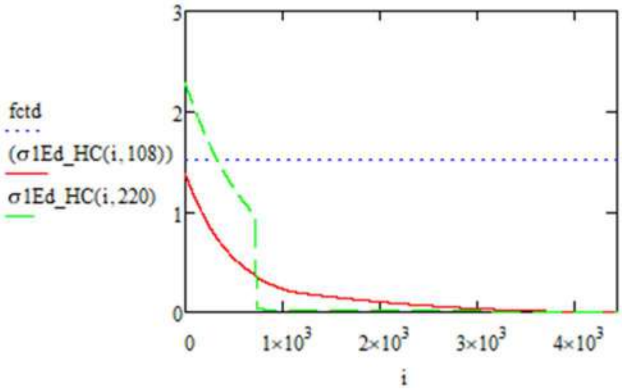
$$\tau Ed\_HC(x,y) := \tau cp(x,y) + Vq\_ULS(x)\cdot\frac{\int_0^y b(y)\cdot(yG - y)\,dy}{Ixo\_cls\cdot b(y)}$$

$$\sigma 1Ed\_HC(x,y) := \frac{\sigma xEd(x,y)}{2} + \sqrt{\left(\frac{\sigma xEd(x,y)}{2}\right)^2 + \tau Ed\_HC(x,y)^2} \qquad \text{§(13.12)}$$


0
53
106
159
212
y
y
y
0
0.6
1.2
1.8
2.4
3
σ1Ed_HC(d,y),σ1Ed_HC(lpt1,y),σ1Ed_HC(0,y)


3
2
1
0
fctd
(σ1Ed_HC(i,108))
σ1Ed_HC(i,220)
0
1×10^3
2×10^3
3×10^3
4×10^3
i

fctd = 1.518



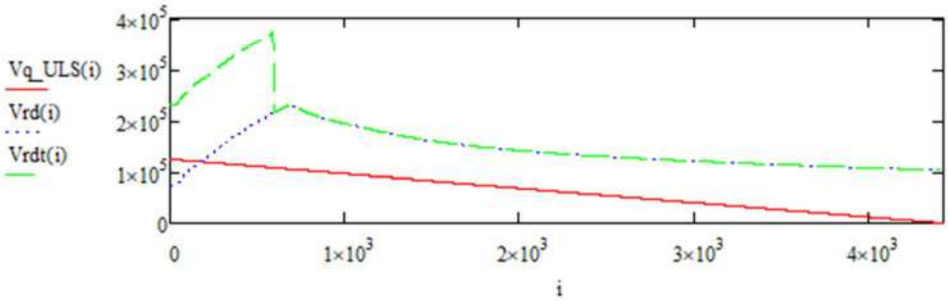
fctd = 1.518

NOTE: the hollowcore element is not checked at all control sections, hence two holes per end are filled with cast-in-situ concrete and threaded-end rebars are screwed into threaded sockets embedded in the proper positions while casting the beam elements. This is also done in order to provide a mechanical slab-to-beam connection useful for robustness criteria and diaphragm action.

$bw\_add := 186 \cdot 4$ mm

$fctd\_C30\_37 := 1.448$ MPa

$$Vrdt(x) := if\left(x < 600, Vrd(x) + \frac{2}{3} \cdot bw\_add \cdot d \cdot fctd\_C30\_37, Vrd(x)\right)$$



MINIMUM REINFORCEMENT (§12.2)

$kh := if[0.8 - 0.6 \cdot (min(bw, Htot) - 0.3) < 0.5, 0.5, if[0.8 - 0.6 \cdot (min(bw, Htot) - 0.3) > 0.8, 0.8, 0.8 - 0.6 \cdot (min(bw, Htot) - 0.3)]] = 0.5$ §9.2.2(2)

$fct\_eff := fctm$

$As\_min\_w1 := 0.2 \cdot kh \cdot fct\_eff \cdot \frac{Ac}{fsk} = 240.078$ mm2 $Ap\_tot = 1.116 \times 10^3$ mm2 CHECK §(9.2)

$rsup \cdot Np\_tot \cdot \frac{(Yp - Yid)}{10^6} + \left(fctm + \frac{Np\_tot \cdot rsup}{Aid}\right) \cdot \frac{Ixo\_id}{(Htot - Yid) \cdot 10^6} = 337.067 < Mrd = 380.537$ kNm CHECK §(12.1)

ANCHORAGE (§11.4)

$klb := 50$

$kcp := 1$ for good bond conditions

$n\sigma := \frac{3}{2}$

$cs := 50$

$cx := 75$

$cy := 40$

$cd(\phi) := \min(0.5 \cdot cs, cx, cy, 3.75 \cdot \phi)$ $cd(12) = 25$

$$lbd(\phi) := \max\left[klb \cdot kcp \cdot \phi \cdot \left(\frac{fsd}{435}\right)^{n\sigma} \cdot \left(\frac{25}{-fck}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1.5 \cdot \phi}{cd(\phi)}\right)^{\frac{1}{2}}, 10 \cdot \phi\right]$$

$lbd(16) =$

length of straight part for 90° bent bars

$lb90(\phi) := \max(70, lbd(\phi) - 15 \cdot \phi, 10 \cdot \phi)$

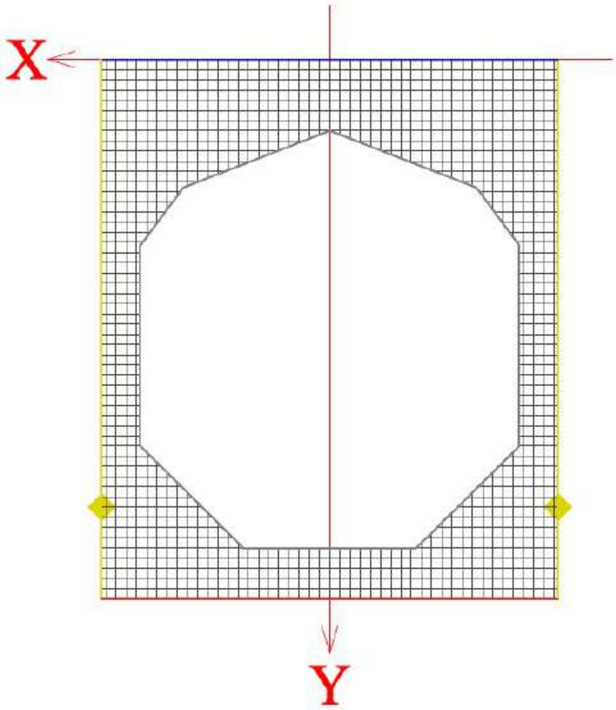
$lb90(12) = 161.872$ $lb90(16) = 339.319$

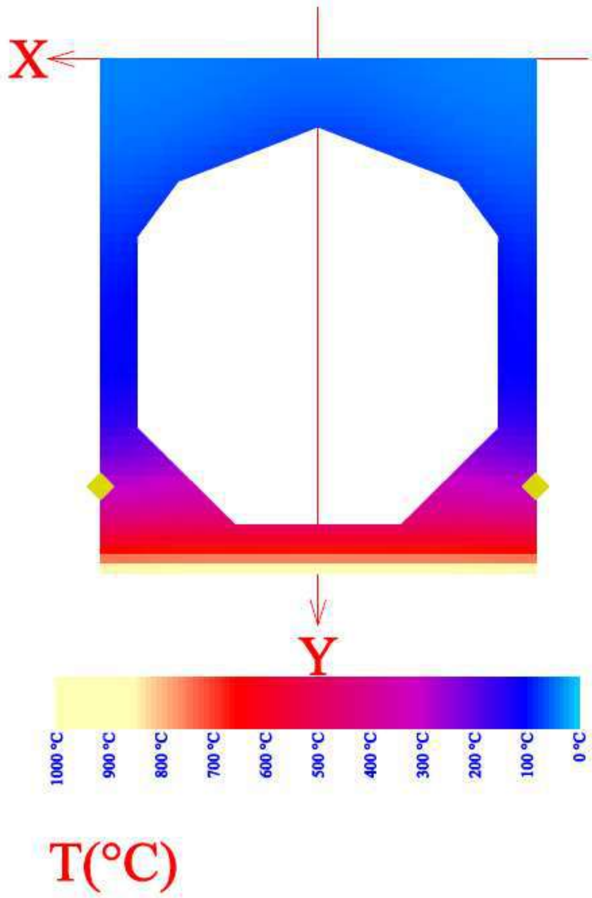
length of straight part for 135° bent bars (stirrups)

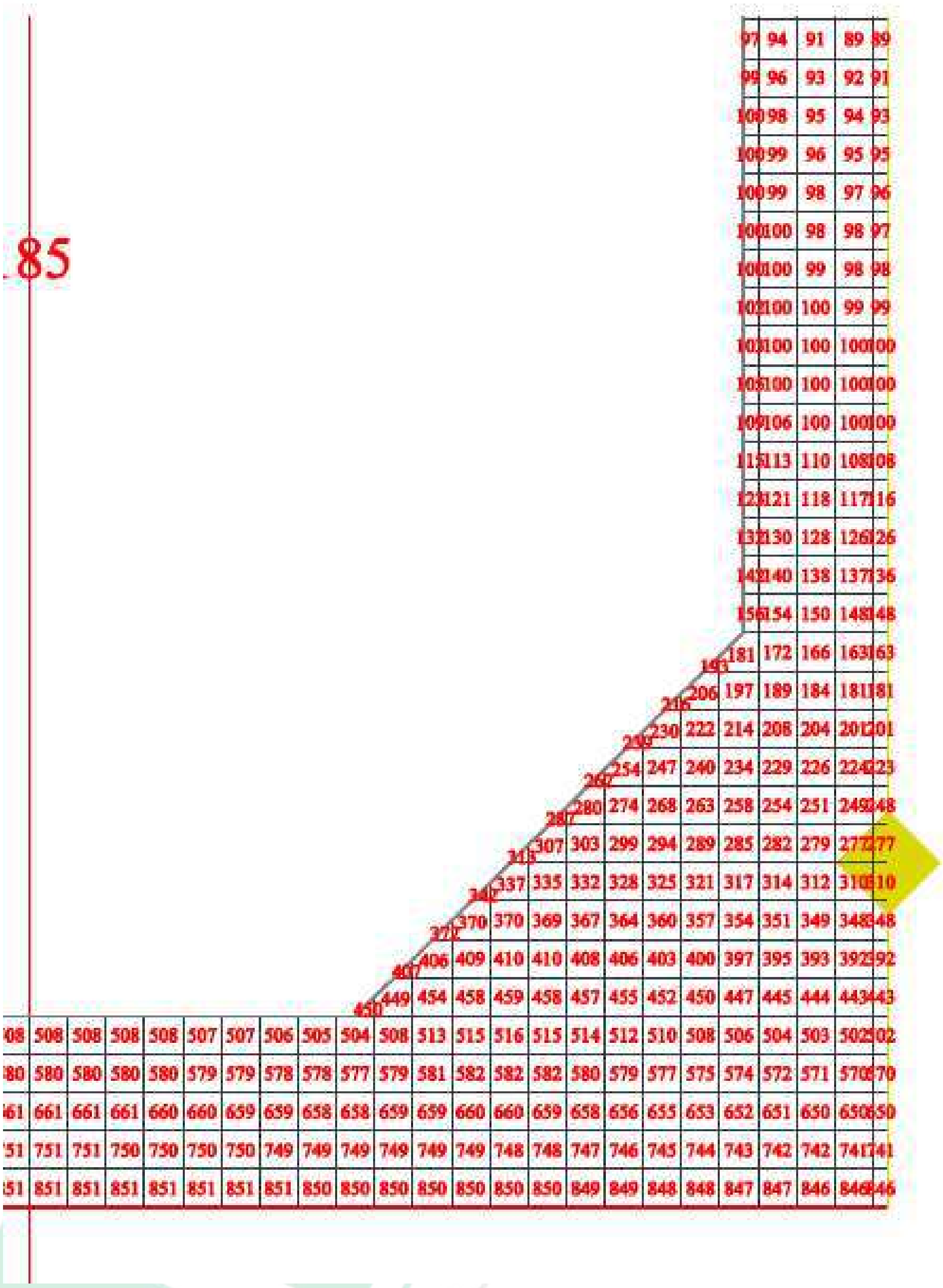
$lb135(\phi) := \max(50, lbd(\phi) - 15 \cdot \phi, 5 \cdot \phi)$

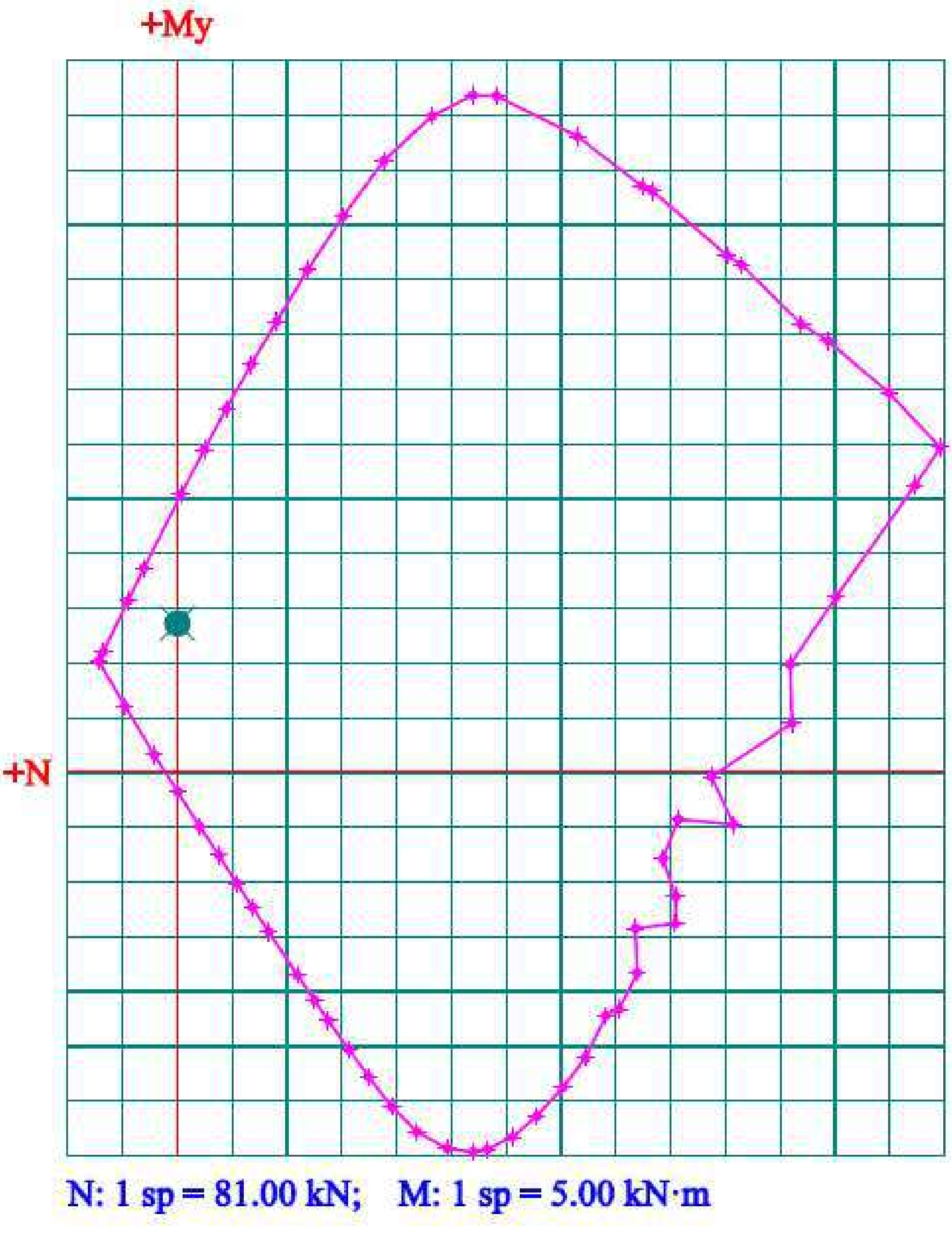
$lb135(12) = 161.872$ $lb135(8) = 50$

8.11 Fire checks





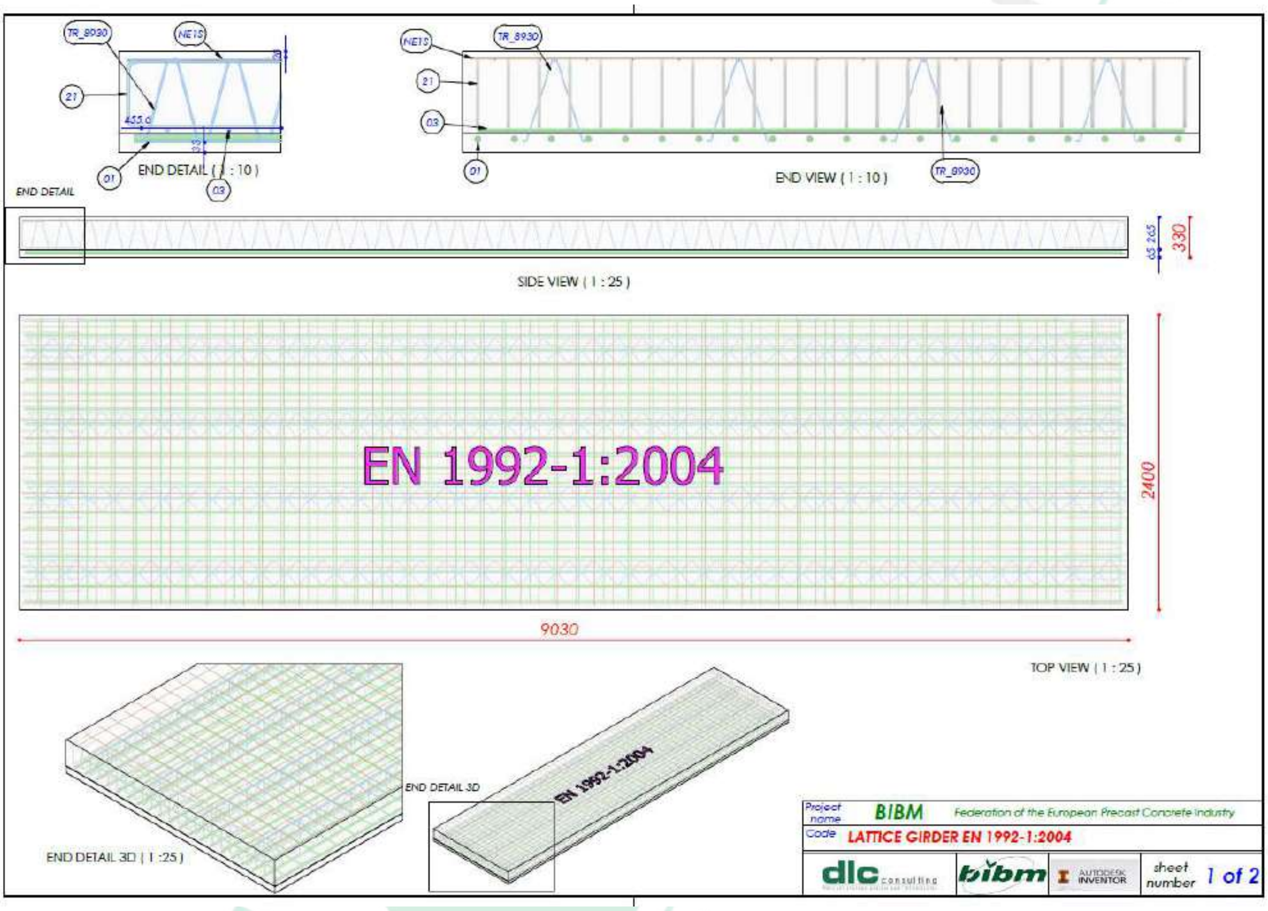




My- =   -1.69 kN·m

9 Lattice girder element –EN1992-1:2004

9.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_	Ø_longitudinal	pattern_T	Ø_transverse	pattern_L
	01	20	31713	634260	24 mm				
	03	45	2042	91890	12 mm				
	21	48	777	37296	12 mm				
	Total mass rebars [kg]			763,45		Incidence kg/m²	106,78		
	NE1S	1	62356	62356		6 mm	150 mm	6 mm	150 mm
Total mass welded-wire-meshes [kg]				62,36		Incidence kg/m²	8,72		
	TR_B750	4	28743	114972					
	Total mass strands [kg]			114,972		Incidence kg/m²	16,08		
	Total mass of steel [kg]			940,77		Concrete volume [m³]		1,41	
						Cast in situ [m³]		5,74	
						Total concrete [m³]		7,15	

Project name	BIBM	Federation of the European Precast Concrete Industry
Code	LATTICE GIRDER EN 1992-1:2004	
dlc consulting / bibm / AUTODESK INVENTOR	sheet number	2 of 2

9.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0 \;\; 330)^T$

$Htot := max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := (2400 \;\; 2400)^T$

$r\_circ := 0$ radius of central void pipe

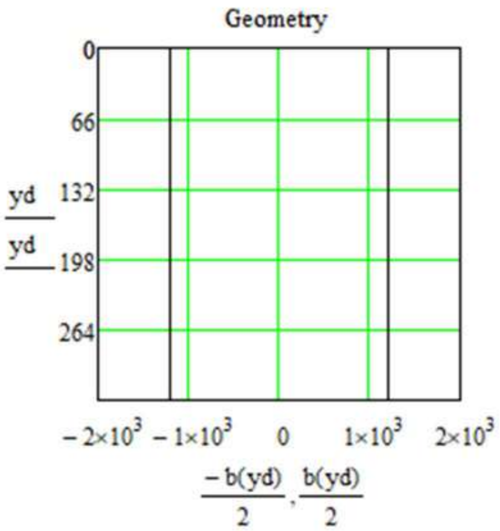
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$b\_lin(y) := linterp(y\_tr, b\_tr, y)$

$b\_circ(y) := linterp(y\_tr, b\_tr, y) - x\_circ(y)$

$$b(y) := if\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$yd := 0 .. Htot$



condensed 1D geometry plot

$u := 2400 \cdot 2 = 4.8 \times 10^3$ exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (30 \;\; 255 \;\; 280 \;\; 295)^T$$

$$As := (4 \cdot A(10) \;\; 0 \cdot A(10) \;\; 8 \cdot A(10) \;\; 20 \cdot A(24))^T = \begin{pmatrix} 314.159 \\ 0 \\ 628.319 \\ 9.048 \times 10^3 \end{pmatrix}$$

$js := rows(As)$ $\qquad js = 4$

$dsmax := max(ds)$ $\qquad dsmax = 295$

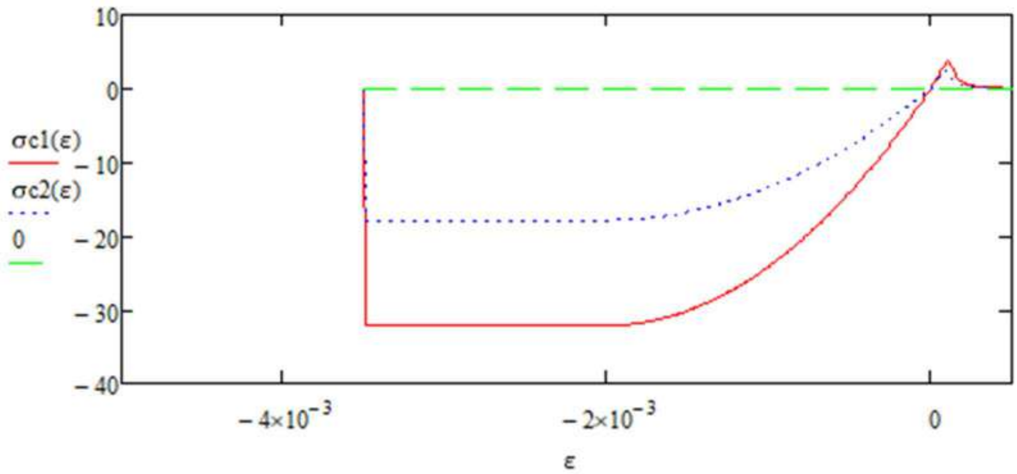
$$As\_tot := \sum_{j=1}^{js} As_j = 9.99 \times 10^3 \quad \text{mm2}$$

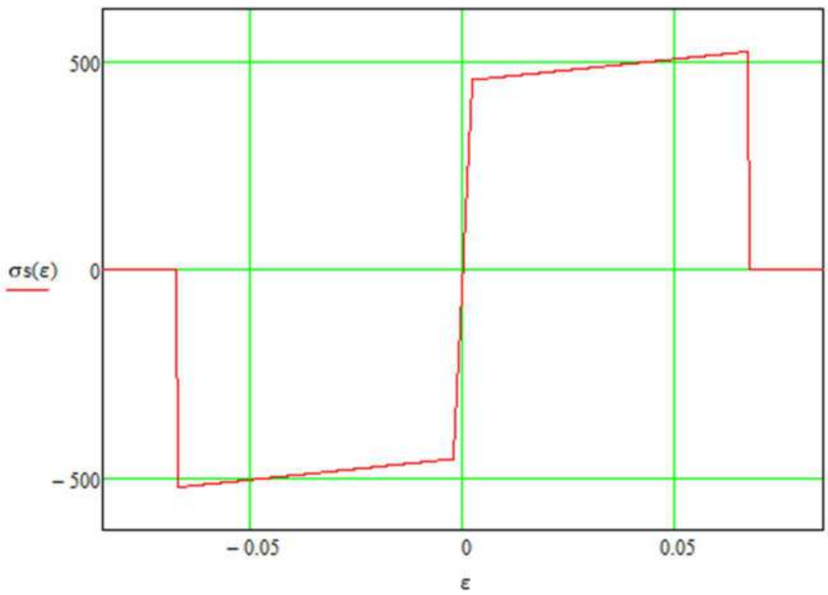
total mild reinforcement area

$$\frac{As\_tot}{2400 \cdot 300} = 0.014$$

Discretisation of the cross-section

9.3 Material constitutive laws employed in the calculation





9.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 7.92 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 0.013$$ geometric ratio for longitudinal mild reinforcement

$$\rho tot := \frac{As\_tot}{Ac} = 0.013$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y)\cdot y\,dy \qquad Syc = 1.307 \times 10^8$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 165$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y)\cdot(y - yG)^2\,dy \qquad Ixo\_cls = 7.187 \times 10^9$$

Idealisation coefficients (elastic)

$$ns := \frac{Es}{Ecm1} \qquad ns = 5.512 \qquad nc := \frac{Ecm2}{Ecm1} = 0.868$$

Area of ideal cross-section

$$Aid := \int_0^{265} nc \cdot b(y)\,dy + \int_{266}^{Htot} b(y)\,dy + (ns-1)\cdot\sum_{j=1}^{js} As_j \qquad Aid = 7.504 \times 10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac \cdot yG + (nc-1)\cdot\int_0^{265} b(y)\cdot y\,dy + (ns-1)\cdot\sum_{j=1}^{js}\left(As_j \cdot ds_j\right) = 1.324 \times 10^8 \qquad Sxid = 1.324 \times 10^8$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 176.429$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y)\cdot(y - Yid)^2\,dy - \int_0^{265} b(y)\cdot(y - Yid)^2\,dy - \sum_{j=1}^{js}\left[As_j \cdot \left(ds_j - Yid\right)^2\right] = 2.201 \times 10^9$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns \cdot \sum_{j=1}^{js}\left[As_j \cdot \left(ds_j - Yid\right)^2\right] \qquad Ixoidcls2 := nc \cdot \int_0^{265} b(y)\cdot(y - Yid)^2\,dy$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidlenta + Ixoidcls2 \qquad Ixo\_id = 7.27 \times 10^9 \quad \text{mm^4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.011$$

9.5 Loads

LOADS

$interaxis := 2400$ mm

$g1 := Ac \cdot 0.000025 = 19.8$ kN/m dead load from self-weight

$g2 := 2 \cdot \frac{interaxis}{1000} = 4.8$ kN/m nonstructural dead load

$q := 3 \cdot \frac{interaxis}{1000} = 7.2$ kN/m live load

$L := 8850$ mm calculation length (span between supports)

$\psi 2 := 0.3$ non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from live load

9.6 Time-dependent behaviour

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$h0 := 2 \cdot \frac{Ac}{2 \cdot u} = 165$ mm notional size of the member

$RH := 50$ % relative humidity

$t0\_T(t0) := t0$

$\alpha := 1$ for cement class R

$$t0\_mod(t0) := \max\left[t0\_T(t0) \cdot \left(\frac{9}{2 + t0\_T(t0)^{1.2}} + 1\right)^{\alpha}, 0.5\right] \qquad t0\_mod(2) = 6.189$$

$$\alpha c1 := \left(\frac{35}{-fcm1}\right)^{0.7} = 0.748$$

$$\alpha c2 := \left(\frac{35}{-fcm1}\right)^{0.2} = 0.92$$

$$\alpha c3 := \left(\frac{35}{-fcm1}\right)^{0.5} = 0.813$$

$$\beta h := if\left[-fcm1 > 35, \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250 \cdot \alpha c3, 1500 \cdot \alpha c3\right], \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250, 1500\right]\right] = 450.684$$

$$\beta t0(t0) := \frac{1}{0.1 + t0\_mod(t0)^{0.2}}$$

$$\beta c(t, t0) := \left(\frac{t - t0\_mod(t0)}{\beta h + t - t0\_mod(t0)}\right)^{0.3}$$

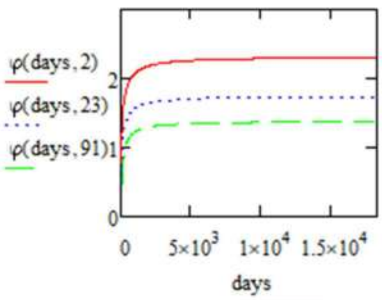
$$\beta fcm := \frac{16.8}{\sqrt{-fcm1}} = 2.308$$

$$\varphi RH := if\left[-fcm1 > 35, \left(1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}} \cdot \alpha c1\right) \cdot \alpha c2, 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}}\right] = 1.548$$

$$\varphi 0(t0) := \varphi RH \cdot \beta fcm \cdot \beta t0(t0)$$

$$\varphi(t, t0) := \varphi 0(t0) \cdot \beta c(t, t0)$$

$$t := 50 \cdot 365 = 1.825 \times 10^{4} \qquad \varphi(t, 2) = 2.303 \qquad \varphi(t, 91) = 1.372$$



Shrinkage

$\beta RH := 1.55\cdot\left[1-\left(\frac{RH}{100}\right)^3\right] = 1.356$

$fcmo := 10$ MPa

$\alpha ds1\_2 := 4$ for cement class N

$\alpha ds2\_2 := 0.12$

$\varepsilon cd\_0\_2 := 0.85\cdot\left[(220+110\cdot\alpha ds1\_2)\cdot e^{-\alpha ds2\_2\cdot\frac{fcm2}{fcmo}}\right]\cdot 10^{-6}\cdot\beta RH = 1.131\times10^{-3}$

$kh := 0.89$ for h0=175 mm

$\varepsilon cd2 := kh\cdot\varepsilon cd\_0\_2 = 1.006\times10^{-3}$

$\varepsilon ca2 := 2.5\cdot(-fck2-10)\cdot10^{-6} = 3.75\times10^{-5}$

$\varepsilon cs2 := \varepsilon cd2+\varepsilon ca2 = 1.044\times10^{-3}$

$\alpha ds1\_1 := 6$ for cement class R

$\alpha ds2\_1 := 0.11$

$\varepsilon cd\_0\_1 := 0.85\cdot\left[(220+110\cdot\alpha ds1\_1)\cdot e^{-\alpha ds2\_1\cdot\frac{fcm1}{fcmo}}\right]\cdot 10^{-6}\cdot\beta RH = 1.817\times10^{-3}$

$\varepsilon cd1 := kh\cdot\varepsilon cd\_0\_1 = 1.617\times10^{-3}$

$\varepsilon ca1 := 2.5\cdot(-fck1-10)\cdot10^{-6} = 8.75\times10^{-5}$

$\varepsilon cs1 := \varepsilon cd1+\varepsilon ca1 = 1.705\times10^{-3}$

it is assumed that the shrinkage effect is compensated by the time slot between casting of the precast girder and the slab and proper expansive admixtures

9.7 Non-linear moment-curvature diagram

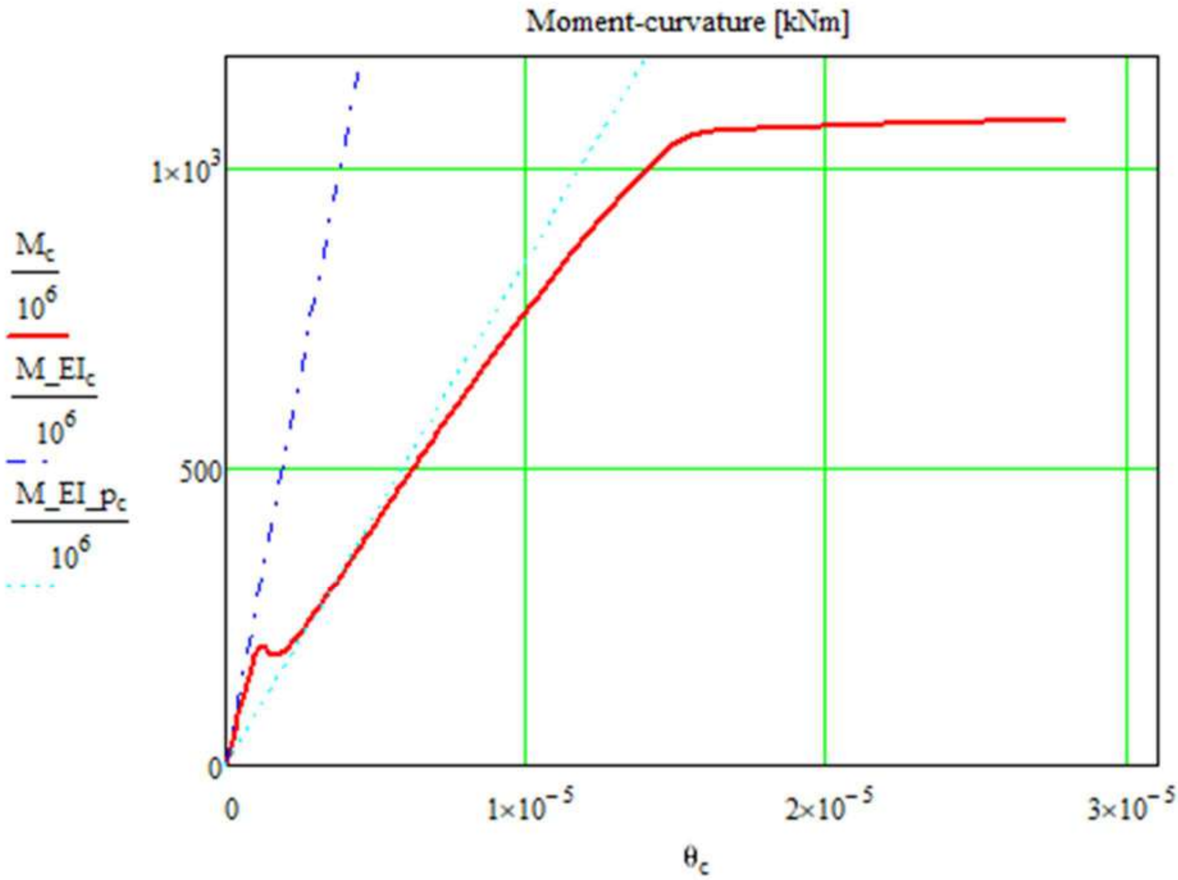
Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

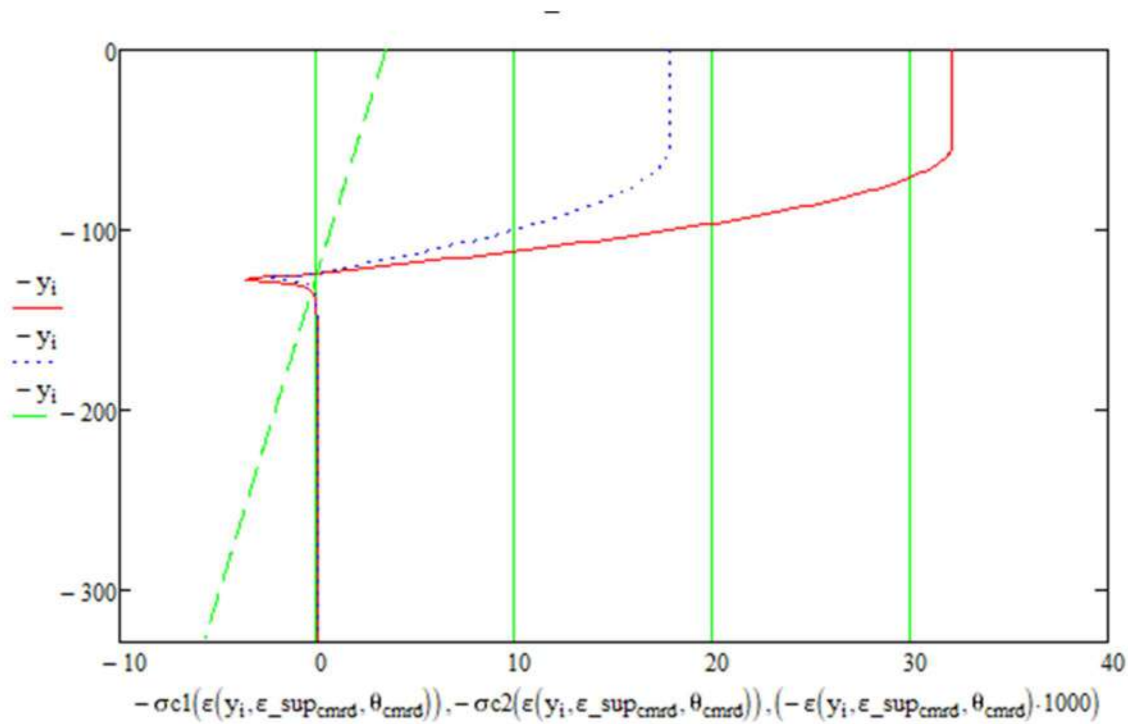
$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{265}\left(\sigma c2\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\right)+\sum_{i=266}^{Htot}\left[\sigma c1\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot\left(b\left(y_i\right)\right)\cdot\Delta y\right]+\sum_{j=1}^{js}\left(\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\right)$$

$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{265}\left[\sigma c2\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\cdot\left(y_i-yG\right)\right]+\sum_{i=266}^{Htot}\left[\sigma c1\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot\left(b\left(y_i\right)\right)\cdot\Delta y\cdot\left(y_i-yG\right)\right]+\sum_{j=1}^{js}\left[\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\cdot\left(ds_j-yG\right)\right]$$

Design external axial load

$NS := -0$


Moment-curvature [kNm]
1×10^3
500
0
0
1×10^-5
2×10^-5
3×10^-5
θc
Mc / 10^6
M_EIc / 10^6
M_EI_pc / 10^6



Condition at resisting (peak) moment
(stress and strain)

9.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformally distributed load q

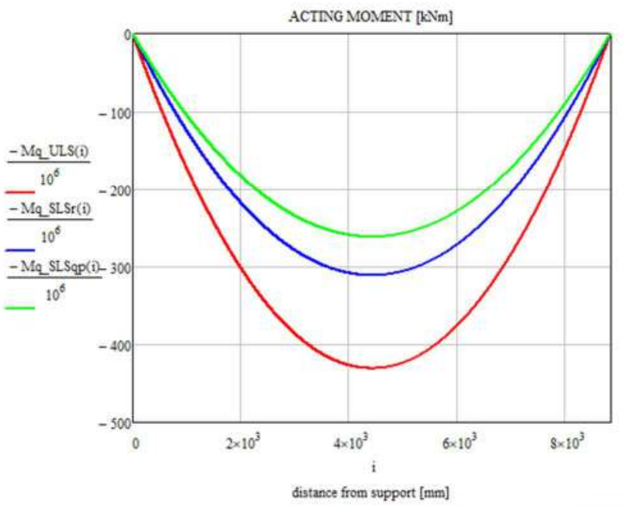
$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformally distributed load q

$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformally distributed load q

$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformally distributed load q

$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformally distributed load q

$i := 0..L$



9.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

v_inf_p(x) := v_SLSg1(x)·(φ(365·50, 14) − φ(365·50, 23)) + v_SLSg2(x)·(1 + φ(365·50, 23))

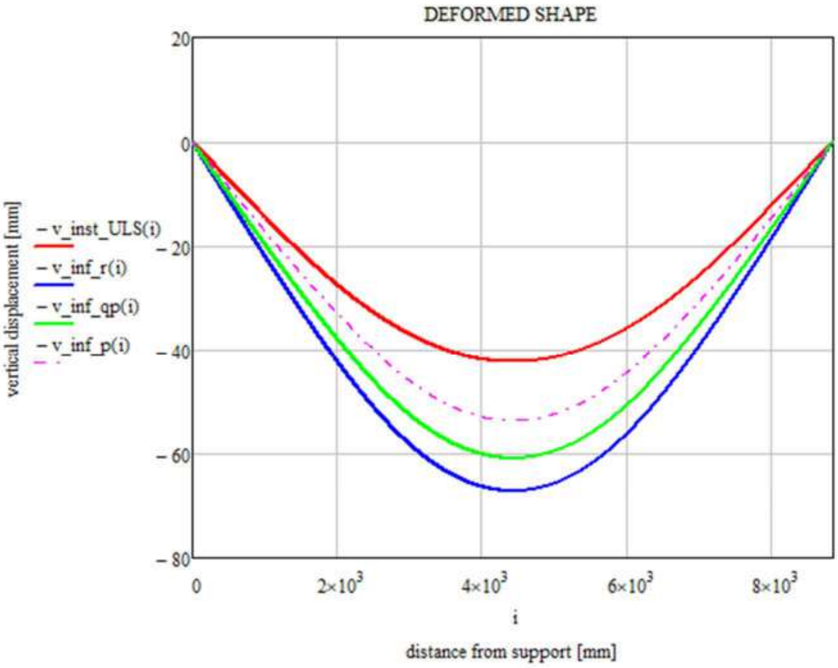
deflection profile at 50 years including creep for permanent load combination

v_inf_qp(x) := v_SLSg1(x)·(φ(365·50, 14) − φ(365·50, 23)) + v_SLSg2(x)·(φ(365·50, 23) − φ(365·50, 91)) + v_SLSqp(x)·(1 + φ(365·50, 91))

deflection profile at 50 years including creep for quasi permanent load combination

v_inf_r(x) := v_SLSg1(x)·(φ(365·50, 14) − φ(365·50, 23)) + v_SLSg2(x)·(φ(365·50, 23) − φ(365·50, 91)) + v_SLSqp(x)·φ(365·50, 91) + v_SLSr(x)

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§7.4.3)

camber := 35 mm	>	$\frac{-L}{250} = -35.4$	CHECK	maximum camber
imposed camber by mould shaping				
$v\_inf\_r\left(\frac{L}{2}\right) - camber = 32.208$	<	$\frac{L}{250} = 35.4$	CHECK	maximum deflection

value calculated from differential equations above

SLS STRESS CONTROL (§7.2)

$k1 := 0.6$

$k2 := 0.45$

$k3 := 0.8$

$k4 := 1$

$k5 := 0.75$

$\sigma cpf\_bot(x) := \dfrac{Mq\_SLSf(x)\cdot(Htot - Yid)}{Ixo\_id}$ $\sigma cpf\_bot\left(\dfrac{L}{2}\right) = 5.832$ $<$ $fctm1 = 3.795$ CHECK

elastic stress of bottom concrete chord for frequent load combination — if not -> cracked

$\sigma sf\_bot(x) := 15\cdot\left[\dfrac{Mq\_SLSf(x)\cdot\left(ds_{js} - Yid\right)}{Ixo\_id}\right]$ $\sigma sf\_bot\left(\dfrac{L}{2}\right) = 67.544$

creep stress of bottom reinforcement layer for frequent load combination

$\sigma cpr\_bot(x) := \dfrac{Mq\_SLSr(x)\cdot(Htot - Yid)}{Ixo\_id}$ $\sigma cpr\_bot\left(\dfrac{L}{2}\right) = 6.577$ $<$ $fctm1 = 3.795$

elastic stress of bottom concrete chord for rare load combination

$\sigma cpr\_top(x) := \dfrac{nc\cdot Mq\_SLSr(x)\cdot(-Yid)}{Ixo\_id}$ $\sigma cpr\_top\left(\dfrac{L}{2}\right) = -6.554$ $>$ $k1\cdot fck2 = -15$ CHECK

$>$ $0.4\cdot fcm2 = -13.2$

elastic stress of top concrete chord for rare load combination

$\sigma cpr\_s(x) := 15\cdot\left[\dfrac{Mq\_SLSr(x)\cdot\left(ds_{js} - Yid\right)}{Ixo\_id}\right]$ $\sigma cpr\_s\left(\dfrac{L}{2}\right) = 76.167$ $<$ $k3\cdot fsk = 400$ CHECK

creep stress of bottom mild steel for rare load combination

SLS CRACK CONTROL (§7.3)

$c\_act := Htot - ds_{js} - 10 = 25$

$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1.25$

$wlim\_cal := 0.2$ mm

$k1c := 0.8$ $\phi := 24$

$k2c := 0.5$

$k3c := 3.4$

$k4c := 0.425$

$cover := Htot - \frac{\phi}{2} - ds_{js} = 23$

$Aceff := b(Htot) \cdot \min\left[2.5 \cdot (Htot - ds_{js}), \frac{Htot - Yn\_n}{3}, \frac{Htot}{2}\right] = 1.907 \times 10^{5}$

$\rho peff := \frac{As_{js} + As_{js-1}}{Aceff} = 0.051$

$srmax := k3c \cdot cover + \frac{k1c \cdot k2c \cdot k4c \cdot \phi}{\rho peff} = 158.595$

$kt := 0.4$ NOTE : 0.6 for sustained loading

$fcteff := fctm1 = 3.795$

$$\varepsilon sm\_\varepsilon cm := \max\left[\frac{\sigma sf\_bot\left(\frac{L}{2}\right) - kt \cdot \frac{fcteff}{\rho peff} \cdot \left(1 + \frac{Es}{Ecm1} \cdot \rho peff\right)}{Es}, 0.6 \cdot \frac{\sigma sf\_bot\left(\frac{L}{2}\right)}{Es}\right] = 2.026 \times 10^{-4}$$

$wk := srmax \cdot \varepsilon sm\_\varepsilon cm = 0.032$ $<$ $wlim\_cal = 0.2$ CHECK

9.10 ULS checks

ULS BENDING-AXIAL CONTROL (§6.1)

$Mrd = 1.077 \times 10^{3}$ $>$ $\frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^{6}} = 430.872$ CHECK

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§6.2)

$Vq\_ULS(x) := \left|(g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2} - x\right)\right|$ shear distribution at Ultimate Limit State (ULS)

$d := ds_{js} = 295$ mm effective depth

$VEd := Vq\_ULS(d) = 1.818\times10^{5}$ N maximum shear at effective depth from support

$bw := 2400$ mm web width

$z := 0.9\cdot d = 265.5$ conventional resultant lever arm

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT

$\rho l := \frac{\sum_{j=1}^{js} As_j}{bw\cdot d} = 0.014$ reinforcement ratio

$\sigma cp(x) := 0$ MPa axial load induced by prestressing

$kv := \min\left(1 + \sqrt{\frac{200}{d}}, 2\right) = 1.678$

$k1v := 0.15$

$Crdc := \frac{0.18}{\gamma cpcred} = 0.129$

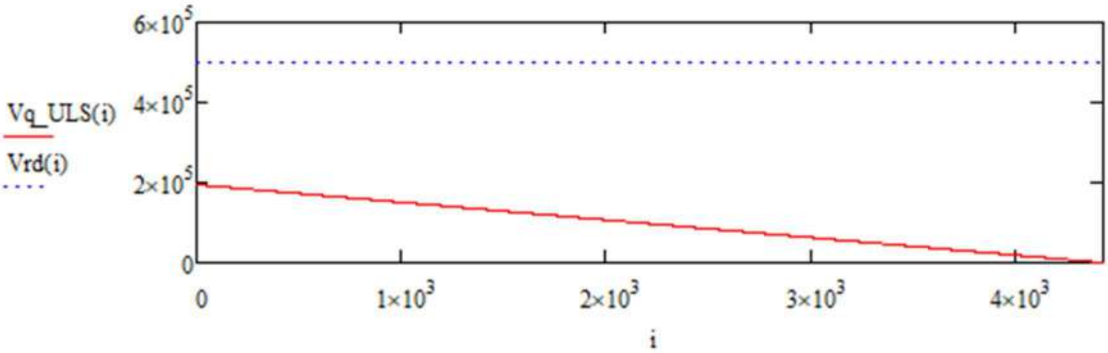
$vmin := 0.035\cdot kv^{\frac{3}{2}}\cdot(-fck2)^{\frac{1}{2}} = 0.38$ §6.3N

bw·d·[Crdc·kv

(vmin + k1v·σ

$VRdc(x) := \max\left[\left[Crdc\cdot kv\cdot(100\cdot\rho l\cdot -fck2)^{\frac{1}{3}} + k1v\cdot\sigma cp(x)\right]\cdot bw\cdot d, (vmin + k1v\cdot\sigma cp(x))\cdot bw\cdot d\right]$ §6.2.a+§6.2.b

$Vrd(x) := VRdc(x)$

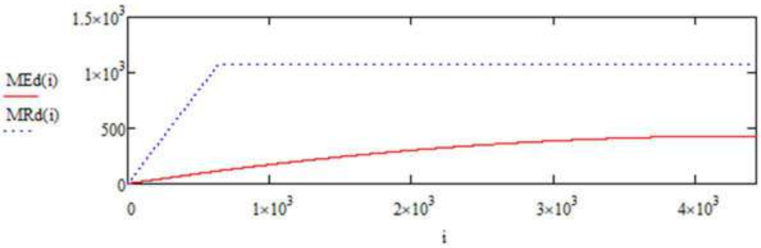


NOTE: there is no need for transverse reinforcement

MOMENT DIAGRAM ACCOUNTING FOR MILD STEEL REBAR ANCHORAGE

$MEd(x) := \frac{Mq\_ULS(x)}{10^6}$

$MRd(x) := if\left(x < 639, Mrd\cdot\frac{x}{639}, Mrd\right)$



MINIMUM REINFORCEMENT

$bt := b(Htot) = 2.4\times 10^3$

§9.1N

§9.2.1.1(3)

$Asmin := max\left(0.26\cdot\frac{fctm2}{fsk}\cdot bt\cdot d, 0.0013\cdot bt\cdot d\right) = 944.317$ mm2 < $\rho l\cdot Ac = 1.118\times 10^4$ mm2 CHECK < $0.04\cdot Ac = 3.168\times 10^4$ CHECK

for longitudinal reinforcement

MINIMUM TRANSVERSE REINFORCEMENT (§9.3.1.1(2))

$\frac{0.2\cdot A(24)}{120} = 0.754$ < $\frac{A(12)}{200} + \frac{A(6)}{150} = 0.754$ CHECK

CHECK OF SUPPORT MILD REBARS (§9.2.1.4(1))

$0.25\cdot Mrd = 269.296$ Nmm < $\left(24\cdot\pi\cdot\frac{12^2}{4}\right)\cdot fsd\cdot 0.9\cdot\frac{250}{10^6} = 277.603$ CHECK

area of support mild steel

ANCHORAGE (§8.4)

$\eta 1 := 1$

$\eta 2 := 1$

$fbd := 2.25\cdot\eta 1\cdot\eta 2\cdot fctd2 = 2.886$

$lbrqd(\phi) := \frac{\phi}{4}\cdot\frac{fsd}{fbd}$

$\alpha 1b := 1$

$\alpha 2b := 1$

$\alpha 3b := 1$

$\alpha 4b := 1$

$\alpha 5b := 1$

$lbd(\phi) := \alpha 1b\cdot\alpha 2b\cdot\alpha 3b\cdot\alpha 4b\cdot\alpha 5b\cdot lbrqd(\phi)$

INTERFACE BETWEEN CONCRETES CAST AT DIFFERENT TIME (§6.2.5)

$\beta inter := 1$

$bi := 2400$ mm

$vEdi(x) := \beta inter \cdot \frac{Vq\_ULS(x)}{z \cdot bi}$ $\qquad vEdi(0) = 0.306$

$c\_inter := 0.4$ $\qquad c\_inter \cdot fctd2 = 0.513$

$\mu := 0.4$

$\alpha inter := 75 \cdot \frac{\pi}{180} = 1.309$ rad

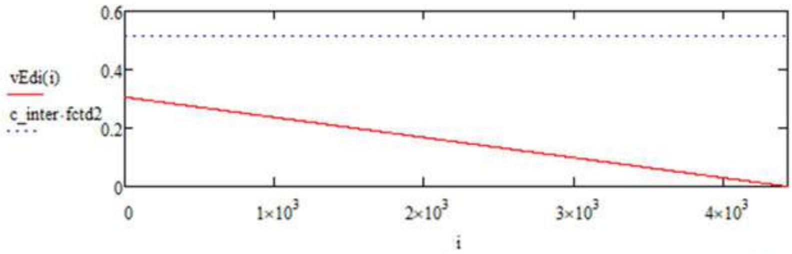
$\sigma n := 0$

$Ainter := bi \cdot 265 = 6.36 \times 10^{5}$

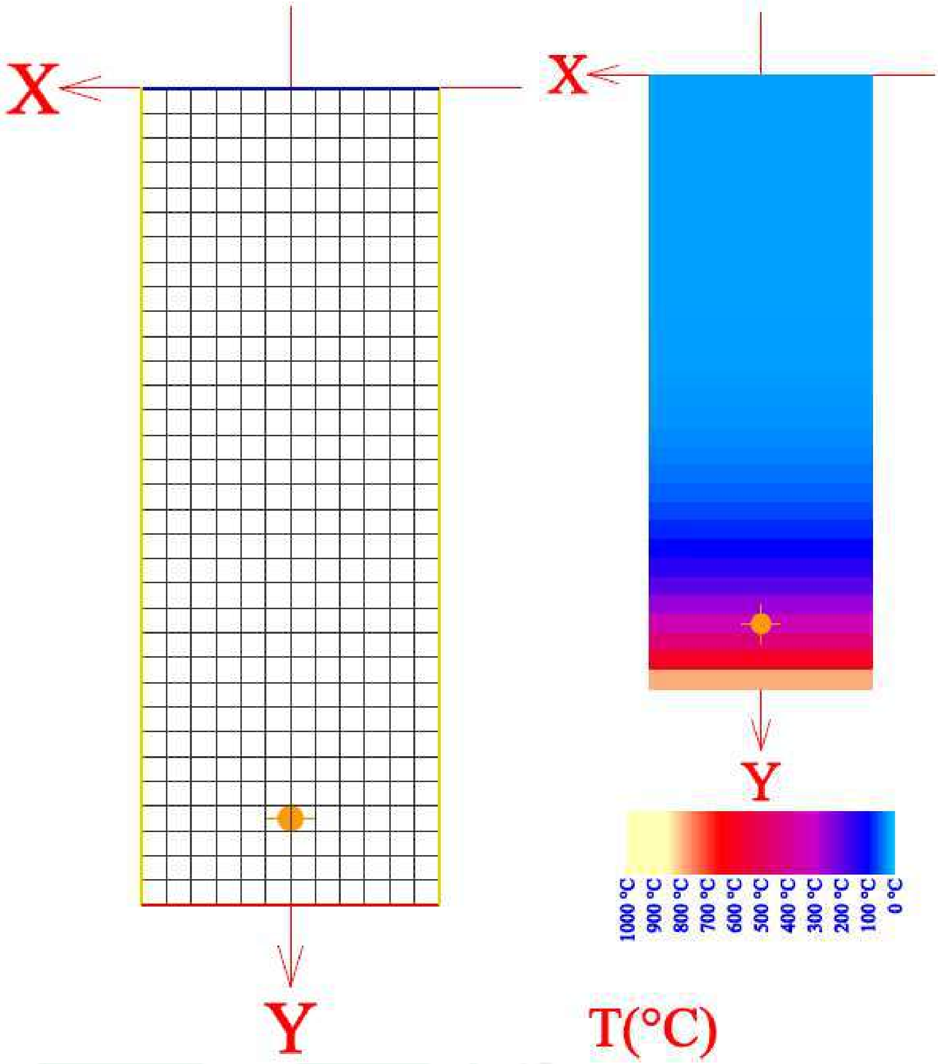
$\rho := \frac{10 \cdot A(6)}{Ainter} = 4.446 \times 10^{-4}$

$\nu := 0.6 \cdot \left(1 - \frac{-fck2}{250}\right) = 0.54$

$vRdi := \min[c\_inter \cdot fctd2 + \mu \cdot \sigma n + \rho \cdot fsd \cdot (\mu \cdot \sin(\alpha inter) + \cos(\alpha inter)), 0.5 \cdot \nu \cdot -fcd2] = 0.643$ MPa $>$ $vEdi(0) = 0.306$ MPa CHECK



9.11 Fire checks



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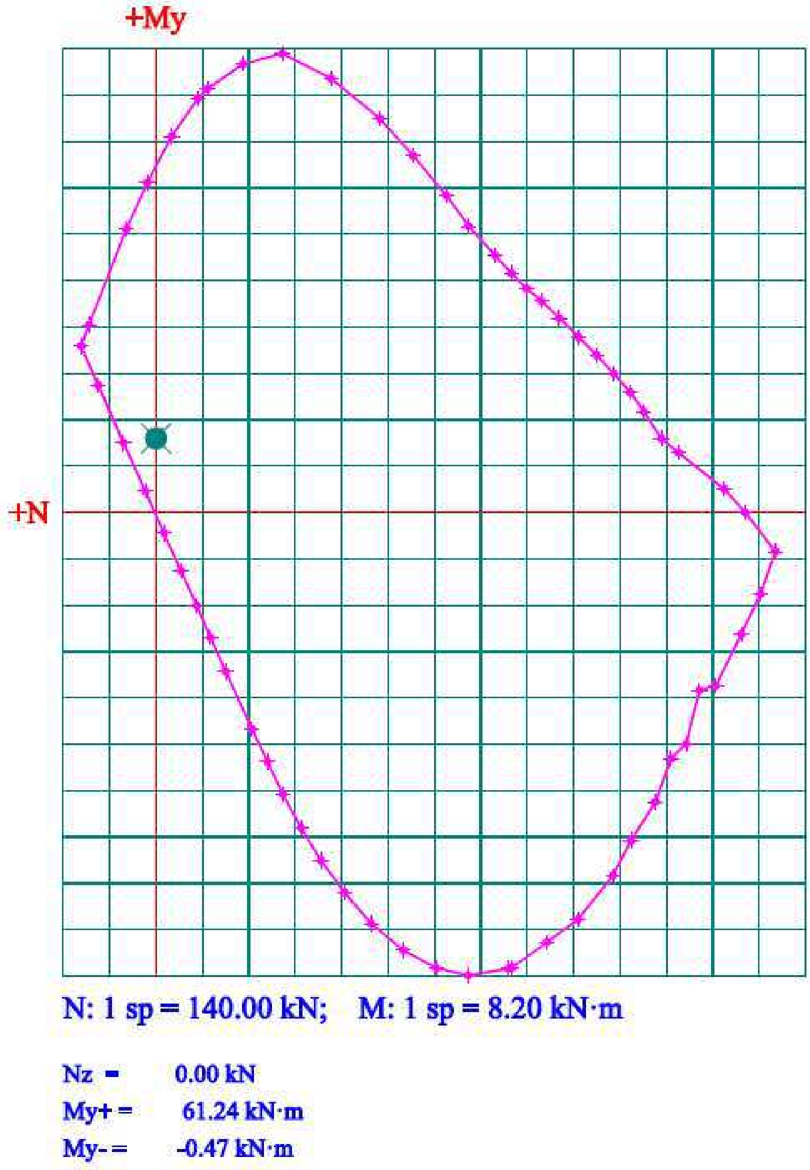
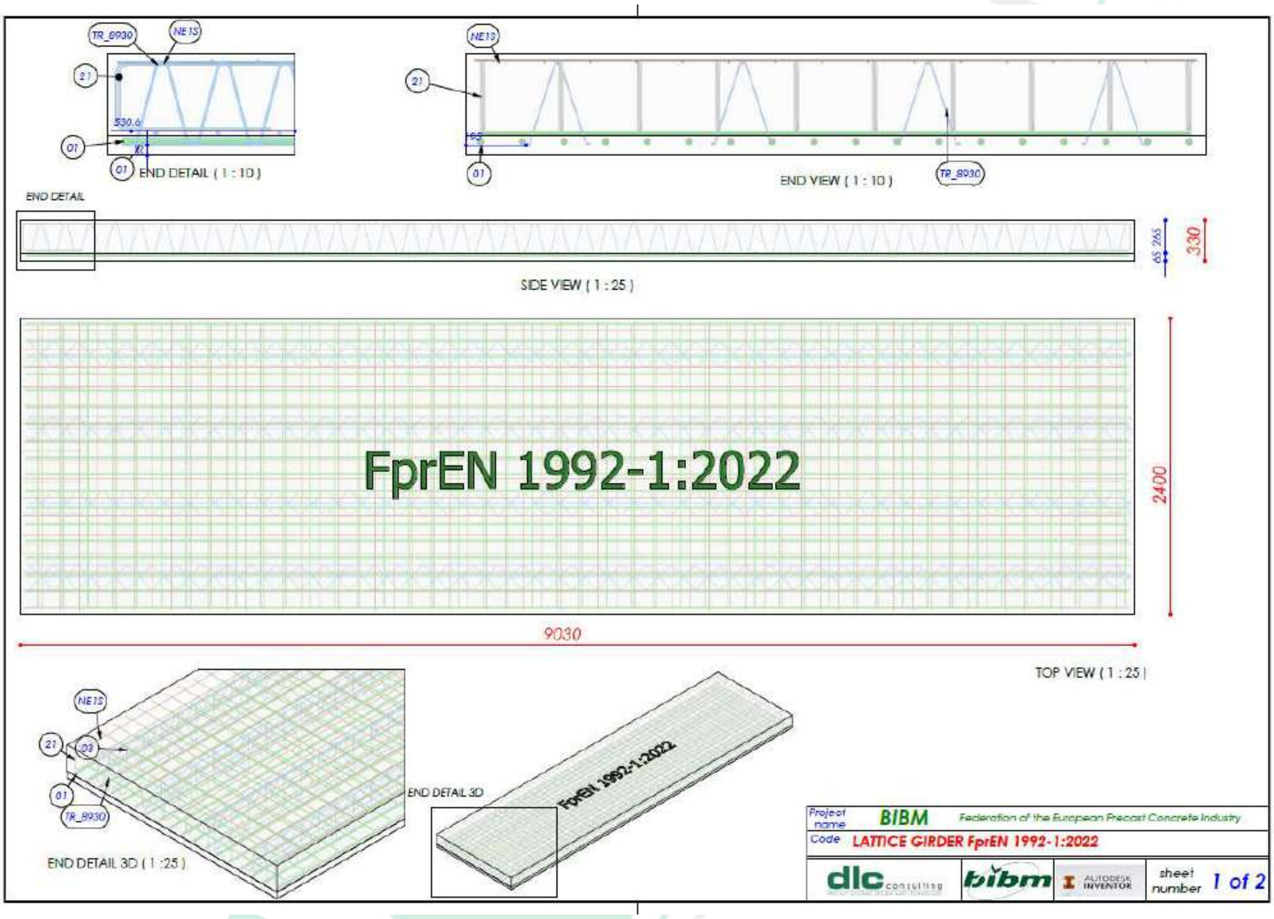

+My
+N
N: 1 sp = 140.00 kN;   M: 1 sp = 8.20 kN·m
Nz = 0.00 kN
My+ = 61.24 kN·m
My- = -0.47 kN·m

10 Lattice girder element – FprEN1992-1:2022

10.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_		
	01	18	31713	570834	24 mm		
	03	43	2042	87806	12 mm		
	21	20	777	15540	12 mm		
Total mass rebars [kg]				674,18	Incidence kg/m³	94,29	
	NE15	1	62356	62356			
Total mass welded-wire-meshes [kg]				62,36	Incidence kg/m³	8,72	
	TR_8750	4	28743	114972			
Total mass strands [kg]				114,972	Incidence kg/m³	16,08	
	Total mass of steel [kg]			851,51		Total concrete volume [m³]	1,41
						Total cast in situ [m³]	5,74
						Total concrete [m³]	7,15

VIEW 5 (1 : 60)

Project name: BIBM — Federation of the European Precast Concrete industry

Code: LATTICE GIRDER FprEN 1992-1:2022

dlc consulting | bibm | AUTODESK INVENTOR | sheet number 2 of 2

10.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0\ \ 330)^T$

$Htot := max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := (2400\ \ 2400)^T$

$r\_circ := 0$ radius of central void pipe

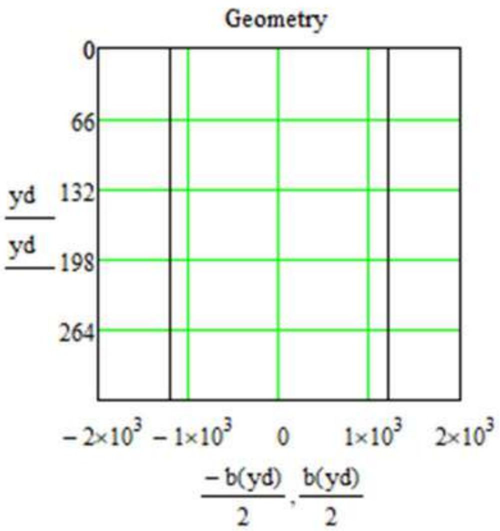
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$b\_lin(y) := linterp(y\_tr, b\_tr, y)$

$b\_circ(y) := linterp(y\_tr, b\_tr, y) - x\_circ(y)$

$$b(y) := if\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$yd := 0..Htot$



condensed 1D geometry plot

$u := 2400 \cdot 2 = 4.8 \times 10^3$ exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (30\ \ 255\ \ 280\ \ 295)^T$$

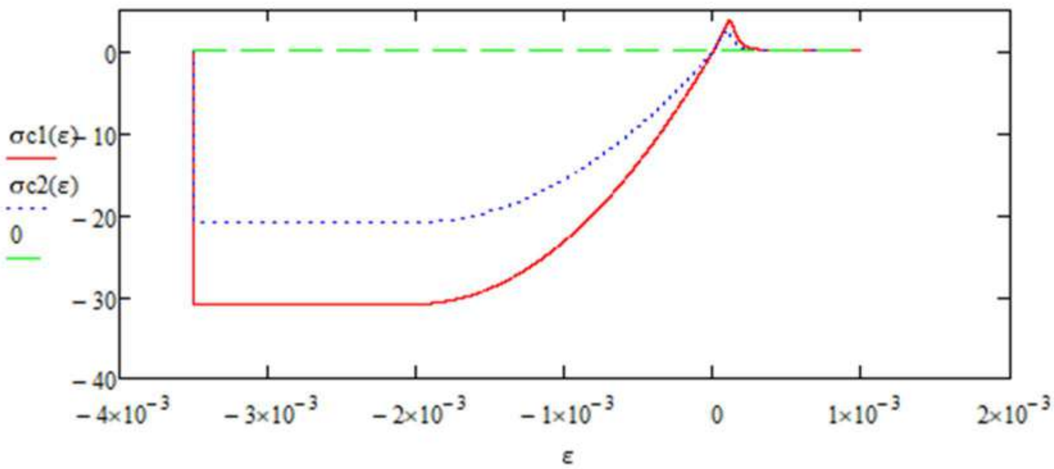
$$As := (4 \cdot A(10)\ \ 0 \cdot A(10)\ \ 8 \cdot A(10)\ \ 18 \cdot A(24))^T = \begin{pmatrix} 314.159 \\ 0 \\ 628.319 \\ 8.143 \times 10^3 \end{pmatrix}$$

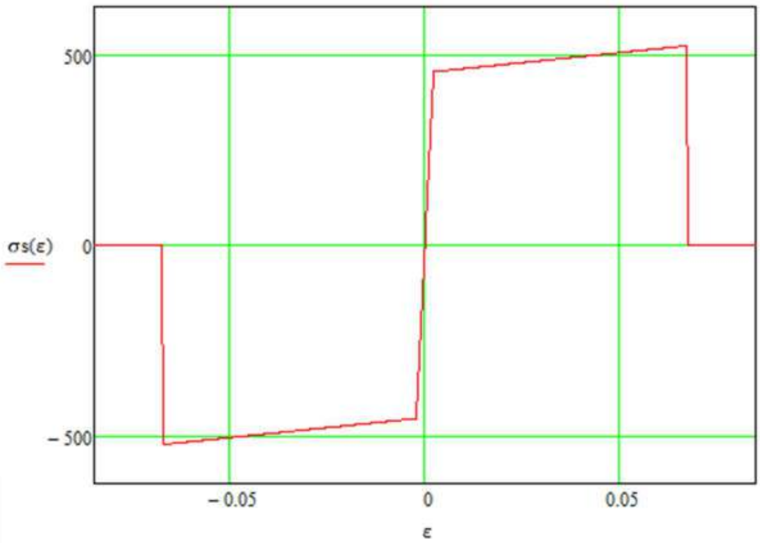
$js := rows(As) \qquad js = 4$

$dsmax := max(ds) \qquad dsmax = 295$

$$As\_tot := \sum_{j=1}^{js} As_j = 9.085 \times 10^3 \qquad \frac{As\_tot}{2400 \cdot 300} = 0.013$$

10.3 Material constitutive laws employed in the calculation





10.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 7.92 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 0.011$$ geometric ratio for longitudinal mild reinforcement

$$\rho tot := \frac{As\_tot}{Ac} = 0.011$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y)\cdot y\,dy \qquad Syc = 1.307 \times 10^8$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 165$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y)\cdot(y - yG)^2\,dy \qquad Ixo\_cls = 7.187 \times 10^9$$

Idealisation coefficients (elastic)

$$ns := \frac{Es}{Ecm1} \qquad ns = 5.605 \qquad nc := \frac{Ecm2}{Ecm1} = 0.854$$

Area of ideal cross-section

$$Aid := \int_0^{265} nc \cdot b(y)\, dy + \int_{266}^{Htot} b(y)\, dy + (ns - 1) \cdot \sum_{j=1}^{js} As_j \qquad Aid = 7.385 \times 10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac \cdot yG + (nc - 1) \cdot \int_0^{265} b(y) \cdot y\, dy + (ns - 1) \cdot \sum_{j=1}^{js} \left(As_j \cdot ds_j\right) = 1.303 \times 10^8 \qquad Sxid = 1.303 \times 10^8$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 176.411$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y) \cdot (y - Yid)^2\, dy - \int_0^{265} b(y) \cdot (y - Yid)^2\, dy - \sum_{j=1}^{js} \left[As_j \cdot \left(ds_j - Yid\right)^2\right] = 2.214 \times 10^9$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns \cdot \sum_{j=1}^{js} \left[As_j \cdot \left(ds_j - Yid\right)^2\right] \qquad Ixoidcls2 := nc \cdot \int_0^{265} b(y) \cdot (y - Yid)^2\, dy$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidlenta + Ixoidcls2 \qquad Ixo\_id = 7.157 \times 10^9 \text{ mm}^4 \qquad \frac{Ixo\_id}{Ixo\_cls} = 0.996$$

10.5 Loads

LOADS

$interaxis := 2400$ mm

$g1 := Ac \cdot 0.000025 = 19.8$ kN/m dead load from self-weight

$g2 := 2 \cdot \frac{interaxis}{1000} = 4.8$ kN/m nonstructural dead load

$q := 3 \cdot \frac{interaxis}{1000} = 7.2$ kN/m live load

$L := 8850$ mm calculation length (span between supports)

$\psi 2 := 0.3$ non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from live load

10.6 Time-dependent behaviour

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$$hn := 2 \cdot \frac{Ac}{u} = 330$$

$$RH := 50$$

$$t0\_adj(t0) := t0$$

$$\beta bc\_fcm1 := \frac{1.8}{(-fcm1)^{0.7}} = 0.112 \qquad \beta bc\_t\_t0(t,t0) := \ln\left[\left(\frac{30}{t0\_adj(t0)} + 0.035\right)^2 \cdot (t - t0) + 1\right]$$

$$\beta dc\_fcm1 := \frac{412}{(-fcm1)^{1.4}} = 1.588$$

$$\beta dc\_RH := \frac{1 - \frac{RH}{100}}{\sqrt[3]{0.1 \cdot \frac{hn}{100}}} = 0.724$$

$$\beta dc\_t0(t0) := \frac{1}{0.1 + t0\_adj(t0)^{0.2}}$$

$$\gamma(t0) := \frac{1}{2.3 + \frac{3.5}{\sqrt{t0\_adj(t0)}}}$$

$$\alpha cm1 := \left(\frac{35}{-fcm1}\right)^{0.5} = 0.813$$

$$\beta h := \min(1.5 \cdot hn + 250 \cdot \alpha cm1, 1500 \cdot \alpha cm1) = 698.159$$

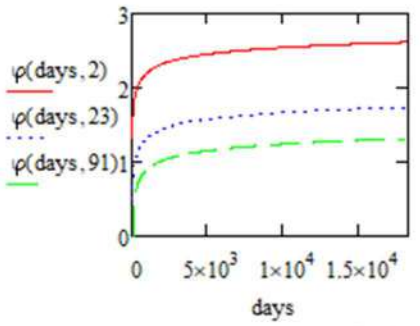
$$\beta dc\_t\_t0(t,t0) := \left[\frac{(t - t0)}{\beta h + (t - t0)}\right]^{\gamma(t0)}$$

$$\varphi dc(t,t0) := \beta dc\_fcm1 \cdot \beta dc\_RH \cdot \beta dc\_t0(t0) \cdot \beta dc\_t\_t0(t,t0)$$

$$\varphi bc(t,t0) := \beta bc\_fcm1 \cdot \beta bc\_t\_t0(t,t0)$$

$$\varphi(t,t0) := \varphi bc(t,t0) + \varphi dc(t,t0)$$

$$t := 50 \cdot 365 = 1.825 \times 10^4 \qquad \varphi(t,2) = 2.615 \qquad \varphi(t,91) = 1.312$$



10.7 Non-linear moment-curvature diagram

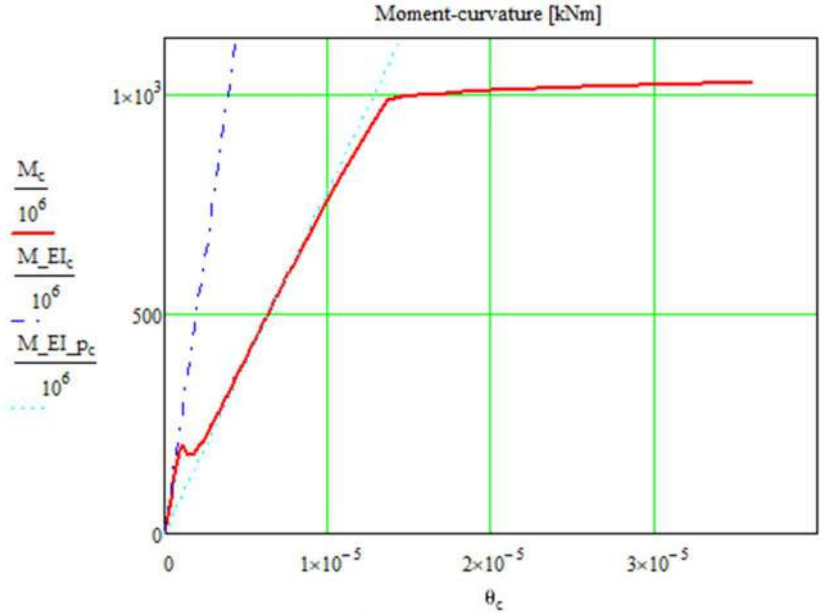
Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

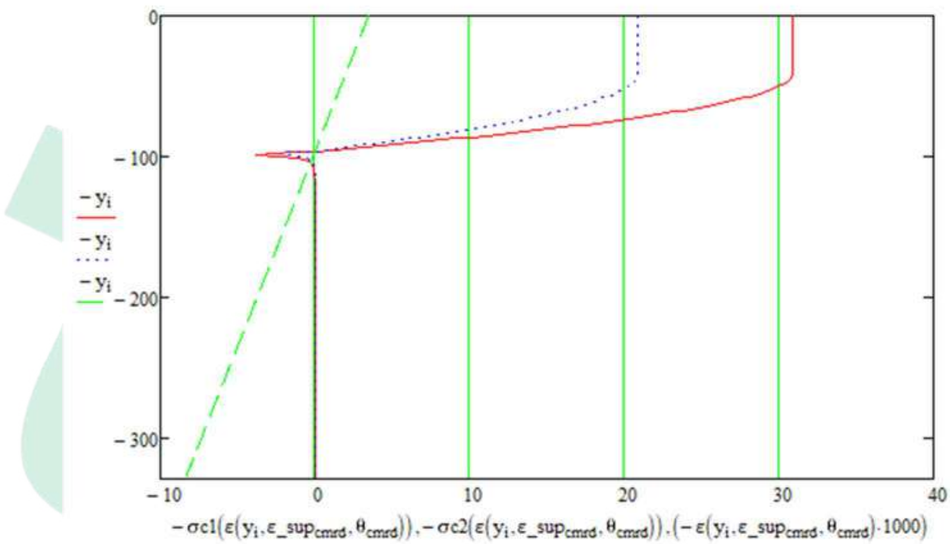
$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{265}\left(\sigma c2\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\right) + \sum_{i=266}^{Htot}\left[\sigma c1\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot\left(b\left(y_i\right)\right)\cdot\Delta y\right] + \sum_{j=1}^{js}\left(\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\right)$$

$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{265}\left[\sigma c2\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\cdot\left(y_i - yG\right)\right] + \sum_{i=266}^{Htot}\left[\sigma c1\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot\left(b\left(y_i\right)\right)\cdot\Delta y\cdot\left(y_i - yG\right)\right] + \sum_{j=1}^{js}\left[\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\cdot\left(ds_j - yG\right)\right]$$

Design external axial load

$NS := -0$





Condition at resisting (peak) moment
(stress and strain)

10.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformally distributed load q

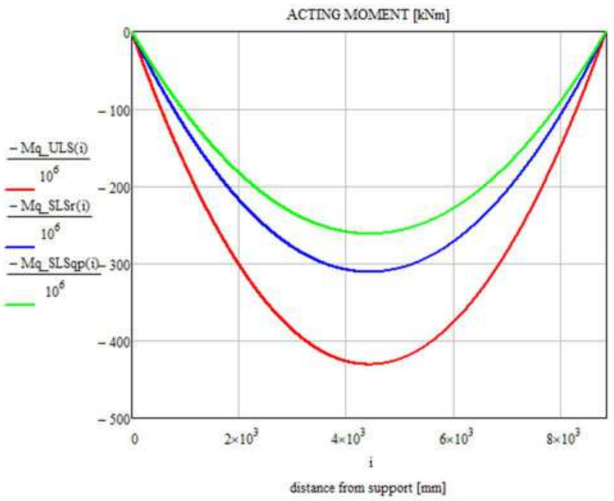
$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformally distributed load q

$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformally distributed load q

$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformally distributed load q

$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformally distributed load q

$i := 0..L$



10.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$$v\_inf\_p(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot 50,14) - \varphi(365\cdot 50,23)) + v\_SLSg2(x)\cdot(1 + \varphi(365\cdot 50,23))$$

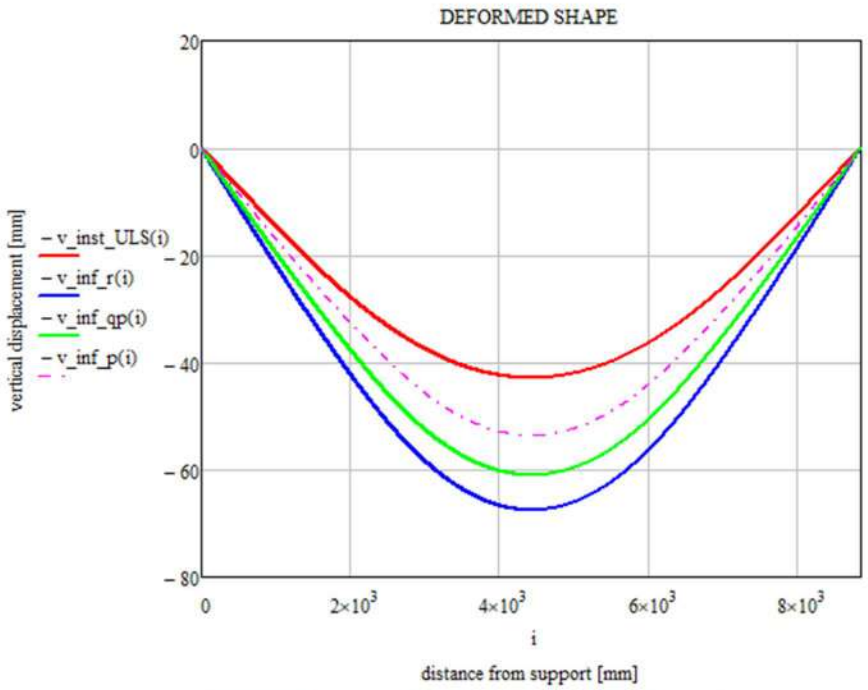
deflection profile at 50 years including creep for permanent load combination

$$v\_inf\_qp(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot 50,14) - \varphi(365\cdot 50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot 50,23) - \varphi(365\cdot 50,91)) + v\_SLSqp(x)\cdot(1 + \varphi(365\cdot 50,91))$$

deflection profile at 50 years including creep for quasi permanent load combination

$$v\_inf\_r(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot 50,14) - \varphi(365\cdot 50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot 50,23) - \varphi(365\cdot 50,91)) + v\_SLSqp(x)\cdot\varphi(365\cdot 50,91) + v\_SLSr(x)$$

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§9.3.4)

camber := 35 mm < $\frac{L}{250} = 35.4$ CHECK maximum deflection

camber induced by shaping the mould

$v\_inf\_r\left(\frac{L}{2}\right) - camber = 32.537$ < $\frac{L}{250} = 35.4$ CHECK maximum camber

value calculated from differential equations above

SLS STRESS CONTROL (§9.2.1)

$k1 := 0.6$ $\quad$ $rsup := 1.05$

$k2 := 0.45$

$k3 := 0.8$ $\quad$ $rinf := 0.95$

$k4 := 1$

$k5 := 0.8$ $\quad$ 0.75 in EN1992-1-1:2002

NOTE: the denomination of the allowable stress coefficients following k factors was kept similar to that of EN1992-1-1:2002

$$\sigma cpf\_bot(x) := \frac{Mq\_SLSf(x)\cdot(Htot - Yid)}{Ixo\_id} \qquad \sigma cpf\_bot\left(\frac{L}{2}\right) = 5.925 \quad < \quad fctm1 = 3.795$$ CHECK

elastic stress of bottom concrete chord for frequent load combination

if not -> cracked

$$\sigma sf\_bot(x) := 15\cdot\left[\frac{Mq\_SLSf(x)\cdot\left(ds_{js} - Yid\right)}{Ixo\_id}\right] \qquad \sigma sf\_bot\left(\frac{L}{2}\right) = 68.62$$

elastic stress of bottom mild steel layer for frequent load combination

$$\sigma cpr\_bot(x) := \frac{Mq\_SLSr(x)\cdot(Htot - Yid)}{Ixo\_id} \qquad \sigma cpr\_bot\left(\frac{L}{2}\right) = 6.681 \quad < \quad fctm1 = 3.795$$

elastic stress of bottom concrete chord for rare load combination

$$\sigma cpr\_top(x) := \frac{nc\cdot Mq\_SLSr(x)\cdot(-Yid)}{Ixo\_id} \qquad \sigma cpr\_top\left(\frac{L}{2}\right) = -6.553 \quad > \quad k1\cdot fck2 = -15$$ CHECK

$$> \quad 0.4\cdot fcm2 = -13.2$$

elastic stress of top concrete chord for rare load combination

$$\sigma cpr\_s(x) := 15\cdot\left[\frac{Mq\_SLSr(x)\cdot\left(ds_{js} - Yid\right)}{Ixo\_id}\right] \qquad \sigma cpr\_s\left(\frac{L}{2}\right) = 77.38 \quad < \quad k3\cdot fsk = 400$$ CHECK

creep stress of bottom mild steel for rare load combination

SLS CRACK CONTROL (§9.2.3)

$$c\_act := Htot - ds_{js} - 10 = 25$$

$$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1.25$$

$$wlim\_cal := 0.2 \cdot ksurf = 0.25 \quad \text{mm}$$

$$kw := 1.7$$

$$ayi := Htot - ds_{js} = 35 \qquad \phi := 24$$

$$k1\_r := \frac{Htot - Yn\_n}{Htot - ayi - Yn\_n} = 1.17$$

$$kfl := \frac{Htot - \min(ayi + 5 \cdot \phi, ayi \cdot 3.5)}{Htot} = 0.629$$

$$kb := 1.2$$

$$Aceff := 0.5 \cdot b(Htot) \cdot \min(ayi + 5 \cdot \phi, ayi \cdot 3.5) = 1.47 \times 10^{5}$$

$$\rho peff := \frac{As_{js} + As_{js-1}}{Aceff} = 0.06$$

$$srmcal := \min\left[1.5 \cdot \left(Htot - ds_{js} + \frac{\phi}{2}\right) + \frac{kfl \cdot kb}{7.2} \cdot \frac{\phi}{\rho peff}, \frac{1.3}{kw} \cdot (Htot - Yn\_n)\right] = 112.652$$

$$kt := 0.4 \qquad \text{NOTE : 0.6 for sustained loading}$$

$$fcteff := fctm1 = 3.795$$

$$\varepsilon sm\_\varepsilon cm := \max\left[\frac{\sigma sf\_bot\left(\frac{L}{2}\right) - kt \cdot \frac{fcteff}{\rho peff} \cdot \left(1 + \frac{Es}{Ecm1} \cdot \rho peff\right)}{Es}, (1 - kt) \cdot \frac{\sigma sf\_bot\left(\frac{L}{2}\right)}{Es}\right] = 2.059 \times 10^{-4}$$

$$wkcal := kw \cdot k1\_r \cdot srmcal \cdot \varepsilon sm\_\varepsilon cm = 0.046 \quad < \quad wlim\_cal = 0.25 \qquad \text{CHECK}$$

10.10 ULS checks

ULS BENDING-AXIAL CONTROL (§8.1)

$$Mrd = 1.026 \times 10^{3} \quad > \quad \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^{6}} = 430.872 \qquad \text{CHECK}$$

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§8.2)

$$Vq\_ULS(x) := \left|(g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2} - x\right)\right|$$

$d := ds_{js} = 295$ mm

$VEd := Vq\_ULS(d) = 1.818 \times 10^5$ N

Dlower := 16 mm

$\gamma v := 1.3$

$bw := 2400$ mm

$z := 0.9\cdot d = 265.5$

$$ddg := \min\left[\mathrm{if}\left[-fck1 > 60, 16 + Dlower\cdot\left(\frac{60}{-fck1}\right)^2, 16 + Dlower\right], 40\right] = 32$$

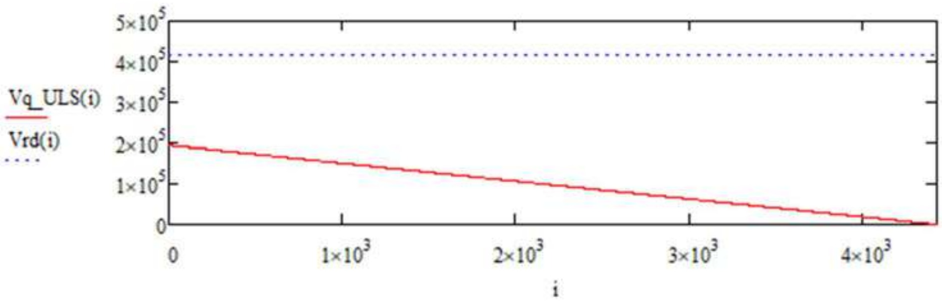
$\tau Ed := \frac{VEd}{bw\cdot z} = 0.285$ MPa

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT

$\tau Rdc\_min := \frac{11}{\gamma v}\cdot\sqrt{\frac{-fck2}{fsd}\cdot\frac{ddg}{d}} = 0.654$ MPa < $\tau Ed = 0.285$ CHECK

no more refined analysis required

$Vrd(x) := bw\cdot z\cdot\tau Rdc\_min$

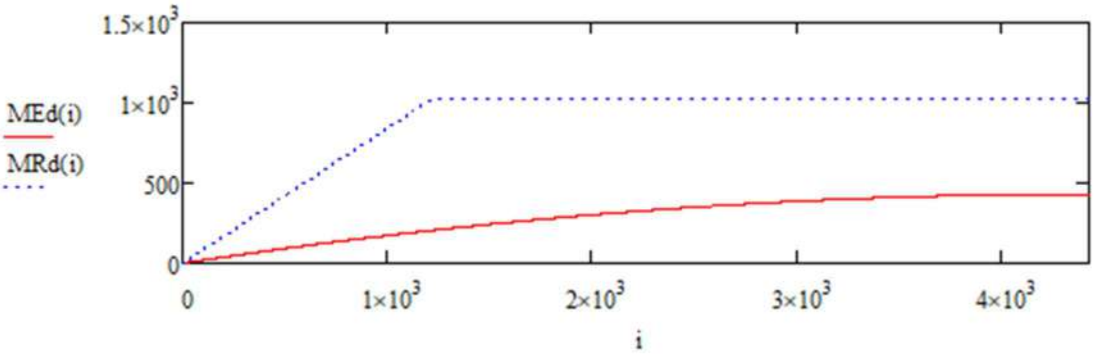


MOMENT DIAGRAM ACCOUNTING DUE TO SHEAR RESISTING MECHANISM (§12.3.2)

$\alpha 3 := 1$ fatigue check not required

$$MEd(x) := \frac{Mq\_ULS(x)}{10^6}$$

$$MRd(x) := \text{if}\left(x < 1220, Mrd \cdot \frac{x}{1220}, Mrd\right)$$



MINIMUM REINFORCEMENT (§12.2)

$kh := \text{if}[0.8 - 0.6\cdot(\min(bw, Htot) - 0.3) < 0.5, 0.5, \text{if}[0.8 - 0.6\cdot(\min(bw, Htot) - 0.3) > 0.8, 0.8, 0.8 - 0.6\cdot(\min(bw, Htot) - 0.3)]] = 0.5$ §9.2.2(2)

$fct\_eff := fctm2$

$As\_min\_w1 := 0.2 \cdot kh \cdot fct\_eff \cdot \frac{Ac}{fsk} = 406.29$ mm2 < $As\_tot = 9.085 \times 10^3$ mm2 CHECK §(9.2)

$fctm1 \cdot \frac{Ixo\_id}{(Htot - Yid) \cdot 10^6} = 176.862$ < $Mrd = 1.026 \times 10^3$ kNm CHECK §(12.1)

MINIMUM LONGITUDINAL REINFORCEMENT IN ORTHOGONAL DIRECTION RESPECT TO SLAB AXIS (§12.2(3))

$\frac{0.2 \cdot 18 \cdot A(24)}{2400} = 0.679$ < $\frac{A(12)}{200} + \frac{A(6)}{200} = 0.707$ CHECK

additional reinforcement in the transverse horizontal direction

CHECK OF SUPPORT MILD REBARS

$0.25 \cdot MEd\left(\frac{L}{2}\right) = 107.718$ kNm < $\left(10 \cdot \pi \cdot \frac{12^2}{4}\right) \cdot fsd \cdot 0.9 \cdot \frac{250}{10^6} = 115.668$ CHECK

§12.1(4)

ANCHORAGE (§11.4)

$klb := 50$

$kcp := 1$ for good bond conditions

$n\sigma := \frac{3}{2}$

$cs := 50$

$cx := 75$

$cy := 40$

$cd(\phi) := \min(0.5 \cdot cs, cx, cy, 3.75 \cdot \phi)$ $\quad cd(12) = 25$

$$lbd2(\phi) := \max\left[klb \cdot kcp \cdot \phi \cdot \left(\frac{fsd}{435}\right)^{n\sigma} \cdot \left(\frac{25}{-fck2}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1.5 \cdot \phi}{cd(\phi)}\right)^{\frac{1}{2}}, 10 \cdot \phi\right]$$

$lbd2(16) = 777.239$

length of straight part for 90° bent bars

$lb90(\phi) := \max(70, lbd2(\phi) - 15 \cdot \phi, 10 \cdot \phi)$

$lb90(12) = 278.67 \quad lb90(8) = 98.105$

length of straight part for 135° bent bars (stirrups)

$lb135(\phi) := \max(50, lbd2(\phi) - 15 \cdot \phi, 5 \cdot \phi)$

$lb135(12) = 278.67 \quad lb135(8) = 98.105$

INTERFACE BETWEEN CONCRETES CAST AT DIFFERENT TIME (§8.2.6)

$Ainter := 265 \cdot 2400 = 6.36 \times 10^{5}$ mm2

$\tau Edi(x) := \dfrac{Vq\_ULS(x)}{Ainter}$ $\tau Edi(0) = 0.306$ MPa

$cv1 := 0.15$ rough surface

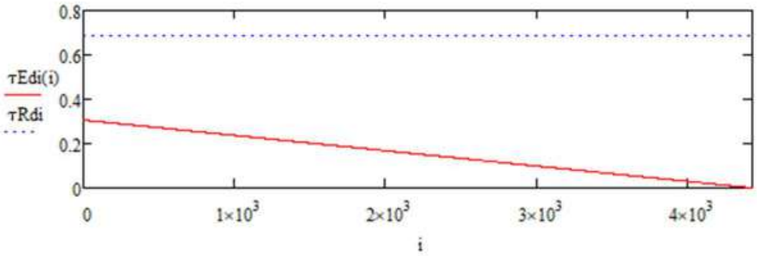
$\mu v := 0.7$

$\alpha inter := 75 \cdot \dfrac{\pi}{180} = 1.309$ rad

$\sigma n := 0$

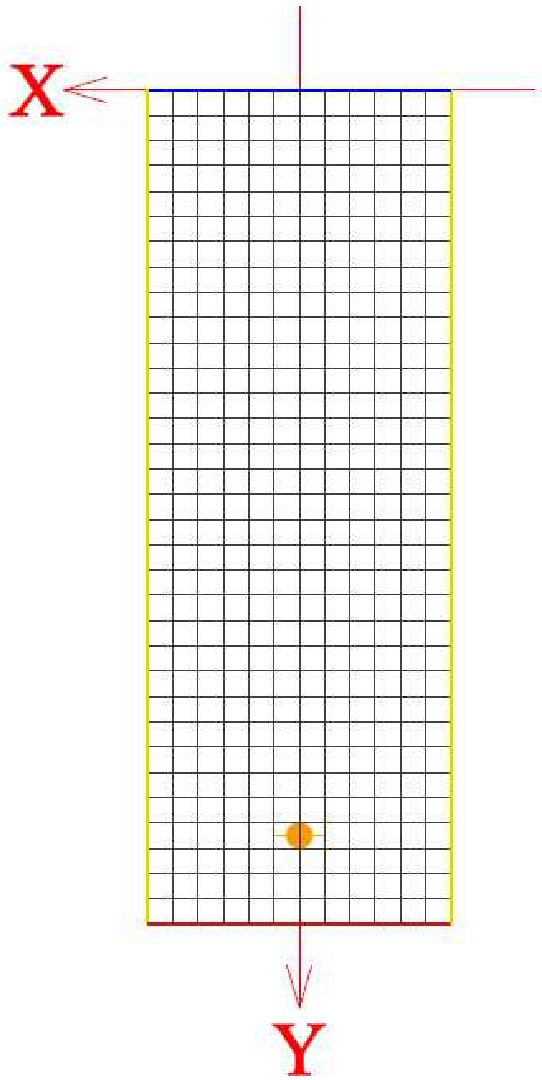
$\rho i := \dfrac{8 \cdot A(6)}{Ainter} = 3.557 \times 10^{-4}$

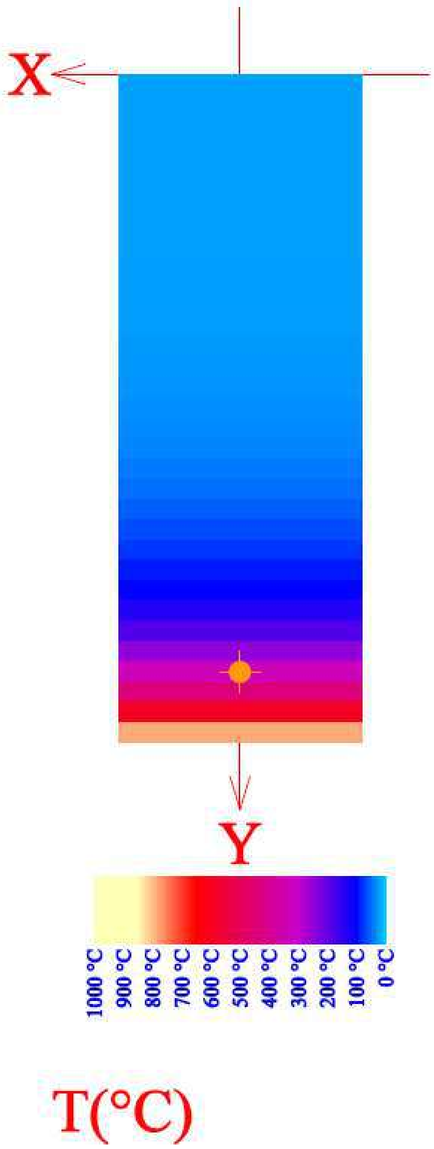
$\tau Rdi := \min\left[cv1 \cdot \dfrac{\sqrt{-fck2}}{\gamma c} + \mu v \cdot \sigma n + \rho i \cdot fsd \cdot (\mu v \cdot \sin(\alpha inter) + \cos(\alpha inter)), 0.3 \cdot -fcd2 + \rho i \cdot fsd \cdot \cos(\alpha inter) \right] = 0.687$ MPa



> $\tau Edi(0) = 0.306$ MPa CHECK

10.11 Fire checks





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100	100	100	100	100	100	100	100	100	100	100	100
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587	587	587	587	587	587	587	587	587	587	587	587
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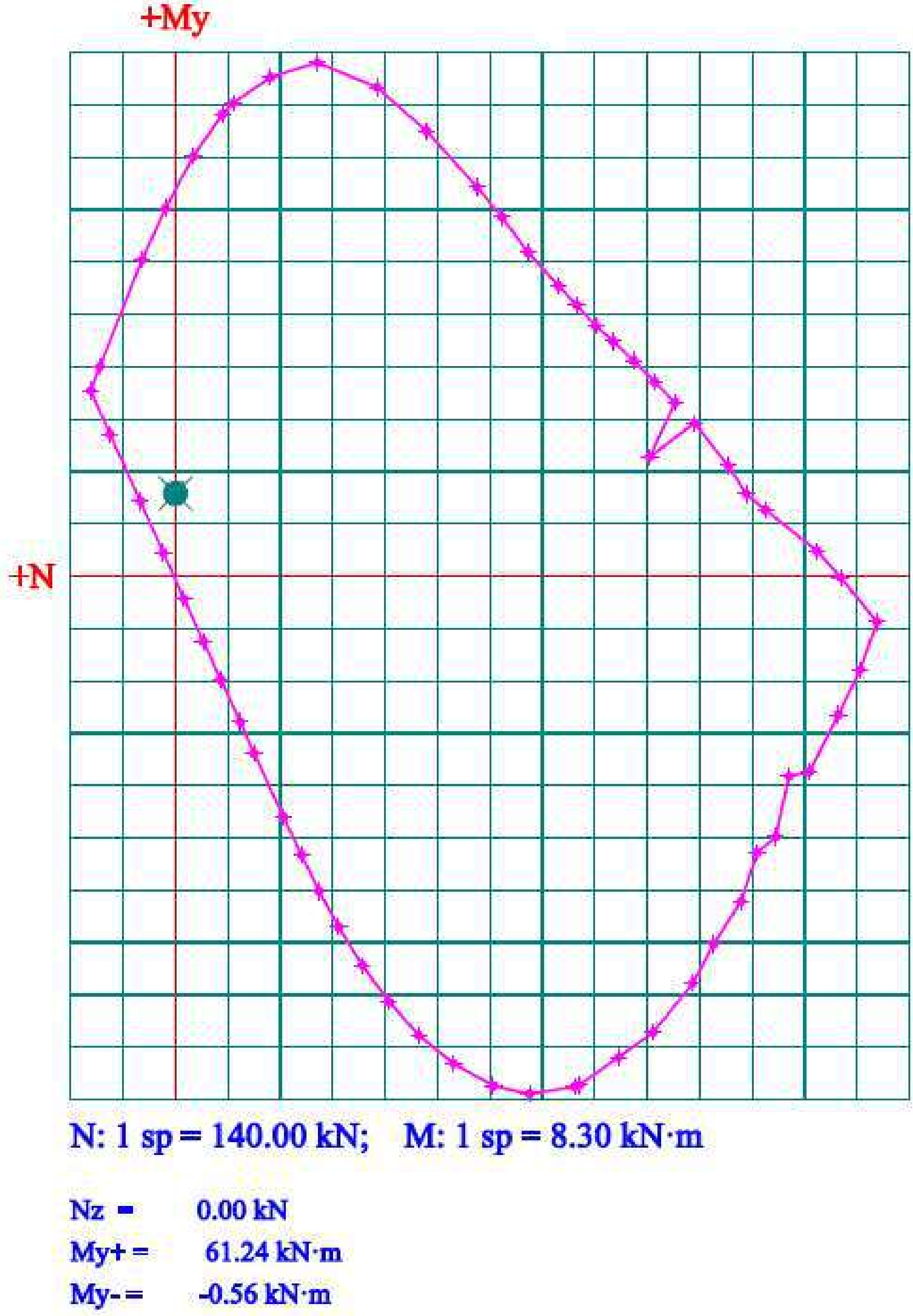
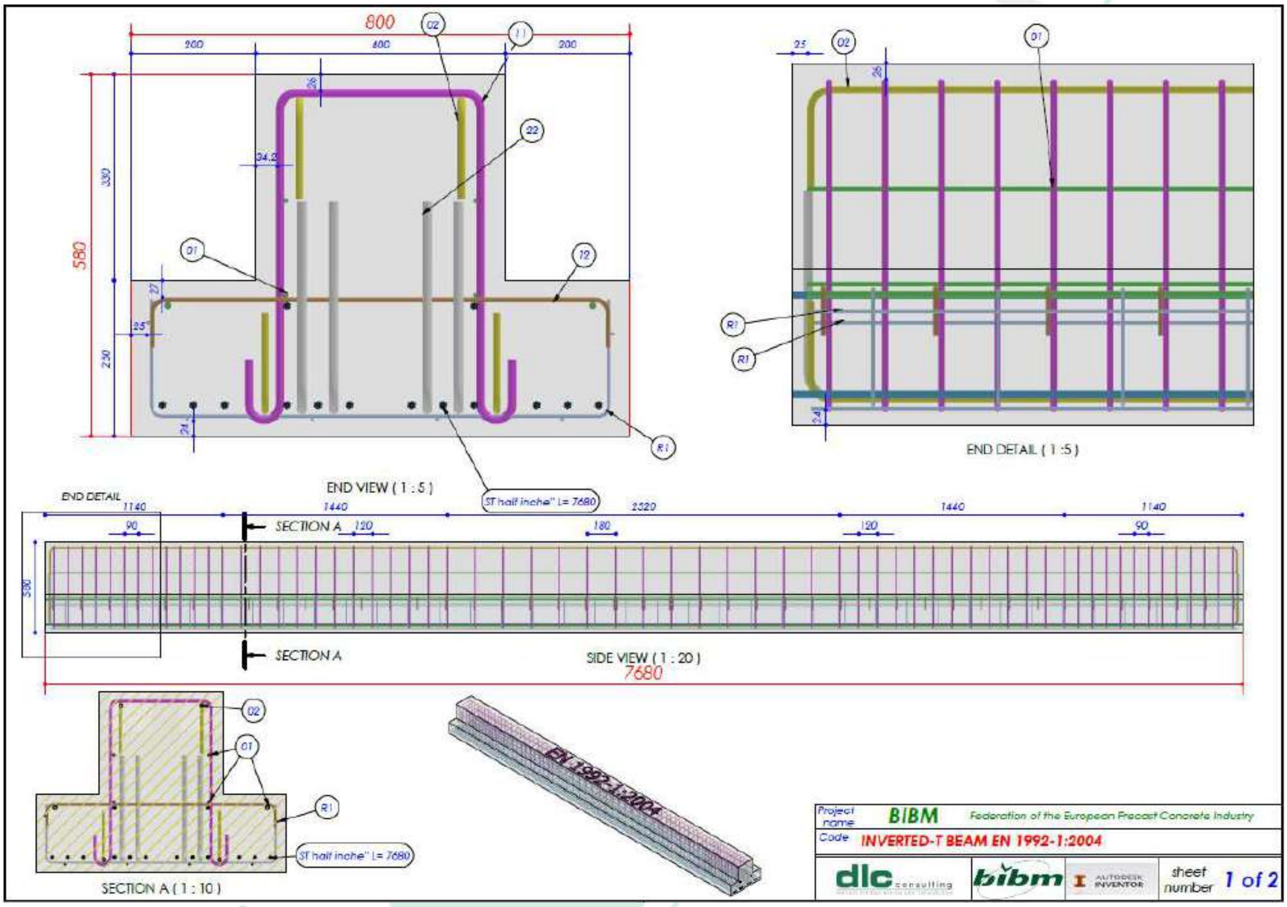
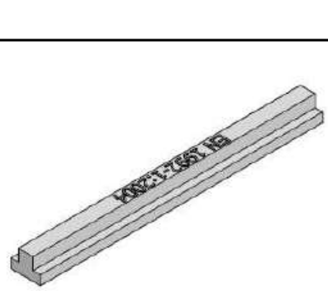

+My
+N
N: 1 sp = 140.00 kN;   M: 1 sp = 8.30 kN·m
Nz = 0.00 kN
My+ = 61.24 kN·m
My- = -0.56 kN·m

11 Prestressed beam element –EN1992-1:2004

11.1 Shop drawings



Thumbnail	art Numbe	QTY	Mass	Total mass	Ø_	_longitudir	pattern_T	_transvers	pattern_L
	01	4	3011	12044	8 mm				
	02	4	7018	28072	12 mm				
	03	2	6774	13548	12 mm				
	11	64	982	62848	10 mm				
	12	46	341	15686	8 mm				
	22	8	2378	19024	16 mm				
Total mass rebars [kg]				151,22	Incidence kg/m³		59,30		
	R1	1	19373	19373		6 mm	200 mm	6 mm	200 mm
mass welded-wire-meshes [kg]				19,37	Incidence kg/m³		7,60		
	t inche" L=	14	5612,5	78575	12,7 mm				
Total mass strands [kg]				78,575	Incidence kg/m³		30,81		
Total mass of steel [kg]				249,17		Total concrete volu	2,55		



Project name	BIBM	Federation of the European Precast Concrete Industry	
Code	INVERTED-T BEAM EN 1992-1:2004		
dlc consulting	bibm	AUTODESK INVENTOR	sheet number 2 of 2

11.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0 \;\; 329.99 \;\; 330 \;\; 580)^T$

$Htot := max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := (400 \;\; 400 \;\; 800 \;\; 800)^T$

$r\_circ := 0$ radius of central void pipe

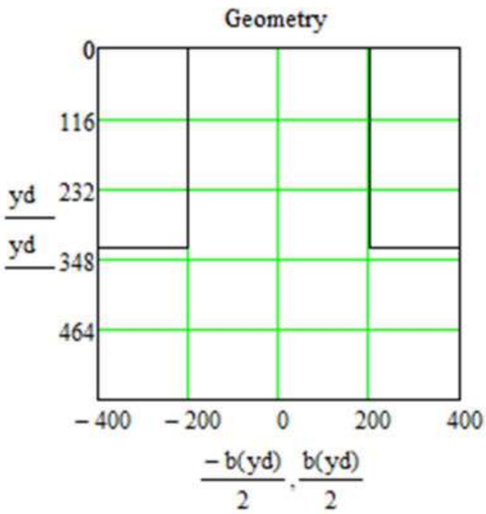
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$b\_lin(y) := linterp(y\_tr, b\_tr, y)$

$b\_circ(y) := linterp(y\_tr, b\_tr, y) - x\_circ(y)$

$$b(y) := if\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$yd := 0..Htot$



condensed 1D geometry plot

$u := 800 \cdot 2 + Htot \cdot 2 = 2.76 \times 10^3$ exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (43 \;\; 202 \;\; 354 \;\; 370 \;\; 538)^T$$

Area of reinforcement at each depth

$$As := (2\cdot A(12) \;\; 2\cdot A(8) \;\; 2\cdot A(8) \;\; 2\cdot A(8) \;\; 2\cdot A(12))^T$$

$js := \text{rows}(As)$ $\qquad js = 5$

$dsmax := \max(ds)$ $\qquad dsmax = 538$

$$As\_tot := \sum_{j=1}^{js} As_j = 753.982$$

Prestressing reinforcement

nc

Area of a single strand:

$Ap0 := 93$ $\qquad \phi p := 12.7$ mm nominal strand diameter

Depth of prestressing strands from upper chord:

$dp := (380 \;\; 480 \;\; 530)^T$

Area of strands at each depth:

$Ap := (2 \cdot Ap0 \;\; 0 \cdot Ap0 \;\; 12 \cdot Ap0)^T$

$\sigma p0 := 1400$ MPa

$\sigma prec := (0.4 \cdot \sigma p0 \;\; \sigma p0 \;\; \sigma p0)^T$ initial prestressing

$perdite := 0 \cdot (1 \;\; 1 \;\; 1)^T$ in percentual **% (losses are introduced later)**

$jp := rows(Ap)$ $\qquad jp = 3$

$k := 1 .. jp$

$$\sigma o_k := \sigma prec_k \cdot \left[\frac{(100 - perdite_k)}{100}\right] \qquad \sigma o = \begin{pmatrix} 560 \\ 1.4 \times 10^3 \\ 1.4 \times 10^3 \end{pmatrix}$$

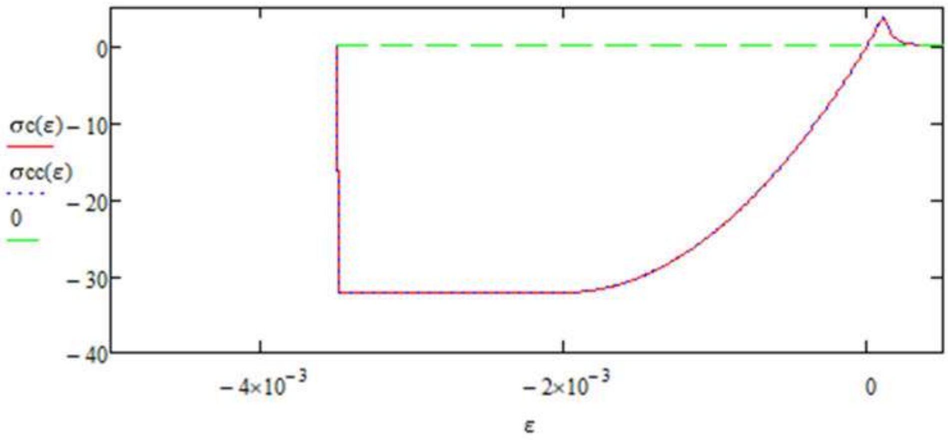
$$Ap\_tot := \sum_{k=1}^{jp} Ap_k \qquad Ap\_tot = 1.302 \times 10^3$$

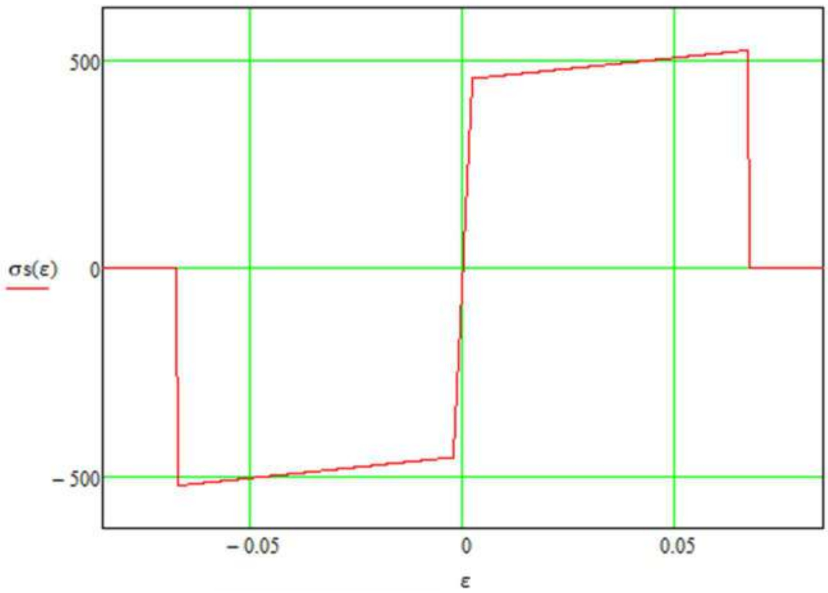
$ypmax := max(dp)$ $\qquad ypmax = 530$

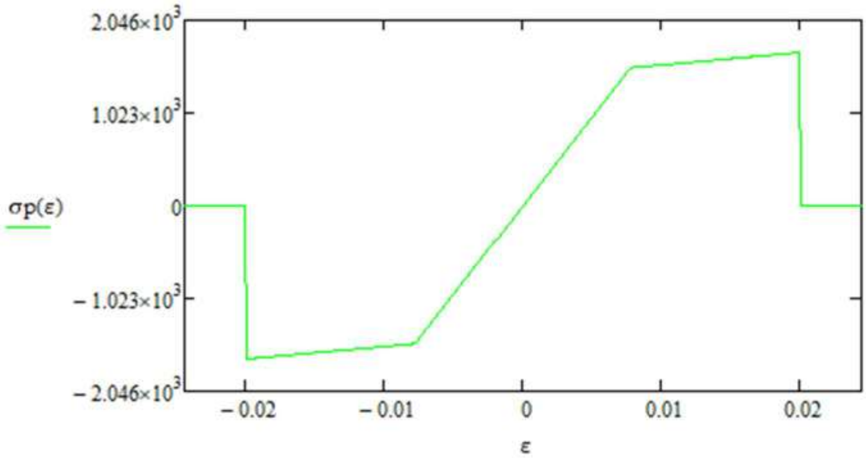
$$Np\_tot := \sum_{k=1}^{jp} \left(\left(Ap_k \cdot \sigma o_k\right)\right) \qquad Np\_tot = 1.667 \times 10^6 \text{ N}$$ total prestressing initial force

$$Yp := \frac{\sum_{k=1}^{jp} \left(dp_k \cdot Ap_k \cdot \sigma o_k\right)}{\sum_{k=1}^{jp} \left(Ap_k \cdot \sigma o_k\right)} = 520.625 \text{ mm}$$ centre of gravity of prestressing

11.3 Material constitutive laws employed in the calculation







11.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 3.321 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 2.27 \times 10^{-3}$$ geometric ratio for longitudinal mild reinforcement

$$\rho p := \frac{Ap\_tot}{Ac} = 3.921 \times 10^{-3}$$ geometric ratio for longitudinal prestressing tendons

$$\rho tot := \frac{As\_tot + Ap\_tot}{Ac} = 6.191 \times 10^{-3}$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y) \cdot y\,dy \qquad Syc = 1.128 \times 10^8$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 339.696$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y) \cdot (y - yG)^2\,dy \qquad Ixo\_cls = 8.927 \times 10^9$$

Global area of all prestressing reinforcement

$$Area\_tr := \left| \begin{array}{l} s \leftarrow 0 \\ \text{for } x \in 1..jp \\ \quad s \leftarrow Ap_x + s \end{array} \right. \qquad Area\_tr = 1.302 \times 10^3$$

First moment of the area referred to prestressing reinforcement only

$$Sxp := \sum_{i=1}^{jp} \left(Ap_i \cdot dp_i\right) \qquad Sxp = 6.622 \times 10^5$$

Centre of gravity of prestressing

$$Yp := \frac{Sxp}{Area\_tr} \qquad Yp = 508.571$$

Idealisation coefficients (elastic)

$$np := \frac{Ep}{Ecm} \qquad np = 5.374$$

$$ns := \frac{Es}{Ecm} \qquad ns = 5.512$$

Area of ideal cross-section

$$Aid := Ac + (np-1)\cdot\sum_{j=1}^{jp} Ap_j + (ns-1)\cdot\sum_{j=1}^{js} As_j \qquad Aid = 3.412\times10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac\cdot yG + (np-1)\cdot(Area\_tr\cdot Yp) + (ns-1)\cdot\sum_{j=1}^{js}\left(As_j\cdot ds_j\right) \qquad Sxid = 1.167\times10^8$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 342.097$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y)\cdot(y-Yid)^2\,dy - \sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right] - \sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the prestressing reinforcement area

$$Ixoidprec := np\cdot\sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right]$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns\cdot\sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidprec + Ixoidlenta \qquad Ixo\_id = 9.242\times10^9 \quad mm^4 \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.035$$

11.5 Loads

LOADS

$g1 := 8.3$ kN/m dead load from self-weight

$g2 := (2 + 2.89) \cdot 9.45 = 46.211$ kN/m nonstructural dead load

$q := 28.35$ kN/m live load

$L := 7500$ mm calculation length (span between supports)

$\psi 2 := 0.3$ non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ SLS bending moment distribution from live load

11.6 Prestressing transfer and time-dependent behaviour

TRANSFER OF PRESTRESS (§8.10.2.2)

$\alpha 1 := 1$ gradual release of prestressing

$\alpha 2 := 0.19$ for 7-wire strands

$\sigma pm0 := \sigma p0 = 1.4 \times 10^3$

$\eta p1 := 3.2$ for 7-wire strands

$\eta 1 := 1$ in favourable position

$fbpt := \eta p1 \cdot \eta 1 \cdot fctdj(2) = 3.51$ MPa equivalent constant bond stress at prestress realease following §(8.15)

$lpt := \frac{\alpha 1 \cdot \alpha 2 \cdot \sigma pm0}{fbpt} \cdot \phi p = 962.587$ mm basic value of the transmission length following §(8.16)

$lpt1 := 0.8 lpt = 770.069$ mm lower-bound transfer length following §(8.17)

$lpt2 := 1.2 \cdot lpt = 1.155 \times 10^3$ mm upper-bound transfer length following §(8.18)

Prestress losses

$$hn := 2 \cdot \frac{Ac}{u} = 240.643$$

$$\varepsilon cs := \frac{0.65}{1000} = 6.5 \times 10^{-4}$$ shrinkage strain assumed as a result of laboratory tests on the specific concrete mix employed

$\rho 1000 := 0.025$ for class 2 (low-relaxation) tendons following §3.3.2(5)

$k\rho := 0.16$

$t := 50 \cdot 365 = 1.825 \times 10^{4}$ days Life span

$$\sigma cpQP2(x) := \frac{-Np\_tot}{Aid} + \frac{[Mq\_SLSg1(x) - Np\_tot \cdot (Yp - Yid)] \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP2\left(\frac{L}{2}\right) = -8.831$$

stress in quasi-permanent load combination at 2 days
(conventional equivalent time for prestressing release)

$$\sigma cpQP23(x) := \frac{Mq\_SLSg2(x) \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP23\left(\frac{L}{2}\right) = 5.852$$

stress in quasi-permanent load combination at 23 days
(conventional time for assemblage of the structure on site)

$$\sigma cpQP91(x) := \frac{Mq\_SLSq(x) \cdot (Yp - Yid)}{Ixo\_id} \qquad \sigma cpQP91\left(\frac{L}{2}\right) = 1.077$$

stress in quasi-permanent load combination at 91 days
(conventional time for enter in use of the structure)

$$\Delta\sigma pr(x,t) := \left[\sigma p0 + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x))\right] \cdot \rho 1000 \cdot \left(\frac{24 \cdot t}{1000}\right)^{k\rho}$$

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$h0 := 2 \cdot \frac{Ac}{u} = 240.643$ mm notional size of the member

$RH := 50$ % relative humidity

$t0\_T(t0) := t0$

$\alpha := 1$ for cement class R

$$t0\_mod(t0) := \max\left[t0\_T(t0) \cdot \left(\frac{9}{2 + t0\_T(t0)^{1.2}} + 1\right)^{\alpha}, 0.5\right]$$

time modification due to type of cement §B.9

$t0\_mod(2) = 6.189$

$$\alpha c1 := \left(\frac{35}{-fcm}\right)^{0.7} = 0.748$$

$$\alpha c2 := \left(\frac{35}{-fcm}\right)^{0.2} = 0.92$$

$$\alpha c3 := \left(\frac{35}{-fcm}\right)^{0.5} = 0.813$$

$$\beta h := \text{if}\left[-fcm > 35, \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250 \cdot \alpha c3, 1500 \cdot \alpha c3\right], \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250, 1500\right]\right] = 564.161$$

$$\beta t0(t0) := \frac{1}{0.1 + t0\_mod(t0)^{0.2}}$$

$$\beta c(t, t0) := \left(\frac{t - t0\_mod(t0)}{\beta h + t - t0\_mod(t0)}\right)^{0.3}$$

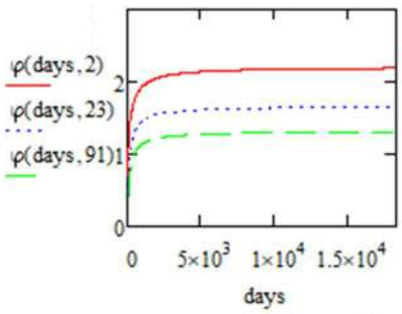
$$\beta fcm := \frac{16.8}{\sqrt{-fcm}} = 2.308$$

$$\varphi RH := \text{if}\left[-fcm > 35, \left(1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}} \cdot \alpha c1\right) \cdot \alpha c2, 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}}\right] = 1.474$$

$\varphi 0(t0) := \varphi RH \cdot \beta fcm \cdot \beta t0(t0)$

$\varphi(t, t0) := \varphi 0(t0) \cdot \beta c(t, t0)$

$\varphi(t, 2) = 2.188$ $\varphi(t, 91) = 1.304$



TIME-DEPENDENT LOSSES OF PRESTRESS (§5.10.6)

$$\Delta\sigma p\_csr(x,t) := \frac{-\varepsilon cs \cdot Ep - 0.8\cdot\Delta\sigma pr(x,t) + \frac{Ep}{Ecm}\cdot(\sigma cpQP2(x)\cdot\varphi(t,2) + \sigma cpQP23(x)\cdot\varphi(t,23) + \sigma cpQP91(x)\cdot\varphi(t,91))}{1 + \frac{Ep}{Ecm}\cdot\frac{Ap\_tot}{Ac}\cdot\left[1 + \frac{Ac}{Ixoidcls}\cdot(Yp - Yid)^2\right]\cdot\left(1 + 0.8\cdot\frac{\varphi(t,2)\cdot\sigma cpQP2(x) + \varphi(t,23)\cdot\sigma cpQP23(x) + \varphi(t,91)\cdot\sigma cpQP91(x)}{\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)}\right)}$$

prestress losses following §(5.46)

NOTE: a weighed creep coefficient was considered accounting for the 3 load phases previously introduced

$$\sigma pm(x,t) := \sigma p0 - \frac{Ep}{Ecm}\cdot(\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)) + \Delta\sigma p\_csr(x,t)$$ prestress considering immediate and delayed losses

$$\frac{\sigma pm\left(\frac{L}{2}, 365\cdot 50\right)}{\sigma p0} = 0.861$$ expected residual prestress ratio after 50 years of life with respect to initial

$$\varepsilon pm := \frac{\sigma pm\left(\frac{L}{2}, 365\cdot 50\right)}{\sigma p0}\cdot\varepsilon p0$$ expected residual strain after 50 years of life with respect to initial

$$\sigma pm\left(\frac{L}{2}, 365\cdot 50\right)\cdot Ap\_tot = 1.569\times 10^6$$ residual prestress force after 50 years of life

$Np\_tot = ▮$ initial prestress force

11.7 Non-linear moment-curvature diagram

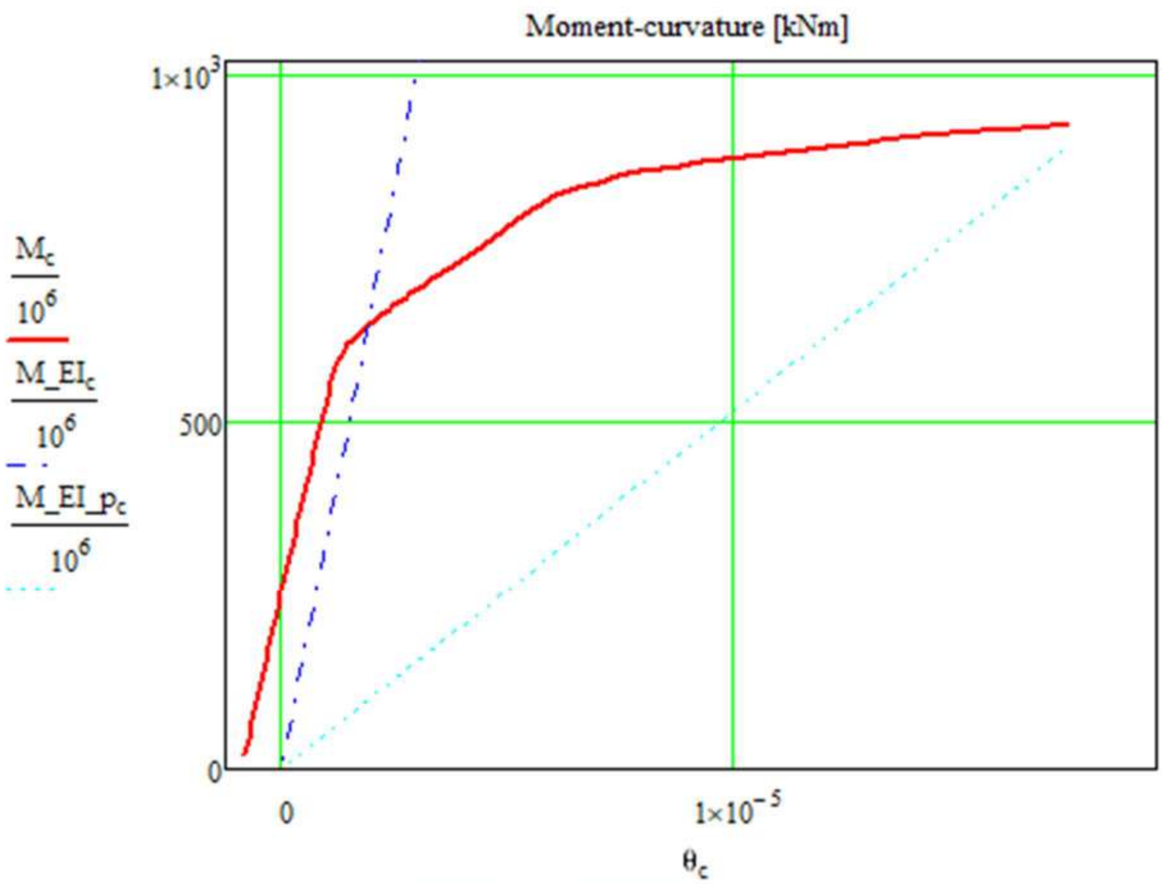
Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

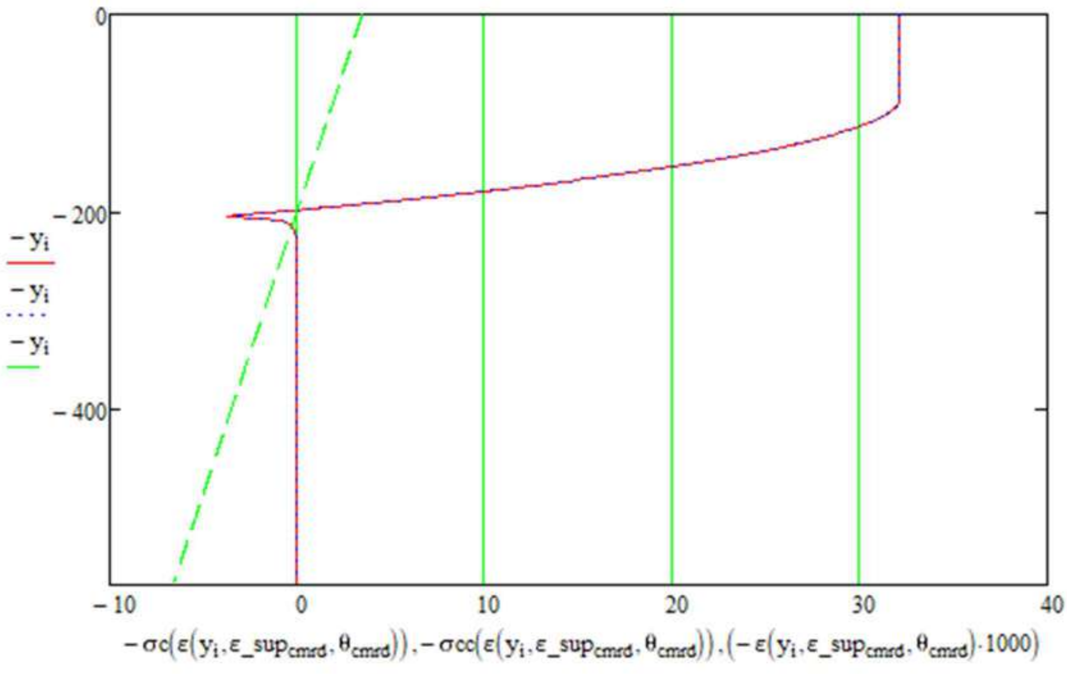
$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot}\left(\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\right) + \sum_{j=1}^{jp}\left(\sigma p\left(\varepsilon\left(dp_j,\varepsilon\_sup,\theta\right) + \varepsilon pm_j\right)\cdot Ap_j\right) + \sum_{j=1}^{js}\left(\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\right)$$

$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot}\left[\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\cdot\left(y_i - yG\right)\right] + \sum_{j=1}^{jp}\left[\sigma p\left(\varepsilon\left(dp_j,\varepsilon\_sup,\theta\right) + \varepsilon pm_j\right)\cdot Ap_j\cdot\left(dp_j - yG\right)\right] + \sum_{j=1}^{js}\left[\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\cdot\left(ds_j - yG\right)\right]$$

Design external axial load

$NS := -0$





Condition at resisting (peak) moment (stress and strain)

11.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformally distributed load q

$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformally distributed load q

$lpt = 962.587$

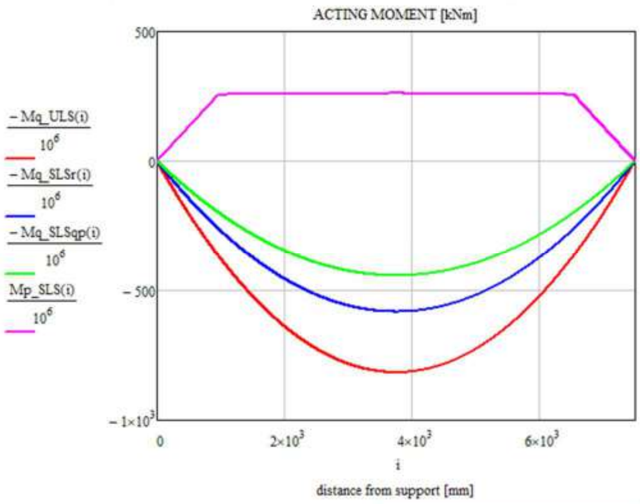
$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformally distributed load q

$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformally distributed load q

$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformally distributed load q

$$Mp\_SLS(x) := if\left[x < lpt, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{x}{lpt}, if\left[x > L - lpt, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid) \cdot \frac{-x + L}{lpt}, \sigma pm(x, 365 \cdot 50) \cdot Ap\_tot \cdot (Yp - Yid)\right]\right]$$

$i := 0..L$ contribution of prestressing equivalent load in SLS (without modification factors)



11.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$v\_inf\_p(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot 50,2) - \varphi(365\cdot 50,23)) + v\_SLSg2(x)\cdot(1 + \varphi(365\cdot 50,23))$

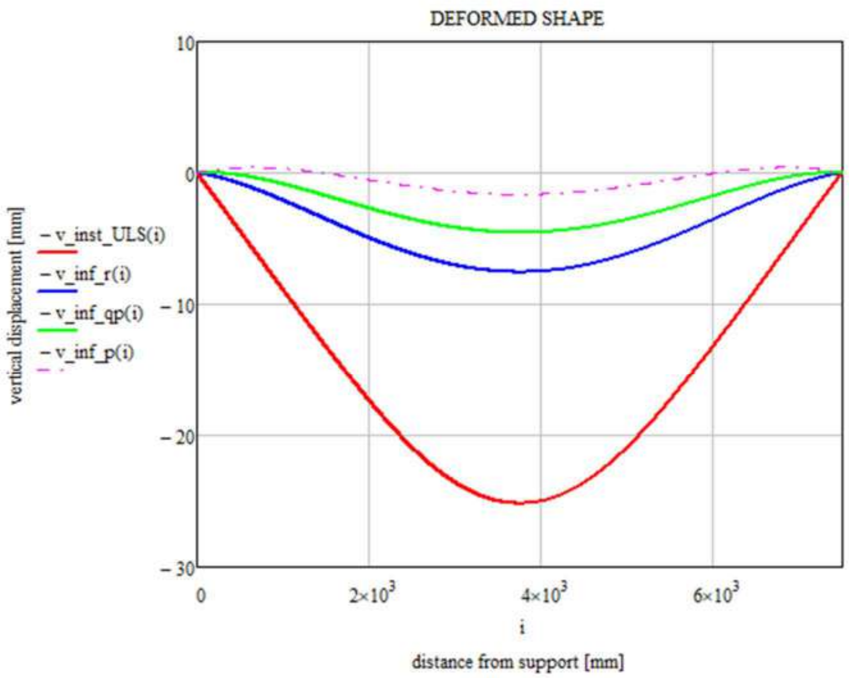
deflection profile at 50 years including creep for permanent load combination

$v\_inf\_qp(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot 50,2) - \varphi(365\cdot 50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot 50,23) - \varphi(365\cdot 50,91)) + v\_SLSqp(x)\cdot(1 + \varphi(365\cdot 50,91))$

deflection profile at 50 years including creep for quasi permanent load combination

$v\_inf\_r(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot 50,2) - \varphi(365\cdot 50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot 50,23) - \varphi(365\cdot 50,91)) + v\_SLSqp(x)\cdot\varphi(365\cdot 50,91) + v\_SLSr(x)$

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§7.4.3)

$v\_inf\_r\left(\frac{L}{2}\right) = 7.564$ < $\frac{L}{250} = 30$ CHECK maximum deflection

values calculated from differential equations above

$v\_inf\_p\left(\frac{L}{2}\right) = 1.638$ > $\frac{-L}{250} = -30$ CHECK maximum camber

SLS STRESS CONTROL (§7.2)

$k1 := 0.6$ $\quad$ $rsup := 1.05$

$k2 := 0.45$ $\quad$ prestressing modification coefficients

$k3 := 0.8$ $\quad$ $rinf := 0.95$

$k4 := 1$

$k5 := 0.75$

$$\sigma cpg1\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for selfweight loads only

$$\sigma cpg1\_top(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id}$$

elastic stress of top concrete chord for selfweight loads only

$$\sigma cpg1\_tops(x) := \frac{Es}{Ecm} \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(ds_1 - Yid\right)}{Ixo\_id} \right]$$

elastic stress of top series of mild steel for selfweight loads only

$$\sigma cpf\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSf(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for frequent load combination

$$\sigma cpr\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for rare load combination

$$\sigma cpr\_top(x) := \frac{-Np\_tot \cdot rinf}{Aid} + \frac{[Mq\_SLSr(x) - rinf \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id}$$

elastic stress of top concrete chord for rare load combination

$$\sigma cpr\_p(x) := \sigma pm(x,t) \cdot rsup + 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(dp_{jp} - Yid\right)}{Ixo\_id} \right]$$

creep stress of bottom prestressing steel for rare load combination

$$\sigma cpr\_s(x) := 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot \left(ds_{js} - Yid\right)}{Ixo\_id} \right]$$

creep stress of bottom mild steel for rare load combination

$\sigma cpg1\_bot(lpt1) = -12.074$	>	$k1 \cdot fckj(2) = -23.178$	CHECK
	>	$k2 \cdot fck = -20.25$	
$\sigma cpg1\_top(lpt1) = 4.858$	<	$fctmj(2) = 2.193$	CHECK
$\sigma cpg1\_tops(lpt1) = 19.858$	<	$k3 \cdot fsk = 400$	
$\sigma cpf\_bot\left(\frac{L}{2}\right) = -0.196$	<	$fctm = 3.795$	CHECK
$\sigma cpr\_bot\left(\frac{L}{2}\right) = 2.369$	<	$fctm = 3.795$	
$\sigma cpr\_top\left(\frac{L}{2}\right) = -16.45$	>	$k1 \cdot fck = -27$	CHECK
	>	$0.4 \cdot fcm = -21.2$	
$\sigma cpr\_p\left(\frac{L}{2}\right) = 1.277 \times 10^3$	<	$k5 \cdot fptk = 1.395 \times 10^3$	CHECK
$\sigma cpr\_s\left(\frac{L}{2}\right) = 15.685$	<	$k3 \cdot fsk = 400$	CHECK

SLS CRACK CONTROL (§7.3)

$c\_act := Htot - ds_{js} - 10 = 32$

$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1.5$

$wlim\_cal := 0.2$ mm

$w\_freq := 0$ < $wlim\_cal = 0.2$ CHECK

11.10 ULS checks

ULS BENDING-AXIAL CONTROL (§6.1)

$$Mrd = 927.011 \quad > \quad \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6} = 816.428$$ CHECK

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§6.2)

$$Vq\_ULS(x) := \left|(g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2} - x\right)\right|$$ shear distribution at Ultimate Limit State (ULS)

$d := Yp = 508.571$ mm — effective depth

$VEd := Vq\_ULS(d) = 3.764 \times 10^5$ N — maximum shear at effective depth from support

$bw := 400$ mm — web width

$z := 0.9\cdot d = 457.714$ — conventional resultant lever arm

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT (§6.2.2)

$$\rho l := \frac{Ap\_tot + \sum_{j=1}^{js} As_j}{bw\cdot d} = 0.01$$

reinforcement ratio

NOTE: the reinforcement ratio is assumed constant due to the introduction of additional support reinforcement which compensates the progressive anchorage of strands

$$\sigma cp(x) := \sigma pm(x,t)\cdot\frac{Ap\_tot}{Ac}$$

axial stress induced by prestressing

$\sigma cp(lpt2) = 4.646$ MPa after full transfer

$$kv := \min\left(1 + \sqrt{\frac{200}{d}}, 2\right) = 1.393$$

$k1v := 0.15$

$$Crdc := \frac{0.18}{\gamma cpcred} = 0.129$$

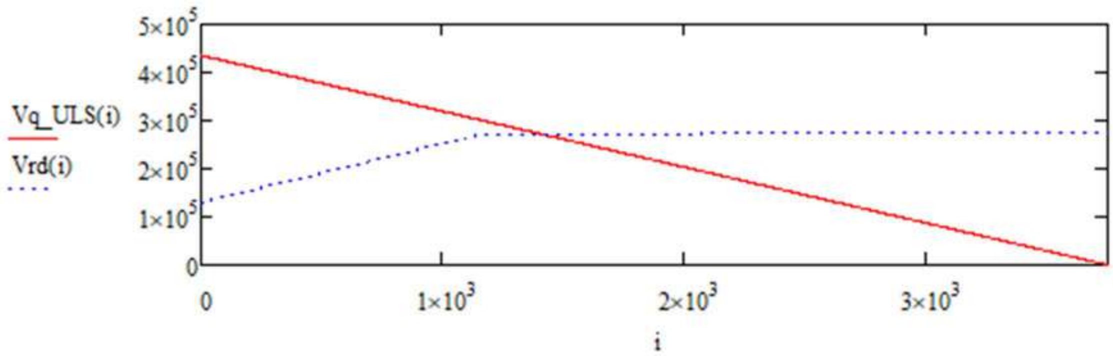
$$vmin := 0.035\cdot k^{\frac{3}{2}}\cdot(-fck)^{\frac{1}{2}} = 0.61$$ §6.3N

$$bw\cdot d\cdot\left[\left\lfloor Crdc\cdot kv\cdot(100\cdot\rho l\cdot -fck)^{\frac{1}{3}} + k1v\cdot\sigma cp(lpt2)\right\rfloor\right] = $$

$$(vmin + k1v\cdot\sigma cp(lpt2))\cdot bw\cdot d = 2.659 \times 10^5$$

$$VRdc(x) := \max\left[\left[\left\lfloor Crdc\cdot kv\cdot(100\cdot\rho l\cdot -fck)^{\frac{1}{3}} + k1v\cdot\sigma cp(x)\right\rfloor\cdot bw\cdot d, (vmin + k1v\cdot\sigma cp(x))\cdot bw\cdot d\right]\right]$$ §6.2.a+§6.2.b

AFTER PRESTRESSING IS



MEMBERS PROVIDED WITH SHEAR REINFORCEMENT (§6.2.3)

$fywd := fsd = 454.545$ MPa — design yield stress of transverse reinforcement

$Asw := 2\cdot\frac{8^2\cdot\pi}{4} = 100.531$ — area of transverse reinforcement (pseudo-vertical stirrups)

$\theta v := atan\left(\frac{1}{2.5}\right) = 0.381$ — angle of inclination of concrete compressed strut

$s1 := 90$ mm — spacing of transverse reinforcement (end field)

$Vrds := \frac{Asw}{s1}\cdot z\cdot fywd\cdot cot(\theta v) = 5.81\times10^5$ N > $Vq\_ULS(0) = 4.354\times10^5$ CHECK — shear resistance on steel side (§6.8)

$\nu 1 := 0.6\cdot\left(1-\frac{fck}{250}\right) = 0.708$ §6.10

$\alpha cw(x) := if\left[\sigma cp(x) < 0.25\cdot -fcd, 1+\frac{\sigma cp(x)}{-fcd}, if\left[\sigma cp(x) > 0.5\cdot -fcd, 2.5\cdot\left(1-\frac{\sigma cp(x)}{-fcd}\right), 1.25\right]\right]$ §6.11

$Vrdmax(x) := \alpha cw(x)\cdot bw\cdot z\cdot \nu 1\cdot\frac{-fcd}{cot(\theta v)+tan(\theta v)}$ — shear resistance on concrete side (§6.9)

$Vrdmax(0) = 1.642\times10^6$ N > $Vq\_ULS(0) = 4.354\times10^5$ CHECK

MOMENT DIAGRAM ACCOUNTING DUE TO SHEAR RESISTING MECHANISM (§9.2.1.3)

$\eta p2 := 1.2$

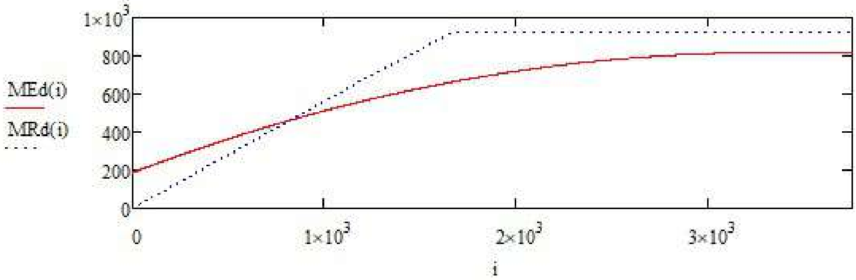
$fbpd := \eta p2\cdot\eta 1\cdot fctd = 2.277$ MPa — given fck < 60 MPa

$$lbpd := lpt2 + \frac{\alpha 2\cdot\left(fptd - \sigma pm\left(\frac{L}{2}, t\right)\right)}{fbpd}\cdot\phi p = 1.67\times10^3$$

$$MRd(x) := if\left[x < lpt2, Mrd\cdot\frac{\sigma pm(lpt2, 365\cdot 50)}{fptd}\cdot\frac{x}{lpt2}, if\left[x > lbpd, Mrd, Mrd\cdot\frac{\sigma pm(lpt2, 365\cdot 50)}{fptd} + \frac{(x-lpt2)}{(lbpd-lpt2)}\cdot\left(Mrd - Mrd\cdot\frac{\sigma pm(lpt2, 365\cdot 50)}{fptd}\right)\right]\right]$$

$$al := z\cdot\left(\frac{cot(\theta v)}{2}\right) = 572.143$$

$$MEd(x) := if\left(x > \frac{L}{2} - al, \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6}, \frac{Mq\_ULS(x + round(z))}{10^6}\right)$$



MINIMUM REINFORCEMENT

$bt := b(Htot) = 800$

§9.1N §9.2.1.1(3)

$Asmin := max\left(0.26 \cdot \frac{fctm}{fsk} \cdot bt \cdot d, 0.0013 \cdot bt \cdot d\right) = 802.986$ mm2 > $\rho l \cdot Ac = 3.356 \times 10^{3}$ mm2 CHECK < $0.04 \cdot Ac = 1.328 \times 10^{4}$ CHECK

$s2 := 120$ mm < $d \cdot 0.75 = 381.429$ mm CHECK for longitudinal reinforcement

$s3 := 180$ mm for shear reinforcement §9.6N

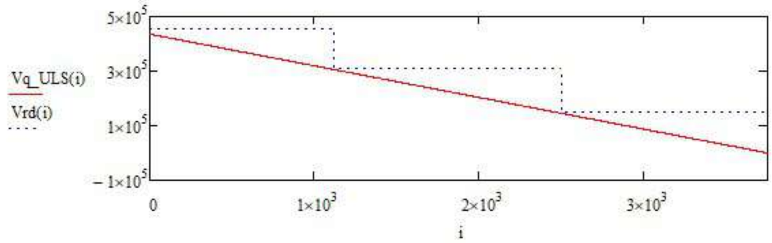
$\rho w\_min := \frac{Asw}{s3 \cdot bw} = 1.396 \times 10^{-3}$ > $0.08 \cdot \frac{\sqrt{fck}}{fsk} = 1.073 \times 10^{-3}$ CHECK

CHECK OF STIRRUPS FOR SUSPENSION LOAD

$\frac{VEd}{cot(\theta v) \cdot bw} + \left(\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5\right) = 485.02$ kN / m < $\frac{Asw \cdot fywd}{s1} = 507.732$ kN / m CHECK BEFORE TRANSFER OF PRESTRESSING

$\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5 = 108.644$ kN / m < $\frac{Asw \cdot fywd}{s2} = 380.799$ kN / m CHECK AT MAXIMUM SPACING

$Vrd(x) := if\left(x < 1120, Vrds, if\left(x < 2500, \frac{s1}{s2} \cdot Vrds, Vrd(x)\right)\right) - \left(\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5\right) \cdot cot(\theta v) \cdot z$ INCLUDING THE EFFECT OF SUSPENSION LOAD



CHECK OF HORIZONTAL SADDLE BAR

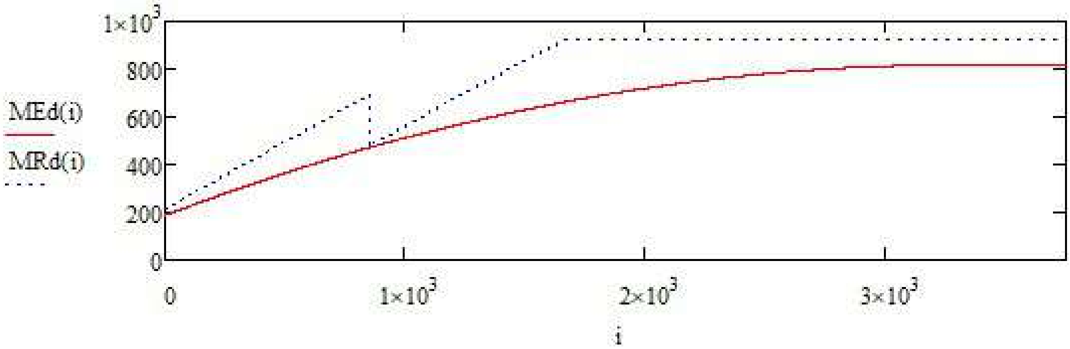
$$\frac{\left(\frac{g1}{3}\cdot 1.35 + g2\cdot 1.35 + q\cdot 1.5\right)}{2}\cdot 100\cdot s2 = 6.519\times 10^{5} \quad \text{Nmm} \quad < \quad \pi\cdot\frac{8^2}{4}\cdot fsd\cdot 0.9\cdot 220 = 4.524\times 10^{6}$$ CHECK

CHECK OF SUPPORT MILD REBARS (§9.2.1.4(1))

$$0.25\cdot Mrd = 231.753 \quad \text{Nmm} \quad < \quad \left(4\cdot\pi\cdot\frac{16^2}{4} + 2\cdot\pi\cdot\frac{12^2}{4}\right)\cdot fsd\cdot 0.9\cdot\frac{550}{10^6} = 231.85$$ CHECK

area of support mild steel

$$MRd(x) := \text{if}\left[x < 850, 0.9\cdot d\cdot\frac{fywd}{10^6}\cdot\left(4\cdot\pi\cdot\frac{16^2}{4} + 2\cdot\pi\cdot\frac{12^2}{4}\right) + MRd(x), MRd(x)\right]$$



ANCHORAGE (§8.4)

$\eta 1 = 1$

$\eta 2 := 1$

$fbd := 2.25\cdot\eta 1\cdot\eta 2\cdot fctd = 4.27$ MPa §8.2

$lbrqd(\phi) := \frac{\phi}{4}\cdot\frac{fsd}{fbd}$ §8.3

$\alpha 1b := 1$

$\alpha 2b := 1$

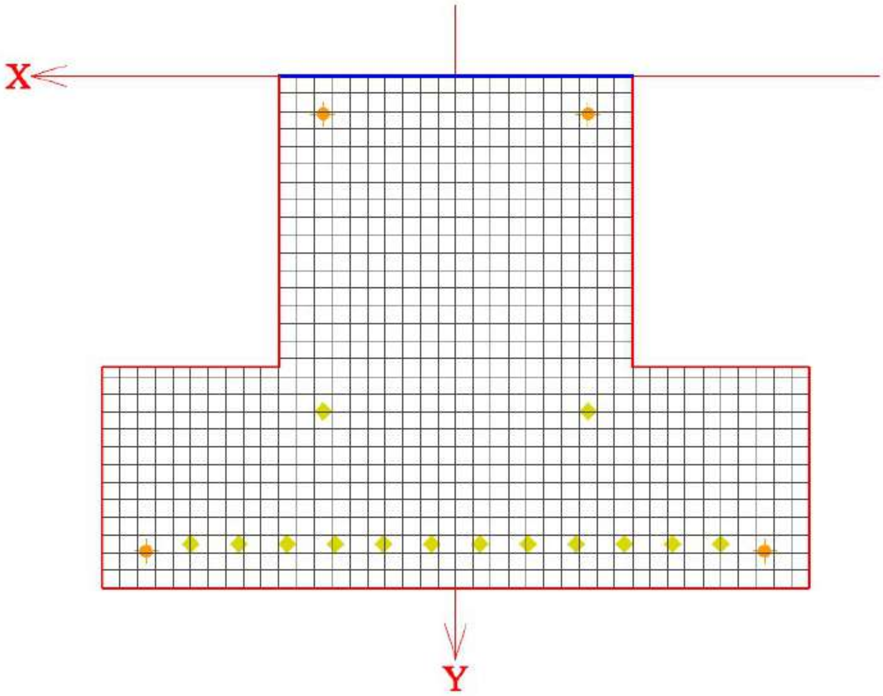
$\alpha 3b := 1$

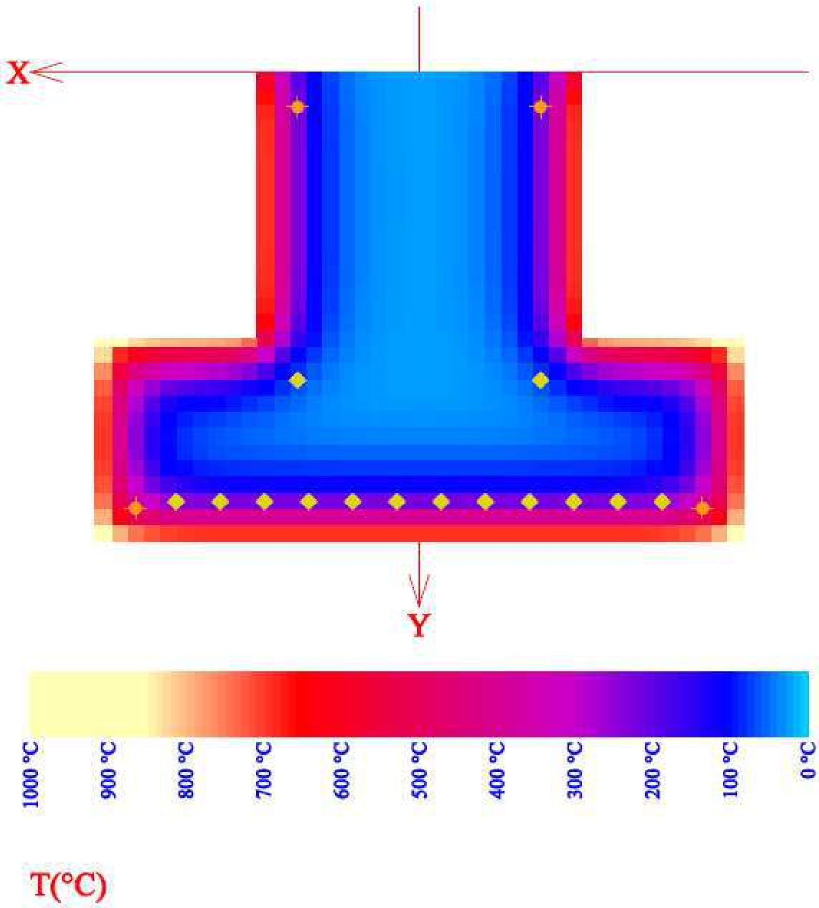
$\alpha 4b := 1$

$\alpha 5b := 1$

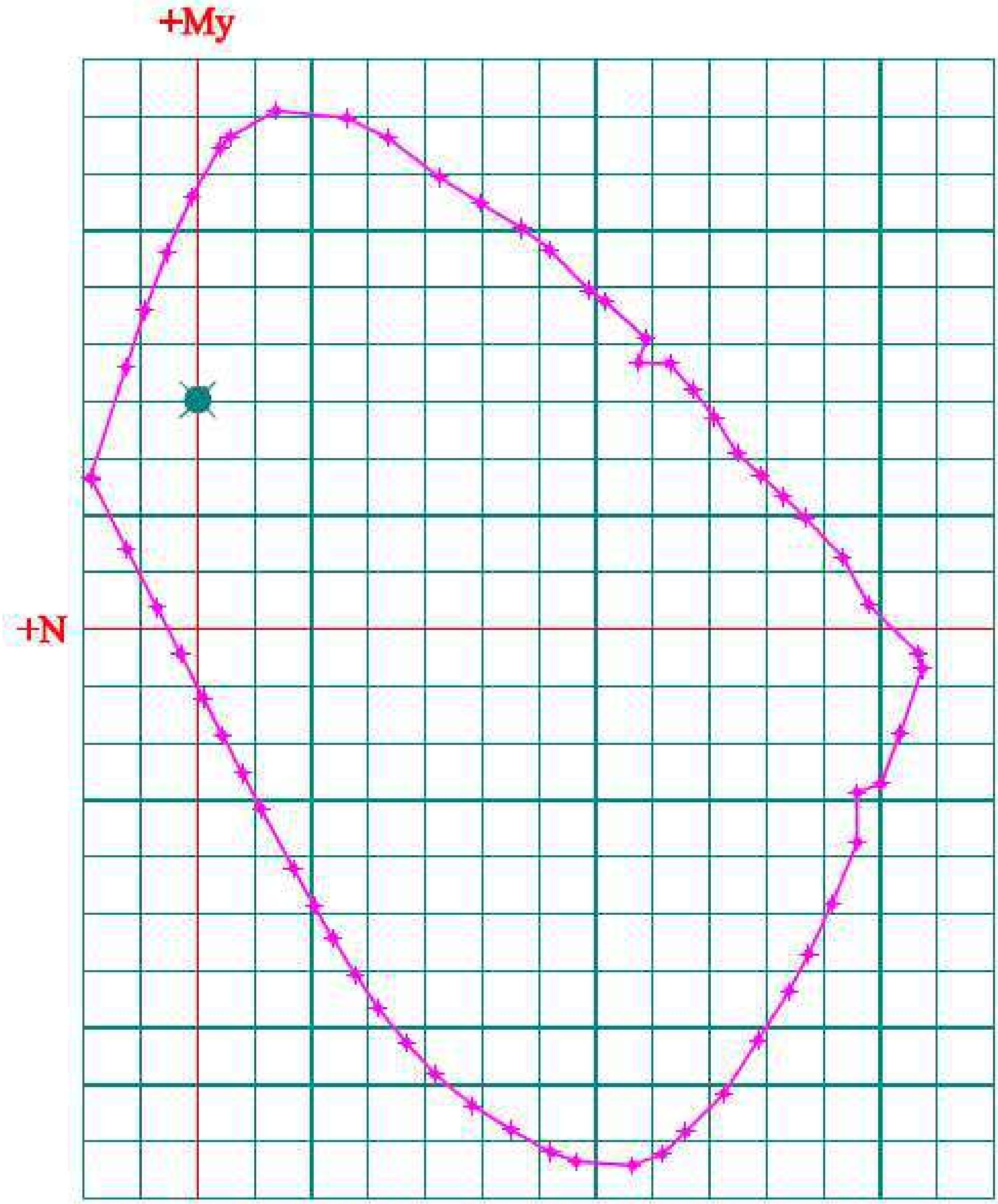
$lbd(\phi) := \alpha 1b\cdot\alpha 2b\cdot\alpha 3b\cdot\alpha 4b\cdot\alpha 5b\cdot lbrqd(\phi)$ §8.4

11.11 Fire checks





27	35	49	73	105	220	391	686										
27	35	49	73	106	220	392	687										
27	35	50	73	107	220	392	687										
27	35	50	73	107	221	392	687										
27	35	50	73	107	221	392	687										
27	35	50	73	107	220	392	687										
27	35	49	73	106	220	392	687										
27	35	49	73	105	220	391	686										
27	35	49	72	102	218	389	685										
27	34	48	71	100	214	383	681										
26	33	46	69	100	205	369	669										
26	32	44	65	100	187	338	632										
25	30	40	58	92	154	275	508	730	771	782	787	790	795	805	822	853	902
24	28	35	49	73	100	186	300	422	476	498	508	517	529	554	600	681	823
23	26	31	41	57	80	104	174	227	261	280	291	303	322	358	425	546	759
23	25	29	35	45	59	78	100	119	144	159	169	181	202	247	328	471	723
24	26	28	31	37	46	56	68	78	89	98	100	100	130	183	274	430	704
28	29	30	32	35	40	45	52	57	63	69	76	86	100	153	247	410	695
35	36	36	38	39	42	44	48	51	55	59	66	77	97	140	237	403	692
50	50	50	51	52	53	54	56	58	60	63	69	80	100	145	241	406	693
73	73	73	74	74	75	75	76	77	78	81	87	97	100	166	259	419	699
107	107	107	107	107	108	108	109	110	112	116	124	140	165	213	299	448	713
221	221	221	221	221	221	221	221	221	222	223	227	237	258	298	373	505	740
392	392	392	392	392	392	392	392	392	393	394	397	403	418	448	505	608	788
687	687	687	687	687	687	687	687	687	687	688	689	692	699	713	740	788	871

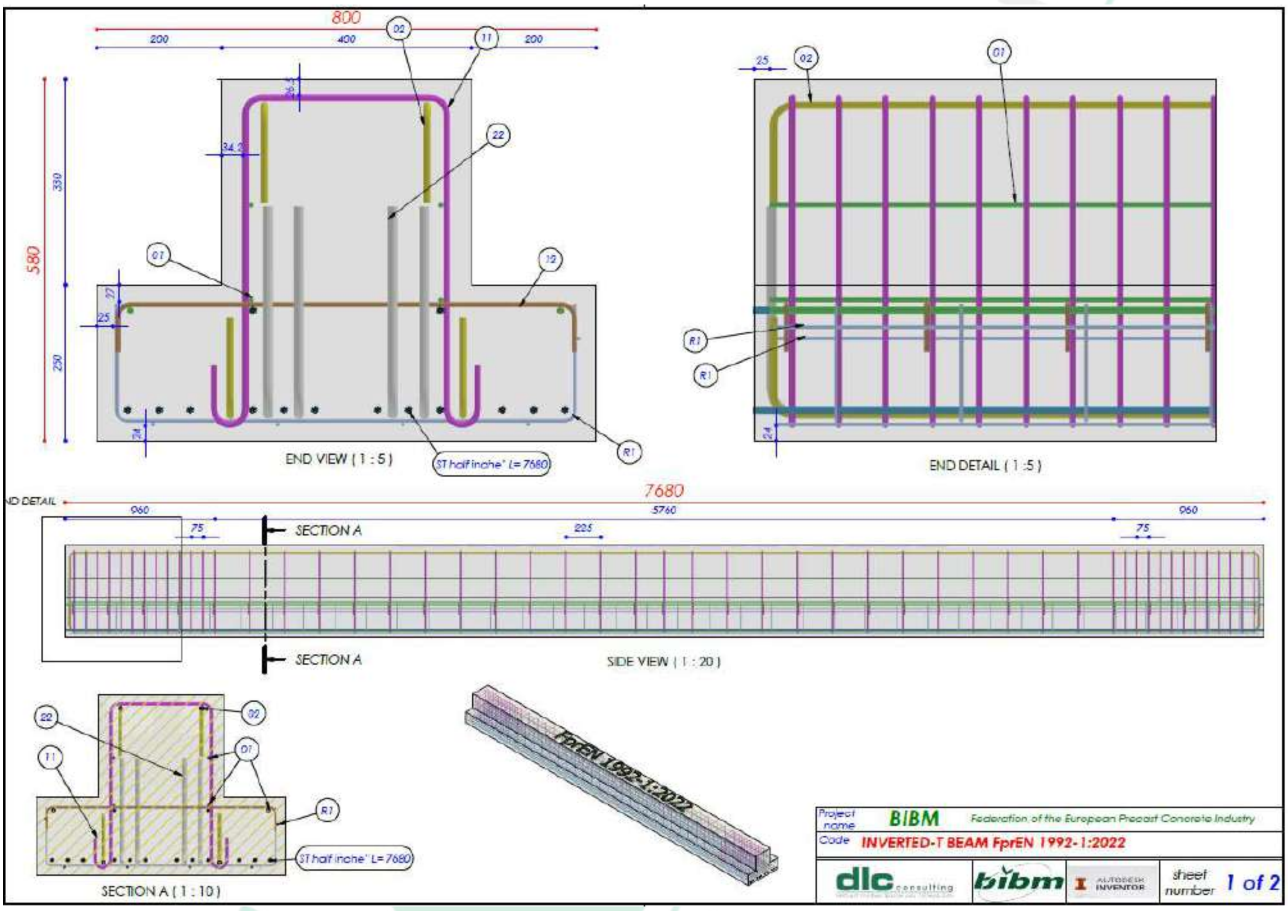


N: 1 sp = 1100.00 kN;   M: 1 sp = 110.00 kN·m

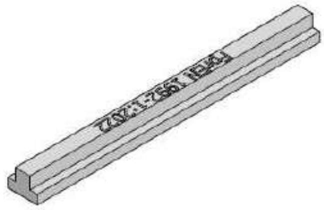
Nz = 0.00 kN
My+ = 855.34 kN·m
My- = -112.43 kN·m

12 Prestressed beam element – FprEN1992-1:2022

12.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_	Ø_longitudinal	pattern_T	Ø_transverse	pattern_L
	01	4	3011	12044	8 mm				
	02	4	7018	28072	12 mm				
	03	2	6774	13548	12 mm				
	11	52	982	51064	10 mm				
	12	35	341	11935	8 mm				
	22	8	2378	19024	16 mm				
Total mass rebars [kg]				135,69		Incidence kg/m³	53,21		
	RI	1	19373	19373		6 mm	200 mm	6 mm	200 mm
Total mass welded-wire-meshes [kg]				19,37		Incidence kg/m³	7,60		
	ST half inche" L= 7680	14	5612.5	78575	12.7 mm				
Total mass strands [kg]				78,575		Incidence kg/m³	30,81		
	Total mass of steel [kg]			233,64		Total concrete volume [r	2,55		



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Code	INVERTED-T BEAM FprEN 1992-1:2022	
dlc consulting	bibm	AUTODESK INVENTOR · sheet number 2 of 2

12.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$$y\_tr := (0 \;\; 329.99 \;\; 330 \;\; 580)^T$$

$$Htot := \max(y\_tr)$$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$$b\_tr := (400 \;\; 400 \;\; 800 \;\; 800)^T$$

$r\_circ := 0$ radius of central void pipe

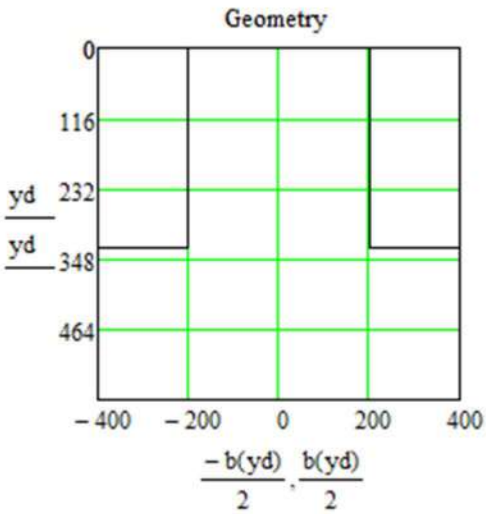
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$$b\_lin(y) := linterp(y\_tr, b\_tr, y)$$

$$b\_circ(y) := linterp(y\_tr, b\_tr, y) - x\_circ(y)$$

$$b(y) := if\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$$yd := 0..Htot$$



condensed 1D geometry plot

$u := 800 \cdot 2 + Htot \cdot 2 = 2.76 \times 10^3$ exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (43 \;\; 202 \;\; 354 \;\; 370 \;\; 538)^T$$

Area of reinforcement at each depth

$$As := (2 \cdot A(12) \;\; 2 \cdot A(8) \;\; 2 \cdot A(8) \;\; 2 \cdot A(8) \;\; 2 \cdot A(12))^T$$

$$js := rows(As) \qquad js = 5$$

$$dsmax := max(ds) \qquad dsmax = 538$$

$$As\_tot := \sum_{j=1}^{js} As_j = 753.982$$

Prestressing reinforcement

Area of a single strand:

$Ap0 := 93$ $\qquad \phi p := 12.7$ mm nominal strand diameter

Depth of prestressing strands from upper chord:

$dp := (380 \;\; 480 \;\; 530)^T$

Area of strands at each depth:

$Ap := (2 \cdot Ap0 \;\; 0 \cdot Ap0 \;\; 12 \cdot Ap0)^T$

$\sigma p0 := 1400$ MPa

$\sigma prec := (0.4 \cdot \sigma p0 \;\; \sigma p0 \;\; \sigma p0)^T$ initial prestressing

$perdite := 0 \cdot (1 \;\; 1 \;\; 1)^T$ in percentual **% (losses are introduced later)**

$jp := rows(Ap)$ $\qquad jp = 3$

$k := 1..jp$

$$\sigma o_k := \sigma prec_k \cdot \left[\frac{(100 - perdite_k)}{100}\right] \qquad \sigma o = \begin{pmatrix} 560 \\ 1.4 \times 10^3 \\ 1.4 \times 10^3 \end{pmatrix}$$

$$Ap\_tot := \sum_{k=1}^{jp} Ap_k \qquad Ap\_tot = 1.302 \times 10^3$$

$ypmax := max(dp)$ $\qquad ypmax = 530$

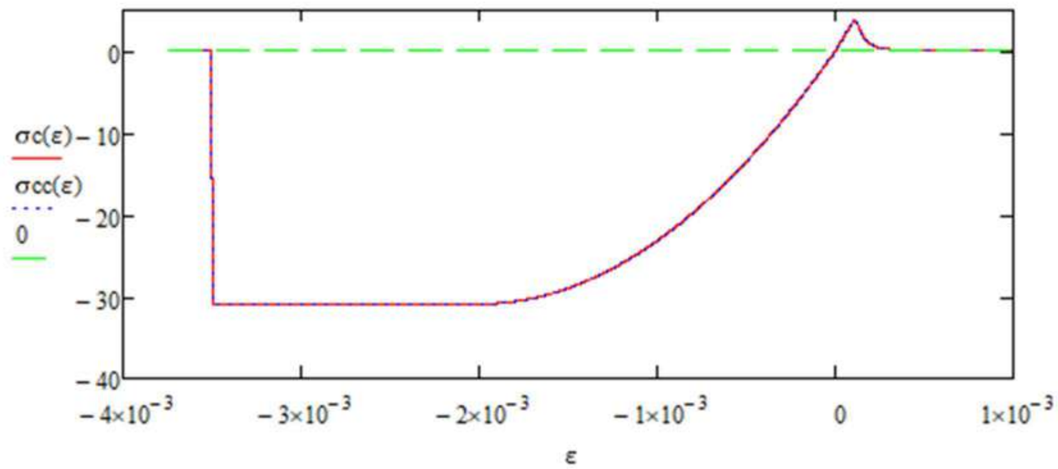
$$Np\_tot := \sum_{k=1}^{jp} \left(\left(Ap_k \cdot \sigma o_k \right) \right) \qquad Np\_tot = 1.667 \times 10^6 \text{ N}$$

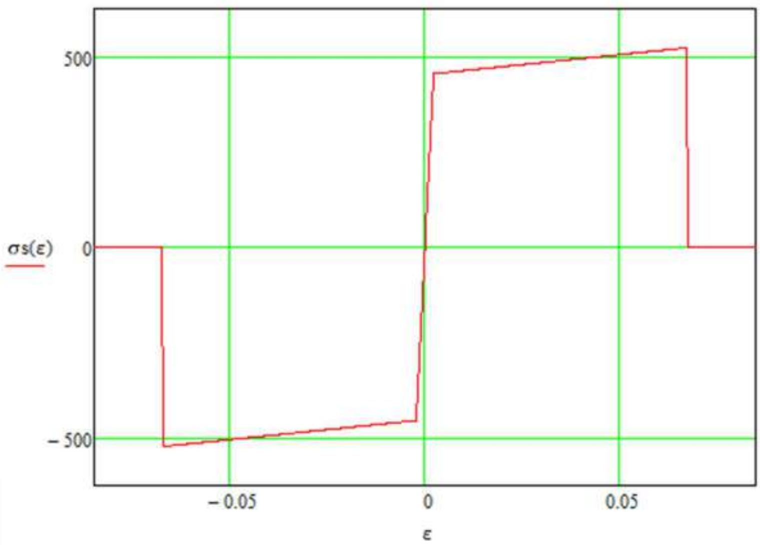
total prestressing initial force

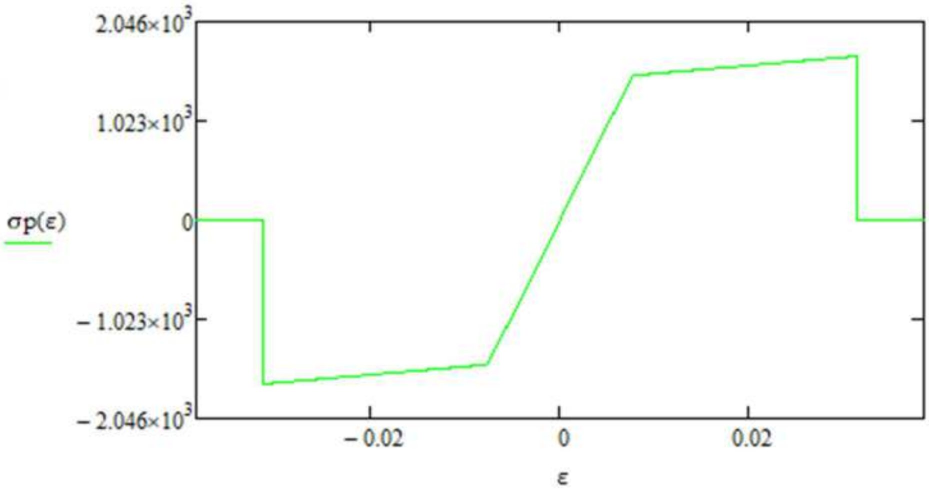
$$Yp := \frac{\sum_{k=1}^{jp} \left(dp_k \cdot Ap_k \cdot \sigma o_k \right)}{\sum_{k=1}^{jp} \left(Ap_k \cdot \sigma o_k \right)} = 520.625 \text{ mm}$$

centre of gravity of prestressing

12.3 Material constitutive laws employed in the calculation







12.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 3.321 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 2.27 \times 10^{-3}$$ geometric ratio for longitudinal mild reinforcement

$$\rho p := \frac{Ap\_tot}{Ac} = 3.921 \times 10^{-3}$$ geometric ratio for longitudinal prestressing tendons

$$\rho tot := \frac{As\_tot + Ap\_tot}{Ac} = 6.191 \times 10^{-3}$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y) \cdot y\,dy \qquad Syc = 1.128 \times 10^8$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 339.696$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y) \cdot (y - yG)^2\,dy \qquad Ixo\_cls = 8.927 \times 10^9$$

Global area of all prestressing reinforcement

$$Area\_tr := \left| \begin{array}{l} s \leftarrow 0 \\ \text{for } x \in 1..jp \\ \quad s \leftarrow Ap_x + s \end{array} \right. \qquad Area\_tr = 1.302 \times 10^3$$

First moment of the area referred to prestressing reinforcement only

$$Sxp := \sum_{i=1}^{jp} \left(Ap_i \cdot dp_i\right) \qquad Sxp = 6.622 \times 10^5$$

Centre of gravity of prestressing

$$Yp := \frac{Sxp}{Area\_tr} \qquad Yp = 508.571$$

Idealisation coefficients (elastic)

$$np := \frac{Ep}{Ecm} \qquad np = 5.465$$

$$ns := \frac{Es}{Ecm} \qquad ns = 5.605$$

Area of ideal cross-section

$$Aid := Ac + (np - 1)\cdot\sum_{j=1}^{jp} Ap_j + (ns - 1)\cdot\sum_{j=1}^{js} As_j \qquad Aid = 3.414\times10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac\cdot yG + (np - 1)\cdot(Area\_tr\cdot Yp) + (ns - 1)\cdot\sum_{j=1}^{js}\left(As_j\cdot ds_j\right) \qquad Sxid = 1.168\times10^8$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 342.145$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y)\cdot(y - Yid)^2\,dy - \sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right] - \sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the prestressing reinforcement area

$$Ixoidprec := np\cdot\sum_{i=1}^{jp}\left[Ap_i\cdot\left(dp_i - Yid\right)^2\right]$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns\cdot\sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidprec + Ixoidlenta \qquad Ixo\_id = 9.249\times10^9 \text{ mm}^4 \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.036$$

12.5 Loads

LOADS

$g1 := 8.3$ kN/m — dead load from self-weight

$g2 := (2 + 2.89) \cdot 9.45 = 46.211$ kN/m — nonstructural dead load

$q := 28.35$ kN/m — live load

$L := 7500$ mm — calculation length (span between supports)

$\psi 2 := 0.3$ — non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ — non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ — SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ — SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ — SLS bending moment distribution from live load

12.6 Prestressing transfer and time-dependent behaviour

TRANSFER OF PRESTRESS (§13.5.3)

$\alpha 1 := 1$ — gradual release of prestressing

$\alpha 2 := 0.26$ — for 7-wire strands

$\sigma pm0 := \sigma p0 = 1.4 \times 10^3$ MPa

$\eta 1 := 1$ — in favourable position

$lpt := \frac{\gamma c}{1.5} \cdot \frac{\alpha 1 \cdot \alpha 2 \cdot \sigma pm0}{\eta 1 \cdot \sqrt{(-fcmj(2) - 8)}} \cdot \phi p = 906.996$ mm — basic value of the transmission length following §(13.4)

$lpt1 := 0.8 lpt = 725.597$ mm — lower-bound transfer length following §(13.6)

$lpt2 := 1.2 \cdot lpt = 1.088 \times 10^3$ mm — upper-bound transfer length following §(13.7)

LOADS

$g1 := 8.3$ kN/m dead load from self-weight

$g2 := (2 + 2.89) \cdot 9.45 = 46.211$ kN/m nonstructural dead load

$q := 28.35$ kN/m live load

$L := 7500$ mm calculation length (span between supports)

$\psi 2 := 0.3$ non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from live load

Prestress losses

$hn := 2 \cdot \frac{Ac}{u} = 240.643$ mm

$A := 0.79 + \frac{(hn - 200)}{(500 - 200)} \cdot (0.75 - 0.79) = 0.785$

$\varepsilon cs := \frac{0.65}{1000} = 6.5 \times 10^{-4}$ shrinkage strain assumed as a result of laboratory tests on the specific concrete mix employed

$\rho 1000 := 0.025$ for class 2 (low-relaxation) tendons

$k\rho := 0.16$

$t := 50 \cdot 365 = 1.825 \times 10^{4}$ days Life span

$\sigma cpQP2(x) := \frac{-Np\_tot}{Aid} + \frac{[Mq\_SLSg1(x) - Np\_tot \cdot (Yp - Yid)] \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP2\left(\frac{L}{2}\right) = -8.823$

stress in quasi-permanent load combination at 2 days
(conventional equivalent time for prestressing release)

$\sigma cpQP23(x) := \frac{Mq\_SLSg2(x) \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP23\left(\frac{L}{2}\right) = 5.847$

stress in quasi-permanent load combination at 23 days
(conventional time for assemblage of the structure on site)

$\sigma cpQP91(x) := \frac{Mq\_SLSq(x) \cdot (Yp - Yid)}{Ixo\_id}$ $\sigma cpQP91\left(\frac{L}{2}\right) = 1.076$

stress in quasi-permanent load combination at 91 days
(conventional time for enter in use of the structure)

$$\Delta\sigma pr(x,t) := \left[\sigma p0 + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x))\right] \cdot \rho 1000 \cdot \left(\frac{24 \cdot t}{1000}\right)^{k\rho}$$

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$RH := 50$

$t0\_adj(t0) := t0$

$\beta bc\_fcm := \frac{1.8}{(-fcm)^{0.7}} = 0.112 \qquad \beta bc\_t\_t0(t,t0) := \ln\left[\left(\frac{30}{t0\_adj(t0)} + 0.035\right)^2 \cdot (t - t0) + 1\right]$

$\beta dc\_fcm := \frac{412}{(-fcm)^{1.4}} = 1.588$

$\beta dc\_RH := \frac{1 - \frac{RH}{100}}{\sqrt[3]{0.1\cdot\frac{hn}{100}}} = 0.804$

$\beta dc\_t0(t0) := \frac{1}{0.1 + t0\_adj(t0)^{0.2}}$

$\gamma(t0) := \frac{1}{2.3 + \frac{3.5}{\sqrt{t0\_adj(t0)}}}$

$\alpha cm := \left(\frac{35}{-fcm}\right)^{0.5} = 0.813$

$\beta h := \min(1.5\cdot hn + 250\cdot\alpha cm, 1500\cdot\alpha cm) = 564.124$

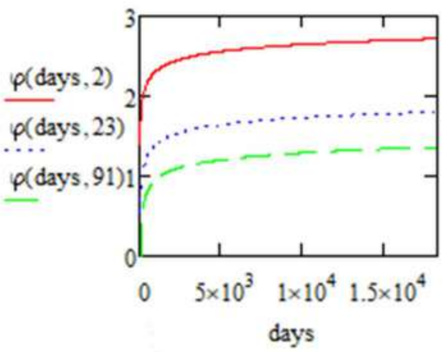
$\beta dc\_t\_t0(t,t0) := \left[\frac{(t - t0)}{\beta h + (t - t0)}\right]^{\gamma(t0)}$

$\varphi dc(t,t0) := \beta dc\_fcm\cdot\beta dc\_RH\cdot\beta dc\_t0(t0)\cdot\beta dc\_t\_t0(t,t0)$

$\varphi bc(t,t0) := \beta bc\_fcm\cdot\beta bc\_t\_t0(t,t0)$

$\varphi(t,t0) := \varphi bc(t,t0) + \varphi dc(t,t0)$

$\varphi(t,2) = 2.718 \qquad \varphi(t,91) = 1.363$



TIME-DEPENDENT LOSSES OF PRESTRESS (§7.6.4)

$$\Delta\sigma p\_csr(x,t) := \frac{-\varepsilon cs \cdot Ep - 0.8 \cdot \Delta\sigma pr(x,t) + \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) \cdot \varphi(t,2) + \sigma cpQP23(x) \cdot \varphi(t,23) + \sigma cpQP91(x) \cdot \varphi(t,91))}{1 + \frac{Ep}{Ecm} \cdot \frac{Ap\_tot}{Ac} \cdot \left[1 + \frac{Ac}{Ixoidcls} \cdot (Yp - Yid)^2\right] \cdot \left(1 + 0.8 \cdot \frac{\varphi(t,2) \cdot \sigma cpQP2(x) + \varphi(t,23) \cdot \sigma cpQP23(x) + \varphi(t,91) \cdot \sigma cpQP91(x)}{\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)}\right)}$$

prestress losses following §(7.35)

NOTE: a weighed creep coefficient was considered accounting for the 3 load phases previously introduced

$$\sigma pm(x,t) := \sigma p0 - \frac{Ep}{Ecm} \cdot (\sigma cpQP2(x) + \sigma cpQP23(x) + \sigma cpQP91(x)) + \Delta\sigma p\_csr(x,t)$$ prestress considering immediate and delayed losses

$$\frac{\sigma pm\left(\frac{L}{2},t\right)}{\sigma p0} = 0.857$$ expected residual prestress ratio after 50 years of life with respect to initial

$$\varepsilon pm := \frac{\sigma pm\left(\frac{L}{2},t\right)}{\sigma p0} \cdot \varepsilon p0$$ expected residual strain after 50 years of life with respect to initial

$$\sigma pm\left(\frac{L}{2},t\right) \cdot Ap\_tot = 1.563 \times 10^6 \quad N$$ residual prestress force after 50 years of life

$$Np\_tot = 1.667 \times 10^6 \quad N$$ initial prestress force

12.7 Non-linear moment-curvature diagram

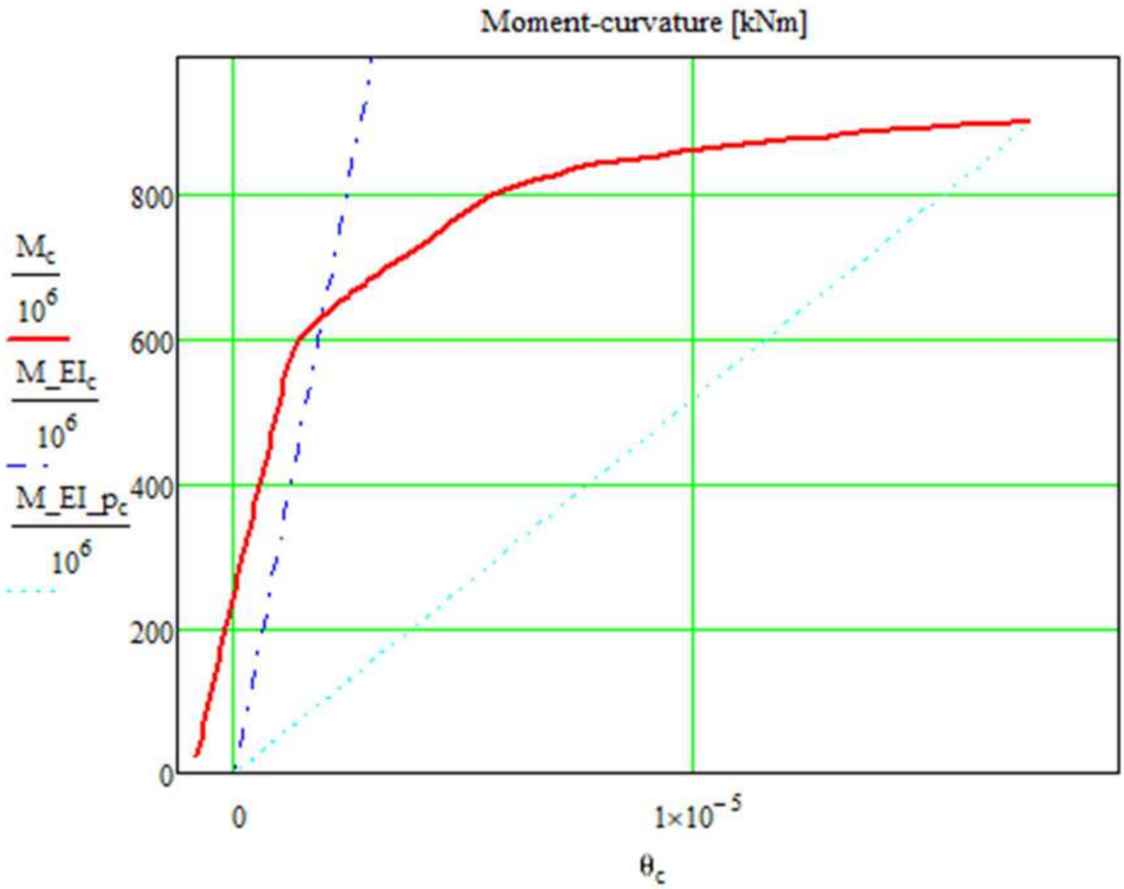
Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

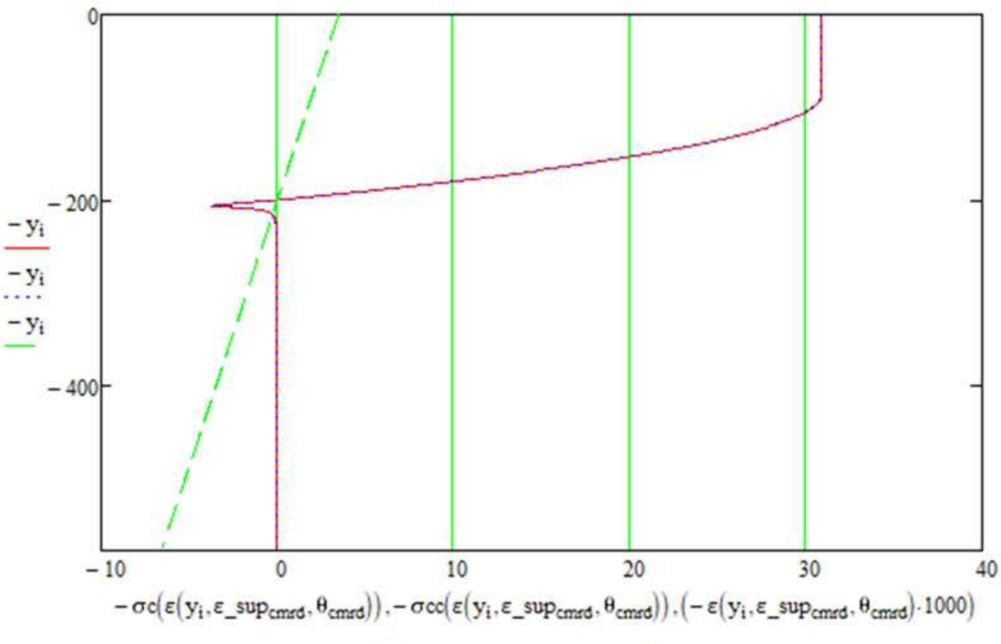
$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot} \left(\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right) \cdot b\left(y_i\right) \cdot \Delta y\right) + \sum_{j=1}^{jp} \left(\sigma p\left(\varepsilon\left(dp_j,\varepsilon\_sup,\theta\right) + \varepsilon pm_j\right) \cdot Ap_j\right) + \sum_{j=1}^{js} \left(\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right) \cdot As_j\right)$$

$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot} \left[\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right) \cdot b\left(y_i\right) \cdot \Delta y \cdot \left(y_i - yG\right)\right] + \sum_{j=1}^{jp} \left[\sigma p\left(\varepsilon\left(dp_j,\varepsilon\_sup,\theta\right) + \varepsilon pm_j\right) \cdot Ap_j \cdot \left(dp_j - yG\right)\right] + \sum_{j=1}^{js} \left[\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right) \cdot As_j \cdot \left(ds_j - yG\right)\right]$$

Design external axial load

$NS := -0$





Condition at resisting (peak) moment (stress and strain)

12.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2}\cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformally distributed load q

$Mq\_SLSr(x) := (g1 + g2 + q)\cdot\left(\frac{L}{2}\cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformally distributed load q

$Mq\_SLSf(x) := (g1 + g2 + \psi 1\cdot q)\cdot\left(\frac{L}{2}\cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformally distributed load q

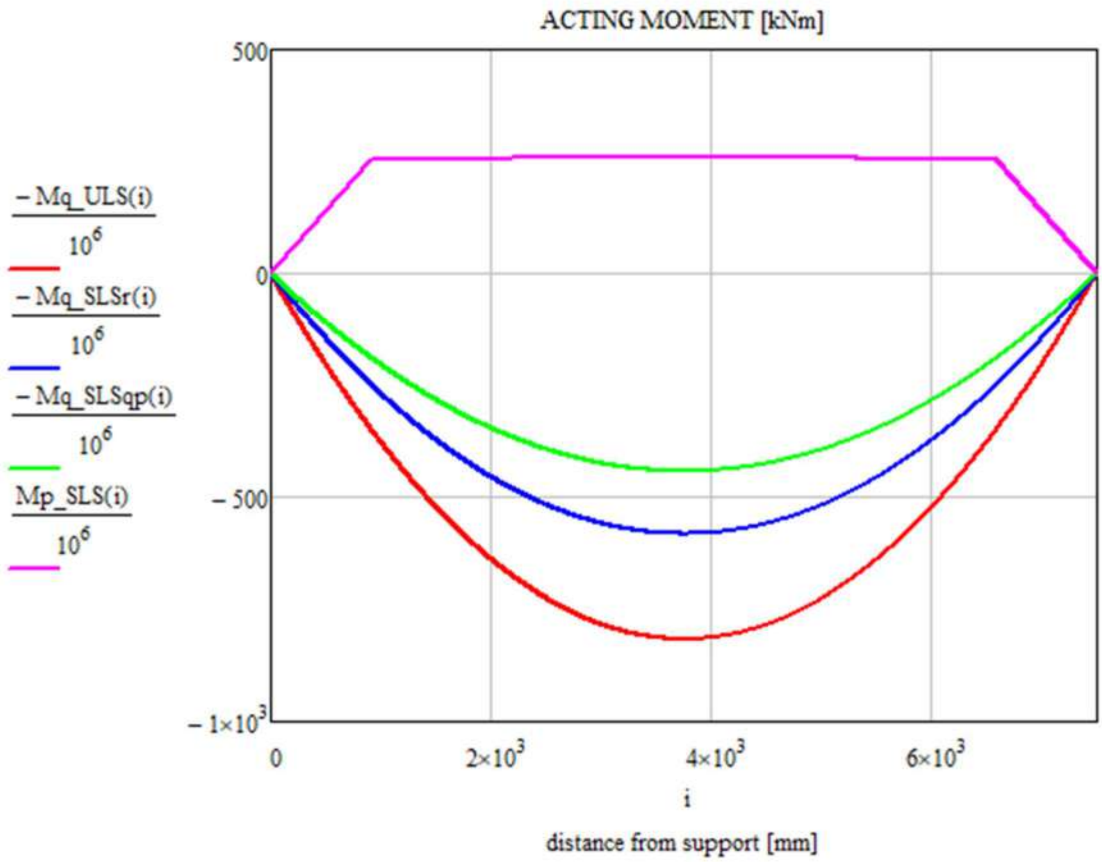
$Mq\_SLSqp(x) := (g1 + g2 + \psi 2\cdot q)\cdot\left(\frac{L}{2}\cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformally distributed load q

$Mq\_SLSg2(x) := (g1 + g2)\cdot\left(\frac{L}{2}\cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformally distributed load q

$Mp\_SLS(x) := \text{if}\left[x < lpt, \sigma pm(x,t)\cdot Ap\_tot\cdot(Yp - Yid)\cdot\frac{x}{lpt}, \text{if}\left[x > L - lpt, \sigma pm(x,t)\cdot Ap\_tot\cdot(Yp - Yid)\cdot\frac{-x + L}{lpt}, \sigma pm(x,t)\cdot Ap\_tot\cdot(Yp - Yid)\right]\right]$

contribution of prestressing equivalent load in SLS (without modification factors)

$i := 0..L$



12.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$$v\_inf\_p(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(1 + \varphi(t,23))}{1.05}$$

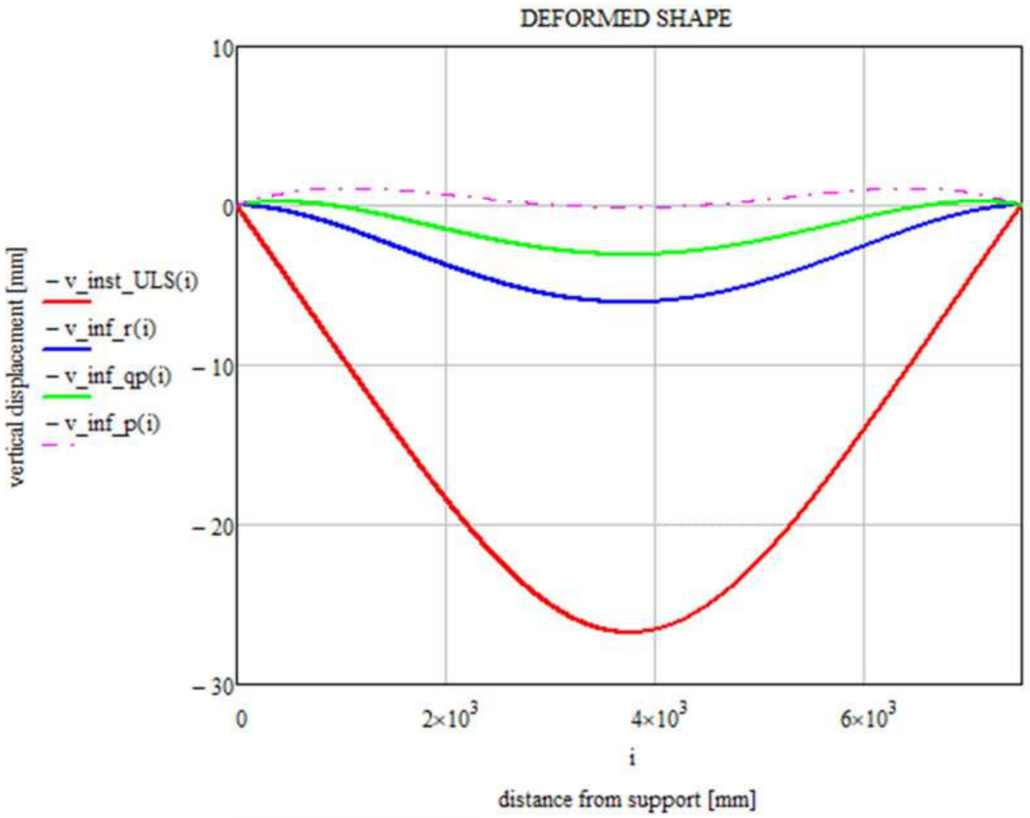
deflection profile at 50 years including creep for permanent load combination

$$v\_inf\_qp(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(\varphi(t,23) - \varphi(t,91)) + v\_SLSqp(x)\cdot(1 + \varphi(t,91))}{1.05}$$

deflection profile at 50 years including creep for quasi permanent load combination

$$v\_inf\_r(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(\varphi(t,23) - \varphi(t,91)) + v\_SLSqp(x)\cdot\varphi(t,91) + v\_SLSr(x)}{1.05}$$

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§9.3.4)

$v\_inf\_r\left(\frac{L}{2}\right) = 6.082$ < $\frac{L}{250} = 30$ CHECK maximum deflection

values calculated from differential equations above

$v\_inf\_p\left(\frac{L}{2}\right) = 0.123$ > $\frac{-L}{250} = -30$ CHECK maximum camber

SLS STRESS CONTROL (§9.2.1)

$k1 := 0.6$ $rsup := 1.05$

$k2 := 0.45$ prestressing modification coefficients

$k3 := 0.8$ $rinf := 0.95$ $Np\_tot = 1.667 \times 10^6$

$k4 := 1$

$k5 := 0.8$ 0.75 in EN1992-1-1:2002

NOTE: the denomination of the allowable stress coefficients following k factors was kept similar to that of EN1992-1-1:2002

$$\sigma cpg1\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpg1\_bot(lpt1) =$$

elastic stress of bottom concrete chord for selfweight loads only

$$\sigma cpg1\_top(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id} \qquad \sigma cpg1\_top(lpt1) =$$

elastic stress of top concrete chord for selfweight loads only

$$\sigma cpg1\_tops(x) := \frac{Es}{Ecm} \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSg1(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (ds_1 - Yid)}{Ixo\_id}\right] \qquad \sigma cpg1\_tops(lpt1)$$

elastic stress of top series of mild steel for selfweight loads only

$$\sigma cpf\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSf(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpf\_bot\left(\frac{L}{2}\right) = -$$

elastic stress of bottom concrete chord for frequent load combination

$$\sigma cpr\_bot(x) := \frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma cpr\_bot\left(\frac{L}{2}\right) = 2.$$

elastic stress of bottom concrete chord for rare load combination

$$\sigma cpr\_top(x) := \frac{-Np\_tot \cdot rinf}{Aid} + \frac{[Mq\_SLSr(x) - rinf \cdot Np\_tot \cdot (Yp - Yid)] \cdot (-Yid)}{Ixo\_id} \qquad \sigma cpr\_top\left(\frac{L}{2}\right) = -$$

elastic stress of top concrete chord for rare load combination

$$\sigma cpr\_p(x) := \sigma pm(x,t) \cdot rsup + 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (dp_{jp} - Yid)}{Ixo\_id}\right] \qquad \sigma cpr\_p\left(\frac{L}{2}\right) = 1.2$$

creep stress of bottom prestressing steel for rare load combination

$$\sigma cpr\_s(x) := 15 \cdot \left[\frac{-Np\_tot \cdot rsup}{Aid} + \frac{[Mq\_SLSr(x) - rsup \cdot Np\_tot \cdot (Yp - Yid)] \cdot (ds_{js} - Yid)}{Ixo\_id}\right] \qquad \sigma cpr\_s\left(\frac{L}{2}\right) = 15.6$$

creep stress of bottom mild steel for rare load combination

EN1992-1-1:2002

$\sigma cpg1\_bot(lpt1) = -12.091$ > $k1 \cdot \beta cc(2)^{\frac{2}{3}} \cdot fck = -18.733$ CHECK

> $k2 \cdot fck = -20.25$

$\sigma cpg1\_top(lpt1) = 4.893$ < $fctmj(2) = 2.731$

$\sigma cpg1\_tops(lpt1) = 20.366$ < $k3 \cdot fsk = 400$

$\sigma cpf\_bot\left(\frac{L}{2}\right) = -0.196$ < $fctm = 3.795$ CHECK

$\sigma cpr\_bot\left(\frac{L}{2}\right) = 2.368$ < $fctm = 3.795$

$\sigma cpr\_top\left(\frac{L}{2}\right) = -16.443$ > $k1 \cdot fck = -27$ CHECK

> $0.4 \cdot fcm = -21.2$

$\sigma cpr\_p\left(\frac{L}{2}\right) = 1.272 \times 10^3$ < $k5 \cdot fptk = 1.488 \times 10^3$ CHECK

$\sigma cpr\_s\left(\frac{L}{2}\right) = 15.667$ < $k3 \cdot fsk = 400$ CHECK

SLS CRACK CONTROL (§9.2.3)

$c\_act := Htot - ds_{js} - 10 = 32$

$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1.5$

$wlim\_cal := 0.2 \cdot ksurf = 0.3$ mm

$w\_freq := 0$ < $wlim\_cal = 0.3$ CHECK

12.10 ULS checks

ULS BENDING-AXIAL CONTROL (§8.1)

$$Mrd = 898.613 \text{ kNm} \quad > \quad \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6} = 816.428 \qquad \text{CHECK}$$

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§8.2)

$Vq\_ULS(x) := \left|(g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} - x\right)\right|$ shear action distribution at Ultimate Limit State (ULS)

$d := Yp = 508.571$ mm effective depth of cross-section

$VEd := Vq\_ULS(d) = 3.764 \times 10^5$ N design shear action at control section at distance d from support

$\gamma v := 1.3$ safety factor for initial shear check

$bw := 400$ mm design web width

$z := 0.9 \cdot d = 457.714$ conventional lever arm of internal stress resultants

$\tau Ed := \frac{VEd}{bw \cdot z} = 2.056$ MPa equivalent mean acting shear stress on control cross-section

$Dlower := 16$ mm maximum aggregate diameter following assumed mix design

$ddg := \min\left[\text{if}\left[-fck > 60, 16 + Dlower \cdot \left(\frac{60}{-fck}\right)^2, 16 + Dlower\right], 40\right] = 32$ size parameter

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT (§8.2.2)

$$\tau Rdc\_min(x) := \frac{11}{\gamma v}\cdot\sqrt{\frac{-fck}{(fptd - \sigma pm(x,t))}\cdot\frac{ddg}{d}}$$ §(8.20)

$\tau Rdc\_min(d) = 0.618$ MPa — not checked with τEd -> detailed evaluation is mandatory following §8.2.1

$$\rho l(x) := \frac{Ap\_tot + \sum_{j=1}^{js} As_j}{bw\cdot d}$$ longitudinal geometric reinforcement ratio §(8.28)

$ep := Yp - Yid = 166.426$ mm — eccentricity of prestressing

$$acs\_0(x) := \max\left(\frac{Mq\_ULS(x)}{Vq\_ULS(x)}, d\right)$$ §(8.30) accounting for comments in §8.2.2(5)

$$k1(x) := \min\left[\frac{0.5}{acs\_0(x)}\cdot\left(ep + \frac{d}{3}\right)\cdot\frac{Ac}{bw\cdot z}, 0.18\cdot\frac{Ac}{bw\cdot z}\right]$$ §(8.34)

$$av\_0(x) := \sqrt{\frac{acs\_0(x)}{4}\cdot d}$$ §(8.29) accounting for comments in §8.2.2(5)

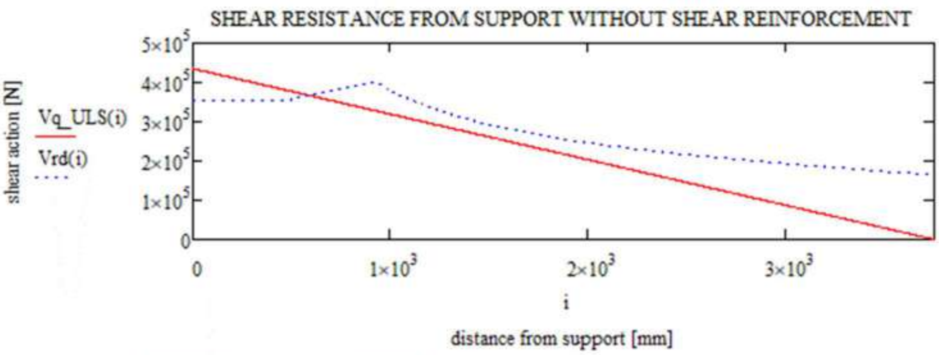
$$\tau Rdc\_0(x) := \frac{0.66}{\gamma v}\cdot\left(100\cdot\rho l(x)\cdot -fck\cdot\frac{ddg}{av\_0(d)}\right)^{\frac{1}{3}}$$ §(8.33)

$$\tau Rdcmax(x) := \min\left[2.15\cdot\tau Rdc\_0(x)\cdot\left(\frac{acs\_0(x)}{d}\right)^{\frac{1}{6}}, 2.7\cdot\tau Rdc\_0(x)\right]$$ §(8.35)

$$\sigma cp(x) := \sigma pm(x,t)\cdot\frac{Ap\_tot}{Ac}$$ §(8.33)

$$\tau Rdc(x) := \max(\min(\tau Rdc\_0(x) + k1(x)\cdot\sigma cp(x), \tau Rdcmax(x)), \tau Rdc\_min(x))$$ §(8.32)

$$Vrd(x) := bw\cdot z\cdot\tau Rdc(x)$$



MEMBERS PROVIDED WITH SHEAR REINFORCEMENT (§8.2.3)

$\theta v := \mathrm{atan}\left(\frac{1}{2}\right) = 0.464$ angle of inclination of concrete compressed strut

$\nu := 0.5$ §8.2.3(6) NOTE: steel grade B500A is used

$\sigma cd := \tau Ed \cdot (\cot(\theta v) + \tan(\theta v)) = 5.139$ MPa $<$ $\nu \cdot -fcd = 15.453$ MPa CHECK §(8.44)

$fywd := fsd = 454.545$ MPa design yield stress of shear reinforcement steel

$Asw := 2 \cdot \frac{8^2 \cdot \pi}{4} = 100.531$ area of transverse shear reinforcement

$s1 := 80$ mm spacing of transverse reinforcement (first field near supports)

$\tau Rd\_sy := \frac{Asw}{bw \cdot s1} \cdot fywd \cdot \cot(\theta v) = 2.856$ MPa $>$ $\tau Ed = 2.056$ MPa CHECK §(8.42)

MOMENT DIAGRAM ACCOUNTING FOR THE SHEAR RESISTING MECHANISM (§12.3.2)

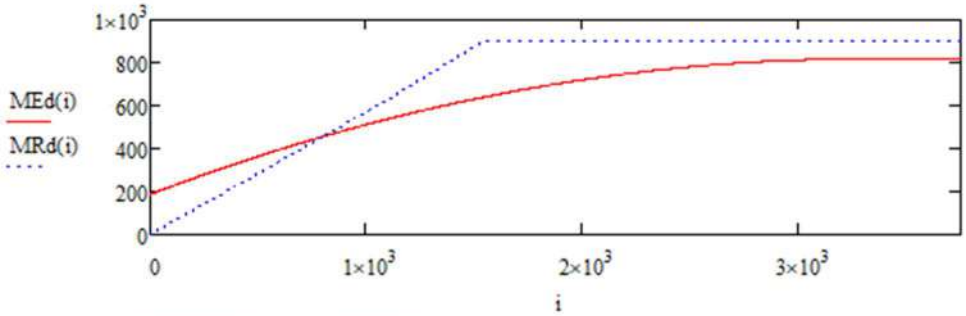
$\alpha 3 := 1$ fatigue check not required

$lbpd := lpt2 + \frac{\gamma c}{1.5} \cdot \frac{2 \cdot \alpha 2 \cdot \alpha 3 \cdot \left(fptd - \sigma pm\left(\frac{L}{2}, t\right)\right)}{\eta 1 \cdot \sqrt{(-fck)}} \cdot \phi p = 1.539 \times 10^3$ mm §13.5.2

$$MRd(x) := \mathrm{if}\left[x < lpt2, Mrd \cdot \frac{\sigma pm(lpt2, t)}{fptd} \cdot \frac{x}{lpt2}, \mathrm{if}\left[x > lbpd, Mrd, Mrd \cdot \frac{\sigma pm(lpt2, t)}{fptd} + \frac{(x - lpt2)}{(lbpd - lpt2)} \cdot \left(Mrd - Mrd \cdot \frac{\sigma pm(lpt2, t)}{fptd}\right)\right]\right]$$

$al := z \cdot \left(\frac{\cot(\theta v)}{2}\right) = 457.714$

$$MEd(x) := \mathrm{if}\left(x > \frac{L}{2} - al, \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6}, \frac{Mq\_ULS(x + \mathrm{round}(z))}{10^6}\right)$$



CHECK OF STIRRUPS FOR SUSPENSION LOAD

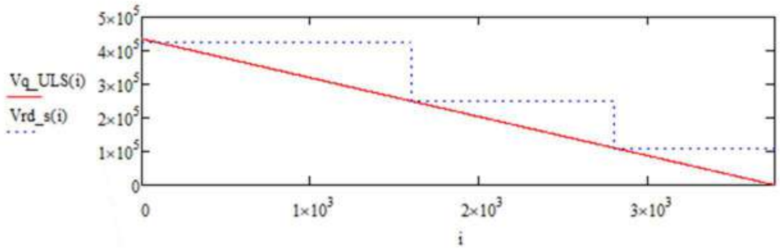
$\frac{\tau Ed}{\cot(\theta v)} \cdot bw + \left(\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5\right) = 519.791$ kN / m < $\frac{Asw \cdot fywd}{s1} = 571.199$ kN / m CHECK BEFORE TRANSFER OF PRESTRESSING

$\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5 = 108.644$ kN / m < $\frac{Asw \cdot fywd}{s3} = 228.479$ kN / m CHECK AT MAXIMUM SPACING

$\tau Rd\_sy2 := \frac{Asw}{bw \cdot s2} \cdot fywd \cdot \cot(\theta v) = 1.904$ MPa

$\tau Rd\_sy3 := \frac{Asw}{bw \cdot s3} \cdot fywd \cdot \cot(\theta v) = 1.142$ MPa

$Vrd\_s(x) := if(x < 1600, \tau Rd\_sy \cdot bw \cdot z, if(x < 2800, \tau Rd\_sy2 \cdot bw \cdot z, \tau Rd\_sy3 \cdot bw \cdot z)) - \left(\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5\right) \cdot \cot(\theta v) \cdot z$



$\frac{Vq\_ULS(d)}{Vrd\_s(d)} = 0.889$

CHECK OF HORIZONTAL SADDLE BAR

$\frac{\left(\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5\right)}{2} \cdot 100 \cdot s2 = 6.519 \times 10^5$ Nmm < $\pi \cdot \frac{8^2}{4} \cdot fsd \cdot 0.9 \cdot 220 = 4.524 \times 10^6$ CHECK

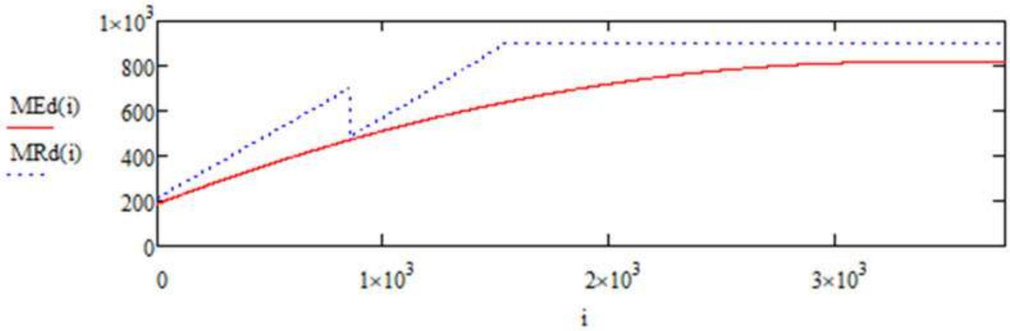
horizontal saddle rebars

CHECK OF SUPPORT MILD REBARS

$MEd(0) = 187.248$ Nmm

$0.25 \cdot MEd\left(\frac{L}{2}\right) = 204.107$ Nmm §12.1(4) < $\left(4 \cdot \pi \cdot \frac{16^2}{4} + 2 \cdot \pi \cdot \frac{12^2}{4}\right) \cdot fsd \cdot 0.9 \cdot \frac{540}{10^6} = 227.634$ CHECK

$MRd(x) := if\left[x < 850, 0.9 \cdot d \cdot \frac{fywd}{10^6} \cdot \left(4 \cdot \pi \cdot \frac{16^2}{4} + 2 \cdot \pi \cdot \frac{12^2}{4}\right) + MRd(x), MRd(x)\right]$



ANCHORAGE (§11.4)

$klb := 50$

$kcp := 1$ for good bond conditions

$n\sigma := \frac{3}{2}$

$cs := 50$

$cx := 75$

$cy := 40$

$cd(\phi) := \min(0.5 \cdot cs, cx, cy, 3.75 \cdot \phi)$ $cd(12) = 25$

$$lbd(\phi) := \max\left[klb \cdot kcp \cdot \phi \cdot \left(\frac{fsd}{435}\right)^{n\sigma} \cdot \left(\frac{25}{-fck}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1.5 \cdot \phi}{cd(\phi)}\right)^{\frac{1}{2}}, 10 \cdot \phi\right]$$

$lbd(12) = 341.872$ $\frac{lbd(12)}{12} = 28.489$

length of straight part for 90° bent bars

$lb90(\phi) := \max(70, lbd(\phi) - 15 \cdot \phi, 10 \cdot \phi)$

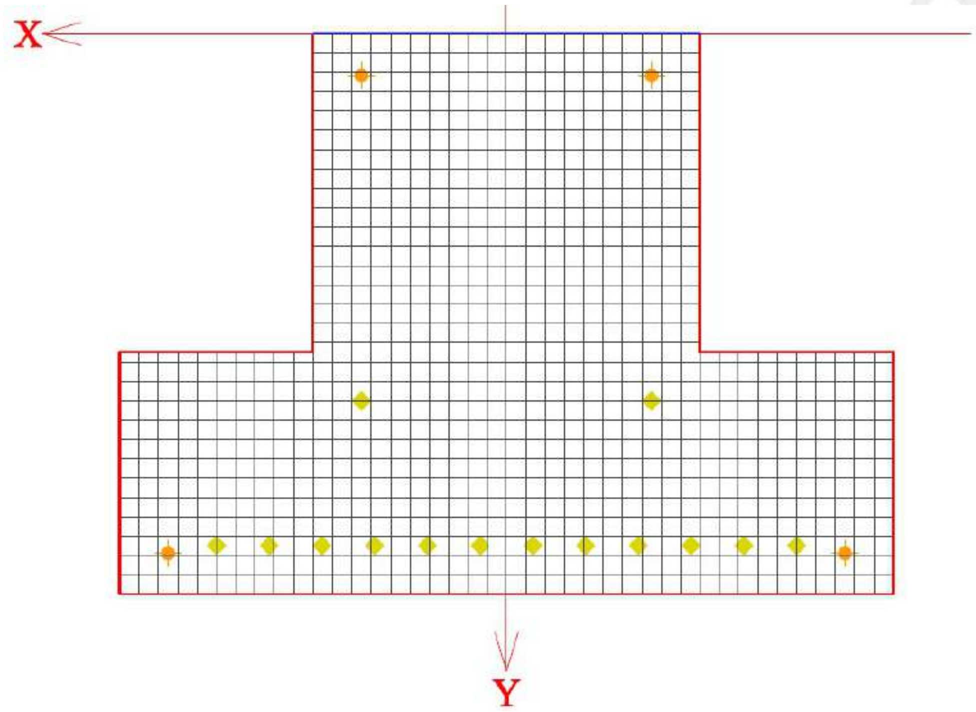
$lb90(12) = 161.872$ $lb90(16) = 339.319$

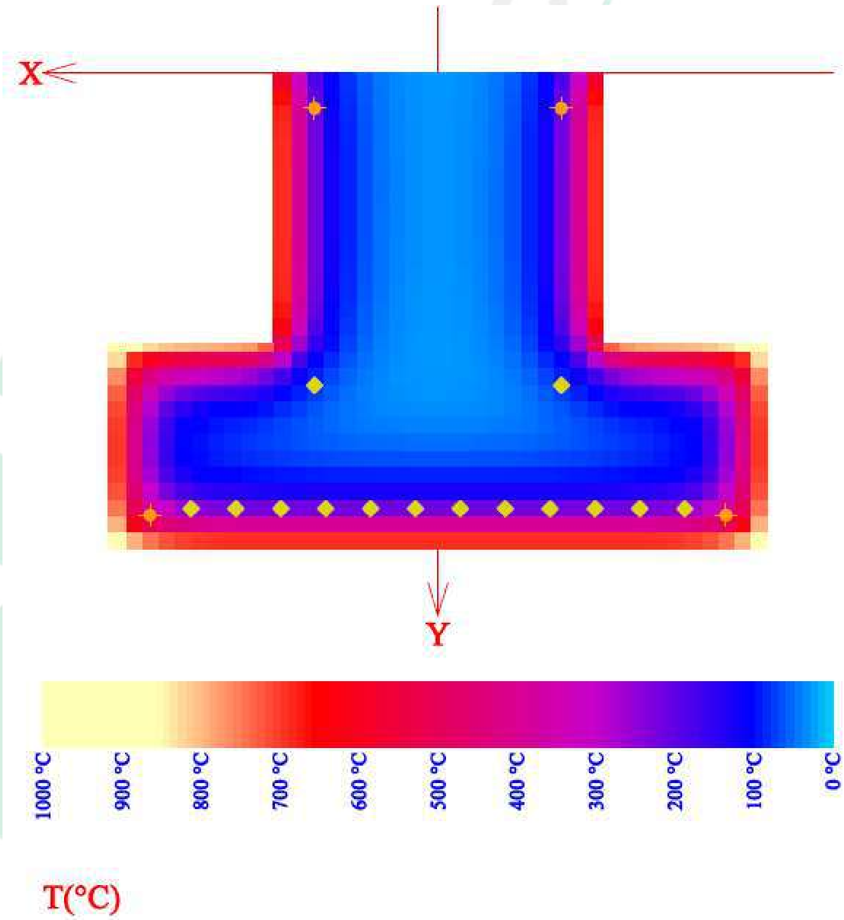
length of straight part for 135° bent bars (stirrups)

$lb135(\phi) := \max(50, lbd(\phi) - 15 \cdot \phi, 5 \cdot \phi)$

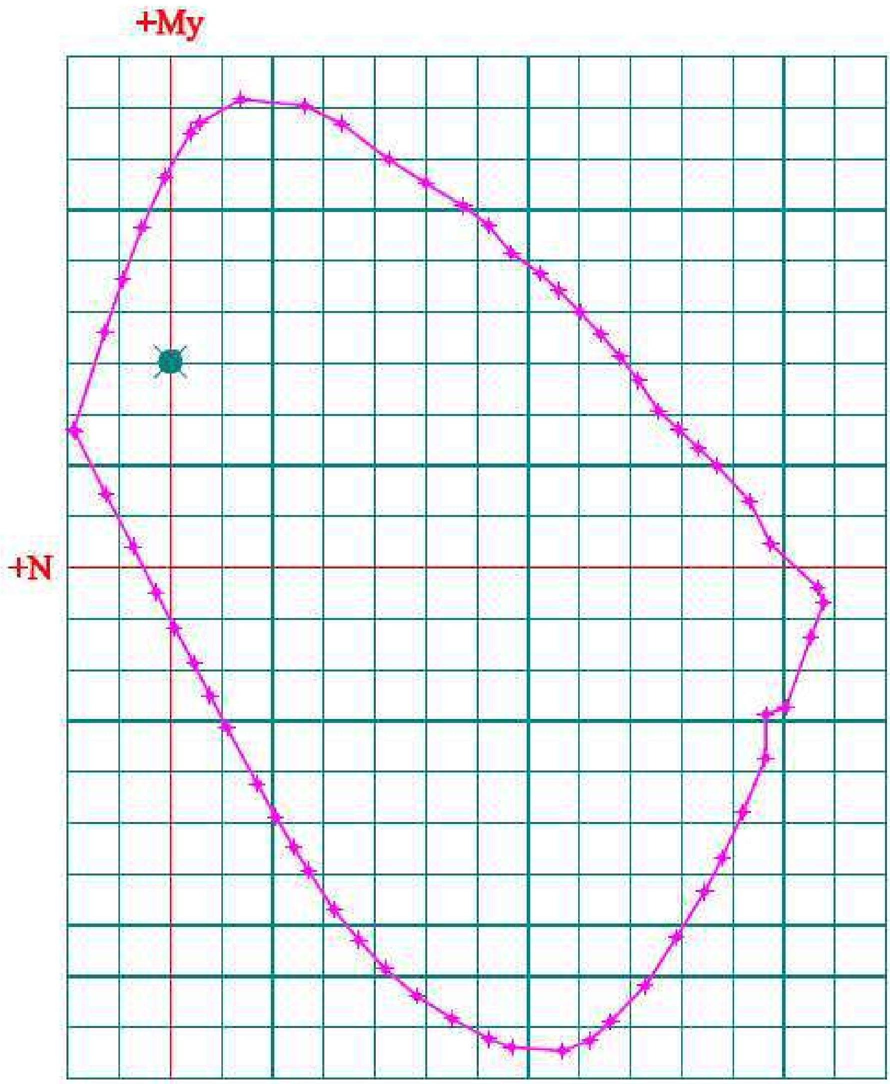
$lb135(12) = 161.872$ $lb135(8) = 50$

12.11 Fire checks





6	29	34	44	60	83	120	212	386	684											
6	29	35	44	60	83	121	212	387	684											
6	29	35	45	60	83	121	213	387	685											
6	29	35	45	60	83	121	213	388	685											
6	29	35	45	60	83	121	213	387	685											
6	29	35	45	60	83	121	213	387	685											
6	29	34	44	60	83	121	212	387	684											
6	29	34	44	59	82	119	211	386	684											
6	28	34	43	59	81	117	209	383	682											
6	28	33	43	57	79	111	205	377	678											
5	27	32	41	56	77	100	196	363	665											
5	27	31	39	53	73	100	178	329	626											
4	26	30	37	49	67	97	149	262	500	726	768	780	786	790	795	805	823	853	903	
4	26	29	35	44	59	80	100	175	290	414	469	493	505	515	530	555	601	683	823	
5	26	28	33	40	51	66	86	113	165	217	252	274	289	303	324	361	428	548	760	
6	27	29	32	37	45	56	69	86	100	129	149	161	170	182	206	253	332	473	724	
9	30	31	33	37	42	49	58	69	80	92	100	100	100	119	145	192	279	432	705	
5	35	36	38	40	43	48	54	60	67	74	81	86	92	100	105	163	251	412	696	
5	45	46	47	48	50	53	56	60	65	69	74	79	86	94	100	155	241	404	692	
0	60	61	61	62	63	65	67	69	72	75	79	83	89	97	100	158	245	407	693	
4	84	84	84	85	85	86	87	89	90	93	96	100	100	100	129	175	264	421	700	
22	122	122	122	122	123	123	124	125	127	129	132	136	143	153	174	220	303	451	714	
13	213	213	213	213	213	213	214	214	215	216	217	220	226	238	261	303	376	507	741	
88	388	388	388	388	388	388	388	388	388	389	389	391	395	403	419	450	507	609	789	
85	685	685	685	685	685	685	685	685	685	685	685	686	688	692	699	713	741	789	871	

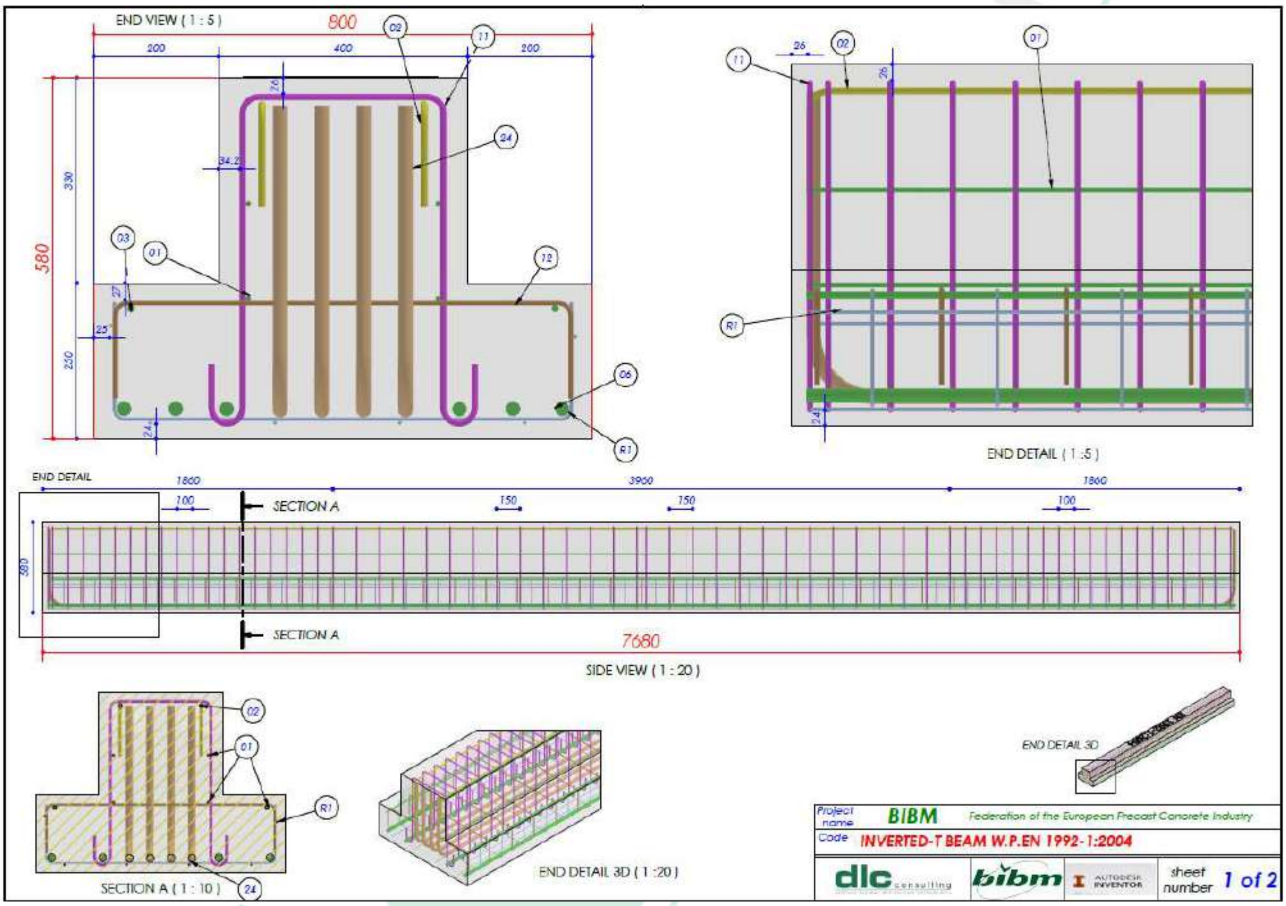


N: 1 sp = 1100.00 kN;   M: 1 sp = 110.00 kN·m

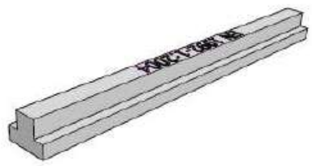
Nz = 0.00 kN
My+ = 861.49 kN·m
My- = -112.50 kN·m

13 Reinforced beam element –EN1992-1:2004

13.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_	Ø_longitudinal	pattern_T	Ø_transverse	pattern_L
	01	4	3011	12044	8 mm				
	02	2	7032	14064	12 mm				
	03	2	6774	13548	12 mm				
	06	4	27096	108384	24 mm				
	08	2	14489	28978	24 mm				
	11	65	982	63830	10 mm				
	12	39	402	15678	8 mm				
	24	4	30184	120736	24 mm				
Total mass rebars [kg]				377.26		Incidence kg/m³	147.95		
	R1	1	19373	19373		6 mm	200 mm	6 mm	200 mm
l mass welded-wire-meshes [kg]				19.37		Incidence kg/m³	7.60		
Total mass of steel [kg]				396.64		Total concrete volume [m³]	2.55		



Project name	BIBM	Federation of the European Precast Concrete Industry
Code	INVERTED-T BEAM W.P.EN 1992-1:2004	
	sheet number	2 of 2

13.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0 \quad 329.99 \quad 330 \quad 580)^T$

$Htot := \max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := (400 \quad 400 \quad 800 \quad 800)^T$

$r\_circ := 0$ radius of central void pipe

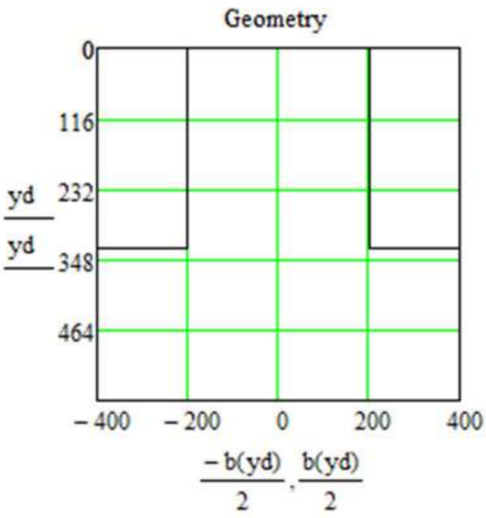
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$b\_lin(y) := \mathrm{linterp}(y\_tr, b\_tr, y)$

$b\_circ(y) := \mathrm{linterp}(y\_tr, b\_tr, y) - x\_circ(y)$

$$b(y) := \mathrm{if}\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$yd := 0..Htot$



condensed 1D geometry plot

$u := 800 \cdot 2 + Htot \cdot 2 = 2.76 \times 10^3$ exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (43\ \ 202\ \ 354\ \ 370\ \ 488\ \ 538)^T$$

Area of reinforcement at each depth

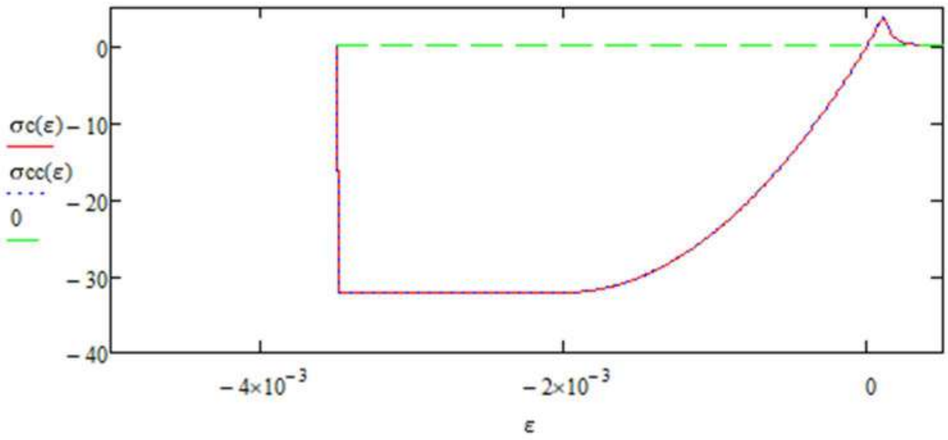
$$As := (2 \cdot A(12)\ \ 2 \cdot A(8)\ \ 2 \cdot A(8)\ \ 2 \cdot A(8)\ \ 0 \cdot A(24)\ \ 10 \cdot A(24))^T$$

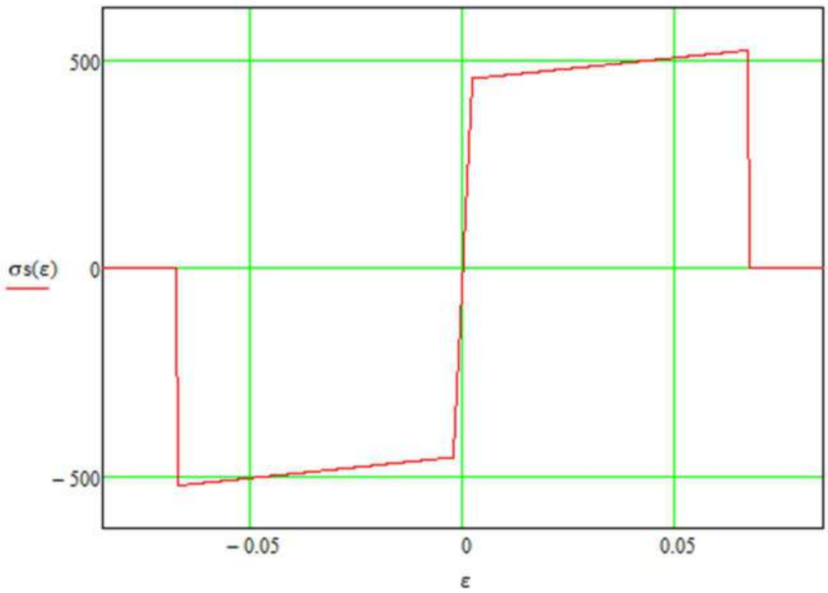
$js := rows(As)$ $\qquad js = 6$

$dsmax := max(ds)$ $\qquad dsmax = 538$

$$As\_tot := \sum_{j=1}^{js} As_j = 5.052 \times 10^3$$

13.3 Material constitutive laws employed in the calculation





13.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 3.321 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 0.015$$ geometric ratio for longitudinal mild reinforcement

$$\rho tot := \frac{As\_tot}{Ac} = 0.015$$ total geometric ratio for longitudinal reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y)\cdot y\,dy \qquad Syc = 1.128 \times 10^8$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 339.696$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y)\cdot(y - yG)^2\,dy \qquad Ixo\_cls = 8.927 \times 10^9$$

Idealisation coefficients (elastic)

$$ns := \frac{Es}{Ecm} \qquad ns = 5.512$$

Area of ideal cross-section

$$Aid := Ac + (ns - 1) \cdot \sum_{j=1}^{js} As_j \qquad Aid = 3.549 \times 10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac \cdot yG + (ns - 1) \cdot \sum_{j=1}^{js} \left(As_j \cdot ds_j\right) \qquad Sxid = 1.243 \times 10^8$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 350.13$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y) \cdot (y - Yid)^2 \, dy - \sum_{j=1}^{js} \left[As_j \cdot \left(ds_j - Yid\right)^2\right]$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns \cdot \sum_{j=1}^{js} \left[As_j \cdot \left(ds_j - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidlenta \qquad Ixo\_id = 9.79 \times 10^9 \quad \text{mm^4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.097$$

13.5 Loads

LOADS

$g1 := Ac \cdot 0.000025 = 8.302$ kN/m — dead load from self-weight

$g2 := (2 + 2.89) \cdot 9.45 = 46.211$ kN/m — nonstructural dead load

$q := 28.35$ kN/m — live load

$L := 7500$ mm — calculation length (span between supports)

$\psi 2 := 0.3$ — non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ — non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ — SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ — SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2} \right)$ — SLS bending moment distribution from live load

13.6 Time-dependent behaviour

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$h0 := 2 \cdot \frac{Ac}{u} = 240.643$ mm notional size of the member

$RH := 50$ % relative humidity

$t0\_T(t0) := t0$

$\alpha := 1$ for cement class R

$$t0\_mod(t0) := \max\left[t0\_T(t0) \cdot \left(\frac{9}{2 + t0\_T(t0)^{1.2}} + 1\right)^{\alpha}, 0.5\right]$$

time modification due to type of cement §B.9

$t0\_mod(2) = 6.189$

$$\alpha c1 := \left(\frac{35}{-fcm}\right)^{0.7} = 0.748$$

$$\alpha c2 := \left(\frac{35}{-fcm}\right)^{0.2} = 0.92$$

$$\alpha c3 := \left(\frac{35}{-fcm}\right)^{0.5} = 0.813$$

$$\beta h := if\left[-fcm > 35, \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250 \cdot \alpha c3, 1500 \cdot \alpha c3\right], \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250, 1500\right]\right] = 564.161$$

$$\beta t0(t0) := \frac{1}{0.1 + t0\_mod(t0)^{0.2}}$$

$$\beta c(t, t0) := \left(\frac{t - t0\_mod(t0)}{\beta h + t - t0\_mod(t0)}\right)^{0.3}$$

$$\beta fcm := \frac{16.8}{\sqrt{-fcm}} = 2.308$$

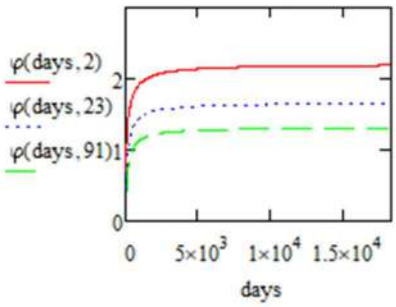
$$\varphi RH := if\left[-fcm > 35, \left(1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}} \cdot \alpha c1\right) \cdot \alpha c2, 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}}\right] = 1.474$$

$\varphi 0(t0) := \varphi RH \cdot \beta fcm \cdot \beta t0(t0)$

$\varphi(t, t0) := \varphi 0(t0) \cdot \beta c(t, t0)$

$t := 50 \cdot 365 = 1.825 \times 10^4$

$\varphi(t, 2) = 2.188$ $\varphi(t, 91) = 1.304$



13.7 Non-linear moment-curvature diagram

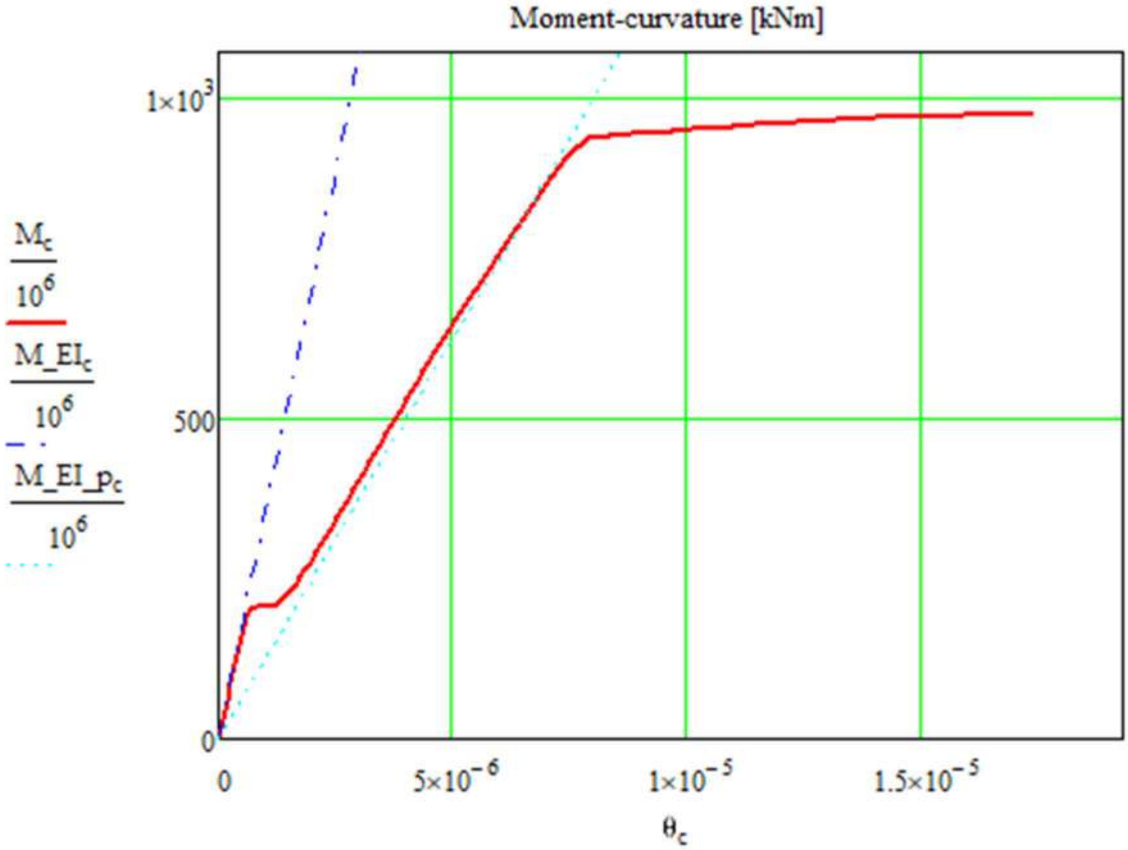
Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

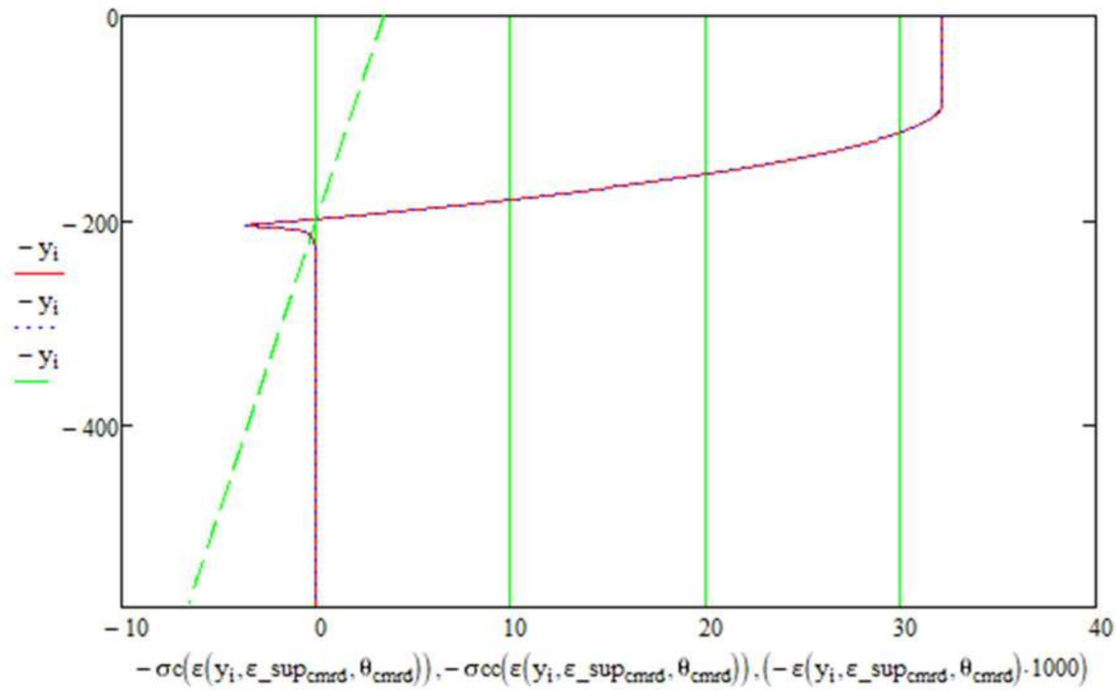
$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot} \left(\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot \Delta y\right) + \sum_{j=1}^{js} \left(\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\right)$$

$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot} \left[\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot \Delta y\cdot\left(y_i - yG\right)\right] + \sum_{j=1}^{js} \left[\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\cdot\left(ds_j - yG\right)\right]$$

Design external axial load

$NS := -0$





Condition at resisting (peak) moment (stress and strain)

13.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformally distributed load q

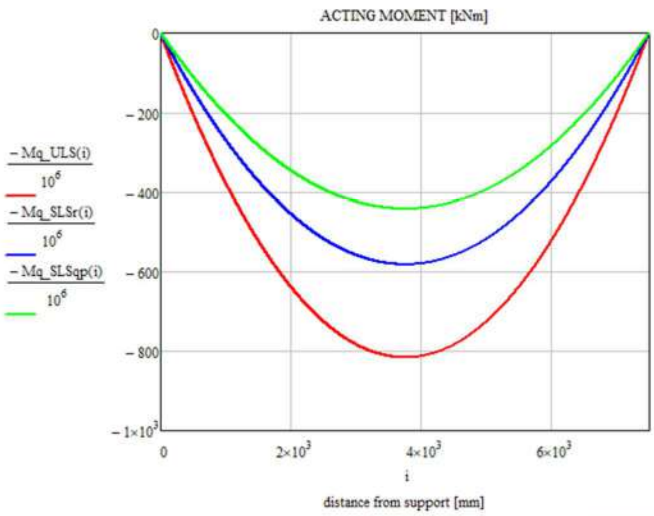
$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformally distributed load q

$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformally distributed load q

$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformally distributed load q

$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformally distributed load q

$i := 0 .. L$



13.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$v\_inf\_p(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(1 + \varphi(365\cdot50,23))$

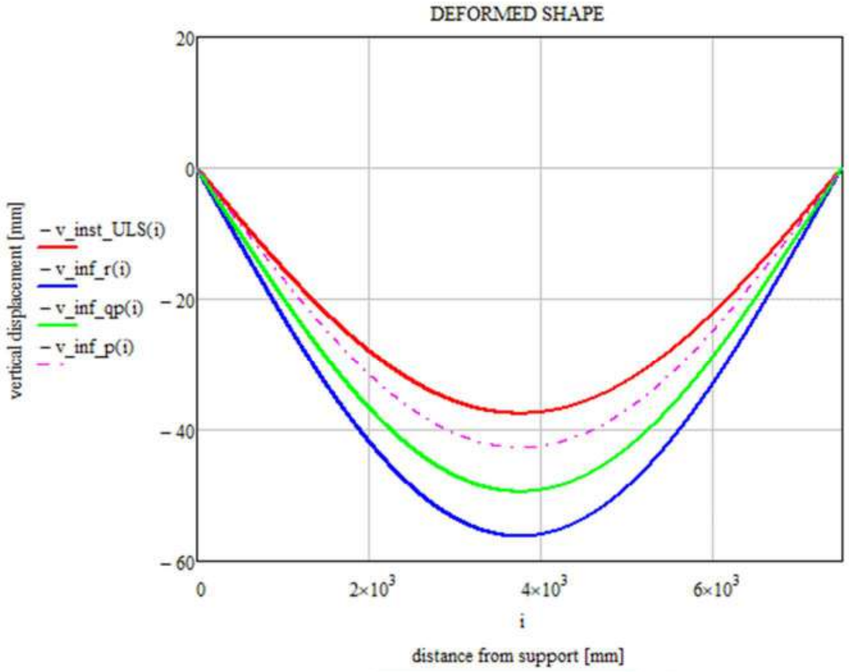
deflection profile at 50 years including creep for permanent load combination

$v\_inf\_qp(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot50,23) - \varphi(365\cdot50,91)) + v\_SLSqp(x)\cdot(1 + \varphi(365\cdot50,91))$

deflection profile at 50 years including creep for quasi permanent load combination

$v\_inf\_r(x) := v\_SLSg1(x)\cdot(\varphi(365\cdot50,2) - \varphi(365\cdot50,23)) + v\_SLSg2(x)\cdot(\varphi(365\cdot50,23) - \varphi(365\cdot50,91)) + v\_SLSqp(x)\cdot\varphi(365\cdot50,91) + v\_SLSr(x)$

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§7.4.3)

$camber := 30$ mm $\quad > \quad \frac{-L}{250} = -30$ CHECK maximum camber

imposed camber by mould shaping

$v\_inf\_r\left(\frac{L}{2}\right) - camber = 26.235 \quad < \quad \frac{L}{250} = 30$ CHECK maximum deflection

value calculated from differential equations above

SLS STRESS CONTROL (§7.2)

$k1 := 0.6$

$k2 := 0.45$

$k3 := 0.8$

$k4 := 1$

$k5 := 0.75$

$\sigma cpf\_bot(x) := \frac{Mq\_SLSf(x)\cdot(Htot - Yid)}{Ixo\_id} \qquad \sigma cpf\_bot\left(\frac{L}{2}\right) = 11.34 \quad < \quad fctm = 3.795$ CHECK

elastic stress of bottom concrete chord for frequent load combination

if not -> cracked

$\sigma sf\_bot(x) := 15\cdot\left[\frac{Mq\_SLSf(x)\cdot\left(ds_{js} - Yid\right)}{Ixo\_id}\right] \qquad \sigma sf\_bot\left(\frac{L}{2}\right) = 139.018$

creep stress of bottom reinforcement layer for frequent load combination

$\sigma cpr\_bot(x) := \frac{Mq\_SLSr(x)\cdot(Htot - Yid)}{Ixo\_id} \qquad \sigma cpr\_bot\left(\frac{L}{2}\right) = 13.68 \quad < \quad fctm = 3.795$

elastic stress of bottom concrete chord for rare load combination

$\sigma cpr\_top(x) := \frac{Mq\_SLSr(x)\cdot(-Yid)}{Ixo\_id} \qquad \sigma cpr\_top\left(\frac{L}{2}\right) = -20.837 \quad > \quad k1\cdot fck = -27$ CHECK

$> \quad 0.4\cdot fcm = -21.2$

elastic stress of top concrete chord for rare load combination

$\sigma cpr\_s(x) := 15\cdot\left[\frac{Mq\_SLSr(x)\cdot\left(ds_{js} - Yid\right)}{Ixo\_id}\right] \qquad \sigma cpr\_s\left(\frac{L}{2}\right) = 167.707 \quad < \quad k3\cdot fsk = 400$ CHECK

creep stress of bottom mild steel for rare load combination

SLS CRACK CONTROL (§7.3)

$c\_act := Htot - ds_{js} - 10 = 32$

$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1.5$

$wlim\_cal := 0.2$ mm

$k1c := 0.8$ $\phi := 24$

$k2c := 0.5$

$k3c := 3.4$

$k4c := 0.425$

$cover := Htot - \frac{\phi}{2} - ds_{js} = 30$

$Aceff := b(Htot)\cdot\min\left[2.5\cdot\left(Htot - ds_{js}\right), \frac{Htot - Yn\_n}{3}, \frac{Htot}{2}\right] = 8.4 \times 10^{4}$

$\rho peff := \frac{As_{js} + As_{js-1}}{Aceff} = 0.054$

$srmax := k3c\cdot cover + \frac{k1c\cdot k2c\cdot k4c\cdot\phi}{\rho peff} = 177.758$

$kt := 0.4$ NOTE : 0.6 for sustained loading

$fcteff := fctm = 3.795$

$\varepsilon sm\_\varepsilon cm := \max\left[\frac{\sigma sf\_bot\left(\frac{L}{2}\right) - kt\cdot\frac{fcteff}{\rho peff}\cdot\left(1 + \frac{Es}{Ecm}\cdot\rho peff\right)}{Es}, 0.6\cdot\frac{\sigma sf\_bot\left(\frac{L}{2}\right)}{Es}\right] = 5.123 \times 10^{-4}$

$wk := srmax\cdot\varepsilon sm\_\varepsilon cm = 0.091$ $<$ $wlim\_cal = 0.2$ CHECK

13.10 ULS checks

ULS BENDING-AXIAL CONTROL (§6.1)

$$Mrd = 974.72 \qquad > \qquad \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6} = 816.449 \qquad \text{CHECK}$$

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§6.2)

$$Vq\_ULS(x) := \left|(g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2} - x\right)\right|$$ shear distribution at Ultimate Limit State (ULS)

$d := ds_{js} = 538$ mm — effective depth

$VEd := Vq\_ULS(d) = 3.73 \times 10^5$ N — maximum shear at effective depth from support

$bw := 400$ mm — web width

$z := 0.9\cdot d = 484.2$ — conventional resultant lever arm

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT (§6.2.2)

$$\rho 1 := \frac{\sum_{j=1}^{js} As_j}{bw\cdot d} = 0.023$$ reinforcement ratio

$\sigma cp(x) := 0$ MPa — axial load induced by prestressing

$$kv := \min\left(1 + \sqrt{\frac{200}{d}}, 2\right) = 1.372$$

$k1v := 0.15$

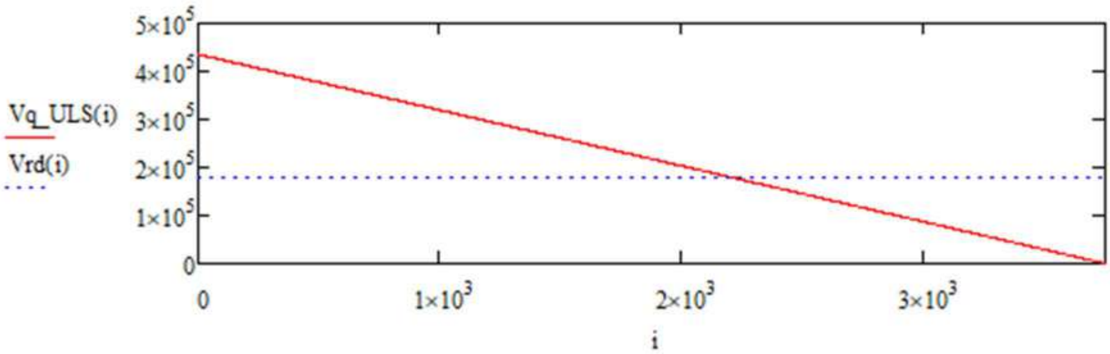
$$Crdc := \frac{0.18}{\gamma cpcred} = 0.129$$

$$vmin := 0.035\cdot k^{\frac{3}{2}}\cdot(-fck)^{\frac{1}{2}} = 0.61 \qquad \text{§6.3N}$$

$$VRdc(x) := \max\left[\left[Crdc\cdot kv\cdot(100\cdot\rho 1\cdot -fck)^{\frac{1}{3}} + k1v\cdot\sigma cp(x)\right]\cdot bw\cdot d, (vmin + k1v\cdot\sigma cp(x))\cdot bw\cdot d\right] \qquad \text{§6.2.a+§6.2.b}$$

bw·d·[Crdc·k

(vmin + k1v·


Vq_ULS(i)
Vrd(i)
5×10^5
4×10^5
3×10^5
2×10^5
1×10^5
0
1×10^3
2×10^3
3×10^3
i

MEMBERS PROVIDED WITH SHEAR REINFORCEMENT (§6.2.3)

$fywd := fsd = 454.545$ MPa design yield stress of transverse reinforcement

$Asw := 2 \cdot \frac{8^2 \cdot \pi}{4} = 100.531$ area of transverse reinforcement (pseudo-vertical stirrups)

$\theta v := \operatorname{atan}\left(\frac{1}{2.5}\right) = 0.381$ angle of inclination of concrete compressed strut

$s1 := 100$ mm spacing of transverse reinforcement (end field)

$Vrds := \frac{Asw}{s1} \cdot z \cdot fywd \cdot \cot(\theta v) = 5.531 \times 10^5$ N > $VEd = 3.73 \times 10^5$ CHECK shear resistance on steel side (§6.8)

$\nu 1 := 0.6 \cdot \left(1 - \frac{fck}{250}\right) = 0.708$ §6.10

$\alpha cw(x) := \text{if}\left[\sigma cp(x) < 0.25 \cdot -fcd, 1 + \frac{\sigma cp(x)}{-fcd}, \text{if}\left[\sigma cp(x) > 0.5 \cdot -fcd, 2.5 \cdot \left(1 - \frac{\sigma cp(x)}{-fcd}\right), 1.25\right]\right]$ §6.11

$Vrdmax(x) := \alpha cw(x) \cdot bw \cdot z \cdot \nu 1 \cdot \frac{-fcd}{\cot(\theta v) + \tan(\theta v)}$ shear resistance on concrete side (§6.9)

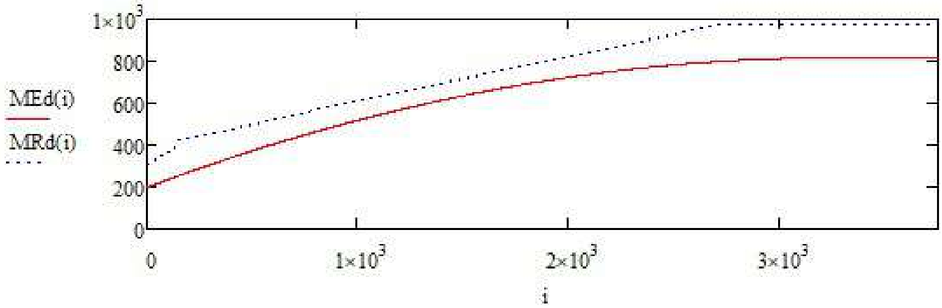
$Vrdmax(d) = 1.52 \times 10^6$ N > $VEd = 3.73 \times 10^5$ CHECK

MOMENT DIAGRAM ACCOUNTING FOR SHIFTING DUE TO SHEAR RESISTING MECHANISM (§9.2.1.3)

$$MRd(x) := \text{if}\left(x < 140, \frac{4}{10} \cdot Mrd \cdot \frac{x + 500}{640}, \text{if}\left(x < 640, \frac{4}{10} \cdot Mrd + \frac{4}{10} \cdot Mrd \cdot \frac{x}{1800}, \text{if}\left(x < 2720, \frac{8}{10} \cdot Mrd + \frac{2}{10} \cdot Mrd \cdot \frac{x - 1800}{2720 - 1800}, Mrd\right)\right)\right)$$

$$al := z \cdot \left(\frac{\cot(\theta v)}{2}\right) = 605.25$$

$$MEd(x) := \text{if}\left(x > \frac{L}{2} - al, \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6}, \frac{Mq\_ULS(x + \text{round}(z))}{10^6}\right)$$



MINIMUM REINFORCEMENT

$bt := b(Htot) = 800$ §9.1N §9.2.1.1(3)

$Asmin := \max\left(0.26 \cdot \frac{fctm}{fsk} \cdot bt \cdot d, 0.0013 \cdot bt \cdot d\right) = 849.451$ mm2 < $\rho l \cdot Ac = 7.796 \times 10^{3}$ mm2 CHECK < $0.04 \cdot Ac = 1.328 \times 10^{4}$ CHECK

$s3 := 200$ mm < $d \cdot 0.75 = 403.5$ mm CHECK for shear reinforcement §9.6N $x3 := 3400$ mm

larger stirrup spacing in the middle of the beam

$\rho w\_min := \frac{Asw}{s3 \cdot bw} = 1.257 \times 10^{-3}$ > $0.08 \cdot \frac{\sqrt{-fck}}{fsk} = 1.073 \times 10^{-3}$ CHECK §9.5N

CHECK OF STIRRUPS FOR SUSPENSION LOAD

$\frac{VEd}{\cot(\theta v) \cdot z} + \left(\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5\right) = 416.756$ kN / m < $\frac{Asw \cdot fywd}{s1} = 456.959$ kN / m CHECK

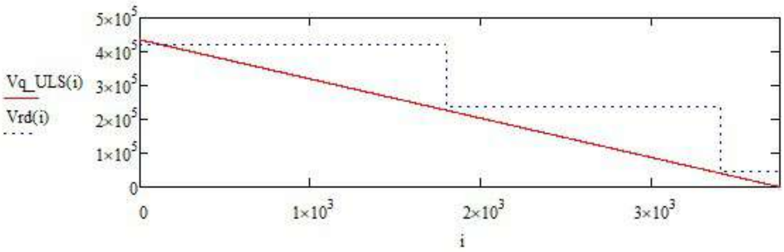
$s2 := 150$ mm $x2 := 1800$

midlle field of stirrup spacing

$Vrds2 := \frac{Asw}{s2} \cdot z \cdot fywd \cdot \cot(\theta v) = 3.688 \times 10^{5}$ N > $Vq\_ULS(x2) = 2.264 \times 10^{5}$ CHECK $\left(\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5\right) = 108.645$

$\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5 = 108.645$ kN / m < $\frac{Asw \cdot fywd}{s2} = 304.639$ kN / m CHECK

$Vrd(x) := \text{if}(x < x2, Vrds, \text{if}(x < x3, Vrds2, Vrd(x))) - \left(\frac{g1}{3} \cdot 1.35 + g2 \cdot 1.35 + q \cdot 1.5\right) \cdot \cot(\theta v) \cdot z$



$s1 = 100$

$s2 = 150$

$s3 = 200$

NOTE: first section to be checked at distance d from support

CHECK OF HORIZONTAL SADDLE BAR

$$\frac{\left(\frac{g1}{3}\cdot 1.35 + g2\cdot 1.35 + q\cdot 1.5\right)}{2}\cdot 100\cdot s2 = 8.148\times 10^{5} \quad \text{Nmm} \quad < \quad \pi\cdot\frac{8^{2}}{4}\cdot fsd\cdot 0.9\cdot 220 = 4.524\times 10^{6} \quad \text{CHECK}$$

CHECK OF SUPPORT MILD REBARS (§9.2.1.4(1))

$$0.25\cdot Mrd = 243.68 \quad \text{Nmm} \quad < \quad \left(3\cdot\pi\cdot\frac{16^{2}}{4} + 4\cdot\pi\cdot\frac{12^{2}}{4}\right)\cdot fsd\cdot 0.9\cdot\frac{540}{10^{6}} = 233.186 \quad \text{CHECK}$$

ANCHORAGE (§8.4)

$\eta 1 := 1$

$\eta 2 := 1$

$fbd := 2.25\cdot\eta 1\cdot\eta 2\cdot fctd = 4.27$

$lbrqd(\phi) := \frac{\phi}{4}\cdot\frac{fsd}{fbd}$

$\alpha 1b := 1$

$\alpha 2b := 1$

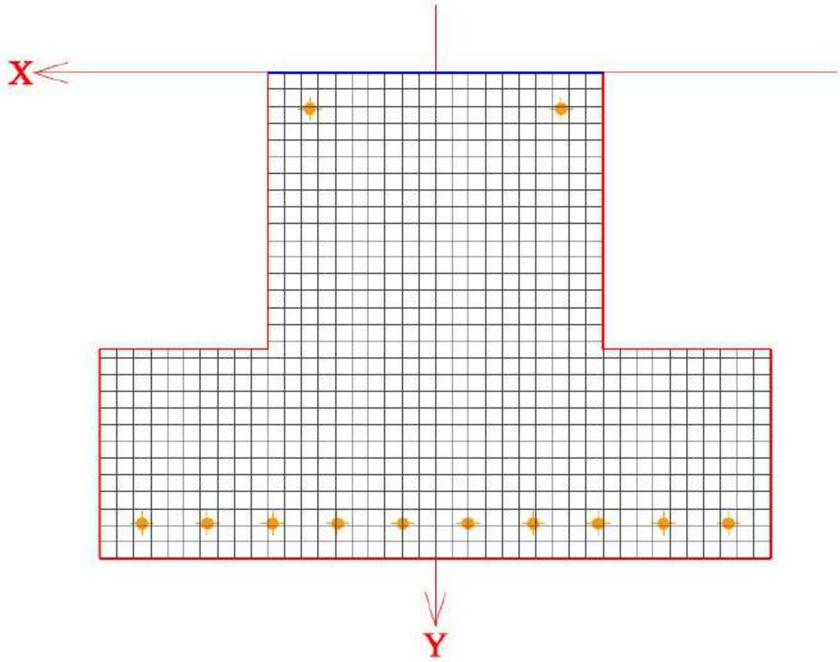
$\alpha 3b := 1$

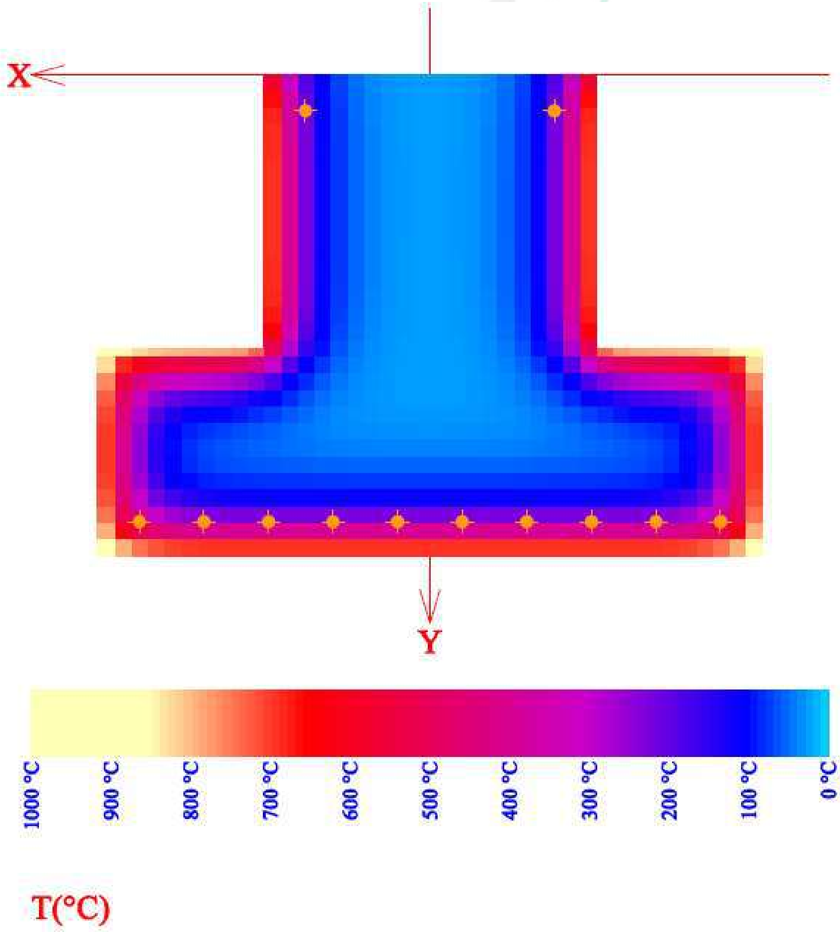
$\alpha 4b := 1$

$\alpha 5b := 1$

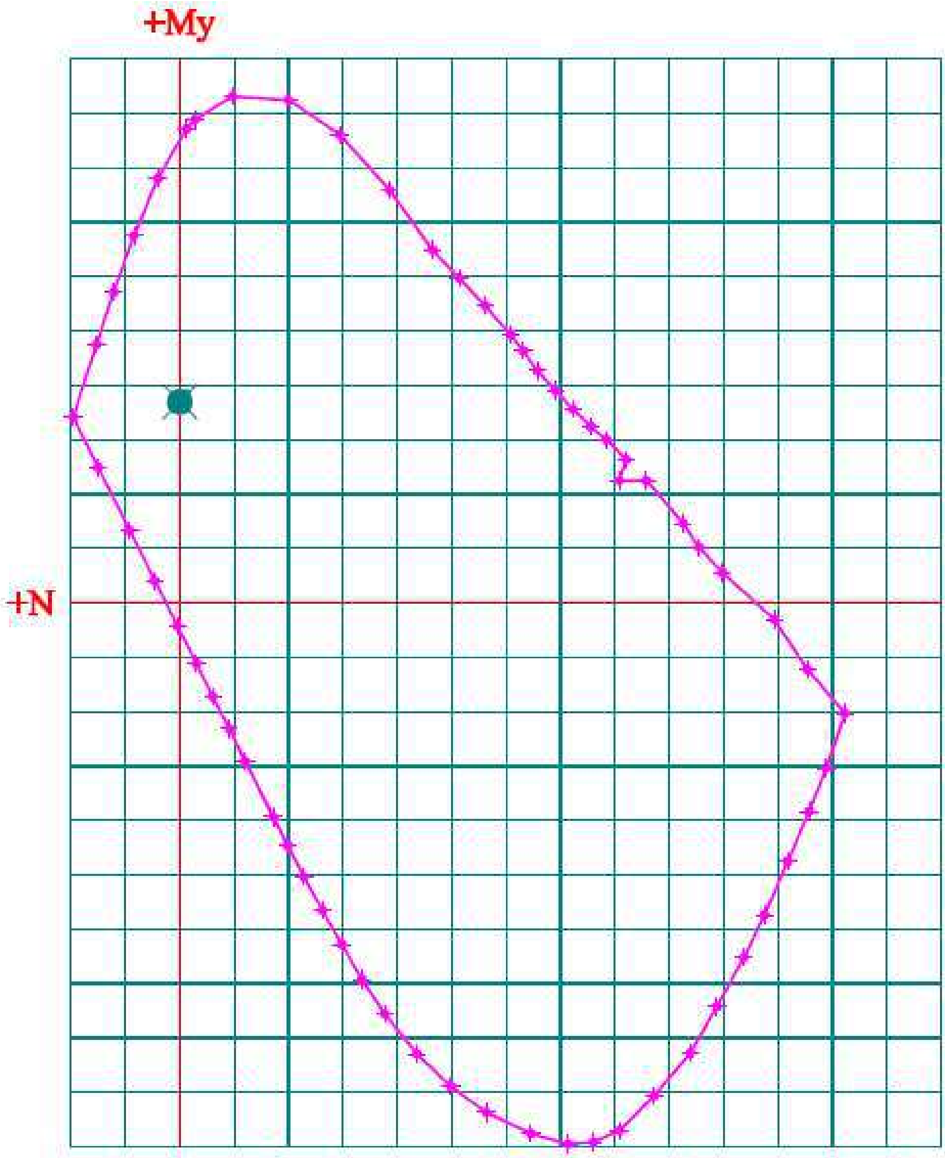
$lbd(\phi) := \alpha 1b\cdot\alpha 2b\cdot\alpha 3b\cdot\alpha 4b\cdot\alpha 5b\cdot lbrqd(\phi)$

13.11 Fire checks





22	23	27	34	47	70	100	204	363	657										
22	23	27	34	48	72	100	214	381	678										
22	23	27	35	49	73	103	218	388	684										
22	23	27	35	49	73	105	220	391	686										
22	24	27	35	49	73	106	220	392	687										
22	24	27	35	50	73	107	220	392	687										
22	24	27	35	50	73	107	221	392	687										
22	24	27	35	50	73	107	221	392	687										
22	24	27	35	50	73	107	220	392	687										
22	24	27	35	49	73	106	220	392	687										
22	23	27	35	49	73	105	220	391	686										
22	23	27	35	49	72	102	218	389	685										
22	23	27	34	48	71	100	214	383	681										
22	23	26	33	46	69	100	205	369	669										
21	23	26	32	44	65	100	187	338	632										
21	22	25	30	40	58	92	154	275	508	730	771	782	787	790	795	805	822	853	902
21	22	24	28	35	49	73	100	186	300	422	476	498	508	517	529	554	600	681	823
21	22	23	26	31	41	57	80	104	174	227	261	280	291	303	322	358	425	546	759
22	22	23	25	29	35	45	59	78	100	119	144	159	169	181	202	247	328	471	723
24	24	24	26	28	31	37	46	56	68	78	89	98	100	100	130	183	274	430	704
27	28	28	29	30	32	35	40	45	52	57	63	69	76	86	100	153	247	410	695
35	35	35	36	36	38	39	42	44	48	51	55	59	66	77	97	140	237	403	692
50	50	50	50	50	51	52	53	54	56	58	60	63	69	80	100	145	241	406	693
73	73	73	73	73	74	74	75	75	76	77	78	81	87	97	100	166	259	419	699
107	107	107	107	107	107	107	108	108	109	110	112	116	124	140	165	213	299	448	713
221	221	221	221	221	221	221	221	221	221	221	222	223	227	237	258	298	373	505	740
392	392	392	392	392	392	392	392	392	392	392	393	394	397	403	418	448	505	608	788
687	687	687	687	687	687	687	687	687	687	687	687	688	689	692	699	713	740	788	871

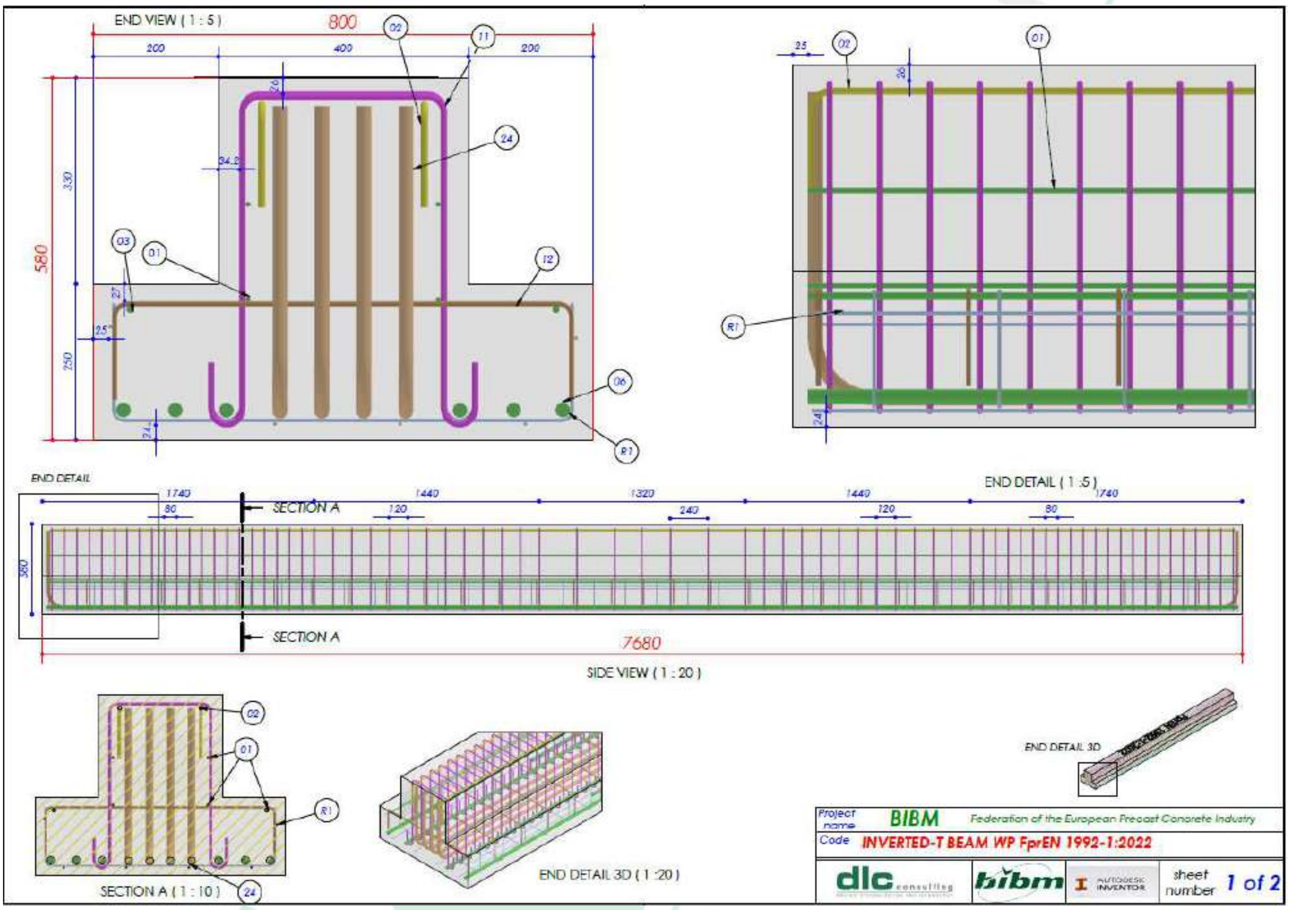


N: 1 sp = 1200.00 kN; M: 1 sp = 120.00 kN·m

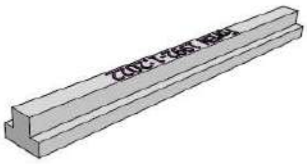
Nz = 0.00 kN
My+ = 1019.80 kN·m
My- = -60.61 kN·m

14 Reinforced beam element – FprEN1992-1:2022

14.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_	Ø_longitudinal	pattern_T	Ø_transverse	pattern_L
	01	4	3011	12044	8 mm				
	02	2	7032	14064	12 mm				
	03	2	6774	13548	12 mm				
	06	4	27096	108384	24 mm				
	07	2	18396	36792	24 mm				
	11	73	982	71686	10 mm				
	12	34	402	13668	8 mm				
	24	4	30184	120736	24 mm				
	Total mass rebars [kg]			390,92		Incidence kg/m³	153,30		
	R1	1	19373	19373		6 mm	200 mm	6 mm	200 mm
	l mass welded-wire-meshes [kg]			19,37		Incidence kg/m³	7,60		
	Total mass of steel [kg]			410,30		Total concrete volume [m³]		2,55	



Project name	BIBM	Federation of the European Precast Concrete Industry
Code	INVERTED-T BEAM WP FprEN 1992-1:2022	
dlc consulting	bibm AUTODESK INVENTOR	sheet number 2 of 2

14.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0 \;\; 329.99 \;\; 330 \;\; 580)^T$

$Htot := \max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := (400 \;\; 400 \;\; 800 \;\; 800)^T$

$r\_circ := 0$ radius of central void pipe

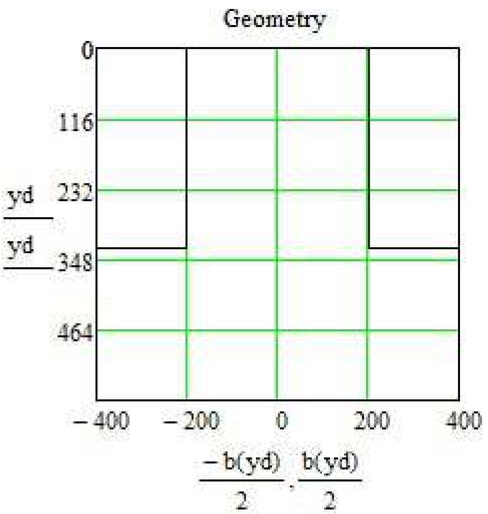
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$b\_lin(y) := linterp(y\_tr, b\_tr, y)$

$b\_circ(y) := linterp(y\_tr, b\_tr, y) - x\_circ(y)$

$$b(y) := if\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$yd := 0..Htot$



condensed 1D geometry plot

$u := 800 \cdot 2 + Htot \cdot 2 = 2.76 \times 10^3$ exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (43 \;\; 202 \;\; 354 \;\; 370 \;\; 488 \;\; 538)^T$$

Area of reinforcement at each depth

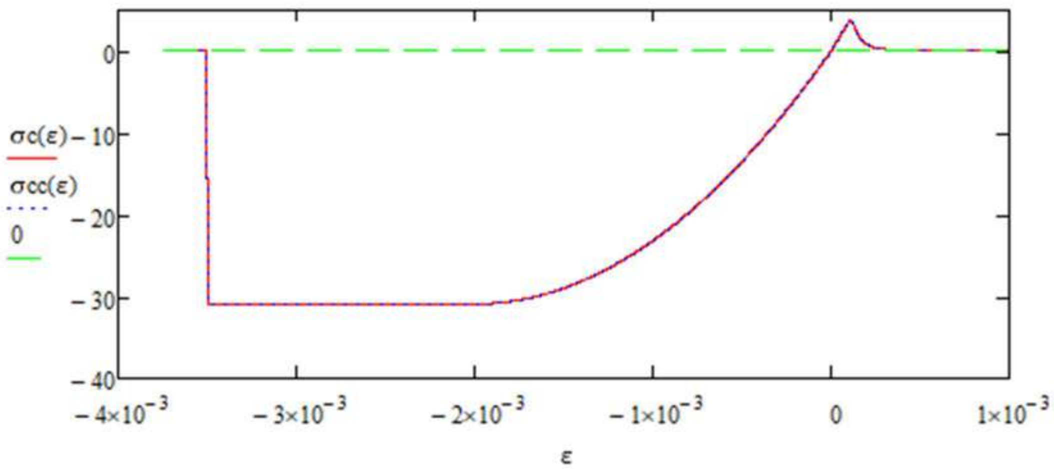
$$As := (2 \cdot A(12) \;\; 2 \cdot A(8) \;\; 2 \cdot A(8) \;\; 2 \cdot A(8) \;\; 0 \cdot A(24) \;\; 10 \cdot A(24))^T$$

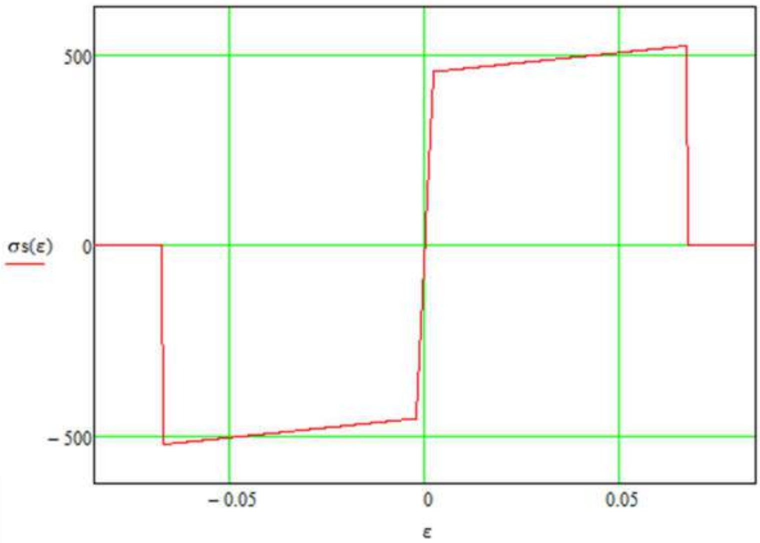
$$js := rows(As) \qquad js = 6$$

$$dsmax := max(ds) \qquad dsmax = 538$$

$$As\_tot := \sum_{j=1}^{js} As_j = 5.052 \times 10^3$$

14.3 Material constitutive laws employed in the calculation





14.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 3.321 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 0.015$$ geometric ratio for longitudinal mild reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y) \cdot y\,dy \qquad Syc = 1.128 \times 10^8$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 339.696$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y) \cdot (y - yG)^2\,dy \qquad Ixo\_cls = 8.927 \times 10^9$$

Idealisation coefficients (elastic)

$$ns := \frac{Es}{Ecm} \qquad ns = 5.605$$

Area of ideal cross-section

$$Aid := Ac + (ns - 1) \cdot \sum_{j=1}^{js} As_j \qquad Aid = 3.553 \times 10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac \cdot yG + (ns - 1) \cdot \sum_{j=1}^{js} \left(As_j \cdot ds_j\right) \qquad Sxid = 1.245 \times 10^8$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 350.33$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y) \cdot (y - Yid)^2 \, dy - \sum_{j=1}^{js} \left[As_j \cdot \left(ds_j - Yid\right)^2\right]$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns \cdot \sum_{j=1}^{js} \left[As_j \cdot \left(ds_j - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidlenta \qquad Ixo\_id = 9.807 \times 10^9 \text{ mm^4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.099$$

14.5 Loads

LOADS

$g1 := Ac \cdot 0.000025 = 8.302$ kN/m dead load from self-weight

$g2 := (2 + 2.89) \cdot 9.45 = 46.211$ kN/m nonstructural dead load

$q := 28.35$ kN/m live load

$L := 7500$ mm calculation length (span between supports)

$\psi 2 := 0.3$ non-contemporaneity factor for quasi-permanent load combination

$\psi 1 := 0.5$ non-contemporaneity factor for frequent load combination

$Mq\_SLSg1(x) := (g1) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from self-weight load

$Mq\_SLSg2(x) := (g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from nonstructural dead load

$Mq\_SLSq(x) := (q \cdot \psi 2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ SLS bending moment distribution from live load

14.6 Time-dependent behaviour

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$$hn := 2 \cdot \frac{Ac}{u} = 240.643$$

$$RH := 50$$

$$t0\_adj(t0) := t0$$

$$\beta bc\_fcm := \frac{1.8}{(-fcm)^{0.7}} = 0.112 \qquad \beta bc\_t\_t0(t,t0) := \ln\left[\left(\frac{30}{t0\_adj(t0)} + 0.035\right)^2 \cdot (t - t0) + 1\right]$$

$$\beta dc\_fcm := \frac{412}{(-fcm)^{1.4}} = 1.588$$

$$\beta dc\_RH := \frac{1 - \frac{RH}{100}}{\sqrt[3]{0.1 \cdot \frac{hn}{100}}} = 0.804$$

$$\beta dc\_t0(t0) := \frac{1}{0.1 + t0\_adj(t0)^{0.2}}$$

$$\gamma(t0) := \frac{1}{2.3 + \frac{3.5}{\sqrt{t0\_adj(t0)}}}$$

$$\alpha cm := \left(\frac{35}{-fcm}\right)^{0.5} = 0.813$$

$$\beta h := \min(1.5 \cdot hn + 250 \cdot \alpha cm, 1500 \cdot \alpha cm) = 564.124$$

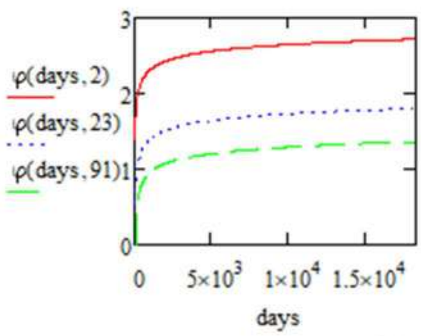
$$\beta dc\_t\_t0(t,t0) := \left[\frac{(t - t0)}{\beta h + (t - t0)}\right]^{\gamma(t0)}$$

$$\varphi dc(t,t0) := \beta dc\_fcm \cdot \beta dc\_RH \cdot \beta dc\_t0(t0) \cdot \beta dc\_t\_t0(t,t0)$$

$$\varphi bc(t,t0) := \beta bc\_fcm \cdot \beta bc\_t\_t0(t,t0)$$

$$\varphi(t,t0) := \varphi bc(t,t0) + \varphi dc(t,t0)$$

$$t := 50 \cdot 365 = 1.825 \times 10^4 \qquad \varphi(t,2) = 2.718 \qquad \varphi(t,91) = 1.363$$



14.7 Non-linear moment-curvature diagram

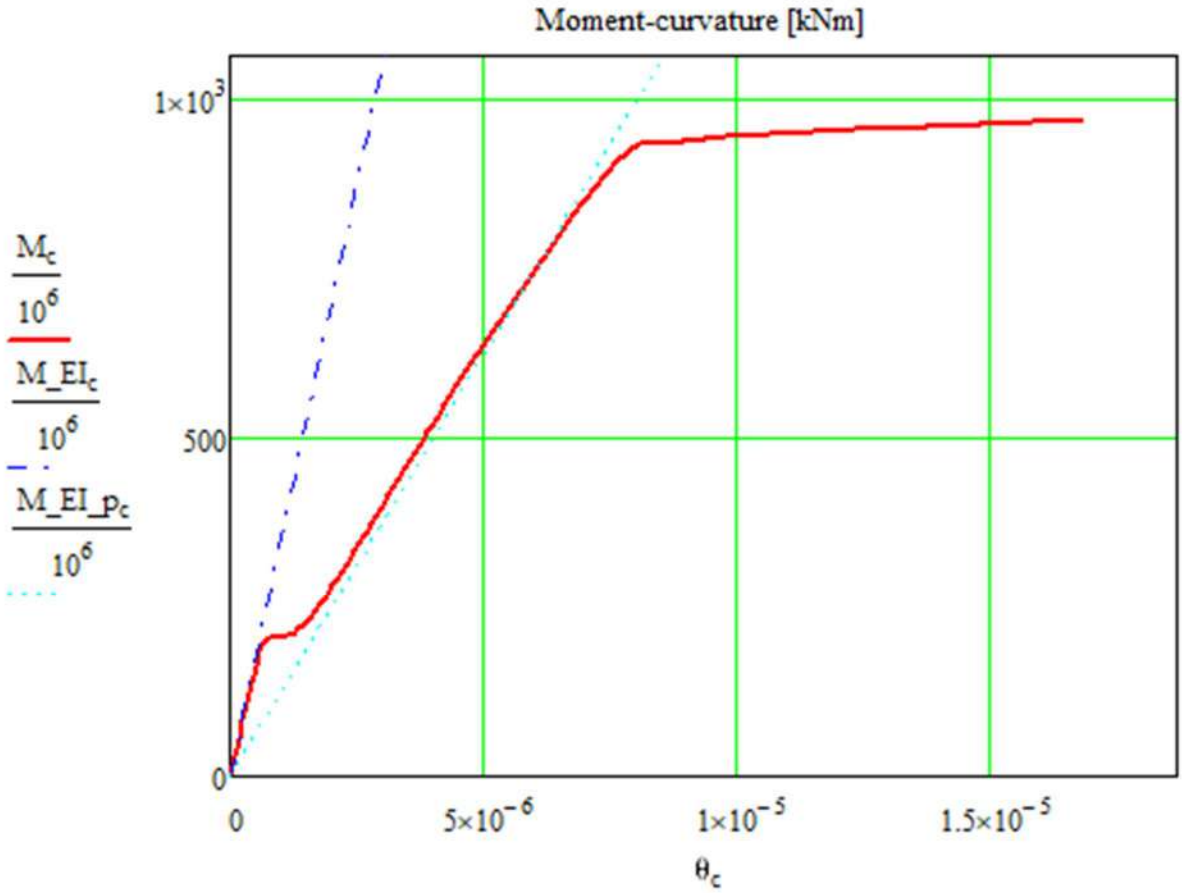
Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

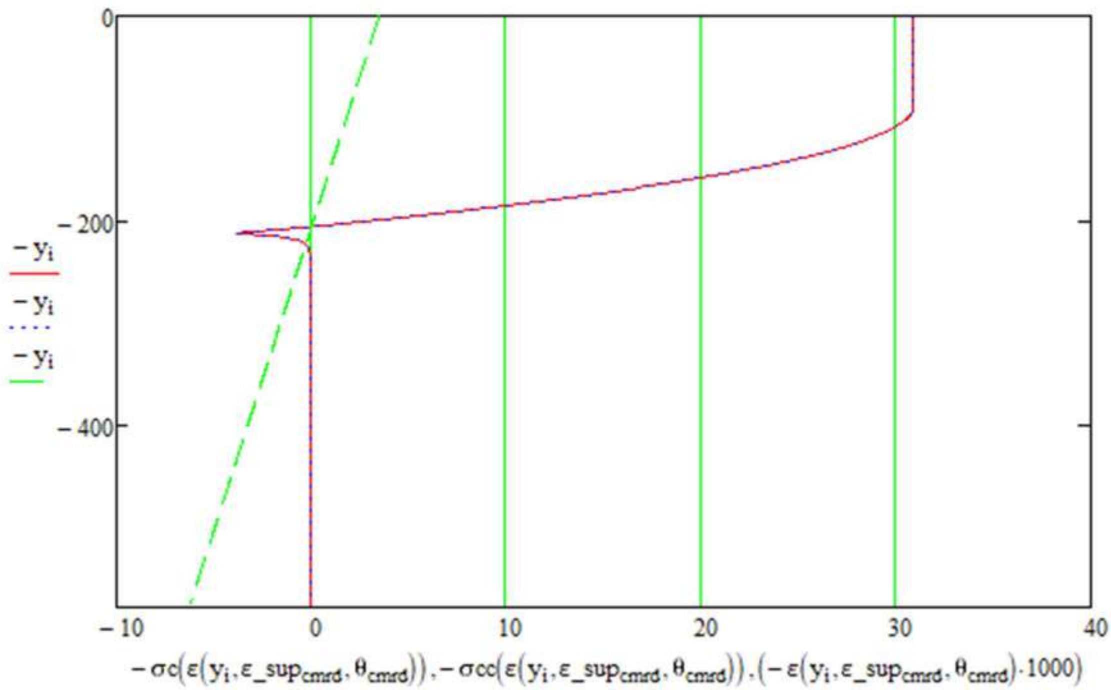
$$N(\varepsilon\_sup, \theta) := \sum_{i=1}^{Htot} \left(\sigma c\left(\varepsilon\left(y_i, \varepsilon\_sup, \theta\right)\right) \cdot b\left(y_i\right) \cdot \Delta y\right) + \sum_{j=1}^{js} \left(\sigma s\left(\varepsilon\left(ds_j, \varepsilon\_sup, \theta\right)\right) \cdot As_j\right)$$

$$M(\varepsilon\_sup, \theta) := \sum_{i=1}^{Htot} \left[\sigma c\left(\varepsilon\left(y_i, \varepsilon\_sup, \theta\right)\right) \cdot b\left(y_i\right) \cdot \Delta y \cdot \left(y_i - yG\right)\right] + \sum_{j=1}^{js} \left[\sigma s\left(\varepsilon\left(ds_j, \varepsilon\_sup, \theta\right)\right) \cdot As_j \cdot \left(ds_j - yG\right)\right]$$

Design external axial load

$NS := -0$





Condition at resisting (peak) moment
(stress and strain)

14.8 Bending moment distribution

$\gamma g1 := 1.35$ partial safety coefficient for self-weight structural loads

$\gamma g2 := 1.35$ partial safety coefficient for non-structural certain dead loads

$\gamma q := 1.5$ partial safety coefficient for live loads or non-structural uncertain dead loads

$Mq\_ULS(x) := (g1 \cdot \gamma g1 + g2 \cdot \gamma g2 + q \cdot \gamma q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Ultimate Limit State (ULS) fundamental load combination following a uniformally distributed load q

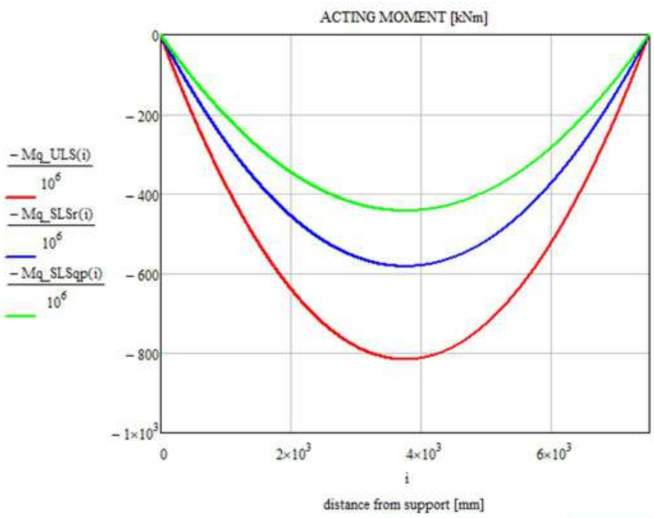
$Mq\_SLSr(x) := (g1 + g2 + q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) rare load combination following a uniformally distributed load q

$Mq\_SLSf(x) := (g1 + g2 + \psi 1 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) frequent load combination following a uniformally distributed load q

$Mq\_SLSqp(x) := (g1 + g2 + \psi 2 \cdot q) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) quasi permanent load combination following a uniformally distributed load q

$Mq\_SLSg2(x) := (g1 + g2) \cdot \left(\frac{L}{2} \cdot x - \frac{x^2}{2}\right)$ moment distribution at Serviceability Limit State (SLS) permanent load combination following a uniformally distributed load q

$i := 0..L$



14.9 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:

$$v\_inf\_p(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(1 + \varphi(t,23))}{1.05}$$

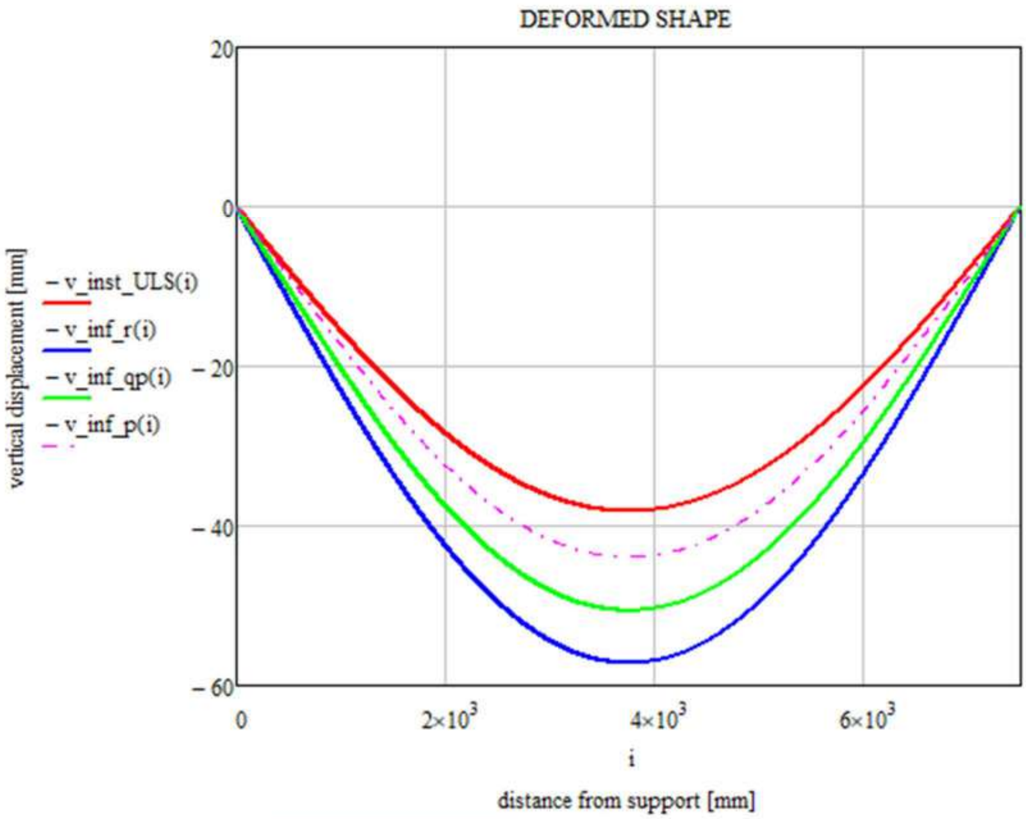
deflection profile at 50 years including creep for permanent load combination

$$v\_inf\_qp(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(\varphi(t,23) - \varphi(t,91)) + v\_SLSqp(x)\cdot(1 + \varphi(t,91))}{1.05}$$

deflection profile at 50 years including creep for quasi permanent load combination

$$v\_inf\_r(x) := \frac{v\_SLSg1(x)\cdot(\varphi(t,2) - \varphi(t,23)) + v\_SLSg2(x)\cdot(\varphi(t,23) - \varphi(t,91)) + v\_SLSqp(x)\cdot\varphi(t,91) + v\_SLSr(x)}{1.05}$$

deflection profile at 50 years including creep for rare load combination



SLS DEFLECTION CONTROL - RIGOROUS METHOD (§9.3.4)

camber := 30 mm < $\frac{L}{250} = 30$ CHECK maximum deflection

made by shaping the mould

$v\_inf\_r\left(\frac{L}{2}\right) - camber = 27.222$ < $\frac{L}{250} = 30$ CHECK maximum camber

SLS STRESS CONTROL (§9.2.1)

$k1 := 0.6$

$k2 := 0.45$

$k3 := 0.8$

$k4 := 1$

$k5 := 0.8$ 0.75 in EN1992-1-1:2002

NOTE: the denomination of the allowable stress coefficients following k factors was kept similar to that of EN1992-1-1:2002

$$\sigma cpf\_bot(x) := \frac{Mq\_SLSf(x)\cdot(Htot - Yid)}{Ixo\_id} \qquad \sigma cpf\_bot\left(\frac{L}{2}\right) = 11.$$

elastic stress of bottom concrete chord for selfweight loads only

$$\sigma sf\_bot(x) := 15\cdot\left[\frac{Mq\_SLSf(x)\cdot\left(ds_{js} - Yid\right)}{Ixo\_id}\right] \qquad \sigma sf\_bot\left(\frac{L}{2}\right) = 138.$$

elastic stress of top concrete chord for selfweight loads only

$$\sigma cpr\_bot(x) := \frac{Mq\_SLSr(x)\cdot(Htot - Yid)}{Ixo\_id} \qquad \sigma cpr\_bot\left(\frac{L}{2}\right) = 13.$$

elastic stress of top series of mild steel for selfweight loads only

$$\sigma cpr\_top(x) := \frac{Mq\_SLSr(x)\cdot(-Yid)}{Ixo\_id} \qquad \sigma cpr\_top\left(\frac{L}{2}\right) = -2($$

$$\sigma cpr\_s(x) := 15\cdot\left[\frac{Mq\_SLSr(x)\cdot\left(ds_{js} - Yid\right)}{Ixo\_id}\right] \qquad \sigma cpr\_s\left(\frac{L}{2}\right) = 167.2$$

$\sigma cpf\_bot\left(\frac{L}{2}\right) = 11.31$ < $fctm = 3.795$ CHECK

if not -> cracked

$\sigma sf\_bot\left(\frac{L}{2}\right) = 138.63$

$\sigma cpr\_bot\left(\frac{L}{2}\right) = 13.644$ < $fctm = 3.795$

$\sigma cpr\_top\left(\frac{L}{2}\right) = -20.813$ > $k1\cdot fck = -27$ CHECK

> $0.4\cdot fcm = -21.2$

$\sigma cpr\_s\left(\frac{L}{2}\right) = 167.239$ < $k3\cdot fsk = 400$ CHECK

SLS CRACK CONTROL (§9.2.3)

$$c\_act := Htot - ds_{js} - 10 = 32$$

$$ksurf := \min\left(1.5, \frac{c\_act}{10 + cmin\_dur\_s}\right) = 1.5$$

$$wlim\_cal := 0.2 \cdot ksurf = 0.3 \quad \text{mm}$$

$$kw := 1.7$$

$$ayi := Htot - ds_{js} = 42 \qquad \phi := 24$$

$$k1\_r := \frac{Htot - Yn\_n}{Htot - ayi - Yn\_n} = 1.121$$

$$kfl := \frac{Htot - \min(ayi + 5 \cdot \phi, ayi \cdot 3.5)}{Htot} = 0.747$$

$$kb := 1.2$$

$$Aceff := 0.5 \cdot b(Htot) \cdot \min(ayi + 5 \cdot \phi, ayi \cdot 3.5) = 5.88 \times 10^{4}$$

$$\rho peff := \frac{As_{js} + As_{js-1}}{Aceff} = 0.077$$

$$srmcal := \min\left[1.5 \cdot \left(Htot - ds_{js} + \frac{\phi}{2}\right) + \frac{kfl \cdot kb}{7.2} \cdot \frac{\phi}{\rho peff}, \frac{1.3}{kw} \cdot (Htot - Yn\_n)\right] = 119.814$$

$kt := 0.4$ NOTE : 0.6 for sustained loading

$$fcteff := fctm = 3.795$$

$$\varepsilon sm\_\varepsilon cm := \max\left[\frac{\sigma sf\_bot\left(\frac{L}{2}\right) - kt \cdot \frac{fcteff}{\rho peff} \cdot \left(1 + \frac{Es}{Ecm} \cdot \rho peff\right)}{Es}, (1 - kt) \cdot \frac{\sigma sf\_bot\left(\frac{L}{2}\right)}{Es}\right] = 5.519 \times 10^{-4}$$

$wkcal := kw \cdot k1\_r \cdot srmcal \cdot \varepsilon sm\_\varepsilon cm = 0.126$ < $wlim\_cal = 0.3$ CHECK

14.10 ULS checks

ULS BENDING-AXIAL CONTROL (§8.1)

$\text{Mrd} = 967.352$ kNm $\quad > \quad \dfrac{\text{Mq\_ULS}\left(\frac{L}{2}\right)}{10^6} = 816.449$ CHECK

resisting moment calculated from moment-curvature diagram above

ULS SHEAR CONTROL (§8.2)

$\text{Vq\_ULS}(x) := \left|(g1\cdot\gamma g1 + g2\cdot\gamma g2 + q\cdot\gamma q)\cdot\left(\frac{L}{2} - x\right)\right|$ — shear action distribution at Ultimate Limit State (ULS)

$d := ds_{js} = 538$ mm — effective depth of cross-section

$\text{VEd} := \text{Vq\_ULS}(d) = 3.73 \times 10^5$ N — design shear action at control section at distance d from support

$\gamma v := 1.3$ — safety factor for initial shear check

$bw := 400$ mm — design web width

$z := 0.9\cdot d = 484.2$ — conventional lever arm of internal stress resultants

$\tau \text{Ed} := \dfrac{\text{VEd}}{bw\cdot z} = 1.926$ MPa — equivalent mean acting shear stress on control cross-section

$\text{Dlower} := 16$ mm — maximum aggregate diameter following assumed mix design

$ddg := \min\left[\text{if}\left[-fck > 60, 16 + \text{Dlower}\cdot\left(\frac{60}{-fck}\right)^2, 16 + \text{Dlower}\right], 40\right] = 32$ — size parameter

MEMBERS NOT PROVIDED WITH SHEAR REINFORCEMENT (§8.2.2)

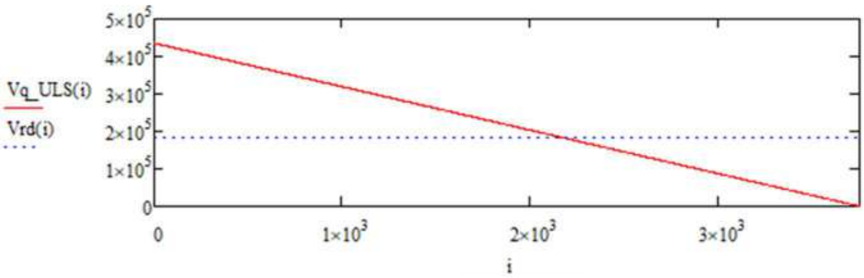
$\tau Rdc\_min := \frac{11}{\gamma v} \cdot \sqrt{\frac{-fck}{fsd} \cdot \frac{ddg}{d}} = 0.649$ MPa §(8.20) not checked with τEd -> detailed evaluation is mandatory following §8.2.1

$\rho l := \frac{\sum_{j=1}^{js} As_j}{bw \cdot d} = 0.023$ longitudinal geometric reinforcement ratio §(8.28)

$\tau Rdc\_0 := \frac{0.66}{\gamma v} \cdot \left(100 \cdot \rho l \cdot -fck \cdot \frac{ddg}{d}\right)^{\frac{1}{3}} = 0.937$ MPa §(8.33) AT SUPPORTS WHERE PRESTRESSING IS NOT ACTIVE

$\tau Rdc := \max(\tau Rdc\_0, \tau Rdc\_min) = 0.937$ MPa §(8.32)

$Vrd(x) := bw \cdot z \cdot \tau Rdc\_0$



$bw \cdot z \cdot \tau Rdc = 1.814 \times 10^5$

MEMBERS PROVIDED WITH SHEAR REINFORCEMENT (§8.2.3)

$\theta v := \operatorname{atan}(0.5) = 0.464$ angle of inclination of concrete compressed strut

$\nu := 0.5$ NOTE: steel grade B500A is used

$\sigma cd := \tau Ed \cdot (\cot(\theta v) + \tan(\theta v)) = 4.814$ MPa $<$ $\nu \cdot -fcd = 15.453$ MPa CHECK §(8.44)

$fywd := fsd = 454.545$ MPa design yield stress of shear reinforcement steel

$Asw := 2 \cdot \frac{8^2 \cdot \pi}{4} = 100.531$ area of transverse shear reinforcement

$s1 := 80$ mm spacing of transverse reinforcement (field near the supports)

$x1 := 1600$ mm end of field 1 from the support

$\tau Rd\_sy1 := \frac{Asw}{bw \cdot s1} \cdot fywd \cdot \cot(\theta v) = 2.856$ MPa $>$ $\tau Ed = 1.926$ MPa CHECK §(8.42)

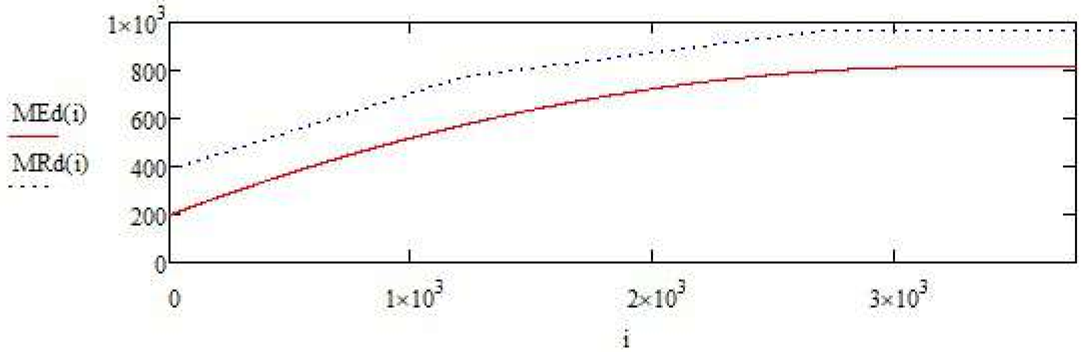
MOMENT DIAGRAM ACCOUNTING FOR THE SHEAR RESISTING MECHANISM (§12.3.2)

$\alpha 3 := 1$ fatigue check not required

$$al := z \cdot \left(\frac{\cot(\theta v)}{2} \right) = 484.2$$

$$MEd(x) := \text{if}\left(x > \frac{L}{2} - al, \frac{Mq\_ULS\left(\frac{L}{2}\right)}{10^6}, \frac{Mq\_ULS(x + \text{round}(z))}{10^6} \right)$$

$$MRd(x) := \text{if}\left(x < 1220, \frac{4}{10} \cdot Mrd + \frac{4}{10} \cdot Mrd \cdot \frac{x}{1220}, \text{if}\left(x < 2720, \frac{8}{10} \cdot Mrd + \frac{2}{10} \cdot Mrd \cdot \frac{x - 1220}{2720 - 1220}, Mrd \right) \right)$$



MINIMUM REINFORCEMENT (§12.2)

$kh := if[0.8 - 0.6\cdot(min(bw, Htot) - 0.3) < 0.5, 0.5, if[0.8 - 0.6\cdot(min(bw, Htot) - 0.3) > 0.8, 0.8, 0.8 - 0.6\cdot(min(bw, Htot) - 0.3)]] = 0.5$ §9.2.2(2)

$fct\_eff := fctm$

$As\_min\_w1 := 0.2\cdot kh\cdot fct\_eff\cdot\frac{Ac}{fsk} = 252.084$ mm2 $As\_tot = 5.052\times10^{3}$ mm2 CHECK §(9.2)

$fctm\cdot\frac{Ixo\_id}{(Htot - Yid)\cdot10^{6}} = 162.068 < Mrd = 967.352$ kNm CHECK §(12.1)

$s3 := 240 < 0.75\cdot(Htot - 30) = 412.5$ CHECK §12.1

$\rho w\_min := \frac{Asw}{s3\cdot bw} = 1.047\times10^{-3} > 0.08\cdot\frac{\sqrt{fck}}{fsk} = 1.073\times10^{-3}$ CHECK §(12.4)

CHECK OF STIRRUPS FOR SUSPENSION LOAD

$\frac{\tau Ed}{cot(\theta v)}\cdot bw + \left(\frac{g1}{3}\cdot1.35 + g2\cdot1.35 + q\cdot1.5\right) = 493.784$ kN / m $<$ $\frac{Asw\cdot fywd}{s1} = 571.199$ kN / m CHECK

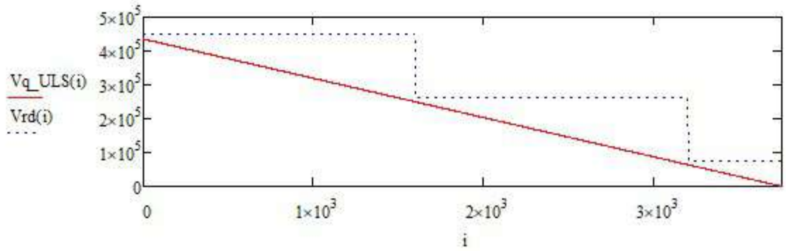
$\frac{g1}{3}\cdot1.35 + g2\cdot1.35 + q\cdot1.5 = 108.645$ kN / m $<$ $\frac{Asw\cdot fywd}{s3} = 190.4$ kN / m CHECK

$s2 := 120$ mm spacing of stirrups within field 2

$x2 := 3200$ mm end of field 2 for stirrups

$\tau Rd\_sy2 := \frac{Asw}{bw\cdot s2}\cdot fywd\cdot cot(\theta v) = 1.904$

$Vrd(x) := if(x < x1, \tau Rd\_sy1\cdot bw\cdot z, if(x < x2, \tau Rd\_sy2\cdot bw\cdot z, Vrd(x))) - \left(\frac{g1}{3}\cdot1.35 + g2\cdot1.35 + q\cdot1.5\right)\cdot cot(\theta v)\cdot z$ INCLUDING THE EFFECT OF SUSPENSION

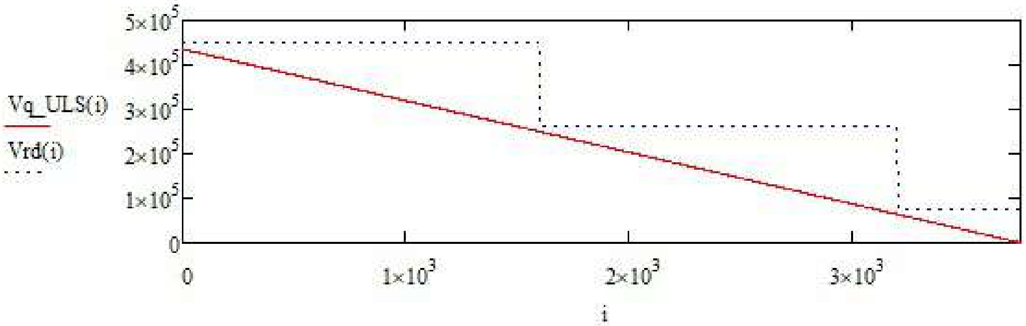


$s1 = 80$

$s2 = 120$

$s3 = 240$

$\frac{Vq\_ULS(1600)}{Vrd(1600)} = 0.947$



$s1 = 80$

$s2 = 12$

$s3 = 24$

CHECK OF HORIZONTAL SADDLE BAR

$$\frac{\left(\frac{g1}{3}\cdot 1.35 + g2\cdot 1.35 + q\cdot 1.5\right)}{2}\cdot 100\cdot s2 = 6.519\times 10^{5} \quad \text{Nmm} \quad < \quad \pi\cdot\frac{8^2}{4}\cdot fsd\cdot 0.9\cdot 220 = 4.524\times 10^{6}$$ CHECK

area of horizontal transverse reinforcement

CHECK OF SUPPORT MILD REBARS

$MEd(0) = 197.152$ Nmm

$0.25\cdot MEd\left(\frac{L}{2}\right) = 204.112$ Nmm §12.1(4)

$$< \left(3\cdot\pi\cdot\frac{16^2}{4} + 4\cdot\pi\cdot\frac{12^2}{4}\right)\cdot fsd\cdot 0.9\cdot\frac{540}{10^6} = 233.186$$ CHECK

area of support mild steel

ANCHORAGE (§11.4)

$klb := 50$

$kcp := 1$ for good bond conditions

$n\sigma := \frac{3}{2}$

$cs := 50$

$cx := 75$

$cy := 40$

$cd(\phi) := \min(0.5\cdot cs, cx, cy, 3.75\cdot\phi)$ $\quad cd(12) = 25$

$$lbd(\phi) := \max\left[klb\cdot kcp\cdot\phi\cdot\left(\frac{fsd}{435}\right)^{n\sigma}\cdot\left(\frac{25}{-fck}\right)^{\frac{1}{2}}\cdot\left(\frac{\phi}{20}\right)^{\frac{1}{3}}\cdot\left(\frac{1.5\cdot\phi}{cd(\phi)}\right)^{\frac{1}{2}}, 10\cdot\phi\right]$$

$lbd(24) = 1.218\times 10^{3}$ $\quad \frac{lbd(12)}{12} = 28.489$

length of straight part for 90° bent bars

$lb90(\phi) := \max(70, lbd(\phi) - 15\cdot\phi, 10\cdot\phi)$

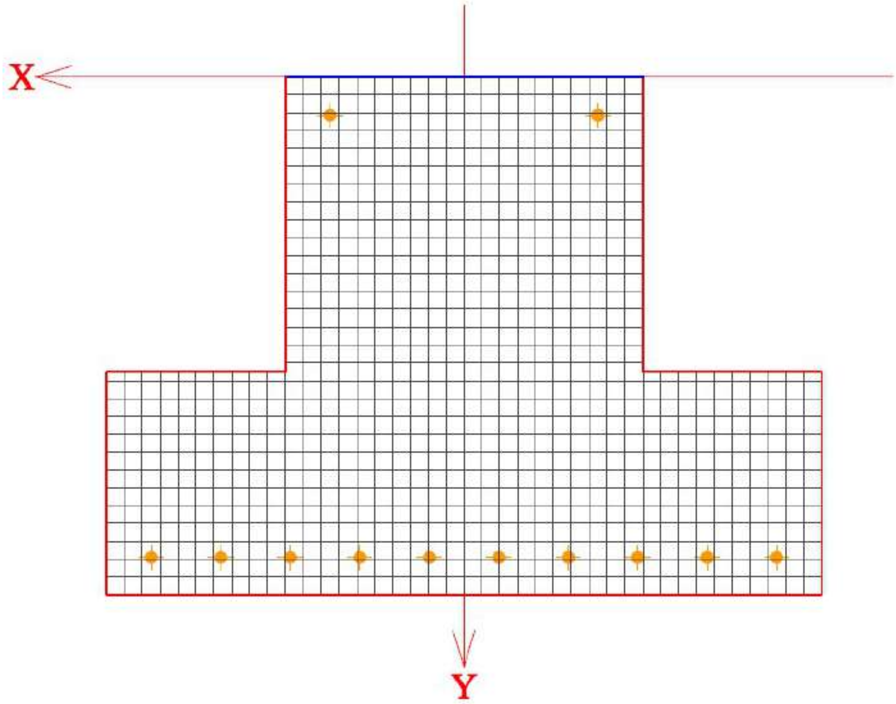
$lb90(12) = 161.872 \quad lb90(8) = 80$

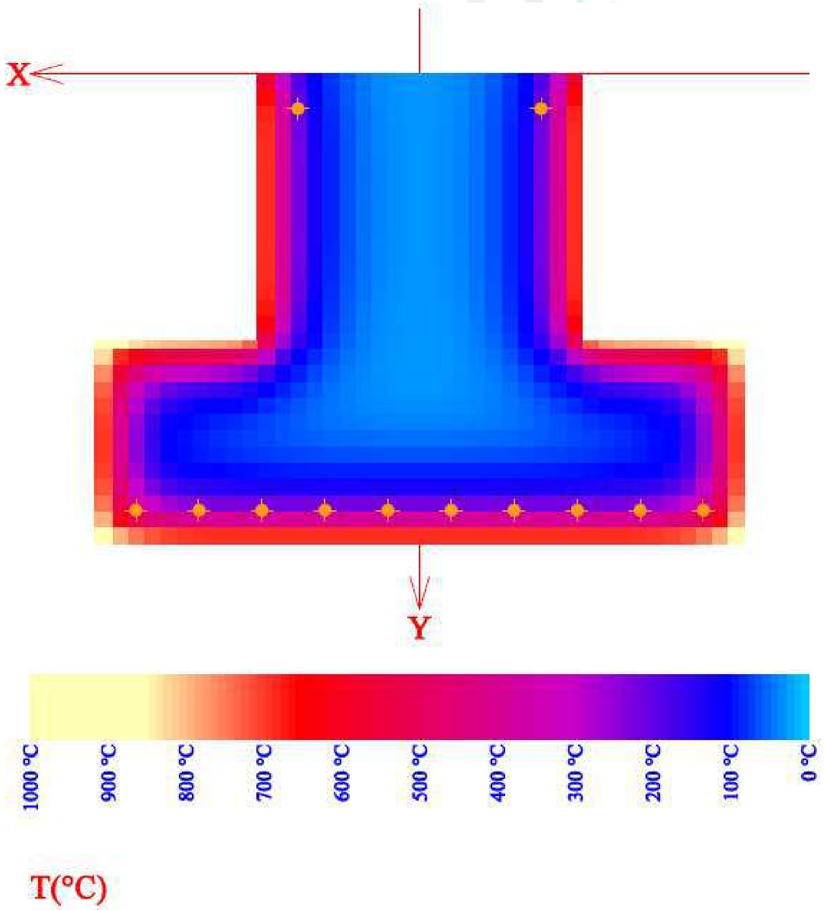
length of straight part for 135° bent bars (stirrups)

$lb135(\phi) := \max(50, lbd(\phi) - 15\cdot\phi, 5\cdot\phi)$

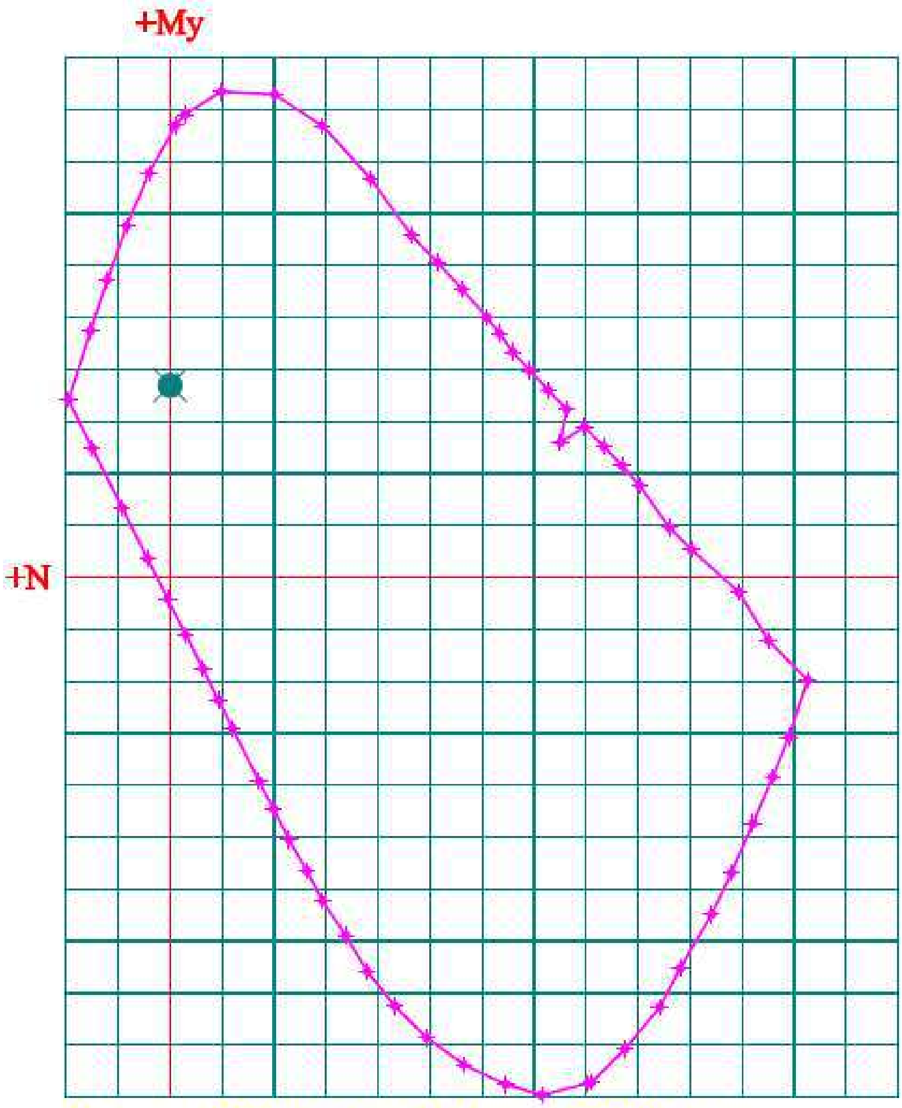
$lb135(12) = 161.872 \quad lb135(8) = 50$

14.11 Fire checks





[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]										
26	26	28	33	42	57	78	101	196	359	655										
26	26	28	34	43	58	80	114	205	377	676										
26	26	29	34	44	59	82	118	210	384	682										
26	26	29	34	44	60	83	120	212	386	684										
26	26	29	35	44	60	83	121	212	387	684										
26	26	29	35	45	60	83	121	213	387	685										
26	26	29	35	45	60	83	121	213	388	685										
26	26	29	35	45	60	83	121	213	387	685										
26	26	29	35	45	60	83	121	213	387	685										
26	26	29	34	44	60	83	121	212	387	684										
26	26	29	34	44	59	82	119	211	386	684										
26	26	28	34	43	59	81	117	209	383	682										
26	26	28	33	43	57	79	111	205	377	678										
25	25	27	32	41	56	77	100	196	363	665										
25	25	27	31	39	53	73	100	178	329	626										
24	24	26	30	37	49	67	97	149	262	500	726	768	780	786	790	795	805	823	853	903
24	24	26	29	35	44	59	80	100	175	290	414	469	493	505	515	530	555	601	683	823
25	25	26	28	33	40	51	66	86	113	165	217	252	274	289	303	324	361	428	548	760
26	26	27	29	32	37	45	56	69	86	100	129	149	161	170	182	206	253	332	473	724
29	29	30	31	33	37	42	49	58	69	80	92	100	100	100	119	145	192	279	432	705
35	35	35	36	38	40	43	48	54	60	67	74	81	86	92	100	105	163	251	412	696
45	45	45	46	47	48	50	53	56	60	65	69	74	79	86	94	100	155	241	404	692
60	60	60	61	61	62	63	65	67	69	72	75	79	83	89	97	100	158	245	407	693
84	84	84	84	84	85	85	86	87	89	90	93	96	100	100	100	129	175	264	421	700
122	122	122	122	122	122	123	123	124	125	127	129	132	136	143	153	174	220	303	451	714
213	213	213	213	213	213	213	213	214	214	215	216	217	220	226	238	261	303	376	507	741
388	388	388	388	388	388	388	388	388	388	388	389	389	391	395	403	419	450	507	609	789
685	685	685	685	685	685	685	685	685	685	685	685	685	686	688	692	699	713	741	789	871

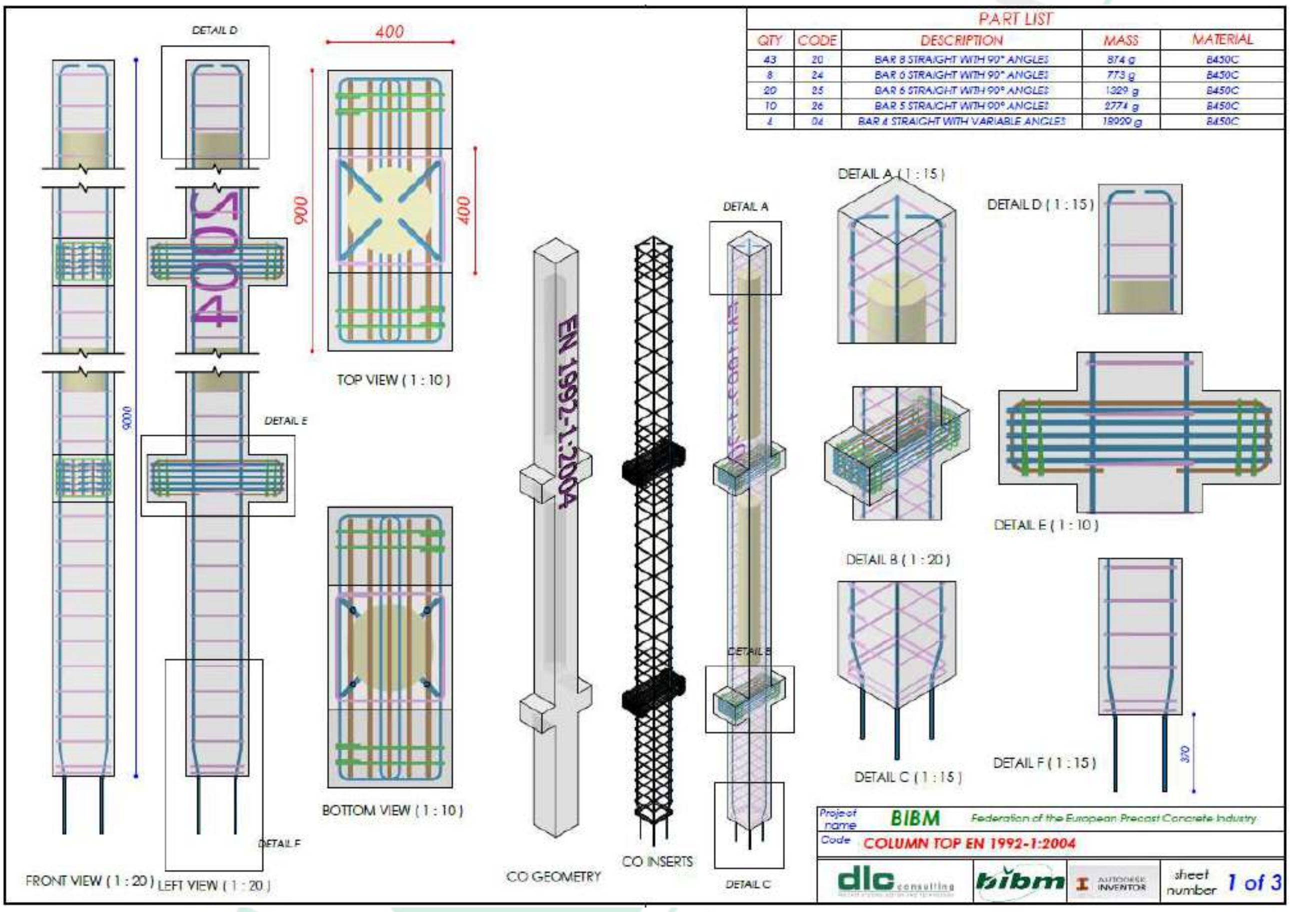


N: 1 sp = 1200.00 kN; M: 1 sp = 120.00 kN·m

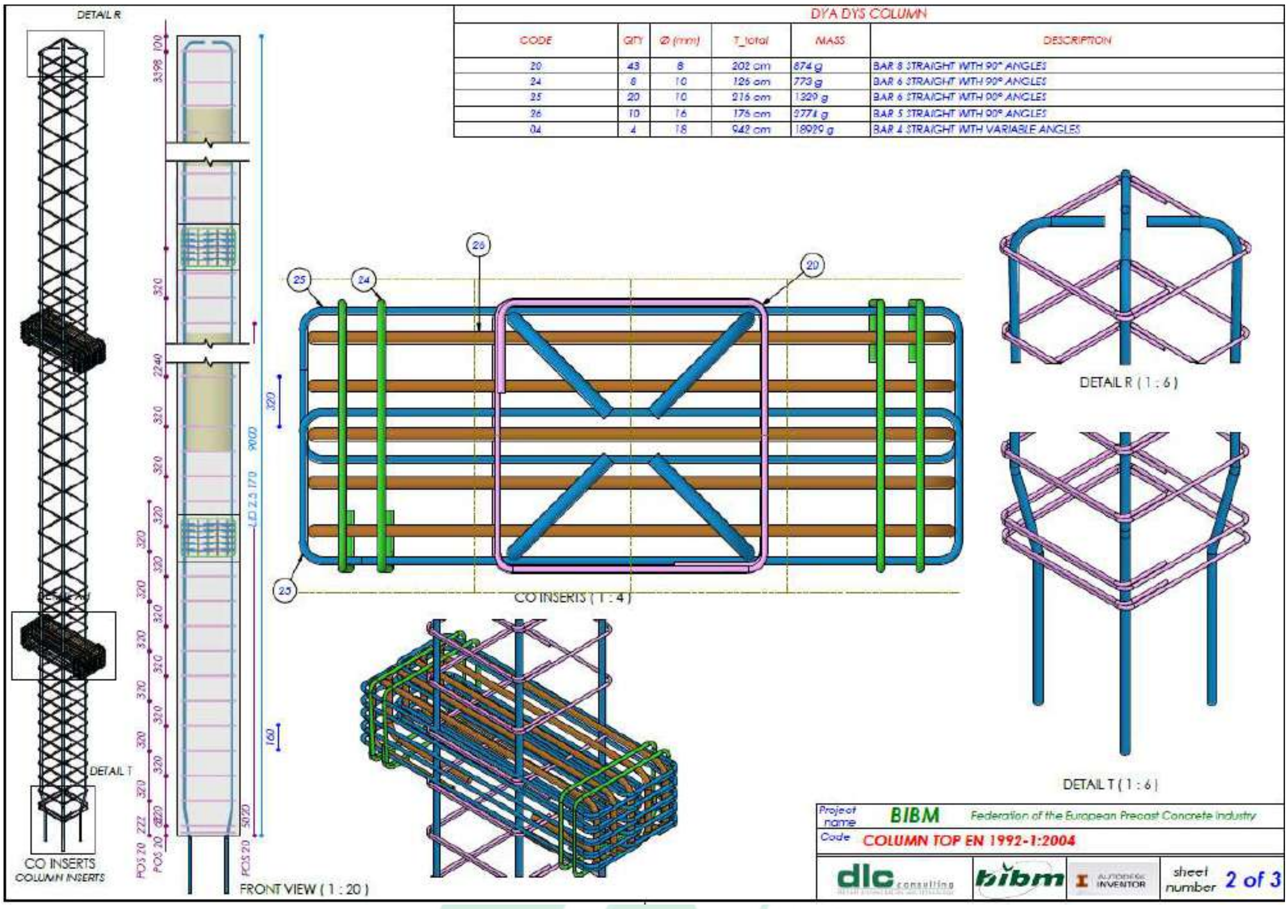
Nz = 0.00 kN
My+ = 1020.71 kN·m
My- = -61.16 kN·m

15 Column element –EN1992-1:2004

15.1 Shop drawings

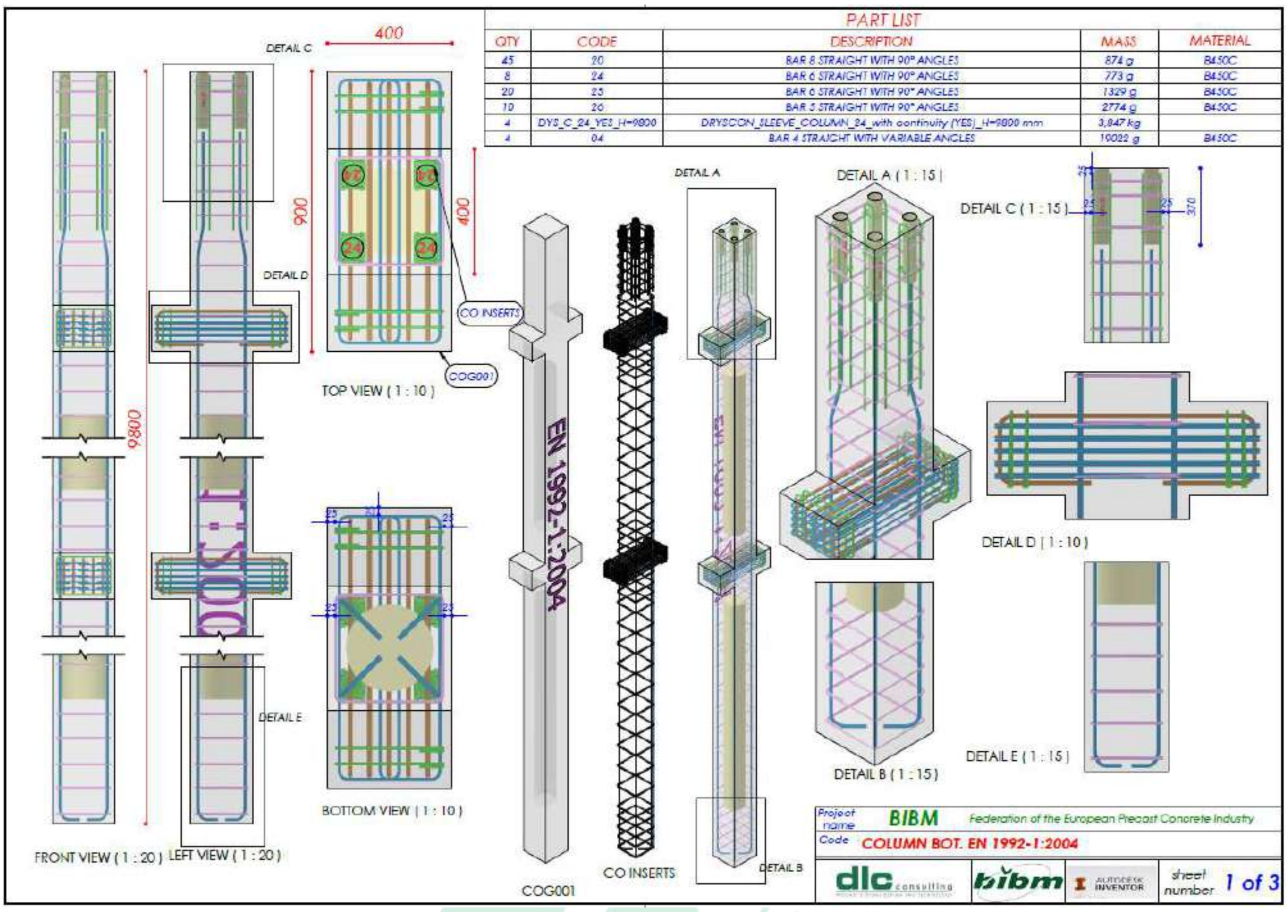


DYA DYS COLUMN					
CODE	QTY	Ø (mm)	T_total	MASS	DESCRIPTION
20	43	8	202 cm	874 g	BAR 8 STRAIGHT WITH 90° ANGLES
24	8	10	126 cm	773 g	BAR 6 STRAIGHT WITH 90° ANGLES
25	20	10	216 cm	1329 g	BAR 6 STRAIGHT WITH 90° ANGLES
26	10	16	176 cm	2774 g	BAR 5 STRAIGHT WITH 90° ANGLES
04	4	18	942 cm	18929 g	BAR 4 STRAIGHT WITH VARIABLE ANGLES



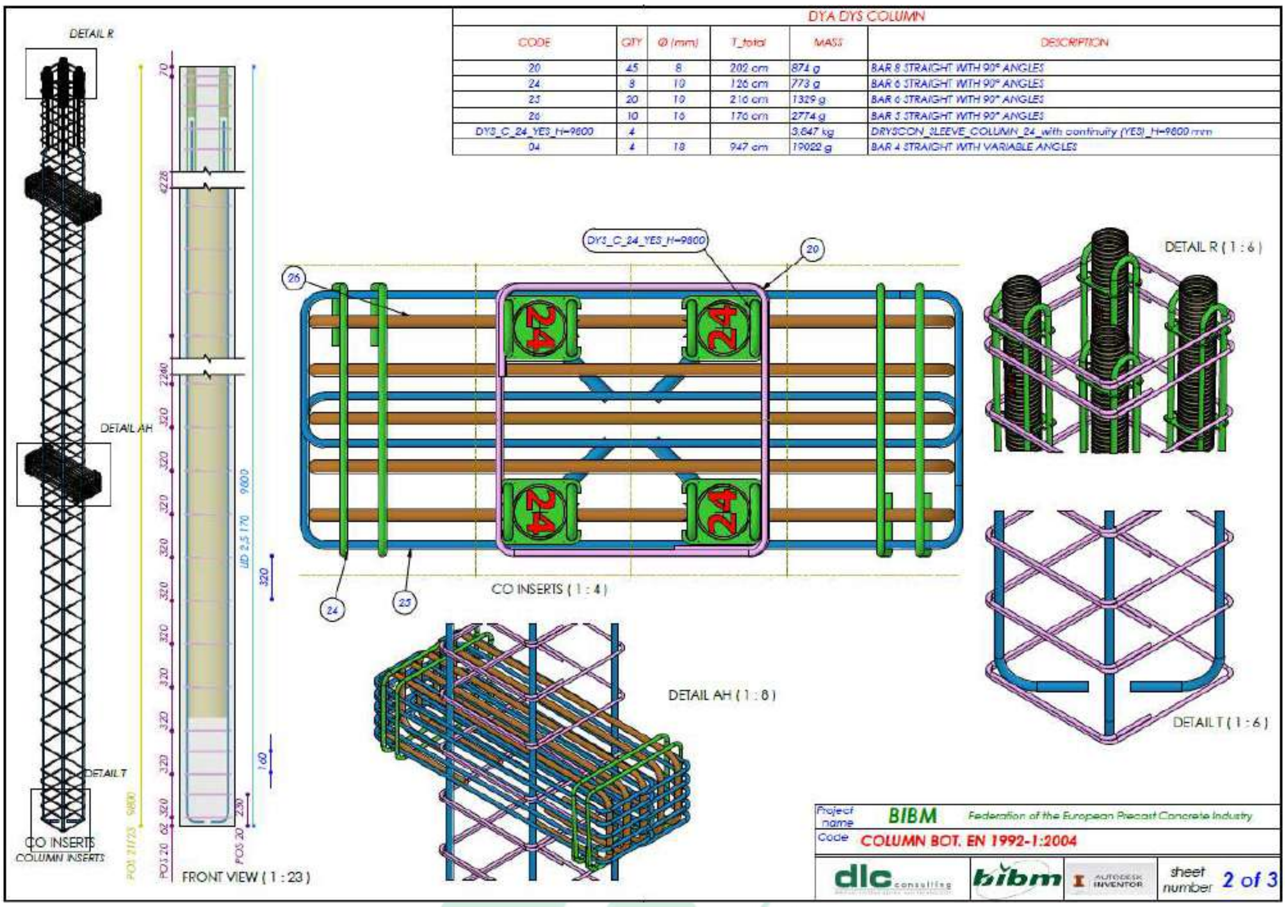
Thumbnail	Part Number	QTY	Mass	Total Mass	Ø_			
	04	4	18929	75716	18 mm			
	20	43	874	37582	8 mm			
	24	8	773	6184	10 mm			
	25	20	1329	26580	10 mm			
	26	10	2774	27740	16 mm			
Total mass rebars [kg]				173,80	Incidence kg/m³	138,82		
	lightening 1	1	7697	7697				
	lightening 2	1	7697	7697				
Total mass lightening [kg]				15,39	Incidence kg/m³	12,30		
	Total mass of steel [kg]			173,80	Total concrete volume [m³]		1,252	

Project name BIBM Federation of the European Precast Concrete Industry
Code COLUMN TOP EN 1992-1:2004
sheet number 3 of 3



PART LIST

QTY	CODE	DESCRIPTION	MASS	MATERIAL
45	20	BAR 8 STRAIGHT WITH 90° ANGLES	874 g	B450C
8	24	BAR 6 STRAIGHT WITH 90° ANGLES	773 g	B450C
20	25	BAR 6 STRAIGHT WITH 90° ANGLES	1329 g	B450C
10	26	BAR 5 STRAIGHT WITH 90° ANGLES	2774 g	B450C
4	DYS_C_24_YES_H=9800	DRYSCON_SLEEVE_COLUMN_24_with continuity (YES)_H=9800 mm	3,847 kg	
4	04	BAR 4 STRAIGHT WITH VARIABLE ANGLES	19022 g	B450C



DYA DYS COLUMN					
CODE	QTY	Ø (mm)	T_total	MASS	DESCRIPTION
20	45	8	202 cm	874 g	BAR 8 STRAIGHT WITH 90° ANGLES
24	8	10	126 cm	773 g	BAR 6 STRAIGHT WITH 90° ANGLES
25	20	10	216 cm	1329 g	BAR 6 STRAIGHT WITH 90° ANGLES
26	10	16	176 cm	2774 g	BAR 5 STRAIGHT WITH 90° ANGLES
DYS_C_24_YES_H=9800	4			3,647 kg	DRYSCON_SLEEVE_COLUMN_24_with continuity (YES)_H=9800 mm
04	4	18	947 cm	19022 g	BAR 4 STRAIGHT WITH VARIABLE ANGLES

Thumbnail	Part Number	QTY	Mass	Total mass	Ø_	Ø_longitudinal	pattern_T	Ø_transverse
	04	4	19022	76088	18 mm			
	20	45	874	39330	8 mm			
	24	8	773	6184	10 mm			
	25	20	1329	26580	10 mm			
	26	10	2774	27740	16 mm			
	DYS_C_24	4	3847	15388				
	Total mass rebars [kg]			191,31	Incidence kg/m³		143,09	
	lightening 1	1	7697	7697				
	lightening 2	1	9852	9852				
	Total mass lightening [kg]			17,55	Incidence kg/m³		13,13	
	Total mass of steel [kg]			191,31		Total concrete volume [m³]	1,337	

VIEW 109 (1 :40)

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sheet number 3 of 3

15.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$$y\_tr := (0\ \ 400)^T$$

$$Htot := \max(y\_tr)$$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$$b\_tr := (400\ \ 400)^T$$

$r\_circ := 140$ radius of central void pipe

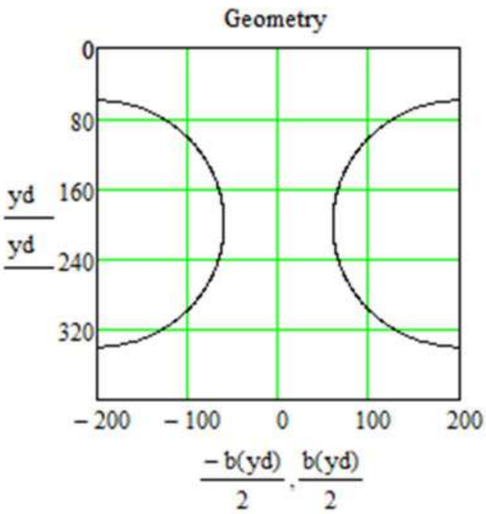
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$$b\_lin(y) := linterp(y\_tr, b\_tr, y)$$

$$b\_circ(y) := linterp(y\_tr, b\_tr, y) - x\_circ(y)$$

$$b(y) := if\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$$yd := 0..Htot$$



condensed 1D geometry plot

$u := 800 \cdot 2 + Htot \cdot 2 = 2.4 \times 10^3$ exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (40 \ \ 200 \ \ 360)^T$$

Area of reinforcement at each depth

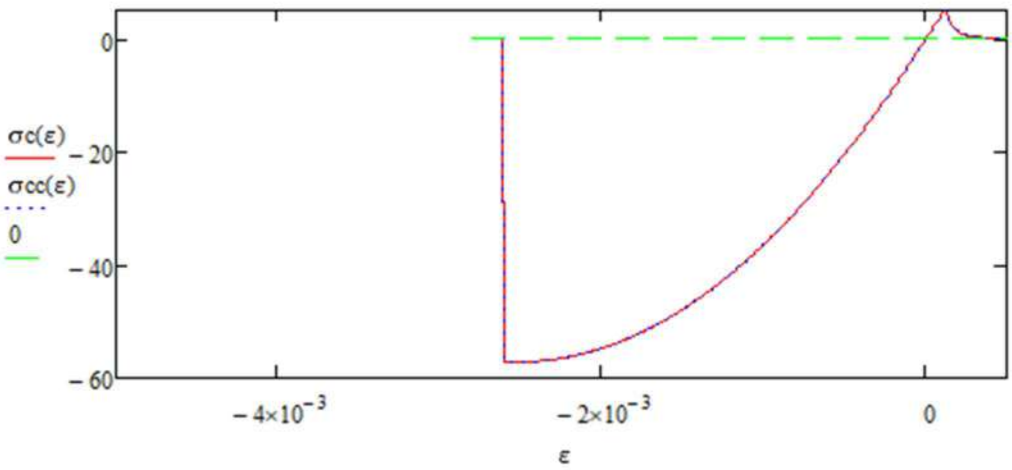
$$As := (2 \cdot A(18) \ \ 0 \cdot A(16) \ \ 2 \cdot A(18))^T$$

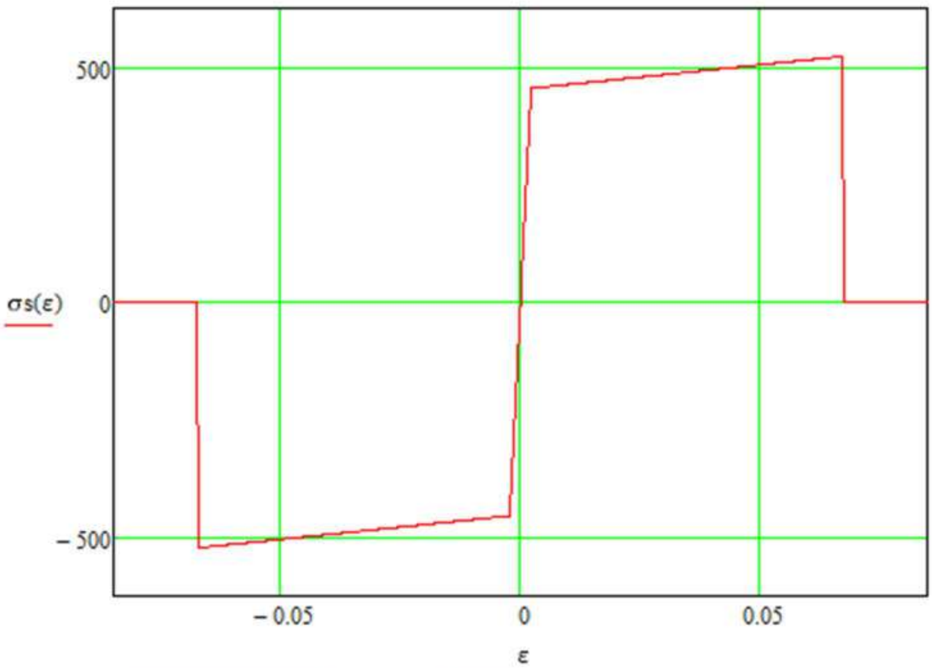
$$js := rows(As) \qquad js = 3$$

$$dsmax := max(ds) \qquad dsmax = 360$$

$$As\_tot := \sum_{j=1}^{js} As_j = 1.018 \times 10^3$$

15.3 Material constitutive laws employed in the calculation





15.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 9.842 \times 10^4$$

$$\rho s := \frac{As\_tot}{Ac} = 8.171 \times 10^{-3}$$ geometric ratio for longitudinal mild reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y) \cdot y\,dy \qquad Syc = 1.968 \times 10^7$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 200$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y) \cdot (y - yG)^2\,dy \qquad Ixo\_cls = 1.832 \times 10^9$$

Idealisation coefficients (elastic)

$$ns := \frac{Es}{Ecm} \qquad ns = 4.734$$

Area of ideal cross-section

$$Aid := Ac + (ns - 1)\cdot\sum_{j=1}^{js} As_j \qquad Aid = 1.014 \times 10^5$$

First moment of the reinforced concrete area

$$Sxid := Ac\cdot yG + (ns - 1)\cdot\sum_{j=1}^{js}\left(As_j\cdot ds_j\right) \qquad Sxid = 2.029 \times 10^7$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 200$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \left[\int_0^{Htot} b(y)\cdot(y - Yid)^2\,dy\right] - \sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns\cdot\sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidlenta \qquad Ixo\_id = 1.908 \times 10^9 \quad \text{mm^4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.042$$

15.5 Time-dependent behaviour

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$h0 := 2 \cdot \frac{Ac}{u} = 82.02$ mm notional size of the member

$RH := 50$ % relative humidity

$t0\_T(t0) := t0$

$\alpha := 1$ for cement class R

$$t0\_mod(t0) := \max\left[t0\_T(t0) \cdot \left(\frac{9}{2 + t0\_T(t0)^{1.2}} + 1\right)^{\alpha}, 0.5\right]$$

$t0\_mod(2) = 6.189$

$$\alpha c1 := \left(\frac{35}{-fcm}\right)^{0.7} = 0.524$$

$$\alpha c2 := \left(\frac{35}{-fcm}\right)^{0.2} = 0.832$$

$$\alpha c3 := \left(\frac{35}{-fcm}\right)^{0.5} = 0.631$$

$$\beta h := if\left[-fcm > 35, \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250 \cdot \alpha c3, 1500 \cdot \alpha c3\right], \min\left[1.5 \cdot \left[1 + (0.012 \cdot RH)^{18}\right] \cdot h0 + 250, 1500\right]\right] = 280.707$$

$$\beta t0(t0) := \frac{1}{0.1 + t0\_mod(t0)^{0.2}}$$

$$\beta c(t, t0) := \left(\frac{t - t0\_mod(t0)}{\beta h + t - t0\_mod(t0)}\right)^{0.3}$$

$$\beta fcm := \frac{16.8}{\sqrt{-fcm}} = 1.791$$

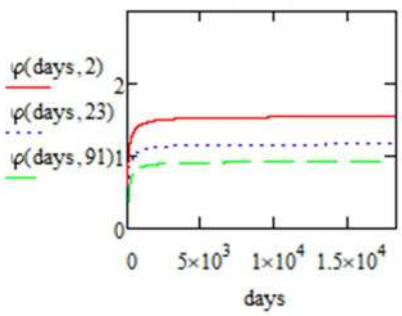
$$\varphi RH := if\left[-fcm > 35, \left(1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}} \cdot \alpha c1\right) \cdot \alpha c2, 1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h0}}\right] = 1.334$$

$\varphi 0(t0) := \varphi RH \cdot \beta fcm \cdot \beta t0(t0)$

$\varphi(t, t0) := \varphi 0(t0) \cdot \beta c(t, t0)$

$t := 50 \cdot 365$ days

$\varphi(t, 2) = 1.544$ $\varphi(t, 91) = 0.92$



15.6 Non-linear moment-curvature diagram

Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot} \left(\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\right) + \sum_{j=1}^{js} \left(\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\right)$$

$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot} \left[\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\cdot\left(y_i - yG\right)\right] + \sum_{j=1}^{js} \left[\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\cdot\left(ds_j - yG\right)\right]$$

Design external axial load

$NS := -4078000$ N

MODEL COLUMN FOR 2nd ORDER EFFECTS (§5.8.8)

$\omega := As\_tot\cdot\dfrac{fsd}{-Ac\cdot fcd} = 0.082$

$rm := 1$

$n := \dfrac{-N}{Ac\cdot -fcd} = 0.725$ $\qquad \dfrac{-N}{Ac\cdot -fcd + As\_tot\cdot fsd} = 0.67 \quad < \quad 1$ CHECK

$A := \dfrac{1}{1 + 0.2\cdot\varphi(t,91)} = 0.845$

$B := \sqrt{1 + 2\cdot\omega} = 1.079$

$C := 1.7 - rm = 0.7$

$\alpha crit := 18.457$

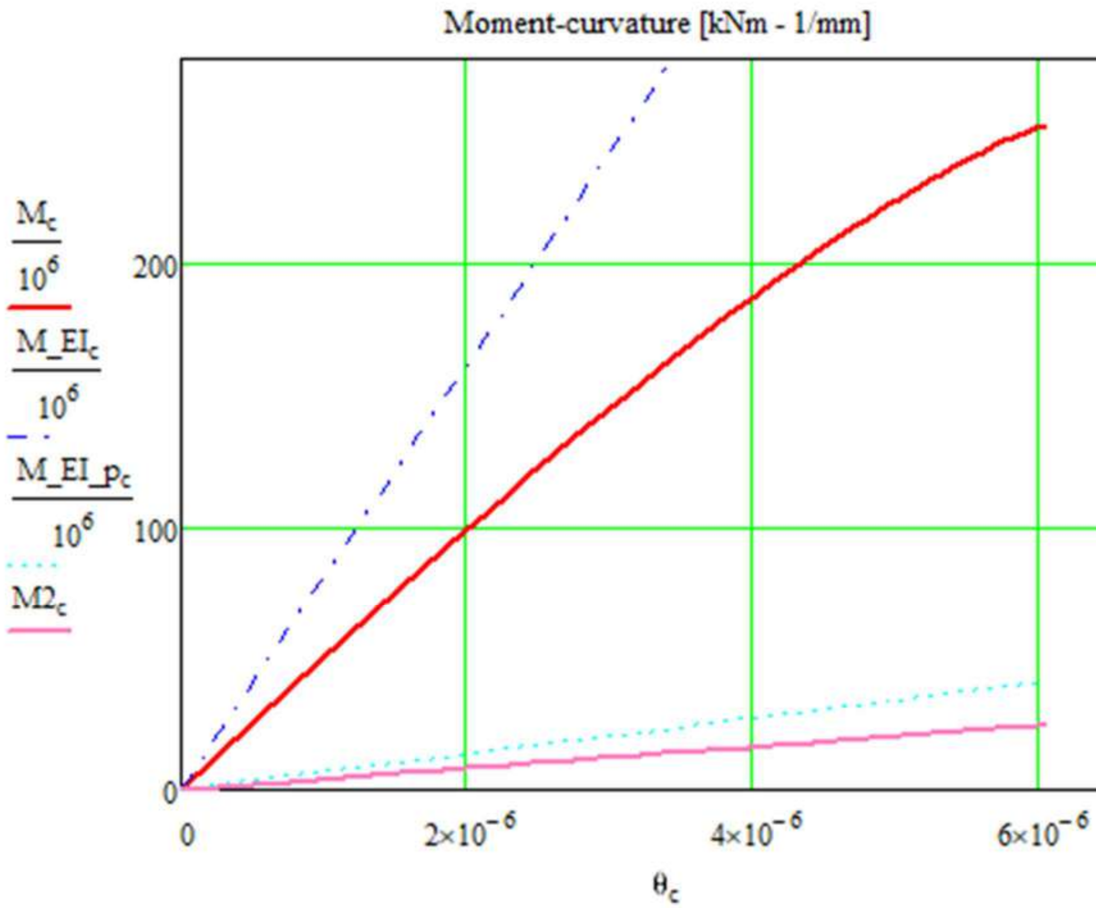
$l0 := \pi\cdot\sqrt{35000\cdot\dfrac{400^4}{12\cdot\alpha crit\cdot -N}} = 3.129\times 10^3$ mm $\qquad \dfrac{l0}{5000} = 0.626$ restraint coefficient β

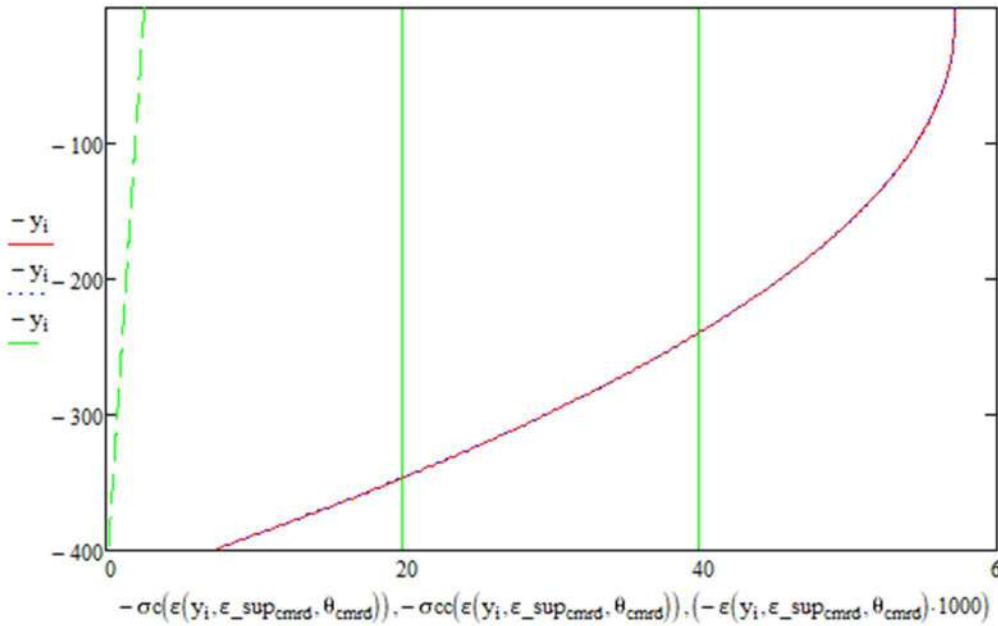
from FEM model -> linear buckling analysis

$\lambda lim := 20\cdot A\cdot B\cdot\dfrac{C}{\sqrt{n}} = 14.985 \quad > \quad \lambda := \dfrac{l0}{\sqrt{\dfrac{Ixo\_cls}{Ac}}} = 22.937$ CHECK

if not 2nd order effects need to be taken into account

$$M2_c := -N\cdot\theta_c\cdot\frac{\left(\frac{l0}{1000}\right)^2}{\pi^2}$$





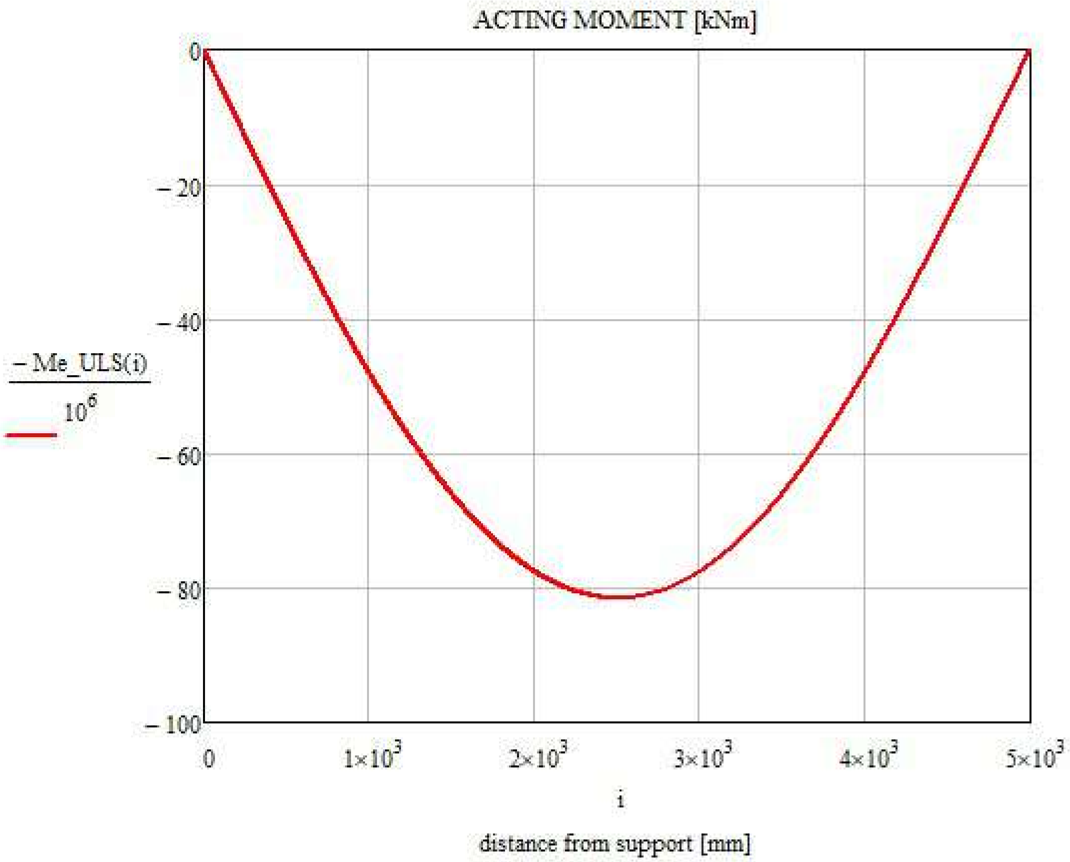
Condition at resisting (peak) moment (stress and strain)

15.7 Bending moment distribution (induced by eccentricity)

$L := 5000$ mm gross column interstorey length

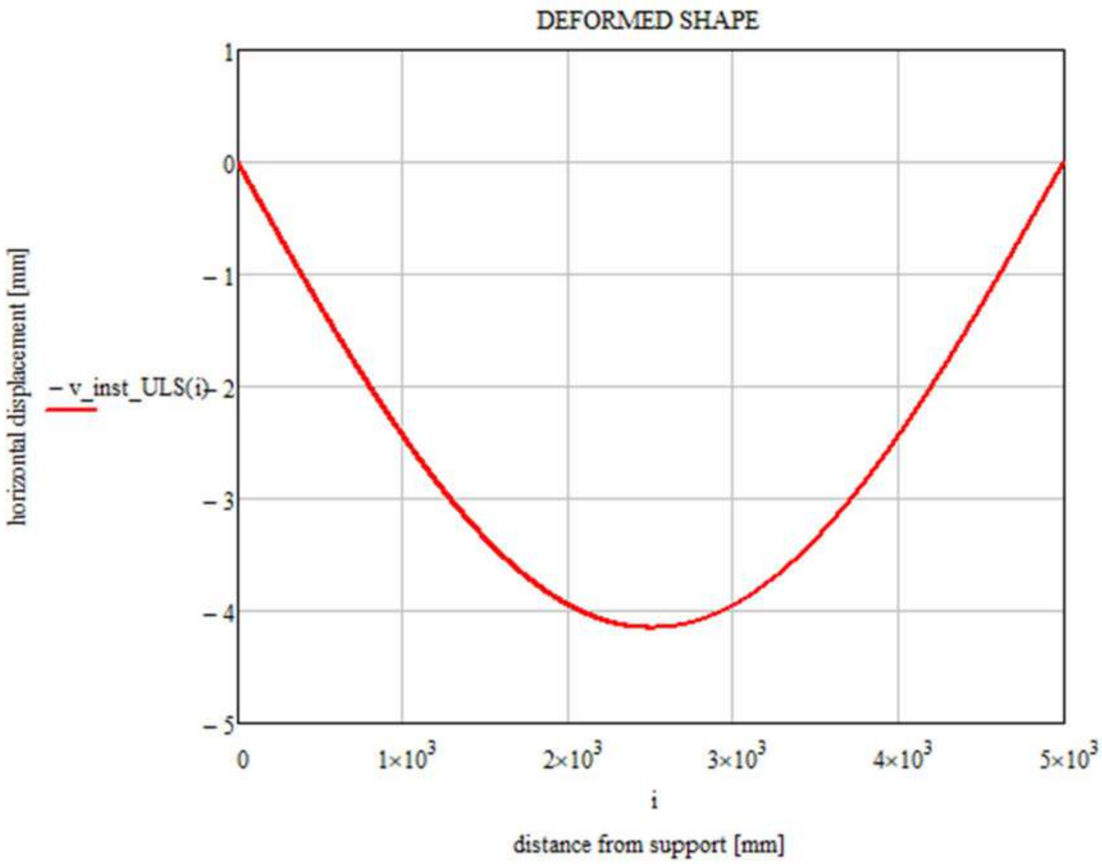
$Me\_ULS(x) := -N \cdot 20 \cdot \sin\left(\frac{\pi}{L} \cdot x\right)$ bending moment induced by geometrical imperfections

$i := 0 .. L$



15.8 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:



SLS STRESS CONTROL (§7.2)

$k1 := 0.6$

$k2 := 0.45$

$k3 := 0.8$

$k4 := 1$

$k5 := 0.75$

$Nr := -2826000$ N axial load in rare combination

$Nqp := -2266000$ N axial load in quasi permanent combination

$$\sigma c\_r\_bot(x) := \frac{Nr}{Aid} + \frac{Me\_ULS(x) \cdot \frac{Nr}{N} \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for rare load combination

$$\sigma c\_r\_top(x) := \frac{Nr}{Aid} + \frac{Me\_ULS(x) \cdot \frac{Nr}{N} \cdot (-Yid)}{Ixo\_id}$$

elastic stress of top concrete chord for rare load combination

$$\sigma c\_qp\_bot(x) := \frac{Nqp}{Aid} + \frac{Me\_ULS(x) \cdot \frac{Nqp}{N} \cdot (Htot - Yid)}{Ixo\_id}$$

elastic stress of bottom concrete chord for quasi permanent load combination

$$\sigma c\_qp\_top(x) := \frac{Nqp}{Aid} + \frac{Me\_ULS(x) \cdot \frac{Nqp}{N} \cdot (-Yid)}{Ixo\_id}$$

elastic stress of top concrete chord for quasi permanent load combination

$\sigma c\_r\_bot\left(\frac{L}{2}\right) = -21.785$ < $fctm = 4.839$ CHECK

if not -> cracked

$\sigma c\_r\_top\left(\frac{L}{2}\right) = -33.505$ > $k1 \cdot fck = -48$ CHECK

> $0.4 \cdot fcm = -35.2$

$\sigma c\_qp\_bot\left(\frac{L}{2}\right) = -17.468$ < $fctm = 4.839$ CHECK

if not -> cracked

$\sigma c\_qp\_top\left(\frac{L}{2}\right) = -26.866$ > $k2 \cdot fck = -36$ CHECK

15.9 ULS checks

MINIMUM REINFORCEMENT

$bt := b(Htot) = 400$ $d := ds_{js} = 360$

$\max\left(0.1 \cdot \frac{-N}{fsd}, 0.002 \cdot Ac\right) = 897.16$ §9.12N mm2 < $As\_tot = 1.018 \times 10^3$ mm2 CHECK < $0.04 \cdot Ac = 3.937 \times 10^3$ CHECK §9.2.1.1(3)

$scltmax := \min(20 \cdot 16, 400, 400) = 320$ > $s2 := 320$ mm CHECK stirrup spacing out of joint/bracket/lap area

$scltmaxred := 0.6 \cdot scltmax = 192$ > $s1 := 160$ mm CHECK stirrup spacing within joint/bracket/lap area

NEAR LAPS AND JOINTS

ANCHORAGE (§8.4)

$\eta 1 := 1$

$\eta 2 := 1$

$fbd := 2.25 \cdot \eta 1 \cdot \eta 2 \cdot fctd = 5.443$

$lbrqd(\phi) := \frac{\phi}{4} \cdot \frac{fsd}{fbd}$

$\alpha 1b := 1$

$\alpha 2b := 1$

$\alpha 3b := 1$

$\alpha 4b := 1$

$\alpha 5b := 1$

$lbd(\phi) := \alpha 1b \cdot \alpha 2b \cdot \alpha 3b \cdot \alpha 4b \cdot \alpha 5b \cdot lbrqd(\phi)$

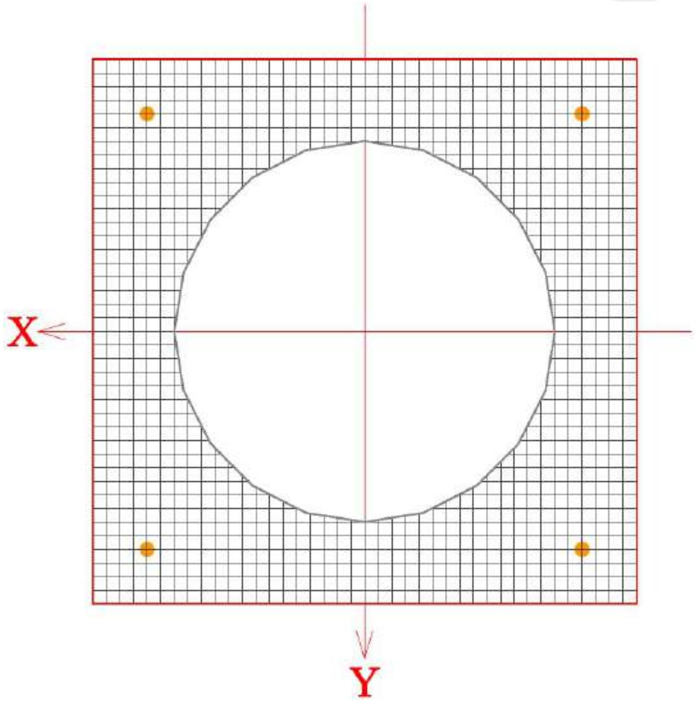
ANCHORAGE OF JOINT REBARS IN M80 MORTAR $lbd(16) = 334.011$

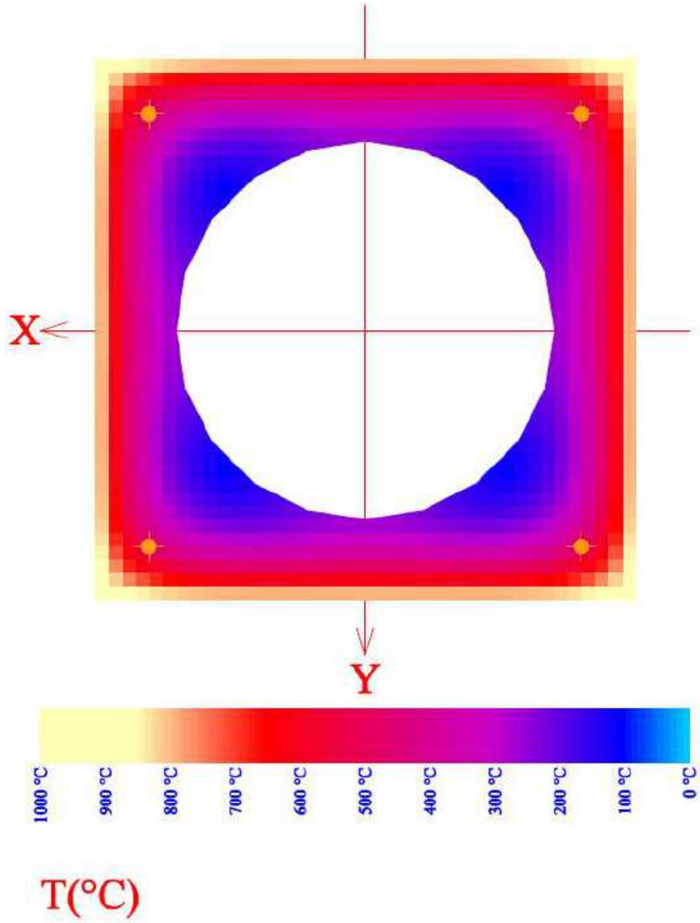
$fcm := -80 \cdot 0.83$ $fck := fcm + 8 = -58.4$

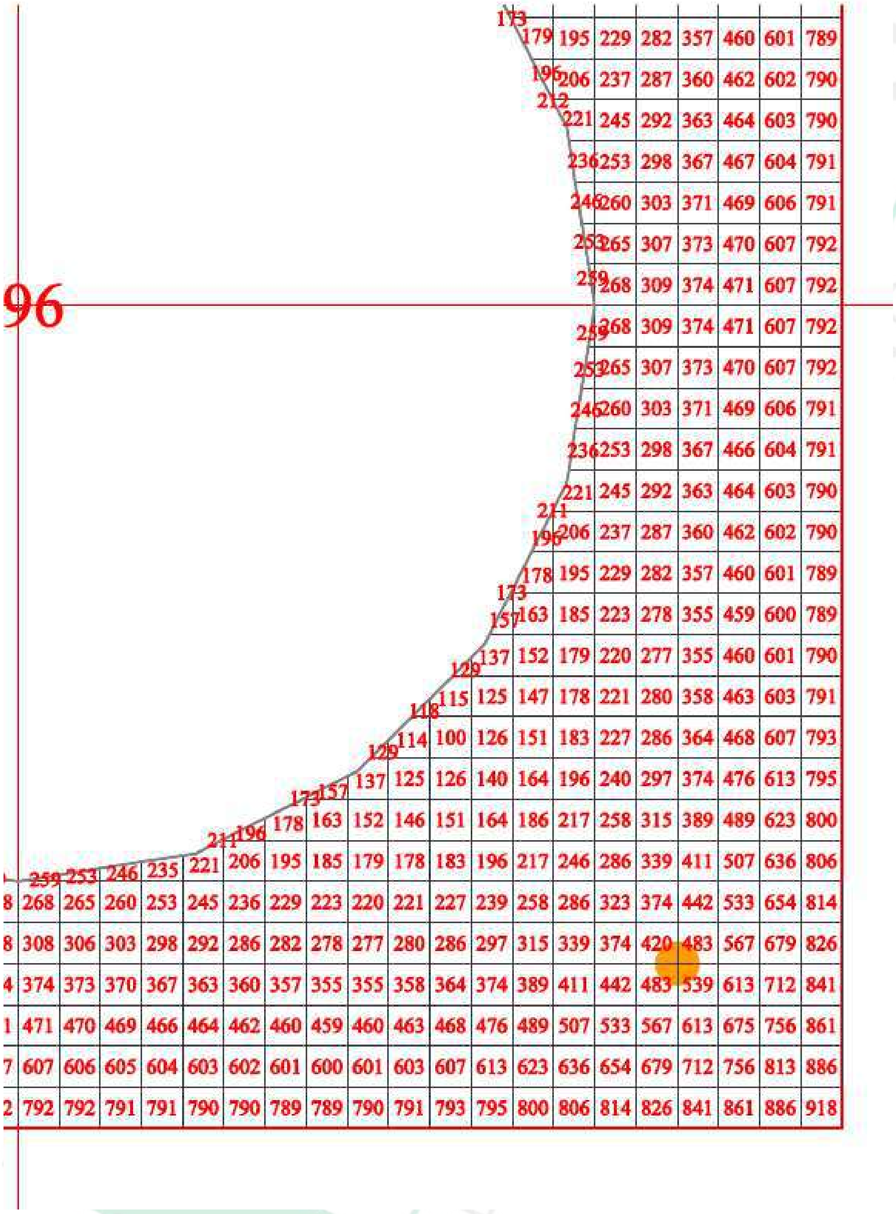
$fmtd := 2.155$ MPa same formula as concrete was used (on the safe side)

$$lbd(16) \cdot \frac{fctd}{fmtd} = 374.979$$

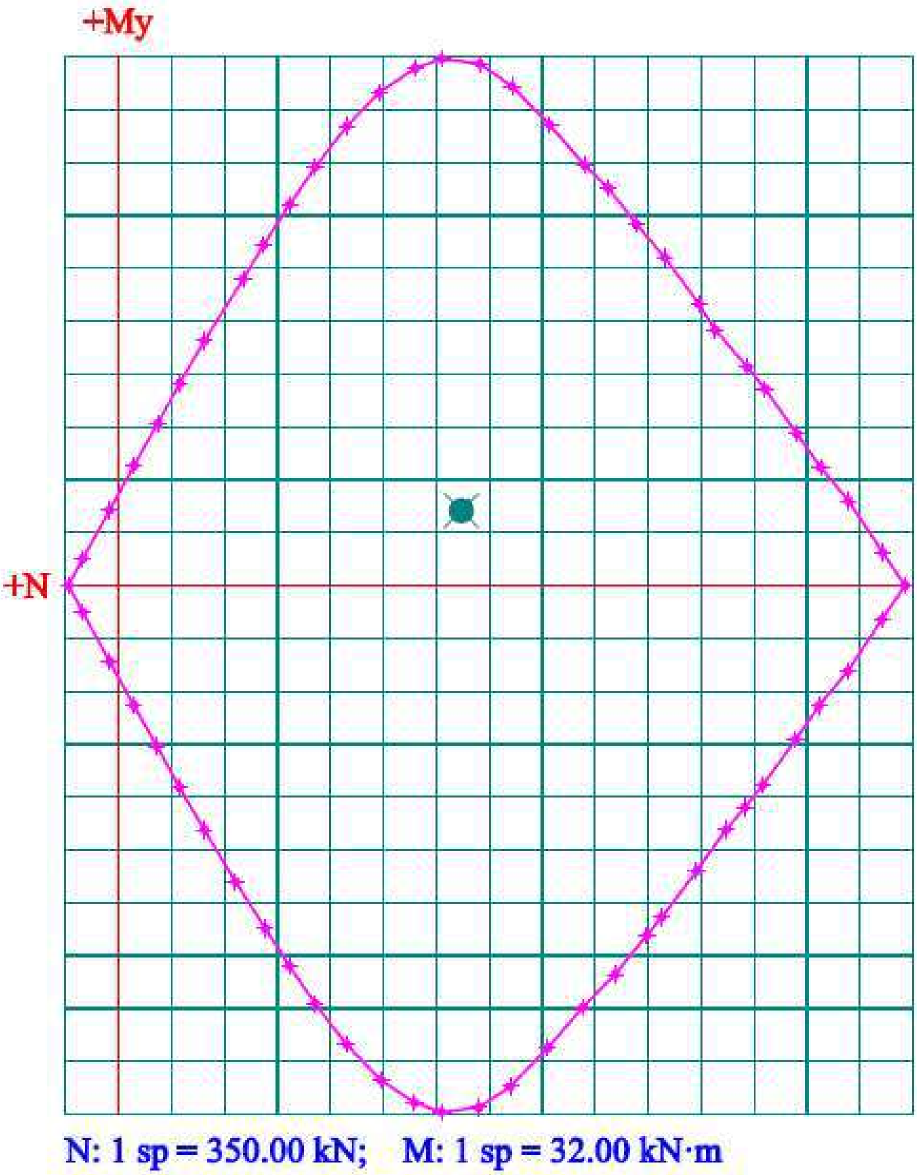
15.10 Fire checks




X
Y
1000 °C
900 °C
800 °C
700 °C
600 °C
500 °C
400 °C
300 °C
200 °C
100 °C
0 °C
T(°C)



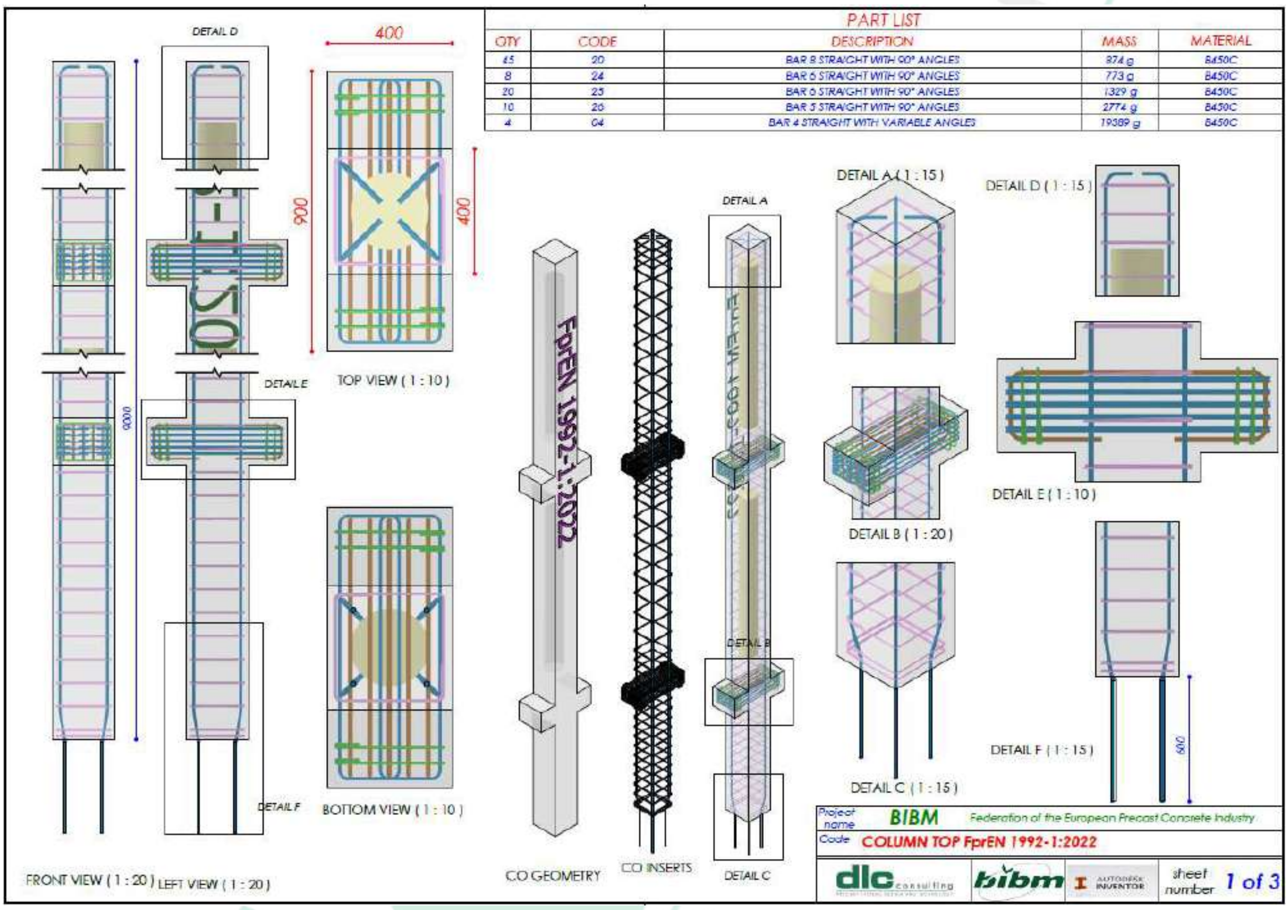
173
179 195 229 282 357 460 601 789
196 206 237 287 360 462 602 790
212
221 245 292 363 464 603 790
236 253 298 367 467 604 791
246 260 303 371 469 606 791
253 265 307 373 470 607 792
259 268 309 374 471 607 792
96
259 268 309 374 471 607 792
253 265 307 373 470 607 792
246 260 303 371 469 606 791
236 253 298 367 466 604 791
221 245 292 363 464 603 790
211
196 206 237 287 360 462 602 790
178 195 229 282 357 460 601 789
173
157 163 185 223 278 355 459 600 789
129 137 152 179 220 277 355 460 601 790
118 115 125 147 178 221 280 358 463 603 791
129 114 100 126 151 183 227 286 364 468 607 793
173 157 137 125 126 140 164 196 240 297 374 476 613 795
178 163 152 146 151 164 186 217 258 315 389 489 623 800
211 196 206 195 185 179 178 183 196 217 246 286 339 411 507 636 806
259 253 246 235 221
8 268 265 260 253 245 236 229 223 220 221 227 239 258 286 323 374 442 533 654 814
8 308 306 303 298 292 286 282 278 277 280 286 297 315 339 374 420 483 567 679 826
4 374 373 370 367 363 360 357 355 355 358 364 374 389 411 442 483 539 613 712 841
1 471 470 469 466 464 462 460 459 460 463 468 476 489 507 533 567 613 675 756 861
7 607 606 605 604 603 602 601 600 601 603 607 613 623 636 654 679 712 756 813 886
2 792 792 791 791 790 790 789 789 790 791 793 795 800 806 814 826 841 861 886 918

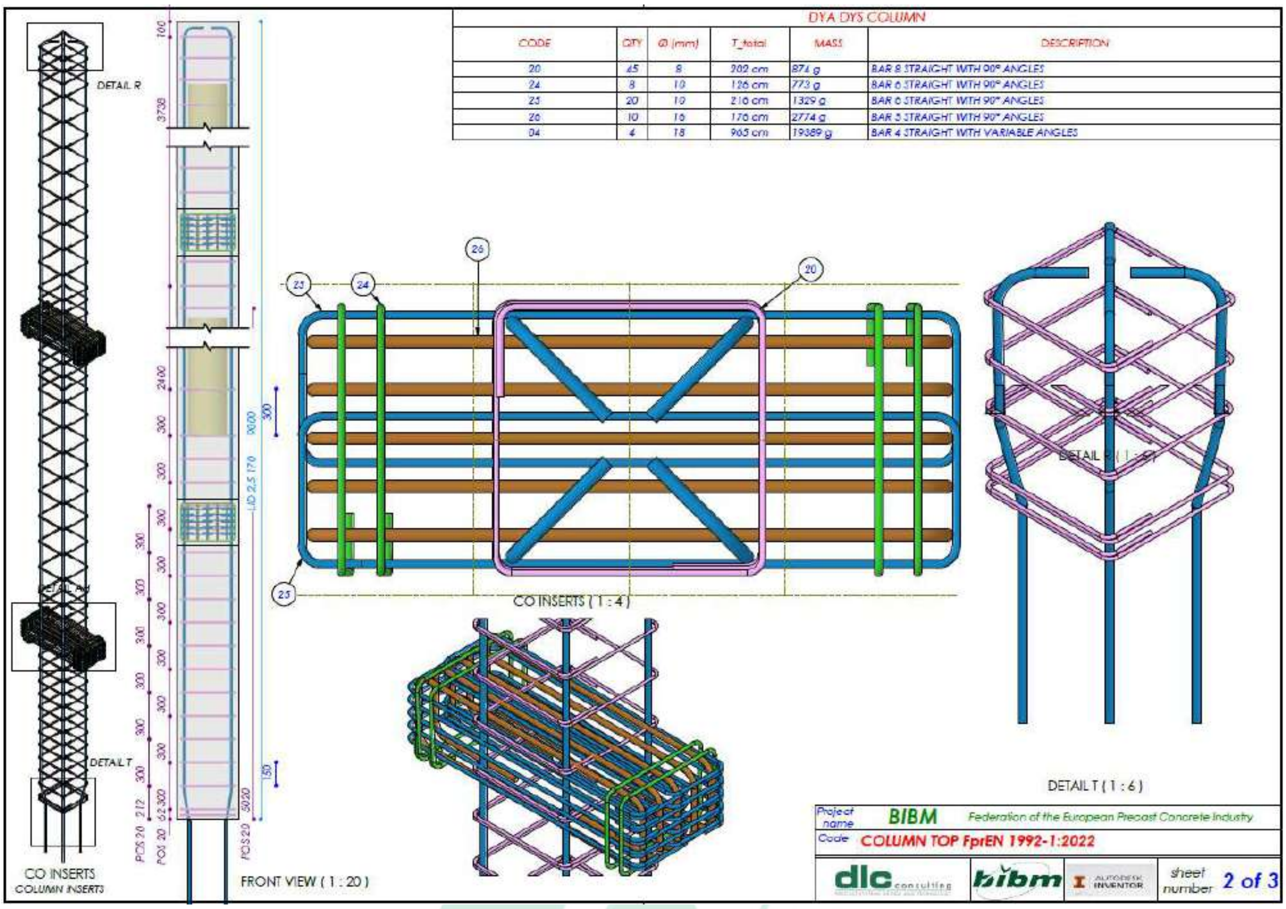


Nz = 0.00 kN
My+ = 55.53 kN·m
My- = -55.68 kN·m

16 Column element – FprEN1992-1:2022

16.1 Shop drawings



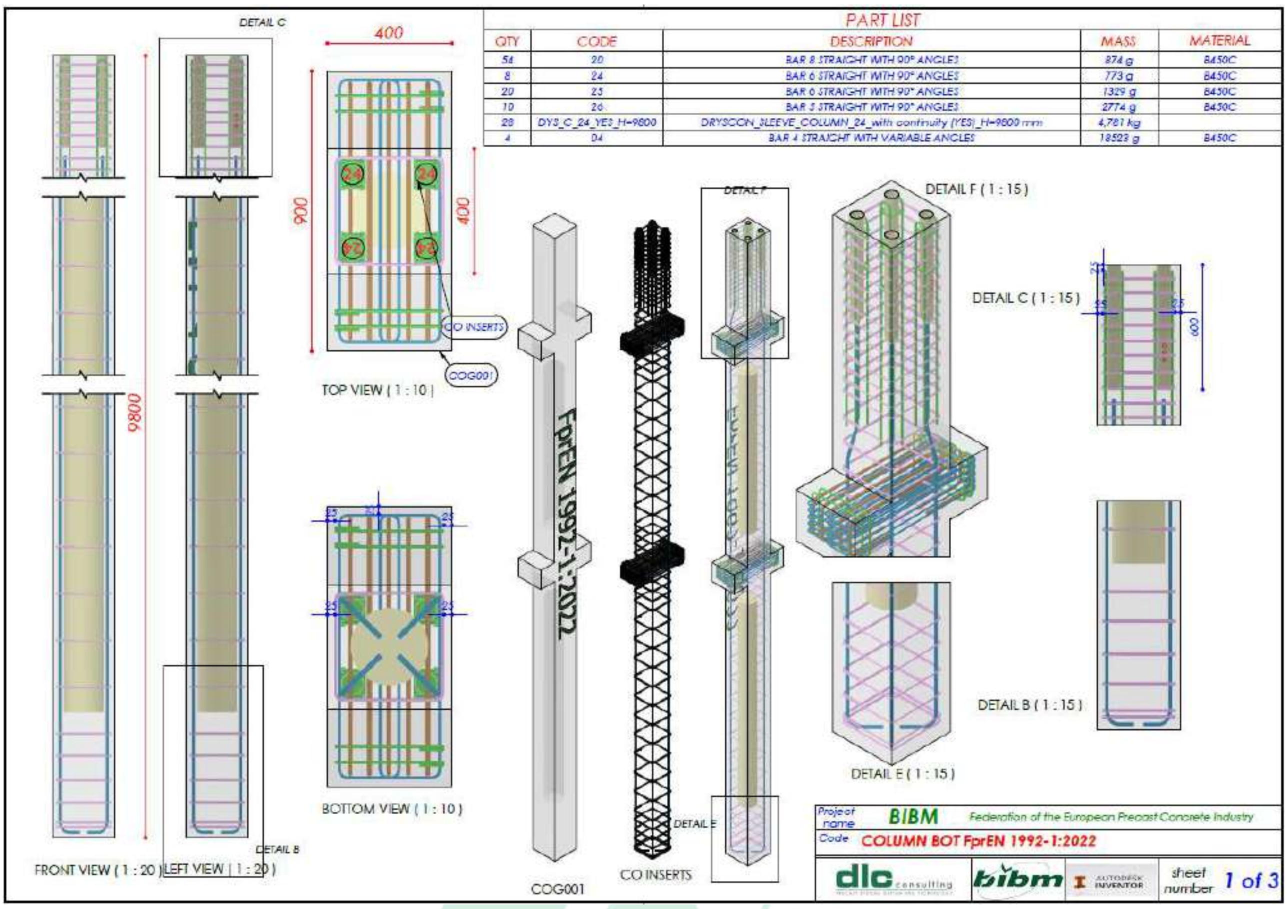


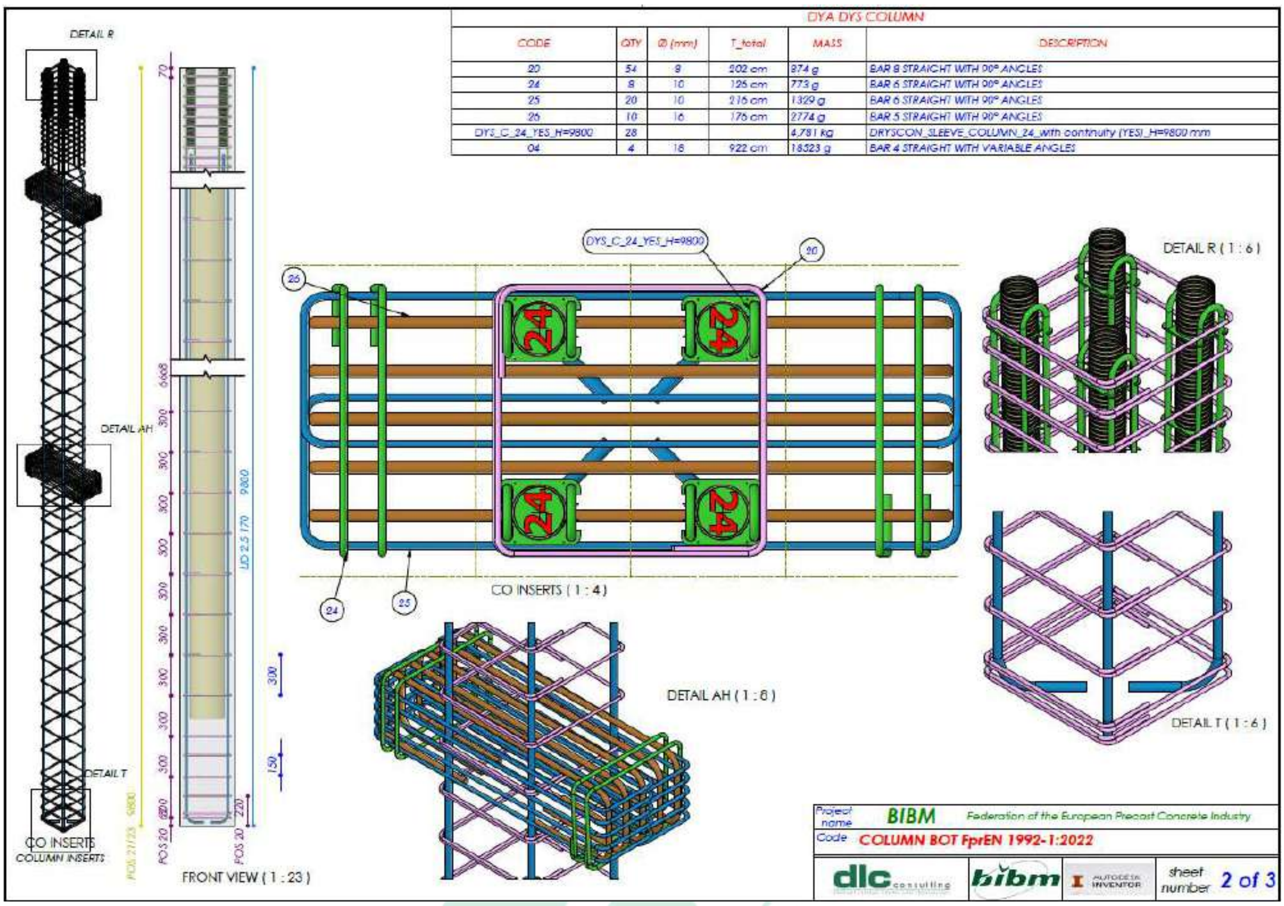
DYA DYS COLUMN					
CODE	QTY	Ø (mm)	T_total	MASS	DESCRIPTION
20	45	8	202 cm	874 g	BAR 8 STRAIGHT WITH 90° ANGLES
24	8	10	126 cm	773 g	BAR 6 STRAIGHT WITH 90° ANGLES
25	20	10	210 cm	1329 g	BAR 6 STRAIGHT WITH 90° ANGLES
26	10	16	176 cm	2774 g	BAR 5 STRAIGHT WITH 90° ANGLES
04	4	18	965 cm	19389 g	BAR 4 STRAIGHT WITH VARIABLE ANGLES

Thumbnail	Part Number	QTY	Mass	Total Mass	Ø_			
	04	4	19389	77556	18 mm			
	20	45	874	39330	8 mm			
	24	8	773	6184	10 mm			
	25	20	1329	26580	10 mm			
	26	10	2774	27740	16 mm			
Total mass rebars [kg]				177,37	Incidence kg/m³		135,21	
	lightening 1	1	6136	6136				
	lightening 2	1	6234	6234				
Total mass lightening [kg]				12,37	Incidence kg/m³		9,43	
	Total mass of steel [kg]			177,39		Total concrete volume [m³]		1,312

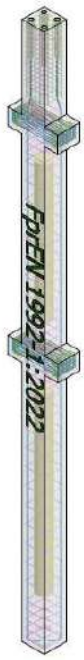
VIEW 107 (1 :40)

Project name	BIBM	Federation of the European Precast Concrete Industry
Code	COLUMN TOP FprEN 1992-1:2022	
dlc consulting / bibm / AUTODESK INVENTOR	sheet number	3 of 3


PART LIST
QTY | CODE | DESCRIPTION | MASS | MATERIAL
54 | 20 | BAR 8 STRAIGHT WITH 90° ANGLES | 874 g | B450C
8 | 24 | BAR 6 STRAIGHT WITH 90° ANGLES | 773 g | B450C
20 | 25 | BAR 6 STRAIGHT WITH 90° ANGLES | 1329 g | B450C
10 | 26 | BAR 5 STRAIGHT WITH 90° ANGLES | 2774 g | B450C
28 | DYS_C_24_YES_H=9800 | DRYSCON_SLEEVE_COLUMN_24_with continuity (YES)_H=9800 mm | 4,781 kg |
4 | D4 | BAR 4 STRAIGHT WITH VARIABLE ANGLES | 18523 g | B450C
DETAIL C
400
900
9800
9886
TOP VIEW (1 : 10)
CO INSERTS
COG001
BOTTOM VIEW (1 : 10)
FRONT VIEW (1 : 20)
LEFT VIEW (1 : 20)
DETAIL B
FprEN 1992-1:2022
DETAIL F
DETAIL F (1 : 15)
DETAIL C (1 : 15)
DETAIL B (1 : 15)
DETAIL E (1 : 15)
DETAIL E
Project name BIBM Federation of the European Precast Concrete Industry
Code COLUMN BOT FprEN 1992-1:2022
sheet number 1 of 3


DYA DYS COLUMN
CODE | QTY | Ø (mm) | T_total | MASS | DESCRIPTION
20 | 54 | 8 | 202 cm | 874 g | BAR 8 STRAIGHT WITH 90° ANGLES
24 | 8 | 10 | 125 cm | 773 g | BAR 6 STRAIGHT WITH 90° ANGLES
25 | 20 | 10 | 216 cm | 1329 g | BAR 6 STRAIGHT WITH 90° ANGLES
26 | 10 | 16 | 175 cm | 2774 g | BAR 5 STRAIGHT WITH 90° ANGLES
DYS_C_24_YES_H=9800 | 28 | | | 4,781 kg | DRYSCON_SLEEVE_COLUMN_24_with continuity (YES)_H=9800 mm
04 | 4 | 16 | 922 cm | 18523 g | BAR 4 STRAIGHT WITH VARIABLE ANGLES
DETAIL R
DETAIL AH
DETAIL T
CO INSERTS
COLUMN INSERTS
FRONT VIEW (1 : 23)
DYS_C_24_YES_H=9800
CO INSERTS (1 : 4)
DETAIL R (1 : 6)
DETAIL AH (1 : 6)
DETAIL T (1 : 6)
Project name BIBM Federation of the European Precast Concrete Industry
Code COLUMN BOT FprEN 1992-1:2022
sheet number 2 of 3

Thumbnail	Part Number	QTY	Mass	Mass Total	Ø_		
	04	4	18523	74092	18 mm		
	20	54	874	47196	8 mm		
	24	8	773	6184	10 mm		
	25	20	1329	26580	10 mm		
	26	10	2774	27740	16 mm		
	DYS_C_24	4	4781	19124			
	Total mass rebars [kg]			200,92	Incidence kg/m³	142,70	
	lightening 1	1	6136	6136			
	lightening 2	1	7854	7854			
	Total mass lightening [kg]			13,99	Incidence kg/m³	9,94	
	Total mass of steel [kg]			200,92	Total concrete volume [m³]	1.408	



Project name	BIBM	Federation of the European Precast Concrete Industry
Code	COLUMN BOT FprEN 1992-1:2022	
dlc consulting · bibm · AUTODESK INVENTOR	sheet number	3 of 3

16.2 Definition of concrete and reinforcement geometry

GEOMETRY

Concrete

Depth from upper chord

$y\_tr := (0 \;\; 400)^T$

$Htot := max(y\_tr)$

$hcopr := 30$ net cover of longitudinal rebars

Width of corresponding chord:

$b\_tr := (400 \;\; 400)^T$

$r\_circ := 125$ radius of central void pipe

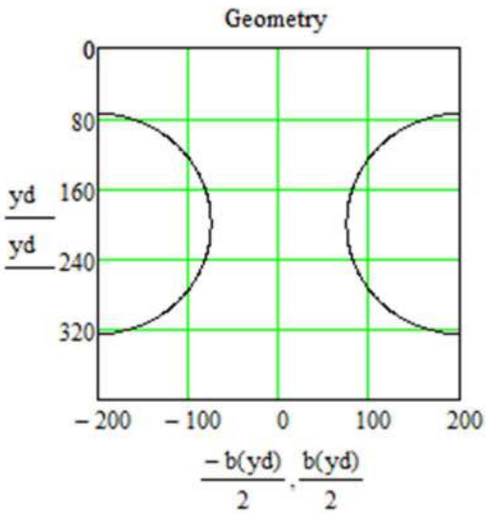
$$x\_circ(y) := 2\sqrt{r\_circ^2 - \left(y - \frac{Htot}{2}\right)^2}$$

$b\_lin(y) := linterp(y\_tr, b\_tr, y)$

$b\_circ(y) := linterp(y\_tr, b\_tr, y) - x\_circ(y)$

$$b(y) := if\left[y \le \left(\frac{Htot}{2} + r\_circ\right) \wedge y \ge \frac{Htot}{2} - r\_circ, b\_circ(y), b\_lin(y)\right]$$

$yd := 0..Htot$



condensed 1D geometry plot

$u := 800 \cdot 2 + Htot \cdot 2 = 2.4 \times 10^3$ exposed perimeter

Longitudinal mild reinforcement

Area of single rebar:

$$A(\phi) := \frac{\phi^2 \cdot \pi}{4}$$

Distance of rebars from upper chord

$$ds := (40 \;\; 200 \;\; 360)^T$$

Area of reinforcement at each depth

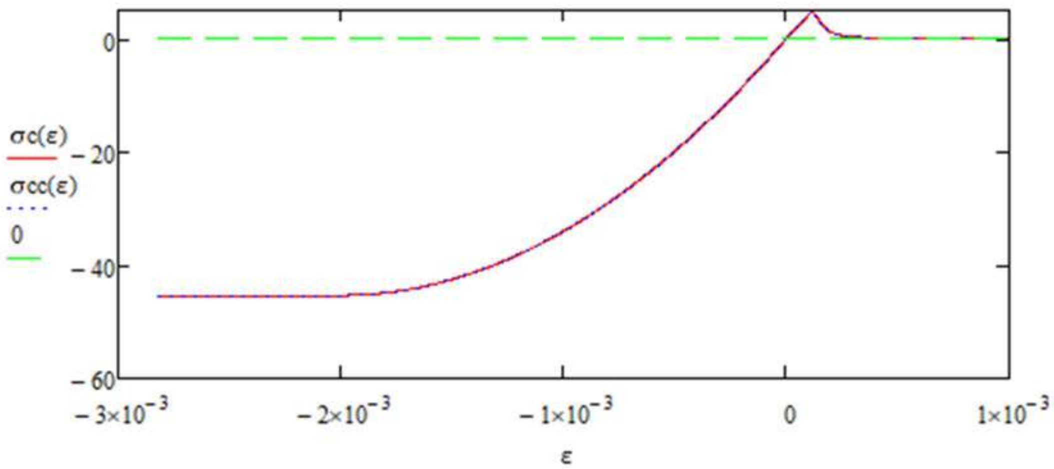
$$As := (2 \cdot A(18) \;\; 0 \cdot A(16) \;\; 2 \cdot A(18))^T$$

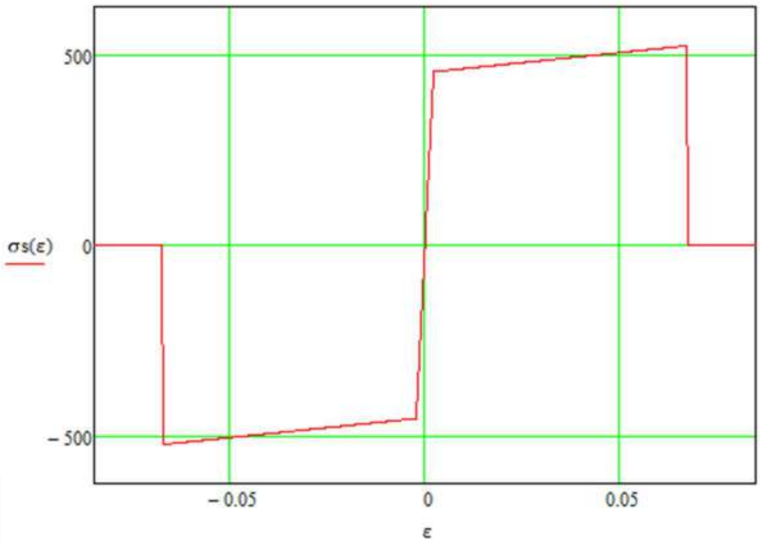
$js := rows(As)$ $\qquad js = 3$

$dsmax := max(ds)$ $\qquad dsmax = 360$

$$As\_tot := \sum_{j=1}^{js} As_j = 1.018 \times 10^3$$

16.3 Material constitutive laws employed in the calculation





16.4 Sectional properties

PROPERTIES OF THE CROSS-SECTION

Assumption of uncracked cross-section

Area of concrete neglecting reinforcement

$$Ac := \int_0^{Htot} b(y)\,dy \qquad Ac = 1.109 \times 10^5$$

$$\rho s := \frac{As\_tot}{Ac} = 9.177 \times 10^{-3}$$ geometric ratio for longitudinal mild reinforcement

First moment of the concrete area

$$Syc := \int_0^{Htot} b(y) \cdot y\,dy \qquad Syc = 2.218 \times 10^7$$

Centre of mass of the concrete area

$$yG := \frac{Syc}{Ac} \qquad yG = 200$$

Second moment of the concrete area

$$Ixo\_cls := \int_0^{Htot} b(y) \cdot (y - yG)^2\,dy \qquad Ixo\_cls = 1.942 \times 10^9$$

Idealisation coefficients (elastic)

$$ns := \frac{Es}{Ecm} \qquad ns = 4.733$$

Area of ideal cross-section

$$Aid := Ac + (ns - 1)\cdot\sum_{j=1}^{js} As_j \qquad Aid = 1.147\times10^{5}$$

First moment of the reinforced concrete area

$$Sxid := Ac\cdot yG + (ns - 1)\cdot\sum_{j=1}^{js}\left(As_j\cdot ds_j\right) \qquad Sxid = 2.294\times10^{7}$$

Centre of mass of the reinforced concrete area

$$Yid := \frac{Sxid}{Aid} \qquad Yid = 200$$

Second moment of the concrete area subtracting the effect of reinforcement

$$Ixoidcls := \int_0^{Htot} b(y)\cdot(y - Yid)^2\,dy - \sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the mild reinforcement area

$$Ixoidlenta := ns\cdot\sum_{j=1}^{js}\left[As_j\cdot\left(ds_j - Yid\right)^2\right]$$

Second moment of the idealised reinforced concrete area

$$Ixo\_id := Ixoidcls + Ixoidlenta \qquad Ixo\_id = 2.039\times10^{9}\ \text{mm^4} \qquad \frac{Ixo\_id}{Ixo\_cls} = 1.05$$

16.5 Time-dependent behaviour

DETAILED EVALUATION OF CREEP COEFFICIENT (ANNEX B)

$$hn := 2\cdot\frac{Ac}{u} = 92.427$$

$$RH := 50$$

$$t0\_adj(t0) := t0$$

$$\beta bc\_fcm := \frac{1.8}{(-fcm)^{0.7}} = 0.078 \qquad \beta bc\_t\_t0(t,t0) := \ln\left[\left(\frac{30}{t0\_adj(t0)} + 0.035\right)^2\cdot(t - t0) + 1\right]$$

$$\beta dc\_fcm := \frac{412}{(-fcm)^{1.4}} = 0.781$$

$$\beta dc\_RH := \frac{1 - \frac{RH}{100}}{\sqrt[3]{0.1\cdot\frac{hn}{100}}} = 1.106$$

$$\beta dc\_t0(t0) := \frac{1}{0.1 + t0\_adj(t0)^{0.2}}$$

$$\gamma(t0) := \frac{1}{2.3 + \frac{3.5}{\sqrt{t0\_adj(t0)}}}$$

$$\alpha cm := \left(\frac{35}{-fcm}\right)^{0.5} = 0.631$$

$$\beta h := \min(1.5\cdot hn + 250\cdot\alpha cm, 1500\cdot\alpha cm) = 296.305$$

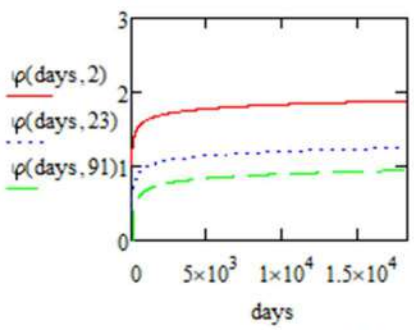
$$\beta dc\_t\_t0(t,t0) := \left[\frac{(t - t0)}{\beta h + (t - t0)}\right]^{\gamma(t0)}$$

$$\varphi dc(t,t0) := \beta dc\_fcm\cdot\beta dc\_RH\cdot\beta dc\_t0(t0)\cdot\beta dc\_t\_t0(t,t0)$$

$$\varphi bc(t,t0) := \beta bc\_fcm\cdot\beta bc\_t\_t0(t,t0)$$

$$\varphi(t,t0) := \varphi bc(t,t0) + \varphi dc(t,t0)$$

$$t := 50\cdot 365 = 1.825\times 10^4 \qquad \varphi(t,2) = 1.883 \qquad \varphi(t,91) = 0.945$$



16.6 Non-linear moment-curvature diagram

Equilibrium equations (rotation with respect to the centre of mass of the concrete section)

$$N(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot} \left(\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\right) + \sum_{j=1}^{js}\left(\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\right)$$

$$M(\varepsilon\_sup,\theta) := \sum_{i=1}^{Htot} \left[\sigma c\left(\varepsilon\left(y_i,\varepsilon\_sup,\theta\right)\right)\cdot b\left(y_i\right)\cdot\Delta y\cdot\left(y_i - yG\right)\right] + \sum_{j=1}^{js}\left[\sigma s\left(\varepsilon\left(ds_j,\varepsilon\_sup,\theta\right)\right)\cdot As_j\cdot\left(ds_j - yG\right)\right]$$

Design external axial load

$NS := -4078000$

MODEL COLUMN FOR 2nd ORDER EFFECTS (§7.4.3.2)

$\omega := As\_tot\cdot\frac{fsd}{-Ac\cdot fcd} = 0.092$

$rm := 1$

$n := \frac{-N}{Ac\cdot -fcd} = 0.811$ $\qquad \frac{-N}{Ac\cdot -fcd + As\_tot\cdot fsd} = 0.742 \quad < \quad 1$ CHECK

$A := \frac{1}{1 + 0.2\cdot\varphi(t,91)} = 0.841$

$B := \sqrt{1 + 2\cdot\omega} = 1.088$

$C := 1.7 - rm = 0.7$

$\alpha crit := 18.457$

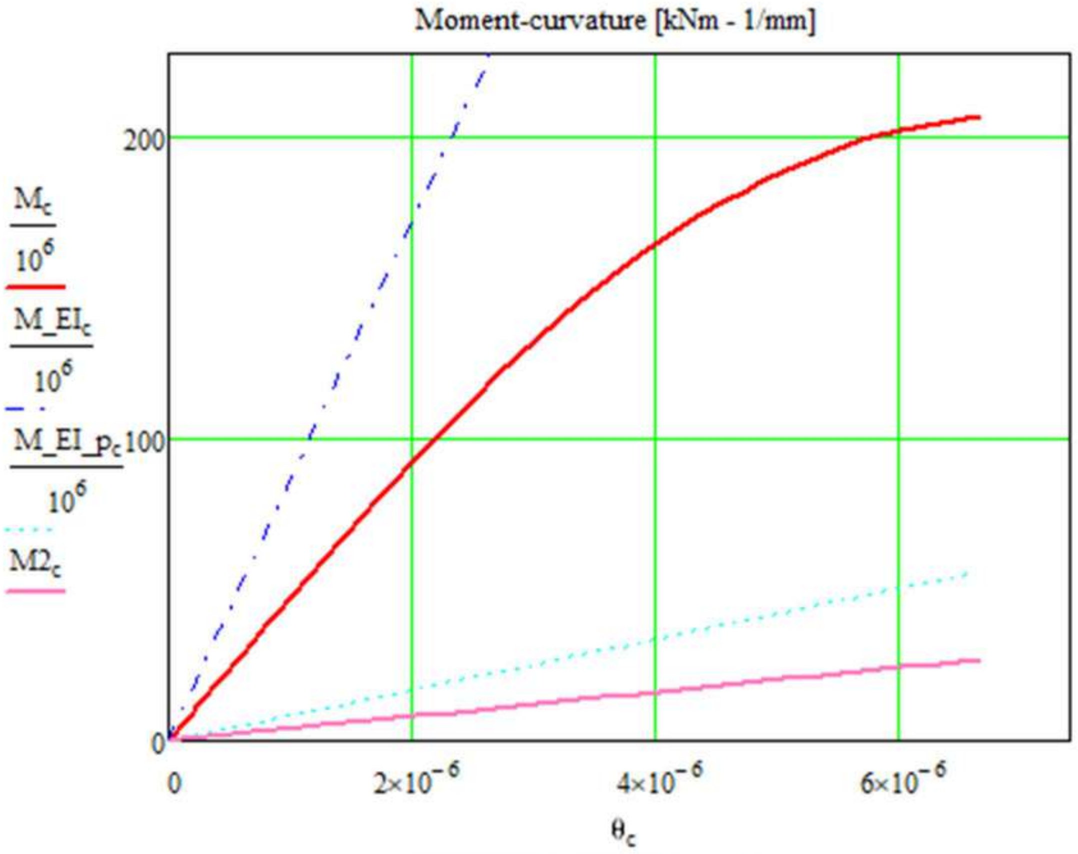
$l0 := \pi\cdot\sqrt{35000\cdot\frac{400^4}{12\cdot\alpha crit\cdot -N}} = 3.129\times 10^3$ mm $\qquad \frac{l0}{5000} = 0.626$ restraint coefficient β

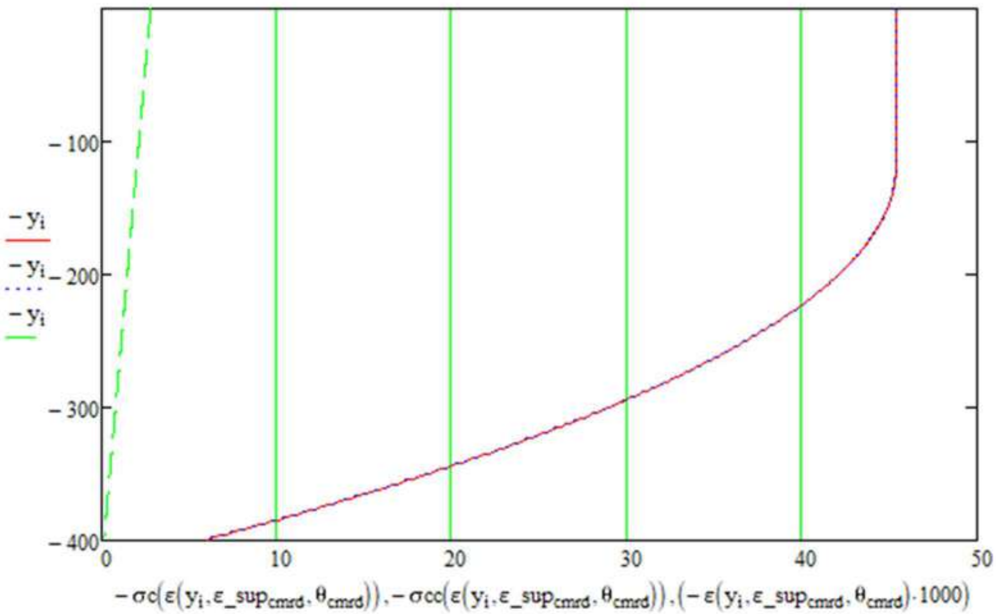
from FEM model -> linear buckling analysis

$\lambda lim := 20\cdot A\cdot B\cdot\frac{C}{\sqrt{n}} = 14.229 \quad > \quad \lambda := \frac{l0}{\sqrt{\frac{Ixo\_cls}{Ac}}} = 23.649$ CHECK

if not 2nd order effects need to be taken into account

$$M2_c := -N\cdot\theta_c\cdot\frac{\left(\frac{l0}{1000}\right)^2}{\pi^2}$$



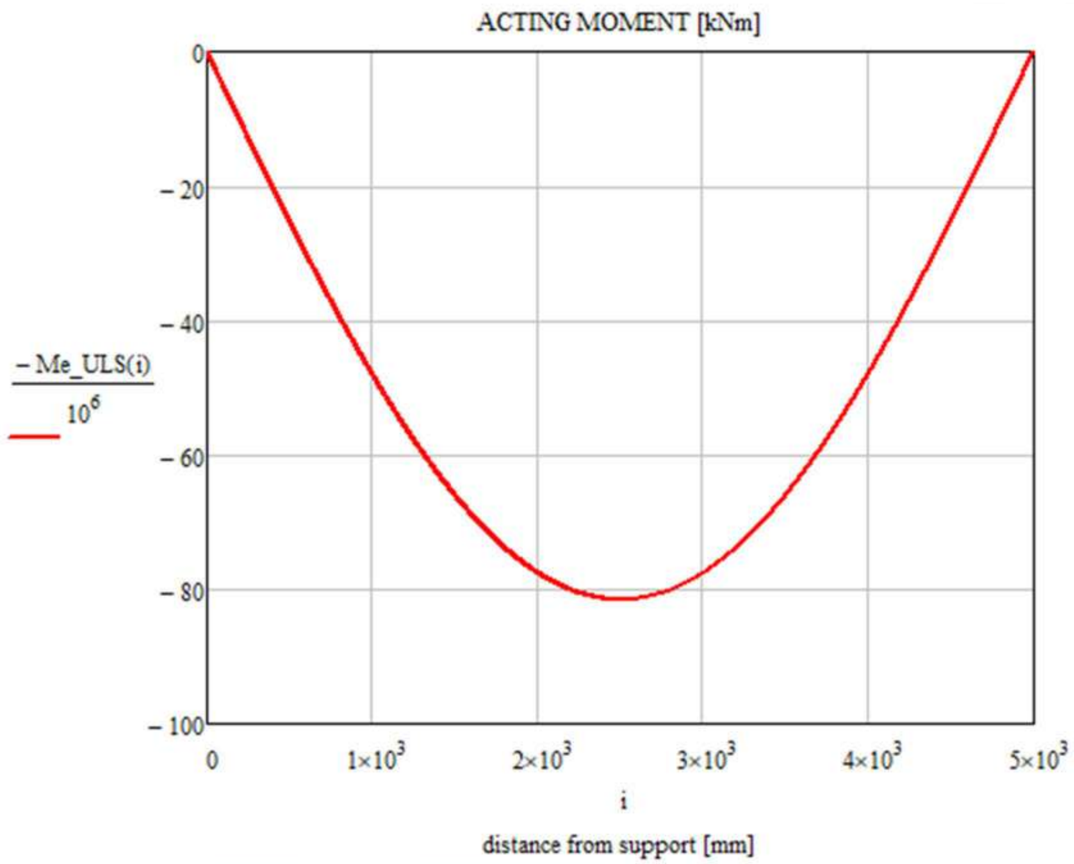


Condition at resisting (peak) moment (stress and strain)

16.7 Bending moment distribution induced by eccentricity

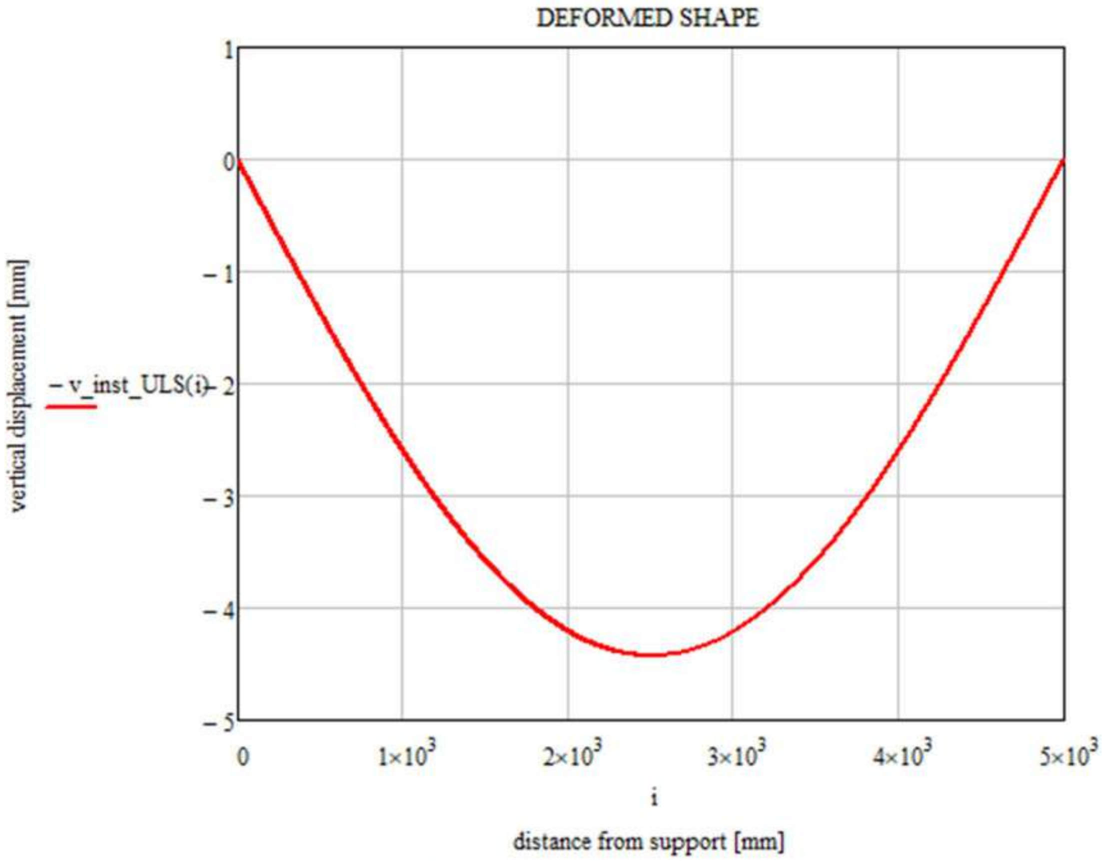
$L := 5000$ mm gross interstorey height

$Me\_ULS(x) := -N \cdot 20 \cdot \sin\left(\frac{\pi}{L} \cdot x\right)$ bending moment distribution induced by imperfections



16.8 SLS checks

NON-LINEAR DEFLECTION PROFILE FOR SIMPLY SUPPORTED BEAM:



SLS STRESS CONTROL (§9.2.1)

$k1 := 0.6$

$k2 := 0.45$

$k3 := 0.8$

$k4 := 1$

$k5 := 0.8$ 0.75 in EN1992-1-1:2002

NOTE: the denomination of the allowable stress coefficients following k factors was kept similar to that of EN1992-1-1:2002

$Nr := -2826000$ N axial load in rare combination

$Nqp := -2266000$ N axial load in quasi permanent combination

$$\sigma c\_r\_bot(x) := \frac{Nr}{Aid} + \frac{Me\_ULS(x) \cdot \frac{Nr}{N} \cdot (Htot - Yid)}{Ixo\_id} \qquad \sigma c\_r\_bot\left(\frac{L}{2}\right) = -19.091 \quad < \quad fctm = 4.74$$

CHECK

if not -> cracked

elastic stress of bottom concrete chord for rare load combination

$$\sigma c\_r\_top(x) := \frac{Nr}{Aid} + \frac{Me\_ULS(x) \cdot \frac{Nr}{N} \cdot (-Yid)}{Ixo\_id} \qquad \sigma c\_r\_top\left(\frac{L}{2}\right) = -30.18 \quad > \quad k1 \cdot fck = -48$$

CHECK

$> \quad 0.4 \cdot fcm = -35.2$

elastic stress of top concrete chord for rare load combination

16.9 ULS checks

ULS BENDING-AXIAL CONTROL (§8.1)

$Mrd = 206.951$ kNm > $\dfrac{Me\_ULS\left(\frac{L}{2}\right)}{10^6} = 81.56$ CHECK

resisting moment calculated from moment-curvature diagram above

$-N = 4.078 \times 10^6$ N < $-Ac\cdot fcd + \left(\sum_{i=1}^{js} As_i\right)\cdot fsd = 5.493 \times 10^6$ N CHECK

MINIMUM REINFORCEMENT (§12.3)

$\max\left(0.1\cdot\frac{-N}{fsd}, 0.002\cdot Ac\right) = 897.16$ $As\_tot = 1.018 \times 10^3$ CHECK §12.3(1)

$smaxcol := \min(20\cdot 18, 400, 300) = 300$ > $s2 := 300$ mm CHECK stirrup spacing out of joint/bracket/lap area §12.3(3)

$smaxcolred := 0.6\cdot smaxcol = 180$ > $s1 := 150$ mm CHECK stirrup spacing within joint/bracket/lap area

NEAR LAPS AND JOINTS

ANCHORAGE (§11.4)

$klb := 50$

$kcp := 1$ for good bond conditions

$n\sigma := \frac{3}{2}$

$cs := 50$

$cx := 75$

$cy := 40$

$cd(\phi) := \min(0.5 \cdot cs, cx, cy, 3.75 \cdot \phi)$ $cd(12) = 25$

$$lbd(\phi) := \max\left[klb \cdot kcp \cdot \phi \cdot \left(\frac{fsd}{435}\right)^{n\sigma} \cdot \left(\frac{25}{-fck}\right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20}\right)^{\frac{1}{3}} \cdot \left(\frac{1.5 \cdot \phi}{cd(\phi)}\right)^{\frac{1}{2}}, 10 \cdot \phi\right]$$

$lbd(18) = 539.211$

ANCHORAGE OF JOINT REBARS IN M80 MORTAR

$fcm := -80 \cdot 0.83$ $fck := fcm + 8 = -58.4$

$fmtd := 1.72$ MPa same formula as concrete was used (on the safe side)

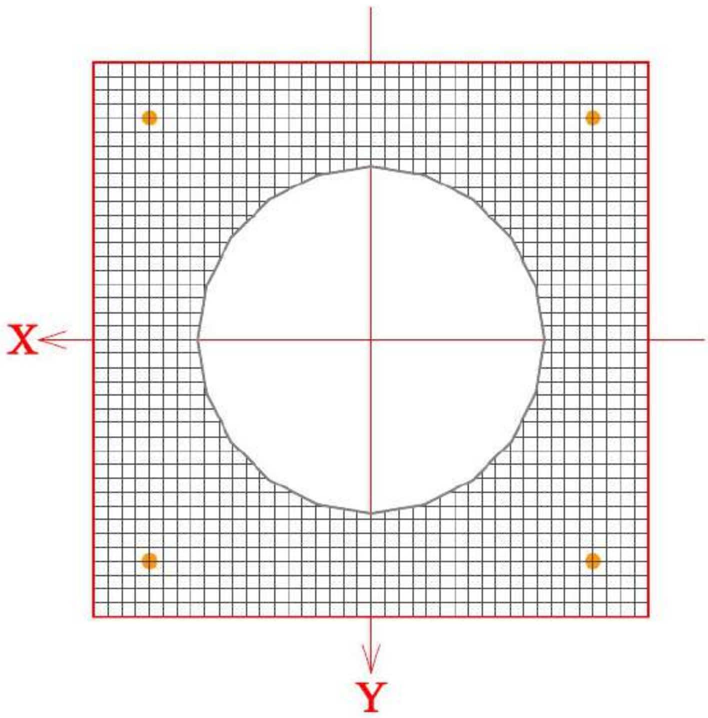
$lbd(30) \cdot \frac{fctd}{fmtd} = 1.516 \times 10^{3}$

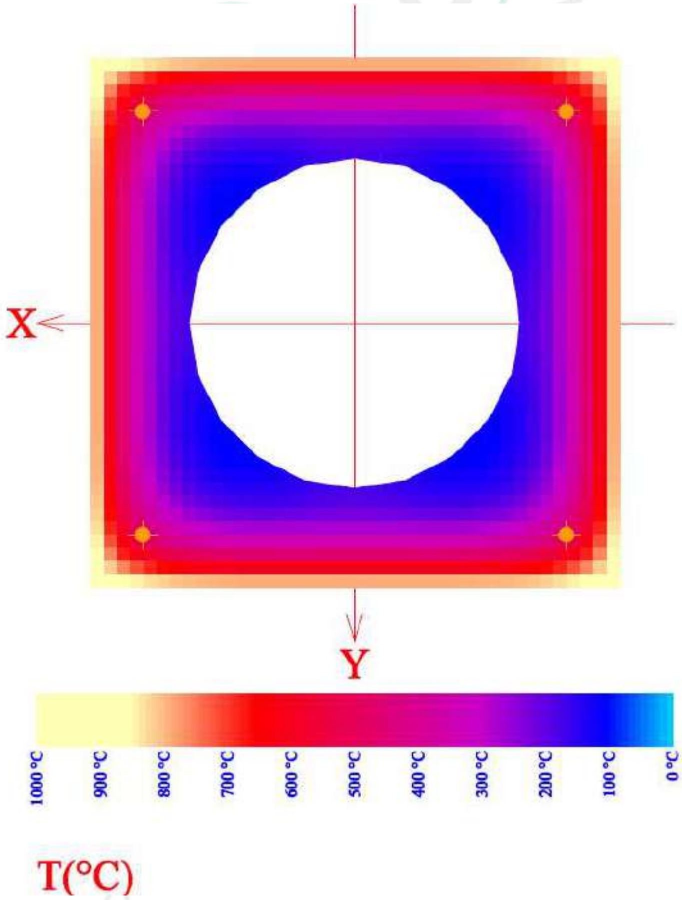
length of straight part for 90° bent bars

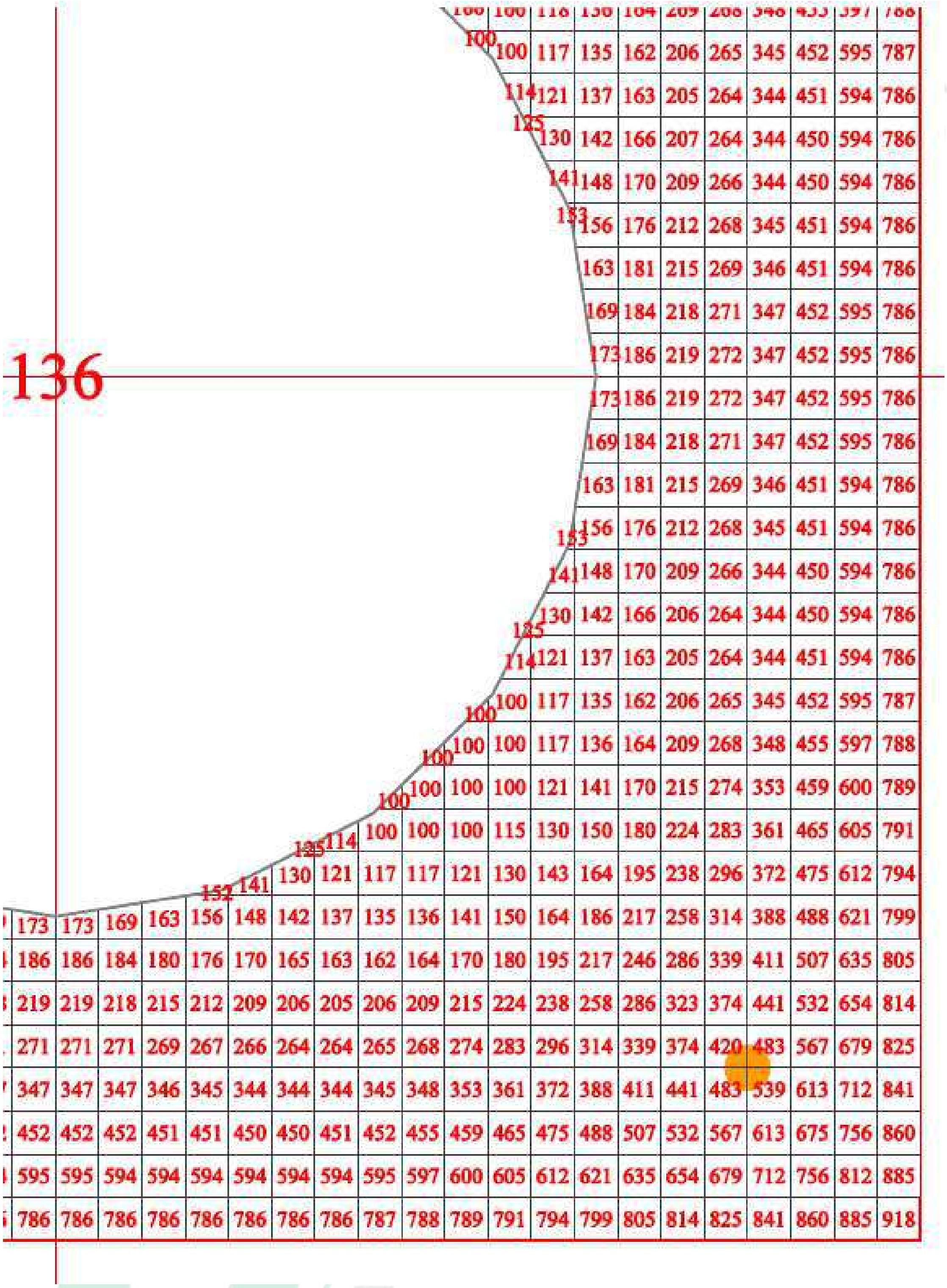
$lb90(\phi) := \max(70, lbd(\phi) - 15 \cdot \phi, 10 \cdot \phi)$

$lb90(12) = 120$ $lb90(8) = 80$

16.10 Fire checks






100 100 117 135 162 206 265 345 452 595 787
114 121 137 163 205 264 344 451 594 786
125 130 142 166 207 264 344 450 594 786
141 148 170 209 266 344 450 594 786
153 156 176 212 268 345 451 594 786
163 181 215 269 346 451 594 786
169 184 218 271 347 452 595 786
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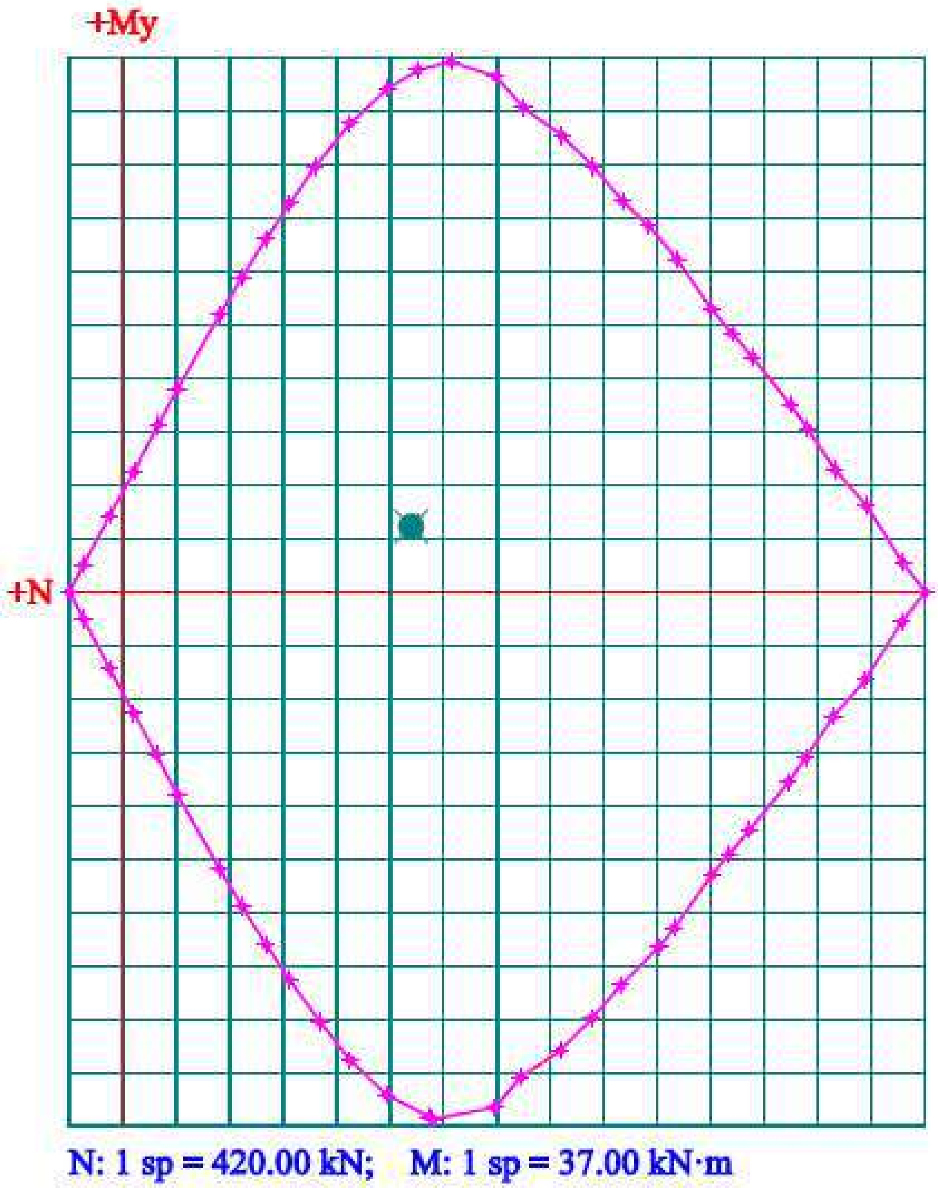
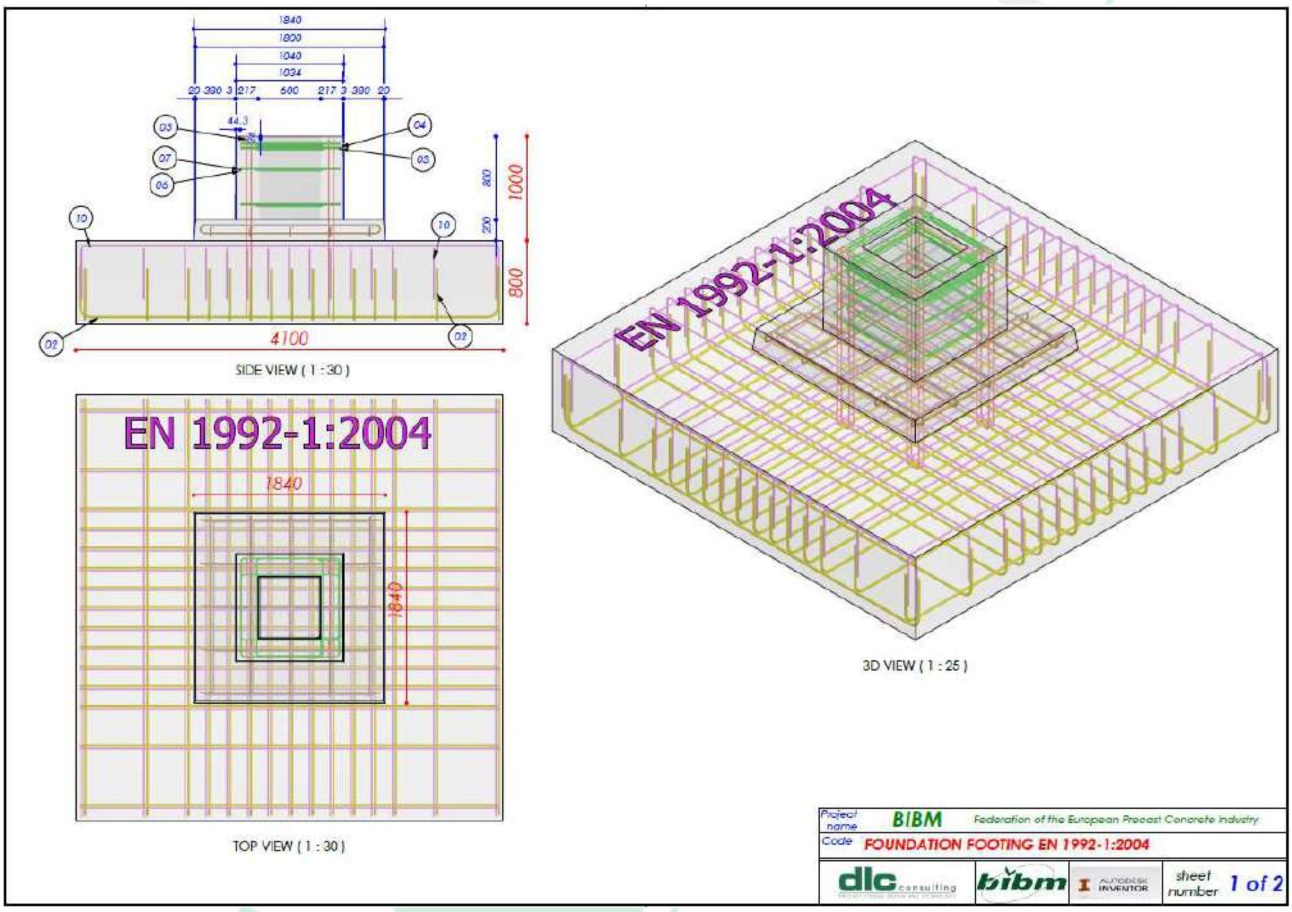
136
173 186 219 272 347 452 595 786
169 184 218 271 347 452 595 786
163 181 215 269 346 451 594 786
153 156 176 212 268 345 451 594 786
141 148 170 209 266 344 450 594 786
125 130 142 166 206 264 344 450 594 786
114 121 137 163 205 264 344 451 594 786
100 100 117 135 162 206 265 345 452 595 787
100 100 100 117 136 164 209 268 348 455 597 788
100 100 100 121 141 170 215 274 353 459 600 789
100 100 100 100 115 130 150 180 224 283 361 465 605 791
125 114 130 121 117 117 121 130 143 164 195 238 296 372 475 612 794
152 141
173 173 169 163 156 148 142 137 135 136 141 150 164 186 217 258 314 388 488 621 799
186 186 184 180 176 170 165 163 162 164 170 180 195 217 246 286 339 411 507 635 805
219 219 218 215 212 209 206 205 206 209 215 224 238 258 286 323 374 441 532 654 814
271 271 271 269 267 266 264 264 265 268 274 283 296 314 339 374 420 483 567 679 825
347 347 347 346 345 344 344 344 345 348 353 361 372 388 411 441 483 539 613 712 841
452 452 452 451 451 450 450 451 452 455 459 465 475 488 507 532 567 613 675 756 860
595 595 594 594 594 594 594 594 595 597 600 605 612 621 635 654 679 712 756 812 885
786 786 786 786 786 786 786 786 787 788 789 791 794 799 805 814 825 841 860 885 918



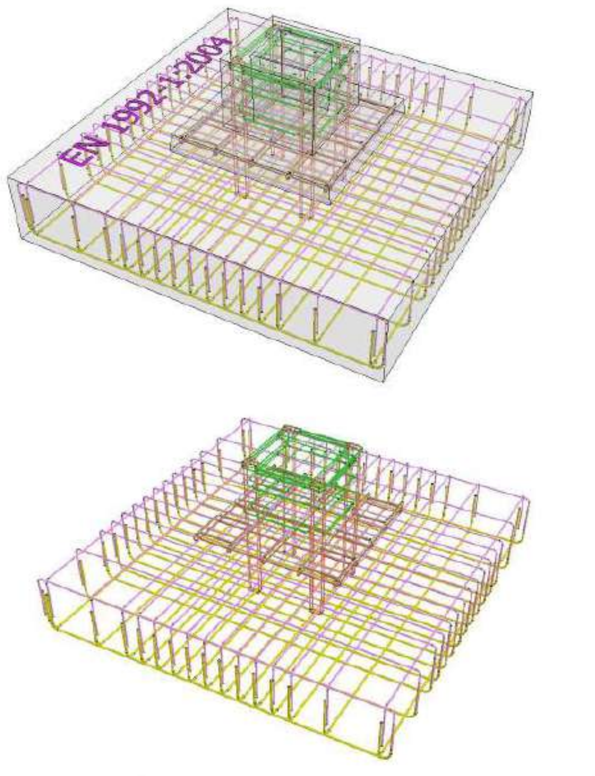
Nz =   0.00 kN
My+ =   69.45 kN·m
My- =   -69.90 kN·m

17 Foundation footing element –EN1992-1:2004

17.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_		
	02	31	17364	538284	24 mm		
	03	6	4088	24528	14 mm		
	04	3	4857	14571	14 mm		
	05	8	4426	35408	14 mm		
	06	2	3060	6120	12 mm		
	07	1	3624	3624	12 mm		
	08	10	4248	42480	16 mm		
	09	10	2588	25880	16 mm		
	10	30	7819	234570	16 mm		
Total mass rebars [kg]				925,47	Incidence kg/m³ 62,97		
	Total mass of steel [kg]			925,47		Concrete volume [m³]	1.248
						Cast in situ [m³]	13.448
						Total concrete [m³]	14.696



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dlc consulting / bibm / AUTODESK INVENTOR	sheet number	2 of 2

17.2 Definition of concrete geometry and material properties

$fck := -25$ MPa characteristic compressive strength of cast-in-situ concrete

$\gamma cpcred := 1.4$ partial safety coefficient for concrete

$$fcd := \frac{fck}{\gamma cpcred} = -17.857 \quad \text{MPa}$$ design compressive strength of cast-in-situ concrete

$$\nu := 0.6\cdot\left(1 + \frac{fck}{250}\right) = 0.54$$ §6.10

$fsk := 500$ MPa characteristic yield strength of mild steel

$\gamma sred := 1.1$ partial safety coefficient for mild steel

$NEd := 4100000$ N Ultimate Limit State (ULS) axial load from column

$L := 4100$ mm side of base square footing

$Lpocket := 1040$ mm side of precast pocket

17.3 Soil bearing stress check

SOIL BEARING STRESS (Winkler soil model with rigid foundation)

$\sigma g := 0.25$ MPa $\quad > \quad \frac{NEd}{L^2} = 0.244$ MPa CHECK

assumed maximum bearing stress of soil

shape of critical perimeter for punching shear (§6.4)

$Hbase := 200$ mm

$Hfound := 800$ mm

$d := Hfound - 55 = 745$ mm

$u := Lpocket \cdot 4 + 2 \cdot d \cdot 2 \cdot \pi = 1.352 \times 10^4$ mm critical perimeter below pocket

$4 \cdot d + 1040 = 4.02 \times 10^3$ mm $<$ $L = 4.1 \times 10^3$ mm

$4 \cdot d + \sqrt{2} \cdot 1040 = 4.451 \times 10^3$ mm $<$ $L \cdot \sqrt{2} = 5.798 \times 10^3$ mm

critical perimeter is inscribed into foundation base CHECK

17.4 Flexural reinforcement design

$$M := \sigma g \cdot L \cdot \frac{\left[\frac{(L - Lpocket)}{2}\right]^2}{2} = 1.2 \times 10^9 \text{ Nmm} < 12 \cdot \pi \cdot \frac{24^2}{4} \cdot 0.9 \cdot (d - Hbase) \cdot \frac{fsk}{\gamma sred} = 1.21 \times 10^9 \text{ Nmm}$$

CHECK

PUNCHING SHEAR AT COLUMN CONTROL PERIMETER

17.5 Punching shear

CHECK

PUNCHING SHEAR AT COLUMN CONTROL PERIMETER

$$\beta := 1 + 1.8\cdot\sqrt{\left(\frac{20}{400}\right)^2\cdot 2} = 1.127$$ §6.4.3

$$vEd\_A := \frac{\beta\cdot NEd\cdot\left[1 - \frac{(8\cdot d + Lpocket)\cdot Lpocket + \pi\cdot(2\cdot d)^2}{L^2}\right]}{Lpocket\cdot 4\cdot d} = 0.227$$ MPa §6.38

$< \quad 0.4\cdot\nu\cdot -fcd = 3.857$ MPa CHECK

§6.4.5(3)

PUNCHING SHEAR AT EXTERNAL CONTROL PERIMETER

$$vEd\_u := \frac{\beta\cdot NEd\cdot\left[1 - \frac{(8\cdot d + Lpocket)\cdot Lpocket + \pi\cdot(2\cdot d)^2}{L^2}\right]}{u\cdot d} = 0.07$$ MPa §6.38

$$\rho l := \min\left[0.02, 0\cdot\pi\cdot\frac{24^2}{4\cdot Hfound\cdot(Lpocket + 6\cdot d)}\right] = 0$$ additional reinforcement ratio for dowel effect

$\sigma cp := 0$ MPa

$$kv := \min\left(1 + \frac{200}{d}, 2\right) = 1.268$$

$k1v := 0.1$

$$Crdc := \frac{0.18}{\gamma cpcred} = 0.129$$

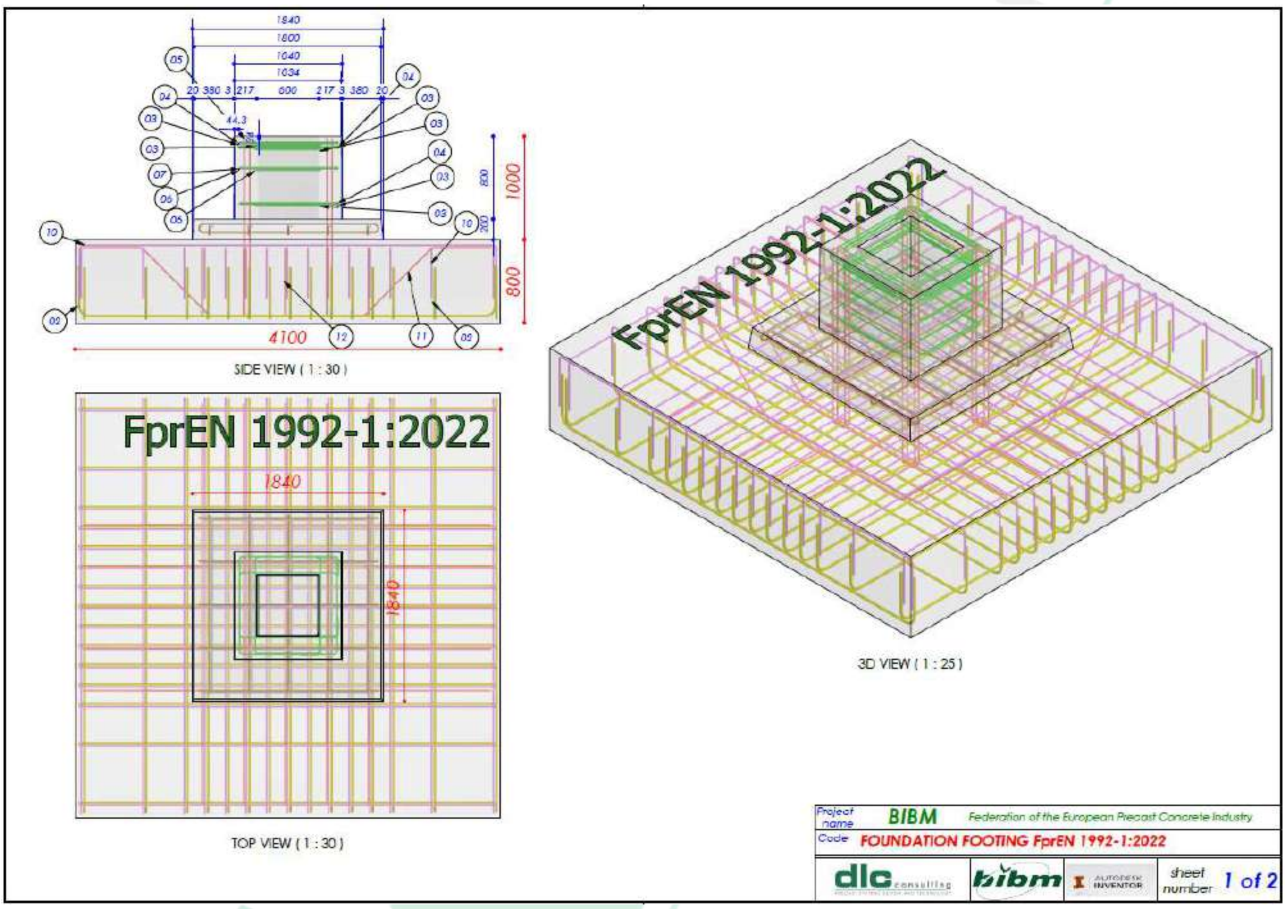
$$vmin := 0.035\cdot kv^{\frac{3}{2}}\cdot(-fck)^{\frac{1}{2}} = 0.25$$ MPa

$$vRdc := \max\left[\left[Crdc\cdot kv\cdot(100\cdot\rho l\cdot -fck)^{\frac{1}{3}} + k1v\cdot\sigma cp\right], (vmin + k1v\cdot\sigma cp)\right] = 0.25$$ MPa §6.47

$> \quad vEd\_u = 0.07$ CHECK

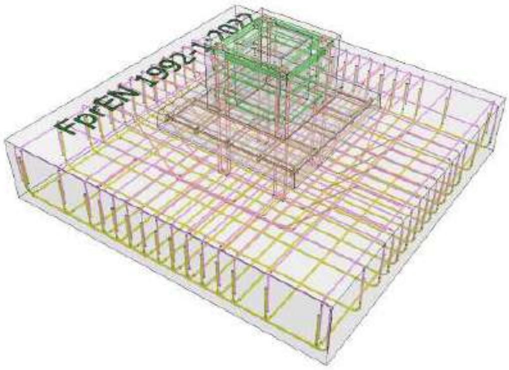
18 Foundation footing element – FprEN1992-1:2022

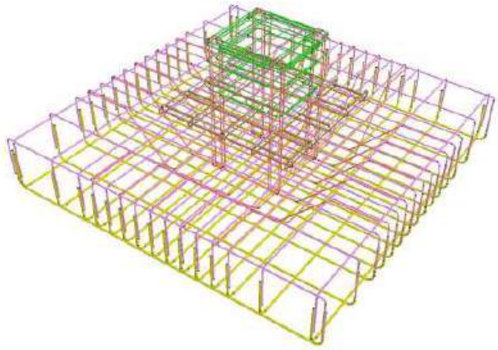
18.1 Shop drawings



Thumbnail	Part Number	QTY	Mass	Total mass	Ø_
	02	30	17364	520920	24 mm
	03	6	4088	24528	14 mm
	04	3	4857	14571	14 mm
	05	8	4426	35408	14 mm
	06	2	3060	6120	12 mm
	07	1	3624	3624	12 mm
	08	10	4248	42480	16 mm
	09	10	2588	25880	16 mm
	10	30	7819	234570	16 mm
	11	5	7048	35240	16 mm
	12	5	6922	34610	16 mm

Total mass rebars [kg]	977,95	Incidence kg/m³	66,55
Total mass of steel [kg]	977,95	Concrete volume [m³]	1,248
		Cast in situ [m³]	13,448
		Total concrete [m³]	14,696





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Code: **FOUNDATION FOOTING FprEN 1992-1:2022**

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18.2 Definition of concrete geometry and material properties

$fck := -25$ MPa characteristic compressive strength of cast-in-situ concrete

$\gamma c := 1.5$ partial safety coefficient for concrete

$fcd := \frac{fck}{\gamma c} = -16.667$ MPa design compressive strength of cast-in-situ concrete

$\gamma v := 1.3$ partial safety coefficient for concrete in shear

$ddg := 32$ mm

$\nu := 0.6 \cdot \left(1 + \frac{fck}{250}\right) = 0.54$

$fsk := 500$ MPa characteristic yield strength of mild steel

$\gamma sred := 1.1$ partial safety coefficient for mild steel

$fywd := \frac{fsk}{\gamma sred} = 454.545$ MPa design yield strength of mild steel web reinforcement

$NEd := 4100000$ N Ultimate Limit State (ULS) axial load from column

$L := 4100$ mm side of base square footing

$Lpocket := 1040$ mm side of precast pocket

18.3 Soil bearing stress check

SOIL BEARING STRESS (Winkler soil model with rigid foundation)

$\sigma g := 0.25$ MPa $>$ $\frac{NEd}{L^2} = 0.244$ MPa CHECK

assumed maximum bearing stress of soil

shape of critical perimeter for punching shear (§8.4)

$Hbase := 200$ mm

$Hfound := 800$ mm

$dv := Hfound - 55 = 745$ mm

$b05 := Lpocket \cdot 4 + 0.5 \cdot dv \cdot 2 \cdot \pi = 6.5 \times 10^3$ mm

$1 \cdot dv + 1040 = 1.785 \times 10^3$ mm $<$ $L = 4.1 \times 10^3$ mm

$1 \cdot dv + \sqrt{2} \cdot 1040 = 2.216 \times 10^3$ mm $<$ $L \cdot \sqrt{2} = 5.798 \times 10^3$ mm

critical perimeter is inscribed into foundation base CHECK

18.4 Flexural reinforcement design

FLEXURAL REINFORCEMENT

$$M := \sigma g \cdot L \cdot \frac{\left[\frac{(L - Lpocket)}{2}\right]^2}{2} = 1.2 \times 10^9 \quad \text{Nmm} \quad < \quad 12 \cdot \pi \cdot \frac{24^2}{4} \cdot 0.9 \cdot (dv - Hbase) \cdot \frac{fsk}{\gamma sred} = 1.21 \times 10^9 \ \text{Nmm}$$

CHECK

18.5 Punching shear

GLOBAL MECHANISM

PUNCHING SHEAR VERIFICATION NEED

$$\tau Rdc\_min := \frac{11}{\gamma v} \cdot \sqrt{\frac{-fck \cdot \gamma sred}{fsk} \cdot \frac{ddg}{dv}} = 0.401 \quad \text{MPa}$$

$$\beta := 1 + 1.8 \cdot \sqrt{\left(\frac{20}{400}\right)^2 \cdot 2} = 1.127$$

$$\tau Ed\_A := \frac{\beta \cdot NEd \cdot \left[1 - \frac{(2.68 \cdot dv + Lpocket) \cdot Lpocket + \pi \cdot (0.67 \cdot dv)^2}{L^2}\right]}{Lpocket \cdot 4 \cdot dv} = 1.067 \quad \text{MPa}$$

$< \quad \tau Rdc\_min = 0.401$ MPa CHECK

if not -> calculation needed

PUNCHING SHEAR AT EXTERNAL CONTROL PERIMETER

$$\tau Ed\_u := \frac{\beta \cdot NEd \cdot \left[1 - \frac{(2.68 \cdot dv + Lpocket) \cdot Lpocket + \pi \cdot (0.67 \cdot dv)^2}{L^2}\right]}{b05 \cdot dv} = 0.67 \quad \text{MPa}$$

$$b0 := 4 \cdot Lpocket = 4.16 \times 10^3 \quad \text{mm}$$

$$kpb := if\left(3.6 \cdot \sqrt{1 - \frac{b0}{b05}} < 1, 1, if\left(3.6 \cdot \sqrt{1 - \frac{b0}{b05}} > 2.5, 2.5, 3.6 \cdot \sqrt{1 - \frac{b0}{b05}}\right)\right) = 2.196$$

$$\rho l := min\left[0.02, 12 \cdot \pi \cdot \frac{24^2}{4 \cdot Hfound \cdot (Lpocket + 6 \cdot dv)}\right] = 1.124 \times 10^{-3}$$

$$\tau Rdc := min\left[\frac{0.6}{\gamma v} \cdot kpb \cdot \left(100 \cdot \rho l \cdot -fck \cdot \frac{ddg}{dv}\right)^{\frac{1}{3}}, \frac{0.5}{\gamma v} \cdot \sqrt{-fck}\right] = 0.492 \quad \text{MPa}$$

$> \quad \tau Ed\_u = 0.67$ CHECK

$$\frac{0.5}{\gamma v} \cdot \sqrt{-fck} = 1.923$$

if not -> punching shear reinforcement is needed

PUNCHING SHEAR REINFORCEMENT CALCULATION

$\phi w := 16$ mm

$n\phi w := 5$

$Asw := n\phi w \cdot \pi \cdot \frac{\phi w^2}{4} = 1.005 \times 10^3$

$st := \frac{2 \cdot 0.67 \cdot dv + Lpocket}{n\phi w} = 418.38$

$\alpha w := \frac{\pi}{4}$

$\rho w := Asw \cdot \frac{\sin(\alpha w)}{dv \cdot st} = 2.164 \times 10^{-3}$

$\eta c := \frac{\tau Rdc}{\tau Ed\_u} = 0.735$

$\eta s := \max\left[0.8, \frac{dv}{150 \cdot \phi w} + \left(15 \cdot \frac{ddg}{dv}\right)^{\frac{1}{2}} \cdot \left(\frac{1}{\eta c \cdot kpb}\right)^{\frac{3}{2}}\right] = 0.8$

$\tau Rdcs := \min[\rho w \cdot fywd, \eta c \cdot \tau Rdc + \eta s \cdot (\rho w \cdot fywd)] = 0.984$ MPa $>$ $\tau Ed\_u = 0.67$ CHECK

$\rho w \cdot fywd = 0.984$

$\eta sys := \max\left[1, 0.5 + 0.63 \cdot \left(\frac{b0}{dv}\right)^{\frac{1}{4}}\right] = 1.456$

$\tau Rdmax := \eta sys \cdot \tau Rdc = 0.717$ MPa $>$ $\tau Ed\_u = 0.67$ CHECK

$cv := 80$ mm

$dvout := dv - cv = 705$ mm

$b05out := b05 \cdot \left(\frac{dv}{dvout \cdot \eta c}\right)^2 = 1.521 \times 10^4$ mm

$\frac{(b05out - 4 \cdot Lpocket)}{2 \cdot \pi} = 1.758 \times 10^3$ mm

extension of the punching shear reinforcement from the control section -> absurdely large and not taking into account the ground reaction

STEP MECHANISM

PUNCHING SHEAR AT STEP CONTROL PERIMETER

$Lbase := Lpocket + 4 \cdot Hbase = 1.84 \times 10^{3}$ mm

$dv2 := dv - Hbase = 585$ mm

$\tau Rdc\_min2 := \frac{11}{\gamma v} \cdot \sqrt{\frac{-fck \cdot \gamma sred}{fsk} \cdot \frac{ddg}{dv2}} = 0.464$ MPa

$b05\_2 := \min(3 \cdot dv2 \cdot 4 + 0.5 \cdot dv2 \cdot 2 \cdot \pi, Lbase \cdot 4 + 0.5 \cdot dv2 \cdot 2 \cdot \pi) = 8.858 \times 10^{3}$ mm

$$\tau Ed\_A2 := \frac{\beta \cdot NEd \cdot \left[1 - \frac{(2.68 \cdot dv + Lbase) \cdot Lbase + \pi \cdot (0.67 \cdot dv2)^{2}}{L^{2}}\right]}{Lbase \cdot 4 \cdot dv} = 0.432 \quad \text{MPa}$$

$< \quad \tau Rdc\_min2 = 0.464$ MPa CHECK

if not -> calculation needed

PUNCHING SHEAR AT EXTERNAL CONTROL PERIMETER

$$\tau Ed\_u2 := \frac{\beta \cdot NEd \cdot \left[1 - \frac{(2.68 \cdot dv2 + Lbase) \cdot Lbase + \pi \cdot (0.67 \cdot dv2)^{2}}{L^{2}}\right]}{b05\_2 \cdot dv2} = 0.534 \quad \text{MPa}$$

$b02 := 4 \cdot Lbase = 7.36 \times 10^{3}$ mm

$$kpb2 := if\left(3.6 \cdot \sqrt{1 - \frac{b02}{b05\_2}} < 1, 1, if\left(3.6 \cdot \sqrt{1 - \frac{b02}{b05\_2}} > 2.5, 2.5, 3.6 \cdot \sqrt{1 - \frac{b02}{b05\_2}}\right)\right) = 1.48$$

$$\rho l2 := \min\left[0.02, 12 \cdot \pi \cdot \frac{24^{2}}{4 \cdot (Hfound - Hbase) \cdot (Lbase + 6 \cdot dv2)}\right] = 1.585 \times 10^{-3}$$

$$\tau Rdc2 := \min\left[\frac{0.6}{\gamma v} \cdot kpb2 \cdot \left(100 \cdot \rho l2 \cdot -fck \cdot \frac{ddg}{dv2}\right)^{\frac{1}{3}}, \frac{0.5}{\gamma v} \cdot \sqrt{-fck}\right] = 0.41 \quad \text{MPa}$$

$> \quad \tau Ed\_u = 0.67$ CHECK

$\frac{0.5}{\gamma v} \cdot \sqrt{-fck} = 1.923$

if not -> punching shear reinforcement is needed

PUNCHING SHEAR REINFORCEMENT CALCULATION

$$st2 := \frac{2 \cdot 0.67 \cdot dv2 + Lpocket}{n\phi w} = 364.78$$

$$\rho w2 := Asw \cdot \frac{\sin(\alpha w)}{dv2 \cdot st2} = 3.331 \times 10^{-3}$$

$$\eta c2 := \frac{\tau Rdc2}{\tau Ed\_A2} = 0.951$$

$$\eta s2 := \max\left[0.8, \frac{dv2}{150 \cdot \phi w} + \left(15 \cdot \frac{ddg}{dv2}\right)^{\frac{1}{2}} \cdot \left(\frac{1}{\eta c2 \cdot kpb2}\right)^{\frac{3}{2}}\right] = 0.8$$

$\tau Rdcs2 := \min[\rho w \cdot fywd, \eta c2 \cdot \tau Rdc + \eta s2 \cdot (\rho w2 \cdot fywd)] = 0.984$ MPa $>$ $\tau Ed\_u = 0.67$ CHECK

$$\rho w2 \cdot fywd = 1.514$$

$$\eta sys2 := \max\left[1, 0.5 + 0.63 \cdot \left(\frac{b02}{dv2}\right)^{\frac{1}{4}}\right] = 1.687$$

$\tau Rdmax2 := \eta sys2 \cdot \tau Rdc2 = 0.692$ MPa $>$ $\tau Ed\_u = 0.67$ CHECK

19 Evaluation of environmental impact

19.1 Methodology

The evaluation of the environmental impact of the analysed structural members is preliminarily carried out considering the consumption of raw materials only. The analysis is carried out through the definition of environmental impact indexes. The environmental indexes considered in this study are, for the sake of conciseness, the compulsory ones as prescribed by the standard EN 15804:2012+A2:2019. Some parameters will be analysed in the following according to the most recurrent of the 10 indexes, i.e. Global-Warming Potential (GWP) in terms of mass of equivalent carbon dioxide associated to the structural bodies. The complete list of the voluntary Environmental Product Declaration (EPD) documents used in this calculation is given in the following table. The considered EPDs are emitted by certified material producers following the instructions of standards ISO 14025 and EN 15804:2012+A2:2019 currently valid.

MATERIAL	EPD	DENSITY (ton/m^3)	GWP ($kgCO_2eq/ton$)	GWP ($kgCO_2eq/m^3$)
CEMENT I 52,5 R	EPDITALY0042	3,15	**891**	2807
CEMENT IV 32,5 N	EPDITALY0042	3,15	**547**	1723
SAND + GRAVEL	EPDITALY0088	1,5	22,5	34
SILICA FUME	EPD636	1,1	52	57
SUPERPLASTICISER	S-P-04323	1,1	**504**	555
WATER	-	1	-	-
REINF BARS B500	EPDITALY0015	7,85	**809**	6351
STRANDS Y1860	S-P-05640	7,85	**2190**	17192
PP MICROFIBRES	MD-21074-EN	0,91	**1770**	1611

The analysis encompasses both the absolute impact of the single element and the specific impact of the elements, obtained by dividing the absolute impact by the influence area within the case study building. The latter value can give an approximate idea about the influence of the impact of the specific structural element over the whole building structure, to be usefully read in a comparative way among the different members.

The values of the influence areas are given in the following:

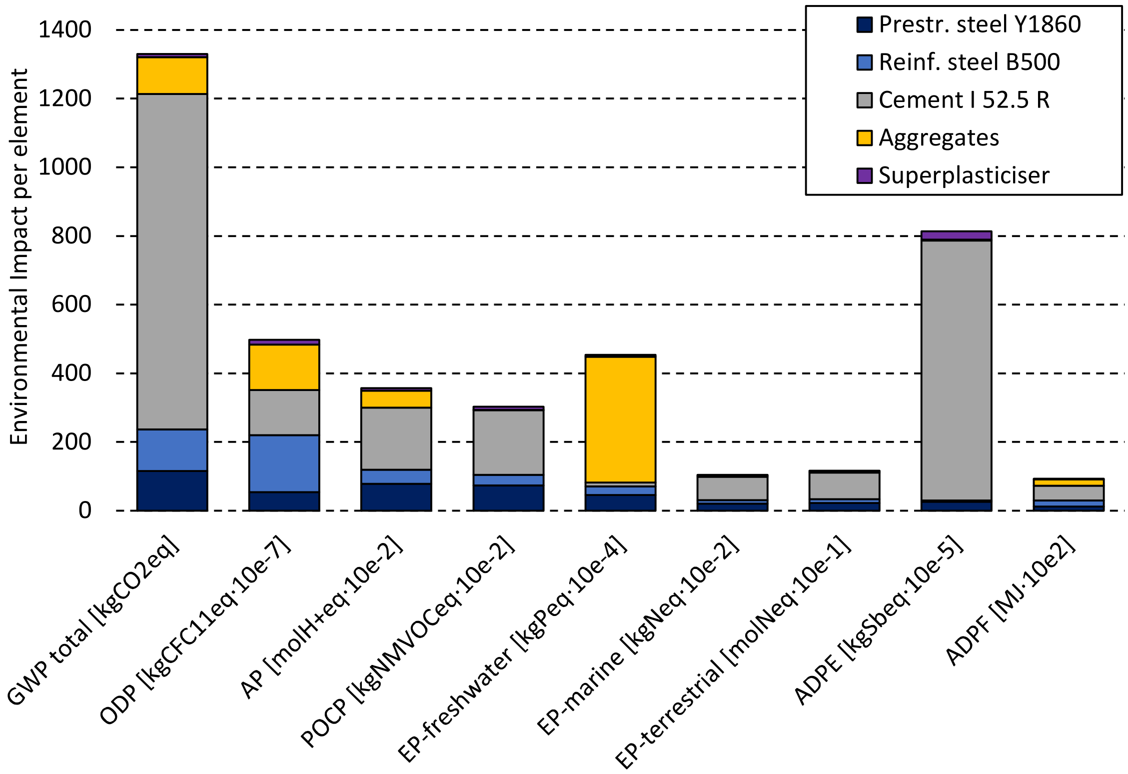
- TT floor element: 22.68 m^2
- Hollowcore floor element: 11.34 m^2
- Lattice girder floor element: 22.68 m^2
- Prestressed and reinforced central beam: 76.55 m^2
- Central column element: 273.38 m^2
- Foundation footing: 273.38 m^2

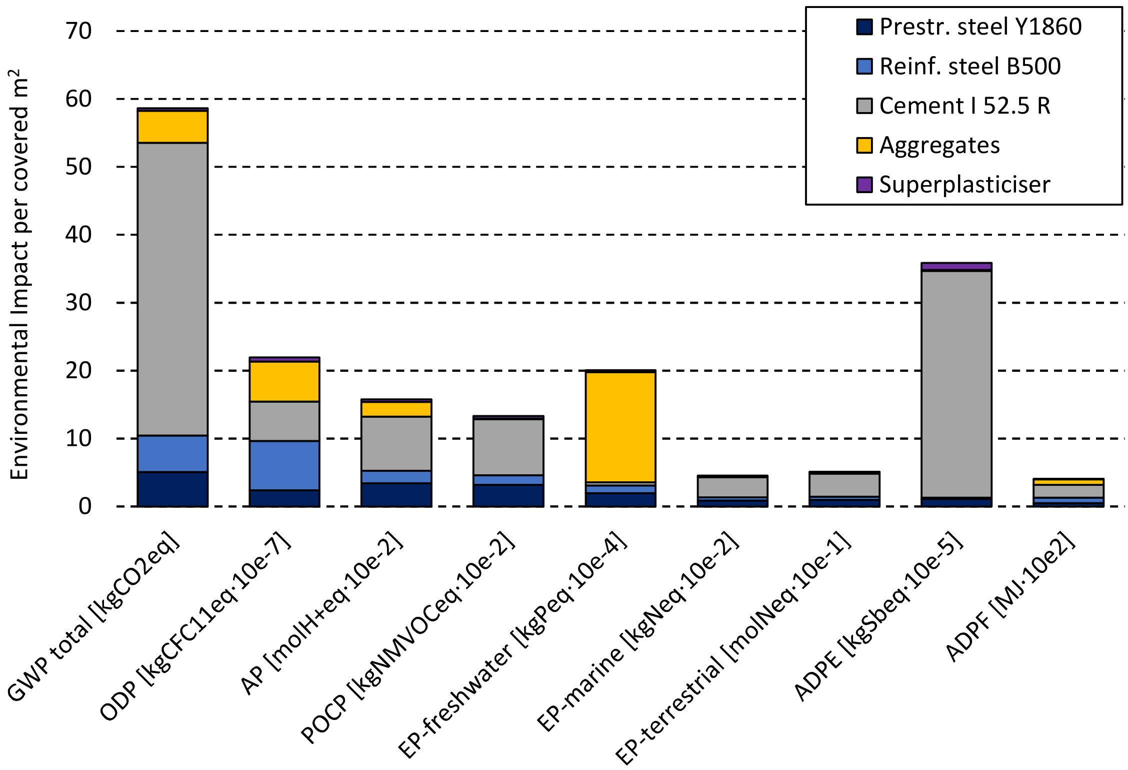
Concerning the different mix designs employed, a generic list used for the purpose of environmental impact evaluation is provided in the following:

	PRECAST (C80/95)	**PRECAST (C45/55)**	**CAST-IN-SITU (C25/30)**
CEM I 52,5 R	540	420	-
CEM IV/B 32,5 N	-	-	350
SAND + GRAVEL	1570	1820	1870
FILLER (SILICA FUME)	90	-	-
PP MICROFIBRES*	2	-	-
SUPERPLASTICISER	7	7	-
WATER	180	150	190
TOTAL (kg/m^3)	**2387**	**2396**	**2400**
W/C RATIO (-)	0,33	0,36	0,54

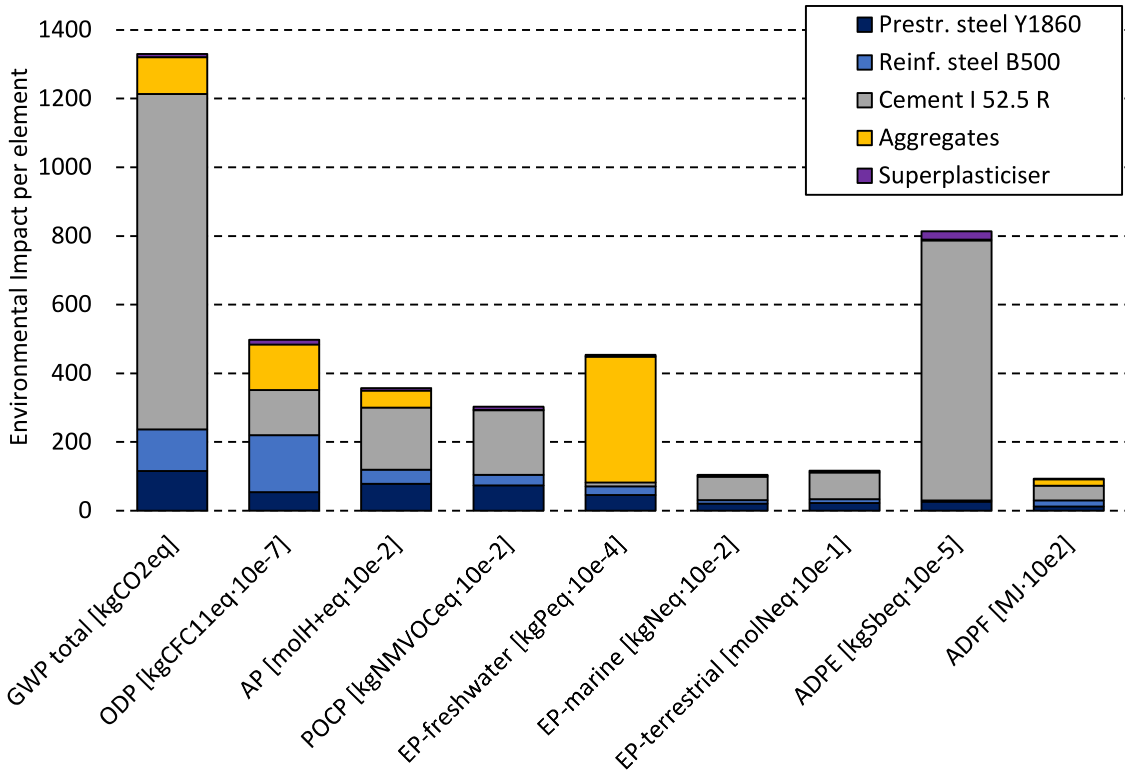
*only for column elements designed according to FprEN1992-1:2022

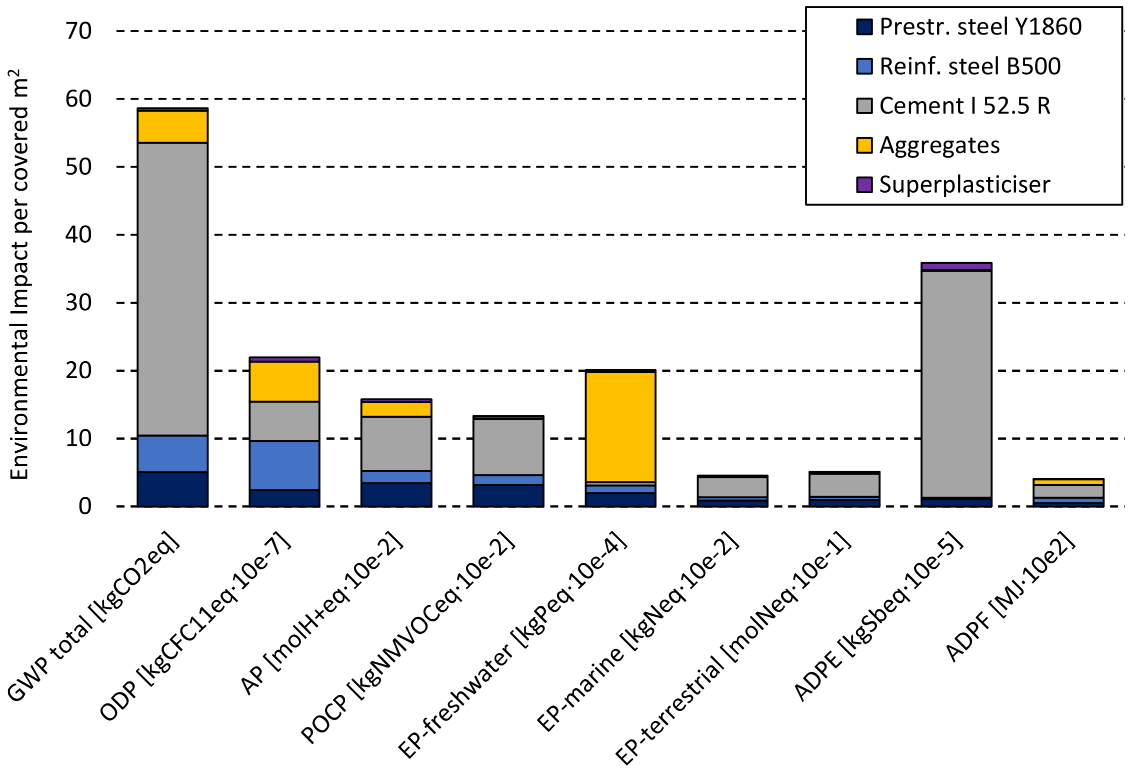
19.2 TT element – EN1992-1:2004



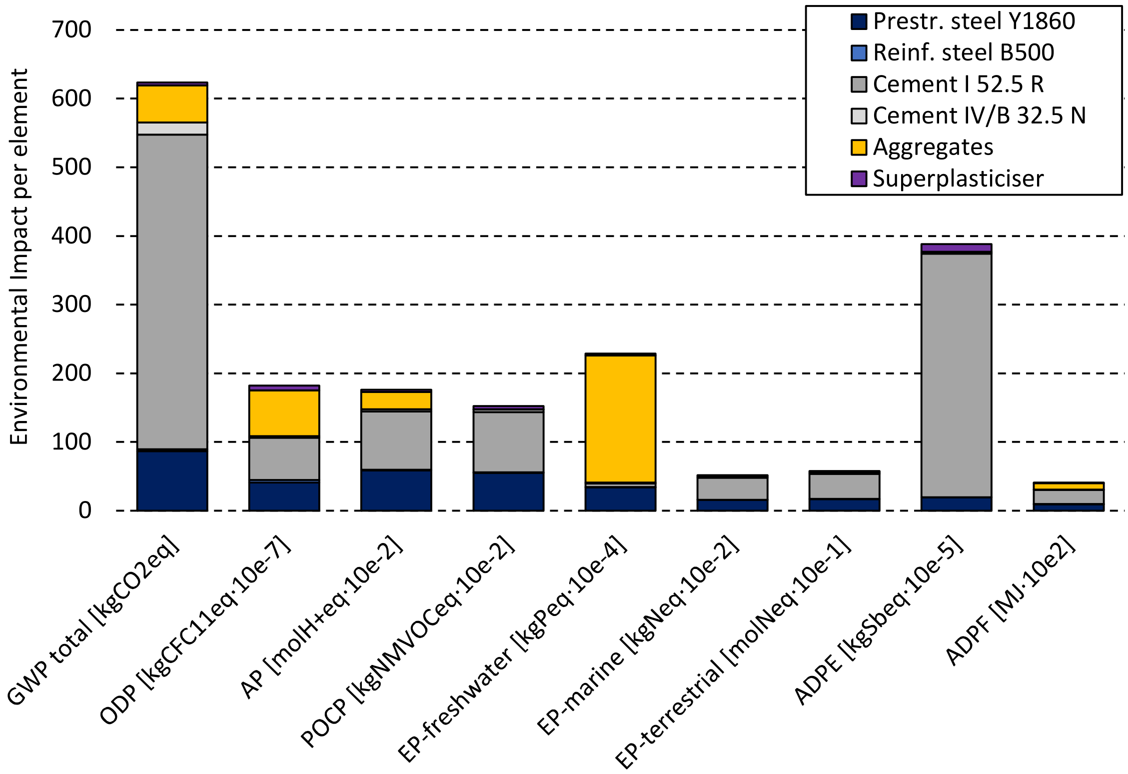


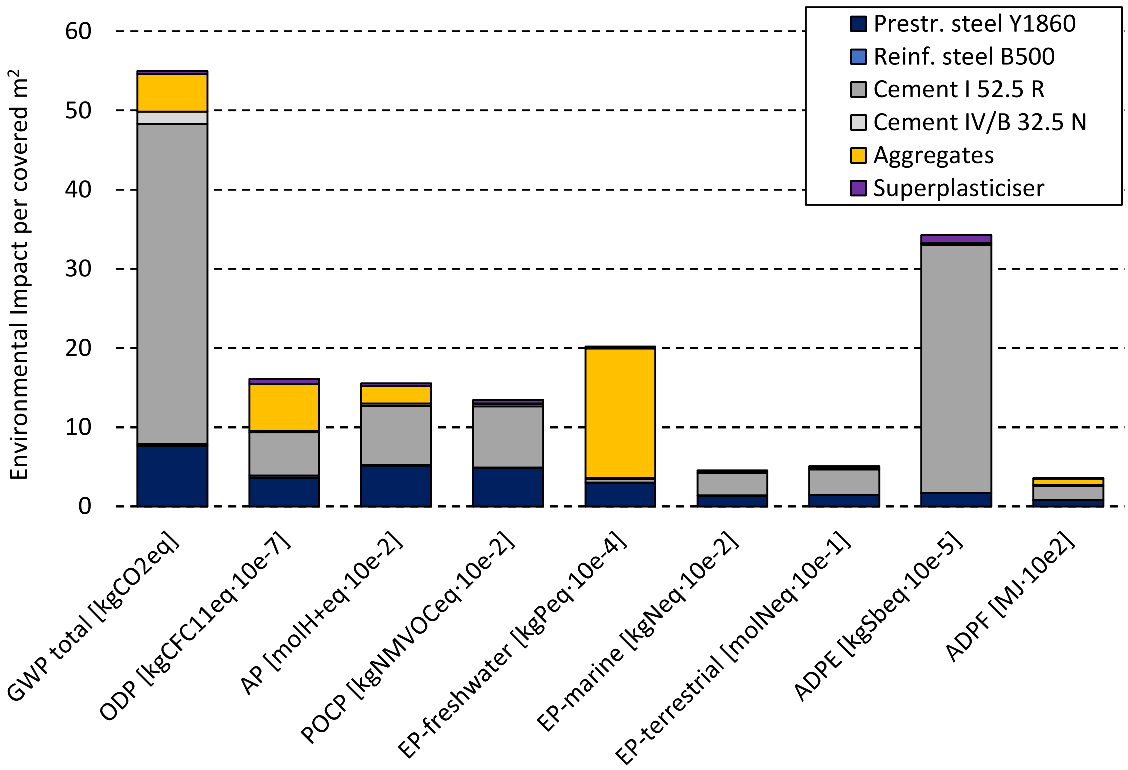
19.3 TT element – FprEN1992-1:2022



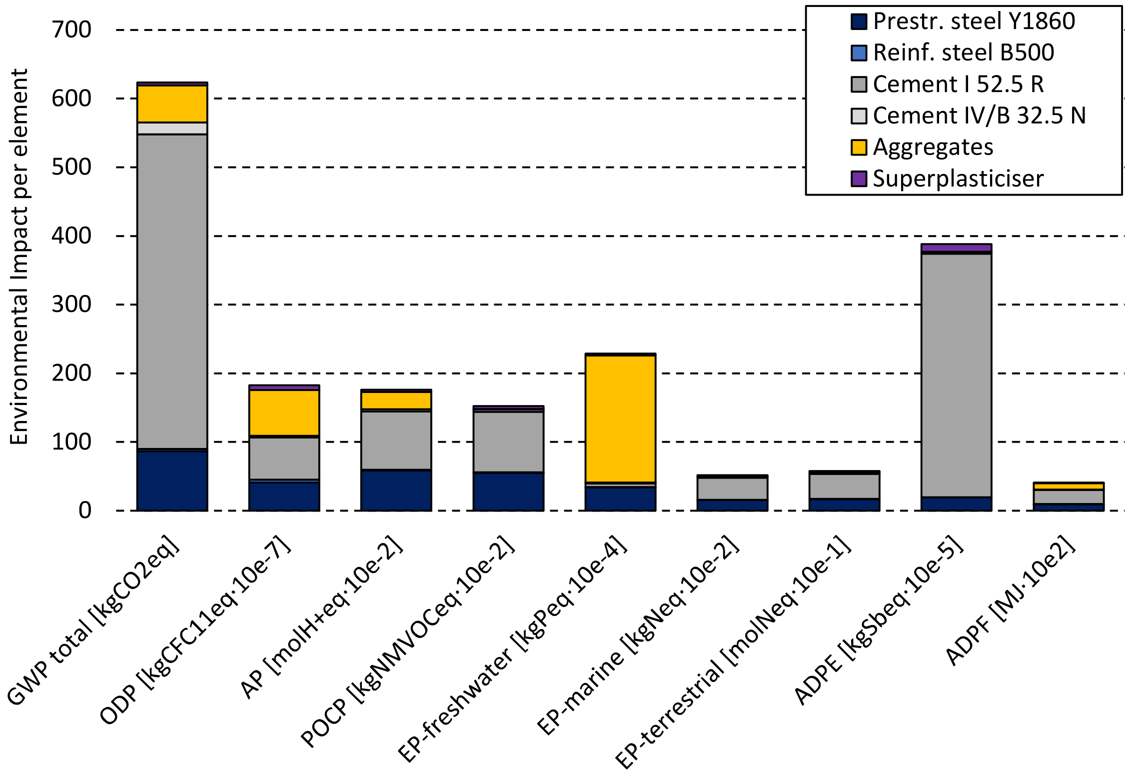


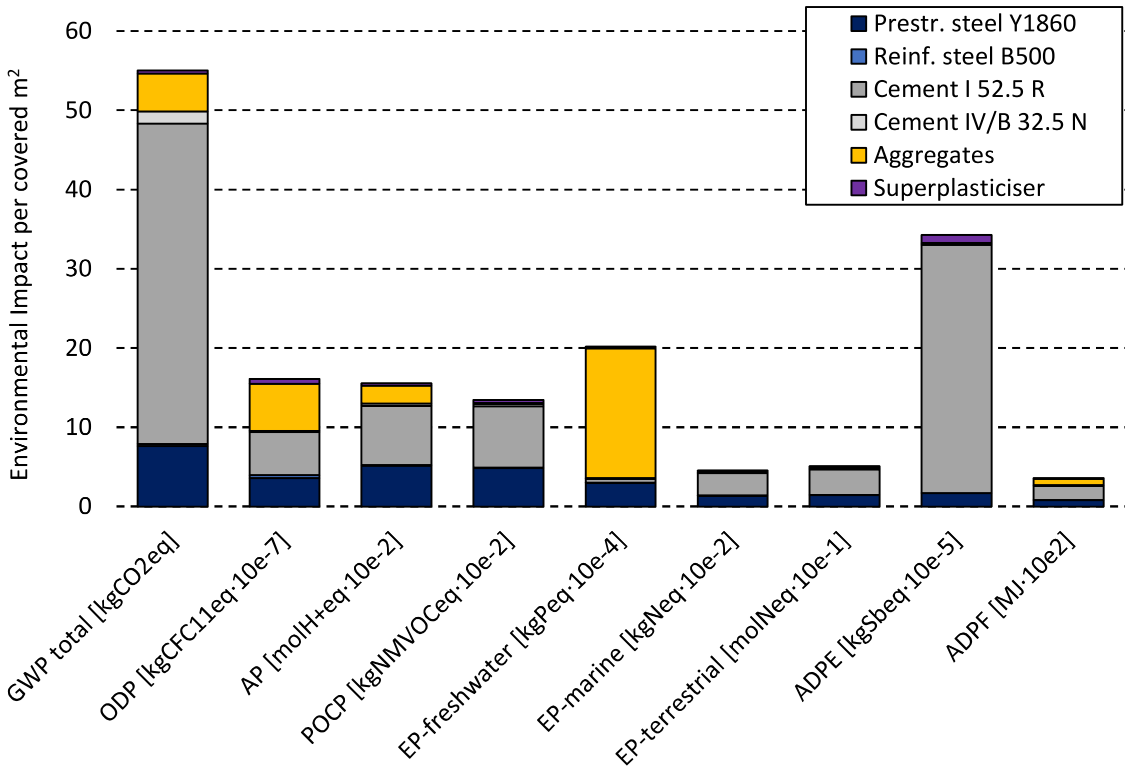
19.4 Hollowcore element –EN1992-1:2004



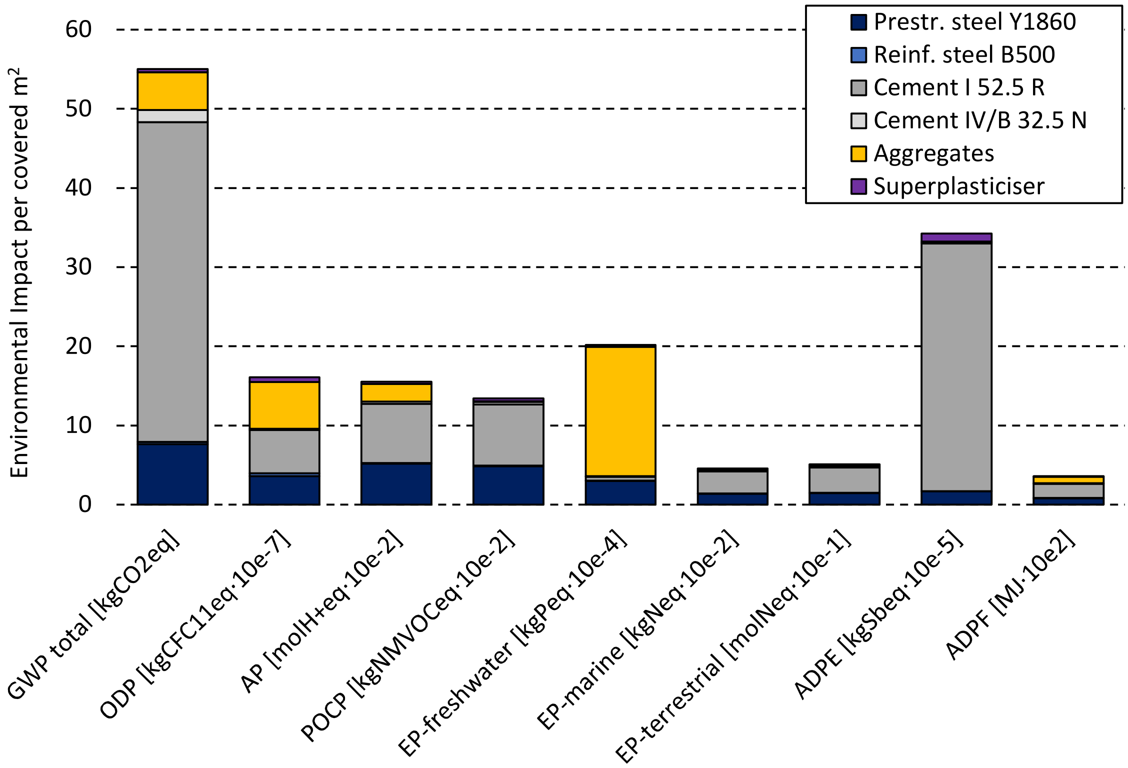


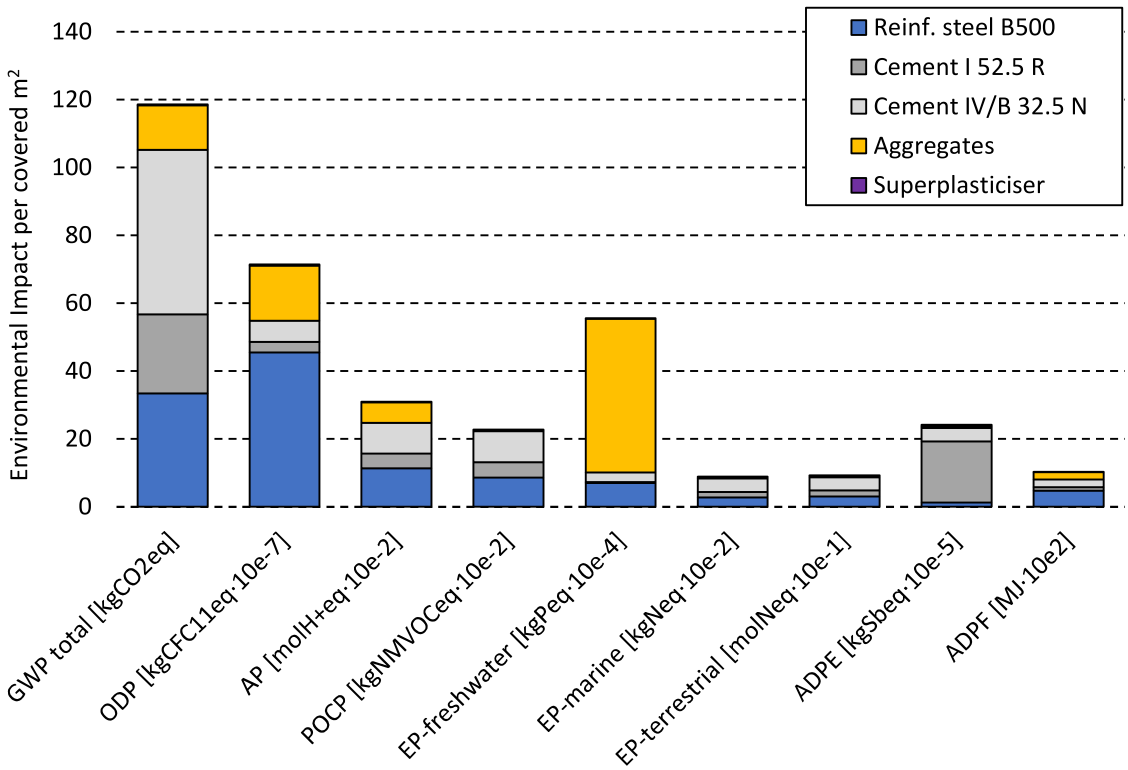
19.5 Hollowcore element – FprEN1992-1:2022



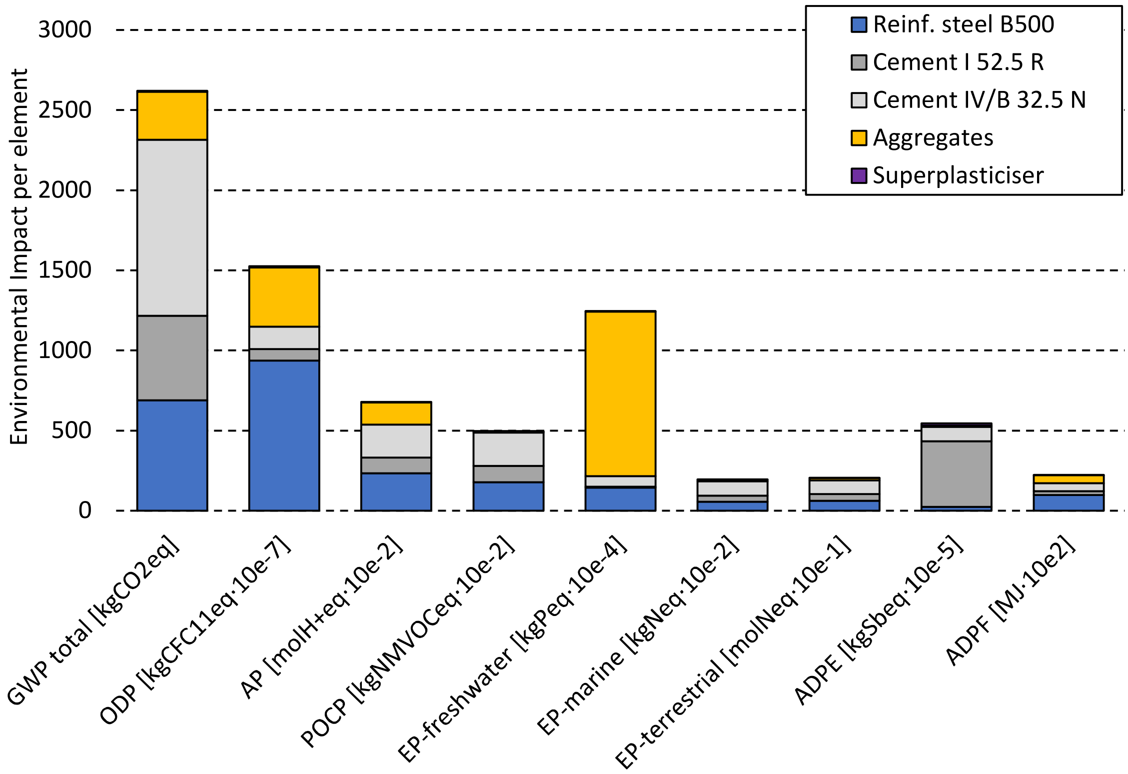


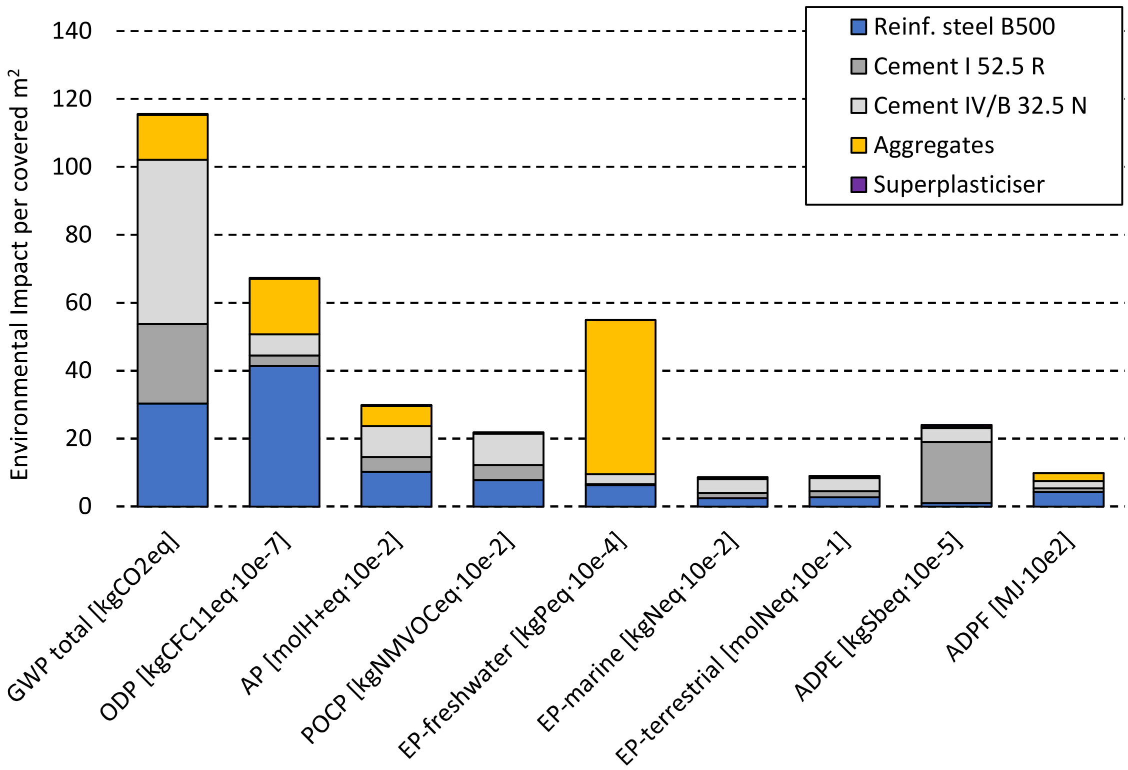
19.6 Lattice girder element –EN1992-1:2004



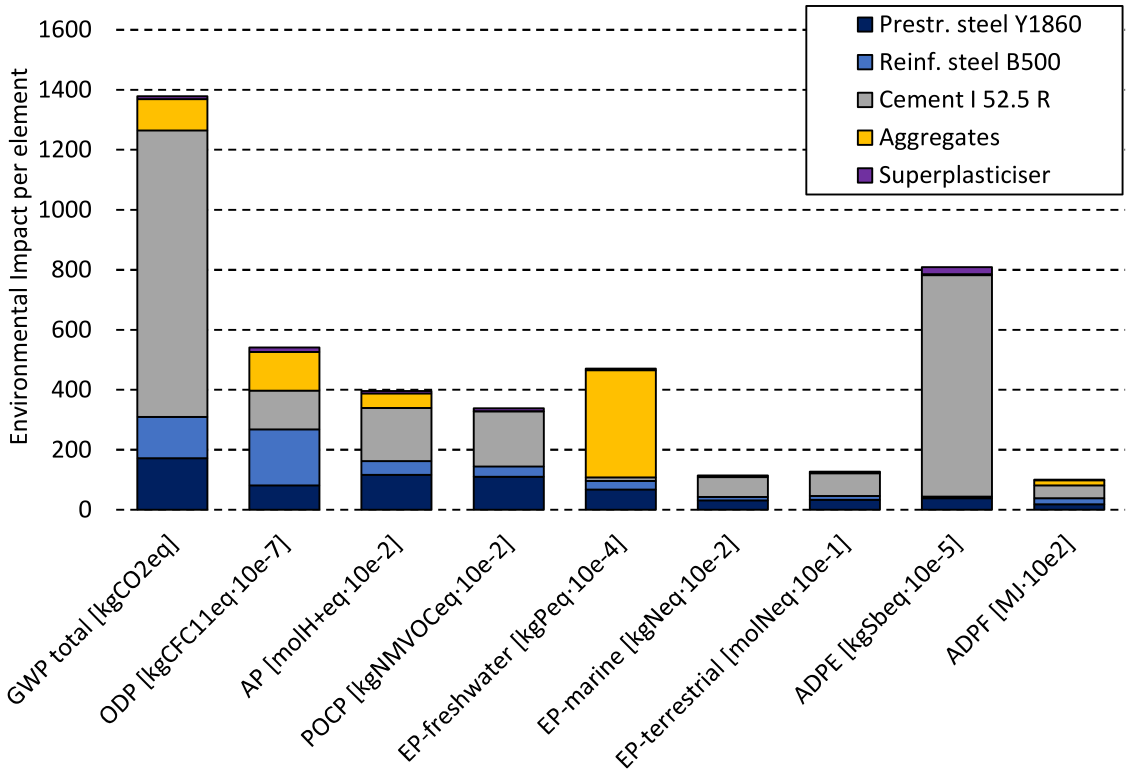


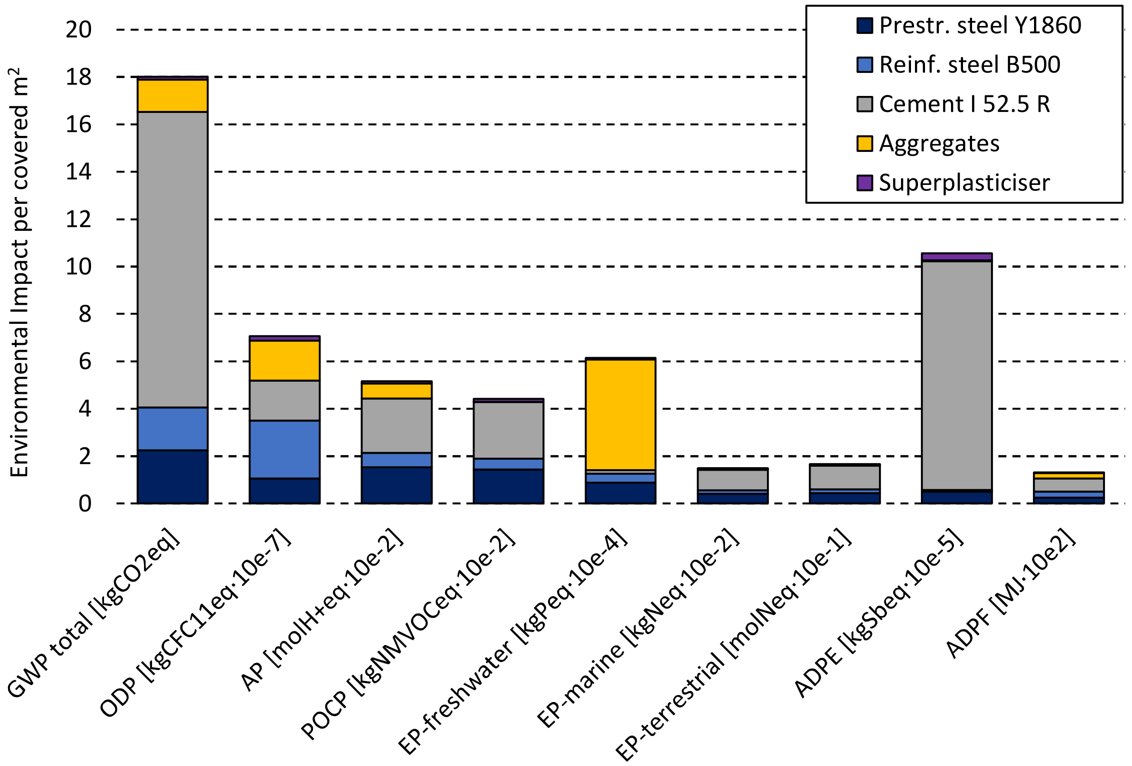
19.7 Lattice girder element – FprEN1992-1:2022



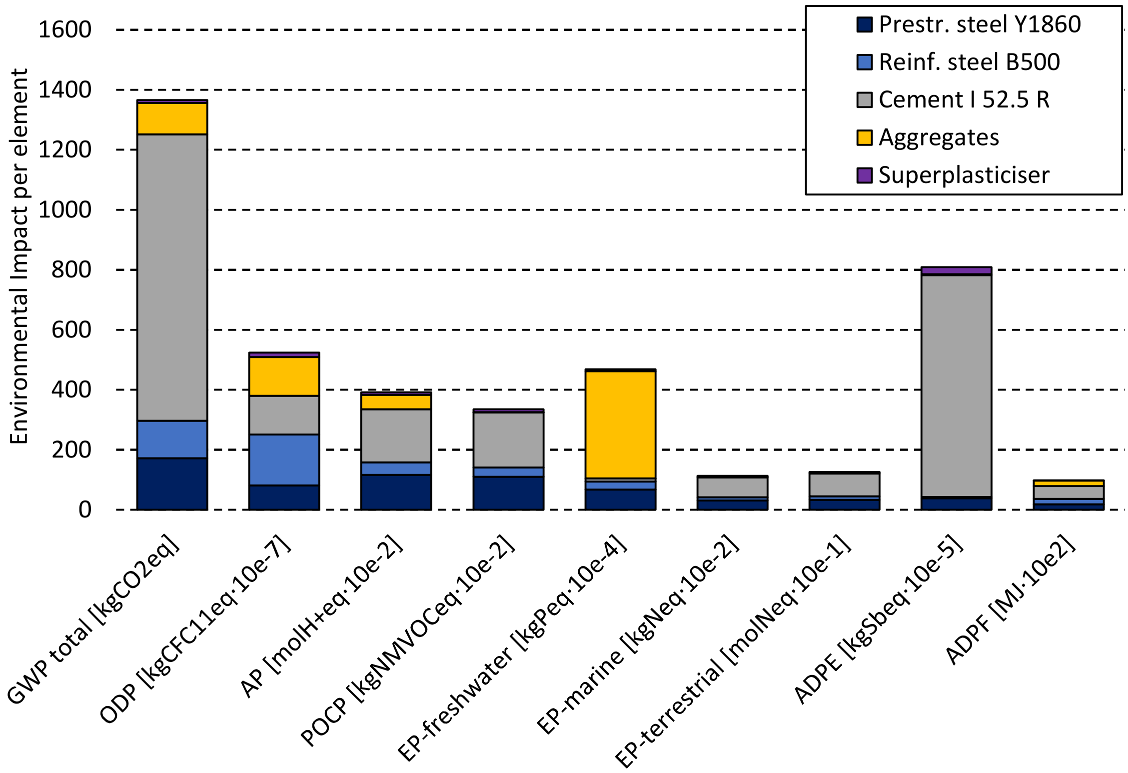


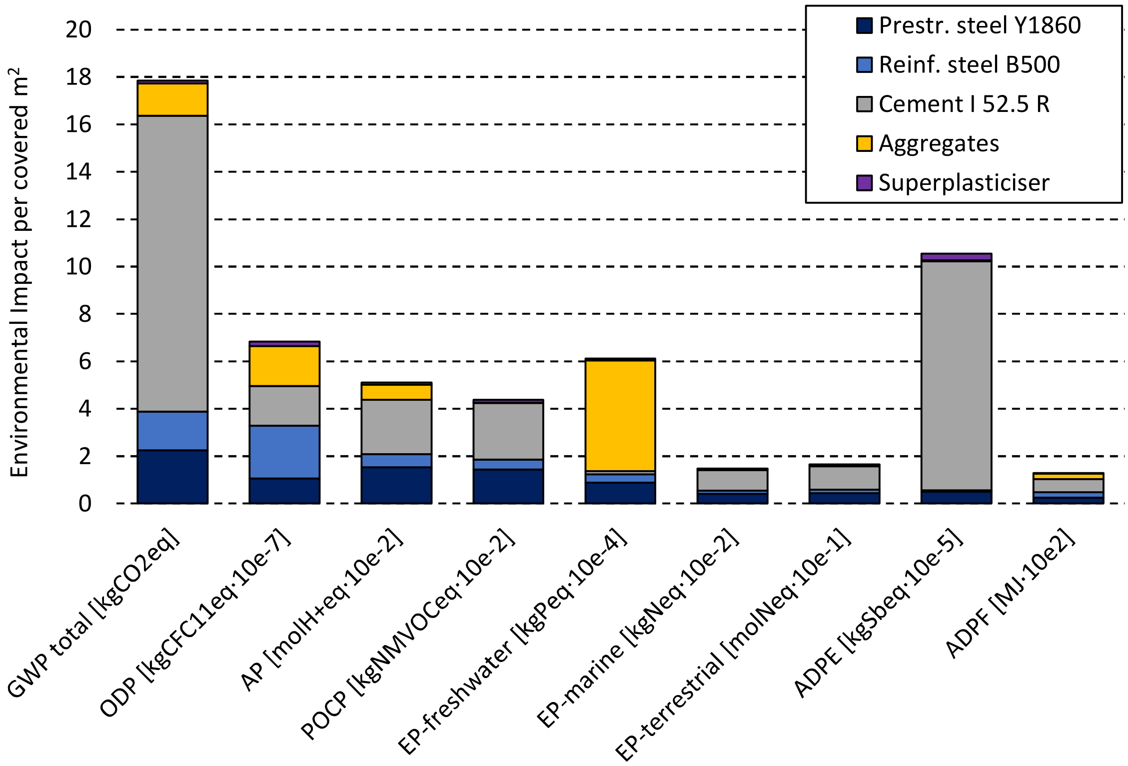
19.8 Prestressed beam element –EN1992-1:2004



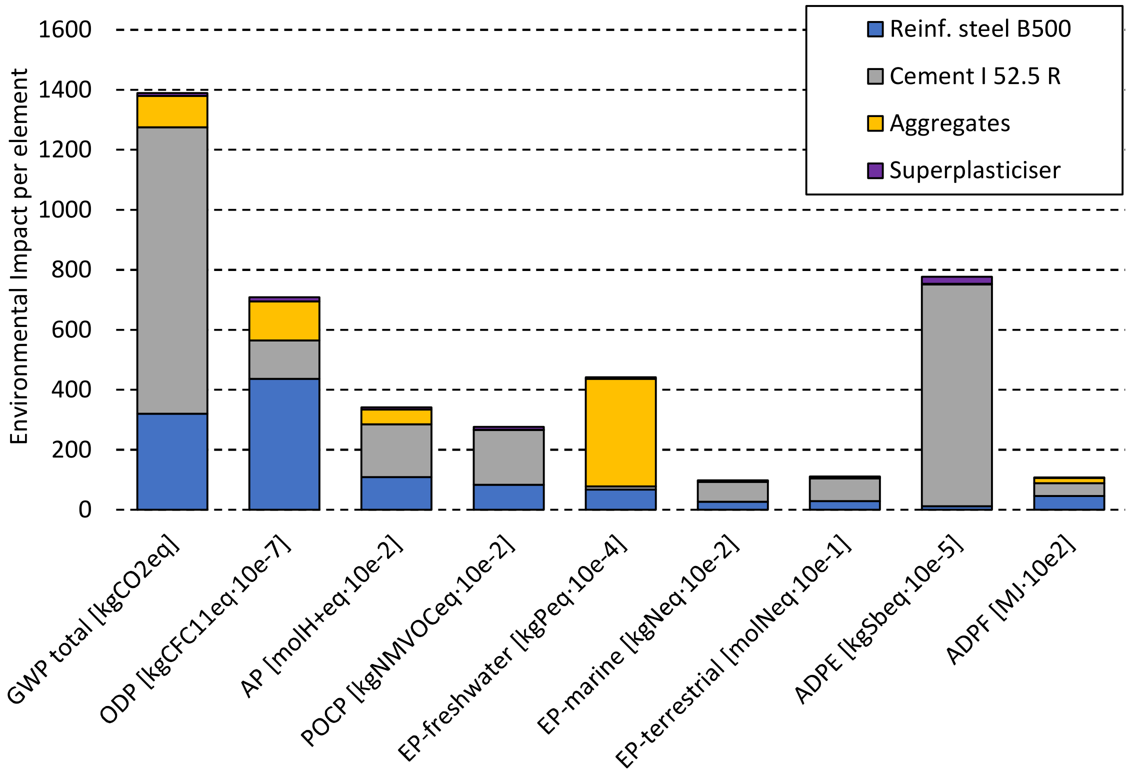


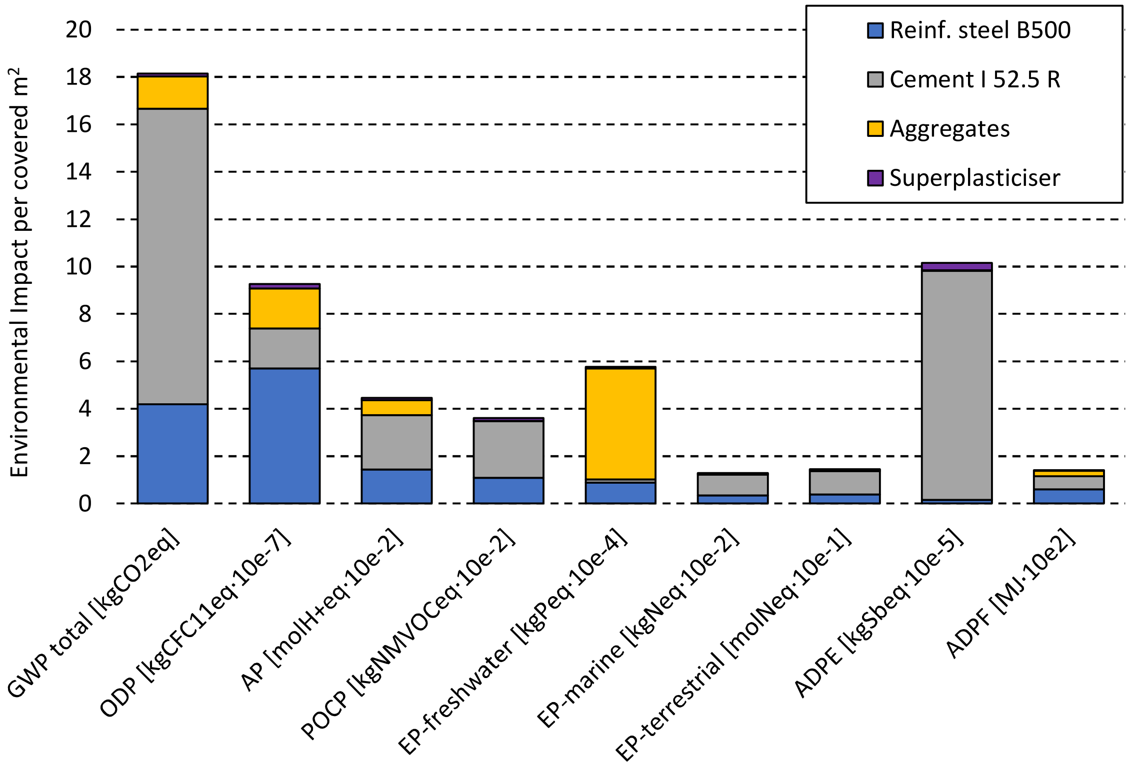
19.9 Prestressed beam element – FprEN1992-1:2022



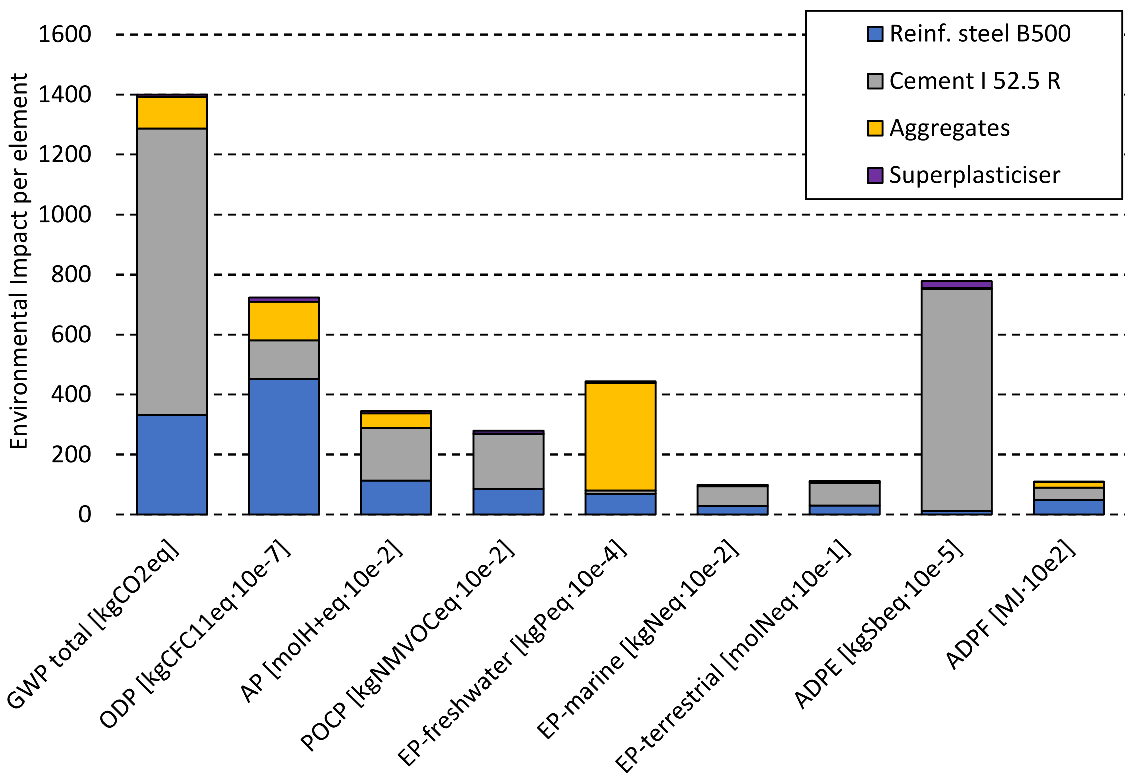


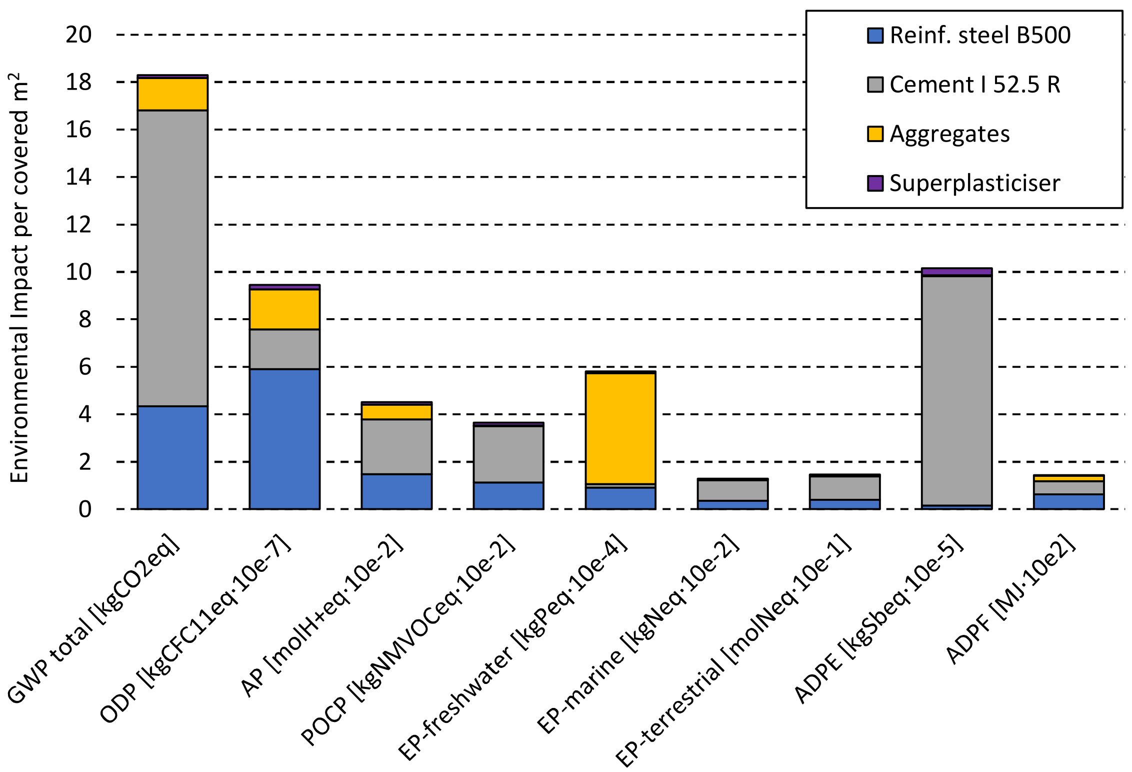
19.10 Reinforced beam element –EN1992-1:2004



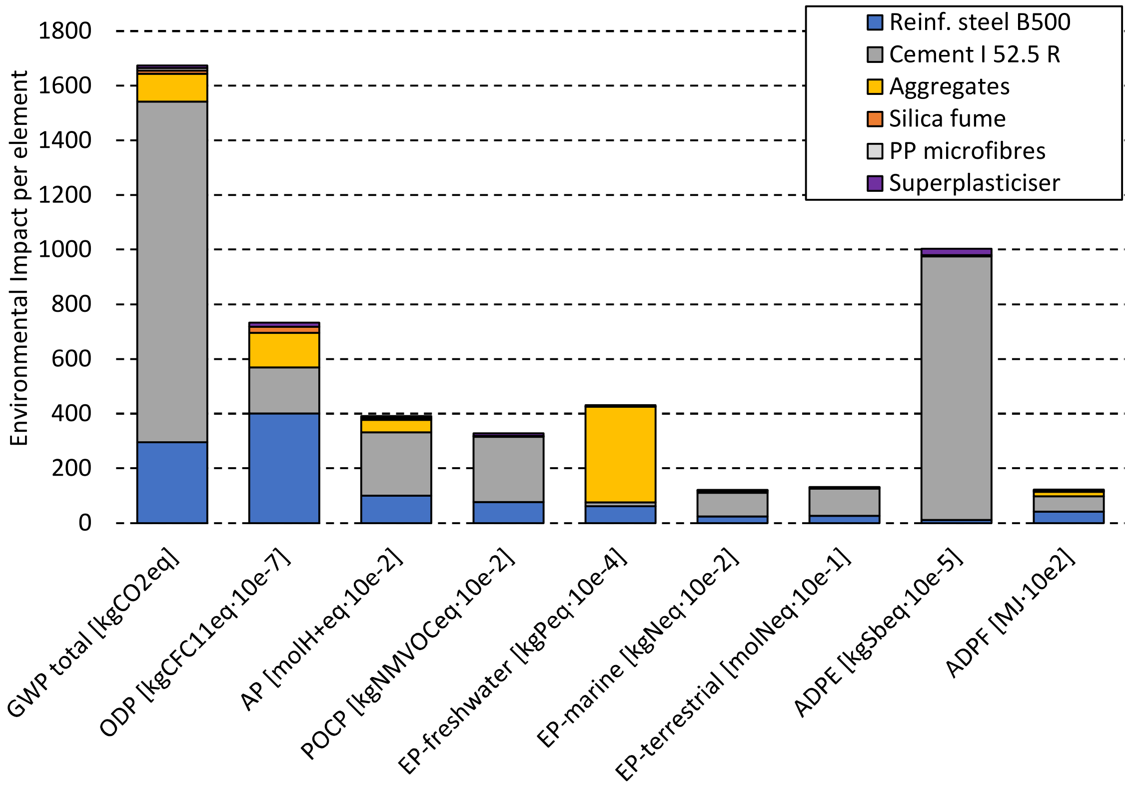


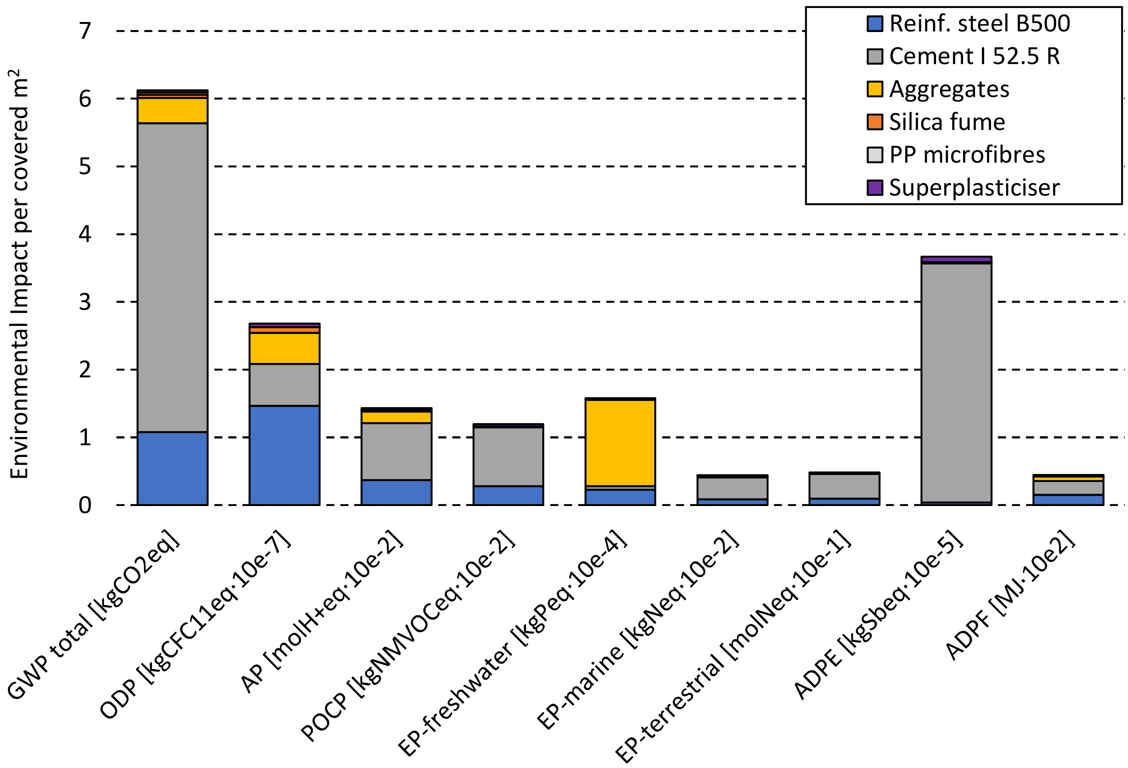
19.11 Reinforced beam element – FprEN1992-1:2022



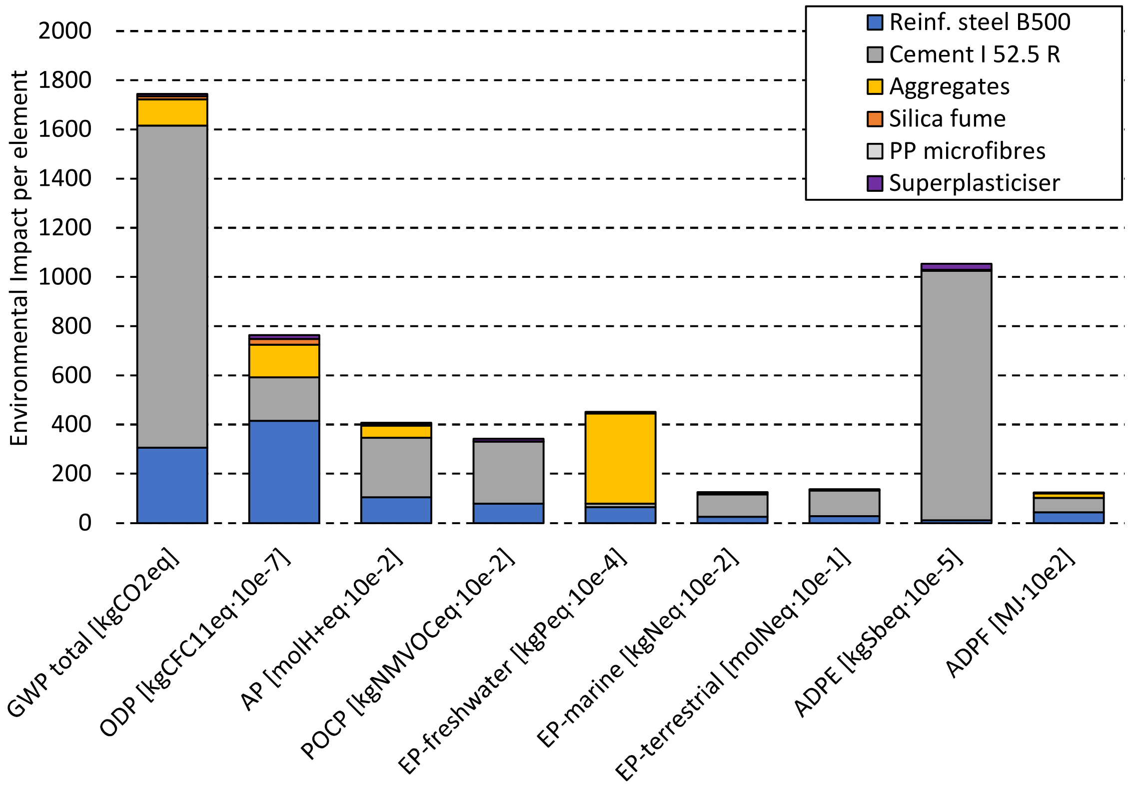


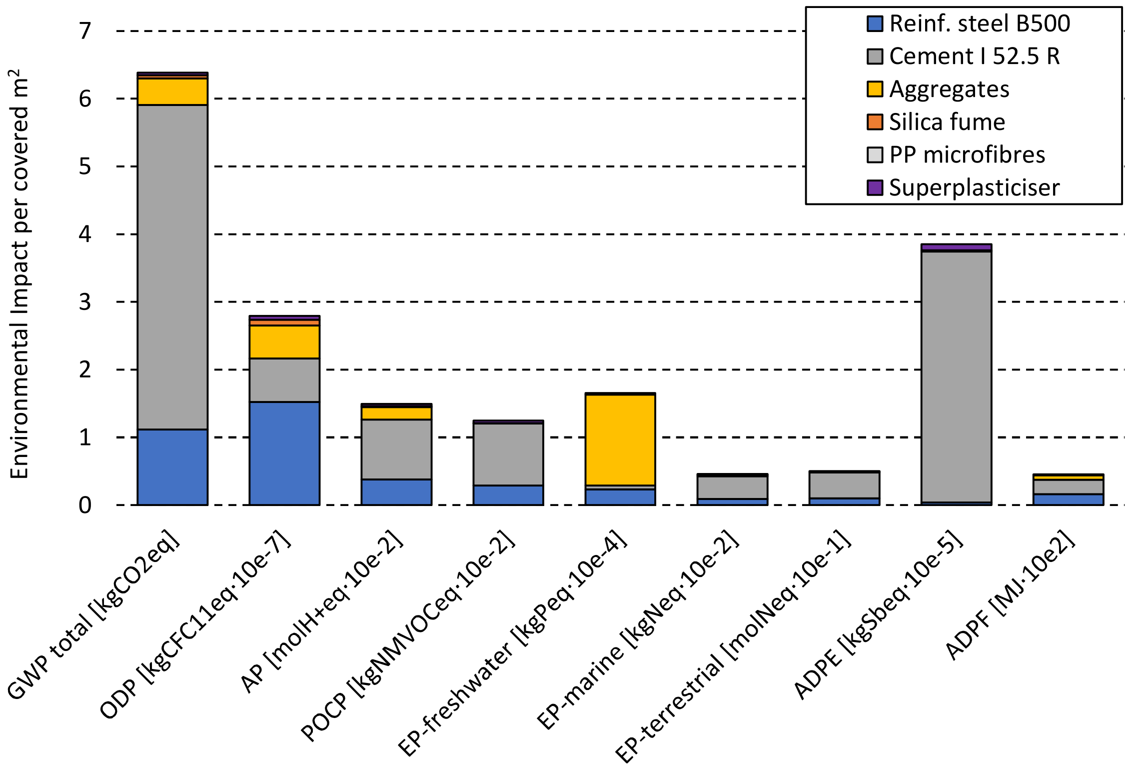
19.12 Column element –EN1992-1:2004



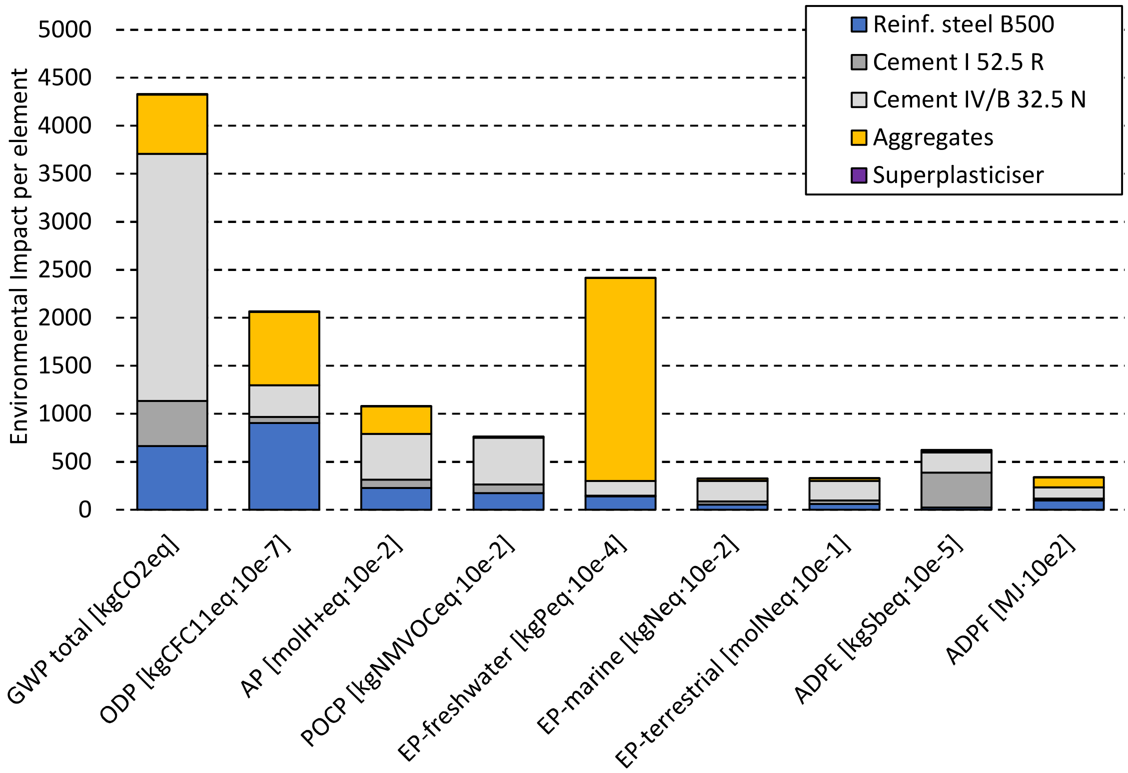


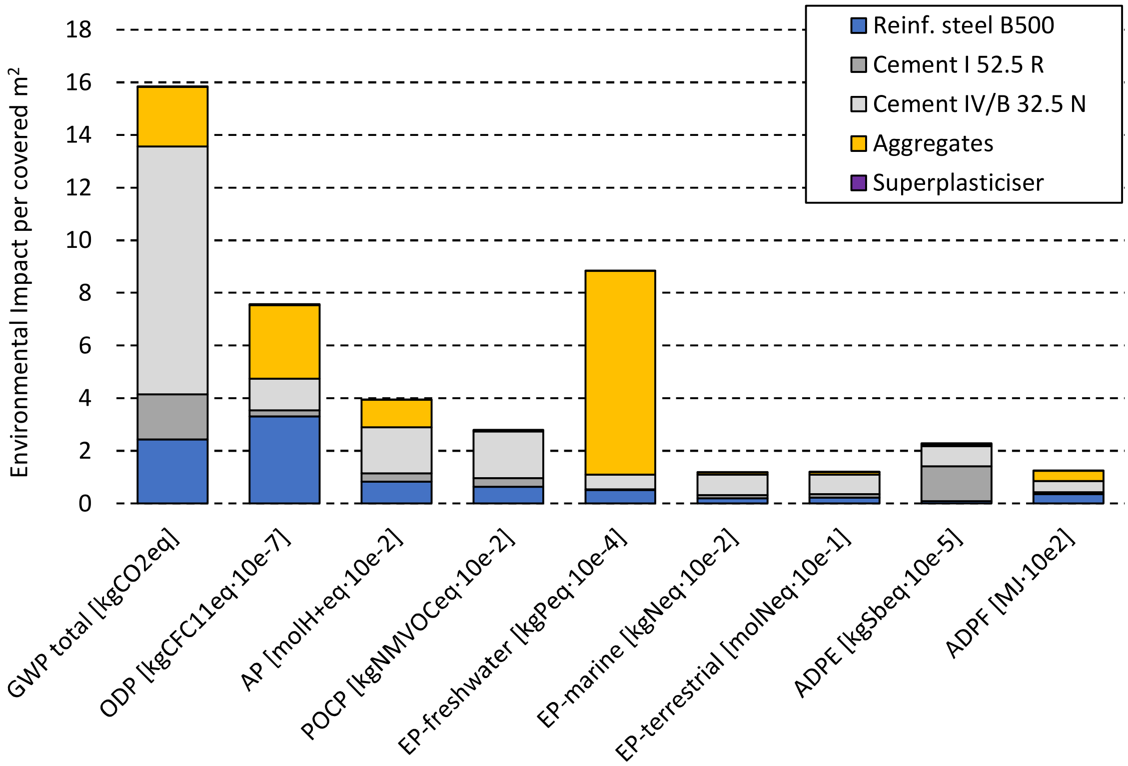
19.13 Column element – FprEN1992-1:2022



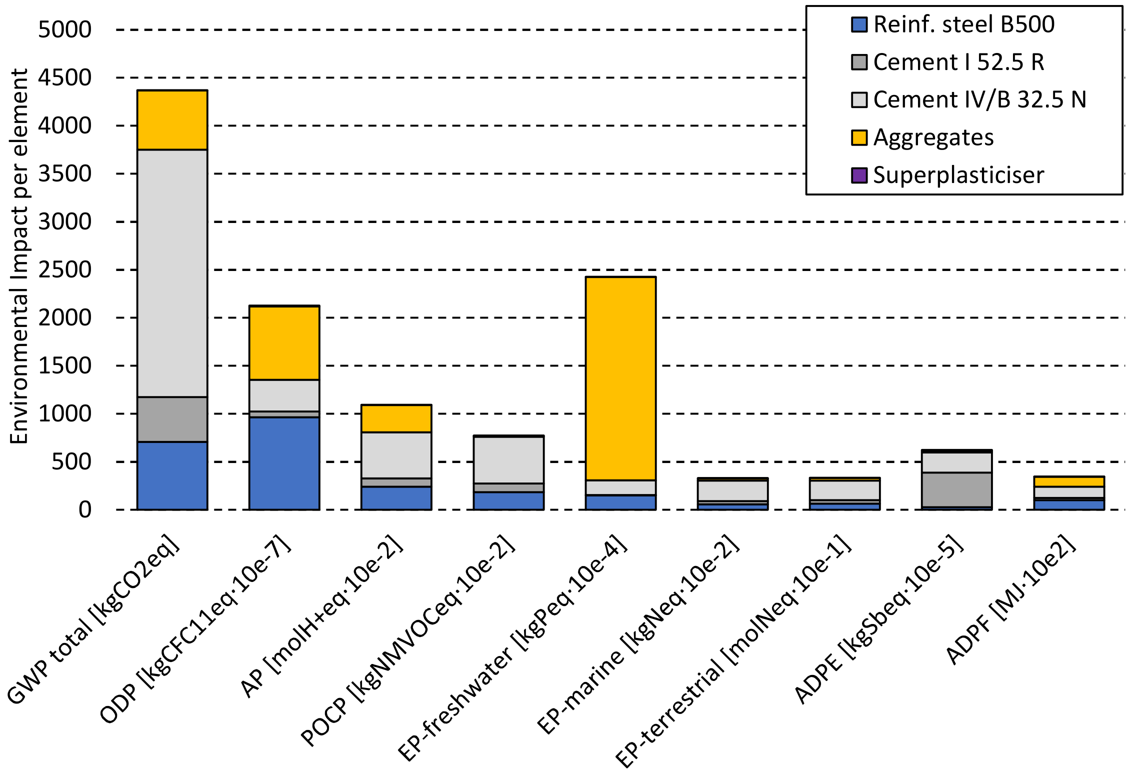


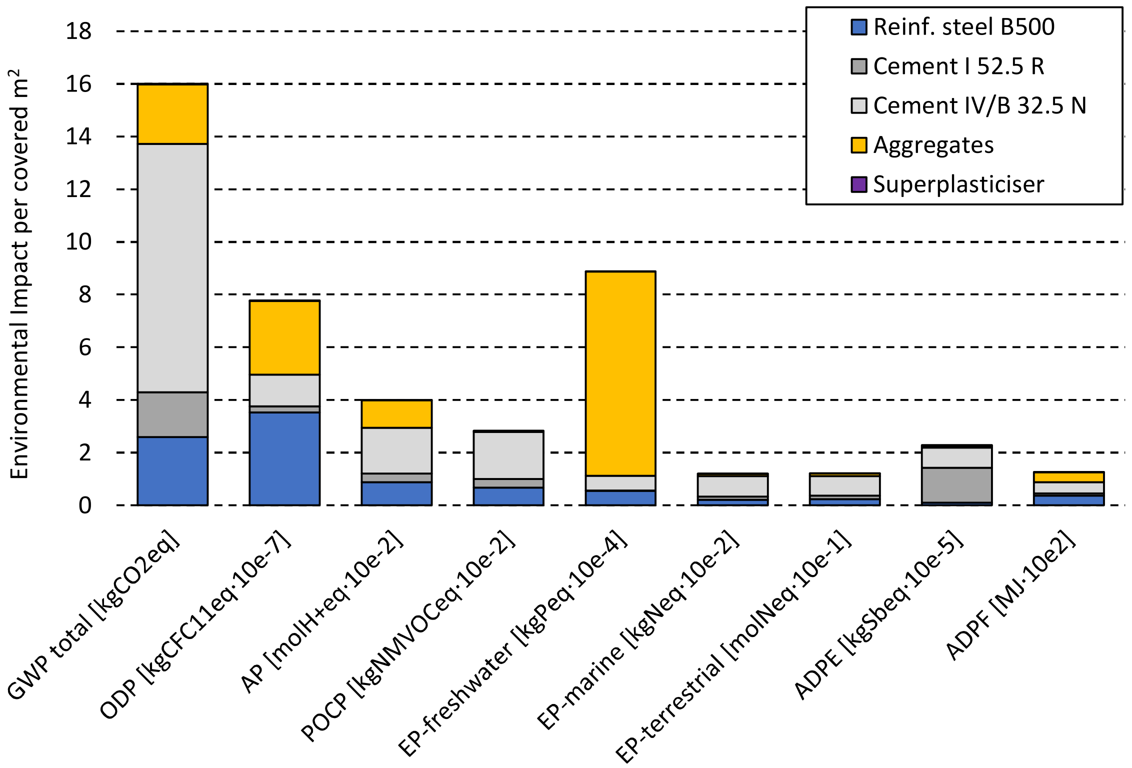
19.14 Foundation footing – EN1992-1:2004





19.15 Foundation footing – FprEN1992-1:2022





20 Comments on main deviations from EN1992:2004 to FprEN1992:2022

Detailed comments about the main deviations found between the structural design flow of the analysed elements following either EN1992-1-1:2004 or FprEN1992-1-1:2022 are listed in the following in the format of lettered point. It is reminded that the list is not intended to highlight every difference between the two codes, but only those differences that came across during the design process of the evaluated structural members.

20.1 Constitutive law of materials

a) **Partial factor for prestressing steel in ULS**: The design prestressing action can be managed following a single partial coefficient equal to 1.0 according to EN1992-1-1:2004 §2.4.2.2, to be multiplied by the mean design prestressing action, apart for the verification of local effects; it shall be managed following two partial coefficients according to FprEN1992-1-1:2022 §4.2.1.5 if the equivalent force approach is used following §7.6.1(1)b). No deviations were introduced in the project, since the stress approach was used. Apparently, to deal with equivalent force approach at ULS, where plasticisation of steel and concrete occurs, looks conceptually wrong and misleading.

b) **Partial safety factors for materials**: two additional partial factors are introduced in FprEN1992-1-1:2022: γ_{ce} for the concrete elastic modulus, and γ_v for the shear and punching resistance of unreinforced concrete.

c) **Design strength of concrete in compression**: a new factor η_{cc} lower than unity is introduced in FprEN1992-1-1:2022, which is intended to lower the concrete strength in compression for mixes of class higher than C40/50. Thus, this affects all elements that were designed in the project. For class C45/55 mix the reduction factor is equal to 0.961. For class C80/95 mix the reduction factor is equal to 0.794. It is pointed out that the formula affects even the most performing mixes classified as Normal Strength Concrete, and not only High Performance or High Strength Concrete.

d) **Formula for the evaluation of Ecm**: the formulae for the evaluation of the mean longitudinal elastic modulus of concrete is different in the two codes. It is reported that results are different but not by much (42.24 Gpa for EN1992-1-1:2004 versus 42.26 GPa for FprEN1992-1-1:2022 for class C80/95 mix).

e) **Formula for the evaluation of εcu1**: the formulae for the evaluation of the ultimate strain of concrete in compression according to the modelling strategy 1 is different in the two codes. It is reported that results are different but not by much (2.803‰ for EN1992-1-1:2004 versus 2.816‰ for FprEN1992-1-1:2022 for class C80/95 mix).

f) **Simplified constitutive law of concrete in compression**: in FprEN1992-1-1:2022 there is a relevant simplification in terms of simplified constitutive laws for concrete: the triangle-rectangle constitutive law was deleted, and the parameters defining parable-rectangle and stress block laws were modified by deleting the dependency of their parameters on the class of concrete.

g) **Ultimate strain of concrete mainly subjected to compression**: the limitation of ultimate strain to ε_{c2} / ε_{c3} contained in EN1992-1-1:2004 is disappeared in FprEN1992-1-1:2022. As a consequence, a potential for better exploitation of steel rebars in compressed column elements is introduced in the new EC2-1-1.

h) **Strength of concrete in tension**: a new formulation is proposed in FprEN1992-1-1:2022. Results do not change significantly with respect to the old formula of EN1992-1-1:2004.

i) **Elastic-perfect-plastic constitutive law of reinforcing and prestressing steel**: this point is a remark rather than an observation of difference. The typical elastic-perfect-plastic relationship is defined in both standards with an indefinite plastic branch. As a consequence, if this typical constitutive law is assumed, cross-sections will never fail on rebar side. To be noted that this is valid for all grades of steel, also for example B500A or for Y1860, despite being them much less ductile than B500C.

j) **Elastic-hardening constitutive law of reinforcing steel**: the value of ε_{ud} in FprEN1992-1-1:2022 is lower with respect to the value in EN1992-1-1:2004. The characteristic ultimate strain to be used for elastic-hardening constitutive law is in the new document divided by the partial safety factor of the steel material γ_s.

k) **Equivalent yield strength of prestressing steel**: the characteristic equivalent yield strength at 0.1% of residual strain at unload in FprEN1992-1-1:2022 is given in Table 5.6 as 1640 MPa, which is lower with respect to the value in EN1992-1-1:2004, equal to 0.9·fptk = 1674 MPa.

l) **Elastic-hardening constitutive law of prestressing steel**: the characteristic ultimate strain of prestressing steel ε_{puk} in FprEN1992-1-1:2022 is equal to 3.5%, much larger with respect to the suggested value in EN1992-1-1:2004, which turns out being 2.2%.

m) **Reduced partial factors thanks to production control**: both standards allow to reduce the partial safety factors for materials for precast concrete members due to the adoption of a production control procedure typical of precast production plants. Despite the procedure is different, the resulting reduced coefficients are the same for both documents (1.4 for concrete and 1.1 for reinforcement). Nevertheless, different strategies may lead to different factors.

n) **Confinement contribution of stirrups for the definition of the concrete core constitutive law**: FprEN1992-1-1:2022 proposes a method to evaluate the contribution from stirrups in the effective confinement of concrete cores in terms of stress σ_2. This method, not cited in EN1992-1-1:2004, helps the designer in understanding how to implement this contribution instead of making reference to methods not included in the standard. Moreover, apart from the method used to evaluate the confining stress, also the formulation concerning the modification of the constitutive law of concrete in compression is radically modified in the new document. As an exemplificative application within the column element (in the areas where it is not lightened by the plastic pipe), the confinement effect resulted in a limited (practically negligible) increase of strength and in a relevant increase of strain capacity (more than double).

o) **Evolution of strength and elastic longitudinal modulus over time**: the formulae for the evaluation of the evolution of compression or tension strength over time are formally similar in the two documents, although the constitutive parameters s, β_{cc} are defined in a different way. Moreover, the exponents of the formulae for the determination of E_{cmj} and f_{ctmj} are slightly different. As a result, at 2 days of age f_{cmj}, f_{ctmj} and E_{cmj} are 30.6 MPa, 2.73 MPa, and 29.7 GPa referred to FprEN1992-1-1:2022, and 30.6 MPa, 2.19 MPa, and 30.7 GPa referred to EN1992-1-1:2004, respectively. Hence, the compressive strength results identical, the tensile strength remarkably higher, and the elastic modulus slightly lower.

20.2 Flexural and compressive strength

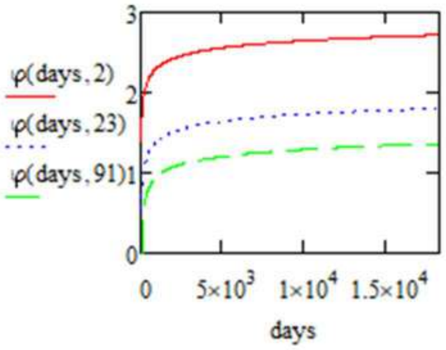
No differences were noted in the procedures for the check of flexural/compressive strength between the two standards.

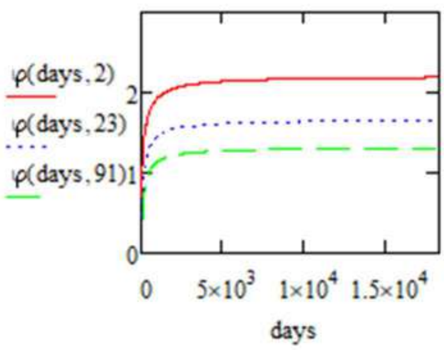
20.3 Serviceability

a) **Method for evaluation of the SLS deflections**: The rigorous method of double integration of the curvature was used following both documents, according to §9.3.4(7) of FprEN1992-1-1:2022 and according to §7.4.3(7) of EN1992-1-1:2004. To be noted that the general non-linear constitutive law of concrete (indicated for structural assessment) should be used to correctly capture the concrete deformability in the pseudo-elastic behaviour phase.

b) **Allowed stress for strands**: The allowed stress for prestressing strands is less severe in FprEN1992-1-1:2022 with respect to the current version. In particular, the allowed stress is increased from 0.75·fpk (§7.2(5) of EN1992-1-1:2004) to 0.80·fpk (Table §9.1 of FprEN1992-1-1:2022).

c) **Creep coefficients**: The method for the calculation of the creep coefficients in classical viscoelasticity regime is completely different in the two documents. In particular, it is pointed out that the creep coefficient at early age of concrete is much higher according to FprEN1992-1-1:2022 with respect to EN1992-1-1:2004 (for two days of age, the linear effects at 50 years of life span are magnified by 2.72 versus 2.19), which affects the contribution of prestressing. Nevertheless, this difference is becoming narrower for loads applied at larger time. In the following, two graph comparing the creep functions for the two documents are reported (concrete C45/55):





Creep coefficient – FprEN1992-1-1:2022 EN1992-1-1:2004

d) **Limit between linear and non-linear creep**: The limit for the consideration of non-linear creep slightly changes from 0.45·fck (§7.2(3) of EN1992-1-1:2004) to 0.40·fcm (§9.1(4) of FprEN1992-1-1:2022).

e) **Maximum crack width**: The limit for the maximum crack width allowed in EN1992-1-1:2004, equal to 0.2 mm for prestressed members, is increased to 0.3 mm in FprEN1992-1-1:2022.

f) **Expected crack width calculation**: The approach is similar in the two documents, although a completely different definition for the spacing of the cracks function sr_{max} is provided. As a result, for the beam with mild reinforcement the crack width in frequent load combination resulted higher in FprEN1992-1-1:2022 (0.126 mm) with respect to the width calculated according to EN1992-1-1:2004 (0.091 mm).

g) **Shrinkage**: Formulae for the evaluation of the shrinkage strain of concrete over time have been deeply modified in FprEN1992-1-1:2022, despite the basic data needed for the calculation is similar to EN1992-1-1:2004. As a result, the final shrinkage at 50 years of life service of the lattice girder element designed according to FprEN1992-1-1:2022 results relevantly lower than what calculated according to EN1992-1-1:2004.

20.4 Shear, punching shear and strut&tie

a) **Type of check for shear**: Different strategies are introduced in FprEN1992-1-1:2022 with regard to ULS shear checks. In particular, following §8.2.1(1), if the maximum tangential stress is lower than a minimum $\tau_{Rdc,min}$, detailed shear check can be omitted; if the maximum tangential stress is lower than τ_{Rdc}, no calculated shear reinforcement can be placed; elsewise, a detailed calculation shall be carried out.

b) **Check procedure for members not reinforced for shear**: Two alternative calculation procedures are suggested, leading to different results. They both differ relevantly from the procedure suggested by EN1992-1-1:2004. The devoted chapter is also not fully clear in some passages concerning the effect of axial forces, including the following "*For the given factor k1 according to Formula (8.34), the effective depth d in Formula (8.33) may be replaced by av,0 where av,0 is determined according to Formulae (8.29) and (8.30), without* *considering in MEd und VEd the effect of prestressing or external load that produces the compressive axial force.*". By reading this chapter, the designer does not fully understand whether *d* is to be replaced in formula (8.34), or in (8.33), or in both. Moreover, in the same chapter the main difference between the two suggested alternative procedures consists in including or excluding the effect of prestressing on bending, which apparently is null in case of straight strands typical of pre-tensioning technique.

c) **Cotangent for the ULS shear check of members with shear reinforcement**: Given stirrups of steel grade B500A are used, as typical for small rebar diameters, the maximum cotangent of a member in bending shall be limited to 2.0 according to §8.2.3(4) of FprEN1992-1-1:2022, instead of 2.5 in EN1992-1-1:2004, following the limitation to 80% of the suggested value.

d) **Basic formulae for the ULS shear check of members with shear reinforcement**: The basic formulae are the same of the Morsch truss model. However, the design shear associated to the failure of stirrups ties or concrete struts is expressed in terms of tangential stress instead of loads over the truss moduli, which is somehow misleading. Moreover, the formula for the concrete strut check is slightly different by means of (i) a single value of the concrete strength reduction coefficient v equal to 0.5 in §8.2.3(6) of FprEN1992-1-1:2022, instead of a slightly larger value as a function of concrete class provided in §6.2.3 of EN1992-1-1:2004, and (ii) the coefficient α_{cw} accounting in EN1992-1-1:2004 for the compression state of stress of the concrete chord, is missing in Fpr1992-1-1:2022.

e) **Formulae for the ULS shear check of precast members without shear reinforcement**: In both documents a specific period is inserted concerning a reduction of shear resistance for precast members not reinforced in shear (mainly hollow core). Despite the objective of the two formulations is the same, i.e. checking plane-state stress to not leave the beam form a crack in shear, the actual proposed formulae yield to different results. In particular, a more general approach is contained in FprEN1992-1-1:2022, which in principle would require the identification of the most stressed chord in combined longitudinal and transverse directions along the depth of the cross-section. The results, in line with other points of the documents, highlight a larger strength associated with the approach of FprEN1992-1-1:2022.

f) **Shear between web and flanges**: The approach is similar in the two documents. The additional horizontal reinforcement is calculated according to the same formula. The check on the compressed concrete struts is calculated according to a different formula, although it differs only for the formal mathematics, since it yields to the same result.

g) **Shear at the interface between concretes cast in different times**: The approach is similar in the two documents. Nevertheless, the formulae of FprEN1992-1-1:2022 are relevantly different from the ones of EN1992-1-1:2004. As a result, coherently with the whole document, the strength at the interface between the two concretes for the lattice girders becomes slightly higher with the new document (0.687 MPa versus 0.643 MPa). To be noted that the values of cohesion and friction coefficients are completely different.

h) **Procedure for calculation of punching shear**: Analogously to the procedure for the check of shear, FprEN1992-1-1:2022 introduces a "gate" severe check with a particularly low punching shear resistance, which, if fulfilled, avoids the need to carry out further calculations.
i) **Definition of control perimeter for punching shear**: The definition of control perimeter for punching shear in FprEN1992-1-1:2022 is similar in terms or approach but much more severe in terms of quantitative results with respect to the current standard: instead of a fixed stress diffusion angle θ=26.6° in EN1992-1-1:2004, with the check to be carried out in correspondence of this perimeter, a lower fixed stress diffusion angle θ=45° is given in FprEN1992-1-1:2022, moreover with the check to be carried out at half of the distance dv from the load area. This highly reduces the control perimeter in the new standard, making the check for punching shear of unreinforced concrete structures relevantly more severe with respect to the current standard. For the case of the designed foundation footing, the new procedure yielded to the need of punching shear reinforcement, placed in the form of bent rebars.
j) **Control sections in punching shear**: FprEN1992-1-1:2022 explicitly cites, also with the aid of an explicative image, that step foundation footings must be checked at all step sections, assuming each step as a loading area, which was not necessary according to EN1992-1-1:2004. For the designed foundation case study, it did not yield to any modification of the element.
k) **Extension of punching shear reinforcement**: The formula for the length of extension of the punching shear reinforcement contained in FprEN1992-1-1:2022 provides large values which do not match with foundation elements resting over soil.
l) **Strut&Tie method for corbels**: The resistance of compressed struts varies significantly between the two standards. In particular, the reduction factor ν' of strut compressive strength, which was related to the concrete class only in EN1992-1-1:2004, is drastically changed in FprEN1992-1-1:2022, where it is related to the angle of the strut. With respect to the previous standard, the coefficient results less severe for large angles, and more severe for small angles. To be noted that the reinforcement and strut&tie schemes provided in annex J of EN1992-1-1:2004 are not proposed in FprEN1992-1-1:2022, apart from a simplified sketch in Fig. 8.5.

20.5 Fire

a) **Thermal conductivity curve for concrete**: The approach of the two alternative curves proposed in EN1992-1-2:2004 was overcome in FprEN1992-1-2:2022 thanks to the introduction of a single curve, which turns out to be intermediate with respect to the ones previously introduced. The other thermal/physical properties of concrete were not changed.
b) **Mechanical stress-strain relationship of concrete with temperature**: the advanced stress-strain relationship of concrete was not changed for concrete classes below C70/85. For class

C70/85 or higher, only a very minor is introduced in FprEN1992-1-1:2022, concerning the strength loss in the range 20 °C – 100 °C.

c) **Concrete spalling**: rules are introduced in §10 of Fpr1992-1-2:2022 concerning the mitigation of concrete spalling. In particular, complex numerical assessments or the introduction of polypropylene microfibres in the concrete mix are requested for members of concrete class higher than C70/85 or for members with thin exposed web. In the project, it turned out that 2.0 kg/m^3 polypropylene microfibres need to be implemented in the concrete mix for the column element, only, due to its high concrete class, given the more standard mix for class C45/55 concrete employed for the other elements does not contain a quantity of silica fume larger than 6% by weight of cement. It is noted that other types of fillers are not apparently included into this restriction.
d) **Checks in bending**: the Isotherm 500 °C method is not anymore explicitly included into Fpr1992-1-2:2022, but a similar procedure is introduced. The dimensions of the concrete area are given by simplified formulations, where it is noted that in §7.3.1(4) concrete classes higher than C70/85 are penalised by means of 15% additional reduction of rim inefficient thickness. Nevertheless, a thermal mapping is always necessary to evaluate the temperature in the steel reinforcement. It is pointed out that the Isotherm 500°C method was not used (advanced method was used instead) apart from the column model method for the evaluation of the fire resistance of the column element including second order effects.
e) **Checks in shear**: the same method of the reduced section is proposed in the two documents. However, the definition of the reference temperature of the transverse reinforcement is different: in EN1992-1-2:2004 it is defined as the position defining the effective tensile area following EN1992-1-1:2004, or other points not fully clearly identified in the procedure and only indicatively positioned in the drawings of Figure D.2. In FprEN1992-1-2:2022 it is defined geometrically on the basis of the shape of the cross-section as the point at mid-depth of the shear-resisting web, which appears being more logical for certain shapes typical of precast concrete elements such as inverted-T-shaped, L-shaped, I-shaped or Y-shaped beams, having larger bottom bulb. Moreover, in the new document the definition of the reduced cross-section may follow the method alternative to the Isotherm 500 °C, and the shear resistance is checked in accordance with the procedure described in FprEN1992-1-1:2004, which provides the differences previously highlighted.

20.6 Additional requirements

a) **Minimum concrete cover**: The approach to evaluate the minimum concrete cover is similar between the two standards, although what are named Structural Classes (SC) in EN1992-1-1:2004 are transformed into Exposure Resistance Classes (ERC) in FprEN1992-1-1:2022 with regards to minimum cover for durability $c_{min,dur}$. Differently from EN1992-1-1:2004, where the basic SC was recommended (S4), in Fpr1992-1-1:2022 there is no suggestion about the basic ERC to select, which is referred to the procedures included in the standard

EN 206:2013. To be noted that the reference to an outer standard different from EC2 is out of the style of the document, which typically is conceived to provide all information to carry out the complete design. Moreover, there is no perfect correspondence between SC and ERC in terms of suggested carbonation-induced minimum cover different. Apparently, the ERC class most similar to S4 is XRC4, despite for XC4 the minimum cover results less. This issue was however overcome by the adoption of the alternative procedure provided in Annex P of FprEN1992-1-1:2022, which is exactly the same of the old version. Thus, no deviations are reported between the position of mild and prestressing reinforcements according to the two EC2 versions.

b) **Minimum amount of main flexural reinforcement**: The procedure for the evaluation of the minimum reinforcement is similar in both documents, although in FprEN1992-1-1:2022 it requires more passages. As a result, it was observed a very remarkable lowering of the minimum amount in the new document (for the beam, slightly more than 3 times). Moreover, an explicit condition of overstrength after cracking is introduced in FprEN1992-1-1:2022, which affects elements predominantly subjected to bending. No changes were observed in the formula for the minimum amount of shear reinforcement.

c) **Minimum amount of reinforcement at support**: FprEN1992-1-1:2022 introduces in Table §12.2(5) a minimum amount of bottom longitudinal reinforcement at the end support equal to $0.25 \cdot A_{s,req,span}$, where it is specified that $A_{s,req,span}$ is the area of steel required for positive bending moments of the span. This definition is not precise, since it does not state that the moment to be considered is the maximum. However, interpreting the definition, the maximum bending moment was used to design the support reinforcement. In EN1992-1-1:2004 a similar concept is introduced, however referring explicitly to the area of steel provided (not required).

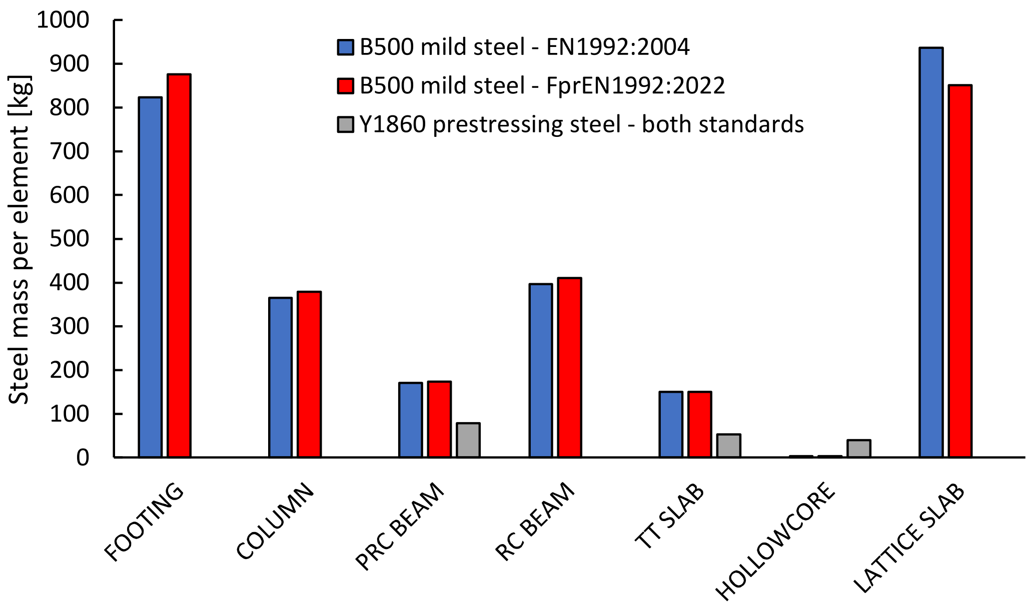
d) **Anchorage length**: The formulae in the two standards appear being completely different. In particular, the formulation included in FprEN1992-1-1:2022 yields to longer anchorage length for mild reinforcement. For the case of the beam, the straight anchorage length is equal to 28.5Φ in the new document, against 26.6Φ in the current one. Surprisingly, although coherently with the lower transfer length of prestressing discussed in the following point, the formula for the anchorage of prestressing tendons yields to the lower length of 1539 mm against 1670 mm. Finally, a more clearer and sound formula is provided in the new standard for what concerns the effect of bends at different angles, which allows to relevantly reduce the total anchorage length of bent rebars.

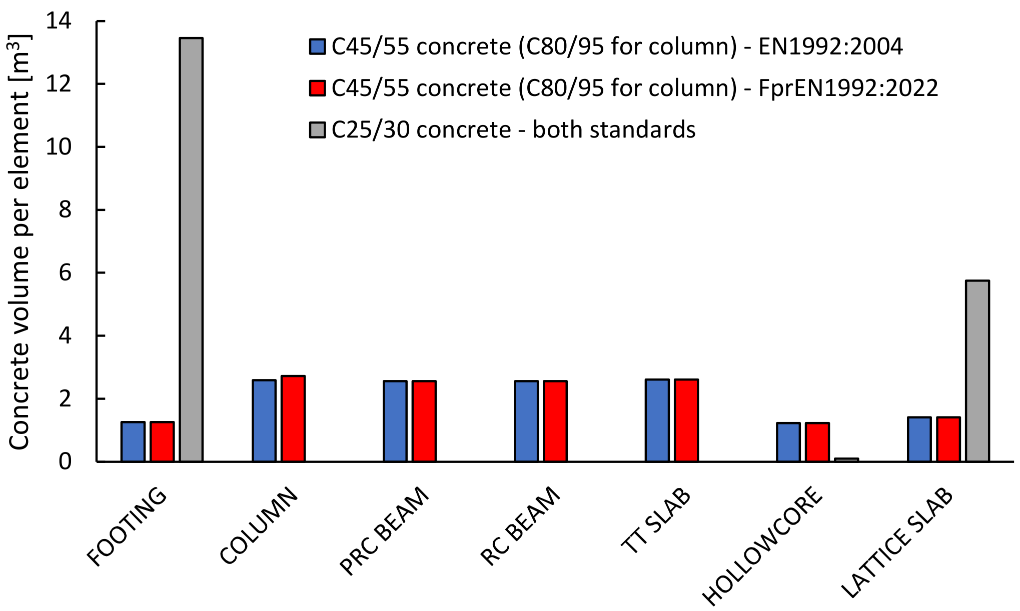
e) **Transfer of prestress**: The formula in FprEN1992-1-1:2022 differs substantially from that of EN1992-1-1:2004, since it is related to the concrete compressive strength rather than its tensile strength, and also several coefficients are different. As a result, the formulation of FprEN1992-1-1:2022 yields to a remarkably lower transfer length of prestressing of 907 mm against 1031 mm from EN1992-1-1:2004.

21 Conclusions

21.1 Comparison in terms of volume of material employed

The conclusions of the work are given in terms of a comparison of the quantities of different materials employed, in the following graphs, and in terms of qualitative indications about the rationale of the proposed concept and structural design and the differences found in the application of the current draft of the new Eurocode FprEN1992-1:2022 with respect to the current standard EN1992-1:2004.





21.2 General comments about the design of the elements

- **TT element**: a commercial cross-section with original 30 cm of depth was selected. However, due to the large distributed floor loads and due to the request to not consider a reinforced concrete topping, making a fully precast slab, the original thickness of 5 cm of the top slab was increased to 8 cm, assuming sides are mounted over the production mould in the precast factory. ULS bending resulted being the critical factor for the selection of the longitudinal prestressing reinforcement. The minimum reinforcement requirement for transverse bars resulted critical for the disposition of the transverse bars, although close to what required for shear resistance close to the end supports. The double mesh of the top slab was integrated with additional straight rebars in the vicinity to the end supports due to shear at web-flange interface. Additional short U-shaped rebars were placed at the supports to cover the bending moment request from the Mörsch truss scheme.
  No differences were adopted between the elements design according to the two standards.

- **Hollowcore element**: a commercial cross-section with 26.5 cm of depth was selected. As required by the extruding production method of hollowcore members, only prestressing tendons were assumed to be incorporated in the precast element. Their design was related to the need for bending check. Two additional straight rebars per member end with end thread are designed to be post-inserted (screwed) into mechanical couplers cast into the beam element, and later filled with in-situ concrete pouring. Their function is double: to provide additional shear strength close to the end supports, and to provide a mechanical connection for robustness and diaphragm stiffening.
  The only difference between the design of the element according to the two standards concerns the length of the additional straight rebars, which following Fpr1992-1:2022 is slightly larger. Indeed, an important difference consists in the shear resistance calculation, which according to Fpr1992-1:2022 would not have needed to cast the selected holes. They were assumed to be cast anyway in order to fulfil their second function, as previously described.

- **Lattice girder element**: the conception of the element starts from a commercial product of precast lattice plank. This element needed to be heavily reinforced in bending for the combination of the following reasons: it is not prestressed, and it turns out being a solid concrete slab, and thus its self-weight is large. In particular, the critical request was the one related to deflection limitation. Despite an initial camber equal to the maximum allowed by the code was set, the large quantity of longitudinal reinforcement in the main working direction of the element was introduced in order to reduce the cracked stiffness of the element and, subsequently, the final viscoelastic deflection. Moreover, a significant quantity of transverse reinforcement was inserted, in order to respect the request by both standards to install a certain percentage of the longitudinal reinforcement in the transverse direction. To be noted that the four lattice trusses cast in the precast plank are not needed in the final

configuration following both standards. Anyhow, they are needed for intermediate construction phases. Finally, short U-shaped rebars were installed as additional end reinforcement to provide bending resistance also near the supports.
The main difference between the design following the two standards refers to the minimum quantity of transverse reinforcement for slabs, which is lower in FprEN1992-1:2022.

- **Prestressed beam element**: the beam element was conceived with proper dimensions to laterally support the slab members. The reinforcement cages were conceived in order to be preassembled in the precast factory, lifted and installed after the tendons and the lower mesh are ready on the mould. The longitudinal reinforcement was designed in order to fulfill the ULS condition for bending moment.
  The main difference between the elements designed following the two standards is in the quantity of stirrups: closer to the end support, the spacing for FprEN1992-1:2022 results narrower due to the difference in the strut angle, but soon after it becomes larger due to the stronger shear resistance of elements not requiring shear reinforcement. In this area, the minimum reinforcement requirement determines the stirrup spacing.

- **Reinforced beam element**: The longitudinal reinforcement was designed in order to fulfill the SLS condition for deflection control. Despite an initial camber equal to the maximum allowed by the code was set, the longitudinal reinforcement was designed in order to reduce the cracked stiffness of the element and, subsequently, the final viscoelastic deflection.
  The main difference between the elements designed following the two standards is in the quantity of stirrups: closer to the end support, the spacing for FprEN1992-1:2022 results narrower due to the difference in the strut angle, but soon after it becomes larger due to the stronger shear resistance of elements not requiring shear reinforcement. In this area, the minimum reinforcement requirement determines the stirrup spacing.

- **Column element**: The side width of the square column element was selected equal to 40 cm in order to be the same of the beam top, for geometrical compatibility. This element was designed with a high-performance concrete class: C80/95. Due to the high compressive strength of concrete, which would have been much over-designed in the configuration of a solid element, the cross-section was lightened by inserting in production long pieces of plastic pipe in central position. The pipes, not intended to work as a rain drainage system, are discrete and interrupted in correspondence of the corbels providing support for the beams. The bottom joint with the foundation in assumed to be a pocket joint, and thus the column is actually longer than what needed in its final configuration. Moreover, the column was deemed to be too long to be transported and erected in a single piece, and thus it was divided at about half of the height of the building, out of the corbel areas. The two elements are assumed to be connected by means of protruding rebars from the top column element, inserted into thin metallic pipes cast into the bottom element, providing large tolerance.

After external bracing and verticality regulation, the joint is completed by pouring high-strength mortar which fills the pipes and the gap. The column element resulted subjected to very low bending actions (moment and shear), being the axial load the clear critical action. Nevertheless, considering a minimum axial load eccentricity of 20 mm, following both standards, the element was checked against the resulting bending moment distribution.
FprEN1992-1:2022 provides a strength reduction coefficient for high performance concrete, which as a consequence allowed to lighten less the column cross-section. Thus, the column design according to EN1992-1:2004 contains less concrete than the one designed according to FprEN1992-1:2022. Moreover, similar considerations apply also to the steel volume, since it was designed according to the minimum reinforcement criterion, as well as the stirrup distribution.

- **Foundation footing**: The foundation footing is assumed to be partially precast, with the pocket element being produced in the precast plant, and the lower slab to be cast-in-situ. The reinforcement of the pocket was selected as the standard minimum for such elements, since very low bending moment and shear actions are expected. U-shaped rebars are assumed to protrude from the precast element and to be connected with the reinforcing cage to be assembled in-situ. Not being the soil object of dedicated design, the reinforced concrete foundation was designed assuming a simplified constant stress distribution from the ground, associated to a Winkler-type soil model with rigid foundation. The reinforcement of the lower cast-in-situ slab was designed following bending actions induced by the cantilevering of the lower slab with respect to the precast. Additional small-diameter rebars were placed above the main bottom longitudinal ones in order to provide stability to the cage in the transitory phases before casting.
The difference between the foundations designed according to the two standards concerns the punching shear reinforcement: indeed, punching shear reinforcement was calculated not to be necessary following EN1992-1:2004, while a series of inclined rebars were inserted in the cage of the element designed according to Fpr1992-1:2022, mainly due to the different approach in the definition of the critical punching perimeter.

21.3 Comparison in terms of fire resistance

Concerning the behaviour of the studied elements in fire, the check of bending resistance was carried out with analytical method as shown in the previous chapters with reference to the exposure time to the nominal standard ISO834 fire curve requested (60 minutes). This check was proved to never be critical concerning the design of concrete sections and reinforcement, apart from the addition of polypropylene microfibres for the high-strength concrete mix of the column elements following FprEN1992:2022 only.

The following table resumes the bending resistance of the investigated elements in terms of time (in minutes) associated with the attainment of their ultimate strength. It can be commented that the effect of the different conductivity curves of concrete (between the lower proposed in EN1992:2004 and the single one proposed in FprEN1992:2022) is limited to very few percentage points and practically negligible, for all cases where the same concrete cross-section was compared. Higher differences in the thermal fields are expected to be observed at early stages of exposure, typically far from being associated to structural safety issues.

	Bending resistance to ISO834 [min]		
	1	2	
Element	EN1992:2004*	FprEN1992:2022	(2 - 1) / 1 [%]
TT element	82	80	-2.44
Hollow core	116	115	-0.86
Lattice girder	272	270	-0.74
Prestressed beam	154	153	-0.65
Reinforced beam	196	195	-0.51
Column‡	130	169	+30.00

*Calculated using the lower conductivity curve of concrete

‡Columns designed with different codes have different concrete section

21.4 Comments about easiness to read and apply FprEN1992-1:2022

Different considerations may be drawn from the comparison of the current and proposed versions of Eurocode 2.

Concerning FprEN1992-1-1:2022, it can be preliminarily observed that the number of pages is relevantly grown to 410 pages from the 227 pages of the current version of EN1992-1-1:2004, which by itself is a factor which determines a difficulty in handling and finding the required information in such a large volume of pages.

In particular, FprEN1992-1-1:2022 is characterised by a very large number of initial pages dedicated to list of content, introduction, normative references and especially definitions and symbols, which ends at page 66. For reference, in the current version of the document this section ends at page 20. Indeed, the section dedicated to definitions and symbols may seem very large in the new version of the document, although more attention has been paid to avoid any duplication of symbols, with potential confusing or unclear meaning by the reader, which sometimes happens in the current version of the code. On the other hand, the meticulous avoidance of symbol repetitions in the new document brought to the definition of a whole new set of symbols, which not always have direct and immediate correspondence to the symbols used in the current version of the document. This adds difficulties in comparing the procedures of the two codes, although the documents contain all information to solve this issue and find the correct corresponding terms/symbol from the current to the new version.

One positive aspect of the organisation of FprEN1992-1-1:2022 concerns the correspondence of the order of the main content with respect to EN1992-1-1:2004: Basis of design ; Materials ; Durability and concrete cover ; Structural analysis ; Ultimate Limit States (ULS) ; Serviceability Limit States (SLS) ; Fatigue ; Detailing of reinforcement and post-tensioning tendons ; Detailing of members and particular rules ; Additional rules for precast concrete elements and structures ; Plain and lightly reinforced concrete structures. The only main chapter which is not recalled in an equal or very similar manner in the current code is the chapter dedicated to Lightweight aggregate concrete structures, which became Annex M in FprEN1992-1-1:2022. Indeed, also the structure of the annexes is very similar between the two documents, despite the number of them has increased significantly with the proposed version, passing from J (10) annexes to S (19) annexes. The large number of changes in the new document version also makes the comparison and the interpretation of the new document rather complex. This consideration has repercussions over the compilers of informatic codes solving structural issues, as well as over design practitioners which created their own design instruments: they will need to deeply revisit their codes/spreadsheets to adapt the many changes from simple terms to completely different formulations and approaches to usual structural design issues. It is also recalled that some chapters, especially those concerning the resistance to shear and punching shear of unreinforced members, appear being more complex and more possibly subjected to interpretation with respect to the previous version of the code.

A quite different scenario concerns the document about fire behaviour and resistance FprEN1992-1-2:2022: the total number of pages of the new document is 88, relevantly less with respect to the 100 pages of the current version, denoting a larger synthesis of the information contained. Also in this case, apart from a more structured introduction and section about terms and definitions, the scheme of the new document remains the same, with some changes in the end of it, where information about simplified design methods, advanced design methods, and spalling, previously contained in specific annexes, was moved (reasonably) to main chapters. Moreover, as previously discussed, the changes between these documents are limited and clearly introduced, making it rather easy to understand and correctly interpret them.

It can be finally observed that all documents share a similar style, which makes easier their comparison and in general the understanding of the differences introduced with the proposed new versions, although the document is clearly devoted to instructed technicians and not to a wider audience.

21.5 Final remarks and future work

As a final consideration, it can be noted that the differences between the two standards are many, and that they include practically all the steps of the design process, starting from the definition of the constitutive laws of the structural materials, down to the definition of details such as the anchorage length. These differences are reflected in the comparative design that was carried out. Nevertheless, the final outcome of the design in terms of element dimensions and reinforcement ratios is reasonably similar between the two standards, with deviations which are in the end limited to few percentage points. To be noted that the differences were found sometimes in favour (smaller volume of materials) of one standard, and sometimes in favour of the other, proving a further balancing between the economical and environmental impact of precast concrete elements designed following the two standards.

Future developments of the work may include an update at the time of the official publication of the new Eurocode 2, when the current draft may have been subjected to modification, and also in the phase of identification of the Nationally Determined Parameters (NDPs) and Non-Contradictory Complementary Information (NCCI) to be selected within the draft of the national annexes by the several mirror groups of the European member countries. Moreover, it would be interesting to extend the comparison also to elements having longer span, typical of industrial buildings.