A STRUCTURAL ANALYSIS OF A FACTORY HALL WITH A TWO-STORY OFFICE BUILDING AS A PRECAST AND PRESTRESSED CONCRETE CONSTRUCTION

By

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ABSTRACT

The objective of this thesis is to develop a precast and prestressed concrete design for a factory hall, which was initially planned as a steel structure. Furthermore, a structural analysis is conducted on several chosen structural elements according to the European Standards and the German Annexes respectively. The analysis is done both by manual calculation and software calculation for comparison and includes the ultimate limit state design, the serviceability limit state design and the design for the state of transportation and assembly of the precast members. Lastly, to illustrate the results of the analysis, an overview drawing with the new concrete design as well as formwork and reinforcement drawings for each of the analyzed structural members are developed.

ACKNOWLEDGEMENTS

I would like to express my gratitude to my supervisor at HsKA, Dr. Christian Enderle, for his persistent support and guidance. His valuable experience and professional expertise was of great help throughout this endeavour. Furthermore, I thank him for always being patient and understanding throughout the process of crafting this thesis.

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1 INTRODUCTION

1.1 PROBLEM DEFINITION

The objective of this MRP is to develop an alternative proposal for the design of a pre-existing factory hall built in Betzweiler-Wälde, Germany, and planned by the company *Schöne + Seeburger / Schwäbisch Gemünd.* The original design of the factory hall was a steel structure and the given layout drawings are based on a steel design. As an alternative proposal, the hall will be designed as a precast and prestressed concrete structure. A structural analysis must be conducted in a form that a test engineer could approve of. Due to the large scale of this building and the number of different structural members, only an example of each of the main structural parts of the building are to be designed.

This includes the design of:

- the roof covering
- a prestressed concrete purlin
- a prestressed concrete truss
- the main concrete columns and the supports of the truss
- the exterior walls
- the concrete slab and one beam of the office section
- a staircase
- the main single and strip foundations.

Additionally, a rough design of a steel canopy for the entry of the building will be completed. All calculations must be done according to the European Standards, *the Eurocodes*, and respectively the German national annexes.

In addition, several layout and reinforcement drawings must be made based on the results of the design calculations.

Following drawings must be included:

- A layout drawing of the roof of the factory hall
- A sections and details drawing of the factory hall
- A formwork and reinforcement plan of the main roof truss
- A formwork and reinforcement plan of one column
- A formwork and reinforcement plan of one beam
- A formwork and reinforcement plan of an excerpt of the slab
- A formwork and reinforcement plan of one individual foundation.

1.2 DESCRIPTION OF THE STRUCTURE

Figure 1 shows the layout of the building, which is divided into two parts. The main part is the factory hall, which functions as a large office furniture factory. The smaller part in the lower right corner is a two-story office section which is framed by the large hall structure.

The hall structure measures about 120 metres in length and 91 metres in width with a maximum height of 10.65 meter. The office section is 98 metres long and 15 metres wide. For an easier overview, the layout of the building is divided into a grid. The short side of the building is divided by an alphabetical axis every 30 meters and the long side is divided by a numbered axis every 15 metres. The main columns are arranged at each intersection of this grid.

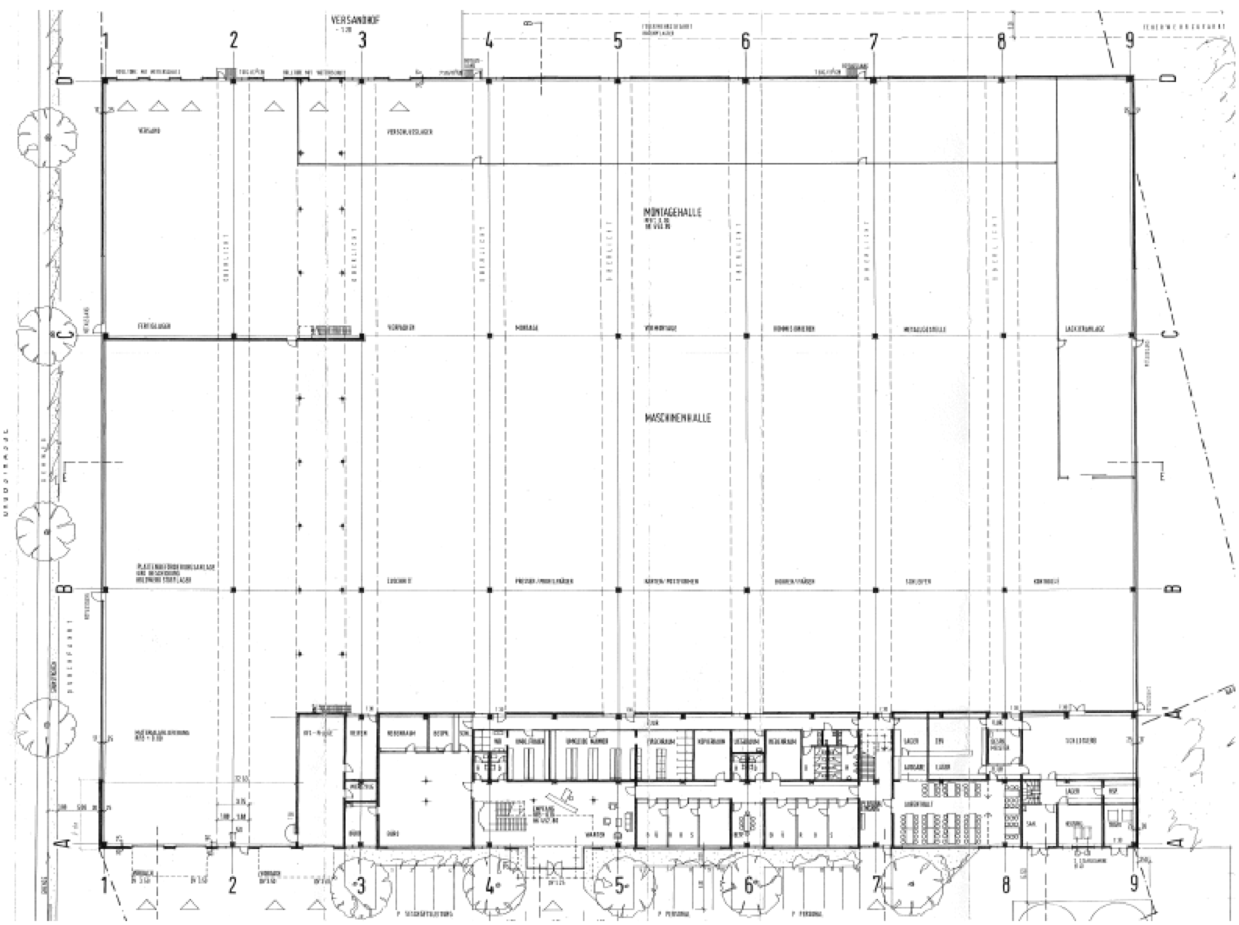


Figure 1 - Layout Drawing of the Factory Hall by Schöne + Seeburger

Figure 2 and Figure 3 show the layout of the first and second floor of the office section, respectively. In addition to the axis of the hall, another axis (A’ between A and B) was added. Along this axis, an additional column row for the office floor is provided, as well as additional columns along the middle axis of the office building. The entry of the office building is between axis 4 and 5, and as it is planned to be a big open space, the slab between the first and second floor is cut out above the entrance area. A large staircase leads to the second floor.

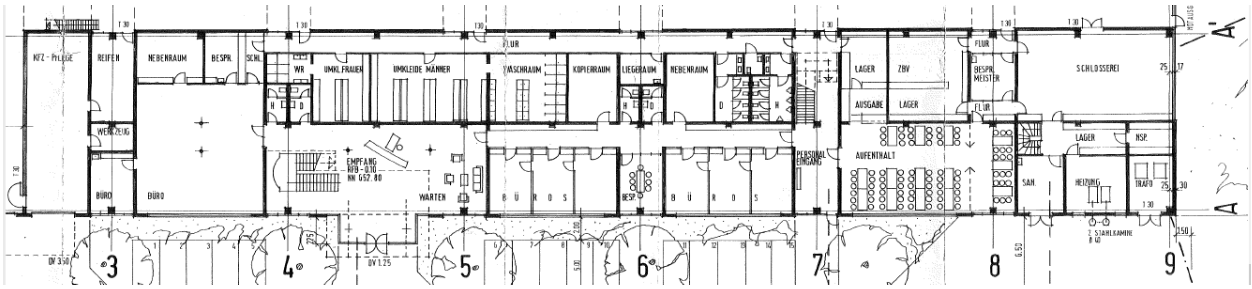


Figure 2 - Layout Drawing of the Ground Floor of the Office Section of the Building by Schöne + Seeburger

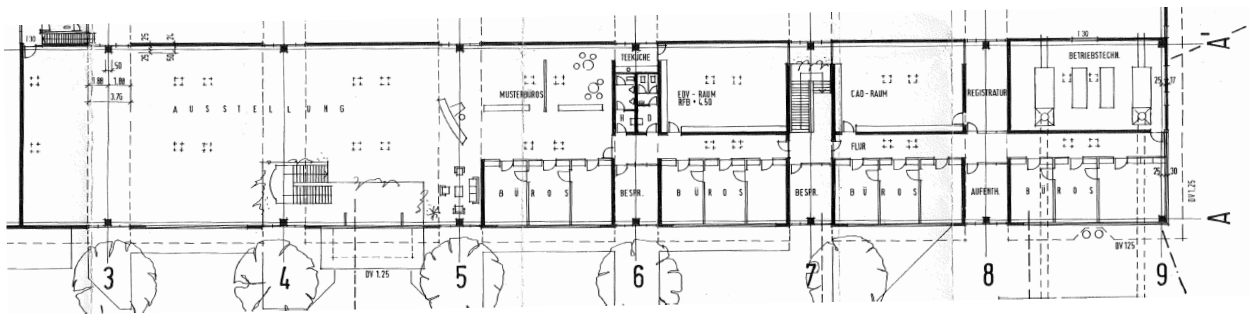


Figure 3 - Layout Drawing of the Second Floor of the Office Section by Schöne + Seeburger

Figure 4 to Figure 6 show different section drawing of the factory hall. Section B-B (Figure 4) cuts along the short side of the building in between axis 4 and 5. As mentioned previously, these diagrams demonstrate the open structure of the entrance to the office building and also shows how the office is framed by the large structure of the hall instead of being added to it as a separate structure.

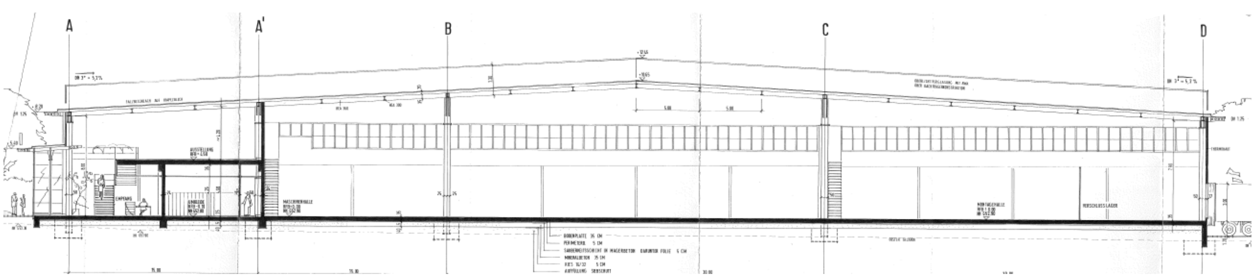


Figure 4 - Section B-B of the Hall by Schöne + Seeburger

Section E-E (Figure 5) is a cut along the long side of the building an shows the big factory hall.

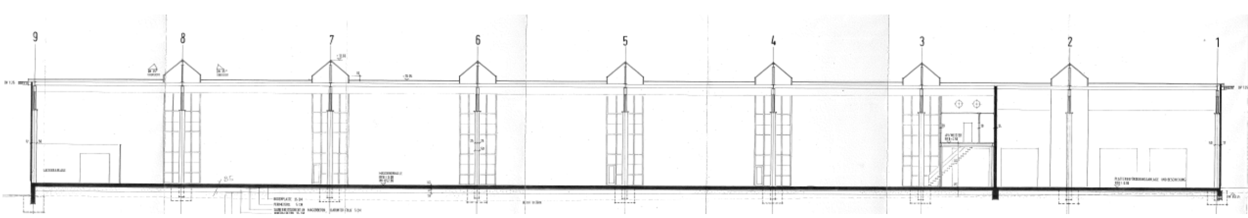


Figure 5 - Section E-E of the hall by Schöne + Seeburger

Section C-C (Figure 6) is another cut through the office building and shows the second staircase.

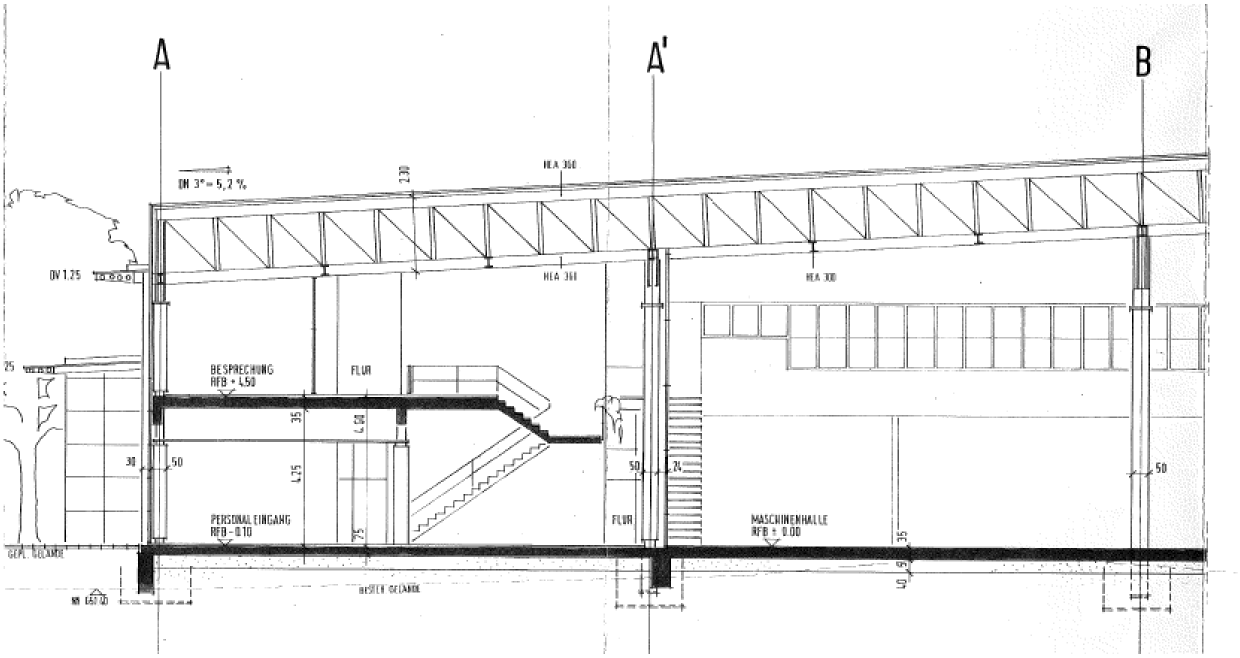


Figure 6 - Section C-C of the Hall by Schöne + Seeburger

1.3 GUIDELINES

Apart from the measurements and the geometry given in the drawings, certain guidelines for the new design of the building as a concrete structure (instead of a steel structure) must be met.

1.3.1 HALL STRUCTURE

The hall structure must consist of:

- Trapezoidal sheets with insulation as roof covering
- Prestressed concrete purlins with a length of approximately 15 metres and in a of 5 metres between each other
- Prestressed concrete trusses with a length of about 30 metres and in a distance of 15 metres between each other
- Clamped concrete columns which are intended to brace the hall structure against wind
- Trapezoidal sheet cladding for the façade.

1.3.2 OFFICE BUILDING

The office building must consist of:

- Trapezoidal sheets with insulation as roof covering
- A composite plank floor with precast concrete beams
- Clamped concrete columns which are intended to brace the hall structure against wind
- Concrete wall panels.

2 LOAD DETERMINATION

2.1 DEAD LOADS

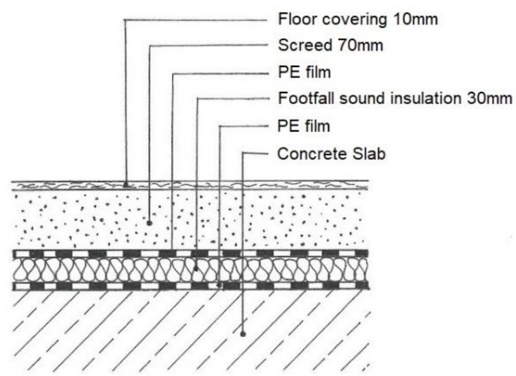
The dead loads mainly result from the roof and floor structure.

2.1.1 FLOOR STRUCTURE

The composition of the dead loads resulting from the floor structure is summarized in Table 1.

Table 1 - Dead Loads from the Floor Structure

	Layer	Thickness [cm]	Specific weight	Loads [kN/m²]
1	Floor covering	1.00	0.22 kN/m² / cm	0.22
2	Screed	7.00	0.22 kN/m² / cm	1.54
3	PE film	≈ 0.1	≈ 0.01 kN/m²	0.01
4	Footfall sound insulation	3.00	0.01 kN/m² /cm	0.03
5	PE film	≈ 0.1	≈ 0.01 kN/m²	0.01
			Σ =	**1.81**
				≈ 2.00

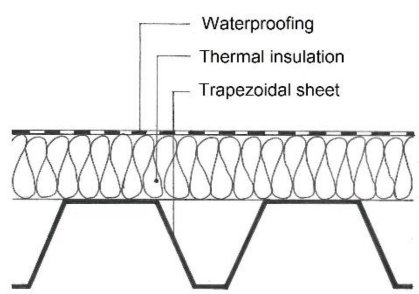


2.1.2 ROOF STRUCTURE

The composition of the dead loads resulting from the roof structure is summarized in Table 2.

Table 2 - Dead Loads from the Roof Structure

	Layer	Thickness [cm]	Specific weight	Loads [kN/m²]
1	Waterproofing	≈ 0.5	0.02 kN/m²	0.02
2	Insulation	10.0	0.02 kN/m² / cm	0.20
3	Trapezoidal sheet		≈ 0.15 kN/m²	0.15
			Σ =	**0.37**
				≈ 0.50



2.2 LIVE LOADS

As per DIN EN 1991-1-1 and DIN EN 1991-1-1/NA the usage category of the building must be defined with the respective live load which has to be considered in the structural analysis. Since the building is of mixed usage, with office areas on one side and storage areas on the other, different usage categories must be observed. They can be seen in Table 3.

Table 3 - Live Loads According to the Usage Categories

Category*	Usage	q_k [kN/m²]	Q_k [kN]
B1	Office areas	2.0	2.0
E1	Warehouses and workshops without forklift traffic	5.0	4.0
E2.5	Warehouses with forklift traffic of the category FL4**	20.0	2 x 90.0
T1	Stair cases and -landings in office buildings	3.0	2.0

*	According to DIN EN 1991-1-1/NA Tab. 6.1DE
**	According to DIN EN 1991-1-1 Tab. 6.5
q_k	Characteristic distributed live load
Q_k	Characteristic concentrated live load

2.3 WIND LOADS

The wind loads are determined according to DIN EN 1991-1-4. The value depends on the location and the height of the building.

Location oft he Building: Betzweiler-Wälde, Baden-Württemberg, Germany

Wind zone (Figure 9): Wind zone 1

Terrain category: Mixed profile “Inland”

Height of the building: z = 10.65 m

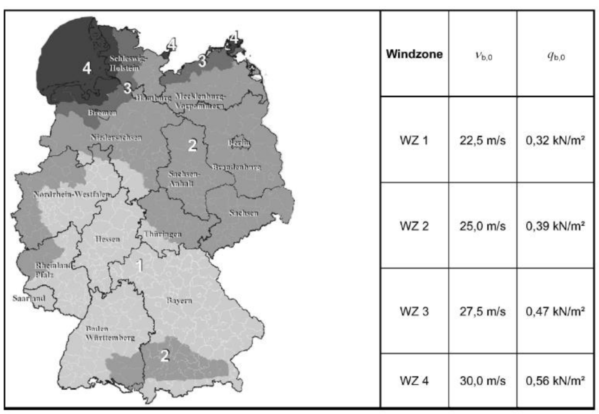


Figure 9 - Wind Zone Map for Germany taken form DIN EN 1991-1-4 NA

The velocity pressure for buildings with a height above 7m and in the category above is calculated according to following formula:

$$q_p(z) = 1.7 \cdot q_{b,0} \cdot \left(\frac{z}{10}\right)^{0,37} \qquad \text{for } 7\text{m} < z \leq 50\text{m}$$

With q_p: Velocity Pressure

z: Height of the building

$q_{b,0}$: Basis velocity pressure.

With this formula the velocity pressure can be calculated as:

$$q_p(10,65) = 1.7 \cdot 0.32 \cdot \left(\frac{10.65}{10}\right)^{0,37} = 0.56 kN/m^2$$

2.3.1 WIND PRESSURE ON THE ROOF

In accordance with DIN EN 1991-1-4 the coefficients for the wind pressure on the roof can be divided into five different areas F to J as it is shown in Figure 10.

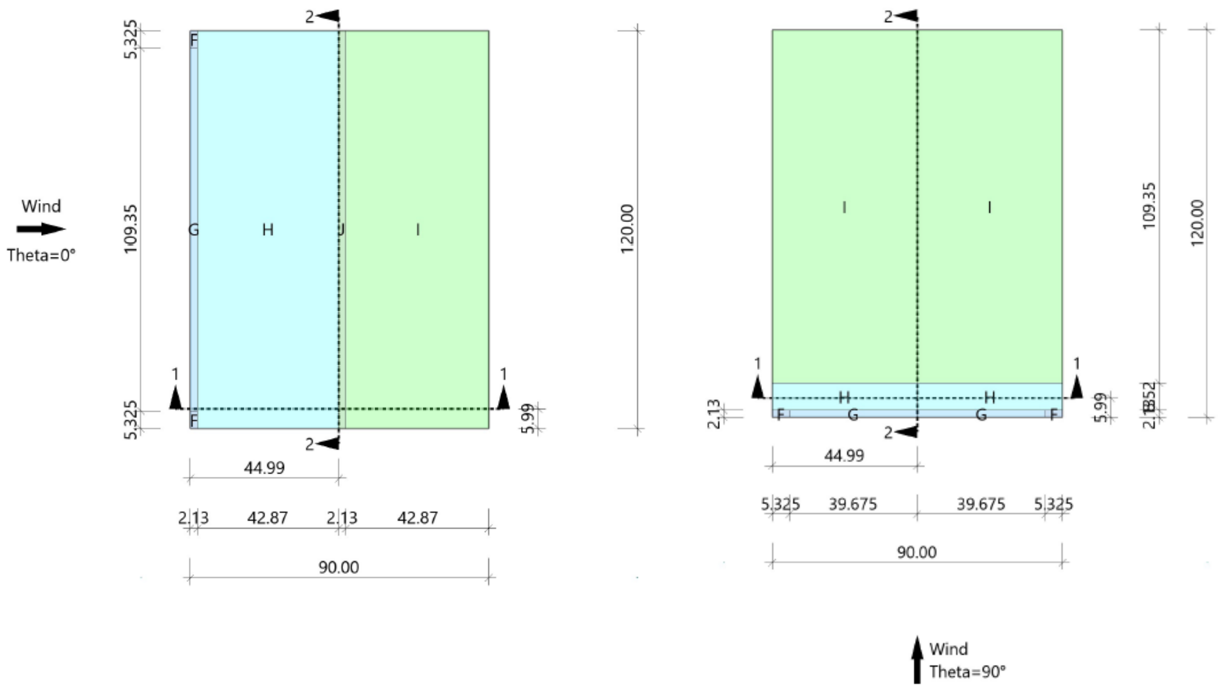


Figure 10 - Definition of Areas on the Roof with different Wind Pressure Coefficients

The following table sums up the different wind pressure coefficients which must be taken into account in the determination of the decisive wind pressure on the roof of the building.

Table 4 - Wind Pressure Coefficients and Wind Pressure on different Areas of the Roof

	Inflow Direction θ = 0°				Inflow Direction θ = 90°			
Area	$c_{pe,10}$ [-]	$c_{pe,1}$ [-]	$w_{e,10}$ [kN/m²]	$w_{e,1}$ [kN/m²]	$c_{pe,10}$ [-]	$c_{pe,1}$ [-]	$w_{e,10}$ [kN/m²]	$w_{e,1}$ [kN/m²]
F	- 1.80	- 2.50	- 1.00	- 1.39	- 1.80	- 2.50	- 1.00	- 1.39
G	- 1.20	- 2.00	- 0.67	- 1.11	- 1.20	- 2.00	- 0.67	- 1.11
H	- 0.70	- 1.20	- 0.39	- 0.67	- 0.70	- 1.20	- 0.39	- 0.67
I	+0.20 - 0.60	+0.20 - 0.60	+0.11 - 0.33	+0.11 - 0.33	+0.20 - 0.60	+0.20 - 0.60	+0.11 - 0.33	+0.11 - 0.33
J	+0.20 - 0.60	+0.20 - 0.60	+0.11 - 0.33	+0.11 - 0.33	-	-	-	-

$c_{pe,10}$ Wind Pressure Coefficient for an Area A > 10 m²
$c_{pe,1}$ Wind Pressure Coefficient for an Area 1 m² < A ≤ 10 m²
$w_{e,10}$ Wind Pressure for an Area A > 10 m²
$w_{e,1}$ Wind Pressure for an Area 1 m² < A ≤ 10 m²

2.3.2 WIND PRESSURE ON THE EXTERIOR WALLS

Like the wind pressure on the roof, the pressure on the walls must be divided into five different areas, A to E, with the respective coefficients. The wind pressure distribution on the walls can be seen in Figure 11.

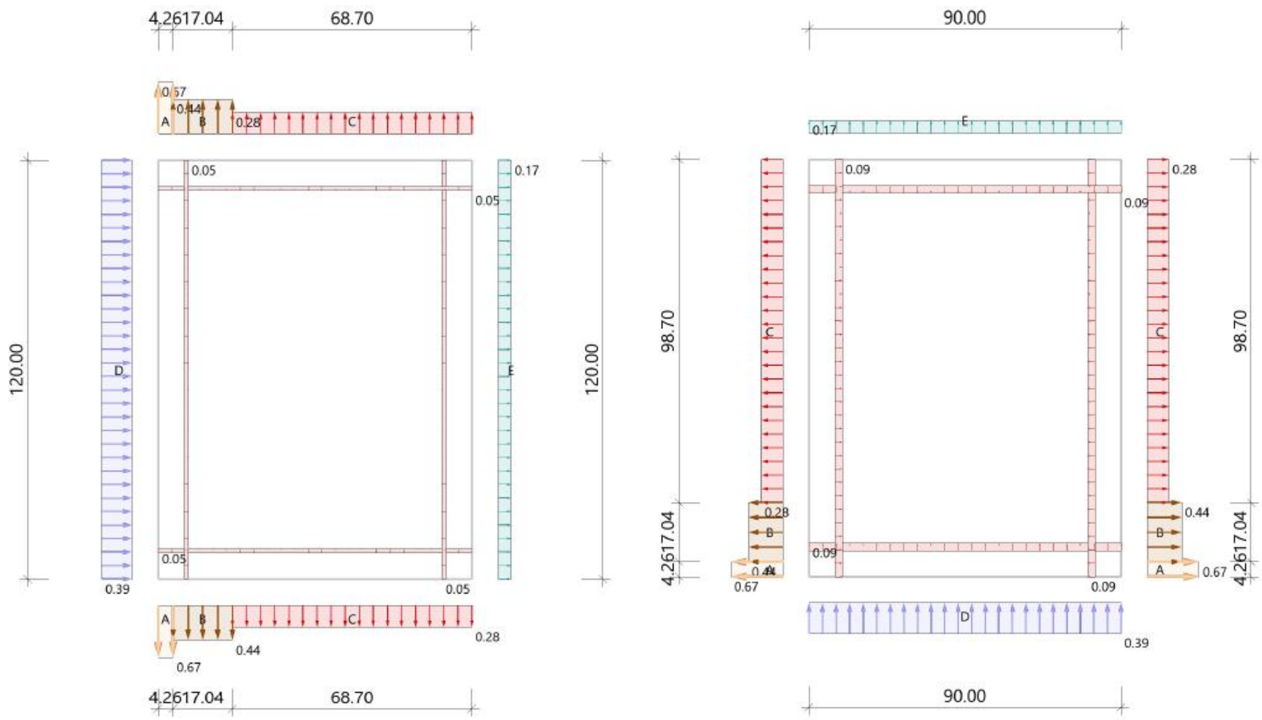


Figure 11 - Wind Pressure Distribution on the Exterior Walls (left: θ = 0°, right: θ = 90°)

Wind pressure coefficients and wind pressure distributions are summed up in following table.

Table 5 - Wind Pressure Coefficients and Wind Pressure on different Areas of the Exterior Walls

	Inflow Direction θ = 0°				Inflow Direction θ = 90°			
Area	$c_{pe,10}$ [-]	$c_{pe,1}$ [-]	$w_{e,10}$ [kN/m²]	$w_{e,1}$ [kN/m²]	$c_{pe,10}$ [-]	$c_{pe,1}$ [-]	$w_{e,10}$ [kN/m²]	$w_{e,1}$ [kN/m²]
A	- 1.20	- 1.40	- 0.67	- 0.78	- 1.20	- 1.40	- 0.67	- 0.78
B	- 0.80	- 1.10	- 0.44	- 0.61	- 0.80	- 1.10	- 0.44	- 0.61
C	- 0.50	- 0.50	- 0.28	- 0.28	- 0.50	- 0.50	- 0.28	- 0.28
D	+0.70	+1.00	+0.39	+0.56	+0.70	+1.00	+0.39	+0.56
E	- 0.30	- 0.50	- 0.17	- 0.28	- 0.30	- 0.50	- 0.17	- 0.28

$c_{pe,10}$ Wind Pressure Coefficient for an Area A > 10 m²
$c_{pe,1}$ Wind Pressure Coefficient for an Area 1 m² < A ≤ 10 m²
$w_{e,10}$ Wind Pressure for an Area A > 10 m²
$w_{e,1}$ Wind Pressure for an Area 1 m² < A ≤ 10 m²

2.4 SNOW LOADS

The snow loads are determined according to DIN EN 1991-1-2 and similar to the wind loads, the snow loads depend on location and height above sea level of the building.

Snow zone: 2a

Height above sea level: A = 660.00 m

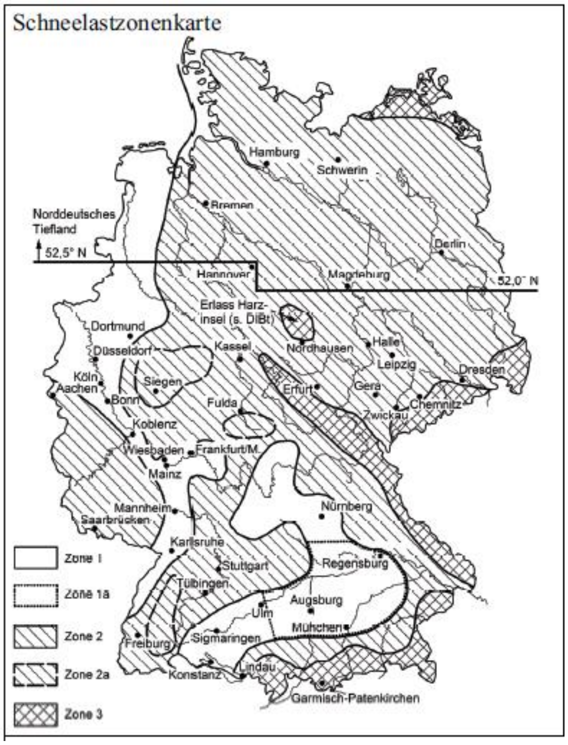


Figure 12 - Snow Zone Map for Germany [1]

The general snow load for snow zone 2a is calculated according to following formula:

$$s_k = 1.25 \cdot \left[0.25 + 1.91 \cdot \left(\frac{A+140}{760} \right)^2 \right] \geq 1.06$$

with A: Height above sea level.

Using this formula, the snow load is:

$$s_k = 1.25 \cdot \left[0.25 + 1.91 \cdot \left(\frac{660+140}{760} \right)^2 \right] = 2.96 kN / m^2 > 1.06 .$$

The shape of the roof of the building is considered with a snow coefficient μ which is depends on the roof inclination α . In the case of a saw-tooth roof, two different coefficients must be defined to take snow drifts into consideration, as is illustrated in Figure 13.

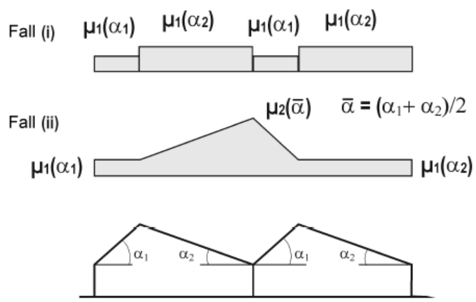


Figure 13 - Snow Loading Arrangement for Saw-Tooth Roofs taken from DIN EN 1991-1-3 Bild 5.4

Since $\alpha_1 = \alpha_2 = 3{,}1°$ the coefficients and respectively the snow loads are:

$\mu_1 = 0.8$ → $s_k = \mu_1 \cdot s_{k,0} = 0.8 \cdot 2.96 kN / m^2 = 2.37 kN / m^2$

$\mu_2 = 0.8 + 0.8 \cdot \alpha / 30° = 0.88$ → $s_k = \mu_2 \cdot s_{k,0} = 0.88 \cdot 2.96 kN / m^2 = 2.60 kN / m^2$

2.5 ACCIDENTAL LOADS

In addition to the imposed loads above, there are accidental loads to be considered because of expected forklift traffic in the building. This causes not only additional vertical loads but also horizontal impact loads on the columns of the hall resulting from a possible collision of the forklift trucks with the columns.

In this case the forklifts have a permissible total weight of 7.5 t, which is 75 kN. According to DIN EN 199111 Tab. 6.5, the forklifts can be classified in the category FL4 with a maximum total weight (including self-weight and lifting capacity) of 100 kN.

DIN EN 1991-1-7 says that the horizontal impact load caused by forklift collision must be considered with an equivalent load *F = 5W*, with W being the permissible total weight of the forklift. The load must be positioned in a height of 0.75 m above ground.

$F = 5W = 5 \cdot 75 kN = 375 kN$

This means that the equivalent load for a forklift collision for this building amounts to 375 kN.

3 STRUCTURAL ANALYSIS

The following chapter gives a quick overview over the main design guidelines and requirements given in the European Standards, the Eurocodes. Most significant for the design of concrete structures are the Eurocodes 0 to 2, and respectively their national annexes, per the german annex.

Eurocode 0 (EC0) describes the basis of structural design and must be applied through all types of designs. It includes the standards for load and resistance factors as well as action combinations. The different types of actions on structures and load cases, like imposed loads for buildings, wind loads, snow loads and others are included in the Eurocode 1 (EC1). Finally, the Eurocode 2 (EC2) regulates the design and construction of reinforced and prestressed concrete structures.

3.1 ACTION COMBINATIONS AND FACTORED LOADS

As mentioned above, the different action combinations for the ultimate limit state design and the serviceability limit state design are introduced in the EC1. The different actions and their formula symbols that are described in this chapter and used throughout this MRP are summarized in Table 6.

Table 6 - Actions on Structures

Permanent Loads		**Variable Loads**	
Dead load	G_k	Live load	Q_k
Prestressing load	P_k	Snow load	S_k , $Q_{k,S}$
		Wind load	W_k , $Q_{k,W}$
Accidental load			A_d

3.1.1 ULTIMATE LIMIT STATE (ULS)

The different action combinations for the ultimate limit sate can be seen in Table 7, where

G_k Characteristic dead load

Q_k Characteristic live load / variable load

γ Safety factor (see Table 8 or DIN EN 1990, Appendix A, Tab. NA.A. 1.2 (B))

ψ_i Combination coefficient (see DIN EN 1990, Appendix A, Tab. A.1.1).

Table 7 - Action Combinations for the ULS

Loading Situation	**Action Combinations**[1)]
Permanent and temporary loads $Q_{Ed} =$	$\sum_{j\geq1}\gamma_{G,j}\cdot G_{k,j}\oplus\gamma_{Q,1}\cdot Q_{k,1}\oplus\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i}$
Accidental loads $Q_{Ad} =$	$\sum_{j\geq1}\gamma_{GA,j}\cdot G_{k,j}\oplus A_d\oplus\gamma_{QA,1}\cdot\psi_{1,1}\cdot Q_{k,i}\oplus\sum_{i>1}\gamma_{QA,i}\cdot\psi_{2,i}\cdot Q_{k,i}$
1) According to DIN EN 1990, 6.4.3	

The safety factors used above are shown in Table 8.

Table 8 - Safety Factors for the ULS

Actions		Final State	State of Construction
Permanent	$\gamma_G =$	1.35 (1.00) 1)	1.15 2)
Variable	$\gamma_Q =$	1.50 (0) 1)	
Prestressing	$\gamma_P =$	1.00 3)	
1) According to DIN EN 1990, Appendix A, Tab. NA.A. 1.2 (B) 2) According to DIN EN 19992-1-1, 10.2 (NA.4) 3) According to DIN EN 1992-1-1, 2.4.2.2			

3.1.2 SERVICEABILITY LIMIT STATE (SLS)

The action combinations for the serviceability limit state are described in Table 9.

Table 9 - Action Combinations for the SLS

Loading situation		Action combinations 1)
Rare situation	$Q_{Ed,,rare} =$	$\sum_{j\geq1} G_{k,j} \oplus Q_{k,1} \oplus \sum_{i>1} \psi_{0,i} \cdot Q_{k,i}$
Frequent situation	$Q_{Ed,frequ} =$	$\sum_{j\geq1} G_{k,j} \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i>1} \psi_{2,i} \cdot Q_{k,i}$
Permanent situation	$Q_{Ed,perm} =$	$\sum_{j\geq1} G_{k,j} \oplus \sum_{i>1} \psi_{2,i} \cdot Q_{k,i}$
1) According to DIN EN 1990, 6.5.3		

3.2 BUILDING MATERIALS AND CHARACTERISTIC VALUES

The building materials used for the structure are concrete, reinforcing steel and prestressing steel. The property values of these materials are defined in the EC 2 and the following chapter will give a short overview of the main properties needed for the structural design.

3.2.1 CONCRETE

The EC 2 distinguishes 15 different concrete strength classes starting at C1215 and ending with C100/115. The first 9 classes and the most relevant for the following structural design are listed in Table 10. The concrete strength classes describe the compressive strength of the concrete based on their characteristic cylinder strength, $f_{ck,}$ and their cube strength, $f_{ck,cube,}$ which are determined at a concrete age of 28 days. The value f_{cd} is the factored cylinder strength with a safety factor of $\gamma_c = 1.5$ and another reduction factor $\alpha_{cc} = 0.85$.

Table 10 - Concrete Properties

[N/mm²]*	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck}	12	16	20	25	30	35	40	45	50
$f_{ck,cube}$	15	20	25	30	37	45	50	55	60
f_{cm}	20	24	28	33	38	43	48	53	58
f_{cd}	6.8	9.1	11.3	14.2	17.0	19.8	22.7	25.5	28.3
f_{ctm}	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1
E_{cm}	27,000	29,000	30,000	31,000	33,000	34,000	35,000	36,000	37,000
	* N/mm² = MPa								

3.2.2 REINFORCING STEEL

The EC2 applies to ribbed and weldable reinforcement and it specifies its requirements. The most important properties of reinforcing steel are a high tensile strength, good bond characteristics and a high ductility. Following table describes the properties of the reinforcing steel used in the structural design.

Table 11 - Reinforcing Steel Properties

Type	Ductility	f_{yk} [Nmm²]*	f_{yd} [N/mm²]	f_{tkcal} [N/mm²]	ε_{ud} [%]	E_s [N/mm²]
B500A	regular	500	435	525	25	200,000

*	N/mm² = MPa	$f_{tk,cal}$	tensile strength
f_{yk}	yield strength	ε_{ud}	strain limit
f_{yd}	design yield strength	E_s	modulus of elasticity

3.2.3 PRESTRESSING STEEL

The prestressing steel that is used for this design is a seven-wire strand and the properties can be taken from the Building Control Certification Nr. Z-12.3-107 from the German Institute for Structural Engineering.

3.3 DURABILITY AND CONCRETE COVER

In order to ensure the bond between the concrete and reinforcing steel, as well as the prestressing steel and to protect the steel against corrosion and fire, a minimum distance between the surface of the concrete and the first layer of reinforcement bars is required. These requirements are defined in DIN EN 1992-1-1 and the national appendix, respectively.

The total concrete cover c_{nom} is composed of two values. The minimum concrete cover c_{min} and an allowance Δc_{dev} which takes possible deviations in the construction process into account.

$$c_{nom} = c_{\min} + \Delta c_{dev}$$

with $c_{\min} = \max \begin{cases} c_{\min,b}(bond) \\ c_{\min,dur}(durability) \\ 10mm \end{cases}$ and $\Delta c_{dev} = 15mm$ (for XC1 only 10mm).

In case of precast concrete, Δc_{dev} can be reduced by 5 mm if a high production quality is ensured and the concrete cover is regularly tested.

The values for $c_{min,b}$ are defined as follows:

For reinforcing steel: $c_{\min,b} = \varnothing_s$ where $\varnothing_s$ is the diameter of the reinforcement bar.

For prestressing steel: $c_{\min,b} = 2.5\varnothing_P$ (in case of immediate bonding).

The value for $c_{min,dur}$ depends on the exposure class of the concrete and can be taken from Table 12. Exposure classes describe different corrosive environments, which may cause damage to the concrete and the reinforcement, and therefore are a decisive factor in choosing the thickness of the concrete cover. The three exposure classes which are significant for the value of $c_{min,dur}$ and which are distinguished in Table 12 are XC1 to XC4 – concrete at risk of carbonation, XD1 to XD3 – concrete exposed to de-icing salt or other chlorides, and XS1 to XS3 – concrete in sea water environments.

Table 12 - Minimum Concrete Cover $c_{min,dur}$ in Relation to the Exposure Class

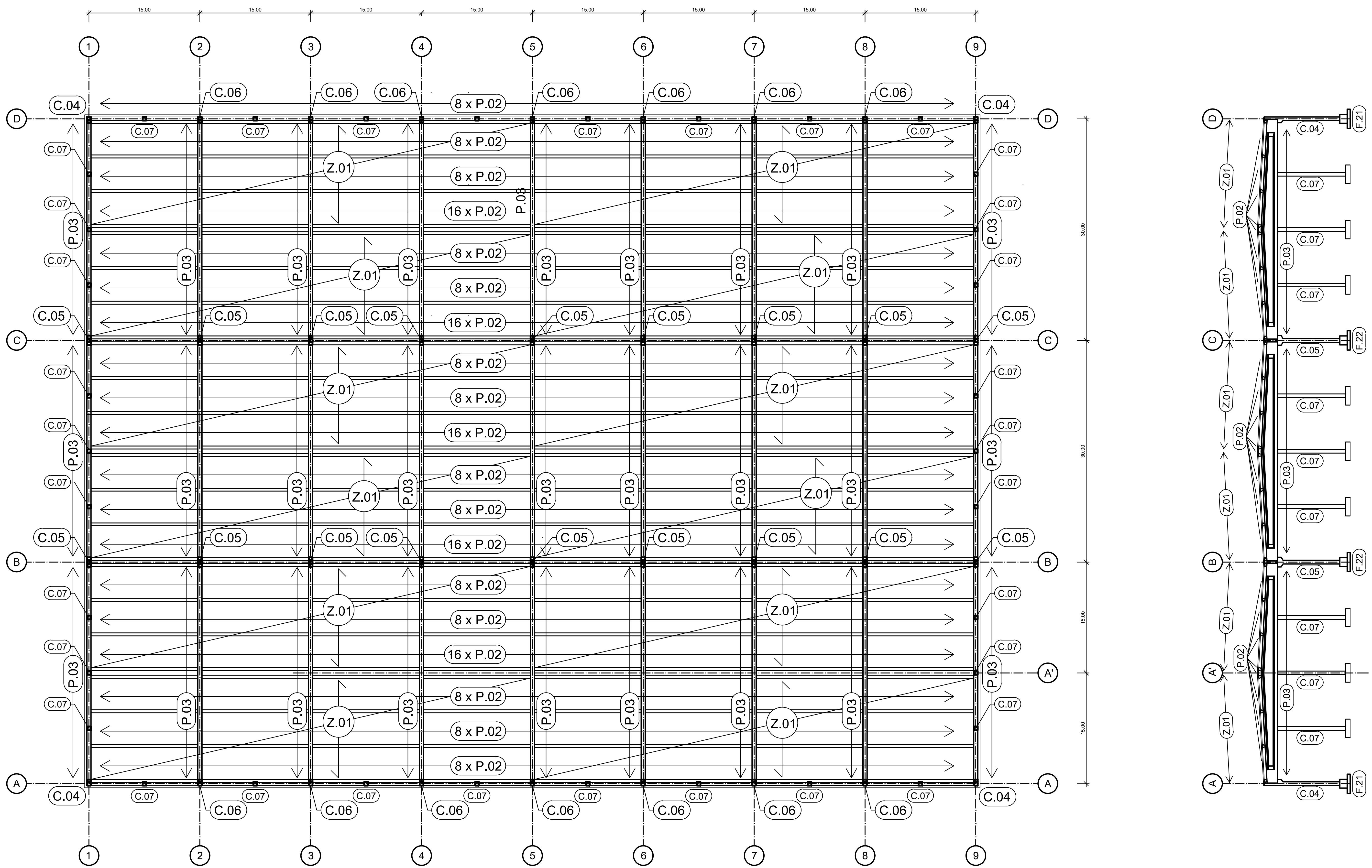
	$c_{min,dur}$ in [mm]			
Exposure Class	**XC1**	**XC2 XC3**	**XC4**	**XD1, XD2, XD3 XS1, XS2, XS3**
Reinforcing Steel	10	20	25	40
Prestressing Steel	20	30	35	50

4 POSITION PLAN

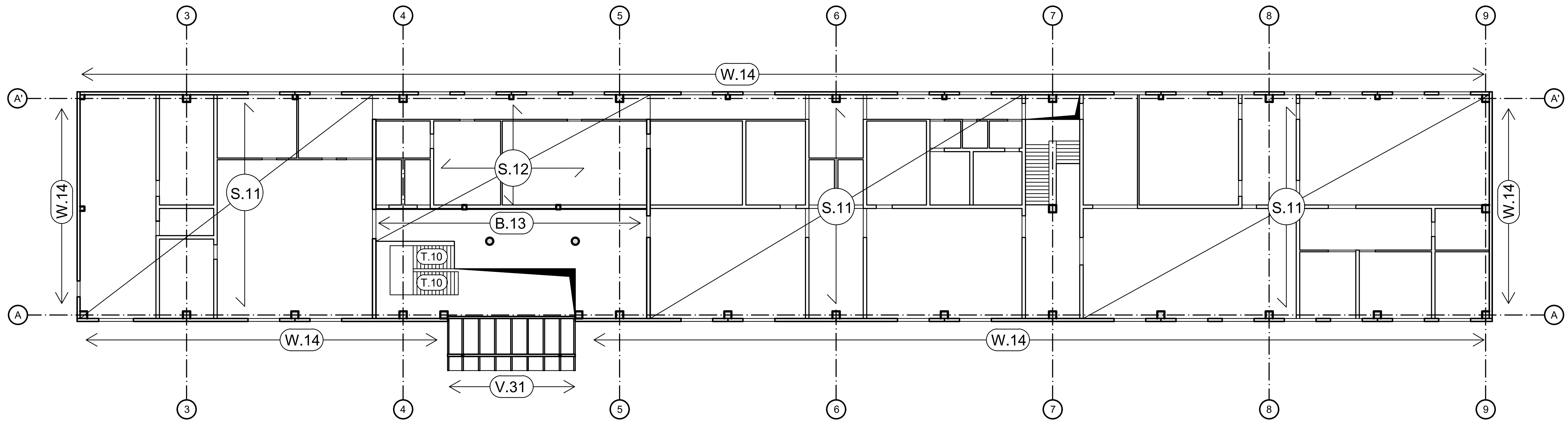
The drawings on the next two pages will give an overview of the different structural elements, the structural positions, which will be analyzed and designed in this paper. Each position is indicated by a letter and a two-digit number. The letters indicate the type of structural member and are assigned as follows:

Z = Trapezoidal sheet covering
P = Prestressed member
C = Column
S = Slab
B = Beam
T = Staircase
F = Footing
W = Wall
V = Steel Structures

The first of the two-digit number stands for different levels or parts of the building. The hall structure is indicated by 0, the office building by 1, the foundation of the building by 2 and any additional structures, like the steel canopy, are indicated by the number 3.



Scale:	Plan:	Page:
1:500	Position Plan of the Hall	16



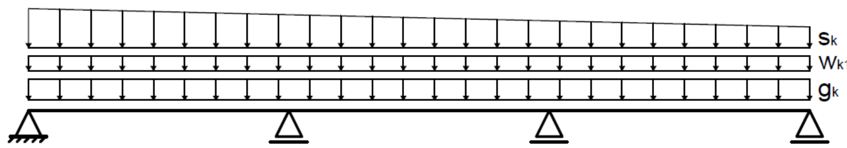
Scale:	Plan:	Page:
1:300	Position Plan of the Office	17

5 POS. Z.01 – TRAPEZOIDAL SHEET COVERING

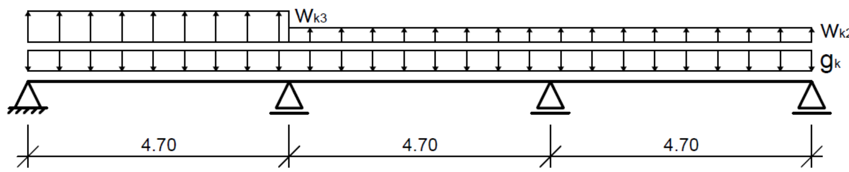
5.1 SYSTEM

System

1) Wind Pressure

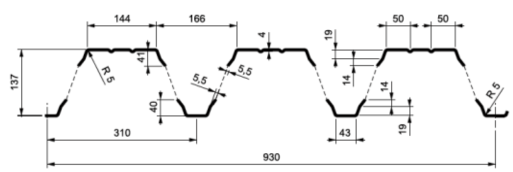


2) Wind Suction



Profile

Hoesch T 135.1, positive position, t_N = 1.00 mm , f_{yk} = 320 N/mm²



Characteristic Loads

Uniformly Distributed Loads		**g_k [kN/m²]**	**q_k [kN/m²]**
Self-weight	g_{k1} =	0.13	-
Roof Structure (Insulation, waterproofing, etc.)	g_{k2} ≈	0.30	-
Snow	min s_k =	-	2.37
Snow	max s_k =	-	2.60
Wind Loads		**g_k [kN/m²]**	**q_k [kN/m²]**
Wind Area I	w_{k1} =	-	0.11
Wind Area G	w_{k2} =	-	- 0.67
Wind Area F	w_{k3} =	-	- 1.00

Load Combinations

LC1:	$1.35 \times g_k + 1.50 \times s_k + 1.50 \times 0.6 \times w_{k1}$	(ULS)
LC2:	$1.00 \times g_k + 1.50 \times w_{k2/3}$	(ULS)
LC3:	$g_k + 0 \times s_k + 0 \times w_k$	(SLS, perm)

Chosen **HOESCH T135.1, positive position, t_N = 1.25 mm**
g = 0.162 kN/m² ,b_A = 40 mm, b_B = 160 mm

5.2 PRELIMINARY DESIGN

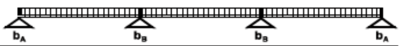
Design Loads 1) Wind PRESSURE: q_k = 0.30 x 1.35/1.5 + 2.49 + 0.11 = 2.87 kN/m²

2) Wind SUCTION: q_k = 0.30 x 1.0/1.5 – 0.67 = - 0.47 kN/m²

The preliminary design of the trapezoidal sheets can be done according to

Table 13 and Table 14 provided by the manufacturer HOESCH. The trapezoidal sheets must be designed for a pressing load as well as for a lifting load resulting from wind suction.

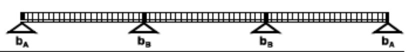
Table 13 - Sizing Table for the T135.1 as a Three-Span System with a Pressing Load



Dreifeldträger, zulässige andrückende Flächenlast zul q [kN/m²]

Stützweite L[m]			2,50	2,75	3,00	3,25	3,50	3,75	4,00	4,25	4,50	4,75	5,00	5,25	5,50	5,75	6,00	6,25	6,50	6,75	7,00	7,25	7,50	7,75	8,00
t_N	g	max f	Endauflagerbreite: b_A = 40 mm						Zwischenauflagerbreite: b_B = 160 mm																
0,75	0,097	*	4,34	3,94	3,62	3,34	3,03	2,75	2,51	2,29	2,11	1,94	1,80	1,67	1,58	1,50	1,37	1,27	1,19	1,12	1,05	1,00	0,94	0,89	0,84
		L/150	4,34	3,94	3,62	3,34	3,03	2,75	2,51	2,29	2,11	1,94	1,80	1,67	1,58	1,50	1,37	1,27	1,19	1,12	1,05	1,00	0,94	0,89	0,84
		L/300	4,34	3,94	3,62	3,34	3,03	2,75	2,51	2,29	2,11	1,94	1,80	1,67	1,58	1,50	1,37	1,24	1,10	0,98	0,88	0,79	0,72	0,65	0,59
		L/500	4,34	3,94	3,62	3,34	3,03	2,75	2,51	2,29	1,99	1,69	1,45	1,25	1,09	0,95	0,84	0,74	0,66	0,59	0,53	0,48	0,43	0,39	0,35
0,88	0,114	*	6,31	5,62	5,01	4,49	4,05	3,67	3,35	3,06	2,82	2,65	2,44	2,23	2,07	1,93	1,81	1,69	1,59	1,49	1,40	1,32	1,25	1,18	1,12
		L/150	6,31	5,62	5,01	4,49	4,05	3,67	3,35	3,06	2,82	2,65	2,44	2,23	2,07	1,93	1,81	1,69	1,59	1,49	1,40	1,32	1,25	1,18	1,12
		L/300	6,31	5,62	5,01	4,49	4,05	3,67	3,35	3,06	2,82	2,65	2,44	2,23	2,07	1,84	1,62	1,43	1,27	1,14	1,02	0,92	0,83	0,75	0,68
		L/500	6,31	5,62	5,01	4,49	4,05	3,67	3,28	2,74	2,30	1,96	1,68	1,45	1,26	1,10	0,97	0,86	0,76	0,68	0,61	0,55	0,50	0,45	0,41
1,00	0,130	*	7,77	6,86	6,11	5,47	4,93	4,48	4,08	3,80	3,54	3,18	2,93	2,71	2,53	2,36	2,25	2,09	1,93	1,82	1,71	1,61	1,52	1,44	1,37
		L/150	7,77	6,86	6,11	5,47	4,93	4,48	4,08	3,80	3,54	3,18	2,93	2,71	2,53	2,36	2,25	2,09	1,93	1,82	1,71	1,61	1,52	1,44	1,37
		L/300	7,77	6,86	6,11	5,47	4,93	4,48	4,08	3,80	3,54	3,18	2,93	2,71	2,37	2,07	1,82	1,61	1,43	1,28	1,15	1,03	0,93	0,85	0,77
		L/500	7,77	6,86	6,11	5,47	4,93	4,48	3,69	3,08	2,59	2,20	1,89	1,63	1,42	1,24	1,09	0,97	0,86	0,77	0,69	0,62	0,56	0,51	0,46
1,25	0,162	*	12,44	10,91	9,94	9,18	8,15	7,11	6,31	5,75	5,27	4,93	4,63	4,24	3,86	3,53	3,29	3,07	2,87	2,69	2,52	2,37	2,24	2,11	1,99
		L/150	12,44	10,91	9,94	9,18	8,15	7,11	6,31	5,75	5,27	4,93	4,63	4,24	3,86	3,53	3,29	3,07	2,87	2,69	2,52	2,37	2,24	2,11	1,95
		L/300	12,44	10,91	9,94	9,18	8,15	7,11	6,31	5,75	5,27	4,66	4,00	3,45	3,00	2,63	2,31	2,05	1,82	1,62	1,46	1,31	1,18	1,07	0,98
		L/500	12,44	10,91	9,94	8,72	6,99	5,68	4,68	3,91	3,29	2,80	2,40	2,07	1,80	1,58	1,39	1,23	1,09	0,97	0,87	0,79	0,71	0,64	0,59
1,50	0,195	*	17,57	15,97	14,27	12,17	10,49	9,33	8,45	7,84	7,30	6,75	6,09	5,52	5,03	4,65	4,32	4,03	3,76	3,52	3,30	3,10	2,92	2,75	2,60
		L/150	17,57	15,97	14,27	12,17	10,49	9,33	8,45	7,84	7,30	6,75	6,09	5,52	5,03	4,65	4,32	4,03	3,76	3,52	3,30	3,10	2,86	2,60	2,36
		L/300	17,57	15,97	14,27	12,17	10,49	9,33	8,45	7,84	6,63	5,64	4,83	4,17	3,63	3,18	2,80	2,47	2,20	1,96	1,76	1,59	1,43	1,30	1,18
		L/500	17,57	15,97	13,42	10,55	8,46	6,88	5,66	4,72	3,98	3,38	2,90	2,51	2,18	1,91	1,68	1,48	1,32	1,18	1,06	0,95	0,86	0,78	0,71

Table 14 - Sizing Table for the T135.1 as a Three-Span System with a Lifting Load



Dreifeldträger, zulässige abhebende Flächenlast zul q [kN/m²]

Stützweite L[m]			2,50	2,75	3,00	3,25	3,50	3,75	4,00	4,25	4,50	4,75	5,00	5,25	5,50	5,75	6,00	6,25	6,50	6,75	7,00	7,25	7,50	7,75	8,00
t_N	g	max f																							
0,75	0,097	*	7,62	6,58	5,72	4,99	4,36	3,83	3,36	2,98	2,66	2,38	2,15	1,95	1,78	1,63	1,49	1,38	1,27	1,18	1,10	1,02	0,96	0,90	0,84
		L/150	7,62	6,58	5,72	4,99	4,36	3,83	3,36	2,98	2,66	2,38	2,15	1,95	1,78	1,63	1,49	1,38	1,27	1,18	1,10	1,02	0,96	0,90	0,84
		L/300	7,62	6,58	5,72	4,99	4,36	3,83	3,36	2,98	2,66	2,38	2,15	1,95	1,78	1,59	1,40	1,24	1,10	0,98	0,88	0,79	0,72	0,65	0,59
		L/500	7,62	6,58	5,72	4,99	4,23	3,44	2,83	2,36	1,99	1,69	1,45	1,25	1,09	0,95	0,84	0,74	0,66	0,59	0,53	0,48	0,43	0,39	0,35
0,88	0,114	*	10,67	9,00	7,61	6,48	5,59	4,87	4,28	3,79	3,38	3,03	2,74	2,48	2,27	2,07	1,90	1,75	1,62	1,50	1,40	1,30	1,22	1,14	1,07
		L/150	10,67	9,00	7,61	6,48	5,59	4,87	4,28	3,79	3,38	3,03	2,74	2,48	2,27	2,07	1,90	1,75	1,62	1,50	1,40	1,30	1,22	1,14	1,07
		L/300	10,67	9,00	7,61	6,48	5,59	4,87	4,28	3,79	3,38	3,03	2,74	2,42	2,10	1,84	1,62	1,43	1,27	1,14	1,02	0,92	0,83	0,75	0,68
		L/500	10,67	9,00	7,61	6,12	4,90	3,98	3,28	2,74	2,30	1,96	1,68	1,45	1,26	1,10	0,97	0,86	0,76	0,68	0,61	0,55	0,50	0,45	0,41
1,00	0,130	*	13,27	10,97	9,23	7,86	6,78	5,91	5,19	4,60	4,10	3,68	3,32	3,01	2,75	2,51	2,31	2,13	1,97	1,82	1,70	1,58	1,48	1,38	1,30
		L/150	13,27	10,97	9,23	7,86	6,78	5,91	5,19	4,60	4,10	3,68	3,32	3,01	2,75	2,51	2,31	2,13	1,97	1,82	1,70	1,58	1,48	1,38	1,30
		L/300	13,27	10,97	9,23	7,86	6,78	5,91	5,19	4,60	4,10	3,67	3,15	2,72	2,36	2,07	1,82	1,61	1,43	1,28	1,15	1,03	0,93	0,85	0,77
		L/500	13,27	10,97	8,75	6,88	5,51	4,48	3,69	3,08	2,59	2,20	1,89	1,63	1,42	1,24	1,09	0,97	0,86	0,77	0,69	0,62	0,56	0,51	0,46
1,25	0,162	*	17,92	14,83	12,46	10,62	9,16	7,97	7,00	6,21	5,54	4,97	4,49	4,07	3,71	3,39	3,11	2,87	2,66	2,46	2,29	2,13	2,00	1,87	1,75
		L/150	17,92	14,83	12,46	10,62	9,16	7,97	7,00	6,21	5,54	4,97	4,49	4,07	3,71	3,39	3,11	2,87	2,66	2,46	2,29	2,13	2,00	1,87	1,75
		L/300	17,92	14,83	12,46	10,62	9,16	7,97	7,00	6,21	5,48	4,66	4,00	3,45	3,00	2,63	2,31	2,04	1,82	1,62	1,46	1,31	1,18	1,07	0,98
		L/500	17,92	14,41	11,09	8,72	6,99	5,68	4,68	3,90	3,29	2,79	2,40	2,07	1,80	1,58	1,39	1,23	1,09	0,97	0,87	0,79	0,71	0,64	0,59
1,50	0,195	*	22,21	18,35	15,43	13,14	11,33	9,87	8,67	7,68	6,85	6,15	5,55	5,03	4,59	4,20	3,85	3,55	3,28	3,05	2,83	2,64	2,47	2,31	2,17
		L/150	22,21	18,35	15,43	13,14	11,33	9,87	8,67	7,68	6,85	6,15	5,55	5,03	4,59	4,20	3,85	3,55	3,28	3,05	2,83	2,64	2,47	2,31	2,17
		L/300	22,21	18,35	15,43	13,14	11,33	9,87	8,67	7,68	6,63	5,64	4,83	4,18	3,63	3,18	2,80	2,47	2,20	1,96	1,76	1,59	1,43	1,30	1,18
		L/500	22,21	17,44	13,42	10,55	8,46	6,87	5,66	4,72	3,98	3,38	2,90	2,51	2,18	1,91	1,68	1,48	1,32	1,18	1,06	0,95	0,86	0,78	0,71

5.3 INTERNAL FORCES

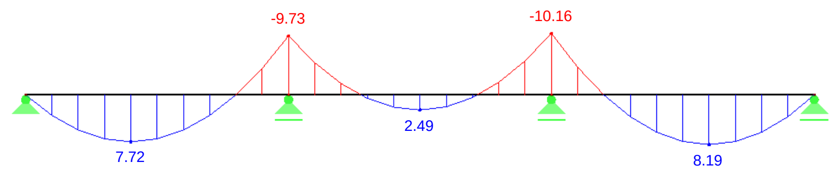
Internal forces

	max $M_{yd,F}$ [kNm/m]	max $M_{yd,S}$ [kNm/m]	max V_{zd} [kN/m]	max $R_{zd,A}$ [kN/m]	max $R_{zd,B}$ [kN/m]
LC1	8.19	- 10.16	12.96	8.73	23.68
LC2	- 1.91	1.84	2.74	- 1.96	- 4.12

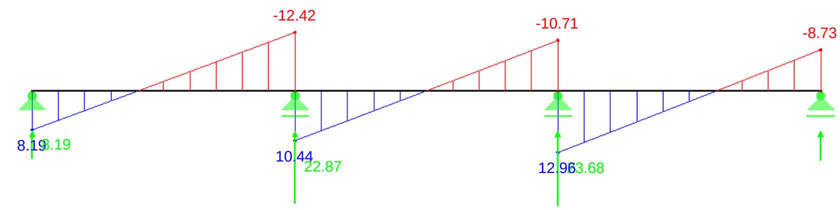
Internal Force Diagrams

LC1

$M_{y,d1}$

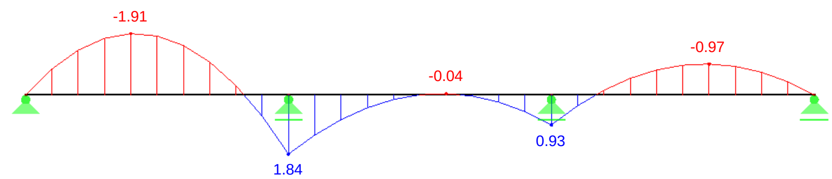


$V_{z,d1}$

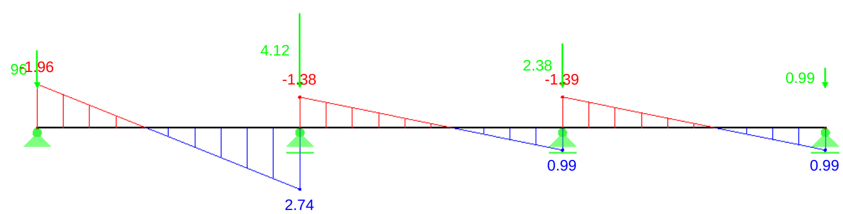


LC2

$M_{y,d2}$



$V_{z,d2}$



5.4 ULTIMATE LIMIT STATE DESIGN

The cross-section values are taken from Table 15 and the static proof is done according to DIN EN 1993- 3, DIN 18807 and DIN 18800.

End support	$R_{A,Ed} \leq R_{A,Rk} / \gamma_M \rightarrow 8.73 \leq 13.30 / 1.25 = 10.64$ ✓
Intermediate support	$R_{B,Ed} \leq R^{w}_{B,Rk} / \gamma_M \rightarrow 23.68 \leq 37.00 / 1.25 = 29.60$ ✓
Field moment	$M_{F,Ed} \leq M^{c}_{F,Rk} / \gamma_M \rightarrow 8.19 \leq 14.80 / 1.25 = 11.84$ ✓
Support moment	$M_{B,Ed} \leq M^{c}_{B,Rk} / \gamma_M \rightarrow 10.16 \leq 15.60 / 1.25 = 12.48$ ✓

Interaction

$$\frac{M_{B,Ed}}{M^0_{B,Rk} / \gamma_M} + \left(\frac{R_{B,Ed}}{R^0_{B,Rk} / \gamma_M}\right)^2 \leq 1.00$$

$$\frac{10.16}{17.40 / 1.25} + \left(\frac{23.68}{47.97 / 1.25}\right)^2 = 1.11 > 1.00$$ ↯ *The design is not fulfilled.*

The sheet thickness must be adjusted to t_N = 1.25 mm!

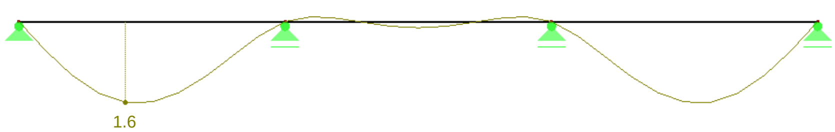
Interaction (t_N=1.25)

$$\frac{10.16}{24.0 / 1.25} + \left(\frac{23.68}{82.79 / 1.25}\right)^2 = 0.66 < 1.00$$ ✓

Span limit *L_{max} = 18.00 m > L = 4.50 m* ✓

5.5 SERVICABILITY LIMIT STATE STATE DESIGN

Deflection of the trapezoidal sheet:



$$\max f = 1.60mm$$

It must b $\max f \leq L / 300$

Here $L / 300 = 450 / 300 = 1.50cm = 15mm > \max f = 1.60mm$ ✓

Table 15 - Cross Section Values for Hoesch T 135.1 Positive Position

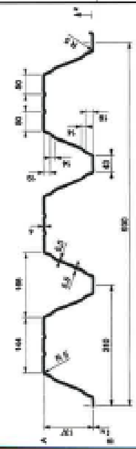
Stahltrapezprofil Typ **Hoesch T 135.1**
Querschnitts- und Bemessungswerte
EN 1993-1-3
Profiltafel in **Positivlage**

Anlage 7.1

Als Typenentwurf
in bautechnischer Hinsicht geprüft
Prüfbescheid-Nr. **T 17-090**
Landesdirektion Sachsen
- Landesstelle für Bautechnik -

Leipzig, den 04.10.2017

Leiter Bearbeiter



Nennstreckgrenze des Stahlkerns $f_{y,k}$ = 320 N/mm²

Charakteristische Tragfähigkeitswerte für andrückende Flächenbelastung 3)

Nennblechdicke 12)	Feldmoment	Endauflagerkraft 6) $l_{a,A1}$ = -	Endauflagerkraft 6) $l_{a,A2}$ = 40 mm	Querkraft	Stützmomente 11) $l_{a,B}$ = 60 mm		Stützmomente 11) $l_{a,B}$ = 160 mm		Stützmomente 11) $l_{a,B}$ = -		Zwischenauflagerkräfte 11) $l_{a,B}$ = 60 mm		Zwischenauflagerkräfte 11) $l_{a,B}$ = 160 mm		Zwischenauflagerkräfte 11) $l_{a,B}$ = -	
t_N	$M_{c,Rk,F}$	$R_{w,Rk,A}$		$V_{w,Rk}$	$M_{0,Rk,B}$	$M_{c,Rk,B}$	$M_{0,Rk,B}$	$M_{c,Rk,B}$	$M_{0,Rk,B}$	$M_{c,Rk,B}$	$R_{0,Rk,B}$	$R_{w,Rk,B}$	$R_{0,Rk,B}$	$R_{w,Rk,B}$	$R_{0,Rk,B}$	$R_{w,Rk,B}$
mm	kNm/m	kN/m			kNm/m						kN/m					
0,75	10,20	-	7,16	26,17	9,44	7,86	10,80	9,56	-	-	20,40	16,60	29,16	22,80	-	-
0,88	12,60	-	10,40	45,40	12,30	10,50	14,30	12,70	-	-	29,07	24,30	39,33	30,20	-	-
1,00	14,80	-	13,30	66,39	14,90	12,90	17,40	15,60	-	-	36,96	31,20	47,97	37,00	-	-
1,25	20,60	-	24,60	123,00	21,30	19,20	24,00	21,90	-	-	66,00	50,70	82,79	60,20	-	-
1,50	26,50	-	36,20	176,82	27,80	25,40	30,60	28,40	-	-	94,91	70,10	117,30	83,10	-	-

Elastisch aufnehmbare Schnittgrößen an Zwischenauflagern 1) 2) 4) 5) – Quadratische Interaktion

Reststützmomente 7)

t_N	$l_{a,B}$ = 60 mm min L	max L	max $M_{R,Rk}$	$l_{a,B}$ = 160 mm min L	max L	max $M_{R,Rk}$	$l_{a,B}$ = - min L	max L	max $M_{R,Rk}$
mm	m	m	kNm/m	m	m	kNm/m	m	m	kNm/m
0,75	6,62	7,46	2,24	8,02	8,83	1,84	-	-	-
0,88	5,08	5,96	3,59	6,00	6,85	3,04	-	-	-
1,00	4,43	5,33	4,82	5,18	6,05	4,14	-	-	-
1,25	4,00	4,92	7,42	4,14	5,05	7,16	-	-	-
1,50	3,82	4,75	10,00	3,75	4,88	10,20	-	-	-

Reststützmomente $M_{R,Rk}$

$M_{R,Rk}$ = 0 für L ≤ min L

$M_{R,Rk}$ = max $M_{R,Rk}$ für L ≥ max L

$$M_{R,Rk} = \frac{L - \min L}{\max L - \min L} \cdot \max M_{R,Rk}$$

Charakteristische Tragfähigkeitswerte für abhebende Flächenbelastung 1) 2)

Nennblechdicke	Feldmoment	Befestigung in jedem anliegenden Gurt: Endauflagerkraft	Zwischenauflager					Befestigung in jedem 2. anliegenden Gurt: Endauflagerkraft	Zwischenauflager				
t_N	$M_{c,Rk,F}$	$R_{w,Rk,A}$	$M_{0,Rk,B}$	$M_{c,Rk,B}$	$R_{0,Rk,B}$	$R_{w,Rk,B}$	$V_{w,Rk}$	$R_{w,Rk,A}$	$M_{0,Rk,B}$	$M_{c,Rk,B}$	$R_{0,Rk,B}$	$R_{w,Rk,B}$	$V_{w,Rk}$
mm	kNm/m	kN/m	kNm/m		kN/m			kN/m	kNm/m		kN/m		
0,75	9,83	26,17	-	6,67	-	-	26,17	14,09	-	4,43	-	-	14,09
0,88	12,80	45,40	-	11,33	-	-	45,40	22,70	-	5,67	-	-	22,70
1,00	15,50	66,39	-	13,66	-	-	66,39	33,20	-	6,83	-	-	33,20
1,25	21,30	123,00	-	18,54	-	-	123,00	61,50	-	9,27	-	-	61,50
1,50	27,20	176,82	-	22,89	-	-	176,82	88,41	-	11,45	-	-	88,41

Fußnoten s. Beiblatt 1/2 bzw. 2/2

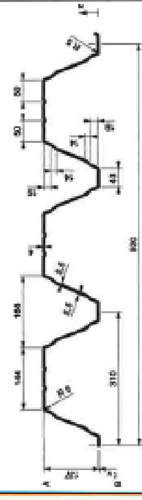
Stahltrapezprofil Typ **Hoesch T 135.1**
Querschnitts- und Bemessungswerte
EN 1993-1-3
Profiltafel in **Positivlage**

Anlage 7.2

Als Typenentwurf
in bautechnischer Hinsicht geprüft
Prüfbescheid-Nr. **T 17-090**
Landesdirektion Sachsen
- Landesstelle für Bautechnik -

Leipzig, den 04.10.2017

Leiter Bearbeiter



Nennstreckgrenze des Stahlkerns $f_{y,k}$ = 320 N/mm²

Maßgebende Querschnittswerte

Nennblechdicke	Eigenlast	Biegung 8)		Normalkraftbeanspruchung: nicht reduzierter Querschnitt			wirksamer Querschnitt 9)			Grenzstützweiten 10) L_{gr} in m: Einfeldträger	Mehrfeldträger
t_N	g	I^+_{ef}	I^-_{ef}	A_g	i_g	z_g	A_{eff}	i_{eff}	z_{eff}		
mm	kN/m²	cm⁴/m		cm²/m	cm		cm²/m	cm			
0,75	0,097	297,00	297,00	11,21	5,03	8,28	3,80	6,01	7,86	5,18	6,48
0,88	0,114	344,00	344,00	13,26	5,03	8,28	4,95	5,98	7,87	10,00	12,50
1,00	0,129	387,00	387,00	15,15	5,03	8,28	6,10	5,95	7,88	11,40	14,30
1,25	0,162	491,00	491,00	19,17	5,03	8,28	8,76	5,89	7,92	14,40	18,00
1,50	0,194	594,00	594,00	23,13	5,03	8,28	11,40	5,78	7,93	17,40	21,70

Schubfeldwerte

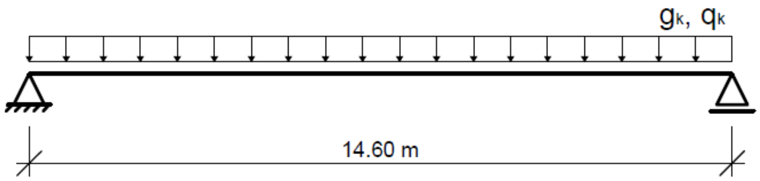
Nennblechdicke		Grenzzustand der Gebrauchstauglichkeit 16)	$T_{3,Rk} = G_s / 750$ [kN/m] 15)	$G_s = 10^4 / (K_1 + K_2 / L_S)$		Grenzzustand der Tragfähigkeit 17)		$F_{t,Rk}$ in kN 18) Einleitungslänge a	
t_N	min L_S 13)	$T_{2,Rk}$	L_G 14)	K_1	K_2	$T_{1,Rk}$	K_3	≥ 130 mm	≥ 280 mm
mm	m	kN/m	m	m/kN	m²/kN	kN/m	-		
	Normalausführung: Verbindung in jedem Untergurt								
0,75	4,98	1,68	6,53	0,274	54,836	2,54	0,884	13,5	18,0
0,88	4,58	2,55	5,54	0,232	36,017	3,27	0,884	16,0	21,3
1,00	4,29	3,56	4,86	0,203	25,795	3,99	0,884	18,3	24,3
1,25	3,82	6,35	3,87	0,161	14,463	5,65	0,884	23,1	30,7
1,50	3,48	10,16	3,48	0,133	9,043	7,49	0,884	27,8	37,0
	Sonderausführung: Verbindung mit 2 Schrauben oder verstärkter Unterlegscheibe in jedem Untergurt 19)								
0,75	5,18	1,60	11,05	0,274	39,674	5,69	0,795	13,5	18,0
0,88	4,76	2,44	9,41	0,232	26,059	7,33	0,795	16,0	21,3
1,00	4,45	3,41	8,26	0,203	18,663	8,95	0,795	18,3	24,3
1,25	3,97	6,08	6,64	0,161	10,464	12,67	0,795	23,1	30,7
1,50	3,61	9,73	5,56	0,133	6,543	16,79	0,795	27,8	37,0

Fußnoten s. Beiblatt 1/2 bzw. 2/2

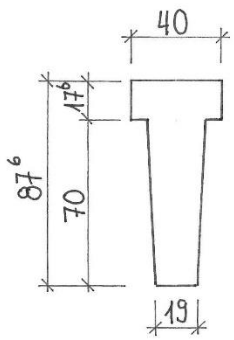
6 POS. P.02 – PRESTRESSED CONCRETE PURLINS

6.1 PRELIMINARY DESIGN

System



Cross-section



Building materials **C 30/37 | St 1570/1770 | B 500A**

Characteristic loads

Uniformly Distributed Loads		**g_k [kN/m]**	**q_k [kN/m]**
Self-weight	g_{k1} = 0.23 cm² x 25 kN/m³ =	5.74	-
Roof Structure – Reaction Force from *TR.01*	g_{k2} =	2.59	-
Snow – Reaction Force from *TR.01*	q_{k1} =	-	13.12
Wind – Reaction Force form *TR.01*	q_{k2} =	-	0.57
		8.33	13.12 0.57

Characteristic bending moment

M_{Gk1} [kNm]	M_{Gk2} [kNm]	M_{Qk1} [kNm]	M_{Qk2} [kNm]
153.0	69.0	349.6	15.2

Assumptions

Prestressing $\sigma_{m,t=0} \approx 1000 N / mm^2$

Prestressing loss $\approx 15\%$

Prestressing with immediate bond

Cross-section Properties

$z_{cu} = 49.76cm$ (*Position of the center of gravity*)

$A_c = 2296.5cm^2$

Moment of inertia $I_{yc} = 1{,}518{,}675cm^4$

Section modulus $W_{cu} = \frac{I_{yc}}{z_{cu}} = \frac{1{,}518{,}675cm^4}{49.76cm} = 30{,}520cm^3$

Bending Compression Zone

Effective depth $d = 87.6cm - 10cm = 77.6cm$

$\delta = h_f / d = 17.6 / 77.6 = 0.227$

$\beta = b_w / b_{eff} = 19cm / 40cm = 0.475$

$$\rightarrow \eta = \frac{1}{(1-\beta)\cdot(1-0.5\delta)\cdot\delta + 0.32\beta} = \underline{\underline{3.88}}$$

ULS $M_{Ed}^{G+Q} = 1.35\cdot(153.0+69.0) + 1.50\cdot 349.6 + 1.5\cdot 0.6\cdot 15.2 = 837.7kNm$

$$b_{eff,req} \geq \frac{M_{Ed}}{f_{cd}}\cdot\eta = \frac{0.8377MNm}{17MN/m^2}\cdot 3.88 / d^2 = \underline{\underline{0.115}}m < b_{eff,prov} = 0.40m \checkmark$$

→ *The provided bending compression zone is sufficient.*

Required Prestressing Steel

SLS $M_{Ed,frequ} = (153.0+69.0) + 0.2\cdot 349.6 + 0\cdot 15.2 = 291.9kNm$

$r_{\inf} = 0.95$, $\alpha_{loss} = 0.15$, $\sigma_{pm,t=0} = 1000N/mm^2$

$z_{cp} = 49.76cm - 10cm = 39.76cm$

Required prestressing steel

$$A_{P,req} \geq \frac{M_{Ed}}{r_{\inf}\cdot(1-\alpha_{loss})\cdot\sigma_{pm,t=0}\cdot\left(\frac{W_{cu}}{A_c} + z_{cp}\right)}$$

$$= \frac{29190kNcm}{0.95\cdot(1-0.15)\cdot 100kN/cm^2\cdot\left(\frac{30520}{2296.5} + 39.76\right)} = \underline{\underline{6.81cm^2}}$$

→ *The required cross-section of the pre-stressing steel is 6.81cm².*

Tension Zone

SLS $\min M_{Gk} = 153.0kNm$ $M_{Ed,frequ} = 291.9kNm$

$$W_{cu,req} \geq \frac{\frac{M_{Ed}}{r_{\inf}\cdot(1-\alpha_{loss})} - \min M_{Gk}}{0.6\cdot f_{ck}} = \frac{\frac{29190}{0.95\cdot(1-0.15)} - 15300}{0.6\cdot 3.0} = \underline{\underline{11{,}584cm^3}}$$

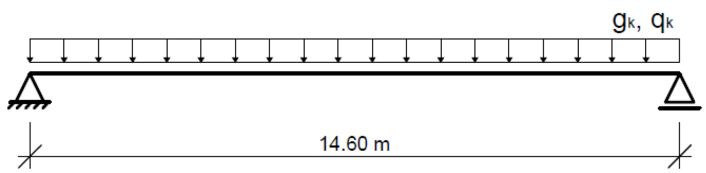
$$W_{cu,prov} = 30{,}520 cm^3 > W_{cu,req} = 11{,}427 cm^3 \checkmark$$

→ *The tension zone of the purlin is sufficient.*

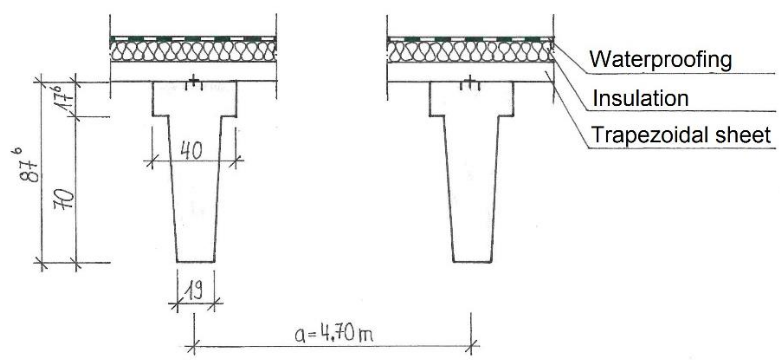
Chosen	**C 30/37 | "T-Profile": h = 876 mm, b_o = 400 mm, b_u = 190 mm** **St 1570/1770 | 8 $Ø_P$ = 12.5mm = 8 x 0.93 cm² = 7.44 cm²**

6.2 SYSTEM

System



Cross-section



A_c [cm²]	U [cm]	$x_{s,u}$ [cm]	$x_{s,o}$ [cm]	I_y [cm⁴]	W_u [cm³]
2297	248	49.8	37.8	1,518,675	30,520

Building materials **C 30/37 | St 1570/1770 | B 500A**

Exposure class XC1, WO

Reinforcing steel $c_{nom,w} = c_{min} + \Delta c_{dev} = 10mm + 10mm = 20mm$

A_s = 2 $Ø_s$ 16 = 4.02 cm² a_u = 4.0 cm

Prestressing steel c_{nom} = 2.5 x 12.5 mm + 10 mm = 42 mm

1. Layer : A_p = 3 $Ø_p$ 12.5 = 3 x 0.93cm² a_p = 7.5 cm

2. Layer : A_p = 3 $Ø_p$ 12.5 = 3 x 0.93cm² a_p = 11.7 cm

3. Layer : A_p = 2 $Ø_p$ 12.5 = 2 x 0.93cm² a_p = 15.9 cm

Characteristic loads

Uniformly Distributed Loads		g_k [kN/m]	q_k [kN/m]
Self-weight	g_{k1} = 0.23 cm² x 25 kN/m³ =	5.74	-
Roof Structure – Reaction Force from *TR.01*	g_{k2} =	2.59	-
Snow – Reaction Force from *TR.01*	q_{k1} =	-	13.12
Wind – Reaction Force form *TR.01*	q_{k2} =	-	0.57
		8.33	13.12 0.57

6.3 INTERNAL FORCES

Characteristic (at x = L/2 = 7.35 m)

M_{gk1} [kNm]	M_{gk2} [kNm]	M_{qk1} [kNm]	M_{qk2} [kNm]
153.0	69.0	349.5	15.2

V_{gk1} [kN]	V_{gk2} [kN]	V_{qk1} [kN]	V_{qk2} [kN]
41.9	18.9	95.8	4.2

Design

ULS	SLS		
M_{Ed} [kNm]	M_{rare} [kNm]	M_{frequ} [kNm]	M_{perm} [kNm]
837.7	580.7	291.9	222.0

ULS	SLS		
V_{Ed} [kN]	V_{rare} [kN]	V_{frequ} [kN]	V_{perm} [kN]
229.5	159.1	80.0	60.8

6.4 PRESTRESSING LOSS

Maximum prestressing force

(1) $P_{m0,\max} = 0.75 \cdot A_p \cdot f_{pk} = 0.75 \cdot 8 \cdot 0.934 \cdot 177 kN/cm^2 = 991.9 kN$

(2) $P_{m0,\max} = 0.85 \cdot A_p \cdot f_{p0,1k} = 0.85 \cdot 8 \cdot 0.934 \cdot 150 kN/cm^2 = \underline{\underline{952.7 kN}}$

→ $P_{m,t=0} = 747.2 kN < P_{m0,\max} = 952.7 kN$ ✓

Effective thickness $h_0 = \frac{2 \cdot A_c}{U} = \frac{2 \cdot 2296.5}{247.9} = 18.53 cm$

Drying shrinkage $\varepsilon_{cd}(\infty) = k_h \cdot \varepsilon_{cd,0} = 0.872 \cdot 0.67‰ = 0.584‰$

Autogenous shrinkage: $\varepsilon_{ca}(\infty) = 2.5 \cdot (f_{ck} - 10) \cdot 10^{-6} = 2.5 \cdot (30 - 10) \cdot 10^{-6} = 0.050‰$

Total shrinkage $\varepsilon_{cs}(\infty) = \varepsilon_{cd}(\infty) + \varepsilon_{ca}(\infty) = 0.585‰ + 0.050‰ = 0.634‰$

Final creep ratio *for t_0 = 5 day:* $\varphi(\infty,t_0) = 2.899$

Prestressing loss due to relaxation

$$\sigma_{p,perm} = \frac{E_p}{E_{cm}} \cdot \left[\frac{M_{Ed,perm}}{I_c} \cdot z_p \right] = \frac{196}{33} \cdot \left[\frac{0.222}{0.01518675} \cdot 0.386 \right] = 33.5 MN/m^2$$

$$N_{p,t=0} = -P_{0,t=0} = -747kN \qquad M_{p,t=0} = N_{p,t=0} \cdot z_p = -747 \cdot 0.386 = -288kNm$$

$$\sigma_{p,t=0} = \frac{P_{0,t=0}}{A_p} + \frac{E_p}{E_{cm}} \cdot \left[\frac{N_{p,t=0}}{A_c} + \frac{M_{p,t=0}}{I_c} \cdot z_p \right]$$

$$= \frac{747}{8 \cdot 0.934} + \frac{196}{33} \cdot \left[\frac{-747}{2297} + \frac{-28800}{1{,}518{,}675} \cdot 38.6 \right] = 937.2 MN/m^2$$

$$R_i / R_m = \frac{\sigma_{p,perm} + \sigma_{p,t=0}}{f_{pk}} = \frac{33.5 + 937.2}{1770} = 0.5484 \qquad \rightarrow \kappa_p = 1.194\%$$

$$\Delta\sigma_{pr} = \kappa \cdot \left(\sigma_{p,perm} + \sigma_{p,t=0} \right) = 0.01194 \cdot (33.5 + 937.2) = 11.6 MN/m^2$$

Total Prestressing Loss

$$\sigma_{cp,perm} = \frac{M_{Ed,perm}}{I_c} \cdot z_p = \frac{22200}{1{,}518{,}675} \cdot 38.6 = 5.64 MN/m^2$$

$$\sigma_{cp,t=0} = \frac{N_{p,t=0}}{A_c} + \frac{M_{p,t=0}}{I_c} \cdot z_p = \frac{-747}{2297} + \frac{-28800}{1{,}518{,}675} \cdot 38.6 = -10.6 MN/m^2$$

$$\alpha_p = \frac{E_p}{E_{cm}} = \frac{196{,}000}{33{,}000} = 5.94$$

$$\Delta\sigma_{p,c+s+r} = \frac{\varepsilon_{cs}(t) \cdot E_p + 0.8 \cdot \Delta\sigma_{pr} + \alpha_p \cdot \varphi(\infty,t_0) \cdot (\sigma_{cp,perm} + \sigma_{cp,t=0})}{1 + \alpha_p \cdot \frac{A_p}{A_c} \cdot \left(1 + \frac{A_c}{I_c} \cdot z_p^2\right) \cdot [1 + 0.8 \cdot \varphi(\infty,t_0)]}$$

$$= -102.9 - 7.7 - 70.4 = -180.9 MN/m^2$$

Shrinkage *- 102.9 MN/m² → 56.9%*

Relaxation *- 7.7 MN/m² → 4.2%*

Creep *- 70.4 MN/m² → 38.9%*

$$\Delta P_{c+s+r} = \Delta\sigma_{p,c+s+r} \cdot A_p = -180.9 \cdot 8 \cdot 0.934 \cdot 10^{-4} = -135.2KN$$

$$\frac{\Delta P_{c+s+r}}{P_{m,t=0}} = \frac{135.2}{747} = 0.142 \qquad \rightarrow \alpha_{loss} = 14.2\% < 15\% \ \checkmark$$

$$P_{m,t=\infty} = 747 - 135.2 = \underline{\underline{612kN}} \qquad \rightarrow \underline{\underline{77kN/strand}}$$

6.5 ULTIMATE LIMIT STATE DESIGN

6.5.1 FLEXURE DESIGN

Compression Zone

The neutral axis lies below the compression flange; therefore the effective width of the flange must be adjusted according to [2].

$$b_i = \lambda_b \cdot b_{eff} = 0.864 \cdot 40 = 34.6cm$$

$$d = 78.9cm \qquad z_r = d - x_{so} = 41.1cm \qquad z_p = x_{su} - a_p = 38.6cm$$

$$M_{Edr} = M_{Ed} + P_{m,t=\infty} \cdot (z_r - z_p) = 837.27 + 612 \cdot (0.411 - 0.386) = 853kNm$$

$$\mu_{Edr} = \frac{M_{Edr}}{b_{eff} \cdot d^2 \cdot f_{cd}} = \frac{85300kNcm}{34.5 \cdot 78.9^2 \cdot 1.7kN/cm^2} = 0.233 < \mu_{Ed,\max} = 0.40 \checkmark$$

→ *The compression zone is sufficient.*

Tension Zone

Using the design table from [3] following values for the height of the compression zone x and the lever arm of the internal forces z can be determined:

Lever arm z $\zeta = 0.859$ → $z = \zeta \cdot d = 0.859 \cdot 78.9 = 67.8cm$

Compression zone $\xi = 0.339$ → $x = \xi \cdot d = 0.339 \cdot 78.6 = 26.8cm$

Rebar strain $\varepsilon_s = \Delta\varepsilon_p = 6.81‰ > 2.17‰$ → $\sigma_{sd} = f_{yd} = 435MN/m^2$

Prestressing steel strain $\varepsilon_p^{(0)} = \frac{P_{m,t=0}}{A_p \cdot E_p} = \frac{612kN}{8 \cdot 0.934cm^2 \cdot 19{,}600kN/cm^2} = 4.18‰$

Total strain $\varepsilon_p = \varepsilon_p^{(0)} + \Delta\varepsilon_p = 4.18‰ + 6.81‰ = 10.99‰$

$$f_{pd} = \frac{f_{p0,1,k}}{\gamma_p} = \frac{1500N/mm^2}{1.15} = 1304N/mm^2$$

Permitted strain $\varepsilon_{p0,1d} = \frac{f_{pd}}{E_p} = \frac{1304N/mm^2}{196{,}000N/mm^2} = 6.7‰$

$$\varepsilon_{p0,1d} = 6.7‰ < \varepsilon_p = 10.99‰ \quad \rightarrow \quad \sigma_{pd} = f_{pd} = 1304N/mm^2$$

Required reinforcement $A_{s1,req} = \left(\frac{M_{Edr}}{z} - A_p \cdot \sigma_{pd}\right) \cdot \frac{1}{\sigma_{sd}} = \left(\frac{85300}{67.8} - 7.47 \cdot 130.4\right) \cdot \frac{1}{43.5} = 6.52cm^2$

Chosen **3 Øs 20 = 9.42 cm²** **lower reinforcement**

The required reinforcement is higher than the previously estimated reinforcement of A_s = 2 $Ø_s$ 16 = 4.02 cm². A new reinforcement of A_s = 3 $Ø_s$ 20 = 9.42 cm² is chosen. Which means the static height as well as the height of the compression zone change:

$$d = 87.6cm - 7.17cm = 80.4cm$$

$$\zeta = 0.864 \quad \rightarrow z = \zeta \cdot d = 0.864 \cdot 80.4 = 69.5m$$

$$\xi = 0.327 \quad \rightarrow x = \xi \cdot d = 0.327 \cdot 70.4 = 26.3cm$$

Precompressed Tension Zone

$$N_{p,t=0} = -747kN \qquad M_{p,t=0} = -288kNm$$

Design normal force $N_{Ed} = \gamma_p \cdot N_{p,t=0} = 1.0 \cdot (-747kN) = -747kN$

Effective depth $d \approx 87.6 - 4 = 83.6cm \quad \rightarrow z_{s2} = d - x_{su} = 83.6 - 49.8 = 33.8cm$

$$M_{Ed} = \gamma_G \cdot \min M_G + \gamma_P \cdot M_{P,t=0} = 1.0 \cdot 153.0 + 1.0 \cdot (-288) = -135.3kNm$$

Design bending moment $M_{Ed,s2} = M_{Ed} - N_{Ed} \cdot z_{s2} = -135.3 - 0.338 \cdot (-747) = 388kNm$

$$\mu_{Eds2} = \frac{M_{Eds2}}{b_{eff} \cdot d^2 \cdot f_{cd}} + \frac{N_{Ed}}{f_{yd}} = 0.095 \quad \rightarrow \omega_2 = 0.099$$

Required reinforcement $A_{s2,req} = \omega_2 \cdot \frac{b_{eff} \cdot d}{f_{yd} / f_{cd}} + \frac{N_{Ed}}{f_{yd}} = 0.099 \cdot \frac{34.6 \cdot 83.6}{43.5 / 1.7} + \frac{-747}{43.5} = -6.00 < 0$

No reinforcement in the compression zone is required.

6.5.2 SHEAR DESIGN

Design location *$x_1 = a_i + d = 0.2 + 0.804 = 1.00$ m (direct support)*

$x_2 = a_i = 0.2$ m

Shear force at x *$V_{Ed,1} = 198.1$ kN*

$V_{Ed,2} = 223.2$ kN

Shear Force Resistance

$$a_r = 7.17cm \quad \rightarrow d = 87.6 - 7.17 = 80.4cm$$

$$k = 1 + \sqrt{\frac{20}{d}} = 1 + \sqrt{\frac{20}{80.4}} = 1.50 < 2.00$$

Longitudinal reinforcement $A_{sl} = A_s + A_p = 9.42 + 7.47 = 16.89cm^2$

Percentage of reinforcement $\rho_1 = \frac{A_{sl}}{b_w \cdot d} = \frac{16.89}{19 \cdot 80.4} = 0.0111 \leq 0.02$

Compressive stress $\sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{0.612}{0.2297} = 2.67MN/m^2 < 0.2 \cdot f_{cd} = 3.40MN/m^2$

$$V_{Rd,c} = \left[0.10 \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp}\right] \cdot b_w \cdot d$$

$$V_{Rd,c} = \left[0.10 \cdot 1.5 \cdot (100 \cdot 0.0111 \cdot 3{,}0)^{1/3} + 0.12 \cdot 2.67\right] \cdot 0.19 \cdot 0.804$$

$$= 122.5kN$$

$$d = 80.4cm \geq 80cm \rightarrow \nu_{min} = 0.025 \cdot 1.5^{3/2} \cdot (30)^{1/2} = 0.251MN/m^2$$

$$V_{Rd,c,\min} = \left[v_{\min} + 0.12 \cdot \sigma_{cp}\right] \cdot b_w \cdot d = \left[0.251 + 0.12 \cdot 2.67\right] \cdot 0.19 \cdot 0.804$$

$$= 87.3kN$$

Shear force resistance $V_{Rd,c} = \underline{\underline{122.5kN}} > V_{Rd,c,\min} = 87.3kN$

$V_{Ed} = 198.1kN > V_{Rd,c} = 122.5kN$ → *Shear force reinforcement is needed!*

Inclination of Concrete Compression Struts

Friction force

$$V_{Rd,cc} = 0.24 \cdot \left(f_{ck}\right)^{1/3} \cdot \left(1 - 1.2 \cdot \frac{\sigma_{cp}}{f_{cd}}\right) \cdot b_w \cdot z$$

$$V_{Rd,cc} = 0.24 \cdot (3.0)^{1/3} \cdot \left(1 - 1.2 \cdot \frac{0.267}{1.7}\right) \cdot 19 \cdot 69.5 = 79.9kN$$

Inclination

$$1.0 \le \cot\Theta = \frac{1.2 + 1.4 \cdot \sigma_{cp} / f_{cd}}{1 - V_{Rd,cc} / V_{Ed}} = \frac{1.2 + 1.4 \cdot 0.267 / 1.7}{1 - 79.9 / 122.5} = 2.38 \le 3.0$$

Required Shear Force Reinforcement

Vertical stirrups $\alpha = 90°$

Required reinforcement

$$a_{sw,req} = \frac{V_{Ed}}{f_{yd} \cdot z \cdot \left(\cot\theta + \cot\alpha\right) \cdot \sin\alpha}$$

$$= \frac{198.1kN}{43.5 \cdot 69.5 \cdot \left(2.38 + 0\right) \cdot 1} = 2.75cm^2/m$$

Chosen **Ø 8 / 25 = 4.02 cm²/m** **stirrups**

Resistance of the Compression Struts

$$V_{Rd,\max} = 0.75 \cdot b_w \cdot z \cdot f_{cd} \cdot \frac{\cot\theta + \cot\alpha}{1 + \cot^2\theta} = 0.75 \cdot 19 \cdot 69.5 \cdot 1.7 \cdot \frac{2.38}{1 + 2.38^2}$$

$$= 601.1kN$$

$V_{Ed,2} = 223kN < V_{Rd,\max} = 601kN$ → *The compression strut is ok.*

Minimum Shear Force Reinforcement

$\min a_{sw} = \rho \cdot b_w \cdot \sin\alpha$ and $\rho = 0.16 \cdot f_{ctm} / f_{yk} = 0.16 \cdot 2.9 / 500 = 9.3‰$

$$\min a_{sw} = 0.00093 \cdot 0.19 \cdot \sin 90° = 1.77 \cdot 10^{-4} m^2/m = \underline{\underline{1.77cm^2/m}}$$

6.5.3 TILTING

According to DIN EN 1992-1-1, 5.9 a simplified proof of the safety against tilting can be conducted with following formula:

$$b_{req} \geq \sqrt[4]{\left(\frac{l_0}{50}\right)^3 \cdot h} \qquad \text{and} \qquad \frac{h}{b} \leq 5.0$$

$$b_{req} \geq \sqrt[4]{\left(\frac{14.6}{50}\right)^3 \cdot 0.876} = 0.384m < b_{prov} = 0.40m \qquad \text{and} \qquad \frac{h}{b} = \frac{87.6}{40} = 2.19 < 5.0 \ ✓$$

This means that the purlin is safe against tilting and no further analysis is necessary. The simplified proof is sufficient.

6.6 SERVICEABILITY LIMIT STATE DESIGN

6.6.1 STRESS LIMITS

First or second order?

$$\min P_{k,t} = r_{\inf} \cdot P_{m,t=\infty} = 0.95 \cdot 612kN = 581.4kN$$

$$N_{Ed,rare} = -\min P_{k,t} = -581.4kN$$

$$M_{Ed,rare} = M_{Ed,rare}^{G+Q} - \min P_{k,t} \cdot z_p = 580.7 - 581.4 \cdot 0.386 = 356.3kNm$$

$$\sigma_{c,rare} = \frac{N_{Ed,rare}}{A_c} + \frac{M_{Ed,rare}}{I_c} \cdot z_u$$

$$\sigma_{c,rare} = \frac{-581.4}{2297} + \frac{35630}{1{,}518{,}675} \cdot 49.8 = 0.91kN/cm^2 = 9.1MN/m^2$$

$$\sigma_{c,rare} = 9.1MN/m^2 > f_{ctm} = 2.90MN/m^2$$

→ *second order must be considered*

Concrete Compression Stresses

$$\frac{2 \cdot A_c \cdot z_p}{z \cdot x \cdot b_{eff}} = \frac{2 \cdot 2297 \cdot 38.6}{69.5 \cdot 26.3 \cdot 34.5} = 2.81 > 1.0 \qquad \rightarrow \min P_{k,t} = 581.4kN$$

$$N_{Ed,perm} = -581.4kN$$

$$M_{Ed,perm} = M_{Ed,perm}^{G+Q} - \min P_{k,t} \cdot z_p = 222.0 - 581 \cdot 0.386 = -2.36kNm$$

$$\sigma_{c,perm} = \frac{N_{Ed,perm}}{A_c} + \frac{2M_{Ed,perm}}{z \cdot x \cdot b_{eff}} \leq 0.45 f_{ck}$$

$$\sigma_{c,perm} = \frac{-581}{2297} + \frac{2 \cdot (-236)}{69.5 \cdot 26.3 \cdot 0.345} = -0.246kN/cm^2 = -2.46MN/m^2$$

$$\left|\sigma_{c,perm}\right| = 2.46MN/m^2 < 0.45 f_{ck} = 0.45 \cdot 30 = 13.5MN/m^2 \ ✓$$

Stresses in the Reinforcement

$$N_{Ed,rare} = -581.4kN$$

$$M_{Ed,rare} = M_{Ed,rare}^{G+Q} - \min P_{k,t} \cdot (z_r - z_p)$$

$$= 581 - 581.4 \cdot (0.426 - 0.386) = 604kNm$$

$$\sigma_{s,rare} = \left[\frac{M_{Ed,rare}}{z} + N_{Ed,rare}\right] \cdot \frac{1}{A_{s1} + A_p} \leq 0.8 f_{yk}$$

$$\sigma_{s,rare} = \left[\frac{60400}{69.5} - 581.4\right] \cdot \frac{1}{9.42 + 7.47} \cdot 10^{-1} = 170.5MN/m^2$$

$$\leq 0.8 \cdot 500 = 400MN/m^2 \checkmark$$

Stresses in the Prestressing Steel

$$\min P_{k,t} = P_{m,t=\infty} = 612kN \qquad N_{Ed,perm} = -\min P_{k,t} = -612kN$$

$$M_{Ed,perm} = M_{Ed,perm}^{G+Q} - \min P_{k,t} \cdot z_p = 222.0 - 612 \cdot 0.386 = 246.5kNm$$

$$\sigma_{p,perm} = \frac{P_{m,t=\infty}}{A_p}\left[\frac{M_{Ed,perm}}{z} + N_{Ed,perm}\right] \cdot \frac{1}{A_{s1} + A_p} \leq 0.65 f_{pk}$$

$$\sigma_{p,perm} = \frac{612}{7.47}\left[\frac{24650}{69.5} - 612\right] \cdot \frac{1}{9.42 + 7.47} \cdot 10^{-1} = 667MN/m^2$$

$$\leq 0.65 \cdot 1770 = 1150MN/m^2 \checkmark$$

6.6.2 MINIMUM REINFORCEMENT AGAINST CRACKING

Reinforcement needed?

$$\sigma_{c,rare} = 9.1MN/m^2 > 0$$

→ *a minimum reinforcement to prevent cracking is necessary*

Area of the Tension Zone

$$\sigma_l = f_{ctm} = \sigma_l^N + \sigma_l^M = 2.9MN/m^2 = \frac{-0.581}{0.2297} + \sigma_l^M$$

$$\sigma_l^M = 5.43MN/m^2 \qquad \rightarrow \sigma_u^M = -5.43MN/m^2$$

$$\sigma_u = \sigma_u^N + \sigma_u^M = \frac{-0.581}{0.2297} - 5.43 = -7.96MN/m^2$$

Area of tension zone $\quad h_t = \frac{2.9}{2.9 + 7.96} \cdot 87.6cm = 23.4cm \qquad \rightarrow A_{ct} = 19.0 \cdot 23.4 = \underline{\underline{444.3cm^2}}$

Minimum Required Cracking Reinforcement

Tensile strength $\quad f_{ct,eff} = f_{ctm} \geq 2.9MN/m^2 \qquad \rightarrow f_{ct,eff} = 2.9MN/m^2 \geq \underline{\underline{2.9MN/m^2}}$

$h^* = h \leq 1.0m \qquad \rightarrow h^* = 0.19m$

for $h < 30cm \qquad \rightarrow k = 0.8$

$$k_1 = 1.5$$

$$k_c = 0.4 \cdot \left[1 - \frac{\sigma_c}{k_1 \cdot h / h^* \cdot f_{ct,eff}}\right] = 0.4 \cdot \left[1 - \frac{581.4 / 2297}{1.5 \cdot 1.0 \cdot 0.29}\right] = 0.167 \leq 1.0$$

Crack width limit $w_k = 0.2\ mm$ *(according to DIN EN 1992-1-1, Tab. NA.7.1.)*

Limit diameter $\phi_s^* = \phi_s \cdot \frac{2.9}{f_{ct,eff}} = 20mm \cdot 1.0 = 20mm$

Reinforcement stresses $\sigma_s = 188.0 MN / m^2$ *(according to DIN EN 1992-1-1, Tab. NA.7.2.)*

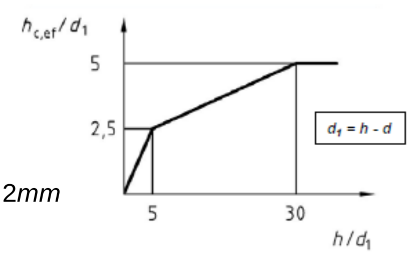
Required reinforcement $A_{s,\min,req} = k_c \cdot k \cdot f_{ct,eff} \cdot \frac{A_{ct}}{\sigma_s} = 0.167 \cdot 0.8 \cdot 2.9 \cdot \frac{444.3}{188.0} = 0.92 cm^2$

$< A_{s,prov} = 9.42 cm^2$ → *The provided reinforcement is sufficient!*

6.6.3 LIMITATION OF THE CRACK WIDTH

Percentage of Reinforcement

$$h / d_1 = 21.9 \quad \rightarrow h_{c,eff} / d_1 = 4.19$$



$$h_{c,eff} = 4.19 \cdot 4.0 cm = 16.8 cm$$

$$A_{c,eff} = 19.0 \cdot 16.8 = 318.4 cm^2$$

$$\xi = 0.6 \qquad \varnothing_s = 20mm \qquad \varnothing_p = 7.2mm$$

$$\xi_1 = \sqrt{\xi \cdot \phi_s / \phi_p} = \sqrt{0.6 \cdot 20 / 7.2} = 1.29$$

$$\rho_{tot} = (A_s + A_p) / A_{c,eff} = (9.42 + 7.47) / 318.4 = 0.0530$$

$$\rho_{p,eff} = (A_s + \xi_1^2 A_p) / A_{c,eff} = (9.42 + 1.29^2 \cdot 7.47) / 318.4 = 0.0687$$

Reinforcement Stresses

$$\sigma_s = \left[\frac{M_{Edr}}{z} + N_{Ed}\right] \cdot \frac{1}{A_s + A_p} + 0.4 \cdot f_{ct,eff} \cdot \left[\frac{1}{\rho_{p,eff}} - \frac{1}{\rho_{tot}}\right]$$

$$\sigma_s = \left[\frac{31520}{69.5} - 581.4\right] \cdot \frac{1}{9.42 + 7.47} + 0.4 \cdot 2.9 \cdot \left[\frac{1}{0.053} - \frac{1}{0.0687}\right] = -8.08 kN / cm^2$$

$$\rightarrow \quad \sigma_s = -80.8 MN / m^2 < 0$$

The stress is negative which means the cross section is under compression so there won't be any cracking.

6.7 DESIGN AND REINFORCEMENT

6.7.1 ANCHORAGE OF TENDONS

Transmission of the Stresses

Transmission length

$l_{pt} = \alpha_1 \cdot \alpha_2 \cdot \varnothing_p \cdot \sigma_{pm0} / f_{bpt}$

$\alpha_1 = 1.25$ *no wire stripping*

$\alpha_2 = 0.19$ *for strands*

$\varnothing_p = 12.5mm$

$f_{bpt} = 3.3MN/m^2$ *for C30/37 and good bond*

$\sigma_{pm0} = 1000MN/m^2$

$l_{pt} = 1.25 \cdot 0.19 \cdot 1.25 \cdot 1000 / 3.3 = 90.0cm$

Modified transmission length $l_{ptd} = 0.8 \cdot l_{pt} = 0.8 \cdot 90.0 = 72.0cm$

Dispersion length $l_{disp} = \sqrt{l_{ptd}^2 + d_p^2} = \sqrt{72.0^2 + 76.4^2} = 105.0cm$

Splitting tensile force

$F_{td} = P_{m,0} - \left|\sigma_{c,m}\right| \cdot A_{c,u}$

$N_{Ed}^P = -P_{m,t=0} = -747kN$ $M_{Ed}^P = N_{Ed}^P \cdot z_p = -747 \cdot 0.386 = -288kNm$

$A_{c,u} = a_p \cdot b = (15.9 - 1.25/2) \cdot 19 = 314cm^2$

$z_{c,u} = z_{s,u} - a_p / 2 = 49.8 - 16.5/2 = 41.5cm$

$$\sigma_{c,m}(x = l_{disp}) = \frac{N_{Ed}^P}{A_c} + \frac{M_{Ed}^P}{I_c} \cdot z_{c,u} = \frac{-747}{2297} + \frac{-28800}{1{,}518{,}675} \cdot 41.5 \cdot 10^{-1}$$

$$= -11.1MN/m^2$$

$F_{td} = 747 - 1.11 \cdot 314 = \underline{\underline{398kN}}$

Required reinforcement

$$A_{sp,req} = \frac{\gamma_{p,sp}}{3} \cdot \frac{F_{td}}{f_{yd}} = \frac{1.35}{3} \cdot \frac{398}{43.5} = 4.18cm^2$$

The reinforcement is to be distributed over the length of

$0.75 \cdot l_{disp} = 0.75 \cdot 105 = 78.8cm$

$a_{sp,req} = 4.18 / 0.788 = \underline{\underline{5.31cm^2/m}}$

Chosen **Ø 8 / 15 = 6.71 cm²/m** **stirrups**

Tensile forces at the edge $Z_{Ed}^P = P_{m0} \cdot \left[\frac{e}{h} - \frac{1}{6}\right] = 747 \cdot \left[\frac{38.6}{100} - \frac{1}{6}\right] = 163.9kN$

Required reinforcement $A_{sz,req} = Z_{Ed} / f_{yd} = 163.9 / 43.5 = \underline{\underline{3.77 cm^2}}$

Chosen **3 Ø 14 = 4.62 cm²** **U-bars**

Anchorage of Tendons in the Ultimate Limit State

Anchorage length

$$l_{bpd} = l_{pt2} + \alpha_2 \cdot \varnothing_P \cdot \frac{\sigma_{pd} - \sigma_{pm,t=\infty}}{f_{bpd}}$$

$$l_{pt2} = 1.2 \cdot l_{pt} = 1.2 \cdot 90.0 = 108.0 cm \qquad f_{bpd} = 3.3 MN / m^2$$

$$\sigma_{pd} = f_{p,01k} / \gamma_p = 1500 / 1.15 = 1304 MN / m^2$$

$$\sigma_{pm,t=\infty} = P_{m,t=\infty} / A_p = 612 / 7.47 = 81.9 kN / cm^2$$

$$l_{bpd} = 108 + 0.19 \cdot 1.25 \cdot \frac{1304 - 81.9}{3.3} = \underline{\underline{143 cm}}$$

Internal forces at x

$$N_{P,t=\infty} = -P_{m,t=\infty} = -612 kN \qquad M_{P,t=\infty} = -612 \cdot 0.386 = -237 kNm$$

$$q_{Ed} = 1.35 \cdot (5.74 + 2.59) + 1.5 \cdot 13.1 + 0.9 \cdot 0.6 = 31.4 kN / m$$

$$x = l_{disp} + a_i = 143 + 21.5 = 164.5 cm$$

$$M_{Ed}(x = 165) = 31.4 \cdot 14.6 \cdot 1.65 / 2 + 31.4 \cdot 1.65^2 / 2 = 336 kNm$$

$$M_{Ed} = M_{Ed}(x) + M_{Ed}^P = 336 - 236 = 99 kNm$$

Concrete stresses

$$\sigma_{c,u} = \frac{N_{Ed}}{A_c} + \frac{M_{Ed}}{I_c} \cdot z_u = \frac{-612}{2297} + \frac{9900}{1{,}518{,}675} \cdot 49.8 = 0.059 kN / m^2$$

$$\sigma_{c,u} = 0.59 MN / m^2 < 0.7 \cdot f_{ctm} = 0.7 \cdot 2.9 = 2.03 MN / m^2 \ \checkmark$$

6.7.2 MINIMUM REINFORCEMENT TO ENSURE ROBUSTNESS

$$M_{cr} = f_{ctm} \cdot W_{cu} \qquad W_{cu} = 30{,}520 cm^3$$

$$M_{cr} = 2.9 \cdot 30{,}520 \cdot 10^{-3} = 88.5 kNm$$

$$A_{s,\min} = \frac{1}{f_{yk}} \cdot \frac{M_{cr}}{z} = \frac{1}{50.0} \cdot \frac{8850}{69.5} = 2.55 cm^2 < A_{s,prov} = 9.42 cm^2 \ \checkmark$$

6.7.3 DESIGN OF THE SUPPORT

Width of the Joints

The width of the joints between two purlins facing each other and between the purlin and the concrete truss can be estimated according to [4] and [5]. The width of the joints is the result of the sum of the different possible dimensional deviations during the process of the production and assembly of the beams.

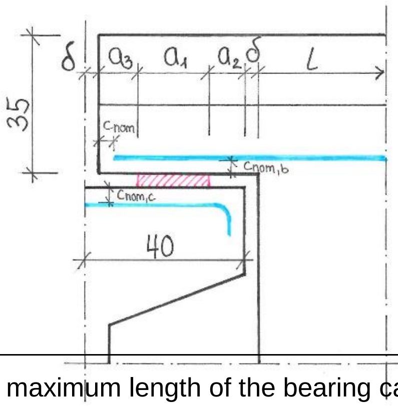
$$\delta_{comb} = \delta_{\max} + \sqrt{\sum \delta_i^2}$$ with δ_{comb} Total allowed dimensional deviation

$\delta_{\max}$ Maximum dimensional deviation

δ_i Other possible dimensional deviations.

The different dimensional deviations can be taken from DIN 18202 and DIN 18203.

Here it is:



$$\delta_{comb} = \delta_{\max} + \sqrt{\sum \delta_i^2} = \frac{1}{2} \cdot (24 + \sqrt{16^2 + 16^2 + 24^2}) = 28.5mm$$

Chosen $\delta = 30mm$

Dimensions of the Bearing

The width of the elastomeric bearing is chosen to be b = 200 mm. The maximum length of the bearing can be calculated as per DIN EN 1992-1-1, 10.9.5.2 with following formula:

$$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2}$$

with

a_1 Length of the elastomeric bearing,

$a_1 > \min a_1$ (DIN EN 1992-1-1, Tab. 10.2)

a_2 Distance of the bearing to the edge of the support

a_3 Distance of the bearing to the edge of the beam

Δa Dimensional deviations.

Here:

Assumption $a_1 \approx 250mm$ → $\sigma_{Ed} = V_{Ed} / a_1 \cdot b_1 = 230kN / 25 \cdot 20 = 0.46kN / cm^2$

$\min a_1 = 70mm$ *for* $0.15 < \sigma_{Ed} / f_{cd} = 0.46 / 1.7 = 0.27 < 0.40$ (DIN EN 1992-1-1, Tab.10.2)

$a_2 = c_{nom} + d_s / 2 + c_{nom,c} / 2 = 20 + 10 / 2 + 20 / 2 = 35mm > \min a_2 = 15mm$ (DIN EN 1992-1-1, Tab.10.3)

$a_3 = c_{nom} + d_s / 2 + c_{nom,b} / 2 = 20 + 12 / 2 + 20 / 2 = 36mm > \min a_3 = 15mm$ (DIN EN 1992-1-1, Tab.10.4)

$\Delta a_2 = l / 1200 = 14700 / 1200 = 12.25mm$ (DIN EN 1992-1-1, Tab.10.5)

$\Delta a_3 = l / 2500 = 14700 / 2500 = 5.88mm$

→ $a_{req} = 250 + 35 + 36 + \sqrt{12.25^2 + 5.88^2} = 334.6mm < a_{prov} = 400 - 30 = \underline{\underline{370mm}}$ ✓

Chosen **a_1 = 280 mm** **length of the bearing**

Elastomeric Bearing

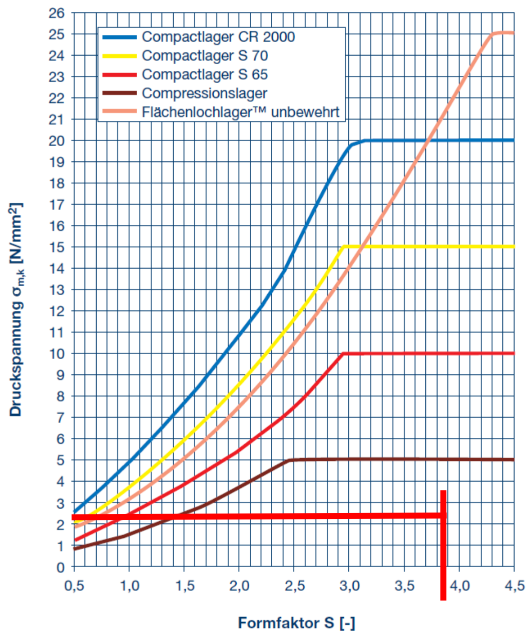
The elastomeric bearing that is used here is by the company *Calenberg Ingenieure* and is chosen in accordance with the technical information and dimensioning tables of the bearings [6].

Size of the bearing l x b x t = 280 x 200 x 15 mm

Bearing compression $\sigma_{m,k} = \dfrac{V_{Ed}}{a_1 \cdot b_1} = \dfrac{230}{28 \cdot 20} = 0.41 kN/cm^2 = 4.1 MN/m^2$

Form factor $S = \dfrac{58}{15} = 3.87$

Chosen **'Compressionslager'** **elastomeric bearing**



6.7.4 DESIGN OF THE NOTCH

Vertical Stirrups

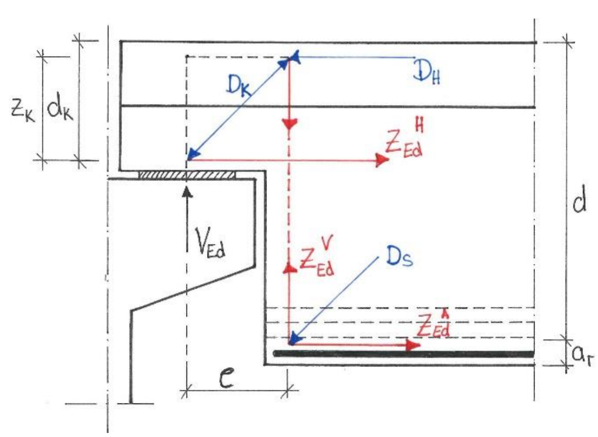
$Z_{Ed}^{V} = V_{Ed} = 230 kN$

$A_{s,req}^{V} = Z_{Ed}^{V} / f_{yd} = 230 / 43.5 = 5.28 cm^2$

Chosen **6 Ø 8 / e = 5cm**

$l_{b,ind} = 5 \cdot 5 + 2 \cdot 0.8 / 2 = 26 cm$

$e = 18.5 + 3 + 2 + 26 / 2 = 36.5$



Design of the Console

Minimum static height $\min d_k = \dfrac{3.4 \cdot V_{Ed}}{b \cdot f_{cd}} = \dfrac{3.4 \cdot 230}{0.24 \cdot 1.7} = 19.2cm$

$d_{k,prov} \approx 30cm > \min d_k = 19.2cm$

Lever arm of the internal forces $z_k \approx 0.9 \cdot d_k = 0.9 \cdot 31 = 27cm$

Horizontal force at support $Z_{Ed}^H = V_{Ed} \cdot \dfrac{e}{z_k} = 230 \cdot \dfrac{36.5}{27} = 311kN$

Required reinforcement $A_{s,req} = \dfrac{Z_{Ed}^H}{f_{yd}} = \dfrac{311}{43.5} = 7.2cm^2$

Chosen **4 Ø 12 =9.05cm²** **reinforcement loops**

Static height $d_{k,prov} = 35 - 2 - 1 - 1.2 - 2/2 = 29.8cm \approx d_{k,est} = 30cm$ ✓

Anchorage of the Reinforcement Loops at the Direct Support

$l_{b,eq} = \alpha_1 \cdot \dfrac{A_{s,req}}{A_{s,prov}} \cdot l_{b,rqd} > l_{b,\min}$ $l_{b,rqd} = 43cm$ *for Ø = 12mm, C30/37 and good bond*

$l_{b,eq} = 0.7 \cdot \dfrac{7.2}{9.05} \cdot 43 = 23.9cm$ $l_{b,\min} = 0.3 \cdot \alpha_1 \cdot l_{b,rqd} = 0.3 \cdot 0.7 \cdot 43 = 9cm > 10 \cdot 1.2 = \underline{\underline{12cm}}$

$l_{b,eq} = 23.9cm > l_{b,\min} = 12cm$ → $l_{b,dir} = \dfrac{2}{3} \cdot l_{b,eq} = \dfrac{2}{3} \cdot 23.9 = \underline{\underline{15.9cm}} > 6.7 \cdot 1.2 = 8.0cm$

$l_{b,dir,prov} = 32.5 - 2 - 1.2 - 2/2 = 28.3cm > l_{b,dir,req} = 15.9cm$ ✓

Anchorage of the Reinforcement Loops at the Indirect Support

$l_{b,eq} = \alpha_1 \cdot \dfrac{A_{s,req}}{A_{s,prov}} \cdot l_{b,rqd} > l_{b,\min}$ $l_{b,rqd} = 61cm$ *for Ø = 12mm, C30/37 and moderate bond*

$l_{b,eq} = 1.0 \cdot \dfrac{7.2}{9.05} \cdot 61 = 48.5cm$ $l_{b,\min} = 0.3 \cdot \alpha_1 \cdot l_{b,rqd} = 0.3 \cdot 61 = \underline{\underline{18.3cm}} > 10 \cdot 1.2 = 12cm$

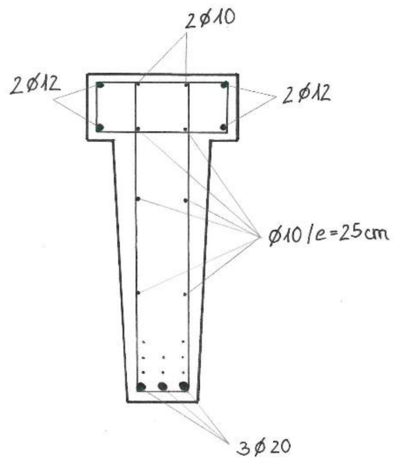
$l_{b,eq} = 48.5cm > l_{b,\min} = 18.3cm$ → $l_{b,ind} = \underline{\underline{48.5cm}}$

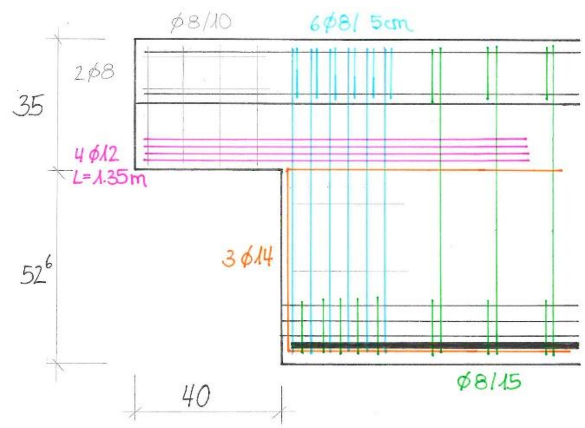
Length of the loops $\min l > l_{b,dir} + a + b + c_{nom} + l_{b,ind}/2 + \left((d - d_k)/\tan\Theta\right) + l_{b,ind,l}/2$

$\min l > 28 + 4.5 + 3 + 2 + 26/2 + \left((80.4 - 30)/\tan 40\right) + 48.5/2$

$= \underline{\underline{135cm}}$

6.7.5 REINFORCEMENT DRAWING



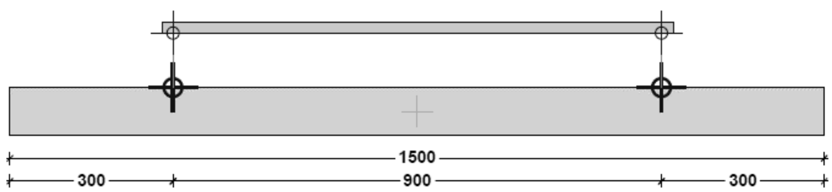


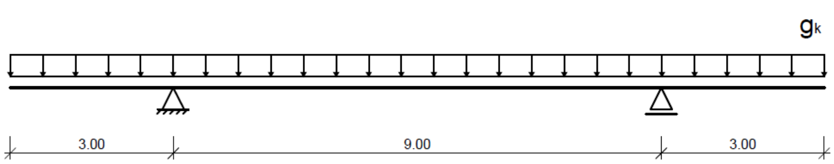
6.8 TRANSPORT AND ASSEMBLY

6.8.1 TRANSPORT AND LIFTING ON THE CONSTRUCTION SITE

The beam will be suspended with the help of anchors at around 1/5 of the length of the beam. Which might cause tension over the anchors and might require upper reinforcement.

In the following the static system during transportation and assembly can be seen.





Lifting out of the Formwork

Lifting loads *self-weight* $g_k = 5.74\ kN/m$

Bending moment $M_{Ed}^{G} = 1.15 \cdot (-5.74) \cdot 3.0^2 / 2 = -29.7kNm$

$M_{Eds} = M_{Ed}^{G} - N_{Ed} \cdot z_{s1} = 29.7 - (-747.0) \cdot (0.378 - 0.04) = 282kNm$

Time of first lifting *approximately after 5 days (t = 5d)*

Compressive strength at time t $f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm}$ $\beta_{cc}(t = 5d) = e^{s\left(1-\sqrt{28/t}\right)} = e^{0.2\left(1-\sqrt{28/5}\right)} = 0.76$

$f_{cm}(t = 5d) = 0.76 \cdot 38 = 28.88 N/mm^2$

$f_{ck}(t = 5d) = f_{cm}(t) - 8 = 28.88 - 8 = 20.88 N/mm^2$

$f_{cd}(t = 5d) = 0.85 \cdot f_{ck}(t) / 1.5 = 0.85 \cdot 20.88 / 1.5 = \underline{\underline{11.8 N/mm^2}}$

Required reinforcement $\mu_{Eds} = \dfrac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \dfrac{28200}{19 \cdot 83.6^2 \cdot 1.18} = 0.18$ → $\omega = 0.2007$

$A_{s2,req} = (\omega \cdot b \cdot d \cdot f_{cd} + N_{Ed}) / \sigma_{sd}$

$= (0.2007 \cdot 19.0 \cdot 83.6 \cdot 1.18 - 747) / 43.5 = \underline{\underline{-8.5 cm^2}} < 0$

No additional upper reinforcement is required!

6.8.2 TILTING

According to DIN EN 1992-1-1, 5.9 a simplified proof of the safety against tilting can be conducted with following formula:

$b_{req} \geq \sqrt[4]{\left(\dfrac{l_0}{70}\right)^3 \cdot h}$ and $\dfrac{h}{b} \leq 5.0$ for tilting during transport and assembly of the beam.

$b_{req} \geq \sqrt[4]{\left(\dfrac{9.0}{70}\right)^3 \cdot 0.876} = 0.21 m < b_{prov} = 0.40 m$ and $\dfrac{h}{b} = \dfrac{87.6}{40} = 2.19 < 5.0$ ✓

This means that the purlin is safe against tilting during transport and assembly and no further analysis is necessary. The simplified proof is sufficient.

6.8.3 TRANSPORT ANCHORS

In order to lift the beam out of its formwork as well as to be able to transport and assemble it on site, transport anchors are needed. Those are installed during the production of the beam and must be designed in accordance with the VDI/BV-BS-Richtlinie 6205. For this project the anchors of the company *Halfen* are chosen.

When choosing the right anchors two different cases must be distinguished. For lifting the beam out of the formwork, a traverse is used which results in only vertical tensile forces on the anchor. For the lifting on site, a suspension gear is used which causes inclined tensile forces.

Lifting out of the Formwork

Self-weight $F_G = 5.74 \cdot 15 = 86.1 kN$

Formwork adhesion $F_{adh} = q_{adh} \cdot A_f$ → *no adhesion here because of hinged formwork!*

Dynamic factor $\psi_{dyn} = 1.3$

Factor for inclination $z = 1.0 \;\; for \;\; \beta = 90°$

Tensile force $$F_z = \frac{F_G \cdot \psi_{dyn} \cdot z}{n} = \frac{86.1 \cdot 1.3 \cdot 1.0}{2} = \underline{\underline{56.0kN}}$$

Lifting on Site

Dynamic factor $\psi_{dyn} = 1.3$

Factor for inclination *and* $z = 1.16$ *for* $\beta = 30°$

Tensile force $$F_z = \frac{F_G \cdot \psi_{dyn} \cdot z}{n} = \frac{86.1 \cdot 1.3 \cdot 1.16}{2} = \underline{\underline{64.9kN}}$$

Chosen **HALFEN DEHA Spherical head anchor 6000-7,5-0300**

Load capacity of the anchor $F_{z,Rd} = 69.9kN > F_{z,Ed} = 64.9kN$ ✓ (see Table 16)

Table 16 – Sizing Table for 'HALFEN DEHA Lifting Anchor Systems'

Spherical head anchors in beams and walls with no special requirements on the reinforcement (load class 1,3 – 7,5)									
Load class	Article number	Anchor length l [mm]	Minimum height of beams B_1 [mm]	Wall thickness $2 \times e_r$ [mm]	Load capacity [kN] at concrete strength f_{ci} for: **Axial pull** up to 30° [β] 15 N/mm²	**Diagonal pull** up to 60° [β] 15 N/mm²	**Axial pull** and **diagonal pull** up to 60° [β] 25 N/mm²	**Axial pull** and **diagonal pull** up to 60° [β] 35 N/mm²	Axial spacing of anchors e_z [mm]
7,5	6000-7,5-0200	200	410	240	45.1	36.0	58.2	68.8	610
				260	47.8	38.3	61.8	73.1	
				280	50.6	40.5	65.3	75.0	
	6000-7,5-0300	300	610	200	54.1	43.3	69.9	75.0	910
				220	58.1	46.5	75.0		
				240	62.2	49.7			
	6000-7,5-0540	540	1090	160	63.2	58.4	75.0	75.0	1630
				180	71.1	63.8			
				200	75.0	69.1			

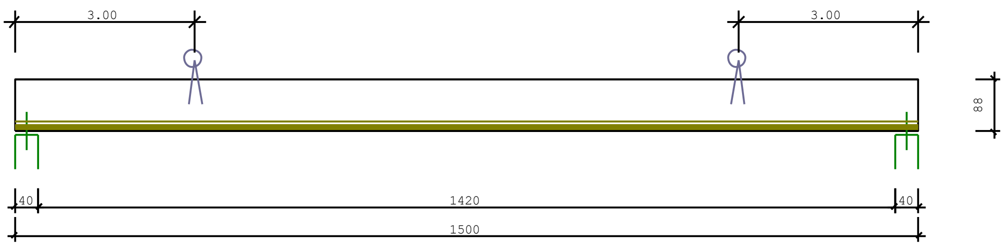
f_{ci} = concrete cube strength at time of lifting

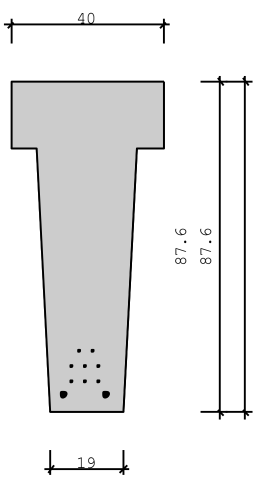
6.9 SOFTWARE CALCULATIONS

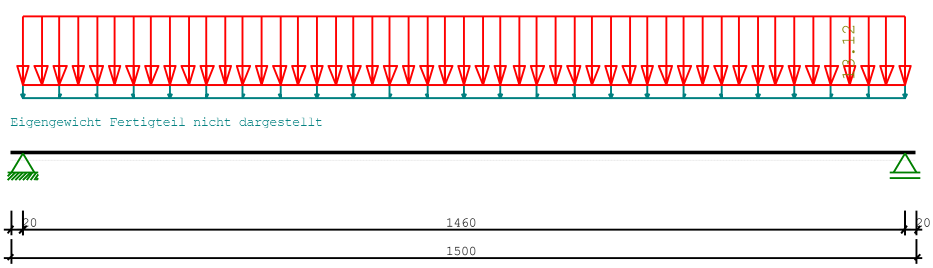
Demo Frilo Nemetschek

Position: P.02 – Prestressed Concrete Purlin

Spannbettbinder B8 01/2018 (Frilo R-2018-1/P12)







Advices:

No continuous reinforcement to 10 cm below UE found!

Material:

Prestressing steel

SpSt 1570/1770 Strand 7 wires

Reinforcing steel:

Longitudinal	Stirrup
B500B	B500B

Concrete:

Precast
C 30/37

Loads:

Self weight

Beam beginning	g11	=	5.70 kN/m
Beam end	g12	=	5.70 kN/m
Total	G	=	85.5 kN
Volume	V	=	3.42 m^3
Surf.	A	=	31.68 m^2

Live loads

Units: Single load[kN] Single moment[kNm] line load[kN/m]

span	type	gle	qle	Dist. a [m]	gri	qri	Length [m]	Fact	Act.	Sim. Pos.
1	1	2.59	13.12					1.00	10	0

Load types: 1 = uniformly distr., 2 = single load at a, 3 = single moment at a
4 = trapezoidal load from a, 5 = triangle load over L

Actions:

Act.	γ_Q	ψ_0	ψ_1	ψ_2	Dep.	Cat.	Description
10	1.50	0.50	0.20	0.00	0	S	Schnee bis NN +1000m

Tendons:

Dist(LE) > 4.8 cm   axis horizontal > 4.3 cm   vertical > 3.7 cm

lay. No.	num- ber	area Ap [cm^2]	Dist.LE Yp [cm]	Prestressing $\sigma_p^{(0)}$ [N/mm^2]	<--- Isolations Count	to x1 [m]	---> from x2 [m]
1	3	2.79	7.5	1000	0		
2	3	2.79	11.7	1000	0		
3	2	1.86	15.9	1000	0		

```
x1 and x2 with respect to the left beginning from joint
The calculation of the losses due to creep, shrinkage and relaxation
following the method from Abelein
--------------------------------------------------------------------------

RESULTS ( summary)
--------------------------------------------------------------------

Reaction forces (t = infinitely):

Units: all [kN] G:perm., Q:variable. ,V: Sum
                                <------char. value-----> <--ULS(PT)---->
Support point               G      min Q      max Q      min V      max V
--------------------------------------------------------------------------
A (left)                61.66       0.00      95.78      61.66     226.90
B (right)               61.66       0.00      95.78      61.66     226.90
--------------------------------------------------------------------------

max. bending moment in erection state(char. value):

    MF =  =   570.36 kNm            at x  =  =   7.50 m

Required shear reinforcement:

Column A:   asw  =   2.73 cm²/m
Column B:   asw  =   2.73 cm²/m

Bursting reinforcement

left   Laying length  =    0.95 m
       from x         =    0.00 m           As  =   3.8 cm²
right  Laying length  =    0.95 m
       from x         =   15.00 m           As  =   3.8 cm²

Check of anchorage

left: Tensile force resistance in anchoring area   Util  =   1.02
         additional reinforcement necessary
right: Tensile force resistance in anchoring area  Util  =   1.02
         additional reinforcement necessary


Overview crit. sections

    Selected basic grid:     20 Sections

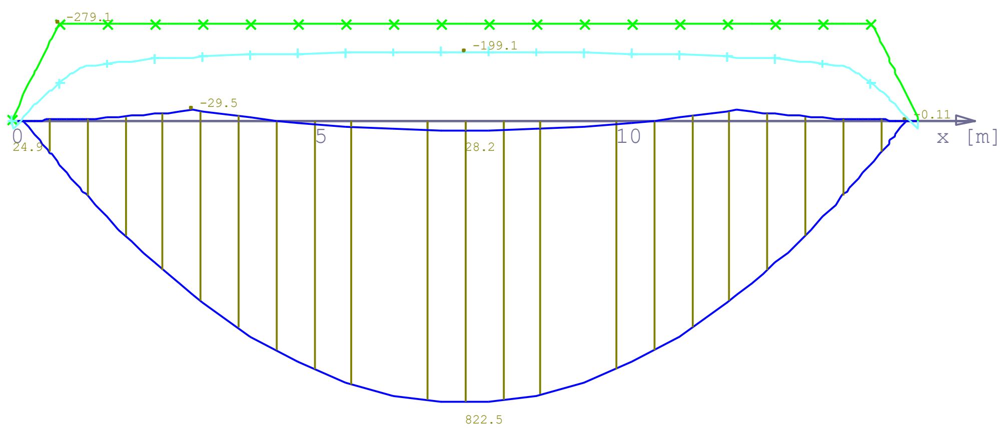
Checkvalue                            Extrem        Utilisation  x [m]
--------------------------------------------------------------------------
Flexural capacity bottom         η =    1.02                 0.98   7.50
Flexural capacity  top           η =    ****                 ****   0.59
Resisting tens force  top        η =    ****                 ****   0.59
Prc.:Compr.stress  Cc           σc =  -17.20 N/mm²           0.96   3.00
Tension prestress. steel     σp,Qc =   963.2 N/mm²           0.84   7.50
Stress in prestress.steel    σp,Cc =  1110.9 N/mm²           0.79   7.50
Stress in rebars                σs =   156.8 N/mm²           0.39   7.50
Crack MinAs+AsDuc bottom     AsMin =     2.4 cm²             0.37   0.02
Crack MinAs+AsDuc top        AsMin =    **** cm²             ****   0.02
Crack width     bottom          wk =    0.00 mm              0.01   7.50
Crack width      top            wk =    **** mm              ****   0.47
Sagging           top           fo =    -1.0 cm              0.17   7.11
Sagging          bottom         fu =     0.1 cm              0.01   0.00
```

Incr.-deflection(Util)	|df|=	0.7 cm	0.24	7.89
Prc.:Shear reinf (web)	asw =	2.73 cm^2/m	1.00	1.23
Concrete strut capacity	η =	2.72	0.37	14.60

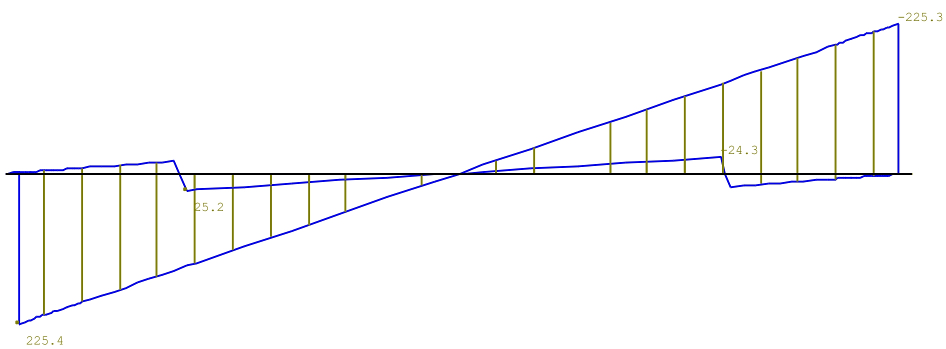
---- Check not required
**** Check not fulfilled
Prc.:Precast member   Add.: in-situ supplement
IS : Installed state   SC : State of construction
AsDuk:Ductility reinforcement

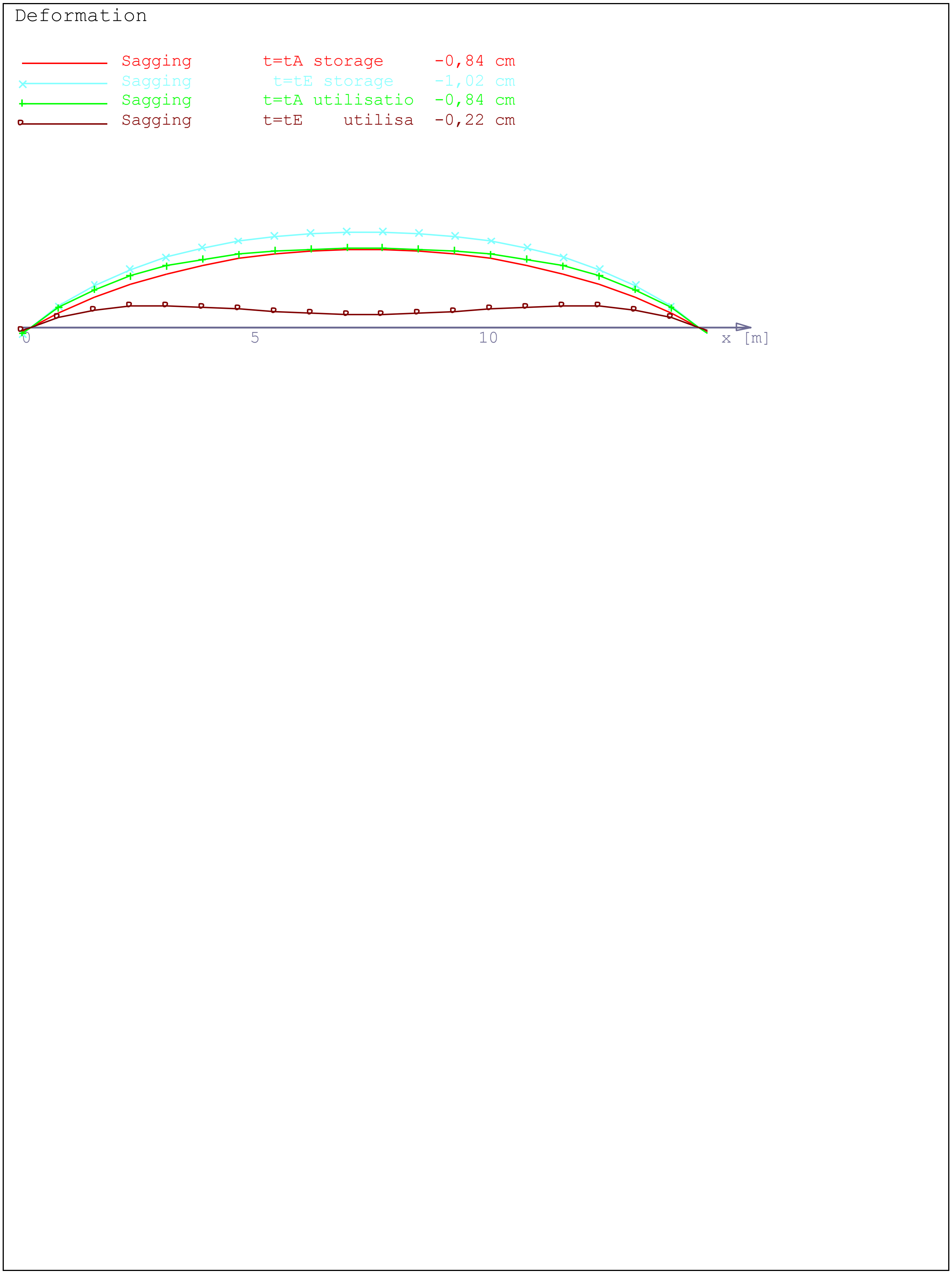
Internal forces

max MEd from external loads (PT)
min MEd from external loads (PT)
Moment from prestressing, t= tA storage
Moment from prestressing, t= tE use



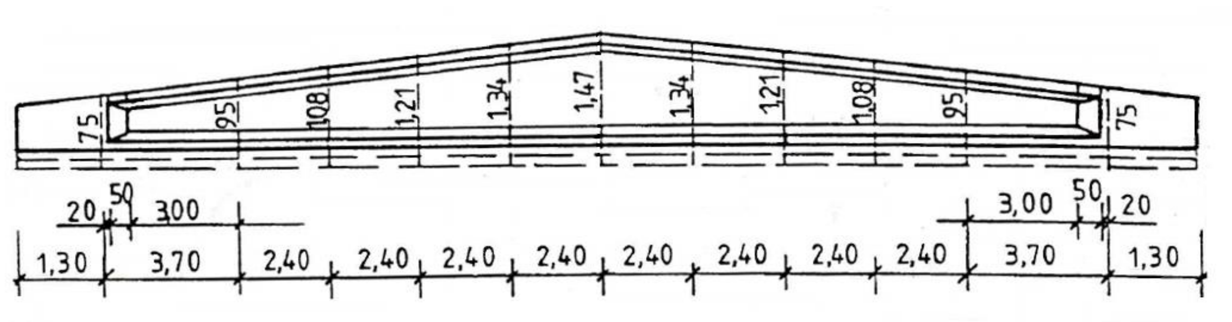
max VEd from external loads (PT)
min VEd from external loads (PT)



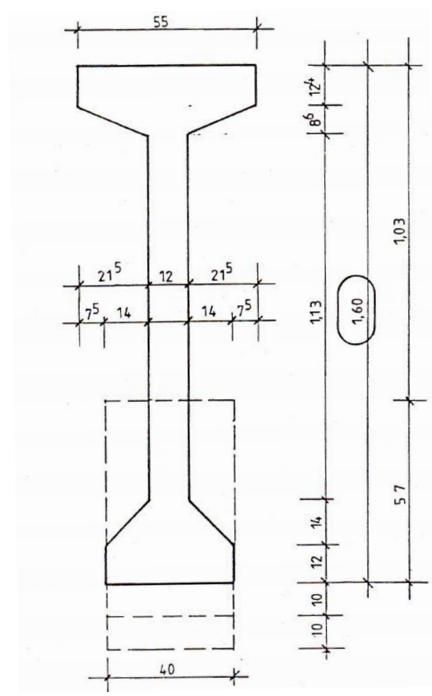

Deformation
Sagging t=tA storage -0,84 cm
Sagging t=tE storage -1,02 cm
Sagging t=tA utilisatio -0,84 cm
Sagging t=tE utilisa -0,22 cm
0
5
10
x [m]

7 POS. P.03 – PRESTRESSED CONCRETE TRUSS

In the following the pre-stressed concrete truss will be designed. To ensure a roof drainage a tapered beam with straight lower chord is chosen with a slope of 3.1°. The design will be made according to following formwork layout.



The respective cross section is shown in the drawing.



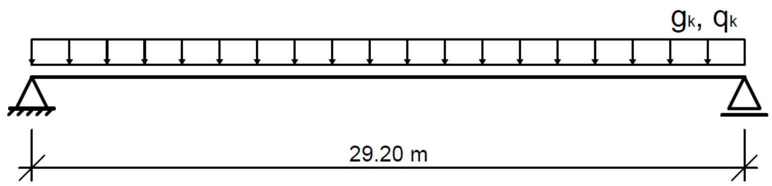
The height of the lower chord as well as the height of the web can be adjusted according to the static analysis and the constructional needs. Though the maximum height of the web cannot exceed 1.13m.

Each truss will span between two columns that are arranged at the intersections of the numerical and alphabetical axes of the building resulting in a span length of approximately 29.2 m. This means that in each numerical axis in total three pre-stressed concrete trusses are positioned.

To be able to approximate the needed dimensions of the truss and the amount of pre-stressing steel a preliminary design must be made.

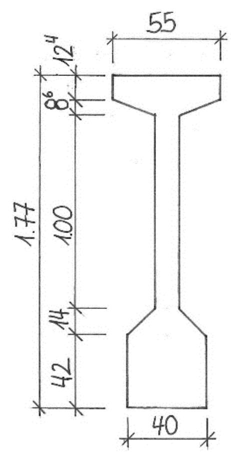
7.1 PRELIMINARY DESIGN

System



Span L = 29.2 m, Distance a = 15 m

Cross-section



Building materials **C 45/55 | St 1570/1770 | B 500A**

Exposure class XC1, WO

Characteristic loads

Uniformly Distributed Loads		g_k **[kN/m]**	q_k **[kN/m]**
Self-weight			
Max. self-weight	$g_{k1,max}$ = 0.42 cm² x 25 kN/m³=	10.50	-
Min. self-weight	$g_{k1,min}$ = 0.33 cm² x 25 kN/m³=	8.25	-
	$g_{k,mean}$ = (10.50 + 8.25)/2=	9.40	
Purlin+Roof Structure			
g_{k2} = (4 x 8.69 + 4 x 6.69) kN/m x 14.70m/29.20m =		31.00	-
Snow			
q_{k1} = (4 x 12.25 + 4 x 4.46) kN/m x 14.7m/29.20m =		-	34.20
Wind (Area I)			
q_{k2} = (4 x 0.57 + 4 x 0.21) kN/m x 14.7m/29.20 =		-	1.60

Design location $a = d_a / \tan\alpha = 0.98 / \tan(3.1°) = 18.08m$

$x = -a + \sqrt{a^2 + a \cdot L} = -18.08 + \sqrt{18.08^2 + 18.08 \cdot 29.2} = 11.16m$

Bending moments At the design location x = 11.16m:

M_{gk1} [kNm]	M_{Gk2} [kNm]	M_{Qk1} [kNm]	M_{Qk2} [kNm]
946.2	3120.0	3442.4	161.1

Assumptions

Prestressing $\sigma_{m,t=0} \approx 1000N / mm^2$

Prestressing loss $\approx 15\%$

Prestressing with immediate bond

Cross-section Properties

At $x = 11.16m$ $h_{x=11.16} = 158cm$

Center of gravity $z_{cu} = 73cm$

$A_c = 3986.1cm^2$ $I_{yc} = 11{,}720{,}979cm^4$

$$W_{cu} = \frac{I_{yc}}{z_{cu}} = \frac{11{,}720{,}979cm^4}{73cm} = 160{,}577cm^3$$

Bending Compression Zone

Effective depth $d = 158cm - 15cm = 143cm$

$\delta = h_f / d = 16.7 / 143 = 0.1168$

$\beta = b_w / b_{eff} = 12cm / 55cm = 0.218$

$$\rightarrow \eta = \frac{1}{(1-\beta)\cdot(1-0.5\delta)\cdot\delta + 0.32\beta} = \underline{\underline{6.42}}$$

$$M_{Ed}^{G+Q} = 1.35 \cdot (946.2 + 3120.3) + 1.50 \cdot 3442.4 + 1.5 \cdot 0.6 \cdot 161.1 = 10{,}940kNm$$

$$b_{eff,req} \geq \frac{M_{Ed}}{f_{cd}} \cdot \eta / d^2 = \frac{10.94MNm}{25.5MN/m^2} \cdot 6.42 / 1.43^2 = 1.35m$$

$b_{eff,prov} = 55cm < b_{eff,req} = 135cm$ ↯

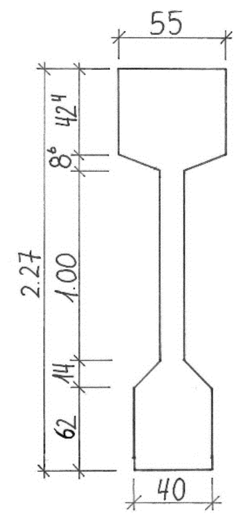
→ *The provided bending compression zone is NOT sufficient.*

A different cross-section has to be chosen or the compression zone must be increased by increasing the height or the length of the compression flange.

7.1.1 CONCRETE TRUSS ALTERNATIVE 1

To strengthen the compression zone, the height of compression flange can be increased. Additional 30 cm of height will be added to the compression flange and 20 cm will be added to the tension zone. So, the maximum height of the truss is 227 cm.

Cross-section



Building materials **C 45/55 | St 1570/1770 | B 500A**

Exposure class XC1, WO

Characteristic loads $\Delta g_k \approx (0.40 \times 0.20 + 0.55 \times 0.30) \times 25\ kN/m^3 = 6.10\ kN/m$

$gk_{1,new} = 9.40 + 6.10 = 15.5\ kN/m$

Design location $a = d_a / \tan\alpha = 1.48 / \tan(3.1°) = 27.31m$

$x = -a + \sqrt{a^2 + a \cdot L} = 11.97m$

Bending moment At the design location x = 11.97m:

M_{gk1} [kNm]	M_{Gk2} [kNm]	M_{Qk1} [kNm]	M_{Qk2} [kNm]
1598.6	3197.2	3527.2	165.0

Cross-section Properties

At $x = 11.97m$ $h_{x=11.97} = 213cm$

Center of gravity $z_{cu} = 113cm$

$A_c = 6218cm^2$ $I_{yc} = 33{,}899{,}542cm^4$

$$W_{cu} = \frac{I_{yc}}{z_{cu}} = \frac{33{,}899{,}542cm^4}{113cm} = 272{,}902cm^3$$

Bending Compression Zone

Effective depth

$$d = 213cm - 15cm = 198cm$$

$$\delta = h_f \;/\; d = 46.2 / 198 = 0.2361$$

$$\beta = b_w \;/\; b_{eff} = 12cm / 55cm = 0.218$$

$$\rightarrow \; \eta = \frac{1}{(1-\beta)\cdot(1-0.5\delta)\cdot\delta + 0.32\beta} = \underline{\underline{4.30}}$$

ULS

$$M_{Ed}^{G+Q} = 1.35 \cdot M_G + 1.50 \cdot M_{Q1} + 1.5 \cdot 0.6 \cdot M_{Q2} = 12{,}154kNm$$

$$b_{eff,req} \geq \frac{M_{Ed}}{f_{cd}} \cdot \eta / d^2 = \frac{12.154MNm}{25.5MN / m^2} \cdot 4.30 / 1.98^2 = 0.523m$$

$$b_{eff,prov} = 55cm > b_{eff,req} = 52.3cm \checkmark$$

→ *The provided bending compression zone is sufficient.*

Required Prestressing Steel

SLS

$$M_{Ed,frequ} = M_G + 0.2 \cdot M_{Q1} + 0 \cdot M_{Q2} = 5501.2kNm$$

$$r_{\inf} = 0.95 \;,\; \alpha_{loss} = 0.15 \;,\; \sigma_{pm,t=0} = 1000N / mm^2$$

$$z_{cp} = 113cm - 15cm = 98cm$$

Required prestressing steel

$$A_{P,req} \geq \frac{M_{Ed}}{r_{\inf} \cdot (1-\alpha_{loss}) \cdot \sigma_{pm,t=0} \cdot \left(\dfrac{W_{cu}}{A_c} + z_{cp}\right)}$$

$$= \frac{550{,}120kNcm}{0.95 \cdot (1-0.15) \cdot 100kN / cm^2 \cdot \left(\dfrac{300{,}176}{6218} + 98.0\right)} = \underline{\underline{46.60cm^2}}$$

→ *The required cross-section of the pre-stressing steel is* ***46.60cm².***

Tension Zone

SLS

$$\min M_{Gk} = 1599kNm \qquad M_{Ed,frequ} = 5501.2kNm$$

$$W_{cu,req} \geq \frac{\dfrac{M_{Ed}}{r_{\inf} \cdot (1-\alpha_{loss})} - \min M_{Gk}}{0.6 \cdot f_{ck}} = \frac{\dfrac{550{,}120}{0.95 \cdot (1-0.15)} - 159{,}900}{0.6 \cdot 4.5} = \underline{\underline{193{,}113cm^3}}$$

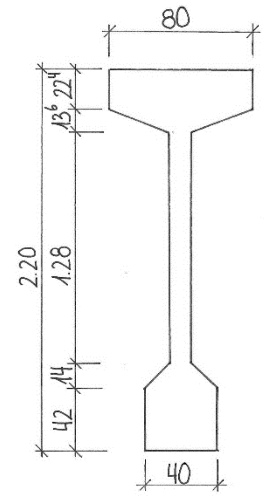
$$W_{cu,prov} = 193{,}113cm^3 > W_{cu,req} = 300{,}176cm^3 \checkmark$$

→ *The tension zone of the purlin is sufficient.*

7.1.2 CONCRETE TRUSS ALTERNATIVE 2

Another option is to use a higher truss with a wider compression flange.

Cross-section



Building materials **C 50/60 | St 1570/1770 | B 500A**

Design location $x = 11.89m$

Cross-section Properties

At $x = 11.89m$ *:* $h_{x=11.89} = 205cm$

Center of gravity $z_{cu} = 114.1cm$

$A_c = 5818cm^2$ $\quad I_{yc} = 31{,}841{,}471cm^4$

$$W_{cu} = \frac{I_{yc}}{z_{cu}} = \frac{31{,}841{,}471cm^4}{114.1cm} = 279{,}176cm^3$$

Bending Compression Zone

$b_{eff,req} = 0.689m < b_{eff,prov} = 80cm$ ✓

→ *The provided bending compression zone is sufficient.*

Required Prestressing Steel

$A_{P,req} = \underline{\underline{45.94cm^2}}$

→ *The required cross-section of the pre-stressing steel is **45,94 cm².***

Tension Zone

$W_{cu,prov} = 279{,}177cm^3 > W_{cu,req} = 172{,}009cm^3$ ✓

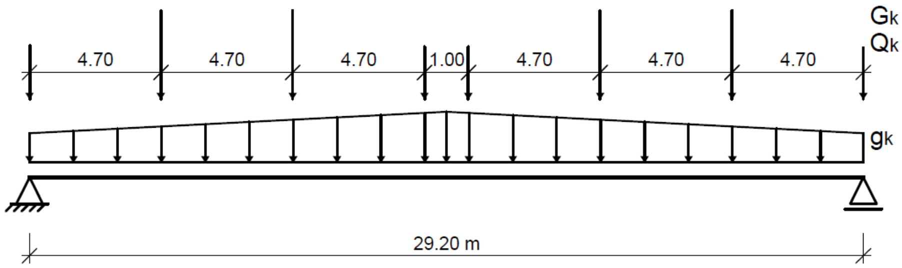
→ *The tension zone of the purlin is sufficient.*

Alternative 2 has a smaller cross-section than alternative 1 that is why it is chosen here.

Chosen	**C 50/60 | "T-Profile": h = 220 mm, b_o = 800 mm, b_u = 400 mm**
	St 1570/1770 | 50 $Ø_P$ = 12.5mm = 50 x 0.934 cm² = 46.70 cm²

7.2 SYSTEM

System



Cross-section See Chapter 7.1.2

Building materials **C 50/60 | St 1570/1770 | B 500A**

Exposure class XC1, WO

Reinforcing steel

$c_{nom,w} = c_{min} + \Delta c_{dev} = 10mm + 10mm = 20mm$

$c_{nom,s} = c_{min} + \Delta c_{dev} = 20mm + 10mm = 30mm$ → $c_{v,w,min} = 20mm$

Chosen $c_v = 20mm$

A_s = 4 $Ø_s$ 16 = 8.04 cm² **a_u = 4.0 cm**

Prestressing steel

c_{nom} = 2.5 x 12.5 mm + 10 mm = 42 mm

1. Layer:	A_p = 8 $Ø_p$ 12.5	a_p = 7.5 cm
2. Layer:	A_p = 8 $Ø_p$ 12.5	a_p = 11.7 cm
3. Layer:	A_p = 8 $Ø_p$ 12.5	a_p = 15.9 cm
4. Layer:	A_p = 8 $Ø_p$ 12.5	a_p = 20.1 cm
5. Layer:	A_p = 8 $Ø_p$ 12.5	a_p = 24.3 cm
6. Layer:	A_p = 8 $Ø_p$ 12.5	a_p = 28.5 cm
7. Layer:	A_p = 2 $Ø_p$ 12.5	a_p = 32.7 cm
∑	**A_p = 50 $Ø_p$ 12.5 = 46.7 cm²**	**a_p = 18.6 cm**

Characteristic loads

Uniformly Distributed Loads		g_k [kN/m]	q_k [kN/m]
Self-weight			-
Max. self-weight	$g_{k1,max}$ = 0.60 cm² x 25 kN/m³ =	15.0	
Min. self-weight	$g_{k1,min}$ = 0.56 cm² x 25 kN/m³ =	14.0	-

Concentrated Loads		G_k [kN]	Q_k [kN]
Purlin – *Reaction Force from P.02*		83.8	-
Roof Structure – *Reaction Force from P.02*	Min =	5.0	-
	Max =	37.8	-
Snow – *Reaction Force from P.02*	Min =	-	65.6
	Max =	-	183.7
Wind – *Reaction Force from P.02*	Min =	-	3.1
	Max =	-	8.3

7.3 INTERNAL FORCES

Characteristic (at x = 11.89)

M_{gk1} [kNm]	M_{gk2} [kNm]	M_{qk1} [kNm]	M_{qk2} [kNm]
1508.0	2790.0	3393.0	155.3

V_{gk1} [kN]	V_{gk2} [kN]	V_{qk1} [kN]	V_{qk2} [kN]
211.7	423.8	502.0	23.0

Design

ULS	SLS		
M_{Ed} [kNm]	M_{rare} [kNm]	M_{frequ} [kNm]	M_{perm} [kNm]
11,031.6	7,784.2	4,976.6	4,298.0

ULS	SLS		
V_{Ed} [kN]	V_{rare} [kN]	V_{frequ} [kN]	V_{perm} [kN]
1,631.6	1151.3	735.9	635.5

7.4 PRESTRESSING LOSS

Maximum prestressing force

(1) $P_{m0,\max} = 0.75 \cdot A_p \cdot f_{pk} = 0.75 \cdot 50 \cdot 0.934 \cdot 177 kN / cm^2 = 6199 kN$

(2) $P_{m0,\max} = 0.85 \cdot A_p \cdot f_{p0,1k} = 0.85 \cdot 50 \cdot 0.934 \cdot 150 kN / cm^2 = \underline{\underline{5954 kN}}$

→ $P_{m,t=0} = 4670 kN < P_{m0,\max} = 5954 kN$ ✓

Effective thickness $h_0 = \frac{2 \cdot A_c}{U} = \frac{2 \cdot 5818}{588} = 19.8cm$

Drying shrinkage $\varepsilon_{cd}(\infty) = k_h \cdot \varepsilon_{cd,0} = 0.853 \cdot 0.54‰ = 0.461‰$

Autogenous shrinkage: $\varepsilon_{ca}(\infty) = 2.5 \cdot (f_{ck} - 10) \cdot 10^{-6} = 2.5 \cdot (50 - 10) \cdot 10^{-6} = 0.100‰$

Total shrinkage $\varepsilon_{cs}(\infty) = \varepsilon_{cd}(\infty) + \varepsilon_{ca}(\infty) = 0.461‰ + 0.100‰ = 0.561‰$

Final creep ratio *for t_0 = 5 day:* $\varphi(\infty, t_0) = 1.89$

Prestressing loss due to relaxation

$$\sigma_{p,perm} = \frac{E_p}{E_{cm}} \cdot \left[\frac{M_{Ed,perm}}{I_c} \cdot z_p\right] = \frac{196}{33} \cdot \left[\frac{4.298}{0.31{,}841{,}471} \cdot 0.955\right] = 68.3MN/m^2$$

$$N_{p,t=0} = -P_{0,t=0} = -4670kN \ ,$$

$$M_{p,t=0} = N_{p,t=0} \cdot z_p = -4670 \cdot 0.955 = -4458kNm$$

$$\sigma_{p,t=0} = \frac{P_{0,t=0}}{A_p} + \frac{E_p}{E_{cm}} \cdot \left[\frac{N_{p,t=0}}{A_c} + \frac{M_{p,t=0}}{I_c} \cdot z_p\right] = 886.7MN/m^2$$

$$R_i / R_m = \frac{\sigma_{p,perm} + \sigma_{p,t=0}}{f_{pk}} = \frac{68.3 + 886.7}{1770} = 0.5395 \quad \rightarrow \kappa_p = 1.158\%$$

$$\Delta\sigma_{pr} = \kappa \cdot \left(\sigma_{p,perm} + \sigma_{p,t=0}\right) = 0.01158 \cdot (68.3 + 886.7) = 11.1MN/m^2$$

Total Prestressing Loss

$$\sigma_{cp,perm} = \frac{M_{Ed,perm}}{I_c} \cdot z_p = \frac{429800}{31{,}841{,}471} \cdot 95.5 = 12.9MN/m^2$$

$$\sigma_{cp,t=0} = \frac{N_{p,t=0}}{A_c} + \frac{M_{p,t=0}}{I_c} \cdot z_p = \frac{-4760}{5818} + \frac{-429800}{31{,}841{,}471} \cdot 95.5 = -21.4MN/m^2$$

$$\alpha_p = \frac{E_p}{E_{cm}} = 5.30$$

$$\Delta\sigma_{p,c+s+r} = \frac{\varepsilon_{cs}(t) \cdot E_p + 0.8 \cdot \Delta\sigma_{pr} + \alpha_p \cdot \varphi(\infty, t_0) \cdot (\sigma_{cp,perm} + \sigma_{cp,t=0})}{1 + \alpha_p \cdot \frac{A_p}{A_c} \cdot \left(1 + \frac{A_c}{I_c} \cdot z_p^2\right) \cdot \left[1 + 0.8 \cdot \varphi(\infty, t_0)\right]}$$

$$= -85.5 - 6.9 - 66.3 = -158.7MN/m^2$$

Shrinkage *- 85.5 MN/m² → 53.9%*

Relaxation *- 6.9 MN/m² → 4.3%*

Creep *- 66.3 MN/m² → 41.8%*

$$\Delta P_{c+s+r} = \Delta\sigma_{p,c+s+r} \cdot A_p = -158.7 \cdot 50 \cdot 0.934 \cdot 10^{-4} = -741.2KN$$

$$\frac{\Delta P_{c+s+r}}{P_{m,t=0}} = \frac{741.2}{4760} = 0.124 \quad \rightarrow \alpha_{loss} = 12.4\% < 15\% \ ✓$$

$$P_{m,t=\infty} = 4760 - 741 = \underline{\underline{3929kN}} \quad \rightarrow \underline{\underline{98kN / strand}}$$

7.5 ULTIMATE LIMIT STATE DESIGN

7.5.1 FLEXURE DESIGN

Compression Zone

$b_{eff} / b_w = 80/12 = 6.67 > 5.0$ → *Slim cross-section, it can be assumed that the compression zone stays within the flange of the beam*

Effective depth $d = 188.6cm \quad z_r = d - x_{so} = 97.6cm$

$$z_p = x_{su} - a_p = 95.5cm$$

$$M_{Edr} = M_{Ed} + P_{m,t=\infty} \cdot (z_r - z_p) = 11{,}032 + 3929 \cdot (0.976 - 0.955)$$

$$= 11{,}116kNm$$

$$\mu_{Edr} = \frac{M_{Edr}}{b_{eff} \cdot d^2 \cdot f_{cd}} = \frac{1111600kNcm}{80.0 \cdot 188.6^2 \cdot 2.83kN / cm^2} = 0.138 < \mu_{Ed,\max} = 0.40 \ ✓$$

→ *The compression zone is sufficient.*

Tension Zone

Using the design table from [3] following values for the height of the compression zone x and the lever arm of the internal forces z can be determined:

Lever arm z $\zeta = 0.923 \quad \rightarrow z = \zeta \cdot d = 0.923 \cdot 188.6 = 174.0cm$

Compression zone $\xi = 0.185 \quad \rightarrow x = \xi \cdot d = 0.185 \cdot 177 = 34.9cm$

Rebar strain $\varepsilon_s = \Delta\varepsilon_p = 15.47‰ > 2.17‰ \quad \rightarrow \sigma_{sd} = f_{yd} = 435MN / m^2$

Prestressing steel strain $\varepsilon_p^{(0)} = \frac{P_{m,t=\infty}}{A_p \cdot E_p} = \frac{3929kN}{50 \cdot 0.934cm^2 \cdot 19{,}600kN / cm^2} = 4.29‰$

Total strain $\varepsilon_p = \varepsilon_p^{(0)} + \Delta\varepsilon_p = 4.29‰ + 15.47‰ = 19.76‰$

$$f_{pd} = \frac{f_{p0,1,k}}{\gamma_p} = \frac{1500N / mm^2}{1.15} = 1304N / mm^2$$

Permitted strain $\varepsilon_{p0,1d} = \frac{f_{pd}}{E_p} = \frac{1304N / mm^2}{196{,}000N / mm^2} = 6.7‰$

$$\varepsilon_{p0,1d} = 6.7‰ < \varepsilon_p = 19.76‰ \quad \rightarrow \sigma_{pd} = f_{pd} = 1304N / mm^2$$

Required reinforcement $A_{s1,req} = \left(\frac{M_{Edr}}{z} - A_p \cdot \sigma_{pd}\right) \cdot \frac{1}{\sigma_{sd}} = \left(\frac{1{,}111{,}600}{174} - 46.7 \cdot 130.4\right) \cdot \frac{1}{43.5}$

$$= 6.80cm^2$$

Chosen **4 Øs 16 = 8.04 cm²** **lower reinforcement**

Precompressed Tension Zone

$N_{p,t=0} = -4670kN$ $M_{p,t=0} = -4458kNm$

Design normal force $N_{Ed} = \gamma_p \cdot N_{p,t=0} = 1.0 \cdot (-4670kN) = -4670kN$

Effective depth $d \approx 205 - 4 = 201cm$ → $z_{s2} = d - x_{su} = 201 - 114.1 = 89.9cm$

$M_{Ed} = \gamma_G \cdot \min M_G + \gamma_P \cdot M_{P,t=0} = 1.0 \cdot 1508.0 + 1.0 \cdot (-4458) = -2950Nm$

Design bending moment $M_{Ed,s2} = M_{Ed} - N_{Ed} \cdot z_{s2} = -2950 - 0.899 \cdot (-4670) = 7011kNm$

$$\mu_{Eds2} = \frac{M_{Eds2}}{b_{eff} \cdot d^2 \cdot f_{cd}} + \frac{N_{Ed}}{f_{yd}} = 0.077$$ → $\omega_2 = 0.0804$

Required reinforcement $$A_{s2,req} = \omega_2 \cdot \frac{b_{eff} \cdot d}{f_{yd} / f_{cd}} + \frac{N_{Ed}}{f_{yd}} = 0.0804 \cdot \frac{80 \cdot 201}{43.5 / 1.7} + \frac{-4760}{43.5} = -23.3 < 0$$

No reinforcement in the compression zone is required.

7.5.2 SHEAR DESIGN

Design location (Direct support)

x_1 = 1.40 m *(≈ end of dispersion length)*

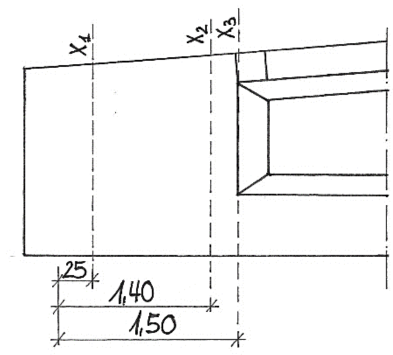
x_2 = a_i = 0.25 m *(face of support)*

x_3 = 1.50 m *(transition of web width)*

$V_{Ed,1}$ = 1374 kN

$V_{Ed,2}$ = 1396 kN

$V_{Ed,3}$ = 1372 kN



Shear Force Resistance

$a_r = 16.5cm$ → $d = 205 - 16.5 = 188.5cm$

$$k = 1 + \sqrt{\frac{20}{d}} = 1 + \sqrt{\frac{20}{188.5}} = 1.39 < 2.00$$

Longitudinal reinforcement $A_{sl} = A_s + A_p = 8.04 + 46.70 = 54.7cm^2$

Percentage of reinforcement $$\rho_1 = \frac{A_{sl}}{b_w \cdot d} = \frac{54.7}{40 \cdot 134} = 0.0102 \leq 0.02$$

Compressive stress $\sigma_{cp} = \frac{N_{Ed}}{A_c} = \frac{0.3929}{40 \cdot 149} \cdot 10 = 6.59 MN/m^2$

$< 0.2 \cdot f_{cd} = 0.2 \cdot 28.3 MN/m^2 = 5.66 MN/m^2$

$V_{Rd,c} = \left[0.10 \cdot 1.39 \cdot (100 \cdot 0.0102 \cdot 50)^{1/3} + 0.12 \cdot 6.59\right] \cdot 0.40 \cdot 1.34 \cdot 10^3$

$= 701 kN$

$d = 134 cm \geq 80 cm$ → $\nu_{\min} = 0.025 \cdot 1.39^{3/2} \cdot (50)^{1/2} = 0.290 MN/m^2$

$V_{Rd,c,\min} = \left[0.290 + 0.12 \cdot 6.59\right] \cdot 0.40 \cdot 1.34 \cdot 10^3 = 580 kN$

Shear force resistance $V_{Rd,c} = \underline{\underline{701 kN}} > V_{Rd,c,\min} = 580 kN$

$V_{Ed} = 1374 kN > V_{Rd,c} = 701 kN$ → *Shear force reinforcement is needed!*

Inclination of Concrete Compression Struts

Friction force $V_{Rd,cc} = 0.24 \cdot (50)^{1/3} \cdot \left(1 - 1.2 \cdot \frac{6.59}{28.3}\right) \cdot 0.4 \cdot 1.21 = 308 kN$

Inclination $1.0 \leq \cot\theta = \frac{1.2 + 1.4 \cdot \sigma_{cp} / f_{cd}}{1 - V_{Rd,cc} / V_{Ed}} = \frac{1.2 + 1.4 \cdot 6.59 / 28.3}{1 - 308 / 1374} = 1.97 \leq 3.0$

Required Shear Force Reinforcement

Vertical stirrups $\alpha = 90°$

Lever arm of the internal forces $z \approx 0.9 \cdot d = 0.9 \cdot 134 = 121 cm$

Required reinforcement $a_{sw,req} = \frac{1374 kN}{43.5 \cdot 1.21 \cdot (1.97 + 0) \cdot 1} = 13.3 cm^2/m$

Chosen **Ø 10 / 10 = 15.71 cm²/m** **stirrups**

Resistance of the Compression Struts

At x_2 = 0.25 m $V_{Rd,\max} = 0.75 \cdot 40 \cdot 121 \cdot 2.83 \cdot \frac{1.97}{1 + 1.97^2} = 4146 kN$

$V_{Ed,2} = 1396 kN < V_{Rd,\max} = 4146 kN$ → *The compression strut is ok.*

At x_3 = 1.50 m *Simplified assumption:* $b_w = 0.12 m$

$\cot\theta = 1.68$

$V_{Rd,\max} = 0.75 \cdot 12 \cdot 121 \cdot 2.83 \cdot \frac{1.68}{1 + 1.68^2} = 1355 kN$

$V_{Ed,3} / V_{Rd,\max} = 1372 kN / 1355 kN = 1.01 \approx 1.00$

An exceedance of 1% is acceptable given that the real web width at 1.50 m is bigger than the assumed 12 cm.

Minimum Shear Force Reinforcement

$\min a_{sw} = \rho \cdot b_w \cdot \sin\alpha$ and $\rho = 0.16 \cdot f_{ctm} / f_{yk} = 0.16 \cdot 4.1 / 500 = 1.31‰$

$\min a_{sw} = 0.00131 \cdot 0.40 \cdot \sin 90° = 5.24 \cdot 10^{-4} m^2 / m = \underline{\underline{5.24 cm^2 / m}}$

7.5.3 TILTING

According to DIN EN 1992-1-1, 5.9 a simplified proof of the safety against tilting can be conducted with following formula:

$$b_{req} \geq \sqrt[4]{\left(\frac{l_0}{50}\right)^3 \cdot h} \quad \text{and} \quad \frac{h}{b} \leq 5.0$$

$$b_{req} \geq \sqrt[4]{\left(\frac{29.2}{50}\right)^3 \cdot 2.20} = 0.81m \cong b_{prov} = 0.80m \quad \text{and} \quad \frac{h}{b} = \frac{220}{80} = 2.75 < 5.0 \ \checkmark$$

This means that the beam is safe against tilting and no further analysis is necessary. The simplified proof is sufficient.

7.6 SERVICEABILITY LIMIT STATE DESIGN

7.6.1 STRESS LIMITS

Second or first order?

$\min P_{k,t} = r_{\inf} \cdot P_{m,t=\infty} = 0.95 \cdot 3929 kN = 3732 kN$

$N_{Ed,rare} = -\min P_{k,t} = -3732 kN$

$M_{Ed,rare} = M_{Ed,rare}^{G+Q} - \min P_{k,t} \cdot z_p = 7784 - 3732 \cdot 0.0.955 = 4221 kNm$

$\sigma_{c,rare} = \frac{-3732}{5818} + \frac{422100}{31841471} \cdot 114 \cdot 10 = 8.70 MN / m^2$

$\sigma_{c,rare} = 8.7 MN / m^2 > f_{ctm} = 4.1 MN / m^2$

→ *Second order must be considered*

Concrete Compression Stresses

$\frac{2 \cdot A_c \cdot z_p}{z \cdot x \cdot b_{eff}} = \frac{2 \cdot 5818 \cdot 95.5}{174 \cdot 34.9 \cdot 80} = 2.29 > 1.0$ → $\min P_{k,t} = 3732 kN$

$N_{Ed,perm} = -3732 kN$

$M_{Ed,perm} = M_{Ed,perm}^{G+Q} - \min P_{k,t} \cdot z_p = 4298 - 3732 \cdot 0.955 = 735 kNm$

$\sigma_{c,perm} = \frac{N_{Ed,perm}}{A_c} + \frac{2 M_{Ed,perm}}{z \cdot x \cdot b_{eff}} \leq 0.45 f_{ck}$

$\sigma_{c,perm} = \frac{-3732}{5818} + \frac{2 \cdot 73500}{174 \cdot 34.9 \cdot 0.80} \cdot 10 = -9.44 MN / m^2$

$$\left|\sigma_{c,perm}\right| = 9.44MN/m^2 < 0.45f_{ck} = 0.45 \cdot 50 = 22.5MN/m^2 \checkmark$$

Stresses in the Reinforcement

$$N_{Ed,rare} = -3732kN$$

$$M_{Ed,rare} = 7784 - 3732 \cdot (0.976 - 0.955) = 7864kNm$$

$$\sigma_{s,rare} = \left[\frac{M_{Ed,rare}}{z} + N_{Ed,rare}\right] \cdot \frac{1}{A_{s1} + A_p} \le 0.8f_{yk}$$

$$\sigma_{s,rare} = \left[\frac{786400}{174} - 3732\right] \cdot \frac{1}{54.7} \cdot 10 = 143.7MN/m^2$$

$$\le 0.8 \cdot 500 = 400MN/m^2 \checkmark$$

Stresses in the Prestressing Steel

$$\min P_{k,t} = P_{m,t=\infty} = 3929kN \qquad N_{Ed,perm} = -\min P_{k,t} = -3929kN$$

$$M_{Ed,perm} = M_{Ed,perm}^{G+Q} - \min P_{k,t} \cdot z_p = 4298 - 3929 \cdot 0.955 = 4382kNm$$

$$\sigma_{p,perm} = \frac{P_{m,t=\infty}}{A_p}\left[\frac{M_{Ed,perm}}{z} + N_{Ed,perm}\right] \cdot \frac{1}{A_{s1} + A_p} \le 0.65f_{pk}$$

$$\sigma_{p,perm} = \frac{3929}{46.7}\left[\frac{438200}{174} - 3929\right] \cdot \frac{1}{54.7} \cdot 10 = 583.6MN/m^2$$

$$\le 0.65 \cdot 1770 = 1150MN/m^2 \checkmark$$

7.6.2 MINIMUM REINFORCEMENT AGAINST CRACKING

Reinforcement needed?

$$\sigma_{c,rare} = 8.70MN/m^2 > 0$$

→ *a minimum reinforcement to prevent cracking is necessary*

Area of the Tension Zone

$$\sigma_l = f_{ctm} = \sigma_l^N + \sigma_l^M = 4.1MN/m^2 = \frac{-3.732}{0.5818} + \sigma_l^M$$

$$\sigma_l^M = 10.51MN/m^2 \quad \rightarrow \quad \sigma_u^M = -10.51MN/m^2$$

$$\sigma_u = \sigma_u^N + \sigma_u^M = \frac{-3.732}{0.5818} - 10.51 = -16.9MN/m^2$$

Area of tension zone $h_t = \frac{4.1}{4.1+16.9} \cdot 205cm = 40cm$ → $A_{ct} = 40 \cdot 19 = \underline{\underline{480cm^2}}$

Minimum Required Cracking Reinforcement

Tensile strength $f_{ct,eff} = f_{ctm} \ge 2.9MN/m^2$ → $f_{ct,eff} = 4.1MN/m^2 \ge \underline{\underline{2.9MN/m^2}}$

$h = 2.05m$ → $h^* = 1.0m$

$for\ h > 80cm$ → $k = 0.50$

$k_1 = 1.5$

$$k_c = 0.4 \cdot \left[1 - \frac{\sigma_c}{k_1 \cdot h / h^* \cdot f_{ct,eff}}\right] = 0.4 \cdot \left[1 - \frac{3.732 / 0.5818}{1.5 \cdot 2.05 / 1.0 \cdot 0.41}\right] = 0.197 \leq 1.0$$

Crack width limit $w_k = 0.2\ mm$ *(according to DIN EN 1992-1-1, Tab. NA.7.1.)*

Limit diameter $\phi_s^* = \phi_s \cdot \frac{2.9}{f_{ct,eff}} = 20mm \cdot \frac{2.9}{4.1} = 14.15mm$ [1]

Reinforcement stresses $\sigma_s = 223MN/m^2$ *(according to DIN EN 1992-1-1, Tab. NA.7.2.)*

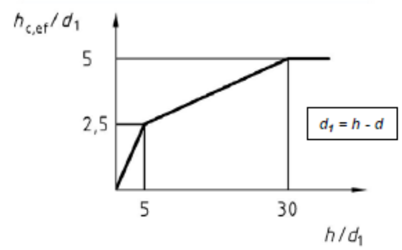
Required Reinforcement $A_{s,\min,req} = k_c \cdot k \cdot f_{ct,eff} \cdot \frac{A_{ct}}{\sigma_s} = 0.197 \cdot 0.5 \cdot 4.1 \cdot \frac{480}{223} = 0.87cm^2$

→ *The provided reinforcement is sufficient!*

7.6.3 LIMITATION OF THE CRACK WIDTH

Percentage of Reinforcement

$h / d_1 = 51.25$ → $h_{c,eff} / d_1 = 5.0$



$h_{c,eff} = 5.0 \cdot 4.0cm = 20cm$

$A_{c,eff} = 12.0 \cdot 20 = 240cm^2$

$\xi = 0.6$ $\varnothing_s = 20mm$ [1] $\varnothing_p = 7.2mm$

$$\xi_1 = \sqrt{\xi \cdot \phi_s / \phi_p} = \sqrt{0.6 \cdot 20/7.2} = 1.29$$

$$\rho_{tot} = (A_s + A_p) / A_{c,eff} = 63.3 / 240 = 0.2600$$

$$\rho_{p,eff} = (A_s + \xi_1^2 A_p) / A_{c,eff} = (15.7 + 1.29^2 \cdot 47.6) / 240 = 0.3897$$

Reinforcement Stresses

$$\sigma_s = \left[\frac{M_{Edr}}{z} + N_{Ed}\right] \cdot \frac{1}{A_s + A_p} + 0.4 \cdot f_{ct,eff} \cdot \left[\frac{1}{\rho_{p,eff}} - \frac{1}{\rho_{tot}}\right]$$

$$\sigma_s = \left[\frac{509300}{174} - \right] \cdot \frac{1}{63.3} + 0.4 \cdot 4.1 \cdot \left[\frac{1}{0.26} - \frac{1}{0.39}\right] = -135.3kN/cm^2$$

→ $\sigma_s = -135.3MN/m^2 < 0$

The stress is negative which means the cross section is under compression so there won't be any cracking.

[1] The diameter of the reinforcement has to be changed to 20mm due to the required minimum reinforcement (see chapter 6.8)

7.7 DESIGN AND REINFORCEMENT

7.7.1 ANCHORAGE OF TENDONS

Transmission of the Stresses

Transmission length $l_{pt} = \alpha_1 \cdot \alpha_2 \cdot \varnothing_p \cdot \sigma_{pm0} / f_{bpt}$

$\alpha_1 = 1.25$ *no wire stripping*

$\alpha_2 = 0.19$ *for strands*

$\varnothing_p = 12.5mm$

$f_{bpt} = 4.6MN / m^2$ *for C50/60 and good bond*

$\sigma_{pm0} = 1000MN / m^2$

$l_{pt} = 1.25 \cdot 0.19 \cdot 1.25 \cdot 1000 / 4.6 = 64.5cm$

Modified transmission length $l_{ptd} = 0.8 \cdot l_{pt} = 0.8 \cdot 64.5 = 51.6cm$

Dispersion length $l_{disp} = \sqrt{l_{ptd}^2 + d_p^2} = \sqrt{51.6^2 + 130.4^2} = 140cm$ *d_p=130.4cm at x = 1.40m*

Splitting tensile force $F_{td} = P_{m,0} - \left|\sigma_{c,m}\right| \cdot A_{c,u}$

$N_{Ed}^P = -P_{m,t=0} = -4760kN$ $M_{Ed}^P = -4760 \cdot (0.745 - 0.186) = -2611kNm$

$A_{c,u} = a_p \cdot b = (32.7 - 1.25 / 2) \cdot 40 = 33.325 \cdot 40 = 1333cm^2$

$z_{c,u} = z_{s,u} - a_p / 2 = 74.5 - 33.33 / 2 = 57.8cm$

$$\sigma_{c,m}(x = l_{disp}) = \frac{N_{Ed}^P}{A_c} + \frac{M_{Ed}^P}{I_c} \cdot z_{c,u} = \frac{-4670}{149 \cdot 40} + \frac{-261100}{40 \cdot 149^3 / 12} \cdot 57.8 \cdot 10$$

$$= -21.5MN / m^2$$

$F_{td} = 4670 - 2.15 \cdot 1333 = \underline{\underline{1804kN}}$

Required reinforcement $A_{sp,req} = \frac{\gamma_{p,sp}}{3} \cdot \frac{F_{td}}{f_{yd}} = \frac{1.35}{3} \cdot \frac{1804}{43.5} = 18.7cm^2$

The reinforcement is to be distributed over the length of

$0.75 \cdot l_{disp} = 0.75 \cdot 140 = 105cm$

$a_{sp,req} = 18.7 / 1.05 = \underline{\underline{17.8cm^2 / m}}$

Chosen **Ø 10 / 7.5 = 20.9 cm²/m** **stirrups**

Tensile forces at the edge $Z_{Ed}^P = P_{m0} \cdot \left[\frac{e}{h} - \frac{1}{6}\right] = 4670 \cdot \left[\frac{55.9}{149} - \frac{1}{6}\right] = 974kN$

Required reinforcement $A_{sz,req} = Z_{Ed} \,/\, f_{yd} = 974 \,/\, 43.5 = \underline{\underline{22.4 cm^2}}$

Chosen **10 Ø 12 = 22.6 cm²** **reinforcement loops**

Anchorage of Tendons in the Ultimate Limit State

Anchorage length $l_{bpd} = l_{pt2} + \alpha_2 \cdot \varnothing_P \cdot \dfrac{\sigma_{pd} - \sigma_{pm,t=\infty}}{f_{bpd}}$

$l_{pt2} = 1.2 \cdot l_{pt} = 1.2 \cdot 64.5 = 77.4 cm$ $f_{bpd} = 4.6 MN \,/\, m^2$

$\sigma_{pd} = f_{p,01k} \,/\, \gamma_p = 1500 \,/\, 1.15 = 1304 MN \,/\, m^2$

$\sigma_{pm,t=\infty} = P_{m,t=\infty} \,/\, A_p = 3929 \,/\, 46.7 = 84.1 kN \,/\, cm^2$

$l_{bpd} = 77.4 + 0.19 \cdot 1.25 \cdot \dfrac{1304 - 841}{4.6} = \underline{\underline{101 cm}}$

Internal forces at x $N_{P,t=\infty} = -P_{m,t=\infty} = -3929 kN$ $N_{P,t=\infty} = -P_{m,t=\infty} = -3929 kN$

$x = l_{bpd} - u = 101 + 25 = 76 cm$

$M_{Ed}(x = 76) = 1064 kNm$

$M_{Ed} = M_{Ed}(x) + M_{Ed}^{P} = 1064 - 2118 = -1054 kNm$

Concrete stresses $\sigma_{c,u} = \dfrac{N_{Ed}}{A_c} + \dfrac{M_{Ed}}{I_c} \cdot z_u = \dfrac{-3929}{145 \cdot 40} + \dfrac{-105400}{40 \cdot 145^3 / 12} \cdot 72.5 = -0.687 kN \,/\, m^2$

$\sigma_{c,u} = -6.9 MN \,/\, m^2 << 0.7 \cdot f_{ctm} = 0.7 \cdot 4.1 = 2.87 MN \,/\, m^2$ ✓

7.7.2 MINIMUM REINFORCEMENT TO ENSURE ROBUSTNESS

$M_{cr} = f_{ctm} \cdot W_{cu}$ $W_{cu} = 279{,}176 cm^3$

$M_{cr} = 4.1 \cdot 279.176 \cdot 10^{-3} = 1145 kNm$

$A_{s,\min} = \dfrac{1}{f_{yk}} \cdot \dfrac{M_{cr}}{z} = \dfrac{1}{50.0} \cdot \dfrac{114500}{174} = 15.2 cm^2 > A_{s,prov} = 8.04 cm^2$ ↯

The provided reinforcement is not enough to cover the minimum reinforcement which is needed to ensure the robustness of the beam, therefore a new reinforcement is chosen.

Chosen **5 Ø 20 = 15.7 cm²** **lower longitudinal reinforcement**

7.7.3 STIRRUPS AT THE RIDGE

Flexure moment $M_{Ed} = 11{,}533 kNm$

Lever arm of the forces $z = 0.9 \cdot d = 0.9 \cdot (220 - 16.5) = 183 cm$

Required Reinforcement $A_{s,req} = \dfrac{2 \cdot M_{Ed} \cdot \sin\alpha}{z \cdot f_{yd}} = \dfrac{2 \cdot 1153300 \cdot \sin 3.1°}{183 \cdot 43.5} = \underline{\underline{15{,}7cm^2}}$

Chosen **7 Ø 12 = 15.84 cm²** **stirrups**

7.7.4 DESIGN OF THE SUPPORT

Width of the Joints

The width of the joints between a concrete truss and a column can be estimated according to [4] and [5]. The width of the joints is the result of the sum of the different possible dimensional deviations during the process of the production and assembly of the beams.

$\delta_{comb} = \delta_{\max} + \sqrt{\sum \delta_i^2}$ with

δ_{comb}	Total allowed dimensional deviation
$\delta_{\max}$	Maximum dimensional deviation
δ_i	Other possible dimensional deviations.

The different dimensional deviations can be taken from DIN 18202 and DIN 18203.

Here it is:

$$\delta_{comb} = \delta_{\max} + \sqrt{\sum \delta_i^2} = \frac{1}{2} \cdot (30 + \sqrt{30^2 + 25^2 + 25^2}) = 38.2mm$$

Chosen $\delta = 40mm$

Dimensions of the Bearing

The width of the elastomeric bearing is chosen to be b = 200 mm. The maximum length of the bearing can be calculated as per DIN EN 1992-1-1, 10.9.5.2 with following formula:

$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2}$ with

a_1	Length of the elastomeric bearing, $a_1 > \min a_1$ (DIN EN 1992-1-1, Tab. 10.2)
a_2	Distance of the bearing to the edge of the support
a_3	Distance of the bearing to the edge of the beam
Δa	Dimensional deviations.

Here:

Assumption $a_1 \approx 500mm$, $b_1 = 350mm$ → $\sigma_{Ed} = V_{Ed} / a_1 \cdot b_1 = 1632kN / 50 \cdot 35 = 0.93kN / cm^2$

$\min a_1 = 70mm$ for $0.15 < \sigma_{Ed} / f_{cd} = 0.93 / 2.83 = 0.33 < 0.40$ (DIN EN 1992-1-1, Tab.10.2)

$a_2 = c_{nom} + d_s / 2 + c_{nom,c} / 2 = 20 + 10 / 2 + 20 / 2 = 35mm > \min a_2 = 15mm$ (DIN EN 1992-1-1, Tab.10.3)

$a_3 = c_{nom} + d_s / 2 + c_{nom,b} / 2 = 20 + 12 / 2 + 20 / 2 = 36mm > \min a_3 = 15mm$ (DIN EN 1992-1-1, Tab.10.4)

$\Delta a_2 = l / 1200 = 29500 / 1200 = 24.6mm$ (DIN EN 1992-1-1, Tab.10.5)

$\Delta a_3 = l / 2500 = 29500 / 2500 = 11.8mm$

→ $a_{req} = 500 + 35 + 36 + \sqrt{24.6^2 + 11.8^2} = 598mm < a_{prov} = 650 - 40 = \underline{\underline{610mm}}$ ✓

Chosen ***a_1 = 500 mm / b_1 = 350 mm*** ***size of the bearing***

Elastomeric Bearing

The elastomeric bearing that is used here is by the company *Calenberg Ingenieure* and is chosen in accordance with the technical information and dimensioning tables of the bearings [6].

Size of the bearing l x b x t = 500 x 350 x 20 mm

Bearing compression $\sigma_{m,k} = \frac{V_{Ed}}{a_1 \cdot b_1} = \frac{1632}{50 \cdot 35} = 0.93kN / cm^2 = 9.3MN / m^2$

Form factor $S = \frac{100}{20} = 5$

Chosen **'Compactlager S65'**

7.7.5 REINFORCEMENT DRAWING

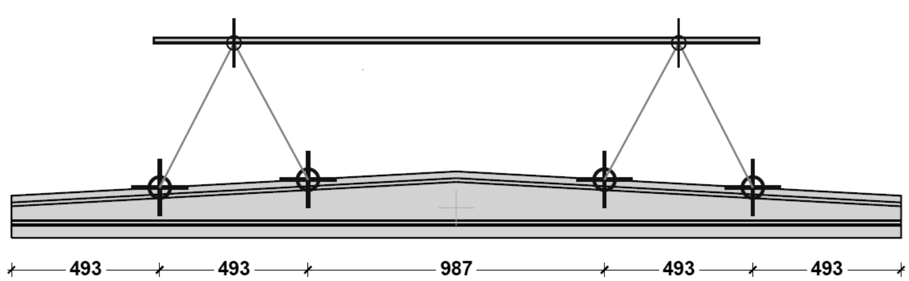
See Appendix A.3, page 189.

7.8 TRANSPORT AND ASSEMBLY

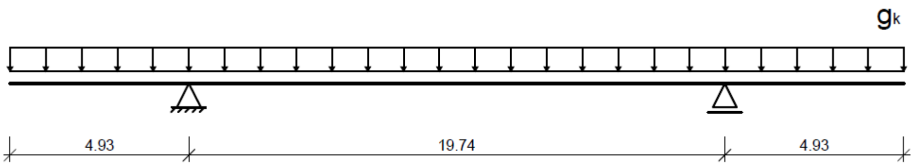
7.8.1 TRANSPORT AND LIFTING ON THE CONSTRUCTION SITE

The beam will be suspended with the help of two anchors on each side above the center of gravity at around 1/6 of the length of the beam. Which might cause tension over the anchors and might require upper reinforcement.

In the following the static system during transportation and assembly can be seen.



Simplified static system:



Lifting out of the Formwork

Lifting loads	*self-weight*	$g_k = 14.5\ kN/m$
Height of the beam at 4.93 m	$h = 1.67\ m$	$x_{so} = 73.1\ cm$

Bending Moment

$$M_{Ed}^{G} = 1.15 \cdot (-14.5) \cdot 4.93^2 / 2 = -203 kNm$$

$$M_{Eds} = M_{Ed}^{G} - N_{Ed} \cdot z_{s1} = 203 - (-4760) \cdot (0.731 - 0.04) = 3492 kNm$$

Time of first lifting approximately after 5 days (t = 5d)

Compressive strength at time t

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \quad \rightarrow \quad \beta_{cc}(t = 5d) = e^{s\left(1-\sqrt{28/t}\right)} = e^{0.2\left(1-\sqrt{28/5}\right)} = 0.76$$

$$f_{cm}(t = 5d) = 0.76 \cdot 58 = 44.1 N/mm^2$$

$$f_{ck}(t = 5d) = f_{cm}(t) - 8 = 44.1 - 8 = 36.1 N/mm^2$$

$$f_{cd}(t = 5d) = 0.85 \cdot f_{ck}(t) / 1.5 = 0.85 \cdot 36.1 / 1.5 = \underline{\underline{17.9 N/mm^2}}$$

Required Reinforcement

$$\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{349200}{40 \cdot 163^2 \cdot 1.79} = 0.18 \quad \rightarrow \omega = 0.2007$$

$$A_{s2,req} = (\omega \cdot b \cdot d \cdot f_{cd} + N_{Ed}) / \sigma_{sd}$$

$$= (0.2007 \cdot 40 \cdot 163 \cdot 1.79 - 4760) / 43.5 = \underline{\underline{-55.6 cm^2}} < 0$$

No additional upper reinforcement is required!

7.8.2 TILTING

According to DIN EN 1992-1-1, 5.9 a simplified proof of the safety against tilting can be conducted with following formula:

$$b_{req} \geq \sqrt[4]{\left(\frac{l_0}{70}\right)^3 \cdot h} \quad \text{and} \quad \frac{h}{b} \leq 5.0$$

for tilting during transport and assembly of the beam.

$$b_{req} \geq \sqrt[4]{\left(\frac{9.87}{70}\right)^3 \cdot 2.2} = 0.28m < b_{prov} = 0.80m \quad \text{and} \quad \frac{h}{b} = \frac{220}{80} = 2.75 < 5.0 \ \checkmark$$

This means that the beam is safe against tilting during transport and assembly and no further analysis is necessary. The simplified proof is sufficient.

7.8.3 TRANSPORT ANCHORS

The transport anchors are designed in accordance with the VDI/BV-BS-Richtlinie 6205. For this project the anchors of the company *HALFEN* are chosen (see Chapter 6.8.3).

Due to the length and the weight of the concrete truss four anchors are needed and a lifting beam is used for both lifting out of the formwork as well as transport and lifting on site.

Maximum Lifting Load on the Anchors

Self-weight $F_G = 14.5 \cdot 29.6 = 429kN$

Formwork adhesion $F_{adh} = q_{adh} \cdot A_f$ → no adhesion here because of hinged formwork!

Dynamic factor $\psi_{dyn} = 1.3$

Factor for inclination $z = 1.16 \quad for \ \beta = 30°$

Lifting load $F_z = \frac{(F_G + F_{adh}) \cdot \psi_{dyn} \cdot z}{n} = \frac{429 \cdot 1.3 \cdot 1.16}{4} = 162kN$

Chosen **HALFEN DEHA Double-headed lifting anchor 6000-32,0-0700D WB**

Load capacity $F_{z,Rd} = 189kN > F_{z,Ed} = 162kN$ ✓ (see Table 17)

Table 17 - Sizing Table for 'HALFEN DEHA Lifting Anchor Systems'

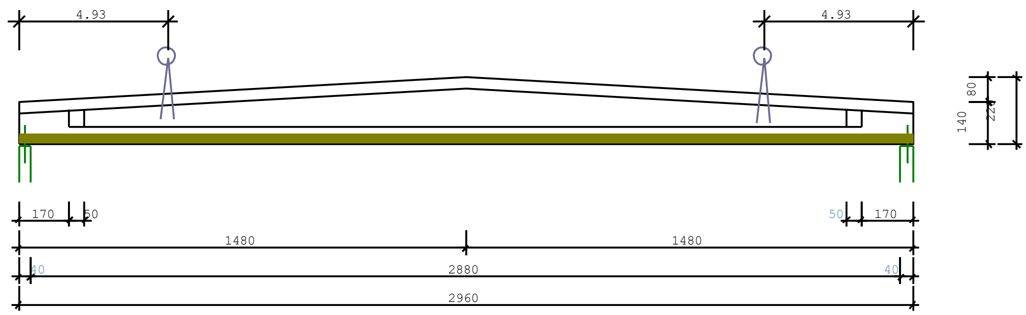
Load capacities for axial pull and diagonal pull up to 60° [ß]					
Load class	Article number	Min. web thickness 2 × e_r [mm]	Axial spacing of anchors e_z [mm]	**Axial pull** and **diagonal pull** up to 60° [β] Load capacity [kN] concrete strength f_{ci} 45 N/mm²	55 N/mm²
10,0	6000-10,0-0340D	120	≥ 1360	88.0	98.0
		140		100.0	100.0
15,0	6000-15,0-0400D	120	≥ 1600	130.0	145.0
		140		150.0	150.0
20,0	6000-20,0-0500D	120	≥ 2000	136.0	151.0
		140		173.0	192.0
		160		197.0	200.0
32,0	6000-32,0-0700D	120	≥ 2800	$\boxed{189.0}$	210.0
		140		220.0	245.0
		160		251.0	280.0
		180		282.0	315.0
f_{ci} = concrete cube strength at time of lifting					

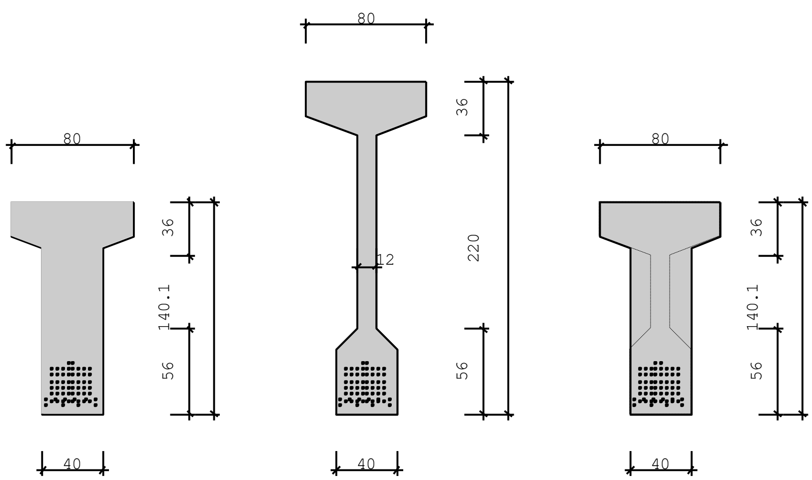
7.9 SOFTWARE CALCULATIONS

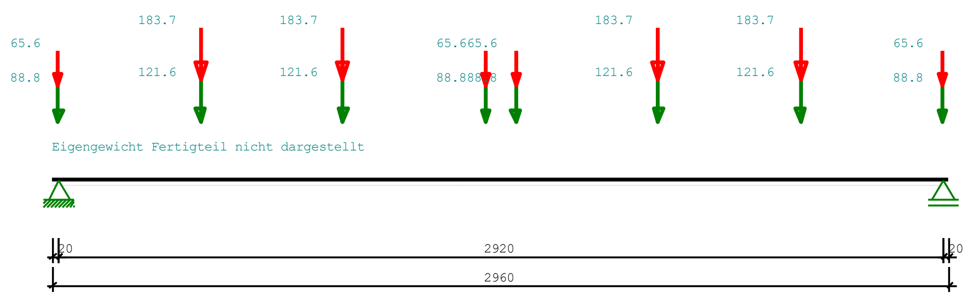
Demo Frilo Nemetschek

Position: P.03 – Prestressed Concrete Purlin

Spannbettbinder B8 01/2018 (Frilo R-2018-1/P12)







Advices:

No continuous reinforcement to 10 cm below UE found!

System:

Double-pitch roof

Material:

Prestressing steel

SpSt 1500/1770 Strand 7 wires

Reinforcing steel:

Longitudinal	Stirrup
B 500 B	B 500 B

Concrete:

Precast
C 50/60

Loads:

Self weight

Total	G	=	425.4 kN
Volume	V	=	17.01 m^3
Surf.	A	=	137.19 m^2

Live loads

Units: Single load[kN] Single moment[kNm] line load[kN/m]

span	type	gle	qle	Dist. a [m]	gri	qri	Length [m]	Fact	Act.	Sim.	Pos.
1	2	88.80	65.60	0.00				1.00	10	0	
1	2	88.80	65.60	14.10				1.00	10	0	
1	2	88.80	65.60	15.10				1.00	10	0	
1	2	88.80	65.60	29.20				1.00	10	0	
1	2	121.60	183.70	4.70				1.00	10	0	
1	2	121.60	183.70	9.40				1.00	10	0	
1	2	121.60	183.70	19.80				1.00	10	0	
1	2	121.60	183.70	24.50				1.00	10	0	

Load types: 1 = uniformly distr., 2 = single load at a, 3 = single moment at a
4 = trapezoidal load from a, 5 = triangle load over L

Actions:

Act.	γ_Q	ψ_0	ψ_1	ψ_2	Dep.	Cat.	Description
10	1.50	0.50	0.20	0.00	0	S	Schnee bis NN +1000m

Tendons:

Dist(LE) > 5.4 cm   axis horizontal > 3.7 cm   vertical > 3.7 cm

lay. No.	number	area A_p [cm²]	Dist.LE Y_p [cm]	Prestressing $\sigma_p^{(0)}$ [N/mm²]	Isolations Count	Isolations to x1 [m]	Isolations from x2 [m]	Type ||
1	8	7.47	7.5	1000	0			LE
2	8	7.47	11.7	1000	0			LE
3	8	7.47	15.9	1000	0			LE
4	8	7.47	20.1	1000	0			LE
5	8	7.47	24.3	1000	0			LE
6	8	7.47	28.5	1000	0			LE
7	2	1.87	32.7	1000	0			LE

x1 and x2 with respect to the left beginning from joint
LE= parallel lower edge, UE= parallel upper edge
The calculation of the losses due to creep, shrinkage and relaxation following the method from Abelein

RESULTS (summary)

Reaction forces (t = infinitely):

Units: all [kN] G:perm., Q:variable. ,V: Sum

Support point	G (char. value)	min Q (char. value)	max Q (char. value)	min V (ULS(PT))	max V (ULS(PT))
A (left)	633.57	0.00	498.60	633.57	1603.23
B (right)	633.57	0.00	433.00	633.57	1603.23

max. bending moment in erection state(char. value):

MF = = 8002.46 kNm   at x = = 14.80 m

Required shear reinforcement:

Column A: asw = 14.61 cm²/m
Column B: asw = 14.61 cm²/m

Bursting reinforcement

left Laying length = 1.13 m
      from x = 0.00 m   As = 21.0 cm²
right Laying length = 1.13 m
      from x = 29.60 m   As = 21.0 cm²

Check of anchorage

left: Tensile force resistance in anchoring area   Util = 0.51
right: Tensile force resistance in anchoring area   Util = 0.51

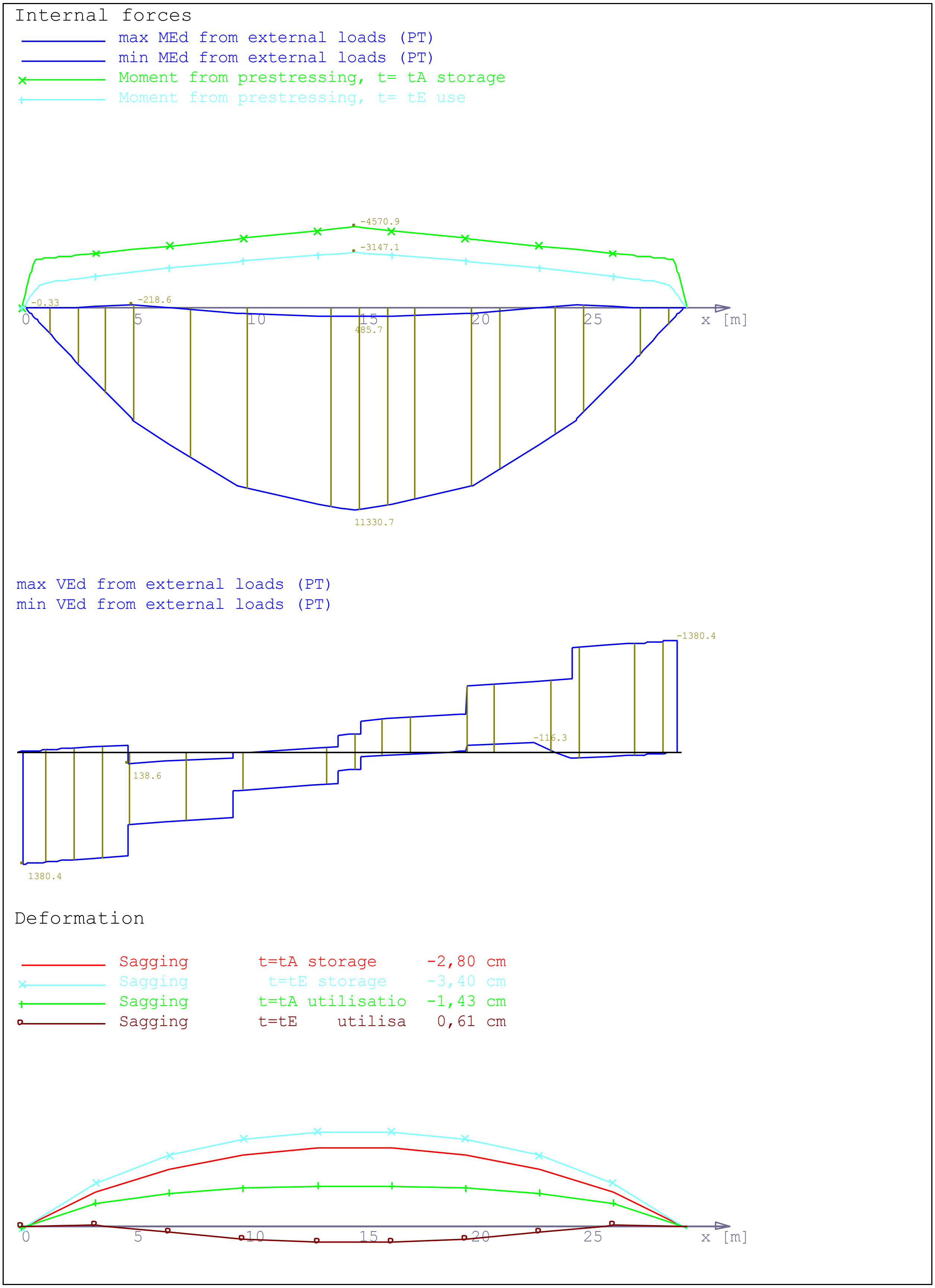
```
Overview crit. sections

    Selected basic grid:     10 Sections

Checkvalue                              Extrem        Utilisation  x [m]
---------------------------------------------------------------------------
Flexural capacity bottom         η =    1.15                 0.87   9.60
Flexural capacity  top           η =    ****                 ****   0.50
Resisting tens force  bot        η =    1.15                 0.87   9.56
Resisting tens force  top        η =    ****                 ****   0.50
Prc.:Compr.stress  Cc           σc =  -20.54 N/mm²           0.68   4.93
Tension prestress. steel     σp,Qc =   936.2 N/mm²           0.81   1.70
Stress in prestress.steel    σp,Cc =  1003.9 N/mm²           0.74   9.60
Stress in rebars                σs =    35.5 N/mm²           0.09   9.60
Crack MinAs+AsDuc bottom     AsMin =    13.5 cm²             0.34  14.80
Crack MinAs+AsDuc top        AsMin =    ---- cm²             ----   ----
Crack width     bottom          wk =    0.00 mm              0.01   9.60
Crack width      top            wk =    **** mm              ****   0.33
Sagging          top            fo =    -3.4 cm              0.29  16.44
Sagging         bottom          fu =     0.6 cm              0.05  13.16
Incr.-deflection(Util)        |df|=     2.0 cm              0.35  13.16
Prc.:Shear reinf (web)         asw =   14.61 cm²/m           1.00   2.20
Concrete strut capacity          η =    1.13                 0.89   2.20
---------------------------------------------------------------------------
---- Check not required
**** Check not fulfilled
Prc.:Precast member        Add.: in-situ supplement
IS  : Installed state       SC : State of construction
AsDuk:Ductility reinforcement
Min. reinforcement width of crack not required  (user defined)
---------------------------------------------------------------------------

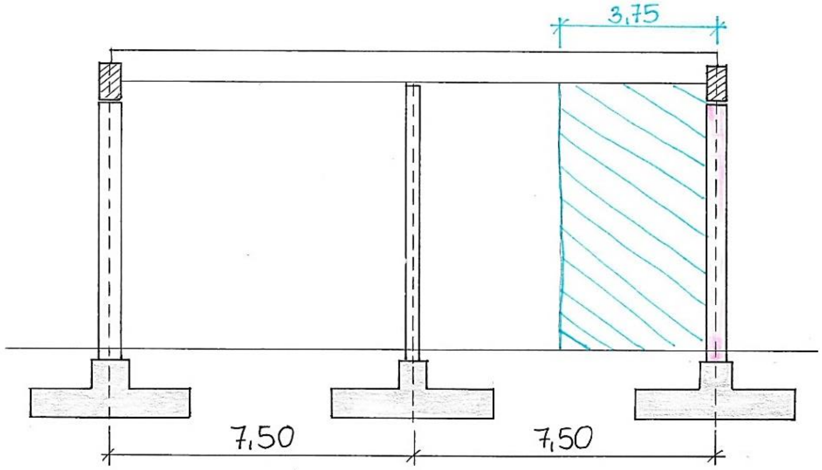
Linear creep limit, informative:            Extrem       Utilisation x [m]
---------------------------------------------------------------------------
Prc.:lin. creep t0(Sto)               σc = -30.13 N/mm²        2.20 28.96
Prc.:Compression quasi-permanent  Lc  σc = -14.35 N/mm²        0.64 27.20
---------------------------------------------------------------------------

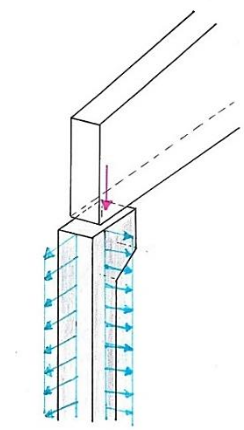
Tensile stress state I, informative:        Extrem       Utilisation x [m]
---------------------------------------------------------------------------
Prc.:Tens.stress (IS)                 σt =  10.80 N/mm²        2.65  9.60
Prc.:Tens.stress (SC)                 σt =   6.01 N/mm²        2.38 28.96
---------------------------------------------------------------------------
```


Internal forces
max MEd from external loads (PT)
min MEd from external loads (PT)
Moment from prestressing, t= tA storage
Moment from prestressing, t= tE use
-4570.9
-3147.1
-0.33
-218.6
485.7
11330.7
0
5
10
15
20
25
x [m]
max VEd from external loads (PT)
min VEd from external loads (PT)
-1380.4
-116.3
138.6
1380.4
Deformation
Sagging t=tA storage -2,80 cm
Sagging t=tE storage -3,40 cm
Sagging t=tA utilisatio -1,43 cm
Sagging t=tE utilisa 0,61 cm
0
5
10
15
20
25
x [m]

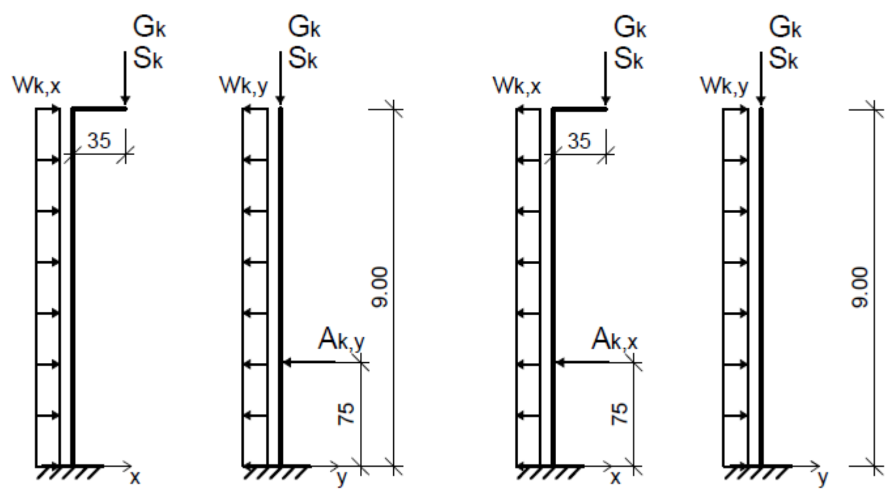
8 POS. C.04 – PRECAST CONCRETE COLUMN WITH BIAXIAL WIND LOADS

8.1 SYSTEM





System | *<u>Load Case I</u>* | *<u>Load Case II</u>*



Cross-section b/h = 50/50cm

Building materials **C 35/45 | B 500A**

Exposure class XC1, WO

Reinforcing Steel $c_{nom,w} = c_{min} + \Delta c_{dev} = 10mm + 10mm = 20mm$

$c_{nom,s} = c_{min} + \Delta c_{dev} = 25mm + 10mm = 35mm$ → $c_{v,w,min} = 25mm$

Chosen $c_v = 25mm$

Characteristic Loads

Vertical Loads	**G_k [kN]**	**Q_k [kN]**
Self-weight G_{k1} = 0.5 x 0.5 x 9.0 x 25 kN/m³ =	56.3	-
Reaction Force from *POS. P.03*		
G_{k2} = R_{gk1} + ½ x R_{gk2} = 211.7 + ½ x 423.8 =	423.6	-
S_{k1} = ½ x (R_{qk1} + R_{qk2}) = ½ x (502 + 23) =	-	262.5
	479.9	262.5

C I) Horizontal Loads – *Wind on short side*	**w_k [kN/m]**	**A_k [kN]**
Wind x – direction w_x = 3.75 x 0.39 =	1.46	-
Wind y – direction w_y = 3.75 x 0.67 =	2.51	-
Forklift collision y – direction A_y =	-	375

C II) Horizontal Loads – *Wind on long side*	**w_k [kN/m]**	**A_k [kN]**
Wind x – direction w_x = 3.75 x 0.67 =	2.51	-
Wind y – direction w_y = 3.75 x 0.39 =	1.46	-
Forklift collision x – direction A_x =	-	375

8.2 INTERNAL FORCES

8.2.1 LOAD COMBINATIONS

Load combinations in the ultimate limit state			ψ
LC1	Maximum vertical load	$Q_{Ed} = 1.35 \cdot G_k \oplus 1.5 \cdot S_k$ $q_{Ed} = 1.5 \cdot \psi_0 \cdot w_k$	$\psi_0 = 0.6$
LC2	Maximum horizontal load	$Q_{Ed} = 1.35 \cdot G_k \oplus 1.5 \cdot \psi_0 \cdot S_k$ $q_{Ed} = 1.5 \cdot w_k$	$\psi_0 = 0.5$
LC3	Minimum vertical load	$Q_{Ed} = 1.0 \cdot G_k \oplus 0 \cdot S_k$ $q_{Ed} = 1.5 \cdot w_k$	
LC4	Maximum vertical load with collision load	$Q_{Ed} = 1.0 \cdot G_k \oplus 1.0 A_k \oplus 1.5 \cdot \psi_1 \cdot S_k$ $q_{Ed} = 1.5 \cdot \psi_2 \cdot w_k$	$\psi_1 = 0.2$ $\psi_2 = 0$
LC5	Maximum horizontal load with collision load	$Q_{Ed} = 1.0 \cdot G_k \oplus 1.0 A_k \oplus 1.5 \cdot \psi_2 \cdot S_k$ $q_{Ed} = 1.5 \cdot \psi_1 \cdot w_k$	$\psi_2 = 0$ $\psi_1 = 0.2$
LC6	Minimum vertical load with collision load	$Q_{Ed} = 1.0 \cdot G_k \oplus 1.0 A_k$ $q_{Ed} = 1.5 \cdot \psi_1 \cdot w_k$	$\psi_1 = 0.2$

8.2.2 ECCENTRICITY

Slenderness of the column $i = 0.289 \cdot h = 0.289 \cdot 50 = 14.45cm$ $\lambda = \frac{l_0}{i} = \frac{2 \cdot 900}{14.45} = 125$

$80 < \lambda = 125 < 160$ → Slender column!

Limit slenderness $|n_{Ed}| = \frac{N_{Ed}}{A_c \cdot f_{cd}} = \frac{1042}{50 \cdot 50 \cdot 1.98} = 0.21 < 0.41$ $\lambda_{\lim} = \frac{16}{\sqrt{|n_{Ed}|}} = \frac{16}{\sqrt{0.21}} = 34.9$

$\lambda = 125 > \lambda_{\lim} = 34.9$ → *Second order effect must be considered*

Eccentricity $e_{tot} = e_i + e_2$

$$e_i = \theta_i \cdot \frac{l_0}{2} = \frac{1}{100 \cdot \sqrt{l}} \cdot \frac{l_0}{2} = \frac{1}{100 \cdot \sqrt{9}} \cdot \frac{18}{2} = 0.03m = 3cm$$

$$e_2 = K_1 \cdot \frac{l_0^2}{10} \cdot \frac{1}{r} = 1.0 \cdot \frac{18^2}{207 \cdot 0.45 \cdot 10} = 0.35m = 35cm$$

$$e_{tot} = e_i + e_2 = 3 + 35 = 38cm$$

8.2.3 CHARACTERISTIC FORCES AT THE SUPPORT

	N_k [kN]	$M_{y,k}$ [kNm]	$V_{x,k}$ [kN]	$M_{x,k}$ [kNm]	$V_{y,k}$ [kN]
G_k	- 479.9	- 168.0	0.0	0.0	0.0
S_k	-262.5	- 91.9	0.0	0.0	0.0
$w_{x,k}$	0.0	- 15.8 / +27.1	+ 3.5 / - 6.0	0.0	0.0
$w_{y,k}$	0.0	0.0	0.0	+ 15.8 / - 27.1	+ 3.5 / - 6.0
$A_{x,k}$	0.0	+ 393.8	- 375.0	0.0	0.0
$A_{y,k}$	0.0	- 393.8	+ 375.0	0.0	0.0

8.2.4 DESIGN FORCES AT THE SUPPORT

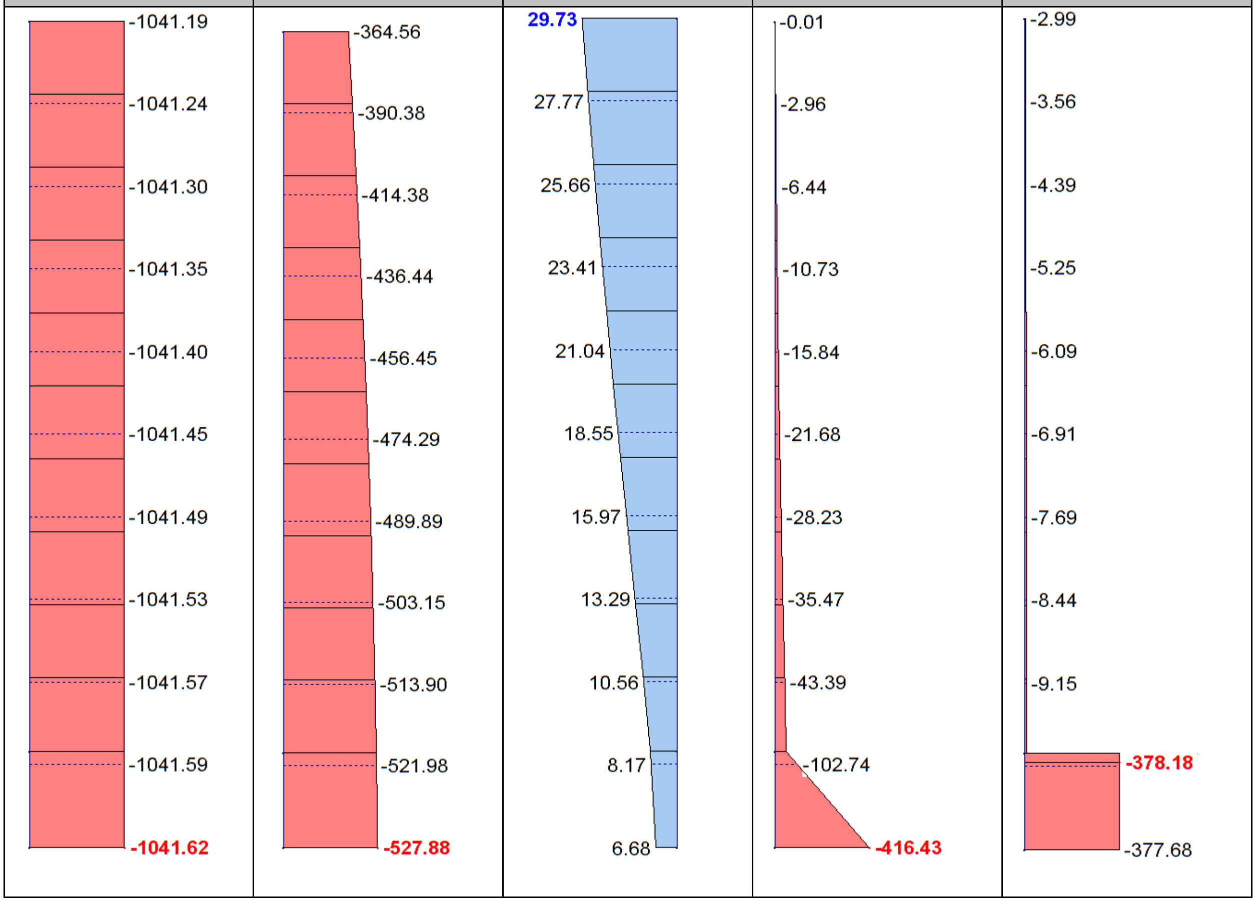
		N_{Ed}	$M_{y,Ed}$	$V_{x,Ed}$	$M_{x,Ed}$	$V_{y,Ed}$
C I	**LC1**	- 1041.6	- 527.9	4.9	- 47.9	-7.2
	LC2	- 844.7	- 416.9	6.7	- 61.1	- 10.5
	LC3	- 479.9	- 224.6	6.1	- 51.4	- 9.9
	LC4	- 558.6	- 237.1	0.9	- 410.3	- 376.0
	LC5	- 480.0	- 204.3	1.9	- 416.4	- 377.7
	LC6	- 479.9	- 204.3	1.9	- 416.4	- 377.7
C II	**LC1**	- 1041.6	- 444.1	- 7.2	35.9	4.9
	LC2	- 844.7	- 314.1	- 10.5	41.8	6.7

	LC3	- 479.9	- 140.0	- 9.9	33.2	6.1
	LC4	- 558.6	182.4	- 376.0	9.3	0.9
	LC5	- 479.9	225.1	- 377.7	12.9	1.9
	LC6	- 479.9	225.1	- 377.7	12.9	1.9

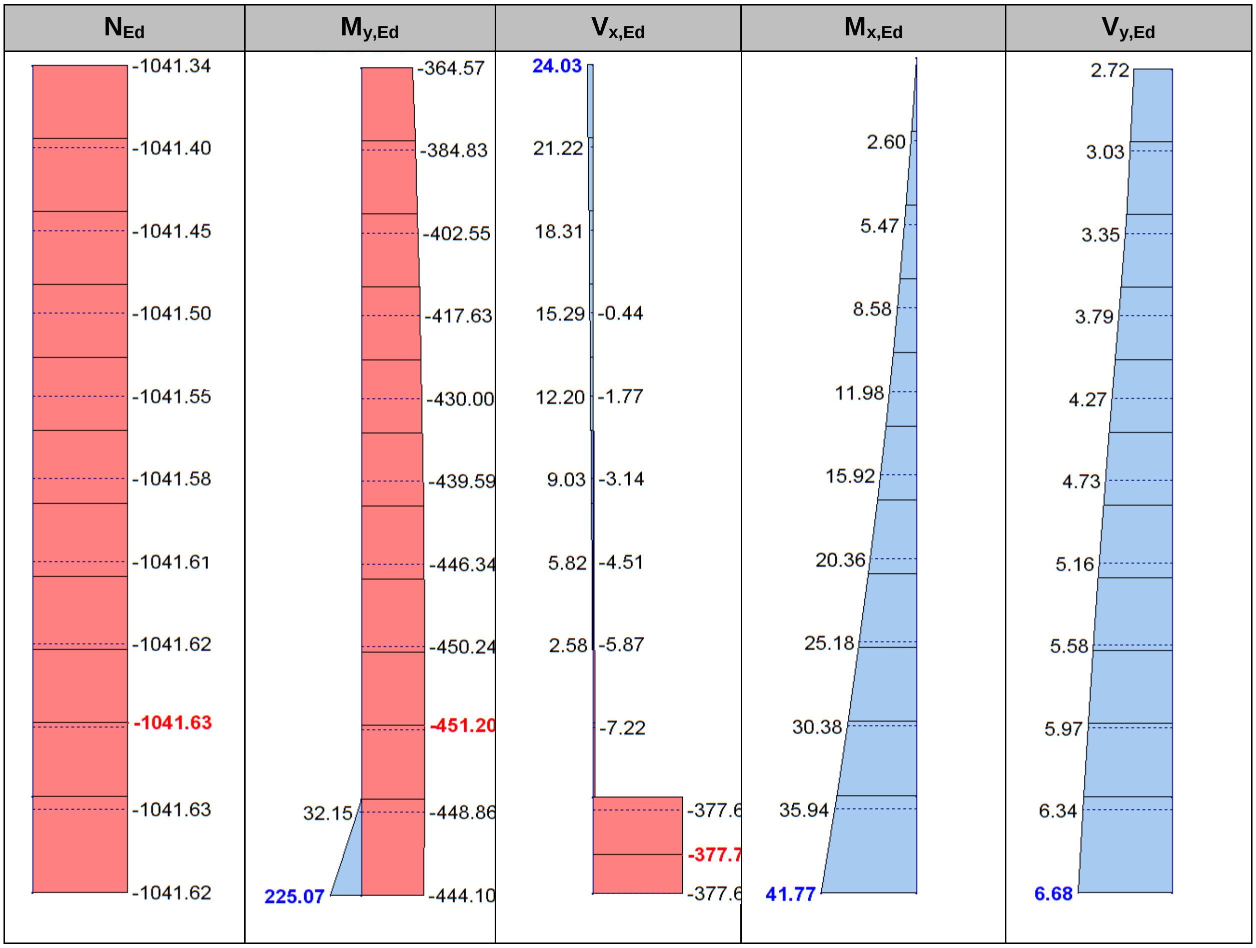
8.2.5 MAXIMUM AND MINIMUM INTERNAL FORCES

C I – Wind on Short Side of the Building

N_{Ed}	$M_{y,Ed}$	$V_{x,Ed}$	$M_{x,Ed}$	$V_{y,Ed}$



C II – Wind on Long Side of the Building



8.3 ULTIMATE LIMIT STATE DESIGN

$$\mu_{Edy} = \frac{M_{Edy}}{b \cdot h^2 \cdot f_{cd}} \qquad \mu_{Edx} = \frac{M_{Edx}}{b^2 \cdot h \cdot f_{cd}} \qquad \rightarrow \mu_1 = \max\{\mu_{Ed,y}, \mu_{Ed,x}\}\ ;\ \mu_2 = \min\{\mu_{Ed,y}, \mu_{Ed,x}\}$$

$$\nu = \nu_{Ed} = \frac{N_{Ed}}{b \cdot h \cdot f_{cd}} \qquad \rightarrow \omega_{tot}$$

$$A_{s,req} = \omega_{tot} \cdot \frac{b \cdot h}{f_{yd} / f_{cd}} \qquad \text{on each side: } \frac{1}{4} A_{s,req}$$

		$\mu_{Ed,y}$	$\mu_{Ed,x}$	ν	$\mu_{Ed,1}$	$\mu_{Ed,2}$	ω_{tot}	$A_{s,req}$	¼ $A_{s,req}$
C I	**LC1**	0.213	0.019	-0.210	0.213	0.019	0.8	**91.03**	**22.76**
	LC2	0.168	0.025	-0.171	0.168	0.025	0.3	**34.14**	**8.53**
	LC3	0.091	0.021	-0.097	0.091	0.021	0.175	**19.91**	**4.98**
	LC4	0.096	0.166	-0.113	0.166	0.096	0.45	**51.21**	**12.80**

	LC5	0.083	0.168	-0.097	0.168	0.083	0.45	**51.21**	**12.80**
	LC6	0.083	0.168	-0.097	0.168	0.083	0.45	**51.21**	**12.80**
C II	**LC1**	0.179	0.015	-0.210	0.179	0.015	0.4	**45.52**	**11.38**
	LC2	0.127	0.017	-0.171	0.127	0.017	0.3	**34.14**	**8.53**
	LC3	0.057	0.013	-0.097	0.057	0.013	0.05	**5.69**	**1.42**
	LC4	0.074	0.004	-0.113	0.074	0.004	0.01	**1.14**	**0.28**
	LC5	0.091	0.005	-0.097	0.091	0.005	0.01	**1.14**	**0.28**
	LC6	0.091	0.005	-0.097	0.091	0.005	0.01	**1.14**	**0.28**

Chosen **4 x 5 Ø 25 = 98.2 cm²** **(5 Ø 25 on each side)**

8.4 SERVICEABILITY LIMIT STATE DESIGN

According to DIN-EN-1992-1-1 no design in the serviceability limit state has to be done. No cracks are expected for the columns at serviceability limit state.

8.5 DESIGN AND REINFORCEMENT

8.5.1 MINIMUM AND MAXIMUM LONGITUDINAL REINFORCEMENT

Maximum reinforcement

$$A_{s,\max} = \frac{1}{2} \cdot 0.09 \cdot A_c = \frac{1}{2} \cdot 0.09 \cdot 50 \cdot 50 = 112.5 cm^2$$

Minimum reinforcement

$$A_{s,\min} = \frac{0.15 \cdot |N_{Ed}|}{f_{yd}} = \frac{0.15 \cdot 1042}{43.5} = 3.59 cm^2$$

8.5.2 TRANSVERSE REINFORCEMENT

Minimum diameter

$$\min d_{s,bü} \geq \begin{cases} 6mm \\ 0.25 \cdot \max d_{s,l} = 0.25 \cdot 25 = \underline{\underline{6.25mm}} \end{cases}$$

Maximum distance

$$\min s_w \leq \begin{cases} 12 \cdot \min d_{s,l} = 12 \cdot 25 = 300mm \\ h_{\min} = 500mm \\ \underline{\underline{300mm}} \end{cases}$$

Chosen **stirrups Ø 8 /25**

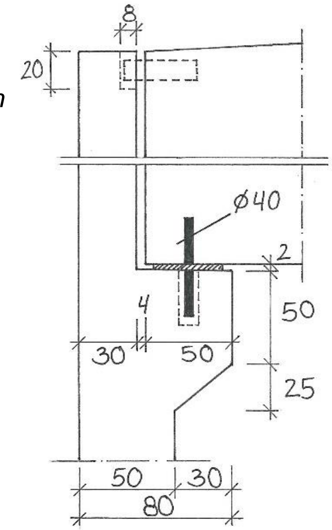
8.5.3 DESIGN OF THE CORBEL

Forces on the corbel $V_{Ed} = 1042kN$

$T_{Ed} = V_{Ed} \cdot l_{eff} / 300 = 1042 \cdot 29.2 / 300 = 101kNm$

$H^T_{Ed,y} = T_{Ed} / z_{HED} = 101kNm / 1.32m = 76.5kN$

$H^W_{Ed,y} = 3kN$ $H^W_{Ed,x} = 30kN$



Flexure Design of the Column

Bending moment $M_{Ed} = 101kNm$

Static height $d = 50 - 2 - 1 - 1 = 46cm$

Flexure design $\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{10100}{30 \cdot 46^2 \cdot 1.98} = 0.08$ → $\omega = 0.0836$

Reinforcement $A_{s,req} = \omega \cdot b \cdot d \cdot f_{cd} / \sigma_{sd} = 0.0836 \cdot 30 \cdot 46 \cdot 19.8 / 435 = 5.25cm^2$

Chosen **2 Ø 20 = 6.28 cm²** **longitudinal reinforcement**

Shear Design of the Column

Shear force $H_{Ed,y} = 76.5kN + 3kN = 79.5kN$

Shear force resistance $V_{Rd,\max} = 0.75 \cdot 30 \cdot 0.9 \cdot 46 \cdot 1.98 \cdot \frac{1.2}{1+1.2^2} = 458kN$ ($\cot\theta \approx 1.2$)

$V_{Ed} = H_{Ed,y} = 79.5kN < V_{Rd,\max} = 458kN$ → The compression strut is ok.

Reinforcement $a_{sw,req} = \frac{V_{Ed}}{f_{yd} \cdot z \cdot \cot\theta} = \frac{79.5kN}{43.5 \cdot 0.9 \cdot 0.46 \cdot 1.2} = 3.68cm^2 / m$

Supplementary Reinforcement

$A_{s,req} = H^T_{Ed} / f_{yd} = 79.5 / 43.5 = 1.83cm^2$

Bearing Load

Size of the support *350 x 350 x 20 mm*

Compression stresses $\sigma_{Ed} = \frac{V_{Ed}}{a_L \cdot b_L} = \frac{1042kN}{35 \cdot 35} \cdot 10 = 8.5MN/m^2 \le 0.85 \cdot f_{cd} = 0.85 \cdot 19.8 = 16.8MN/m^2$ ✓

Main Reinforcement of the Corbel

Static height $d \approx 75 - 6 = 69cm$

Tensile force $F_{sd} = \frac{1.1 \cdot a}{0.85 \cdot d} \cdot V_{Ed} + H_{Ed} > V_{Ed}/2$

$$F_{sd} = \frac{1.1 \cdot 27}{0.85 \cdot 69} \cdot 1042 + 30 = 558kN > \frac{1042}{2} = 521kN$$

Reinforcement $A_{s,req} = \frac{F_{sd}}{f_{yd}} = \frac{521}{43.5} = 12.0cm^2$

Chosen **6 Ø 14 = 18.48 cm²** **reinforcement loop (3 layers)**

Provided static height $d_{prov} = 75 - 2 - 1 - 1.4 - 2 - 1.4/2 = 75 - 7.1 = 67.9cm < d_{est} = 69cm$

Reinforcement $F_{sd} = \frac{1.1 \cdot 27}{0.85 \cdot 67.9} \cdot 1040 + 30 = 565kN$

$$A_{s,req} = \frac{F_{sd}}{f_{yd}} = \frac{565}{43.5} = 13.0cm^2 < A_{s,prov} = 18.48cm^2$$ ✓

Anchorage at the support $l_{b,rqd} = 64cm$ (Ø14) *for C35/45 and moderate bond*

$$l_{bd} = \alpha_1 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 0.7 \cdot 64 \cdot (20.5/24.6) = 37.2cm$$

$$> l_{b,\min} = 0.3 \cdot 0.7 \cdot 64 = 13.4cm > 10\phi_s = \underline{\underline{14.0cm}}$$

$$l_{b,dir} = \frac{2}{3} \cdot l_{bd} = \frac{2}{3} \cdot 37.2 = \underline{\underline{24.8cm}} > 6.7\phi_s = 9.4cm$$

$$l_{b,dir,prov} == 23 + 17.5 - 2 - 1.4 - 1 = 36.1cm > l_{b,dir,req} = 24.8cm$$ ✓

Overlap length $l_{b,rqd} = 81cm$ (Ø25) *for C35/45 and good bond*

$$l_{bd} = \alpha_1 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 1.0 \cdot 81 \cdot (20.5/24.6) = 67.5cm$$

$$l_0 = \alpha_6 \cdot l_{bd} = 2.0 \cdot 67.5 = \underline{\underline{135cm}} > l_{0,\min}$$

$$l_{0,\min} = 0.3 \cdot \alpha_6 \cdot l_{b,rqd} = 0.3 \cdot 2.0 \cdot 81 = \underline{\underline{48.6cm}} \ge \begin{cases} 15 \cdot 2.5 = 37.5cm \\ 20cm \end{cases}$$

➔ $l_0 = 135cm$

Secondary Reinforcement

Crosswise reinforcement $A_{s,net,req} = 0.5 \cdot A_s = 0.5 \cdot 13.0 = 7.5cm^2$

Chosen **5 Ø 10 = 7.85 cm²** **stirrups in a crosswise arrangement**

Design of the Shear Force Dowel

Horizontal force $H_{Ed} = \sqrt{30^2 + 79.5^2} = 85kN$

Shear force resistance $W_{dowel} = r^3 \cdot \pi / 4 = 2.0^3 \cdot \pi / 4 = 6.28cm^3$

$$H_{Rd} = 1.25 \cdot f_{yd} \cdot \frac{W_{dowel}}{t/2 \cdot d_{dowel}} = 1.25 \cdot 43.5 \cdot \frac{6.28}{1.0 + 4.0} = 68.3kN \text{ (per dowel)}$$

2 dowels → $H_{Rd} = 2 \cdot 68.3kN = 137kN > H_{Ed} = 85kN$ ✓

Chosen **2 dowels Ø 40**

Distance to the edge *x - direction:* $3 \cdot d_s = 12cm < u_x = 23cm < 8 \cdot d_s = 32cm$

Supplementary reinforcement necessary:

$A_{s,x,req} = H_{Ed} / f_{yd} = 30.0 / 43.5 = 0.7cm^2$

y – direction: $3 \cdot d_s = 12cm < u_y = 17cm < 8 \cdot d_s = 32cm$

Supplementary reinforcement necessary:

$A_{s,x,req} = H_{Ed} / f_{yd} = 79.5 / 43.5 = 1.83cm^2$

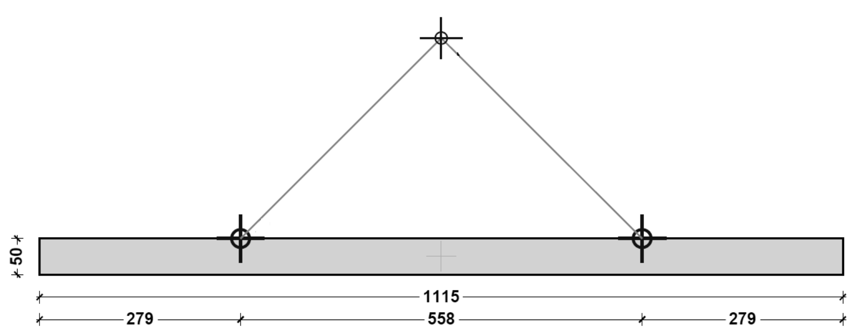
8.6 TRANSPORT AND ASSEMBLY

8.6.1 TRANSPORT AND LIFTING ON THE CONSTRUCTION SITE

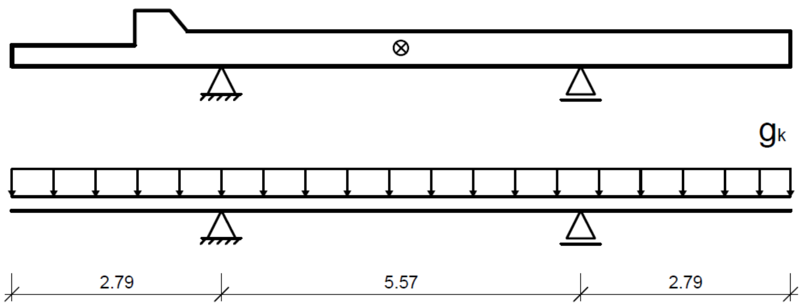
The beam will be suspended with the help of two anchors on each side above the center of gravity at around 1/6 of the length of the beam. Which might cause tension over the anchors and might require upper reinforcement.

System

In the following the static system during transportation and assembly can be seen.



Simplified static system:



Design

Lifting loads *A = 5.65 m²* *V = 5.65 x 0.50 = 2.825 m³*

self-weight g_k *= 2.825 m³ x 25 kN/m³/ 11.15 m = 6.33 kN/m*

Bending moment $M_{Ed}^{G} = 1.15 \cdot (-6.33) \cdot 2.79^2 / 2 = -28.3kNm$

Time of first lifting approximately after 5 days (t = 5d)

Compressive strength at time t

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \quad \rightarrow \quad \beta_{cc}(t = 5d) = e^{s\left(1-\sqrt{28/t}\right)} = e^{0.2\left(1-\sqrt{28/5}\right)} = 0.76$$

$$f_{cm}(t = 5d) = 0.76 \cdot 43 = 32.68N/mm^2$$

$$f_{ck}(t = 5d) = f_{cm}(t) - 8 = 32.68 - 8 = 24.68N/mm^2$$

$$f_{cd}(t = 5d) = 0.85 \cdot f_{ck}(t)/1.5 = 0.85 \cdot 24.68/1.5 = \underline{\underline{14.0N/mm^2}}$$

Required reinforcement

$$\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{2830}{50 \cdot 45^2 \cdot 1.4} = 0.02 \quad \rightarrow \omega = 0.0203$$

$$A_{s2,req} = (\omega \cdot b \cdot d \cdot f_{cd} + N_{Ed}) / \sigma_{sd}$$

$$= 0.0203 \cdot 50 \cdot 45 \cdot 1.40 / 43.5 = \underline{\underline{1.47cm^2}} < A_{s,prov} = 24.55cm^2$$

No additional reinforcement is required!

8.6.2 TRANSPORT ANCHORS

In order to lift the beam out of its formwork as well as to be able to transport and assemble it on site, transport anchors are needed. Those are installed during the production of the beam and must be designed in accordance with the VDI/BV-BS-Richtlinie 6205. For this project the anchors of the company *Halfen* are chosen.

Self-weight $F_G = 6.33kN/m \cdot 11.15m = 70.6kN$

Formwork adhesion $F_{adh} = q_{adh} \cdot A_f$ → no adhesion here because of hinged formwork!

Dynamic factor $\psi_{dyn} = 1.3$

Factor for inclination $z = 1.41 \text{ for } \beta = 45°$

Tensile force $F_z = \dfrac{F_G \cdot \psi_{dyn} \cdot z}{n} = \dfrac{70.6 \cdot 1.3 \cdot 1.41}{2} = \underline{\underline{64.7kN}}$

Chosen **HALFEN DEHA Spherical head anchor 6000-10,0-0170**

Load capacity of the anchor $F_{z,Rd} = 67.3kN > F_{z,Ed} = 64.7kN$ ✓ (see Table 18)

Table 18 - Sizing table for 'HALFEN DEHA Lifting Anchor Systems'

Spherical head anchors in beams and walls with no special requirements on the reinforcement (load class 1,3 – 7,5)

Load class	Article number	Anchor length l [mm]	Minimum height of beams B_1 [mm]	Wall thickness $2 \times e_r$ [mm]	Load capacity [kN] at concrete strength f_{ci} for: **Axial pull** up to 30° [β] 15 N/mm²	**Diagonal pull** up to 60° [β] 15 N/mm²	**Axial pull** and **diagonal pull** up to 60° [β] 25 N/mm²	**Axial pull** and **diagonal pull** up to 60° [β] 35 N/mm²	Axial spacing of anchors e_z [mm]
10,0	6000-10,0-0170	170	340	300	46.4	37.2	60.0	70.9	520
				350	52.1	41.7	67.3	79.6	
				400	57.6	46.1	74.4	88.0	
	6000-10,0-0340	340	680	280	76.6	61.3	98.9	100.0	1030
				300	80.7	64.5	100.0		
				320	84.7	67.7			
	6000-10,0-0680	680	1360	160	73.7	70.0	95.2	100.0	2050
				180	83.0	76.5	100.0		
				200	92.2	82.8			

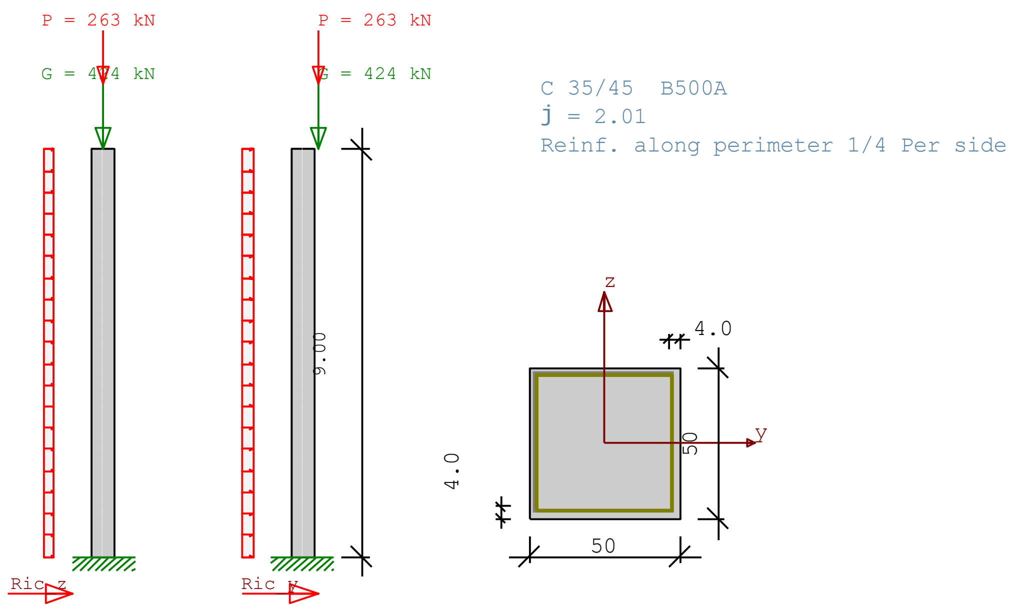
8.7 SOFTWARE CALCULATIONS

Demo Frilo Nemetschek

Position: C.04 – Precast Concrete Column with Biaxial Wind Loads

Reinforced Concrete Column B5 01/2018 (Frilo R-2018-1/P12)

```
CANTILEVER COLUMN, Rectangle, 2-axial strained
--------------------------------------------------------------------
Calculation base: DIN EN 1992-1-1/NA/A1:2015-12
E = 34000 N/mm2    ρ = 2500 kg/m3
```




```
 1     Mcry =      66.87 kNm     Mcrz =      66.87 kNm
```

NODES - LOADS :

LcNo.	KNo.	V (kN)	ey (cm)	ez (cm)	Py (kN)	Pz (kN)	My (kNm)	Mz (kNm)	act	con	alt
1	2	423.60	35.0	.	.	.	.	.		.	g
		263.00	35.0	.	.	.	.	.	J	.	p
		56.25	(dead load)								

MEMBER - LOADS :

LcNo	MNo	type	dir	g1 (kN/m	g2 , kN)	dist (m)	length (m)	actGrp	con	alt
2	.	Uniform loa	z	1.46	1.46	.00	9.00	I	.	p
3	.	Uniform loa	y	2.51	2.51	.00	9.00	I	.	p

Actions:

```
No. Cl Name                            ψ0    ψ1    ψ2     γ
-------------------------------------------------------------------
I  4  Wind loads                       0.60  0.20  0.00  1.50
J  3  Snow loads <1000m                0.50  0.20  0.00  1.50

All actions are use like independent.

Further design fundamentals:
-------------------------------------------------------------------
Accuracy Gkn = 6.14e-5
Number of sub-element per member section: 6
Stress-strain-curve of concr. for deform. analy. EN 1992-1-1 3.1.5
Calc. of compr. force in concr.without deduction of reinf.
If n > -0.10 : eff EI acc.toEN2  7.4.2  (7.19)
Creep effects are considered by modified stress-strain-
curve.
φeff = φ0 * M0/Med (M0 By permanent combination with ei)
consequency class acc. EN 1990 tab b.1CC2 -> KFi = 1.0 (Tab B.3)

FLBemBn.DLL: version9.0.1.121

NKi/N =  5.18 dir_y  NKi/N =  5.18 dir_z  cross sect. of concr. only


CALCULATED COMBINATIONS by 3 Loads Kombi_D

Lc- Comb K1  K2  K3  K4  K5  K6  K7  K8
-------------------------------------------------------------------
          g   g   g   g   g   g   g   g
          J   I   I   J   I   I       I
   1      x   .   x   x   x   x   x   .
   2      x   .   x   .   x   .   x   x
   3      x   x   x   x   .   x   x   .

Partial safety factor    γC = 1.50  γS = 1.15    γG = 1.35 / 1.00

Proof according DIN EN 1992-1-1/NA/A1:2015-12
γC = 1.50   γS = 1.15   φeff = 1.37

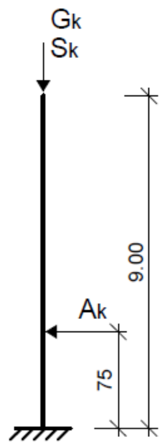
Design values    LcCom = 1   in :   y-direction    z-direction
-------------------------------------------------------------------
System                                    Displacable
Buckling length             sk  =     18.00         18.00 m
Slenderness                 λ   =    124.6         124.6
Normal force                N   =  -1042.30      -1042.30 kN
Specific normal force       n   =      -.21          -.21
Inter. moment  h =   .00 m ,  M =   -429.72        -53.22 kNm
Methodical  eccentr. e = M / N  =     41.23          5.11 cm
Related eccentr.   e/b and e/d  =    0.8246        0.1021
Unintentional eccentricity  ei  =      3.00          3.00 cm
Displacement Th.2.Ord.      e2  =     37.11          0.18 cm
Design moment            M des. =   -847.80        -23.80 kNm

Reinforcement            tot ω  =     .8827
                             ρ  =      4.03 %
                         Req As =    100.66 cm2
The influence of creep will considered acc. EN 1992-1-1  5.8.4 curve.
```

9 POS. C.05 – PRECAST CONCRETE COLUMN WITH MAXIMUM VERTICAL LOAD

9.1 SYSTEM

System



Cross-section b/h = 50/50cm

Building materials **C 35/45 | B 500A**

Exposure class XC1, WO

Reinforcing steel $c_{nom,w} = c_{min} + \Delta c_{dev} = 10mm + 10mm = 20mm$

$c_{nom,s} = c_{min} + \Delta c_{dev} = 25mm + 10mm = 35mm$ → $c_{v,w,min} = 25mm$

Chosen $c_{nom} = 25mm$

Characteristic loads

Vertical Loads		**G_k [kN]**	**Q_k [kN]**
Self-weight	G_{k1} = 0.5 x 0.5 x 9.0 x 25 kN/m³ =	56.3	-
Reaction Force from *POS. P.03*			
	G_{k2} = 2 x (211.7 + 423.8) =	1271.0	-
	S_k = 2 x (502 + 23) =	-	1050.0
		1327.3	1050.0

Horizontal Loads			**A_k [kN]**
Forklift collision	A_k =	-	375.0

9.2 INTERNAL FORCES

9.2.1 LOAD COMBINATIONS

Load combinations in the ultimate limit state			ψ
LC1	Maximum vertical load	$Q_{Ed} = 1.35 \cdot G_k \oplus 1.5 \cdot S_k$	
LC2	Vertical load with collision load	$Q_{Ed} = 1.0 \cdot G_k \oplus 1.0 A_k \oplus 1.5 \cdot \psi_1 \cdot S_k$	$\psi_1 = 0.2$
LC3	Minimum vertical load with collision load	$Q_{Ed} = 1.0 \cdot G_k \oplus 1.0 A_k$	

9.2.2 ECCENTRICITY

Slenderness of the column $\quad i = 0.289 \cdot h = 0.289 \cdot 50 = 14.45cm \quad \lambda = \frac{l_0}{i} = \frac{2 \cdot 900}{14.45} = 125$

$80 < \lambda = 125 < 160$ → *Slender column!*

Limit slenderness $\quad |n_{Ed}| = \frac{N_{Ed}}{A_c \cdot f_{cd}} = \frac{3367}{50 \cdot 50 \cdot 1.98} = 0.68 > 0.41 \quad \lambda_{\lim} = 25$

$\lambda = 125 > \lambda_{\lim} = 25$ → *Second order effect must be considered!*

Eccentricity

$$e_i = \theta_i \cdot \frac{l_0}{2} = \frac{1}{100 \cdot \sqrt{l}} \cdot \frac{l_0}{2} = \frac{1}{100 \cdot \sqrt{9}} \cdot \frac{18}{2} = 0.03m = 3cm$$

$$e_2 = K_1 \cdot \frac{l_0^2}{10} \cdot \frac{1}{r} = 1.0 \cdot \frac{18^2}{207 \cdot 0.45 \cdot 10} = 0.35m = 35cm$$

$$e_{tot} = e_i + e_2 = 3 + 35 = 38cm$$

9.2.3 CHARACTERISTIC FORCES AT THE SUPPORT

	N_k [kN]	M_k [kNm]	V_k [kN]
G_k	- 1327.3	0.0	0.0
S_k	- 1050.0	0.0	0.0
A_k	0.0	- 393.8	+ 375.0

9.2.4 DESIGN FORCES AT THE SUPPORT

	N_{Ed}	M_{Ed}	V_{Ed}
LC1	- 3366.9	- 151.7	+ 5.6
LC2	- 1537.4	- 451.0	+ 380.4
LC3	- 1327.3	- 440.5	+ 379.6

9.3 ULTIMATE LIMIT STATE DESIGN

$$\mu_{Ed} = \frac{M_{Ed}}{b \cdot h^2 \cdot f_{cd}} \qquad \nu = \nu_{Ed} = \frac{N_{Ed}}{b \cdot h \cdot f_{cd}} \qquad \rightarrow \omega_{tot}$$

$$A_{s,req} = \omega_{tot} \cdot \frac{b \cdot h}{f_{yd} / f_{cd}}$$ on each side: $\frac{1}{4} A_{s,req}$

		$\mu_{Ed,y}$	$\mu_{Ed,x}$	ν	$\mu_{Ed,1}$	$\mu_{Ed,2}$	ω_{tot}	$A_{s,req}$	¼ $A_{s,req}$
C I	LC1	-3366.9	-151.7	5.6	0.061	-0.680	0.00	**0.00**	**0.00**
	LC2	-1537.4	-451.0	380.4	0.182	-0.311	0.30	**34.14**	**8.53**
	LC3	-1327.3	-440.5	379.6	0.178	-0.268	0.25	**28.45**	**7.11**

Chosen **4 x 3 Ø 20 = 37.7 cm²** **(3 Ø 20 on each side)**

9.4 SERVICEABILITY LIMIT STATE DESIGN

According to DIN-EN-1992-1-1 no design in the serviceability limit state has to be done. No cracks are expected for the columns at serviceability limit state.

9.5 DESIGN AND REINFORCEMENT

9.5.1 MINIMUM AND MAXIMUM LONGITUDINAL REINFORCEMENT

Maximum reinforcement $$A_{s,\max} = \frac{1}{2} \cdot 0.09 \cdot A_c = \frac{1}{2} \cdot 0.09 \cdot 50 \cdot 50 = 112.5 cm^2$$

Minimum reinforcement $$A_{s,\min} = \frac{0.15 \cdot |N_{Ed}|}{f_{yd}} = \frac{0.15 \cdot 3367}{43.5} = 11.6 cm^2$$

9.5.2 TRANSVERSE REINFORCEMENT

Minimum diameter $$\min d_{s,bü} \geq \begin{cases} 6mm \\ 0.25 \cdot \max d_{s,l} = 0.25 \cdot 25 = \underline{\underline{6.25mm}} \end{cases}$$

Maximum distance $$\min s_w \leq \begin{cases} 12 \cdot \min d_{s,l} = 12 \cdot 25 = 300mm \\ h_{\min} = 500mm \\ \underline{\underline{300mm}} \end{cases}$$

Chosen **Ø 8 /25 = 4.02 cm²/m** **stirrups**

9.5.3 DESIGN OF THE CORBEL

Forces on the corbel $V_{Ed} = \frac{1}{2} \cdot 3291kN = 1646kN$

$T_{Ed} = V_{Ed} \cdot l_{eff} / 300 = 1646 \cdot 29.2 / 300 = 160kNm$

$H^T_{Ed,y} = T_{Ed} / z_{HED} = 160kNm / 1.32m = 121kN$

Flexure Design of the Column

Bending moment $M_{Ed} = 160kNm$

Static height $d = 50 - 2 - 1 - 1 = 46cm$

Flexure design $\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{16000}{30 \cdot 46^2 \cdot 1.98} = 0.13$ → $\omega = 0.1401$

Reinforcement $A_{s,req} = \omega \cdot b \cdot d \cdot f_{cd} / \sigma_{sd} = 0.1404 \cdot 30 \cdot 46 \cdot 19.8 / 435 = 8.80cm^2$

Chosen **3 Ø 20 = 9.42 cm²** **longitudinal reinforcement**

Shear Design of the Column

Shear force $H_{Ed,y} = 121kN$

Shear force resistance $V_{Rd,\max} = 0.75 \cdot 30 \cdot 0.9 \cdot 46 \cdot 1.98 \cdot \frac{1.2}{1 + 1.2^2} = 458kN$ $(\cot\theta \approx 1.2)$

$V_{Ed} = H_{Ed,y} = 121kN < V_{Rd,\max} = 458kN$ → *The compression strut is ok.*

Reinforcement $a_{sw,req} = \frac{V_{Ed}}{f_{yd} \cdot z \cdot \cot\theta} = \frac{121kN}{43.5 \cdot 0.9 \cdot 0.46 \cdot 1.2} = 5.6cm^2 / m$

Chosen **Ø 8 / 10 = 10.05 cm²/m**

Bearing Load

Size of the support *350 x 350 x 2 mm*

Compression stresses $\sigma_{Ed} = \frac{V_{Ed}}{a_L \cdot b_L} = \frac{1646kN}{35 \cdot 35} \cdot 10 = 13.4MN / m^2 \leq 0.85 \cdot f_{cd} = 0.85 \cdot 19.8 = 16.8MN / m^2$ ✓

Main Reinforcement of the Corbel

Static height $d \approx 75 - 9 = 66cm$

Tensile force $F_{sd} = \frac{1.1 \cdot a}{0.85 \cdot d} \cdot V_{Ed} + H_{Ed} > V_{Ed} / 2$

$F_{sd} = \frac{1.1 \cdot 27}{0.85 \cdot 66} \cdot 1646 = 871kN > \frac{1646}{2} = 823kN$

Reinforcement $A_{s,req} = \frac{F_{sd}}{f_{yd}} = \frac{871}{43.5} = 20.0cm^2$

Chosen **8 Ø 14 = 24.64 cm²** **reinforcement loop (4 layers)**

Provided static height $d_{prov} = 75 - 2 - 1 - 1.4 - 2 - 1.4 - 2.0/2 = 75 - 8.8 = 66.2cm > d_{est} = 66cm$

Anchorage at the support $l_{b,rqd} = 64cm$ (Ø14) *for C35/45 and moderate bond*

$$l_{bd} = \alpha_1 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 0.7 \cdot 64 \cdot (20.5/24.6) = 37.2cm$$

$$> l_{b,\min} = 0.3 \cdot 0.7 \cdot 64 = 13.4cm > 10\phi_s = \underline{\underline{14.0cm}}$$

$$l_{b,dir} = \tfrac{2}{3} \cdot l_{bd} = \tfrac{2}{3} \cdot 37.2 = \underline{\underline{24.8cm}} > 6.7\phi_s = 9.4cm$$

$$l_{b,dir,prov} == 23 + 17.5 - 2 - 1.4 - 1 = 36.1cm > l_{b,dir,req} = 24.8cm \checkmark$$

Overlap length $l_{b,rqd} = 64cm$ *(Ø20)* *for C35/45 and good bond*

$$l_{bd} = \alpha_1 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 1.0 \cdot 64 \cdot (20/24.6) = 52cm$$

$$l_0 = \alpha_6 \cdot l_{bd} = 2.0 \cdot 52 = \underline{\underline{104cm}} > l_{0,\min}$$

$$l_{0,\min} = 0.3 \cdot \alpha_6 \cdot l_{b,rqd} = 0.3 \cdot 2.0 \cdot 64 = \underline{\underline{38.4cm}} \geq \begin{cases} 15 \cdot 2 = 30cm \\ 20cm \end{cases}$$

➔ $l_0 = 104cm$

Secondary Reinforcement

Crosswise reinforcement $A_{s,net,req} = 0.5 \cdot A_s = 0.5 \cdot 20 = 10cm^2$

Chosen **5 Ø 12 = 11.3 cm²** **stirrups in a crosswise arrangement**

Design of the Shear Force Dowel

Horizontal force $H_{Ed} = 121kN$

Shear force resistance $W_{dowel} = r^3 \cdot \pi / 4 = 2.0^{30} \cdot \pi / 4 = 6.28cm^3$

$$H_{Rd} = 1.25 \cdot f_{yd} \cdot \frac{W_{dowel}}{\tfrac{t}{2} \cdot d_{dowel}} = 1.25 \cdot 43.5 \cdot \frac{6.28}{1.0 + 4.0} = 68.3kN \text{ (per dowel)}$$

2 dowels → $H_{Rd} = 2 \cdot 68.3kN = 137kN > H_{Ed} = 121kN$ ✓

Chosen **2 dowels Ø 40**

Distance to the edge *x - direction:* $3 \cdot d_s = 12cm < u_x = 23cm < 8 \cdot d_s = 32cm$

y – direction: $3 \cdot d_s = 12cm < u_y = 17cm < 8 \cdot d_s = 32cm$

Supplementary reinforcement necessary:

$$A_{s,req} = H_{Ed} / f_{yd} = 121 / 43.5 = 2.78cm^2$$

9.5.4 REINFORCEMENT DRAWING

See Appendix A.4, page 190.

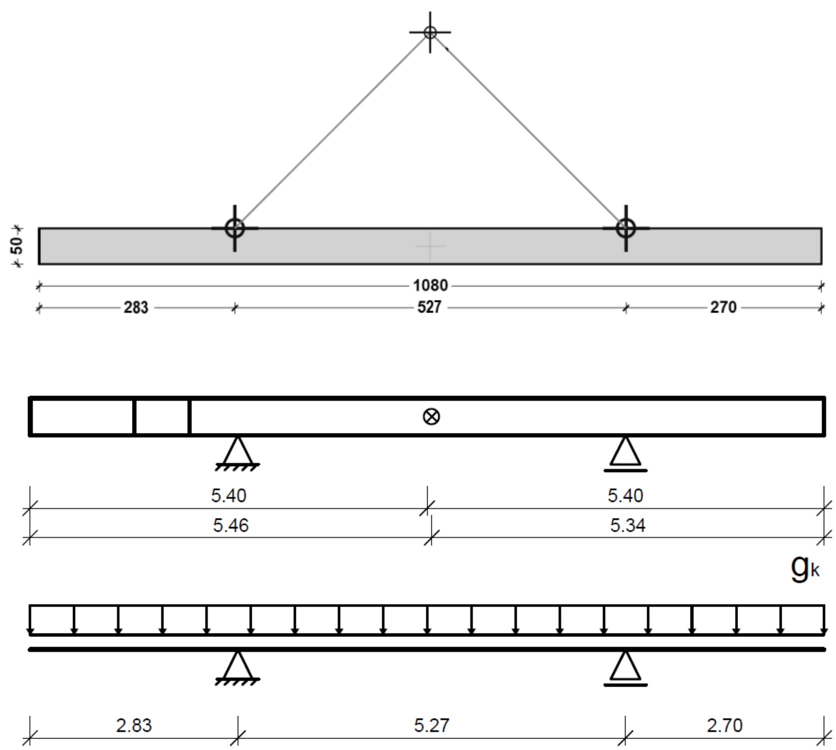
9.6 TRANSPORT AND ASSEMBLY

9.6.1 TRANSPORT AND LIFTING ON THE CONSTRUCTION SITE

The beam will be suspended with the help of two anchors on each side above the center of gravity at around 1/6 of the length of the beam. Which might cause tension over the anchors and might require upper reinforcement.

System

In the following the static system during transportation and assembly can be seen.



Design

Lifting loads	*A = 5.65 m²*	*V = 5.65 x 0.50 = 2.825 m³*
	self-weight	*g_k = 2.825 m³ x 25 kN/m³/ 11.15 m = 6.33 kN/m*
Bending Moment	$M_{Ed}^{G} = 1.15 \cdot (-6.33) \cdot 2.83^2 / 2 = -29.2 kNm$	
Time of first lifting	approximately after 5 days (t = 5d)	

Compressive strength at time t

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \quad \rightarrow \quad \beta_{cc}(t = 5d) = e^{s\left(1-\sqrt{28/t}\right)} = e^{0.2\left(1-\sqrt{28/5}\right)} = 0.76$$

$$f_{cm}(t = 5d) = 0.76 \cdot 43 = 32.68 N/mm^2$$

$$f_{ck}(t = 5d) = f_{cm}(t) - 8 = 32.68 - 8 = 24.68 N/mm^2$$

$$f_{cd}(t = 5d) = 0.85 \cdot f_{ck}(t) / 1.5 = 0.85 \cdot 24.68 / 1.5 = \underline{\underline{14.0 N/mm^2}}$$

Required reinforcement

$$\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{2920}{50 \cdot 45^2 \cdot 1.4} = 0.021 \quad \rightarrow \omega = 0.0203$$

$$A_{s2,req} = (\omega \cdot b \cdot d \cdot f_{cd} + N_{Ed}) / \sigma_{sd}$$

$$= 0.0203 \cdot 50 \cdot 45 \cdot 1.40 / 43.5 = \underline{\underline{1.47 cm^2}} < A_{s,prov} = 9.42 cm^2$$

No additional reinforcement is required!

9.6.2 TRANSPORT ANCHORS

In order to lift the beam out of its formwork as well as to be able to transport and assemble it on site, transport anchors are needed. Those are installed during the production of the beam and must be designed in accordance with the VDI/BV-BS-Richtlinie 6205. For this project the anchors of the company *Halfen* are chosen.

Self-weight $\quad F_G = 6.33 kN/m \cdot 10.8 m = 68.4 kN$

Formwork adhesion $\quad F_{adh} = q_{adh} \cdot A_f \quad \rightarrow$ no adhesion here because of hinged formwork!

Dynamic factor $\quad \psi_{dyn} = 1.3$

Factor for inclination $\quad z = 1.41 \text{ for } \beta = 45°$

Tensile force

$$F_z = \frac{F_G \cdot \psi_{dyn} \cdot z}{n} = \frac{68.4 \cdot 1.3 \cdot 1.41}{2} = \underline{\underline{62.7 kN}}$$

Chosen $\quad$ **HALFEN DEHA Spherical head anchor 6000-7,5-0200**

Load capacity of the anchor $\quad F_{z,Rd} = 65.3 kN > F_{z,Ed} = 62.7 kN$ ✓ $\quad$ (see Table 19)

Table 19 - Sizing Table for 'HALFEN DEHA Lifting Anchor Systems'

Spherical head anchors in beams and walls with no special requirements on the reinforcement (load class 1,3 – 7,5)

Load class	Article number	Anchor length l [mm]	Minimum height of beams B_1 [mm]	Wall thickness $2 \times e_r$ [mm]	Load capacity [kN] at concrete strength f_{ci} for: Axial pull up to 30° [β] 15 N/mm²	Diagonal pull up to 60° [β] 15 N/mm²	Axial pull and diagonal pull up to 60° [β] 25 N/mm²	Axial pull and diagonal pull up to 60° [β] 35 N/mm²	Axial spacing of anchors e_z [mm]
7,5	6000-7,5-0200	200	410	240	45.1	36.0	58.2	68.8	610
				260	47.8	38.3	61.8	73.1	
				280	50.6	40.5	65.3	75.0	
	6000-7,5-0300	300	610	200	54.1	43.3	69.9	75.0	910
				220	58.1	46.5	75.0		
				240	62.2	49.7			
	6000-7,5-0540	540	1090	160	63.2	58.4	75.0	75.0	1630
				180	71.1	63.8			
				200	75.0	69.1			

f_{ci} = concrete cube strength at time of lifting

9.7 SOFTWARE CALCULATIONS

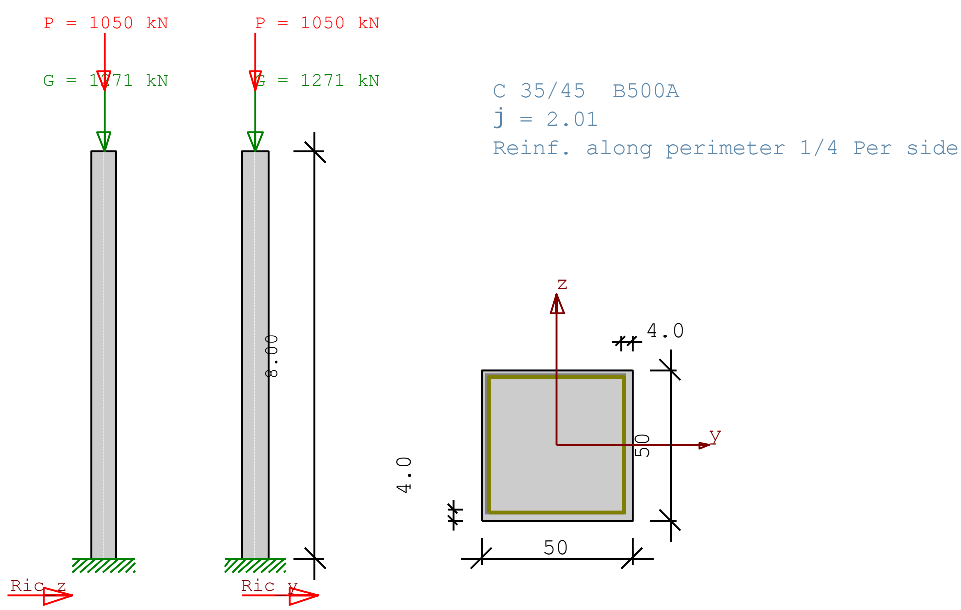
Demo Frilo Nemetschek

Position: C.05 – Precast Concrete Column with Maximum Vertical Load

Reinforced Concrete Column B5 01/2018 (Frilo R-2018-1/P12)

CANTILEVER COLUMN, Rectangle, 2-axial strained

Calculation base: DIN EN 1992-1-1/NA/A1:2015-12
E = 34000 N/mm2 ρ = 2500 kg/m3



1 Mcry = 66.87 kNm Mcrz = 66.87 kNm

NODES - LOADS :

LcNo.	KNo.	V (kN)	ey (cm)	ez (cm)	Py (kN)	Pz (kN)	My (kNm)	Mz (kNm)	act	con	alt
1	2	1271.0	.	.	.	.	.	.		.	g
		1050.0	.	.	.	.	.	.	J	.	p
		50.00 (dead load)									

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ
J	3	Snow loads <1000m	0.50	0.20	0.00	1.50

Further design fundamentals:

Accuracy Gkn = 8.48e-5
Number of sub-element per member section: 6
Stress-strain-curve of concr. for deform. analy. EN 1992-1-1 3.1.5

```
Calc. of compr. force in concr.without deduction of reinf.
If n > -0.10 : eff EI acc.toEN2  7.4.2  (7.19)
Creep effects are considered by modified stress-strain-
curve.
φeff = φ0 * M0/Med (M0 By permanent combination with ei)
consequency class acc. EN 1990 tab b.1CC2 -> KFi = 1.0 (Tab B.3)

FLBemBn.DLL: version9.0.1.121
NKi/N =  2.03 dir_y  NKi/N =  2.03 dir_z  cross sect. of concr. only


CALCULATED COMBINATIONS by 1 Loads Kombi_D

Lc- Comb K1  K2
--------------------------------------------------------------------
          g   g
          J
   1      x   .

Partial safety factor    γC = 1.50  γS = 1.15    γG = 1.35 / 1.00

Proof according DIN EN 1992-1-1/NA/A1:2015-12
γC = 1.50  γS = 1.15  φeff = .80

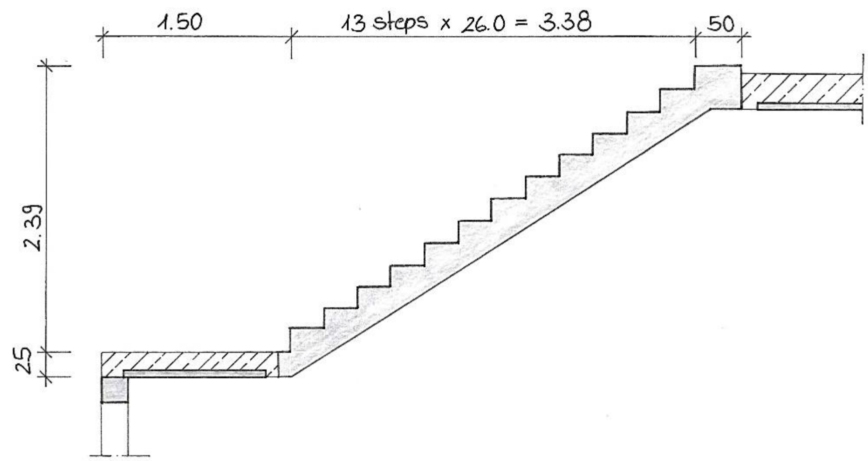
Design values    LcCom = 1   in :  y-direction    z-direction
--------------------------------------------------------------------
System                                    Displacable
Buckling length             sk  =     16.00         16.00 m
Slenderness                 λ   =    110.7         110.7
Normal force                N   =  -3358.35      -3358.35 kN
Specific normal force       n   =      -.68          -.68
Inter. moment  h =   .00 m ,  M =      0.00          0.00 kNm
Methodical  eccentr. e = M / N  =      0.00          0.00 cm
Related eccentr.   e/b and e/d  =    0.0000        0.0000
Unintentional eccentricity  ei  =      2.83          2.83 cm
Displacement Th.2.Ord.      e2  =     10.06         10.06 cm
Design moment            M des. =   -432.74       -432.75 kNm

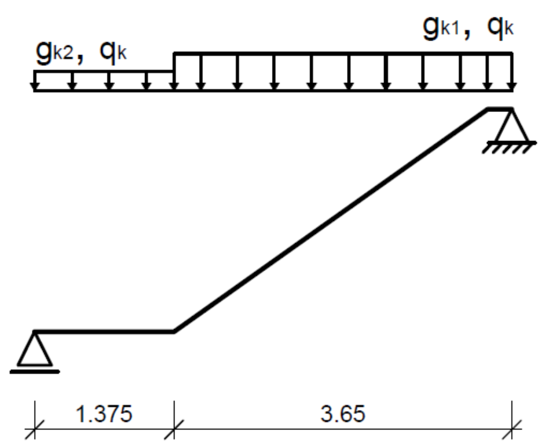
Reinforcement            tot ω  =     .9541
                             ρ  =      4.35 %
                         Req As =    108.81 cm2
The influence of creep will considered acc. EN 1992-1-1  5.8.4 curve.
```

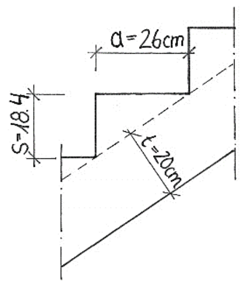
10 POS. T.10 – PRECAST STAIRCASE

10.1 SYSTEM

System







Cross-section	13 steps x 18.4/26 cm, t = 20 cm
Building materials	**C 30/37 | B 500A**
Exposure class	XC1, WO
Reinforcing steel	c_{nom} = 10 mm + 10 mm = 20 mm

Characteristic Loads

Uniformly Distributed Loads		g_k [kN/m]	q_k [kN/m]
Self-weight steps	$g_{k1,1}$ = ½ x 0.184 x 25 kN/m³ =	2.30	-
Self-weight slab	$g_{k1,2}$ = 0.20 x 25 kN/m³/cos(35°) =	6.10	-
Self-weight landing	$g_{k,2}$ = 0.25 x 25 kN/m³ =	6.25	
Floor structure	g_{k1+2} =	1.50	-
Live load (Cat. T1 – Stair cases)	q_{k1} =	-	3.00
	g_{k1} =	8.40	3.00
	g_{k2} =	7.75	

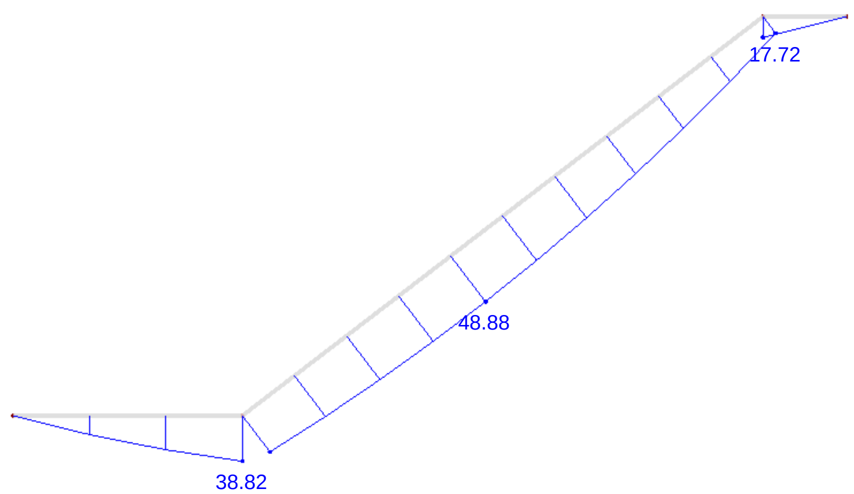
Design Loads

$$q_{Ed} = 1.35 \cdot g_k + 1.5 \cdot q_k$$

$$q_{perm} = 1.0 \cdot g_k + 1.0 \cdot \psi_2 \cdot q_k \qquad \psi_2 = 0.5 \text{ (Cat. T1)}$$

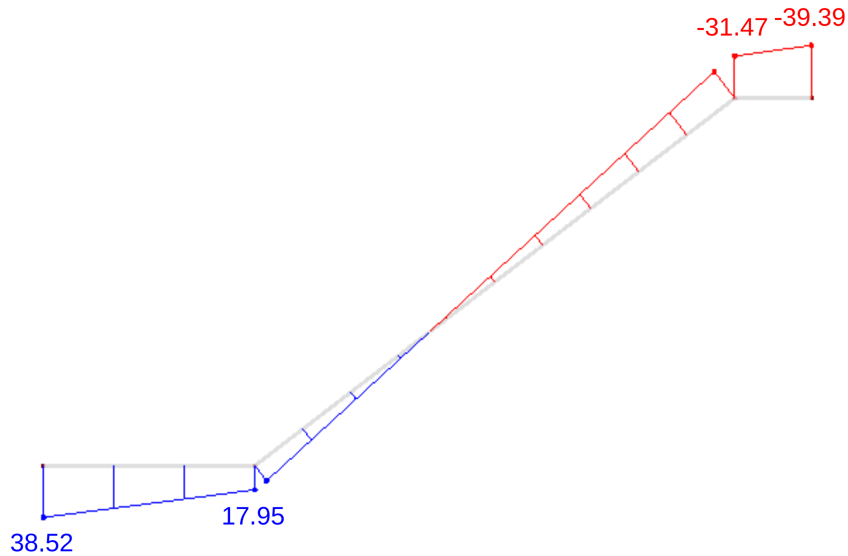
10.2 INTERNAL FORCES

Flexure moment



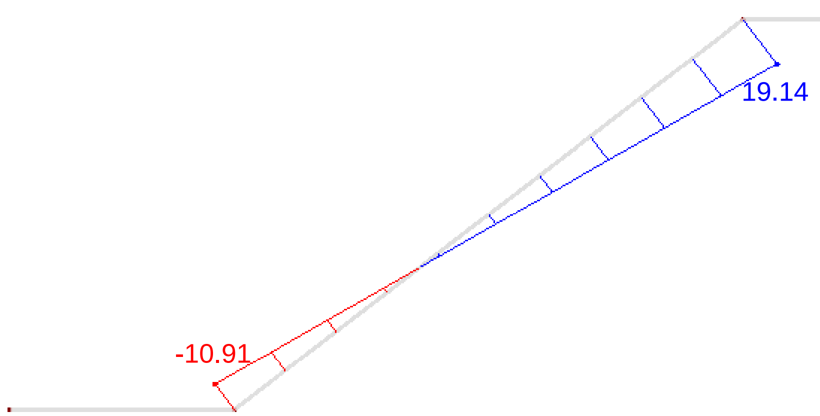
	m_{gk} [kNm/m]	m_{qk} [kNm/m]	m_{Ed} [ULS]	m_{perm} [SLS]
m_L	20.5	7.5	38.8	24.2
m_S	25.8	9.3	48.9	30.5

Shear force



	v_{gk} [kN/m]	v_{qk} [kN/m]	v_{Ed} [ULS]	v_{Ed} [SLS]
v_{max}	20.9	7.5	39.4	24.6

Normal force



	n_{gk} [kN/m]	n_{qk} [kN/m]	n_{Ed} [ULS]	n_{Ed} [SLS]
n_L	- 5.8	- 2.1	- 10.9	- 6.8
n_R	10.1	3.6	19.1	12.0

10.3 ULTIMATE LIMIT STATE DESIGN

10.3.1 FLEXURE DESIGN

Effective depth $d = h - c_{nom} - d_{s,quer} - d_{s,längs} / 2 = 20 - 2.0 - 1.0 - 1.0 / 2 = 16.5cm$

Reinforcement in the stairway $\mu_{Eds} = \dfrac{m_{Eds}}{b_{eff} \cdot d^2 \cdot f_{cd}} = \dfrac{4890kNcm}{100 \cdot 16.5^2 \cdot 1.7kN / cm^2} = 0.11$ → $\omega = 0.0728$

Longitudinal reinforcement $a_{sl,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \dfrac{1}{\sigma_{sd}} = 0.1170 \cdot 100 \cdot 16.5 \cdot 1.7 \cdot \dfrac{1}{43.5} = 7.54cm^2 / m$

Chosen **Ø 10/10 = 7.85 cm²/m** **longitudinal reinforcement**

Transverse reinforcement $a_{sq,req} = 0.2 \cdot a_{sl} = 0.2 \cdot 4.69 = 0.94cm^2 / m$

Chosen **Ø 8/25 = 2.01 cm²/m** **transverse reinforcement**

10.3.2 SHEAR DESIGN

Scale factor $k = 1 + \sqrt{\dfrac{20}{d}} = 1 + \sqrt{\dfrac{20}{16.5}} = 2.10 < \underline{\underline{2.00}}$

Percentage of reinforcement $\rho_1 = \dfrac{a_{sl}}{b_w \cdot d} = \dfrac{7.85}{100 \cdot 16.5} = 0.0048 \leq 0.02$

Shear force resistance $v_{Rd,c} = \left[0.10 \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp}\right] \cdot b_w \cdot d$

$$v_{Rd,c} = \left[0.10 \cdot 2.0 \cdot (100 \cdot 0.0048 \cdot 30)^{1/3}\right] \cdot 1.0 \cdot 0.165 \cdot 10^3 = 80.3kN / m$$

$$d = 16.5cm < 60cm \rightarrow v_{\min} = 0.035 \cdot 2.00^{3/2} \cdot (30)^{1/2} = 0.542MN / m^2$$

$$v_{Rd,c,\min} = 0.542 \cdot 1.00 \cdot 0.165 \cdot 10^3 = \underline{\underline{89.4kN}} / m > v_{Rd,c} = 80.3kN / m$$

$$v_{Ed} = 39.4kN / m < v_{Rd,c} = 89.4kN / m \checkmark$$

No further shear force reinforcement is needed!

10.4 SERVICEABILITY LIMIT STATE DESIGN

10.4.1 LIMITATION OF THE CRACK WIDTH

Maximum diameter of the reinforcement

Crack width w_k = 0.4 mm (according to DIN EN 1992-1-1, Tab. NA.7.1.)

Lever arm of the forces $\zeta = 0.940$ → $z = 0.940 \cdot 16.5 = 15.5cm$

Reinforcement stresses $\sigma_s = \dfrac{m_{Eds}}{z \cdot a_{sl}} = \dfrac{3050kNcm / m}{15.5 \cdot 7.85cm^2 / m} = 25.1kN / cm^2 = 251N / mm^2$

Limit diameter $d_s^* = 22.35$

$d_s = d_s^* \cdot f_{ct,eff} / 2.9 = 22.35 \cdot 1.0 = 22.35mm > d_{s,prov} = 10mm$ ✓

Minimum Reinforcement to Prevent Cracking

Tensile strength $f_{ct,eff} = f_{ctm} \geq 3.0MN/m^2$ → $f_{ct,eff} = 2.9MN/m^2 < \underline{\underline{3.0MN/m^2}}$

for $h < 30cm$ → $k = 0.80$

for flexure → $k_c = 0.4$

Tensile stress area $A_{ct} = b \cdot h / 2 = 100 \cdot 20 / 2 = 1000cm^2$

Reinforcement stresses $\sigma_s = 380MN/m^2$ (w_k = 0.4mm, d_s = 10mm)

Required Reinforcement $a_{s,\min,req} = k_c \cdot k \cdot f_{ct,eff} \cdot \frac{A_{ct}}{\sigma_s} = 0.4 \cdot 0.8 \cdot 3.0 \cdot \frac{1000}{380} = 2.53cm^2/m$

$< a_{s,prov} = 7.85cm^2/m$ ✓

The provided reinforcement is sufficient!

10.4.2 LIMITATION OF THE DEFORMATION

$l/d \leq K \cdot 35 = 1.0 \cdot 35$

$l/d_{prov} = 503/16.5 = 30.5 < 35$ ✓

The provided slab thickness is sufficient!

10.5 DESIGN AND REINFORCEMENT

10.5.1 MINIMUM REINFORCEMENT TO ENSURE ROBUSTNESS

$m_{cr} = f_{ctm} \cdot W_{cu}$ $W_{cu} = 100 \cdot 20^2 / 6 = 6667cm^3$

$m_{cr} = 2.9 \cdot 6667 \cdot 10^{-3} = 19.33kNm/m$

$a_{s,\min} = \frac{1}{f_{yk}} \cdot \frac{m_{cr}}{z} = \frac{1}{50.0} \cdot \frac{1933}{15.9} = 2.43cm^2/m > a_{s,prov} = 7.85cm^2/m$ ✓

The provided reinforcement is sufficient!

10.5.2 ANCHORAGE LENGTH AT THE SUPPORT

Global offset $a_l = 1.0 \cdot d = 16.5cm$

$F_{Ed} = \frac{V_{Ed} \cdot a_l}{z} = \frac{39.4 \cdot 16.5}{15.5} = 41.9kN$

$a_{s,req} = \frac{F_{Ed}}{f_{yd}} = \frac{41.9}{43.5} = 0.96cm^2/m$

Anchorage length $l_{b,rqd} = 36cm$ *for C30/37 and good bond*

$$l_{bd} = \alpha_1 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd} \cdot \left(a_{s,req} / a_{s,prov} \right) = 1.0 \cdot 36 \cdot (0.96 / 7.85) = 4.4cm$$

$$> l_{b,\min} = 0.3 \cdot 1.0 \cdot 36 = \underline{\underline{10.8cm}} > 10\phi_s = 10.0cm$$

$$l_{b,dir} = \tfrac{2}{3} \cdot l_{bd} = \tfrac{2}{3} \cdot 10.8 = \underline{\underline{7.2cm}} > 6.7\phi_s = 6.7cm$$

10.5.3 OVERLAP LENGTH

$$l_0 = \alpha_1 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \cdot \left(a_{s,req} / a_{s,prov} \right) = 1.0 \cdot 1.4 \cdot (0.96 / 7.85) \cdot 36 = 5.3cm$$

$$l_{0,\min} = 0.3 \cdot \alpha_1 \cdot \alpha_6 \cdot l_{b,rqd} = 0.3 \cdot 1.0 \cdot 1.4 \cdot 36 = 15.1cm > \begin{cases} 15\phi_s = 15cm \\ \underline{\underline{20cm}} \end{cases}$$

➔ $l_0 = 20cm$

10.5.4 DESIGN OF THE SUPPORT TRONSOLE

To improve the sound insulation of the stairs a the Tronsole® of the manufacturer *Schöck* has to be installed at the connection between the stairway and the upper slab. The sound insulating Tronsole® of the type T as can be seen in Figure 15.

Table 20 - Sizing Table for the Schöck Tronsole® Type T

Schöck Tronsole Typ	T-V2-NF	T-V4-NF	T-V6-NF	T-V7-NF	T-V8-NF
Bemessungswerte bei	Betonfestigkeit Podest ≥ C20/25, Treppenlauf ≥ C30/37				
Tronsole®-Höhe H [mm]	$V_{Rd,z}$ [kN/Element]				
160 - 170 (h_A ≥ 180 mm)	14,3	28,6	42,9	50,1	57,2
180 - 320	17,4	34,8	52,2	60,9	69,6
	$V_{Rd,y}$ [kN/Element]				
160 - 320	±1,6	±3,3	±5,0	±5,8	±6,6

Shear force at the support $v_{Ed} = 39.4kN/m$

Chosen **Schöck Tronsole® type T-V6-NF-H320-L1000**

Maximum shear force $v_{Rd} = 52.2kN/m > v_{Ed} = 39.4kN/m$ ✓ (see Table 20)

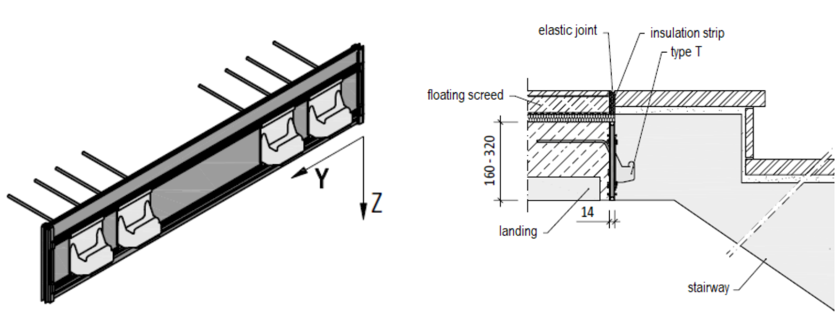
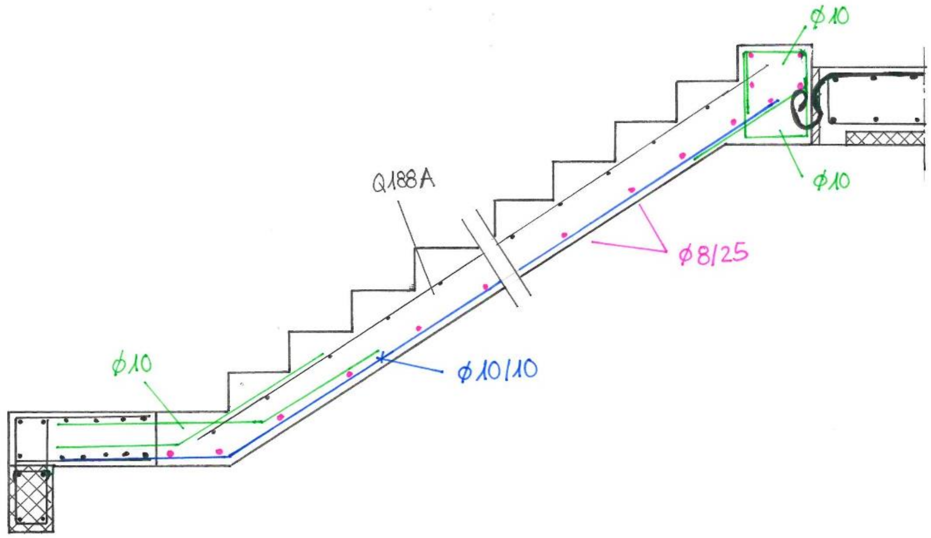


Figure 15 - Schöck Tronsole® Type T

10.5.5 REINFORCEMENT DRAWING



10.6 SOFTWARE CALCULATIONS

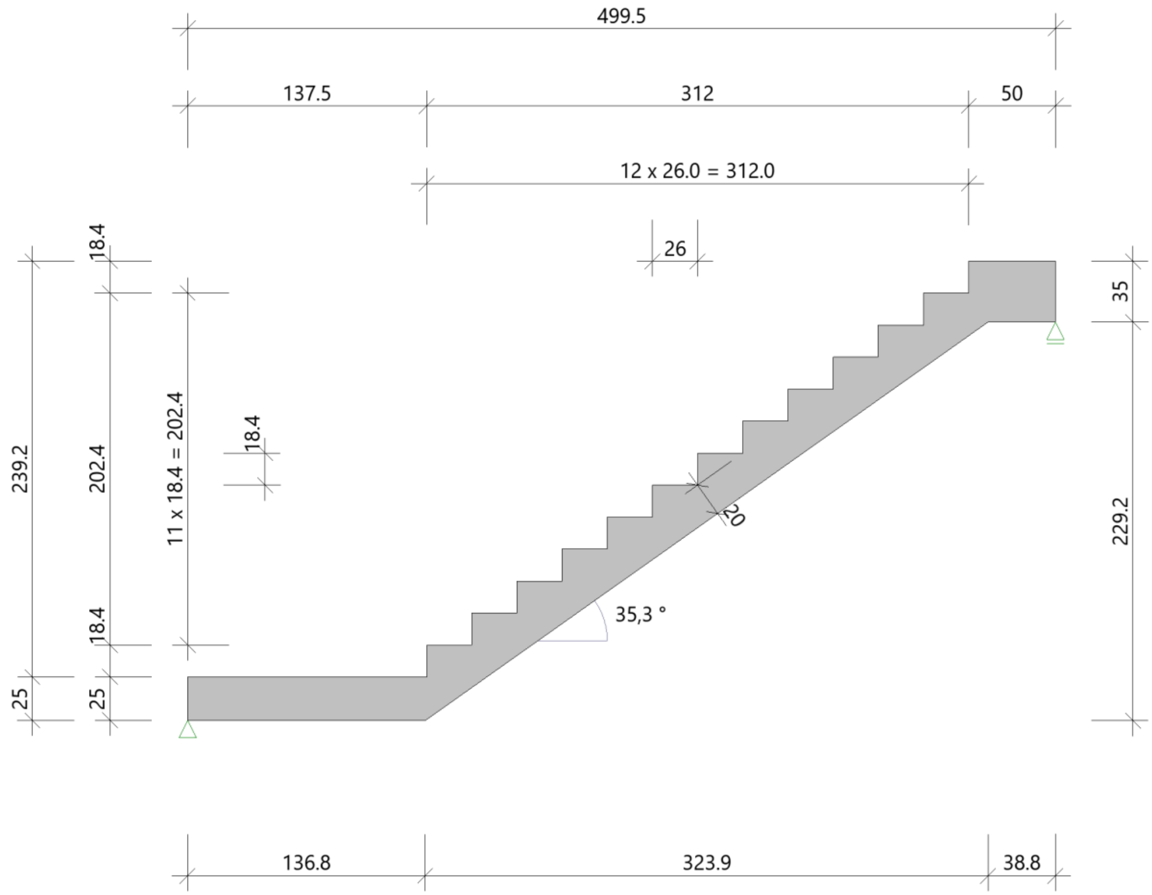
Demo Frilo Nemetschek

Position: T.10 – Precast Staircase

Treppenlauf B7+ 01/18A 01/2018 (Frilo R-2018-1/P12)

System

System graphics



Loads

Location -	Type -	g kN/m²	q kN/m²
Stairway	Covering	1.50	-
	Live load	-	3.00
Landing/console bottom	Covering	1.50	-
	Live load	-	3.00
Landing/console top	Covering	1.50	-
	Live load	-	3.00

Resulting loading (relative to the horizontal surface)

Location	Type	g	q
-	-	kN/m²	kN/m²
Stairway	Self weight	8.43	-
	Covering	1.50	-
	Live load	-	3.00
	Total	9.93	3.00
Landing/console bottom	Self weight	6.25	-
	Covering	1.50	-
	Live load	-	3.00
	Total	7.75	3.00
Landing/console top	Self weight	8.75	-
	Covering	1.50	-
	Live load	-	3.00
	Total	10.25	3.00

The dead weight is with gamma = 25.00 kN/m³ considered.

Results

Bending design

All design results per m stair width!

Flexural reinforcement

Location	d	M_{Ed}	N_{Ed}	req. a_{sb}	req. a_{st}	&About
-	cm	kNm/m	kN/m	cm²/m	cm²/m	-
lower landing, span reinforcement	25.0	42.48	0.0	4.4	0.0	
Stairway, span reinforcement	20.0	54.46	0.3	7.5	0.0	
upper landing, span reinforcement	35.0	6.73	0.0	4.1	0.0	*)

*) Minimum longitudinal reinforcement is decisive

exist. reinforcement

Stairway, span reinforcement 7 Ø 12 / 15.0 cm (Suggestion from program for number Ø)
exist. a_{sbtm} = 7.54 cm²/m

Shear design

Shear reinforcement B500A

Location	V_{Ed}	N_{Ed}	k_z	θ	a_{sL}	$V_{Rd,c}$	$V_{Rd,max}$	req. a_{sStir}	&About
-	kN/m	kN/m	-	Degree	cm^2/m	kN/m	kN/m	cm^2/m^2	-
lower landing left	41.2	0.0	0.82	18.4	0.0	115.1	688.5	9.3	*)
lower landing right	20.7	0.0	0.82	18.4	4.4	115.1	688.5	9.3	*)
Stairway left	16.9	-12.0	0.76	18.4	5.6	93.4	497.3		0.0
Stairway right	-31.4	22.3	0.76	18.4	2.9	89.9	497.3		0.0
upper landing left	-38.5	0.0	0.88	18.4	4.1	147.0	1071.0	9.3	*)
upper landing right	-44.3	0.0	0.88	18.4	0.0	147.0	1071.0	9.3	*)

*) Minimum shear reinforcement is decisive

crack width verification

The check is carried out with the quasi-permanent action combination

Crack width limitation stairs:

Location	d	M_{Ed}	N_{Ed}	exist. A_{sb}	exist. A_{st}	Env.Cl	$d_{s,exist}$	$d_{s,limit}$	exist. w	perm. w
-	cm	kNm	kN	cm^2	cm^2	-	mm	mm	mm	mm
Stairway, bottom side	20.0	32.75	0.1	7.9	0.0	XC1	12	24	0.20	0.40

Deformation

The calculation will be done with quasi permanent Action combination at state I (E_{cm} = 33000 N/mm²).

max. f = 0.4 cm (in staircase at x = 1.45 m)

Note: The deflection value is to be understood perpendicular to the corresponding member axis. The x-value refers to the beginning of the member (beginning lower platform, staircase or upper platform) and runs in the direction of the member axis.

11 POS. S.11 – SEMI-PRECAST DOUBLE-TEE SLAB

11.1 SYSTEM ALTERNATIVES

For the design of the slab of the office, a few possible solutions have been developed. The biggest issues of this slab are the large span length, 7.50 metres and 15.0 metres between the axes, as well as the opening in the slab above the entrance area. The two final possible solutions are described in the following chapters.

11.1.1 COMBINATION OF A SLAB-BEAM SYSTEM AND A FLAT SLAB

The first proposal is a combination of a slab-beam system and a flat slab between the axes 4 and 5, where the large opening is located. The original layout of the office includes additional columns at every 7.50 metres in axis A' as well as additional columns in the middle axis between A and A'. Beams span between each column along axis A, A' and the middle column row with span lengths of 7.50 m and 15.00 metres as can be seen in Figure 16. The slab spans between those beams creating a double-span system with a span length of 7.50 m. Because of the opening over the entrance, the beam system in axis A has to be interrupted. The walls next to axis 4 and 5 are used as additional support for the flat slab in-between. This system would require beams with fairly large cross-sections and a slab with a minimum thickness of 25 cm.

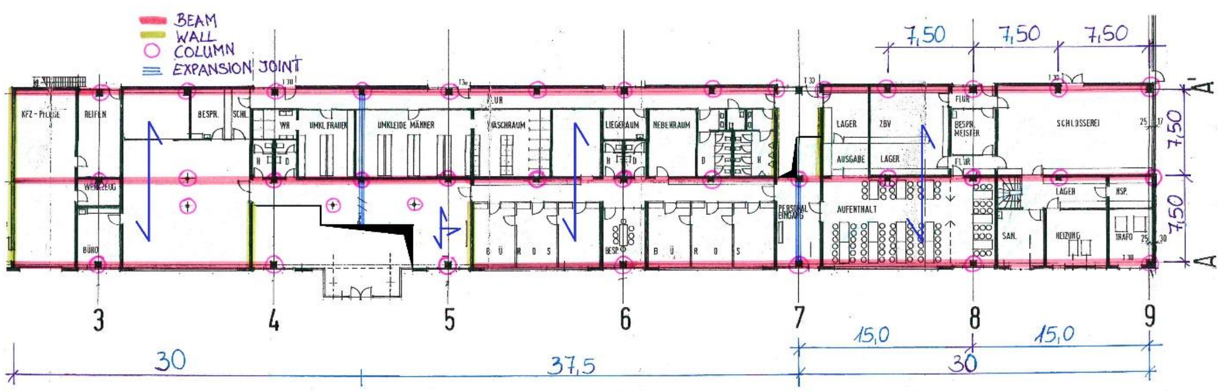


Figure 16 - Proposal of a Combination of a Slab-Beam System and a Flat Slab

11.1.2 COMBINATION OF A DOUBLE-TEE BEAM SLAB AND A TWO-WAY SLAB

Another possible solution is a combination of a double-tee beam system and a two-way slab as illustrated in Figure 17. The double-tee beams span between axis A and A'. That way large span lengths are possible and the column row in the middle, as well as the beams, are no longer necessary. Which creates more space and more flexibility on the ground floor of the office. In addition, instead of beams the concrete wall panels around the office are used as supports for the beams. Between axis 4 and 5, a two-way slab is used supported by the two load bearing walls next to the axes, a beam between those two and again the concrete wall panel in axis A'. The advantage of this system is that almost no columns and beams are needed inside the office area. Moreover, compared to the previous proposal, the construction of this slab system is easier and faster due to the high degree of prefabricated members. Therefore this system is chosen and will be analyzed in the following chapters.

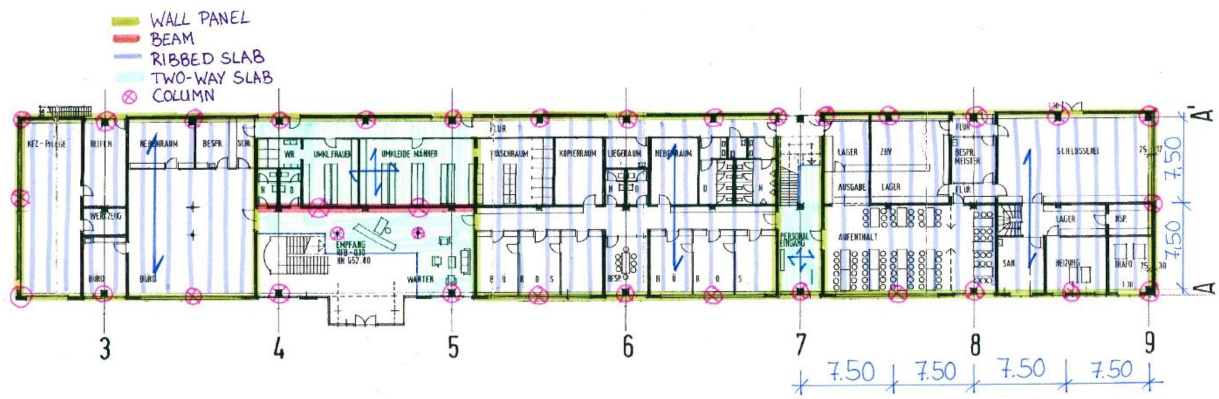
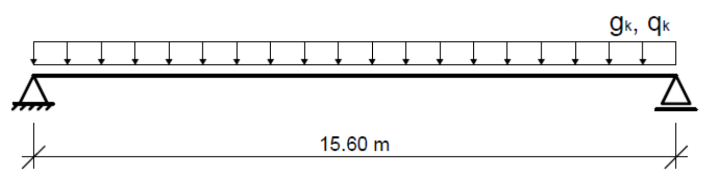


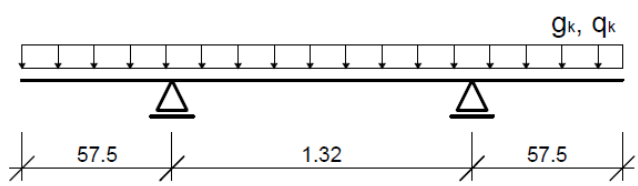
Figure 17 - Proposal of a Combination of a Double-Tee Slab System and A Two-Way Slab

11.2 SYSTEM

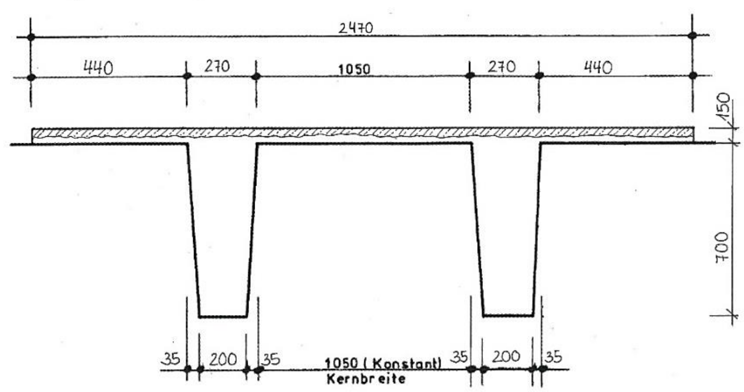
Longitudinal system



Transverse system



Cross-section



A_c [cm²]	$W_{c,u}$ [cm³]	I_y [cm⁴]	$z_{s,u}$ [cm]	$z_{s,o}$ [cm]	d [cm]
6,995	73,694	4,298,400	58.3	26.7	77.5

Building materials **C 35/45 | B 500A**

Exposure class XC1, WO

Reinforcing steel c_{nom} = 10 mm + 10 mm = 20 mm *Chosen:* c_{nom} = 25 mm

Characteristic loads

Uniformly Distributed Loads		**g_k [kN/m²]**	**q_k [kN/m²]**
Self-weight	g_{k1} = 0.6995 m² x 25 kN/m³ =	17.5	-
Floor structure	g_{k2} =	2.0	-
Live load (Cat. B2 – Office Building)	q_{k1} =	-	2.0
Load from partition walls	q_{k2} =		1.2
		19.5	3.2

11.3 INTERNAL FORCES

11.3.1 LONGITUDINAL DIRECTION

Loads in longitudinal direction

$$g_k = 17.5 + 2.47m \cdot 2.0 = 22.5kN/m$$

$$q_k = 2.47m \cdot 3.2 = 7.9kN/m$$

$$q_{Ed} = 1.35 \cdot g_k + 1.50 \cdot q_k = 42.2kN/m \quad \text{(ULS)}$$

$$q_{perm} = g_k + 0.3 \cdot q_k = 24.9kN/m \quad \text{(SLS)}$$

Flexure moment

M_{gk} [kNm]	M_{qk} [kNm]	M_{Ed} [ULS]	M_{perm} [SLS]
684.5	240.3	1283.7	757.5

Shear force

V_{gk} [kN]	V_{qk} [kN]	V_{Ed} [ULS]	V_{perm} [SLS]
175.5	61.6	329.2	194.2

11.3.2 TRANSVERSE DIRECTION

Loads in transverse direction

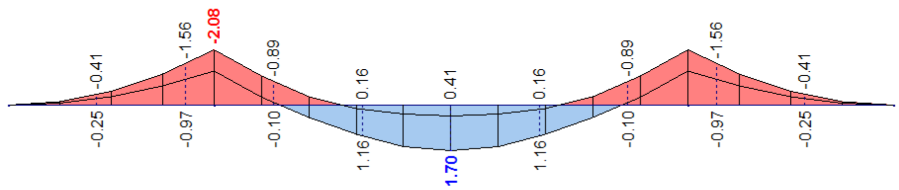
$$g_k = 0.15 \cdot 25kN/m^3 + 2.0 = 5.75kNm/m$$

$$q_k = 3.20kNm/m$$

$$q_{Ed} = 1.35 \cdot g_k + 1.50 \cdot q_k = 12.6kN/m \quad \text{(ULS)}$$

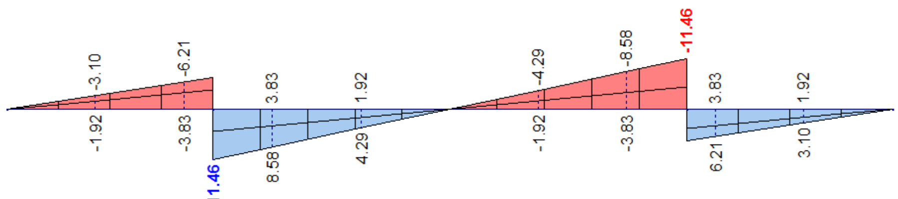
$$q_{perm} = g_k + 0.3 \cdot q_k = 6.7kN/m \quad \text{(SLS)}$$

Flexure moment



	m_{gk} [kNm/m]	m_{qk} [kNm/m]	m_{Ed} [ULS]	M_{perm} [SLS]
Midspan	0.30	0.70	1.70	0.51
Support	- 0.95	- 0.53	- 2.08	- 1.11

Shear force



v_{gk} [kN/m]	v_{qk} [kN/m]	v_{Ed} [ULS]	V_{perm} [SLS]
3.79	2.11	11.46	4.42

11.4 ULTIMATE LIMIT STATE DESIGN

11.4.1 FLEXURE DESIGN

Longitudinal direction – Design of the T-Beam

Effective width

$$b_{eff,1} = 0.2 \cdot 44 + 0.1 \cdot 1560 = 164.8cm \; < \; 0.2 \cdot 1560 = 312cm$$

$$< \; b_1 = \underline{\underline{44cm}}$$

$$b_{eff,2} = 0.2 \cdot 52.5 + 0.1 \cdot 1560 = 160.5cm \; < \; 0.2 \cdot 1560 = 312cm$$

$$< \; b_2 = \underline{\underline{52.5cm}}$$

$$b_{eff} = 2 \cdot (b_{eff,1} + b_w + b_{eff,2}) = \underline{\underline{2.47m}}$$

Longitudinal reinforcement

Compression zone at the height of the in-situ concrete → C 20/25!

$$\mu_{Eds} = \frac{M_{Eds}}{b_{eff} \cdot d^2 \cdot f_{cd}} = \frac{128370kNcm}{247 \cdot 77.5^2 \cdot 1.13kN / cm^2} = 0.08 \quad \rightarrow \; \omega = 0.0836$$

$$\xi = 0.107 \quad \rightarrow \; x = 0.107 \cdot 77.5 = 8.3cm < h_f = 15cm \; \checkmark$$

$$A_{sl,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = 0.0836 \cdot 247 \cdot 77.5 \cdot 1.13 \cdot \frac{1}{43.5} = 41.6cm^2$$

Chosen **2 x 5 Ø 25 = 49.1 cm²** **longitudinal reinforcement**

New static height $d = h - c_{nom} - d_{s,bü} - d_{s,lä} / 2 = 85 - 2.0 - 1.0 - 3.25 = 78.75cm$

Transverse direction – Design of the slab

Static height $d = h - c_{nom} - d_{s,quer} - d_{s,längs} / 2 = 15 - 2.0 - 1.0 - 1.0 / 2 = 11.5cm$

Reinforcement at Support $\mu_{Eds} = \frac{m_{Eds}}{b_{eff} \cdot d^2 \cdot f_{cd}} = \frac{208kNcm}{100 \cdot 11.5^2 \cdot 1.98kN / cm^2} = 0.008$ → $\omega = 0.0101$

$$a_{so,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = 0.0101 \cdot 100 \cdot 11.5 \cdot 1.98 \cdot \frac{1}{43.5} = 0.53cm^2 / m$$

Reinforcement at Midspan Compression zone at the height of the in-situ concrete → C 20/25!

$$\mu_{Eds} = \frac{m_{Eds}}{b_{eff} \cdot d^2 \cdot f_{cd}} = \frac{170kNcm}{100 \cdot 11.5^2 \cdot 1.13kN / cm^2} = 0.011 \rightarrow \omega = 0.0101$$

$$a_{so,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = 0.0101 \cdot 100 \cdot 11.5 \cdot 1.98 \cdot \frac{1}{43.5} = 0.53cm^2 / m$$

Chosen **Q188A** **upper transverse reinforcement**

Q188A **lower transverse reinforcement**

11.4.2 SHEAR DESIGN

Longitudinal direction – Design of the T-Beam

Design location $x = a_i + d = 0.1 + 78.8 = 79.8cm$

Shear force at design location $V_{Ed,x} = 329.2 - 0.798 \cdot 42.2 = 295.5kN$

Scale factor $k = 1 + \sqrt{\frac{20}{d}} = 1 + \sqrt{\frac{20}{78.8}} = 1.50 < \underline{\underline{2.00}}$

Percentage of reinforcement $\rho_1 = \frac{A_{sl}}{b_w \cdot d} = \frac{49.1}{40 \cdot 78.8} = 0.016 \leq \underline{\underline{0.02}}$

Shear force resistance $V_{Rd,c} = \left[0.10 \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp}\right] \cdot b_w \cdot d$

$$V_{Rd,c} = \left[0.10 \cdot 1.5 \cdot (100 \cdot 0.016 \cdot 35)^{1/3}\right] \cdot 0.78 \cdot 0.40 \cdot 10^3 = 179kN$$

$$d = 78.8cm > 80cm \rightarrow \nu_{\min} = 0.025 \cdot 1.50^{3/2} \cdot (35)^{1/2} = 0.272MN / m^2$$

$$V_{Rd,c,\min} = 0.272 \cdot 0.40 \cdot 0.788 \cdot 10^3 = 85.7kN < V_{Rd,c} = \underline{\underline{179kN}}$$

$$V_{Ed} = 295.5kN > V_{Rd,c} = 179kN$$

➔ Shear reinforcement is needed!

Lever arm of the internal forces $z = \zeta \cdot d = 0.956 \cdot 78.8 = 75.3cm$

Friction Force: $V_{Rd,cc} = 0.24 \cdot (35)^{1/3} \cdot 0.4 \cdot 0.753 \cdot 10^3 = 236.5kN$

Inclination: $1.0 \leq \cot\theta = \dfrac{1.2}{1 - V_{Rd,cc} / V_{Ed}} = \dfrac{1.2}{1 - 236.5 / 295.5} = 6.0 \leq \underline{\underline{3.0}}$

Vertical stirrups $\alpha = 90°$

Required reinforcement $a_{sw,req} = \dfrac{295.5kN}{43.5 \cdot 0.753 \cdot 3.0} = 3.0cm^2 / m$

Chosen **Ø 8 / 20 = 5.03 cm²/m** **stirrups**

Resistance of the Compression Struts

$$V_{Rd,\max} = 0.75 \cdot 40 \cdot 75.3 \cdot 1.98 \cdot \frac{3.00}{1 + 3.00^2} = 1342kN$$

$V_{Ed,\max} = 329.2kN < V_{Rd,\max} = 1342kN$ → The compression strut is ok.

Minimum shear reinforcement $\min a_{sw} = \rho \cdot b_w \cdot \sin\alpha$ and $\rho = 0.16 \cdot f_{ctm} / f_{yk} = 0.16 \cdot 3.2 / 500 = 1.02‰$

$$\min a_{sw} = 0.00102 \cdot 0.40 \cdot \sin 90° = 4.08 \cdot 10^{-4} m^2 / m = \underline{\underline{4.08cm^2 / m}}$$

$$\min a_{sw} = 4.08cm^2 / m < a_{sw,prov} = 5.03cm^2 / m \checkmark$$

Transverse direction – Design of the slab (C20/25)

Scale factor $k = 1 + \sqrt{\dfrac{20}{d}} = 1 + \sqrt{\dfrac{20}{11.5}} = 2.32 < \underline{\underline{2.00}}$

Percentage of reinforcement $\rho_1 = \dfrac{a_{sl}}{b_w \cdot d} = \dfrac{2.01}{100 \cdot 11.5} = 0.0017 \leq 0.02$

Shear force resistance $v_{Rd,c} = \left[0.10 \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp}\right] \cdot b_w \cdot d$

$$v_{Rd,c} = \left[0.10 \cdot 2.0 \cdot (100 \cdot 0.0017 \cdot 20)^{1/3}\right] \cdot 1.0 \cdot 0.115 \cdot 10^3 = 34.6kN / m$$

$$d = 11.5cm < 60cm \rightarrow v_{\min} = 0.035 \cdot 2.00^{3/2} \cdot (20)^{1/2} = 0.443MN / m^2$$

$$v_{Rd,c,\min} = 0.443 \cdot 1.00 \cdot 0.115 \cdot 10^3 = \underline{\underline{50.9kN}} / m > v_{Rd,c} = 34.6kN / m$$

$$v_{Ed} = 11.5kN / m < v_{Rd,c} = 50.9kN / m \checkmark$$

No further shear force reinforcement is needed!

11.5 SERVICEABILITY LIMIT STATE DESIGN

11.5.1 LIMITATION OF THE CRACK WIDTH

Maximum diameter of the reinforcement

Crack width $w_k = 0.4$ mm (according to DIN EN 1992-1-1, Tab. NA.7.1.)

Lever arm of the forces $z = \zeta \cdot d = 0.956 \cdot 78.8 = 75.3cm$

Reinforcement stresses $\sigma_s = \frac{M_{perm}}{z \cdot A_{sl}} = \frac{75750kNcm}{75.3 \cdot 49.1cm^2} = 20.5kN / cm^2 = 205N / mm^2$

Limit diameter $d_s^* = 33.63$

$d_s = d_s^* \cdot f_{ct,eff} / 2.9 = 33.63 \cdot 3.2 / 2.9 = 37.1mm > d_{s,prov} = 25mm$ ✓

Minimum reinforcement to prevent cracking

Tensile strength $f_{ct,eff} = f_{ctm} \geq 3.0MN / m^2$ → $f_{ct,eff} = 3.2MN / m^2 > \underline{\underline{3.0MN / m^2}}$

for $h > 80cm$ → $k = 0.50$

for flexure → $k_c = 0.4$

Tensile stress area $A_{ct} = b \cdot h / 2 = 40 \cdot 85 / 2 = 1700cm^2$

Reinforcement stresses $\sigma_s = 236MN / m^2$ (w_k = 0.4mm, d_s = 25mm)

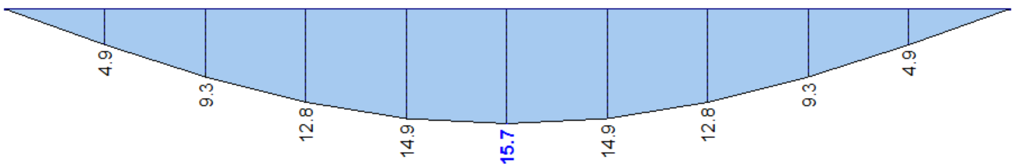
Required Reinforcement $A_{s,min,req} = k_c \cdot k \cdot f_{ct,eff} \cdot \frac{A_{ct}}{\sigma_s} = 0.4 \cdot 0.5 \cdot 3.2 \cdot \frac{1700}{236} = 4.6cm^2$

$< A_{s,prov} = 49.1cm^2$ ✓

The provided reinforcement is sufficient!

11.5.2 DEFORMATION OF THE SLAB

Deformation of the Slab under q_{perm}



$f_{max} = 15.7mm < l / 250 = 15600 / 200 = 62.4mm$ ✓

The deformation does not exceed the maximum allowed deformation!

11.6 DESIGN AND REINFORCEMENT

11.6.1 MINIMUM REINFORCEMANT TO ENSURE ROBUSTNESS

Longitudinal direction – Design of the T-Beam

$M_{cr} = f_{ctm} \cdot W_{cu}$ $\qquad W_{cu} = 73{,}694 cm^3$

$M_{cr} = 3.2 \cdot 73{,}694 \cdot 10^{-3} = 235.8 kNm$

$$A_{s,\min} = \frac{1}{f_{yk}} \cdot \frac{M_{cr}}{z} = \frac{1}{50.0} \cdot \frac{23580}{75.3} = 6.3 cm^2 < A_{s,prov} = 49.1 cm^2 \ \checkmark$$

Transverse direction – Design of the slab

$M_{cr} = f_{ctm} \cdot W_{cu}$ $\qquad W_{cu} = 100 \cdot 15^2 / 6 = 3750 cm^3$

$M_{cr} = 2.2 \cdot 3750 \cdot 10^{-3} = 8.25 kNm / m$

$$a_{s,\min} = \frac{1}{f_{yk}} \cdot \frac{M_{cr}}{z} = \frac{1}{50.0} \cdot \frac{8.25}{11.4} = 0.02 cm^2 / m < a_{s,prov} = 2.01 cm^2 / m \ \checkmark$$

11.6.2 DESIGN OF THE INTERCONNECTING JOINT

Design location $\qquad x = 0.1 + 0.115 - 0.09 = 0.125 m$

Shear force $\qquad V_{Ed,x} = 329.2 - 0.125 \cdot 42.2 = 323.9 kN$

$$\upsilon_{Ed,i} = \frac{323.9}{247 \cdot 11.4} = 0.115 kN / cm^2$$

Rough joint $\qquad c = 0.4 \qquad \mu = 0.7 \qquad \nu = 0.5$

Maximum shear force $\qquad \upsilon_{Rdi,c} = c \cdot 0.12 \cdot (f_{ck})^{2/3} = 0.4 \cdot 0.12 \cdot (20)^{2/3} \cdot 10 = 3.54 kN / cm^2$

$> \upsilon_{Ed,i} = 0.115 kN / cm^2 \ \checkmark$

No reinforcement at the interconnection joint necessary!

11.6.3 CONNECTION OF THE COMPRESSION FLANGE

Design location $\qquad \Delta x = 1/2 \cdot l/2 = 15.6/4 = 3.90 m$

Flexure moment $\qquad M_{Ed}(x = 3.9) = 329.2 \cdot 3.9 - 42.2 \cdot {}^{3.9^2}\!/_{2} = 962.9 kNm = \Delta M_{Ed}$

Bond force in the joint $\qquad v_{Ed} = \dfrac{\Delta M_{Ed}}{z \cdot h_f \cdot \Delta x} \cdot \dfrac{b_a}{b_{eff}} = \dfrac{962.9}{0.735 \cdot 0.15 \cdot 3.90} \cdot \dfrac{97.5}{247} = 884 kN / m^2$

Shear force resistance $\qquad \cot\theta = 1.2$

$$v_{Rd,\max} = \frac{0.75 \cdot f_{cd}}{\cot\theta + \tan\theta} = \frac{0.75 \cdot 11.3}{1.2 + 1/1.2} \cdot 10^3 = 4170 kN / m^2 > v_{Ed} = 884 kN / m^2$$

Transverse reinforcement $a_{sw,req} = \frac{v_{Ed} \cdot h_f}{f_{yd} \cdot \cot\theta} = \frac{884 \cdot 0.15}{43.5 \cdot 1.2} = 2.54 cm^2 / m$

Chosen	**Q188A**	**upper reinforcement**
	Q188 A	**lower reinforcement**

11.6.4 REINFORCEMENT DRAWING

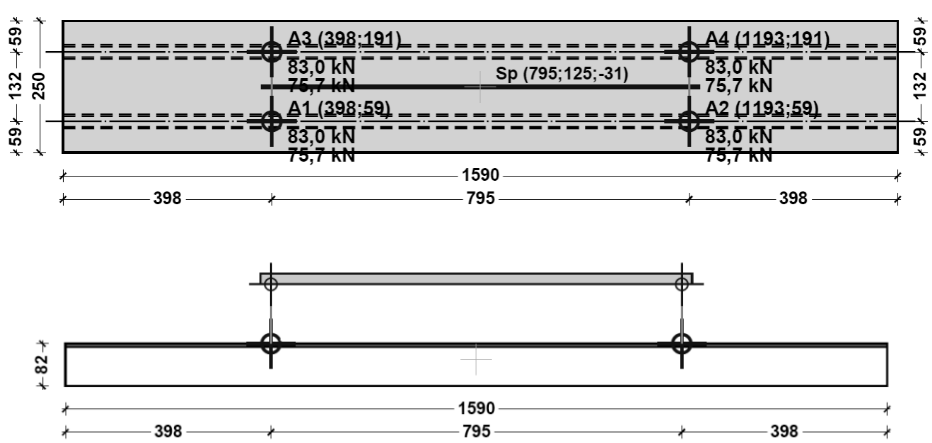
See Appendix A.5, page 191.

11.7 TRANSPORT AND ASSEMBLY

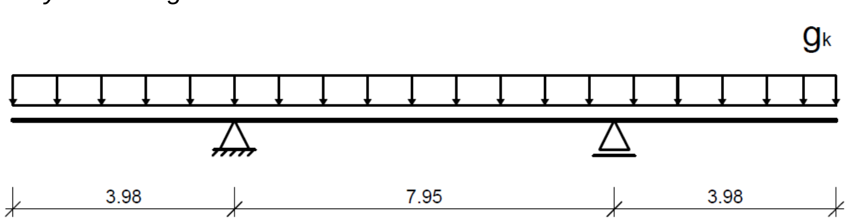
11.7.1 TRANSPORT TO THE SITE

Each Double-Tee Slab will be suspended with the help of four anchors evenly placed across from each other.

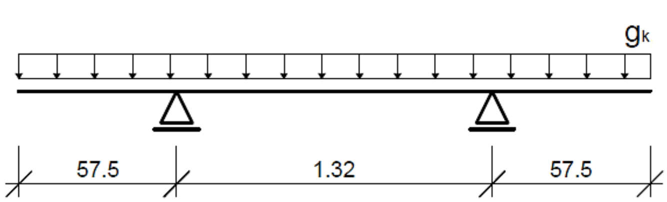
System



Static System Longitudinal Direction



Static System Transverse Direction



Design in Longitudinal Direction

Self-weight — *V = 0.70 m³/m*

g_k = 0.70 m³ x 25 kN/m³ = 17.5 kN/m

Flexure Moment — $M_{Ed}^{G} = 1.15 \cdot (-17.5) \cdot 3.98^2 / 2 = -159.4 kNm$

Time of first lifting — approximately after 5 days (t = 5d)

Compressive strength at time t

$$f_{cm}(t) = \beta_{cc}(t) \cdot f_{cm} \quad \rightarrow \quad \beta_{cc}(t = 5d) = e^{s\left(1-\sqrt{28/t}\right)} = e^{0.2\left(1-\sqrt{28/5}\right)} = 0.76$$

$$f_{cm}(t = 5d) = 0.76 \cdot 43 = 32.68 N / mm^2$$

$$f_{ck}(t = 5d) = f_{cm}(t) - 8 = 32.68 - 8 = 24.68 N / mm^2$$

$$f_{cd}(t = 5d) = 0.85 \cdot f_{ck}(t) / 1.5 = 0.85 \cdot 24.68 / 1.5 = \underline{\underline{14.0 N / mm^2}}$$

Effective depth — *d ≈ 76 – 5 = 71 cm*

Required Reinforcement

$$\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{15940}{248 \cdot 71^2 \cdot 1.4} = 0.01 \quad \rightarrow \quad \omega = 0.0101$$

$$A_{s2,req} = (\omega \cdot b \cdot d \cdot f_{cd} + N_{Ed}) / \sigma_{sd}$$

$$= 0.0101 \cdot 248 \cdot 71 \cdot 1.40 / 43.5 = \underline{\underline{5.72 cm^2}}$$

Chosen — **3 Ø 16 = 6.03 cm²** — **upper reinforcement**

Design in Transverse Direction

Load on flange — *g_k = 0.15 m x 25 kN/m³ = 3.75 kN/m/m*

Flexure moment — $M_{Ed}^{G} = 1.15 \cdot (-3.75) \cdot 0.575^2 / 2 = -0.71 kNm / m$

Effective depth — *d ≈ 4 cm*

Required reinforcement

$$\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{71}{100 \cdot 4^2 \cdot 1.4} = 0.03 \quad \rightarrow \quad \omega = 0.0306$$

$$a_{s2,req} = (\omega \cdot b \cdot d \cdot f_{cd} + N_{Ed}) / \sigma_{sd}$$

$$= 0.0306 \cdot 100 \cdot 4 \cdot 1.40 / 43.5 = \underline{\underline{0.39 cm^2}} / m$$

11.7.2 TRANSPORT ANCHORS

In order to lift the double-tee slab out of its formwork as well as to be able to transport and assemble it on site, transport anchors are needed. Those are installed during the production of the beam and must be designed in accordance with the VDI/BV-BS-Richtlinie 6205. For this project the anchors of the company *Halfen* are chosen.

Self-weight — $F_G = (17.5 kN / m - 0.08 \cdot 25 kN / m^3) \cdot 15.9 m = 246.5 kN$

Formwork adhesion — $F_{adh} = q_{adh} \cdot A_f$ → no adhesion here because of hinged formwork!

Dynamic factor $\psi_{dyn} = 1.3$

Factor for inclination $z = 1.16 \ \ for\ \beta = 30°$

Tensile force $F_z = \dfrac{F_G \cdot \psi_{dyn} \cdot z}{n} = \dfrac{246.5 \cdot 1.3 \cdot 1.16}{4} = \underline{\underline{92.9kN}}$

Chosen **HALFEN DEHA Spherical head anchor 6000-10,0-0340**

Load capacity of the anchor $F_{z,Rd} = 92.9kN > F_{z,Ed} = 98.9kN$ ✓ (see Table 21)

Table 21 - Sizing Table for 'HALFEN DEHA Lifting Anchor Systems'

Spherical head anchors in beams and walls with no special requirements on the reinforcement (load class 1,3 – 7,5)									
Load class	Article number	Anchor length l [mm]	Minimum height of beams B_1 [mm]	Wall thickness $2 \times e_r$ [mm]	Load capacity [kN] at concrete strength f_{ci} for: **Axial pull** up to 30° [β] 15 N/mm²	**Diagonal pull** up to 60° [β] 15 N/mm²	**Axial pull** and **diagonal pull** up to 60° [β] 25 N/mm²	**Axial pull** and **diagonal pull** up to 60° [β] 35 N/mm²	Axial spacing of anchors e_z [mm]
10,0	6000-10,0-0170	170	340	300	46.4	37.2	60.0	70.9	520
				350	52.1	41.7	67.3	79.6	
				400	57.6	46.1	74.4	88.0	
	6000-10,0-0340	340	680	280	76.6	61.3	98.9	100.0	1030
				300	80.7	64.5	100.0		
				320	84.7	67.7			
	6000-10,0-0680	680	1360	160	73.7	70.0	95.2	100.0	2050
				180	83.0	76.5	100.0		
				200	92.2	82.8			

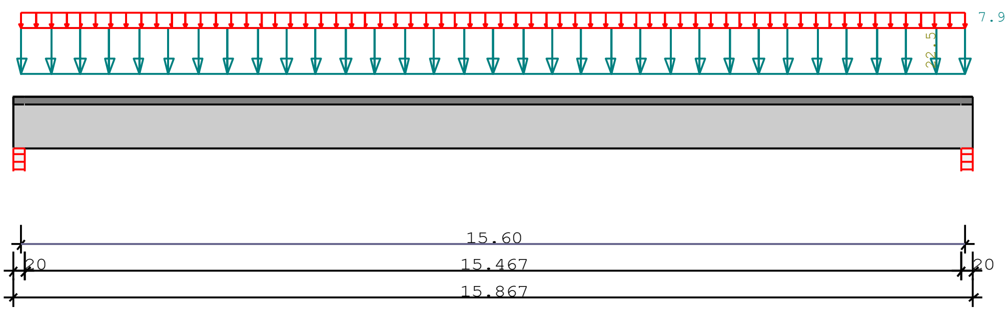
11.8 SOFTWARE CALCULATIONS

Demo Frilo Nemetschek

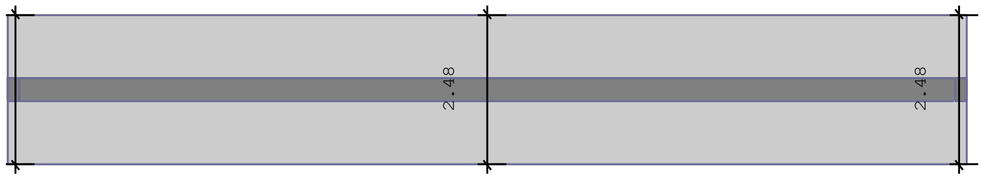
Position: S.11 – Semi-Precast Double-Tee Slab

Durchlaufträger DLT10 01/2018 (Frilo R-2018-1/P12)

Scale 1 : 125



effective breadth and design



Results for 1-times loads

Span moments maximum (kNm , kN)

Span		Mf	M le	M ri	V le	V ri	comb
1	x0 = 7.80	924.77	0.00	0.00	237.12	-237.12	2

Support moments maximum (kNm , kN)

Column	M le	M ri	V le	V ri	max F	min F	comb
1	0.00	0.00	0.00	237.12	237.12	175.50	2
2	0.00	0.00	-237.12	0.00	237.12	175.50	2

Moment boundary diagram

x/L =	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
Span											
1	0.00	246	438	575	657	684	657	575	438	246	0.00
1	0.00	333	592	777	888	925	888	777	592	333	0.00

Support reactions (kN)

Column	by g	max q	min q	Fulload	max	min
1	175.50	61.62	0.00	237.12	237.12	175.50
2	175.50	61.62	0.00	237.12	237.12	175.50
Total:	351.00	123.24	0.00	474.24	474.24	351.00

```
S
upport reactions                                                   ( kN )
--------------------------------------------------------------------
       Column 1         Column 2
CA      max      min      max      min
 g     175.5    175.5    175.5    175.5
 A      61.6      0.0     61.6      0.0
--------------------------------------------------------------------
tot    237.1    175.5    237.1    175.5

Deflections calculated according to uncracked concrete!

Deflections        maximum                    minimum
--------------------------------------------------------------------
Span No.       x (m)       f (cm)  comb     x (m)      f(cm)  comb
   1            7.80      1.68     2        0.00      0.00     0

Results for γ-times loads
Partial safety factor γG * KFi = 1.35 constant over whole girder length

Span moments maximum                                      ( kNm , kN )
--------------------------------------------------------------------
Span                 Mfd      Mdle      Mdri      V le      V ri     comb
  1    x0 =  7.80  1284.48     0.00      0.00    329.35   -329.36    A  2


Support moments maximum                                   ( kNm , kN )
--------------------------------------------------------------------
Support      Mdle      Mdri      Vdle      Vdri      max F     min F    comb
  1          0.00      0.00      0.00    329.35    329.35    175.50    A  2
  2          0.00      0.00   -329.35      0.00    329.36    175.50    A  2


Moment boundary diagram
--------------------------------------------------------------------
x/L = .0     .1     .2     .3     .4     .5     .6     .7     .8     .9    1.0
Span
  1   0.00    246    438    575    657    684    657    575    438    246   0.00
  1   0.00    462    822   1079   1233   1284   1233   1079    822    462   0.00



Scale 1 : 150
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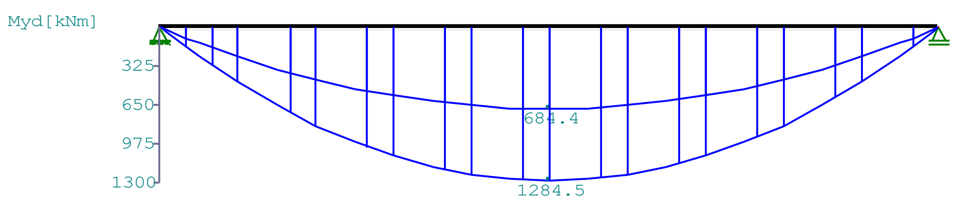
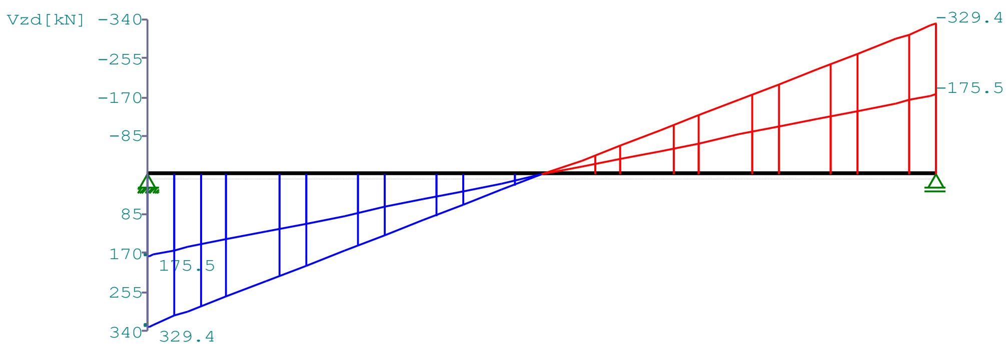




Minimum reinforcement EN2 9.2.1.1 (9.1) fctm = 3.21 N/mm2
Calculating Wy, breadth of slab is limited to 2*b0.

Q.No.	min Mb (kNm)	req As (cm2)	min Mo (kNm)	req As (cm2)	
1	222.06	6.25	-303.60	8.33	120.0/15.0/40.0/85.0

Span reinforcement

Span No.	x (m)	Myd (kNm)	min Myd (kNm)	d (cm)	kx	Asb (cm2)	Ast (cm2)	comb
1	7.80	1284.5		79.0	0.07	36.5	0.0	A 2

On first support are at least 11.3 cm2 to be anchored.
On last support are at least 11.3 cm2 to be anchored.
Shear force VK-support is with F = V,Ed * Cot(Theta) / 2 considered.

shear force reinforcement B500A DIN EN 1992-1-1/NA/A1:2015-12 6.2

column No.	dist (m)	kz	VEd (kN)	Θ (°)	VRd,c (kN)	VRd,max (kN)	a_max (cm)	asw (cm2/m)	comb
1 ri	0.86	0.92	293.2	18.4	163.1	1297.7	30.0	4.1~	A 2
1 *	1.65	0.92	259.8	18.4	163.1	1297.7	30.0	4.1~	A 2
2 le	0.86	0.92	-293.2	18.4	163.1	1297.7	30.0	4.1~	A 2
2 *	1.65	0.92	-259.8	18.4	163.1	1297.7	30.0	4.1~	A 2

~ at the end of line: Minimum stirrup reinforcement
max distance of stirrups will with Θ >= 40° investigated (paper DAfStb 525).

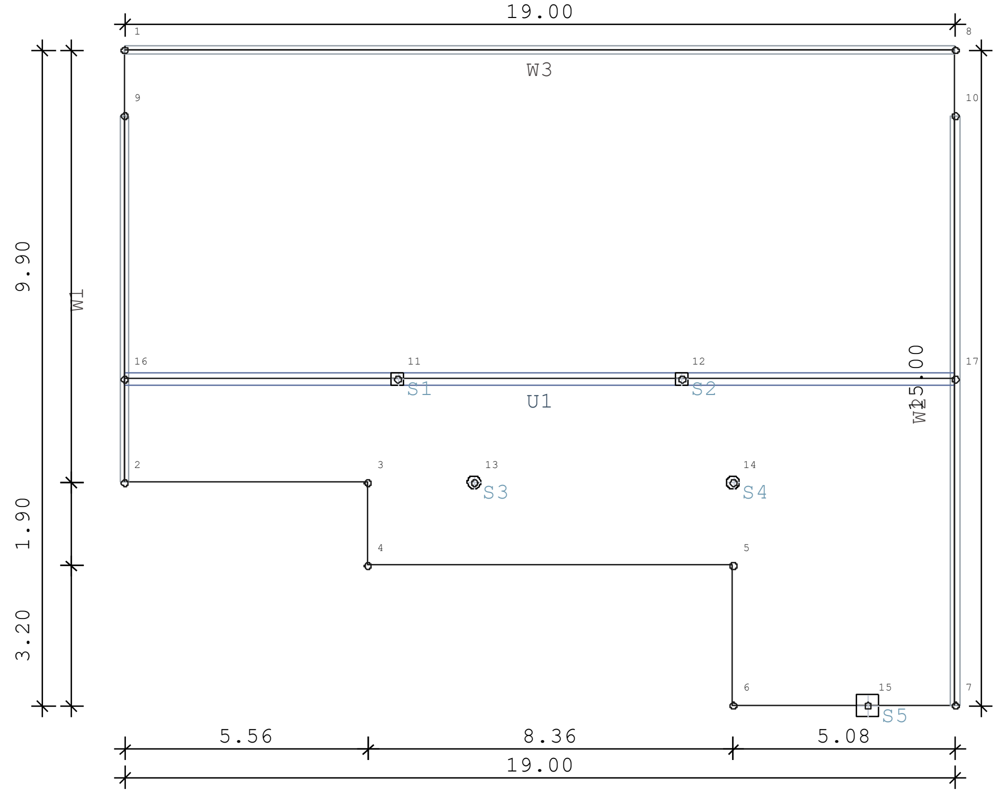
shoulder shear

Span	xa (cm)	xe (cm)	mle (kNm)	mri (kNm)	av (cm)	beff (cm)	dFcd (kN)	vEd (kN/m2)	vEd,perm.	asf (cm2/m)
1	0	390	0.0	963.4	390	248	554	947	7325	2.7
1	390	780	963.4	1284.5	390	248	185	316	7325	0.9
1	780	1170	1284.5	963.4	390	248	185	316	7325	0.9
1	1170	1560	963.4	3.3	390	248	552	944	7325	2.7

12 POS. S.12 – SEMI-PRECAST CONCRETE SLAB

12.1 SYSTEM

System



Cross-section b / h / d = 100 / 35 cm

Building materials **C 30/37 | B 500A**

Exposure class XC1, WO

Reinforcing steel c_{nom} = 10 mm + 10 mm = 20 mm

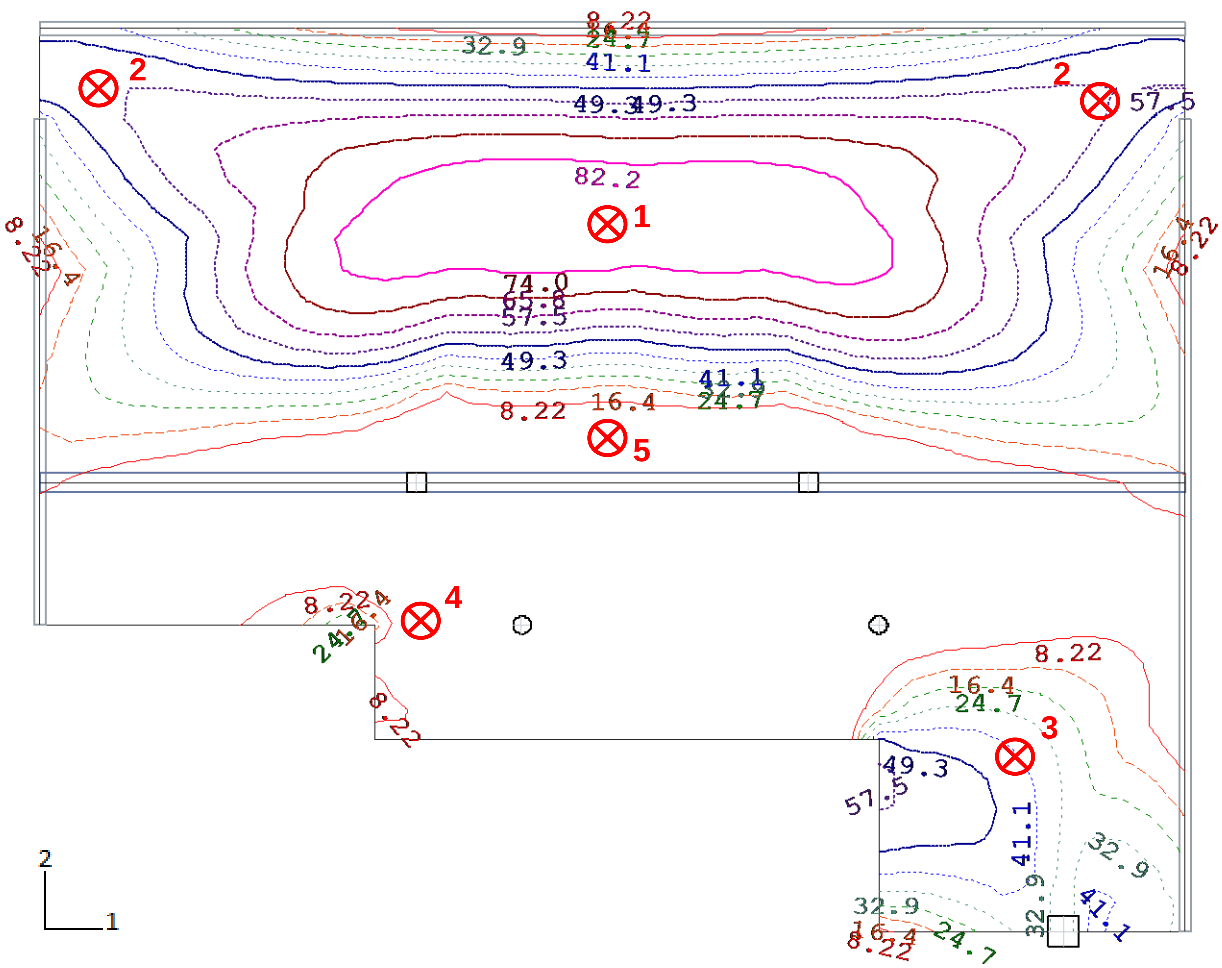
Characteristic loads

Uniformly Distributed Loads		**g_k [kN/m²]**	**q_k [kN/m²]**
Self-weight	g_{k1} = 0.35 x 25 kN/m³ =	8.75	-
Floor structure	g_{k2} =	2.0	-
Live load (Cat. B2 – Office Building)	q_{k1} =	-	2.0
Load from partition walls	q_{k2} =		1.2
		10.75	3.2
Linear Loads		**g_k [kN/m]**	**q_k [kN/m]**
Reaction force from *POS. T.10*		19.1	5.9

12.2 ULTIMATE LIMIT STATE DESIGN

12.2.1 FLEXURE DESIGN

Lower Design Bending Moment mB – 2



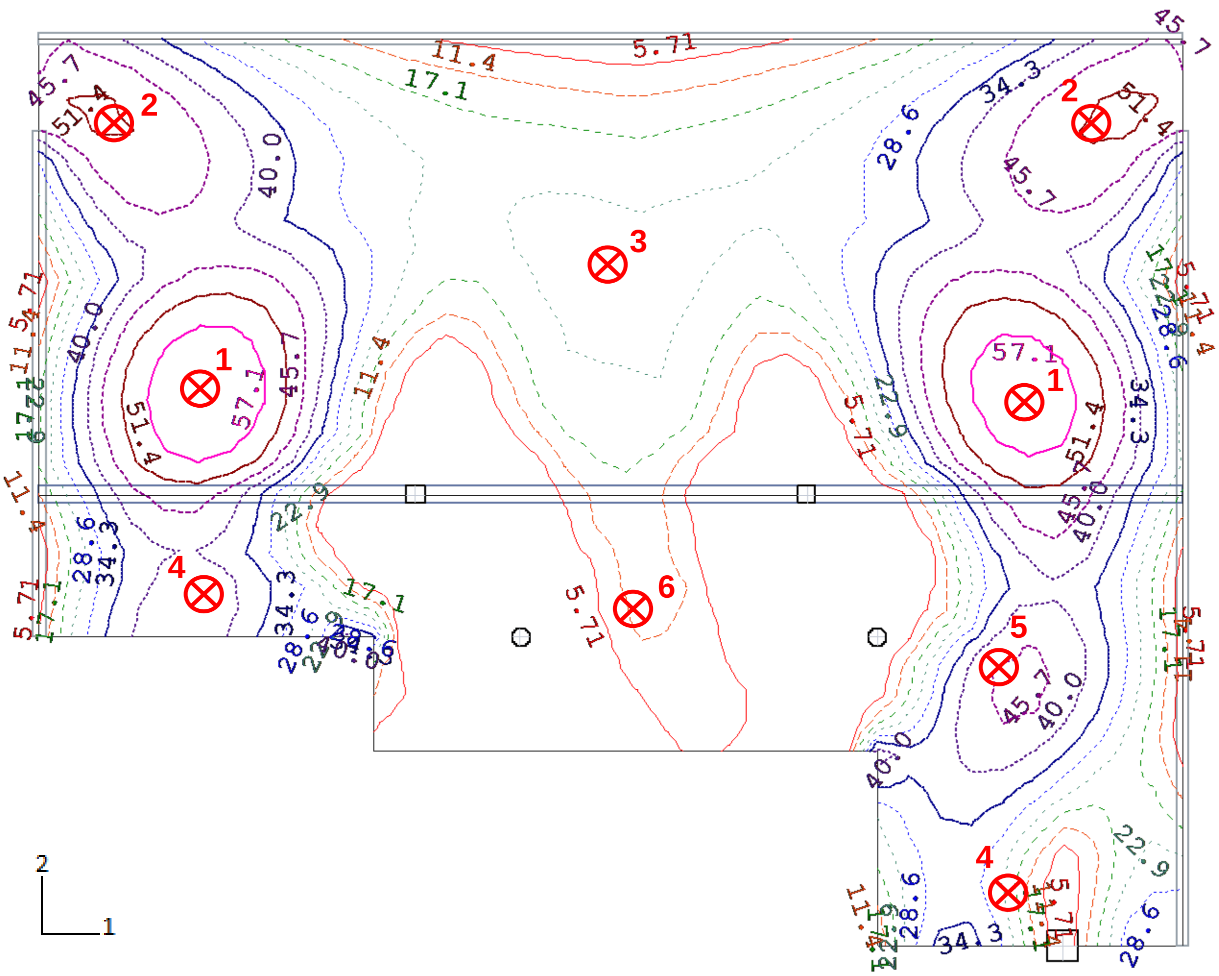
$d_1 = h - c_{nom} - d_s / 2 = 35 - 2.0 - 1.0 / 2 = 32.5cm$ (Reinforcement in the precast concrete)

$\mu_{Eds} = \dfrac{m_{Ed,l\_2}}{100 \cdot 32.5^2 \cdot 1.98} = 4.78 \cdot 10^{-6} \cdot m_{Ed,l\_2}$ → ω → $a_{s,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \dfrac{1}{\sigma_{sd}} = \omega \cdot 147.9cm^2$

Chosen Lattice girder **KT 800** (lower rebars 2 Ø 10, distance s_T = 45 cm, a_s = 3.49cm²/m)

	m_{Ed,l_2} [kNm/m]	μ_{Ed} [-]	ω [-]	$a_{s,l\_2,req}$ [cm²/m]	Chosen KT 800	Chosen Additional	$a_{s,l\_2,prov}$ [cm²/m]
1	82.20	0.0458	0.04625	5.78	2 Ø 10 / 45	Ø 8 / 20	6.00
2	57.50	0.0320	0.0306	3.83	2 Ø 10 / 45	Ø 8 / 25	5.50
3	49.30	0.0275	0.0306	3.83	2 Ø 10 / 45	Ø 8 / 25	5.50
4	24.70	0.0138	0.0152	1.90	2 Ø 10 / 45	Ø 8 / 25	5.50
5	8.22	0.0046	0.0101	1.26	2 Ø 10 / 45	Ø 8 / 25	5.50

Lower Design Bending Moment mB - 1



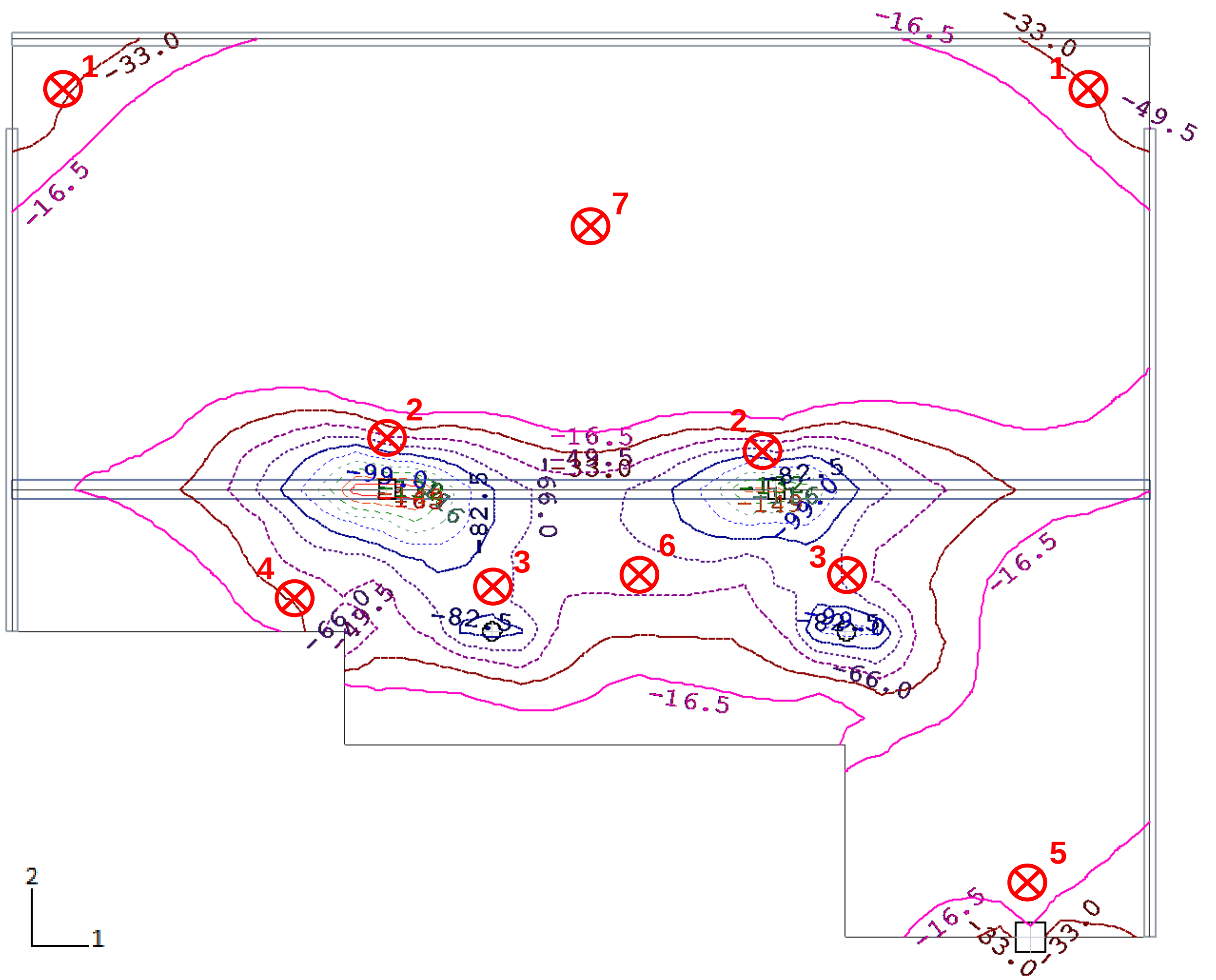
$d_1 = h - h_F - d_s / 2 = 35 - 4.0 - 1.0 / 2 = 30.5cm$ (Reinforcement in the in-situ concrete)

$\mu_{Eds} = \frac{m_{Ed,l\_1}}{100 \cdot 30.5^2 \cdot 1.98} = 4.12 \cdot 10^{-6} \cdot m_{Ed,l\_1}$ → ω → $a_{s,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = \omega \cdot 159.31 cm^2$

	m_{Ed,l_1} [kNm/m]	μ_{Ed} [-]	ω [-]	$a_{s,l\_1,req}$ [cm²/m]	Chosen Reinforcement	$a_{s,l\_1,prov}$ [cm²/m]
1	57.10	0.0361	0.0358	4.48	Ø 8 / 10	5.03
2	51.40	0.0325	0.0306	3.83	Ø 8 / 10[2]	5.03
3	22.90	0.0145	0.0152	1.90	Ø 8 / 10[2]	5.03
4	34.30	0.0217	0.0203	2.54	Ø 8 / 10[2]	5.03
5	45.70	0.0289	0.0306	3.83	Ø 8 / 10[2]	5.03
6	5.71	0.0036	0.0101	1.26	Ø 8 / 10[2]	5.03

[2] Chosen due to a necessary minimum reinforcement (see Chapter 11.4.1)

Upper Design Bending Moment mB - 2

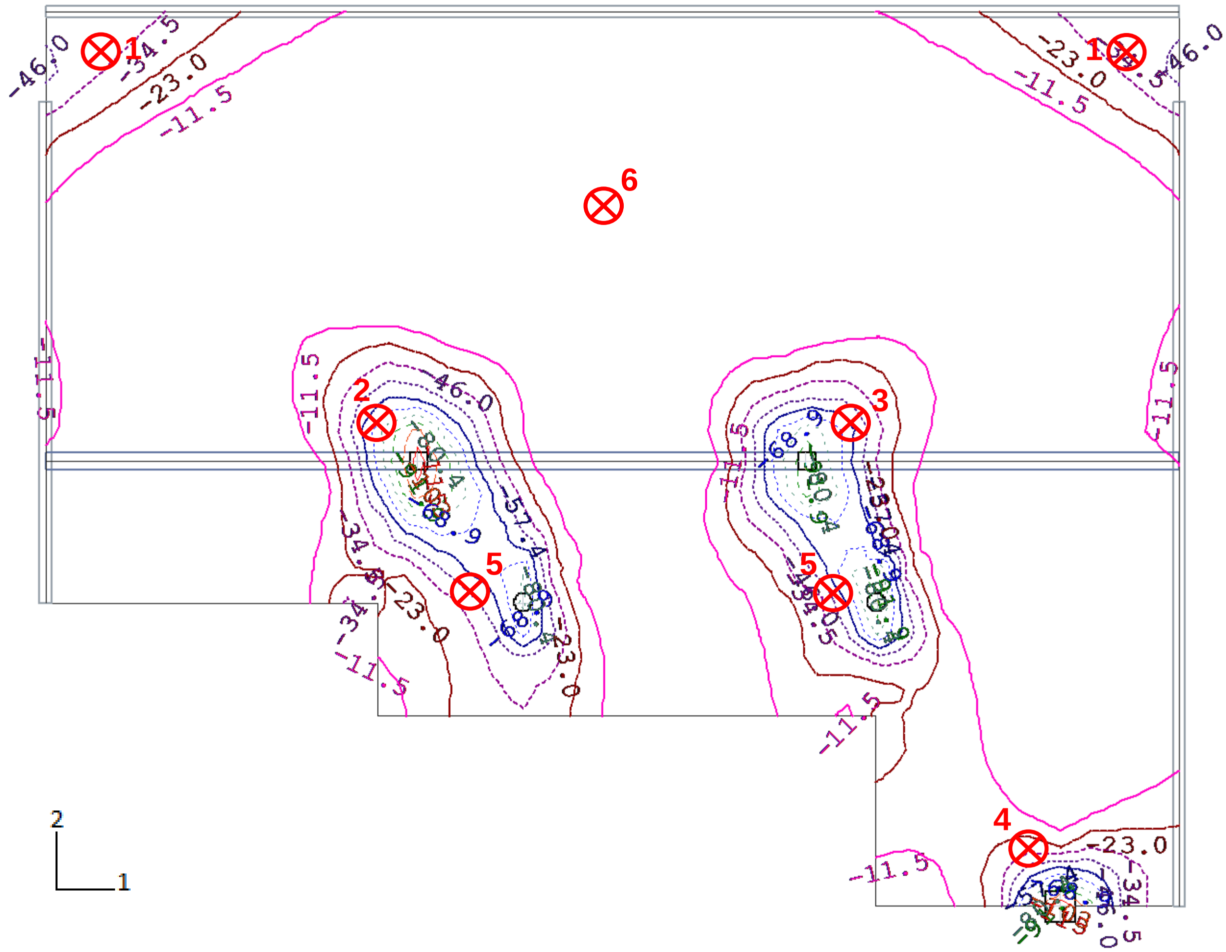


$d_1 = h - c_{com} - d_s / 2 = 35 - 2.0 - 1.0 / 2 = 32.5cm$ (Reinforcement in the in-situ concrete)

$$\mu_{Eds} = \frac{m_{Ed,u\_2}}{100 \cdot 32.5^2 \cdot 1.98} = 4.78 \cdot 10^{-6} \cdot m_{Ed,u\_2} \rightarrow \omega \qquad \rightarrow a_{s,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = \omega \cdot 147.9cm^2$$

	m_{Ed,l_2} [kNm/m]	μ_{Ed} [-]	ω [-]	$a_{s,l\_2,req}$ [cm²/m]	Chosen Reinforcement		$a_{s,l\_2,prov}$ [cm²/m]
					Mesh	Additional	
1	33.00	0.0184	0.0203	2.54	Q 335 A	-	3.35
2	149.00	0.0830	0.0891	11.14	Q 335 A	Ø 10 / 7.5	13.82
3	82.50	0.0459	0.04625	5.78	Q 524 A	Ø 8 / 25	7.25
4	66.00	0.0368	0.0358	4.48	Q 524 A	-	5.24
5	33.00	0.0184	0.0203	2.54	Q 524 A	-	5.24
6	49.50	0.0276	0.0306	3.83	Q 524 A	-	5.24
7	16.50	0.0092	0.0101	1.26	Q 335 A	-	3.35

Upper Design Bending Moment mB - 1

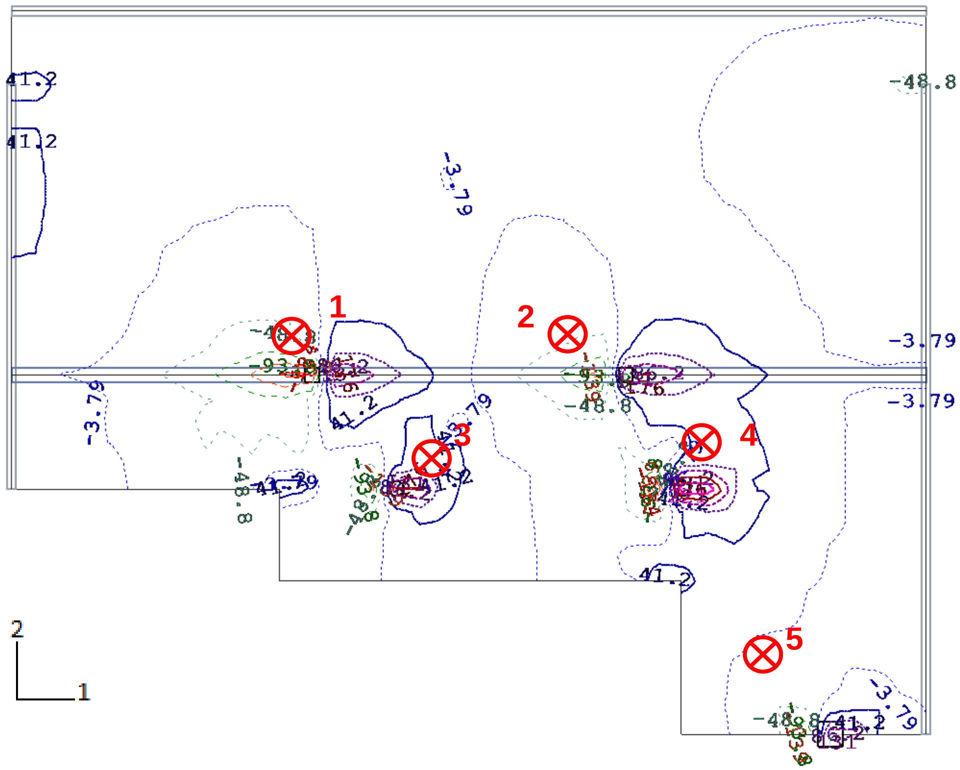


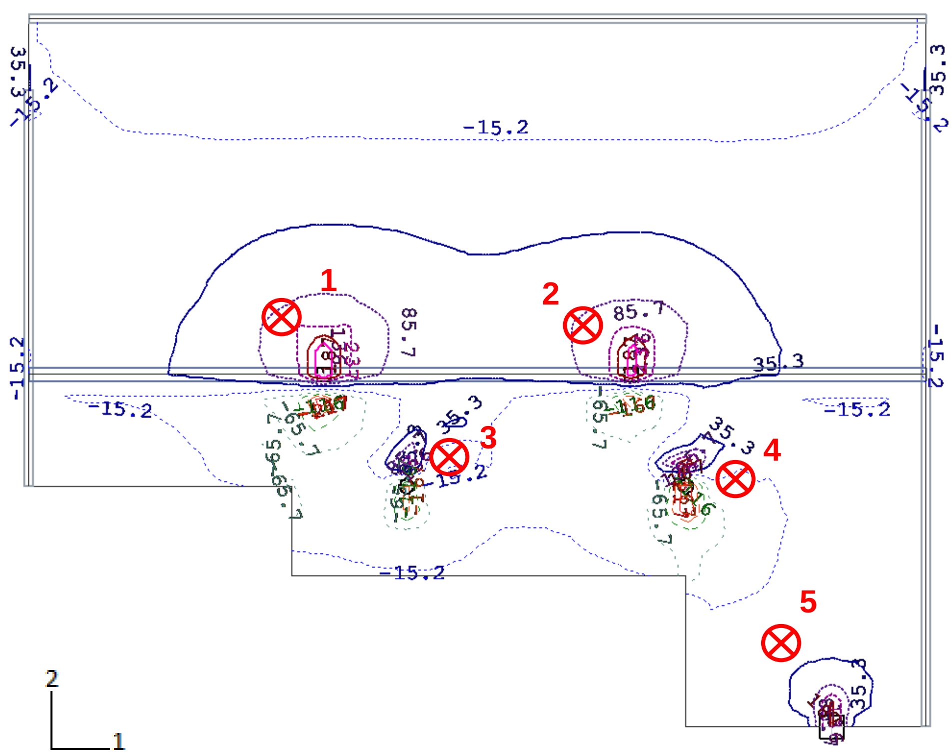
$d_1 = h - c_{com} - d_{s,2} - d_{s,1} / 2 = 35 - 2.0 - 1.0 - 1.0 / 2 = 31.5cm$ (Reinforcement in the in-situ concrete)

$\mu_{Eds} = \frac{m_{Ed,u\_1}}{100 \cdot 31.5^2 \cdot 1.98} = 5.09 \cdot 10^{-6} \cdot m_{Ed,u\_1}$ → ω → $a_{s,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = \omega \cdot 143.4cm^2$

	m_{Ed,u_1} [kNm/m]	μ_{Ed} [-]	ω [-]	$a_{s,u\_1,req}$ [cm²/m]	Chosen Reinforcement		$a_{s,u\_1,prov}$ [cm²/m]
					Mesh	Additional	
1	46.00	0.0273	0.0306	3.83	Q 335 A	4 Ø 10 / 10	6.49
2	103.00	0.0611	0.06745	8.44	Q 335 A	Ø 10 / 15	8.59
3	80.40	0.0477	0.0515	6.44	Q 335 A	Ø 10 / 15	8.59
4	91.90	0.0545	0.0568	7.10	Q 524A	3 Ø 10 / 10	7.60
5	68.90	0.0408	0.041	5.13	Q 524 A	-	5.24
6	11.50	0.0068	0.0101	1.26	Q 335 A	-	3.35

12.2.2 SHEAR DESIGN





	1	2	3	4	5
$v_{Ed,1}$ [kN/m]	93.6	93.6	86.1	48.9	48.9
$v_{Ed,2}$ [kN/m]	136.0	136.0	65.7	65.7	35.3

Shear force at design location $v_{Ed,2} = 136.0kN/m$

Scale factor $k = 1+\sqrt{\frac{20}{d}} = 1+\sqrt{\frac{20}{31}} = 1.80 < \underline{\underline{2.00}}$

Percentage of reinforcement $\rho_1 = \frac{A_{sl}}{b_w \cdot d} = \frac{3.35cm^2/m}{100\cdot 31} = 0.0011 \leq \underline{\underline{0.02}}$

Shear force resistance $V_{Rd,c} = \left[0.10\cdot k\cdot(100\cdot\rho_l\cdot f_{ck})^{1/3} + 0.12\cdot\sigma_{cp}\right]\cdot b_w\cdot d$

$V_{Rd,c} = \left[0.10\cdot 1.8\cdot(100\cdot 0.0011\cdot 30)^{1/3}\right]\cdot 0.31\cdot 1.0\cdot 10^3 = 83.1kN/m$

$d = 31cm < 60cm \rightarrow \nu_{\min} = 0.035\cdot 1.80^{3/2}\cdot(30)^{1/2} = 0.463MN/m^2$

$v_{Rd,c,\min} = 0.463\cdot 1.00\cdot 0.31\cdot 10^3 = \underline{\underline{143.5kN/m}} > v_{Rd,c} = 83.1kN/m$

$v_{Ed} = 136kN/m < v_{Rd,c} = 143.5kN/m$

➔ *No shear reinforcement is needed!*

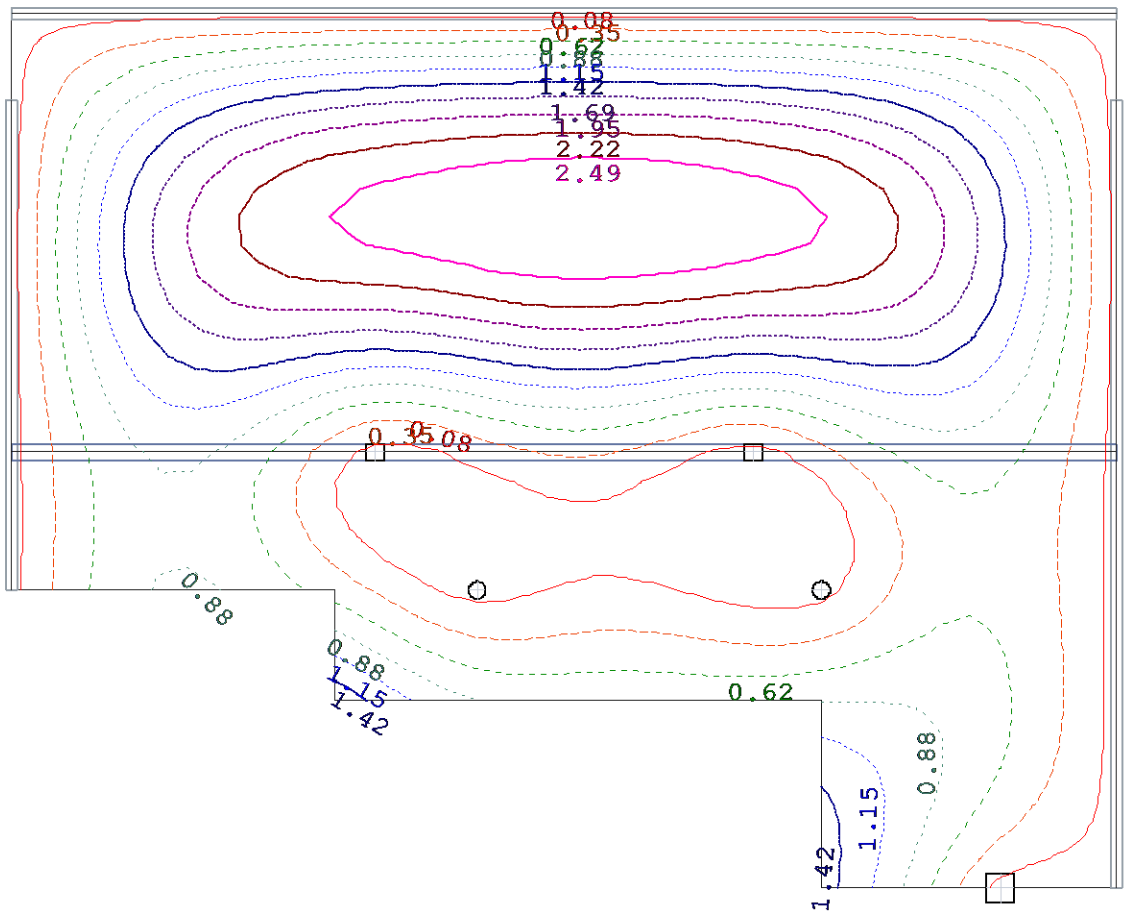
Minimum shear reinforcement $\min a_{sw} = \rho\cdot b_w\cdot\sin\alpha$ and $\rho = 0.16\cdot f_{ctm}/f_{yk} = 0.16\cdot 3.2/500 = 1.02‰$

$\min a_{sw} = 0.00102\cdot 0.40\cdot\sin 90° = 4.08\cdot 10^{-4} m^2/m = \underline{\underline{4.08cm^2/m}}$

$\min a_{sw} = 4.08cm^2/m < a_{sw,prov} = 5.03cm^2/m$ ✓

12.3 SERVICEABILITY LIMIT STATE DESIGN

Deformation of the Slab under q_{perm}



$f_{\max} = 2.49mm < l / 250 = 7500 / 250 = 30mm$ ✓

The deformation does not exceed the maximum allowed deformation!

12.4 DESIGN AND REINFORCEMENT

12.4.1 MINIMUM REINFORCEMANT TO ENSURE ROBUSTNESS

$M_{cr} = f_{ctm} \cdot W_{cu}$ $\qquad W_{cu} = 100 \cdot 35^2 / 6 = 20,417cm^3$

$M_{cr} = 2.9 \cdot 20,417 \cdot 10^{-3} = 59.2kNm$

Direction of the longer span: $a_{s,\min,1} = \frac{1}{f_{yk}} \cdot \frac{M_{cr}}{z} = \frac{1}{50.0} \cdot \frac{5920}{0.9 \cdot 30.5} = 4.31cm^2 / m < a_{s,1,prov} = 5.5cm^2 / m$ ✓

Direction of the shorter span: $a_{s,\min,2} = \frac{1}{f_{yk}} \cdot \frac{M_{cr}}{z} = \frac{1}{50.0} \cdot \frac{5920}{0.9 \cdot 32.5} = 4.04cm^2 / m < a_{s,2,prov} = 5.03cm^2 / m$ ✓

12.5 SOFTWARE CALCULATIONS

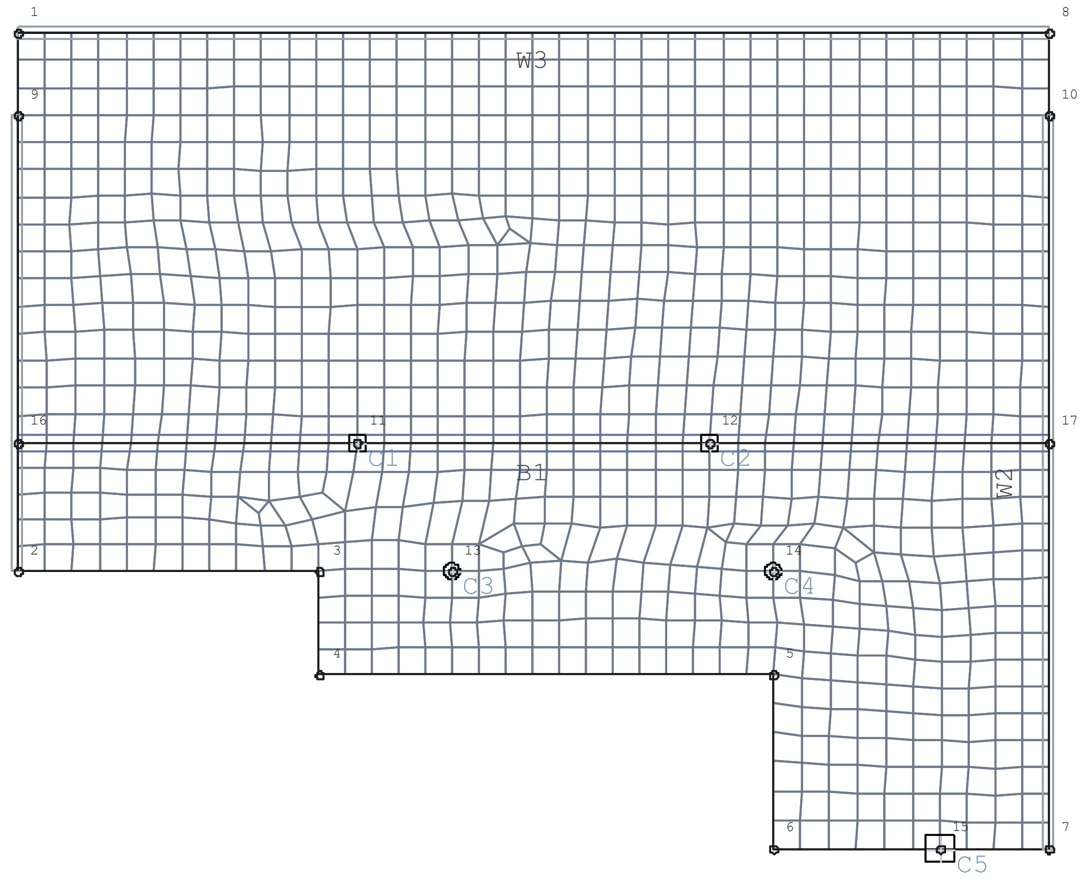
Demo Frilo Nemetschek

Position: S.12 – Semi-Precast Concrete Slab

Plates by Finite Elements PLT 01/2018 (Frilo R-2018-1/P12)

System: Floor Plan with FE Mesh

Scale 1 : 150



LOADCASE 1 "Self-Weight"

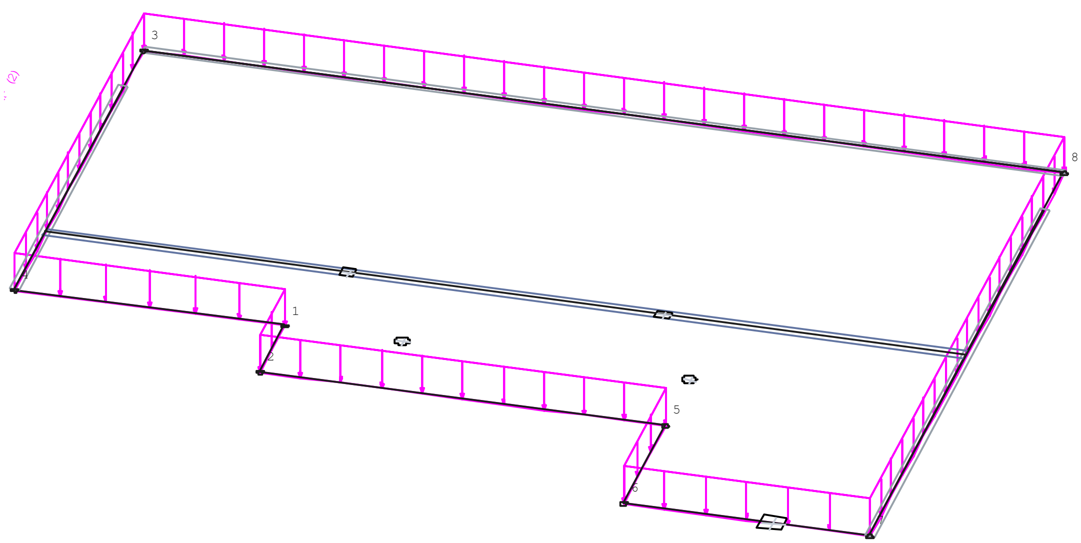
```
----------------------------------------------------------------------

Type:                                                   permanent
Dead loads due to
   Plate , beams and parapets
are included:                                                 YES
Action:                                           Ständige Lasten
Partial safety factor action:                                1.35
Partial safety factor concrete:                              1.50
Partial safety factor steel:                                 1.15
Load points:                                                    8
Point loads:                                                    0
Line loads:                                                     1
Area loads:                                                     1
Temperature loads:                                              0

Total of input loads:                                     496 [kN]
 (portion on the plate)
Dead loads due to
   plate and beams:                                      2061 [kN]
Total of all loads:                                      2558 [kN]

Total of all support reactions:                          2558 [kN]
```

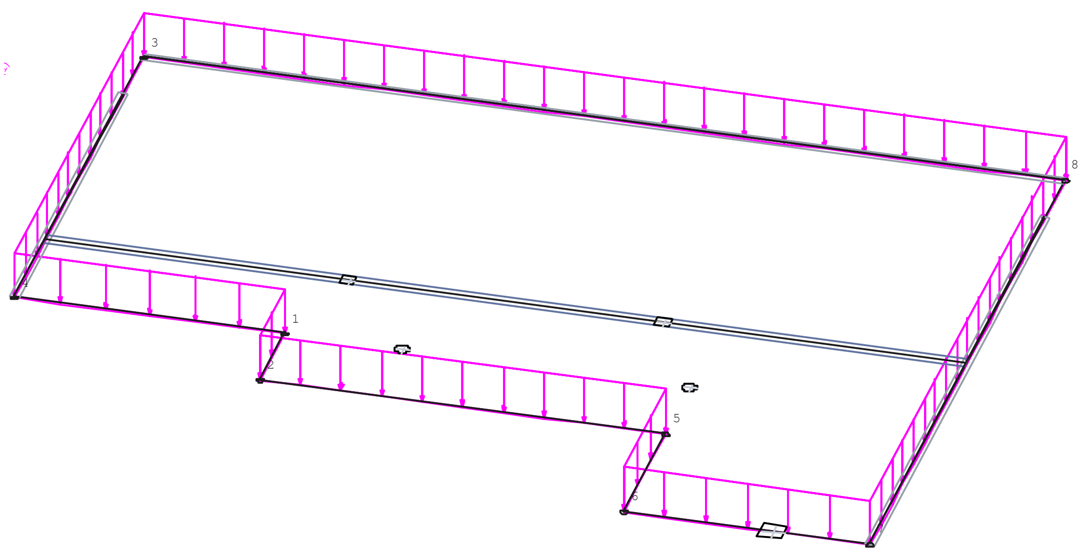
Load case 1 "Self-Weight"
Area Loads
Scale 1 : 150



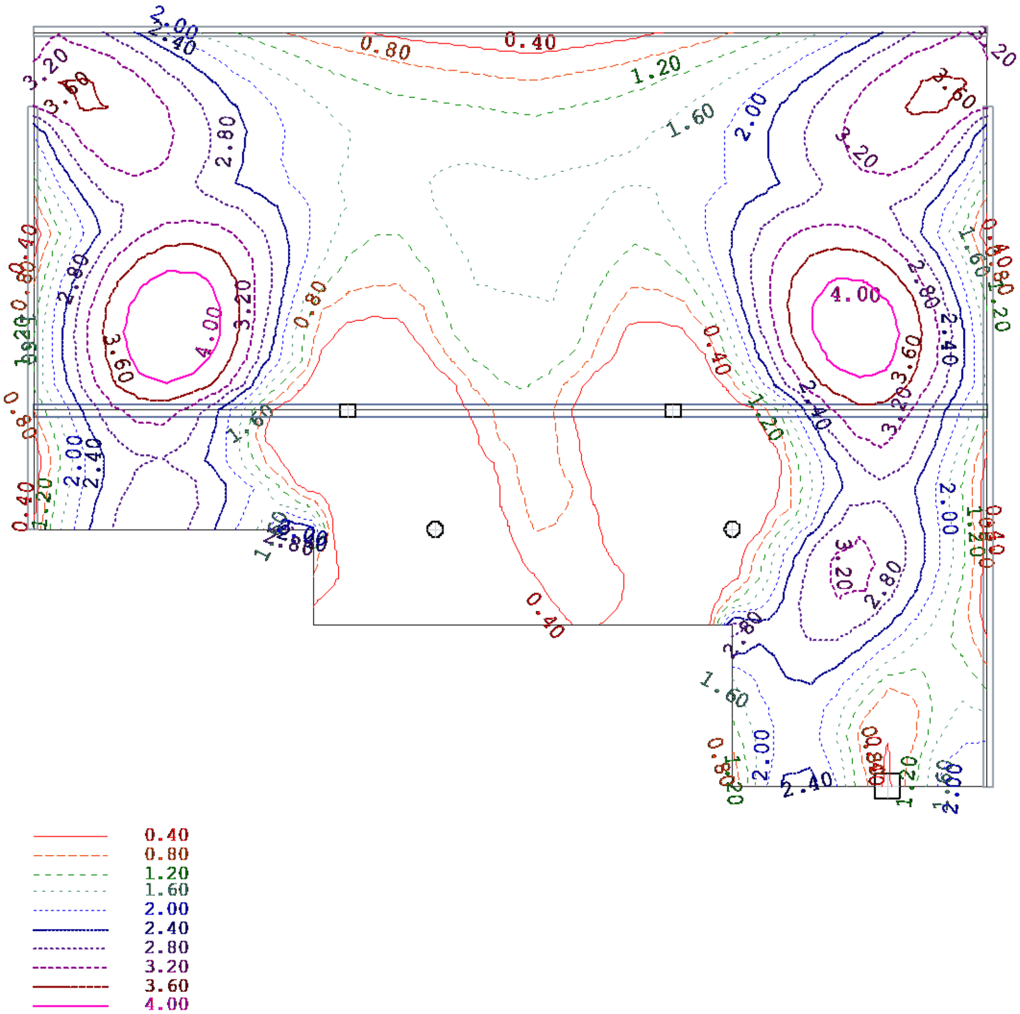
LOADCASE 2 "Live Load"

Type:	non permanent
Dead loads due to Plate , beams and parapets are included:	NO
Action:	Büros
Partial safety factor action:	1.50
Partial safety factor concrete:	1.50
Partial safety factor steel:	1.15
Load points:	8
Point loads:	0
Line loads:	1
Area loads:	1
Temperature loads:	0
Total of input loads: (portion on the plate)	747 [kN]
Total of all support reactions:	747 [kN]

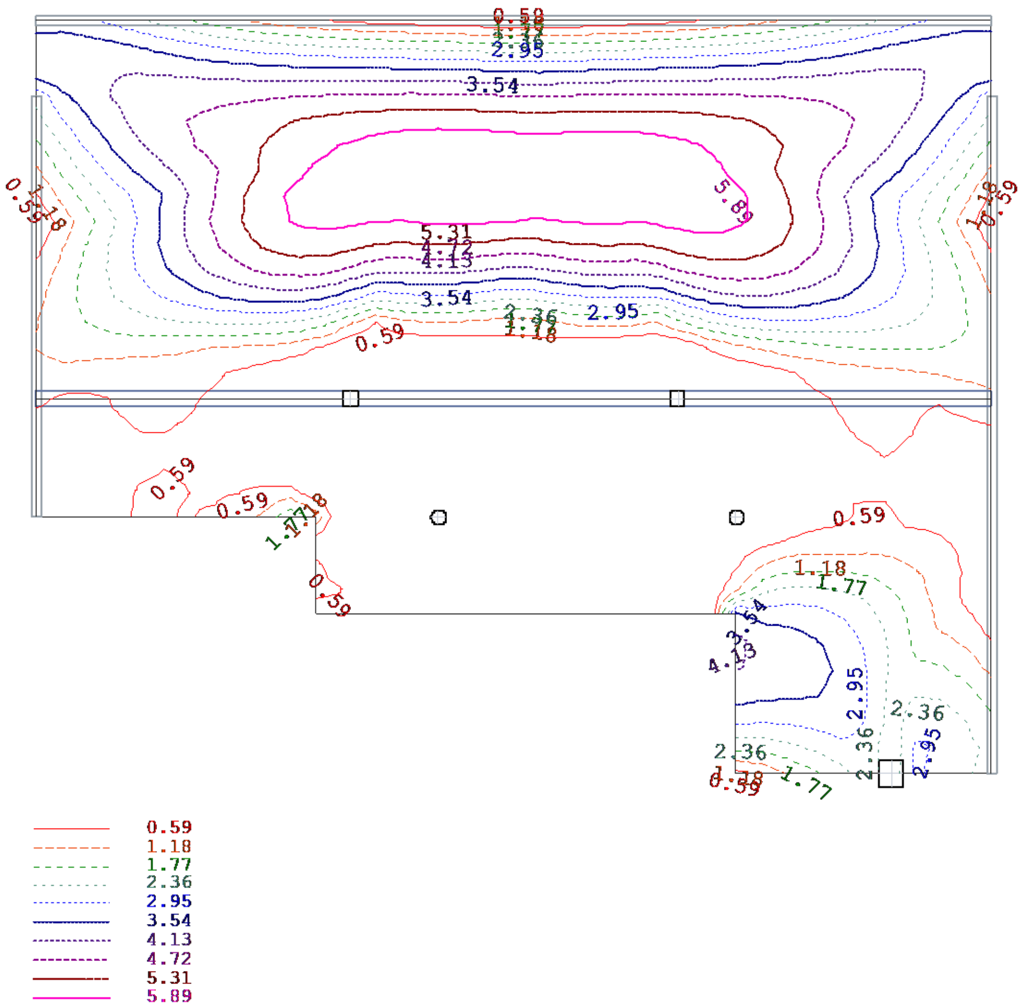
Load case 2 "Live Load"
Area Loads
Scale 1 : 150



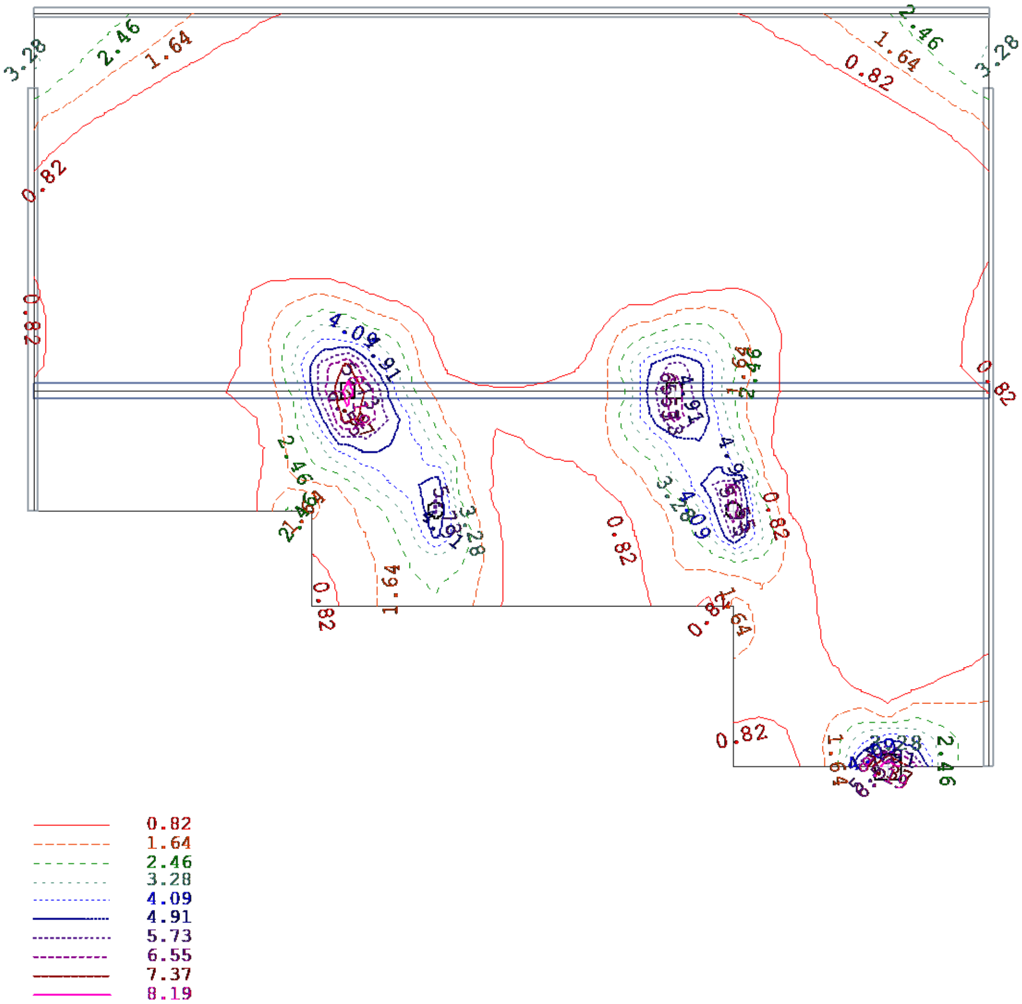
Superposition 2 "USL Permanent/Transient"
Reinforcement, bottom aS-1 [cm²/m]
Scale 1 : 150



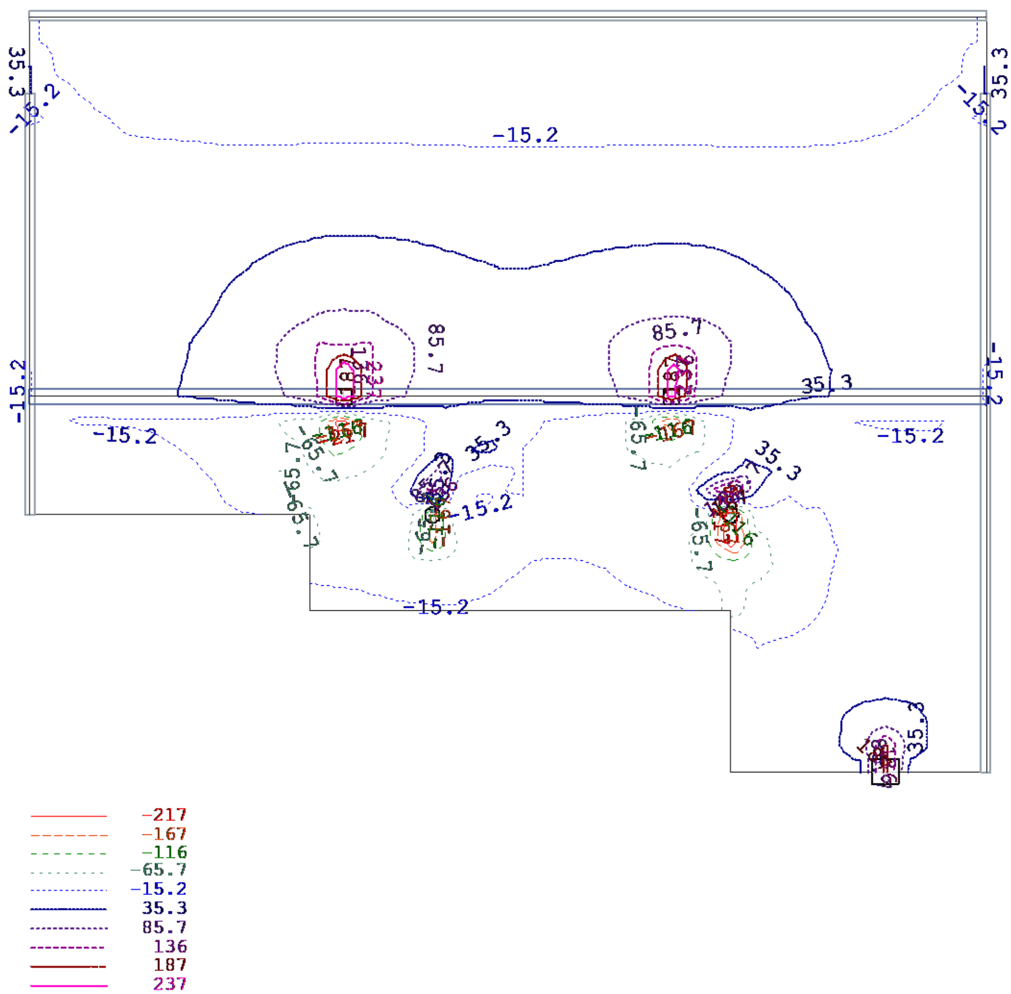
Superposition 2 "USL Permanent/Transient"
Reinforcement, bottom aS-2 [cm²/m]
Scale 1 : 150



Superposition 2 "USL Permanent/Transient"
Reinforcement, top aS-1 [cm²/m]
Scale 1 : 150



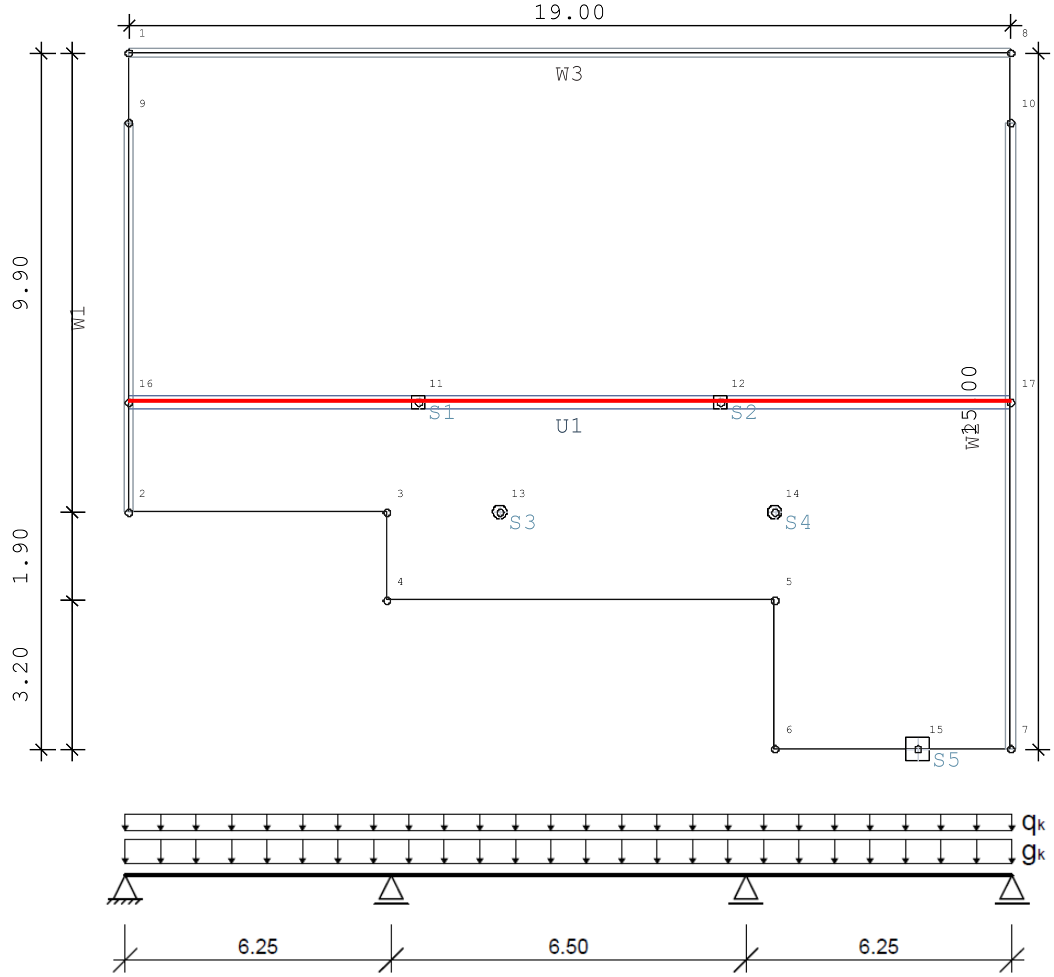
Superposition 2 "USL Permanent/Transient"
Reinforcement, top aS-2 [cm²/m]
Scale 1 : 125



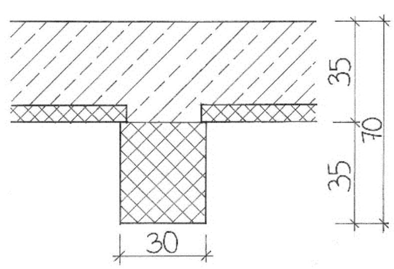
13 POS. B.13 – SEMI-PRECAST CONCRETE BEAM

13.1 SYSTEM

System



Cross-section



Building materials **C 30/37 | B 500A**

Exposure class XC1, WO

Reinforcing steel c_{nom} = 10 mm + 10 mm = 20 mm *Chosen:* c_{nom} = 25 mm

Characteristic loads

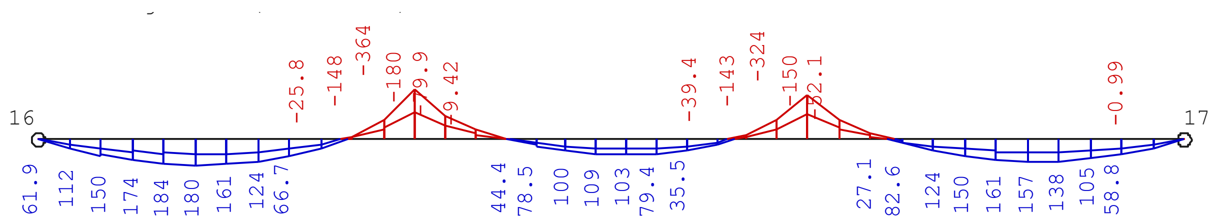
Uniformly Distributed Loads	**g_k [kN/m]**	**q_k [kN/m]**
see *POS. S.12*	-	-

13.2 INTERNAL FORCES

Design Forces

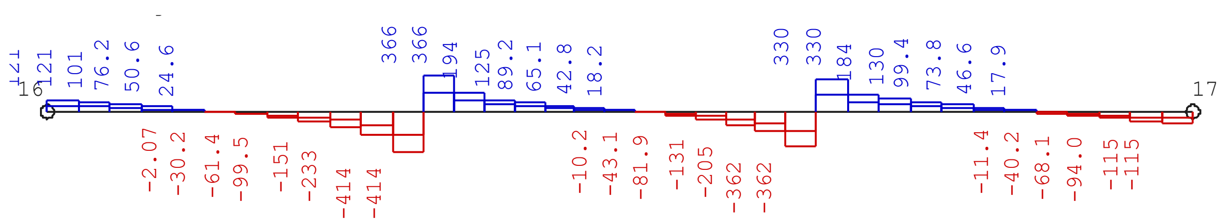
Bending moment

	Span 1	Support B	Span 2	Support C	Span 3
max M_{Ed} [kNm]	184	- 364	109	- 324	161



Shear force

	Support A	Support B_L	Support B_R	Support C_L	Support C_R	Support D
maxV_{Ed} [kN]	121	- 414	366	- 362	330	- 115

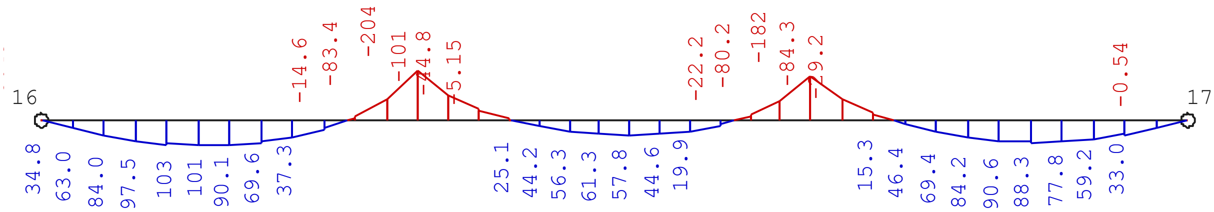


Characteristic forces

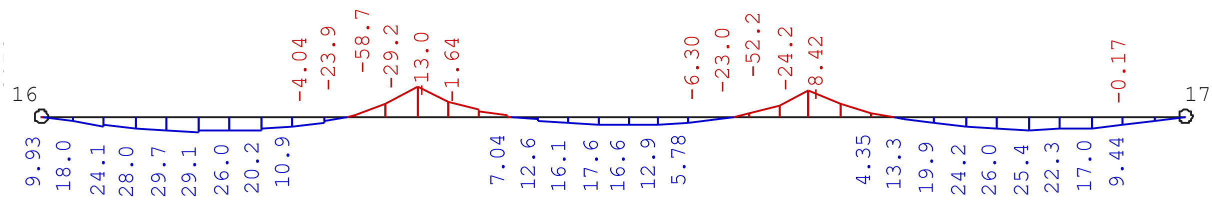
Bending moment

	Span 1	Support B	Span 2	Support C	Span 3
M_{gk} [kNm]	103	- 204	61.3	- 182	90.6
M_{qk} [kNm]	29.7	- 58.7	17.6	- 52.2	26.0
M_{perm} [kNm]	111.9	- 221.6	66.6	- 197.7	98.1

Bending moment M_{gk}



Bending moment M_{qk}



13.3 ULTIMATE LIMIT STATE DESIGN

13.3.1 FLEXURE DESIGN

Lower reinforcement at span 1

Effective width

$$b_{eff,1} = 0.2 \cdot 360 + 0.1 \cdot 0.85 \cdot 625 = 125cm < \begin{cases} 0.2 \cdot 0.85 \cdot 625 = \underline{\underline{106cm}} \\ b_1 = 360cm \end{cases}$$

$$b_{eff,2} = 0.2 \cdot 220 + 0.1 \cdot 0.85 \cdot 625 = \underline{\underline{97cm}} < \begin{cases} 106cm \\ b_2 = 220cm \end{cases}$$

$$b_{eff} = b_{eff,1} + b_w + b_{eff,2} = 1.06 + 0.30 + 0.97 = \underline{\underline{2.33m}}$$

Lower reinforcement

$$\mu_{Eds} = \frac{M_{Eds}}{b_{eff} \cdot d^2 \cdot f_{cd}} = \frac{18400kNcm}{233 \cdot 65^2 \cdot 1.70kN/cm^2} = 0.011 \quad \rightarrow \omega = 0.0101$$

$$\xi = 0.030 \quad \rightarrow x = 0.030 \cdot 65 = 1.95cm < h_f = 35cm \ \checkmark$$

$$A_{sl,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = 0.0101 \cdot 233 \cdot 65 \cdot 1.70 \cdot \frac{1}{43.5} = 5.98cm^2$$

Chosen **4 Ø 14 = 6.16 cm²** **lower reinforcement**

New static height

$$d = h - c_{nom} - d_{s,bü} - d_{s,lä}/2 = 70 - 2.0 - 1.0 - 1.6/2 = 66.2cm$$

$$> d_{est} = 65cm \ \checkmark$$

Lower Reinforcement at Span 2 and Span 3

	M_{Ed} [kNm]	b_{eff} [cm]	d [cm]	μ_{Ed} [-]	ω [-]	x [cm]	z [cm}	$A_{s,req}$ [cm²]	Chosen	$A_{s,prov}$ [cm²]
Span 2	109.0	189	65	0.0080	0.0081	1.56	51.48	3.88	**4 Ø 14**	**6.16**
Span 3	161.0	211	65	0.0106	0.0107	2.00	64.34	5.74	**4 Ø 14**	**6.16**

Upper Reinforcement at Support B

Upper reinforcement $\mu_{Eds} = \frac{M_{Eds}}{b_w \cdot d^2 \cdot f_{cd}} = \frac{36400 kNcm}{30 \cdot 65^2 \cdot 1.70 kN/cm^2} = 0.169$ → $\omega = 0.1882$

$$A_{sl,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = 0.1882 \cdot 30 \cdot 65 \cdot 1.70 \cdot \frac{1}{43.5} = 14.3 cm^2$$

Chosen **5 Ø 20 = 15.7 cm²** **upper reinforcement**

New static height $d = h - c_{nom} - d_{s,l} - d_{s,q} - d_{s,b}/2 = 70 - 2.0 - 1.0 - 2.0/2 = 65 cm$

$> d_{est} = 65 cm$ ✓

Upper Reinforcement at Support C

	M_{Ed} [kNm]	b_w [cm]	d [cm]	μ_{Ed} [-]	ω [-]	x [cm]	z [cm}	$A_{s,req}$ [cm²]	Chosen	$A_{s,prov}$ [cm²]
Supp C	-324.0	30	65	0.1504	0.1638	13.13	59.54	12.48	**5 Ø 20**	**15.70**

13.3.2 SHEAR DESIGN

Shear Design Calculations at Support B_L

Shear force $V_{Ed} = 414 kN$

Scale factor $k = 1 + \sqrt{\frac{20}{d}} = 1 + \sqrt{\frac{20}{66}} = 1.55 < \underline{\underline{2.00}}$

Percentage of reinforcement $\rho_1 = \frac{A_{sl}}{b_w \cdot d} = \frac{15.7}{30 \cdot 65} = 0.0081 \leq \underline{\underline{0.02}}$

Shear force resistance $V_{Rd,c} = \left[0.10 \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp}\right] \cdot b_w \cdot d$

$$V_{Rd,c} = \left[0.10 \cdot 1.55 \cdot (100 \cdot 0.0081 \cdot 30)^{1/3}\right] \cdot 0.65 \cdot 0.30 \cdot 10^3 = 87.6 kN$$

$d = 65 cm > 60 cm$ → $v_{min} = 0.0325 \cdot 1.55^{3/2} \cdot (30)^{1/2} = 0.345 MN/m^2$

$$V_{Rd,c,min} = 0.345 \cdot 0.30 \cdot 0.65 \cdot 10^3 = 67.3 kN < V_{Rd,c} = \underline{\underline{87.6 kN}}$$

$V_{Ed} = 414 kN > V_{Rd,c} = 87.6 kN$ → Shear reinforcement is needed!

Lever arm of the internal forces $z = \zeta \cdot d = 0.903 \cdot 65 = 58.7 cm$

Friction Force: $V_{Rd,cc} = 0.24 \cdot (30)^{1/3} \cdot 0.3 \cdot 0.587 \cdot 10^3 = 131.3 kN$

Inclination: $1.0 \le \cot\theta = \dfrac{1.2}{1-V_{Rd,cc}/V_{Ed}} = \dfrac{1.2}{1-131.3/414} = 1.76 \le \underline{\underline{3.0}}$

Vertical stirrups $\alpha = 90°$

Required reinforcement $a_{sw,req} = \dfrac{414kN}{43.5 \cdot 0.587 \cdot 1.75} = 9.3cm^2/m$

Chosen **Ø 8 / 10 = 10.05 cm²/m** **stirrups**

Resistance of the compression struts

$$V_{Rd,\max} = 0.75 \cdot 30 \cdot 58.7 \cdot 1.7 \cdot \frac{1.76}{1+1.76^2} = 965kN$$

$V_{Ed,\max} = 414kN < V_{Rd,\max} = 965kN$ → The compression strut is ok.

Minimum shear reinforcement $\min a_{sw} = \rho \cdot b_w \cdot \sin\alpha$ and $\rho = 0.16 \cdot f_{ctm} / f_{yk} = 0.16 \cdot 2.9/500 = 0.93‰$

$\min a_{sw} = 0.0093 \cdot 0.30 \cdot \sin 90° = 0.28 \cdot 10^{-4} m^2/m = \underline{\underline{0.28cm^2/m}}$

$\min a_{sw} = 0.28cm^2/m < a_{sw,prov} = 10.48cm^2/m$ ✓

Shear Design Calculations at other Supports

	V_{Ed} [kN]	$V_{Rd,c}$ [kN]	$V_{Rd,cc}$ [kN]	cot θ [-]	$a_{sw,req}$ [cm²/m]	Chosen	$a_{sw,prov}$ [cm²/m]	$V_{Rd,max}$ [kN]
Supp A_R	121	67.29	130.88	1.200	3.96	**Ø 8 / 20**	**5.03**	1100.5
Supp B_L	- 414	87.63	131.31	1.757	9.23	**Ø 8 / 10**	**10.05**	965.0
Supp B_R	366	87.63	131.31	1.871	7.66	**Ø 8 / 12.5**	**8.04**	933.2
Supp C_L	- 362	87.63	133.20	1.899	7.36	**Ø 8 / 12.5**	**8.04**	939.0
Supp C_R	330	87.63	133.20	2.012	6.33	**Ø 8 / 15**	**6.71**	907.6
Supp D_L	- 115	67.29	130.88	1.200	3.77	**Ø 8 / 20**	**5.03**	1100.5

13.4 SERVICEABILITY LIMIT STATE DESIGN

13.4.1 LIMITATION OF THE CRACK WIDTH

Maximum Diameter of the Reinforcement

Crack width w_k = 0.4 mm (according to DIN EN 1992-1-1, Tab. NA.7.1.)

Reinforcement stresses $\sigma_s = \dfrac{M_{perm}}{z \cdot A_{sl}}$ → d_s^*

Limit diameter $d_s = d_s^* \cdot f_{ct,eff} / 2.9 > d_{s,prov}$

	M_{perm} [kNm]	$A_{s,prov}$ [cm²]	z [cm]	σ_s [N/mm²]	d_s* [mm]	max d_s [mm]	prov d_s [mm]
Span 1	111.9	6.16	64.35	282.29	17.77	**17.77**	**> 14**
Supp B	- 221.6	15.7	58.70	240.47	24.00	**24.00**	**> 20**
Span 2	66.6	6.16	51.48	210.02	32.25	**32.25**	**> 14**
Supp C	- 197.7	15.7	59.54	211.49	31.84	**31.84**	**> 20**
Span 3	98.1	6.16	64.35	247.49	16.88	**16.88**	**> 14**

13.4.2 MINIMUM REINFORCEMENT TO PREVENT CRACKING (SUPPORT B)

Minimum Reinforcement of the Flange

Tensile strength $f_{ct,eff} = f_{ctm} \geq 3.0 MN/m^2$ → $f_{ct,eff} = 2.9 MN/m^2 > \underline{\underline{3.0 MN/m^2}}$

Tensile stresses at the flange

$$\sigma_c = -\frac{(21.5-17.5)}{21.5} \cdot 3.0 = -0.558 MN/m^2$$

$h = 35cm$ → $h^* = 35cm$

$k_1 = 1.5$

$$k_c = 0.4 \cdot \left[1 - \frac{\sigma_c}{k_1 \cdot h / h^* \cdot f_{ct,eff}}\right] = 0.4 \cdot \left[1 - \frac{-0.558}{1.5 \cdot 3.0}\right] = 0.45 \leq 1.0$$

for $h = 35cm > 30cm$ → $k = 0.77$

Tensile stress area $A_{ct} = 0.215 \cdot 1.0 = 0.215 m^2/m$

Reinforcement stresses $d_s = 10mm$ → $d_s^* = \frac{2.9}{f_{ct,eff}} \cdot d_s = \frac{2.9}{3.0} \cdot 10 = 9.7$

$\sigma_s = 386 MN/m^2$ (w_k = 0.4mm, d_s = 9.7mm)

Required reinforcement $a_{s,min,req} = k_c \cdot k \cdot f_{ct,eff} \cdot \frac{A_{ct}}{\sigma_s} = 0.45 \cdot 0.77 \cdot 3.0 \cdot \frac{0.215}{386} \cdot 10^4 = 5.79 cm^2/m$

$< a_{s,prov} = 8.59 cm^2/m$ *(Q335A + Ø 10/15)*[3]

Minimum Reinforcement of the Web

Compression stresses at the web

$$\sigma_c = -3.0 + \frac{35.0}{21.5} \cdot 3.0 = +1.883 MN/m^2$$

$k_1 = 1.5$

[3] See Chapter 12.2.1 (Upper reinforcement of the slab in direction 1)

$$k_c = 0.4 \cdot \left[1 - \frac{\sigma_c}{k_1 \cdot h / h^* \cdot f_{ct,eff}}\right] = 0.4 \cdot \left[1 - \frac{1.883}{1.5 \cdot 3.0}\right] = 0.23 \leq 1.0$$

for $h = 70cm > 30cm$ → $k = 0.56$

Tensile stress area $A_{ct} = 0.215 \cdot 0.3 = 0.0645m^2$

Reinforcement stresses $d_s = 20mm$ → $d_s^* = \dfrac{2.9}{f_{ct,eff}} \cdot d_s = \dfrac{2.9}{3.0} \cdot 20 = 19.33$

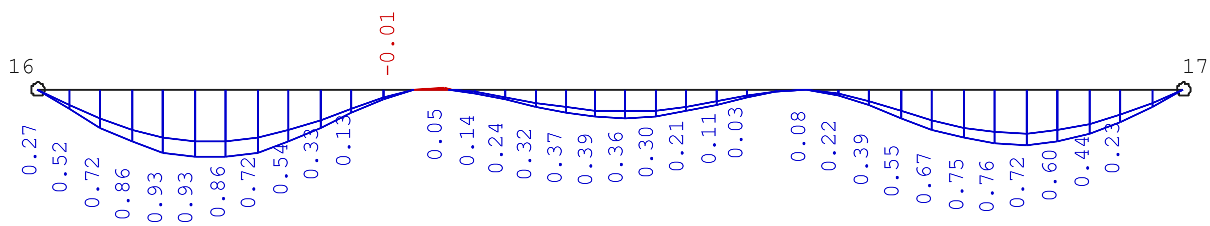
$\sigma_s = 271MN / m^2$ (w_k = 0.4mm, d_s = 19.33mm)

Required reinforcement $A_{s,\min,req} = k_c \cdot k \cdot f_{ct,eff} \cdot \dfrac{A_{ct}}{\sigma_s} = 0.23 \cdot 0.56 \cdot 3.0 \cdot \dfrac{0.0645}{271} \cdot 10^4 = 0.92cm^2$

$< A_{s,prov} = 15.7cm^2$ *(5 Ø 20)*[4]

13.4.3 DEFORMATION OF THE SLAB

Deformation of the slab under q_{perm}



$f_{\max} = 0.93mm < l / 250 = 6250 / 200 = 31.25mm$ ✓

The deformation does not exceed the maximum allowed deformation!

13.5 DESIGN AND REINFORCEMENT

13.5.1 MINIMUM REINFORCEMANT TO ENSURE ROBUSTNESS

$M_{cr} = f_{ctm} \cdot W_{cu}$ $W_{cu} = I_y / z_s$ $A_{s,\min} = \dfrac{1}{f_{yk}} \cdot \dfrac{M_{cr}}{z}$

	b_{eff} [cm]	$z_{s,u}$ [cm]	I_y [cm^4]	W_u [cm³]	W_o [cm³]	M_{cr} [kNcm]	$A_{s,min,req}$ [cm²]	$A_{s,prov}$ [cm²]
Span 1	233	48.51	2079207	42864	-	12430	**3.86**	**6.16**
Supp B	106	44.78	1488435	-	59017	17115	**5.83**	**15.70**
Span 2	189	47.71	1892520	39671	-	11505	**4.47**	**6.16**
Supp C	106	44.78	1488435	-	59017	17115	**5.75**	**15.70**
Span 3	211	48.14	1987209	41277	-	11970	**3.72**	**6.16**

[4] See Chapter 13.3.1 (Upper reinforcement of the beam at support B)

13.5.2 DESIGN OF THE INTERCONNECTING JOINT

Design location $x = \frac{0.15}{2} + 0.65 - 0.29 = 0.435m$

Shear force $V_{Ed,x} \approx 414kN$

$$\upsilon_{Ed,i} = \frac{F_{cdj}}{F_{cd}} \cdot \frac{V_{Ed}}{z \cdot b_F} = 1.0 \cdot \frac{414}{0.26 \cdot 0.587} = 2713kN/m^2$$

Rough joint $c = 0.4 \quad \mu = 0.7 \quad \nu = 0.5$

Maximum shear force $\upsilon_{Rdi,c} = c \cdot 0.12 \cdot (f_{ck})^{2/3} = 0.4 \cdot 0.12 \cdot (30)^{2/3} \cdot 10^3 = 463kN/m^2$

$< \upsilon_{Ed,i} = 2713kN/m^2$ ✓

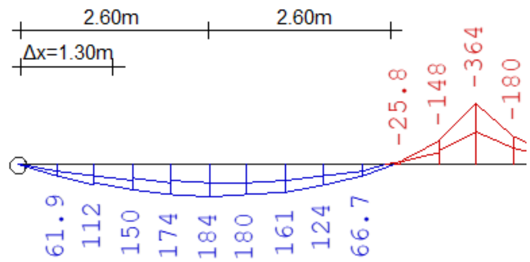
Reinforcement at the interconnection joint necessary!

Reinforcement $a_{s,F} = \frac{(\upsilon_{Edi} - \upsilon_{Rdi,c}) \cdot b_F}{f_{yd} \cdot 1.2 \cdot \mu} = \frac{(2713-463) \cdot 0.26}{43.5 \cdot 1.2 \cdot 0.7} = \underline{\underline{16.0cm^2/m}}$

Maximum shear force $\upsilon_{Rdi,\max} = 0.5 \cdot \nu \cdot f_{cd} = 0.5 \cdot 0.5 \cdot 17 \cdot 10^3 = 4250kN/m^2 > \upsilon_{Edi} = 2713kN/m^2$

13.5.3 CONNECTION OF THE COMPRESSION FLANGE (SPAN 1)

Design location $\Delta x = 1.30m$



Flexure moment $M_{Ed}(x = 1.30) \approx 140kNm = \Delta M_{Ed}$

Bond force in the joint $v_{Ed} = \frac{\Delta M_{Ed}}{z \cdot h_f \cdot \Delta x} \cdot \frac{b_a}{b_{eff}} = \frac{140}{0.644 \cdot 0.35 \cdot 1.30} \cdot \frac{97}{233} = 199kN/m^2$

Shear force resistance $\cot\theta = 1.2$

$$v_{Rd,\max} = \frac{0.75 \cdot f_{cd}}{\cot\theta + \tan\theta} = \frac{0.75 \cdot 17}{1.2 + 1/1.2} \cdot 10^3 = 6271kN/m^2 > v_{Ed} = 199kN/m^2$$

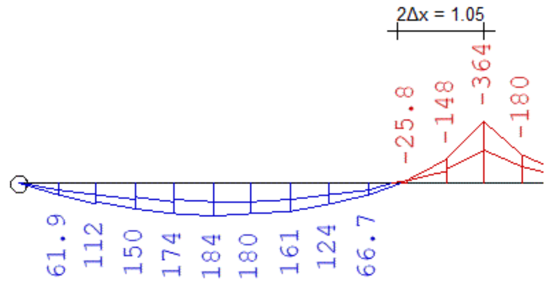
Transverse reinforcement $a_{sw,req} = \frac{v_{Ed} \cdot h_f}{f_{yd} \cdot \cot\theta} = \frac{199 \cdot 0.35}{43.5 \cdot 1.2} = 1.33cm^2/m$

$a_{s,u\_prov} = 3.35cm^2/m > a_{s,u\_req} = 1.33/2 = 0.67cm^2/m$ ✓

$a_{s,l\_prov} = 5.5cm^2/m > a_{s,l\_req} = 1.33/2 = 0.67cm^2/m$ ✓

13.5.4 CONNECTION OF THE TENSION FLANGE

Design location $\Delta x = 1.05/2 = 0.525m$



Flexure moment $\Delta M_{Ed} = 364 - 148 = 216kNm$

Bond force in the joint $v_{Ed} = \frac{\Delta M_{Ed}}{z \cdot h_f \cdot \Delta x} \cdot \frac{A_{sa}}{A_s} = \frac{216}{0.587 \cdot 0.35 \cdot 0.525} \cdot \frac{6.28}{15.7} = 801kN/m^2$

Shear force resistance $\cot\theta = 1.0$

$$v_{Rd,\max} = \frac{0.75 \cdot f_{cd}}{\cot\theta + \tan\theta} = \frac{0.75 \cdot 17}{1.0 + 1/1.0} \cdot 10^3 = 6375kN/m^2 > v_{Ed} = 801kN/m^2$$

Transverse reinforcement $a_{sw,req} = \frac{v_{Ed} \cdot h_f}{f_{yd} \cdot \cot\theta} = \frac{801 \cdot 0.35}{43.5 \cdot 1.0} = 6.44cm^2/m$

$$a_{s,u\_prov} = 3.35cm^2/m > a_{s,u\_req} = 6.44/2 = 3.22cm^2/m \;\checkmark$$

$$a_{s,l\_prov} = 5.5cm^2/m > a_{s,l\_req} = 6.44/2 = 3.22cm^2/m \;\checkmark$$

13.5.5 ANCHORAGE LENGTH AT THE SUPPORT

End Support A and D

Global offset $a_l = 0.9 \cdot 65 \cdot 1.2/2 = 35.1cm$ $F_{Ed} = \frac{V_{Ed} \cdot a_l}{z} = \frac{121 \cdot 35.1}{0.9 \cdot 65} = 72.6kN$

$$A_{s,req} = \frac{F_{Ed}}{f_{yd}} = \frac{72.6}{43.5} = 1.67cm^2$$

Anchorage length $l_{b,rqd} = 36cm$ *for C30/37 and good bond*

$$l_{bd} = \alpha_1 \cdot l_{b,rqd} \cdot \left(A_{s,req}/A_{s,prov}\right) = 0.7 \cdot 50 \cdot (5.98/6.16) = 33.98cm$$

$$> l_{b,\min} = 0.3 \cdot 0.7 \cdot 50 = 10.5cm > 10\phi_s = 14.0cm$$

$$l_{b,dir} = 2/3 \cdot l_{bd} = 2/3 \cdot 33.98 = \underline{\underline{22.7cm}} > 6.7\phi_s = 9.4cm$$

Intermediate Support B and C

$$l_{bd} = 6 \cdot \phi = 6 \cdot 1.4 = 8.4cm$$

13.5.6 OVERLAP LENGTH

Øs = 14

$$l_0 = \alpha_1 \cdot \alpha_6 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 1.0 \cdot 1.4 \cdot 50 \cdot (5.98 / 6.16) = 68cm$$

$$> l_{0,\min} = 21cm$$

Øs = 20

$$l_0 = \alpha_1 \cdot \alpha_6 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 1.0 \cdot 2.0 \cdot 89 \cdot (14.34 / 15.7) = 163cm$$

$$> l_{0,\min} = 53cm$$

13.5.7 REINFORCEMENT DRAWING

See Appendix A.7, page 193.

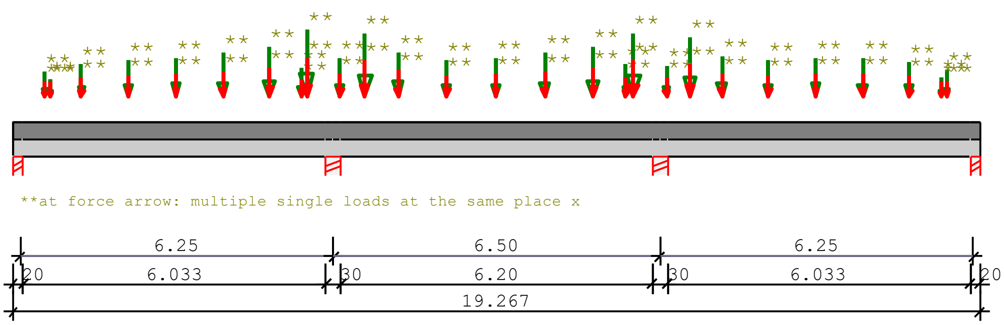
13.6 SOFTWARE CALCULATIONS

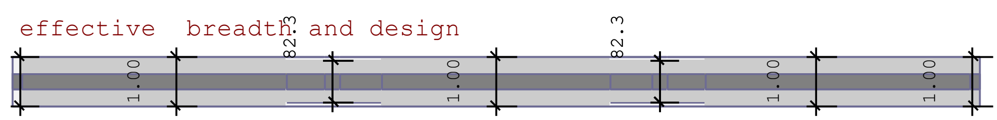
Demo Frilo Nemetschek

Position: B.13 – Semi-Precast Concrete Beam

Durchlaufträger DLT10 01/2018 (Frilo R-2018-1/P12)

Scale 1 : 150





Reinforced concrete girder over 3 Spans C30/37 E = 33000 N/mm2
DIN EN 1992-1-1/NA/A1:2015-12

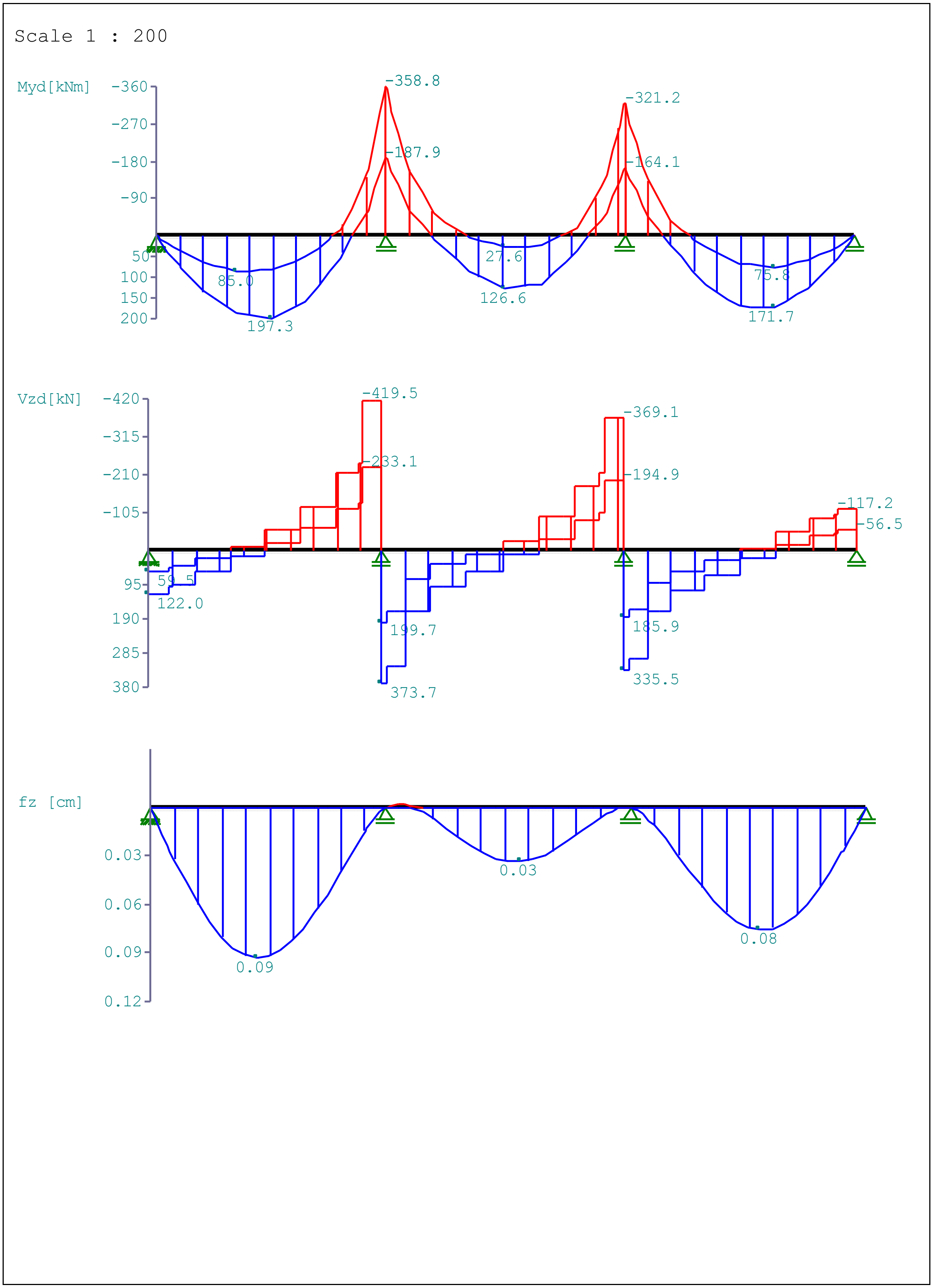
Decke über NG (Normalgeschoss) von Gebäudemodell

System length cross-section values

Span	l (m)		bt	ht	b0	h0	bb	hb
1	6.25	constant	100.0	35.0	30.0	70.0		
2	6.50	constant	100.0	35.0	30.0	70.0		
3	6.25	constant	100.0	35.0	30.0	70.0		

Cross-sections with eff. active breadth

x (m)	bt (cm)	ht (cm)	b0 (cm)	h0 (cm)	bb (cm)	hb (cm)	Wyb (m3)	Wyt (m3)
0.00	100.0	35.0	30.0	70.0			0.0327	0.0568
5.31	100.0	35.0	30.0	70.0			0.0327	0.0568
5.31	82.3	35.0	30.0	70.0			0.0311	0.0500
6.25	82.3	35.0	30.0	70.0			0.0311	0.0500
7.22	82.3	35.0	30.0	70.0			0.0311	0.0500
7.23	100.0	35.0	30.0	70.0			0.0327	0.0568
11.77	100.0	35.0	30.0	70.0			0.0327	0.0568
11.78	82.3	35.0	30.0	70.0			0.0311	0.0500
12.75	82.3	35.0	30.0	70.0			0.0311	0.0500
13.69	82.3	35.0	30.0	70.0			0.0311	0.0500
13.69	100.0	35.0	30.0	70.0			0.0327	0.0568
19.00	100.0	35.0	30.0	70.0			0.0327	0.0568


Scale 1 : 200
Myd[kNm]
-360
-270
-180
-90
50
100
150
200
-358.8
-187.9
-321.2
-164.1
85.0
197.3
27.6
126.6
75.8
171.7
Vzd[kN]
-420
-315
-210
-105
95
190
285
380
-419.5
-233.1
-369.1
-194.9
-117.2
-56.5
59.5
122.0
199.7
373.7
185.9
335.5
fz [cm]
0.03
0.06
0.09
0.12
0.09
0.03
0.08

```
Design DIN EN 1992-1-1/NA/A1:2015-12
FLBemBn.DLL: Version 9.0.1.121 (1)
C30/37  B500A normaly ductil
---------------------------------------------------------------------

Minimum reinforcement EN2 9.2.1.1  (9.1)   fctm =  2.90 N/mm2
Calculating Wy, breadth of slab is limited to 2*b0.
Q.No.   min Mb   req As    min Mo  req As
         (kNm)   (cm2)      (kNm)   (cm2)
---------------------------------------------------------------------
  1     94.80    3.19    -153.75    5.18   90.0/35.0/30.0/70.0

Span reinforcement
---------------------------------------------------------------------
Span      x       Myd    min Myd    d      kx     Asb   Ast
 No.     (m)     (kNm)     (kNm)   (cm)              (cm2)      comb
   1     3.13    197.3             66.0   0.05    6.7    0.0    B  2
         5.73   -150.7   -150.7    66.0   0.09    0.0    5.2    B  2
   2     3.25    126.6             66.0   0.04    4.3    0.0    B  2
         0.65   -154.5   -154.5    66.0   0.10    0.0    5.3    B  2
   3     4.06    171.7             66.0   0.05    5.8    0.0    B  2
         0.94    -86.1    -86.1    66.0   0.06    0.0    5.2 *  B  2
* Minimum reinforcement acc.to DIN EN 1992-1 9.2.1.1 (1)
On first support are at least 4.2 cm2 to be anchored.
On last support are at least 4.0 cm2 to be anchored.
Shear force VK-support is with F = V,Ed * Cot(Theta) / 2 considered.

Support reinforcement DIN EN 1992:2015 5.5
---------------------------------------------------------------------
Column    x      Myd    des.. Myd   d      kx      Asb   Ast
 No.     (m)    (kNm)     (kNm)   (cm)                (cm2)     comb
  1 ri   0.00      0.0                                              1
  2 le   0.15   -350.1   -236.2    66.0   0.14    0.0    8.4    B  2
  2 ri   0.15   -350.1   -244.5    66.0   0.14    0.0    8.7    B  2
  3 le   0.15   -310.7   -211.7    66.0   0.12    0.0    7.4    B  2
  3 ri   0.15   -310.7   -215.2    66.0   0.13    0.0    7.5    B  2
  4 le   0.00      0.0                                              1

shear force reinforcement B500A   DIN EN 1992-1-1/NA/A1:2015-12 6.2
---------------------------------------------------------------------
column  dist   kz     VEd      Θ    VRd,c   VRd,max a_max  asw
 No.    (m)            (kN)     (°  (kN)    (kN)    (cm)  (cm2/m) comb
  1 ri  0.52  0.90   122.0    18.4   67.0   685.1   30.0   2.8~  B  2
  1 ri  0.73  0.90    94.5    18.4   67.0   685.1   30.0   2.8~  B  2
  1 *   1.39  0.90    60.0    18.4   67.0   685.1   30.0   2.8~  B  2
  2 le  0.52  0.90  -419.5    29.6   71.6   980.7   30.0   9.2   B  2
  2 le  0.81  0.90  -213.6    29.6   71.6   980.7   30.0   4.7   B  2
  2 *   1.47  0.90  -122.6    29.6   67.0   980.7   30.0   2.8~  B  2
  2 ri  0.65  0.90   322.2    26.0   72.6   899.9   30.0   6.1   B  2
  2 ri  0.81  0.90   172.2    26.0   72.6   899.9   30.0   3.2   B  2
  2 *   1.47  0.90   102.0    26.0   72.6   899.9   30.0   2.8~  B  2
  3 le  0.50  0.90  -369.1    28.0   68.8   946.7   30.0   7.6   B  2
  3 le  0.81  0.90  -180.2    28.0   68.8   946.7   30.0   3.7   B  2
  3 *   1.47  0.90   -93.4    28.0   67.0   946.7   30.0   2.8~  B  2
  3 ri  0.62  0.90   303.6    25.0   69.2   875.1   30.0   5.5   B  2
  3 ri  0.81  0.90   172.8    25.0   69.2   875.1   30.0   3.1   B  2
  3 *   1.47  0.90   114.9    25.0   69.2   875.1   30.0   2.8~  B  2
  4 le  0.52  0.90  -117.2    18.4   67.0   685.1   30.0   2.8~  B  2
  4 le  0.73  0.90   -88.5    18.4   67.0   685.1   30.0   2.8~  B  2
  4 *   1.39  0.90   -51.3    18.4   67.0   685.1   30.0   2.8~  B  2
```

~ at the end of line: Minimum stirrup reinforcement
max distance of stirrups will with Θ >= 40° investigated (paper DAfStb 525).
reduction of single loads is deactivated

shoulder shear

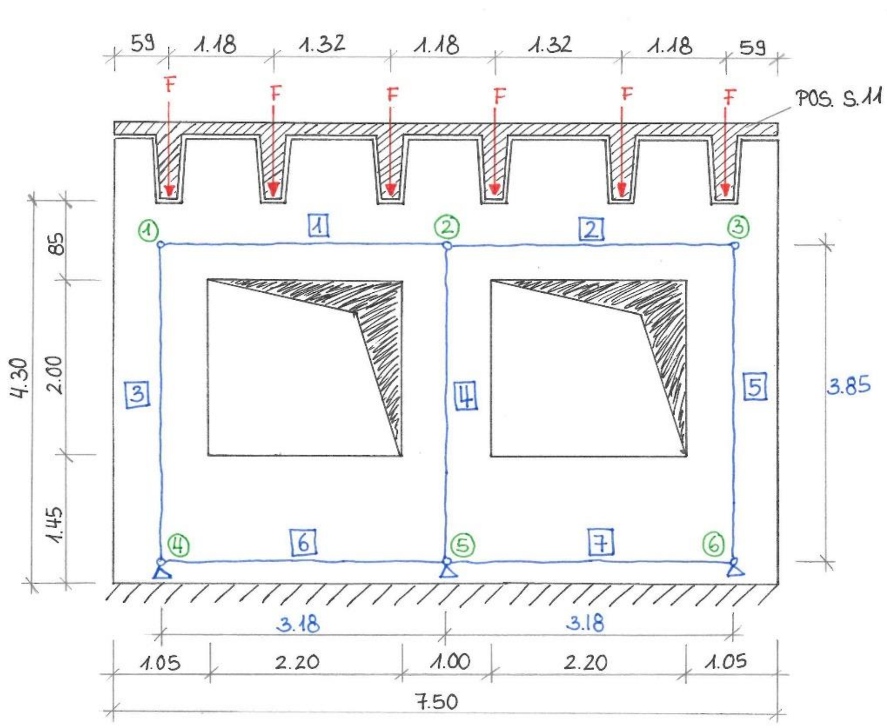
Span	xa (cm)	xe (cm)	mle (kNm)	mri (kNm)	av (cm)	beff (cm)	dFcd (kN)	vEd (kN/m2)	vEd,perm.	asf (cm2/m)
1	0	156	0.0	150.8	156	100	89	162	6278	1.1
1	156	313	150.8	197.3	156	100	27	50	6278	0.3
1	313	422	197.3	139.3	110	100	34	89	6278	0.6
1	422	532	139.3	-0.1	110	100	82	214	6278	1.4
2	125	225	1.1	84.0	100	100	49	139	6278	0.9
2	225	325	84.0	126.6	100	100	25	71	6278	0.5
2	325	437	126.6	103.9	112	100	13	34	6278	0.2
2	437	549	103.9	-0.2	112	100	61	156	6278	1.0
3	107	257	0.2	137.8	150	100	81	155	6278	1.0
3	257	406	137.8	171.7	150	100	20	38	6278	0.3
3	406	516	171.7	112.2	109	100	35	91	6278	0.6
3	516	625	112.2	1.2	109	100	65	171	6278	1.1
3	516	625	112.2	1.2	109	100	65	171	6278	1.1
3	516	625	112.2	1.2	109	100	65	171	6278	1.1

14 POS. W.14 – LOAD BEARING PRECAST CONCRETE WALL PANEL

14.1 SYSTEM

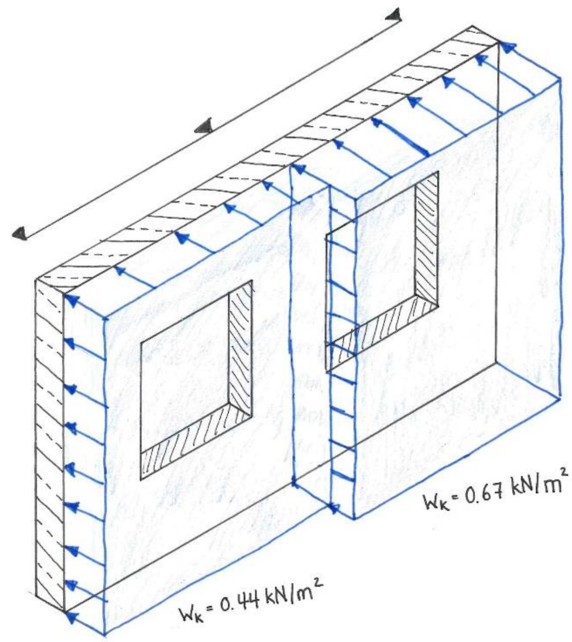
The concrete wall panel functions as the support of the double-tee slab and has to carry the resulting vertical loads as well as horizontal wind loads. For the design of the wall panel a simplified system as shown below is sufficient to describe the load paths and calculate the required reinforcement. A more in-depth analysis of the concrete wall will be done in the following chapters.

System



Cross-section	h = 20 cm
Building materials	**C 35/45 | B 500A**
Exposure class	XC1, WO
Reinforcing steel	c_{nom} = 10 mm + 10 mm = 20 mm *Chosen*: c_{nom} = 25 mm
Characteristic loads	

Concentrated vertical loads	**G_k [kN]**	**Q_k [kN]**
Self-weight (generated by the software)	-	-
½ x Reaction Force from *POS. S.11*	88.0	31.0



Uniformly distributed horizontal loads			w_k [kN/m]
Column 3/5	$w_{k,A} \approx 0.40 \times 3.18 \times 0.67$ kN/m² =	-	0.85
Column 4	$w_{k,A} \approx 0.60 \times 3.18 \times (0.67 + 0.44)$kN/m² =	-	2.12

14.2 INTERNAL FORCES

14.2.1 INTERNAL FORCES RESULTING FROM VERTICAL LOADS

Flexure moment

	M_{Gk} [kNm]	M_{Qk} [kNm]	M_{Ed} [kNm]
Truss 1 and 2	64.2	21.4	118.7
Node 2	- 79.8	- 26.6	- 147.7
Truss 6 and 7	4.8	0.0	6.45
Node 5	- 7.4	0.0	- 10.0

Shear force

	V_{Gk} [kN]	V_{Qk} [kN]	V_{Ed} [kN]
Node 1 and 3	50.9	16.0	92.8
Node 2	- 138.6	- 46.0	- 256.1
Node 4 and 6	9.7	0.0	13.1
Node 5	- 13.3	0.0	- 18.0

Normal force

	N_{Gk} [kNm]	N_{Qk} [kNm]	N_{Ed} [kNm]
Truss 3 and 5	- 138.9	- 47.0	- 258.1
Truss 4	- 277.2	- 91.9	- 512.1

14.2.2 INTERNAL FORCES RESULTING FROM HORIZONTAL LOADS

Flexure moment

	M_{wk} [kNm]	M_{Ed} [kNm]
Column A / C	2.0	2.9
Column B	4.9	7.4

Shear force

	V_{wk} [kN]	V_{Ed} [kN]
Column A / C	1.8	2.8
Column B	4.6	6.8

14.3 ULTIMATE LIMIT STATE DESIGN

14.3.1 FLEXURE DESIGN OF TRUSS 1 AND 2

Lower Reinforcement Truss 1 and 2

Cross-section *b / h = 20 / 85 cm*

Effective depth *d = 85 – 2.5 – 1 – 2.0/2 = 80.5 cm*

Lower reinforcement

$$\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{11870 kNcm}{20 \cdot 80.5^2 \cdot 1.98 kN / cm^2} = 0.046 \quad \rightarrow \omega = 0.0476$$

$$A_{sl,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = 0.0476 \cdot 20 \cdot 80.5 \cdot 1.98 \cdot \frac{1}{43.5} = 3.5 cm^2$$

Chosen **2 Ø 16 = 4.02 cm²** **lower reinforcement**

Upper Reinforcement Node 2

Cross-section *b / h = 20 / 85 cm*

Effective depth *d = 85 – 2.5 – 1 – 2.0/2 = 80.5 cm*

Lower reinforcement

$$\mu_{Eds} = \frac{M_{Eds}}{b \cdot d^2 \cdot f_{cd}} = \frac{14770 kNcm}{20 \cdot 80.5^2 \cdot 1.98 kN / cm^2} = 0.058 \quad \rightarrow \omega = 0.060$$

$$A_{sl,req} = \omega \cdot b \cdot d \cdot f_{cd} \cdot \frac{1}{\sigma_{sd}} = 0.060 \cdot 20 \cdot 80.5 \cdot 1.98 \cdot \frac{1}{43.5} = 4.40 cm^2$$

Chosen **2 Ø 20 = 6.28 cm²** **upper reinforcement**

14.3.2 DESIGN OF COLUMN A/C AND B

The design of column B is decisive for the structure due to its smaller cross-section and higher loads.

Column B

Cross-section — *b / h = 100 / 20 cm*

Slenderness of the column — $\beta = 1.0 \qquad l_0 = 1.0 \cdot 4.30 = 4.30m$

$$i = 0.289 \cdot h = 0.289 \cdot 20 = 5.78cm \qquad \lambda = \frac{l_0}{i} = \frac{430}{5.78} = 74.4$$

Limit slenderness

$$|n_{Ed}| = \frac{N_{Ed}}{A_c \cdot f_{cd}} = \frac{512.1}{100 \cdot 20 \cdot 1.98} = 0.13 < 0.41$$

$$\lambda_{\lim} = \frac{16}{\sqrt{|n_{Ed}|}} = \frac{16}{\sqrt{0.13}} = 44.4$$

$\lambda = 74.4 > \lambda_{\lim} = 44.4$ → second order effects must be considered!

Eccentricity due to imperfections

$$\alpha_h = \frac{2}{\sqrt{l}} = \frac{2}{\sqrt{4.30}} = 0.96 \qquad \theta_i = \frac{1}{200} \cdot \alpha_h = \frac{1}{200} \cdot 0.96 = 4.82 \cdot 10^{-3}$$

$$e_i = \theta_i \cdot \frac{l_0}{2} = 4.82 \cdot 10^{-3} \cdot \frac{4.30}{2} = 0.01m$$

Eccentricity due to second order effects

$$e_2 = K_1 \cdot \frac{l_0^2}{10} \cdot \frac{1}{r} = 1.0 \cdot \frac{4.30^2 \cdot 0.0022}{10 \cdot 0.45 \cdot 0.155} = 0.06m$$

Total eccentricity — $e_{tot} = e_i + e_2 = 0.01 + 0.06 = 0.07m$

Load combinations

LC1: $1.35 \cdot G_k \oplus 1.50 \cdot Q_k \oplus 1.50 \cdot \psi_0 \cdot w_k \qquad \psi_0 = 0.6$

LC2: $1.0 \cdot G_K \oplus 1.5 \cdot w_k$

Flexure moment — $M_{Eds} = M_{Ed} + N_{Ed} \cdot e_{tot}$

LC1: $M_{Eds} = 0.6 \cdot 7.4 + 0.07 \cdot 512.1 = 40.3kNm$

LC2: $M_{Eds} = 7.4 + 0.07 \cdot 1.35 \cdot 277.2 = 33.6kNm$

The flexure Moment is very small and can be neglected for the design of the column.

Design — $N_{Ed} = 512.1kN < \max N_{Ed} = A_c \cdot f_{cd} = 100 \cdot 20 \cdot 1.98 = 3960kN$ ✓

14.4 DESIGN AND REINFORCEMENT

14.4.1 MINIMUM REINFORCEMANT FOR CONCRETE WALLS

Normal force $N_{Ed} = 512.1kN < 0.3 \cdot f_{cd} \cdot A_c = 0.3 \cdot 1.98 \cdot 100 \cdot 20 = 1188kN$

Minimum reinforcement $a_{s,v,\min} = 0.15 \cdot \frac{|N_{Ed}|}{f_{yd}} = 0.15 \cdot \frac{512.1}{43.5} = 1.77cm^2/m$ *vertical reinforcement*

$$\geq 0.003 \cdot A_c = 0.003 \cdot 20 = 0.06cm^2/m$$

$a_{s,h,\min} \geq 0.5 \cdot a_{s,v} = 0.5 \cdot 1.77 = 0.885cm^2/m$ *horizontal reinforcement*

15 POS. W.14 A – STRUT-AND-TIE MODEL OF THE CONCRETE WALL

In Chapter 14 the design calculations for the concrete wall panel are made based on a very simple static system. For comparison a more detailed analysis based on a strut and tie model is made in the following chapter. Finite element analysis is used to generate the stress trajectories of the wall and in accordance with these a strut and tie model is developed which can be used to design the reinforcement of the wall.

15.1 SYSTEM

Further see Chapter 14.1.

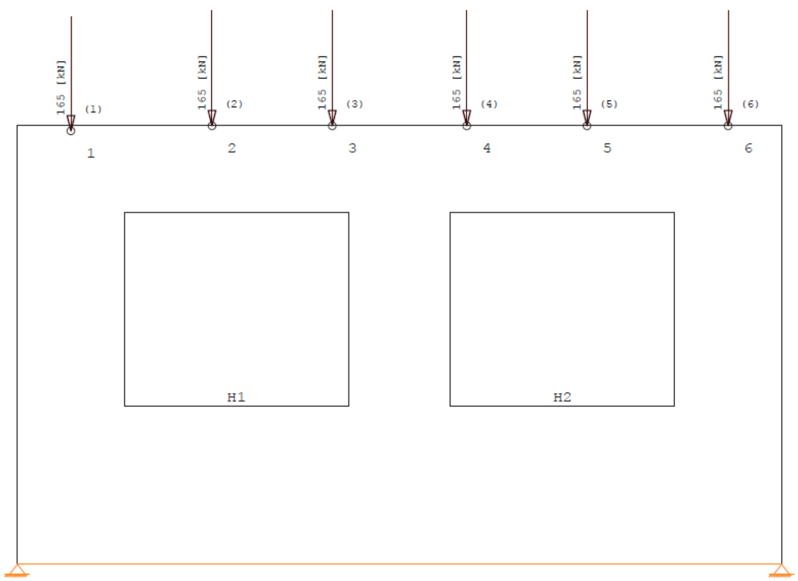


Figure 18 - Static System of the Concrete Wall

15.2 STRESS TRAJECTORIES

The stress trajectories are computed with the program *Nemetschek FRILO*. To find the best matching stress trajectories and therefore strut and tie model, three different systems are analyzed and at the end one is chosen to be used for the design calculations. The three system that are analyzed are a single-span deep beam, a double-span deep beam and a wall with a line support.

Due to their low impact, the wind loads are neglected in this analysis and only the concentrated loads from the double-tee slab are regarded.

15.2.1 SINGLE-SPAN DEEP BEAM

Figure 19 shows the stress trajectories for the single-span deep beam.

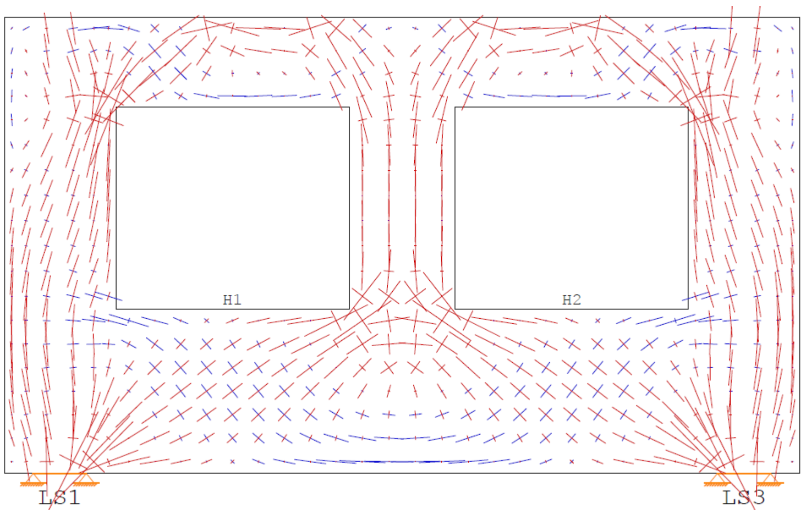


Figure 19 - Stress Trajectories for a Single-Span Deep Beam

The stresses above the window holes look similar to the stress trajectories of a double span beam. The 'columns' between the window holes act as supports for the beam and tension ties form above them. Another thing that must be noted is the development of a big tension tie at the lower edge of the wall between the two supports. The bottle-shape of the two compression struts coming from the intermediate column leading to the two supports can be seen clearly.

15.2.2 DOUBLE-SPAN DEEP BEAM

In Figure 20 an intermediate support is added, making it a double-span deep beam.

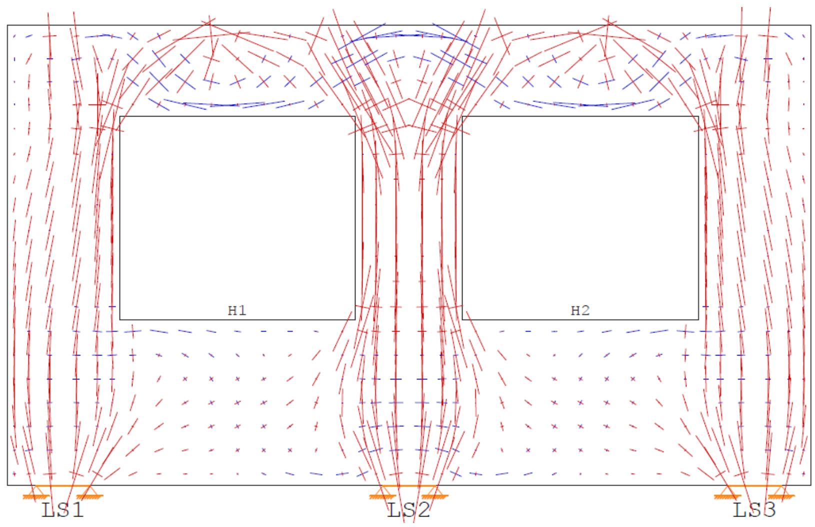


Figure 20 - Stress Trajectories for a Double-Span Deep Beam

The stress trajectories above the windows are very similar to the ones of the single-span deep beam. The part above the windows acts like a regular double-span beam. The compression struts go directly to the support and there is almost no tension at the lower edge of the wall. Most of the tension results from the transverse tension of the compression struts due to their bottle-shape.

15.2.3 CONTINUOUS SUPPORT

Finally, the third system can be seen in Figure 21 – a deep beam with a continuous support.

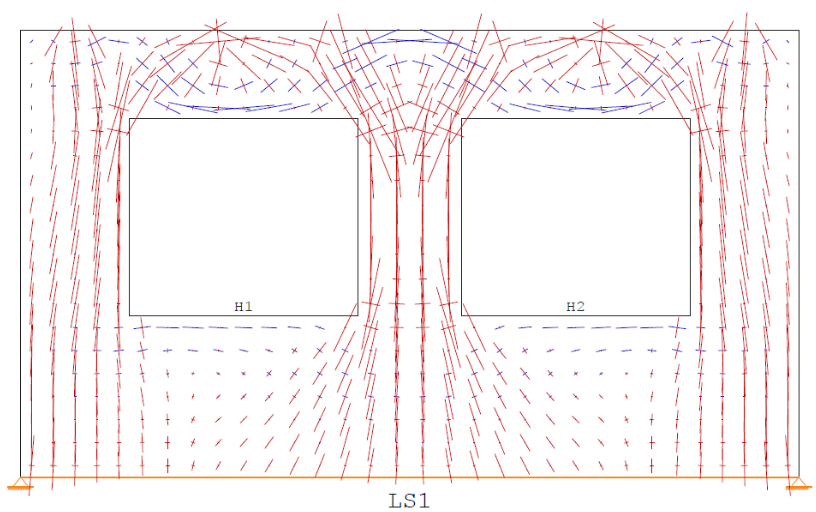


Figure 21 - Stress Trajectories for a Deep Beam with a Continuous Support

Again, the stresses above the windows resemble the previous two systems. The outer compression struts go to the support in a straight line and so is the compression strut in the middle for the most part. The compression stresses fan out closer to the lower edge of the wall. Tension ties develop horizontally under the window holes.

15.2.4 CONCLUSION

The stresses above the window resemble each other in all three systems and act like a double-span beam so for that part the path of the loads is very clear. System two and three clearly show that the compression struts will then follow along the 'columns' between the holes in the wall and directly lead to the support. A development seen in system one is unlikely to occur since the loads will follow the shortest path to the supports. Therefore, system two (Figure 20) seems to depict the load paths and stress trajectories of the wall the best and will be used to develop a strut and tie model and to design the reinforcement of the wall.

15.3 THE STRUT-AND-TIE MODEL

15.3.1 REACTION FORCES AT THE SUPPORT

The reaction forces can be assumed to be the mean value between the reaction forces of a double-span beam and two single-span beams.

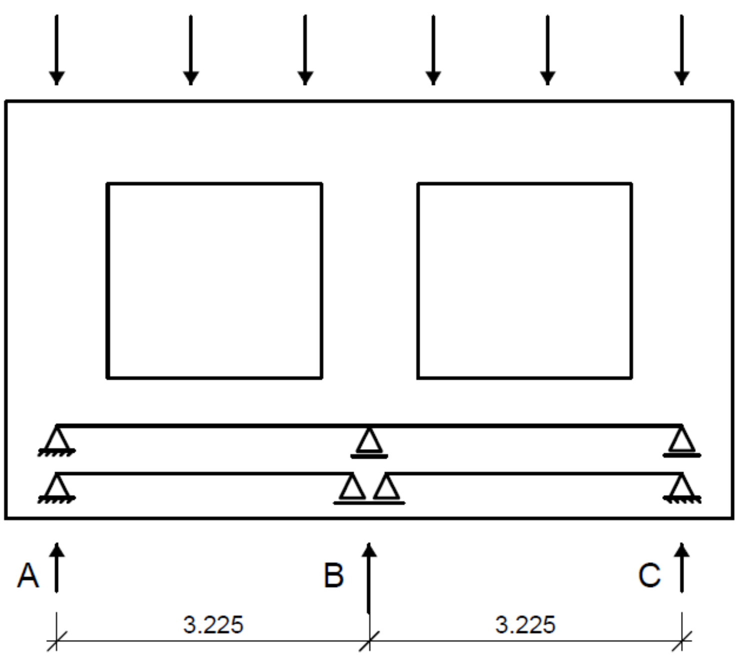
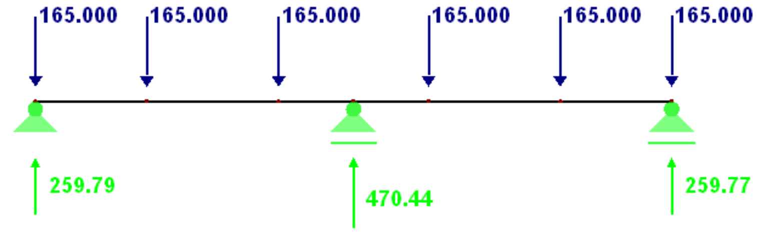
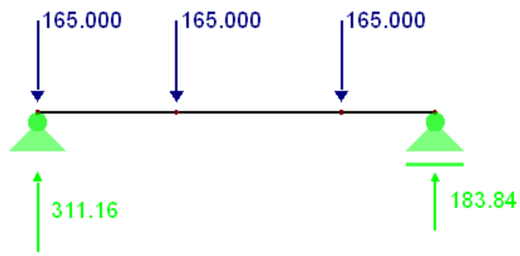


Figure 22 - Static System to Calculate the Reaction Forces of the Concrete Wall

Reaction forces for a double-span beam



Reaction forces for a single-span beam

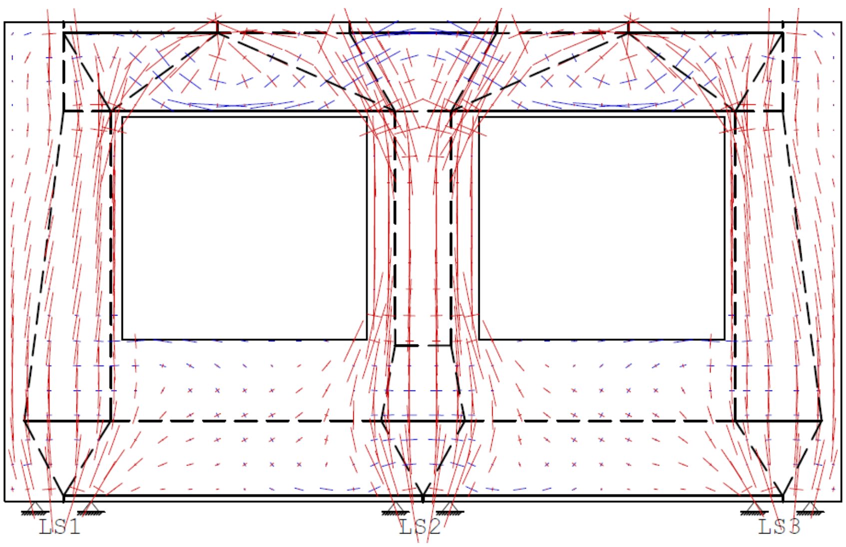


Reaction forces of the wall

	A	B	C
Double-Span	259.8 kN	470.4 kN	259.8 kN
2 x Single-Span	311.2 kN	367.7 kN	311.2 kN
Average	**285.5 kN**	**419.1 kN**	**285.5 kN**

15.3.2 STRUT-AND-TIE MODEL

The final strut and tie model can be seen in Figure 23. Dashed lines illustrate the compression struts and solid lines illustrate the tension ties.



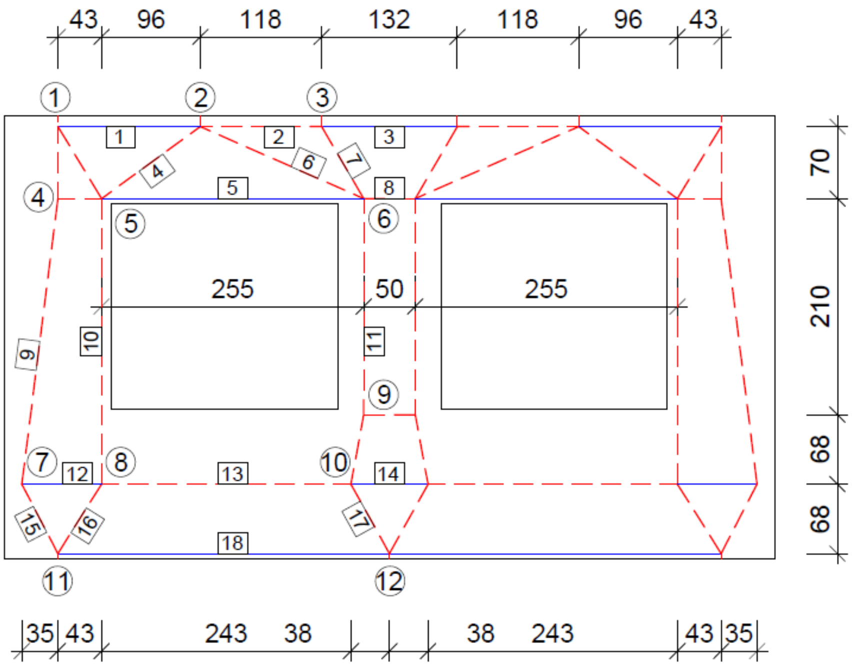


Figure 23 - Final Strut-and-Tie Model

15.3.3 RESULTING FORCES

The resulting forces in the struts and ties are calculated with the program *Dlubal R-Stab* and are summarized in Table 22. Compression struts are shown in red and with a negative sign and the tension ties are shown in blue and with a positive sign.

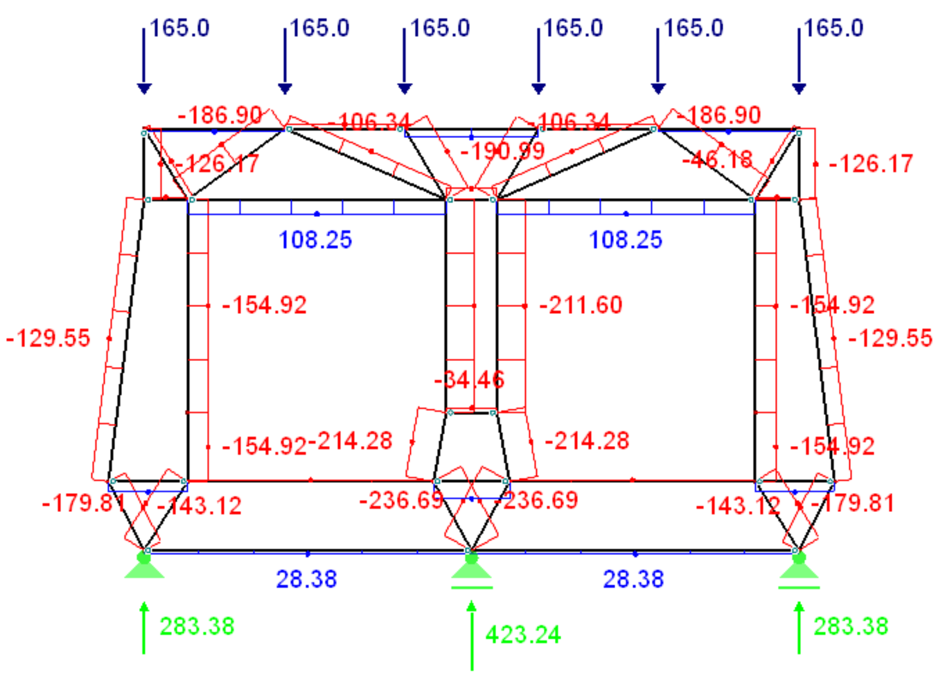


Figure 24 - Result of the Dlubal R-Stab Calculation

Table 22 - Resulting Forces of the Strut-and-Tie Model

Truss	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
Force [kN]	21.2	- 31.0	65.2	- 186.9	108.3	- 106.3	- 191.0	- 83.0	- 129.6

10	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**
- 154.9	- 211.6	79.7	- 10.2	130.3	- 143.1	- 179.8	- 236.7	28.4

15.4 DESIGN AND REINFORCEMENT

15.4.1 TENSION TIES

Truss 1 $A_{s,req} = \frac{F_{Ed}}{f_{yd}} = \frac{21.2}{43.5} = 0.49cm^2$ → ***2 Ø 8 = 1.01 cm²***

Truss 3 $A_{s,req} = \frac{F_{Ed}}{f_{yd}} = \frac{65.2}{43.5} = 1.50cm^2$ → ***2 Ø10 = 1.57 cm²***

Truss 5 $A_{s,req} = \frac{F_{Ed}}{f_{yd}} = \frac{108.3}{43.5} = 2.49cm^2$ → ***2 x 2 Ø10 = 3.14 cm²***

Truss 12 $A_{s,req} = \frac{F_{Ed}}{f_{yd}} = \frac{79.7}{43.5} = 1.83cm^2$ → ***2 x 2 Ø10 =3.14 cm²***

Truss 14 $A_{s,req} = \frac{F_{Ed}}{f_{yd}} = \frac{130.3}{43.5} = 3.00cm^2$ → ***2 x 2 Ø10 =3.14 cm²***

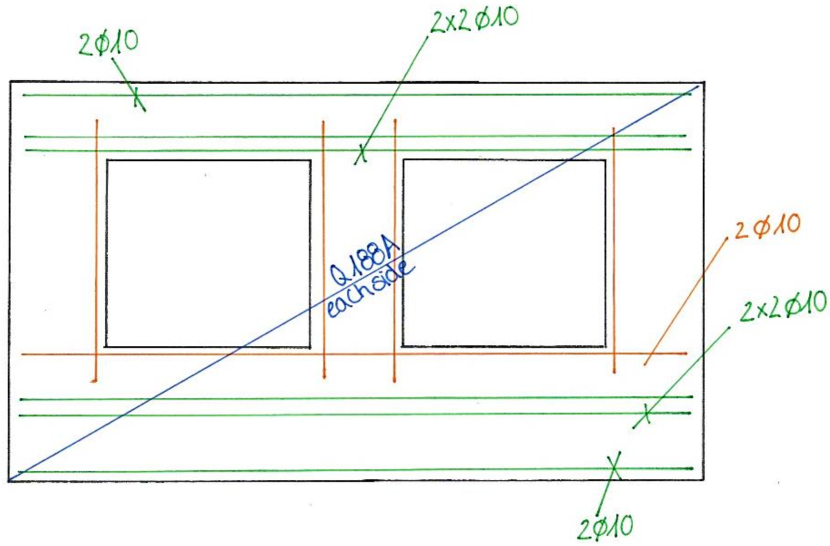
Truss 18 $A_{s,req} = \frac{F_{Ed}}{f_{yd}} = \frac{28.4}{43.5} = 0.65cm^2$ → ***2 Ø 8 = 1.01 cm²***

15.4.2 MINIMUM REINFORCEMENT

$$a_{s,\min} = \max\begin{cases} 0.75 \cdot 10^{-3} \cdot a_c = 0.75 \cdot 10^{-3} \cdot 20 \cdot 100 = 1.5cm^2/m \\ 1.5cm^2/m \end{cases}$$

Chosen **Q188 A = 1.88 cm²/m** **on both sides of the wall**

15.4.3 REINFORCEMENT DRAWING



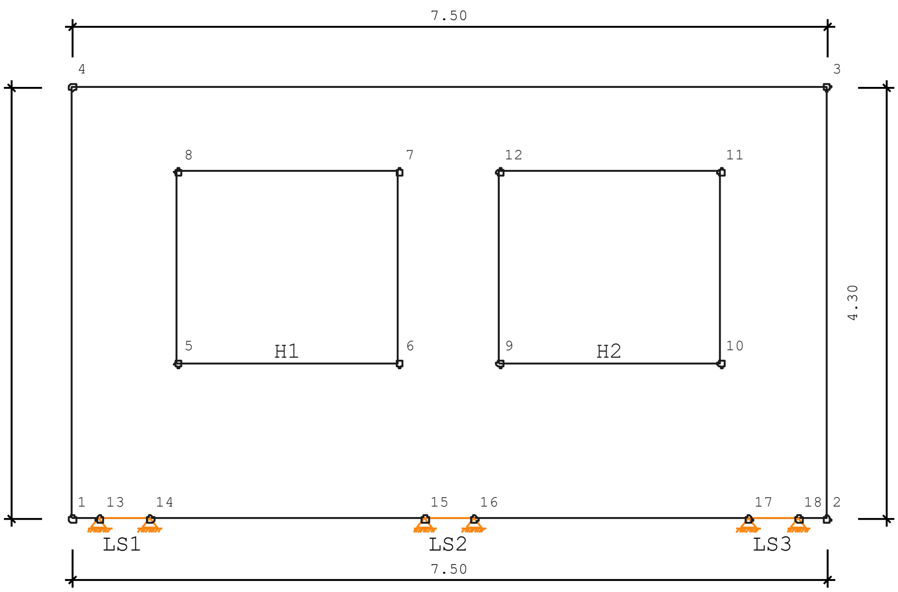
15.5 SOFTWARE CALCULATIONS

Demo Frilo Nemetschek

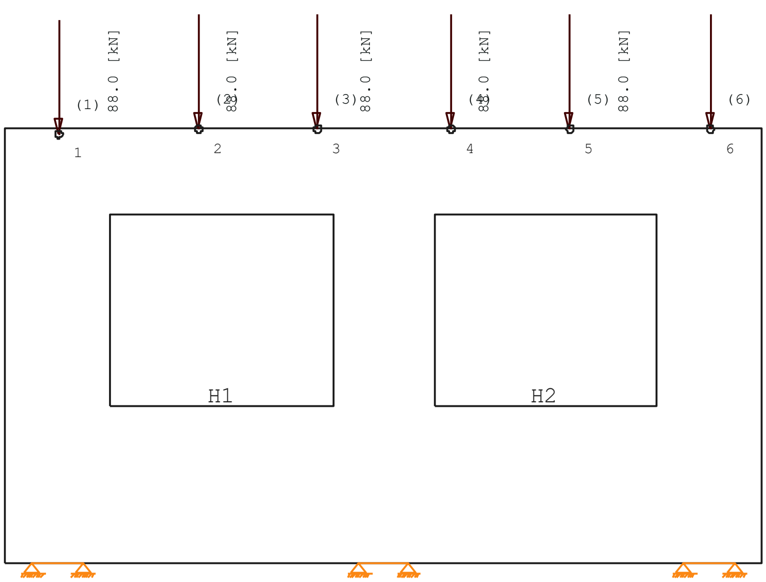
Position: W.14a – Load Bearing Concrete Wall

Panels by Finite Elements SCN 01/2018 (Frilo R-2018-1/P12)

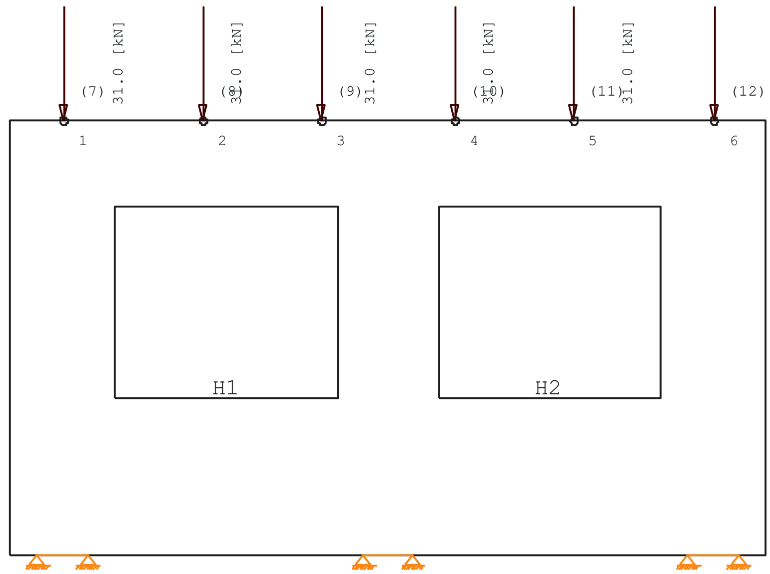
System
Scale 1 : 75



Load case 1 "POS. S.11 - Dead Load"
Point loads
Scale 1 : 75



Load case 2 "POS. S.11 - Live Load"
Point loads
Scale 1 : 75



SUPERPOSITION 1 "Characteristic"

Load Cases Involved

Number	Load case	Type	Self Self-Weight	Action Shrt Hnd	Action Name	Alter-native Group
1	POS. S.11 - Dea...	permanent	yes	g	Ständige ...	-
2	POS. S.11 - Liv...	non perm	no	1	Wohnräume	0

Action

Number	Shrt Hnd	Name	Type
1	1	Wohnräume	non perm
2	g	Ständige Lasten	permanent

SUPERPOSITION 2 "ULS Permanent/Transient"

Load Cases Involved

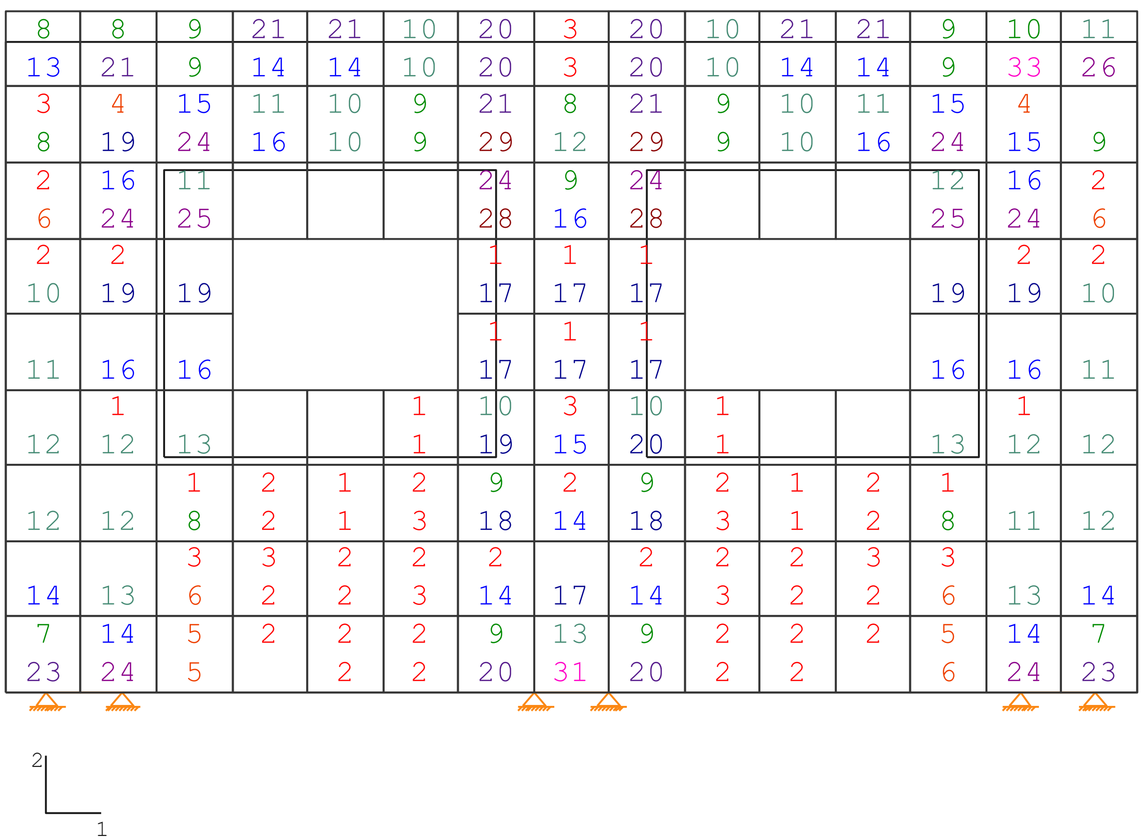
Number	Load case	Type	Self Self-Weight	Action Shrt Hnd	Action Name	Alter-native Group
1	POS. S.11 - Dea...	permanent	yes	g	Ständige ...	-
2	POS. S.11 - Liv...	non perm	no	1	Wohnräume	0

Action

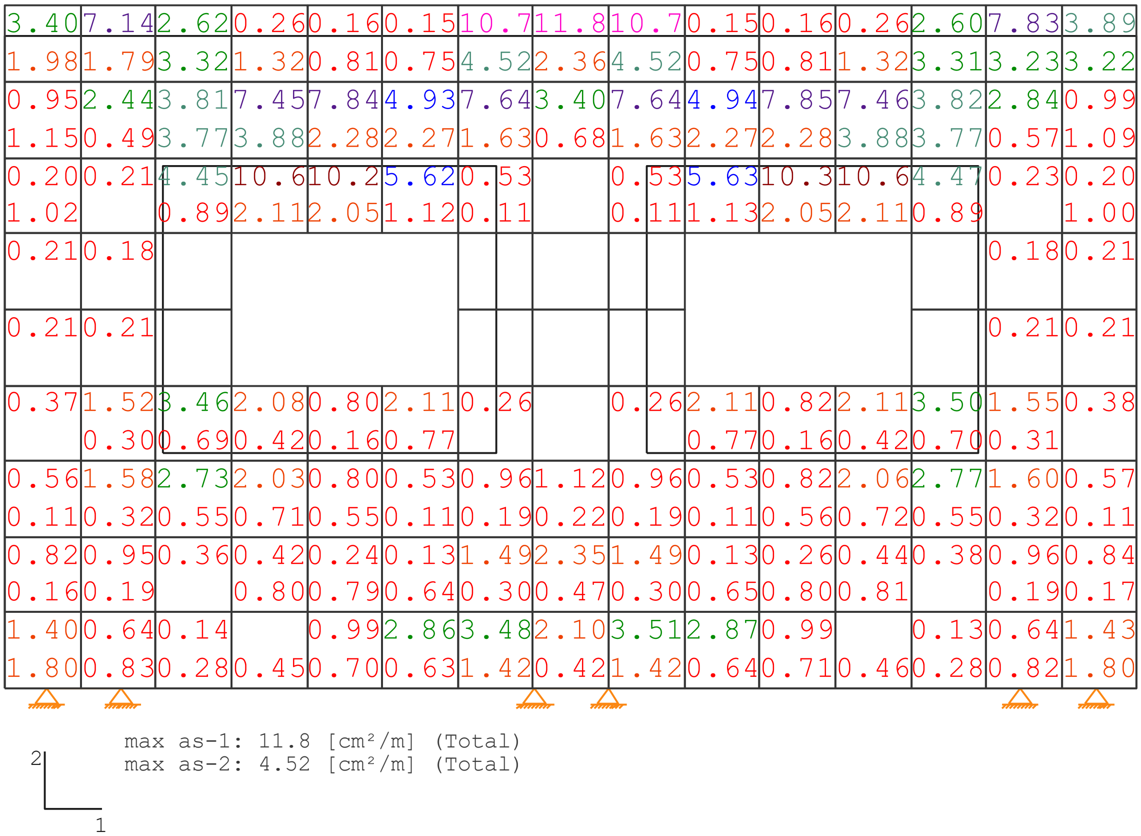
```
--------------------------------------------------------------------
Number Shrt Name                    Type Partial Safety  Combination
        Hnd                                sup   inf       dom   ndo
--------------------------------------------------------------------
     1     1 Wohnräume            non perm      1.50 0.00    1.00 0.70
     2     g Ständige Lasten     permanent      1.35 1.00    1.00 1.00
```

NOTE: Design Values
All results of a superposition of load cases
Include both partial safety and combination
factors: DIN EN 1990/NA:2010-12

NOTE: Combination Factors
With the combination of independent, variable actions
the individual dominant action is determined for both each
location and for each action quantity.
In geneneral, the dominant actions differ with each
location and each quantity
.
The individually found dominant action receives the
conbination factor 1.00. In case of only one variable action
this action is considered dominant.
Superposition 2 "ULS Permanent/Transient"
Concrete Load (Compressive) uC-1, uC-2 [%]
Scale 1 : 50



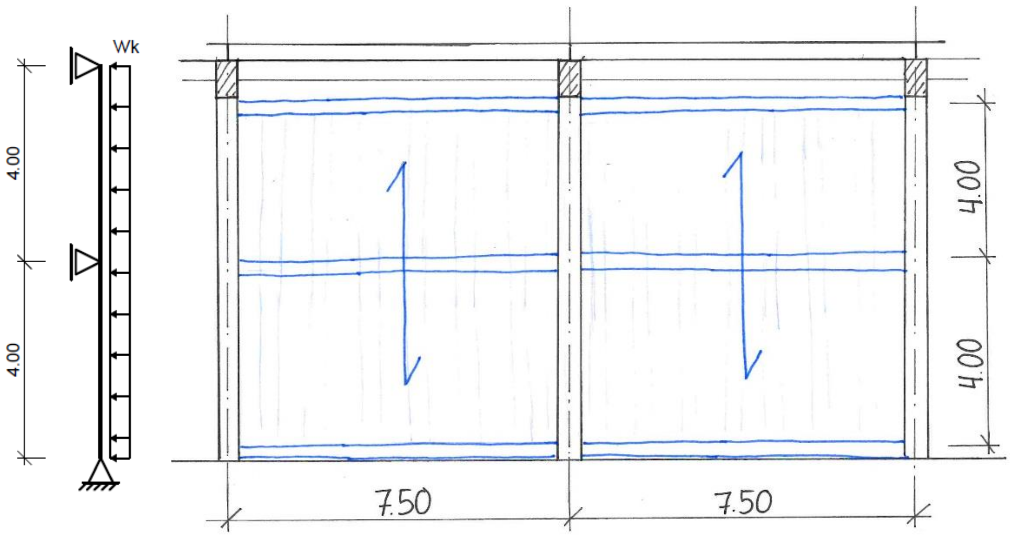
Superposition 2 "ULS Permanent/Transient"
Reinforcement, Total aS-1, aS-2 [cm²/m] Total
Scale 1 : 50



16 POS. W.15 – TRAPEZOIDAL SHEET CLADDING

16.1 SYSTEM

System



Profile

Hoesch Thermowand VL 80

$t_{N,a}$ = 0.75 mm (external sheet thickness)

$t_{N,i}$ = 0.50 mm (internal sheet thickness)

Materials

f_{yk} = 320 N/mm²

Characteristic loads

Uniformly Distributed Loads			g_k [kN/m²]	q_k [kN/m²]
Wind Area A	(wind suction)	$w_{k,A}$ =	-	- 0.67
Wind Area D	(wind pressure)	$w_{k,D}$ =	-	+ 0.39

16.2 ULTIMATE LIMIT STATE DESIGN

16.2.1 WIND SUCTION

The design of the sheet cladding is done according to the sizing table of the producer HOESCH.

$$w_{k,A} = -0.67 kN/m^2 < w_{k,\max} = -1.21 kN/m^2 \checkmark$$

Zweifeldträger, zul. We [kN/m²] - Wind, abhebend

Stützweite L[m]		1,50	1,75	2,00	2,25	2,50	2,75	3,00	3,25	3,50	3,75	4,00	4,25	4,50	4,75	5,00	5,25	5,50	5,75	6,00	6,25	6,50	6,75	7,00
We,10	Farbgruppe I	5,52	4,64	3,99	3,51	3,09	2,56	2,15	1,83	1,58	1,38	1,21	1,07	0,96	0,86	0,77	0,70	0,64	0,58	0,54	0,50	0,46		
$n_A \le 3$ 3)	Farbgruppe I	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3		
We,1	Farbgruppe I	5,52	4,64	3,99	3,51	3,12	2,82	2,57	2,36	2,18	2,03	1,81	1,61	1,43	1,29	1,16	1,05	0,96	0,88	0,81	0,74	0,69		
$n_B \le 6$	Farbgruppe I	6	6	6	6	6	6	6	6	6	6	6	6	6	5	5	5	5	5	4	4	4		
We,10	Farbgruppe II	5,38	4,52	3,89	3,42	3,04	2,56	2,15	1,83	1,58	1,38	1,21	1,07	0,96	0,86	0,77	0,70	0,64	0,58	0,54	0,50	0,46		
$n_A \le 3$	Farbgruppe II	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3		
We,1	Farbgruppe II	5,38	4,52	3,89	3,42	3,04	2,75	2,50	2,30	2,13	1,99	1,81	1,61	1,43	1,29	1,16	1,05	0,96	0,88	0,81	0,74	0,69		
$n_B \le 6$	Farbgruppe II	6	6	6	6	6	6	6	6	6	6	6	6	6	6	5	5	5	5	5	4	4		
We,10	Farbgruppe III	5,17	4,34	3,73	3,28	2,92	2,56	2,15	1,83	1,58	1,38	1,21	1,07	0,96	0,86	0,77	0,70	0,64	0,58	0,54	0,50	0,46		
$n_A \le 3$	Farbgruppe III	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3		
We,1	Farbgruppe III	5,17	4,34	3,73	3,28	2,92	2,64	2,41	2,22	2,06	1,92	1,80	1,61	1,43	1,29	1,16	1,05	0,96	0,88	0,81	0,74	0,69		
$n_B \le 6$	Farbgruppe III	6	6	6	6	6	6	6	6	6	6	6	6	6	6	5	5	5	5	5	5	4		

Figure 26 - Sizing Table of the Sheet Cladding 'Hoesch Thermowand VL 80' for Wind Suction

16.2.2 WIND PRESSURE

The design of the sheet cladding is done according to the sizing table of the producer HOESCH.

$$w_{k,A} = +0.39 kN/m^2 < w_{k,\max} = +0.56 kN/m^2 \checkmark$$

Zweifeldträger, zul. We [kN/m²] - Wind, andrückend

Stützweite L[m]	1,50	1,75	2,00	2,25	2,50	2,75	3,00	3,25	3,50	3,75	4,00	4,25	4,50	4,75	5,00	5,25	5,50	5,75	6,00	6,25	6,50	6,75	7,00
Breite b_A = 40 [mm]	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40		
We,10	1,50	1,29	1,13	1,00	0,90	0,82	0,75	0,69	0,64	0,60	0,56	0,53	0,50	0,47	0,45	0,43	0,41	0,39	0,38	0,33	0,29		
Breite b_B 3) = 60 [mm]	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60		
Breite $b_A \le 60$ [mm]	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40		
We,10	2,00	1,72	1,50	1,33	1,20	1,09	1,00	0,92	0,86	0,80	0,75	0,71	0,67	0,63	0,60	0,57	0,51	0,44	0,38	0,33	0,29		
Breite $b_B \le 80$ [mm]	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	80	74	67	61	60	60		
Breite $b_A \le 80$ [mm]	63	62	62	63	62	63	63	62	63	62	62	63	60	53	47	41	40	40	40	40	40		
We,10	3,13	2,68	2,35	2,09	1,88	1,71	1,57	1,44	1,34	1,25	1,17	1,11	1,01	0,84	0,70	0,59	0,51	0,44	0,38	0,33	0,29		
Breite $b_B \le 125$[mm]	125	125	125	125	125	125	125	125	125	125	125	125	121	106	93	83	74	67	61	60	60		

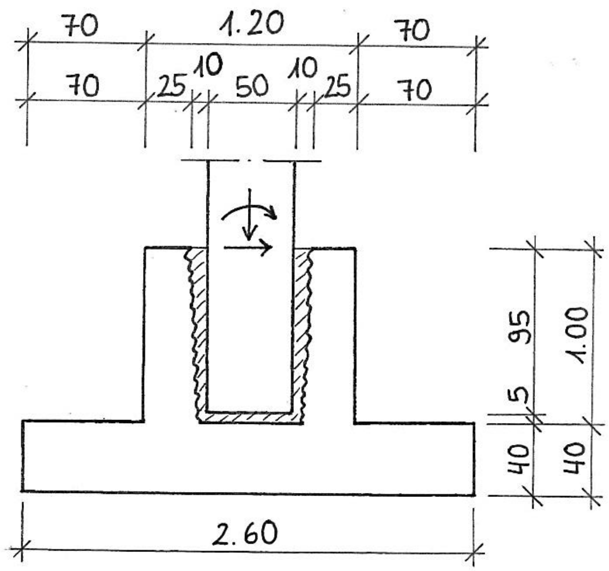
Figure 27 - Sizing Table of the Sheet Cladding 'Hoesch Thermowand VL 80' for Wind Pressure

Chosen	**Hoesch Thermowand VL 80**

17 POS. F.21 – POCKET FOUNDATION OF COLUMN C.04

17.1 SYSTEM

System



Cross-section	b_x x b_y x h = 2.60 x 2.60 x 0.40 m
Building materials	**C 35/45 | B 500A**
Exposure class	XC2
Concrete cover	c_{nom} = 20 mm + 15 mm = 35 mm
Characteristic loads	Reaction forces from *POS. C.04* – see Chapter 8.2.4

17.2 SOIL-BEARING CAPACITY

The maximum characteristic bearing capacity of the soil is $\sigma_{R,k}$ = 400 kN/m². The size of the shallow foundation has to be chosen so the stresses caused by the load on the footing do not exceed the maximum bearing capacity, which means

$$\sigma_{Ed} = \frac{N_{Ed,s}}{A'} \leq \sigma_{Rd} = 1.4 \cdot \sigma_{Rk} = 1.4 \cdot 400 = 560 kN/m^2 \quad \text{and} \quad A' = (b_x - 2e_x)(b_y - 2e_y)$$

$$N_{Ed,s} = N_{Ed} + 1.0/1.35 \cdot G_k \qquad G_k = 2.6 \cdot 2.6 \cdot 0.4 \cdot 25 kN/m^3 = 67.6 kN \quad \text{(self-weight of the footing)}$$

$$M_{Ed,s} = M_{Ed} + h \cdot V_{Ed} \qquad e = \frac{M_{Ed,s}}{N_{Ed,s}}$$

		$N_{Ed,s}$ [kN]	$M_{x,Ed,s}$ [kNm]	$M_{y,Ed,s}$ [kNm]	e_x [m]	e_y [m]	A' [m²]	σ_{Ed} [kN/m²]	
C I	LC1	1110.05	50.06	529.37	0.045	0.477	4.13	235.53	42.1%
	LC2	913.15	64.25	418.91	0.070	0.459	4.14	187.61	33.5%
	LC3	530.60	54.37	226.43	0.102	0.427	4.18	98.36	17.6%
	LC4	609.30	523.1	237.37	0.859	0.390	1.61	304.88	54.4%
	LC5	530.66	529.71	204.87	0.998	0.386	1.10	373.00	66.6%
	LC6	530.60	529.71	204.87	0.998	0.386	1.10	373.10	66.6%
C II	LC1	1110.05	37.37	446.26	0.034	0.402	4.55	213.95	38.2%
	LC2	913.15	43.81	317.25	0.048	0.347	4.77	162.72	29.1%
	LC3	530.60	35.03	142.97	0.066	0.269	5.09	80.89	14.4%
	LC4	609.30	9.57	295.2	0.016	0.484	4.19	117.00	20.9%
	LC5	530.60	13.47	338.41	0.025	0.638	3.38	121.87	21.8%
	LC6	530.60	13.47	338.41	0.025	0.638	3.38	121.87	21.8%

17.3 LIMITATION OF THE OPEN GAP

It has to be ensured that no rotation in the shallow foundation occurs due to horizontal loads and bending moments. A simplified and conservative approach is to proof that the eccentricities caused by those loads will not exceed a certain value. These limitations can be described with following formulas for rectangular footings [7]:

(1) $\frac{e_{x,k}}{b_x} + \frac{e_{y,k}}{b_y} \leq \frac{1}{6}$

with	$e_{x,k}, e_{y,k}$	Eccentricities caused by horizontal characteristic dead loads
	b_x, b_y	Width of the shallow foundation

(2) $\left(\frac{e_{x,k}}{b_x}\right)^2 + \left(\frac{e_{y,k}}{b_y}\right)^2 \leq \frac{1}{9}$

with	$e_{x,k}, e_{y,k}$	Eccentricities caused by horizontal characteristic dead and live loads
	b_x, b_y	Width of the shallow foundation

$N_{k,s} = N_k + G_k$ $\qquad M_{k,s} = M_k + h \cdot V_k$ $\qquad e = \frac{M_{Ed,s}}{N_{Ed,s}}$

	Characteristic forces			Eccentricities	
	$N_{k,s}$ [kN]	$M_{x,k,s}$ [kNm]	$M_{y,k,s}$ [kNm]	$e_{x,k}$ [m]	$e_{y,k}$ [m]
G_k	530.6	0.0	168.0	0	0.317
$G_k + S_k + w_k + A_k$	793.1	28.9	670.6	0.036	0.845

(1) $\frac{e_{x,k}}{b_x} + \frac{e_{y,k}}{b_y} = \frac{0}{2.6} + \frac{0.317}{2.6} = 0.122 \leq \frac{1}{6} = 0.17$ ✓ → 73.1 %

(2) $\left(\frac{e_{x,k}}{b_x}\right)^2 + \left(\frac{e_{y,k}}{b_y}\right)^2 = \left(\frac{0.036}{2.6}\right)^2 + \left(\frac{0.845}{2.6}\right)^2 = 0.106 \leq \frac{1}{9} = 0.111$ ✓ → 95.3 %

17.4 TILTING

Shallow foundations must be prevented from tilting. In order to proof that the foundations is safe from tilting again a conservative approach is to limit the eccentricities that occur from the factored horizontal loads. This can be done by meeting following conditions for rectangular footings [7]:

(1) $\frac{e_{x,d}}{b_x} \leq \frac{1}{2}$

(2) $\frac{e_{y,d}}{b_y} \leq \frac{1}{2}$ with $e_{x,k}, e_{y,k}$ Eccentricities caused by horizontal factored loads.

		$N_{Ed,s}$ [kN]	$M_{x,Ed,s}$ [kNm]	$M_{y,Ed,s}$ [kNm]	$e_{x,d}$ [m]	$e_{y,d}$ [m]	$e_{x,d}/b_x$ [-]		$e_{y,d}/b_y$ [-]	
C I	LC1	1110.05	50.06	529.37	0.045	0.477	0.017	3.5%	0.183	36.7%
	LC2	913.15	64.25	418.91	0.070	0.459	0.027	5.4%	0.176	35.3%
	LC3	530.60	54.37	226.43	0.102	0.427	0.039	7.9%	0.164	32.8%
	LC4	609.30	523.1	237.37	0.859	0.390	0.330	66.0%	0.150	30.0%
	LC5	530.66	529.71	204.87	0.998	0.386	0.384	76.8%	0.148	29.7%
	LC6	530.60	529.71	204.87	0.998	0.386	0.384	76.8%	0.149	29.7%
C II	LC1	1110.05	37.37	446.26	0.034	0.402	0.013	2.6%	0.155	30.9%
	LC2	913.15	43.81	317.25	0.048	0.347	0.018	3.7%	0.134	26.7%
	LC3	530.60	35.03	142.97	0.066	0.269	0.025	5.1%	0.104	20.7%
	LC4	609.30	9.57	295.2	0.016	0.484	0.006	1.2%	0.186	37.3%
	LC5	530.60	13.47	338.41	0.025	0.638	0.010	2.0%	0.245	49.1%
	LC6	530.60	13.47	338.41	0.025	0.638	0.010	2.0%	0.245	49.1%

17.5 CAPACITY UTILIZATION OF THE FOOTING

	Maximum utilization [%]
Bearing Capacity	66.6 %
Open Gap	95.3 %
Tilting	37.3 %

As can be seen in the table above the limitation of the open gap is the decisive criteria for choosing the size of the footing.

17.6 FLEXURE DESIGN

The flexure design of the footing is done according to [7].

	Verteilungsbeiwerte k_x für Moment [a] M_x im Schnitt *1-1*			
	c_y/b_y	0,1	0,2	0,3
Ort	Streifen 1	0,07	0,08	0,09
	Streifen 2	0,10	0,10	0,11
	Streifen 3	0,14	0,14	0,14
	Streifen 4	0,19	0,18	0,16
	Σ	0,50	0,50	0,50

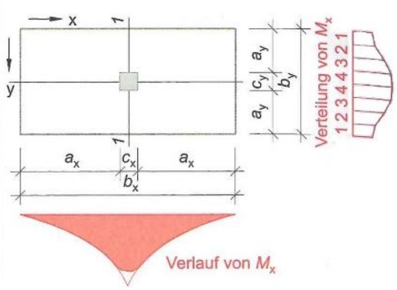


Figure 28 - Distribution of the Flexure Moment [7]

17.6.1 FLEXURE DESIGN

Maximum soil stress $\sigma_{Ed,\max} = 373.0kN/m^2$ (see Chapter 17.2)

Flexure moment $M_{Ed} = \sigma_{Ed} \cdot b_y \cdot \frac{b_x^2}{8} = 373.0 \cdot 2.6 \cdot \frac{2.6^2}{8} = 819.5kNm$

Distribution of the flexure moment

$$\frac{c}{b} = \frac{50}{260} = 0.192 \approx 0.2 \quad \rightarrow k_4 = 0.18$$

$$\Delta M_{Ed} = k_4 \cdot M_{Ed} = 0.18 \cdot 819.5 = 147.5kNm$$

$$d_{est} = h - c_{nom} - d_s/2 = 40 - 3.5 - 2.0 - 2.0/2 = 33cm$$

Required reinforcement $\mu_{Eds} = \frac{8 \cdot \Delta M_{Edx}}{b \cdot d^2 \cdot f_{cd}} = \frac{8 \cdot 14750}{260 \cdot 33^2 \cdot 1.98} = 0.21 \quad \rightarrow \omega = 0.2395$

$$A_{s4,req} = 0.2395 \cdot 260 \cdot 33 \cdot 19.8/8 \cdot 441 = 11.53cm^2$$

$$A_{s3,req} = \frac{k_{xi}}{k_{x4}} \cdot A_{sx4} = \frac{0.14}{0.18} \cdot 11.53cm^2 = 8.97cm^2$$

$$A_{s2,req} = \frac{k_{xi}}{k_{x4}} \cdot A_{sx4} = \frac{0.10}{0.18} \cdot 11.53cm^2 = 6.41cm^2$$

$$A_{s1,req} = \frac{k_{xi}}{k_{x4}} \cdot A_{sx4} = \frac{0.08}{0.18} \cdot 11.53cm^2 = 5.12cm^2$$

Chosen	**Ø 16 / 5 = 13.07 cm²/strip**	**strip 4**
	Ø 16 / 7 = 9.33 cm²/strip	**strip 3**
	Ø 16 / 9 = 7.25 cm²/strip	**strip 2**
	Ø 16 / 12.5 = 5.23 cm²/strip	**strip 1**

17.7 DESIGN OF THE PEDESTAL SOCKET

17.7.1 DEPTH OF THE SOCKET

Design forces

The decisive load for the depth of the socket is Case I Load Combination 6 with a minimum normal force and a maximum bending moment.[5]

N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$V_{x,Ed}$ [kN]	$M_{x,Ed}$ [kNm]	$V_{y,Ed}$ [kN]
- 479.9	- 204.3	1.9	- 416.4	- 377.7

Minimum depth

According to [8] the depth of the socket can be estimated using following formulas:

$$\frac{M_{Ed}}{N_{Ed} \cdot h} \leq 0.15 \quad \rightarrow \quad t \geq 1.2 \cdot h \qquad\qquad \frac{M_{Ed}}{N_{Ed} \cdot h} = 2.0 \quad \rightarrow \quad t \geq 2.0 \cdot h$$

But $t_{\min} = 1.5 \cdot h$

Here: $\frac{M_{Ed}}{N_{Ed} \cdot h} = \frac{416.4}{479.9 \cdot 0.5} = 1.74 \quad \rightarrow \quad t \geq 1.9 \cdot 50 = 95cm$

Total depth of the socket $h = 95 + 5 = \underline{\underline{100cm}}$ (+ 5cm for levelling shims)

17.7.2 DESIGN OF THE WALLS

$$\min d_k = \frac{W}{3} = \frac{10+50+10}{3} = 23.3cm > 20cm$$

Chosen $d_k = 25cm$

17.7.3 REINFORCEMENT OF THE SOCKET

Maximum Horizontal Forces

Upper horizontal force $H_{Ed}^{U} = \frac{6}{5} \cdot \frac{M_{Ed}}{t} + \frac{6}{5} \cdot H_{Ed} = \frac{6}{5} \cdot \frac{416.4}{0.95} + \frac{6}{5} \cdot 377.7 = 979.2kN$

[5] See Chapter 8.2.4

Lower horizontal force $H_{Ed}^{L} = \frac{6}{5} \cdot \frac{M_{Ed}}{t} + \frac{1}{5} \cdot H_{Ed} = \frac{6}{5} \cdot \frac{416.4}{0.95} + \frac{1}{5} \cdot 377.7 = 658.4kN$

Upper Reinforcement

$$A_{s,req}^{U} = \frac{H_{Ed}^{U}}{2} \cdot \frac{1}{f_{yd}} = \frac{979.2}{2} \cdot \frac{1}{43.5} = 11.3cm^2$$

Chosen **4 reinforcement loops Ø 14 = 12.32 cm²** **per wall**

Lap length *$Ø_s$ = 10 and moderate bond*

$$l_0 = \alpha_1 \cdot \alpha_6 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 1.0 \cdot 1.4 \cdot 58 \cdot (11.3/12.64) = 50.1cm$$

$$> l_{0,\min} = 20cm$$

Vertical Reinforcement

Vertical tension force $F_{sd}^{V} = H_{Ed}^{U} \cdot \frac{f_k}{g_k} = 979.2 \cdot \frac{84.2}{91.4} = 902.1kN$

$$f_k = h_k - \frac{t}{6} = 100 - \frac{95}{6} = 84.2cm \;;\; g_k = 0.85 \cdot (b_k - \frac{d_k}{2}) = 0.85 \cdot (120 - \frac{25}{2}) = 91.4cm$$

Vertical reinforcement $A_{s,req}^{V} = \frac{F_{sd}^{V}}{2} \cdot \frac{1}{f_{yd}} = \frac{902.1}{2} \cdot \frac{1}{43.5} = 10.4cm^2$

Chosen **4 vertical stirrups Ø 14 = 12.32 cm²** **per corner**

Lap length *$Ø_s$ = 10 and good bond*

$$l_0 = \alpha_1 \cdot \alpha_6 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 1.0 \cdot 1.4 \cdot 40 \cdot (10.4/11.06) = 53cm$$

$$> l_{0,\min} = 20cm$$

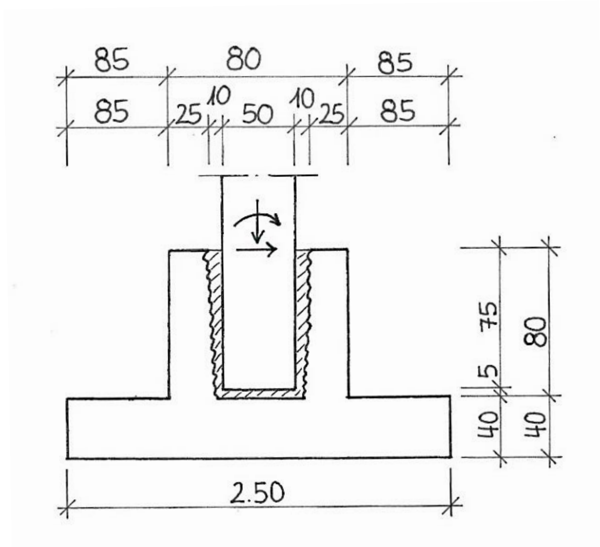
17.7.4 REINFORCEMENT DRAWING

See Appendix A.8, page 194.

18 POS. F.22 – POCKET FOUNDATION OF COLUMN C.05

18.1 SYSTEM

System:



Cross-section	b_x x b_y x h = 2.50 x 2.50 x 0.40 m
Building materials	**C 35/45 | B 500A**
Exposure class	XC2
Reinforcing steel	c_{nom} = 20 mm + 15 mm = 35 mm
Characteristic loads	Reaction forces from *POS. C.04* – see Chapter 8.2.4

18.2 SOIL-BEARING CAPACITY

The maximum characteristic bearing capacity of the soil is $\sigma_{R,k}$ = 400 kN/m². The size of the shallow foundation has to be chosen so the stresses caused by the load on the footing do not exceed the maximum bearing capacity, which means

$$\sigma_{Ed} = \frac{N_{Ed,s}}{A'} \leq \sigma_{Rd} = 1.4 \cdot \sigma_{Rk} = 1.4 \cdot 400 = 560 kN/m^2 \quad \textit{and} \quad A' = (b_x - 2e_x)(b_y - 2e_y)$$

$$N_{Ed,s} = N_{Ed} + 1.0/1.35 \cdot G_k \qquad G_k = 2.5 \cdot 2.5 \cdot 0.4 \cdot 25 kN/m^3 = 62.5 kN \quad \text{(self-weight of the footing)}$$

$$M_{Ed,s} = M_{Ed} + h \cdot V_{Ed} \qquad e = \frac{M_{Ed,s}}{N_{Ed,s}}$$

The decisive load combination for the soil-bearing capacity is LC1 (see Chapter 9.2.4).

	$N_{Ed,s}$ [kN]	$M_{Ed,s}$ [kNm]	e_x [m]	A' [m²	σ_{Ed} [kN/m²]	
LC1	3451.28	153.94	0.045	6.03	544.64	97.3%

18.3 LIMITATION OF THE OPEN GAP

It has to be ensured that no rotation in the shallow foundation occurs due to horizontal loads and bending moments. A simplified and conservative approach is to proof that the eccentricities caused by those loads will not exceed a certain value. These limitations can be described with following formulas for rectangular footings [7]:

(1) $\frac{e}{b} \leq \frac{1}{6}$ for dead loads (2) $\frac{e}{b} \leq \frac{1}{3}$ for dead and live loads

$N_{k,s} = N_k + G_k$ $M_{k,s} = M_k + h \cdot V_k$ $e = \frac{M_{Ed,s}}{N_{Ed,s}}$

	Characteristic forces		Eccentricities
	$N_{k,s}$ [kN]	$M_{k,s}$ [kNm]	e_k [m]
G_k	1389.8	0.0	0
G_k + S_k + A_k	2439.8	543.8	0.223

(1) $\frac{e_k}{b} = 0 \leq \frac{1}{6} = 0.17$ ✓ → 0.0 %

(2) $\frac{e_k}{b} = \frac{0.223}{2.50} = 0.008 \leq \frac{1}{3} = 0.33$ ✓ → 7.2 %

18.4 TILTING

Shallow foundations must be prevented from tilting. In order to proof that the foundations is safe from tilting again a conservative approach is to limit the eccentricities that occur from the factored horizontal loads. This can be done by meeting following conditions for rectangular footings [7]:

(3) $\frac{e_d}{b} \leq \frac{1}{2}$

	$N_{Ed,s}$ [kN]	$M_{Ed,s}$ [kNm]	e_d [m]	e_d/b [-]	
LC1	3451.28	153.94	0.045	0.018	3.6%
LC2	1599.90	603.16	0.377	0.151	30.2%
LC3	1389.80	592.34	0.426	0.170	34.1%

18.5 CAPACITY UTILIZATION OF THE FOOTING

	Maximum utilization [%]
Bearing Capacity	97.3 %
Open Gap	7.2 %
Tilting	34.1 %

As can be seen in the table above the bearing capacity is the decisive criteria for choosing the size of the footing.

18.6 FLEXURE DESIGN

The flexure design of the footing is done according to [7].

		Verteilungsbeiwerte k_x für Moment [a] M_x im Schnitt I-I		
	c_y/b_y	0,1	0,2	0,3
Ort	Streifen 1	0,07	0,08	0,09
	Streifen 2	0,10	0,10	0,11
	Streifen 3	0,14	0,14	0,14
	Streifen 4	0,19	0,18	0,16
	Σ	0,50	0,50	0,50

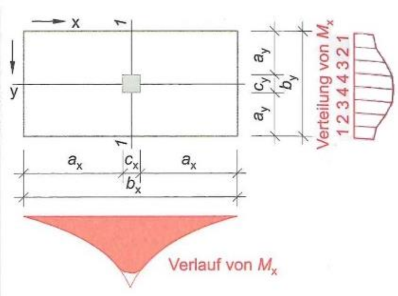


Figure 29 - Distribution of the Flexure Moment [7]

18.6.1 FLEXURE DESIGN

Maximum soil stress $\sigma_{Ed,\max} = 544.6 kN/m^2$ (see Chapter 17.2)

Flexure moment $M_{Ed} = \sigma_{Ed} \cdot b_y \cdot \frac{b_x^2}{8} = 554.6 \cdot 2.5 \cdot \frac{2.5^2}{8} = 1083 kNm$

Distribution of the flexure moment

$$\frac{c}{b} = \frac{50}{250} = 0.2 \quad \rightarrow k_4 = 0.18$$

$$\Delta M_{Ed} = k_4 \cdot M_{Ed} = 0.18 \cdot 1083 = 194.9 kNm$$

$$d_{est} = h - c_{nom} - d_s/2 = 40 - 3.5 - 2.0 - 2.0/2 = 33 cm$$

Required reinforcement

$$\mu_{Eds} = \frac{8 \cdot \Delta M_{Edx}}{b \cdot d^2 \cdot f_{cd}} = \frac{8 \cdot 19490}{250 \cdot 33^2 \cdot 1.98} = 0.29 \quad \rightarrow \omega = 0.3546$$

$$A_{s4,req} = 0.3546 \cdot 250 \cdot 33 \cdot 19.8 / 8 \cdot 437 = 16.6 cm^2$$

$$A_{s3,req} = \frac{k_{xi}}{k_{x4}} \cdot A_{sx4} = \frac{0.14}{0.18} \cdot 16.6 cm^2 = 12.9 cm^2$$

$$A_{s2,req} = \frac{k_{xi}}{k_{x4}} \cdot A_{sx4} = \frac{0.10}{0.18} \cdot 16.6 cm^2 = 9.2 cm^2$$

$$A_{s1,req} = \frac{k_{xi}}{k_{x4}} \cdot A_{sx4} = \frac{0.08}{0.18} \cdot 16.6cm^2 = 7.4cm^2$$

Chosen	**Ø 20 / 5 = 19.6 cm²/strip**	**strip 4**
	Ø 20 / 7.5 = 13.1 cm²/strip	**strip 3**
	Ø 20 / 10 = 9.8 cm²/strip	**strip 2**
	Ø 20 / 12.5 = 7.9 cm²/strip	**strip 1**

18.7 DESIGN OF THE PEDESTAL SOCKET

18.7.1 DEPTH OF THE SOCKET

Design forces

The decisive load for the depth of the socket is LC 5 with a minimum normal force and a maximum bending moment (see Chapter 9.2.4).

N_{Ed} [kN]	$M_{y,Ed}$ [kNm]	$V_{x,Ed}$ [kN]
-1327.3	-440.5	379.6

Minimum depth

According to [8] the depth of the socket can be estimated using following formulas:

$$\frac{M_{Ed}}{N_{Ed} \cdot h} \le 0.15 \quad \rightarrow \; t \ge 1.2 \cdot h \qquad\qquad \frac{M_{Ed}}{N_{Ed} \cdot h} = 2.0 \quad \rightarrow \; t \ge 2.0 \cdot h$$

But $t_{\min} = 1.5 \cdot h$

Here: $\frac{M_{Ed}}{N_{Ed} \cdot h} = \frac{440.5}{1327.3 \cdot 0.5} = 0.66 \quad \rightarrow \; t \ge 1.5 \cdot 50 = 75cm$

Total depth of the socket $h = 75 + 5 = \underline{\underline{80cm}}$ (+ 5cm for levelling shims)

18.7.2 DESIGN OF THE WALLS

$$\min d_k = \frac{W}{3} = \frac{10+50+10}{3} = 23.3cm > 20cm$$

Chosen $d_k = 25cm$

18.7.3 REINFORCEMENT OF THE SOCKET

Maximum Horizontal Forces

Upper horizontal force $H_{Ed}^U = \frac{6}{5} \cdot \frac{M_{Ed}}{t} + \frac{6}{5} \cdot H_{Ed} = \frac{6}{5} \cdot \frac{440.5}{0.75} + \frac{6}{5} \cdot 379.6 = 1160kN$

Lower horizontal force $H_{Ed}^L = \frac{6}{5} \cdot \frac{M_{Ed}}{t} + \frac{1}{5} \cdot H_{Ed} = \frac{6}{5} \cdot \frac{440.5}{0.75} + \frac{1}{5} \cdot 379.6 = 781kN$

Upper Reinforcement

$$A_{s,req}^{U} = \frac{H_{Ed}^{U}}{2} \cdot \frac{1}{f_{yd}} = \frac{1160}{2} \cdot \frac{1}{43.5} = 13.3cm^2$$

Chosen **9 reinforcement loops Ø 10 = 14.14 cm²** **per wall**

Lap length *$Ø_s$ = 10 and moderate bond*

$$l_0 = \alpha_1 \cdot \alpha_6 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 1.0 \cdot 1.4 \cdot 58 \cdot (11.3/12.64) = 50.1cm$$

$$> l_{0,\min} = 20cm$$

Vertical reinforcement

Vertical tension force

$$F_{sd}^{V} = H_{Ed}^{U} \cdot \frac{f_k}{g_k} = 1160 \cdot \frac{67.5}{91.4} = 857kN$$

$$f_k = h_k - \frac{t}{6} = 80 - \frac{75}{6} = 67.5cm \;;\; g_k = 0.85 \cdot (b_k - \frac{d_k}{2}) = 0.85 \cdot (120 - \frac{25}{2}) = 91.4cm$$

Vertical reinforcement

$$A_{s,req}^{V} = \frac{F_{sd}^{V}}{2} \cdot \frac{1}{f_{yd}} = \frac{857}{2} \cdot \frac{1}{43.5} = 9.9cm^2$$

Chosen **7 vertical stirrups Ø 10 m = 11.06 cm²** **per corner**

Lap length *$Ø_s$ = 10 and good bond*

$$l_0 = \alpha_1 \cdot \alpha_6 \cdot l_{b,rqd} \cdot \left(A_{s,req} / A_{s,prov}\right) = 1.0 \cdot 1.4 \cdot 40 \cdot (10.4/11.06) = 53cm$$

$$> l_{0,\min} = 20cm$$

19 POS. F.23 – STRIP FOUNDATION

19.1 SYSTEM

Cross-section	b / h = 50 / 80 cm
Building materials	**C 30/37 | B 500A**
Exposition class	XC2
Reinforcing steel	c_{nom} = 20 mm + 15 mm = 35 mm

Characteristic loads

Uniformly Distributed Loads		**g_k [kN/m]**	**q_k [kN/m]**
Self-weight	g_{k1} = 0.3 x 0.8 x 25 kN/m³ =	6.00	-
Reaction Force from POS. W.*14*			
	g_{k2} = 4.30 x 0.2 x 25kN/m³ + 6 x 88.0kN / 7.50m =	91.9	-
	q_k = 6 x 31.0kN / 7.50m =	-	24.8
		97.9	24.8

19.2 SOIL-BEARING CAPACITY

Factored load

$$N_{Ed} = 1.35 \cdot 97.9 + 1.50 \cdot 24.8 = 169.4 kN$$

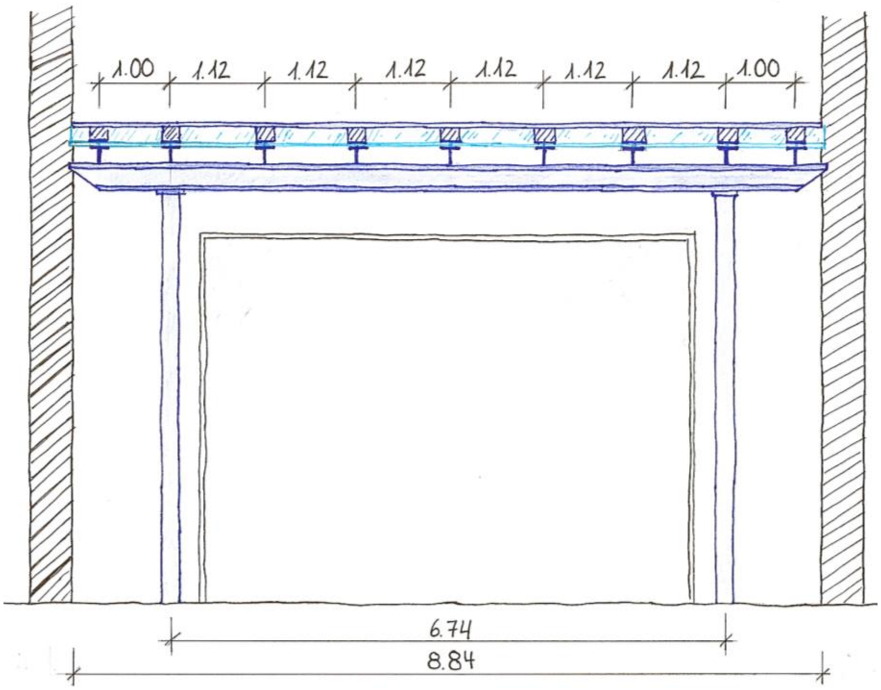
Soil-bearing capacity

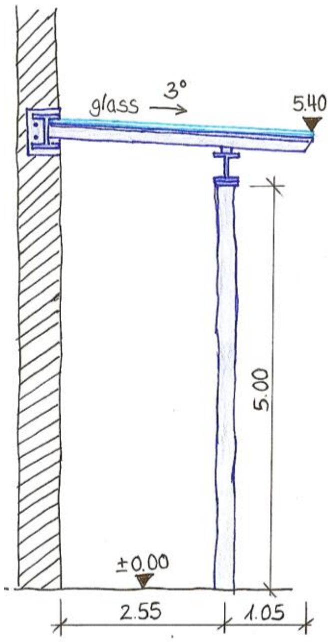
$$\sigma_{Ed} = \frac{N_{Ed}}{A} = \frac{169.4}{0.5 \cdot 1.0} = 338.8 kN/m^2 \leq \sigma_{Rd} = 1.4 \cdot \sigma_{Rk} = 1.4 \cdot 400 = 560 kN/m^2$$

20 POS. V.31 – STEEL CANOPY

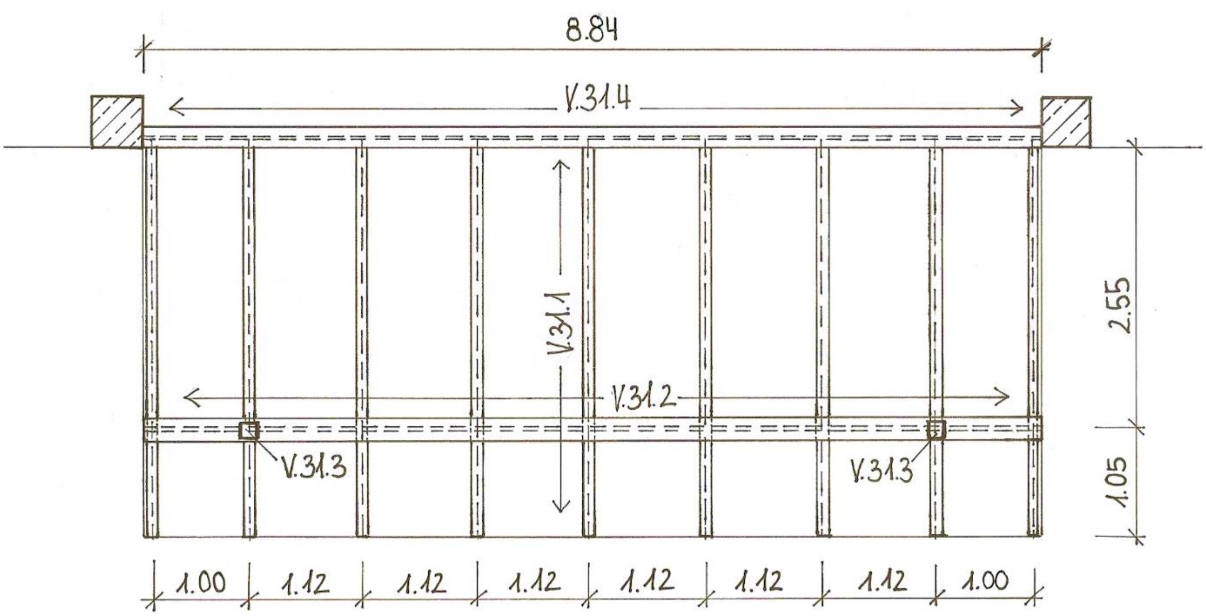
20.1 OVERVIEW DRAWING

20.1.1 SKETCH





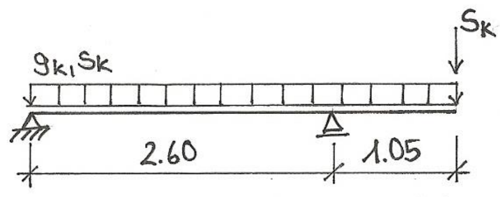
20.1.2 POSITION PLAN



20.2 DESIGN CALCULATIONS

20.2.1 POSITION V.31.1

System



Cross-section **2 x L100x50x6 (S 235)**

a [mm]	b [mm]	t [mm]	r [mm]	A [cm²]	I_y [cm³]	$W_{el,y}$ [cm³]	g_k [kN/m]
100	50	6	8	8.71	89.9	13.8	0.0684

Characteristic loads

Uniformly Distributed Loads		g_k [kN/m]	q_k [kN/m]
Self-weight	g_{k1} = 2 x 0.068 kN/m =	0.14	-
Self-weight glass roof	g_{k2} = 1.12 x 0.02 x 25 kN/m³ =	0.70	-
Snow load	s_k = 1.12 x 0.8 x 2.96 kN/m² =	-	2.65
		0.84	2.64

Concentrated Loads		**Gk [kN]**	**Qk [kN]**
Snow overhang	S_k = 0.4 x 0.8 x 2.96 x 1.12/3.0 =	-	0.35

Design loads

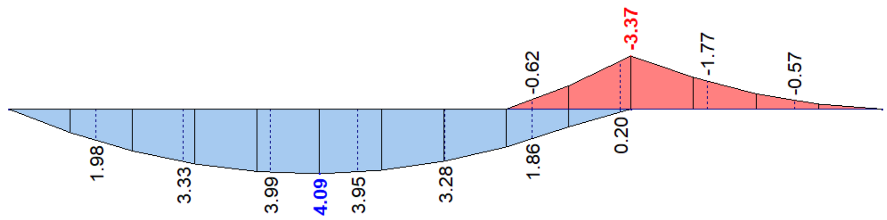
LC1:
$$q_{Ed} = 1.35 \cdot g_k + 1.50 \cdot s_k = 1.35 \cdot 0.84 + 1.50 \cdot 2.64 = 5.09 kN/m$$
$$Q_{Ed} = 1.50 \cdot S_k = 1.50 \cdot 0.35 = 0.53 kN$$

LC2:
$$q_{Ed} = 1.00 \cdot g_k + 0 \cdot s_k = 1.00 \cdot 0.84 = 0.84 kN/m$$
$$Q_{Ed} = 0 \cdot S_k = 0 kN$$

Internal Forces

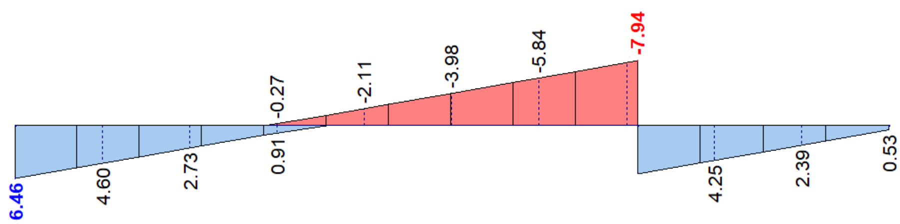
Flexure moment

	M_{gk} [kNm]	M_{sk} [kNm]	min M_{Ed} [kNm]	max M_{Ed} [kNm]
Span	0.50	1.42	2.80	4.09
Cantilever	-0.46	-1.83	-0.46	-3.37



Shear force

	V_{gk} [kN]	V_{sk} [kN]	min V_{Ed} [kN]	maxVM_{Ed} [kN]
Support A	0.91	2.74	5.35	6.46
Support B_L	-1.27	-4.15	-6.82	-7.94
Support B_R	0.88	3.13	0.88	5.89



Ultimate Limit State Design

Plastic internal forces $N_{pl} = 205\ kN$ $M_{pl,y,Rd} = 6.00\ kNm$ $V_{pl,z,Rd} = 66.0\ kN$

Maximum flexure moment $\frac{M_{Ed}}{M_{pl,Rd}} = \frac{4.09}{6.00} = 0.68 < 1.00$ ✓

Maximum shear force $\frac{V_{Ed}}{V_{pl,z,Rd}} = \frac{7.94kN}{66.0} = 0.12 < 1.00 < 0.50$ ✓ → No M-N-interaction necessary!

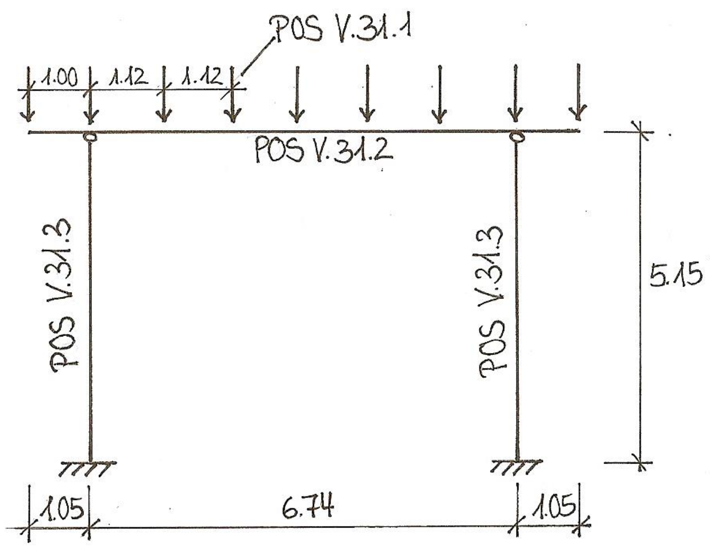
Maximum Deflection

Span $\max f = 5.0mm < L/300 = 2600/300 = 8.7mm$ → $\eta = 0.57$

Cantilever $\max f = 5.7mm < L/150 = 1050/150 = 7.0mm$ → $\eta = 0.81$

20.2.2 POSITION V.31.2

System



Cross-section **IPE 300 (S235)**

h [mm]	b [mm]	t_w [mm]	t_f [mm]	A [cm²]	I_y [cm³]	$W_{el,y}$ [cm³]	g_k [kN/m]
300	150	7.1	10.7	53.8	8360	557	0.422

Characteristic loads

Uniformly Distributed Loads		g_k [kN/m]	q_k [kN/m]
Self-weight	(generated by the software)	-	-

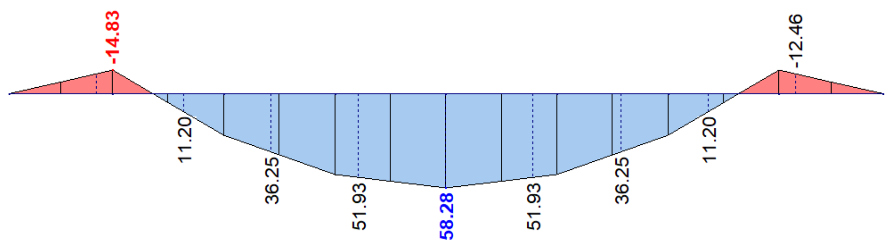
Concentrated Loads	Gk [kN]	Qk [kN]
Reaction forces from POS. V.31.1	2.15	7.28

Design loads $Q_{Ed} = 1.35 \cdot G_k + 1.50 \cdot Q_k = 1.35 \cdot 2.15 + 1.50 \cdot 7.28 = 14.42 kN$

Internal Forces

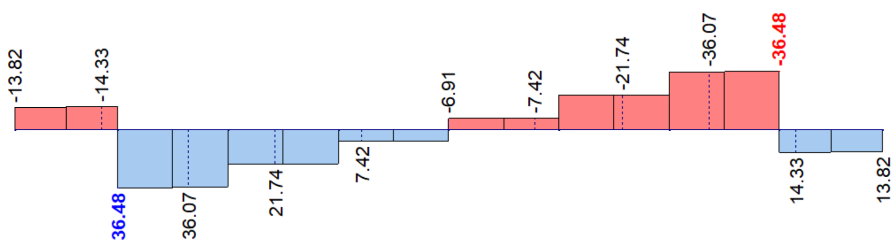
Flexure moment

	M_{gk} [kNm]	M_{sk} [kNm]	M_{Ed} [kNm]
Span	10.78	29.16	58.28
Cantilever	-2.49	-7.64	-14.83



Shear force

	V_{gk} [kN]	V_{sk} [kN]	V_{Ed} [kN]
Support A_L	-2.59	-7.28	-14.33
Support A_R	6.80	18.20	36.48



Ultimate Limit State Design

Plastic internal forces $N_{pl} = 2656\ kN$ $M_{pl,y,Rd} = 325\ kNm$ $V_{pl,z,Rd} = 512\ kN$

Maximum flexure moment $\frac{M_{Ed}}{M_{pl,Rd}} = \frac{58.3kNm}{325kNm} = 0.18 < 1.00$ ✓

Maximum shear force $\frac{V_{Ed}}{V_{pl,z,Rd}} = \frac{36.5kN}{512kN} = 0.07 < 1.00 < 0.50$ ✓ → No M-N-interaction necessary!

Maximum Deflection

$\max f = 15.6mm < L/300 = 6740/300 = 22.5mm$ → $\eta = 0.69$

20.2.3 POSITION V.31.3

System

See Chapter 20.2.2

Cross-section **QRO 100x5 (S235)**

b [mm]	t [mm]	A [cm²]	I_y [cm³]	i [cm]	g_k [kN/m]
100	5	18.4	271	3.84	0.144

Internal Forces

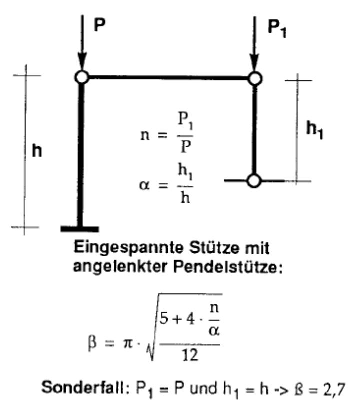
Normal force

N_{gk} [kN]	N_{sk} [kN]	N_{Ed} [kN]
-12.46	-32.76	-69.95

Ultimate Limit State Design

Buckling length *Simplified assumption:* $\beta \approx 2.7$ (see Figure 30)

$$L_{cr} = \beta \cdot L_0 = 2.7 \cdot 5.15 = 13.9m$$



Reduction coefficient

$$\bar{\lambda} = \frac{L_{cr}}{93.9 \cdot i} = \frac{13.9m}{93.9 \cdot 3.84} = 0.039$$

Imperfection coefficient $\alpha = 0.49$

$$\phi = 0.5 \cdot \left[1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$$

$$= 0.5 \cdot \left[1 + 0.49 \cdot (0.039 - 0.2) + 0.039^2\right]$$

$$= 0.46$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}}$$

$$= \frac{1}{0.46 + \sqrt{0.46^2 - 0.039^2}} = 1.09 \leq \underline{\underline{1.0}}$$

Maximum normal force

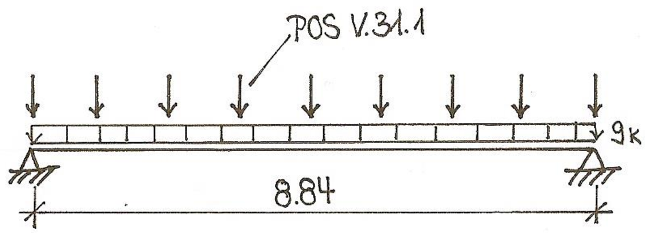
$$N_{cr,Rd} = \chi \cdot N_{pl,Rd} = 1.0 \cdot 18.4 \cdot 23.5 = 432.4kN$$

Utilisation

$$\eta = \frac{N_{Ed}}{N_{cr,Rd}} = \frac{69.95}{432.4} = 0.16 < 1.00 \checkmark$$

20.2.4 POSITION V.31.4

System



Cross-section **IPE270 (S235)**

h [mm]	b [mm]	t_w [mm]	t_f [mm]	A [cm²]	I_y [cm³]	$W_{el,y}$ [cm³]	g_k [kN/m]
270	135	6.6	10.2	45.9	5790	429	0.361

Characteristic loads

Uniformly Distributed Loads		g_k [kN/m]	q_k [kN/m]
Self-weight	(generated by the software)	-	-
Concentrated Loads		**Gk [kN]**	**Qk [kN]**
Reaction forces from POS. V.31.1		0.91	2.74

Design loads $Q_{Ed} = 1.35 \cdot G_k + 1.50 \cdot S_k = 1.35 \cdot 0.91 + 1.50 \cdot 2.74 = 5.34 kN$

Internal Forces

Flexure moment

M_{gk} [kNm]	M_{sk} [kNm]	M_{Ed} [kNm]
11.46	23.92	51.36

Shear force

V_{gk} [kN]	V_{sk} [kN]	V_{Ed} [kN]
4.78	9.59	20.32

Ultimate Limit State Design

Plastic internal forces N_{pl} = 1079 kN $M_{pl,y,Rd}$ = 113.7 kNm $V_{pl,z,Rd}$ = 299.8 kN

Maximum flexure moment $\frac{M_{Ed}}{M_{pl,Rd}} = \frac{51.36kNm}{113.7kNm} = 0.45 < 1.00$ ✓

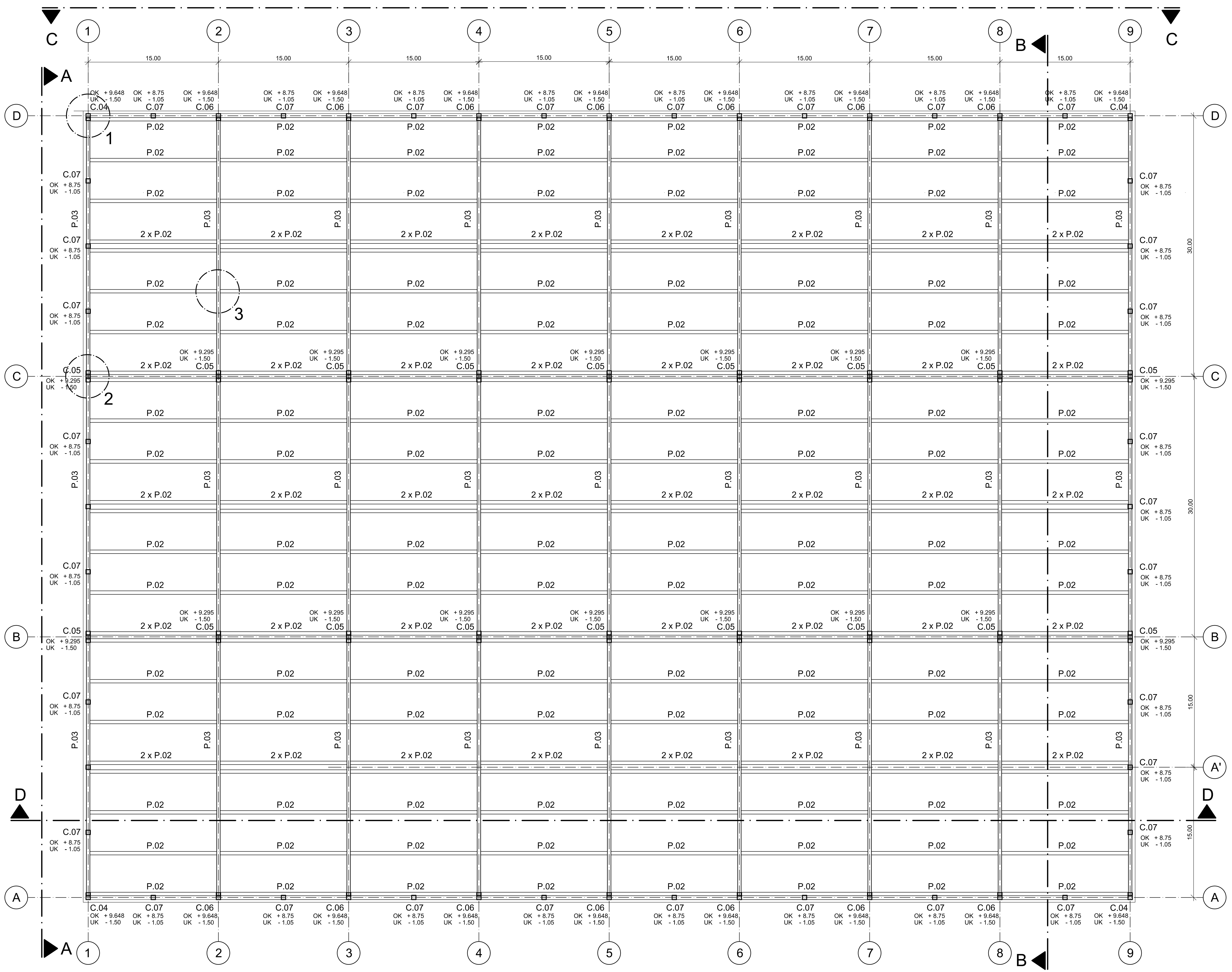
Maximum shear force $\frac{V_{Ed}}{V_{pl,z,Rd}} = \frac{20.32kN}{229.8kN} = 0.07 < 1.00 < 0.50$ ✓→ No M-N-interaction necessary!

Maximum Deflection

$$\max f = 23.7mm < L/300 = 8840/300 = 29.5mm \rightarrow \eta = 0.80$$

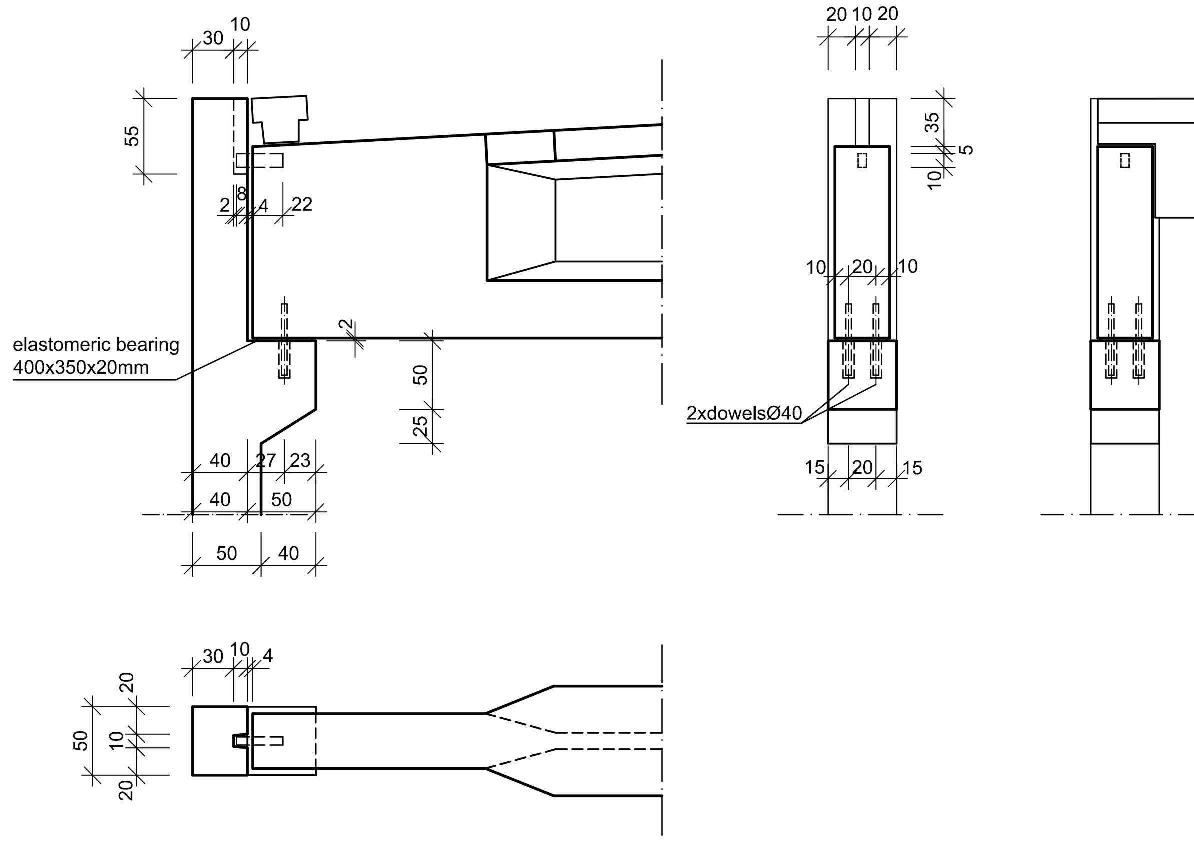
APPENDICES

APPENDIX A DESIGN DRAWINGS



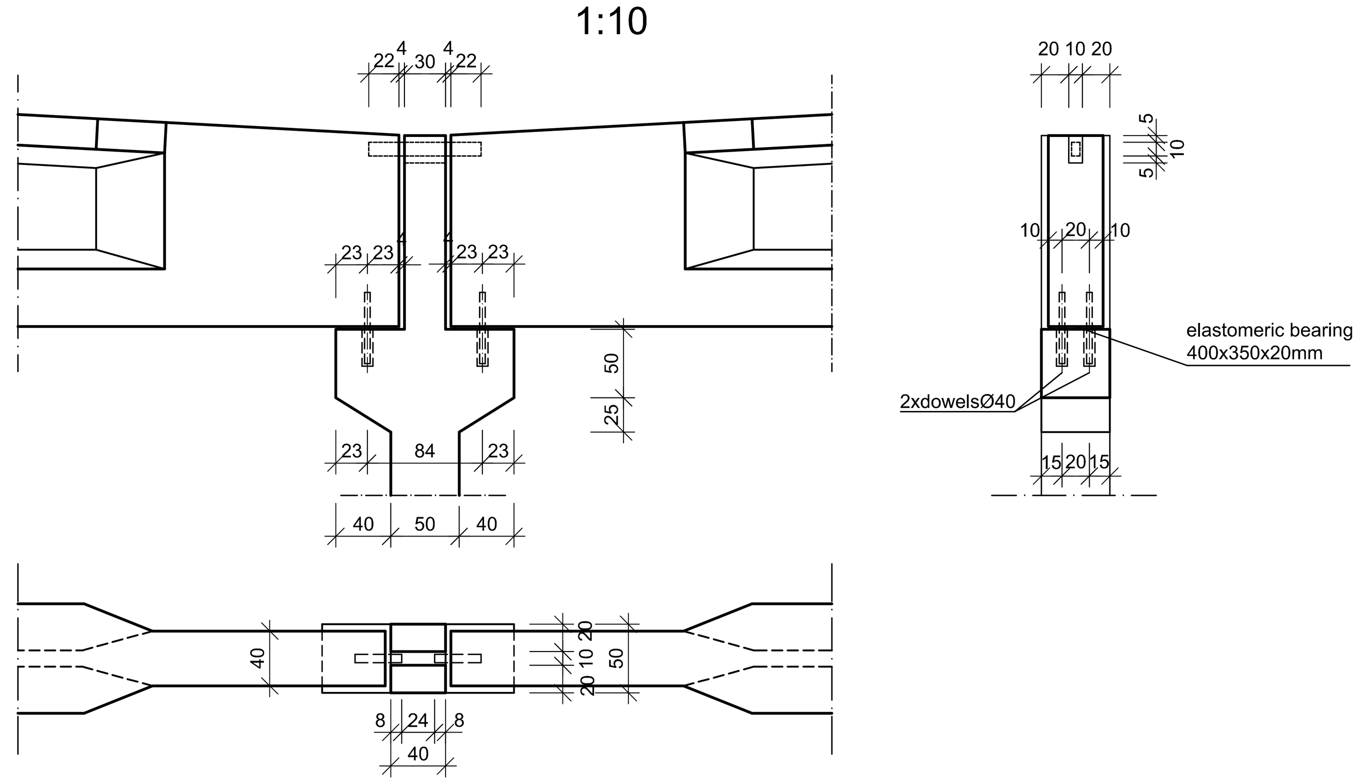
Detail '1'

1:10



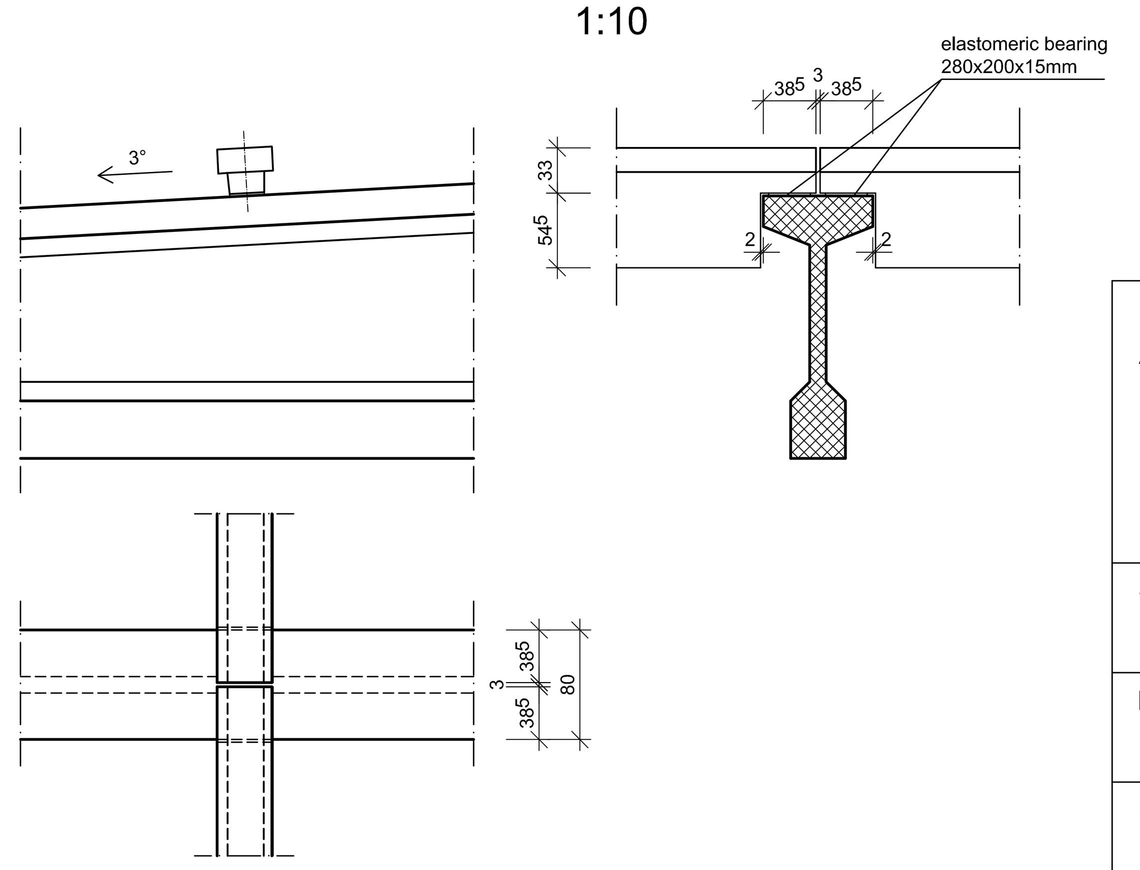
Detail '2'

1:10



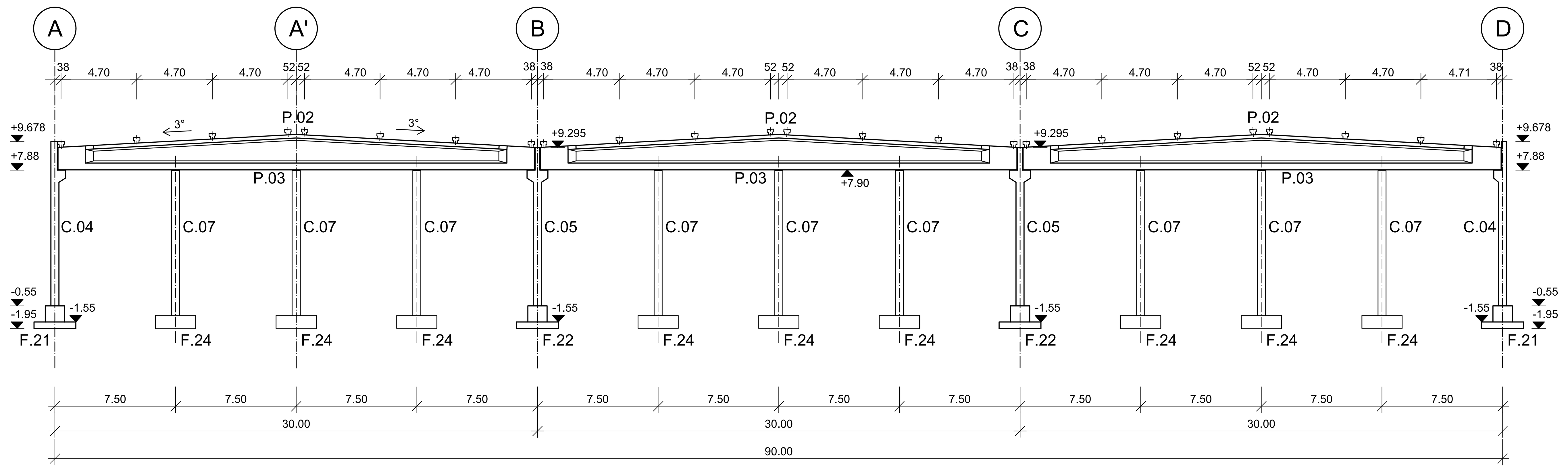
Detail '3'

1:10

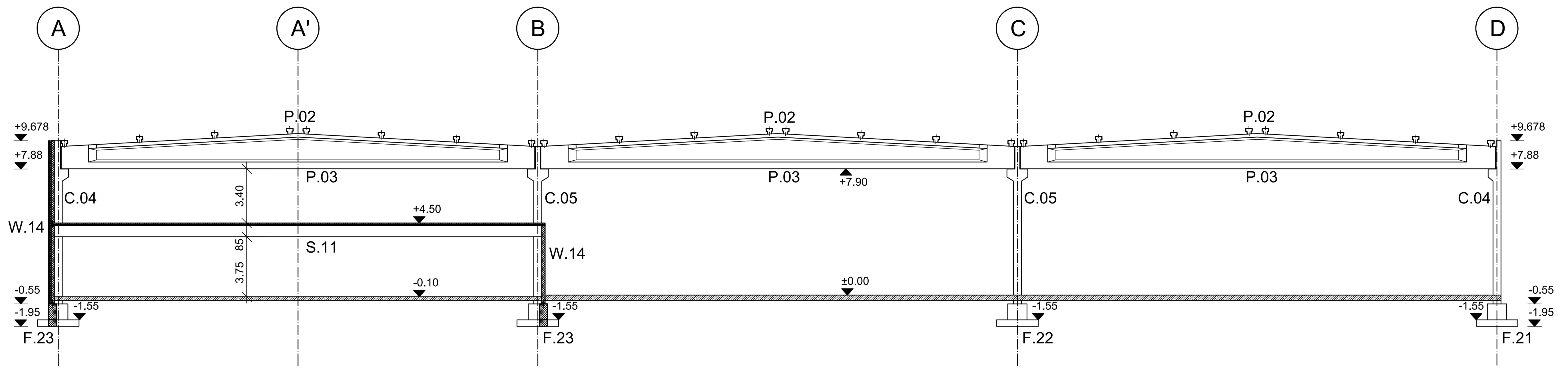


Karlsruhe University of Applied Sciences Ryerson Universtiy Faculty of Civil Engineering	Master Project 2018 A Structural Analysis of a Factory Hall as a Precast Concrete Structure by Olesja Befus
Supervisor:	Prof. Dr.-Ing. Ch. Enderle Prof. Dr. R. Kianoush
Project:	Construction of a Factory Hall in Betzweiler-Wälde, Germany
Plan type:	Overview Drawing
Drawn by: Olesja Befus Date: 31.07.2018	Scale: 1:200 - m — Plan-Nr.: A.1

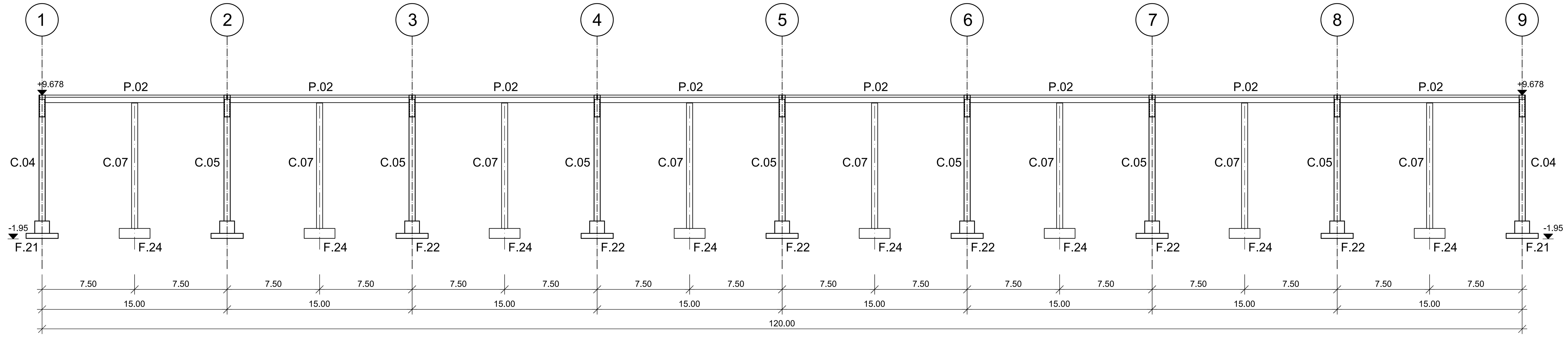
View A-A



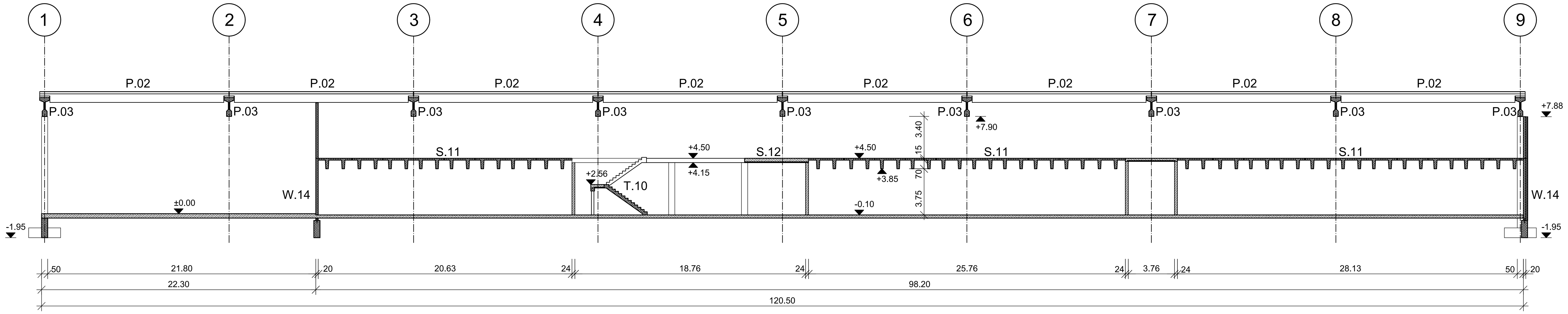
Section B-B



View C-C

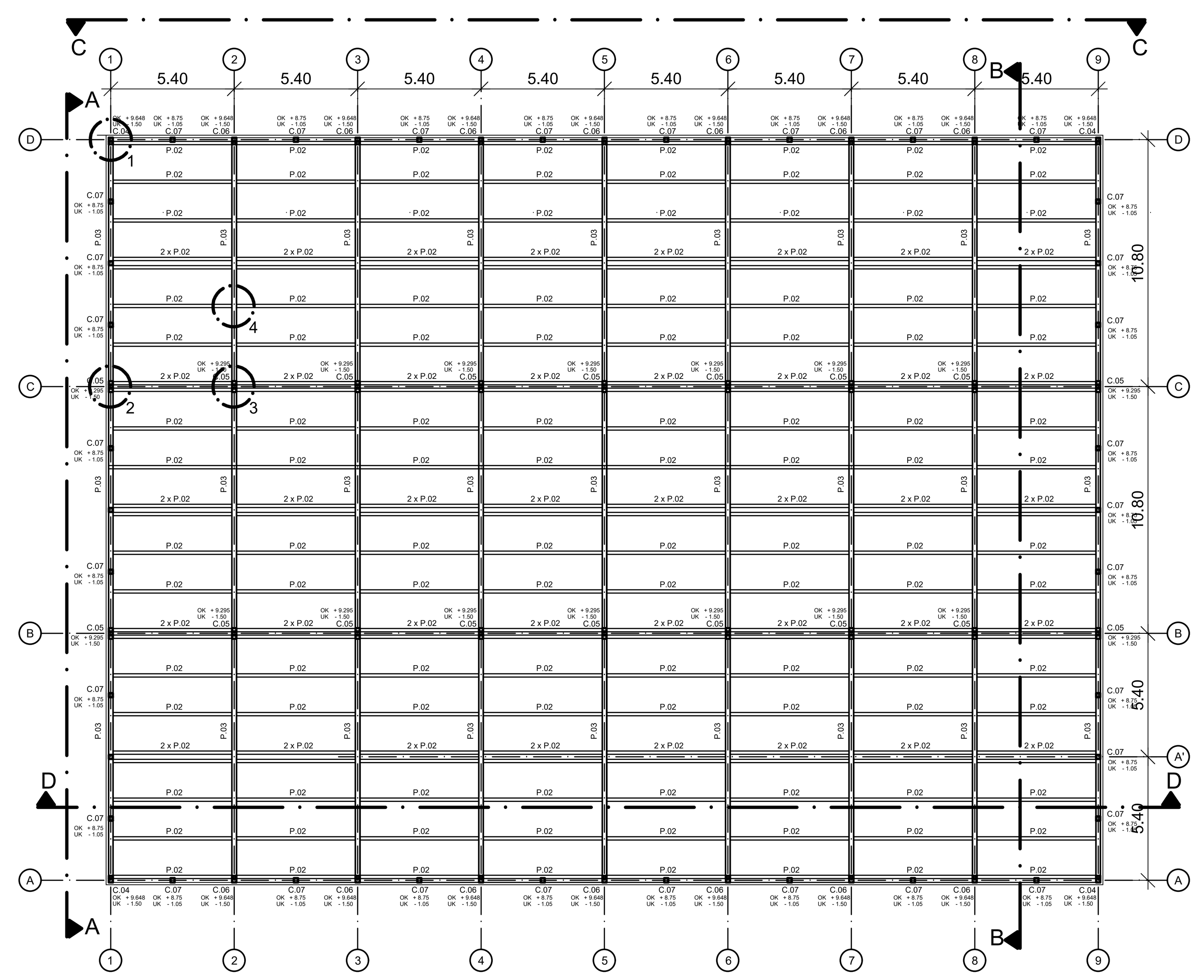


Section D-D



Overview Drawing see Plan Nr. A.1

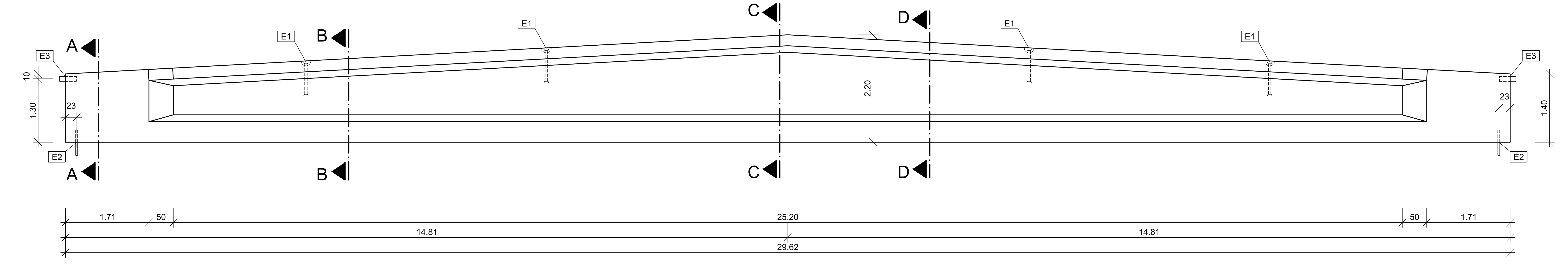
N.T.S



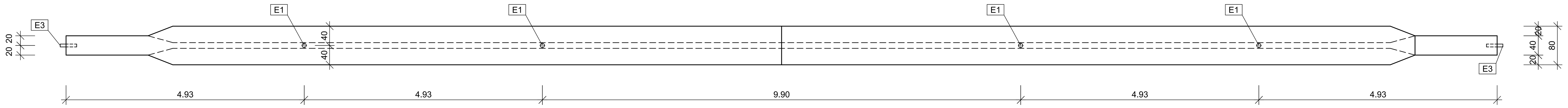
Karlsruhe University of Applied Sciences	Master Project 2018
Ryerson Universtiy	A Structural Analysis of a Factory Hall as a Precast Concrete Structure
Faculty of Civil Engineering	by Olesja Befus
Supervisor:	Prof. Dr.-Ing. Ch. Enderle Prof. Dr. R. Kianoush
Project:	Construction of a Factory Hall in Betzweiler-Wälde, Germany
Plan type:	Sections and Views

Drawn by:	Scale:	Plan-Nr.:
Olesja Befus	1:200 - m	A.2
Date: 31.07.2018		

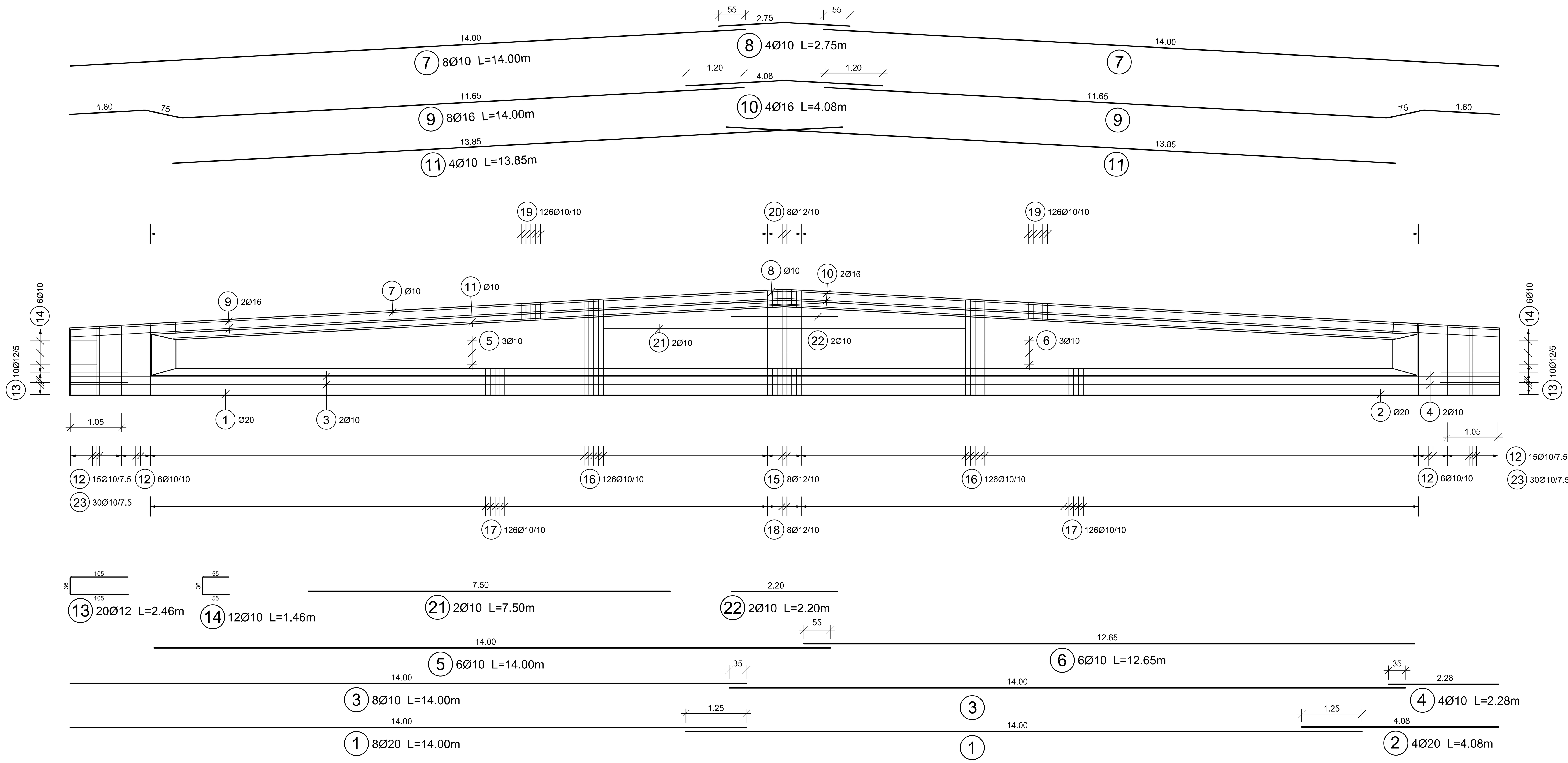
Prestressed Concrete Truss



Top View

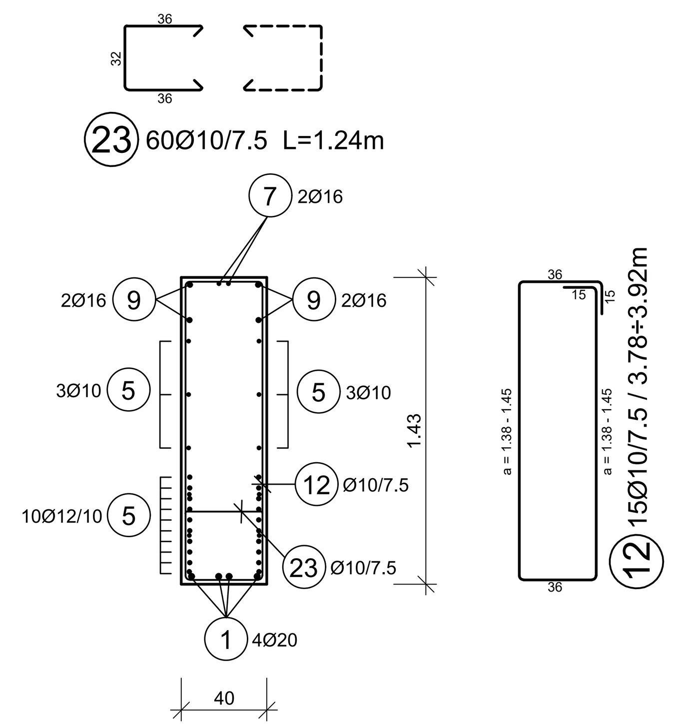


Reinforcement



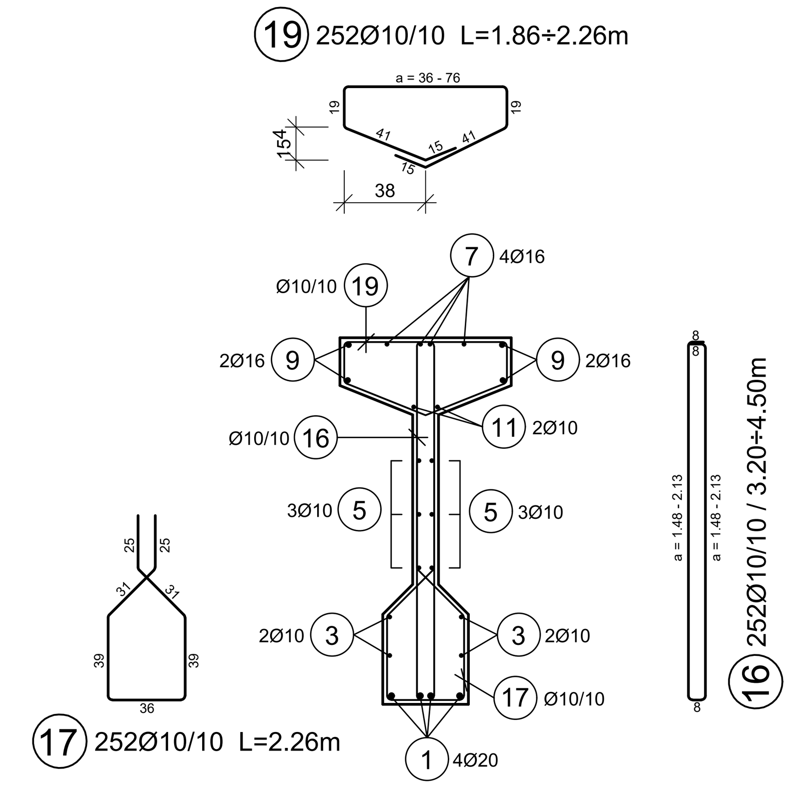
Section A-A

1:25



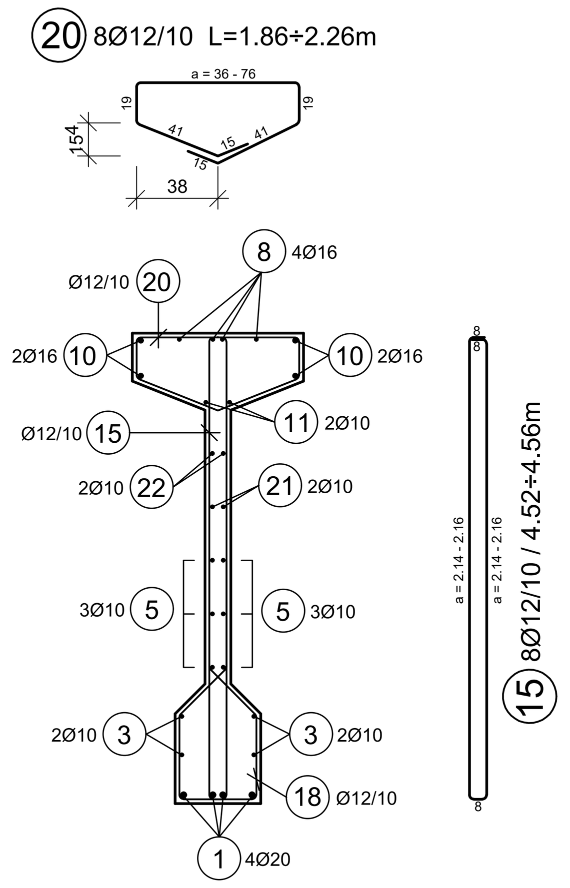
Section B-B

1:25



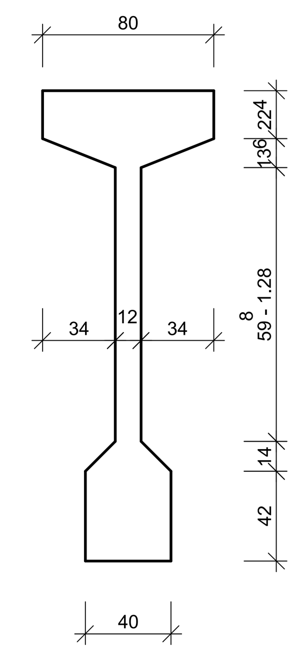
Section C-C

1:25

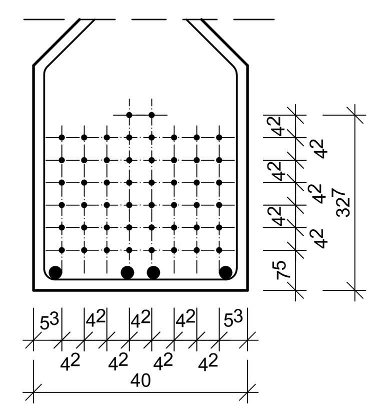


Section D-D

1:25



Prestressing Information



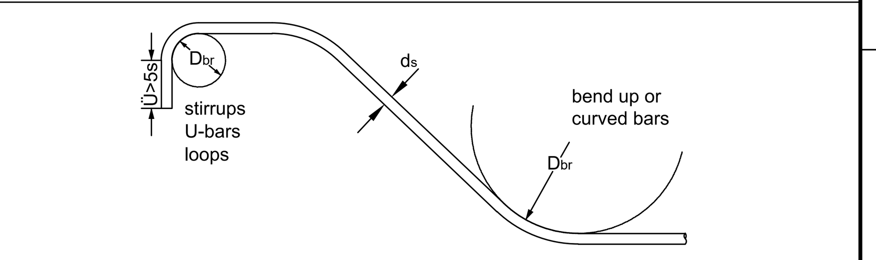
Prestressing with immediate bond
7 Layers, 50 strands 0.5" (Ø12.5mm)
St 1570/1770 (Z-12.3-107)
$\sigma_{p,max}$=1000N/mm²
P_{max}=93kN/strand

1. Layer 8 strands
2. Layer 8 strands
3. Layer 8 strands
4. Layer 8 strands
5. Layer 8 strands
6. Layer 8 strands
7. Layer 2 strands

Installation Parts

E1	HALFEN DEHA Double-headed lifting anchor 6000-32,0-0700D WB	4 Pcs
E2	Shear force dowel Ø40	4 Pcs
E3	MSH 100x50x5	2 Pcs

Bending of the Reinforcment Bars (DIN EN 1992-1-1)



Diameter of the bar d_s [mm]			Concrete cover		
	< 20	D_{br} = 4 d_s		> 100 mm and > 7 d_s	D_{br} = 10 d_s
	≥ 20	D_{br} = 7 d_s		> 50 mm and > 3 d_s	D_{br} = 15 d_s
				≤ 50 mm and ≤ 3 d_s	D_{br} = 20 d_s

Concrete strength class: C50/60	Exposure class: XC1, WO
Concrete cover:	Stirrups: c_{nom} = 25 mm
Chamfer strip:	10 mm / 10 mm
Reinforcing and Prestressing Steel:	
Reinforcement bars:	B500A
Reinforcement mesh:	B500A
Prestressing steel:	St 1570/1770 (Z-12.3-107)

Surface finish		rough	rubbed	sanded	troweld	special formwork
	Filling side					
	Formwork side	exposed concrete smooth	no requirements	texture	exposed aggregate concrete	special formwork

Karlsruhe University of Applied Sciences

Ryerson Universtiy

Faculty of Civil Engineering

Master Project 2018

A Structural Analysis of a Factory Hall as a Precast Concrete Structure

by Olesja Befus

Supervisor:	Prof. Dr.-Ing. Ch. Enderle Prof. Dr. R. Kianoush
Project:	Construction of a Factory Hall in Betzweiler-Wälde, Germany
Plan type:	Formwork and Reinforcement Plan of the Prestressed Concrete Truss P.03

Drawn by:	Scale:	Plan-Nr.:
Olesja Befus	1:50 - m	A.3
Date: 31.07.2018		

Installation Parts		
E1	HALFEN DEHA Spherical head anchor 6000-7,5-0200	2 Pcs
E2	Shear force dowel Ø40	4 Pcs

Bending of the Reinforcment Bars (DIN EN 1992-1-1)

Diameter of the bar d_s [mm]	< 20	$D_{br} = 4\ d_s$	Concrete cover	> 100 mm and > 7 d_s	$D_{br} = 10\ d_s$
				> 50 mm and > 3 d_s	$D_{br} = 15\ d_s$
	≥ 20	$D_{br} = 7\ d_s$		≤ 50 mm and ≤ 3 d_s	$D_{br} = 20\ d_s$

Concrete strength class:	Exposure class:
C35/45	XC1, WO
Concrete cover:	Stirrups: c_{nom} = 25 mm
Chamfer strip:	10 mm / 10 mm
Reinforcing and Prestressing Steel:	
Reinforcement bars:	B500A
Reinforcement mesh:	B500A
Prestressing steel:	St 1570/1770 (Z-12.3-107)

Surface finish						
	Filling side	rough	rubbed	sanded	troweld	special formwork
	Formwork side	exposed concrete smooth	no requirements	texture	exposed aggregate concrete	special formwork

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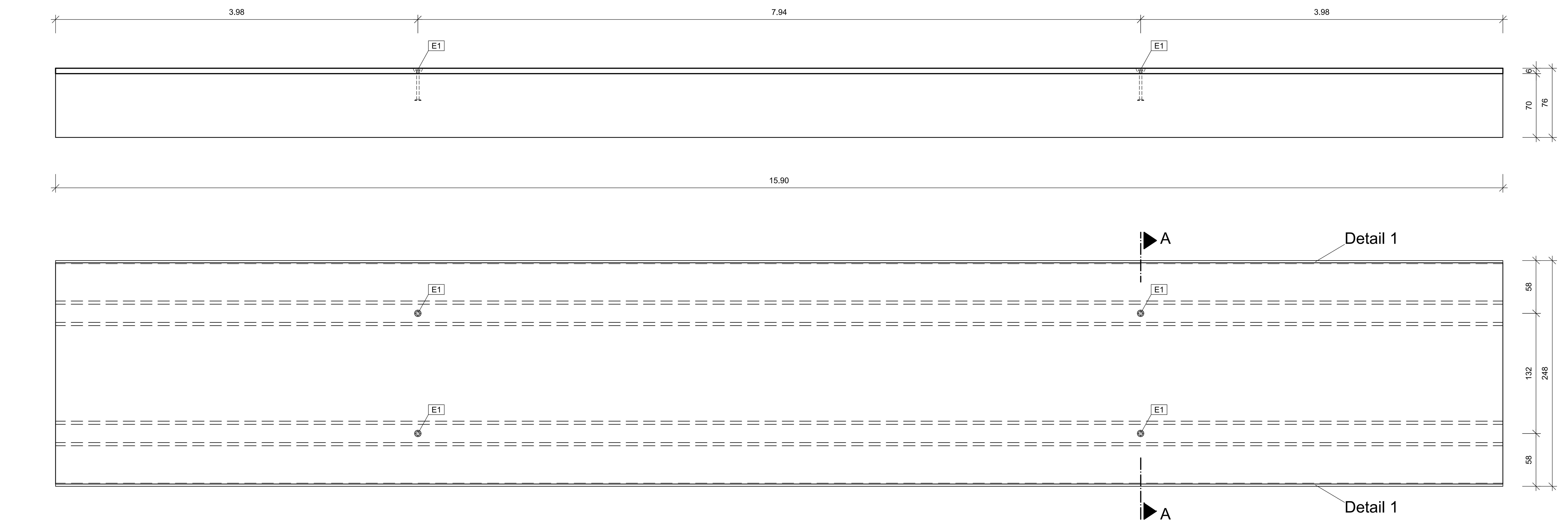
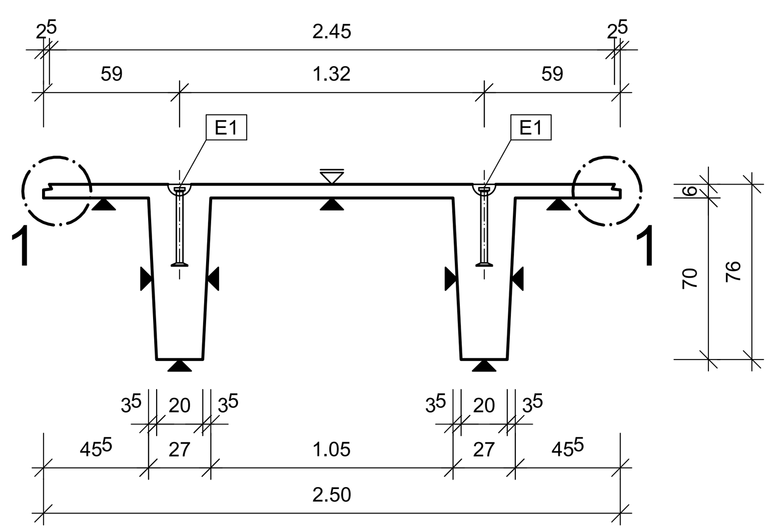
Supervisor:	Prof. Dr.-Ing. Ch. Enderle Prof. Dr. R. Kianoush
Project:	Construction of a Factory Hall in Betzweiler-Wälde, Germany
Plan type:	Formwork and Reinforcement Plan of the Column C.05

Drawn by:	Scale:	Plan-Nr.:
Olesja Befus	1:25 - m	A.4
Date: 31.07.2018		

Double-Tee Slab

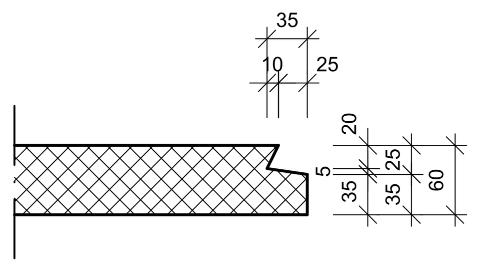


Section A-A

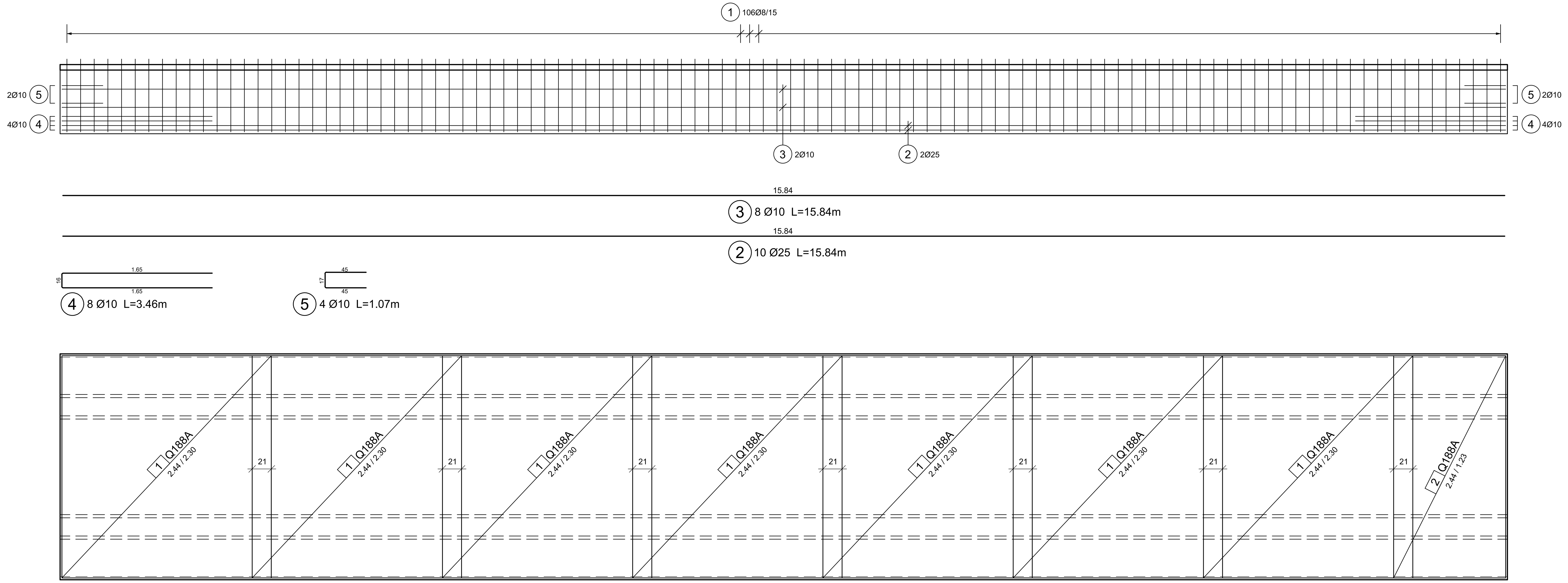


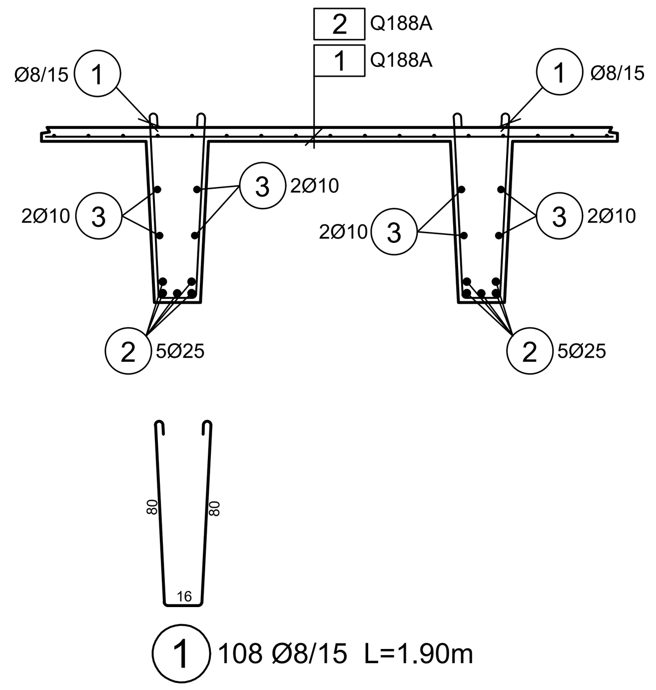
Detail 1

1:5 - mm



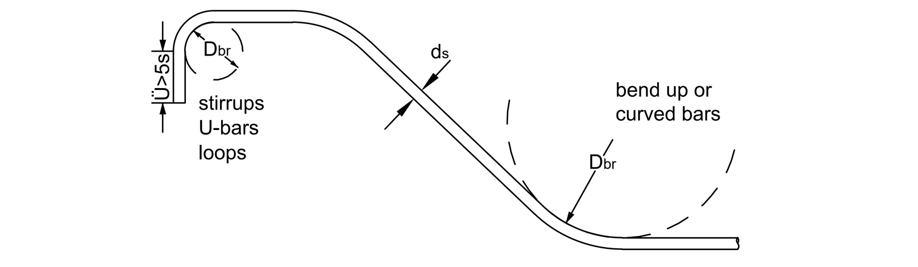
Reinforcement





Installation Parts		
E1	HALFEN DEHA Spherical head anchor 6000-10,0-0340	4 Pcs

Bending of the Reinforcment Bars (DIN EN 1992-1-1)



Diameter of the bar d_s [mm]	< 20	$D_{br} = 4\ d_s$	Concrete cover	> 100 mm and > 7 d_s	$D_{br} = 10\ d_s$
	≥ 20	$D_{br} = 7\ d_s$		> 50 mm and > 3 d_s	$D_{br} = 15\ d_s$
				≤ 50 mm and ≤ 3 d_s	$D_{br} = 20\ d_s$

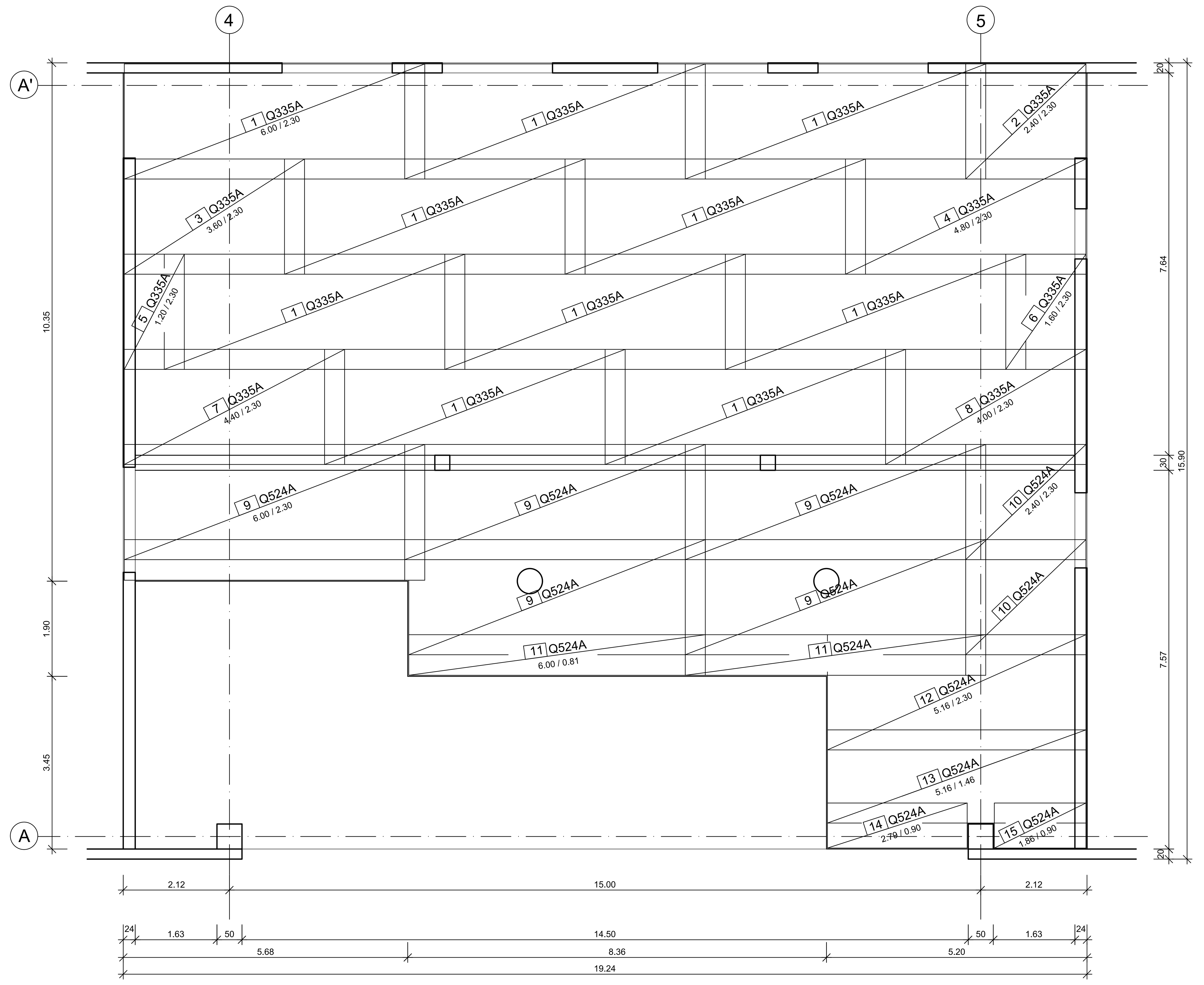
Concrete strength class:	Exposure class:
C35/45	XC1, WO
Concrete cover:	$c_{nom,slab}$ = 20 mm, $c_{nom,beam}$ = 25 mm
Chamfer strip:	10 mm / 10 mm
Reinforcing and Prestressing Steel:	
Reinforcement bars:	B500A
Reinforcement mesh:	B500A
Prestressing steel:	St 1570/1770 (Z-12.3-107)

Surface finish	Filling side	rough	rubbed	sanded	troweld	special formwork
	Formwork side	exposed concrete smooth	no requirements	texture (ST)	exposed aggregate concrete (WB)	special formwork (XX)

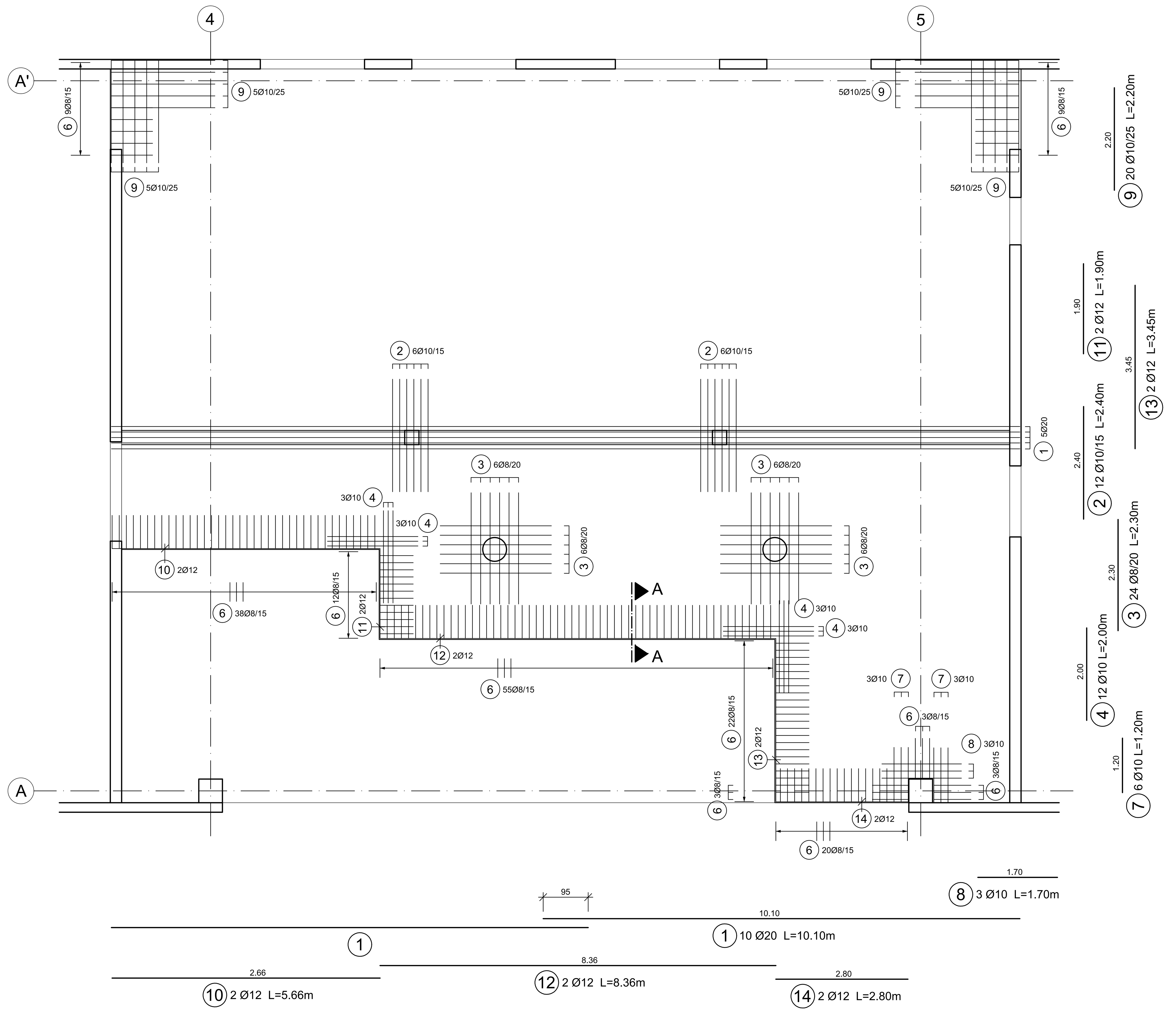
Karlsruhe University of Applied Sciences Ryerson Universtiy Faculty of Civil Engineering	**Master Project 2018** A Structural Analysis of a Factory Hall as a Precast Concrete Structure by Olesja Befus
Supervisor:	Prof. Dr.-Ing. Ch. Enderle Prof. Dr. R. Kianoush
Project:	Construction of a Factory Hall in Betzweiler-Wälde, Germany
Plan type:	Formwork and Reinforcement Plan of a Double-Tee Slab

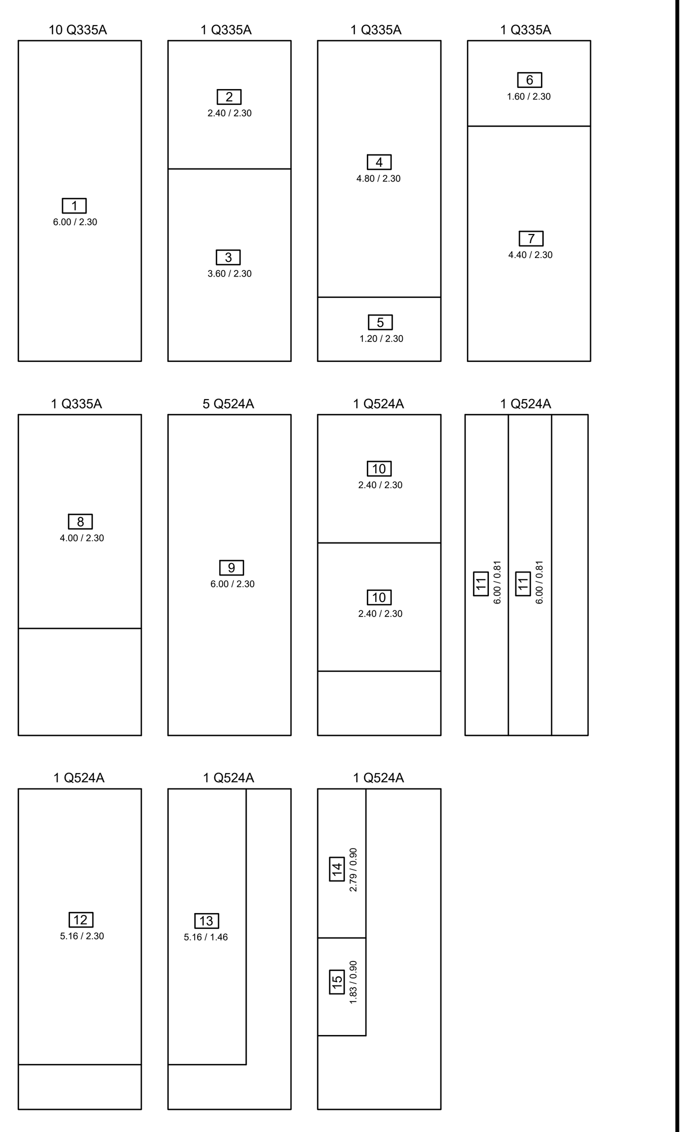
Drawn by: Olesja Befus	Scale: 1:25 - m	Plan-Nr.: A.5
Date: 31.07.2018		

Upper Mesh Reinforcement



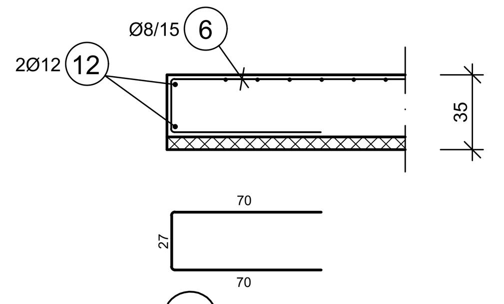
Upper Reinforcement Bars

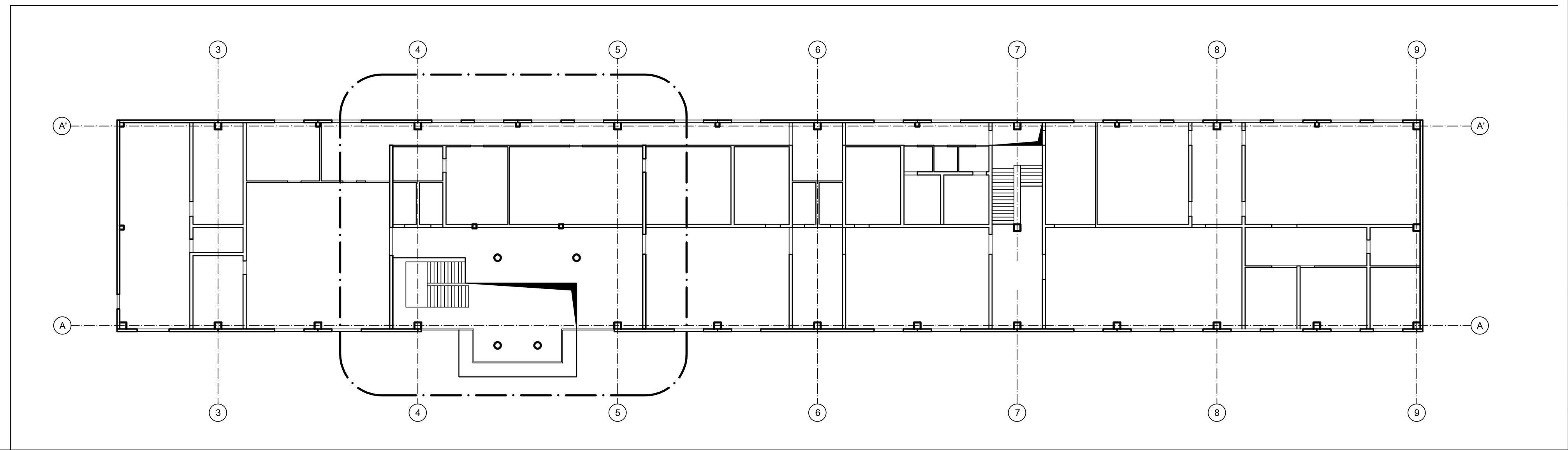




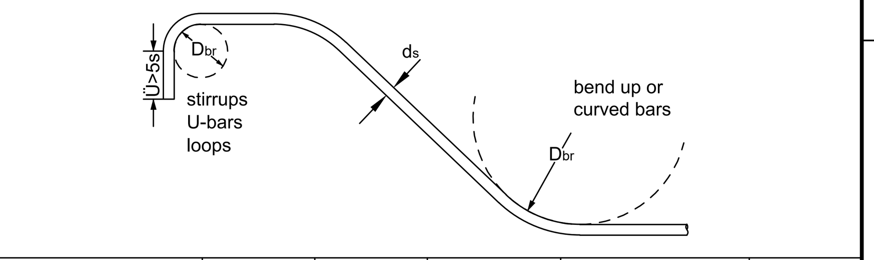
Section A-A

1:25





Bending of the Reinforcment Bars (DIN EN 1992-1-1)



Diameter of the bar d_s [mm]	< 20	$D_{br} = 4\ d_s$	Concrete cover	> 100 mm and > 7 d_s	$D_{br} = 10\ d_s$
				> 50 mm and > 3 d_s	$D_{br} = 15\ d_s$
	≥ 20	$D_{br} = 7\ d_s$		≤ 50 mm and ≤ 3 d_s	$D_{br} = 20\ d_s$

Concrete strength class: C30/37	Exposure class: XC1, WO
Concrete cover:	c_{nom} = 20 mm
Chamfer strip:	10 mm / 10 mm
Reinforcing and Prestressing Steel:	
Reinforcement bars:	B500A
Reinforcement mesh:	B500A
Prestressing steel:	St 1570/1770 (Z-12.3-107)

Surface finish						
	Filling side	rough	rubbed	sanded	troweld	special formwork (XX)
	Formwork side	exposed concrete smooth	no requirements	texture (ST)	exposed aggregate concrete (WB)	special formwork (XX)

Karlsruhe University of Applied Sciences

Ryerson Universtiy

Faculty of Civil Engineering

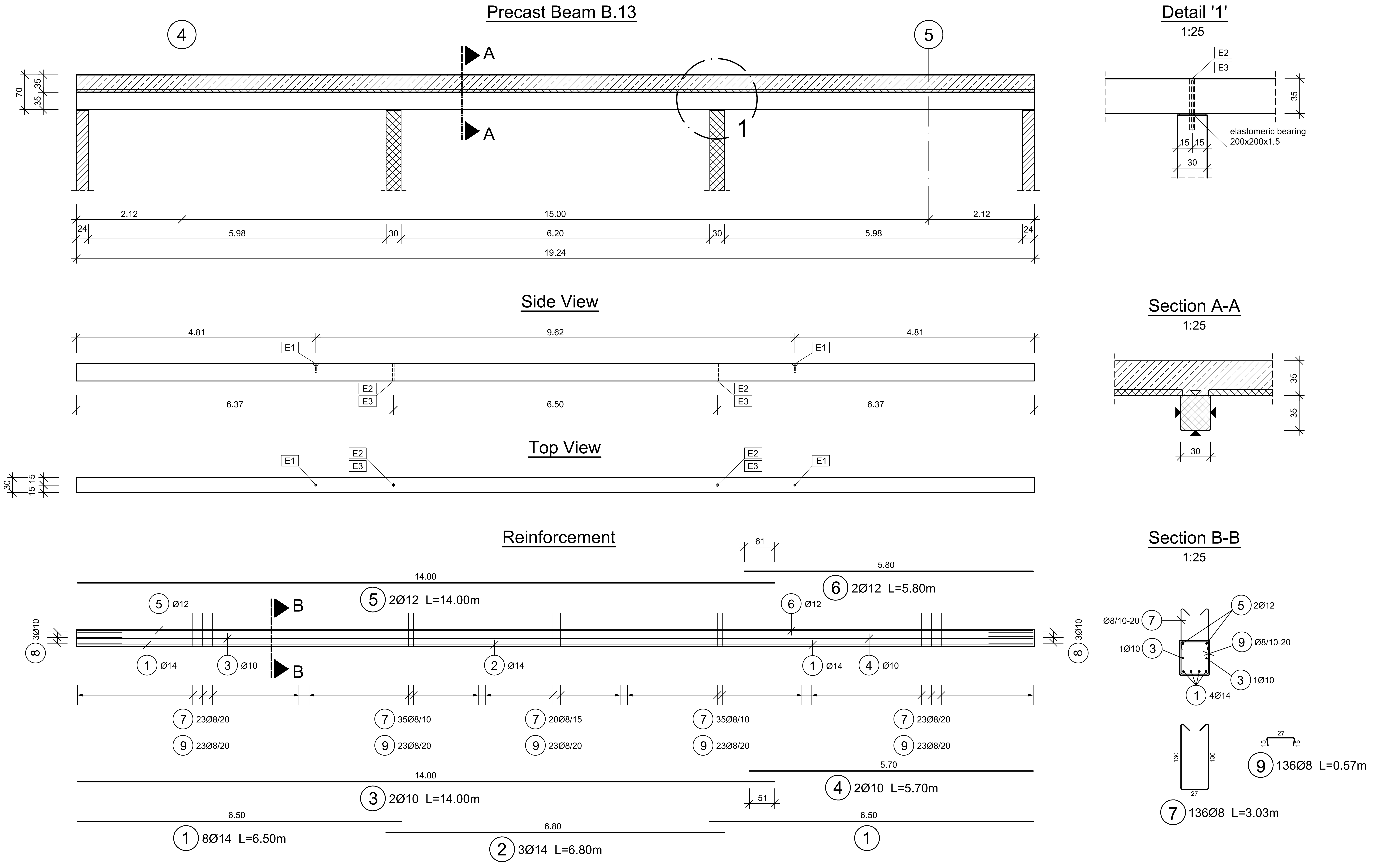
Master Project 2018

A Structural Analysis of a Factory Hall as a Precast Concrete Structure

by Olesja Befus

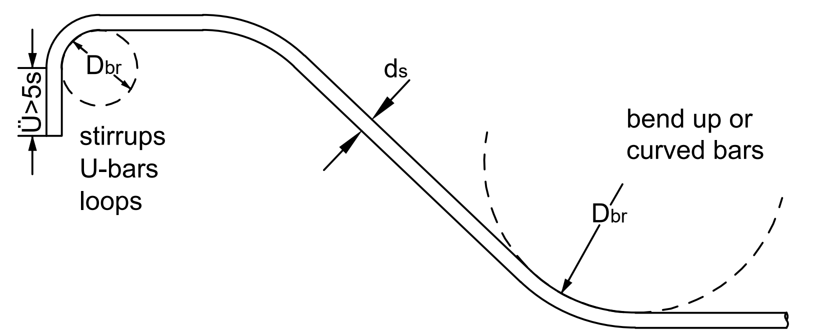
Supervisor:	Prof. Dr.-Ing. Ch. Enderle Prof. Dr. R. Kianoush
Project:	Construction of a Factory Hall in Betzweiler-Wälde, Germany
Plan type:	Formwork and Reinforcement Plan of Part of the Slab S.12

Drawn by: Olesja Befus	Scale:	Plan-Nr.:	
Date: 31.07.2018	1:50 - m	A.6	



Installation Parts		
E1	HALFEN DEHA Spherical head anchor 6000-4,0-0170	2 Pcs
E2	Corrugated pipe Ø50	2 Pcs
E3	Dowel Ø20	2 Pcs

Bending of the Reinforcment Bars (DIN EN 1992-1-1)



Diameter of the bar d_s [mm]	< 20	$D_{br} = 4\ d_s$	Concrete cover	> 100 mm and > 7 d_s	$D_{br} = 10\ d_s$
				> 50 mm and > 3 d_s	$D_{br} = 15\ d_s$
	≥ 20	$D_{br} = 7\ d_s$		≤ 50 mm and ≤ 3 d_s	$D_{br} = 20\ d_s$

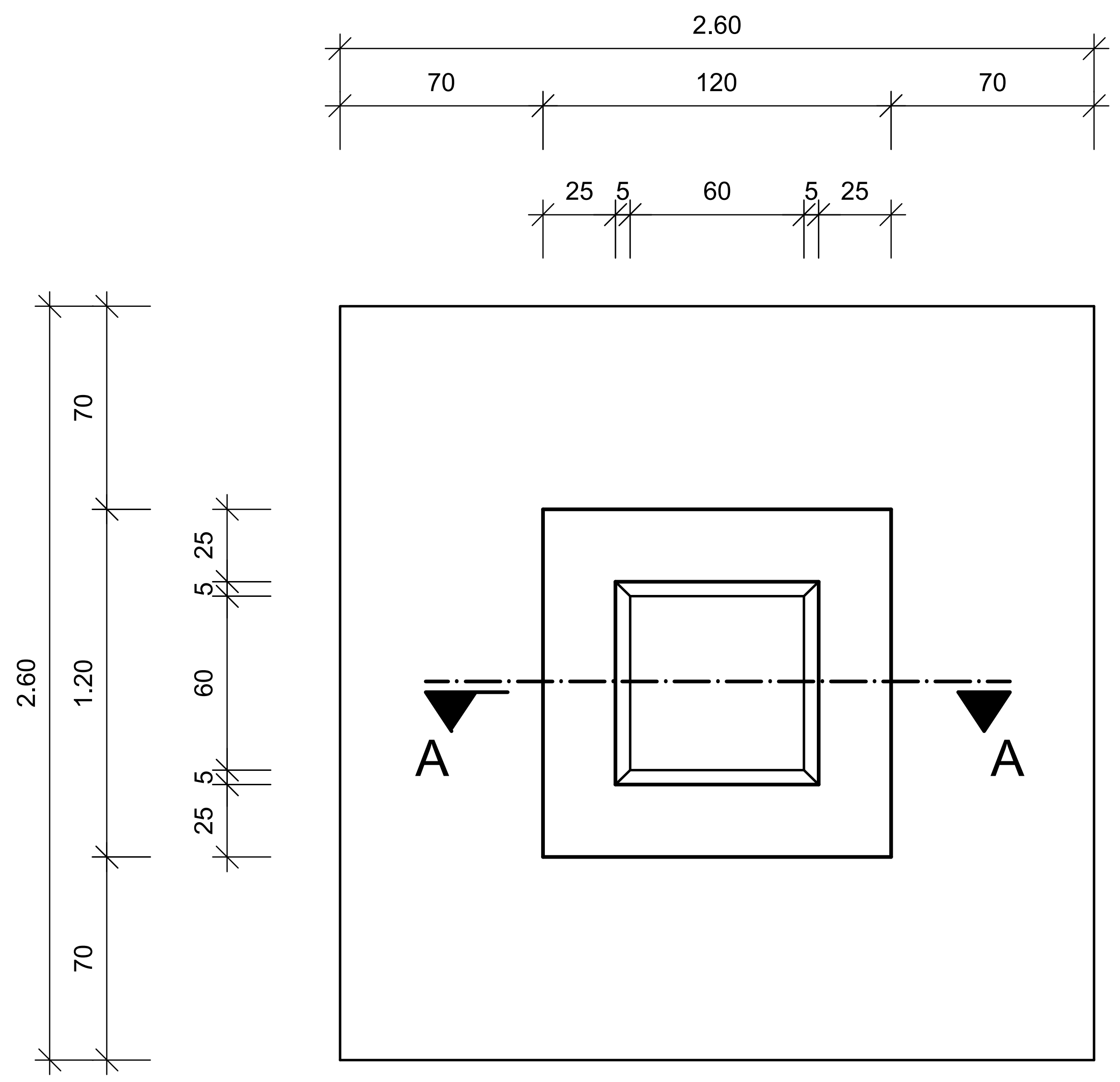
Concrete strength class:	Exposure class:
C30/37	XC1, WO
Concrete cover:	Stirrups: c_{nom} = 25 mm
Chamfer strip:	10 mm / 10 mm
Reinforcing and Prestressing Steel:	
Reinforcement bars:	B500A
Reinforcement mesh:	B500A
Prestressing steel:	St 1570/1770 (Z-12.3-107)

Surface finish						
	Filling side	rough	rubbed	sanded	troweld	special formwork
	Formwork side	exposed concrete smooth	no requirements	texture	exposed aggregate concrete	special formwork

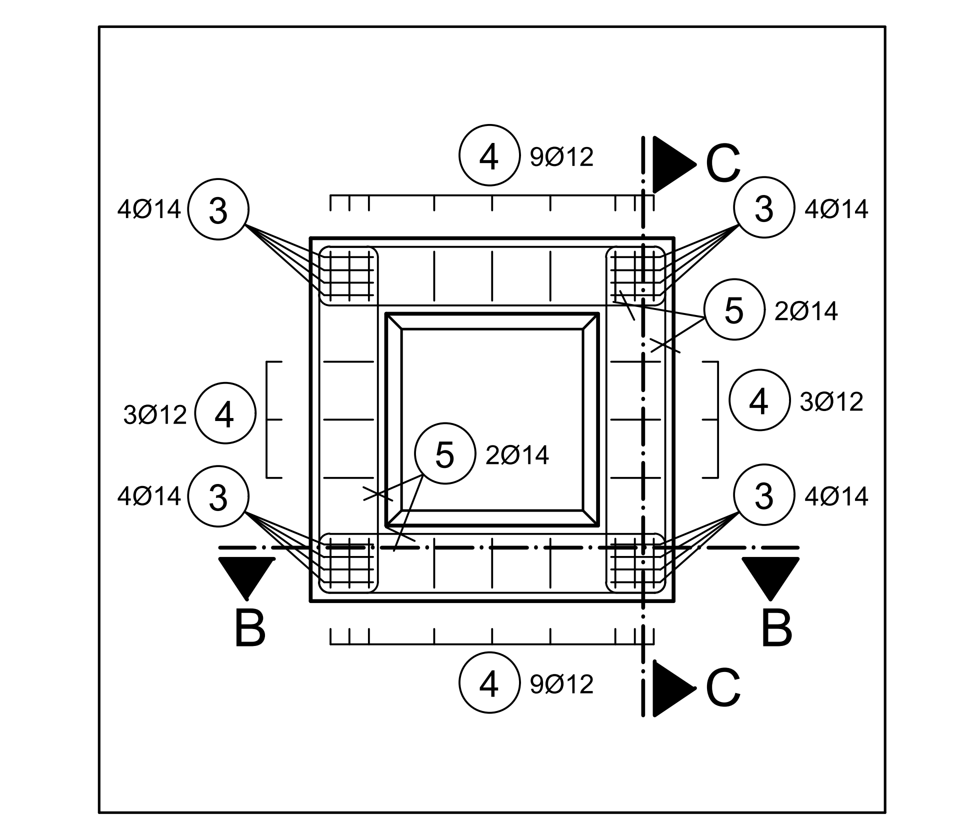
Karlsruhe University of Applied Sciences	Master Project 2018
Ryerson Universtiy	A Structural Analysis of a Factory Hall as a Precast Concrete Structure
Faculty of Civil Engineering	by Olesja Befus
Supervisor:	Prof. Dr.-Ing. Ch. Enderle Prof. Dr. R. Kianoush
Project:	Construction of a Factory Hall in Betzweiler-Wälde, Germany
Plan type:	Formwork and Reinforcement Plan of the Precast Beam B.13

Drawn by: Olesja Befus	Scale:	Plan-Nr.:
Date: 31.07.2018	1:50 - m	A.7

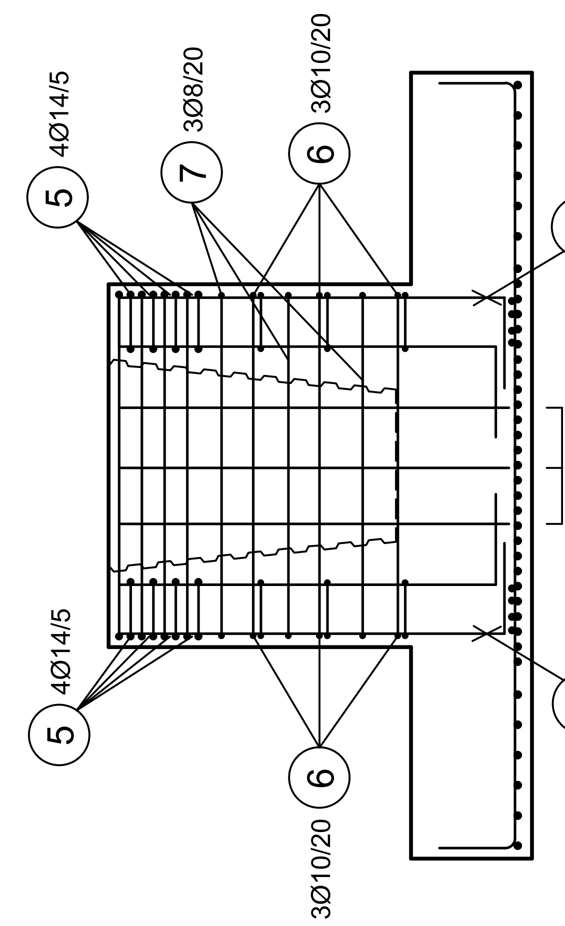
Pocket Foundation F.21



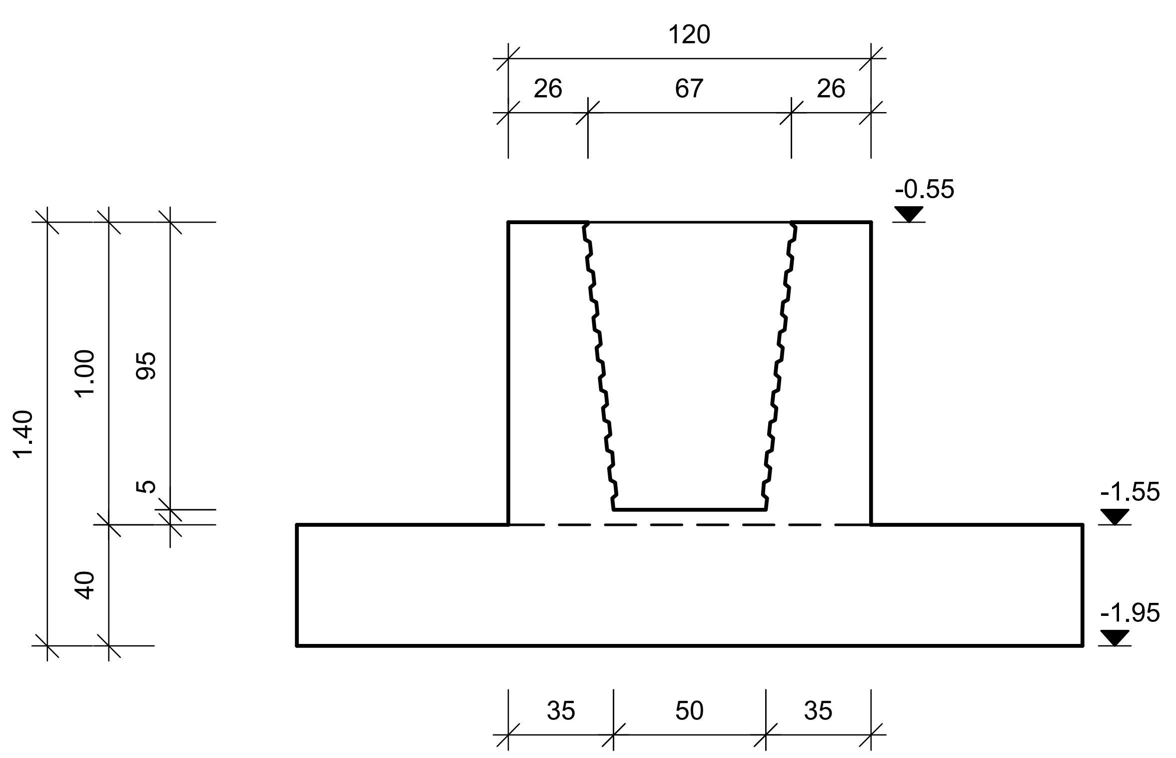
Reinforcement



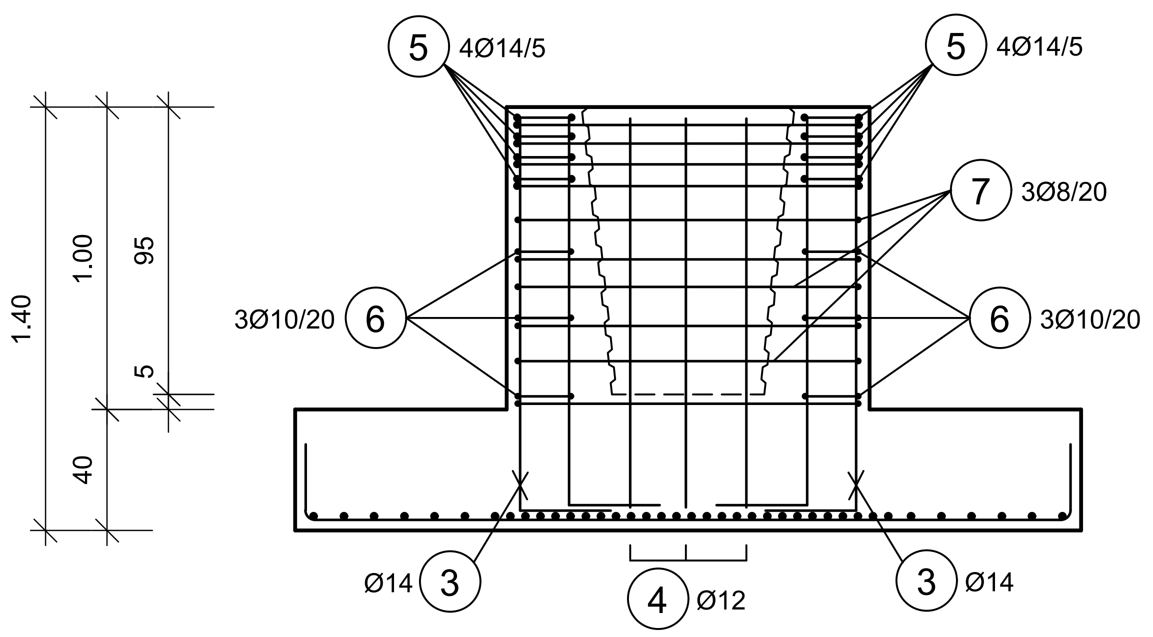
Section C-C

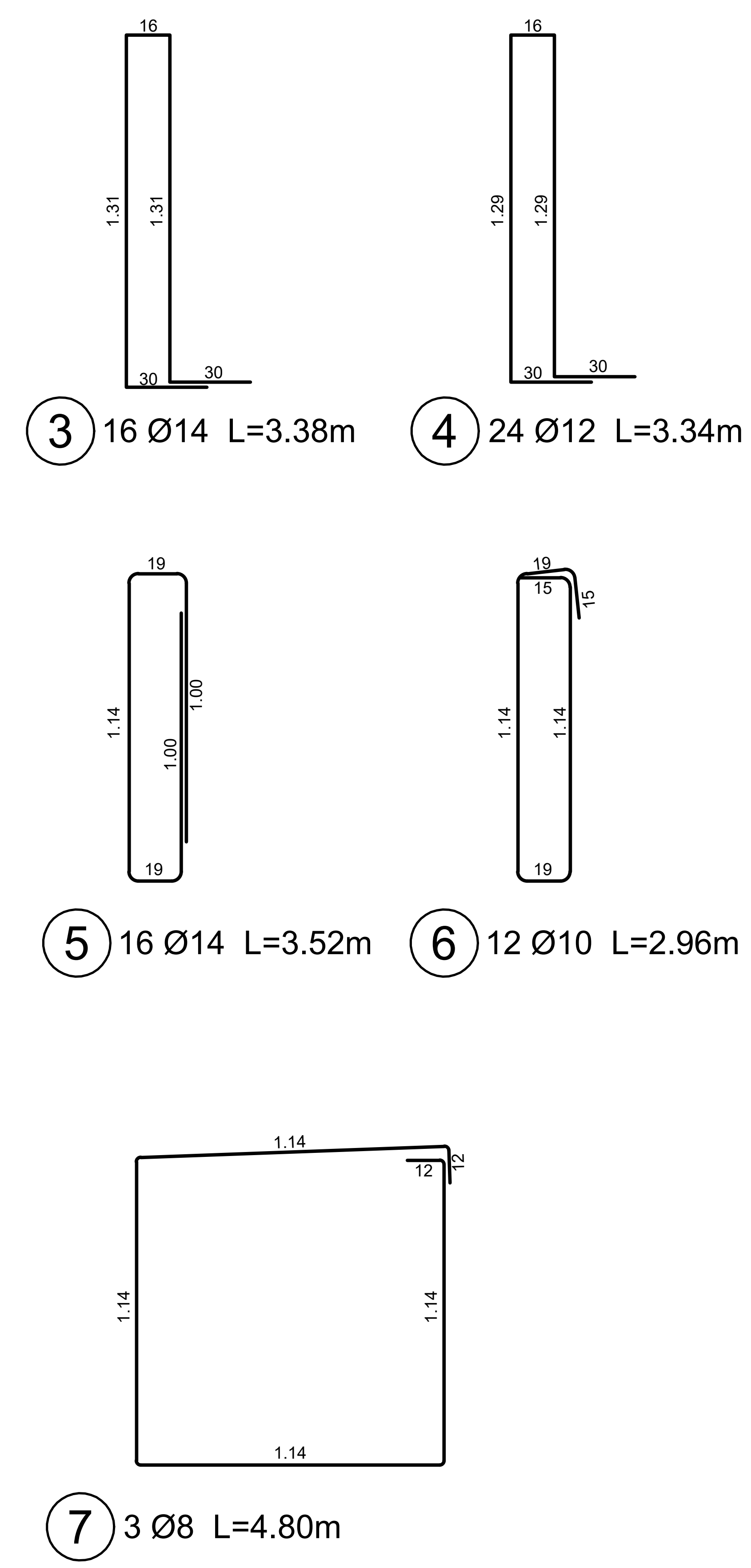


Section A-A

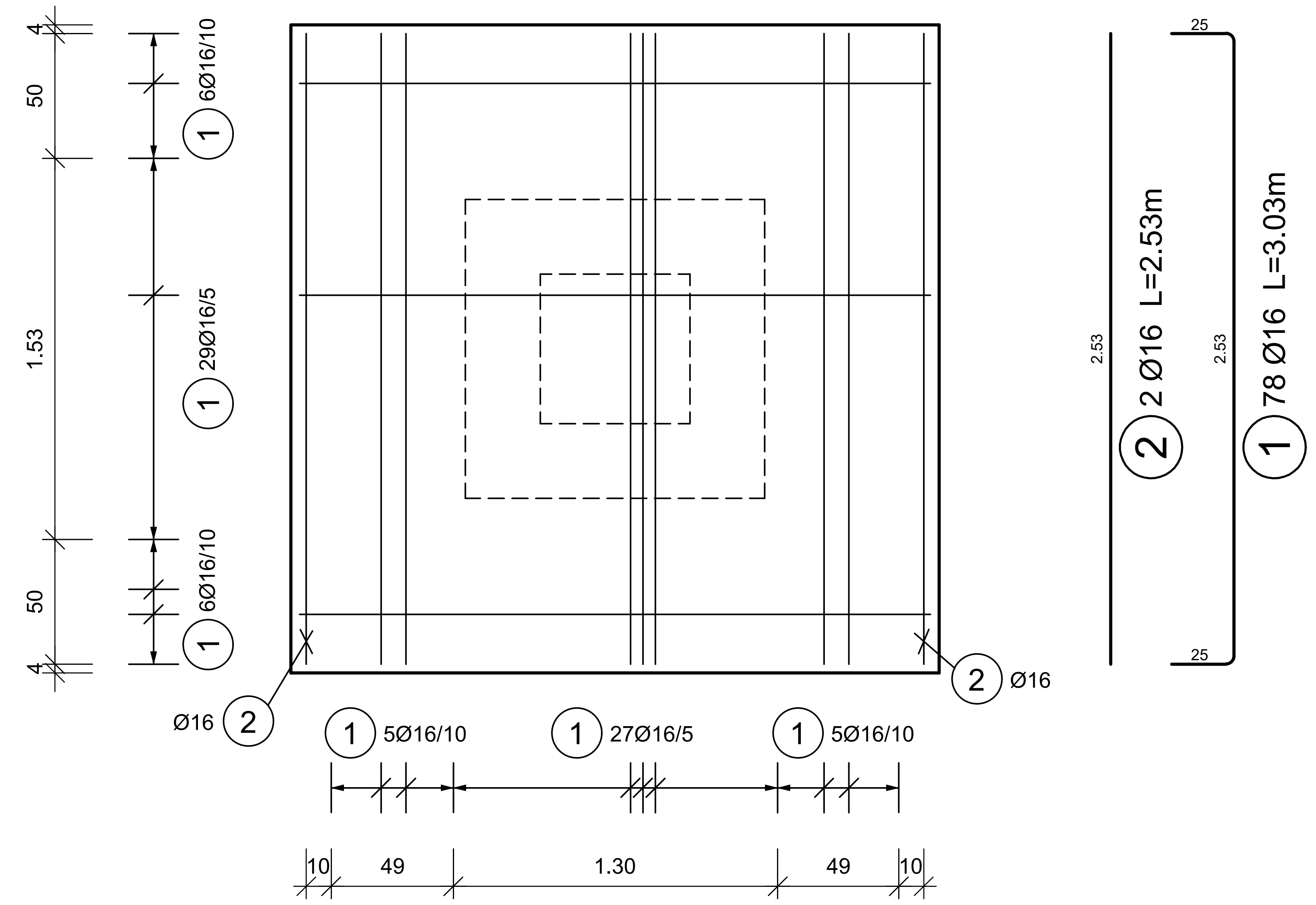


Section B-B

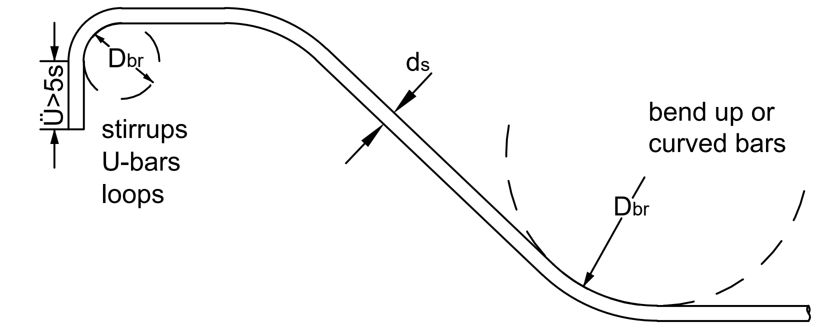




Top View Slab



Bending of the Reinforcment Bars (DIN EN 1992-1-1)



Diameter of the bar d_s [mm]	< 20	$D_{br} = 4\ d_s$	Concrete cover	> 100 mm and > 7 d_s	$D_{br} = 10\ d_s$
				> 50 mm and > 3 d_s	$D_{br} = 15\ d_s$
	≥ 20	$D_{br} = 7\ d_s$		≤ 50 mm and ≤ 3 d_s	$D_{br} = 20\ d_s$

Concrete strength class:	Exposure class:
C35/45	XC2, WF
Concrete cover:	c_{nom} = 20 mm + 15 mm = 35 mm
Chamfer strip:	10 mm / 10 mm
Reinforcing and Prestressing Steel:	
Reinforcement bars:	B500A
Reinforcement mesh:	B500A
Prestressing steel:	St 1570/1770 (Z-12.3-107)

Surface finish						
	Filling side	rough	rubbed	sanded	troweld	special formwork XX
	Formwork side	exposed concrete smooth	no requirements	texture ST	exposed aggregate concrete WB	special formwork XX

Karlsruhe University of Applied Sciences Ryerson Universtiy Faculty of Civil Engineering	Master Project 2018 A Structural Analysis of a Factory Hall as a Precast Concrete Structure by Olesja Befus
Supervisor:	Prof. Dr.-Ing. Ch. Enderle Prof. Dr. R. Kianoush
Project:	Construction of a Factory Hall in Betzweiler-Wälde, Germany
Plan type:	Formwork and Reinforcement Plan of the Pocket Foundation F.21

Drawn by: Olesja Befus	Scale:	Plan-Nr.:
Date: 31.07.2018	1:25 - m	A.8

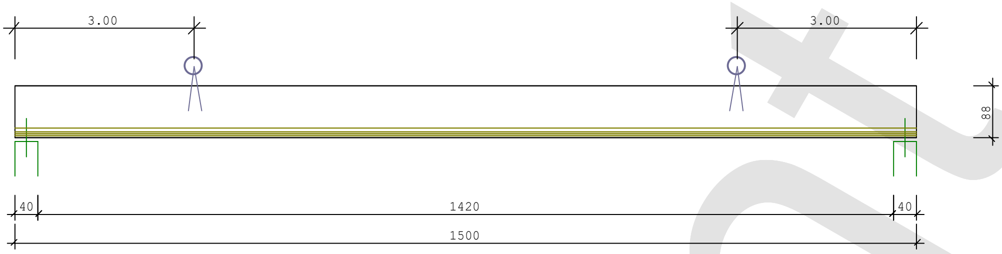
APPENDIX B SOFTWARE CALCULATIONS

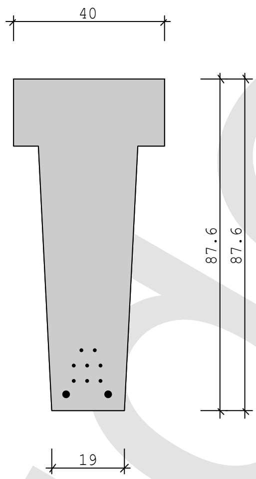
B.1 DESIGN RESULTS FROM 'NEMETSCHEK FRILO'

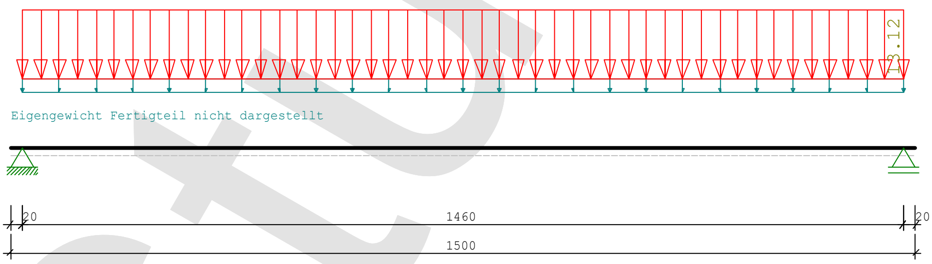
B.1.1. Design Calculations POS. P.02 – Prestressed Concrete Purlin

Position: P.02 - Prestressed Concrete Purlin

Spannbettbinder B8 01/2018 (Frilo R-2018-1/P12)







Advices:

No continuous reinforcement to 10 cm below UE found!

<u>System:</u>

Parallel flange beam

Basics:

Load combinatorics:	DIN EN 1990/NA:2010-12 + EN 1990:2002/AC:2010 ULS: Structural safety checks(STR) permanent/variable design situation with equation 6.10
Design code:	DIN EN 1992-1-1/NA/A1:2015-12 + EN 1992-1-1:2004 /AC:2010 Prestressing for pretensioning

System Geometry:

Total $L = 15.00$ m Effective $L1 = 14.60$ m
Outstand left $L0 = 0.20$ m right $L2 = 0.20$ m
Height beam : $H2 = 87.6$ cm
Relation eff.span to height of beam:
$L1/H2 = 16.67$

Erection attachment, distance from the beginning resp. end of beam:
Hook $L8 = 3.00$ m right $L9 = 3.00$ m

Cross-section Precast :

Layer of cross-section from top to bottom			
Nr	Width [cm]	Distance [cm]	Remarks
1	40.0	0.0	Web begin
2	40.0	17.6	
3	26.0	17.7	
4	19.0	87.6	Web end

Material:

Prestressing steel

SpSt 1570/1770 Strand 7 wires
$d_d = 4.1$ mm $d_p = 12.3$ mm
$E_p = 196000$ N/mm² $A_p = 0.930$ cm²
$f_{p0.1k} = 1570$ N/mm² $f_{pk} = 1770$ N/mm²
$\varepsilon_{uk} = 35.0$ ‰ $\varepsilon_{ud} = 25.0$ ‰ + $\varepsilon_p^{(0)}$

Partial safety factor :
$\gamma_s = 1.15$

Coeff. prestress:
charact. value $r_{sup} = 1.05$ $r_{inf} = 0.95$
Design value $\gamma_{p,max} = 1.00$ $\gamma_{p,min} = 1.00$

Proof of crack width
Equ. diameter $d_{pV} = 7.20$ mm $\xi = 0.60$ (Tab. 6.2)

Relaxation (from approval)				
σ_{p0}/f_{pk}	10 h	200 h	1000 h	1000000 h
0.60	0.3	0.6	0.8	2.8
0.70	0.8	1.6	2.0	7.0
0.80	2.0	4.0	5.0	14.0
Losses in % as a function of time and stress				

Permitted stresses:
in formwork $\sigma_p \leq 1413.0$ N/mm² $(0.90^* f_{p0.1k})$
after anchor. release $\sigma_p \leq 1327.5$ N/mm² $(0.75^* f_{pk})$
char. Lc $\sigma_p \leq 1413.0$ N/mm² $(0.90^* f_{p0.1k})$
q.perm.Lc $\sigma_p \leq 1150.5$ N/mm² $(0.65^* f_{pk})$

Transmission length
$\eta_{p1} = 2.85$ $\eta_1 = 1.00$
$\alpha_1 = 1.25$ $\alpha_2 = 0.19$
$\sigma_{pm0} = 945$ N/mm²
PT: $f_{ctdt} = 1.02$ N/mm² $f_{bpt} = 2.91$ N/mm²
$l_{pt} = 0.95$ m

Dispersion_length:
$d = 0.83$ m $l_{disp} = 1.26$ m

Reinforcing steel:

	Longitudinal B500B			Stirrup B500B	
E_s =	200000	N/mm²	E_s =	200000	N/mm²
f_{yk} =	500	N/mm²	f_{yk} =	500	N/mm²
f_{tk} =	540	N/mm²	f_{tk} =	540	N/mm²
ε_{uk} =	50.0	‰	ε_{uk} =	50.0	‰
ε_{ud} =	25.0	‰	ε_{ud} =	25.0	‰

Partial safety factor :
γ_s = 1.15 γ_s = 1.15

permitted stresses in SLS :
$\sigma_s \leq$ 400 N/mm² $\sigma_s \leq$ 400 N/mm² (0.80 * f_{yk})

Requirements durability:

attack on concrete W0
attack on reinforc. XC1
min. concrete class C 16/20
stirrup ϕ,l = 10 mm
long. reinforcement ϕ,m = 16 mm
prestressed steel d_p = 12.3 mm strand , s >=2.5* dp ,$\sigma_p^{(0)}$ <=1000 N/mm2
allowance in design Δc_{dev} = 10 mm
stirrup $c_{min,l}$ = 10 mm *5
concrete coverage $c_{nom,l}$ = 20 mm *5
longitudinal bars $c_{min,m}$ = 16 mm *5
concrete coverage $c_{nom,m}$ = 30 mm *1
prestressing steel : $c_{min,p}$ = 25 mm *5
concrete coverage $c_{nom,p}$ = 35 mm
laying dist. link $c_{,l}$ = 20 mm
all. crack width w_{max} = 0.20 mm
decompression not req.
*1:with cmin,l
*5: bond decisive

Concrete:

Precast
C 30/37
f_{ck} = 30.00 N/mm²
α_{cc} = 0.85
$f_{ctk0.05}$ = 2.03 N/mm²
α_{ct} = 0.85
γ = 25.00 kN/m³ Unit
E_{cm} = 33000 N/mm²
α_E = 1.00 Coeff. E-module
G_{cm} = 13200 N/mm²

Partial safety factor :
γ_c = 1.50

permitted stresses in SLS :
char. Lc $\sigma_c \geq$ -18.00 N/mm²
q.perm.Lc $\sigma_c \geq$ -13.50 N/mm²
Removal the anchor t= t0T(sto)
$f_{cm(t)}$ = 25.18 N/mm²
$f_{ck(t)}$ = 17.18 N/mm²
linear creep $\sigma_c \geq$ -7.73 N/mm² (k2=0.45)
maximum $\sigma_c \geq$ -10.31 N/mm² (k6=0.60)

Creep modulus & shrinkage strain

no heat treatment, t0T = t0
CementStrenght class 42,5R;52,5
ρ= 0.5 (Aging coefficient)
Reference point for t0 is the start of the concreting of the precast

Creep	t0 Days	RH %
Storage	3	70
Utilization precast	180	50

L.	Segment	Part-cross-section	t0	t	α	$t_{0,eff}$ B.9	β_{t0} B.5	$ß_H$ B.8	$\beta_{c(t,t0)}$ B.7	ϕ_{RH} B.3	β_{fcm} B.4	$\phi(t,t0)$ B.1
1	Storage	PcC	3.0	180.0	1	7.7	0.62	527.5	0.66	1.47	2.73	1.65
2	Utilization precast	PcC	180.0	26000.0	1	7.7	0.62	515.6	0.33	1.80	2.73	1.01

L.	A [cm^2]	U [cm]	h0 [cm]	$ß_{ds}(t0,ts)$	$ß_{ds}(t,ts)$ 3.10	$ß_{RH}$ B.12	$\varepsilon_{cd,0}$ B.11	$ß_{as}$ 3.13	ε_{ca}/10e6 3.12	$\varepsilon_{cs}(t,t0)$ [‰]
1	2280.05	248.2	183.7	0.000	0.640	1.02	501.5	0.93	50.00	0.327
2	2280.05	248.2	183.7	0.640	0.996	1.36	667.9	1.00	50.00	0.212

Loads:

Self weight

Beam beginning g11 = 5.70 kN/m
Beam end g12 = 5.70 kN/m

Total G = 85.5 kN
Volume V = 3.42 m^3
Surf. A = 31.68 m^2

Live loads

Units: Single load[kN] Single moment[kNm] line load[kN/m]

span	type	gle	qle	Dist. a [m]	gri	qri	Length [m]	Fact	Act.	Sim.	Pos.
1	1	2.59	13.12					1.00	10	0	

Load types: 1 = uniformly distr., 2 = single load at a, 3 = single moment at a
4 = trapezoidal load from a, 5 = triangle load over L

Actions:

Act.	γ_Q	ψ_0	ψ_1	ψ_2	Dep.	Cat.	Description
10	1.50	0.50	0.20	0.00	0	S	Schnee bis NN +1000m

Tendons:

Dist(LE) > 4.8 cm axis horizontal > 4.3 cm vertical > 3.7 cm

lay. No.	num-ber	area Ap [cm^2]	Dist.LE Yp [cm]	Prestressing $\sigma_p^{(0)}$ [N/mm^2]	<--- Isolations ---> Count	to x1 [m]	from x2 [m]
1	3	2.79	7.5	1000	0		
2	3	2.79	11.7	1000	0		
3	2	1.86	15.9	1000	0		

x1 and x2 with respect to the left beginning from joint
The calculation of the losses due to creep, shrinkage and relaxation
following the method from Abelein

Untensioned reinforcement:

Layer No.	num-ber	diam. Φs,l [mm]	area As [cm^2]	Dist.LE Ys [cm]	effective range from xA [m]	to xE [m]
1	2	16	4.02	4.0	0.00	15.00

xA and xE with respect to the left beginning from joint

Surface reinforcement acc.to Tab. NA.J.41 (B0 < D0) :

Web (Z1/S3) AsS = 1.21 cm^2/m (UwkS <= XC4) (per side)
Edge top (Z2/S3) AsO = 0.00 cm^2/m (UwkS <= XC4)

Settings for shear resistance check

Bearing width, distance bearing edge, effective height of the bearing line
left bAl = 0.40 m al = 0.20 m dAl = 0.83 m
right bAr = 0.40 m ar = 0.20 m dAR = 0.83 m
For shear reinforcement not decisive ranges over support A and B:
xaRe=1.03 m direct bearing (width of bearing/2 + eff. depth)
xbLi=1.03 m direct bearing (width of bearing/2 + eff. depth)

Check the limit deformation:

Total sagging	f ≤ L/ 250	Increase deflection	|df| ≤ L/ 500
Cantilever left	f ≤ 0.2 cm		|df| ≤ 0.1 cm
Span	f ≤ 5.8 cm		|df| ≤ 2.9 cm
Cantilever right	f ≤ 0.2 cm		|df| ≤ 0.1 cm

quasi- permanent combination and eff. char. prestress
Deflection due to shrinkage considered
Tension stiffening: Member rigidity, Characteristic combination

RESULTS (summary)

Reaction forces (t = infinitely):

Units: all [kN] G:perm., Q:variable. ,V: Sum					
	<------char. value----->			<--ULS(PT)---->	
Support point	G	min Q	max Q	min V	max V
A (left)	61.66	0.00	95.78	61.66	226.90
B (right)	61.66	0.00	95.78	61.66	226.90

max. bending moment in erection state(char. value):

MF = = 570.36 kNm at x = = 7.50 m

Checks are not complied with:

Checkvalue		Extrem		Utilisation	x [m]
Resisting tens force bot	η =	0.98		1.02	14.80
Crack MinAs+AsDuc top	AsMin =	****	cm^2	****	0.02
Prc.:Compr.stress t0(sto)	σc =	-28.39	N/mm^2	2.75	3.00

Warning

Prc.:lin. creep t0(Sto) σc =-15.73 N/mm2 x= 0.79 m
σc <0.45*fck(t)= -7.73 N/mm2
_disproportional creeping by increased creep modulus considered(fk= 2.01)

Required shear reinforcement:

Column A: asw = 2.73 cm^2/m
Column B: asw = 2.73 cm^2/m

Bursting reinforcement

left Laying length = 0.95 m
from x = 0.00 m As = 3.8 cm^2
right Laying length = 0.95 m
from x = 15.00 m As = 3.8 cm^2

Check of anchorage

left: Tensile force resistance in anchoring area Util = 1.02
additional reinforcement necessary
right: Tensile force resistance in anchoring area Util = 1.02
additional reinforcement necessary

Overview crit. sections

Selected basic grid: 20 Sections

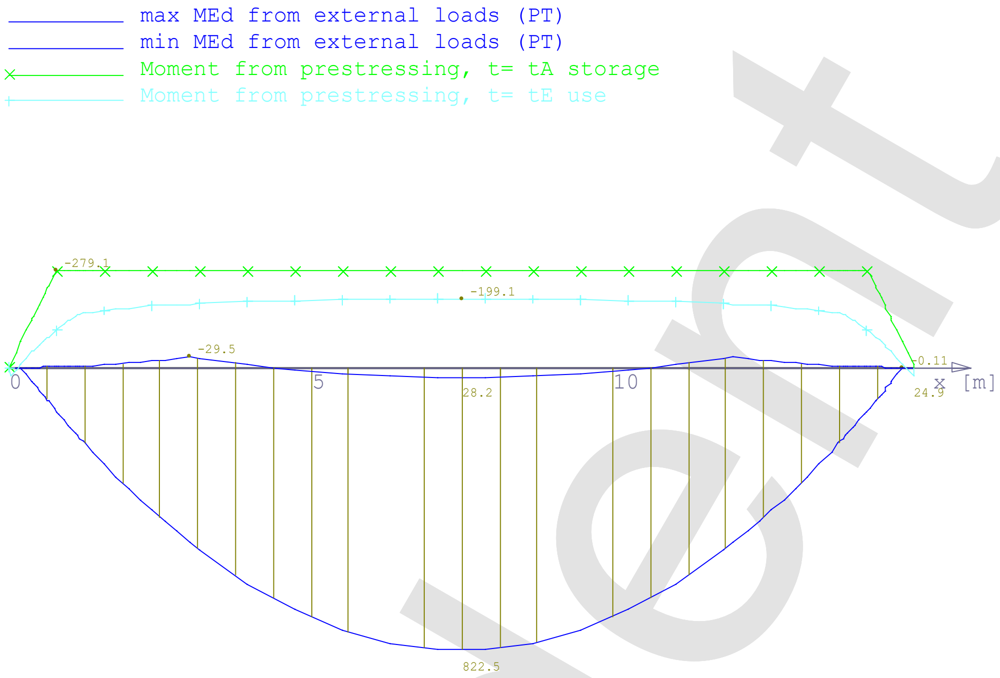
Checkvalue		Extrem		Utilisation	x [m]	
Flexural capacity bottom	η =	1.02		0.98	7.50	
Flexural capacity top	η =	****		****	0.59	
Resisting tens force bot	η =	0.98		1.02	14.80	!
Resisting tens force top	η =	****		****	0.59	
Prc.:Compr.stress t0(sto)	σc =	-28.39	N/mm²	2.75	3.00	!
Prc.:Compr.stress Cc	σc =	-17.20	N/mm²	0.96	3.00	
Tension prestress. steel	σp,Qc =	963.2	N/mm²	0.84	7.50	
Stress in prestress.steel	σp,Cc =	1110.9	N/mm²	0.79	7.50	
Stress in rebars	σs =	156.8	N/mm²	0.39	7.50	
Crack MinAs+AsDuc bottom	AsMin =	2.4	cm²	0.37	0.02	
Crack MinAs+AsDuc top	AsMin =	****	cm²	****	0.02	!
Crack width bottom	wk =	0.00	mm	0.01	7.50	
Crack width top	wk =	****	mm	****	0.47	
Sagging top	fo =	-1.0	cm	0.17	7.11	
Sagging bottom	fu =	0.1	cm	0.01	0.00	
Incr.-deflection(Util)	|df|=	0.7	cm	0.24	7.89	
Prc.:Shear reinf (web)	asw =	2.73	cm²/m	1.00	1.23	
Concrete strut capacity	η =	2.72		0.37	0.40	

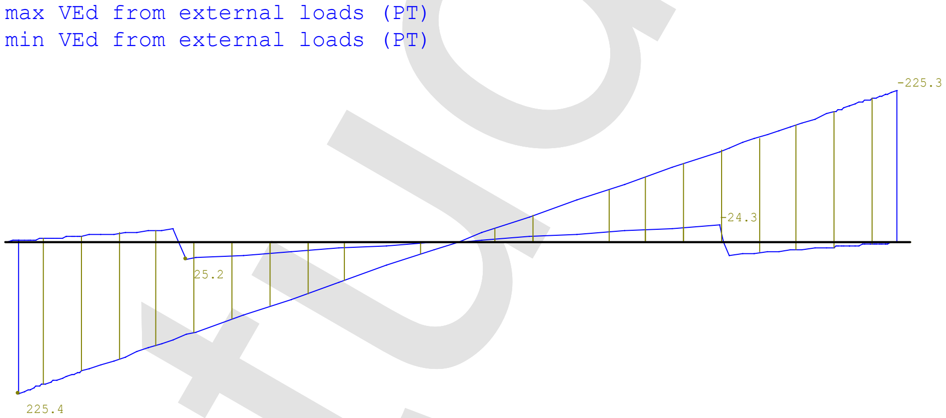
---- Check not required
**** Check not fulfilled
Prc.:Precast member Add.: in-situ supplement
IS : Installed state SC : State of construction
AsDuk:Ductility reinforcement

Linear creep limit, informative:		Extrem		Utilisation	x [m]
Prc.:lin. creep t0(Sto)	σc =	-15.73	N/mm²	2.03	0.79
Prc.:Compression quasi-permanent Lc	σc =	-7.07	N/mm²	0.52	2.17

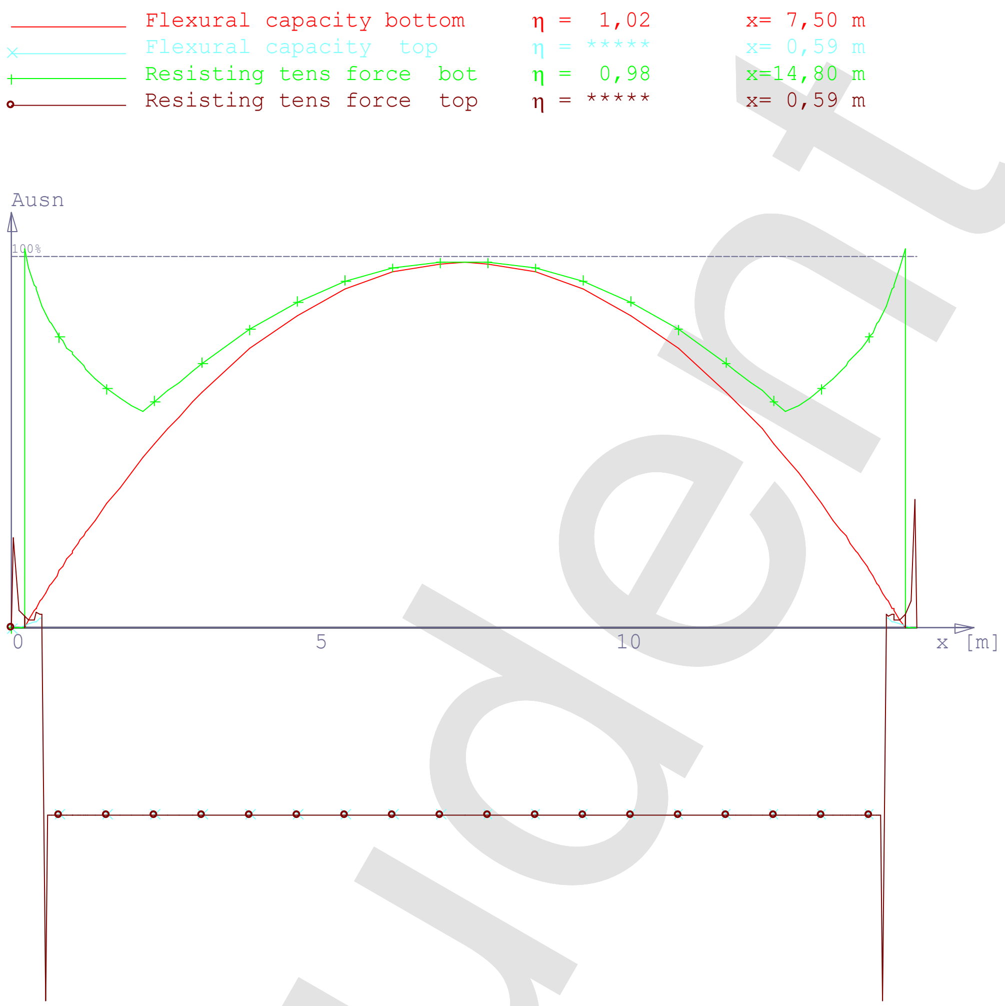
Tensile stress state I, informative:		Extrem		Utilisation	x [m]
Prc.:Tens.stress (IS)	σt =	9.34	N/mm²	3.23	7.50
Prc.:Tens.stress (SC)	σt =	3.17	N/mm²	1.76	0.79

Internal forces

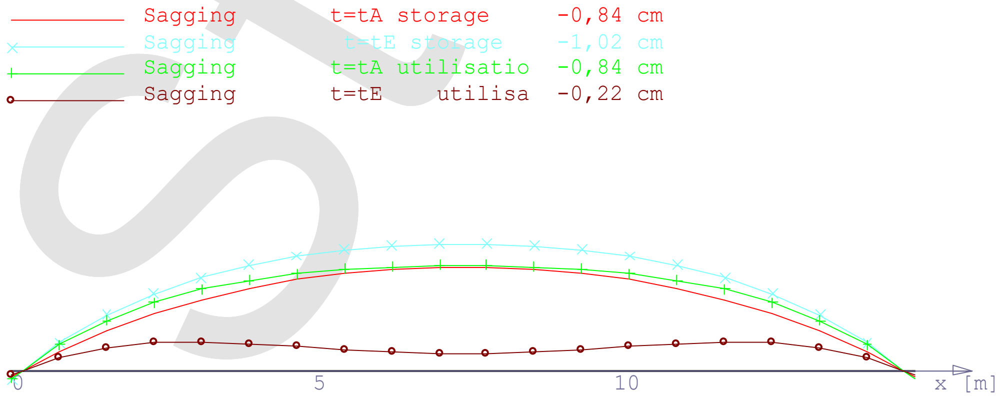




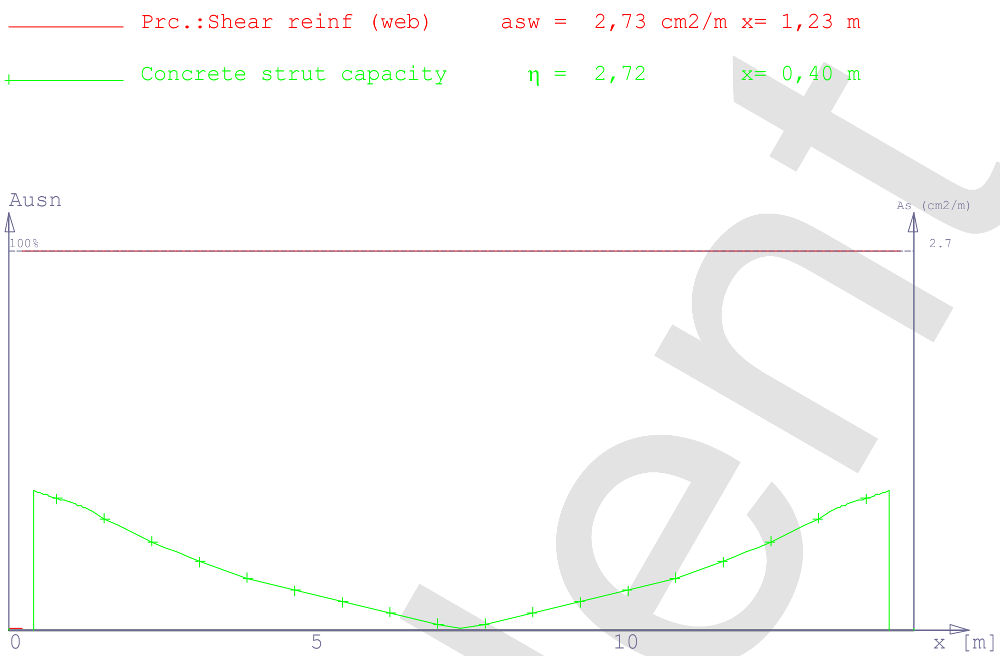
Bending resitance (failure safety)



Deformation



Shear force resistance (shear covering)



Anchorage by bond (over the left bearing)

l_{pt2} = 1.14 m Distance first bending crack l_r = 1.78 m
$f_{ctd}(t)$ = 1.02 N/mm² t= release anchorage
f_{bpt} = 2.91 N/mm²
η_{P2} = 1.40 f_{bpd} = 1.61 N/mm²

x [m]	Z_p [kN]	Z_s [kN]	T_{Ed} [kN]	η= $(Z_p+Z_s)/T_{Ed}$	Util
0.20	92.5	176.8	273.7	0.98	1.02
0.39	179.3	176.7	325.8	1.09	0.92
0.59	271.3	176.6	374.0	1.20	0.84
0.79	363.3	176.5	421.1	1.28	0.78
0.99	455.2	176.4	466.8	1.35	0.74
1.18	544.6	176.4	509.0	1.42	0.71
1.38	646.7	176.4	551.9	1.49	0.67
1.58	748.8	176.5	593.4	1.56	0.64
1.78	850.9	176.5	633.6	1.62	0.62
1.97	947.9	176.5	670.5	1.68	0.60
2.17	1044.8	176.5	708.0	1.73	0.58
2.37	1044.9	176.5	744.4	1.64	0.61
2.57	1045.0	176.5	778.8	1.57	0.64
2.76	1045.1	176.5	808.4	1.51	0.66

Z_p: resisting tensile force by the prestressed steel
Z_s: resisting tensile force by the rebars
T_{Ed}: tensile force to be anchored

No. Lay.	Dist.LE [cm]	XA [m]	σ_p [N/mm²]	Eq.	l_{bpd} [m]	xk [cm²]	ΣZ_p [kN]	ΣZ_s [kN]	T_{Ed} [kN]	add. As [cm²]	
1	7.5	0.00	693.42	8.21	2.37	0.20	92.5	176.8	273.7	0.1	(PT)
2	11.7	0.00	706.41	8.21	2.35	0.20	92.5	176.8	273.7	0.1	(PT)
3	15.9	0.00	719.33	8.21	2.34	0.20	92.5	176.8	273.7	0.1	(PT)

XA: Beginning of the anchoring area of the steel layer (dist. from the corresp. binder side)
Eq. 8.21.1: Anchorage area uncracked, σ_p acc.to fig. 8.17DE (b)
Eq. 8.21.1: Anchorage area cracked, σ_p acc.to fig. 8.17DE (b)
xk: decisive section in the anchoring area of the layer (distance from the beginning of the binder)
add. As: Additional sagging reinforcement required for anchorage

Anchorage by bond (over the right bearing)

l_{pt2} = 1.14 m   Distance first bending crack l_r = 1.78 m
$f_{ctd}(t)$ = 1.02 N/mm²   t= release anchorage
f_{bpt} = 2.91 N/mm²
η_{P2} = 1.40 f_{bpd} = 1.61 N/mm²

x [m]	Z_p [kN]	Z_s [kN]	T_{Ed} [kN]	η= $(Z_p+Z_s)/T_{Ed}$	Util
12.24	1045.1	176.5	808.4	1.51	0.66
12.43	1045.0	176.5	778.8	1.57	0.64
12.63	1044.9	176.5	744.4	1.64	0.61
12.83	1044.8	176.5	708.0	1.73	0.58
13.03	947.9	176.5	670.5	1.68	0.60
13.22	850.9	176.5	633.6	1.62	0.62
13.42	748.8	176.5	593.4	1.56	0.64
13.62	646.7	176.4	551.9	1.49	0.67
13.82	544.6	176.4	509.0	1.42	0.71
14.01	455.2	176.4	466.8	1.35	0.74
14.21	363.3	176.5	421.1	1.28	0.78
14.41	271.3	176.6	374.0	1.20	0.84
14.61	179.3	176.7	325.8	1.09	0.92
14.80	92.5	176.8	273.7	0.98	1.02

Z_p: resisting tensile force by the prestressed steel
Z_s: resisting tensile force by the rebars
T_{Ed}: tensile force to be anchored

No. Lay.	Dist.LE [cm]	XA [m]	σp [N/mm²]	Eq.	l_{bpd} [m]	xk [cm²]	ΣZp [kN]	ΣZs [kN]	T_{Ed} [kN]	add. As [cm²]	
1	7.5	0.00	693.42	8.21	2.37	14.80	92.5	176.8	273.7	0.1	(PT)
2	11.7	0.00	706.41	8.21	2.35	14.80	92.5	176.8	273.7	0.1	(PT)
3	15.9	0.00	719.33	8.21	2.34	14.80	92.5	176.8	273.7	0.1	(PT)

XA: Beginning of the anchoring area of the steel layer (dist. from the corresp. binder side)
Eq. 8.21.1: Anchorage area uncracked, σp acc.to fig. 8.17DE (b)
Eq. 8.21.1: Anchorage area cracked, σp acc.to fig. 8.17DE (b)
xk: decisive section in the anchoring area of the layer (distance from the beginning of the binder)
add. As: Additional sagging reinforcement required for anchorage

Bursting Reinforcement at beginning of beam

$\gamma_{p,unfav}$ = 1.35 l_{disp} = 1.26 m

	Initiation zone		Section over the last effective position of tension. member					
No.	from [m]	to [m]	Dist.LE [cm]	N_c [kN]	N_p [kN]	T_p [kN]	Factor Interpolation	req. As [cm²]
1	0.00	1.26	16.9	-369.9	699.3	329.4	0.376	3.8

The bursting reinforcement must be arranged in zone of reduced dispersion length.
red. dispersion length indented wire w.o. strand 3/4*l $_{disp}$=0.95 m

Burstring Reinforcement at end of beam

$\gamma_{p,unfav}$ = 1.35 l_{disp} = 1.26 m

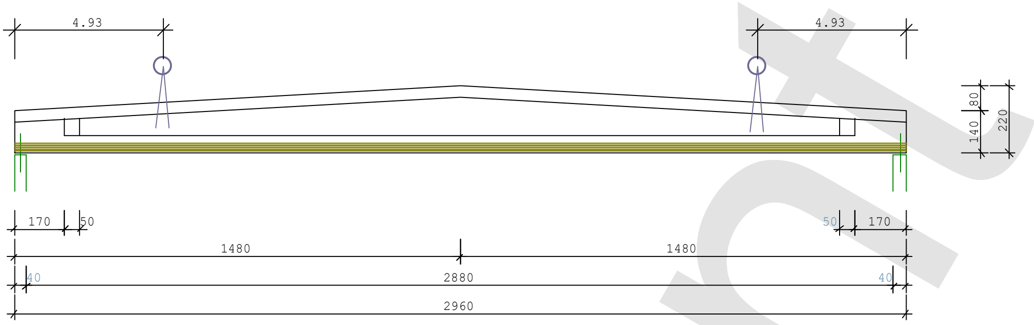
	Initiation zone		Section over the last effective position of tension. member					
No.	from [m]	to [m]	Dist.LE [cm]	N_c [kN]	N_p [kN]	T_p [kN]	Factor Interpolation	req. As [cm²]
1	15.00	13.74	16.9	-369.9	699.3	329.4	0.376	3.8

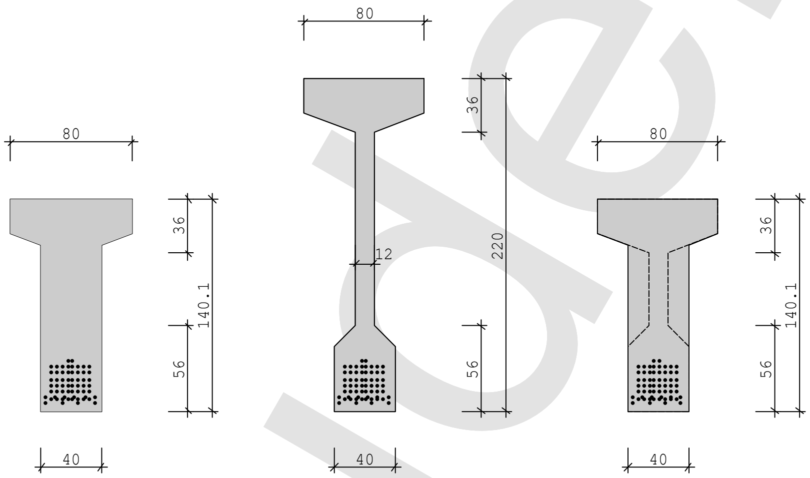
The bursting reinforcement must be arranged in zone of reduced dispersion length.
red. dispersion length indented wire w.o. strand 3/4*l $_{disp}$=0.95 m

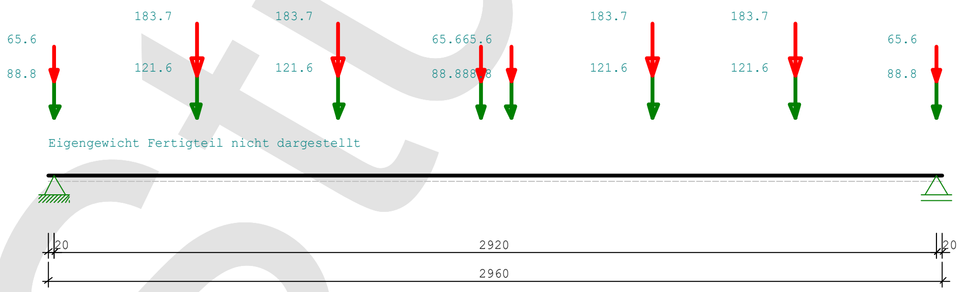
B.1.2. Design Calculations POS. P.03 – Prestressed Concrete Truss

Position: P.03 - Prestressed Concrete Truss

Spannbettbinder B8 01/2018 (Frilo R-2018-1/P12)







Advices:

No continuous reinforcement to 10 cm below UE found!

System:

Double-pitch roof

Basics:

Load combinatorics: DIN EN 1990/NA:2010-12 + EN 1990:2002/AC:2010
ULS: Structural safety checks(STR)
permanent/variable design situation with equation 6.10

Design code: DIN EN 1992-1-1/NA C1:2012-06 + EN 1992-1-1:2004 /AC:2010
Prestressing for pretensioning

System Geometry:

Total L = 29.60 m Effective L1 = 29.20 m
Outstand left L0 = 0.20 m right L2 = 0.20 m
Distance Ridge L3 = 14.80 m
Height beam :
left H1 = 140.0 cm Ridge H2 = 220.0 cm
right H3 = 140.0 cm
Relation eff.span to height of beam:
L1/H2 = 13.27

Erection attachment, distance from the beginning resp. end of beam:
Hook L8 = 4.93 m right L9 = 4.93 m

Cross-section Precast :

Layer of cross-section from top to bottom			
Nr	Width [cm]	Distance [cm]	Remarks
1	80.0	0.0	
2	80.0	22.4	
3	12.0	36.0	Web begin
4	12.0	164.0	Web end
5	40.0	178.0	
6	40.0	220.0	
Web height over beam length constant			

Bearing strengthening:

	Length [cm]	Haunch [cm]	Width [cm]
left	170.0	50.0	40.0
right	170.0	50.0	40.0

Material:

Prestressing steel

SpSt 1500/1770 Strand 7 wires
d_d = 4.1 mm d_p = 12.4 mm
E_p = 195000 N/mm² A_p = 0.934 cm²
$f_{p0.1k}$ = 1500 N/mm² f_{pk} = 1770 N/mm²
ε_{uk} = 35.0 ‰ ε_{ud} = 25.0 ‰ + $\varepsilon_p^{(0)}$

Partial safety factor :
γ_s = 1.15

Coeff. prestress:
charact. value r_{sup} = 1.05 r_{inf} = 0.95
Design value $\gamma_{p,max}$ = 1.00 $\gamma_{p,min}$ = 1.00

Proof of crack width
Equ. diameter d_{pv} = 7.21 mm ξ = 0.60 (Tab. 6.2)

Relaxation (from approval)				
σ_{p0}/f_{pk}	10 h	200 h	1000 h	1000000 h
0.60	0.3	0.6	0.8	2.8
0.70	0.8	1.6	2.0	7.0
0.80	2.0	4.0	5.0	14.0
Losses in % as a function of time and stress				

Permitted stresses:
in formwork σ_p ≤ 1350.0 N/mm² (0.90* $f_{p0.1k}$)
after anchor. release σ_p ≤ 1275.0 N/mm² (0.85* $f_{p0.1k}$)
char. Lc σ_p ≤ 1350.0 N/mm² (0.90* $f_{p0.1k}$)
q.perm.Lc σ_p ≤ 1150.5 N/mm² (0.65* f_{pk})

Transmission length

	η_{p1} = 2.85		η_1 = 1.00
	α_1 = 1.25		α_2 = 0.19
	σ_{pm0} = 917 N/mm²		
PT:	f_{ctdt} = 1.36 N/mm²		f_{bpt} = 3.86 N/mm²
	l_{pt} = 0.70 m		

Dispersion_length:
d = 1.34 m l_{disp} = 1.51 m

Reinforcing steel:

	Longitudinal		Stirrup
	B 500 B		B 500 B
E_s =	200000 N/mm²	E_s =	200000 N/mm²
f_{yk} =	500 N/mm²	f_{yk} =	500 N/mm²
f_{tk} =	540 N/mm²	f_{tk} =	540 N/mm²
ε_{uk} =	50.0 ‰	ε_{uk} =	50.0 ‰
ε_{ud} =	25.0 ‰	ε_{ud} =	25.0 ‰

Partial safety factor :
γ_s = 1.15 γ_s = 1.15

permitted stresses in SLS :
$\sigma_s \leq$ 400 N/mm² $\sigma_s \leq$ 400 N/mm²   (0.80 * f_{yk})

Requirements durability:

		top			bottom	
attack on concrete		W0			W0	
attack on reinforc.		XC1			XC1	
min. concrete class		C 16/20			C 16/20	
stirrup	ϕ,l =	10 mm				
long. reinforcement	ϕ,m =	16 mm		ϕ,m =	20 mm	
prestressed steel	d_p =	12.4 mm	strand ,$\sigma_p^{(0)}$ <=1000 N/mm2			
allowance in design	Δc_{dev} =	10 mm		Δc_{dev} =	10 mm	
stirrup	$c_{min,l}$ =	10 mm	*5	$c_{min,l}$ =	10 mm	*5 *5
concrete coverage	$c_{nom,l}$ =	20 mm	*5 *5	$c_{nom,l}$ =	20 mm	*5 *5
longitudinal bars	$c_{min,m}$ =	16 mm	*5	$c_{min,m}$ =	20 mm	*5
concrete coverage	$c_{nom,m}$ =	30 mm	*1	$c_{nom,m}$ =	30 mm	
prestressing steel :	$c_{min,p}$ =	31 mm	*5	$c_{min,p}$ =	31 mm	*5
concrete coverage	$c_{nom,p}$ =	41 mm		$c_{nom,p}$ =	41 mm	
laying dist. link	$c_{,l}$ =	20 mm		$c_{,l}$ =	20 mm	
all. crack width	w_{max} =	0.20 mm		w_{max} =	0.20 mm	
decompression		not req.			not req.	

*1:with cmin,l
*5: bond decisive

Concrete:

Precast

	C 50/60	
f_{ck} =	50.00 N/mm²	
α_{cc} =	0.85	
$f_{ctk0.05}$ =	2.85 N/mm²	
α_{ct} =	0.85	
γ =	25.00 kN/m³	Unit
E_{cm} =	37000 N/mm²	
α_E =	1.00	Coeff. E-module
G_{cm} =	14800 N/mm²	

Partial safety factor :
γ_c = 1.50

permitted stresses in SLS :

char. Lc	$\sigma_c \geq$	-30.00 N/mm²	
q.perm.Lc	$\sigma_c \geq$	-22.50 N/mm²	
Removal the anchor t= t0T(sto)			
	$f_{cm(t)}$ =	38.43 N/mm²	
	$f_{ck(t)}$ =	30.43 N/mm²	
linear creep	$\sigma_c \geq$	-13.69 N/mm²	(k2=0.45)
maximum	$\sigma_c \geq$	-18.26 N/mm²	(k6=0.60)

Creep modulus & shrinkage strain

no heat treatment, t0T = t0
CementStrenght class 42,5R;52,5
ρ= 0.5 (Aging coefficient)
Reference point for t0 is the start of the concreting of the precast

Creep	t0 Days	RH %
Storage	3	70
Utilization precast	180	50

L.	Segment	Part-cross-section	t0	t	α	$t_{0,eff}$ B.9	β_{t0} B.5	β_H B.8	$\beta_{c(t,t0)}$ B.7	ϕ_{RH} B.3	β_{fcm} B.4	φ(t,t0) B.1
1	Storage	PcC	3.0	180.0	1	7.7	0.62	498.2	0.67	1.23	2.21	1.13
2	Utilization precast	PcC	180.0	26000.0	1	7.7	0.62	485.6	0.32	1.45	2.21	0.64

L.	A [cm²]	U [cm]	h0 [cm]	β_{ds}(t0,ts)	β_{ds}(t,ts) 3.10	β_{RH} B.12	$\varepsilon_{cd,0}$ B.11	β_{as} 3.13	ε_{ca}/10e6 3.12	ε_{cs}(t,t0) [‰]
1	5997.60	617.6	194.2	0.000	0.620	1.02	402.5	0.93	100.00	0.308
2	5997.60	617.6	194.2	0.620	0.996	1.36	536.0	1.00	100.00	0.180

Loads:

Self weight

Beam beginning	g11	=	16.65	kN/m
Support reinforcement end left	g12	=	17.56	kN/m
Support reinforcement haunch left	g13	=	12.95	kN/m
Ridge	g14	=	14.99	kN/m
Support reinforcement haunch right	g15	=	12.95	kN/m
Support reinforcement end right	g16	=	17.56	kN/m
Beam end	g17	=	16.65	kN/m

Total	G	=	425.4	kN
Volume	V	=	17.01	m³
Surf.	A	=	137.19	m²

Live loads

Units: Single load[kN] Single moment[kNm] line load[kN/m]

span	type	gle	qle	Dist. a [m]	gri	qri	Length [m]	Fact	Act.	Sim.	Pos.
1	2	88.80	65.60	0.00				1.00	10	0	
1	2	88.80	65.60	14.10				1.00	10	0	
1	2	88.80	65.60	15.10				1.00	10	0	
1	2	88.80	65.60	29.20				1.00	10	0	
1	2	121.60	183.70	4.70				1.00	10	0	
1	2	121.60	183.70	9.40				1.00	10	0	
1	2	121.60	183.70	19.80				1.00	10	0	
1	2	121.60	183.70	24.50				1.00	10	0	

Load types: 1 = uniformly distr., 2 = single load at a, 3 = single moment at a
4 = trapezoidal load from a, 5 = triangle load over L

Actions:

Act.	γ_Q	ψ_0	ψ_1	ψ_2	Dep.	Cat.	Description
10	1.50	0.50	0.20	0.00	0	S	Schnee bis NN +1000m

Tendons:

Dist(LE) > 5.4 cm axis horizontal > 3.7 cm vertical > 3.7 cm

lay. No.	num- ber	area Ap [cm²]	Dist.LE Yp [cm]	Prestressing $\sigma_p^{(0)}$ [N/mm²]	<--- Isolations ---> Count	to x1 [m]	from x2 [m]	Type ||
1	8	7.47	7.5	1000	0			LE
2	8	7.47	11.7	1000	0			LE
3	8	7.47	15.9	1000	0			LE
4	8	7.47	20.1	1000	0			LE
5	8	7.47	24.3	1000	0			LE
6	8	7.47	28.5	1000	0			LE
7	2	1.87	32.7	1000	0			LE

x1 and x2 with respect to the left beginning from joint
LE= parallel lower edge, UE= parallel upper edge
The calculation of the losses due to creep, shrinkage and relaxation
following the method from Abelein

Untensioned reinforcement:

Layer No.	num- ber	diam. Φs,l [mm]	area As [cm²]	Dist.LE Ys [cm]	effective range from xA [m]	to xE [m]	Type ||
1	4	20	12.57	4.0	0.00	29.60	LE
2	6	16	12.06	8.0	0.00	29.60	LE

xA and xE with respect to the left beginning from joint
LE= parallel lower edge, UE= parallel upper edge

Min. reinforcement width of crack not required (user defined)

Surface reinforcement acc.to Tab. NA.J.41 (B0 < D0) :

Web (Z1/S3) AsS = 0.78 cm²/m (UwkS <= XC4) (per side)
Top flange (Z3/S1) AsO = 0.00 cm²/m (UwkS <= XC4)

Settings for shear resistance check

Bearing width, distance bearing edge, effective height of the bearing line
left bAl = 0.40 m al = 0.20 m dAl = 1.35 m
right bAr = 0.40 m ar = 0.20 m dAR = 1.35 m
For shear reinforcement not decisive ranges over support A and B:
xaRe=1.55 m direct bearing (width of bearing/2 + eff. depth)
xbLi=1.55 m direct bearing (width of bearing/2 + eff. depth)

Check the limit deformation:

Total sagging f ≤ L/ 250 Increase deflection |df| ≤ L/ 500
Cantilever left f ≤ 0.2 cm |df| ≤ 0.1 cm
Span f ≤ 11.7 cm |df| ≤ 5.8 cm
Cantilever right f ≤ 0.2 cm |df| ≤ 0.1 cm
quasi- permanent combination and eff. char. prestress
Deflection due to shrinkage considered
Tension stiffening: Member rigidity, Characteristic combination

RESULTS (summary)

Reaction forces (t = infinitely):

Units: all [kN] G:perm., Q:variable. ,V: Sum

Support point	<------char. value-----> G	min Q	max Q	<--ULS(PT)----> min V	max V
A (left)	633.57	0.00	498.60	633.57	1603.23
B (right)	633.57	0.00	433.00	633.57	1603.23

max. bending moment in erection state(char. value):

MF = = 8002.46 kNm at x = = 14.80 m

Checks are not complied with:

Checkvalue		Extrem		Utilisation	x [m]
Prc.:Compr.stress t0(sto)	σc =	-41.26	N/mm²	2.26	4.93

Warning

Prc.:lin. creep t0(Sto) σc =-30.13 N/mm2 x= 28.96 m
σc <0.45*fck(t)=-13.69 N/mm2
_disproportional creeping by increased creep modulus considered(fk= 2.25)

Required shear reinforcement:

Column A: asw = 14.61 cm^2/m
Column B: asw = 14.61 cm^2/m

Bursting reinforcement

left	Laying length	=	1.13	m					
	from x	=	0.00	m	As	=	21.0	cm^2
right	Laying length	=	1.13	m					
	from x	=	29.60	m	As	=	21.0	cm^2

Check of anchorage

left: Tensile force resistance in anchoring area Util = 0.51
right: Tensile force resistance in anchoring area Util = 0.51

Overview crit. sections

Selected basic grid: 10 Sections

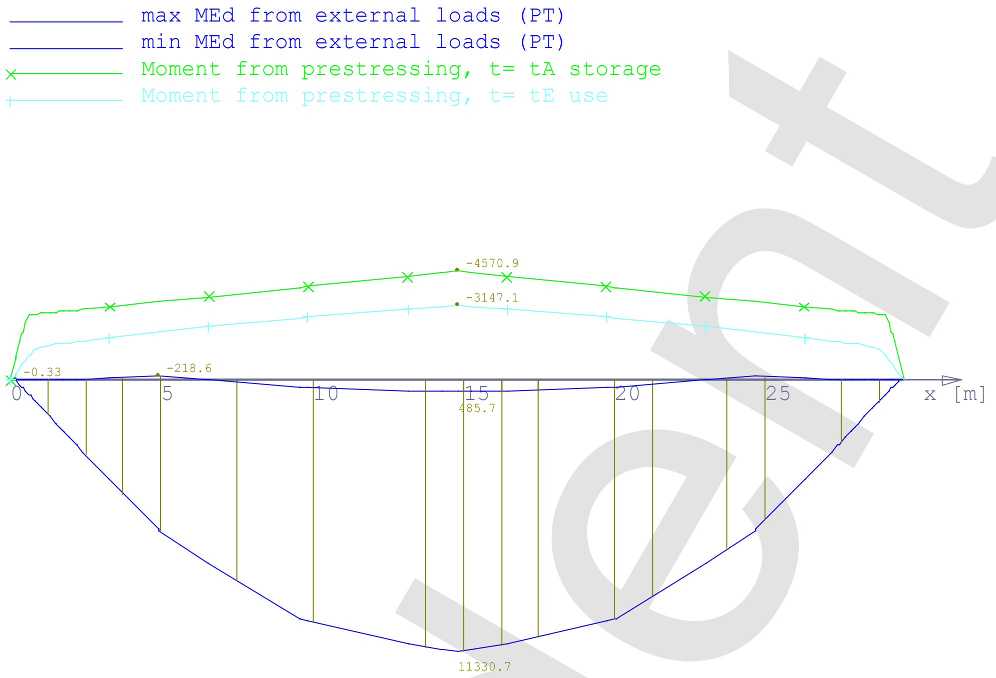
Checkvalue		Extrem		Utilisation	x [m]	
Flexural capacity bottom	η =	1.15		0.87	9.60	
Flexural capacity top	η =	****		****	0.50	
Resisting tens force bot	η =	1.15		0.87	9.56	
Resisting tens force top	η =	****		****	0.50	
Prc.:Compr.stress t0(sto)	σc =	-41.26	N/mm^2	2.26	4.93	!
Prc.:Compr.stress Cc	σc =	-20.54	N/mm^2	0.68	4.93	
Tension prestress. steel	σp,Qc =	936.2	N/mm^2	0.81	1.70	
Stress in prestress.steel	σp,Cc =	1003.9	N/mm^2	0.74	9.60	
Stress in rebars	σs =	35.5	N/mm^2	0.09	9.60	
Crack MinAs+AsDuc bottom	AsMin =	13.5	cm^2	0.34	14.80	
Crack MinAs+AsDuc top	AsMin =	----	cm^2	----	----	
Crack width bottom	wk =	0.00	mm	0.01	9.60	
Crack width top	wk =	****	mm	****	0.33	
Sagging top	fo =	-3.4	cm	0.29	16.44	
Sagging bottom	fu =	0.6	cm	0.05	13.16	
Incr.-deflection(Util)	|df|=	2.0	cm	0.35	13.16	
Prc.:Shear reinf (web)	asw =	14.61	cm^2/m	1.00	2.20	
Concrete strut capacity	η =	1.13		0.89	2.20	

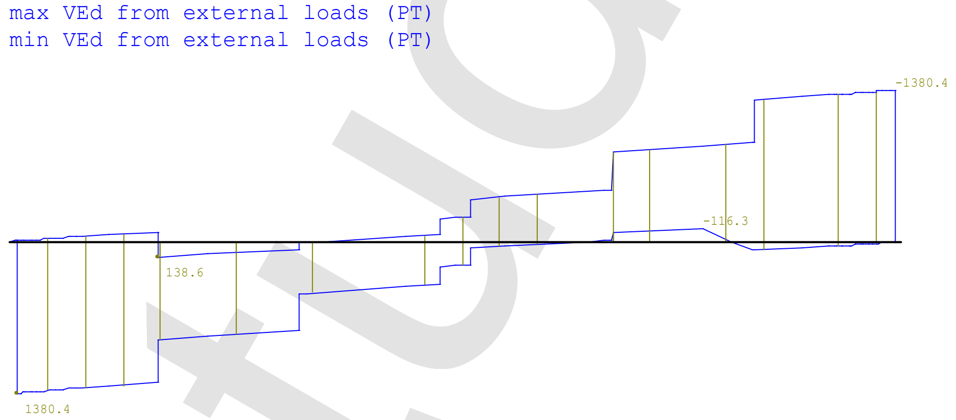
---- Check not required
**** Check not fulfilled
Prc.:Precast member Add.: in-situ supplement
IS : Installed state SC : State of construction
AsDuk:Ductility reinforcement
Min. reinforcement width of crack not required (user defined)

Linear creep limit, informative:		Extrem		Utilisation	x [m]
Prc.:lin. creep t0(Sto)	σc =	-30.13	N/mm^2	2.20	28.96
Prc.:Compression quasi-permanent Lc	σc =	-14.35	N/mm^2	0.64	27.20

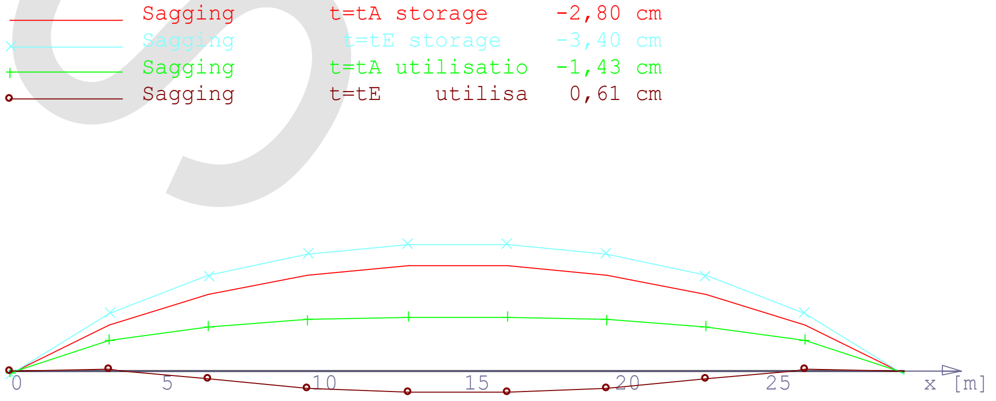
Tensile stress state I, informative:		Extrem		Utilisation	x [m]
Prc.:Tens.stress (IS)	σt =	10.80	N/mm^2	2.65	9.60
Prc.:Tens.stress (SC)	σt =	6.01	N/mm^2	2.38	28.96

Internal forces





Deformation



Anchorage by bond (over the left bearing)

l_{pt2} = 0.84 m Distance first bending crack l_r = 1.97 m
$f_{ctd}(t)$ = 1.36 N/mm² t= release anchorage
f_{bpt} = 3.86 N/mm²
η_{P2} = 1.40 f_{bpd} = 2.26 N/mm²

x [m]	Z_p [kN]	Z_s [kN]	T_{Ed} [kN]	η= $(Z_p+Z_s)/T_{Ed}$	Util
0.20	1727.6	1078.7	1440.7	1.95	0.51
0.33	2834.9	1077.6	1597.6	2.45	0.41
0.66	5669.9	1076.3	1825.6	3.70	0.27
0.99	6233.4	1076.3	2174.4	3.36	0.30
1.32	6237.2	1076.5	2484.4	2.94	0.34
1.64	6258.3	1077.3	2810.0	2.61	0.38
1.97	6223.2	1075.9	3144.0	2.32	0.43

Z_p: resisting tensile force by the prestressed steel
Z_s: resisting tensile force by the rebars
T_{Ed}: tensile force to be anchored

No. Lay.	Dist.LE [cm]	XA [m]	σ_p [N/mm²]	Eq.	l_{bpd} [m]	xk [cm²]	ΣZ_p [kN]	ΣZ_s [kN]	T_{Ed} [kN]	add. As [cm²]	
1	7.5	0.00	621.60	8.21	1.79	Anchorage range uncracked					(PT)
2	11.7	0.00	632.42	8.21	1.78	Anchorage range uncracked					(PT)
3	15.9	0.00	643.25	8.21	1.77	Anchorage range uncracked					(PT)
4	20.1	0.00	654.07	8.21	1.76	Anchorage range uncracked					(PT)
5	24.3	0.00	664.90	8.21	1.75	Anchorage range uncracked					(PT)
6	28.5	0.00	675.72	8.21	1.73	Anchorage range uncracked					(PT)
7	32.7	0.00	686.55	8.21	1.82	Anchorage range uncracked					(PT)

XA: Beginning of the anchoring area of the steel layer (dist. from the corresp. binder side)
Eq. 8.21.1: Anchorage area uncracked, σp acc.to fig. 8.17DE (b)
Eq. 8.21.1: Anchorage area cracked, σp acc.to fig. 8.17DE (b)
xk: decisive section in the anchoring area of the layer (distance from the beginning of the binder)
add. As: Additional sagging reinforcement required for anchorage

Anchorage by bond (over the right bearing)

l_{pt2} = 0.84 m Distance first bending crack l_r = 1.97 m
$f_{ctd}(t)$ = 1.36 N/mm² t= release anchorage
f_{bpt} = 3.86 N/mm²
η_{P2} = 1.40 f_{bpd} = 2.26 N/mm²

x [m]	Z_p [kN]	Z_s [kN]	T_{Ed} [kN]	η= $(Z_p+Z_s)/T_{Ed}$	Util
27.63	6223.2	1075.9	3144.0	2.32	0.43
27.96	6258.3	1077.3	2810.0	2.61	0.38
28.28	6237.2	1076.5	2484.4	2.94	0.34
28.61	6233.4	1076.3	2174.4	3.36	0.30
28.94	5669.9	1076.3	1825.6	3.70	0.27
29.27	2834.9	1077.6	1597.6	2.45	0.41
29.40	1727.6	1078.7	1440.7	1.95	0.51

Z_p: resisting tensile force by the prestressed steel
Z_s: resisting tensile force by the rebars
T_{Ed}: tensile force to be anchored

No. Lay.	Dist.LE [cm]	XA [m]	σ_p [N/mm²]	Eq.	l_{bpd} [m]	xk [cm²]	ΣZ_p [kN]	ΣZ_s [kN]	T_{Ed} [kN]	add. As [cm²]	
1	7.5	0.00	621.60	8.21	1.79	Anchorage range uncracked					(PT)
2	11.7	0.00	632.42	8.21	1.78	Anchorage range uncracked					(PT)
3	15.9	0.00	643.25	8.21	1.77	Anchorage range uncracked					(PT)
4	20.1	0.00	654.07	8.21	1.76	Anchorage range uncracked					(PT)
5	24.3	0.00	664.90	8.21	1.75	Anchorage range uncracked					(PT)
6	28.5	0.00	675.72	8.21	1.73	Anchorage range uncracked					(PT)
7	32.7	0.00	686.55	8.21	1.82	Anchorage range uncracked					(PT)

XA: Beginning of the anchoring area of the steel layer (dist. from the corresp. binder side)
Eq. 8.21.1: Anchorage area uncracked, σp acc.to fig. 8.17DE (b)
Eq. 8.21.1: Anchorage area cracked, σp acc.to fig. 8.17DE (b)
xk: decisive section in the anchoring area of the layer (distance from the beginning of the binder)
add. As: Additional sagging reinforcement required for anchorage

Bursting Reinforcement at beginning of beam

$\gamma_{p,unfav}$ = 1.35 l_{disp} = 1.51 m

	Initiation zone		Section over the last effective position of tension. member					
No.	from [m]	to [m]	Dist.LE [cm]	N_c [kN]	N_p [kN]	T_p [kN]	Factor Interpolation	req. As [cm²]
1	0.00	1.51	33.7	-2434.6	4221.8	1787.2	0.378	21.0

The bursting reinforcement must be arranged in zone of reduced dispersion length.
red. dispersion length indented wire w.o. strand 3/4*l_{disp}=1.13 m

Burstring Reinforcement at end of beam

$\gamma_{p,unfav}$ = 1.35 l_{disp} = 1.51 m

	Initiation zone		Section over the last effective position of tension. member					
No.	from [m]	to [m]	Dist.LE [cm]	N_c [kN]	N_p [kN]	T_p [kN]	Factor Interpolation	req. As [cm²]
1	29.60	28.09	33.7	-2434.6	4221.8	1787.2	0.378	21.0

The bursting reinforcement must be arranged in zone of reduced dispersion length.
red. dispersion length indented wire w.o. strand 3/4*l_{disp}=1.13 m

B.1.3. Design Calculations POS. T.10 – Precast Staircase

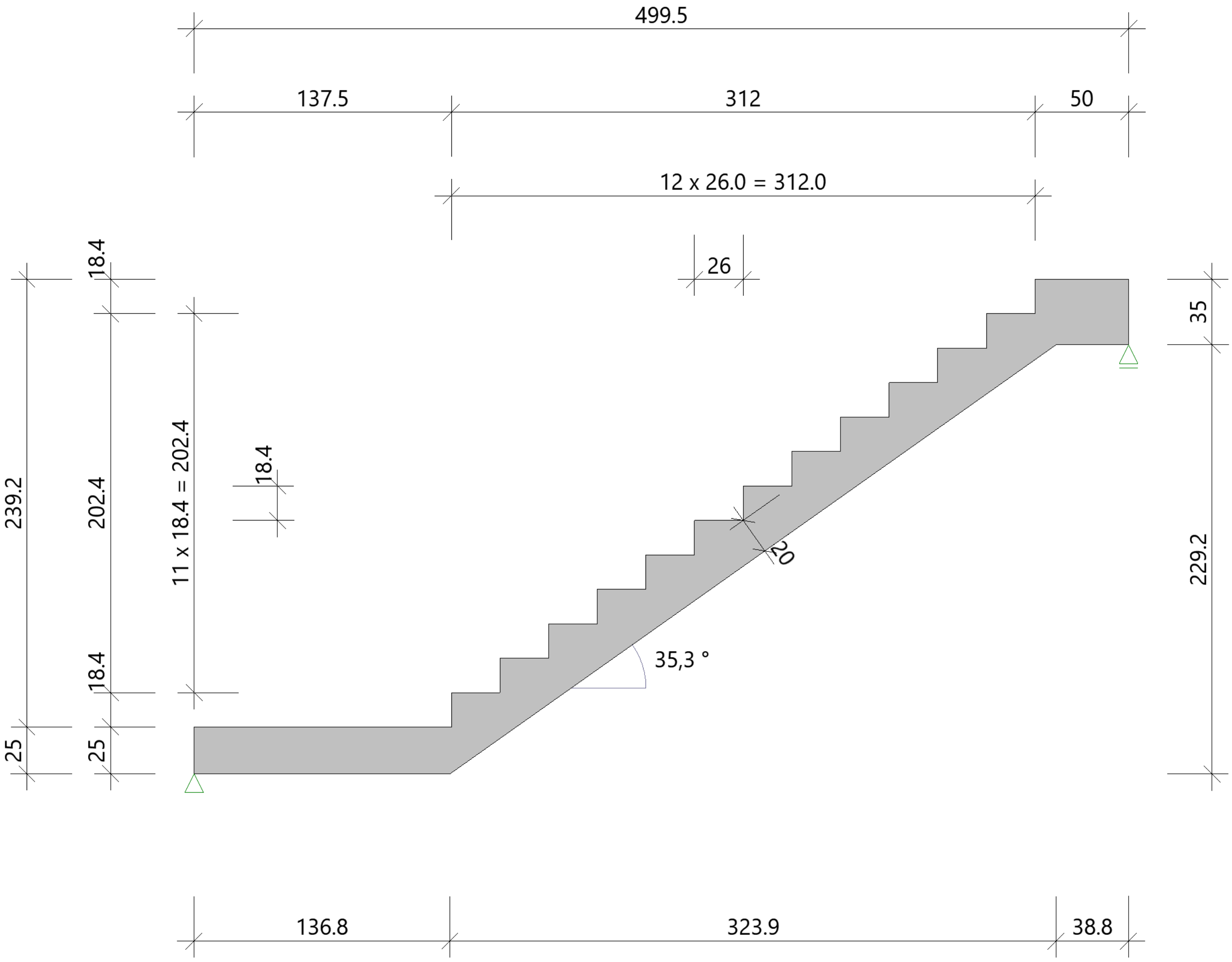
Position: T.10 - Precast Staircase

Treppenlauf (neu) B7+ 01/18A (Frilo R-2018-1/P12)

System

System graphics

Scale 1 : 38.1



Geometry

Rfb floor landing top - Rfb floor landing bottom	H_1	=	239.2	cm
Length from 1-st to to last step tread	L_1	=	312.0	cm
Length lower landing to FE support	L_2	=	137.5	cm
Length upper landing to FE support	L_3	=	50.0	cm
width of flight	B_1	=	100.0	cm
Width of cover	B_2	=	100.0	cm
Live load width	B_3	=	100.0	cm
Number of rises	n_s	=	13	
Height of step bottom	H_u	=	18.4	cm
Height of step top	H_o	=	18.4	cm
Stair steps	H_s / L_s	=	18.4 / 26.0	cm
Thickness of staircase	D_1	=	20.0	cm
Thickness lower landing	D_2	=	25.0	cm
Thickness upper landing	D_3	=	35.0	cm
Length of the staircase bottom view in plan	L_4	=	323.9	cm
distance 1st step to inflexion point bottom	L_5	=	-0.7	cm

bearing

bttm: hinged without console
top: hinged without console

Support

Location -	horizontal kN/m	vertical kN/m	turning kNm/rad
left right	rigid free	rigid rigid	free free

Durability

Requirements durability

attack on concrete			X0		
attack on reinforc.			XC1		
min. concrete class			C 16/20		
long. reinforcement	$\phi_{,m}$	=	10	mm	
allowance in design	Δc_{dev}	=	10	mm	
longitudinal bars	$c_{min,m}$	=	10	mm	*5
concrete coverage	$c_{nom,m}$	=	20	mm	
laying dist. link	$c_{,l}$	=	20	mm	
all. crack width	w_{max}	=	0.40	mm	

*5: bond decisive

Loads

safety and combination factors

Action group	γ_G	γ_Q	ψ_0	ψ_1	ψ_2
Kat. B: Bürogebäude	1,35	1,5	0,7	0,5	0,3

Load

Location -	Type -	g kN/m²	q kN/m²
Stairway	Covering Live load	1.50 -	- 3.00
Landing/console bottom	Covering Live load	1.50 -	- 3.00
Landing/console top	Covering Live load	1.50 -	- 3.00

Resulting loading (relative to the horizontal surface)

Location -	Type -	g kN/m²	q kN/m²
Stairway	Self weight Covering Live load Total	8.43 1.50 - 9.93	- - 3.00 3.00
Landing/console bottom	Self weight Covering Live load Total	6.25 1.50 - 7.75	- - 3.00 3.00
Landing/console top	Self weight Covering Live load Total	8.75 1.50 - 10.25	- - 3.00 3.00

The dead weight is with gamma = 25.00 kN/m^3 considered.

Standard, Materials und Reinforcement layer

Design acc.to DIN EN 1992-1-1/NA/A1:2015-12

Construction materials: Concrete C30/37, γ_c = 1.50, f_{ck} = 30.0 N/mm² ; Steel B500A, γ_s = 1.15, f_{yk} = 500.0 N/mm²

Individual lengths

	Lower landing	Stairway	Upper landing
dimension	1.37 m	4.05 m (L_{tot}) 3.31 m (L_{hor}) 2.34 m (L_{vert})	0.31 m

Reinforcement layer bottom d_1 = 3.0 cm
Reinforcement upper layer d_2 = 3.0 cm

Results

Bending design

All design results per m stair width!

Flexural reinforcement

Location -	d cm	M_{Ed} kNm/m	N_{Ed} kN/m	req. a_{sb} cm²/m	req. a_{st} cm²/m	&About -
lower landing, span reinforcement	25.0	42.48	0.0	4.4	0.0	
Stairway, span reinforcement	20.0	54.46	0.3	7.5	0.0	
upper landing, span reinforcement	35.0	6.73	0.0	4.1	0.0	*)
*) Minimum longitudinal reinforcement is decisive						

exist. reinforcement

Stairway, span reinforcement 7 Ø 12 / 15.0 cm (Suggestion from program for number Ø)
exist. a_{sbtm} = 7.54 cm²/m

Shear design

Shear reinforcement B500A

Location -	V_{Ed} kN/m	N_{Ed} kN/m	k_z -	θ Degree	a_{sL} cm²/m	$V_{Rd,c}$ kN/m	$V_{Rd,max}$ kN/m	req. a_{sStir} cm²/m²	&About -
lower landing left	41.2	0.0	0.82	18.4	0.0	115.1	688.5	9.3	*)
lower landing right	20.7	0.0	0.82	18.4	4.4	115.1	688.5	9.3	*)
Stairway left	16.9	-12.0	0.76	18.4	5.6	93.4	497.3	0.0	
Stairway right	-31.4	22.3	0.76	18.4	2.9	89.9	497.3	0.0	
upper landing left	-38.5	0.0	0.88	18.4	4.1	147.0	1071.0	9.3	*)
upper landing right	-44.3	0.0	0.88	18.4	0.0	147.0	1071.0	9.3	*)
*) Minimum shear reinforcement is decisive									

crack width verification
The check is carried out with the quasi-permanent action combination

Crack width limitation stairs:

Location -	d cm	M_{Ed} kNm	N_{Ed} kN	exist. A_{sb} cm²	exist. A_{st} cm²	Env.Cl -	$d_{s,exist}$ mm	$d_{s,limit}$ mm	exist. w mm	perm. w mm
Stairway, bottom side	20.0	32.75	0.1	7.9	0.0	XC1	12	24	0.20	0.40

Deformation
The calculation will be done with quasi permanent Action combination at state I (E_{cm} = 33000 N/mm²).

max. f = 0.4 cm (in staircase at x = 1.45 m)
Note: The deflection value is to be understood perpendicular to the corresponding member axis. The x-value refers to the beginning of the member (beginning lower platform, staircase or upper platform) and runs in the direction of the member axis.

Support reactions

Definition supporting forces

(A) support left (v) vertical supporting force
(B) support right (v) horizontal supporting force

Support reactions per m stair width

	A_v kN/m	A_h kN/m	B_v kN/m	B_h kN/m
γ = 1.0				
Total	29.7	0.0	32.0	0.0
from g	22.2	0.0	24.5	0.0
from q	7.5	0.0	7.5	0.0
γ-times				
Total	41.2	0.0	44.3	0.0
from g	30.0	0.0	33.0	0.0
from q	11.2	0.0	11.2	0.0

Self weight of stairs

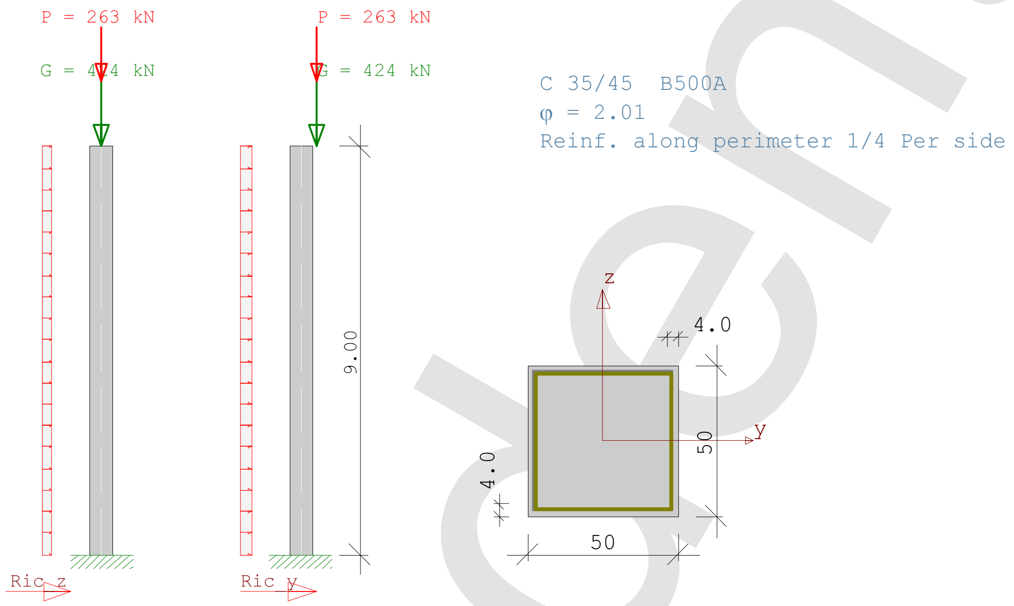
The self-weight of the stair (without covering) G_k is 39.2 kN

B.1.4. Design Calculations POS. C.04 – Precast Concrete Column with Biaxial Wind Loads

Position: C.04 - Precast Concrete Column

Reinforced Concrete Column B5 01/2018 (Frilo R-2018-1/P12)

CANTILEVER COLUMN, Rectangle, 2-axial strained
Calculation base: DIN EN 1992-1-1/NA/A1:2015-12 E = 34000 N/mm2 ρ = 2500 kg/m3



1 Mcry = 66.87 kNm Mcrz = 66.87 kNm

NODES - LOADS :

LcNo.	KNo.	V (kN)	ey (cm)	ez (cm)	Py (kN)	Pz (kN)	My (kNm)	Mz (kNm)	act	con	alt
1	2	423.60	35.0	.	.	.	.	.		.	g
		263.00	35.0	.	.	.	.	.	J	.	p
		56.25	(dead load)								

MEMBER - LOADS :

LcNo	MNo	type	dir	g1 (kN/m	g2 , kN)	dist (m)	length (m)	actGrp	con	alt
2	.	Uniform loa	z	1.46	1.46	.00	9.00	I	.	p
3	.	Uniform loa	y	2.51	2.51	.00	9.00	I	.	p

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ
I	4	Wind loads	0.60	0.20	0.00	1.50
J	3	Snow loads <1000m	0.50	0.20	0.00	1.50

All actions are use like independent.

Further design fundamentals:

Accuracy Gkn = 6.14e-5
Number of sub-element per member section: 6
Stress-strain-curve of concr. for deform. analy. EN 1992-1-1 3.1.5
Calc. of compr. force in concr.without deduction of reinf.
If n > -0.10 : eff EI acc.toEN2 7.4.2 (7.19)
Creep effects are considered by modified stress-strain-curve.
ϕeff = ϕ0 * M0/Med (M0 By permanent combination with ei)
consequency class acc. EN 1990 tab b.1CC2 -> KFi = 1.0 (Tab B.3)

FLBemBn.DLL: version9.0.1.121

NKi/N = 5.18 dir_y NKi/N = 5.18 dir_z cross sect. of concr. only

CALCULATED COMBINATIONS by 3 Loads Kombi_D

Lc- Comb	K1	K2	K3	K4	K5	K6	K7	K8
	g	g	g	g	g	g	g	g
	J	I	I	J	I	I		I
1	x	.	x	x	x	x	x	.
2	x	.	x	.	x	.	x	x
3	x	x	x	x	.	x	x	.

Partial safety factor γC = 1.50 γS = 1.15 γG = 1.35 / 1.00

Proof according DIN EN 1992-1-1/NA/A1:2015-12
γC = 1.50 γS = 1.15 ϕeff = 1.37

Design values	LcCom = 1 in :	y-direction	z-direction
System		Displacable	
Buckling length	sk =	18.00	18.00 m
Slenderness	λ =	124.6	124.6
Normal force	N =	-1042.30	-1042.30 kN
Specific normal force	n =	-.21	-.21
Inter. moment	h = .00 m , M =	-429.72	-53.22 kNm
Methodical eccentr.	e = M / N =	41.23	5.11 cm
Related eccentr.	e/b and e/d =	0.8246	0.1021
Unintentional eccentrici	ty ei =	3.00	3.00 cm
Displacement Th.2.Ord.	e2 =	37.11	0.18 cm
Design moment	M des. =	-847.80	-23.80 kNm
Reinforcement	tot ω =	.8827	
	ρ =	4.03 %	
	Req As =	100.66 cm2	

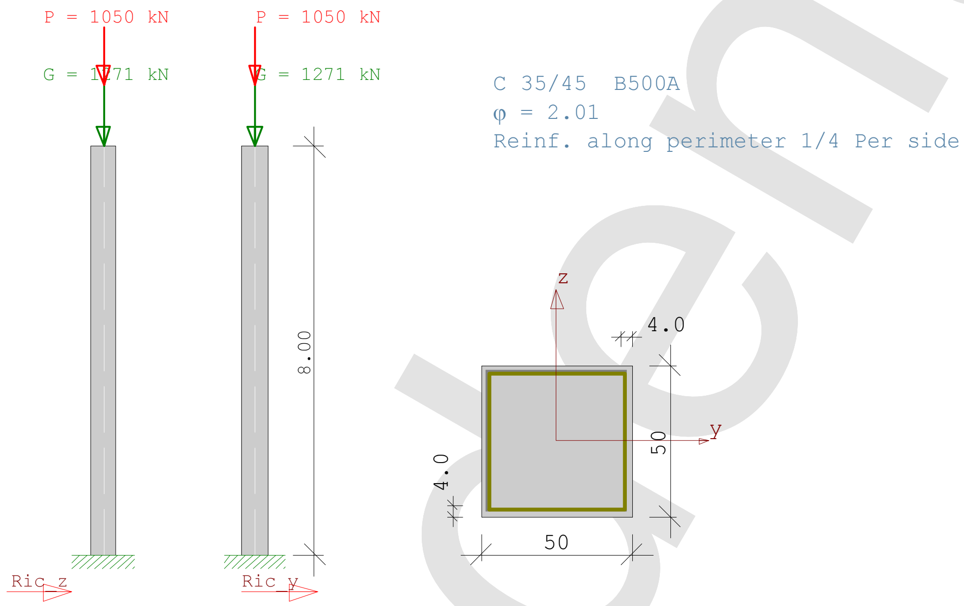
The influence of creep will considered acc. EN 1992-1-1 5.8.4 curve.

B.1.5. Design Calculations POS. C.05 – Precast Concrete Column

Position: C.05 - Precast Concrete Column

Reinforced Concrete Column B5 01/2018 (Frilo R-2018-1/P12)

CANTILEVER COLUMN, Rectangle, 2-axial strained
Calculation base: DIN EN 1992-1-1/NA/A1:2015-12 E = 34000 N/mm2 ρ = 2500 kg/m3



1 Mcry = 66.87 kNm Mcrz = 66.87 kNm

NODES - LOADS :

LcNo.	KNo.	V (kN)	ey (cm)	ez (cm)	Py (kN)	Pz (kN)	My (kNm)	Mz (kNm)	act	con	alt
1	2	1271.0	.	.	.	.	.	.		.	g
		1050.0	.	.	.	.	.	.	J	.	p
		50.00	(dead load)								

Actions: No.	Cl	Name	ψ0	ψ1	ψ2	γ
J	3	Snow loads <1000m	0.50	0.20	0.00	1.50

Further design fundamentals:

Accuracy Gkn = 8.48e-5
Number of sub-element per member section: 6
Stress-strain-curve of concr. for deform. analy. EN 1992-1-1 3.1.5
Calc. of compr. force in concr.without deduction of reinf.
If n > -0.10 : eff EI acc.toEN2 7.4.2 (7.19)
Creep effects are considered by modified stress-strain-curve.
ϕeff = ϕ0 * M0/Med (M0 By permanent combination with ei)
consequency class acc. EN 1990 tab b.1CC2 -> KFi = 1.0 (Tab B.3)

FLBemBn.DLL: version9.0.1.121
mininmum moments acc. to 6.1 (4) are not considered.

NKi/N = 2.03 dir_y NKi/N = 2.03 dir_z cross sect. of concr. only

CALCULATED COMBINATIONS by 1 Loads Kombi_D

Lc- Comb	K1	K2
	g J	g
1	x	.

Partial safety factor γC = 1.50 γS = 1.15 γG = 1.35 / 1.00

Proof according DIN EN 1992-1-1/NA/A1:2015-12
γC = 1.50 γS = 1.15 ϕeff = .80

Design values LcCom = 1 in : y-direction z-direction

System		Displacable	
Buckling length	sk =	16.00	16.00 m
Slenderness	λ =	110.7	110.7
Normal force	N =	-3358.35	-3358.35 kN
Specific normal force	n =	-.68	-.68
Inter. moment	h = .00 m , M =	0.00	0.00 kNm
Methodical eccentr.	e = M / N =	0.00	0.00 cm
Related eccentr.	e/b and e/d =	0.0000	0.0000
Unintentional eccentrici	ty ei =	2.83	2.83 cm
Displacement Th.2.Ord.	e2 =	10.06	10.06 cm
Design moment	M des. =	-432.74	-432.75 kNm
Reinforcement	tot ω =	.9541	
	ρ =	4.35 %	
	Req As =	108.81 cm2	

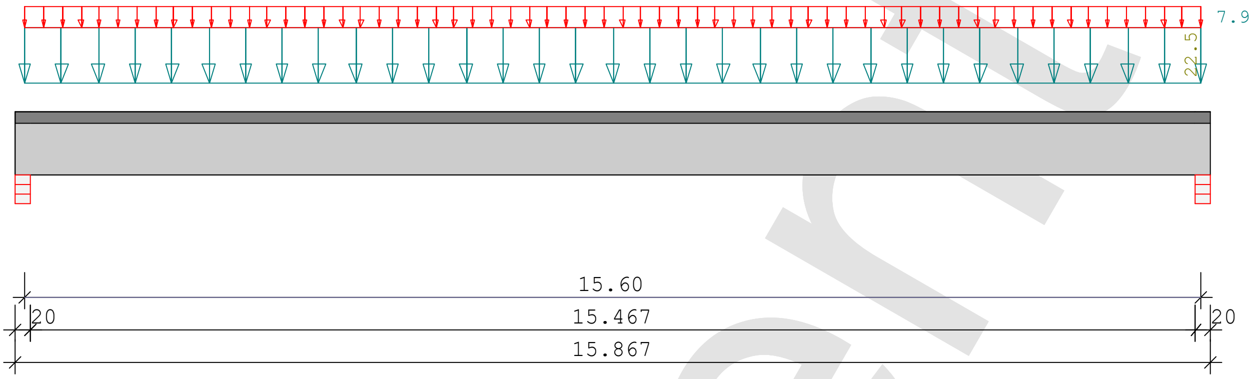
The influence of creep will considered acc. EN 1992-1-1 5.8.4 curve.

B.1.6. Design Calculations POS. S.11 – Semi-Precast Double-Tee Slab

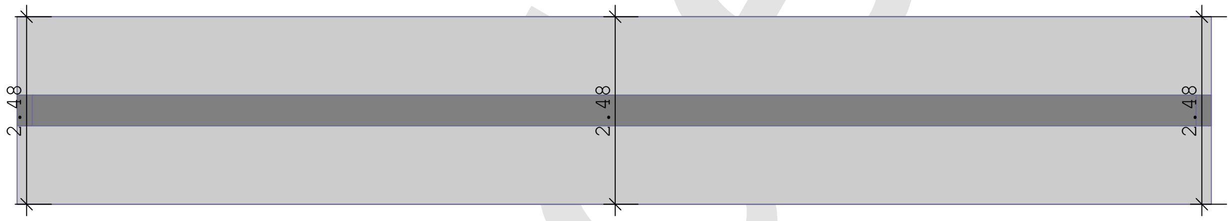
Position: S.11 - Semi-Precast Double-Tee Slab

Durchlaufträger DLT10 01/2018 (Frilo R-2018-1/P12)

Scale 1 : 100



effective breadth and design



Reinforced concrete girder C35/45 E = 34000 N/mm2 DIN EN 1992-1-1/NA/A1:2015-12								
System	length		cross-section values					
Span	l (m)		bt	ht	b0	h0	bb	hb
1	15.60	constant	248.0	15.0	40.0	85.0		

Load type (kN,m) : 1=uniform over L, 2=concentrated at a, 3=single moment at a, 4=trapezoidal btw. a, a+b, 5=triangular over L, 6=trapezoidal over L										
Span	Type	AG G	r g_l/r	q_l/r	factor	distanc	e length	from	Item	Phi
1	1	A	22.50	7.90	1.00					

Actions:					
No.	Cl Name	ψ0	ψ1	ψ2	γ
A	1 Cat A - domestic	0.70	0.50	0.30	1.50

Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3
In following tables the last cell the row is a reference to
the number of the related superposition (see below).
In tables with internal forces multiplied by Gamma is additionally
a reference to the main action.

Results for 1-times loads

Span moments maximum							(kNm , kN)
Span		Mf	M le	M ri	V le	V ri	comb
1	x0 = 7.80	924.77	0.00	0.00	237.12	-237.12	2

Support moments maximum							(kNm , kN)
Column	M le	M ri	V le	V ri	max F	min F	comb
1	0.00	0.00	0.00	237.12	237.12	175.50	2
2	0.00	0.00	-237.12	0.00	237.12	175.50	2

Moment boundary diagram											
x/L = .0 Span		.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
1	0.00	246	438	575	657	684	657	575	438	246	0.00
1	0.00	333	592	777	888	925	888	777	592	333	0.00

Support reactions						(kN)
Column	by g	max q	min q	Fulload	max	min
1	175.50	61.62	0.00	237.12	237.12	175.50
2	175.50	61.62	0.00	237.12	237.12	175.50
Total:	351.00	123.24	0.00	474.24	474.24	351.00

Support reactions				(kN)
	Column 1		Column 2	
CA	max	min	max	min
g	175.5	175.5	175.5	175.5
A	61.6	0.0	61.6	0.0
tot	237.1	175.5	237.1	175.5

Deflections calculated according to uncracked concrete!

Deflections	maximum			minimum		
Span No.	x (m)	f (cm)	comb	x (m)	f(cm)	comb
1	7.80	1.68	2	0.00	0.00	0

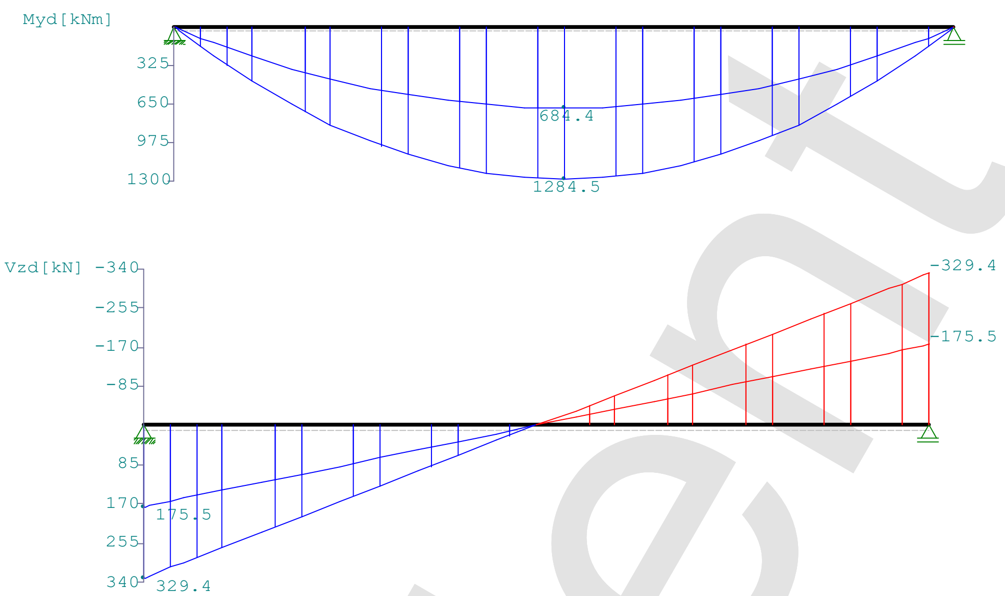
Results for γ-times loads
Partial safety factor $\gamma G * K_{Fi}$ = 1.35 constant over whole girder length

Span moments maximum						(kNm , kN)	
Span		Mfd	Mdle	Mdri	V le	V ri	comb
1	x0 = 7.80	1284.48	0.00	0.00	329.35	-329.36	A 2

Support moments maximum							(kNm , kN)
Support	Mdle	Mdri	Vdle	Vdri	max F	min F	comb
1	0.00	0.00	0.00	329.35	329.35	175.50	A 2
2	0.00	0.00	-329.35	0.00	329.36	175.50	A 2

Moment boundary diagram											
x/L = .0 Span		.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
1	0.00	246	438	575	657	684	657	575	438	246	0.00
1	0.00	462	822	1079	1233	1284	1233	1079	822	462	0.00

Scale 1 : 150



Design DIN EN 1992-1-1/NA/A1:2015-12
FLBemBn.DLL: Version 9.0.1.121 (1)
C35/45 B500A normaly ductil

Concrete cover: cv = 2.5 cm >= req. cv
Reinforcement location: dt = 4.0 cm dB = 8 dS = 14
Db = 6.0 cm dB = 8 dS = 12
Span reinforcement is not curtailed.
The ductility reinforcement by 9.2.1.1 is contained in the required reinforcement.

Creep factor ϕ = 2.28 εcs = 0.38 ‰ h0 = 22.50 cm

All supports identical:Brickwork b = 20.0 cm

Minimum reinforcement EN2 9.2.1.1 (9.1) fctm = 3.21 N/mm2
Calculating Wy, breadth of slab is limited to 2*b0.

Q.No.	min Mb (kNm)	req As (cm2)	min Mo (kNm)	req As (cm2)	
1	222.06	6.25	-303.60	8.33	120.0/15.0/40.0/85.0

Span reinforcement

Span No.	x (m)	Myd (kNm)	min Myd (kNm)	d (cm)	kx	Asb (cm2)	Ast (cm2)	comb	
1	7.80	1284.5		79.0	0.07	36.5	0.0	A	2

On first support are at least 11.3 cm2 to be anchored.
On last support are at least 11.3 cm2 to be anchored.
Shear force VK-support is with F = V,Ed * Cot(Theta) / 2 considered.

shear force reinforcement B500A DIN EN 1992-1-1/NA/A1:2015-12 6.2

column No.		dist (m)	kz	VEd (kN)	Θ (°)	VRd,c (kN)	VRd,max (kN)	a_max (cm)	asw (cm2/m)	comb	
1	ri	0.86	0.92	293.2	18.4	163.1	1297.7	30.0	4.1~	A	2
1	*	1.65	0.92	259.8	18.4	163.1	1297.7	30.0	4.1~	A	2
2	le	0.86	0.92	-293.2	18.4	163.1	1297.7	30.0	4.1~	A	2
2	*	1.65	0.92	-259.8	18.4	163.1	1297.7	30.0	4.1~	A	2

~ at the end of line: Minimum stirrup reinforcement
max distance of stirrups will with Θ >= 40° investigated (paper DAfStb 525).

shoulder shear

Span	xa (cm)	xe (cm)	mle (kNm)	mri (kNm)	av (cm)	beff (cm)	dFcd (kN)	vEd (kN/m2)	vEd,perm.	asf (cm2/m)
1	0	390	0.0	963.4	390	248	554	947	7325	2.7
1	390	780	963.4	1284.5	390	248	185	316	7325	0.9
1	780	1170	1284.5	963.4	390	248	185	316	7325	0.9
1	1170	1560	963.4	3.3	390	248	552	944	7325	2.7

At the following table the loads are specified by their internal numeration.
The following table of calculated combinations referenced.
to these numbers

Load type (kN,m) :
1=uniform over L
2=concentrated at a
3=single moment at a
4=trapezoidal btw. a, a+b
5=triangular over L
6=trapezoidal over L

No.	span	Type	Grp	g1	q1	g2	q2	factor	distance	length
1	1	1	A 1	22.50	7.90			1.00		

Calculated combinations from 1 Loads

lc	K1	K2
	g	g
1	.	x

The combinations above will be managed as followed :
Calculating ULS the dead loads will be exceeded
all at once alternating by GammaG = 1,00 / 1,35.
If in one combination live-loads from different actions
exists , then will be investigated, which action is
the dominating one.
The effect of the duration of action will be checked too.

B.1.7. Design Calculations POS. S.12 – Semi-Precast Concrete Slab

Position: S.12 - Semi-Precast Concrete Slab

Plates by Finite Elements PLT 01/2018A (Frilo R-2018-1/P12)

System: Floor Plan
-> Siehe Anhang Pläne

System: Floor Plan with FE Mesh
-> Siehe Anhang Pläne

LOADCASE 1 "Self-Weight"

Type:	permanent
Dead loads due to	
Plate , beams and parapets	
are included:	YES
Action:	Ständige Lasten
Partial safety factor action:	1.35
Partial safety factor concrete:	1.50
Partial safety factor steel:	1.15
Load points:	8
Point loads:	0
Line loads:	1
Area loads:	1
Temperature loads:	0
Total of input loads:	496
(portion on the plate)	
Dead loads due to	
plate and beams:	2061
Total of all loads:	2558
Total of all support reactions:	2558

NOTE
All effects of actions (like moments, shear forces, support reactions,deflections, etc.) of an individual load case are, unlike the results of a superposition of load cases, plain, i.e. characteristic values.
Design results are based on design quantities including the partial safety coefficients.

Load case 1 "Self-Weight"
Area Loads
-> Siehe Anhang Pläne

LOADCASE 2 "Live Load"

Type:	non permanent
Dead loads due to Plate , beams and parapets are included:	NO
Action:	Büros
Partial safety factor action:	1.50
Partial safety factor concrete:	1.50
Partial safety factor steel:	1.15
Load points:	8
Point loads:	0
Line loads:	1
Area loads:	1
Temperature loads:	0
Total of input loads: (portion on the plate)	747
Total of all support reactions:	747

NOTE
All effects of actions (like moments, shear forces,
support reactions,deflections, etc.) of an individual load case
are, unlike the results of a superposition of load cases,
plain, i.e. characteristic values.
Design results are based on design quantities including the partial
safety coefficients.

Load case 2 "Live Load"
Area Loads
-> Siehe Anhang Pläne

SUPERPOSITION 1 "Characteristic"

Load Cases Involved

Number	Load case	Type	Dead Loads included	Action Shrt Name Hnd	Alter-native Group
1	Self-Weight	permanent	yes	g Ständige ...	-
2	Live Load	variable	no	2 Büros	0

Actions Concerned

Number	Shrt Name Hnd	Type
1	g Ständige Lasten	permanent
2	2 Büros	variable

Superposition 1 "Characteristic"
Deflection [mm] MAX
-> Siehe Anhang Pläne

Superposition 1 "Characteristic"
Deflection [mm] MIN
-> Siehe Anhang Pläne

Superposition 1 "Characteristic"
Beam B1
-> Siehe Anhang Pläne

Superposition 1 "Characteristic"

Beam B1
Start: 16 (0.000 /2.365) End: 17 (19.000 /2.365)

x	Bending Moment		Shear Force		Torsion moment		Deflec-tion	
	MAX	MIN	MAX	MIN	MAX	MIN	MAX	MIN
[m]	[kNm]	[kNm]	[kN]	[kN]	[kNm]	[kNm]	[mm]	[mm]
0.00	-0.5	-0.6	87.2	67.8	1.2	0.9	0.0	0.0
0.52	44.8	34.8	87.2	67.8	1.2	0.9	0.3	0.2
0.52	43.2	33.6	72.7	56.5	1.3	1.0	0.3	0.2
1.04	81.0	63.0	72.7	56.5	1.3	1.0	0.5	0.4
1.04	79.4	61.8	55.1	42.7	1.3	1.0	0.5	0.4
1.56	108.1	84.0	55.1	42.7	1.3	1.0	0.7	0.6
1.56	106.5	82.8	36.6	28.3	1.3	1.0	0.7	0.6
2.08	125.5	97.5	36.6	28.3	1.3	1.0	0.9	0.7
2.08	123.7	96.2	17.8	13.7	1.3	1.0	0.9	0.7
2.60	133.0	103.3	17.8	13.7	1.3	1.0	0.9	0.7
2.60	130.9	101.7	-1.3	-1.5	1.3	1.0	0.9	0.7
3.12	130.1	101.0	-1.3	-1.5	1.3	1.0	0.9	0.7
3.12	127.5	99.0	-17.1	-21.8	1.3	1.0	0.9	0.7
3.65	116.2	90.1	-17.1	-21.8	1.3	1.0	0.9	0.7
3.65	112.9	87.6	-34.5	-44.4	1.3	1.0	0.9	0.7
4.17	89.8	69.6	-34.5	-44.4	1.3	1.0	0.7	0.6
4.17	85.6	66.4	-55.9	-71.9	1.3	1.0	0.7	0.6
4.69	48.2	37.3	-55.9	-71.9	1.3	1.0	0.5	0.4
4.69	43.4	33.6	-85.0	-109.4	1.3	1.0	0.5	0.4
5.21	-10.7	-13.6	-85.0	-109.4	1.3	1.0	0.3	0.3
5.21	-14.6	-18.7	-130.8	-168.4	1.3	1.0	0.3	0.3
5.73	-82.7	-106.4	-130.8	-168.4	1.3	1.0	0.1	0.1
5.73	-83.4	-107.2	-232.0	-299.0	1.4	1.0	0.1	0.1
6.25	-204.2	-262.9	-232.0	-299.0	1.4	1.0	0.0	0.0
6.25	-204.1	-262.8	264.8	205.8	1.3	1.0	0.0	0.0
6.75	-101.2	-130.4	264.8	205.8	1.3	1.0	-0.0	-0.0
6.75	-99.3	-127.9	140.3	109.1	1.2	0.9	-0.0	-0.0
7.25	-44.8	-57.8	140.3	109.1	1.2	0.9	0.1	0.0
7.25	-40.4	-52.2	90.7	70.5	0.9	0.7	0.1	0.0
7.75	-5.2	-6.8	90.7	70.5	0.9	0.7	0.1	0.1
7.75	-1.9	-2.6	64.5	50.1	0.5	0.4	0.1	0.1
8.25	29.6	23.1	64.5	50.1	0.5	0.4	0.2	0.2
8.25	32.1	25.1	47.0	36.5	0.2	0.1	0.2	0.2
8.75	55.7	43.3	47.0	36.5	0.2	0.1	0.3	0.2
8.75	56.8	44.2	31.0	24.0	-0.1	-0.1	0.3	0.2
9.25	72.3	56.2	31.0	24.0	-0.1	-0.1	0.4	0.3
9.25	72.3	56.3	13.2	10.1	-0.1	-0.2	0.4	0.3
9.75	78.9	61.3	13.2	10.1	-0.1	-0.2	0.4	0.3
9.75	78.1	60.7	-5.8	-7.4	-0.1	-0.2	0.4	0.3
10.25	74.4	57.8	-5.8	-7.4	-0.1	-0.2	0.4	0.3
10.25	73.0	56.7	-24.3	-31.2	-0.1	-0.1	0.4	0.3
10.75	57.4	44.6	-24.3	-31.2	-0.1	-0.1	0.3	0.2
10.75	55.3	42.9	-46.0	-59.2	0.0	0.0	0.3	0.2
11.25	25.7	19.9	-46.0	-59.2	0.0	0.0	0.2	0.2
11.25	22.6	17.5	-73.7	-94.9	0.1	0.1	0.2	0.2
11.75	-19.3	-24.8	-73.7	-94.9	0.1	0.1	0.1	0.1
11.75	-22.2	-28.5	-115.0	-148.0	0.2	0.1	0.1	0.1
12.25	-79.7	-102.5	-115.0	-148.0	0.2	0.1	0.0	0.0
12.25	-80.2	-103.2	-203.3	-261.7	0.1	0.1	0.0	0.0
12.75	-181.9	-234.0	-203.3	-261.7	0.1	0.1	0.0	0.0
12.75	-180.9	-232.8	238.7	185.6	-0.1	-0.1	0.0	0.0
13.27	-84.3	-108.5	238.7	185.6	-0.1	-0.1	0.1	0.1
13.27	-83.1	-106.9	133.1	103.4	-0.4	-0.5	0.1	0.1
13.79	-29.2	-37.6	133.1	103.4	-0.4	-0.5	0.2	0.2
13.79	-25.6	-33.0	93.7	72.8	-0.8	-1.1	0.2	0.2

Beam B1
Start: 16 (0.000 /2.365) End: 17 (19.000 /2.365)

x	Bending Moment		Shear Force		Torsion moment		Deflection	
[m]	MAX [kNm]	MIN [kNm]	MAX [kN]	MIN [kN]	MAX [kNm]	MIN [kNm]	MAX [mm]	MIN [mm]
14.31	15.9	12.4	93.7	72.8	-0.8	-1.1	0.4	0.3
14.31	19.6	15.3	71.9	55.8	-1.2	-1.6	0.4	0.3
14.83	57.1	44.3	71.9	55.8	-1.2	-1.6	0.5	0.4
14.83	59.7	46.4	53.3	41.4	-1.5	-2.0	0.5	0.4
15.35	87.5	68.0	53.3	41.4	-1.5	-2.0	0.7	0.5
15.35	89.4	69.4	33.7	26.2	-1.6	-2.1	0.7	0.5
15.87	106.9	83.1	33.7	26.2	-1.6	-2.1	0.7	0.6
15.87	108.4	84.2	12.9	10.1	-1.7	-2.2	0.7	0.6
16.40	115.2	89.5	12.9	10.1	-1.7	-2.2	0.8	0.6
16.40	116.6	90.6	-6.3	-8.2	-1.6	-2.1	0.8	0.6
16.92	112.3	87.3	-6.3	-8.2	-1.6	-2.1	0.7	0.6
16.92	113.7	88.3	-22.5	-29.1	-1.6	-2.0	0.7	0.6
17.44	98.6	76.6	-22.5	-29.1	-1.6	-2.0	0.6	0.5
17.44	100.1	77.8	-38.2	-49.2	-1.5	-2.0	0.6	0.5
17.96	74.5	57.9	-38.2	-49.2	-1.5	-2.0	0.4	0.3
17.96	76.2	59.2	-52.7	-68.0	-1.5	-1.9	0.4	0.3
18.48	40.8	31.7	-52.7	-68.0	-1.5	-1.9	0.2	0.2
18.48	42.5	33.0	-64.5	-82.9	-1.3	-1.7	0.2	0.2
19.00	-0.5	-0.7	-64.5	-82.9	-1.3	-1.7	0.0	0.0

SUPERPOSITION 2 "USL Permanent/Transient"

Load Cases Involved

Number	Load case	Type	Dead Loads included	Action Shrt Name Hnd	Alter-native Group
1	Self-Weight	permanent	yes	g Ständige ...	-
2	Live Load	variable	no	2 Büros	0

Actions Concerned

Number	Shrt Name Hnd	Type	Partial Safety sup	inf	Combination dom	ndom
1	g Ständige Lasten	permanent	1.35	1.00	1.00	1.00
2	2 Büros	variable	1.50	0.00	1.00	0.70

Partial safety factor concrete: 1.50
Partial safety factor steel: 1.15

NOTE: Design Values
All results of a superposition of load cases include both using both the partial safety and combination factors.: DIN EN 1990/NA:2010-12

NOTE: Combination Factors
With the combination of independent, variable actions the individual main action is determined for both each location and for each action quantity.
In geneneral, the main actions differ with each location and each quantity.
the leading effect.
The individually found main action receives the conbination factor 1.00. In case of only one variable action this action is considered the main action.

Superposition 2 "USL Permanent/Transient"
Moment m-1 [kNm/m] MAX
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Moment m-1 [kNm/m] MIN
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Moment m-2 [kNm/m] MAX
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Moment m-2 [kNm/m] MIN
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Moment m-12 [kNm/m] MAX
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Moment m-12 [kNm/m] MIN
Design Values (x Gamma)

-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Design Moment, bottom mB-1 [kNm/m]
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Design Moment, bottom mB-2 [kNm/m]
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Design Moment, top mB-1 [kNm/m]
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Design Moment, top mB-2 [kNm/m]
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Reinforcement, bottom aS-1 [cm²/m]
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Reinforcement, bottom aS-2 [cm²/m]
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Reinforcement, top aS-1 [cm²/m]
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Reinforcement, top aS-2 [cm²/m]
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Shear Force q-1z [kN/m]
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Shear Force q-2z [kN/m]
Design Values (x Gamma)
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Shear reinforcement [cm²/m²]
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"
Beam B1
-> Siehe Anhang Pläne

Superposition 2 "USL Permanent/Transient"

Beam B1
Start: 16 (0.000 /2.365) End: 17 (19.000 /2.365)

x [m]	Bending Moment MAX [kNm]	Bending Moment MIN [kNm]	As bottom [cm²]	As top [cm²]	Torsional Moment MAX [kNm]	Torsional Moment MIN [kNm]	As Longitudinal Torsion [cm²]
0.00	-0.5	-0.9	0.00	5.54	1.7	0.9	0.71
0.52	61.9	34.8	3.19	0.00	1.7	0.9	0.71
0.52	59.7	33.6	3.19	0.00	1.8	1.0	0.77
1.04	112.1	63.0	3.77	0.00	1.8	1.0	0.77
1.04	109.8	61.8	3.69	0.00	1.8	1.0	0.77
1.56	149.5	84.0	5.04	0.00	1.8	1.0	0.77
1.56	147.3	82.8	4.96	0.00	1.8	1.0	0.77
2.08	173.7	97.5	5.86	0.00	1.8	1.0	0.77
2.08	171.2	96.2	5.78	0.00	1.8	1.0	0.77
2.60	184.0	103.3	6.22	0.00	1.8	1.0	0.77
2.60	181.1	101.7	6.12	0.00	1.8	1.0	0.76
3.12	180.0	101.0	6.08	0.00	1.8	1.0	0.76
3.12	176.4	99.0	5.96	0.00	1.8	1.0	0.00
3.65	160.7	90.1	5.42	0.00	1.8	1.0	0.00
3.65	156.1	87.6	5.27	0.00	1.8	1.0	0.75
4.17	124.2	69.6	4.18	0.00	1.8	1.0	0.75
4.17	118.5	66.4	3.99	0.00	1.8	1.0	0.75
4.69	66.7	37.3	3.19	0.00	1.8	1.0	0.75
4.69	60.1	33.6	3.19	0.00	1.8	1.0	0.76
5.21	-10.7	-18.8	0.00	5.54	1.8	1.0	0.76
5.21	-14.6	-25.8	0.00	5.54	1.8	1.0	0.70
5.73	-82.7	-147.2	0.00	5.54	1.8	1.0	0.70
5.73	-83.4	-148.4	0.00	5.54	1.9	1.0	0.47
6.25	-204.2	-363.8	0.00	13.67	1.9	1.0	0.46
6.25	-204.1	-363.6	0.00	13.66	1.9	1.0	0.49
6.75	-101.2	-180.4	0.00	6.26	1.9	1.0	0.49
6.75	-99.3	-176.9	0.00	6.14	1.7	0.9	0.70
7.25	-44.8	-79.9	0.00	5.54	1.7	0.9	0.70
7.25	-40.4	-72.2	0.00	5.54	1.3	0.7	0.54
7.75	-5.2	-9.4	0.00	5.54	1.3	0.7	0.54
7.75	-1.9	-3.7	0.00	5.54	0.8	0.4	0.32
8.25	41.0	23.1	3.19	0.00	0.8	0.4	0.32
8.25	44.4	25.1	3.19	0.00	0.3	0.1	0.11
8.75	77.0	43.3	3.19	0.00	0.3	0.1	0.11
8.75	78.5	44.2	3.19	0.00	-0.1	-0.1	0.00
9.25	100.0	56.2	3.36	0.00	-0.1	-0.1	0.00
9.25	100.0	56.3	3.36	0.00	-0.1	-0.3	0.00
9.75	109.2	61.3	3.67	0.00	-0.1	-0.3	0.00
9.75	108.1	60.7	3.63	0.00	-0.1	-0.3	0.00
10.25	103.0	57.8	3.46	0.00	-0.1	-0.3	0.00
10.25	101.0	56.7	3.39	0.00	-0.1	-0.1	0.00
10.75	79.4	44.6	3.19	0.00	-0.1	-0.1	0.00
10.75	76.5	42.9	3.19	0.00	0.0	0.0	0.01
11.25	35.5	19.9	3.19	0.00	0.0	0.0	0.01
11.25	31.3	17.5	3.19	0.00	0.2	0.1	0.07
11.75	-19.3	-34.3	0.00	5.54	0.2	0.1	0.07
11.75	-22.2	-39.4	0.00	5.54	0.3	0.1	0.11
12.25	-79.7	-141.8	0.00	5.54	0.3	0.1	0.11
12.25	-80.2	-142.7	0.00	5.54	0.2	0.1	0.05
12.75	-181.9	-323.7	0.00	11.96	0.2	0.1	0.05
12.75	-180.9	-322.1	0.00	11.90	-0.1	-0.2	0.04
13.27	-84.3	-150.1	0.00	5.54	-0.1	-0.2	0.04
13.27	-83.1	-147.9	0.00	5.54	-0.4	-0.7	0.31
13.79	-29.2	-52.1	0.00	5.54	-0.4	-0.7	0.31
13.79	-25.6	-45.6	0.00	5.54	-0.8	-1.5	0.64

Beam B1
Start: 16 (0.000 /2.365) End: 17 (19.000 /2.365)

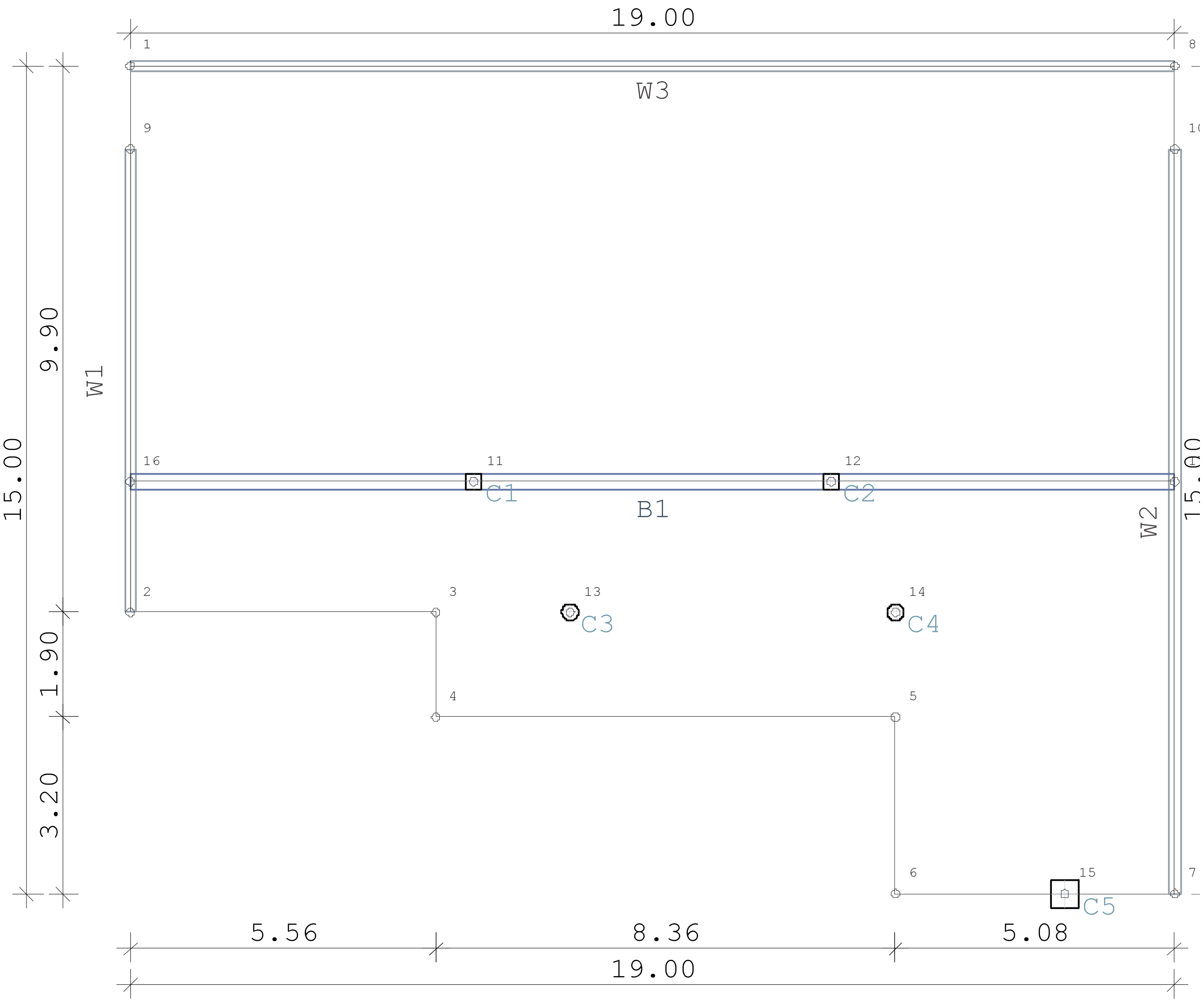
x	Bending Moment MAX	MIN	As bottom	As top	Torsional Moment MAX	MIN	As Longitudinal Torsion
[m]	[kNm]	[kNm]	[cm²]	[cm²]	[kNm]	[kNm]	[cm²]
14.31	21.9	12.4	3.19	0.00	-0.8	-1.5	0.64
14.31	27.1	15.3	3.19	0.00	-1.2	-2.2	0.95
14.83	78.9	44.3	3.19	0.00	-1.2	-2.2	0.95
14.83	82.6	46.4	3.19	0.00	-1.5	-2.7	1.16
15.35	121.0	68.0	4.07	0.00	-1.5	-2.7	1.16
15.35	123.6	69.4	4.16	0.00	-1.6	-2.9	1.25
15.87	147.9	83.1	4.99	0.00	-1.6	-2.9	1.25
15.87	150.0	84.2	5.06	0.00	-1.7	-3.0	1.27
16.40	159.4	89.5	5.38	0.00	-1.7	-3.0	1.27
16.40	161.3	90.6	5.44	0.00	-1.6	-2.9	1.24
16.92	155.4	87.3	5.24	0.00	-1.6	-2.9	1.24
16.92	157.3	88.3	5.31	0.00	-1.6	-2.8	1.20
17.44	136.4	76.6	4.59	0.00	-1.6	-2.8	1.20
17.44	138.5	77.8	4.67	0.00	-1.5	-2.7	1.16
17.96	103.0	57.9	3.46	0.00	-1.5	-2.7	1.16
17.96	105.4	59.2	3.54	0.00	-1.5	-2.7	1.13
18.48	56.4	31.7	3.19	0.00	-1.5	-2.7	1.13
18.48	58.8	33.0	3.19	0.00	-1.3	-2.4	1.03
19.00	-0.5	-1.0	0.00	5.54	-1.3	-2.4	1.03

Beam B1
Start: 16 (0.000 /2.365) End: 17 (19.000 /2.365)

x	Shear Force MAX	MIN	Compress strut	VEd / VRd,c	VEd / VRd,max	As Stirrup
[m]	[kN]	[kN]	cot [1]	[1]	[1]	[cm²/m]
0.00	120.7	67.8	3.00	1.80	0.17	2.78
0.52	120.7	67.8	3.00	1.80	0.17	2.78
0.52	100.6	56.5	3.00	1.50	0.14	2.78
1.04	100.6	56.5	3.00	1.50	0.14	2.78
1.04	76.2	42.7	3.00	1.14	0.11	2.78
1.56	76.2	42.7	3.00	1.14	0.11	2.78
1.56	50.6	28.3	3.00	0.76	0.07	2.78
2.08	50.6	28.3	3.00	0.76	0.07	2.78
2.08	24.6	13.7	3.00	0.37	0.04	2.78
2.60	24.6	13.7	3.00	0.37	0.04	2.78
2.60	-1.3	-2.1	3.00	0.03	0.00	2.78
3.12	-1.3	-2.1	3.00	0.03	0.00	2.78
3.12	-17.1	-30.2	3.00	0.45	0.04	2.78
3.65	-17.1	-30.2	3.00	0.45	0.04	2.78
3.65	-34.5	-61.4	3.00	0.92	0.09	2.78
4.17	-34.5	-61.4	3.00	0.92	0.09	2.78
4.17	-55.9	-99.5	3.00	1.48	0.14	2.78
4.69	-55.9	-99.5	3.00	1.48	0.14	2.78
4.69	-85.0	-151.4	3.00	2.26	0.22	2.78
5.21	-85.0	-151.4	3.00	2.26	0.22	2.78
5.21	-130.8	-233.1	2.67	3.48	0.30	3.40
5.73	-130.8	-233.1	2.67	3.48	0.30	3.40
5.73	-232.0	-413.6	1.76	6.17	0.41	9.00
6.25	-232.0	-413.6	1.74	4.91	0.42	9.29
6.25	366.3	205.8	1.85	4.35	0.38	7.79
6.75	366.3	205.8	1.87	5.47	0.38	7.53
6.75	194.0	109.1	3.00	2.90	0.28	2.78
7.25	194.0	109.1	3.00	2.90	0.28	2.78
7.25	125.5	70.5	3.00	1.87	0.18	2.78
7.75	125.5	70.5	3.00	1.87	0.18	2.78
7.75	89.2	50.1	3.00	1.33	0.13	2.78
8.25	89.2	50.1	3.00	1.33	0.13	2.78
8.25	65.1	36.5	3.00	0.97	0.09	2.78
8.75	65.1	36.5	3.00	0.97	0.09	2.78
8.75	42.8	24.0	3.00	0.64	0.06	2.78

Beam B1
Start: 16 (0.000 /2.365) End: 17 (19.000 /2.365)

x [m]	Shear Force MAX [kN]	MIN [kN]	Compress strut cot [1]	VEd / VRd,c [1]	VEd / VRd,max [1]	As Stirrup [cm²/m]
9.25	42.8	24.0	3.00	0.64	0.06	2.78
9.25	18.2	10.1	3.00	0.27	0.03	2.78
9.75	18.2	10.1	3.00	0.27	0.03	2.78
9.75	-5.8	-10.2	3.00	0.15	0.01	2.78
10.25	-5.8	-10.2	3.00	0.15	0.01	2.78
10.25	-24.3	-43.1	3.00	0.64	0.06	2.78
10.75	-24.3	-43.1	3.00	0.64	0.06	2.78
10.75	-46.0	-81.9	3.00	1.22	0.12	2.78
11.25	-46.0	-81.9	3.00	1.22	0.12	2.78
11.25	-73.7	-131.3	3.00	1.96	0.19	2.78
11.75	-73.7	-131.3	3.00	1.96	0.19	2.78
11.75	-115.0	-204.8	3.00	3.06	0.29	2.78
12.25	-115.0	-204.8	3.00	3.06	0.29	2.78
12.25	-203.3	-362.0	1.92	5.40	0.38	7.09
12.75	-203.3	-362.0	1.91	4.49	0.38	7.19
12.75	330.2	185.6	2.03	4.10	0.36	6.18
13.27	330.2	185.6	2.04	4.93	0.36	6.08
13.27	184.1	103.4	3.00	2.75	0.26	2.78
13.79	184.1	103.4	3.00	2.75	0.26	2.78
13.79	129.7	72.8	3.00	1.94	0.18	2.78
14.31	129.7	72.8	3.00	1.94	0.18	2.78
14.31	99.4	55.8	3.00	1.48	0.14	2.78
14.83	99.4	55.8	3.00	1.48	0.14	2.78
14.83	73.8	41.4	3.00	1.10	0.11	2.78
15.35	73.8	41.4	3.00	1.10	0.11	2.78
15.35	46.6	26.2	3.00	0.70	0.07	2.78
15.87	46.6	26.2	3.00	0.70	0.07	2.78
15.87	17.9	10.1	3.00	0.27	0.03	2.78
16.40	17.9	10.1	3.00	0.27	0.03	2.78
16.40	-6.3	-11.4	3.00	0.17	0.02	2.78
16.92	-6.3	-11.4	3.00	0.17	0.02	2.78
16.92	-22.5	-40.2	3.00	0.60	0.06	2.78
17.44	-22.5	-40.2	3.00	0.60	0.06	2.78
17.44	-38.2	-68.1	3.00	1.02	0.10	2.78
17.96	-38.2	-68.1	3.00	1.02	0.10	2.78
17.96	-52.7	-94.0	3.00	1.40	0.13	2.78
18.48	-52.7	-94.0	3.00	1.40	0.13	2.78
18.48	-64.5	-114.7	3.00	1.71	0.16	2.78
19.00	-64.5	-114.7	3.00	1.71	0.16	2.78



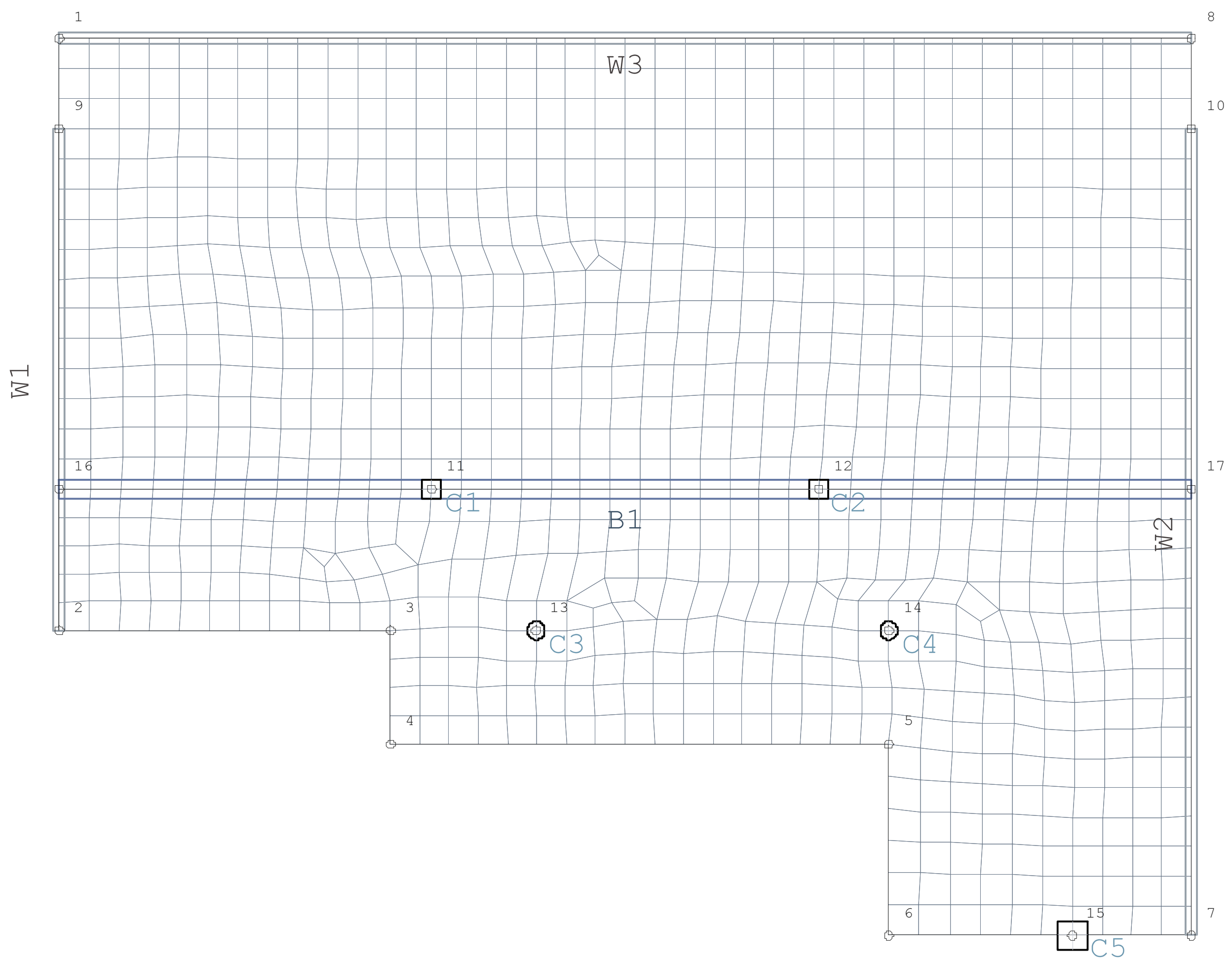
Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

System: Floor Plan

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 1
1 : 125
26.07.2018



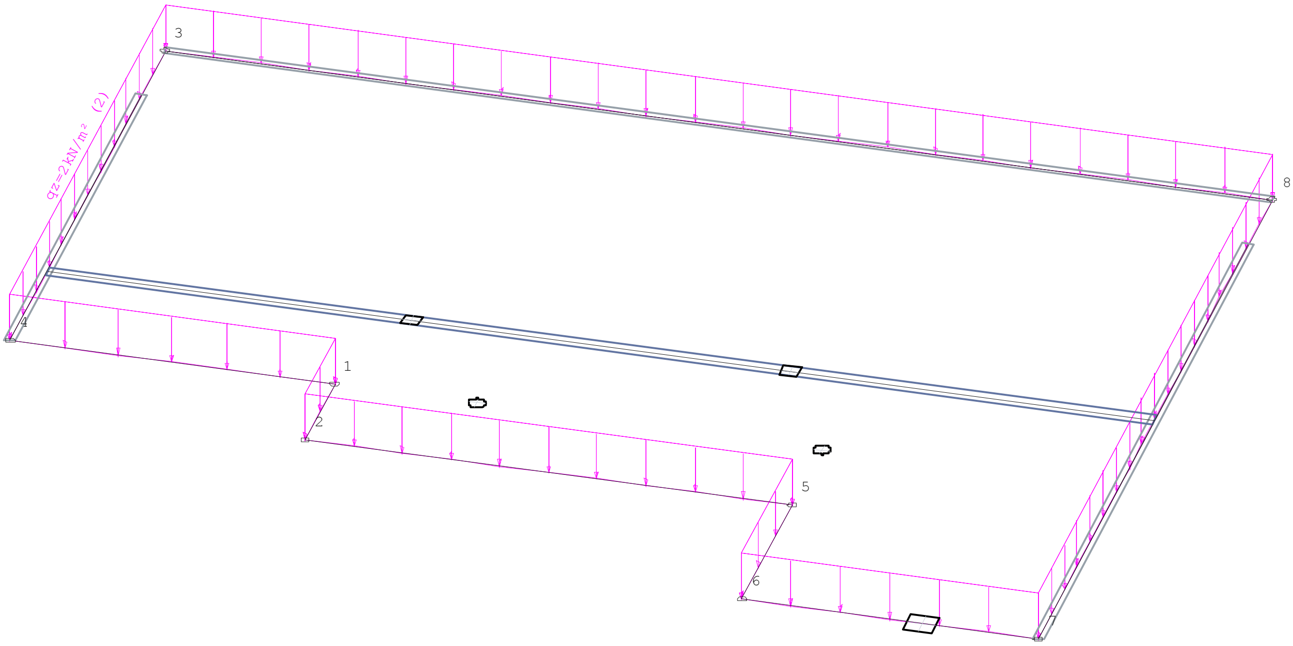
Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

System: Floor Plan with FE Mesh

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 2
1 : 125
26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Load case 1 "Self-Weight"

Area Loads

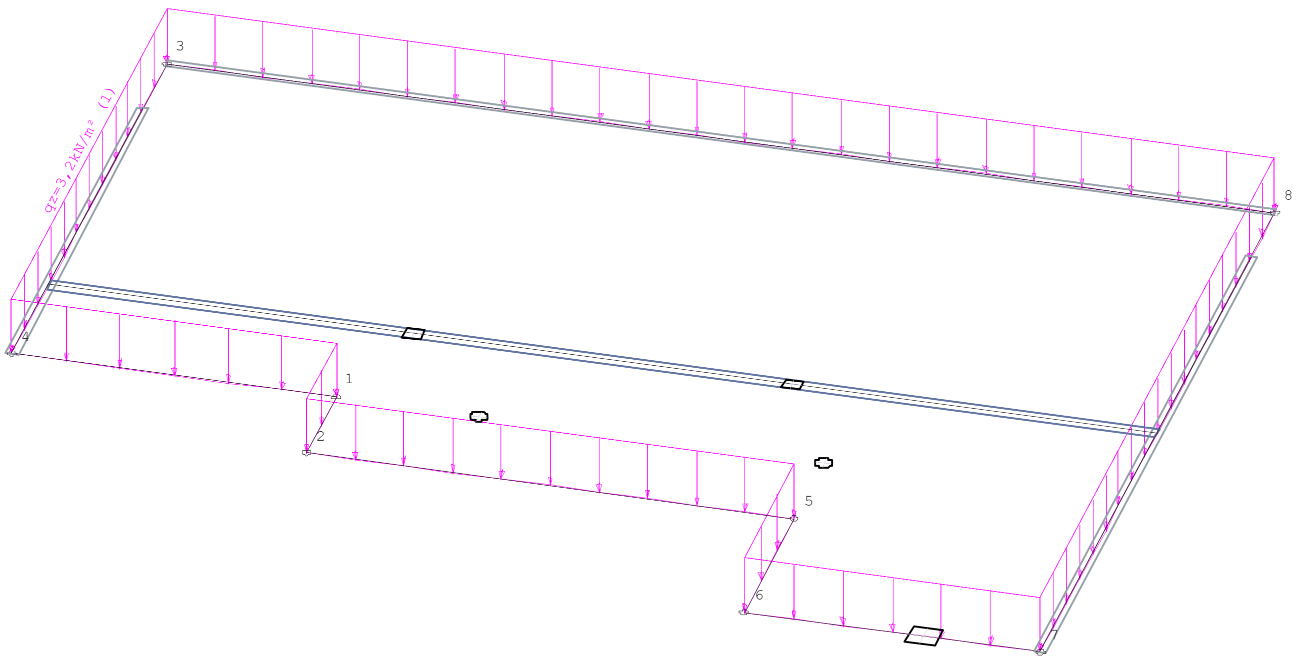
Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

Seite: 3

1 : 125

26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Load case 2 "Live Load"

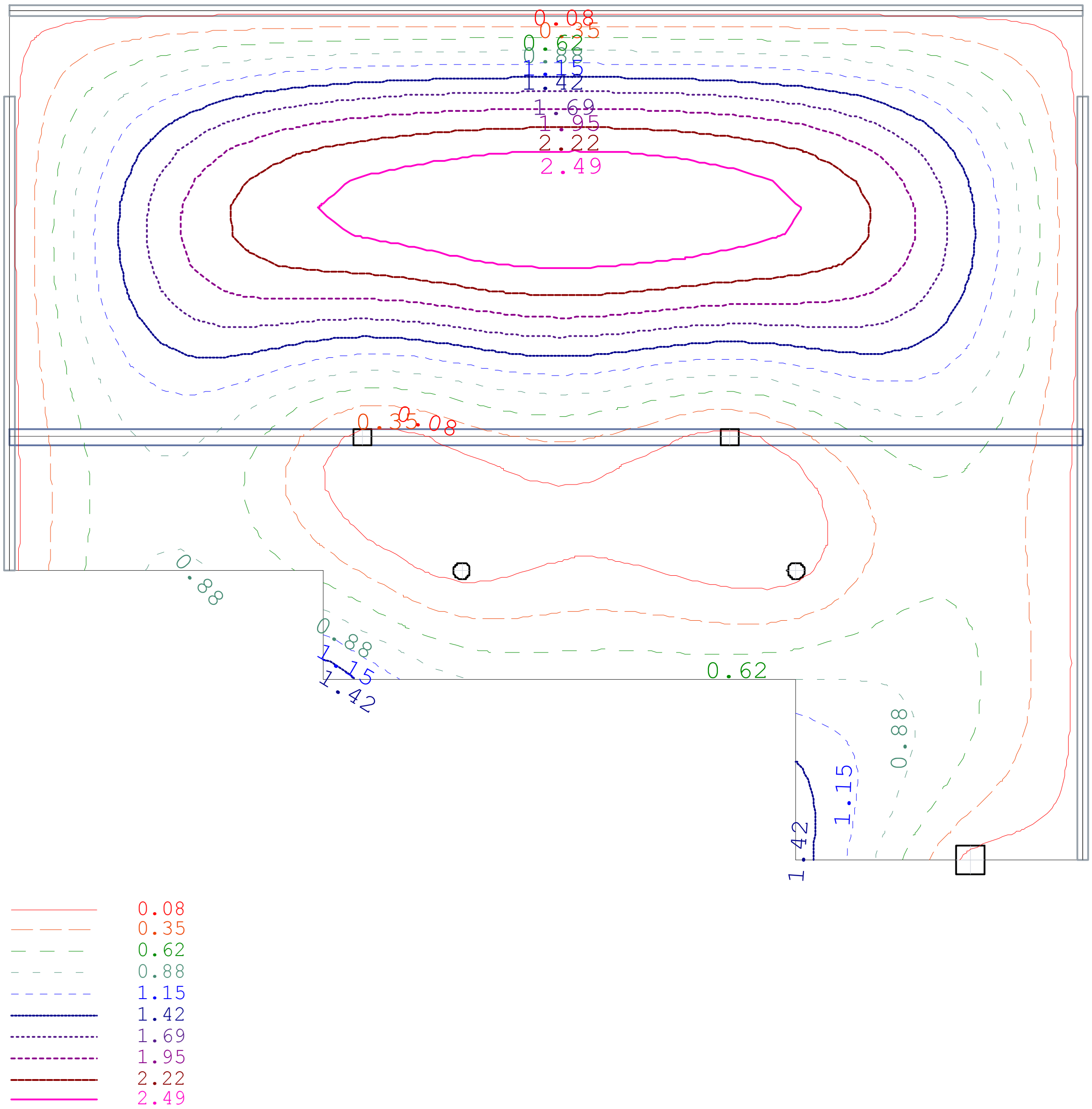
Area Loads

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

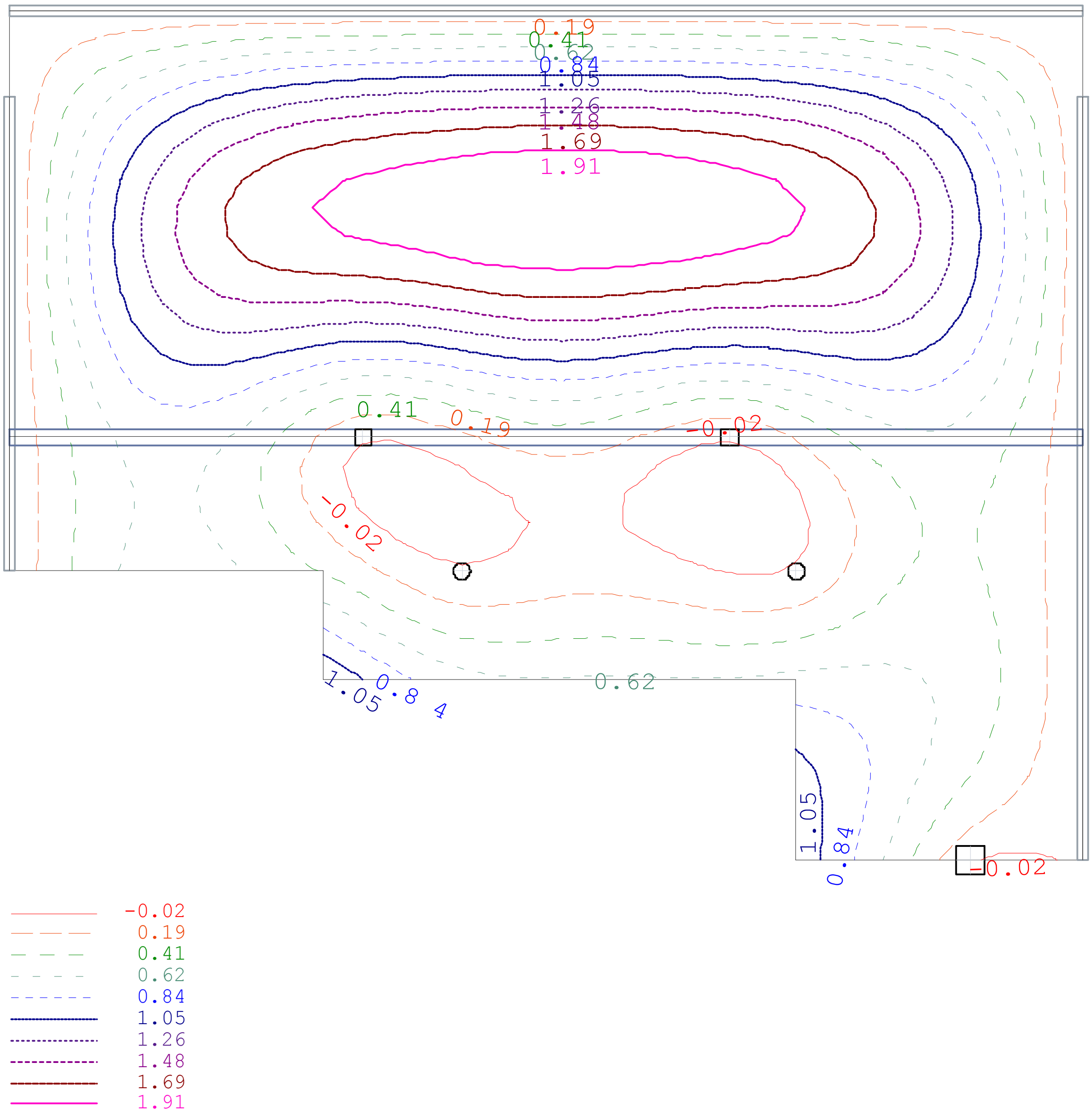
Prj.Nr.:

1 : 125

26.07.2018



Projekt: Design Calculations	
Position: S.12 - Semi-Precast Concrete Slab	
Superposition 1 "Characteristic" **Deflection [mm] MAX**	
Demo Frilo Stuttgarter Straße 40 70469 Stuttgart Tel.: 0711 810020 Email: info@frilo.de	Prj.Nr.: Seite: 5 1 : 125 26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

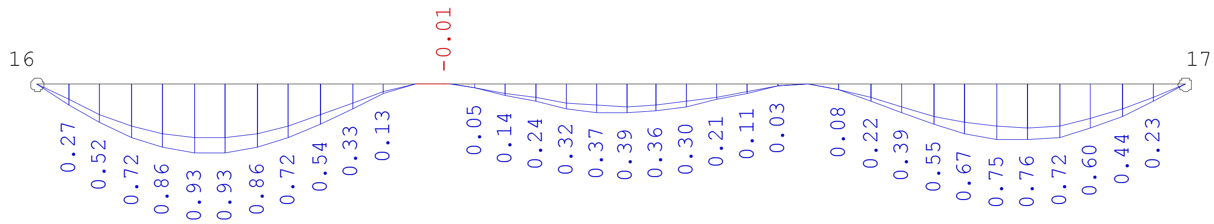
Superposition 1 "Characteristic"

Deflection [mm] MIN

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 6
1 : 125
26.07.2018

Beam B1: Deflection [mm]



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 1 "Characteristic"

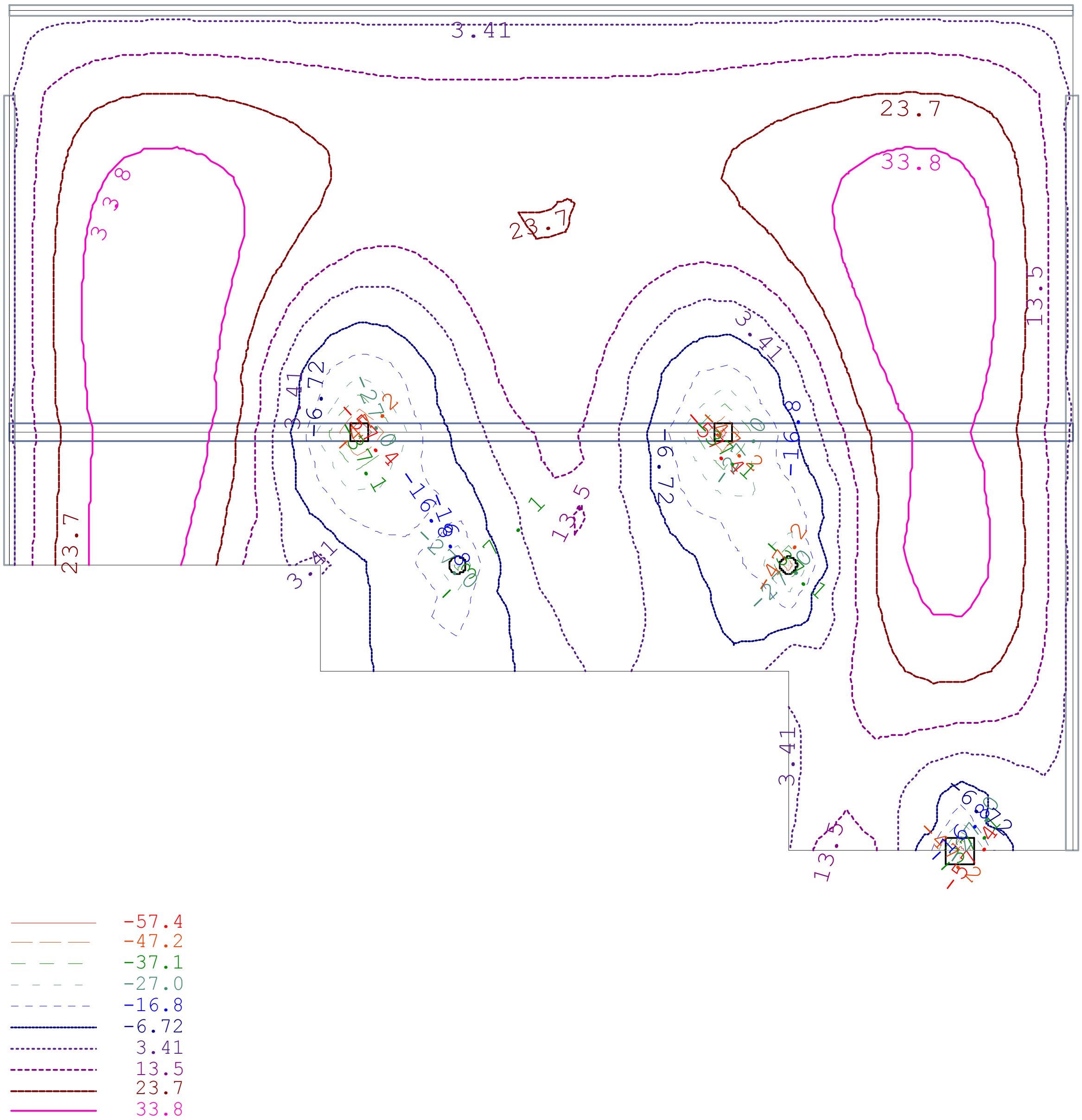
Beam B1

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125

26.07.2018



Projekt: Design Calculations

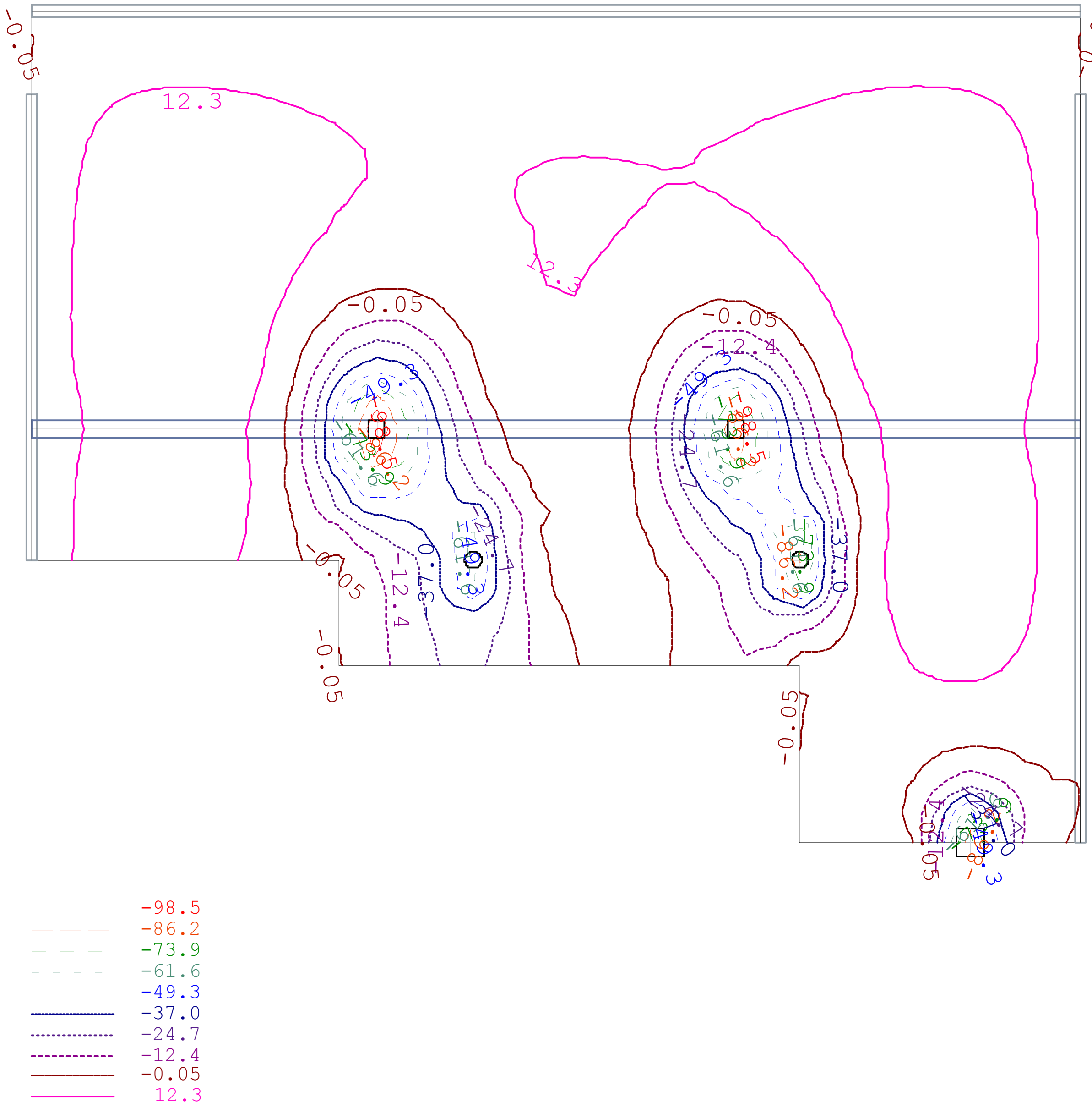
Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Moment m-1 [kNm/m] MAX

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125
26.07.2018



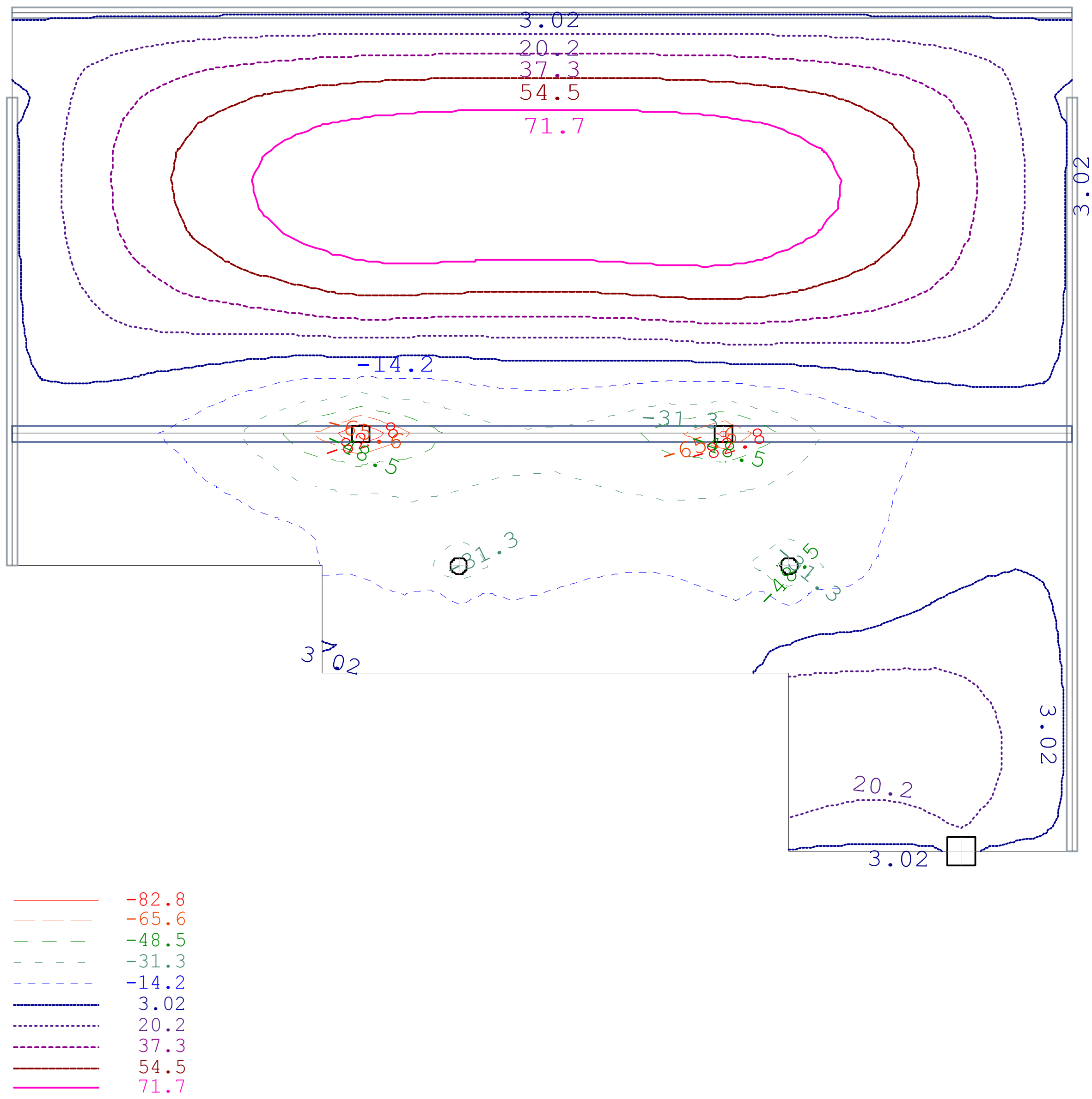
Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Moment m-1 [kNm/m] MIN

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 9
1 : 125
26.07.2018



Projekt: Design Calculations

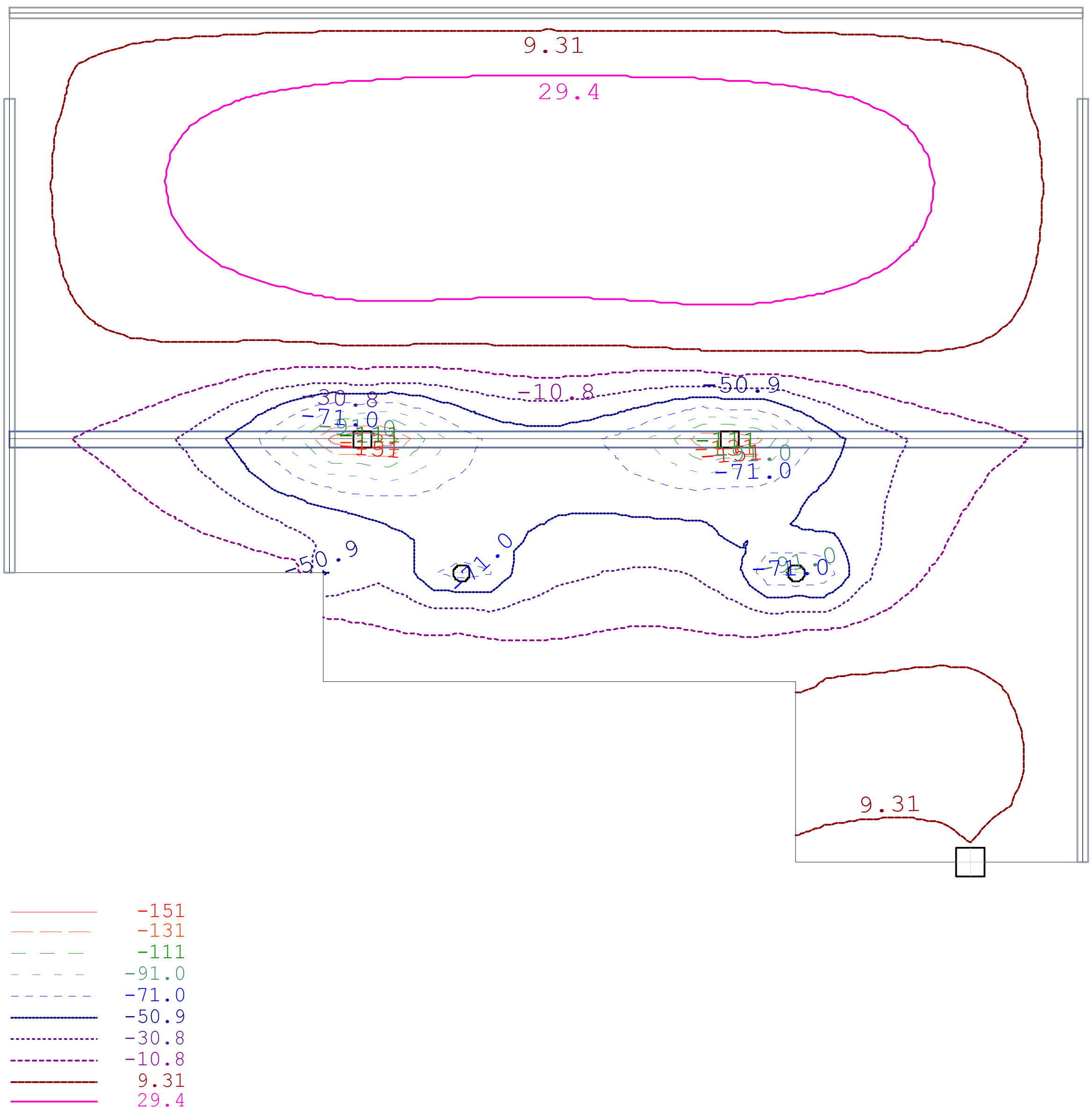
Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Moment m-2 [kNm/m] MAX

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125
26.07.2018



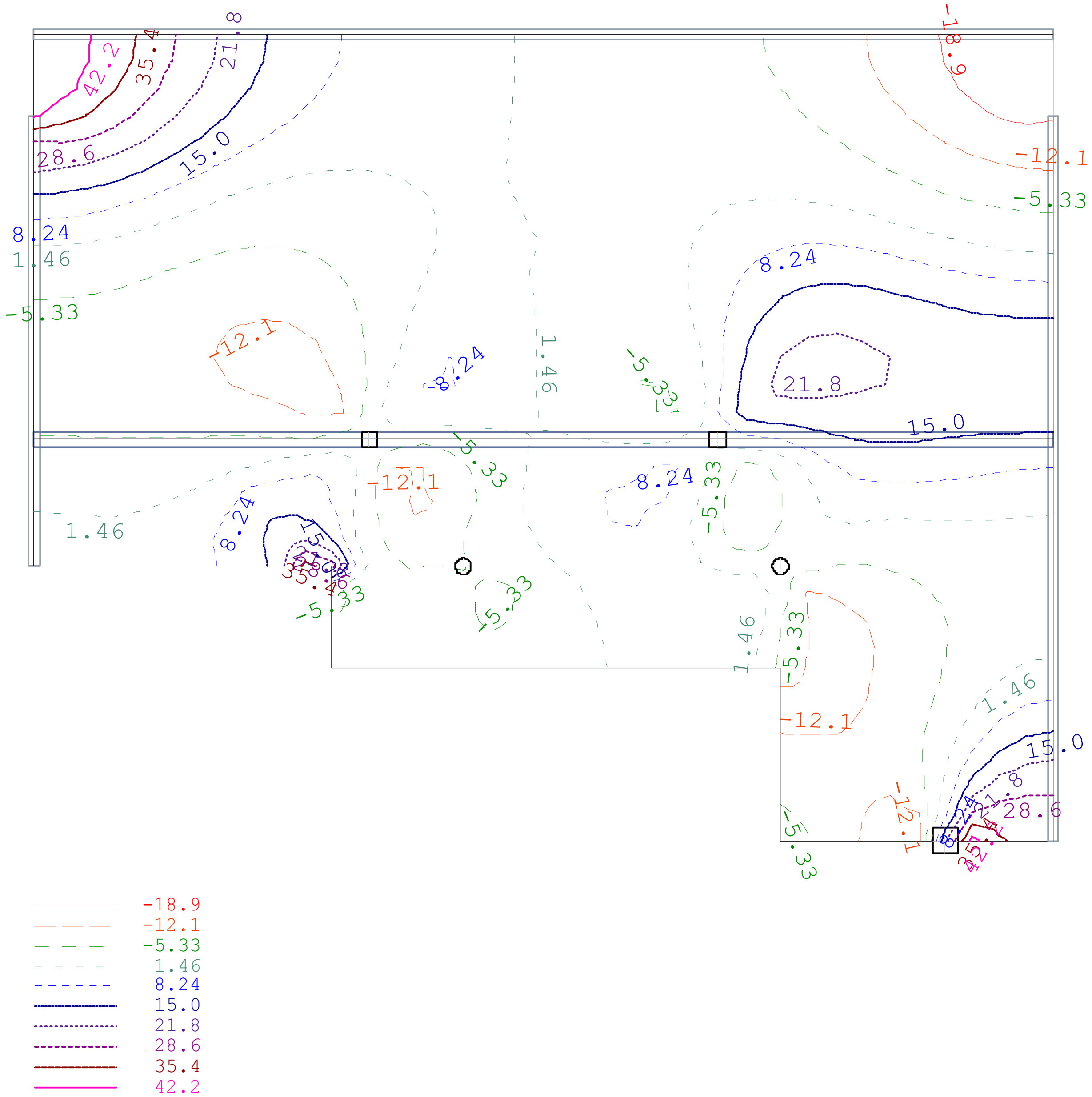
Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Moment m-2 [kNm/m] MIN

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 11
1 : 125
26.07.2018



Projekt: Design Calculations

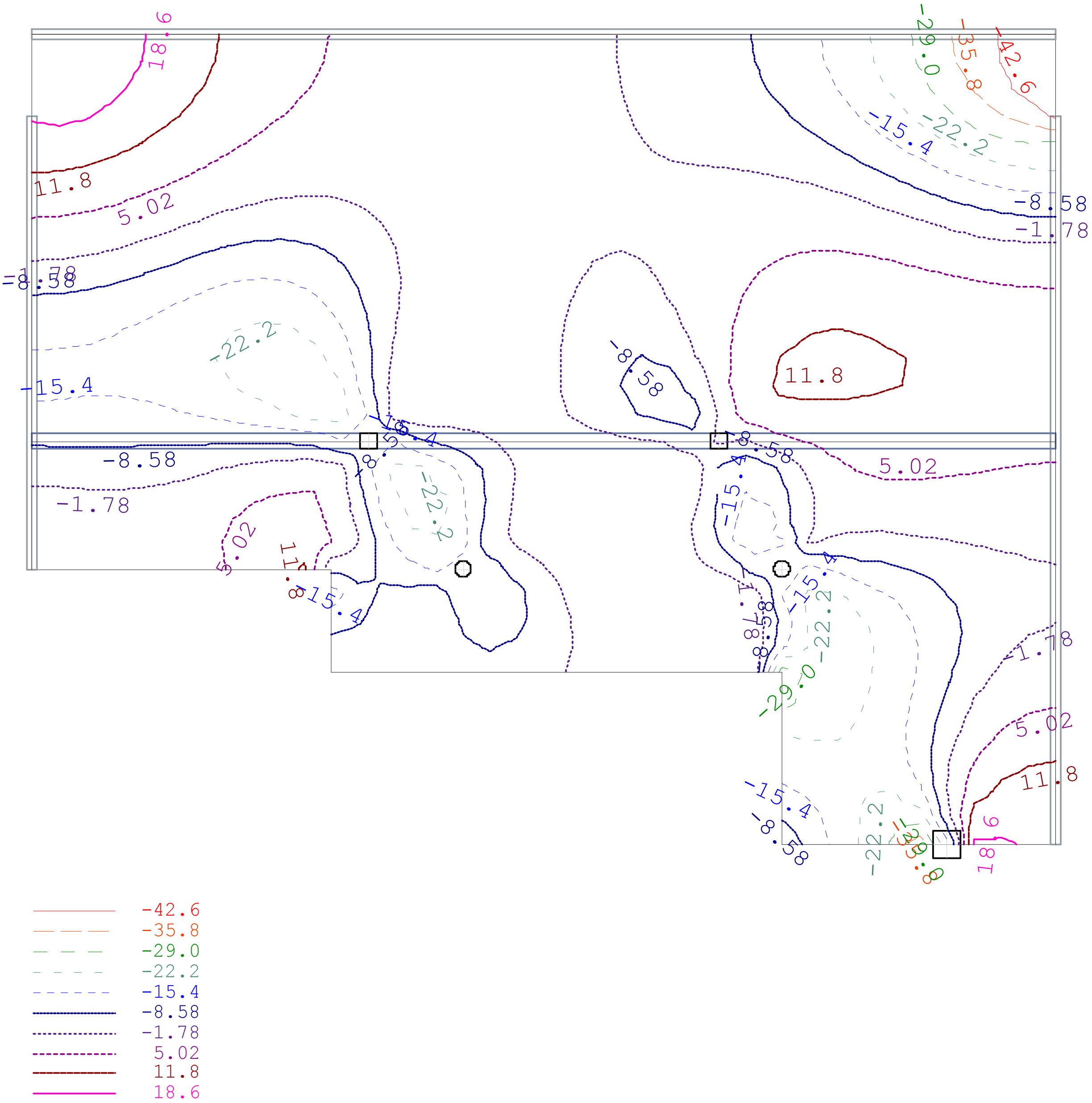
Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Moment m-12 [kNm/m] MAX

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125
26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"

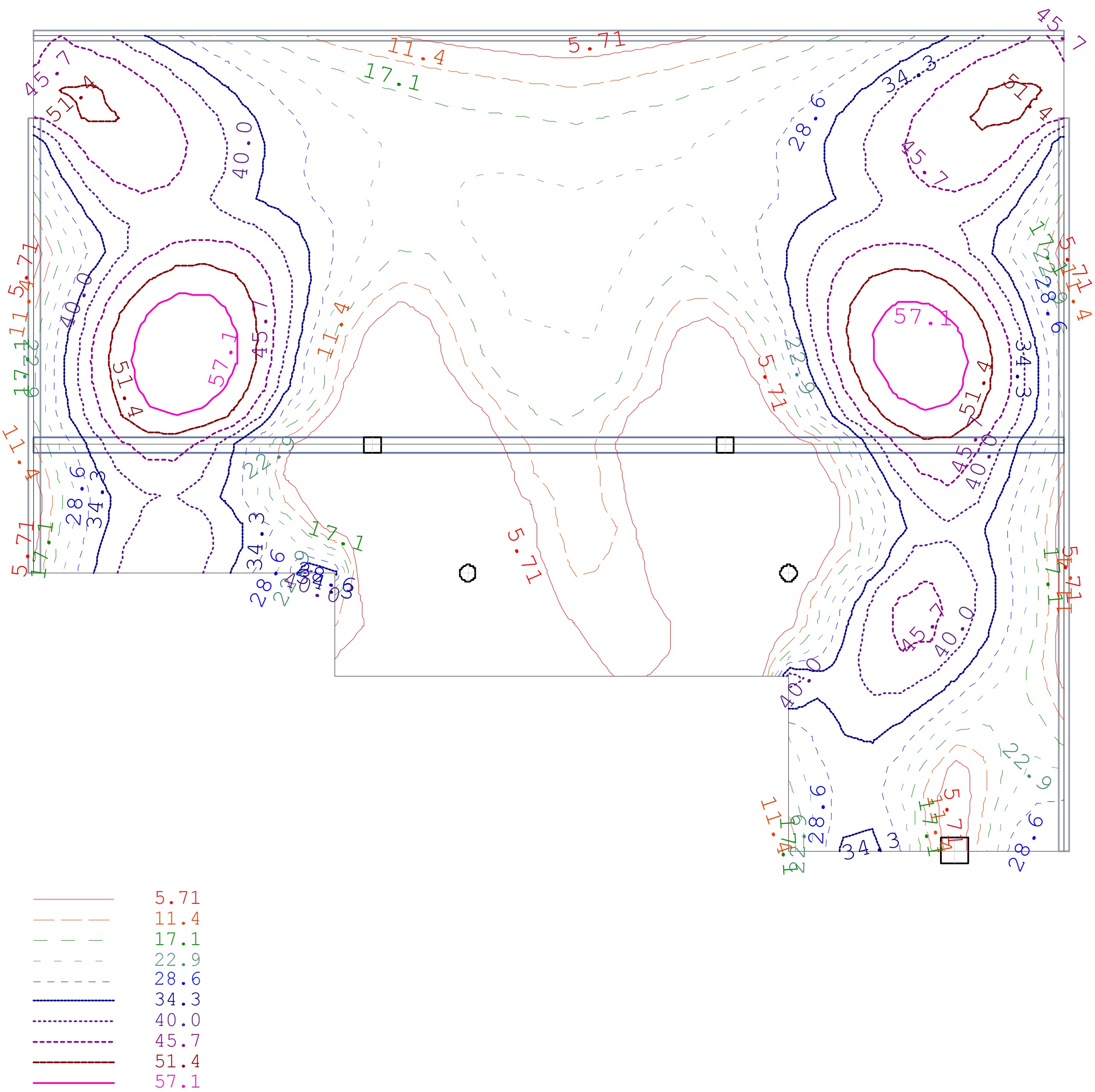
Moment m-12 [kNm/m] MIN

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125

26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"

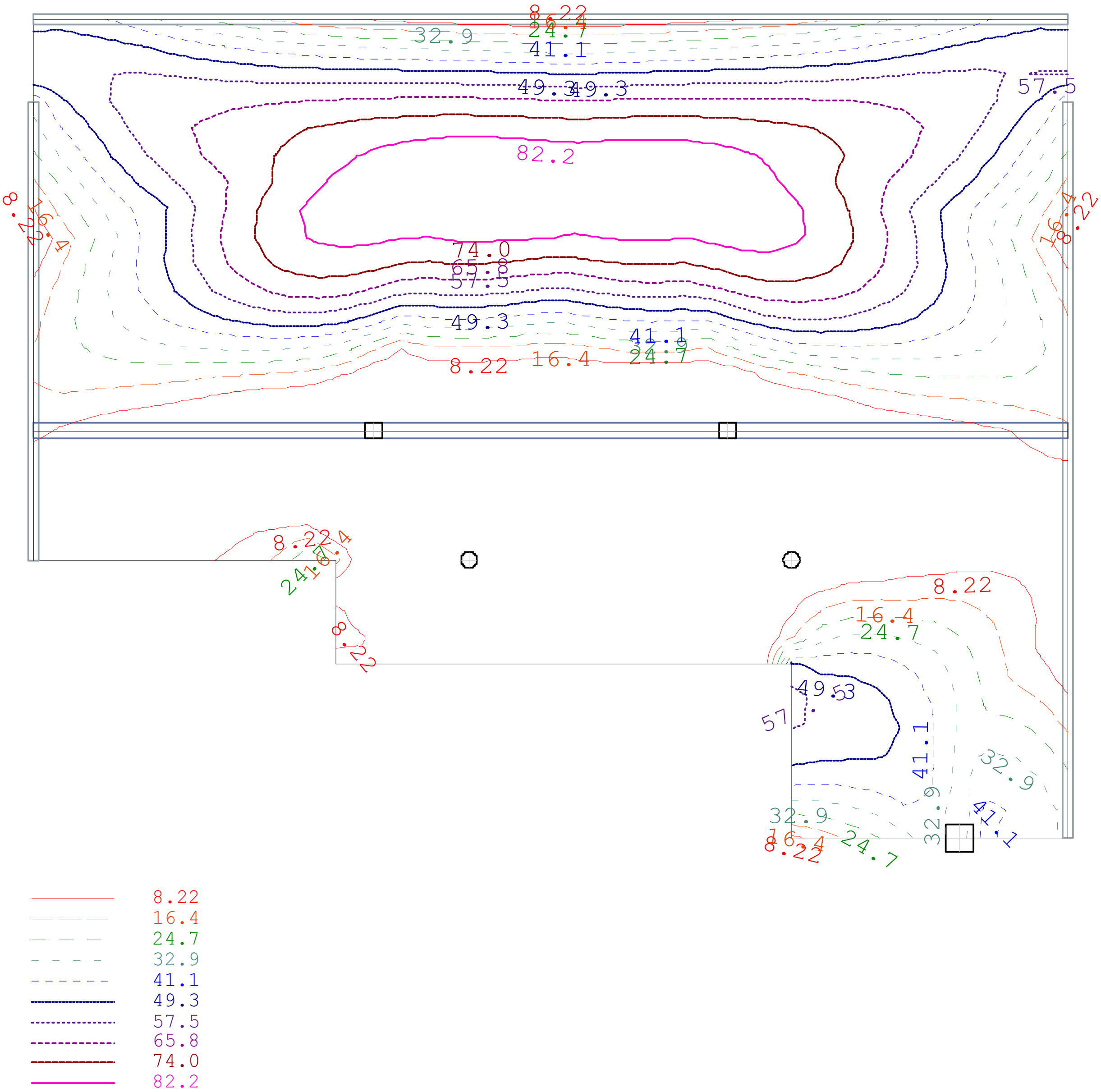
Design Moment, bottom mB-1 [kNm/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125

26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"

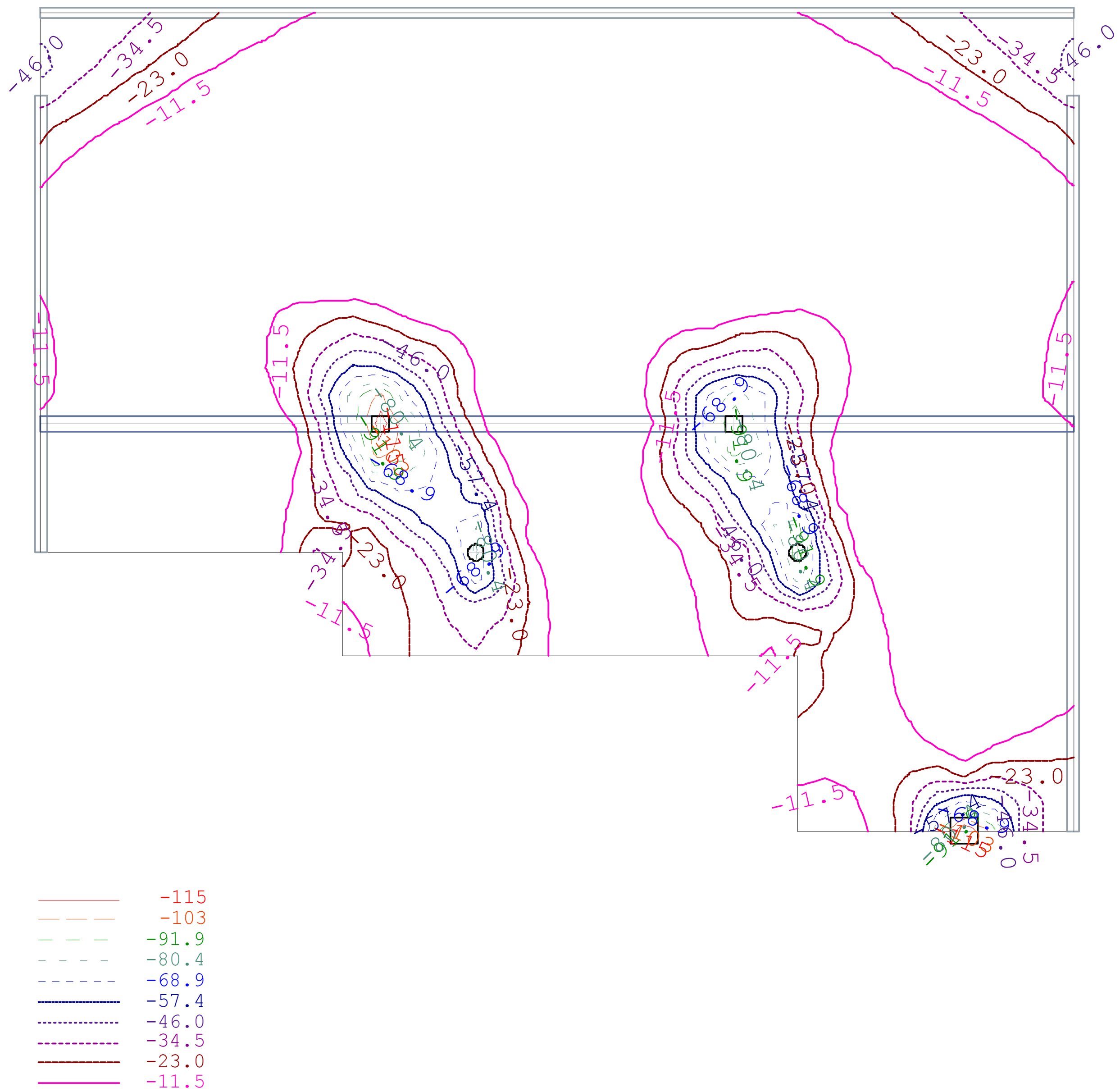
Design Moment, bottom mB-2 [kNm/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125

26.07.2018



Projekt: Design Calculations

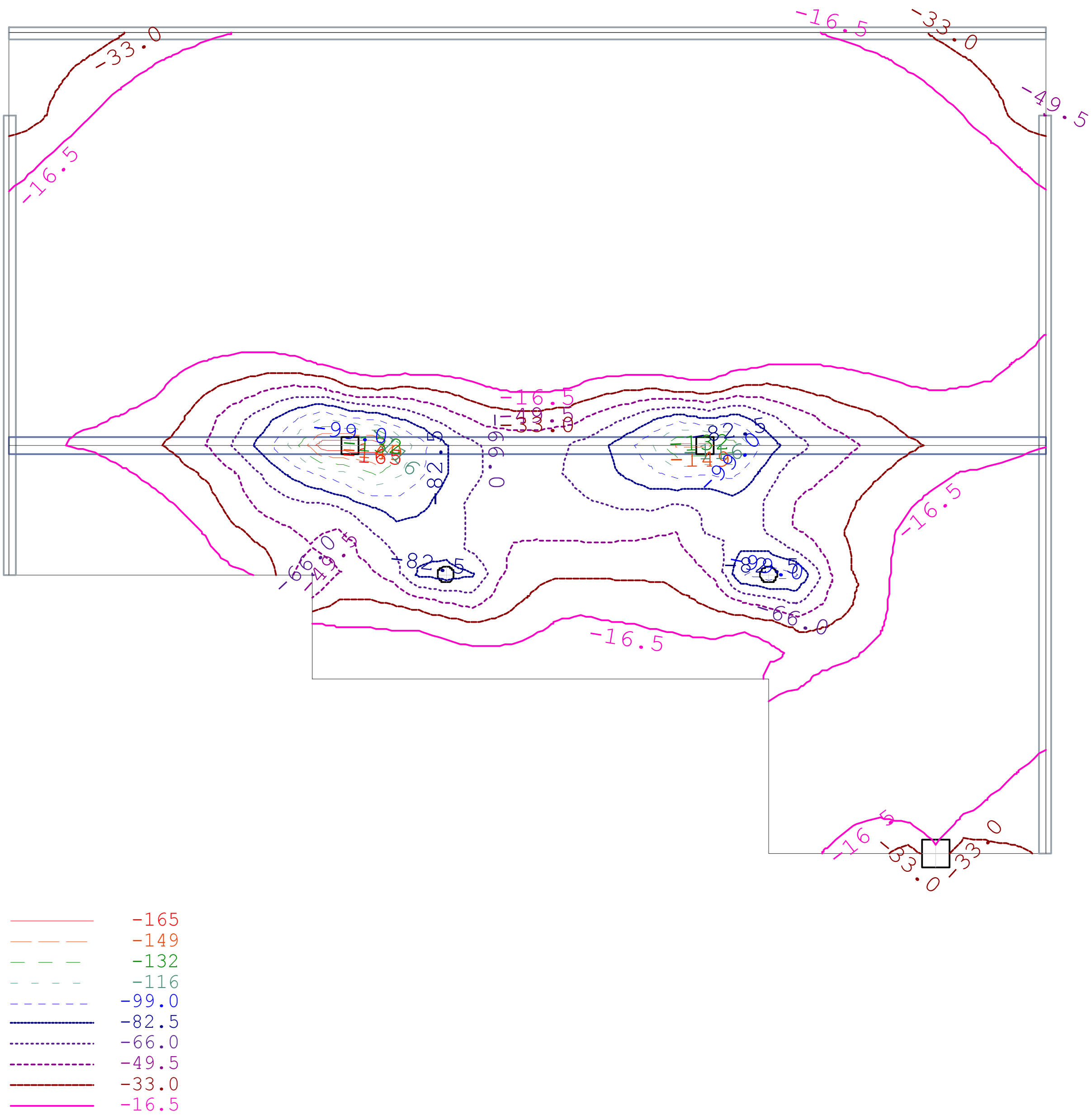
Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Design Moment, top mB-1 [kNm/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125
26.07.2018



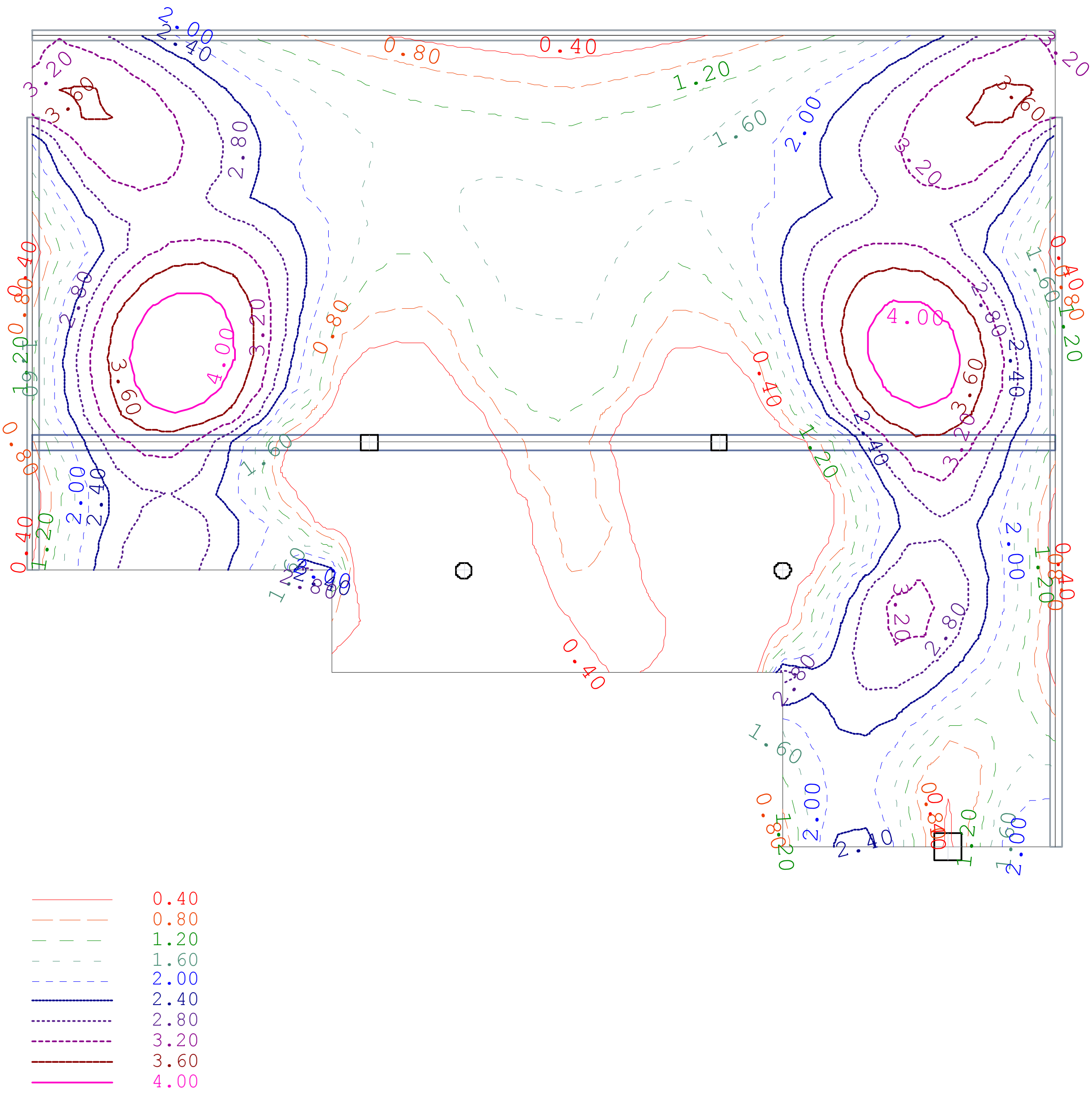
Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Design Moment, top mB-2 [kNm/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 17
1 : 125
26.07.2018



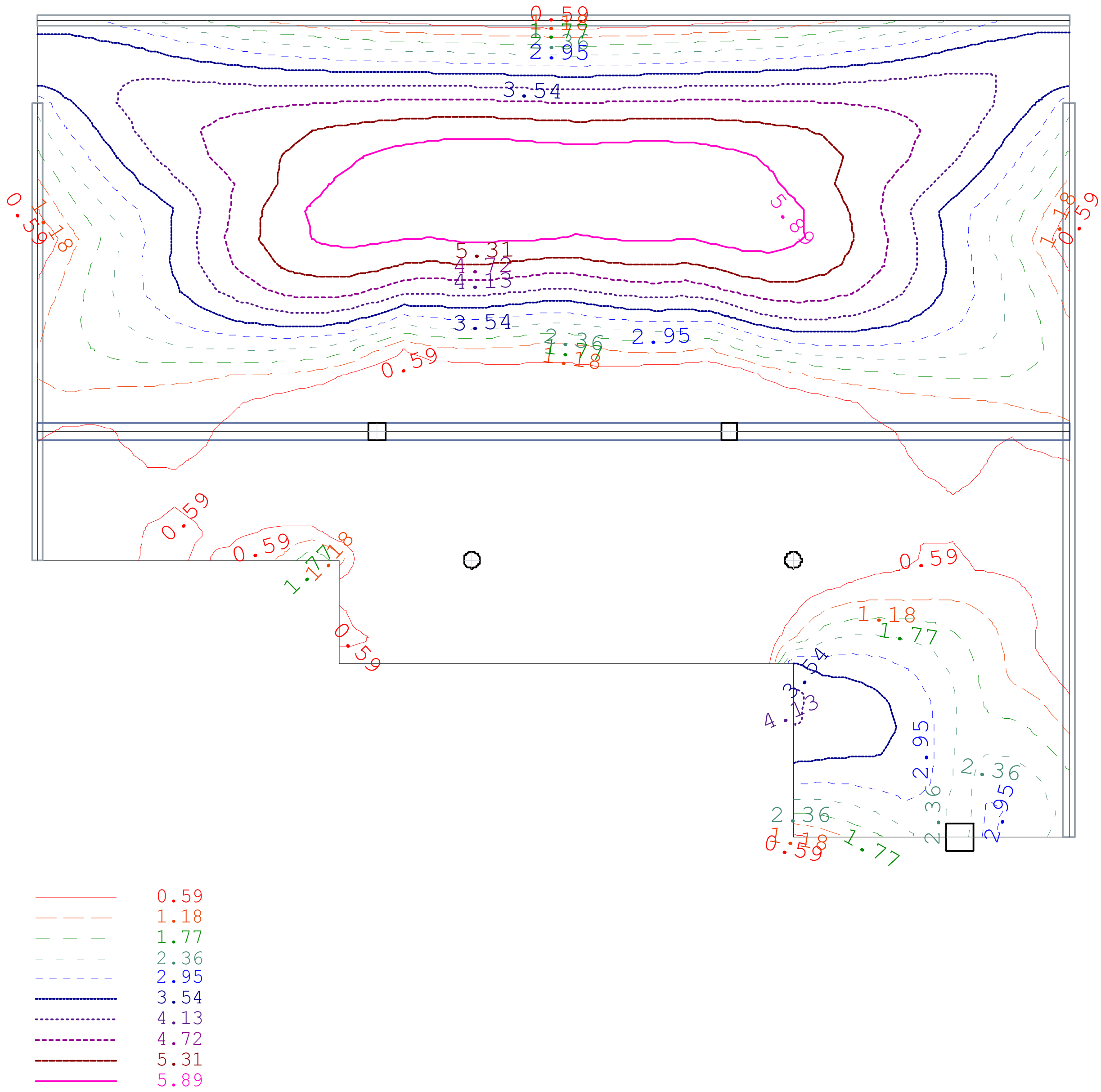
Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Reinforcement, bottom aS-1 [cm²/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 18
1 : 125
26.07.2018



Projekt: Design Calculations

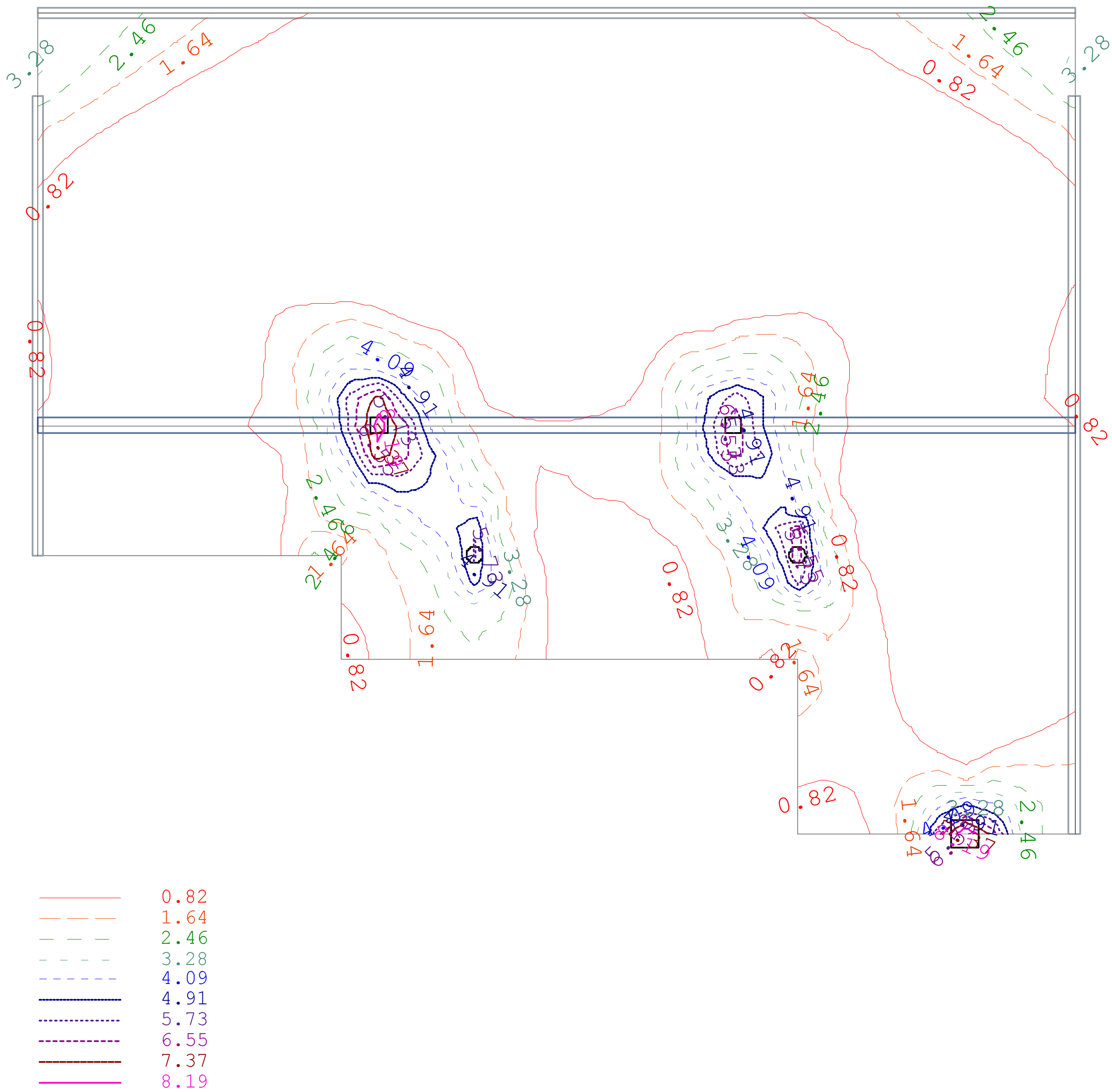
Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Reinforcement, bottom aS-2 [cm²/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125
26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"

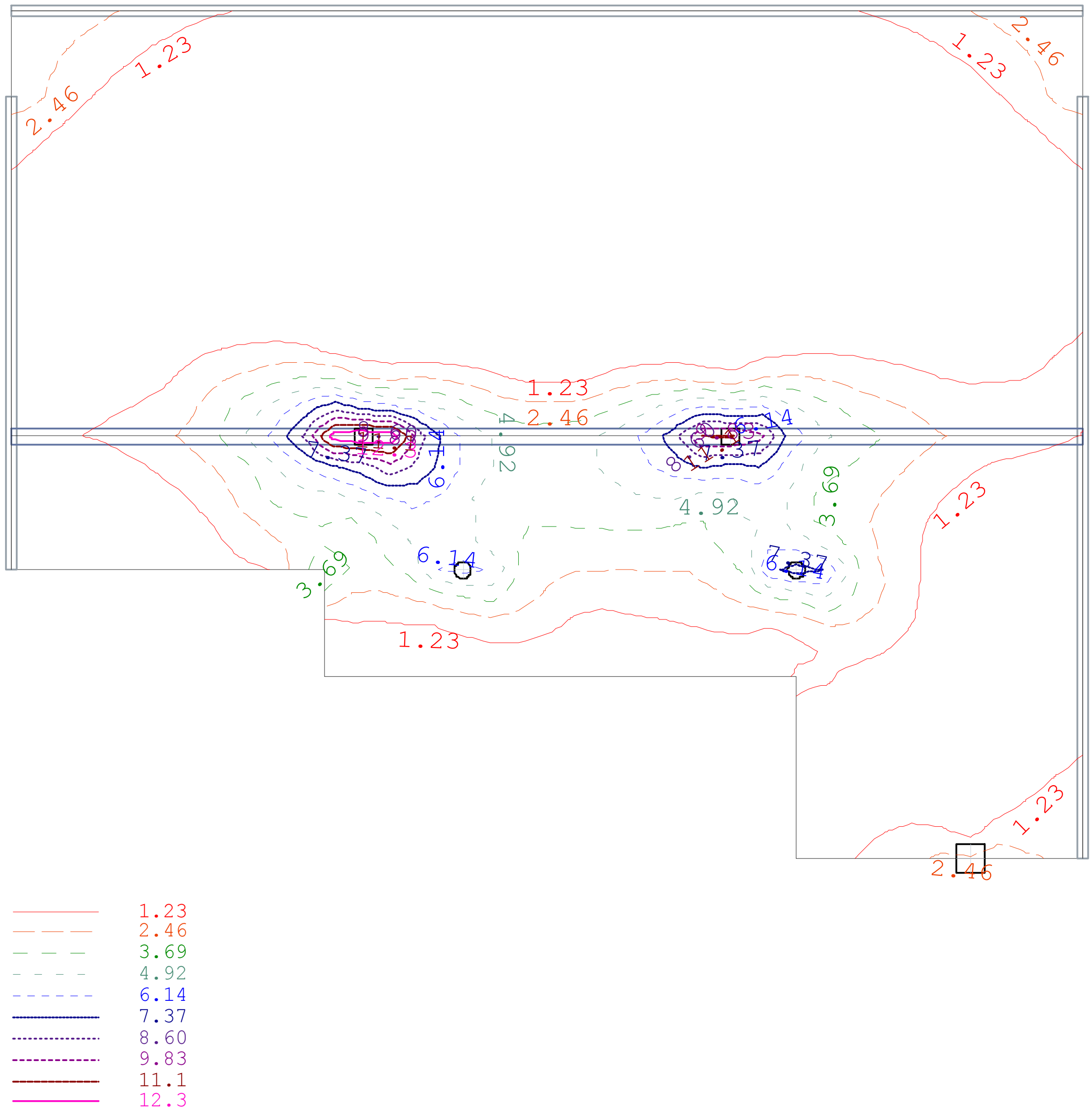
Reinforcement, top aS-1 [cm²/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125

26.07.2018



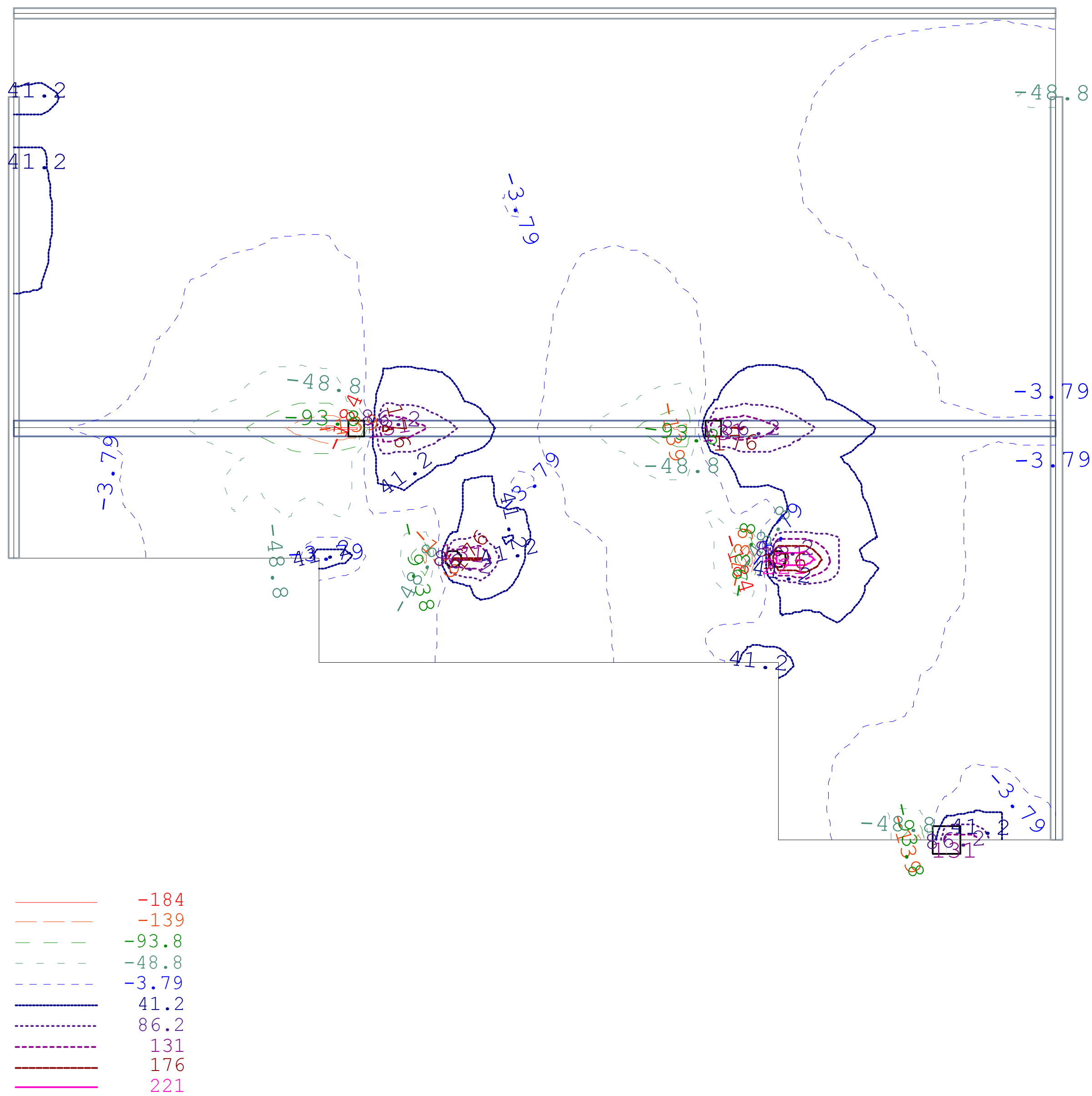
Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Reinforcement, top aS-2 [cm²/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 21
1 : 125
26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"

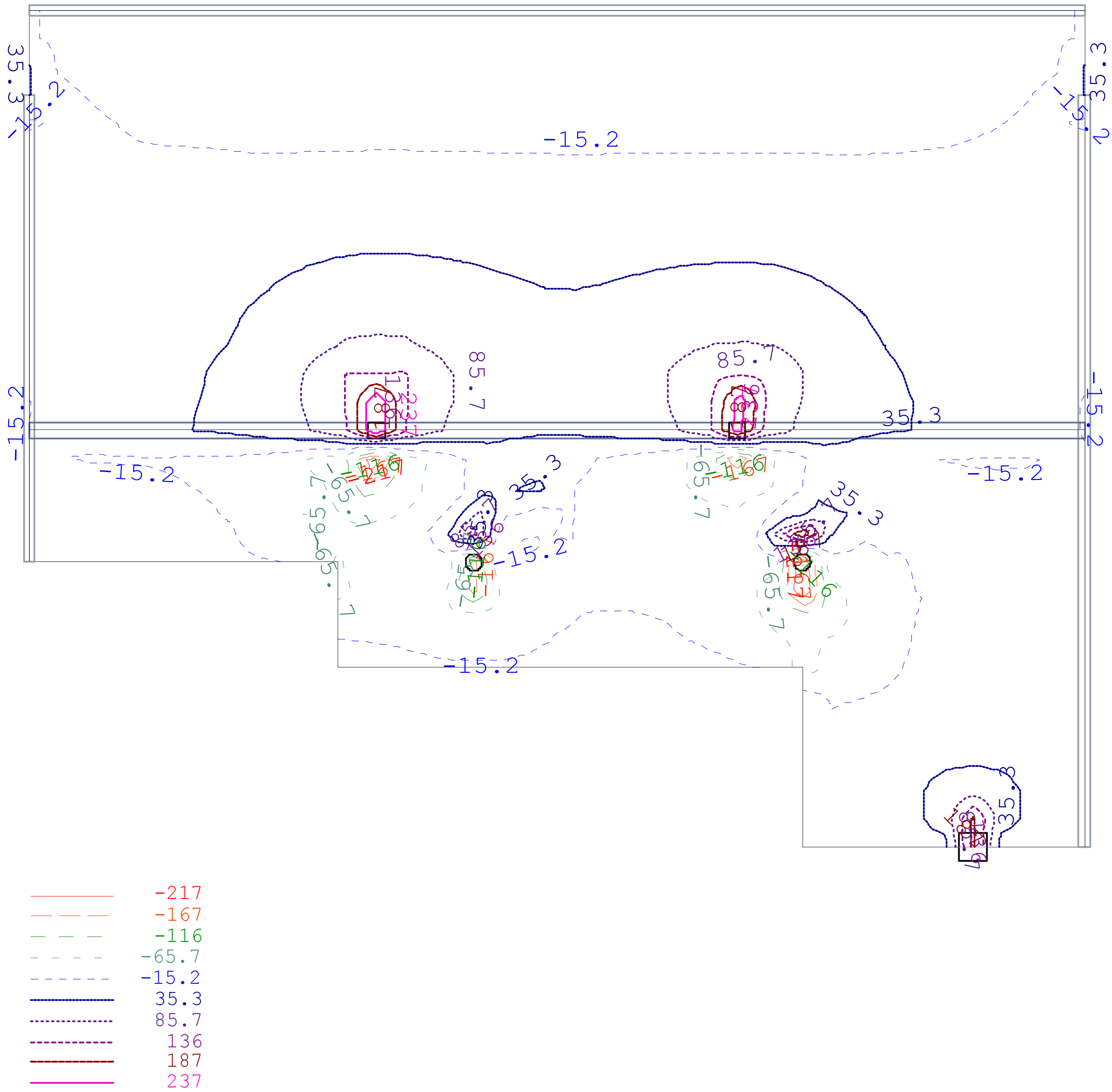
Shear Force q-1z [kN/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125

26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"

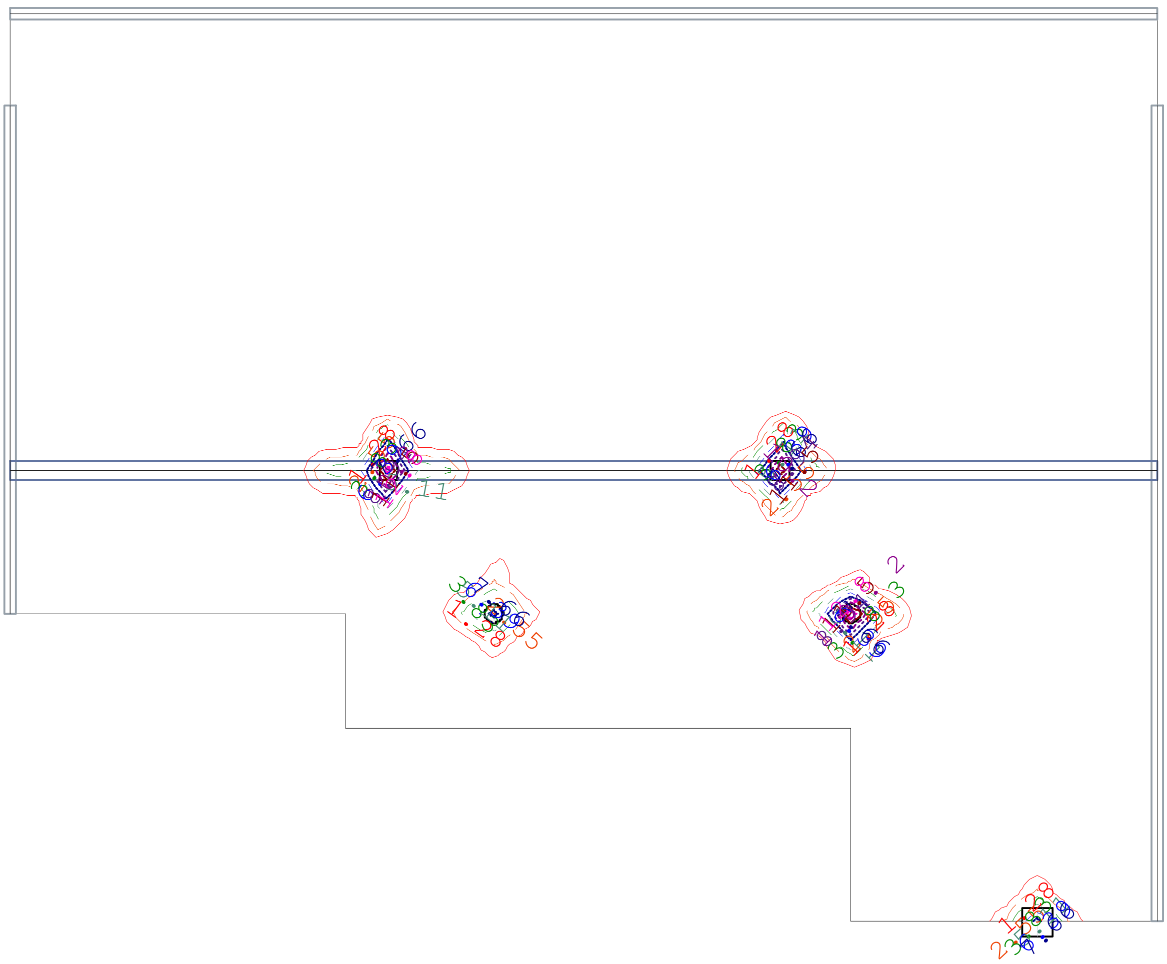
Shear Force q-2z [kN/m]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125

26.07.2018



	1.28
	2.55
	3.83
	5.11
	6.39
	7.66
	8.94
	10.2
	11.5
	12.8

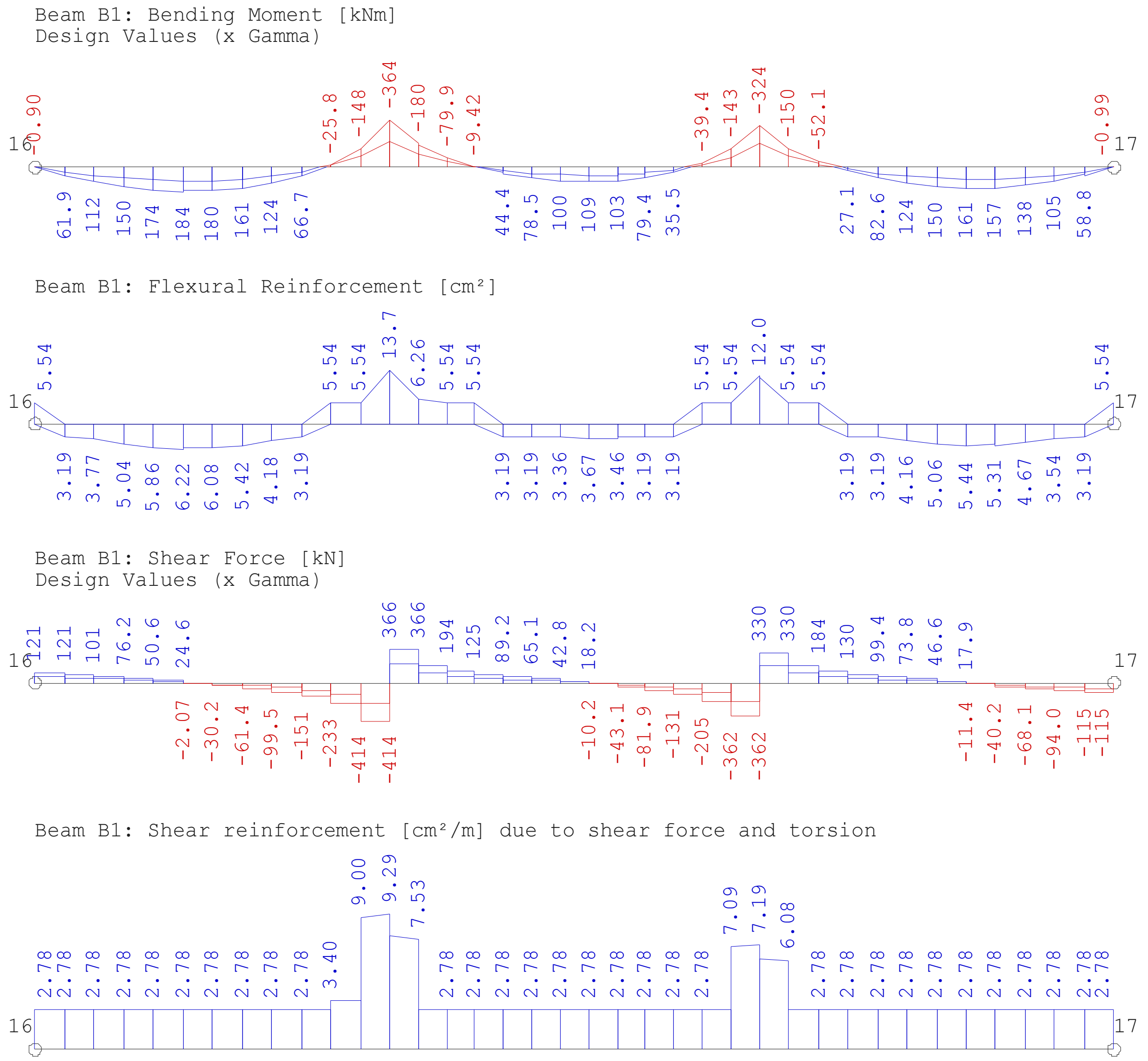
Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"
Shear reinforcement [cm²/m²]

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:
Seite: 24
1 : 125
26.07.2018



Projekt: Design Calculations

Position: S.12 - Semi-Precast Concrete Slab

Superposition 2 "USL Permanent/Transient"

Beam B1

Demo Frilo
Stuttgarter Straße 40
70469 Stuttgart
Tel.: 0711 810020
Email: info@frilo.de

Prj.Nr.:

1 : 125

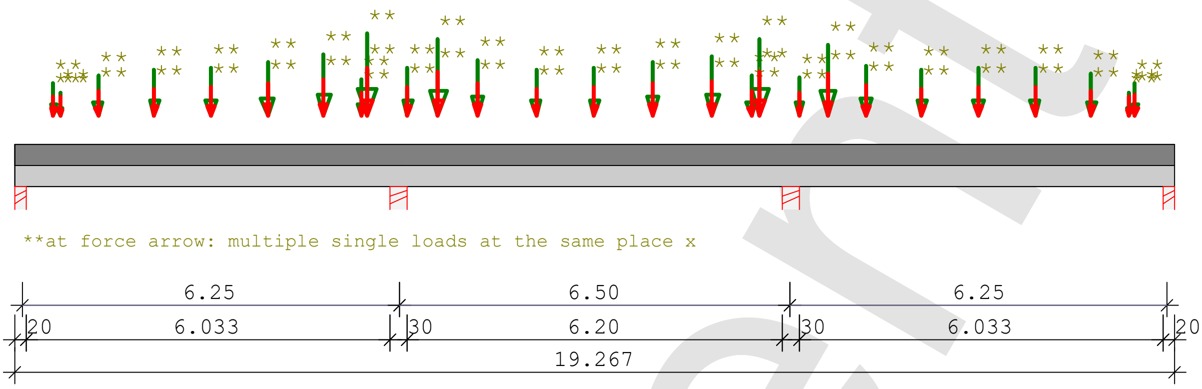
26.07.2018

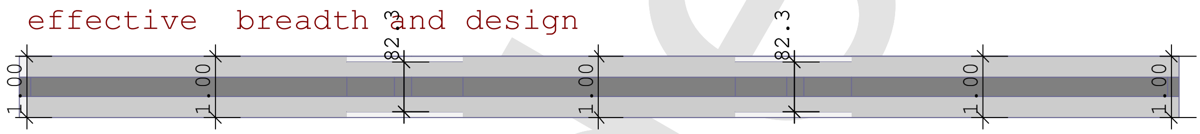
B.1.8. Design Calculations POS. B.13 – Semi-Precast Concrete Beam

Position: B.13 - Semi-Precast Concrete Beam

Durchlaufträger DLT10 01/2018 (Frilo R-2018-1/P12)

Scale 1 : 125





Reinforced concrete girder over 3 Spans C30/37 E = 33000 N/mm2
DIN EN 1992-1-1/NA/A1:2015-12

Decke über NG (Normalgeschoss) von Gebäudemodell

System	length		cross-section values					
Span	l (m)		bt	ht	b0	h0	bb	hb
1	6.25	constant	100.0	35.0	30.0	70.0		
2	6.50	constant	100.0	35.0	30.0	70.0		
3	6.25	constant	100.0	35.0	30.0	70.0		

Cross-sections with eff. active breadth								
x (m)	bt (cm)	ht (cm)	b0 (cm)	h0 (cm)	bb (cm)	hb (cm)	Wyb (m3)	Wyt (m3)
0.00	100.0	35.0	30.0	70.0			0.0327	0.0568
5.31	100.0	35.0	30.0	70.0			0.0327	0.0568
5.31	82.3	35.0	30.0	70.0			0.0311	0.0500
6.25	82.3	35.0	30.0	70.0			0.0311	0.0500
7.22	82.3	35.0	30.0	70.0			0.0311	0.0500
7.23	100.0	35.0	30.0	70.0			0.0327	0.0568
11.77	100.0	35.0	30.0	70.0			0.0327	0.0568
11.78	82.3	35.0	30.0	70.0			0.0311	0.0500
12.75	82.3	35.0	30.0	70.0			0.0311	0.0500
13.69	82.3	35.0	30.0	70.0			0.0311	0.0500
13.69	100.0	35.0	30.0	70.0			0.0327	0.0568
19.00	100.0	35.0	30.0	70.0			0.0327	0.0568

Beam-related loads (kN,m)

Load type (kN,m) : 1=uniform over L 2=concentrated at a
3=single moment at a 4=trapezoidal btw. a, a+b
5=triangular over L 6=trapezoidal over L

Type	EG	Gr	VK	g_l/r	q_l/r	fac.	dist. Lb/Lc	fromItem	Phi
2	A		0.00	11.37	0.00	1.00	0.52	G0_LF1	
2	A		0.00	4.58	0.00	1.00	0.63	G0_LF1	
2	A		0.00	20.40	0.00	1.00	1.25	G0_LF1	
2	A		0.00	26.10	0.00	1.00	2.19	G0_LF1	
2	A		0.00	30.21	0.00	1.00	3.13	G0_LF1	
2	A		0.00	38.41	0.00	1.00	4.06	G0_LF1	
2	A		0.00	52.30	0.00	1.00	5.00	G0_LF1	
2	A		0.00	15.23	0.00	1.00	5.63	G0_LF1	
2	A		0.00	101.28	0.00	1.00	5.73	G0_LF1	
2	A		0.00	29.01	0.00	1.00	6.40	G0_LF1	
2	A		0.00	85.47	0.00	1.00	6.90	G0_LF1	
2	A		0.00	40.81	0.00	1.00	7.55	G0_LF1	
2	A		0.00	27.16	0.00	1.00	8.53	G0_LF1	
2	A		0.00	29.30	0.00	1.00	9.50	G0_LF1	
2	A		0.00	39.59	0.00	1.00	10.48	G0_LF1	
2	A		0.00	50.36	0.00	1.00	11.45	G0_LF1	
2	A		0.00	19.03	0.00	1.00	12.10	G0_LF1	
2	A		0.00	88.31	0.00	1.00	12.25	G0_LF1	
2	A		0.00	17.98	0.00	1.00	12.90	G0_LF1	
2	A		0.00	74.32	0.00	1.00	13.38	G0_LF1	
2	A		0.00	33.38	0.00	1.00	14.00	G0_LF1	
2	A		0.00	26.94	0.00	1.00	14.94	G0_LF1	
2	A		0.00	30.16	0.00	1.00	15.88	G0_LF1	
2	A		0.00	28.71	0.00	1.00	16.81	G0_LF1	
2	A		0.00	21.95	0.00	1.00	17.75	G0_LF1	
2	A		0.00	4.85	0.00	1.00	18.38	G0_LF1	
2	A		0.00	11.75	0.00	1.00	18.48	G0_LF1	
2	B	1	0.00	0.00	3.10	1.00	0.52	G0_LF2	
2	B	1	0.00	0.00	1.30	1.00	0.63	G0_LF2	
2	B	1	0.00	0.00	5.81	1.00	1.25	G0_LF2	
2	B	1	0.00	0.00	7.48	1.00	2.19	G0_LF2	
2	B	1	0.00	0.00	8.70	1.00	3.13	G0_LF2	
2	B	1	0.00	0.00	11.11	1.00	4.06	G0_LF2	
2	B	1	0.00	0.00	15.15	1.00	5.00	G0_LF2	
2	B	1	0.00	0.00	4.41	1.00	5.63	G0_LF2	
2	B	1	0.00	0.00	29.23	1.00	5.73	G0_LF2	
2	B	1	0.00	0.00	8.35	1.00	6.40	G0_LF2	
2	B	1	0.00	0.00	24.58	1.00	6.90	G0_LF2	
2	B	1	0.00	0.00	11.63	1.00	7.55	G0_LF2	
2	B	1	0.00	0.00	7.69	1.00	8.53	G0_LF2	
2	B	1	0.00	0.00	8.41	1.00	9.50	G0_LF2	
2	B	1	0.00	0.00	11.43	1.00	10.48	G0_LF2	
2	B	1	0.00	0.00	14.52	1.00	11.45	G0_LF2	
2	B	1	0.00	0.00	5.48	1.00	12.10	G0_LF2	
2	B	1	0.00	0.00	25.36	1.00	12.25	G0_LF2	
2	B	1	0.00	0.00	5.15	1.00	12.90	G0_LF2	
2	B	1	0.00	0.00	21.28	1.00	13.38	G0_LF2	
2	B	1	0.00	0.00	9.51	1.00	14.00	G0_LF2	
2	B	1	0.00	0.00	7.72	1.00	14.94	G0_LF2	
2	B	1	0.00	0.00	8.73	1.00	15.88	G0_LF2	
2	B	1	0.00	0.00	8.31	1.00	16.81	G0_LF2	
2	B	1	0.00	0.00	6.30	1.00	17.75	G0_LF2	
2	B	1	0.00	0.00	1.38	1.00	18.38	G0_LF2	
2	B	1	0.00	0.00	3.22	1.00	18.48	G0_LF2	
Total				958.96	275.34				

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ
A	1	Cat A - domestic	0.70	0.50	0.30	1.50
B	1	Cat B - offices	0.70	0.50	0.30	1.50

Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3
In following tables the last cell the row is a reference to
the number of the related superposition (see below).
In tables with internal forces multiplied by Gamma is additionally
a reference to the main action.

Results for 1-times loads

Span moments maximum (kNm , kN)

Span				Mf	M le	M ri	V le	V ri	comb
1	x0	=	3.13	129.43	0.00	-253.07	83.98	-302.18	2
2	x0	=	3.25	71.31	-253.07	-224.61	265.40	-261.08	2
3	x0	=	4.06	114.55	-224.61	0.00	241.32	-80.33	2

Support moments maximum (kNm , kN)

Column	M le	M ri	V le	V ri	max F	min F	comb
1	0.00	0.00	0.00	83.98	83.98	65.36	2
2	-253.07	-253.07	-302.18	265.40	567.59	440.74	2
3	-224.61	-224.61	-261.08	241.32	502.39	390.37	2
4	0.00	0.00	-80.33	0.00	80.33	62.48	2

Moment boundary diagram

x/L = Span	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
1	0.00	39.7	70.5	88.7	98.6	100	83.4	54.3	13.2	-77.8	-253
1	0.00	51.0	90.8	114	127	129	108	70.1	17.2	-60.5	-197
2	-253	-99.2	-22.5	15.7	40.0	55.5	51.9	35.4	6.11	-72.0	-225
2	-197	-77.0	-17.4	20.1	51.4	71.3	66.8	45.6	7.92	-55.9	-175
3	-225	-84.8	-8.16	32.4	62.6	84.5	87.4	81.5	66.5	37.8	0.00
3	-175	-65.8	-6.31	41.7	80.6	109	113	105	85.6	48.7	0.00

Support reactions (kN)

Column	by g	max q	min q	Fulload	max	min
1	65.36	18.62	0.00	83.98	83.98	65.36
2	440.74	126.84	0.00	567.59	567.59	440.74
3	390.37	112.03	0.00	502.39	502.39	390.37
4	62.48	17.85	0.00	80.33	80.33	62.48
Total:	958.95	275.34	0.00	1234.30	1234.30	958.95

The system was imported from FEM-calculation
For load transfer do not use the above listed values,
but the bearing loads of FEM-calculation.

Support reactions (kN)

	Column 1		Column 2		Column 3		Column 4	
CA	max	min	max	min	max	min	max	min
g	65.4	65.4	440.7	440.7	390.4	390.4	62.5	62.5
A	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
B	18.6	0.0	126.8	0.0	112.0	0.0	17.9	0.0
tot	84.0	65.4	567.6	440.7	502.4	390.4	80.3	62.5

Results for γ-times loads
Partial safety factor γG * K_{Fi} = 1.35 spanwise constant

Span moments maximum (kNm , kN)

Span				Mfd	Mdle	Mdri	V le	V ri	comb	
1	x0	=	3.13	197.27	0.00	-313.74	121.99	-412.28	B	2
2	x0	=	3.25	126.56	-317.72	-287.33	365.78	-362.58	B	2
3	x0	=	4.06	171.67	-273.05	0.00	327.81	-117.16	B	2

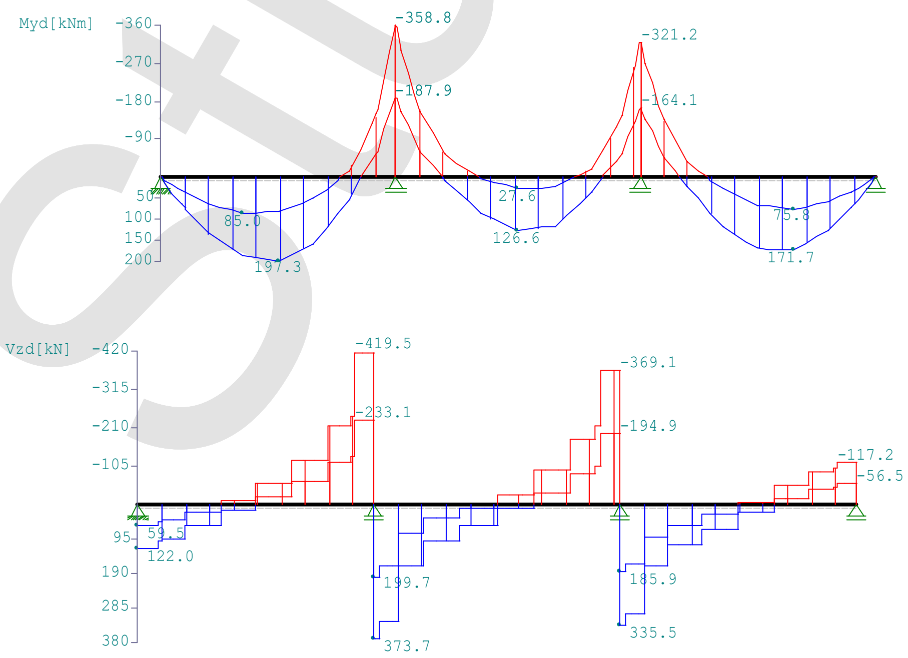
Support moments maximum (kNm , kN)

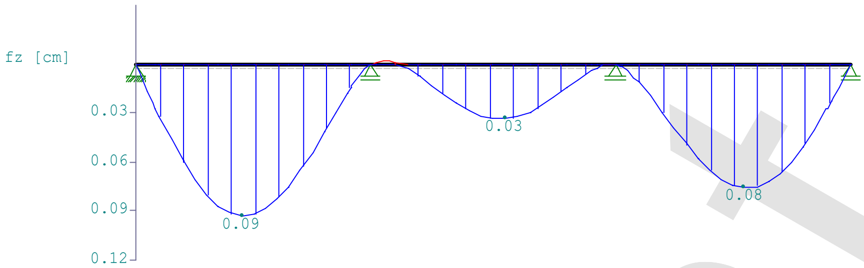
Support	Mdle	Mdri	Vdle	Vdri	max F	min F	comb	
1	0.00	0.00	0.00	121.99	121.99	59.54	B	2
2	-358.75	-358.75	-419.48	373.71	793.19	432.82	B	2
3	-321.19	-321.19	-369.12	335.52	704.63	380.77	B	2
4	0.00	0.00	-117.16	0.00	117.16	56.46	B	2

Moment boundary diagram

x/L = Span	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
1	0.00	36.0	63.3	77.8	84.1	82.3	61.6	28.8	-15.9	-127	-359
1	0.00	74.2	133	169	190	197	171	122	53.0	-40.7	-188
2	-359	-154	-55.9	-14.0	11.2	27.6	24.9	9.32	-19.3	-117	-321
2	-188	-60.0	7.30	57.5	99.8	127	119	89.2	36.3	-38.4	-164
3	-321	-136	-39.3	5.98	40.0	65.6	72.4	70.2	59.0	34.1	0.00
3	-164	-47.1	21.7	84.0	134	169	171	156	126	71.1	0.00

Scale 1 : 200





Design DIN EN 1992-1-1/NA/A1:2015-12
FLBemBn.DLL: Version 9.0.1.121 (1)
C30/37 B500A normaly ductil

Concrete cover:	cv	=	3.0	cm	>=	req. cv							
Reinforcement location:	dt	=	4.0	cm	dB	=	8	dS	=	14			
	Db	=	4.0	cm	dB	=	8	dS	=	14			

In Spans with kx > 0.45 is EN 1992 5.5 (5)to be considered.
Span reinforcement is not curtailed.
The ductility reinforcement by 9.2.1.1 is contained in the required reinforcement.

Creep factor $\phi = 2.58$ $\varepsilon cs = 0.39$ ‰ h0 = 22.50 cm

Support conditions

support	width (cm)	support	type
1	20.0	Concrete	direct
2	30.0	Concrete	direct
3	30.0	Concrete	direct
4	20.0	Concrete	direct

Reduction of the moments of supports <= 15 %

Minimum reinforcement EN2 9.2.1.1 (9.1) fctm = 2.90 N/mm2
Calculating Wy, breadth of slab is limited to 2*b0.

Q.No.	min Mb (kNm)	req As (cm2)	min Mo (kNm)	req As (cm2)	
1	94.80	3.19	-153.75	5.18	90.0/35.0/30.0/70.0

Span reinforcement

Span No.	x (m)	Myd (kNm)	min Myd (kNm)	d (cm)	kx	Asb (cm2)	Ast (cm2)		comb	
1	3.13	197.3		66.0	0.05	6.7	0.0		B	2
	5.73	-150.7	-150.7	66.0	0.09	0.0	5.2		B	2
2	3.25	126.6		66.0	0.04	4.3	0.0		B	2
	0.65	-154.5	-154.5	66.0	0.10	0.0	5.3		B	2
3	4.06	171.7		66.0	0.05	5.8	0.0		B	2
	0.94	-86.1	-86.1	66.0	0.06	0.0	5.2	*	B	2

* Minimum reinforcement acc.to DIN EN 1992-1 9.2.1.1 (1)
On first support are at least 4.2 cm2 to be anchored.
On last support are at least 4.0 cm2 to be anchored.
Shear force VK-support is with F = V,Ed * Cot(Theta) / 2 considered.

Support reinforcement DIN EN 1992:2015 5.5

Column No.	x (m)	Myd (kNm)	des.. Myd (kNm)	d (cm)	kx	Asb (cm2)	Ast (cm2)	comb	
1 ri	0.00	0.0							1
2 le	0.15	-350.1	-236.2	66.0	0.14	0.0	8.4	B	2
2 ri	0.15	-350.1	-244.5	66.0	0.14	0.0	8.7	B	2
3 le	0.15	-310.7	-211.7	66.0	0.12	0.0	7.4	B	2
3 ri	0.15	-310.7	-215.2	66.0	0.13	0.0	7.5	B	2

Support reinforcement DIN EN 1992:2015 5.5

Column No.	x (m)	Myd (kNm)	des.. Myd (kNm)	d (cm)	kx	Asb (cm2)	Ast (cm2)	comb
4 le	0.00	0.0						1

shear force reinforcement B500A DIN EN 1992-1-1/NA/A1:2015-12 6.2

column No.	dist (m)	kz	VEd (kN)	Θ (°)	VRd,c (kN)	VRd,max (kN)	a_max (cm)	asw (cm2/m)		comb
1 ri	0.52	0.90	122.0	18.4	67.0	685.1	30.0	2.8~	B	2
1 ri	0.73	0.90	94.5	18.4	67.0	685.1	30.0	2.8~	B	2
1 *	1.39	0.90	60.0	18.4	67.0	685.1	30.0	2.8~	B	2
2 le	0.52	0.90	-419.5	29.6	71.6	980.7	30.0	9.2	B	2
2 le	0.81	0.90	-213.6	29.6	71.6	980.7	30.0	4.7	B	2
2 *	1.47	0.90	-122.6	29.6	67.0	980.7	30.0	2.8~	B	2
2 ri	0.65	0.90	322.2	26.0	72.6	899.9	30.0	6.1	B	2
2 ri	0.81	0.90	172.2	26.0	72.6	899.9	30.0	3.2	B	2
2 *	1.47	0.90	102.0	26.0	72.6	899.9	30.0	2.8~	B	2
3 le	0.50	0.90	-369.1	28.0	68.8	946.7	30.0	7.6	B	2
3 le	0.81	0.90	-180.2	28.0	68.8	946.7	30.0	3.7	B	2
3 *	1.47	0.90	-93.4	28.0	67.0	946.7	30.0	2.8~	B	2
3 ri	0.62	0.90	303.6	25.0	69.2	875.1	30.0	5.5	B	2
3 ri	0.81	0.90	172.8	25.0	69.2	875.1	30.0	3.1	B	2
3 *	1.47	0.90	114.9	25.0	69.2	875.1	30.0	2.8~	B	2
4 le	0.52	0.90	-117.2	18.4	67.0	685.1	30.0	2.8~	B	2
4 le	0.73	0.90	-88.5	18.4	67.0	685.1	30.0	2.8~	B	2
4 *	1.39	0.90	-51.3	18.4	67.0	685.1	30.0	2.8~	B	2

~ at the end of line: Minimum stirrup reinforcement
max distance of stirrups will with Θ >= 40° investigated (paper DAfStb 525).
reduction of single loads is deactivated

shoulder shear

Span	xa (cm)	xe (cm)	mle (kNm)	mri (kNm)	av (cm)	beff (cm)	dFcd (kN)	vEd (kN/m2)	vEd,perm. (kN/m2)	asf (cm2/m)
1	0	156	0.0	150.8	156	100	89	162	6278	1.1
1	156	313	150.8	197.3	156	100	27	50	6278	0.3
1	313	422	197.3	139.3	110	100	34	89	6278	0.6
1	422	532	139.3	-0.1	110	100	82	214	6278	1.4
2	125	225	1.1	84.0	100	100	49	139	6278	0.9
2	225	325	84.0	126.6	100	100	25	71	6278	0.5
2	325	437	126.6	103.9	112	100	13	34	6278	0.2
2	437	549	103.9	-0.2	112	100	61	156	6278	1.0
3	107	257	0.2	137.8	150	100	81	155	6278	1.0
3	257	406	137.8	171.7	150	100	20	38	6278	0.3
3	406	516	171.7	112.2	109	100	35	91	6278	0.6
3	516	625	112.2	1.2	109	100	65	171	6278	1.1
3	516	625	112.2	1.2	109	100	65	171	6278	1.1
3	516	625	112.2	1.2	109	100	65	171	6278	1.1

At the following table the loads are specified by their internal numeration.
The following table of calculated combinations referenced.
to these numbers

Load type (kN,m) : 1=uniform over L, 2=concentrated at a, 3=single moment at a, 4=trapezoidal btw. a, a+b, 5=triangular over L, 6=trapezoidal over L

No.	span	Type	Grp	g1	q1	g2	q2	factor	distance	length
1	1	2	A 2	11.37	0.00			1.00	0.52	
2		2	A 2	4.58	0.00			1.00	0.63	
3		2	A 2	20.40	0.00			1.00	1.25	
4		2	A 2	26.10	0.00			1.00	2.19	
5		2	A 2	30.21	0.00			1.00	3.13	
6		2	A 2	38.41	0.00			1.00	4.06	

At the following table the loads are specified by their internal numeration.
The following table of calculated combinations referenced.
to these numbers

Load type (kN,m) : 1=uniform over L 2=concentrated at a
3=single moment at a 4=trapezoidal btw. a, a+b
5=triangular over L 6=trapezoidal over L

No.	span	Type	Grp		g1	q1	g2	q2	factor	distance	length
7		2	A	2	52.30	0.00			1.00	5.00	
8		2	A	2	15.23	0.00			1.00	5.63	
9		2	A	2	101.28	0.00			1.00	5.73	
28		2	B	1	0.00	3.10			1.00	0.52	
29		2	B	1	0.00	1.30			1.00	0.63	
30		2	B	1	0.00	5.81			1.00	1.25	
31		2	B	1	0.00	7.48			1.00	2.19	
32		2	B	1	0.00	8.70			1.00	3.13	
33		2	B	1	0.00	11.11			1.00	4.06	
34		2	B	1	0.00	15.15			1.00	5.00	
35		2	B	1	0.00	4.41			1.00	5.63	
36		2	B	1	0.00	29.23			1.00	5.73	
10	2	2	A	2	29.01	0.00			1.00	0.15	
11		2	A	2	85.47	0.00			1.00	0.65	
12		2	A	2	40.81	0.00			1.00	1.30	
13		2	A	2	27.16	0.00			1.00	2.28	
14		2	A	2	29.30	0.00			1.00	3.25	
15		2	A	2	39.59	0.00			1.00	4.23	
16		2	A	2	50.36	0.00			1.00	5.20	
17		2	A	2	19.03	0.00			1.00	5.85	
18		2	A	2	88.31	0.00			1.00	6.00	
37		2	B	1	0.00	8.35			1.00	0.15	
38		2	B	1	0.00	24.58			1.00	0.65	
39		2	B	1	0.00	11.63			1.00	1.30	
40		2	B	1	0.00	7.69			1.00	2.28	
41		2	B	1	0.00	8.41			1.00	3.25	
42		2	B	1	0.00	11.43			1.00	4.23	
43		2	B	1	0.00	14.52			1.00	5.20	
44		2	B	1	0.00	5.48			1.00	5.85	
45		2	B	1	0.00	25.36			1.00	6.00	
19	3	2	A	2	17.98	0.00			1.00	0.15	
20		2	A	2	74.32	0.00			1.00	0.63	
21		2	A	2	33.38	0.00			1.00	1.25	
22		2	A	2	26.94	0.00			1.00	2.19	
23		2	A	2	30.16	0.00			1.00	3.13	
24		2	A	2	28.71	0.00			1.00	4.06	
25		2	A	2	21.95	0.00			1.00	5.00	
26		2	A	2	4.85	0.00			1.00	5.63	
27		2	A	2	11.75	0.00			1.00	5.73	
46		2	B	1	0.00	5.15			1.00	0.15	
47		2	B	1	0.00	21.28			1.00	0.63	
48		2	B	1	0.00	9.51			1.00	1.25	
49		2	B	1	0.00	7.72			1.00	2.19	
50		2	B	1	0.00	8.73			1.00	3.13	
51		2	B	1	0.00	8.31			1.00	4.06	
52		2	B	1	0.00	6.30			1.00	5.00	
53		2	B	1	0.00	1.38			1.00	5.63	
54		2	B	1	0.00	3.22			1.00	5.73	

Calculated combinations from 54 Loads

lc	K1	K2
	g	g
1	.	.
2	.	.
3	.	.
4	.	.
5	.	.
6	.	.
7	.	.
8	.	.
9	.	.
10	.	.
11	.	.
12	.	.
13	.	.
14	.	.
15	.	.
16	.	.
17	.	.
18	.	.
19	.	.
20	.	.
21	.	.
22	.	.
23	.	.
24	.	.
25	.	.
26	.	.
27	.	.
28	.	x
29	.	x
30	.	x
31	.	x
32	.	x
33	.	x
34	.	x
35	.	x
36	.	x
37	.	x
38	.	x
39	.	x
40	.	x
41	.	x
42	.	x
43	.	x
44	.	x
45	.	x
46	.	x
47	.	x
48	.	x
49	.	x
50	.	x
51	.	x
52	.	x
53	.	x
54	.	x

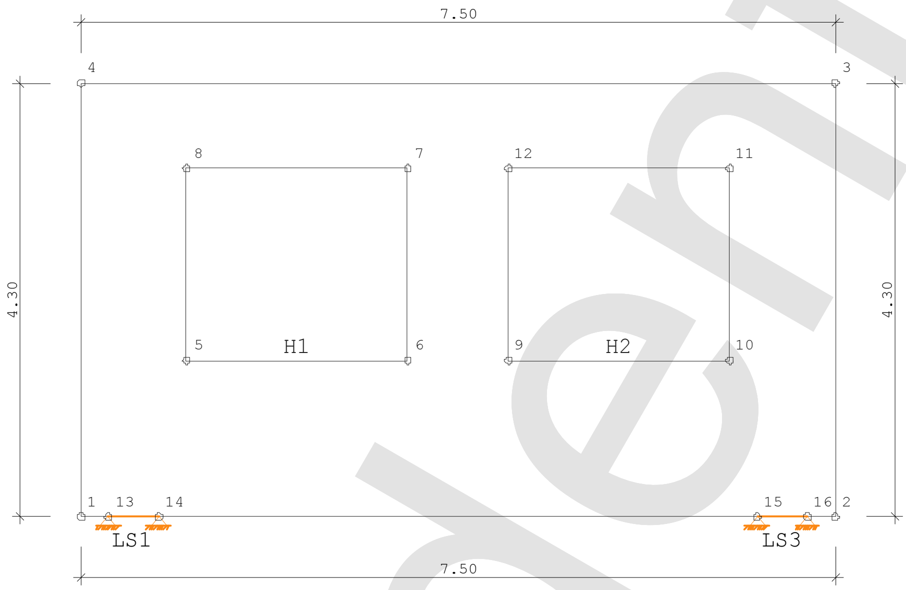
The combinations above will be managed as followed :
Calculating ULS the dead loads will be exceeded
one by one alternating by GammaG = 1,00 / 1,35.
If in one combination live-loads from different actions
exists , then will be investigated, which action is
the dominating one.
The effect of the duration of action will be checked too.

B.1.9. Design Calculations POS. W.14a – Load Bearing Concrete Wall

Position: W.14a - Load Bearing Concrete Wall_Single

Panels by Finite Elements SCN 01/2018 (Frilo R-2018-1/P12)

System
Scale 1 : 75



LOADCASE 1 "POS. S.11 - Dead Load"

Type:	permanent
Self-weight of the slab is included:	YES
Action:	Ständige Lasten
Partial safety factor:	1.35
Load points:	6
Point loads:	6

Forces, vertical

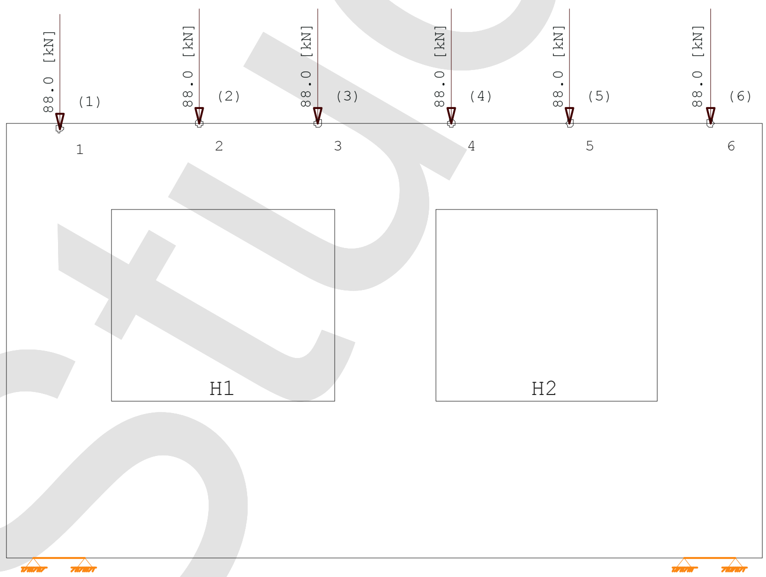
Total of input loads:	528
(portion on the slab)	
Self-weight of the slab:	119
Total of all loads:	647
Total of all support reactions:	647

Forces, horizontal

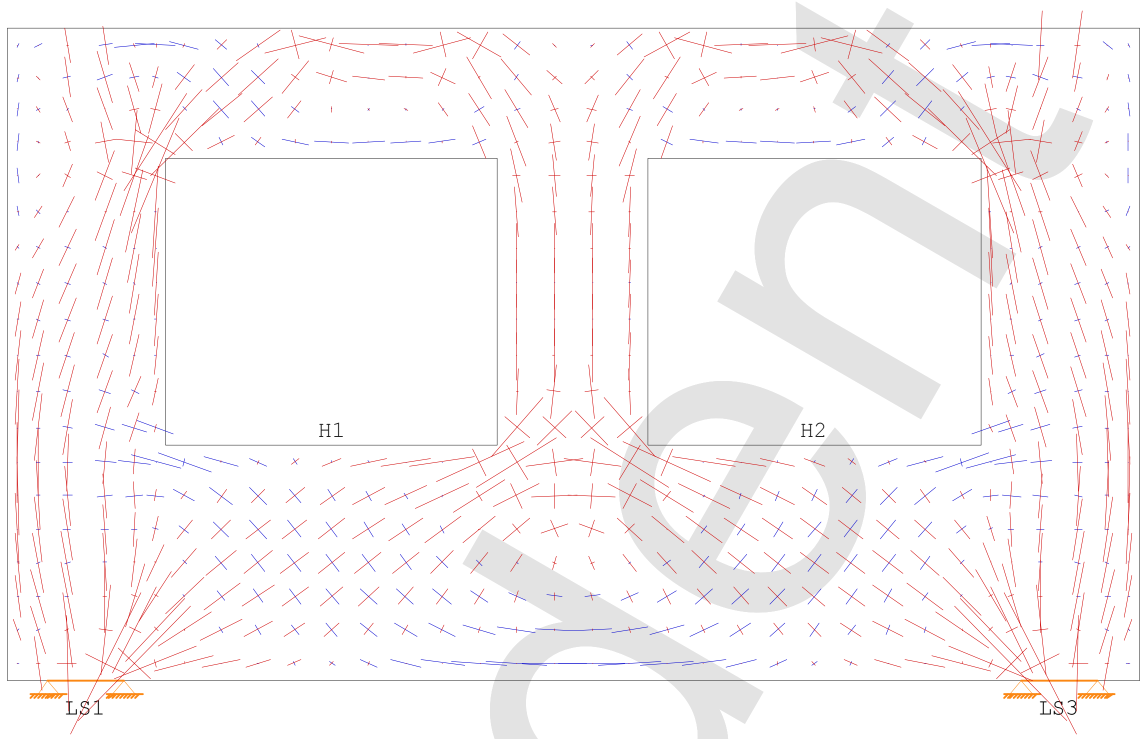
Total of input loads:	0
(portion on the slab)	
Total of all support reactions:	0

NOTE
All effects of actions (like internal forces, support reactions, displacements, etc.) of an individual load case are, unlike the results of a superposition of load cases, simple, i.e. characteristic, values
.
Design results are based on design quantities including the partial safety coefficients.

Load case 1 "POS. S.11 - Dead Load"
Point loads
Scale 1 : 75



Load case 1 "POS. S.11 - Dead Load"
Principal Forces [kN/m]
Scale 1 : 50



LOADCASE 2 "POS. S.11 - Live Load"

Type:	non permanent
Self-weight of the slab is included:	NO
Action:	Wohnräume
Partial safety factor:	1.50
Load points:	6
Point loads:	6

Forces, vertical

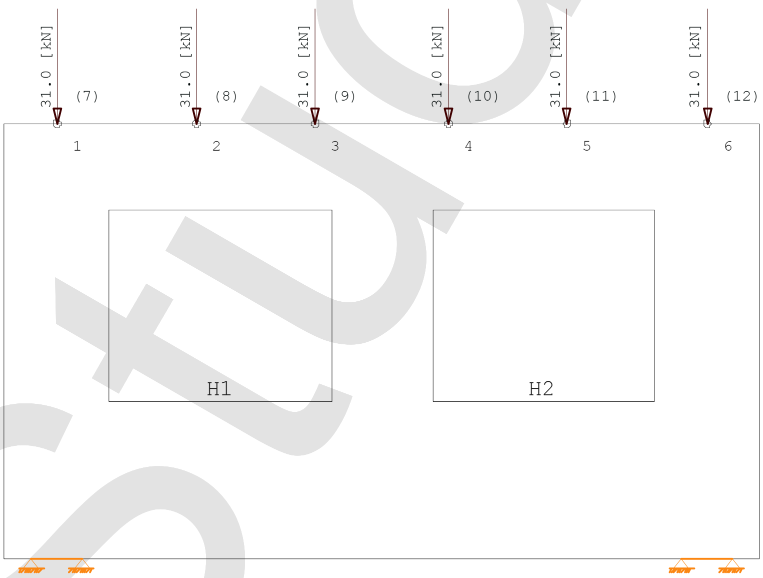
Total of input loads: (portion on the slab)	186
Total of all support reactions:	186

Forces, horizontal

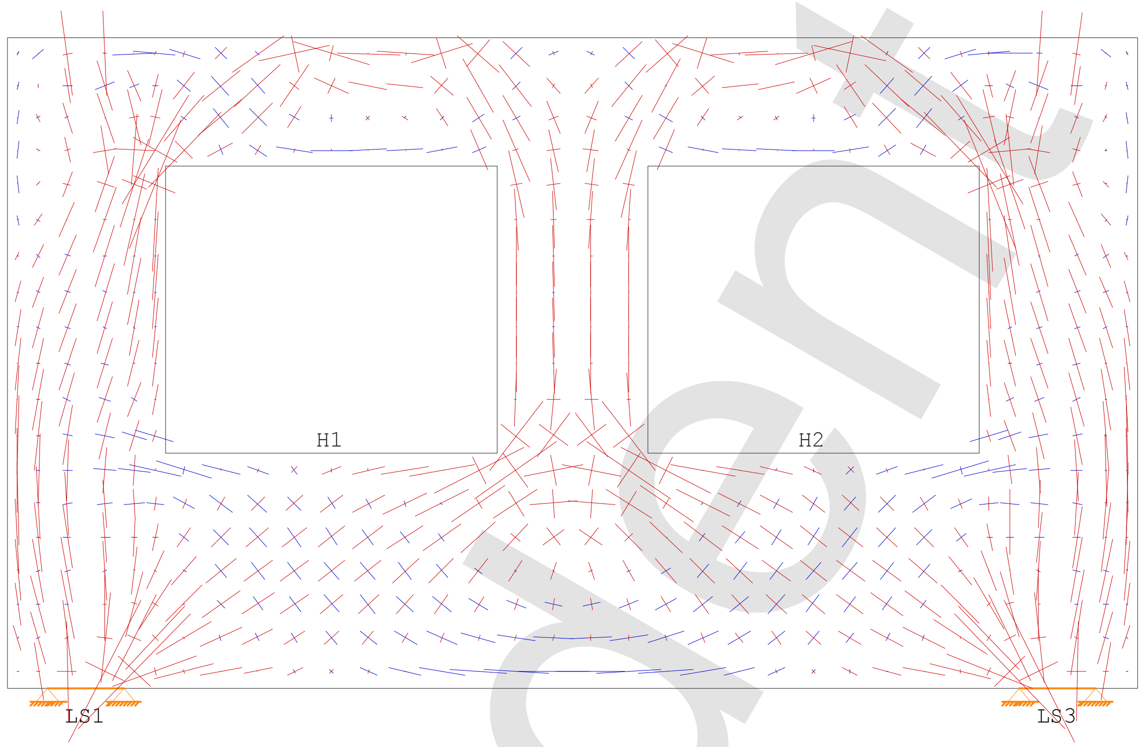
Total of input loads: (portion on the slab)	0
Total of all support reactions:	0

NOTE
All effects of actions (like internal forces, support reactions, displacements, etc.) of an individual load case are, unlike the results of a superposition of load cases, simple, i.e. characteristic, values
.
Design results are based on design quantities including the partial safety coefficients.

Load case 2 "POS. S.11 - Live Load"
Point loads
Scale 1 : 75



Load case 2 "POS. S.11 - Live Load"
Principal Forces [kN/m]
Scale 1 : 50



SUPERPOSITION 1 "Characteristic"

Load Cases Involved

Number	Load case	Type	Self Self-Weight	Action Shrt Name Hnd	Alter-native Group
1	POS. S.11 - Dea...	permanent	yes	g Ständige ...	-
2	POS. S.11 - Liv...	non perm	no	1 Wohnräume	0

Action

Number	Shrt Name Hnd	Type
1	g Ständige Lasten	permanent
2	1 Wohnräume	non perm

SUPERPOSITION 2 "ULS Permanent/Transient"

Load Cases Involved

Number	Load case	Type	Self Self-Weight	Action Shrt Name Hnd	Alter-native Group
1	POS. S.11 - Dea...	permanent	yes	g Ständige ...	-
2	POS. S.11 - Liv...	non perm	no	1 Wohnräume	0

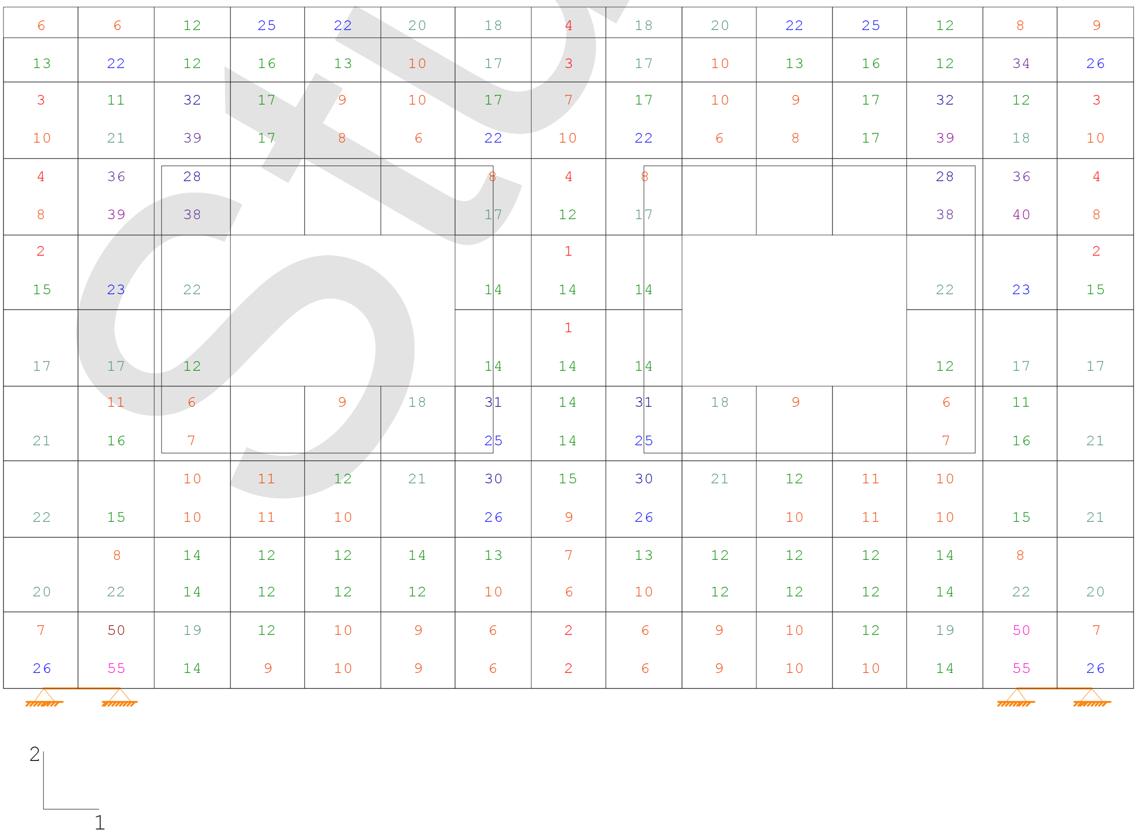
Action

Number	Shrt Name Hnd	Type	Partial Safety sup	inf	Combination dom	ndo
1	g Ständige Lasten	permanent	1.35	1.00	1.00	1.00
2	1 Wohnräume	non perm	1.50	0.00	1.00	0.70

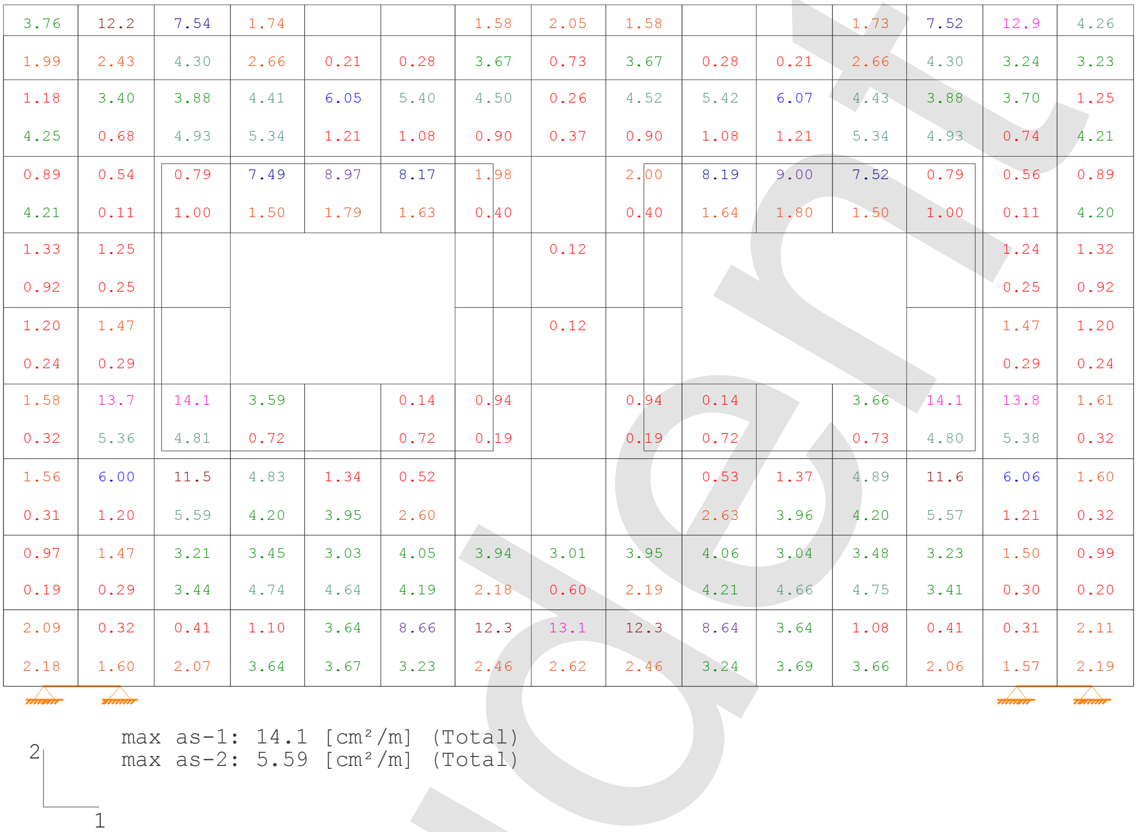
NOTE: Design Values
All results of a superposition of load cases
Include both partial safety and combination
factors: DIN EN 1990/NA:2010-12

NOTE: Combination Factors
With the combination of independent, variable actions
the individual dominant action is determined for both each
location and for each action quantity.
In geneneral, the dominant actions differ with each
location and each quantity
.
The individually found dominant action receives the
conbination factor 1.00. In case of only one variable action
this action is considered dominant.

Superposition 2 "ULS Permanent/Transient"
Concrete Load (Compressive) uC-1, uC-2 [%]
Scale 1 : 50



Superposition 2 "ULS Permanent/Transient"
Reinforcement, Total aS-1, aS-2 [cm²/m] Total
Scale 1 : 50

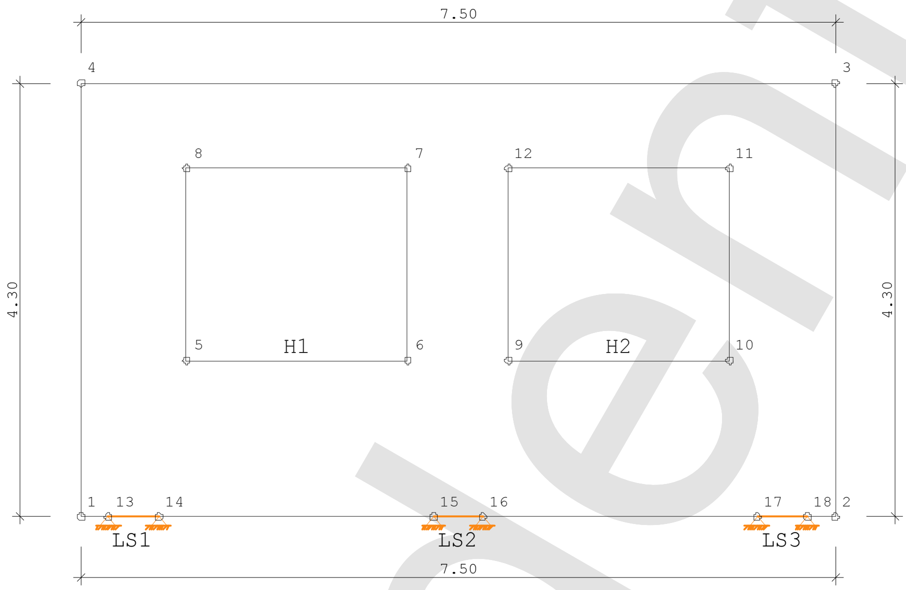


max as-1: 14.1 [cm²/m] (Total)
max as-2: 5.59 [cm²/m] (Total)

Position: W.14a - Load Bearing Concrete Wall_Double

Panels by Finite Elements SCN 01/2018 (Frilo R-2018-1/P12)

System
Scale 1 : 75



LOADCASE 1 "POS. S.11 - Dead Load"

Type:	permanent
Self-weight of the slab is included:	YES
Action:	Ständige Lasten
Partial safety factor:	1.35
Load points:	6
Point loads:	6

Forces, vertical

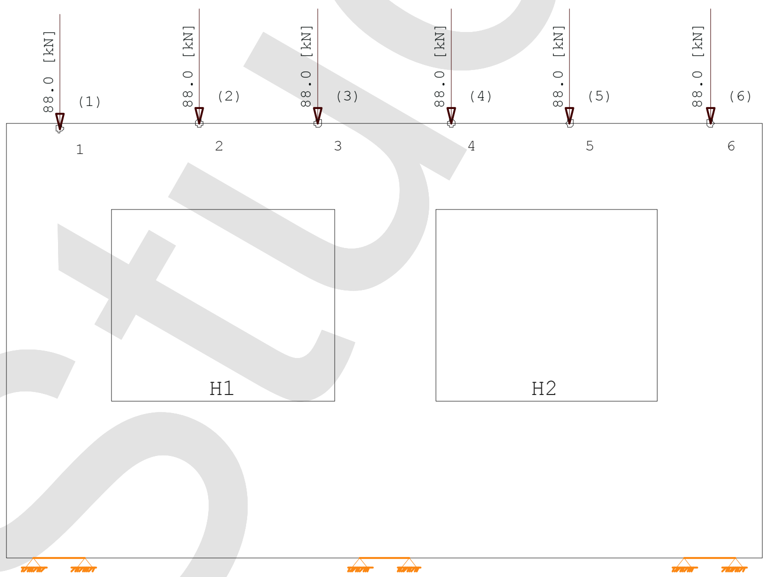
Total of input loads:	528
(portion on the slab)	
Self-weight of the slab:	119
Total of all loads:	647
Total of all support reactions:	647

Forces, horizontal

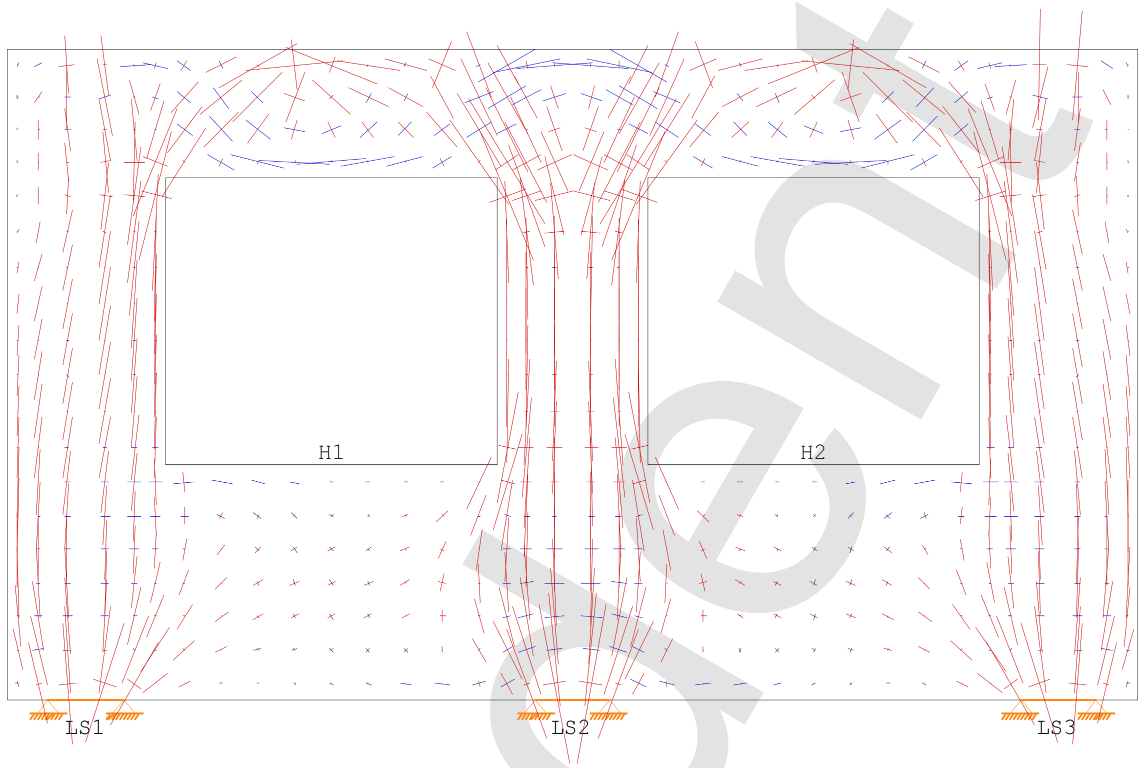
Total of input loads:	0
(portion on the slab)	
Total of all support reactions:	0

NOTE
All effects of actions (like internal forces, support reactions, displacements, etc.) of an individual load case are, unlike the results of a superposition of load cases, simple, i.e. characteristic, values
.
Design results are based on design quantities including the partial safety coefficients.

Load case 1 "POS. S.11 - Dead Load"
Point loads
Scale 1 : 75



Load case 1 "POS. S.11 - Dead Load"
Principal Forces [kN/m]
Scale 1 : 50



LOADCASE 2 "POS. S.11 - Live Load"

Type:	non permanent
Self-weight of the slab is included:	NO
Action:	Wohnräume
Partial safety factor:	1.50
Load points:	6
Point loads:	6

Forces, vertical

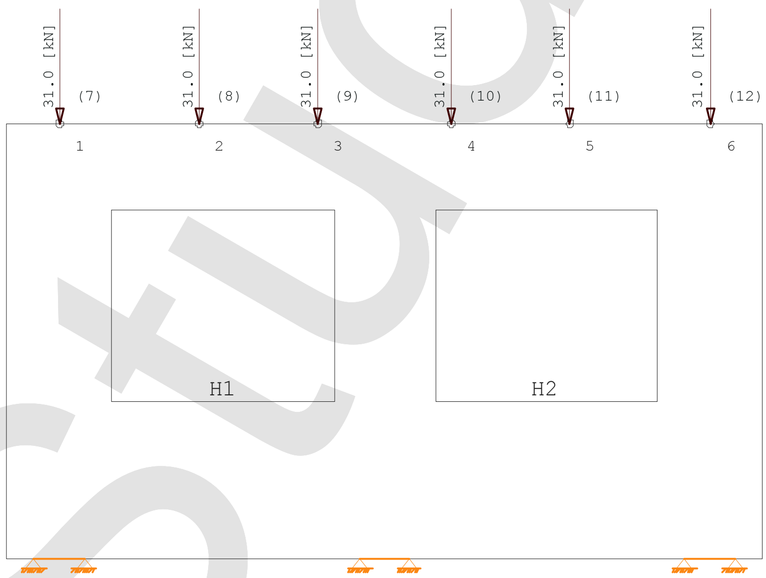
Total of input loads: (portion on the slab)	186
Total of all support reactions:	186

Forces, horizontal

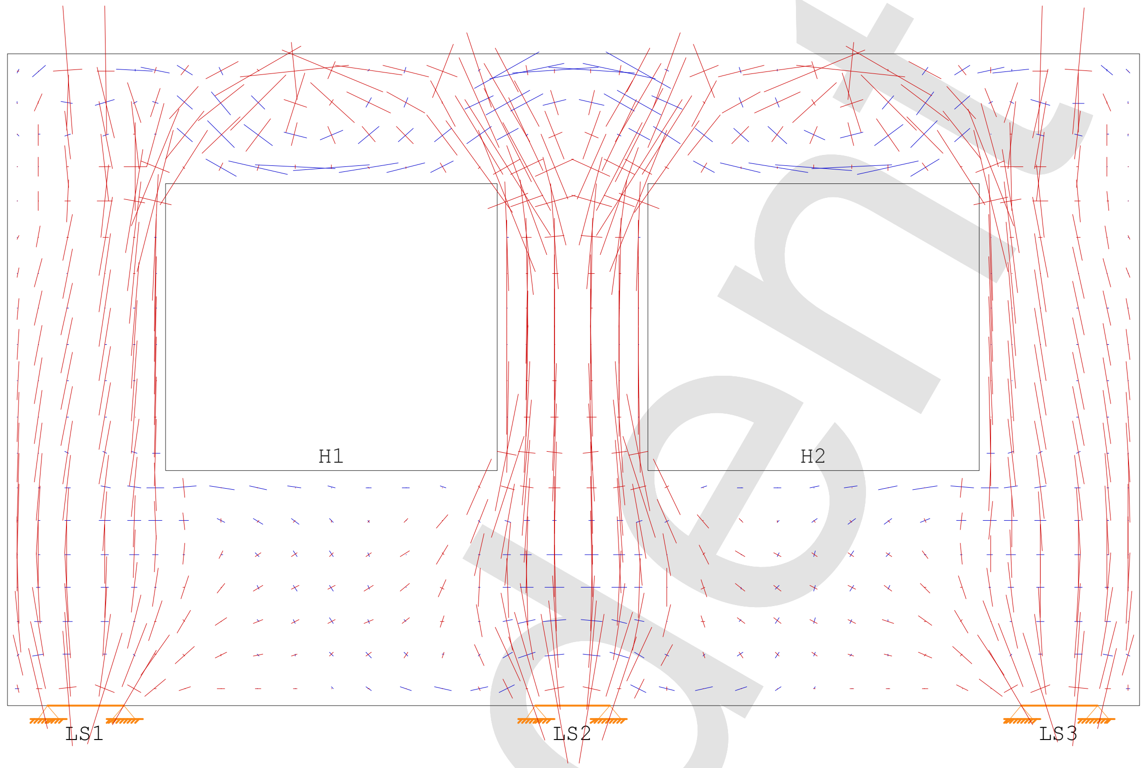
Total of input loads: (portion on the slab)	0
Total of all support reactions:	0

NOTE
All effects of actions (like internal forces, support reactions, displacements, etc.) of an individual load case are, unlike the results of a superposition of load cases, simple, i.e. characteristic, values
.
Design results are based on design quantities including the partial safety coefficients.

Load case 2 "POS. S.11 - Live Load"
Point loads
Scale 1 : 75



Load case 2 "POS. S.11 - Live Load"
Principal Forces [kN/m]
Scale 1 : 50



SUPERPOSITION 1 "Characteristic"

Load Cases Involved					
Number	Load case	Type	Self Self-Weight	Action Shrt Name Hnd	Alter-native Group
1	POS. S.11 - Dea...	permanent	yes	g Ständige ...	-
2	POS. S.11 - Liv...	non perm	no	1 Wohnräume	0

Action		
Number	Shrt Name Hnd	Type
1	1 Wohnräume	non perm
2	g Ständige Lasten	permanent

SUPERPOSITION 2 "ULS Permanent/Transient"

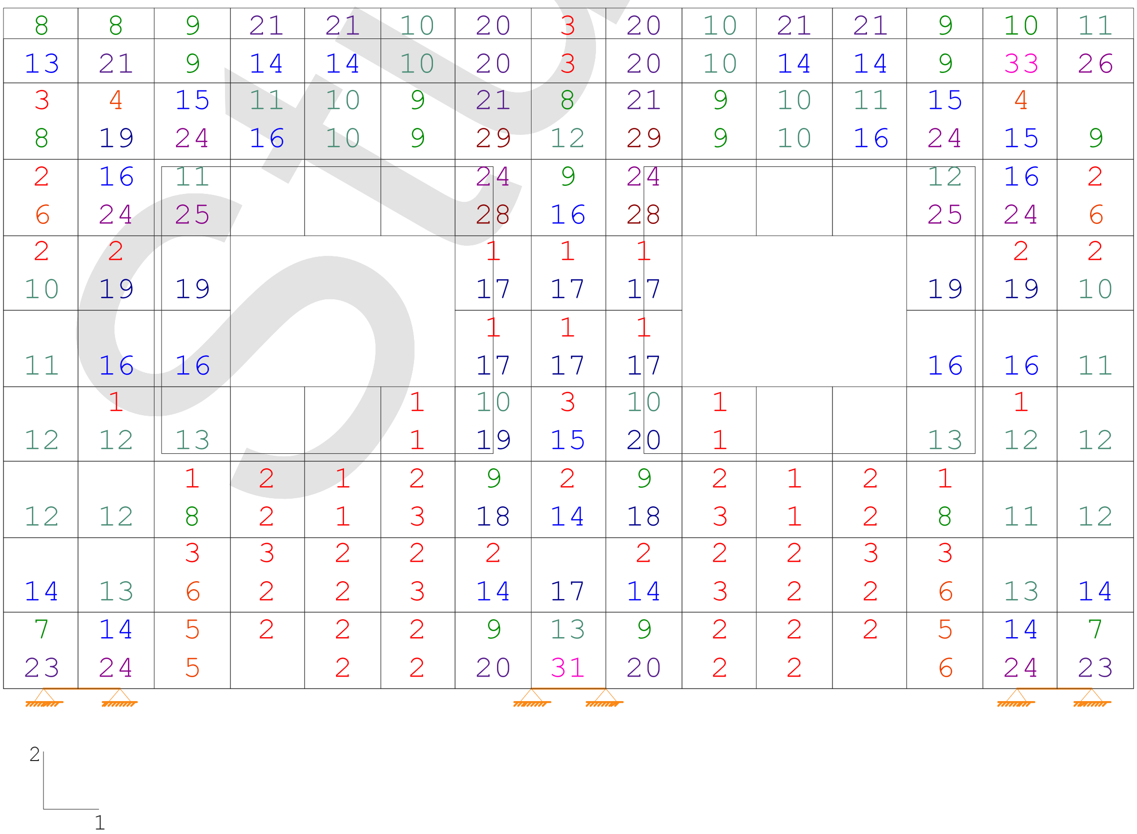
Load Cases Involved					
Number	Load case	Type	Self Self-Weight	Action Shrt Name Hnd	Alter-native Group
1	POS. S.11 - Dea...	permanent	yes	g Ständige ...	-
2	POS. S.11 - Liv...	non perm	no	1 Wohnräume	0

Action						
Number	Shrt Name Hnd	Type	Partial Safety sup	inf	Combination dom	ndo
1	1 Wohnräume	non perm	1.50	0.00	1.00	0.70
2	g Ständige Lasten	permanent	1.35	1.00	1.00	1.00

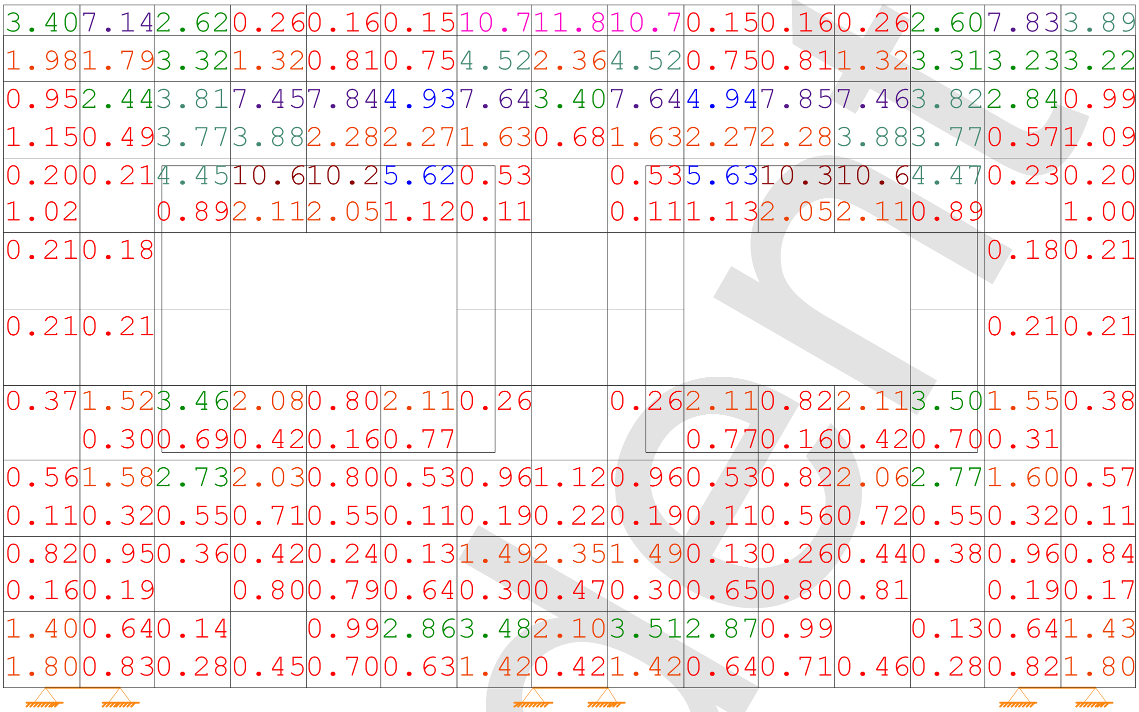
NOTE: Design Values
All results of a superposition of load cases
Include both partial safety and combination
factors: DIN EN 1990/NA:2010-12

NOTE: Combination Factors
With the combination of independent, variable actions
the individual dominant action is determined for both each
location and for each action quantity.
In geneneral, the dominant actions differ with each
location and each quantity
.
The individually found dominant action receives the
conbination factor 1.00. In case of only one variable action
this action is considered dominant.

Superposition 2 "ULS Permanent/Transient"
Concrete Load (Compressive) uC-1, uC-2 [%]
Scale 1 : 50



Superposition 2 "ULS Permanent/Transient"
Reinforcement, Total aS-1, aS-2 [cm²/m] Total
Scale 1 : 50



max as-1: 11.8 [cm²/m] (Total)
max as-2: 4.52 [cm²/m] (Total)

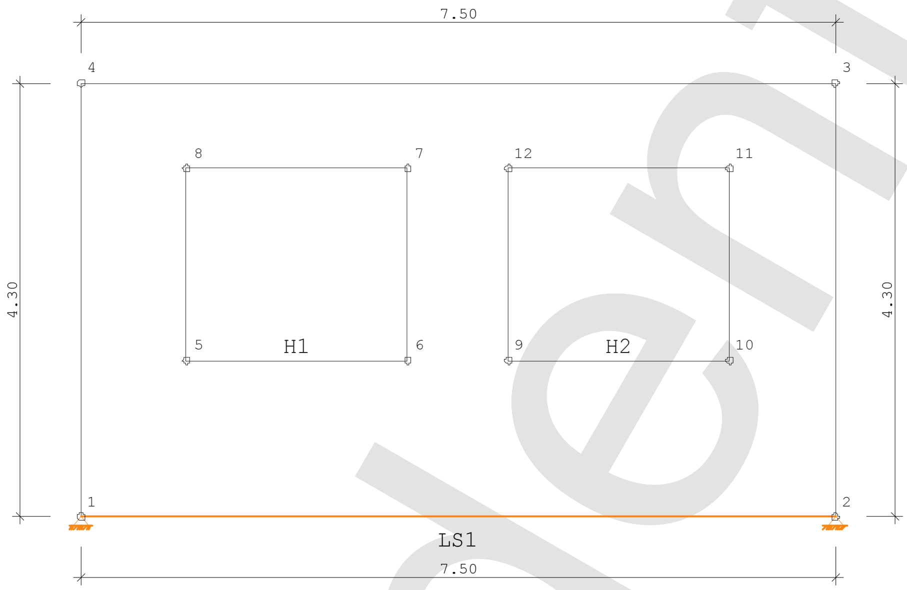
2

1

Position: W.14a - Load Bearing Concrete Wall_Contin

Panels by Finite Elements SCN 01/2018 (Frilo R-2018-1/P12)

System
Scale 1 : 75



LOADCASE 1 "POS. S.11 - Dead Load"

Type:	permanent
Self-weight of the slab is included:	YES
Action:	Ständige Lasten
Partial safety factor:	1.35
Load points:	6
Point loads:	6

Forces, vertical

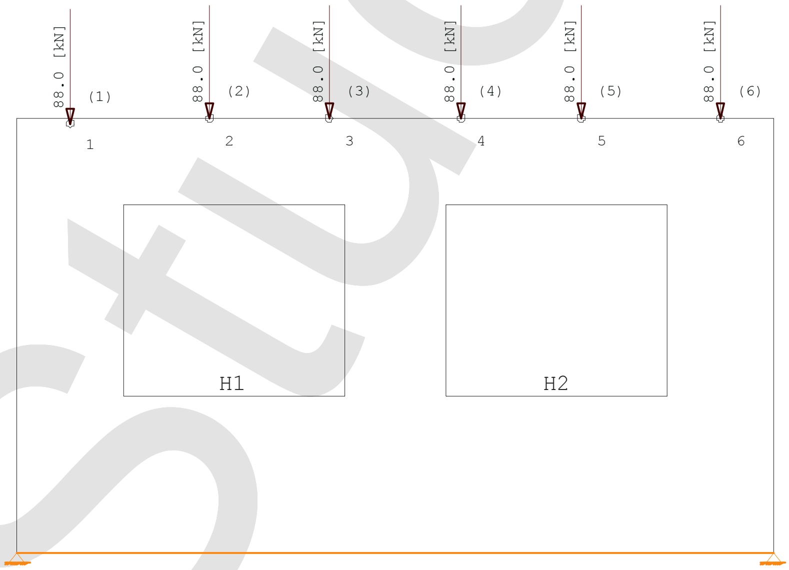
Total of input loads: (portion on the slab)	528
Self-weight of the slab:	119
Total of all loads:	647
Total of all support reactions:	647

Forces, horizontal

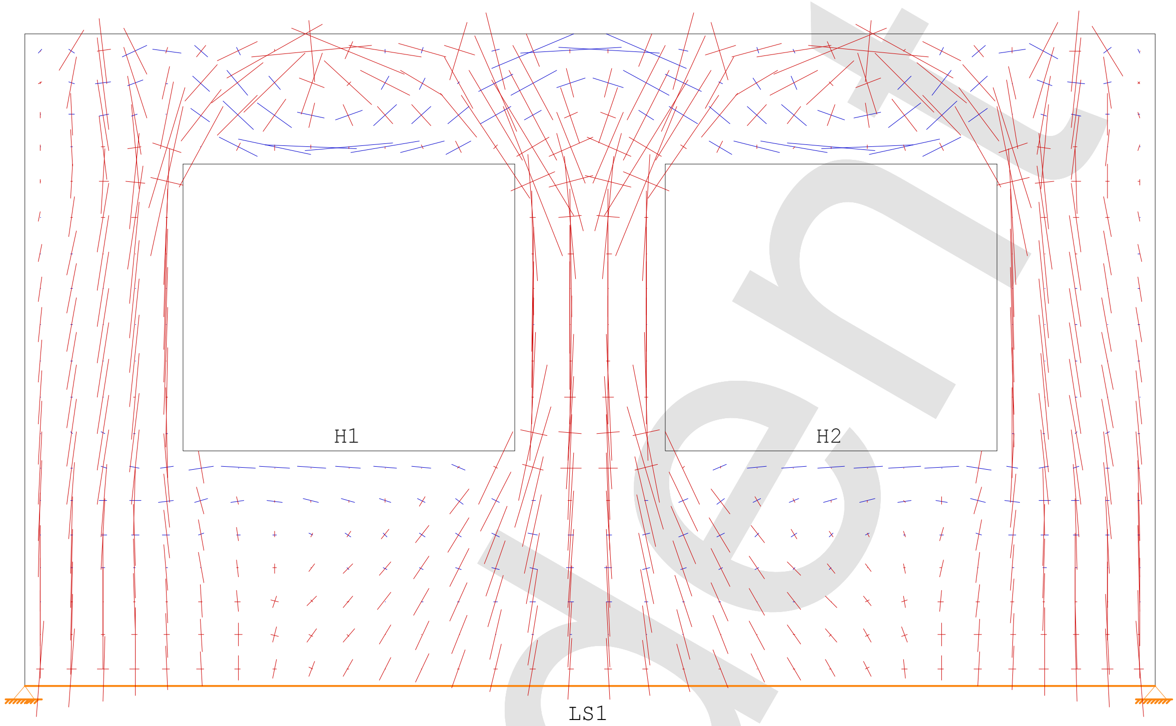
Total of input loads: (portion on the slab)	0
Total of all support reactions:	0

NOTE
All effects of actions (like internal forces, support reactions, displacements, etc.) of an individual load case are, unlike the results of a superposition of load cases, simple, i.e. characteristic, values
.
Design results are based on design quantities including the partial safety coefficients.

Load case 1 "POS. S.11 - Dead Load"
Point loads
Scale 1 : 75



Load case 1 "POS. S.11 - Dead Load"
Principal Forces [kN/m]
Scale 1 : 50



LOADCASE 2 "POS. S.11 - Live Load"

Type:	non permanent
Self-weight of the slab is included:	NO
Action:	Wohnräume
Partial safety factor:	1.50
Load points:	6
Point loads:	6

Forces, vertical

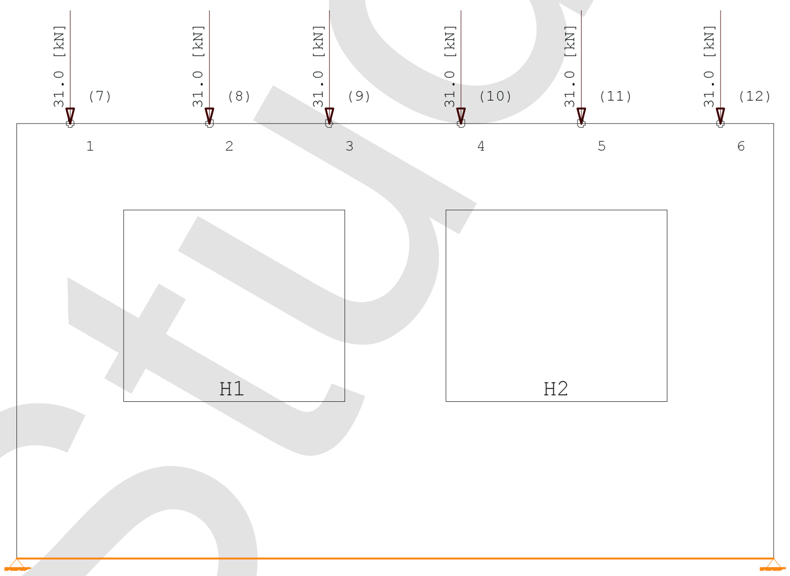
Total of input loads: (portion on the slab)	186
Total of all support reactions:	186

Forces, horizontal

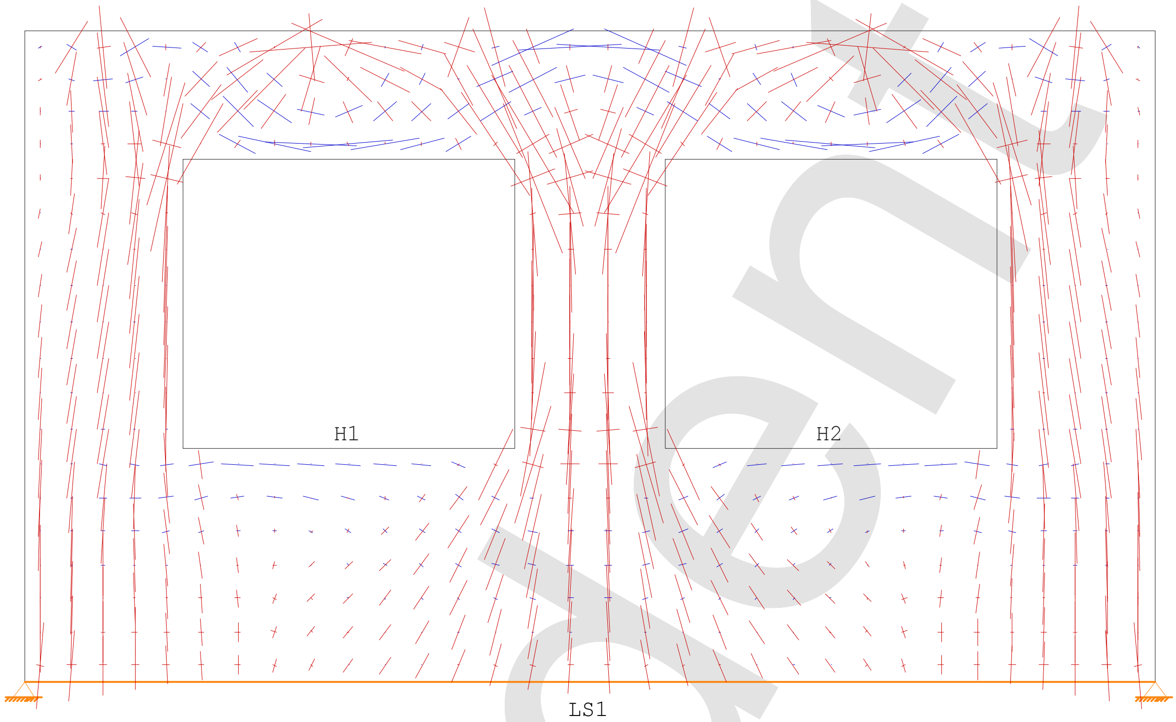
Total of input loads: (portion on the slab)	0
Total of all support reactions:	0

NOTE
All effects of actions (like internal forces, support reactions, displacements, etc.) of an individual load case are, unlike the results of a superposition of load cases, simple, i.e. characteristic, values
.
Design results are based on design quantities including the partial safety coefficients.

Load case 2 "POS. S.11 - Live Load"
Point loads
Scale 1 : 75



Load case 2 "POS. S.11 - Live Load"
Principal Forces [kN/m]
Scale 1 : 50



SUPERPOSITION 1 "Characteristic"

Load Cases Involved

Number	Load case	Type	Self Self-Weight	Action Shrt Name Hnd	Alter-native Group
1	POS. S.11 - Dea...	permanent	yes	g Ständige ...	-
2	POS. S.11 - Liv...	non perm	no	1 Wohnräume	0

Action

Number	Shrt Name Hnd	Type
1	g Ständige Lasten	permanent
2	1 Wohnräume	non perm

SUPERPOSITION 2 "ULS Permanent/Transient"

Load Cases Involved

Number	Load case	Type	Self Self-Weight	Action Shrt Name Hnd	Alter-native Group
1	POS. S.11 - Dea...	permanent	yes	g Ständige ...	-
2	POS. S.11 - Liv...	non perm	no	1 Wohnräume	0

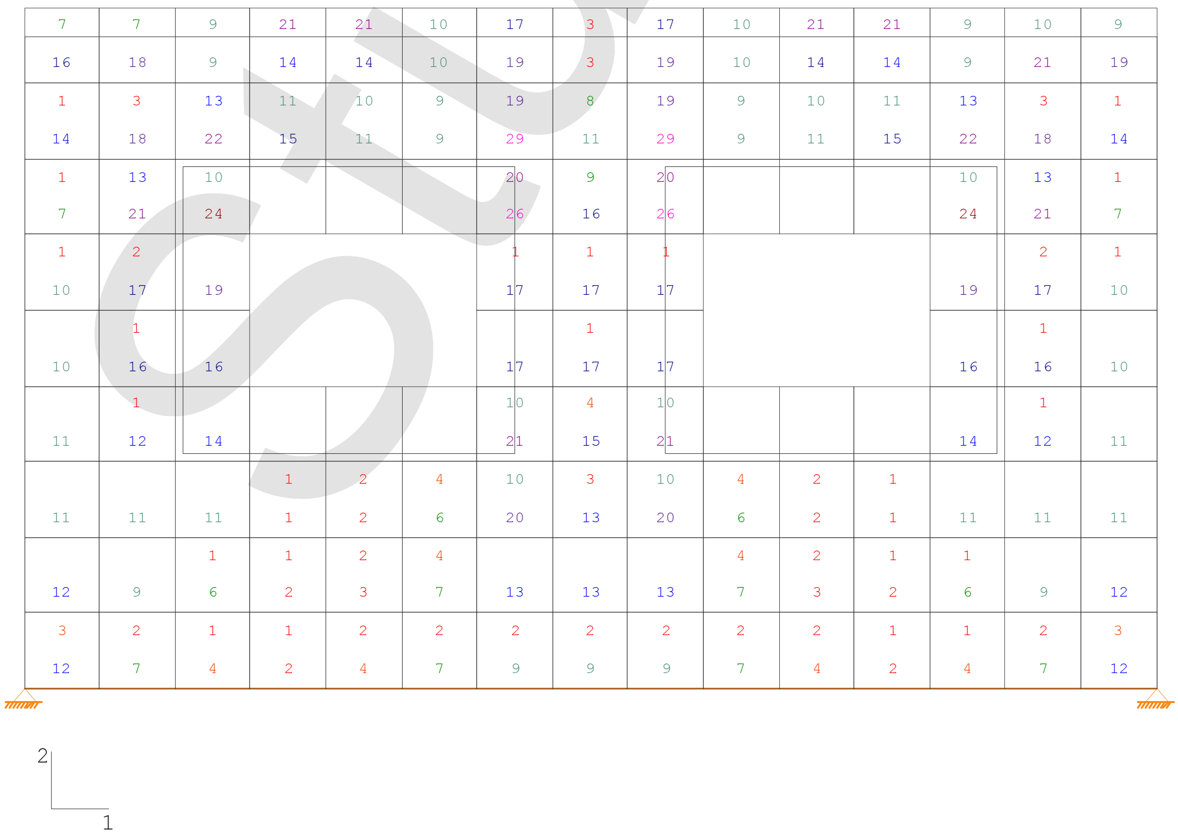
Action

Number	Shrt Name Hnd	Type	Partial Safety sup	inf	Combination dom	ndo
1	g Ständige Lasten	permanent	1.35	1.00	1.00	1.00
2	1 Wohnräume	non perm	1.50	0.00	1.00	0.70

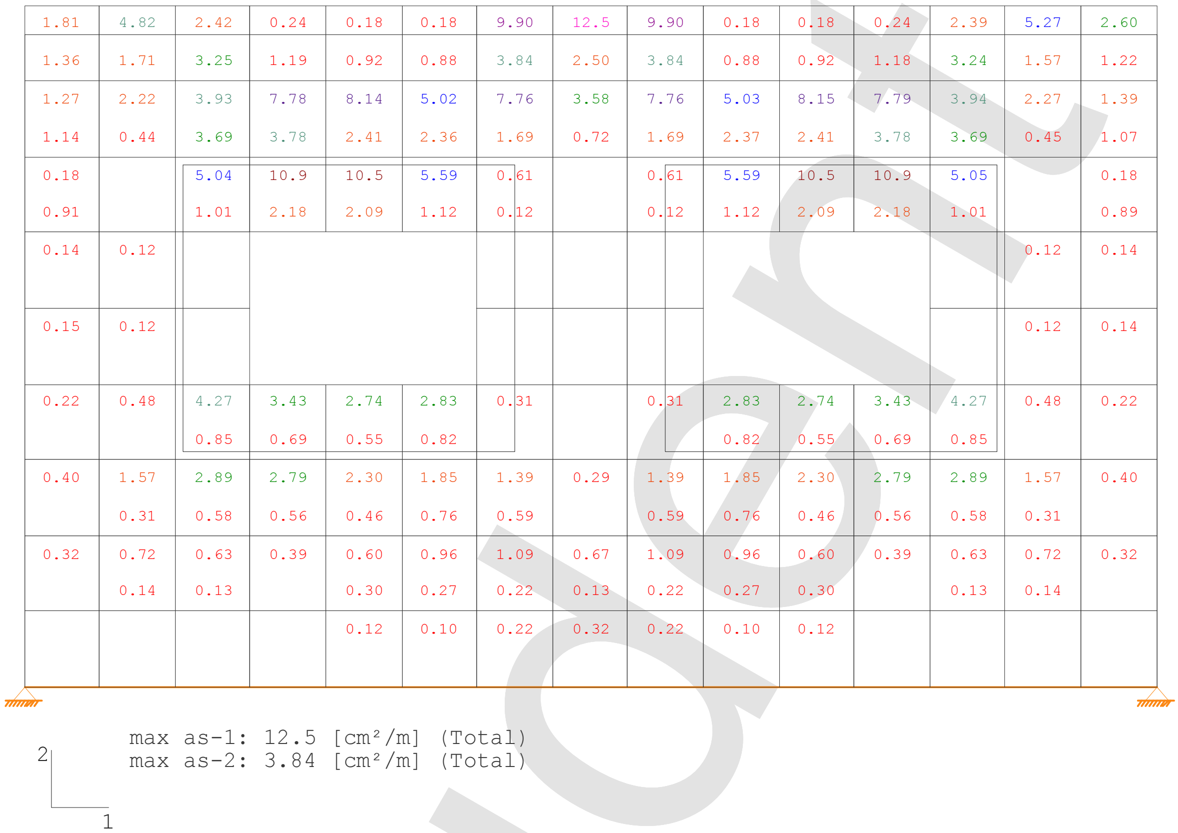
NOTE: Design Values
All results of a superposition of load cases
Include both partial safety and combination
factors: DIN EN 1990/NA:2010-12

NOTE: Combination Factors
With the combination of independent, variable actions
the individual dominant action is determined for both each
location and for each action quantity.
In geneneral, the dominant actions differ with each
location and each quantity
.
The individually found dominant action receives the
conbination factor 1.00. In case of only one variable action
this action is considered dominant.

Superposition 2 "ULS Permanent/Transient"
Concrete Load (Compressive) uC-1, uC-2 [%]
Scale 1 : 50



Superposition 2 "ULS Permanent/Transient"
Reinforcement, Total aS-1, aS-2 [cm²/m] Total
Scale 1 : 50

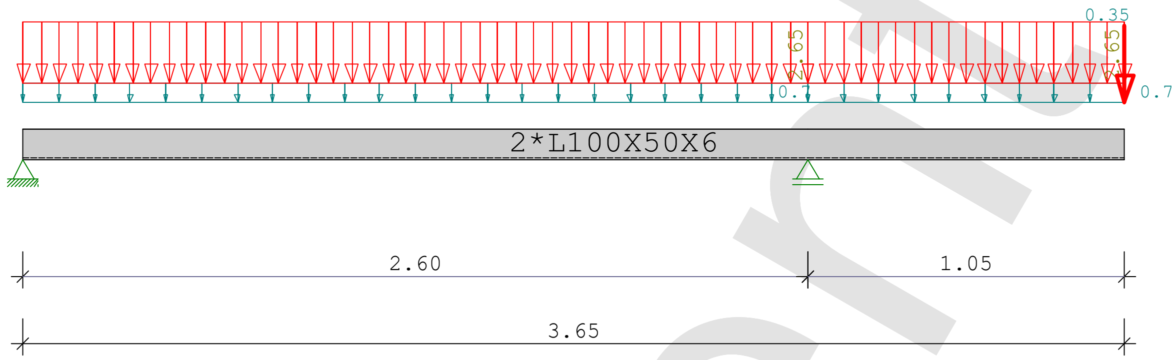


B.1.10. Design Calculations POS. V.31 – Steel Canopy

Position: V.31.1 - Steel Canopy

Durchlaufträger DLT10 01/2018 (Frilo R-2018-1/P12)

Scale 1 : 25



Steel girder S235 DIN EN 1993-1-1/NA:2010-12 E-modulus E =210000 N/mm2

System	length		cross-section values				
Span	L (m)		CsNo.	I (cm4)	St (cm3)	Sb (cm3)	
1	2.600	constant	1	179.4	27.6	2	L100X50X6
Cantilever right	1.050	constant	1	179.4	27.6	2	L100X50X6

Load type (kN,m) : 1=uniform over L 2=concentrated at a
3=single moment at a 4=trapezoidal btw. a, a+b
5=triangular over L 6=trapezoidal over L

Span	Type	AG	G	r	g_l/r	q_l/r	factor	distanc	e length	fromItem	Phi
1	1	J			0.700	2.650	1.000				
Cantilever CaRi	1	J			0.700	2.650	1.000				
	2	J			0.000	0.350	1.000	1.050			

Dead load of girder is considered with Gamma = 78.5 kN/m3 .

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ
J	3	Snow loads <1000m	0.50	0.20	0.00	1.50

Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3
In following tables the last cell the row is a reference to
the number of the related superposition (see below).
In tables with internal forces multiplied by Gamma is additionally
a reference to the main action.

Results for 1-times loads

Span moments maximum (kNm , kN)

Span		Mf	M le	M ri	V le	V ri	comb
1	x0 = 1.250	2.72	0.00	-0.46	4.36	-4.71	2

Support moments maximum							(kNm , kN)
Column	M le	M ri	V le	V ri	max F	min F	comb
1	0.00	0.00	0.00	4.36	4.36	0.21	2
2	-2.29	-2.29	-5.41	4.01	9.43	2.14	3

Support reactions						(kN)
Column	by g	max q	min q	Fulload	max	min
1	0.91	3.45	-0.70	3.65	4.36	0.21
2	2.14	7.28	0.00	9.43	9.43	2.14
Total:	3.06	10.73	-0.70	13.08	13.78	2.35

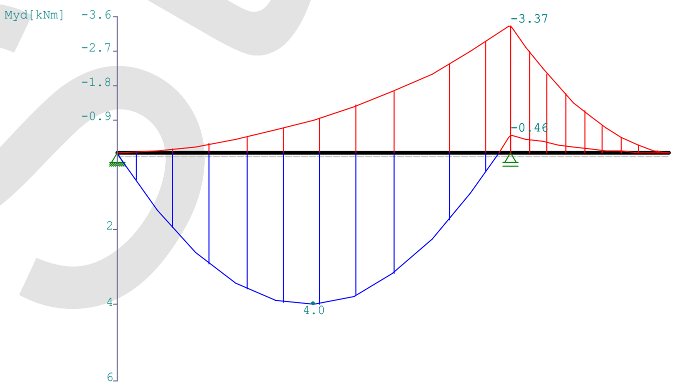
Support reactions				(kN)
	Column 1		Column 2	
CA	max	min	max	min
g	0.9	0.9	2.1	2.1
J	3.4	-0.7	7.3	0.0
tot	4.4	0.2	9.4	2.1

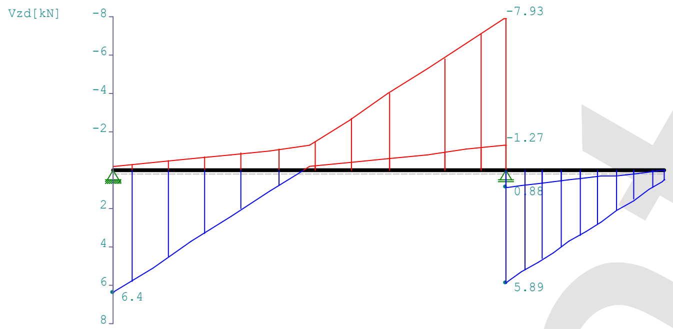
Results for γ-times loads
Partial safety factor γG * K_{Fi} = 1.35 constant over whole girder length

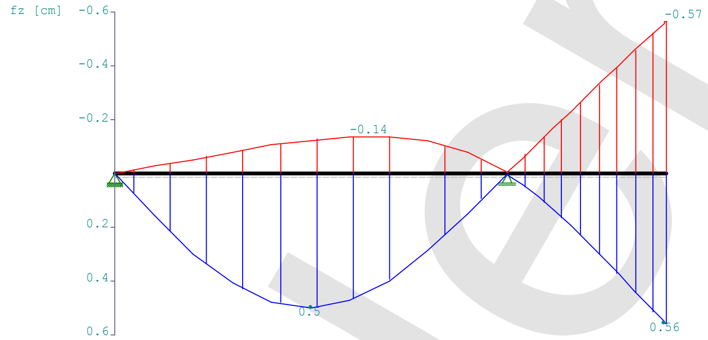
Span moments maximum							(kNm , kN)
Span		Mfd	Mdle	Mdri	V le	V ri	comb
1	x0 = 1.250	4.01	0.00	-0.62	6.40	-6.88	J 2

Support moments maximum							(kNm , kN)
Support	Mdle	Mdri	Vdle	Vdri	max F	min F	comb
1	0.00	0.00	0.00	6.40	6.40	-0.14	J 2
2	-3.37	-3.37	-7.93	5.89	13.82	2.14	J 3

Scale 1 : 50







Cross sectionsS235		fyk =	235 N/mm2			
type	name	Npl	Mplyd	Vplzd	Mplzd	Vplyd
10	L100X50X6	205	6	66	2	32

proof acc. DIN EN 1993-1-1/NA:2010-12 6.2.1 (6.1) γM0 = 1.00

Span No.	x (m)	csno.	My,ed (kNm)	Vz,ed (kN)	σv (N/mm2)	τ	CSCL	η	comb	
1	0.000	1	0.0	6.4	13	8	1	0.06	J	2
	1.250	1	4.0	0.0	165	0	1	0.70	J	2
	2.600	1	-3.4	-2.8	138	0	1	0.59	J	3
CaRi	0.000	1	-3.4	5.9	138	0	1	0.59	J	3
	1.049	1	0.0	0.5	1	1	1	0.00	J	4
	1.050	1	0.0	0.5	1	1	1	0.00	J	4

proof acc. DIN EN 1993-1-1/NA:2010-12 6.2.1 (6.2) γM0 = 1.00

Span No.	x (m)	My,ed (kNm)	Vz,ed (kN)	CSCL (-)	ρ (-)	M,Rd (kNm)	η	comb	
1	0.000	0.0	6.4	1	0.00	5.8	0.05	J	2
	1.250	4.0	0.0	1	0.00	4.7	0.43	J	2
	2.600	-3.4	-2.8	1	0.00	4.7	0.36	J	3
CaRi	0.000	-3.4	5.9	1	0.00	4.7	0.36	J	3
	1.049	0.0	0.5	1	0.00	5.8	0.00	J	4
	1.050	0.0	0.5	1	0.00	5.8	0.00	J	4

Compression flange is supported continuously.
Proof of torsional-flexural buckling is not necessary.

Permissible deflection : in span perm.f = L / 300
characteristic combination Cantilever L / 150

Span No.	x (m)	fg (cm)	ftot (cm)	f (cm)	perm.f (cm)	η	comb
1	1.300	0.08	0.50	0.499	0.867	0.58	2
CaRi	1.049	-0.03	-0.57	-0.567	0.700	0.81	2

At the following table the loads are specified by their internal numeration.
The following table of calculated combinations referenced.
to these numbers

Load type (kN,m) : 1=uniform over L 2=concentrated at a
3=single moment at a 4=trapezoidal btw. a, a+b
5=triangular over L 6=trapezoidal over L

No.	span	Type		Grp	g1	q1	g2	q2	factor	distance	length
1	1	1	J	1	0.70	2.65			1.00		
Cantilever											
2	CaRi	1	J	2	0.70	2.65			1.00		
3		2	J	2	0.00	0.35			1.00	1.05	

Calculated combinations from 3 Loads

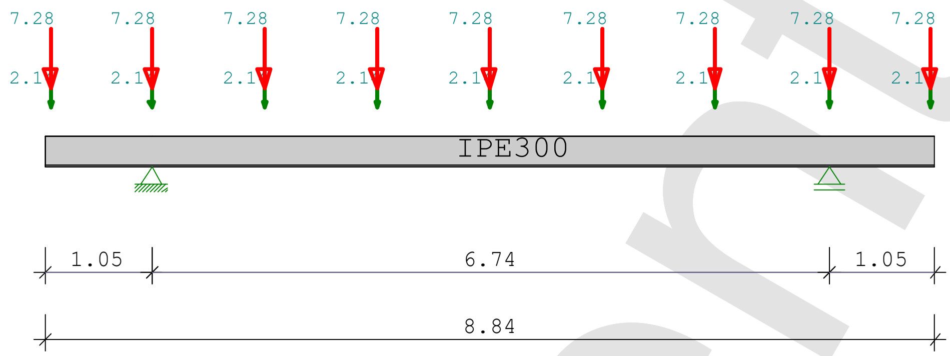
lc	K1	K2	K3	K4
	g	g	g	g
1	.	x	.	x
2	.	.	x	x
3	.	.	x	x

The combinations above will be managed as followed :
Calculating ULS the dead loads will be exceeded
all at once alternating by GammaG = 1,00 / 1,35.
If in one combination live-loads from different actions
exists , then will be investigated, which action is
the dominating one.
The effect of the duration of action will be checked too.

Position: V.31.2 - Steel Canopy

Durchlaufträger DLT10 01/2018 (Frilo R-2018-1/P12)

Scale 1 : 75



Steel girder	S235 DIN EN 1993-1-1/NA:2010-12
E-modulus	E =210000 N/mm2

System	length		cross-section values				
Span	L (m)		CsNo.	I (cm4)	St (cm3)	Sb (cm3)	
1	6.740	constant	1	8360.0	557.0	557.0	IPE300
Cantilever							
left	1.050	constant	1	8360.0	557.0	557.0	IPE300
right	1.050	constant	1	8360.0	557.0	557.0	IPE300

Load type (kN,m) : 1=uniform over L 2=concentrated at a
3=single moment at a 4=trapezoidal btw. a, a+b
5=triangular over L 6=trapezoidal over L

Span	Type	AG	G	r g_l/r	q_l/r	factor	distanc	e length	fromItem	Phi
1	2	A		2.140	7.280	1.000	0.000			
	2	A		2.140	7.280	1.000	1.120			
	2	A		2.140	7.280	1.000	2.240			
	2	A		2.140	7.280	1.000	3.360			
	2	A		2.140	7.280	1.000	4.480			
	2	A		2.140	7.280	1.000	5.600			
	2	A		2.140	7.280	1.000	6.740			
Cantilever										
CaLe	2	A		2.140	7.280	1.000	0.050			
CaRi	2	A		2.140	7.280	1.000	1.000			

Dead load of girder is considered with Gamma = 78.5 kN/m3 .

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ
A	1	Cat A - domestic	0.70	0.50	0.30	1.50

Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3
In following tables the last cell the row is a reference to
the number of the related superposition (see below).
In tables with internal forces multiplied by Gamma is additionally
a reference to the main action.

Results for 1-times loads

Span moments maximum (kNm , kN)

Span				Mf	M le	M ri	V le	V ri	comb
1	x0	=	3.361	47.73	-2.37	-2.37	25.04	-24.90	3

Support moments maximum (kNm , kN)

Column	M le	M ri	V le	V ri	max F	min F	comb
1	-9.65	-9.65	-9.86	26.12	45.41	10.43	2
2	-9.65	-9.65	-25.98	9.86	45.27	10.40	2

Support reactions (kN)

Column	by g	max q	min q	Fulload	max	min
1	11.51	33.89	-1.08	44.33	45.41	10.43
2	11.48	33.79	-1.08	44.19	45.27	10.40
Total:	22.99	67.68	-2.16	88.51	90.67	20.83

Support reactions (kN)

	Column 1		Column 2	
CA	max	min	max	min
g	11.5	11.5	11.5	11.5
A	33.9	-1.1	33.8	-1.1
tot	45.4	10.4	45.3	10.4

Results for γ-times loads
Partial safety factor γG * K_{Fi} = 1.35 constant over whole girder length

Span moments maximum (kNm , kN)

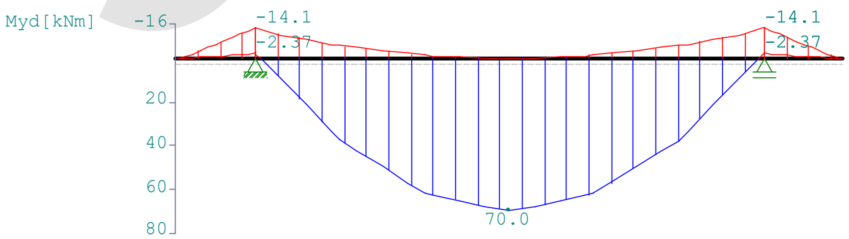
Span				Mfd	Mdle	Mdri	V le	V ri	comb
1	x0	=	3.361	69.97	-3.20	-3.20	36.55	-36.34	A 3

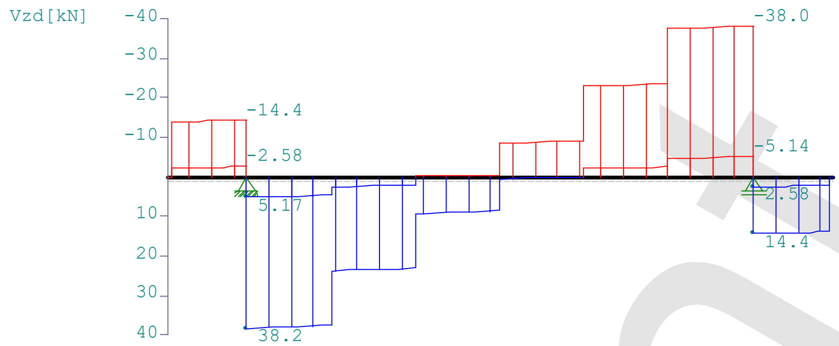
Support moments maximum (kNm , kN)

Support	Mdle	Mdri	Vdle	Vdri	max F	min F	comb
1	-14.12	-14.12	-14.41	38.17	66.38*	9.89*	A 2
2	-14.12	-14.12	-37.96	14.41	66.18*	9.86*	A 2

* -> value for F results by an another combination.

Scale 1 : 100





Cross sectionsS235	fyk =	235 N/mm2			
type name	Npl	Mplyd	Vplzd	Mplzd	Vplyd
2 IPE300	1264	148	348	29	436

proof acc. DIN EN 1993-1-1/NA:2010-12 6.2.1 (6.1) γM0 = 1.00

Span No.	x (m)	csno.	My,ed (kNm)	Vz,ed (kN)	σv (N/mm2)	τ	CSCL	η		comb
CaLe	0.000	1	0.0	0.0	0	0	1	0.00		1
	0.049	1	0.0	0.0	0	0	1	0.00		1
	0.051	1	0.0	-13.8	12	7	1	0.05	A	2
	1.050	1	-14.1	-14.4	26	2	1	0.11	A	2
1	0.000	1	-14.1	38.2	35	16	1	0.15	A	5
	0.001	1	-14.1	38.2	35	16	1	0.15	A	5
	1.119	1	37.3	35.9	67	5	1	0.29	A	3
	1.121	1	37.4	22.1	67	3	1	0.29	A	3
	2.239	1	61.8	21.5	111	3	1	0.47	A	3
	2.241	1	61.8	7.7	111	1	1	0.47	A	3
	3.359	1	70.0	7.0	126	1	1	0.53	A	3
	3.361	1	70.0	-6.8	126	1	1	0.53	A	3
	4.479	1	62.0	-7.4	111	1	1	0.47	A	3
	4.481	1	62.0	-21.2	111	3	1	0.47	A	3
	5.599	1	37.9	-21.9	68	3	1	0.29	A	3
	5.601	1	37.8	-35.7	68	5	1	0.29	A	3
	6.739	1	-14.1	-38.0	35	16	1	0.15	A	7
	6.740	1	-14.1	-38.0	35	16	1	0.15	A	7
CaRi	0.000	1	-14.1	14.4	26	2	1	0.11	A	6
	0.999	1	0.0	13.8	12	7	1	0.05	A	7
	1.001	1	0.0	0.0	0	0	1	0.00	A	7
	1.050	1	0.0	0.0	0	0	1	0.00	A	7

proof acc. DIN EN 1993-1-1/NA:2010-12 6.2.1 (6.2) γM0 = 1.00

Span No.	x (m)	My,ed (kNm)	Vz,ed (kN)	CSCL (-)	ρ (-)	M,Rd (kNm)	η		comb
CaLe	0.000	0.0	0.0	1	0.00	148.0	0.00		1
	0.049	0.0	0.0	1	0.00	148.0	0.00		1
	0.051	0.0	-13.8	1	0.00	148.0	0.04	A	2
	1.050	-14.1	-14.4	1	0.00	148.0	0.10	A	2
1	0.000	-14.1	38.2	1	0.00	148.0	0.11	A	5
	0.001	-14.1	38.2	1	0.00	148.0	0.11	A	5
	1.119	37.3	35.9	1	0.00	148.0	0.25	A	3
	1.121	37.4	22.1	1	0.00	148.0	0.25	A	3
	2.239	61.8	21.5	1	0.00	148.0	0.42	A	3
	2.241	61.8	7.7	1	0.00	148.0	0.42	A	3
	3.359	70.0	7.0	1	0.00	148.0	0.47	A	3
	3.361	70.0	-6.8	1	0.00	148.0	0.47	A	3
	4.479	62.0	-7.4	1	0.00	148.0	0.42	A	3
	4.481	62.0	-21.2	1	0.00	148.0	0.42	A	3
	5.599	37.9	-21.9	1	0.00	148.0	0.26	A	3

proof acc. DIN EN 1993-1-1/NA:2010-12 6.2.1 (6.2) γM0 = 1.00

Span No.	x (m)	My,ed (kNm)	Vz,ed (kN)	CSCL (-)	ρ (-)	M,Rd (kNm)	η	comb
	5.601	37.8	-35.7	1	0.00	148.0	0.26	A 3
	6.739	-14.1	-38.0	1	0.00	148.0	0.11	A 7
	6.740	-14.1	-38.0	1	0.00	148.0	0.11	A 7
CaRi	0.000	-14.1	14.4	1	0.00	148.0	0.10	A 6
	0.999	0.0	13.8	1	0.00	148.0	0.04	A 7
	1.001	0.0	0.0	1	0.00	148.0	0.00	A 7
	1.050	0.0	0.0	1	0.00	148.0	0.00	A 7

Compression flange is supported continuously.
Proof of torsional-flexural buckling is not necessary.

Permissible deflection : in span perm.f = L / 300
characteristic combination Cantilever L / 150

Span No.	x (m)	fg (cm)	ftot (cm)	f (cm)	perm.f (cm)	η	comb
CaLe	0.000	-0.12	-0.60	-0.603	0.700	0.86	3
1	3.370	0.27	1.25	1.245	2.247	0.55	3
CaRi	1.050	-0.12	-0.60	-0.603	0.700	0.86	3

At the following table the loads are specified by their internal numeration.
The following table of calculated combinations referenced.
to these numbers

Load type (kN,m) : 1=uniform over L 2=concentrated at a
3=single moment at a 4=trapezoidal btw. a, a+b
5=triangular over L 6=trapezoidal over L

No.	span	Type	Grp	g1	q1	g2	q2	factor	distance	length
2	1	2	A 2	2.14	7.28			1.00	0.00	
3		2	A 2	2.14	7.28			1.00	1.12	
4		2	A 2	2.14	7.28			1.00	2.24	
5		2	A 2	2.14	7.28			1.00	3.36	
6		2	A 2	2.14	7.28			1.00	4.48	
7		2	A 2	2.14	7.28			1.00	5.60	
8		2	A 2	2.14	7.28			1.00	6.74	
Cantilever										
1	CaLe	2	A 1	2.14	7.28			1.00	0.05	
9	CaRi	2	A 3	2.14	7.28			1.00	1.00	

Calculated combinations from 9 Loads

lc	K1	K2	K3	K4	K5	K6	K7
	g	g	g	g	g	g	g
1	.	x	.	x	x	.	.
2	.	.	x	.	x	.	x
3	.	.	x	.	x	.	x
4	.	.	x	.	x	.	x
5	.	.	x	.	x	.	x
6	.	.	x	.	x	.	x
7	.	.	x	.	x	.	x
8	.	.	x	.	x	.	x
9	.	x	.	.	.	x	x

Calculated combinations from 9 Loads

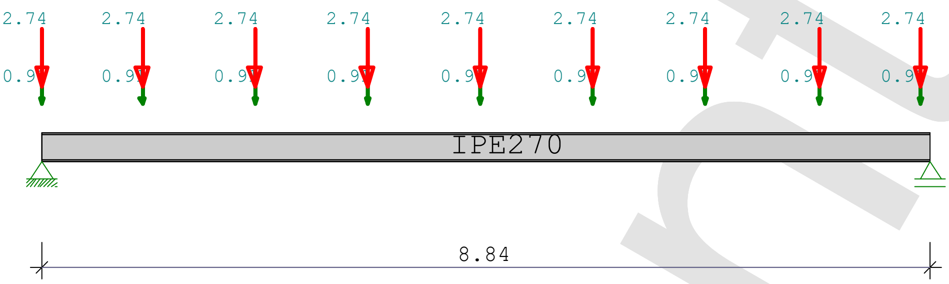
lc	K1	K2	K3	K4	K5	K6	K7

The combinations above will be managed as followed :
Calculating ULS the dead loads will be exceeded
all at once alternating by GammaG = 1,00 / 1,35.
If in one combination live-loads from different actions
exists , then will be investigated, which action is
the dominating one.
The effect of the duration of action will be checked too.

Position: V.31.4 - Steel Canopy

Durchlaufträger DLT10 01/2018 (Frilo R-2018-1/P12)

Scale 1 : 75



Steel girder S235 DIN EN 1993-1-1/NA:2010-12
E-modulus E =210000 N/mm2

System	length		cross-section values				
Span	L (m)		CsNo.	I (cm4)	St (cm3)	Sb (cm3)	
1	8.840	constant	1	5790.0	429.0	429.0	IPE270

Load type (kN,m) : 1=uniform over L 2=concentrated at a
3=single moment at a 4=trapezoidal btw. a, a+b
5=triangular over L 6=trapezoidal over L

Span	Type	AG	G	r	g_l/r	q_l/r	factor	distance	length	fromItem	Phi
1	2	A			0.910	2.740	1.000	0.000			
	2	A			0.910	2.740	1.000	1.000			
	2	A			0.910	2.740	1.000	2.120			
	2	A			0.910	2.740	1.000	3.240			
	2	A			0.910	2.740	1.000	4.360			
	2	A			0.910	2.740	1.000	5.480			
	2	A			0.910	2.740	1.000	6.600			
	2	A			0.910	2.740	1.000	7.740			
	2	A			0.910	2.740	1.000	8.740			

Dead load of girder is considered with Gamma = 78.5 kN/m3 .

Actions:

No.	Cl	Name	ψ0	ψ1	ψ2	γ
A	1	Cat A - domestic	0.70	0.50	0.30	1.50

Consequency class CC 2 acc. EN 1990 Tab. B1 -> K_{Fi} = 1.0 Tab. B3
In following tables the last cell the row is a reference to
the number of the related superposition (see below).
In tables with internal forces multiplied by Gamma is additionally
a reference to the main action.

Results for 1-times loads

Span moments maximum (kNm , kN)

Span		Mf	M le	M ri	V le	V ri	comb
1	x0 = 4.361	35.59	0.00	0.00	14.57	-17.81	2

Support moments maximum							(kNm , kN)
Column	M le	M ri	V le	V ri	max F	min F	comb
1	0.00	0.00	0.00	14.57	18.22	5.74	2
2	0.00	0.00	-17.81	0.00	17.81	5.64	2

Support reactions							(kN)
Column	by g	max q	min q	Fulload	max	min	
1	5.74	12.48	0.00	18.22	18.22	5.74	
2	5.64	12.18	0.00	17.81	17.81	5.64	
Total:	11.38	24.66	0.00	36.04	36.04	11.38	

Support reactions				(kN)
	Column 1		Column 2	
CA	max	min	max	min
g	5.7	5.7	5.6	5.6
A	12.5	0.0	12.2	0.0
tot	18.2	5.7	17.8	5.6

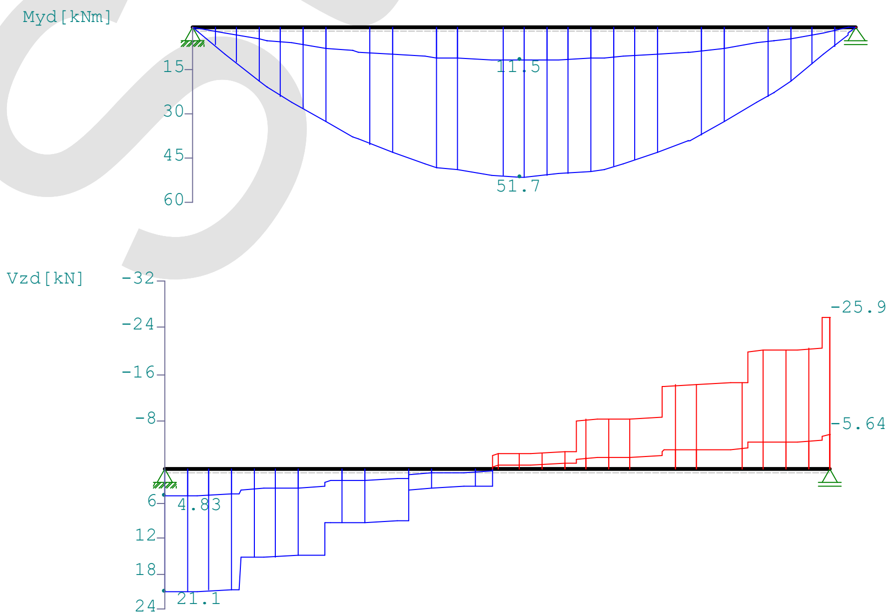
Results for γ-times loads
Partial safety factor γG * K_{Fi} = 1.35 constant over whole girder length

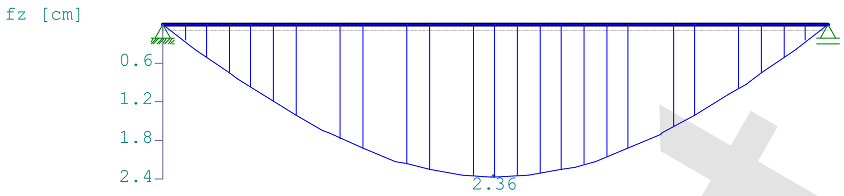
Span moments maximum							(kNm , kN)
Span		Mfd	Mdle	Mdri	V le	V ri	comb
1	x0 = 4.361	51.66	0.00	0.00	21.14	-25.87	A 2

Support moments maximum							(kNm , kN)
Support	Mdle	Mdri	Vdle	Vdri	max F	min F	comb
1	0.00	0.00	0.00	21.14	26.48*	5.74*	A 2
2	0.00	0.00	-25.87	0.00	25.87	5.64	A 2

* -> value for F results by an another combination.

Scale 1 : 100





Cross sectionsS235	fyk =	235 N/mm2			
type name	Npl	Mplyd	Vplzd	Mplzd	Vplyd
2 IPE270	1079	114	300	23	374

proof acc. DIN EN 1993-1-1/NA:2010-12 6.2.1 (6.1) γM0 = 1.00

Span No.	x (m)	csno.	My,ed (kNm)	Vz,ed (kN)	σv (N/mm2)	τ	CSCL	η	comb	
1	0.000	1	0.0	21.1	22	13	1	0.10	A	2
	0.001	1	0.0	21.1	22	13	1	0.10	A	2
	0.999	1	20.9	20.7	49	3	1	0.21	A	2
	1.001	1	20.9	15.3	49	2	1	0.21	A	2
	2.119	1	37.7	14.8	88	2	1	0.37	A	2
	2.121	1	37.7	9.4	88	1	1	0.37	A	2
	3.239	1	48.0	8.9	112	1	1	0.48	A	2
	3.241	1	48.0	3.5	112	1	1	0.48	A	2
	4.359	1	51.7	3.0	120	0	1	0.51	A	2
	4.361	1	51.7	-2.3	120	0	1	0.51	A	2
	5.479	1	48.7	-2.9	114	0	1	0.48	A	2
	5.481	1	48.7	-8.2	114	1	1	0.48	A	2
	6.599	1	39.2	-8.8	91	1	1	0.39	A	2
	6.601	1	39.2	-14.1	92	2	1	0.39	A	2
	7.739	1	22.8	-14.7	53	2	1	0.23	A	2
	7.741	1	22.8	-20.0	53	3	1	0.23	A	2
	8.739	1	2.6	-20.5	22	13	1	0.09	A	2
	8.741	1	2.6	-25.8	27	16	1	0.12	A	2
	8.840	1	0.0	-25.9	28	16	1	0.12	A	2

proof acc. DIN EN 1993-1-1/NA:2010-12 6.2.1 (6.2) γM0 = 1.00

Span No.	x (m)	My,ed (kNm)	Vz,ed (kN)	CSCL (-)	ρ (-)	M,Rd (kNm)	η	comb	
1	0.000	0.0	21.1	1	0.00	114.1	0.07	A	2
	0.001	0.0	21.1	1	0.00	114.1	0.07	A	2
	0.999	20.9	20.7	1	0.00	114.1	0.18	A	2
	1.001	20.9	15.3	1	0.00	114.1	0.18	A	2
	2.119	37.7	14.8	1	0.00	114.1	0.33	A	2
	2.121	37.7	9.4	1	0.00	114.1	0.33	A	2
	3.239	48.0	8.9	1	0.00	114.1	0.42	A	2
	3.241	48.0	3.5	1	0.00	114.1	0.42	A	2
	4.359	51.7	3.0	1	0.00	114.1	0.45	A	2
	4.361	51.7	-2.3	1	0.00	114.1	0.45	A	2
	5.479	48.7	-2.9	1	0.00	114.1	0.43	A	2
	5.481	48.7	-8.2	1	0.00	114.1	0.43	A	2
	6.599	39.2	-8.8	1	0.00	114.1	0.34	A	2
	6.601	39.2	-14.1	1	0.00	114.1	0.34	A	2
	7.739	22.8	-14.7	1	0.00	114.1	0.20	A	2
	7.741	22.8	-20.0	1	0.00	114.1	0.20	A	2
	8.739	2.6	-20.5	1	0.00	114.1	0.07	A	2
	8.741	2.6	-25.8	1	0.00	114.1	0.09	A	2
	8.840	0.0	-25.9	1	0.00	114.1	0.09	A	2

Compression flange is supported continuously.
Proof of torsional-flexural buckling is not necessary.

Permissible deflection : in span perm.f = L / 300
characteristic combination

Span No.	x (m)	fg (cm)	ftot (cm)	f (cm)	perm.f (cm)	η	comb
1	4.420	0.77	2.36	2.359	2.947	0.80	2

At the following table the loads are specified by their internal numeration.
The following table of calculated combinations referenced.
to these numbers

Load type (kN,m) : 1=uniform over L; 2=concentrated at a; 3=single moment at a; 4=trapezoidal btw. a, a+b; 5=triangular over L; 6=trapezoidal over L

No.	span	Type	Grp	g1	q1	g2	q2	factor	distance	length
1	1	2	A 1	0.91	2.74			1.00	0.00	
2		2	A 1	0.91	2.74			1.00	1.00	
3		2	A 1	0.91	2.74			1.00	2.12	
4		2	A 1	0.91	2.74			1.00	3.24	
5		2	A 1	0.91	2.74			1.00	4.36	
6		2	A 1	0.91	2.74			1.00	5.48	
7		2	A 1	0.91	2.74			1.00	6.60	
8		2	A 1	0.91	2.74			1.00	7.74	
9		2	A 1	0.91	2.74			1.00	8.74	

Calculated combinations from 9 Loads

lc	K1	K2
	g	g
1	.	x
2	.	x
3	.	x
4	.	x
5	.	x
6	.	x
7	.	x
8	.	x
9	.	x

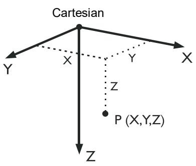
The combinations above will be managed as followed :
Calculating ULS the dead loads will be exceeded
all at once alternating by GammaG = 1,00 / 1,35.
If in one combination live-loads from different actions
exists , then will be investigated, which action is
the dominating one.
The effect of the duration of action will be checked too.

B.2 INTERNAL FORCES FROM 'DLUBAL RSTAB'

B.2.1. Internal Forces POS. Z.01 - Trapezoidal Sheet Covering

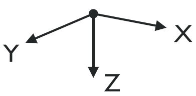
Project: Master's Project   Model: POS. Z.01 - Trapezoidal Sheet Covering

CONTENTS



1.1 NODES

Node No.	Reference Node	Coordinate System	Node Coordinates X [m]	Z [m]	Comment
1	-	Cartesian	0.000	0.000	
2	-	Cartesian	4.700	0.000	
3	-	Cartesian	9.400	0.000	
4	-	Cartesian	14.100	0.000	



1.8 NODAL SUPPORTS

Support No.	Nodes No.	Rotation [°] about Y	Support or Spring [kN/m] [kNm/rad] $u_{X'}$	$u_{Z'}$	$\varphi_{Y'}$	Comment
1	1	0.00	☒	☒	☐	
2	2-4	0.00	☐	☒	☐	

2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | DIN Action Category	Active	Self-Weight - Factor in Direction X	Y	Z
LC1	Self Weight + Roof Covering	Permanent	☐			
LC2	Snow	Snow (H ≤ 1000 m a.s.l.)	☐			
LC3	Wind PRESSURE	Wind	☐			
LC4	Wind SUCTION	Wind	☐			

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		ULS - Pressure g+s+w	1	1.35	LC1	Self Weight + Roof Covering
			2	1.50	LC2	Snow
			3	0.90	LC3	Wind PRESSURE
CO2		ULS - Suction g+w	1	1.00	LC1	Self Weight + Roof Covering
			2	1.50	LC4	Wind SUCTION

LC1
Self Weight + Roof Covering

3.2 MEMBER LOADS

LC1: Self Weight + Roof Covering

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	1-3	Force	Uniform	Z	True Length	p	0.500	kN/m

LC2
Snow

3.2 MEMBER LOADS

LC2: Snow

No.	Reference to	On Members No.		Load Distribution	Load Direction	Symbol	Load Parameters Value	Unit	Over Tot. Length
1	List of members	1-3	Force	Trapezoidal	Z	True Length	p_1	2.370	kN/m
							p_2	2.600	kN/m

LC3
Wind PRESSURE

3.2 MEMBER LOADS

LC3: Wind PRESSURE

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	1-3	Force	Uniform	Z	True Length	p	0.110	kN/m

LC4
Wind SUCTION

3.2 MEMBER LOADS

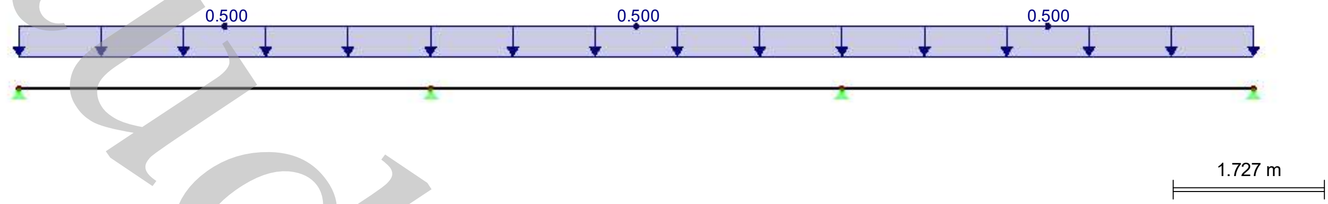
LC4: Wind SUCTION

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	1	Force	Uniform	Z	True Length	p	-1.000	kN/m
2	Members	2,3	Force	Uniform	Z	True Length	p	-0.670	kN/m

LC1: SELF WEIGHT + ROOF COVERING

LC 1: Self Weight + Roof Covering
Loads [kN/m]

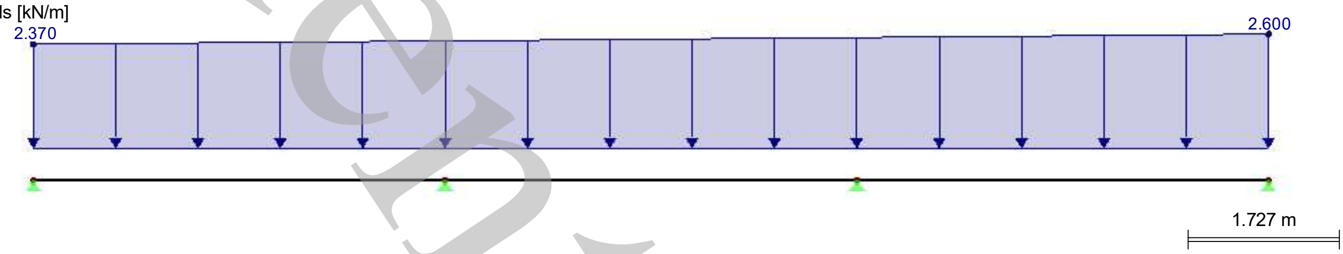
Against Y-direction



LC2: SNOW

LC 2: Snow
Loads [kN/m]

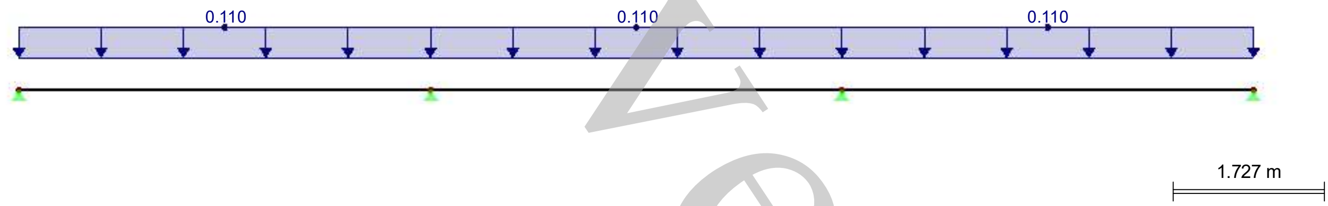
Against Y-direction



LC3: WIND PRESSURE

LC 3: Wind PRESSURE
Loads [kN/m]

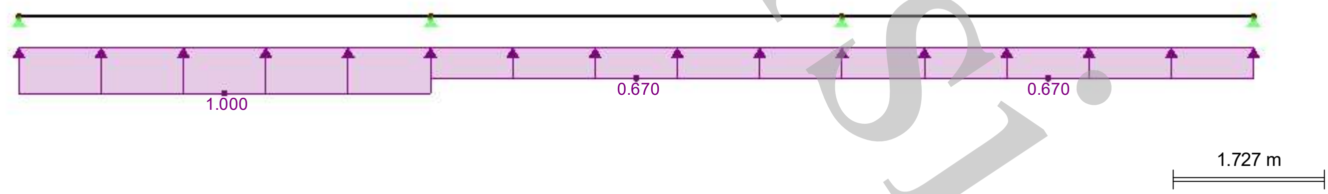
Against Y-direction



LC4: WIND SUCTION

LC 4: Wind SUCTION
Loads [kN/m]

Against Y-direction

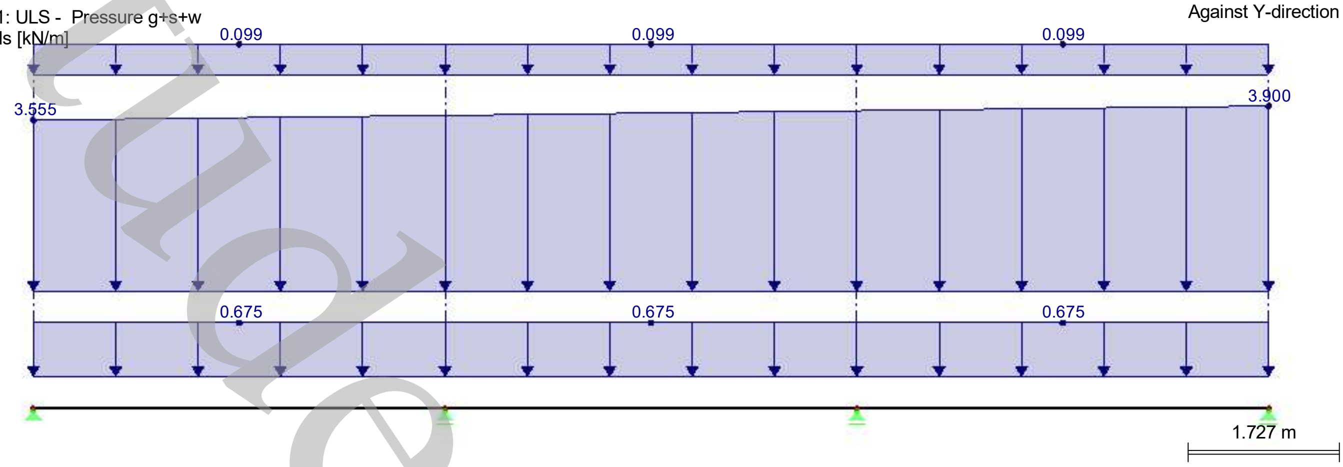


Project: Master's Project Model: POS. Z.01 - Trapezoidal Sheet Covering

■ CO1: ULS - PRESSURE G+S+W

CO 1: ULS - Pressure g+s+w
Loads [kN/m]

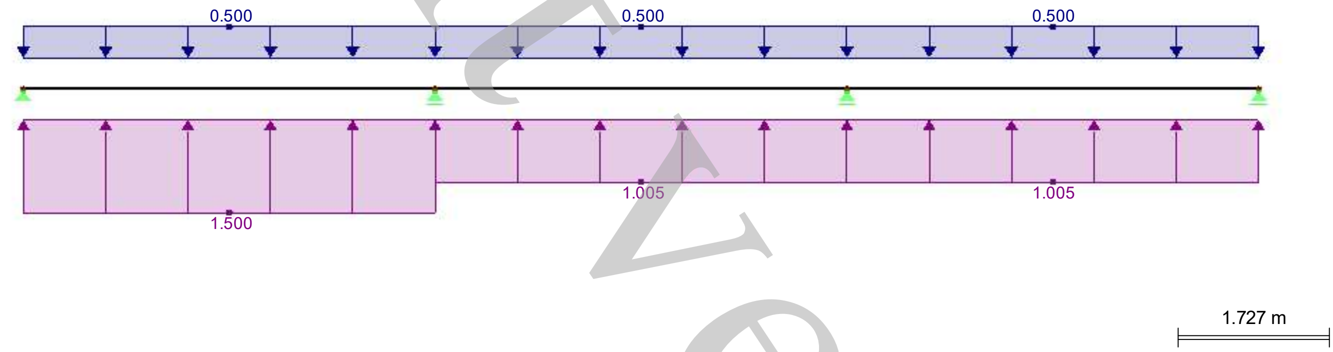
Against Y-direction



■ CO2: ULS - SUCTION G+W

CO 2: ULS - Suction g+w
Loads [kN/m]

Against Y-direction



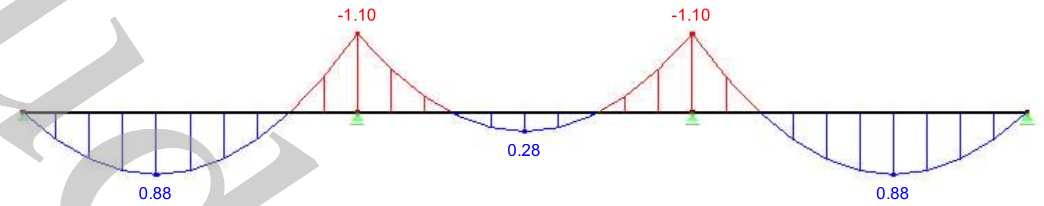
■ 4.3 CROSS-SECTIONS - INTERNAL FORCES

Member No.	LC/CO	Node No.	Location x [m]	Forces [kN] N	Forces [kN] V_z	Moments M_y [kNm]
	Section No. 2: HSW + E 135 - 1.25 (b: 1000.0 mm) | Hoesch E					
3	CO1	MAX N	4.700	0.10	-8.73	0.00
2	CO1	MIN N	4.700	-0.04	-10.71	-10.16
3	CO1	MAX V_z	0.000	0.05	12.96	-10.16
1	CO1	MIN V_z	4.700	0.05	-12.42	-9.73
3	CO1	MAX M_y	2.820	0.00	0.01	8.19
2	CO1	MIN M_y	4.700	-0.04	-10.71	-10.16

INTERNAL FORCES M_y

LC 1: Self Weight + Roof Covering
Internal Forces M-y

Against Y-direction



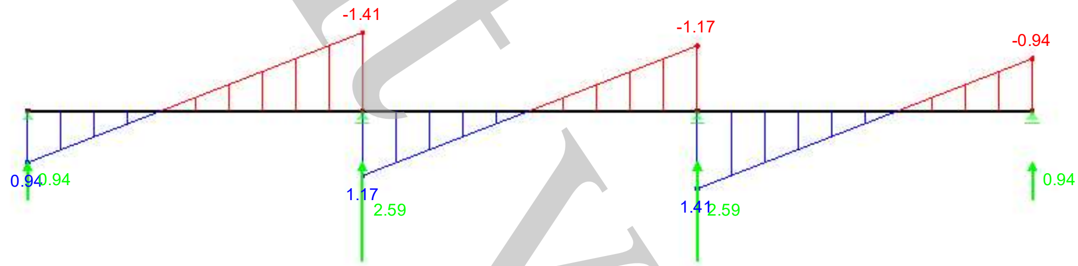
2.122 m

Max M-y: 0.88, Min M-y: -1.10 [kNm]

INTERNAL FORCES V_z, SUPPORT REACTIONS

LC 1: Self Weight + Roof Covering
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



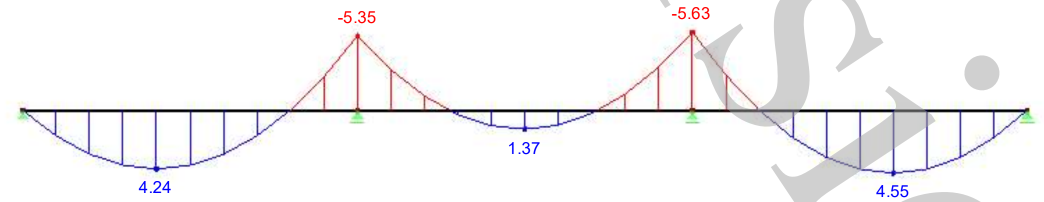
Max V-z: 1.41, Min V-z: -1.41 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 2.59, Min P-Z': 0.94 kN

2.122 m

INTERNAL FORCES M_y

LC 2: Snow
Internal Forces M-y

Against Y-direction



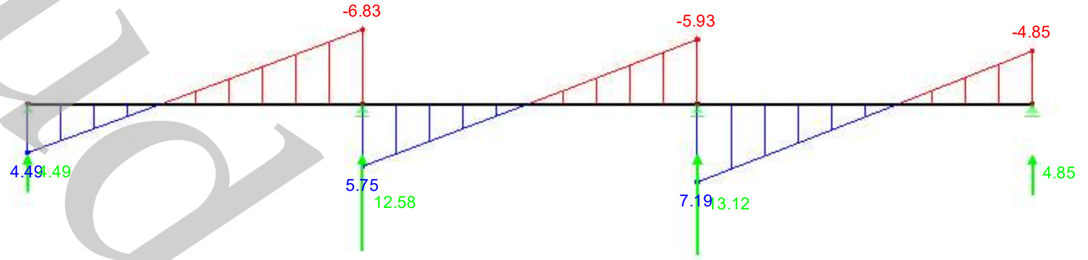
2.122 m

Max M-y: 4.55, Min M-y: -5.63 [kNm]

INTERNAL FORCES V_z, SUPPORT REACTIONS

LC 2: Snow
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



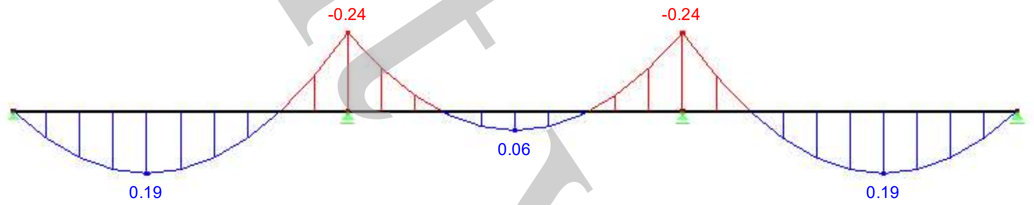
Max V-z: 7.19, Min V-z: -6.83 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 13.12, Min P-Z': 4.49 kN

2.122 m

INTERNAL FORCES M_y

LC 3: Wind PRESSURE
Internal Forces M-y

Against Y-direction



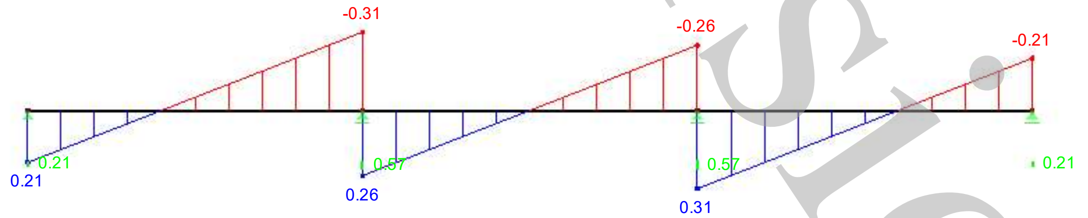
Max M-y: 0.19, Min M-y: -0.24 [kNm]

2.122 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

LC 3: Wind PRESSURE
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



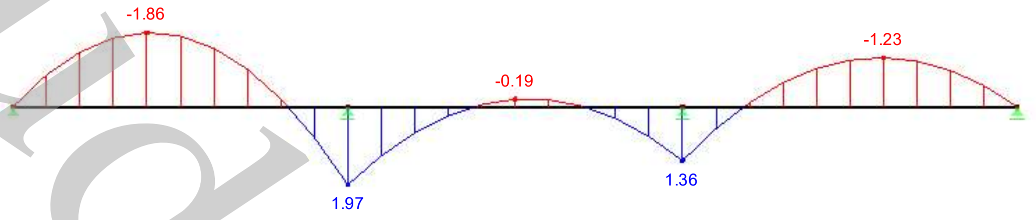
Max V-z: 0.31, Min V-z: -0.31 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 0.57, Min P-Z': 0.21 kN

2.122 m

INTERNAL FORCES M_y

LC 4: Wind SUCTION
Internal Forces M-y

Against Y-direction



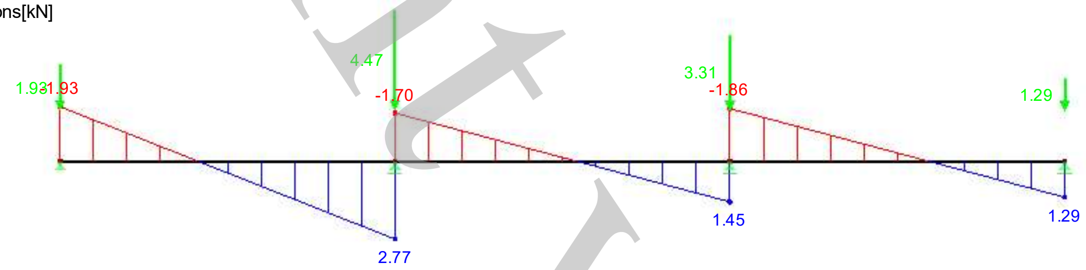
Max M-y: 1.97, Min M-y: -1.86 [kNm]

2.122 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

LC 4: Wind SUCTION
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



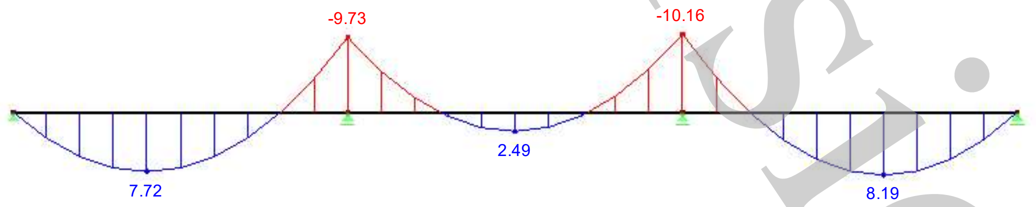
Max V-z: 2.77, Min V-z: -1.93 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': -1.29, Min P-Z': -4.47 kN

2.122 m

INTERNAL FORCES M_y

CO 1: ULS - Pressure g+s+w
Internal Forces M-y

Against Y-direction



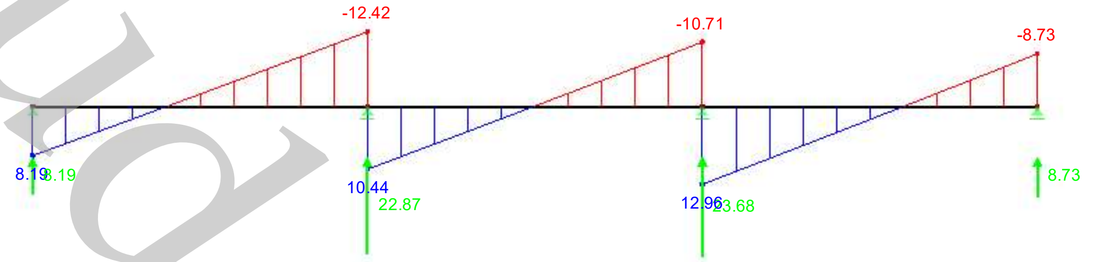
Max M-y: 8.19, Min M-y: -10.16 [kNm]

2.122 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

CO 1: ULS - Pressure g+s+w
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



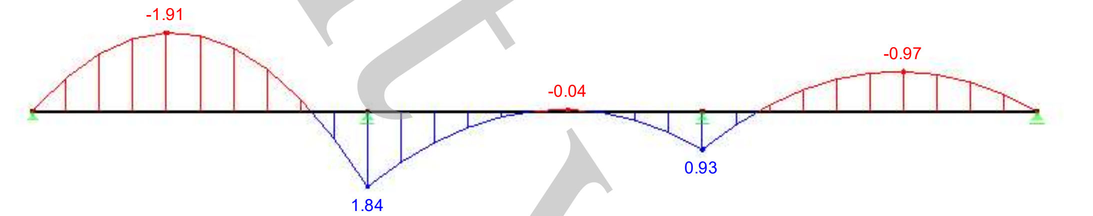
Max V-z: 12.96, Min V-z: -12.42 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 23.68, Min P-Z': 8.19 kN

2.122 m

INTERNAL FORCES M_y

CO 2: ULS - Suction g+w
Internal Forces M-y

Against Y-direction



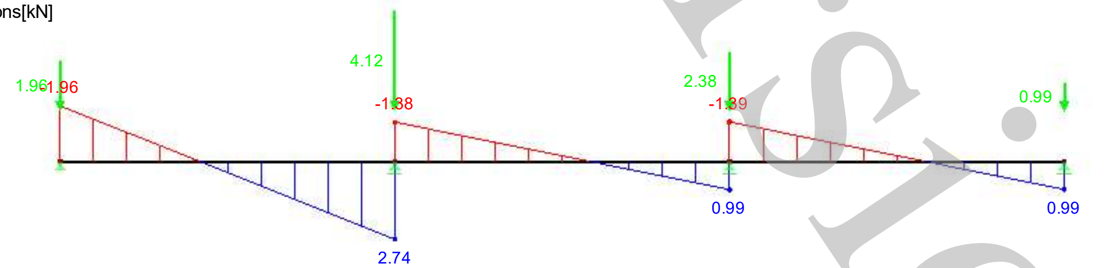
Max M-y: 1.84, Min M-y: -1.91 [kNm]

2.122 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

CO 2: ULS - Suction g+w
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



Max V-z: 2.74, Min V-z: -1.96 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': -0.99, Min P-Z': -4.12 kN

2.122 m

GLOBAL DEFORMATIONS u

LC 1: Self Weight + Roof Covering
Global Deformations u

Against Y-direction

Max u: 1.6, Min u: 0.0 [mm]
Factor of deformations: 880.00

1.727 m

GLOBAL DEFORMATIONS u

CO 1: ULS - Pressure g+s+w
Global Deformations u

Against Y-direction

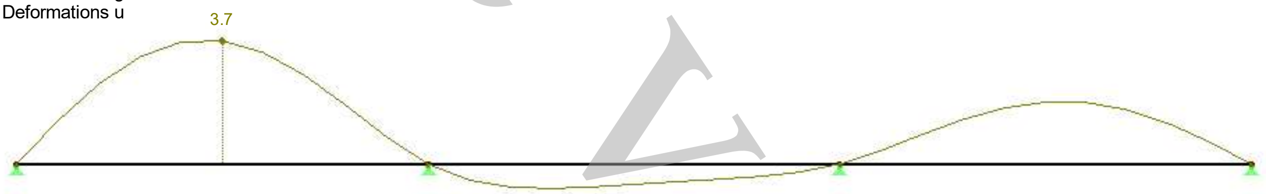
Max u: 14.9, Min u: 0.0 [mm]
Factor of deformations: 95.00

1.727 m

GLOBAL DEFORMATIONS u

CO 2: ULS - Suction g+w
Global Deformations u

Against Y-direction



Max u: 3.7, Min u: 0.0 [mm]
Factor of deformations: 380.00

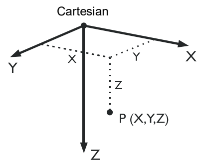
1.727 m

B.2.2. Internal Forces POS. P.03 – Prestressed Concrete Truss

Project: Master Project   Model: POS.P.03 - Prestressed Concrete Truss

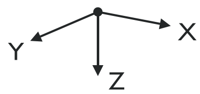
CONTENTS

1.1 NODES



Node No.	Reference Node	Coordinate System	Node Coordinates X [m]	Z [m]	Comment
2	-	Cartesian	0.000	0.000	
3	-	Cartesian	4.700	0.000	
4	-	Cartesian	9.400	0.000	
5	-	Cartesian	14.100	0.000	
6	-	Cartesian	15.100	0.000	
7	-	Cartesian	19.800	0.000	
8	-	Cartesian	24.500	0.000	
9	-	Cartesian	29.200	0.000	
10	-	Cartesian	14.600	0.000	
11	-	Cartesian	11.890	0.000	

1.8 NODAL SUPPORTS

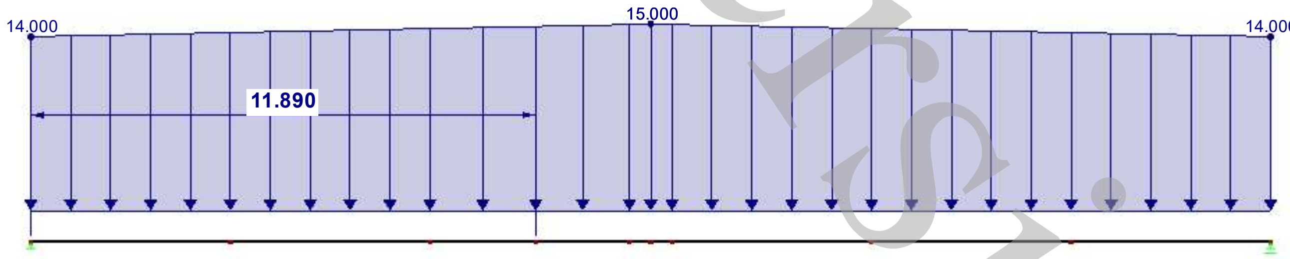


Support No.	Nodes No.	Rotation [°] about Y	Support or Spring [kN/m] [kNm/rad] $u_{X'}$	$u_{Z'}$	$\varphi_{Y'}$	Comment
1	2	0.00	☒	☒	☐	
2	9	0.00	☐	☒	☐	

MODEL

LC 1: Self-weight
Loads [kN/m]

Against Y-direction

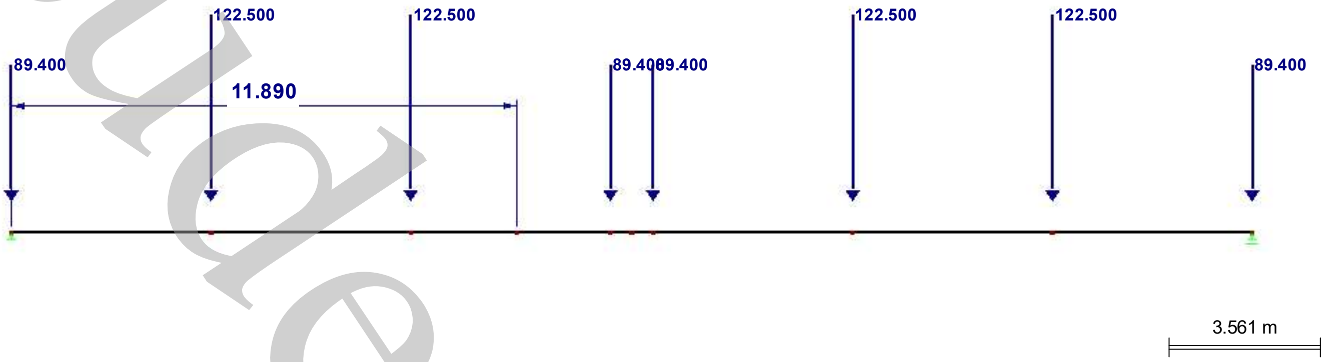


3.561 m

MODEL

LC 2: Roof structure
Loads [kN]

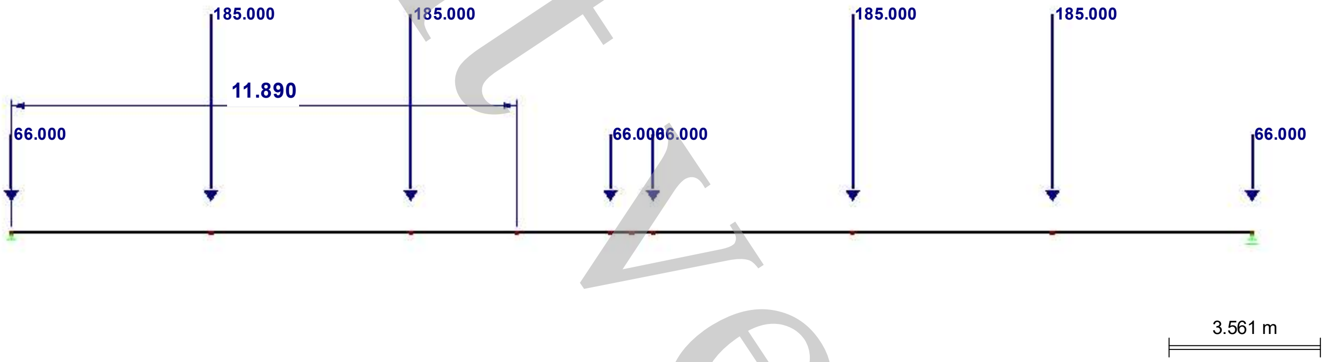
Against Y-direction



MODEL

LC 3: Snow
Loads [kN]

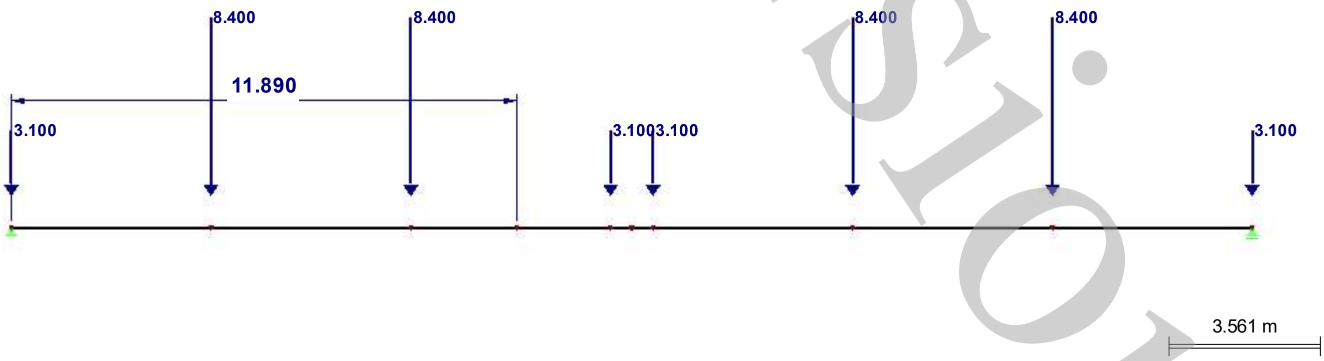
Against Y-direction



MODEL

LC 4: Wind pressure
Loads [kN]

Against Y-direction

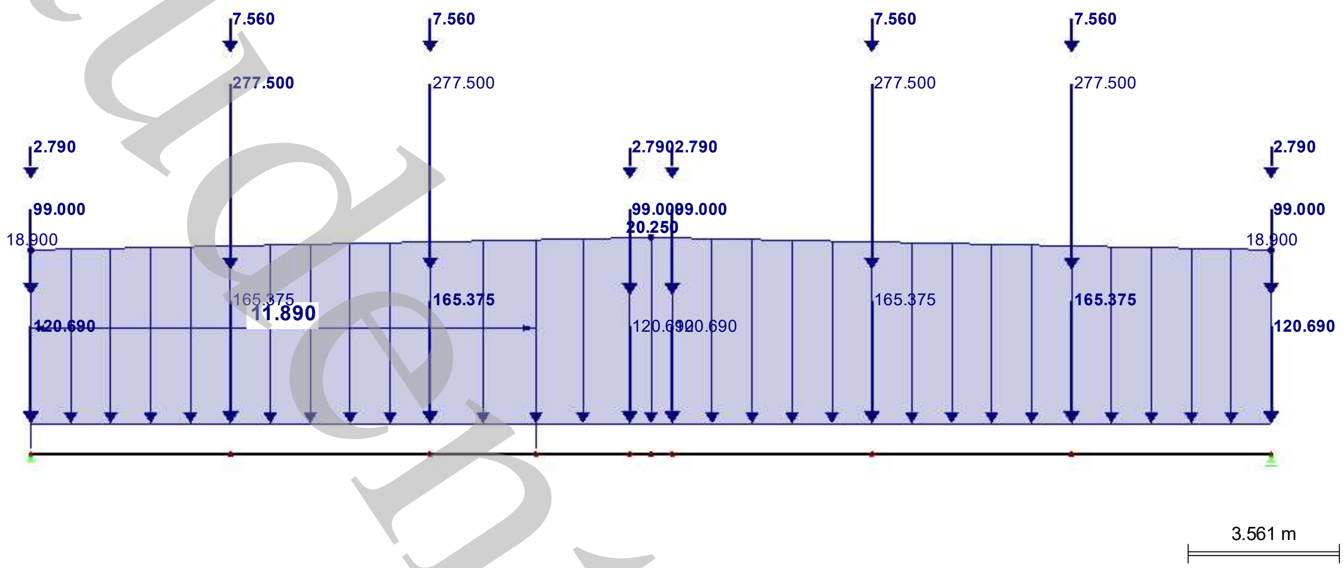


Project: Master Project Model: POS.P.03 - Prestressed Concrete Truss

MODEL

CO 1: ULS G+S+W
Loads [kN/m], [kN]

Against Y-direction



2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | DIN Action Category	Active	Self-Weight - Factor in Direction X	Y	Z
LC1	Self-weight	Permanent	☐			
LC2	Roof structure	Permanent/Imposed	☐			
LC3	Snow	Snow (H ≤ 1000 m a.s.l.)	☐			
LC4	Wind pressure	Wind	☐			

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		ULS G+S+W	1	1.35	LC1	Self-weight
			2	1.35	LC2	Roof structure
			3	1.50	LC3	Snow
			4	0.90	LC4	Wind pressure
CO2		SLS frequ	1	1.00	LC1	Self-weight
			2	1.00	LC2	Roof structure
			3	0.20	LC3	Snow

LC1
Self-weight

3.2 MEMBER LOADS

LC1: Self-weight

No.	Reference to	On Members No.		Load Distribution	Load Direction	Symbol	Load Parameters Value	Over Tot. Unit	Length
1	List of members	1,2,9,10,4	Force	Trapezoidal	Z	True Length	p_1	14.000	kN/m
							p_2	15.000	kN/m
2	List of members	8,5-7	Force	Trapezoidal	Z	True Length	p_1	15.000	kN/m
							p_2	14.000	kN/m

LC2
Roof structure

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC2: Roof structure

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Z / P_W	Moment M_Y / M_V [kNm]
1	3,4,7,8	0 | Global XYZ	0.000	122.500	0.000
2	2,5,6,9	0 | Global XYZ	0.000	89.400	0.000

Project: Master Project Model: POS.P.03 - Prestressed Concrete Truss

LC3
Snow

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC3: Snow

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Z / P_W	Moment M_Y / M_V [kNm]
1	3,4,7,8	0 | Global XYZ	0.000	185.000	0.000
2	2,5,6,9	0 | Global XYZ	0.000	66.000	0.000

LC4
Wind pressure

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC4: Wind pressure

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Z / P_W	Moment M_Y / M_V [kNm]
1	3,4,7,8	0 | Global XYZ	0.000	8.400	0.000
2	2,5,6,9	0 | Global XYZ	0.000	3.100	0.000

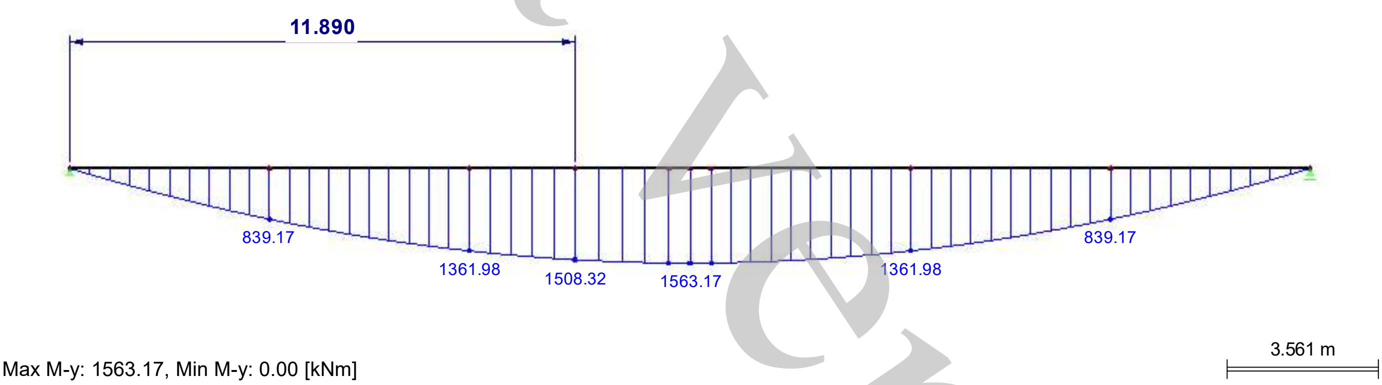
4.3 CROSS-SECTIONS - INTERNAL FORCES

Member No.	LC/CO	Node No.	Location x [m]	Forces [kN] N	V_z	Moments M_y [kNm]
	Section No. 2: SBD 1670/550/124/210/120/400/320/460					
1	CO1	MAX N	0.000	42.97	1408.49	0.00
1	LC1	MIN N	0.000	0.00	211.70	0.00
1	CO1	MAX V_z	0.000	42.97	1408.49	0.00
7	CO1	MIN V_z	4.700	42.97	-1408.49	0.00
4	CO1	MAX M_y	0.500	0.00	0.00	11598.40
1	LC2	MIN M_y	0.000	0.00	334.40	0.00

INTERNAL FORCES M_y

LC 1: Self-weight
Internal Forces M-y

Against Y-direction

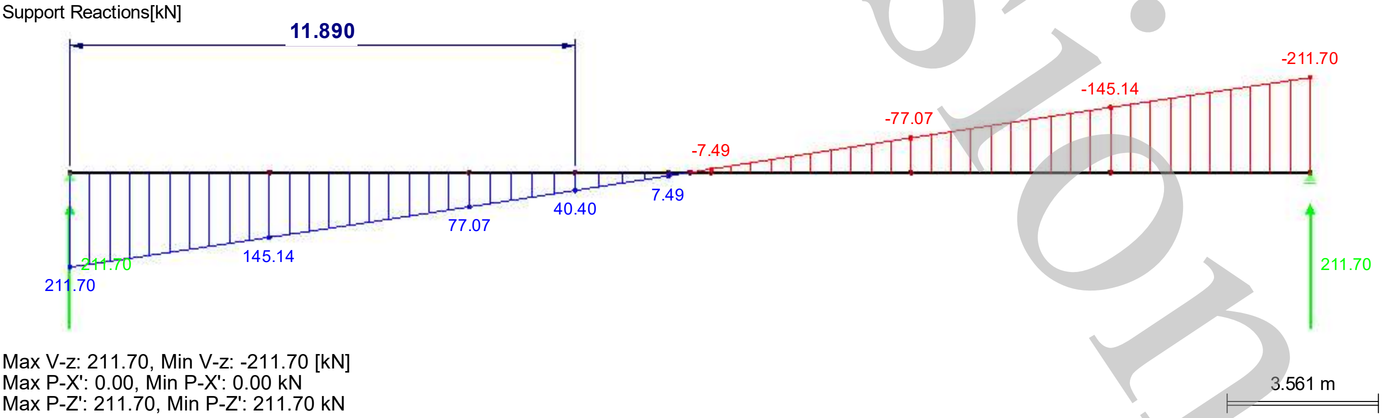


Max M-y: 1563.17, Min M-y: 0.00 [kNm]

INTERNAL FORCES V_z, SUPPORT REACTIONS

LC 1: Self-weight
Internal Forces V-z
Support Reactions[kN]

Against Y-direction

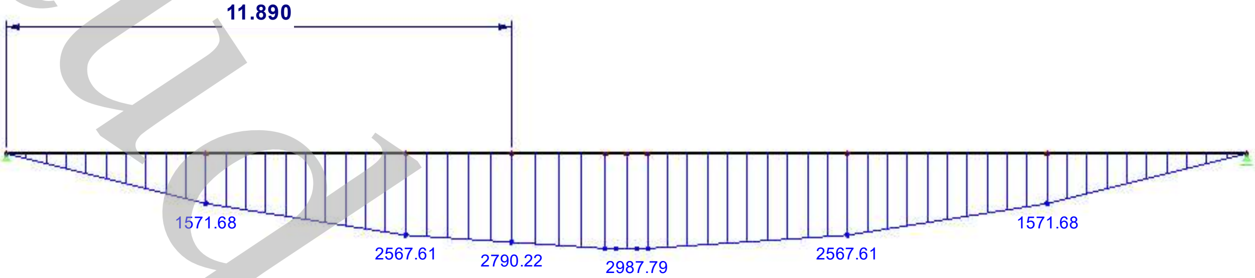


Max V-z: 211.70, Min V-z: -211.70 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 211.70, Min P-Z': 211.70 kN

INTERNAL FORCES M_y

LC 2: Roof structure
Internal Forces M-y

Against Y-direction



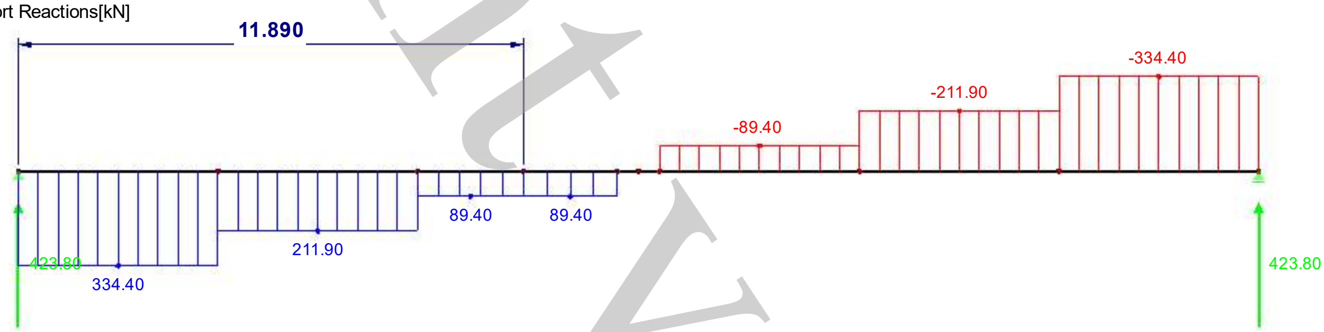
Max M-y: 2987.79, Min M-y: 0.00 [kNm]

3.561 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

LC 2: Roof structure
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



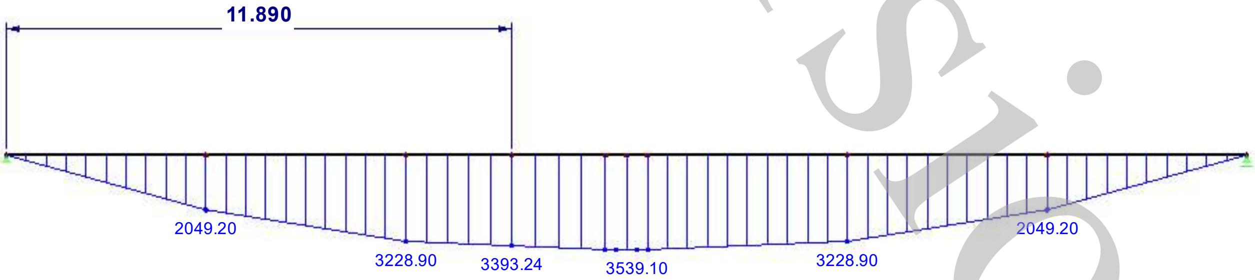
Max V-z: 334.40, Min V-z: -334.40 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 423.80, Min P-Z': 423.80 kN

3.561 m

INTERNAL FORCES M_y

LC 3: Snow
Internal Forces M-y

Against Y-direction



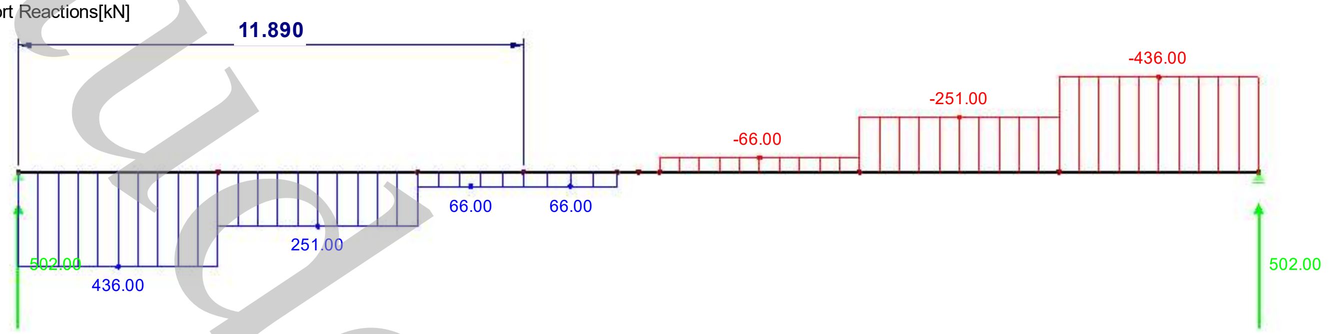
Max M-y: 3539.10, Min M-y: 0.00 [kNm]

3.561 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

LC 3: Snow
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



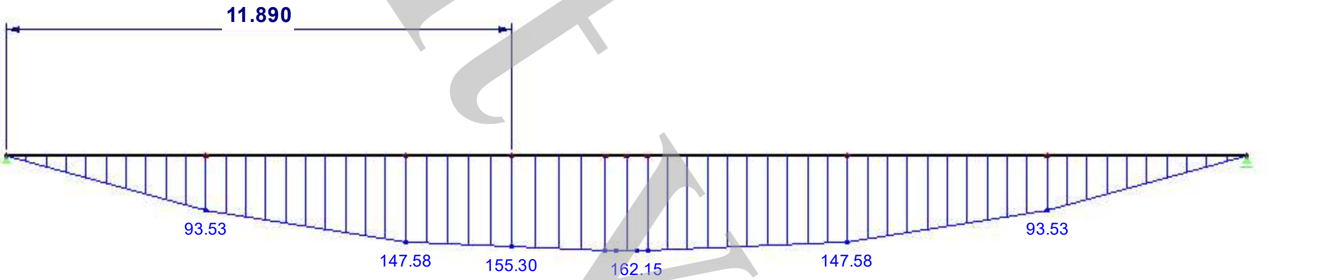
Max V-z: 436.00, Min V-z: -436.00 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 502.00, Min P-Z': 502.00 kN

3.561 m

INTERNAL FORCES M_y

LC 4: Wind pressure
Internal Forces M-y

Against Y-direction



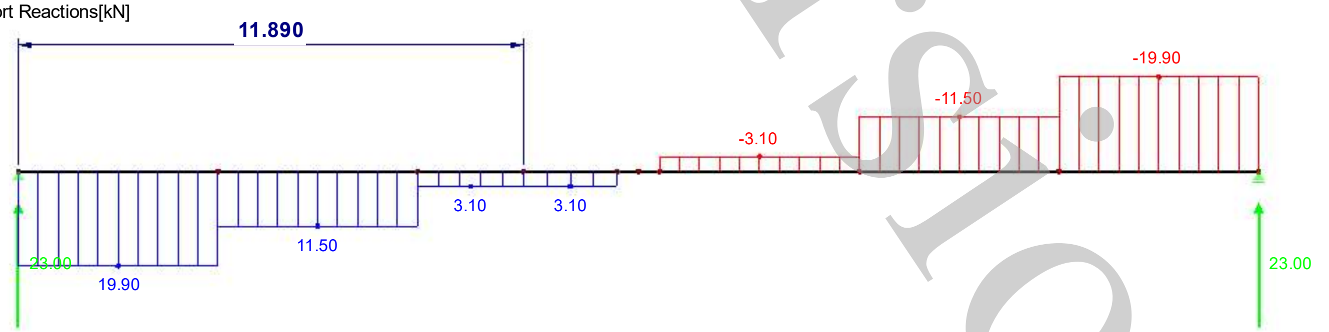
Max M-y: 162.15, Min M-y: 0.00 [kNm]

3.561 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

LC 4: Wind pressure
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



Max V-z: 19.90, Min V-z: -19.90 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 23.00, Min P-Z': 23.00 kN

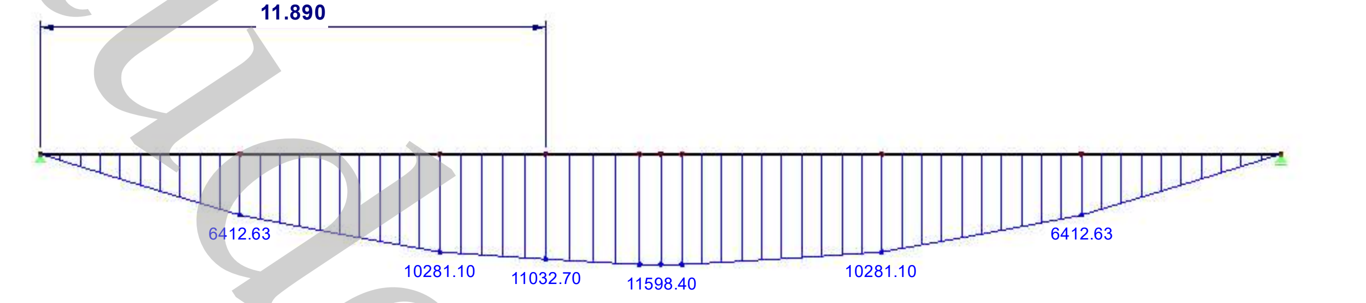
3.561 m

Project: Master Project Model: POS.P.03 - Prestressed Concrete Truss

INTERNAL FORCES M_y

CO 1: ULS G+S+W
Internal Forces M-y

Against Y-direction



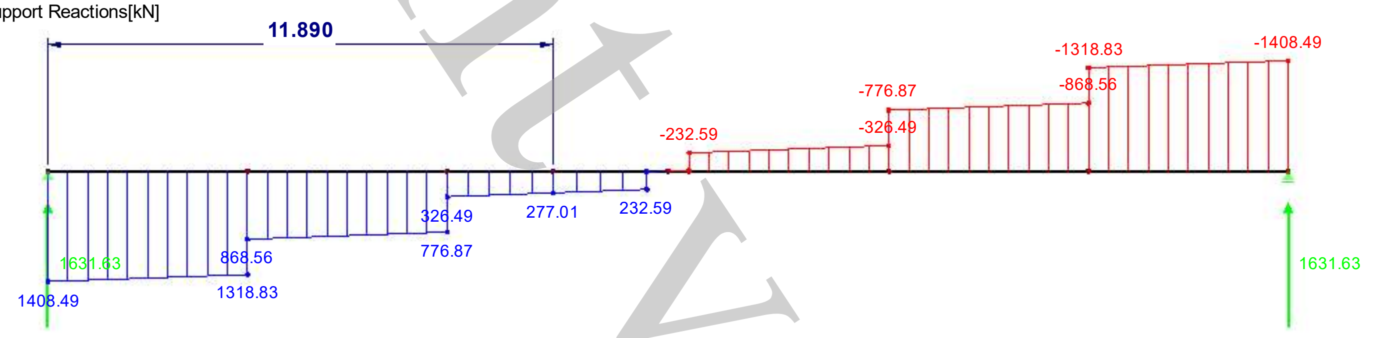
Max M-y: 11598.38, Min M-y: 0.00 [kNm]

3.561 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

CO 1: ULS G+S+W
Internal Forces V-z
Support Reactions[kN]

Against Y-direction

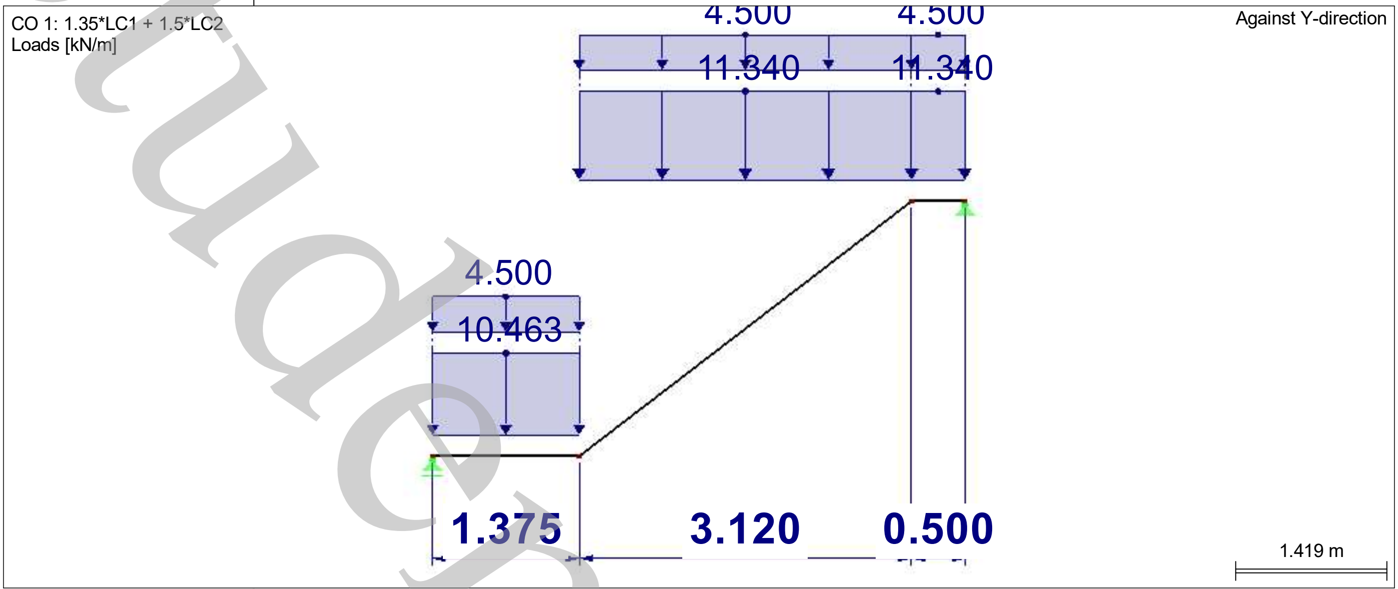


Max V-z: 1408.49, Min V-z: -1408.49 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 1631.63, Min P-Z': 1631.63 kN

3.561 m

B.2.3. Internal Forces POS. T.10 – Precast Concrete Staircase

MODEL



1.2 MATERIALS

Matl. No.	Modulus E [kN/cm²]	Modulus G [kN/cm²]	Spec. Weight γ [kN/m³]	Coeff. of Th. Exp. α [1/°C]	Partial Factor γ_M [-]	Material Model
1	Steel S 235 | EN 1993-1-1:2005-05					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic
2	Concrete C30/37 | DIN 1045-1:2008-08					
	2830.00	1179.17	25.00	1.00E-05	1.00	Isotropic Linear Elastic
3	Concrete C30/37 | DIN 1045-1:2008-08					
	2830.00	1179.17	25.00	1.00E-05	1.00	Isotropic Linear Elastic

1.3 CROSS-SECTIONS

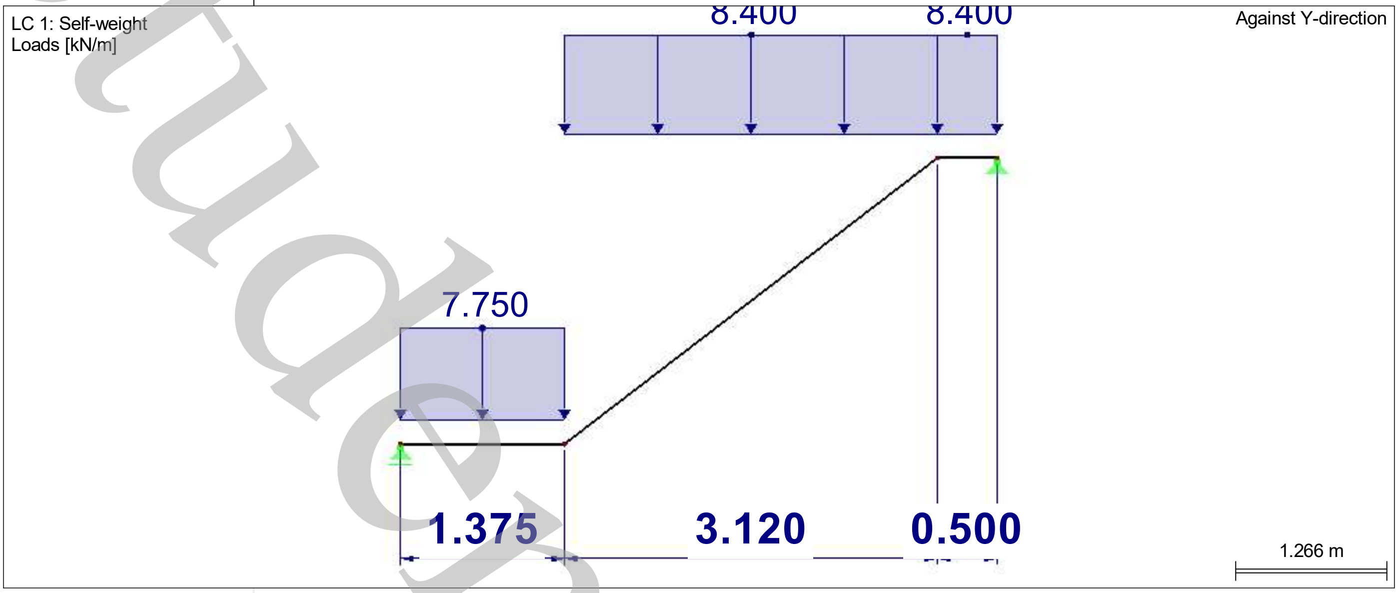
Section No.	Matl. No.	J [cm⁴]	A [cm²]	I_y [cm⁴]	A_y [cm²]	I_z [cm⁴]	A_z [cm²]	Principal Axes α [°]	Rotation α' [°]	Overall Dimensions [mm] Width b	Height h
1	Rectangle 20/100										
	2		20.00	166.67			16.67	0.00	0.00	20.0	100.0
2	Rectangle 100/25										
	2		25.00	13.02			20.83	0.00	0.00	100.0	25.0
3	Rectangle 1000/200										
	2		2000.00	66666.67			1666.67	0.00	0.00	1000.0	200.0

2.1 LOAD CASES

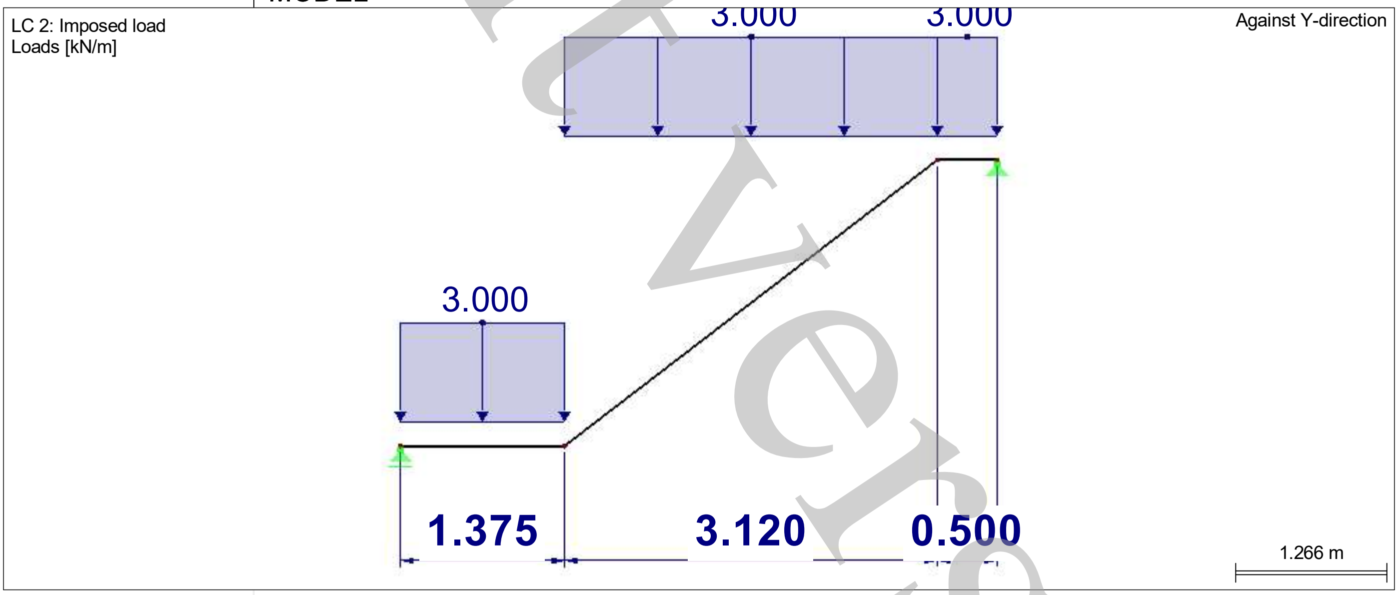
Load Case	Load Case Description	EN 1990 | DIN Action Category	Active	Self-Weight - Factor in Direction X	Y	Z
LC1	Self-weight	Permanent	☐			
LC2	Imposed load	Imposed - Category B: office areas	☐			

Project: Master Project Model: POS. T.10 - Concrete Staircase

MODEL



MODEL



2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		1.35*LC1 + 1.5*LC2	1	1.35	LC1	Self-weight
			2	1.50	LC2	Imposed load
CO2		LC1 + 0.5*LC2	1	1.00	LC1	Self-weight
			2	0.50	LC2	Imposed load

LC1
Self-weight

3.2 MEMBER LOADS

LC1: Self-weight

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	2	Force	Uniform	Z	Projection Z	p	8.400	kN/m
2	Members	1	Force	Uniform	Z	Projection Z	p	7.750	kN/m
3	Members	3	Force	Uniform	Z	Projection Z	p	8.400	kN/m

LC2
Imposed load

3.2 MEMBER LOADS

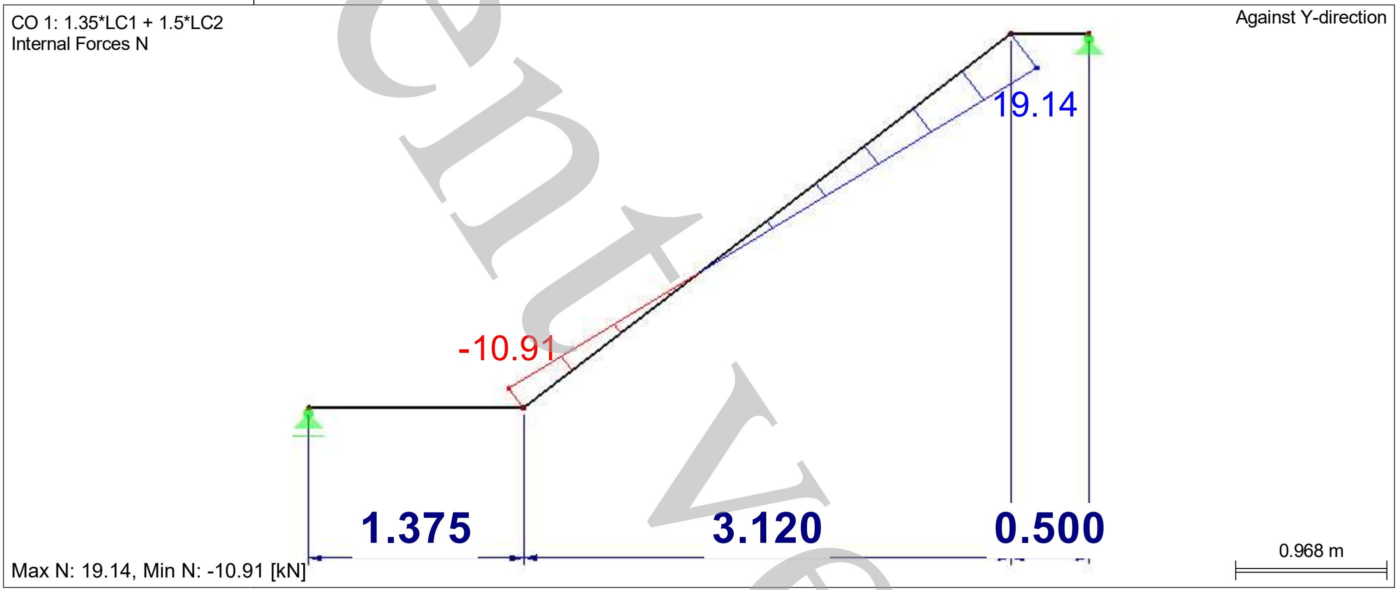
LC2: Imposed load

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	2,3	Force	Uniform	Z	Projection Z	p	3.000	kN/m
2	Members	1	Force	Uniform	Z	Projection Z	p	3.000	kN/m

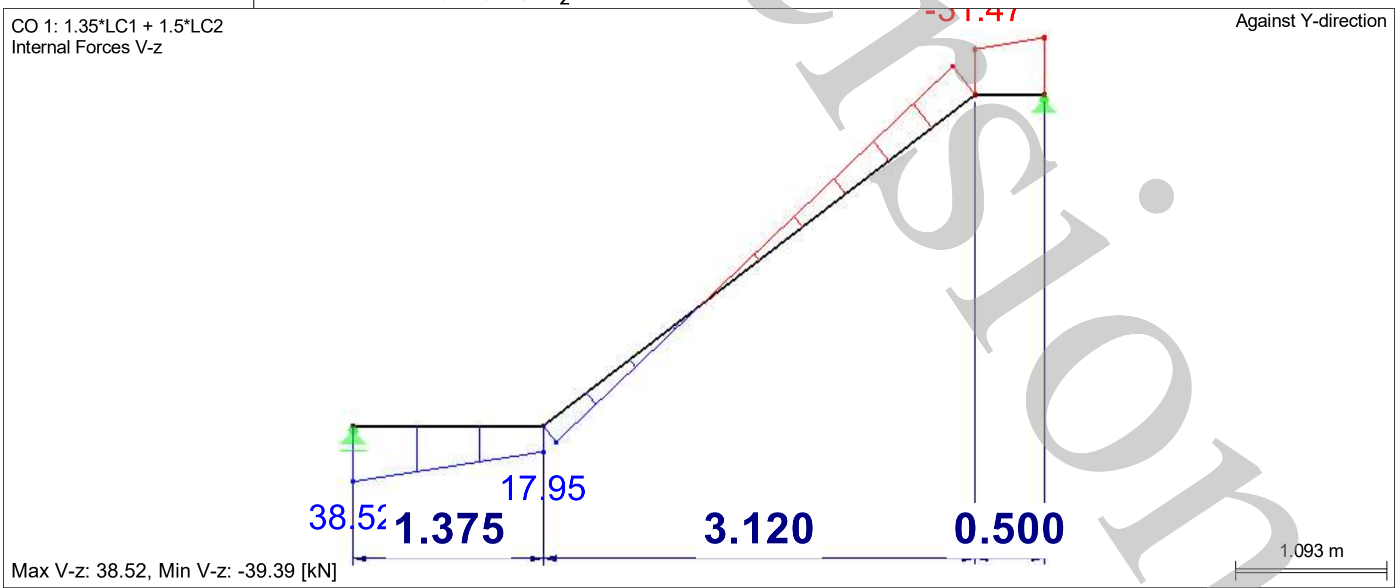
4.3 CROSS-SECTIONS - INTERNAL FORCES

Member No.	LC/CO	Node No.	Location x [m]	Forces [kN] N	V_z	Moments M_y [kNm]
	Section No. 3: Rectangle 1000/200					
2	CO1	MAX N	3.930	19.14	-24.99	17.72
2	CO1	MIN N	0.000	-10.91	14.25	38.82
1	CO1	MAX V_z	0.000	0.00	38.52	0.00
3	CO1	MIN V_z	0.500	0.00	-39.39	0.00
2	CO1	MAX M_y	1.572	1.11	-1.45	48.88
1	CO1	MIN M_y	0.000	0.00	38.52	0.00

INTERNAL FORCES N



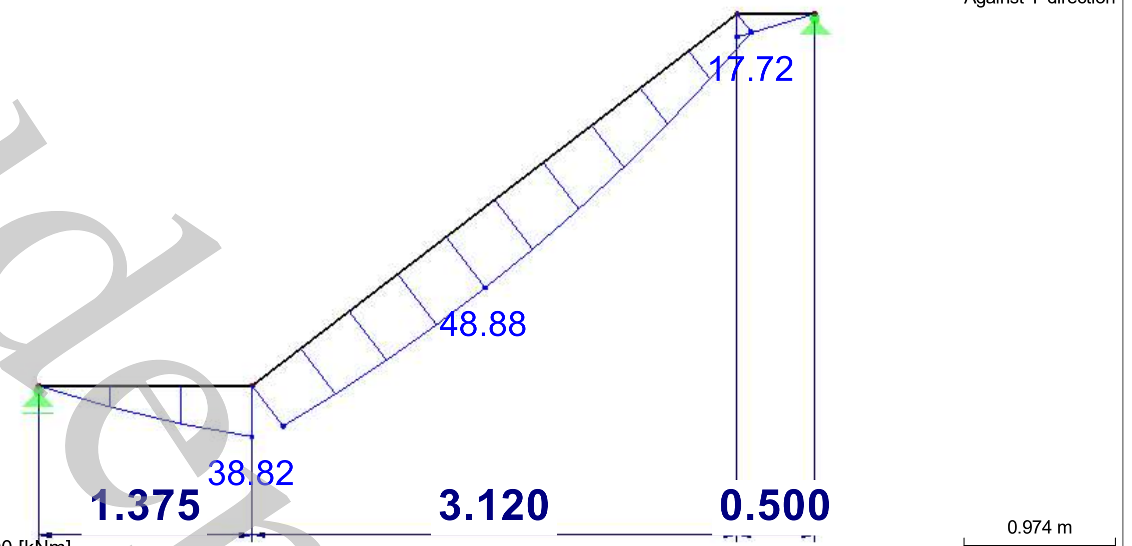
INTERNAL FORCES V_z



INTERNAL FORCES M_y

CO 1: 1.35*LC1 + 1.5*LC2
Internal Forces M-y

Against Y-direction



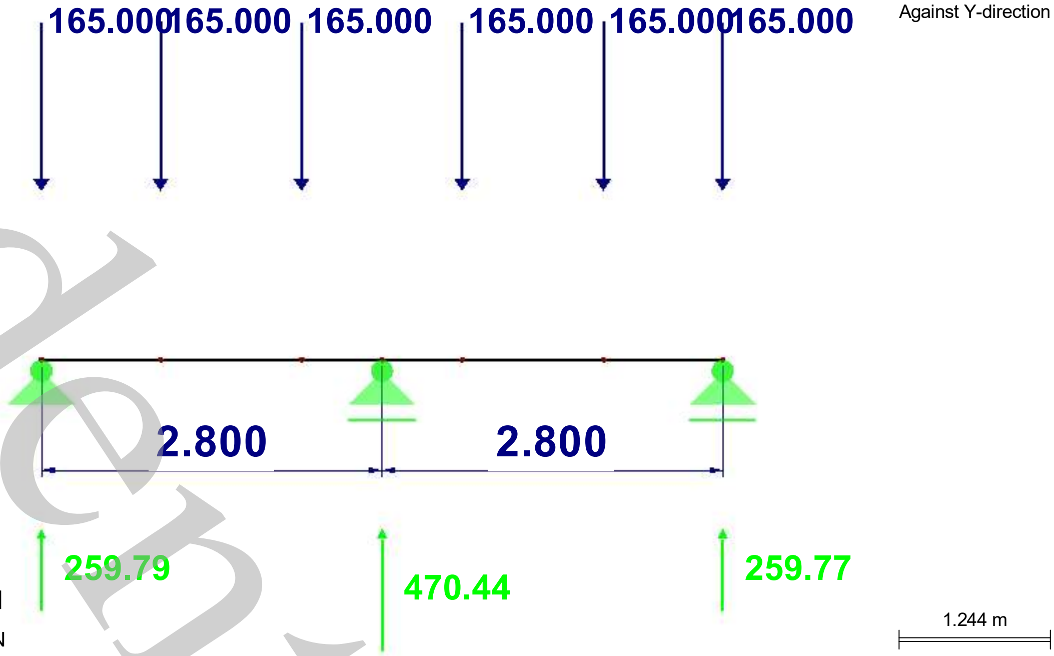
Max M-y: 48.88, Min M-y: 0.00 [kNm]

B.2.4. Internal Forces POS. W.14a – Strut-and-Tie Model Load Bearing Wall

INTERNAL FORCES N, SUPPORT REACTIONS

LC 1: Loads from Slab S.11
Loads [kN]
Internal Forces N
Support Reactions[kN]

Against Y-direction

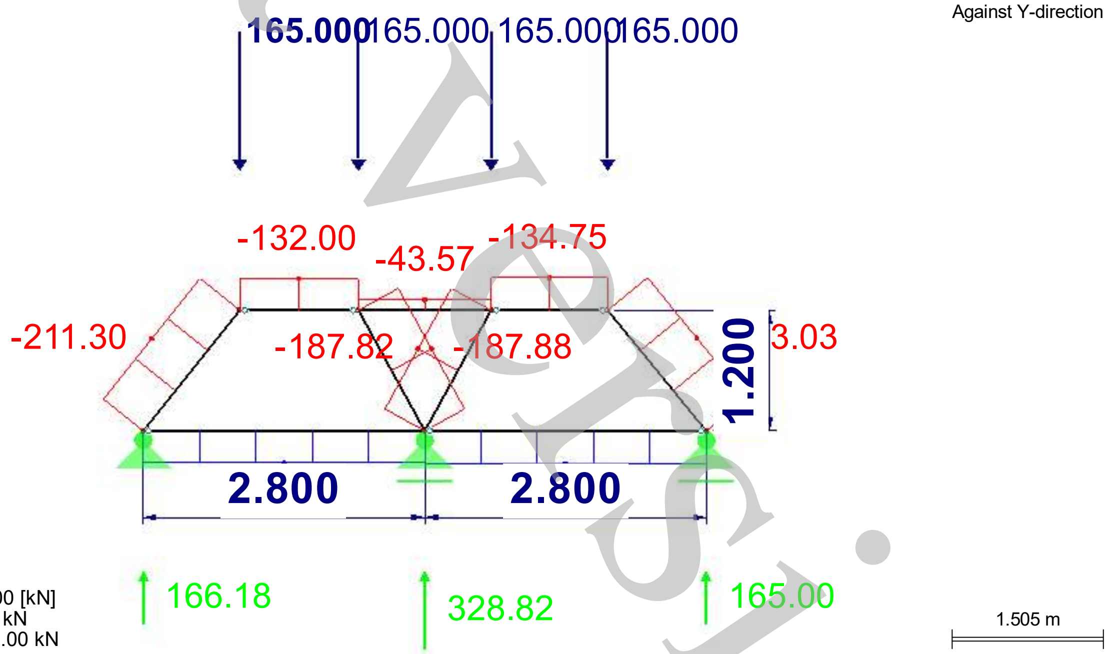


Max N: 701.10, Min N: -2158.00 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 3393.02, Min P-Z': 0.00 kN

INTERNAL FORCES N, SUPPORT REACTIONS

LC 1: Loads from Slab S.11
Loads [kN]
Internal Forces N
Support Reactions[kN]

Against Y-direction

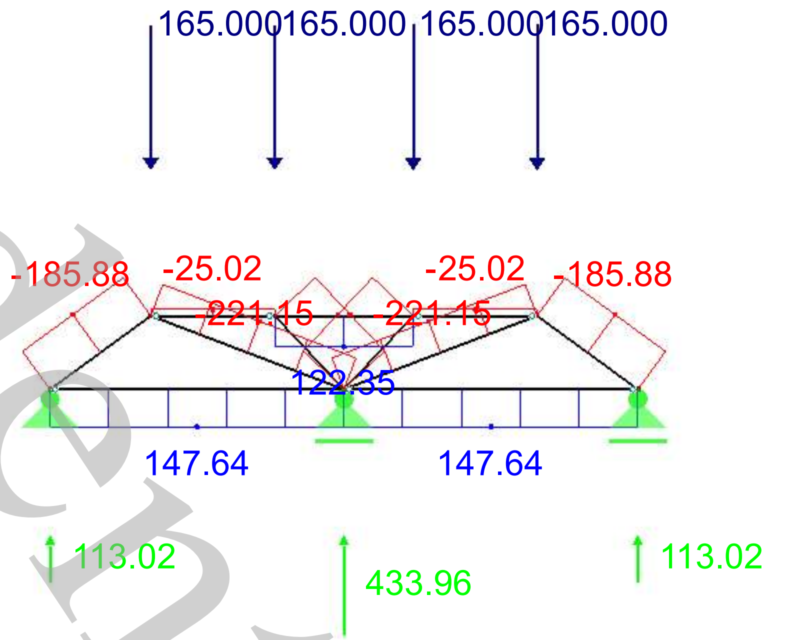


Max N: 701.10, Min N: -2158.00 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 3393.02, Min P-Z': 0.00 kN

INTERNAL FORCES N, SUPPORT REACTIONS

LC 1: Loads from Slab S.11
Loads [kN]
Internal Forces N
Support Reactions[kN]

Against Y-direction



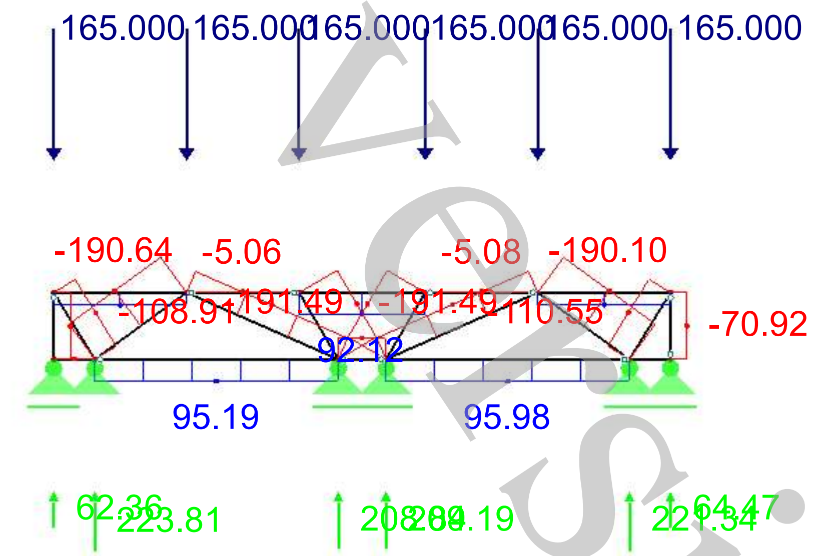
Max N: 701.10, Min N: -2158.00 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 3393.02, Min P-Z': 0.00 kN

1.441 m

INTERNAL FORCES N, SUPPORT REACTIONS

LC 1: Loads from Slab S.11
Loads [kN]
Internal Forces N
Support Reactions[kN]

Against Y-direction



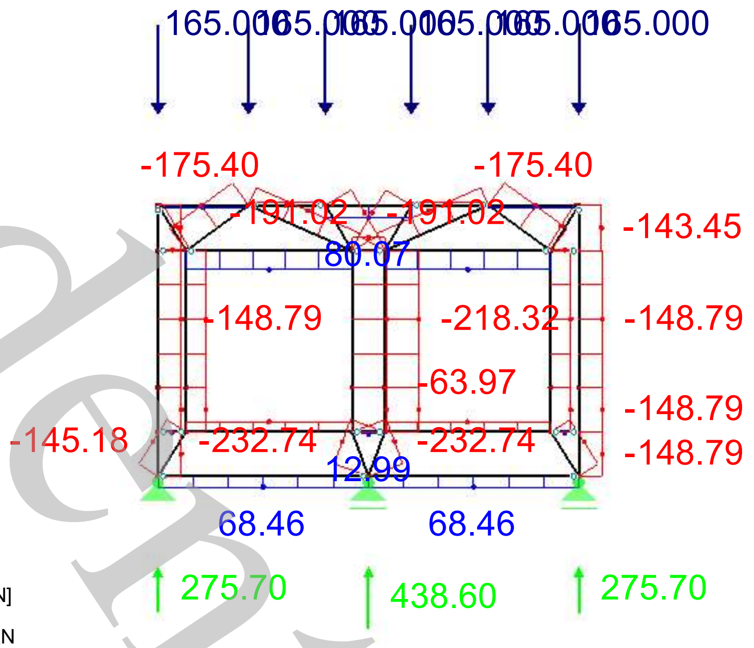
Max N: 701.10, Min N: -2158.00 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 3393.02, Min P-Z': 0.00 kN

1.585 m

■ INTERNAL FORCES N, SUPPORT REACTIONS

LC 1: Loads from Slab S.11
Loads [kN]
Internal Forces N
Support Reactions[kN]

Against Y-direction



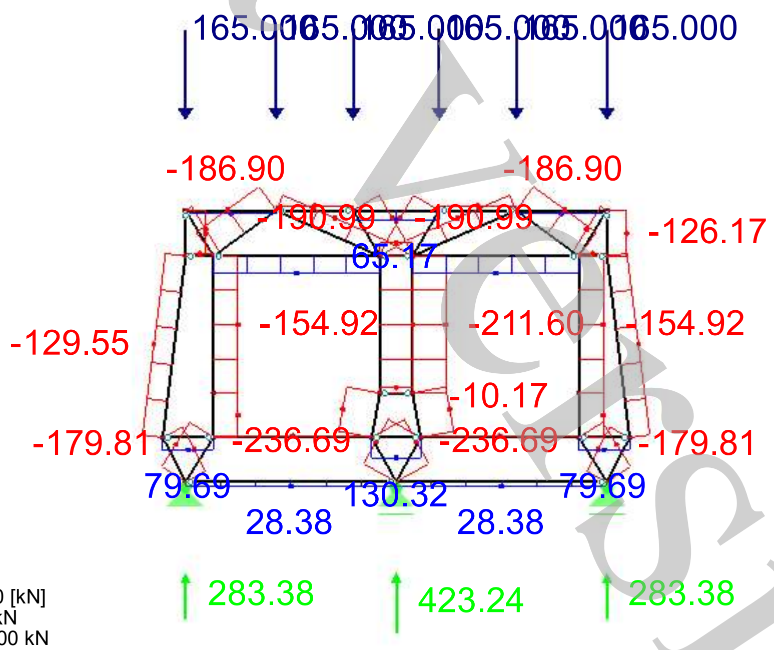
Max N: 701.10, Min N: -2158.00 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 3393.02, Min P-Z': 0.00 kN

2.321 m

■ INTERNAL FORCES N, SUPPORT REACTIONS

LC 1: Loads from Slab S.11
Loads [kN]
Internal Forces N
Support Reactions[kN]

Against Y-direction



Max N: 701.10, Min N: -2158.00 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 3393.02, Min P-Z': 0.00 kN

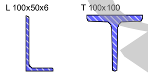
2.321 m

B.2.5. Internal Forces POS. V.31 – Steel Canopy

1.2 MATERIALS

Matl. No.	Modulus E [kN/cm²]	Modulus G [kN/cm²]	Spec. Weight γ [kN/m³]	Coeff. of Th. Exp. α [1/°C]	Partial Factor γ_M [-]	Material Model
1	Steel S 235 | EN 1993-1-1:2005-05					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic

1.3 CROSS-SECTIONS



Section No.	Matl. No.	J [cm⁴] A [cm²]	I_y [cm⁴] A_y [cm²]	I_z [cm⁴] A_z [cm²]	Principal Axes α [°]	Rotation α' [°]	Overall Dimensions [mm] Width b	Height h
1	L 100x50x6 | EN 10056-1:1998							
	1	8.71	95.40	4.70	0.00	0.00	50.0	100.0
2	T 100x100 | EN 10055:1995							
	1	20.90	179.00	8.03	0.00	0.00	100.0	100.0

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		1.35*LC1 + 1.35*LC2 + 1.5*LC3 + 1.5*LC4	1	1.35	LC1	Self-weight Span
			2	1.35	LC2	Self-weight Cantilever
			3	1.50	LC3	Snow Span
			4	1.50	LC4	Snow Cantilever
CO2		1.35*LC1 + LC2 + 1.5*LC3	1	1.35	LC1	Self-weight Span
			2	1.00	LC2	Self-weight Cantilever
			3	1.50	LC3	Snow Span
CO3		Self-weight	1	1.00	LC1	Self-weight Span
			2	1.00	LC2	Self-weight Cantilever
CO4		Snow	1	1.00	LC3	Snow Span
			2	1.00	LC4	Snow Cantilever

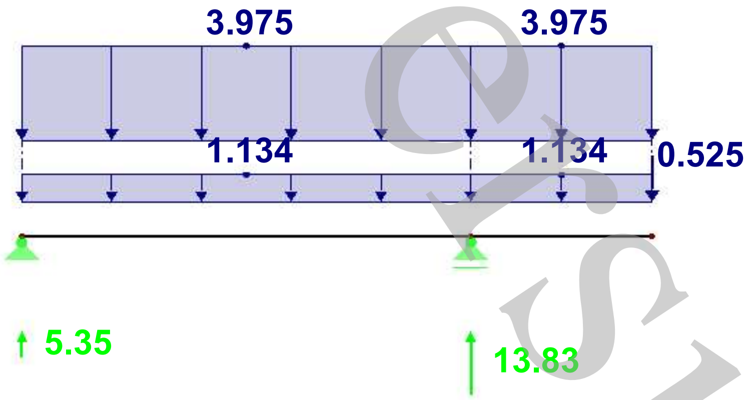
2.6 RESULT COMBINATIONS

Result Combin	Description	Loading
RC1		CO1 or CO2

SUPPORT REACTIONS

CO 1: 1.35*LC1 + 1.35*LC2 + 1.5*LC3 + 1.5*LC4
Loads [kN/m], [kN]
Support Reactions[kN]

Against Y-direction



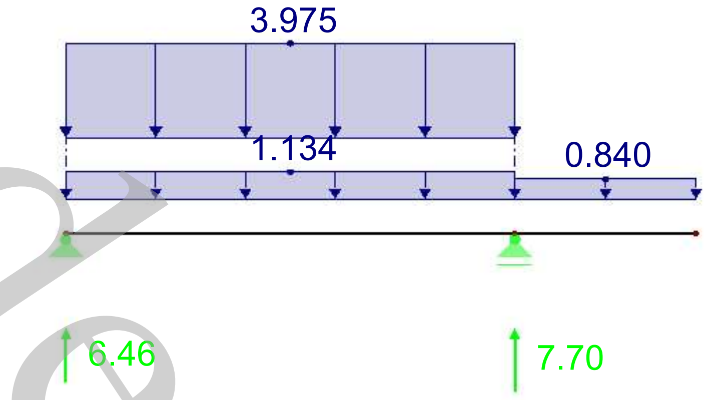
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 13.83, Min P-Z': 5.35 kN

0.877 m

SUPPORT REACTIONS

CO 2: 1.35*LC1 + LC2 + 1.5*LC3
Loads [kN/m]
Support Reactions[kN]

Against Y-direction



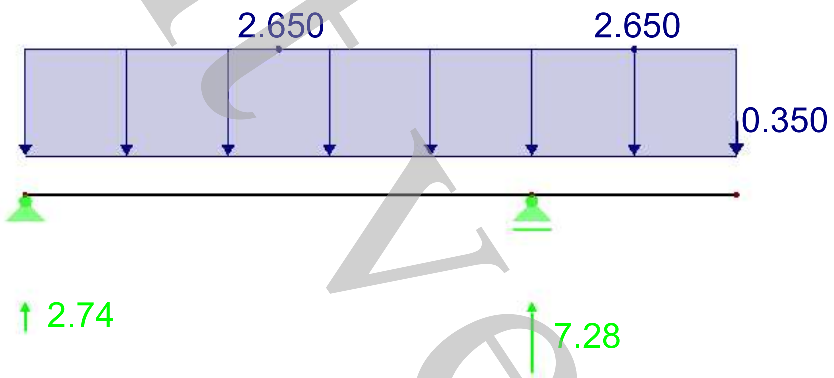
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 7.70, Min P-Z': 6.46 kN

0.877 m

SUPPORT REACTIONS

CO 4: Snow
Loads [kN/m], [kN]
Support Reactions[kN]

Against Y-direction



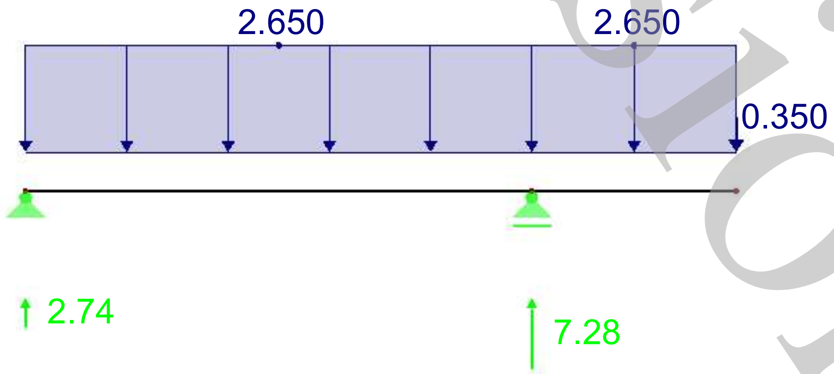
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 7.28, Min P-Z': 2.74 kN

0.776 m

SUPPORT REACTIONS

CO 4: Snow
Loads [kN/m], [kN]
Support Reactions[kN]

Against Y-direction



Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 7.28, Min P-Z': 2.74 kN

0.776 m

LC1
Self-weight Span

3.2 MEMBER LOADS
LC1: Self-weight Span

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	1	Force	Uniform	Z	True Length	p	0.840	kN/m

LC2
Self-weight Cantilever

3.2 MEMBER LOADS
LC2: Self-weight Cantilever

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	2	Force	Uniform	Z	True Length	p	0.840	kN/m

LC3
Snow Span

3.2 MEMBER LOADS
LC3: Snow Span

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	1	Force	Uniform	Z	True Length	p	2.650	kN/m

LC4
Snow Cantilever

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM
LC4: Snow Cantilever

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Z / P_W	Moment M_Y / M_V [kNm]
1	3	0 | Global XYZ	0.000	0.350	0.000

3.2 MEMBER LOADS
LC4: Snow Cantilever

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	2	Force	Uniform	Z	True Length	p	2.650	kN/m

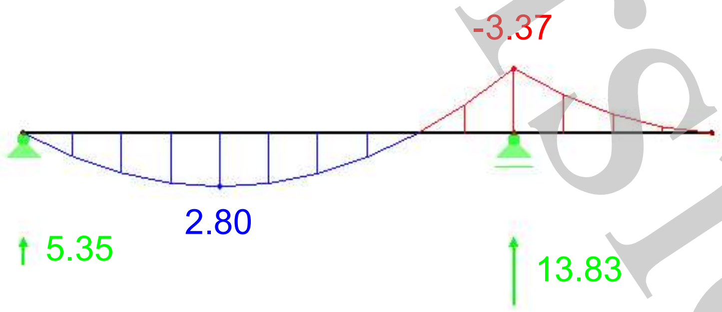
4.3 CROSS-SECTIONS - INTERNAL FORCES

Member No.	LC/CO	Node No.	Location x [m]	Forces [kN] N	V_z	Moments M_y [kNm]
Section No. 2: T 100x100 | EN 10055:1995						
1	CO4	MAX N	0.000	0.01	2.74	0.00
2	CO4	MIN N	0.000	0.00	3.13	-1.83
1	CO2	MAX V_z	0.000	0.00	6.46	0.00
1	CO1	MIN V_z	2.600	0.00	-7.94	-3.37
1	CO2	MAX M_y	1.300	0.00	-0.18	4.09
1	CO1	MIN M_y	2.600	0.00	-7.94	-3.37

INTERNAL FORCES M_y, SUPPORT REACTIONS

CO 1: 1.35*LC1 + 1.35*LC2 + 1.5*LC3 + 1.5*LC4
Internal Forces M-y
Support Reactions[kN]

Against Y-direction



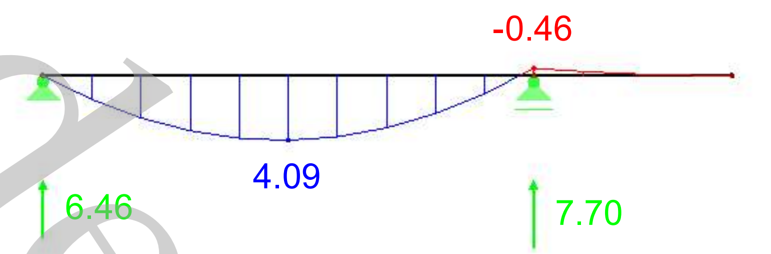
Max M-y: 2.80, Min M-y: -3.37 [kNm]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 13.83, Min P-Z': 5.35 kN

0.8 m

INTERNAL FORCES M_y, SUPPORT REACTIONS

CO 2: 1.35*LC1 + LC2 + 1.5*LC3
Internal Forces M-y
Support Reactions[kN]

Against Y-direction



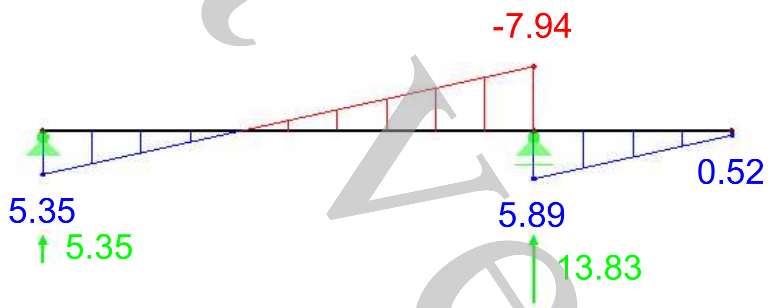
Max M-y: 4.09, Min M-y: -0.46 [kNm]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 7.70, Min P-Z': 6.46 kN

0.8 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

CO 1: 1.35*LC1 + 1.35*LC2 + 1.5*LC3 + 1.5*LC4
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



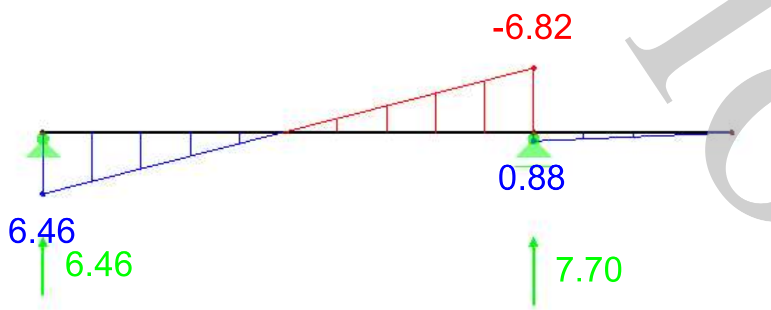
Max V-z: 5.89, Min V-z: -7.94 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 13.83, Min P-Z': 5.35 kN

0.8 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

CO 2: 1.35*LC1 + LC2 + 1.5*LC3
Internal Forces V-z
Support Reactions[kN]

Against Y-direction



Max V-z: 6.46, Min V-z: -6.82 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 7.70, Min P-Z': 6.46 kN

0.8 m

4.3 CROSS-SECTIONS - INTERNAL FORCES

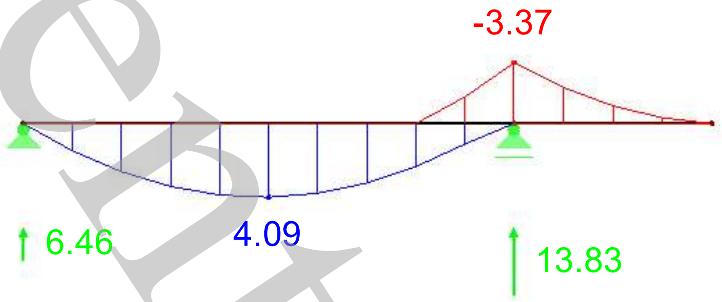
Result Combinations

Member No.	RC	Node No.	Location x [m]		Forces [kN] N	V_z	Moments M_y [kNm]	Corresponding Load Cases
	Section No. 2: T 100x100 | EN 10055:1995							
1	RC1		0.000	MAX N	0.00	0.00	0.00	
1	RC1		0.000	MIN N	0.00	0.00	0.00	
1	RC1		0.000	MAX V_z	0.00	6.46	0.00	CO 2
1	RC1		2.600	MIN V_z	0.00	-7.94	-3.37	CO 1
1	RC1		1.300	MAX M_y	0.00	-0.18	4.09	CO 2
1	RC1		2.600	MIN M_y	0.00	-7.94	-3.37	CO 1

INTERNAL FORCES M_y, SUPPORT REACTIONS

Against Y-direction

RC 1: CO1 or CO2
Internal Forces M-y
Support Reactions[kN]
Result Combinations: Max and Min Values



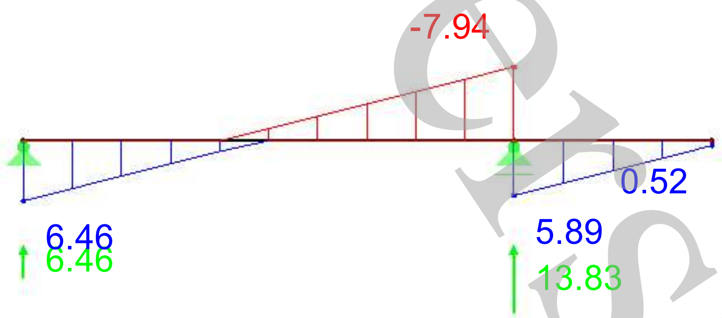
Max M-y: 4.09, Min M-y: -3.37 [kNm]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 13.83, Min P-Z': 0.00 kN

0.8 m

INTERNAL FORCES V_z, SUPPORT REACTIONS

Against Y-direction

RC 1: CO1 or CO2
Internal Forces V-z
Support Reactions[kN]
Result Combinations: Max and Min Values



Max V-z: 6.46, Min V-z: -7.94 [kN]
Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Z': 13.83, Min P-Z': 0.00 kN

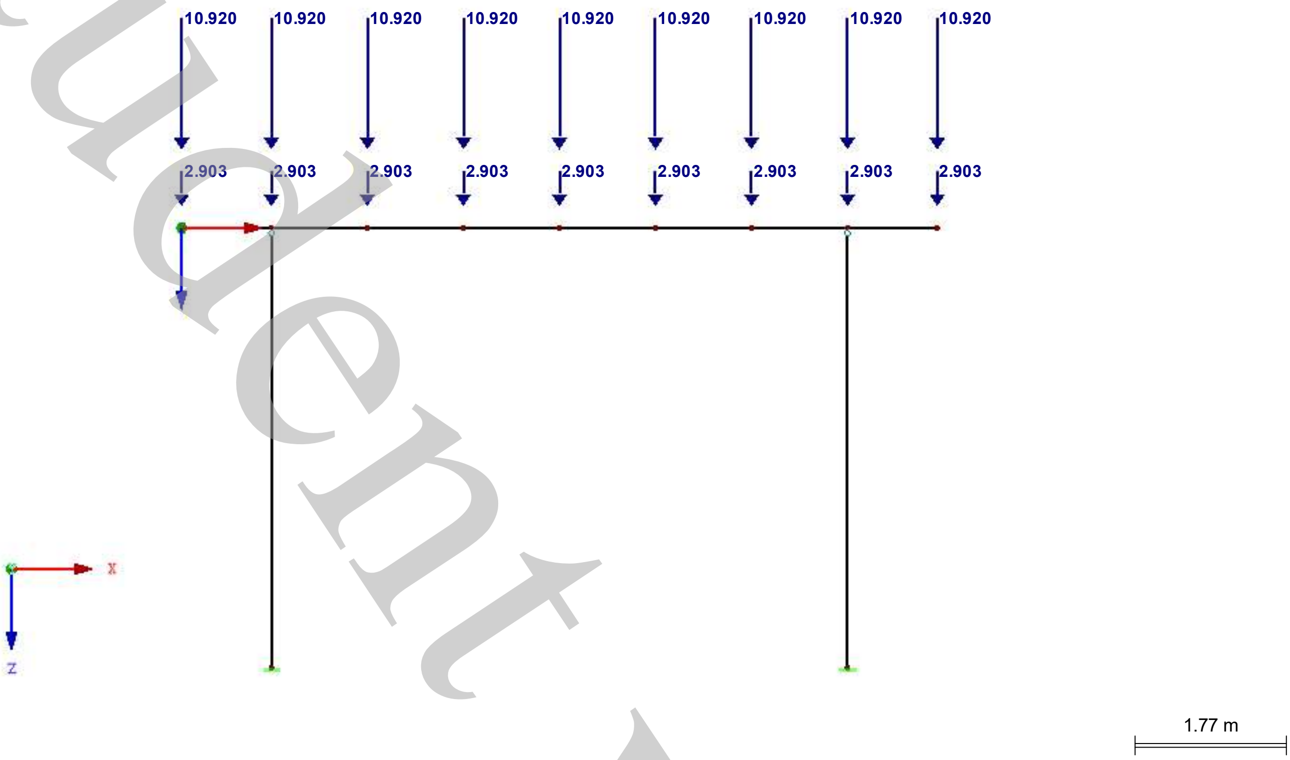
0.8 m

Project: Master Project Model: POS. V.31.2 - Steel Canopy

MODEL

CO 1: 1.35*LC1 + 1.5*LC2
Loads [kN]

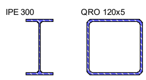
Against Y-direction



1.2 MATERIALS

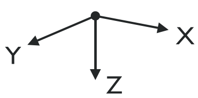
Matl. No.	Modulus E [kN/cm²]	Modulus G [kN/cm²]	Spec. Weight γ [kN/m³]	Coeff. of Th. Exp. α [1/°C]	Partial Factor γ_M [-]	Material Model
1	Steel S 235 | EN 1993-1-1:2005-05					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic

1.3 CROSS-SECTIONS



Section No.	Matl. No.	J [cm⁴] A [cm²]	I_y [cm⁴] A_y [cm²]	I_z [cm⁴] A_z [cm²]	Principal Axes α [°]	Rotation α' [°]	Overall Dimensions [mm] Width b	Height h
1	IPE 300 | Euronorm 19-57							
	1	53.80	8360.00	19.82	0.00	0.00	150.0	300.0
2	QRO 120x5 | DIN 59410:1974							
	1	22.60	495.00	9.68	0.00	0.00	120.0	120.0

1.8 NODAL SUPPORTS



Support No.	Nodes No.	Rotation [°] about Y	Support or Spring [kN/m] [kNm/rad] $u_{X'}$	$u_{Z'}$	$\varphi_{Y'}$	Comment
1	5,6	0.00	☒	☒	☒	

2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | DIN Action Category	Active	Self-Weight - Factor in Direction X	Y	Z
LC1	Permanent	Permanent	☒	0.000		1.000
LC2	Snow	Snow (H ≤ 1000 m a.s.l.)	☐			

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		1.35*LC1 + 1.5*LC2	1	1.35	LC1	Permanent
			2	1.50	LC2	Snow

LC1
Permanent

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC1: Permanent

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Z / P_W	Moment M_Y / M_V [kNm]
1	1-4,7-11	0 | Global XYZ	0.000	2.150	0.000

LC2
Snow

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC2: Snow

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Z / P_W	Moment M_Y / M_V [kNm]
1	1-4,7-11	0 | Global XYZ	0.000	7.280	0.000

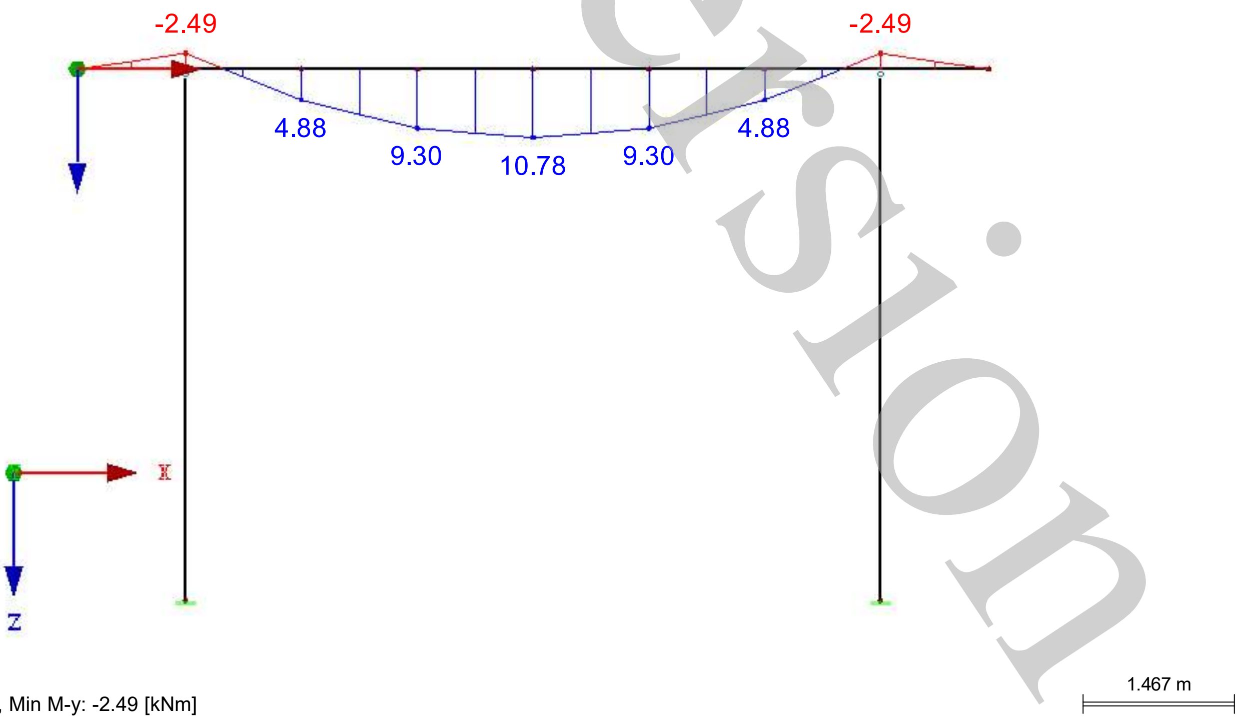
4.3 CROSS-SECTIONS - INTERNAL FORCES

Member No.	LC/CO	Node No.	Location x [m]	Forces [kN] N	V_z	Moments M_y [kNm]
	Section No. 1: IPE 300 | Euronorm 19-57					
1	LC1	MAX N	0.000	0.00	-2.15	0.00
1	LC1	MIN N	0.000	0.00	-2.15	0.00
2	CO1	MAX V_z	0.000	0.00	36.48	-14.83
10	CO1	MIN V_z	1.123	0.00	-36.48	-14.83
7	CO1	MAX M_y	1.123	0.00	6.91	58.28
1	CO1	MIN M_y	1.050	0.00	-14.42	-14.83
	Section No. 2: QRO 120x5 | DIN 59410:1974					
4	LC1	MAX N	0.000	-11.54	0.00	0.00
4	CO1	MIN N	5.150	-65.95	0.00	0.00
4	LC1	MAX V_z	0.000	-11.54	0.00	0.00
4	LC1	MIN V_z	0.000	-11.54	0.00	0.00
4	LC1	MAX M_y	0.000	-11.54	0.00	0.00
4	LC1	MIN M_y	0.000	-11.54	0.00	0.00

INTERNAL FORCES M_y

LC 1: Permanent
Internal Forces M-y

Against Y-direction

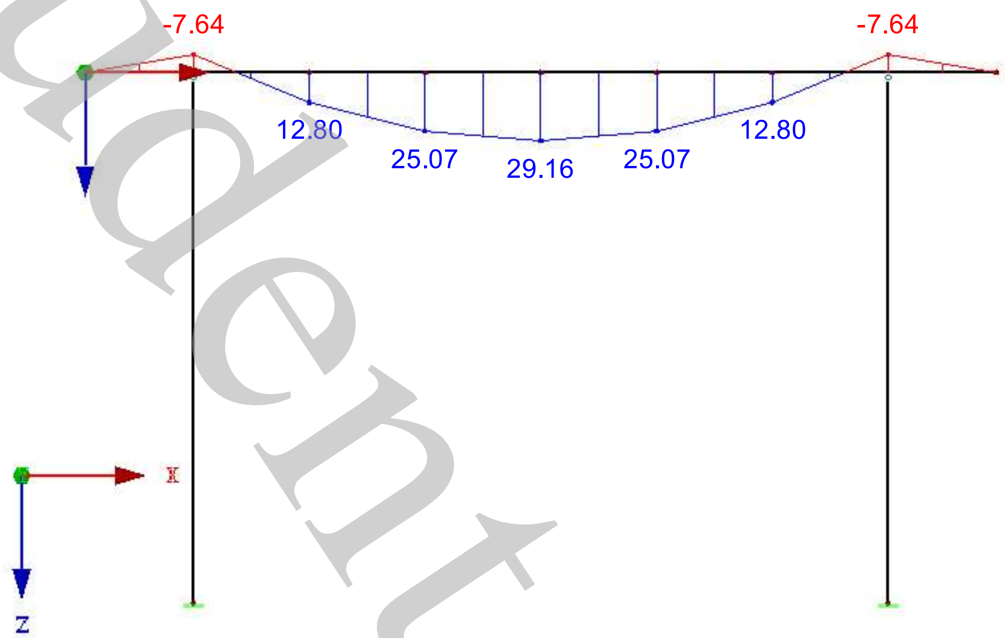


Max M-y: 10.78, Min M-y: -2.49 [kNm]

INTERNAL FORCES M_y

LC 2: Snow
Internal Forces M-y

Against Y-direction



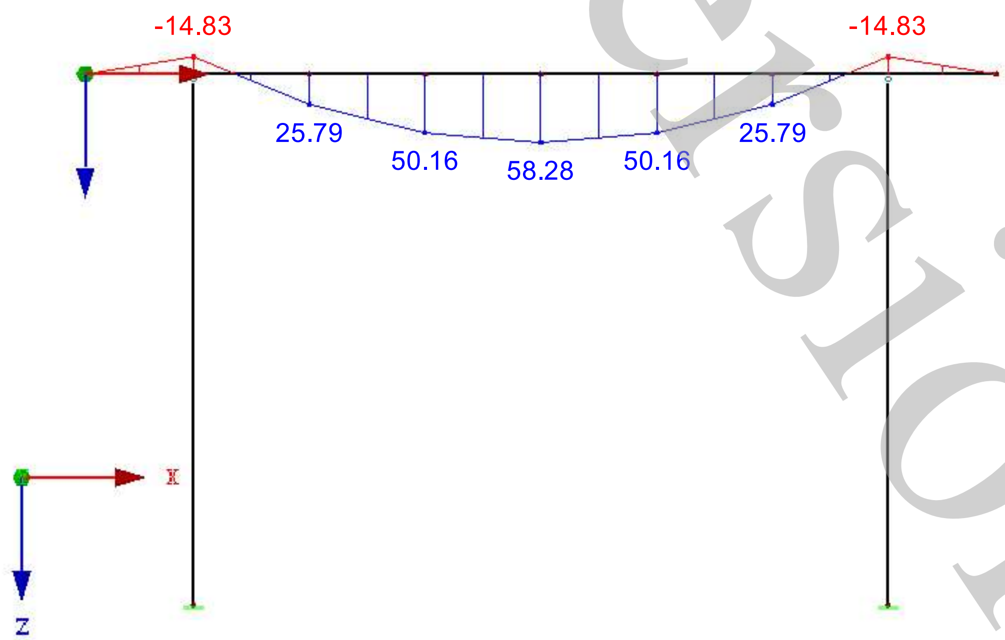
Max M-y: 29.16, Min M-y: -7.64 [kNm]

1.467 m

INTERNAL FORCES M_y

CO 1: 1.35*LC1 + 1.5*LC2
Internal Forces M-y

Against Y-direction



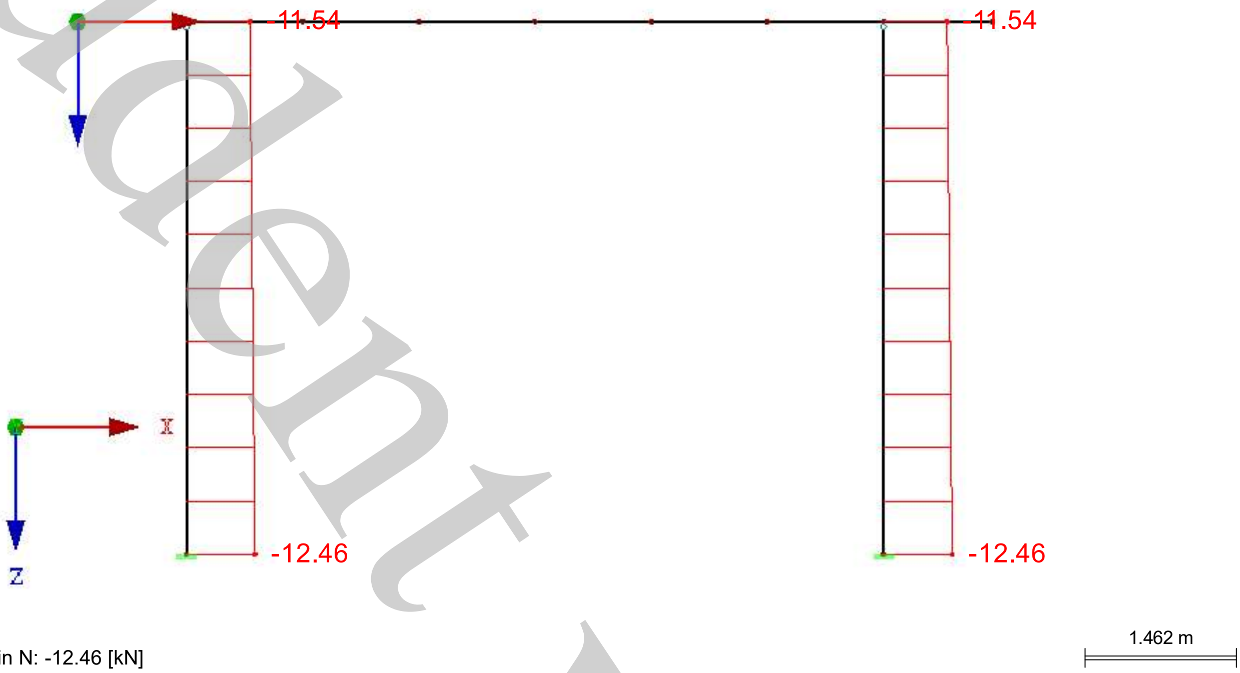
Max M-y: 58.28, Min M-y: -14.83 [kNm]

1.467 m

INTERNAL FORCES N

LC 1: Permanent
Internal Forces N

Against Y-direction

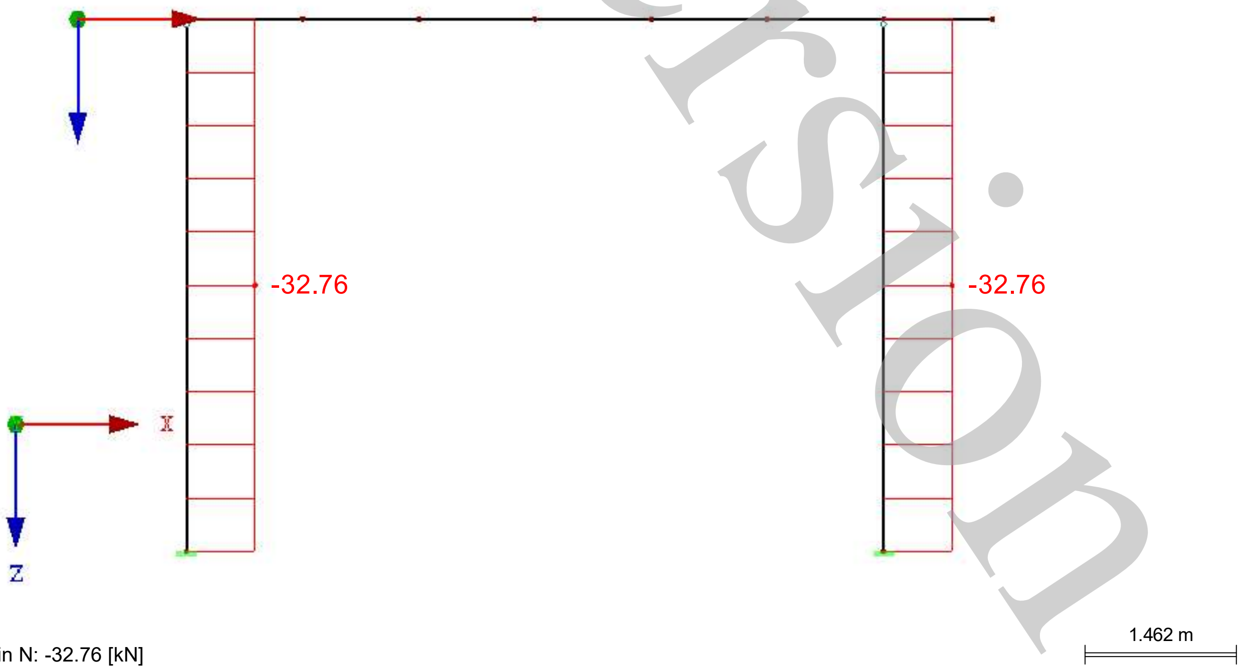


Max N: 0.00, Min N: -12.46 [kN]

INTERNAL FORCES N

LC 2: Snow
Internal Forces N

Against Y-direction

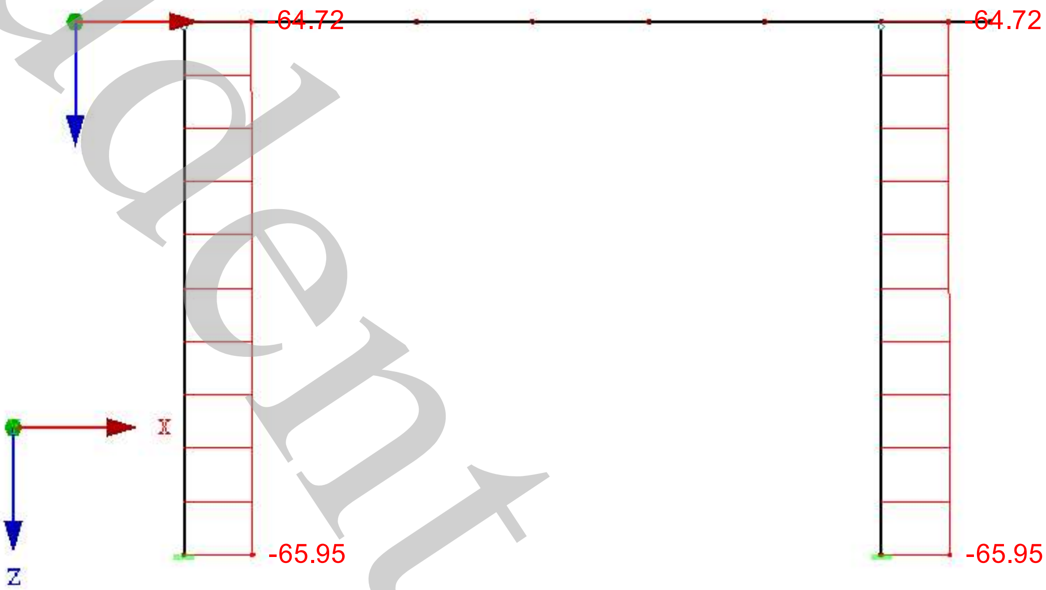


Max N: 0.00, Min N: -32.76 [kN]

INTERNAL FORCES N

CO 1: 1.35*LC1 + 1.5*LC2
Internal Forces N

Against Y-direction



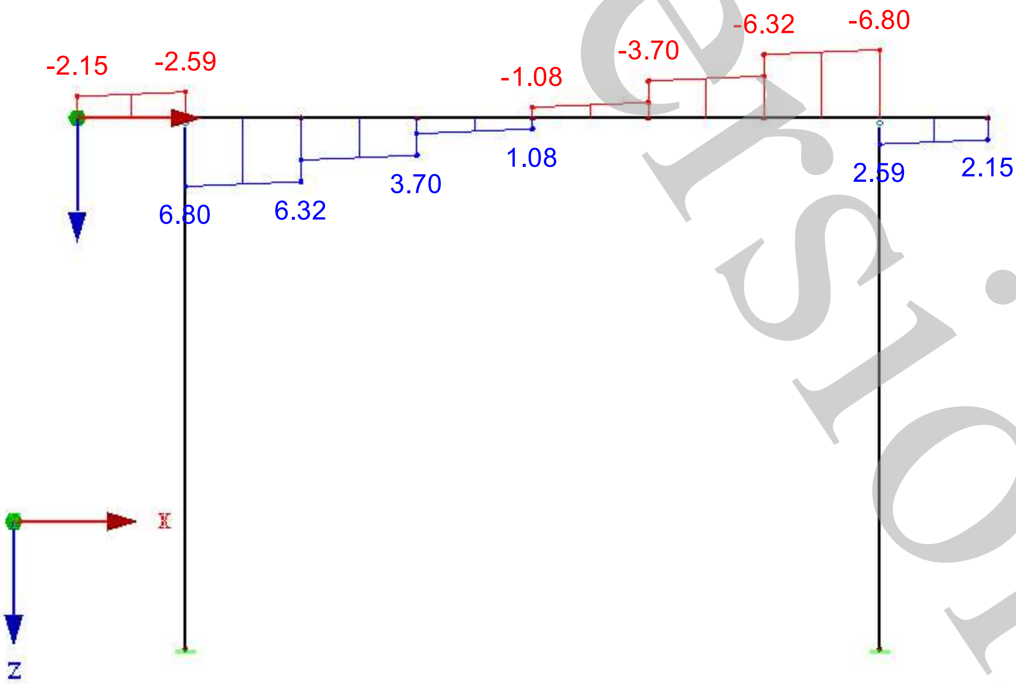
Max N: 0.00, Min N: -65.95 [kN]

1.462 m

INTERNAL FORCES V_z

LC 1: Permanent
Internal Forces V-z

Against Y-direction



Max V-z: 6.80, Min V-z: -6.80 [kN]

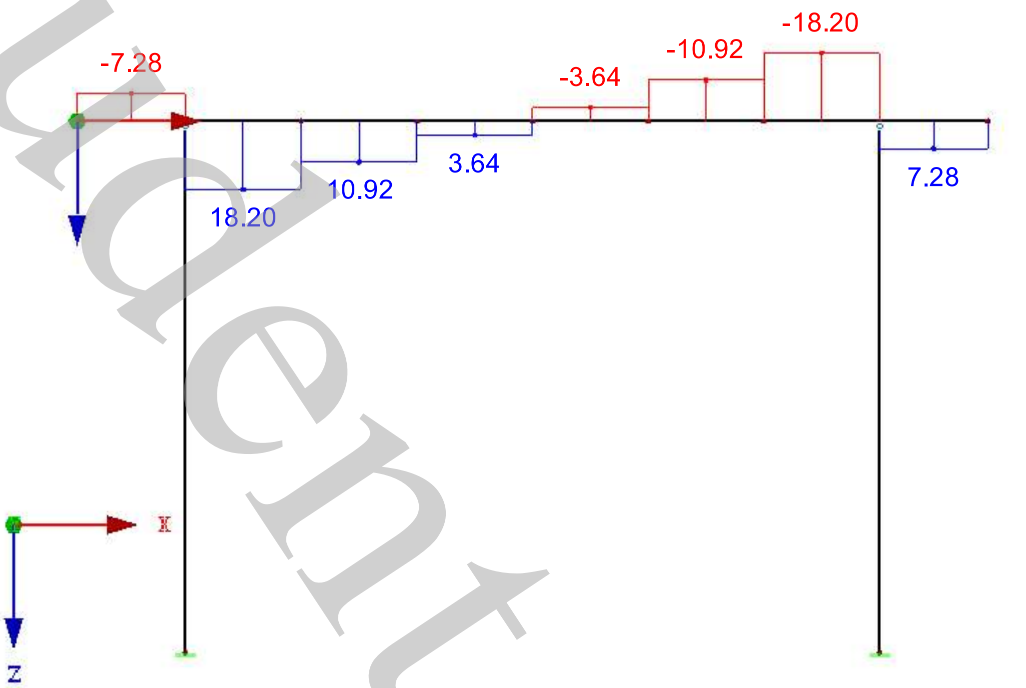
1.467 m

Project: Master Project Model: POS. V.31.2 - Steel Canopy

INTERNAL FORCES V_z

LC 2: Snow
Internal Forces V-z

Against Y-direction



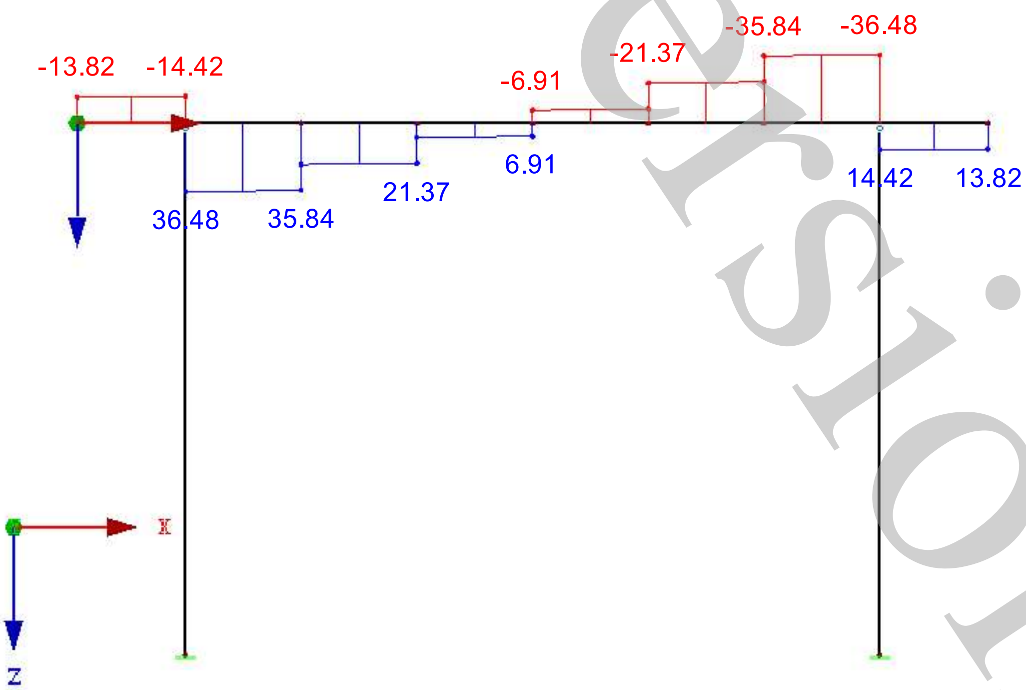
Max V-z: 18.20, Min V-z: -18.20 [kN]

1.467 m

INTERNAL FORCES V_z

CO 1: 1.35*LC1 + 1.5*LC2
Internal Forces V-z

Against Y-direction



Max V-z: 36.48, Min V-z: -36.48 [kN]

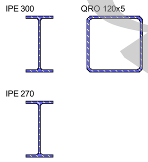
1.467 m

Project: Master Project   Model: POS. V.31.4 - Steel Canopy

1.2 MATERIALS

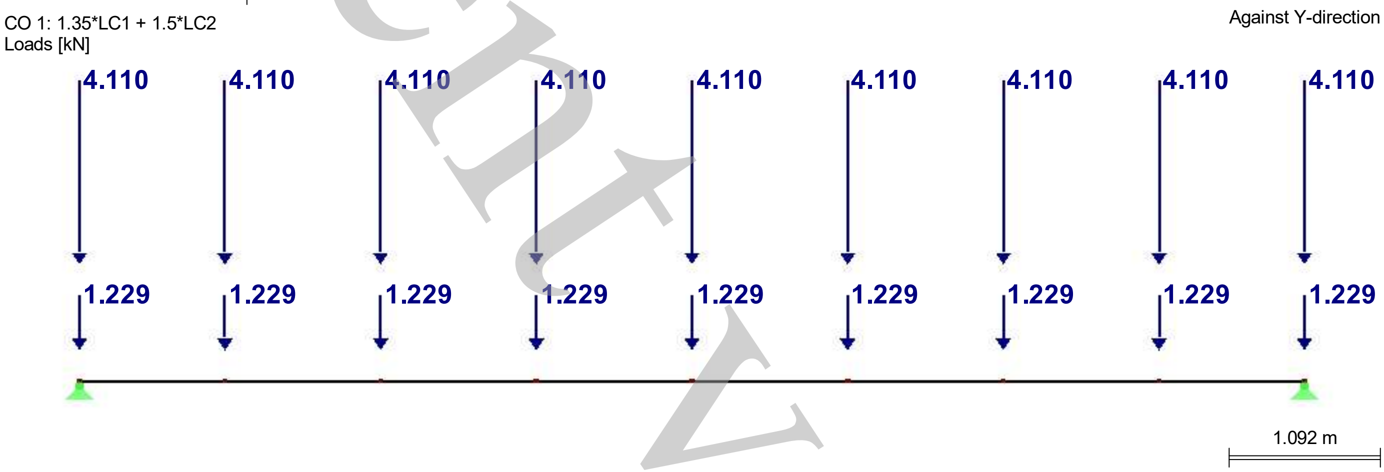
Matl. No.	Modulus E [kN/cm²]	Modulus G [kN/cm²]	Spec. Weight γ [kN/m³]	Coeff. of Th. Exp. α [1/°C]	Partial Factor γ_M [-]	Material Model
1	Steel S 235 | EN 1993-1-1:2005-05					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic

1.3 CROSS-SECTIONS



Section No.	Matl. No.	J [cm⁴]	A [cm²]	I_y [cm⁴]	A_y [cm²]	I_z [cm⁴]	A_z [cm²]	Principal Axes α [°]	Rotation α' [°]	Overall Dimensions [mm] Width b	Height h
1	IPE 300 | Euronorm 19-57										
	1		53.80	8360.00			19.82	0.00	0.00	150.0	300.0
2	QRO 120x5 | DIN 59410:1974										
	1		22.60	495.00			9.68	0.00	0.00	120.0	120.0
3	IPE 270 | Euronorm 19-57										
	1		45.90	5790.00			16.57	0.00	0.00	135.0	270.0

MODEL



2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | DIN Action Category	Active	Self-Weight - Factor in Direction X	Y	Z
LC1	Permanent	Permanent	☒	0.000		1.000
LC2	Snow	Snow (H ≤ 1000 m a.s.l.)	☐			

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters		
LC1	Permanent	Method of analysis	: ⦿	Geometrically linear analysis
		Activate stiffness factors of:	: ☒	Cross-sections (factor for J, I_y, I_z, A, A_y, A_z)
			: ☒	Members (factor for GJ, EI_y, EI_z, EA, GA_y, GA_z)
LC2	Snow	Method of analysis	: ⦿	Geometrically linear analysis
		Activate stiffness factors of:	: ☒	Cross-sections (factor for J, I_y, I_z, A, A_y, A_z)
			: ☒	Members (factor for GJ, EI_y, EI_z, EA, GA_y, GA_z)

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1		1.35*LC1 + 1.5*LC2	1	1.35	LC1	Permanent
			2	1.50	LC2	Snow
CO2		LC1 + LC2	1	1.00	LC1	Permanent
			2	1.00	LC2	Snow

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC1: Permanent

LC1
Permanent

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	Force [kN] P_Z / P_W	Moment M_Y / M_V [kNm]
1	1-4,7-11	0 | Global XYZ	0.000	0.910	0.000

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC2: Snow

LC2
Snow

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	Force [kN] P_Z / P_W	Moment M_Y / M_V [kNm]
1	1-4,7-11	0 | Global XYZ	0.000	2.740	0.000

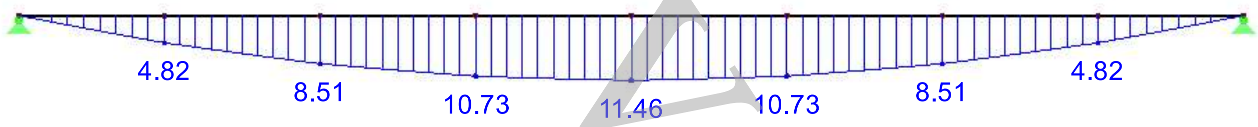
4.3 CROSS-SECTIONS - INTERNAL FORCES

Member No.	LC/CO	Node No.	Location x [m]	Forces [kN] N	Forces [kN] V_z	Moments M_y [kNm]
	Section No. 3: IPE 270 | Euronorm 19-57					
1	CO2	MAX N	0.000	⊳ 0.12	14.37	0.00
1	LC1	MIN N	0.000	⊳ 0.00	4.78	0.00
1	CO1	MAX V_z	0.000	0.00	⊳ 20.83	0.00
3	CO1	MIN V_z	1.050	0.00	⊳ -20.83	0.00
7	CO1	MAX M_y	1.123	0.00	2.67	⊳ 51.36
1	LC2	MIN M_y	0.000	0.00	9.59	⊳ 0.00

INTERNAL FORCES M_y

LC 1: Permanent
Internal Forces M-y

Against Y-direction



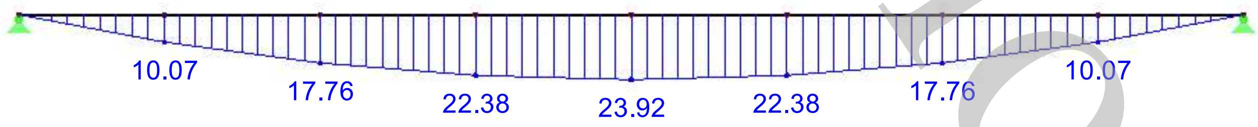
1.092 m

Max M-y: 11.46, Min M-y: 0.00 [kNm]

INTERNAL FORCES M_y

LC 2: Snow
Internal Forces M-y

Against Y-direction



1.092 m

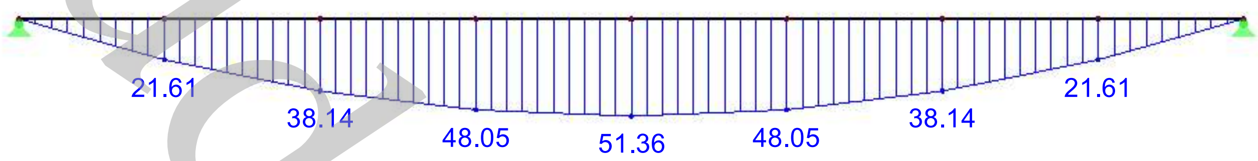
Max M-y: 23.92, Min M-y: 0.00 [kNm]

Project: Master Project Model: POS. V.31.4 - Steel Canopy

INTERNAL FORCES M_y

CO 1: 1.35*LC1 + 1.5*LC2
Internal Forces M-y

Against Y-direction



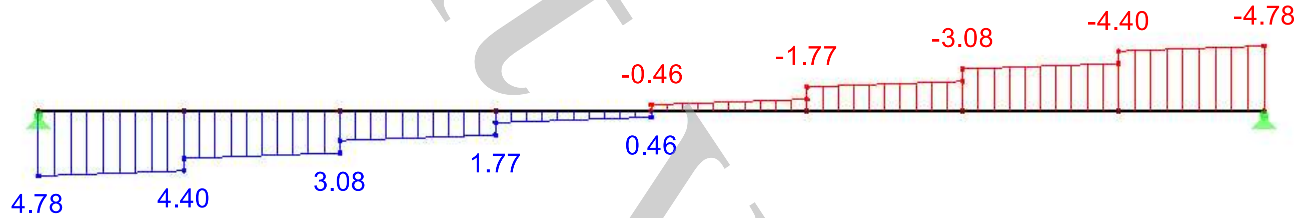
Max M-y: 51.36, Min M-y: 0.00 [kNm]

1.092 m

INTERNAL FORCES V_z

LC 1: Permanent
Internal Forces V-z

Against Y-direction



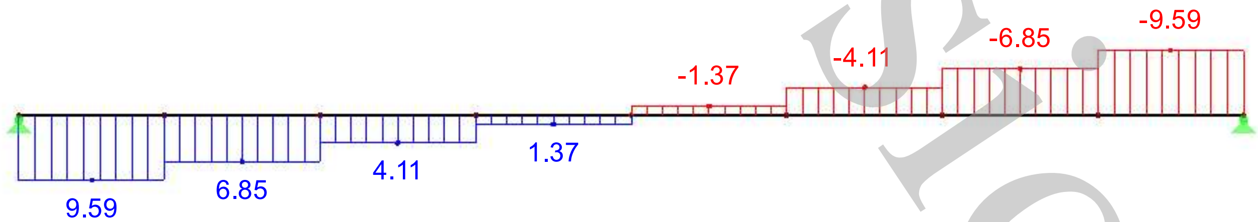
Max V-z: 4.78, Min V-z: -4.78 [kN]

1.092 m

INTERNAL FORCES V_z

LC 2: Snow
Internal Forces V-z

Against Y-direction



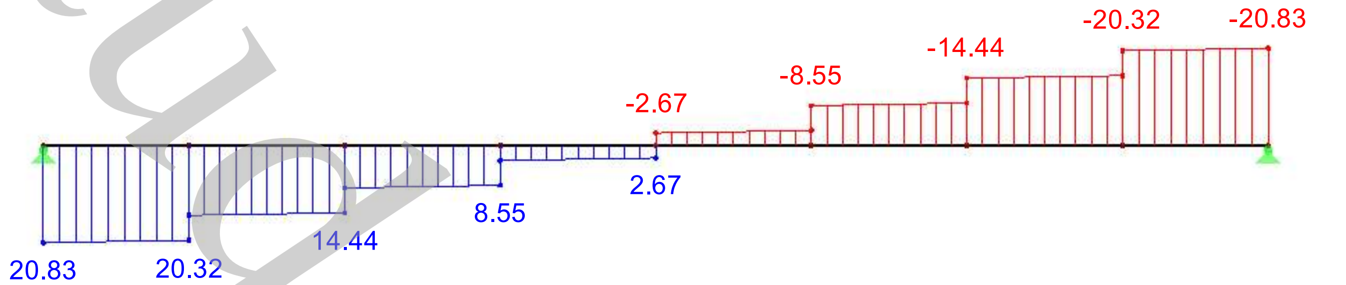
Max V-z: 9.59, Min V-z: -9.59 [kN]

1.092 m

INTERNAL FORCES V_z

CO 1: 1.35*LC1 + 1.5*LC2
Internal Forces V-z

Against Y-direction



Max V-z: 20.83, Min V-z: -20.83 [kN]

REFERENCES

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[3] A. Goris and U. P. Schmitz, Bemessungstafeln nach Eurocode 2: Normalbeton, hochfester Beton, Leichtbeton., Köln: Bundesanzeiger-Verag, 2014.

[4] A. Steinle, H. Bachmann und M. Tillmann, Bauen mit Betonfertigteilen im Hochbau, Berlin: Ernst & Sohn, 2015.

[5] F. D. B. e.V., "Merkblatt Nr. 6 - Toleranzen und Passungsberechnungen für Betonfertigteile," Bonn, 2015.

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[8] C. Enderle, Lecture notes "Stahlbetonfertigteilbau", Karlsruhe: Hochschule Karlsruhe, 2017.

[9] R. Avak und K. Meiss, Spannbetonbau. Theorie, Praxis, Berechnungsbeispiele nach Eurocode 2., Berlin: Bauwerk - Beuth Verlag, 2015.

STANDARDS AND DATA SHEETS

STANDARDS

DIN EN 1990:2010-12 *Eurocode: Basis of structural design and* DIN EN 1990/NA:2010-12 *National Annex – Nationally determined paramteers for DIN EN 1990*

DIN EN 1991-1-1:2010-12 *Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings; and* DIN EN 1991-1-1/NA:2010-12 *National Annex – Nationally determined paramteers for DIN EN 1991-1-1*

DIN EN 1991-1-3:2010-12 *Eurocode 1: Actions on structures – Part 1-3: General actions – Snow loads; and* DIN EN 1991-1-3/NA:2010-12 *National Annex – Nationally determined paramteers for DIN EN 1991 - 1- 3*

DIN EN 1991-1-4:2010-12 *Eurocode 1: Actions on structures – Part 1-4: General actions – Wind actions; and* DIN EN 1991-1-4/NA:2010-12 *National Annex – Nationally determined paramteers for DIN EN 1991 - 1- 4*

DIN EN 1992-1-1:2011-01 *Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings; and* DIN EN 1992-1-1/NA:2011-01 *National Annex – Nationally determined paramteers for DIN EN 1992 - 1- 1*

DATA SHEETS

- HOESCH Sizing Tabele for Trapezoidal Sheets
- HOESCH Cross-Section Properties Table for Trapezoidal Sheets
- Technical Product Information for HALFEN Transport Anchor System
- Calenberg Ingenieure Sizing Tables for Elastomeric Bearings
- Sizing Table for SCHÖCK Tronsole Type T
- Building Control Certification for Prestressing Steel Z-12.3-107