GOLDBECK

Structural Calculation

Project name: **MSCP Neratovice**

Project number: **PR0460**

Client:
Město Neratovice
Kojetická 1028
277 11 Neratovice
Czech Republic

Construction site address:
Na Výsluní 1368
parc. č, 92/15, 92/16
k.ú. Neratovice
Czech Republic

Date: Leipzig, 2025-09-04

Author: Primary Engineer

[redacted]

[redacted]

GOLDBECK Deutschland GmbH
Debyestraße 10
04329 Leipzig
GERMANY

[redacted]

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Date: 2025-09-04

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Explanatory report

The present construction project is a 3-bay parking garage in split level / full storey construction. For each parking garage bay 2 (upgradable to 3) ceiling levels (steel levels) are planned. The top ramp is roofed. Staircases are planned on gridlines 1 and 28.

The parking garage is approx. 69 m long and approx. 49 m wide. The storey height is 2,8 m, while the column grid is 2500 mm. In the course of the structural design, all optional extensions and superstructures (e.g. curtained facade) are taken into account in the load assumptions.

The entrance and exit of the parking garage is located in gridline V / 2-5 (longitudinal gridline). The individual parking levels are accessed via access and exit ramps, which each bridge half the floor height. The load-bearing structure of the parking garage consists of 10 cm thick prefabricated reinforced concrete slabs, which span over 2500 mm as single-span beams and transfer their load to the ceiling beams.

The slabs are joined above the beam. The beams are designed as steel or steel composite beams and transfer their loads to supports or interception beams. In the case of steel composite beams, the bond is created by means of head bolt dowels welded onto the beams. These are arranged in recesses at the edge of the precast concrete slabs. By casting the joint and the recesses with a swellable and low-shrinkage casting concrete, a force-fit connection is created between the head bolt dowels and the precast concrete element. During assembly, the beams are supported in order to achieve a self-weight bond in the final state.

Steel columns with articulated beam connections and articulated base points are used as supports. The parking garage is stabilized in the longitudinal and transverse directions by the diaphragms, as well as with compression-slack vertical bracing in the longitudinal and ramp gridlines. All staircases are constructed using prefabricated elements and are completely decoupled from the parking garage. No stabilization loads are transferred from the multi-storey parking garage to the reinforced concrete structures or vice versa. All reinforced concrete structures and the foundation are not part of this static calculation.

Exposure classes, minimum concrete quality, concrete cover and minimum axial distances

	Description			DIN EN 1992-1-1							
	Details of the component			Exposure classes					minimum concrete quality	Concrete cover 1)	Allowance
	Type	Remark	used concrete quality	XC	XD	XF	XM	W		[mm]	[mm]
1	Ceiling & ramp slabs	Bottom of slab	C40/50	XC3		XF1		WF	C25/30	25 3),6)	10 2)
		Top of slab		XC4	XD3	XF4		WA	C30/37	45 3),6)	10 2)
2	Columns										
	for chambered concrete columns see chapter 6										
3	Retaining wall plates	earth side	C40/50	XC2	XD2	XF1		WA	C30/37 4)	50	10 2)
		air side		XC4		XF1		WA	C25/30	30 3),6)	10 2)
	for façade wall panels see chapter 8										
4	Precast concrete foundations	top/side	C40/50	XC3	XD3			WA/WF	C30/37 4)	50	10 2)
		bottom		XC3	XD3			WA/WF		30 5)	10 2)
	In-situ concrete foundations		C30/37	XC3	XD3	XF2		WA	C30/37 4)	55	15

[1] Laying dimension at least bar diameter + allowance

[2] due to quality control (prefabricated parts), the allowance is reduced by 5 mm.

[3] Due to the two strength classes higher concrete quality used, the concrete cover is reduced by 5mm.

[4] For air-entrained concrete, one strength class lower (originally C35/45) due to simultaneous requirement from exposure class XF.

[5] See floor measures Concrete design

[6] for XS1 near the coast, concrete cover: 45 mm

Rebar steel: B500 (A) or (B)

Calculation principles

The structural analysis provides verification of the load-bearing capacity of all relevant components for the preparation of the technical documentation (TE).

Goldbeck Deutschland GmbH

- Views, floor plans and sections

Applicable Standards (valid versions):

- EN 1990 + NA – Basis of structural design
- EN 1991 + NA – Actions on structures
- EN 1992 + NA – Design of concrete structures
- EN 1993 + NA – Design of steel structures
- EN 1994 + NA – Design of composite steel and concrete structures
- EN 1995 + NA – Design of timber structures
- EN 1996 + NA – Design of masonry structures
- EN 1997 + NA – Geotechnical design
- EN 1998 + NA – Design of structures for earthquake resistance
- EN 1999 + NA – Design of aluminium structures
-

As well as all other currently valid standards, guidelines, and regulations.

The calculation methods used comply with the above-listed standards.

Software Used for Structural Analysis:

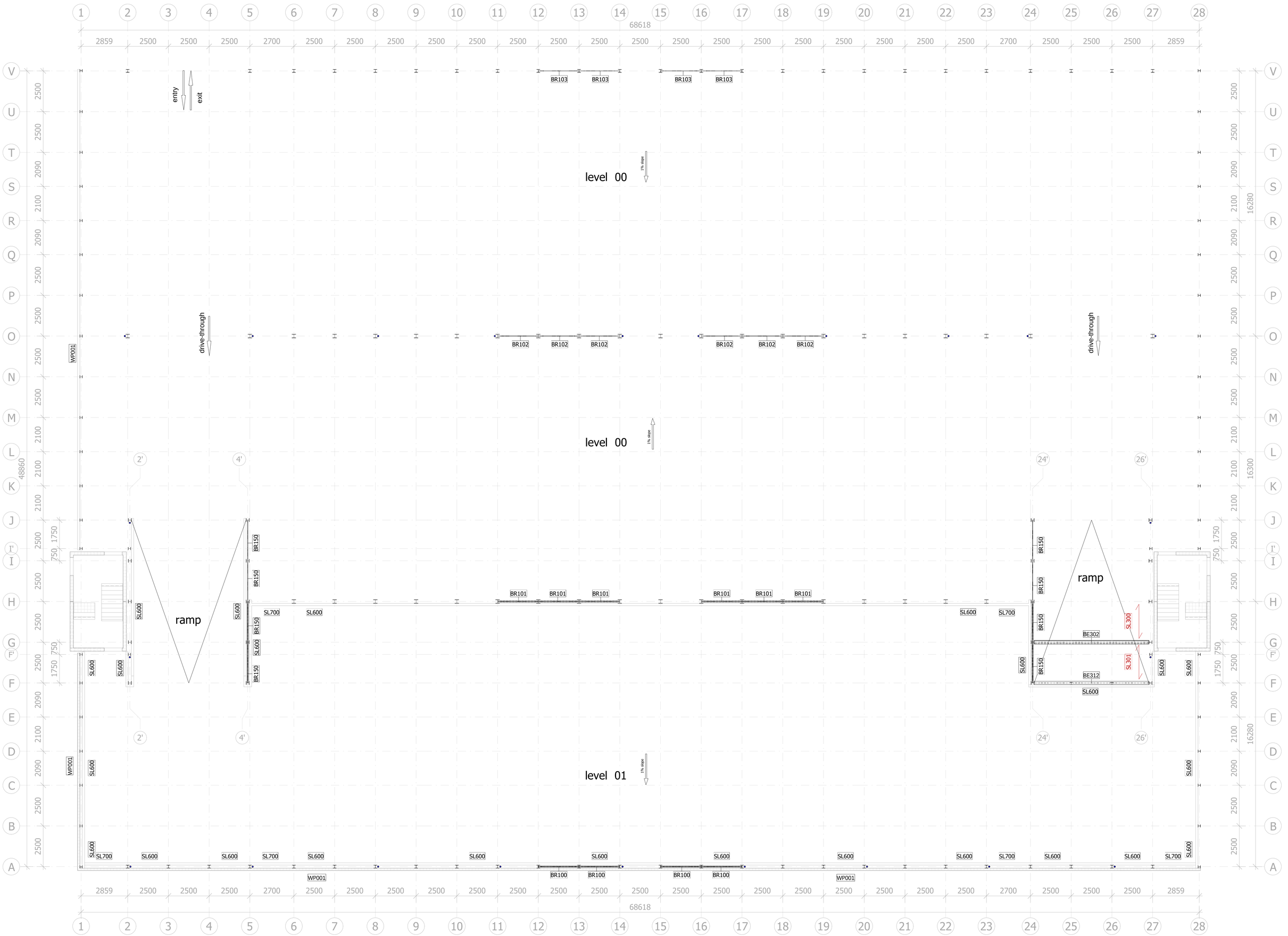
- R-Stab 8.23
- R-FEM 5.23
- In-house Excel 365 calculation tools

Construction materials:

-	Profiled steel	S235 S355
-	For in-situ concrete	see the following pages
-	For precast concrete components	see the following pages
-	For reinforced concrete	see the following pages

Any changes to the structure, such as slots, openings, or recesses, may only be made after consultation with the structural engineer.

Further details are provided in the structural analysis and the position drawings.



legend of position numbers	legend of symbols
BE = Beam (Deckenträger) BR = Bracing (Aussteifung) CO = Column (Stütze) RO = Roof (Dach) SL = Slab (Platte) SP = Special design (Sonderkonstruktion)	pinned column stub column connection impact mat transfer column tension band = coupling element + continuous rebar loop 2 Ø 10

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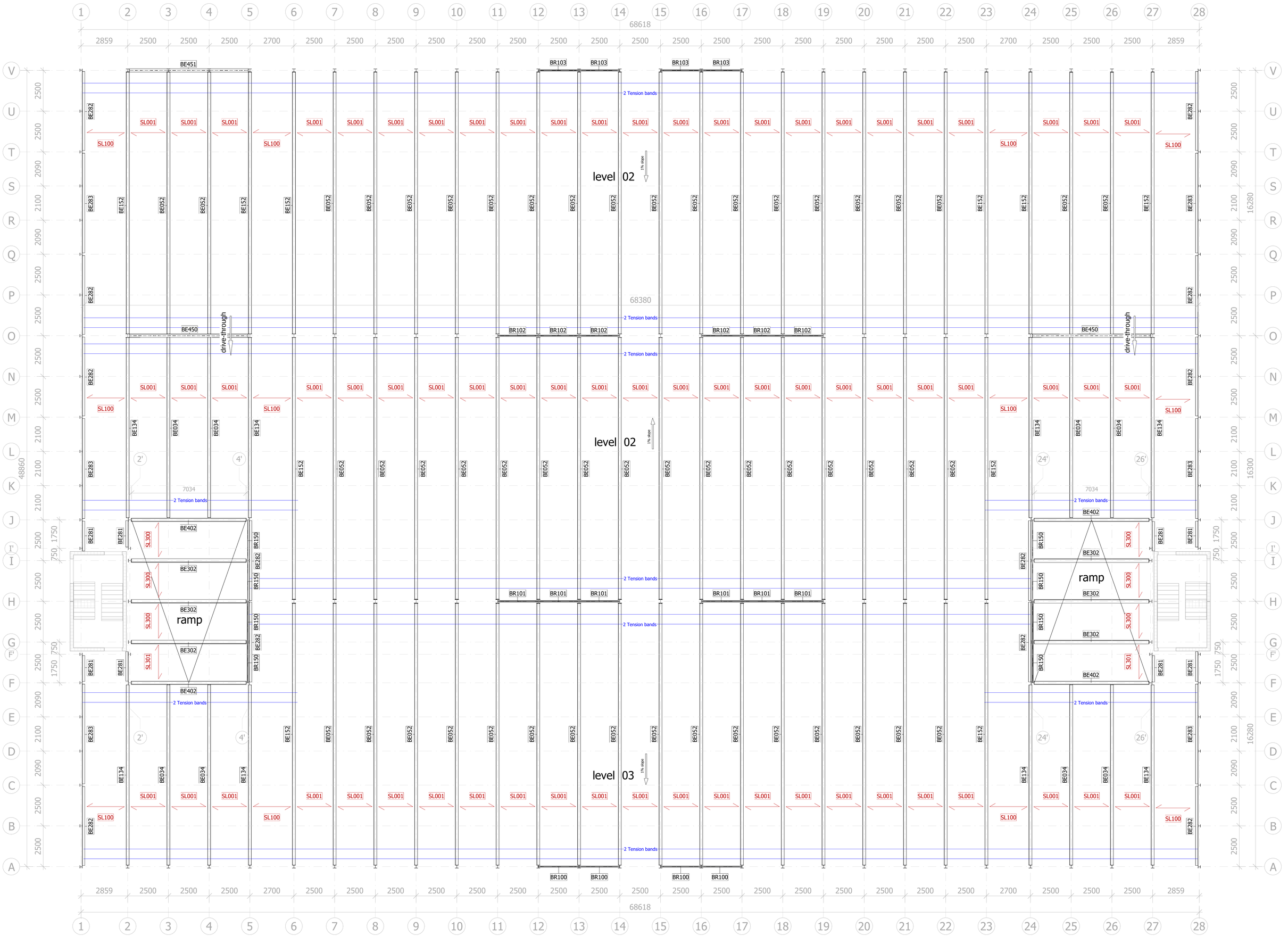
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gez.:	04.09.2025	[redacted]	
Statiker:	04.09.2025	[redacted]	
Maßstab:	o.M.		

GOLDBECK
Goldbeckstraße 7, 69493 Hirschberg a. d. Bergstraße Telefon: +49 (6201) 8777 - 0

Projekt	Auftrag Nr.
Parkovací d ům Neratovice	PR0460

Planinhalt	Phase	Planer	Art
structural position plan 1. construction phase level 00 and 01	5	TW	PO
	Ebene	laufende Nr.	Index
	xx	2005	0

Format: DIN A1+, 841 x 690 mm



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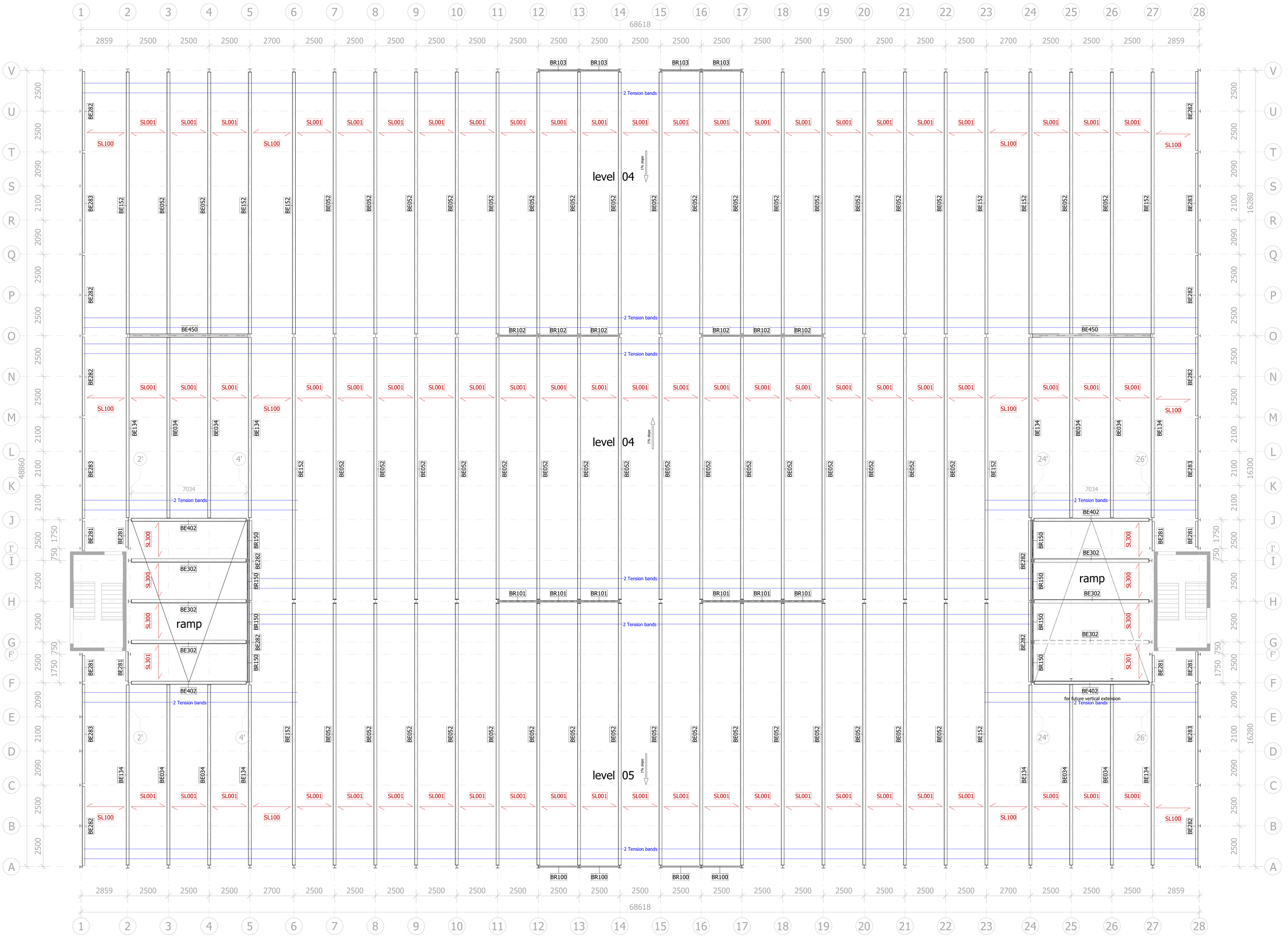
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Projekt	Auftrag Nr.
Parkovací d ům Neratovice	PR0460

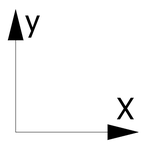
Planinhalt	Phase	Planer	Art
structural position plan 1. construction phase level 02 and 03	5	TW	PO
	Ebene	laufende Nr.	Index
	xx	2004	0

Format: DIN A1+, 841 x 690 mm

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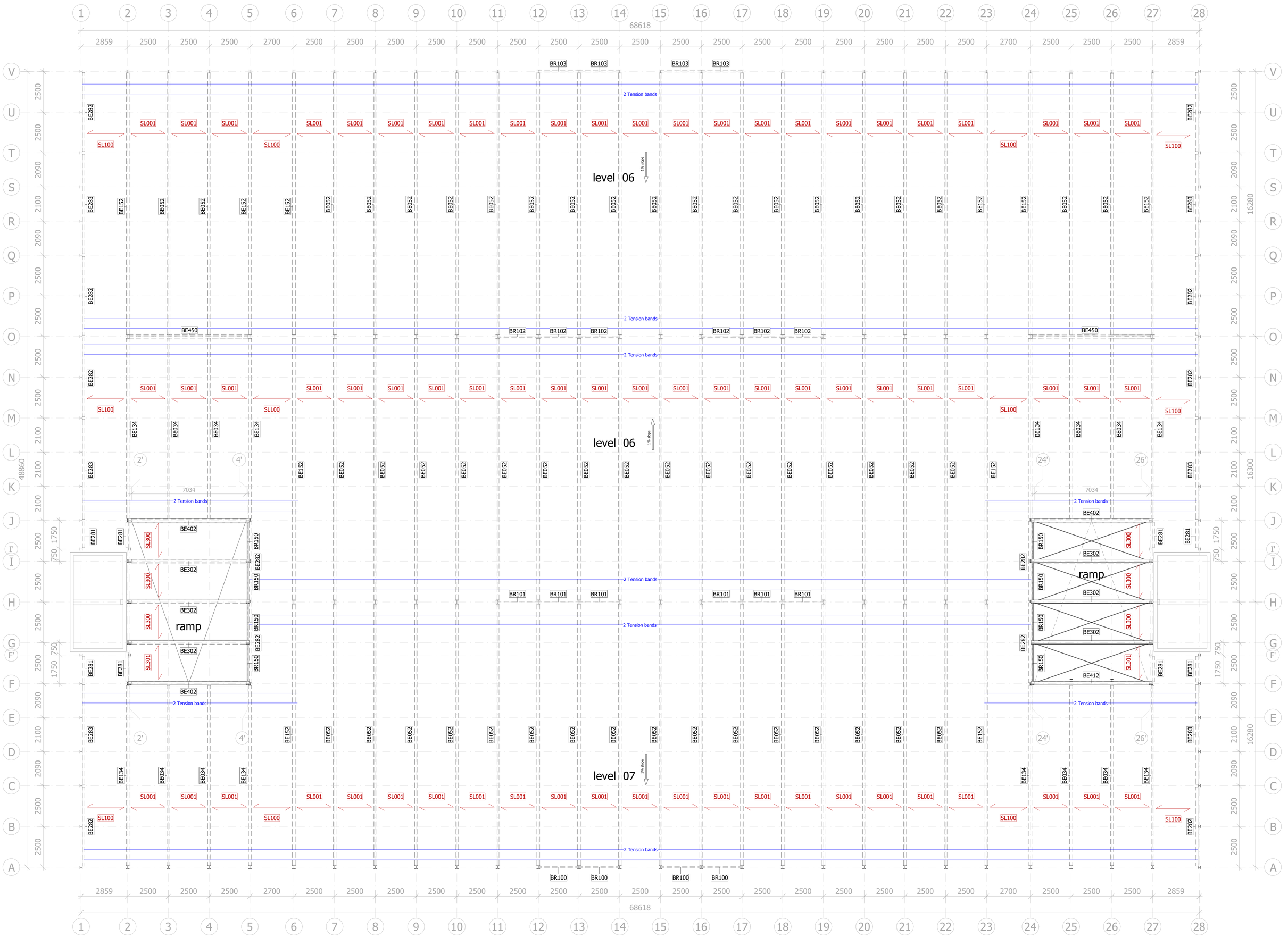
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Parkovací d ům Neratovice	PR0460

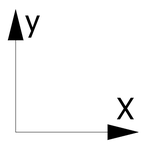
Planinhalt	Phase	Planer	Art
structural position plan 1. construction phase level 04 and 05	5	TW	PO
	Ebene	laufende Nr.	Index
	xx	2003	0

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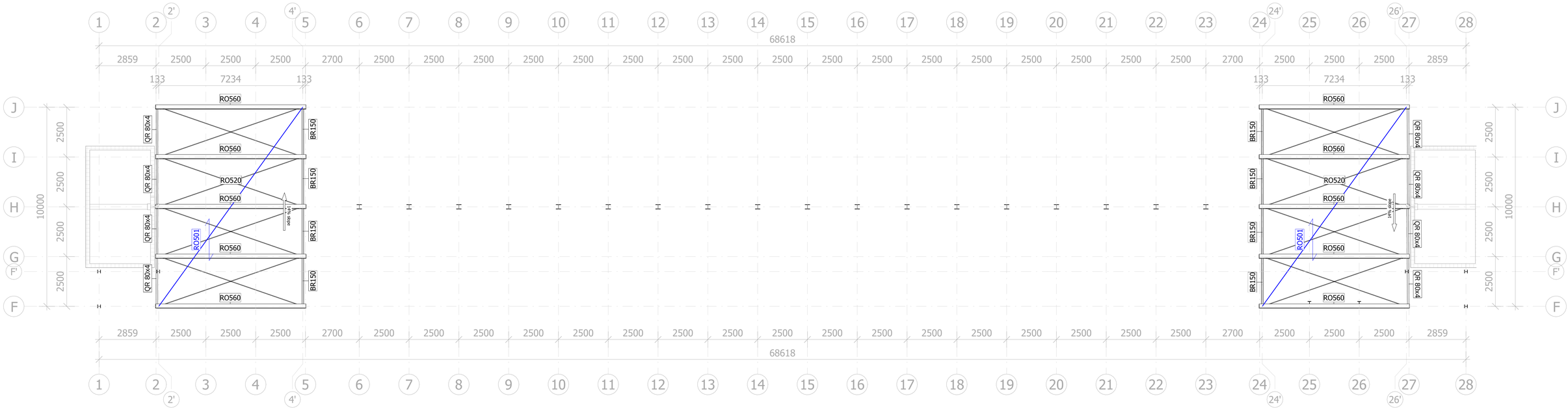
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Projekt	Parkovací d ům Neratovice
Auftrag Nr.	PR0460
Planinhalt	structural position plan extension level 06 and 07

Phase	Planer	Art
5	TW	PO
Ebene	laufende Nr.	Index
xx	2002	0

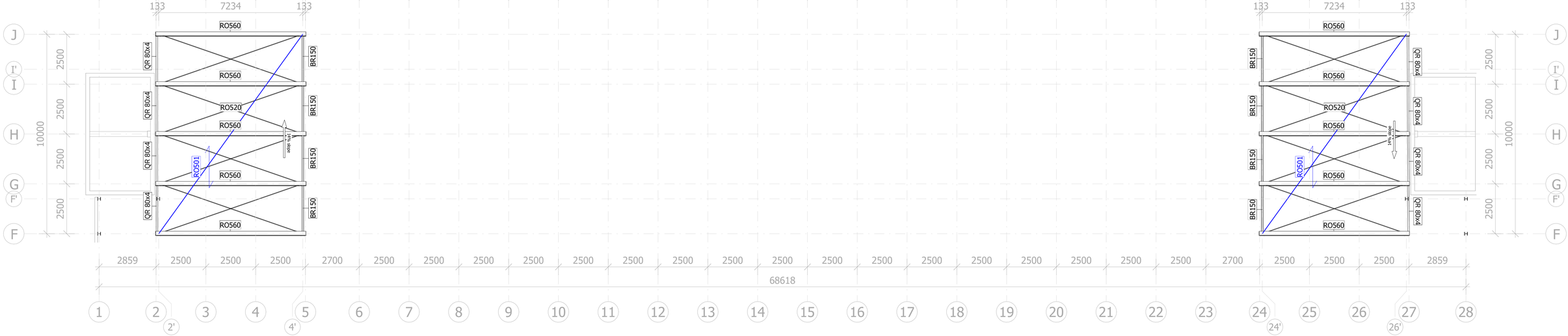
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ramp roof **1.** construction phase

scale 1 : 100



ramp roof extension

scale 1 : 100

Legend of structural position numbers
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BR = Bracing (Aussteifung)
CO = Column (Stütze)
RO = Roof (Dach)
SL = Slab (Platte)
SP = Special design (Sonderkonstruktion)

Legend of symbols

- pinned column
- stub column connection impact mat
- transfer column
- tension band = coupling element + continuous rebar loop 2 Ø 10

Index	Datum	Name	Änderung
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Projekt	Auftrag Nr.
Parkovací d ům Neratovice	**PR0460**

Planinhalt	Phase	Planer	Art
structural position plan	5	TW	PO
ramp roofs 1. cp ramp roofs extension	Ebene	laufende Nr.	Index
	xx	2001	0

Format: DIN A1, 841 x 594 mm

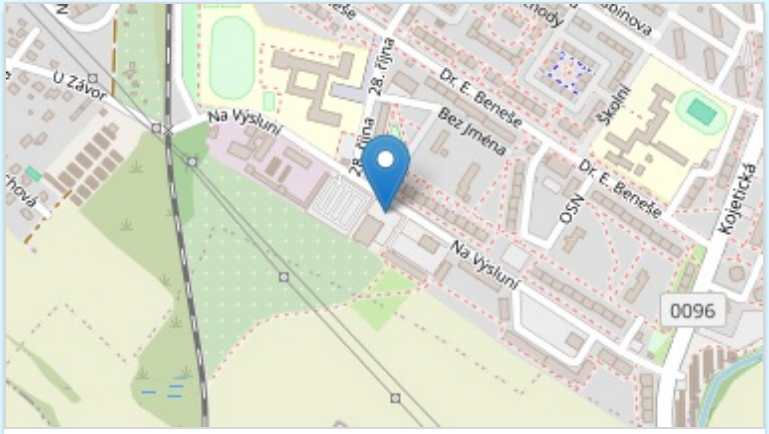
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Chapter 1: Loads

Load assumption on structures

Self-weight:	floor weight of 10.3 cm reinforced concrete	g_1 = 2.58 kN/m²
Installation:	Installation load (e.g. suspended ceiling)	g_2 = 0.15 kN/m²
	E-mobility: cable ducts go along main axes	g_3 = 0.6 kN/m
	The load is to be applied per level (optional).	
Facade:	Expanded metal or similar (e.g. panel facade)	g_4 = 1.00 kN/m²
	The load specifications also include facade elements and their substructure. The loads from the facade are considered as point loads up to the UE anchor plate.	
Fire spread walls:	weight of 14.0 cm reinforced concrete	g_6 = 3.50 kN/m²
	Fire spread walls are located along axis A and axis 1. They are directly suspended from the structural columns.	
Pay loads:	private vehicles up to 3.0 to at the floor area according to CSN EN 1991-1-1, 6.3.3 + CSN EN 1991-1-1/NA:	
	load for parking areas and ramps	q_p = 2,5 kN/m²
	or two single loads with each 10 kN on an area of 20 cm x 20 cm with 1.8 m in unfavourable position.	
Snow loads:	according to CSN EN 1991-1-3 → separate load determination	
Wind loads:	according to CSN EN 1991-1-4 → separate load determination	
Impact loads:	according to CSN EN 1991-1-7:	
	in driving direction:	F_k = 50 kN
	perpendicular to the driving direction:	F_k = 25 kN
	The proofs due to vehicle impact are carried out exclusively for traffic within the parking garage. Any impact from outside must always be prevented by on-site measures (independent of the parking garage).	
Earthquake:	according to CSN EN 1998:	
	Neratovice in the Czech Republic does not belong to any earthquake zone.	
Fire protection:	fire protection requirements R15 (steel construction of the car park)	

Geo-Zone Tool: Snow Load, Wind Speed, and Seismic Zone Maps



ONLINE MAP

Location Detail

[redacted]

Na Výsluní
27711
Neratovice
Czechia

LATITUDE	LONGITUDE	ALTITUDE [M]
50.253°	**14.512°**	**170 m**

Report Results **(3)**

Snow

STANDARD
EN 1991-1-3

COUNTRY | TABLE
Czech Republic | ČSN EN 1991-1-3

Characteristic Value of Snow Load

s_k = 0.70 kN/m²



SHOW ONLINE MAP

I II III IV V VI VII VIII

I Snow Load Zone

Wind

STANDARD
EN 1991-1-4

COUNTRY | TABLE
Czech Republic | ČSN EN 1991-1-4

Fundamental Basic Wind Velocity

$v_{b,0}$ = 22.5 m/s

Basic Velocity Pressure

q_b = 0.32 kN/m^2



I Windzone

Seismic

STANDARD
EN 1998-1

COUNTRY | TABLE
Czech Republic | ČSN EN 1998-1

Reference Value of Peak Ground Acceleration

a_{gR} = ?



SHOW ONLINE MAP

0.03·g | 0.04·g | 0.05·g | 0.06·g | 0.07·g | N/A

N/A Seismic Zone

No defined seismic load
Mělník

Disclaimer

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Sources

ČSN EN 1991-1-3 NA ed. A:2017-01 | http://www.gadm.org/download | http://srtm.csi.cgiar.org
ČSN EN 1991-1-4 NA ed. A:2013-07 | http://www.gadm.org/download | http://srtm.csi.cgiar.org
ČSN EN 1998-1 NA:2016-09 ed. A | http://www.openstreetmap.org | https://osm-boundaries.com/

Snow loads according to CSN EN 1991-1-3 NA

SNOW LOAD ON FLAT ROOFS

Elevation above sea level: **170 m**

snow load zone: **1**

Building length: **69 m**
Building width: **49 m**

} μ_1 = 0,80

Snow loads:

- on the ground: s_k = 1,00 * { 0,19 + 0,91 * [(170 + 140) / 760] ² } ≥ 0,70 kN/m²

s_k = 0,70 kN/m²

- on the main roof: s_1 = μ_1 * s_k = 0,80 * 0,70 kN/m²

s_1 = **0,56 kN/m²**

Live load including the effects of snow, technical installations, wind pressure, and vehicular traffic

In principle, no traffic flow is possible on the top parking deck as long as the design snow load is present and has not been removed. However, vehicles may already be parked during snowfall.
Therefore, the following section investigates the governing load combination.

To assess the influence of traffic load components, seven load case combinations are considered:

1. Leading action: snow (without traffic)
2. Leading action: installations (without traffic)
3. Leading action: wind (without traffic)
4. Leading action: traffic over the full area (without snow)
5. Leading action: snow (with traffic on parking areas only)
6. Leading action: traffic on parking areas only (with snow)
7. Exceptional snowfall (with traffic on parking areas only)

Snow load on the main roof:	s_1 =	1,40 kN/m²
Exeptional snow load: ($s_{Ad} = C_{esl} * s_k$):	s_{Ad} =	0,00 kN/m²
Installation load:	q =	0,15 kN/m²
Wind pressure:	w_{dd} =	0,11 kN/m²
Traffic:	v =	2,50 kN/m²

Load Case Combination 1:	1,50 (s + 0,8 q + 0,6 w) = 2,38 kN/m²
Load Case Combination 2:	1,50 (q + 0,5 s + 0,6 w) = 1,37 kN/m²
Load Case Combination 3:	1,50 (w + 0,5 s + 0,8 q) = 1,40 kN/m²
Load Case Combination 4:	1,50 (v + 0,8 q + 0,6 w) = 4,03 kN/m²
Load Case Combination 5:	1,50 (s + 0,8 q + 0,6 w + 0,7 (10m/16m) v) = 4,02 kN/m²
Load Case Combination 6:	1,50 ((10m/16m) v + 0,8 q + 0,6 w + 0,5 s) = 3,68 kN/m²
Load Case Combination 7:	sAd + 0,7 (10m/16m) v + 0,5 q = 1,17 kN/m²

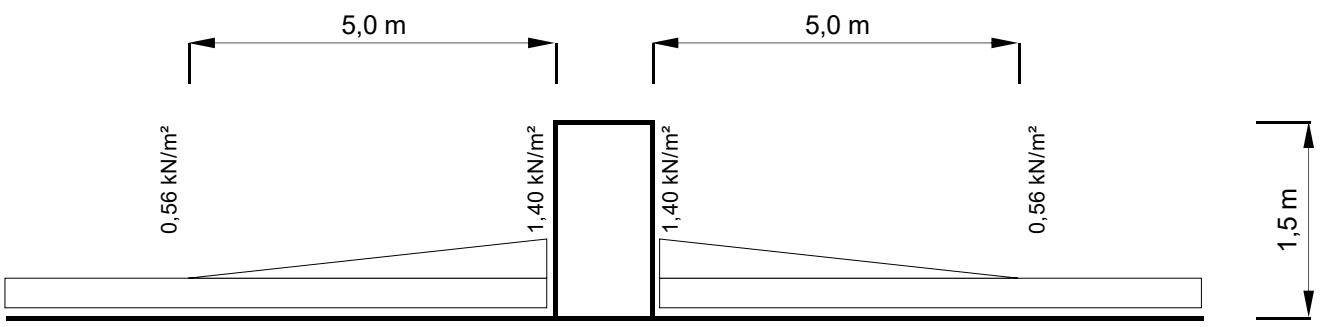
governing: q = **4,03 kN/m²**

Load Case Combination 4 is the governing combination. Since snow and traffic can only act together to a limited extent (see LC5 and LC6), the snow load is not considered in the design of the uppermost slab level. (This does not apply to canopy and ramp roofs.)

SNOW DRIFT AGAINST WALLS AND ROOF STRUCTURES

Remark: **parapet**

Snow load on the ground: s_k = 0,70 kN/m²
Snow load on the main roof: s_1 = 0,56 kN/m²



Permanent/variable load combination:

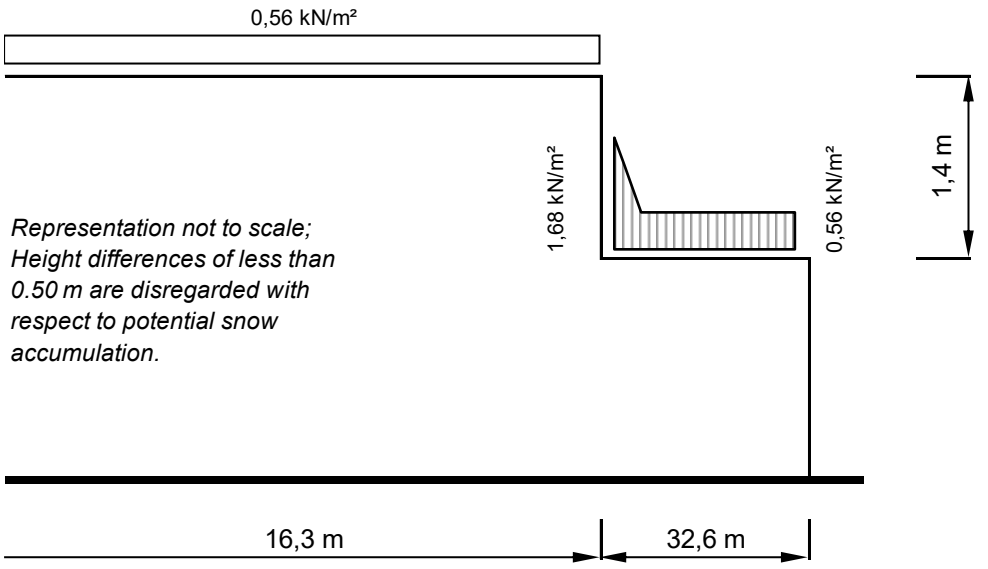
μ_2 = $\gamma * h / s_k$ with γ = 2,0 kN/m³ (snow)
μ_2 = 2,00 with $0,8 \leq \mu_2 \leq 2,0$

$s_{2, max}$ = $\mu 2 * sk$
$s_{2, max}$ = **1,40 kN/m²**

SNOW DRIFT ON A LOWER BUILDING SECTION

Remark: **Split-level offset and staircase**

Snow load on the ground:	s_k =	0,70 kN/m²
Snow load on the main roof:	s_1 =	0,56 kN/m²



Representation not to scale; Height differences of less than 0.50 m are disregarded with respect to potential snow accumulation.

The snow wedge is 5.0 m long.

μ_w =	(16,3 + 32,6) / (2 * 1,4) ≤ 2,0 * 1,4 / 0,70	with γ = 2,0 kN/m³ (snow)
μ_w =	4,00	with 0,8 ≤ μ_w

Permanent/variable load combination:

μ_w =	2,40	with 0,8 ≤ μ_w ≤ 2,4
$s_{max} = \mu_w * s_k$ =	**1,68 kN/m²**	
s_{min} =	**0,56 kN/m²**	

WIND LOADS ACCORDING TO CSN EN 1991-1-4 NA

for h ≤ b

External wind pressure w_e for flat roofs

1. Building Information

Building length:	(Eaves)	b =	69,00	m
Building width	(Gable)	d =	49,00	m
Reference elevation:		±0 =	170,00	m above sea level
Top of parapet:		ze =	+ 11,30	m
Top of roof surface:		h =	+ 11,31	m
Height hp:		hp =	+ 10,00	m
Design Auxiliary Value:		e =	min (b; 2h)	
Influence Area of Component:			> 10	m²

2. Calculation of the peak velocity pressure q_p

The building is located in wind load zone 1 (Inland).

Peak velocity pressure: **q_p = 0,57 kN/m²** according to height-dependent method

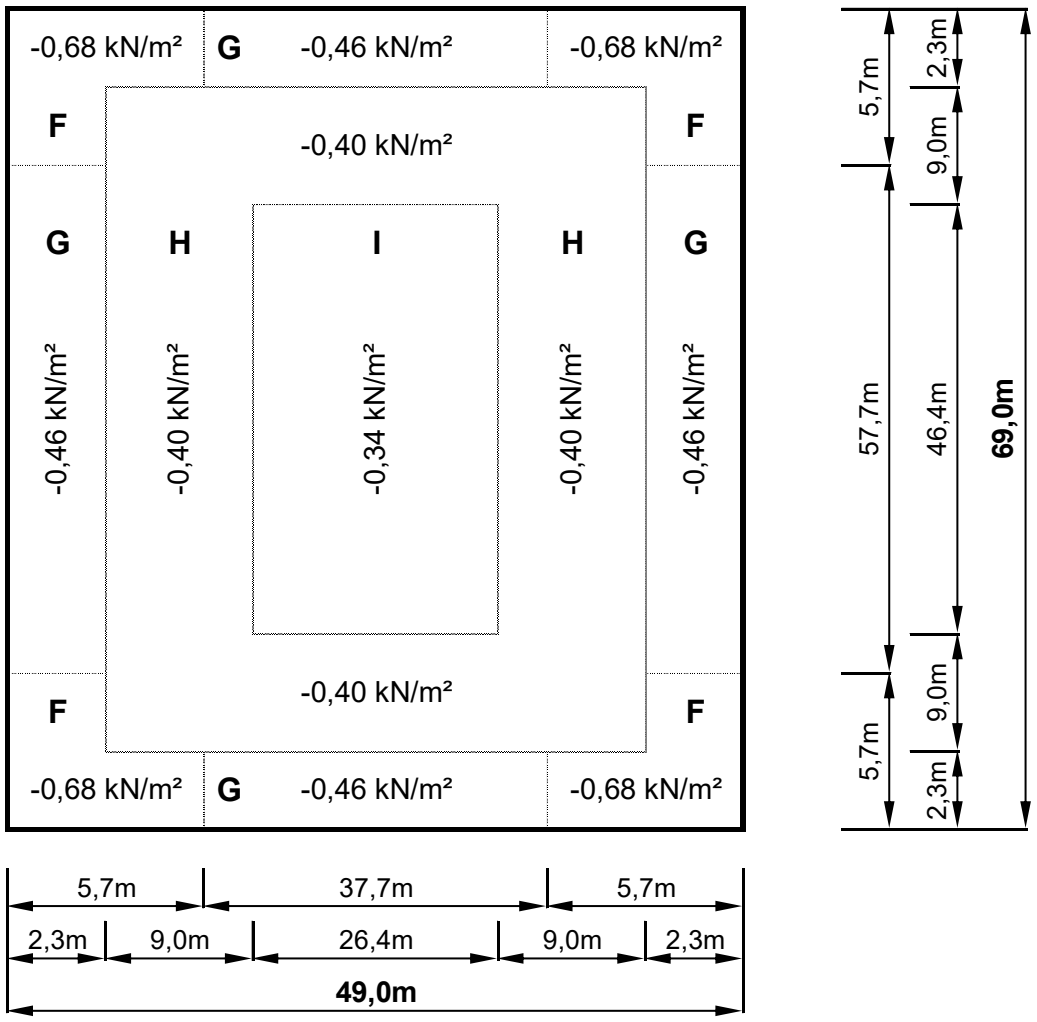
3. Determination of the negative external pressure coefficients c_{pe}

(depending on the building geometry and wind direction of approach)

Section F:	c_{pe} (F)	=	-1,20
Section G:	c_{pe} (G)	=	-0,80
Section H:	c_{pe} (H)	=	-0,70
Section I:	c_{pe} (I)	=	-0,60

4. Overview of maximum negative external pressure w_e for each roof section ($w_e = c_{pe} * q_p$)

(This diagram shows a superposition of all four incident wind directions.)



5. Consideration of the positive external pressure w_e for each roof section ($w_e = c_{pe} * q_p$)

Apart from the suction loads, the structural design must also take into account the pressure loads.

The following applies: c_{pe} = 0,20 → **we =** **0,11 kN/m²**

WIND LOADS ACCORDING TO CSN EN 1991-1-4 NA

for h ≤ b

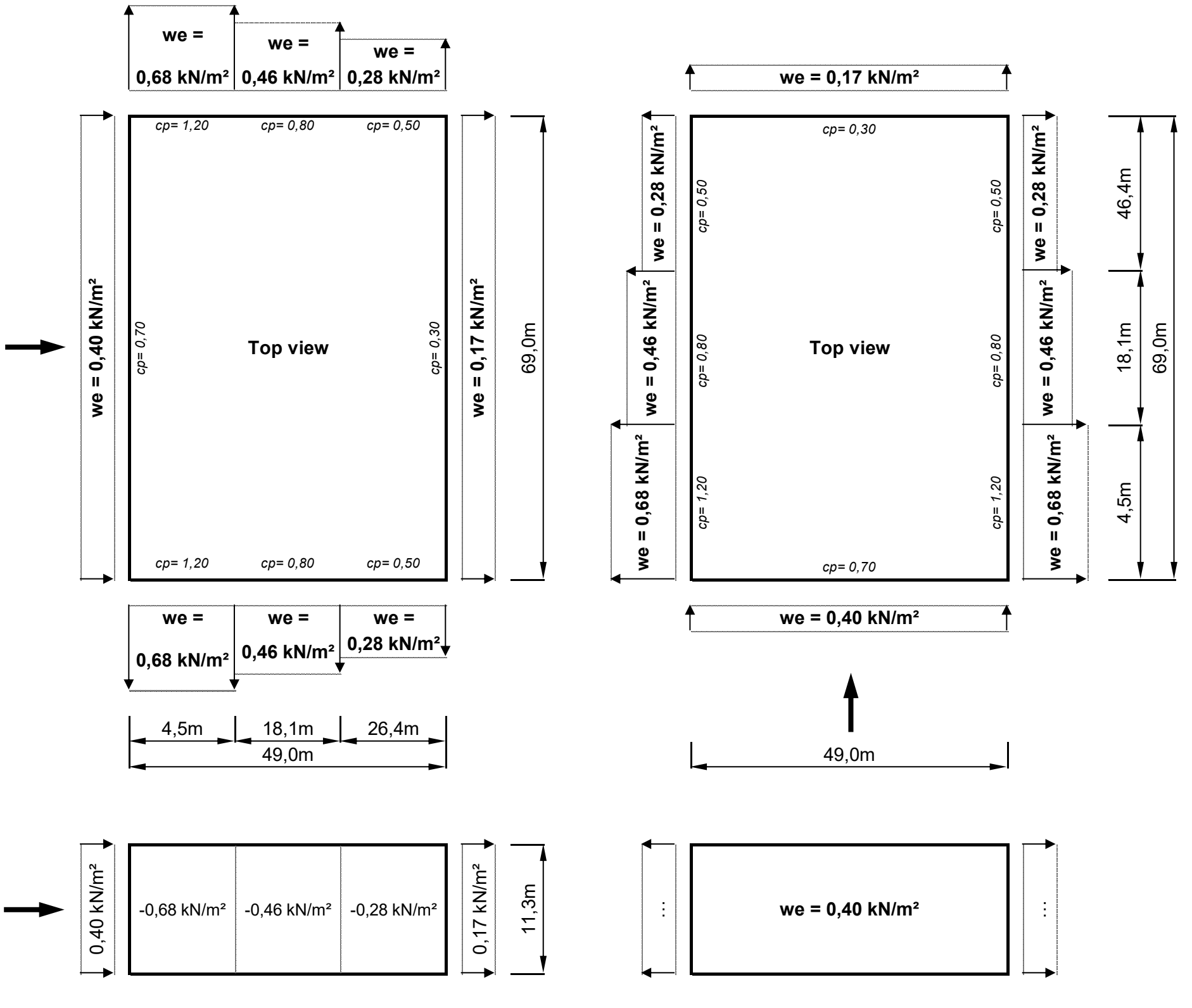
External pressure w_e for enclosed structures acting on the exterior walls

Building length:	(Eaves)	b =	69,00	m
Building width	(Gable)	d =	49,00	m
Reference elevation:		±0 =	170,00	m above sea level
Top of parapet:		ze =	+ 11,30	m
Top of Eaves:		h_T =	+ 11,21	m
Design Auxiliary Value:		e =	min (b; 2h)	
Influence Area of Component:			> 10	m²

The building is located in wind load zone 1 (Inland).

Peak velocity pressure: **q_p = 0,57 kN/m²** according to height-dependent method

The external pressure coefficients c_{pe} are determined based on the building geometry and the tributary area of the structural element.
The external pressure **$w_e = c_{pe} * q$** is specified depending on the wind direction, for each wall side and, if applicable, for each wall surface.



Wind loads on free-standing roofs

according to EN 1991-1-4, section 7.3 and table 7.6

Slope	$\alpha \approx 8°$
Maximum dynamic pressure	$q_p = 0{,}60$ kN/m²

Wind pressure (degree of obstruction max ϕ)

Area	A	B	C
$c_{p,net}$	1,20	2,40	1,60
w_k [kN/m^2]	0,72	1,44	0,96

Wind suction (degree of obstruction ϕ=1)

Area	A	B	C
$c_{p,net}$	-1,60	-2,60	-2,70
w_k [kN/m^2]	-0,96	-1,56	-1,62

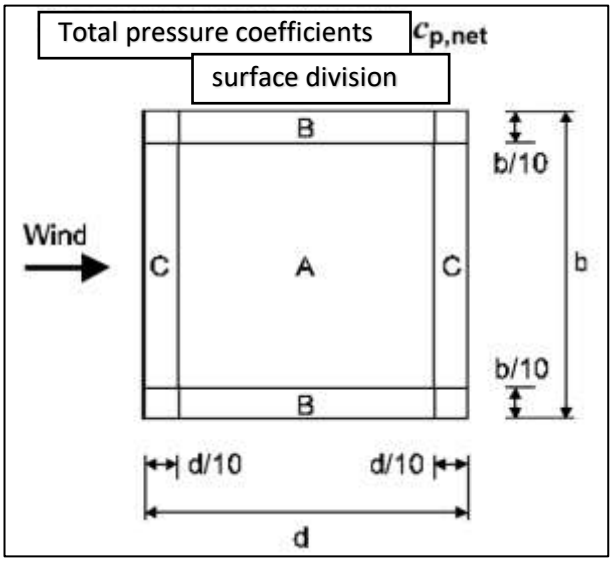


Table 7.6 – $c_{p,net}$ and c_f values for free-standing mono-pitch roofs

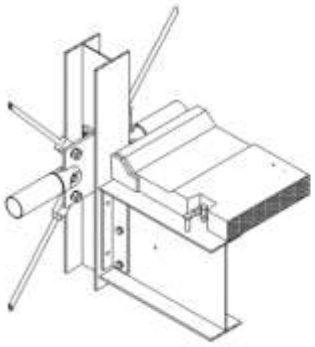
Chapter 2: Bracing

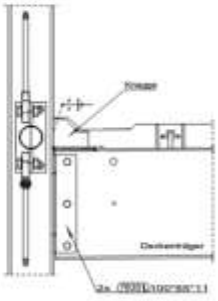
Pos. BR001 ff.

Conventional design:

The building is stiffened using shear-resistant ceiling panels in conjunction with non-compression vertical bracing (tension diagonals) in the longitudinal and transverse axes (e.g. ramp or gable)

The load is introduced into the bracing in the longitudinal axes of the car park using shear lugs which are already welded to the supports in the factory and are grouted with the joint.





In the transverse and ramp axes the forces are introduced from the ceiling panel via the edge angles with welded-on head bolts.

The car park is designed as a closed building.

All solid staircases are completely decoupled from the steel construction.

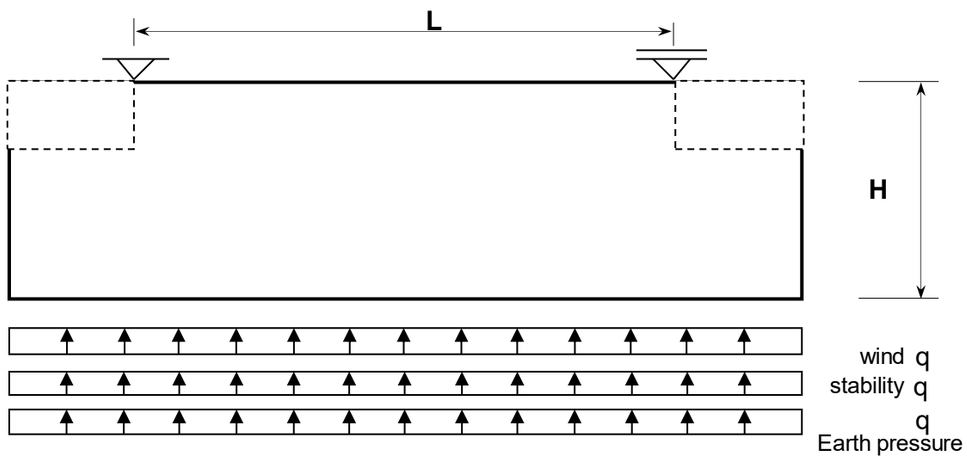
The calculation is carried separately for the longitudinal direction (X) and transverse direction (Y).

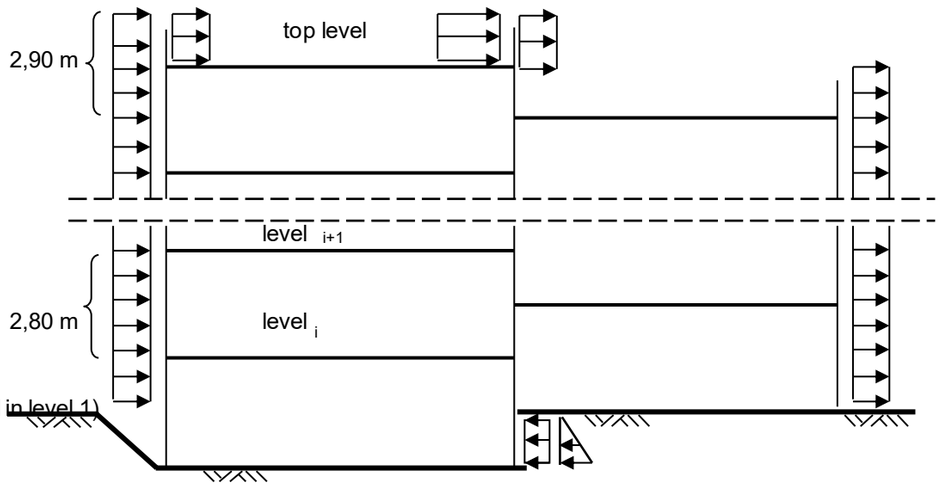
Standard dimensioning considers wind and misalignment loads, and possibly earth pressure.

Earthquake design: (not applicable!)

Pos BR001 Verification of the diaphragm

System:





System parameters:

Total length of parking garage	L_{ges} = 69,0 m	storey loads:	
Span of the diaphragm	L = 47,9 m	g_k =	**3,00 kN/m²**
Width of parking garage bay	H = 16,0 m	p_k =	**2,50 kN/m²**
Thickness of slab	h = 10,3 cm		
Concrete strength	C 40/50		no roof loads
			~~2,50 kN/m²~~
Column spacing =	2,50 m		~~0,92 kN/m²~~
Roof design	without roof		
Number of bays	multy-bay		

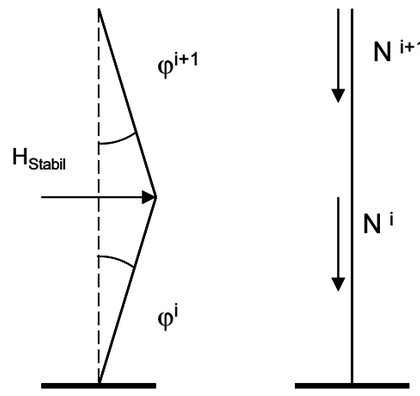
Level	Height [m]	
1	4,30 m	bottom
2	2,80 m	
3	2,80 m	
4	0,00 m	
5	0,00 m	
6	0,00 m	
7	0,00 m	
8	0,00 m	
9	0,00 m	
10	0,00 m	top

1

Stabilization loads:

Center axis columns		Outer axis columns	
φ_0= 1/200 =	0,005	φ_0= 1/200 =	0,005
α_h= 2/√h =	1	α_h= 2/√h =	0,964485644
α_m= √(0,5*(1+1/m))	0,73	α_m= √(0,5*(1+1/m))	0,72
max. storey height h =	2,15	max. storey height h =	4,30 m
number of columns in a row m=	18,16	number of columns in a row m=	26,6

αm= governs in the central axis



==> $\phi = \phi_0 * \alpha_h * \alpha_m$ = **0,0036**

Storey	$q_{stabil,g}$ [kN/m]	$q_{stabil,p}$ [kN/m]
1	0,78	0,68
2	0,52	0,46
3	0,26	0,23
4	0,00	0,00
5	0,00	0,00
6	0,00	0,00
7	0,00	0,00
8	0,00	0,00
9	0,00	0,00
10	0,00	0,00

2

Wind load:

For all storeys, a wind band of full storey height is applied.

In the standard storey, wind pressure is applied with superposition with

the stabilization loads.

The upper level receives additional wind load due to the parapet (1.5*pressure + 1.0*suction).
The lower level can optionally be calculated with wind suction only
for superposition with earth pressure.

Relevant for design in the lowest level: earth pressure + wind pressure

$q_{h,Wind}$ = **0,58 kN/m²** $c_{p,\ Druck}$ = **0,70** $c_{p,\ Sog}$ = **0,30**

Storey	$q_{Wind,k}$ [kN/m]		$q_{h,Wind}$	c_p	Influencing height
1	1,44	pressure	= 0,6 kN/m²	x 0,70	x 3,55 m
2	1,14	pressure	= 0,6 kN/m²	x 0,70	x 2,80 m
3	2,27	1,5*pressure + su	= 0,6 kN/m²	x 1,35	x 2,90 m
4	0,00	0	= 0,0 kN/m²	x 0,00	x 0,00 m
5	0,00	0	= 0,0 kN/m²	x 0,00	x 0,00 m
6	0,00	0	= 0,0 kN/m²	x 0,00	x 0,00 m
7	0,00	0	= 0,0 kN/m²	x 0,00	x 0,00 m
8	0,00	0	= 0,0 kN/m²	x 0,00	x 0,00 m
9	0,00	0	= 0,0 kN/m²	x 0,00	x 0,00 m
10	0,00	0	= 0,0 kN/m²	x 0,00	x 0,00 m

3

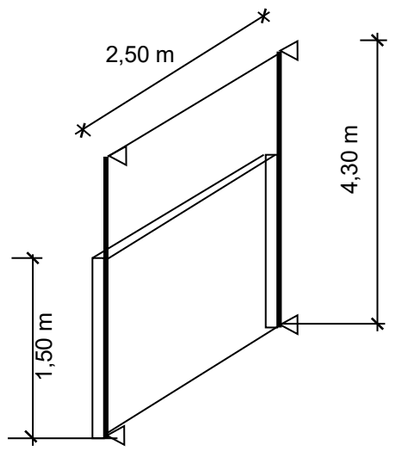
Earth pressure:

specific soil weight=	19,0 kN/m³	= γ
internal friction angle =	30,0 °	= ϕ
live load =	**3,0 kN/m²**	= p

passive pressure ratio: $K_{0'}$: 75,0 %

$K_{0'}$ = 0,25 * Ka + 0,75 * Ko

Storey height:	4,30 m
Column spacing:	2,50 m
Earth pressure height:	**1,50 m**



Storey	$q_{ehg,k}$ [kN/m]	$q_{ehp,k}$ [kN/m]
1	1,12	0,35

4

Load case superposition:

for storey 1: max q_d = max (LK1; LK2; LK3)

LK1 = $1,5*q_{Wind,k} + 1,35*q_{stabil,g} + 1,35*q_{eh,g} + 0,7*1,5*q_{eh,p} + 0,7*1,5*q_{stabil,p}$

LK2 = $0,6*1,5*q_{Wind,k} + 1,35*q_{stabil,g} + 1,35*q_{eh,g} + 1,5*q_{eh,p} + 0,7*1,5*q_{stabil,p}$

LK3 = $0,6*1,5*q_{Wind,k} + 1,35*q_{stabil,g} + 1,35*q_{eh,g} + 0,7*1,5*q_{eh,p} + 1,5*q_{stabil,p}$

for storey 2-10: max q_d = max (LK4; LK5)

LK 4 = $1,5*q_{Wind,k} + 1,35*q_{stabil,g} + 0,7*1,5*q_{stabil,p}$;

LK5 = $0,6*1,5*q_{Wind,k} + 1,35*q_{stabil,g} + 1,5*q_{stabil,p})$]

Storey	q_d [kN/m]	maßgeb. LK
1	5,82	1
2	2,89	4
3	4,00	4
4	0,00	
5	0,00	
6	0,00	
7	0,00	
8	0,00	
9	0,00	
10	0,00	

governs for the standard design!

System: **Single span beam between the ramp braces.**

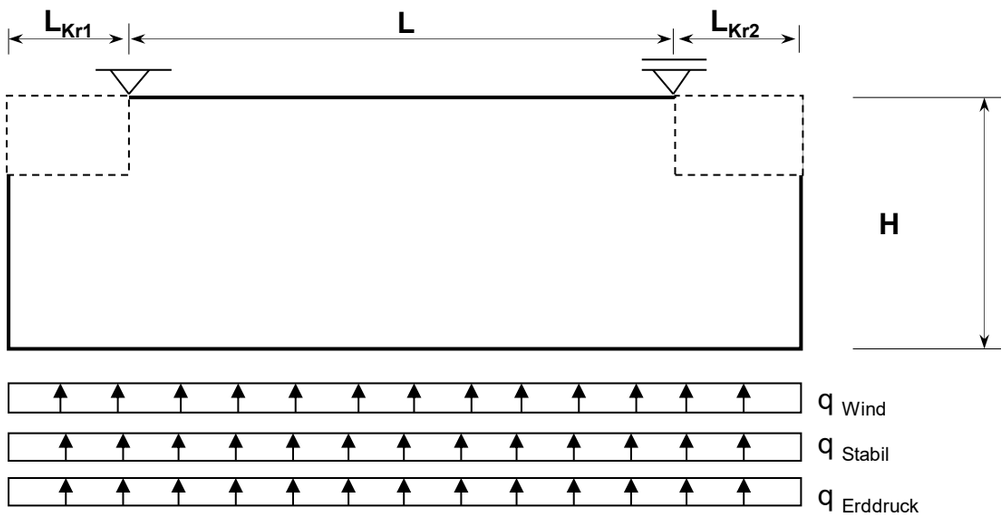
The cantilevers act to reduce the field moment.

Earthquake governs? **No**

Do cantilevers reduce? **Yes**

q_d = **5,82 kN/m** see 4 - Summary

L_{Kr1} = 10,3 m L = 47,9 m L_{Kr2} = 10,3 m



Cantilever arms:

moment at support, left M_{1d} = -308,9 kNm
moment at support, right M_{2d} = -308,9 kNm

Shear force left: V_{1Ld} = -60,0 kN
Shear force right: V_{2Rd} = 60,0 kN

Support reaction left: A = 199,5 kN
Support reaction left: B = 199,5 kN

Field area:

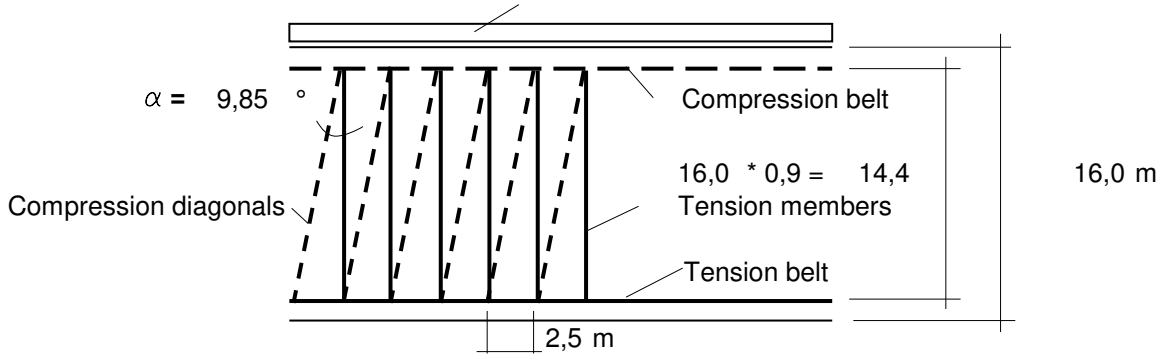
max $M_d = q_d * L^2/8$ = 1361,3 kNm

Shear force left: V_{1Rd} = 139,5 kN
Shear force right: V_{2Ld} = -139,5 kN

Permissible chord forces according to type test T 14 - 105 of 29. 07. 2014.						**Verification of the diaphragm**			
H = 16 m	zul V_k =	250 kN	-->	zul V_d =	375 kN	vorh V_d / zul V_d =	0,37	< 1	
	zul M_k =	900 kNm	-->	zul M_d =	1350 kNm	vorh M_d / zul M_d =	1,01	special verification required	
H = 11 m	zul V_k =	400 kN	-->	zul V_d =	600 kN	vorh V_d / zul V_d =	0,10	< 1	
	zul M_k =	600 kNm	-->	zul M_d =	900 kNm	vorh M_d / zul M_d =	0,34	< 1 -->	2 coupling elements in accordance with type testing

Influences from diaphragm forces in the ceiling slab (level 3) b = 16,0 m

In the final state, the ceiling slabs create a structure which is used to remove the horizontal loads in vertical reinforcing construction elements. A framework model is used as the load-bearing model in which the compression diagonals and the compression belt run through the concrete slabs. The tension members are formed by the composite beams, the tension belt by two closed steel loop on the edge of the prefabricated part.



Continuous loops:

The two outer loops are closed and laid through the slab in order to absorb and conduct ceiling slab forces. The outer loop is no longer required to remove shear force which is why all the loads from the diaphragm forces can be assigned to it. A maximum of

1,30 cm²/m is required from the second loop for bracing girder.
(cf. calculation of composite girder IPE 400S355; connection concrete belts)

Load γ_G= **1,35** γ_Q = **1,5**

$V_{E,\,diaphragm,\,d}$ = 140 kN loads diaphragm

$M_{E,\,diaphragm,\,d}$ = 1365 kNm (--> Moment redistribution of 85% considered)

Tension belt:

F_{Ed} = $M_{E,\,diaphragm,\,d}$ / ($h_{diaphragm}$ * 0,9)

1365 kNm / 14,4 m = 94,8 kN

erf A_{slice} = 94,8 kN / 43,4 kN/cm² = 2,18 cm²

erf $A_{composite\ force\ transfer}$ = 1,30 cm²/m * 0,6 m*(n-1) = 0,78 cm²

erf A_{total} = 0,78 cm² + 2,18 cm² = 2,97 cm²

chosen: n = **2** coupling elements **ϕ 10** A = 3,14 cm²

Compression belt:

vorh $A_{concrete}$ = ($h_{diaphragm}$ - 0,9 * $h_{diaphragm}$) * d_{slab}

(16,0 m - 14,4) m * 0,103 m

= 0,165 m²

F_{Rd} = 2,3 kN/cm² * 1648 cm² = 3735 kN

utility: 94,8 / 3735 = 0,03 < 1

Compression diagonals:

F_{Ed} = $V_{E,\,diaphragm,\,d}$ / cos α

140 kN / cos 9,85 = 142,1 kN

exist. $A_{concrete}$ (conservative estimate) 0,3 m * 0,103 m

= 0,0309 m²

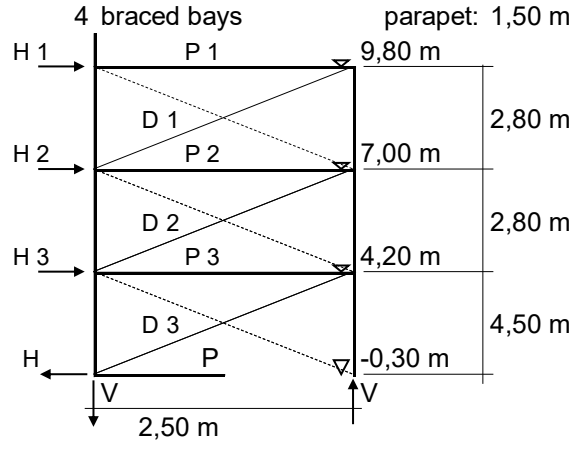
F_{Rd} = 2,3 kN/cm² * 309 cm² = 700,4 kN

utility: 142,1 / 700,4 = 0,20 < 1

Pos BR100 Vertical bracing

in gridline A — influence width per load introduction node: EFB = 8,16 m

System — parking garage length: L = 69,00 m



VV 3

EXC 1 welded version

Divide bracing field at H=0,33*story height

Additional compression and tension members see design below

Loads

	H 1	imperfection	3,9 kN	=	1/200*(g+q)*EFB*L/4	
		wind	5,4 kN	=	(cpd+cps)*q*EFB*H/4	
	H 2	imperfection	3,9 kN	=	1/200*(g+q)*EFB*L/4	
		wind	3,4 kN	=	(cpd+cps)*q*EFB*H/4	
Soil pressure	H 3	imperfection	6,3 kN	=	1/200*(g+q)*EFB*L/4	
Soil pressure		wind	5,3 kN	=	(cpd+cps)*q*EFB*H/4	
characteristic bearing reaction			**$\Sigma H_{E,k}$**	**=**	**28,2 kN**	
			+/- $V_{E,k}$	**=**	**79,6 kN**	

total load on gridline		**K=20 mm**
H1d, ges	48,6 kN	4
H2d, ges	36,8 kN	3
H3d, ges	58,2 kN	4
$\Sigma H_{E,k}$ =	112,6 kN	
$\Sigma H_{E,d}$ =	143,6 kN	

internal forces and design — γ_G = 1,35 — γ_Q = 1,50

	selected		N_{Ed}	N_{Rd}	utilisation
P 1	**QR 80*4,0**	**S355**	13,3 kN	190,9 kN	0,07
P 2	**QR 80*4,0**	**S355**	23,7 kN	190,9 kN	0,12
P 3	**QR 80*4,0**	**S355**	40,1 kN	190,9 kN	0,21

			N_{Ed}	N_{Rd}	utilisation
D 1	**1 Ø 22,0 - M24**	**S355**	20,0 kN	124,5 kN	0,16
D 2	**1 Ø 22,0 - M24**	**S355**	35,5 kN	124,5 kN	0,29
D 3	**1 Ø 22,0 - M24**	**S355**	82,6 kN	124,5 kN	0,66

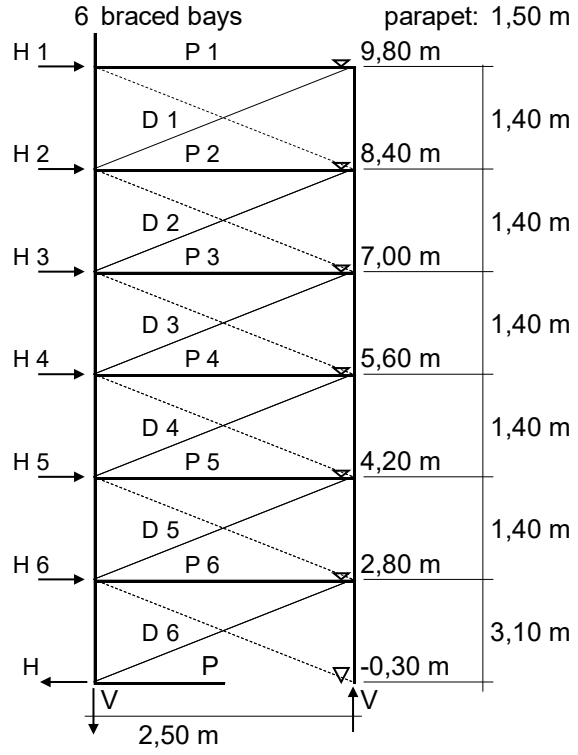
Pos BR101 Vertical bracing

in gridline H — influence width per load introduction node: EFB = 8,16 m

System — parking garage length: L = 69,00 m

6 braced bays — parapet: 1,50 m — VV 6

EXC 1 welded version



Divide bracing field at H=0,5*story height

Additional compression and tension members see design below

Loads

	H 1	imperfection	2,6 kN	=	1/200*(g+q)*EFB*L/6	**total load on gridline**	**K=20 mm**
		wind	3,6 kN	=	(cpd+cps)*q*EFB*H/6	H1d, ges 48,6 kN	4
	H 2	imperfection	2,6 kN	=	1/200*(g+q)*EFB*L/6		
		wind	3,6 kN	=	(cpd+cps)*q*EFB*H/6	H2d, ges 48,6 kN	4
	H 3	imperfection	2,6 kN	=	1/200*(g+q)*EFB*L/6		
		wind	2,3 kN	=	(cpd+cps)*q*EFB*H/6	H3d, ges 36,8 kN	3
	H 4	imperfection	2,6 kN	=	1/200*(g+q)*EFB*L/6		
		wind	2,3 kN	=	(cpd+cps)*q*EFB*H/6	H4d, ges 36,8 kN	3
	H 5	imperfection	2,6 kN	=	1/200*(g+q)*EFB*L/6		
		wind	2,3 kN	=	(cpd+cps)*q*EFB*H/6	H5d, ges 36,8 kN	3
Soil pressure	H 6	imperfection	4,2 kN	=	1/200*(g+q)*EFB*L/6		
Soil pressure		wind	3,0 kN	=	(cpd+cps)*q*EFB*H/6	H6d, ges 53,8 kN	4
characteristic bearing reaction			**$\Sigma H_{E,k}$**	**=**	**34,1 kN**	**$\Sigma H_{E,k}$ =** 204,6 kN	
			+/- $V_{E,k}$	**=**	**89,7 kN**	**$\Sigma H_{E,d}$ =** 261,5 kN	

internal forces and design — γ_G = 1,35 — γ_Q = 1,50

	selected		N_{Ed}	N_{Rd}	utilisation
P 1	**QR 80*4,0**	**S355**	8,9 kN	190,9 kN	0,05
P 2	**QR 80*4,0**	**S355**	17,7 kN	190,9 kN	0,09
P 3	**QR 80*4,0**	**S355**	24,6 kN	190,9 kN	0,13
P 4	**QR 80*4,0**	**S355**	31,6 kN	190,9 kN	0,17
P 5	**QR 80*4,0**	**S355**	38,5 kN	190,9 kN	0,20
P 6	**QR 80*4,0**	**S355**	48,6 kN	190,9 kN	0,25

			N_{Ed}	N_{Rd}	utilisation
D 1	**1 Ø 22,0 - M24**	**S355**	10,2 kN	124,5 kN	0,08
D 2	**1 Ø 22,0 - M24**	**S355**	20,3 kN	124,5 kN	0,16
D 3	**1 Ø 22,0 - M24**	**S355**	28,3 kN	124,5 kN	0,23
D 4	**1 Ø 22,0 - M24**	**S355**	36,2 kN	124,5 kN	0,29
D 5	**1 Ø 22,0 - M24**	**S355**	44,1 kN	124,5 kN	0,35
D 6	**1 Ø 22,0 - M24**	**S355**	77,4 kN	124,5 kN	0,62

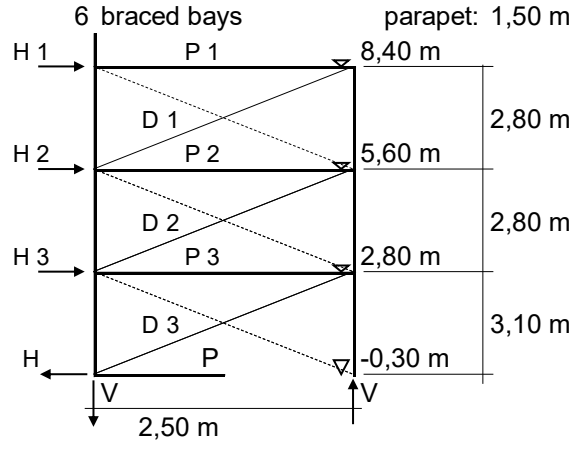
Pos BR102 Vertical bracing

in gridline O — influence width per load introduction node: EFB = 16,32 m

System — parking garage length: L = 69,00 m

6 braced bays — parapet: 1,50 m — VV 3

EXC 1 welded version



Loads

					total load on gridline	**K=20 mm**
H 1	imperfection	5,2 kN	=	1/200*(g+q)*EFB*L/6	H1d, ges 97,1 kN	7
	wind	7,2 kN	=	(cpd+cps)*q*EFB*H/6		
H 2	imperfection	5,2 kN	=	1/200*(g+q)*EFB*L/6	H2d, ges 73,6 kN	6
	wind	4,6 kN	=	(cpd+cps)*q*EFB*H/6		
H 3	imperfection	5,2 kN	=	1/200*(g+q)*EFB*L/6	H3d, ges 75,8 kN	6
	wind	4,8 kN	=	(cpd+cps)*q*EFB*H/6		

characteristic bearing reaction

				total load on gridline	
	$\Sigma H_{E,k}$	**=**	**32,0 kN**	**$\Sigma H_{E,k}$ =**	192,3 kN
	+/- $V_{E,k}$	**=**	**78,3 kN**	**$\Sigma H_{E,d}$ =**	246,6 kN

internal forces and design — γ_G = 1,35 — γ_Q = 1,50

	selected		N_{Ed}	N_{Rd}	utilisation
P 1	**QR 80*4,0**	**S355**	17,7 kN	190,9 kN	0,09
P 2	**QR 80*4,0**	**S355**	31,6 kN	190,9 kN	0,17
P 3	**QR 80*4,0**	**S355**	45,8 kN	190,9 kN	0,24

			N_{Ed}	N_{Rd}	utilisation
D 1	**1 Ø 22,0 - M24**	**S355**	26,6 kN	124,5 kN	0,21
D 2	**1 Ø 22,0 - M24**	**S355**	47,4 kN	124,5 kN	0,38
D 3	**1 Ø 22,0 - M24**	**S355**	72,9 kN	124,5 kN	0,59

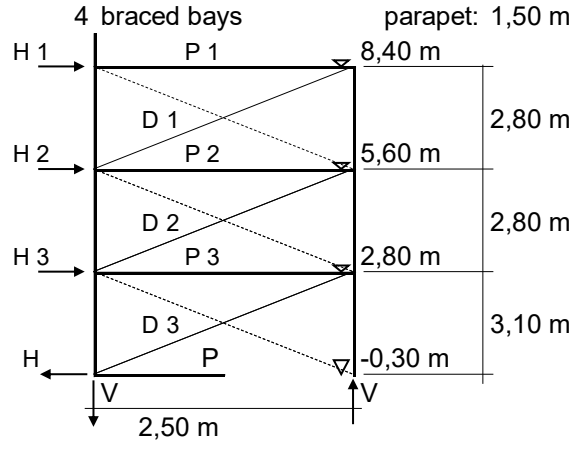
Pos BR103 Vertical bracing

in gridline V — influence width per load introduction node: EFB = 8,16 m

System — parking garage length: L = 69,00 m

4 braced bays — parapet: 1,50 m — VV 3

EXC 1 welded version



Loads

					total load on gridline	**K=20 mm**
H 1	imperfection	3,9 kN	=	1/200*(g+q)*EFB*L/4	H1d, ges 48,6 kN	4
	wind	5,4 kN	=	(cpd+cps)*q*EFB*H/4		
H 2	imperfection	3,9 kN	=	1/200*(g+q)*EFB*L/4	H2d, ges 36,8 kN	3
	wind	3,4 kN	=	(cpd+cps)*q*EFB*H/4		
H 3	imperfection	3,9 kN	=	1/200*(g+q)*EFB*L/4	H3d, ges 37,9 kN	3
	wind	3,6 kN	=	(cpd+cps)*q*EFB*H/4		

characteristic bearing reaction

				total load on gridline	
	$\Sigma H_{E,k}$	**=**	**24,0 kN**	**$\Sigma H_{E,k}$ =**	96,1 kN
	+/- $V_{E,k}$	**=**	**58,7 kN**	**$\Sigma H_{E,d}$ =**	123,3 kN

internal forces and design — γ_G = 1,35 — γ_Q = 1,50

	selected		N_{Ed}	N_{Rd}	utilisation
P 1	**QR 80*4,0**	**S355**	13,3 kN	190,9 kN	0,07
P 2	**QR 80*4,0**	**S355**	23,7 kN	190,9 kN	0,12
P 3	**QR 80*4,0**	**S355**	34,3 kN	190,9 kN	0,18

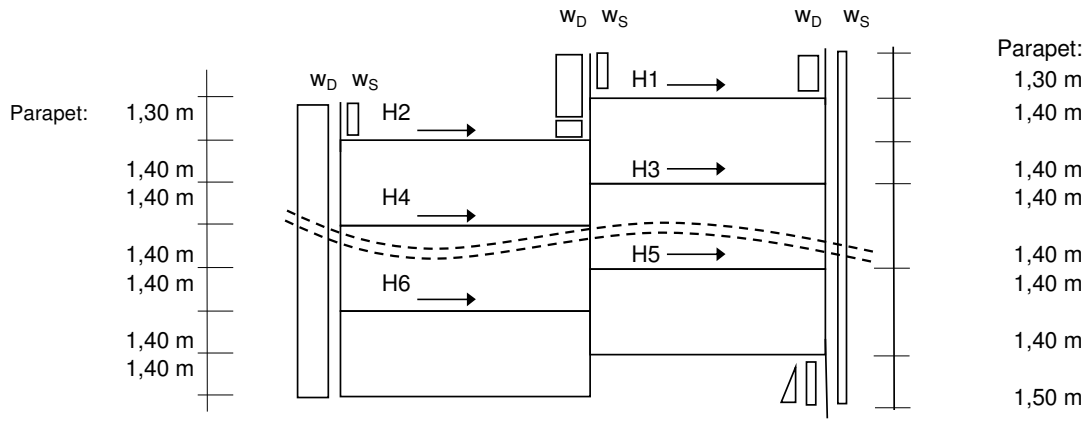
			N_{Ed}	N_{Rd}	utilisation
D 1	**1 Ø 22,0 - M24**	**S355**	20,0 kN	124,5 kN	0,16
D 2	**1 Ø 22,0 - M24**	**S355**	35,5 kN	124,5 kN	0,29
D 3	**1 Ø 22,0 - M24**	**S355**	54,7 kN	124,5 kN	0,44

Pos. 150 Vertical bracing without roof
Determination of horizontal loads

Width of influence: 35,00 m

				pressure			suction	
Wind:	q =	0,6	kN/m²	$c_{p,pr.}$ =	0,7		$c_{p,suc.}$ =	0,3
	w_D =	0,42	kN/m²	w_S =	0,18	kN/m²		
	from PV system: hw =		0,00	kN/m				
earth pressure:	e_g =	1,20	kN/m	e_p =	0,40	kN/m	Pos. SL600	
	concrete slab							
inclination:	g_{slab} =	3,0	kN/m²	p_{Decke} =	2,5	kN/m²		

Geometry: **H- load** →



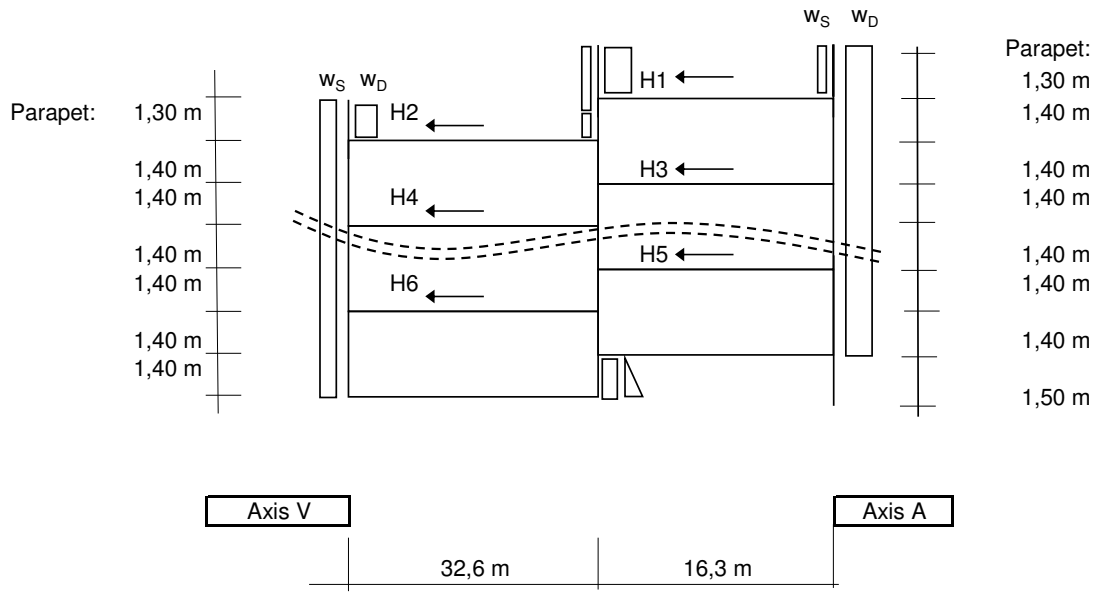


Load:

Wind w		inclination g		inclination p		Earth pressure g	Earth pressure p
$H1_w$ =	73,71 kN	$H1_{sg}$ =	8,57 kN	$H1_{sp}$ =	7,14 kN		
$H2_w$ =	58,17 kN	$H2_{sg}$ =	17,12 kN	$H2_{sp}$ =	14,26 kN		
$H3_w$ =	17,64 kN	$H3_{sg}$ =	8,57 kN	$H3_{sp}$ =	7,14 kN		
$H4_w$ =	41,16 kN	$H4_{sg}$ =	17,12 kN	$H4_{sp}$ =	14,26 kN		
$H5_w$ =	17,64 kN	$H5_{sg}$ =	8,57 kN	$H5_{sp}$ =	7,14 kN	$H5_{Earth\ pr.\ g}$ = -42,00 kN	$H5_{Earth\ pr.\ p}$ = -14,00 kN
$H6_w$ =	41,16 kN	$H6_{sg}$ =	17,12 kN	$H6_{sp}$ =	14,26 kN		

Pos 150

Geometry: **H- load ←**



Load:

Wind w		inclination g		inclination p		Earth pressure g	Earth pressure p
$H1_w$ =	79,6 kN	$H1_{sg}$ =	8,57 kN	$H1_{sp}$ =	7,14 kN		
$H2_w$ =	40,5 kN	$H2_{sg}$ =	17,12 kN	$H2_{sp}$ =	14,26 kN		
$H3_w$ =	41,2 kN	$H3_{sg}$ =	8,57 kN	$H3_{sp}$ =	7,14 kN		
$H4_w$ =	17,6 kN	$H4_{sg}$ =	17,12 kN	$H4_{sp}$ =	14,26 kN		
$H5_w$ =	41,2 kN	$H5_{sg}$ =	8,57 kN	$H5_{sp}$ =	7,14 kN		
$H6_w$ =	17,6 kN	$H6_{sg}$ =	17,12 kN	$H6_{sp}$ =	14,26 kN	$H6_{Earth\ pr.\ g}$ = 42,0 kN	$H6_{Earth\ pr.\ p}$ = 14,0 kN

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MODEL

Project: PR0460 Model: 02-BR150_Ramp bracing 4' and 24'_1 Date: 04.09.2025

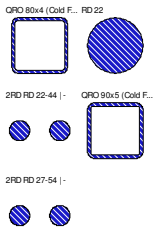
MODEL - GENERAL DATA

General	Model name	:	02-BR150_Ramp bracing 4' and 24'_1
	Type of model	:	3D
	Positive direction of global axis Z	:	Downward
	Classification of load cases and combinations	:	According to Standard: EN 1990 National Annex: CSN - Czech Republic
Options	☐ Use CQC Rule		
	☐ Enable CAD/BIM model		
	Standard Gravity		
	g	:	10.00 m/s²

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 355 | DIN EN 1993-1-1:2010-12					
	21000.00	8100.00	78.50	1.20E-05	1.00	Isotropic Linear Elastic
3	Steel S 460 NL | DIN EN 1993-1-1:2010-12					
	21000.00	8100.00	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
3	QRO 80x4 (kaltgefertigt)							
	1	180.00 11.70	111.00 5.13	111.00 5.13	0.00	0.00	80.0	80.0
4	RD 22							
	1	2.30 3.80	1.15 3.19	1.15 3.19	0.00	0.00	22.0	22.0
5	2RD RD 22-44							
	1	4.60 7.60	2.30 6.38	39.08 6.38	0.00	0.00	66.0	22.0
8	QRO 90x5 (kaltgefertigt)							
	1	316.00 16.40	193.00 7.19	193.00 7.19	0.00	0.00	90.0	90.0
9	2RD RD 27-54							
	3	10.43 11.46	5.22 9.63	88.76 9.63	0.00	0.00	81.0	27.0

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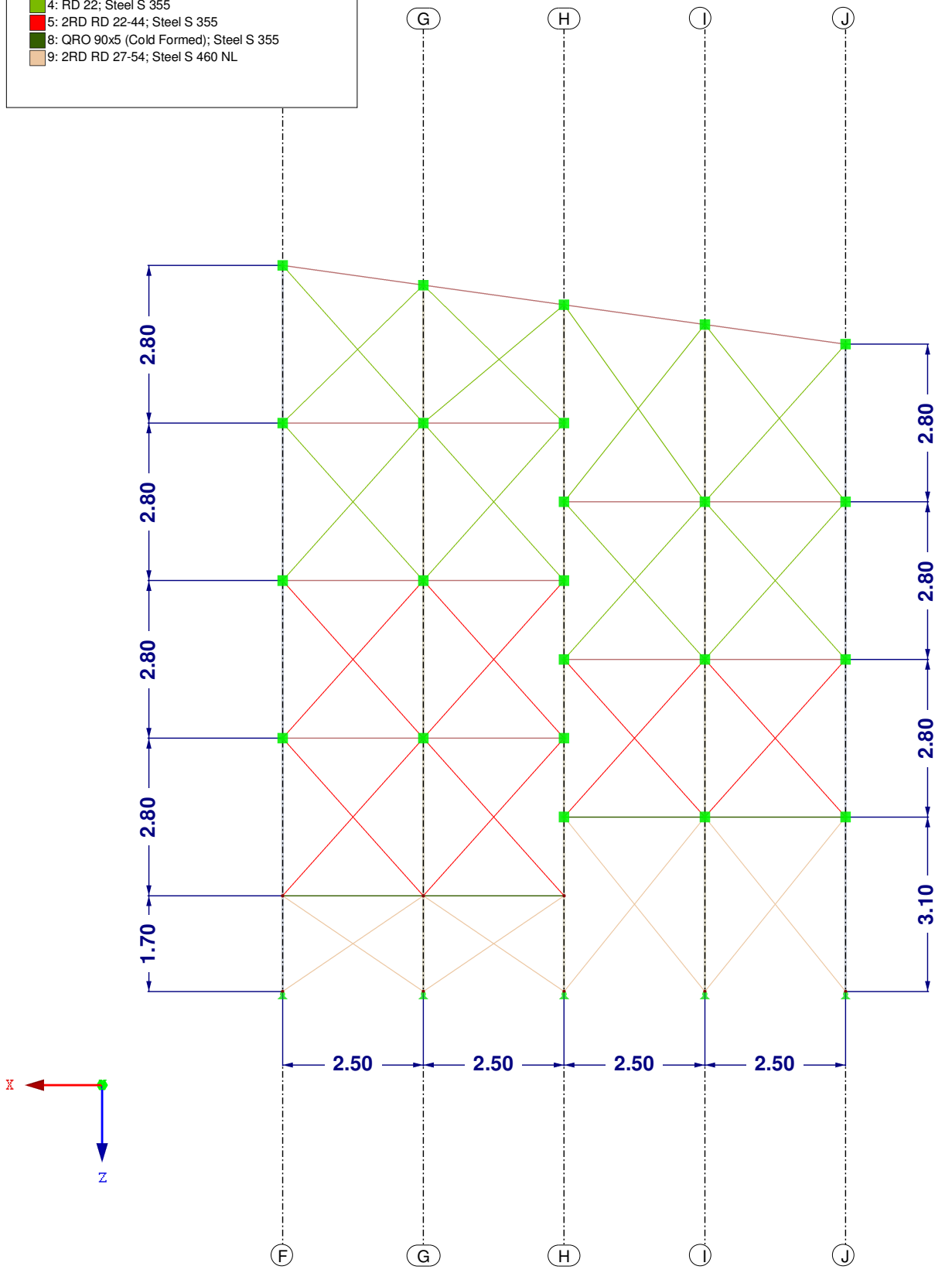
MODEL

Project: PR0460 Model: 02-BR150_Ramp bracing 4' and 24'_1 Date: 04.09.2025

MODEL

In Y-direction

Cross-Sections
- 3: QRO 80x4 (Cold Formed); Steel S 355
- 4: RD 22; Steel S 355
- 5: 2RD RD 22-44; Steel S 355
- 8: QRO 90x5 (Cold Formed); Steel S 355
- 9: 2RD RD 27-54; Steel S 460 NL



2.08 m

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LOADS

Project: PR0460 Model: 02-BR150_Ramp bracing 4' and 24'_1 Date: 04.09.2025

2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | CSN Action Category	Self-Weight - Factor in Direction Active	X	Y	Z
LC1	Wind ==>	Wind	☐			
LC2	Wind <==	Wind	☐			
LC3	Inclination g ==>	Permanent	☐			
LC4	Inclination p ==>	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			
LC6	Inclination g <==	Permanent	☐			
LC7	Inclination p <==	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			
LC9	Earth pressure g ==>	Permanent	☐			
LC10	Earth pressure p ==>	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			
LC11	Earth pressure g <==	Permanent	☐			
LC12	Earth pressure p <==	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters	
LC1	Wind ==>	Method of analysis	: ⊙ Geometrically linear analysis
LC2	Wind <==	Method of analysis	: ⊙ Geometrically linear analysis
LC3	Inclination g ==>	Method of analysis	: ⊙ Geometrically linear analysis
LC4	Inclination p ==>	Method of analysis	: ⊙ Geometrically linear analysis
LC6	Inclination g <==	Method of analysis	: ⊙ Geometrically linear analysis
LC7	Inclination p <==	Method of analysis	: ⊙ Geometrically linear analysis
LC9	Earth pressure g ==>	Method of analysis	: ⊙ Geometrically linear analysis
LC10	Earth pressure p ==>	Method of analysis	: ⊙ Geometrically linear analysis
LC11	Earth pressure g <==	Method of analysis	: ⊙ Geometrically linear analysis
LC12	Earth pressure p <==	Method of analysis	: ⊙ Geometrically linear analysis

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case
CO1		Design combination ==>	1	1.50	LC1 Wind ==>
			2	1.35	LC3 Inclination g ==>
			3	1.50	LC4 Inclination p ==>
			4	1.35	LC9 Earth pressure g ==>
			5	1.50	LC10 Earth pressure p ==>
			6	1.50	LC5 Inclination s (roof) ==>
CO2		Design combination <==	1	1.50	LC2 Wind <==
			2	1.35	LC6 Inclination g <==
			3	1.50	LC7 Inclination p <==
			4	1.35	LC11 Earth pressure g <==
			5	1.50	LC12 Earth pressure p <==
			6	1.50	LC8 Inclination s (roof) <==
CO3		Char. combination ==>	1	1.00	LC1 Wind ==>
			2	1.00	LC3 Inclination g ==>
			3	1.00	LC4 Inclination p ==>
			4	1.00	LC5 Inclination s (roof) ==>
			5	1.00	LC9 Earth pressure g ==>
			6	1.00	LC10 Earth pressure p ==>
CO4		Char. combination <==	1	1.00	LC2 Wind <==
			2	1.00	LC6 Inclination g <==
			3	1.00	LC7 Inclination p <==
			4	1.00	LC8 Inclination s (roof) <==
			5	1.00	LC11 Earth pressure g <==
			6	1.00	LC12 Earth pressure p <==
CO7		thermal design combination ==>	1	0.60	LC1 Wind ==>
			2	1.00	LC3 Inclination g ==>
			3	0.70	LC4 Inclination p ==>
			4	1.00	LC9 Earth pressure g ==>
			5	0.70	LC10 Earth pressure p ==>
CO8		thermal design combination <==	1	0.60	LC2 Wind <==
			2	1.00	LC6 Inclination g <==
			3	0.70	LC7 Inclination p <==
			4	1.00	LC11 Earth pressure g <==
			5	0.70	LC12 Earth pressure p <==

2.5.2 LOAD COMBINATIONS - CALCULATION PARAMETERS

Load Combin.	Description	Calculation Parameters	
CO1	Design combination ==>	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γM)
			: ☒ 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)
CO2	Design combination <==	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T

2.5.2 LOAD COMBINATIONS - CALCULATION PARAMETERS

Load Combin.	Description	Calculation Parameters
		Activate stiffness factors of: : ☒ Materials (partial factor γM) : ☒ 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)
CO3	Char. combination ==>	Method of analysis : ⊙ Second order analysis (P-Delta) Options : ☒ Consider favorable effects due to tension : ☒ Refer internal forces to deformed system for: ☒ Normal forces N ☒ Shear forces V_y and V_z ☒ Moments M_y, M_z and M_T Activate stiffness factors of: : ☒ Materials (partial factor γM) : ☒ 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)
CO4	Char. combination <==	Method of analysis : ⊙ Second order analysis (P-Delta) Options : ☒ Consider favorable effects due to tension : ☒ Refer internal forces to deformed system for: ☒ Normal forces N ☒ Shear forces V_y and V_z ☒ Moments M_y, M_z and M_T Activate stiffness factors of: : ☒ Materials (partial factor γM) : ☒ 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)
CO7	thermal design combination ==>	Method of analysis : ⊙ Second order analysis (P-Delta) Options : ☒ Consider favorable effects due to tension : ☒ Refer internal forces to deformed system for: ☒ Normal forces N ☒ Shear forces V_y and V_z ☒ Moments M_y, M_z and M_T Activate stiffness factors of: : ☒ Materials (partial factor γM) : ☒ 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)
CO8	thermal design combination <==	Method of analysis : ⊙ Second order analysis (P-Delta) Options : ☒ Consider favorable effects due to tension : ☒ Refer internal forces to deformed system for: ☒ Normal forces N ☒ Shear forces V_y and V_z ☒ Moments M_y, M_z and M_T Activate stiffness factors of: : ☒ Materials (partial factor γM) : ☒ 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)

2.6 RESULT COMBINATIONS

Result Combin	Description	Loading
RC1	Design combination CO1 or CO2	CO1/p or CO2/p
RC2	Char. combination CO3 or CO4	CO3/p or CO4/p
RC4	thermal design combination CO7 or CO8	CO7 or CO8

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC1
Wind ==>

LC1: Wind ==>

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	26	0 | Global XYZ	-9.00	0.00	0.00	0.00	0.00	0.00
2	1	0 | Global XYZ	-4.00	0.00	0.00	0.00	0.00	0.00
3	27	0 | Global XYZ	-80.00	0.00	0.00	0.00	0.00	0.00
4	28	0 | Global XYZ	-41.50	0.00	0.00	0.00	0.00	0.00
5	29	0 | Global XYZ	-52.50	0.00	0.00	0.00	0.00	0.00
6	2	0 | Global XYZ	-41.00	0.00	0.00	0.00	0.00	0.00
7	3	0 | Global XYZ	-18.00	0.00	0.00	0.00	0.00	0.00
8	4	0 | Global XYZ	-18.00	0.00	0.00	0.00	0.00	0.00

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC2
Wind <==

LC2: Wind <==

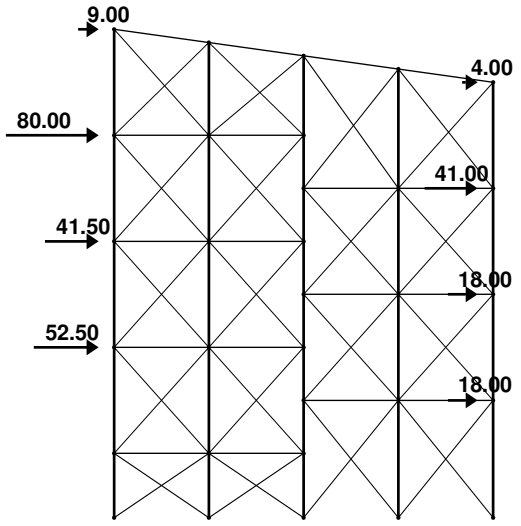
No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	26	0 | Global XYZ	4.00	0.00	0.00	0.00	0.00	0.00
2	1	0 | Global XYZ	9.00	0.00	0.00	0.00	0.00	0.00
3	27	0 | Global XYZ	74.00	0.00	0.00	0.00	0.00	0.00
4	28	0 | Global XYZ	18.00	0.00	0.00	0.00	0.00	0.00
5	29	0 | Global XYZ	22.50	0.00	0.00	0.00	0.00	0.00
6	2	0 | Global XYZ	58.50	0.00	0.00	0.00	0.00	0.00
7	4	0 | Global XYZ	41.50	0.00	0.00	0.00	0.00	0.00
8	3	0 | Global XYZ	41.50	0.00	0.00	0.00	0.00	0.00

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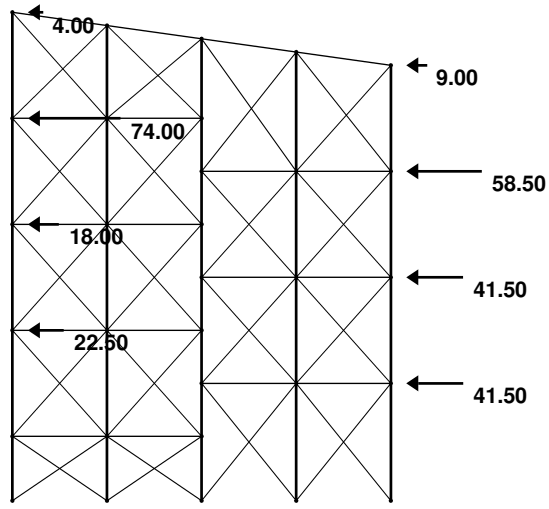
LC1 AND LC2: WIND IN BOTH DIRECTIONS

LC1 : Wind ==> In Y-direction
Loads [kN]



4 m
M 1:200

LC2 : Wind <== In Y-direction
Loads [kN]



4 m
M 1:200

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC3
Inclination g ==>

LC3: Inclination g ==>

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	1,26	0 | Global XYZ	-1.90	0.00	0.00	0.00	0.00	0.00
2	27-29	0 | Global XYZ	-9.00	0.00	0.00	0.00	0.00	0.00
3	2-4	0 | Global XYZ	-17.50	0.00	0.00	0.00	0.00	0.00

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

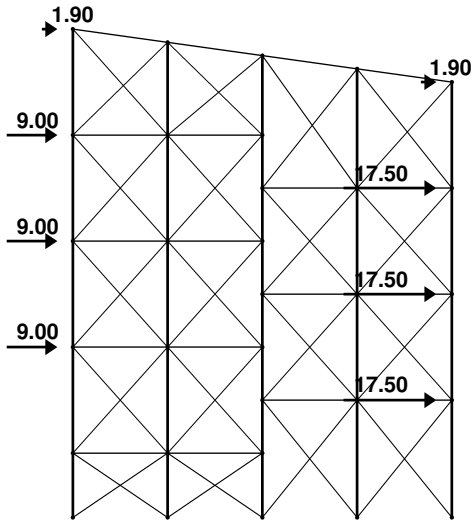
LC4
Inclination p ==>

LC4: Inclination p ==>

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	1,26	0 | Global XYZ	-1.60	0.00	0.00	0.00	0.00	0.00
2	27-29	0 | Global XYZ	-7.50	0.00	0.00	0.00	0.00	0.00
3	2-4	0 | Global XYZ	-14.50	0.00	0.00	0.00	0.00	0.00

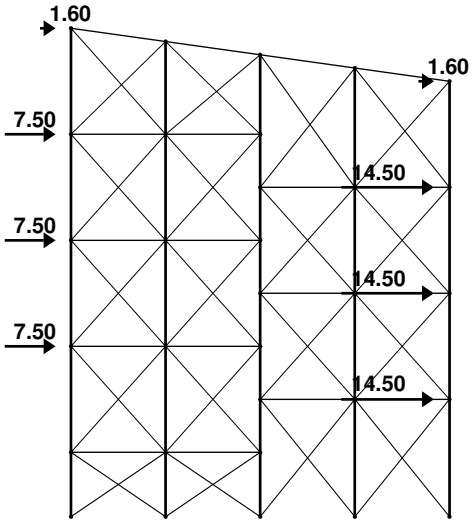
LC3 AND LC4: INCLINATION G + P ==>

LC3 : Inclination g ==> In Y-direction
Loads [kN]



4 m
M 1:200

LC4 : Inclination p ==> In Y-direction
Loads [kN]



4 m
M 1:200

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC6
Inclination g <==

LC6: Inclination g <==

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	1,26	0 | Global XYZ	1.90	0.00	0.00	0.00	0.00	0.00
2	27-29	0 | Global XYZ	9.00	0.00	0.00	0.00	0.00	0.00
3	2-4	0 | Global XYZ	17.50	0.00	0.00	0.00	0.00	0.00

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

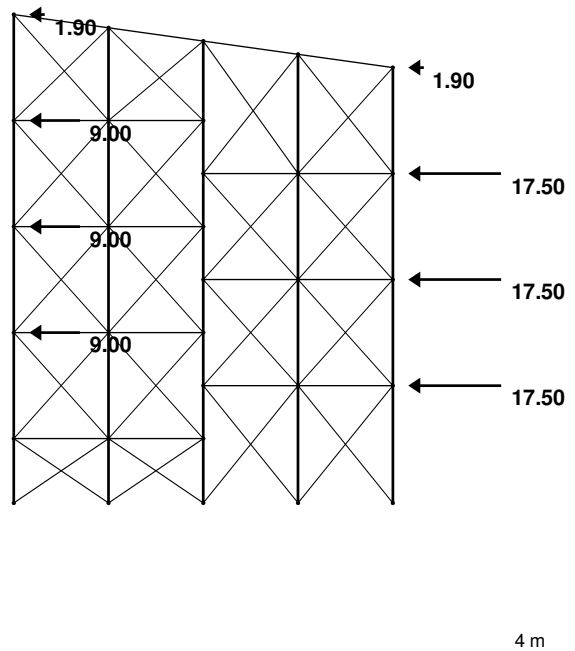
LC7
Inclination p <==

LC7: Inclination p <==

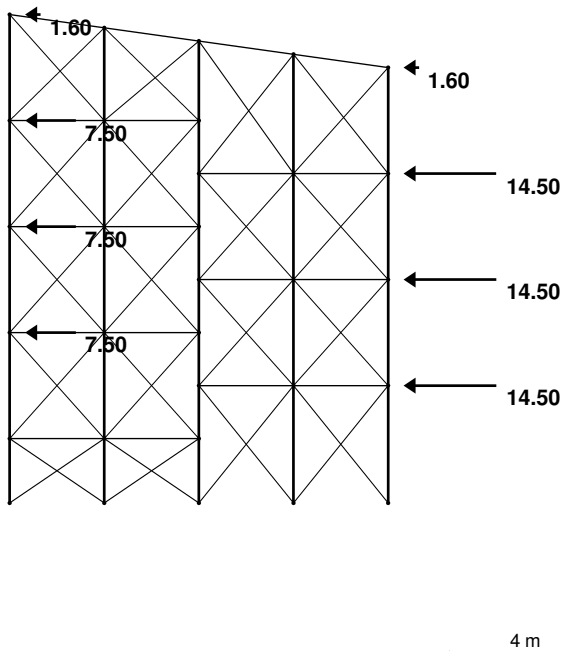
No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	1,26	0 | Global XYZ	1.60	0.00	0.00	0.00	0.00	0.00
2	27-29	0 | Global XYZ	7.50	0.00	0.00	0.00	0.00	0.00
3	2-4	0 | Global XYZ	14.50	0.00	0.00	0.00	0.00	0.00

LC6 AND LC7: INCLINATION G + P <==

LC6 : Inclination g <==
Loads [kN]
In Y-direction



LC7 : Inclination p <==
Loads [kN]
In Y-direction



3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC9
Earth pressure g ==>

LC9: Earth pressure g ==>

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	9,17	0 | Global XYZ	-21.00	0.00	0.00	0.00	0.00	0.00

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC10
Earth pressure p ==>

LC10: Earth pressure p ==>

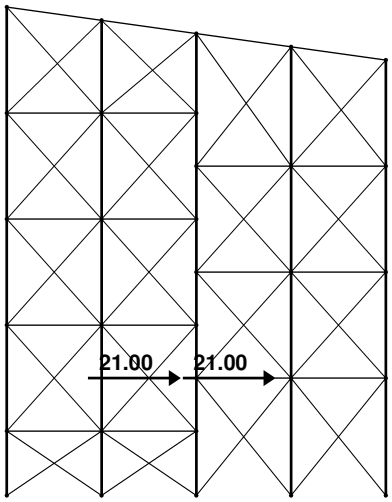
No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	9,17	0 | Global XYZ	-7.00	0.00	0.00	0.00	0.00	0.00

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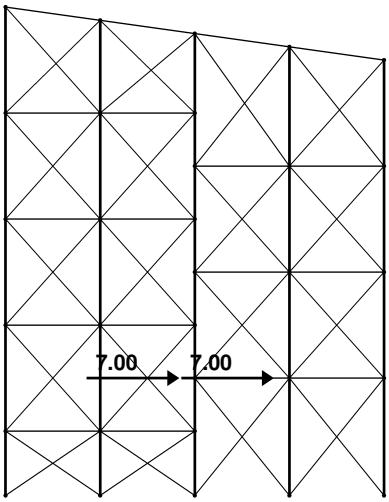
LC9 AND LC10: EARTH PRESSURE G + P ==>

LC9 : Earth pressure g ==> In Y-direction
Loads [kN]



4 m
M 1:200

LC10 : Earth pressure p ==> In Y-direction
Loads [kN]



4 m
M 1:200

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC11
Earth pressure g <==

LC11: Earth pressure g <==

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	23,29	0 | Global XYZ	21.00	0.00	0.00	0.00	0.00	0.00

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC12
Earth pressure p <==

LC12: Earth pressure p <==

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	23,29	0 | Global XYZ	7.00	0.00	0.00	0.00	0.00	0.00

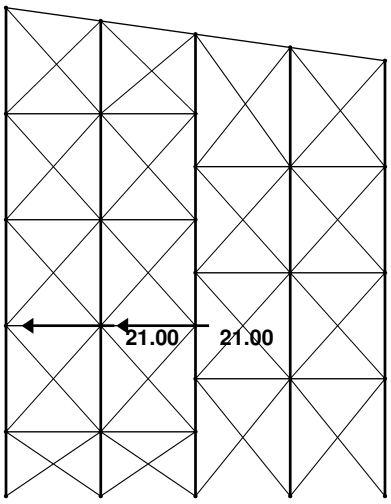
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LC11 AND LC12: EARTH PRESSURE G + P <==

LC11 : Earth pressure g <==
Loads [kN]

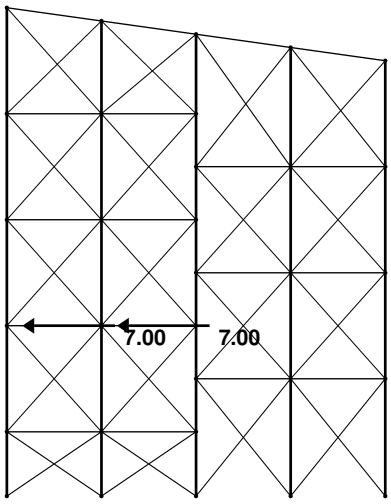
In Y-direction



4 m
M 1:200

LC12 : Earth pressure p <==
Loads [kN]

In Y-direction



4 m
M 1:200

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC13
Unit load

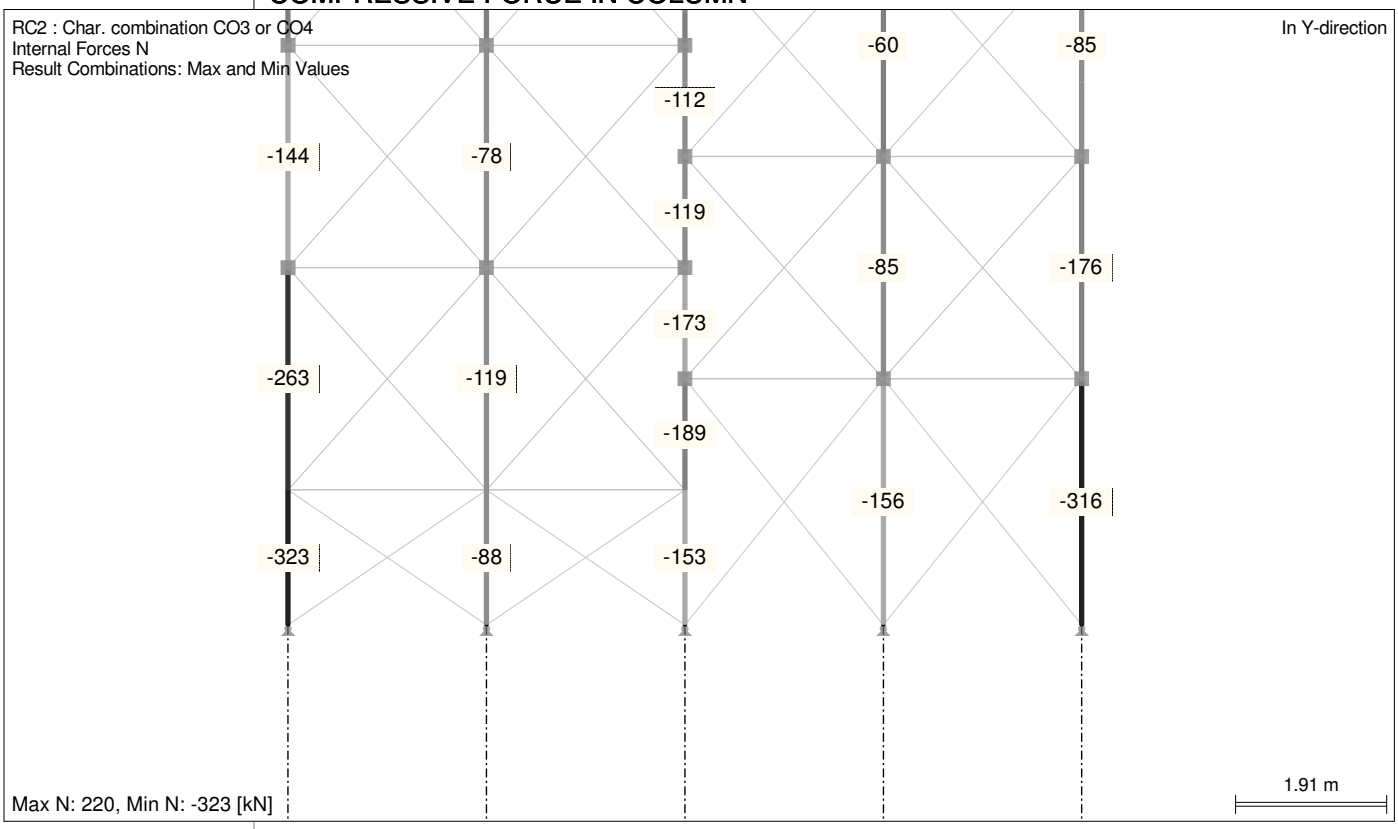
LC13: Unit load

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	26	0 | Global XYZ	-100.00	0.00	0.00	0.00	0.00	0.00

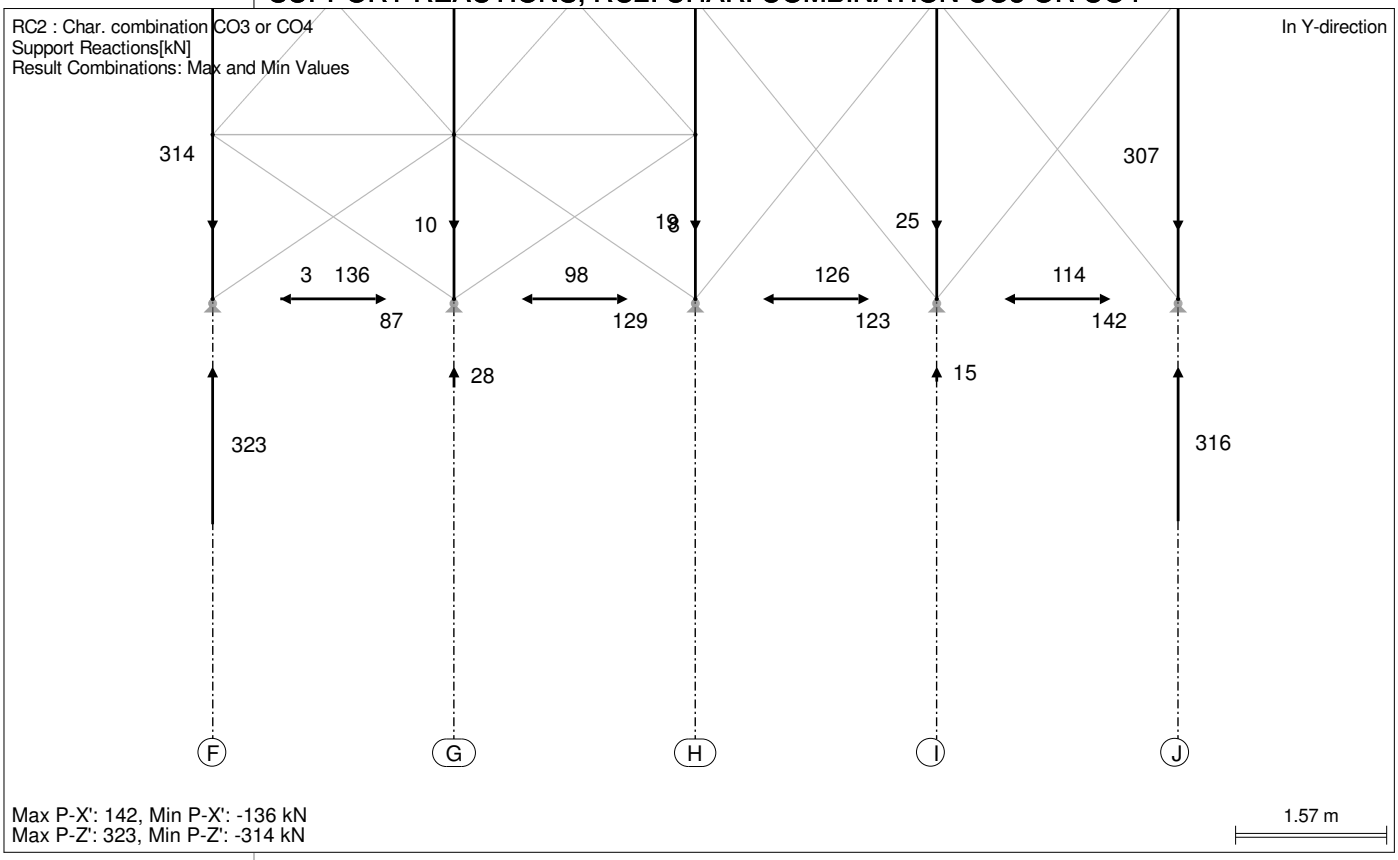
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INTERNAL FORCES N, RC2: CHAR. COMBINATION CO3 OR CO4; MAX. AXIAL COMPRESSIVE FORCE IN COLUMN



SUPPORT REACTIONS, RC2: CHAR. COMBINATION CO3 OR CO4



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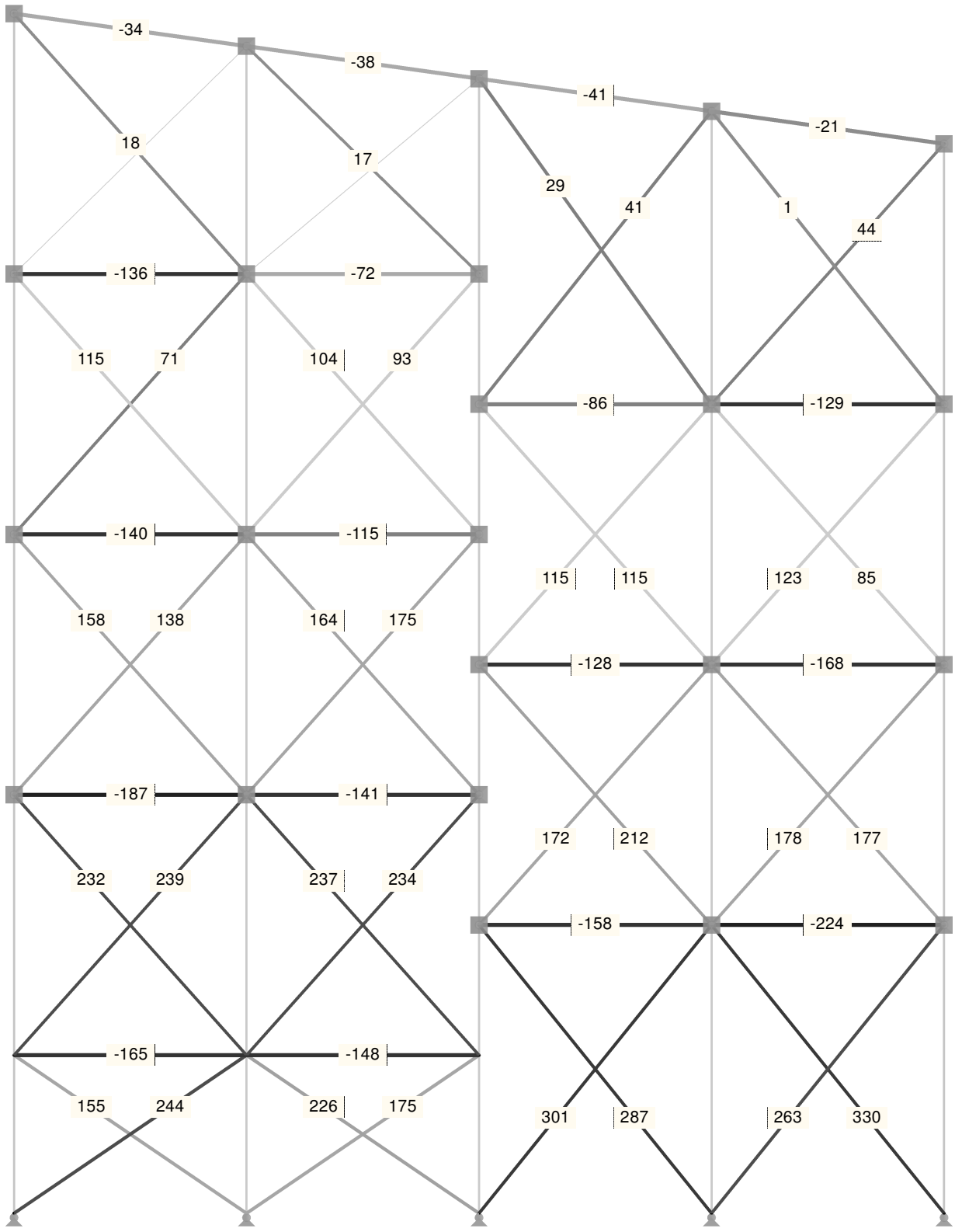
RESULTS

Project: PR0460 Model: 02-BR150_Ramp bracing 4' and 24'_1 Date: 04.09.2025

INTERNAL FORCES N, RC1: DESIGN COMBINATION CO1 OR CO2 FOR TENSION AND COUPLING MEMBERS

RC1 : Design combination CO1 or CO2
Internal Forces N
Result Combinations: Max and Min Values

In Y-direction



1.26 m

Max N: 330, Min N: -224 [kN]

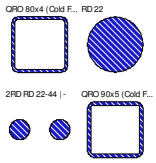
1.1 GENERAL DATA

Members to design:	5-16,21-32,41-55,61-75	
Sets of members to design:		
National Annex:	CSN	
Ultimate Limit State Design Result combinations to design:	RC1	Design combination CO1 or CO2

1.2 MATERIALS

Matl. No.	Material Description	E- Modulus E [kN/cm²]	Shear Modulus G [kN/cm²]	Poisson's Ratio ν [-]	Yield Stress f_{yk} [kN/cm²]	Max. Thickness t [mm]
1	Steel S 355 | DIN EN 1993-1-1:2010-12	21000.00	8076.92	0.300	35.50	40.0
					33.50	80.0
					31.50	100.0
					29.50	150.0
					28.50	200.0
					27.50	250.0
3	Steel S 460 NL | DIN EN 1993-1-1:2010-12	21000.00	8076.92	0.300	46.00	40.0
					43.00	80.0
					40.00	100.0
					38.00	150.0
					37.00	200.0

1.3 CROSS-SECTIONS



Sect. No.	Matl. No.	Cross-Section Description	Cross-Section Type	Max Design Ratio	Comment
3	1	QRO 80x4 (Cold Formed)	Box rolled	0.89	
4	1	RD 22	Round bar	0.91	
5	1	2RD RD 22-44	General	0.88	
	Type General - Only Class 3 and Class 4 possible				
8	1	QRO 90x5 (Cold Formed)	Box rolled	0.68	
9	3	2RD RD 27-54	General	0.63	
	Type General - Only Class 3 and Class 4 possible				



2.2 DESIGN BY CROSS-SECTION

Sect. No.	Member No.	Location x [m]	LC/CO/ RC	Design		Equation No.	Description
3	QRO 80x4 (Cold Formed)						
	64	0.00	RC1	0.12	≤ 1	CS101)	Cross-section check - Tension acc. to 6.2.3
	70	0.00	RC1	0.45	≤ 1	CS102)	Cross-section check - Compression acc. to 6.2.4
	41	0.00	RC1	0.02	≤ 1	ST301)	Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	70	0.00	RC1	0.89	≤ 1	ST302)	Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2
	41	0.00	RC1	0.02	≤ 1	ST311)	Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	70	0.00	RC1	0.89	≤ 1	ST312)	Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2
4	RD 22						
	6	0.00	RC1	0.00	≤ 1	CS100)	Negligible internal forces
	9	0.00	RC1	0.91	≤ 1	CS101)	Cross-section check - Tension acc. to 6.2.3
5	2RD RD 22-44						
	12	0.00	RC1	0.00	≤ 1	CS100)	Negligible internal forces
	72	0.00	RC1	0.88	≤ 1	CS101)	Cross-section check - Tension acc. to 6.2.3
8	QRO 90x5 (Cold Formed)						
	14	0.00	RC1	0.38	≤ 1	CS102)	Cross-section check - Compression acc. to 6.2.4
	14	0.00	RC1	0.68	≤ 1	ST302)	Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2
	14	0.00	RC1	0.68	≤ 1	ST312)	Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2
9	2RD RD 27-54						
	15	0.00	RC1	0.00	≤ 1	CS100)	Negligible internal forces
	16	0.00	RC1	0.63	≤ 1	CS101)	Cross-section check - Tension acc. to 6.2.3

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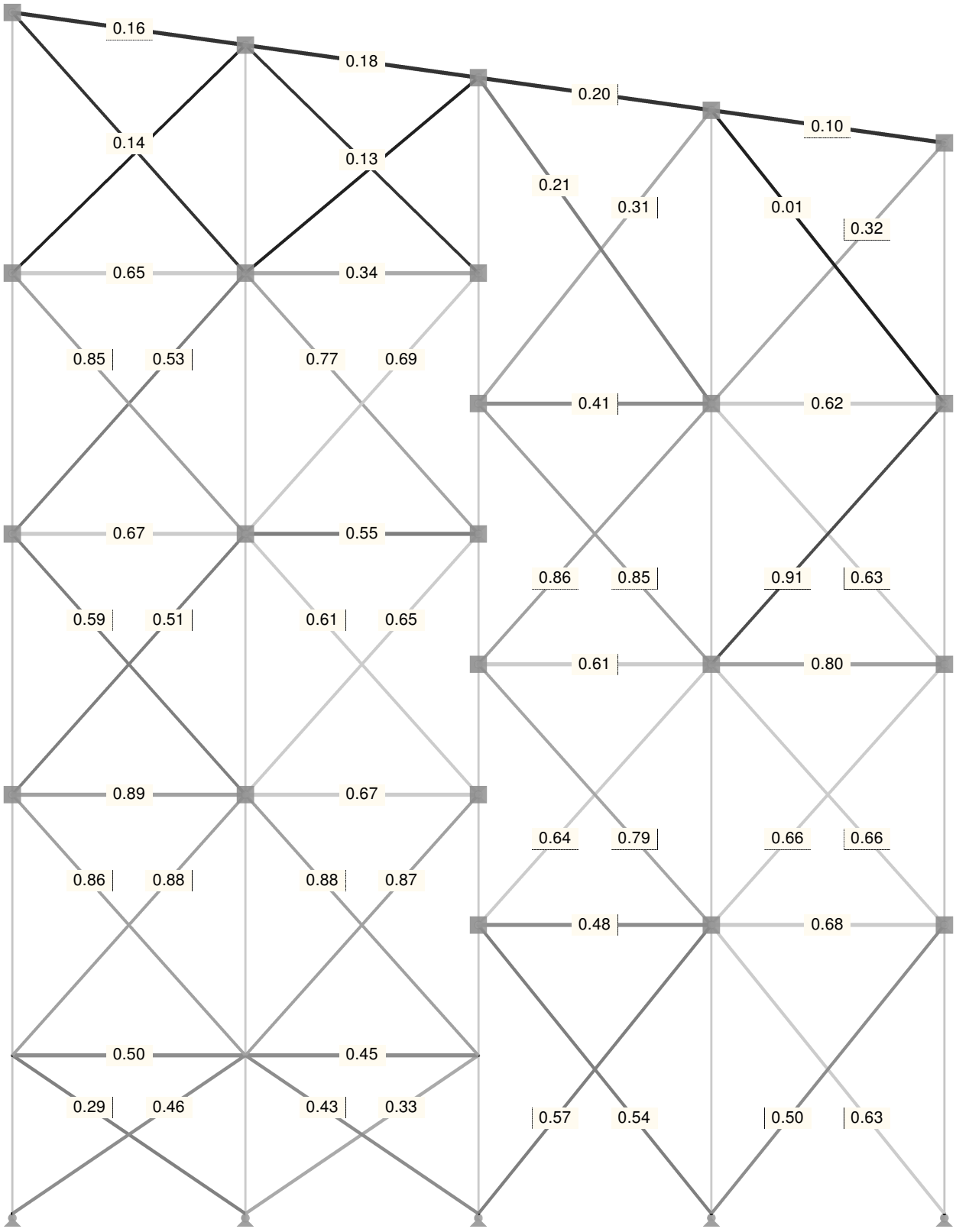
RS-STAHLEC3

Project: PR0460 Model: 02-BR150_Ramp bracing 4' and 24'_1 Date: 04.09.2025

STEEL EC3: DESIGN RATIO

STEEL EC3 CA7
Ultimate Limit State: Cross-Section Design, Stability Design, Weld Design, Pressure Design, Plastic Design

In Y-direction



Max Design Ratio: 0.91

1.24 m

Chapter 3: Roof

Pos. RO001 ff.

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Pos RO501 metal roof deck L = 5,00 m

v25.33

2-field-beam

field 1: L_1 = 2,50 m
field 2: L_2 = 2,50 m

permanent loads γ_G = 1,35

photovoltaic:
green roof:
roof build up:
dead load sheet metal: 0,09 kN/m²
g = 0,09 kN/m²

boundary conditions

normal roof deck negative orientation
deflection limit: 1 / 300 of span
intermediate support width: 200 mm

variable loads γ_Q = 1,5

snow: s = 0,60 kN/m² with Ψ = 0,5
installation: inst = 0,15 kN/m² with Ψ = 0,8
wind suction: w_{sd} = -1,65 kN/m²
wind pressure: w_{dd} = 1,45 kN/m² with Ψ = 1,0 deflection with wind pressure

notes:
The variable loads act together over the beam length.
With the guiding effect "wind pressure" the design load results to 2,93 kN/m².

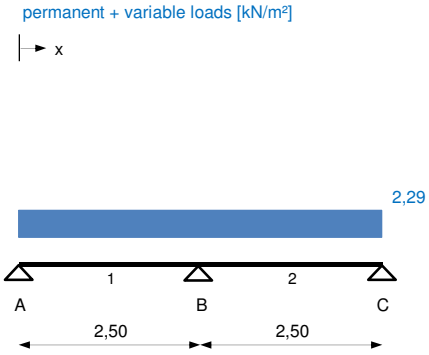
deflecion (LSS, rare combination)

support reactions + internal forces

	g [kN/m]	s [kN/m]	inst [kN/m]	w_{sd} [kN/m]	w_{dd} [kN/m]	$V_{d,li}$ [kN/m]	$V_{d,re}$ [kN/m]	V_d [kN/m]
A	0,09	0,56	0,14	-1,55	1,36		2,75	2,75
B	0,29	1,88	0,47	-5,16	4,53	-4,58	4,58	9,16
C	0,09	0,56	0,14	-1,55	1,36	-2,75		2,75
Σ	0,47	3,00	0,75	-8,25	7,25			

$M_{A,d}$ =
$M_{B,d}$ = -2,29 kNm/m
$M_{C,d}$ =

max M_d = 1,29 kNm/m (x= 0,94 m)



design

selected: **85 / 0,88 NEG**

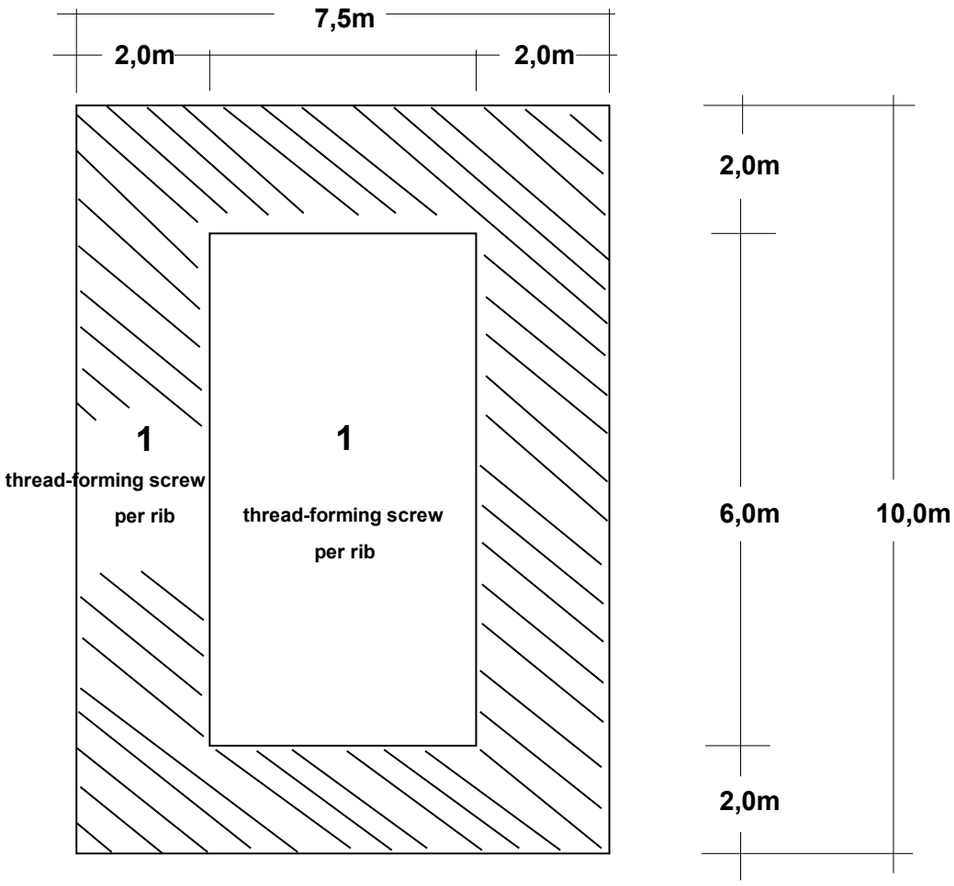
field moment		support moment			shear force		end support		intermediate support			interaction		max. support width		deflection	
M_{Ed}	$M_{c,Rd,F}$	M_{Ed}	$M_{c,Rd,B}$	$M_{0,Rd,B}$	V_{Ed}	$V_{w,Rd}$	$R_{Ed,A}$	$R_{w,Rd,A}$	$R_{Ed,B}$	$R_{w,Rd,B}$	$R_{0,Rd,B}$	M-V	M-R	avail.	allow.	avail.	L / 300
[kNm/m]		[kNm/m]			[kN/m]		[kN/m]		[kN/m]			[-]		[m]		[cm]	
Arcelor A 85/280/0,88 NEG (Normaldachblech Negativlage)																**max. utilisation**	**55%**
1,29	5,78	2,29	6,56	8,21	4,58	52,68	2,75	8,27	9,16	27,49	34,36	0,00 <1	0,55 <1	2,50	6,55	0,18	0,83
0,22 <1		0,35 <1			0,09 <1		0,33 <1		0,33 <1					0,38 <1		0,22 <1	
kein Trapezprofil von FISCHER möglich																	
Das FISCHER-Produktportfolio umfasst auch 200 mm hohe Dachbleche. Die Tragfähigkeitswerte entsprechen denen von SAB.																	
Münker M 85/280 NEG /0,88 (Normaldachblech Negativlage)																**max. utilisation**	**45%**
1,29	7,09	2,29	6,42	6,42	4,58	51,53	2,75	8,17	9,16	27,16	30,26	0,00 <1	0,45 <1	2,50	7,34	0,17	0,83
0,18 <1		0,36 <1			0,09 <1		0,34 <1		0,34 <1					0,34 <1		0,20 <1	
kein Trapezprofil von PRUSZYNSKI möglich																	
SAB 85R/1120/0,88 NEG (Normaldachblech Negativlage)																**max. utilisation**	**59%**
1,29	5,79	2,29	6,37	7,97	4,58	51,77	2,75	8,22	9,16	24,39	30,49	0,00 <1	0,59 <1	2,50	5,24	0,18	0,83
0,22 <1		0,36 <1			0,09 <1		0,33 <1		0,38 <1					0,48 <1		0,22 <1	
Salzgitter PS85/280/0,88 NEG (Normaldachblech Negativlage)																**max. utilisation**	**42%**
1,29	7,60	2,29	7,27	7,27	4,58	52,23	2,75	9,27	9,16	30,91	38,05	0,00 <1	0,37 <1	2,50	5,89	0,18	0,83
0,17 <1		0,31 <1			0,09 <1		0,30 <1		0,30 <1					0,42 <1		0,22 <1	

Project:	PR0460	MSCP Neratovice

Fastening scheme 2 - roof trapezoidal sheet

(for open canopies) Cold roof with thread-forming screws

Top of eaves: 12,90 m



Hatched area along all outer roof edges

1. End support and intermediate support

thread-forming screw EJOT JZ3-6,3*L E16 or equivalent, quantity according to sketch

2. Transverse joint

thread-forming screw EJOT JZ3-6,3*L E16 or equivalent, quantity according to sketch

3. Moment-resisting connection

Execution according to DIN EN 1090-4, Annex B.8

4. Longitudinal joint

Self-drilling screw EJOT-JT3-2H-Plus-5,5*25-E16 or equivalent, $e_L \leq 500$mm

5. Edge stiffening profile / Eaves purlin / Skylight frame

Self-drilling screw EJOT-JT3-2H-Plus-5,5*25-E16 or eq., $e_R \leq 500$ mm

RO520 Tension Rod Roof

Structural Specification:

- 1ø18.3 – M20, S355
- Intended for structural bracing of the ramp roof

Installation Concept:

Tension rods will be installed in all bays of the ramp roofs to provide structural reinforcement.

Additionally, square hollow sections (QR 80 × 4.0) will be mounted along axis 2’ and axis 26’ to contribute to the overall structural stiffening.

Reference:

For detailed positioning and installation, please refer to the corresponding layout drawings (position plans).

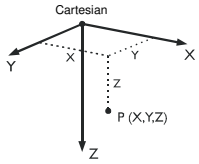
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MODEL - GENERAL DATA

	General	Model name	:	03-RO560_Ramp roof beam
		Type of model	:	3D
		Positive direction of global axis Z	:	Downward
		Classification of load cases and combinations	:	According to Standard: EN 1990 National Annex: CSN - Czech Republic
	Options	☐ Use CQC Rule		
		☐ Enable CAD/BIM model		
		Standard Gravity		
		g	:	10.00 m/s²



1.1 NODES

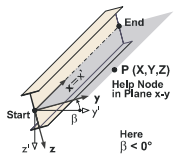
Node No.	Reference Node	Coordinate System	Node Coordinates X [m]	Y [m]	Z [m]	Comment
1	-	Cartesian	0.000	0.000	0.000	Supported
2	-	Cartesian	7.500	0.000	0.000	Supported

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 γM [-]	Material Model
1	Steel S 355 | DIN EN 1993-1-1:2010-12					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic

1.3 CROSS-SECTIONS

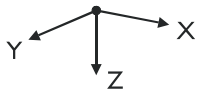
Section No.	Matl. No.	J [cm⁴] / A [cm²]	I_y [cm⁴] / A_y [cm²]	I_z [cm⁴] / A_z [cm²]	Principal Axes α [°]	Rotation α' [°]	Overall Dimensions [mm] Width b	Height h
2	HEA 200							
	1	20.98 53.83	3692.00 33.30	1336.00 10.77	0.00	0.00	200.0	190.0



1.7 MEMBERS

Mbr. No.	Member	Node Start	Node End	Rotation Type	Rotation β [°]	Cross-Section Start	Cross-Section End	Hinge No. Start	Hinge No. End	Ecc. No.	Div. No.	Length L [m]	
1	Beam	1	2	Angle	8.00	1	1	-	-	-	-	7.500	X

1.8 NODAL SUPPORTS



Support No.	Nodes No.	Sequen.	Rotation [°] about X	about Y	about Z	Column in Z	Support Conditions $u_{X'}$	$u_{Y'}$	$u_{Z'}$	$\varphi_{X'}$	$\varphi_{Y'}$	$\varphi_{Z'}$
1	1	XYZ	0.00	0.00	0.00	☐	☒	☒	☒	Spring	☐	Spring
2	2	XYZ	0.00	0.00	0.00	☐	☐	☒	☒	Spring	☐	Spring

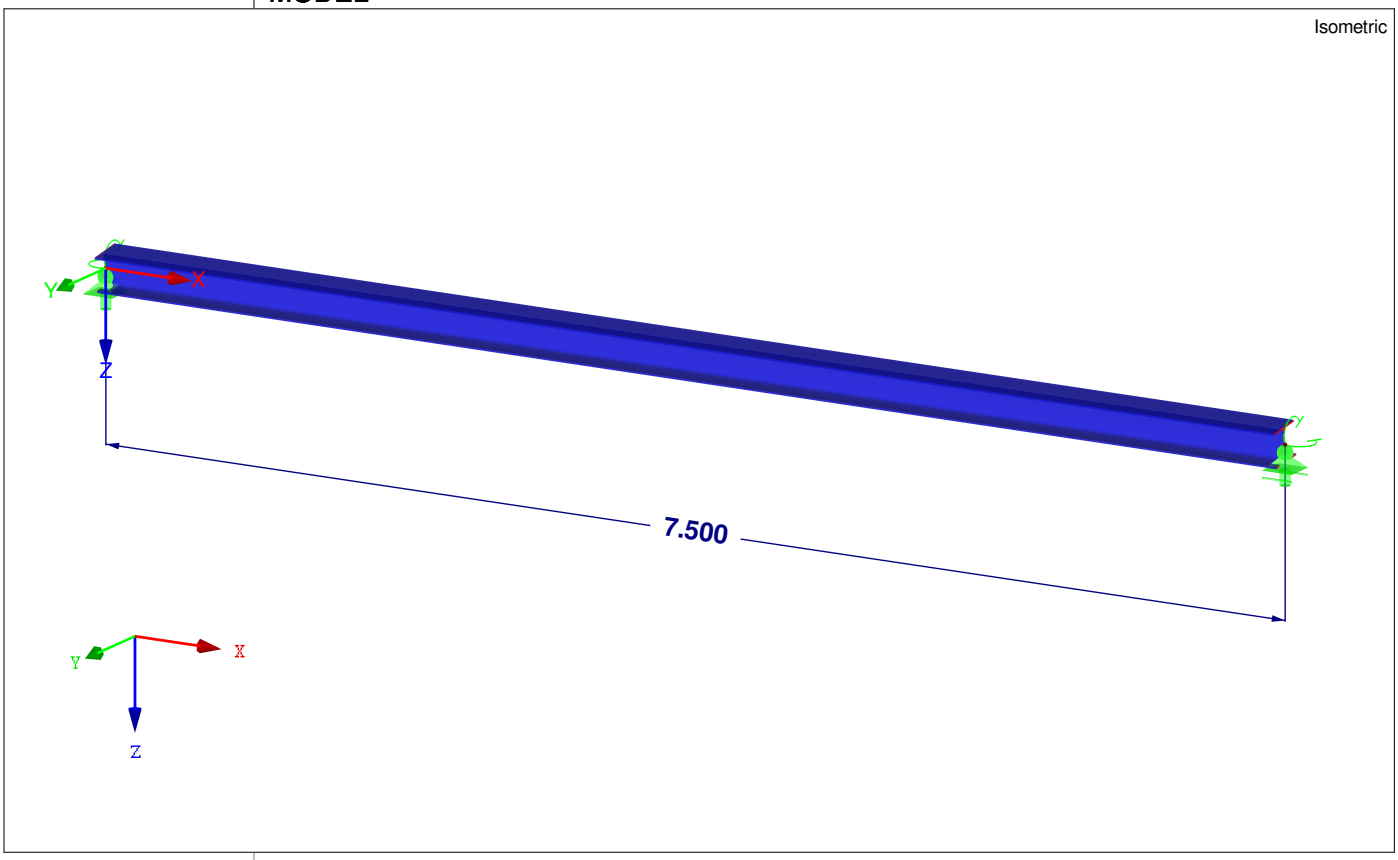
1.8.2 NODAL SUPPORTS - SPRINGS

Support No.	Nodes No.	Translation Spring [kN/m] $C_{u,X'}$	$C_{u,Y'}$	$C_{u,Z'}$	Rotation Spring [kNm/rad] $C_{\varphi,X'}$	$C_{\varphi,Y'}$	$C_{\varphi,Z'}$
1	1	-	-	-	1.000	-	1.000
2	2	-	-	-	1.000	-	1.000

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MODEL



PRECAMBER

Precamber f = 10 mm

2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | CSN Action Category	Self-Weight - Factor in Direction Active	X	Y	Z
LC1	Self-weight	Permanent	☒	0.000	0.000	1.000
LC2	Snow	Snow (H ≤ 1000 m a.s.l.)	☐			
LC3	Wind	Wind	☐			

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters		
LC1	Self-weight	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)
LC3	Wind	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	STR	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	1	1.35	LC1	Self-weight
			2	1.50	LC2	Snow
			3	0.90	LC3	Wind
CO2	STR	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	1	1.35	LC1	Self-weight
			2	0.75	LC2	Snow
			3	1.50	LC3	Wind

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2.5 LOAD COMBINATIONS

Load Combin.	Load Combination DS	Description	No.	Factor	Load Case	
CO3	S Ch	SLS - Characteristic	1	1.00	LC1	Self-weight
			2	1.00	LC2	Snow
			3	0.60	LC3	Wind
CO4	S Ch	SLS - Characteristic	1	1.00	LC1	Self-weight
			2	0.50	LC2	Snow
			3	1.00	LC3	Wind
CO5	ACC	ULS (EQU) - Accidental - psi-1,1	1	1.00	LC1	Self-weight
			2	0.50	LC2	Snow
			3	0.20	LC3	Wind
CO6	ACC	ULS (EQU) - Accidental - psi-1,1	1	1.00	LC1	Self-weight
			2	0.20	LC2	Snow
			3	0.60	LC3	Wind

2.5.2 LOAD COMBINATIONS - CALCULATION PARAMETERS

Load Combin.	Description	Calculation Parameters	
CO1	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γ_M)
			: ☒ 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)
CO2	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γ_M)
			: ☒ 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)
CO3	SLS - Characteristic	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γ_M)
			: ☒ 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)
CO4	SLS - Characteristic	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γ_M)
			: ☒ 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)
CO5	ULS (EQU) - Accidental - psi-1,1	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γ_M)
			: ☒ 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)
CO6	ULS (EQU) - Accidental - psi-1,1	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γ_M)
			: ☒ 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)

2.6 RESULT COMBINATIONS

Result Combin	Description	Loading
RC1	ULS (STR/GEO) - Permanent / transient - Eq. 6.10a and 6.10b	CO1 or CO2
RC2	SLS - Characteristic	CO3 or CO4
RC3	ULS (EQU) - Accidental - psi-1,1 - thermal design	CO5 or CO6

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LOADS

Project: PR0460 Model: 03-RO560_Ramp roof beam Date: 03.09.2025

LC1
Self-weight

3.2 MEMBER LOADS

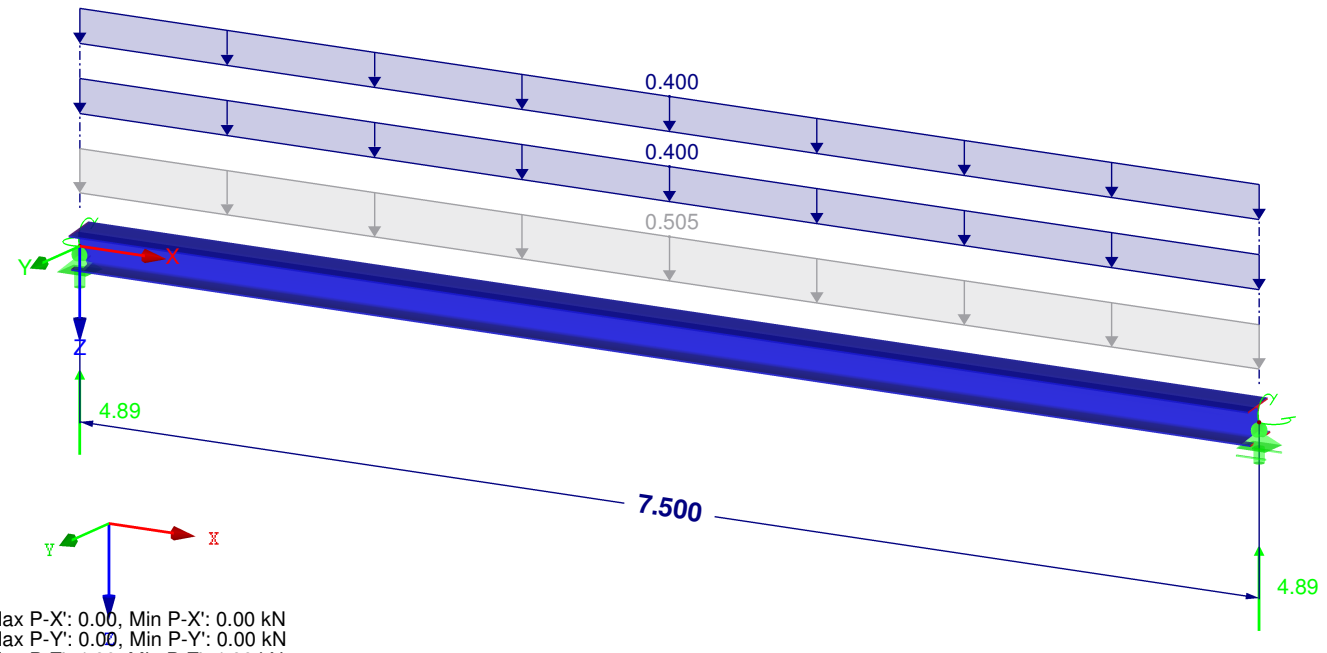
LC1: Self-weight

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	1	Force	Uniform	Z	True Length	p	0.400	kN/m
	installation: 0,15 kN/m² · 2,50 m = 0,375 kN/m								
2	Members	1	Force	Uniform	Z	True Length	p	0.400	kN/m
	trapezoidal sheet metal: 0,15 kN/m² · 2,50 m = 0,375 kN/m								

LC3: SELF-WEIGHT, SUPPORT REACTIONS

LC1 : Self-weight
Loads [kN/m]
Support Reactions[kN], [kNm]

Isometric



Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Y': 0.00, Min P-Y': 0.00 kN
Max P-Z': 4.89, Min P-Z': 4.89 kN
Max M-X': 0.00, Min M-X': 0.00 kNm
Max M-Y': 0.00, Min M-Y': 0.00 kNm
Max M-Z': 0.00, Min M-Z': 0.00 kNm

LC2
Snow

3.2 MEMBER LOADS

LC2: Snow

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	1.500	kN/m
	snow: 0,60 kN/m² · 2,50 m = 1,50 kN/m								

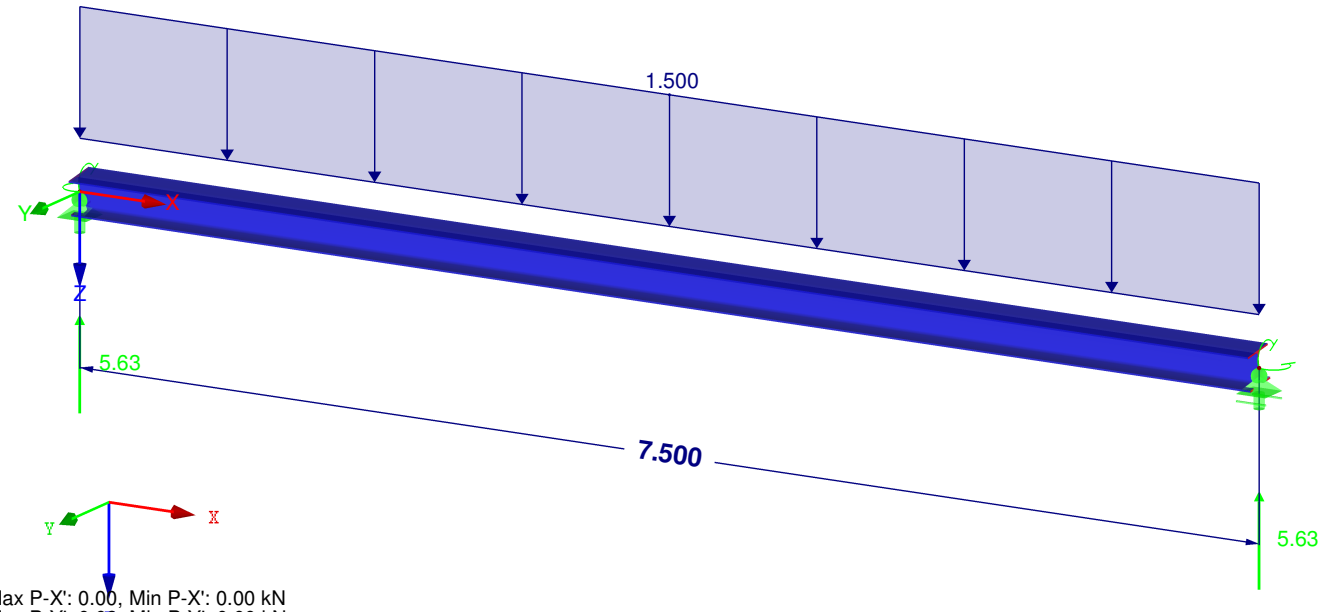
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LC2: SNOW, SUPPORT REACTIONS

LC2 : Snow
Loads [kN/m]
Support Reactions[kN], [kNm]

Isometric



Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Y': 0.00, Min P-Y': 0.00 kN
Max P-Z': 5.63, Min P-Z': 5.63 kN
Max M-X': 0.00, Min M-X': 0.00 kNm
Max M-Y': 0.00, Min M-Y': 0.00 kNm
Max M-Z': 0.00, Min M-Z': 0.00 kNm

LC3
Wind

3.2 MEMBER LOADS

LC3: Wind

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	2.600	kN/m
	wind: 0,60 kN/m² · 2,50 m · (1,20 + 2,40 + 1,60)/3 = 2,60 kN/m								

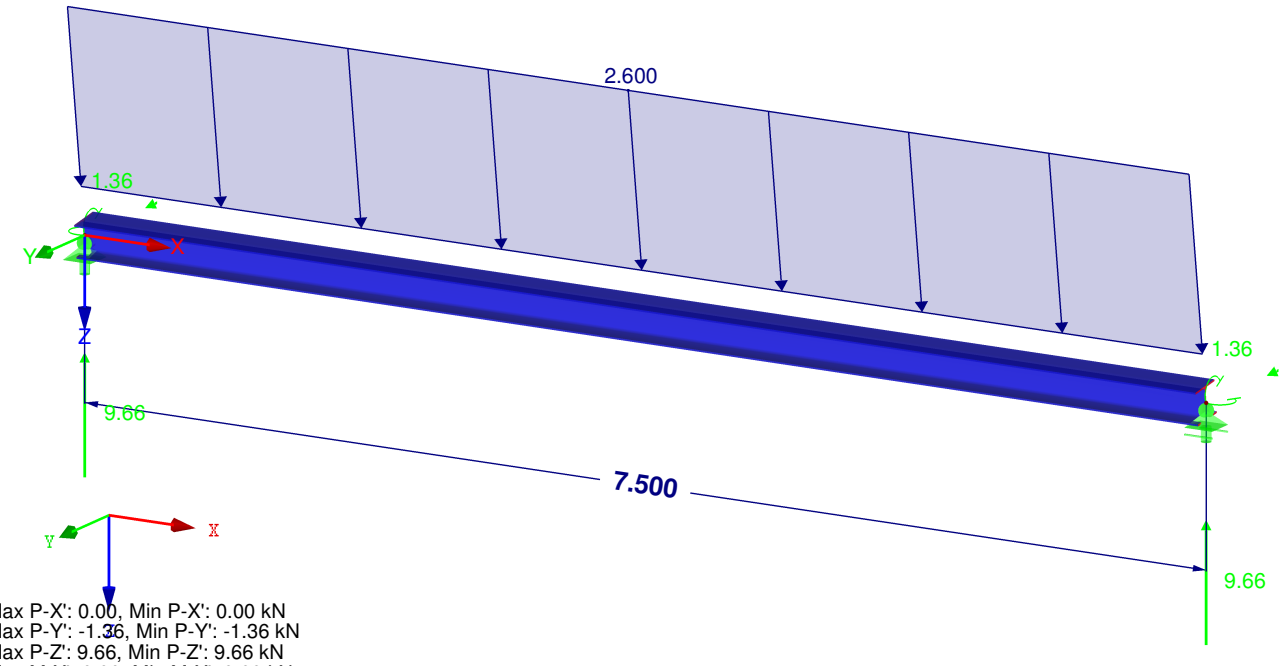
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LC3: WIND, SUPPORT REACTIONS

LC3 : Wind
Loads [kN/m]
Support Reactions[kN], [kNm]

Isometric



Max P-X': 0.00, Min P-X': 0.00 kN
Max P-Y': -1.36, Min P-Y': -1.36 kN
Max P-Z': 9.66, Min P-Z': 9.66 kN
Max M-X': 0.00, Min M-X': 0.00 kNm
Max M-Y': 0.00, Min M-Y': 0.00 kNm
Max M-Z': 0.00, Min M-Z': 0.00 kNm

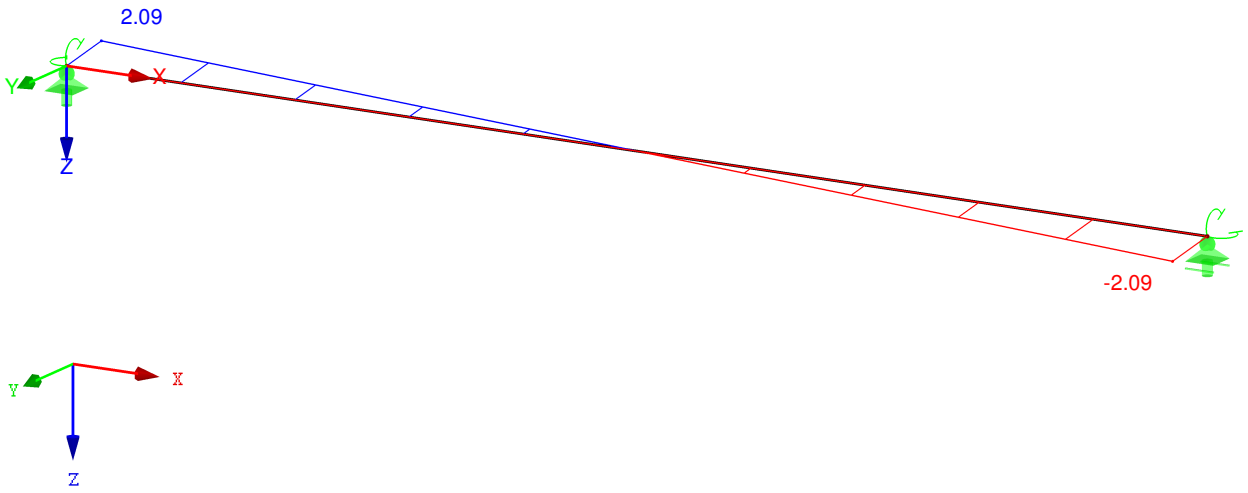
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INTERNAL FORCES V_y

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10a and 6.10b
Internal Forces V-y
Result Combinations: Max and Min Values

Isometric

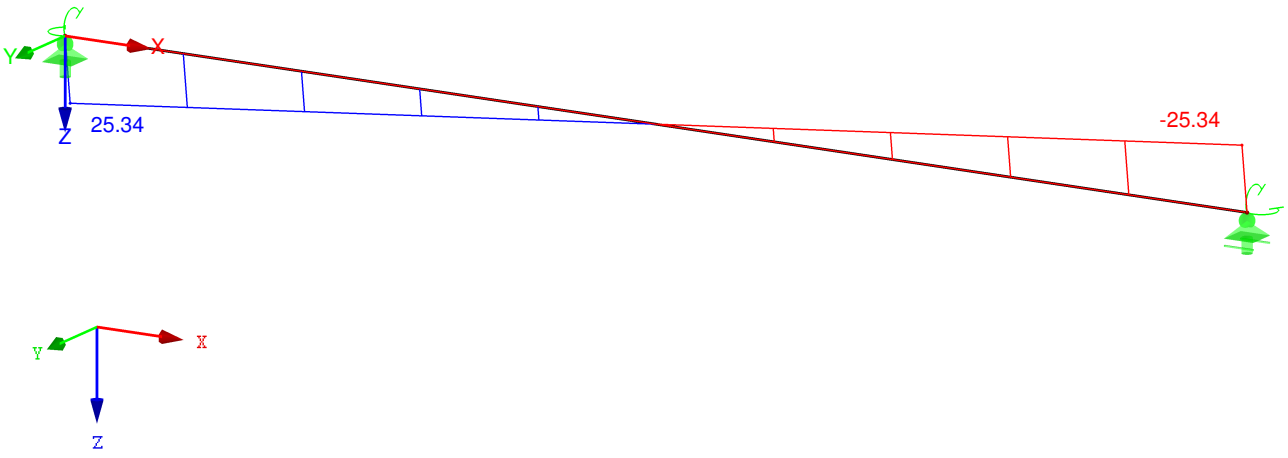


Max V-y: 2.09, Min V-y: -2.09 [kN]

INTERNAL FORCES V_z

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10a and 6.10b
Internal Forces V-z
Result Combinations: Max and Min Values

Isometric



Max V-z: 25.34, Min V-z: -25.34 [kN]

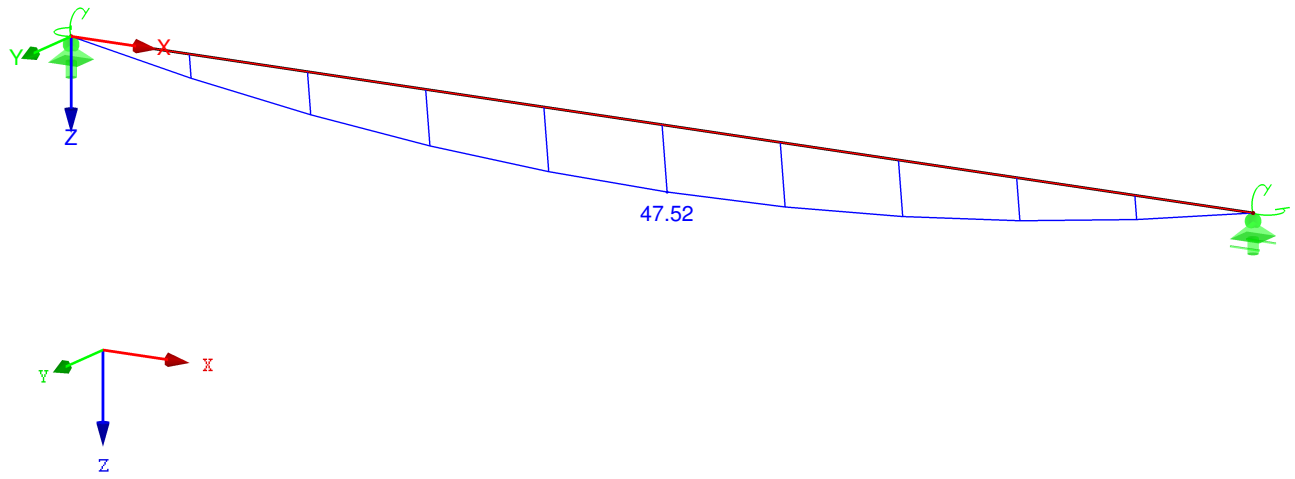
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INTERNAL FORCES M_y

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10a and 6.10b
Internal Forces M-y
Result Combinations: Max and Min Values

Isometric

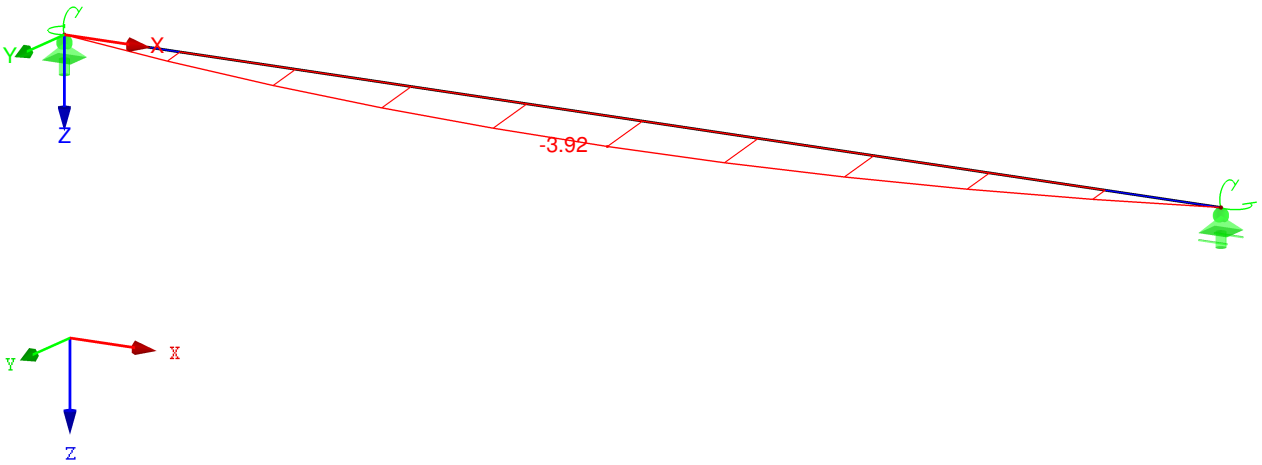


Max M-y: 47.52, Min M-y: 0.00 [kNm]

INTERNAL FORCES M_z

RC1 : ULS (STR/GEO) - Permanent / transient - Eq. 6.10a and 6.10b
Internal Forces M-z
Result Combinations: Max and Min Values

Isometric



Max M-z: 0.00, Min M-z: -3.92 [kNm]

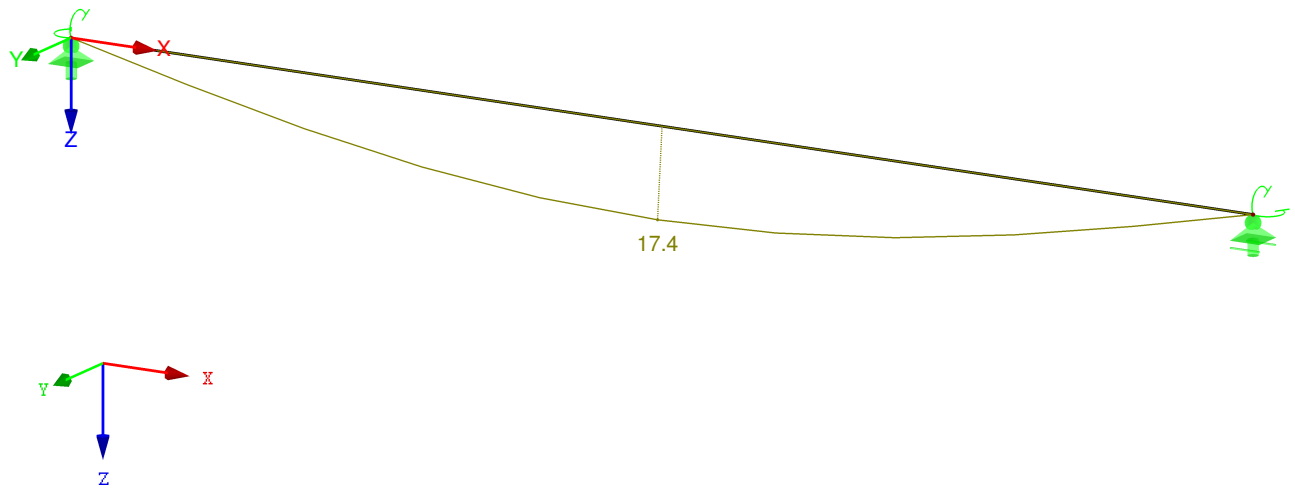
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GLOBAL DEFORMATIONS u

RC2 : SLS - Characteristic
Global Deformations u [mm]
Result Combinations: Max and Min Values

Isometric



Max u: 17.4, Min u: 0.0 [mm]
Factor of deformations: 30.00

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STEEL EC3
CA1
Design ratio according EC3

1.1 GENERAL DATA

Members to design:	All	
Sets of members to design:		
National Annex:	CSN	
Ultimate Limit State Design Result combinations to design:	RC1	ULS (STR/GEO) - Permanent / transient - Eq. 6.10a and 6.10b
Serviceability Limit State Design Result combinations to design:	RC2	SLS - Characteristic
Fire Resistance Design Result combinations to design:	RC3	ULS (EQU) - Accidental - psi-1,1 - thermal design

1.2 MATERIALS

Matl. No.	Material Description	E- Modulus E [kN/cm²]	Shear Modulus G [kN/cm²]	Poisson's Ratio ν [-]	Yield Stress f_{yk} [kN/cm²]	Max. Thickness t [mm]
1	Steel S 355 | DIN EN 1993-1-1:2010-12	21000.00	8076.92	0.300	35.50	40.0
					33.50	80.0
					31.50	100.0
					29.50	150.0
					28.50	200.0
					27.50	250.0

1.5 EFFECTIVE LENGTHS - MEMBERS

Member No.	Buckling Possible	Buckling About Axis y			Buckling About Axis z			Lateral-Torsional Buckling				
		Possible	$k_{cr,y}$	$L_{cr,y}$ [m]	Possible	$k_{cr,z}$	$L_{cr,z}$ [m]	Possible	k_z	k_w	L_w [m]	L_T [m]
1	☒	☒	1.00	7.500	☒	1.00	7.500	☒	1.0	1.0	7.500	7.500

1.9 SERVICEABILITY DATA

No.	Reference to	Members/Sets No.	Reference Length Manually	Reference Length l [m]	Direct.	Precamber e_0 [mm]	Beam Type
1	Member	1	☐	7.500	y, z	10.0	Beam

1.10 FIRE RESISTANCE - MEMBERS

No.	Members N	Req. Time $t_{fi,req}$ [min]	Fire Exposure	Fire Protection	Protection Type	Unit Mass ρ_p [kg/m³]	Thermal Cond. λ_p [W/m*K]	Specific Heat c_p [J/(kg*K)]	Thickness d_p [mm]
1	1	15	All Sides	☐					

1.12 PARAMETERS - MEMBERS

Member No.	Description	Parameter
1	Cross-Section	1 - HEA 220
	Shear panel	☐
	Rotational restraint	☐
	Cross-sectional area for tension design	☐

2.4 DESIGN BY MEMBER

Member No.	Location x [m]	LC/CO/RC	Design		Equation No.	Description
1	**Cross-section No. 1 - HEA 220**					
	0.000	RC1	0.06	≤ 1	CS121)	Cross-section check - Shear force in z-axis acc. to 6.2.6
	0.000	RC1	0.00	≤ 1	CS123)	Cross-section check - Shear force in y-axis acc. to 6.2.6
	0.000	RC1	0.00	≤ 1	CS126)	Cross-section check - Shear buckling acc. to 6.2.6(6)
	3.750	RC1	0.09	≤ 1	CS161)	Cross-section check - Biaxial bending and shear force acc. to 6.2.6, 6.2.7 and 6.2.9
	0.750	RC1	0.44	≤ 1	ST363)	Stability analysis - Biaxial bending acc. to 6.3.3, Method 2
	0.000	RC2	0.00	≤ 1	SE400)	Serviceability - Negligible deformations
	3.750	RC2	0.19	≤ 1	SE403)	Serviceability - Combination of actions 'Quasi-permanent' - z-direction
	3.750	RC2	0.10	≤ 1	SE408)	Serviceability - Combination of actions 'Quasi-permanent' - y-direction
	0.000	RC3	0.06	≤ 1	FC621)	Fire design - Cross-section check - Shear force in z-axis acc. to EN 1993-1-2, 4.2.3.3
	0.750	RC3	0.05	≤ 1	FC622)	Fire design - Cross-section check - Shear force in z-axis acc. to EN 1993-1-2, 4.2.3.4 - Class 3 or 4
	3.750	RC3	0.32	≤ 1	FC662)	Fire design - Cross-section check - Biaxial bending and shear force acc. to EN 1993-1-2, 4.2.3.4 - Class 3
	0.750	RC3	0.70	≤ 1	FS832)	Fire design - Stability analysis - Lateral torsional buckling acc. to EN 1993-1-2, 4.2.3.4
	3.750	RC3	0.91	≤ 1	FS853)	Fire design - Stability analysis - Biaxial bending acc. to EN 1993-1-2, 4.2.3.5

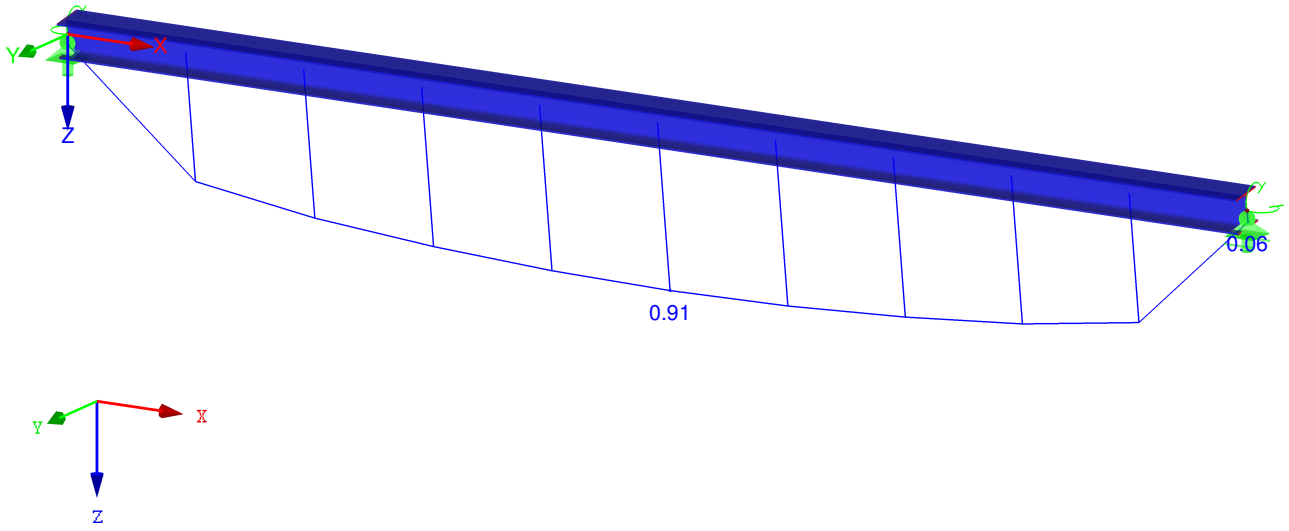
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DESIGN RATIO

STEEL EC3 CA1
Ultimate Limit State: Cross-Section Design, Stability Design, Weld Design, Pressure Design, Plastic Design
Serviceability Limit State: Deformations, Web Breathing
Fire Resistance: Cross-Section Design, Stability Design

Isometric



Max Design Ratio: 0.91

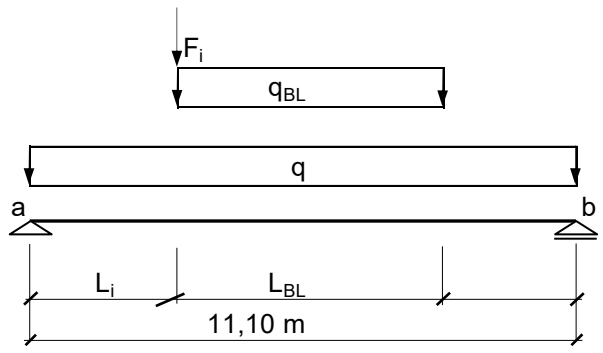
Chapter 4: Ceiling beams

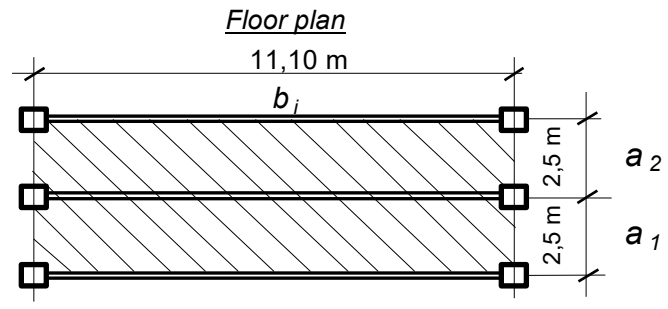
Pos. BE001 ff.

Pos BE034 Composite beam Individuell

Version: 16

SYSTEM





Width carried: 2,5 m -
Influence width: 5,0 m
Influence area: 55,5 m²

Influence area according to DBV booklet 36

$A_{EF} = b_i * (a_1 + a_2)$

LOADS

Partial safety factors for the loads γ_G = 1,35 γ_F = 1,5

Area loads

$g_{0,k}$ - permanent loads (characteristic)

Concrete slab	0,10 m *	25 kN/m³	* 2,5 m =	6,4 kN/m	-
Self weight of beam		-	-	0,6 kN/m	
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m	-
			Σ g_0 =	7,0 kN/m	

$g_{1,k}$ - permanent loads from finishing (characteristic)

Finishing, installation		0,15 kN/m²	* 2,5 m =	0,4 kN/m	-
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m	-
			Σ g_1 =	0,4 kN/m	

$p_{,k}$ - variable loads (characteristic)

Live load		2,50 kN/m²	* 2,5 m =	6,3 kN/m	-
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m	-
-			Σ p =	6,3 kN/m	

Point and uniformly distributed loads (characteristic)

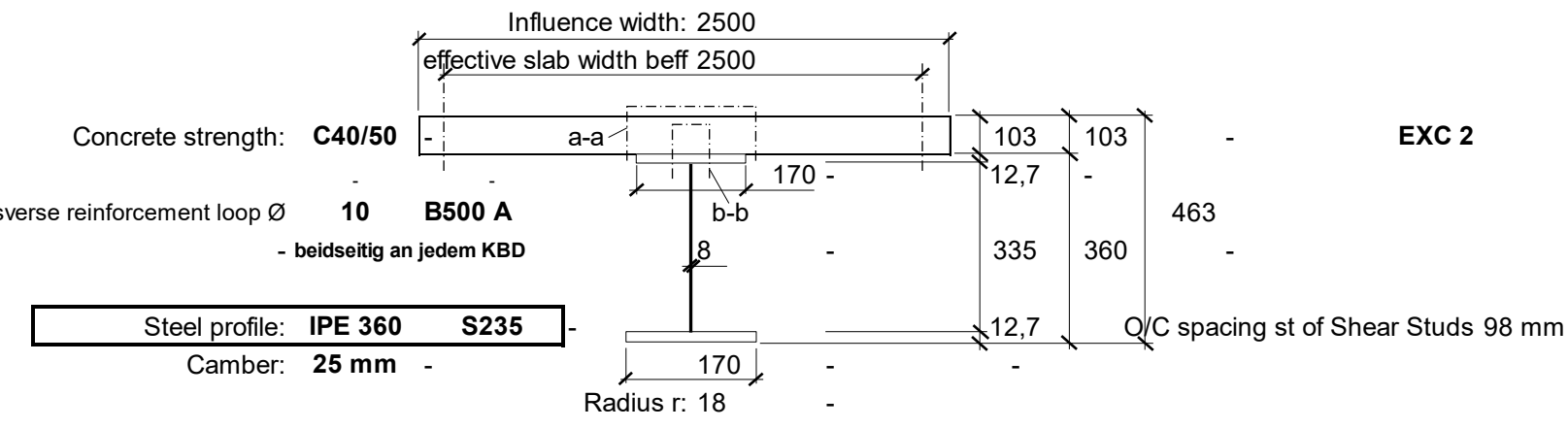
	$g_{0,k}$	$g_{1,k}$	$p_{,k}$	L_i	L_{BL}
F_1	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_2	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_3					-
F_4					-
F_5					-
q_{BL}					

SUPPORT REACTIONS

characteristic **($\gamma_{G,F}$ = 1,0):**

$A_{g0,k}$ =	38,9 kN			$B_{g0,k}$ =	38,9 kN		
$A_{g1,k}$ =	2,1 kN	$A_{g,k}$ =	**41,0 kN**	$B_{g1,k}$ =	2,1 kN	$B_{g,k}$ =	**41,0 kN**
		$A_{p,k}$ =	**34,7 kN**			$B_{p,k}$ =	**34,7 kN**
Design loads:		A_{Ed} =	**107,4 kN**			B_{Ed} =	**107,4 kN**

CROSS-SECTION



Concrete strength: **C40/50** - **EXC 2**

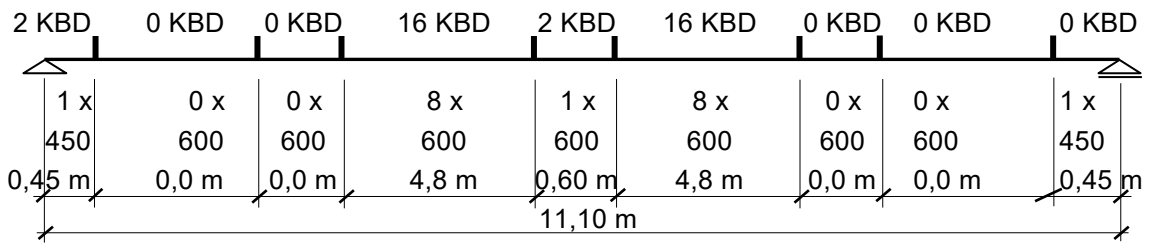
Transverse reinforcement loop Ø **10** **B500 A**
- beidseitig an jedem KBD

Steel profile: **IPE 360** **S235** -
Camber: **25 mm** -
Radius r: 18 -

O/C spacing st of Shear Studs 98 mm

SHEAR STUD ARRANGEMENT

Shear Studs: **doppelreihig** **double-row** ∅ **19** Height: **76** Total number: 36 KBD



Pos BE034

INTERNAL DESIGN FORCES

Final state

max M_{Ed} (γ_G * (M_{g0} + M_{g1}) + γ_F * M_p) = **297,9 kNm** at x = 5,5 m

max V_{Ed} (γ_G * (V_{g0} + V_{g1}) + γ_F * V_p) = **107,4 kN**

Construction state

max M_{Ed} (γ_G * M_{g0}) = 145,7 kNm

MATERIAL PROPERTIES

for design

Concrete			Reinforcement			Steel			Shear studs			
γ_C =	1,5		γ_S =	1,15		γ_{M0} =	1,00		γ_{Va} =	1,25		
α_{cc} =	0,85								γ_{Vc} =	1,50		
f_{ck} =	40	N/mm²	f_{yk} =	500	N/mm²	f_{yk} =	235	N/mm²	α =	1,00		
f_{cd} =	23	N/mm²	f_{yd} =	435	N/mm²	f_{yd} =	235	N/mm²	f_u =	470	N/mm²	-
f_{ctm} =	3,5	N/mm²							P_{Rd} =	75,0	kN	
E_{cm} =	35000	N/mm²										
ν =	0,675											

SECTION PROPERTIES

for design

Concrete		Steel		Composite cross section	
A_c = 2575,0 cm²	N_{cd} = 5836,7 kN	A_a = 72,7 cm²	$N_{pl,a,Rd}$ = 1709,1 kN	z_{pl} = 3,0 cm	--> neutral line in concrete slab
		A_{aS} = 32,6 cm²	$V_{pl,a,Rd}$ = 442,3 kN	$M_{pl,Rd}$ = 457,9 kNm	
		$z_{el,a}$ = 18,0 cm	$z_{el',a}$ = 18,0 cm		
		$z_{pl,a}$ = 18,0 cm	$M_{pl,a,Rd}$ = 239,7 kNm		

DESIGN

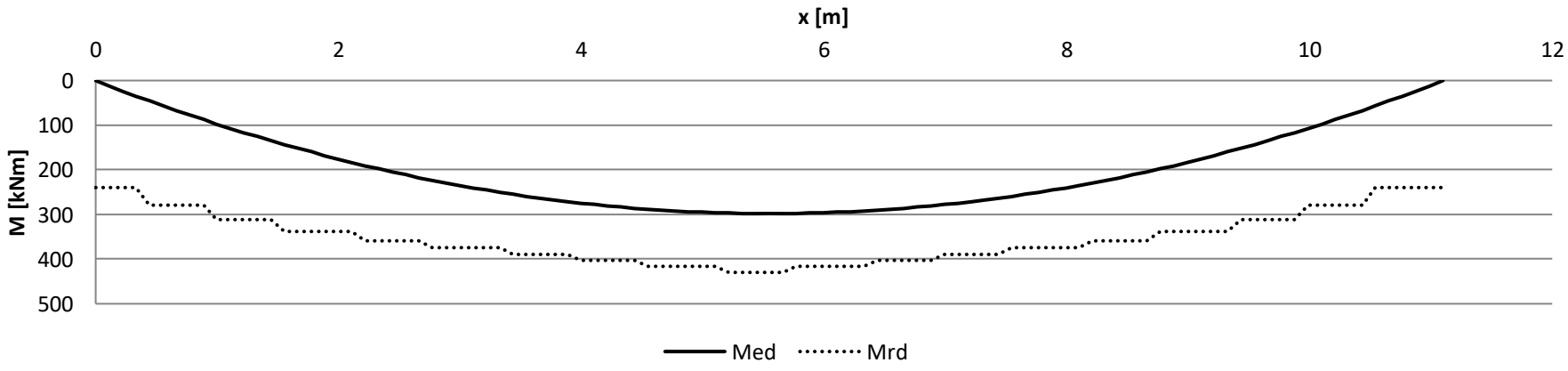
Cross-section classification $\varepsilon = \sqrt{(235/f_{yk})}$ = 1,00

Shear force resistance

Web: vorh h_w/t = 37,3 < 72*ε = 72,0 --> no shear buckling analysis necessary

NW: $V_{Ed}/V_{pl,a,Rd}$ = 0,24 <1; <0,5 → ρ= 0,00 --> shear force without influence on bending

Moment resistance (plastic with partial anchoring along the beam axis)



NW: max (M_{Ed}/M_{Rd}) = 0,71 < 1 at x = 6,5 m

Application limits for: anchoring partially; Shear stud distribution equidistant

Cross-section class 1 or 2: **fulfilled**

Verification of ductility according to DIN-EN 1994-1-1 (possible according to 6.6.1.1 or 6.6.1.2, one criterion must be fulfilled)

According to 6.6.1.2:

Minimum degree of shear connec: min η = 0,40 < vorh η = 0,79 **fulfilled**

According to 6.6.1.1:

Ductility: Deformation capacity according to separate verification > 6.0 mm **fulfilled**

∅ KBD: 16 mm < 19 mm < 25 mm **fulfilled**

$h_{KBD}/\varnothing_{KBD} \geq 4$: **fulfilled**

Longitudinal shear resistance (Ansatz θ_f= 21,8°) mit : σ_{cd} = 0 N/mm² $V_{Rd,cc}$ = 938,4 kN [=$N_{pl,a,Rd}$] V_{Ed} = 1709 kN cotθ= 2,7

min e_{KBD}= Δx = 0,60 m ΔF_d = 150,0 kN $h_{f,a-a}$ = 206 mm --> $\nu_{Ed,a-a}$ = 1,21 N/mm²

$h_{f,b-b}$ = 269 mm --> $\nu_{Ed,b-b}$ = 0,93 N/mm²

Tension strut s_f = 0,60 m A_{sf} = 3,14 cm² $\rho_w = A_{sf}/(s_f * h_f)$ $\nu_{Rd} = (A_{sf} * f_{yd}/s_f) * \cot\theta_f/h_f$

Section a-a: $\rho_{w,a-a}$ = 0,003 --> $\nu_{Rd,a-a}$ = 2,76 N/mm²

Section b-b: $\rho_{w,b-b}$ = 0,002 --> $\nu_{Rd,b-b}$ = 2,12 N/mm²

$\nu_{Rd} = \nu * f_{cd} * \sin\theta_f * \cos\theta_f$ = 5,28 N/mm²

NW: ν_{Ed}/ν_{Rd} = 0,44 < 1

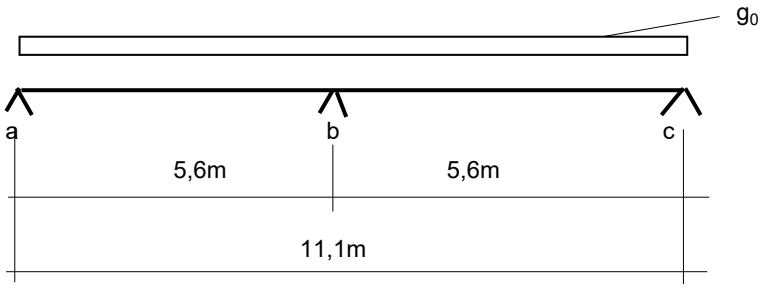
Minimum reinforcement $\rho_{w,min}$ = **0,001** < **0,002** = ρ_w

DEFORMATION CALCULATION WITH AN AUXILIARY SUPPORT

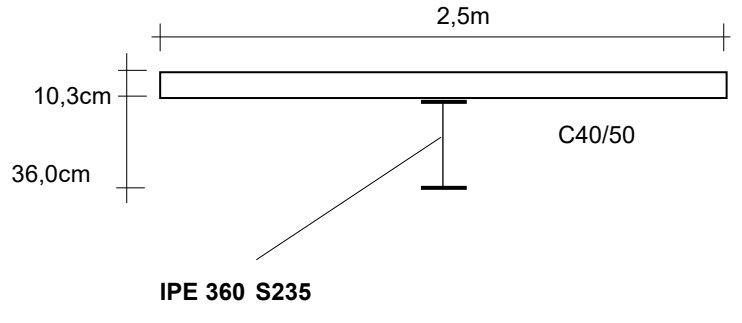
Pos BE034

During the construction phase (placement of the ceiling slabs, pouring of the joint), the beam receives assembly support.
This support is removed after the grout has set. This means that the beam,
spanning over 11,1m , also acts as a composite beam for its self-weight.

The system during assembly:



cross-section:



IPE 360 S235

load:

dead load:	g_0 =	7,0kN/m				
finishing loa	g_1 =	0,4kN/m				
live load:	p =	6,3kN/m	permanently from it:	40%	->p_s =	2,5kN/m
			variable from it:	60%	->p_v =	3,8kN/m

cross-sectional values:

$\varphi\ (t=\infty,t_0)$ = 1,96

$\varepsilon_{cs,\ unendl.}$ = 0,0052%

		t_0	$t_{unendl.}$	
			creep	shrinkage
A_a	[cm²]	72,7		
I_a	[cm⁴]	16298		
E	[N/mm²]	21000	35220,46	
ψ_P			1,10	
ψ_{PT}			0,55	
ψ_S				0,55
n_0		5,96		
n_P			18,82	
n_{PT}			12,39	
n_S				12,39
A_c	[cm²]	431,87	136,85	207,84
I_c	[cm⁴]	3818,09	1209,84	1837,45
A_i	[cm²]	504,60	209,58	280,57
I_i	[cm⁴]	53475,7	41958,81	45452,27

deformations / support forces: (deflections at the center of the beam)

A = **14,6kN** B = **48,6kN** C = **14,6kN**

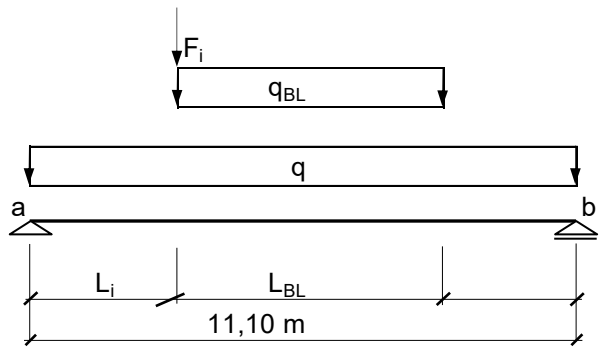
steel beam deformation		0,0cm
composite beam deformation due to	g_0	1,2cm
composite beam deformation due to	g_1+p_s	0,5cm
composite beam deformation due to creep	g_0, g_1, p_s	0,5cm
composite beam deformation due to shrinkage		1,8cm
composite beam deformation due to	p_v	0,5cm
total deformation t= ∞		4,5cm
	overheight	**2,5cm**
total sag		2,0cm = L/548

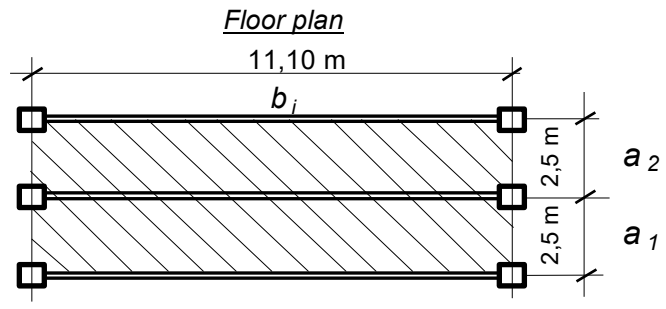
Pos BE034 (Alt.) Composite beam Individuell

*** alternative position ***

Version: 16

SYSTEM





Influence area according to DBV booklet 36

$A_{EF} = b_i * (a_1 + a_2)$

Width carried: 2,5 m -
Influence width: 5,0 m
Influence area: 55,5 m²

LOADS

Partial safety factors for the loads γ_G = 1,35 γ_F = 1,5

Area loads

$g_{0,k}$ - permanent loads (characteristic)

Concrete slab	0,10 m *	25 kN/m³	* 2,5 m =	6,4 kN/m
Self weight of beam		-	-	0,7 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
			Σ g_0 =	7,1 kN/m

$g_{1,k}$ - permanent loads from finishing (characteristic)

Finishing, installation		0,15 kN/m²	* 2,5 m =	0,4 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
			Σ g_1 =	0,4 kN/m

$p_{,k}$ - variable loads (characteristic)

Live load		2,50 kN/m²	* 2,5 m =	6,3 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
-			Σ p =	6,3 kN/m

Point and uniformly distributed loads (characteristic)

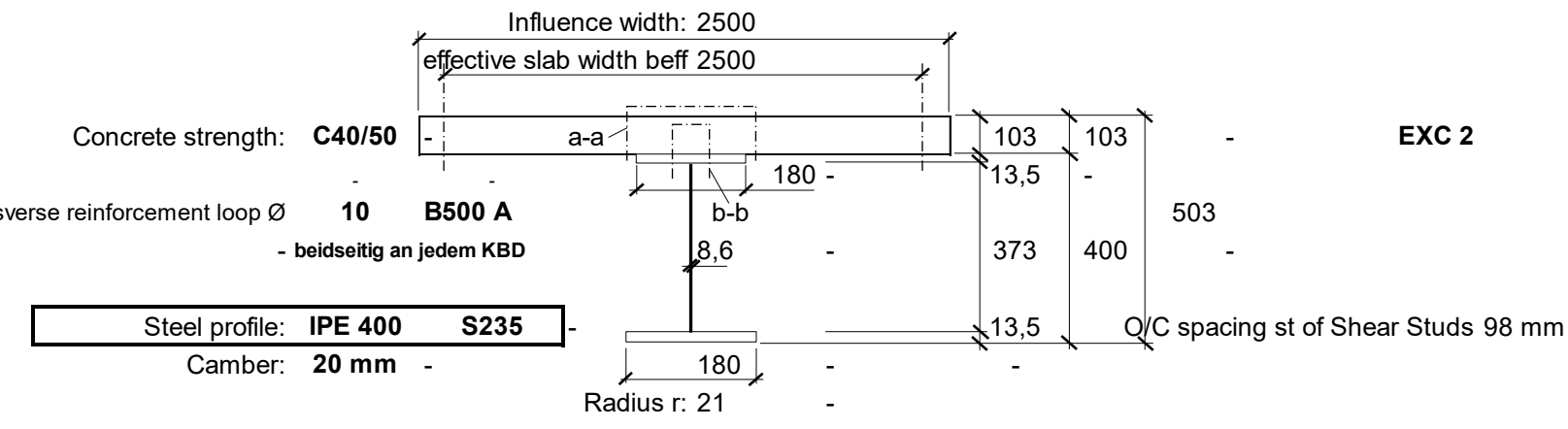
	$g_{0,k}$	$g_{1,k}$	$p_{,k}$	L_i	L_{BL}
F_1	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_2	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_3					-
F_4					-
F_5					-
q_{BL}					

SUPPORT REACTIONS

characteristic **($\gamma_{G,F}$ = 1,0):**

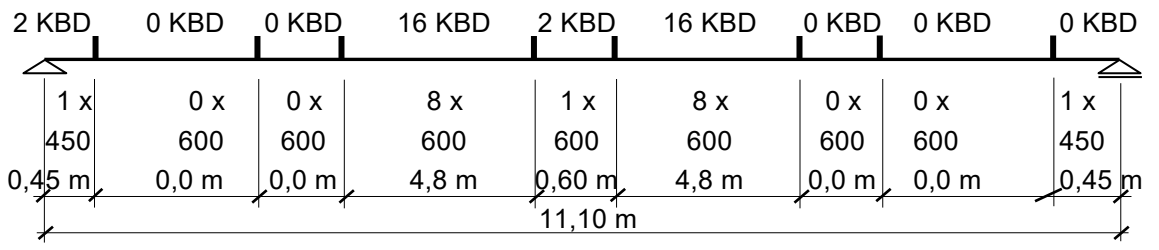
$A_{g0,k}$ =	39,4 kN			$B_{g0,k}$ =	39,4 kN		
$A_{g1,k}$ =	2,1 kN	$A_{g,k}$ =	**41,5 kN**	$B_{g1,k}$ =	2,1 kN	$B_{g,k}$ =	**41,5 kN**
		$A_{p,k}$ =	**34,7 kN**			$B_{p,k}$ =	**34,7 kN**
Design loads:		A_{Ed} =	**108,0 kN**			B_{Ed} =	**108,0 kN**

CROSS-SECTION



SHEAR STUD ARRANGEMENT

Shear Studs: **doppelreihig** **double-row** ∅ **19** Height: **76** Total number: 36 KBD



INTERNAL DESIGN FORCES

Pos BE034 (Alt.)

Final state

max M_{Ed} (γ_G * (M_{g0} + M_{g1}) + γ_F * M_p) = **299,8 kNm** at x = 5,5 m

max V_{Ed} (γ_G * (V_{g0} + V_{g1}) + γ_F * V_p) = **108,0 kN**

Construction state

max M_{Ed} (γ_G * M_{g0}) = 147,6 kNm

MATERIAL PROPERTIES

for design

Concrete			Reinforcement			Steel			Shear studs			
γ_C =	1,5		γ_S =	1,15		γ_{M0} =	1,00		γ_{Va} =	1,25		
α_{cc} =	0,85								γ_{Vc} =	1,50		
f_{ck} =	40	N/mm²	f_{yk} =	500	N/mm²	f_{yk} =	235	N/mm²	α =	1,00		
f_{cd} =	23	N/mm²	f_{yd} =	435	N/mm²	f_{yd} =	235	N/mm²	f_u =	470	N/mm²	-
f_{ctm} =	3,5	N/mm²							P_{Rd} =	75,0	kN	
E_{cm} =	35000	N/mm²										
ν =	0,675											

SECTION PROPERTIES

for design

Concrete			Steel			Composite cross section	
A_c = 2575,0 cm²	N_{cd} = 5836,7 kN		A_a = 84,5 cm²	$N_{pl,a,Rd}$ = 1984,9 kN		z_{pl} = 3,5 cm	--> neutral line in concrete slab
			A_{aS} = 39,3 cm²	$V_{pl,a,Rd}$ = 532,9 kN		$M_{pl,Rd}$ = 566,7 kNm	
			$z_{el,a}$ = 20,0 cm	$z_{el',a}$ = 20,0 cm			
			$z_{pl,a}$ = 20,0 cm	$M_{pl,a,Rd}$ = 307,5 kNm			

DESIGN

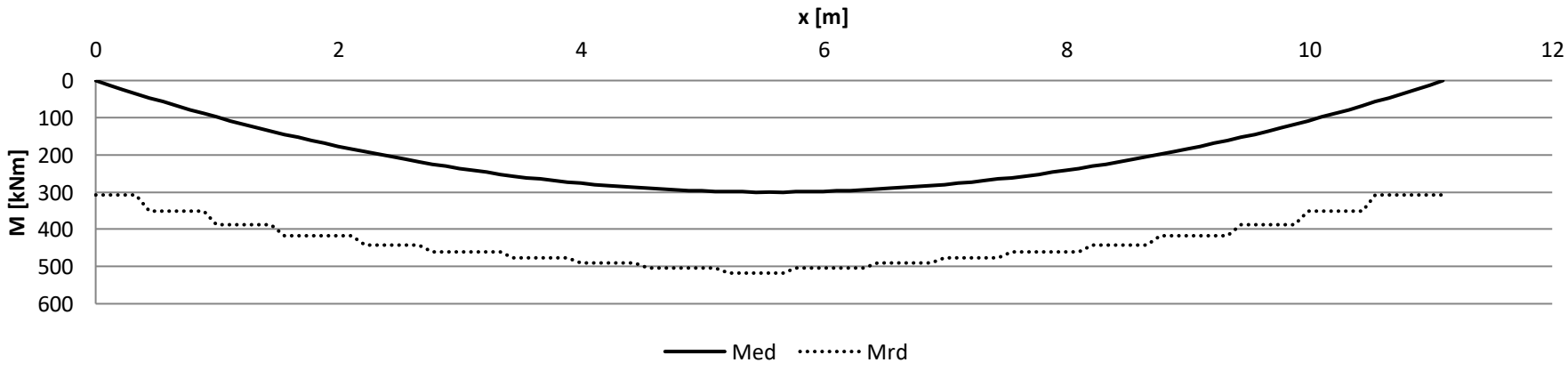
Cross-section classification $\varepsilon = \sqrt{(235/f_{yk})}$ = 1,00

Shear force resistance

Web: vorh h_w/t = 38,5 < 72*ε = 72,0 --> no shear buckling analysis necessary

NW: $V_{Ed}/V_{pl,a,Rd}$ = 0,20 **<1; <0,5 → ρ=** 0,00 --> shear force without influence on bending

Moment resistance (plastic with partial anchoring along the beam axis)



NW: max (M_{Ed}/M_{Rd}) = 0,59 < 1 at x = 5,2 m

Application limits for: anchoring: partially; Shear stud distribution: equidistant

Cross-section class 1 or 2: **fulfilled**

Verification of ductility according to DIN-EN 1994-1-1 (possible according to 6.6.1.1 or 6.6.1.2, one criterion must be fulfilled)

According to 6.6.1.2:

Minimum degree of shear connec: min η = 0,40 < vorh η = 0,68 **fulfilled**

According to 6.6.1.1:

Ductility: Deformation capacity according to separate verification > 6.0 mm **fulfilled**

∅ KBD: 16 mm < 19 mm < 25 mm **fulfilled**

$h_{KBD}/\varnothing_{KBD} \geq 4$: **fulfilled**

Longitudinal shear resistance (Ansatz θ_f= 23,7°) mit : σ_{cd} = 0 N/mm² $V_{Rd,cc}$ = 938,4 kN [=$N_{pl,a,Rd}$] V_{Ed} = 1985 kN cotθ= 2,3

min e_{KBD}= Δx = 0,60 m ΔF_d = 150,0 kN $h_{f,a-a}$ = 206 mm --> $\nu_{Ed,a-a}$ = 1,21 N/mm²

$h_{f,b-b}$ = 269 mm --> $\nu_{Ed,b-b}$ = 0,93 N/mm²

Tension strut s_f = 0,60 m A_{sf} = 3,14 cm² $\rho_w = A_{sf}/(s_f * h_f)$ $\nu_{Rd} = (A_{sf} * f_{yd}/s_f) * \cot\theta_f/h_f$

Section a-a: $\rho_{w,a-a}$ = 0,003 --> $\nu_{Rd,a-a}$ = 2,52 N/mm²

Section b-b: $\rho_{w,b-b}$ = 0,002 --> $\nu_{Rd,b-b}$ = 1,93 N/mm²

$\nu_{Rd} = \nu * f_{cd} * \sin\theta_f * \cos\theta_f$ = 5,63 N/mm²

NW: ν_{Ed}/ν_{Rd} = 0,48 < 1

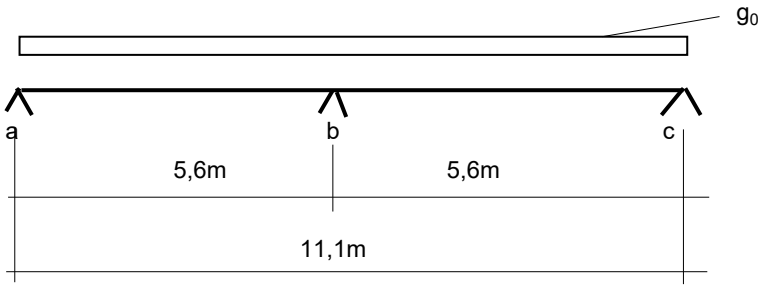
Minimum reinforcement $\rho_{w,min}$ = **0,001** **<** **0,002** = ρ_w

DEFORMATION CALCULATION WITH AN AUXILIARY SUPPORT

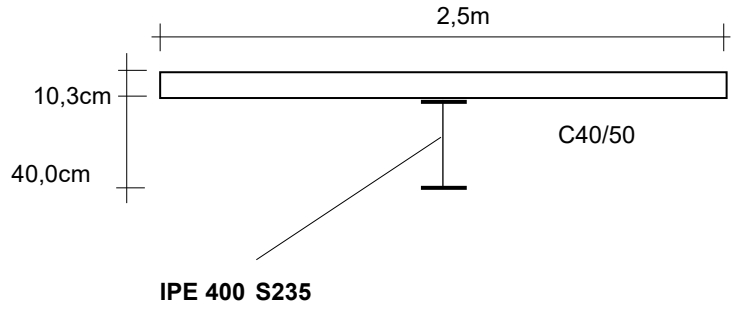
Pos BE034 (Alt.)

During the construction phase (placement of the ceiling slabs, pouring of the joint), the beam receives assembly support.
This support is removed after the grout has set. This means that the beam,
spanning over 11,1m , also acts as a composite beam for its self-weight.

The system during assembly:



cross-section:



IPE 400 S235

load:

dead load:	g_0 =	7,1kN/m				
finishing loa	g_1 =	0,4kN/m				
live load:	p =	6,3kN/m	permanently from it:	40%	->p_s =	2,5kN/m
			variable from it:	60%	->p_v =	3,8kN/m

cross-sectional values:

$\varphi\ (t=\infty,t_0)$ = 1,96

$\varepsilon_{cs,\ unendl.}$ = 0,0052%

		t_0	$t_{unendl.}$	
			creep	shrinkage
A_a	[cm²]	84,5		
I_a	[cm⁴]	23185		
E	[N/mm²]	21000	35220,46	
ψ_P			1,10	
ψ_{PT}			0,55	
ψ_S				0,55
n_0		5,96		
n_P			18,82	
n_{PT}			12,39	
n_S				12,39
A_c	[cm²]	431,87	136,85	207,84
I_c	[cm⁴]	3818,09	1209,84	1837,45
A_i	[cm²]	516,33	221,31	292,30
I_i	[cm⁴]	71689,1	56442,00	61464,98

deformations / support forces: (deflections at the center of the beam)

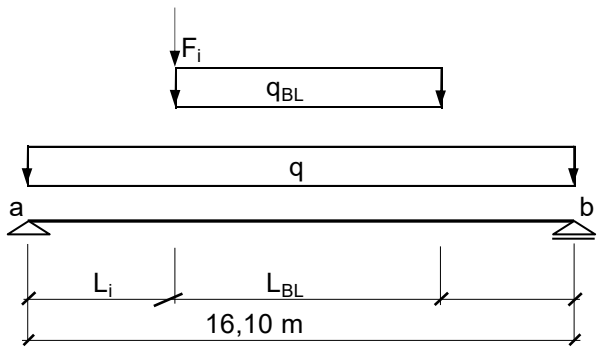
A = **14,8kN** B = **49,3kN** C = **14,8kN**

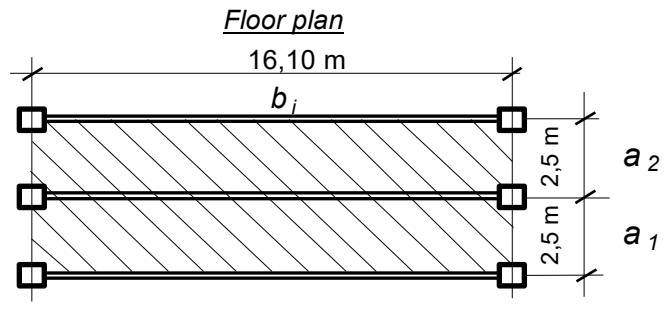
steel beam deformation		0,0cm
composite beam deformation due to	g_0	0,9cm
composite beam deformation due to	g_1+p_s	0,4cm
composite beam deformation due to creep	g_0, g_1, p_s	0,4cm
composite beam deformation due to shrinkage		1,7cm
composite beam deformation due to	p_v	0,3cm
total deformation t= ∞		3,7cm
	overheight	**2,0cm**
total sag		1,7cm = L/668

Pos BE052 Composite beam Individuell

Version: 16

SYSTEM





Width carried: 2,5 m -
Influence width: 5,0 m
Influence area: 80,5 m²

Influence area according to DBV booklet 36

$A_{EF} = b_i * (a_1 + a_2)$

LOADS

Partial safety factors for the loads γ_G = 1,35 γ_F = 1,5

Area loads

$g_{0,k}$ - permanent loads (characteristic)

Concrete slab	0,10 m *	25 kN/m³	* 2,5 m =	6,4 kN/m -
Self weight of beam		-	-	0,7 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m -
			Σ g_0 =	7,1 kN/m

$g_{1,k}$ - permanent loads from finishing (characteristic)

Finishing, installation		0,15 kN/m²	* 2,5 m =	0,4 kN/m -
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m -
			Σ g_1 =	0,4 kN/m

$p_{,k}$ - variable loads (characteristic)

Live load		2,50 kN/m²	* 2,5 m =	6,3 kN/m -
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m -
-			Σ p =	6,3 kN/m

Point and uniformly distributed loads (characteristic)

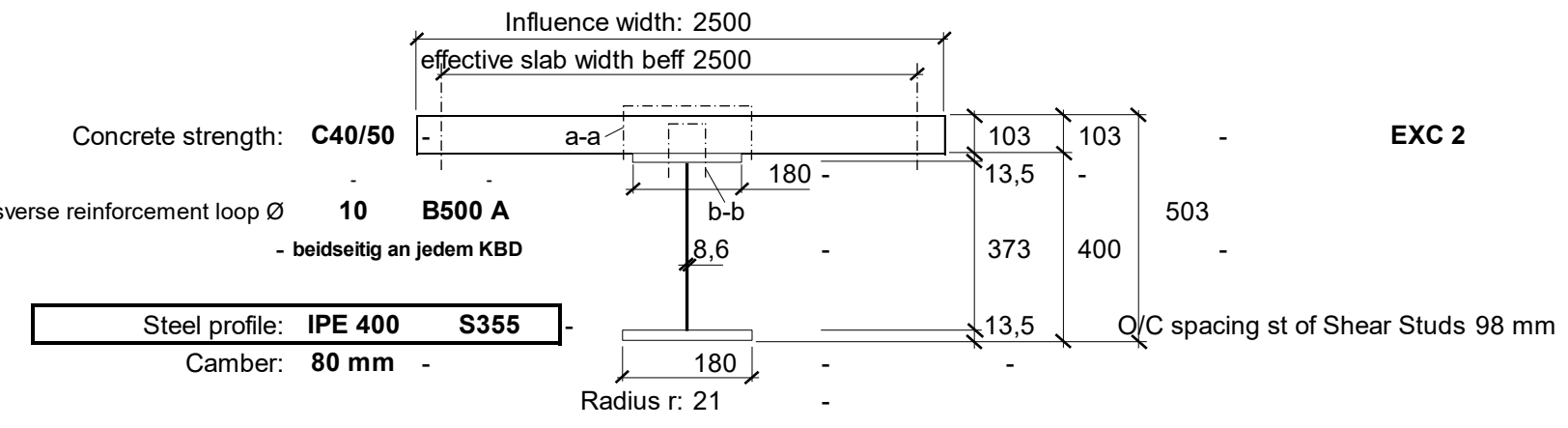
	$g_{0,k}$	$g_{1,k}$	$p_{,k}$	L_i	L_{BL}
F_1	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_2	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_3					-
F_4					-
F_5					-
q_{BL}					

SUPPORT REACTIONS

characteristic **($\gamma_{G,F}$ = 1,0):**

$A_{g0,k}$ =	57,2 kN			$B_{g0,k}$ =	57,2 kN		
$A_{g1,k}$ =	3,0 kN	$A_{g,k}$ =	**60,2 kN**	$B_{g1,k}$ =	3,0 kN	$B_{g,k}$ =	**60,2 kN**
		$A_{p,k}$ =	**50,3 kN**			$B_{p,k}$ =	**50,3 kN**
Design loads:		A_{Ed} =	**156,7 kN**			B_{Ed} =	**156,7 kN**

CROSS-SECTION



Concrete strength: **C40/50** - **EXC 2**

Transverse reinforcement loop Ø **10** **B500 A**

- beidseitig an jedem KBD

Steel profile: **IPE 400** **S355** -

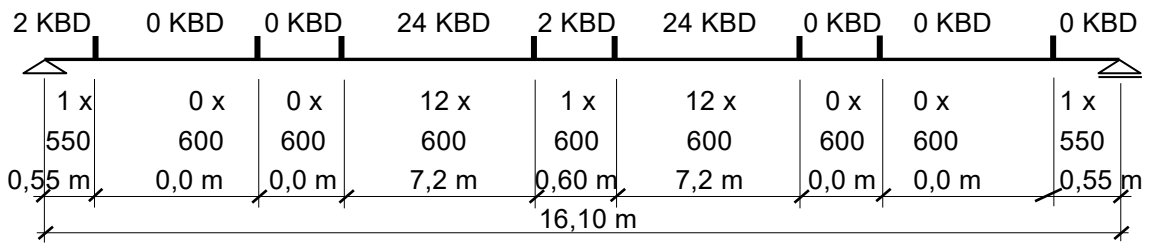
Camber: **80 mm** -

O/C spacing st of Shear Studs 98 mm

Radius r: 21 -

SHEAR STUD ARRANGEMENT

Shear Studs: **doppelreihig** **double-row** ∅ **19** Height: **76** Total number: 52 KBD



INTERNAL DESIGN FORCES

Pos BE052

Final state

max M_{Ed} (γ_G * (M_{g0} + M_{g1}) + γ_F * M_p) = **630,8 kNm** at x = 8,0 m

max V_{Ed} (γ_G * (V_{g0} + V_{g1}) + γ_F * V_p) = **156,7 kN**

Construction state

max M_{Ed} (γ_G * M_{g0}) = 310,6 kNm

MATERIAL PROPERTIES

for design

Concrete			Reinforcement			Steel			Shear studs			
γ_C =	1,5		γ_S =	1,15		γ_{M0} =	1,00		γ_{Va} =	1,25		
α_{cc} =	0,85								γ_{Vc} =	1,50		
f_{ck} =	40	N/mm²	f_{yk} =	500	N/mm²	f_{yk} =	355	N/mm²	α =	1,00		
f_{cd} =	23	N/mm²	f_{yd} =	435	N/mm²	f_{yd} =	355	N/mm²	f_u =	470	N/mm²	-
f_{ctm} =	3,5	N/mm²							P_{Rd} =	75,0	kN	
E_{cm} =	35000	N/mm²										
ν =	0,675											

SECTION PROPERTIES

for design

Concrete		Steel		Composite cross section	
A_c = 2575,0 cm²	N_{cd} = 5836,7 kN	A_a = 84,5 cm²	$N_{pl,a,Rd}$ = 2998,5 kN	z_{pl} = 5,3 cm	--> neutral line in concrete slab
		A_{aS} = 39,3 cm²	$V_{pl,a,Rd}$ = 805,0 kN	$M_{pl,Rd}$ = 829,2 kNm	
		$z_{el,a}$ = 20,0 cm	$z_{el',a}$ = 20,0 cm		
		$z_{pl,a}$ = 20,0 cm	$M_{pl,a,Rd}$ = 464,6 kNm		

DESIGN

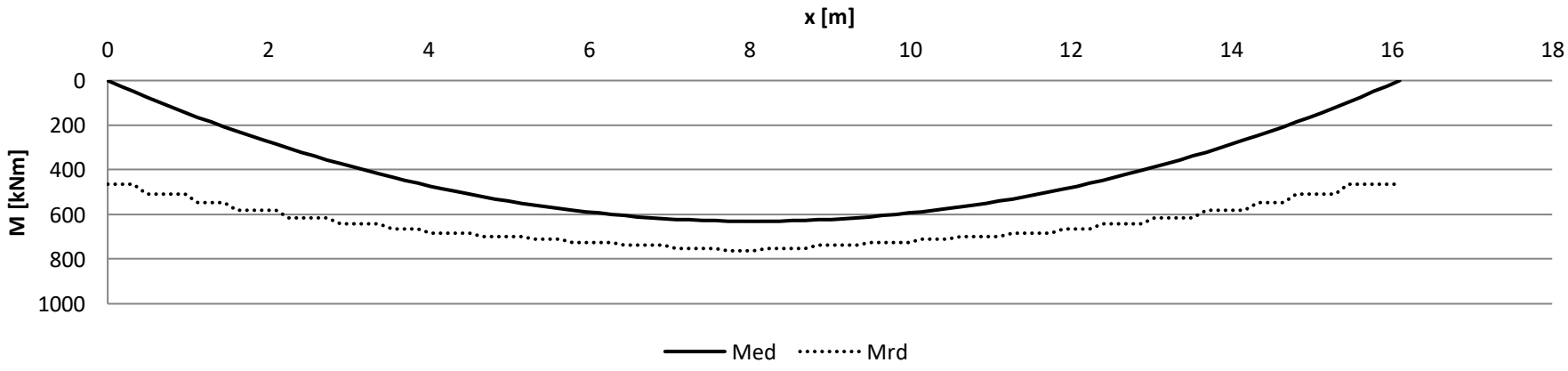
Cross-section classification $\varepsilon = \sqrt{(235/f_{yk})}$ = 0,81

Shear force resistance

Web: vorh h_w/t = 38,5 < 72*ε = 58,6 --> no shear buckling analysis necessary

NW: $V_{Ed}/V_{pl,a,Rd}$ = 0,19 <1; <0,5 → ρ= 0,00 --> shear force without influence on bending

Moment resistance (plastic with partial anchoring along the beam axis)



NW: max (M_{Ed}/M_{Rd}) = 0,84 < 1 at x = 9,0 m

Application limits for	anchoring	Shear stud distribution
	partially	equidistant
Cross-section class 1 or 2:	**fulfilled**	

Verification of ductility according to DIN-EN 1994-1-1 (possible according to 6.6.1.1 or 6.6.1.2, one criterion must be fulfilled)

According to 6.6.1.2:

Minimum degree of shear connec: min η = 0,73 > vorh η = 0,65 **not fulfilled**

According to 6.6.1.1:

Ductility: Deformation capacity according to separate verification > 6.0 mm **fulfilled**

∅ KBD: 16 mm < 19 mm < 25 mm **fulfilled**

$h_{KBD}/\varnothing_{KBD} \geq 4$: **fulfilled**

[=$N_{pl,a,Rd}$]

Longitudinal shear resistance (Ansatz θ_f= 24,5°) mit : σ_{cd} = 0 N/mm² $V_{Rd,cc}$ = 1361,1 kN V_{Ed} = 2998 kN cotθ= 2,2

min e_{KBD}= Δx = 0,60 m ΔF_d = 150,0 kN $h_{f,a-a}$ = 206 mm --> $\nu_{Ed,a-a}$ = 1,21 N/mm²

$h_{f,b-b}$ = 269 mm --> $\nu_{Ed,b-b}$ = 0,93 N/mm²

Tension strut s_f = 0,60 m A_{sf} = 3,14 cm² $\rho_w = A_{sf} / (s_f * h_f)$ $\nu_{Rd} = (A_{sf} * f_{yd}/s_f) * \cot \theta_f/h_f$

Section a-a: $\rho_{w,a-a}$ = 0,003 --> $\nu_{Rd,a-a}$ = 2,43 N/mm²

Section b-b: $\rho_{w,b-b}$ = 0,002 --> $\nu_{Rd,b-b}$ = 1,86 N/mm²

$\nu_{Rd} = \nu * f_{cd} * \sin \theta_f * \cos \theta_f$ = 5,77 N/mm²

NW: ν_{Ed}/ν_{Rd} = 0,50 < 1

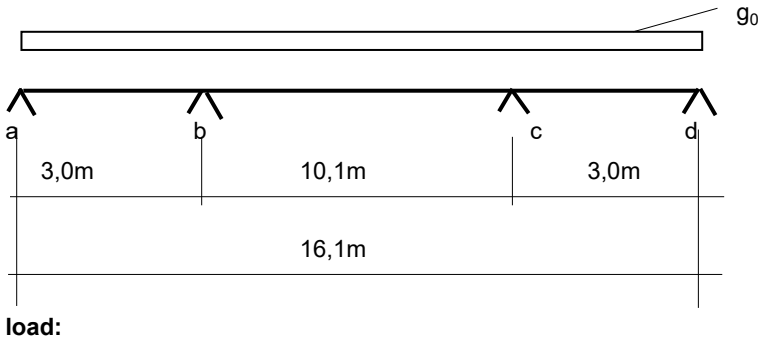
Minimum reinforcement $\rho_{w,min}$ = **0,001 < 0,002** = ρ_w

DEFORMATION CALCULATION WITH TWO AUXILIARY SUPPORT

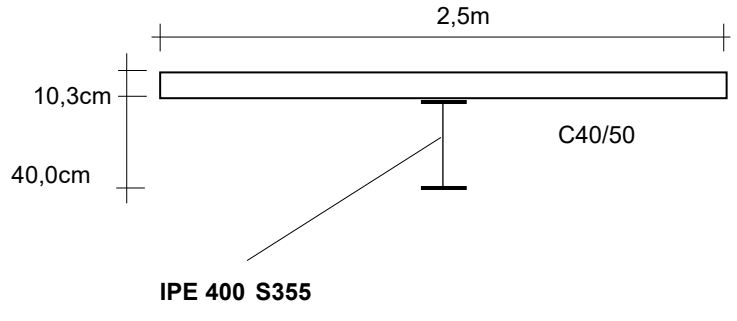
Pos BE052

During the construction phase (placement of the ceiling slabs, pouring of the joint), the beam receives assembly support.
This support is removed after the grout has set. This means that the beam,
spanning over 16,1m , also acts as a composite beam for its self-weight.

The system during assembly:



cross-section:



load:

dead load: g_0 = 7,1kN/m
finishing loa g_1 = 0,4kN/m
live load: p = 6,3kN/m permanently from it: 40% ->p_s = 2,5kN/m
variable from it: 60% ->p_v = 3,8kN/m

cross-sectional values:

$\varphi\ (t=\infty,t_0)$ = 1,96
$\varepsilon_{cs,\ unendl.}$ = 0,0052%

		t_0	$t_{unendl.}$	
			creep	shrinkage
A_a	[cm²]	84,5 cm²		
I_a	[cm^4]	23185,402		
E	[N/mm²]	21000	35220,46	
ψ_P			1,10	
ψ_{PT}			0,55	
ψ_S				0,55
n_0		5,96		
n_P			18,82	
n_{PT}			12,39	
n_S				12,39
A_c	[cm²]	431,87	136,85	207,84
I_c	[cm^4]	3818,1	1209,84	1837,45
A_i	[cm²]	516,33	221,31	292,30
I_i	[cm^4]	71689	56442,00	61464,98

deformations / support forces: (deflections at the center of the beam)

A = **-6,6kN** B = **63,7kN** C = **63,7kN** D = **-6,6kN**

steel beam deformation		0,6cm
composite beam deformation due to	g_0	3,9cm
composite beam deformation due to	g_1+p_s	1,7cm
composite beam deformation due to creep	g_0, g_1, p_s	1,5cm
composite beam deformation due to shrinkage		3,5cm
composite beam deformation due to	p_v	1,5cm
total deformation t= ∞		12,7cm

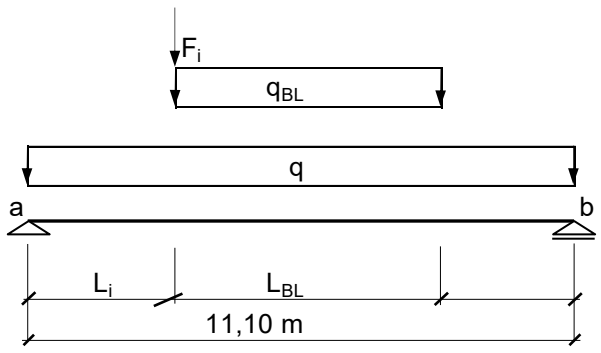
overheight **8,0cm**

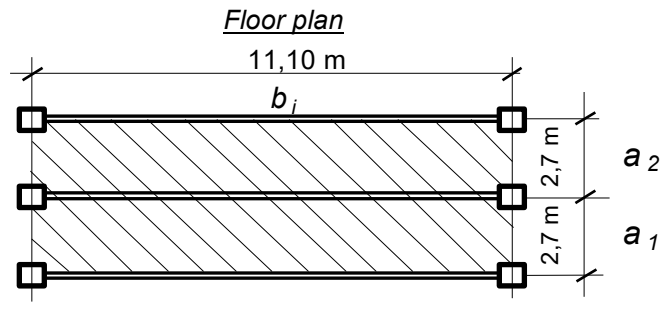
total sag 4,7cm = L/340

Pos BE134 Composite beam Individuell

Version: 16

SYSTEM





Width carried: 2,7 m -
Influence width: 5,4 m
Influence area: 59,9 m²

Influence area according to DBV booklet 36

$A_{EF} = b_i * (a_1 + a_2)$

LOADS

Partial safety factors for the loads γ_G = 1,35, γ_F = 1,5

Area loads

$g_{0,k}$ - permanent loads (characteristic)

Concrete slab	0,10 m *	25 kN/m³	* 2,7 m =	7,0 kN/m -
Self weight of beam		-	-	0,6 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m -
			Σ g_0 =	7,5 kN/m

$g_{1,k}$ - permanent loads from finishing (characteristic)

Finishing, installation		0,15 kN/m²	* 2,7 m =	0,4 kN/m -
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m -
			Σ g_1 =	0,4 kN/m

$p_{,k}$ - variable loads (characteristic)

Live load		2,50 kN/m²	* 2,7 m =	6,8 kN/m -
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m -
-			Σ p =	6,8 kN/m

Point and uniformly distributed loads (characteristic)

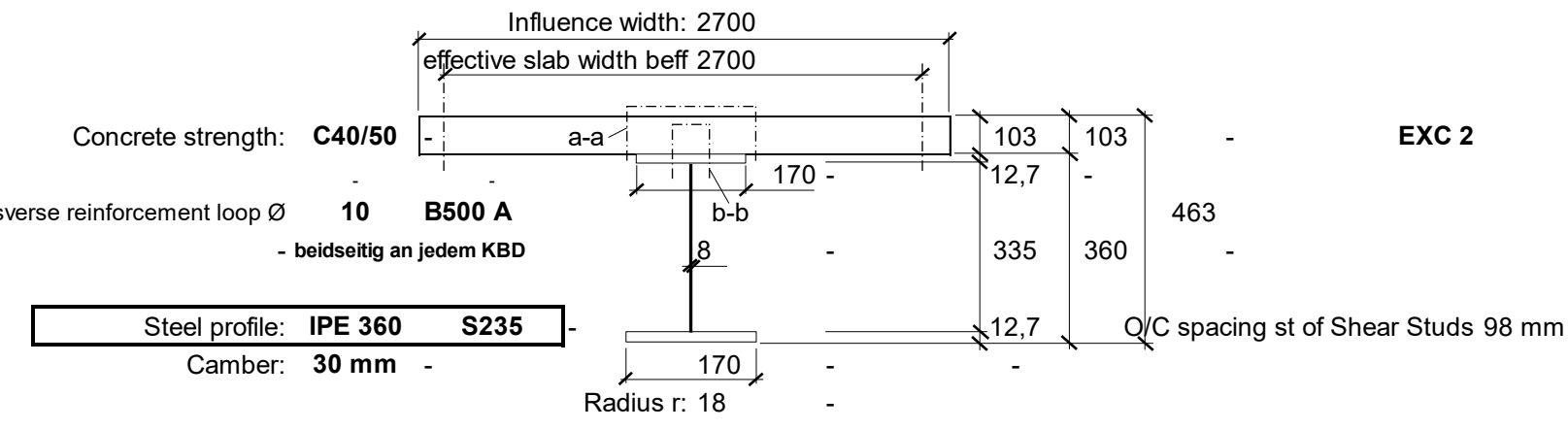
	$g_{0,k}$	$g_{1,k}$	$p_{,k}$	L_i	L_{BL}
F_1	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_2	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_3					-
F_4					-
F_5					-
q_{BL}					

SUPPORT REACTIONS

characteristic **($\gamma_{G,F}$ = 1,0):**

$A_{g0,k}$ =	41,8 kN			$B_{g0,k}$ =	41,8 kN		
$A_{g1,k}$ =	2,2 kN	$A_{g,k}$ =	**44,0 kN**	$B_{g1,k}$ =	2,2 kN	$B_{g,k}$ =	**44,0 kN**
		$A_{p,k}$ =	**37,5 kN**			$B_{p,k}$ =	**37,5 kN**
Design loads:		A_{Ed} =	**115,6 kN**			B_{Ed} =	**115,6 kN**

CROSS-SECTION



Concrete strength: **C40/50** - - -

Transverse reinforcement loop Ø **10** **B500 A**

- beidseitig an jedem KBD

Steel profile: **IPE 360** **S235** -

Camber: **30 mm** -

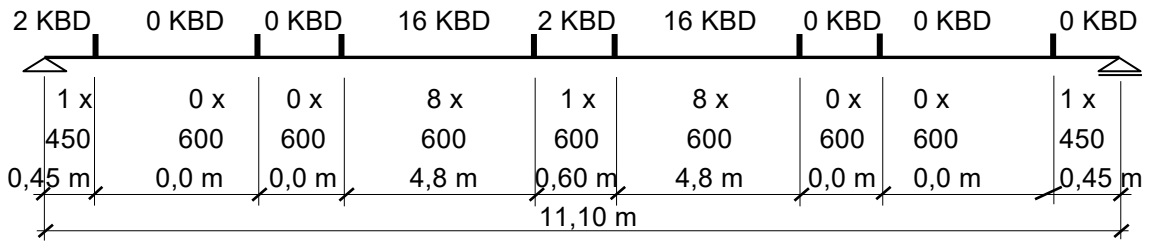
Radius r: 18 -

EXC 2

O/C spacing st of Shear Studs 98 mm

SHEAR STUD ARRANGEMENT

Shear Studs: **doppelreihig** **double-row** ∅ **19** Height: **76** Total number: 36 KBD



INTERNAL DESIGN FORCES

Pos BE134

Final state

max M_{Ed} (γ_G * (M_{g0} + M_{g1}) + γ_F * M_p) = **320,8 kNm** at x = 5,5 m

max V_{Ed} (γ_G * (V_{g0} + V_{g1}) + γ_F * V_p) = **115,6 kN**

Construction state

max M_{Ed} (γ_G * M_{g0}) = 156,4 kNm

MATERIAL PROPERTIES

for design

Concrete			Reinforcement			Steel			Shear studs			
γ_C =	1,5		γ_S =	1,15		γ_{M0} =	1,00		γ_{Va} =	1,25		
α_{cc} =	0,85								γ_{Vc} =	1,50		
f_{ck} =	40	N/mm²	f_{yk} =	500	N/mm²	f_{yk} =	235	N/mm²	α =	1,00		
f_{cd} =	23	N/mm²	f_{yd} =	435	N/mm²	f_{yd} =	235	N/mm²	f_u =	470	N/mm²	-
f_{ctm} =	3,5	N/mm²							P_{Rd} =	75,0	kN	
E_{cm} =	35000	N/mm²										
ν =	0,675											

SECTION PROPERTIES

for design

Concrete		Steel		Composite cross section	
A_c = 2781,0 cm²	N_{cd} = 6303,6 kN	A_a = 72,7 cm²	$N_{pl,a,Rd}$ = 1709,1 kN	z_{pl} = 2,8 cm	--> neutral line in concrete slab
		A_{aS} = 32,6 cm²	$V_{pl,a,Rd}$ = 442,3 kN	$M_{pl,Rd}$ = 459,8 kNm	
		$z_{el,a}$ = 18,0 cm	$z_{el',a}$ = 18,0 cm		
		$z_{pl,a}$ = 18,0 cm	$M_{pl,a,Rd}$ = 239,7 kNm		

DESIGN

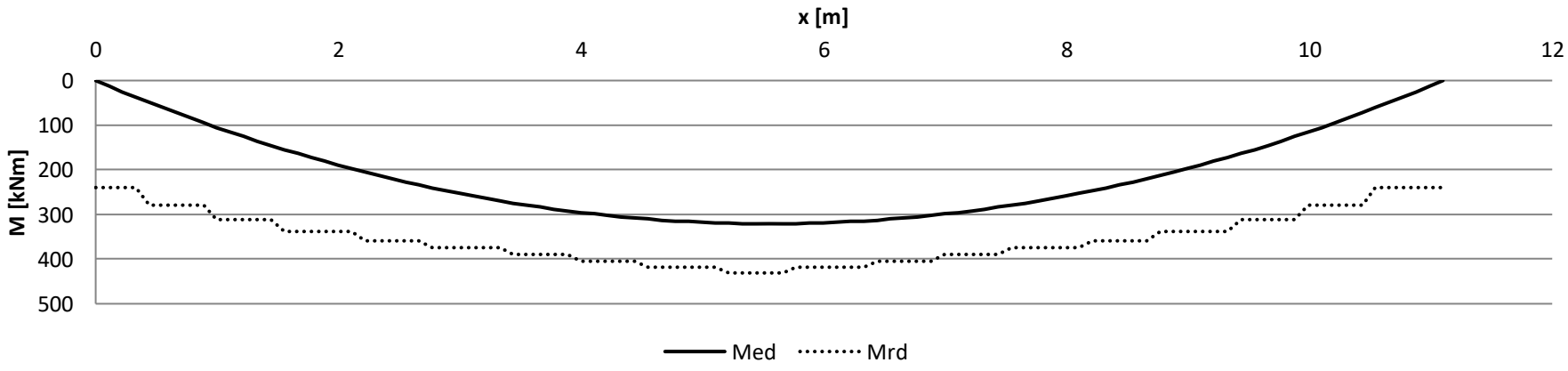
Cross-section classification $\varepsilon = \sqrt{(235/f_{yk})}$ = 1,00

Shear force resistance

Web: vorh h_w/t = 37,3 < 72*ε = 72,0 --> no shear buckling analysis necessary

NW: $V_{Ed}/V_{pl,a,Rd}$ = 0,26 <1; <0,5 → ρ= 0,00 --> shear force without influence on bending

Moment resistance (plastic with partial anchoring along the beam axis)



NW: max (M_{Ed}/M_{Rd}) = 0,77 < 1 at x = 6,5 m

Application limits for	anchoring	Shear stud distribution
	partially	equidistant
Cross-section class 1 or 2:	**fulfilled**	

Verification of ductility according to DIN-EN 1994-1-1 (possible according to 6.6.1.1 or 6.6.1.2, one criterion must be fulfilled)

According to 6.6.1.2:

Minimum degree of shear connec: min η = 0,40 < vorh η = 0,79 **fulfilled**

According to 6.6.1.1:

Ductility: Deformation capacity according to separate verification > 6.0 mm **fulfilled**

∅ KBD: 16 mm < 19 mm < 25 mm **fulfilled**

$h_{KBD}/\varnothing_{KBD} \geq 4$: **fulfilled**

[=$N_{pl,a,Rd}$]

Longitudinal shear resistance (Ansatz θ_f= 21,8°) mit : σ_{cd} = 0 N/mm² $V_{Rd,cc}$ = 938,4 kN V_{Ed} = 1709 kN cotθ= 2,7

min e_{KBD}= Δx = 0,60 m ΔF_d = 150,0 kN $h_{f,a-a}$ = 206 mm --> $\nu_{Ed,a-a}$ = 1,21 N/mm²

$h_{f,b-b}$ = 269 mm --> $\nu_{Ed,b-b}$ = 0,93 N/mm²

Tension strut s_f = 0,60 m A_{sf} = 3,14 cm² $\rho_w = A_{sf} / (s_f * h_f)$ $\nu_{Rd} = (A_{sf} * f_{yd}/s_f) * \cot \theta_f/h_f$

Section a-a: $\rho_{w,a-a}$ = 0,003 --> $\nu_{Rd,a-a}$ = 2,76 N/mm²

Section b-b: $\rho_{w,b-b}$ = 0,002 --> $\nu_{Rd,b-b}$ = 2,12 N/mm²

$\nu_{Rd} = \nu * f_{cd} * \sin \theta_f * \cos \theta_f$ = 5,28 N/mm²

NW: ν_{Ed}/ν_{Rd} = 0,44 < 1

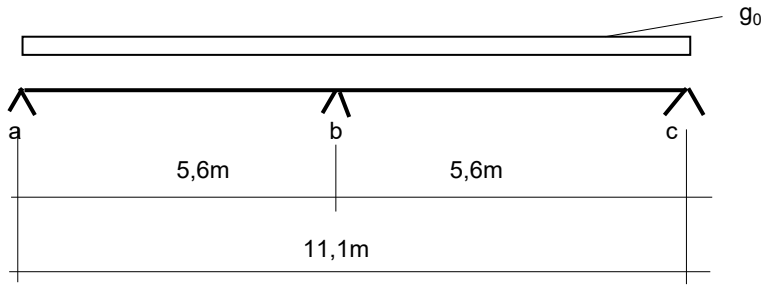
Minimum reinforcement $\rho_{w,min}$ = **0,001 < 0,002** = ρ_w

DEFORMATION CALCULATION WITH AN AUXILIARY SUPPORT

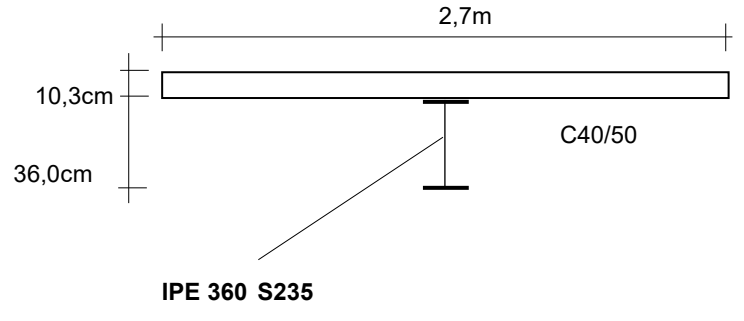
Pos BE134

During the construction phase (placement of the ceiling slabs, pouring of the joint), the beam receives assembly support.
This support is removed after the grout has set. This means that the beam,
spanning over 11,1m , also acts as a composite beam for its self-weight.

The system during assembly:



cross-section:



IPE 360 S235

load:

dead load:	g_0 =	7,5kN/m				
finishing loa	g_1 =	0,4kN/m				
live load:	p =	6,8kN/m	permanently from it:	40%	->p_s =	2,7kN/m
			variable from it:	60%	->p_v =	4,1kN/m

cross-sectional values:

$\varphi\ (t=\infty,t_0)$ = 1,96

$\varepsilon_{cs,\ unendl.}$ = 0,0052%

		t_0	$t_{unendl.}$	
			creep	shrinkage
A_a	[cm²]	72,7		
I_a	[cm⁴]	16298		
E	[N/mm²]	21000	35220,46	
ψ_P			1,10	
ψ_{PT}			0,55	
ψ_S				0,55
n_0		5,96		
n_P			18,82	
n_{PT}			12,39	
n_S				12,39
A_c	[cm²]	466,42	147,79	224,46
I_c	[cm⁴]	4123,54	1306,63	1984,44
A_i	[cm²]	539,15	220,52	297,19
I_i	[cm⁴]	54141,2	42641,26	46034,17

deformations / support forces: (deflections at the center of the beam)

A = **15,7kN** B = **52,2kN** C = **15,7kN**

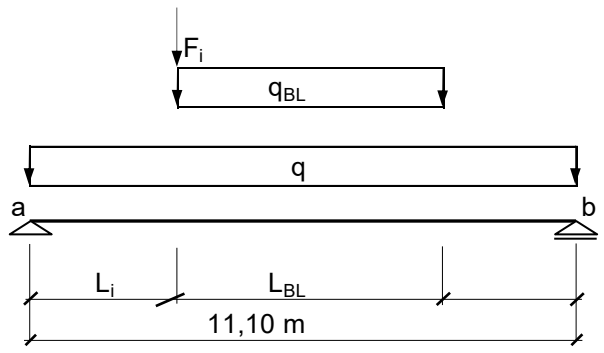
steel beam deformation		0,0cm
composite beam deformation due to	g_0	1,3cm
composite beam deformation due to	g_1+p_s	0,5cm
composite beam deformation due to creep	g_0, g_1, p_s	0,5cm
composite beam deformation due to shrinkage		1,9cm
composite beam deformation due to	p_v	0,5cm
total deformation t= ∞		4,7cm
	overheight	**3,0cm**
total sag		1,7cm = L/654

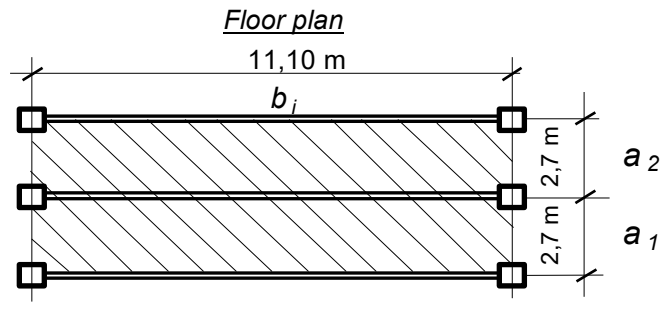
Pos BE134 (Alt.) Composite beam Individuell

*** alternative position ***

Version: 16

SYSTEM





Influence area according to DBV booklet 36

$A_{EF} = b_i * (a_1 + a_2)$

Width carried: 2,7 m -
Influence width: 5,4 m
Influence area: 59,9 m²

LOADS

Partial safety factors for the loads γ_G = 1,35, γ_F = 1,5

Area loads

$g_{0,k}$ - permanent loads (characteristic)

Concrete slab	0,10 m *	25 kN/m³	* 2,7 m =	7,0 kN/m
Self weight of beam		-	-	0,7 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
			Σ g_0 =	7,6 kN/m

$g_{1,k}$ - permanent loads from finishing (characteristic)

Finishing, installation		0,15 kN/m²	* 2,7 m =	0,4 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
			Σ g_1 =	0,4 kN/m

$p_{,k}$ - variable loads (characteristic)

Live load		2,50 kN/m²	* 2,7 m =	6,8 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
			Σ p =	6,8 kN/m

Point and uniformly distributed loads (characteristic)

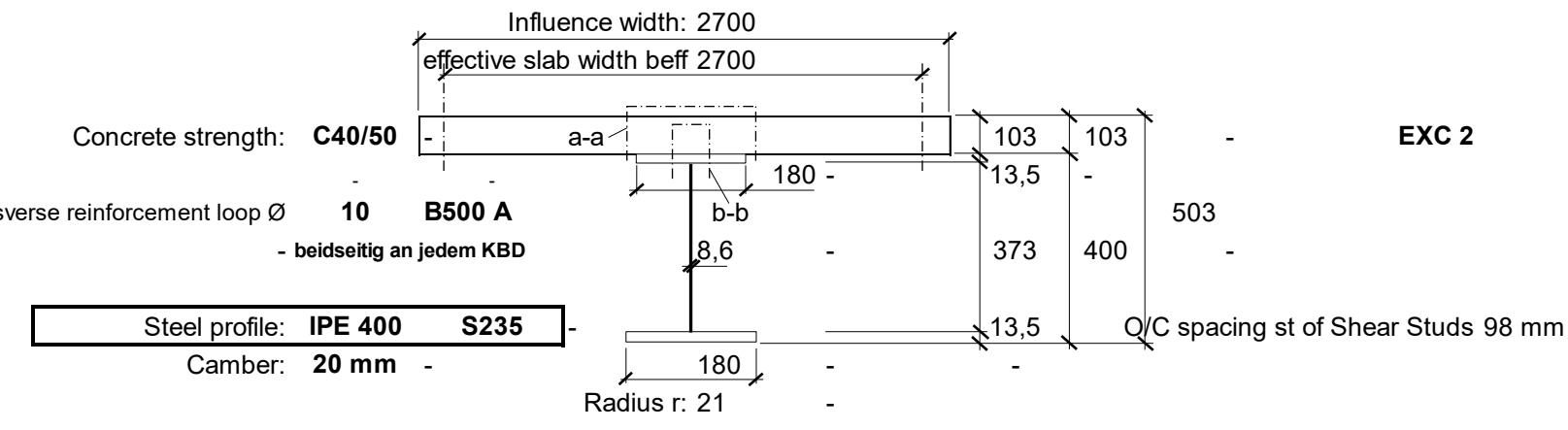
	$g_{0,k}$	$g_{1,k}$	$p_{,k}$	L_i	L_{BL}
F_1	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_2	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_3					-
F_4					-
F_5					-
q_{BL}					

SUPPORT REACTIONS

characteristic **($\gamma_{G,F}$ = 1,0):**

$A_{g0,k}$ =	42,3 kN			$B_{g0,k}$ =	42,3 kN		
$A_{g1,k}$ =	2,2 kN	$A_{g,k}$ =	**44,5 kN**	$B_{g1,k}$ =	2,2 kN	$B_{g,k}$ =	**44,5 kN**
		$A_{p,k}$ =	**37,5 kN**			$B_{p,k}$ =	**37,5 kN**
Design loads:		A_{Ed} =	**116,3 kN**			B_{Ed} =	**116,3 kN**

CROSS-SECTION



Concrete strength: **C40/50**

Transverse reinforcement loop Ø **10** **B500 A**

- beidseitig an jedem KBD

Steel profile: **IPE 400** **S235**

Camber: **20 mm**

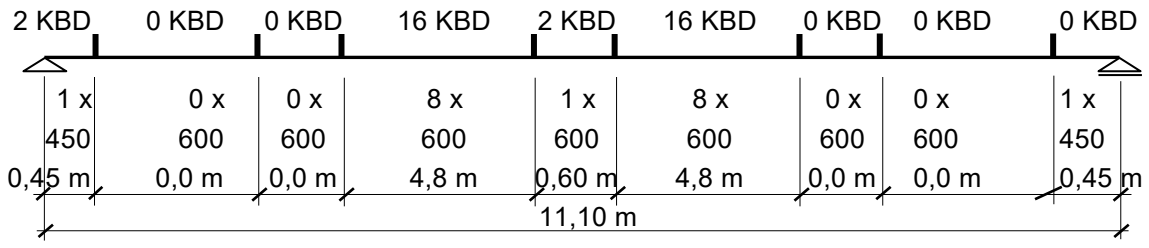
Radius r: 21

EXC 2

O/C spacing st of Shear Studs 98 mm

SHEAR STUD ARRANGEMENT

Shear Studs: **doppelreihig** **double-row** ∅ **19** Height: **76** Total number: 36 KBD



INTERNAL DESIGN FORCES

Pos BE134 (Alt.)

Final state

max M_{Ed} (γ_G * (M_{g0} + M_{g1}) + γ_F * M_p) = **322,7 kNm** at x = 5,5 m

max V_{Ed} (γ_G * (V_{g0} + V_{g1}) + γ_F * V_p) = **116,3 kN**

Construction state

max M_{Ed} (γ_G * M_{g0}) = 158,3 kNm

MATERIAL PROPERTIES

for design

Concrete			Reinforcement			Steel			Shear studs			
γ_C =	1,5		γ_S =	1,15		γ_{M0} =	1,00		γ_{Va} =	1,25		
α_{cc} =	0,85								γ_{Vc} =	1,50		
f_{ck} =	40	N/mm²	f_{yk} =	500	N/mm²	f_{yk} =	235	N/mm²	α =	1,00		
f_{cd} =	23	N/mm²	f_{yd} =	435	N/mm²	f_{yd} =	235	N/mm²	f_u =	470	N/mm²	-
f_{ctm} =	3,5	N/mm²							P_{Rd} =	75,0	kN	
E_{cm} =	35000	N/mm²										
ν =	0,675											

SECTION PROPERTIES

for design

Concrete		Steel		Composite cross section	
A_c = 2781,0 cm²	N_{cd} = 6303,6 kN	A_a = 84,5 cm²	$N_{pl,a,Rd}$ = 1984,9 kN	z_{pl} = 3,2 cm	--> neutral line in concrete slab
		A_{aS} = 39,3 cm²	$V_{pl,a,Rd}$ = 532,9 kN	$M_{pl,Rd}$ = 569,2 kNm	
		$z_{el,a}$ = 20,0 cm	$z_{el',a}$ = 20,0 cm		
		$z_{pl,a}$ = 20,0 cm	$M_{pl,a,Rd}$ = 307,5 kNm		

DESIGN

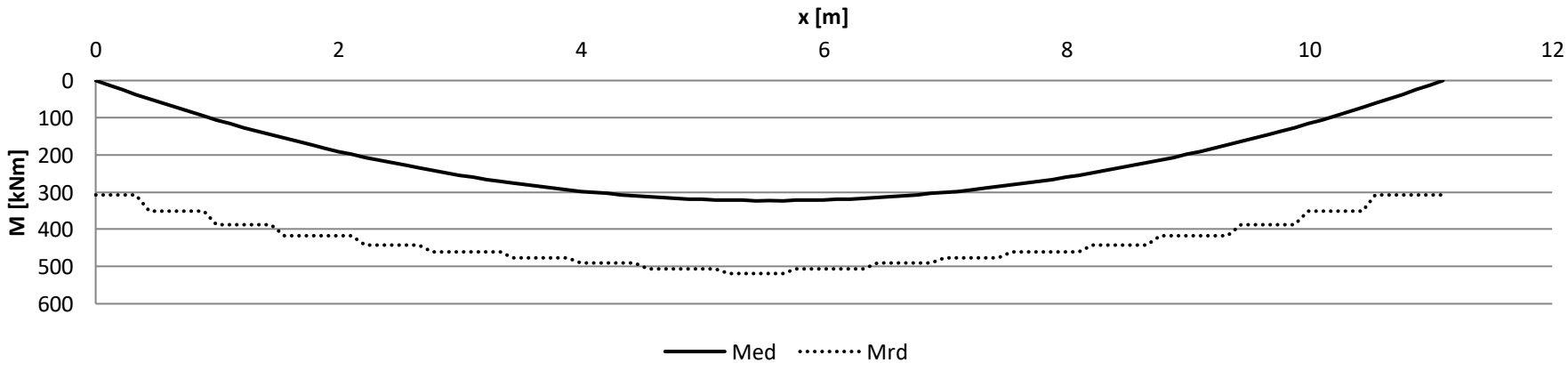
Cross-section classification $\varepsilon = \sqrt{(235/f_{yk})}$ = 1,00

Shear force resistance

Web: vorh h_w/t = 38,5 < 72*ε = 72,0 --> no shear buckling analysis necessary

NW: $V_{Ed}/V_{pl,a,Rd}$ = 0,22 <1; <0,5 → ρ= 0,00 --> shear force without influence on bending

Moment resistance (plastic with partial anchoring along the beam axis)



NW: max (M_{Ed}/M_{Rd}) = 0,64 < 1 at x = 5,9 m

Application limits for: anchoring partially; Shear stud distribution equidistant

Cross-section class 1 or 2: **fulfilled**

Verification of ductility according to DIN-EN 1994-1-1 (possible according to 6.6.1.1 or 6.6.1.2, one criterion must be fulfilled)

According to 6.6.1.2:

Minimum degree of shear connec: min η = 0,40 < vorh η = 0,68 **fulfilled**

According to 6.6.1.1:

Ductility: Deformation capacity according to separate verification > 6.0 mm **fulfilled**

∅ KBD: 16 mm < 19 mm < 25 mm **fulfilled**

$h_{KBD}/\varnothing_{KBD} \geq 4$: **fulfilled**

Longitudinal shear resistance (Ansatz θ_f= 23,7°) mit : σ_{cd} = 0 N/mm² $V_{Rd,cc}$ = 938,4 kN [=$N_{pl,a,Rd}$] V_{Ed} = 1985 kN cotθ= 2,3

min e_{KBD}= Δx = 0,60 m ΔF_d = 150,0 kN $h_{f,a-a}$ = 206 mm --> $\nu_{Ed,a-a}$ = 1,21 N/mm²

$h_{f,b-b}$ = 269 mm --> $\nu_{Ed,b-b}$ = 0,93 N/mm²

Tension strut s_f = 0,60 m A_{sf} = 3,14 cm² $\rho_w = A_{sf} / (s_f * h_f)$ $\nu_{Rd} = (A_{sf} * f_{yd}/s_f) * \cot \theta_f/h_f$

Section a-a: $\rho_{w,a-a}$ = 0,003 --> $\nu_{Rd,a-a}$ = 2,52 N/mm²

Section b-b: $\rho_{w,b-b}$ = 0,002 --> $\nu_{Rd,b-b}$ = 1,93 N/mm²

$\nu_{Rd} = \nu * f_{cd} * \sin \theta_f * \cos \theta_f$ = 5,63 N/mm²

NW: ν_{Ed}/ν_{Rd} = 0,48 < 1

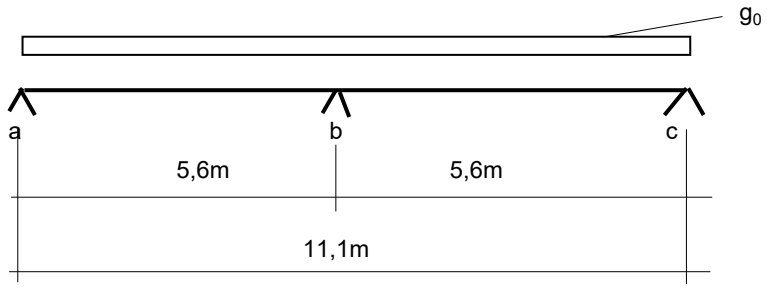
Minimum reinforcement $\rho_{w,min}$ = **0,001 < 0,002** = ρ_w

DEFORMATION CALCULATION WITH AN AUXILIARY SUPPORT

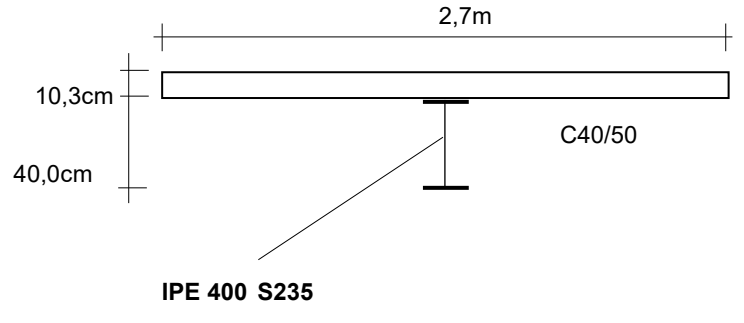
Pos BE134 (Alt.)

During the construction phase (placement of the ceiling slabs, pouring of the joint), the beam receives assembly support.
This support is removed after the grout has set. This means that the beam,
spanning over 11,1m , also acts as a composite beam for its self-weight.

The system during assembly:



cross-section:



IPE 400 S235

load:

dead load:	g_0 =	7,6kN/m				
finishing loa	g_1 =	0,4kN/m				
live load:	p =	6,8kN/m	permanently from it:	40%	->p_s =	2,7kN/m
			variable from it:	60%	->p_v =	4,1kN/m

cross-sectional values:

$\varphi\ (t=\infty,t_0)$ = 1,96

$\varepsilon_{cs,\ unendl.}$ = 0,0052%

		t_0	$t_{unendl.}$	
			creep	shrinkage
A_a	[cm²]	84,5		
I_a	[cm⁴]	23185		
E	[N/mm²]	21000	35220,46	
ψ_P			1,10	
ψ_{PT}			0,55	
ψ_S				0,55
n_0		5,96		
n_P			18,82	
n_{PT}			12,39	
n_S				12,39
A_c	[cm²]	466,42	147,79	224,46
I_c	[cm⁴]	4123,54	1306,63	1984,44
A_i	[cm²]	550,88	232,26	308,93
I_i	[cm⁴]	72542,7	57414,05	62312,49

deformations / support forces: (deflections at the center of the beam)

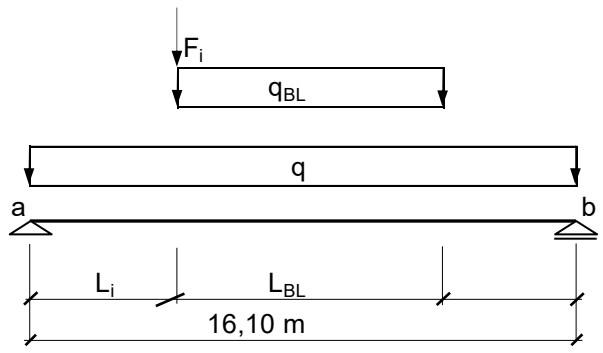
A = **15,8kN** B = **52,8kN** C = **15,8kN**

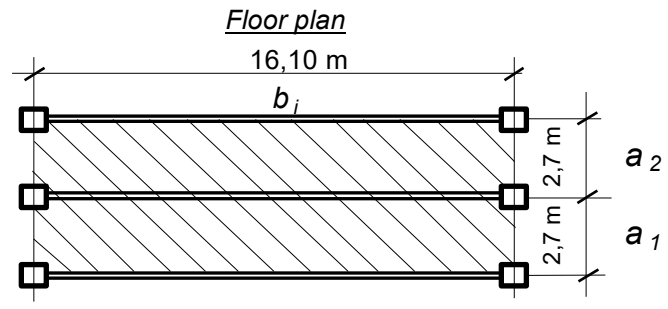
steel beam deformation		0,0cm
composite beam deformation due to	g_0	1,0cm
composite beam deformation due to	g_1+p_s	0,4cm
composite beam deformation due to creep	g_0, g_1, p_s	0,4cm
composite beam deformation due to shrinkage		1,7cm
composite beam deformation due to	p_v	0,4cm
total deformation t= ∞		3,8cm
	overheight	**2,0cm**
total sag		1,8cm = L/619

Pos BE152 Composite beam Individuell

Version: 16

SYSTEM





Width carried: 2,7 m -
Influence width: 5,4 m
Influence area: 86,9 m²

Influence area according to DBV booklet 36

$A_{EF} = b_i * (a_1 + a_2)$

LOADS

Partial safety factors for the loads γ_G = 1,35, γ_F = 1,5

Area loads

$g_{0,k}$ - permanent loads (characteristic)

Concrete slab	0,10 m *	25 kN/m³	* 2,7 m =	7,0 kN/m
Self weight of beam		-	-	0,7 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
			Σ g_0 =	7,6 kN/m

$g_{1,k}$ - permanent loads from finishing (characteristic)

Finishing, installation		0,15 kN/m²	* 2,7 m =	0,4 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
			Σ g_1 =	0,4 kN/m

$p_{,k}$ - variable loads (characteristic)

Live load		2,50 kN/m²	* 2,7 m =	6,8 kN/m
Additional load		0,0 kN/m²	* 0,0 m =	0,0 kN/m
-			Σ p =	6,8 kN/m

Point and uniformly distributed loads (characteristic)

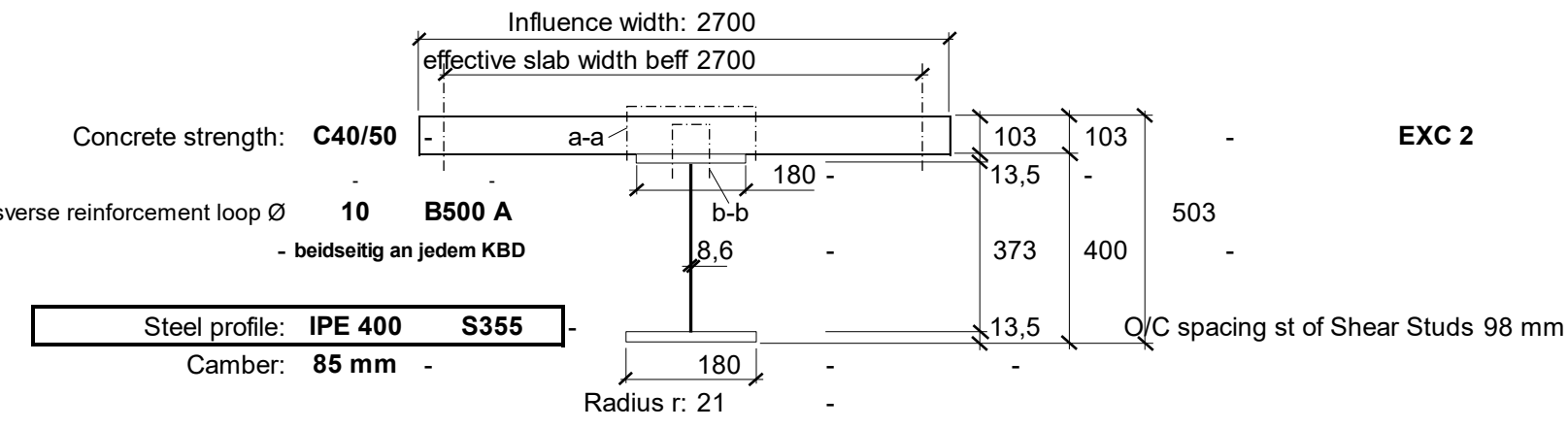
	$g_{0,k}$	$g_{1,k}$	$p_{,k}$	L_i	L_{BL}
F_1	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_2	0,0 kN	0,0 kN	0,0 kN	0,0 m	-
F_3					-
F_4					-
F_5					-
q_{BL}					

SUPPORT REACTIONS

characteristic **($\gamma_{G,F}$ = 1,0):**

$A_{g0,k}$ =	61,3 kN			$B_{g0,k}$ =	61,3 kN		
$A_{g1,k}$ =	3,3 kN	$A_{g,k}$ =	**64,6 kN**	$B_{g1,k}$ =	3,3 kN	$B_{g,k}$ =	**64,6 kN**
		$A_{p,k}$ =	**54,3 kN**			$B_{p,k}$ =	**54,3 kN**
Design loads:		A_{Ed} =	**168,7 kN**			B_{Ed} =	**168,7 kN**

CROSS-SECTION



Concrete strength: **C40/50**

Transverse reinforcement loop Ø **10** **B500 A**

- beidseitig an jedem KBD

Steel profile: **IPE 400** **S355**

Camber: **85 mm**

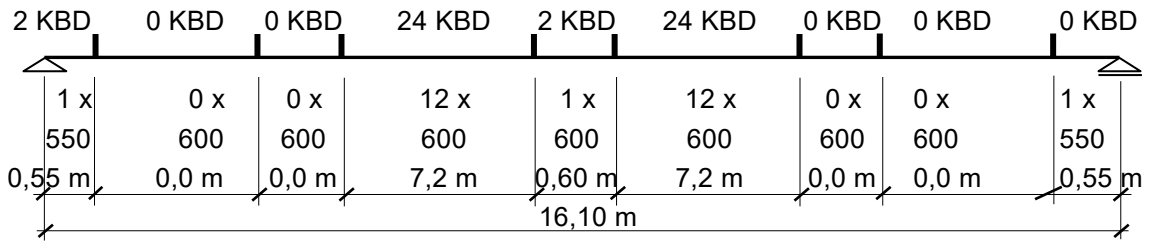
Radius r: 21

EXC 2

O/C spacing st of Shear Studs 98 mm

SHEAR STUD ARRANGEMENT

Shear Studs: **doppelreihig** **double-row** ∅ **19** Height: **76** Total number: 52 KBD



INTERNAL DESIGN FORCES

Pos BE152

Final state

max M_{Ed} (γ_G * (M_{g0} + M_{g1}) + γ_F * M_p) = **678,9 kNm** at x = 8,0 m

max V_{Ed} (γ_G * (V_{g0} + V_{g1}) + γ_F * V_p) = **168,7 kN**

Construction state

max M_{Ed} (γ_G * M_{g0}) = 333,1 kNm

MATERIAL PROPERTIES

for design

Concrete			Reinforcement			Steel			Shear studs			
γ_C =	1,5		γ_S =	1,15		γ_{M0} =	1,00		γ_{Va} =	1,25		
α_{cc} =	0,85								γ_{Vc} =	1,50		
f_{ck} =	40	N/mm²	f_{yk} =	500	N/mm²	f_{yk} =	355	N/mm²	α =	1,00		
f_{cd} =	23	N/mm²	f_{yd} =	435	N/mm²	f_{yd} =	355	N/mm²	f_u =	470	N/mm²	-
f_{ctm} =	3,5	N/mm²							P_{Rd} =	75,0	kN	
E_{cm} =	35000	N/mm²										
ν =	0,675											

SECTION PROPERTIES

for design

Concrete		Steel		Composite cross section	
A_c = 2781,0 cm²	N_{cd} = 6303,6 kN	A_a = 84,5 cm²	$N_{pl,a,Rd}$ = 2998,5 kN	z_{pl} = 4,9 cm	--> neutral line in concrete slab
		A_{aS} = 39,3 cm²	$V_{pl,a,Rd}$ = 805,0 kN	$M_{pl,Rd}$ = 835,1 kNm	
		$z_{el,a}$ = 20,0 cm	$z_{el',a}$ = 20,0 cm		
		$z_{pl,a}$ = 20,0 cm	$M_{pl,a,Rd}$ = 464,6 kNm		

DESIGN

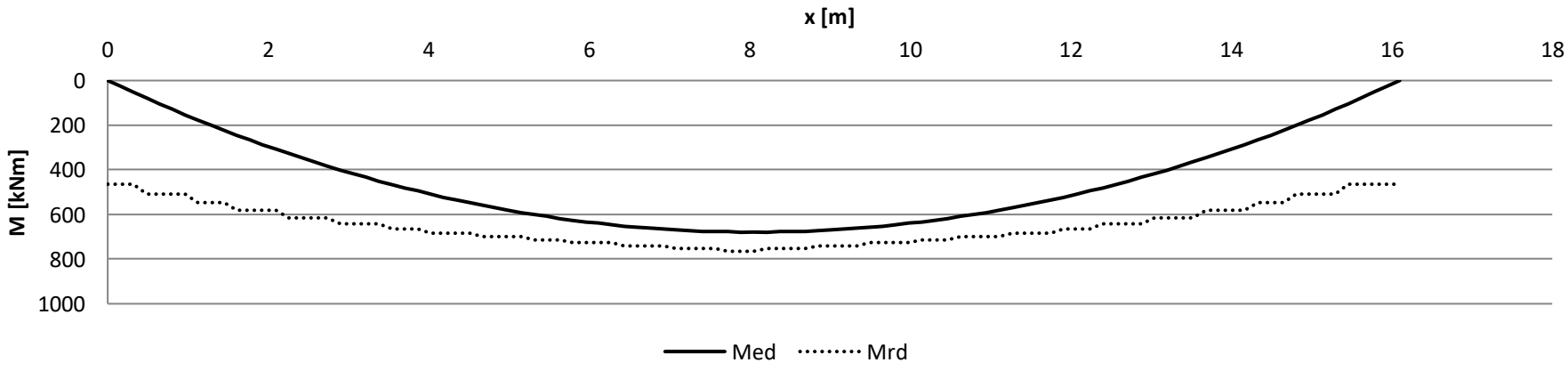
Cross-section classification $\varepsilon = \sqrt{(235/f_{yk})}$ = 0,81

Shear force resistance

Web: vorh h_w/t = 38,5 < 72*ε = 58,6 --> no shear buckling analysis necessary

NW: $V_{Ed}/V_{pl,a,Rd}$ = 0,21 <1; <0,5 → ρ= 0,00 --> shear force without influence on bending

Moment resistance (plastic with partial anchoring along the beam axis)



NW: max (M_{Ed}/M_{Rd}) = 0,90 < 1 at x = 9,0 m

Application limits for	anchoring	Shear stud distribution
	partially	equidistant
Cross-section class 1 or 2:	**fulfilled**	

Verification of ductility according to DIN-EN 1994-1-1 (possible according to 6.6.1.1 or 6.6.1.2, one criterion must be fulfilled)

According to 6.6.1.2:

Minimum degree of shear connec: min η = 0,73 > vorh η = 0,65 **not fulfilled**

According to 6.6.1.1:

Ductility: Deformation capacity according to separate verification > 6.0 mm **fulfilled**

∅ KBD: 16 mm < 19 mm < 25 mm **fulfilled**

$h_{KBD}/\varnothing_{KBD} \geq 4$: **fulfilled**

[=$N_{pl,a,Rd}$]

Longitudinal shear resistance (Ansatz θ_f= 24,5°) mit : σ_{cd} = 0 N/mm² $V_{Rd,cc}$ = 1361,1 kN V_{Ed} = 2998 kN cotθ= 2,2

min e_{KBD}= Δx = 0,60 m ΔF_d = 150,0 kN $h_{f,a-a}$ = 206 mm --> $\nu_{Ed,a-a}$ = 1,21 N/mm²

$h_{f,b-b}$ = 269 mm --> $\nu_{Ed,b-b}$ = 0,93 N/mm²

Tension strut s_f = 0,60 m A_{sf} = 3,14 cm² $\rho_w = A_{sf} / (s_f * h_f)$ $\nu_{Rd} = (A_{sf} * f_{yd}/s_f) * \cot \theta_f/h_f$

Section a-a: $\rho_{w,a-a}$ = 0,003 --> $\nu_{Rd,a-a}$ = 2,43 N/mm²

Section b-b: $\rho_{w,b-b}$ = 0,002 --> $\nu_{Rd,b-b}$ = 1,86 N/mm²

$\nu_{Rd} = \nu * f_{cd} * \sin \theta_f * \cos \theta_f$ = 5,77 N/mm²

NW: ν_{Ed}/ν_{Rd} = 0,50 < 1

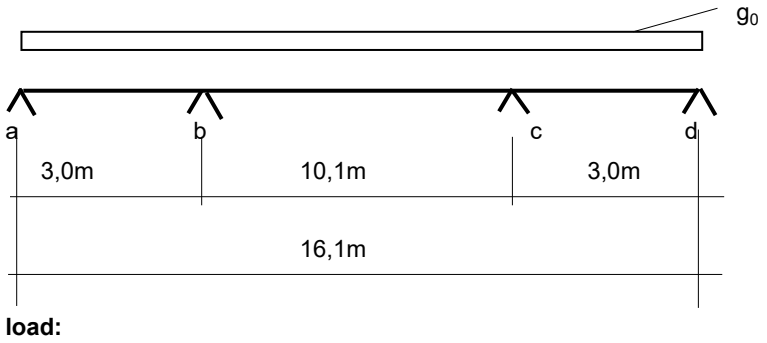
Minimum reinforcement $\rho_{w,min}$ = **0,001 < 0,002** = ρ_w

DEFORMATION CALCULATION WITH TWO AUXILIARY SUPPORT

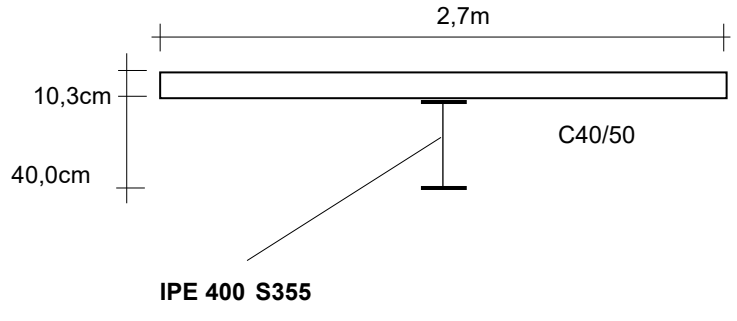
Pos BE152

During the construction phase (placement of the ceiling slabs, pouring of the joint), the beam receives assembly support.
This support is removed after the grout has set. This means that the beam,
spanning over 16,1m , also acts as a composite beam for its self-weight.

The system during assembly:



cross-section:



load:

dead load: g_0 = 7,6kN/m
finishing loa g_1 = 0,4kN/m
live load: p = 6,8kN/m permanently from it: 40% ->p_s = 2,7kN/m
variable from it: 60% ->p_v = 4,1kN/m

cross-sectional values:

$\varphi\ (t=\infty,t_0)$ = 1,96
$\varepsilon_{cs,\ unendl.}$ = 0,0052%

		t_0	$t_{unendl.}$	
			creep	shrinkage
A_a	[cm²]	84,5 cm²		
I_a	[cm^4]	23185,402		
E	[N/mm²]	21000	35220,46	
ψ_P			1,10	
ψ_{PT}			0,55	
ψ_S				0,55
n_0		5,96		
n_P			18,82	
n_{PT}			12,39	
n_S				12,39
A_c	[cm²]	466,42	147,79	224,46
I_c	[cm^4]	4123,5	1306,63	1984,44
A_i	[cm²]	550,88	232,26	308,93
I_i	[cm^4]	72543	57414,05	62312,49

deformations / support forces: (deflections at the center of the beam)

A = **-7,1kN** B = **68,4kN** C = **68,4kN** D = **-7,1kN**

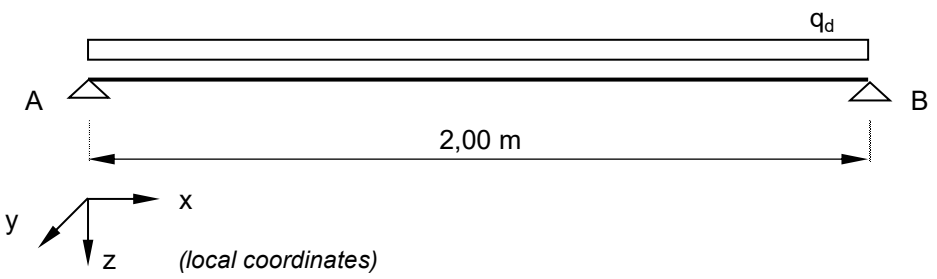
steel beam deformation		0,7cm
composite beam deformation due to	g_0	4,2cm
composite beam deformation due to	g_1+p_s	1,8cm
composite beam deformation due to creep	g_0, g_1, p_s	1,6cm
composite beam deformation due to shrinkage		3,5cm
composite beam deformation due to	p_v	1,6cm
total deformation t= ∞		13,3cm

overheight **8,5cm**

total sag 4,8cm = L/335

Pos BE281 Edge angle L = 2,00 m

SYSTEM AND LOADS:



Influence width: 1,45 m

Rotation of the beam axis α = 0,0 °

permanent loads:

$g_{0,k}$ - permanent loads (characteristic)			
Concrete slab	2,58 kN/m² * 1,45 m =		3,7 kN/m
self weight of beam			0,2 kN/m
		g =	4,0 kN/m

variable loads

$p_{,k}$ - variable loads (characteristic)			
Live load	2,50 kN/m² * 1,45 m =		3,6 kN/m
Installation	0,15 kN/m² * 1,45 m =		0,2 kN/m
		p =	3,8 kN/m

q_d = **11,1 kN/m**

INTERNAL FORCES:

Bearing reactions:	A_g =	**4,0 kN**	B_g =	**4,0 kN**
	A_p =	**3,8 kN**	B_p =	**3,8 kN**
	A_{Ed} =	**11,1 kN**	B_{Ed} =	**11,1 kN**
Governing internal design forces:	$V_{y,Ed}$ =	0,0 kN	$M_{y,Ed}$ =	**5,6 kNm**
	$V_{z,Ed}$ =	**11,1 kN**	$M_{z,Ed}$ =	0,0 kNm

DESIGN

selected: **L 150*10 S 235** **EXC2**

Cross section class 3

γ_M = 1,00

alternatively L200x150x12 S235 if geometrically required

σ_{Ed}	= $M_{y,Ed}$ / I_y * z	=	555,7 / 624 * 10,966	= 9,77 kN/cm²
σ_{Ed} / σ_{Rd}		=	9,77 / 23,5	= **0,42 < 1**
max τ_{Ed}	= 1,5* max $V_{z,Ed}$ / A (vertical leg)	=	1,5 * 11 / 15	= 1,1 kN/cm²
τ_{Ed} / τ_{Rd}		=	1,1 / 13,6	= **0,05 < 1**

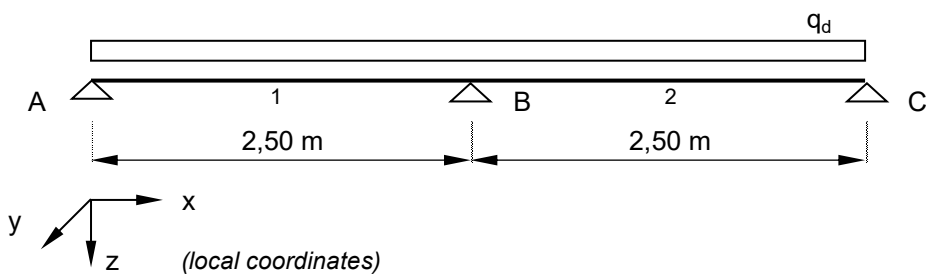
Camber:

Final deflection: fz = 0,1 cm = L / 1610

The angle is stabilised by the concrete slab through the shear studs.

Pos BE282 Edge angle L = 5,00 m

SYSTEM AND LOADS:



Influence width: 1,45 m

Rotation of the beam axis: α = 0,0 °

permanent loads:

$g_{0,k}$ - permanent loads (characteristic)			
Concrete slab	2,58 kN/m² * 1,45 m =		3,7 kN/m
Self weight of beam			0,2 kN/m
		g =	4,0 kN/m

variable loads

$p_{,k}$ - variable loads (characteristic)			
Live loads	2,50 kN/m² * 1,45 m =		3,6 kN/m
Installation	0,15 kN/m² * 1,45 m =		0,2 kN/m
The variable loads act only on individual spans.		p =	3,8 kN/m

q_{Ed} = 11,1 kN/m

INTERNAL FORCES:

Bearing reaction:	A_g =	**3,716 kN**	B_g =	**12,4 kN**	C_g =	**3,7 kN**
	$A_{p,max}$ =	**4,203 kN**	$B_{p,max}$ =	**12,0 kN**	$C_{p,max}$ =	**4,2 kN**
	$A_{p,min}$ =	**-0,6 kN**	$B_{p,min}$ =	0,0 kN	$C_{p,min}$ =	**-0,6 kN**
	$A_{Ed,max}$ =	**11,3 kN**	$B_{Ed,max}$ =	**34,7 kN**	$C_{Ed,max}$ =	**11,3 kN**
	$A_{Ed,min}$ =	3,7 kN	$B_{Ed,min}$ =	12,4 kN	$C_{Ed,min}$ =	3,7 kN
Internal design forces:			$V_{B,Ed,li}$ =	-17,4 kN	$M_{B,Ed}$ =	-8,7 kNm
			$V_{B,Ed,re}$ =	17,4 kN	$M_{1,Ed}$ =	5,8 kNm
					$M_{2,Ed}$ =	5,8 kNm
Governing internal design forces:	{		$M_{y,Ed}$ =	**8,7 kNm**	$M_{z,Ed}$ =	0,0 kNm
			$V_{z,Ed}$ =	**17,4 kN**	$V_{y,Ed}$ =	0,0 kN

DESIGN

selected: **L 150*10 S 235** **EXC2**

cross section class 3

γ_M = 1,00

alternatively L200x150x12 S235 if geometrically required

σ_{Ed}	= $M_{y,Ed} / I_y * z$	=	868,3 / 624 * 10,966	=	15,3 kN/cm²
$\sigma_{Ed} / \sigma_{Rd}$		=	15,26 / 23,5	=	**0,65 < 1**
max τ_{Ed}	= 1,5 * max $V_{z,Ed} / A$ (vertical leg)	=	1,5 * 17 / 15	=	1,7 kN/cm²
τ_{Ed} / τ_{Rd}		=	1,7 / 13,6	=	**0,13 < 1**

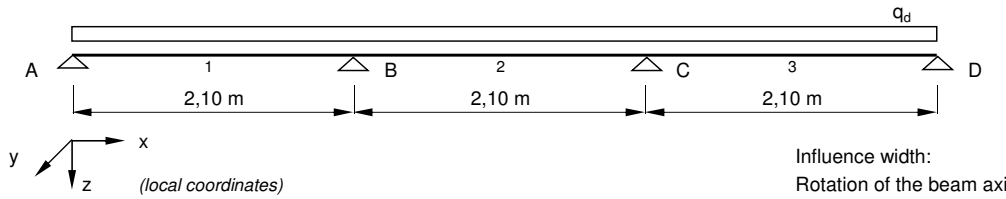
Final deflection:

Span 1	f_z =	0,2 cm	=	L / 1480
Span 2	f_z =	0,2 cm	=	L / 1480

The angle is stabilised by the concrete slab through the shear studs.

Pos BE283 Edge angle L = 6,30 m

SYSTEM AND LOADS:



Influence width: 1,45 m
Rotation of the beam axis: α = 0,0 °

permanent loads:

$g_{0,k}$ - permanent loads (characteristic)			
Concrete slab	2,58 kN/m² * 1,45 m =		3,7 kN/m
self weight of beam			0,2 kN/m
		g =	4,0 kN/m

variable loads

$p_{,k}$ - variable loads (characteristic)			
Live Load	2,50 kN/m² * 1,45 m =		3,6 kN/m
Installation	0,15 kN/m² * 1,45 m =		0,2 kN/m
The variable loads act only on individual spans.		p =	3,8 kN/m

For load transfer, traffic loads may be reduced to 2,5 kN/m².

q_{Ed} = 11,1 kN/m

INTERNAL FORCES:

Bearing reactions:

A_g =	**3,335 kN**	B_g =	**9,172 kN**	C_g =	**9,2 kN**	D_g =	**3,3 kN**
$A_{p,max}$ =	**3,631 kN**	$B_{p,max}$ =	**9,683 kN**	$C_{p,max}$ =	**9,7 kN**	$D_{p,max}$ =	**3,6 kN**
$A_{p,min}$ =	**-0,4 kN**	$B_{p,min}$ =	**-0,8 kN**	$C_{p,min}$ =	**-0,8 kN**	$D_{p,min}$ =	**-0,4 kN**
$A_{Ed,max}$ =	**9,9 kN**	$B_{Ed,max}$ =	**26,9 kN**	$C_{Ed,max}$ =	**26,9 kN**	$D_{Ed,max}$ =	**9,9 kN**
$A_{Ed,min}$ =	3,3 kN	$B_{Ed,min}$ =	9,2 kN	$C_{Ed,min}$ =	9,2 kN	$D_{Ed,min}$ =	3,3 kN

Internal design forces:		-14,2 kN	$V_{C,Ed,li}$ =	-12,7 kN	$M_{1,Ed}$ =	4,4 kNm
		12,7 kN	$V_{C,Ed,re}$ =	14,2 kN	$M_{2,Ed}$ =	2,5 kNm
	$M_{B,Ed}$ =	-5,3 kNm	$M_{C,Ed}$ =	-5,3 kNm	$M_{3,Ed}$ =	4,4 kNm
Governing internal design forces:	$M_{y,Ed}$ =	**5,3 kNm**	$M_{z,Ed}$ =	0,0 kNm		
	$V_{z,Ed}$ =	**14,2 kN**	$V_{y,Ed}$ =	0,0 kN		

DESIGN

selected: **L 150*10 S 235** **EXC2**

cross section class 3
γ_M = 1,00

alternatively L200x150x12 S235 if geometrically required

σ_{Ed}	= $M_{y,Ed} / I_y * z$	= 532,9 / 624 * 10,966	=	9,4 kN/cm²
$\sigma_{Ed} / \sigma_{Rd}$		= 9,4 / 23,5	=	**0,40 < 1**
max τ_{Ed}	= 1,5 * max $V_{z,Ed}$ / A (vertical leg)	= 1,5 * 14 / 15	=	1,4 kN/cm²
τ_{Ed} / τ_{Rd}		= 1,4 / 13,6	=	**0,10 < 1**

Deflection:

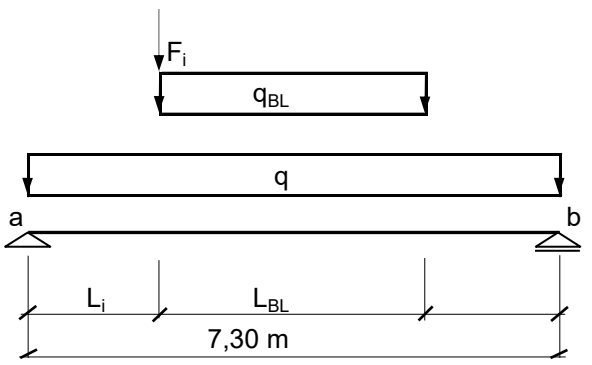
Span 1	f_z =	0,1 cm	=	L / 2160
Span 2	f_z =	0,0 cm	=	L / 5050
Span 3	f_z =	0,1 cm	=	L / 2160

The angle is stabilised by the concrete slab through the shear studs.

Pos BE302 Composite girder L = 7,30 m

Ramp beam

SYSTEM



Influenced width: 2,5 m

LOAD

Partial safety factors of the load γ_G = 1,35

γ_F = 1,5

Area loads

$g_{0,k}$ - constant loads (characteristic)

concrete slab	0,10 m *	25 kN/m³	* 2,5 m =	6,4 kN/m
Dead load of ceiling girder				0,4 kN/m
Additional load		0,5 kN/m²	* 2,5 m =	1,3 kN/m
			Σ g_0 =	8,1 kN/m

$g_{1,k}$ - constant expansion loads (characteristic)

Construction, Installation	0,2 kN/m²	* 2,5 m =	0,4 kN/m
Additional load	0,0 kN/m²	* 0,0 m =	0,0 kN/m
		Σ g_1 =	0,4 kN/m

$p_{,k}$ - variable loads (characteristic)

Payload	2,5 kN/m²	* 2,5 m =	6,3 kN/m
Additional load	0,0 kN/m²	* 0,0 m =	0,0 kN/m
		Σ p =	6,3 kN/m

Individual loads and block loads (characteristic)

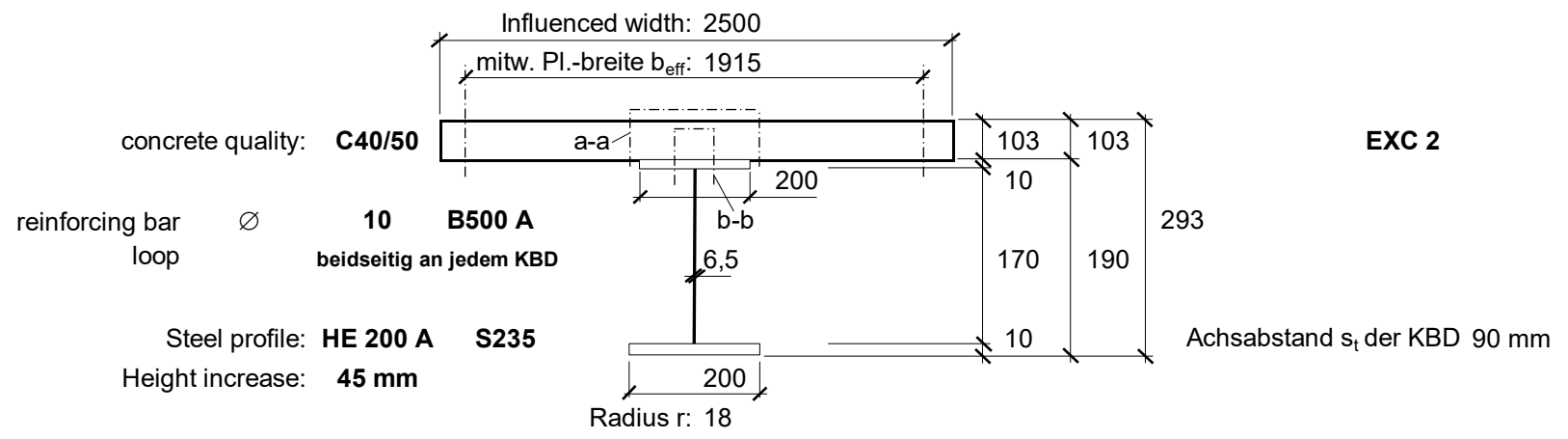
	$g_{0,k}$	$g_{1,k}$	$p_{,k}$	+	L_{BL}
F_1	5,5 kN			0,6 m	
F_2	5,5 kN			6,8 m	
F_3					
F_4					
F_5					
q_{BL}					

SUPPORT FORCES

characteristic **($\gamma_{G,F}$ = 1,0):**

$A_{g0,k}$ =	35,1 kN				$B_{g0,k}$ =	35,1 kN			
$A_{g1,k}$ =	1,4 kN	$A_{g,k}$ =	**36,5 kN**		$B_{g1,k}$ =	1,4 kN	$B_{g,k}$ =	**36,5 kN**	
		$A_{p,k}$ =	**22,8 kN**				$B_{p,k}$ =	**22,8 kN**	
Dimensioning loads:		A_{Ed} =	**83,5 kN**				B_{Ed} =	**83,5 kN**	

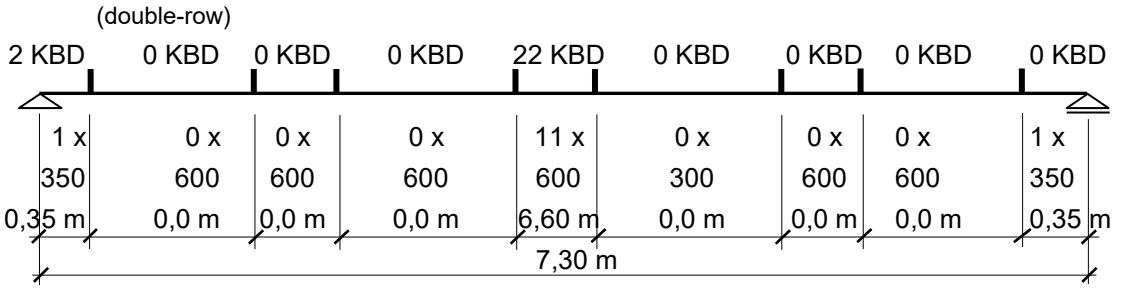
CROSS-SECTION



DOWEL ARRANGEMENT

Dowel: **doppelreihig** ∅ **19** Height: **76** Total: 24 KBD

(double-row)



DIMENSIONING INTERNAL FORCES

Final status: max M_{Ed} (γ_G * (M_{g0} + M_{g1}) + γ_F * M_p) = **142,8 kNm** at x = 3,7 m

max V_{Ed} (γ_G * (V_{g0} + V_{g1}) + γ_F * V_p) = **83,5 kN**

Stage of construction: max M_{Ed} (γ_G * M_{g0}) = 77,0 kNm

MATERIAL PARAMETERS for dimensioning

Concrete			Reinforcing bar			Steel			Shear studs		
γ_C =	1,5		γ_S =	1,15		γ_{M0} =	1,0		γ_{Va} =	1,25	
α_{cc} =	0,85								γ_{Vc} =	1,5	
f_{ck} =	40	N/mm²	f_{yk} =	500	N/mm²	f_{yk} =	235	N/mm²	α =	1,00	
f_{cd} =	23	N/mm²	f_{yd} =	435	N/mm²	f_{yd} =	235	N/mm²	f_u =	470	N/mm²
f_{ctm} =	3,5	N/mm²							P_{Rd} =	75,0	kN
E_{cm} =	35000	N/mm²									
ν =	0,675										

PROFILE CHARACTERISTICS for dimensioning

Concrete			Steel			Composite cross-section		
A_c =	1972,5 cm²	N_{cd} = 4470,9 kN	A_a =	53,8 cm²	$N_{pl,a,Rd}$ = 1265,0 kN	z_{pl} =	2,9 cm	--> Zero line in the concrete slab
			A_{aS} =	15,5 cm²	$V_{pl,a,Rd}$ = 210,5 kN	$M_{pl,Rd}$ =	232,0 kNm	
			$z_{el,a}$ =	9,5 cm	$z_{el',a}$ = 9,5 cm			
			$z_{pl,a}$ =	9,5 cm	$M_{pl,a,Rd}$ = 101,2 kNm			

DIMENSIONING

Cross-section classification $\varepsilon = \sqrt{(235/f_{yk})}$ = 1,00

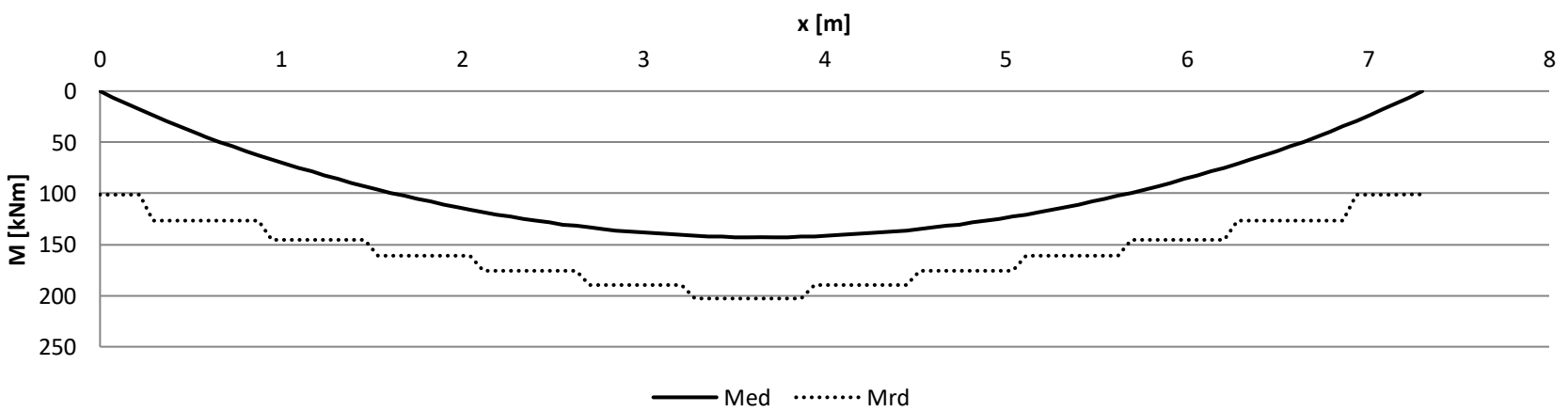
plastic zero line in the concrete slab, beam completely in tensile range → Cross-section class 1

Shear capacity

Web: vorh h_w/t = 20,6 < 72*ε = 72,0 --> no shear buckling analysis

NW: $V_{Ed}/V_{pl,a,Rd}$ = 0,40 <1; <0,5 → ρ= 0,00 --> shear force without affecting flexure

Torque bearing capacity (plastic under partial anchoring along the support shaft)



NW: max (M_{Ed}/M_{Rd}) = 0,76 < 1 at x = 2,7 m

Application limits for						partially anchoring	equidistant dowel distribution	
Cross-section class 1 or 2:							**fullfilled**	
Minimum dowel joint level	min η = 0,40	<	vorh η = 0,71				**fullfilled**	
∅ KBD:	16 mm	<	19 mm	<	25 mm	**fullfilled**	-	
$h_{KBD}/\varnothing_{KBD} \geq 4$:						**fullfilled**	-	

Longitudinal shear strength (Ansatz θ_f= 23,1°) mit : σ_{cd} = 0 N/mm² $V_{Rd,cc}$ = 617,2 kN V_{Ed} = 1265 kN [=$N_{pl,a,Rd}$] cotθ= 2,3

min e_{KBD}= Δx = 0,60 m ΔF_d = 150,0 kN $h_{f,a\text{-}a}$ = 206 mm --> $\nu_{Ed,a\text{-}a}$ = 1,21 N/mm²

$h_{f,b\text{-}b}$ = 261 mm --> $\nu_{Ed,b\text{-}b}$ = 0,96 N/mm²

Tension strut: s_f = 0,60 m A_{sf} = 3,14 cm² ρ_w = A_{sf} / (s_f * h_f) ν_{Rd} = (A_{sf} * f_{yd}/s_f) * cot θ_f/h_f

section a-a $\rho_{w,a\text{-}a}$ = 0,025 --> $\nu_{Rd,a\text{-}a}$ = 2,59 N/mm²

section b-b $\rho_{w,b\text{-}b}$ = 0,020 --> $\nu_{Rd,b\text{-}b}$ = 2,04 N/mm²

Compression strut ν_{Rd} = ν * f_{cd} * sin θ_f * cos θ_f = 5,52 N/mm²

NW: ν_{Ed}/ν_{Rd} = 0,47 < 1

Min. reinforcement $\rho_{w,min}$ = **0,001 < 0,020** = ρ_w

Thrust-bearing capacity of the joint: precast - grout (approach keyed joint) --> c = 0,5 μ = 0,9

ν_{Edi} = $\nu_{Ed,a\text{-}a}$ = 1,2 N/mm² ν_{Fuge} = 0,675

ν_{Rdi} = min(c * f_{ctd} + ρ * f_{yd} * 1,2 * μ; 0,5 * ν * f_{cd}) ; $f_{ctk;0,05}$ = 2,5 N/mm² --> f_{ctd} = 1,4 N/mm²

ν_{Rdi} = 7,7 N/mm² ρ = $\rho_{w,a\text{-}a}$= 0,025

NW: ν_{Edi}/ν_{Rdi} = 0,16 < 1 -

d

DIMENSIONING - steel girders during construction

Sectional classification, verification of shear force resistance see proof composite section

Torque bearing capacity

NW: max ($M_{Ed}/M_{pl,a,Rd}$) = 0,76 < 1

DEFORMATION CALCULATION open-air parking deck slab (Girder wirhout weight composite)

Content of the quasi-continuous load on the live load 40% creep coefficient ϕ_{unend} 2,16 (for expansion and quasi-continuous live load)

shrinkage strain ε_{cs} -0,0002

Material parameters for calculating the deformation

Concrete		**steel**		**Composite cross-section**				
E_{cm} =	35000 N/mm²	E_a =	210000 N/mm²	t = 0		n_0 =	6,00	
				t = ∞		α_A =	0,038	α_I = 0,561
					creep coefficient	$\psi_{A,L}$ =	1,043	$\psi_{I,L}$ = 1,954
						$n_{A,L}$ =	19,51	$n_{I,L}$ = 31,32
					shrinkage coefficient	$\psi_{A,L}$ =	0,507	$\psi_{I,L}$ = 0,597
						$n_{A,L}$ =	12,57	$n_{I,L}$ = 13,74

Profile characteristic for calculating the deformation

Concrete		**steel**		**Composite cross-section**					
A_c =	1972,45 cm²	A_a =	53,8 cm²	t = 0	A_{i0} =	383 cm²			
z_c =	5,2 cm	$z_{el,a}$ =	9,5 cm		z_{i0} =	7,2 cm			
I_c =	17438 cm⁴	I_a =	3709 cm⁴		I_{i0} =	16543 cm⁴			
				t = ∞		creepage			shrinkage
					$A_{i\infty,K}$ =	155 cm²		$A_{i\infty,S}$ =	211 cm²
					$z_{i\infty,K}$ =	10,2 cm		$z_{i\infty,S}$ =	8,9 cm
					$I_{i\infty;K}$ =	11804 cm⁴		$I_{i\infty;S}$ =	13581 cm⁴

Deformation component

Deformation content from height increase	-45 mm
Deformation content from slab dead weight (without joint I_a)	41 mm
Deformation content as a result of expansion and quasi-continuous load (with joint I_{i0})	3 mm
Deformation content from creepage coefficient as a result of expansion and quasi-continuous load $I_{i\infty,K}$)	1 mm
Deformation content from shrinkage (joint $I_{i\infty,S}$)	6 mm
Deformation content from live load (joint; I_{i0})	4 mm
Maximum total bending at x = 3,7 m	10 mm
equivalent to:	L / 722

DIMENSIONING - Steel beams at the edge with biaxial bending

The ramp girders are connected to the pillars using face plates.
Due to the ramp inclination of 8,5 °, the edge area of the steel girder
(from the column up to the first head bolt dowel) is subjected to biaxial bending stress.
The bending torque around the z-axis will only be allocated to the half of the profile on the concrete slab.
The ramp girders are provided with head bolts at a distance of approximately 60 cm from the middle of the pillar.

Verification at the first shear stud:

$W_{y,el}$ = 390,39 cm³ $W_{OG,el}$ = 66,7 cm³

M_{zd} = 83,5 kN * 0,6 m * sin 8,5 ° = 7,4 kNm

M_{yd} = 83,5 kN * 0,6 m * cos 8,5 °

\- 20,8 kN/m * 0,6 ² m² / 2 * cos 8,5 ° = 45,8 kNm

$\sigma_{e,d}$ = 4581,421 / 390,39 + 740,1 / 66,7

= 11,7 + 11,1 = 22,8 kN/cm²

NW: $\sigma_{Ed}/f_{y,d}$ = 0,97 < 1

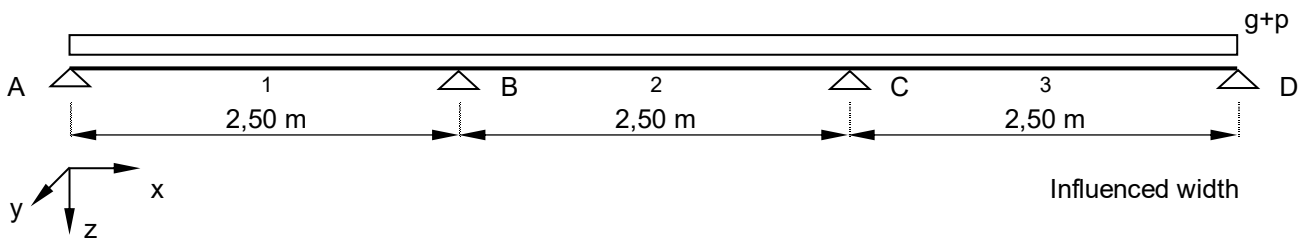
Pos BE312 Steel girder L = 7,50 m

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pavement transition

SYSTEM AND LOADS



Influenced width 1,25 m

constant loads	γ = 1,35			
concrete slab		2,58 kN/m² * 1,25 m =		3,23 kN/m
Additional load + Installation		0,65 kN/m² * 1,25 m =		0,81 kN/m
Dead load				0,36 kN/m
			g =	4,39 kN/m
variable loads	γ = 1,5			
Live loads		2,50 kN/m² * 1,25 m =		3,13 kN/m
The variable loads act only on individual spans.			p =	3,13 kN/m
			$q_{z,Ed}$ =	10,62 kN/m

INTERNAL FORCES

Support forces

$A_{g,k}$ =	**4,4 kN**	$B_{g,k}$ =	**12,1 kN**	$C_{g,k}$ =	**12,1 kN**	$D_{g,k}$ =	**4,4 kN**
$A_{p,k}$ =	**3,5 kN**	$B_{p,k}$ =	**9,4 kN**	$C_{p,k}$ =	**9,4 kN**	$D_{p,k}$ =	**3,5 kN**
$A_{p,k,min}$ =	-0,4 kN	$B_{p,k,min}$ =	-0,8 kN	$C_{p,k,min}$ =	-0,8 kN	$D_{p,k,min}$ =	-0,4 kN
A_{Ed} =	**11,2 kN**	B_{Ed} =	**30,4 kN**	C_{Ed} =	**30,4 kN**	D_{Ed} =	**11,2 kN**

governing internal forces

$M_{B,Ed}$ =	-7,1 kNm	$M_{C,Ed}$ =	-7,1 kNm	$M_{1,Ed}$ =	5,9 kNm
$V_{B,Ed,li}$ =	-16,1 kN	$V_{C,Ed,li}$ =	-14,2 kN	$M_{2,Ed}$ =	3,1 kNm
$V_{B,Ed,re}$ =	14,2 kN	$V_{C,Ed,re}$ =	16,1 kN	$M_{3,Ed}$ =	5,9 kNm

DIMENSIONING

selected: **HE 180 A S235** **EXC2**

Cross-section class 1

γ_{M0} = 1,00

Verification according 6.2.1 (6.1)

x [m]	$N_{x,Ed}$ [kN]	$M_{y,Ed}$ [kNm]	$V_{z,Ed}$ [kN]	$M_{z,Ed}$ [kNm]	$V_{y,Ed}$ [kN]	σ_v [kN/cm²]	τ [kN/cm²]	η
0,00	0,0	0,0	11,2	0,0	0,0	2,09	1,21	**0,09**
1,06	0,0	5,9	0,0	0,0	0,0	2,01	0,00	**0,09**
2,50	0,0	-7,1	-16,1	0,0	0,0	3,30	1,62	**0,14**
3,75	0,0	3,1	0,0	0,0	0,0	1,06	0,00	**0,05**
5,00	0,0	-7,1	16,1	0,0	0,0	3,30	1,62	**0,14**
6,44	0,0	5,9	0,0	0,0	0,0	2,01	0,00	**0,09**
7,50	0,0	0,0	-11,2	0,0	0,0	2,09	1,21	**0,09**

Deformation

field	f_z [cm]	L/300 [cm]
1	0,00	0,83
2	0,00	0,83
3	0,00	0,83

Nachweis nach 6.2.1 (6.2)

x [m]	$N_{x,Ed}$ [kN]	$M_{y,Ed}$ [kNm]	$V_{z,Ed}$ [kN]	$M_{z,Ed}$ [kNm]	$V_{y,Ed}$ [kN]	$N_{pl,Rd}$ [kN]	$M_{pl,y,Rd}$ [kNm]	$V_{pl,z,Rd}$ [kN]	$M_{pl,z,Rd}$ [kNm]	$V_{pl,y,Rd}$ [kN]	η
0,00	0,0	0,0	11,2	0,0	0,0	1063,4	76,3	196,3	36,8	417,6	**0,06**
1,06	0,0	5,9	0,0	0,0	0,0	1063,4	76,3	196,3	36,8	417,6	**0,08**
2,50	0,0	-7,1	-16,1	0,0	0,0	1063,4	76,3	196,3	36,8	417,6	**0,09**
3,75	0,0	3,1	0,0	0,0	0,0	1063,4	76,3	196,3	36,8	417,6	**0,04**
5,00	0,0	-7,1	16,1	0,0	0,0	1063,4	76,3	196,3	36,8	417,6	**0,09**
6,44	0,0	5,9	0,0	0,0	0,0	1063,4	76,3	196,3	36,8	417,6	**0,08**
7,50	0,0	0,0	-11,2	0,0	0,0	1063,4	76,3	196,3	36,8	417,6	**0,06**

stability check, see next page η = **0,12**

Pos BE312 Steel girder L = 7,50 m

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STABILITY CHECK

HE 180 A S235 (cross-section class 1)

load application at top chord z_p = -8,6 cm The beam is not laterally supported

γ_{M1} = 1,10

Lateral-torsional buckling

buckling mode b α_{LT} = 0,34

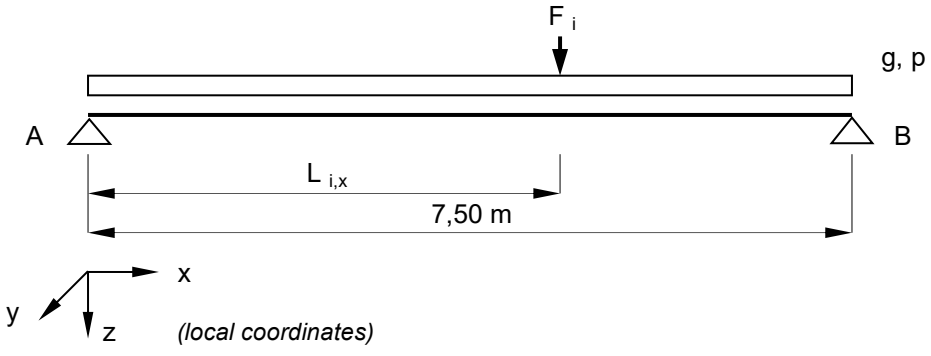
Feld	$L_{cr,y}$ [m]	$L_{cr,z}$ [m]	$M_{y,Ed}$ [kNm]	$N_{cr,z}$ [kN]	ζ_0	α_{cr}	M_{cr} [kNm]	λ_{LT}	Φ_{LT}	χ_{LT}	k_c	f	$\chi_{LT,mod}$	$M_{b,Rd}$ [kNm]	η
1	2,50	2,50	7,1	3066,2	2,06	40,80	290,7	0,51	0,62	0,96	0,91	0,96	1,00	69,2	**0,10**
2	2,50	2,50	8,3	3066,2	5,15	52,25	433,4	0,42	0,57	0,99	0,90	0,95	1,00	69,4	**0,12**
3	2,50	2,50	7,1	3066,2	2,06	40,80	290,7	0,51	0,62	0,96	0,91	0,96	1,00	69,2	**0,10**

$\eta = M_{y,Ed} / M_{b,Rd}$

Pos BE402 Transfer Beam L = 7,5m

Ramp transfer beam

SYSTEM AND LOADS



Floor beam spacing of 2,50 m Grid

Influence width (line load): 1,25 m
Rotation of the member a α = 0,0 °

For the connecting floor beams
Approach PH loads according to Tab. 6.8 EN (NA 201
Σ Length of floor beams at Point 1 32,0 m
S Länge Deckenträger Stelle 2: 22,0 m

load influence area A_{EF}: 6.3.1.2 (135 m²

Area loads **Point loads** q_k = 2,50 kN/m²

$g_{0,k}$ - Permanent loads (characteristic)

Concrete slab	0,103 m * 25 kN/m³ * 1,25 m =	3,2 kN/m	
Self weight beam:		0,8 kN/m	
	g =	4,1 kN/m	

$p_{,k}$ - variable loads (characteristic)

Live Loads	2,50 kN/m² * 1,25 m =	3,1 kN/m	
Installation	0,15 kN/m² * 1,25 m =	0,2 kN/m	
	p =	3,3 kN/m	

i	$F_{i,g}$ [kN]	$F_{i,p}$ [kN]	$L_{i,x}$ [m]	*Remarks*
1	41,6	34,8	2,50	*from Pos. BE034*
2	41,6	34,8	5,00	*from Pos. BE034*
3	2,8		0,55	*from curb*
4	2,8		6,95	*from curb*
5				

Normal force in beam $N_{x,Ed}$ = 0 kN

INTERNAL FORCES

characteristic bearing reactions:

	A_g	=	**59,6 kN**		B_g	=	**59,6 kN**
	A_p	=	**47,2 kN**		B_p	=	**47,2 kN**
	A_{Ed}	=	**151,4 kN**		B_{Ed}	=	**151,4 kN**
	$V_{Ed,re}$	=	151,4 kN		$V_{Ed,li}$	=	-151,4 kN
	max M_{Ed}	=	346,5 kNm	(bei x= 3,75m)	max | $V_{z,Ed}$ |	=	151,4 kN
					max | $V_{y,Ed}$ |	=	0,0 kN

γ_G = 1,35
γ_Q = 1,50

governing design internal forces:

$M_{y,Ed}$	=	**335,3 kNm**	(at x= 2,48m)	$M_{z,Ed}$	=	0,0 kNm
$V_{z,Ed}$	=	**121,7 kN**		$V_{y,Ed}$	=	0,0 kN

DESIGN

welded beam

selected: **Schweißprofil S 355** **EXC2**

Section classification 1
γ_M = 1,10

Define Profile:

profile height:	h =	**360**	**mm**	Flange width:	bf =	**190**	**mm**
web thickness:	tw =	**10**	**mm**	Flange thickness	tf =	**20**	**mm**

Pos BE402 Transfer Beam L = 7,5m

Ramp transfer beam

Profile parameters (for M)

$M_{pl,y,Rd}$	=	500	kNm
$M_{pl,z,Rd}$	=	119	kNm
$N_{pl,Rd}$	=	3485	kN
$V_{pl,y,Rd}$	=	1342	kN
$V_{pl,z,Rd}$	=	671	kN

Profile parameters (for M + V)

$M_{V,y,Rd}$	=	500	kNm
$M_{V,z,Rd}$	=	119	kNm
$N_{V,Rd}$	=	3485	kN

Profile parameters (for M + V + N)

$M_{V,N,y,Rd}$	=	500	kNm
$M_{V,N,z,Rd}$	=	119	kNm

Verifications

$V_{y,Ed} / V_{pl,y,Rd}$	=	0,00 < 1
$V_{z,Ed} / V_{pl,z,Rd}$	=	**0,18 < 1**
$M_{y,Ed} / M_{V,N,y,Rd}$	=	**0,67 < 1**

Verification **only** for biaxial bending (incl. normal force): $(M_{y,Ed} / M_{V,N,y,Rd})^{\alpha} + (M_{z,Ed} / M_{V,N,z,Rd})^{\beta}$

Camber (z): **1,0 cm**

Final deflection (x= 3,75m) : fz = 1,8 cm = L / 410

Stability verifications see following page

Utilization: **0,75 < 1**

Pos BE402 Transfer Beam L = 7,5m

VERIFICATION OF STABILITY

Schweißprofil S 355 Cross section class 1

L	=	7,50	m	Lateral load	on the upper chord		
N_{Ed}	=	0	kN		z_p	=	-18,0 cm
$V_{y,Ed}$	=	0	kN				
$V_{z,Ed}$	=	122	kN				
$M_{y,Ed}$	=	335,3	kNm	**The beam is**	**supported at the 1/3-points**		
$M_{z,Ed}$	=	0,0	kNm				
γ_{M1}	=	1,10					

Buckling around the z-z axis

Buckling line	c		
α	=	0,49	
Buckling length	=	250	cm
$N_{cr,z}$	=	7591	kN
λ_z'	=	0,71	
Φ	=	0,88	
χ_z	=	0,72	
$N_{b,z,Rd}$	=	2503	kN
$N_{Ed} / N_{b,z,Rd}$	=	**0,00 < 1**	

buckling around the y-y axis

Buckling line	b		
α	=	0,34	
Buckling length	=	750	cm
$N_{cr,y}$	=	9108	kN
λ_y'	=	0,65	
Φ	=	0,79	
χ_y	=	0,81	
$N_{b,y,Rd}$	=	2830	kN
$N_{Ed} / N_{b,y,Rd}$	=	**0,00 < 1**	

lateral torsional buckling without axial force

Buckling line	c		
α_{LT}	=	0,49	
c^2	=	408	cm^2
$N_{cr,z}$	=	7591	kN
M_{cr}	=	155598	kNcm
λ_{LT}'	=	0,59	
$\lambda_{LT,0}'$	=	0,40	
β	=	0,75	
Φ_{LT}	=	0,68	
χ_{LT}	=	0,89	
k_c	=	0,99	
f	=	0,99	
$\chi_{LT,mod}$	=	0,89	
$M_{b,Rd}$	=	44694	kNcm
$M_{z,Rd}$	=	11909	kNcm
$M_{y,Ed} / M_{b,Rd}$	=	**0,75 < 1**	

lateral torsional buckling with axial force

n_y	=	0,00
n_z	=	0,00
λ_y'	=	0,65
λ_z'	=	0,71
C_{my}	=	1,00
C_{mz}	=	1,00
C_{mLT}	=	1,00
k_{yy}	=	1,00
k_{yz}	=	0,60
k_{zy}	=	1,00
k_{zz}	=	1,00

$N_{Ed} / N_{b,z,Rd} + k_{zy} * M_{y,Ed} / M_{b,Rd} + k_{zz} * M_{z,Ed} / M_{z,Rd}$

= 0,00 + 0,75 + 0,00 = **0,75 < 1**

$N_{Ed} / N_{b,y,Rd} + k_{yy} * M_{y,Ed} / M_{b,Rd} + k_{yz} * M_{z,Ed} / M_{z,Rd}$

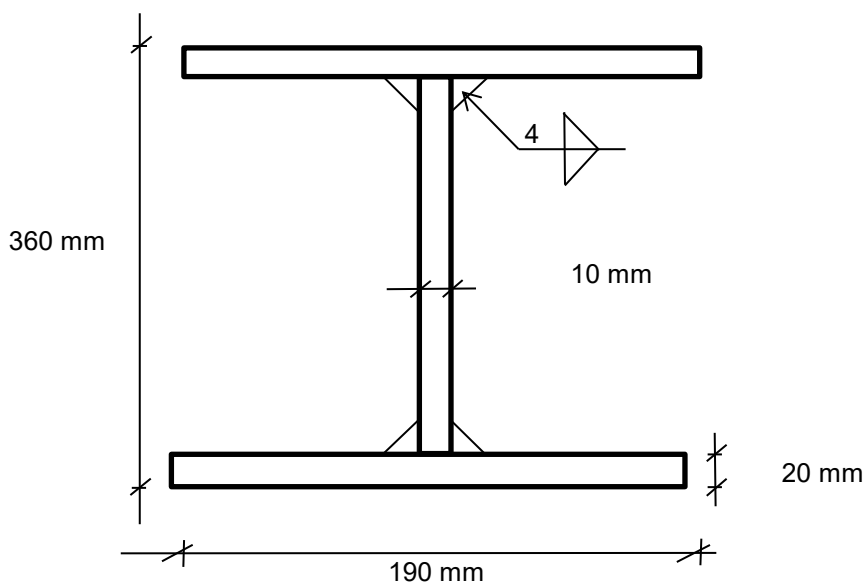
= 0,00 + 0,75 + 0,00 = **0,75 < 1**

BE402 Welded beam: Weld seam

V_{Ed} = 151,4 kN

$\tau_{II} = \frac{V * S_{Fl}}{I * \sum a}$

Welded profile



I	=	24720,0	cm^4
S_{Fl}	=	646,0	cm^3
a	=	0,4	cm

S 355

$\tau_{II} = \frac{151 * 646,0}{24720 * 0,8} = 4,94\ kN/cm^2 \leq \frac{\frac{f_u}{\sqrt{3}}}{\beta_w * \gamma_{m2}} = F_{w,Rd}$

$\tau_{II} = 4,94\ kN/cm^2 \leq \frac{\frac{f_u}{\sqrt{3}}}{\beta_w * \gamma_{m2}} = 25,10\ kN/cm^2$

$\frac{\tau_{II}}{F_{w,Rd}} = \frac{4,94\ kN/cm^2}{25,10\ kN/cm^2} = 0,20 < 1$

Weld seam

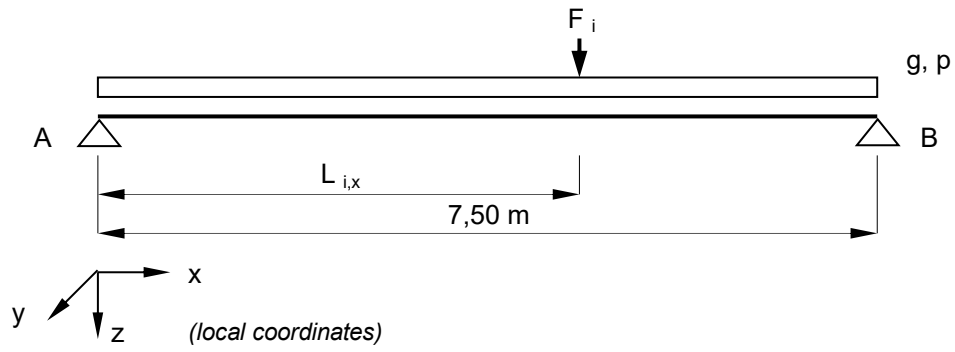
a_{max} = 4 mm ≤ a ≤ 0,7 * t_{min} = 7

$a_{min} = \sqrt{\max t}$ - 0,5 = 4

Pos BE412 Transfer Beam L = 7,5m

ramp end

SYSTEM AND LOADS



Area loads

$g_{0,k}$ - Permanent loads (characteristic)

Concrete slab	0,103 m * 25 kN/m³ * 0,00 m =		kN/m
Self weight beam:		0,8	kN/m
	g =	0,8	kN/m

$p_{,k}$ - variable loads (characteristic)

Live Loads	2,50 kN/m² * 0,00 m =		kN/m
Installation	0,15 kN/m² * 0,00 m =		kN/m
	p =	0,0	kN/m

Floor beam spacing of 2,50 m Grid

Influence width (line load): m

Rotation of the member a: α = 0,0 °

For the connecting floor beams

Approach PH loads according to Tab. 6.8 EN (NA 2014-

Σ Length of floor beams at Point 1 32,0 m

S Länge Deckenträger Stelle 2: 22,0 m

load influence area A_{EF}: 6.3.1.2 (135 m²

q_k = 2,50 kN/m²

Point loads

i	$F_{i,g}$ [kN]	$F_{i,p}$ [kN]	$L_{i,x}$ [m]	*Remarks*
1	41,6	34,8	2,50	*from Pos. BE034*
2	41,6	34,8	5,00	*from Pos. BE034*
3	2,0		2,50	*from column stub*
4	2,0		5,00	*from column stub*
5				

Normal force in beam $N_{x,Ed}$ = 0 kN

INTERNAL FORCES

characteristic bearing reactions:

A_g = **46,8 kN** B_g = **46,8 kN**

A_p = **34,8 kN** B_p = **34,8 kN**

γ_G = 1,35

γ_Q = 1,50

A_{Ed} = **115,4 kN** B_{Ed} = **115,4 kN**

$V_{Ed,re}$ = 115,4 kN $V_{Ed,li}$ = -115,4 kN

max M_{Ed} = 285,7 kNm (at x= 3,75m) max | $V_{z,Ed}$ | = 115,4 kN

max | $V_{y,Ed}$ | = 0,0 kN

governing design internal forces:

$M_{y,Ed}$ = **282,0 kNm** (at x= 2,48m) $M_{z,Ed}$ = 0,0 kNm

$V_{z,Ed}$ = **112,5 kN** $V_{y,Ed}$ = 0,0 kN

DESIGN

welded beam

selected: **Schweißprofil S 355** **EXC2** Section classification 1

γ_M = 1,10

Define Profile:

profile height: h = **360 mm** Flange width: bf = **190 mm**

web thickness: tw = **10 mm** Flange thickness: tf = **20 mm**

Profile parameters (for M)

$M_{pl,y,Rd}$ = 500 kNm

$M_{pl,z,Rd}$ = 119 kNm

$N_{pl,Rd}$ = 3485 kN

$V_{pl,y,Rd}$ = 1342 kN

$V_{pl,z,Rd}$ = 671 kN

Profile parameters (for M + V)

$M_{V,y,Rd}$ = 500 kNm

$M_{V,z,Rd}$ = 119 kNm

$N_{V,Rd}$ = 3485 kN

Profile parameters (for M + V + N)

$M_{V,N,y,Rd}$ = 500 kNm

$M_{V,N,z,Rd}$ = 119 kNm

Verifications

$V_{y,Ed}$ / $V_{pl,y,Rd}$ = 0,00 < 1

$V_{z,Ed}$ / $V_{pl,z,Rd}$ = **0,17 < 1**

$M_{y,Ed}$ / $M_{V,N,y,Rd}$ = **0,56 < 1**

Verification **only** for biaxial bending (incl. normal force): $(M_{y,Ed} / M_{V,N,y,Rd})^{\alpha} + (M_{z,Ed} / M_{V,N,z,Rd})^{\beta}$

Camber (z): **1,0 cm**

Final deflection (x= 3,75m) : fz = 1,3 cm = L / 560

Stability verifications see following page

Utilization: **0,63 < 1**

Pos BE412 Transfer Beam L = 7,5m

VERIFICATION OF STABILITY

Schweißprofil S 355 Cross section class 1

L	=	7,50	m	Lateral load		on the upper chord	
N_{Ed}	=	0	kN		z_p	=	-18,0 cm
$V_{y,Ed}$	=	0	kN				
$V_{z,Ed}$	=	113	kN				
$M_{y,Ed}$	=	282,0	kNm	**The beam is**	**supported at the 1/3-points**		
$M_{z,Ed}$	=	0,0	kNm				
γ_{M1}	=	1,10					

Buckling around the z-z axis

Buckling line	c		
α	=	0,49	
Buckling length	=	250	cm
$N_{cr,z}$	=	7591	kN
λ_z'	=	0,71	
Φ	=	0,88	
χ_z	=	0,72	
$N_{b,z,Rd}$	=	2503	kN
$N_{Ed} / N_{b,z,Rd}$	=	**0,00 < 1**	

buckling around the y-y axis

Buckling line	b		
α	=	0,34	
Buckling length	=	750	cm
$N_{cr,y}$	=	9108	kN
λ_y'	=	0,65	
Φ	=	0,79	
χ_y	=	0,81	
$N_{b,y,Rd}$	=	2830	kN
$N_{Ed} / N_{b,y,Rd}$	=	**0,00 < 1**	

lateral torsional buckling without axial force

Buckling line	c		
α_{LT}	=	0,49	
c^2	=	408	cm^2
$N_{cr,z}$	=	7591	kN
M_{cr}	=	154692	kNcm
λ_{LT}'	=	0,60	
$\lambda_{LT,0}'$	=	0,40	
β	=	0,75	
Φ_{LT}	=	0,68	
χ_{LT}	=	0,89	
k_c	=	1,00	
f	=	1,00	
$\chi_{LT,mod}$	=	0,89	
$M_{b,Rd}$	=	44483	kNcm
$M_{z,Rd}$	=	11909	kNcm
$M_{y,Ed} / M_{b,Rd}$	=	**0,63 < 1**	

lateral torsional buckling with axial force

n_y	=	0,00
n_z	=	0,00
λ_y'	=	0,65
λ_z'	=	0,71
C_{my}	=	1,00
C_{mz}	=	1,00
C_{mLT}	=	1,00
k_{yy}	=	1,00
k_{yz}	=	0,60
k_{zy}	=	1,00
k_{zz}	=	1,00

$N_{Ed} / N_{b,z,Rd} + k_{zy} * M_{y,Ed} / M_{b,Rd} + k_{zz} * M_{z,Ed} / M_{z,Rd}$

= 0,00 + 0,63 + 0,00 = **0,63 < 1**

$N_{Ed} / N_{b,y,Rd} + k_{yy} * M_{y,Ed} / M_{b,Rd} + k_{yz} * M_{z,Ed} / M_{z,Rd}$

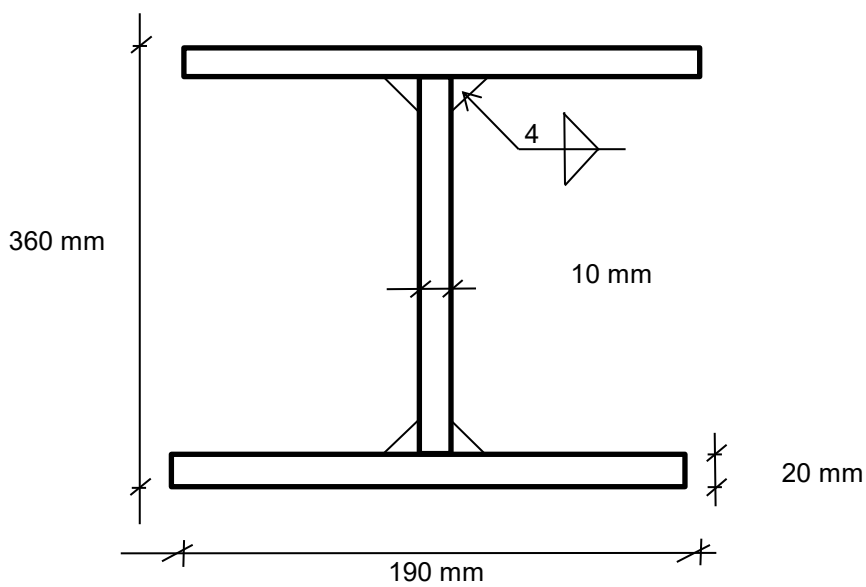
= 0,00 + 0,63 + 0,00 = **0,63 < 1**

BE412 Welded beam: Weld seam

V_{Ed} = 115,4 kN

$\tau_{II} = \frac{V * S_{Fl}}{I * \sum a}$

Welded profile



I	=	24720,0	cm^4
S_{Fl}	=	646,0	cm^3
a	=	0,4	cm

S 355

$\tau_{II} = \frac{115 * 646,0}{24720 * 0,8} = 3,77\ kN/cm^2 \leq \frac{\frac{f_u}{\sqrt{3}}}{\beta_w * \gamma_{m2}} = F_{w,Rd}$

$\tau_{II} = 3,77\ kN/cm^2 \leq \frac{\frac{f_u}{\sqrt{3}}}{\beta_w * \gamma_{m2}} = 25,10\ kN/cm^2$

$\frac{\tau_{II}}{F_{w,Rd}} = \frac{3,77\ kN/cm^2}{25,10\ kN/cm^2} = 0,15 < 1$

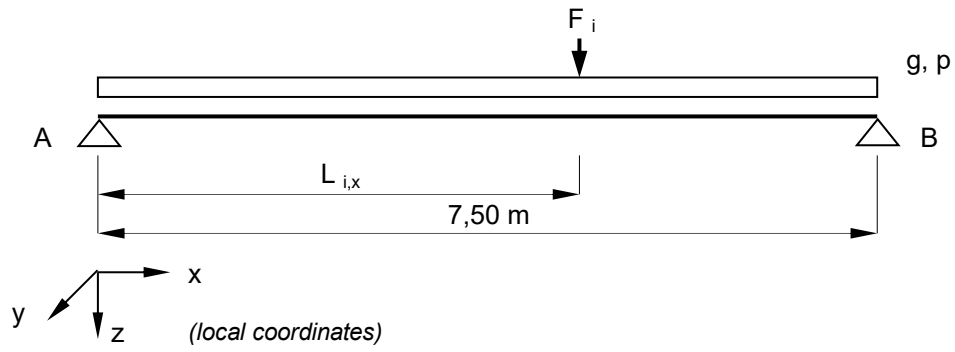
Weld seam

a_{max} = 4 mm ≤ a ≤ 0,7 * t_{min} = 7

$a_{min} = \sqrt{\max t}$ - 0,5 = 4

Pos BE450 Transfer Beam L = 7,5m

SYSTEM AND LOADS



Area loads

$g_{0,k}$ - Permanent loads (characteristic)

Concrete slab	0,103 m * 25 kN/m³ * 0,00 m =		kN/m
Self weight beam:		1,1	kN/m
	g =	1,1	kN/m

$p_{,k}$ - variable loads (characteristic)

Live Loads	2,50 kN/m² * 0,00 m =		kN/m
Installation	0,15 kN/m² * 0,00 m =		kN/m
	p =	0,0	kN/m

Floor beam spacing of 2,50 m Grid

Influence width (line load): m

Rotation of the member a: α = 0,0 °

For the connecting floor beams

Approach PH loads according to Tab. 6.8 EN (NA 2014-

Σ Length of floor beams at Point 1 32,0 m

S Länge Deckenträger Stelle 2: 22,0 m

load influence area A_{EF}: 6.3.1.2 (135 m²

Point loads q_k = 2,50 kN/m²

i	$F_{i,g}$ [kN]	$F_{i,p}$ [kN]	$L_{i,x}$ [m]	*Remarks*
1	41,6	34,8	2,50	*from Pos. BE034*
2	60,3	50,4	2,50	*from Pos. BE052*
3	41,6	34,8	5,00	*from Pos. BE034*
4	60,3	50,4	5,00	*from Pos. BE052*
5				

Normal force in beam $N_{x,Ed}$ = 0 kN

INTERNAL FORCES

characteristic bearing reactions:	A_g = **106,1 kN**			B_g = **106,1 kN**		
	A_p = **85,2 kN**			B_p = **85,2 kN**		
γ_G = 1,35	A_{Ed} = **271,0 kN**			B_{Ed} = **271,0 kN**		
γ_Q = 1,50	$V_{Ed,re}$ = 271,0 kN			$V_{Ed,li}$ = -271,0 kN		
	max M_{Ed} = 674,0 kNm	(at x= 3,75m)		max | $V_{z,Ed}$ | = 271,0 kN		
				max | $V_{y,Ed}$ | = 0,0 kN		
governing design internal forces:	$M_{y,Ed}$ = **666,1 kNm**	(at x= 2,48m)		$M_{z,Ed}$ = 0,0 kNm		
	$V_{z,Ed}$ = **267,3 kN**			$V_{y,Ed}$ = 0,0 kN		

DESIGN

welded beam

selected: **Schweißprofil S 355** **EXC2**

Section classification 1

γ_M = 1,00

Define Profile:

profile height:	h =	**400 mm**	Flange width:	bf =	**220 mm**
web thickness:	tw =	**15 mm**	Flange thickness:	tf =	**20 mm**

Profile parameters (for M)

$M_{pl,y,Rd}$	=	766 kNm
$M_{pl,z,Rd}$	=	179 kNm
$N_{pl,Rd}$	=	5041 kN
$V_{pl,y,Rd}$	=	1701 kN
$V_{pl,z,Rd}$	=	1209 kN

Profile parameters (for M + V)

$M_{V,y,Rd}$	=	766 kNm
$M_{V,z,Rd}$	=	179 kNm
$N_{V,Rd}$	=	5041 kN

Profile parameters (for M + V + N)

$M_{V,N,y,Rd}$	=	766 kNm
$M_{V,N,z,Rd}$	=	179 kNm

Verifications

$V_{y,Ed}$ / $V_{pl,y,Rd}$	=	0,00 < 1
$V_{z,Ed}$ / $V_{pl,z,Rd}$	=	**0,22 < 1**
$M_{y,Ed}$ / $M_{V,N,y,Rd}$	=	**0,87 < 1**

Verification **only** for biaxial bending (incl. normal force): $(M_{y,Ed} / M_{V,N,y,Rd})^{\alpha} + (M_{z,Ed} / M_{V,N,z,Rd})^{\beta}$

Camber (z): **2,0 cm**

Final deflection (x= 3,75m) : fz = 1,6 cm = L / 470

Stability verifications see following page

Utilization: **0,95 < 1**

Pos BE450 Transfer Beam L = 7,5m

VERIFICATION OF STABILITY

Schweißprofil S 355 Cross section class 1

L	=	7,50	m	Lateral load		on the upper chord	
N_{Ed}	=	0	kN		z_p	=	-20,0 cm
$V_{y,Ed}$	=	0	kN				
$V_{z,Ed}$	=	267	kN				
$M_{y,Ed}$	=	666,1	kNm	**The beam is**	**supported at the 1/3-points**		
$M_{z,Ed}$	=	0,0	kNm				
γ_{M1}	=	1,00					

Buckling around the z-z axis

Buckling line	c		
α	=	0,49	
Buckling length	=	250	cm
$N_{cr,z}$	=	11804	kN
λ_z'	=	0,65	
Φ	=	0,82	
χ_z	=	0,75	
$N_{b,z,Rd}$	=	3797	kN
$N_{Ed} / N_{b,z,Rd}$	=	**0,00 < 1**	

buckling around the y-y axis

Buckling line	b		
α	=	0,34	
Buckling length	=	750	cm
$N_{cr,y}$	=	13865	kN
λ_y'	=	0,60	
Φ	=	0,75	
χ_y	=	0,84	
$N_{b,y,Rd}$	=	4212	kN
$N_{Ed} / N_{b,y,Rd}$	=	**0,00 < 1**	

lateral torsional buckling without axial force

Buckling line	c		
α_{LT}	=	0,49	
c^2	=	468	cm^2
$N_{cr,z}$	=	11804	kN
M_{cr}	=	257532	kNcm
λ_{LT}'	=	0,55	
$\lambda_{LT,0}'$	=	0,40	
β	=	0,75	
Φ_{LT}	=	0,65	
χ_{LT}	=	0,92	
k_c	=	1,00	
f	=	1,00	
$\chi_{LT,mod}$	=	0,92	
$M_{b,Rd}$	=	70462	kNcm
$M_{z,Rd}$	=	17901	kNcm
$M_{y,Ed} / M_{b,Rd}$	=	**0,95 < 1**	

lateral torsional buckling with axial force

n_y	=	0,00
n_z	=	0,00
λ_y'	=	0,60
λ_z'	=	0,65
C_{my}	=	1,00
C_{mz}	=	1,00
C_{mLT}	=	1,00
k_{yy}	=	1,00
k_{yz}	=	0,60
k_{zy}	=	1,00
k_{zz}	=	1,00

$N_{Ed} / N_{b,z,Rd} + k_{zy} * M_{y,Ed} / M_{b,Rd} + k_{zz} * M_{z,Ed} / M_{z,Rd}$

= 0,00 + 0,95 + 0,00 = **0,95 < 1**

$N_{Ed} / N_{b,y,Rd} + k_{yy} * M_{y,Ed} / M_{b,Rd} + k_{yz} * M_{z,Ed} / M_{z,Rd}$

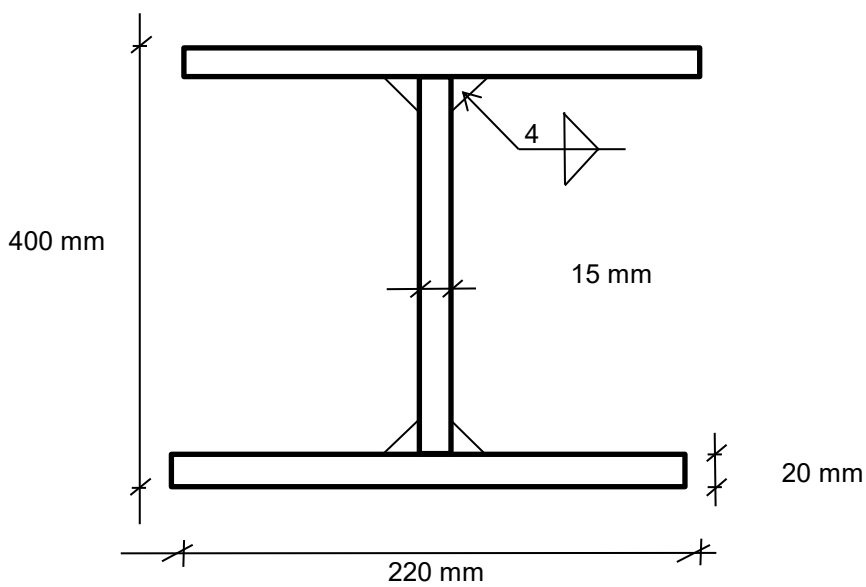
= 0,00 + 0,95 + 0,00 = **0,95 < 1**

BE450 Welded beam: Weld seam

V_{Ed} = 271,0 kN

$\tau_{II} = \frac{V * S_{Fl}}{I * \sum a}$

Welded profile



I	=	37629,3	cm^4
S_{Fl}	=	836,0	cm^3
a	=	0,4	cm

S 355

$\tau_{II} = \frac{271 * 836,0}{37629,3 * 0,8} = 7,53\ kN/cm^2 \leq \frac{\frac{f_u}{\sqrt{3}}}{\beta_w * \gamma_{m2}} = F_{w,Rd}$

$\tau_{II} = 7,53\ kN/cm^2 \leq \frac{\frac{f_u}{\sqrt{3}}}{\beta_w * \gamma_{m2}} = 25,10\ kN/cm^2$

$\frac{\tau_{II}}{F_{w,Rd}} = \frac{7,53\ kN/cm^2}{25,10\ kN/cm^2} = 0,30 < 1$

Weld seam

a_{max} = 4 mm ≤ a ≤ 0,7 * t_{min} = 11

$a_{min} = \sqrt{\max t}$ - 0,5 = 4

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MODEL

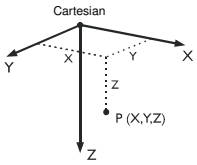
Project: PR0460 Model: 04-BE451_Truss beam 3 fields Date: 03.09.2025

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MODEL - GENERAL DATA

General	Model name	:	04-BE451_Truss beam 3 fields
	Type of model	:	3D
	Positive direction of global axis Z	:	Downward
	Classification of load cases and combinations	:	According to Standard: EN 1990 National Annex: CSN - Czech Republic
Options	☐ Use CQC Rule		
	☐ Enable CAD/BIM model		
	Standard Gravity		
	g	:	10.00 m/s²



1.1 NODES

Node No.	Reference Node	Coordinate System	Node Coordinates X [m]	Y [m]	Z [m]	Comment
1	3	Cartesian	0.00	0.00	-0.60	
2	3	Cartesian	0.00	0.00	-0.15	
3	-	Cartesian	12.09	0.00	0.00	Supported
4	2	Cartesian	0.00	0.10	0.00	Supported
5	6	Cartesian	0.00	0.00	-0.30	
6	9	Cartesian	0.00	0.00	-2.95	
7	9	Cartesian	0.00	0.00	-2.50	
8	9	Cartesian	0.00	0.00	-0.15	
9	3	Cartesian	2.50	0.00	0.00	
10	6	Cartesian	0.00	0.10	0.00	Supported
11	8	Cartesian	0.00	0.10	0.00	Supported
12	13	Cartesian	0.00	0.00	-0.30	
13	16	Cartesian	0.00	0.00	-2.95	
14	16	Cartesian	0.00	0.00	-2.50	
15	16	Cartesian	0.00	0.00	-0.15	
16	3	Cartesian	5.00	0.00	0.00	
17	13	Cartesian	0.00	0.10	0.00	Supported
18	15	Cartesian	0.00	0.10	0.00	Supported
19	21	Cartesian	0.00	0.00	-0.60	
20	21	Cartesian	0.00	0.00	-0.15	
21	16	Cartesian	2.50	0.00	0.00	Supported
22	20	Cartesian	0.00	0.10	0.00	Supported

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
3	Steel S 355 | DIN EN 1993-1-1:2010-12					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic

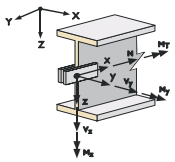
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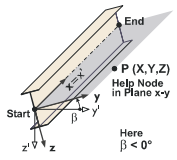
1.3 CROSS-SECTIONS

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
10	HEB 180							
	3	42.16 / 65.25	3831.00 / 42.05	1363.00 / 12.98	0.0	0.0	180.0	180.0



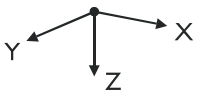
1.4 MEMBER HINGES

Release No.	Reference System	Force Release or Spring [kN/m] u_x	u_y	u_z	Moment Release or Spring [kNm/rad] φ_x	φ_y	φ_z
1	Local x,y,z	☐	☐	☐	☐	☒	☒
	Nonlinearity	-	-	-	-	-	-



1.7 MEMBERS

Mbr. No.	Member	Node Start	Node End	Rotation Type	Rotation β [°]	Cross-Section Start	Cross-Section End	Hinge No. Start	Hinge No. End	Ecc. No.	Div. No.	Length L [m]	
1	Beam	2	1	Angle	90.0	10	10	-	-	-	-	0.45	Z
2	Beam	3	2	Angle	90.0	10	10	-	-	15	-	0.25	Z
3	Beam	4	2	Angle	0.0	9	9	-	-	25	-	0.01	Y
4	Beam	3	7	Angle	90.0	10	10	1	1	19	-	3.21	XZ
5	Beam	3	9	Angle	90.0	10	10	-	-	14	-	2.40	X
6	Beam	6	5	Angle	90.0	10	10	-	-	-	-	0.30	Z
7	Beam	7	6	Angle	90.0	10	10	-	-	-	-	0.45	Z
8	Beam	8	7	Angle	90.0	10	10	-	-	-	-	2.35	Z
9	Beam	9	8	Angle	90.0	10	10	-	-	28	-	0.05	Z
10	Beam	10	6	Angle	0.0	9	9	-	-	25	-	0.01	Y
11	Beam	11	8	Angle	0.0	9	9	-	-	25	-	0.01	Y
12	Beam	7	16	Angle	90.0	10	10	1	1	20	-	3.21	XZ
13	Beam	7	14	Angle	90.0	10	10	-	-	-	-	2.50	X
14	Beam	9	16	Angle	90.0	10	10	-	-	-	-	2.50	X
15	Beam	13	12	Angle	90.0	10	10	-	-	-	-	0.30	Z
16	Beam	14	13	Angle	90.0	10	10	-	-	-	-	0.45	Z
17	Beam	15	14	Angle	90.0	10	10	-	-	-	-	2.35	Z
18	Beam	16	15	Angle	90.0	10	10	-	-	28	-	0.05	Z
19	Beam	17	13	Angle	0.0	9	9	-	-	25	-	0.01	Y
20	Beam	18	15	Angle	0.0	9	9	-	-	25	-	0.01	Y
21	Beam	16	21	Angle	90.0	10	10	-	-	13	-	2.40	X
22	Beam	14	21	Angle	90.0	10	10	1	1	20	-	3.21	XZ
23	Beam	20	19	Angle	90.0	10	10	-	-	-	-	0.45	Z
24	Beam	21	20	Angle	90.0	10	10	-	-	15	-	0.25	Z
25	Beam	22	20	Angle	0.0	9	9	-	-	25	-	0.01	Y



1.8 NODAL SUPPORTS

Support No.	Nodes No.	Sequen.	Rotation [°] about X	about Y	about Z	Column in Z	Support Conditions $u_{X'}$	$u_{Y'}$	$u_{Z'}$	$\varphi_{X'}$	$\varphi_{Y'}$	$\varphi_{Z'}$
5	4,10,11,17, 18,22	XYZ	0.0	0.0	180.0	☐	☐	☒	☐	☐	☐	☐
100	3,21	XYZ	0.0	0.0	0.0	☐	Spring	☐	☒	☐	☐	☐

1.8.2 NODAL SUPPORTS - SPRINGS

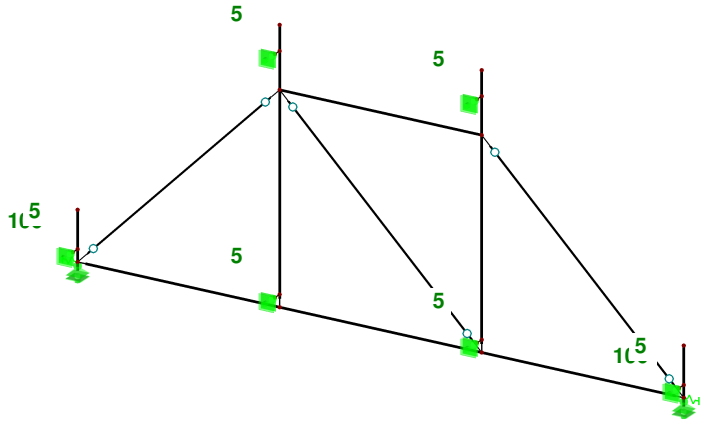
Support No.	Nodes No.	Translation Spring [kN/m] $C_{u,X'}$	$C_{u,Y'}$	$C_{u,Z'}$	Rotation Spring [kNm/rad] $C_{\varphi,X'}$	$C_{\varphi,Y'}$	$C_{\varphi,Z'}$
100	3,21	1000.000	-	-	-	-	-

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MODEL

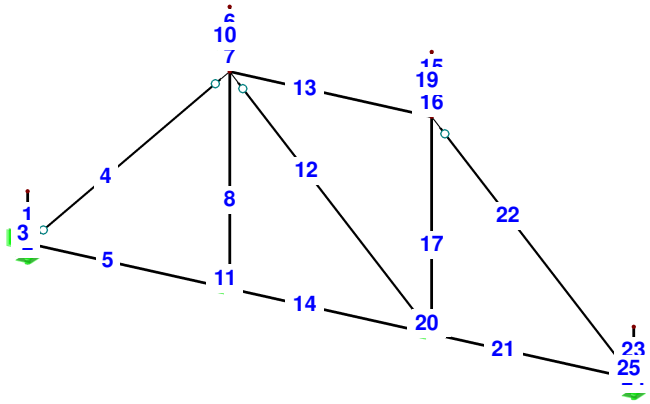
Support Numbering — Isometric



All profiles: HEB 180, S355

MODEL

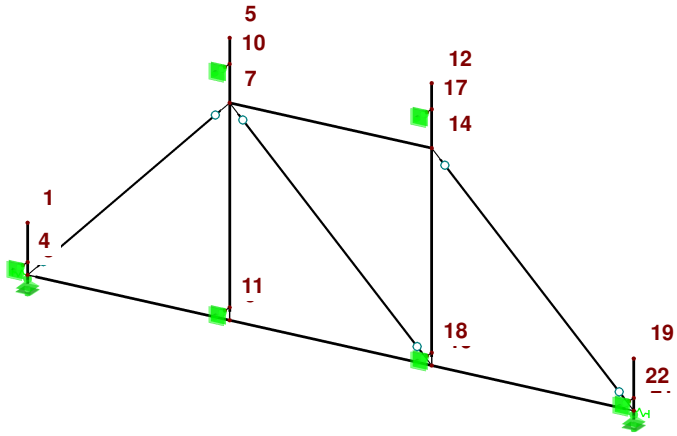
Member Numbering — Isometric



All profiles: HEB 180, S355

MODEL

Node Numbering — Isometric



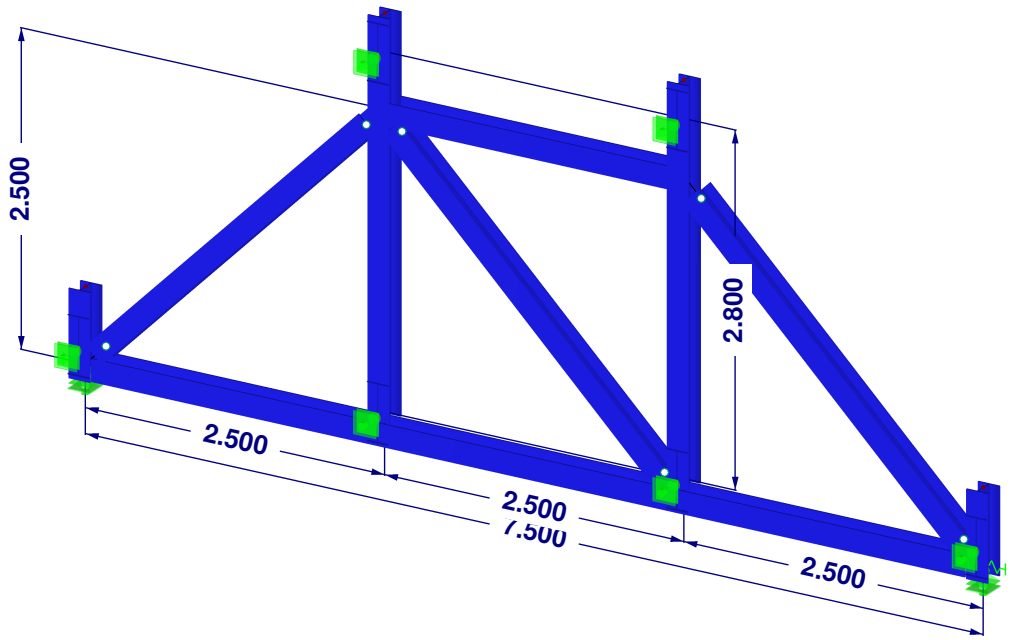
All profiles: HEB 180, S355

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MODEL

Isometric



All profiles: HEB 180, S355

2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | CSN Action Category	Self-Weight - Factor in Direction Active	X	Y	Z
LC1	Self-weight	Permanent	☒	0.000	0.000	1.000
LC2	Imposed load left	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			
LC3	Imposed load right	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			
LC4	vehicle impact 1	Accidental	☐			
LC5	vehicle impact 2	Accidental	☐			
LC6	vehicle impact 3	Accidental	☐			

2.5 LOAD COMBINATIONS

Load Combin.	DS	Load Combination Description	No.	Factor	Load Case	
CO1	STR	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	1	1.35	LC1	Self-weight
			2	1.50	LC2	Imposed load left
CO2	STR	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	1	1.35	LC1	Self-weight
			2	1.50	LC2	Imposed load left
			3	1.50	LC3	Imposed load right
CO3	STR	ULS (STR/GEO) - Permanent / transient - Eq. 6.10	1	1.35	LC1	Self-weight
			2	1.50	LC3	Imposed load right
CO4	ACC	ULS (EQU) - Accidental - psi-1,1	1	1.00	LC1	Self-weight
			2	0.70	LC2	Imposed load left
			3	0.70	LC3	Imposed load right
			4	1.00	LC4	vehicle impact 1
CO5	ACC	ULS (EQU) - Accidental - psi-1,1	1	1.00	LC1	Self-weight
			2	0.70	LC2	Imposed load left
			3	0.70	LC3	Imposed load right
			4	1.00	LC5	vehicle impact 2
CO6	ACC	ULS (EQU) - Accidental - psi-1,1	1	1.00	LC1	Self-weight
			2	0.70	LC2	Imposed load left
			3	0.70	LC3	Imposed load right
			4	1.00	LC6	vehicle impact 3
CO7	S Ch	SLS - Characteristic imposed load	1	1.00	LC2	Imposed load left
			2	1.00	LC3	Imposed load right

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2.6 RESULT COMBINATIONS

Result Combin	Description	Loading
RC1	Governing Result Combination	CO1 or to CO3
RC2	Unusual combination	CO4 or to CO6
RC3	thermal design	LC1/p + 0.7*LC2 + 0.7*LC3

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC1
Self-weight

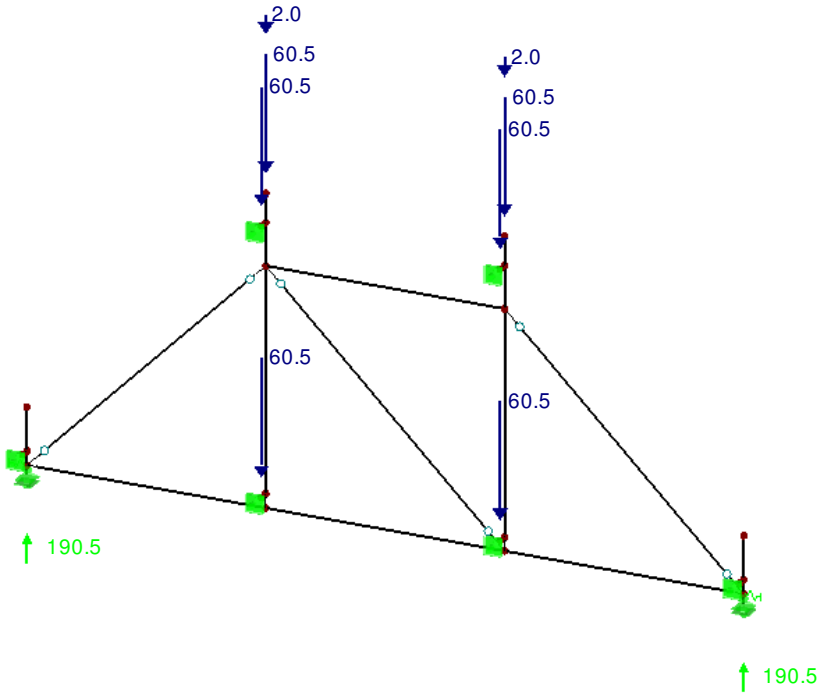
LC1: Self-weight

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	10,17	0 | Global XYZ	0.0	0.0	60.5	0.0	0.0	0.0
	1 x Pos. BE052							
2	5,12	0 | Global XYZ	0.0	0.0	60.5	0.0	0.0	0.0
	1 x Pos. BE052							
3	11	0 | Global XYZ	0.0	0.0	60.5	0.0	0.0	0.0
	1 x Pos. BE052							
4	18	0 | Global XYZ	0.0	0.0	60.5	0.0	0.0	0.0
	1 x Pos. BE052							
17	5,12	0 | Global XYZ	0.0	0.0	2.0	0.0	0.0	0.0
	Dead load column= 1,5 x 2,80 m x 0,45 kN/m = 1,89 kN							

SUPPORT FORCES, PERMANENT LOAD

LC1 : Self-weight
Loads [kN]
Support Reactions[kN]

Isometric



Max P-Z': 190.5, Min P-Z': 0.0 kN

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC2
Imposed load left

LC2: Imposed load left

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	5	0 | Global XYZ	0.0	0.0	50.5	0.0	0.0	0.0
	1 x Pos. BE052							
2	5	0 | Global XYZ	0.0	0.0	17.5	0.0	0.0	0.0
	facade: 2,5 x 2,80 m x 2,50 m x 1 kN/m² = 17,5 kN							
4	10	0 | Global XYZ	0.0	0.0	50.5	0.0	0.0	0.0
	1 x Pos. BE052							
6	11	0 | Global XYZ	0.0	0.0	50.5	0.0	0.0	0.0
	1 x Pos. BE052							

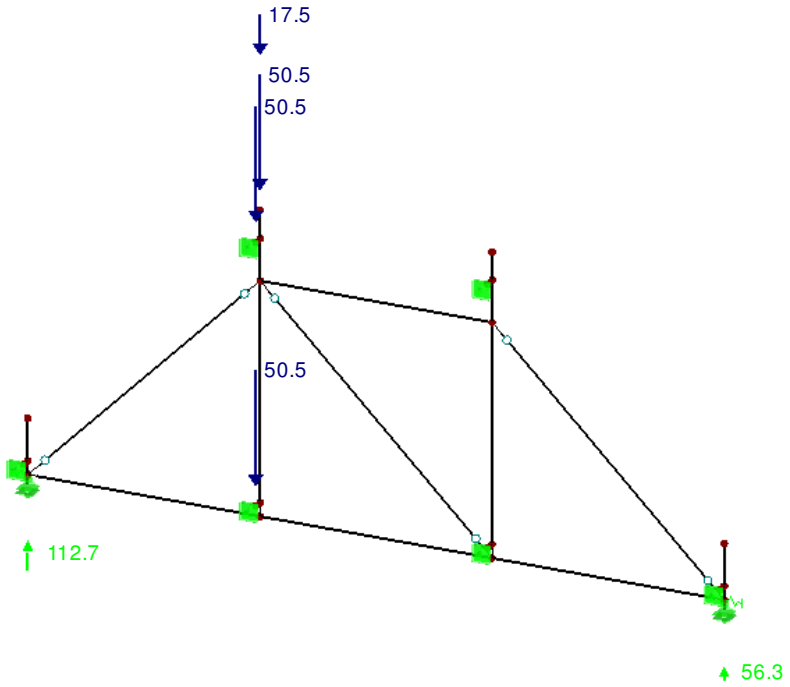
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SUPPORT FORCES, LIVE LOAD LEFT

LC2 : Imposed load left
Loads [kN]
Support Reactions[kN]

Isometric



Max P-Z': 112.7, Min P-Z': 0.0 kN

3.1 NODAL LOADS - BY COMPONENTS - COORDINATE SYSTEM

LC3
Imposed load right

LC3: Imposed load right

No.	On Nodes No.	Coordinate System	Force [kN] P_X / P_U	P_Y / P_V	P_Z / P_W	Moment [kNm] M_X / M_U	M_Y / M_V	M_Z / M_W
1	12,17	0 | Global XYZ	0.0	0.0	50.5	0.0	0.0	0.0
	1 x Pos. BE052							
2	12	0 | Global XYZ	0.0	0.0	17.5	0.0	0.0	0.0
	facade: 2,5 x 2,80 m x 2,50 m x 1 kN/m² = 17,5 kN							
3	18	0 | Global XYZ	0.0	0.0	50.5	0.0	0.0	0.0
	1 x Pos. BE052							

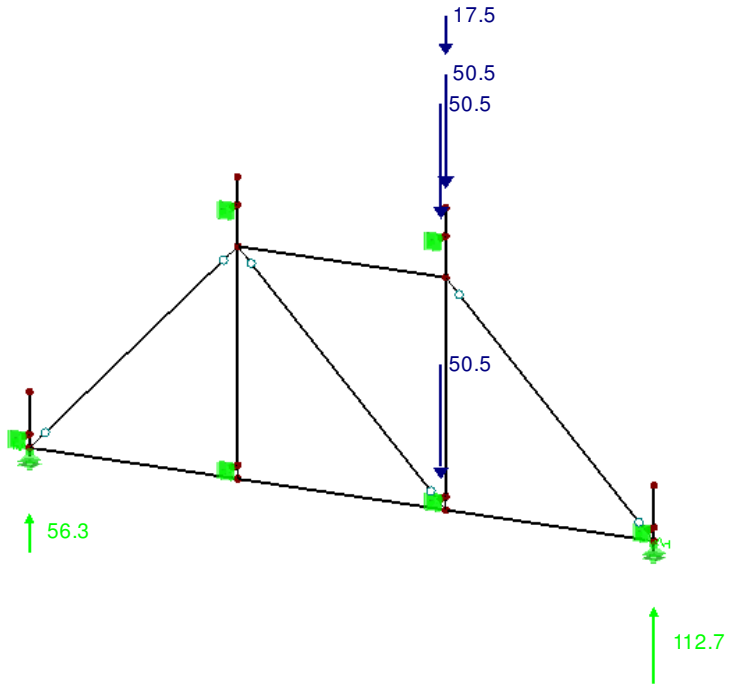
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SUPPORT FORCES, LIVE LOAD RIGHT

LC3 : Imposed load right
Loads [kN]
Support Reactions[kN]

Isometric



Max P-Z': 112.7, Min P-Z': 0.0 kN

LC4
vehicle impact 1

3.2 MEMBER LOADS

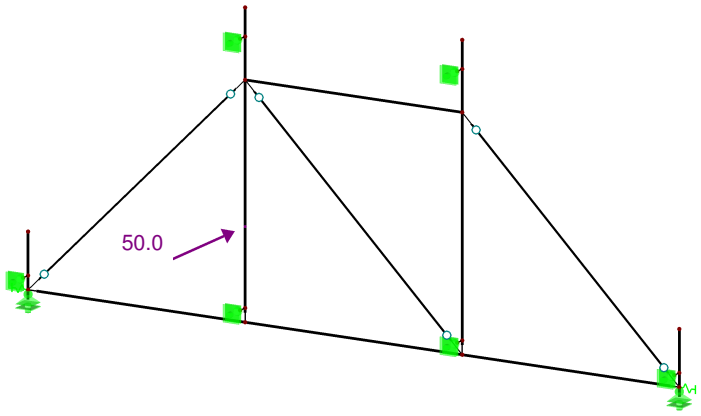
LC4: vehicle impact 1

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
3	Members	8	Force	Concentr.	Y	Projection Y	P	-50.0	kN
							A	0.84	m

LC4: VEHICLE IMPACT 1

LC4 : vehicle impact 1
Loads [kN]

Isometric



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LC5
vehicle impact 2

3.2 MEMBER LOADS

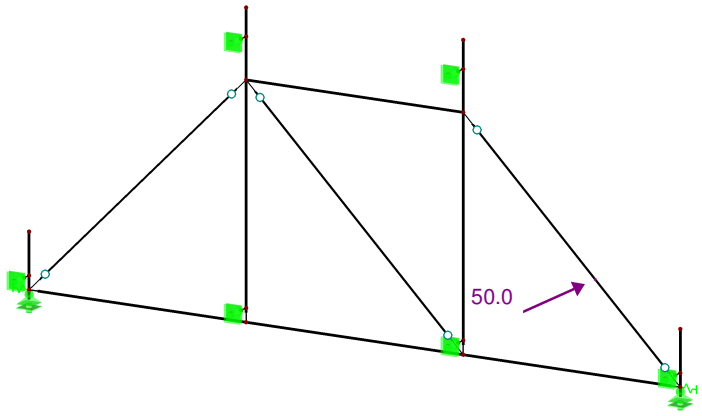
LC5: vehicle impact 2

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
4	Members	22	Force	Concentr.	Y	Projection Y	P	-50.0	kN
							A	2.00	m

LC5: VEHICLE IMPACT 2

LC5 : vehicle impact 2
Loads [kN]

Isometric



LC6
vehicle impact 3

3.2 MEMBER LOADS

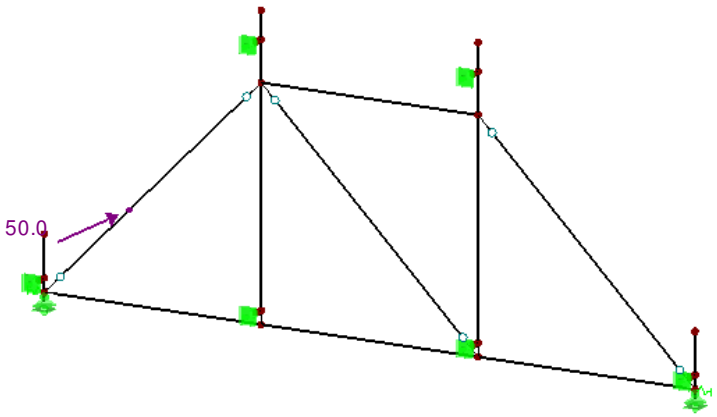
LC6: vehicle impact 3

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
5	Members	4	Force	Concentr.	Y	Projection Y	P	-50.0	kN
							A	1.211	m

LC6: VEHICLE IMPACT 3

LC6 : vehicle impact 3
Loads [kN]

Isometric



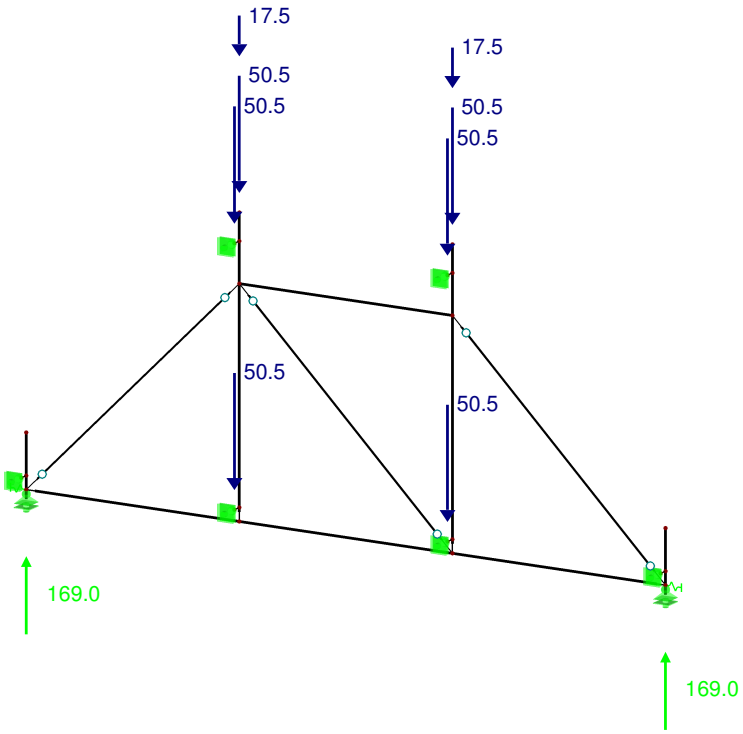
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CO7: SUPPORT REACTIONS - IMPOSED LOAD

CO7 : SLS - Characteristic imposed load
Loads [kN]
Support Reactions[kN]

Isometric

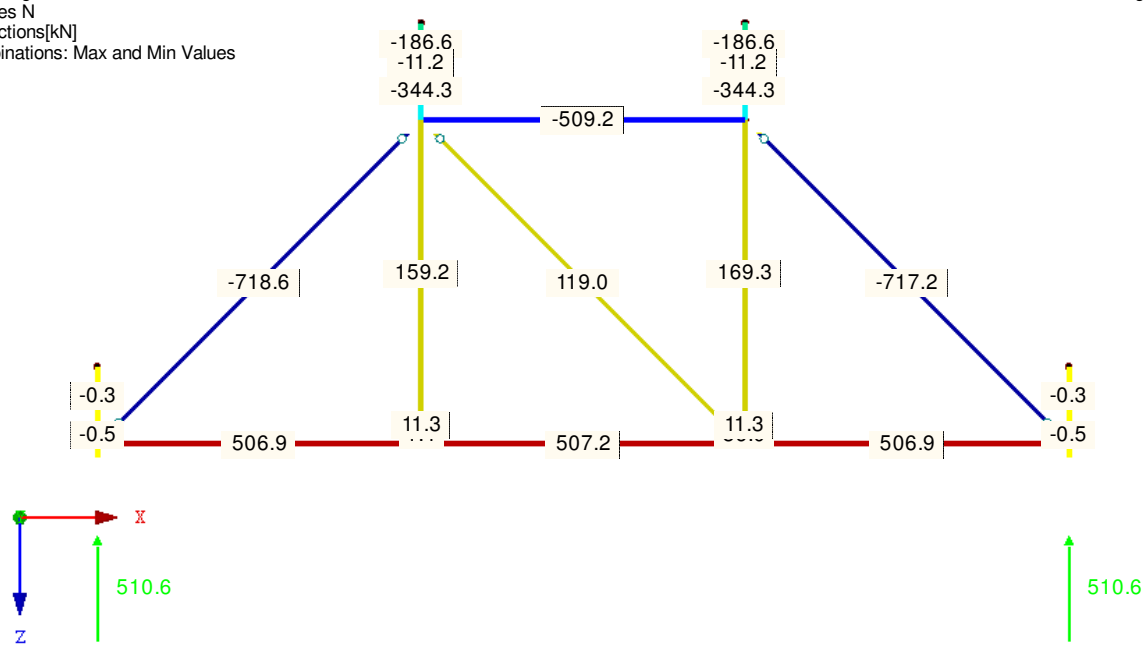


Max P-Z': 169.0, Min P-Z': 0.0 kN

INTERNAL FORCES N, SUPPORT FORCES, DESIGN COMBINATION

RC1 : Governing Result Combination
Internal Forces N
Support Reactions[kN]
Result Combinations: Max and Min Values

Against Y-direction



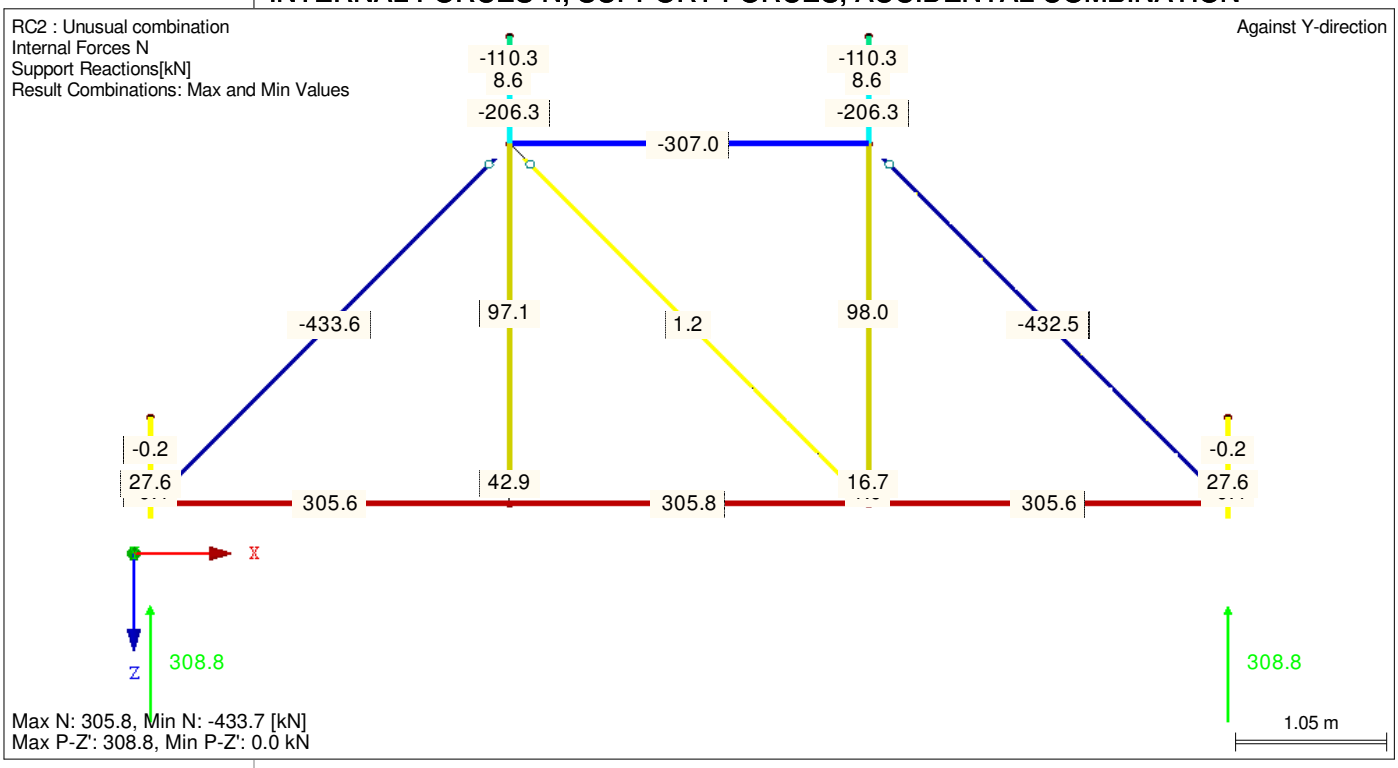
Max N: 507.2, Min N: -718.8 [kN]
Max P-Z': 510.6, Min P-Z': 0.0 kN

1.17 m

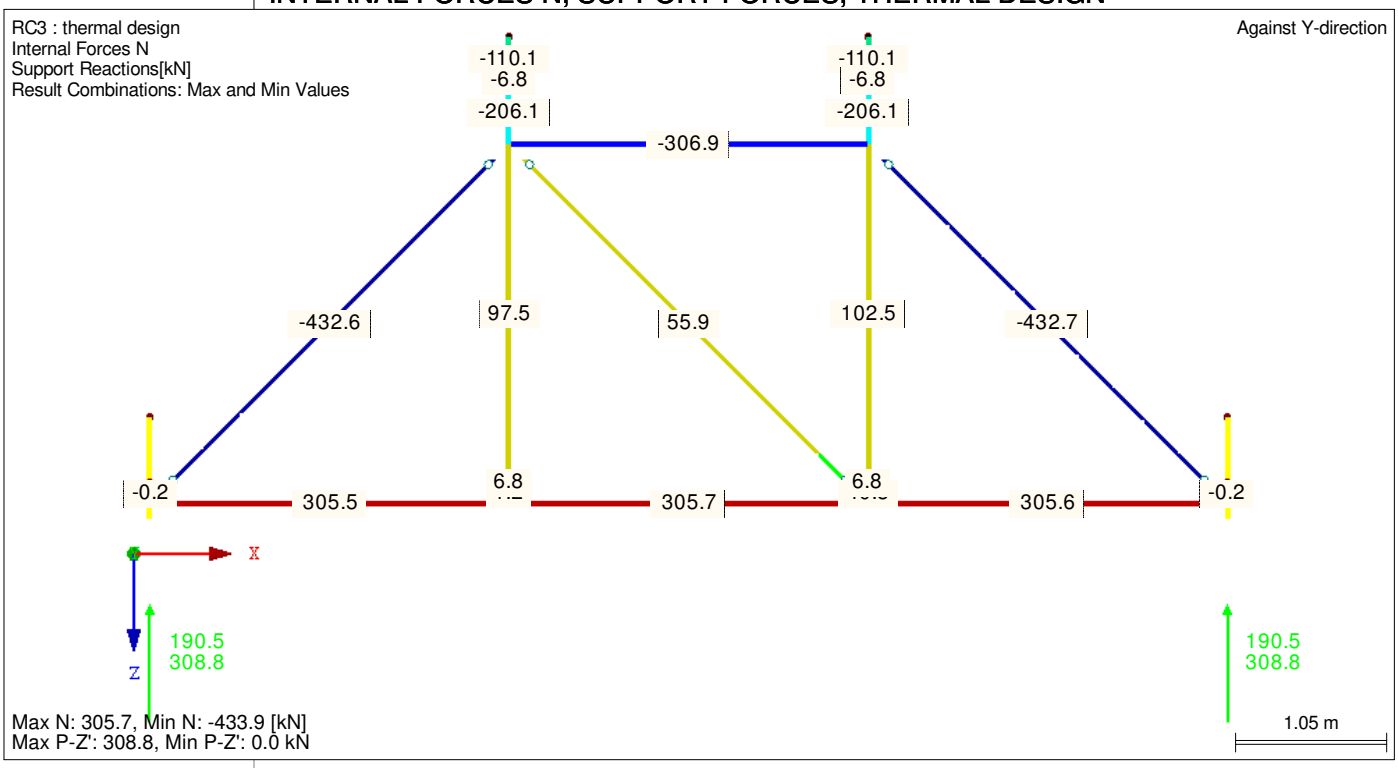
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INTERNAL FORCES N, SUPPORT FORCES, ACCIDENTAL COMBINATION



INTERNAL FORCES N, SUPPORT FORCES, THERMAL DESIGN



STEEL
CA1
General stress analysis

1.1.1 GENERAL DATA

Members to design:		1,2,4-9,12-18,21-24	
Result combinations to design:		RC1	Governing Result Combination
		RC2	Unusual combination

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Project: PR0460 Model: 04-BE451_Truss beam 3 fields Date: 03.09.2025

1.2 MATERIALS

Matl. No.	Material Description	Safety Factor γ_M [-]	Yield Strength f_{yk} [kN/cm²]	Limit Stresses [kN/cm²] Manually	Limit σ_x	Limit τ	Limit σ_{eqv}
3	Baustahl S 355	1.00	35.50	☐	35.50	20.50	35.50

1.3.1 CROSS-SECTIONS



Sect. No.	Matl. No.	Cross-section Description	I_t [cm⁴] / A [cm²]	I_y [cm⁴] / $\alpha_{pl,y}$	I_z [cm⁴] / $\alpha_{pl,z}$	Comment
10	3	HEB 180	42.16 65.25	3831.00 1.13	1363.00 1.53	

2.1 STRESSES BY CROSS-SECTION

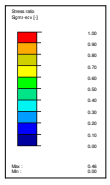
Sect. No.	Member No.	Location x [m]	S-Point No.	Load Case	Stress Type	Stress [kN/cm²] Existing	Limit	Stress Ratio
10	**HEB 180**							
	22	2.00	1	RC2	Sigma Total	-16.25	35.50	0.46
	8	0.84	13	RC2	Tau Total	3.12	20.50	0.15
	22	2.00	1	RC2	Sigma-eqv	16.26	35.50	0.46

3.1 GOVERNING INTERNAL FORCES BY MEMBER

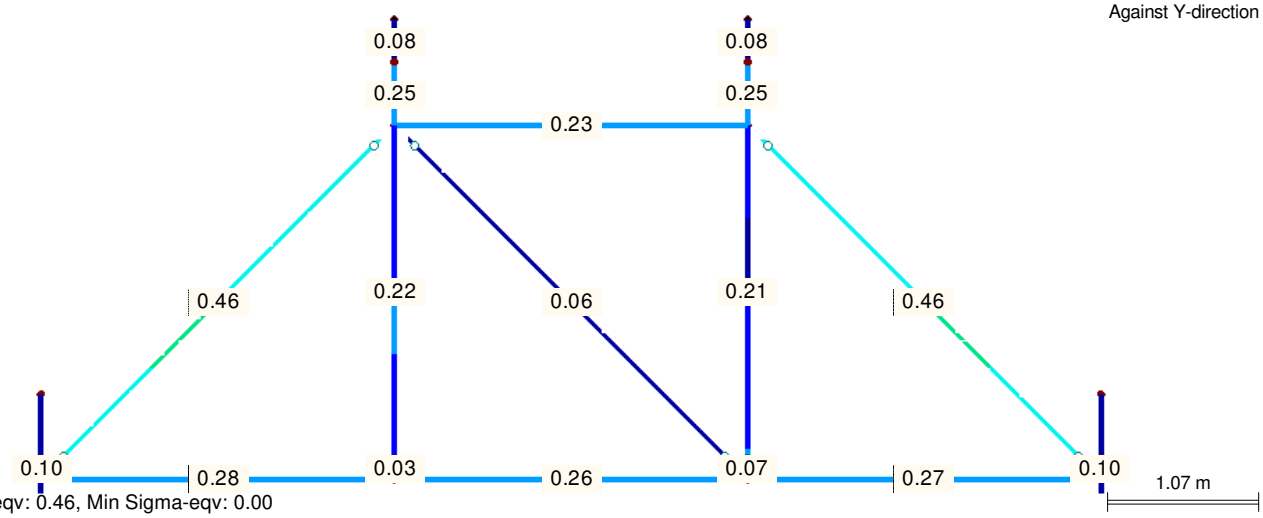
Member No.	Location x [m]	Load Case	Forces [kN] N	V_y	V_z	Moments [kNm] M_T	M_y	M_z
1	0.00	RC1	-0.3	0.0	0.0	0.0	0.0	0.0
2	0.00	RC2	-0.4	0.0	-27.6	0.0	6.8	0.0
4	1.21	RC2	-433.1	0.3	31.7	-0.1	39.4	-0.5
5	2.40	RC1	506.9	0.0	0.0	0.0	0.0	-3.1
6	0.00	RC1	-186.6	0.0	-0.2	0.0	0.1	0.0
7	0.45	RC1	-344.0	0.0	10.9	0.0	15.8	0.0
8	0.84	RC2	96.3	-0.2	41.9	0.0	25.4	-0.5
9	0.00	RC1	1.4	-0.7	-0.1	0.0	0.0	-1.4
12	1.61	RC1	118.2	0.0	0.0	0.0	0.0	-0.6
13	0.88	RC1	-509.2	0.0	0.0	0.0	0.0	-0.6
14	0.75	RC1	507.2	-0.1	0.0	0.0	0.0	-2.1
15	0.00	RC1	-186.6	0.0	-0.2	0.0	0.1	0.0
16	0.45	RC1	-344.0	0.0	10.9	0.0	15.8	0.0
17	0.00	RC1	158.9	1.0	11.4	0.0	-15.7	1.8
18	0.00	RC1	86.0	1.2	-0.1	0.0	0.0	1.9
21	0.00	RC1	506.9	0.2	0.0	0.0	0.0	-2.8
22	2.00	RC2	-433.2	-0.1	18.3	0.1	39.4	-0.5
23	0.00	RC1	-0.3	0.0	0.0	0.0	0.0	0.0
24	0.00	RC2	-0.4	0.0	-27.6	0.0	6.8	0.0

STRESS RATIO Sigma-eqv

STEEL CA1



Against Y-direction



Max Sigma-eqv: 0.46, Min Sigma-eqv: 0.00

STEEL EC3
CA1
Design according to EC3

1.1 GENERAL DATA

Members to design:	4,5,8,12-14,17,21,22	
Sets of members to design:		
National Annex:	CSN	
Ultimate Limit State Design Result combinations to design:	RC1 RC2	Governing Result Combination Unusual combination

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1.1 GENERAL DATA

	Fire Resistance Design Result combinations to design:	RC3	thermal design

1.2 MATERIALS

Matl. No.	Material Description	E- Modulus E [kN/cm²]	Shear Modulus G [kN/cm²]	Poisson's Ratio ν [-]	Yield Stress f_{yk} [kN/cm²]	Max. Thickness t [mm]
3	Steel S 355 | DIN EN 1993-1-1:2010-12	21000.00	8076.92	0.300	35.50	40.0
					33.50	80.0
					31.50	100.0
					29.50	150.0
					28.50	200.0
					27.50	250.0

1.3 CROSS-SECTIONS



Sect. No.	Matl. No.	Cross-Section Description	Cross-Section Type	Max Design Ratio	Comment
10	3	HEB 180	I-section rolled	0.71	

1.10 FIRE RESISTANCE - MEMBERS

No.	Members N	Req. Time $t_{fi,req}$ [min]	Fire Exposure	Fire Protection	Protection Type	Unit Mass ρ_p [kg/m³]	Thermal Cond. λ_p [W/m*K]	Specific Heat c_p [J/(kg*K)]	Thickness d_p [mm]
1	4,5,8, 12-14,17, 21,22	15	All Sides	☐					

2.1 DESIGN BY LOAD CASE

LC/CO/ RC	Load Case or CO/RC Description	Member No.	Location x [m]	Design		Equation No.	Description
	Ultimate Limit State Design						
RC1	Governing Result Combination	22	3.05	0.54	≤ 1	ST364)	PT
RC2	Unusual combination	22	3.05	0.55	≤ 1	ST364)	AC
	Fire Resistance Design						
RC3	thermal design	22	3.21	0.71	≤ 1	FS812)	

2.2 DESIGN BY CROSS-SECTION

Sect. No.	Member No.	Location x [m]	LC/CO/ RC	Design		Equation No.	Description
10	**HEB 180**						
	12	3.05	RC1	0.00	≤ 1	CS100)	Negligible internal forces
	14	0.00	RC1	0.22	≤ 1	CS101)	Cross-section check - Tension acc. to 6.2.3
	22	3.21	RC1	0.31	≤ 1	CS102)	Cross-section check - Compression acc. to 6.2.4
	12	1.61	RC1	0.01	≤ 1	CS116)	Cross-section check - Bending about z-axis acc. to 6.2.5 - Class 1 or 2
	8	0.84	RC2	0.10	≤ 1	CS121)	Cross-section check - Shear force in z-axis acc. to 6.2.6
	4	0.00	RC2	0.00	≤ 1	CS126)	Cross-section check - Shear buckling acc. to 6.2.6(6)
	12	1.61	RC1	0.01	≤ 1	CS151)	Cross-section check - Bending about z-axis and shear force acc. to 6.2.5 and 6.2.8
	22	3.05	RC2	0.21	≤ 1	CS181)	Cross-section check - Bending, shear and axial force acc. to 6.2.9.1
	5	2.40	RC1	0.04	≤ 1	CS201)	Cross-section check - Bending about z-axis, shear and axial force acc. to 6.2.9.1
	4	1.21	RC2	0.07	≤ 1	CS221)	Cross-section check - Biaxial bending, shear and axial force acc. to 6.2.10 and 6.2.9
	13	0.00	RC1	0.20	≤ 1	ST301)	Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2(4)
	22	3.21	RC1	0.36	≤ 1	ST302)	Stability analysis - Flexural buckling about y-axis acc. to 6.3.1.1 and 6.3.1.2
	22	3.21	RC1	0.53	≤ 1	ST312)	Stability analysis - Flexural buckling about z-axis acc. to 6.3.1.1 and 6.3.1.2
	13	0.00	RC2	0.16	≤ 1	ST321)	Stability analysis - Torsional buckling acc. to 6.3.1.4 and 6.3.1.2(4)
	22	3.21	RC1	0.39	≤ 1	ST322)	Stability analysis - Torsional buckling acc. to 6.3.1.4 and 6.3.1.2
	8	0.84	RC2	0.15	≤ 1	ST331)	Stability analysis - Lateral torsional buckling acc. to 6.3.2.1 and 6.3.2.3 - I-Section
	17	0.00	RC1	0.10	≤ 1	ST363)	Stability analysis - Biaxial bending acc. to 6.3.3, Method 2
	22	3.05	RC2	0.55	≤ 1	ST364)	Stability analysis - Bending and compression acc. to 6.3.3, Method 2
	12	3.05	RC3	0.00	≤ 1	FC600)	Fire design - Negligible internal forces
	14	0.00	RC3	0.23	≤ 1	FC601)	Fire design - Cross-section check - Tension acc. to EN 1993-1-2, 4.2.3.1
	22	3.21	RC3	0.32	≤ 1	FC602)	Fire design - Cross-section check - Compression acc. to EN 1993-1-2, 4.2.3.2
	12	1.61	RC3	0.01	≤ 1	FC616)	Fire design - Cross-section check - Bending about z-axis acc. to EN 1993-1-2, 4.2.3.3 - Class 1 or 2
	8	1.29	RC3	0.03	≤ 1	FC621)	Fire design - Cross-section check - Shear force in z-axis acc. to EN 1993-1-2, 4.2.3.3
	12	1.61	RC3	0.01	≤ 1	FC651)	Fire design - Cross-section check - Bending about z-axis a

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2.2 DESIGN BY CROSS-SECTION

Sect. No.	Member No.	Location x [m]	LC/CO/ RC	Design		Equation No.	Description
							and shear force acc. to EN 1993-1-2, 4.2.3.3
	17	2.24	RC3	0.13	≤ 1	FC681)	Fire design - Cross-section check - Bending, shear and axial force acc. to EN 1993-1-2, 4.2.3.3
	22	1.61	RC3	0.33	≤ 1	FC701)	Fire design - Cross-section check - Bending about z-axis, shear and axial force acc. to EN 1993-1-2, 4.2.3.3
	17	0.00	RC3	0.03	≤ 1	FC721)	Fire design - Cross-section check - Biaxial bending, shear and axial force acc. to EN 1993-1-2, 4.2.3.3
	22	3.21	RC3	0.47	≤ 1	FS802)	Fire design - Stability analysis - Flexural buckling about y-axis acc. to EN 1993-1-2, 4.2.3.2
	22	3.21	RC3	0.71	≤ 1	FS812)	Fire design - Stability analysis - Flexural buckling about z-axis acc. to EN 1993-1-2, 4.2.3.2
	22	3.21	RC3	0.49	≤ 1	FS822)	Fire design - Stability analysis - Torsional buckling acc. to EN 1993-1-2, 4.2.3.2
	8	0.00	RC3	0.12	≤ 1	FS831)	Fire design - Stability analysis - Lateral torsional buckling acc. to EN 1993-1-2, 4.2.3.3
	17	0.00	RC3	0.14	≤ 1	FS853)	Fire design - Stability analysis - Biaxial bending acc. to EN 1993-1-2, 4.2.3.5

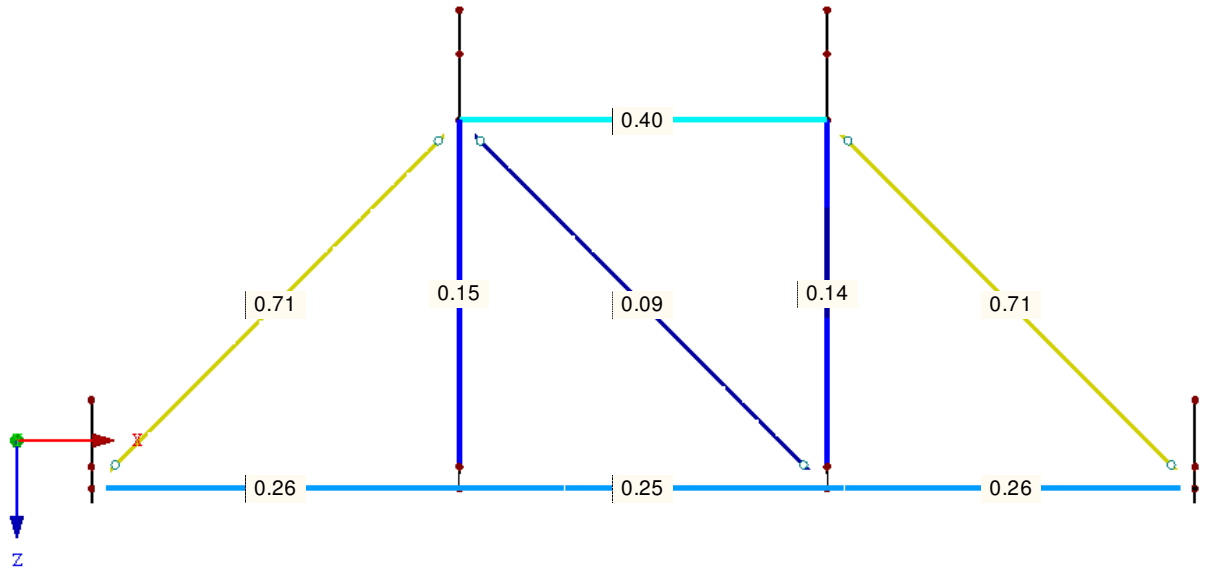
4.1 PARTS LIST BY MEMBER

Part No.	Cross-Section Description	Number Members	Length [m]	Tot. Length [m]	Surf. Area [m^2]	Volume [m^3]	Unit Wt. [kg/m]	Weight [kg]	Total Wt. [t]
1	10 - HEB 180	3	3.21	9.63	10.02	0.06	51.22	164.45	0.493
2	10 - HEB 180	2	2.40	4.80	4.99	0.03	51.22	122.93	0.246
3	10 - HEB 180	2	2.35	4.71	4.89	0.03	51.22	120.52	0.241
4	10 - HEB 180	2	2.50	5.00	5.20	0.03	51.22	128.05	0.256
Sum		9		24.14	25.10	0.16			1.236

VERIFICATION

STEEL EC3 CA1
Ultimate Limit State: Cross-Section Design, Stability Design, Weld Design
Fire Resistance: Cross-Section Design

Against Y-direction



Max Design Ratio: 0.71

Chapter 5: concrete slabs

Pos. SL001 ff.

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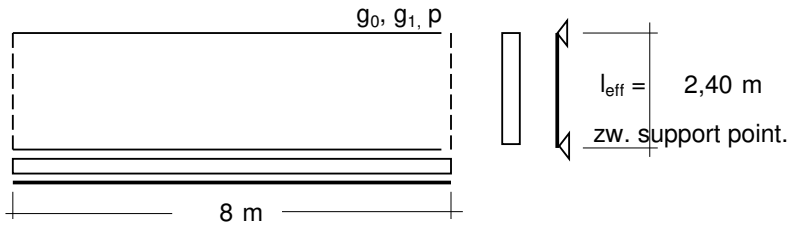
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Pos SL001 Ceiling tile L = 2,50 m

Stand: 10.03.2025

Single span panels are formed which are coupled to the composite beam by loops.
with the composite girder. The slabs are to be verified for one distributed load or for max. two concentrated loads.

1. system:



2. durability:

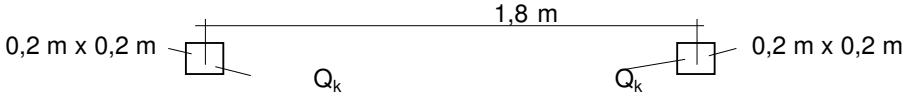
Slab underside: Exposure classes XC3, XF1, WF
--> Concrete quality: required C25/30 existing = C40/50
--> concrete cover: 20 - 5 + 15 - 5 = c_{nom} = 25 mm < cexst = 25 mm

Top side of slab: Exposure classes XC4, XD3, XF4, WA
--> Concrete quality: erf. Air-entrained concrete C30/37; existing air-entrained concrete C40/50
--> Concrete cover 40 - 5 + 15 - 5 = c_{nom} = 45 mm < cexst = 45 mm
(Reduction c_{min} by 5 mm, as the pre. concrete strength class is 2 strength classes higher than the
than the minimum concrete strength class. Reduction Δ_c by 5 mm is achieved by
a corresponding quality control in planning, design and construction.)

3. load:

vertical:

from d =	10,3 cm reinforced concrete:	0,103	m *	25	kN/m³	= g_0 =	2,58 kN/m²
from installation load						g_1=	0,15 kN/m²
						g =	2,73 kN/m²
from live load for cars < 3.0 t, catchment area A < 20 m²						p =	2,50 kN/m²
alternatively concentrated loads for cars < 3.0 t						Q_k =	10,0 kN
Contact area for concentrated load						a =	0,20 m



4. summary of existing reinforcement

above:	**Normal area**	**List mat with**	**a_{sl} =**	**0,94**	**cm²/m**	
	free edge at 0.34m:	**vorh asl = 3 ø 6 =**		**1,88**	**cm²/m**	*see proof 6.3 ff*
bottom:	**Normal area**	**List mat with**	**a_{sl} =**	**3,35**	**cm²/m**	*see proof 5+6.2*
	free edge at 1.(1,06m:	**vorh asl = 10 ø 8 =**		**4,75**	**cm²/m**	*see proof 6.1*
	free edge at 0.34m:	**vorh asl = 5 ø 8 =**		**7,38**	**cm²/m**	*see proof 6.3 ff*
at the free edge:		**2 continuous loops**	**ø 10 =**	**3,14**	**cm²**	

Slab thickness : 10,3 cm **Air-entrained concrete C40/50 ; B500 A (B)**
Exposure classes: XC4, XD3, XF4, WF
cbottom = 25 mm **cup= 45 mm**

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5. calculation of reinforcement due to area load Pos. SL001

Internal forces: γ_F = 1,0 for area load:

$A_{g,k} = B_{g,k}$ =			2,4	m / 2 *	2,7	kN/m²	=	3,27 kN/m
$A_{p,k} = B_{p,k}$ =			2,4	m / 2 *	2,5	kN/m²	=	3,00 kN/m

Internal forces: γ_G = 1,35 γ_Q = 1,5 **for area load**

$A_d = B_d$ =	(	1,35	* g +	1,50	* p) *	2,4 / 2	=	8,9 kN/m
max M_d =	(	1,35	* g +	1,50	* p) *	2,4 ² / 8	=	5,3 kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 68 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B500 A (B)

σ_{sd} = (Limitation of steel sp. to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 5,3 *10^6/(1000 * 68 ² * 25,1852) = 0,046

ω = 0,047

erf. as = 1000 * 0,047 * 68 * 25,1852 / 457,0 = **1,77** cm²/m

foregoing as = 3,35 cm²/m

6. calculation of reinforcement due to wheel load

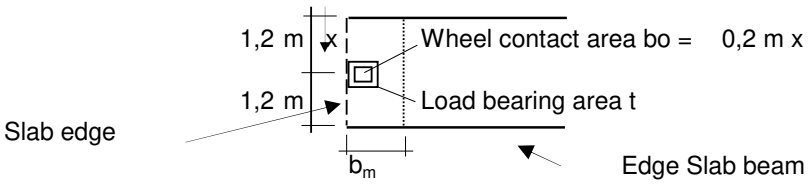
Pos. SL001

The wheel loads are to be placed at the most unfavourable point for the respective internal forces.
The internal force is calculated on a detached beam. The width of the beam is
determined according to Figure 2.2 of DAfStb booklet 631. For the load transfer area
the wheel contact area may be increased.

Load transfer area = t_y x t_x = (0,2 + h) m x (0,2 + h) m = 0,303 m x 0,303 m

6.1 Bending reinforcement at the free slab edge

decisive load position:



effective slab width bm:

$b_{m,M} = t_y + 2{,}5 * x * (1- x /L) / 2$

maximum moment at x = L/2 = 2,4 m / 2 = 1,2 m

$b_{m,M}$ = 0,303 + 2,5 * 1,2 m * (1 - 1,2 m / 2,4 m) / 2 = 1,060 m

internal forces: γ_G = **1,35** γ_Q = **1,5**

$m_{g,d}$	*(EG slab) =*	1,35	*	2,7 kN/m² *	2,4 ²m / 8	=	2,6	kNm/m
$M_{p,d}/b_{m,M}$	*(concentrated load) =*	1,5	*	10,0 kN *	2,4 m / 4			
		* 1 /	1,06 m			=	8,5	kNm/m
$m_{ges,d}$						=	11,1	kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 68 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B500 A (B)

σ_{sd} = (Limitation of steel sp. to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 11,1 *10^6/(1000 * 68 ² * 25,1852) = 0,096

ω = 0,101

erf. as = 1000 * 0,101 * 68 * 25,1852 / 457,0 = **3,78** cm²/m

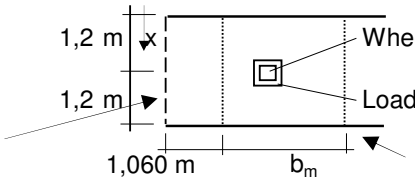
vorh. a_{sl} auf 1,0 m = 4,75 cm²/m

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6.2 Bending reinforcement **1,060** **m from free edge** Pos. SL001

decisive load position:



effective slab width bm:

$b_{m,M} = t_y + 2{,}5 * x * (1- x/L_{eff})$

maximum moment at $x = L_{eff}/2$ = 2,4 m / 2 = 1,2 m

$b_{m,M}$ = 0,303 + 2,5 * 1,2 m * (1 - 1,2 m / 2,4 m) = 1,8 m

internal forces: γ_G = **1,35** γ_Q = **1,5**

$m_{g,d}$ *(EG slab)* = 1,35 * 2,7 kN/m² * 2,4 ²m / 8 = 2,6 kNm/m

$M_{p,d}/b_{m,M}$ *(concentrated load)*= 1,5 * 10,0 kN * 2,4 m / 4 * 1 / 1,8 m = 5,0 kNm/m

$m_{ges,d}$ = 7,6 kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 68 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B500 A (B)

σ_{sd} = (Limitation of steel sp. to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 7,6 $*10^6$/(1000 * 68 ² * 25,1852) = 0,066

ω = 0,068

erf. as = 1000 * 0,068 * 68 * 25,1852 / 457,0 = **2,55** cm²/m

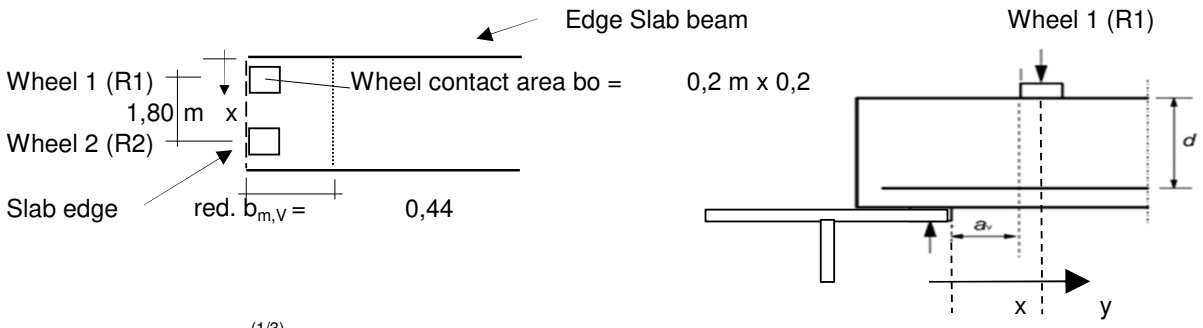
vorh a_s = 3,35 cm²/m

6.3 Shear force design with load at the free edge of the slab

Pos. SL001

decisive load position:

Several load positions are investigated. In the case of concentrated loads close to the support, the the shear force is reduced, so that a concentrated load somewhat further away from the edge can become decisive.



$v_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_1 * f_{ck})^{(1/3)}) * b_w * d$

$C_{Rd,c} = (0,15 / \gamma_C)$ = 0,11

k = 1 + root(200/d) = 2,71 --> 2,00

$\rho_1 = A_{s1}/(b_w * d)$ = 7,38 cm²/m / (100 * 6,8) = 0,011

5 ø 8 mm on 0.34 m

b_w = 1,0 m

$v_{Ed} = (1,35 * V_{k,Eg} + 1,50 * (V_{k,R1} + V_{k,R2})) * b_{m,V}$

Co-supporting width according to booklet 631:

for: x ≤ 2,5d = 0,17 m red. $b_{m,V} = 3,5 * d + 1,0 * b_o$ = 0,44 m *red. bm close to support (2.3b)*

for: x > 2,5d = 0,17 m red. $b_{m,V} = 3,5 * d + 0,5 * b_o$ = 0,34 m *red. bm general (2.3a)*

$V_{k,Eg} = A_{g,k} - g * x_v$ *Shear force from EGPlate*

$x_v = d$

$V_{k,R1(2)} = \beta * R1(2) * y / I_{eff}$ *Shear force from wheel 1(2)*

min $\beta = a_v/(2,0 * d) \leq 1$

min $a_v \geq 0,5 * d$ bzw. $< 2 * d$

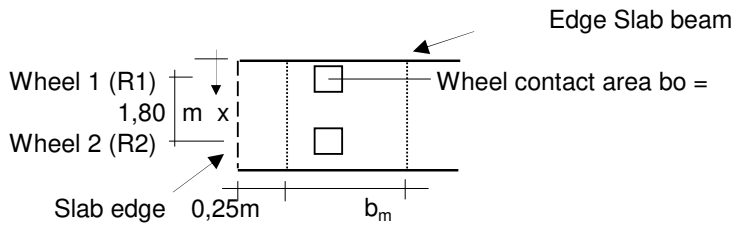
x	a_v	β	$v_{k,Eg}$	red. $b_{m,V,R1}$	$V_{k,R1}$	$v_{k,R1}$	red. $b_{m,V,R2}$	$V_{k,R2}$	$v_{k,R2}$	v_{Ed}	$v_{Rd,c}$	NW
m	m		kN/m	m	kN	kN/m	m	kN	kN/m	kN/m	kN/m	
0,1	0,034	0,25	3,08	0,44	2,40	5,47	0,44	2,08	4,76	19,50	53,1	0,37
0,17	0,070	0,51	3,08	0,44	4,78	10,92	0,44	1,79	4,09	26,68	53,1	0,50
0,18	0,080	0,59	3,08	0,34	5,44	16,10	0,34	1,75	5,18	36,08	53,1	0,68
0,2	0,100	0,74	3,08	0,34	6,74	19,94	0,34	1,67	4,93	41,47	53,1	0,78
0,23	0,130	0,96	3,08	0,34	8,64	25,57	0,34	1,54	4,56	49,36	53,1	0,93
0,24	0,136	1,00	3,08	0,34	9,00	26,63	0,34	1,50	4,44	50,76	53,1	0,96
0,25	0,136	1,00	3,08	0,34	8,96	26,50	0,34	1,46	4,31	50,39	53,1	0,95

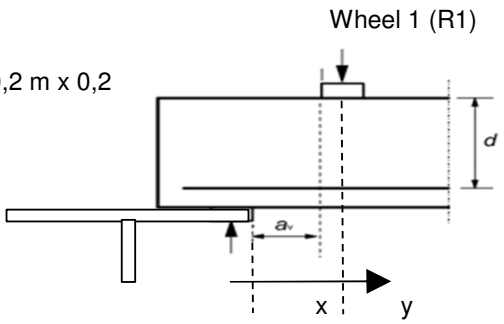
6.4 Shear force calculation for loads further than 0.25 m from edge

Pos. SL001

decisive load position:

Several load positions are investigated. In the case of concentrated loads close to the support, the the shear force is reduced, so that a concentrated load slightly further away from the edge can become decisive.





$v_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_1 * f_{ck})^{(1/3)}) * b_w * d$

$C_{Rd,c} = (0,15 / \gamma_C) =$ 0,11

$k = 1 + \text{root}(200/d) =$ 2,71 --> 2,00

$\rho_1 = A_{s1}/(b_w * d) =$ 3,35 cm²/m / (100 * 6,8) = 0,0049

$b_w =$ 1,0 m

$v_{Ed} = (1,35 * V_{k,Eg} + 1,50 * (V_{k,R1} + V_{k,R2})) * b_{m,V}$

Co-supporting width according to booklet 631:

für: x < 2,5d = 0,17 m $b_{m,V} = 7,0 * d + 1,0 * b_o =$ 0,68 m *close to support (2.3b)*

für: x > 2,5d = 0,17 m $b_{m,V} = 7,0 * d + 0,5 * b_o =$ 0,58 m *general (2.3a)*

$V_{k,Eg} = A_{g,k} - g * x_v$ *Shear force from slab*

$x_v = d$

$V_{k,R1(2)} = \beta * R1(2) * y / I_{eff}$ *Shear force from wheel 1(2)*

min $\beta = a_v/(2,0 * d) \le 1$

min $a_v \ge 0,5 * d$ bzw. $< 2 * d$

x	a_v	β	$v_{k,Eg}$	$b_{m,V,R1}$	$V_{k,R1}$	$v_{k,R1}$	$b_{m,V,R2}$	$V_{k,R2}$	$v_{k,R2}$	v_{Ed}	$v_{Rd,c}$	NW
m	m		kN/m	m	kN	kN/m	m	kN	kN/m	kN/m	kN/m	
0,1	0,034	0,25	3,08	0,68	2,40	3,54	0,68	2,08	3,08	14,10	40,8	0,35
0,17	0,070	0,51	3,08	0,58	4,78	8,30	0,58	1,79	3,11	21,28	40,8	0,52
0,18	0,080	0,59	3,08	0,58	5,44	9,45	0,58	1,75	3,04	22,89	40,8	0,56
0,2	0,100	0,74	3,08	0,58	6,74	11,70	0,58	1,67	2,89	26,06	40,8	0,64
0,23	0,130	0,96	3,08	0,58	8,64	15,00	0,58	1,54	2,68	30,69	40,8	0,75
0,24	0,136	1,00	3,08	0,58	9,00	15,63	0,58	1,50	2,60	31,51	40,8	0,77
0,25	0,136	1,00	3,08	0,58	8,96	15,55	0,58	1,46	2,53	31,29	40,8	0,77

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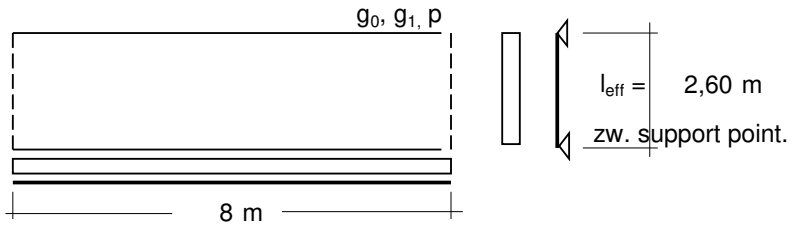
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Pos SL100 Ceiling tile L = 2,70 m

Stand: 10.03.2025

Single span panels are formed which are coupled to the composite beam by loops.
with the composite girder. The slabs are to be verified for one distributed load or for max. two concentrated loads.

1. system:



2. durability:

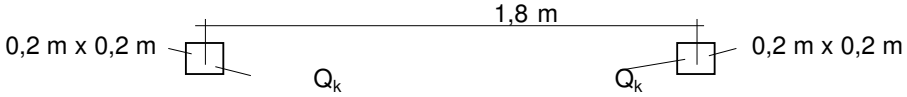
Slab underside: Exposure classes XC3, XF1, WF
--> Concrete quality: required C25/30 existing = C40/50
--> concrete cover: 20 - 5 + 15 - 5 = c_{nom} = 25 mm < cexst = 25 mm

Top side of slab: Exposure classes XC4, XD3, XF4, WA
--> Concrete quality: erf. Air-entrained concrete C30/37; existing air-entrained concrete C40/50
--> Concrete cover 40 - 5 + 15 - 5 = c_{nom} = 45 mm < cexst = 45 mm
(Reduction c_{min} by 5 mm, as the pre. concrete strength class is 2 strength classes higher than the
than the minimum concrete strength class. Reduction Δ_c by 5 mm is achieved by
a corresponding quality control in planning, design and construction.)

3. load:

vertical:

from d =	10,3 cm reinforced concrete:	0,103	m *	25	kN/m³	= g_0 =	2,58 kN/m²
from installation load						g_1=	0,15 kN/m²
						g =	2,73 kN/m²
from live load for cars < 3.0 t, catchment area A < 20 m²						p =	3,00 kN/m²
alternatively concentrated loads for cars < 3.0 t						Q_k =	10,0 kN
Contact area for concentrated load						a =	0,20 m



4. summary of existing reinforcement

above:	**Normal area**	**List mat with**	**a_{sl} =**	**0,94**	**cm²/m**	
	free edge at 0.34m:	**vorh asl = 3 ø 6 =**		**1,88**	**cm²/m**	*see proof 6.3 ff*
bottom:	**Normal area**	**List mat with**	**a_{sl} =**	**3,35**	**cm²/m**	*see proof 5+6.2*
	free edge at 1.(1,12m:	**vorh asl = 11 ø 8 =**		**4,94**	**cm²/m**	*see proof 6.1*
	free edge at 0.34m:	**vorh asl = 6 ø 8 =**		**8,88**	**cm²/m**	*see proof 6.3 ff*
at the free edge:		**2 continuous loops**	**ø 10 =**	**3,14**	**cm²**	

Slab thickness : 10,3 cm **Air-entrained concrete C40/50 ; B500 A (B)**
Exposure classes: XC4, XD3, XF4, WF
cbottom = 25 mm **cup= 45 mm**

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5. calculation of reinforcement due to area load Pos. SL100

Internal forces: γ_F = 1,0 for area load:

$A_{g,k} = B_{g,k}$ =			2,6	m / 2 *	2,7	kN/m²	=	3,54 kN/m
$A_{p,k} = B_{p,k}$ =			2,6	m / 2 *	3,0	kN/m²	=	3,90 kN/m

Internal forces: γ_G = 1,35 γ_Q = 1,5 **for area load**

$A_d = B_d$ =	(	1,35	* g +	1,50	* p) *	2,6 / 2	=	10,6 kN/m
max M_d =	(	1,35	* g +	1,50	* p) *	2,6 ² / 8	=	6,9 kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 68 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B500 A (B)

σ_{sd} = (Limitation of steel sp. to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 6,9 *10^6/(1000 * 68 ² * 25,1852) = 0,059

ω = 0,061

erf. as = 1000 * 0,061 * 68 * 25,1852 / 457,0 = **2,30** cm²/m

foregoing as = 3,35 cm²/m

6. calculation of reinforcement due to wheel load

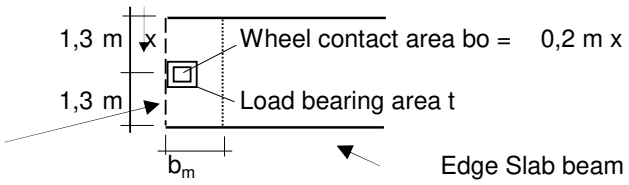
Pos. SL100

The wheel loads are to be placed at the most unfavourable point for the respective internal forces.
The internal force is calculated on a detached beam. The width of the beam is
determined according to Figure 2.2 of DAfStb booklet 631. For the load transfer area
the wheel contact area may be increased.

Load transfer area = t_y x t_x = (0,2 + h) m x (0,2 + h) m = 0,303 m x 0,303 m

6.1 Bending reinforcement at the free slab edge

decisive load position:



effective slab width bm:

$b_{m,M} = t_y + 2{,}5 * x * (1- x /L) / 2$

maximum moment at x = L/2 = 2,6 m / 2 = 1,3 m

$b_{m,M}$ = 0,303 + 2,5 * 1,3 m * (1 - 1,3 m / 2,6 m) / 2 = 1,120 m

internal forces: γ_G = **1,35** γ_Q = **1,5**

$m_{g,d}$ *(EG slab)* =	1,35	*	2,7 kN/m² *	2,6 ²m / 8	=	3,1 kNm/m
$M_{p,d}/b_{m,M}$ *(concentrated load)* =	1,5	*	10,0 kN *	2,6 m / 4		
	* 1 /	1,12 m			=	8,7 kNm/m
$m_{ges,d}$					=	11,8 kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 68 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B500 A (B)

σ_{sd} = (Limitation of steel sp. to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 11,8 $*10^6/(1000 *$ 68 ² * 25,1852) = 0,101

ω = 0,107

erf. as = 1000 * 0,107 * 68 * 25,1852 / 457,0 = **4,03** cm²/m

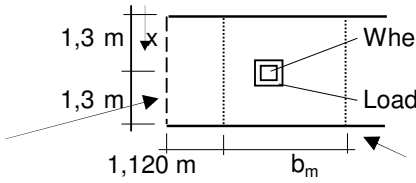
vorh. a_{sl} auf 1,0 m = 4,94 cm²/m

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6.2 Bending reinforcement **1,120** **m from free edge** Pos. SL100

decisive load position:



effective slab width bm:

$b_{m,M} = t_y + 2{,}5 * x * (1- x/L_{eff})$

maximum moment at $x = L_{eff}/2$ = 2,6 m / 2 = 1,3 m

$b_{m,M}$ = 0,303 + 2,5 * 1,3 m * (1 - 1,3 m / 2,6 m) = 1,8 m

internal forces: γ_G = **1,35** γ_Q = **1,5**

$m_{g,d}$ *(EG slab)* = 1,35 * 2,7 kN/m² * 2,6 ²m / 8 = 3,1 kNm/m

$M_{p,d}/b_{m,M}$ *(concentrated load)*= 1,5 * 10,0 kN * 2,6 m / 4

* 1 / 1,8 m = 5,4 kNm/m

$m_{ges,d}$ = 8,5 kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 68 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B500 A (B)

σ_{sd} = (Limitation of steel sp. to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 8,5 *10^6/(1000 * 68 ² * 25,1852) = 0,073

ω = 0,076

erf. as = 1000 * 0,076 * 68 * 25,1852 / 457,0 = **2,86** cm²/m

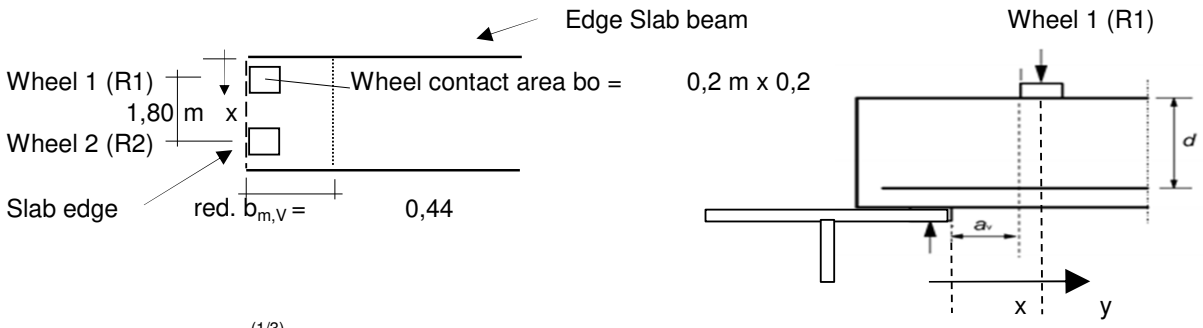
vorh a_s = 3,35 cm²/m

6.3 Shear force design with load at the free edge of the slab

Pos. SL100

decisive load position:

Several load positions are investigated. In the case of concentrated loads close to the support, the the shear force is reduced, so that a concentrated load somewhat further away from the edge can become decisive.



$v_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_1 * f_{ck})^{(1/3)}) * b_w * d$

$C_{Rd,c} = (0{,}15 / \gamma_C)$ = 0,11

k = 1 + root(200/d) = 2,71 --> 2,00

$\rho_1 = A_{s1}/(b_w * d)$ = 8,88 cm²/m / (100 * 6,8) = 0,013

6 ø 8 mm on 0.34 m

b_w = 1,0 m

$v_{Ed} = (1{,}35 * V_{k,Eg} + 1{,}50 * (V_{k,R1} + V_{k,R2})) * b_{m,V}$

Co-supporting width according to booklet 631:

for: x ≤ 2,5d = 0,17 m red. $b_{m,V} = 3{,}5 * d + 1{,}0 * b_o$ = 0,44 m *red. bm close to support (2.3b*

for: x > 2,5d = 0,17 m red. $b_{m,V} = 3{,}5 * d + 0{,}5 * b_o$ = 0,34 m *red. bm general (2.3a)*

$V_{k,Eg} = A_{g,k} - g * x_v$ *Shear force from EGPlate*

$x_v = d$

$V_{k,R1(2)} = \beta * R1(2) * y / l_{eff}$ *Shear force from wheel 1(2)*

min $\beta = a_v/(2{,}0 * d) \le 1$

min $a_v \ge 0{,}5 * d$ bzw. $< 2 * d$

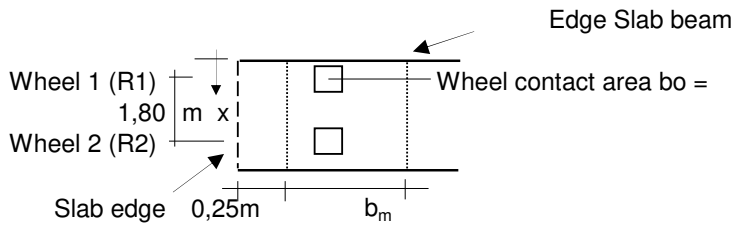
x	a_v	β	$v_{k,Eg}$	red. $b_{m,V,R1}$	$V_{k,R1}$	$v_{k,R1}$	red. $b_{m,V,R2}$	$V_{k,R2}$	$v_{k,R2}$	v_{Ed}	$v_{Rd,c}$	NW
m	m		kN/m	m	kN	kN/m	m	kN	kN/m	kN/m	kN/m	
0,1	0,034	0,25	3,36	0,44	2,40	5,49	0,44	2,69	6,15	21,98	56,5	0,39
0,17	0,070	0,51	3,36	0,44	4,81	10,98	0,44	2,42	5,53	29,30	56,5	0,52
0,18	0,080	0,59	3,36	0,34	5,48	16,20	0,34	2,38	7,06	39,41	56,5	0,70
0,2	0,100	0,74	3,36	0,34	6,79	20,08	0,34	2,31	6,83	44,89	56,5	0,79
0,23	0,130	0,96	3,36	0,34	8,71	25,78	0,34	2,19	6,49	52,93	56,5	0,94
0,24	0,136	1,00	3,36	0,34	9,08	26,85	0,34	2,15	6,37	54,37	56,5	0,96
0,25	0,136	1,00	3,36	0,34	9,04	26,74	0,34	2,12	6,26	54,03	56,5	0,96

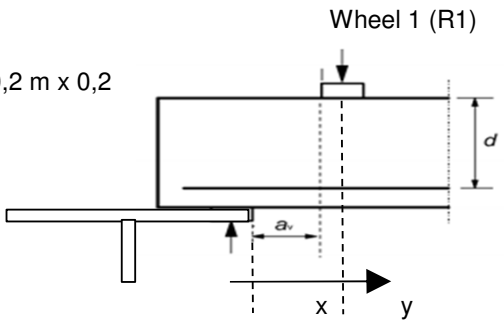
6.4 Shear force calculation for loads further than 0.25 m from edge

Pos. SL100

decisive load position:

Several load positions are investigated. In the case of concentrated loads close to the support, the the shear force is reduced, so that a concentrated load slightly further away from the edge can become decisive.





$v_{Rd,c} = (C_{Rd,c} * k * (100 * \rho_1 * f_{ck})^{(1/3)}) * b_w * d$

$C_{Rd,c} = (0,15 / \gamma_C) =$ 0,11

k = 1 + root(200/d) = 2,71 --> 2,00

$\rho_1 = A_{s1}/(b_w * d) =$ 3,35 cm²/m / (100 * 6,8) = 0,0049

$b_w =$ 1,0 m

$v_{Ed} = (1,35 * V_{k,Eg} + 1,50 * (V_{k,R1} + V_{k,R2})) * b_{m,V}$

Co-supporting width according to booklet 631:

für: x < 2,5d = 0,17 m $b_{m,V} = 7,0 * d + 1,0 * b_o =$ 0,68 m *close to support (2.3b)*

für: x > 2,5d = 0,17 m $b_{m,V} = 7,0 * d + 0,5 * b_o =$ 0,58 m *general (2.3a)*

$V_{k,Eg} = A_{g,k} - g * x_v$ *Shear force from slab*

$x_v = d$

$V_{k,R1(2)} = \beta * R1(2) * y / l_{eff}$ *Shear force from wheel 1(2)*

min $\beta = a_v/(2,0 * d) \leq 1$

min $a_v \geq 0,5 * d$ bzw. $< 2 * d$

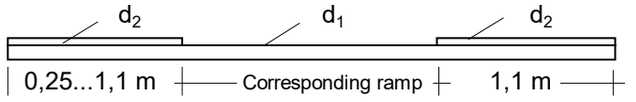
x	a_v	β	$v_{k,Eg}$	$b_{m,V,R1}$	$V_{k,R1}$	$v_{k,R1}$	$b_{m,V,R2}$	$V_{k,R2}$	$v_{k,R2}$	v_{Ed}	$v_{Rd,c}$	NW
m	m		kN/m	m	kN	kN/m	m	kN	kN/m	kN/m	kN/m	
0,1	0,034	0,25	3,36	0,68	2,40	3,56	0,68	2,69	3,98	15,84	40,8	0,39
0,17	0,070	0,51	3,36	0,58	4,81	8,35	0,58	2,42	4,21	23,37	40,8	0,57
0,18	0,080	0,59	3,36	0,58	5,48	9,51	0,58	2,38	4,14	25,00	40,8	0,61
0,2	0,100	0,74	3,36	0,58	6,79	11,78	0,58	2,31	4,01	28,22	40,8	0,69
0,23	0,130	0,96	3,36	0,58	8,71	15,13	0,58	2,19	3,81	32,93	40,8	0,81
0,24	0,136	1,00	3,36	0,58	9,08	15,76	0,58	2,15	3,74	33,78	40,8	0,83
0,25	0,136	1,00	3,36	0,58	9,04	15,69	0,58	2,12	3,67	33,58	40,8	0,82

Pos SL 300 Ramp panels L = 2.5 m

Simply supported panels are placed, which are connected with reinforcement loops to the composite beams
The panels shall be designed for a distributed load or for two concentrated loads.

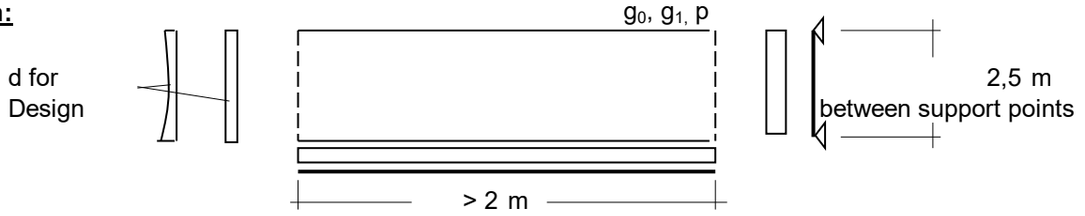
Pos SL 300

Cross-section:



Thickness d1 = 10,3 cm
Thickness d2 = 23,95 cm

System:



Durability:

Slab underside: exposure classes XC3, XF1
--> concrete strength: required C2 vorh = C40/50
--> concrete cover 20 - 5 + 15 - 5 = c_{nom} = 25 mm ≤ c_{vorh} = 25 mm

Slab topside: exposure classes XC4, XD3, XF4
--> Concrete strength: Air-entrained concrete C30/37 req.; actual: air-ent C40/50
--> concrete cover 40 - 5 + 15 - 5 = c_{nom} = 45 mm ≤ c_{vorh} = 45 mm

(Decrease cmin by 5 mm, as the actual concrete strength class is 2 classes higher than the minimum required class. A reduction of Dc by 5 mm is achieved by a corresponding quality control during planning, design and construction.)

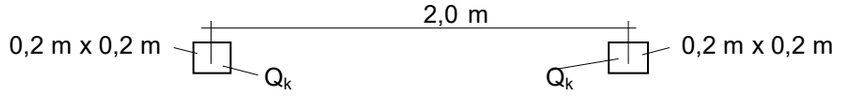
Load:

from d1	10,3 cm	reinforced concrete:	0,16	m *	25	kN/m³ =	g_0 =	4,0 kN/m²
from installation load			thicker slab in the walkway				g_1 =	0,1 kN/m²
							g =	4,1 kN/m²
from d2	24,0 cm	reinforced concrete:	0,2395	m *	25	kN/m³ =	g_0 =	6,0 kN/m²
from installation load							g_1 =	0,1 kN/m²
							g =	6,1 kN/m²

from live load for passenger car < 3.0 t, influence area A < 20 m² p = 5,0 kN/m²

alternatively concentrated loads for passenger cars < 3.0 t Q_k = 10,0 kN

a = 0,20 m



Summary

Top:	**Custom mesh with Asl =**	**1,88**	**cm²/m**
top free edge:	**present Asl at 0.45 m =**	**1,88**	**cm²/m**
bottom:	**Custom mesh with Asl =**	**3,85**	**cm²/m**
bottom free edge:	**Asl present at 1 m =**	**3,85**	**cm²/m**
	present Asl at 0.45 m =	**3,85**	**cm²/m**

Slab thickness: 10,3 cm Air-entrained concrete C40/50 ; B500 A (B)
Exposure classes: XC4, XD3; XF4
c_{unten} = 25 mm c_{oben} = 45 mm

Calculation of reinforcement due to surface load

Cross section with slab thickness d1 Pos SL 300

Internal forces gF = 1.0 for distributed load:

$A_{g,k} = B_{g,k}$ = 2,5 m / 2 * 4,1 kN/m² = 5,13 kN/m

$A_{p,k} = B_{p,k}$ = 2,5 m / 2 * 5,0 kN/m² = 6,25 kN/m

Internal forces gG= 1,35 γ_Q = 1,5 for distributed load

$A_d = B_d$ = (1,35 * g + 1,50 * p) * 2,5 / 2 = 16,3 kN/m

max M_d = (1,35 * g + 1,50 * p) * 2,5 ² / 8 = 10,2 kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 71 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B 500 S/M (A)

σ_{sd} = (Limitation of steel stress to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 10,2 *10°/(1000 * 71 ² * 25,1852) = 0,080

ω = 0,084

erf. a_s = 1000 * 0,084 * 71 * 25,1852 / 457,0 = **3,28** cm²/m

vorh. a_s = 3,85 cm²/m

Cross section with slab thickness d2

Internal forces gF = 1.0 for distributed load:

$A_{g,k} = B_{g,k}$ = 2,5 m / 2 * 6,1 kN/m² = 7,61 kN/m

$A_{p,k} = B_{p,k}$ = 2,5 m / 2 * 5,0 kN/m² = 6,25 kN/m

Internal forces gG= 1,35 γ_Q = 1,5 for distributed load

$A_d = B_d$ = (1,35 * g + 1,50 * p) * 2,5 / 2 = 19,6 kN/m

max M_d = (1,35 * g + 1,50 * p) * 2,5 ² / 8 = 12,3 kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 188 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B 500 S/M (A)

σ_{sd} = (Limitation of steel stress to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 12,3 *10°/(1000 * 188 ² * 25,1852) = 0,014

ω = 0,014

erf. a_s = 1000 * 0,014 * 188 * 25,1852 / 457,0 = **1,45** cm²/m

vorh. a_s = 3,85 cm²/m

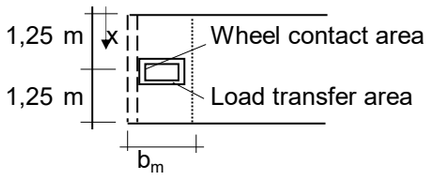
Calculation of reinforcement due to wheel load Pos SL 300
governing cross-section with slab thickness d1

The wheel loads are to be placed at the most unfavorable point for the respective internal forces.
The internal forces are determined on a effective beam. The width of the beam is
determined according to table 2.1 of booklet 631 of the DAfStb. For the load application area
the load transfer area may be increased.

Load transfer area = ty x tx = (0.2 + h) m x (0.2 + h) m = 0,303 m x 0,303 m

Bending reinforcement at the free edge of the slab

governing load position:



effective beam width bm:

b_m = ty + 2,5 * x * (1- x/L)/2

maximum moment at x = L/2 = 2,5 m / 2 = 1,25 m

b_m = 0,303 + 2,5 * 1,25 m * (1 - 1,25 m / 2,5 m) / 2+ 0 m = 1,0843 m

Internal forces gG= 1,35 **γ_Q = 1,35**

m_g (inf. EG Platte) =	1,35	*	4,1	kN/m² *	2,5	²m / 8	=	4,3 kNm/m
Mp/bm (from point load) =	1,5	*	10,0	kN *	2,5	m / 4		
	* 1 /	1,08425	m				=	8,6 kNm/m
m_{ges}							=	13,0 kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 111 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B 500 S/M (A)

σ_{sd} = (Limitation of steel stress to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 13,0 *10^o/(1000 * 111 ² * 25,1852) = 0,042

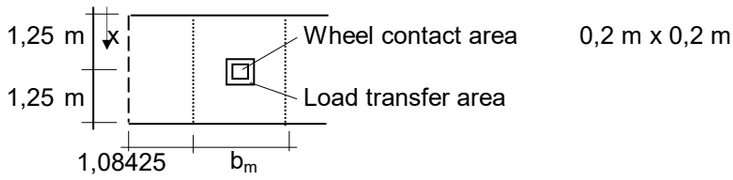
ω = 0,043

erf. a_s = 1000 * 0,043 * 111 * 25,1852 / 457,0 = **2,62** cm²/m

vorh. a_{sl} auf 1,0 m = 3,85 cm²/m

Verification of bending reinforcement **1,0843 m from free edge** Pos SL 300

governing load position:



effective slab width bm:

b_m = ty + 2,5 * x * (1- x/L)

maximum moment at x = L/2 = 2,5 m / 2 = 1,25 m

b_m = 0,303 + 2,5 * 1,25 m * (1 - 1,25 m / 2,5 m) = 1,9 m

Internal forces gG= 1,35 **γ_Q = 1,5**

mg (from panel s.w.) =	1,35	*	4,1	kN/m² *	2,5	²m / 8	=	4,3	kNm/m
Mp/bm (from point load) =	1,5	*	10,0	kN *	2,5	m / 4			
	* 1 /	1,8655	m				=	5,0	kNm/m
m_{ges}							=	9,3	kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 111 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B 500 S/M (A)

σ_{sd} = (limitation of steel stress to ftd,cal) = 457,0 N/mm²

μ_{Eds} = 9,3 *10^o/(1000 * 111 ² * 25,1852) = 0,030

ω = 0,031

erf. a_s = 1000 * 0,031 * 111 * 25,1852 / 457,0 = **1,88** cm²/m

vorh a_s = 3,85 cm²/m

Shear force design with load at free slab edge Pos SL 300

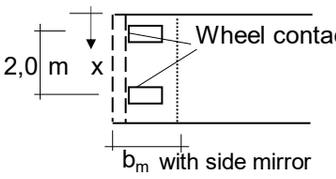
governing load position:

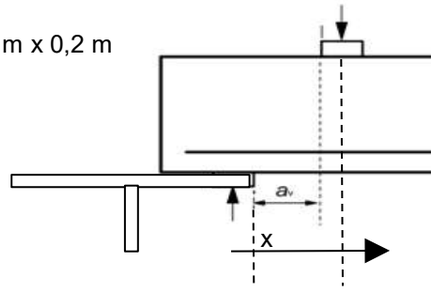
Several load positions are considered. For concentrated loads near the support, the shear force is reduced, so that a concentrated load slightly further away from the edge can govern. The width of the side mirror is assumed to be 11.2 cm.

Rad 1 (R1)

(car to car) Row 1 (R1)

Row 2 (R2)





$V_{Rd,ct} = (C_{Rd,c} * 0{,}1 * k * (100 * \rho_1 * f_{ck})^{(1/3)}) * b_w * d$

$C_{Rd,c}$ = 0,11 b_w = 1 m

k = 1 + wurzel(200/d) = 2,68 --> 2,00

$\rho_1 = A_{s1}/(b_w * d)$ = (3,85 + 0,00)/(100* 7,10) = 0,0054

$v_{E,d} = (1{,}35 * V_{Eg} + 1{,}50 * (V_{E,R1} + V_{E,R2})) * b_m$

$V_{Eg} = A_{g,k} - g * x_v$

$V_{E,R1(2)} = \beta * R1(2) * y / I_{eff}$

$b_m = 7 * d_f + 0{,}5 * b_0$ general (2.3a)

$b_m = 7 * d_f + 1{,}0 * b_0$ close to support (2.3b)

$x_v = d$

min $\beta = a_v/(2{,}0 * d) \le 1$

min $a_v \ge 0{,}5 * d$

V_{Eg} = Shear force from panel

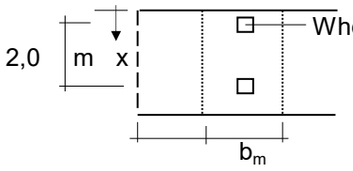
VE,R1(2) = shear force from whee

x	a_v	β	V_{Eg}	$b_{m,R1}$	$V_{E,R1}$	$v_{E,R1}$	$b_{m,R2}$	$V_{E,R2}$	$V_{E,R2}$	$v_{E,d}$	$v_{Rd,ct}$	NW
m	m		kN/m	m	kN	kN/m	m	kN	kN/m	kN/m	kN/m	
0,10	0,04	0,25	4,83	0,56	2,40	4,28	0,46	1,54	3,34	17,96	44,0	0,41
0,20	0,10	0,70	4,83	0,46	6,48	14,07	0,46	1,14	2,48	31,34	44,0	0,71
0,30	0,20	1,00	4,83	0,46	8,80	19,11	0,46	0,74	1,61	37,60	44,0	0,85
0,35	0,25	1,00	4,83	0,46	8,60	18,68	0,46	0,54	1,17	36,30	44,0	0,82

shear force design for load more than 0.25 m from edge

governing load position:

Several load positions are considered. For concentrated loads near the support, the shear force is reduced, so that a concentrated load slightly further away from the edge can govern.



Wheel contact area 0,2 m x 0,2 m

$V_{Rd,ct} = (C_{Rd,c} * 0{,}1 * k(100 * \rho_1 * f_{ck})^{(1/3)}) * b_w * d$

$C_{Rd,c}$ = 0,11 b_w = 1 m

k = 1 + wurzel(200/d) = 2,68 --> 2,00

$\rho_1 = A_{s1}/(b_w * d)$ = (3,85 + 0,00)/(100* 7,1) = 0,0054

$v_{E,d} = (1{,}35 * V_{Eg} + 1{,}50 * (V_{E,R1} + V_{E,R2})) * b_m$

$b_m = 7 * d_f + 0{,}5 * b_0$ general (2.3a)

$b_m = 7 * d_f + 1{,}0 * b_0$ close to support (2.3b)

$V_{Eg} = A_{g,k} - g * x_v$ $x_v = d$ V_{Eg} = Shear force from panel

$V_{E,R1(2)} = \beta * R1(2) * y / I_{eff}$ min $\beta = a_v/(2{,}0 * d) \le 1$ VE,R1(2) = shear force from whee

min $a_v \ge 0{,}5 * d$

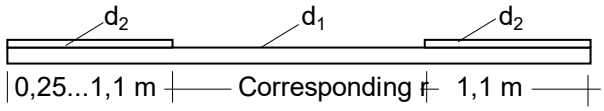
x	a_v	β	V_{Eg}	$b_{m,R1}$	$V_{E,R1}$	$v_{E,R1}$	$b_{m,R2}$	$V_{E,R2}$	$V_{E,R2}$	$v_{E,d}$	$v_{Rd,ct}$	NW
m	m		kN/m	m	kN	kN/m	m	kN	kN/m	kN/m	kN/m	
0,10	0,04	0,25	4,83	0,70	2,40	3,44	0,60	1,54	2,58	15,56	44,0	0,35
0,20	0,10	0,70	4,83	0,60	6,48	10,85	0,60	1,14	1,91	25,67	44,0	0,58
0,30	0,20	1,00	4,83	0,60	8,80	14,74	0,60	0,74	1,24	30,50	44,0	0,69
0,35	0,25	1,00	4,83	0,60	8,60	14,41	0,60	0,54	0,90	29,49	44,0	0,67

Pos SL301 Ramp panel

Simply supported panels are placed, which are connected with reinforcement loops to the composite beams

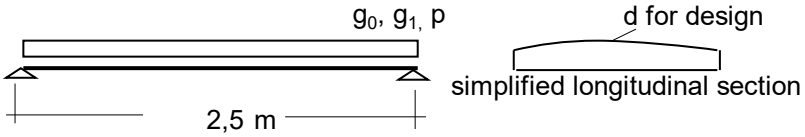
Pos SL301

Cross section:



Thickness d1 = 14 cm
Thickness d2 = 22 cm

System:



Durability:

Bottom side of slab: XC3, XF1 --> C25/30 required; C40/50 present

c_{min} = 20 - 5 = 15mm; $\triangle c$ = 15 - 5 = 10mm --> 15 + 10 = $\mathbf{c_{nom}}$ = **25 mm**

Top side of slab: XC4, XD3; XF4 --> air-entrained concrete C30/37 req.; air-entrained concrete C40/50 prese

c_{min} = 40 - 5 = 35mm; $\triangle c$ = 15 - 5 = 10mm --> 35 + 10 = $\mathbf{c_{nom}}$ = **45 mm**

(Decrease cmin by 5 mm, as the actual concrete strength class is 2 classes higher than the minimum required class. A reduction of Dc by 5 mm is achieved by a corresponding quality control during planning, design and construction.)

Design for cross-section area with slab thickness d1

load:

from d =	14 cm reinforced concrete:	0,14	m	*	26	kN/m³	= g =	3,6 kN/m²
from passenger car live load < 3.0 t							p =	5,0 kN/m²

Internal forces gF = 1.0:

$A_{g,k} = B_{g,k}$ =	2,5	m / 2 *	3,6	kN/m²	=	4,55	kN/m
$A_{p,k} = B_{p,k}$ =	2,5	m / 2 *	5,0	kN/m²	=	6,25	kN/m

Internal forces gG= 1,35 γ_Q = 1,5

$A_d = B_d$ =	(	1,35	* g +	1,50	* p)*	2,5	/ 2	=	15,5	kN/m
max M_d =	(	1,35	* g +	1,50	* p)*	2,5 ²	/ 8	=	9,7	kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with: C40/50 c_{nom} = 25 mm Height d (constr.) = 111 mm

f_{cd} = 0,85 * 40,0 N/mm²/ 1,35 = 25,2 N/mm²

B 500A S/M

σ_{sd} = 500 N/mm²/ 1,15 = 434,8 N/mm²

μ_{Eds} = 9,7 *10°/(1000 * 111 ² * 25,185) = 0,031

ω = 0,032

erf. a_s = 1000 * 0,032 * 111 * 25,185 / 434,8 = **2,05** cm²/m

Pos SL301

Design for cross-section area with slab thickness d2
load:

from d =	22 cm reinforced concrete:	0,22	m	*	26	kN/m³	= g =	5,7 kN/m²
from passenger car live load < 3.0 t							p =	5,0 kN/m²

Internal forces gF = 1.0:

$A_{g,k} = B_{g,k}$ =	2,5	m / 2 *	5,7	kN/m²	=	7,15	kN/m
$A_{p,k} = B_{p,k}$ =	2,5	m / 2 *	5,0	kN/m²	=	6,25	kN/m

Internal forces gG= 1,35 γ_Q = 1,5

$A_d = B_d$ =	(	1,35	* g +	1,50	$*p_{bem.}$)* a =	/ 2	=	19,0	kN/m
max M_d =	(	1,35	* g +	1,50	$*p_{bem.}$)* a =	² / 8	=	11,9	kNm/m

Design: γ_C = 1,35 α = 0,85 γ_S = 1,15

Calculation with:	C40/50	c_{nom} =	25	mm	Height d (constr.) =	188	mm	
	f_{cd} =	0,85 *	40,0	N/mm²/	1,35	=	25,2	N/mm²
	B 500A S/M							
	σ_{sd} =	500	N/mm²/	1,15		=	434,8	N/mm²

μ_{Eds} = 11,9 $*10^o$/(1000 * 188 ² * 25,185) = 0,013

ω = 0,014

erf. a_s = 1000 * 0,014 * 188 * 25,185 / 434,8 = **1,47** cm²/m

reinforcement:

top	**Custom mesh with Asl = 1.31 cm²/m**		
bottom:	**Custom mesh with vorh asl = 3.79 cm²/m**	**>**	**2,05 cm²/m**
Slab thickness:	**14,0 cm**	**Air-entrained concrete C40/50**	**B500 A M/S**
Exposure classes: XC4, XD3; XF4			
cnom bottom =	**25 mm**	**cnom top =**	**45 mm**

Due to the greater slab thickness, if the reinforcement is the same, the verification for the wheel loads as concentrated loads does not govern (cf. Pos SL300).

Pos SL600 Soil pressure load distribution

In Gridline A the lowest level is approx. **1,50 m** in the ground.

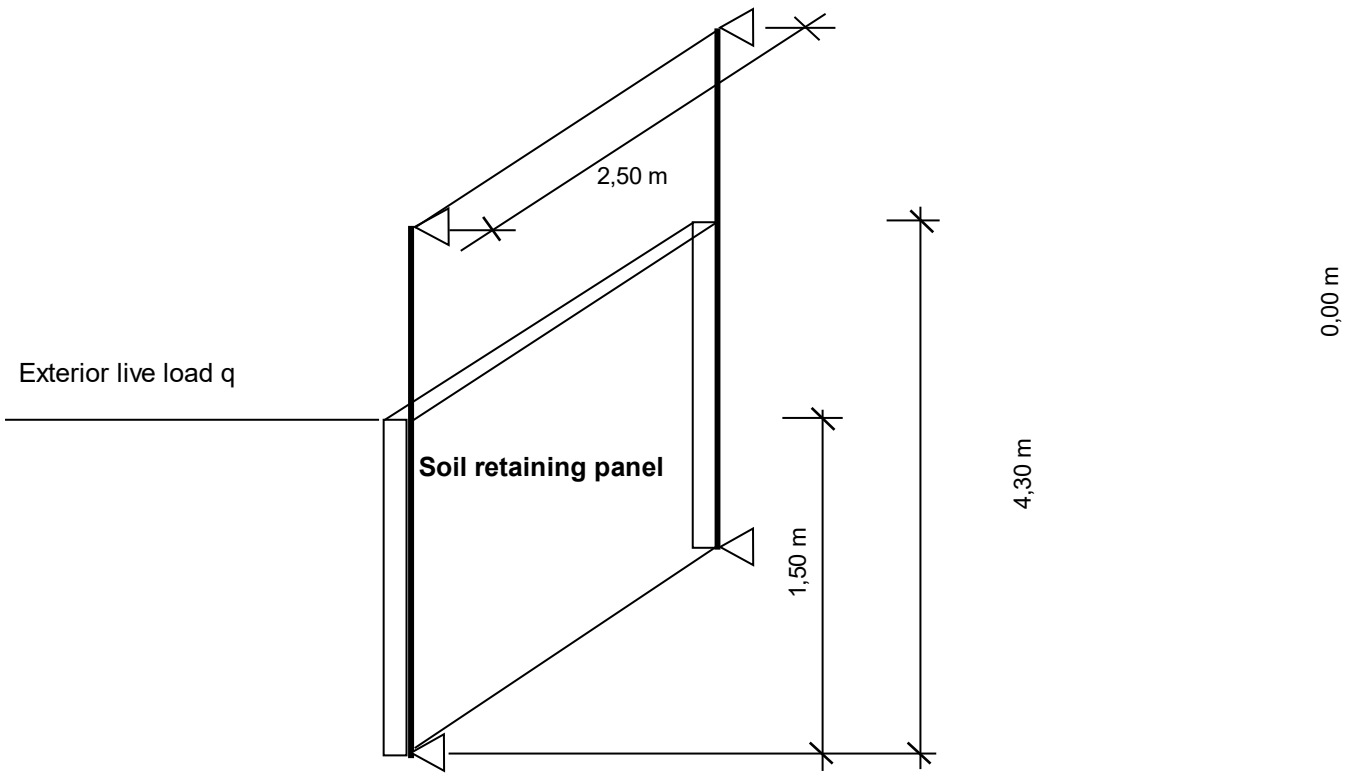
1. Construction method:

wall panel supported of the column

2.) Geometry specifications:

Precast concrete wall panels are supported in front of the columns to retain the soil pressure, which then transfer the load into the columns.

Storey height:	4,30 m	
Soil pressure height:	1,50 m	
Column spacing:	2,50 m	= Span wall panel



3. Soil pressure specifications:

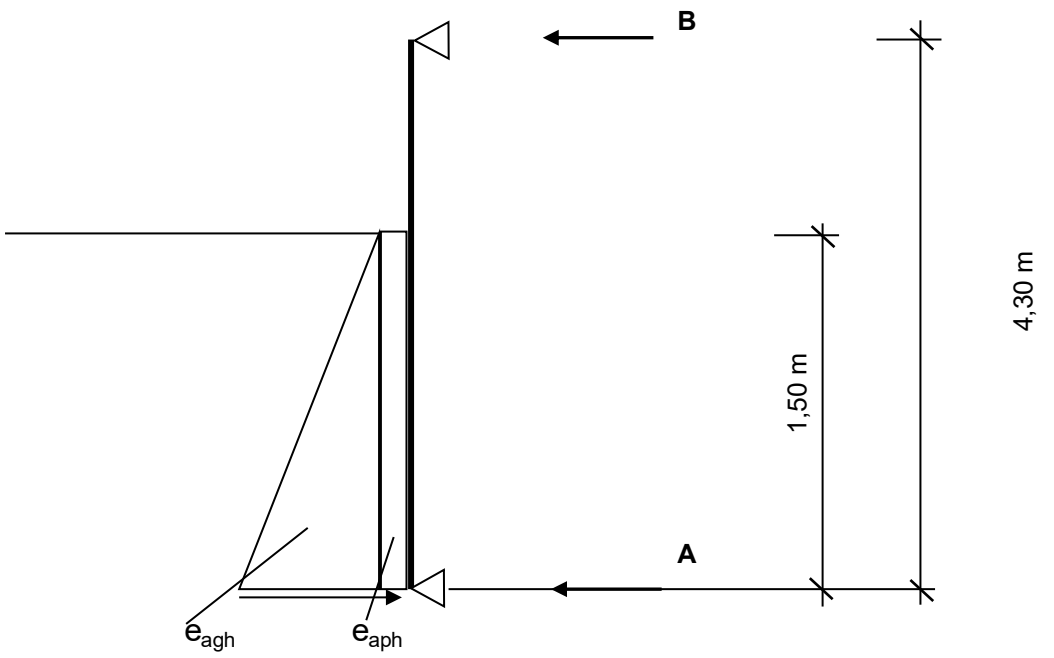
specific gravity	γ =	19,0 kN/m³	
live load	q =	3,0 kN/m²	
internal friction angle	ϕ =	30,0 °	
wall friction angle	δ =	10,0 °	= 1/3 * ϕ
Slope of terrain	β =	0,0 °	

4. Soil pressure coefficients:

active soil pressure coefficient	K_a =	0,30
resting soil pressure coefficient	K_0 =	0,50

The load is applied with the increased active earth pressure coefficient K0' :			component of the resting pressure:	75,0 %
increased active soil pressure coefficient	K_0' =	**0,45**		= 0,25 * Ka + 0,75 * Ko

5. Soil pressure load figure:



Load from soil pressure:	e_{agh} =	12,9 kN/m²	= 19,0 kN/m³	*	1,50 m *	0,45
Load from live load:	e_{aqh} =	1,4 kN/m²	= 3,0 kN/m²	*	0,45	

Total pressure: **e = 14,2 kN/m²**

---

6. Bending on columns:

(influence width: 2,50 m)

Support reactions of the colum	γ_F=1,0-fach		per column
Support reaction at column base f	e_{agh}:	A =	21,3 kN
Support reaction at column base f	e_{aqh}:	A =	4,2 kN
		Total at base of column:	**25,5 kN**
Support reaction at column top fro	e_{agh}:	B =	2,8 kN
Support reaction at column top fro	e_{aqh}:	B =	0,9 kN
		Total at top of column:	**3,7 kN**

Loads on diaphragm:	γ_F=1,0-fach		per linear meter
load on diaphragm from:	e_{agh}:	B =	1,1 kN/m
load on diaphragm from:	e_{aqh}:	B =	0,4 kN/m
		Total on diaphragm:	**1,5 kN/m**

Bending moment in column:	γ_F=1,0-fach		per column
maximum moment from	e_{agh}:	M_{max} =	8,8 kNm
maximum moment from	e_{aqh}:	M_{max} =	2,6 kNm
		Total:	**11,4 kNm**

γ_G= 1,35 $\gamma_{G,Eo}$= 1,2 the soil pressure portion is from 75% of the earth pressure height **γ_G'= 1,24**

Design moment: (γ_F= 1,24 / 1,50 - fach) **Md = 14,8 kNm** = 1,24 * 8,8 + 1,5 * 2,6 kNm

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7. Transfer to foundation:

Linear load on foundation:	γ_F=1,0-fach		per linear meter	
Dead load from precast wall panels:		g_k =	**0,0 kN/m**	
equivalent load in steel column:	γ_F=1,0-fach		Load influence width	1,25 m
Dead load from precast wall panels:		G_k =	**7,1 kN**	

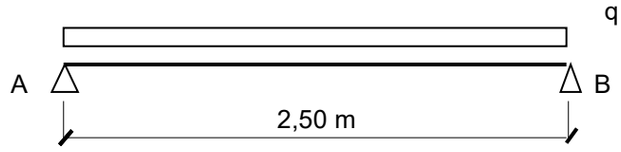
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8. Design of precast soil retaining panels:

see next Page

Pos SL600 Wall panel with soil pressure (supported on two sides)

1. SYSTEM Simply supported precast panels are placed along the columns.



h = 14 cm

2. LOADS

from soil pressure e_{agh} = 12,9 kN/m²
from live load e_{aqh} = 1,4 kN/m²

3. INTERNAL FORCES

γ = 1,00

$A_{g,k} = B_{g,k}$ = 12,9 * 2,50 / 2 = **16,1 kN/m**
$A_{q,k} = B_{q,k}$ = 1,4 * 2,50 / 2 = **1,7 kN/m**

4. INTERNAL FORCES

γ_G'= 1,24
γ_Q = 1,50

e_d = 1,24 * 12,9 + 1,50 * 1,4 = 17,9 kN/m²
$A_d = B_d$ = 17,9 * 2,50 / 2 = **22,4 kN/m**

The bending moment of the wall plate due to soil pressure is multiplied by the factor a in order to account for the triangular shape of the load.

at l/h= 1,67 -->a = 0,58

m_{Eds} = (1,24*12,9*a+1,50 *1,4) *2,5²/8 = **8,8 kNm/m**

5. DESIGN

Partial safety factors for resistance:

Concrete		γ_c =	1,35	precast element
Reinforcing steel		γ_s =	1,15	
Long-term factor concrete:		αcc =	0,85	
Concrete	C40/50	f_{ck} =	40 N/mm²	f_{ctm} = 3,5 N/mm²
		f_{cd} =	25 N/mm²	
Reinforcing steel	B 500	f_{yd} =	435 N/mm²	

Exposure class:

airside:	XC4	c_{nom} =	3,0 cm	reduce by 5 mm because pre-cast-element
				reduce by 5 mm because higher concrete quality
soil side:	XD2	c_{nom} =	5,0 cm	reduce by 5 mm because pre-cast-element
Fire resistance:	REI 30	c_{nom} =	1,0 cm	
		d_L =	10,5 cm	z = 10,3 cm

$\mu_{Eds} = m_{Eds} / (b * d_L^2 * f_{cd})$ = 0,0315
ω_1 = 0,0322
σ_{sd} = 456,5 N/mm²
$\zeta = z / d_L$ = 0,979

$a_{s1} = \omega_1 * b * d_L$' = **1,86 cm²/m**
Mindestbew. min as = 2,43 cm²/m

selected: d = 14 cm C40/50 B 500
airside: Q 257 A = 2,57 cm²/m
c_{nom} = 3,0 cm XC4, XF1
earth side: Q 188 A = 1,88 cm²/m
c_{nom} = 5,0 cm XD2, XC2, XF1
Top & bottom edges: 2 Ø 8 + Bü Ø 6 / 25 all around
Side edges: 2 Ø 8 + Bü Ø 6 / 20 all around

alternatively: standard mesh with the corresponding cross-section

Pos SL700 Soil pressure load distribution

In Gridline A the lowest level is approx. **1,50 m** in the ground.

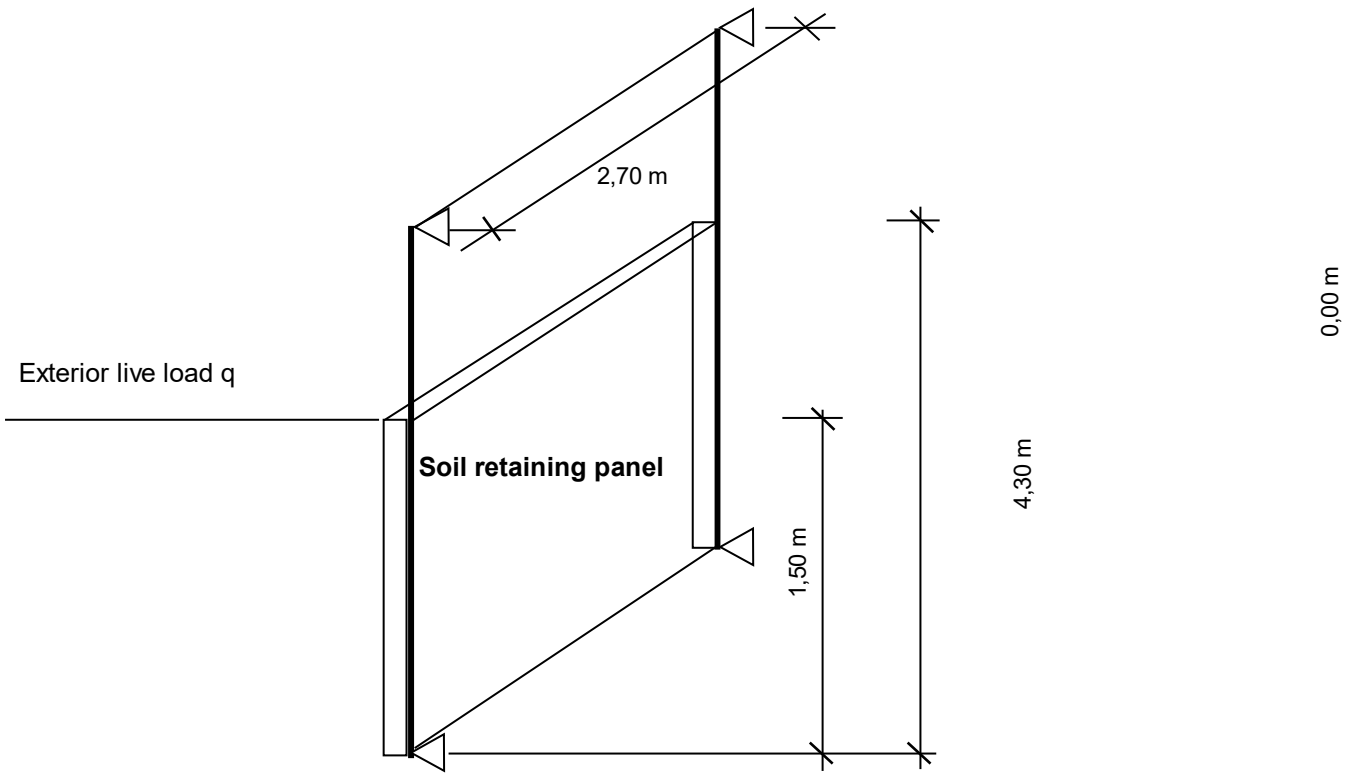
1. Construction method:

wall panel supported of the column

2.) Geometry specifications:

Precast concrete wall panels are supported in front of the columns to retain the soil pressure, which then transfer the load into the columns.

Storey height:	4,30 m	
Soil pressure height:	1,50 m	
Column spacing:	2,70 m	= Span wall panel



3. Soil pressure specifications:

specific gravity	γ =	19,0 kN/m³	
live load	q =	3,0 kN/m²	
internal friction angle	ϕ =	30,0 °	
wall friction angle	δ =	10,0 °	= 1/3 * ϕ
Slope of terrain	β =	0,0 °	

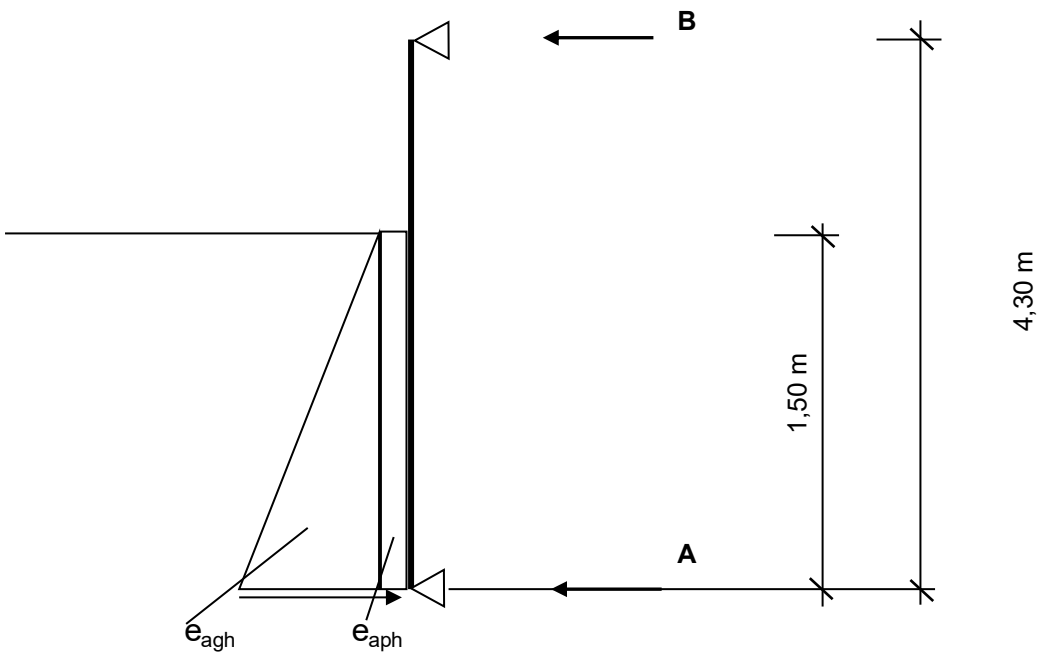
4. Soil pressure coefficients:

active soil pressure coefficient	K_a =	0,30
resting soil pressure coefficient	K_0 =	0,50

The load is applied with the increased active earth pressure coefficient K0' : component of the resting pressure: 75,0 %

increased active soil pressure coefficient	K_0' =	**0,45**	= 0,25 * Ka + 0,75 * Ko

5. Soil pressure load figure:



Load from soil pressure:	e_{agh} =	12,9 kN/m²	= 19,0 kN/m³	*	1,50 m *	0,45
Load from live load:	e_{aqh} =	1,4 kN/m²	= 3,0 kN/m²	*	0,45	

Total pressure: **e = 14,2 kN/m²**

---

6. Bending on columns:

(influence width: 2,70 m)

Support reactions of the colum γ_F=1,0-fach per column

Support reaction at column base f	e_{agh}:	A =	23,0 kN
Support reaction at column base f	e_{aqh}:	A =	4,5 kN
		Total at base of column:	**27,5 kN**
Support reaction at column top fro	e_{agh}:	B =	3,0 kN
Support reaction at column top fro	e_{aqh}:	B =	1,0 kN
		Total at top of column:	**4,0 kN**

Loads on diaphragm: γ_F=1,0-fach per linear meter

load on diaphragm from:	e_{agh}:	B =	1,1 kN/m
load on diaphragm from:	e_{aqh}:	B =	0,4 kN/m
		Total on diaphragm:	**1,5 kN/m**

Bending moment in column: γ_F=1,0-fach per column

maximum moment from	e_{agh}:	M_{max} =	9,5 kNm
maximum moment from	e_{aqh}:	M_{max} =	2,8 kNm
		Total:	**12,3 kNm**

γ_G= 1,35 $\gamma_{G,Eo}$= 1,2 the soil pressure portion is from 75% of the earth pressure height **γ_G'= 1,24**

Design moment: (γ_F= 1,24 / 1,50 - fach) **Md = 16,0 kNm** = 1,24 * 9,5 + 1,5 * 2,8 kNm

---

7. Transfer to foundation:

Linear load on foundation: γ_F=1,0-fach per linear meter

Dead load from precast wall panels: g_k = **0,0 kN/m**

equivalent load in steel column: γ_F=1,0-fach Load influence width 1,35 m

Dead load from precast wall panels: G_k = **7,6 kN**

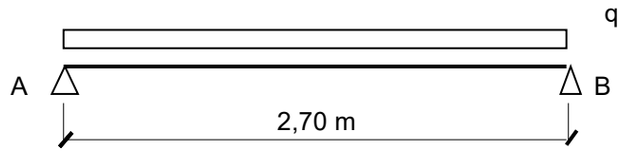
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8. Design of precast soil retaining panels:

see next Page

Pos SL700 Wall panel with soil pressure (supported on two sides)

1. SYSTEM Simply supported precast panels are placed along the columns.



h = 14 cm

2. LOADS

from soil pressure e_{agh} = 12,9 kN/m²
from live load e_{aqh} = 1,4 kN/m²

3. INTERNAL FORCES

γ = 1,00

$A_{g,k} = B_{g,k}$ = 12,9 * 2,70 / 2 = **17,4 kN/m**
$A_{q,k} = B_{q,k}$ = 1,4 * 2,70 / 2 = **1,8 kN/m**

4. INTERNAL FORCES

γ_G'= 1,24
γ_Q = 1,50

e_d = 1,24 * 12,9 + 1,50 * 1,4 = 17,9 kN/m²
$A_d = B_d$ = 17,9 * 2,70 / 2 = **24,2 kN/m**

The bending moment of the wall plate due to soil pressure is multiplied by the factor a in order to account for the triangular shape of the load.

at l/h= 1,80 -->a = 0,57

m_{Eds} = (1,24*12,9*a+1,50 *1,4) *2,7²/8 = **10,1 kNm/m**

5. DESIGN

Partial safety factors for resistance:

Concrete		γ_c =	1,35		precast element
Reinforcing steel		γ_s =	1,15		
Long-term factor concrete:		αcc =	0,85		
Concrete	C40/50	→ f_{ck} =	40 N/mm²	f_{ctm} =	3,5 N/mm²
		f_{cd} =	25 N/mm²		
Reinforcing steel	B 500	→ f_{yd} =	435 N/mm²		
Exposure class:					
airside:	XC4	→ c_{nom} =	3,0 cm		reduce by 5 mm because pre-cast-element
					reduce by 5 mm because higher concrete quality
soil side:	XD2	→ c_{nom} =	5,0 cm		reduce by 5 mm because pre-cast-element
Fire resistance:	REI 30	→ c_{nom} =	1,0 cm		
		d_L =	10,5 cm	z =	10,3 cm

$\mu_{Eds} = m_{Eds} / (b * d_L^2 * f_{cd})$ = 0,0363
ω_1 = 0,0372
σ_{sd} = 456,5 N/mm²
$\zeta = z / d_L$ = 0,977

$a_{s1} = \omega_1 * b * d_L$' = **2,15 cm²/m**
Mindestbew. min as = 2,43 cm²/m

selected:	**d =**	**14 cm C40/50 B 500**	
airside:	**Q 257 A**	=	**2,57 cm²/m**
	c_{nom} = 3,0 cm XC4, XF1		
earth side:	**Q 188 A**	=	**1,88 cm²/m**
	c_{nom} = 5,0 cm XD2, XC2, XF1		
Top & bottom edges:	**2 Ø 8 + Bü Ø 6 / 25 all around**		
Side edges:	**2 Ø 8 + Bü Ø 6 / 20 all around**		

alternatively: standard mesh with the corresponding cross-section

Chapter 6: Steel columns

Pos. CO001 ff.

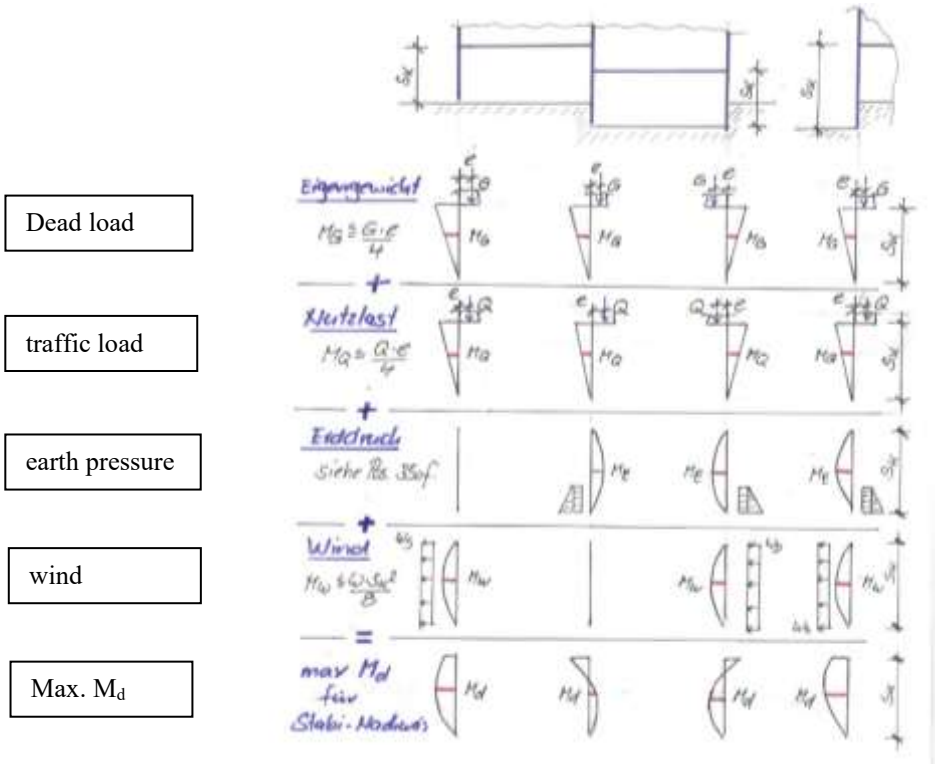
All columns of the car park are summarized under this position. First, the bearing loads of the individual girder positions are compiled.

Load case: standard dimensioning:

- Dead loads
- Loads from traffic and installation
- Bracing loads from wind, earth pressure and misalignment
- Wind loads on the columns

→ Partial safety factors γ_F= 1,35 and 1,50

Since the traffic loads are consistently applied with a factor of 1.50, slight exceedances of the load-bearing capacity can be accepted. The girder loads are assigned to the columns by floor. The vertical loads from the bracing and from the facade are unfavourably applied to the column head. In addition to the vertical loads, bending moments from wind, earth pressure and eccentric girder connection are also considered.



The column design is presented in tabular form for the lowest (critical) storey using the equivalent member method. In addition, a separate dimensioning is carried out for the load case vehicle impact.

Pos CO001	column design						
project:	PR0460	MSCP Neratovice					
summary of the support reactions							v25.30
Pos	profile	steel grade	length	G_k	P_k	wind suction	A_d
					γ = 1,00		γ = 1,35/1,50
			[m]		[kN]		[kN]
ramp roof							
RO560	HE 220 A	**S 355**	7,30	5,0	15,5		**30,0**
floor beams							
BE034	IPE 360	S 235	11,10	42,0	35,0		**109,2**
BE052	IPE 400	**S 355**	16,10	60,5	50,5		**157,4**
BE134	IPE 360	S 235	11,10	45,0	38,0		**117,8**
BE152	IPE 400	**S 355**	16,10	65,0	55,0		**170,3**
BE281	L 150*10	S 235	2,00	4,5	4,0		**12,1**
BE282A	L 150*10	S 235	5,00	4,0	4,5		**12,2**
BE282B	L 150*10	S 235	5,00	12,5	12,5		**35,6**
BE283A	L 150*10	S 235	6,30	3,5	4,0		**10,7**
BE283B	L 150*10	S 235	6,30	9,5	10,0		**27,8**
BE302	HE 200 A	S 235	7,30	37,0	23,0		**84,5**
BE312A	HE 180 A	S 235	6,00	4,5	4,0		**12,1**
BE312B	HE 180 A	S 235	6,00	12,5	9,5		**31,1**
BE402	welded beam	**S 355**	7,50	60,0	47,5		**152,3**
BE412	welded beam	**S 355**	7,50	47,0	35,0		**116,0**
BE450	welded beam	**S 355**	7,50	107,5	85,5		**273,4**
BE451	HE 180 B	**S 355**	7,50	191,0	169,5		**512,1**

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project: PR0460 MSCP Neratovice | **Pos CO001**

load composition per column [kN or kNm] γ_F = 1,0 v25.30

n = number of floors | **full load + wind + imperfection**

location	n	loads	roof	1,0							ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing			moments from wind & eccentricity		wind suction on roof
	3	from-to	G	P							G	P	G	P	G	P	G	± V	± H_x	± H_y	$M_{y,k}$	$M_{z,k}$	W_{sd}
																					kNm	kNm	kN
A / 2	3	GF-roof	45	38							45	38	45	38		77	7			24	19,1		
A / 3	3	GF-roof	42	35							42	35	42	35		77	7			22	17,7		
A / 4	3	GF-roof	42	35							42	35	42	35		77	7			22	16,5		
A / 5	3	GF-roof	45	38							45	38	45	38		77	7			24	17,9		
A / 6	3	GF-roof	65	55							65	55	65	55		77	7			24	19,2		
A / 7	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 8	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 9	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 10	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 11	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 12	3	GF-roof	61	51							61	51	61	51		77	7	80	29	22	17,8		
A / 13	3	GF-roof	61	51							61	51	61	51		77	7	40	29	22	17,8		
A / 14	3	GF-roof	61	51							61	51	61	51		77	7	80	29	22	17,8		
A / 15	3	GF-roof	61	51							61	51	61	51		77	7	80	29	22	17,8		
A / 16	3	GF-roof	61	51							61	51	61	51		77	7	40	29	22	17,8		
A / 17	3	GF-roof	61	51							61	51	61	51		77	7	80	29	22	17,8		
A / 18	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 19	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 20	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 21	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 22	3	GF-roof	61	51							61	51	61	51		77	7			22	17,8		
A / 23	3	GF-roof	65	55							65	55	65	55		77	7			24	19,2		
A / 24	3	GF-roof	45	38							45	38	45	38		77	7			24	17,9		
A / 25	3	GF-roof	42	35							42	35	42	35		77	7			22	16,5		
A / 26	3	GF-roof	42	35							42	35	42	35		77	7			22	17,7		
A / 27	3	GF-roof	45	38							45	38	45	38		77	7			24	19,1		
H / 6	3	GF-roof	130	110							130	110	130	110		9	5			22	21,7		
H / 7	3	GF-roof	121	101							121	101	121	101		9	5			20	20,1		
H / 8	3	GF-roof	121	101							121	101	121	101		9	5			20	20,1		
H / 9	3	GF-roof	121	101							121	101	121	101		9	5			20	20,1		
H / 10	3	GF-roof	121	101							121	101	121	101		9	5			20	20,1		
H / 11	3	GF-roof	121	101							121	101	121	101		9	5	90	35	20	20,1		
H / 12	3	GF-roof	121	101							121	101	121	101		9	5	45	35	20	20,1		
H / 13	3	GF-roof	121	101							121	101	121	101		9	5	45	35	20	20,1		
H / 14	3	GF-roof	121	101							121	101	121	101		9	5	90	35	20	20,1		
H / 15	3	GF-roof	121	101							121	101	121	101		9	5			20	20,1		
H / 16	3	GF-roof	121	101							121	101	121	101		9	5	90	35	20	20,1		
H / 17	3	GF-roof	121	101							121	101	121	101		9	5	45	35	20	20,1		
H / 18	3	GF-roof	121	101							121	101	121	101		9	5	45	35	20	20,1		

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project: PR0460 MSCP Neratovice — **Pos CO001**

load composition per column [kN or kNm] γ_F = 1,0 — v25.30

location	n	loads	full load + wind + imperfection																				moments from wind & eccentricity		wind suction on roof
			roof	1,0									ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing					
	3	from-to	G	P									G	P	G	P	G	P	G	± V	± H_x	± H_y	$M_{y,k}$	$M_{z,k}$	W_{sd}
																							kNm	kNm	kN
H / 19	3	GF-roof	121	101									121	101	121	101		9	5	90	35	20	20,1		
H / 20	3	GF-roof	121	101									121	101	121	101		9	5			20	20,1		
H / 21	3	GF-roof	121	101									121	101	121	101		9	5			20	20,1		
H / 22	3	GF-roof	121	101									121	101	121	101		9	5			20	20,1		
H / 23	3	GF-roof	130	110									130	110	130	110		9	5			22	21,7		
O / 2	3	GF-roof	218	179									218	179	218	179		9	6				6,0	15,4	
O / 5	3	GF-roof	218	179									218	179	218	179		9	6				6,0	15,4	
O / 6	3	GF-roof	130	110									130	110	130	110		9	5				4,4		
O / 7	3	GF-roof	121	101									121	101	121	101		9	5				4,0		
O / 8	3	GF-roof	121	101									121	101	121	101		9	5				4,0		
O / 9	3	GF-roof	121	101									121	101	121	101		9	5				4,0		
O / 10	3	GF-roof	121	101									121	101	121	101		9	5				4,0		
O / 11	3	GF-roof	121	101									121	101	121	101		9	5	79	33		4,0		
O / 12	3	GF-roof	121	101									121	101	121	101		9	5	40	33		4,0		
O / 13	3	GF-roof	121	101									121	101	121	101		9	5	40	33		4,0		
O / 14	3	GF-roof	121	101									121	101	121	101		9	5	79	33		4,0		
O / 15	3	GF-roof	121	101									121	101	121	101		9	5				4,0		
O / 16	3	GF-roof	121	101									121	101	121	101		9	5	79	33		4,0		
O / 17	3	GF-roof	121	101									121	101	121	101		9	5	40	33		4,0		
O / 18	3	GF-roof	121	101									121	101	121	101		9	5	40	33		4,0		
O / 19	3	GF-roof	121	101									121	101	121	101		9	5	79	33		4,0		
O / 20	3	GF-roof	121	101									121	101	121	101		9	5				4,0		
O / 21	3	GF-roof	121	101									121	101	121	101		9	5				4,0		
O / 22	3	GF-roof	121	101									121	101	121	101		9	5				4,0		
O / 23	3	GF-roof	130	110									130	110	130	110		9	5				4,4		
O / 24	3	GF-roof	218	179									218	179	218	179		9	6				6,0	15,4	
O / 27	3	GF-roof	218	179									218	179	218	179		9	6				6,0	15,4	
V / 2	3	GF-roof	65	55									65	55	256	225		29	4				9,6		
V / 3	3	GF-roof	61	51									61	51				22	4				8,9		
V / 4	3	GF-roof	61	51									61	51				22	4				8,9		
V / 5	3	GF-roof	65	55									65	55	256	225		29	4				9,6		
V / 6	3	GF-roof	65	55									65	55	65	55		29	4				9,6		
V / 7	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 8	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 9	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 10	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 11	3	GF-roof	61	51									61	51	61	51		29	4				8,9		

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project: PR0460 MSCP Neratovice — **Pos CO001**

load composition per column [kN or kNm] γ_F = 1,0 — v25.30

location	n	loads	full load + wind + imperfection: roof	roof 1,0									ceiling above 1.FL	ceiling above 1.FL 1,0	ceiling above GF	ceiling above GF 1,0	facade, canopy etc.		column	bracing			moments from wind & eccentricity		wind suction on roof
	3	from-to	G	P									G	P	G	P	G	P	G	± V	± H_x	± H_y	$M_{y,k}$	$M_{z,k}$	W_{sd}
																							kNm	kNm	kN
V / 12	3	GF-roof	61	51									61	51	61	51		29	4	59	25		8,9		
V / 13	3	GF-roof	61	51									61	51	61	51		29	4	30	25		8,9		
V / 14	3	GF-roof	61	51									61	51	61	51		29	4	59	25		8,9		
V / 15	3	GF-roof	61	51									61	51	61	51		29	4	59	25		8,9		
V / 16	3	GF-roof	61	51									61	51	61	51		29	4	30	25		8,9		
V / 17	3	GF-roof	61	51									61	51	61	51		29	4	59	25		8,9		
V / 18	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 19	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 20	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 21	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 22	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 23	3	GF-roof	65	55									65	55	65	55		29	4				9,6		
V / 24	3	GF-roof	65	55									65	55	65	55		29	4				9,6		
V / 25	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 26	3	GF-roof	61	51									61	51	61	51		29	4				8,9		
V / 27	3	GF-roof	65	55									65	55	65	55		29	4				9,6		
1 / A	3	GF-roof	4	5									4	5	4	5		72	6		12	12	8,5	8,1	
1 / B	3	GF-roof	13	13									13	13	13	13		72	6		22		16,0		
1 / C	3	GF-roof	8	9									8	9	8	9		72	6		22		15,7		
1 / D	3	GF-roof	10	10									10	10	10	10		72	6		22		14,6		
1 / E	3	GF-roof	10	10									10	10	10	10		72	6		22		14,6		
1 / F	3	GF-roof	8	8									8	8	8	8		72	6		22		14,5		
1 / F'	3	GF-roof	5	4									5	4	5	4		36	6		11		8,7		
1 / I'	3	GF-roof	5	4									5	4	5	4		34	3				2,0		
1 / J	3	GF-roof	8	8									8	8	8	8		69	3				2,3		
1 / K	3	GF-roof	10	10									10	10	10	10		69	3				2,5		
1 / L	3	GF-roof	10	10									10	10	10	10		69	3				2,5		
1 / M	3	GF-roof	8	9									8	9	8	9		69	3				2,3		
1 / N	3	GF-roof	13	13									13	13	13	13		69	3				2,8		
1 / O	3	GF-roof	8	9									8	9	8	9		69	3				2,4		
1 / P	3	GF-roof	13	13									13	13	13	13		69	3				2,8		
1 / Q	3	GF-roof	8	9									8	9	8	9		69	3				2,3		
1 / R	3	GF-roof	10	10									10	10	10	10		69	3				2,5		
1 / S	3	GF-roof	10	10									10	10	10	10		69	3				2,5		
1 / T	3	GF-roof	8	9									8	9	8	9		69	3				2,9		
1 / U	3	GF-roof	13	13									13	13	13	13		69	3				3,4		
1 / V	3	GF-roof	4	5									4	5	4	5		69	3				1,5	1,1	
28 / A	3	GF-roof	4	5									4	5	4	5		50	6		12	12	8,5	8,1	

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project: PR0460 MSCP Neratovice | **Pos CO001**

load composition per column [kN or kNm] γ_F = 1,0 v25.30

n = number of floors: 3

full load + wind + imperfection

location	n	loads	roof	1,0							ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing			moments from wind & eccentricity		wind suction on roof
	3	from-to	G	P							G	P	G	P	G	P	G	± V	± H_x	± H_y	$M_{y,k}$	$M_{z,k}$	W_{sd}
																					kNm	kNm	kN
28 / B	3	GF-roof	13	13							13	13	13	13		29	6		22		16,0		
28 / C	3	GF-roof	8	9							8	9	8	9		29	6		22		15,7		
28 / D	3	GF-roof	10	10							10	10	10	10		29	6		22		14,6		
28 / E	3	GF-roof	10	10							10	10	10	10		29	6		22		14,6		
28 / F	3	GF-roof	8	8							8	8	8	8		29	6		22		14,5		
28 / F'	3	GF-roof	5	4							5	4	5	4		14	6		11		8,7		
28 / I'	3	GF-roof	5	4							5	4	5	4		12	3				2,0		
28 / J	3	GF-roof	8	8							8	8	8	8		25	3				2,3		
28 / K	3	GF-roof	10	10							10	10	10	10		25	3				2,5		
28 / L	3	GF-roof	10	10							10	10	10	10		25	3				2,5		
28 / M	3	GF-roof	8	9							8	9	8	9		25	3				2,3		
28 / N	3	GF-roof	13	13							13	13	13	13		25	3				2,8		
28 / O	3	GF-roof	8	9							8	9	8	9		25	3				2,4		
28 / P	3	GF-roof	13	13							13	13	13	13		25	3				2,8		
28 / Q	3	GF-roof	8	9							8	9	8	9		25	3				2,3		
28 / R	3	GF-roof	10	10							10	10	10	10		25	3				2,5		
28 / S	3	GF-roof	10	10							10	10	10	10		25	3				2,5		
28 / T	3	GF-roof	8	9							8	9	8	9		25	3				2,9		
28 / U	3	GF-roof	13	13							13	13	13	13		25	3				3,4		
28 / V	3	GF-roof	4	5							4	5	4	5		25	3				1,5	1,1	
2' / F	3	GF-roof	115	105							110	90	110	90			8		20		11,5		-11
2' / F'	3	GF-roof	5	4							5	4	5	4			4		20		9,4		
2' / G	3	GF-roof	42	39							37	23	37	23			5		20		11,6		-11
2' H	3	GF-roof	42	39							37	23	37	23			5		20		11,6		-11
2' / I	3	GF-roof	42	39							37	23	37	23			4		20		11,6		-11
2' / I'	3	GF-roof	5	4							5	4	5	4			3		20		9,4		
2' / J	3	GF-roof	115	105							110	90	110	90			5		20		7,1		-11
4' / F	3	GF-roof	114	106							109	90	109	90			8	324	20	137	11,5		-11
4' / G	3	GF-roof	55	51							50	36	50	36			5	120	20	99	11,0		-11
4' / H	3	GF-roof	50	48							45	32	45	32		5	5	190	10	140	11,4	4,5	-11
4' / I	3	GF-roof	55	51							50	36	50	36			4	157	20	124	11,0		-11
4' / J	3	GF-roof	114	106							109	90	109	90			5	317	20	143	11,5		-11
24' / F	3	GF-roof	101	93							109	90	114	94			5	324	10	147	7,2	4,5	-11
24' / G	3	GF-roof	55	51							50	36	50	36			4	120	20	99	11,0		-11
24' H	3	GF-roof	50	48							45	32	45	32		5	4	190	10	140	7,0	4,5	-11
24' / I	3	GF-roof	55	51							50	36	50	36			4	157		124	3,8		-11
24' / J	3	GF-roof	114	106							109	90	109	90			5	317		143	4,7		-11

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project: PR0460 MSCP Neratovice																						Pos CO001
load composition per column [kN or kNm] γ_F = 1,0																						v25.30
n = number of floors	full load + wind + imperfection																			moments from wind & eccentricity		wind suction on roof
location	n	loads	roof	1,0									ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing			$M_{y,k}$	$M_{z,k}$	W_{sd}
	3	from-to	G	P									G	P	G	P	G	P	G	± V	± H_x	± H_y	kNm	kNm	kN
26' / F	3	GF-roof	102	93									110	90	114	94			5		10	10	7,1	4,5	-11
26' / F'	3	GF-roof	5	4									5	4	5	4			4		10		4,9		
26' / G	3	GF-roof	5	16									37	23	37	23			4				4,8		-11
26' H	3	GF-roof	42	39									37	23	37	23			4				4,8		-11
26' / I	3	GF-roof	42	39									37	23	37	23			4				4,8		-11
26' / I'	3	GF-roof	5	4									5	4	5	4			3				0,7		
26' / J	3	GF-roof	115	105									110	90	110	90			5				4,6		-11
F / 25	3	GF-roof													13	10			3			20	9,9		
F / 26	3	GF-roof													13	10			3			20	9,9		

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project: PR0460 MSCP Neratovice — **Pos CO001**

column design — **full load + wind + imperfection** — γ_{M1} = 1,00 K_{FI} = 1,00 — v25.30

location	joint at storey	number of floors	design below joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,35*G + 1,50*(P+0,6*V)				280	280							Nd= 1,35*G + 1,50*(P+0,6*V)				280	280					
A / 2		3	GF-roof	477	26,6		430	430	HE 180 B	S 355	0,64	(6.62)	S 355												
A / 3		3	GF-roof	452	24,7		430	430	HE 180 B	S 355	0,60	(6.62)	S 355												
A / 4		3	GF-roof	452	22,9		430	430	HE 180 B	S 355	0,59	(6.62)	S 355												
A / 5		3	GF-roof	477	24,8		430	430	HE 180 B	S 355	0,63	(6.62)	S 355												
A / 6		3	GF-roof	635	26,7		430	430	HE 180 B	S 355	0,80	(6.62)	S 355												
A / 7		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 8		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 9		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 10		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 11		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 12		3	GF-roof	668	24,7		430	430	HE 180 B	S 355	0,82	(6.62)	S 355												
A / 13		3	GF-roof	632	24,7		430	430	HE 180 B	S 355	0,79	(6.62)	S 355												
A / 14		3	GF-roof	668	24,7		430	430	HE 180 B	S 355	0,82	(6.62)	S 355												
A / 15		3	GF-roof	668	24,7		430	430	HE 180 B	S 355	0,82	(6.62)	S 355												
A / 16		3	GF-roof	632	24,7		430	430	HE 180 B	S 355	0,79	(6.62)	S 355												
A / 17		3	GF-roof	668	24,7		430	430	HE 180 B	S 355	0,82	(6.62)	S 355												
A / 18		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 19		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 20		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 21		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 22		3	GF-roof	596	24,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 23		3	GF-roof	635	26,7		430	430	HE 180 B	S 355	0,80	(6.62)	S 355												
A / 24		3	GF-roof	477	24,8		430	430	HE 180 B	S 355	0,63	(6.62)	S 355												
A / 25		3	GF-roof	452	22,9		430	430	HE 180 B	S 355	0,59	(6.62)	S 355												
A / 26		3	GF-roof	452	24,7		430	430	HE 180 B	S 355	0,60	(6.62)	S 355												
A / 27		3	GF-roof	477	26,6		430	430	HE 180 B	S 355	0,64	(6.62)	S 355												
H / 6		3	GF-roof	1.042	30,3		280	280	HE 240 A	S 355	0,59	(6.62)	S 355												
H / 7		3	GF-roof	965	28,0		280	280	HE 240 A	S 355	0,55	(6.62)	S 355												
H / 8		3	GF-roof	965	28,0		280	280	HE 240 A	S 355	0,55	(6.62)	S 355												
H / 9		3	GF-roof	965	28,0		280	280	HE 240 A	S 355	0,55	(6.62)	S 355												
H / 10		3	GF-roof	965	28,0		280	280	HE 240 A	S 355	0,55	(6.62)	S 355												
H / 11		3	GF-roof	1.046	28,0		280	280	HE 240 A	S 355	0,59	(6.62)	S 355												
H / 12		3	GF-roof	1.005	28,0		280	280	HE 240 A	S 355	0,57	(6.62)	S 355												
H / 13		3	GF-roof	1.005	28,0		280	280	HE 240 A	S 355	0,57	(6.62)	S 355												
H / 14		3	GF-roof	1.046	28,0		280	280	HE 240 A	S 355	0,59	(6.62)	S 355												
H / 15		3	GF-roof	965	28,0		280	280	HE 240 A	S 355	0,55	(6.62)	S 355												
H / 16		3	GF-roof	1.046	28,0		280	280	HE 240 A	S 355	0,59	(6.62)	S 355												
H / 17		3	GF-roof	1.005	28,0		280	280	HE 240 A	S 355	0,57	(6.62)	S 355												
H / 18		3	GF-roof	1.005	28,0		280	280	HE 240 A	S 355	0,57	(6.62)	S 355												

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project: PR0460 MSCP Neratovice — **Pos CO001**

column design — **full load + wind + imperfection** — γ_{M1} = 1,00 K_{FI} = 1,00 — v25.30

location	joint at storey	number of floors	design below joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,35*G + 1,50*(P+0,6*V)				280	280							Nd= 1,35*G + 1,50*(P+0,6*V)				280	280					
H / 19		3	GF-roof	1.046	28,0		280	280	HE 240 A	S 355	0,59	(6.62)	S 355												
H / 20		3	GF-roof	965	28,0		280	280	HE 240 A	S 355	0,55	(6.62)	S 355												
H / 21		3	GF-roof	965	28,0		280	280	HE 240 A	S 355	0,55	(6.62)	S 355												
H / 22		3	GF-roof	965	28,0		280	280	HE 240 A	S 355	0,55	(6.62)	S 355												
H / 23		3	GF-roof	1.042	30,3		280	280	HE 240 A	S 355	0,59	(6.62)	S 355												
O / 2		3	GF-roof	1.706	8,8	21,9	280	280	HE 220 B	S 355	0,87	(6.62)	S 355												
O / 5		3	GF-roof	1.706	8,8	21,9	280	280	HE 220 B	S 355	0,87	(6.62)	S 355												
O / 6		3	GF-roof	1.042	6,6		280	280	HE 240 A	S 355	0,51	(6.62)	S 355												
O / 7		3	GF-roof	965	6,1		280	280	HE 240 A	S 355	0,47	(6.62)	S 355												
O / 8		3	GF-roof	965	6,1		280	280	HE 240 A	S 355	0,47	(6.62)	S 355												
O / 9		3	GF-roof	965	6,1		280	280	HE 240 A	S 355	0,47	(6.62)	S 355												
O / 10		3	GF-roof	965	6,1		280	280	HE 240 A	S 355	0,47	(6.62)	S 355												
O / 11		3	GF-roof	1.036	6,1		280	280	HE 240 A	S 355	0,51	(6.62)	S 355												
O / 12		3	GF-roof	1.000	6,1		280	280	HE 240 A	S 355	0,49	(6.62)	S 355												
O / 13		3	GF-roof	1.000	6,1		280	280	HE 240 A	S 355	0,49	(6.62)	S 355												
O / 14		3	GF-roof	1.036	6,1		280	280	HE 240 A	S 355	0,51	(6.62)	S 355												
O / 15		3	GF-roof	965	6,1		280	280	HE 240 A	S 355	0,47	(6.62)	S 355												
O / 16		3	GF-roof	1.036	6,1		280	280	HE 240 A	S 355	0,51	(6.62)	S 355												
O / 17		3	GF-roof	1.000	6,1		280	280	HE 240 A	S 355	0,49	(6.62)	S 355												
O / 18		3	GF-roof	1.000	6,1		280	280	HE 240 A	S 355	0,49	(6.62)	S 355												
O / 19		3	GF-roof	1.036	6,1		280	280	HE 240 A	S 355	0,51	(6.62)	S 355												
O / 20		3	GF-roof	965	6,1		280	280	HE 240 A	S 355	0,47	(6.62)	S 355												
O / 21		3	GF-roof	965	6,1		280	280	HE 240 A	S 355	0,47	(6.62)	S 355												
O / 22		3	GF-roof	965	6,1		280	280	HE 240 A	S 355	0,47	(6.62)	S 355												
O / 23		3	GF-roof	1.042	6,6		280	280	HE 240 A	S 355	0,51	(6.62)	S 355												
O / 24		3	GF-roof	1.706	8,8	21,9	280	280	HE 220 B	S 355	0,87	(6.62)	S 355												
O / 27		3	GF-roof	1.706	8,8	21,9	280	280	HE 220 B	S 355	0,87	(6.62)	S 355												
V / 2	1	1	GF-1.FL	1.072	13,6		300	300	HE 180 B	S 355	0,80	(6.62)	S 355	2	1.FL-roof	390	13,6		280	280	HE 200 A	S 355	0,37	(6.62)	S 355
V / 3		3	GF-roof	353	12,6		300	300	HE 200 A	S 355	0,35	(6.62)	S 355												
V / 4		3	GF-roof	353	12,6		300	300	HE 200 A	S 355	0,35	(6.62)	S 355												
V / 5	1	1	GF-1.FL	1.072	13,6		300	300	HE 180 B	S 355	0,80	(6.62)	S 355	2	1.FL-roof	390	13,6		280	280	HE 200 A	S 355	0,37	(6.62)	S 355
V / 6		3	GF-roof	560	13,6		300	300	HE 200 A	S 355	0,52	(6.62)	S 355												
V / 7		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 8		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 9		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 10		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 11		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												

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project: PR0460 MSCP Neratovice — **Pos CO001**

column design — **full load + wind + imperfection** — γ_{M1} = 1,00 K_{FI} = 1,00 — v25.30

location	joint at storey	number of floors	design below joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,35*G + 1,50*(P+0,6*V)				280	280							Nd= 1,35*G + 1,50*(P+0,6*V)				280	280					
V / 12		3	GF-roof	574	12,6		300	300	HE 200 A	S 355	0,52	(6.62)	S 355												
V / 13		3	GF-roof	548	12,6		300	300	HE 200 A	S 355	0,50	(6.62)	S 355												
V / 14		3	GF-roof	574	12,6		300	300	HE 200 A	S 355	0,52	(6.62)	S 355												
V / 15		3	GF-roof	574	12,6		300	300	HE 200 A	S 355	0,52	(6.62)	S 355												
V / 16		3	GF-roof	548	12,6		300	300	HE 200 A	S 355	0,50	(6.62)	S 355												
V / 17		3	GF-roof	574	12,6		300	300	HE 200 A	S 355	0,52	(6.62)	S 355												
V / 18		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 19		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 20		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 21		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 22		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 23		3	GF-roof	560	13,6		300	300	HE 200 A	S 355	0,52	(6.62)	S 355												
V / 24		3	GF-roof	560	13,6		300	300	HE 200 A	S 355	0,52	(6.62)	S 355												
V / 25		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 26		3	GF-roof	521	12,6		300	300	HE 200 A	S 355	0,48	(6.62)	S 355												
V / 27		3	GF-roof	560	13,6		300	300	HE 200 A	S 355	0,52	(6.62)	S 355												
1 / A		3	GF-roof	153	11,8	11,3	460	460	HE 160 B	S 355	0,61	(6.62)	S 355												
1 / B		3	GF-roof	223	22,3		460	460	HE 160 B	S 355	0,54	(6.62)	S 355												
1 / C		3	GF-roof	185	21,8		460	460	HE 160 B	S 355	0,48	(6.62)	S 355												
1 / D		3	GF-roof	200	20,3		460	460	HE 160 B	S 355	0,49	(6.62)	S 355												
1 / E		3	GF-roof	200	20,3		460	460	HE 160 B	S 355	0,49	(6.62)	S 355												
1 / F		3	GF-roof	185	20,1		460	460	HE 160 B	S 355	0,46	(6.62)	S 355												
1 / F'		3	GF-roof	98	12,2		460	460	HE 160 B	S 355	0,26	(6.62)	S 355												
1 / I'		3	GF-roof	92	2,9		320	320	HE 160 A	S 355	0,16	(6.62)	S 355												
1 / J		3	GF-roof	175	3,4		320	320	HE 160 A	S 355	0,29	(6.62)	S 355												
1 / K		3	GF-roof	190	3,6		320	320	HE 160 A	S 355	0,31	(6.62)	S 355												
1 / L		3	GF-roof	190	3,6		320	320	HE 160 A	S 355	0,31	(6.62)	S 355												
1 / M		3	GF-roof	175	3,4		320	320	HE 160 A	S 355	0,29	(6.62)	S 355												
1 / N		3	GF-roof	214	4,0		320	320	HE 160 A	S 355	0,35	(6.62)	S 355												
1 / O		3	GF-roof	180	3,5		320	320	HE 160 A	S 355	0,29	(6.62)	S 355												
1 / P		3	GF-roof	214	4,0		320	320	HE 160 A	S 355	0,35	(6.62)	S 355												
1 / Q		3	GF-roof	175	3,4		320	320	HE 160 A	S 355	0,29	(6.62)	S 355												
1 / R		3	GF-roof	190	3,6		320	320	HE 160 A	S 355	0,31	(6.62)	S 355												
1 / S		3	GF-roof	190	3,6		320	320	HE 160 A	S 355	0,31	(6.62)	S 355												
1 / T		3	GF-roof	175	4,3		320	320	HE 160 A	S 355	0,30	(6.62)	S 355												
1 / U		3	GF-roof	214	4,9		320	320	HE 160 A	S 355	0,36	(6.62)	S 355												
1 / V		3	GF-roof	143	2,2	1,7	320	320	HE 160 A	S 355	0,26	(6.62)	S 355												
28 / A		3	GF-roof	120	11,8	11,3	460	460	HE 160 B	S 355	0,54	(6.62)	S 355												

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project: PR0460 MSCP Neratovice — **Pos CO001**

column design — **full load + wind + imperfection** — γ_{M1} = 1,00 K_{FI} = 1,00 — v25.30

location	joint at storey	number of floors	design below joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,35*G + 1,50*(P+0,6*V)				280	280							Nd= 1,35*G + 1,50*(P+0,6*V)				280	280					
28 / B		3	GF-roof	158	22,3		460	460	HE 160 B	S 355	0,44	(6.62)	S 355												
28 / C		3	GF-roof	119	21,8		460	460	HE 160 B	S 355	0,37	(6.62)	S 355												
28 / D		3	GF-roof	134	20,3		460	460	HE 160 B	S 355	0,38	(6.62)	S 355												
28 / E		3	GF-roof	134	20,3		460	460	HE 160 B	S 355	0,38	(6.62)	S 355												
28 / F		3	GF-roof	119	20,1		460	460	HE 160 B	S 355	0,36	(6.62)	S 355												
28 / F'		3	GF-roof	66	12,2		460	460	HE 160 B	S 355	0,21	(6.62)	S 355												
28 / I'		3	GF-roof	59	2,9		320	320	HE 160 A	S 355	0,12	(6.62)	S 355												
28 / J		3	GF-roof	109	3,4		320	320	HE 160 A	S 355	0,19	(6.62)	S 355												
28 / K		3	GF-roof	125	3,6		320	320	HE 160 A	S 355	0,22	(6.62)	S 355												
28 / L		3	GF-roof	125	3,6		320	320	HE 160 A	S 355	0,22	(6.62)	S 355												
28 / M		3	GF-roof	110	3,4		320	320	HE 160 A	S 355	0,19	(6.62)	S 355												
28 / N		3	GF-roof	148	4,0		320	320	HE 160 A	S 355	0,25	(6.62)	S 355												
28 / O		3	GF-roof	114	3,5		320	320	HE 160 A	S 355	0,20	(6.62)	S 355												
28 / P		3	GF-roof	148	4,0		320	320	HE 160 A	S 355	0,25	(6.62)	S 355												
28 / Q		3	GF-roof	110	3,4		320	320	HE 160 A	S 355	0,19	(6.62)	S 355												
28 / R		3	GF-roof	125	3,6		320	320	HE 160 A	S 355	0,22	(6.62)	S 355												
28 / S		3	GF-roof	125	3,6		320	320	HE 160 A	S 355	0,22	(6.62)	S 355												
28 / T		3	GF-roof	110	4,3		320	320	HE 160 A	S 355	0,20	(6.62)	S 355												
28 / U		3	GF-roof	148	4,9		320	320	HE 160 A	S 355	0,26	(6.62)	S 355												
28 / V		3	GF-roof	78	2,2	1,7	320	320	HE 160 A	S 355	0,16	(6.62)	S 355												
2' / F		3	GF-roof	887	15,9		420	420	HE 200 B	S 355	0,71	(6.62)	S 355												
2' / F'		3	GF-roof	42	12,7		460	460	HE 160 A	S 355	0,24	(6.62)	S 355												
2' / G		3	GF-roof	290	15,8		405	405	HE 200 A	S 355	0,40	(6.62)	S 355												
2' H		3	GF-roof	290	15,8		365	365	HE 200 A	S 355	0,36	(6.62)	S 355												
2' / I		3	GF-roof	289	15,8		325	325	HE 200 A	S 355	0,34	(6.62)	S 355												
2' / I'		3	GF-roof	40	12,7		320	320	HE 160 A	S 355	0,20	(6.62)	S 355												
2' / J		3	GF-roof	883	10,0		280	280	HE 200 B	S 355	0,49	(6.62)	S 355												
4' / F		3	GF-roof	1.244	15,9		420	280	HE 200 B	S 355	0,69	(6.62)	S 355												
4' / G		3	GF-roof	522	15,1		405	280	HE 200 A	S 355	0,48	(6.62)	S 355												
4' / H		3	GF-roof	602	15,6	6,1	365	280	HE 200 A	S 355	0,61	(6.62)	S 355												
4' / I		3	GF-roof	576	15,1		325	310	HE 200 A	S 355	0,55	(6.62)	S 355												
4' / J		3	GF-roof	1.230	15,9		280	280	HE 200 B	S 355	0,68	(6.62)	S 355												
24' / F		3	GF-roof	1.221	10,0	6,1	280	280	HE 200 B	S 355	0,71	(6.62)	S 355												
24' / G		3	GF-roof	520	15,1		280	280	HE 200 A	S 355	0,48	(6.62)	S 355												
24' H		3	GF-roof	601	9,7	6,1	280	280	HE 200 A	S 355	0,58	(6.62)	S 355												
24' / I		3	GF-roof	576	5,4		280	310	HE 200 A	S 355	0,49	(6.62)	S 355												
24' / J		3	GF-roof	1.230	6,9		280	280	HE 200 B	S 355	0,65	(6.62)	S 355												

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project:	PR0460	MSCP Neratovice														Pos CO001																	
column design			full load + wind + imperfection													γ_{M1} = 1,00		K_{FI} = 1,00															v25.30
location	joint at storey	number of floors	design below joint loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment								
			Nd= 1,35*G + 1,50*(P+0,6*V)												Nd= 1,35*G + 1,50*(P+0,6*V)																	
							280	280											280	280													
26' / F		3	GF-roof	859	10,0	6,1	280	280	HE 200 B	S 355	0,52	(6.62)	S 355																			
26' / F'		3	GF-roof	42	6,6		460	460	HE 160 A	S 355	0,17	(6.62)	S 355																			
26' / G		3	GF-roof	204	6,8		280	280	HE 200 A	S 355	0,19	(6.62)	S 355																			
26' H		3	GF-roof	288	6,8		280	280	HE 200 A	S 355	0,26	(6.62)	S 355																			
26' / I		3	GF-roof	288	6,8		280	280	HE 200 A	S 355	0,26	(6.62)	S 355																			
26' / I'		3	GF-roof	40	1,0		320	320	HE 160 A	S 355	0,07	(6.62)	S 355																			
26' / J		3	GF-roof	883	6,8		280	280	HE 200 B	S 355	0,48	(6.62)	S 355																			
F / 25		3	GF-roof	35	13,5		280	280	HE 160 A	S 355	0,20	(6.62)	S 355																			
F / 26		3	GF-roof	35	13,5		280	280	HE 160 A	S 355	0,20	(6.62)	S 355																			

Load case: vehicle impact

- Dead load
- Loads from vehicle impact:

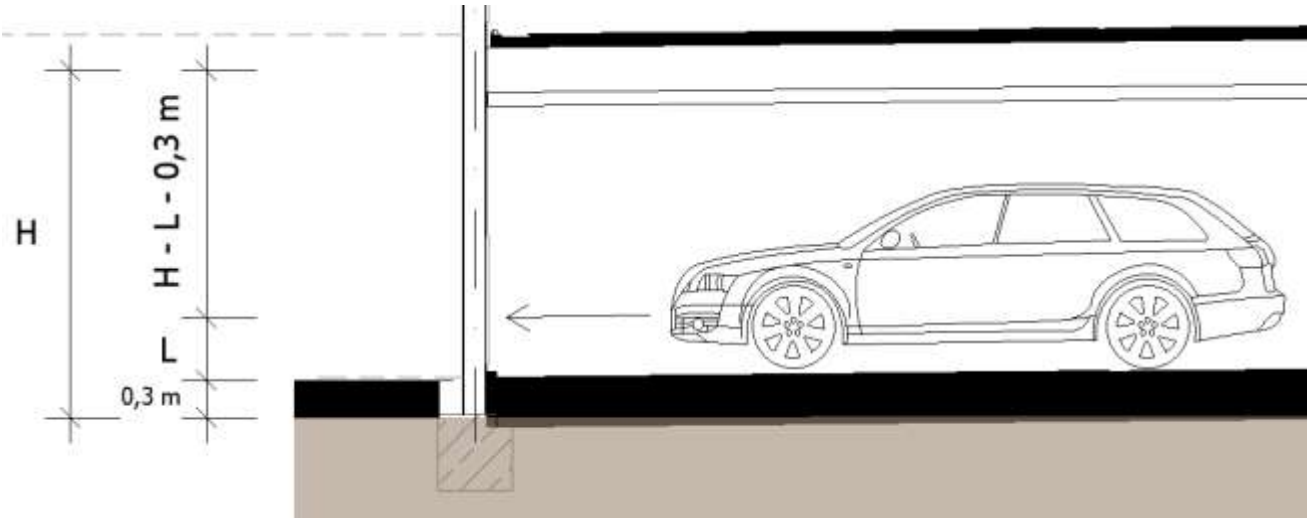
In the direction of travel: horizontal load of 50 kN at L = 0.50 m above the road, H = 2,80 m, H – L – 0,3 m = 2.00 m:
$M_{y/z} = 50 \cdot 0.8 \cdot 2.00/2.80 = 28.6$ kNm
$M_{y/z} = 50 \cdot 2.125 \cdot 2.00/4.125 = 51,52$ kNm

Across the direction of travel: horizontal load of 25 kN at L = 0.50 m above the road, H = 2,80 m, H – L – 0,3 m = 2.00 m:
$M_{y/z} = 25 \cdot 0,8 \cdot 2.00/2.80 = 14.3$ kNm

- Traffic and installation loads are reduced to 60%
- no wind loads
- loads of bracing (only inclination + earth pressure)

→ Partial safety factors $\gamma_F = 1.0$

The verifications for the steel columns are satisfied for all load cases. For the anchoring (i.e. base plate, stud bolt plate and anchor plate) and the foundation loads, the standard design load case is decisive.



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project: PR0460 MSCP Neratovice																							Pos CO001
load composition per column [kN or kNm] γ_F = 1,0																							v25.30
n = number of floors	unusual load case (impact)																				moments from impact & eccentricity		wind suction on roof
location	n	loads	roof	1,0									ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing			$M_{y,k}$	$M_{z,k}$	W_{sd}
	3	from-to	G	P									G	P	G	P	G	P	G	± V	± H_x	± H_y	kNm	kNm	kN
A / 2	3	GF-roof	45	38									45	38	45	38		77	7			24	68,4		
A / 3	3	GF-roof	42	35									42	35	42	35		77	7			22	67,3		
A / 4	3	GF-roof	42	35									42	35	42	35		77	7			22	67,3		
A / 5	3	GF-roof	45	38									45	38	45	38		77	7			24	68,4		
A / 6	3	GF-roof	65	55									65	55	65	55		77	7			24	69,7		
A / 7	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 8	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 9	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 10	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 11	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 12	3	GF-roof	61	51									61	51	61	51		77	7	80	29	22	68,5		
A / 13	3	GF-roof	61	51									61	51	61	51		77	7	40	29	22	68,5		
A / 14	3	GF-roof	61	51									61	51	61	51		77	7	80	29	22	68,5		
A / 15	3	GF-roof	61	51									61	51	61	51		77	7	80	29	22	68,5		
A / 16	3	GF-roof	61	51									61	51	61	51		77	7	40	29	22	68,5		
A / 17	3	GF-roof	61	51									61	51	61	51		77	7	80	29	22	68,5		
A / 18	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 19	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 20	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 21	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 22	3	GF-roof	61	51									61	51	61	51		77	7			22	68,5		
A / 23	3	GF-roof	65	55									65	55	65	55		77	7			24	69,7		
A / 24	3	GF-roof	45	38									45	38	45	38		77	7			24	68,4		
A / 25	3	GF-roof	42	35									42	35	42	35		77	7			22	67,3		
A / 26	3	GF-roof	42	35									42	35	42	35		77	7			22	67,3		
A / 27	3	GF-roof	45	38									45	38	45	38		77	7			24	68,4		
H / 6	3	GF-roof	130	110									130	110	130	110		9	5			22	54,9		
H / 7	3	GF-roof	121	101									121	101	121	101		9	5			20	53,4		
H / 8	3	GF-roof	121	101									121	101	121	101		9	5			20	53,4		
H / 9	3	GF-roof	121	101									121	101	121	101		9	5			20	53,4		
H / 10	3	GF-roof	121	101									121	101	121	101		9	5			20	53,4		
H / 11	3	GF-roof	121	101									121	101	121	101		9	5	90	35	20	53,4		
H / 12	3	GF-roof	121	101									121	101	121	101		9	5	45	35	20	53,4		
H / 13	3	GF-roof	121	101									121	101	121	101		9	5	45	35	20	53,4		
H / 14	3	GF-roof	121	101									121	101	121	101		9	5	90	35	20	53,4		
H / 15	3	GF-roof	121	101									121	101	121	101		9	5			20	53,4		
H / 16	3	GF-roof	121	101									121	101	121	101		9	5	90	35	20	53,4		
H / 17	3	GF-roof	121	101									121	101	121	101		9	5	45	35	20	53,4		
H / 18	3	GF-roof	121	101									121	101	121	101		9	5	45	35	20	53,4		

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project: PR0460 MSCP Neratovice — **Pos CO001**

load composition per column [kN or kNm] γ_F = 1,0 — v25.30

location	n	loads	unusual load case (impact)																				moments from impact & eccentricity		wind suction on roof
			roof	1,0									ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing					
	3	from-to	G	P									G	P	G	P	G	P	G	± V	± H_x	± H_y	$M_{y,k}$	$M_{z,k}$	W_{sd}
																							kNm	kNm	kN
H / 19	3	GF-roof	121	101									121	101	121	101		9	5	90	35	20	53,4		
H / 20	3	GF-roof	121	101									121	101	121	101		9	5			20	53,4		
H / 21	3	GF-roof	121	101									121	101	121	101		9	5			20	53,4		
H / 22	3	GF-roof	121	101									121	101	121	101		9	5			20	53,4		
H / 23	3	GF-roof	130	110									130	110	130	110		9	5			22	54,9		
O / 2	3	GF-roof	218	179									218	179	218	179		9	6				6,0	43,1	
O / 5	3	GF-roof	218	179									218	179	218	179		9	6				6,0	43,1	
O / 6	3	GF-roof	130	110									130	110	130	110		9	5				37,2		
O / 7	3	GF-roof	121	101									121	101	121	101		9	5				37,0		
O / 8	3	GF-roof	121	101									121	101	121	101		9	5				37,0		
O / 9	3	GF-roof	121	101									121	101	121	101		9	5				37,0		
O / 10	3	GF-roof	121	101									121	101	121	101		9	5				37,0		
O / 11	3	GF-roof	121	101									121	101	121	101		9	5	79	33		37,0		
O / 12	3	GF-roof	121	101									121	101	121	101		9	5	40	33		37,0		
O / 13	3	GF-roof	121	101									121	101	121	101		9	5	40	33		37,0		
O / 14	3	GF-roof	121	101									121	101	121	101		9	5	79	33		37,0		
O / 15	3	GF-roof	121	101									121	101	121	101		9	5				37,0		
O / 16	3	GF-roof	121	101									121	101	121	101		9	5	79	33		37,0		
O / 17	3	GF-roof	121	101									121	101	121	101		9	5	40	33		37,0		
O / 18	3	GF-roof	121	101									121	101	121	101		9	5	40	33		37,0		
O / 19	3	GF-roof	121	101									121	101	121	101		9	5	79	33		37,0		
O / 20	3	GF-roof	121	101									121	101	121	101		9	5				37,0		
O / 21	3	GF-roof	121	101									121	101	121	101		9	5				37,0		
O / 22	3	GF-roof	121	101									121	101	121	101		9	5				37,0		
O / 23	3	GF-roof	130	110									130	110	130	110		9	5				37,2		
O / 24	3	GF-roof	218	179									218	179	218	179		9	6				6,0	43,1	
O / 27	3	GF-roof	218	179									218	179	218	179		9	6				6,0	43,1	
V / 2	3	GF-roof	65	55									65	55	256	225		29	4				9,6	37,5	
V / 3	3	GF-roof	61	51									61	51				22	4				42,1		
V / 4	3	GF-roof	61	51									61	51				22	4				42,1		
V / 5	3	GF-roof	65	55									65	55	256	225		29	4				9,6	37,5	
V / 6	3	GF-roof	65	55									65	55	65	55		29	4				42,5		
V / 7	3	GF-roof	61	51									61	51	61	51		29	4				42,1		
V / 8	3	GF-roof	61	51									61	51	61	51		29	4				42,1		
V / 9	3	GF-roof	61	51									61	51	61	51		29	4				42,1		
V / 10	3	GF-roof	61	51									61	51	61	51		29	4				42,1		
V / 11	3	GF-roof	61	51									61	51	61	51		29	4				42,1		

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project: PR0460 MSCP Neratovice — **Pos CO001**

load composition per column [kN or kNm] γ_F = 1,0 — v25.30

n = number of floors

unusual load case (impact)

location	n	loads	roof	1,0							ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing			moments from impact & eccentricity		wind suction on roof
	3	from-to	G	P							G	P	G	P	G	P	G	± V	± H_x	± H_y	$M_{y,k}$	$M_{z,k}$	W_{sd}
																					kNm	kNm	kN
V / 12	3	GF-roof	61	51							61	51	61	51		29	4	59	25		42,1		
V / 13	3	GF-roof	61	51							61	51	61	51		29	4	30	25		42,1		
V / 14	3	GF-roof	61	51							61	51	61	51		29	4	59	25		42,1		
V / 15	3	GF-roof	61	51							61	51	61	51		29	4	59	25		42,1		
V / 16	3	GF-roof	61	51							61	51	61	51		29	4	30	25		42,1		
V / 17	3	GF-roof	61	51							61	51	61	51		29	4	59	25		42,1		
V / 18	3	GF-roof	61	51							61	51	61	51		29	4				42,1		
V / 19	3	GF-roof	61	51							61	51	61	51		29	4				42,1		
V / 20	3	GF-roof	61	51							61	51	61	51		29	4				42,1		
V / 21	3	GF-roof	61	51							61	51	61	51		29	4				42,1		
V / 22	3	GF-roof	61	51							61	51	61	51		29	4				42,1		
V / 23	3	GF-roof	65	55							65	55	65	55		29	4				42,5		
V / 24	3	GF-roof	65	55							65	55	65	55		29	4				42,5		
V / 25	3	GF-roof	61	51							61	51	61	51		29	4				42,1		
V / 26	3	GF-roof	61	51							61	51	61	51		29	4				42,1		
V / 27	3	GF-roof	65	55							65	55	65	55		29	4				42,5		
1 / A	3	GF-roof	4	5							4	5	4	5		72	6		12	12	63,8	5,9	
1 / B	3	GF-roof	13	13							13	13	13	13		72	6		22		69,5		
1 / C	3	GF-roof	8	9							8	9	8	9		72	6		22		69,1		
1 / D	3	GF-roof	10	10							10	10	10	10		72	6		22		69,2		
1 / E	3	GF-roof	10	10							10	10	10	10		72	6		22		69,2		
1 / F	3	GF-roof	8	8							8	8	8	8		72	6		22		69,1		
1 / F'	3	GF-roof	5	4							5	4	5	4		36	6		11		63,3		
1 / I'	3	GF-roof	5	4							5	4	5	4		34	3				40,3		
1 / J	3	GF-roof	8	8							8	8	8	8		69	3				40,6		
1 / K	3	GF-roof	10	10							10	10	10	10		69	3				40,8		
1 / L	3	GF-roof	10	10							10	10	10	10		69	3				40,8		
1 / M	3	GF-roof	8	9							8	9	8	9		69	3				40,6		
1 / N	3	GF-roof	13	13							13	13	13	13		69	3				41,0		
1 / O	3	GF-roof	8	9							8	9	8	9		69	3				40,7		
1 / P	3	GF-roof	13	13							13	13	13	13		69	3				41,0		
1 / Q	3	GF-roof	8	9							8	9	8	9		69	3				40,6		
1 / R	3	GF-roof	10	10							10	10	10	10		69	3				40,8		
1 / S	3	GF-roof	10	10							10	10	10	10		69	3				40,8		
1 / T	3	GF-roof	8	9							8	9	8	9		69	3				40,6		
1 / U	3	GF-roof	13	13							13	13	13	13		69	3				41,0		
1 / V	3	GF-roof	4	5							4	5	4	5		69	3				40,3		
28 / A	3	GF-roof	4	5							4	5	4	5		50	6		12	12	63,8	5,9	

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project: PR0460 MSCP Neratovice — **Pos CO001**

load composition per column [kN or kNm] γ_F = 1,0 — v25.30

n = number of floors: 3

unusual load case (impact)

location	n	loads	roof	1,0							ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing			moments from impact & eccentricity		wind suction on roof
	3	from-to	G	P							G	P	G	P	G	P	G	± V	± H_x	± H_y	$M_{y,k}$	$M_{z,k}$	W_{sd}
																					kNm	kNm	kN
28 / B	3	GF-roof	13	13							13	13	13	13		29	6		22		69,5		
28 / C	3	GF-roof	8	9							8	9	8	9		29	6		22		69,1		
28 / D	3	GF-roof	10	10							10	10	10	10		29	6		22		69,2		
28 / E	3	GF-roof	10	10							10	10	10	10		29	6		22		69,2		
28 / F	3	GF-roof	8	8							8	8	8	8		29	6		22		69,1		
28 / F'	3	GF-roof	5	4							5	4	5	4		14	6		11		63,3		
28 / I'	3	GF-roof	5	4							5	4	5	4		12	3				40,3		
28 / J	3	GF-roof	8	8							8	8	8	8		25	3				40,6		
28 / K	3	GF-roof	10	10							10	10	10	10		25	3				40,8		
28 / L	3	GF-roof	10	10							10	10	10	10		25	3				40,8		
28 / M	3	GF-roof	8	9							8	9	8	9		25	3				40,6		
28 / N	3	GF-roof	13	13							13	13	13	13		25	3				41,0		
28 / O	3	GF-roof	8	9							8	9	8	9		25	3				40,7		
28 / P	3	GF-roof	13	13							13	13	13	13		25	3				41,0		
28 / Q	3	GF-roof	8	9							8	9	8	9		25	3				40,6		
28 / R	3	GF-roof	10	10							10	10	10	10		25	3				40,8		
28 / S	3	GF-roof	10	10							10	10	10	10		25	3				40,8		
28 / T	3	GF-roof	8	9							8	9	8	9		25	3				40,6		
28 / U	3	GF-roof	13	13							13	13	13	13		25	3				41,0		
28 / V	3	GF-roof	4	5							4	5	4	5		25	3				40,3		
2' / F	3	GF-roof	115	105							110	90	110	90			8		20		11,5	52,5	-11
2' / F'	3	GF-roof	5	4							5	4	5	4			4		20		66,9		
2' / G	3	GF-roof	42	39							37	23	37	23			5		20		62,1		-11
2' H	3	GF-roof	42	39							37	23	37	23			5		20		57,1		-11
2' / I	3	GF-roof	42	39							37	23	37	23			4		20		52,1		-11
2' / I'	3	GF-roof	5	4							5	4	5	4			3		20		49,4		
2' / J	3	GF-roof	115	105							110	90	110	90			5		20		7,1	35,0	-11
4' / F	3	GF-roof	114	106							109	90	109	90			8	324	20	137	11,5	35,0	-11
4' / G	3	GF-roof	55	51							50	36	50	36			5	120	20	99	61,6		-11
4' / H	3	GF-roof	50	48							45	32	45	32		5	5	190	10	140	56,9	4,5	-11
4' / I	3	GF-roof	55	51							50	36	50	36			4	157	20	124	51,6		-11
4' / J	3	GF-roof	114	106							109	90	109	90			5	317	20	143	11,5	35,0	-11
24' / F	3	GF-roof	101	93							109	90	114	94			5	324	10	147	7,2	39,5	-11
24' / G	3	GF-roof	55	51							50	36	50	36			4	120	20	99	46,0		-11
24' H	3	GF-roof	50	48							45	32	45	32		5	4	190	10	140	41,8	4,5	-11
24' / I	3	GF-roof	55	51							50	36	50	36			4	157		124	36,9		-11
24' / J	3	GF-roof	114	106							109	90	109	90			5	317		143	4,7	35,0	-11

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project: PR0460 MSCP Neratovice																						Pos CO001		
load composition per column [kN or kNm] γ_F = 1,0																						v25.30		
n = number of floors			unusual load case (impact)																			moments from impact & eccentricity		wind suction on roof
location	n	loads	roof	1,0									ceiling above 1.FL	1,0	ceiling above GF	1,0	facade, canopy etc.		column	bracing			$M_{y,k}$	$M_{z,k}$	W_{sd}
	3	from-to	G	P									G	P	G	P	G	P	G	± V	± H_x	± H_y	kNm	kNm	kN
26' / F	3	GF-roof	102	93									110	90	114	94			5		10	10	7,1	39,5	-11
26' / F'	3	GF-roof	5	4									5	4	5	4			4		10		62,4		
26' / G	3	GF-roof	5	16									37	23	37	23			4				37,4		-11
26' H	3	GF-roof	42	39									37	23	37	23			4				37,4		-11
26' / I	3	GF-roof	42	39									37	23	37	23			4				37,4		-11
26' / I'	3	GF-roof	5	4									5	4	5	4			3				40,3		
26' / J	3	GF-roof	115	105									110	90	110	90			5				4,6	35,0	-11
F / 25	3	GF-roof													13	10			3			20	9,9		
F / 26	3	GF-roof													13	10			3			20	9,9		

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project: PR0460 MSCP Neratovice — **Pos CO001**

column design — **unusual load case (impact)** — γ_{M1} = 1,00 K_{FI} = 1,00 — v25.30

location	joint at storey	number of floors	design below joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,0*G + 1,0*(V+0,6*Q)				280	280							Nd= 1,0*G + 1,0*(V+0,6*Q)				280	280					
A / 2		3	GF-roof	256	68,4		430	430	HE 180 B	S 355	0,66	(6.62)	S 355												
A / 3		3	GF-roof	242	67,3		430	430	HE 180 B	S 355	0,64	(6.62)	S 355												
A / 4		3	GF-roof	242	67,3		430	430	HE 180 B	S 355	0,64	(6.62)	S 355												
A / 5		3	GF-roof	256	68,4		430	430	HE 180 B	S 355	0,66	(6.62)	S 355												
A / 6		3	GF-roof	347	69,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 7		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 8		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 9		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 10		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 11		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 12		3	GF-roof	405	68,5		430	430	HE 180 B	S 355	0,80	(6.62)	S 355												
A / 13		3	GF-roof	365	68,5		430	430	HE 180 B	S 355	0,76	(6.62)	S 355												
A / 14		3	GF-roof	405	68,5		430	430	HE 180 B	S 355	0,80	(6.62)	S 355												
A / 15		3	GF-roof	405	68,5		430	430	HE 180 B	S 355	0,80	(6.62)	S 355												
A / 16		3	GF-roof	365	68,5		430	430	HE 180 B	S 355	0,76	(6.62)	S 355												
A / 17		3	GF-roof	405	68,5		430	430	HE 180 B	S 355	0,80	(6.62)	S 355												
A / 18		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 19		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 20		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 21		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 22		3	GF-roof	325	68,5		430	430	HE 180 B	S 355	0,73	(6.62)	S 355												
A / 23		3	GF-roof	347	69,7		430	430	HE 180 B	S 355	0,75	(6.62)	S 355												
A / 24		3	GF-roof	256	68,4		430	430	HE 180 B	S 355	0,66	(6.62)	S 355												
A / 25		3	GF-roof	242	67,3		430	430	HE 180 B	S 355	0,64	(6.62)	S 355												
A / 26		3	GF-roof	242	67,3		430	430	HE 180 B	S 355	0,64	(6.62)	S 355												
A / 27		3	GF-roof	256	68,4		430	430	HE 180 B	S 355	0,66	(6.62)	S 355												
H / 6		3	GF-roof	598	54,9		280	280	HE 240 A	S 355	0,48	(6.62)	S 355												
H / 7		3	GF-roof	555	53,4		280	280	HE 240 A	S 355	0,46	(6.62)	S 355												
H / 8		3	GF-roof	555	53,4		280	280	HE 240 A	S 355	0,46	(6.62)	S 355												
H / 9		3	GF-roof	555	53,4		280	280	HE 240 A	S 355	0,46	(6.62)	S 355												
H / 10		3	GF-roof	555	53,4		280	280	HE 240 A	S 355	0,46	(6.62)	S 355												
H / 11		3	GF-roof	645	53,4		280	280	HE 240 A	S 355	0,50	(6.62)	S 355												
H / 12		3	GF-roof	600	53,4		280	280	HE 240 A	S 355	0,48	(6.62)	S 355												
H / 13		3	GF-roof	600	53,4		280	280	HE 240 A	S 355	0,48	(6.62)	S 355												
H / 14		3	GF-roof	645	53,4		280	280	HE 240 A	S 355	0,50	(6.62)	S 355												
H / 15		3	GF-roof	555	53,4		280	280	HE 240 A	S 355	0,46	(6.62)	S 355												
H / 16		3	GF-roof	645	53,4		280	280	HE 240 A	S 355	0,50	(6.62)	S 355												
H / 17		3	GF-roof	600	53,4		280	280	HE 240 A	S 355	0,48	(6.62)	S 355												
H / 18		3	GF-roof	600	53,4		280	280	HE 240 A	S 355	0,48	(6.62)	S 355												

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project: PR0460 MSCP Neratovice — **Pos CO001**

column design — **unusual load case (impact)** — γ_{M1} = 1,00 K_{FI} = 1,00 — v25.30

location	joint at storey	number of floors	design below joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,0*G + 1,0*(V+0,6*Q)				280	280							Nd= 1,0*G + 1,0*(V+0,6*Q)				280	280					
H / 19		3	GF-roof	645	53,4		280	280	HE 240 A	S 355	0,50	(6.62)	S 355												
H / 20		3	GF-roof	555	53,4		280	280	HE 240 A	S 355	0,46	(6.62)	S 355												
H / 21		3	GF-roof	555	53,4		280	280	HE 240 A	S 355	0,46	(6.62)	S 355												
H / 22		3	GF-roof	555	53,4		280	280	HE 240 A	S 355	0,46	(6.62)	S 355												
H / 23		3	GF-roof	598	54,9		280	280	HE 240 A	S 355	0,48	(6.62)	S 355												
O / 2		3	GF-roof	985	6,0	43,1	280	280	HE 220 B	S 355	0,67	(6.62)	S 355												
O / 5		3	GF-roof	985	6,0	43,1	280	280	HE 220 B	S 355	0,67	(6.62)	S 355												
O / 6		3	GF-roof	598	37,2		280	280	HE 240 A	S 355	0,41	(6.62)	S 355												
O / 7		3	GF-roof	555	37,0		280	280	HE 240 A	S 355	0,39	(6.62)	S 355												
O / 8		3	GF-roof	555	37,0		280	280	HE 240 A	S 355	0,39	(6.62)	S 355												
O / 9		3	GF-roof	555	37,0		280	280	HE 240 A	S 355	0,39	(6.62)	S 355												
O / 10		3	GF-roof	555	37,0		280	280	HE 240 A	S 355	0,39	(6.62)	S 355												
O / 11		3	GF-roof	634	37,0		280	280	HE 240 A	S 355	0,43	(6.62)	S 355												
O / 12		3	GF-roof	595	37,0		280	280	HE 240 A	S 355	0,41	(6.62)	S 355												
O / 13		3	GF-roof	595	37,0		280	280	HE 240 A	S 355	0,41	(6.62)	S 355												
O / 14		3	GF-roof	634	37,0		280	280	HE 240 A	S 355	0,43	(6.62)	S 355												
O / 15		3	GF-roof	555	37,0		280	280	HE 240 A	S 355	0,39	(6.62)	S 355												
O / 16		3	GF-roof	634	37,0		280	280	HE 240 A	S 355	0,43	(6.62)	S 355												
O / 17		3	GF-roof	595	37,0		280	280	HE 240 A	S 355	0,41	(6.62)	S 355												
O / 18		3	GF-roof	595	37,0		280	280	HE 240 A	S 355	0,41	(6.62)	S 355												
O / 19		3	GF-roof	634	37,0		280	280	HE 240 A	S 355	0,43	(6.62)	S 355												
O / 20		3	GF-roof	555	37,0		280	280	HE 240 A	S 355	0,39	(6.62)	S 355												
O / 21		3	GF-roof	555	37,0		280	280	HE 240 A	S 355	0,39	(6.62)	S 355												
O / 22		3	GF-roof	555	37,0		280	280	HE 240 A	S 355	0,39	(6.62)	S 355												
O / 23		3	GF-roof	598	37,2		280	280	HE 240 A	S 355	0,41	(6.62)	S 355												
O / 24		3	GF-roof	985	6,0	43,1	280	280	HE 220 B	S 355	0,67	(6.62)	S 355												
O / 27		3	GF-roof	985	6,0	43,1	280	280	HE 220 B	S 355	0,67	(6.62)	S 355												
V / 2	1	1	GF-1.FL	608	9,6	37,5	300	300	HE 180 B	S 355	0,89	(6.62)	S 355	2	1.FL-roof	273	9,6	37,5	280	280	HE 200 A	S 355	0,64	(6.62)	S 355
V / 3		3	GF-roof	199	42,1		300	300	HE 200 A	S 355	0,42	(6.62)	S 355												
V / 4		3	GF-roof	199	42,1		300	300	HE 200 A	S 355	0,42	(6.62)	S 355												
V / 5	1	1	GF-1.FL	608	9,6	37,5	300	300	HE 180 B	S 355	0,89	(6.62)	S 355	2	1.FL-roof	273	9,6	37,5	280	280	HE 200 A	S 355	0,64	(6.62)	S 355
V / 6		3	GF-roof	315	42,5		300	300	HE 200 A	S 355	0,51	(6.62)	S 355												
V / 7		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 8		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 9		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 10		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 11		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												

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project: PR0460 MSCP Neratovice — **Pos CO001**

column design — **unusual load case (impact)** — γ_{M1} = 1,00 K_{FI} = 1,00 — v25.30

location	joint at storey	number of floors	design below joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,0*G + 1,0*(V+0,6*Q)				280	280							Nd= 1,0*G + 1,0*(V+0,6*Q)				280	280					
V / 12		3	GF-roof	353	42,1		300	300	HE 200 A	S 355	0,53	(6.62)	S 355												
V / 13		3	GF-roof	323	42,1		300	300	HE 200 A	S 355	0,51	(6.62)	S 355												
V / 14		3	GF-roof	353	42,1		300	300	HE 200 A	S 355	0,53	(6.62)	S 355												
V / 15		3	GF-roof	353	42,1		300	300	HE 200 A	S 355	0,53	(6.62)	S 355												
V / 16		3	GF-roof	323	42,1		300	300	HE 200 A	S 355	0,51	(6.62)	S 355												
V / 17		3	GF-roof	353	42,1		300	300	HE 200 A	S 355	0,53	(6.62)	S 355												
V / 18		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 19		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 20		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 21		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 22		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 23		3	GF-roof	315	42,5		300	300	HE 200 A	S 355	0,51	(6.62)	S 355												
V / 24		3	GF-roof	315	42,5		300	300	HE 200 A	S 355	0,51	(6.62)	S 355												
V / 25		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 26		3	GF-roof	294	42,1		300	300	HE 200 A	S 355	0,49	(6.62)	S 355												
V / 27		3	GF-roof	315	42,5		300	300	HE 200 A	S 355	0,51	(6.62)	S 355												
1 / A		3	GF-roof	69	63,8	5,9	460	460	HE 160 B	S 355	0,76	(6.62)	S 355												
1 / B		3	GF-roof	109	69,5		460	460	HE 160 B	S 355	0,75	(6.62)	S 355												
1 / C		3	GF-roof	87	69,1		460	460	HE 160 B	S 355	0,71	(6.62)	S 355												
1 / D		3	GF-roof	96	69,2		460	460	HE 160 B	S 355	0,73	(6.62)	S 355												
1 / E		3	GF-roof	96	69,2		460	460	HE 160 B	S 355	0,73	(6.62)	S 355												
1 / F		3	GF-roof	88	69,1		460	460	HE 160 B	S 355	0,71	(6.62)	S 355												
1 / F'		3	GF-roof	48	63,3		460	460	HE 160 B	S 355	0,61	(6.62)	S 355												
1 / I'		3	GF-roof	44	40,3		320	320	HE 160 A	S 355	0,52	(6.62)	S 355												
1 / J		3	GF-roof	82	40,6		320	320	HE 160 A	S 355	0,57	(6.62)	S 355												
1 / K		3	GF-roof	91	40,8		320	320	HE 160 A	S 355	0,58	(6.62)	S 355												
1 / L		3	GF-roof	91	40,8		320	320	HE 160 A	S 355	0,58	(6.62)	S 355												
1 / M		3	GF-roof	82	40,6		320	320	HE 160 A	S 355	0,57	(6.62)	S 355												
1 / N		3	GF-roof	104	41,0		320	320	HE 160 A	S 355	0,60	(6.62)	S 355												
1 / O		3	GF-roof	84	40,7		320	320	HE 160 A	S 355	0,57	(6.62)	S 355												
1 / P		3	GF-roof	104	41,0		320	320	HE 160 A	S 355	0,60	(6.62)	S 355												
1 / Q		3	GF-roof	82	40,6		320	320	HE 160 A	S 355	0,57	(6.62)	S 355												
1 / R		3	GF-roof	91	40,8		320	320	HE 160 A	S 355	0,58	(6.62)	S 355												
1 / S		3	GF-roof	91	40,8		320	320	HE 160 A	S 355	0,58	(6.62)	S 355												
1 / T		3	GF-roof	82	40,6		320	320	HE 160 A	S 355	0,57	(6.62)	S 355												
1 / U		3	GF-roof	104	41,0		320	320	HE 160 A	S 355	0,60	(6.62)	S 355												
1 / V		3	GF-roof	64	40,3		320	320	HE 160 A	S 355	0,55	(6.62)	S 355												
28 / A		3	GF-roof	56	63,8	5,9	460	460	HE 160 B	S 355	0,73	(6.62)	S 355												

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project: PR0460 MSCP Neratovice — **Pos CO001**

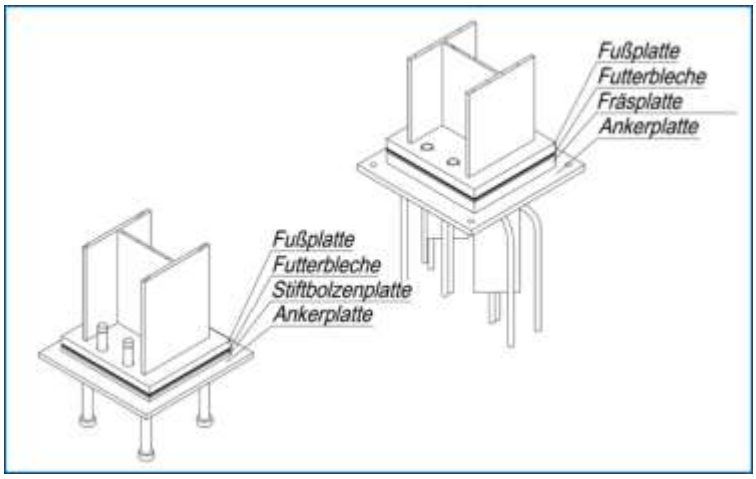
column design — **unusual load case (impact)** — γ_{M1} = 1,00 K_{FI} = 1,00 — v25.30

location	joint at storey	number of floors	design below joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint: loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,0*G + 1,0*(V+0,6*Q)				280	280							Nd= 1,0*G + 1,0*(V+0,6*Q)				280	280					
28 / B		3	GF-roof	83	69,5		460	460	HE 160 B	S 355	0,71	(6.62)	S 355												
28 / C		3	GF-roof	61	69,1		460	460	HE 160 B	S 355	0,67	(6.62)	S 355												
28 / D		3	GF-roof	69	69,2		460	460	HE 160 B	S 355	0,69	(6.62)	S 355												
28 / E		3	GF-roof	69	69,2		460	460	HE 160 B	S 355	0,69	(6.62)	S 355												
28 / F		3	GF-roof	61	69,1		460	460	HE 160 B	S 355	0,67	(6.62)	S 355												
28 / F'		3	GF-roof	35	63,3		460	460	HE 160 B	S 355	0,59	(6.62)	S 355												
28 / I'		3	GF-roof	31	40,3		320	320	HE 160 A	S 355	0,51	(6.62)	S 355												
28 / J		3	GF-roof	56	40,6		320	320	HE 160 A	S 355	0,54	(6.62)	S 355												
28 / K		3	GF-roof	64	40,8		320	320	HE 160 A	S 355	0,55	(6.62)	S 355												
28 / L		3	GF-roof	64	40,8		320	320	HE 160 A	S 355	0,55	(6.62)	S 355												
28 / M		3	GF-roof	56	40,6		320	320	HE 160 A	S 355	0,54	(6.62)	S 355												
28 / N		3	GF-roof	78	41,0		320	320	HE 160 A	S 355	0,57	(6.62)	S 355												
28 / O		3	GF-roof	58	40,7		320	320	HE 160 A	S 355	0,54	(6.62)	S 355												
28 / P		3	GF-roof	78	41,0		320	320	HE 160 A	S 355	0,57	(6.62)	S 355												
28 / Q		3	GF-roof	56	40,6		320	320	HE 160 A	S 355	0,54	(6.62)	S 355												
28 / R		3	GF-roof	64	40,8		320	320	HE 160 A	S 355	0,55	(6.62)	S 355												
28 / S		3	GF-roof	64	40,8		320	320	HE 160 A	S 355	0,55	(6.62)	S 355												
28 / T		3	GF-roof	56	40,6		320	320	HE 160 A	S 355	0,54	(6.62)	S 355												
28 / U		3	GF-roof	78	41,0		320	320	HE 160 A	S 355	0,57	(6.62)	S 355												
28 / V		3	GF-roof	38	40,3		320	320	HE 160 A	S 355	0,51	(6.62)	S 355												
2' / F		3	GF-roof	512	11,5	52,5	420	420	HE 200 B	S 355	0,88	(6.62)	S 355												
2' / F'		3	GF-roof	25	66,9		460	460	HE 160 A	S 355	0,90	(6.62)	S 355												
2' / G		3	GF-roof	172	62,1		405	405	HE 200 A	S 355	0,58	(6.62)	S 355												
2' H		3	GF-roof	171	57,1		365	365	HE 200 A	S 355	0,52	(6.62)	S 355												
2' / I		3	GF-roof	171	52,1		325	325	HE 200 A	S 355	0,48	(6.62)	S 355												
2' / I'		3	GF-roof	24	49,4		320	320	HE 160 A	S 355	0,60	(6.62)	S 355												
2' / J		3	GF-roof	509	7,1	35,0	280	280	HE 200 B	S 355	0,53	(6.62)	S 355												
4' / F		3	GF-roof	835	11,5	35,0	420	280	HE 200 B	S 355	0,74	(6.62)	S 355												
4' / G		3	GF-roof	352	61,6		405	280	HE 200 A	S 355	0,67	(6.61)	S 355												
4' / H		3	GF-roof	404	56,9	4,5	365	280	HE 200 A	S 355	0,74	(6.62)	S 355												
4' / I		3	GF-roof	388	51,6		325	310	HE 200 A	S 355	0,64	(6.62)	S 355												
4' / J		3	GF-roof	825	11,5	35,0	280	280	HE 200 B	S 355	0,73	(6.62)	S 355												
24' / F		3	GF-roof	819	7,2	39,5	280	280	HE 200 B	S 355	0,75	(6.62)	S 355												
24' / G		3	GF-roof	350	46,0		280	280	HE 200 A	S 355	0,55	(6.62)	S 355												
24' H		3	GF-roof	403	41,8	4,5	280	280	HE 200 A	S 355	0,65	(6.62)	S 355												
24' / I		3	GF-roof	387	36,9		280	310	HE 200 A	S 355	0,53	(6.62)	S 355												
24' / J		3	GF-roof	825	4,7	35,0	280	280	HE 200 B	S 355	0,71	(6.62)	S 355												

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project:	PR0460	MSCP Neratovice																							Pos CO001
column design		unusual load case (impact)																							v25.30
location	joint at storey	number of floors	design below joint loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment	number of floors	design above joint loads from - to	N_d [kN]	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	l_{ky} [cm]	l_{kz} [cm]	column profile EXC 2	steel grade	utility <1,0	governing verification	comment
			Nd= 1,0*G + 1,0*(V+0,6*Q)												Nd= 1,0*G + 1,0*(V+0,6*Q)										
							280	280											280	280					
26' / F		3	GF-roof	495	7,1	39,5	280	280	HE 200 B	S 355	0,55	(6.62)	S 355												
26' / F'		3	GF-roof	25	62,4		460	460	HE 160 A	S 355	0,84	(6.62)	S 355												
26' / G		3	GF-roof	119	37,4		280	280	HE 200 A	S 355	0,33	(6.62)	S 355												
26' H		3	GF-roof	170	37,4		280	280	HE 200 A	S 355	0,37	(6.62)	S 355												
26' / I		3	GF-roof	170	37,4		280	280	HE 200 A	S 355	0,37	(6.62)	S 355												
26' / I'		3	GF-roof	24	40,3		320	320	HE 160 A	S 355	0,50	(6.62)	S 355												
26' / J		3	GF-roof	509	4,6	35,0	280	280	HE 200 B	S 355	0,52	(6.62)	S 355												
F / 25		3	GF-roof	21	9,9		280	280	HE 160 A	S 355	0,14	(6.62)	S 355												
F / 26		3	GF-roof	21	9,9		280	280	HE 160 A	S 355	0,14	(6.62)	S 355												

Column bases, anchor plates and foundation loads



Special proofs for base plates / foundation pressure and anchor plates are given in the attached positions.

In the stuctural analysys, it is assumed that the foundations will be constructed as strip foundations.

Concrete quality:	**C35/45**
Exposure classes:	**XC3, XD3, XF2 (c_{nom} = 55 mm)**
Concrete moisture class:	**WA**

It is a monitoring class 2 concrete.

If the in-situ concrete foundations are replaced by precast concrete part foundations in some areas, proof of anchoring is provided in the precast foundations!

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project:	PR0460	MSCP Neratovice														Pos CO001
columns - baseplate design / selection of pin plates												full load + wind + imperfection				v25.30
	column					base plate							pin bolt plate				foundation anchorage
	design values		Nd= 1,35*G + 1,50*(P+0,6*V)														
location	max N_d [kN]	min N_d [kN]	max H_d [kN]	column profile	concrete class foundation C30/37	GW-Nr.		thickness s ≥ [mm]	length [mm]	width [mm]	steel grade	welding detail	GW-Nr.	welding detail	remark	BOCAD-type	type / GW-No
A / 2	496		35	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 3	469		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 4	469		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 5	496		35	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 6	654		35	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 7	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 8	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 9	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 10	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 11	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 12	686		44	HE 180 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
A / 13	650		44	HE 180 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
A / 14	686		44	HE 180 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
A / 15	686		44	HE 180 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
A / 16	650		44	HE 180 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
A / 17	686		44	HE 180 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
A / 18	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 19	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 20	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 21	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 22	614		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 23	654		35	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 24	496		35	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 25	469		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 26	469		33	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
A / 27	496		35	HE 180 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
H / 6	1.061		33	HE 240 A	C30/37	78574	f	**25**	320	300	S 355	**all around**	**78652**	**all around**		**TYP 7**	**Q3 / 78607**
H / 7	983		30	HE 240 A	C30/37	78574	f	**25**	320	300	S 355	**all around**	**78652**	**all around**		**TYP 7**	**Q3 / 78607**
H / 8	983		30	HE 240 A	C30/37	78574	f	**25**	320	300	S 355	**all around**	**78652**	**all around**		**TYP 7**	**Q3 / 78607**
H / 9	983		30	HE 240 A	C30/37	78574	f	**25**	320	300	S 355	**all around**	**78652**	**all around**		**TYP 7**	**Q3 / 78607**
H / 10	983		30	HE 240 A	C30/37	78574	f	**25**	320	300	S 355	**all around**	**78652**	**all around**		**TYP 7**	**Q3 / 78607**
H / 11	1.064		53	HE 240 A	C30/37	78578	f	**25**	320	300	S 355	**all around**	**78669**	**all around**		**TYP 12**	**H5 / 78619**
H / 12	1.023		53	HE 240 A	C30/37	78578	f	**25**	320	300	S 355	**all around**	**78669**	**all around**		**TYP 12**	**H5 / 78619**
H / 13	1.023		53	HE 240 A	C30/37	78578	f	**25**	320	300	S 355	**all around**	**78669**	**all around**		**TYP 12**	**H5 / 78619**
H / 14	1.064		53	HE 240 A	C30/37	78578	f	**25**	320	300	S 355	**all around**	**78669**	**all around**		**TYP 12**	**H5 / 78619**
H / 15	983		30	HE 240 A	C30/37	78578	f	**25**	320	300	S 355	**all around**	**78669**	**all around**		**TYP 12**	**H5 / 78619**
H / 16	1.064		53	HE 240 A	C30/37	78578	f	**25**	320	300	S 355	**all around**	**78669**	**all around**		**TYP 12**	**H5 / 78619**
H / 17	1.023		53	HE 240 A	C30/37	78578	f	**25**	320	300	S 355	**all around**	**78669**	**all around**		**TYP 12**	**H5 / 78619**

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project:	PR0460	MSCP Neratovice															Pos CO001
columns - baseplate design / selection of pin plates												full load + wind + imperfection					v25.30
	column					base plate							pin bolt plate				foundation anchorage
	design values		Nd= 1,35*G + 1,50*(P+0,6*V)		concrete class foundation												
location	max N_d	min N_d	max H_d	column profile		GW-Nr.		thickness s ≥	length	width	steel grade	welding detail	GW-Nr.	welding detail	remark	BOCAD-type	type / GW-No
	[kN]	[kN]	[kN]		C30/37			[mm]	[mm]	[mm]							
H / 18	1.023		53	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
H / 19	1.064		53	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
H / 20	983		30	HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
H / 21	983		30	HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
H / 22	983		30	HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
H / 23	1.061		33	HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 2	1.706			HE 220 B	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 5	1.706			HE 220 B	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 6	1.042			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 7	965			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 8	965			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 9	965			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 10	965			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 11	1.036		50	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 12	1.000		50	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 13	1.000		50	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 14	1.036		50	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 15	965			HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 16	1.036		50	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 17	1.000		50	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 18	1.000		50	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 19	1.036		50	HE 240 A	C30/37	78578	f	25	320	300	S 355	all around	78669	all around		TYP 12	H5 / 78619
O / 20	965			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 21	965			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 22	965			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 23	1.042			HE 240 A	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 24	1.706			HE 220 B	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
O / 27	1.706			HE 220 B	C30/37	78574	f	25	320	300	S 355	all around	78652	all around		TYP 7	Q3 / 78607
V / 2	1.072			HE 180 B	C30/37	78572	f	20	270	250	S 355	all around	78651	all around		TYP 5	Q2 / 78605
V / 3	353			HE 200 A	C30/37												
V / 4	353			HE 200 A	C30/37							entry / exit					
V / 5	1.072			HE 180 B	C30/37	78572	f	20	270	250	S 355	all around	78651	all around		TYP 5	Q2 / 78605
V / 6	560			HE 200 A	C30/37	78572	f	20	270	250	S 355	all around	78651	all around		TYP 5	Q2 / 78605
V / 7	521			HE 200 A	C30/37	78572	f	20	270	250	S 355	all around	78651	all around		TYP 5	Q2 / 78605
V / 8	521			HE 200 A	C30/37	78572	f	20	270	250	S 355	all around	78651	all around		TYP 5	Q2 / 78605
V / 9	521			HE 200 A	C30/37	78572	f	20	270	250	S 355	all around	78651	all around		TYP 5	Q2 / 78605

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project:	PR0460	MSCP Neratovice														Pos CO001
columns - baseplate design / selection of pin plates												full load + wind + imperfection				v25.30
	column					base plate							pin bolt plate				foundation anchorage
	design values		Nd= 1,35*G + 1,50*(P+0,6*V)														
location	max N_d	min N_d	max H_d	column profile	concrete class foundation	GW-Nr.		thickness s ≥	length	width	steel grade	welding detail	GW-Nr.	welding detail	remark	BOCAD-type	type / GW-No
	[kN]	[kN]	[kN]		C30/37			[mm]	[mm]	[mm]							
V / 10	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 11	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 12	574		38	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
V / 13	548		38	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
V / 14	574		38	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
V / 15	574		38	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
V / 16	548		38	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
V / 17	574		38	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**H4 / 78618**
V / 18	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 19	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 20	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 21	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 22	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 23	560			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 24	560			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 25	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 26	521			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
V / 27	560			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / A	172		18	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / B	241		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / C	203		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / D	218		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / E	218		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / F	202		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / F'	116		16	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / I'	92			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / J	175			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / K	190			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / L	190			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / M	175			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / N	214			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / O	180			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / P	214			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / Q	175			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / R	190			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / S	190			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / T	175			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
1 / U	214			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**

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project:	PR0460	MSCP Neratovice														Pos CO001
columns - baseplate design / selection of pin plates													full load + wind + imperfection			v25.30
	column					base plate							pin bolt plate				foundation anchorage
	design values		Nd= 1,35*G + 1,50*(P+0,6*V)														
location	max N_d	min N_d	max H_d	column profile	concrete class foundation	GW-Nr.		thickness s ≥	length	width	steel grade	welding detail	GW-Nr.	welding detail	remark	BOCAD-type	type / GW-No
	[kN]	[kN]	[kN]		C30/37			[mm]	[mm]	[mm]							
1 / V	143			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / A	139		18	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / B	175		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / C	137		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / D	152		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / E	152		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / F	137		33	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / F'	83		16	HE 160 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / I'	59			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / J	109			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / K	125			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / L	125			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / M	110			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / N	148			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / O	114			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / P	148			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / Q	110			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / R	125			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / S	125			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / T	110			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / U	148			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
28 / V	78			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
2' / F	904		30	HE 200 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
2' / F'	60		30	HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
2' / G	308		30	HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
2' H	307		30	HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
2' / I	307		30	HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
2' / I'	58		30	HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
2' / J	892		30	HE 200 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
4' / F	1.262	-185	206	HE 200 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
4' / G	558		149	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
4' / H	637		210	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
4' / I	594		186	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
4' / J	1.239	-183	215	HE 200 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
24' / F	1.238	-195	221	HE 200 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**

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project:	PR0460	MSCP Neratovice															Pos CO001
columns - baseplate design / selection of pin plates													full load + wind + imperfection				v25.30
	column					base plate							pin bolt plate				foundation anchorage
	design values		Nd= 1,35*G + 1,50*(P+0,6*V)														
location	max N_d	min N_d	max H_d	column profile	concrete class foundation	GW-Nr.		thickness s ≥	length	width	steel grade	welding detail	GW-Nr.	welding detail	remark	BOCAD-type	type / GW-No
	[kN]	[kN]	[kN]		C30/37			[mm]	[mm]	[mm]							
24' / G	538		149	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
24' H	618		210	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
24' / I	576		186	HE 200 A	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
24' / J	1.230	-189	215	HE 200 B	C30/37	78577	f	**25**	270	250	S 355	**all around**	**78668**	**all around**		**TYP 11**	**Pos. 78666**
26' / F	877		15	HE 200 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
26' / F'	51		15	HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
26' / G	204			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
26' H	288			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
26' / I	288			HE 200 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
26' / I'	40			HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
26' / J	883			HE 200 B	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
F / 25	52		30	HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**
F / 26	52		30	HE 160 A	C30/37	78572	f	**20**	270	250	S 355	**all around**	**78651**	**all around**		**TYP 5**	**Q2 / 78605**

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project: PR0460 MSCP Neratovice															Pos CO001		
foundation loads + anchoring [kN]							full load + wind + imperfection						v25.30				
	representative values										design values						
	perm.	variable loads							total		1,35 / 1,5(·0,6) -fold				anchorage	splitting tensile reinforcement	remark
location	loads	traffic	roof	sum	support reactions from bracings			wind suction	max N	min N	max N_d	min N_d	column profile	steel grade			
	G	P	P	Q	± V	± H_x	± H_y	W_s	G+Q+|V|	G - |V| + W_s	γG / γQ 1,35 / 1,5	γG / γQ 0,9 / 1,5			type / GW-No		
A / 2	156	153	38	191			24		347	156	496	140	HE 180 B	**S 355**	Q2 / 78605	–	
A / 3	146	147	35	182			22		327	146	469	131	HE 180 B	**S 355**	Q2 / 78605	–	
A / 4	146	147	35	182			22		327	146	469	131	HE 180 B	**S 355**	Q2 / 78605	–	
A / 5	156	153	38	191			24		347	156	496	140	HE 180 B	**S 355**	Q2 / 78605	–	
A / 6	216	187	55	242			24		458	216	654	194	HE 180 B	**S 355**	Q2 / 78605	–	
A / 7	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 8	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 9	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 10	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 11	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 12	201	178	51	228	80	29	22		509	121	686	61	HE 180 B	**S 355**	H4 / 78618	–	
A / 13	201	178	51	228	40	29	22		469	161	650	121	HE 180 B	**S 355**	H4 / 78618	–	
A / 14	201	178	51	228	80	29	22		509	121	686	61	HE 180 B	**S 355**	H4 / 78618	–	
A / 15	201	178	51	228	80	29	22		509	121	686	61	HE 180 B	**S 355**	H4 / 78618	–	
A / 16	201	178	51	228	40	29	22		469	161	650	121	HE 180 B	**S 355**	H4 / 78618	–	
A / 17	201	178	51	228	80	29	22		509	121	686	61	HE 180 B	**S 355**	H4 / 78618	–	
A / 18	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 19	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 20	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 21	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 22	201	178	51	228			22		429	201	614	181	HE 180 B	**S 355**	Q2 / 78605	–	
A / 23	216	187	55	242			24		458	216	654	194	HE 180 B	**S 355**	Q2 / 78605	–	
A / 24	156	153	38	191			24		347	156	496	140	HE 180 B	**S 355**	Q2 / 78605	–	
A / 25	146	147	35	182			22		327	146	469	131	HE 180 B	**S 355**	Q2 / 78605	–	
A / 26	146	147	35	182			22		327	146	469	131	HE 180 B	**S 355**	Q2 / 78605	–	
A / 27	156	153	38	191			24		347	156	496	140	HE 180 B	**S 355**	Q2 / 78605	–	
H / 6	409	229	110	339			22		748	409	1.061	368	HE 240 A	**S 355**	Q3 / 78607	–	
H / 7	381	211	101	312			20		693	381	983	343	HE 240 A	**S 355**	Q3 / 78607	–	
H / 8	381	211	101	312			20		693	381	983	343	HE 240 A	**S 355**	Q3 / 78607	–	
H / 9	381	211	101	312			20		693	381	983	343	HE 240 A	**S 355**	Q3 / 78607	–	
H / 10	381	211	101	312			20		693	381	983	343	HE 240 A	**S 355**	Q3 / 78607	–	
H / 11	381	211	101	312	90	35	20		783	291	1.064	208	HE 240 A	**S 355**	H5 / 78619	–	
H / 12	381	211	101	312	45	35	20		738	336	1.023	276	HE 240 A	**S 355**	H5 / 78619	–	
H / 13	381	211	101	312	45	35	20		738	336	1.023	276	HE 240 A	**S 355**	H5 / 78619	–	
H / 14	381	211	101	312	90	35	20		783	291	1.064	208	HE 240 A	**S 355**	H5 / 78619	–	
H / 15	381	211	101	312			20		693	381	983	343	HE 240 A	**S 355**	H5 / 78619	–	
H / 16	381	211	101	312	90	35	20		783	291	1.064	208	HE 240 A	**S 355**	H5 / 78619	–	
H / 17	381	211	101	312	45	35	20		738	336	1.023	276	HE 240 A	**S 355**	H5 / 78619	–	

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project: PR0460 MSCP Neratovice															Pos CO001		
foundation loads + anchoring [kN]								full load + wind + imperfection						v25.30			
	representative values										design values						
	perm.	variable loads							total		1,35 / 1,5(·0,6) -fold				anchorage	splitting tensile reinforcement	remark
location	loads	traffic	roof	sum	support reactions from bracings			wind suction	max N	min N	max N_d	min N_d	column profile	steel grade			
	G	P	P	Q	± V	± H_x	± H_y	W_s	G+Q+|V|	G - |V| + W_s	γG / γQ 1,35 / 1,5	γG / γQ 0,9 / 1,5			type / GW-No		
H / 18	381	211	101	312	45	35	20		738	336	1.023	276	HE 240 A	**S 355**	H5 / 78619	–	
H / 19	381	211	101	312	90	35	20		783	291	1.064	208	HE 240 A	**S 355**	H5 / 78619	–	
H / 20	381	211	101	312			20		693	381	983	343	HE 240 A	**S 355**	Q3 / 78607	–	
H / 21	381	211	101	312			20		693	381	983	343	HE 240 A	**S 355**	Q3 / 78607	–	
H / 22	381	211	101	312			20		693	381	983	343	HE 240 A	**S 355**	Q3 / 78607	–	
H / 23	409	229	110	339			22		748	409	1.061	368	HE 240 A	**S 355**	Q3 / 78607	–	
O / 2	659	366	179	545					1.203	659	1.706	593	HE 220 B	**S 355**	Q3 / 78607	–	
O / 5	659	366	179	545					1.203	659	1.706	593	HE 220 B	**S 355**	Q3 / 78607	–	
O / 6	395	229	110	339					734	395	1.042	356	HE 240 A	**S 355**	Q3 / 78607	–	
O / 7	368	211	101	312					680	368	965	331	HE 240 A	**S 355**	Q3 / 78607	–	
O / 8	368	211	101	312					680	368	965	331	HE 240 A	**S 355**	Q3 / 78607	–	
O / 9	368	211	101	312					680	368	965	331	HE 240 A	**S 355**	Q3 / 78607	–	
O / 10	368	211	101	312					680	368	965	331	HE 240 A	**S 355**	Q3 / 78607	–	
O / 11	368	211	101	312	79	33			759	289	1.036	213	HE 240 A	**S 355**	H5 / 78619	–	
O / 12	368	211	101	312	40	33			720	329	1.000	272	HE 240 A	**S 355**	H5 / 78619	–	
O / 13	368	211	101	312	40	33			720	329	1.000	272	HE 240 A	**S 355**	H5 / 78619	–	
O / 14	368	211	101	312	79	33			759	289	1.036	213	HE 240 A	**S 355**	H5 / 78619	–	
O / 15	368	211	101	312					680	368	965	331	HE 240 A	**S 355**	H5 / 78619	–	
O / 16	368	211	101	312	79	33			759	289	1.036	213	HE 240 A	**S 355**	H5 / 78619	–	
O / 17	368	211	101	312	40	33			720	329	1.000	272	HE 240 A	**S 355**	H5 / 78619	–	
O / 18	368	211	101	312	40	33			720	329	1.000	272	HE 240 A	**S 355**	H5 / 78619	–	
O / 19	368	211	101	312	79	33			759	289	1.036	213	HE 240 A	**S 355**	H5 / 78619	–	
O / 20	368	211	101	312					680	368	965	331	HE 240 A	**S 355**	Q3 / 78607	–	
O / 21	368	211	101	312					680	368	965	331	HE 240 A	**S 355**	Q3 / 78607	–	
O / 22	368	211	101	312					680	368	965	331	HE 240 A	**S 355**	Q3 / 78607	–	
O / 23	395	229	110	339					734	395	1.042	356	HE 240 A	**S 355**	Q3 / 78607	–	
O / 24	659	366	179	545					1.203	659	1.706	593	HE 220 B	**S 355**	Q3 / 78607	–	
O / 27	659	366	179	545					1.203	659	1.706	593	HE 220 B	**S 355**	Q3 / 78607	–	
V / 2	390	309	55	364					754	390	1.072	351	HE 180 B	**S 355**	Q2 / 78605	–	
V / 3	**entry / exit**															–	
V / 4																–	
V / 5	390	309	55	364					754	390	1.072	351	HE 180 B	**S 355**	Q2 / 78605	–	
V / 6	199	139	55	194					393	199	560	179	HE 200 A	**S 355**	Q2 / 78605	–	
V / 7	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 8	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 9	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	

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project: PR0460 MSCP Neratovice															Pos CO001		
foundation loads + anchoring [kN]								full load + wind + imperfection						v25.30			
	representative values										design values						
	perm.	variable loads							total		1,35 / 1,5(·0,6) -fold				anchorage	splitting tensile reinforcement	remark
location	loads	traffic	roof	sum	support reactions from bracings			wind suction	max N	min N	max N_d	min N_d	column profile	steel grade			
	G	P	P	Q	± V	± H_x	± H_y	W_s	G+Q+|V|	G - |V| + W_s	γG / γQ 1,35 / 1,5	γG / γQ 0,9 / 1,5			type / GW-No		
V / 10	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 11	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 12	185	130	51	181	59	25			425	126	574	78	HE 200 A	**S 355**	H4 / 78618	–	
V / 13	185	130	51	181	30	25			396	156	548	123	HE 200 A	**S 355**	H4 / 78618	–	
V / 14	185	130	51	181	59	25			425	126	574	78	HE 200 A	**S 355**	H4 / 78618	–	
V / 15	185	130	51	181	59	25			425	126	574	78	HE 200 A	**S 355**	H4 / 78618	–	
V / 16	185	130	51	181	30	25			396	156	548	123	HE 200 A	**S 355**	H4 / 78618	–	
V / 17	185	130	51	181	59	25			425	126	574	78	HE 200 A	**S 355**	H4 / 78618	–	
V / 18	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 19	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 20	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 21	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 22	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 23	199	139	55	194					393	199	560	179	HE 200 A	**S 355**	Q2 / 78605	–	
V / 24	199	139	55	194					393	199	560	179	HE 200 A	**S 355**	Q2 / 78605	–	
V / 25	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 26	185	130	51	181					366	185	521	167	HE 200 A	**S 355**	Q2 / 78605	–	
V / 27	199	139	55	194					393	199	560	179	HE 200 A	**S 355**	Q2 / 78605	–	
1 / A	32	81	5	86		12	12		118	32	172	29	HE 160 B	**S 355**	Q2 / 78605	–	
1 / B	57	97	13	110		22			166	57	241	51	HE 160 B	**S 355**	Q2 / 78605	–	
1 / C	42	89	9	98		22			139	42	203	37	HE 160 B	**S 355**	Q2 / 78605	–	
1 / D	48	92	10	102		22			150	48	218	43	HE 160 B	**S 355**	Q2 / 78605	–	
1 / E	48	92	10	102		22			150	48	218	43	HE 160 B	**S 355**	Q2 / 78605	–	
1 / F	43	88	8	96		22			139	43	202	39	HE 160 B	**S 355**	Q2 / 78605	–	
1 / F'	33	44	4	48		11			81	33	116	29	HE 160 B	**S 355**	Q2 / 78605	–	
1 / I'	16	42	4	46					63	16	92	15	HE 160 A	**S 355**	Q2 / 78605	–	
1 / J	27	85	8	93					119	27	175	24	HE 160 A	**S 355**	Q2 / 78605	–	
1 / K	31	89	10	99					130	31	190	28	HE 160 A	**S 355**	Q2 / 78605	–	
1 / L	31	89	10	99					130	31	190	28	HE 160 A	**S 355**	Q2 / 78605	–	
1 / M	25	86	9	94					119	25	175	23	HE 160 A	**S 355**	Q2 / 78605	–	
1 / N	40	94	13	106					146	40	214	36	HE 160 A	**S 355**	Q2 / 78605	–	
1 / O	27	87	9	96					122	27	180	24	HE 160 A	**S 355**	Q2 / 78605	–	
1 / P	40	94	13	106					146	40	214	36	HE 160 A	**S 355**	Q2 / 78605	–	
1 / Q	25	86	9	94					119	25	175	23	HE 160 A	**S 355**	Q2 / 78605	–	
1 / R	31	89	10	99					130	31	190	28	HE 160 A	**S 355**	Q2 / 78605	–	
1 / S	31	89	10	99					130	31	190	28	HE 160 A	**S 355**	Q2 / 78605	–	
1 / T	25	86	9	94					119	25	175	23	HE 160 A	**S 355**	Q2 / 78605	–	
1 / U	40	94	13	106					146	40	214	36	HE 160 A	**S 355**	Q2 / 78605	–	

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project: PR0460 MSCP Neratovice															Pos CO001		
foundation loads + anchoring [kN]						full load + wind + imperfection								v25.30			
	representative values										design values						
		variable loads							total		1,35 / 1,5(·0,6) -fold				anchorage	splitting tensile reinforcement	remark
location	perm. loads	traffic	roof	sum	support reactions from bracings			wind suction	max N	min N	max N_d	min N_d	column profile	steel grade			
	G	P	P	Q	± V	± H_x	± H_y	W_s	G+Q+|V|	G - |V| + W_s	γG / γQ 1,35 / 1,5	γG / γQ 0,9 / 1,5			type / GW-No		
1 / V	15	78	5	82					97	15	143	13	HE 160 A	**S 355**	Q2 / 78605	–	
28 / A	32	59	5	64		12	12		96	32	139	29	HE 160 B	**S 355**	Q2 / 78605	–	
28 / B	57	54	13	66		22			123	57	175	51	HE 160 B	**S 355**	Q2 / 78605	–	
28 / C	42	46	9	54		22			96	42	137	37	HE 160 B	**S 355**	Q2 / 78605	–	
28 / D	48	49	10	59		22			106	48	152	43	HE 160 B	**S 355**	Q2 / 78605	–	
28 / E	48	49	10	59		22			106	48	152	43	HE 160 B	**S 355**	Q2 / 78605	–	
28 / F	43	45	8	53		22			96	43	137	39	HE 160 B	**S 355**	Q2 / 78605	–	
28 / F'	33	22	4	26		11			59	33	83	29	HE 160 B	**S 355**	Q2 / 78605	–	
28 / I'	16	20	4	24					41	16	59	15	HE 160 A	**S 355**	Q2 / 78605	–	
28 / J	27	41	8	49					76	27	109	24	HE 160 A	**S 355**	Q2 / 78605	–	
28 / K	31	45	10	55					86	31	125	28	HE 160 A	**S 355**	Q2 / 78605	–	
28 / L	31	45	10	55					86	31	125	28	HE 160 A	**S 355**	Q2 / 78605	–	
28 / M	25	42	9	50					76	25	110	23	HE 160 A	**S 355**	Q2 / 78605	–	
28 / N	40	50	13	62					103	40	148	36	HE 160 A	**S 355**	Q2 / 78605	–	
28 / O	27	43	9	52					79	27	114	24	HE 160 A	**S 355**	Q2 / 78605	–	
28 / P	40	50	13	62					103	40	148	36	HE 160 A	**S 355**	Q2 / 78605	–	
28 / Q	25	42	9	50					76	25	110	23	HE 160 A	**S 355**	Q2 / 78605	–	
28 / R	31	45	10	55					86	31	125	28	HE 160 A	**S 355**	Q2 / 78605	–	
28 / S	31	45	10	55					86	31	125	28	HE 160 A	**S 355**	Q2 / 78605	–	
28 / T	25	42	9	50					76	25	110	23	HE 160 A	**S 355**	Q2 / 78605	–	
28 / U	40	50	13	62					103	40	148	36	HE 160 A	**S 355**	Q2 / 78605	–	
28 / V	15	34	5	38					53	15	78	13	HE 160 A	**S 355**	Q2 / 78605	–	
2' / F	354	179	105	284		20		-11	638	343	904	302	HE 200 B	**S 355**	Q2 / 78605	–	
2' / F'	31	8	4	12		20			43	31	60	28	HE 160 A	**S 355**	Q2 / 78605	–	
2' / G	134	46	39	85		20		-11	219	123	308	104	HE 200 A	**S 355**	Q2 / 78605	–	
2' H	134	46	39	85		20		-11	218	123	307	104	HE 200 A	**S 355**	Q2 / 78605	–	
2' / I	133	46	39	85		20		-11	218	122	307	103	HE 200 A	**S 355**	Q2 / 78605	–	
2' / I'	30	8	4	12		20			42	30	58	27	HE 160 A	**S 355**	Q2 / 78605	–	
2' / J	345	179	105	284		20		-11	629	334	892	294	HE 200 B	**S 355**	Q2 / 78605	–	
4' / F	353	180	106	286	324	20	137	-11	962	18	1.262	-185	HE 200 B	**S 355**	Pos. 78666	–	
4' / G	185	71	51	122	29	20	99	-11	336	145	558	106	HE 200 A	**S 355**	Pos. 78666	–	
4' / H	171	69	48	116	20	10	140	-11	307	140	637	107	HE 200 A	**S 355**	Pos. 78666	–	
4' / I	171	71	51	122	26	20	124	-11	319	134	594	98	HE 200 A	**S 355**	Pos. 78666	–	
4' / J	344	180	106	286	317	20	143	-11	946	16	1.239	-183	HE 200 B	**S 355**	Pos. 78666	–	
24' / F	342	184	93	277	324	10	147	-11	943	7	1.238	-195	HE 200 B	**S 355**	Pos. 78666	–	

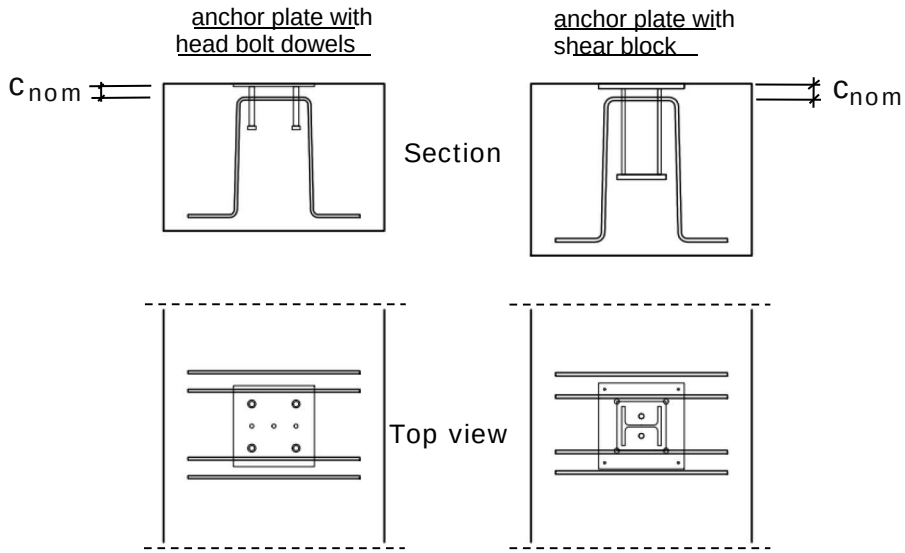
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project: PR0460 MSCP Neratovice															Pos CO001		
foundation loads + anchoring [kN] full load + wind + imperfection v25.30																	
	representative values										design values						
		variable loads							total		1,35 / 1,5(·0,6) -fold						
location	perm. loads	traffic	roof	sum	support reactions from bracings			wind suction	max N	min N	max N_d	min N_d	column profile	steel grade	anchorage	splitting tensile reinforcement	remark
	G	P	P	Q	± V	± H_x	± H_y	W_s	G+Q+|V|	G - |V| + W_s	γG / γQ 1,35 / 1,5	γG / γQ 0,9 / 1,5			type / GW-No		
24' / G	170	71	51	122	29	20	99	-11	321	130	538	93	HE 200 A	**S 355**	Pos. 78666	–	
24' H	157	69	48	116	20	10	140	-11	293	126	618	95	HE 200 A	**S 355**	Pos. 78666	–	
24' / I	157	71	51	122	26		124	-11	305	120	576	86	HE 200 A	**S 355**	Pos. 78666	–	
24' / J	337	180	106	286	317		143	-11	940	9	1.230	-189	HE 200 B	**S 355**	Pos. 78666	–	
26' / F	343	183	93	276		10	10	-11	619	332	877	292	HE 200 B	**S 355**	Q2 / 78605	–	
26' / F'	24	8	4	12		10			36	24	51	22	HE 160 A	**S 355**	Q2 / 78605	–	
26' / G	83	46	16	62				-11	144	72	204	58	HE 200 A	**S 355**	Q2 / 78605	–	
26' H	120	46	39	85				-11	204	109	288	91	HE 200 A	**S 355**	Q2 / 78605	–	
26' / I	120	46	39	85				-11	204	109	288	91	HE 200 A	**S 355**	Q2 / 78605	–	
26' / I'	16	8	4	12					28	16	40	15	HE 160 A	**S 355**	Q2 / 78605	–	
26' / J	339	179	105	284				-11	623	328	883	288	HE 200 B	**S 355**	Q2 / 78605	–	
F / 25	28	10		10			20		38	28	52	25	HE 160 A	**S 355**	Q2 / 78605	–	
F / 26	28	10		10			20		38	28	52	25	HE 160 A	**S 355**	Q2 / 78605	–	

Pos. CO100 Type: 78605 ff. Standard / Special anchor plates

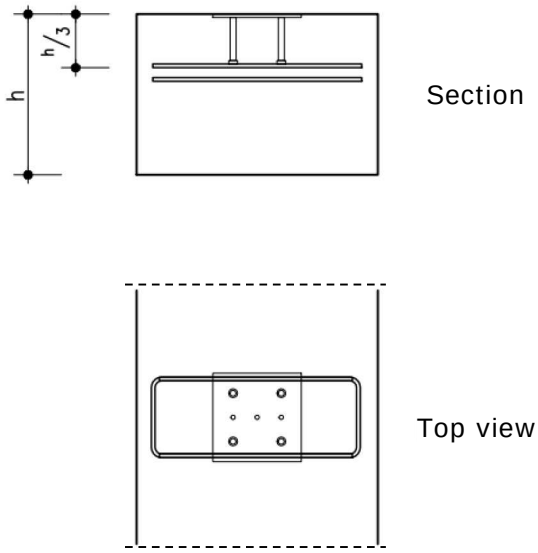
Standard details for additional reinforcement at anchor plates

1. Suspended reinforcement
function:Anchoring of uplifting vertical loads

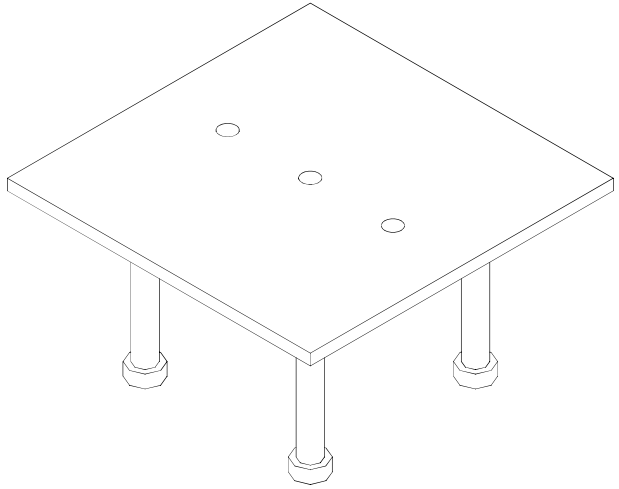


Information on the required amount of reinforcement can be found in the structural analysis.

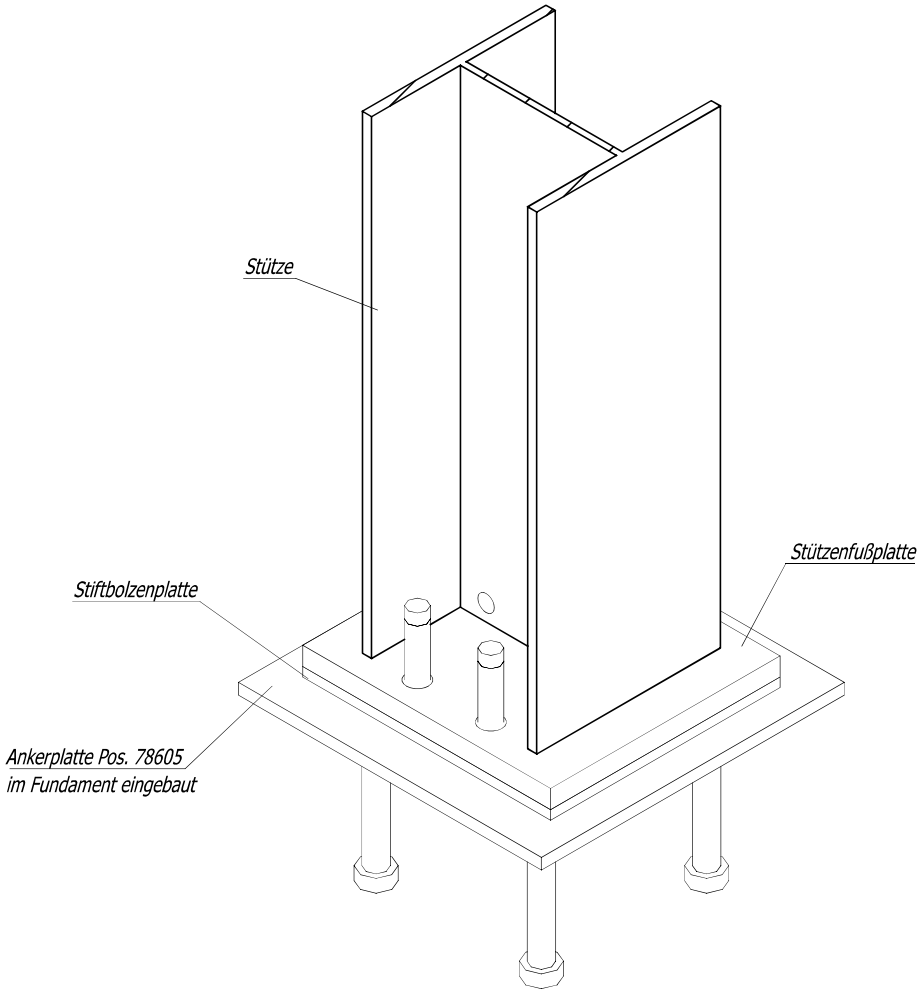
2. Reinforcement for splitting tension
Function: Increase of permissible concrete stress (local bearing pressure) due to high compressive vertical loads



Information on the required amount of reinforcement can be found in the structural analysis.



Perspektive der Ankerplatte Pos. 78605



Einbausituation

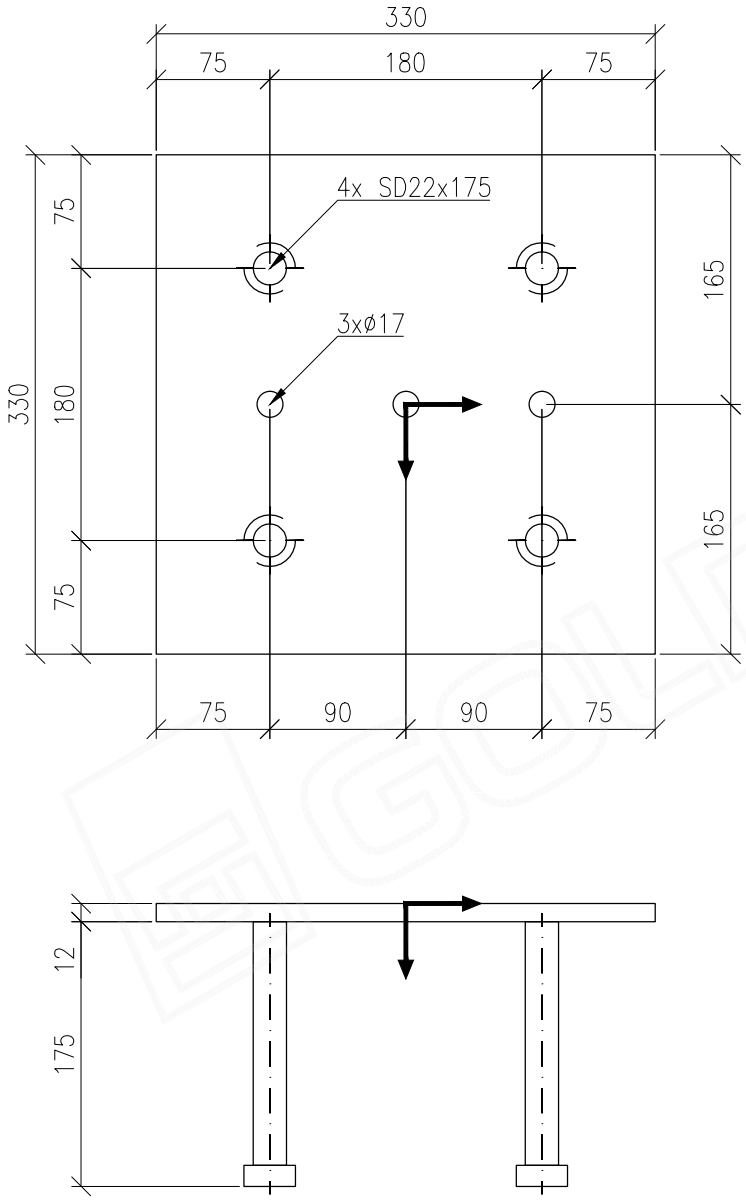
		.	GOLDBECK

Ummelner Str. 4-6, 33649 Bielefeld, Telefon 0521/9488-0

Planinhalt:
Hauptposition 78605
AP - ANKERPLATTE
Einbausituation

Phase	Planer	Art
5	WP	FE
Ebene	Laufende-Nr.:	Index
XX	78605	-

bocad-3d, Rev. 24.1.82/8



*EXC 2 nach EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78605	Ankerplatte	s. Versandliste	BL12x330	330	S235JR	
Stahlgewicht:	10,30 kg	Mantelfläche:	0,23 m²	Versandmaße:	(H x B x L)	330 x 187 x 330

Stückzahl	Schraube	DIN	Material	Gewicht	Bemerkung
4	SD22x175	13918	S235J2G3+C450	0,6 kg	Werkstatt
Schraubengewicht:				2,4 kg	
Beschichtung:	schwarz			Gesamtgewicht (kg):	12,70

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Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

Ankerplatte
330mm x 330mm t=12mm

gezeichnet	30.05.2012	[redacted]	
geprüft			
Maßstab	1:5	DWG-Name	[redacted]

a	22.05.2015	Weisters / Wisotzky	Stückliste	
Zusatz		(27)		c19
Tabelle 0		Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G40	78605	a

Blattgröße: 210mm x 297mm

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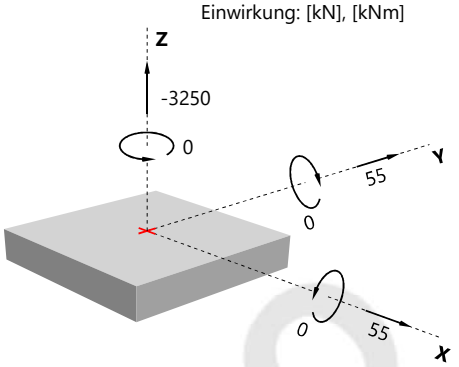
Köco StudCalc-Pro 3.1.0 - Erweiterter Ausdruck

Firma:	Goldbeck Südwest GmbH	E-Mail:	
Bearbeiter:		Tel.:	
Adresse:		Fax:	
Projekt:		Datum:	21.07.2022
Kommentar:	Q2 / 78605	Seite:	1 / 5

1. Eingabedaten

Untergrund:
- Gerissener Beton, Bauteildicke h=600mm, c_{nom}=55mm
  Festigkeitsklasse C30/37, $f_{ck,cube}$=37,0N/mm²
- Weite Flächenbewehrung
  Stababstand a≥150mm für alle Ø, oder a≥100mm für Ø≤10mm
- Mit gerader Randbewehrung (Ø ≥ 12mm)

Einwirkung: [kN], [kNm]



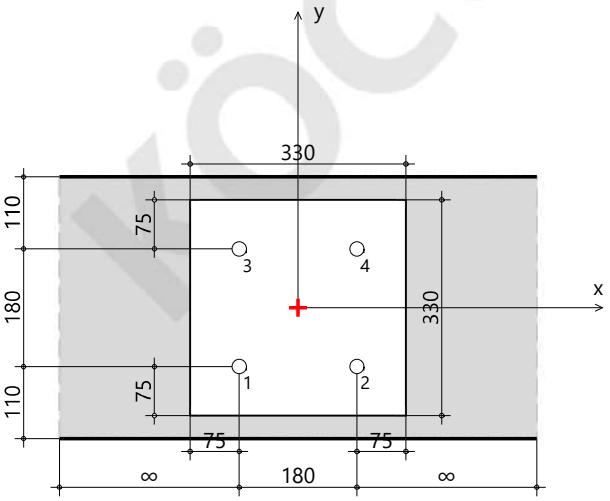
Zug Zusatzbewehrung
- Keine

Quer Zusatzbewehrung
- Keine

Einwirkung:
- Vorwiegend ruhende Designlasten

Montageart:
- Ankerplatte liegt ebenbündig im Beton
- Gute Betonverdichtung unter der Ankerplatte

Ankerplatte:
- S 235 (St 37), E=210000N/mm²
  f_{yk}=235N/mm², γ_s=1,5
- Angenommen: biegestarre Ankerplatte
- Verwendete Dicke: 12,0mm
- Erforderliche Dicke wurde nicht berechnet.
- Rechteckform
  Seitenlänge: 330 x 330 mm

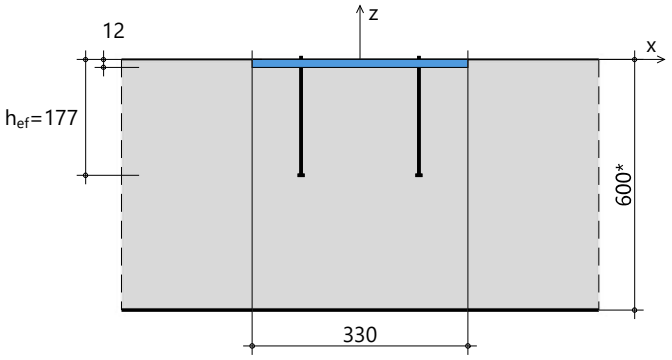


Profil:
- kein Profil
  Lasteinwirkungspunkt [mm]: [0, 0]

Ankerkoordinaten [mm]:

Nr.	x	y	Langloch L-x	Langloch L-y
1	-90,0	-90,0		
2	90,0	-90,0		
3	-90,0	90,0		
4	90,0	90,0		

Gewählte Anker:
- Köco SD1 d22 x 175
  Kopfbolzen
  Baustahl S235J2+C470
  Bemessung gemäß CEN/TS 1992-4-2
- Bewertung ETA-03/0039
  erteilt von DIBt, am 05.06.2018
- Verankerungstiefe h_{ef} = 177 mm

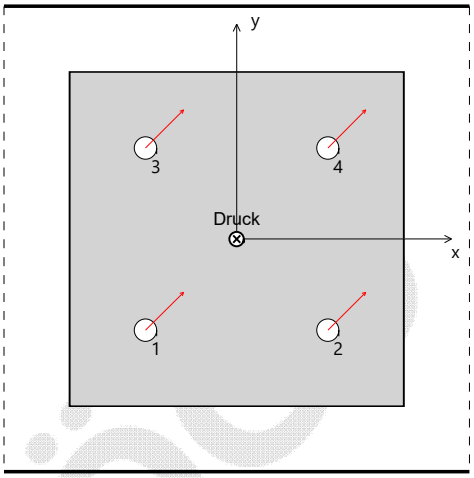


(* nicht maßstäblich gezeichnet)

2. Ankerschnittkräfte und Nachweis der Ankerplatten-Biegesteifigkeit

Ankerschnittkräfte [kN]

Anker-Nr.	Normalkraft	Querkraft	Querkraft x	Querkraft y
1	0,000	19,445	13,750	13,750
2	0,000	19,445	13,750	13,750
3	0,000	19,445	13,750	13,750
4	0,000	19,445	13,750	13,750



maximale Betonstauchung [‰]: 0,9044
maximale Betondruckspannung: 29,84 [N/mm²]
resultierende Zugkraft in (x/y=0,0/0,0): 0,000 [kN]
resultierende Druckkraft in (x/y=0,0/0,0): 3250,000 [kN]
Bemerkung: Der Randabstand ist nicht maßstäblich.

Nachweisbedingungen:

a) $\sigma \leq f_{yd}$

b) $N_{r,max} \approx N_{e,max}$

$N_{r,max}$: maximale Ankerzugkraft bei biegestarrer Ankerplatte

$N_{e,max}$: maximale Ankerzugkraft bei elastischer Ankerplatte

Der Nachweis für die Ankerplatten-Biegesteifigkeit wurde nicht durchgeführt.

3. Nachweis für den Grenzzustand der Tragfähigkeit gemäß CEN/TS 1992-4-2

3.1 Zugbeanspruchung

	bezogene Anker	Einwirkung [kN]	Widerstand [kN]	Ausnutzung [%]	Status
Stahlversagen	-	-	-	-	Ohne Nachweis
Herausziehen	-	-	-	-	Ohne Nachweis
Betonausbruch	-	-	-	-	Ohne Nachweis
Spaltversagen	-	-	-	-	Ohne Nachweis
Lokaler Betonausbruch	-	-	-	-	Ohne Nachweis
Stahlversagen Bew	-	-	-	-	Ohne Nachweis
Verbundversagen Bew	-	-	-	-	Ohne Nachweis

3.2 Querbeanspruchung

	bezogene Anker	Einwirkung [kN]	Widerstand [kN]	Ausnutzung [%]	Status
Stahlversagen (o.H.)	1,2,3,4	19,445	85,600	22,7	√
Pryout	1,2,3,4	77,782	134,971	57,6	√
Betonkantenbruch (y+)	3,4	38,891	39,261	**99,1**	√
Stahlversagen Bew	-	-	-	-	Ohne Nachweis
Verbundversagen Bew	-	-	-	-	Ohne Nachweis

Stahlversagen ohne Hebelarm

$V_{Rd,s} = V_{Rk,s} \cdot k_2 / \gamma_{Ms}$ $\beta_{V,s} = V_{Sd} / V_{Rd,s}$

$V_{Rk,s}$ [kN]	k_2	γ_{Ms}	$V_{Rd,s}$ [kN]	V_{Sd} [kN]	$\beta_{V,s}$
107,0	1,0	1,25	85,600	19,445	0,227

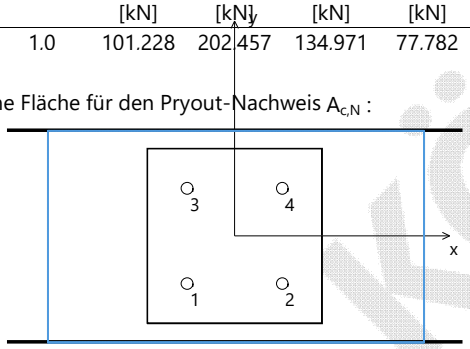
Pryout-Versagen (Betonausbruch auf der lastabgewandten Seite)

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,V,cp}$ $N^0_{Rk,c} = k_1 \cdot (f_{ck,cube})^{0.5} \cdot h_{ef}^{1.5}$ [N] $\psi_{A,N} = A_{c,N}/A^0_{c,N}$ $V_{Rk,cp}=k_3 \cdot N_{Rk,c}$ $V_{Rd,cp} = V_{Rk,cp}/\gamma_{Mc}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	$\psi_{s,N}$	$\psi_{re,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
121.753	284400	281961	1.009	0.824	1.0	177.0	531.0	265.5

$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,c}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V_{Sd} [kN]	k_1	k_3	γ_{Mc}	$\beta_{V,cp}$
0.0	0.0	1.0	1.0	1.0	101.228	202.457	134.971	77.782	8.5	2.0	1.5	0.576

Bezogene Fläche für den Pryout-Nachweis $A_{c,N}$:



Betonkantenbruch, Richtung y+

$V_{Rk,c}=V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $V^0_{Rk,c} = k_1 \cdot d_{nom}^{\alpha} \cdot l_f^{\beta} \cdot (f_{ck,cube})^{0.5} \cdot c_1^{1.5}$ [N] $\psi_{A,V} = A_{c,V}/A^0_{c,V}$ $V_{Rd,c} = V_{Rk,c}/\gamma_{Mc}$

$\alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\beta = 0.1 \cdot (d / c_1)^{0.2}$

h_{ef} [mm]	k_1	$f_{ck,cube}$ [N/mm²]	γ_{Mc}	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d_{nom} [mm]	l_f [mm]
177,0	1,6	37	1,5	110,0	-	0,127	0,072	24,184	1,000	22,0	177,0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V_{Sd} [kN]	$\beta_{V,c}$
84150	54450	1,545	1,000	1,313	0,0	1,000	1,200	58,891	39,261	38,891	0,991

Stahlversagen der Zusatzbewehrung

Nachweis ist nicht erforderlich, weil keine Zusatzbewehrung für Querlasten vorhanden ist.

Verbundversagen der Zusatzbewehrung

Nachweis ist nicht erforderlich, weil keine Zusatzbewehrung für Querlasten vorhanden ist.

3.3 Kombinierte Zug- und Querbeanspruchung

Interaktion ist nicht erforderlich.

Ankerbezogene Ausnutzung

A-Nr.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,ec}$	$\beta_{N,sp}$	$\beta_{N,cb}$	$\beta_{N,ecb}$	$\beta_{N,re}$	$\beta_{N,a}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{V,re}$	$\beta_{V,a}$	$\beta_{N,c,max,E}$	$\beta_{V,c,max,E}$	$\beta_{combi,c,E}$	$\beta_{combi,s,E}$
1	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,227	0,576	0,736	0,000	0,000	0,000	0,736	-	-
2	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,227	0,576	0,736	0,000	0,000	0,000	0,736	-	-
3	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,227	0,576	0,991	0,000	0,000	0,000	0,991	-	-
4	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,227	0,576	0,991	0,000	0,000	0,000	0,991	-	-

4. Verschiebung

Zugbeanspruchung: $N_{Sk}^{h} = N_{Sd}^{h} / 1{,}4$

Kurzzeit-Verschiebung: $\delta_N^{0} = (N_{Sk}^{h} / N_0) \cdot \delta_{N0}$

Langzeit-Verschiebung: $\delta_N^{\infty} = (N_{Sk}^{h} / N_0) \cdot \delta_{N\infty}$

N_{Sd}^{h} [kN]	N_0 [kN]	δ_{N0} [mm]	$\delta_{N\infty}$ [mm]	δ_N^{0} [mm]	δ_N^{∞} [mm]
0,000	35,0	0,7	1,8	**0,000**	**0,000**

Querbeanspruchung: $V_{Sk}^{h} = V_{Sd}^{h} / 1{,}4$

Kurzzeit-Verschiebung: $\delta_V^{0} = (V_{Sk}^{h} / V_0) \cdot \delta_{V0}$

Langzeit-Verschiebung: $\delta_V^{\infty} = (V_{Sk}^{h} / V_0) \cdot \delta_{V\infty}$

V_{Sd}^{h} [kN]	V_0 [kN]	δ_{V0} [mm]	$\delta_{V\infty}$ [mm]	δ_V^{0} [mm]	δ_V^{∞} [mm]
19,445	60,0	1,5	2,0	**0,347**	**0,463**

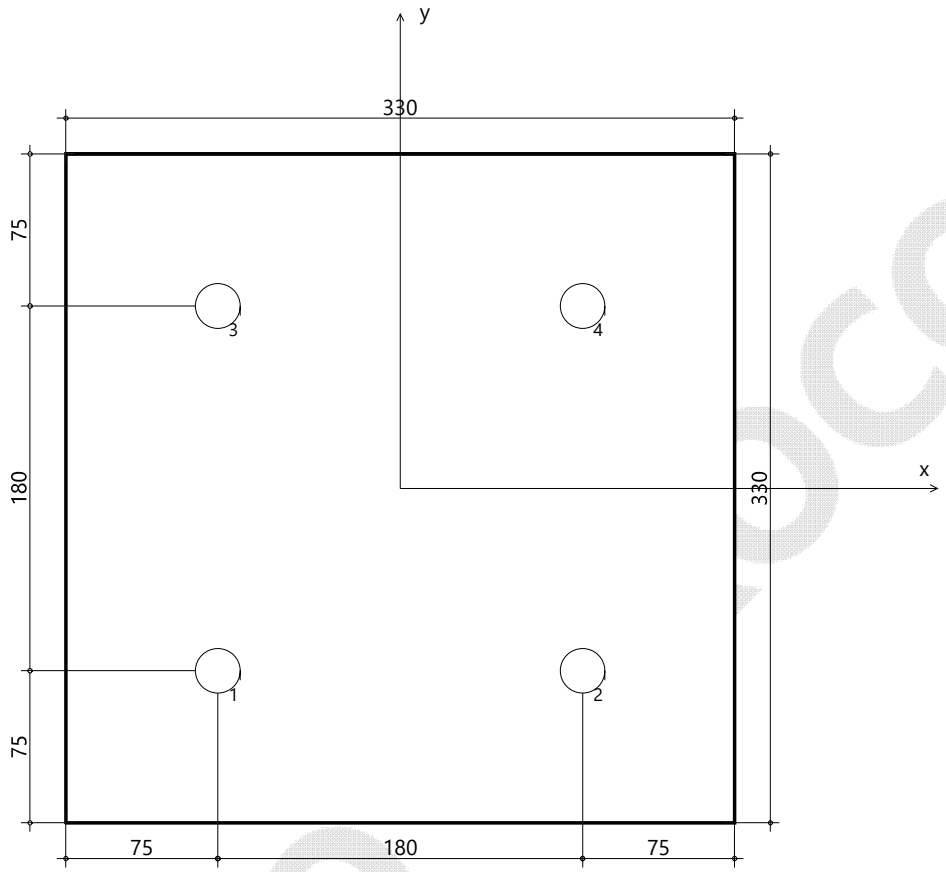
5. Bemerkungen

- Der Nachweis im Abschnitt 3 wurde wie folgt durchgeführt: Die von CET/TS gedeckten Fälle wurden nach CET/TS bemessen; Die über CET/TS hinausgehenden Fälle wurden auf CET/TS basierend bemessen.
- Bei der Befestigung mit biegestarrer Ankerplatte wird vorausgesetzt, dass die Ankerplatte ausreichend steif ist. Für den Nachweis der ausreichenden Ankerplattensteifigkeit in dieser Software wird die Steifigkeitsbedingung anhand der Veröffentlichung „Required Thickness of Flexurally Rigid Baseplate for Anchor Fastenings“ (fib Symposium 2017 Maastricht) verwendet.
- Bei der Befestigung mit elastischer Ankerplatte werden die Ankerzugkräfte mit Finite-Elemente-Methode unter Berücksichtigung der Verformungen von Ankerplatte, Ankern und Beton berechnet. Hintergrund zur Bemessung der elastischen Ankerplatten wird im Aufsatz "Bemessung von Befestigungen mit elastischen Ankerplatten unter Zug- und Biegebeanspruchung" beschrieben. Dieser Aufsatz wurde in "Stahlbau 88 (2019), Heft 8" und "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017" veröffentlicht.
  Die Ankerquerkräfte werden im Programm unter der Annahme der starren Ankerplatte berechnet. Dies ist besonders bei schmaler Ankerplatte mit dem Verhältnis Breit/Länge kleiner als 1/3 zu beachten.
- Der Nachweis für den Grenzzustand der Tragfähigkeit und die berechnete Verschiebung unter der Gebrauchslast gelten nur, wenn die bemessenen Anker gemäß ETA-Zulassung richtig installiert sind.
- Der Nachweis der örtlichen Einleitung der Ankerlasten in das als Verankerungsgrund dienende Betonbauteil ist nach CEN/TS 1992-4-1 Anhang A zu erbringen.

Die Tragfähigkeit der Verankerung ist: **nachgewiesen !**

Die Bemessung ist nur gültig für KÖCO-Kopfbolzen. Sie gilt nicht für Kopfbolzen, deren Festigkeitswerte nur den geringeren Anforderungen der Norm entsprechen.

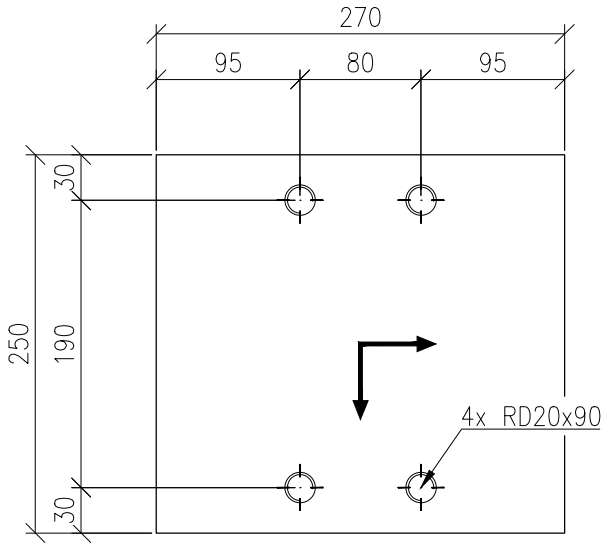
Ankerplatte, Stahlgüte: S 235 (St 37)

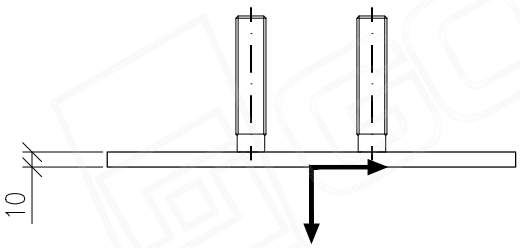


Material-Liste

Bezeichnung	Abmessungen [mm]	Anzahl	Gewicht [kg]
Stahlplatte	330 x 330 x 12	1	10,27
KÖCO-Kopfbolzen S235J2+C470 SD1 d22 x 175 Artikel-Nr. 017-0049-001	22 x 175	4	2,33

Die Bemessung ist nur gültig für KÖCO-Kopfbolzen. Sie gilt nicht für Kopfbolzen, deren Festigkeitswerte nur den geringeren Anforderungen der Norm entsprechen.





*EXC 2 nach EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78651	Stiftbolzenplatte	s. Versandliste	FL250x10	270	S235JR	
Stahlgewicht:	5,30 kg	Mantelfläche:	0,14 m²	Versandmaße:	(H x B x L)	250 x 100 x 270

Stückzahl	Schraube	DIN	Material	Gewicht	Bemerkung
4	RD20x90	13918	4.8	0,18	Werkstatt
Schraubengewicht:				0,72	
Beschichtung:	schwarz			Gesamtgewicht (kg):	6,02

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

Stiftbolzenplatte
250mm x 270mm t=10mm

gezeichnet	30.05.2012	[redacted]	
geprüft			
Maßstab	1:5	DWG-Name	[redacted]

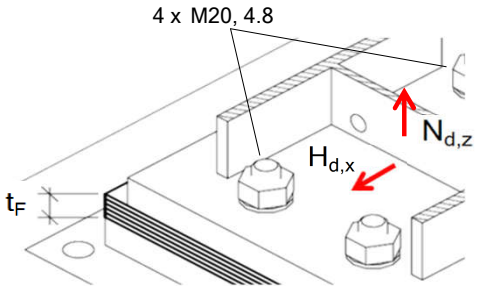
a	19.06.2015	Weisters / Wisotzky	Stückliste	Typ	Tabelle 2	Normteilnummer	Index
Zusatz		(27)	c19	P	G41	78651	a
Tabelle 0		Tabelle 1	Tabelle 2	Tabelle 3			

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FUSSPUNKTANSCHLUSS MIT FUTTERBLECHE (EN 1993-1-8:2010-12) 2021-06-24

Auftragsnummer: Stiftbolzenplatte Positionsnummer: 78651

Nationales Anwendungsdokument: Sonstige (EU)



Vertikalkraft $N_{d,z}$:	0,0 kN
max. Horizontalkraft $H_{d,x}$:	40,0 kN
Verbindungsmittel:	M20, 4.8
max. Futterblechstärke t_F:	12 mm
Gewinde in der Scherf.:	ja

1. Belastung je Schraube

$$M_{Ed} = H_{Ed} \cdot t_F = \frac{H_{d,x}}{4} \cdot t_F = \frac{40{,}0\ \text{kN}}{4} \cdot 1{,}2\ \text{cm} = 12{,}0\ \text{kNcm}$$

$$H_{Ed} = \frac{H_{d,x}}{4} = \frac{40{,}0\ \text{kN}}{4} = 10{,}0\ \text{kN}$$

2. Statische Werte

2.1 Flächenträgheitsmoment

$$I_y = \frac{r^4 \cdot \pi}{4} = \frac{(0{,}83\ \text{cm})^4 \cdot \pi}{4} = 0{,}3638\ \text{cm}^4$$

2.2 Schwerpunkt

$$z_u = \frac{4 \cdot r}{3 \cdot \pi} = \frac{4 \cdot 0{,}83\ \text{cm}}{3 \cdot \pi} = 0{,}350\ \text{cm}$$

3. Beanspruchung

3.1 Normalkraft

3.1.1 Zugkraft (Biegung aus Horizontalkraft)

$$F_{t,Hd} = \frac{M_{Ed}}{I_y} \cdot r \cdot A_S = \frac{12{,}0\ \text{kNcm}}{0{,}3638\ \text{cm}^4} \cdot 0{,}83\ \text{cm} \cdot \frac{(1{,}65\ \text{cm})^2 \cdot \pi}{4} = 58{,}2\ \text{kN}$$

3.1.2 Zugkraft (aus Vertikalkraft)

$$F_{t,Nd} = \frac{N_{d,z}}{4} = \frac{0{,}0\ \text{kN}}{4} = 0{,}0\ \text{kN}$$

3.1.3 Gesamte Zugkraft

$$F_{t,Ed} = F_{t,Hd} + F_{t,Nd} = 58{,}2\ \text{kN} + 0{,}0\ \text{kN} = 58{,}2\ \text{kN}$$

3.2 Abscherkraft

$$F_{v,Hd} = \frac{H_{d,x}}{4} = \frac{40{,}0\ \text{kN}}{4} = 10{,}0\ \text{kN}$$

4. Beanspruchbarkeit

4.1 Grenzabscherkraft

$$F_{v,Rd} = \frac{\alpha_v \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}6 \cdot 2{,}14\ cm^2 \cdot 42{,}0\ kN/cm^2}{1{,}35} = 39{,}9\ kN$$

4.2 Grenzzugkraft

$$F_{t,Rd} = \frac{k_2 \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}9 \cdot 2{,}14\ cm^2 \cdot 42{,}0\ kN/cm^2}{1{,}35} = 59{,}9\ kN$$

5. Nachweise

5.1 Abscheren

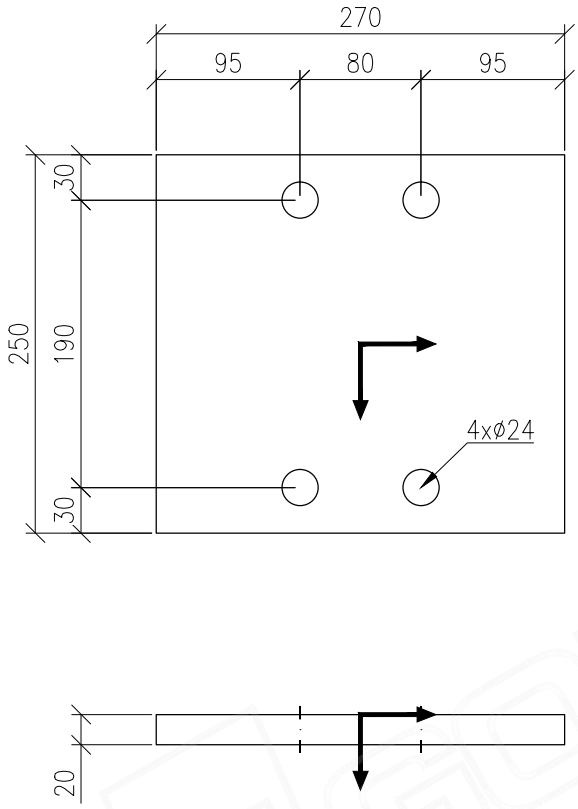
$F_{v,Hd} \overset{!}{\leq} F_{v,Rd}$ erfüllt: 10,0 kN < 39,9 kN

5.2 Zug

$F_{t,Ed} \overset{!}{\leq} F_{t,Rd}$ erfüllt: 58,2 kN < 59,9 kN

5.3 Zug und Abscheren

$$\frac{F_{v,Hd}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} \overset{!}{\leq} 1$$ erfüllt: 0,94 < 1



*EXC 2 nach EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78572	Fußplatte	s. Stückliste	FL250x20	270	S355J2	Ultraschallprüfung
Stahlgewicht:	10,60 kg	Mantelfläche:	0,15 m²	Versandmaße:	(H x B x L)	250 x 20 x 270
Beschichtung:	–				Gesamtgewicht (kg):	10,60

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

Fußplatte
250mm x 270mm t=20mm

gezeichnet 18.10.2012 [redacted]
geprüft
Maßstab 1:5
DWG-Name [redacted]

b	22.05.2015	Weisters / Wisotzky	Stückliste	
Zusatz		(27)	G10	c19
Tabelle 0		Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G42	78572	b

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Pos 78572 Teilflächenbelastung für Fußplatte 270 x 250 x 20 [mm]

BOCAD - TYP 5 | Fußplatte Pos. 78572, t = 20 mm, S355 | Stiftbolzenplatte Pos. 78651, t = 10 mm, S235

v20.12

EINGANGSWERTE

Baustoffe

Beton	C30/37	→	f_{cd} =	17 N/mm²
Stahl	B500	→	f_{yd} =	435 N/mm²

Belastung

F_{Ed} = 1600 kN

Lastausmitte

e = 0 mm

unterstützendes Bauteil

a = 500 mm
b = 2500 mm

Fuß- bzw. Zwischenplatte

a_0 = 270 mm
b_0 = 250 mm

Ankerplatte Pos. 78605

a_1 = 330 mm
b_1 = 330 mm
t_1 = 12 mm

Verteilungsfläche

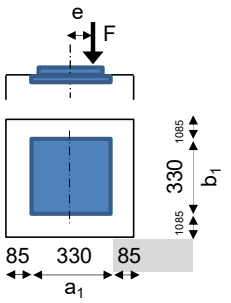
a_2 = 500 mm
b_2 = 469 mm

BEMESSUNG

Es werden eine gleichmäßige Pressung unterseitig der Fuß- bzw. Zwischenplatte sowie ein Lastausbreitungswinkel 1:2 innerhalb der Ankerplatte unterstellt.

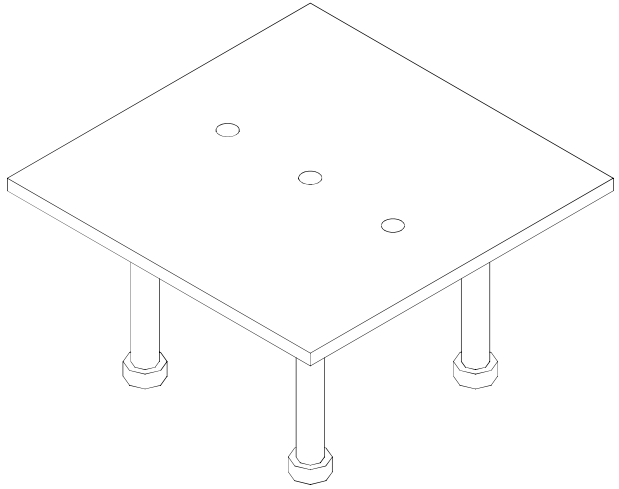
Unterseitig der Ankerplatte ergibt sich die Belastungsfläche zu:

a_1' = 318 mm
b_1' = 298 mm

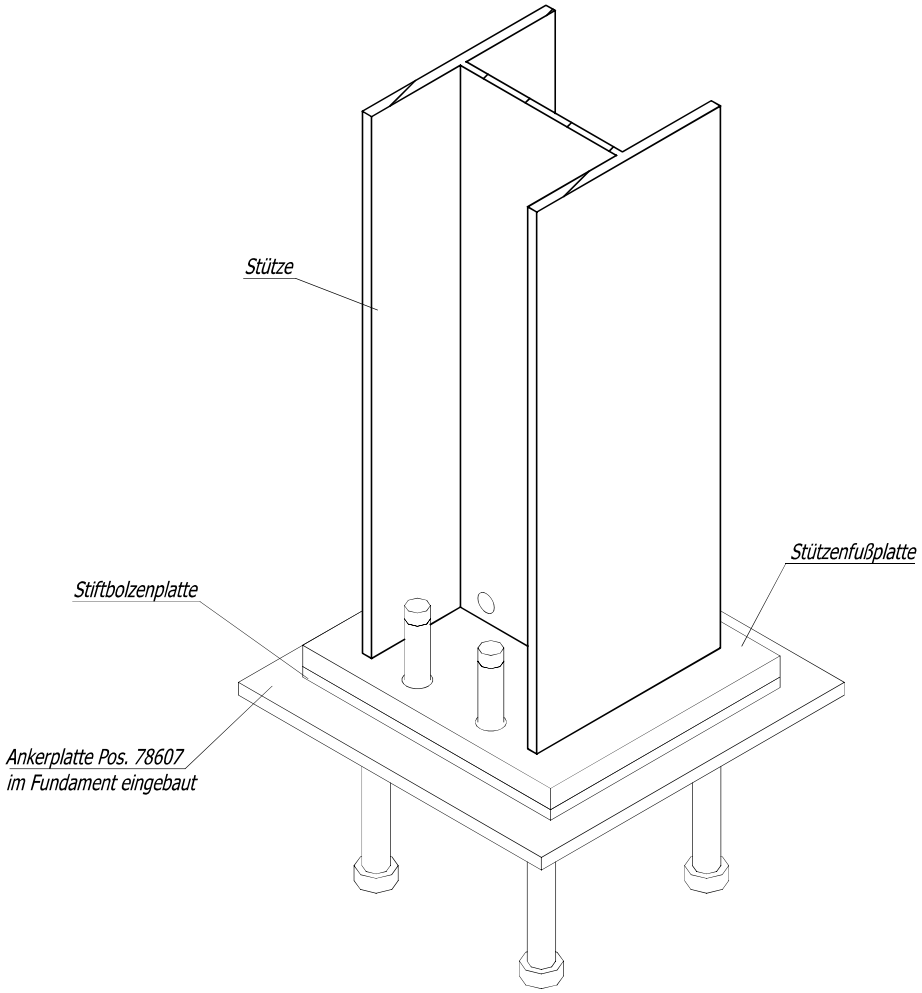


F_{Ed} = 1600 kN < 2533 kN = F_{Rdu}

A_{c0} = 94764 mm²
$a_{1,red}$ = 318 mm → a_2, b_2
$A_{c0,red}$ = 94764 mm²
$\sigma_{Ed,0}$ = = 16,9 N/mm² < 17,0 N/mm² = f_{cd} → kein Nachweis der Teilflächenbelastung erforderlich



Perspektive der Ankerplatte Pos. 78607



Einbausituation

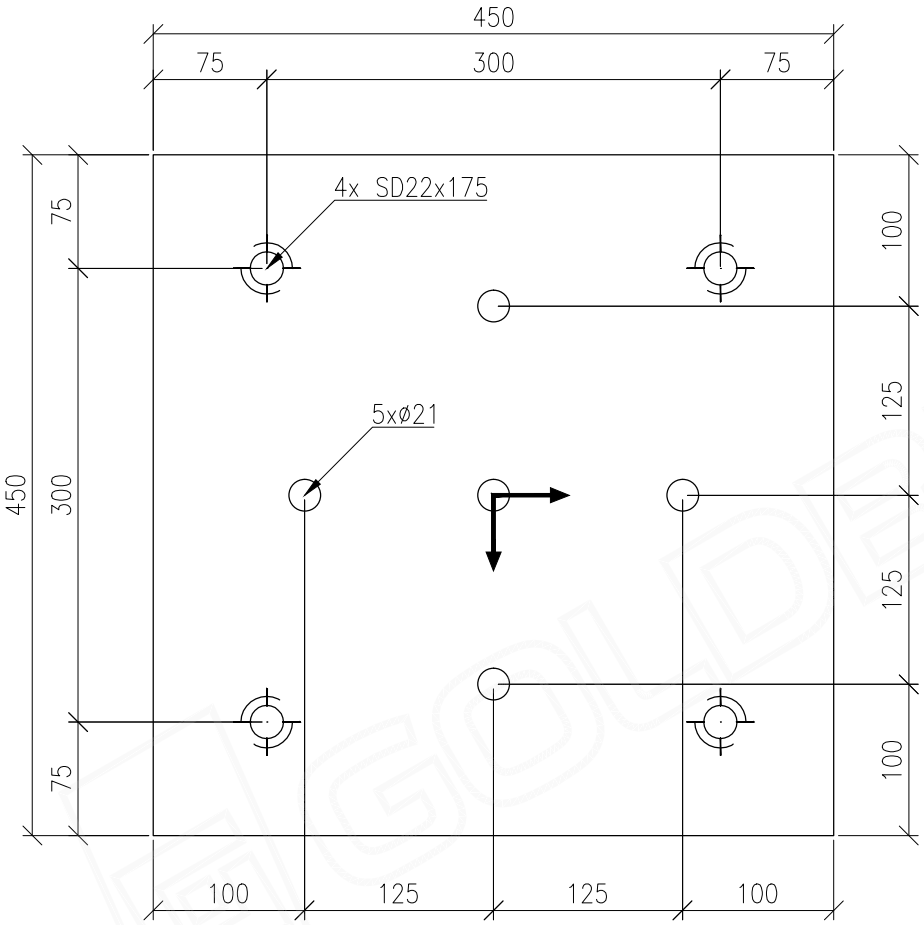
-	-	-	
		-	

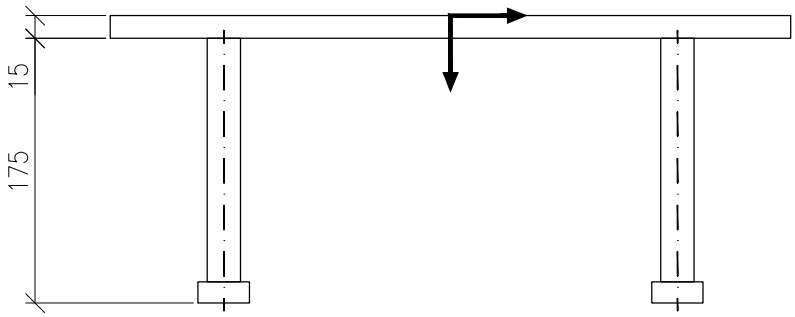
GOLDBECK
Ummelner Str. 4-6, 33649 Bielefeld, Telefon 0521/9488-0

Planinhalt:
Hauptposition 78607
AP - ANKERPLATTE
Einbausituation

Phase	Planer	Art
5	WP	FE
Ebene	Laufende-Nr.:	Index
XX	78607	-

bocad-3d, Rev. 24.1.82/8





*EXC 2 nach EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78607	Ankerplatte	s. Versandliste	BL15x450	450	S235JR	
Stahlgewicht:	23,80 kg	Mantelfläche:	0,42 m²	Versandmaße:	(H x B x L)	450 x 190 x 450

Stückzahl	Schraube	DIN	Material	Gewicht	Bemerkung
4	SD22x175	13918	S235J2G3+C450	0,6 kg	Werkstatt
Schraubengewicht:				2,4 kg	
Beschichtung:	schwarz			Gesamtgewicht (kg):	26,20

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

Ankerplatte
450mm x 450mm t=15mm

gezeichnet	30.05.2012
geprüft	
Maßstab	1:5
DWG-Name	

a 22.05.2015	Weisters / Wisotzky	a	
Zusatz	(27)		c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G40	78607	a

Blattgröße: 210mm x 297mm

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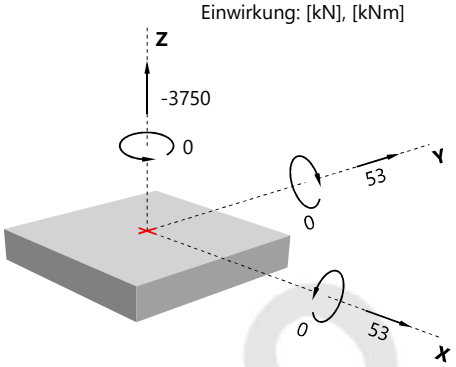
Köco StudCalc-Pro 3.1.0 - Erweiterter Ausdruck

Firma:	Goldbeck Südwest GmbH	E-Mail:	
Bearbeiter:		Tel.:	
Adresse:		Fax:	
Projekt:		Datum:	21.07.2022
Kommentar:	Q3 / 78607	Seite:	1 / 5

1. Eingabedaten

Untergrund:
- Gerissener Beton, Bauteildicke h=500mm, c_{nom}=55mm
  Festigkeitsklasse C30/37, $f_{ck,cube}$=37,0N/mm²
- Weite Flächenbewehrung
  Stababstand a≥150mm für alle Ø, oder a≥100mm für Ø≤10mm
- Keine Rand- und Aufhängebewehrung

Einwirkung: [kN], [kNm]



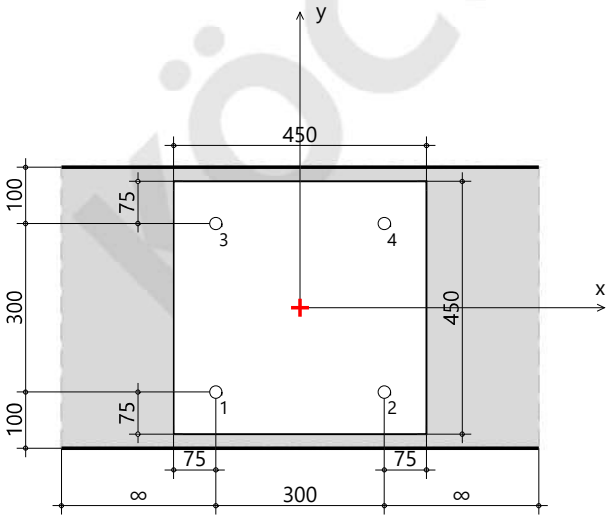
Zug Zusatzbewehrung
- Keine

Quer Zusatzbewehrung
- Keine

Einwirkung:
- Vorwiegend ruhende Designlasten

Montageart:
- Ankerplatte liegt ebenbündig im Beton
- Gute Betonverdichtung unter der Ankerplatte

Ankerplatte:
- S 235 (St 37), E=210000N/mm²
  f_{yk}=235N/mm², γ_s=1,5
- Angenommen: biegestarre Ankerplatte
- Verwendete Dicke: 12,0mm
- Erforderliche Dicke wurde nicht berechnet.
- Rechteckform
  Seitenlänge: 450 x 450 mm

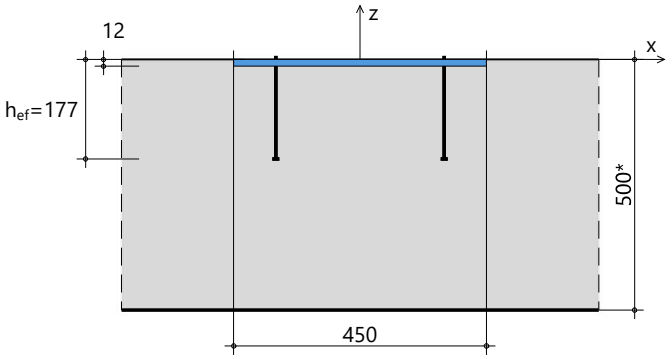


Profil:
- kein Profil
  Lasteinwirkungspunkt [mm]: [0, 0]

Ankerkoordinaten [mm]:

Nr.	x	y	Langloch L-x	Langloch L-y
1	-150,0	-150,0		
2	150,0	-150,0		
3	-150,0	150,0		
4	150,0	150,0		

Gewählte Anker:
- Köco SD1 d22 x 175
  Kopfbolzen
  Baustahl S235J2+C470
  Bemessung gemäß CEN/TS 1992-4-2
- Bewertung ETA-03/0039
  erteilt von DIBt, am 05.06.2018
- Verankerungstiefe h_{ef} = 177 mm

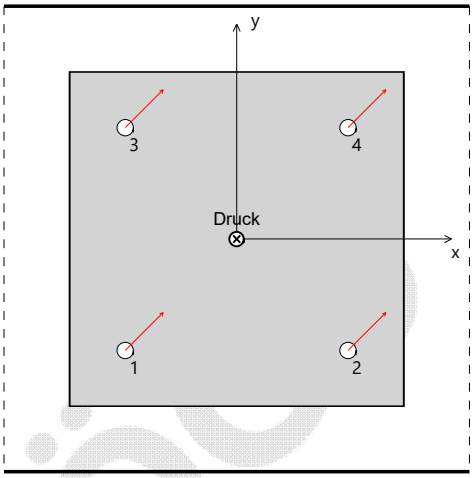


(* nicht maßstäblich gezeichnet)

2. Ankerschnittkräfte und Nachweis der Ankerplatten-Biegesteifigkeit

Ankerschnittkräfte [kN]

Anker-Nr.	Normalkraft	Querkraft	Querkraft x	Querkraft y
1	0,000	18,738	13,250	13,250
2	0,000	18,738	13,250	13,250
3	0,000	18,738	13,250	13,250
4	0,000	18,738	13,250	13,250



maximale Betonstauchung [‰]: 0,5612
maximale Betondruckspannung: 18,52 [N/mm²]
resultierende Zugkraft in (x/y=0,0/0,0): 0,000 [kN]
resultierende Druckkraft in (x/y=0,0/0,0): 3750,000 [kN]
Bemerkung: Der Randabstand ist nicht maßstäblich.

Nachweisbedingungen:

a) $\sigma \leq f_{yd}$

b) $N_{r,max} \approx N_{e,max}$

$N_{r,max}$: maximale Ankerzugkraft bei biegestarrer Ankerplatte

$N_{e,max}$: maximale Ankerzugkraft bei elastischer Ankerplatte

Der Nachweis für die Ankerplatten-Biegesteifigkeit wurde nicht durchgeführt.

3. Nachweis für den Grenzzustand der Tragfähigkeit gemäß CEN/TS 1992-4-2

3.1 Zugbeanspruchung

	bezogene Anker	Einwirkung [kN]	Widerstand [kN]	Ausnutzung [%]	Status
Stahlversagen	-	-	-	-	Ohne Nachweis
Herausziehen	-	-	-	-	Ohne Nachweis
Betonausbruch	-	-	-	-	Ohne Nachweis
Spaltversagen	-	-	-	-	Ohne Nachweis
Lokaler Betonausbruch	-	-	-	-	Ohne Nachweis
Stahlversagen Bew	-	-	-	-	Ohne Nachweis
Verbundversagen Bew	-	-	-	-	Ohne Nachweis

3.2 Querbeanspruchung

	bezogene Anker	Einwirkung [kN]	Widerstand [kN]	Ausnutzung [%]	Status
Stahlversagen (o.H.)	1,2,3,4	18,738	85,600	21,9	√
Pryout	1,2,3,4	74,953	194,486	38,5	√
Betonkantenbruch (y+)	3,4	37,477	37,680	**99,5**	√
Stahlversagen Bew	-	-	-	-	Ohne Nachweis
Verbundversagen Bew	-	-	-	-	Ohne Nachweis

Stahlversagen ohne Hebelarm

$V_{Rd,s} = V_{Rk,s} \cdot k_2 / \gamma_{Ms}$ $\beta_{V,s} = V_{Sd} / V_{Rd,s}$

$V_{Rk,s}$ [kN]	k_2	γ_{Ms}	$V_{Rd,s}$ [kN]	V_{Sd} [kN]	$\beta_{V,s}$
107,0	1,0	1,25	85,600	18,738	0,219

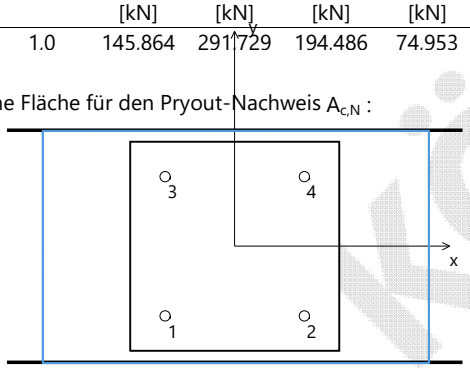
Pryout-Versagen (Betonausbruch auf der lastabgewandten Seite)

$N_{Rk,c}=N^0_{Rk,c} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,V,cp}$ $N^0_{Rk,c} = k_1 \cdot (f_{ck,cube})^{0.5} \cdot h_{ef}^{1.5}$ [N] $\psi_{A,N} = A_{c,N}/A^0_{c,N}$ $V_{Rk,cp}=k_3 \cdot N_{Rk,c}$ $V_{Rd,cp} = V_{Rk,cp}/\gamma_{Mc}$

$N^0_{Rk,c}$ [kN]	$A_{c,N}$ [mm²]	$A^0_{c,N}$ [mm²]	$\psi_{A,N}$	$\psi_{s,N}$	$\psi_{re,N}$	h_{ef} [mm]	$s_{cr,N}$ [mm]	$c_{cr,N}$ [mm]
121.753	415500	281961	1.474	0.813	1.0	177.0	531.0	265.5

$e_{V,cp,x}$ [mm]	$e_{V,cp,y}$ [mm]	$\psi_{ec,V,cp,x}$	$\psi_{ec,V,cp,y}$	$\psi_{ec,V,cp}$	$N_{Rk,c}$ [kN]	$V_{Rk,cp}$ [kN]	$V_{Rd,cp}$ [kN]	V_{Sd} [kN]	k_1	k_3	γ_{Mc}	$\beta_{V,cp}$
0.0	0.0	1.0	1.0	1.0	145.864	291.729	194.486	74.953	8.5	2.0	1.5	0.385

Bezogene Fläche für den Pryout-Nachweis $A_{c,N}$:



Betonkantenbruch, Richtung y+

$V_{Rk,c}=V^0_{Rk,c} \cdot \psi_{A,V} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{re,V}$ $V^0_{Rk,c} = k_1 \cdot d_{nom}^{\alpha} \cdot l_f^{\beta} \cdot (f_{ck,cube})^{0.5} \cdot c_1^{1.5}$ [N] $\psi_{A,V} = A_{c,V}/A^0_{c,V}$ $V_{Rd,c} = V_{Rk,c}/\gamma_{Mc}$

$\alpha = 0.1 \cdot (l_f / c_1)^{0.5}$ $\beta = 0.1 \cdot (d / c_1)^{0.2}$

h_{ef} [mm]	k_1	$f_{ck,cube}$ [N/mm²]	γ_{Mc}	c_1 [mm]	c'_1 [mm]	α	β	$V^0_{Rk,c}$ [kN]	$\psi_{s,V}$	d_{nom} [mm]	l_f [mm]
177,0	1,6	37	1,5	100,0	-	0,133	0,074	21,522	1,000	22,0	177,0

$A_{c,V}$ [mm²]	$A^0_{c,V}$ [mm²]	$\psi_{A,V}$	$\psi_{h,V}$	$\psi_{\alpha,V}$	e_V [mm]	$\psi_{ec,V}$	$\psi_{re,V}$	$V_{Rk,c}$ [kN]	$V_{Rd,c}$ [kN]	V_{Sd} [kN]	$\beta_{V,c}$
90000	45000	2,000	1,000	1,313	0,0	1,000	1,000	56,520	37,680	37,477	0,995

Stahlversagen der Zusatzbewehrung

Nachweis ist nicht erforderlich, weil keine Zusatzbewehrung für Querlasten vorhanden ist.

Verbundversagen der Zusatzbewehrung

Nachweis ist nicht erforderlich, weil keine Zusatzbewehrung für Querlasten vorhanden ist.

3.3 Kombinierte Zug- und Querbeanspruchung

Interaktion ist nicht erforderlich.

Ankerbezogene Ausnutzung

A-Nr.	$\beta_{N,s}$	$\beta_{N,p}$	$\beta_{N,c}$	$\beta_{N,ec}$	$\beta_{N,sp}$	$\beta_{N,cb}$	$\beta_{N,ecb}$	$\beta_{N,re}$	$\beta_{N,a}$	$\beta_{V,s}$	$\beta_{V,cp}$	$\beta_{V,c}$	$\beta_{V,re}$	$\beta_{V,a}$	$\beta_{N,c,max,E}$	$\beta_{V,c,max,E}$	$\beta_{combi,c,E}$	$\beta_{combi,s,E}$
1	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,219	0,385	0,587	0,000	0,000	0,000	0,587	-	-
2	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,219	0,385	0,587	0,000	0,000	0,000	0,587	-	-
3	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,219	0,385	0,995	0,000	0,000	0,000	0,995	-	-
4	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,000	0,219	0,385	0,995	0,000	0,000	0,000	0,995	-	-

4. Verschiebung

Zugbeanspruchung: $N_{Sk}^{h} = N_{Sd}^{h} / 1,4$

Kurzzeit-Verschiebung: $\delta_N^{0} = (N_{Sk}^{h} / N_0) \cdot \delta_{N0}$

Langzeit-Verschiebung: $\delta_N^{\infty} = (N_{Sk}^{h} / N_0) \cdot \delta_{N\infty}$

N_{Sd}^{h} [kN]	N_0 [kN]	δ_{N0} [mm]	$\delta_{N\infty}$ [mm]	δ_N^{0} [mm]	δ_N^{∞} [mm]
0,000	35,0	0,7	1,8	**0,000**	**0,000**

Querbeanspruchung: $V_{Sk}^{h} = V_{Sd}^{h} / 1,4$

Kurzzeit-Verschiebung: $\delta_V^{0} = (V_{Sk}^{h} / V_0) \cdot \delta_{V0}$

Langzeit-Verschiebung: $\delta_V^{\infty} = (V_{Sk}^{h} / V_0) \cdot \delta_{V\infty}$

V_{Sd}^{h} [kN]	V_0 [kN]	δ_{V0} [mm]	$\delta_{V\infty}$ [mm]	δ_V^{0} [mm]	δ_V^{∞} [mm]
18,738	60,0	1,5	2,0	**0,335**	**0,446**

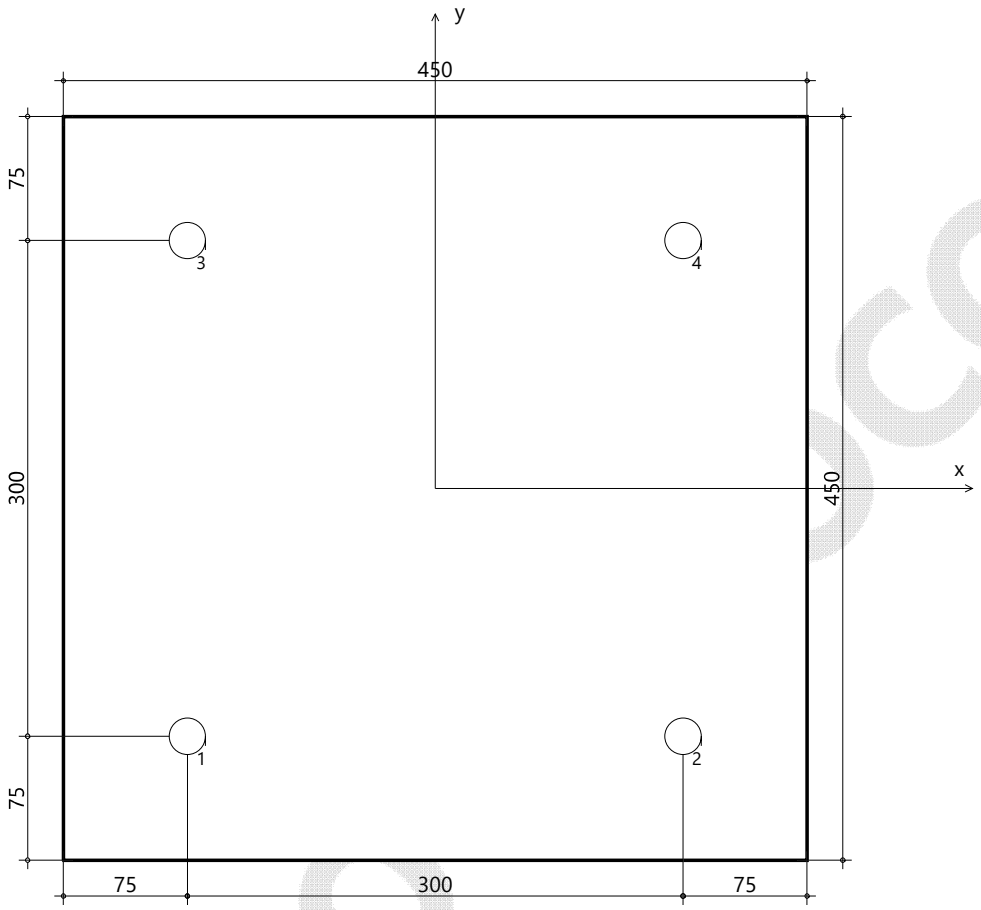
5. Bemerkungen

- Der Nachweis im Abschnitt 3 wurde wie folgt durchgeführt: Die von CET/TS gedeckten Fälle wurden nach CET/TS bemessen; Die über CET/TS hinausgehenden Fälle wurden auf CET/TS basierend bemessen.
- Bei der Befestigung mit biegestarrer Ankerplatte wird vorausgesetzt, dass die Ankerplatte ausreichend steif ist. Für den Nachweis der ausreichenden Ankerplattensteifigkeit in dieser Software wird die Steifigkeitsbedingung anhand der Veröffentlichung „Required Thickness of Flexurally Rigid Baseplate for Anchor Fastenings“ (fib Symposium 2017 Maastricht) verwendet.
- Bei der Befestigung mit elastischer Ankerplatte werden die Ankerzugkräfte mit Finite-Elemente-Methode unter Berücksichtigung der Verformungen von Ankerplatte, Ankern und Beton berechnet. Hintergrund zur Bemessung der elastischen Ankerplatten wird im Aufsatz "Bemessung von Befestigungen mit elastischen Ankerplatten unter Zug- und Biegebeanspruchung" beschrieben. Dieser Aufsatz wurde in "Stahlbau 88 (2019), Heft 8" und "5. Jahrestagung des Deutschen Ausschusses für Stahlbeton - DAfStb 2017" veröffentlicht.
  Die Ankerquerkräfte werden im Programm unter der Annahme der starren Ankerplatte berechnet. Dies ist besonders bei schmaler Ankerplatte mit dem Verhältnis Breit/Länge kleiner als 1/3 zu beachten.
- Der Nachweis für den Grenzzustand der Tragfähigkeit und die berechnete Verschiebung unter der Gebrauchslast gelten nur, wenn die bemessenen Anker gemäß ETA-Zulassung richtig installiert sind.
- Der Nachweis der örtlichen Einleitung der Ankerlasten in das als Verankerungsgrund dienende Betonbauteil ist nach CEN/TS 1992-4-1 Anhang A zu erbringen.

Die Tragfähigkeit der Verankerung ist: **nachgewiesen !**

Die Bemessung ist nur gültig für KÖCO-Kopfbolzen. Sie gilt nicht für Kopfbolzen, deren Festigkeitswerte nur den geringeren Anforderungen der Norm entsprechen.

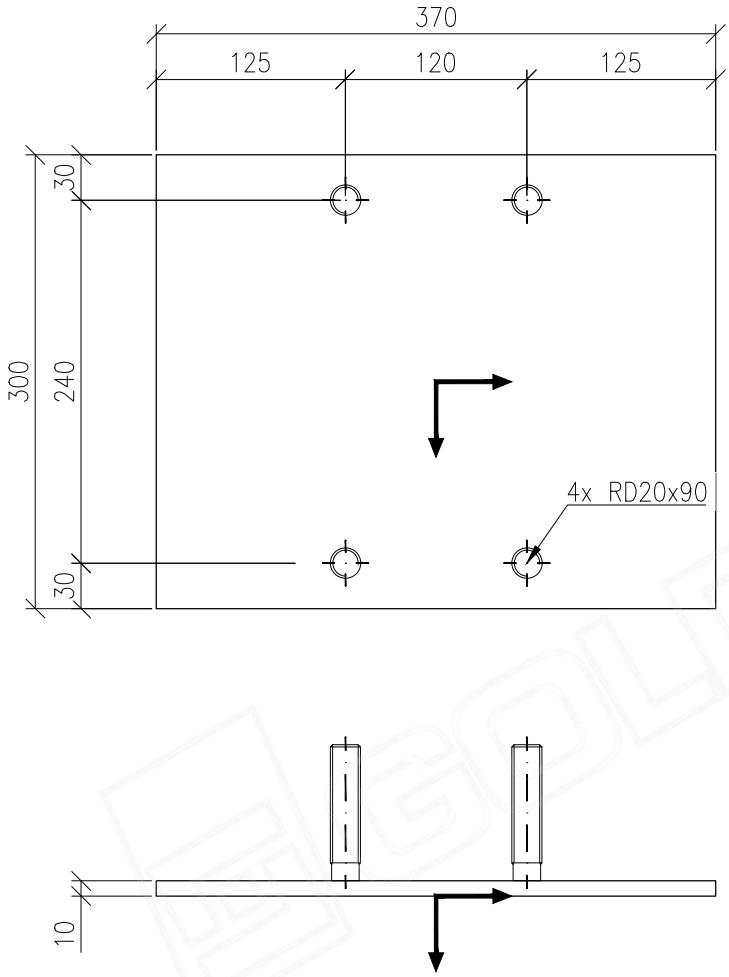
Ankerplatte, Stahlgüte: S 235 (St 37)



Material-Liste

Bezeichnung	Abmessungen [mm]	Anzahl	Gewicht [kg]
Stahlplatte	450 x 450 x 12	1	19,10
KÖCO-Kopfbolzen S235J2+C470 SD1 d22 x 175 Artikel-Nr. 017-0049-001	22 x 175	4	2,33

Die Bemessung ist nur gültig für KÖCO-Kopfbolzen. Sie gilt nicht für Kopfbolzen, deren Festigkeitswerte nur den geringeren Anforderungen der Norm entsprechen.



*EXC 2 nach EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78652	Stiftbolzenplatte	s. Versandliste	FL300x10	370	S235JR	
Stahlgewicht:	8,70 kg	Mantelfläche:	0,23 m²	Versandmaße:	(H x B x L)	300 x 100 x 370

Stückzahl	Schraube	DIN	Material	Gewicht	Bemerkung
4	RD20x90	13918	4.8	0,18	Werkstatt
Schraubengewicht:				0,72	
Beschichtung:	schwarz			Gesamtgewicht (kg):	9,42

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Stiftbolzenplatte
300mm x 370mm t=10mm

gezeichnet	30.05.2012		
geprüft			
Maßstab	1:5	DWG-Name	

a	19.06.2015	Weisters / Wisotzky	Stückliste	
Zusatz		(27)		c19
Tabelle 0		Tabelle 1	Tabelle 2	Tabelle 3

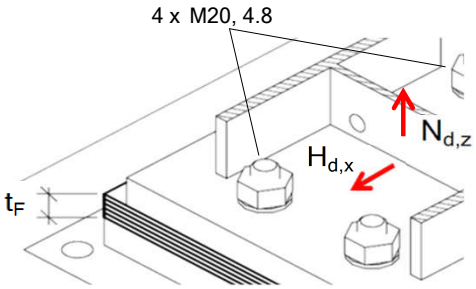
Typ	Tabelle 2	Normteilnummer	Index
P	G41	78652	a

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FUSSPUNKTANSCHLUSS MIT FUTTERBLECHE (EN 1993-1-8:2010-12) 2021-06-24

Auftragsnummer: Stiftbolzenplatte Positionsnummer: 78652

Nationales Anwendungsdokument: Sonstige (EU)



Vertikalkraft $N_{d,z}$:	0,0 kN
max. Horizontalkraft $H_{d,x}$:	40,0 kN
Verbindungsmittel:	M20, 4.8
max. Futterblechstärke t_F:	12 mm
Gewinde in der Scherf.:	ja

1. Belastung je Schraube

$$M_{Ed} = H_{Ed} \cdot t_F = \frac{H_{d,x}}{4} \cdot t_F = \frac{40{,}0\ \text{kN}}{4} \cdot 1{,}2\ \text{cm} = 12{,}0\ \text{kNcm}$$

$$H_{Ed} = \frac{H_{d,x}}{4} = \frac{40{,}0\ \text{kN}}{4} = 10{,}0\ \text{kN}$$

2. Statische Werte

2.1 Flächenträgheitsmoment

$$I_y = \frac{r^4 \cdot \pi}{4} = \frac{(0{,}83\ \text{cm})^4 \cdot \pi}{4} = 0{,}3638\ \text{cm}^4$$

2.2 Schwerpunkt

$$z_u = \frac{4 \cdot r}{3 \cdot \pi} = \frac{4 \cdot 0{,}83\ \text{cm}}{3 \cdot \pi} = 0{,}350\ \text{cm}$$

3. Beanspruchung

3.1 Normalkraft

3.1.1 Zugkraft (Biegung aus Horizontalkraft)

$$F_{t,Hd} = \frac{M_{Ed}}{I_y} \cdot r \cdot A_S = \frac{12{,}0\ \text{kNcm}}{0{,}3638\ \text{cm}^4} \cdot 0{,}83\ \text{cm} \cdot \frac{(1{,}65\ \text{cm})^2 \cdot \pi}{4} = 58{,}2\ \text{kN}$$

3.1.2 Zugkraft (aus Vertikalkraft)

$$F_{t,Nd} = \frac{N_{d,z}}{4} = \frac{0{,}0\ \text{kN}}{4} = 0{,}0\ \text{kN}$$

3.1.3 Gesamte Zugkraft

$$F_{t,Ed} = F_{t,Hd} + F_{t,Nd} = 58{,}2\ \text{kN} + 0{,}0\ \text{kN} = 58{,}2\ \text{kN}$$

3.2 Abscherkraft

$$F_{v,Hd} = \frac{H_{d,x}}{4} = \frac{40{,}0\ \text{kN}}{4} = 10{,}0\ \text{kN}$$

4. Beanspruchbarkeit

4.1 Grenzabscherkraft

$$F_{v,Rd} = \frac{\alpha_v \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}6 \cdot 2{,}14\ cm^2 \cdot 42{,}0\ kN/cm^2}{1{,}35} = 39{,}9\ kN$$

4.2 Grenzzugkraft

$$F_{t,Rd} = \frac{k_2 \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}9 \cdot 2{,}14\ cm^2 \cdot 42{,}0\ kN/cm^2}{1{,}35} = 59{,}9\ kN$$

5. Nachweise

5.1 Abscheren

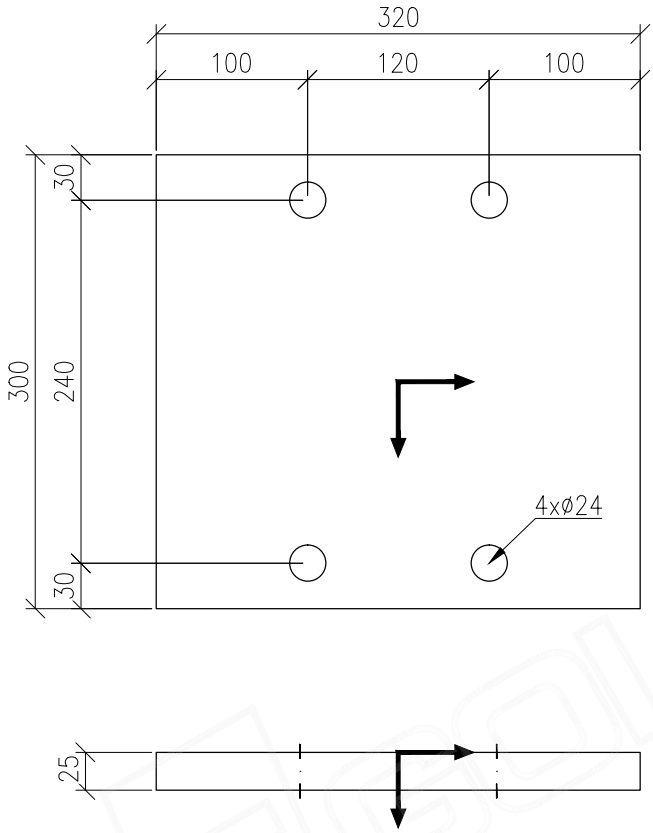
$F_{v,Hd} \overset{!}{\leq} F_{v,Rd}$ erfüllt: 10,0 kN < 39,9 kN

5.2 Zug

$F_{t,Ed} \overset{!}{\leq} F_{t,Rd}$ erfüllt: 58,2 kN < 59,9 kN

5.3 Zug und Abscheren

$$\frac{F_{v,Hd}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} \overset{!}{\leq} 1 \quad \text{erfüllt:} \quad 0{,}94 < 1$$



*EXC 2 nach EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78574	Fußplatte	s. Stückliste	FL300x25	320	S355J2	Ultraschallprüfung
Stahlgewicht:	18,84 kg	Mantelfläche:	0,21 m²	Versandmaße:	(H x B x L)	300 x 25 x 320
Beschichtung:	–				Gesamtgewicht (kg):	18,84

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Fußplatte
300mm x 320mm t=25mm

gezeichnet 18.10.2012 [redacted]
geprüft
Maßstab 1:5
DWG-Name [redacted]

b	22.05.2015	Weisters / Wisotzky	Stückliste	
Zusatz		(27)	G10	c19
		Tabelle 1	Tabelle 2	Tabelle 3
Tabelle 0				

Typ	Tabelle 2	Normteilnummer	Index
P	G42	78574	b

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Pos 78574 Teilflächenbelastung für Fußplatte 320 x 300 x 20 [mm]

BOCAD - TYP 7 | Fußplatte Pos. 78574, t = 25 mm, S355 | Stiftbolzenplatte Pos. 78652, t = 10 mm, S235

v20.12

EINGANGSWERTE

Baustoffe

Beton	C30/37	→	f_{cd} =	17	N/mm²
Stahl	B500	→	f_{yd} =	435	N/mm²

Belastung

F_{Ed} = 2625 kN

Lastausmitte

e = 0 mm

unterstützendes Bauteil

a	=	500	mm
b	=	2500	mm

Zwischenplatte Pos. 78652

a_0 =	370	mm
b_0 =	300	mm

Ankerplatte Pos. 78607

a_1 =	450	mm
b_1 =	450	mm
t_1 =	15	mm

Verteilungsfläche

a_2 =	500	mm
b_2 =	419	mm

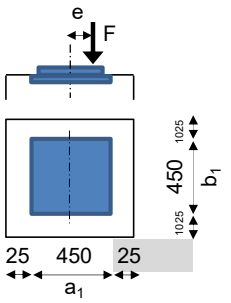
BEMESSUNG

Es werden eine gleichmäßige Pressung unterseitig der Fuß- bzw. Zwischenplatte sowie ein Lastausbreitungswinkel 1:2 innerhalb der Ankerplatte unterstellt.

Unterseitig der Ankerplatte ergibt sich die Belastungsfläche zu:

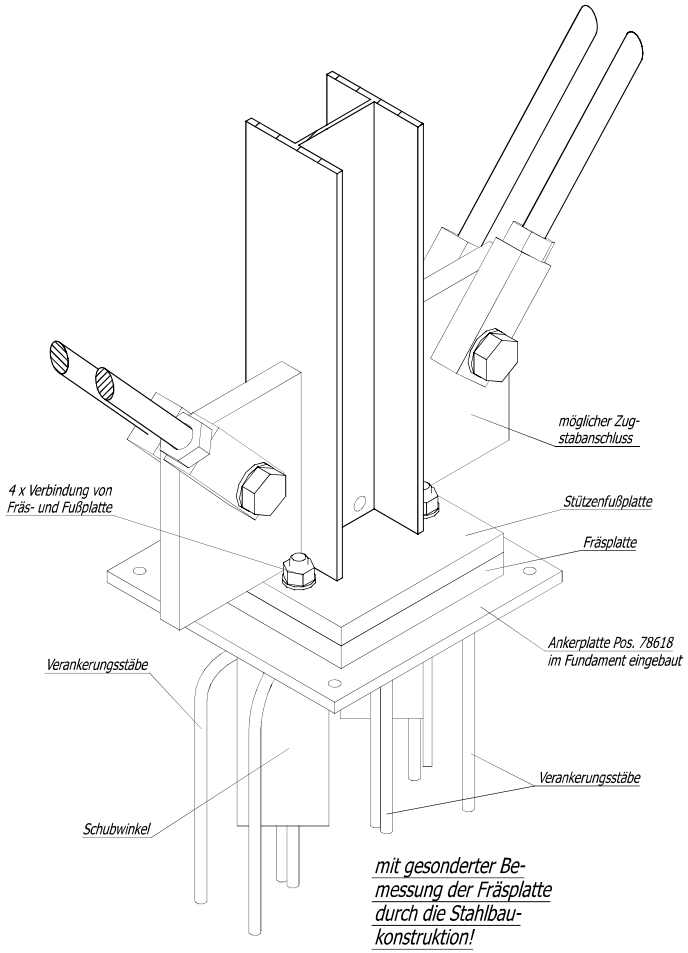
a_1' = 430 mm

b_1' = 360 mm



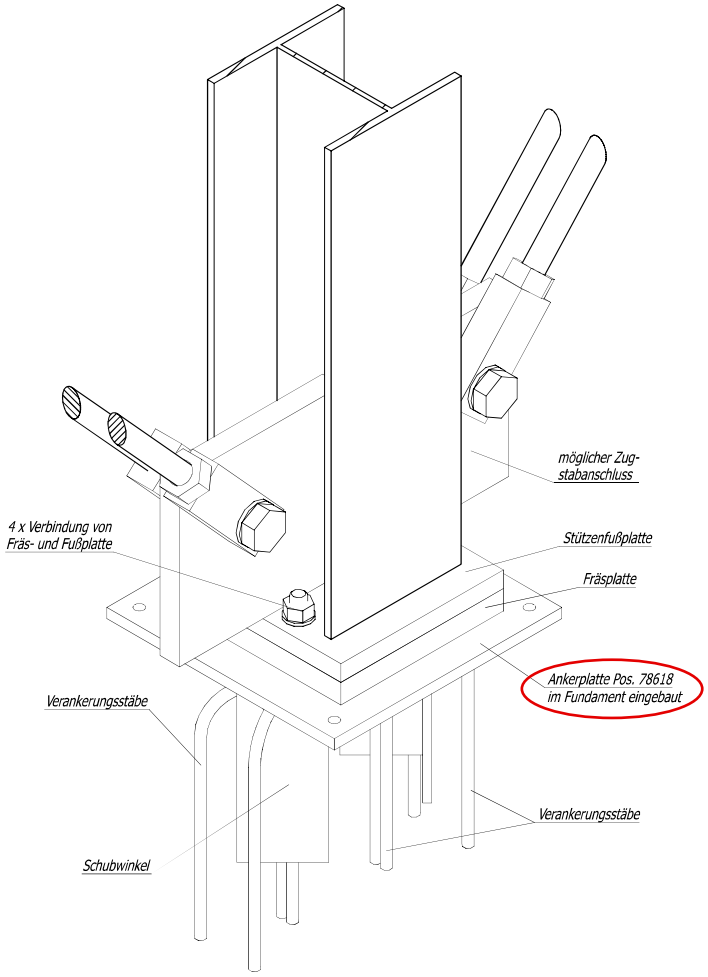
F_{Ed}	=	2625	kN	<	3060	kN	= F_{Rdu}
A_{c0}	=	154800	mm²				
$a_{1,red}$	=	430	mm	→	a_2, b_2		
$A_{c0,red}$	=	154800	mm²				
$\sigma_{Ed,0}$ =	=	17,0	N/mm²	<	17,0	N/mm²	= f_{cd}

→ kein Nachweis der Teilflächenbelastung erforderlich

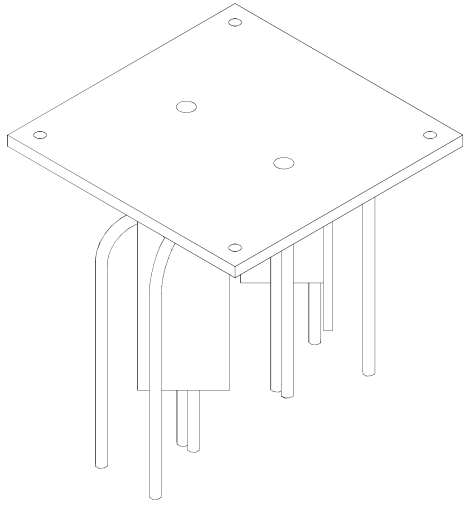


mit gesonderter Be-
messung der Fräsplatte
durch die Stahlbau-
konstruktion!

Einbausituation (gedrehte Stütze, Spezialfall)



Einbausituation (Regelfall)



Perspektive der Ankerplatte Pos. 78618

-	-	-	-
Index	Datum	Name	
		-	

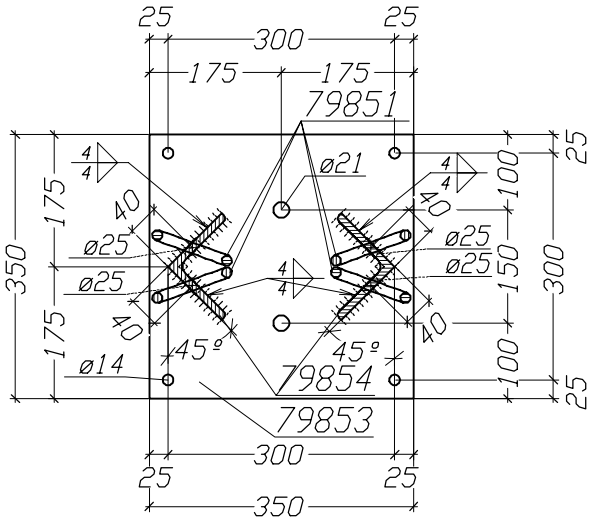
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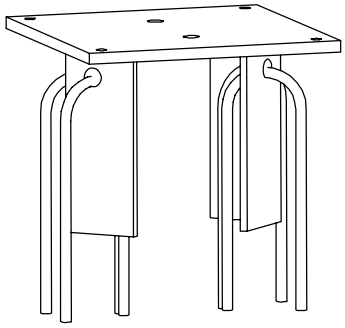
Planinhalt:
Hauptposition 78618
AP - ANKERPLATTE
Einbausituation

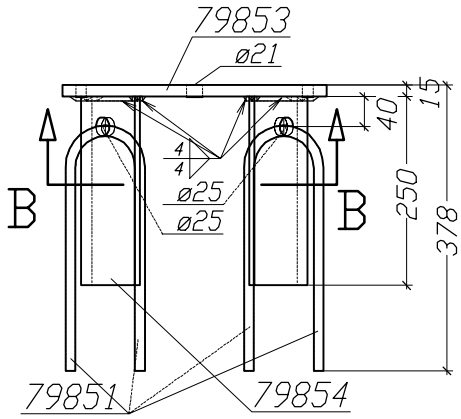
Phase	Planer	Art
5	WP	FE
Ebene	Laufende-Nr.:	Index
XX	78618	-

Vorlage-Stat_Rev_24.1.8078



Schnitt B-B





Ansicht A-A

*Werkstoff: siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAZM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung: siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 20mm = 5%

Nähte ohne Angabe: Kehlnaht, a=4mm

Position	Benennung	Profil	Stückzahl	Material	Länge	Bemerkung
78618	Ankerplatte H4	H4	s.Versandliste			
79851	Bewehrung	Betonstahl Ø14	4	B500B	691	für Transport fixiert
79853	PL	FL350*15	1	S355J2	350	
79854	WI	L100*10	2	S355J2	250	
Stahlgewicht:	25,32	Mantelfläche:	0,57	Versandmaße:	(H x B x L)	350*350*378
Beschichtung:	schwarz				Gesamtgewicht (kg):	25,32

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Ankerplatte Typ H4
350 x 350 mm
mit Schubwinkel und Bewehrungsschlaufe

gezeichnet	03.06.2015
geprüft	03.06.2015
Maßstab	1:10
DWG-Name	78618-78619.dwg

Zusatz			
	(27)		c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

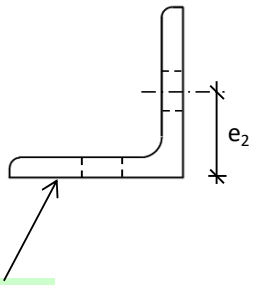
Typ	Tabelle 2	Normteilnummer	Index
P	G40	78618	–

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Nachweis der Verankerung H4 Pos. 78618

Winkelprofil:

Profil:	L 100 x 100 x 10		Fundamentbeton:	C 30/37
Material:	S 355		γ_C =	1,5
Länge l_L =	25	[cm]		
d_L =	25	[mm]	(Bohrung für Zugverankerung)	
e_2 =	40	[mm]		
a_L =	30	[cm]	(Abstand der L-Profile)	



Verschweißung mit umlaufender Kehlnaht: a = 4 [mm]

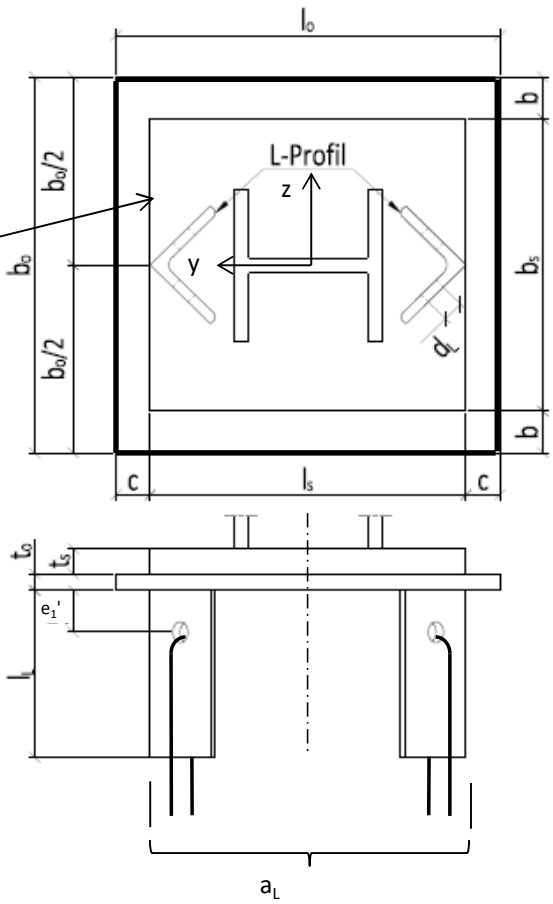
e_1' = 40 [mm]

Stiftbolzenplatte:

Material:	S 235	
l_S =	25	[cm]
b_S =	27	[cm]
t_S =	15	[mm]
a =	4	[mm]

Ankerplatte, oben:

Material:	S 355	
l_O =	35	[cm]
b_O =	35	[cm]
t_O =	1,5	[cm]



Fundamentabmessungen:

b =	50	[cm]
d =	60	[cm]

Betondeckung, Fundament:

c_{nom} = 35 [mm]

Bewehrungswahl Zugverankerung:

Ø =	14	[mm]	
l =	32,5	[cm]	Länge des Bewehrungsstabes
γ_S =	1,15		
f_y =	500	[N/mm²]	

max. gleichz. wirk. Bemessungskräfte, projektbezogene Last: s. akt. Tragwerkspl.

Z_{Ed} = 190,0 [kN]

$H_{y,Ed}$ = 150,0 [kN]

$H_{z,Ed}$ = 150,0 [kN]

vorh. A_S = 12,32 [cm²] erf. $A_S = Z_{Ed} / (f_y / \gamma_s)$ = 4,37 [cm²]

Biegerollendurchmesser: $D_{min} = 4 \cdot Ø$ = 56,0 [mm]

gewählt: D = 6 · Ø = 84,0 [mm]

Verbundbedingung: (◉) mäßig ( ) gut

f_{ck} = 30 [N/mm²]

α_C = 0,85

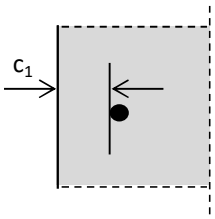
γ_C = 1,5

$f_{cd} = \alpha_C \cdot f_{ck} / \gamma_C$ = 17,0 [N/mm²]

$f_{ctk;0,05}$ = 2 [N/mm²]

η_1 = 0,7

η_2 = 1,0



Bemessungswert der Verbundfestigkeit: $f_{bd} = 2{,}25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}$ = 2,10 [N/mm²]

Grundwert der Verankerungslänge: $l_{b,rqd} = (Ø/4) \cdot (f_{yd}/f_{bd})$ = 725 [mm]

c_d = min {a/2; c_1} hier: $c_d = c_1$ = 120,8 [mm]

$$\alpha_2 = 1 - 0{,}15 \cdot \frac{c_d - Ø}{Ø}$$

α_1 = 1,0

α_2 = -0,1

$\alpha_1 \cdot \alpha_2 \cdot \alpha_5 \geq 0{,}7$ = 0,7

α_3 = 1,0

α_4 = 1,0

α_5 = 1,0

Mindestwert der Verankerungslänge:

$l_{b,min} = \max \{0{,}3 \cdot \alpha_1 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd};\ 10 \cdot Ø\}$ = 217 [mm]

Bemessungswert der Verankerungslänge $l_{bd} \geq l_{b,min}$:

$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd}$ = 507 [mm]

l_{bd} = 507 [mm]

Abminderung der Verankerungslänge um erf. A_s / vorh. $A_s \geq 0{,}3$:

erf. A_S / vorh A_S = 0,35

Abgeminderter Bemessungswert der Verankerungslänge l_{bd}:

l_{bd} = 18,0 [cm]

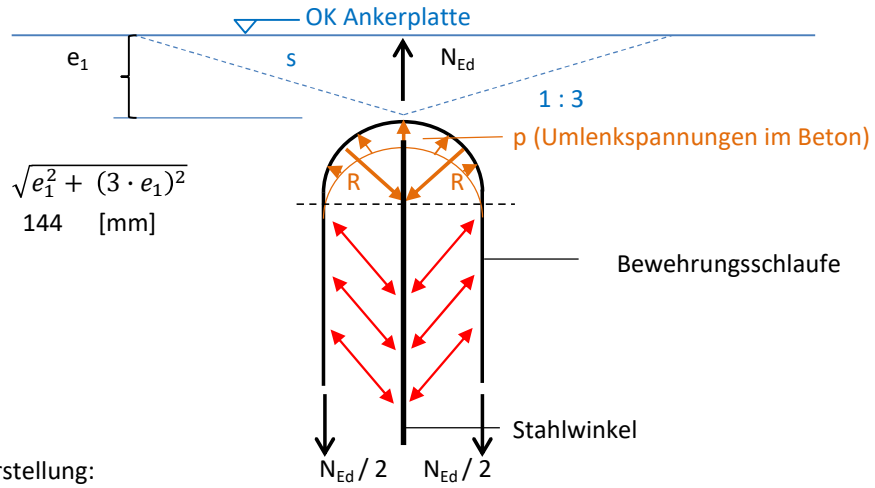
Vorhandene Verankerungslänge, l_{vorh}:

Nachweis: l_{vorh} = 32,5 > 18,0 = l_{bd} Ausnutzung: 0,55

Nachweis des Bewehrungsquerschnitts und Lasteinleitung:

γ_S =	1,15		$e_1 = e_1' + 0{,}5 \cdot d_L - 0{,}5 \cdot Ø$ =	45,5	[mm]	
f_y =	500	[N/mm²]	Ø =	14	[mm]	
$N_{Ed} = Z_{Ed} / 4$ =	47,5	[kN]	A =	1,54	[cm²]	
			d_{Br} =	84,0	[mm]	≥ $4 \cdot Ø$ = 56 mm

Modell:



$s = \sqrt{e_1^2 + (3 \cdot e_1)^2}$

s = 144 [mm]

Modellvorstellung:

Die Zugkraft des Winkelprofils wird über Reibung in den Beton eingeleitet und durch die Bewehrungsschlaufe verankert.

Resultierende Horizontalbeanspruchung R_H, die gegen den Stahlwinkel drückt.

$$R_H = \int_0^{90} \cos\alpha \cdot N_{Ed}/2 \quad = 24 \quad [kN]$$

Die Horizontalkomponente R_H wirkt auf die Fläche, die sich aus den Abmessungen des Biegeradius mal der Profilbreite ergibt.

$\sigma_N = R_H / A$ = 5,65 [N/mm²]

Fuge zwischen Stahlprofil und Beton: sehr glatt

$$V_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_N + \rho \cdot f_{yd} \cdot 1{,}2 \cdot \mu \cdot \sin\alpha + \cos\alpha) \quad \left[\begin{array}{l} \leq 0{,}5 \cdot \upsilon \cdot f_{cd} \\ \leq 0{,}1 \cdot f_{cd} \end{array} \right]$$

(6.25)

NCI zu 6.2.2 (6)

υ = 0,20

μ = 0,50

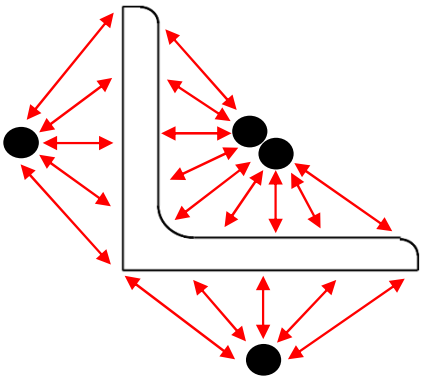
c = 0,00

Werte für sehr glatt

$\mu \cdot \sigma_N \leq 0{,}1 \cdot f_{cd}$ Reibungsanteil für sehr glatte Fugen nach (6.25)

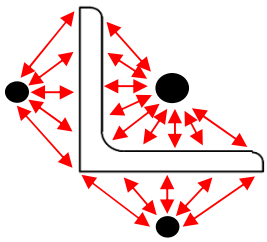
2,83 [N/mm²] ≤ 1,70 [N/mm²] → maßgebend: 1,70 [N/mm²]

Schematische Skizze Trajektorien vom Kraftverlauf aus dem Profil über Reibung durch den Beton in die Bewehrungsschlaufe.



8.9

Stabbündel:
Die aneinanderliegenden Bewehrungsstähle im Innenbereich des Winkelprofils dürfen als einen Stab (Ersatzstab) mit gleicher Querschnittsfläche angesehen werden.



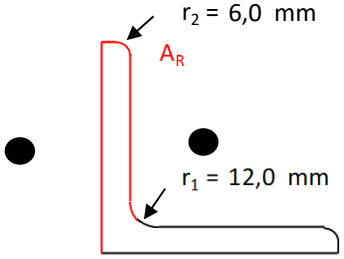
Die übertragbare Last einer Bewehrungsschlaufe wird duch die unten abgebildete rote Fläche A_R eingeleitet. Daraus ergibt sich eine übertragebare Beanspruchung von:

$A_R = l_L \cdot (a + a - t - r_1 / \sqrt{2} + 1/8 \cdot 2 \cdot \pi \cdot r_1 + 1/4 \cdot 2 \cdot \pi \cdot r_2 + t - 2 \cdot r_2 / \sqrt{2})$

A_R = 50470 [mm]

$V_{Rdi} = A_R \cdot \min \{\mu \cdot \sigma_N; 0{,}1 \cdot f_{cd}\}$

V_{Rdi} = 85,8 [kN]



Nachweis Reibungsübertragung:

N_{Ed} / V_{Rdi} = 0,55

Nachweis des Bewehrungsquerschnitts:

$(N_{Ed} / 2) / N_{Rd}$ = 0,35 mit $N_{Rd} = A_s \cdot f_{yd}$ = 66,9 [kN]

Z_{Ed} = 190 [kN]

$H_{y,Ed}$ = 150 [kN]

$H_{z,Ed}$ = 150 [kN]

Aus H_{yd} und H_{zd} Resultierende Last R_d:

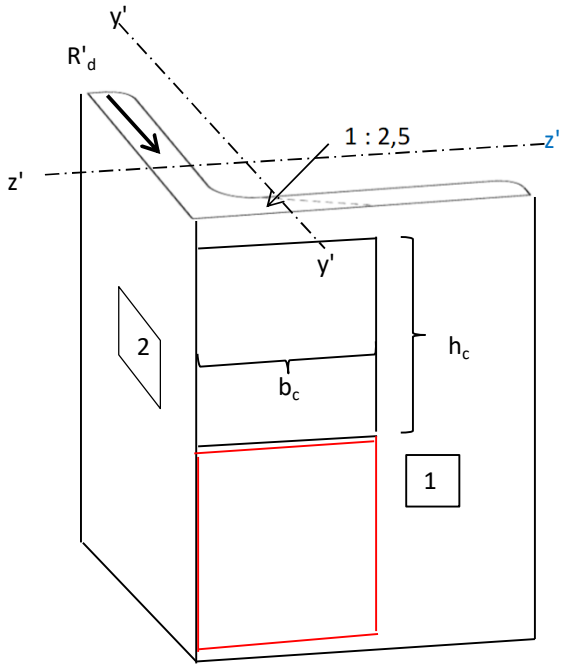
$$R_{Ed} = \sqrt{H_{y,Ed}^2 + H_{z,Ed}^2} = 212 \text{ [kN]}$$

Die Resultierende R_d wirkt hälftig auf beide Profile; als Maximalwert wirkt R'_d auf einen Schenkel des Winkels.

$R'_{Ed} = 0{,}5 \cdot R_{Ed}$ = 106 [kN]

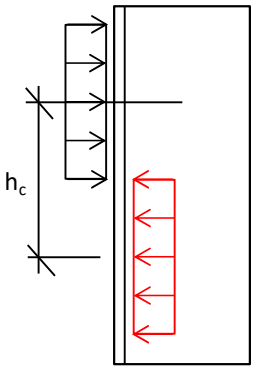
$f_{cd} = \alpha_C \cdot f_{ck} / \gamma_C$ = 17,0 [N/mm²]

$b_c = t + r_1 + 2{,}5 \cdot t$ = 47 [mm]



Teilflächenpressung:

erf. $h_c = R'_{Ed} / (3 \cdot f_{cd} \cdot b_c)$ = 44,2 [mm]



Der Winkel ist einbetoniert. Ein Verdrehen wird behindert, da die gleiche Beanspruchung aus Symetriegründen an der Fläche 2 wirkt und mit Fläche 1 im Gleichgewicht steht.

Für den Winkel resultiert daraus eine Biegebeanspruchung um die lokale Achsen y' - y' und z' - z'.

$M_{y',Ed} = M_{z',Ed} = R'_{Ed} \cdot h_c$ = 469,3 [kNcm]

Länge des Winkelprofils ok!

Vergrößerung um mindestens: nicht erforderlich!

Ausnutzung: $\eta = \dfrac{R'_{Ed}}{b_c \cdot l_L \cdot f_{cd}} =$ 0,53

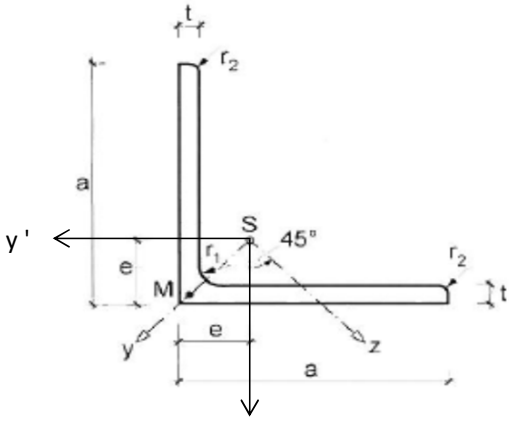
Auf der sicheren Seite liegend: H_{yd} und H_{zd} gleichzeitig in voller Größe wirkend.

Z_{Ed} =	190	[kN]	γ_{M0} =	1,0	f_y =	355	[N/mm²]
R_{Ed} =	212	[kN]	γ_{M1} =	1,1	f_u =	490	[N/mm²]
$M_{y',Ed} = M_{z',Ed}$ =	469,3	[kNcm]	γ_{M2} =	1,25			

Profilkennwerte:

A =	19,15	[cm²]
a =	100	[mm]
e =	2,82	[cm]
t =	10	[mm]
W_y =	39,65	[cm³]
W_z =	18,29	[cm³]
r_1 =	12,0	[mm]
$M_{pl,y,Rd}$ =	2234,0	[kNcm]
$M_{pl,z,Rd}$ =	1148,0	[kNcm]

siehe S. H4-17



$V_{pl,y,Rd}$ =	277,5	[kN]	$V_{pl,y',Rd}$ =	196,2	[kN]
$V_{pl,z,Rd}$ =	277,5	[kN]	$V_{pl,z',Rd}$ =	196,2	[kN]

$A_F = 0{,}5 \cdot A =$ 9,6 [cm²] $A_{net} =$ 14,2 [cm²]

Je nach dem, ob die Horizontalbeanspruchungen in + und/oder - der Hauptachsen wirken, addieren sich die Biegebeanspruchungen aus der Betonpressung zur Resultierenden Biegebenaspruchung in Richtung der Hauptachse y oder der Hauptachse z; positiv, oder negativ. Die Biegebeanspruchung ist unabhängig von der Zugbeanspruchung wird deshalb ungünstigst berücksichtigt.

Die Zugkraft Z_d verteilt sich gleichmäßig auf alle vier Bewehrungsnadeln.

$Z_{Ed}'' = 1/4 \cdot Z_{Ed} =$ 47,5 [kN]

Die Bewehrungsnadel (Rückhängebewehrung für die Zugkraft) ist nicht in der Schwerachse des Winkels angeordnet. Daraus resultiert ein Versatz e' zur Schwerachse.

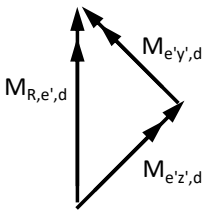
$e' = e - e_2 =$ 1,18 [cm]

Berechnung des Versatzmomentes $M_{e',d}$ aus der Exzentrizität e'. Das Moment wirkt um beide lokalen Achsen y' und z'.

$M_{e',Ed} = M_{e'y',Ed} = M_{e'z',Ed} = e' \cdot Z_{Ed'}' =$ 56,0 [kNcm]

Das resultierende Versatzmoment $M_{R,e',d}$ wirkt um die globale z-Achse.

$M_{R,e',d} =$ 79,1 [kNcm]

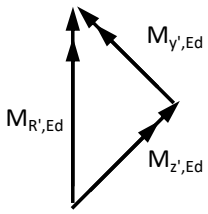


Zusammenfassung der Einwirkungen:
Auf der sicheren Seite liegend wirken die Horizontalbeanspruchungen gleichzeitig in voller Größe. Je nach dem, ob in positiver oder negativer Richtung der Hauptachsen, wirkt die Resultierende R_{Ed} hälftig auf einen Schenkel der L-Profile. Die daraus resultierende Biegung der Winkel $M_{y',Ed}$ und $M_{z',Ed}$ wirkt in der vektoriellen Addition entweder um die gloabe y- oder globale z-Achse (positiv oder negativ).

$M_{y',Ed} = M_{z',Ed} =$ 469,3 [kNcm] (resultierend aus der Betonpressung)
$M_{R,e',Ed} =$ 79,1 [kNcm]

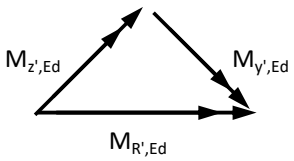
Aus der Betonpressung hervorgerufene, gleichzeitig wirkende Momentenbeanspruchung $M_{y'd}$ und $M_{z'd}$ wirken gleichzeitig. Es sind zwei Fälle zu unterscheiden:

Fall a)



$M_{R'd}$ als Resultierende um die globael z-Achse

Fall b)



$M_{R'd}$ als Resultierende um die globael y-Achse

Für beide Fälle gilt: $|M_{R',Ed}| = \sqrt{M_{z',Ed}^2 + M_{y',Ed}^2}$ = ± 664 [kNcm]

Fall a)

Z'_d = 95 [kN]

R'_d = 106 [kN]

$M_{z,Ed} = M_{R,e',Ed} + M_{R',Ed}$ = 743 [kNcm]

Fall b)

Z'_d = 95 [kN]

R'_d = 106 [kN]

$M_{z,Ed} = M_{R,e',Ed}$ = 79,1 [kNcm]

$M_{y,Ed} = M_{R',Ed}$ = 664 [kNcm]

Nachweis auf Biegung, Normalkraft und Querkraft:

$$\tau_{Rd} = \frac{f_{yk}}{\sqrt{3} \cdot \gamma_{M0}} = 20{,}5 \text{ [kN/cm²]}$$

$\sigma_{x,Rd} = f_y / \gamma_{M0}$ = 35,5 [kN/cm²]

$N_{pl,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 679,8 [kN]

$N_{u,Rd} = (0{,}9 \cdot A_{net} \cdot f_u) / \gamma_{M2}$ = 499,2 [kN]

$V_{pl,y,Rd} = V_{pl,z,Rd}$ = 277,5 [kN]

$V_{Ed} / V_{pl,Rd}$ = 0,38 < 0,5 Abminderung kann vernachlässigt werden!

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2 = 0{,}00 \qquad (6.29)$$

Abminderung der Streckgrenze für die Momententragfähigkeit durch die Querkraft:

$f'_y = (1 - \rho) \cdot f_y$ = 355 [N/mm²]

$M_{pl,y,Rd}$ = 2234,0 [kNcm]

$M_{pl,z,Rd}$ = 1148,0 [kNcm]

(berechnet mit abgeminderter Streckgrenze infolge Querkraft)

$n = N_{Ed} / \min\{N_{u,Rd}; N_{pl,Rd}\}$ = 0,19 mit: $N_{Ed} = Z'_d$, gem. vorheriger Seite (Fall a bzw. Fall b)

$N_{u,Rd}$ und $N_{pl,Rd}$ siehe oben auf dieser Seite

$$M_{N,Rd} = M_{pl,Rd} \cdot \left[1 - \left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2\right]$$ Interaktion Biegung, Querkraft, Normalkraft (6.32)

$M_{N,y,Rd}$ = 2153,1 [kNcm]

$M_{N,z,Rd}$ = 1106,4 [kNcm]

berechnet mit Gl. (6.32) und $M_{pl,y,Rd}$ bzw. $M_{pl,z,Rd}$ mit abgeminderter Streckgrenze

Nachweis (2-achsige Biegung, Querkraft, Normalkraft)

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1$$

mit: $\alpha = 2$

$\beta = 5 \cdot n \geq 1$ (6.41)

Fall a)

0,67 ≤ 1

Fall b)

mit: $\beta =$ 1,00

0,10 + 0,07 = 0,17 ≤ 1

Die Zugbeanspruchung wird über Reibung vom Winkelprofil in den Beton eingeleitet.
Auf der sicheren Seite liegend führe ich hier zusätzlich ein Nachweis des Winkelprofils auf Lochleibung. Als Modellvorstellung gehe ich hier davon aus, dass die eingehangene Bewehrungsschlaufe das Winkelprofil auf Lochleibung beansprucht.

Das Profil ist vollständig im Beton eingebettet. Es kann sich im Beton nicht verformen oder auf- bzw. ausreißen. Der Nachweis liegt somit auf der sicheren Seite.

			d = Ø =	14	[mm]
			$d_0 = d_L$ =	25	[mm]
			t =	10	[mm]
Randabstand $e_1 \geq 1{,}2\ d_0$ =	30	[mm]	$e_1 = l_L - e_1'$ =	285	[mm]
Randabstand $e_2 \geq 1{,}2\ d_0$ =	30	[mm]	e_2 =	40	[mm]
			f_u =	490	[N/mm²]
hier für den Bewehrungsstahl (DIN 488): $f_{ub} = 1{,}05 \cdot f_{yb}$:			f_{ub} =	525	[N/mm²]
			k_1 =	2,5	
			γ_{M2} =	1,25	

für am Rand liegende Schraube: $\alpha_d = \frac{e_1}{3\ d_0} =$ 3,80

1,00

f_{ub} / f_u = 1,07

α_b = MIN (α_d; f_{ub}/f_u; 1,0) = 1,00 [kN]

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \quad 137{,}2 \text{ [kN]}$$

Mit $F_{v,Ed} = Z'_{,Ed}$ = 47,5 [kN]

Nachweis: $F_{v,Ed} / F_{b,Rd}$ = 0,35 < 1

$f_{y,min}$ = 355

Für den Nachweis der Schweißnaht wird auf der sicheren Seite liegend die maximal und gleichzeitig auftretenden Bemessunslasten zu Grunde gelegt.
Die Horizontalkräfte wirken in maximaler Größe als Resultierende hälftig auf einen Schenkel des L-Profils.

Material Winkel:	S 355	f_u =	490	[N/mm²]	β_W = 0,9
Material Ankerplatte:	S 355	f_u =	490	[N/mm²]	β_W = 0,9

Umlaufende Kehlnaht: gew. a = 4 [mm]

Auf einen Flansch (Schenkel) angesetzte Schweißnahtlänge l_W:
l_W = Flanschlänge - t - r_1 = 78,0 [mm]

Schweißnahtfläche A_W pro Schenkel:
$A_W = 2 \cdot a \cdot l_W$ = 624 [mm²]

Flächenmoment zweiten Grades J_W der Schweißnaht pro Schenkel:
$J_W = 2 \cdot a \cdot l^3_W \cdot 1/12$ = 31,6 [cm^4]

Widerstandsmoment der Schweißnaht pro Schenkel:
$W_w = J_W / (l_W/2)$ = 8,1 [cm³]

Korrelationsbeiwert: β_W = 0,9 γ_{M2} = 1,25

Zugfestigkeit des schwächeren Materials: f_u = 490 [N/mm²]

Einwirkende Bemessungslasten auf das Profil, auf einen Schenkel:

Normalkraft: siehe Seite H4-8 $N_{Ed} = Z''_{Ed}$ = 47,5 [kN]

Querkraft: siehe Seite H4-7 $V_{Ed} = R'_{Ed}$ = 106,1 [kN]

Beanspruchungen:

$\sigma_{x,Ed} = N_{Ed} / A_W + M_{e',d} / W_w$ = 14,5 [kN/cm²] $M_{e',d}$ siehe Seite H4-8

$\tau_{IIEd} = V_{Ed} / A_w$ = 17,0 [kN/cm²]

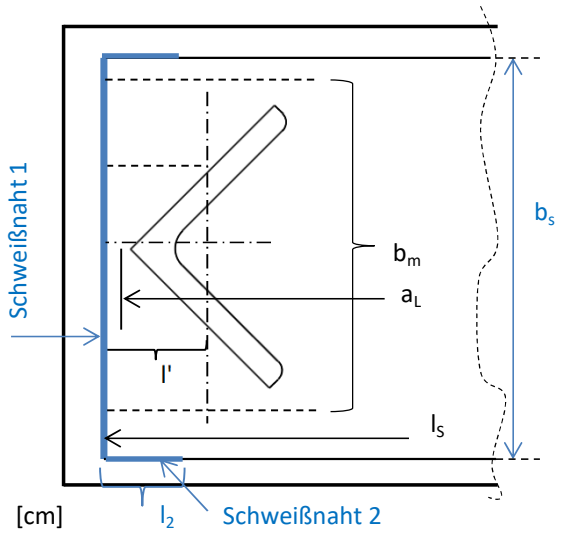
Nachweis:

$\sigma_V = (\sigma^2_x + 3\,\tau^2_{II})^{0,5}$ = 32,8 [kN/cm²] ≤ 43,6 [kN/cm²] $= f_u / (\beta_W \cdot \gamma_{M2})$

η = 0,75

$\sigma_{x,Ed}$ = 14,5 [kN/cm²] ≤ 35,3 [kN/cm²] $= 0,9 \cdot f_u / \gamma_{M2}$ η = 0,41

t_O = 1,5 [cm]
f_y = 355 [N/mm²]
γ_{M0} = 1,0
l_s = 25 [cm]
a_L = 30 [cm]
b_s = 27 [cm]
l_2 = 6,5 [cm]
a = 5 [mm]



$l' = 0,5 \cdot (l_s - a_L) + e / \sqrt{2}$ = -0,50 [cm]

Pro Winkel wirkt die halbe einwirkende Bemessungszugkraft:

Z'_{Ed} = 95 [kN]

Als Versatz l' zur Einleitung der Zugbeanspruchung wird der Abstand von der Schweißnaht 1 zum Schwerpunkt des L-Profils angesetzt.
Daraus resultiert eine Momentenbeanspruchung M_{Ed}.

$M_{Ed} = I\,Z'_{Ed} \cdot l'\,I$ = 47,9 [kNcm]

M_{pl} der Ankerplatte: $M_{pl} = t^2_O / 4 \cdot (f_y / \gamma_{M0})$ = 20,0 [kNcm/cm]

mittragende Breite b_m (mit a = Breite des Schnekels vom L-Profil):

$b_m = 2 \cdot a / \sqrt{2} + 2 \cdot 2,5 \cdot t_O$ = 21,6 [cm]

$M_{pl,Rd} = M_{pl} \cdot b_m$ = 432,2 [kNcm]

Nachweis:

$M_{Ed} / M_{pl,Rd}$ = 0,11 ≤ 1

Einwirkungen: Z_{Ed} = 190 [kN]
$H_{y,Ed}$ = 150 [kN]
$H_{z,Ed}$ = 150 [kN]

Material Ankerplatte: S 355 f_u = 490 [N/mm²] β_W = 0,9
Material Stiftbolzenpl.: S 235 f_u = 360 [N/mm²] β_W = 0,8

Korrelationsbweiwert: β_W = 0,8 γ_{M2} = 1,25

Zugfestigkeit des schwächeren Materials: f_u = 360 [N/mm²]

Die Beanspruchung Z_{Ed} und $H_{z,Ed}$ wird von der Schweißnaht 1 und die Beanspruchung $H_{y,Ed}$ von der Schweißnaht 2 aufgenommen.

Nachweis Schweißnaht 1:
$A_{w1} = a \cdot b_s$ = (a und b_s siehe vorherige Seite) 1350 [mm²]
$\sigma_{x,Ed} = (Z_{Ed} / 2) / A_{w1}$ = 70,4 [N/mm²]
$\tau_{IIEd} = (H_{z,Ed} / 2) / A_{w1}$ = 55,6 [N/mm²]

Nachweis:
$\sigma_V = (\sigma^2_x + 3\,\tau^2_{II})^{0,5}$ = 119,2 [N/mm²] ≤ 360,0 [N/mm²] = $f_u / (\beta_W \cdot \gamma_{M2})$
η = 0,33

$\sigma_{x,Ed}$ = 70,4 [N/mm²] ≤ 259,2 [N/mm²] = $0,9 \cdot f_u / \gamma_{M2}$ η = 0,27

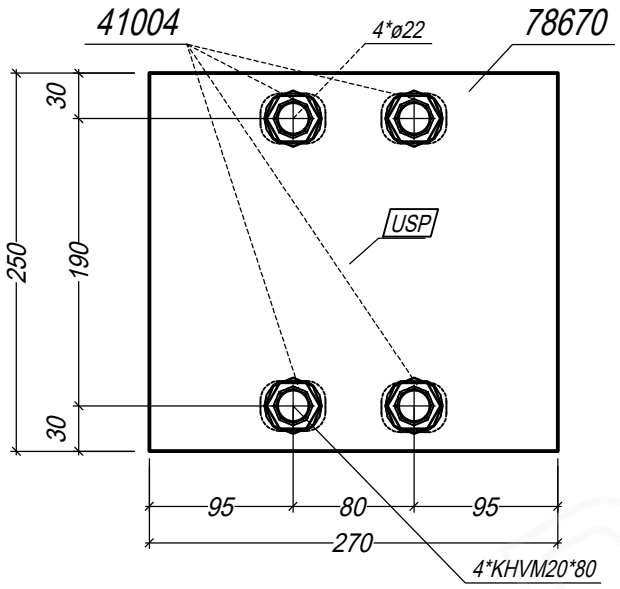
Nachweis Schweißnaht 2:
Die Beanspruchung $H_{y,Ed}$ teilt sich gleihmäßig auf die vier Einzelnähte der Länge l_2 auf.

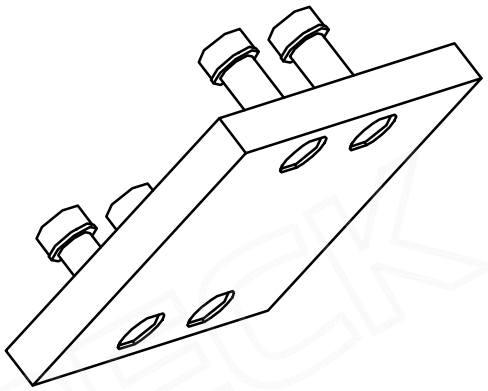
$F_{w,Ed} = (H_{y,Ed} / 4) / l_2$ = 576,9 [N/mm]

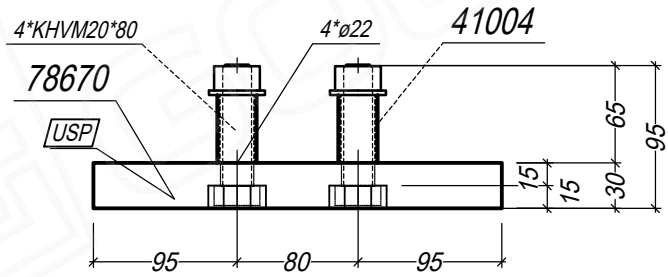
$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} =$$ 208 [N/mm²]

$F_{w,Rd} = f_{vw,d} \cdot a$ = 1039 [N/mm]

Nachweis:
$F_{w,Ed}$ = 576,9 [N/mm] ≤ 1039 [N/mm] = $F_{w,Rd}$ η = 0,56







*Blech ist nach EN 10164 durch UT (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung:siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3 für Standardbeschichtung
*Vorbereitungsgrad P3 nach ISO 8501-3 für Sonderbeschichtung
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78668	FR	s.Stückliste	NormBGR			
41004	RO	4	ROHR26*1.5	45	PE	
78670	FR	1	FL250*30	270	S355J2	UT (USP)
Stahlgewicht:	15,42 kg	Mantelfläche:	0,167 m²	Versandmaße:	(L x H x B)	270*250*95

Stückzahl	Schraube	DIN	Werkstoff	Gewicht	Bemerkung
4	KHVM20*80	EN 14399-4/-6	10.9 tZn	0,36	Werkstatt
Schraubengewicht:				1,44 kg	
Beschichtung:	–			Gesamtgewicht (kg):	16,86 kg

GOLDBECK

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FR | Fräsplatte (Stiftplatte) 270*250mm, t=30mm mit 4 HV–Schrauben M20*80

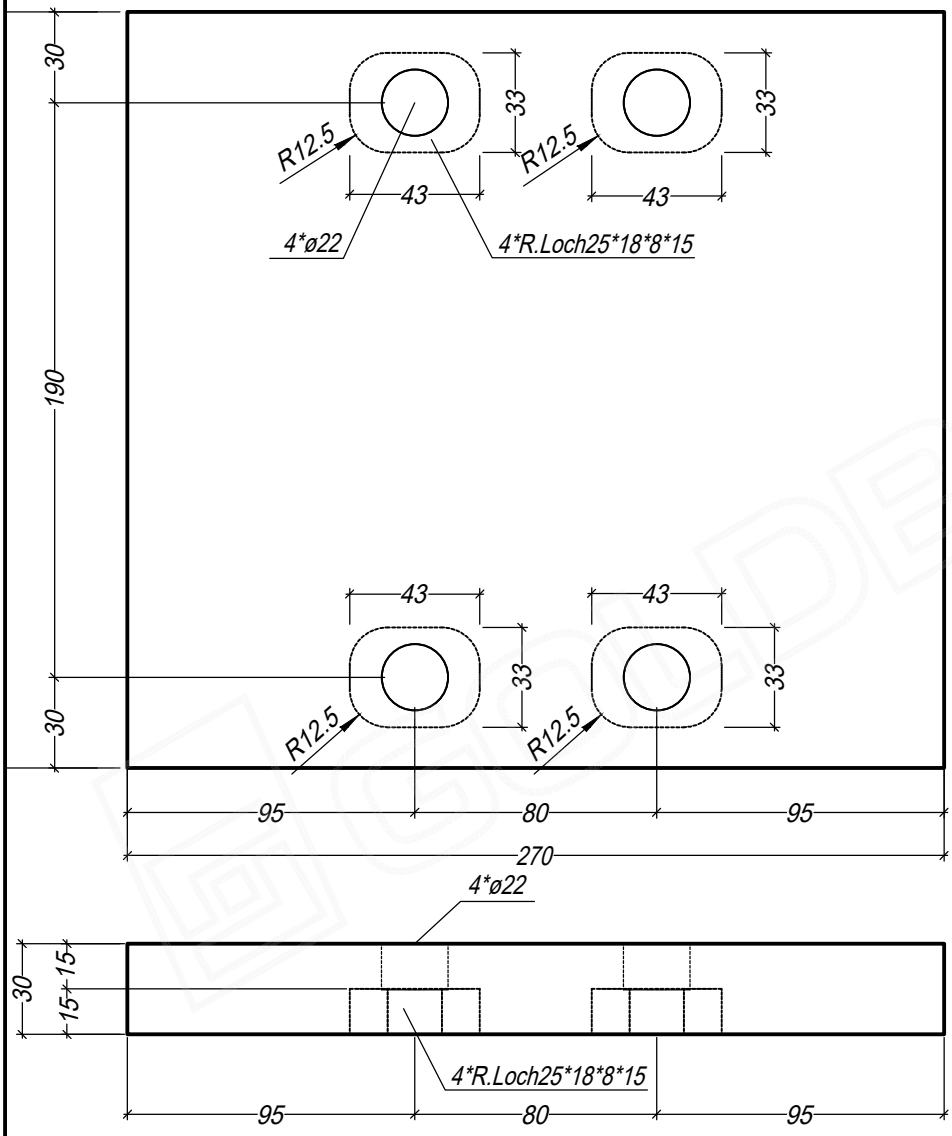
gezeichnet	25.02.2022	[redacted]
geprüft	25.02.2022	[redacted]
Maßstab	1:5	DWG–Name GW78600–78699.dwg

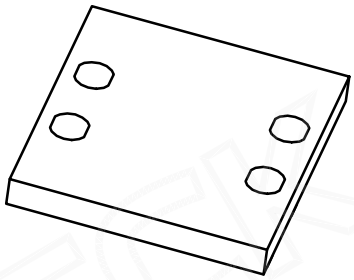
Zusatz			
23; 25; 27	(27)		c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G41	78668	–

Blattgröße: 210mm x 297mm

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*Blech ist nach EN 10164 durch UT (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung:siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3 für Standardbeschichtung
*Vorbereitungsgrad P3 nach ISO 8501-3 für Sonderbeschichtung
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78670	FR	1	FL250*30	270	S355J2	UT (USP)
Stahlgewicht:	15,40 kg	Mantelfläche:	0,151 m²	Versandmaße:	(L x H x B)	270*250*30
Beschichtung:	–				Gesamtgewicht (kg):	15,40 kg

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FR | Fräsplatte 270*250mm, t=30mm UP zu GW78668

gezeichnet	25.02.2022	[redacted]
geprüft	25.02.2022	[redacted]
Maßstab	1:2.5	DWG-Name GW78600–78699.dwg

Zusatz			
23; 25; 27	(27)	G75	c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G41	78670	–

Blattgröße: 210mm x 297mm

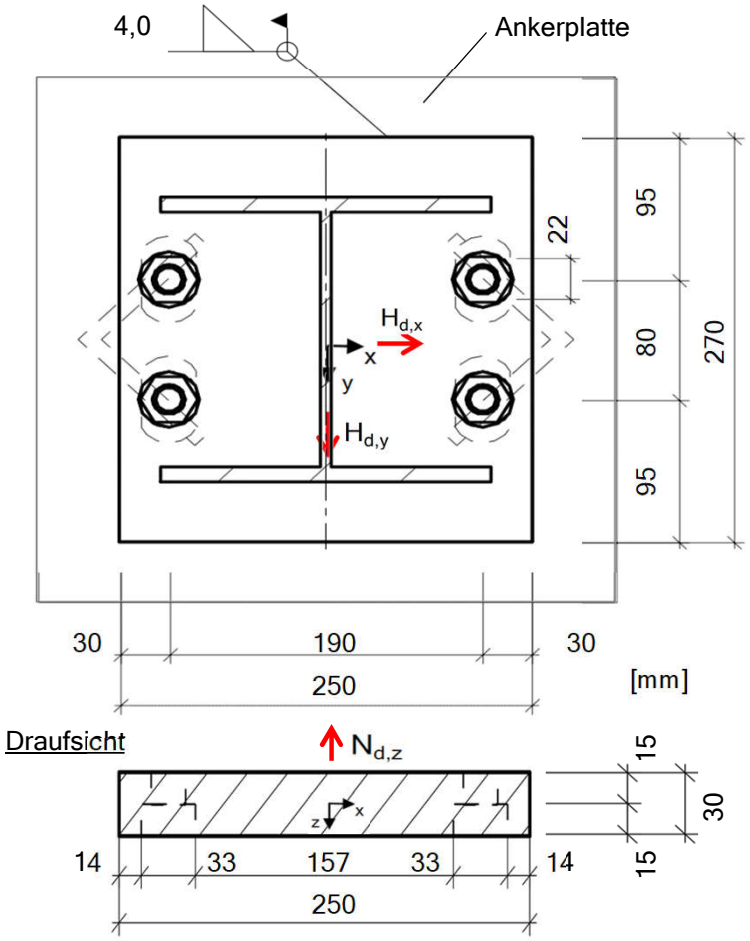
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FRÄSPLATTENBEMESSUNG (EN 1993-1-8:2010-12)

2022-05-02

Auftragsnummer: Grenzlast (projektbez. Last: s. Statik) Positionsnummer: Versandpos. 78668 (=78670)

Nationales Anwendungsdokument: Sonstige (EU)



Draufsicht

Schnitt (durch Fräsung)

Vertikalkraft	$N_{d,z}$:	350,0 kN
Horizontalkraft	$H_{d,x}$:	275,0 kN
Horizontalkraft	$H_{d,y}$:	200,0 kN
Montagenaht	$a_{Fräspl.}$:	4,0 mm
Verbindungsmittel:		M20, 10.9
Werkstoff der Platte:		S 355
Lochabstand	p_x:	19,0 cm
Lochabstand	p_y:	8,0 cm
Fräsplattenbreite	x_F:	25,0 cm
Fräsplattenlänge	y_F:	27,0 cm
Fräsplattendicke	t_p:	30 mm
Ankerplattentyp:		
Ankerplatte mit Schubwinkel (in X)		
Werkstoff der Ankerplatte:		S 355

1. Nachweis der Schweißnaht

$\sigma_{v,w,d} \overset{!}{\leq} f_{vw,d}$ erfüllt: 18,66 kN/cm² < 22,33 kN/cm²

$\tau_{w,d,x} \overset{!}{\leq} f_{vw,d}$ erfüllt: 13,75 kN/cm² < 22,33 kN/cm²

mit $A_{w,z}$ = 21,60 cm², $A_{w,y}$ = 21,60 cm², $A_{w,x}$ = 20,0 cm²

2. Nachweis der Schraubenverbindung

$F_{v,Ed} \overset{!}{\leq} F_{v,Rd}$ erfüllt: 85,0 kN < 139,6 kN

$F_{t,Ed} \overset{!}{\leq} F_{t,Rd}$ erfüllt: 87,5 kN < 163,3 kN

$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1,4 \cdot F_{t,Rd}} \overset{!}{\leq} 1$ erfüllt: 0,99 < 1

3. Nachweis der Fräsplatte (einachsig in x-Richtung gespannte Platte)

$\sigma_d \overset{!}{\leq} \sigma_{R,d}$ erfüllt: 12,68 kN/cm² < 31,36 kN/cm²

mit $M_{d,y}$ = 525,0 kNcm, $W_{pl,y}$ = 41,4 cm³

1. Nachweis der Schweißnaht

1.1 Beanspruchbarkeit

$$f_{vw,d} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = \frac{47{,}0\ \text{kN/cm}^2}{\sqrt{3} \cdot 0{,}90 \cdot 1{,}35} = 22{,}33\ \text{kN/cm}^2$$

1.2 Beanspruchung

1.2.1 Beanspruchung in y- und z-Richtung

$$\sigma_{w,d,z} = \frac{N_{d,z}}{A_{w,z}} = \frac{N_{d,z}}{2 \cdot a \cdot y_F} = \frac{350{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 27{,}0\ \text{cm}} = \frac{350{,}0\ \text{kN}}{21{,}6\ \text{cm}^2} = 16{,}20\ \text{kN/cm}^2$$

$$\tau_{w,d,y} = \frac{H_{d,y}}{A_{w,y}} = \frac{H_{d,y}}{2 \cdot a \cdot y_F} = \frac{200{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 27{,}0\ \text{cm}} = \frac{200{,}0\ \text{kN}}{21{,}6\ \text{cm}^2} = 9{,}26\ \text{kN/cm}^2$$

$$\sigma_{v,w,d} = \sqrt{\sigma_{w,d}^2 + \tau_{w,d}^2} = 18{,}66\ \text{kN/cm}^2$$

1.2.2 Beanspruchung in x-Richtung

$$\tau_{w,d,x} = \frac{H_{d,x}}{A_{w,x}} = \frac{H_{d,x}}{2 \cdot a \cdot x_F} = \frac{275{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 25{,}0\ \text{cm}} = \frac{275{,}0\ \text{kN}}{20{,}0\ \text{cm}^2} = 13{,}75\ \text{kN/cm}^2$$

1.3 Nachweise

$\sigma_{v,w,d} \overset{!}{\leq} f_{vw,d}$ erfüllt: $18{,}66\ \text{kN/cm}^2 < 22{,}33\ \text{kN/cm}^2$

$\tau_{w,d,x} \overset{!}{\leq} f_{vw,d}$ erfüllt: $13{,}75\ \text{kN/cm}^2 < 22{,}33\ \text{kN/cm}^2$

2. Nachweis der Schraubenverbindung

2.1 Abscheren

2.1.1 Beanspruchung

Komponenten der Schraubenkräfte aus $V_{d,z}$

$$V_{Vd,z} = \frac{H_{d,y}}{n} = \frac{200{,}0\ \text{kN}}{4} = 50{,}0\ \text{kN}$$

Komponenten der Schraubenkräfte aus $V_{d,y}$

$$V_{Vd,y} = \frac{H_{d,x}}{n} = \frac{275{,}0\ \text{kN}}{4} = 68{,}8\ \text{kN}$$

Maximale Schraubenkraft

$$F_{v,Ed} = \sqrt{(V_{Vd,z})^2 + (V_{Vd,y})^2} = \sqrt{(50{,}0\ \text{kN})^2 + (68{,}8\ \text{kN})^2} = 85{,}0\ \text{kN}$$

2.1.2 Beanspruchbarkeit

$$F_{v,Rd} = \frac{\alpha_v \cdot A \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}6 \cdot 3{,}14\ \text{cm}^2 \cdot 100{,}0\ \text{kN/cm}^2}{1{,}35} = 139{,}6\ \text{kN}$$

2.1.3 Nachweis

$F_{v,Ed} \overset{!}{\leq} F_{v,Rd}$ erfüllt: $85{,}0\ \text{kN} < 139{,}6\ \text{kN}$

2.2 Schraubenzugkraft

2.2.1 Beanspruchbarkeit

$$F_{t,Rd} = \frac{k_2 \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}9 \cdot 2{,}45\ cm^2 \cdot 100{,}0\ kN/cm^2}{1{,}35} = 163{,}3\ kN$$

2.2.2 Beanspruchung

$$F_{t,Ed} = \frac{N_{d,z}}{n} = \frac{350{,}0\ kN}{4} = 87{,}5\ kN$$

2.2.3 Nachweis

$$F_{t,Ed} \overset{!}{\leq} F_{t,Rd} \qquad \text{erfüllt:} \qquad 87{,}5\ kN < 163{,}3\ kN$$

2.3 Zug und Abscheren

2.3.1 Beanspruchung

$F_{v,Ed} = 85{,}0\ kN$ $F_{t,Ed} = 87{,}5\ kN$

2.3.2 Beanspruchbarkeit

$F_{v,Rd} = 139{,}6\ kN$ $F_{t,Rd} = 163{,}3\ kN$

2.3.3 Nachweis

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} \overset{!}{\leq} 1 \qquad \text{erfüllt:} \qquad 0{,}99 < 1$$

3. Nachweis der Fräsplatte (einachsig in x-Richtung gespannte Platte)

3.1 Schnittgrößen

$$M_{d,y} = \frac{N_{d,z}}{2} \cdot \frac{x_F - p_x}{2} = \frac{350{,}0\ kN}{2} \cdot \frac{25{,}0\ cm - 19{,}0\ cm}{2} = 525{,}0\ kNcm$$

3.2 Querschnittswerte

$$W_{pl,y} = \frac{(y_F - 2 \cdot a_{y,Fräs.}) \cdot t_P^2}{4} = \frac{(27{,}0\ cm - 2 \cdot 4{,}3\ cm) \cdot (3{,}0\ cm)^2}{4} = 41{,}4\ cm^3$$

Konstruktionsdetails zur Fräsung: s. Abschnitt 4.

3.3 Beanspruchbarkeit

$$\sigma_{R,d} = \frac{f_y}{\gamma_{M0}} = \frac{34{,}5\ kN/cm^2}{1{,}10} = 31{,}36\ kN/cm^2$$

3.4 Beanspruchung

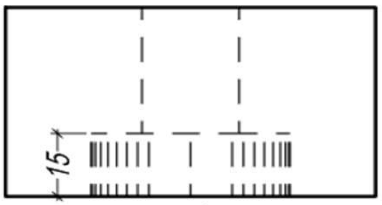
$$\sigma_d = \frac{M_{d,y}}{W_{pl,y}} = \frac{525{,}0\ kNcm}{41{,}4\ cm^3} = 12{,}68\ kN/cm^2$$

3.5 Nachweis

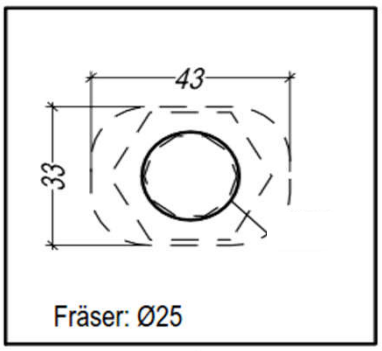
$$\sigma_d \overset{!}{\leq} \sigma_{R,d} \qquad \text{erfüllt:} \qquad 12{,}68\ kN/cm^2 < 31{,}36\ kN/cm^2$$

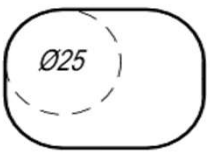
4. Konstruktionsdetails

Verbindungsmittel: M20, 10.9



Ansicht





Draufsicht



Ausführungsdetail der Fräsung

Pos 78577 Teilflächenbelastung für Fußplatte 270 x 250 x 25 [mm]

BOCAD - TYP 11 | Fußplatte Pos. 78577, t = 25 mm, S355 | Fräsplatte Pos. 78668, t = 30 mm, S355

v20.12

EINGANGSWERTE

Baustoffe
Beton C30/37 → f_{cd} = 17 N/mm²

Belastung
F_{Ed} = 1725 kN

Lastausmitte
e = 0 mm

unterstützendes Bauteil
a = 500 mm
b = 2500 mm

Fuß- bzw. Zwischenplatte
a_0 = 270 mm
b_0 = 250 mm

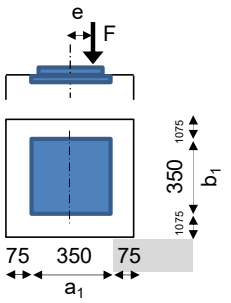
Ankerplatte Pos. 78618
a_1 = 350 mm
b_1 = 350 mm
t_1 = 15 mm

Verteilungsfläche
a_2 = 500 mm
b_2 = 470 mm

BEMESSUNG

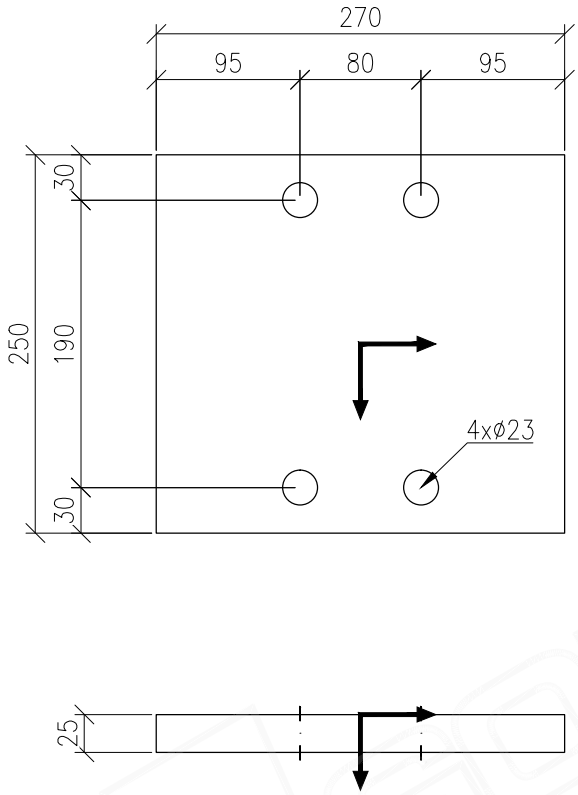
Es werden eine gleichmäßige Pressung unterseitig der Fuß- bzw. Zwischenplatte sowie ein Lastausbreitungswinkel 1:2 innerhalb der Ankerplatte unterstellt.
Unterseitig der Ankerplatte ergibt sich die Belastungsfläche zu:

a_1' = 330 mm
b_1' = 310 mm



F_{Ed} = 1725 kN < 2635 kN = F_{Rdu}

A_{c0} = 102300 mm²
$a_{1,red}$ = 330 mm → a_2, b_2
$A_{c0,red}$ = 102300 mm²
$\sigma_{Ed,0}$ = = 16,9 N/mm² < 17,0 N/mm² = f_{cd} → kein Nachweis der Teilflächenbelastung erforderlich



*Blech ist nach EN 10164 durch
UT (Ultraschallprüfung)
im Raster 200x200mm zu prüfen

*Werkstoff: siehe Stückliste
*Schrauben:
KM..*..4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAZM..*..8.8 nach EN ISO 4017, 4032, 7089
KHVM..*..10.9 nach DIN EN 14399-4/-6
*Vorspannung: siehe Übersichtszeichnung
*Lochspiel: 2mm
*Vorbereitungsgrad P1 nach ISO 8501-3
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur bei S355 (C_{AE} >0.42%
Materialdicke: 25...40mm 120°C
>40mm 180°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78577	Fußplatte	s. Stückliste	FL250x25	270	S355J2	Ultraschallprüfung
Stahlgewicht:	13,25 kg	Mantelfläche:	0,15 m²	Versandmaße:	(H x B x L)	250 x 25 x 270
Beschichtung:	–				Gesamtgewicht (kg):	13,25

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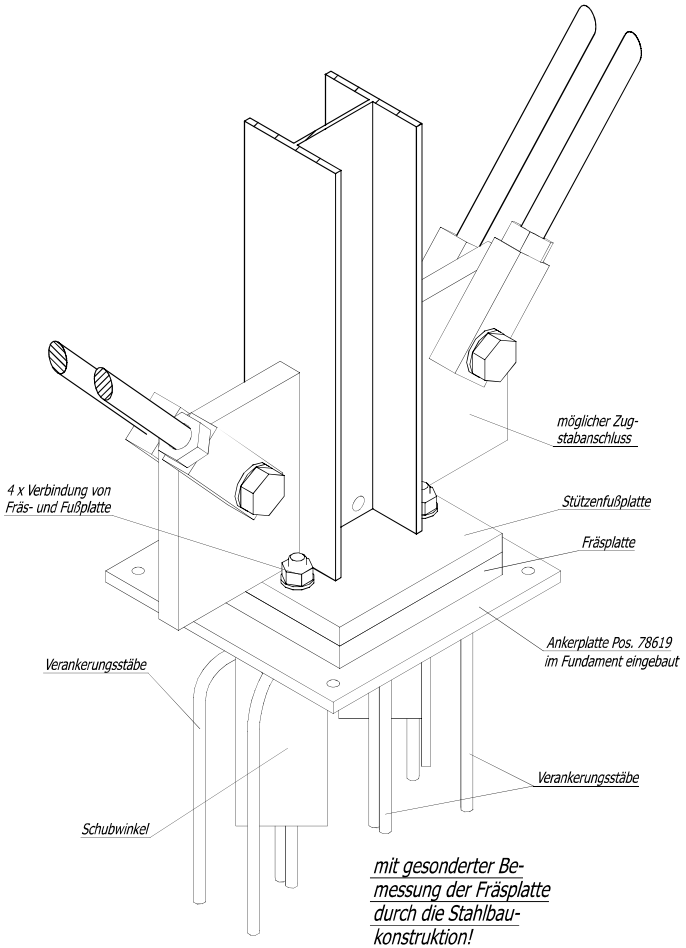
Fußplatte
250mm x 270mm t=25mm

gezeichnet 18.10.2012 [redacted]
geprüft
Maßstab 1:5
DWG-Name [redacted]

a	22.05.2015	Weisters / Wisotzky	Stückliste	
Zusatz		(27)	G10	c19
		Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

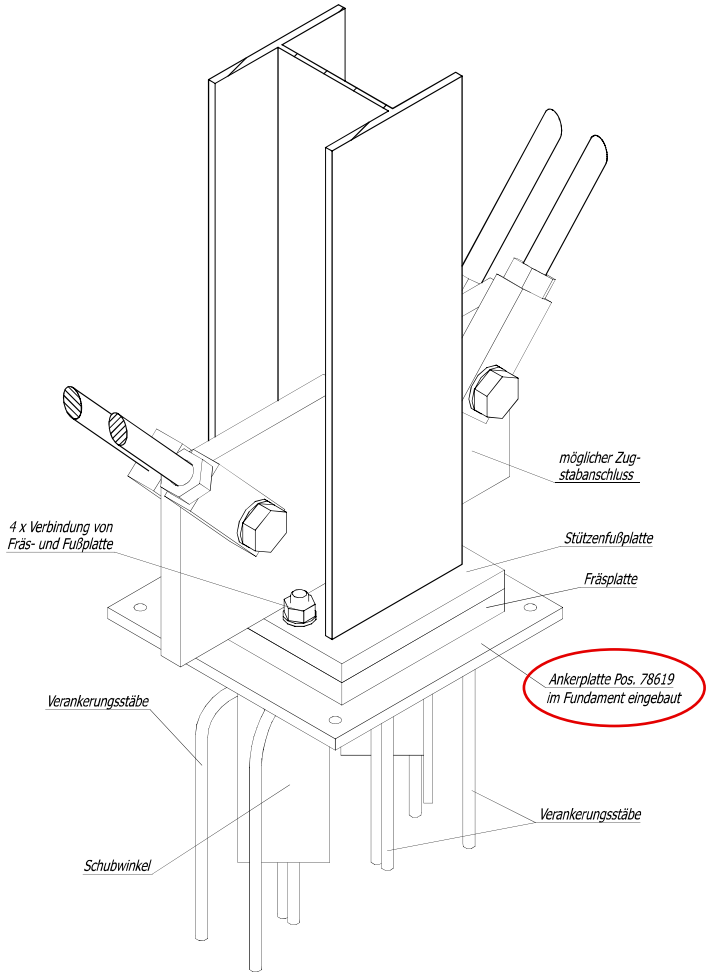
Typ	Tabelle 2	Normteilnummer	Index
P	G42	78577	a

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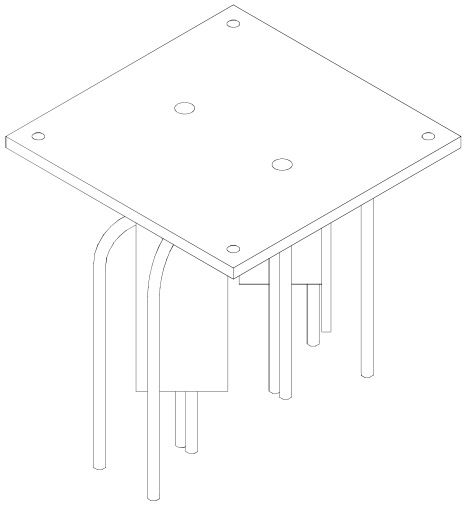


mit gesonderter Be-
messung der Fräsplatte
durch die Stahlbau-
konstruktion!

Einbausituation (gedrehte Stütze, Spezialfall)



Einbausituation (Regelfall)



Perspektive der Ankerplatte Pos. 78619

Index	Datum	Name
-	-	-
		-

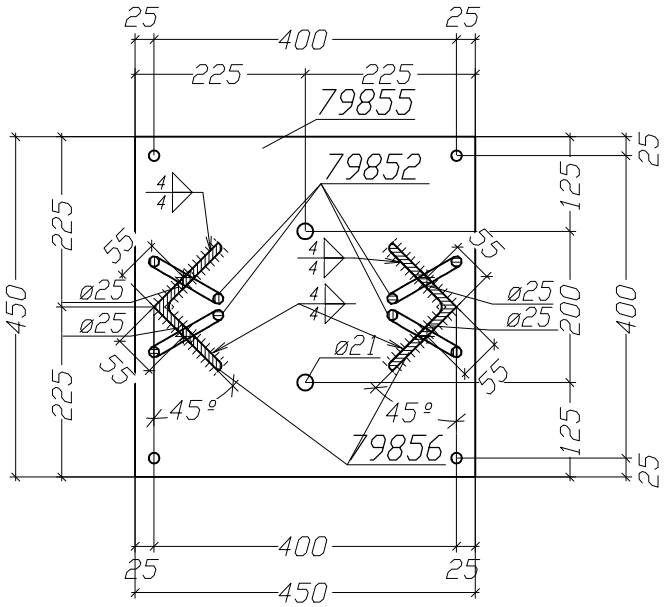
GOLDBECK

Ummelner Str. 4-6, 33649 Bielefeld, Telefon 0521/9488-0

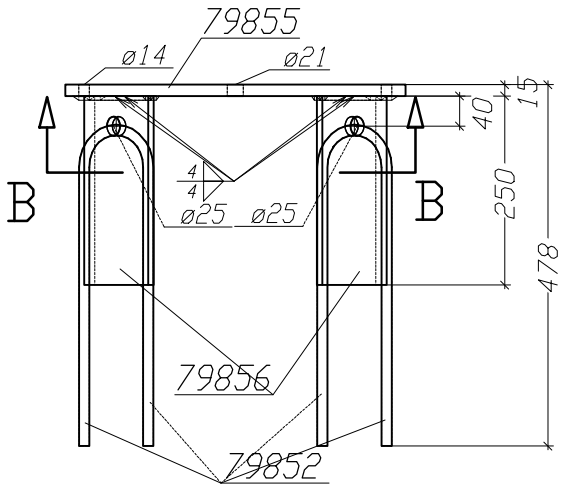
Planinhalt:
Hauptposition 78619
AP - ANKERPLATTE
Einbausituation

Phase	Planer	Art
5	WP	FE
Ebene	Laufende-Nr.:	Index
XX	78619	-

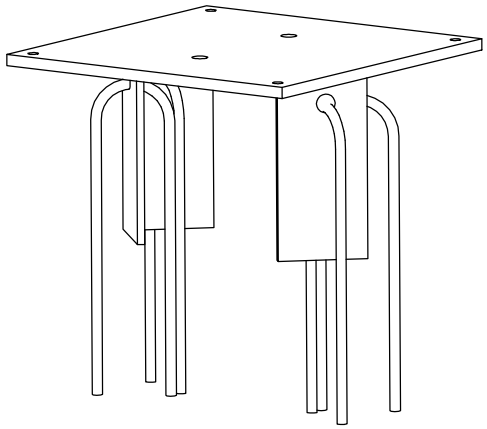
bocad-3d /rev. 24.1.8078



Schnitt B-B



Ansicht A-A



*Werkstoff: siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAZM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung: siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 20mm = 5%

Nähte ohne Angabe: Kehlnaht, a=4mm

Position	Benennung	Profil	Stückzahl	Material	Länge	Bemerkung
78619	Ankerplatte H5	H5	s.Versandliste			
79852	Bewehrung	Betonstahl Ø14	4	B500B	891	für Transport fixiert
79855	PL	FL450*15	1	S355J2	450	
79856	WI	L120*10	2	S355J2	250	
Stahlgewicht:	37,25	Mantelfläche:	0,809	Versandmaße:	(H x B x L)	450*450*478
Beschichtung:	schwarz				Gesamtgewicht (kg):	37,25

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4-6, 33649 Bielefeld

Ankerplatte Typ H5
450 x 450 mm
mit Schubwinkel und Bewehrungsschlaufe

gezeichnet	03.06.2015
geprüft	03.06.2015
Maßstab	1:10
DWG-Name	78618-78619.dwg

Zusatz			
	(27)		c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G40	78619	–

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Nachweis der Verankerung H5 Pos. 78619

Winkelprofil:

Profil: L 120 x 120 x 10 Fundamentbeton: C 30/37

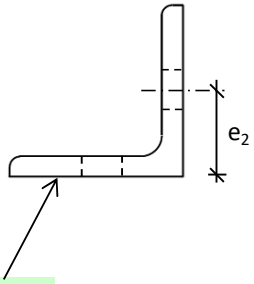
Material: S 355 γ_C = 1,5

Länge l_L = 25 [cm]

d_L = 25 [mm] (Bohrung für Zugverankerung)

e_2 = 55 [mm]

a_L = 40 [cm] (Abstand der L-Profile)



Verschweißung mit umlaufender Kehlnaht: a = 4 [mm]

e_1' = 40 [mm]

Stiftbolzenplatte:

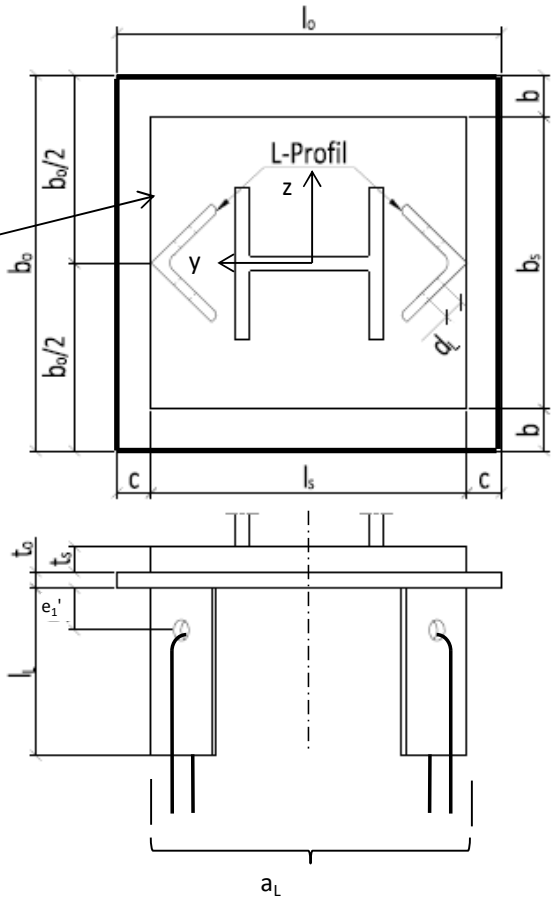
Material: S 235

l_S = 37 [cm]

b_S = 35 [cm]

t_S = 15 [mm]

a = 5 [mm]



Ankerplatte, oben:

Material: S 355

l_O = 45 [cm]

b_O = 45 [cm]

t_O = 1,5 [cm]

Fundamentabmessungen:

b = 55 [cm]

d = 60 [cm]

Betondeckung, Fundament:

c_{nom} = 35 [mm]

Bewehrungswahl Zugverankerung:

Ø = 14 [mm]

l = 42,5 [cm] Länge des Bewehrungsstabes

γ_S = 1,15

f_y = 500 [N/mm²]

max. gleichz. wirk. Bemessungskräfte (projektbezogene Last: s. akt. Tragwerkspl.)

Z_{Ed} = 240,0 [kN]

$H_{y,Ed}$ = 180,0 [kN]

$H_{z,Ed}$ = 180,0 [kN]

vorh. A_S = 12,32 [cm²] erf. $A_S = Z_{Ed} / (f_y / \gamma_s)$ = 5,52 [cm²]

Biegerollendurchmesser: $D_{min} = 4 \cdot Ø$ = 56,0 [mm]

gewählt: D = 6 · Ø = 84,0 [mm]

Verbundbedingung: (◉) mäßig ( ) gut

f_{ck} = 30 [N/mm²]

α_C = 0,85

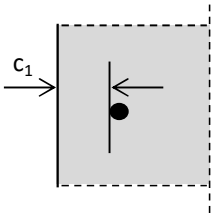
γ_C = 1,5

$f_{cd} = \alpha_C \cdot f_{ck} / \gamma_C$ = 17,0 [N/mm²]

$f_{ctk;0,05}$ = 2 [N/mm²]

η_1 = 0,7

η_2 = 1,0



Bemessungswert der Verbundfestigkeit: $f_{bd} = 2{,}25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd}$ = 2,10 [N/mm²]

Grundwert der Verankerungslänge: $l_{b,rqd} = (Ø/4) \cdot (f_{yd}/f_{bd})$ = 725 [mm]

c_d = min {a/2; c_1} hier: $c_d = c_1$ = 85,8 [mm]

α_1 = 1,0

α_2 = 0,2

$\alpha_1 \cdot \alpha_2 \cdot \alpha_5 \geq 0{,}7$ = 0,7

α_3 = 1,0

α_4 = 1,0

α_5 = 1,0

$$\alpha_2 = 1 - 0{,}15 \cdot \frac{c_d - Ø}{Ø}$$

Mindestwert der Verankerungslänge:

$l_{b,min} = \max \{0{,}3 \cdot \alpha_1 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd};\ 10 \cdot Ø\}$ = 217 [mm]

Bemessungswert der Verankerungslänge $l_{bd} \geq l_{b,min}$:

$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,rqd}$ = 507 [mm]

l_{bd} = 507 [mm]

Abminderung der Verankerungslänge um erf. A_s / vorh. $A_s \geq 0{,}3$:

erf. A_S / vorh A_S = 0,45

Abgeminderter Bemessungswert der Verankerungslänge l_{bd}:

l_{bd} = 22,7 [cm]

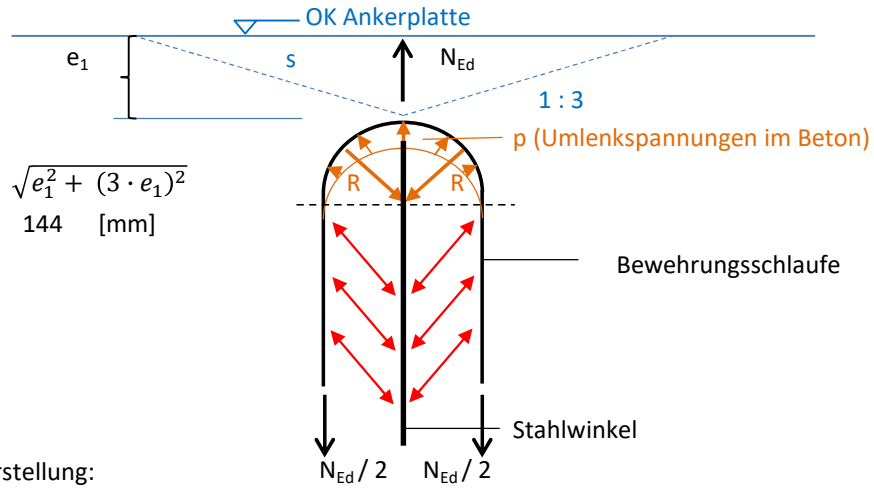
Vorhandene Verankerungslänge, l_{vorh}:

Nachweis: l_{vorh} = 42,5 > 22,7 = l_{bd} Ausnutzung: 0,53

Nachweis des Bewehrungsquerschnitts und Lasteinleitung:

γ_S =	1,15		$e_1 = e_1' + 0{,}5 \cdot d_L - 0{,}5 \cdot Ø$ =	45,5	[mm]
f_y =	500	[N/mm²]	Ø =	14	[mm]
$N_{Ed} = Z_{Ed} / 4$ =	60	[kN]	A =	1,54	[cm²]
			d_{Br} = 84,0 [mm] ≥ $4 \cdot Ø$ =	56	mm

Modell:



$s = \sqrt{e_1^2 + (3 \cdot e_1)^2}$

s = 144 [mm]

Modellvorstellung:

Die Zugkraft des Winkelprofils wird über Reibung in den Beton eingeleitet und durch die Bewehrungsschlaufe verankert.

Resultierende Horizontalbeanspruchung R_H, die gegen den Stahlwinkel drückt.

$$R_H = \int_0^{90} \cos\alpha \cdot N_{Ed}/2 \quad = 30 \quad [kN]$$

Die Horizontalkomponente R_H wirkt auf die Fläche, die sich aus den Abmessungen des Biegeradius mal der Profilbreite ergibt.

$\sigma_N = R_H / A$ = 5,95 [N/mm²]

Fuge zwischen Stahlprofil und Beton: sehr glatt

$V_{Rdi} = c \cdot f_{ctd} + \mu \cdot \sigma_N + \rho \cdot f_{yd} \cdot 1{,}2 \cdot \mu \cdot \sin\alpha + \cos\alpha)$ $\left\{ \begin{array}{l} \leq 0{,}5 \cdot \upsilon \cdot f_{cd} \\ \leq 0{,}1 \cdot f_{cd} \end{array} \right\}$ (6.25)

NCI zu 6.2.2 (6)

$\upsilon =$ 0,20

$\mu =$ 0,50

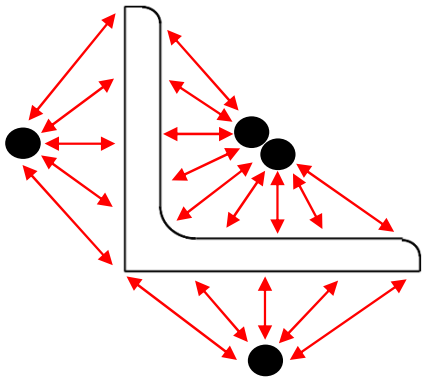
$c =$ 0,00

Werte für sehr glatt

$\mu \cdot \sigma_N \leq 0{,}1 \cdot f_{cd}$ Reibungsanteil für sehr glatte Fugen nach (6.25)

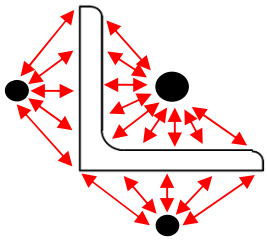
2,98 [N/mm²] ≤ 1,70 [N/mm²] → maßgebend: 1,70 [N/mm²]

Schematische Skizze Trajektorien vom Kraftverlauf aus dem Profil über Reibung durch den Beton in die Bewehrungsschlaufe.



8.9

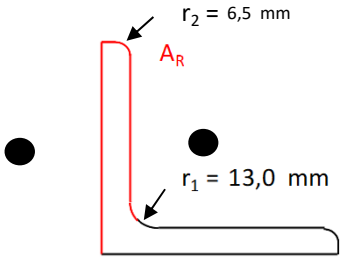
Stabbündel:
Die aneinanderliegenden Bewehrungsstähle im Innenbereich des Winkelprofils dürfen als einen Stab (Ersatzstab) mit gleicher Querschnittsfläche angesehen werden.



Die übertragbare Last einer Bewehrungsschlaufe wird duch die unten abgebildete rote Fläche A_R eingeleitet. Daraus ergibt sich eine übertragebare Beanspruchung von:

$A_R = l_L \cdot (a + a - t - r_1 / \sqrt{2} + 1/8 \cdot 2 \cdot \pi \cdot r_1 + 1/4 \cdot 2 \cdot \pi \cdot r_2 + t - 2 \cdot r_2 / \sqrt{2})$

$A_R =$ 60509 [mm]



$V_{Rdi} = A_R \cdot \min \{\mu \cdot \sigma_N; 0{,}1 \cdot f_{cd}\}$

$V_{Rdi} =$ 102,9 [kN]

Nachweis Reibungsübertragung:

$N_{Ed} / V_{Rdi} =$ 0,58

Nachweis des Bewehrungsquerschnitts:

$(N_{Ed} / 2) / N_{Rd}$ = 0,45 mit $N_{Rd} = A_s \cdot f_{yd}$ = 66,9 [kN]

Z_{Ed} = 240 [kN]

$H_{y,Ed}$ = 180 [kN]

$H_{z,Ed}$ = 180 [kN]

Aus H_{yd} und H_{zd} Resultierende Last R_d:

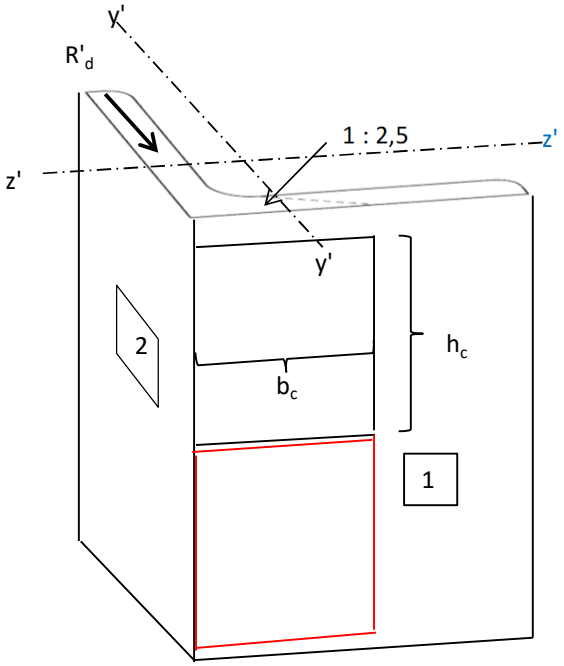
$$R_{Ed} = \sqrt{H_{y,Ed}^2 + H_{z,Ed}^2} = 255 \text{ [kN]}$$

Die Resultierende R_d wirkt hälftig auf beide Profile; als Maximalwert wirkt R'_d auf einen Schenkel des Winkels.

$R'_{Ed} = 0{,}5 \cdot R_{Ed}$ = 127 [kN]

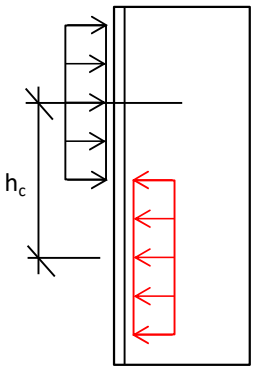
$f_{cd} = \alpha_C \cdot f_{ck} / \gamma_C$ = 17,0 [N/mm²]

$b_c = t + r_1 + 2{,}5 \cdot t$ = 48 [mm]



Teilflächenpressung:

erf. $h_c = R'_{Ed} / (3 \cdot f_{cd} \cdot b_c)$ = 52,0 [mm]



Der Winkel ist einbetoniert. Ein Verdrehen wird behindert, da die gleiche Beanspruchung aus Symetriegründen an der Fläche 2 wirkt und mit Fläche 1 im Gleichgewicht steht.

Für den Winkel resultiert daraus eine Biegebeanspruchung um die lokale Achsen y' - y' und z' - z'.

$M_{y',Ed} = M_{z',Ed} = R'_{Ed} \cdot h_c$ = 661,8 [kNcm]

Länge des Winkelprofils ok!

Vergrößerung um mindestens: nicht erforderlich!

Ausnutzung: $\eta = \dfrac{R'_{Ed}}{b_c \cdot l_L \cdot f_{cd}} =$ 0,62

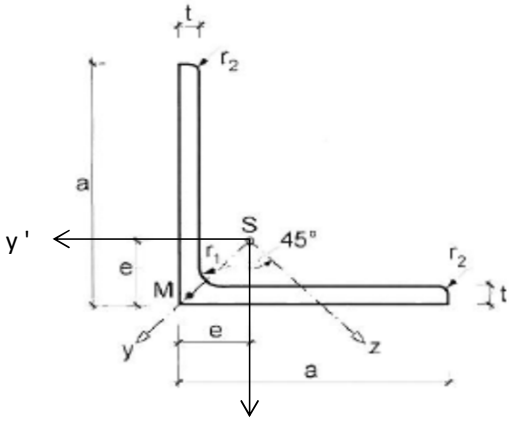
Auf der sicheren Seite liegend: H_{yd} und H_{zd} gleichzeitig in voller Größe wirkend.

Z_{Ed} =	240	[kN]	γ_{M0} =	1,0		f_y =	355	[N/mm²]
R_{Ed} =	255	[kN]	γ_{M1} =	1,1		f_u =	490	[N/mm²]
$M_{y',Ed} = M_{z',Ed}$ =	661,8	[kNcm]	γ_{M2} =	1,25				

Profilkennwerte:

A =	23,18	[cm²]
a =	120	[mm]
e =	3,31	[cm]
t =	10	[mm]
W_y =	58,57	[cm³]
W_z =	27,51	[cm³]
r_1 =	13,0	[mm]
$M_{pl,y,Rd}$ =	3273,0	[kNcm]
$M_{pl,z,Rd}$ =	1681,0	[kNcm]
$V_{pl,y,Rd}$ =	335,9	[kN]
$V_{pl,z,Rd}$ =	335,9	[kN]

siehe S. H5-17



$V_{pl,y',Rd}$ = 237,5 [kN]

$V_{pl,z',Rd}$ = 237,5 [kN]

$A_F = 0{,}5 \cdot A$ = 11,6 [cm²]

A_{net} = 18,2 [cm²]

Je nach dem, ob die Horizontalbeanspruchungen in + und/oder - der Hauptachsen wirken, addieren sich die Biegebeanspruchungen aus der Betonpressung zur Resultierenden Biegebenaspruchung in Richtung der Hauptachse y oder der Hauptachse z; positiv, oder negativ. Die Biegebeanspruchung ist unabhängig von der Zugbeanspruchung wird deshalb ungünstigst berücksichtigt.

Die Zugkraft Z_d verteilt sich gleichmäßig auf alle vier Bewehrungsnadeln.

$Z_{Ed}'' = 1/4 \cdot Z_{Ed}$ = 60,0 [kN]

Die Bewehrungsnadel (Rückhängebewehrung für die Zugkraft) ist nicht in der Schwerachse des Winkels angeordnet. Daraus resultiert ein Versatz e' zur Schwerachse.

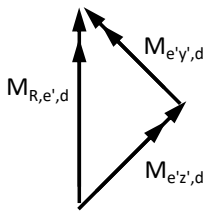
$e' = e - e_2$ = 2,19 [cm]

Berechnung des Versatzmomentes $M_{e',d}$ aus der Exzentrizität e'. Das Moment wirkt um beide lokalen Achsen y' und z'.

$M_{e',Ed} = M_{e'y',Ed} = M_{e'z',Ed} = e' \cdot Z_{Ed}' =$ 131,2 [kNcm]

Das resultierende Versatzmoment $M_{R,e',d}$ wirkt um die globale z-Achse.

$M_{R,e',d} =$ 185,5 [kNcm]

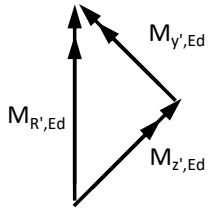


Zusammenfassung der Einwirkungen:
Auf der sicheren Seite liegend wirken die Horizontalbeanspruchungen gleichzeitig in voller Größe. Je nach dem, ob in positiver oder negativer Richtung der Hauptachsen, wirkt die Resultierende R_{Ed} hälftig auf einen Schenkel der L-Profile. Die daraus resultierende Biegung der Winkel $M_{y',Ed}$ und $M_{z',Ed}$ wirkt in der vektoriellen Addition entweder um die gloabe y- oder globale z-Achse (positiv oder negativ).

$M_{y',Ed} = M_{z',Ed} =$ 661,8 [kNcm] (resultierend aus der Betonpressung)
$M_{R,e',Ed} =$ 185,5 [kNcm]

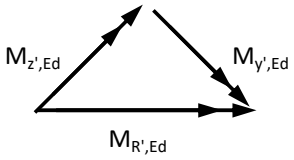
Aus der Betonpressung hervorgerufene, gleichzeitig wirkende Momentenbeanspruchung $M_{y'd}$ und $M_{z'd}$ wirken gleichzeitig. Es sind zwei Fälle zu unterscheiden:

Fall a)



$M_{R'd}$ als Resultierende um die globael z-Achse

Fall b)



$M_{R'd}$ als Resultierende um die globael y-Achse

Für beide Fälle gilt: $|M_{R',Ed}| = \sqrt{M_{z',Ed}^2 + M_{y',Ed}^2}$ = ± 936 [kNcm]

Fall a)

Z'_d = 120 [kN]

R'_d = 127 [kN]

$M_{z,Ed} = M_{R,e',Ed} + M_{R',Ed}$ = 1121 [kNcm]

Fall b)

Z'_d = 120 [kN]

R'_d = 127 [kN]

$M_{z,Ed} = M_{R,e',Ed}$ = 185,5 [kNcm]

$M_{y,Ed} = M_{R',Ed}$ = 936 [kNcm]

Nachweis auf Biegung, Normalkraft und Querkraft:

$$\tau_{Rd} = \frac{f_{yk}}{\sqrt{3} \cdot \gamma_{M0}} = 20{,}5 \text{ [kN/cm²]}$$

$\sigma_{x,Rd} = f_y / \gamma_{M0}$ = 35,5 [kN/cm²]

$N_{pl,Rd} = (A \cdot f_y) / \gamma_{M0}$ = 822,9 [kN]

$N_{u,Rd} = (0{,}9 \cdot A_{net} \cdot f_u) / \gamma_{M2}$ = 641,4 [kN]

$V_{pl,y,Rd} = V_{pl,z,Rd}$ = 335,9 [kN]

$V_{Ed} / V_{pl,Rd}$ = 0,38 < 0,5 Abminderung kann vernachlässigt werden!

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2 = 0{,}00 \qquad (6.29)$$

Abminderung der Streckgrenze für die Momententragfähigkeit durch die Querkraft:

$f'_y = (1 - \rho) \cdot f_y$ = 355 [N/mm²]

$M_{pl,y,Rd}$ = 3273,0 [kNcm]

$M_{pl,z,Rd}$ = 1681,0 [kNcm]

(berechnet mit abgeminderter Streckgrenze infolge Querkraft)

$n = N_{Ed} / \min\{N_{u,Rd}; N_{pl,Rd}\}$ = 0,19 mit: $N_{Ed} = Z'_d$, gem. vorheriger Seite (Fall a bzw. Fall b)

$N_{u,Rd}$ und $N_{pl,Rd}$ siehe oben auf dieser Seite

$$M_{N,Rd} = M_{pl,Rd} \cdot \left[1 - \left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2\right]$$ Interaktion Biegung, Querkraft, Normalkraft (6.32)

$M_{N,y,Rd}$ = 3158,4 [kNcm]

$M_{N,z,Rd}$ = 1622,2 [kNcm]

berechnet mit Gl. (6.32) und $M_{pl,y,Rd}$ bzw. $M_{pl,z,Rd}$ mit abgeminderter Streckgrenze

Nachweis (2-achsige Biegung, Querkraft, Normalkraft)

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1$$

mit: $\alpha = 2$

$\beta = 5 \cdot n \geq 1$ (6.41)

Fall a)

0,69 ≤ 1

Fall b)

mit: $\beta =$ 1,00

0,09 + 0,11 = 0,20 ≤ 1

Die Zugbeanspruchung wird über Reibung vom Winkelprofil in den Beton eingeleitet.
Auf der sicheren Seite liegend führe ich hier zusätzlich ein Nachweis des Winkelprofils auf Lochleibung. Als Modellvorstellung gehe ich hier davon aus, dass die eingehangene Bewehrungsschlaufe das Winkelprofil auf Lochleibung beansprucht.

Das Profil ist vollständig im Beton eingebettet. Es kann sich im Beton nicht verformen oder auf- bzw. ausreißen. Der Nachweis liegt somit auf der sicheren Seite.

			d = Ø =	14	[mm]
			$d_0 = d_L$ =	25	[mm]
			t =	10	[mm]
Randabstand $e_1 \geq 1{,}2\ d_0$ =	30	[mm]	$e_1 = l_L - e_1'$ =	385	[mm]
Randabstand $e_2 \geq 1{,}2\ d_0$ =	30	[mm]	e_2 =	55	[mm]
			f_u =	490	[N/mm²]
hier für den Bewehrungsstahl (DIN 488): $f_{ub} = 1{,}05 \cdot f_{yb}$:			f_{ub} =	525	[N/mm²]
			k_1 =	2,5	
			γ_{M2} =	1,25	

für am Rand liegende Schraube: $\alpha_d = \frac{e_1}{3\ d_0} =$ 5,13

1,00

f_{ub} / f_u = 1,07

α_b = MIN (α_d; f_{ub}/f_u; 1,0) = 1,00 [kN]

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = 137{,}2 \text{ [kN]}$$

Mit $F_{v,Ed} = Z'_{,Ed}$ = 60,0 [kN]

Nachweis: $F_{v,Ed} / F_{b,Rd}$ = 0,44 < 1

$f_{y,min}$ = 355

Für den Nachweis der Schweißnaht wird auf der sicheren Seite liegend die maximal und gleichzeitig auftretenden Bemessunslasten zu Grunde gelegt.
Die Horizontalkräfte wirken in maximaler Größe als Resultierende hälftig auf einen Schenkel des L-Profils.

Material Winkel:	S 355	f_u =	490	[N/mm²]	β_W = 0,9
Material Ankerplatte:	S 355	f_u =	490	[N/mm²]	β_W = 0,9

Umlaufende Kehlnaht: gew. a = 4 [mm]

Auf einen Flansch (Schenkel) angesetzte Schweißnahtlänge l_W:
l_W = Flanschlänge - t - r_1 = 97,0 [mm]

Schweißnahtfläche A_W pro Schenkel:
$A_W = 2 \cdot a \cdot l_W$ = 776 [mm²]

Flächenmoment zweiten Grades J_W der Schweißnaht pro Schenkel:
$J_W = 2 \cdot a \cdot l^3_W \cdot 1/12$ = 60,8 [cm⁴]

Widerstandsmoment der Schweißnaht pro Schenkel:
$W_w = J_W / (l_W/2)$ = 12,5 [cm³]

Korrelationsbeiwert: β_W = 0,9 γ_{M2} = 1,25

Zugfestigkeit des schwächeren Materials: f_u = 490 [N/mm²]

Einwirkende Bemessungslasten auf das Profil, auf einen Schenkel:

Normalkraft: siehe Seite H5-8 $N_{Ed} = Z''_{Ed}$ = 60,0 [kN]

Querkraft: siehe Seite H5-7 $V_{Ed} = R'_{Ed}$ = 127,3 [kN]

Beanspruchungen:

$\sigma_{x,Ed} = N_{Ed} / A_W + M_{e',d} / W_w$ = 18,2 [kN/cm²] $M_{e',d}$ siehe Seite H5-8

$\tau_{IIEd} = V_{Ed} / A_w$ = 16,4 [kN/cm²]

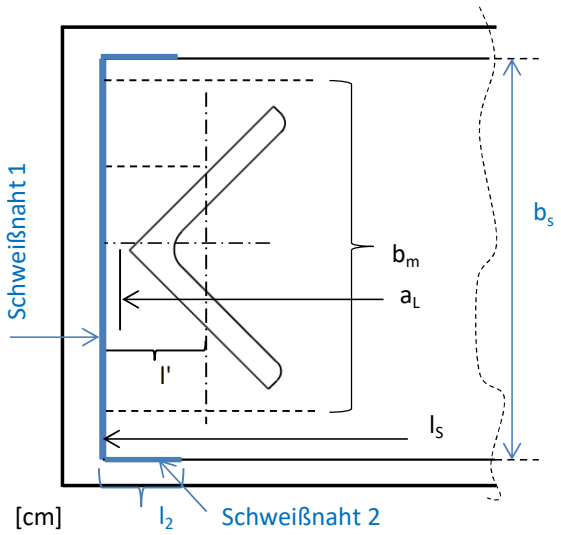
Nachweis:

$\sigma_V = (\sigma^2_x + 3\,\tau^2_{II})^{0,5}$ = 33,7 [kN/cm²] ≤ 43,6 [kN/cm²] $= f_u / (\beta_W \cdot \gamma_{M2})$

η = 0,77

$\sigma_{x,Ed}$ = 18,2 [kN/cm²] ≤ 35,3 [kN/cm²] $= 0,9 \cdot f_u / \gamma_{M2}$ η = 0,52

t_O =	1,5	[cm]
f_y =	355	[N/mm²]
γ_{M0} =	1,0	
l_s =	37	[cm]
a_L =	40	[cm]
b_s =	35	[cm]
l_2 =	8	[cm]
a =	5	[mm]



$l' = 0,5 \cdot (l_s - a_L) + e / \sqrt{2}$ = 0,84 [cm]

Pro Winkel wirkt die halbe einwirkende Bemessungszugkraft:

Z'_{Ed} = 120 [kN]

Als Versatz l' zur Einleitung der Zugbeanspruchung wird der Abstand von der Schweißnaht 1 zum Schwerpunkt des L-Profils angesetzt.
Daraus resultiert eine Momentenbeanspruchung M_{Ed}.

$M_{Ed} = I\, Z'_{Ed} \cdot l' \,I$ = 101,2 [kNcm]

M_{pl} der Ankerplatte: $M_{pl} = t^2_O / 4 \cdot (f_y / \gamma_{M0})$ = 20,0 [kNcm/cm]

mittragende Breite b_m (mit a = Breite des Schnekels vom L-Profil):

$b_m = 2 \cdot a / \sqrt{2} + 2 \cdot 2,5 \cdot t_O$ = 24,5 [cm]

$M_{pl,Rd} = M_{pl} \cdot b_m$ = 488,6 [kNcm]

Nachweis:

$M_{Ed} / M_{pl,Rd}$ = 0,21 ≤ 1

Einwirkungen: Z_{Ed} = 240 [kN]
$H_{y,Ed}$ = 180 [kN]
$H_{z,Ed}$ = 180 [kN]

Material Ankerplatte: S 355 f_u = 490 [N/mm²] β_W = 0,9
Material Stiftbolzenpl.: S 235 f_u = 360 [N/mm²] β_W = 0,8

Korrelationsbweiwert: β_W = 0,8 γ_{M2} = 1,25

Zugfestigkeit des schwächeren Materials: f_u = 360 [N/mm²]

Die Beanspruchung Z_{Ed} und $H_{z,Ed}$ wird von der Schweißnaht 1 und die Beanspruchung $H_{y,Ed}$ von der Schweißnaht 2 aufgenommen.

Nachweis Schweißnaht 1:
$A_{w1} = a \cdot b_s$ = (a und b_s siehe vorherige Seite) 1750 [mm²]
$\sigma_{x,Ed} = (Z_{Ed} / 2) / A_{w1}$ = 68,6 [N/mm²]
$\tau_{IIEd} = (H_{z,Ed} / 2) / A_{w1}$ = 51,4 [N/mm²]

Nachweis:
$\sigma_V = (\sigma^2_x + 3\,\tau^2_{II})^{0,5}$ = 112,4 [N/mm²] ≤ 360,0 [N/mm²] $= f_u / (\beta_W \cdot \gamma_{M2})$
η = 0,31

$\sigma_{x,Ed}$ = 68,6 [N/mm²] ≤ 259,2 [N/mm²] $= 0,9 \cdot f_u / \gamma_{M2}$ η = 0,26

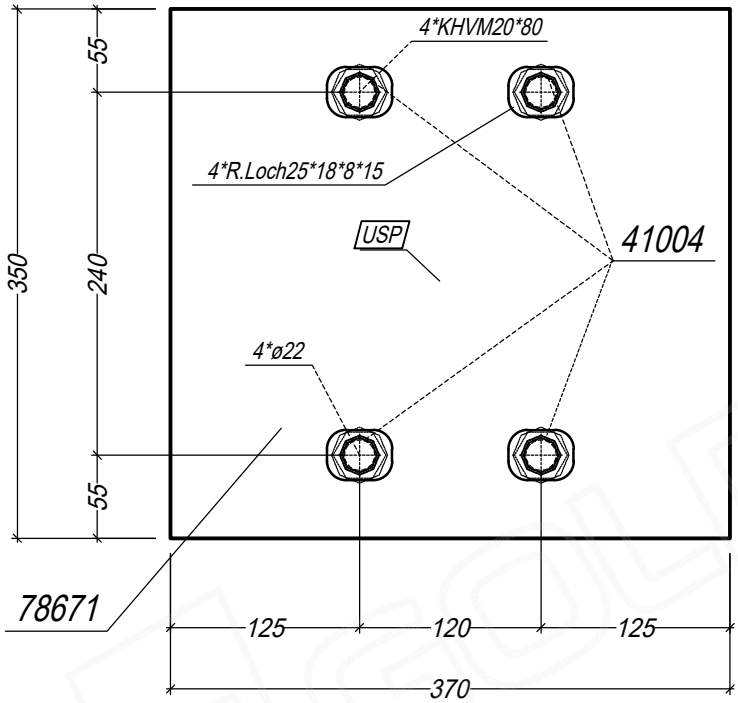
Nachweis Schweißnaht 2:
Die Beanspruchung $H_{y,Ed}$ teilt sich gleihmäßig auf die vier Einzelnähte der Länge l_2 auf.

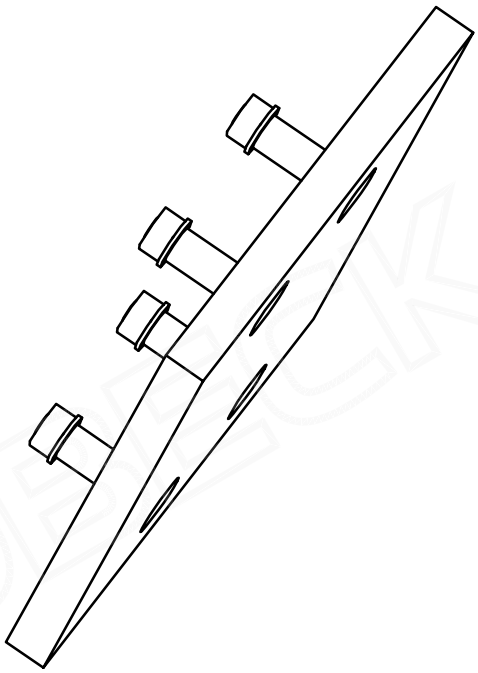
$F_{w,Ed} = (H_{y,Ed} / 4) / l_2$ = 562,5 [N/mm]

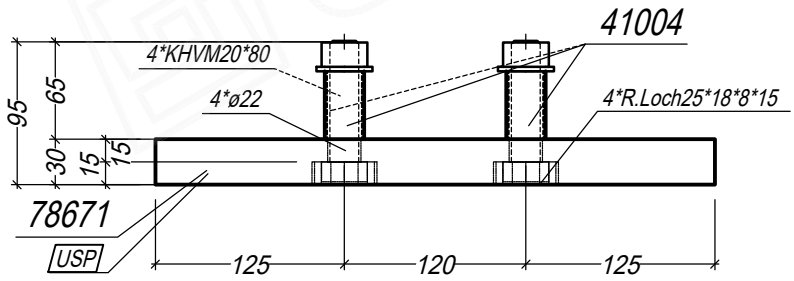
$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \cdot \gamma_{M2}} =$$ 208 [N/mm²]

$F_{w,Rd} = f_{vw,d} \cdot a$ = 1039 [N/mm]

Nachweis:
$F_{w,Ed}$ = 562,5 [N/mm] ≤ 1039 [N/mm] $= F_{w,Rd}$ η = 0,54







*Blech ist nach EN 10164 durch UT (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung:siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3 für Standardbeschichtung
*Vorbereitungsgrad P3 nach ISO 8501-3 für Sonderbeschichtung
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78669	FR	s.Stückliste	NormBGR			
41004	RO	4	ROHR26*1.5	45	PE	
78671	FR	1	FL350*30	370	S355J2	UT (USP)
Stahlgewicht:	30,51 kg	Mantelfläche:	0,297 m²	Versandmaße:	(L x H x B)	370*350*95

Stückzahl	Schraube	DIN	Werkstoff	Gewicht	Bemerkung
4	KHVM20*80	EN 14399-4/-6	10.9 tZn	0,36	Werkstatt
Schraubengewicht:				1,44 kg	
Beschichtung:	–			Gesamtgewicht (kg):	31,95kg

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

FR | Fräsplatte (Stiftplatte) 370*350mm, t=30mm mit 4 HV–Schrauben M20*80

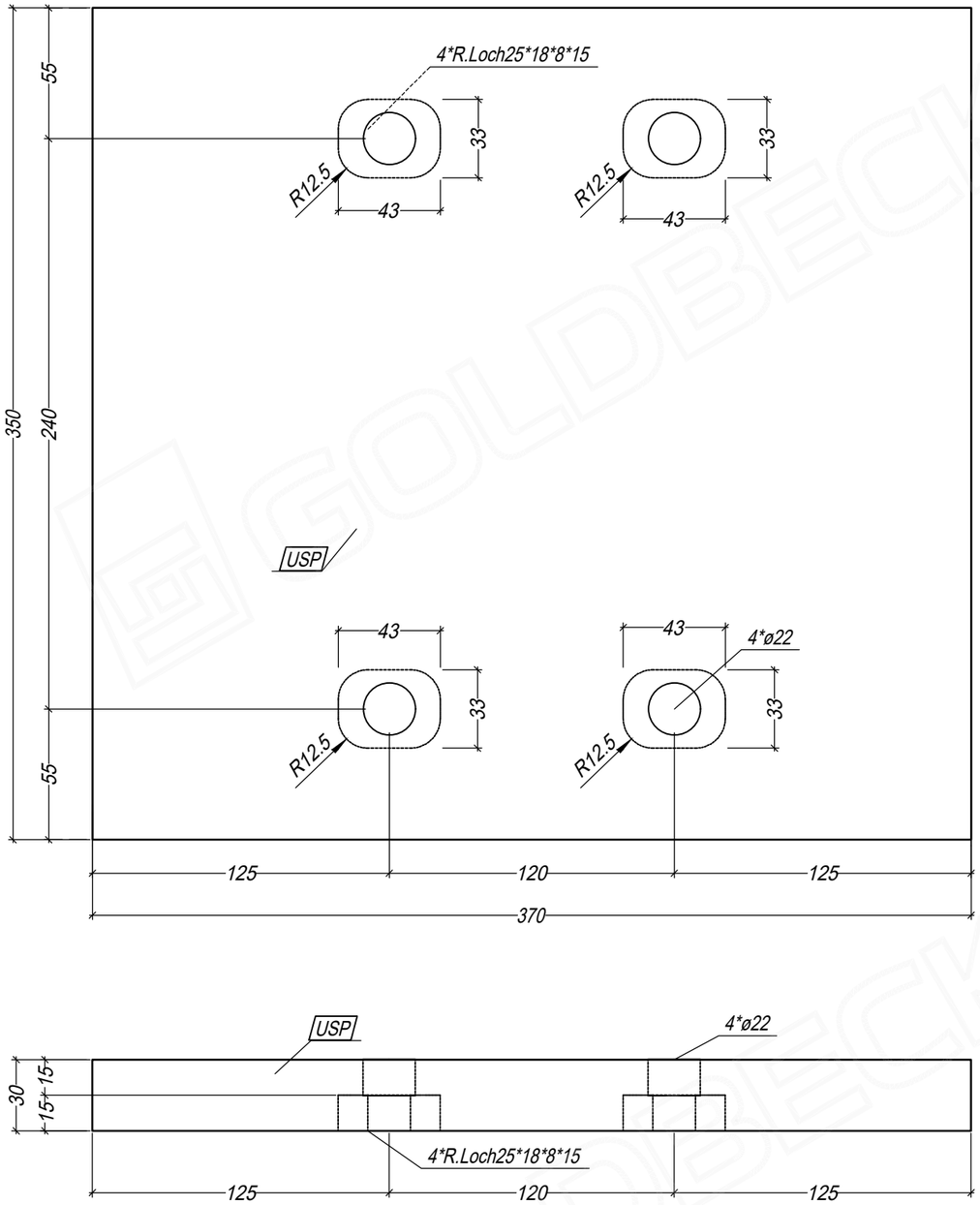
gezeichnet	25.02.2022
geprüft	25.02.2022
Maßstab	1:5
DWG–Name	GW78600–78699.dwg

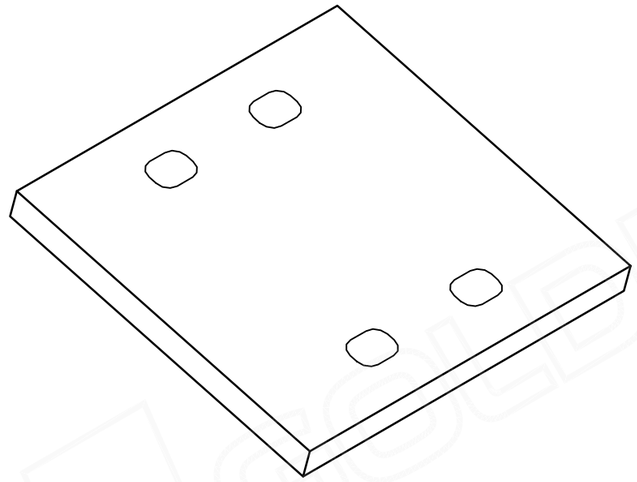
Zusatz			
12; 23; 25; 27	(27)		c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G41	78669	–

Blattgröße: 210mm x 297mm

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*Blech ist nach EN 10164 durch **UT** (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
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*Vorspannung:siehe Übersichtszeichnung
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*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78671	FR	1	FL350*30	370	S355J2	UT (USP)
Stahlgewicht:	30,50 kg	Mantelfläche:	0,281 m²	Versandmaße:	(L x H x B)	370*350*30
Beschichtung:	–				Gesamtgewicht (kg):	30,50 kg

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummeiner Str. 4–6, 33649 Bielefeld

FR | Fräsplatte 370*350mm, t=30mm UP zu GW78669

gezeichnet 25.02.2022 [redacted]
geprüft 25.02.2022 T[redacted]
Maßstab 1:2.5
DWG-Name GW78600–78699.dwg

Zusatz			
23; 25; 27	(27)	G75	c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G41	78671	–

Blattgröße: 420mm x 297mm

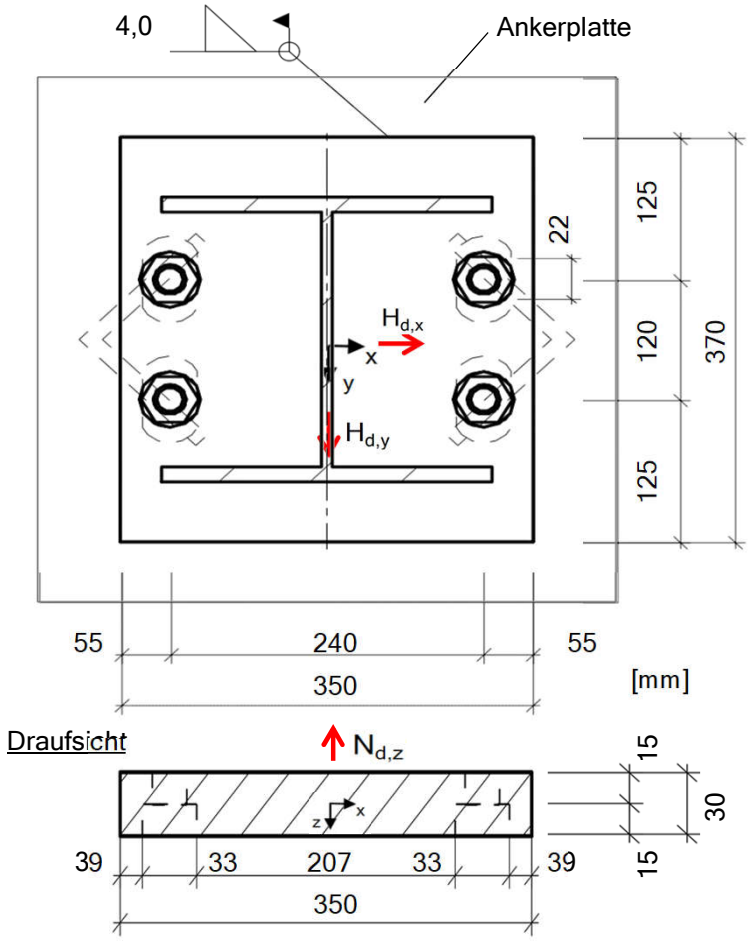
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FRÄSPLATTENBEMESSUNG (EN 1993-1-8:2010-12)

2022-04-29

Auftragsnummer: Grenzlast (projektbez. Last: s. Statik) Positionsnummer: Versandpos. 78669 (=78671)

Nationales Anwendungsdokument: Sonstige (EU)



Draufsicht

Schnitt (durch Fräsung)

Vertikalkraft	$N_{d,z}$:	375,0 kN
Horizontalkraft	$H_{d,x}$:	300,0 kN
Horizontalkraft	$H_{d,y}$:	135,0 kN
Montagenaht	$a_{Fräspl.}$:	4,0 mm
Verbindungsmittel:		M20, 10.9
Werkstoff der Platte:		S 355
Lochabstand	p_x:	24,0 cm
Lochabstand	p_y:	12,0 cm
Fräsplattenbreite	x_F:	35,0 cm
Fräsplattenlänge	y_F:	37,0 cm
Fräsplattendicke	t_P:	30 mm
Ankerplattentyp:		
Ankerplatte mit Schubwinkel (in X)		
Werkstoff der Ankerplatte:		S 355

1. Nachweis der Schweißnaht

$\sigma_{v,w,d} \overset{!}{\leq} f_{vw,d}$ erfüllt: 13,47 kN/cm² < 22,33 kN/cm²

$\tau_{w,d,x} \overset{!}{\leq} f_{vw,d}$ erfüllt: 10,71 kN/cm² < 22,33 kN/cm²

mit $A_{w,z}$ = 29,60 cm², $A_{w,y}$ = 29,60 cm², $A_{w,x}$ = 28,0 cm²

2. Nachweis der Schraubenverbindung

$F_{v,Ed} \overset{!}{\leq} F_{v,Rd}$ erfüllt: 82,3 kN < 139,6 kN

$F_{t,Ed} \overset{!}{\leq} F_{t,Rd}$ erfüllt: 93,8 kN < 163,3 kN

$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1,4 \cdot F_{t,Rd}} \overset{!}{\leq} 1$ erfüllt: 1 = 1

3. Nachweis der Fräsplatte (einachsig in x-Richtung gespannte Platte)

$\sigma_d \overset{!}{\leq} \sigma_{R,d}$ erfüllt: 16,14 kN/cm² < 31,36 kN/cm²

mit $M_{d,y}$ = 1031,3 kNcm, $W_{pl,y}$ = 63,9 cm³

1. Nachweis der Schweißnaht

1.1 Beanspruchbarkeit

$$f_{vw,d} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = \frac{47{,}0\ \text{kN/cm}^2}{\sqrt{3} \cdot 0{,}90 \cdot 1{,}35} = 22{,}33\ \text{kN/cm}^2$$

1.2 Beanspruchung

1.2.1 Beanspruchung in y- und z-Richtung

$$\sigma_{w,d,z} = \frac{N_{d,z}}{A_{w,z}} = \frac{N_{d,z}}{2 \cdot a \cdot y_F} = \frac{375{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 37{,}0\ \text{cm}} = \frac{375{,}0\ \text{kN}}{29{,}6\ \text{cm}^2} = 12{,}67\ \text{kN/cm}^2$$

$$\tau_{w,d,y} = \frac{H_{d,y}}{A_{w,y}} = \frac{H_{d,y}}{2 \cdot a \cdot y_F} = \frac{135{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 37{,}0\ \text{cm}} = \frac{135{,}0\ \text{kN}}{29{,}6\ \text{cm}^2} = 4{,}56\ \text{kN/cm}^2$$

$$\sigma_{v,w,d} = \sqrt{\sigma_{w,d}^2 + \tau_{w,d}^2} = 13{,}47\ \text{kN/cm}^2$$

1.2.2 Beanspruchung in x-Richtung

$$\tau_{w,d,x} = \frac{H_{d,x}}{A_{w,x}} = \frac{H_{d,x}}{2 \cdot a \cdot x_F} = \frac{300{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 35{,}0\ \text{cm}} = \frac{300{,}0\ \text{kN}}{28{,}0\ \text{cm}^2} = 10{,}71\ \text{kN/cm}^2$$

1.3 Nachweise

$\sigma_{v,w,d} \overset{!}{\leq} f_{vw,d}$ erfüllt: $13{,}47\ \text{kN/cm}^2 < 22{,}33\ \text{kN/cm}^2$

$\tau_{w,d,x} \overset{!}{\leq} f_{vw,d}$ erfüllt: $10{,}71\ \text{kN/cm}^2 < 22{,}33\ \text{kN/cm}^2$

2. Nachweis der Schraubenverbindung

2.1 Abscheren

2.1.1 Beanspruchung

Komponenten der Schraubenkräfte aus $V_{d,z}$

$$V_{Vd,z} = \frac{H_{d,y}}{n} = \frac{135{,}0\ \text{kN}}{4} = 33{,}8\ \text{kN}$$

Komponenten der Schraubenkräfte aus $V_{d,y}$

$$V_{Vd,y} = \frac{H_{d,x}}{n} = \frac{300{,}0\ \text{kN}}{4} = 75{,}0\ \text{kN}$$

Maximale Schraubenkraft

$$F_{v,Ed} = \sqrt{(V_{Vd,z})^2 + (V_{Vd,y})^2} = \sqrt{(33{,}8\ \text{kN})^2 + (75{,}0\ \text{kN})^2} = 82{,}3\ \text{kN}$$

2.1.2 Beanspruchbarkeit

$$F_{v,Rd} = \frac{\alpha_v \cdot A \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}6 \cdot 3{,}14\ \text{cm}^2 \cdot 100{,}0\ \text{kN/cm}^2}{1{,}35} = 139{,}6\ \text{kN}$$

2.1.3 Nachweis

$F_{v,Ed} \overset{!}{\leq} F_{v,Rd}$ erfüllt: $82{,}3\ \text{kN} < 139{,}6\ \text{kN}$

2.2 Schraubenzugkraft

2.2.1 Beanspruchbarkeit

$$F_{t,Rd} = \frac{k_2 \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}9 \cdot 2{,}45\ cm^2 \cdot 100{,}0\ kN/cm^2}{1{,}35} = 163{,}3\ kN$$

2.2.2 Beanspruchung

$$F_{t,Ed} = \frac{N_{d,z}}{n} = \frac{375{,}0\ kN}{4} = 93{,}8\ kN$$

2.2.3 Nachweis

$$F_{t,Ed} \overset{!}{\leq} F_{t,Rd} \qquad \text{erfüllt:} \qquad 93{,}8\ kN < 163{,}3\ kN$$

2.3 Zug und Abscheren

2.3.1 Beanspruchung

$F_{v,Ed} = 82{,}3\ kN$ $F_{t,Ed} = 93{,}8\ kN$

2.3.2 Beanspruchbarkeit

$F_{v,Rd} = 139{,}6\ kN$ $F_{t,Rd} = 163{,}3\ kN$

2.3.3 Nachweis

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} \overset{!}{\leq} 1 \qquad \text{erfüllt:} \qquad 1 = 1$$

3. Nachweis der Fräsplatte (einachsig in x-Richtung gespannte Platte)

3.1 Schnittgrößen

$$M_{d,y} = \frac{N_{d,z}}{2} \cdot \frac{x_F - p_x}{2} = \frac{375{,}0\ kN}{2} \cdot \frac{35{,}0\ cm - 24{,}0\ cm}{2} = 1031{,}3\ kNcm$$

3.2 Querschnittswerte

$$W_{pl,y} = \frac{(y_F - 2 \cdot a_{y,Fräs.}) \cdot t_P^2}{4} = \frac{(37{,}0\ cm - 2 \cdot 4{,}3\ cm) \cdot (3{,}0\ cm)^2}{4} = 63{,}9\ cm^3$$

Konstruktionsdetails zur Fräsung: s. Abschnitt 4.

3.3 Beanspruchbarkeit

$$\sigma_{R,d} = \frac{f_y}{\gamma_{M0}} = \frac{34{,}5\ kN/cm^2}{1{,}10} = 31{,}36\ kN/cm^2$$

3.4 Beanspruchung

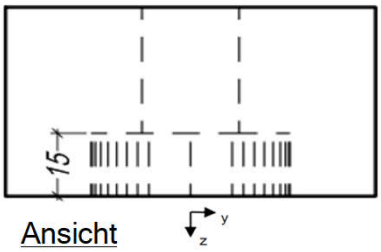
$$\sigma_d = \frac{M_{d,y}}{W_{pl,y}} = \frac{1031{,}3\ kNcm}{63{,}9\ cm^3} = 16{,}14\ kN/cm^2$$

3.5 Nachweis

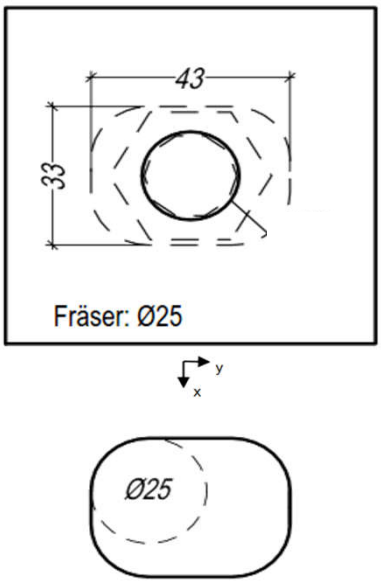
$$\sigma_d \overset{!}{\leq} \sigma_{R,d} \qquad \text{erfüllt:} \qquad 16{,}14\ kN/cm^2 < 31{,}36\ kN/cm^2$$

4. Konstruktionsdetails

Verbindungsmittel: M20, 10.9



Ansicht



Draufsicht



Ausführungsdetail der Fräsung

Pos 78578 Teilflächenbelastung für Fußplatte 320 x 300 x 25 [mm]

BOCAD - TYP 12 | Fußplatte Pos. 78578, t = 25 mm, S355 | Fräsplatte Pos. 78669, t = 30 mm, S355

v20.12

EINGANGSWERTE

Baustoffe
Beton C30/37 → f_{cd} = 17,0 N/mm²

Belastung
F_{Ed} = 2975,0 kN

Lastausmitte
e = 0 mm

unterstützendes Bauteil
a = 500 mm
b = 2500 mm

Fuß- bzw. Zwischenplatte
a_0 = 370 mm
b_0 = 350 mm

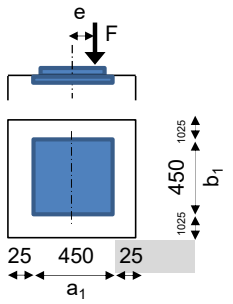
Ankerplatte Pos. 78632
a_1 = 450 mm
b_1 = 450 mm
t_1 = 15 mm

Verteilungsfläche
a_2 = 500 mm
b_2 = 477 mm

BEMESSUNG

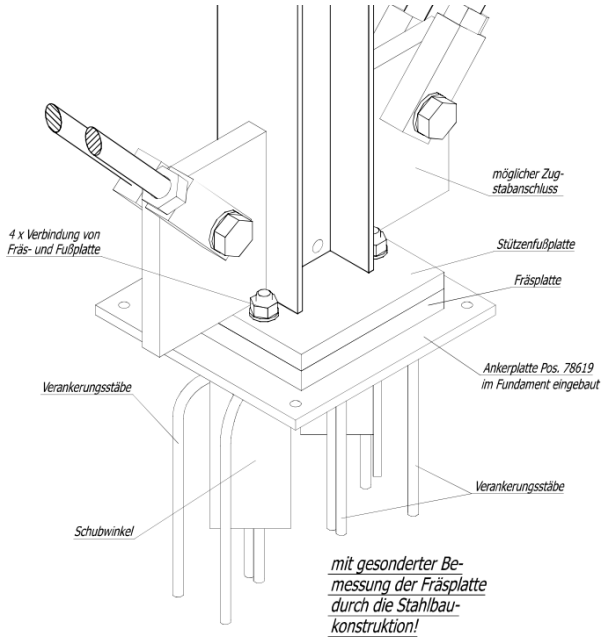
Es werden eine gleichmäßige Pressung unterseitig der Fuß- bzw. Zwischenplatte sowie ein Lastausbreitungswinkel 1:2 innerhalb der Ankerplatte unterstellt.
Unterseitig der Ankerplatte ergibt sich die Belastungsfläche zu:

a_1' = 430 mm
b_1' = 410 mm

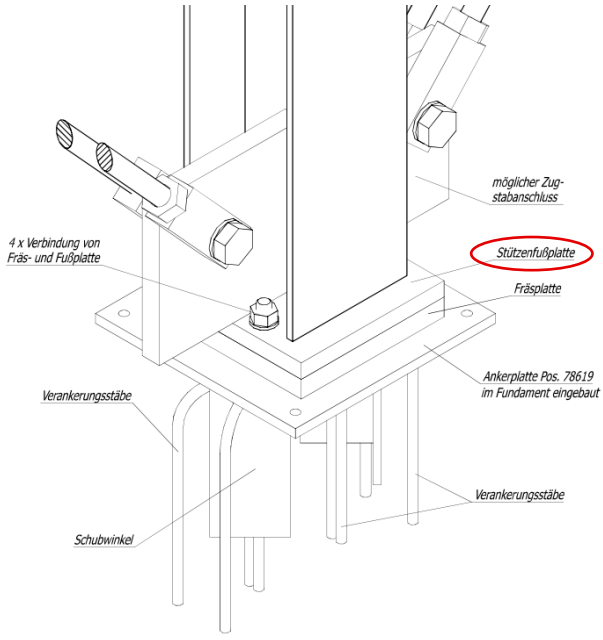


F_{Ed} = 2975 kN < 3485 kN = F_{Rdu}

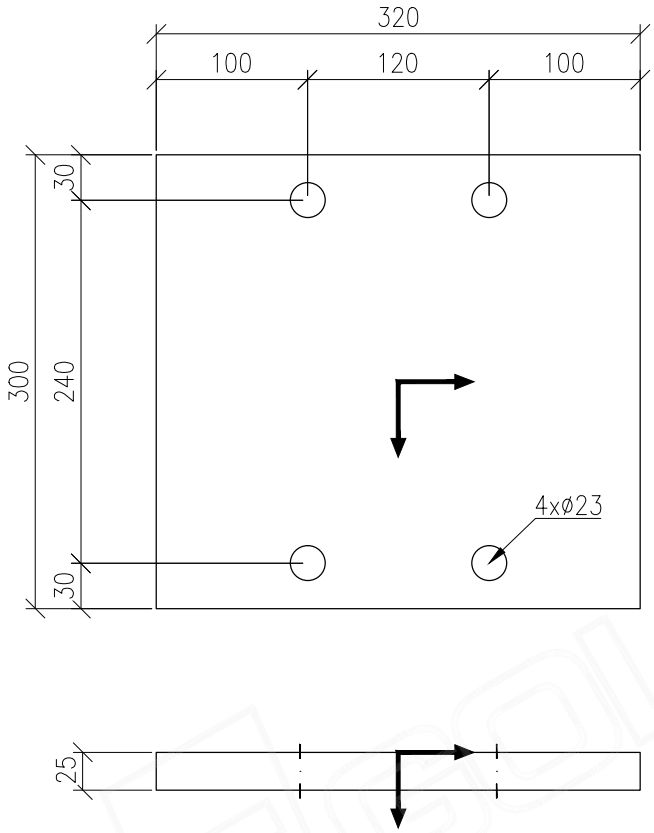
A_{c0} = 176300 mm²
$a_{1,red}$ = 430 mm → a_2, b_2
$A_{c0,red}$ = 176300 mm²
$\sigma_{Ed,0}$ = = 16,9 N/mm² < 17,0 N/mm² = f_{cd} → kein Nachweis der Teilflächenbelastung erforderlich



Einbausituation (gedrehte Stütze, Spezialfall)



Einbausituation (Regelfall)



o. w. Nachweis

*Blech ist nach EN 10164 durch
UT (Ultraschallprüfung)
im Raster 200x200mm zu prüfen

*Werkstoff: siehe Stückliste
*Schrauben:
KM..*..4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAZM..*..8.8 nach EN ISO 4017, 4032, 7089
KHVM..*..10.9 nach DIN EN 14399-4/-6
*Vorspannung: siehe Übersichtszeichnung
*Lochspiel: 2mm
*Vorbereitungsgrad P1 nach ISO 8501-3
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur bei S355 (C_{AE} >0.42%
Materialdicke: 25...40mm 120°C
>40mm 180°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78578	Fußplatte	s. Stückliste	FL300x25	320	S355J2	Ultraschallprüfung
Stahlgewicht:	18,84 kg	Mantelfläche:	0,21 m²	Versandmaße:	(H x B x L)	300 x 25 x 320
Beschichtung:	–				Gesamtgewicht (kg):	18,84

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Fußplatte
300mm x 320mm t=25mm

gezeichnet 18.10.2012
geprüft
Maßstab 1:5
DWG-Name

a	22.05.2015	Weisters / Wisotzky	Stückliste	
Zusatz		(27)	G10	c19
		Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

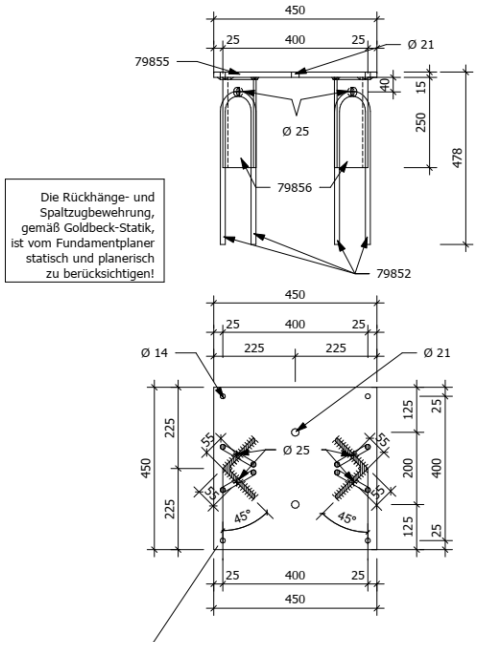
Typ	Tabelle 2	Normteilnummer	Index
P	G42	78578	a

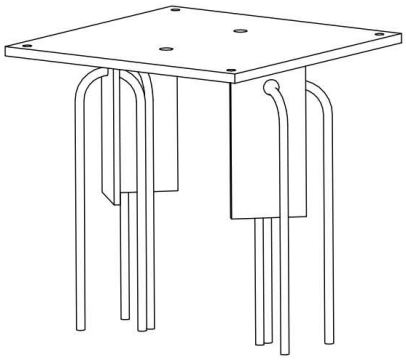
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Regeldetail Zulagebewehrung AP 78619 - H5

(gilt nur als Vorgabe für die Gründungsplanung)

- Die endgültigen Biegeformen und Stückzahlen sind in der Gründungsplanung anzugeben, in den Stahllisten aufzuführen und ausschließlich diesen zu entnehmen!
- Die Zulage-Eisen sind KEIN Bestandteil der Ankerplatten und es erfolgt keine Lieferung von GOLDBECK!
- Diese Details umfassen nicht die Spaltzugbewehrung infolge Vertikallast. Diese ind vom Gründungsstatiker zu bestimmen!

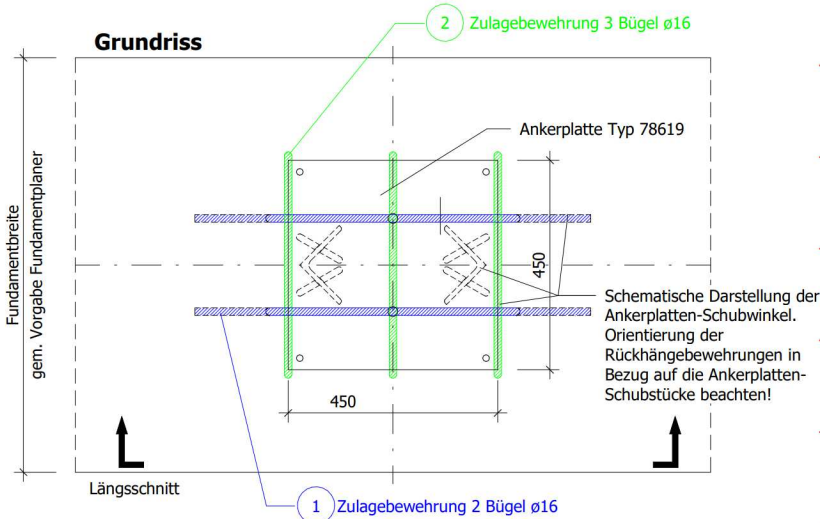


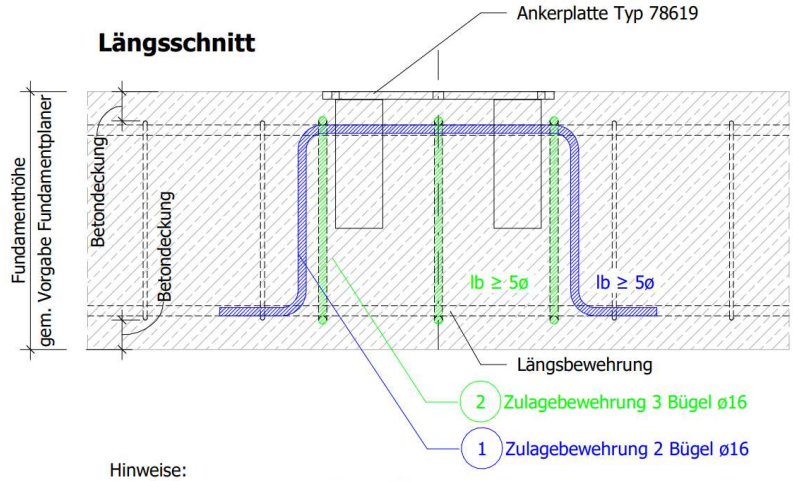


Typenprüfung T20-041

Bemessungskräfte	
$H_{y,Rd}$	180 kN
$H_{z,Rd}$	180 kN
Z_{Rd}	240 kN
Die Bemessungskräfte können gleichzeitig wirken!	

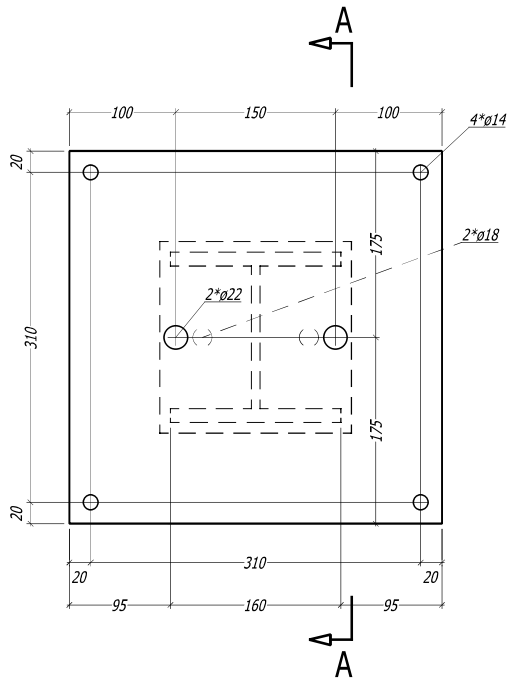
Für AP 78619

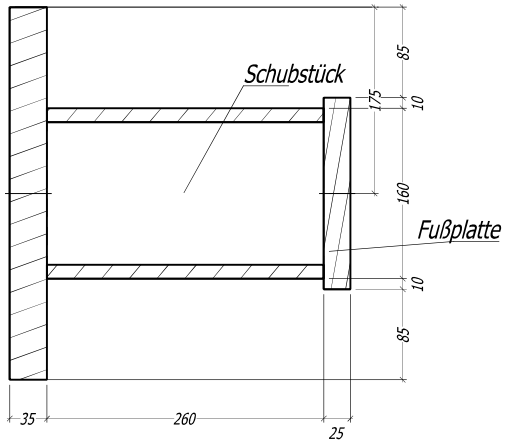




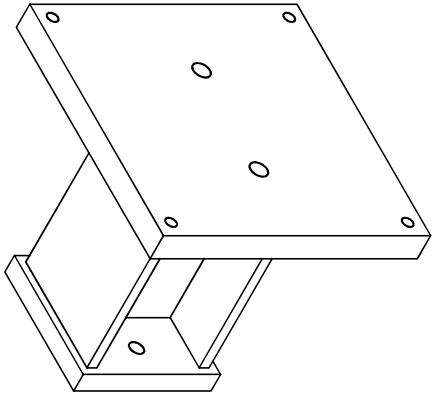
Hinweise:

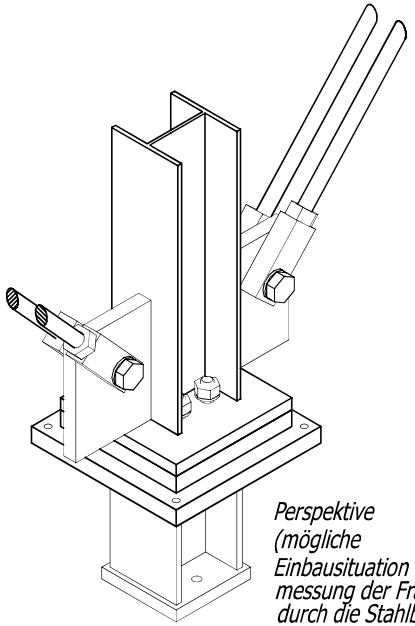
- Vorhandene Bewehrung (Bügel) kann angerechnet werden!
- Bei Bodenplatten ist die Ausführung der Bügel gegebenefalls anzupassen.



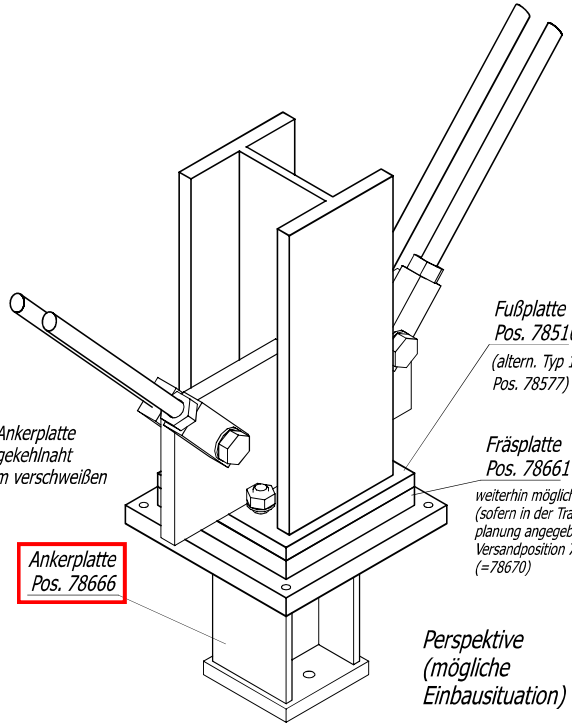


Schnitt A-A





Perspektive
(mögliche
Einbausituation mit neuer Bemessung der Fräsplatte
durch die Stahlbaukonstruktion)

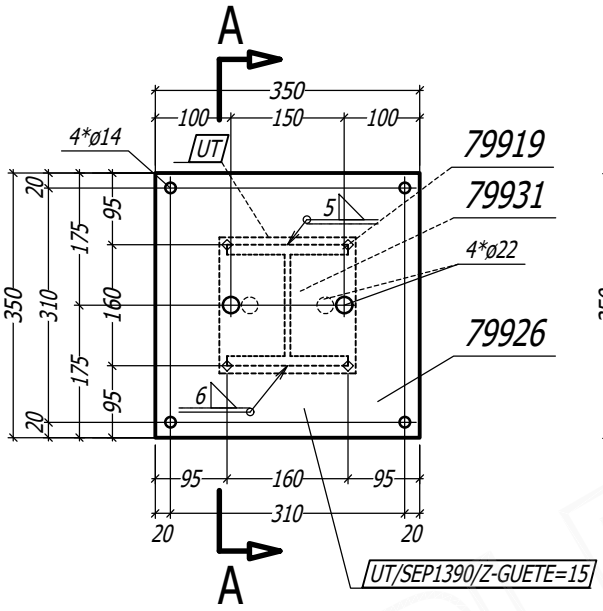


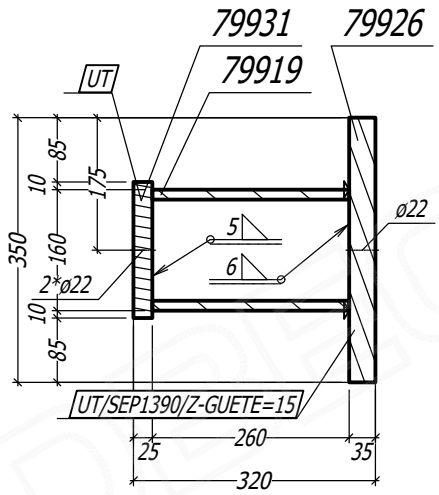
Perspektive
(mögliche
Einbausituation)

Einbaurichtung des Schubstückes in Abhängigkeit der Verbandsrichtung beachten!

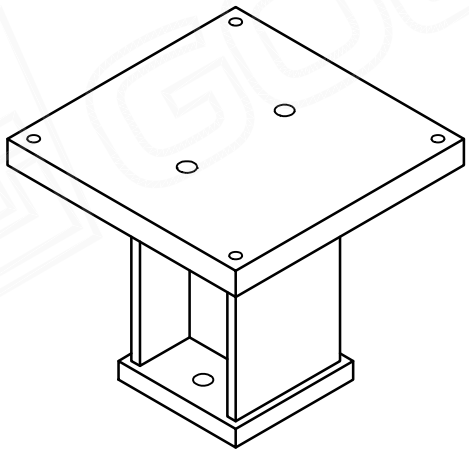
Planinhalt:	Phase	Planer	Art
Hauptposition 78666 AP - ANKERPLATTE -	5	WP	FE
	Ebene	Laufende-Nr.:	Index
	XX	78666	-

bocad-3d, Rev. 24.1.82/8





Schnitt 78666 , A-A



roh

*Blech ist nach EN 10164 durch UT (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung:siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3 für Standardbeschichtung
*Vorbereitungsgrad P3 nach ISO 8501-3 für Sonderbeschichtung
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78666	AP	s.Stückliste	NormBGR		S355J2	
79919	SST	1	HEB160	260	S355J2	
79926	AP	1	BL35*350	350	S355J2	UT (USP)
79931	PL	1	FL180*25	180	S355J2	UT (USP)
Stahlgewicht:	51,10 kg	Mantelfläche:	0,583	Versandmaße:	(L x H x B)	350*320*350
Beschichtung:	–				Gesamtgewicht (kg):	51,10 kg

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

AP | Ankerplatte
350 x 350 mm mit Schubstück HEB160
für Parkhausstützen, Systemfußpunkte

gezeichnet	24.11.2022
geprüft	24.11.2022
Maßstab	1:10
DWG-Name	GW78600-78699.dwg

Zusatz			
125	(27)		c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G40	78666	–

Blattgröße: 210mm x 297mm

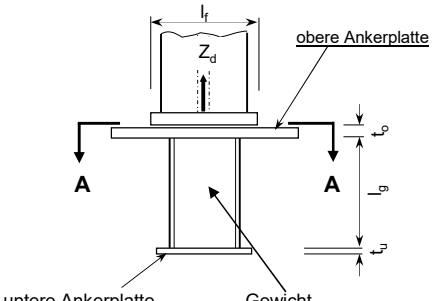
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Pos. 78666 Ankerplatte mit Schubstück

Belastung

maximal gleichzeitig wirkende Kräfte (mit Lastfaktoren):

max Z_d	=	(abhebend)		=	396,0 kN
max $H_{y,d}$	=	(Flanschebene)		=	176,0 kN
max $H_{z,d}$	=	(Stegebene)	Verbandsrichtung	=	256,0 kN



Bauteile

Alle Nachweise sind erfüllt.

Schubstück: HEB 160 - S 355

Länge Profil (tatsächliche): l_g = 26,0 cm

max. Länge Fußplatte Stütze ≤ l_f = 30,0 cm

obere Ankerplatte bo*lo*to: S 355

Dicke Ankerplatte:	t_o =	3,5 cm
Länge Ankerplatte:	l_o =	35,0 cm
Breite Ankerplatte:	b_o =	35,0 cm
Flanschnaht:	a_F =	5,5 mm
Stegnaht:	a_{St} =	5,5 mm

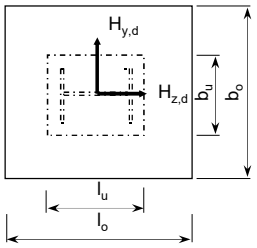
untere Ankerplatte bu*lu*tu: S 355

Dicke Ankerplatte:	t_u =	2,5 cm
Länge Ankerplatte:	l_u =	18,0 cm
Breite Ankerplatte:	b_u =	18,0 cm
Flanschnaht:	a_F =	4,5 mm
Stegnaht:	a_{St} =	4,5 mm

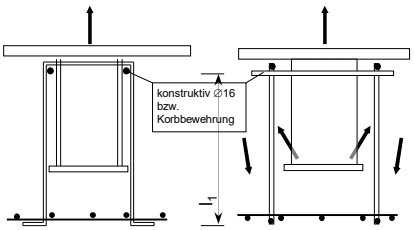
Betondruckfestigkeitsklasse (Einbauort): C30/37

=> Ausführung als Ortbeton

A - A:



Rückverankerung
Betonstabstahl B500 gew.: 6 Bü ∅ 12



zur Anordnung der Rückverankerung

Höhe/Tiefe des Betons: h_F = 60 cm

Ausführungsklasse nach DIN EN 1090-2: EXC2

Profilkennwerte:

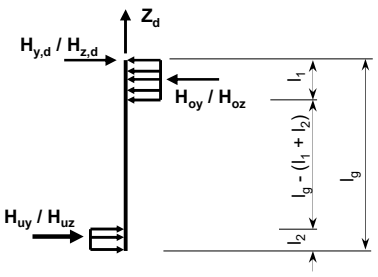
Profilhöhe:	h =	16,0 cm	starke Achse:	I_y =	2490 cm^4
red. Steghöhe:	$d = h-2*(t_f+r)$ =	10,4 cm	schwache Achse:	I_z =	889 cm^4
Profilbreite:	b =	16,0 cm	starke Achse:	W_y =	311 cm^3
Flanschdicke:	t_f =	1,3 cm	schwache Achse:	W_z =	111 cm^3
Stegdicke:	t_w =	0,8 cm	Fläche:	A =	54,3 cm^2
Walzradius:	r =	1,5 cm	Stegfläche inkl. Flanschanteil:	A_V =	17,6 cm^2

Nachweise:

statische Hilfswerte:

Länge oberer Druckbereich:	l_1 =	14,0 cm
Länge unterer Druckbereich:	l_2 =	3,5 cm
$H_{oy} = [H_{y,d}*(l_g-l_2/2) / (l_g-(l_1+l_2)/2)]$ =		247,4 kN
$H_{uy} = H_{oy} - H_{y,d}$ =		71,4 kN
$H_{oz} = [H_{z,d}*(l_g-l_2/2) / (l_g-(l_1+l_2)/2)]$ =		359,9 kN
$H_{uz} = H_{oz} - H_{z,d}$ =		103,9 kN

statisches System für $H_{y,d}$, $H_{z,d}$ und Z_d:

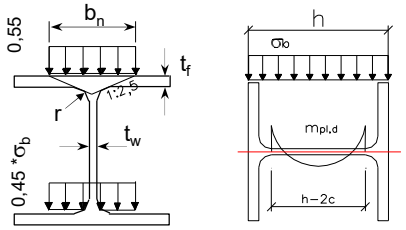


1. Betonpressung

Betongüte: C30/37 f_{cd} = 1,70 kN/cm²

γ_C = 1,5 Ortbeton

mitwirkende Breite: α_{cc} = 0,85



in z-Richtung Verteilung auf beide Flansche
infolge elastischer Stauchung des Betons (vgl. Skizze):

$\Rightarrow \bar{b}_n$ = (tw+1,6r+5tf)/0,55 = 17,6 cm

in y-Richtung Ansatz der gesamten Profilhöhe h (vgl. Skizze):

Betonpressung oben

infolge	H_{oz}:	$\sigma_{cd,o}$ =	$H_{oz}/(\bar{b}_n * I_1)$ =	1,46	kN/cm²	<	1,70	kN/cm²	= f_{cd}
infolge	H_{oy}	$\sigma_{cd,o}$ =	$H_{oy}/(h * I_1)$ =	1,10	kN/cm²	<	1,70	kN/cm²	= f_{cd}

Betonpressung unten

infolge	H_{uz}	$\sigma_{cd,u}$ =	$H_{uz}/(\bar{b}_n * I_2)$ =	1,68	kN/cm²	<	1,70	kN/cm²	= f_{cd}
infolge	H_{uy}	$\sigma_{cd,u}$ =	$H_{uy}/(h * I_2)$ =	1,28	kN/cm²	<	1,70	kN/cm²	= f_{cd}

2. Rückverankerung

Betonstabstahl B500 mit $f_{y,d} = f_{y,k}/\gamma_s$ = 43,48 kN/cm² (mit γ_s = 1,15)

gew.: 6 Bü ∅ 12 mit $A_{s,vorh}$ = 13,57 cm²

Z_d = - 396,00 kN $A_{s,erf} = Z_d / f_{y,d}$ = 9,11 cm²

$A_{s,erf}$ = 9,11 cm² < 13,57 cm² = $A_{s,vorh}$

Verankerungslänge

Betongüte: C30/37 f_{bd} = 3,04 N/mm²

γ_C = 1,5 Ortbeton

α_{cc} = 0,85

guter Verbund

α_1 = 0,7

und α_2; α_3; α_4; α_5 = 1,0

$l_{b,rqd} = \phi_s/4 * f_{yd}/f_{bd}$	43	cm
$l_{b,min} \geq \max\{0,3 * l_{b,rqd}; 10 * \phi_S; 100mm\}$ =	13	cm
$l_{bd} = \alpha_1 * (A_{s,erf}/A_{s,vorh}) * l_{b,rqd}$ =	21	cm
$l_{bd,max} = \min\{l_g - 4*\phi_S; h_F - 5 - l_g - 4*\phi_S - t_o\}$ =	21	cm

l_{bd} = 21 cm < 21 cm = $l_{bd,max}$

Anmerkung:
Die Rückverankerungsschlaufen sind bis zur unteren Fundamentbewehrung durchzuführen!

3. Profil

Anmerkung zur Querschnittsklassifizierung nach DIN EN 1993-1-1 (5.5.2):
Eine Querschnittsklassifizierung für das einbetonierte Stahlprofil ist hinfällig. Jegliches örtliche Beulen des Querschnitts wird durch den umgebenden Beton verhindert.

3a. Nachweis um die starke Achse

Bemessungsschnittgrößen:

max. Spannungen am Rundungsbeginn im Steg:

Z_d =	396,0	kN	$\sigma_{x,Ed} = Z_d/A + (M_{y,d}/W_y) * (d/h)$	11,04	kN/cm²
$H_{z,d}$ =	256,0	kN	$\sigma_{z,Ed} = -0,55 * H_{z,d}/(I_1 * t_w)$	-12,57	kN/cm²
$M_{y,d}$ = $H_{z,d} * I_1/2$ =	1792,0	kNcm	$\tau_{Ed} = H_{z,d}/Av$	14,51	kN/cm²

$\Rightarrow \sigma_v = (\sigma_{x,Ed}^2 + \sigma_{z,Ed}^2 - (\sigma_{x,Ed} * \sigma_{z,Ed}) + 3\tau_{Ed}^2)^{1/2}$ = 32,41 kN/cm² < 35,5 kN/cm² = $\sigma_{Rd} = f_y/\gamma_{M0}$

mit γ_{M0} = 1,00

3b. Nachweis um die schwache Achse

Z_d =	396,0	kN	$\sigma_{x,Ed} = Z_d/A + M_{z,d}/W_z$	18,39	kN/cm²
$H_{y,d}$ =	176,0	kN	$\tau_{Ed} = 1,5 * (H_{y,d}/2)/(b * t_f)$	6,35	kN/cm²
$M_{z,d}$ = $H_{y,d} * I_1/2$ =	1232	kNcm			

$\Rightarrow \sigma_v = (\sigma_{x,Ed}^2 + 3\tau_{Ed}^2)^{1/2}$ = 21,43 kN/cm² < 35,5 kN/cm² = $\sigma_{Rd} = f_y/\gamma_{M0}$

mit γ_{M0} = 1,00

Stegnachweis (mit W_{pl}):

$\sigma_{y,d}$ =	$H_{y,d}/(I_1 * h)$ =	0,79	kN/cm²
$m_{pl,Ed}$ =	$\sigma_{y,d} * d^2/16$ =	5,31	kNcm/cm
$W_{pl,Steg}$ =	$t_w^2/4$ =	0,16	cm³/cm

$\Rightarrow \sigma_{steg} = m_{pl,Ed}/W_{pl,Steg}$ = 33,20 kN/cm² < 35,5 kN/cm² = $\sigma_{Rd} = f_y/\gamma_{M0}$

mit γ_{M0} = 1,00

4. Schweißnähte an oberer Ankerplatte

Vereinfachtes Verfahren nach DIN EN 1993-1-8 (4.5.3.3) auf Spannungsniveau

Steg: a_{St} = 5,5 mm (beidseitig zur Aufnahme von Hz)

$A_{w,Hz}$ = $2*a_{St}*(d+2r)$ = 14,7 cm²

$\Rightarrow \tau_{w,Hz}$ = $H_{z,d} / Aw_{,Hz}$ = 17,37 kN/cm² < 25,15 kN/cm² = $f_{vw,d}$

mit γ_{M2} = 1,25

Flansche: a_F = 5,5 mm (innen + außen zur Aufnahme von Hy und Z)

$A_{w,Hy+Z}$ = $2*a_F*[b+(b-2r-t_w)]$ = 31,0 cm²

$\Rightarrow \sigma_{w,Ed}$ = $[(Z_d/A_{v,Hy+Z})^2 + (H_{y,d}/A_{w,Hy+Z})^2]^{1/2}$ = 13,97 kN/cm² < 25,15 kN/cm² = $f_{vw,d}$

mit γ_{M2} = 1,25

5. Ankerplatte oben

Plattenbiegung infolge abhebender Last z_d:

$M_{Ed} = Z_d /2 * (l_r-h)/2$ = 1386 kNcm

$M_{pl,Rd} = b *(t_o)^2/4*(f_y/\gamma_{M1})$ = 1581 kNcm

mit γ_{M1} = 1,10

M_{Ed} = 1386 kNcm < 1581 kNcm = $M_{pl,Rd}$

6. Schweißnähte an unterer Ankerplatte

Vereinfachtes Verfahren nach DIN EN 1993-1-8 (4.5.3.3) auf Spannungsniveau

Steg: a_{St} = 4,5 mm (beidseitig konstruktiv o. w. N.)

Flansche: Die abhebende Last Zd wird durch die Flanschnähte aufgenommen.

a_F = 4,5 mm (innen + außen)

$A_{W,Z}$ = $2*a_F*[b+(b-2r-t_s)]$ = 25,4 cm²

$\Rightarrow \sigma_{w,Ed}$ = $Z_d /A_{W,Z}$ = 15,60 kN/cm² < 25,15 kN/cm² = $f_{vw,d}$

mit γ_{M2} = 1,25

7. Ankerplatte unten

Plattenbiegung infolge abhebender Last z_d:

$\Rightarrow \sigma_{cd,Z}$ = $Z_d / [(b*l_u)-A]$ = 1,69 kN/cm² < 1,70 kN/cm² = f_{cd}

Plattenbiegung infolge abhebender Last z_d:

Platte kragt, bezogen auf Mitte Flansch über :

l_{krag} = $(l_u-(h-t_f))/2$ = 1,65 cm

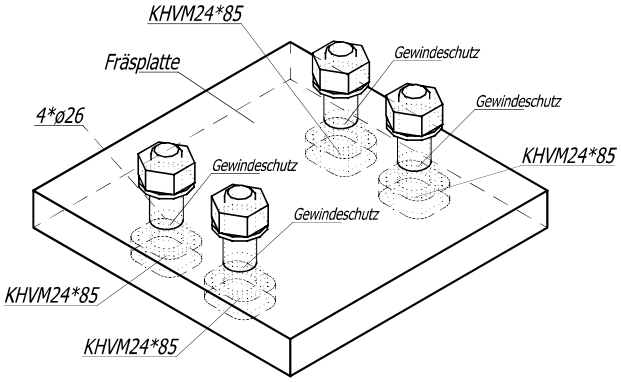
$M_{Ed,krag}$ = $\sigma_{cd,Z}*b*[(l_{krag})^2/2]$ = 37 kNcm

$M_{Ed,feld}$ = $[\sigma_{cd,Z}*b*(h-t_f)^2/8]-M_{Ed,krag}$ = 695,41 kNcm

$M_{pl,Rd}$ = $b *(t_u)^2/4*(f_y/\gamma_{M1})$ = 807 kNcm

mit γ_{M1} = 1,10

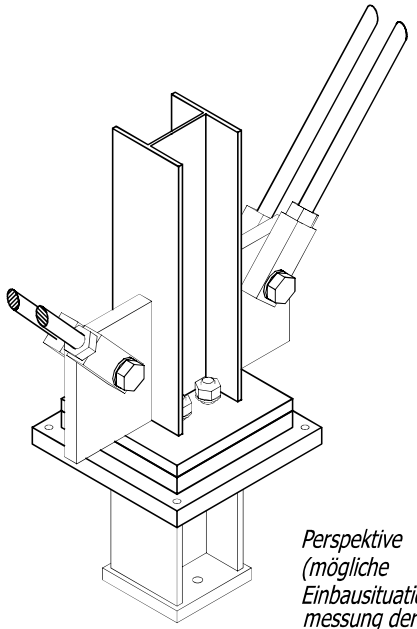
max M_{Ed} = 695 kNcm < 807 kNcm = $M_{pl,Rd}$



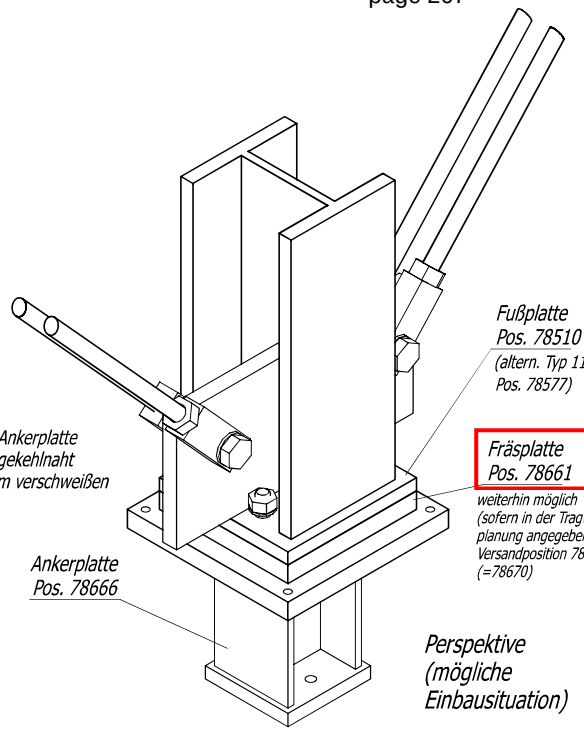
Fräsplatte mit Gewindeschutz und Verbindungsmittel KHV M24x85 versandfertig ausstatten



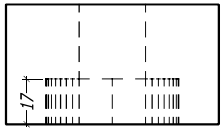
Ausführungsdetail der Fräsung



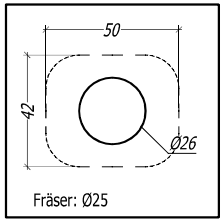
Perspektive (mögliche Einbausituation mit neuer Bemessung der Fräsplatte durch die Stahlbaukonstruktion)



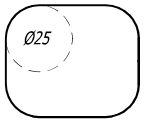
Perspektive (mögliche Einbausituation)



Ansicht

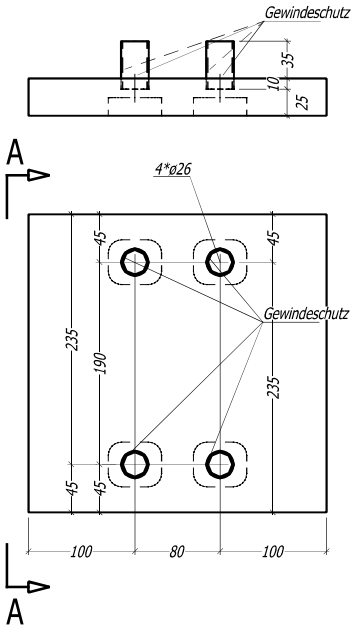


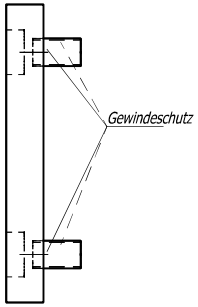
Draufsicht



Draufsicht Fräskörper

M24



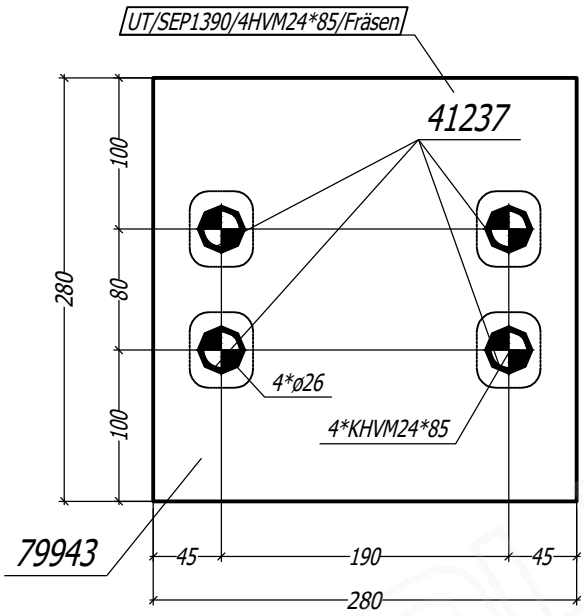


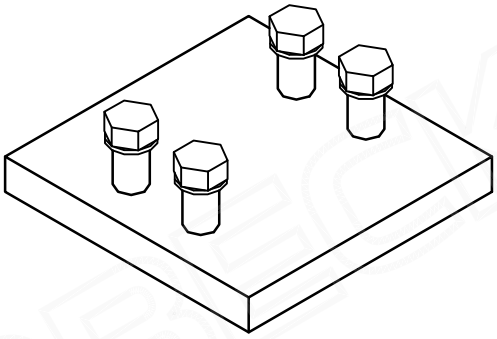
Ansicht von links, A-A

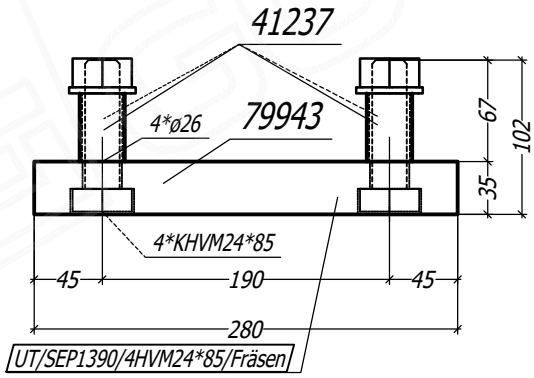
Einbaurichtung des Schubstückes in Abhängigkeit der Verbandsrichtung beachten!

Planinhalt:	Phase	Planer	Art
Hauptposition 78661 FR | Fräsplatte	5	WP	FE
	Ebene	Laufende-Nr.:	Index
	XX	78661	-

bocad-3d, Rev. 24.1.82/8







roh

*Blech ist nach EN 10164 durch UT (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung:siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3 für Standardbeschichtung
*Vorbereitungsgrad P3 nach ISO 8501-3 für Sonderbeschichtung
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78661	FR	s.Stückliste	NormBGR		S355J2	
41237	RO	4	ROHR30*1.5	45	PE	
79943	FR	1	FL280*35	280	S355J2	UT (USP)
Stahlgewicht:	20,52 kg	Mantelfläche:	0,192 m²	Versandmaße:	(L x H x B)	280*102*280

Stückzahl	Schraube	DIN	Werkstoff	Gewicht	Bemerkung
4	KHVM24*85	EN 14399-4/-6	10.9 tZn	2,46	Werkstatt
Schraubengewicht:				2,46 kg	
Beschichtung:	–			Gesamtgewicht (kg):	22,98 kg

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

FR | Fräsplatte (Stiftplatte) 280*280mm, t=35mm mit 4 HV-Schrauben M24*85

gezeichnet	24.11.2022
geprüft	24.11.2022
Maßstab	1:5
DWG-Name	GW78600-78699.dwg

Zusatz			
23; 25; 27	(27)		c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G41	78661	–

Blattgröße: 210mm x 297mm

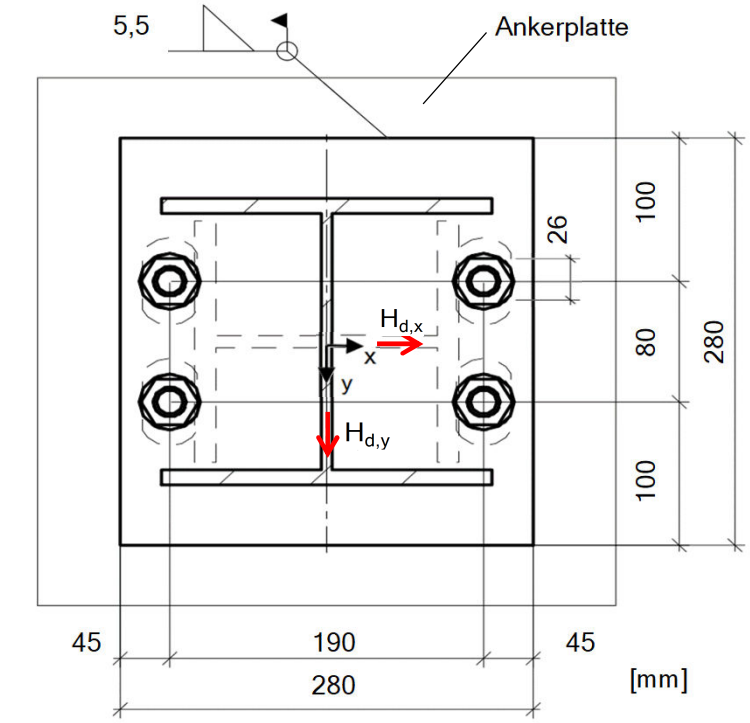
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FRÄSPLATTENBEMESSUNG (EN 1993-1-8:2010-12)

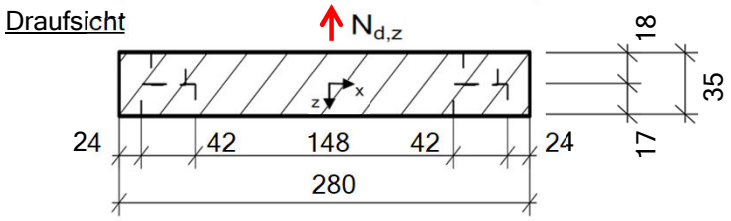
2023-02-27

Auftragsnummer: | Positionsnummer: Versandpos. 78661 (=79943)

Nationales Anwendungsdokument: Sonstige (EU)



Draufsicht



Schnitt (durch Fräsung)

Vertikalkraft	$N_{d,z}$:	396,0 kN
Horizontalkraft	$H_{d,x}$:	256,0 kN
Horizontalkraft	$H_{d,y}$:	176,0 kN
Montagenaht	$a_{Fräspl.}$:	5,5 mm
Verbindungsmittel:		M24, 10.9
Werkstoff der Platte:		S 355
Lochabstand	p_x:	19,0 cm
Lochabstand	p_y:	8,0 cm
Fräsplattenbreite	x_F:	28,0 cm
Fräsplattenlänge	y_F:	28,0 cm
Fräsplattendicke	t_p:	35 mm
Ankerplattentyp:		
Ankerplatte mit Schubstück (in X)		
Werkstoff der Ankerplatte:		S 355

1. Nachweis der Schweißnaht

$\sigma_{v,w,d} \overset{!}{\leq} f_{vw,d}$ erfüllt: 14,07 kN/cm² < 22,33 kN/cm²

$\tau_{w,d,x} \overset{!}{\leq} f_{vw,d}$ erfüllt: 8,31 kN/cm² < 22,33 kN/cm²

mit $A_{w,z}$ = 30,80 cm², $A_{w,y}$ = 30,80 cm², $A_{w,x}$ = 30,8 cm²

2. Nachweis der Schraubenverbindung

$F_{v,Ed} \overset{!}{\leq} F_{v,Rd}$ erfüllt: 77,7 kN < 200,9 kN

$F_{t,Ed} \overset{!}{\leq} F_{t,Rd}$ erfüllt: 99,0 kN < 235,3 kN

$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1,4 \cdot F_{t,Rd}} \overset{!}{\leq} 1$ erfüllt: 0,69 < 1

3. Nachweis der Fräsplatte (einachsig in x-Richtung gespannte Platte)

$\sigma_d \overset{!}{\leq} \sigma_{R,d}$ erfüllt: 16,16 kN/cm² < 31,36 kN/cm²

mit $M_{d,y}$ = 891,0 kNcm, $W_{pl,y}$ = 55,1 cm³

1. Nachweis der Schweißnaht

1.1 Beanspruchbarkeit

$$f_{vw,d} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = \frac{47{,}0\ \text{kN/cm}^2}{\sqrt{3} \cdot 0{,}90 \cdot 1{,}35} = 22{,}33\ \text{kN/cm}^2$$

1.2 Beanspruchung

1.2.1 Beanspruchung in y- und z-Richtung

$$\sigma_{w,d,z} = \frac{N_{d,z}}{A_{w,z}} = \frac{N_{d,z}}{2 \cdot a \cdot y_F} = \frac{396{,}0\ \text{kN}}{2 \cdot 0{,}55\ \text{cm} \cdot 28{,}0\ \text{cm}} = \frac{396{,}0\ \text{kN}}{30{,}8\ \text{cm}^2} = 12{,}86\ \text{kN/cm}^2$$

$$\tau_{w,d,y} = \frac{H_{d,y}}{A_{w,y}} = \frac{H_{d,y}}{2 \cdot a \cdot y_F} = \frac{176{,}0\ \text{kN}}{2 \cdot 0{,}55\ \text{cm} \cdot 28{,}0\ \text{cm}} = \frac{176{,}0\ \text{kN}}{30{,}8\ \text{cm}^2} = 5{,}71\ \text{kN/cm}^2$$

$$\sigma_{v,w,d} = \sqrt{\sigma_{w,d}^2 + \tau_{w,d}^2} = 14{,}07\ \text{kN/cm}^2$$

1.2.2 Beanspruchung in x-Richtung

$$\tau_{w,d,x} = \frac{H_{d,x}}{A_{w,x}} = \frac{H_{d,x}}{2 \cdot a \cdot x_F} = \frac{256{,}0\ \text{kN}}{2 \cdot 0{,}55\ \text{cm} \cdot 28{,}0\ \text{cm}} = \frac{256{,}0\ \text{kN}}{30{,}8\ \text{cm}^2} = 8{,}31\ \text{kN/cm}^2$$

1.3 Nachweise

$\sigma_{v,w,d}$	$\overset{!}{\leq}$	$f_{vw,d}$	erfüllt:	14,07 kN/cm²	<	22,33 kN/cm²
$\tau_{w,d,x}$	$\overset{!}{\leq}$	$f_{vw,d}$	erfüllt:	8,31 kN/cm²	<	22,33 kN/cm²

2. Nachweis der Schraubenverbindung

2.1 Abscheren

2.1.1 Beanspruchung

Komponenten der Schraubenkräfte aus $V_{d,z}$

$$V_{Vd,z} = \frac{H_{d,y}}{n} = \frac{176{,}0\ \text{kN}}{4} = 44{,}0\ \text{kN}$$

Komponenten der Schraubenkräfte aus $V_{d,y}$

$$V_{Vd,y} = \frac{H_{d,x}}{n} = \frac{256{,}0\ \text{kN}}{4} = 64{,}0\ \text{kN}$$

Maximale Schraubenkraft

$$F_{v,Ed} = \sqrt{(V_{Vd,z})^2 + (V_{Vd,y})^2} = \sqrt{(44{,}0\ \text{kN})^2 + (64{,}0\ \text{kN})^2} = 77{,}7\ \text{kN}$$

2.1.2 Beanspruchbarkeit

$$F_{v,Rd} = \frac{\alpha_v \cdot A \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}6 \cdot 4{,}52\ \text{cm}^2 \cdot 100{,}0\ \text{kN/cm}^2}{1{,}35} = 200{,}9\ \text{kN}$$

2.1.3 Nachweis

$F_{v,Ed}$	$\overset{!}{\leq}$	$F_{v,Rd}$	erfüllt:	77,7 kN	<	200,9 kN

2.2 Schraubenzugkraft

2.2.1 Beanspruchbarkeit

$$F_{t,Rd} = \frac{k_2 \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}9 \cdot 3{,}53\ cm^2 \cdot 100{,}0\ kN/cm^2}{1{,}35} = 235{,}3\ kN$$

2.2.2 Beanspruchung

$$F_{t,Ed} = \frac{N_{d,z}}{n} = \frac{396{,}0\ kN}{4} = 99{,}0\ kN$$

2.2.3 Nachweis

$$F_{t,Ed} \overset{!}{\leq} F_{t,Rd} \qquad \text{erfüllt:} \qquad 99{,}0\ kN < 235{,}3\ kN$$

2.3 Zug und Abscheren

2.3.1 Beanspruchung

$F_{v,Ed} = 77{,}7\ kN$ $F_{t,Ed} = 99{,}0\ kN$

2.3.2 Beanspruchbarkeit

$F_{v,Rd} = 200{,}9\ kN$ $F_{t,Rd} = 235{,}3\ kN$

2.3.3 Nachweis

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} \overset{!}{\leq} 1 \qquad \text{erfüllt:} \qquad 0{,}69 < 1$$

3. Nachweis der Fräsplatte (einachsig in x-Richtung gespannte Platte)

3.1 Schnittgrößen

$$M_{d,y} = \frac{N_{d,z}}{2} \cdot \frac{x_F - p_x}{2} = \frac{396{,}0\ kN}{2} \cdot \frac{28{,}0\ cm - 19{,}0\ cm}{2} = 891{,}0\ kNcm$$

3.2 Querschnittswerte

$$W_{pl,y} = \frac{(y_F - 2 \cdot a_{y,Fräs.}) \cdot t_P^2}{4} = \frac{(28{,}0\ cm - 2 \cdot 5{,}0\ cm) \cdot (3{,}5\ cm)^2}{4} = 55{,}1\ cm^3$$

Konstruktionsdetails zur Fräsung: s. Abschnitt 4.

3.3 Beanspruchbarkeit

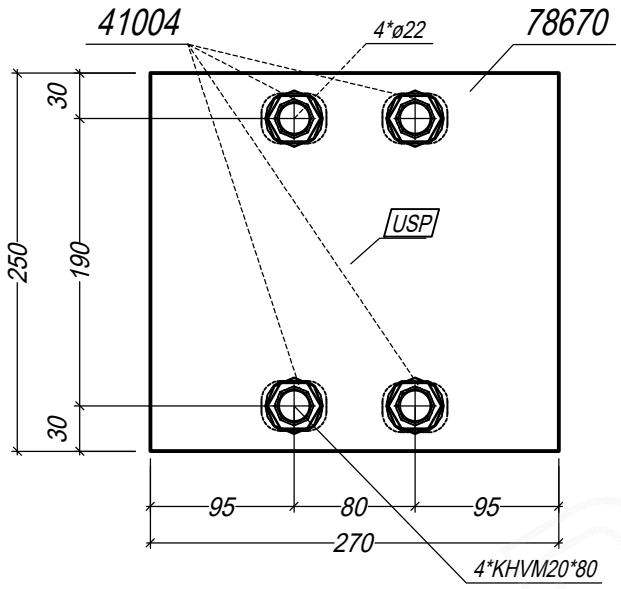
$$\sigma_{R,d} = \frac{f_y}{\gamma_{M0}} = \frac{34{,}5\ kN/cm^2}{1{,}10} = 31{,}36\ kN/cm^2$$

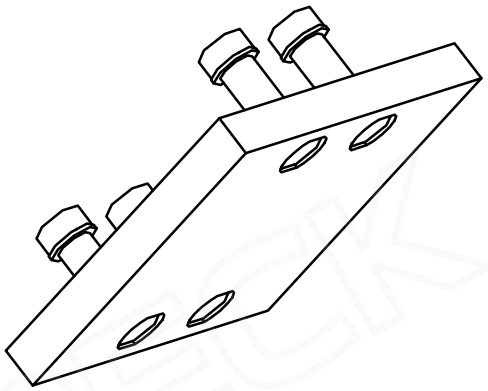
3.4 Beanspruchung

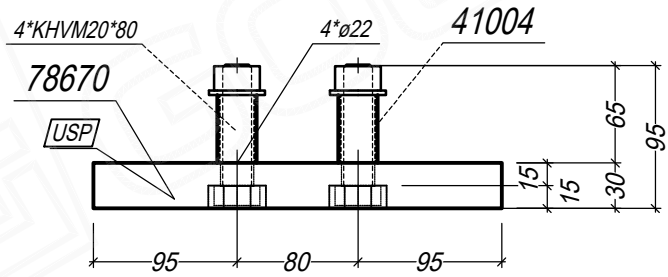
$$\sigma_d = \frac{M_{d,y}}{W_{pl,y}} = \frac{891{,}0\ kNcm}{55{,}1\ cm^3} = 16{,}16\ kN/cm^2$$

3.5 Nachweis

$$\sigma_d \overset{!}{\leq} \sigma_{R,d} \qquad \text{erfüllt:} \qquad 16{,}16\ kN/cm^2 < 31{,}36\ kN/cm^2$$







weiterhin möglich (sofern in der Tragwerks-planung angegeben): Versandposition 78668 (=78670)

*Blech ist nach EN 10164 durch UT (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung:siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3 für Standardbeschichtung
*Vorbereitungsgrad P3 nach ISO 8501-3 für Sonderbeschichtung
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78668	FR	s.Stückliste	NormBGR			
41004	RO	4	ROHR26*1.5	45	PE	
78670	FR	1	FL250*30	270	S355J2	UT (USP)
Stahlgewicht:	15,42 kg	Mantelfläche:	0,167 m²	Versandmaße:	(L x H x B)	270*250*95

Stückzahl	Schraube	DIN	Werkstoff	Gewicht	Bemerkung
4	KHVM20*80	EN 14399-4/-6	10.9 tZn	0,36	Werkstatt
Schraubengewicht:				1,44 kg	
Beschichtung:	–			Gesamtgewicht (kg):	16,86 kg

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FR | Fräsplatte (Stiftplatte) 270*250mm, t=30mm mit 4 HV–Schrauben M20*80

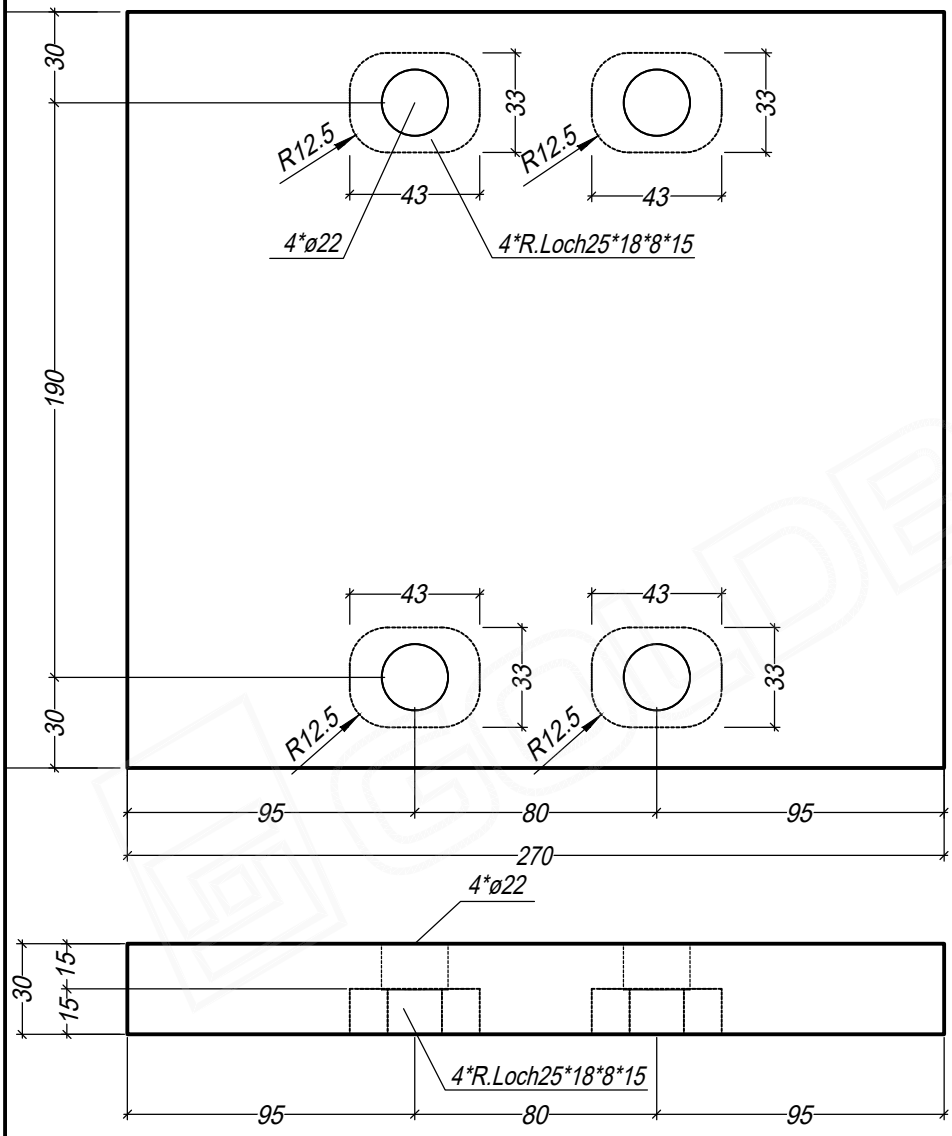
gezeichnet	25.02.2022	[redacted]
geprüft	25.02.2022	
Maßstab	1:5	DWG–Name GW78600–78699.dwg

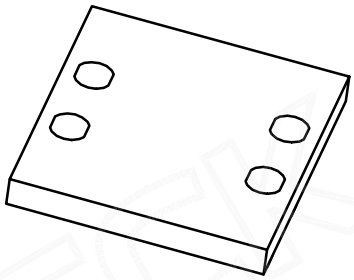
Zusatz			
23; 25; 27	(27)		c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G41	78668	–

Blattgröße: 210mm x 297mm

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*Blech ist nach EN 10164 durch UT (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung:siehe Übersichtszeichnung
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*Vorbereitungsgrad P1 nach ISO 8501-3 für Standardbeschichtung
*Vorbereitungsgrad P3 nach ISO 8501-3 für Sonderbeschichtung
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*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78670	FR	1	FL250*30	270	S355J2	UT (USP)
Stahlgewicht:	15,40 kg	Mantelfläche:	0,151 m²	Versandmaße:	(L x H x B)	270*250*30
Beschichtung:	–				Gesamtgewicht (kg):	15,40 kg

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

FR | Fräsplatte 270*250mm, t=30mm UP zu GW78668

gezeichnet	25.02.2022
geprüft	25.02.2022
Maßstab	1:2.5
DWG-Name	GW78600–78699.dwg

Zusatz			
23; 25; 27	(27)	G75	c19
Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G41	78670	–

Blattgröße: 210mm x 297mm

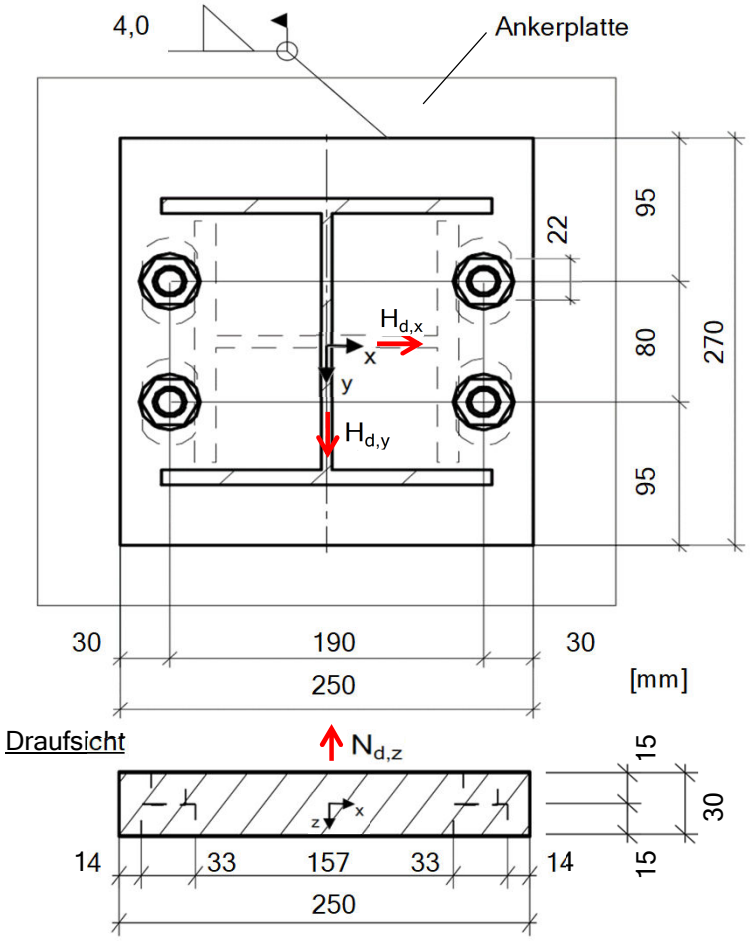
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FRÄSPLATTENBEMESSUNG (EN 1993-1-8:2010-12)

2023-02-27

Auftragsnummer: Grenzlast (projektbez. Last: s. Statik) Positionsnummer: Versandpos. 78668 (=78670)

Nationales Anwendungsdokument: Sonstige (EU)



Draufsicht

Schnitt (durch Fräsung)

Vertikalkraft	$N_{d,z}$:	350,0 kN
Horizontalkraft	$H_{d,x}$:	256,0 kN
Horizontalkraft	$H_{d,y}$:	176,0 kN
Montagenaht	$a_{Fräspl.}$:	4,0 mm
Verbindungsmittel:		M20, 10.9
Werkstoff der Platte:		S 355
Lochabstand	p_x:	19,0 cm
Lochabstand	p_y:	8,0 cm
Fräsplattenbreite	x_F:	25,0 cm
Fräsplattenlänge	y_F:	27,0 cm
Fräsplattendicke	t_p:	30 mm
Ankerplattentyp:		
Ankerplatte mit Schubstück (in X)		
Werkstoff der Ankerplatte:		S 355

1. Nachweis der Schweißnaht

$\sigma_{v,w,d} \overset{!}{\leq} f_{vw,d}$ erfüllt: 18,13 kN/cm² < 22,33 kN/cm²

$\tau_{w,d,x} \overset{!}{\leq} f_{vw,d}$ erfüllt: 12,80 kN/cm² < 22,33 kN/cm²

mit $A_{w,z}$ = 21,60 cm², $A_{w,y}$ = 21,60 cm², $A_{w,x}$ = 20,0 cm²

2. Nachweis der Schraubenverbindung

$F_{v,Ed} \overset{!}{\leq} F_{v,Rd}$ erfüllt: 77,7 kN < 139,6 kN

$F_{t,Ed} \overset{!}{\leq} F_{t,Rd}$ erfüllt: 87,5 kN < 163,3 kN

$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1,4 \cdot F_{t,Rd}} \overset{!}{\leq} 1$ erfüllt: 0,94 < 1

3. Nachweis der Fräsplatte (einachsig in x-Richtung gespannte Platte)

$\sigma_d \overset{!}{\leq} \sigma_{R,d}$ erfüllt: 12,68 kN/cm² < 31,36 kN/cm²

mit $M_{d,y}$ = 525,0 kNcm, $W_{pl,y}$ = 41,4 cm³

1. Nachweis der Schweißnaht

1.1 Beanspruchbarkeit

$$f_{vw,d} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}} = \frac{47{,}0\ \text{kN/cm}^2}{\sqrt{3} \cdot 0{,}90 \cdot 1{,}35} = 22{,}33\ \text{kN/cm}^2$$

1.2 Beanspruchung

1.2.1 Beanspruchung in y- und z-Richtung

$$\sigma_{w,d,z} = \frac{N_{d,z}}{A_{w,z}} = \frac{N_{d,z}}{2 \cdot a \cdot y_F} = \frac{350{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 27{,}0\ \text{cm}} = \frac{350{,}0\ \text{kN}}{21{,}6\ \text{cm}^2} = 16{,}20\ \text{kN/cm}^2$$

$$\tau_{w,d,y} = \frac{H_{d,y}}{A_{w,y}} = \frac{H_{d,y}}{2 \cdot a \cdot y_F} = \frac{176{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 27{,}0\ \text{cm}} = \frac{176{,}0\ \text{kN}}{21{,}6\ \text{cm}^2} = 8{,}15\ \text{kN/cm}^2$$

$$\sigma_{v,w,d} = \sqrt{\sigma_{w,d}^2 + \tau_{w,d}^2} = 18{,}13\ \text{kN/cm}^2$$

1.2.2 Beanspruchung in x-Richtung

$$\tau_{w,d,x} = \frac{H_{d,x}}{A_{w,x}} = \frac{H_{d,x}}{2 \cdot a \cdot x_F} = \frac{256{,}0\ \text{kN}}{2 \cdot 0{,}40\ \text{cm} \cdot 25{,}0\ \text{cm}} = \frac{256{,}0\ \text{kN}}{20{,}0\ \text{cm}^2} = 12{,}80\ \text{kN/cm}^2$$

1.3 Nachweise

$\sigma_{v,w,d} \overset{!}{\leq} f_{vw,d}$ erfüllt: 18,13 kN/cm² < 22,33 kN/cm²

$\tau_{w,d,x} \overset{!}{\leq} f_{vw,d}$ erfüllt: 12,80 kN/cm² < 22,33 kN/cm²

2. Nachweis der Schraubenverbindung

2.1 Abscheren

2.1.1 Beanspruchung

Komponenten der Schraubenkräfte aus $V_{d,z}$

$$V_{Vd,z} = \frac{H_{d,y}}{n} = \frac{176{,}0\ \text{kN}}{4} = 44{,}0\ \text{kN}$$

Komponenten der Schraubenkräfte aus $V_{d,y}$

$$V_{Vd,y} = \frac{H_{d,x}}{n} = \frac{256{,}0\ \text{kN}}{4} = 64{,}0\ \text{kN}$$

Maximale Schraubenkraft

$$F_{v,Ed} = \sqrt{(V_{Vd,z})^2 + (V_{Vd,y})^2} = \sqrt{(44{,}0\ \text{kN})^2 + (64{,}0\ \text{kN})^2} = 77{,}7\ \text{kN}$$

2.1.2 Beanspruchbarkeit

$$F_{v,Rd} = \frac{\alpha_v \cdot A \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}6 \cdot 3{,}14\ \text{cm}^2 \cdot 100{,}0\ \text{kN/cm}^2}{1{,}35} = 139{,}6\ \text{kN}$$

2.1.3 Nachweis

$F_{v,Ed} \overset{!}{\leq} F_{v,Rd}$ erfüllt: 77,7 kN < 139,6 kN

2.2 Schraubenzugkraft

2.2.1 Beanspruchbarkeit

$$F_{t,Rd} = \frac{k_2 \cdot A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0{,}9 \cdot 2{,}45\ cm^2 \cdot 100{,}0\ kN/cm^2}{1{,}35} = 163{,}3\ kN$$

2.2.2 Beanspruchung

$$F_{t,Ed} = \frac{N_{d,z}}{n} = \frac{350{,}0\ kN}{4} = 87{,}5\ kN$$

2.2.3 Nachweis

$$F_{t,Ed} \overset{!}{\leq} F_{t,Rd} \qquad \text{erfüllt:} \qquad 87{,}5\ kN < 163{,}3\ kN$$

2.3 Zug und Abscheren

2.3.1 Beanspruchung

$F_{v,Ed} = 77{,}7\ kN$ $F_{t,Ed} = 87{,}5\ kN$

2.3.2 Beanspruchbarkeit

$F_{v,Rd} = 139{,}6\ kN$ $F_{t,Rd} = 163{,}3\ kN$

2.3.3 Nachweis

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} \overset{!}{\leq} 1 \qquad \text{erfüllt:} \qquad 0{,}94 < 1$$

3. Nachweis der Fräsplatte (einachsig in x-Richtung gespannte Platte)

3.1 Schnittgrößen

$$M_{d,y} = \frac{N_{d,z}}{2} \cdot \frac{x_F - p_x}{2} = \frac{350{,}0\ kN}{2} \cdot \frac{25{,}0\ cm - 19{,}0\ cm}{2} = 525{,}0\ kNcm$$

3.2 Querschnittswerte

$$W_{pl,y} = \frac{(y_F - 2 \cdot a_{y,Fräs.}) \cdot t_P^2}{4} = \frac{(27{,}0\ cm - 2 \cdot 4{,}3\ cm) \cdot (3{,}0\ cm)^2}{4} = 41{,}4\ cm^3$$

Konstruktionsdetails zur Fräsung: s. Abschnitt 4.

3.3 Beanspruchbarkeit

$$\sigma_{R,d} = \frac{f_y}{\gamma_{M0}} = \frac{34{,}5\ kN/cm^2}{1{,}10} = 31{,}36\ kN/cm^2$$

3.4 Beanspruchung

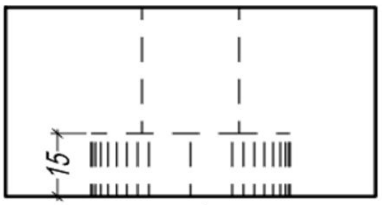
$$\sigma_d = \frac{M_{d,y}}{W_{pl,y}} = \frac{525{,}0\ kNcm}{41{,}4\ cm^3} = 12{,}68\ kN/cm^2$$

3.5 Nachweis

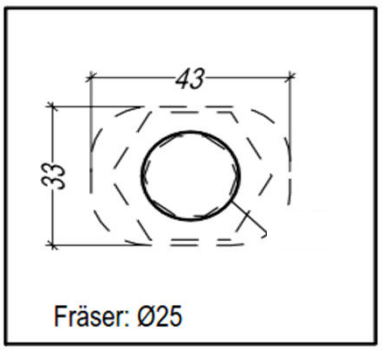
$$\sigma_d \overset{!}{\leq} \sigma_{R,d} \qquad \text{erfüllt:} \qquad 12{,}68\ kN/cm^2 < 31{,}36\ kN/cm^2$$

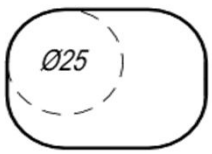
4. Konstruktionsdetails

Verbindungsmittel: M20, 10.9



Ansicht





Draufsicht



Ausführungsdetail der Fräsung

Pos 78510 Teilflächenbelastung für Fußplatte 280 x 280 x 25 [mm]

| Fußplatte Pos. 78510, t = 25 mm, S355 | Fräsplatte Pos. 78661, t = 35 mm, S355

v20.12

EINGANGSWERTE

Baustoffe
Beton C30/37 → f_{cd} = 17,0 N/mm²

Belastung
F_{Ed} = 2075,0 kN

Lastausmitte
e = 0 mm

unterstützendes Bauteil
a = 500 mm
b = 2500 mm

Fuß- bzw. Zwischenplatte
a_0 = 280 mm
b_0 = 280 mm

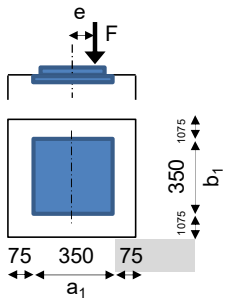
Ankerplatte Pos. 78666
a_1 = 350 mm
b_1 = 350 mm
t_1 = 35 mm

Verteilungsfläche
a_2 = 500 mm
b_2 = 500 mm

BEMESSUNG

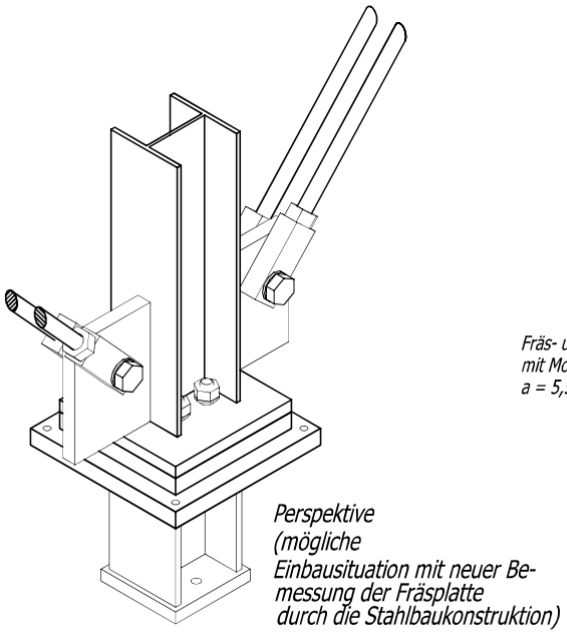
Es werden eine gleichmäßige Pressung unterseitig der Fuß- bzw. Zwischenplatte sowie ein Lastausbreitungswinkel 1:2 innerhalb der Ankerplatte unterstellt.
Unterseitig der Ankerplatte ergibt sich die Belastungsfläche zu:

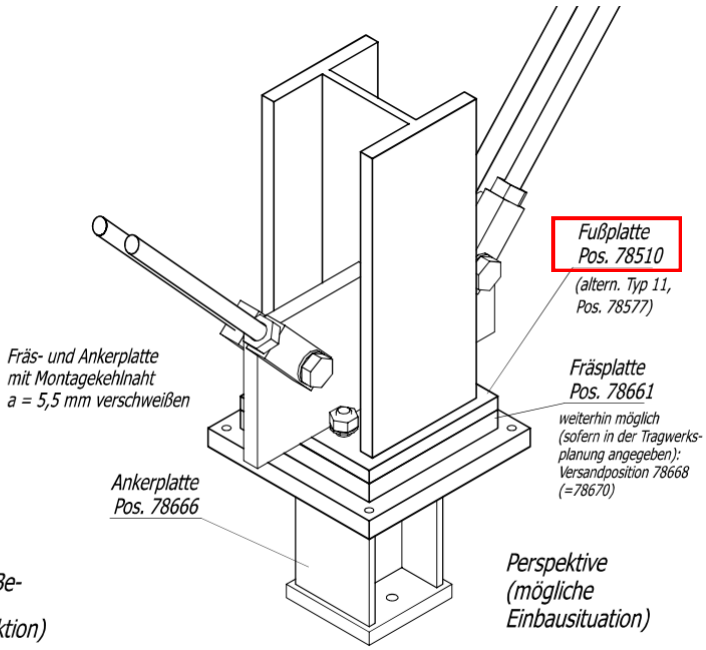
a_1' = 350 mm
b_1' = 350 mm



F_{Ed} = 2075 kN < 2975 kN = F_{Rdu}

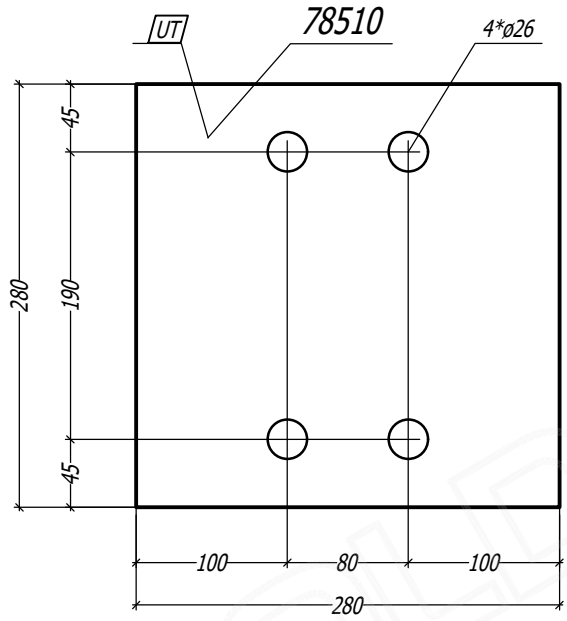
A_{c0} = 122500 mm²
$a_{1,red}$ = 350 mm → a_2, b_2
$A_{c0,red}$ = 122500 mm²
$\sigma_{Ed,0}$ = = 16,9 N/mm² < 17,0 N/mm² = f_{cd} → kein Nachweis der Teilflächenbelastung erforderlich

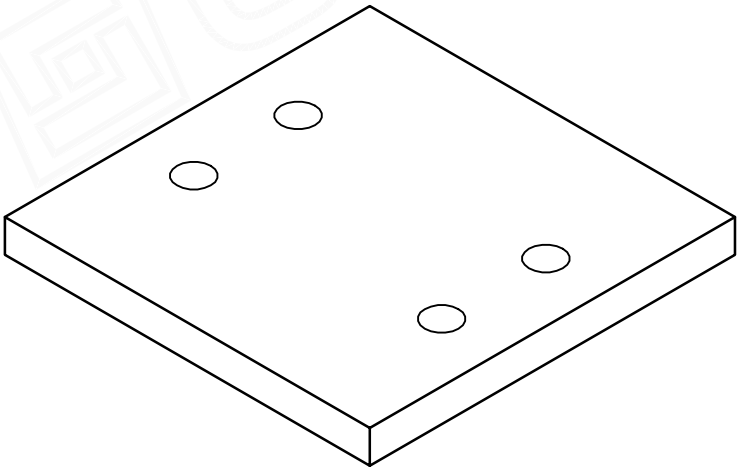

Perspektive (mögliche Einbausituation mit neuer Bemessung der Fräsplatte durch die Stahlbaukonstruktion)



Perspektive (mögliche Einbausituation)

Einbaurichtung des Schubstückes in Abhängigkeit der Verbandsrichtung beachten!





roh

*Blech ist nach EN 10164 durch UT (Ultraschallprüfung) im Raster 200x200mm zu prüfen

*Werkstoff:siehe Stückliste
*Schrauben:
KM..*...4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAM..*...8.8 nach EN ISO 4017, 4032, 7089
KHVM..*...10.9 nach DIN EN 14399-4/-6
*Vorspannung:siehe Übersichtszeichnung
*Nennlochspiel nach DIN EN 1090-2
*Vorbereitungsgrad P1 nach ISO 8501-3 für Standardbeschichtung
*Vorbereitungsgrad P3 nach ISO 8501-3 für Sonderbeschichtung
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur für S355 bei einer Materialdicke >25mm = 120°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfumfang (ZfP) für querverlaufende Kehlnähte bei einer Materialdicke > 30mm = 5%

Position	Benennung	Stückzahl	Profil	Länge [mm]	Werkstoff	Gewicht [kg]	Bemerkung
78510	Fußplatte	s.Stückliste	BL25x280	280	S355J2	15,39	UT (USP)

Beschichtung: Lieferzustand	Farbe: –	Dicke [µm]:	15,39 kg Gewicht
Versandmaße (L x H x B): 280x25x280 mm	Mantelfläche: 0,171 m²		Gesamtgewicht: 15,39 kg

Index	Datum	Name	Änderung
gez.:	24.11.2022	[redacted]	
geprüft:	24.11.2022		
Maßstab:	1:5	DWG-Name GW78500-78599.dwg	

GOLDBECK
Ummelner Str. 4-6, 33649 Bielefeld
Innovationen Produkte Systeme

Fußplatte

für Parkhausstützen Systemfußpunkte

Typ	Tabelle 2	Normteilnummer	Index
P	G16	78510	–

Blattgröße: 210mm x 297mm

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Pos 78577 Teilflächenbelastung für Fußplatte 270 x 250 x 25 [mm]

BOCAD - TYP 11 | Fußplatte Pos. 78577, t = 25 mm, S355 | Fräsplatte Pos. 78654, t = 30 mm, S355

v20.12

EINGANGSWERTE

Baustoffe
Beton C30/37 → f_{cd} = 17,0 N/mm²

Belastung
F_{Ed} = 2075,0 kN

Lastausmitte
e = 0 mm

unterstützendes Bauteil
a = 500 mm
b = 2500 mm

Fuß- bzw. Zwischenplatte
a_0 = 270 mm
b_0 = 250 mm

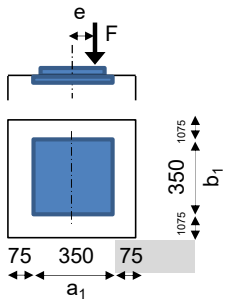
Ankerplatte Pos. 78666
a_1 = 350 mm
b_1 = 350 mm
t_1 = 35 mm

Verteilungsfläche
a_2 = 500 mm
b_2 = 500 mm

BEMESSUNG

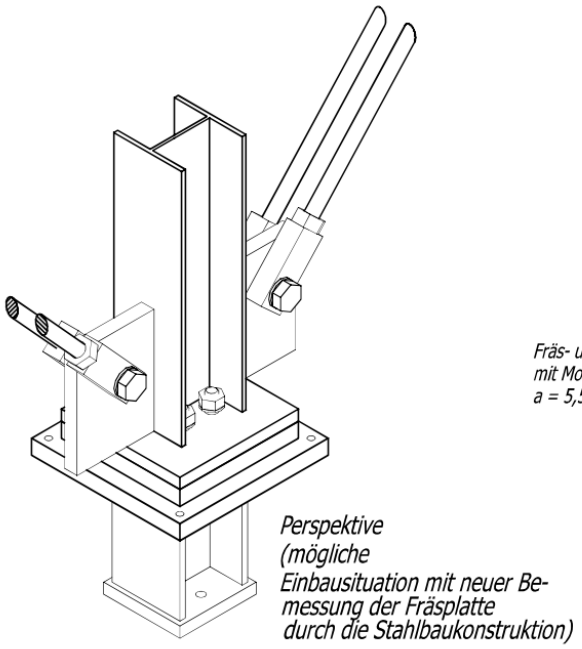
Es werden eine gleichmäßige Pressung unterseitig der Fuß- bzw. Zwischenplatte sowie ein Lastausbreitungswinkel 1:2 innerhalb der Ankerplatte unterstellt.
Unterseitig der Ankerplatte ergibt sich die Belastungsfläche zu:

a_1' = 350 mm
b_1' = 350 mm

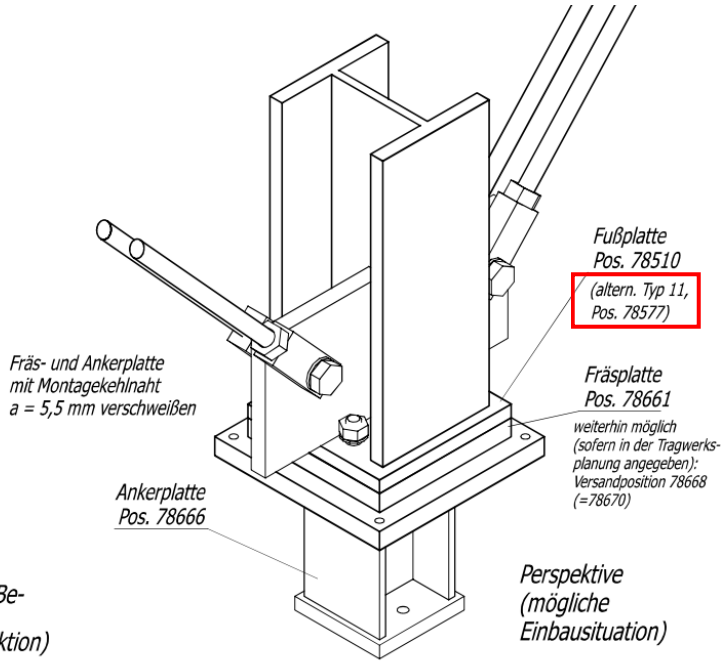


F_{Ed} = 2075 kN < 2975 kN = F_{Rdu}

A_{c0} = 122500 mm²
$a_{1,red}$ = 350 mm → a_2, b_2
$A_{c0,red}$ = 122500 mm²
$\sigma_{Ed,0}$ = = 16,9 N/mm² < 17,0 N/mm² = f_{cd} → kein Nachweis der Teilflächenbelastung erforderlich

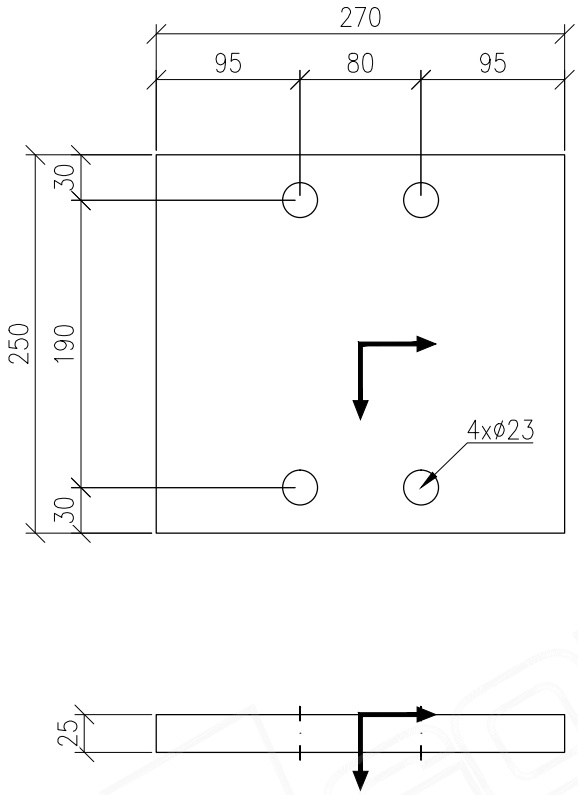


Perspektive (mögliche Einbausituation mit neuer Bemessung der Fräsplatte durch die Stahlbaukonstruktion)



Perspektive (mögliche Einbausituation)

Einbaurichtung des Schubstückes in Abhängigkeit der Verbandsrichtung beachten!



*Blech ist nach EN 10164 durch
UT (Ultraschallprüfung)
im Raster 200x200mm zu prüfen

*Werkstoff: siehe Stückliste
*Schrauben:
KM..*..4.6 nach DIN 7990, 7989, EN ISO 4032
KSGKAZM..*..8.8 nach EN ISO 4017, 4032, 7089
KHVM..*..10.9 nach DIN EN 14399-4/-6
*Vorspannung: siehe Übersichtszeichnung
*Lochspiel: 2mm
*Vorbereitungsgrad P1 nach ISO 8501-3
*Bewertungsgruppe C nach DIN EN ISO 5817
*Vorwärmtemperatur bei S355 (C_{AE} >0.42%
Materialdicke: 25...40mm 120°C
>40mm 180°C
*Toleranzklasse 1 nach DIN EN 1090-2

*EXC 2 nach DIN EN 1090-2
*Schweißnahtprüfung (ZfP)
bei Materialdicke >20mm =5%

Position	Benennung	Stückzahl	Profil	Länge	Werkstoff	Bemerkung
78577	Fußplatte	s. Stückliste	FL250x25	270	S355J2	Ultraschallprüfung
Stahlgewicht:	13,25 kg	Mantelfläche:	0,15 m²	Versandmaße:	(H x B x L)	250 x 25 x 270
Beschichtung:	–				Gesamtgewicht (kg):	13,25

GOLDBECK

Goldbeck GmbH Entwicklung Bausysteme, Ummelner Str. 4–6, 33649 Bielefeld

Fußplatte
250mm x 270mm t=25mm

gezeichnet 18.10.2012
geprüft
Maßstab 1:5
DWG-Name

a	22.05.2015	Weisters / Wisotzky	Stückliste	
Zusatz		(27)	G10	c19
		Tabelle 0	Tabelle 1	Tabelle 2	Tabelle 3

Typ	Tabelle 2	Normteilnummer	Index
P	G42	78577	a

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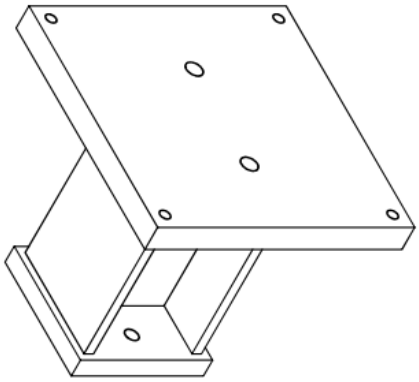
Regeldetail Zulagebewehrung AP 78666 SSHEB160

(gilt nur als Vorgabe für die Gründungsplanung)

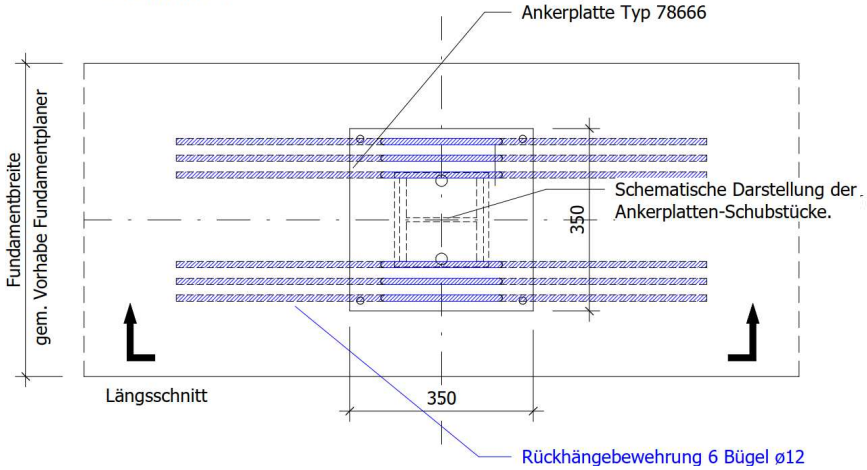
- Die endgültigen Biegeformen und Stückzahlen sind in der Gründungsplanung anzugeben, in den Stahllisten aufzuführen und ausschließlich diesen zu entnehmen!
- Die Zulage-Eisen sind KEIN Bestandteil der Ankerplatten und es erfolgt keine Lieferung von GOLDBECK!
- Diese Details umfassen nicht die Spaltzugbewehrung infolge Vertikallast. Diese ind vom Gründungsstatiker zu bestimmen!

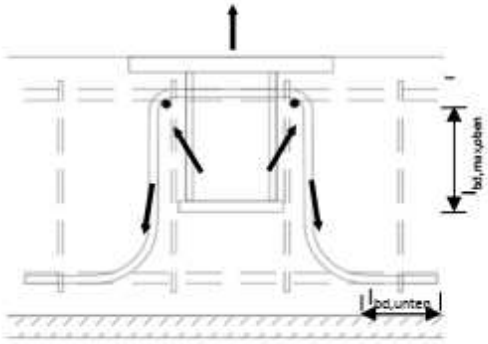
Die Zulagebewehrung ist nur erforderlich, wenn durch die Ankerplatte Zugkräfte in die Gründung eingeleitet werden.

Für AP 78666

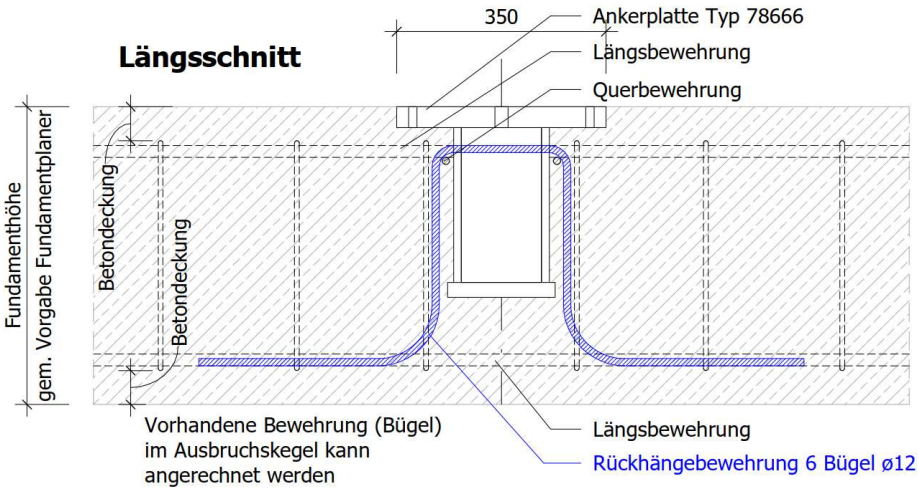


Grundriss

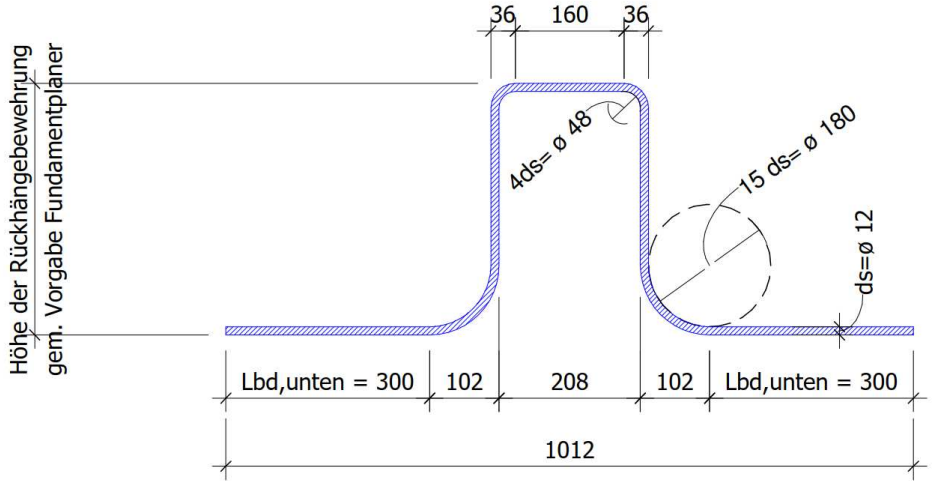




Längsschnitt



Auszug Rückhängebewehrung für AP 78666



Chapter 7: Special construction

- omitted -

Chapter 8: Facade

Pos. WP001 ff.

Pos. WP001 ff. – precast concrete façade wall panels

The dimensions of the façade wall panels can be taken from the views.

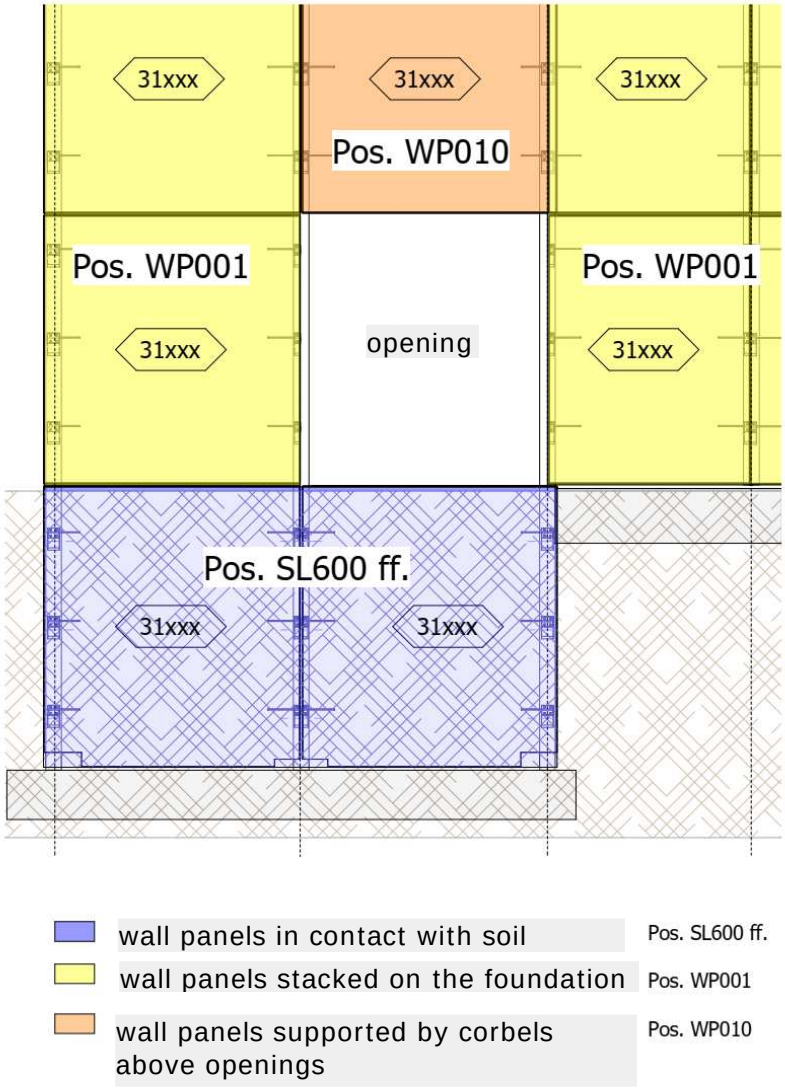
Load assumptions:
wind load: see load assumptions

materials:
Walls: C35/45, XC4, XF1, WF, cnom = 3,0 cm, d = 14cm
consoles: S355, design class EXC2

Explanation

The precast concrete façade wall panels are available in three design variants and are available in the positions 350 ff. WP001 and WP010.

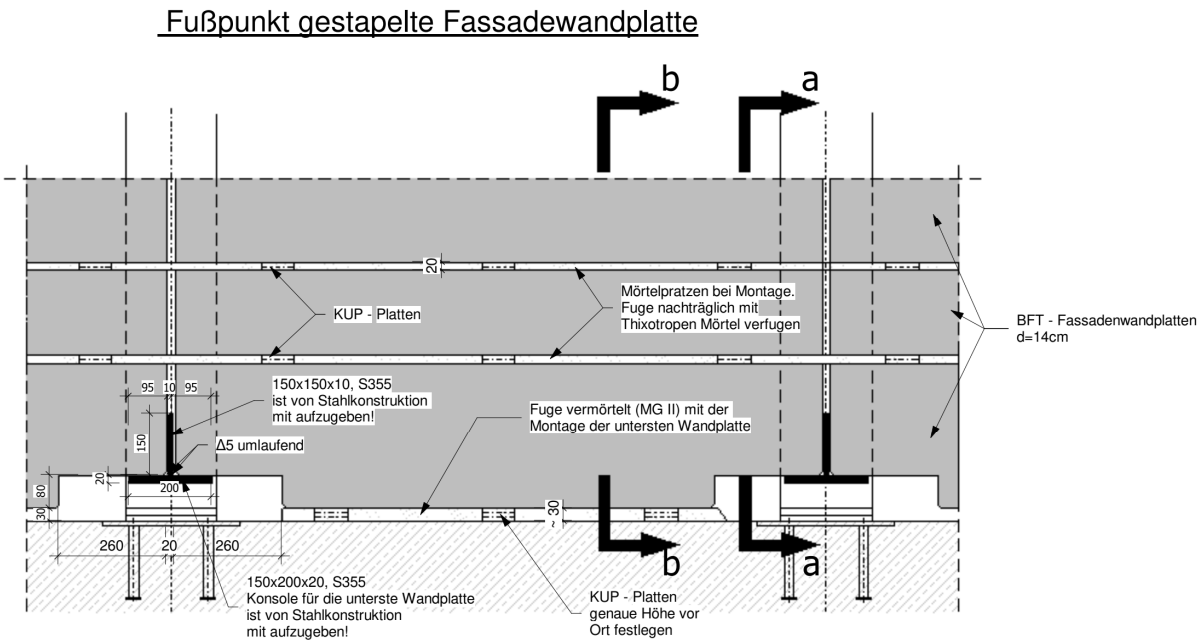
Figure 1



Depending on the design of the façade wall panels in the context of this project, only the relevant section is relevant. Designs that do not correspond to any section in Figure 1, such as wall plates with door recesses, are dealt with in Item WP100 ff.

Pos. WP001: FWP stacked on foundation without earth pressure

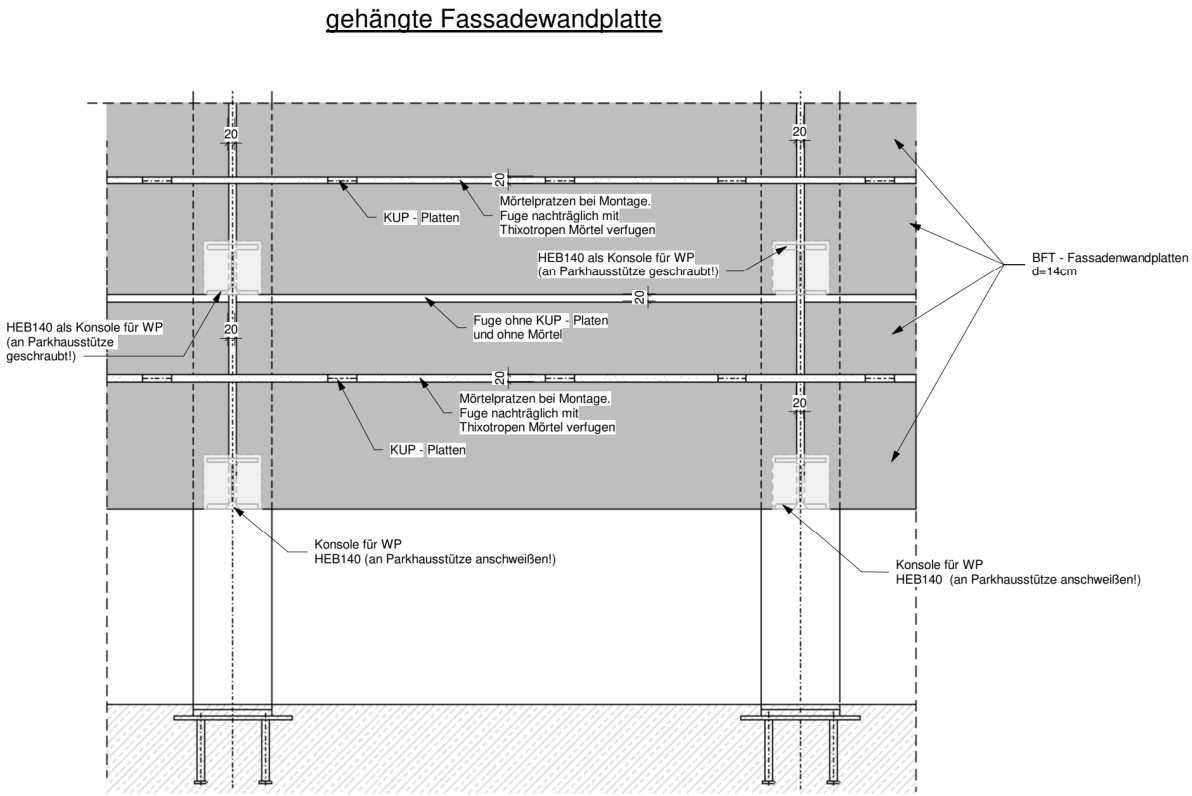
If the wall panels are designed as a stacked façade (see Figure 1, yellow area), the dead weight is transferred directly into the foundation. With this design variant, support brackets are only located in the foot area of the columns. They only serve to align the first wall panel level and have no function in the final state, therefore no calculation is required.



In the area of closed wall surfaces, the dead weight loads of the wall panels are transferred into the foundation as linear loads. To simplify matters, however, all dead weight loads of the wall panels are already included in the dimensioning of the steel supports and the specified foundation loads (see Item CO001 ff.). When dimensioning the foundations, the façade is therefore always considered as an individual load, the statics of the foundation is on the safe side (most unfavourable reinforcement).

Pos. WP010: FWP stacked on brackets without earth pressure

If openings are provided in the wall area (see figure 1, orange area), the dead weight of the wall panels above the openings is supported by steel brackets. These are welded or screwed to the supports.



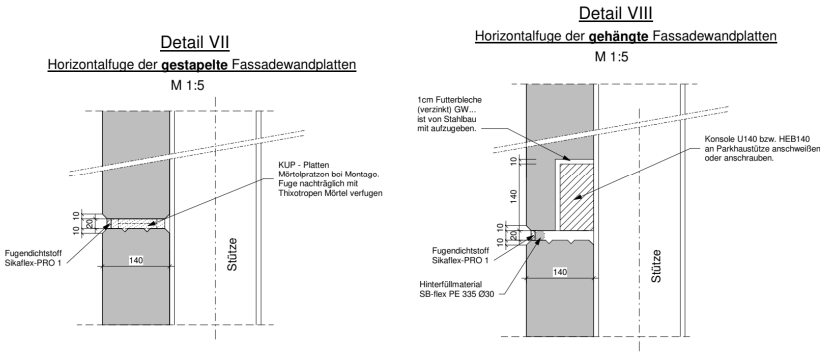
In the area of the above-mentioned openings, the dead weight loads of the wall panels are transferred via the steel supports and are taken into account accordingly in the statics.

Pos. 350 ff. Wall panels with earth pressure

The stacked or suspended wall panels (see figure 1, blue area), which are provided in the earth pressure area, are covered with pos. 350 ff. of the statics.

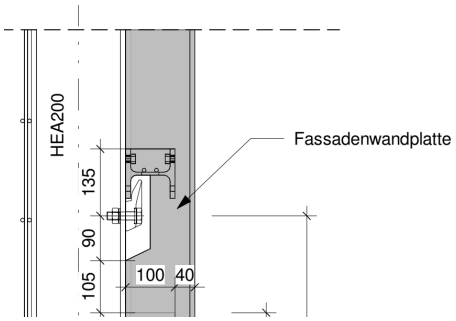
Joints

The horizontal joints are not mortarised during installation; in the case of F90 requirements, they are subsequently grouted. For F90 requirements, the vertical joint is subsequently filled with Thixo mortar, otherwise only a permanently elastic joint is sufficient.



For F90 requirements, the wall panels are additionally doubled in the horizontal joint and the vertical joint is grouted.

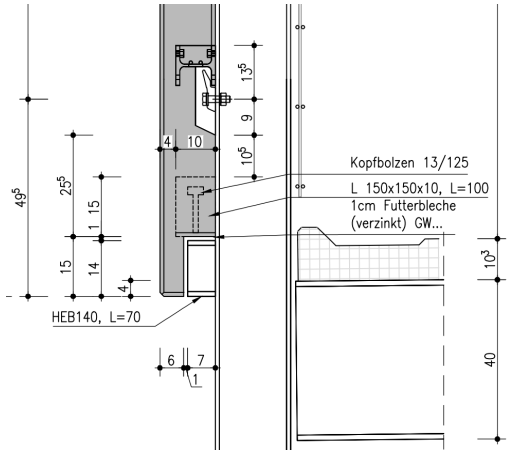
The wind and impact loads on the wall panels are absorbed by at least 4 uniformly arranged wall brackets.

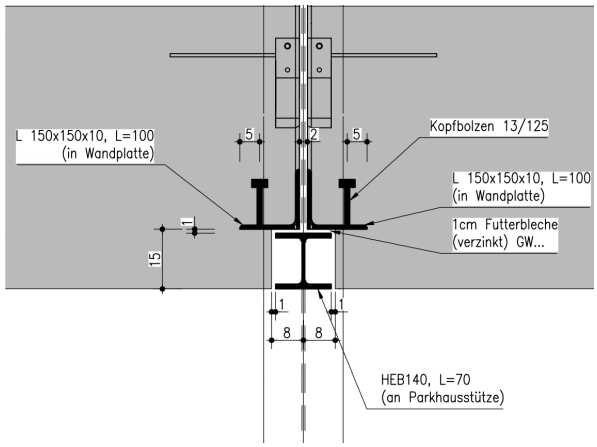


Support consoles

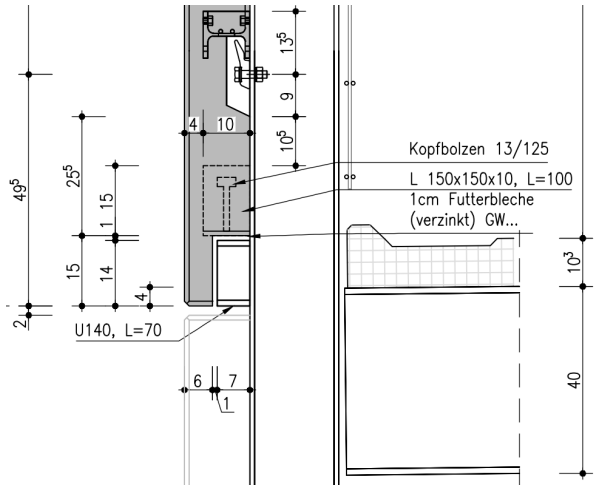
As a rule, the steel consoles are fixed to the flange of the columns and verified with a limit load by means of sample verification. Deviations of the concrete edges compared to these details are possible.

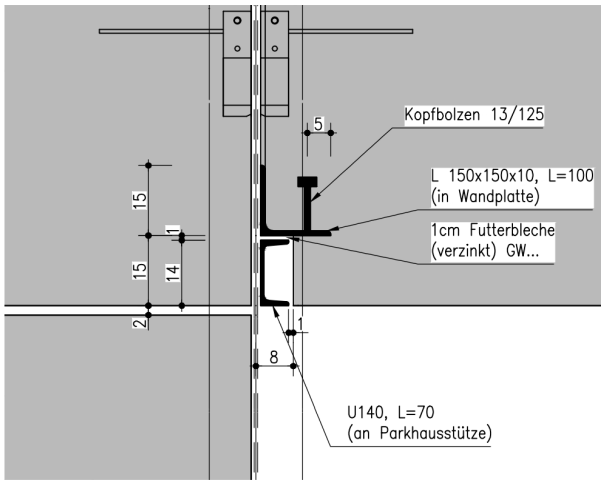
a) Variant welded console HEB 140:



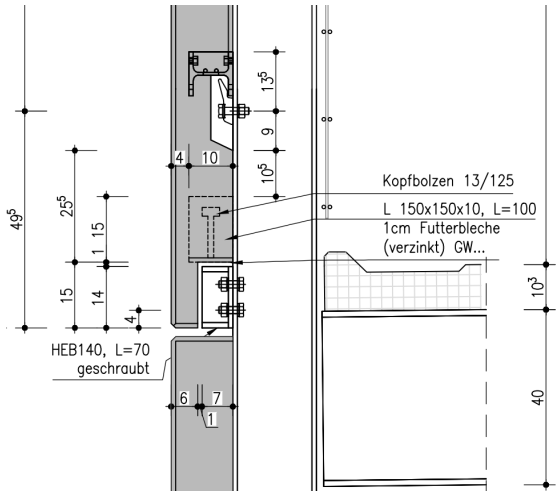


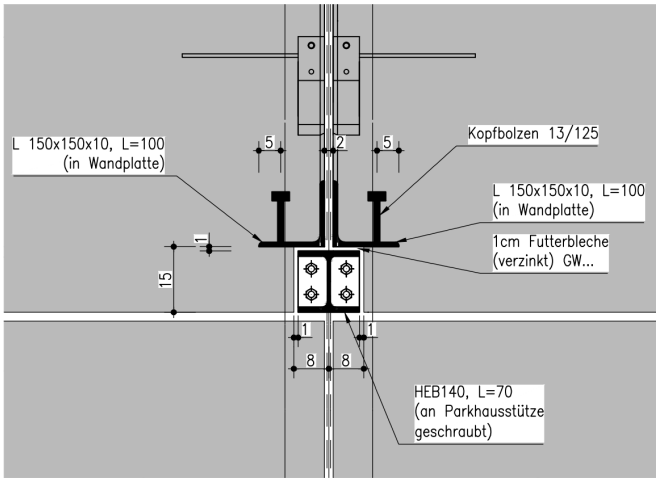
b) Variant welded console U 140:



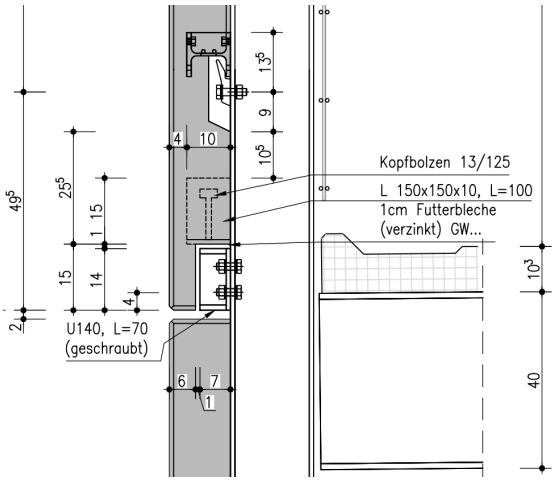


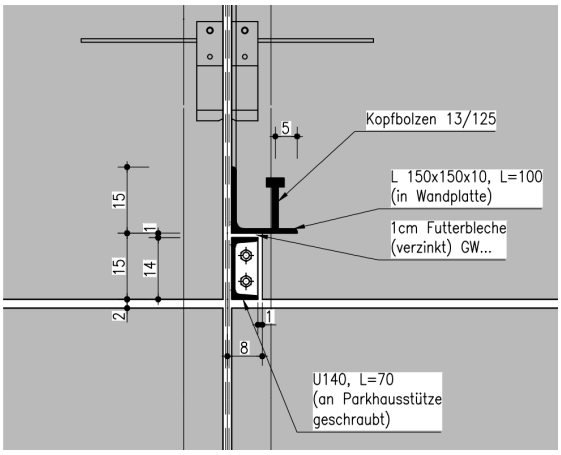
c) <u>Version screwed console HEB 140:</u>





d) <u>Version screwed console U 140:</u>



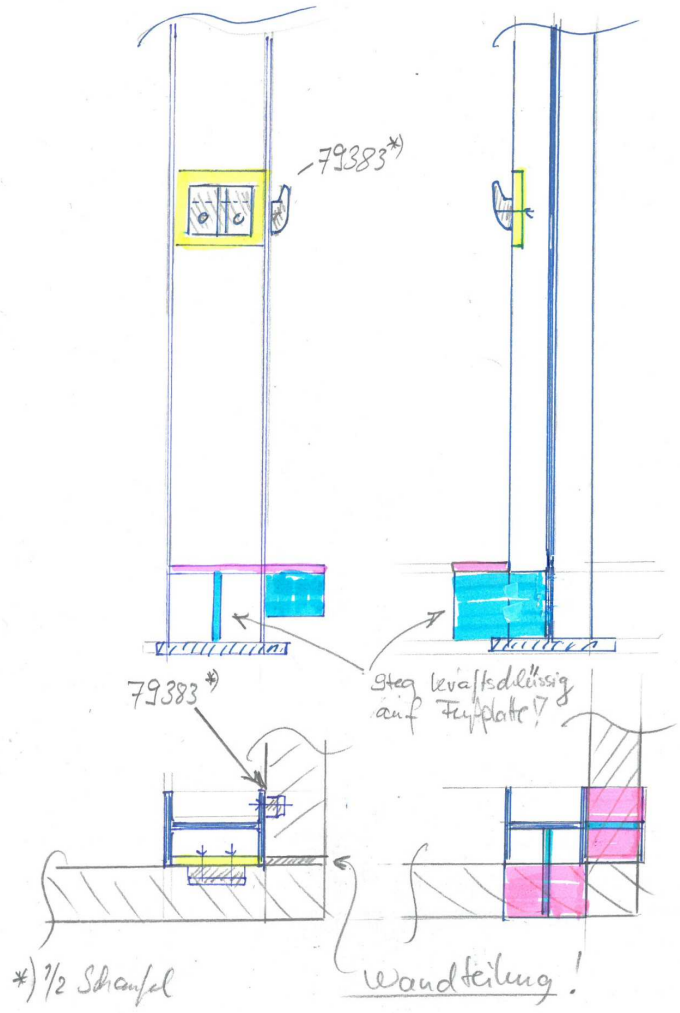


Special cases

At corner supports and in special cases, the load transmission of the consoles and wall brackets is not always possible at the flange. In these cases, project-related substructures are required according to the following examples. The additional connection verifications are carried out with the steel construction and are not part of these statics. Further variants are possible.

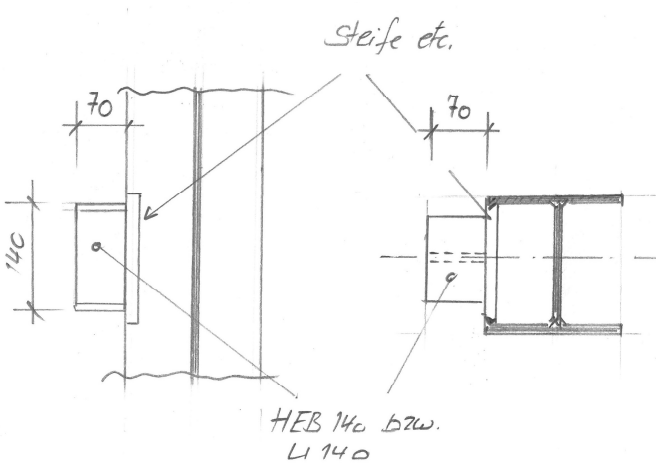
a) Corner support

If the façade panels are attached to both sides of a corner support, the horizontal brackets must be designed with the adjustments shown in the sketch. The console at the base of the support is only required for height adjustment during installation and must also be adapted to the local situation; if necessary, the vertical web is force-fitted to the base plate.



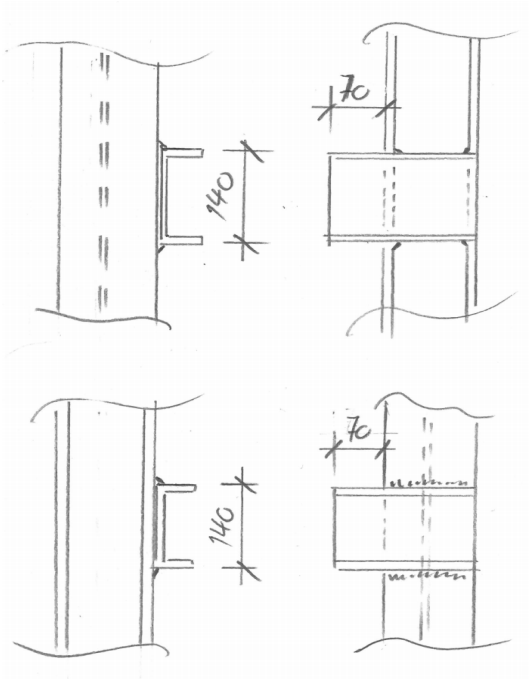
b) <u>Console on rotated support</u>

The consoles are arranged according to plan in the column axis on the flange side. In special cases (twisted column etc.) the cantilever length of the consoles must be limited to 70 mm (with welded or bolted connection) and an individual substructure (stiffeners etc.) must be provided. This substructure must be flush with the outer edge of the support.



c) <u>Console next to the support</u>

The consoles are arranged according to plan in the column axis on the flange side. In special cases, the consoles can also be attached to the sides of the columns. The U-profile is extended; the load is transferred via the flanges of the columns.

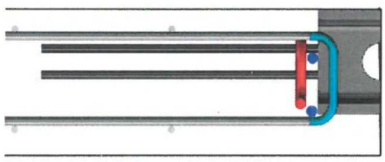


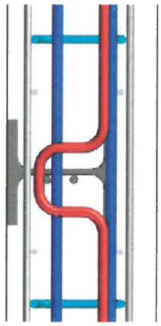
Reinforcement details

a) Wall bracket

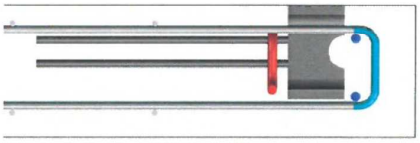
The wall brackets in the façade wall panels (GW 79355 and GW 79356) differ in size and location. An additional bracket ∅12 mm (pos. 4) is fitted to each wall bracket as shown in the sketch. The position of the edge brackets and corner brackets ∅10 mm must be adjusted to the position of the wall brackets (detail 1-3).

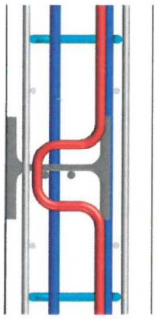
Detail 1



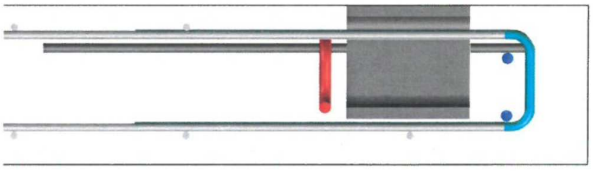


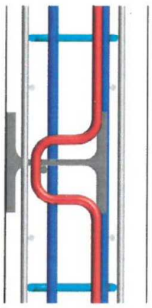
Detail 2

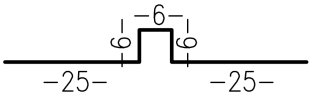




Detail 3



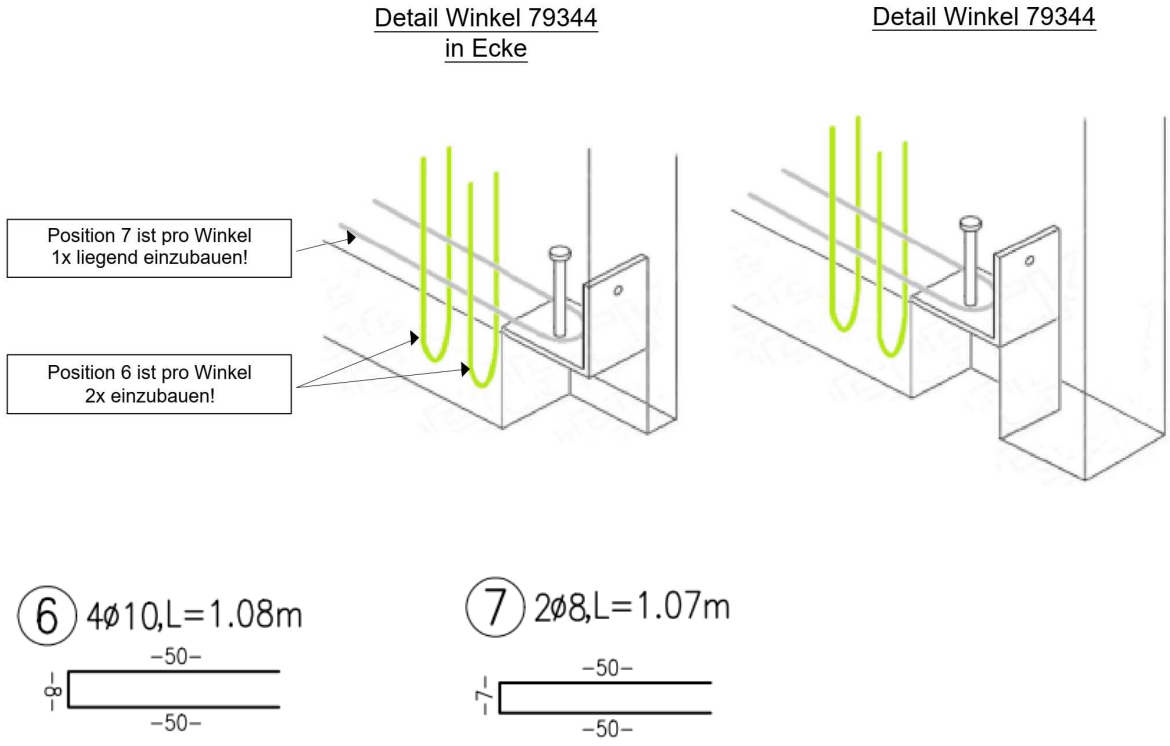




④ 6ø12,L=0.68m

b) Recess on console bearing

In addition to the basic reinforcement, two vertical stirrups ∅8 mm, l=500m are installed in front of each recess with L-bracket (GW 79344) at a distance of e=10 cm (pos. 6). In addition, a horizontal stirrup ∅8 mm, l=500m (pos. 7) is placed on each angle, surrounding the shear bolt.



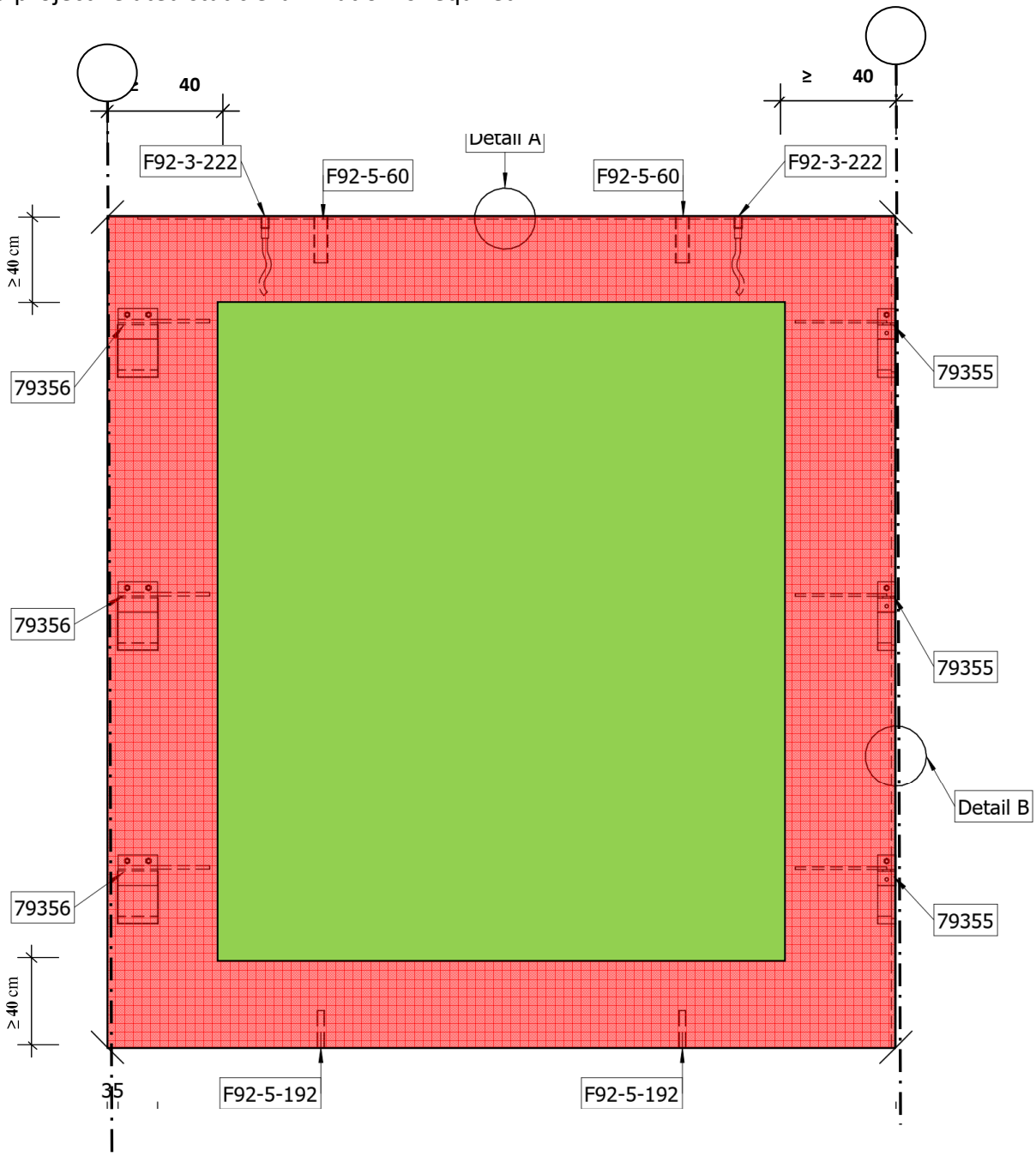
Loads on wall panels and anchor areas

Lightweight façade constructions without spacer installation (e.g. sheet metal façade, climbing aids with "hand-tight" prestressing etc.) with a weight of up to 10 kg/m2 can be excluded from the construction without further testing. Anchors for their substructures must be placed at a **minimum distance of 40 cm from each edge of the building component** (in the green area). If **drill holes are** required **in the edge area (red area)**, their position has to be adapted to the installation parts located there (see examples in sketch). The proof of the dowels for fixing the facade, wall brackets etc. must be provided by the designer of the facade.

For **larger vertical loads** on the BFT façade wall panels such as:

- Clinker facades
- Brackets with higher individual loads
- Support of other components
- etc.

a project-related static examination is required.



Fire resistance class F90

Proof of fire resistance according to DIN EN 1992-1-2/NA:2010-12 (DIN 4102-4):

Proof of the minimum wall thickness:

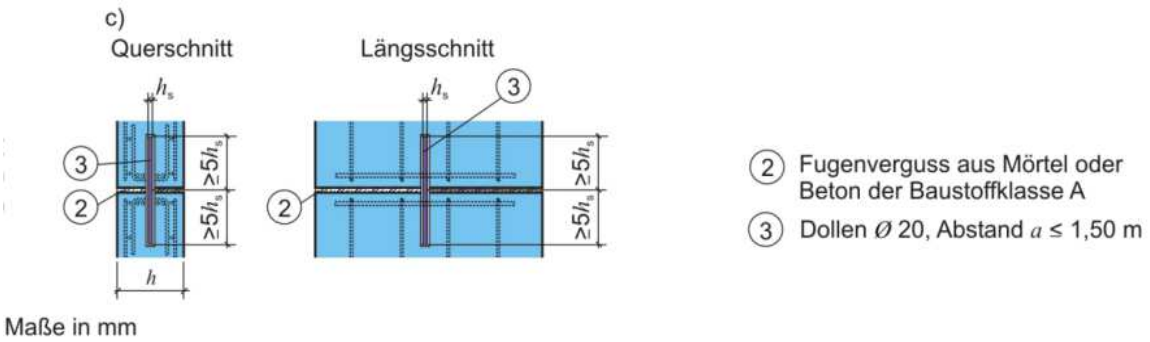
Stahlbetonwände, tragend

(Tab. 5.4)

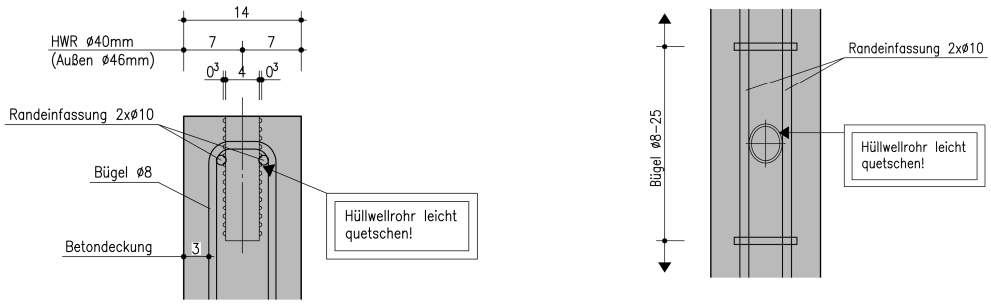
Anforderung				**Ausführung**				
REI 90, einseitige Brandbeanspruchung					c_{nom}	=	30	mm
Ausnutzungsgrad:	μ_{fi}	=	0,3					
Mindestwanddicke:	d	=	140 mm	Wanddicke:	d_{min}	=	140	mm
Achsabstand:	a	=	25 mm	Achsabstand:	a	~	35	mm

Execution of the joint:

The horizontal joints are fully mortared and executed with dowels d=20 mm, e ≤ 1,50m and corrugated pipes d=40 mm. The vertical joints are subsequently grouted with mortar:

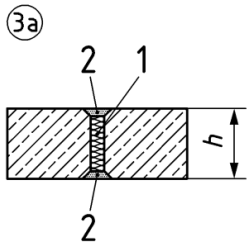


Maße in mm



Connection to staircases etc.:

Connection as F90 wall according to DIN4102-4, Figure 5.11, 3a. The construction corresponds to the design according to DIN4102-4, Figure 5.10b for f ≥ 15mm and is therefore also permissible for expansion joints.



Legende

1 Mineralwolledämmschicht nach 5.11.2 (4)
2 Fugendichtstoffe nach DIN EN ISO 6927
3 Fugenverguss aus Mörtel oder Beton — nichtbrennbar —

Bild 5.11 — Wandfugen (Schema-Skizzen für die Ausführungen 1 bis 3 b)

Calculation:

This section contains the verifications of the items WP001 (stacked on foundations, without earth pressure) and WP010 (stacked on consoles, without earth pressure). The verification of the wall plates with earth pressure has already been carried out in pos. SL600 ff.
The verification includes the concrete components and, if necessary, the necessary support consoles for the connection to the steel construction. The load introduction and transmission in the steel construction is not subject of this verification.
In the following, the **limit forces** are verified in each case. This means that the verifications are carried out using the **most unfavourable dimensions** that occur in the Goldbeck system and **unfavourable load assumptions.**

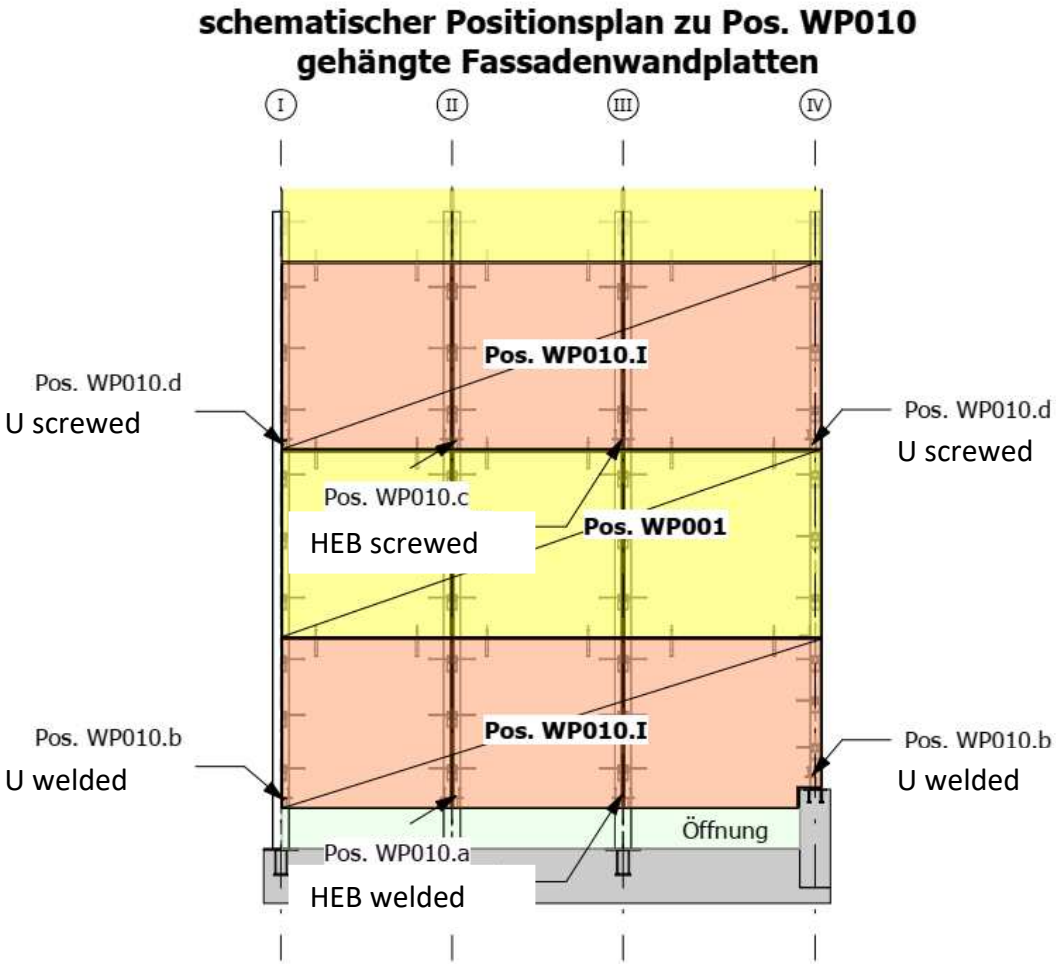
Item WP001

For stacked construction, proof of the BFT-FWP is required.

Item WP010

Through openings, the loads are transferred into the supports via consoles. The lowest row of consoles over openings is usually welded to the columns. In all horizontal joints above, these are screwed for assembly reasons (see sketch). The maximum load per console is limited so that consoles are usually required in every 2nd horizontal joint. The facade wall panels, which rest on the consoles, are assigned to item WP010.I. The wall plates above are stacked on this position and therefore corresponds to pos. WP001 (see figure 2).
If only one panel rests laterally on a console, for example at the edge of the wall plate bay, it is designed as a U-profile. If there are panels resting on a console on both sides, it is designed as HEB 140 (see figure 2).

Figure 2



Required evidence:

- **Pos. WP001.I**: wall-like carrier
  Wall plate rests on consoles, here the proof for the most unfavourable ratio h/b and the maximum possible load is provided by pos. WP001
- **Pos. WP001**: Stacked FWP
- **Item WP010.a-d**: Support consoles:
  - H (Pos. WP010.a welded and WP010.c screwed)
  - U (Pos. WP010.b welded and WP010.d screwed)

GOLDBECK

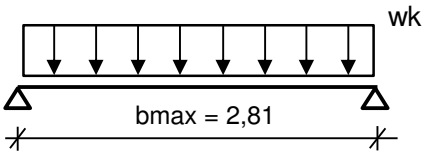
Pos WP001 **reinf. con. wall panel, d = 14 cm**

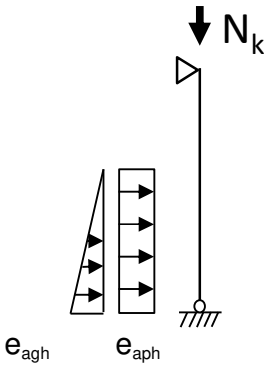
C 35/45, XF 1 / XC 4 / WF, c_{nom} = 3,0 cm *V. 09.08.19*

System: → **stacked wall panel**

two-sided held wall panel with windload

width b = 2,81 m





Loads:

from dead weight	0,14 m	*	25,0 kN/m³	=	3,50 kN/m²
				g_k =	**3,50 kN/m²**
wind:				**max. w_k ≤**	**± 2,00 kN/m²**

Internal forces+dimensioning earth pressure: siehe Pos. SL600 ff.

Internal forces facade:

max. A_{Hk} = B_{Hk} =		± 2,00 kN/m²	*	$\frac{2,8\ m}{2}$	=	**± 2,81 kN/m**
max. M_d =	1,5 *	± 2,00 kN/m²	*	$\frac{(2,8\ m)^2}{8}$	=	**± 2,96 kNm/m**

Dimensioning facade: (Normal force can be neglected)

$k_d = \frac{d}{\sqrt{\frac{M_{Eds}}{b_{eff}}}}$ = 10,5 / $\sqrt{2,96}$ = 6,2 → k_s = 2,3

erf. a_s = 2,3 * 2,96 / 10,5 = **0,65 cm²/m each side**

sel.	**# Q 257 two-sided**
	edges 2 Ø 10 + stirrups Ø 8 / 25
Execution F90: + Dowels in horizontal joint	

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Pos WP010.I reinf. con. wall panel, d = 14 cm

C 35/45, XF 1 / XC 4 / WF, c_{nom} = 3,0 cm *V. 09.08.19*

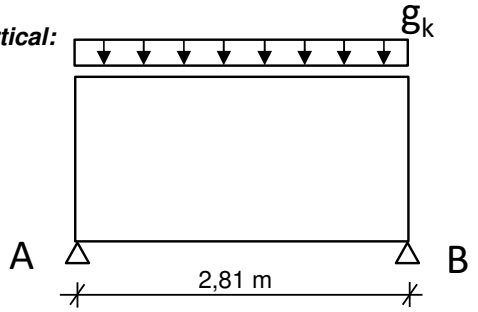
System: → **deep beam**

calculation worst case ration h/b

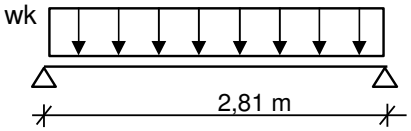
Width b =	2,81 m	h / b =	1,41	/	2,81	=	**0,50**
Height h =	1,4 m	→ z_F = 0,3 * h * (3-h / b)				=	**1,06**

> 0,5 und < 1,0

vertical:



horizontal:



Loads:

from dead weight		1,41 m	*	0,14 m	*	25,0 kN/m³	=	4,94 kN/m
from pos.	max.	5,00 m	*	0,14 m	*	25,0 kN/m³	=	17,50 kN/m

g_k = 22,44 kN/m

wind: **max. w_k = ± 2,00 kN/m²**

Internal forces:

max. $A_k = B_k$ =			22,44 kN/m	*	2,8 m / 2	=	**31,52 kN**
max. M_d =	1,35	*	22,44 kN/m	*	(2,8 m)² / 8	=	**29,89 kNm**
max. $M_{d\ Wind}$ =	1,5	*	2,00 kN/m	*	(2,8 m)² / 8	=	**2,96 kNm/m**

Dimensioning:

Tensile reinforcement deep beam:

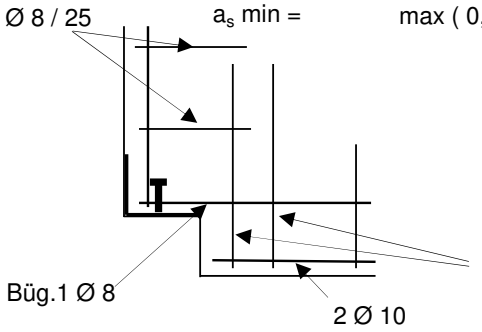
max. Z_d =	29,9	/	1,06			=	**28,29 kN**
erf. A_{sf} =	28,3	/	43,5	=	**0,65 cm²**	<	**2,29 cm²**
	(2 Ø 10)				(min b | h)		

A_s vorh. = 1,57 cm² + 2*0,1* 1,41 * 2,57 cm² = 2,29 cm²

minimum area reinforcement deep beam:

a_s min = max (0,075* 14cm ; 1,50 cm²) = **1,5 cm²/m < 2,57 cm²/m**

(grösser als Mind.-Bew. aus Windlast)



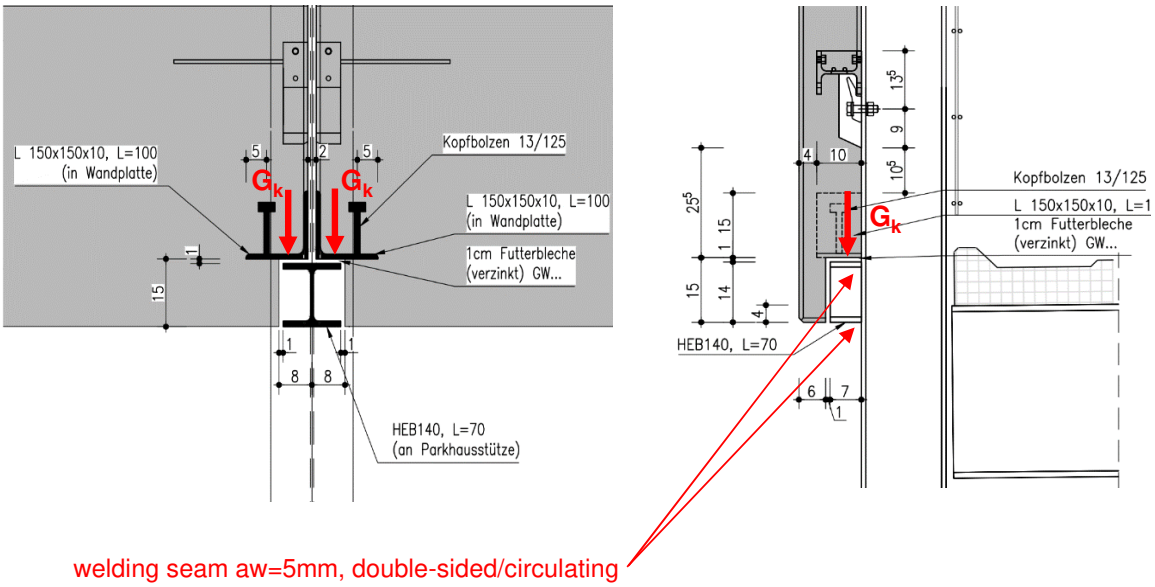
chosen # Q 257 two-sided, edges 2 Ø 10 + stirrups Ø 8 / 25
Allowances support: 1 stirrup Ø 8, horizontal around shear
2 stirrups Ø 8/10, vertical
Execution F90: + Dowels in horizontal joint

GOLDBECK

Pos WP010.a console HEB 140 (welded)

V. 09.08.19

System:



Loads: aus Pos. WP010.l — **max. G_k = 35,0 kN**

Internal forces: 1,35 * 2 * 35,0 kN — **max. V_d = 94,6 kN**

94,6 kN * 0,05 m — **max. M_d = 4,3 kN**

t

Dimensioning: see computing

HEB 140, S335

weld at column aw = 5 mm, circulating

Load introduction:

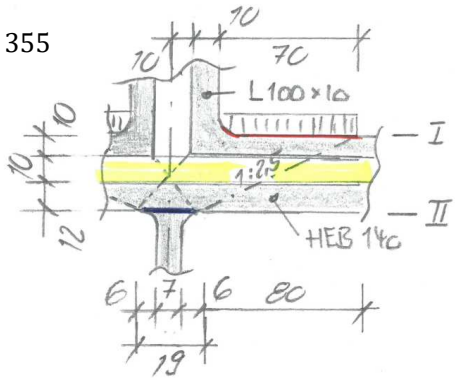
About the contact surface of the L-angle of the concrete wall and the lining plates is a direct load application of the bearing force in the web of the support brackets, so that no flange bending occurs.

Concrete: $vorh.\ \delta_d = \frac{47{,}30\ kN * 1000}{70mm * 70\ mm} = 9{,}65\ \frac{N}{mm^2} < 19{,}80 = fcd_{C35/45}$

(Schnitt I-I)

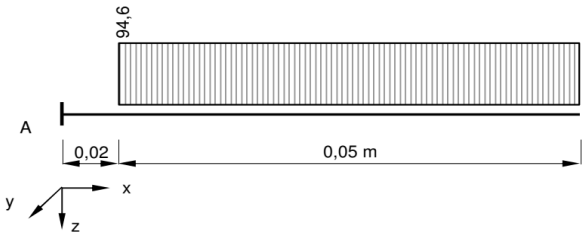
Steel: $vorh.\ \delta_d = \frac{2 * 47{,}30\ kN\ * 1000}{19\ mm\ * 70\ mm} = 71{,}20\ \frac{N}{mm^2} < 355$

(Schnitt II-II)



GOLDBECK

SYSTEM AND LOADS



Einflussbreite: 1,00 m

Area Loads

Dead load Structure	2 * 35 kN	70,00 kN	70,0 kN
			0,1 kN
		G =	70,1 kN
Payloads			
		Q =	0,0 kN/m
		Q_d =	**94,6 kN**

INTERNAL FORCES

Characteristic bearing forces: A_g = **70,0 kN**; A_p = **0,0 kN**; A_{Ed} = **94,6 kN**

Decesive design internal forces: $V_{z,Ed}$ = **94,6 kN**; $M_{y,Ed}$ = **4,3 kNm**

DIMENSIONING

selected: **HE 140 B S 355** [-]

Cross-section classification 1

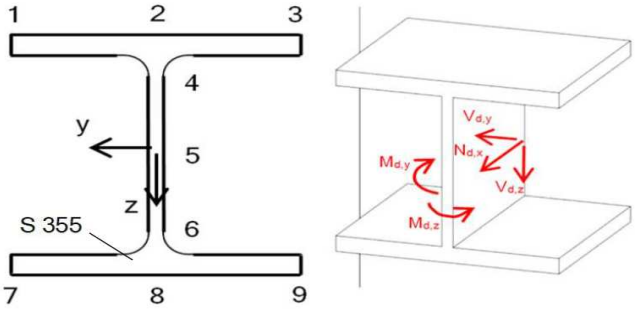
γ_M = 1,00

σ_{Ed}	$= N_{Ed} / A + M_{y,Ed} / I_y * z - M_{z,Ed} / I_z * y$	=	0,0 + 0,0 + 0,0	=	0,0 kN/cm²
$\sigma_{Ed} / \sigma_{Rd}$		=	0,0 / 35,5	=	**0,00 < 1**
max τ_{Ed}	$= \max V_{z,Ed} * S_y / (I_y * t_w)$	=	95 * 123 / (1509 * 0,70)	=	11,0 kN/cm²
τ_{Ed} / τ_{Rd}		=	11,0 / 20,5	=	**0,54 < 1**
$\sigma_{i,Ed}$	$= N_{Ed} / A + M_{y,Ed} / I_y * z - M_{z,Ed} / I_z * y$	=	0,0 + 0,0 + 0,0	=	0,0 kN/cm²
$\tau_{i,Ed}$		=	95 * 116 / (1509 * 0,70)	=	11,0 kN/cm²
σ_v		=	$\sqrt{}$ (0,0² + 3 * 11,0²)	=	19,1 kN/cm²
σ_v / σ_{Rd}		=	19,1 / 35,5	=	**0,54 < 1**

maßgebend: "Steg mittig"

GOLDBECK

MOMENT RESISTANT BEAM SPLICES (EN 1993-1-8:2010-12) 2020-07-02



Horizontal force	$N_{d,x}$:	0,0 kN
Vertical force	$V_{d,z}$:	94,6 kN
Vertical force	$V_{d,y}$:	0,0 kN
Moment	$M_{d,y}$:	4,3 kNm
Moment	$M_{d,z}$:	0,0 kNm
Weld thickn. (flan.)	a_F:	5,0 mm
Weld thickn. (web)	a_S:	5,0 mm
Girder profile:		HEB 140
Girder material:		S 355
Column material:		S 355

1. Cross-section values

1.1 Efficient weld area

$A_y = 2 \cdot [\, a_F \cdot (\, 2 \cdot b + t_{fb,o} + t_{fb,u} - 2 \cdot r - t_S)\,] = 27{,}3\ cm^2$

$A_z = 2 \cdot a_S \cdot d = 2 \cdot 0{,}5\ cm \cdot 9{,}2\ cm = 9{,}2\ cm^2$

$A = A_y + A_z = 27{,}3\ cm^2 + 9{,}2\ cm^2 = 36{,}5\ cm^2$

1.2 Moments of inertia

$I_y = 1.215{,}9\ cm^4$

$I_z = 572{,}5\ cm^4$

2. Stress

$$\sigma_w = \frac{N_{d,x}}{A} + \frac{M_{d,y}}{I_y} \cdot z - \frac{M_{d,z}}{I_z} \cdot y \qquad \sigma_{\perp,Ed} = \sigma_w \cdot \frac{1}{2} \cdot \sqrt{2}$$

$$\tau_{y,w} = \frac{V_{d,y}}{A_y} = \tau_{II} \quad \text{(flange weld)} \qquad \tau_{z,w} = \frac{V_{d,z}}{A_z} = \tau_{II} \quad \text{(web weld)}$$

$$\sigma_{V,w,Ed} = \sqrt{\sigma_\perp^2 + 3 \cdot \tau_\perp^2 + 3 \cdot \tau_{II}^2} = \sqrt{2 \cdot \sigma_w^2 + 3 \cdot \tau_{II}^2}$$

3. Resistance

$$f_{1,w,Rd} = \frac{f_u}{\beta_w \cdot \gamma_{M2}} = \frac{47{,}0\ kN/cm^2}{0{,}90 \cdot 1{,}25} = 41{,}8\ kN/cm^2$$

$$f_{2,w,Rd} = \frac{0{,}9 \cdot f_u}{\gamma_{M2}} = \frac{0{,}9 \cdot 47{,}0\ kN/cm^2}{1{,}25} = 33{,}8\ kN/cm^2$$

4. Requirements

$\sigma_{V,w,Ed} \overset{!}{\leq} f_{1,w,Rd}$ fulfilled: $18{,}0\ kN/cm^2 < 41{,}8\ kN/cm^2$

$\sigma_{\perp,Ed} \overset{!}{\leq} f_{2,w,Rd}$ fulfilled: $1{,}8\ kN/cm^2 < 33{,}8\ kN/cm^2$

GOLDBECK

1. Cross-section values

1.1 Length of weld (web)

$$d = h - t_{fb,o} - t_{fb,u} - 2 \cdot r = 14{,}0\ cm - 1{,}2\ cm - 1{,}2\ cm - 2 \cdot 1{,}2\ cm = 9{,}2\ cm$$

1.2 Efficient weld area (in y-direction)

$$A_y = 2 \cdot [\, a_F \cdot (2 \cdot b + t_{fb,o} + t_{fb,u} - 2 \cdot r - t_S)\,]$$

$$A_y = 2 \cdot [\, 0{,}5\ cm \cdot (2 \cdot 14{,}0\ cm + 1{,}20\ cm + 1{,}20\ cm - 2 \cdot 1{,}2\ cm - 0{,}7\ cm)\,]$$

$$A_y = 27{,}3\ cm^2$$

1.3 Efficient weld area (in z-direction)

$$A_z = 2 \cdot a_S \cdot d = 2 \cdot 0{,}5\ cm \cdot 9{,}2\ cm = 9{,}2\ cm^2$$

1.4 Total weld area

$$A = A_y + A_z = 27{,}3\ cm^2 + 9{,}2\ cm^2 = 36{,}5\ cm^2$$

1.5 Moment of inertia (y-axis)

$$I_y = a_F \cdot b \cdot \frac{h^2}{2} + a_F \cdot (b - 2 \cdot r - t_S) \cdot \frac{(h - 2 \cdot t_F)^2}{2} + 2 \cdot a_S \cdot \frac{d^3}{12} + 2 \cdot a_F \cdot t_F \cdot \frac{(h - t_F)^2}{2}$$

$$I_y = 1.215{,}9\ cm^4$$

1.6 Moment of inertia (z-axis)

$$I_z = 4 \cdot a_F \cdot \frac{b^3}{12} - 2 \cdot a_F \cdot \frac{(2 \cdot r + t_S)^3}{12} + 2 \cdot a_F \cdot t_F \cdot \frac{b^2}{2}$$

$$I_z = 572{,}5\ cm^4$$

2. Stress

$$\sigma_w = \frac{N_{d,x}}{A} + \frac{M_{d,y}}{I_y} \cdot z - \frac{M_{d,z}}{I_z} \cdot y$$

$$\sigma_{\perp,Ed} = \sigma_w \cdot \frac{1}{2} \cdot \sqrt{2}$$

$$\tau_{y,w} = \frac{V_{d,y}}{A_y} = \tau_{\parallel} \quad \text{(flange weld)}$$

$$\tau_{z,w} = \frac{V_{d,z}}{A_z} = \tau_{\parallel} \quad \text{(web weld)}$$

$$\sigma_{V,w,Ed} = \sqrt{\sigma_{\perp}^2 + 3 \cdot \tau_{\perp}^2 + 3 \cdot \tau_{\parallel}^2} = \sqrt{2 \cdot \sigma_w^2 + 3 \cdot \tau_{\parallel}^2}$$

Table 1: Stress

Pos.-Nr.	y_i [cm]	z_i [cm]	σ_w [kN/cm²]	$\sigma_{\perp,Ed}$ [kN/cm²]	$\tau_{y,w}$ [kN/cm²]	$\tau_{z,w}$ [kN/cm²]	$\sigma_{V,w,Ed}$ [kN/cm²]
1	7,0	-7,0	-2,48	-1,75	0,00	0,00	3,51
2	0,0	-7,0	-2,48	-1,75	0,00	10,28	3,51
3	-7,0	-7,0	-2,48	-1,75	0,00	0,00	3,51
4	0,0	-4,6	-1,63	-1,15	0,00	10,28	17,95
5	0,0	0,0	0,00	0,00	0,00	10,28	17,81
6	0,0	4,6	1,63	1,15	0,00	10,28	17,95
7	7,0	7,0	2,48	1,75	0,00	0,00	3,51
8	0,0	7,0	2,48	1,75	0,00	10,28	3,51
9	-7,0	7,0	2,48	1,75	0,00	0,00	3,51

3. Resistance

$$f_{1,w,Rd} = \frac{f_u}{\beta_w \cdot \gamma_{M2}} = \frac{47{,}0\ kN/cm^2}{0{,}90 \cdot 1{,}25} = 41{,}8\ kN/cm^2$$

$$f_{2,w,Rd} = \frac{0{,}9 \cdot f_u}{\gamma_{M2}} = \frac{0{,}9 \cdot 47{,}0\ kN/cm^2}{1{,}25} = 33{,}8\ kN/cm^2$$

4. Requirements

$\sigma_{V,w,Ed} \overset{!}{\leq} f_{1,w,Rd}$ fulfilled: 18,0 kN/cm² < 41,8 kN/cm²

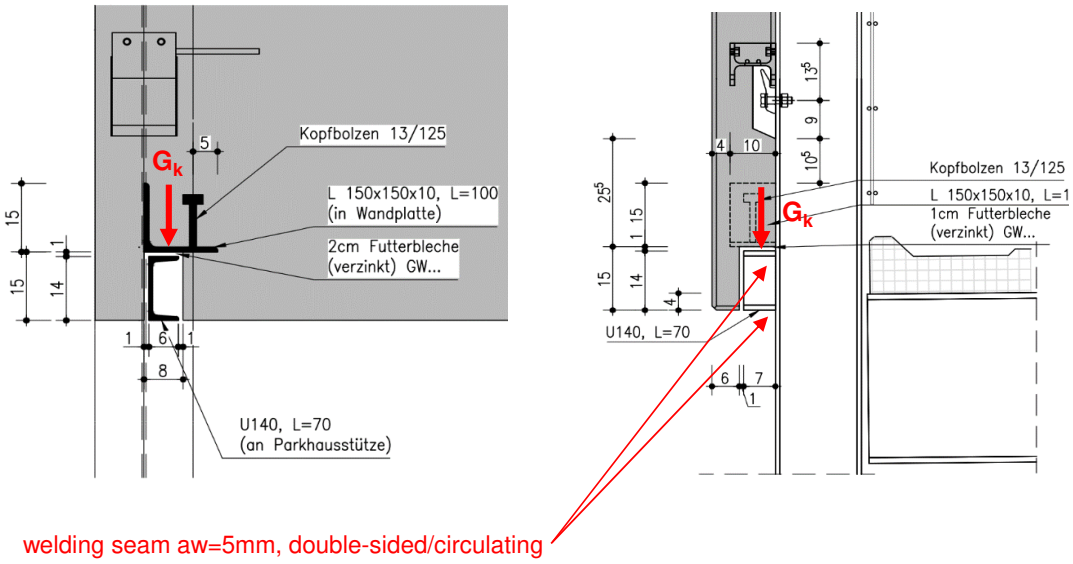
$\sigma_{\perp,Ed} \overset{!}{\leq} f_{2,w,Rd}$ fulfilled: 1,8 kN/cm² < 33,8 kN/cm²

GOLDBECK

Pos WP010.b console U 140 (welded)

V. 09.08.19

System:



Loads: from Pos. WP010.l — **max. G_k = 35,0 kN**

Internal forces: 1,35 * 35,0 kN — **max. V_d = 47,3 kN**
47,3 kN * 0,05 m — **max. M_d = 2,36 kN**

Dimensioning: see computing

U 140, S355

weld at column aw = 5 mm, circulating

Profil: $W_y = 14{,}0^2 * 0{,}70 / 6 = 22{,}90\ cm^2$ (approach only web)

Bending: $$vorh.\ \delta_d = \frac{2{,}36\ kNm}{22{,}9\ cm^2} = 103{,}0\ \frac{N}{mm^2} < 355$$

Shear force: $$vorh.\ T\parallel,_d = \frac{47{,}30\ kN}{140\ mm\ * 7{,}0\ mm} = 48{,}2\ \frac{N}{mm^2} < 204$$

Weld: $W_{w,r} = 14{,}0^2 * 2 * 0{,}50 / 6 = 32{,}70\ cm^2$ (approach only web)

Bending: $$vorh.\ \delta_{v,d} = \frac{2{,}36\ kNm}{32{,}7\ cm^2} = 72{,}2\ \frac{N}{mm^2} < 355$$

Shear force: $$vorh.\ T\parallel,_d = \frac{47{,}30\ kN}{2\ * 140\ mm\ * 5{,}0\ mm} = 33{,}8\ \frac{N}{mm^2} < 204$$

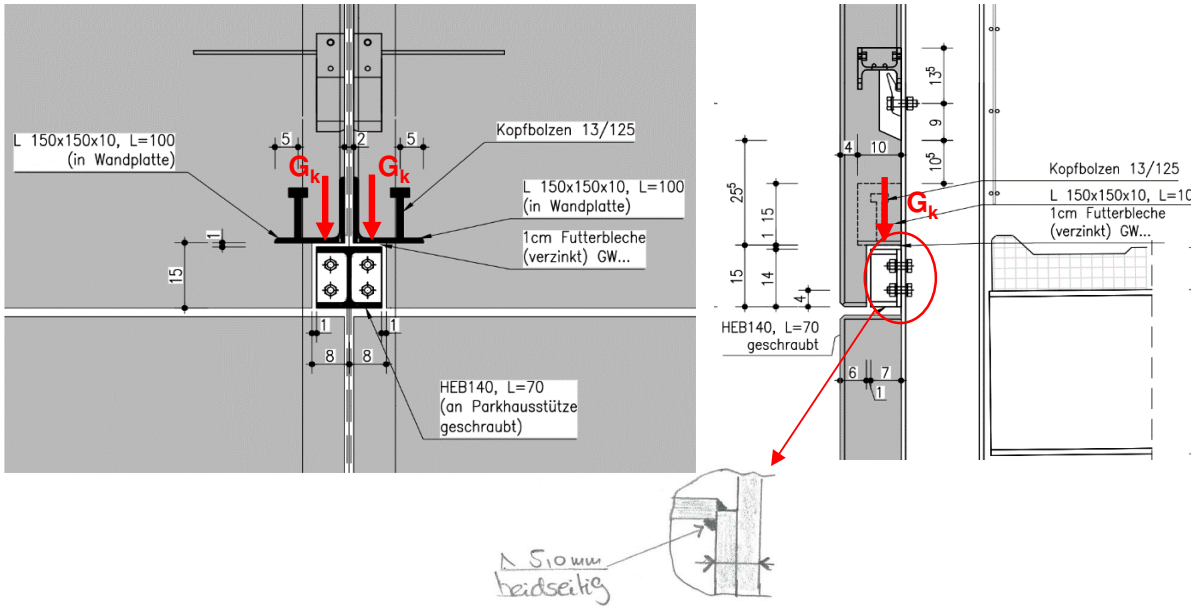
Load transmission: see analysis of console HEB 140

GOLDBECK

Pos WP010.c console HEB 140 (screwed)

V. 09.08.19

System:



Loads: from Pos. WP010.l — **max. G_k = 35,0 kN**

Internal forces: 1,35 * 2 * 35,0 kN — **max. V_d = 94,6 kN**

94,6 kN * 0,05 m — **max. M_d = 4,3 kN**

Dimensioning: see computing

HEB 140, S355

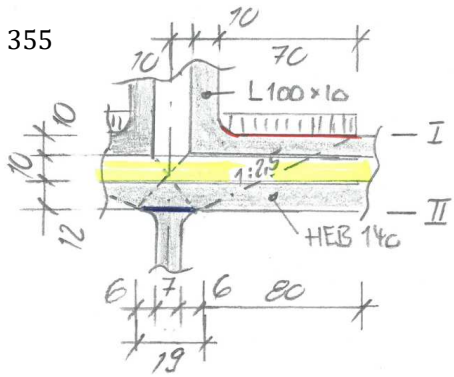
4 screws M16 10.9
welding seam at end plate aw = 5 mm, circulating

Load transmission:

About the contact surface of the L-angle of the concrete wall and the lining plate is a direct load application of the bearing force in the web of the support brackets, so that no flange bending occurs.

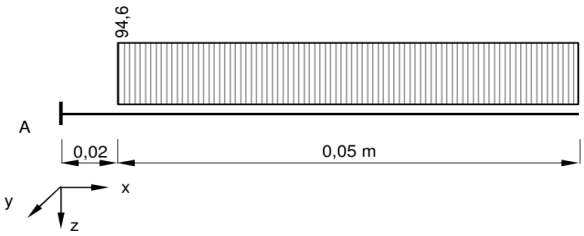
Concrete $vorh.\ \delta_d = \frac{47{,}30\ kN}{70\ mm\ *70\ mm} = 9{,}65\frac{N}{mm^2} < 19{,}80 = fcd_{,\ C35/45}$

Steel: $vorh.\ \delta_d = \frac{2 * 47{,}30\ kN\ * 1000}{19\ mm\ *70\ mm} = 71{,}20\frac{N}{mm^2} < 355$



GOLDBECK

SYSTEM AND LOADS



Einflussbreite: 1,00 m

Area Loads

Dead load Structure	2 * 35 kN	70,00 kN	70,0 kN
			0,1 kN
		G =	70,1 kN
Payloads			
		Q =	0,0 kN/m
		Q_d =	**94,6 kN**

INTERNAL FORCES

Characteristic bearing forces: A_g = **70,0 kN**, A_p = **0,0 kN**, A_{Ed} = **94,6 kN**

Decesive design internal forces: $V_{z,Ed}$ = **94,6 kN**, $M_{y,Ed}$ = **4,3 kNm**

DIMENSIONING

selected: **HE 140 B S 355** [-]

Cross-section classification 1

γ_M = 1,00

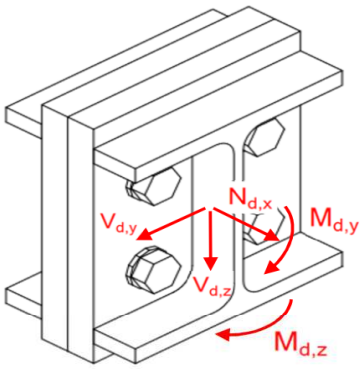
σ_{Ed}	$= N_{Ed} / A + M_{y,Ed} / I_y * z - M_{z,Ed} / I_z * y$	=	0,0 + 0,0 + 0,0	=	0,0 kN/cm²
$\sigma_{Ed} / \sigma_{Rd}$		=	0,0 / 35,5	=	**0,00 < 1**
max τ_{Ed}	$= \max V_{z,Ed} * S_y / (I_y * t_w)$	=	95 * 123 / (1509 * 0,70)	=	11,0 kN/cm²
τ_{Ed} / τ_{Rd}		=	11,0 / 20,5	=	**0,54 < 1**
$\sigma_{i,Ed}$	$= N_{Ed} / A + M_{y,Ed} / I_y * z - M_{z,Ed} / I_z * y$	=	0,0 + 0,0 + 0,0	=	0,0 kN/cm²
$\tau_{i,Ed}$		=	95 * 116 / (1509 * 0,70)	=	11,0 kN/cm²
σ_v		=	$\sqrt{}$ (0,0² + 3 * 11,0²)	=	19,1 kN/cm²
σ_v / σ_{Rd}		=	19,1 / 35,5	=	**0,54 < 1**

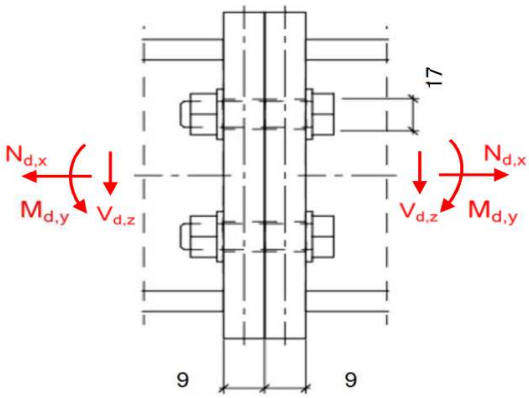
maßgebend: "Steg mittig"

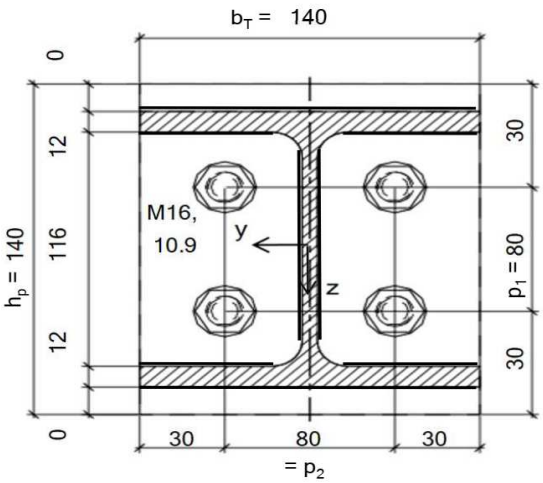
GOLDBECK

MOMENT RESISTANT BEAM SPLICES (EN 1993-1-8:2010-12) 2020-07-02

National Annex: Deutschland







Horizontal force	$N_{d,x}$:	0,0 kN
Vertical force	$V_{d,z}$:	94,6 kN
Vertical force	$V_{d,y}$:	0,0 kN
Moment	$M_{d,y}$:	4,3 kNm
Moment	$M_{d,z}$:	0,0 kNm
Weld thickn. (flange)	a_F:	5,0 mm
Weld thickn. (web)	a_S:	5,0 mm
Girder profile:		HEB 140
Girder material:		S 355
Thickness of plates	t_P:	9,0 mm
Material of plates:		S 235
Height of plates	h_P:	14,0 cm
Width of end plate	b_T:	14,0 cm
Hole pitch	p_1:	8,0 cm
Hole pitch	p_2:	8,0 cm
Bolt:		M16, 10.9
Hole clearances	Δd:	1 mm
Type of preload:		nicht voll vorgesp.

GOLDBECK

Summary

1. End plate connection (equivalent T-stub)

1.1 Tensile force resistance (plate)

$$F_{T,1,Rd} = \frac{4 \cdot M_{pl,1,Rd}}{m} = \frac{4 \cdot 92{,}1\ \text{kNcm}}{3{,}08\ \text{cm}} = 119{,}6\ \text{kN}$$

$$F_{T,2,Rd} = \frac{2 \cdot M_{pl,2,Rd} + n \cdot \sum F_{t,Rd}}{m + n} = 146{,}5\ \text{kN}$$

$$F_{T,3,Rd} = \sum F_{t,Rd} = 2 \cdot 113{,}0\ \text{kN} = 226{,}0\ \text{kN}$$

$$F_{T,4,Rd} = \frac{2 \cdot M_{pl,4,Rd}}{m} = \frac{2 \cdot 92{,}1\ \text{kNcm}}{3{,}08\ \text{cm}} = 59{,}8\ \text{kN}$$

$$F_{t,wb,Rd} = b_{eff,t,wb} \cdot t_{wb} \cdot \frac{f_{y,wb}}{\gamma_{M0}} = 19{,}4\ \text{cm} \cdot 0{,}70\ \text{cm} \cdot \frac{35{,}5\ \text{kN/cm}^2}{1{,}00} = 480{,}8\ \text{kN}$$

$$F_{w,Rd} = b_{eff,t,wb} \cdot \sum a \cdot \frac{f_u}{\sqrt{2} \cdot \beta_w \cdot \gamma_{M2}} = 492{,}6\ \text{kN}$$

1.2 Moment resistance of the plate

$$M_{y,j,Rd} = F_{S,Rd,y} \cdot (p_1 + e_z - u_1 - 0{,}5\, t_{fb}) = 1.243{,}9\ \text{kNcm}$$

1.3 Tensile force resistance

$$N_{j,Rd,t} = \sum F_{t,Rd} = 4 \cdot 113{,}0\ \text{kN} = 452{,}0\ \text{kN}$$

1.4 Compression force resistance

$$N_{j,Rd,p} = A \cdot \sigma_{Rd} = A \cdot \frac{f_y}{\gamma_{M0}} = 43{,}0\ \text{cm}^2 \cdot \frac{35{,}5\ \text{kN/cm}^2}{1{,}00} = 1526{,}5\ \text{kN}$$

1.5 Moment resistance of the joint

$$M_{z,j,Rd} = \left(\frac{p_2}{2} + \frac{b_{fb}}{4} \right) \cdot F_{S,Rd,z} = 897{,}1\ \text{kNcm}$$

1.6 Proof

$$\frac{M_{y,Ed}^*}{M_{y,j,Rd}} + \frac{N_{ed}^*}{N_{j,Rd}} + \frac{M_{z,Ed}^*}{M_{z,j,Rd}} \overset{!}{\leq} 1 \qquad \text{fulfilled:} \quad 0{,}35 < 1$$

2. Weld connection

$\sigma_{V,w,Ed} \overset{!}{\leq} f_{1,w,Rd}$ fulfilled: 18,0 kN/cm² < 36,0 kN/cm²

$\sigma_{\perp,Ed} \overset{!}{\leq} f_{2,w,Rd}$ fulfilled: 1,8 kN/cm² < 25,9 kN/cm²

3. Bolt connection

3.1 Combined shear and tension

$$F_{v,Ed} = \sqrt{(V_{Vd,z})^2 + (V_{Vd,y})^2} = \sqrt{(23{,}7\ \text{kN})^2 + (0{,}0\ \text{kN})^2} = 23{,}7\ \text{kN}$$

$F_{t,Ed} = 20{,}7$ kN $F_{v,Rd} = 96{,}5$ kN $F_{t,Rd} = 113{,}0$ kN

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1{,}4 \cdot F_{t,Rd}} \overset{!}{\leq} 1 \qquad \text{fulfilled:} \quad 0{,}38 < 1$$

3.2 Bearing

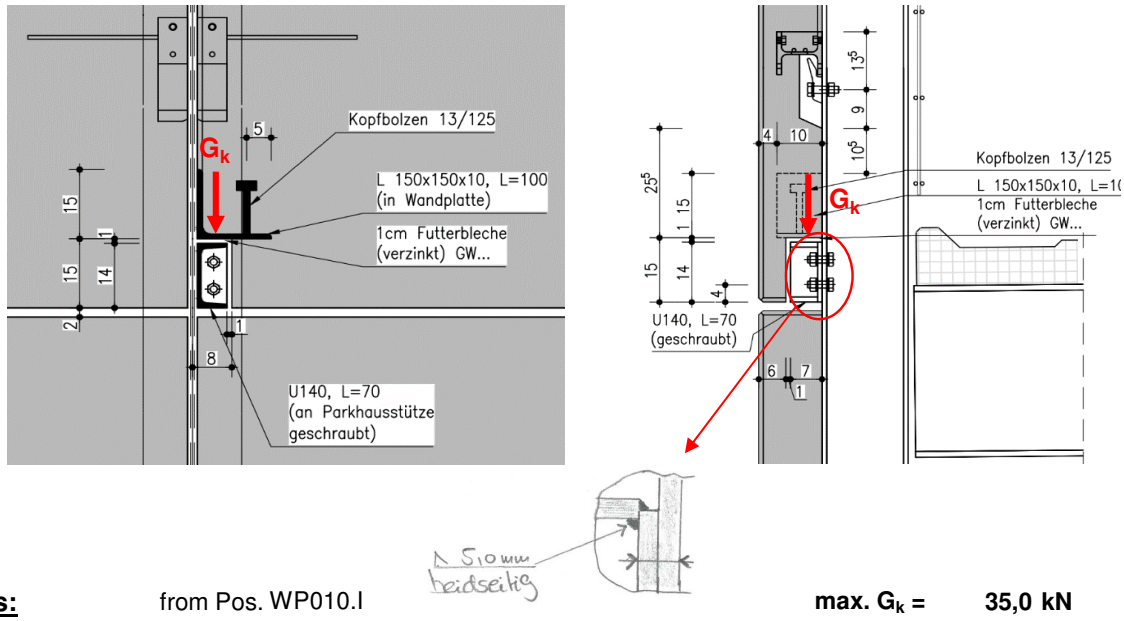
$F_{v,Ed,z} \overset{!}{\leq} F_{b,Rd,z}$ fulfilled: 23,7 kN < 61,0 kN 0

GOLDBECK

Pos WP010.d Konsole U 140 (screwed)

V. 09.08.19

System:



Loads: from Pos. WP010.I — **max. G_k = 35,0 kN**

Internal forces: 1,35 * 35,0 kN — **max. V_d = 47,3 kN**
47,3 kN * 0,05 m — **max. M_d = 2,3 kN**

Dimensioning: see computing

U 140, S355
2 screws M16 10.9
welding seam at end plate aw = 5 mm, circulating

Profil: $W_y = 14{,}0^2 * 0{,}70 / 6 = 22{,}90\ cm^2$ (approach only web)

Bending: $$vorh.\ \delta_d = \frac{2{,}36\ kNm}{22{,}9\ cm^2} = 103{,}0\ \frac{N}{mm^2} < 355$$

Shear force: $$vorh.\ T_{\parallel,d} = \frac{47{,}30\ kN}{140\ mm\ * 7{,}0\ mm} = 48{,}2\ \frac{N}{mm^2} < 204$$

Weld: $W_{w,r} = 14{,}0^2 * 2 * 0{,}50 / 6 = 32{,}70\ cm^2$ (approach only web)

Bending: $$vorh.\ \delta_{v,d} = \frac{2{,}36\ kNm}{32{,}7\ cm^2} = 72{,}2\ \frac{N}{mm^2} < 355$$

Shear force: $$vorh.\ T_{\parallel,d} = \frac{47{,}30\ kN}{2\ * 140\ mm\ * 5{,}0\ mm} = 33{,}8\ \frac{N}{mm^2} < 204$$

Load transmission: see analysis of console HEB 140

analysis end-plate + screws

at half load and similar execution as console HEB 140, there is no additional analysis required

Chapter 9: thermal design

The governing columns in each row are verified individually.

Fire protection verification for RO560 and BE451 is provided in Chapter 4.

FP001 **Structural fire design - steel column at** **external axis**

axis: **A / 12** **floor:** **1** **facade:** **permeable**

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system Version: 21.07.2025

profile: **HE 180 B** **S 355**

effective length	
l_{ky}	l_{kz}
430 cm	430 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
301 cm	301 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
188 kN	228 kN	80 kN	15,0 kNm	3,4 kNm	0,0 kNm	0,0 kNm	2,9 kNm

combination values:

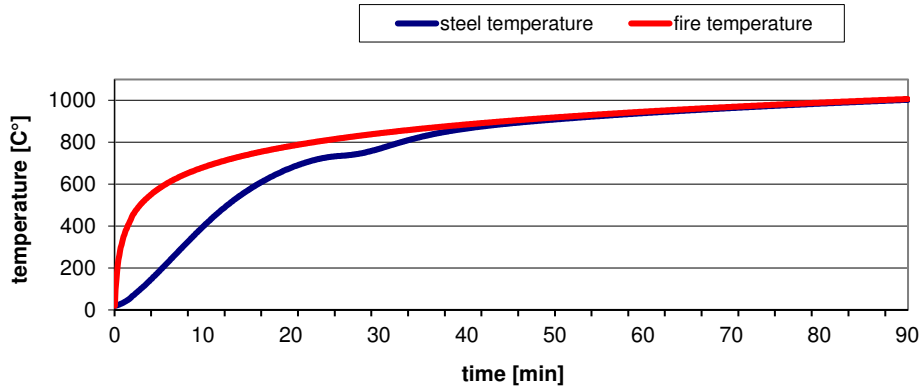
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
716 kN	**29,6 kNm**	**0,0 kNm**	**347,9 kN**	**17,4 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 180 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,17 < 1
flexural buckling y-axis:	0,40 < 1
flexural buckling z-axis:	0,74 < 1
lateral torsional buckling y-axis:	0,53 < 1
lateral torsional buckling z-axis:	0,88 < 1
determinant design values:	**0,88 < 1**

5. structural design on fire

profile factor Am/V: 160 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 562,9 °C

bending:	0,17 < 1
flexural buckling y-axis:	0,37 < 1
flexural buckling z-axis:	0,53 < 1
lateral torsional buckling:	0,75 < 1
determinant design values:	**0,75 < 1**

FP002 Structural fire design - steel column at external axis

axis: A / 23 **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system

Version: 21.07.2025

profile:	HE 180 B	S 355

effective length	
l_{ky}	l_{kz}
430 cm	430 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
301 cm	301 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
202 kN	242 kN	0 kN	16,2 kNm	3,7 kNm	0,0 kNm	0,0 kNm	3,1 kNm

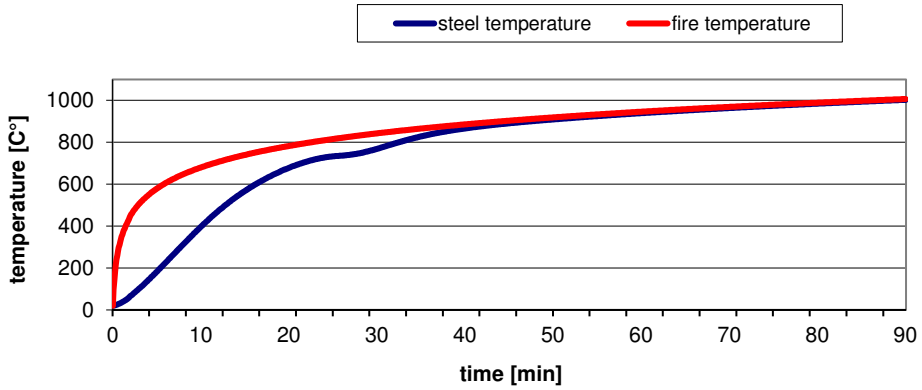
combination values:

$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
leading action:	**imposed load**			$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
635 kN	**32,0 kNm**	**0,0 kNm**	**370,8 kN**	**18,8 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 180 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,19 < 1
flexural buckling y-axis:	0,36 < 1
flexural buckling z-axis:	0,65 < 1
lateral torsional buckling y-axis:	0,50 < 1
lateral torsional buckling z-axis:	0,81 < 1
determinant design values:	**0,81 < 1**

5. structural design on fire

profile factor Am/V: 160 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 562,9 °C

bending:	0,19 < 1
flexural buckling y-axis:	0,39 < 1
flexural buckling z-axis:	0,57 < 1
lateral torsional buckling:	0,80 < 1
determinant design values:	**0,80 < 1**

FP003 Structural fire design - steel column at middle axis

axis: H / 11 **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 240 A | S 355

effective length	
l_{ky}	l_{kz}
280 cm	280 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
196 cm	196 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
368 kN	312 kN	90 kN	18,7 kNm	8,1 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:

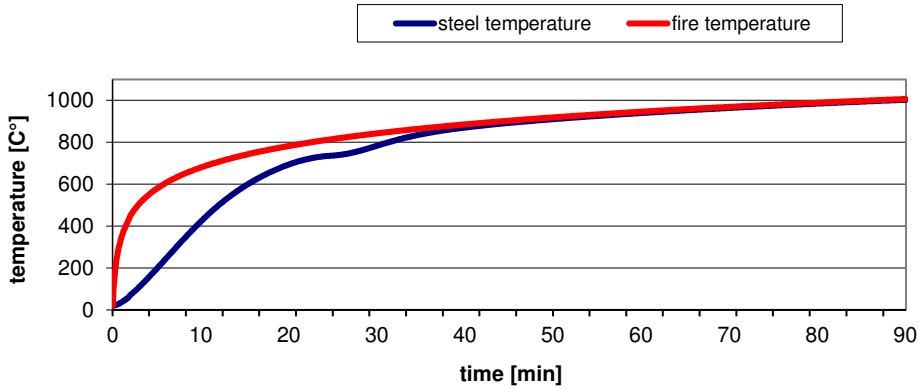
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
1100 kN	**37,4 kNm**	**0,0 kNm**	**586,5 kN**	**24,4 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 240 A

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	2	flange:	3
profile:	**2**	**profile:**	**3**

4. structural design

bending:	0,14 < 1
flexural buckling y-axis:	0,43 < 1
flexural buckling z-axis:	0,52 < 1
lateral torsional buckling y-axis:	0,52 < 1
lateral torsional buckling z-axis:	0,65 < 1
determinant design values:	**0,65 < 1**

5. structural design on fire

profile factor Am/V: 179 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 586,6 °C

bending:	0,18 < 1
flexural buckling y-axis:	0,49 < 1
flexural buckling z-axis:	0,56 < 1
lateral torsional buckling:	0,80 < 1
determinant design values:	**0,80 < 1**

FP004 Structural fire design - steel column at middle axis

axis: H / 6 **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 240 A | S 355

effective length	
l_{ky}	l_{kz}
280 cm	280 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
196 cm	196 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
395 kN	339 kN	0 kN	20,2 kNm	8,8 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:

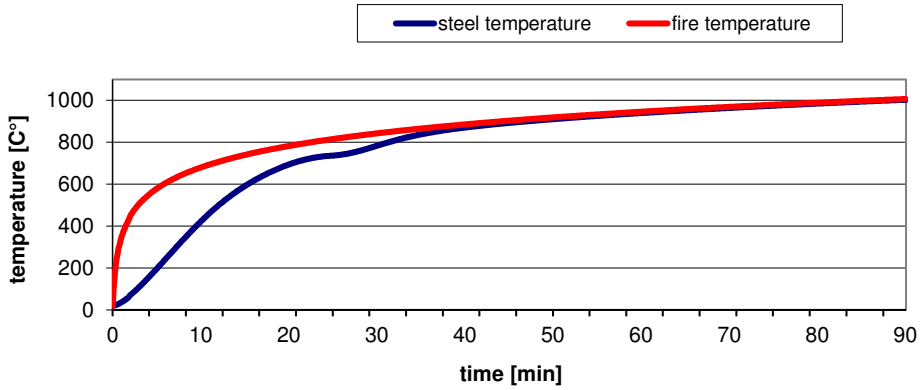
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
1042 kN	**40,4 kNm**	**0,0 kNm**	**632,4 kN**	**26,3 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 240 A

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	2	flange:	3
profile:	**2**	**profile:**	**3**

4. structural design

bending:	0,15 < 1
flexural buckling y-axis:	0,41 < 1
flexural buckling z-axis:	0,49 < 1
lateral torsional buckling y-axis:	0,50 < 1
lateral torsional buckling z-axis:	0,63 < 1
determinant design values:	**0,63 < 1**

5. structural design on fire

profile factor Am/V: 179 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 586,6 °C

bending:	0,19 < 1
flexural buckling y-axis:	0,53 < 1
flexural buckling z-axis:	0,61 < 1
lateral torsional buckling:	0,86 < 1
determinant design values:	**0,86 < 1**

FP005 Structural fire design - steel column at middle axis

axis: O / 2 **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 220 B | S 355

effective length	
l_{ky}	l_{kz}
280 cm	280 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
196 cm	196 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
659 kN	545 kN	0 kN	1,6 kNm	4,4 kNm	8,6 kNm	6,8 kNm	0,0 kNm

combination values:

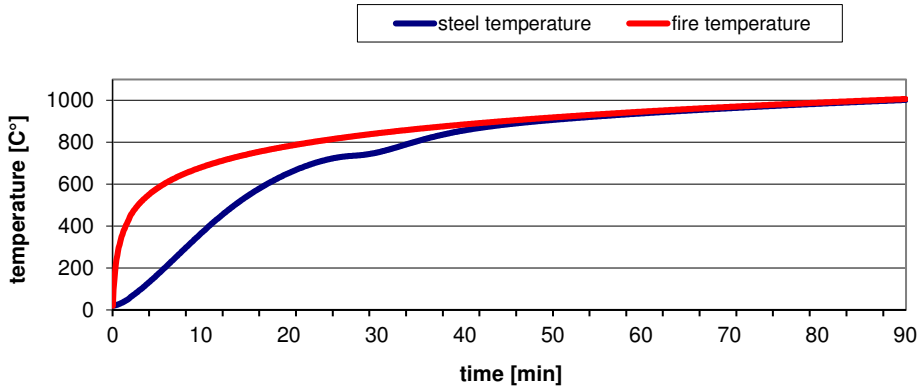
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
1706 kN	**8,8 kNm**	**21,9 kNm**	**1039,7 kN**	**4,7 kNm**	**13,4 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 220 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,03 < 1
flexural buckling y-axis:	0,57 < 1
flexural buckling z-axis:	0,70 < 1
lateral torsional buckling y-axis:	0,67 < 1
lateral torsional buckling z-axis:	0,87 < 1
determinant design values:	**0,87 < 1**

5. structural design on fire

profile factor Am/V: 140 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 530,2 °C

bending:	0,02 < 1
flexural buckling y-axis:	0,55 < 1
flexural buckling z-axis:	0,63 < 1
lateral torsional buckling:	0,75 < 1
determinant design values:	**0,75 < 1**

FP006 Structural fire design - steel column at middle axis

axis: O / 6 **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 240 A | S 355

effective length	
l_{ky}	l_{kz}
280 cm	280 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
196 cm	196 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
395 kN	339 kN	0 kN	0,0 kNm	4,4 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:

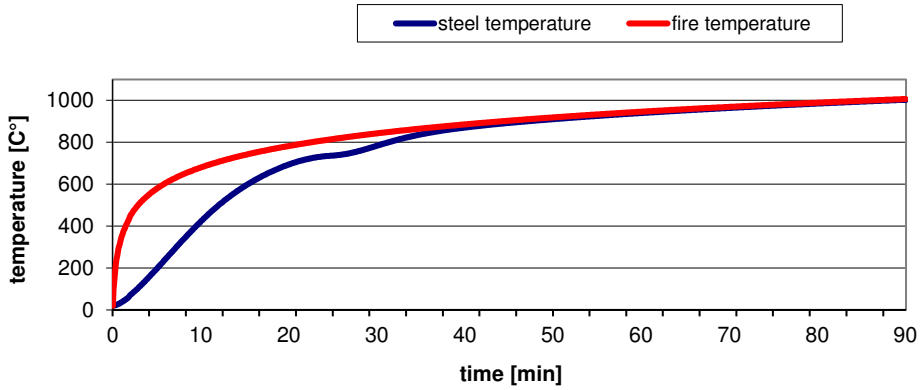
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
1042 kN	**6,6 kNm**	**0,0 kNm**	**632,4 kN**	**3,1 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 240 A

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	2	flange:	3
profile:	**2**	**profile:**	**3**

4. structural design

bending:	0,02 < 1
flexural buckling y-axis:	0,41 < 1
flexural buckling z-axis:	0,49 < 1
lateral torsional buckling y-axis:	0,42 < 1
lateral torsional buckling z-axis:	0,51 < 1
determinant design values:	**0,51 < 1**

5. structural design on fire

profile factor Am/V: 179 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 586,6 °C

bending:	0,02 < 1
flexural buckling y-axis:	0,53 < 1
flexural buckling z-axis:	0,61 < 1
lateral torsional buckling:	0,64 < 1
determinant design values:	**0,64 < 1**

FP007 Structural fire design - steel column at external axis

axis: V / 2 **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 180 B | S 355

effective length	
l_{ky}	l_{kz}
300 cm	300 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
210 cm	210 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
390 kN	364 kN	0 kN	5,2 kNm	4,4 kNm	0,0 kNm	0,0 kNm	1,9 kNm

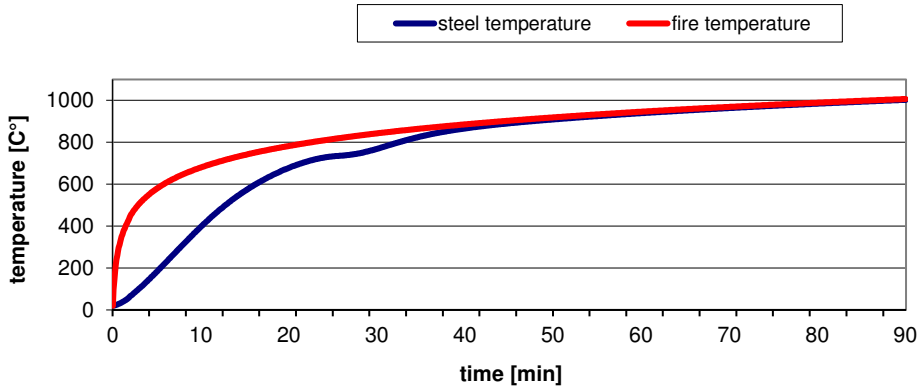
combination values:

$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
leading action:	**imposed load**			$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
1072 kN	**16,4 kNm**	**0,0 kNm**	**644,5 kN**	**8,3 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 180 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,10 < 1
flexural buckling y-axis:	0,53 < 1
flexural buckling z-axis:	0,74 < 1
lateral torsional buckling y-axis:	0,59 < 1
lateral torsional buckling z-axis:	0,82 < 1
determinant design values:	**0,82 < 1**

5. structural design on fire

profile factor Am/V: 160 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 562,9 °C

bending:	0,08 < 1
flexural buckling y-axis:	0,60 < 1
flexural buckling z-axis:	0,74 < 1
lateral torsional buckling:	0,84 < 1
determinant design values:	**0,84 < 1**

FP008 Structural fire design - steel column at external axis

axis: V / 2 **floor:** 2 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 200 A | S 355

effective length	
l_{ky}	l_{kz}
280 cm	280 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
196 cm	196 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
130 kN	110 kN	0 kN	5,2 kNm	4,4 kNm	0,0 kNm	0,0 kNm	1,9 kNm

combination values:

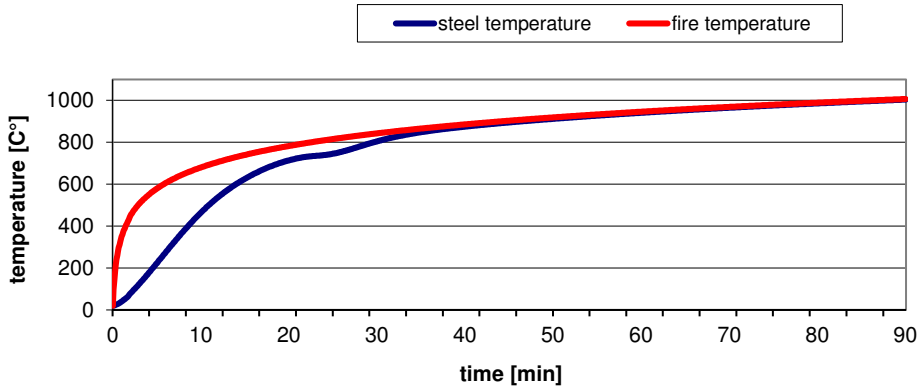
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
341 kN	**16,4 kNm**	**0,0 kNm**	**207,0 kN**	**8,3 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 A

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	2	flange:	3
profile:	**2**	**profile:**	**3**

4. structural design

bending:	0,11 < 1
flexural buckling y-axis:	0,20 < 1
flexural buckling z-axis:	0,25 < 1
lateral torsional buckling y-axis:	0,26 < 1
lateral torsional buckling z-axis:	0,36 < 1
determinant design values:	**0,36 < 1**

5. structural design on fire

profile factor Am/V: 212 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 621,3 °C

bending:	0,13 < 1
flexural buckling y-axis:	0,32 < 1
flexural buckling z-axis:	0,38 < 1
lateral torsional buckling:	0,55 < 1
determinant design values:	**0,55 < 1**

FP009 Structural fire design - steel column at external axis

axis: V / 6 **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system

Version: 21.07.2025

profile:		
HE 200 A	S 355	

effective length	
l_{k_y}	l_{k_z}
300 cm	300 cm

effective lengthh on fire	
$l_{k\Theta_y}$	$l_{k\Theta_z}$
210 cm	210 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
199 kN	194 kN	0 kN	5,2 kNm	4,4 kNm	0,0 kNm	0,0 kNm	1,3 kNm

combination values:

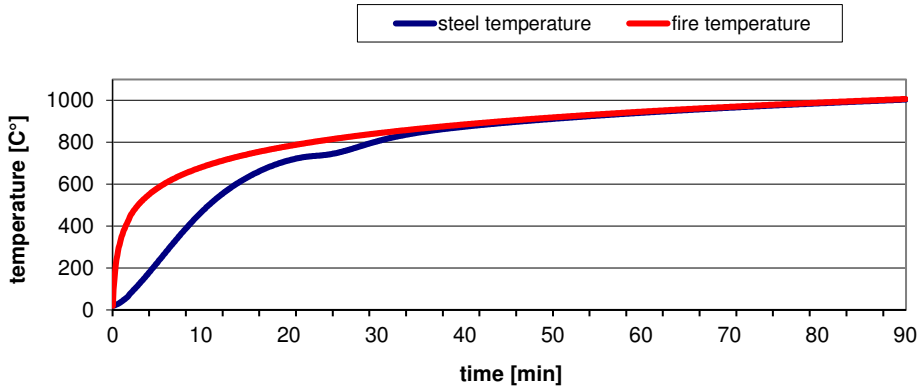
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
560 kN	**15,6 kNm**	**0,0 kNm**	**334,8 kN**	**8,3 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 A

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	2	flange:	3
profile:	**2**	**profile:**	**3**

4. structural design

bending:	0,10 < 1
flexural buckling y-axis:	0,33 < 1
flexural buckling z-axis:	0,44 < 1
lateral torsional buckling y-axis:	0,39 < 1
lateral torsional buckling z-axis:	0,53 < 1
determinant design values:	**0,53 < 1**

5. structural design on fire

profile factor Am/V: 212 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 621,3 °C

bending:	0,13 < 1
flexural buckling y-axis:	0,52 < 1
flexural buckling z-axis:	0,64 < 1
lateral torsional buckling:	0,81 < 1
determinant design values:	**0,81 < 1**

FP010 **Structural fire design - steel column at** **gable axis**

axis: **1 / A** **floor:** **1** **facade:** **permeable**

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system Version: 21.07.2025

profile: **HE 160 B** **S 355**

effective length	
l_{ky}	l_{kz}
460 cm	460 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
322 cm	322 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
18 kN	86 kN	0 kN	6,2 kNm	0,4 kNm	5,9 kNm	2,2 kNm	2,2 kNm

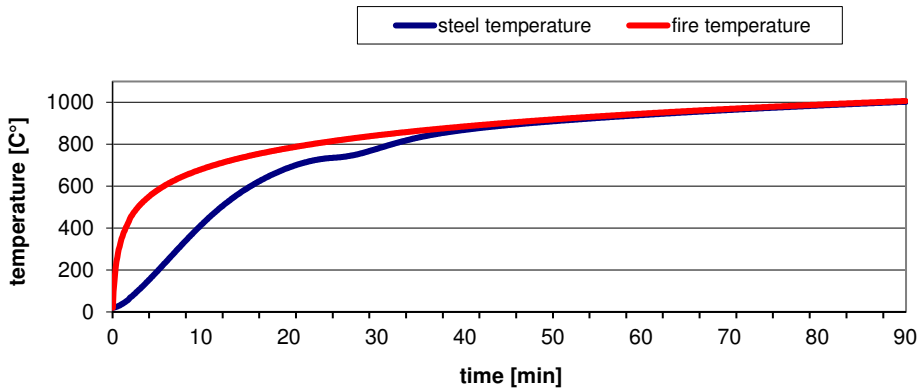
combination values:

$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
leading action:	**imposed load**			$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
153 kN	**12,2 kNm**	**11,3 kNm**	**77,9 kN**	**6,5 kNm**	**7,5 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 160 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,10 < 1
flexural buckling y-axis:	0,12 < 1
flexural buckling z-axis:	0,25 < 1
lateral torsional buckling y-axis:	0,38 < 1
lateral torsional buckling z-axis:	0,60 < 1
determinant design values:	**0,60 < 1**

5. structural design on fire

profile factor Am/V: 170 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 578,5 °C

bending:	0,10 < 1
flexural buckling y-axis:	0,12 < 1
flexural buckling z-axis:	0,20 < 1
lateral torsional buckling:	0,56 < 1
determinant design values:	**0,56 < 1**

FP011 Structural fire design - steel column at gable axis

axis: 1 / B **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system Version: 21.07.2025

profile:		
HE 160 B	S 355	

effective length	
l_{ky}	l_{kz}
460 cm	460 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
322 cm	322 cm

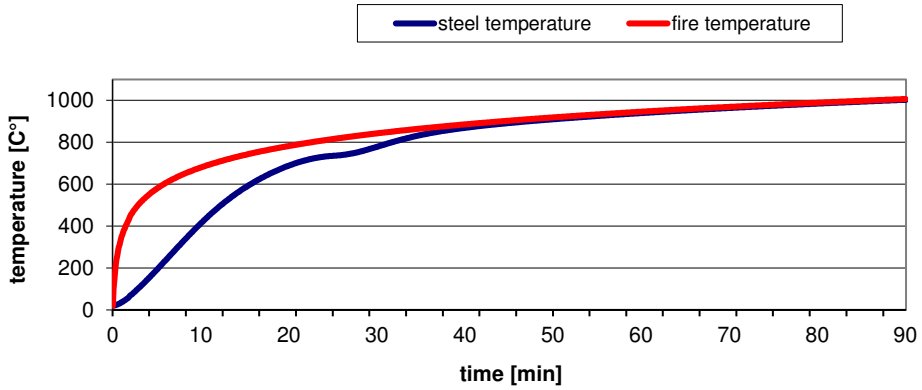
2. actions

characteristic actions:	$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
	43 kN	110 kN	0 kN	12,0 kNm	1,0 kNm	0,0 kNm	0,0 kNm	4,0 kNm

combination values:								
	$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
	γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
leading action:	**imposed load**				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
					$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

design actions:	N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
	223 kN	**23,7 kNm**	**0,0 kNm**	**120,2 kN**	**12,7 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 160 B	$f_{y,d}$ = 35,5 kN/cm²	$f_{y,fi}$ = 35,5 kN/cm²
	ε = 0,81	ε = 0,69

section properties:		
web:	1	1
flange:	1	1
profile:	**1**	**1**

4. structural design

bending:	0,19 < 1
flexural buckling y-axis:	0,17 < 1
flexural buckling z-axis:	0,36 < 1
lateral torsional buckling y-axis:	0,40 < 1
lateral torsional buckling z-axis:	0,55 < 1
determinant design values:	**0,55 < 1**

5. structural design on fire

profile factor Am/V: 170 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 578,5 °C

bending:	0,19 < 1
flexural buckling y-axis:	0,19 < 1
flexural buckling z-axis:	0,31 < 1
lateral torsional buckling:	0,55 < 1
determinant design values:	**0,55 < 1**

FP012 Structural fire design - steel column at gable axis

axis: 28 / A **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 160 B | S 355

effective length	
l_{ky}	l_{kz}
460 cm	460 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
322 cm	322 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
18 kN	64 kN	0 kN	6,2 kNm	0,4 kNm	5,9 kNm	2,2 kNm	2,2 kNm

combination values:

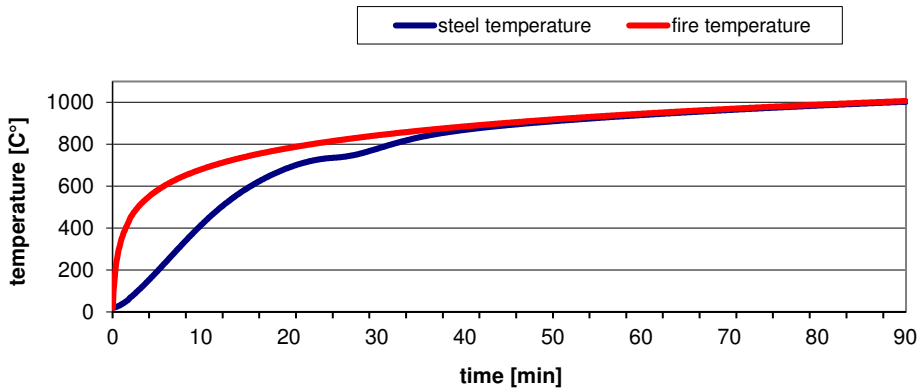
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
120 kN	**12,2 kNm**	**11,3 kNm**	**62,6 kN**	**6,5 kNm**	**7,5 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 160 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,10 < 1
flexural buckling y-axis:	0,09 < 1
flexural buckling z-axis:	0,20 < 1
lateral torsional buckling y-axis:	0,35 < 1
lateral torsional buckling z-axis:	0,53 < 1
determinant design values:	**0,53 < 1**

5. structural design on fire

profile factor Am/V: 170 1/m four-sided fire

design at t = 15 min

fire temperatur = 738,6 °C

steel temperature = 578,5 °C

bending:	0,10 < 1
flexural buckling y-axis:	0,10 < 1
flexural buckling z-axis:	0,16 < 1
lateral torsional buckling:	0,52 < 1
determinant design values:	**0,52 < 1**

FP013 Structural fire design - steel column at gable axis

axis: 28 / B **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 160 B | S 355

effective length	
l_{ky}	l_{kz}
460 cm	460 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
322 cm	322 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
43 kN	66 kN	0 kN	12,0 kNm	1,0 kNm	0,0 kNm	0,0 kNm	4,0 kNm

combination values:

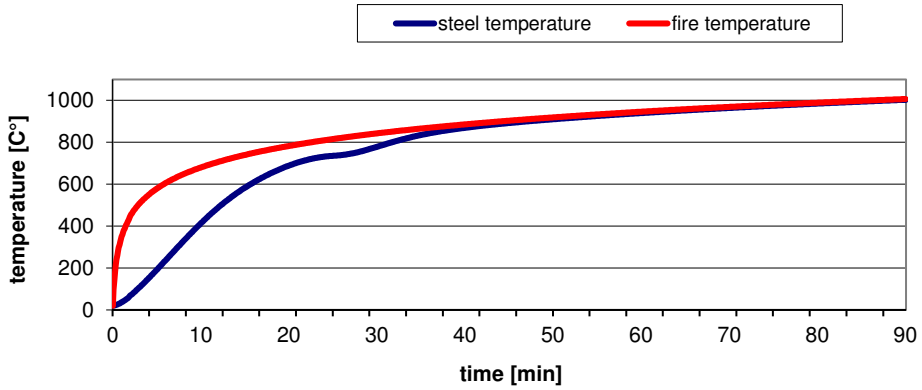
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
158 kN	**23,7 kNm**	**0,0 kNm**	**89,6 kN**	**12,7 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 160 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,19 < 1
flexural buckling y-axis:	0,12 < 1
flexural buckling z-axis:	0,26 < 1
lateral torsional buckling y-axis:	0,34 < 1
lateral torsional buckling z-axis:	0,45 < 1
determinant design values:	**0,45 < 1**

5. structural design on fire

profile factor Am/V: 170 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 578,5 °C

bending:	0,19 < 1
flexural buckling y-axis:	0,14 < 1
flexural buckling z-axis:	0,23 < 1
lateral torsional buckling:	0,48 < 1
determinant design values:	**0,48 < 1**

FP014 Structural fire design - steel column at gable axis

axis: 2' / F **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system Version: 21.07.2025

profile:		
HE 200 B	S 355	

effective length	
l_{k_y}	l_{k_z}
420 cm	420 cm

effective lengthh on fire	
$l_{k\Theta_y}$	$l_{k\Theta_z}$
294 cm	294 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
341 kN	284 kN	0 kN	9,9 kNm	3,8 kNm	0,0 kNm	0,0 kNm	0,0 kNm

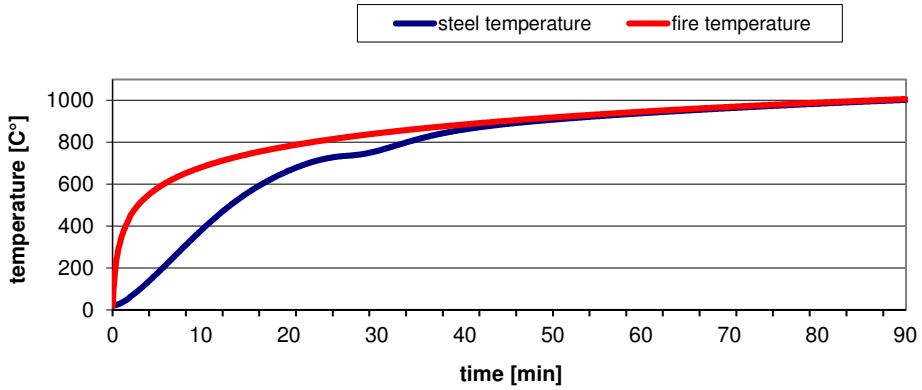
combination values:

$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
leading action:	imposed load			$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
887 kN	**19,0 kNm**	**0,0 kNm**	**540,0 kN**	**12,5 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,08 < 1
flexural buckling y-axis:	0,39 < 1
flexural buckling z-axis:	0,65 < 1
lateral torsional buckling y-axis:	0,45 < 1
lateral torsional buckling z-axis:	0,72 < 1
determinant design values:	**0,72 < 1**

5. structural design on fire

profile factor Am/V: 148 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 544,9 °C

bending:	0,09 < 1
flexural buckling y-axis:	0,41 < 1
flexural buckling z-axis:	0,55 < 1
lateral torsional buckling:	0,66 < 1
determinant design values:	**0,66 < 1**

FP015 Structural fire design - steel column at bracing axis

axis: 4' / F **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system

Version: 21.07.2025

profile:		
HE 200 B	S 355	

effective length	
l_{ky}	l_{kz}
420 cm	280 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
294 cm	294 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
340 kN	286 kN	324 kN	9,9 kNm	3,8 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:

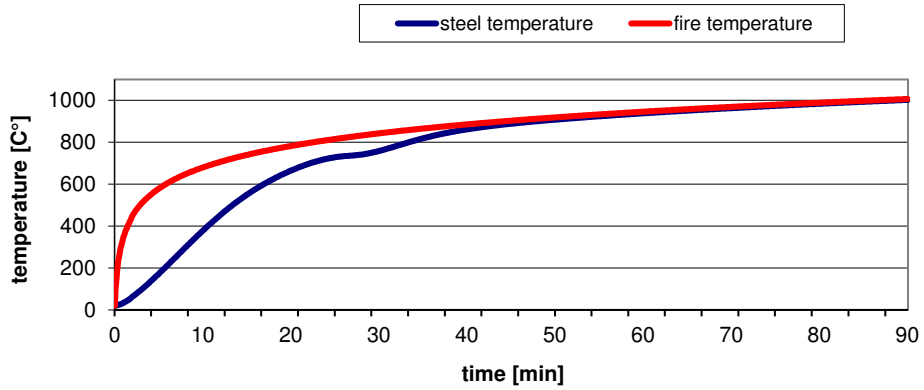
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **Wind**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
1244 kN	**19,1 kNm**	**0,0 kNm**	**575,8 kN**	**12,2 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,08 < 1
flexural buckling y-axis:	0,55 < 1
flexural buckling z-axis:	0,63 < 1
lateral torsional buckling y-axis:	0,61 < 1
lateral torsional buckling z-axis:	0,70 < 1
determinant design values:	**0,70 < 1**

5. structural design on fire

profile factor Am/V: 148 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 544,9 °C

bending:	0,08 < 1
flexural buckling y-axis:	0,44 < 1
flexural buckling z-axis:	0,59 < 1
lateral torsional buckling:	0,70 < 1
determinant design values:	**0,70 < 1**

FP016 Structural fire design - steel column at bracing axis

axis: 24' / F **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system

Version: 21.07.2025

profile:		
HE 200 B	S 355	

effective length	
l_{k_y}	l_{k_z}
280 cm	280 cm

effective lengthh on fire	
$l_{k\Theta_y}$	$l_{k\Theta_z}$
196 cm	294 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
329 kN	277 kN	324 kN	5,4 kNm	3,8 kNm	4,5 kNm	0,0 kNm	0,0 kNm

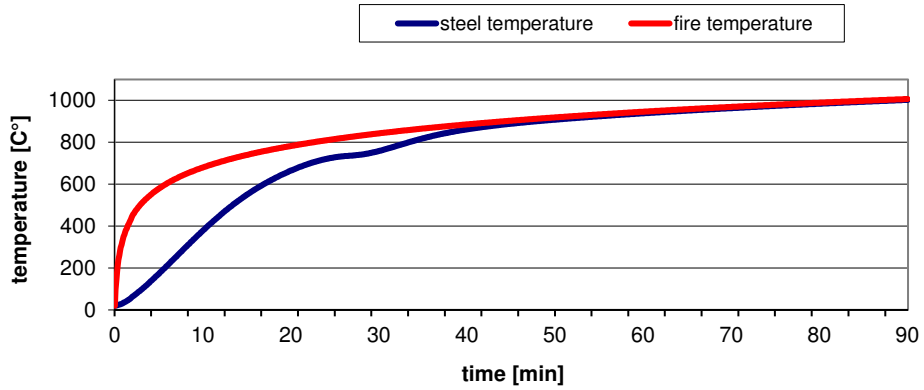
combination values:

$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
leading action:	Wind			$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
1221 kN	**13,0 kNm**	**6,1 kNm**	**559,6 kN**	**7,7 kNm**	**4,5 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,06 < 1
flexural buckling y-axis:	0,48 < 1
flexural buckling z-axis:	0,62 < 1
lateral torsional buckling y-axis:	0,55 < 1
lateral torsional buckling z-axis:	0,72 < 1
determinant design values:	**0,72 < 1**

5. structural design on fire

profile factor Am/V: 148 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 544,9 °C

bending:	0,05 < 1
flexural buckling y-axis:	0,38 < 1
flexural buckling z-axis:	0,57 < 1
lateral torsional buckling:	0,70 < 1
determinant design values:	**0,70 < 1**

FP017 Structural fire design - steel column at bracing axis

axis: 24' / J **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

Version: 21.07.2025

1. static system

profile: HE 200 B | S 355

effective length	
l_{ky}	l_{kz}
280 cm	280 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
196 cm	196 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
337 kN	286 kN	317 kN	0,9 kNm	3,8 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:

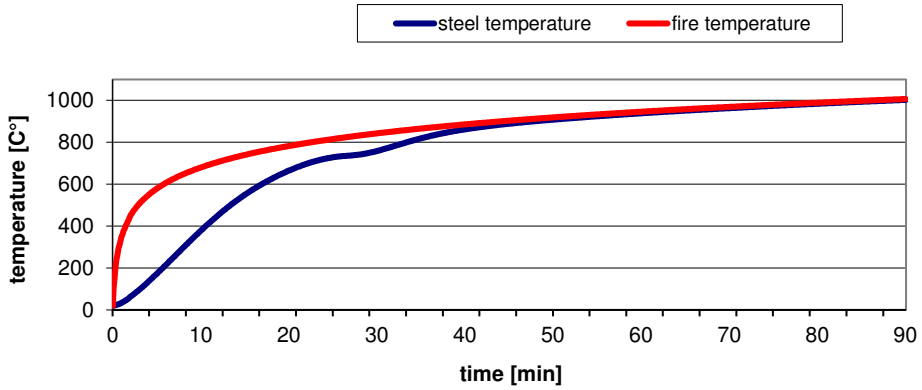
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **Wind**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
1230 kN	**6,9 kNm**	**0,0 kNm**	**571,8 kN**	**3,2 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 B

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	1
profile:	**1**	**profile:**	**1**

4. structural design

bending:	0,03 < 1
flexural buckling y-axis:	0,49 < 1
flexural buckling z-axis:	0,63 < 1
lateral torsional buckling y-axis:	0,51 < 1
lateral torsional buckling z-axis:	0,65 < 1
determinant design values:	**0,65 < 1**

5. structural design on fire

profile factor Am/V: 148 1/m four-sided fire

design at t = 15 min

fire temperatur = 738,6 °C

steel temperature = 544,9 °C

bending:	0,02 < 1
flexural buckling y-axis:	0,39 < 1
flexural buckling z-axis:	0,46 < 1
lateral torsional buckling:	0,48 < 1
determinant design values:	**0,48 < 1**

FP018 **Structural fire design - steel column at** **bracing axis**

axis: **2' / G** **floor:** **1** **facade:** **permeable**

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system Version: 21.07.2025

profile: **HE 200 A** **S 355**

effective length	
l_{ky}	l_{kz}
405 cm	405 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
284 cm	284 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
121 kN	85 kN	0 kN	12,0 kNm	1,8 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:

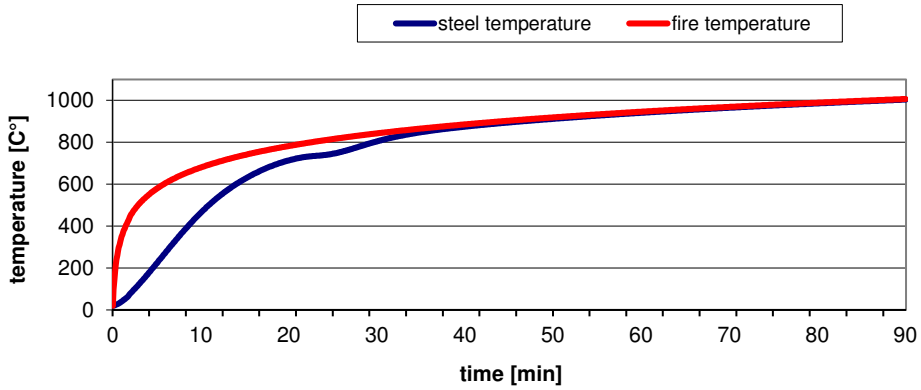
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
290 kN	**19,0 kNm**	**0,0 kNm**	**180,3 kN**	**13,3 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 A

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	2	flange:	3
profile:	**2**	**profile:**	**3**

4. structural design

bending:	0,12 < 1
flexural buckling y-axis:	0,19 < 1
flexural buckling z-axis:	0,30 < 1
lateral torsional buckling y-axis:	0,27 < 1
lateral torsional buckling z-axis:	0,42 < 1
determinant design values:	**0,42 < 1**

5. structural design on fire

profile factor Am/V: 212 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 621,3 °C

bending:	0,21 < 1
flexural buckling y-axis:	0,31 < 1
flexural buckling z-axis:	0,43 < 1
lateral torsional buckling:	0,72 < 1
determinant design values:	**0,72 < 1**

FP019 **Structural fire design - steel column at** **bracing axis**

axis: **4' / G** **floor:** **1** **facade:** **permeable**

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system Version: 21.07.2025

profile: **HE 200 A** **S 355**

effective length	
l_{ky}	l_{kz}
405 cm	280 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
284 cm	315 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
159 kN	122 kN	120 kN	11,0 kNm	1,8 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:

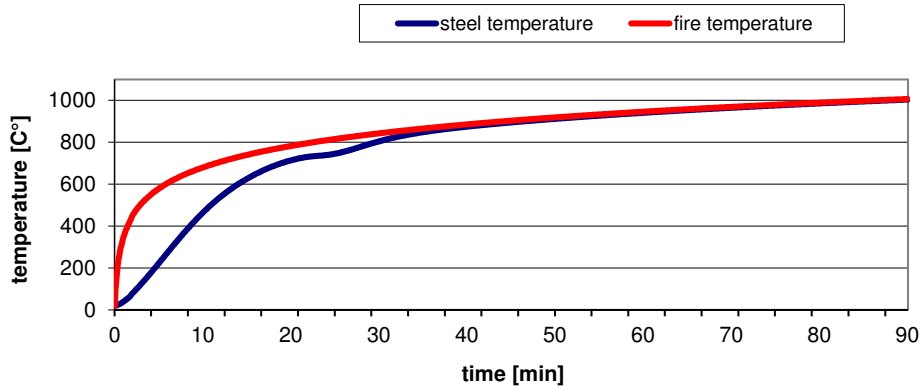
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **Wind**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
522 kN	**17,6 kNm**	**0,0 kNm**	**255,8 kN**	**12,1 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 A

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	2	flange:	3
profile:	**2**	**profile:**	**3**

4. structural design

bending:	0,12 < 1
flexural buckling y-axis:	0,33 < 1
flexural buckling z-axis:	0,39 < 1
lateral torsional buckling y-axis:	0,41 < 1
lateral torsional buckling z-axis:	0,50 < 1
determinant design values:	**0,50 < 1**

5. structural design on fire

profile factor Am/V: 212 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 621,3 °C

bending:	0,19 < 1
flexural buckling y-axis:	0,44 < 1
flexural buckling z-axis:	0,67 < 1
lateral torsional buckling:	0,94 < 1
determinant design values:	**0,94 < 1**

FP020 Structural fire design - steel column at bracing axis

axis: 4' / H **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system

Version: 21.07.2025

profile:		
HE 200 A	S 355	

effective length	
l_{k_y}	l_{k_z}
365 cm	280 cm

effective lengthh on fire	
$l_{k\Theta_y}$	$l_{k\Theta_z}$
256 cm	217 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
145 kN	116 kN	190 kN	11,7 kNm	1,8 kNm	4,5 kNm	0,0 kNm	0,0 kNm

combination values:

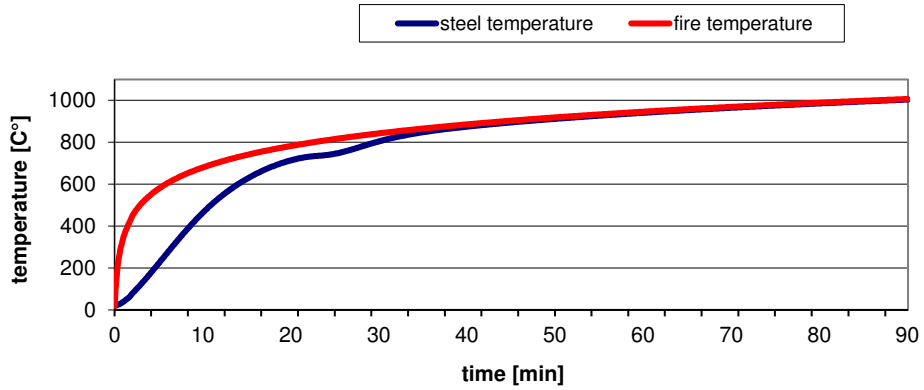
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **Wind**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
602 kN	**18,5 kNm**	**6,1 kNm**	**252,2 kN**	**12,8 kNm**	**4,5 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 A

$f_{y,d}$ =	35,5 kN/cm²
ε =	0,81

$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,69

section properties:

web:	1
flange:	2
profile:	**2**

web:	1
flange:	3
profile:	**3**

4. structural design

bending:	0,12 < 1
flexural buckling y-axis:	0,37 < 1
flexural buckling z-axis:	0,45 < 1
lateral torsional buckling y-axis:	0,50 < 1
lateral torsional buckling z-axis:	0,63 < 1
determinant design values:	**0,63 < 1**

5. structural design on fire

profile factor Am/V: 212 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 621,3 °C

bending:	0,20 < 1
flexural buckling y-axis:	0,42 < 1
flexural buckling z-axis:	0,49 < 1
lateral torsional buckling:	0,95 < 1
determinant design values:	**0,95 < 1**

FP021 **Structural fire design - steel column at** **bracing axis**

axis: **4' / I** **floor:** **1** **facade:** **permeable**

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 **Czech Republic**

1. static system Version: 21.07.2025

profile:			effective length		effective lengthh on fire	
	HE 200 A	**S 355**	l_{ky}	l_{kz}	$l_{k\Theta y}$	$l_{k\Theta z}$
			325 cm	310 cm	228 cm	217 cm

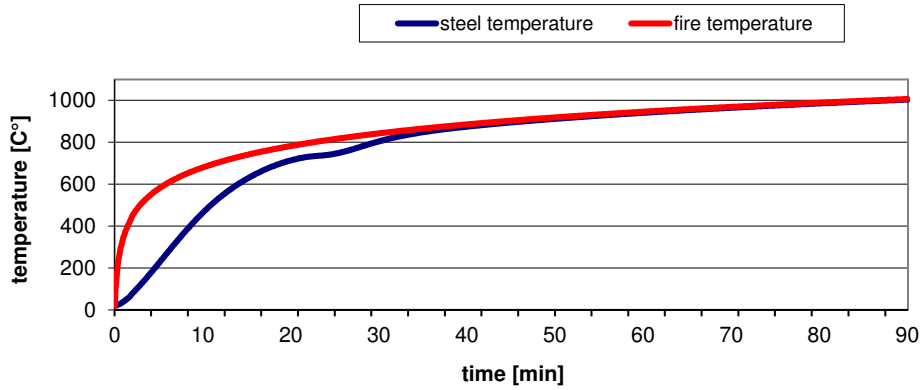
2. actions

characteristic actions:	$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
	158 kN	122 kN	157 kN	11,0 kNm	1,8 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:								
	$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
	γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
leading action:	**Wind**				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
					$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

design actions:	N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
	576 kN	**17,6 kNm**	**0,0 kNm**	**262,2 kN**	**12,1 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 200 A					
	$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²	
	ε =	0,81	ε =	0,69	
section properties:	web:	1	web:	1	
	flange:	2	flange:	3	
	profile:	**2**	**profile:**	**3**	

4. structural design

bending:	0,12 < 1
flexural buckling y-axis:	0,34 < 1
flexural buckling z-axis:	0,46 < 1
lateral torsional buckling y-axis:	0,42 < 1
lateral torsional buckling z-axis:	0,56 < 1
determinant design values:	**0,56 < 1**

5. structural design on fire

profile factor Am/V: 212 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 621,3 °C

bending:	0,19 < 1
flexural buckling y-axis:	0,42 < 1
flexural buckling z-axis:	0,51 < 1
lateral torsional buckling:	0,77 < 1
determinant design values:	**0,77 < 1**

FP022 Structural fire design - steel column at bracing axis

axis: 2' / F' **floor:** 1 **facade:** permeable

For unprotected steel cross-section-class 1-3. Analysis by EN1993-1-1 (2010) and EN1993-1-2 (2005) **Czech Republic**

1. static system

Version: 21.07.2025

profile: HE 160 A | S 355

effective length	
l_{ky}	l_{kz}
460 cm	460 cm

effective lengthh on fire	
$l_{k\Theta y}$	$l_{k\Theta z}$
322 cm	322 cm

2. actions

characteristic actions:

$N_{k,g}$	$N_{k,p}$	$N_{k,Wind}$	$M_{kg,y}$	$M_{kp,y}$	$M_{kg,z}$	$M_{kp,z}$	$M_{kw,y}$
18 kN	12 kN	0 kN	9,4 kNm	0,3 kNm	0,0 kNm	0,0 kNm	0,0 kNm

combination values:

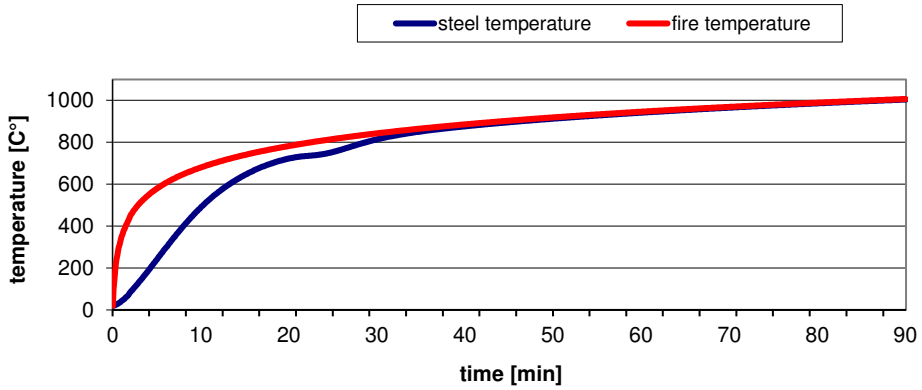
$\gamma_{M0,}$ =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00	Q: $\psi_{f,Q}$ =	0,70
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00	Q: $\psi_{f,Wind}$ =	0,00
				$\gamma_{fi,Q}$ =	1,00	W: $\psi_{f,Wind}$ =	0,20
				$\gamma_{fi,Wind}$ =	1,00	W: $\psi_{f,Q}$ =	0,60

leading action: **imposed load**

design actions:

N_{Ed}	$M_{Ed,y}$	$M_{Ed,z}$	$N_{Ed,fi}$	$M_{Ed,y,fi}$	$M_{Ed,z,fi}$
42 kN	**13,2 kNm**	**0,0 kNm**	**26,1 kN**	**9,6 kNm**	**0,0 kNm**

thermal actions: ETK



3. section properties / material skills

profile: HE 160 A

$f_{y,d}$ =	35,5 kN/cm²	$f_{y,fi}$ =	35,5 kN/cm²
ε =	0,81	ε =	0,69

section properties:

web:	1	web:	1
flange:	1	flange:	2
profile:	**1**	**profile:**	**2**

4. structural design

bending:	0,15 < 1
flexural buckling y-axis:	0,05 < 1
flexural buckling z-axis:	0,10 < 1
lateral torsional buckling y-axis:	0,15 < 1
lateral torsional buckling z-axis:	0,26 < 1
determinant design values:	**0,26 < 1**

5. structural design on fire

profile factor Am/V: 234 1/m four-sided fire

design at t = 15 min fire temperatur = 738,6 °C

steel temperature = 640,0 °C

bending:	0,30 < 1
flexural buckling y-axis:	0,09 < 1
flexural buckling z-axis:	0,14 < 1
lateral torsional buckling:	0,57 < 1
determinant design values:	**0,57 < 1**

Pos. FP100 Structural fire design - composite steel beam

V. 21.07.2025

floor beam BE034 **Area:** **parkingplace**

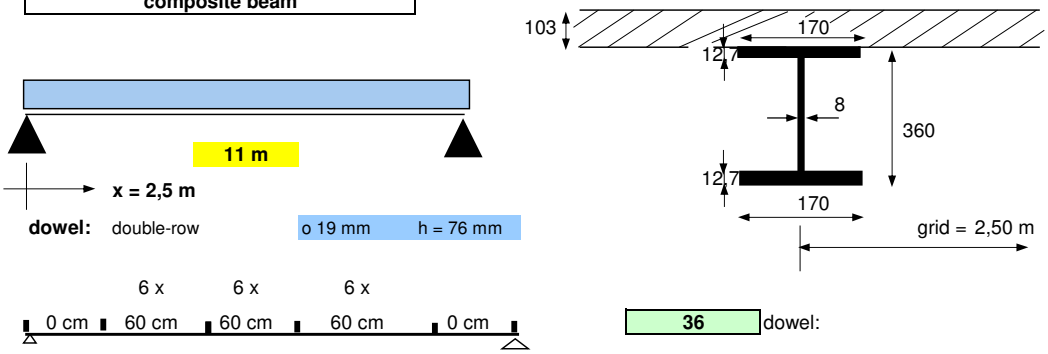
For unprotected steel cross-section-class 1-3. Analysis by EN1994-1-1 (2010) and EN1994-1-2 (2026) **Czech Republic**

1. static system

Version: 21.07.2025

profile: **IPE 360** | **S 235** | **composite beam**

concrete: **C40/50**



36 dowel:

2. actions

grid = 2,50 m

characteristic actions:

g_k	p_k	
7,259 kN/m	6,250 kN/m	Full load

combination values:

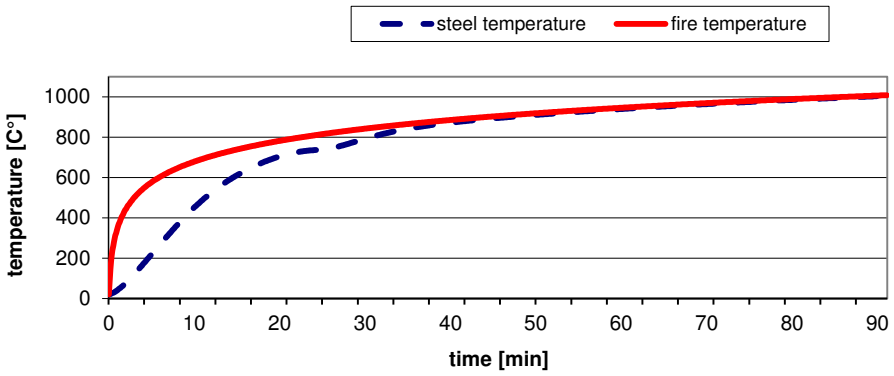
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00
				$\gamma_{fi,Q}$ =	1,00
				ψ_{fi} =	0,70

$M_{Ed,fi,y}$ design at x = **2,5 m**

design actions:

$M_{Ed,y}$	A_d	B_d	$M_{Ed,y,fi\,(x)}$	$A_{d,fi}$	$B_{d,fi}$	$V_{Ed,fi\,(x)}$
203,7 kNm	105,5 kN	105,5 kN	123,6 kNm	64,0 kN	64,0 kN	34,9 kN

thermal actions: ETK



3. section properties / material skills

profile: IPE 360

$f_{y,d}$ = 23,5 kN/cm²

$f_{y,fi}$ = 23,5 kN/cm²

4. structural design

determinant design values: **0,66 < 1**

5. structural design on fire

design at t = 15 min fire temperatur = 739 °C

maximal steel temperature bottom flange	**592 °C**	**after**	**15 min**	
moment load capacity:	124 kNm / 247 kNm	=		0,50 < 1
Interaction:	124 kNm / 247 kNm	=		0,50 < 1
maximal steel temperature web	**592 °C**	**after**	**15 min**	
shear force resistance:	35 kN / 236 kN	=		0,15 < 1

determinant design values: **0,50 < 1**

detailed design - composite steel beam (moment load capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
upper flange:	432 °C	0,67	0,40	0,93	0,93	140216	94	218	218
web:	592 °C	0,33	0,19	0,50	0,50	70034	46	116	116
bottom flange:	592 °C	0,33	0,19	0,50	0,50	70034	46	116	116

calculation of positively moment load capacity

T^+ = 1034,0 kN
y_T = 217 mm

$T<N^*P_{fi,Rd}$ $P_{Rd,1}$ = 113,4 kN $k_{u,\Theta}$ = 1,14 $Pfi_{,Rd,1}$ = 103,0 kN => gov.: 103.0 kN
$P_{Rd,2}$ = 124,3 kN $k_{c,\Theta}$ = 0,96 $Pfi_{,Rd,2}$ = 119,7 kN

$N^*P_{fi,Rd}$ = 1030,1 kN

=> gov T^+ = **1030,1 kN**

calculation of pressure zones height

b_{eff} = 192,5 cm (hc - hu) = 8,96 cm > hcr = 1,00 cm
f_c = 4,0 kN/cm²

moment load capacity:

y_F = 45,6 cm
$M_{fi,Rd}$ = 246,6 kNm NW: 124 kNm / 247 kNm = 0,50 < 1

detailed design - composite steel beam (shear force capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
web:	592 °C	0,33	0,19	0,50	0,50	70034	45,7	116,4	116,4

shear force resistance:

Av = 35,108 cm²
$f_{ay,\Theta,web}$ = 11,64 kN/cm²

$V_{fi,Rd}$ = 235,84 kN NW: 35 kN / 236 kN = 0,15 < 1

Pos. FP101 Structural fire design - composite steel beam

V. 21.07.2025

floor beam BE052 **Area:** **parkingplace**

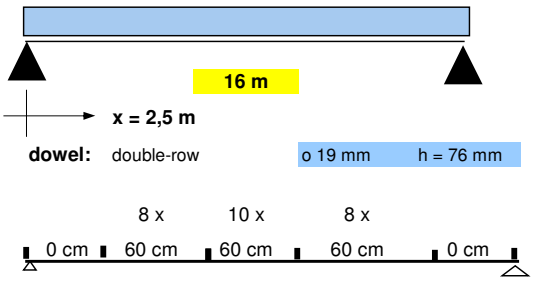
For unprotected steel cross-section-class 1-3. Analysis by EN1994-1-1 (2010) and EN1994-1-2 (2006) **Czech Republic**

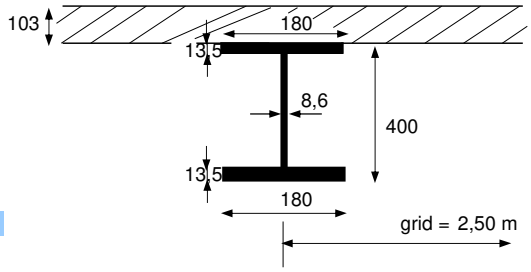
Version: 21.07.2025

1. static system

profile: **IPE 400** | **S 355** | **composite beam**

concrete: **C40/50**





52 dowel:

2. actions

grid = 2,50 m

characteristic actions:

g_k	p_k	
7,351 kN/m	6,250 kN/m	Full load

combination values:

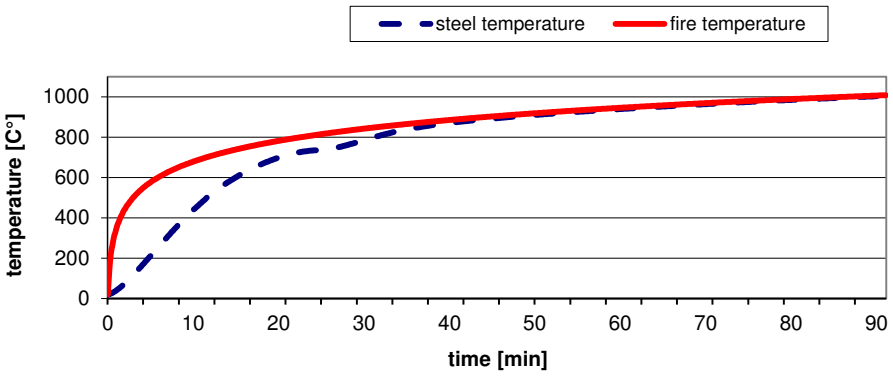
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00
				$\gamma_{fi,Q}$ =	1,00
				ψ_{fi} =	0,70

$M_{Ed,fi,y}$ design at x = **2,5 m**

design actions:

$M_{Ed,y}$	A_d	B_d	$M_{Ed,y,fi\,(x)}$	$A_{d,fi}$	$B_{d,fi}$	$V_{Ed,fi\,(x)}$
325,7 kNm	**154,4 kN**	**154,4 kN**	**197,9 kNm**	**93,8 kN**	**93,8 kN**	**64,5 kN**

thermal actions: ETK



3. section properties / material skills

profile: IPE 400

$f_{y,d}$ = 35,5 kN/cm²

$f_{y,fi}$ = 35,5 kN/cm²

4. structural design

determinant design values: **0,78 < 1**

5. structural design on fire

design at t = 15 min fire temperatur = 739 °C

maximal steel temperature bottom flange	**581 °C**	**after**	**15 min**	
moment load capacity:	198 kNm / 273 kNm	=		0,72 < 1
Interaction:	198 kNm / 273 kNm	=		0,72 < 1
maximal steel temperature web	**581 °C**	**after**	**15 min**	
shear force resistance:	64 kN / 464 kN	=		0,14 < 1

determinant design values: **0,72 < 1**

detailed design - composite steel beam (moment load capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
upper flange:	419 °C	0,68	0,41	0,96	0,96	143042	145	340	340
web:	581 °C	0,37	0,21	0,53	0,53	76822	76	188	188
bottom flange:	581 °C	0,37	0,21	0,53	0,53	76822	76	188	188

calculation of positively moment load capacity

T^+ = 1887,0 kN
y_T = 238 mm

$T<N^*P_{fi,Rd}$ $P_{Rd,1}$ = 113,4 kN $k_{u,\Theta}$ = 1,16 $Pfi_{,Rd,1}$ = 105,5 kN => gov.: 105,5 kN
$P_{Rd,2}$ = 124,3 kN $k_{c,\Theta}$ = 0,97 $Pfi_{,Rd,2}$ = 120,1 kN

$N^*P_{fi,Rd}$ = 1054,6 kN

=> gov T^+ = **1054,6 kN**

calculation of pressure zones height

b_{eff} = 217,5 cm (hc - hu) = 9,09 cm > hcr = 1,00 cm
f_c = 4,0 kN/cm²

moment load capacity:

y_F = 49,7 cm
$M_{fi,Rd}$ = 273,2 kNm **NW: 198 kNm / 273 kNm = 0,72 < 1**

detailed design - composite steel beam (shear force capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
web:	581 °C	0,37	0,21	0,53	0,53	76822	76,2	188,0	188,0

shear force resistance:

Av = 42,731 cm²
$f_{ay,\Theta,web}$ = 18,80 kN/cm²

$V_{fi,Rd}$ = 463,89 kN **NW: 64 kN / 464 kN = 0,14 < 1**

Pos. FP102 **Structural fire design - composite steel beam** V. 21.07.2025

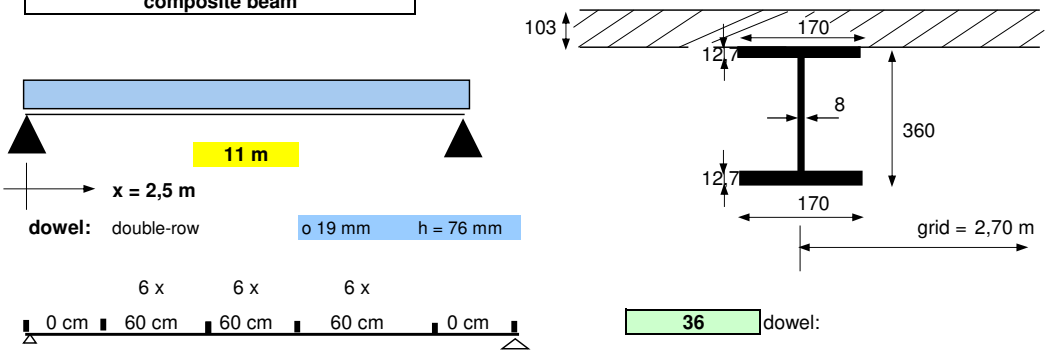
floor beam BE134 **Area:** **parkingplace**

For unprotected steel cross-section-class 1-3. Analysis by EN1994-1-1 (2010) and EN1994-1-2 (2026) **Czech Republic**

1. static system Version: 21.07.2025

profile: **IPE 360** | **S 235** — **composite beam**

concrete: **C40/50**



36 dowel:

2. actions grid = 2,70 m

characteristic actions:

g_k	p_k	
7,794 kN/m	6,750 kN/m	Full load

combination values:

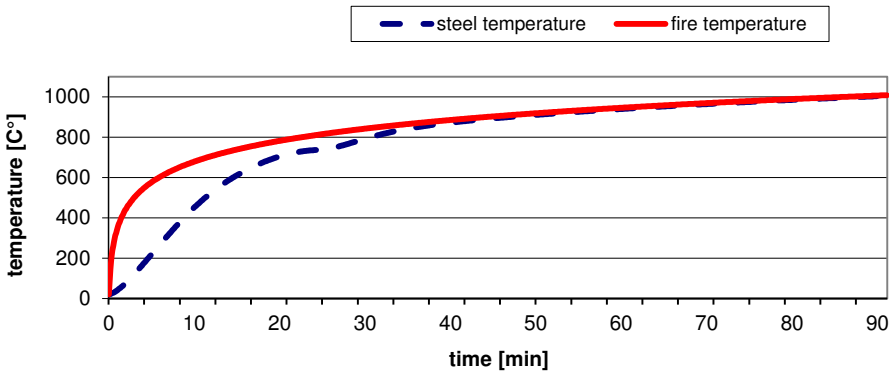
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00
				$\gamma_{fi,Q}$ =	1,00
				ψ_{fi} =	0,70

$M_{Ed,fi,y}$ design at x = **2,5 m**

design actions:

$M_{Ed,y}$	A_d	B_d	$M_{Ed,y,fi\,(x)}$	$A_{d,fi}$	$B_{d,fi}$	$V_{Ed,fi\,(x)}$
219,4 kNm	113,6 kN	113,6 kN	133,0 kNm	68,9 kN	68,9 kN	37,6 kN

thermal actions: ETK



3. section properties / material skills

profile: IPE 360 $f_{y,d}$ = 23,5 kN/cm² $f_{y,fi}$ = 23,5 kN/cm²

4. structural design

determinant design values: **0,71 < 1**

5. structural design on fire

design at t = 15 min fire temperatur = 739 °C

maximal steel temperature bottom flange	**592 °C**	**after**	**15 min**	
moment load capacity:	133 kNm / 247 kNm	=		0,54 < 1
Interaction:	133 kNm / 247 kNm	=		0,54 < 1
maximal steel temperature web	**592 °C**	**after**	**15 min**	
shear force resistance:	38 kN / 236 kN	=		0,16 < 1

determinant design values: **0,54 < 1**

detailed design - composite steel beam (moment load capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
upper flange:	432 °C	0,67	0,40	0,93	0,93	140216	94	218	218
web:	592 °C	0,33	0,19	0,50	0,50	70034	46	116	116
bottom flange:	592 °C	0,33	0,19	0,50	0,50	70034	46	116	116

calculation of positively moment load capacity

T^+ = 1034,0 kN
y_T = 217 mm

$T<N^*P_{fi,Rd}$ $P_{Rd,1}$ = 113,4 kN $k_{u,\Theta}$ = 1,14 $Pfi_{,Rd,1}$ = 103,0 kN => gov.: 103.0 kN
$P_{Rd,2}$ = 124,3 kN $k_{c,\Theta}$ = 0,96 $Pfi_{,Rd,2}$ = 119,7 kN

$N^*P_{fi,Rd}$ = 1030,1 kN

=> gov T^+ = **1030,1 kN**

calculation of pressure zones height

b_{eff} = 203,5 cm (hc - hu) = 9,03 cm > hcr = 1,00 cm
f_c = 4,0 kN/cm²

moment load capacity:

y_F = 45,7 cm
$M_{fi,Rd}$ = 246,9 kNm NW: 133 kNm / 247 kNm = 0,54 < 1

detailed design - composite steel beam (shear force capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
web:	592 °C	0,33	0,19	0,50	0,50	70034	45,7	116,4	116,4

shear force resistance:

Av = 35,108 cm²
$f_{ay,\Theta,web}$ = 11,64 kN/cm²

$V_{fi,Rd}$ = 235,84 kN NW: 38 kN / 236 kN = 0,16 < 1

Pos. FP103 Structural fire design - composite steel beam

V. 21.07.2025

floor beam BE152 **Area:** **parkingplace**

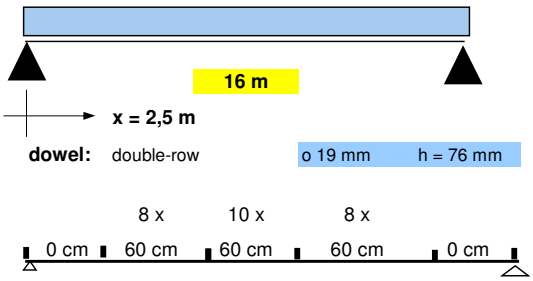
For unprotected steel cross-section-class 1-3. Analysis by EN1994-1-1 (2010) and EN1994-1-2 (2026) **Czech Republic**

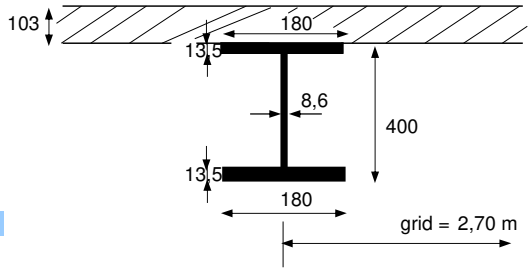
1. static system

Version: 21.07.2025

profile: **IPE 400** | **S 355** | **composite beam**

concrete: **C40/50**





52 dowel:

2. actions

grid = 2,70 m

characteristic actions:

g_k	p_k	
7,886 kN/m	6,750 kN/m	Full load

combination values:

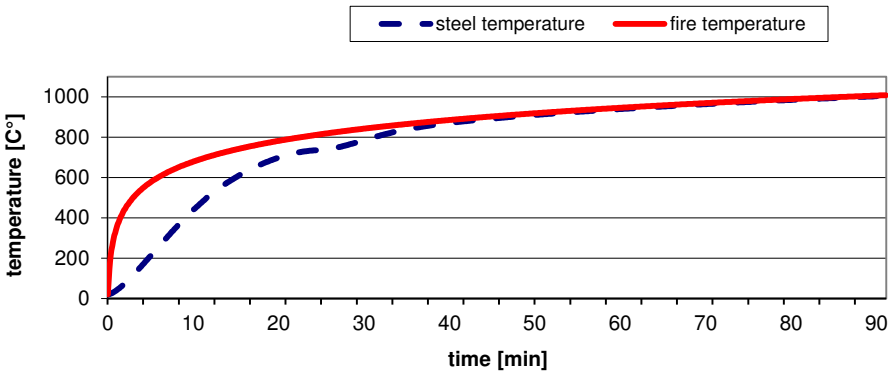
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00
				$\gamma_{fi,Q}$ =	1,00
				ψ_{fi} =	0,70

$M_{Ed,fi,y}$ design at x = **2,5 m**

design actions:

$M_{Ed,y}$	A_d	B_d	$M_{Ed,y,fi\,(x)}$	$A_{d,fi}$	$B_{d,fi}$	$V_{Ed,fi\,(x)}$
350,5 kNm	166,2 kN	166,2 kN	212,8 kNm	100,9 kN	100,9 kN	69,4 kN

thermal actions: ETK



3. section properties / material skills

profile: IPE 400

$f_{y,d}$ = 35,5 kN/cm²

$f_{y,fi}$ = 35,5 kN/cm²

4. structural design

determinant design values: **0,83 < 1**

5. structural design on fire

design at t = 15 min fire temperatur = 739 °C

maximal steel temperature bottom flange	**581 °C**	**after**	**15 min**	
moment load capacity:	213 kNm / 273 kNm	=		0,78 < 1
Interaction:	213 kNm / 273 kNm	=		0,78 < 1
maximal steel temperature web	**581 °C**	**after**	**15 min**	
shear force resistance:	69 kN / 464 kN	=		0,15 < 1

determinant design values: **0,78 < 1**

detailed design - composite steel beam (moment load capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
upper flange:	419 °C	0,68	0,41	0,96	0,96	143042	145	340	340
web:	581 °C	0,37	0,21	0,53	0,53	76822	76	188	188
bottom flange:	581 °C	0,37	0,21	0,53	0,53	76822	76	188	188

calculation of positively moment load capacity

T^+ = 1887,0 kN
y_T = 238 mm

$T<N^*P_{fi,Rd}$ $P_{Rd,1}$ = 113,4 kN $k_{u,\Theta}$ = 1,16 $Pfi_{,Rd,1}$ = 105,5 kN => gov.: 105,5 kN
$P_{Rd,2}$ = 124,3 kN $k_{c,\Theta}$ = 0,97 $Pfi_{,Rd,2}$ = 120,1 kN

$N^*P_{fi,Rd}$ = 1054,6 kN

=> gov T^+ = **1054,6 kN**

calculation of pressure zones height

b_{eff} = 228,5 cm (hc - hu) = 9,15 cm > hcr = 1,00 cm
f_c = 4,0 kN/cm²

moment load capacity:

y_F = 49,7 cm
$M_{fi,Rd}$ = 273,5 kNm NW: 213 kNm / 273 kNm = 0,78 < 1

detailed design - composite steel beam (shear force capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
web:	581 °C	0,37	0,21	0,53	0,53	76822	76,2	188,0	188,0

shear force resistance:

Av = 42,731 cm²
$f_{ay,\Theta,web}$ = 18,80 kN/cm²

$V_{fi,Rd}$ = 463,89 kN NW: 69 kN / 464 kN = 0,15 < 1

Pos. FP104 **Structural fire design - composite steel beam** V. 21.07.2025

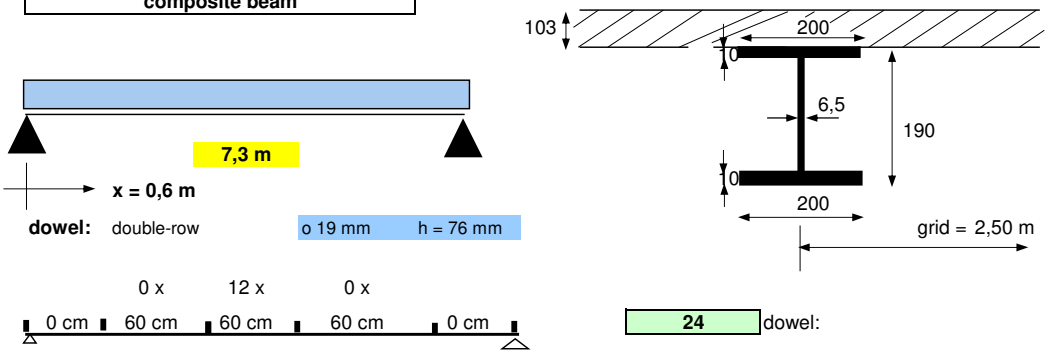
floor beam BE302 **Area:** **drivinglane**

For unprotected steel cross-section-class 1-3. Analysis by EN1994-1-1 (2010) and EN1994-1-2 (2026) **Czech Republic**

1. static system Version: 21.07.2025

profile: **HEA 200** | **S 235** | **composite beam**

concrete: **C40/50**



24 dowel:

2. actions grid = 2,50 m

characteristic actions:

g_k	p_k	
8,486 kN/m	6,250 kN/m	Full load

combination values:

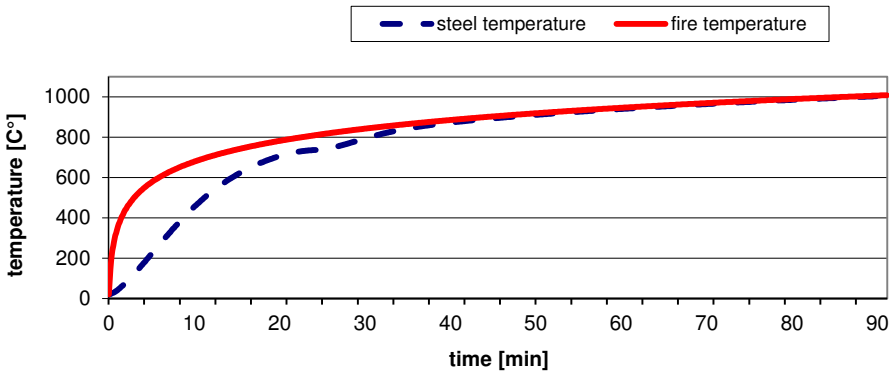
γ_{M0} =	1,0	$\gamma_{F,G}$ =	1,35	$\gamma_{M,fi}$ =	1,00
γ_{M1} =	1,0	$\gamma_{F,Q}$ =	1,50	$\gamma_{fi,G}$ =	1,00
				$\gamma_{fi,Q}$ =	1,00
				ψ_{fi} =	0,70

$M_{Ed,fi,y}$ design at x = **0,6 m**

design actions:

$M_{Ed,y}$	A_d	B_d	$M_{Ed,y,fi\,(x)}$	$A_{d,fi}$	$B_{d,fi}$	$V_{Ed,fi\,(x)}$
41,9 kNm	**76,0 kN**	**76,0 kN**	**25,8 kNm**	**46,9 kN**	**46,9 kN**	**39,2 kN**

thermal actions: ETK



3. section properties / material skills

profile: HEA 200 $f_{y,d}$ = 23,5 kN/cm² $f_{y,fi}$ = 23,5 kN/cm²

4. structural design

determinant design values: **0,58 < 1**

5. structural design on fire

design at t = 15 min fire temperatur = 739 °C

maximal steel temperature bottom flange	**595 °C**	**after**	**15 min**	
moment load capacity:	26 kNm / 36 kNm	=		0,73 < 1
Interaction:	26 kNm / 36 kNm	=		0,73 < 1
maximal steel temperature web	**595 °C**	**after**	**15 min**	
shear force resistance:	39 kN / 119 kN	=		0,33 < 1

determinant design values: **0,73 < 1**

detailed design - composite steel beam (moment load capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
upper flange:	431 °C	0,67	0,40	0,93	0,93	140535	94	219	219
web:	595 °C	0,32	0,19	0,49	0,49	68059	44	114	114
bottom flange:	595 °C	0,32	0,19	0,49	0,49	68059	44	114	114

calculation of positively moment load capacity

T^+ = 792,1 kN
y_T = 119 mm

T<N*$P_{fi,Rd}$ $P_{Rd,1}$ = 113,4 kN $k_{u,\Theta}$ = 1,14 $Pfi_{,Rd,1}$ = 103,3 kN => gov.: 103,3 kN
$P_{Rd,2}$ = 124,3 kN $k_{c,\Theta}$ = 0,96 $Pfi_{,Rd,2}$ = 119,8 kN

N*$P_{fi,Rd}$ = 206,6 kN

=> gov T^+ = **206,6 kN**

calculation of pressure zones height

b_{eff} = 136,875 cm (hc - hu) = 9,92 cm > hcr = 1,00 cm
f_c = 4,0 kN/cm²

moment load capacity:

y_F = 29,1 cm
$M_{fi,Rd}$ = 35,6 kNm NW: 26 kNm / 36 kNm = 0,73 < 1

detailed design - composite steel beam (shear force capacity)

design at t = 15 min

structural design on fire

		reduction factors				material skills on fire			
	Temp.	$k_{E,\Theta}$	$k_{P,\Theta}$	$k_{y,\Theta}$	$k_{u,\Theta}$	$E_{a,\Theta}$	$f_{ap,\Theta}$	$f_{ay,\Theta}$	$f_{au,\Theta}$
web:	595 °C	0,32	0,19	0,49	0,49	68059	44,4	114,0	114,0

shear force resistance:

Av = 18,05 cm²
$f_{ay,\Theta,web}$ = 11,40 kN/cm²

$V_{fi,Rd}$ = 118,79 kN NW: 39 kN / 119 kN = 0,33 < 1

GOLDBECK

GOLDBECK Südwest GmbH - Tragwerksplanung -

Goldbeckstraße 7, 69493 Hirschberg an der Bergstraße

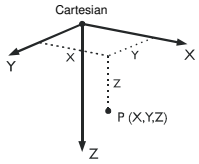
[redacted]

MODEL

Project: PR0460 | Model: 09-FP105_ETK_R15_BE402_Ramp transfer beam | Date: 03.09.2025

MODEL - GENERAL DATA

General	Model name	:	09-FP105_PTK_R30_BE402_Ramp transfer beam
	Type of model	:	3D
	Positive direction of global axis Z	:	Downward
	Classification of load cases and combinations	:	According to Standard: EN 1990 National Annex: CSN - Czech Republic
Options	☐ Use CQC Rule		
	☐ Enable CAD/BIM model		
	Standard Gravity		
	g	:	10.00 m/s²



1.1 NODES

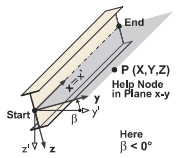
Node No.	Reference Node	Coordinate System	Node Coordinates X [m]	Y [m]	Z [m]	Comment
1	-	Cartesian	0.000	0.000	0.000	Supported
2	-	Cartesian	7.500	0.000	0.000	Supported

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 355 | DIN EN 1993-1-1:2010-12					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic

1.3 CROSS-SECTIONS

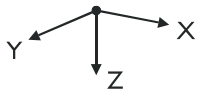
Section No.	Matl. No.	J [cm⁴] / A [cm²]	I_y [cm⁴] / A_y [cm²]	I_z [cm⁴] / A_z [cm²]	Principal Axes α [°]	Rotation α' [°]	Overall Dimensions [mm] Width b	Height h
2	IU 360/190/20/10/190/20/4/4							
	1	105.95 108.00	24720.00 63.51	2289.00 32.58	0.00	0.00	190.0	360.0



1.7 MEMBERS

Mbr. No.	Member	Node Start	Node End	Rotation Type	Rotation β [°]	Cross-Section Start	Cross-Section End	Hinge No. Start	Hinge No. End	Ecc. No.	Div. No.	Length L [m]	
1	Beam	1	2	Angle	0.00	2	2	-	-	-	-	7.500	X

1.8 NODAL SUPPORTS

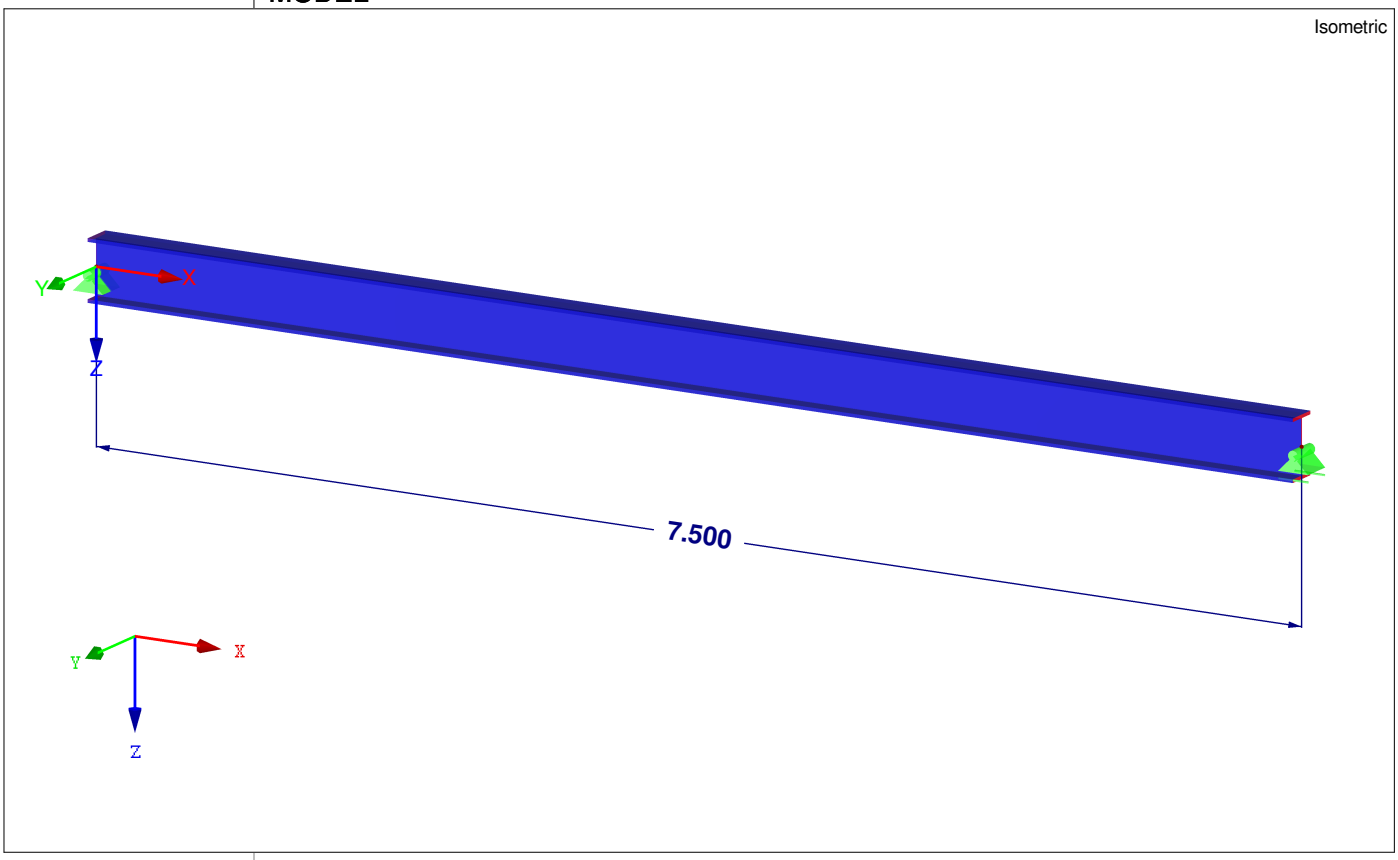


Support No.	Nodes No.	Sequen.	Rotation [°] about X	about Y	about Z	Column in Z	Support Conditions $u_{X'}$	$u_{Y'}$	$u_{Z'}$	$\varphi_{X'}$	$\varphi_{Y'}$	$\varphi_{Z'}$
1	1	XYZ	0.00	0.00	0.00	☐	☒	☒	☒	☒	☐	☒
2	2	XYZ	0.00	0.00	0.00	☐	☐	☒	☒	☒	☐	☒

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MODEL



2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | CSN Action Category	Self-Weight - Factor in Direction Active	X	Y	Z
LC1	Self-weight	Permanent	☒	0.000	0.000	1.000
LC2	Imposed load	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters	
LC1	Self-weight	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	Imposed load	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	
CO3	ACC	ULS (EQU) - Accidental - psi-1,1	1	1.00	LC1	Self-weight
			2	0.70	LC2	Imposed load

2.5.2 LOAD COMBINATIONS - CALCULATION PARAMETERS

Load Combin.	Description	Calculation Parameters	
CO3	ULS (EQU) - Accidental - psi-1,1	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γM)
			: ☒ 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)

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2.6 RESULT COMBINATIONS

Result Combin	Description	Loading
RC3	ULS (EQU) - Accidental - psi-1,1 - thermal design	CO3

LC1
Self-weight

3.2 MEMBER LOADS

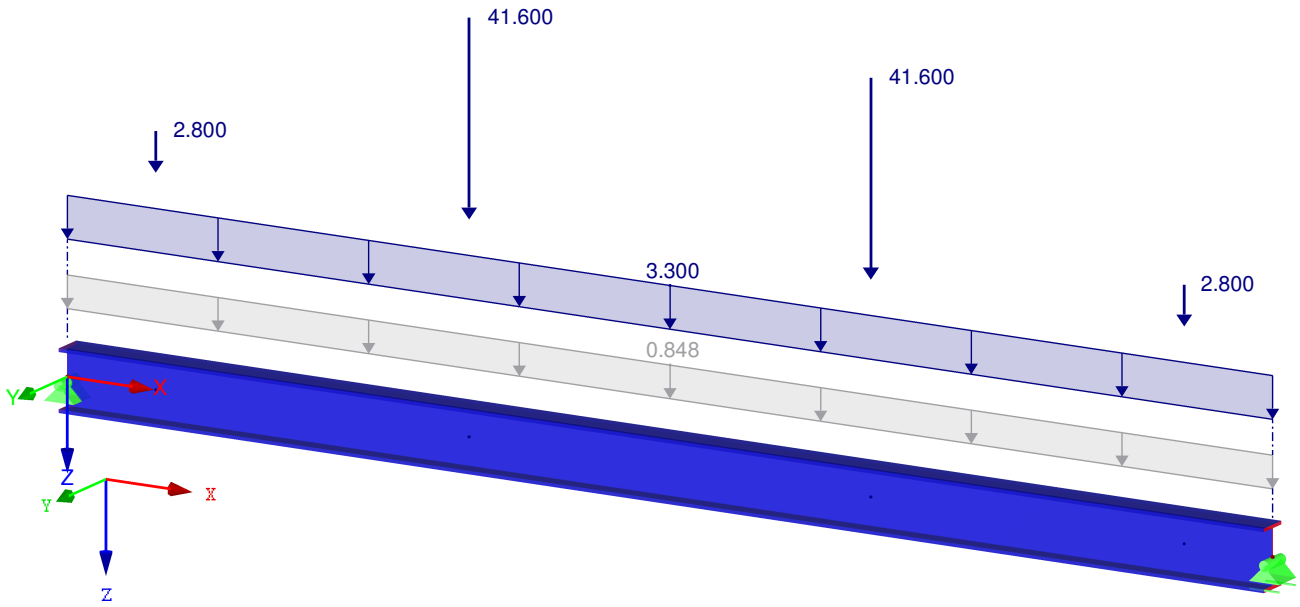
LC1: Self-weight

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	1	Force	Uniform	Z	True Length	p	3.300	kN/m
	concrete slab: 0,103 m · 25 kN/m³ · 1,25 m = 3,22 kN/m								
2	Members	1	Force	Concentr.	Z	True Length	P	41.600	kN
							A	2.500	m
	from Pos. BE034								
3	Members	1	Force	Concentr.	Z	True Length	P	41.600	kN
							A	5.000	m
	from Pos. BE034								
4	Members	1	Force	Concentr.	Z	True Length	P	2.800	kN
							A	6.950	m
	from scraping board: 2,80 kN on each side								
5	Members	1	Force	Concentr.	Z	True Length	P	2.800	kN
							A	0.550	m
	from scraping board: 2,80 kN on each side								

LC3: SELF-WEIGHT

LC1 : Self-weight
Loads [kN/m], [kN]

Isometric



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	1	Force	Uniform	Z	True Length	p	3.200	kN/m
	live load: 2,50 kN/m² · 1,25 m = 3,13 kN/m								
2	Members	1	Force	Uniform	Z	True Length	p	0.200	kN/m
	installation: 0,15 kN/m² · 1,25 m = 0,19 kN/m								
3	Members	1	Force	Concentr.	Z	True Length	P	34.800	kN
							A	2.500	m
	from Pos. BE034								
4	Members	1	Force	Concentr.	Z	True Length	P	34.800	kN
							A	5.000	m
	from Pos. BE034								

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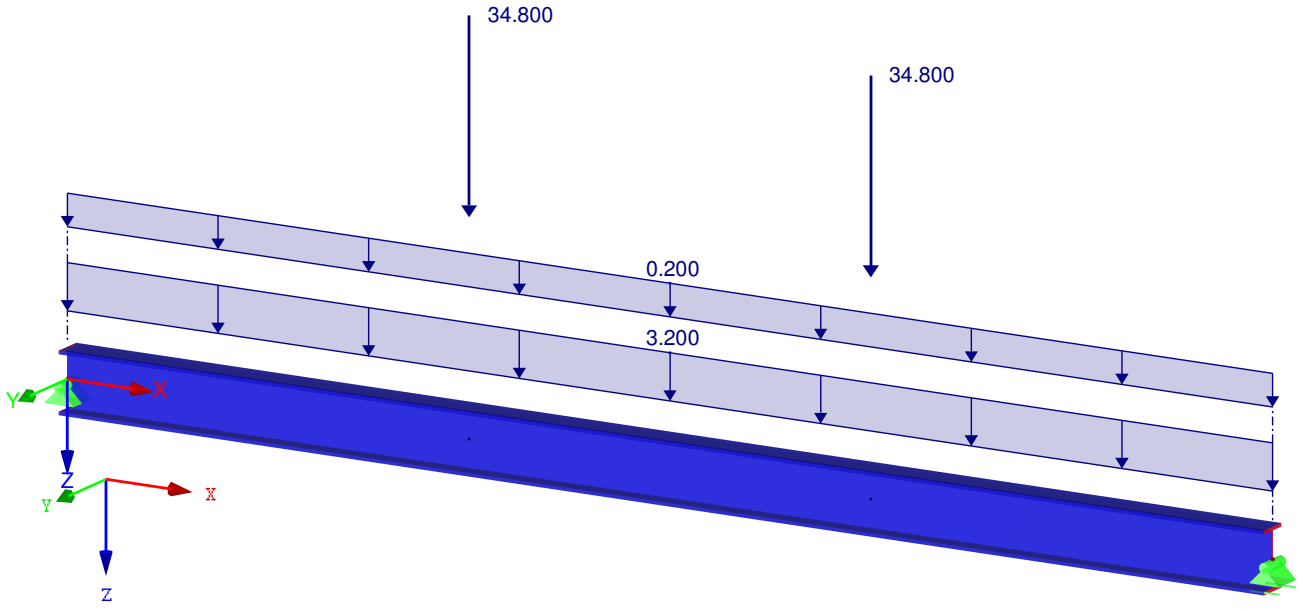
LOADS

Project: PR0460 Model: 09-FP105_ETK_R15_BE402_Ramp transfer beam Date: 03.09.2025

LC2: IMPOSED LOAD

LC2 : Imposed load
Loads [kN/m], [kN]

Isometric



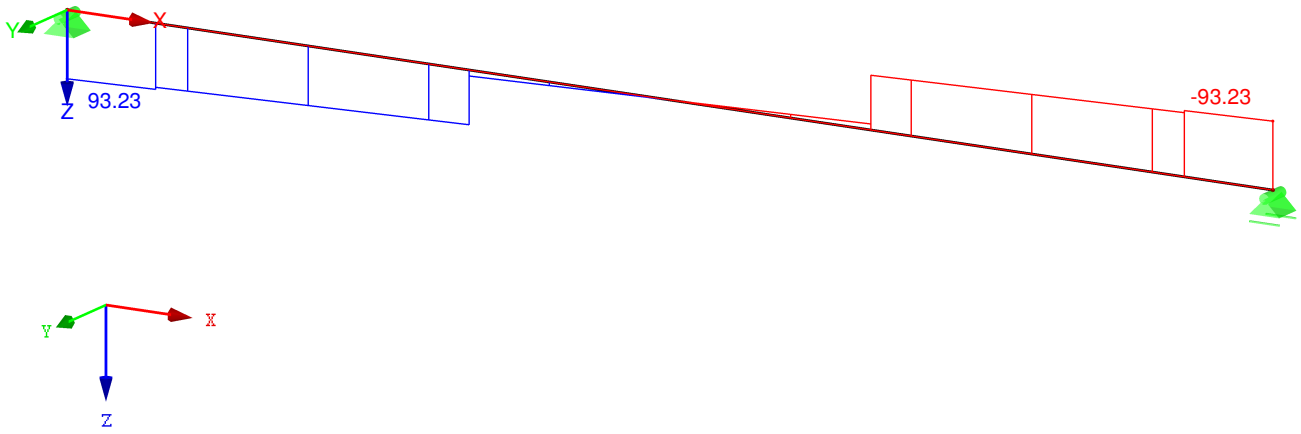
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INTERNAL FORCES V_z

RC3 : ULS (EQU) - Accidental - psi-1,1 - thermal design
Internal Forces V-z
Result Combinations: Max and Min Values

Isometric

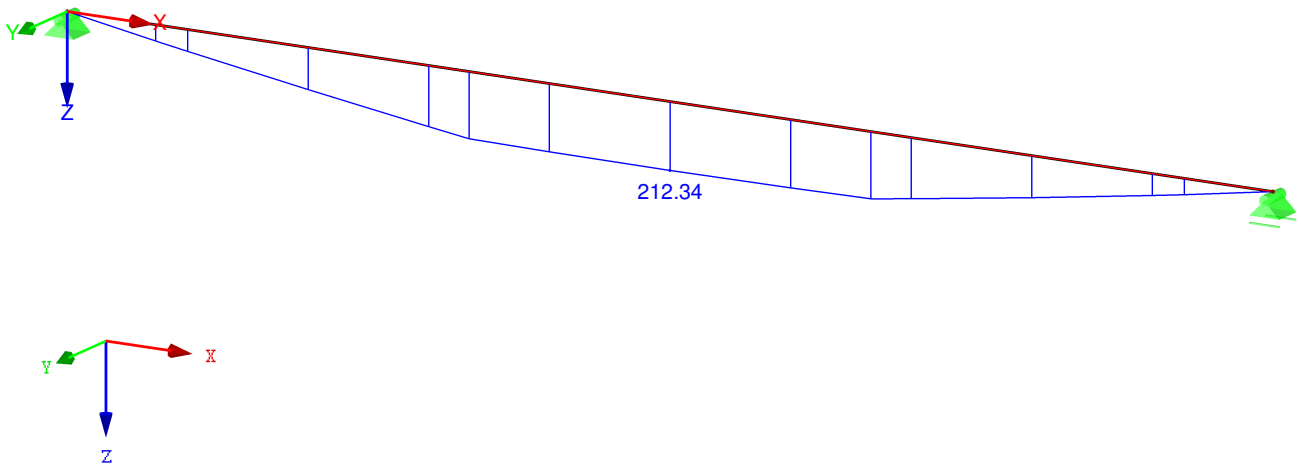


Max V-z: 93.23, Min V-z: -93.23 [kN]

INTERNAL FORCES M_y

RC3 : ULS (EQU) - Accidental - psi-1,1 - thermal design
Internal Forces M-y
Result Combinations: Max and Min Values

Isometric



Max M-y: 212.34, Min M-y: 0.00 [kNm]

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STEEL EC3
CA1
Design ratio according EC3

1.1 GENERAL DATA

Members to design:	All	
Sets of members to design:		
National Annex:	CSN	
Fire Resistance Design		
Result combinations to design:	RC3	ULS (EQU) - Accidental - psi-1,1 - thermal design

1.2 MATERIALS

Matl. No.	Material Description	E- Modulus E [kN/cm²]	Shear Modulus G [kN/cm²]	Poisson's Ratio ν [-]	Yield Stress f_{yk} [kN/cm²]	Max. Thickness t [mm]
1	Steel S 355 | DIN EN 1993-1-1:2010-12	21000.00	8076.92	0.300	35.50	40.0
					33.50	80.0
					31.50	100.0
					29.50	150.0
					28.50	200.0
					27.50	250.0

1.3 CROSS-SECTIONS

Sect. No.	Matl. No.	Cross-Section Description	Cross-Section Type	Max Design Ratio	Comment
2	1	IU 360/190/20/10/190/20/4/4	I-section welded IU	0.86	

1.4 INTERMEDIATE LATERAL RESTRAINTS

Member No.	Restrain Type	Length L [m]	Intermediate Lateral Restraints [-]									
			Number	x_1	x_2	x_3	x_4	x_5	x_6	x_7	x_8	x_9
1	Lateral and torsional	7.500	2	0.333	0.667							

1.5 EFFECTIVE LENGTHS - MEMBERS

Member No.	Buckling Possible	Buckling About Axis y			Buckling About Axis z			Lateral-Torsional Buckling				
		Possible	$k_{cr,y}$	$L_{cr,y}$ [m]	Possible	$k_{cr,z}$	$L_{cr,z}$ [m]	Possible	k_z	k_w	L_w [m]	L_T [m]
1	☒	☒	1.00	7.500	☒	1.00	2.500	☒	1.0	1.0	2.500	2.500

1.9 SERVICEABILITY DATA

No.	Reference to	Members/Sets No.	Reference Length Manually	Reference Length l [m]	Direct.	Precamber e_0 [mm]	Beam Type
1	Member	1	☐	7.500	z	10.0	Beam

1.10 FIRE RESISTANCE - MEMBERS

No.	Members N	Req. Time $t_{fi,req}$ [min]	Fire Exposure	Fire Protection	Protection Type	Unit Mass ρ_p [kg/m³]	Thermal Cond. λ_p [W/m*K]	Specific Heat c_p [J/(kg*K)]	Thickness d_p [mm]
1	1	15	All Sides	☐					

1.12 PARAMETERS - MEMBERS

Member No.	Description	Parameter
1	Cross-Section	2 - IU 360/190/20/10/190/20/4/4
	Shear panel	☐
	Rotational restraint	☐
	Cross-sectional area for tension design	☐

2.4 DESIGN BY MEMBER

Member No.	Location x [m]	LC/CO/RC	Design		Equation No.	Description
1	**Cross-section No. 2 - IU 360/190/20/10/190/20/4/4**					
	3.750	RC3	0.60	≤ 1	FC611)	Fire design - Cross-section check - Bending about y-axis acc. to EN 1993-1-2, 4.2.3.3 - Class 1 or 2
	0.000	RC3	0.18	≤ 1	FC621)	Fire design - Cross-section check - Shear force in z-axis acc. to EN 1993-1-2, 4.2.3.3
	3.750	RC3	0.60	≤ 1	FC641)	Fire design - Cross-section check - Bending and shear force acc. to EN 1993-1-2, 4.2.3.3
	3.750	RC3	0.86	≤ 1	FS831)	Fire design - Stability analysis - Lateral torsional buckling acc. to EN 1993-1-2, 4.2.3.3

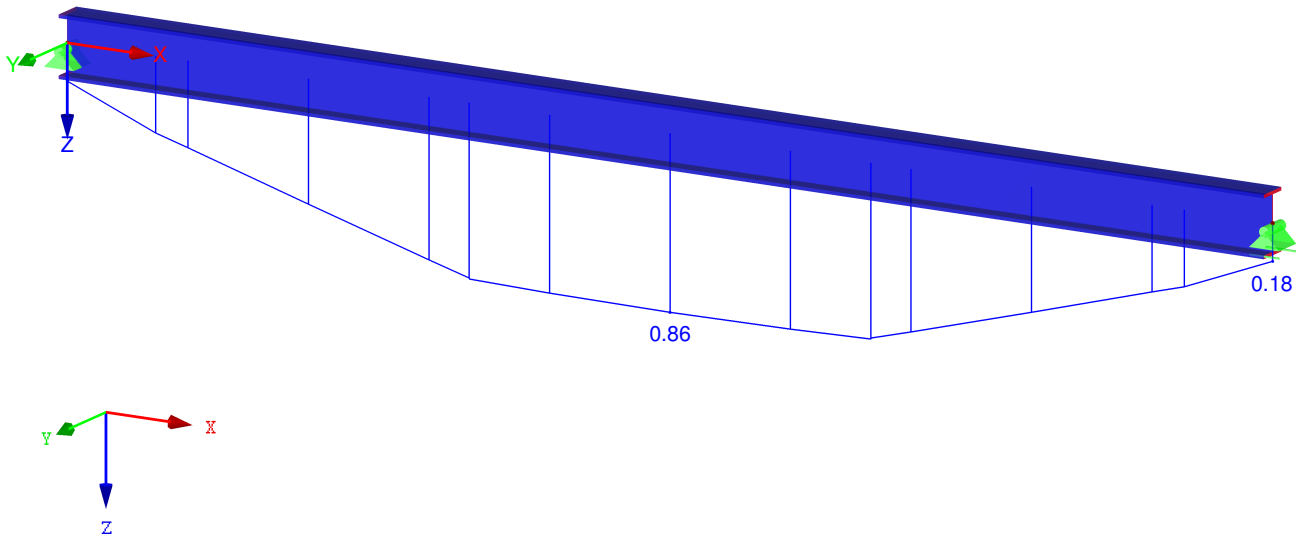
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DESIGN: FIRE RESISTANCE - CROSS-SECTION DESIGN + STABILITY DESIGN

STEEL EC3 CA1
Fire Resistance: Cross-Section Design, Stability Design

Isometric



Max Design Ratio: 0.86

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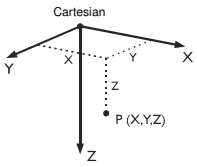
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MODEL

Project: PR0460 Model: 09-FP106_ETK_R15_BE412_Ramp transfer beam .. Date: 03.09.2025

MODEL - GENERAL DATA

General	Model name	:	09-FP106_ETK_R15_BE412_Ramp transfer beam (ramp end)
	Type of model	:	3D
	Positive direction of global axis Z	:	Downward
	Classification of load cases and combinations	:	According to Standard: EN 1990 National Annex: CSN - Czech Republic
Options	☐ Use CQC Rule		
	☐ Enable CAD/BIM model		
	Standard Gravity		
	g	:	10.00 m/s²



1.1 NODES

Node No.	Reference Node	Coordinate System	Node Coordinates X [m]	Y [m]	Z [m]	Comment
1	-	Cartesian	0.000	0.000	0.000	Supported
2	-	Cartesian	7.500	0.000	0.000	Supported

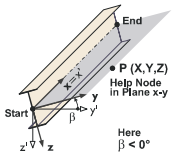
1.2 MATERIALS

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 355 | DIN EN 1993-1-1:2010-12					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic



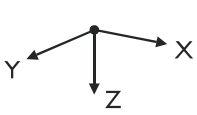
1.3 CROSS-SECTIONS

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
2	IU 360/190/20/10/190/20/4/4							
	1	105.95 108.00	24720.00 63.51	2289.00 32.58	0.00	0.00	190.0	360.0



1.7 MEMBERS

Mbr. No.	Member	Node Start	Node End	Rotation Type	Rotation β [°]	Cross-Section Start	Cross-Section End	Hinge No. Start	Hinge No. End	Ecc. No.	Div. No.	Length L [m]	
1	Beam	1	2	Angle	0.00	2	2	-	-	-	-	7.500	X



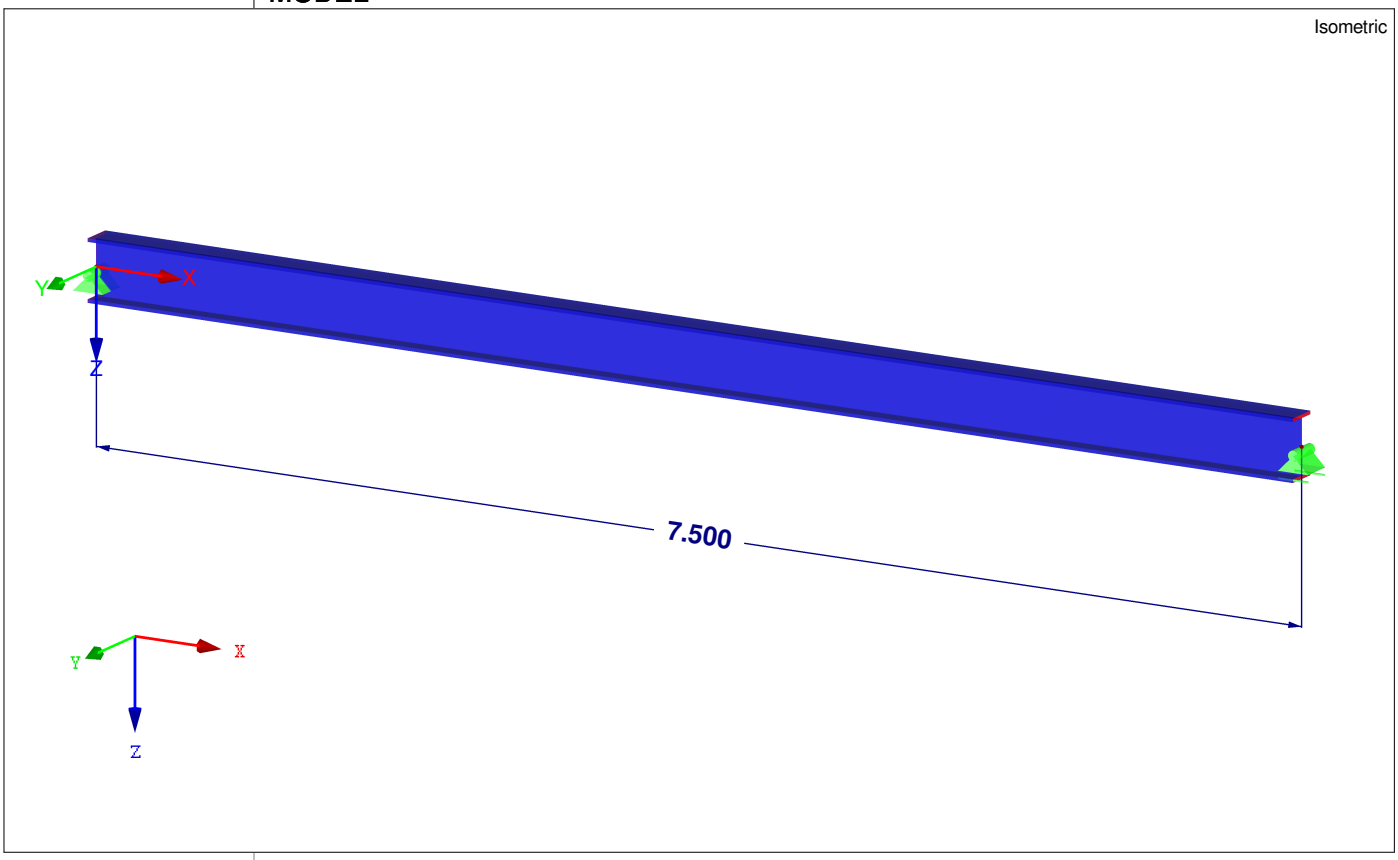
1.8 NODAL SUPPORTS

Support No.	Nodes No.	Sequen.	Rotation [°] about X	about Y	about Z	Column in Z	Support Conditions $u_{X'}$	$u_{Y'}$	$u_{Z'}$	$\varphi_{X'}$	$\varphi_{Y'}$	$\varphi_{Z'}$
1	1	XYZ	0.00	0.00	0.00	☐	☒	☒	☒	☒	☐	☒
2	2	XYZ	0.00	0.00	0.00	☐	☐	☒	☒	☒	☐	☒

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MODEL



2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | CSN Action Category	Self-Weight - Factor in Direction Active	X	Y	Z
LC1	Self-weight	Permanent	☒	0.000	0.000	1.000
LC2	Imposed load	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters	
LC1	Self-weight	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	Imposed load	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	
CO3	ACC	ULS (EQU) - Accidental - psi-1,1 - thermal design	1	1.00	LC1	Self-weight
			2	0.70	LC2	Imposed load

2.5.2 LOAD COMBINATIONS - CALCULATION PARAMETERS

Load Combin.	Description	Calculation Parameters	
CO3	ULS (EQU) - Accidental - psi-1, 1 - thermal design	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γM)
			: ☒ 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)

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2.6 RESULT COMBINATIONS

Result Combin	Description	Loading
RC3	ULS (EQU) - Accidental - psi-1,1 - thermal design	CO3

LC1
Self-weight

3.2 MEMBER LOADS

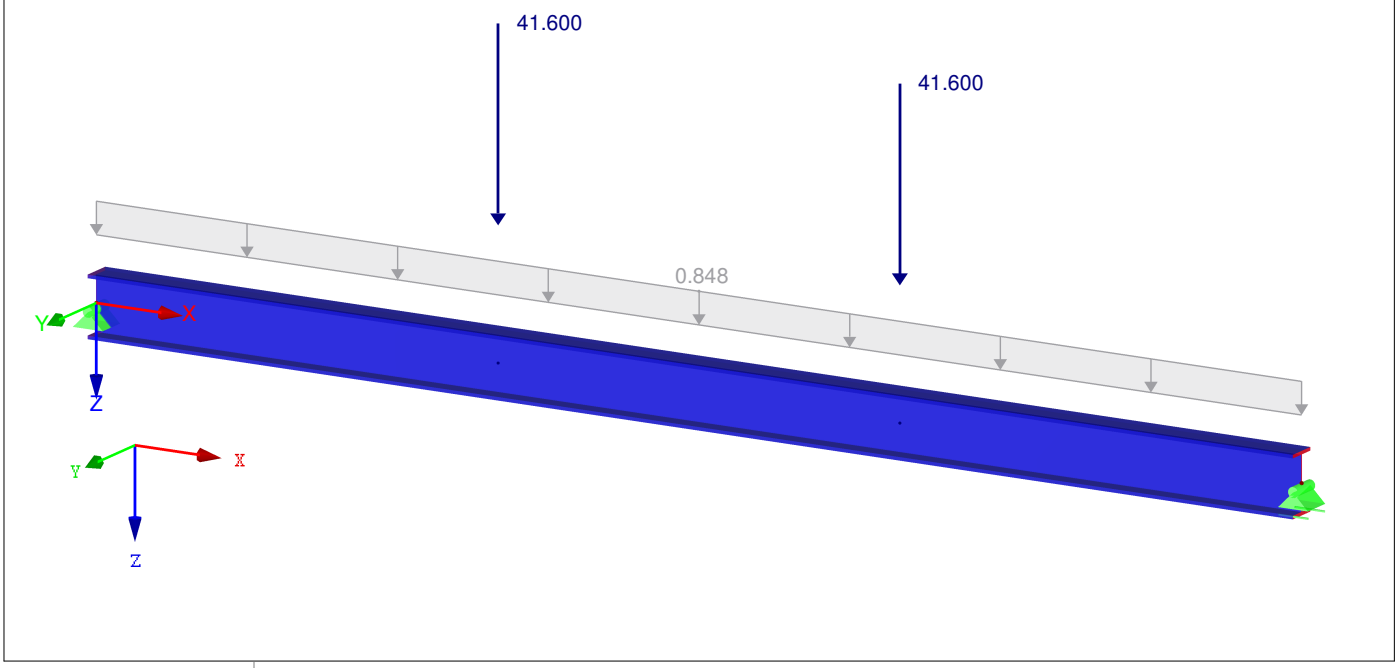
LC1: Self-weight

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
2	Members	1	Force	Concentr.	Z	True Length	P	41.600	kN
							A	2.500	m
	from Pos. BE034								
3	Members	1	Force	Concentr.	Z	True Length	P	41.600	kN
							A	5.000	m
	from Pos. BE034								

LC1: SELF-WEIGHT

LC1 : Self-weight
Loads [kN/m], [kN]

Isometric



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
3	Members	1	Force	Concentr.	Z	True Length	P	34.800	kN
							A	2.500	m
	from Pos. BE034								
4	Members	1	Force	Concentr.	Z	True Length	P	34.800	kN
							A	5.000	m
	from Pos. BE034								

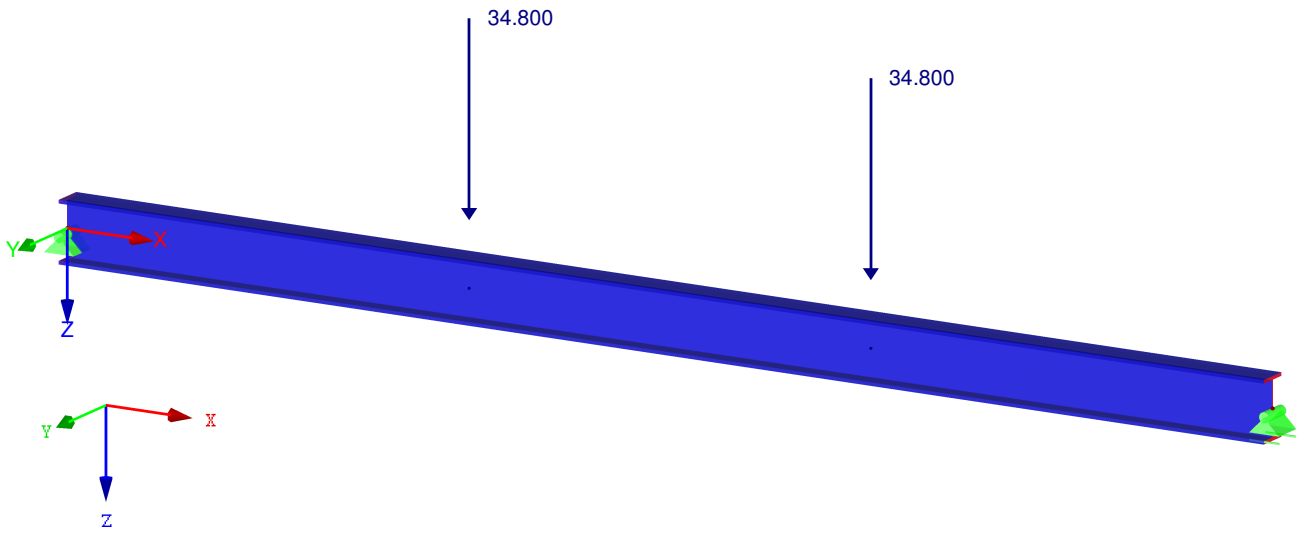
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[redacted]

LC2: IMPOSED LOAD

LC2 : Imposed load
Loads [kN]

Isometric



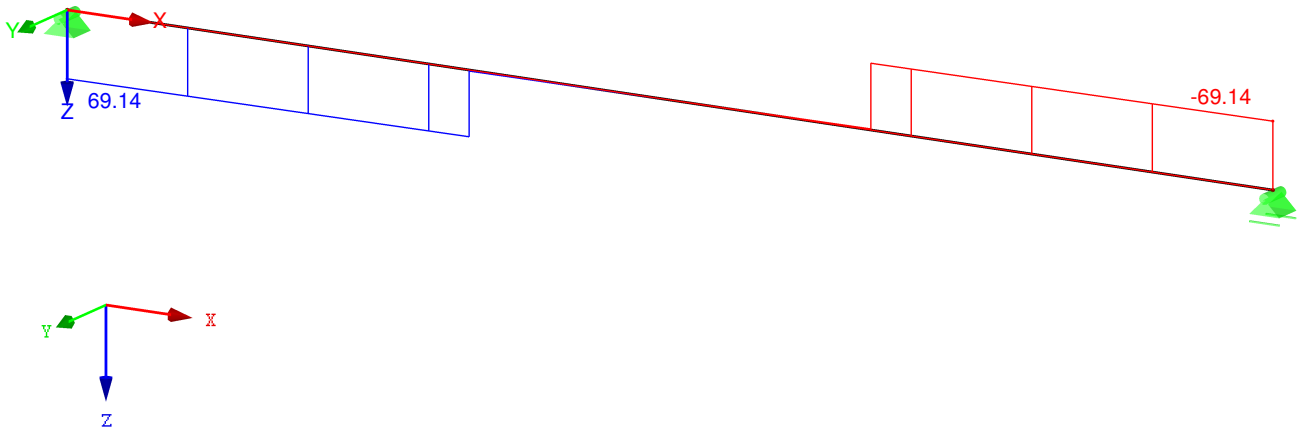
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[redacted]

INTERNAL FORCES V_z

RC3 : ULS (EQU) - Accidental - psi-1,1 - thermal design
Internal Forces V-z
Result Combinations: Max and Min Values

Isometric

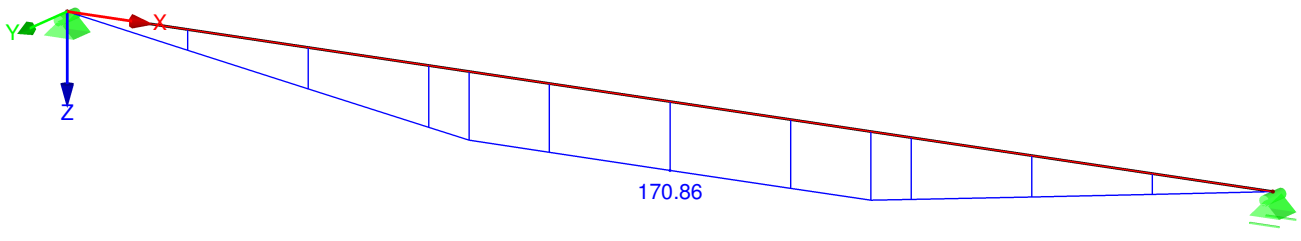


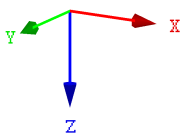
Max V-z: 69.14, Min V-z: -69.14 [kN]

INTERNAL FORCES M_y

RC3 : ULS (EQU) - Accidental - psi-1,1 - thermal design
Internal Forces M-y
Result Combinations: Max and Min Values

Isometric





Max M-y: 170.86, Min M-y: 0.00 [kNm]

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STEEL EC3
CA1
Design ratio according EC3

1.1 GENERAL DATA

Members to design:	All	
Sets of members to design:		
National Annex:	CSN	
Fire Resistance Design		
Result combinations to design:	RC3	ULS (EQU) - Accidental - psi-1,1 - thermal design

1.2 MATERIALS

Matl. No.	Material Description	E- Modulus E [kN/cm²]	Shear Modulus G [kN/cm²]	Poisson's Ratio ν [-]	Yield Stress f_{yk} [kN/cm²]	Max. Thickness t [mm]
1	Steel S 355 | DIN EN 1993-1-1:2010-12	21000.00	8076.92	0.300	35.50	40.0
					33.50	80.0
					31.50	100.0
					29.50	150.0
					28.50	200.0
					27.50	250.0

1.3 CROSS-SECTIONS



Sect. No.	Matl. No.	Cross-Section Description	Cross-Section Type	Max Design Ratio	Comment
2	1	IU 360/190/20/10/190/20/4/4	I-section welded IU	0.69	

1.4 INTERMEDIATE LATERAL RESTRAINTS

Member No.	Restrain Type	Length L [m]	Intermediate Lateral Restraints [-]									
			Number	x_1	x_2	x_3	x_4	x_5	x_6	x_7	x_8	x_9
1	Lateral and torsional	7.500	2	0.333	0.667							

1.5 EFFECTIVE LENGTHS - MEMBERS

Member No.	Buckling Possible	Buckling About Axis y			Buckling About Axis z			Lateral-Torsional Buckling				
		Possible	$k_{cr,y}$	$L_{cr,y}$ [m]	Possible	$k_{cr,z}$	$L_{cr,z}$ [m]	Possible	k_z	k_w	L_w [m]	L_T [m]
1	☒	☒	1.00	7.500	☒	1.00	2.500	☒	1.0	1.0	2.500	2.500

1.9 SERVICEABILITY DATA

No.	Reference to	Members/Sets No.	Reference Length Manually	Reference Length l [m]	Direct.	Precamber e_0 [mm]	Beam Type
1	Member	1	☐	7.500	z	10.0	Beam

1.10 FIRE RESISTANCE - MEMBERS

No.	Members N	Req. Time $t_{fi,req}$ [min]	Fire Exposure	Fire Protection	Protection Type	Unit Mass ρ_p [kg/m³]	Thermal Cond. λ_p [W/m*K]	Specific Heat c_p [J/(kg*K)]	Thickness d_p [mm]
1	1	15	All Sides	☐					

1.12 PARAMETERS - MEMBERS

Member No.	Description	Parameter
1	Cross-Section	2 - IU 360/190/20/10/190/20/4/4
	Shear panel	☐
	Rotational restraint	☐
	Cross-sectional area for tension design	☐

2.4 DESIGN BY MEMBER

Member No.	Location x [m]	LC/CO/RC	Design		Equation No.	Description
1	**Cross-section No. 2 - IU 360/190/20/10/190/20/4/4**					
	3.750	RC3	0.48	≤ 1	FC611)	Fire design - Cross-section check - Bending about y-axis acc. to EN 1993-1-2, 4.2.3.3 - Class 1 or 2
	0.000	RC3	0.14	≤ 1	FC621)	Fire design - Cross-section check - Shear force in z-axis acc. to EN 1993-1-2, 4.2.3.3
	3.750	RC3	0.48	≤ 1	FC641)	Fire design - Cross-section check - Bending and shear force acc. to EN 1993-1-2, 4.2.3.3
	3.750	RC3	0.69	≤ 1	FS831)	Fire design - Stability analysis - Lateral torsional buckling acc. to EN 1993-1-2, 4.2.3.3

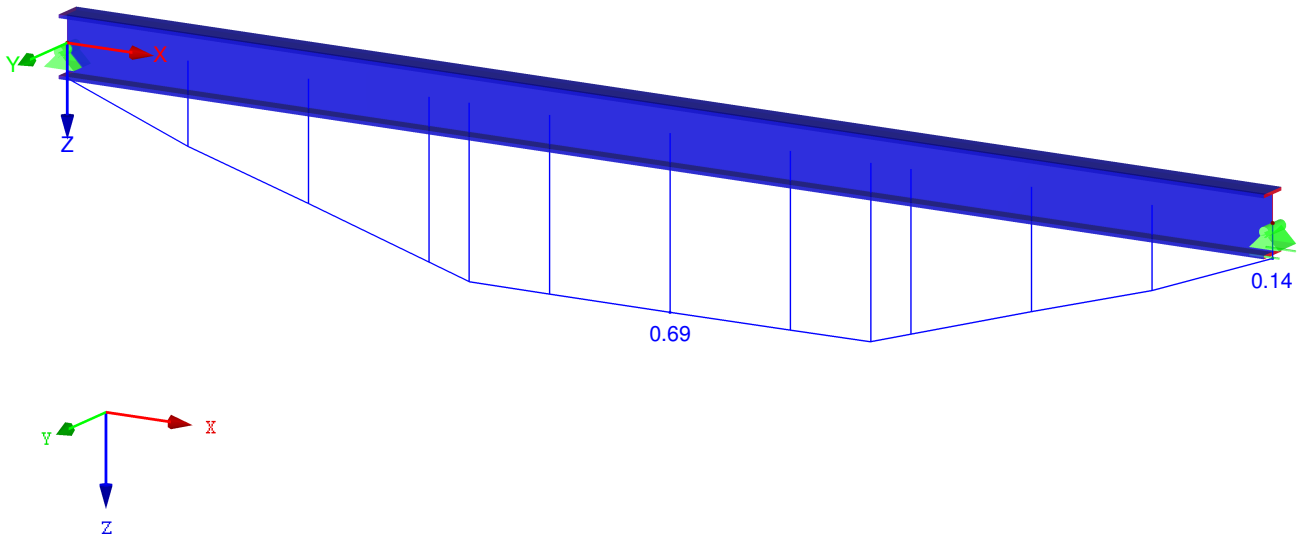
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DESIGN: FIRE RESISTANCE - CROSS-SECTION DESIGN + STABILITY DESIGN

STEEL EC3 CA1
Fire Resistance: Cross-Section Design, Stability Design

Isometric



Max Design Ratio: 0.69

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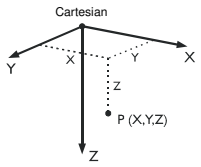
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MODEL - GENERAL DATA

	General	Model name	:	09-FP107_ETK_R15_BE450_Transfer beam Longitudinal direction
		Type of model	:	3D
		Positive direction of global axis Z	:	Downward
		Classification of load cases and combinations	:	According to Standard: EN 1990 National Annex: CSN - Czech Republic
	Options	☐ Use CQC Rule		
		☐ Enable CAD/BIM model		
		Standard Gravity		
		g	:	10.00 m/s²



1.1 NODES

Node No.	Reference Node	Coordinate System	Node Coordinates X [m]	Y [m]	Z [m]	Comment
1	-	Cartesian	0.000	0.000	0.000	Supported
2	-	Cartesian	7.500	0.000	0.000	Supported

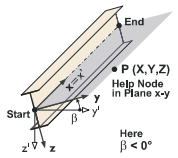
1.2 MATERIALS

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 355 | DIN EN 1993-1-1:2010-12					
	21000.00	8076.92	78.50	1.20E-05	1.00	Isotropic Linear Elastic



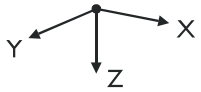
1.3 CROSS-SECTIONS

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
2	IU 400/220/20/15/220/20/4/4							
	1	153.36 142.00	37629.33 73.81	3559.46 53.62	0.00	0.00	220.0	400.0



1.7 MEMBERS

Mbr. No.	Member	Node Start	Node End	Rotation Type	Rotation β [°]	Cross-Section Start	Cross-Section End	Hinge No. Start	Hinge No. End	Ecc. No.	Div. No.	Length L [m]	
1	Beam	1	2	Angle	0.00	2	2	-	-	-	-	7.500	X



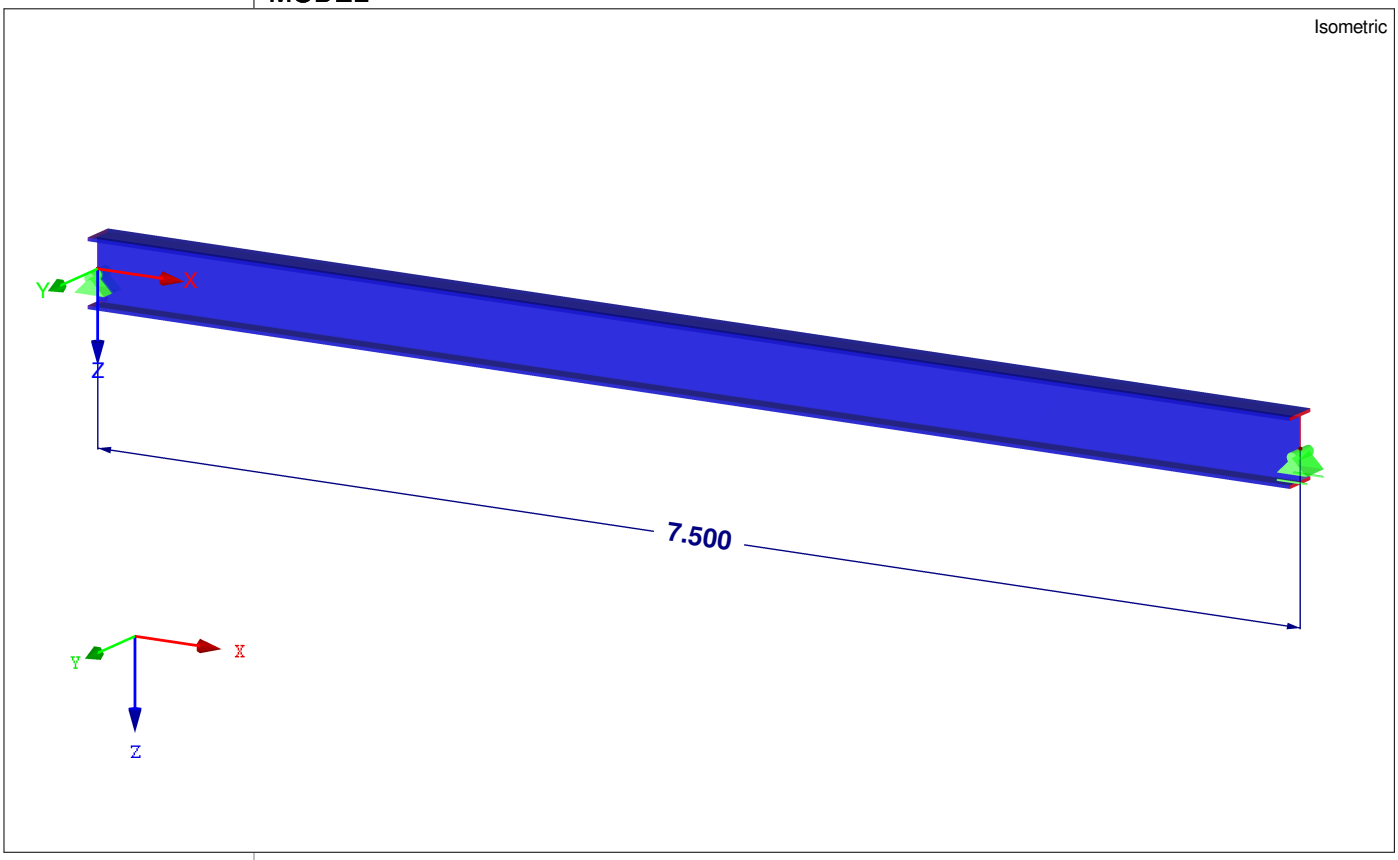
1.8 NODAL SUPPORTS

Support No.	Nodes No.	Sequen.	Rotation [°] about X	about Y	about Z	Column in Z	Support Conditions $u_{X'}$	$u_{Y'}$	$u_{Z'}$	$\varphi_{X'}$	$\varphi_{Y'}$	$\varphi_{Z'}$
1	1	XYZ	0.00	0.00	0.00	☐	☒	☒	☒	☒	☐	☒
2	2	XYZ	0.00	0.00	0.00	☐	☐	☒	☒	☒	☐	☒

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MODEL



2.1 LOAD CASES

Load Case	Load Case Description	EN 1990 | CSN Action Category	Self-Weight - Factor in Direction Active	X	Y	Z
LC1	Self-weight	Permanent	☒	0.000	0.000	1.000
LC2	Imposed load	Imposed - Category F: traffic area - vehicle weight ≤ 30 kN	☐			

2.1.1 LOAD CASES - CALCULATION PARAMETERS

Load Case	Load Case Description	Calculation Parameters	
LC1	Self-weight	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	Imposed load	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	
CO3	ACC	ULS (EQU) - Accidental - psi-1,1	1	1.00	LC1	Self-weight
			2	0.70	LC2	Imposed load

2.5.2 LOAD COMBINATIONS - CALCULATION PARAMETERS

Load Combin.	Description	Calculation Parameters	
CO3	ULS (EQU) - Accidental - psi-1,1	Method of analysis	: ⊙ Second order analysis (P-Delta)
		Options	: ☒ Consider favorable effects due to tension
			: ☒ Refer internal forces to deformed system for:
			☒ Normal forces N
			☒ Shear forces V_y and V_z
			☒ Moments M_y, M_z and M_T
		Activate stiffness factors of:	: ☒ Materials (partial factor γM)
			: ☒ 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)

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2.6 RESULT COMBINATIONS

Result Combin	Description	Loading
RC3	ULS (EQU) - Accidental - psi-1,1 - thermal design	CO3

LC1
Self-weight

3.2 MEMBER LOADS

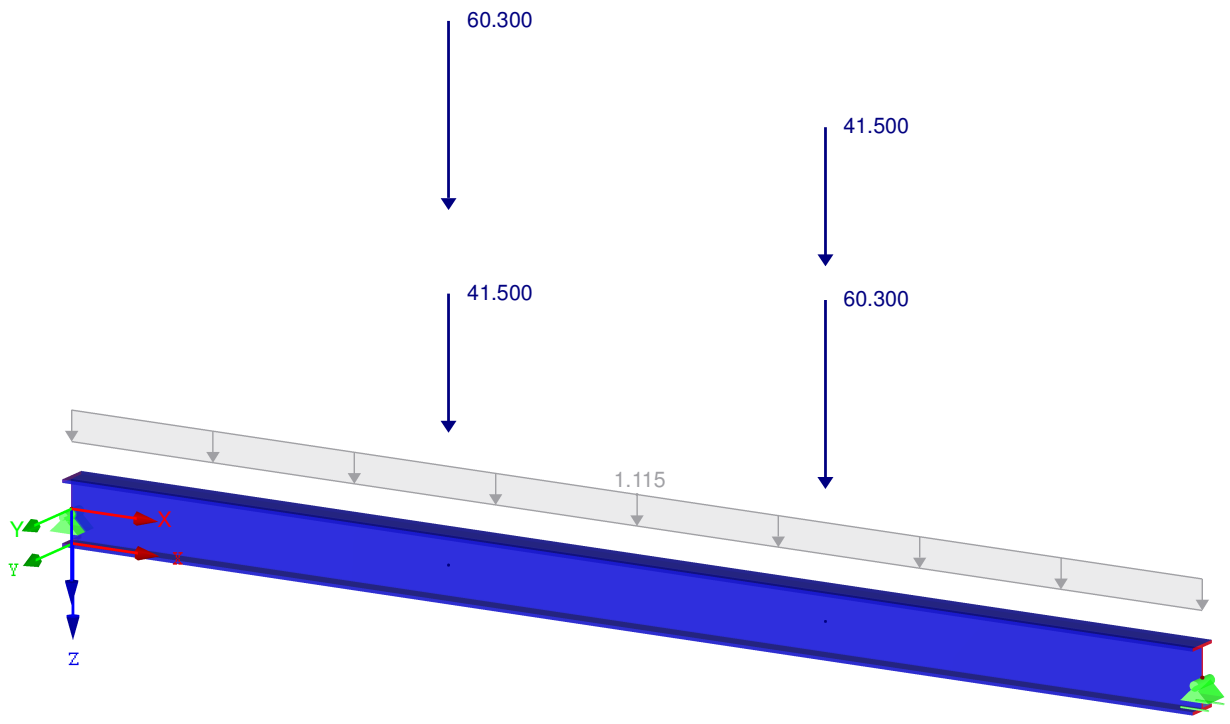
LC1: Self-weight

No.	Reference to	On Members No.	Load Type	Load Distribution	Load Direction	Reference Length	Load Parameters Symbol	Value	Unit
1	Members	1	Force	Concentr.	Z	True Length	P	60.300	kN
							A	5.000	m
	from Pos. BE052								
2	Members	1	Force	Concentr.	Z	True Length	P	41.500	kN
							A	2.500	m
	from Pos. BE034								
3	Members	1	Force	Concentr.	Z	True Length	P	41.500	kN
							A	5.000	m
	from Pos. BE034								
4	Members	1	Force	Concentr.	Z	True Length	P	60.300	kN
							A	2.500	m
	from Pos. BE052								

LC1: SELF-WEIGHT

LC1 : Self-weight
Loads [kN/m], [kN]

Isometric



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	1	Force	Concentr.	Z	True Length	P	50.400	kN
							A	5.000	m
	from Pos. BE052								
3	Members	1	Force	Concentr.	Z	True Length	P	34.800	kN
							A	2.500	m
	from Pos. BE034								
4	Members	1	Force	Concentr.	Z	True Length	P	34.800	kN
							A	5.000	m
	from Pos. BE034								
5	Members	1	Force	Concentr.	Z	True Length	P	50.400	kN
							A	2.500	m
	from Pos. BE052								

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Sheet: 1

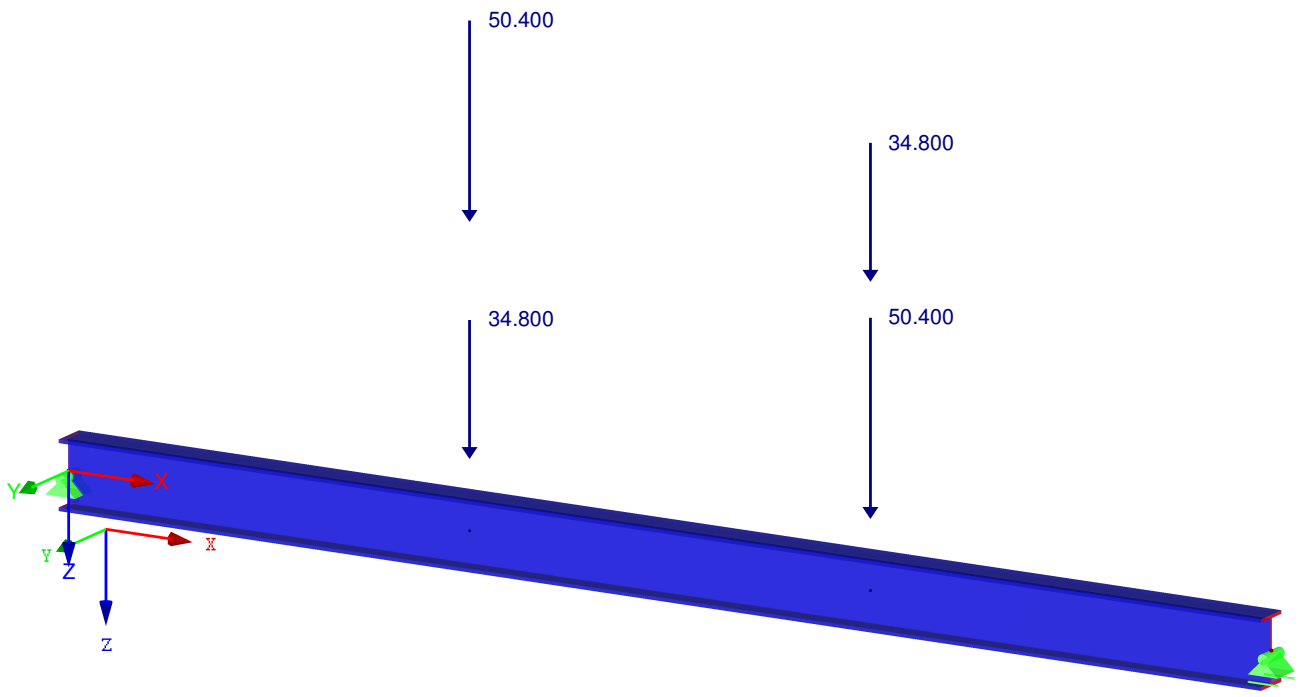
LOADS

Project: PR0460 Model: 09-FP107_ETK_R15_BE450_Transfer beam .. Date: 03.09.2025

LC2: IMPOSED LOAD

LC2 : Imposed load
Loads [kN]

Isometric



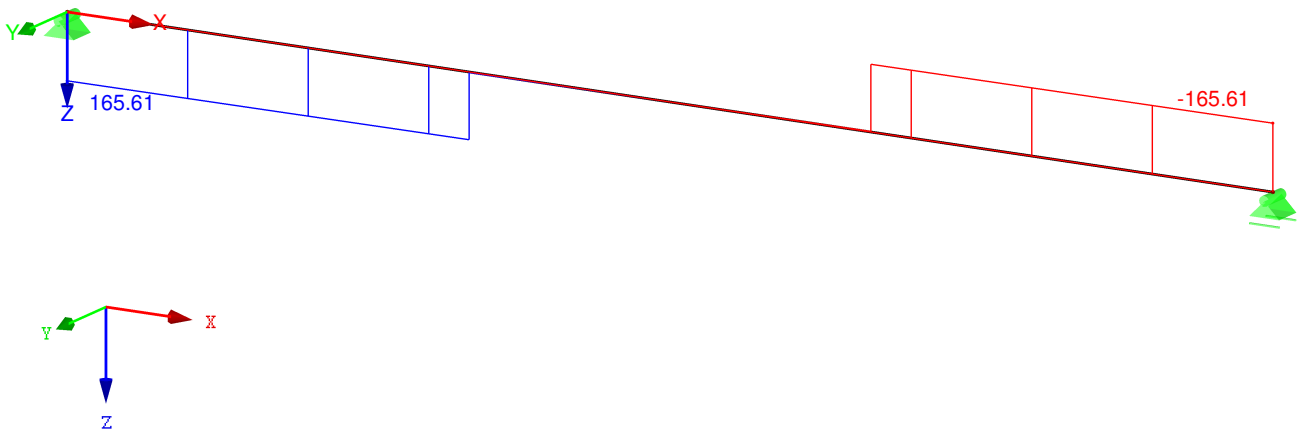
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INTERNAL FORCES V_z

RC3 : ULS (EQU) - Accidental - psi-1,1 - thermal design
Internal Forces V-z
Result Combinations: Max and Min Values

Isometric

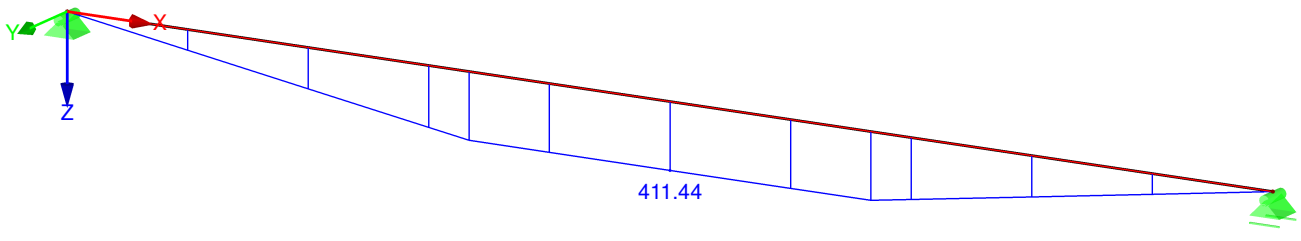


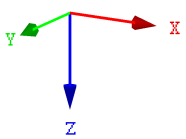
Max V-z: 165.61, Min V-z: -165.61 [kN]

INTERNAL FORCES M_y

RC3 : ULS (EQU) - Accidental - psi-1,1 - thermal design
Internal Forces M-y
Result Combinations: Max and Min Values

Isometric





Max M-y: 411.44, Min M-y: 0.00 [kNm]

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STEEL EC3
CA1
Design ratio according EC3

1.1 GENERAL DATA

	Members to design:	All	
	Sets of members to design:		
	National Annex:	CSN	
	Fire Resistance Design		
	Result combinations to design:	RC3	ULS (EQU) - Accidental - psi-1,1 - thermal design

1.2 MATERIALS

Matl. No.	Material Description	E- Modulus E [kN/cm²]	Shear Modulus G [kN/cm²]	Poisson's Ratio ν [-]	Yield Stress f_{yk} [kN/cm²]	Max. Thickness t [mm]
1	Steel S 355 | DIN EN 1993-1-1:2010-12	21000.00	8076.92	0.300	35.50	40.0
					33.50	80.0
					31.50	100.0
					29.50	150.0
					28.50	200.0
					27.50	250.0

IU 400/220/20/15/...

1.3 CROSS-SECTIONS

Sect. No.	Matl. No.	Cross-Section Description	Cross-Section Type	Max Design Ratio	Comment
2	1	IU 400/220/20/15/220/20/4/4	I-section welded IU	0.77	

1.4 INTERMEDIATE LATERAL RESTRAINTS

Member No.	Restrain Type	Length L [m]	Intermediate Lateral Restraints [-]									
			Number	x_1	x_2	x_3	x_4	x_5	x_6	x_7	x_8	x_9
1	Lateral and torsional	7.500	2	0.333	0.667							

1.5 EFFECTIVE LENGTHS - MEMBERS

Member No.	Buckling Possible	Buckling About Axis y			Buckling About Axis z			Lateral-Torsional Buckling				
		Possible	$k_{cr,y}$	$L_{cr,y}$ [m]	Possible	$k_{cr,z}$	$L_{cr,z}$ [m]	Possible	k_z	k_w	L_w [m]	L_T [m]
1	☒	☒	1.00	7.500	☒	0.33	0.833	☒	1.0	1.0	0.833	0.833

1.9 SERVICEABILITY DATA

No.	Reference to	Members/Sets No.	Reference Length Manually	Reference Length l [m]	Direct.	Precamber e_0 [mm]	Beam Type
1	Member	1	☐	7.500	z	20.0	Beam

1.10 FIRE RESISTANCE - MEMBERS

No.	Members N	Req. Time $t_{fi,req}$ [min]	Fire Exposure	Fire Protection	Protection Type	Unit Mass ρ_p [kg/m³]	Thermal Cond. λ_p [W/m*K]	Specific Heat c_p [J/(kg*K)]	Thickness d_p [mm]
1	1	15	All Sides	☐					

1.12 PARAMETERS - MEMBERS

Member No.	Description	Parameter
1	Cross-Section	2 - IU 400/220/20/15/220/20/4/4
	Shear panel	☐
	Rotational restraint	☐
	Cross-sectional area for tension design	☐

2.4 DESIGN BY MEMBER

Member No.	Location x [m]	LC/CO/RC	Design		Equation No.	Description
1	Cross-section No. 2 - IU 400/220/20/15/220/20/4/4					
	3.750	RC3	0.70	≤ 1	FC611)	Fire design - Cross-section check - Bending about y-axis acc. to EN 1993-1-2, 4.2.3.3 - Class 1 or 2
	0.000	RC3	0.16	≤ 1	FC621)	Fire design - Cross-section check - Shear force in z-axis acc. to EN 1993-1-2, 4.2.3.3
	3.750	RC3	0.70	≤ 1	FC641)	Fire design - Cross-section check - Bending and shear force acc. to EN 1993-1-2, 4.2.3.3
	3.750	RC3	0.77	≤ 1	FS831)	Fire design - Stability analysis - Lateral torsional buckling acc. to EN 1993-1-2, 4.2.3.3

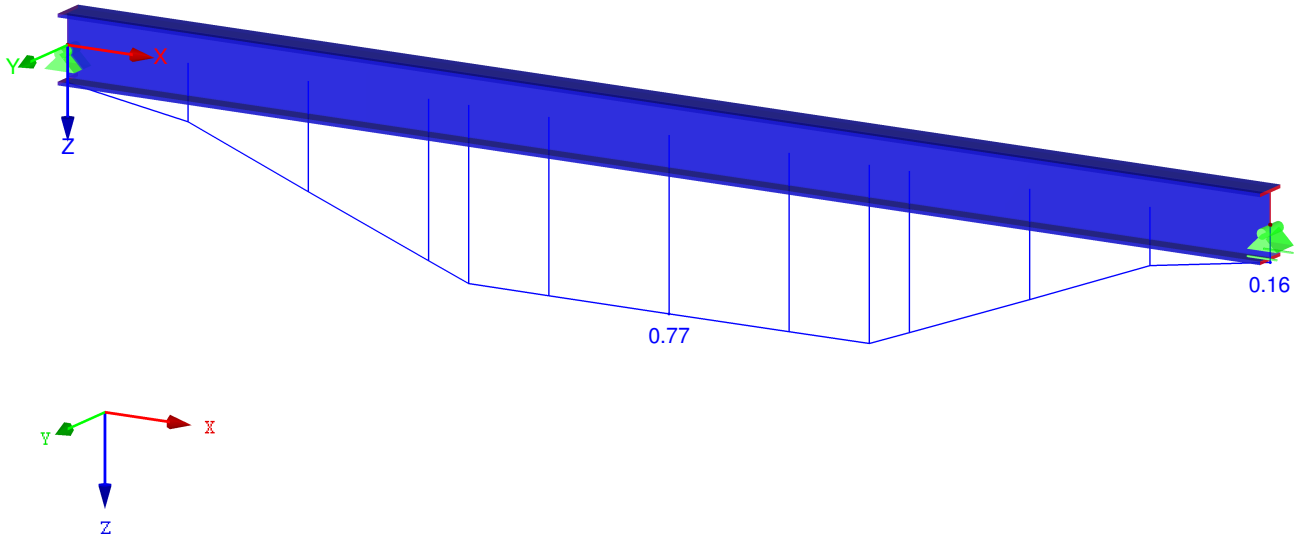
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DESIGN: FIRE RESISTANCE - CROSS-SECTION DESIGN + STABILITY DESIGN

STEEL EC3 CA1
Fire Resistance: Cross-Section Design, Stability Design

Isometric



Max Design Ratio: 0.77