UNIVERCITY OF ARCHITECTURE, CIVIL ENGINEERING AND GEODESY

Department of Steel, Timber and Plastic Structures

GRADUATION PROJECT

Project name:

"Structural design of the steel structure of multi-storey building in Sofia“

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I. In general

1. Architecture

The architecture of the building for this project to be made was borrowed from a skyscraper, built in Taipei, Taiwan in 2004. It is designed for commercial, entertainment and administrative purposes. It consists of a high building with 37 overground floors, reaching 160.6 m in height, and a low building with 7 overground floors, reaching the elevation +45.20 m.

There are 7 underground levels for commercial areas and parking lots. The upper part is served by 8 passenger and 2 service elevators to the 20th floor, where 4 of the elevators provided for passengers are interrupted. In the lower part of the building, intended for a commercial area, the vertical communications are carried out by means of 10 elevators, both for visitors and service. Escalators and several stairwells are provided.

At the top of the building at elevation of +143.90 m is placed an outdoor pool measuring 7.50 m x 20.00 m. The pool is placed in the center of an architectural structure with several storeys, giving an iconic look to the building.

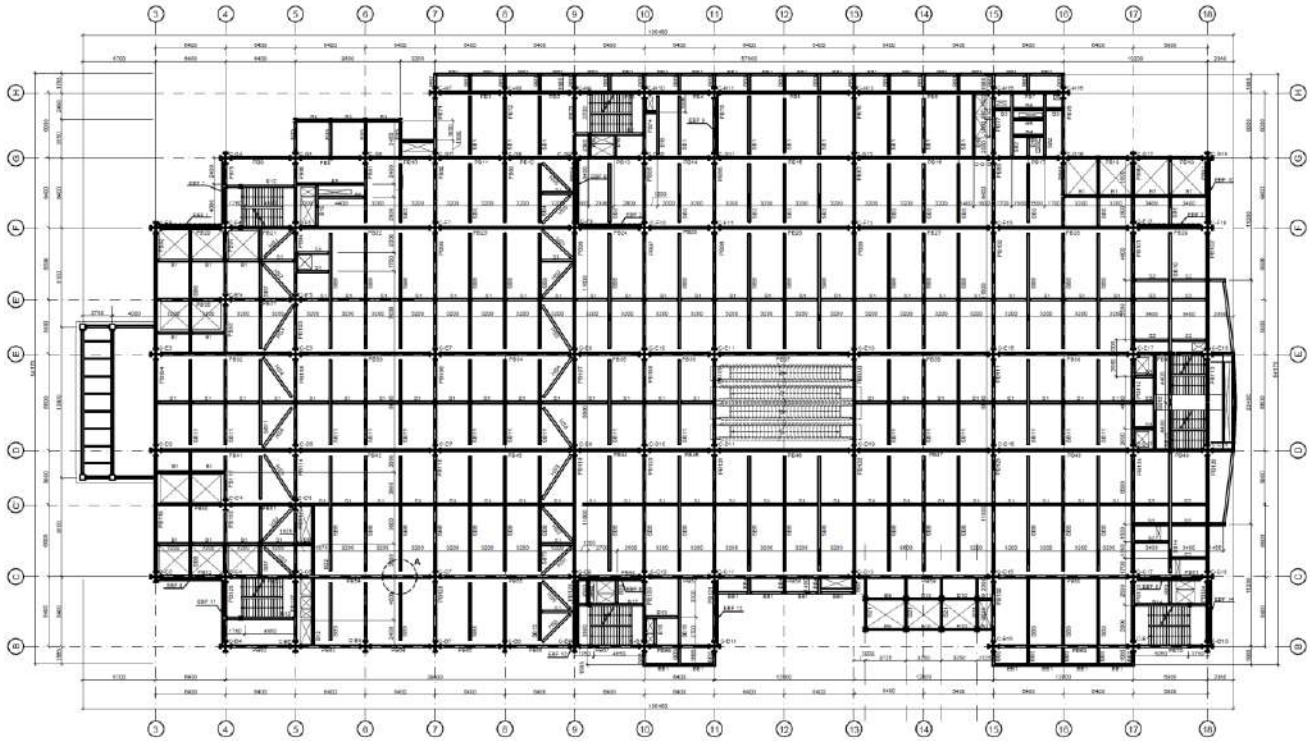
The thesis h being developed for the territory of Sofia, Mladost district. The overall architectural appearance and the number of storeys above the elevation of +0.20 m are preserved. The underground levels are reduced to 3, reaching an elevation of -13.40 m, in order to simplify the foundation and strengthen the excavation.

2. Structural system

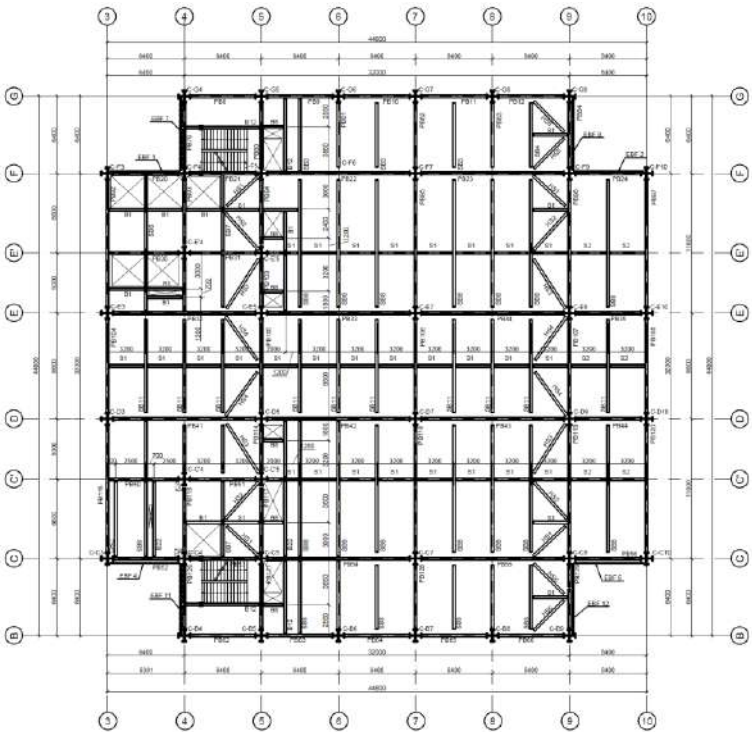
Building slabs are composite. Composite slabs are constructed from reinforced concrete cast on top of profiled steel decking, which works as a remaining formwork and reduces the amount of lower reinforcement required in the field. Such combined slabs are a good solution from a constructive, technological and economic point of view, as they reduce the thickness of the concrete in the slab, its own weight, as well as the construction time. Special profiled steel sheets are used, providing mechanical interaction with the concrete.

A down stand beam is connected to a composite slab by the use of through-deck welded shear studs. These studs are to take the shear forces between the concrete slab and the steel profile.

Plan of the floor construction on a standard floor up to elevation +35.70 m

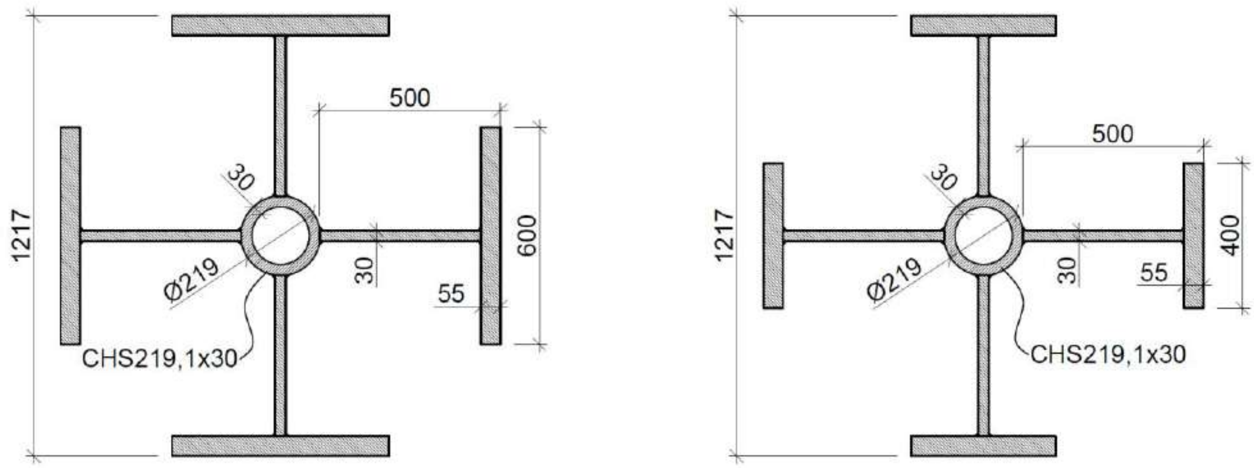


Plan of a standard floor construction over the elevation +35.70 m

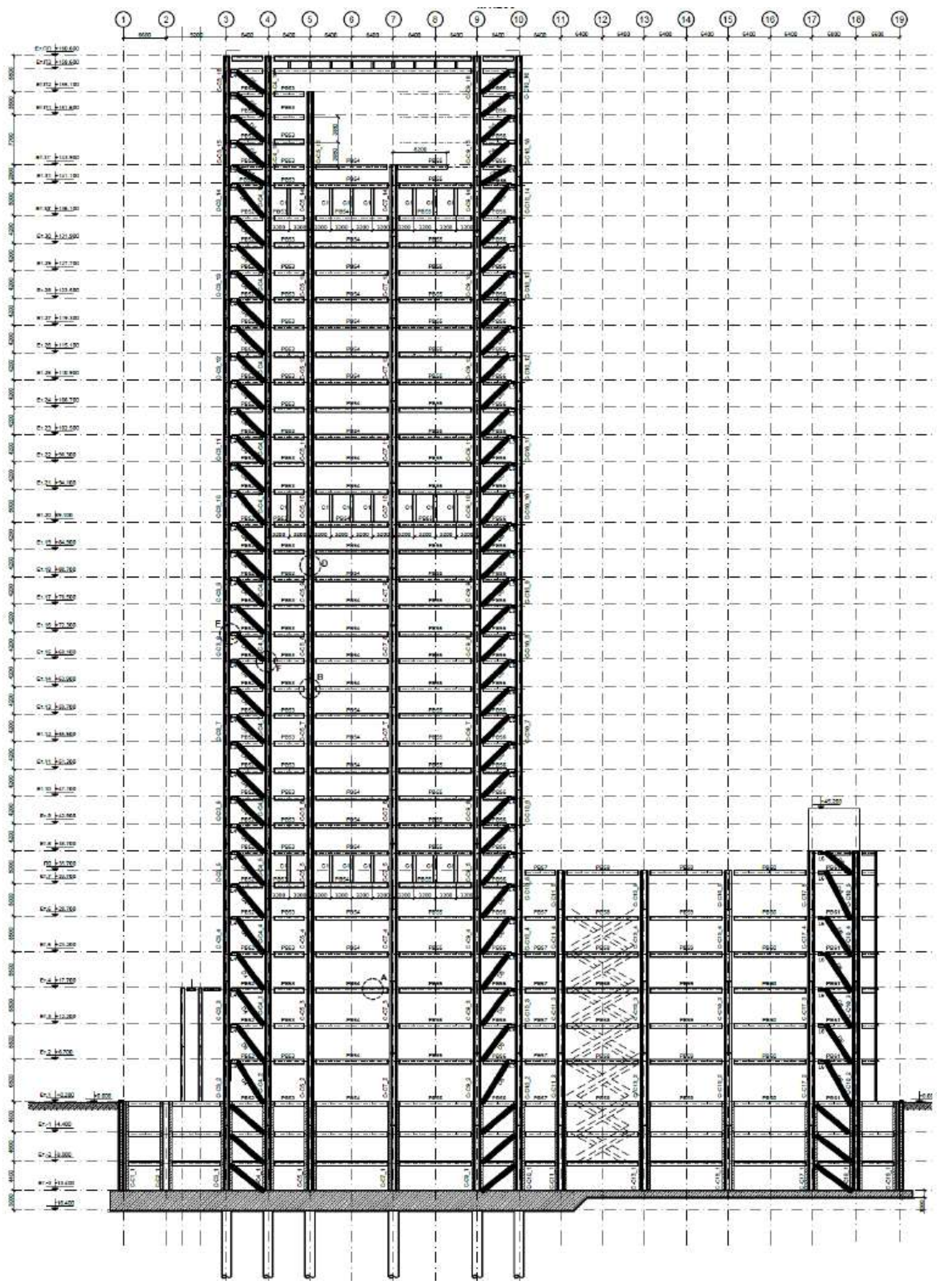


The vertical loads from the floor constructions are transported to the base by powerful columns with a composite cross section. Due to the high axial forces and bending moments from horizontal impacts (wind and earthquake), the columns are designed with high-strength steel Histar S460JR.

<u>*Column section*</u>

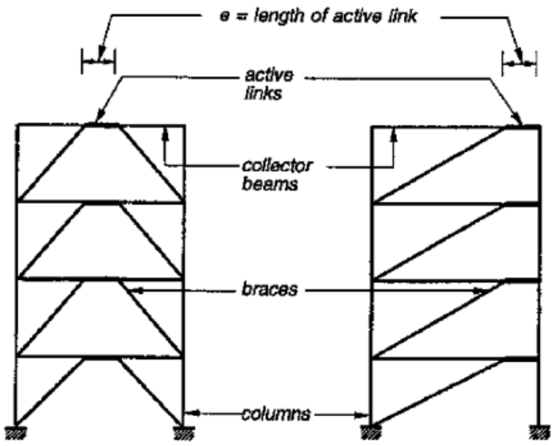


<u>*Section through the structure along the "C" axis*</u>



The assumption of the horizontal load caused by wind and earthquake is provided by a mixed (dual) MRF-EBF system, as in both directions moment resisting frames and eccentrically braced frame are constructed.

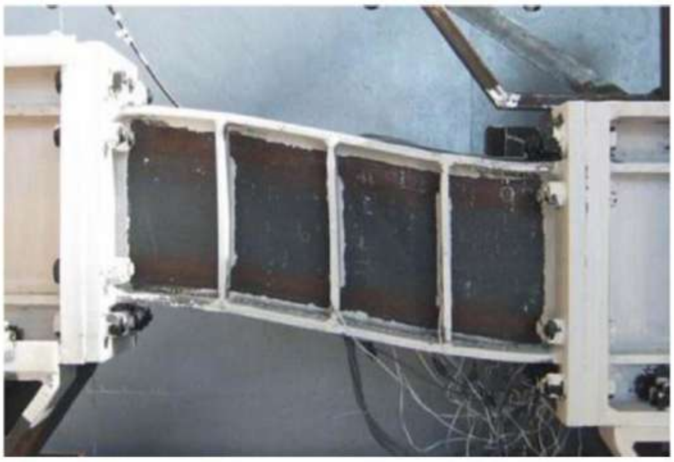
The system of moment resisting frames consists of rigidly connected beams and columns, forming a space frame that acts as a support system. The eccentrically braced frames are capable of resisting most of the design forces. This type of dual construction system combines the advantages of the two main systems. The structure has a high static uncertainty, very good global ductility and takes advantage of the frame system to distribute the shear force generated by wind and earthquake at multiple anchor points.



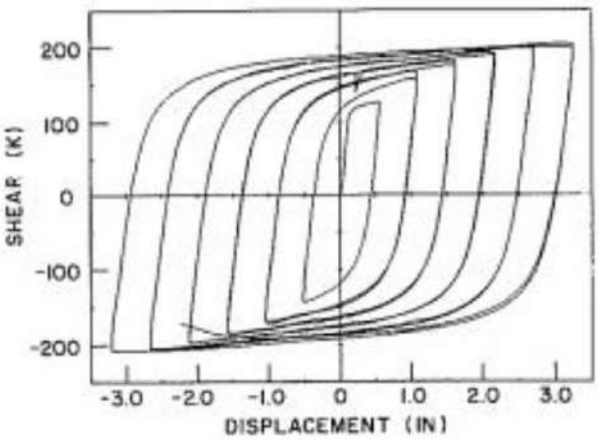
The brace systems for resisting the horizontal loads are realized through Eccentrically braced frames (EBF). They consist of columns, beams and braces and also of a link element.

The design seeks assuring the development of a collapse mechanism of global type by relying on yielding at the link beam between braces to dissipate seismic energy. Depending on the design, the link element yields by shear, bending or both. This project is considering a solution with EBF with replaceable shear links. According to studies, short shear links have better dissipative behavior than longer links, which dissipate energy by yielding in bending.

Ductile deformation in a shear link element



Hysteresis loop of a shear link element



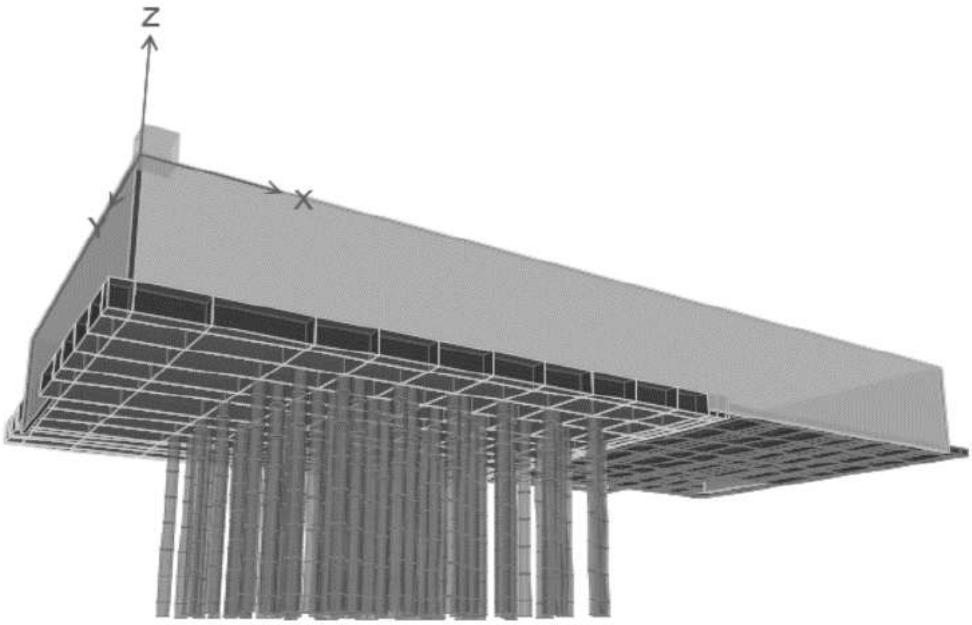
Replaceable seismic link elements are a very good solution in seismically active areas, because after seismic impact they can be replaced with new uncompromised elements. This type of connection is proven to be effective and cost efficient, because it prevents the complete reconstruction of the structural elements and leads to much lower costs for the restoration of buildings after seismic impact. Link elements are easy to replace as they can be connected by bolted end plates, which also reduces installation time.

3. Foundation

The foundation construction of the building is a joint foundation slab with variable thickness and pouring concrete piles, placed under the multi-storey part of the building (the part of the building reaching an elevation of +160.6 m.)

Piles are placed under each column of the multi-storey building and between the columns as well to observe the minimum required distances, ensuring the independent behavior of each pile. The total number of piles is 85 and they are united by a foundation slab with a thickness of *3m* In the area of the low building the foundation slab changes its thickness to *1m.*

A solution of the building with 3 underground levels and basement walls is considered. The basement walls together with the foundation slab act as a rigid box-like structure and distribute the horizontal earthquake forces, which are completely balanced by the ground pressure at rest and the friction forces in the foundation slab. As a result the piles take only axial forces from the vertical loads and insignificant bending moments from the bending of the foundation slab.



4. Construction technology

In the current graduation project is considered an exemplary technology for installation of the structure and an exemplary technology for concrete work above elevation +0.00.

For the installation of structural elements are provided tower cranes. For the implementation of the structure up to elevation +45.20 m there are three tower cranes provided, one of which is located on the outside of the multi-storey building (160.6 m), and the other two are to serve the parts of the building, reaching elevation +45.20 m. After reaching this elevation, the two tower cranes are disassembled and only one tower crane remains, serving the growing structure up to 160.6 m. It has a horizontal boom, high load capacity and increases itself in height. Supports are necessary among the built up floors, due to the increasing high of the tower crane.

The concrete works are realized with two stationary concrete pumps with distribution booms. Their delivery rate is selected on the basis of the required hourly inflow of concrete for monolithic execution of floor slabs. One concrete pump serves the multi-storey building and the other the lower structure. The concrete reaches the required elevation by means of a special construction, reinforced at the levels of the completed composite floor structures

II. Loads

1. Roof loads

The architectural design of the building foresees an outdoor swimming pool at elevation +143.9m. The roof is to be exploit as well and live loads and water loads are taken into account.

1.1. *Dead loads*

- Weigts of materials are taken into account while creating the 3D model of the structure with ETABS
- Water load

$V_p = b_1.b_2.h = 7{,}5.20.1{,}20 = 180\ m^3$

$b_1 = 7{,}5\ m$

$b_2 = 20\ m$

$h = 1{,}20\ m$

$\gamma_w = 9{,}81\ kN/m^3$

$F_{G,p} = V_p.\ \gamma_w = 180.9{,}81 = 1\ 765{,}8\ kN \rightarrow \boldsymbol{F_{G,p} = 1\ 765{,}8\ kN}$ – Water load is applied as a concentrated force in the geometrical center of the roof;

1.2. *Live loads*

$\boldsymbol{q_k = 3{,}0\ kN}$ – Human and furniture loads are distributed at the roof floor.

2. Floor loads

Натоварване върху типов етаж					
В експлоатационно състояние			В монтажно състояние		
Постоянни товари		[kN/m^2]	**Постоянни товари**		[kN/m^2]
1	Настилка	0,50	1	Армировка	0,03
2	Циментова замазка	0,36	2	Прясна бетонна смес	2,71
3	Бетон	2,60	3	Профилирана ламарина	0,11
4	Армировка	0,03	**Временни въздействия**		[kN/m^2]
5	Проф. ламарина	0,11	1	Натоварване в работен участък	1,5
6	Инсталации	0,50			
7	Окачен таван	0,20	2	Натоварване извън работен участък	0,75
8	Преградни стени	1,00			
Временни въздействия		[kN/m^2]			
*	Хора и оборудване	5,00			

3. Facade loads

It’s been chosen a glass facade E85, ETEM.

Facade loads are applied on the structure by linear distributed load, applied on the contour beams of the floor structure.

Facade loads by floors:

Етаж	z [m]	Q_i [kN/m]	Етаж	z [m]	Q_i [kN/m]
1	0,2	**1,7**	-	-	-
2	6,7	**3,0**	**20**	89,1	**2,3**
3	12,2	**2,8**	**21**	94,1	**2,3**
4	17,7	**2,8**	**22**	98,3	**2,1**
5	23,2	**2,8**	**23**	102,5	**2,1**
6	28,7	**2,6**	**24**	106,7	**2,1**
7	35,7	**1,3**	**25**	110,9	**2,1**
8	38,7	**1,8**	**26**	115,1	**1,2**
9	42,9	**1,5**	**27**	119,3	**1,9**
10	47,1	**1,6**	**28**	123,5	**2,1**
11	51,3	**2,1**	**29**	127,7	**2,1**
12	55,5	**2,1**	**30**	131,9	**2,1**
13	59,7	**2,1**	**30'**	136,1	**2,3**
14	63,9	**2,1**	**31**	141,1	**2,0**
15	68,1	**2,1**	**31'**	143,9	**2,6**
16	72,3	**2,1**	**P1**	151,6	**2,8**
17	76,5	**2,1**	**P2**	155,1	**1,7**
18	80,7	**2,1**	**P3**	158,6	**1,4**
19	84,9	**2,1**	**PP**	160,6	**0,5**

$Q_{f,i} = q_f.(h_i/2 + h_{i+1}/2)$ $[kN/m]$,

Where:

$q_f = 0{,}5$ kN/m^2 – façade loads;

h_i – high of the i-floor;

h_{i+1} – high of the (i+1)-floor;

4. Snow load

$s = \mu_i .C_e .C_t .s_k$, where:

For roofs with slope: $0^\circ \leq \alpha \leq 30^\circ \rightarrow \mu_i = 0{,}8$ - coefficient for snow load according to the roof shape;

$C_e = 1{,}0$

$C_t = 1{,}0$

$s_k = 1{,}28$ kN/m^2 – snow load for Sofia region;

$s = \mu_i .C_e .C_t .s_k = 0{,}8.1{,}0.1{,}0.1{,}28 = 1{,}024$ $kN/m^2 \rightarrow$ $\mathbf{s = 1{,}024}$ **kN/m²**

5. Wind load

Calculations are made according to BDS EN 1991-1-4.

1.3. Wind velocity and velocity pressure

5.1.1. Basis for calculation

$v_b = c_{dir} .c_{season} .v_{b,0}$, where:

$c_{dir} = 1{,}0$ – БДС EN1991-1-4/NA;

$c_{season} = 1{,}0$ – БДС EN 1991-1-4/NA;

$v_{b,0} = 26{,}1$ m/s - БДС EN 1991-1-4/NA;

$v_b = 1.1.26{,}1 = 26{,}1$ $m/s \rightarrow$ $\mathbf{v_b = 26{,}1}$ **m/s**

5.1.2. Mean wind

$\mathbf{v_m (z) = c_r (z).c_0 (z).v_b}$, where:

$c_0 (z) = 1{,}0$

$c_r(z) = k_r \, . \, ln\left(\frac{z}{z_0}\right)$

$k_r = 0{,}19.\left(\frac{z_0}{z_{0,II}}\right)^{0{,}07}$

$z_{0,II} = 0{,}05\ m$

$z_0 = 0{,}3\ m$

$z_{min} = 5\ m$ } III category terrain according to БДС EN 1991-1-4

$z_{max} = 200\ m$

$k_r = 0{,}19.\left(\frac{0{,}3}{0{,}05}\right)^{0{,}07} = 0{,}2154$

$\boldsymbol{k_r = 0{,}2154}$

Категория на терена		Z_0, m	z_{min}, m
0	Море или крайбрежни местности с изложение към открито море	0,003	1
I	Езера или равнинна хоризонтална местност с пренебрежимо малка растителност и без препятствия	0,01	1
II	Местност с ниска растителност, например трева, и отделни препятствия (дървета, сгради), които са отдалечени на разстояние, най-малко 20 пъти височината на препятствието.	0,05	2
III	Местност, която е равномерно покрита с растителност или сгради или с изолирани препятствия, отдалечени на разстояние най-много 20 пъти височината на препятствията (например села, крайградски зони, гори)	0,3	5
IV	Местност, най-малко 15 % от повърхността на която е покрита със сгради, чиято средна височина превишава 15 m	1,0	10
ЗАБЕЛЕЖКА : Категориите терени са показани в А.1.			

5.1.3. Wind turbulence

Standard deviation: $\sigma_v = k_r \, .v_b \, .k_I$, където:

$k_I = 1{,}0$ - БДС EN 1991-1-4/NA;

$\sigma_v = 0{,}2154 \, .26{,}1 \, .1{,}0 = 5{,}62\ m/s$

$\boldsymbol{I_v(z) = \frac{\sigma_v}{v_m(z)}}$

5.1.4. Peak velocity pressure

$q_p(z) = [1 + 7{,}0.\ I_v(z)].1/2.\rho.v_m^2(z) = c_e(z).q_b$

$c_e(z) = q_p(z)/\ q_b$

$q_b = 0{,}5.\rho.v_b^2 = 0{,}5.1{,}25.26{,}1^2 = 425{,}8\ N/m^2 \rightarrow \boldsymbol{q_b = 425{,}8\ N/m^2}$

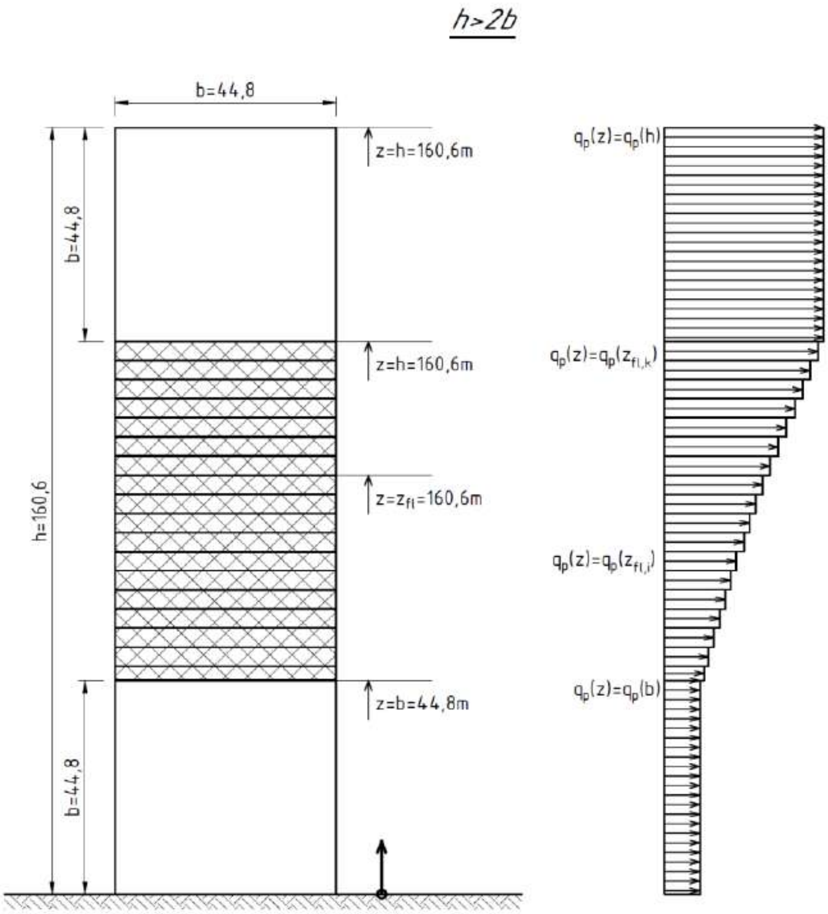
$\rho = 1{,}25\ kg/m^3$

1.4. <u>Wind pressure on surfaces</u>

$\boldsymbol{w_e = q_p(z).c_{pe,10}}$ – wind pressure on external surfaces

$c_{pe,10}$ – pressure coefficient for the external pressure;

5.1.5. Pressure on vertical surfaces

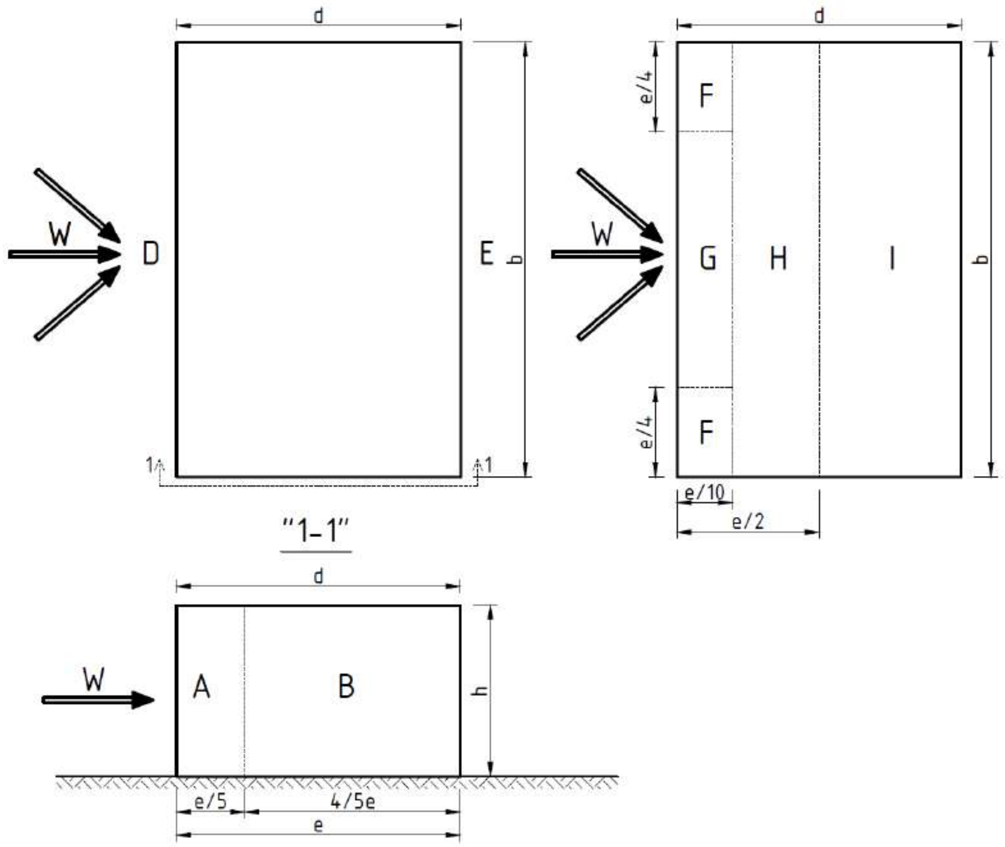


High of the building *(h = 160,6 m)* is bigger than it width (*b = 44,8m),* multiplied by two, and this is why pressure is separated in three. First zone reaches high of *z = b*, where wind pressure is constant and it's velocity is $q(z) = q(b)$. Last zone is placed on a distance *b* from the top of the building and is as wide as the first zone. Wind pressuer is constant as well and it's velocity is $q(z) = q(h)$. Between these two zones there is a third one, where wind pressure is increasing as a function of high $q(z) = q(z_{fl,i})$.

5.1.6. Wind zones

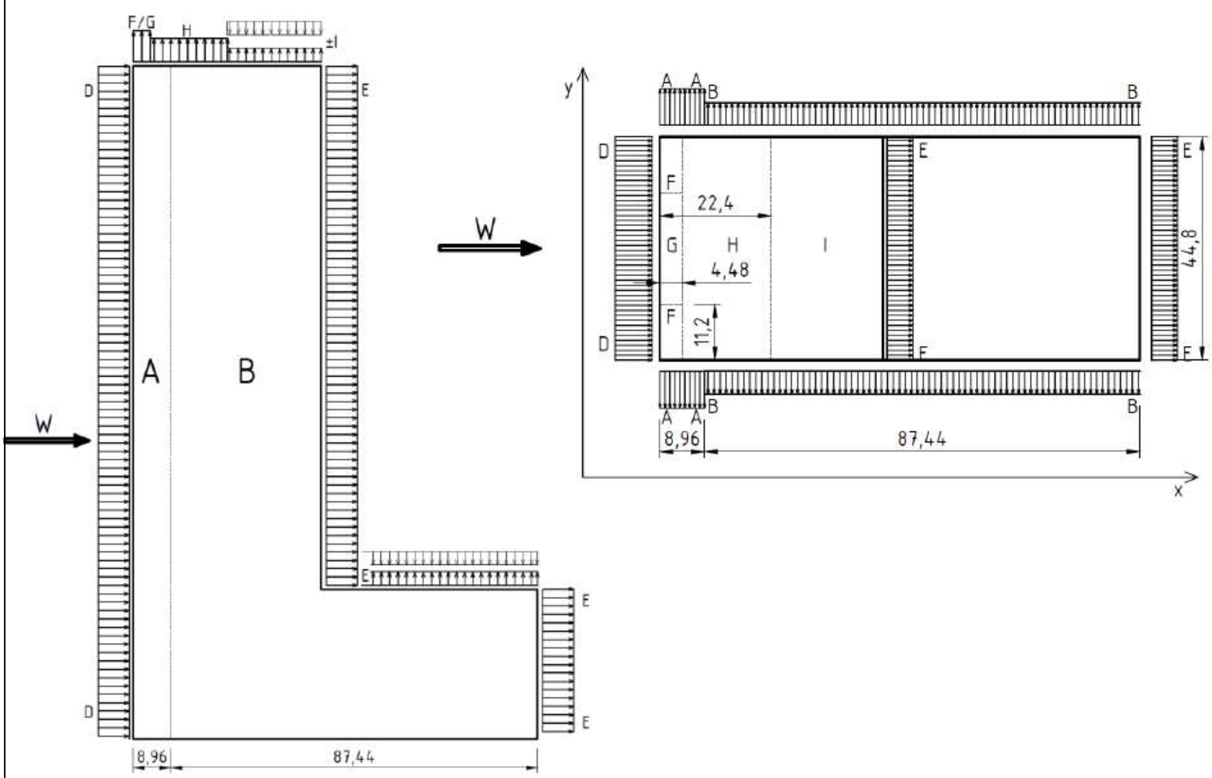
Zone from *A÷E* show loads on vertical surfaces of the structure.

Zone from *F÷I* show loads on the roof of the structure. In this case, due to the complicated shape of the roof, in calculations is assumed that the roof is flat.



5.1.7. Distribution of wind load due to wind direction

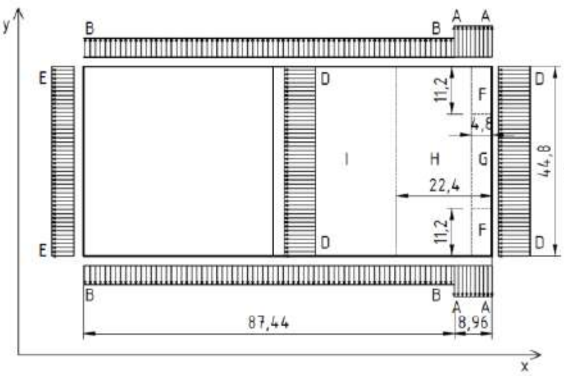
- *Wind with direction* ***+X***

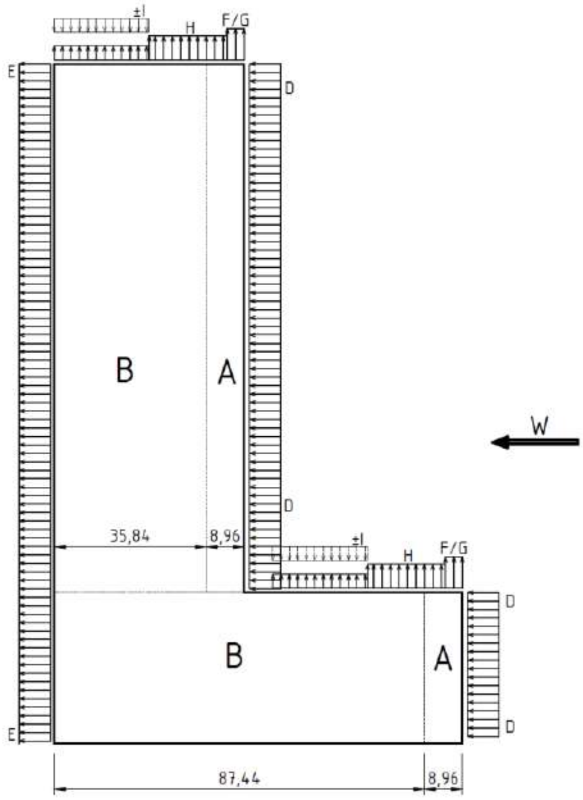


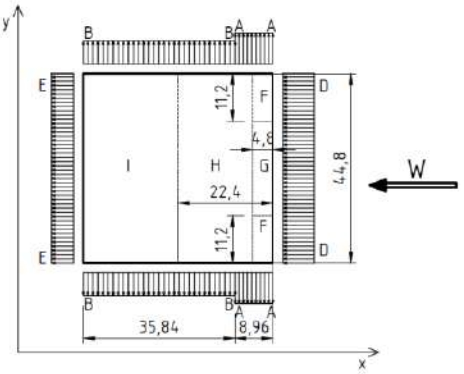
Wind loads in +X direction

z	cr(z) [-]	vm(z) [m/s]	Iv(z) [-]	qp(z) [kN/m^2]	A	B	D	E
					w_e [kN/m^2]			
0	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
0,2	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
6,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
12,2	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
17,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
23,2	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
28,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
33,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
35,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
38,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
42,9	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
44,8	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
47,1	1,09	30,93	0,18	1,36	-1,63	-1,09	1,09	-0,86
51,3	1,11	31,45	0,18	1,39	-1,67	-1,11	1,11	-0,88
55,5	1,12	31,93	0,18	1,42	-1,71	-1,14	1,14	-0,90
59,7	1,14	32,38	0,17	1,45	-1,74	-1,16	1,16	-0,91
63,9	1,15	32,80	0,17	1,48	-1,77	-1,18	1,18	-0,93
68,1	1,17	33,19	0,17	1,50	-1,81	-1,20	1,20	-0,95
72,3	1,18	33,55	0,17	1,53	-1,83	-1,22	1,22	-0,96
76,5	1,19	33,90	0,17	1,55	-1,86	-1,24	1,24	-0,98
80,7	1,21	34,22	0,16	1,57	-1,89	-1,26	1,26	-0,99
84,9	1,22	34,54	0,16	1,59	-1,91	-1,28	1,28	-1,00
89,1	1,23	34,83	0,16	1,61	-1,94	-1,29	1,29	-1,02
94,1	1,24	35,16	0,16	1,64	-1,96	-1,31	1,31	-1,03
98,3	1,25	35,43	0,16	1,66	-1,99	-1,32	1,32	-1,04
102,5	1,26	35,69	0,16	1,67	-2,01	-1,34	1,34	-1,05
106,7	1,27	35,93	0,16	1,69	-2,03	-1,35	1,35	-1,07
110,9	1,27	36,17	0,16	1,71	-2,05	-1,37	1,37	-1,08
115,1	1,28	36,40	0,15	1,72	-2,07	-1,38	1,38	-1,09
115,8	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
119,3	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
123,5	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
127,7	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
131,9	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
136,1	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
141,1	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
143,9	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
151,6	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
155,1	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
158,6	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
160,6	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18

➢ *Wind with direction* ***–X***







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					w_e [kN/m²]			
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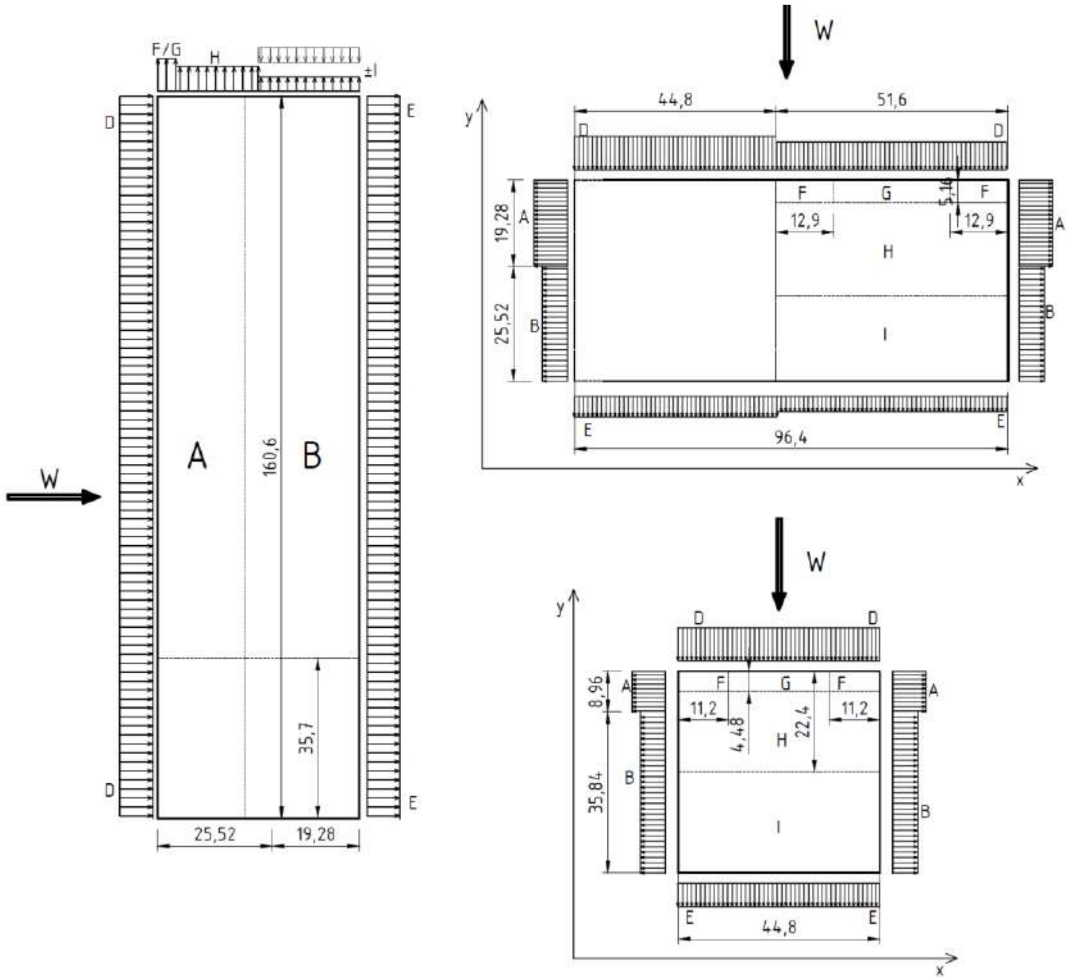
17,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
23,2	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
28,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
33,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
35,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
38,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
42,9	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
44,8	1,08	30,62	0,18	1,34	-1,61	-1,07	1,07	-0,84
47,1	1,09	30,93	0,18	1,36	-1,63	-1,09	1,09	-0,86
51,3	1,11	31,45	0,18	1,39	-1,67	-1,11	1,11	-0,88
55,5	1,12	31,93	0,18	1,42	-1,71	-1,14	1,14	-0,90
59,7	1,14	32,38	0,17	1,45	-1,74	-1,16	1,16	-0,91
63,9	1,15	32,80	0,17	1,48	-1,77	-1,18	1,18	-0,93
68,1	1,17	33,19	0,17	1,50	-1,81	-1,20	1,20	-0,95
72,3	1,18	33,55	0,17	1,53	-1,83	-1,22	1,22	-0,96
76,5	1,19	33,90	0,17	1,55	-1,86	-1,24	1,24	-0,98
80,7	1,21	34,22	0,16	1,57	-1,89	-1,26	1,26	-0,99
84,9	1,22	34,54	0,16	1,59	-1,91	-1,28	1,28	-1,00
89,1	1,23	34,83	0,16	1,61	-1,94	-1,29	1,29	-1,02
94,1	1,24	35,16	0,16	1,64	-1,96	-1,31	1,31	-1,03
98,3	1,25	35,43	0,16	1,66	-1,99	-1,32	1,32	-1,04
102,5	1,26	35,69	0,16	1,67	-2,01	-1,34	1,34	-1,05

106,7	1,27	35,93	0,16	1,69	-2,03	-1,35	1,35	-1,07
110,9	1,27	36,17	0,16	1,71	-2,05	-1,37	1,37	-1,08
115,1	1,28	36,40	0,15	1,72	-2,07	-1,38	1,38	-1,09
115,8	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
119,3	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
123,5	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
127,7	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
131,9	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
136,1	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
141,1	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
143,9	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
151,6	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
155,1	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
158,6	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18
160,6	1,35	38,43	0,15	1,87	-2,24	-1,49	1,49	-1,18

- *Roof wind loads when wind is in X direction*

z	***F***	***G***	***H***	***I+***	***I-***
	w_e [kN/m2]				
35,7	-1,78	-1,22	-0,78	0,22	-0,22
160,6	-1,98	-1,32	-1,15	0,33	-0,33

- *Wind with direction ±**Y***

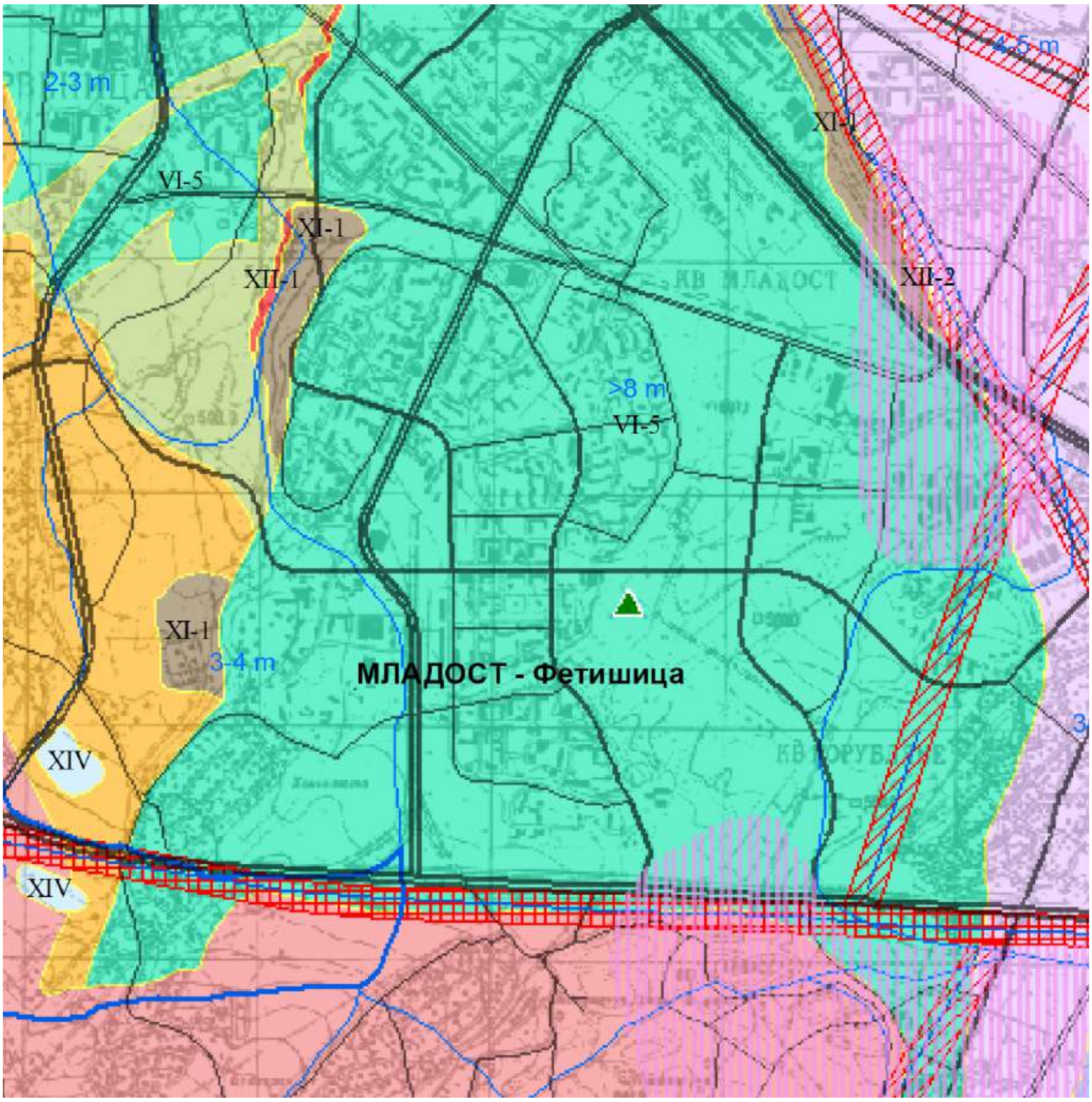


Wind loads in Y direction

z	cr(z) [-]	vm(z) [m/s]	Iv(z) [-]	qp(z) [kN/m ^{2}]	A	B	D'	E'	D	E
					w_e [kN/m ^{2}]					
0	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
0,2	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
6,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
12,2	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
17,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
23,2	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
28,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
33,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
35,7	1,08	30,62	0,18	1,34	-1,61	-1,07	1,02	-0,56	1,07	-0,80
38,7	1,08	30,62	0,18	1,34	-1,61	-1,07	-	-	1,07	-0,80
42,9	1,08	30,62	0,18	1,34	-1,61	-1,07	-	-	1,07	-0,80
44,8	1,08	30,62	0,18	1,34	-1,61	-1,07	-	-	1,07	-0,80
47,1	1,09	30,93	0,18	1,36	-1,63	-1,09	-	-	1,09	-0,82
51,3	1,11	31,45	0,18	1,39	-1,67	-1,11	-	-	1,11	-0,84
55,5	1,12	31,93	0,18	1,42	-1,71	-1,14	-	-	1,14	-0,85
59,7	1,14	32,38	0,17	1,45	-1,74	-1,16	-	-	1,16	-0,87
63,9	1,15	32,80	0,17	1,48	-1,77	-1,18	-	-	1,18	-0,89
68,1	1,17	33,19	0,17	1,50	-1,81	-1,20	-	-	1,20	-0,90
72,3	1,18	33,55	0,17	1,53	-1,83	-1,22	-	-	1,22	-0,92
76,5	1,19	33,90	0,17	1,55	-1,86	-1,24	-	-	1,24	-0,93
80,7	1,21	34,22	0,16	1,57	-1,89	-1,26	-	-	1,26	-0,94
84,9	1,22	34,54	0,16	1,59	-1,91	-1,28	-	-	1,28	-0,96
89,1	1,23	34,83	0,16	1,61	-1,94	-1,29	-	-	1,29	-0,97
94,1	1,24	35,16	0,16	1,64	-1,96	-1,31	-	-	1,31	-0,98
98,3	1,25	35,43	0,16	1,66	-1,99	-1,32	-	-	1,32	-0,99
102,5	1,26	35,69	0,16	1,67	-2,01	-1,34	-	-	1,34	-1,00
106,7	1,27	35,93	0,16	1,69	-2,03	-1,35	-	-	1,35	-1,01
110,9	1,27	36,17	0,16	1,71	-2,05	-1,37	-	-	1,37	-1,02
115,1	1,28	36,40	0,15	1,72	-2,07	-1,38	-	-	1,38	-1,03
115,8	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
119,3	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
123,5	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
127,7	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
131,9	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
136,1	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
141,1	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
143,9	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
151,6	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
155,1	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
158,6	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12
160,6	1,35	38,43	0,15	1,87	-2,24	-1,49	-	-	1,49	-1,12

6. Seismic loads

1.5. Ground



ТЕРЕНИ БЛАГОПРИЯТНИ ЗА СТРОИТЕЛНИ ДЕЙНОСТИ

А1 - Магмени скали, Донеогенски магмени и седиментни скали

А2 - Вулканогенно-седиментни скали, Донеогенски магмени и седиментни скали

……………………………………………………………..

VI-5 - Алувиални глини и чакъли в кв. Младост и кв. Дървеница

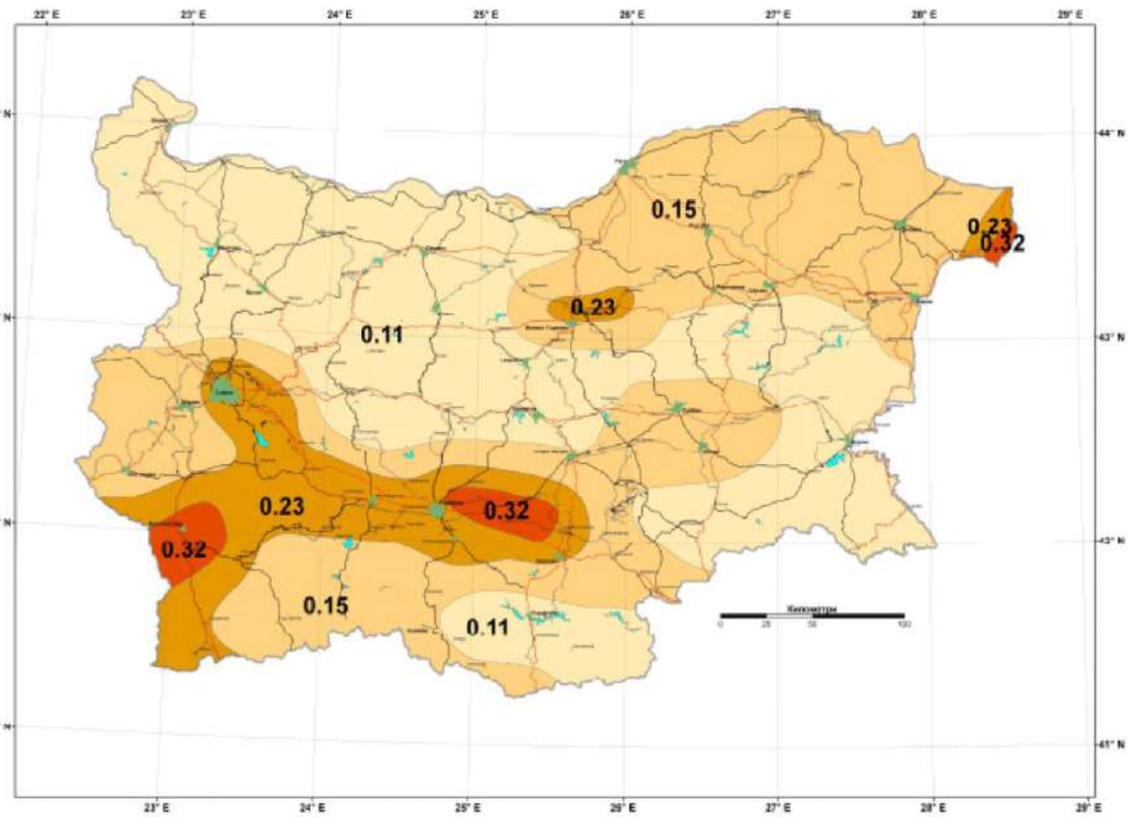
According to the map for "Engineering-geological and hydro-geological zoning of Sofia", the terrain of the neighborhood "Mladost" lays in an area with alluvial clays and gravels and is classified as land suitable for construction. According to the map, the groundwater level for the whole region is below 8 m depth. The characteristics of the ground base belong to category “C”, according to БДС EN 1998 - 1, item 3.1.2.

Тип земна основа	Описание на стратиграфския профил	Параметри		
		$v_{S,30}$ [m/s]	N_{SPT} [удари/30 cm]	C_U [kPa]
A	Скала или друг вид скални образувания, включващи най-много 5 m по-слаби материали под повърхността	>800	-	-
B	Пластове от много плътен пясък, чакъл или много твърда глина с дебелина поне няколко десетки метра, характеризиращи се с постепенно нарастване на механичните характеристики в дълбочина	360-800	>50	>250
C	Дълбоки депозити от плътен или средно плътен пясък, чакъл или твърда глина с дълбочина от няколко десетки до стотици метри	180-360	15-50	70-250
D	Депозити от слаби до средно сбити несвързани почви (със или без слаби прослойки от свързани почви) или предимно меки до твърди свързани почви	<180	<15	<70
E	Почвени профили от повърхностни алувиални пластове с v_S на тип C или D и дълбочина, варираща между 5 и 20 m, лежащи върху твърда основа с v_S > 800 m/s			
S_1	Депозити, съставени или включващи пласт с дебелина най-малко 10 m от меки глини/наноси с висок показател на пластичност (PI > 40) и голямо водно съдържание	<100	-	10 - 20
S_2	Депозити от втечняващи се почви, от чувствителни глини или други почвени профили, невключени в типове от A до E или S_1			

1.6. Peak ground acceleration

6.1.1. Seismic hazard for PGA in 475-year return period

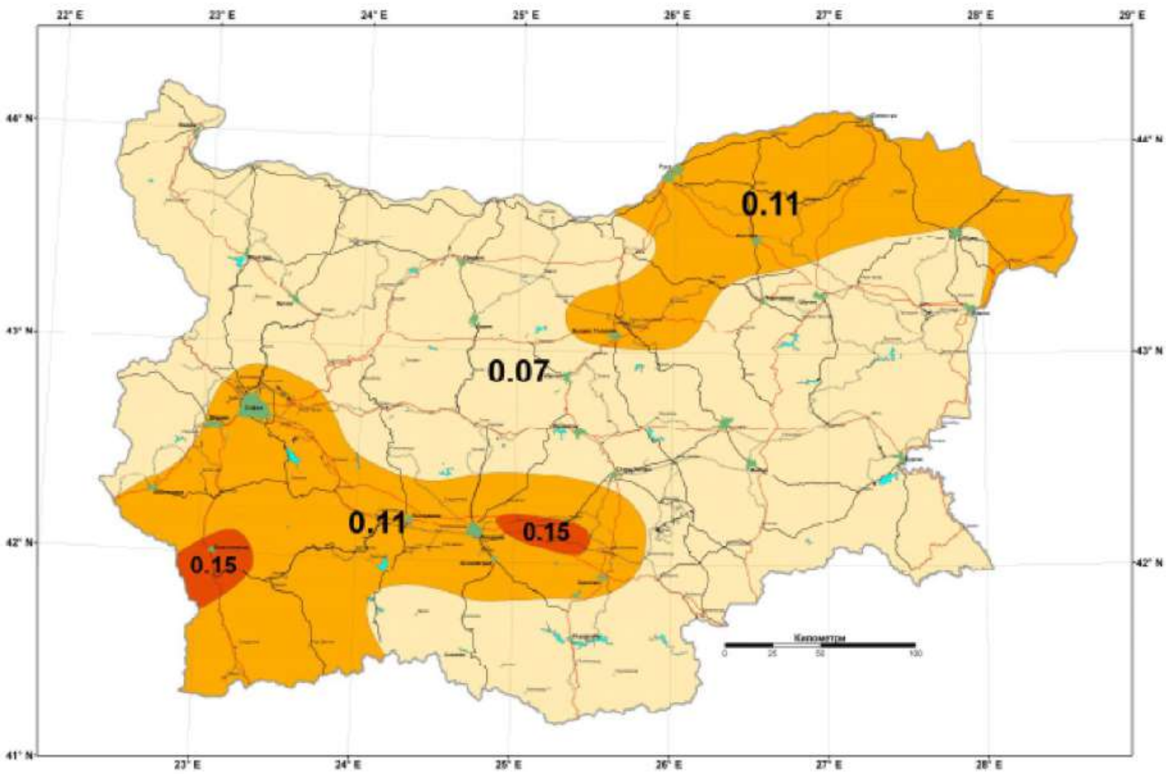
According to NA.D of БДС EN 1998 – 1: 2005/NA:2012, the region of Sofia has $a_{gr} = 0,23g\ m/s^2$.



According to the map for “Microseismic zoning on the territory of the city of Sofia and Sofia Municipality”, the area for construction of the building lays within the zone with PGA in 475-year return period $\boldsymbol{a_{gr} = 0,30g\ m/s^2}$.

6.1.1. Seismic hazard for PGA in 95-year return period

To increase safety, the PGA in 95-year return period won't be taken from the NA map, where it is due to be $a_{gr} = 0,11g$.



It's used the following formula:

$$T = 95\ г\!: a_{gr} = a_{gr}^{95} = 0,4.\ a_{gr}^{475} = 0,4.0,30 = 0,12s \rightarrow \boldsymbol{a_{gr} = 0,12g}$$

6.1.2. Designed value of PGA

Buildings are classified into 4 classes of significance depending on the consequences of complete destruction for human life, their significance for public safety and civil protection after the earthquake, as well as the social and economic consequences as a result of destruction. According to these conditions and according to the area in which the building will be located, classified in class of importance III.

Клас на значимост	Сгради
I	Сгради с малка значимост за обществена безопасност, например селскостопански сгради и др.
II	Обикновени сгради, непринадлежащи към другите категории
III	Сгради, чиято сеизмична носеща способност е от значение от гледна точка на последиците от пълно разрушаване, например училища, зали, културни институции и др.
IV	Сгради, чиято цялост по време на земетресения е от жизнено значение за защита на населението, например болници, противопожарна охрана, електроцентрали и др.

According to table NA.4.3. на БДС EN 1998 – 1, the value of the importance factor $\boldsymbol{\gamma_I = 1,2}$.

Клас на значимост	I	II	III	IV
Коефициент на значимост γ_I	0,8	1,0	1,2	1,4

Designed value for PGA is:

$\boldsymbol{a_g = \gamma_I.a_{gr}}$

- Designed value for PGA *(T = 475yars)*

$a_g = \gamma_I.a_{gr} = 1,2.0,30g = 0,36g \rightarrow \underline{\boldsymbol{a_g = 0,36g}}$

- Designed value for PGA *(T = 95yars)*

$a_g = \gamma_I.a_{gr} = 1,2.0,12g = 0,144g \rightarrow \underline{\boldsymbol{a_g = 0,144g}}$

1.7. *Behavior factor*

The stiffness for horizontal loads is similar in the two main directions, and the structural type is the same in these directions. It is dual system consisting of MRF and EBF.

According to БДС EN 1998-1, т. 6.3.2. for Dual systems and DCM the value for the behavior factor is $\boldsymbol{q = 3,0.\alpha_u/\alpha_1}$.

$\alpha_u/\alpha_1 = 1,2$ – frame-equivalent dual structures;

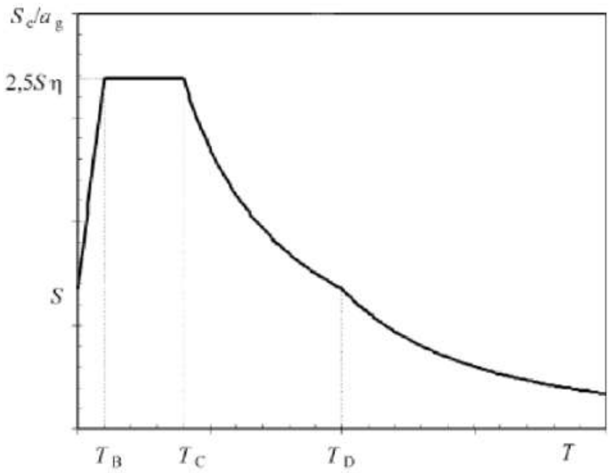
$q_0 = 3,0.\alpha_u/\alpha_1 = 3,0.1,2 = 3,6$

For buildings, which are not regular in elevation, the value q_0 should be reduced by 20%!

$q = 0,8.q_0 = 0,8.3,6 = 3,2 \rightarrow$ *Value of the behavior factor for the structure:* $\underline{\boldsymbol{q = 3,2}}$

1.8. *Horizontal elastic response spectrum*

Type 1 response spectrum is used for the territory of Bulgaria. For parts of Northern Bulgaria type 3 response spectrum, typical for the Vrancea outbreak, Romania, is also applied, but the building does not stand in this area and such a spectrum should not be considered.



При $0 \le T \le T_B$:

$S_e(T) = a_g.S.[1 + T/T_B.(\eta.2,5 - 1)]$

При $T_B \le T \le T_C$:

$S_e(T) = a_g.S.\ \eta.2,5$

При $T_C \le T \le T_D$:

$S_e(T) = a_g.S.\ \eta.2,5.[T_C/T_B]$

При $T_D \le T \le 4_S$:

$S_e(T) = a_g.S.\ \eta.2,5.[T_C.\ T_D/T^2]$

$S_e(T)$ – elastic spectrum;

T – is the vibration period of a linear single-degree-of-freedom system;

a_g – is the design ground acceleration on type A ground;

T_B, T_C и T_D – determines limits of the period of the constant spectral acceleration branch;

S – is the soil factor;

$\eta = 1,0$ – is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping;

The values of the periods and the soil factor S, T_B, T_C и T_D, describing the shape of the elastic response spectrum are given in NA.3.2. of БДС EN 1998 – 1.

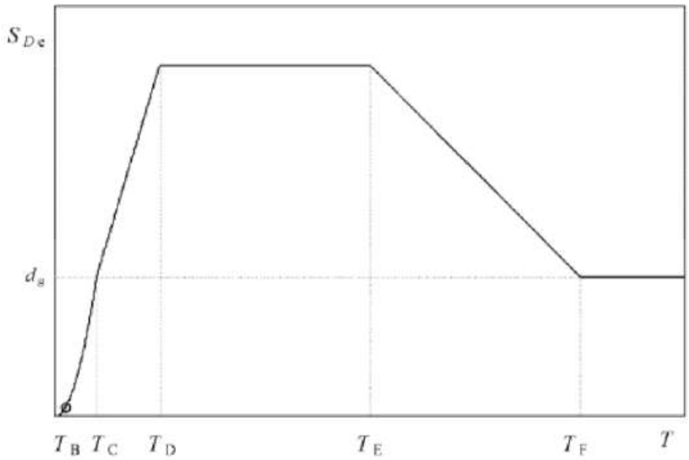
Тип земна основа	S	T_B(s)	T_C(s)	T_D(s)
A	1,00	0,10	0,3	2
B	1,3	0,10	0,4	2
C	1,2	0,10	0,5	2
D	1	0,10	0,6	2
E	1,2	0,10	0,5	2

S = 1,2

$T_B = 0,1\ s$

$T_C = 0,5\ s$

$T_D = 2,0\ s$



For structures with long vibration period the seismic action shall be represented by the expression for $S_{De}(T)$ according to “A” of БДС EN 1998 – 1.

Before control period T_E is reached:

$S_{De}(T) = S_e(T).[T/2\pi]^2$

For soil type „C“ control period has the value $T_E = 6,0\ s$.

Тип земна основа	T_E (s)	T_F (s)
A	4,5	10,0
B	5,0	10,0
C	6,0	10,0
D	6,0	10,0
E	6,0	10,0

→ При $T_D \leq T = 4{,}16\ s\ \leq 6{,}0\ s$:

$S_{De}(T = 4{,}16\ s) = [T/2\pi]^2 .a_g.S.\ \eta.2{,}5.[T_C.\ T_D/T^2] = [4{,}16/2\pi]^2 .0{,}144.1{,}2.\ 1{,}0.2{,}5.[0{,}5.\ 2{,}0/4{,}16^2]$

$S_{De}(T = 4{,}16\ s) = 0{,}0109$

1.9. <u>Design horizontal response spectrum</u>

According to БДС EN 1998 - 1, item 3.2.2.5, the ability of structures to withstand seismic impact in a nonlinear region allows their design for lower seismic forces than those corresponding to their linear - elastic response. In order to avoid non-linear design analysis, the ability of the structure to dissipate energy through ductile behavior of its elements is taken into account by means of a linear analysis based on a reduced elastic response spectrum called a "designed spectrum". The reduction is achieved by introducing the behavior factor ***q = 3,2***.

При $T_D \leq T$:

$S_e(T) = a_g.S.\ \eta.2{,}5/q.[T_C.\ T_D/T^2] \geq \beta.a_g$

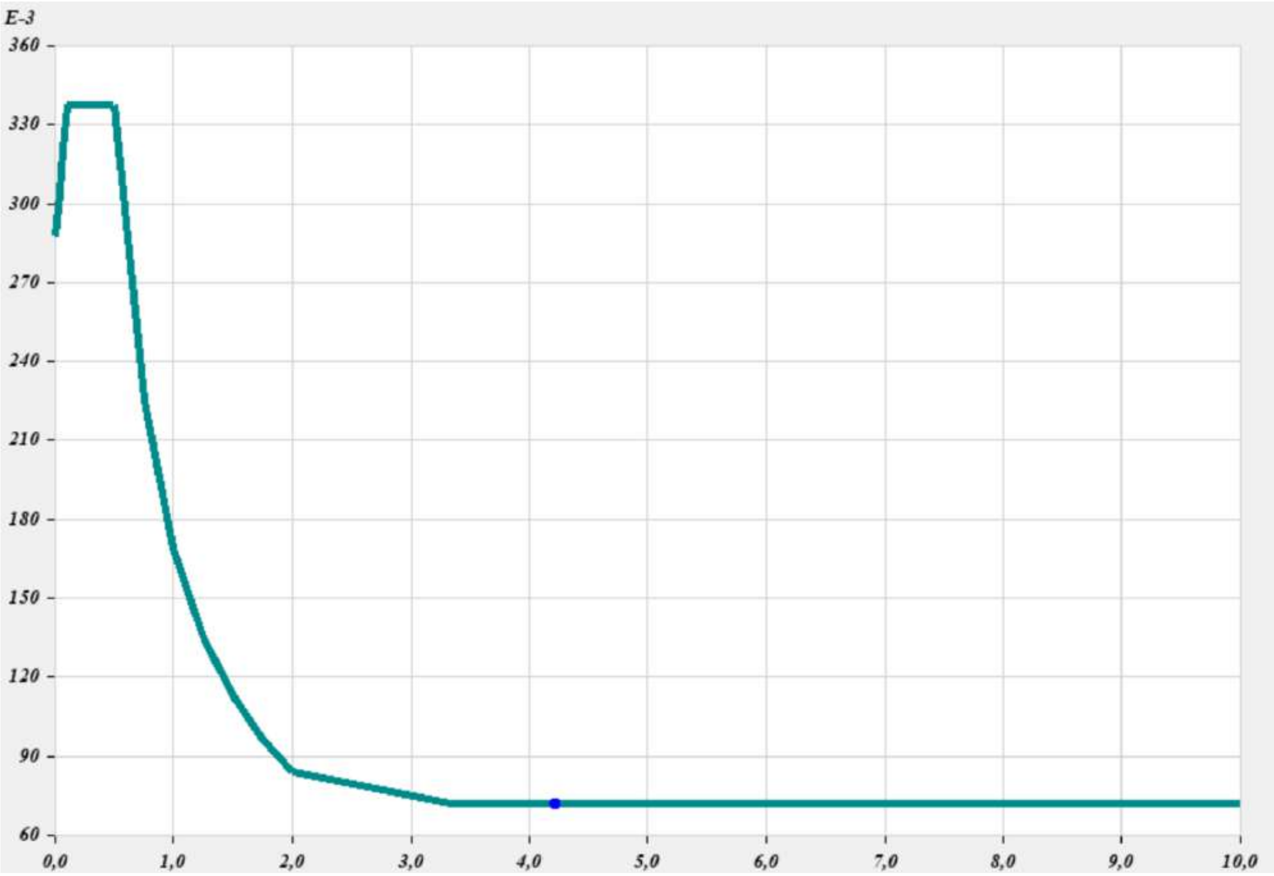
Където:

q = 3,2 – behavior factor ;

$\beta = 0{,}2$

$S_e(T = 4{,}16\ s) = a_g.S.\ \eta.2{,}5.[T_C.\ T_D/T^2] = 0{,}36.1{,}2.\ 1{,}0.2{,}5/3{,}2.[0{,}5.\ 2{,}0/4{,}16^2] = 0{,}0195 \geq 0{,}2.0{,}36$

$S_e(T = 4{,}16\ s) = 0{,}0195 < 0{,}072 \rightarrow$ **S_e(T = 4, 16 s) = 0,072**

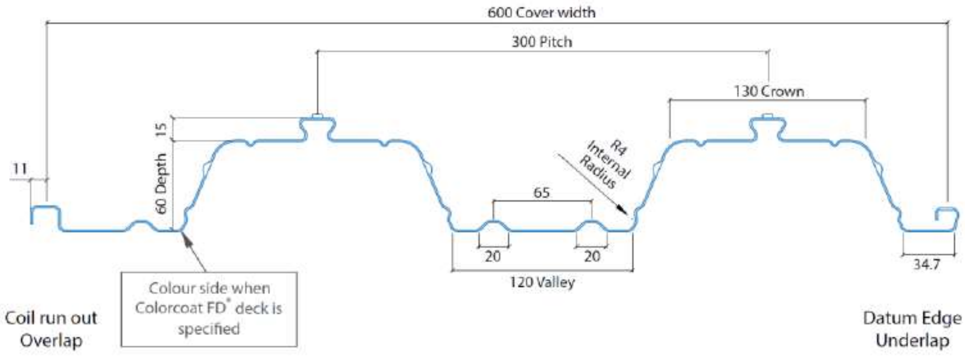


III. Design of the floor structure

1. Floor slab

The floor slab is calculated as a composite slab. Two stages of work are considered. In the first stage - the concrete placing stage - the profiled steel decking is used as a remaining formwork and works independently, requiring to withstand the load of fresh concrete mix and construction loads. During the second stage of work (operational) a composite slab - steel decking and concrete work together -the concrete has reached the design strength. At this stage, all operational impacts are applied - dead and live loads.

It's been chosen the ComFlor 60 steel decking – a TATA Steel product. The steel decking meets the requirements for the implementation of composite floor constructions. Sufficient space is provided for the installation of shear studs



For slab thickness is chosen: $h_{пл} = 140mm$

1.10. Loads

1.10.1. First stage – concrete placing stage

➢ *Dead loads*

- Steel decking load: $g_{LT} = 0,11\ kN/m^2$
- Load of the fresh concrete:
  Taken from ComFlor 60 catalog: $g_{c,wet} = 2,71\ kN/m^2$
- Reinforcement load:
  Load for A393: $g_r = 0,03\ kN/m^2$

$$\boldsymbol{g_{tot} = 2,82\ kN/m^2}$$

➢ *Live loads*

- Work zone load (3,0x3,0 m): $q = 1,5\ kN/m^2$
- Loads, out of the work zone: $q' = 0,75\ kN/m^2$

1.10.2. Second stage – composite slab

➢ *Dead loads*

- Steel decking load: $g_{LT} = 0,11\ kN/m^2$
- Concrete load: $g_{c,dry} = 2,60\ kN/m^2$
- Reinforcement load: $g_r = 0,03\ kN/m^2$
- Paste load: $g_c = t_c.\gamma_c = 0,03.12 \rightarrow g_c = 0,36\ kN/m^2$
- Flooring load: $g_f = t_f.\gamma_f = 0,02.25 \rightarrow g_f = 0,50\ kN/m^2$
- Ceiling: $g_s = 0,2\ kN/m^2$

- Installations: $g_i = 0{,}5\ kN/m^2$
- Walls: $g_w = 1{,}0\ kN/m^2$

$\boldsymbol{g_{tot} = 5{,}30\ kN/m^2}$

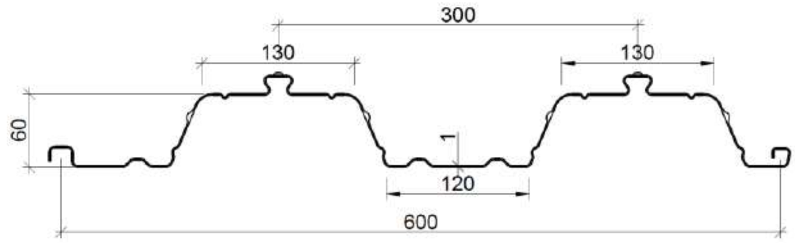
➢ *Live loads*

- Live loads for rooms category "C" and "D", according to БДС EN1991-1-1, т. 6.3.1.2.: $\boldsymbol{q = 5{,}0\ kN/m^2}$

1.11. <u>Design in first stage – concrete placing</u>

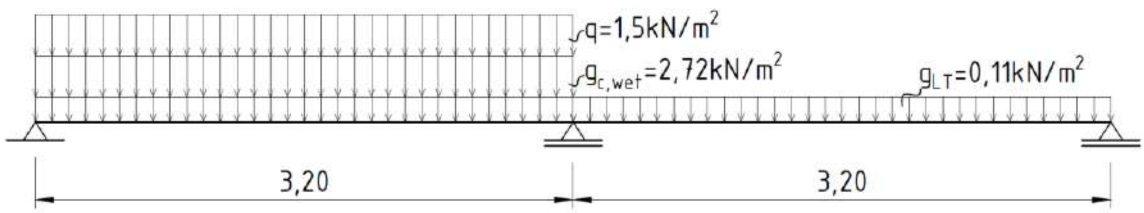
1.11.1. Static scheme

The static scheme is „continuous beam of two spans". Calculations are made with width of *1m = 1000mm.* Thickness of the steel decking is chosen *1mm*. Geometry of the steel section of the decking is shown:

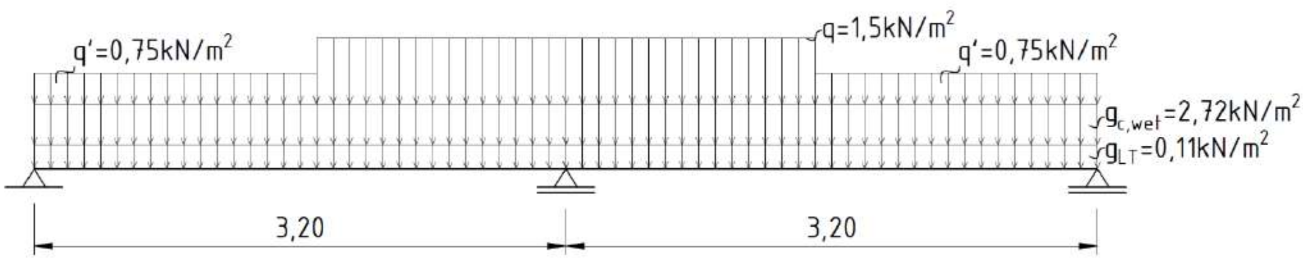


1.11.2. Load cases

1) For maximal span bending:



2) Form maximal support moment and maximal force in the support:



1.11.3. Load combinations

➢ *SLS (ЕГС)*

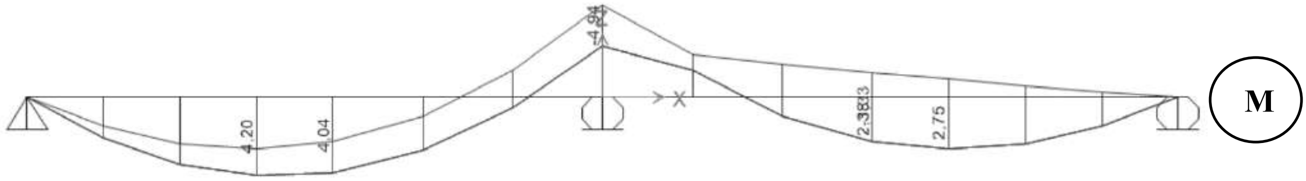
γ_G."G" + γ_Q."Q" = 1,00."G" + 1,00."Q"

➢ *ULS (КГС)*

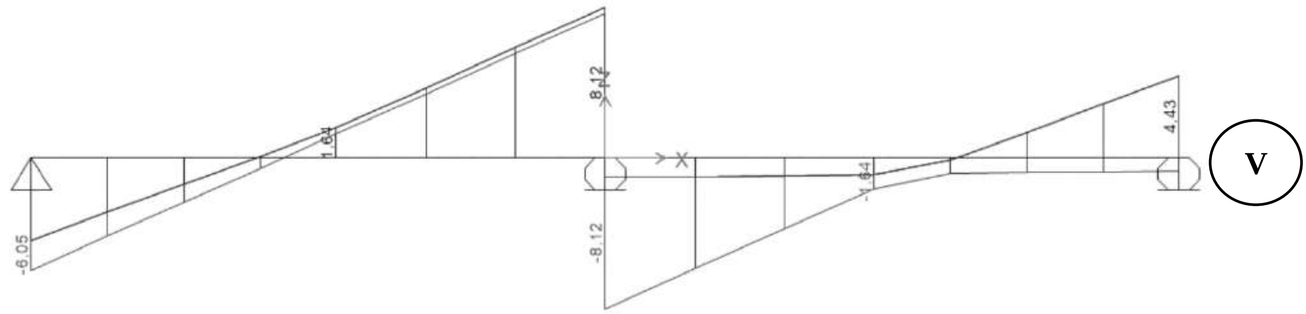
γ_G."G" + γ_Q."Q" = 1,35."G" + 1,50."Q"

1.11.4. Envelope diagrams of the moments and forces

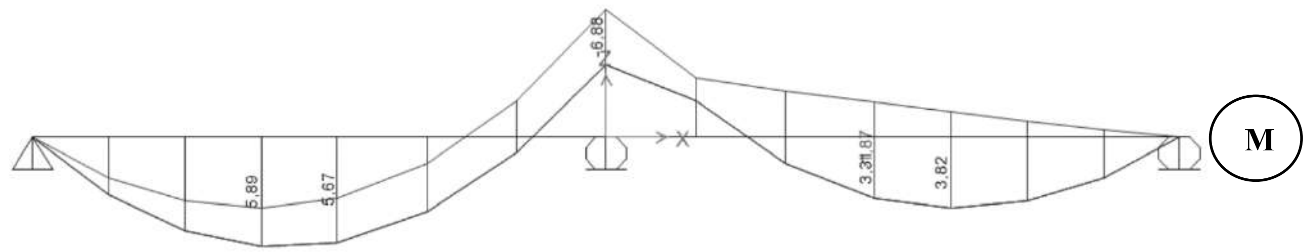
- *SLS – M_{Ed}*



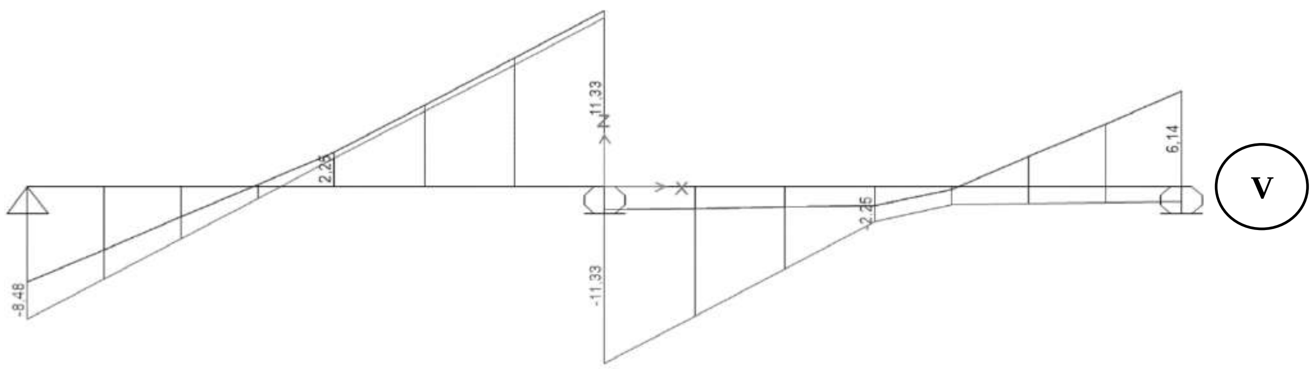
- *SLS – V_{Ed}*



- *ULS – M_{Ed}*



- *ULS – V_{Ed}*



1.11.5. Bending resistance (ULS)

- *Positive sign moment (M_{Ed} = 589 kNcm):*

$M_{Ed} \leq M_{c,Rd}$

Section class 3: $M_{c,Rd} = W_{el}.f_{ya}/\gamma_{M0}$

$f_{ya} = f_{yb} = 350\ MPa = 35\ kN/cm^2$

$W_{el} = I_y/z_{top} = 106{,}15/(6{,}0 - 3{,}375) \rightarrow \mathbf{W_{el} = 40{,}44\ cm^3}$

$M_{c,Rd} = 40{,}44.35/1{,}05 \rightarrow \mathbf{M_{c,Rd} = 1347{,}9\ kN/cm^2 > M_{Ed} = 589\ kN/cm^2}$

Satisfied!

- *Negative sign moment ($M_{Ed} = 688\ kNcm$):*

$M_{Ed} \leq M_{c,Rd}$

Section class 3: $M_{c,Rd} = W_{el}.f_{ya}/\gamma_{M0}$

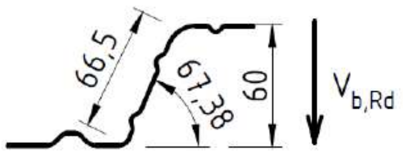
$f_{ya} = f_{yb} = 350\ MPa = 35\ kN/cm^2$

$W_{el} = I_y/z_{bot} = 106{,}15/3{,}375 \rightarrow \mathbf{W_{el} = 31{,}45\ cm^3}$

$M_{c,Rd} = 31{,}45.35/1{,}05 \rightarrow \mathbf{M_{c,Rd} = 1048\ kN/cm^2 > M_{Ed} = 688\ kN/cm^2}$

Satisfied!

1.11.6. Shear resistance (ULS)



$V_{b,Rd} = \frac{h_w}{sin\varphi}.t.f_{by}/\gamma_{M0}$ – Shear resistance of one rib of the steel decking;

$f_{bv} = f\,(\overline{\lambda_w})$

For ribs with longitudinal stiffeners: $\overline{\lambda_w} = 0{,}346.\frac{s_d}{t}.\sqrt{\frac{5{,}34.f_{yb}}{k_\tau.E}}$, where:

$s_d = 66{,}5\ mm = 6{,}65\ cm$

$k_\tau = 5{,}34$

$\overline{\lambda_w} = 0{,}346.\frac{66{,}5}{0{,}96}.\sqrt{\frac{5{,}34.35}{5{,}34.21\ 000}} = 0{,}978 \rightarrow 0{,}83 < \overline{\lambda_w} = 0{,}978 < 1{,}40 \rightarrow f_{bv} = 0{,}48.f_{yb}/\overline{\lambda_w}$

$f_{bv} = 0{,}48.35/0{,}978 \rightarrow \mathbf{f_{bv} = 17{,}17\ kN/cm^2}$

$V_{b,Rd} = \frac{60}{sin67{,}38}.0{,}96.17{,}17/1{,}05 \rightarrow \mathbf{V_{b,Rd} = 10{,}2\ kN}$

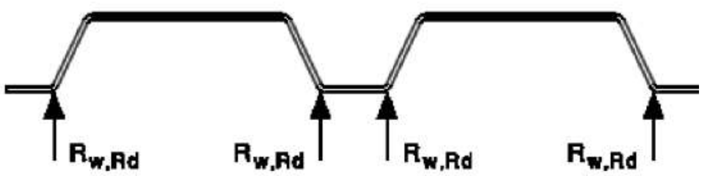
$V_{Ed} \leq n_w.V_{b,Rd}$

$n_w = 2x1000/300 = 6{,}67$ *бр. стебла за 1,0 m*

$V_{Ed} \leq n_w.V_{b,Rd} = 6{,}67.10{,}2 = 68\ kN \rightarrow \mathbf{V_{Ed} = 11{,}33\ kN < V_{Rd} = 68\ kN}$

Проверката е удовлетворена!

1.11.7. Local buckling resistance – according to БДС EN1993-1-3, т. 6.1.7.3.



$F_{Ed} \leq n_w.R_{w,Rd}$

Requirements:

$r/t \leq 10 \rightarrow 4/0{,}96 = 4{,}167 < 10$

$h_w/t \leq 200.sin\varphi \rightarrow$

60/0,96 = 63,82 < 200.sin67,38 = 184,6

$45^\circ \leq \varphi \leq 90^\circ \rightarrow 45^\circ \leq 67,38 \leq 90^\circ$

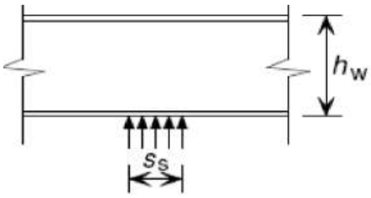
The requirements are satisfied!

$\rightarrow R_{w,Rd} = \alpha.t^2.\sqrt{f_{yb}.E}.\left(1 - 0{,}1.\sqrt{\frac{r}{t}}\right).\left[0{,}5 + \sqrt{0{,}02.\frac{l_a}{t}}\right].\left(2{,}4 + \left(\frac{\varphi}{90}\right)^2\right)$, където:

l_a – length of the support print, according to the category;

α – coefficient;

- *Category(2)*



$\alpha = 0{,}15$

$l_a = f(\beta_v)$; $\beta_v = \frac{|V_{Ed,1}|-|V_{Ed,2}|}{|V_{Ed,1}|+|V_{Ed,2}|}$; $|V_{Ed,1}|$ *u* $|V_{Ed,2}|$ - absolute values of the shear forces on both sides of the local load;

The local load refers to the support reaction, transmitted by the load print. The size s_s of the load print refers to the flange width of the secondary beam. Section chosen: IPE 200 → s_s = 100 mm

$|V_{Ed,1}| = 10{,}34\ kN$; $|V_{Ed,2}| = 10{,}52\ kN \rightarrow \beta_v = \frac{10{,}52-10{,}34}{10{,}52+10{,}34} = 0{,}009 < 0{,}2 \rightarrow l_a = s_s = 100\ mm$

$R_{w,Rd} = 0{,}15.0{,}096^2.\sqrt{35.21000}.\left(1 - 0{,}1.\sqrt{\frac{0{,}4}{0{,}096}}\right).\left[0{,}5 + \sqrt{0{,}02.\frac{10}{0{,}096}}\right].\left(2{,}4 + \left(\frac{67{,}38}{90}\right)^2\right)$

$\mathbf{R_{w,Rd} = 5{,}61\ kN}$ ***;*** $\mathbf{n_w = 6{,}67}$ ***;*** $F_{Ed} = 22{,}7\ kN \rightarrow 22{,}7\ kN < 6{,}67.5{,}61 = 37{,}5\ kN$

→22,7 kN < 37,5 kN

The requirements are satisfied!

1.11.8. Interaction of the inner support forces check

- *Combination of shear force and bending moment*

$V_{Ed} \leq 0{,}5.n_w.V_{b,Rd} = 0{,}5.6{,}67.10{,}2 = 34\ kN \rightarrow$ $\mathbf{V_{Ed} = 11{,}33\ kN < 34\ kN}$ ***→ interaction between*** $\mathbf{M_{Ed}}$ ***u*** $\mathbf{V_{Ed}}$ ***is not taken into account!***

- *Combination of support force and bending moment*

$\frac{M_{Ed}}{M_{Rd}} + \frac{F_{Ed}}{n_w.F_{Rd}} \leq 1{,}25 \rightarrow \frac{688}{1051} + \frac{22{,}7}{6{,}67.5{,}61} = 1{,}25 = 1{,}25$

The requirement is satisfied!

1.11.9. Calculation of the deflection of the steel decking during first stage

$\delta = \frac{2{,}13}{384}.\frac{g.L^4}{E.I_{eff}} = \frac{2{,}13}{384}.\frac{g.L^4}{E.0{,}78.I_{el}} = \frac{2{,}13}{384}.\frac{2{,}82.0{,}01.320^4}{21\,000.0{,}78.106{,}15} = 0{,}94\ cm = 9{,}4\ mm$

$\delta_{max} \leq L/180 = 3200/180 = 17{,}78\ mm \rightarrow$ $\mathbf{\delta_{max} = 9{,}4\ mm < 17{,}78\ mm}$

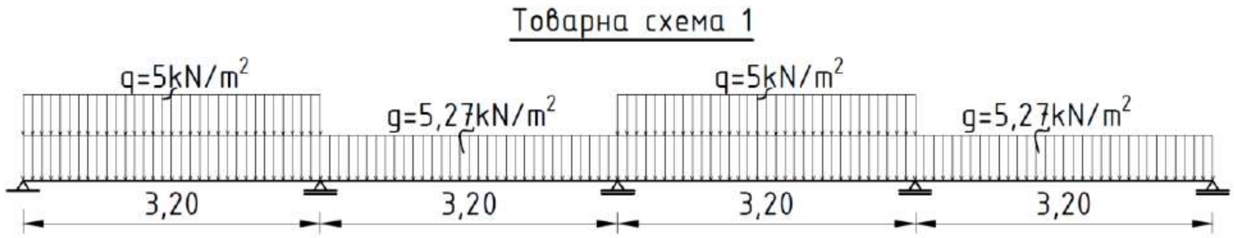
Check: $\delta \leq 1/10.(h_p + h_c) = 1/10.(60+80) = 14\ mm \rightarrow$ $\mathbf{\delta = 9{,}4\ mm < 14\ mm}$

→ "Slope effect" is not taken into account!

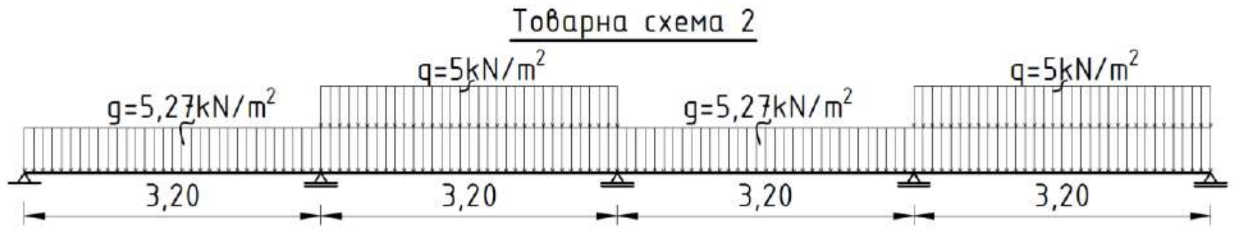
1.12. Design in second stage – composite slab – Calculations for static scheme "continuous beam on four spans"

1.12.1. Load schemes

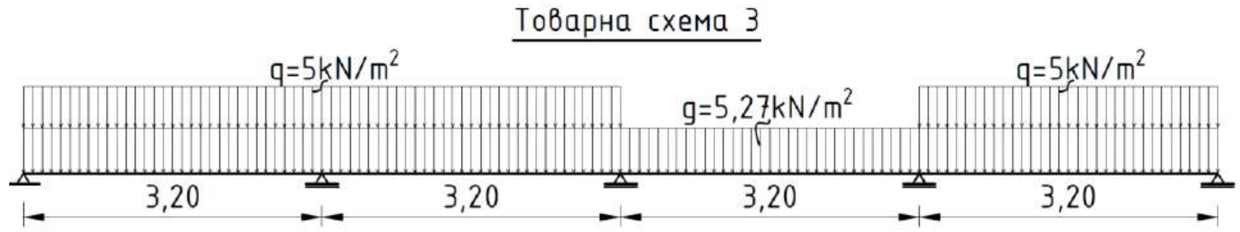
- *Load scheme 1 – $M_{поле\ 1,max}$ и $M_{поле\ 3,max}$*



- *Load scheme 2 – $M_{поле\ 2,max}$ и $M_{поле\ 4,max}$*

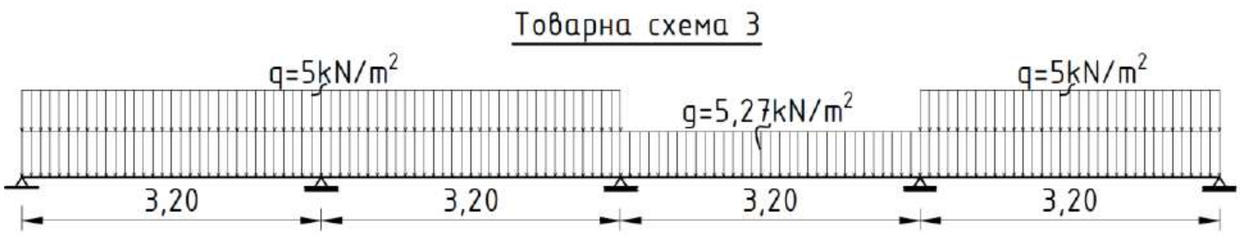


- *Load scheme 3 – $M_{on\ 1,max}$ и $R_{on\ 1,max}$*



- *Load scheme 4 – $M_{on\ 2,max}$ и $R_{on\ 2,max}$*

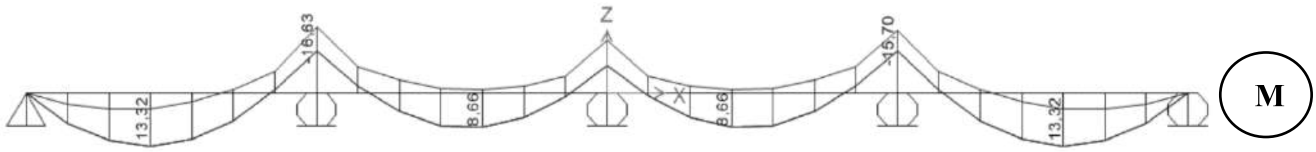
- *Load scheme 5 – full load*



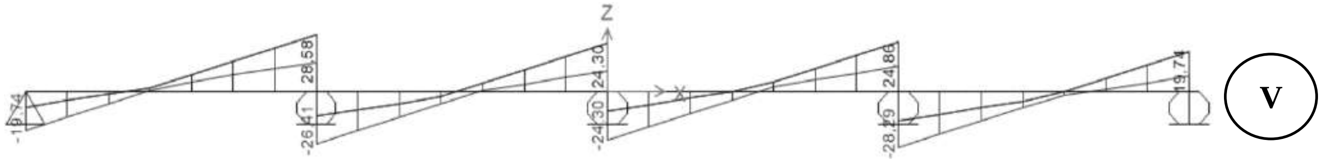
Load combination: γ_G."G" + γ_Q."Q" = 1,35."G" + 1,50."Q"

1.12.2. Analysis results

- *Envelope diagram of the bending moment*



- *Envelope diagram of the shear force*

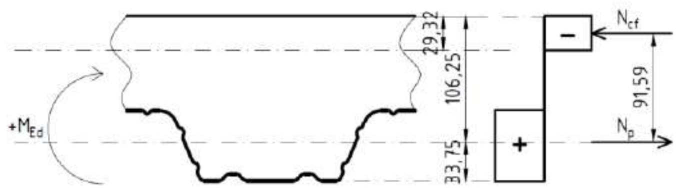


1.12.3. Materials used

- *Concrete class C30/37*
  - f_{ck} = 30 MPa = 3 kN/cm^2
  - f_{cd} = 20 MPa = 2 kN/cm^2
  - f_{ctm} = 2,9 MPa = 0,29 kN/cm^2
  - E_{cm} = 32 800 MPa = 3 280 kN/cm^2
  - γ_c = 1,5
- *Reinforcement B500B*
  - f_{yk} = 500 MPa = 50 kN/cm^2
  - f_{yd} = 435 MPa = 43,5 kN/cm^2
  - E = 210 000 MPa = 21 000 kN/cm^2
  - γ_c = 1,15

1.12.4. Bending resistance of the composite slab when *+M is applied*

- *Determination of the position of the zero line*



$N_p = N_{cf}$; $N_p = f_{yp}.A_p$; $N_{cf} = x.b.f_{cd}$

A_p = 1424 mm^2 ; A_c = 108 000 mm^2

$N_p = N_{cf} \rightarrow f_{yp}.A_p = x.b.f_{cd} \rightarrow x = f_{yp}.A_p /(b.f_{cd})$ = 350.1424/(0,85.1000.20)

→ *x* = 29,32 mm

→Zero line is located inside the slab!

- *Bending resistance*

$M_{pl,Rd} = N_p.z$; z = 160 – 33,75 – 29,32/2 → z = 106,25 mm

$M_{pl,Rd} = A_p.f_{yp}.z$ = 1424.350.106,25 → $\mathbf{M_{pl,Rd}}$ **= 52,95 kNm**

$\mathbf{M_{Ed}^+}$ **= 13,32 kNm <** $\mathbf{M_{pl,Rd}^+}$ **= 41,55 kNm** ***→ The requirement is satisfied!***

Minimum reinforcing coefficient is accepted for bottom reinforcement: ρ = 0,0013

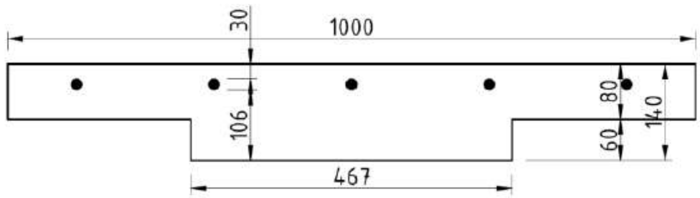
$A_s = \rho.A_c$ = 0,0013.108 000 → A_s = 140,4 mm^2

Accepted reinforcement: ***3N8****/333 mm* ; $A_{s,prov}$ = 150,8 mm^2 *→ In every rib of the steel decking is placed one reinforcing bar N8!*

The reinforcement is provided in case the profiled steel decking excludes (in case of fire) or its load-bearing capacity is reduced (in case of corrosion).

1.12.5. Bending resistance of the composite slab when *-M is applied*

The combined section is reduced to an equivalent slab section with equal area. The contribution of profiled steel decking, located in the pressure zone, is neglected to increase safety. Calculating the width of the beam: $b_c = (A_c – h_c.b)/h_p = (108\,000 – 80.1000)/60 \rightarrow b_c = 467$ mm



$d = h_{пл} – c_{nom} – 0,5.ф$

$d = 140 – 30 – 0,5.8 \rightarrow$ **d = 106 mm**

<u>Design procedure:</u>

1) *Relative bending moment:*

$$m_{Ed} = \frac{M_{Ed}}{b.d^2.f_{cd}} = \frac{16,63.10^6}{467.106^2.20} = 0,158$$

Where $b = 467$ mm is the width of the slab section;

2) *Check of the condition to prevent fragile destruction of concrete (x < 0,45d):*

$m_{Ed} \leq m_{Ed,lim} = 0,2952 \rightarrow 0,158 < 0,2952 \rightarrow$ *Satisfied!*

3) *For* $m_{Ed} = 0,158 \rightarrow$ $\boldsymbol{\zeta = 0,9243}$
4) *Required reinforcement:*

$$\boldsymbol{A_{s,req}} = \frac{M_{Ed}}{\zeta.d.f_{yd}} = \frac{16,63.10^6}{0,9243.106.435} = \mathbf{390,2\ mm^2/m}$$

5) *Minimum reinforcement:*

$$A_{s,min} = 0,26.\frac{f_{ctm}}{f_{yk}}.b.d \geq 0,002.b.d,$$

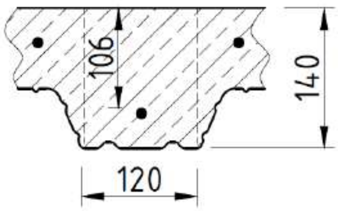
$f_{ctm} = 2,9$ MPa

$$\rightarrow 0,26.\frac{2,9}{435}.467.106 = 0,0017.467.106 = 85,80\ mm^2 < 0,002.467.106$$

$\boldsymbol{A_{s,min} = 99,00\ mm^2/m}$

6) $A_{s1} = \max(A_{s,req}; A_{s,min}) \rightarrow A_{s1} = \max(390,2; 99,00) \rightarrow \boldsymbol{A_{s1} = 390,2\ mm^2/m}$
7) *Defined rinforcement:* **5N10/s = 200 mm** $\rightarrow \boldsymbol{A_{s,prov} = 393\ mm^2/m}$
8) *Reinforcing coefficient:* $\rho = A_{,prov}/(b.d) = 393/(10000.106) \rightarrow \rho = 0,0037 > \rho_{min} = 0,002$

1.12.6. Vertical shear resistance of the composite slab



The contribution of profiled steel decking is neglected. The design resistance of the slab section (width of the section is equal to the space between ribs of the steel decking) is determined:

Resistencia is determined according to

БДС EN1992-1-1, т. 6.2.2.

$V_{ed} \leq n.V_{Rd,c}$

$V_{Rd,c} = C_{Rd,c}.k.(100.\rho_1.f_{ck})^{1/3}.b.d \geq min\ V_{Rd,c} = v_{min}.b.d\ [N/m],$

където:

$C_{Rd,c} = \frac{0{,}18}{\gamma_c} = \frac{0{,}18}{1{,}5} = 0{,}12;$

$k = 1 + \sqrt{\frac{200}{d[mm]}} \leq 2{,}0 \rightarrow k = 1 + \sqrt{\frac{200}{106}} = 2{,}37 > 2{,}0 \rightarrow k = 2{,}0$

$\rho_l \leq 0{,}02 \rightarrow \rho_l = 0{,}0037 + 50{,}26/(120.106) = 0{,}0076$

$f_{ck} = 30\ N/mm^2$

$v_{min} = 0{,}035.k^{3/2}.f_{ck}^{1/2} = 0{,}035.2^{3/2}.30^{1/2} = 0{,}542\ MPa$

$b = 120\ mm$

$v_{Rd,c} = 0{,}12.2{,}0.(100.0{,}0055.30)^{1/3} \rightarrow \boldsymbol{v_{Rd,c} = 0{,}680\ MPa > v_{min} = 0{,}542\ MPa}$

$V_{Rd,c} = v_{Rd,c}.b.d = 0{,}680.120.106 \rightarrow$ ***$V_{Rd,c}$ = 8,656 kN/1 ребро***

$n = b/b_n = 1000/300 = 3{,}33$

$V_{Ed} \leq n.V_{Rd,c} = 3{,}33.8{,}66 = 28{,}85\ kN \rightarrow \boldsymbol{V_{Ed} = 28{,}58\ kN < V_{Rd,c} = 28{,}85\ kN}$

Requirement is satisfied!

1.12.7. Horizontal (longitudinal) shear check

Composite slab is considered as a slab with “fragile behavior”. “m-k” method is used for calculating the horizontal (longitudinal) shear resistance

Check is made without taking into consideration the anchoring in the ends, according to

БДС EN1994-1-1, т. 9.7.3.

$V_{L,Rd} = \frac{b.d_p}{\gamma_{vs}}.\left(\frac{m.A_p}{b.l_s} + k\right)$, where:

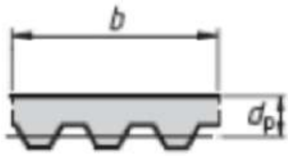
$l_s = L/4 = 3{,}20/4 \rightarrow l_s = 0{,}80\ m \rightarrow$ sliding length for slabs, loaded with distributed load;

$A_p = 1424\ mm^2$ – nominal section of the steel decking;

m и k – design values of empiric coefficients, determined by testing the slab meeting the requirements of the *“m-k”* method. Values are given by the manufacturer.

$m = 178{,}39\ MPa;\quad k = 0{,}1\ MPa$

$\gamma_{vs} = 1{,}25$ – ULS safety coefficient;



$V_{L,Rd} = \frac{1000.100}{1{,}25}.\left(\frac{178{,}39.1424}{1000.800} + 0{,}1\right) = 33\ 402{,}7\ N \rightarrow$ ***$V_{L,Rd}$ = 33,4 kN***

V_{Ed} = 28,58 kN < $V_{L,Rd}$ = 33,4 kN → The requirement is satisfied!

2. Secondary beam SB8

The secondary beams have a static scheme “simple beam”. They receive load from the slab and distribute it by supporting reactions to the main beams. The beams are considered to be composite and the connection between the plate and the beam is realized thanks to welded shear studs, resisting the sliding (horizontal shear) forces and resisting the shear force.

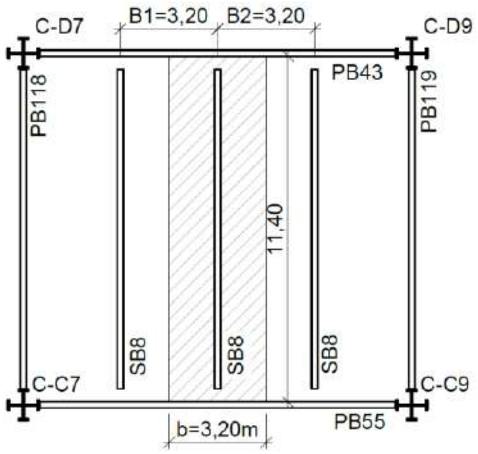
Secondary beams *SB8 are taken into consideration.* Their length is $l = 11,60$ *m*, and the space in-between is $B = 3,20$ *m*.

There are two stages of work considered, similar to the calculation of the composite slab. First stage – concrete placing stage, and Second stage – composite slab is resisting the loads (operational mode).

2.1. Analyzing beam in first stage

2.1.1. Defining the loads, affecting the beam

It is considered an area load, influencing the beam. The size of the area load is calculated: $(b = 1/2.(B_1+B_2) = 1/2.(3,20+3,20) \rightarrow \mathbf{b = 3,20\ m})$. Area load is approximated to a linear distributed load.



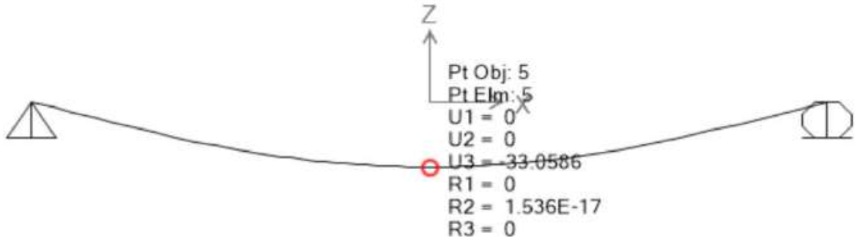
- *Dead loads*
  - Steel decking: $g_{LT} = 0,11\ kN/m^2 \rightarrow g'_{LT} = b.\ g_{LT}$
    $g'_{LT} = 3,20.\ 0,11 \rightarrow \mathbf{g'_{LT} = 0,352\ kN/m}$
  - Fresh concrete: $g_{c,wet} = 2,71\ kN/m^2 \rightarrow$
    $g'_{c,wet} = b.g_{c,wet} = 3,20.2,71 \rightarrow \mathbf{g'_{c,wet} = 8,672\ kN/m}$
    $\mathbf{\sum g = g = 9,024\ kN/m}$
- *Live loads*
  - Loading in the work area (3,0x3,0 m): $q = 1,5\ kN/m^2 \rightarrow$
    $q' = b.q = 3,20.1,5 \rightarrow \mathbf{q' = 4,80\ kN/m}$
  - Loading out of the work zone: $q' = 0,75\ kN/m^2 \rightarrow$
    $q'' = b.q' = 3,20.0,75 \rightarrow \mathbf{q'' = 2,40\ kN/m}$
- *Design values of the loads*
  - $g_d = \gamma_G.g = 1,35.9,024 \rightarrow \mathbf{g_d = 12,18\ kN/m}$
  - $q'_d = \gamma_q.q' = 1,50.4,80 \rightarrow \mathbf{q'_d = 7,20\ kN/m}$
  - $q''_d = \gamma_q.q'' = 1,50.2,40 \rightarrow \mathbf{q''_d = 3,60\ kN/m}$

2.1.2. Pre-selection of the beam section by satisfying the deflection requirements

The check is made with the characteristic values for SLS combination.

Selected section: ***IPE 500***, g_{IPE} = *0,776 kN/m*

Verifying if the vertical deflection in the middle of the span is less than the limit deflection.

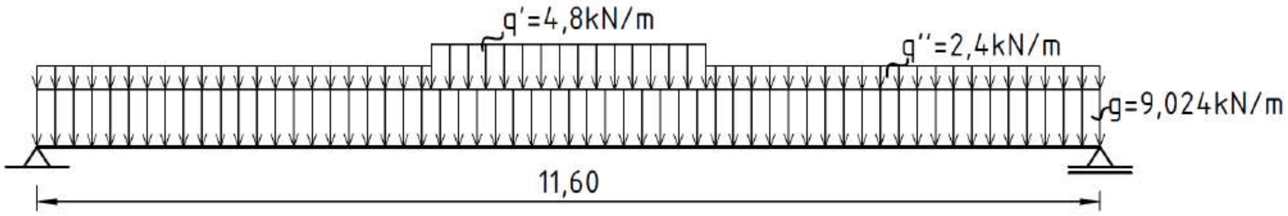


$\delta_{max} \leq L/250 = 11\,600/250 = 46{,}4$ mm $\rightarrow$ $\boldsymbol{\delta_{max}}$ **= 33,1 mm < 46,4 mm**

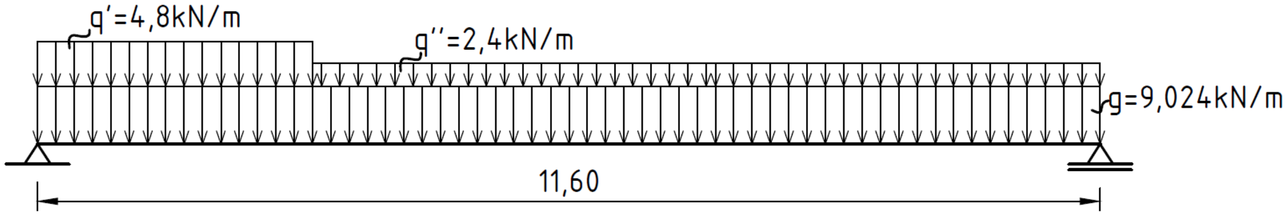
The deflection satisfies the requirements of the limit state! The ULS requirements would be checked for a beam section IPE500!

2.1.3. Load schemes

- *For maximal bending moment in the middle of the span*



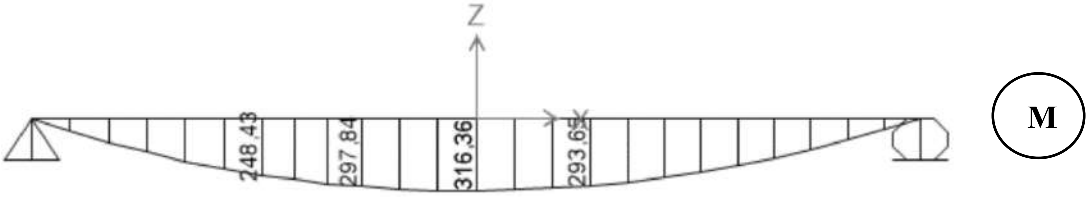
- *For a maximal support reaction and maximum shear force*



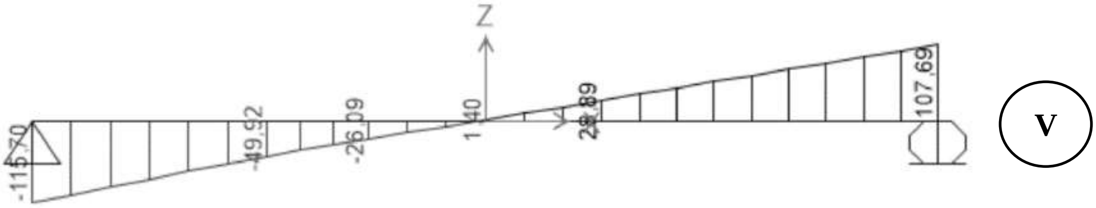
Note: *Schemes are showing the characteristic values of the loads!*

2.1.4. Moment and forces diagrams

- *Design values of bending moment*



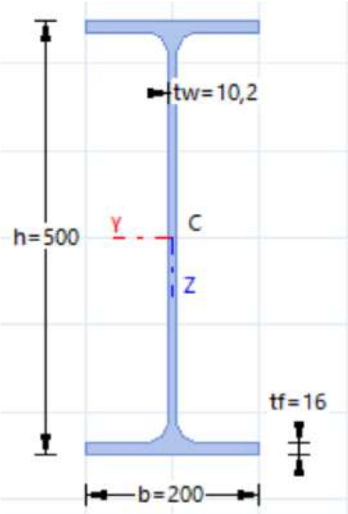
- *Design values of shear force*



2.1.5. Defining the steel section class

Steel class selected: S275 → $\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{275}} = 0{,}924$

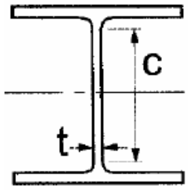
- *Steel cross section characteristics*



h =	500,0	b2=	200,0	b =	200,0	ri=	21
tw=	10,2	tf2=	16,0	tf=	16,0	ro=	0

Инерционни характеристики

A = 115.522 cm	
Cz = 25.000 cm	Cy = 10.000 cm
Iy = 48198.535 cm4	Iz = 2141.688 cm4
Wy = 1927.941 cm3	Wz = 214.169 cm3
Wply= 2194.118 cm3	Wplz= 335.879 cm3
ry = 20.426 cm	rz = 4.306 cm
Avy = 64.000 cm2	Avz = 59.874 cm2
It = 89.441 cm4	Wt = 41.668 cm3

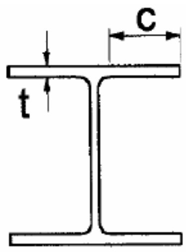


- *Slenderness of the web*

$\lambda = c/t = (h - 2.t_f - 2.r_i)/t_w = (500 - 2.16 - 2.21)/10{,}2 = 426/10{,}2$

$\rightarrow \lambda = 41{,}8$

$\boldsymbol{\lambda = 41{,}8 < 72.\varepsilon} = 72.0{,}924 = 66{,}5 \rightarrow$ *Cross section class 1*



- *Slenderness of the flanges*

$\lambda = c/t = (b/2 - t_w/2 - r_i)/t_f = (200/2 - 10{,}2/2 - 21)/16 = 73{,}9/16$

$\rightarrow \lambda = 4{,}61$

$\boldsymbol{\lambda = 4{,}61 < 9.\varepsilon} = 9.0{,}924 = 8{,}32 \rightarrow$ *Cross section class 1*

→ The cross section is classified as class 1!

2.1.6. Resistance of the cross-section

In order to increase safety checks are made in linear (elastic) state (elastic distribution of strains across the cross-section is considered)!

- *Bending resistance*

$M_{y,el,Rd} = \frac{W_{y,el} \cdot f_y}{\gamma_{M0}} = \frac{1928.27{,}5}{1{,}05} = 50\ 495\ kN.cm \rightarrow \boldsymbol{M_{y,el,Rd} = 504{,}9\ kNm}$

$\frac{M_{y,Ed}}{M_{y,el,Rd}} \leq 1{,}0 \rightarrow \frac{316{,}4}{504{,}9} = 0{,}63 < 1{,}0 \rightarrow$ ***Requirement is satisfied!***

➢ *Shear resistance*

$V_{el,Rd} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{59,87.27,5}{\sqrt{3}.1,05}$ → ***$V_{el,Rd}$ = 905,3 kN***

$\frac{V_{z,Ed}}{V_{el,Rd}} \leq 1,0 \rightarrow \frac{115,7}{905,3} = 0,13 < 1,0$ → ***Requirement is satisfied!***

2.1.7. Buckling resistance

➢ Check for stabilized upper flange

Check if the upper flange is stabilized by the steel decking, whose stiffeners are perpendicular to the axis of the beam. This circumstance improves the buckling resistance of the beam. Requirements according to БДС EN1993-1-3, т. 10.1:

$S \geq \left(EI_w.\frac{\pi^2}{L^2} + GI_t + EI_z.\frac{\pi^2}{L^2}.0,25.h^2\right).\frac{70}{h^2}$, where:

$S = 1000.\sqrt{t^3}.\left(50 + 10.\sqrt[3]{b_{roof}}\right).\frac{s}{h_w}$ *[N]* – shear stiffness, provided from the steel decking;

$S = 1000.\sqrt{0,96^3}.\left(50 + 10.\sqrt[3]{44\,800}\right).\frac{3200}{60} = 20\,325\,154\ N$

$\left(EI_w.\frac{\pi^2}{L^2} + GI_t + EI_z.\frac{\pi^2}{L^2}.0,25.h^2\right).\frac{70}{h^2} = \left(210\,000.1\,249\,000.10^6.\frac{\pi^2}{11600^2} + 80\,769.89,29.10^4 + 210\,000.2142.10^4.\frac{\pi^2}{11600^2}.0,25.500^2\right).\frac{70}{500^2} = 31\,342\,000\ N$ → ***S = 20 325 kN < 31 342 kN***

→ ***Requirement is not satisfied! Stabilization of the flange wont be taken into consideration!***

➢ *Lateral torsional buckling*

$\frac{M_{Ed}}{M_{b,Rd}} \leq 1,0 \rightarrow \quad M_{b,Rd} = \chi_{LT}.W_y.\frac{f_y}{\gamma_{M1}}$,where:

$W_y = W_{pl} = 2194\ cm^3$ → cross-section class 1

$\gamma_{M1} = 1,05$

$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta.\overline{\lambda}_{LT}^2}} \leq \begin{cases} 1,0 \\ \frac{1}{\overline{\lambda}_{LT}^2} \end{cases}$

$\Phi_{LT} = 0,5.[1 + \alpha_{LT}.(\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta.\overline{\lambda}_{LT}^2]$, where: α_{LT} – imperfection factor

$\overline{\lambda}_{LT,0} = 0,4\ ;\ \ \beta = 0,75$

h/b = 500/200 = 2,5 > 2,0 → For welded I-sections the buckling curve is *c;*

→ $\alpha_{LT} = 0,49$

$\overline{\lambda}_{LT} = \sqrt{\frac{W_y.f_y}{M_{cr}}}$

$M_{cr} = C_1.\frac{\pi^2.E.I_z}{(k_z.L)^2}.\sqrt{\left(\frac{k_z}{k_w}\right)^2.\frac{I_w}{I_z} + \frac{(k_z.L)^2.G.I_t}{\pi^2.E.I_z}}$

$C_1 = 1,112$

$k_z = 1,0$

$k_w = 1,0$

$G = 8\,100\ kN/cm^2$

For length of the beam $L = 11{,}60\ m \rightarrow$ $\boldsymbol{M_{cr} = 19\ 663\ kN.cm}$

$\overline{\lambda_{LT}} = 1{,}75;\ \Phi_{LT} = 2{,}30;\ \chi_{LT} = 0{,}26;\ \rightarrow\ M_{b,Rd} = 15\ 180\ kN.cm$

$M_{Ed} = 31\ 400\ kN.cm$

$\frac{M_{Ed}}{M_{b,Rd}} = \frac{31\ 400}{15\ 180} = 2{,}07 > 1{,}0$

→ Requirement is not satisfied! → Transversal restrains are needed along the beam! Horizontal deflections are limited through transversal beams and horizontal braced frames!

Buckling length is reduced:

L = 6,60 m

Elastic critical moment as a result of analysis with LTBeamN: $\boldsymbol{M_{cr} = 56\ 831\ kN.cm} \rightarrow$

$\overline{\lambda_{LT}} = 1{,}03;\ \Phi_{LT} = 1{,}00;\ \chi_{LT} = 0{,}68;\ \rightarrow\ M_{b,Rd} = 39\ 136\ kN.cm$

$M_{Ed} = 31\ 600\ kN.cm\ \rightarrow \frac{M_{Ed}}{M_{b,Rd}} = \frac{31\ 600}{39\ 136} = 0{,}81 < 1{,}0$

→ Requirement is satisfied! → The beam will remain stable.

2.2. *Analyzing the beam in second stage – composite beam*

2.2.1. Defining the loads, affecting the beam

- *Dead loads*
  - Steel decking: $g_{LT} = 0{,}11\ kN/m^2$
  - Concrete: $g_{c,dry} = 2{,}60\ kN/m^2$
  - Paste: $g_c = 0{,}36\ kN/m^2$
  - Flooring: $g_f = 1{,}10\ kN/m^2$
  - Ceiling: $g_s = 0{,}2\ kN/m^2$
  - Installations: $g_i = 0{,}4\ kN/m^2$
  - Walls: $g_w = 0{,}5\ kN/m^2$

$g_{tot} = 5{,}27\ kN/m^2$

$g'_{tot} = b.g_{tot} + g_{IPE500} = 3{,}20.5{,}27+0{,}907 \rightarrow \boldsymbol{g'_{tot} = 17{,}77\ kN/m}$

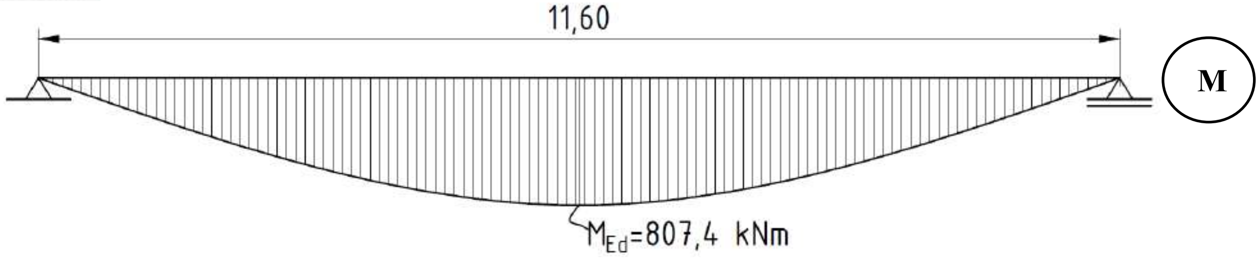
- *Live loads*
  - Operational load for rooms of category “C” и “D”, according to БДС EN1991-1-1, т. 6.3.1.2.: $q = 5{,}0\ kN/m^2$

    $q' = 3{,}80.5{,}0 \rightarrow \boldsymbol{q' = 16{,}0\ kN/m}$

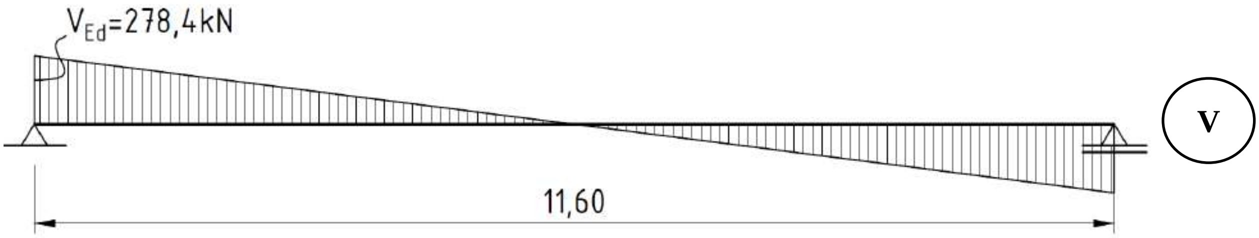
- *Design values of the loads*
  - $g_d = \gamma_G.g = 1{,}35.17{,}77 \rightarrow \boldsymbol{g_d = 24{,}0\ kN/m}$
  - $q'_d = \gamma_q.q' = 1{,}50.4{,}80 \rightarrow \boldsymbol{q'_d = 24{,}0\ kN/m}$

2.2.2. Design values of the bending moment and shear force

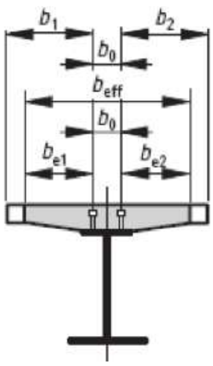
$M_{Ed} = (g_d + q'_d).l^2/8 = (24{,}0 + 24{,}0).11{,}60^2/8 \rightarrow \boldsymbol{M_{Ed} = 807{,}4\ kNm}$



$V_{Ed} = (g_d + q'_d).l/2 = (24,0 + 24,0).11,60/2 \rightarrow \boldsymbol{V_{Ed} = 278,4\ kN}$



2.2.3. Effective width



$b_{eff} = b_o + \sum b_{ei}$

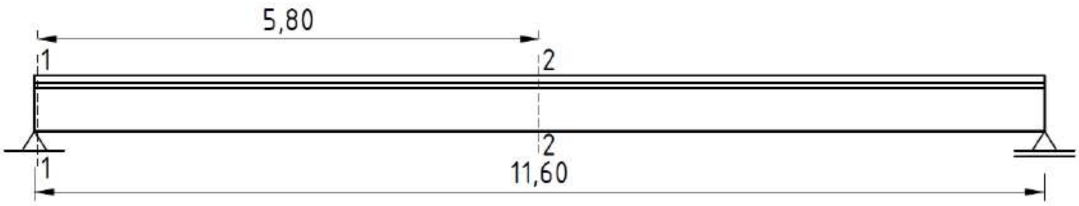
$b_{ei} = L_e/8 \leq b_i \rightarrow b_{ei} = 11,60/8 \leq 3,20/2 \rightarrow b_{ei} = 1,45 < 1,60\ m = 145\ cm$

$b_1 = b_2 = b_{ei} = 145\ cm\ ;\ \ b_0 = 0\ ; \rightarrow b_{eff} = 0 + 2.145 = 290\ cm$

$\boldsymbol{b_{eff} = 290\ cm}$

2.2.4. Resistance of cross-sections of beams – examination of the critical sections

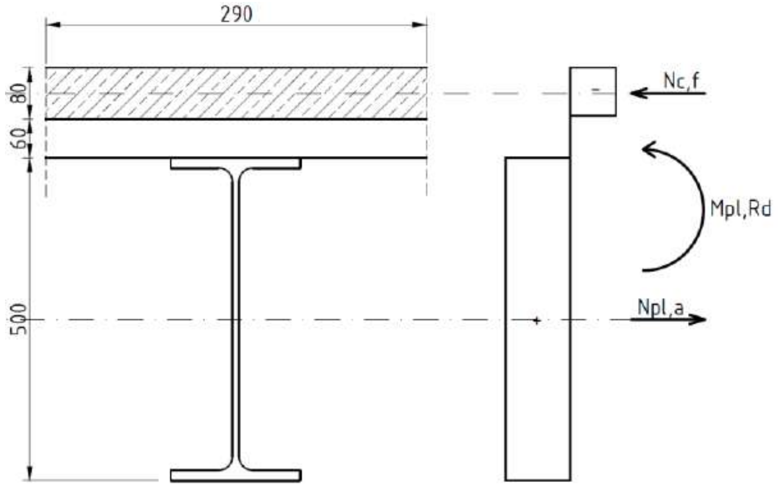
Resistance checks in the critical sections „1-1“ и „2-2“ have to be done. These are respectively the section with maximal shear force and the section with maximal bending moment.



Critical sections define the so called „critical length“, which is equal to the distance between two critical sections $\rightarrow l_{cr} = L/2 = 11,60/2 \rightarrow \boldsymbol{l_{cr} = 5,80\ m}$

- *Bending resistance (section „2-2“)*

It is considered a full interaction between steel and concrete. Calculations in plastic stage.



$N_{c,f} = 0,85.f_{cd}.b_{eff}.h_c = 0,85.2,0.290.8 \rightarrow \boldsymbol{N_{c,f} = 3\ 944\ kN}$

$N_{pl,a} = A_a.f_{yd} = 115,5.27,5/1,05 \rightarrow \boldsymbol{N_{pl,a} = 3\ 025\ kN}$

$N_{pl,a} < N_{c,f}$ → "Zero line" is located in the slab!

Definition of the "zero line" location.: $x = \frac{N_a}{b_{eff}.0,85.f_{cd}} = \frac{3\ 025}{290.0,85.2} \rightarrow \boldsymbol{x = 6,14\ cm}$

$M_{pl,Rd} = N_{pl,a}.(h/2 + h_p + h_c - x/2) = 3\ 025.(50/2+8+6 - 6,14/2) = 108\ 688\ kN.cm$

$\boldsymbol{M_{pl,Rd} = 1\ 087\ kNm}$

$M_{Ed}/M_{pl,Rd} = 807,4/1\ 087 = 0,75$ → Requirement is satisfied!

- *Shear resistance*

$V_{el,Rd} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{59,87.27,5}{\sqrt{3}.1,05} \rightarrow \boldsymbol{V_{el,Rd} = 905,3\ kN}$

$\frac{V_{z,Ed}}{V_{el,Rd}} \leq 1,0 \rightarrow \frac{278,4}{905,3} = 0,31 < 1,0$ → ***Requirement is satisfied!***

$h_w/t_w = 426/10,2 = 41,8$ – slenderness of the web;

$h_w/t_w \leq 72.\varepsilon/\eta = 72.0,924/1,00 = 66,5$

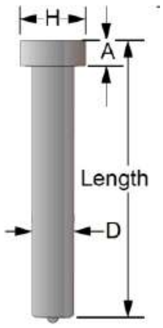
$\eta = 1,00$ – for steel S275;

→ $h_w/t_w = 41,8 < 66,5$ → Shear buckling resistance of web check is not necessary!

2.2.5. Design of shear studs

Shear studs resist the longitudinal shear (sliding forces) and prevents longitudinal splitting. In vertical direction the shear studs should resist the separation forces, which are trying to separate the composite slab from the steel section.

Shear studs KÖCO SD 19x100 have the following characteristics:



$D = d = 19\ mm$

$L = h_{sc} = 100\ mm$

$H = 32\ mm$

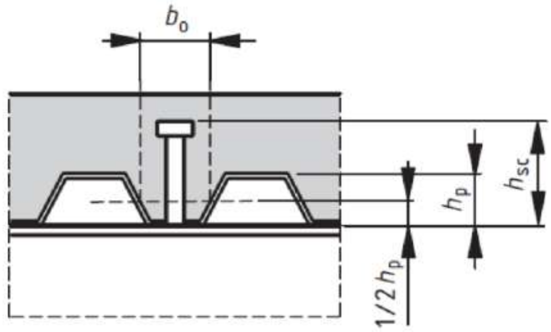
$F_u = 450\ MPa$

- *Shear stud connection resistance*

$$P_{Rd} = min\begin{cases}\dfrac{0,8.f_u.\pi.D^2/4}{\gamma_v}\\[2ex]\dfrac{0,29.\alpha.D^2.\sqrt{f_{ck}.E_{cm}}}{\gamma_v}\end{cases} = min\begin{cases}\dfrac{0,8.45.\pi.1,9^2/4}{1,25}\\[2ex]\dfrac{0,29.1,00.1,9^2.\sqrt{3.3280}}{1,25}\end{cases} = min\begin{cases}81,7\\83,1\end{cases} \rightarrow \boldsymbol{P_{Rd} = 81,7\ kN}$$

When $h_{sc}/d \geq 4 \rightarrow \alpha = 1,0$

$$k_t = \frac{0,7}{\sqrt{n_r}}.\frac{b_0}{h_p}.\left(\frac{h_{sc}}{h_p} - 1\right) = \frac{0,7}{\sqrt{2}}.\frac{140}{60}.\left(\frac{100}{60} - 1\right) = 0,80$$



$b_0 = 140\ mm$

$n = 2$ – two shear studs in every rib

$k = 0,8 < k_{max} = 0,85$

$P_{Rd} = k_t.P_{Rd} = 0,80.81,7$

$\boldsymbol{P_{Rd} = 65,4\ kN}$

In order to provide full interaction, the number of the studs in the critical length should be:

$n_f \geq min\{N_{c,f}\ ;N_{pl,a}\}/P_{Rd} \rightarrow n_f \geq min\{3\ 944;\ 3\ 025\}/65,4 = 3\ 025/65,4 \approx 46\ studs$

$\rightarrow$ $\boldsymbol{n_f = 46\ studs\ /l_{cr}}$

The extreme number of the shear studs, which can be placed along the beam is equal to the number of the steel decking's ribs, located on the beam flange.

$n = l_{cr}/b_p = 5800/300 \approx 19$

$b_p = 300\ mm$ – space between two ribs of the steel decking

Extreme number of shear studs, which can be placed inside the critical length is 19x2 = 38 бр. Full interaction of the composite slab and the steel section can't be reached. Level of interaction (η) should be determined.

$\eta = n/n_f = 38/46 \rightarrow \boldsymbol{\eta = 0,83}$

According to БДС EN1994-1-1, т. 6.6.1.2: $\eta_{min} = 1 - \left(\frac{355}{f_y}\right).(1,0 - 0,04.L_e)$

$\eta_{min} = 1 - \left(\frac{355}{275}\right).(1,0 - 0,04.11,60) = 0,31 \rightarrow \boldsymbol{\eta = 0,83 > \eta_{min} = 0,31}$ $\rightarrow$ ***The shear studs are considered ductile!***

- *Bending resistance in plastic stage, when partial interaction is provided*

$M_{Rd} = M_{pl,a,Rd} + (M_{pl,Rd} - M_{pl,a,Rd}).\eta$

$M_{pl,a,Rd} = W_{pl,y} .f_y/\gamma_{M0} = 2194{,}2.27{,}5/1{,}05 = 57\ 467{,}2\ kNcm$

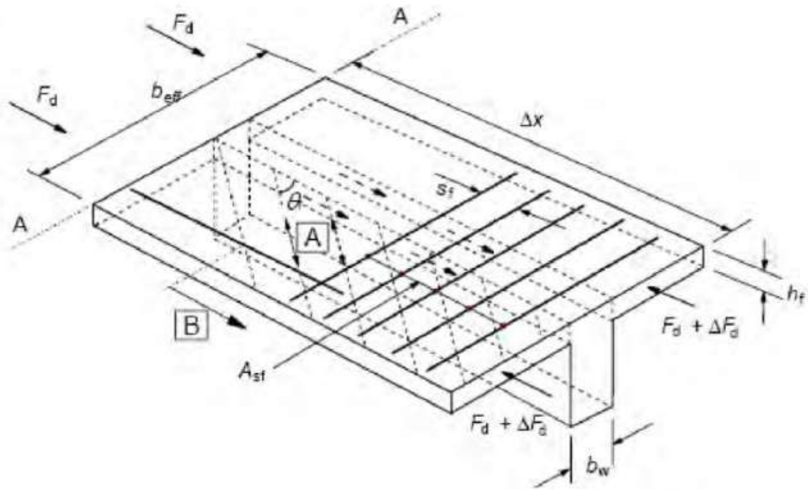
$M_{pl,Rd} = 108\ 700\ kNcm$

$M_{Rd} = 57\ 467 + (108\ 700 - 57\ 467).0{,}83 = 99\ 990\ kNcm \rightarrow$ $\boldsymbol{M_{Rd} = 999{,}9\ kNm}$

$M_{Ed} / M_{Rd} = 807{,}4/999{,}9 = 0{,}81 \rightarrow$ Requirement is satisfied!

2.2.6. Check for longitudinal shear

Check according to БДС EN1992-1-1, т. 6.2.4.



Contribution of the steel decking is not taken into consideration.

The check is made for section „a-a“, where reinforcement mesh *5N10* helps resisting the longitudinal shear. Reinforcement cross-section area: $A_t = A_{sf}/s_f = 3{,}93\ \ cm^2/m$

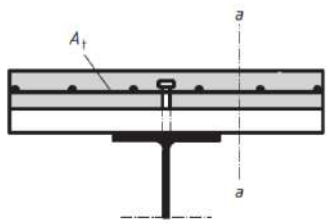
v_{Ed} – cross shear stress in the surface of interaction between the shear stud and the concrete slab;

$v_{Ed} = \Delta F_d/(2.h_f) = 436/(2.0{,}08) \rightarrow$ $\boldsymbol{v_{Ed} = 2\ 725\ kN/m^2}$

$\Delta F_d = 2.P_{Rd}/s = 2.65{,}4/0{,}3 = 436\ kN/m$ – longitudinal shear force;

$s = 0{,}3\ m$ – space between the ribs of the steel decking;

<u>To prevent crushing of the compression struts in the flange, the following condition should be satisfied:</u>



$v_{Ed} \leq v.f_{cd}.\sin\theta.\cos\theta = 0{,}6.2.\sin 45^\circ.\cos 45^\circ = 0{,}6\ kN/cm^2$

Прието: $\theta = 45^\circ$; $v = 0{,}6$

$v_{Ed} = 2\ 725\ kN/m^2 < 6000\ kN/m^2 \rightarrow$ Condition is satisfied!

<u>The transverse reinforcement per unit length may be determined as follows:</u>

$(A_{sf}.f_{yd}/s_f) + A_{s1}.f_{yd}/s_f \geq v_{Ed}.h_f/\cot g\theta$

A_{sf} и A_{sf1} – reinforcement in section „*a-a*“;

$(3{,}93.43{,}5) + (3.50{,}3/100).43{,}5 \geq 2\ 725.0{,}08/\cot g 45 \rightarrow$ $\boldsymbol{236\ kN/m > 218\ kN/m \rightarrow}$

Condition is satisfied! → The reinforcement in the section is enough for resisting the longitudinal shear! No extra reinforcement is needed!

2.2.7. Serviceability limit states

➢ *Composite section properties*

$n_0 = E_a/E_{cm} = 21000/3280 = 6,4$

$n_{eff} = 2.n_0 = 2.6,4 = 12,8$

$A_{c,eff} = A_c/n_{eff} = 2\,320/12,8 \rightarrow \mathbf{A_{c,eff} = 181,25\ cm^2}$

$A_c = b_{eff}.h_f = 290.8 = 2\,320\ cm^2$

$I_{c,eff} = b_{eff}.h_c^3/(12.n_{eff}) = 290.8^3/(12.12,8)$

$\rightarrow \mathbf{I_{c,eff} = 966,7\ cm^4}$

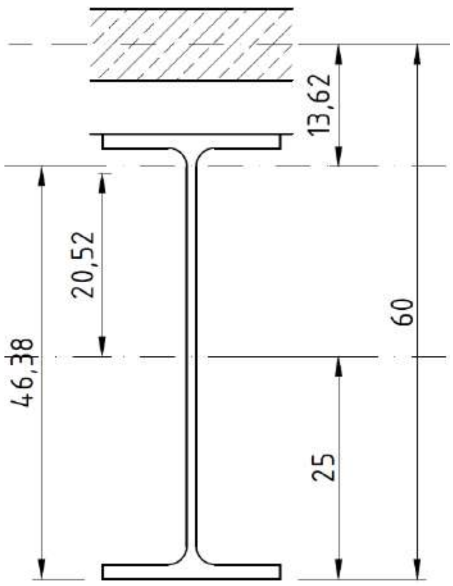
$A_{comb} = A_{c,eff} + A_a = 181,25 + 115,5 = 296,75\ cm^2$

$\mathbf{A_{comb} = 296,75\ cm^2}$

$I_{comb} = I_a + A_a.z_a^2 + I_{c,eff} + A_{c,eff}.z_c^2$

$I_{comb} = 48\,198,53 + 115,5.20,52^2 + 966,7 + 181,25.13,62^2$

$\mathbf{I_{comb} = 131\,421,5\ cm^4}$



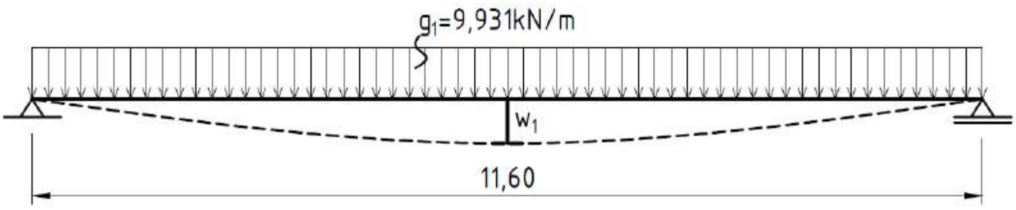
➢ *Deflection*

$w_{total} = w_1 + w_2 + w_3$

$w_1 = \frac{5}{384}.\frac{g_{k1}.L^4}{E_a.I_a} = \frac{5}{384}.\frac{9{,}931.10^{-2}.1160^4}{21000.48\,198{,}53} = 2{,}31\ cm$

$g_1 = 9,024 + 0,907 = 9,931\ kN/m$

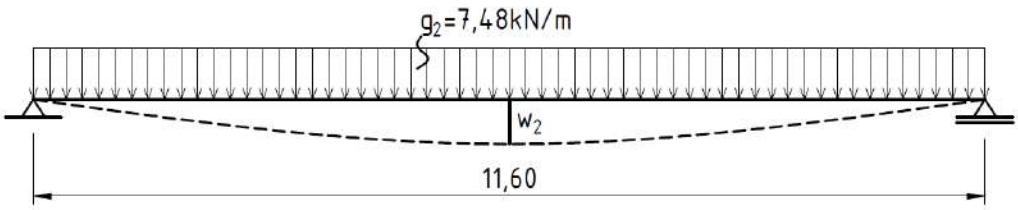
$\mathbf{w_1 = 2,31\ cm}$



$w_2 = \frac{5}{384}.\frac{g_{k2}.L^4}{E_a.I_{comb}} = \frac{5}{384}.\frac{7{,}48.10^{-2}.1160^4}{21000.131\,421{,}5} = 0{,}63\ cm$

$g_2 = 7,48\ kN/m$

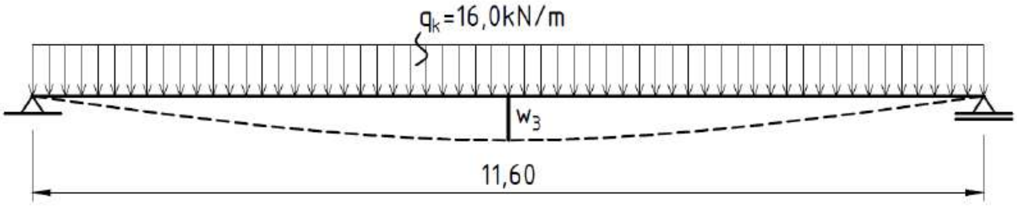
$\mathbf{w_2 = 0,63\ cm}$



$w_2 = \frac{5}{384}.\frac{q_k.L^4}{E_a.I_{comb}} = \frac{5}{384}.\frac{16.10^{-2}.1160^4}{21000.131\,421{,}5} = 1{,}37\ cm$

$q_k = 16\ kN/m$

$\mathbf{w_3 = 1,37\ cm}$



$w_f = w_2 + w_3$ = 0,63 + 1,37 = 2,0 cm

Increasing the deflection in second stage, because of the partial interaction:

$w_p = w_f.\left(1+\beta.(1-\eta).\left(\frac{w_a}{w_f}-1\right)\right) =$

w_f *= 2,0 cm* – deflection in second stage

$w_a = = \frac{5}{384}.\frac{(g_{k2}+q_k).L^4}{E_a.I_a}$ *- deflection in stage 2, without taking the contribution of the slab into account (steel section resists the deformations of it's own)*

$w_a = \frac{5}{384}.\frac{23{,}48.10^{-2}.1\,160^4}{21\,000.48\,198{,}53} \rightarrow w_a = 5{,}47$ *cm*

$\beta = 0{,}3$ –

$\eta = 0{,}83$ – level of interaction;

$w_p = w_f.\left(1+0{,}3.(1-0{,}83).\left(\frac{5{,}47}{2{,}0}-1\right)\right) = 2{,}18$ *cm*

Total deflection of the beam, taking the partial interaction into account:

$w_{total} = w_1 + w_p = 2{,}31 + 2{,}18 \rightarrow$ **w_{total} = 4,49 cm**

$w_{max} = L/250 = 11600/250 \rightarrow$ **w_{max} = 4,64 cm**

w_{total} = 4,49 cm < w_{max} = 4,64 cm → Deflection check is satisfied!

- *Control of the vibrations*

$f_1 \geq f_{1,SLS}$

$f_{1,SLS}$ *=2,5 Hz* – for slabs in buildings and parking and garages;

$f_1 \approx 18/\sqrt{\delta}$ *[Hz]* – frequency of first form of free oscillation (vertical surface) for simple beams;

$\delta = \frac{5}{384}.\frac{(g_k+0{,}2.q_k).L^4}{E_a.I_{comb}} = \frac{5}{384}.\frac{(9{,}93+7{,}48+0{,}2.16).10^{-2}.1160^4}{21000.131\,421{,}5}$ ***→ δ = 1,76 cm = 17,6 mm***

$f_1 \approx 18/\sqrt{17{,}6}$ → ***f_1 ≈ 4,3 Hz > $f_{1,SLS}$ =2,5 Hz → No unacceptable vibrations!***

3. Secondary beam SB11

The secondary beams have a static scheme “simple beam”. They receive load from the slab and distribute it by supporting reactions to the main beams. The beams are considered to be composite and the connection between the plate and the beam is realized thanks to welded shear studs, resisting the sliding (horizontal shear) forces and resisting the shear force.

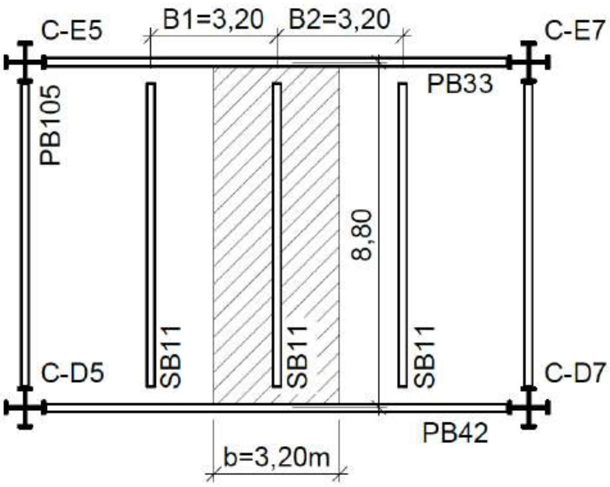
Secondary beams *SB8 are taken into consideration.* Their length is $l = 8{,}80\ m$, and the space in-between is $B = 3{,}20\ m$.

There are two stages of work considered, similar to the calculation of the composite slab. First stage – concrete placing stage, and Second stage – composite slab is resisting the loads (operational mode).

3.1. Analyzing beam in first stage

3.1.1. Defining the loads, affecting the beam

It is considered an area load, influencing the beam. The size of the area load is calculated: $(b = 1/2.(B_1+B_2) = 1/2.(3{,}20+3{,}20) \rightarrow \mathbf{b = 3{,}20\ m})$. Area load is approximated to a linear distributed load.



- *Dead loads*
  - Steel decking: $g_{LT} = 0{,}11\ kN/m^2 \rightarrow g'_{LT} = b.\ g_{LT}$
    $g'_{LT} = 3{,}20.\ 0{,}11 \rightarrow \mathbf{g'_{LT} = 0{,}352\ kN/m}$
  - Fresh concrete: $g_{c,wet} = 2{,}71\ kN/m^2 \rightarrow$
    $g'_{c,wet} = b.g_{c,wet} = 3{,}20.2{,}71 \rightarrow \mathbf{g'_{c,wet} = 8{,}672\ kN/m}$
    $\sum \mathbf{g = g = 9{,}024\ kN/m}$

- *Live loads*
  - Loading in the work area (3,0x3,0 m): $q = 1{,}5\ kN/m^2 \rightarrow$
    $q' = b.q = 3{,}20.1{,}5 \rightarrow \mathbf{q' = 4{,}80\ kN/m}$
  - Loading out of the work zona: $q' = 0{,}75\ kN/m^2 \rightarrow$
    $q'' = b.q' = 3{,}20.0{,}75 \rightarrow \mathbf{q'' = 2{,}40\ kN/m}$

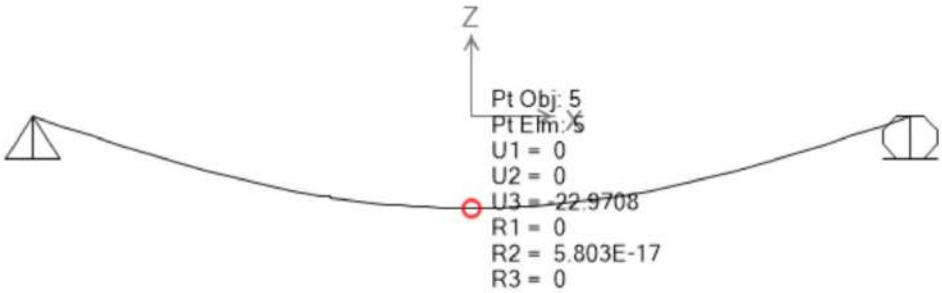
- *Design values of the loads*
  - $g_d = \gamma_G.g = 1{,}35.9{,}024 \rightarrow \mathbf{g_d = 12{,}18\ kN/m}$
  - $q'_d = \gamma_q.q' = 1{,}50.4{,}80 \rightarrow \mathbf{q'_d = 7{,}20\ kN/m}$
  - $q''_d = \gamma_q.q'' = 1{,}50.2{,}40 \rightarrow \mathbf{q''_d = 3{,}60\ kN/m}$

3.1.2. Pre-selection of the beam section by satisfying the deflection requirements

The check is made with the characteristic values for SLS combination.

Selected section: ***IPE 400***, $g_{IPE} = 0{,}663\ kN/m$

Verifying if the vertical deflection in the middle of the span is less than the limit deflection.

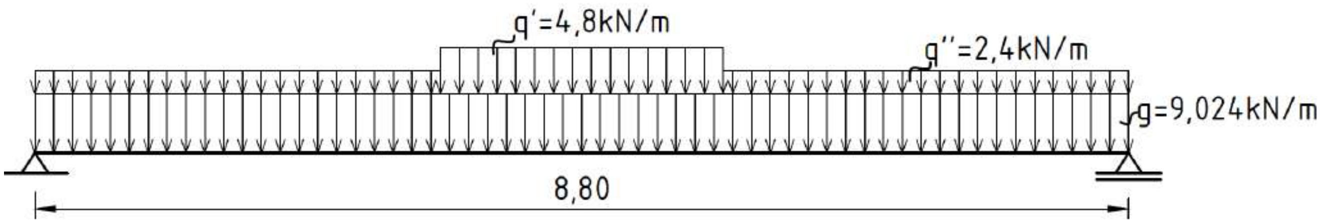


$\delta_{max} \leq L/250 = 880/250 = 35{,}2\ mm \rightarrow \boldsymbol{\delta_{max} = 23\ mm < 35{,}2\ mm}$

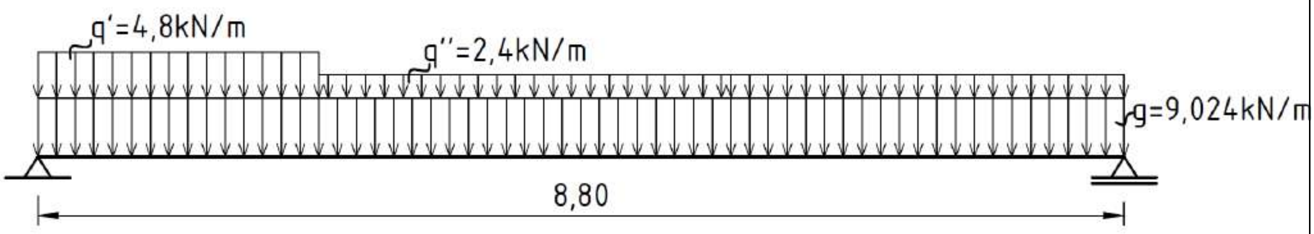
The deflection satisfies the requirements of the limit state! The ULS requirements would be checked for a beam section IPE400!

3.1.3. Load schemes

- *For maximal bending moment in the middle of the span*



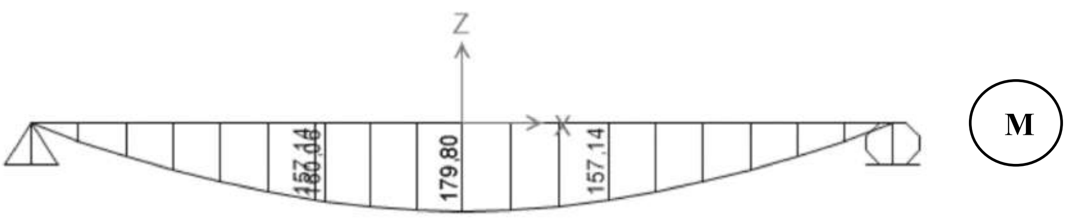
- *For a maximal support reaction and maximum shear force*



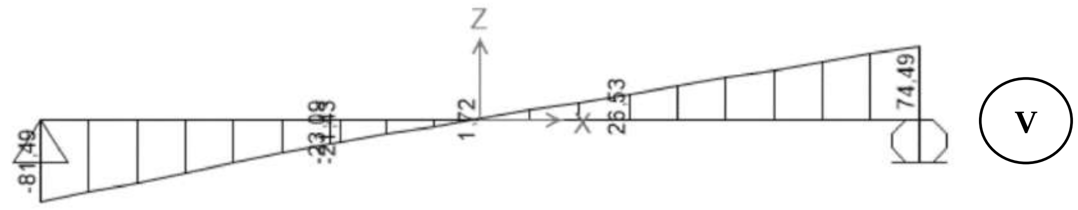
Note: *Schemes are showing the characteristic values of the loads!*

3.1.4. Moments and forces diagrams

- *Design values of bending moment*



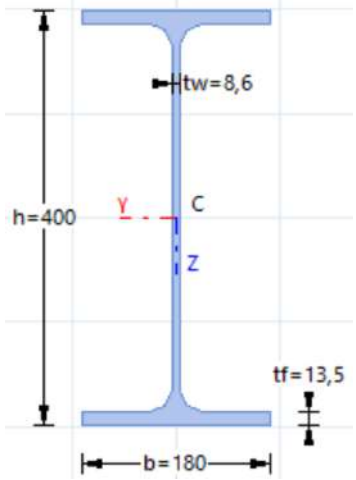
➢ *Design values of shear force*



3.1.5. Defining the steel section class

Steel class selected: S275 → $\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{275}} = 0{,}924$

➢ *Steel cross section characteristics*



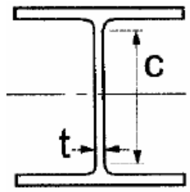
h =	400,0	b2=	180,0	b =	180,0	ri=	21
tw=	8,6	tf2=	13,5	tf=	13,5	ro=	0

Инерционни характеристики	
A = 84.464 cm	
Cz = 20.000 cm	Cy = 9.000 cm
Iy = 23128.369 cm4	Iz = 1317.824 cm4
Wy = 1156.418 cm3	Wz = 146.425 cm3
Wply= 1307.148 cm3	Wplz= 229.000 cm3
ry = 16.548 cm	rz = 3.950 cm
Avy = 48.600 cm2	Avz = 42.695 cm2
It = 50.451 cm4	Wt = 26.625 cm3

➢ *Slenderness of the web*

$\lambda = c/t = (h - 2.t_f - 2.r_i)/t_w = (400 - 2.13{,}5 - 2.21)/8{,}6 \rightarrow \lambda = 38{,}5$

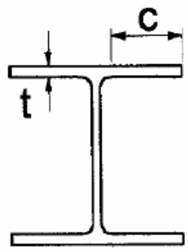
$\boldsymbol{\lambda = 38{,}5 < 72.\varepsilon} = 72.0{,}924 = 66{,}5 \rightarrow$ *Cross section class 1*



➢ *Slenderness of the flanges*

$\lambda = c/t = (b/2 - t_w/2 - r_i)/t_f = (180/2 - 8{,}6/2 - 21)/13{,}5 \rightarrow \lambda = 4{,}79$

$\boldsymbol{\lambda = 4{,}79 < 9.\varepsilon} = 9.0{,}924 = 8{,}31 \rightarrow$ *Cross section class 1*



→ ***The cross section is classified as class 1!***

3.1.6. Resistance of the cross-section.

➢ *Bending resistance*

$M_{y,el,Rd} = \frac{W_{y,el}.f_y}{\gamma_{M0}} = \frac{1156{,}4.27{,}5}{1{,}05} = 30\ 286{,}7\ kN.cm \rightarrow \boldsymbol{M_{y,el,Rd} = 302{,}9\ kNm}$

$\frac{M_{y,Ed}}{M_{y,el,Rd}} \leq 1{,}0 \rightarrow \frac{179{,}80}{302{,}9} = 0{,}6 < 1{,}0 \rightarrow$ ***Requirement is satisfied!***

- *Shear resistance*

$V_{el,Rd} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{42,7.27,5}{\sqrt{3}.1,05} \rightarrow$ ***$V_{el,Rd}$ = 645,6 kN***

$\frac{V_{z,Ed}}{V_{el,Rd}} \leq 1,0 \rightarrow \frac{81,5}{645,6} = 0,13 < 1,0 \rightarrow$ ***Requirement is satisfied!***

3.1.1. Buckling resistance

- Check for *stabilized* upper flange

Check if the upper flange is stabilized by the steel decking, whose stiffeners are perpendicular to the axis of the beam. This circumstance improves the buckling resistance of the beam. Requirements according to БДС EN1993-1-3, т. 10.1:

$S \geq \left(EI_w.\frac{\pi^2}{L^2} + GI_t + EI_z.\frac{\pi^2}{L^2}.0,25.h^2\right).\frac{70}{h^2}$, where:

$S = 1000.\sqrt{t^3}.\left(50 + 10.\sqrt[3]{b_{roof}}\right).\frac{s}{h_w}$ *[N]* – shear stiffness, provided from the steel decking;

$S = 1000.\sqrt{0,96^3}.\left(50 + 10.\sqrt[3]{44\,800}\right).\frac{3200}{60}$ = *20 325 154 N*

$\left(EI_w.\frac{\pi^2}{L^2} + GI_t + EI_z.\frac{\pi^2}{L^2}.0,25.h^2\right).\frac{70}{h^2} = \left(210\,000.492\,149.10^6.\frac{\pi^2}{8800^2} + 80\,769.50,27.10^4 + 210\,000.1317,8.10^4.\frac{\pi^2}{8800^2}.0,25.400^2\right).\frac{70}{400^2}$ = *29 686 N* → ***S = 20 325 kN < 29 686 kN***

→ ***Requirement is not satisfied! Stabilization of the flange wont be taken into consideration!***

- *Lateral torsional buckling*

$\frac{M_{Ed}}{M_{b,Rd}} \leq 1,0 \rightarrow \quad M_{b,Rd} = \chi_{LT}.W_y.\frac{f_y}{\gamma_{M1}}$, where:

$W_y = W_{pl} = 1019\ cm^3$ → cross-section class 1

$\gamma_{M1} = 1,05$

$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta.\bar{\lambda}_{LT}^2}} \leq \begin{cases} 1,0 \\ \frac{1}{\bar{\lambda}_{LT}^2} \end{cases}$

$\Phi_{LT} = 0,5.[1 + \alpha_{LT}.(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta.\bar{\lambda}_{LT}^2]$, where: α_{LT} – imperfection factor

$\bar{\lambda}_{LT,0} = 0,4\ ;\ \beta = 0,75$

$h/b = 400/180 = 2,22 > 2,0$ → For welded I-sections the buckling curve is *c;*

→ $\alpha_{LT} = 0,49$

$\bar{\lambda}_{LT} = \sqrt{\frac{W_y.f_y}{M_{cr}}}$

$M_{cr} = C_1.\frac{\pi^2.E.I_z}{(k_z.L)^2}.\sqrt{\left(\frac{k_z}{k_w}\right)^2.\frac{I_w}{I_z} + \frac{(k_z.L)^2.G.I_t}{\pi^2.E.I_z}}$

$C_1 = 1,112$

$k_z = 1,0$

$k_w = 1,0$

$G = 8\,100\ kN/cm^2$

For length of the beam $L = 8{,}80\ m \rightarrow$ $\boldsymbol{M_{cr} = 15\,416\ kN.cm}$

$\overline{\lambda_{LT}} = 1{,}53;\ \Phi_{LT} = 1{,}65;\ \chi_{LT} = 0{,}38;\ \rightarrow\ M_{b,Rd} = 12\,977\ kN.cm$

$M_{Ed} = 17\,980\ kN.cm$

$$\frac{M_{Ed}}{M_{b,Rd}} = \frac{17\,980}{12\,977} = \mathbf{1{,}38 > 1{,}0}$$

→ Requirement is not satisfied! → Transversal restrains are needed along the beam! Horizontal deflections are limited through transversal beams and horizontal braced frames!

Buckling length is reduced:

$L = 4{,}40\ m$

Elastic critical moment as a result of analysis with LTBeamN: $\boldsymbol{M_{cr} = 49\,304\ kN.cm \rightarrow}$

$\overline{\lambda_{LT}} = 0{,}85;\ \Phi_{LT} = 0{,}88;\ \chi_{LT} = 0{,}73;\ \rightarrow\ M_{b,Rd} = 24\,985\ kN.cm$

$$M_{Ed} = 17\,980\ kN.cm \rightarrow \frac{M_{Ed}}{M_{b,Rd}} = \frac{17980}{23\,679} = \mathbf{0{,}72 < 1{,}0}$$

→ Requirement is satisfied! → The beam will remain stable.

3.2. Analyzing the beam in second stage – composite beam

3.2.1. Defining the loads, affecting the beam

- *Dead loads*
  - Steel decking: $g_{LT} = 0{,}11\ kN/m^2$
  - Concrete: $g_{c,dry} = 2{,}60\ kN/m^2$
  - Paste: $g_c = 0{,}36\ kN/m^2$
  - Flooring: $g_f = 1{,}10\ kN/m^2$
  - Ceiling: $g_s = 0{,}2\ kN/m^2$
  - Installations: $g_i = 0{,}4\ kN/m^2$
  - Walls: $g_w = 0{,}5\ kN/m^2$

  $g_{tot} = 5{,}27\ kN/m^2$

  $g'_{tot} = b.g_{tot} + g_{IPE400} = 3{,}20.5{,}27+0{,}907 \rightarrow \boldsymbol{g'_{tot} = 17{,}77\ kN/m}$

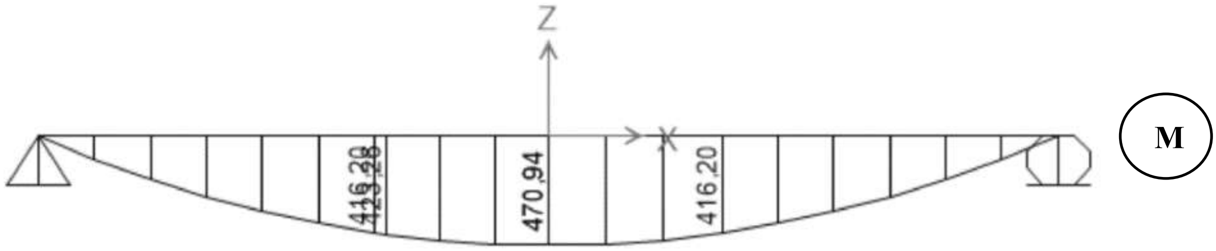
- *Live loads*
  - Operational load for rooms of category “C” и “D”, according to БДС EN1991-1-1, т. 6.3.1.2.: $q = 5{,}0\ kN/m^2$

    $q' = 3{,}80.5{,}0 \rightarrow \boldsymbol{q' = 16{,}0\ kN/m}$

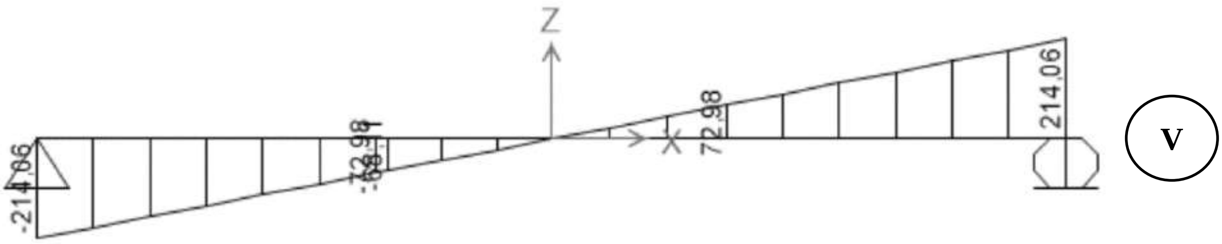
- *Design values of the loads*
  - $g_d = \gamma_G.g = 1{,}35.17{,}77 \rightarrow \boldsymbol{g_d = 24{,}0\ kN/m}$
  - $q'_d = \gamma_q.q' = 1{,}50.4{,}80 \rightarrow \boldsymbol{q'_d = 24{,}0\ kN/m}$

3.2.2. Design values of the bending moment and shear force

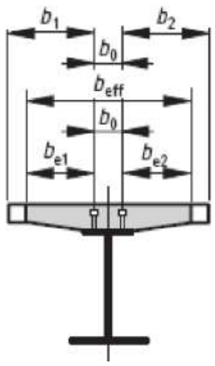
M_{Ed} = 470,9 kNm



V_{Ed} = 214,1 kN



3.2.3. Effective width



$b_{eff} = b_o + \sum b_{ei}$

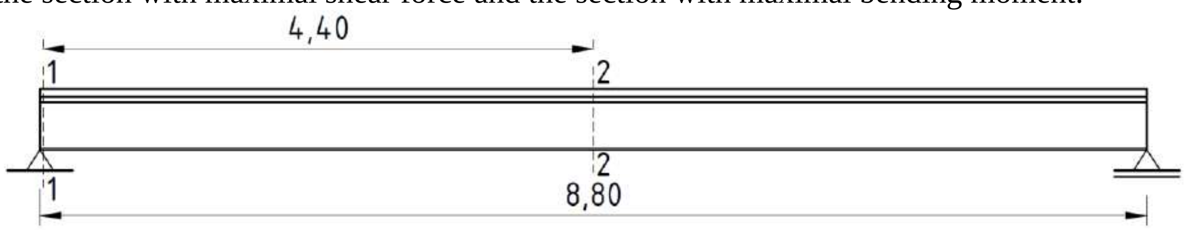
$b_{ei} = L_e/8 \leq b_i \ \rightarrow b_{ei} = 8,80/8 \leq 3,20/2 \rightarrow b_{ei} = 1,10 < 1,60\ m = 110\ cm$

$b_1 = b_2 = b_{ei} = 110\ cm\ ;\ \ b_0 = 0\ ; \rightarrow b_{eff} = 0 + 2.110 = 220\ cm$

$\boldsymbol{b_{eff} = 220\ cm}$

3.2.4. Resistance of cross-sections of beams – examination of the critical sections

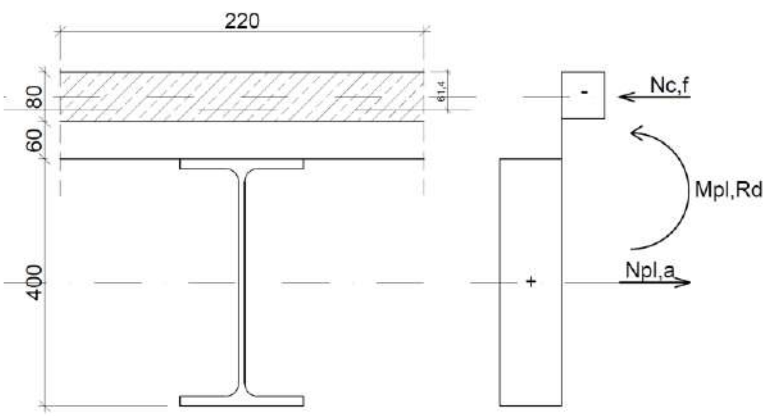
Resistance checks in the critical sections „1-1“ и „2-2“ have to be done. These are respectively the section with maximal shear force and the section with maximal bending moment.



Critical sections define the so called „critical length“, which is equal to the distance between two critical sections → $l_{cr} = L/2 = 8,80/2 \rightarrow$ $\boldsymbol{l_{cr} = 4,40\ m}$

➢ *Bending resistance (section „2-2“)*

It is considered a full interaction between steel and concrete. Calculations in plastic stage.



$N_{c,f} = 0{,}85.f_{cd}.b_{eff}.h_c = 0{,}85.2{,}0.220.8 \rightarrow \boldsymbol{N_{c,f} = 2\ 992\ kN}$

$N_{pl,a} = A_a.f_{yd} = 84{,}5.27{,}5/1{,}05 \rightarrow \boldsymbol{N_{pl,a} = 2\ 213\ kN}$

$N_{pl,a} < N_{c,f}$ → "Zero line" is located in the slab!

Definition of the "zero line" location: $x = \frac{N_a}{b_{eff}.0{,}85.f_{cd}} = \frac{2\ 213}{220.0{,}85.2} \rightarrow \boldsymbol{x = 5{,}91\ cm}$

$M_{pl,Rd} = N_{pl,a}.(h/2 + h_p + h_c - x/2) = 2\ 213.(40/2+8+6 - 5{,}91/2) = 68\ 702{,}6\ kN.cm$

$\boldsymbol{M_{pl,Rd} = 687\ kNm}$

$M_{Ed}/M_{pl,Rd} = 470{,}9/687 = 0{,}68$ → Requirement is satisfied!

➢ *Shear resistance*

$V_{Rd} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{42{,}7.27{,}5}{\sqrt{3}.1{,}05} \rightarrow \boldsymbol{V_{Rd} = 645{,}7\ kN}$

$\frac{V_{z,Ed}}{V_{el,Rd}} \leq 1{,}0 \rightarrow \frac{214}{645{,}7} = 0{,}33 < 1{,}0 \rightarrow \boldsymbol{v}$

$h_w/t_w = 331/8{,}6 = 38{,}5$ – slenderness of the web;

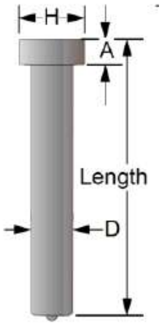
$h_w/t_w \leq 72.\varepsilon/\eta = 72.0{,}924/1{,}00 = 66{,}5$

$\eta = 1{,}00$ – for steel S275;

→ $h_w/t_w = 38{,}5 < 66{,}5$ → Shear buckling resistance of web check is not necessary!

3.2.5. Design of shear studs

Shear studs resist the longitudinal shear (sliding forces) and prevents longitudinal splitting. In vertical direction the shear studs should resist the separation forces, which are trying to separate the composite slab from the steel section.



Shear studs KÖCO SD 19x100 have the following characteristics:

$D = d = 19\ mm$

$L = h_{sc} = 100\ mm$

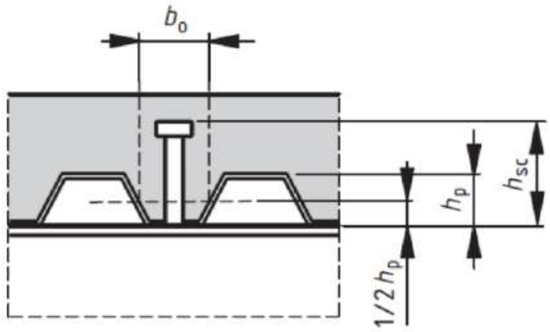
$H = 32\ mm$

$f_u = 450\ MPa$

- *Shear stud connection resistance*

$$P_{Rd} = min\begin{cases}\dfrac{0,8.f_u.\pi.D^2/4}{\gamma_v}\\ \dfrac{0,29.\alpha.D^2.\sqrt{f_{ck}.E_{cm}}}{\gamma_v}\end{cases} = min\begin{cases}\dfrac{0,8.45.\pi.1,9^2/4}{1,25}\\ \dfrac{0,29.1,00.1,9^2.\sqrt{3.3280}}{1,25}\end{cases} = min\begin{cases}81,7\\83,1\end{cases} \rightarrow \boldsymbol{P_{Rd} = 81,7\ kN}$$

When $h_{sc}/d \geq 4 \rightarrow \alpha = 1,0$



$$k_t = \frac{0,7}{\sqrt{n_r}}.\frac{b_0}{h_p}.\left(\frac{h_{sc}}{h_p} - 1\right) = \frac{0,7}{\sqrt{2}}.\frac{140}{60}.\left(\frac{100}{60} - 1\right)$$

$b_0 = 140\ mm$

$n = 2$ – two shear studs in every rib $k = 0,8 < k_{max} = 0,85$

$P_{Rd} = k_t.P_{Rd} = 0,80.81,66$

$\boldsymbol{P_{Rd} = 65,4\ kN}$

In order to provide full interaction, the number of the studs in the critical length should be:

$n_f \geq min\{N_{c,f}\ ;N_{pl,a}\}/P_{Rd} \rightarrow n_f \geq min\{2\ 992;\ 2\ 213\}/66,3 = 2\ 213/65,4 \approx 34\ studs$

$\rightarrow$ ***n_f = 34 studs /l_{cr}***

The extreme number of the shear studs, which can be placed along the beam is equal to the number of the steel decking's ribs, located on the beam flange.

$n = l_{cr}/b_p = 4400/300 \approx 14$

$b_p = 300\ mm$ – space between two ribs of the steel decking

Extreme number of shear studs, which can be placed inside the critical length is 14x2 = 28 studs. Full interaction of the composite slab and the steel section can't be reached. Level of interaction (η) should be determined.

$\eta = n/n_f = 28/34 \rightarrow \boldsymbol{\eta = 0,82}$

According to БДС EN1994-1-1, т. 6.6.1.2: $\eta_{min} = 1 - \left(\frac{355}{f_y}\right).(1,0 - 0,04.L_e)$

$\eta_{min} = 1 - \left(\frac{355}{275}\right).(1,0 - 0,04.8,80) = 0,16 \rightarrow \boldsymbol{\eta = 0,82 > \eta_{min} = 0,16}$ ***→ The shear studs are considered ductile!***

- *Bending resistance in plastic stage, when partial interaction is provided*

$M_{Rd} = M_{pl,a,Rd} + (M_{pl,Rd} - M_{pl,a,Rd}).\eta$

$M_{pl,a,Rd} = W_{pl,y}\ .f_y/\gamma_{M0} = 1307.27,5/1,05 = 34\ 236\ kNcm$

$M_{pl,Rd} = 68\ 702\ kNcm$

$M_{Rd} = 34\ 236 + (68\ 702 - 34\ 236).0,82 = 62\ 498\ kNcm \rightarrow \boldsymbol{M_{Rd} = 625\ kNm}$

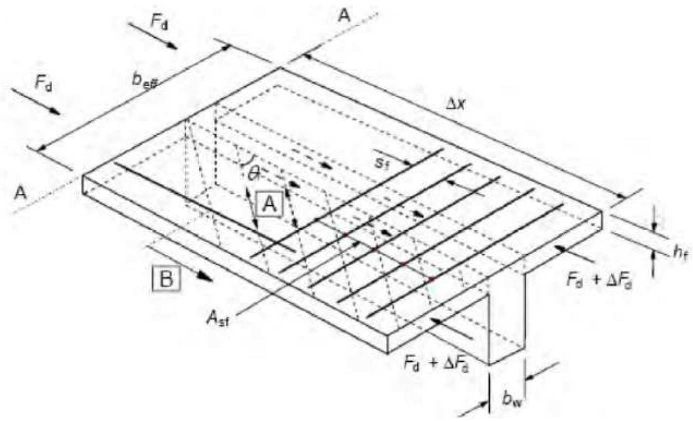
M_{Ed} / M_{Rd} = 470,9/625 = 0,75 → Requirement is satisfied!

3.2.6. Check for longitudinal shear

Check according to БДС EN1992-1-1, т. 6.2.4.

Contribution of the steel decking is not taken into consideration.

The check is made for section „a-a“, where reinforcement mesh *5N10* helps resisting the longitudinal shear. Reinforcement cross-section area:



$A_t = A_{sf}/s_f = 3{,}93\ \ cm^2/m$

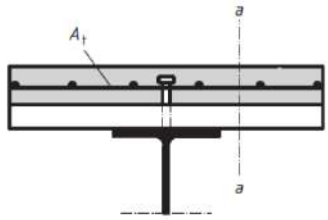
v_{Ed} – cross shear stress in the surface of interaction between the shear stud and the concrete slab;

$v_{Ed} = \Delta F_d/(2.h_f) = 436/(2.0{,}08) \rightarrow$ $\boldsymbol{v_{Ed} = 2\ 725\ kN/m^2}$

$\Delta F_d = 2.P_{Rd}/s = 2.65{,}4/0{,}3 = 436\ kN/m$ – longitudinal shear force;

$s = 0{,}3\ m$ – space between the ribs of the steel decking;

<u>To prevent crushing of the compression struts in the flange, the following condition should be satisfied:</u>



$v_{Ed} \leq v.f_{cd}.sin\theta.cos\theta = 0{,}6.2.sin45^\circ.cos45^\circ = 0{,}6\ kN/cm^2$

Прието: $\theta = 45^\circ$; $v = 0{,}6$

$\boldsymbol{v_{Ed} = 2\ 725\ kN/m^2 < 6000\ kN/m^2}$ → ***Condition is satisfied!***

<u>The transverse reinforcement per unit length may be determined as follows</u>

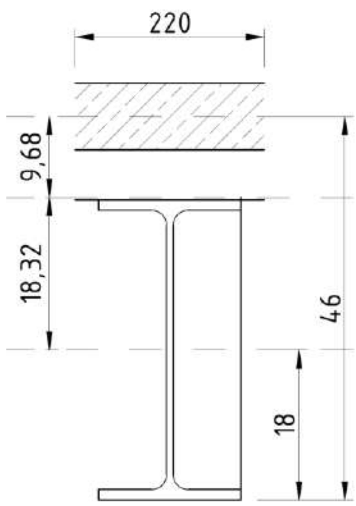
$(A_{sf}.f_{yd}/s_f) + A_{s1}.f_{yd}/s_f \geq v_{Ed}.h_f/cotg\theta$

A_{sf} и A_{sf1} – reinforcement in section „*a-a*“;

$(3{,}93.43{,}5) + (3.50{,}3/100).43{,}5 \geq 2\ 725.0{,}08/cotg45$ → ***236 kN/m > 218 kN/m → Condition is satisfied! → The reinforcement in the section is enough for resisting the longitudinal shear! No extra reinforcement is needed!***

3.2.7. Serviceability limit states

- *Composite section properties*



$n_0 = E_a/E_{cm} = 21000/3280 = 6{,}4$

$n_{eff} = 2.n_0 = 2.6{,}4 = 12{,}8$

$A_{c,eff} = A_c/n_{eff} = 1\ 760/12{,}8$ → $\boldsymbol{A_{c,eff} = 137{,}5\ cm^2}$

$A_c = b_{eff}.h_f = 220.8 = 1\ 760\ cm^2$

$I_{c,eff} = b_{eff}.h_c^3/(12.n_{eff}) = 220.8^3/(12.12{,}8)$ → $\boldsymbol{I_{c,eff} = 733{,}3\ cm^4}$

$A_{comb} = A_{c,eff} + A_a = 137{,}5 + 84{,}5 = 222\ cm^2$

$\boldsymbol{A_{comb} = 222\ cm^2}$

$z_{comb} = (A_a.z_a + A_{c,eff}.z_c)/A_{comb} = 38{,}58\ cm$

$I_{comb} = I_a + A_a.z_a^2 + I_{c,eff} + A_{c,eff}.z_c^2$

$I_{comb} = 23\,128{,}4 + 84{,}5.18{,}58^2 + 733{,}3 + 137{,}5.11{,}42^2$

$I_{comb} = 70\,965\ cm^4$

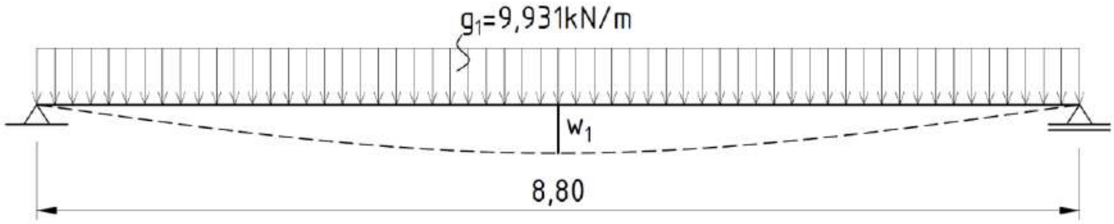
- *Deflection*

$w_{total} = w_1 + w_2 + w_3$

$w_1 = \frac{5}{384}.\frac{g_{k1}.L^4}{E_a.I_a} = \frac{5}{384}.\frac{9{,}6.10^{-2}.880^4}{21000.23\,128} = 1{,}54\ cm$

$g_1 = 9{,}024 + 0{,}571 = 9{,}6\ kN/m$

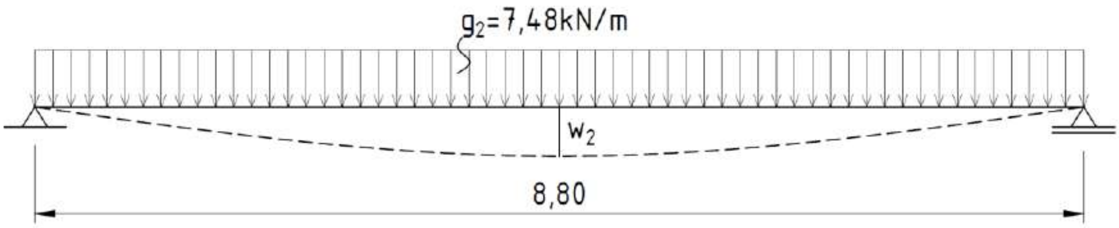
$w_1 = 1{,}54\ cm$



$w_2 = \frac{5}{384}.\frac{g_{k2}.L^4}{E_a.I_{comb}} = \frac{5}{384}.\frac{7{,}48.10^{-2}.880^4}{21000.70\,965} = 0{,}39\ cm$

$g_2 = 7{,}48\ kN/m$

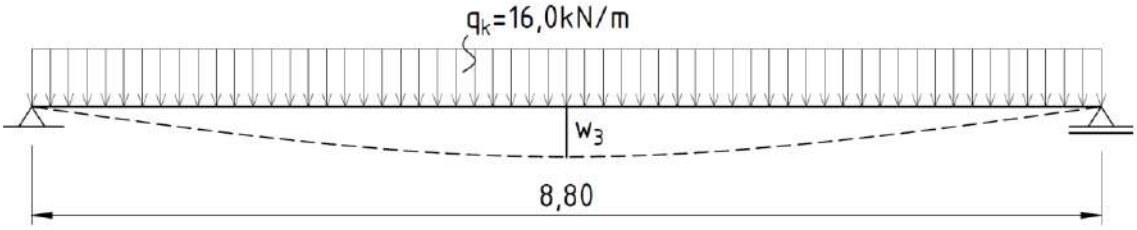
$w_2 = 0{,}39\ cm$



$w_3 = \frac{5}{384}.\frac{q_k.L^4}{E_a.I_{comb}} = \frac{5}{384}.\frac{16.10^{-2}.880^4}{21000.70\,965} = 0{,}84\ cm$

$q_k = 16\ kN/m$

$w_3 = 0{,}84\ cm$



$w_f = w_2 + w_3 = 0{,}39 + 0{,}84 = 1{,}23\ cm$

Increasing the deflection in second stage, because of the partial interaction:

$$w_p = w_f.\left(1+\beta.(1-\eta).\left(\frac{w_a}{w_f}-1\right)\right)$$

$w_f = 1{,}23\ cm$ – deflection in second stage

$w_a = = \frac{5}{384}.\frac{(g_{k2}+q_k).L^4}{E_a.I_a}$ – deflection in stage 2, without taking the contribution of the slab into account (steel section resists the deformations of it's own)

$w_a = \frac{5}{384}.\frac{23{,}48.10^{-2}.880^4}{21\,000.23\,128} \rightarrow w_a = 3{,}77\ cm$

$\beta = 0{,}3$

$\eta = 0{,}82$ – level of interaction;

$w_p = w_f.\left(1+0{,}3.(1-0{,}82).\left(\frac{3{,}77}{1{,}23}-1\right)\right) = 1{,}43\ cm$

Total deflection of the beam, taking the partial interaction into account:

$w_{total} = w_1 + w_p = 1{,}54 + 1{,}43 \rightarrow \mathbf{w_{total} = 2{,}97\ cm}$

$w_{max} = L/250 = 880/250 \rightarrow \mathbf{w_{max} = 3{,}52\ cm}$

$\mathbf{w_{total} = 2{,}97\ cm < w_{max} = 3{,}52\ cm}$ → ***Deflection check is satisfied!***

- *Control of the vibrations*

$f_1 \geq f_{1,SLS}$

$f_{1,SLS} = 4{,}0\ Hz$ – for slabs in buildings and parking and garages;

$f_1 \approx 18/\sqrt{\delta}\ [Hz]$ – frequency of first form of free oscillation (vertical surface) for simple beams;

$\delta = \frac{5}{384}.\frac{(g_k+0{,}2.q_k).L^4}{E_a.I_{comb}} = \frac{5}{384}.\frac{(9{,}93+7{,}48+0{,}2.16).10^{-2}.880^4}{21000.70\,964} \rightarrow \boldsymbol{\delta = 1{,}08\ cm = 10{,}8\ mm}$

$f_1 \approx 18/\sqrt{10{,}8} \rightarrow \mathbf{f_1 \approx 5{,}48\ Hz > f_{1,SLS} = 4{,}0\ Hz}$ → ***No unacceptable vibrations!***

4. Primary beam

The primary beams are considered to be composite and the connection between the plate and the beam is realized thanks to welded shear studs, resisting the sliding (horizontal shear) forces and resisting the shear force. They are loaded with the reactions of the secondary beams connected to them. The static scheme is a bilaterally elastically restrained beam, as the beams are part of frames, rigidly connected in the nodes. The calculation assumes a stiffness of the joint equal to the stiffness of the beam. The calculations are made in order to pre-select the cross sections for the beams.

A spring constant is accepted $k = EI_b = 21000.422075 = 8{,}864.10^9\ kN.cm^2$

The primary beam between axles *"7"* and *"8"* is considered. Length is $l = 12{,}80\ m$.

There are two stages of work considered, similar to the calculation of the composite slab. First stage – concrete placing stage, and Second stage – composite slab is resisting the loads (operational mode).

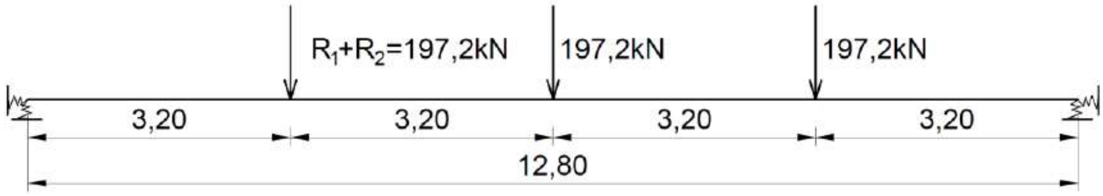
4.1. *Analyzing beam in first stage*

It is assumed that the load from the secondary beams and its own weight is realized on the beam. Reactions from secondary beams are transferred as action force on the main beam.

The ULS checks will be made for primary beam with cross-section HE900A and class S355JR!

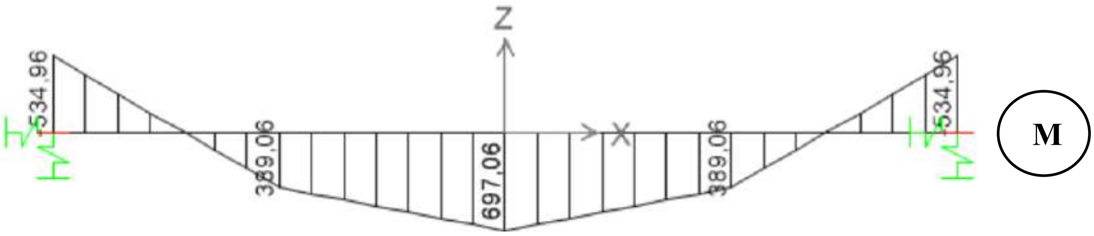
4.1.1. Loads

$R_1 = R_{IPE500} = 103{,}58\ kN$; $R_2 = R_{IPE400} = 78{,}25\ kN \rightarrow R = R_1 + R_2 = 103{,}58 + 78{,}25 = 181{,}83\ kN$

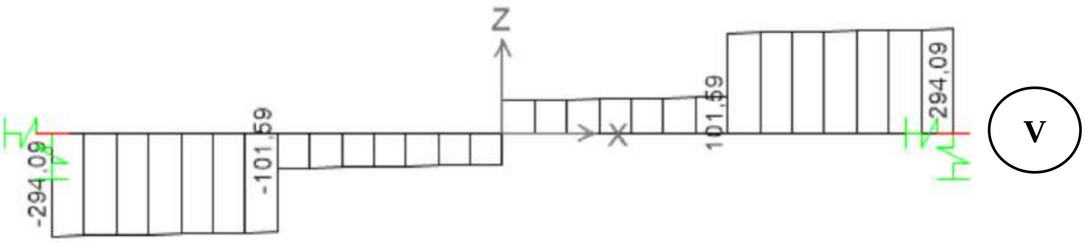


4.1.2. Inner forces

- *Design values of bending moment*



- *Design values of shear force*



4.1.1. Defining the steel section class

Steel class selected: S275 $\rightarrow \varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0{,}81$

- *Steel cross section characteristics*

h =	890,0	b2=	300,0	b =	300,0	ri=	30
tw=	16,0	tf2=	30,0	tf=	30,0	ro=	0

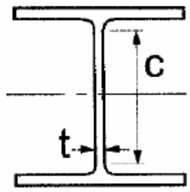
Инерционни характеристики

A = 320.526 cm	
Cz = 44.500 cm	Cy = 15.000 cm
Iy =422075.018 cm4	Iz = 13547.472 cm4
Wy = 9484.832 cm3	Wz = 903.165 cm3
Wply= 10811.038 cm3	Wplz= 1414.478 cm3
ry = 36.288 cm	rz = 6.501 cm
Avy = 180.000 cm2	Avz = 163.326 cm2
It = 760.075 cm4	Wt = 205.079 cm3

- *Slenderness of the web*

$\lambda = c/t = (h - 2.t_f - 2.r_i)/t_w = (890 - 2.30 - 2.30)/16 \rightarrow \lambda = 48{,}1$

$\boldsymbol{\lambda = 48{,}1 < 72.\varepsilon} = 72.0{,}81 = 58{,}3 \rightarrow$ *Cross section class 1*



- *Slenderness of the flanges*

$\lambda = c/t = (b/2 - t_w/2 - r_i)/t_f = (300/2 - 16/2 - 30)/30 \rightarrow \lambda = 3{,}73$

$\boldsymbol{\lambda = 3{,}73 < 9.\varepsilon} = 9.0{,}81 = 7{,}29 \rightarrow$ *Cross section class 1*

$\rightarrow$ ***The cross section is classified as class 1!***

4.1.2. Resistance of the cross-section

In order to increase safety checks are made in linear (elastic) state (elastic distribution of strains across the cross-section is considered)!

- *Bending resistance*

$M_{y,el,Rd} = \frac{W_{y,el}.f_y}{\gamma_{M0}} = \frac{9484{,}8.27{,}5}{1{,}05} = 248\,411\ kN.cm \rightarrow \boldsymbol{M_{y,el,Rd} = 2\,484\ kNm}$

$\frac{M_{y,Ed}}{M_{y,el,Rd}} \leq 1{,}0 \rightarrow \frac{697}{2484} = 0{,}28 < 1{,}0 \rightarrow$ ***Requirement is satisfied!***

- *Shear resistance*

$V_{el,Rd} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{163{,}3.27{,}5}{\sqrt{3}.1{,}05} \rightarrow \boldsymbol{V_{Rd} = 2\,469\ kN}$

$\frac{V_{z,Ed}}{V_{el,Rd}} \leq 1{,}0 \rightarrow \frac{294}{2\,469} = 0{,}12 < 1{,}0 \rightarrow$ ***Requirement is satisfied!***

4.1.1. Lateral torsional buckling resistance

Stiffeners of the decking are parallel to the axis of the beam, which does not improves the buckling resistance of the beam.

$\frac{M_{Ed}}{M_{b,Rd}} \leq 1{,}0 \rightarrow \quad M_{b,Rd} = \chi_{LT}.W_y.\frac{f_y}{\gamma_{M1}}$, where:

$W_y = W_{pl} = 6136{,}3\ cm^3 \rightarrow$ *Cross section class 1*

$\gamma_{M1} = 1{,}05$

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \beta.\overline{\lambda_{LT}^2}}} \leq \begin{cases} 1{,}0 \\ \frac{1}{\overline{\lambda_{LT}}^2} \end{cases}$$

$\Phi_{LT} = 0{,}5.[1 + \alpha_{LT}.(\overline{\lambda_{LT}} - \overline{\lambda}_{LT,0}) + \beta.\overline{\lambda_{LT}}^2]$, where: α_{LT} – imperfection factor

$\overline{\lambda}_{LT,0} = 0{,}4\ ;\ \beta = 0{,}75$

$h/b = 890/300 = 2{,}97 > 2{,}0 \rightarrow$ For welded I-sections the buckling curve is *c;*

$\rightarrow \alpha_{LT} = 0{,}49$

$\overline{\lambda_{LT}} = \sqrt{\frac{W_y.f_y}{M_{cr}}}$

The primary beam is stiffened from the secondary beam in transversal direction.

→ length between secondary beams: $L = 3{,}20\ m$ →

$\boldsymbol{M_{cr} = 1\ 101\ 300\ kN.cm}$ → Elastic critical moment as a result of analysis with LTBeamN.

$\overline{\lambda_{LT}} = 0{,}52;\ \ \Phi_{LT} = 0{,}69;\ \ \chi_{LT} = 0{,}87;\ \ \rightarrow\ \ M_{b,Rd} = 247\ 877\ kN.cm$

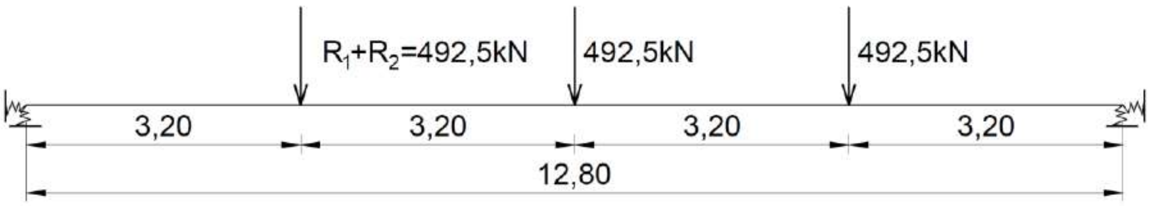
$M_{Ed} = 75\ 284\ kN.cm\ \ \rightarrow \frac{M_{Ed}}{M_{b,Rd}} = \frac{69\ 700}{247\ 877} = 0{,}28\ \ < 1{,}0$ → ***Requirement is satisfied!***

→ ***The beam will remain stable.***

4.2. Analyzing the beam in second stage – composite beam

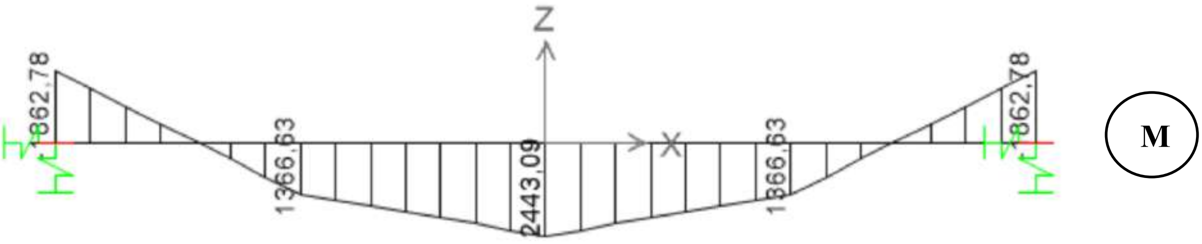
4.2.1. Defining the loads, affecting the beam

$R_1 = R_{IPE500} = 278{,}4\ kN\ ;\ R_2 = R_{IPE400} = 214{,}06\ kN \rightarrow R = R_1 + R_2 = 278{,}4 + 214{,}06 = 492{,}5\ kN$

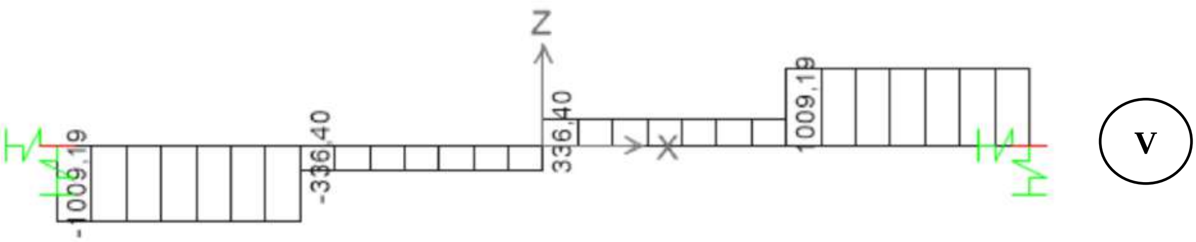


4.2.2. Design values of the bending moment and shear force

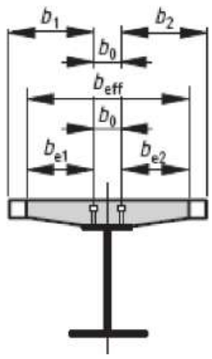
$\boldsymbol{M_{Ed}^{+} = 2\ 443\ kNm;\ M_{Ed}^{-} = 1\ 863\ kNm}$



$\boldsymbol{V_{Ed} = 1\ 009{,}2\ kN}$



4.2.3. Effective width



➢ *For positive bending moment*

$b_{eff} = b_o + \sum b_{ei}$; $L_e = 8{,}30\ m$ → Result from SAP2000.

$b_{ei} = L_e/8 \leq b_i \rightarrow b_{ei} = 8{,}30/8 \rightarrow b_{ei} = 1{,}04 \rightarrow b_{ei} = 104\ cm$

$b_1 = b_2 = b_{ei} = 104\ cm$; $b_0 = 0$; → $b_{eff} = 0 + 2.104 = 208\ cm$

$\boldsymbol{b_{eff} = 208\ cm}$

➢ *For negative bending moment*

$b_{eff} = b_o + \sum b_{ei}$; $L_e = 2.2{,}25 = 4{,}5\ m$ → Result from SAP2000.

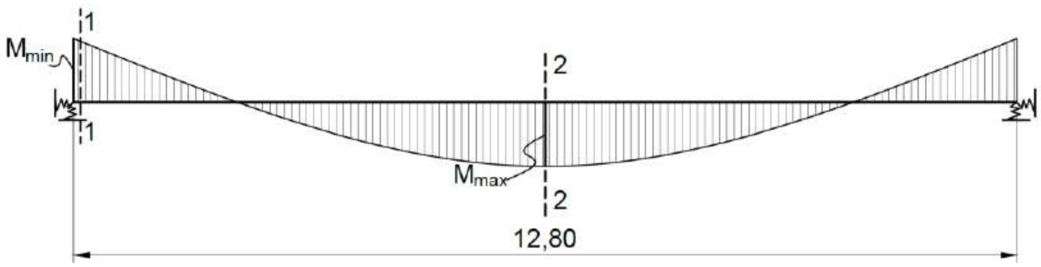
$b_{ei} = L_e/8 \leq b_i \rightarrow b_{ei} = 4{,}5/8 \rightarrow b_{ei} = 0{,}56 \rightarrow b_{ei} = 56\ cm$

$b_1 = b_2 = b_{ei} = 56\ cm$; $b_0 = 0$; → $b_{eff} = 0 + 2.56 = 112\ cm$

$\boldsymbol{b_{eff} = 112\ cm}$

4.2.4. ULS checks in critical sections

Resistance checks in the critical sections „1-1“ и „2-2“ have to be done. These are the sections with maximum shear force and maximum bending moment (positive and negative).



Critical sections define the so called „critical length“, which is equal to the distance between two critical sections → $l_{cr} = L/2 = 12{,}80/2$ → $\boldsymbol{l_{cr} = 6{,}40\ m}$

➢ *Bending resistance (section „2-2“)*

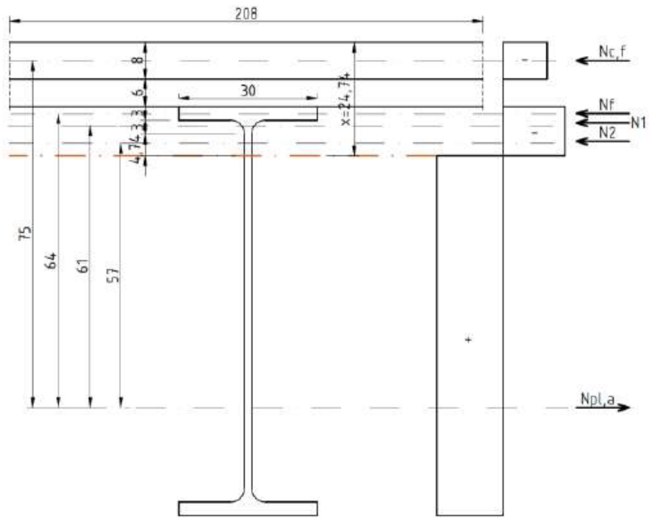
It is considered a full interaction between steel and concrete. Calculations in plastic stage:

$N_{c,f} = 0{,}85.f_{cd}.b_{eff}.h_c = 0{,}85.2{,}0.208.8$ → $\boldsymbol{N_{c,f} = 2\ 828{,}8\ kN}$

$N_{pl,a} = A_a.f_{yd} = 320{,}5.27{,}5/1{,}05$ → $\boldsymbol{N_{pl,a} = 8\ 394\ kN}$

$\boldsymbol{N_{pl,a} > N_{c,f}}$ → ***“Zero line” is located in the slab!***

Definition of the "zero line" location.:



$\Delta N = N_{pl,a} - N_{cf} = 8\,394 - 2\,828,8$

$\rightarrow \Delta N = 5\,565,2\ kN \rightarrow \Delta N/2 = 2\,782,6\ kN$

$N_f = b_f.t_f.f_{yd} = 30.3.27,5/1,05 = 2\,357\ kN$

$N_1 = A_1.f_{yd} = 8,66.\ 27,5/1,05 = 226,8\ kN$

$N_2 = \Delta N/2 - N_f - N_1 = 2\,782,6 - 2\,357 - 226,8$

$N_2 = 198,8\ kN$

$x_2 = \frac{N_2}{t_w \cdot f_{yd}} = \frac{198,8.1,05}{1,6.27,5} \rightarrow x_2 = 4,74\ cm$

$x = h_p + h_c + t_f + R + x_2 = 6+8+3+3+4,74$
$\rightarrow$ **$x = 24,74$ cm**

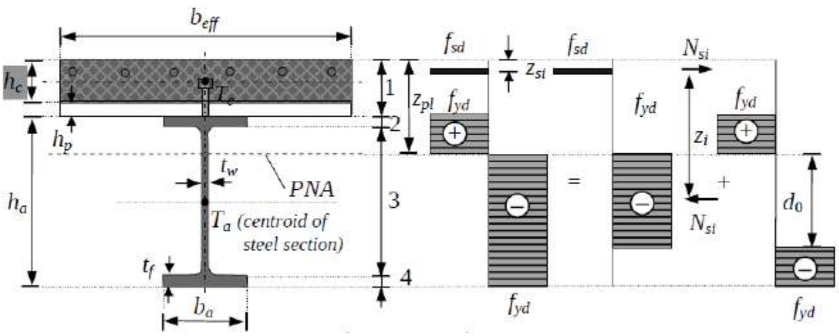
$M_{pl,Rd} = N_{cf}.z_{cf} + N_f.z_f + N_1.z_1 + N_2.z_2 =$

$2\,828,8.75+2\,357.64+226,8.61+198,8.57$

$M_{pl,Rd} = 3\,882$ kNm

$M_{Ed}/M_{pl,Rd} = 2443/3\,882 = 0,63 \rightarrow$ *Requirement is satisfied!*

- *Bending resistance (section „1-1")*



$d_0 = N_s/(t_w.f_{yd})$

$N_s = A_{sl}.f_{sd}$; $A_{sl} = A_s.b_{eff}$; $A_s = 393\ mm^2/m$ – мрежа 5N10; $b_{eff} = 1,12\ m$

$A_{sl} = 393.1,12 = 440,16\ mm^2 \rightarrow N_s = 440,16.435 = 192\,270\ N \rightarrow$ **$N_s = 192,3$ kN**

$d_0 = 192,3.1,05/(1,6.27,5) \rightarrow$ **$d_0 = 4,59$ cm**

$z_{pl} = h_c + h_p + h_a/2 - d_0/2 = 8+6+89/2 - 4,59/2 \rightarrow$ **$z_{pl} = 56,2$ cm**

$z_i = h_c + h_p + h_a/2 - z_{si} = 8+6+89/2 - 3 \rightarrow$ **$z_{pl} = 55,5$ cm**

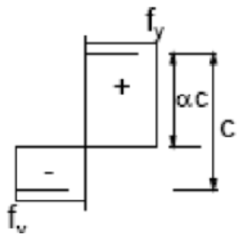
$M_{pl,Rd} = N_{sl}.z_i + M_{pl,a,Rd} - f_{yd}.t_w.d_0^2/2 = 192,3.55,5 + 283\,150 - 27,5.1,6.4,59^2/(2.1,05)$

$M_{pl,a,Rd} = 283\,150\ kNcm$

$M_{pl,Rd} = 293\,381\ kNcm \rightarrow$ **$M_{pl,Rd} = 2\,933,8$ kNm $> M_{Ed}^- = 1\,862$ kNm**

Deffinition of the web class:

$$\alpha = \frac{1}{c}.\left(\frac{h_a}{2} + \frac{1}{2}.\frac{N_{Ed}}{t_w.f_y} - (t_f + r)\right)$$



c = d = 77 cm; h_a *= 89 cm;*

$N_{Ed} = N_s = 192{,}3$ *kN;* $t_w = 1{,}6$ *cm;* $t_f = 3$ *cm; r = 3 cm;*

$$\alpha = \frac{1}{77}.\left(\frac{89}{2} + \frac{1}{2}.\frac{192{,}3.1{,}05}{1{,}6.27{,}5} - (3+3)\right) = 0{,}53 \rightarrow \alpha > 0{,}5 \rightarrow$$

За клас 1: $\frac{c}{t} \leq \frac{369.\varepsilon}{13.\alpha - 1} \rightarrow \frac{77}{1{,}6} = 48{,}1 \leq \frac{369.0{,}81}{13.0{,}53-1} = 53{,}1 \rightarrow$ ***48,1 < 53,1 → Web is class 1!***

- *Shear resistance*

$$V_{Rd} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{A_v.f_y}{\sqrt{3}.\gamma_{M0}} = \frac{163{,}3.27{,}5}{\sqrt{3}.1{,}05} \rightarrow \mathbf{V_{Rd} = 2\,469\ kN}$$

$\frac{V_{z,Ed}}{V_{el,Rd}} \leq 1{,}0 \rightarrow \frac{1\,009{,}2}{2\,469} = 0{,}4 < 1{,}0 \rightarrow$ ***Requirement is satisfied!***

$h_w/t_w = 890/16 = 55{,}6$ – slenderness of the web;

$h_w/t_w \leq 72.\varepsilon/\eta = 72.0{,}81/1{,}00 = 58{,}3$

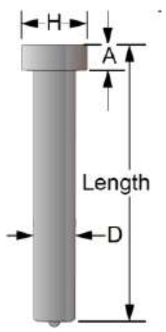
$\eta = 1{,}00$ – for steel S275;

→ $\mathbf{h_w/t_w = 55{,}6 < 58{,}3}$ → ***Requirement is satisfied!***

Shear buckling resistance of web check is not necessary!

4.2.5. Design of shear studs

Shear studs resist the longitudinal shear (sliding forces) and prevents longitudinal splitting. In vertical direction the shear studs should resist the separation forces, which are trying to separate the composite slab from the steel section.



Shear studs KÖCO SD 19x100 have the following characteristics:

D = d = 19 mm

$L = h_{sc} = 100$ *mm*

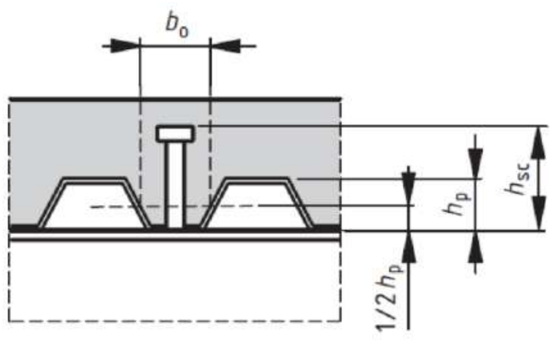
H = 32 mm

$f_u = 450$ *MPa*

- *Shear stud connection resistance*

$$P_{Rd} = min\begin{cases}\dfrac{0{,}8.f_u.\pi.D^2/4}{\gamma_v}\\ \dfrac{0{,}29.\alpha.D^2.\sqrt{f_{ck}.E_{cm}}}{\gamma_v}\end{cases} =$$

$$min\begin{cases}\dfrac{0{,}8.45.\pi.1{,}9^2/4}{1{,}25}\\ \dfrac{0{,}29.1{,}00.1{,}9^2.\sqrt{3.3280}}{1{,}25}\end{cases} = min\begin{cases}81{,}7\\83{,}1\end{cases} \rightarrow \mathbf{P_{Rd} = 81{,}7\ kN}$$



When $h_{sc}/d \geq 4 \rightarrow \alpha = 1{,}0$

$k_l = 0{,}6.\frac{b_0}{h_p}.\left(\frac{h_{sc}}{h_p} - 1\right) = 0{,}6.\frac{140}{60}.\left(\frac{100}{60} - 1\right) = 0{,}93$

$b_0 = 140\ mm$

$n = 2$ – two shear studs in every rib;

$k_l = 0{,}93 < 1{,}0$

$P_{Rd} = k_l.P_{Rd} = 0{,}93.81{,}7 = 76\ kN$

$\boldsymbol{P_{Rd} = 76\ kN}$

In order to provide full interaction, the number of the studs in the critical length should be:

$n_f \geq min\{N_{c,f}\ ;N_{pl,a}\}/P_{Rd} \rightarrow n_f \geq min\{2\ 828;\ 8\ 394\}/76 = 2\ 828/66{,}3 \approx 38$ *бр. дюбела*

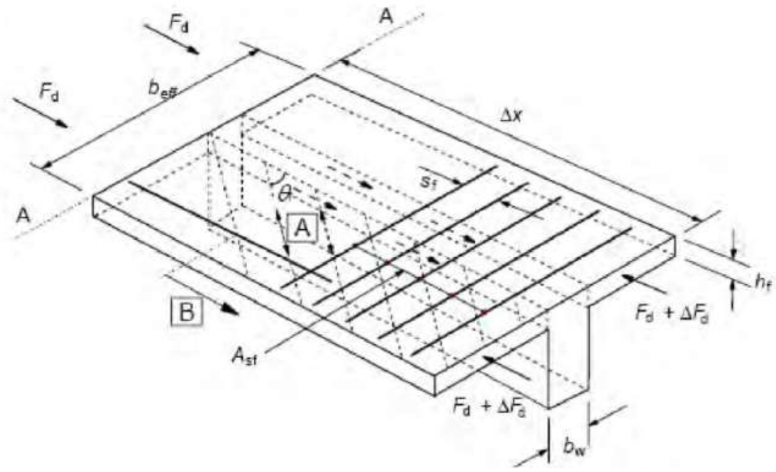
→ $\boldsymbol{n_f = 38}$ ***бр. дюбела/l_{cr}***

$n = l_{cr}/b_p = 6400/100 = 64$ *бр*$/l_{cr}$ – the extreme number of the shear studs, which can be placed along the beam

Extreme number of shear studs, which can be placed inside the critical length is 64 бр. Full interaction of the composite slab and the steel section is reached!

4.2.6. Check for longitudinal shear

Check according to БДС EN1992-1-1, т. 6.2.4.



Contribution of the steel decking is not taken into consideration.

The check is made for section „a-a", where reinforcement mesh *5N10* helps resisting the longitudinal shear. Reinforcement cross-section area:

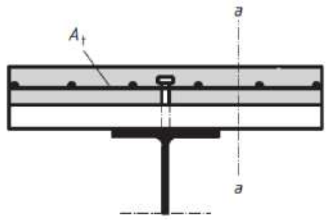
$A_t = A_{sf}/s_f = 3{,}93\ \ cm^2/m$

v_{Ed} – cross shear stress in the surface of interaction between the shear stud and the concrete slab;

$v_{Ed} = \Delta F_d/(2.h_f) = 934/(2.0{,}08) \rightarrow \boldsymbol{v_{Ed} = 5839\ kN/m^2}$

$\Delta F_d = 2.P_{Rd}/s = 2.65{,}4/0{,}14 = 934\ kN/m$ – longitudinal shear force;

$s = 0{,}14\ m$ – space between the ribs of the steel decking;



<u>To prevent crushing of the compression struts in the flange, the following condition should be satisfied:</u>

$v_{Ed} \leq v.f_{cd}.sin\theta.cos\theta = 0{,}6.2.sin45^\circ.cos45^\circ = 0{,}6\ kN/cm^2$

Приeто: $\theta = 45^\circ$; $v = 0{,}6$

$\boldsymbol{v_{Ed} = 5893\ kN/m^2 < 6000\ kN/m^2}$ → ***Requirement is satisfied!***

The transverse reinforcement per unit length may be determined as follows:

$(A_{sf}.f_{yd}/s_f) \geq v_{Ed}.h_f/cotg\theta$

A_{sf} – reinforcement in section „*a-a*“;

$(3{,}93.43{,}5) \geq 5839.0{,}08/cotg45$ → ***147 kN/m < 467 kN/m*** → ***The reinforcement in the section is not enough for resisting the longitudinal shear! Additional reinforcement is needed!***

In the area of primary beams (at effective width b_{eff}) between the reinforcement bars of the main mesh is provided additional reinforcement 5N14/m!

Check: $A_{sf}.f_{yd}/s_f) + A_{s1}.f_{yd}/s_f \geq v_{Ed}.h_f/cotg\theta \rightarrow$

$(3{,}93.43{,}5) + (5.153{,}9/100).43{,}5 \geq 467$ *kN/m* → ***505 kN/m > 467 kN/m***

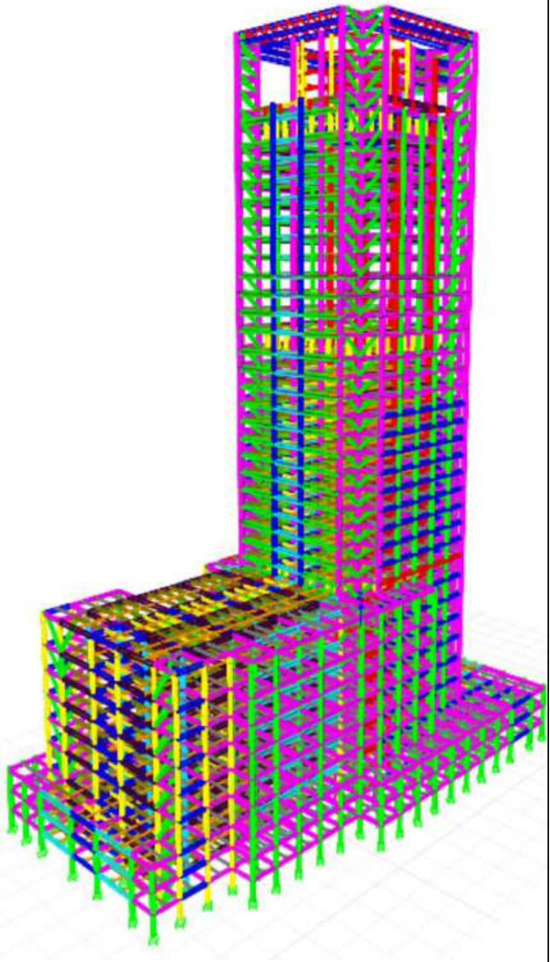
→ ***Requirement is satisfied!***

IV. Analysis model with ETABS

The construction of the building is dual system. Two systems work to resist the forces of seismic impact – moment - resisting frames and eccentrically braced frames. The energy is dissipated both from the plastic hinges, formed at the ends of the beams of the MRF, and from the plastification in the link element, part of the EBF.

Due to the complexity of the structural system, the unclarity regarding the distribution of forces, as well as the lack of regularity in plan and height, a 3D model has been prepared, which gives a clearer idea of the behavior of the structure

1. Modeling the construction



The analysis model of the structure is made with the ETABS program. It consists entirely of frame - elements interconnected rigidly. Exceptionally the secondary beams, which are connected to the primary beams (beams of the MRFs) by hinges. The structure is fixed at Base3 level (foundation slab level). The lateral displacements of the underground floor diaphragms are limited by linear supports in the respective directions. Supports are defined in each node of the peripheral underground structure. In this way, the behavior of a rigid "box"-type basement is simulated.

The floor constructions are modeled as rigid diaphragms on each floor level, which is imposed by the premise that the floor slabs absorb and distribute the inertial forces between the vertical elements. However, with such modeling the beams of the floor diaphragm do not absorb axial forces, which could lead to unrealistic results and behavior. In order to take into account and check the action of the axial forces in the beams, an additional model has been made, in which the floor constructions are deformable and the diaphragm behavior is not taken into account.

1.1.<u>Materials</u>

1.1.1. Construction steel

- Beams of the MRF – S355
- Secondary beams – S275
- Link elements – S355/S275
- Braces – S355
- Columns – Histar S460
- For some support elements – S235

1.1.2. Concrete – C30/37

1.2.<u>Cross - sections</u>

The cross-sections of the elements are selected on the basis of preliminary analysis, using 2D frames along the axis „B” и “G”, „C” и “F” и „D” и “E”. According to the results of the analysis and the performed ULS checks, cross-sections for the 3D model have been selected.

1.3. Loads

1.3.1. Vertical loads

Vertical loads of the slab, together with all finishing works and installations described in point II. – 2., as well as the operational load and the snow load, are presented as a linear loads, distributed on the beams in the direction of the numerical axes. The area load is distributed between the beams, based on the deductible width of the slab.

1.3.2. Horizontal wind load

Presented in item *II. - 5.2.* wind loads for every zone of the structure are surface loads. In order to avoid the modeling of shell elements, which would unnecessarily complicate the model, the area wind load is distributed between the floor levels and transformed into a linear distributed load. Such approach is allowed due to the way of distribution of the loads from the facade construction. The considered solution consists of vertical load-bearing facade elements, supported on each floor level and distributing the wind load between the floor diaphragms.

For this purpose, the intensity of the wind load was determined for each elevation. Rigid floor diaphragms distribute the wind load between the individual frames in proportion to their stiffness.

2. Modal analysis

2.1. Seismic mass

The inertial forces from the calculated seismic impact are determined after defining the masses of the structure. The masses are bound up with the gravity loads in the structure and according to БДС EN 1998-1, т. 3.2.4. the combination of loads is:

$$\Sigma G_{k,j} \text{ “+” } \Sigma \psi_{E,i} . Q_{k,i}$$

Where:

$\psi_{E,i} = \varphi . \psi_{2,I};$

The coefficients take into account the probability that the temporary loads (live, snow, wind, etc.) will not act on the whole structure during seismic impact.

$\psi_{2,i}$ – combination coefficient, according to A1 на EN 1990:2002

Values for φ are given in the following table:

Вид на временното натоварване	Етаж	φ
Категории A-C*	Покрив	1,0
	Етажи с взаимнозависимо обитаване	0.8
	Независимо обитавани етажи	0,5
Категории D-F* и архиви		1,0

$\varphi_{roof} = 1,0$

$\varphi_{Q} = 0,5$

Въздействия	ψ_0	ψ_1	ψ_2
Експлоатационни натоварвания в сгради от категория (виж EN 1991-1-1):			
Категория A: жилищни сгради	0,7	0,5	0,3
Категория B: административни сгради и офиси	0,7	0,5	0,3
Категория C: участъци, в които е възможно струпване на хора	0,7	0,7	0,6
Категория D: търговски помещения	0,7	0,7	0,6
Категория E: складови помещения	1,0	0,9	0,8
Категория F: участъци за преминаване на превозни средства с тегла до 30 kN	0,7	0,7	0,6
Категория G: участъци за преминаване на превозни средства с тегла над 30 kN, но не повече от 160 kN	0,7	0,5	0,3
Категория H: недостъпни (освен за обичайно поддържане и ремонт) покриви	0,6	0,2	0
Категория K: плоски покриви с площадки за кацане на вертолети:			
- за вертолетите върху покривите	1,0	0,9	0,5
- за другите натоварвания върху покрива (товари, персонал, съоръжения)	0,7	0,5	0,3
Натоварвания от сняг върху сгради (виж EN 1991-1-3):			
- на терени с височина над морското равнище до 1 000 m	0,5	0,2	0
- на терени с височина над морското равнище над 1 000 m	0,7	0,5	0,2
Въздействия от вятър върху сгради (виж EN 1991-1-4)	0,6	0,2	0
Температурни въздействия (без пожар) в сгради (виж EN 1991-1-5)	0,6	0,5	0
ЗАБЕЛЕЖКА 1: Когато в една сграда има участъци от различни категории по натоварване, които не могат да бъдат ясно разграничени, се приемат тези стойности на коефициентите ψ, които водят до най-неблагоприятен резултат. ЗАБЕЛЕЖКА 2: За категория I (достъпни покриви на сгради от категории A – D, виж EN 1991) се приемат същите стойности на коефициентите ψ, както са за самата сграда, а ако в нея има помещения от няколко категории, приема се най-неблагоприятната група стойности на коефициентите ψ.			

$\psi_{2,Q} = 0,3$

$\psi_{2,S} = 0$

$\psi_{2,roof} = 0,3$

- Combination coefficients

$\psi_{E,Q} = \varphi.\psi_{2,Q} = 0,5.0,3 = 0,15$

$\psi_{E,S} = \varphi.\psi_{2,S} = 1,0.0 = 0$

$\psi_{E,Qroof} = \varphi.\psi_{2,Qroof} = 1,0.0,3 = 0,3$

$\rightarrow \boldsymbol{\Sigma G_{k,j}}$ **“+” $\boldsymbol{\Sigma 0,15.Q_{k,i}}$ “+” $\boldsymbol{\Sigma 0,3.Q_{k,roof}}$**

2.2. Effective modal masses

According to БДС EN 1998-1, т. 4.3.3.3. calculations must include every oscillation form, which contribute to the general behavior of the building. This requirement is satisfied if the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure. Condition should be satisfied in every direction (X и Y).

For the current construction requirement is satisfied and it could be proved with the results of the modal analysis, which are given in the following table:

TABLE: Modal Participating Mass Ratios					
Mode	**Period**	**UX**	**UY**	**Sum UX**	**Sum UY**
	[sec]	[%]	[%]	[%]	[%]
1	4,16	0,23	65,53	0,23	65,53
2	3,76	67,29	0,26	67,51	65,79
3	2,50	0,03	2,22	67,54	68
4	1,36	0,00	15,26	67,55	83,27
5	1,31	19,59	0,01	87,14	83,28
6	1,08	0,08	6,30	87,22	89,58
7	0,76	0,48	3,47	87,7	93,05
8	0,76	5,39	0,20	93,09	93,25
9	0,70	0,00	0,00	93,09	93,26
10	0,52	0,57	0,90	93,66	94,16
11	0,52	1,19	0,37	94,86	94,52
12	0,48	0,00	0,03	94,86	94,55

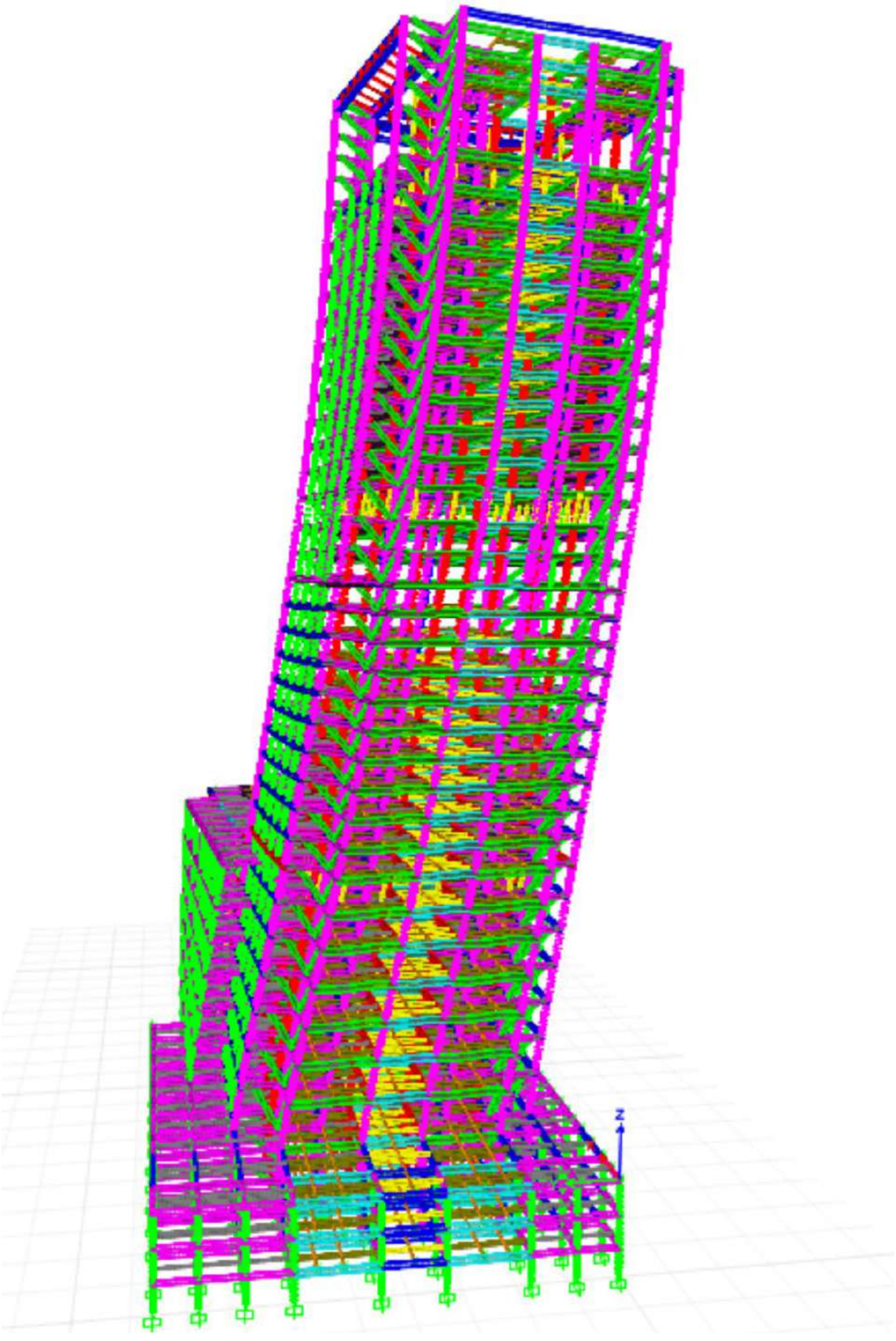
This table shows that the sum of the effective modal masses for both of the main directions exceeds 90% still at 8^{th} form of modal response.

2.3.Modal responses of the structure

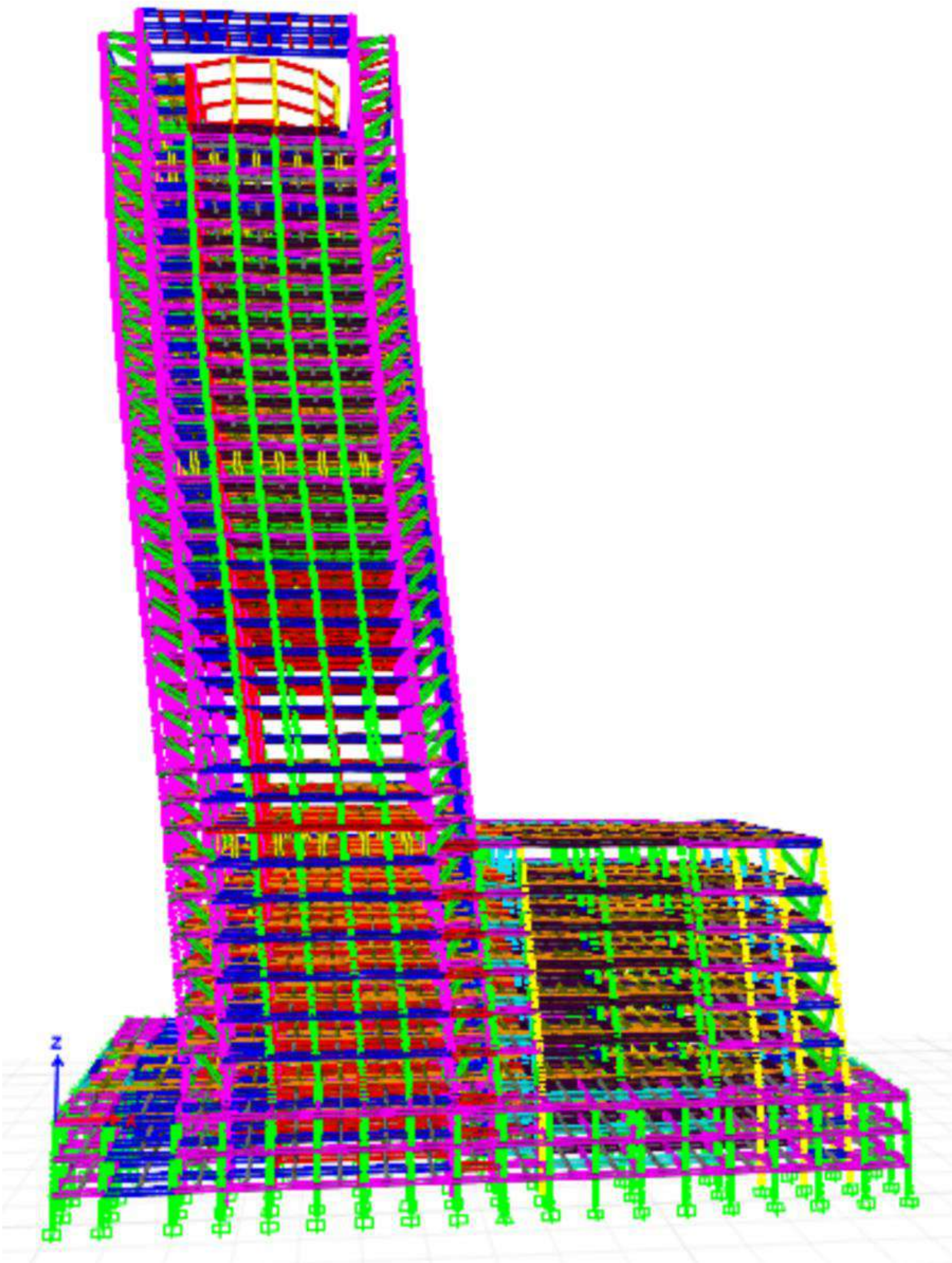
Vibration periods and frequencies for each mode, which take part in the analysis are given in the following table. The first three modes go with graphical representation.

TABLE: Modal Periods and Frequencies		
Mode	**Period**	**Frequency**
	sec	cyc/sec
1	4,16	0,24
2	3,76	0,27
3	2,50	0,40
4	1,36	0,74
5	1,31	0,76
6	1,08	0,92
7	0,76	1,31
8	0,76	1,32
9	0,70	1,44
10	0,52	1,91
11	0,52	1,93

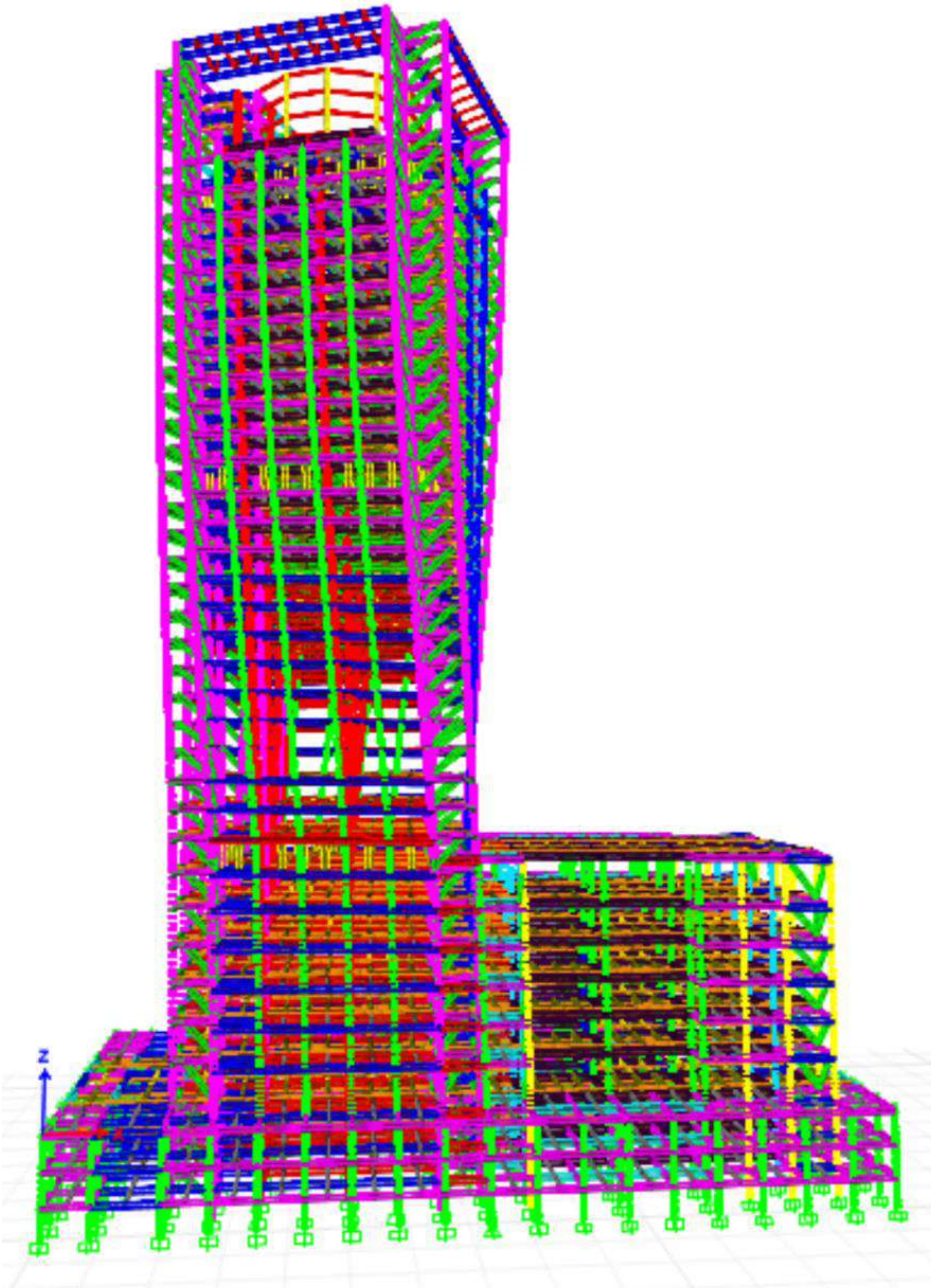
➢ **First mode** – translation parallel to Y - T_1 = 4,16 s



- **Second mode** – translation parallel to X – T_2 = 3,76 s



- **Third mode** – rotation – T_3 = 2,50 s



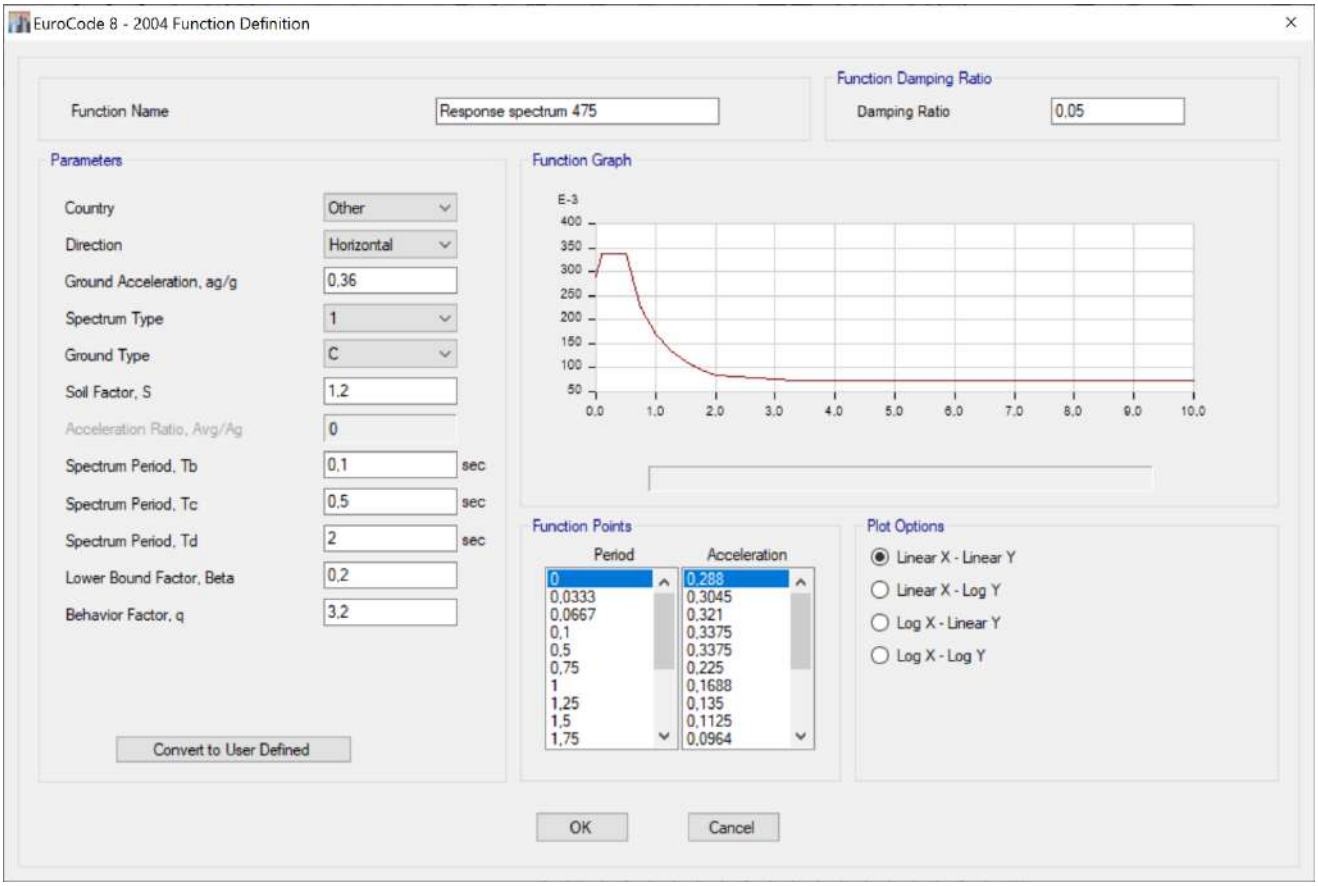
3. Linear spectral analysis

3.1.<u>Response spectrum</u>

To define the earthquake motion in the 3D model of the structure are used two types of design response spectrum with different characteristics.

First response spectrum is define to satisfy the no-collapse requirement (ULS requirements). It refers to a seismic motion with reference return period of 475 years. According to БДС EN 1998 – 1, т. 2.1., construction shall withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seisll1ic events.

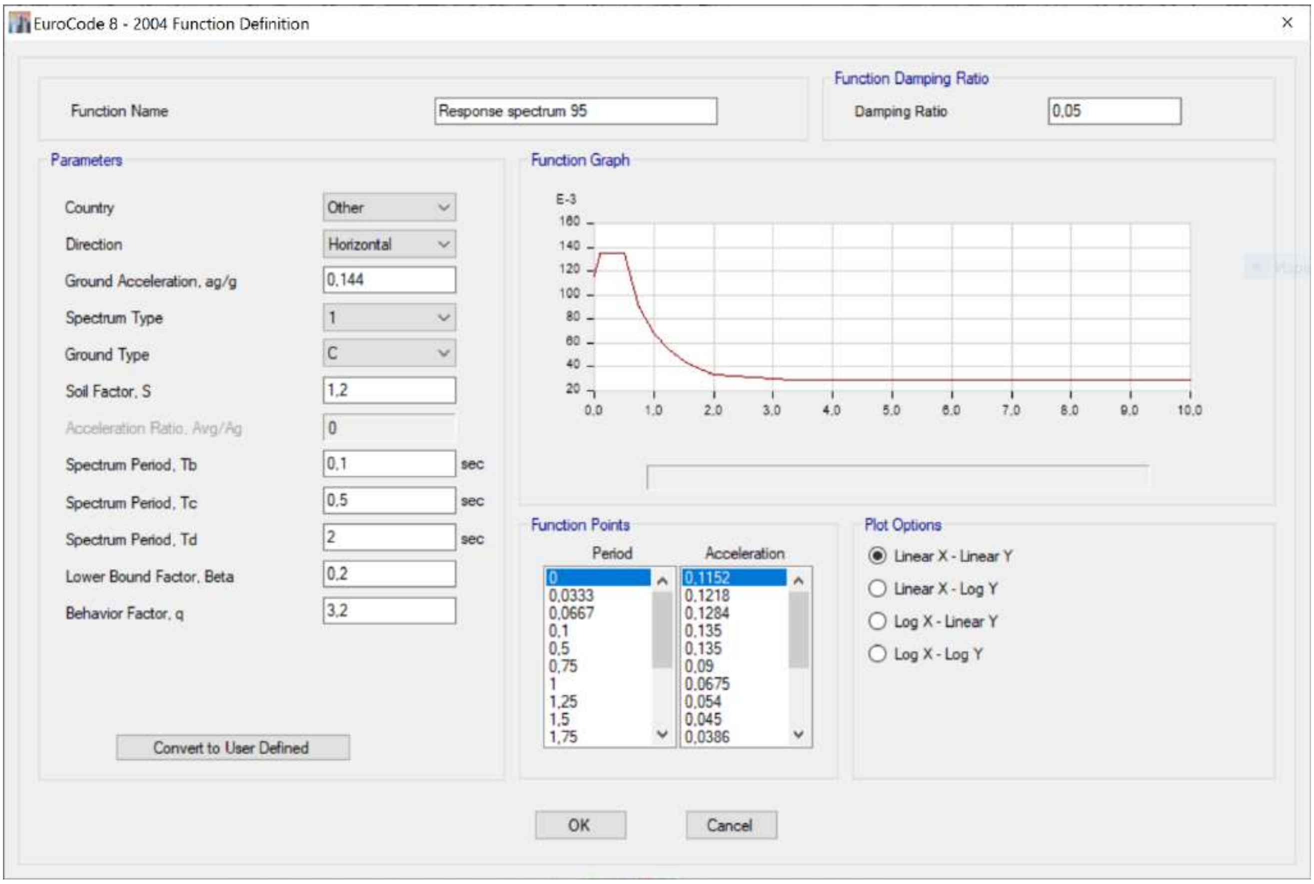
<u>Definition of the response spectrum for strong earthquake</u>



Second response spectrum is defined to satisfy the damage limitation requirement. It refers to a seismic motion with reference return period of 95 years. According to БДС EN 1998 – 1, т. 2.1., the structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action, without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself.

Characteristics of the defined spectrum are calculated in detail and given in point *II* of this thesis.

Definition of the response spectrum for weak earthquake



3.2. Definition of the spectral method for taking into account the accidental eccentricity

After the definition of the response spectrum in the program it is defined Load Case SEISMIC, in which the seismic action is defined for both of the main directions. Displacements at every point of the structure should then be calculated combining the translational and rotational displacements using CQC. Combination of the horizontal components of the seismic action should be done as a sum of the squares (SRSS).

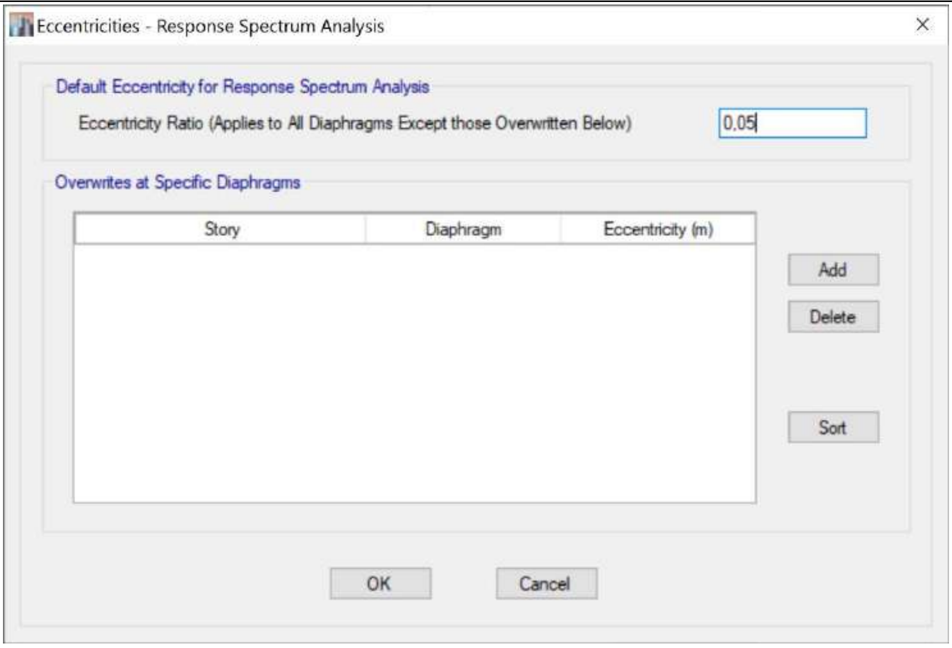
According to БДС EN 1998 – 1, т. 4.3.3.3.2., the response based on two vibration modes may be considered independent if their vibration periods satisfy the following requirement:

$T_j \leq 0{,}9.T_i$, when $T_j < T_i$

Modes with minimal difference between periods are considered. These are the vibration period of first mode $T_1 = 4{,}16\ s$ and the vibration period of second mode $T_2 = 3{,}76\ s$.

$T_2 \leq 0{,}9.T_1 \rightarrow 3{,}76 \leq 0{,}9.4{,}16 \rightarrow 3{,}76 > 3{,}74 \rightarrow$ *Requirement is not satisfied and the use of more precise methods is necessary! CQC is used to define the peaks!*

Torsion effects due to accidental eccentricities of the mass of each elevation can be automatically detected by the program ETABS.



4. Load combination

4.1. *Ultimate limit states*

The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions:

$\boldsymbol{\Sigma\gamma_{G,i} . G_{k,i} + \gamma_{Q,1} . Q_{k,1} + \Sigma\gamma_{Q,i} . \psi_{0,i} . Q_{k,I}}$

Where:

$\gamma_{G,i} = 1{,}35$ – safety coefficient for dead loads;

$\gamma_{Q,i} = 1{,}50$ – safety coefficient for live loads;

$\psi_{0,i}$ – combination coefficient, according to A1 на EN 1990: 2002

For live loads in buildings category „B“ – administration and office buildings: $\psi_0 = 0{,}7$

For wind loads: $\psi_0 = 0{,}6$

For snow loads of buildings, located below 1000m over sea level: $\psi_0 = 0{,}5$

For roof loads: $\psi_0 = 0{,}7$

4.1.1. When live load is the main load

ULS 1: $1{,}35.G_k + 1{,}5.Q + 1{,}5.Q_{roof}$

ULS 2: $1{,}35.G_k + 1{,}5.Q + 1{,}5.Q_{roof} + 1{,}5.0{,}6.W_x$

ULS 3: $1{,}35.G_k + 1{,}5.Q + 1{,}5.Q_{roof} + 1{,}5.0{,}6.W_y$

ULS 4: $1{,}35.G_k + 1{,}5.Q + 1{,}5.0{,}7.Q_{roof} + 1{,}5.0{,}6.W_x + 1{,}5.0{,}5.S$

ULS 5: $1{,}35.G_k + 1{,}5.Q + 1{,}5.0{,}7.Q_{roof} + 1{,}5.0{,}6.W_y + 1{,}5.0{,}5.S$

4.1.2. When wind load is the main load

ULS 6: $1{,}35.G_k + 1{,}5.W_x + 1{,}5.0{,}7.Q + 1{,}5.0{,}7.Q_{roof}$

ULS 7: $1{,}35.G_k + 1{,}5.W_y + 1{,}5.0{,}7.Q + 1{,}5.0{,}7.Q_{roof}$

ULS 8: $1{,}35.G_k + 1{,}5.W_x + 1{,}5.0{,}7.Q + 1{,}5.0{,}7.Q_{roof} + 1{,}5.0{,}5.S$

ULS 9: $1{,}35.G_k + 1{,}5.W_y + 1{,}5.0{,}7.Q + 1{,}5.0{,}7.Q_{roof} + 1{,}5.0{,}5.S$

Combination with snow load as main load is less likely to occur, than a combination with extreme live load as a leading load. This is why it won’t be considered.

4.2. *Accidental design situation*

$\boldsymbol{\Sigma G_{k,i} + A_{Ed} + \Sigma\psi_{2,i}.Q_{k,i}}$

Where:

$\psi_{2,i}$ – combination coefficient, according to A1 of EN 1990: 2002

For live loads in buildings category „B“ – administration and office buildings: $\psi_2 = 0{,}3$

For wind loads: $\psi_2 = 0$

For snow loads of buildings, located below 1000m over sea level: $\psi_2 = 0$

For roof loads at operating roofs: $\psi_2 = 0{,}3$

Defined combination in ETABS - **Accidental:** $G + SEISMIC + 0{,}3.Q + 0{,}3.Q_{roof}$

4.3. *Serviceability limit states*

4.3.1. Characteristic combination

$\boldsymbol{\Sigma G_{k,I} + Q_{k,1} + \Sigma\psi_{0,i}.Q_{k,i}}$

SLS 1: $G_k + Q_k + Q_{roof}$

SLS 2: $G_k + Q_k + Q_{roof} + 0{,}6.W_x$

SLS 3: $G_k + Q_k + Q_{roof} + 0{,}6.W_y$

SLS 4: $G_k + Q_k + Q_{roof} + 0{,}6.W_x + 0{,}5.S$

SLS 5: $G_k + Q_k + Q_{roof} + 0{,}6.W_y + 0{,}5.S$

SLS 6: $G_k + W_x + 0{,}7.Q_k + 0{,}7.Q_{roof}$

SLS 7: $G_k + W_y + 0{,}7.Q_k + 0{,}7.Q_{roof}$

SLS 8: $G_k + W_x + 0{,}7.Q_k + 0{,}7.Q_{roof} + 0{,}5.S$

SLS 9: $G_k + W_y + 0{,}7.Q_k + 0{,}7.Q_{roof} + 0{,}5.S$

4.3.2. Frequent combination

$\boldsymbol{\Sigma G_{k,I} + \psi_{1,1}.Q_{k,1} + \Sigma\psi_{2,i}.Q_{k,i}}$

Вхере:

$\psi_{1,i}$ - combination coefficient, according to A1 на EN 1990: 2002

For live loads in buildings category „B“: $\psi_1 = 0,5$

For wind loads: $\psi_1 = 0,2$

or snow loads of buildings, located below 1000m over sea level: $\psi_1 = 0,2$

For roof loads at operating roofs: $\psi_1 = 0,5$

SLS 10: $G_k + 0,5.Q_k + 0,5.Q_{roof}$

SLS 11: $G_k + 0,2.W_x + 0,3.Q_k + 0,3.Q_{roof}$

SLS 12: $G_k + 0,2.W_y + 0,3.Q_k + 0,3.Q_{roof}$

4.3.3. Quasi-permanent combination

$\boldsymbol{\Sigma G_{k,i} + \Sigma \psi_{2,i}.Q_{k,i}}$

SLS 13: $G_k + 0,3.Q_k + 0,3.Q_{roof}$

5. Limitation of interstorey drifts

According to БДС EN 1998 – 1, т. 4.4.3.2., for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$d_r .v \leq 0,01.h$

where:

d_r – design interstorey drift;

h – storey height;

$v = 0,4$ – is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement. The given value is for importance classes I and II;

$d_r = d_{s,j} - d_{s,i}$

$d_s = q_d.d_e$

While modeling the structure is defined response spectrum for seismic motion with reference return period of 95 years, so it won’t be necessary a reduction of the analysis results with the reduction factor v.

In ETABS there is a function, which gives results for interstorey drifts like ratio of d_r/h. The results are given in the following table:

Interstorey drifts check

Етаж	*Кота*	h *Ет*	*Ет. премествания* X-Dir	*Ет. премествания* Y-Dir	*Междует. премествания* (d_r/h)	(d_r/h)	(d_r/h)	(d_r/h)	$d_{r,lim}$	*Проверка* X-Dir	*Проверка* Y-Dir
	[m]	[m]	[mm]	[mm]	[-]	[-]	[-]	[-]	[-]	[-]	[-]
Story PP	160,6	2	184,1	240,6	0,0003	0,0006	**0,0011**	**0,0020**	0,01	✓	✓
Story P3	158,6	3,5	183,5	239,3	0,0006	0,0006	**0,0021**	**0,0019**	0,01	✓	✓
Story P2	155,1	3,5	181,4	237,4	0,0008	0,0007	**0,0025**	**0,0023**	0,01	✓	✓
Story P1	151,6	7,7	178,9	235,3	0,0009	0,0009	**0,0030**	**0,0030**	0,01	✓	✓
Story31'	143,9	2,8	172,4	228,2	0,0006	0,0007	**0,0020**	**0,0023**	0,01	✓	✓
Story31	141,1	5	170,8	226,2	0,0006	0,0008	**0,0020**	**0,0027**	0,01	✓	✓
Story30'	136,1	4,2	168,0	222,4	0,0009	0,0010	**0,0028**	**0,0031**	0,01	✓	✓
Story30	131,9	4,2	164,7	218,6	0,0010	0,0011	**0,0033**	**0,0035**	0,01	✓	✓
Story29	127,7	4,2	160,9	214,4	0,0011	0,0012	**0,0037**	**0,0038**	0,01	✓	✓
Story28	123,5	4,2	156,7	209,7	0,0012	0,0012	**0,0038**	**0,0040**	0,01	✓	✓
Story27	119,3	4,2	152,3	204,9	0,0012	0,0013	**0,0039**	**0,0041**	0,01	✓	✓
Story26	115,1	4,2	147,8	199,9	0,0013	0,0014	**0,0041**	**0,0043**	0,01	✓	✓
Story25	110,9	4,2	142,9	194,6	0,0013	0,0014	**0,0043**	**0,0046**	0,01	✓	✓
Story24	106,7	4,2	137,8	188,9	0,0014	0,0015	**0,0045**	**0,0048**	0,01	✓	✓
Story23	102,5	4,2	132,5	183,0	0,0014	0,0016	**0,0046**	**0,0050**	0,01	✓	✓
Story22	98,3	4,2	127,0	176,8	0,0014	0,0016	**0,0046**	**0,0053**	0,01	✓	✓
Story21	94,1	5	121,4	170,3	0,0014	0,0017	**0,0044**	**0,0056**	0,01	✓	✓
Story20	89,1	4,2	115,1	162,1	0,0015	0,0017	**0,0048**	**0,0055**	0,01	✓	✓
Story19	84,9	4,2	109,2	155,2	0,0015	0,0017	**0,0048**	**0,0054**	0,01	✓	✓
Story18	80,7	4,2	103,3	148,5	0,0014	0,0017	**0,0046**	**0,0053**	0,01	✓	✓
Story17	76,5	4,2	97,7	141,7	0,0014	0,0017	**0,0044**	**0,0054**	0,01	✓	✓
Story16	72,3	4,2	92,2	134,9	0,0014	0,0017	**0,0044**	**0,0055**	0,01	✓	✓
Story15	68,1	4,2	86,7	127,9	0,0014	0,0017	**0,0045**	**0,0056**	0,01	✓	✓
Story14	63,9	4,2	81,1	120,7	0,0014	0,0018	**0,0046**	**0,0057**	0,01	✓	✓
Story13	59,7	4,2	75,4	113,4	0,0014	0,0018	**0,0046**	**0,0058**	0,01	✓	✓
Story12	55,5	4,2	69,6	105,9	0,0015	0,0018	**0,0047**	**0,0059**	0,01	✓	✓
Story11	51,3	4,2	63,8	98,3	0,0015	0,0019	**0,0047**	**0,0060**	0,01	✓	✓
Story10	47,1	4,2	57,9	90,6	0,0014	0,0019	**0,0046**	**0,0060**	0,01	✓	✓
Story9	42,9	4,2	52,1	82,9	0,0013	0,0019	**0,0042**	**0,0059**	0,01	✓	✓
Story8	38,7	5	46,9	75,1	0,0012	0,0019	**0,0038**	**0,0061**	0,01	✓	✓
Story7	33,7	5	41,2	65,6	0,0013	0,0020	**0,0041**	**0,0065**	0,01	✓	✓
Story6	28,7	5,5	34,9	55,5	0,0013	0,0021	**0,0043**	**0,0069**	0,01	✓	✓
Story5	23,2	5,5	27,6	43,7	0,0013	0,0022	**0,0043**	**0,0069**	0,01	✓	✓
Story4	17,7	5,5	20,3	31,8	0,0013	0,0021	**0,0043**	**0,0068**	0,01	✓	✓
Story3	12,2	5,5	12,9	20,0	0,0013	0,0020	**0,0041**	**0,0065**	0,01	✓	✓
Story2	6,7	6,7	5,8	8,9	0,0009	0,0014	**0,0029**	**0,0044**	0,01	✓	✓

6. P-Δ effects

According to БДС EN 1998 – 1, т. 4.4.3., second-order effects (P-Δ). effects) need not be taken into account if the following condition is fulfilled in all storeys:

$$\theta = \frac{P_{tot} \cdot d_r}{V_{tot} \cdot h_{ет}} \leq 0{,}1$$

where:

P_{tot} – is the total gravity load at and above the storey considered in the seismic design situation;

V_{tot} – total seismic storey shear;

h_{em} – interstorey height;

d_r – is the interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration;

To define the P-Δ effects are used the interstorey drifts, due to seismic motion with reference return period of 475 years.

Drifts are calculated for the center of mass of each elevation.

Storey forces are calculated automatically by the program.

Етаж	$V_{X,tot}$	$V_{Y,tot}$	P_{tot}	***Относители премествания (q.d$_r$/h)***			θ_x	θ_y	θ_{lim}	***Проверка***
	[kN]	*[kN]*	*[kN]*	*h[m]*	*x-dir*	*y-dir*	*[-]*	*[-]*	*[-]*	*[-]*
Story PP	167	136	864	160,6	0,0062	0,0061	0,03	0,04	0,10	✓
Story P3	923	716	4695	158,6	0,0056	0,0049	0,03	0,03	0,10	✓
Story P2	1789	1397	9096	155,1	0,0059	0,0085	0,03	0,06	0,10	✓
Story P1	2559	2024	13300	151,6	0,0059	0,0076	0,03	0,05	0,10	✓
Story31'	3597	2939	20298	143,9	0,0046	0,0117	0,03	0,08	0,10	✓
Story31	5395	4564	33239	141,1	0,0045	0,0046	0,03	0,03	0,10	✓
Story30'	7400	6390	49467	136,1	0,0055	0,0050	0,04	0,04	0,10	✓
Story30	8947	7833	64157	131,9	0,0062	0,0058	0,04	0,05	0,10	✓
Story29	10293	9132	78847	127,7	0,0067	0,0062	0,05	0,05	0,10	✓
Story28	11468	10308	93538	123,5	0,0071	0,0066	0,06	0,06	0,10	✓
Story27	12513	11387	108228	119,3	0,0075	0,0070	0,06	0,07	0,10	✓
Story26	13469	12396	122918	115,1	0,0079	0,0075	0,07	0,07	0,10	✓
Story25	14370	13355	137608	110,9	0,0082	0,0078	0,08	0,08	0,10	✓
Story24	15235	14277	152298	106,7	0,0085	0,0082	0,09	0,09	0,10	✓
Story23	16071	15163	166989	102,5	0,0088	0,0085	0,09	0,09	0,10	✓
Story22	16875	16010	181679	98,3	0,0088	0,0090	0,09	0,10	0,10	✓
Story21	17660	16827	196939	94,1	0,0085	0,0097	0,09	0,11	0,10	×
Story20	18390	17568	211629	89,1	0,0092	0,0096	0,11	0,12	0,10	×
Story19	19066	18252	226319	84,9	0,0097	0,0098	0,11	0,12	0,10	×
Story18	19707	18905	241009	80,7	0,0096	0,0101	0,12	0,13	0,10	×
Story17	20335	19548	255799	76,5	0,0093	0,0102	0,12	0,13	0,10	×
Story16	20959	20185	270588	72,3	0,0094	0,0104	0,12	0,14	0,10	×
Story15	21584	20823	285378	68,1	0,0094	0,0106	0,12	0,15	0,10	×
Story14	22205	21458	300167	63,9	0,0095	0,0108	0,13	0,15	0,10	×
Story13	22812	22080	314957	59,7	0,0096	0,0109	0,13	0,16	0,10	×
Story12	23388	22676	329746	55,5	0,0096	0,0110	0,14	0,16	0,10	×
Story11	23920	23235	344536	51,3	0,0096	0,0110	0,14	0,16	0,10	×
Story10	24400	23748	359325	47,1	0,0094	0,0112	0,14	0,17	0,10	×
Story9	24828	24218	374115	42,9	0,0086	0,0126	0,13	0,19	0,10	×
Story8	23493	21017	388971	38,7	0,0047	0,0109	0,08	0,20	0,10	×
Story7	25601	24641	428500	33,7	0,0080	0,0115	0,13	0,20	0,10	×
Story6	27509	26509	462094	28,7	0,0078	0,0108	0,13	0,19	0,10	×
Story5	28708	27477	495881	23,2	0,0079	0,0098	0,14	0,18	0,10	×
Story4	29851	28382	530059	17,7	0,0079	0,0077	0,14	0,14	0,10	×
Story3	30754	29090	564236	12,2	0,0079	0,0075	0,14	0,15	0,10	×
Story2	31264	29494	599084	6,7	0,0066	0,0064	0,13	0,13	0,10	×

After analyzing the results form calculations according to БДС EN 1998-1, it is clear that second-order effects should be taken into account.

Due to the complexity of the structure, its behavior was studied by "buckling analysis", which shows values of the coefficient $\alpha_{cr} > 10$, i.e. the so-called "10% criterion" is satisfied.

$\alpha_{cr} \geq F_{cr}/F \geq 10$ – requirement for not taking the p-Δ effects into consideration;

F_{cr} – critical load of the construction in elastic stage;

F – design value of the vertical load on the construction;

Despite these results, second order effects are taken into account in the analysis model of the building in ETABS. Comparing the results from different analysis with and without P-Δ effects, taken into account, slight differences in the values of the inner forces are obtained.

7. Nonlinear analysis – PUSHOVER ANALYSIS

For the approximate examination of the behavior of the structure in the plastic stage it is designed in the basis of a nonlinear analysis or Pushover-analysis, which aims to predict the sequence of formation of plastic hinges and redistribution of forces between the MRF and EBF-systems, The analysis is performed after applying vertical loads from the seismic combination and monotonically increasing horizontal load imitating floor seismic forces. Seismic motion is examined in direction parallel to X-direction.

The results of this analysis are to serve as a landmark, as the structure is multi-storey and has a high vibration period for the first mode. Even if the structure has fully elastic, higher modal forms have a significant contribution to the dynamic response. However, non-linear static analysis, although approximate, can provide additional valuable information on the development of plastic hinges in dissipative elements and on the verification of plastic rotations in link elements of the EBFs.

Advantages and disadvantages of the nonlinear analysis are discussed in detail in the research of Helmut Krawinkler и G.D.P.K. Seneviratna *“Pros and Cons of a pushover analysis of seismic performance evaluation”, Engineering Structures, vo. 20, 4-6, 1998*.

Two nonlinear analyzes of the structure were made. The first aims to test whether the link elements of the EBF, which are part of the dual system for resisting seismic motion, will work only in the elastic stage when earthquake with a reference return period of 95y occurs. The second nonlinear analysis monitors the formation of plastic hinges in the link elements of the EBFs and in the ends of the beams in MRFs. The calculated plastic rotations of the link elements are compared with the rotational capacity prescribed in Eurocode 8 for "short" links.

For this purpose, plastic hinges for bending moment M3 are defined in the zones of the link elements and at the ends of the beams, which are part of the MRFs. The “bending moment – rotation” diagrams for plastic hinges are generated automatically by the program according to the defined cross sections in accordance with American standards ASCE 41-13. It should be noted that the plastic hinges at the ends of the link elements are defined by the bending resistance capacity in the plastic stage, which is a derivative of the shear capacity of the link element in the plastic stage. This simulates the exceeding of shear yield strenght of the link elements.

The distribution of the floor seismic forces along the height of the building is obtained by the floor shear forces determined by the spectral method. The model is loaded with normalized values of these horizontal forces applied in the center of masses on each floor level.

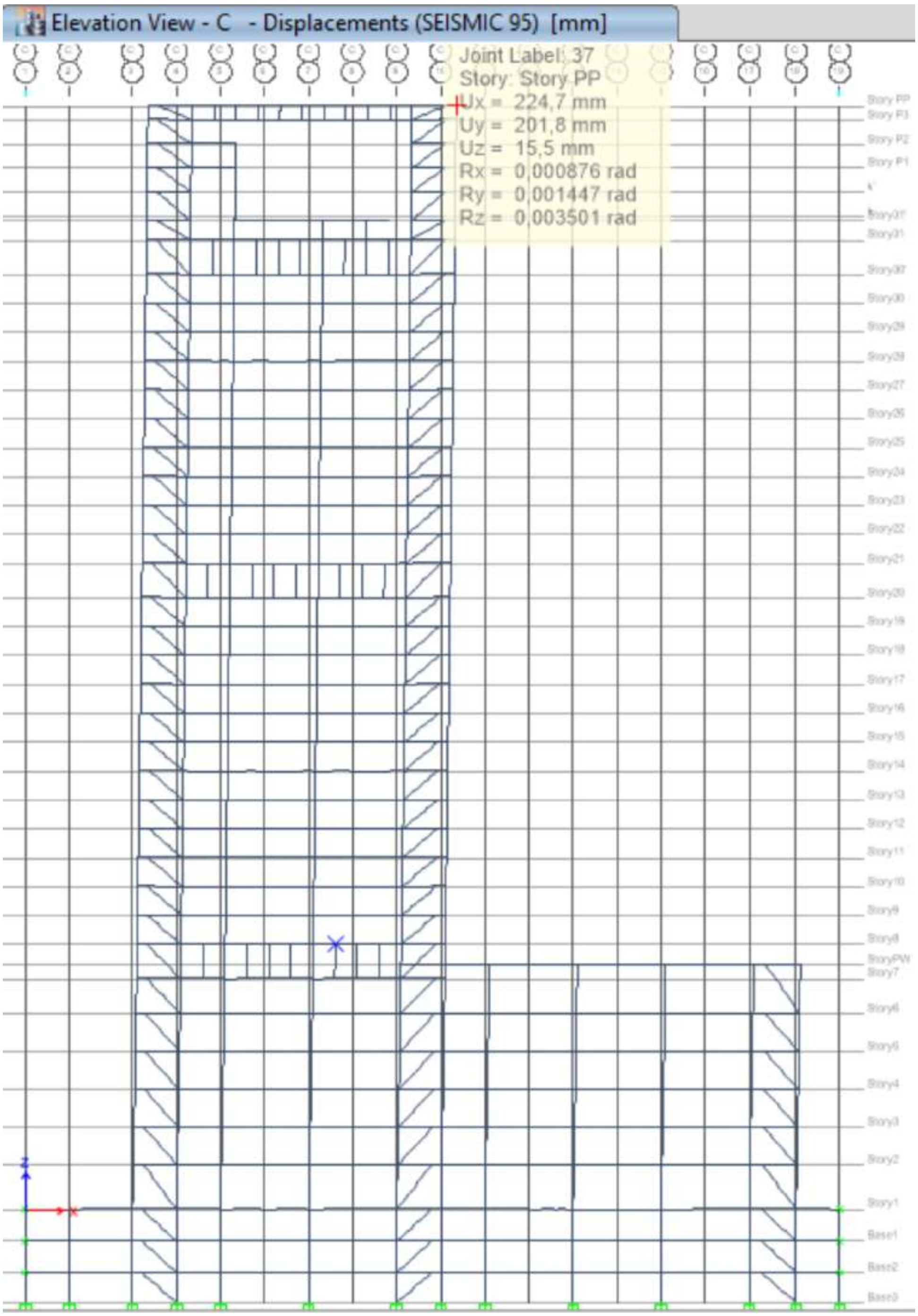
The first nonlinear load case is defined, including vertical loads and dead loads. It serves as the initial load case preceding the Pushover - the analysis of horizontal forces.

For the purposes of Pushover analysis, there is a target displacement of the roof level (Displacement control) set, which is determined by spectral method. Alternatively, the target displacement can be determined by the procedure “Displacement coefficient method” by FEMA 356.

7.1. Serviceability limit state check

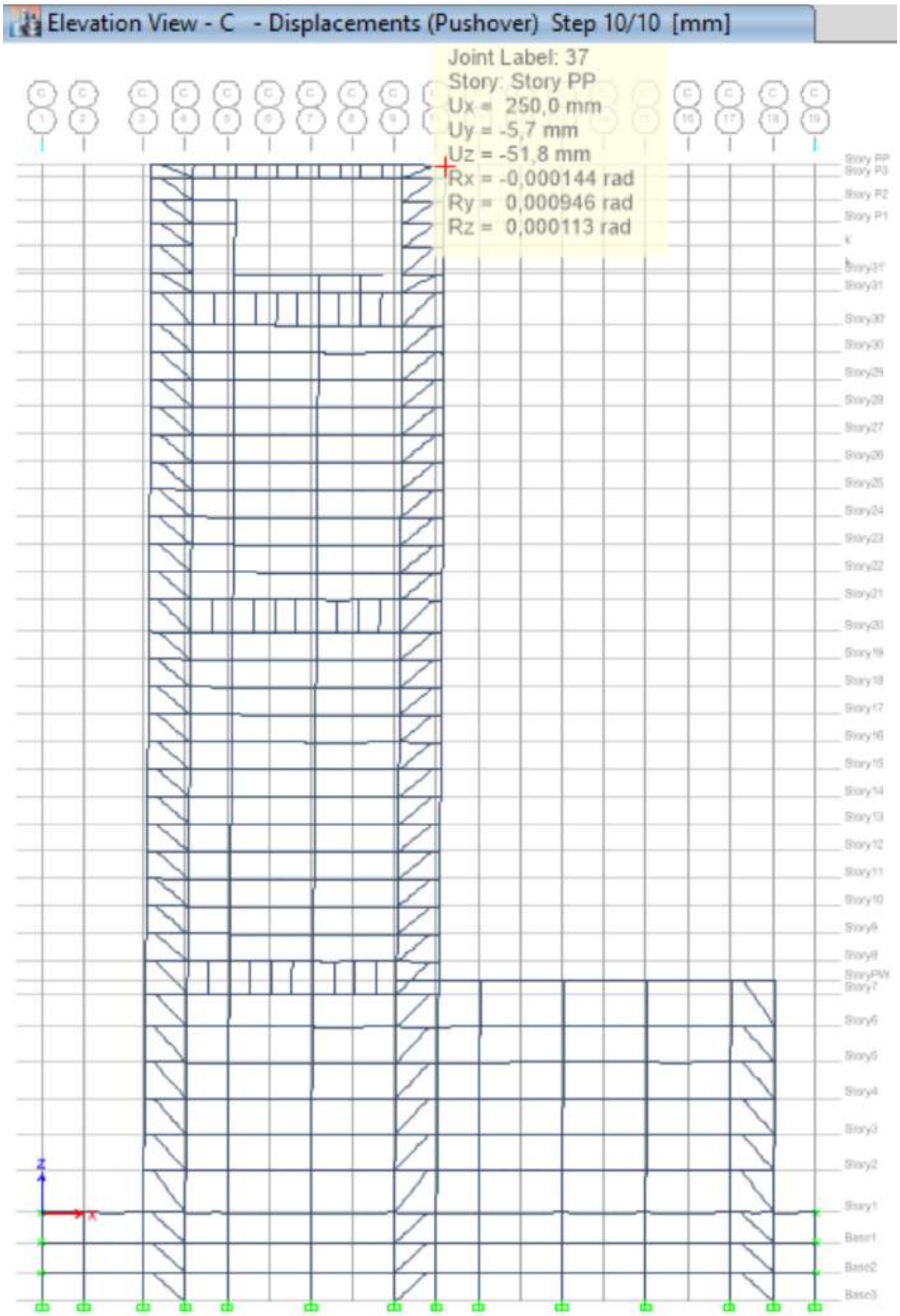
Static nonlinear analysis verifies that the “link” elements operate in the elastic stage during a target displacement determined by spectral analysis for an earthquake with a reference return period of 95 years

Displacements from “weak” earthquake (Modal response spectrum analysis):



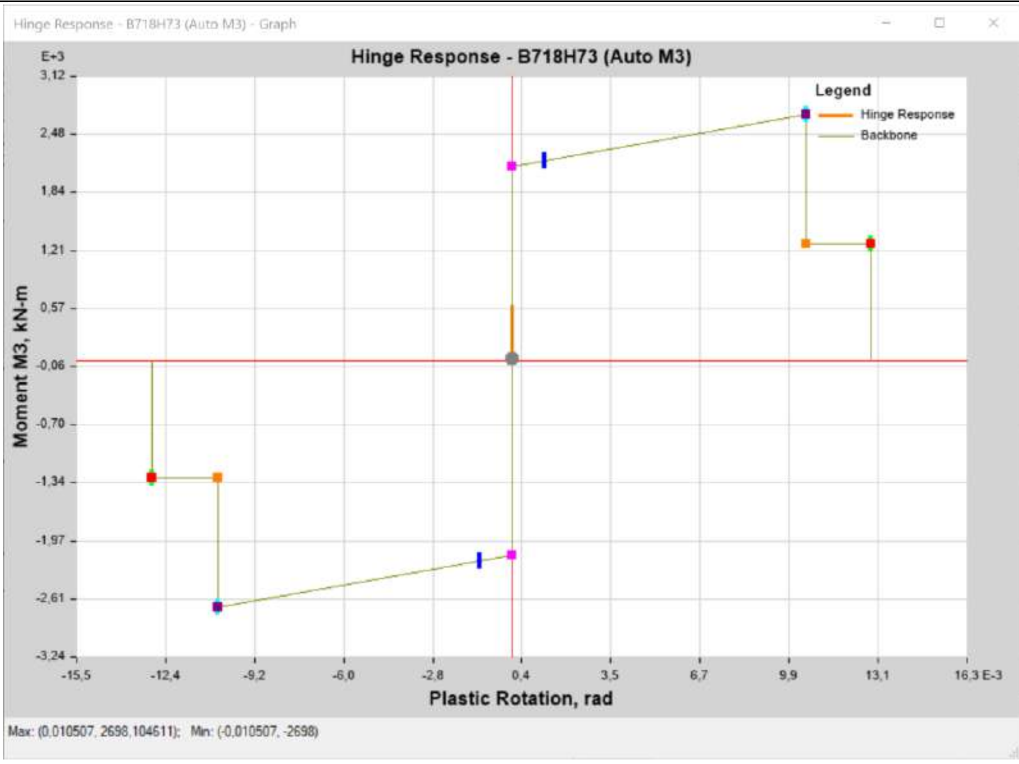
It is defined a target displacement of *250 mm* while defining the nonlinear analysis!

Displacements as a result of the nonlinear analysis:



From the presented deformed scheme it is clear that when reaching the limit displacement obtained by the spectral method, no plastic deformations are observed and the elements work in the elastic stage. This result satisfies the requirement to limit the damage from the defined earthquake with a reference return period of 95 years.

This is also proved by the following graphic, which shows the behavior of the most loaded seismic connecting element (floor 2). It can be seen that the element works in an elastic stage and does not yield until the target displacement is achieved.



„Force-displacement" curve (shows the behavior of plastic hinges)

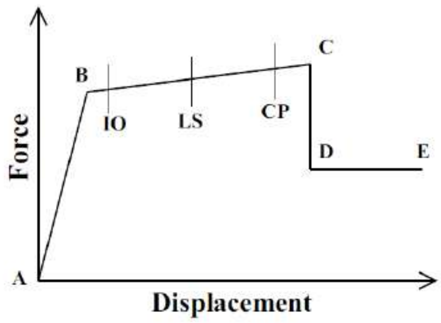


Figure 40
The A-B-C-D-E curve for Force vs. Displacement
The same type of curve is used for Moment vs. Rotation

т. A – начално състояние;

т. B – начало на пластични деформации;

т. C – гранична носимоспособност;

т. D – остатъчна носимоспособност;

т. E – разрушаване на елемента;

7.2.Maximal plastic rotation in link elements

With the second Pushover analysis plastic deformations as a result of the "strong" earthquake - with a reference return period of 475 years, could be observed.

In the response spectrum analysis, a coefficient of behavior q = 3.2 was adopted. The horizontal displacements obtained from the program are underestimated. The target displacement of the structure at roof level can be obtained approximately by multiplying the displacement from response spectrum analysis by the behavior factor:

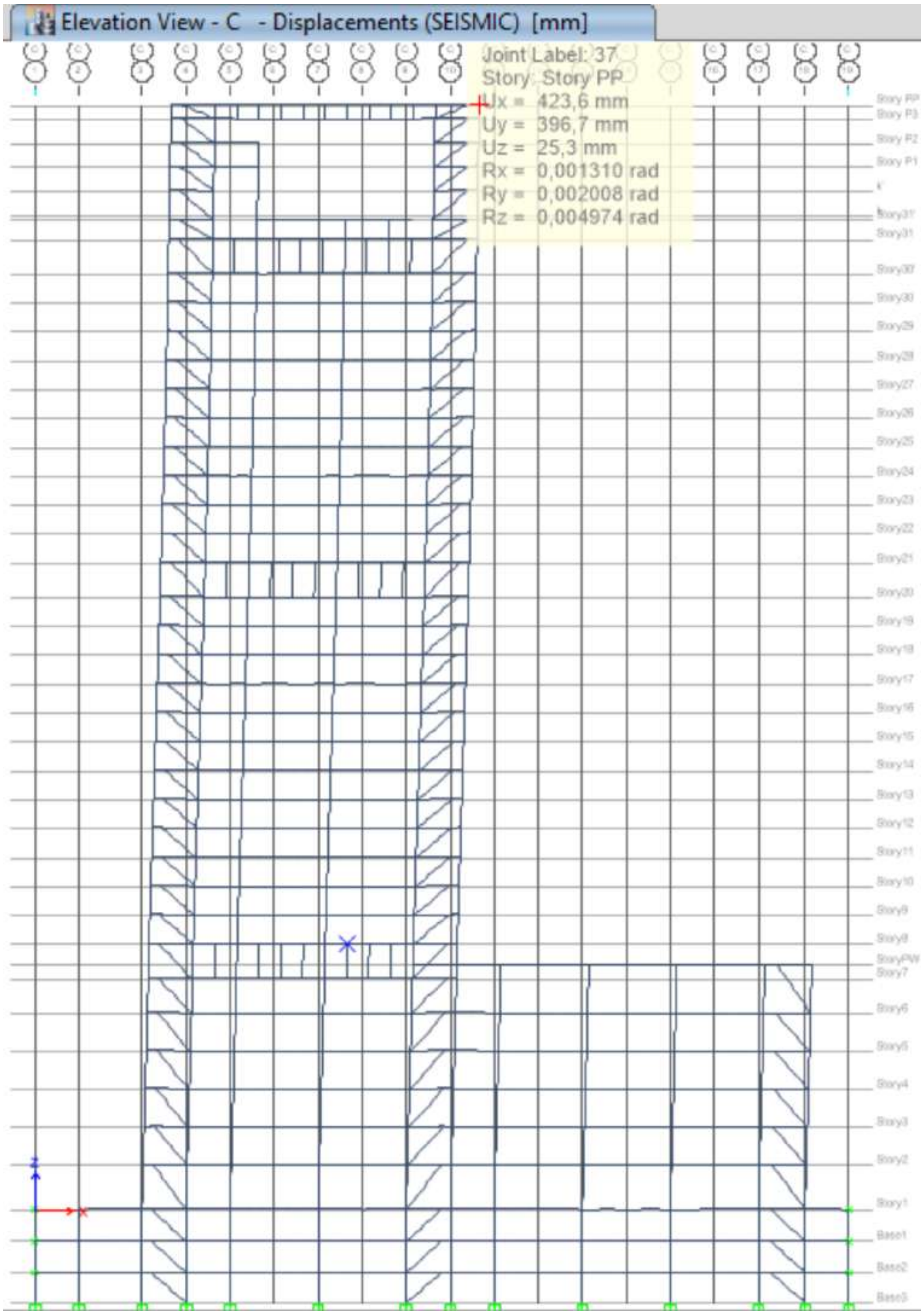
$\Delta_{pl} = q\ \Delta_{el}$.

As reference is used the horizontal displacement of the highest point of the structure. Node 37 with displacement *U_1 = 423,6 mm.* is considered.

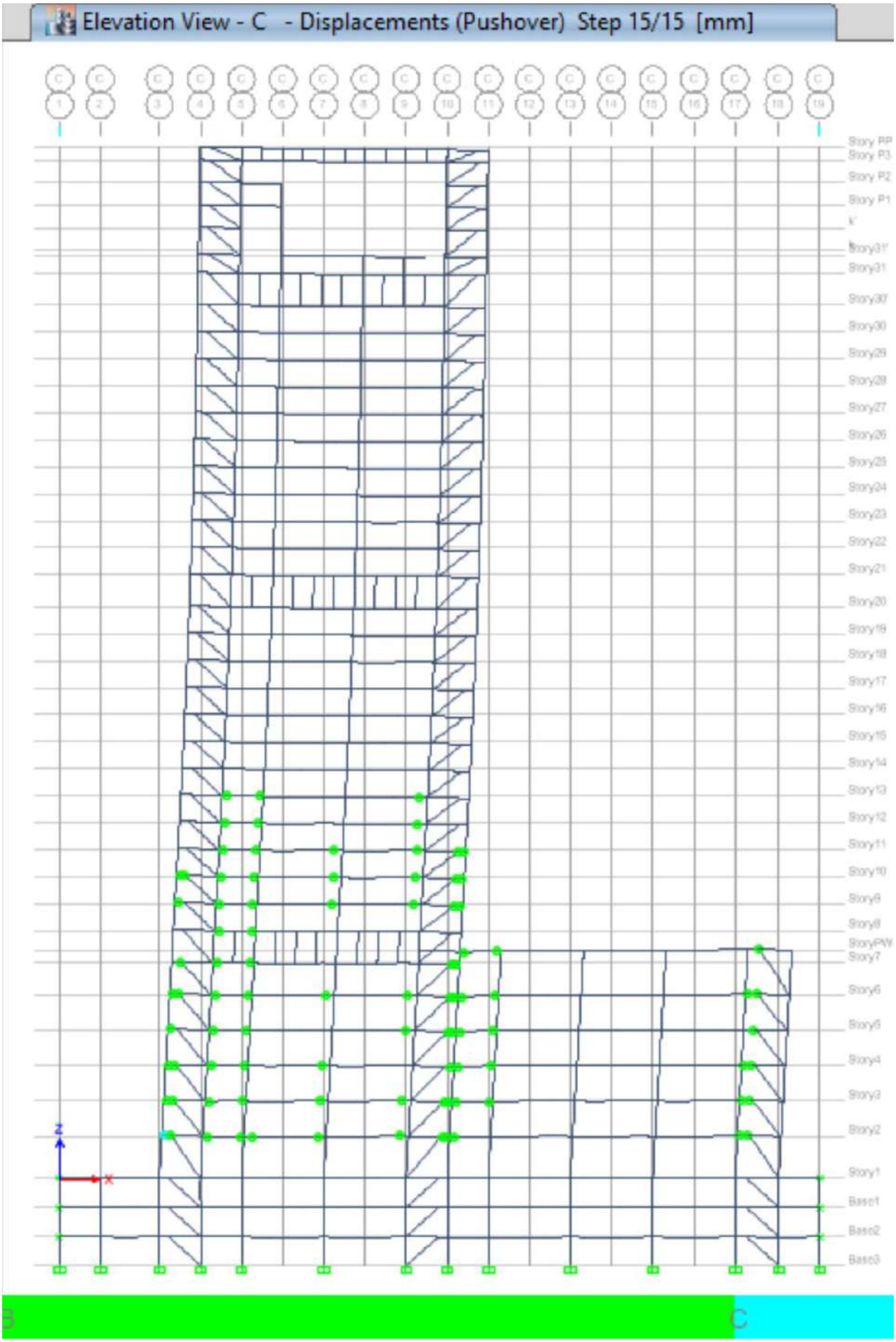
The target displacement defined in the non linear analysis is:

Δ_{pl} = 3,2.423,6 = 1355 mm

Results for the peak displacement (response spectrum analysis):



<u>Results from the nonlinear analysis of the sructure:</u>



In the last step of the analysis beginning of yielding is observed, both in the link elements and in the ends of the beams of the MRFs. At this stage, yielding should only be observed in the active link element, as EBFs are the main system for resisting seismic impact and MRFs are a secondary system. This may be a result of inaccurate definition of plastic hinges in the model, because the program generates "automatic" plastic hinges with unrealistically low rotational capacity compared to the available capacity for "short" link elements. Also this could be a result of the redistribution of inner forces that occurs after reaching the "residual load bearing capacity" of these hinges. The depletion of the rotational capacity and the decrease of the load bearing capacity in the plastic hinges of the link elements leads to an earlier transfer of part of the seismic forces from the EBFs to the MRFs in the dual system.

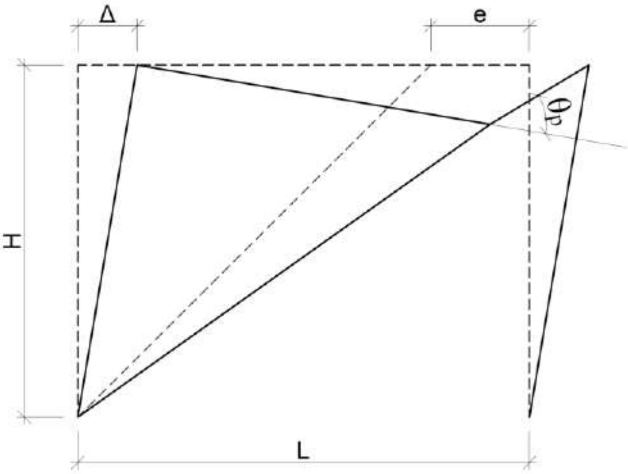
TABLE: Hinge States

Story	***Load Case/Combo***	***Assigned Hinged***	***Generated Hinge***	***Relative Distance***	***M3*** kN-m	***R3 Plastic*** rad	***Hinge State***	***Hinge Status***
Story11	Pushover Max	Auto M3	B722H55	0,1	1176	**0,0006**	B to C	A to IO
Story11	Pushover Min	Auto M3	B722H56	0,9	-1197	**-0,0013**	A to B	A to IO
Story10	Pushover Max	Auto M3	B718H57	0,1	1190	**0,0014**	B to C	A to IO
Story10	Pushover Min	Auto M3	B718H58	0,9	-1177	**-0,0009**	A to B	A to IO
Story10	Pushover Max	Auto M3	B722H57	0,1	1252	**0,0029**	B to C	A to IO
Story10	Pushover Min	Auto M3	B722H58	0,9	-1274	**-0,0037**	A to B	A to IO
Story9	Pushover Max	Auto M3	B718H59	0,1	1169	**0,0005**	B to C	A to IO
Story9	Pushover Min	Auto M3	B718H60	0,9	-1046	**0**	A to B	A to IO
Story9	Pushover Max	Auto M3	B722H59	0,1	1198	**0,0015**	B to C	A to IO
Story9	Pushover Min	Auto M3	B722H60	0,9	-1242	**-0,0027**	A to B	A to IO
Story8	Pushover Max	Auto M3	B718H61	0,1	865	**0**	A to B	A to IO
Story8	Pushover Min	Auto M3	B718H62	0,9	-587	**0**	A to B	A to IO
Story8	Pushover Max	Auto M3	B722H61	0,1	722	**0**	A to B	A to IO
Story8	Pushover Min	Auto M3	B722H62	0,9	-1135	**0**	A to B	A to IO
Story7	Pushover Max	Auto M3	B718H63	0,1	1079	**0**	A to B	A to IO
Story7	Pushover Min	Auto M3	B718H64	0,9	-1175	**-0,0009**	A to B	A to IO
Story7	Pushover Max	Auto M3	B722H63	0,1	1269	**0,0037**	B to C	A to IO
Story7	Pushover Min	Auto M3	B722H64	0,9	-1262	**-0,0032**	A to B	A to IO
Story6	Pushover Max	Auto M3	B718H65	0,1	1288	**0,0040**	B to C	A to IO
Story6	Pushover Min	Auto M3	B718H66	0,9	-1263	**-0,0035**	A to B	A to IO
Story6	Pushover Max	Auto M3	B722H65	0,1	1382	**0,0067**	B to C	A to IO
Story6	Pushover Min	Auto M3	B722H66	0,9	-1365	**-0,0062**	A to B	A to IO
Story5	Pushover Max	Auto M3	B718H67	0,1	2116	**0,0001**	B to C	A to IO
Story5	Pushover Min	Auto M3	B718H68	0,9	-1665	**0**	A to B	A to IO
Story5	Pushover Max	Auto M3	B722H67	0,1	2137	**0,0006**	B to C	A to IO
Story5	Pushover Min	Auto M3	B722H68	0,9	-2178	**-0,0012**	A to B	A to IO
Story4	Pushover Max	Auto M3	B718H69	0,1	2255	**0,0024**	B to C	A to IO
Story4	Pushover Min	Auto M3	B718H70	0,9	-2169	**-0,0008**	A to B	A to IO
Story4	Pushover Max	Auto M3	B722H69	0,1	1960	**0**	A to B	A to IO
Story4	Pushover Min	Auto M3	B722H70	0,9	-2169	**-0,0010**	A to B	A to IO
Story3	Pushover Max	Auto M3	B718H71	0,1	2510	**0,0072**	B to C	A to IO
Story3	Pushover Min	Auto M3	B718H72	0,9	-2401	**-0,0055**	A to B	A to IO
Story3	Pushover Max	Auto M3	B722H71	0,1	2350	**0,0045**	B to C	A to IO
Story3	Pushover Min	Auto M3	B722H72	0,9	-2383	**-0,0050**	A to B	A to IO
Story2	Pushover Max	Auto M3	B718H73	0,1	2693	**0,0104**	C to D	A to IO
Story2	Pushover Min	Auto M3	B718H74	0,9	-2565	**-0,0083**	A to B	A to IO
Story2	Pushover Max	Auto M3	B722H73	0,1	2472	**0,0064**	B to C	A to IO
Story2	Pushover Min	Auto M3	B722H74	0,9	-2526	**-0,0075**	A to B	A to IO

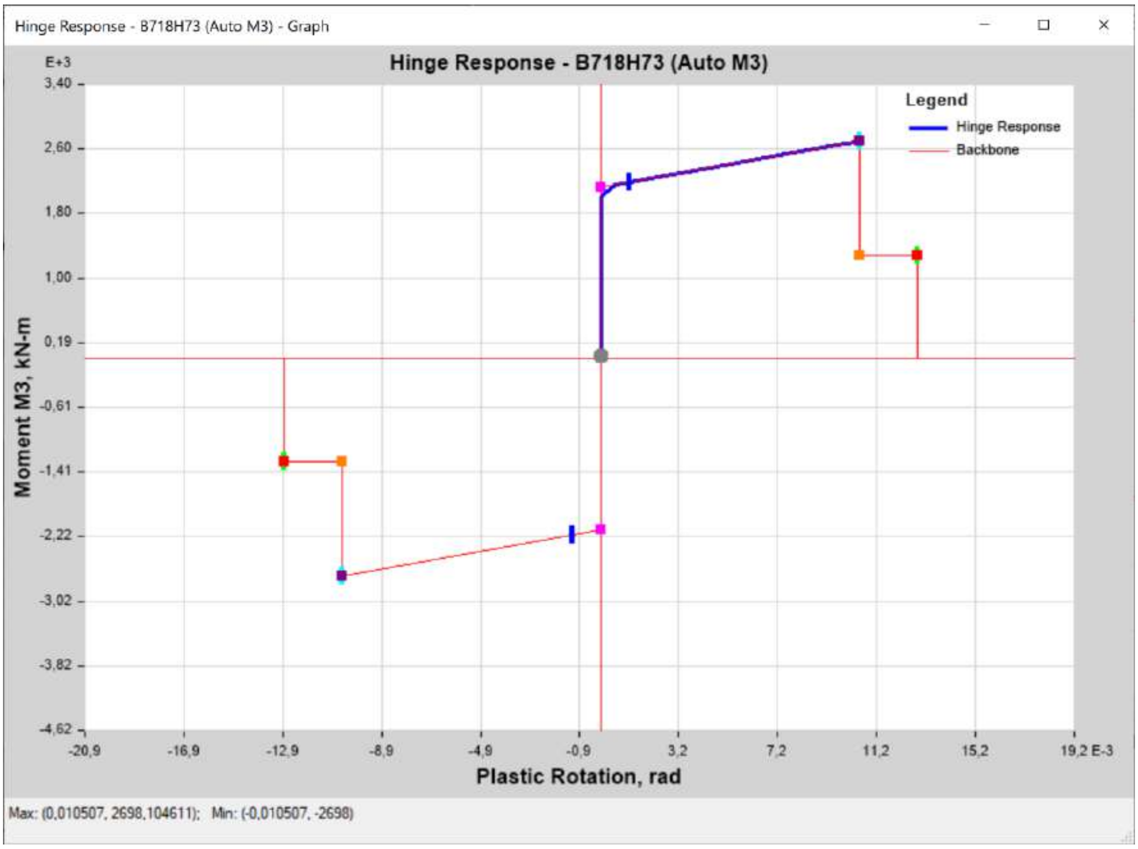
The deformations (angle of plastic rotation) in the active link elements, in which plastic deformations occurred before the considered control displacement, were studied.

It can be seen that the value of the plastic rotation in the most loaded link element does not exceed the allowable one.

$\boldsymbol{\Theta_{max} = 0{,}0104\ rad < \Theta_p = 0{,}08\ rad}$



The given value for θ_{max} refers to plastic hinge *B718H73*, generated in link element with cross section *HE550B* at floor 2. Its behavior is presented graphically.



Plastic deformation curve

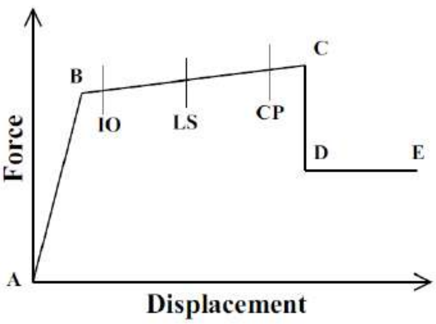


Figure 40
The A-B-C-D-E curve for Force vs. Displacement
The same type of curve is used for Moment vs. Rotation

т. A – начално състояние;

т. B – начало на пластични деформации;

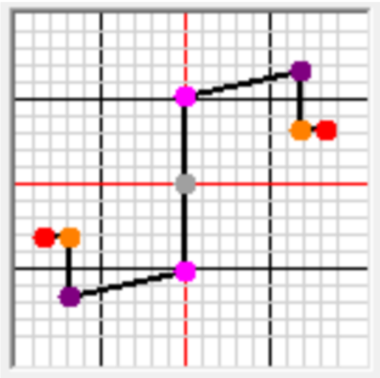
т. C – гранична носимоспособност;

т. D – остатъчна носимоспособност;

т. E – разрушаване на елемента;

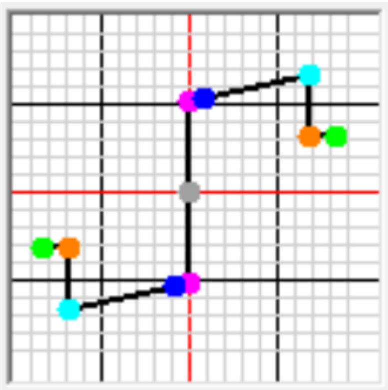
Представени са относителните стойности на огъващия момент в пластичната става за различните точки. Като базова стойност се залага носимоспособност на огъване в пластичен стадий ($M_{p,link}$), която е производна на носимоспособността на срязване на съответния линк-елемент в пластичен стадий ($V_{p,link}$).

The relative values of the bending moment in the plastic hinge for different points are presented. The basic value is the bending resistance in the plastic stage (Mp, link), which is a derivative of the shear resistance of the link element in the plastic stage (Vp, link).



Point	Moment/SF	Rotation/SF
E-	-0,6	-11
D-	-0,6	-9
C-	-1,27	-9
B-	-1	0
A	0	0
B	1	0
C	1,27	9
D	0,6	9
E	0,6	11

Operational level of the structure and the structursl elements is defined due to points *IO*, *LS* и *CP*.

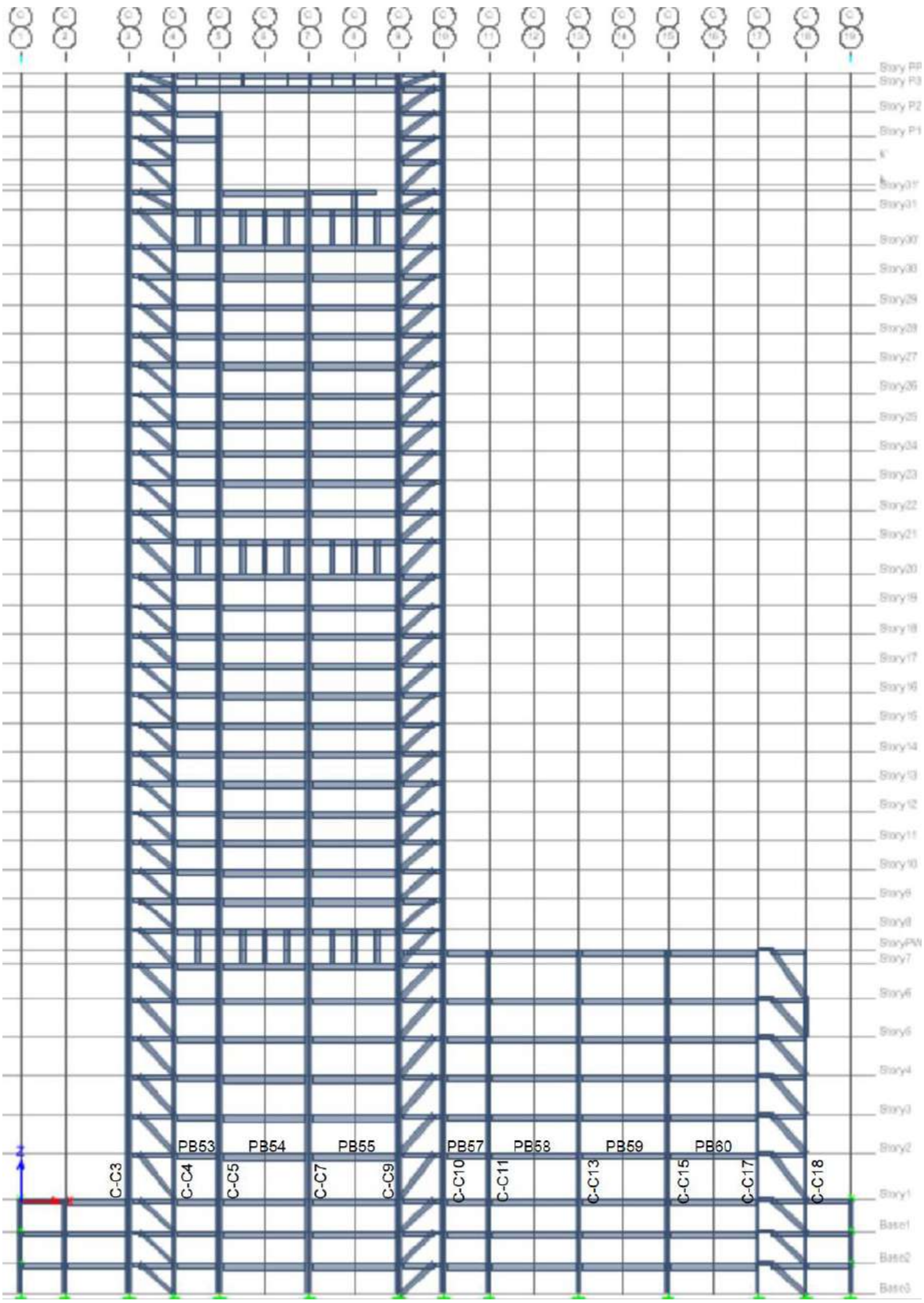


IO – Immediate occupancy level

lS - Life Safety level

CP - Collapse Prevention

V. Design of the elements from frame parallel to axis „C”



1. Elements of EBF 4

Results received from the analysis show values of the inner forces as a result of design seismic combination. For the purposes of capacitive design, the forces of this combination are divided into seismic forces and forces, due to vertical (non-seismic) loads, which are involved in the seismic combination *Accidental.*

4.3. Active link elements – L4

1.1.1. Design checks

The selected cross sections should have load bearing resistance, according to the following conditions:

$N_{Ed}/N_{pl,Rd} \leq 0{,}15 \rightarrow$ *If not satisfied, reduced shear and bending resistance is defined!*

$V_{Ed} \leq V_{p,\ link}$

$M_{Ed} \leq M_{p,\ link}$

- *Shear resistance*

$V_{p,\ link} = (f_y/\sqrt{3}).t_w.(d - t_f)$

- *Bending resistance*

$M_{p,\ link} = f_y.b.t_f\ .(d - t_f)$

1.1.2. Classification of the link elements according to their length

Short link elements are designed. They dissipate energy by shear yielding. Length og the link elements shall be limited to: $e < e_s$

$e_s = 1{,}6.M_{p,link}/V_{p,link}$

1.1.3. Global dissipative behavior requirement

According to БДС EN 1998 – 1, т. 6.8.2., to guarantee global dissipative behavior of the structure, the values Ω_i should not exceed the minimal value of the resistance reserve Ω_{min} more than 25%.

$\Omega_{min} = min\{min(\Omega_{i,V});\ min(\Omega_{i,M})\}$

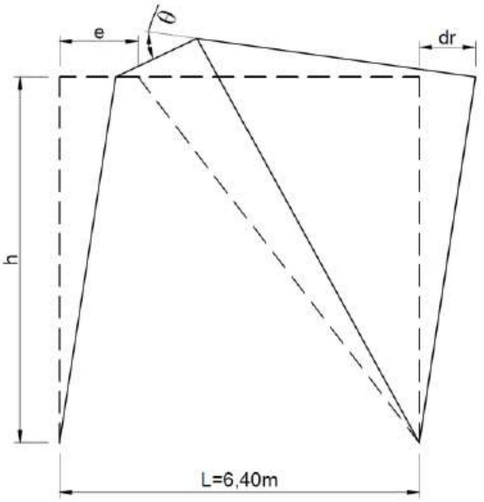
$\Omega_{i,V} = 1{,}5.(V_{p,\ link,\ i}/V_{Ed,\ i})$

$\Omega_{i,M} = 1{,}5.(M_{p,\ link,\ i}/M_{Ed,\ i})$

In order to be satisfied the presented above, the following condition should be satisfied:

$\Omega_{max}/\Omega_{min} \leq 1{,}25$

1.1.4. Angle of rotation



Another important criteria while designing the EBFs is limitation of the plastic rotation. In order to be defined the angle of rotation, nonlinear analysis, observing the behavior of the construction in plastic stage is necessary. In the current thesis results of the nonlinear analysis wont be consider in order to increase safety. This is why the angle of rotation will be calculated on the bases of geometry and using the interstorey drifts. The following formula is used:

$$\theta_p = \frac{\Delta.L}{H.e}$$

Where:

$\Delta = k.q.d_{r,i}$ – relative interstorey drift in elasto-plastic stage;

k – coefficient, showing the ratio of deformations in plastic and deformations in elasto-plastic stage. According to ASCE-7-16:

$k = c_d/R = 4/8 = 0,5$ – За EBF

$k = c_d/R = 5,5/8 = 0,7$ – За MRF

→ It's assumed value of k, reffering to MRF: $k = 0,7$

$h = h_i$ – interstorey height;

e – length of the „link"- element;

L = 6,40 m – span of the EBF;

$\boldsymbol{\theta_p \leq \theta_{p,R} = 0,08}$ ***rad*** **–** restriction for short active link elements;

$q.d_{r,I,max}/h_i = 0,0144$

$$\boldsymbol{\theta_{p,max} = \frac{0,7.0,0114.6,40}{0,85} = 0,06\ rad < \theta_{p,R} = 0,08\ rad}$$

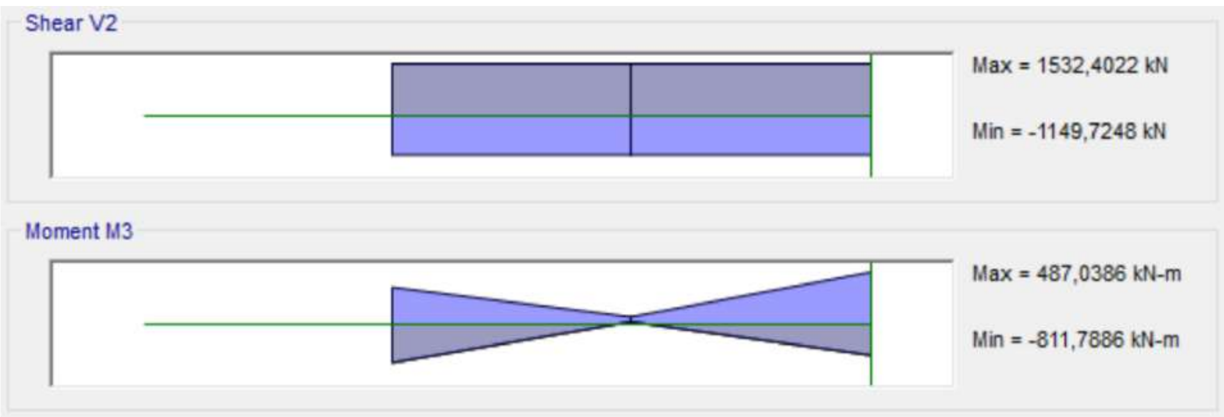
1.1.5. Cross-section characteristics

Характеристики на напречното сечение					
LINK - HE550B					
A [cm²]	***b [mm]***	***tw [mm]***	***tf [mm]***	***d [mm]***	***Fy [kN/cm2]***
254,1	300	15	29	550	35,5
LINK - HE500A					
A [cm²]	***b [mm]***	***tw [mm]***	***tf [mm]***	***d [mm]***	***Fy [kN/cm2]***
197,5	300	12	23	490	35,5
LINK - HE400A					
A [cm²]	***b [mm]***	***tw [mm]***	***tf [mm]***	***d [mm]***	***Fy [kN/cm2]***
159	300	11	19	390	35,5
LINK - HE360B					
A [cm²]	***b [mm]***	***tw [mm]***	***tf [mm]***	***d [mm]***	***Fy [kN/cm2]***
180,6	300	12,5	22,5	360	35,5
LINK -HE300A					
A [cm²]	***b [mm]***	***tw [mm]***	***tf [mm]***	***d [mm]***	***Fy [kN/cm2]***
112,5	300	8,5	14	290	35,5
LINK - HE240A					
A [cm²]	***b [mm]***	***tw [mm]***	***tf [mm]***	***d [mm]***	***Fy [kN/cm2]***
76,8	240	7,5	12	230	35,5

1.1.6. Presentation of the design results

Оразмеряване на сеизмичните свързващи елемнти в рамка по ос "C"															
Етаж	**Напр. сечение**	**Клас стомана**	N_{Ed} [kN]	V_{Ed} [kN]	M_{Ed} [kNm]	N_{Rd} [kN]	$V_{pl,link}$ [kN]	$M_{pl,link}$ [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{p,link}}$	$\frac{M_{Ed,y}}{M_{p,link}}$	e_s [m]	$e_{s,lim}$ [m]	Ω -	θ [rad]
Ет. 31	**HE300A**	S355	108	391	191	3804	481	412	0,03	0,81	0,46	1,30	1,37	**1,85**	0,02
Ет. 30'	**HE240A**	S275	115	243	109	2011	260	173	0,06	0,93	0,63	1,00	1,06	**1,60**	0,02
Ет. 30		S355	20	273	132	2597	335	223	0,01	0,81	0,59	1,00	1,06	**1,84**	0,03
Ет. 29		S355	27	334	163	2597	335	223	0,01	1,00	0,73	0,85	1,06	**1,51**	0,03
Ет. 28	**HE300A**	S355	16	390	192	3804	481	412	0,00	0,81	0,47	0,85	1,37	**1,85**	0,03
Ет. 27		S355	14	442	218	3804	481	412	0,00	0,92	0,53	0,85	1,37	**1,63**	0,04
Ет. 26		S355	13	481	244	3804	481	412	0,00	1,00	0,59	0,85	1,37	**1,50**	0,04
Ет. 25	**HE360A**	S275	12	540	269	6106	670	626	0,00	0,81	0,43	0,85	1,50	**1,86**	0,04
Ет. 24		S275	11	588	294	6106	670	626	0,00	0,88	0,47	0,85	1,50	**1,71**	0,04
Ет. 23		S355	13	629	313	6106	670	809	0,00	0,94	0,39	0,85	1,93	**1,60**	0,04
Ет. 22		S275	59	624	307	6106	670	626	0,01	0,93	0,49	0,85	1,50	**1,61**	0,05
Ет. 21		S275	130	574	303	6106	670	626	0,02	0,86	0,48	0,85	1,50	**1,75**	0,05
Ет. 20	**HE400A**	S355	163	691	357	5376	836	751	0,03	0,83	0,48	0,85	1,44	**1,82**	0,05
Ет. 19		S355	13	784	396	5376	836	751	0,00	0,94	0,53	0,85	1,44	**1,60**	0,05
Ет. 18		S355	48	755	371	5376	836	751	0,01	0,90	0,49	0,85	1,44	**1,66**	0,05
Ет. 17		S355	27	709	348	5376	836	751	0,00	0,85	0,46	0,85	1,44	**1,77**	0,05
Ет. 16		S355	9	726	364	5376	836	751	0,00	0,87	0,48	0,85	1,44	**1,73**	0,06
Ет. 15		S355	8	762	384	5376	836	751	0,00	0,91	0,51	0,85	1,44	**1,65**	0,06
Ет. 14		S355	8	804	408	5376	836	751	0,00	0,96	0,54	0,85	1,44	**1,56**	0,06
Ет. 13		S355	7	833	432	5376	836	751	0,00	1,00	0,58	0,85	1,44	**1,51**	0,06
Ет. 12	**HE500A**	S275	6	893	457	5173	890	886	0,00	1,00	0,52	0,85	1,59	**1,49**	0,06
Ет. 11		S355	7	936	481	6677	1149	1144	0,00	0,82	0,42	0,85	1,59	**1,84**	0,06
Ет. 10		S355	8	964	495	6677	1149	1144	0,00	0,84	0,43	0,85	1,59	**1,79**	0,06
Ет. 9		S275	69	898	447	5173	890	886	0,01	1,01	0,50	0,85	1,59	**1,49**	0,06
Ет. 8	**HE400A**	S355	148	823	387	5376	836	751	0,03	0,98	0,52	0,85	1,44	**1,52**	0,06
Ет. 7	**HE500A**	S355	159	1037	529	6677	1149	1144	0,02	0,90	0,46	0,85	1,59	**1,66**	0,06
Ет. 6		S355	10	1150	595	6677	1149	1144	0,00	1,00	0,52	0,85	1,59	**1,50**	0,06
Ет. 5	**HE550B**	S275	32	1227	668	6655	1241	1246	0,00	0,99	0,54	0,85	1,61	**1,52**	0,06
Ет. 4		S355	19	1339	739	8591	1602	1609	0,00	0,84	0,46	0,85	1,61	**1,79**	0,06
Ет. 3		S355	53	1488	818	8591	1602	1609	0,01	0,93	0,51	0,85	1,61	**1,61**	0,06
Ет. 2		S355	248	1532	812	8591	1602	1609	0,03	0,96	0,50	0,85	1,61	**1,57**	0,05

Design inner forces for link elements are a result of *Accidental* combination. The following graphic presents diagrams of the shear forces and bending moments in the active link element, as a result of the analysis of the structure. In this case are given the results for s link element located on the second floor level.



In order to satisfy the requirements given in point 1.1.3.,and also to reduce the number of different sections, there are two steel classes selected for the design of link elements: S275J2 и S355J2.

In order to guarantee the plastic hinge formation within the link element and to avoid resizing the other elements of the EBF, which are operating in the elastic stage, cross sections are chosen in order to minimize the load bearing capacity.

Ω_{min} = 1,50; Ω_{max} = 1,86 → $\Omega_{max}/\Omega_{min} = 1,86/1,50 = 1,25$ → *The requirement placed in point 1.1.3. is satisfied! Link elements till floor 31 are considered! Under the ground level EBF with link elements are replaced with ordinary braced frames.*

$\theta_p = \theta_{p,max} = 0,06 \leq \theta_{p,R} = 0,08$ rad → *Requirement from point. 1.1.4. is satisfied!*

1.1.7. Web stiffeners for active link elements

According to БДС EN 1998 – 1, т. 6.8.2., full-depth web stiffeners should be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners should have a combined width of not less than $(b_f - 2t_w)$ and a thickness not than $0,75t_w$ nor 10 mm, whichever is larger:

$b_r \leq b_f - 2.t_w$

$t_r \geq max\{0,75.t_w;10\ mm\}$

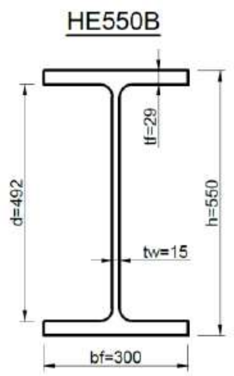
According to the same standard, short links should be provided with intermediate web stiffeners spaced at intervals, in relation with the rotation angle.

For elements with $\theta = 0,02 \rightarrow l_r \leq 52.t_w - d/5$

For elements with $\theta = 0,08 \rightarrow l_r \leq 30.t_w - d/5$

To satisfy these requirements, web stiffeners are provided at the ends of the link. These stiffeners are designed according to the cross section of the link element. Intermediate stiffeners are also provided, spaced st the minimal intervals. The following ribs are designed for link elements with cross sections *HE500B* and *HE400A.*

За HE550B:



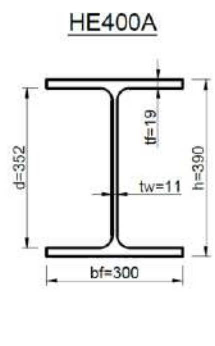
$\theta = 0,06 \rightarrow l_r \leq 37,3.t_w - d/5 = 37,3.15 - 492/5 = 461\ mm$ → For length of the link: $e = 850\ mm$ → $\mathbf{l_r = 425\ mm}$

$b_r \leq 300 - 2.15 = 270$ → $\mathbf{b_r = 270\ mm}$

$t_r \geq max\{0,75.15\ ;\ 10mm\} = max\{11,25\ mm;\ 10\ mm\} = 11,25\ mm$

→ $\mathbf{t_r = 12\ mm}$

За HE400A:



$\theta = 0,06 \rightarrow l_r \leq 37,3.t_w - d/5 = 37,3.11 - 352/5 = 340\ mm$ → For length of the link: $e = 850\ mm$ → $\mathbf{l_r = 283\ mm}$

$b_r \leq 300 - 2.11 = 278$ → $\mathbf{b_r = 278\ mm}$

$t_r \geq max\{0,75.11\ ;\ 10mm\} = max\{8,25\ mm;\ 10\ mm\} = 8,25\ mm$

→ $\mathbf{t_r = 10\ mm}$

1.1.8. Welds between stiffeners and link element

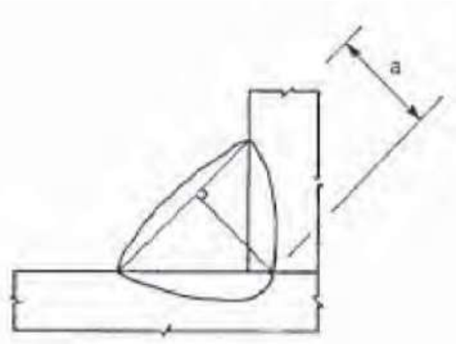
Welds of the most loaded link elements are designed. These links has cross sections *HE550B* и *HE400A.*

- Welds between web and stiffener

Welds between web and stiffener are calculated using the following formula:

$\gamma_{ov}.A_{st}.f_y$,

Where: $A_{st} = t_r.b_r$ – Stiffeners cross section area;



According to БДС EN 1993–1–8, т. 4.5.3.3., weld's resistance can be determined with the equation:

$F_{w,Rd} = f_{vw,d}.a,$

Where:

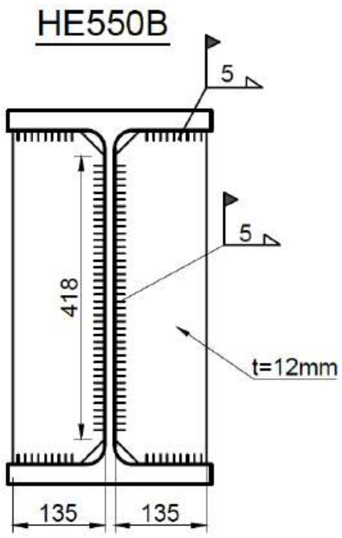
$f_{vw,a} = \frac{f_u/\sqrt{3}}{\beta_w.\gamma_{M2}}$

$F_u = 355\ MPa = 35,5\ kN/cm^2$ – nominal tensile strength of the lower class material;

$\beta_w = 0,9$ – correlation coefficient according to 4.1. на БДС EN 1993-1-1, т. 4.5.3.2.;

$f_{vw,a} = \frac{35,5/\sqrt{3}}{0,9.1,25} = 18,22\ kN/cm$

Weld's thickness: $a \geq \frac{\gamma_{ov}.A_{st}.f_y}{2.l_w.f_{vw,d}}$



За HE550B:

$A_{st} = t_r.b_r/2 = 1,2.27/2 = 16,2\ cm^2 \rightarrow A_{st} = 16,2\ cm^2$

$a \geq \frac{\gamma_{ov}.A_{st}.f_y}{2.l_w.f_{vw,d}} = \frac{1,25.16,2.35,5}{2.41,8.18,22} = 0,47\ cm$

$\rightarrow$ ***a = 5 mm***

$l_w = h_w - 20mm = 418\ mm = 41,8\ cm$

За HE400A:

$A_{st} = t_r.b_r/2 = 1,0.27,8/2 = 13,9\ cm^2 \rightarrow A_{st} = 13,9\ cm^2$

$a \geq \frac{\gamma_{ov}.A_{st}.f_y}{2.l_w.f_{vw,d}} = \frac{1,25.13,9.35,5}{2.27,8.18,22} = 0,6\ cm$

$\rightarrow$ ***a = 6 mm***

$l_w = h_w - 20mm = 278\ mm = 27,8\ cm$

- Welds between flange and stiffener

Weld should resist the following force:

$F_{Ed} = \gamma_{ov}.A_{st}.f_y/4$

$l_w = b_f/2 - t_w - r$

$a \geq \frac{\gamma_{ov}.A_{st}.f_y}{2.l_w.f_{vw,d}.4}$

<u>За HE550B:</u>

$A_{st} = 16{,}2\ cm^2$

$l_w = b_f/2 - t_w - r = 300/2 - 15 - 27 = 108\ mm = 10{,}8\ cm$

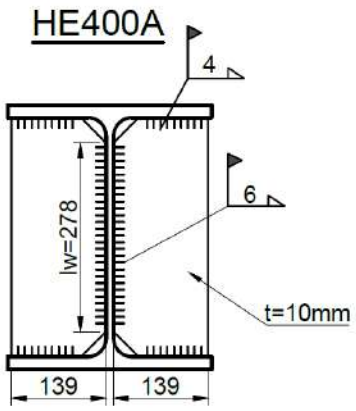
$a \geq \frac{\gamma_{ov} \cdot A_{st} \cdot f_y}{2.l_w \cdot f_{vw,d} \cdot 4} = \frac{1{,}25.16{,}2.35{,}5}{2.10{,}8.18{,}22.4} = 0{,}46\ cm \rightarrow \boldsymbol{a = 5\ mm}$

<u>За HE400A:</u>

$A_{st} = 13{,}9\ cm^2$

$l_w = b_f/2 - t_w - r = 300/2 - 11 - 27 = 112\ mm = 11{,}2\ cm$

$a \geq \frac{\gamma_{ov} \cdot A_{st} \cdot f_y}{2.l_w \cdot f_{vw,d} \cdot 4} = \frac{1{,}25.13{,}9.35{,}5}{2.11{,}2.18{,}22.4} = 0{,}38\ cm \rightarrow \boldsymbol{a = 4\ mm}$



4.4. <u>Beams of EBF 4 – PB52</u>

1.1.9. Design checks

Beams part of the EBFs are considered non-dissipative . It is assumed that they operate in elastic stage and plastic deformations and yielding is not allowed. This acceptance is realized in practice by capacitively multiplying the efforts in the elements. The efforts caused by the seismic impact shall be multiplied according to the following formula:

$\boldsymbol{M_{Rd}\ (N_{Ed}, V_{Ed}) \geq M_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.M_{Ed,E}}$

Където:

$\boldsymbol{M_{Rd}\ (N_{Ed}, V_{Ed})}$ – design bending resistance of the beam with the axial force and shear force taken into account;

The influence of both axial and shear force should not be taken into consideration if the following is satisfied:

$N_{Ed} \leq 0{,}15.N_{Rd}$

$V_{Ed} \leq 0{,}5.V_{Rd}$

$\gamma_{ov} = 1{,}25$

$M_{Rd} = f_y.W/\gamma_{M0}$ – design bending resistance of the beam;

$V_{Rd} = f_y.A_{vz}/(\gamma_{M0}.\sqrt{3})$ – design shear resistance of the beam;

$N_{Rd} = f_y.A/\gamma_{M0}$ – design compression/ tensile resistance of the beam;

$\gamma_{M0} = 1{,}05$

1.1.10. Beams cross section characteristics

Характеристики на напречното сечение			
Греди - HEA800			
A [cm^2]	***Avz [cm^2]***	***Wy [cm^3]***	***fy [kN/cm^2]***
285,8	138,8	8699	35,5
Греди -HEA700			
A [cm^2]	***Avz [cm^2]***	***Wy [cm^3]***	***fy [kN/cm^2]***
260,5	117	7032	35,5
Греди - HEA650			
A [cm^2]	***Avz [cm^2]***	***Wy [cm^3]***	***fy [kN/cm^2]***
241,6	103,2	6136	35,5
Греди - HEA600			
A [cm^2]	***Avz [cm^2]***	***Wy [cm^3]***	***fy [kN/cm^2]***
226,5	93,21	5350	35,5
Греди - HEA550			
A [cm^2]	***Avz [cm^2]***	***Wy [cm^3]***	***fy [kN/cm^2]***
211,8	83,72	4622	35,5
Греди - HEA500			
A [cm^2]	***Avz [cm^2]***	***Wy [cm^3]***	***fy [kN/cm^2]***
197,5	74,72	3949	35,5

The cross sections for the beams are selected so that when connecting them with link elements through a flange connection, to be able to place a bolt between the flanges of the two sections.

1.1.11. Presentation of the design results

Оразмеряване на ригелите на EBF в рамка по ос "C"

Етаж	***Напр. сечение***	***Клас стом.***	$N_{Ed,G}$ [kN]	$N_{Ed,E}$ [kN]	$\boldsymbol{N_{Ed}}$ [kN]	$V_{Ed,G}$ [kN]	$V_{Ed,E}$ [kN]	$\boldsymbol{V_{Ed}}$ [kN]	$M_{Ed,G}$ [kNm]	$M_{Ed,E}$ [kNm]	$\boldsymbol{M_{Ed}}$ [kNm]	$\boldsymbol{N_{Rd}}$ [kN]	$\boldsymbol{V_{Rd}}$ [kN]	$\boldsymbol{M_{Rd}}$ [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$
Ет. 31	**HE500A**	S355	72	152	*385*	57	19	*96*	46	62	*175*	7161	1634	1563	0,00	0,04	0,11
Ет. 30'		S355	11	121	*260*	72	23	*118*	50	66	*186*	7161	1634	1563	0,01	0,04	0,12
Ет. 30		S355	46	18	*82*	73	24	*122*	46	89	*230*	7161	1634	1563	0,00	0,05	0,15
Ет. 29		S355	50	32	*116*	74	26	*127*	44	105	*260*	7161	1634	1563	0,01	0,06	0,17
Ет. 28	**HE550A**	S355	51	22	*96*	75	28	*133*	42	116	*280*	7161	1634	1563	0,01	0,07	0,18
Ет. 27		S355	52	19	*92*	76	31	*140*	39	124	*295*	7161	1634	1563	0,01	0,08	0,19
Ет. 26		S355	53	19	*91*	77	33	*146*	37	130	*305*	7658	1819	1809	0,01	0,07	0,17
Ет. 25	**HE600A**	S355	53	18	*90*	78	36	*153*	35	135	*313*	7658	1819	1809	0,01	0,07	0,17
Ет. 24		S355	51	15	*83*	79	38	*159*	32	140	*320*	7658	1819	1809	0,01	0,08	0,18
Ет. 23		S355	43	15	*74*	81	39	*162*	30	142	*322*	7658	1819	1809	0,01	0,08	0,18
Ет. 22		S355	42	85	*218*	81	34	*152*	28	133	*301*	7658	1819	1809	0,01	0,07	0,17
Ет. 21		S355	111	163	*447*	88	40	*169*	22	131	*291*	7658	1819	1809	0,01	0,07	0,16
Ет. 20	**HE650A**	S355	30	209	*461*	81	53	*190*	32	144	*328*	7658	1819	1809	0,01	0,08	0,18
Ет. 19		S355	43	20	*83*	86	47	*183*	18	146	*319*	8168	2014	2075	0,00	0,07	0,15
Ет. 18		S355	54	64	*185*	86	40	*168*	18	132	*290*	8168	2014	2075	0,01	0,07	0,14
Ет. 17		S355	54	39	*134*	88	38	*166*	20	121	*269*	8168	2014	2075	0,01	0,06	0,13
Ет. 16		S355	54	12	*78*	89	43	*178*	22	123	*275*	8168	2014	2075	0,01	0,06	0,13
Ет. 15		S355	56	11	*79*	90	45	*183*	23	128	*287*	8168	2014	2075	0,01	0,06	0,14
Ет. 14		S355	57	11	*79*	91	47	*189*	25	135	*303*	8168	2014	2075	0,01	0,07	0,15
Ет. 13		S355	58	10	*78*	93	49	*194*	27	143	*323*	8168	2014	2075	0,01	0,07	0,16

Ет. 12	HE700A	S355	60	8	*77*	94	51	*200*	30	153	*345*	8807	2284	2377	0,01	0,07	0,14
Ет. 11		S355	62	9	*80*	95	53	*204*	32	161	*365*	8807	2284	2377	0,01	0,07	0,15
Ет. 10		S355	60	9	*79*	97	51	*202*	34	165	*374*	8807	2284	2377	0,01	0,07	0,16
Ет. 9		S355	45	99	*249*	95	40	*178*	29	149	*337*	8807	2284	2377	0,01	0,07	0,14
Ет. 8		S355	101	202	*518*	98	59	*220*	25	145	*323*	8807	2284	2377	0,01	0,06	0,14
Ет. 7		S355	24	212	*462*	100	80	*265*	30	202	*446*	8807	2284	2377	0,01	0,09	0,19
Ет. 6		S355	41	13	*67*	108	75	*263*	41	235	*527*	8807	2284	2377	0,00	0,10	0,22
Ет. 5	HEA800A	S355	78	31	*143*	114	80	*279*	55	270	*612*	9663	2709	2941	0,00	0,10	0,21
Ет. 4		S355	113	20	*154*	118	83	*290*	69	303	*694*	9663	2709	2941	0,01	0,11	0,24
Ет. 3		S355	167	56	*281*	124	88	*306*	87	336	*781*	9663	2709	2941	0,01	0,12	0,27
Ет. 2		S355	199	361	*944*	141	41	*225*	132	273	*696*	9663	2709	2941	0,02	0,10	0,24

$$\Omega = 1{,}5 \rightarrow E_{Ed} = E_{Ed,G} + 1{,}1.1{,}25.1{,}5.E_{Ed,E}$$

The design inner forces for the beams are calculated according to the formula above. The following figures show typical diagrams of shear forces and bending moments as a result of seismic effect and vertical loads (respectively $E_{Ed,G}$ и $E_{Ed,E}$), obtained from the analysis of the structure. In this case the efforts are for an element of the 2nd floor.

Inner forces due to vertical loads

Inner forces due to seismic effect

4.5. Braces of EBF 4 – D4

1.1.12. Design checks

Braces of the EBFs belong to the elements that do not contain seismic link element and are calculated to resist the capacitively increased values of seismic forces and those of gravitational loads, which are part of seismic combination. The calculations are performed with increased inner forces, according to БДС EN 1998 - 1, item 6.8.2., Analogous to the calculations for the beams in item 1.2.

Diagonal elements are designed for non-centric loads. The influence of shear force should not be taken into account if: $V_{Ed} \leq 0{,}5.V_{Rd}$

$$N_{Ed} = N_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.N_{Ed,E}$$

$$V_{Ed} = V_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.V_{Ed,E}$$

$$M_{Ed} = M_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.M_{Ed,E}$$

The following requirement should be satisfied:

$$\sigma_{max} = \frac{N_{Ed}}{A.\chi_z} + \frac{M_{Ed,y}}{W_y} \leq f_y/\gamma_{M0}\ ,$$

Where:

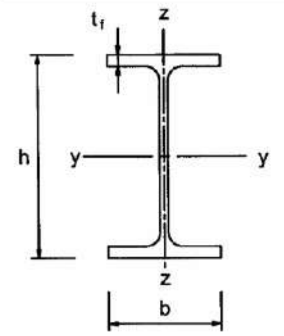
$$\chi_z = \frac{1}{\Phi+\sqrt{\Phi^2+\lambda^2}} \leq 1{,}0$$

$$\Phi = 0{,}5.[1+\alpha.(\bar{\lambda}-0{,}2) + \lambda^2]$$

α – imperfection factor

Крива на изкълчване	a_0	a	b	c	d
Коефициент за несъвършенства α	0,13	0,21	0,34	0,49	0,76

Buckling curve for sections HEA и HEB is determined as it follows:



Валцувани сечения					
	h/b>1,2	$t_f \leq 40$ mm	y-y z-z	a b	a_0 a_0
		40 mm < $t_f \leq 100$	y-y z-z	b c	a a
	h/b≤1,2	$t_f \leq 100$ mm	y-y z-z	b c	a a
		$t_f > 100$ mm	y-y z-z	d d	c c

Buckling curve is ***c*** → $\underline{\alpha = 0{,}49}$

$$\bar{\lambda} = \sqrt{\frac{A.f_y}{N_{cr}}};\quad N_{cr} = \frac{\pi^2.E.I}{l_{eff}^2}$$

$L_{eff} = \beta.L$ – effective length of the brace;

To increase safety it's accepted:

$\beta = 1{,}0 \rightarrow L_{eff} = L$

1.1.13. Cross sections of the braces

Характеристики на напречното сечение							
Диагонали - HE650B				**Диагонали - HE400A**			
A [cm²]	*Avz [cm²]*	*Wy [cm²]*	fy [kN/cm²]	*A [cm²]*	*Avz [cm²]*	*Wy [cm²]*	fy [kN/cm²]
286,3	122	7320	35,5	159	57,33	2562	35,5
Диагонали - HE650A				**Диагонали - HE300A**			
A [cm²]	*Avz [cm²]*	*Wy [cm²]*	fy [kN/cm²]	*A [cm²]*	*Avz [cm²]*	*Wy [cm²]*	fy [kN/cm²]
241,6	103,2	6136	35,5	112,5	37,28	1383	35,5
Диагонали - HE500A				**Диагонали - HE240A**			
A [cm²]	*Avz [cm²]*	*Wy [cm²]*	fy [kN/cm²]	*A [cm²]*	*Avz [cm²]*	*Wy [cm²]*	fy [kN/cm²]
197,5	74,72	3949	35,5	76,8	25,18	744,6	35,5

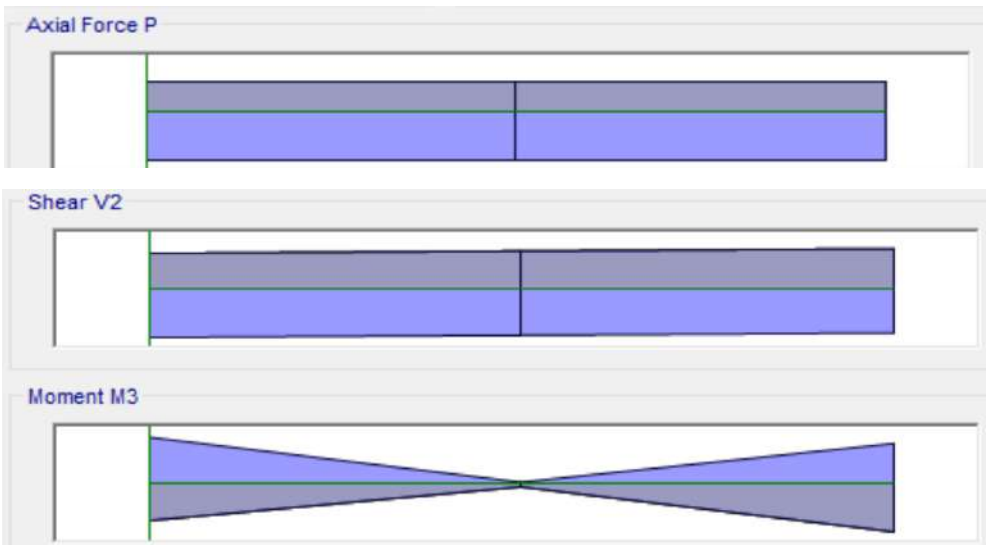
1.1.14. Presentation of the results

$\Omega = 1{,}5 \rightarrow E_{Ed} = N_{Ed,G} + 1{,}1.1{,}25.1{,}5.N_{Ed,E}$

Оразмеряване на диагоналите от EBF в рамка по ос "C"

Етаж	Напр. сечение	Клас стом.	N_{Ed} [kN]	V_{Ed} [kN]	$M_{Ed,y}$ [kNm]	N_{Rd} [kN]	V_{Rd} [kN]	$M_{Rd,y}$ [kNm]	l [m]	χ_z [-]	$\frac{N_{Ed}}{N_{Rd}\cdot\chi_z}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$	$\frac{\sigma_{Ed}}{f_{y,Rd}}$
Ет. 31'	HE300A	S355	*1004*	*20*	*152*	3804	728	468	5,7	0,79	0,33	0,03	0,33	***0,66***
Ет. 31		S355	*1135*	*45*	*240*	3804	728	468	7,1	0,71	0,42	0,06	0,51	***0,93***
Ет. 30'		S355	*823*	*33*	*185*	3804	728	468	6,5	0,74	0,29	0,05	0,40	***0,69***
Ет. 30		S355	*955*	*35*	*187*	3804	728	468	6,5	0,74	0,34	0,05	0,40	***0,74***
Ет. 29		S355	*1169*	*41*	*220*	3804	728	468	6,5	0,74	0,41	0,06	0,47	***0,88***
Ет. 28	HE400A	S355	*1366*	*47*	*251*	5376	1119	866	6,5	0,74	0,34	0,04	0,29	***0,63***
Ет. 27		S355	*1548*	*53*	*280*	5376	1119	866	6,5	0,84	0,34	0,05	0,32	***0,67***
Ет. 26		S355	*1723*	*57*	*308*	5376	1119	866	6,5	0,84	0,38	0,05	0,36	***0,74***
Ет. 25		S355	*1894*	*62*	*337*	5376	1119	866	6,5	0,84	0,42	0,06	0,39	***0,81***
Ет. 24		S355	*2062*	*67*	*364*	5376	1119	866	6,5	0,84	0,46	0,06	0,42	***0,88***
Ет. 23		S355	*2199*	*73*	*385*	5376	1119	866	6,5	0,84	0,49	0,06	0,44	***0,93***
Ет. 22		S355	*2173*	*77*	*369*	5376	1119	866	6,5	0,84	0,48	0,07	0,43	***0,91***
Ет. 21		S355	*1896*	*63*	*376*	5376	1119	866	7,1	0,81	0,43	0,06	0,43	***0,87***
Ет. 20	HE500A	S355	*2440*	*74*	*441*	6677	1459	1335	6,5	0,89	0,41	0,05	0,33	***0,74***
Ет. 19		S355	*2748*	*90*	*473*	6677	1459	1335	6,5	0,89	0,46	0,06	0,35	***0,82***
Ет. 18		S355	*2622*	*89*	*444*	6677	1459	1335	6,5	0,84	0,47	0,06	0,33	***0,80***
Ет. 17		S355	*2452*	*82*	*425*	6677	1459	1335	6,5	0,84	0,44	0,06	0,32	***0,76***
Ет. 16		S355	*2529*	*83*	*445*	6677	1459	1335	6,5	0,89	0,42	0,06	0,33	***0,76***
Ет. 15		S355	*2667*	*87*	*467*	6677	1459	1335	6,5	0,89	0,45	0,06	0,35	***0,80***
Ет. 14		S355	*2821*	*91*	*492*	6677	1459	1335	6,5	0,89	0,47	0,06	0,37	***0,84***
Ет. 13		S355	*2985*	*95*	*517*	6677	1459	1335	6,5	0,89	0,50	0,07	0,39	***0,89***
Ет. 12		S355	*3154*	*99*	*543*	6677	1459	1335	6,5	0,89	0,53	0,07	0,41	***0,94***
Ет. 11		S355	*3317*	*104*	*567*	6677	1459	1335	6,5	0,89	0,56	0,07	0,42	***0,98***
Ет. 10		S355	*3415*	*111*	*576*	6677	1459	1335	6,5	0,89	0,57	0,08	0,43	***1,00***
Ет. 9		S355	*3134*	*112*	*514*	6677	1459	1335	6,5	0,89	0,53	0,08	0,39	***0,91***
Ет. 8		S355	*2440*	*82*	*371*	6677	1459	1335	7,1	0,93	0,39	0,06	0,28	***0,67***
Ет. 7	HE650A	S355	*3179*	*99*	*678*	8168	2014	2075	7,1	0,93	0,42	0,05	0,33	***0,75***
Ет. 6		S355	*3472*	*110*	*739*	8168	2014	2075	7,4	0,92	0,46	0,05	0,36	***0,82***
Ет. 5		S355	*3814*	*117*	*788*	8168	2014	2075	7,4	0,92	0,51	0,06	0,38	***0,89***
Ет. 4		S355	*4183*	*126*	*832*	8168	2014	2075	7,4	0,92	0,56	0,06	0,40	***0,96***
Ет. 3		S355	*4555*	*132*	*883*	9680	2381	2075	7,4	0,92	0,51	0,06	0,43	***0,94***
Ет. 2	**HE650B**	S355	*4129*	*175*	*660*	8168	2014	2475	8,2	0,90	0,56	0,09	0,27	***0,83***

Diagrams of the design inner forces in a brace



4.6. Column C-C3

1.1.15. Design checks

Analogous to the calculations for the braces, the column, which is part of the EBF should be calculated to resist the capacitively increased values with coefficient $\Omega_{min} = 1{,}5$, define on the basis of cross sections of link elements

Columns are designed to resist bi-axial bending with compression. Influence of the shear force should not be taken into consideration if: $V_{Ed} \leq 0{,}5.V_{Rd}$

$N_{Ed} = N_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.N_{Ed,E}$

$V_{Ed} = V_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.V_{Ed,E}$

$M_{Ed} = M_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.M_{Ed,E}$

Buckling resistance of the columns should be also checked. The following conditions should be satisfied:

$$\frac{N_{Ed}}{\frac{\chi_y.N_{Rk}}{\gamma_{M1}}} + \frac{k_{yy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{yz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$\frac{N_{Ed}}{\frac{\chi_z.N_{Rk}}{\gamma_{M1}}} + \frac{k_{zy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{zz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1,$$

Where:

$N_{Rk} = f_y.A$; $M_{y,Rk} = f_y.W_y$; $M_{z,Rk} = f_y.W_z$ - bending and axial resistances of the cross section

k_{yy}, k_{zz}, k_{yz} и k_{zy} – interaction coefficients

Columns, which are part of the EBFs and MRFs are heavily loaded with compression force and bending moment due to seismic motion. This leads to the use of non-standard composite sections for heavily loaded columns. Such sections are not sensitive to buckling and for them the value of the displacement coefficient $\chi_{Lt} > 1{,}0$, which allows it to be assumed $\chi_{Lt} = 1{,}0$. For sections of this type, the values of the interaction coefficients are not defined by EC 1993-1-1. To increase safety and for more proper calculations in the present project, the values of these coefficients are accepted:

$k_{yy} = k_{zz} = k_{yz} = k_{zy} = 1{,}0$

All these assumptions allow the above formulas containing the coefficients of interaction to be reduced to a modified, simplified formula:

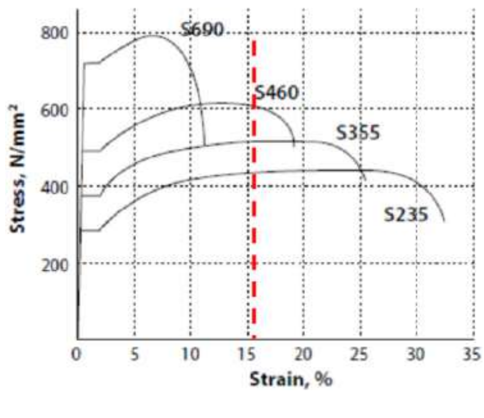
$$\boldsymbol{\sigma_{max} = \frac{N_{Ed}}{A.\chi} + \frac{M_{Ed,y}}{W_{pl,y}} + \frac{M_{Ed,z}}{W_{pl,z}} \leq f_y/\gamma_{M1}}$$

The values of buckling factor are equal for the main axes, because of the double symmetry and the equal inertial characteristics of the cross sections in both directions.

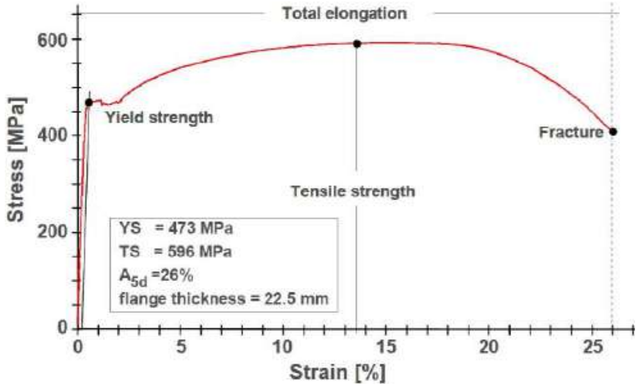
1.1.16. Characteristics of the cross sections

The columns are designed with high-strength steel *HISTAR – S460.* This type of steel is characterized by lower alloy content, good weldability and excellent behavior at low temperatures.

With the increase of the steel grade (tensile strength), the yielding zone in the stress-deformation diagram of the steel decreases and the tensile strength is reached faster, ie. steel has a more fragile behavior.



However, in HISTAR high - strength steel, the relative strains when tensile strength is reached are significantly higher and the steel has a high dissipative behavior. Such characteristics of steel are suitable for heavily loaded columns in buildings, located in seismic active areas. For such structures it is necessary to guarantee dissipative behavior.



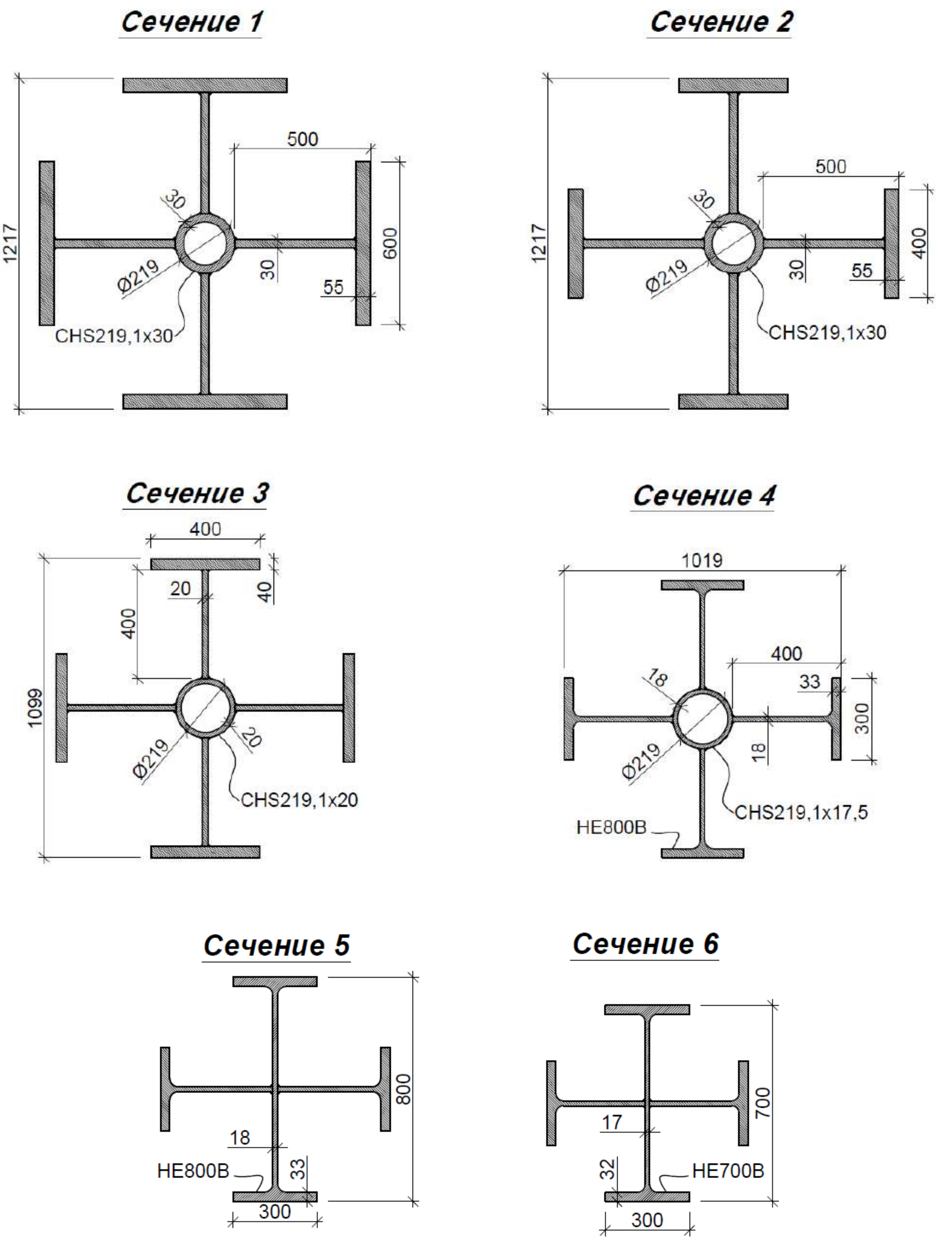
S355 steel is also included in the composite cross-sections, as European *CHS* profiles are used, which are not available from the high-strength steel manufacturer *HISTAR*.

➢ *Geometrical and inertial characteristics of the cross sections*

Характеристики на напречното сечение										
Колони-2x900X600X55X30+CHS219,1X30 - 1										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A csh***	***W csh***	***f y,csh***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
121,7	1843,4	268	2680407	2680407	44049	44049	46	178	1080	35,5
Колони-2x900X400X55X30+CHS219,1X30 - 2										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A csh***	***W csh***	***f y,csh***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
121,7	1576,6	268	1886296	1886296	30999	30999	46	178	1080	35,5
Колони-2x800X400X40X20+CHS219,1X20 - 3										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A csh***	***W csh***	***f y,csh***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
109,9	960,2	160	1115156	1115156	20294	20294	46	125	795	35,5
Колони-2xHE800B+CHS219,1X17,5 - 4										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A csh***	***W csh***	***f y,csh***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
101,9	779,25	161,8	638211	638211,3	12525	12525	46	111	713	35,5
Колони-2xHE800B - 5										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***			
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]			
80	665	161,8	373987	373986,5	9350	9350	46			
Колони-2xHE700B - 6										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***			
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]			
70	609,9	137,1	271328	271328	7752	7752	46			

- *Types of cross sections*

The inner forces in the columns decrease in height, which requires different sections within the storeys in order to achieve a more economical construction. The different sections, the inertial characteristics of which are described in the previous point, are presented graphically as follows:



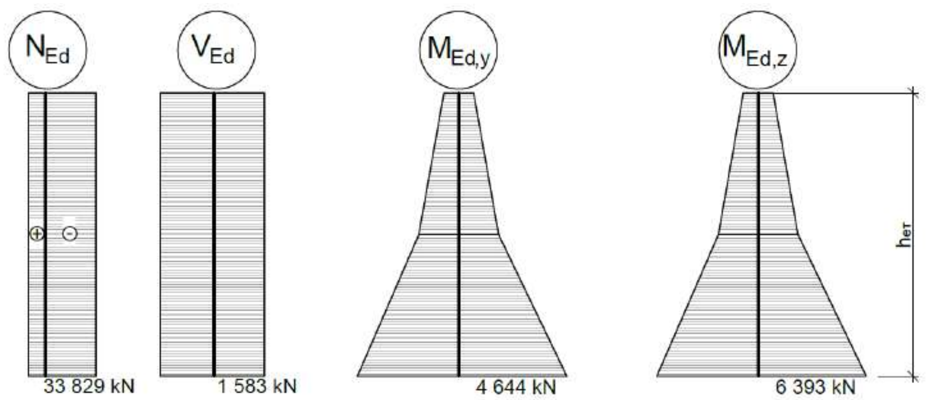
1.1.17. Presentation of the results for the inner forces

Ω =1,5 → $E_{Ed} = N_{Ed,G} + 1{,}1.1{,}25.1{,}5.N_{Ed,E}$

Оразмеряване на колона С-С3

Етаж	Напр. сечение	Клас стомана	N_{Ed} [kN]	V_{Ed} [kN]	$M_{Ed,y}$ [kNm]	$M_{Ed,z}$ [kNm]	$N_{b,Rd}$ [kN]	V_{Rd} [kN]	$M_{Rd,y}$ [kNm]	$M_{Rd,z}$ [kNm]	$\frac{N_{Ed}}{N_{Rd}\cdot\chi}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{y,Ed}}{M_{y,Rd}}$	$\frac{M_{z,Ed}}{M_{z,Rd}}$	$\frac{\sigma_{Ed}}{f_{y,Rd}}$
Ет. ПП	5	S460	*250*	*913*	*196*	*1288*	28746	3468	3396	3396	0,01	0,26	0,06	0,38	***0,45***
Ет. П3		S460	*955*	*687*	*446*	*1244*	28746	3468	3396	3396	0,04	0,20	0,13	0,37	***0,53***
Ет. П2		S460	*1425*	*704*	*371*	*1098*	28746	3468	3396	3396	0,06	0,20	0,11	0,32	***0,49***
Ет. П1		S460	*1968*	*491*	*383*	*1271*	28746	3468	3396	3396	0,08	0,14	0,11	0,37	***0,56***
Ет. К		S460	*2138*	*236*	*729*	*368*	28746	3468	3396	3396	0,08	0,07	0,21	0,11	***0,41***
Ет. 31'		S460	*2394*	*869*	*371*	*1232*	28746	3468	3396	3396	0,09	0,25	0,11	0,36	***0,56***
Ет. 31	4	S460	*3346*	*600*	*695*	*1347*	31343	4092	4096	4096	0,12	0,15	0,17	0,33	***0,62***
Ет. 30'		S460	*4146*	*830*	*324*	*1565*	31343	4092	4096	4096	0,15	0,20	0,08	0,38	***0,61***
Ет. 30		S460	*4806*	*900*	*436*	*1731*	31343	4092	4096	4096	0,17	0,22	0,11	0,42	***0,70***
Ет. 29		S460	*5415*	*972*	*339*	*1891*	31343	4092	4096	4096	0,19	0,24	0,08	0,46	***0,74***
Ет. 28		S460	*6007*	*1040*	*348*	*2041*	31343	4092	4096	4096	0,21	0,25	0,08	0,50	***0,80***
Ет. 27		S460	*6600*	*1105*	*391*	*2185*	31343	4092	4096	4096	0,23	0,27	0,10	0,53	***0,86***
Ет. 26		S460	*7205*	*1168*	*433*	*2323*	31343	4092	4096	4096	0,26	0,29	0,11	0,57	***0,93***
Ет. 25		S460	*7834*	*1227*	*476*	*2455*	31343	4092	4096	4096	0,28	0,30	0,12	0,60	***0,99***
Ет. 24	3	S460+S355	*8497*	*1284*	*524*	*2580*	40765	4092	5728	5728	0,23	0,31	0,09	0,45	***0,77***
Ет. 23		S460+S355	*9204*	*1335*	*604*	*2689*	40765	4092	5728	5728	0,25	0,33	0,11	0,47	***0,83***
Ет. 22		S460+S355	*9927*	*1387*	*755*	*2780*	40765	4092	5728	5728	0,27	0,34	0,13	0,49	***0,89***
Ет. 21		S460+S355	*10561*	*1338*	*386*	*3049*	40765	4092	5728	5728	0,29	0,33	0,07	0,53	***0,89***
Ет. 20		S460+S355	*11334*	*1473*	*604*	*3059*	40765	4092	5728	5728	0,31	0,36	0,11	0,53	***0,95***
Ет. 19	2	S460+S355	*12241*	*1511*	*825*	*3130*	49803	4047	9159	9159	0,27	0,37	0,09	0,34	***0,70***
Ет. 18		S460+S355	*13179*	*1528*	*790*	*3177*	49803	4047	9159	9159	0,29	0,38	0,09	0,35	***0,73***
Ет. 17		S460+S355	*14108*	*1541*	*646*	*3198*	49803	4047	9159	9159	0,31	0,38	0,07	0,35	***0,73***
Ет. 16		S460+S355	*15061*	*1573*	*676*	*3256*	49803	4047	9159	9159	0,34	0,39	0,07	0,36	***0,77***
Ет. 15		S460+S355	*16073*	*1598*	*710*	*3312*	49803	4047	9159	9159	0,36	0,39	0,08	0,36	***0,80***
Ет. 14		S460+S355	*17156*	*1621*	*754*	*3365*	49803	4047	9159	9159	0,38	0,40	0,08	0,37	***0,83***
Ет. 13		S460+S355	*18325*	*1641*	*804*	*3414*	49803	4047	9159	9159	0,41	0,41	0,09	0,37	***0,87***
Ет. 12		S460+S355	*19590*	*1658*	*862*	*3457*	49803	4047	9159	9159	0,44	0,41	0,09	0,38	***0,91***
Ет. 11		S460+S355	*20957*	*1671*	*924*	*3494*	49803	4047	9159	9159	0,47	0,41	0,10	0,38	***0,95***
Ет. 10	1	S460+S355	*22401*	*1683*	*1105*	*3538*	80783	6779	13946	13946	0,31	0,25	0,08	0,25	***0,64***
Ет. 9		S460+S355	*23713*	*1681*	*1239*	*3513*	80783	6779	13946	13946	0,33	0,25	0,09	0,25	***0,67***
Ет. 8		S460+S355	*24656*	*1400*	*818*	*3116*	80783	6779	13946	13946	0,34	0,21	0,06	0,22	***0,62***
Ет. 7		S460+S355	*25522*	*1534*	*1120*	*3682*	80783	6779	13946	13946	0,35	0,23	0,08	0,26	***0,70***
Ет. 6		S460+S355	*26570*	*1380*	*1065*	*3778*	80783	6779	13946	13946	0,37	0,20	0,08	0,27	***0,71***
Ет. 5		S460+S355	*27829*	*1368*	*1167*	*3797*	80783	6779	13946	13946	0,38	0,20	0,08	0,27	***0,74***
Ет. 4		S460+S355	*29373*	*1334*	*1391*	*3745*	80783	6779	13946	13946	0,40	0,20	0,10	0,27	***0,77***
Ет. 3		S460+S355	*31174*	*1290*	*1129*	*3728*	80783	6779	13946	13946	0,43	0,19	0,08	0,27	***0,78***
Ет. 2	1+	S460+S355	*33829*	*1583*	*4644*	*6393*	93358	6779	19663	19663	0,40	0,23	0,24	0,33	***0,96***
Ет. 1		S460+S355	*34019*	*1177*	*3293*	*2688*	93358	6779	19663	19663	0,40	0,17	0,17	0,14	***0,71***
Ет. -1		S460+S355	*34330*	*270*	*730*	*456*	93358	6779	19663	19663	0,41	0,04	0,04	0,02	***0,47***
Ет. -2		S460+S355	*34138*	*143*	*270*	*166*	93385	6779	19663	19663	0,41	0,02	0,01	0,01	***0,43***

Diagrams of the design inner forces in the column at floor 2



4.7. *Column C-C4*

As well as *C-C3*, this column is also part of the EBF. It is calculated to resist the capacitively increased values with coefficient $\Omega_{min} = 1{,}5$, defined while designing the link elements.

1.1.18. Design checks

Design procedure from 1.4.1. can be applied here as well!

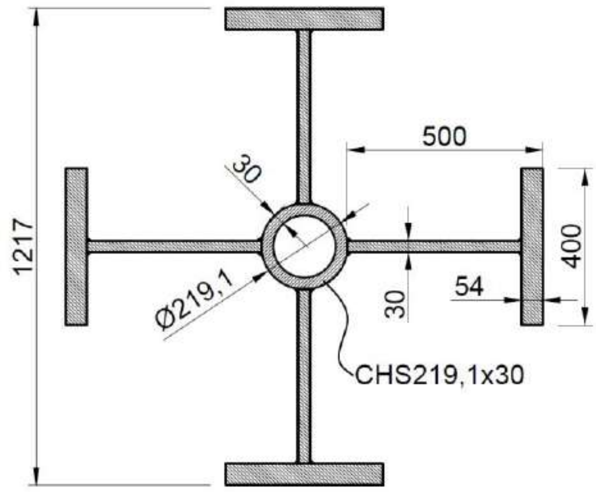
1.1.19. Characteristics of the cross sections

- *Geometrical and inertial characteristics*

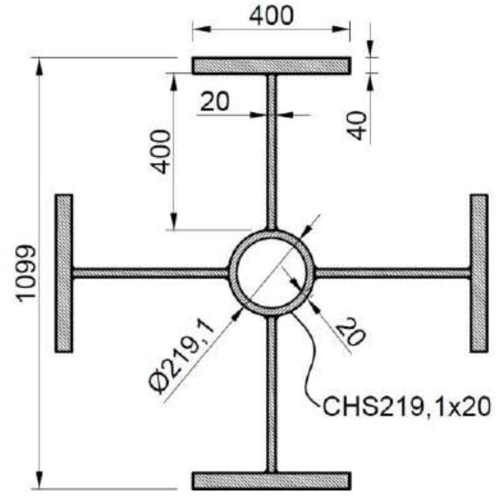
Характеристики на напречното сечение										
Колони-2x900X400X54X30+CHS219,1X30 - 1										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A_{csh}***	***W_{csh}***	***$f_{y,csh}$***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
121,7	1576,6	259,3	1886296	1886296	30999	30999	46	178	1080	35,5
Колони-2x800X400X40X20+CHS219,1X20 - 2										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A_{csh}***	***W_{csh}***	***$f_{y,csh}$***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
109,9	960,2	219,8	1115156	1115156	20294	20294	46	125	795	35,5
Колони-2xHE800B+CHS219,1X17,5 - 3										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A_{csh}***	***W_{csh}***	***$f_{y,csh}$***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
101,9	779,25	178,3	638211	638211	12525	12525	46	111	713	35,5
Колони-2xHE700B - 4										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***			
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]			
700	609,9	161,8	271328	271328	7752	7752	46			
Колони-2xHE600B - 5										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***			
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]			
60	537,5	161,8	184571	184571	6152	6152	46			
Колони-2xHE500B - 6										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***			
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]			
50	475	161,8	119799	119799	4792	4792	46			

- *Type of the cross sections*

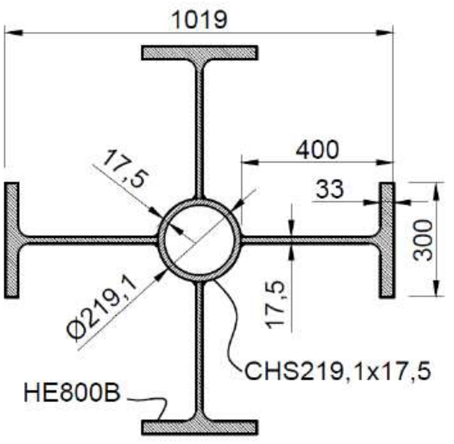
Сечение 1



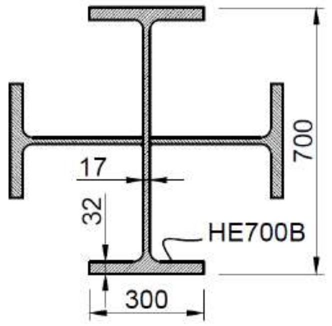
Сечение 2



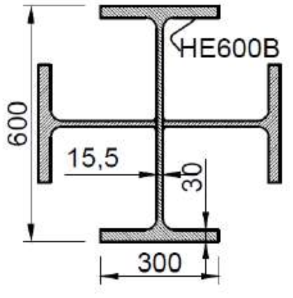
Сечение 3



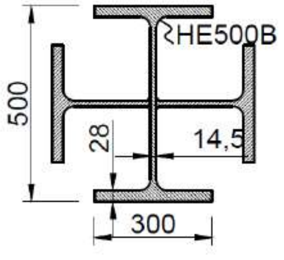
Сечение 4



Сечение 5



Сечение 6



1.1.20. Presentation of the results for inner forces

$\Omega = 1,5 \rightarrow E_{Ed} = N_{Ed,G} + 1,1.1,25.1,5.N_{Ed,E}$

Оразмеряване на колона С-С4

Етаж	Напр. сечение	Клас стомана	N_{Ed} [kN]	V_{Ed} [kN]	$M_{Ed,y}$ [kNm]	$M_{Ed,z}$ [kNm]	$N_{b,Rd}$ [kN]	V_{Rd} [kN]	$M_{Rd,y}$ [kNm]	$M_{Rd,z}$ [kNm]	$\frac{N_{Ed}}{N_{Rd} \cdot \chi}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{y,Ed}}{M_{y,Rd}}$	$\frac{M_{z,Ed}}{M_{z,Rd}}$	$\frac{\sigma_{Ed}}{f_{y,Rd}}$
Ет. ПП		S460	*97*	*363*	*284*	*645*	22388	4092	2099	2099	0,00	0,09	0,14	0,31	***0,45***
Ет. П3		S460	*325*	*434*	*771*	*657*	22388	4092	2099	2099	0,02	0,11	0,37	0,31	***0,70***
Ет. П2	**6**	S460	*898*	*325*	*446*	*724*	22388	4092	2099	2099	0,04	0,08	0,21	0,34	***0,60***
Ет. П1		S460	*1820*	*336*	*701*	*540*	22388	4092	2099	2099	0,09	0,08	0,33	0,26	***0,68***
Ет. К		S460	*1818*	*205*	*704*	*475*	22388	4092	2099	2099	0,09	0,05	0,34	0,23	***0,65***
Ет. 31'		S460	*1921*	*232*	*569*	*350*	22388	4092	2099	2099	0,10	0,06	0,27	0,17	***0,53***
Ет. 31		S460	*2643*	*219*	*648*	*342*	25334	4092	2695	2695	0,12	0,05	0,24	0,13	***0,48***
Ет. 30'		S460	*3899*	*329*	*788*	*448*	25334	4092	2695	2695	0,17	0,08	0,29	0,17	***0,63***
Ет. 30	**5**	S460	*4804*	*375*	*740*	*473*	25334	4092	2695	2695	0,21	0,09	0,27	0,18	***0,66***
Ет. 29		S460	*5538*	*415*	*754*	*506*	25334	4092	2695	2695	0,24	0,10	0,28	0,19	***0,71***
Ет. 28		S460	*6181*	*458*	*862*	*538*	25334	4092	2695	2695	0,27	0,11	0,32	0,20	***0,79***
Ет. 27		S460	*6766*	*497*	*956*	*571*	28746	4092	3396	3396	0,26	0,12	0,28	0,17	***0,71***
Ет. 26		S460	*7308*	*534*	*1042*	*603*	28746	4092	3396	3396	0,28	0,13	0,31	0,18	***0,77***
Ет. 25		S460	*7813*	*570*	*1123*	*634*	28746	4092	3396	3396	0,30	0,14	0,33	0,19	***0,82***
Ет. 24	**4**	S460	*8287*	*605*	*1207*	*677*	28746	4092	3396	3396	0,32	0,15	0,36	0,20	***0,87***
Ет. 23		S460	*8733*	*652*	*1348*	*738*	28746	6559	3396	3396	0,34	0,10	0,40	0,22	***0,95***
Ет. 22		S460	*9154*	*693*	*1567*	*715*	40765	4510	5728	5728	0,25	0,15	0,27	0,12	***0,65***
Ет. 21		S460	*9615*	*485*	*1032*	*1110*	40765	4510	5728	5728	0,26	0,11	0,18	0,19	***0,64***
Ет. 20		S460	*10597*	*789*	*1389*	*954*	40765	4510	5728	5728	0,29	0,18	0,24	0,17	***0,70***
Ет. 19		S460+S355	*11029*	*801*	*1677*	*992*	40765	4510	5728	5728	0,30	0,18	0,29	0,17	***0,77***
Ет. 18		S460+S355	*11405*	*736*	*1630*	*1028*	40765	4510	5728	5728	0,31	0,16	0,28	0,18	***0,77***
Ет. 17		S460+S355	*11815*	*669*	*1451*	*1046*	40765	4510	5728	5728	0,32	0,15	0,25	0,18	***0,76***
Ет. 16		S460+S355	*12311*	*720*	*1494*	*1087*	40765	4510	5728	5728	0,34	0,16	0,26	0,19	***0,79***
Ет. 15	**3**	S460+S355	*12798*	*742*	*1537*	*1129*	40765	4510	5728	5728	0,35	0,16	0,27	0,20	***0,81***
Ет. 14		S460+S355	*13252*	*763*	*1586*	*1173*	40765	4510	5728	5728	0,36	0,17	0,28	0,20	***0,84***
Ет. 13		S460+S355	*13662*	*784*	*1634*	*1218*	40765	4510	5728	5728	0,37	0,17	0,29	0,21	***0,87***
Ет. 12		S460+S355	*14022*	*802*	*1683*	*1264*	40765	4510	5728	5728	0,38	0,18	0,29	0,22	***0,90***
Ет. 11		S460+S355	*14323*	*819*	*1738*	*1308*	40765	4510	5728	5728	0,39	0,18	0,30	0,23	***0,92***
Ет. 10		S460+S355	*14552*	*868*	*1947*	*1389*	40765	4510	5728	5728	0,40	0,19	0,34	0,24	***0,98***
Ет. 9		S460+S355	*14688*	*859*	*2263*	*1266*	49803	5559	9159	9159	0,33	0,15	0,25	0,14	***0,71***
Ет. 8		S460+S355	*14660*	*664*	*1164*	*915*	49803	5559	9159	9159	0,33	0,12	0,13	0,10	***0,55***
Ет. 7		S460+S355	*15963*	*757*	*1650*	*1460*	49803	5559	9159	9159	0,36	0,14	0,18	0,16	***0,70***
Ет. 6	**2**	S460+S355	*16215*	*591*	*1425*	*1561*	49803	5559	9159	9159	0,36	0,11	0,16	0,17	***0,69***
Ет. 5		S460+S355	*16364*	*587*	*1506*	*1635*	49803	5559	9159	9159	0,37	0,11	0,16	0,18	***0,71***
Ет. 4		S460+S355	*16397*	*581*	*1564*	*1677*	49803	5559	9159	9159	0,37	0,10	0,17	0,18	***0,72***
Ет. 3		S460+S355	*16460*	*584*	*1516*	*1767*	49803	5559	9159	9159	0,37	0,11	0,17	0,19	***0,73***
Ет. 2		S460+S355	*16862*	*1010*	*4447*	*4921*	80783	6559	13946	13946	0,23	0,15	0,32	0,35	***0,90***
Ет. 1	**1**	S460+S355	*18742*	*1151*	*3368*	*2943*	80783	6559	13946	13946	0,26	0,18	0,24	0,21	***0,71***
Ет. -1		S460+S355	*18984*	*209*	*589*	*464*	80783	6559	13946	13946	0,26	0,03	0,04	0,03	***0,34***
Ет. -2		S460+S355	*19429*	*57*	*133*	*76*	80806	6559	13946	13946	0,27	0,01	0,01	0,01	***0,28***

2. Design of the elements of MRF

4.8. Beam PB53

The designed beam resists the loads applied together with the reinforced concrete slab and should be considered as composite beam. According to the prescriptions of БДС EN 1998-1, item 7.6.2., during a seismic event is necessary to maintain the integrity of the concrete slab, while yielding takes place in the bottom part of the steel section and/or in the reinforcement in the concrete slab. When the usage of the advantages of the composite cross-section of the beam is not foreseen for energy dissipation, only the steel cross-section may be taken into account for the calculation of the plastic bearing capacity of the beam in the areas intended for the formation of plastic hinges, according to БДС EN 1998-1, item 7.7.5. Such are the areas with large negative moments at the ends of the beams. There, the slab is considered completely separated from the steel frame in an area around the column with a diameter of $2.b_{eff}$ - twice the effective width of the reinforced concrete slab.

A calculation was performed only for a seismic combination (for maximum negative bending moment) and a check was made for a combination at the ultimate limit state containing only vertical loads (for maximum positive moment). The steel cross-section for the designed beam should be determined.

2.1.1. Design of the beam for negative bending moment

➢ *Design procedure*

Check for the bearing resistance of the beam:

$M_{y,Ed} \leq M_{y,Rd} = W_y.f_y/\gamma_{M0}$

Shear force and axial force may not be taken into consideration if the following requirements are satisfied:

$N_{Ed} \leq 0,15.N_{Rd}$

$V_{Ed} \leq 0,5.V_{Rd}$,

Където:

$V_{Ed} = V_{Ed,G} + V_{Ed,M}$

$V_{Ed,G}$ – Shear force, due to gravity loads in seismic combination;

$V_{Ed,M}$ – design value of the shear force, due to the plastic moments $M_{pl,Rd,A}$ и $M_{pl,Rd,B}$ applied in the end sections of the beam *A* и *B.*

$V_{Ed,M} = (M_{pl,Rd,A} + M_{pl,Rd,B})/L$ – most unpleasant condition for defining shear force, reffering to a beam with a span *L* and dissipative zones in the ends.

To increase safety, for defining $V_{Ed,M}$ is used the span between plastic hinges of the beam.

➢ *Characteristics of the cross sections*

Характеристики на напречното сечение							
Греди - HEA 600				**Греди - HEB 400**			
A [cm²]	*Avz [cm²]*	*Wy [cm³]*	**Fy [kN/cm²]**	*A [cm²]*	*Avz [cm²]*	*Wy [cm³]*	**Fy [kN/cm²]**
226,5	93,21	5350	35,5	159	57,33	3232	35,5
Греди - HEA550				**Греди - HEA 280**			
A [cm²]	*Avz [cm²]*	*Wy [cm³]*	**Fy [kN/cm²]**	*A [cm²]*	*Avz [cm²]*	*Wy [cm³]*	**Fy [kN/cm²]**
211,8	83,72	4622	35,5	97,3	31,74	1112	35,5

➢ *Presentation of the results*

Оразмеряване на греда PB53														
Етаж	**Напр. сечение**	**Клас стом.**	N_{Ed} [kN]	$V_{Ed,G}$ [kN]	$V_{Ed,M}$ [kN]	V_{Ed} [kN]	M_{Ed} [kNm]	N_{Rd} [kN]	V_{Rd} [kN]	M_{Rd} [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$	Ω [-]
Ет. П2	**HE400B**	S355	154	154	171	324	701	5376	1119	1093	0,03	0,29	0,64	**1,56**
Ет. П1		S355	196	160	171	331	782	5376	1119	1093	0,04	0,30	0,72	**1,40**
Ет. 31'		S355	-	-	-	-	-	-	-	-	-	-	-	**-**
Ет. 31		S355	637	231	171	430	654	5376	1119	1093	0,12	0,38	0,60	**1,67**
Ет. 30'		S355	594	260	171	341	740	5376	1119	1093	0,11	0,30	0,68	**1,48**
Ет. 30		S355	220	170	171	343	864	5376	1119	1093	0,04	0,31	0,79	**1,27**
Ет. 29		S355	79	172	171	343	973	5376	1119	1093	0,01	0,31	0,89	**1,12**
Ет. 28		S355	61	173	171	344	1059	5376	1119	1093	0,01	0,31	0,97	**1,03**
Ет. 27	**HE550A**	S355	59	173	244	417	1135	7161	1634	1563	0,01	0,26	0,73	**1,38**
Ет. 26		S355	58	173	244	417	1203	7161	1634	1563	0,01	0,26	0,77	**1,30**
Ет. 25		S355	58	173	244	416	1268	7161	1634	1563	0,01	0,25	0,81	**1,23**
Ет. 24		S355	59	172	244	416	1328	7161	1634	1563	0,01	0,25	0,85	**1,18**
Ет. 23		S355	53	171	244	414	1382	7161	1634	1563	0,01	0,25	0,88	**1,13**
Ет. 22		S355	174	170	244	466	1413	7161	1634	1563	0,02	0,29	0,90	**1,11**
Ет. 21		S355	732	222	244	545	1303	7161	1634	1563	0,10	0,33	0,83	**1,20**
Ет. 20		S355	824	301	244	410	1507	7161	1634	1563	0,12	0,25	0,96	**1,04**
Ет. 19		S355	241	166	244	408	1566	7161	1634	1563	0,03	0,25	1,00	**1,00**
Ет. 18	**HE600A**	S355	93	164	283	442	1590	7658	1819	1809	0,01	0,24	0,88	**1,14**
Ет. 17		S355	99	159	283	440	1531	7658	1819	1809	0,01	0,24	0,85	**1,18**
Ет. 16		S355	38	158	283	437	1528	7658	1819	1809	0,00	0,24	0,85	**1,18**
Ет. 15		S355	49	154	283	433	1557	7658	1819	1809	0,01	0,24	0,86	**1,16**
Ет. 14		S355	51	151	283	429	1587	7658	1819	1809	0,01	0,24	0,88	**1,14**
Ет. 13		S355	52	147	283	425	1615	7658	1819	1809	0,01	0,23	0,89	**1,12**
Ет. 12		S355	53	142	283	420	1640	7658	1819	1809	0,01	0,23	0,91	**1,10**
Ет. 11		S355	55	137	283	415	1657	7658	1819	1809	0,01	0,23	0,92	**1,09**
Ет. 10		S355	34	132	283	409	1663	7658	1819	1809	0,00	0,22	0,92	**1,09**
Ет. 9		S355	220	127	283	529	1610	7658	1819	1809	0,03	0,29	0,89	**1,12**
Ет. 8	**HE550A**	S355	906	246	244	577	1345	7161	1634	1563	0,13	0,35	0,86	**1,16**
Ет. 7		S355	703	333	244	373	1459	7161	1634	1563	0,10	0,23	0,93	**1,07**
Ет. 6		S355	102	129	244	360	1458	7161	1634	1563	0,01	0,22	0,93	**1,07**
Ет. 5		S355	84	115	244	343	1446	7161	1634	1563	0,01	0,21	0,93	**1,08**
Ет. 4		S355	114	98	244	326	1456	7161	1634	1563	0,02	0,20	0,93	**1,07**
Ет. 3		S355	150	81	244	303	1446	7161	1634	1563	0,02	0,19	0,93	**1,08**
Ет. 2		S355	307	59	244	258	1428	7161	1634	1563	0,04	0,16	0,91	**1,09**
Ет. 1	**HE320A**	S355	757	14	59	107	297	3290	620	376	0,23	0,17	0,79	**1,27**
Ет. -1		S355	561	48	59	105	149	3290	620	376	0,17	0,17	0,40	**2,52**
Ет. -2		S355	190	46	59	59	81	3290	620	376	0,06	0,09	0,21	**4,66**

2.1.2. Desogn of the beam for positive bending moment

The values of the positive bending moments in the beam in combination *1.35.G + 1.5.Q* do not exceed those of the negative ones, obtained in seismic combination. The steel section chosen to resist the negative moments would be able to resist the forces, result of ULS combination, on its own, without taking into account the assistance of the slab. The results for the moments of the two combinations are presented in the following table:

Extreme values of the negative bending moment (A_{Ed}) and positive bending moment (ULS)

Максимални отрицателни (A_{Ed}) и положителни моменти в гредата (ULS)					
Етаж	**Напр. сечение**	**Клас стом.**	$M_{ed,min}$ [kNm]	$M_{ed,max}$ [kNm]	M_{Rd} [kNm]
Ет. П2	**HE400B**	S355	701	496	1093
Ет. П1		S355	782	585	1093
Ет. 31'		S355	-	-	-
Ет. 31		S355	654	686	1093
Ет. 30'		S355	740	632	1093
Ет. 30		S355	864	571	1093
Ет. 29		S355	973	586	1093
Ет. 28		S355	1059	589	1093
Ет. 27	**HE550A**	S355	1135	592	1563
Ет. 26		S355	1203	592	1563
Ет. 25		S355	1268	592	1563
Ет. 24		S355	1328	589	1563
Ет. 23		S355	1382	585	1563
Ет. 22		S355	1413	579	1563
Ет. 21		S355	1303	734	1563
Ет. 20		S355	1507	680	1563
Ет. 19		S355	1566	557	1563
Ет. 18	**HE600A**	S355	1590	548	1809
Ет. 17		S355	1531	512	1809
Ет. 16		S355	1528	505	1809
Ет. 15		S355	1557	487	1809
Ет. 14		S355	1587	468	1809
Ет. 13		S355	1615	445	1809
Ет. 12		S355	1640	421	1809
Ет. 11		S355	1657	394	1809
Ет. 10		S355	1663	367	1809
Ет. 9		S355	1610	336	1809
Ет. 8	**HE550A**	S355	1345	704	1563
Ет. 7		S355	1459	854	1563
Ет. 6		S355	1458	334	1563
Ет. 5		S355	1446	262	1563
Ет. 4		S355	1456	171	1563
Ет. 3		S355	1446	85	1563
Ет. 2		S355	1428	87	1563
Ет. 1	**HE320A**	S355	297	1	376
Ет. -1		S355	149	121	376
Ет. -2		S355	81	115	376

4.9. *Beams PB54 и PB55*

The designed beam resists the loads applied together with the reinforced concrete slab and should be considered as composite beam. According to the prescriptions of БДС EN 1998-1, item 7.6.2., during a seismic event is necessary to maintain the integrity of the concrete slab, while yielding takes place in the bottom part of the steel section and/or in the reinforcement in the concrete slab.

When the usage of the advantages of the composite cross-section of the beam is not foreseen for energy dissipation, only the steel cross-section may be taken into account for the calculation of the plastic bearing capacity of the beam in the areas intended for the formation of plastic hinges, according to БДС EN 1998-1, item 7.7.5. Such are the areas with large negative moments at the ends of the beams. Within these areas, the slab is considered completely separated from the steel frame in an area around the column with a diameter of $2.b_{eff}$ - twice the effective wifth of the reinforced concrete slab.

The beam will be designed as steel for areas with large negative moments - the ends of the beams, receiving maximum bending moment in a design seismic combination (Accidental). In areas with a positive bending moment, the beam will be dimensioned as a composite steel - reinforced concrete beam. A calculation was performed only for a seismic combination (for maximum negative bending moment) and a check was made for a combination at the ultimate limit state containing only vertical loads (for maximum positive moment). The steel cross-section for the beam considered should be determined.

2.1.3. Design of the beams for negative bending moment

Design procedure is analogical to the one given in *point 2.1.1*.

- *Characteristics of the cross sections*

Характеристики на напречното сечение							
Греди - HEA 900				**Греди - HEA 700**			
A [cm^2]	*Avz [cm^2]*	*Wy [cm^3]*	**Fy [kN/cm^2]**	*A [cm^2]*	*Avz [cm^2]*	*Wy [cm^3]*	**Fy [kN/cm^2]**
320,5	163,3	10810	35,5	260,5	117	7032	35,5
Греди - HEA 800				**Греди - HEA 650**			
A [cm^2]	*Avz [cm^2]*	*Wy [cm^3]*	**Fy [kN/cm^2]**	*A [cm^2]*	*Avz [cm^2]*	*Wy [cm^3]*	**Fy [kN/cm^2]**
285,8	138,8	8699	35,5	241,6	103,2	6136	35,5
		Греди - HEA 500					
		A [cm^2]	*Avz [cm^2]*	*Wy [cm^3]*	**Fy [kN/cm^2]**		
		197,5	74,72	3949	35,5		

- *Presentation of the results*

Analogically to the calculations for beam *PB53*, the values of the coefficient Ω are tabulated, which should capacitive increase the seismic forces in the columns, part of the MRFs.

$\Omega = \Omega_{i,min} = M_{Rd}/M_{Ed}$

Оразмеряване на греда PB54 и PB55

Етаж	Напр. сечение	Клас стомана	N_{Ed}	$V_{Ed,G}$	$V_{Ed,M}$	V_{Ed}	M_{Ed}	N_{Rd}	V_{Rd}	M_{Rd}	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$	Ω
			[kN]	[kN]	[kN]	[kN]	[kNm]	[kN]	[kN]	[kNm]				-
Ет. 31'	H650A	S355	615	197	352	549	1597	8168	2014	2075	0,08	0,27	0,77	**1,30**
Ет. 31		S355	765	358	352	710	1561	8168	2014	2075	0,09	0,35	0,75	**1,33**
Ет. 30'		S355	829	387	352	739	1726	8168	2014	2075	0,10	0,37	0,83	**1,20**
Ет. 30		S355	186	379	352	731	2019	8168	2014	2075	0,02	0,36	0,97	**1,03**
Ет. 29	HE700A	S355	92	380	403	783	2135	8807	2284	2377	0,01	0,34	0,90	**1,11**
Ет. 28		S355	77	381	403	784	2228	8807	2284	2377	0,01	0,34	0,94	**1,07**
Ет. 27		S355	74	381	403	784	2308	8807	2284	2377	0,01	0,34	0,97	**1,03**
Ет. 26		S355	72	381	403	784	2379	8807	2284	2377	0,01	0,34	1,00	**1,00**
Ет. 25	HE800A	S355	71	381	498	879	2443	9663	2709	2941	0,01	0,32	0,83	**1,20**
Ет. 24		S355	68	380	498	879	2500	9663	2709	2941	0,01	0,32	0,85	**1,18**
Ет. 23		S355	62	380	498	878	2545	9663	2709	2941	0,01	0,32	0,87	**1,16**
Ет. 22		S355	134	379	498	877	2576	9663	2709	2941	0,01	0,32	0,88	**1,14**
Ет. 21		S355	1033	412	498	910	2471	9663	2709	2941	0,11	0,34	0,84	**1,19**
Ет. 20		S355	1148	576	498	1074	2791	9663	2709	2941	0,12	0,40	0,95	**1,05**
Ет. 19		S355	209	376	498	875	2681	9663	2709	2941	0,02	0,32	0,91	**1,10**
Ет. 18		S355	60	375	498	874	2700	9663	2709	2941	0,01	0,32	0,92	**1,09**

Оразмеряване на греда PB54 и PB55

Етаж	Напр. сечение	Клас стомана	N_{Ed}	$V_{Ed,G}$	$V_{Ed,M}$	V_{Ed}	M_{Ed}	N_{Rd}	V_{Rd}	M_{Rd}	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$	Ω
			[kN]	[kN]	[kN]	[kN]	[kNm]	[kN]	[kN]	[kNm]				-
Ет. 17	HE900A	S355	80	407	619	1027	3454	10836	3188	3655	0,01	0,32	0,95	**1,06**
Ет. 16		S355	60	406	619	1025	3446	10836	3188	3655	0,01	0,32	0,94	**1,06**
Ет. 15		S355	66	403	619	1023	3452	10836	3188	3655	0,01	0,32	0,94	**1,06**
Ет. 14		S355	67	401	619	1020	3460	10836	3188	3655	0,01	0,32	0,95	**1,06**
Ет. 13		S355	67	398	619	1017	3463	10836	3188	3655	0,01	0,32	0,95	**1,06**
Ет. 12		S355	67	395	619	1014	3459	10836	3188	3655	0,01	0,32	0,95	**1,06**
Ет. 11		S355	66	391	619	1011	3444	10836	3188	3655	0,01	0,32	0,94	**1,06**
Ет. 10		S355	56	388	619	1007	3411	10836	3188	3655	0,01	0,32	0,93	**1,07**
Ет. 9		S355	160	384	619	1004	3311	10836	3188	3655	0,01	0,31	0,91	**1,10**
Ет. 8	HE800A	S355	1297	245	619	864	2749	9663	3188	3655	0,13	0,27	0,75	**1,33**
Ет. 7		S355	932	739	619	1359	3173	9663	3188	3655	0,10	0,43	0,87	**1,15**
Ет. 6		S355	255	385	619	1005	2953	9663	3188	3655	0,03	0,32	0,81	**1,24**
Ет. 5		S355	138	377	619	997	2863	9663	3188	3655	0,01	0,31	0,78	**1,28**
Ет. 4		S355	113	366	619	986	2809	9663	3188	3655	0,01	0,31	0,77	**1,30**
Ет. 3		S355	156	355	619	974	2728	9663	3188	3655	0,02	0,31	0,75	**1,34**
Ет. 2		S355	321	340	619	960	2611	9663	3188	3655	0,03	0,30	0,71	**1,40**
Ет. 1	HE500A	S355	577	347	226	573	1140	6677	1459	1335	0,09	0,39	0,85	**1,17**
Ет. -1		S355	356	346	226	572	807	6677	1459	1335	0,05	0,39	0,60	**1,65**
Ет. -2		S355	134	342	226	568	726	6677	1459	1335	0,02	0,39	0,54	**1,84**

2.1.4. Buckling resistance check

The heaviest loaded beam is examined in between floor 9 and floor 18. It is designed with cross-section HE900A.

According to БДС EN1993-1-1, item 6.3.5, structures can be calculated in the plastic stage, in case that buckling is prevented by stiffeners in the places of plastic hinges or by proving the buckling length. It is necessary to prove that the length between the stiffeners is less than the "stable" length L_{stable}.

for beams with: $h/t_f \leq 40.\varepsilon$

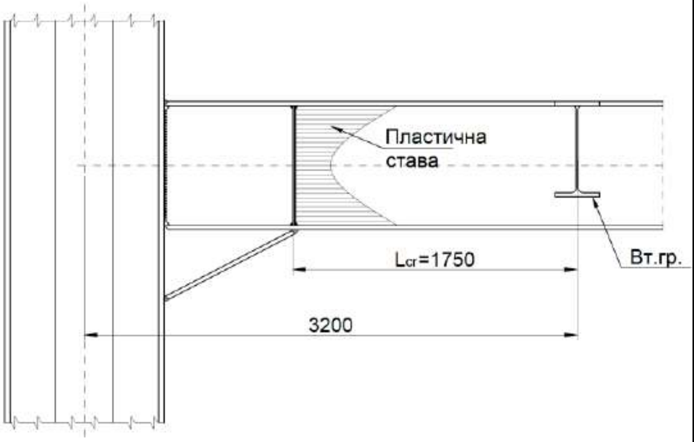
$\rightarrow 890/30 = 29,67 < 40.0,81 = 32,4$

$\psi = \frac{M_{Ed,min}}{M_{pl,Rd}} = \frac{-3463}{3655} = -0,95$

$L_{stable} = (60 - 40.\psi).\varepsilon.i_z$, при $-1 \leq \psi \leq 0,625$

$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0,81$

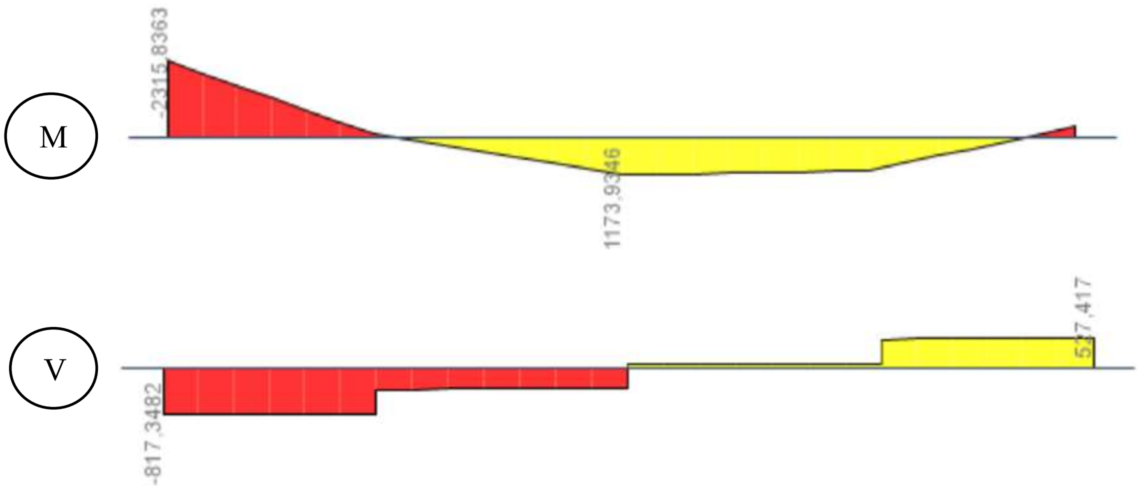
$L_{stable} = (60 - 40.(-0,95)).0,81.6,50 = 515\ cm > L_{cr} = 175\ cm$ → ***Buckling is not taken into consideration!***



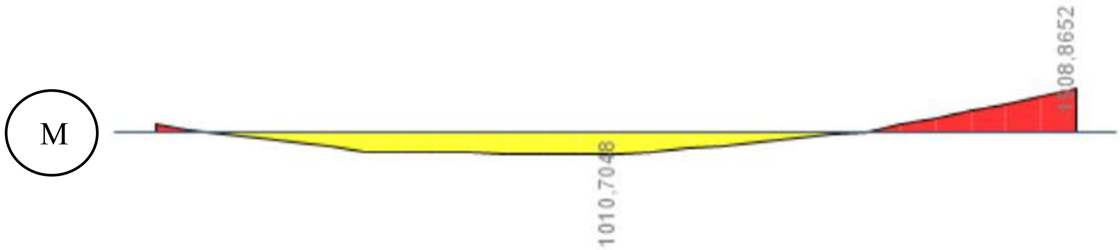
2.1.5. Design of the beam for positive bending moment

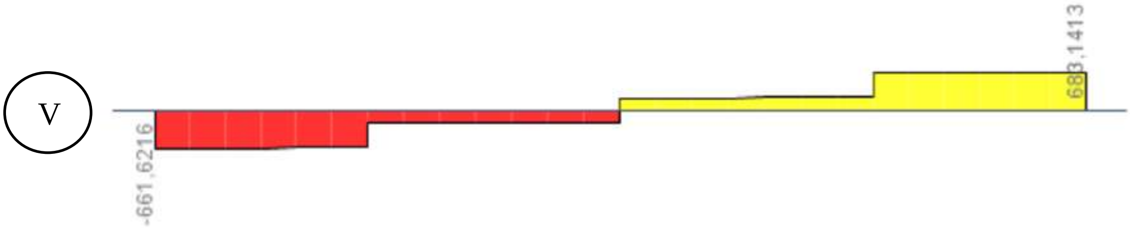
The values of the positive bending moments in the beam in combination *1.35.G + 1.5.Q* do not exceed those of the negative ones obtained in seismic combination. The steel section chosen to resist the negative moments would be able to resist the forces of the ULS combination on its own, without taking into account the assistance of the plate. The increase in field positive moments due to the formation of plastic hinges at the ends of the beam should be checked. The heaviest loaded beam between *floor 9* and *floor 18* with steel section *HE900A* is considered.

- *Inner forces in the considered beam as a result of different combinations*
  - Combination *ULS – 1,35.G + 1,5.Q*

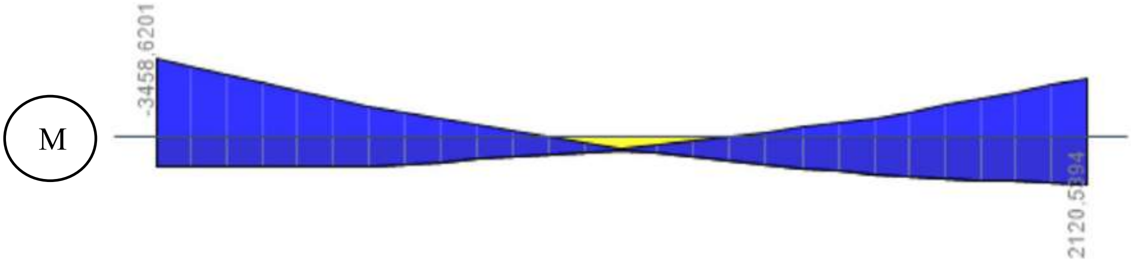


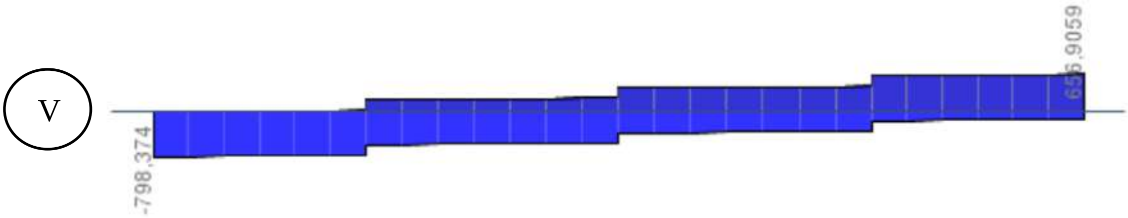
  - Combination *ULS – 1,35.G + 1,05.Q + 1,5.Wx*





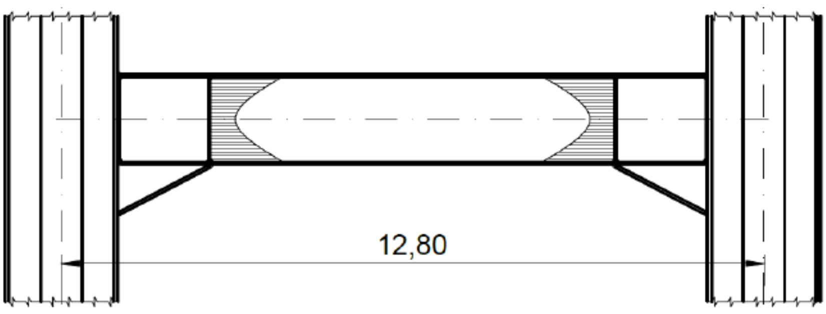
- Seismic combination– *Accidental*





➢ *Defining the value of space bending moment after the formation of plastic hinge in the ends of the beam*

Based on the presented graphical results (diagrams for the combinations), it can be said that the positive moments in the beam do not exceed in absolute value the moments at the ends of the beam from a calculated seismic situation. However, in the case of seismic impact, plastic hinges may form at the ends of the beams, as a result of which the value of the moment whithin the span will be higher than that of a rigid connection at the ends of the beam.

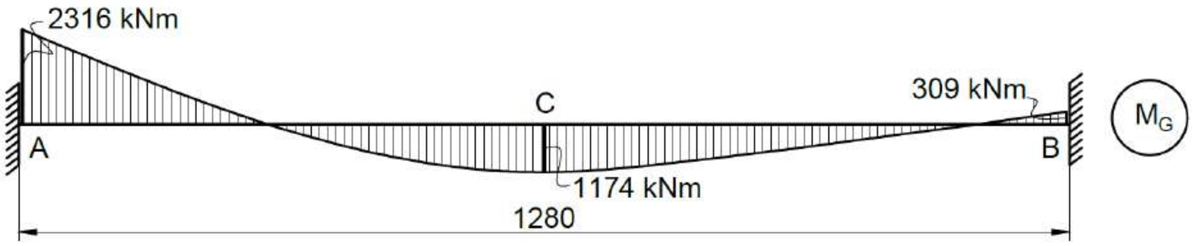


The moment within the span should be determined in case that plastic hinges have been formed at the ends of the beam.

- Formation of the first plastic hinge

The first plastic hinge is expected to form at the end *A*.

Diagram of the bending moments due to gravity loads:



Determining the required value of the bending moments of seismic impact for the formation of the first plastic hinge in item A:

$M_G + M_E^I = M_{pl,b,Rd} \rightarrow M_E^I = M_{pl,b,Rd} - M_G$,

Where:

$M_G = 2316\ kNm$ – bending moment as a result of gravity loads A;

$M_{pl,b,Rd} = \frac{W_{pl,y} \cdot f_y}{\gamma_{M0}} = \frac{10810.35,5}{1,05} = 3655\ kNm$ – plastic bending resistance of the beam;

$M_E^I = 3655 - 2316 = 1339\ kNm$ → value of the moment of seismic impact, leading to the formation of the first plastic hinge;

Diagram of the the seismic bending moment leading go plastic hinge formation:

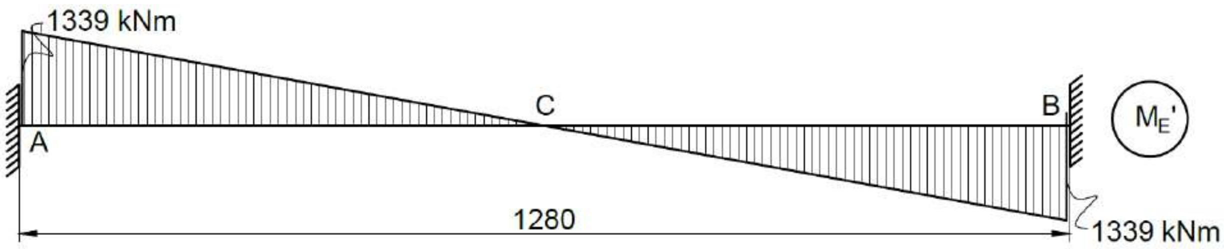
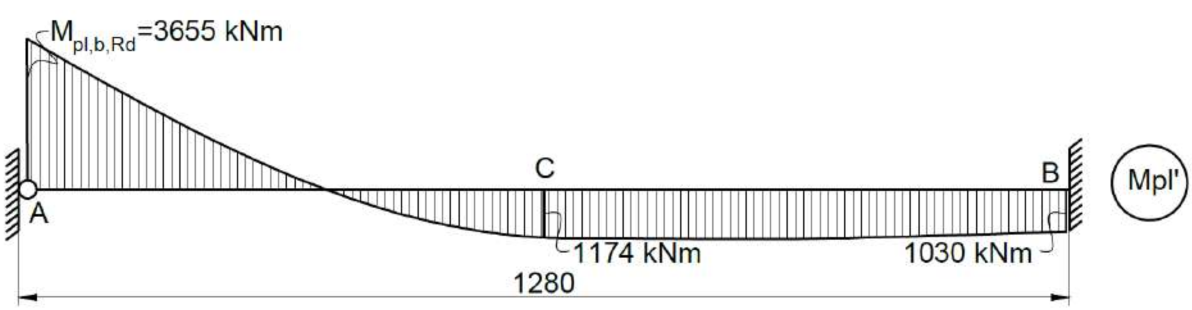


Diagram of the inner forces in the beam after the formation of the first plastic hinge:

M_{pl}^I "=" M_G "+" M_E'



- Second plastic hinge formation

The second and final plastic hinge is expected to form at end B. The formation of more plastic hinges is not allowed, as the system becomes a mechanism!

The required value of the bending moments of seismic impact for the formation of a second plastic joint in point B is:

$M_{pl}^{I} + M_{E}^{II} = M_{pl,b,Rd} \rightarrow M_{E}^{II} = M_{pl,b,Rd} - M_{pl}^{I}$,

$M_{E}^{II} = 3655 - 1030 = 2316\ kNm$ → value of the bending moment of seismic impact, leading to the formation of the first plastic hinge;

Diagram of the moments, result of seismic forces and leading to the formation of second plastic hinge:

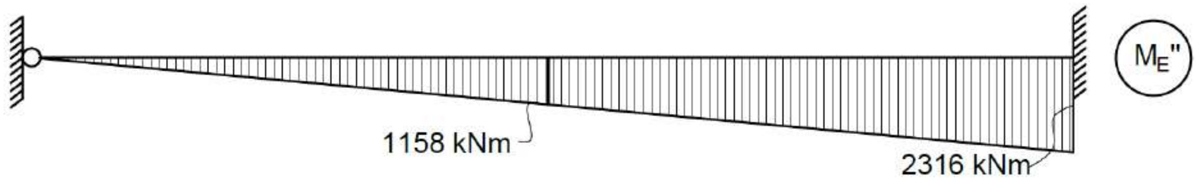
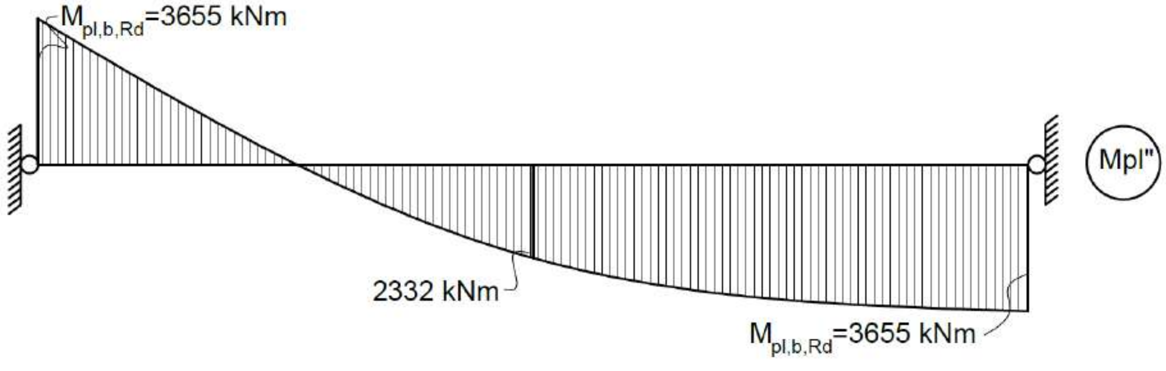
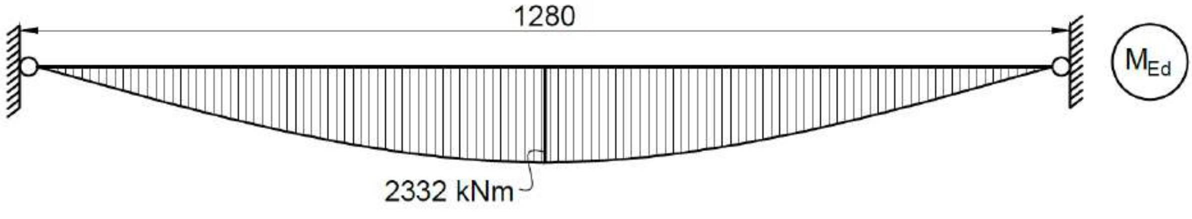


Diagram of the inner forces in the beam after the formation of second plastic hinge:

$\underline{\boldsymbol{M_{pl}^{II} \text{ “=” } M_{pl}^{I} \text{ “+” } M_{E}^{II}}}$

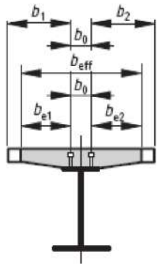


- Final moment diagram



The moment in the field after plasticizing the ends of the beam does not exceed the load-bearing capacity of the steel section! The resistence of the combined section should not be checked.

- *Determination of the effective width of the beam*



$b_{eff} = b_o + \sum b_{ei}$; $L_e = 8{,}00\ m \rightarrow$ *An ETABS result.*

$b_{ei} = L_e/8 \leq b_i \rightarrow b_{ei} = 8{,}00/8 \rightarrow b_{ei} = 1{,}00 \rightarrow b_{ei} = 100\ cm$

$b_1 = b_2 = b_{ei} = 100\ cm$; $b_0 = 0$; $\rightarrow b_{eff} = 0 + 2.100 = 200\ cm$

$\boldsymbol{b_{eff} = 200\ cm}$

- *Defining the compression resistances of both of the materials*

$N_{c,f} = 0{,}85.f_{cd}.b_{eff}.h_c = 0{,}85.2{,}0.200.8$

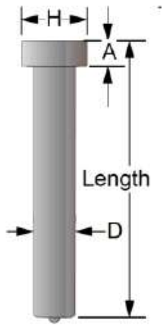
$\rightarrow \boldsymbol{N_{c,f} = 2829\ kN}$

$N_{pl,a} = A_a.f_{yd} = 285{,}8.35{,}5/1{,}05$

$\rightarrow \boldsymbol{N_{pl,a} = 9\ 663\ kN}$

- *Definition of the studs amount*

Studs KÖCO SD 19x100 are selected. They have the following characteristics:



$D = d = 19\ mm$ $\qquad H = 32\ mm$
$L = h_{sc} = 100\ mm$ $\qquad f_u = 450\ MPa$

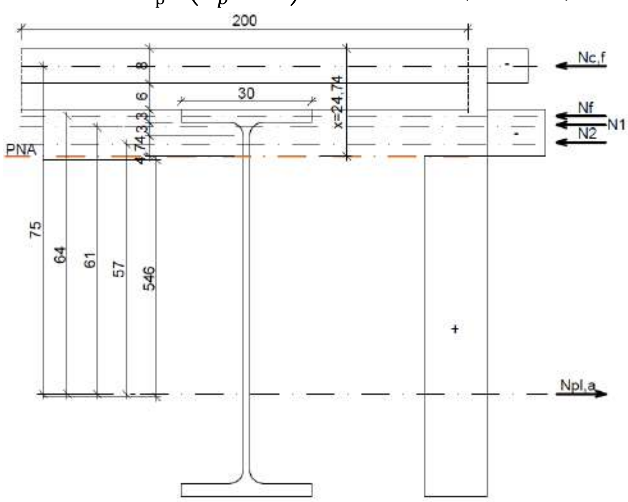
- Resistance of the stud connection:

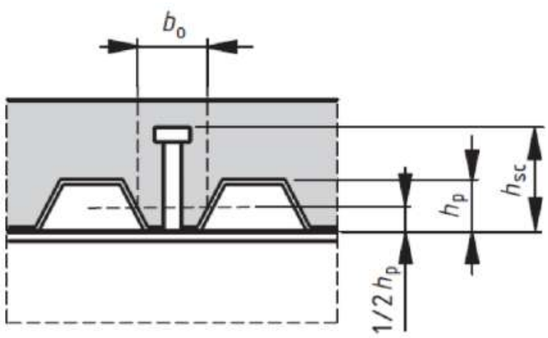
$$P_{Rd} = min \begin{cases} \frac{0{,}8.f_u.\pi.D^2/4}{\gamma_v} \\ \frac{0{,}29.\alpha.D^2.\sqrt{f_{ck}.E_{cm}}}{\gamma_v} \end{cases} = min \begin{cases} \frac{0{,}8.45.\pi.1{,}9^2/4}{1{,}25} \\ \frac{0{,}29.1{,}26.1{,}9^2.\sqrt{3.3280}}{1{,}25} \end{cases} = min \begin{cases} 81{,}7 \\ 104{,}7 \end{cases} \rightarrow \boldsymbol{P_{Rd} = 81{,}7\ kN}$$

$\alpha = 0{,}2.(h_{sc}/d + 1) = 0{,}2.(100/19 + 1) = 1{,}25$

In the case of slabs in which the ribs of the profiled steel decking are parallel to the axis of the beam, the bearing capacity of the stud is reduced by a factor of k_l.

$$k_l = 0{,}6.\frac{b_0}{h_p}.\left(\frac{h_{sc}}{h_p} - 1\right) = 0{,}6.\frac{140}{60}.\left(\frac{100}{60} - 1\right) = 0{,}93$$





$b_0 = 140\ m$

$k_l = 0{,}93 < k_{max} = 1{,}0$

$P_{Rd} = k_l.P_{Rd} = 0{,}93.81{,}7$

$\boldsymbol{P_{Rd} = 76\ kN}$

The beam is able to resist the forces of vertical loads without taking into account the composite action, but additional sliding forces due to seismic impact in the duts, which occur due to the diaphragmatic action of the floor slab, should be taken into account. A full interaction is accepted between steel studs and concrete slab.

For a full interaction to be provided, the numbure of the studs within the critical legth should be:

$n_f \geq min\{N_{c,f}\ ;N_{pl,a}\}/P_{Rd} \rightarrow n_f \geq min\{2829;\ 9663\}/76 = 2829/76 \approx 38$ *studs*

$\rightarrow$ ***n_f = 38 studs/l_{cr}***

The maximum number of studs that can be located along the length of the beam is proportional to the minimum allowable distances between the studs, as the ribs of the steel decking are parallel to the axis of the beam.

$L_{cr} \approx L_{pl}/2 = L/2 - b_{eff} = 6{,}40 - 2{,}00 \approx 4{,}40\ m \rightarrow$ Length for placing of studs. No studs are located in the area around the column with radius $2.b_{eff}$, according to БДС EN1998-1, as this area is considered dissipative.

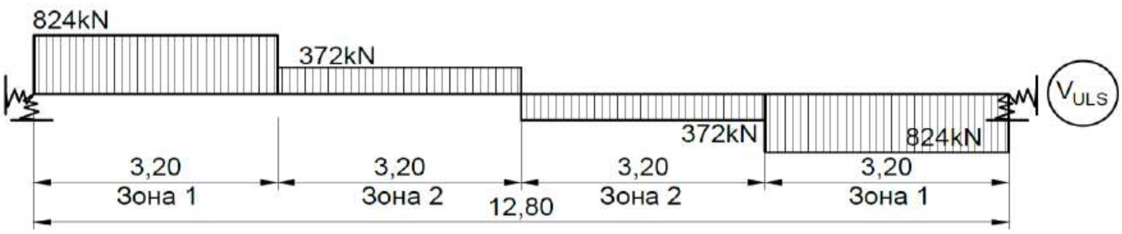
$n = l_{cr}/b_p = 4400/100 = 44$ *бр/*l_{cr} – maximum number of studs that can be located along the length of the beam.

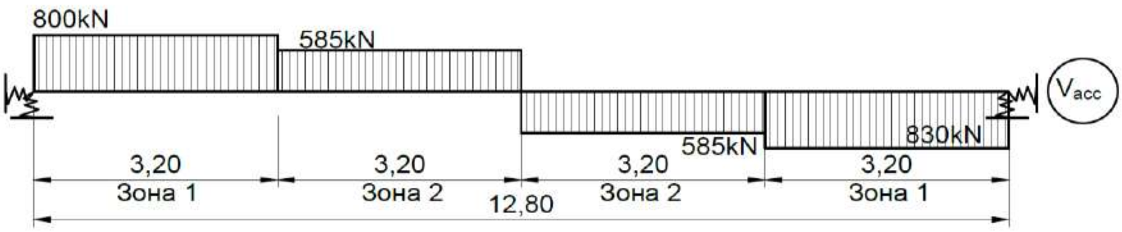
The maximum number of studs that can be placed in the critical length, so as to meet the necessary requirements for minimum distances, is 44 pcs. The required number of studs is n_f = 38 pcs. and can be located within the critical length. A full interaction is provided!

- *Definition of the required number of studs for every zone*

The studs should be positioned in proportion to the diagram of the shear or sliding forces in the beam. The beam is divided into 4 zones, for which the required number of studs is determined.

The following diagrams presend shear forces in the beam from ULS combination and from Accidental combination:





$n_1/n_2 = V_1/V_2$

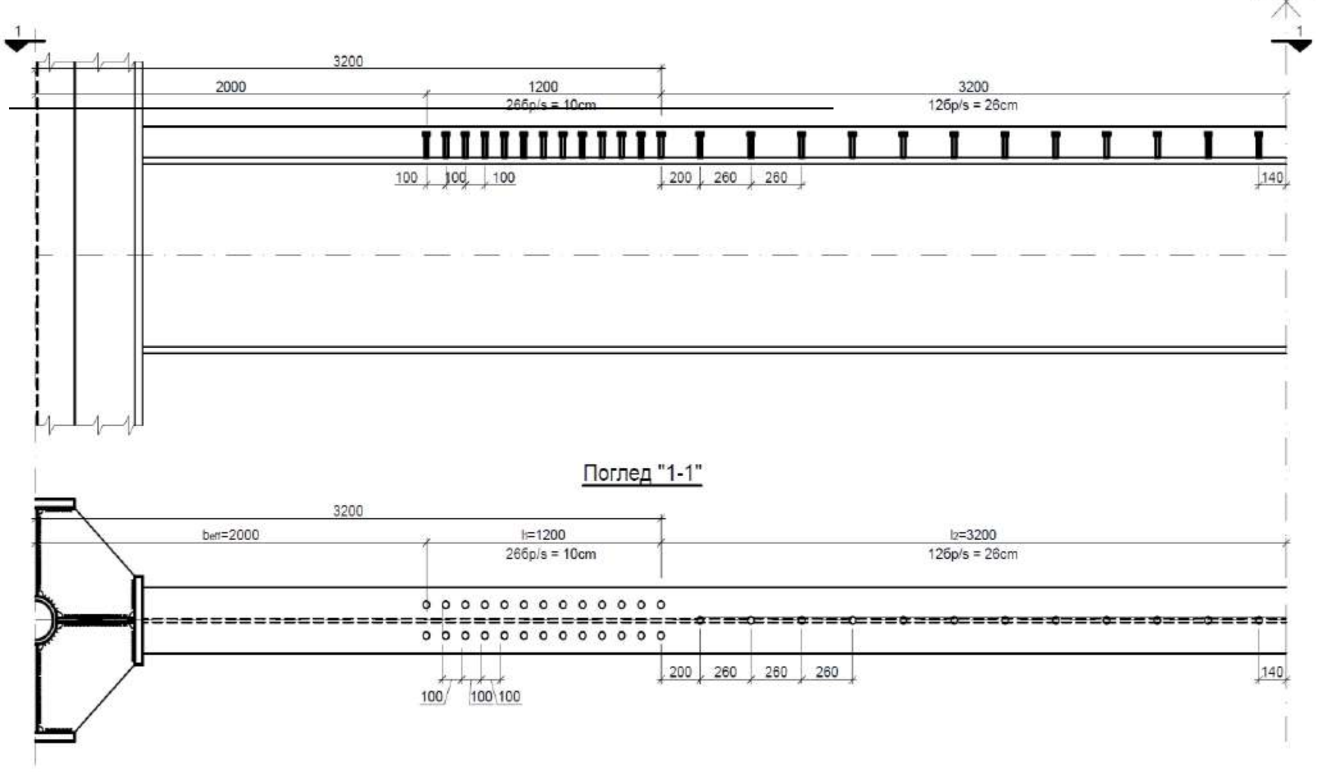
$n_1\ = x\ ;\ \ n_2 = n - x$

$\frac{x}{38-x} = \frac{824}{372} \rightarrow x = 26$ *бр.*

n_1 = 26 бр. $\rightarrow$ number of studs in zone 1;

n_2 = 12 бр. $\rightarrow$ number of studs in zone 2;

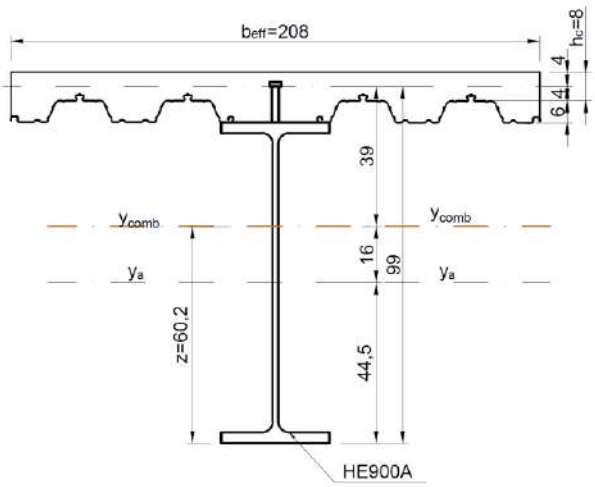
Placement of the studs within the length of the beam



- *Definition of the longitudinal shear:*

$$T_{Ed} = \frac{V_{z,Ed} \cdot S_{c,eff}}{I_{comb}} = \frac{V_{Ed} \cdot A_{c,eff} \cdot z_c}{I_{comb}}$$

Inertial characteristics of the section:



$n_0 = E_a/E_{cm} = 21000/3280 = 6{,}4$

$n_{eff} = 2.n_0 = 2.6{,}4 = 12{,}8$

$A_{c,eff} = A_c/n_{eff} = 1\,664/12{,}8 \rightarrow \boldsymbol{A_{c,eff} = 130\ cm^2}$

$A_c = b_{eff}.h_c = 208.8 = 1\,664\ cm^2$

$I_{c,eff} = b_{eff}.h_c^3/(12.n_{eff}) = 208.8^3/(12.12{,}8)$

$\rightarrow \boldsymbol{I_{c,eff} = 693{,}33\ cm^4}$

$A_{comb} = A_{c,eff} + A_a = 130 + 320{,}5 = 450{,}5\ cm^2$

$\boldsymbol{A_{comb} = 450{,}5\ cm^2}$

$I_{comb} = I_a + A_a.z_a^2 + I_{c,eff} + A_{c,eff}.z_c^2$

$z = 60{,}22\ cm$

$I_{comb} = 422\,075 + 320{,}5.16^2 + 693{,}33 + 130.39^2$

$\boldsymbol{I_{comb} = 702\,546\ cm^4}$

Zone 1: $T_{Ed} = 824.130.39/702\,546 = 5{,}91\ kN/cm$

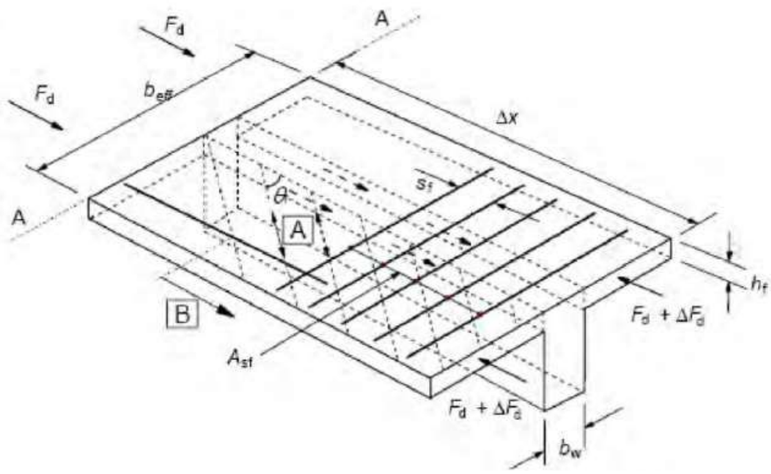
Zone 2: $T_{Ed} = 585.130.39/702\,546 = 4{,}22\ kN/cm$

- Design check for longitudinal shear resistance

The check is performed in accordance with БДС EN1992-1-1, item 6.2.4.

The contribution of profiled steel decking is taken into account, as it passes continuously over the secondary beam.

The section *"a-a"* is checked, where the reinforcement mesh *5N10* contributes to resisting of the longitudinal shear force and has cross section area: $A_t = A_{sf}/s_f = 3,93\ cm^2/m$.



v_{Ed} – *longitudinal shear stress in the contact between the shear stud and the reinforced concrete slab;*

$v_{Ed} = T_{Ed,1}/(2.h_f) = 591/(2.0,08) \rightarrow \boldsymbol{v_{Ed} = 3693\ kN/m^2}$

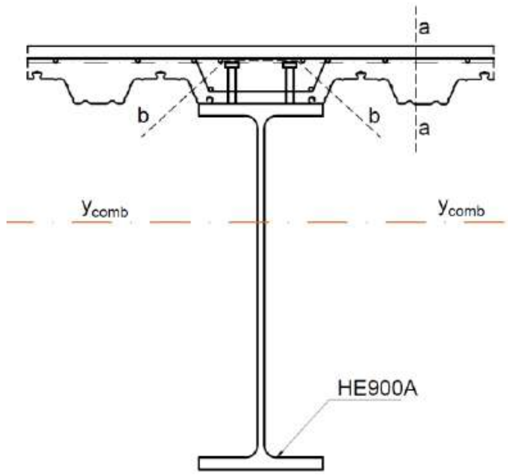
Check for the resistance of compression diagonal:

$v_{Ed} \le v.f_{cd}.\sin\theta.\cos\theta = 0,6.2.\sin 45^\circ.\cos 45^\circ = 0,6\ kN/cm^2$

Accepted: $\theta = 45^\circ$; $v = 0,6$

$\boldsymbol{v_{Ed} = 3693\ kN/m^2 < 6000\ kN/m^2}$ **→ *Requirement is satisfied***

Check for existing reinforcement in section „a-a":



$(A_{sf}.f_{yd}/s_f) \ge v_{Ed}.h_f/cotg\theta$

A_{sf}– area of the top reinforcement in section „*a-a";*

$(3,93.43,5) \ge 3693.0,08/cotg45$

→ *170 kN/m < 295,5kN/m* → The resistance of longitudinal shear is not ensured by the available transverse reinforcement! It is necessary to be added additional reinforcement to the slab!

$A_{sf,req}/s_f > 2,86\ cm^2/m$

Check for existing reinforcement in section „b-b":

$v_{Ed} = T_{Ed,1}/(2.l_c) = 591/(2.0,30) \rightarrow \boldsymbol{v_{Ed} = 985\ kN/m^2}$

$2.(A_{sf,req}.f_{yd}/s_f) \ge v_{Ed}.h_f/cotg\theta$

$2.(A_{sf,req}/s_f.43,5) \ge 985.0,30/cotg45 \rightarrow A_{sf,req}/s_f > 3,39\ cm^2/m$

Amplifiers are required: $A_{sf,req}/s_f > 3,39\ cm^2/m$

In the area of the main beams (with width b_{eff}) between the bars of the main mesh are placed amplifiers *5N10 / m*!

Check: $A_{sf}.f_{yd}/s_f + A_{s1}.f_{yd}/s_f \geq v_{Ed}.h_f/cotg\theta \rightarrow$

(3,93.43,5) + (3,93.43,5) ≥ 342 kN/m → ***342 kN/m > 295 kN/m***

→ Requirement is satisfied!

4.10. Column C-C5

The column is part of a moment resistant frame. In order to ensure the global ductile behavior of the structure, the frames resisting bending must be designed so as to form plastic hinges in the beams, and the columns to work entirely in the elastic stage. This requirement is presented in БДС EN 1998-1, item 6.6.

To ensure the formation of plastic hinges in the beams and to prevent plasticization in the columns, the columns of the frame are designed for increased inner forces.

2.1.6. Design checks

Columns are designed to resist bi-axial bending with compression. Influence of the shear force should not be taken into consideration if: $V_{Ed} \leq 0{,}5.V_{Rd}$

$N_{Ed} = N_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.N_{Ed,E}$

$V_{Ed} = V_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.V_{Ed,E}$

$M_{Ed} = M_{Ed,G} + 1{,}1.\gamma_{ov}.\Omega.M_{Ed,E}$

$\Omega = \Omega_{i,min} = M_{Rd,i}/M_{Ed,i} = 1{,}0$

Buckling resistance of the columns should be also checked. The following conditions should be satisfied:

$$\frac{N_{Ed}}{\frac{\chi_y.N_{Rk}}{\gamma_{M1}}} + \frac{k_{yy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{yz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$\frac{N_{Ed}}{\frac{\chi_z.N_{Rk}}{\gamma_{M1}}} + \frac{k_{zy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{zz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1,$$

Where:

$N_{Rk} = f_y.A;\ \ M_{y,Rk} = f_y.W_y\ ;\ \ M_{z,Rk} = f_y.W_z$ - bending and axial resistances of the cross section

k_{yy}, k_{zz}, k_{yz} и k_{zy} – interaction coefficients

Columns, which are part of the EBFs and MRFs are heavily loaded with compression force and bending moment due to seismic motion. This leads to the use of non-standard composite sections for heavily loaded columns. Such sections are not sensitive to buckling and for them the value of the displacement coefficient $\chi_{Lt} > 1{,}0$, which allows it to be assumed $\chi_{Lt} = 1{,}0$. For sections of this type, the values of the interaction coefficients are not defined by EC 1993-1-1. To increase safety and for more proper calculations in the present project, the values of these coefficients are accepted:

$k_{yy} = k_{zz} = k_{yz} = k_{zy} = 1{,}0$

All these assumptions allow the above formulas containing the coefficients of interaction to be reduced to a modified, simplified formula:

$$\sigma_{max} = \frac{N_{Ed}}{A.\chi} + \frac{M_{Ed,y}}{W_{pl,y}} + \frac{M_{Ed,z}}{W_{pl,z}} \leq f_y/\gamma_{M1}$$

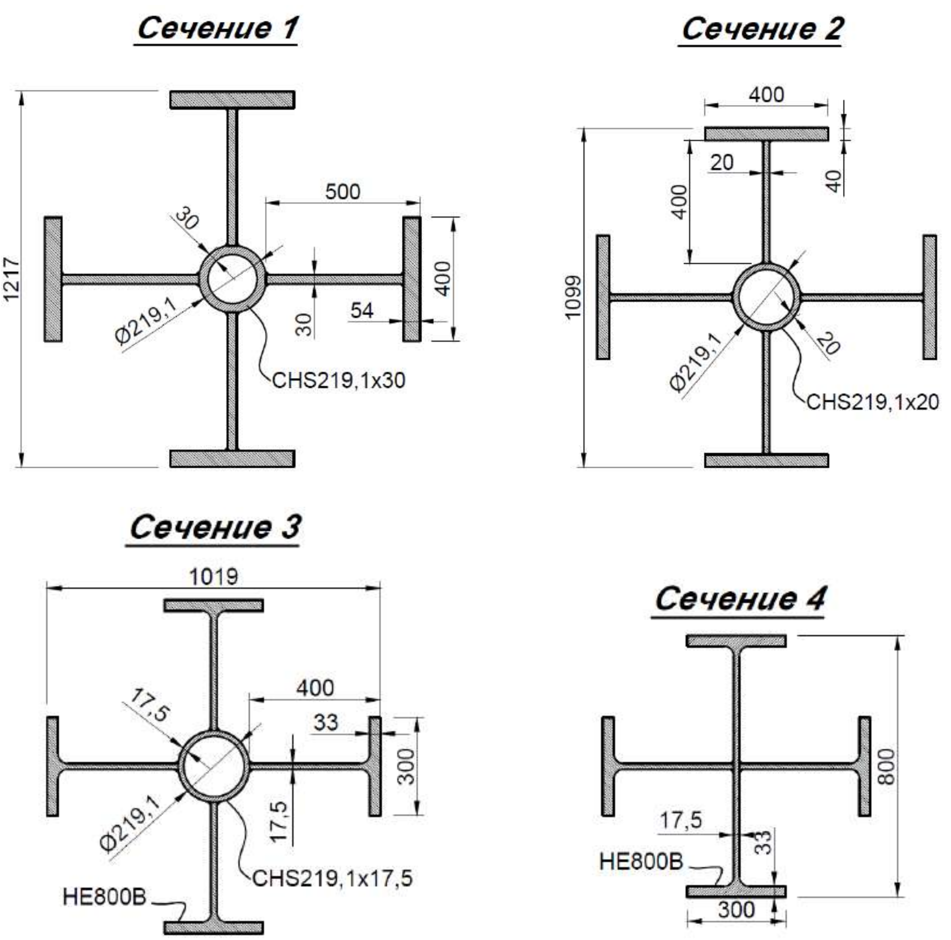
The values of buckling factor are equal for the main axes, because of the double symmetry and the equal inertial characteristics of the cross sections in both directions.

2.1.7. Characteristics of the cross sections

- *Geometrical and inertial characteristics of the cross sections*

Характеристики на напречното сечение										
Колони-2x900X400X54X30+CHS219,1X30 - 1										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A csh***	***W csh***	***f y,csh***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
121,7	1576,6	259,3	1886296	1886296	30999	30999	46	178	1080	35,5
Колони-2x800X400X40X20+CHS219,1X20 - 2										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A csh***	***W csh***	***f y,csh***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
109,9	960,2	219,8	1115156	1115156	20294	20294	46	125	795	35,5
Колони-2xHE800B+CHS219,1X17,5 - 3										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A csh***	***W csh***	***f y,csh***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
101,9	779,25	178,3	638211	638211	12525	12525	46	111	713	35,5
Колони-2xHE800B - 4										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***			
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]			
80	665	161,8	373987	373987	9350	9350	46			

- *Type of cross section*



2.1.8. Presentation of the results from the checks

Ω =1,0 → $E_{Ed} = N_{Ed,G} + 1{,}1.1{,}25.1{,}0.N_{Ed,E}$

Оразмеряване на колона C-C5															
Етаж	**Напр. сечение**	**Клас стомана**	***N_{Ed}*** *[kN]*	***V_{Ed}*** *[kN]*	***$M_{Ed,y}$*** *[kNm]*	***$M_{Ed,z}$*** *[kNm]*	**N_{Rd}** [kN]	**V_{Rd}** [kN]	**$M_{Rd,y}$** [kNm]	**$M_{Rd,z}$** [kNm]	$\frac{N_{Ed}}{N_{Rd}\cdot\chi}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{y,Ed}}{M_{y,Rd}}$	$\frac{M_{z,Ed}}{M_{z,Rd}}$	$\frac{\sigma_{Ed}}{f_{y,Rd}}$
Ет. П2	4	S460	*323*	*353*	*568*	*474*	27080	4092	3116	3116	0,01	0,09	0,18	0,15	***0,35***
Ет. П1		S460	*707*	*242*	*607*	*594*	27080	4092	3116	3116	0,03	0,06	0,19	0,19	***0,41***
Ет. К		S460	*747*	*248*	*924*	*625*	27080	4092	3116	3116	0,03	0,06	0,30	0,20	***0,52***
Ет. 31'		S460	*1105*	*791*	*1569*	*518*	27080	4092	3116	3116	0,04	0,19	0,50	0,17	***0,71***
Ет. 31		S460	*1614*	*481*	*1108*	*511*	27080	4092	3116	3116	0,06	0,12	0,36	0,16	***0,58***
Ет. 30'		S460	*2424*	*867*	*1700*	*598*	27080	4092	3116	3116	0,09	0,21	0,55	0,19	***0,83***
Ет. 30		S460	*3069*	*930*	*1790*	*630*	27080	4092	4096	4096	0,11	0,23	0,44	0,15	***0,70***
Ет. 29		S460	*3692*	*996*	*1966*	*660*	27080	4092	4096	4096	0,14	0,24	0,48	0,16	***0,78***
Ет. 28	3	S460+S355	*4300*	*1057*	*2112*	*718*	35220	4510	5728	5728	0,12	0,23	0,37	0,13	***0,62***
Ет. 27		S460+S355	*4895*	*1112*	*2240*	*785*	35220	4510	5728	5728	0,14	0,25	0,39	0,14	***0,67***
Ет. 26		S460+S355	*5478*	*1162*	*2355*	*849*	35220	4510	5728	5728	0,16	0,26	0,41	0,15	***0,71***
Ет. 25		S460+S355	*6062*	*1209*	*2460*	*911*	35220	4510	5728	5728	0,17	0,27	0,43	0,16	***0,76***
Ет. 24		S460+S355	*6648*	*1254*	*2568*	*970*	35220	4510	5728	5728	0,19	0,28	0,45	0,17	***0,81***
Ет. 23		S460+S355	*7234*	*1287*	*2659*	*1021*	35220	4510	5728	5728	0,21	0,29	0,46	0,18	***0,85***
Ет. 22		S460+S355	*7822*	*1413*	*3166*	*995*	35220	4510	5728	5728	0,22	0,31	0,55	0,17	***0,95***
Ет. 21		S460+S355	*8257*	*1056*	*2605*	*1350*	35220	4510	5728	5728	0,23	0,23	0,45	0,24	***0,92***
Ет. 20		S460+S355	*9161*	*1498*	*2821*	*1232*	43029	4510	9159	9159	0,21	0,33	0,31	0,13	***0,66***
Ет. 19		S460+S355	*9709*	*1423*	*2961*	*1266*	43029	4510	9159	9159	0,23	0,32	0,32	0,14	***0,69***
Ет. 18	2	S460+S355	*10269*	*1524*	*3445*	*1298*	43029	5559	9159	9159	0,24	0,27	0,38	0,14	***0,76***
Ет. 17		S460+S355	*10887*	*1610*	*3386*	*1325*	43029	5559	9159	9159	0,25	0,29	0,37	0,14	***0,77***
Ет. 16		S460+S355	*11501*	*1628*	*3400*	*1363*	43029	5559	9159	9159	0,27	0,29	0,37	0,15	***0,79***
Ет. 15		S460+S355	*12114*	*1649*	*3439*	*1402*	43029	5559	9159	9159	0,28	0,30	0,38	0,15	***0,81***
Ет. 14		S460+S355	*12725*	*1666*	*3477*	*1441*	43029	5559	9159	9159	0,30	0,30	0,38	0,16	***0,83***
Ет. 13		S460+S355	*13335*	*1679*	*3510*	*1481*	43029	5559	9159	9159	0,31	0,30	0,38	0,16	***0,85***
Ет. 12		S460+S355	*13945*	*1688*	*3537*	*1520*	43029	5559	9159	9159	0,32	0,30	0,39	0,17	***0,88***
Ет. 11		S460+S355	*14556*	*1694*	*3565*	*1558*	43029	5559	9159	9159	0,34	0,30	0,39	0,17	***0,90***
Ет. 10		S460+S355	*15170*	*1700*	*3647*	*1606*	43029	5559	9159	9159	0,35	0,31	0,40	0,18	***0,93***
Ет. 9		S460+S355	*15785*	*1703*	*3949*	*1594*	43029	5559	9159	9159	0,37	0,31	0,43	0,17	***0,97***
Ет. 8		S460+S355	*15874*	*1169*	*1540*	*963*	43029	5559	9159	9159	0,37	0,21	0,17	0,11	***0,64***
Ет. 7		S460+S355	*16977*	*1372*	*3191*	*1695*	43029	5559	9159	9159	0,39	0,25	0,35	0,19	***0,93***
Ет. 6		S460+S355	*17612*	*1062*	*2950*	*1814*	43029	5559	9159	9159	0,41	0,19	0,32	0,20	***0,93***
Ет. 5		S460+S355	*18253*	*1087*	*2984*	*1880*	43029	5559	9159	9159	0,42	0,20	0,33	0,21	***0,96***
Ет. 4		S460+S355	*18897*	*1076*	*2971*	*1926*	43029	5559	9159	9159	0,44	0,19	0,32	0,21	***0,97***
Ет. 3		S460+S355	*19546*	*1056*	*2933*	*1928*	43029	5559	9159	9159	0,45	0,19	0,32	0,21	***0,98***
Ет. 2	1	S460+S355	*20217*	*1147*	*4690*	*4233*	69795	6559	13946	13946	0,29	0,17	0,34	0,30	***0,93***
Ет. 1		S460+S355	*20807*	*1133*	*3001*	*2286*	69795	6559	13946	13946	0,30	0,17	0,22	0,16	***0,68***
Ет. -1		S460+S355	*21397*	*329*	*862*	*341*	69795	6559	13946	13946	0,31	0,05	0,06	0,02	***0,39***
Ет. -2		S460+S355	*21993*	*144*	*329*	*69*	69795	6559	13946	13946	0,32	0,02	0,02	0,00	***0,34***

3. Elements form the MRFs, located in the low storey part of the building

All design procedures are given in item 2. The design method won't be described in detail. Only the results of design for seismic combination will be presented.

4.11. Beam PB57

- Characteristics of the cross sections

Характеристики на напречното сечение											
Греди - HEA 900				**Греди - HEA 700**				**Греди - HEA 360**			
A [cm²]	*Avz [cm²]*	*Wy [cm³]*	Fy [kN/cm²]	*A [cm²]*	*Avz [cm²]*	*Wy [cm³]*	Fy [kN/cm²]	*A [cm²]*	*Avz [cm²]*	*Wy [cm³]*	Fy [kN/cm²]
320,5	161,8	10230	35,5	260,5	117	7032	35,5	142,8	48,96	2088	35,5

- *Presentation of the results*

Етаж	Напр. сечение	Клас стом.	N_{Ed} [kN]	$V_{Ed,G}$ [kN]	$V_{Ed,M}$ [kN]	V_{Ed} [kN]	M_{Ed} [kNm]	N_{Rd} [kN]	V_{Rd} [kN]	M_{Rd} [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{\chi_{LT}.M_{Rd}}$	Ω [-]
Ет. PW	**HE900A**	S355	1443	424	1274	1698	2977	10775	3158	3439	0,13	0,54	0,87	**1,16**
Ет. 6		S355	640	347	1280	1627	2641	10826	3158	3456	0,06	0,52	0,76	**1,31**
Ет. 5		S355	113	300	1281	1581	2429	10836	3158	3459	0,01	0,50	0,70	**1,42**
Ет. 4	**HE800A**	S355	41	252	881	1133	2142	8807	2284	2377	0,00	0,50	0,90	**1,11**
Ет. 3		S355	34	212	881	1093	1876	8807	2284	2377	0,00	0,48	0,79	**1,27**
Ет. 2		S355	69	184	881	1065	1641	8807	2284	2377	0,01	0,47	0,69	**1,45**
Ет. 1	**HE360A**	S355	775	165	261	426	697	4828	956	706	0,16	0,45	0,99	**1,01**
Ет. -1		S355	272	146	261	407	356	4828	956	706	0,06	0,43	0,50	**1,98**
Ет. -2		S355	95	149	261	410	336	4828	956	706	0,02	0,43	0,48	**2,10**

The table shows that the requirement: $V_{Ed} \leq 0,5.V_{Rd}$ is not satisfied for some of the floors.

When $V_{Ed} > 0,5.V_R$, the design axial force resistance and bending moment resistance of the section shall be calculated using a reduced yield strength for the shear area. The reduced yield strength is determined by the formula:

$(1 - \rho).f_y$,

Where:

$\rho = (2.V_{Ed}/V_{pl,Rd} - 1)^2$

Етаж	ρ	f_{yd}
Ет. PW	0,0056	35,30
Ет. 6	0,0009	35,47

The results with the reduced yield strength for the last two floors are presented. There the shear test is not satisfied.

4.12. <u>Beams PB58, PB59 и PB60</u>

3.1.1. Design for negative bending moment

- *Characteristics of the cross sections*

Характеристики на напречното сечение							
Греди - HEA 550				**Греди - HEA 450**			
A [cm²]	***Avz [cm²]***	***Wy [cm²]***	***Fy [kN/cm²]***	***A [cm²]***	***Avz [cm²]***	***Wy [cm²]***	***Fy [kN/cm²]***
211,8	83,72	4622	35,5	178	65,78	3216	35,5

- Presentation of the results

Етаж	Напр. сечение	Клас стом.	N_{Ed} [kN]	$V_{Ed,G}$ [kN]	$V_{Ed,M}$ [kN]	V_{Ed} [kN]	M_{Ed} [kNm]	N_{Rd} [kN]	V_{Rd} [kN]	M_{Rd} [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{\chi_{LT}.M_{Rd}}$	Ω -
Ет. PW	**HE450A**	S355	978	217	183	401	1067	6018	1284	1064	0,16	0,31	1,00	**1,00**
Ет. 6	**HE550A**	S355	522	214	269	484	1347	7161	1634	1563	0,07	0,30	0,86	**1,16**
Ет. 5		S355	184	218	269	488	1439	7161	1634	1563	0,03	0,30	0,92	**1,09**
Ет. 4		S355	91	216	269	486	1427	7161	1634	1563	0,01	0,30	0,91	**1,09**
Ет. 3		S355	92	215	269	484	1421	7161	1634	1563	0,01	0,30	0,91	**1,10**
Ет. 2		S355	245	217	269	487	1393	7161	1634	1563	0,03	0,30	0,89	**1,12**
Ет. 1	**HE450A**	S355	831	343	187	530	1001	6018	1284	1087	0,14	0,41	0,92	**1,09**
Ет. -1		S355	433	341	187	528	811	6018	1284	1087	0,07	0,41	0,75	**1,34**
Ет. -2		S355	114	340	187	528	776	6018	1284	1087	0,02	0,41	0,71	**1,40**

For roof beams, the axial force limit $N_{Ed} \leq 0,15.N$is not satisfied. It is necessary to determine the moment resistance in the plastic stage, reduced by the axial force.

$M_{N,y,Rd} = \frac{M_{pl,y,Rd}.(1-n)}{1-0,5.a} \leq M_{pl,y,Rd}$,

Where:

$n = \frac{N_{Ed}}{N_{pl,Rd}}$

$a = \frac{A-2.b.t_f}{A} \leq 0,5$

For the designed cross sections:

$A = 178\ cm^2; \quad b = 30\ cm; \quad t_f = 2,1\ cm$

$n = \frac{978}{6018} = 0,163$

$a = \frac{178-2.30.2,1}{178} = 0,29 < 0,5$

$M_{N,y,Rd} = \frac{1087.(1-0,163)}{1-0,5.0,29} = 1064\ kNm \leq M_{pl,y,Rd} = 1087kNm$

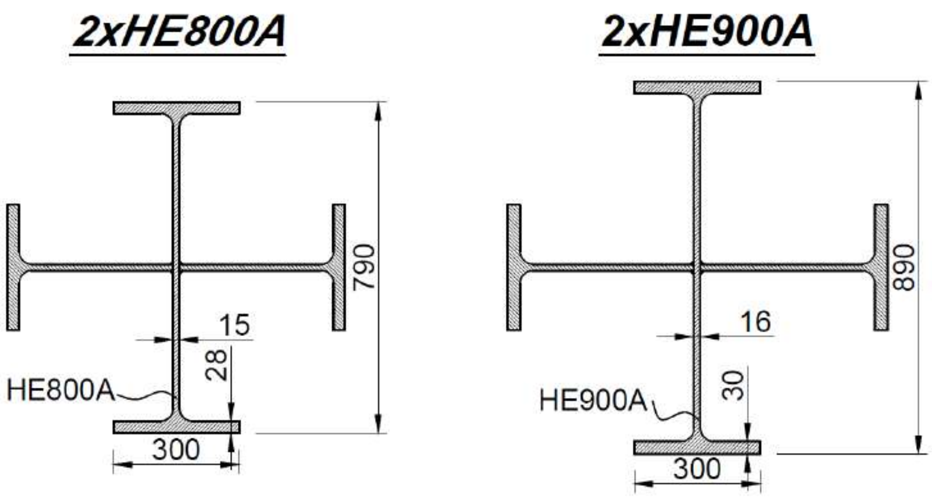
$\mathbf{M_{N,y,Rd} = 1064\ kNm}$

4.13. Column C-C15

All design procedures applied to the other columns are valid. The coefficient for capacitive increase of the forces in the column is $\Omega_{min} = 1,0$, determined by the calculations for beams *PB58, PB59 and PB60.*

- Characteistics of the cross sections

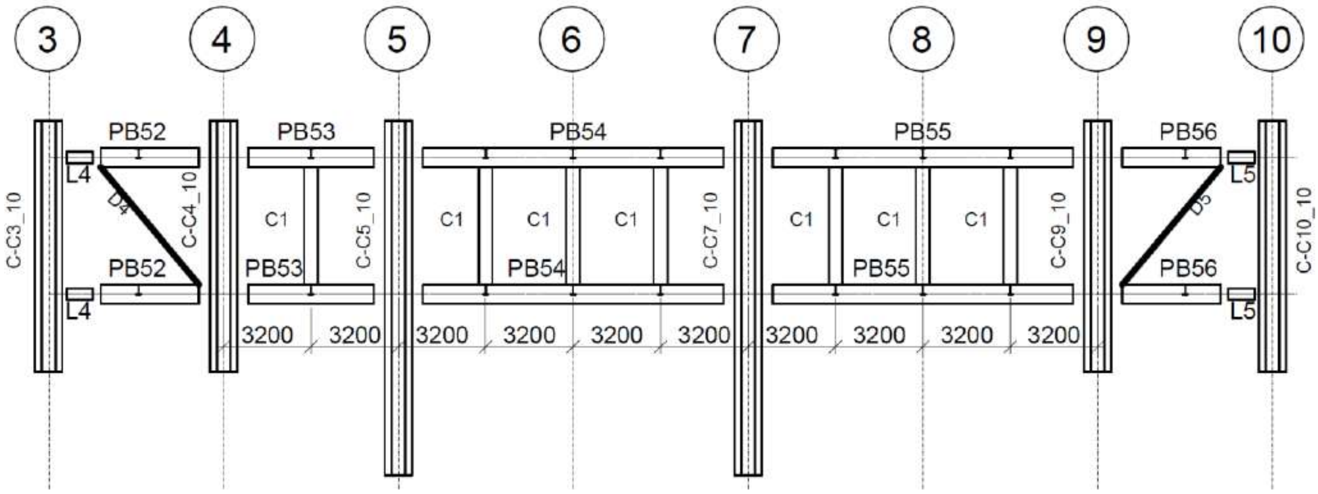
Колони-2xHE900A							
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]
89	638,5	163,3	435622	435622	9789	9789	46
Колони-2xHE800A							
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]
79	569,4	138,8	316081	316081	8002	8002	46



➢ Presentation of the results

Етаж	Напр. сечение	Клас стом.	N_{Ed} [kN]	V_{Ed} [kN]	$M_{Ed,y}$ [kNm]	$M_{Ed,z}$ [kNm]	N_{Rd} [kN]	V_{Rd} [kN]	$M_{Rd,y}$ [kNm]	$M_{Rd,z}$ [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed,y}}{M_{Rd,y}}$	$\frac{M_{Ed,z}}{M_{Rd,z}}$	$\frac{\sigma_{max}}{f_{y,Rd}}$
Ет. 7	**2xHE800A**	S460	*769*	*1659*	*1329*	*919*	23187	3511	3506	3506	0,03	0,47	0,38	0,26	***0,67***
Ет. 6		S460	*1512*	*1255*	*1697*	*1035*	23187	3511	3506	3506	0,07	0,36	0,48	0,30	***0,84***
Ет. 5		S460	*2250*	*1291*	*1646*	*1049*	23187	3511	3506	3506	0,10	0,37	0,47	0,30	***0,87***
Ет. 4		S460	*2985*	*1299*	*1619*	*1073*	23187	3511	3506	3506	0,13	0,37	0,46	0,31	***0,90***
Ет. 3		S460	*3721*	*1291*	*1630*	*1093*	23187	3511	3506	3506	0,16	0,37	0,46	0,31	***0,94***
Ет. 2	**2xHE900A**	S460	*4473*	*1409*	*1652*	*1184*	26000	4130	4289	4289	0,17	0,34	0,39	0,28	***0,83***
Ет. 1		S460	*5425*	*1118*	*604*	*364*	26000	4130	4289	4289	0,21	0,27	0,14	0,08	***0,43***
Ет. -1		S460	*6360*	*185*	*128*	*123*	26000	4130	4289	4289	0,24	0,04	0,03	0,03	***0,30***
Ет. -2		S460	*7303*	*39*	*24*	*56*	26000	4130	4289	4289	0,28	0,01	0,01	0,01	***0,30***

4. Short columns from „Vierendeel belt trusses”



The shear forces in the considered vertical elements are maximal when applying a design seismic situation. The most heavily loaded column should be considered.

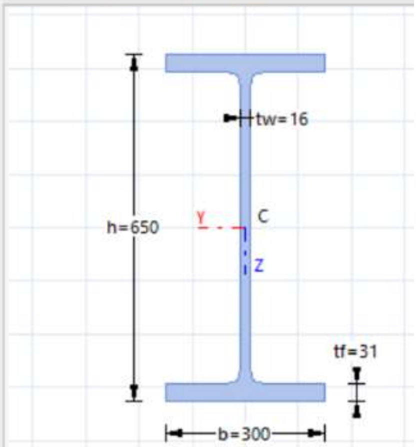
➢ *Design inner forces*

$M_{y,Ed} = 1\ 750$ kNm; $M_{z,Ed} = 83$ kNm; $V_{Ed} = 820$ kN; $N_{Ed} = 480$ kN

➢ *Selected cross section*

The selected cross section is ***HE650B***. Characteristics of the section are as it follows:

Cross Section - HE 650 B



h =	650,0	b2=	300,0	b =	300,0	ri=	27
tw=	16,0	tf2=	31,0	tf=	31,0	ro=	0

Section Properties

A = 286.338 cm	
Cz = 32.500 cm	Cy = 15.000 cm
Iy =210616.100 cm4	Iz = 13983.994 cm4
Wy = 6480.495 cm3	Wz = 932.266 cm3
Wply= 7319.881 cm3	Wplz= 1441.412 cm3
ry = 27.121 cm	rz = 6.988 cm
Avy = 186.000 cm2	Avz = 122.038 cm2
It = 761.606 cm4	Wt = 204.511 cm3

$N_{Rk} = f_{yk}.A = 35{,}5.286{,}3 = 10\ 163$ kN → $\boldsymbol{N_{Rk} = 10\ 163}$ **kN**

$M_{y,Rk} = f_{yk}.W_{pl,y} = 35{,}5.\ 6480{,}5 = 2\ 596\ kNm \rightarrow \boldsymbol{M_{y,Rk} = 2\ 596\ kNm}$

$M_{z,Rk} = f_{yk}.W_{pl,z} = 35{,}5.932.3 = 331\ kNm \rightarrow \boldsymbol{M_{z,Rk} = 331\ kNm}$

$V_{Rd} = f_{yd}.A_{vz}/\sqrt{3} = 35{,}5.122/(1{,}05.\sqrt{3}) = 2\ 381\ kN \rightarrow \boldsymbol{V_{Rd} = 2\ 381\ kN}$

- *Design checks*

The element is loaded with double-axial bending and compression. The possibility of buckling is taken into account. It is also necessary to check the influence of shear forces on the load bearing capacity of the element.

$V_{Ed} \leq 0{,}5.V_{Rd} \rightarrow 480 \leq 0{,}5.2\ 381 \rightarrow \boldsymbol{V_{Ed} = 480\ kN < 0{,}5.V_{Rd} = 1\ 190\ kN} \rightarrow$ ***Influence of shear force is not taken into account!***

$$\frac{N_{Ed}}{\frac{\chi_y.N_{Rk}}{\gamma_{M1}}} + \frac{k_{yy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{yz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$\frac{N_{Ed}}{\frac{\chi_z.N_{Rk}}{\gamma_{M1}}} + \frac{k_{zy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{zz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1,$$

Where:

$N_{Rk} = f_y.A;\ \ M_{y,Rk} = f_y.W_y;\ \ M_{z,Rk} = f_y.W_z$ - compression and bending resistances of cross sections, defined according to the geometrical characteristics of the selected sections.

k_{yy}, k_{zz}, k_{yz} и k_{zy} – interaction coefficients;

Section is class 1! → Section works in plastic stage. The following formulas are used to define the interaction coefficients.

$$k_{yy} = c_{my}.\left(1 + \left(\overline{\lambda_y} - 0{,}2\right).\frac{N_{Ed}}{\chi_y.N_{Rk}/\gamma_{M1}}\right) \leq c_{my}.\left(1 + 0{,}8.\frac{N_{Ed}}{\chi_y.N_{Rk}/\gamma_{M1}}\right)$$

$$k_{zz} = c_{mz}.\left(1 + \left(\overline{2.\lambda_z} - 0{,}6\right).\frac{N_{Ed}}{\chi_z.N_{Rk}/\gamma_{M1}}\right) \leq c_{mz}.\left(1 + 1{,}4.\frac{N_{Ed}}{\chi_z.N_{Rk}/\gamma_{M1}}\right)$$

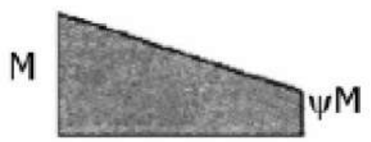
$k_{yz} = 0{,}6.k_{zz}$

$k_{zy} = 0{,}6.k_{yy}$

$C_{my} = C_{mz} = C_{mLT} = 0{,}91$

$c_{mLT} = 0{,}6 + 0{,}4.\psi = 0{,}6 + 0{,}4.0{,}77 = 0{,}908$

$\psi = 0{,}77$



The parameters, taking into account the possibility of buckling of the selected cross section at the critical length $Lcr = 5.0\ m$, have the following values:

$\boldsymbol{\lambda_y = 0{,}22}$	$\boldsymbol{\chi_y = 0{,}99}$
$\boldsymbol{\lambda_z = 0{,}95}$	$\boldsymbol{\chi_z = 0{,}63}$
	$\boldsymbol{\chi_{LT} = 0{,}85}$

$$k_{yy} = 0{,}91\left(1 + (0{,}22 - 0{,}2).\frac{480}{0{,}99.10\ 163/1{,}05}\right) \leq 0{,}91.\left(1 + 0{,}8.\frac{480}{0{,}99.10\ 163/1{,}05}\right)$$

$k_{yy} = 0,9 < 0,94$

$$k_{zz} = c_{mz}.\left(1 + (2.0,95 - 0,6).\frac{480}{0,63.10\,163/1,05}\right) \leq c_{mz}.\left(1 + 1,4.\frac{480}{0,63.10\,163/1,05}\right)$$

$k_{zz} = 0,99 < 1,016$

$k_{yz} = 0,6.0,99 = 0,59$

$k_{zy} = 0,6.0,9 = 0,54$

Interaction coefficients:

k_{yy} = 0,90 **k_{zz} = 0,99**
k_{yz} = 0,59 **k_{zy} = 0,54**

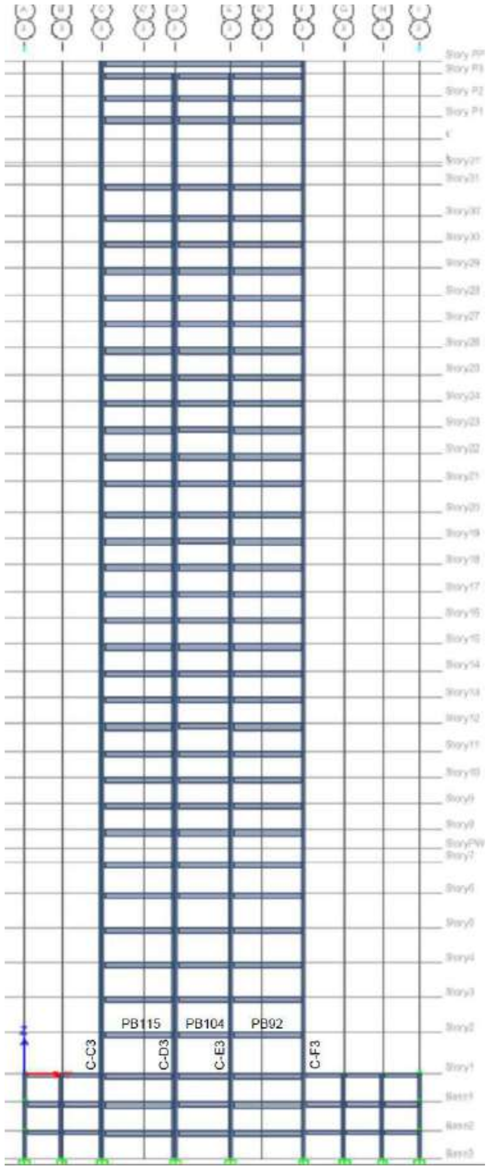
$$\frac{N_{Ed}}{\frac{\chi_y.N_{Rk}}{\gamma_{M1}}} + \frac{k_{yy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{yz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \rightarrow \frac{480}{\frac{0,99.10\,163}{1,05}} + \frac{0,9.1750}{\frac{0,85.2589}{1,05}} + \frac{0,59.83}{\frac{331}{1,05}} = \mathbf{0,95 < 1,0}$$

$$\frac{N_{Ed}}{\frac{\chi_z.N_{Rk}}{\gamma_{M1}}} + \frac{k_{zy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{zz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \rightarrow \frac{480}{\frac{0,63.10\,163}{1,05}} + \frac{0,54.1750}{\frac{0,85.2589}{1,05}} + \frac{0,99.83}{\frac{331}{1,05}} = \mathbf{0,79 < 1,0}$$

→ ***Requirements are satisfied!***

→ *Confirmed cross section for column C1 is* ***HE650B!***

VI. Design of the elements from frame parallel to axes „3”



1. Elements from EBFs

Columns *C-C3* и *C-F3* are the considered elements, part of EBF, from frame parallel to axes „3“. Their designing is presented in point V.1.4. and won’t be given in this point.

2. Elements from MRF

2.1. <u>Beam PB104</u>

The presented results and selected cross sections are designed with the inner forces, which are result of seismic combination (for maximum negative bending moment).

➢ *Design procedure*

Check for the bearing resistance of the beam:

$M_{y,Ed} \leq M_{y,Rd} = W_y.f_y/\gamma_{M0}$

Shear force and axial force may not be taken into consideration if the following requirements are satisfied:

$N_{Ed} \leq 0{,}15.N_{Rd}$

$V_{Ed} \leq 0{,}5.V_{Rd}$,

Where

$V_{Ed} = V_{Ed,G} + V_{Ed,M}$

$V_{Ed,G}$ – shear force, due to gravity loads in seismic combination;

$V_{Ed,M}$ – design value of the shear force, due to the plastic moments $M_{pl,Rd,A}$ и $M_{pl,Rd,B}$ applied in the end sections of the beam *A* и *B*.

$V_{Ed,M} = (M_{pl,Rd,A} + M_{pl,Rd,B})/L$ – most unpleasant condition for defining shear force, reffering to a beam with a span *L* and dissipative zones in the ends.

To increase safety, for defining $V_{Ed,M}$ is used the span between plastic hinges of the beam.

➢ *Characteristics of the cross sections*

Характеристики на напречното сечение											
Греди - HEA 700				**Греди - HEA 550**				**Греди - HEA400**			
A [cm²]	*Avz [cm2]*	*Wy [cm3]*	*Fy [kN/cm2]*	*A [cm²]*	*Avz [cm²]*	*Wy [cm³]*	*Fy [kN/cm²]*	*A [cm²]*	*Avz [cm²]*	*Wy [cm³]*	*Fy [kN/cm²]*
260,5	117	7032	35,5	211,8	83,72	4622	35,5	159	57,33	2562	35,5
Греди - HEA 600				**Греди - HEA 450**				**Греди - HEA 500**			
A [cm²]	*Avz [cm²]*	*Wy [cm³]*	*Fy [kN/cm²]*	*A [cm²]*	*Avz [cm²]*	*Wy [cm³]*	*Fy [kN/cm²]*	*A [cm²]*	*Avz [cm²]*	*Wy [cm³]*	*Fy [kN/cm²]*
226,5	93,21	5350	35,5	178	65,78	3216	35,5	197,5	74,72	3949	35,5

➢ *Presentation of the results*

Оразмеряване на греда PB104														
Етаж	**Напр. сечение**	**Клас стом.**	**N_{Ed}** [kN]	**$V_{Ed,G}$** [kN]	**$V_{Ed,M}$** [kN]	**V_{Ed}** [kN]	**M_{Ed}** [kNm]	**N_{Rd}** [kN]	**V_{Rd}** [kN]	**M_{Rd}** [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$	**Ω** -
Ет. П3	**H400A**	S355	394	54	222	276	239	5376	1119	866	0,07	0,25	0,28	**3,63**
Ет. П2		S355	78	55	222	277	358	5376	1119	866	0,01	0,25	0,41	**2,42**
Ет. П1		S355	392	55	222	277	589	5376	1119	866	0,07	0,25	0,68	**1,47**
Ет. 31'	-	-		-	-	-		-	-	-	-	-	-	-
Ет. 31	**H400A**	S355	296	46	222	268	800	5376	1119	866	0,06	0,24	0,92	**1,08**
Ет. 30'		S355	113	55	222	277	750	5376	1119	866	0,02	0,25	0,87	**1,15**
Ет. 30		S355	42	55	222	277	779	5376	1119	866	0,01	0,25	0,90	**1,11**
Ет. 29	**HE450A**	S355	41	55	279	334	853	6018	1284	1087	0,01	0,26	0,78	**1,27**
Ет. 28		S355	48	55	279	334	933	6018	1284	1087	0,01	0,26	0,86	**1,17**
Ет. 27		S355	53	55	279	334	1010	6018	1284	1087	0,01	0,26	0,93	**1,08**
Ет. 26		S355	57	56	279	334	1082	6018	1284	1087	0,01	0,26	1,00	**1,00**
Ет. 25	**HE550A**	S355	60	56	401	456	1151	7161	1634	1563	0,01	0,28	0,74	**1,36**
Ет. 24		S355	63	56	401	456	1216	7161	1634	1563	0,01	0,28	0,78	**1,28**
Ет. 23		S355	65	56	401	457	1281	7161	1634	1563	0,01	0,28	0,82	**1,22**
Ет. 22		S355	72	56	401	457	1354	7161	1634	1563	0,01	0,28	0,87	**1,15**
Ет. 21		S355	104	56	401	457	1474	7161	1634	1563	0,01	0,28	0,94	**1,06**
Ет. 20		S355	124	56	401	457	1530	7161	1634	1563	0,02	0,28	0,98	**1,02**
Ет. 19		S355	66	56	401	457	1512	7161	1634	1563	0,01	0,28	0,97	**1,03**
Ет. 18		S355	70	56	401	457	1543	7161	1634	1563	0,01	0,28	0,99	**1,01**

Оразмеряване на греда PB104														
Етаж	**Напр. сечение**	**Клас стом.**	N_{Ed} [kN]	$V_{Ed,G}$ [kN]	$V_{Ed,M}$ [kN]	V_{Ed} [kN]	M_{Ed} [kNm]	N_{Rd} [kN]	V_{Rd} [kN]	M_{Rd} [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$	Ω -
Ет. 17	**HE600A**	S355	71	56	464	520	1582	7658	1819	1809	0,01	0,29	0,87	**1,14**
Ет. 16		S355	71	56	464	520	1620	7658	1819	1809	0,01	0,29	0,90	**1,12**
Ет. 15		S355	72	56	464	520	1654	7658	1819	1809	0,01	0,29	0,91	**1,09**
Ет. 14		S355	71	56	464	520	1683	7658	1819	1809	0,01	0,29	0,93	**1,07**
Ет. 13		S355	71	56	464	520	1707	7658	1819	1809	0,01	0,29	0,94	**1,06**
Ет. 12		S355	70	56	464	520	1726	7658	1819	1809	0,01	0,29	0,95	**1,05**
Ет. 11		S355	68	56	464	520	1739	7658	1819	1809	0,01	0,29	0,96	**1,04**
Ет. 10		S355	66	56	464	520	1751	7658	1819	1809	0,01	0,29	0,97	**1,03**
Ет. 9		S355	67	56	464	519	1754	7658	1819	1809	0,01	0,29	0,97	**1,03**
Ет. 8	**HE700B**	S355	92	56	610	665	1881	8807	2284	2377	0,01	0,29	0,79	**1,26**
Ет. 7		S355	58	56	610	665	2001	8807	2284	2377	0,01	0,29	0,84	**1,19**
Ет. 6		S355	63	56	610	665	2077	8807	2284	2377	0,01	0,29	0,87	**1,14**
Ет. 5		S355	46	55	610	665	2131	8807	2284	2377	0,01	0,29	0,90	**1,12**
Ет. 4		S355	40	55	610	665	2122	8807	2284	2377	0,00	0,29	0,89	**1,12**
Ет. 3		S355	36	55	610	665	2101	8807	2284	2377	0,00	0,29	0,88	**1,13**
Ет. 2		S355	58	55	610	664	1979	8807	2284	2377	0,01	0,29	0,83	**1,20**
Ет. 1	**HE500A**	S355	140	175	342	518	1217	6677	1459	1335	0,02	0,36	0,91	**1,10**
Ет. -1		S355	39	174	342	517	345	6677	1459	1335	0,01	0,35	0,26	**2,82**
Ет. -2		S355	14	174	342	517	231	6677	1459	1335	0,00	0,35	0,17	**2,82**

The values of the positive bending moments in the beam in combination *1.35.G + 1.5.Q* do not exceed those of the negative ones, obtained in seismic combination. This is why they won't be considered. In the current point composite behavior of the beam won't be taken into account.

2.2. Beams PB92 and PB115

The presented calculations and the selected cross sections are obtained due to the forces, result of seismic situation (maximal negative bending moment).

- *Design procedure*

Design procedure is analogical to the one for *PB104*.

- *Characteristics of the cross sections*

Характеристики на напречното сечение			
Греди - HEA 900			
A [cm²]	*Avz [cm²]*	*Wy [cm³]*	*fy [kN/cm²]*
320,5	163,3	10810	35,5
Греди - HEA 800			
A [cm²]	*Avz [cm²]*	*Wy [cm³]*	*fy [kN/cm²]*
285,8	138,8	8699	35,5
Греди - HEA 650			
A [cm²]	*Avz [cm²]*	*Wy [cm³]*	*fy [kN/cm²]*
241,6	103,2	6136	35,5
Греди - HEA 550			
A [cm²]	*Avz [cm²]*	*Wy [cm³]*	*fy [kN/cm²]*
211,8	83,72	4622	35,5

- *Presentation of the results*

Оразмеряване на греди PB92 и PB115

Етаж	Напр. сечение	Клас стомана	N_{Ed} [kN]	$V_{Ed,G}$ [kN]	$V_{Ed,M}$ [kN]	V_{Ed} [kN]	M_{Ed} [kNm]	N_{Rd} [kN]	V_{Rd} [kN]	M_{Rd} [kNm]	$\frac{N_{Ed}}{N_{Rd}}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{Ed}}{M_{Rd}}$	Ω -
Ет. П3	H650A	S355	79	218	391	609	1510	8168	2014	2075	0,01	0,30	0,73	**1,37**
Ет. П2		S356	27	216	391	607	1589	8168	2014	2075	0,00	0,30	0,77	**1,31**
Ет. П1		S357	255	210	391	601	1658	8168	2014	2075	0,03	0,30	0,80	**1,25**
Ет. 31'	-	-		-	-	-	-	-	-	-	-	-	-	-
Ет. 31	H650A	S355	183	194	391	586	1678	8168	2014	2075	0,02	0,29	0,81	**1,24**
Ет. 30'		S355	79	211	391	603	1872	8168	2014	2075	0,01	0,30	0,90	**1,11**
Ет. 30		S355	28	212	391	604	2030	8168	2014	2075	0,00	0,30	0,98	**1,02**
Ет. 29	HE700A	S355	23	212	555	767	2166	9663	2709	2941	0,00	0,28	0,74	**1,36**
Ет. 28		S355	23	212	555	767	2296	9663	2709	2941	0,00	0,28	0,78	**1,28**
Ет. 27		S355	24	211	555	766	2419	9663	2709	2941	0,00	0,28	0,82	**1,22**
Ет. 26		S355	26	210	555	765	2537	9663	2709	2941	0,00	0,28	0,86	**1,16**
Ет. 25		S355	27	209	555	764	2649	9663	2709	2941	0,00	0,28	0,90	**1,11**
Ет. 24		S355	28	208	555	763	2755	9663	2709	2941	0,00	0,28	0,94	**1,07**
Ет. 23		S355	29	207	555	762	2855	9663	2709	2941	0,00	0,28	0,97	**1,03**
Ет. 22		S355	34	205	555	760	2955	9663	2709	2941	0,00	0,28	1,00	**1,00**
Ет. 21		S355	65	202	690	892	3057	10836	3188	3655	0,01	0,28	0,84	**1,20**
Ет. 20		S355	66	200	690	889	3137	10836	3188	3655	0,01	0,28	0,86	**1,17**
Ет. 19		S355	32	199	690	888	3178	10836	3188	3655	0,00	0,28	0,87	**1,15**
Ет. 18		S355	31	196	690	886	3214	10836	3188	3655	0,00	0,28	0,88	**1,14**
Ет. 17	HE900A	S355	30	193	690	883	3242	10836	3188	3655	0,00	0,28	0,89	**1,13**
Ет. 16		S355	30	190	690	880	3276	10836	3188	3655	0,00	0,28	0,90	**1,12**
Ет. 15		S355	30	187	690	877	3314	10836	3188	3655	0,00	0,28	0,91	**1,10**
Ет. 14		S355	30	184	690	873	3350	10836	3188	3655	0,00	0,27	0,92	**1,09**
Ет. 13		S355	30	180	690	870	3379	10836	3188	3655	0,00	0,27	0,92	**1,08**
Ет. 12		S355	30	176	690	865	3403	10836	3188	3655	0,00	0,27	0,93	**1,07**
Ет. 11		S355	29	172	690	861	3419	10836	3188	3655	0,00	0,27	0,94	**1,07**
Ет. 10		S355	29	167	690	856	3425	10836	3188	3655	0,00	0,27	0,94	**1,07**
Ет. 9		S355	33	163	690	852	3418	10836	3188	3655	0,00	0,27	0,94	**1,07**
Ет. 8	HE800B	S355	56	158	690	847	3395	9663	3188	3655	0,01	0,27	0,93	**1,08**
Ет. 7		S355	35	157	690	846	3482	9663	3188	3655	0,00	0,27	0,95	**1,05**
Ет. 6		S355	34	146	690	835	3599	9663	3188	3655	0,00	0,26	0,98	**1,02**
Ет. 5		S355	24	136	690	826	3583	9663	3188	3655	0,00	0,26	0,98	**1,02**
Ет. 4		S355	19	126	690	816	3506	9663	3188	3655	0,00	0,26	0,96	**1,04**
Ет. 3		S355	37	115	690	805	3378	9663	3188	3655	0,00	0,25	0,92	**1,08**
Ет. 2		S355	142	102	690	792	3155	9663	3188	3655	0,01	0,25	0,86	**1,16**
Ет. 1	HE550A	S355	980	250	295	545	1376	7161	1634	1563	0,14	0,33	0,88	**1,14**
Ет. -1		S355	67	247	295	541	538	7161	1634	1563	0,01	0,33	0,34	**2,90**
Ет. -2		S355	62	243	295	538	396	7161	1634	1563	0,01	0,33	0,25	**3,04**

2.3. Columns C-D3 and C-E3

The column is part of a moment resistant frame. In order to ensure the global ductile behavior of the structure, the frames resisting bending must be designed so as to form plastic hinges in the beams, and the columns to work entirely in the elastic stage. This requirement is presented in БДС EN 1998-1, item 6.6.

To ensure the formation of plastic hinges in the beams and to prevent plasticization in the columns, the columns of the frame are designed for increased inner forces.

2.3.1. Design checks

Columns are designed to resist bi-axial bending with compression. Influence of the shear force should not be taken into consideration if: $V_{Ed} \leq 0,5.V_{Rd}$

$$N_{Ed} = N_{Ed,G} + 1,1.\gamma_{ov}.\Omega.N_{Ed,E}$$

$$V_{Ed} = V_{Ed,G} + 1,1.\gamma_{ov}.\Omega.V_{Ed,E}$$

$$M_{Ed} = M_{Ed,G} + 1,1.\gamma_{ov}.\Omega.M_{Ed,E}$$

$$\Omega = \Omega_{i,min} = M_{Rd,i}/M_{Ed,i} = 1,0$$

Buckling resistance of the columns should be also checked. The following conditions should be satisfied:

$$\frac{N_{Ed}}{\frac{\chi_y.N_{Rk}}{\gamma_{M1}}} + \frac{k_{yy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{yz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

$$\frac{N_{Ed}}{\frac{\chi_z.N_{Rk}}{\gamma_{M1}}} + \frac{k_{zy}.M_{y,Ed}}{\frac{\chi_{LT}.M_{y,Rk}}{\gamma_{M1}}} + \frac{k_{zz}.M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1,$$

Where:

$N_{Rk} = f_y.A$; $M_{y,Rk} = f_y.W_y$; $M_{z,Rk} = f_y.W_z$ - bending and axial resistances of the cross section;

k_{yy}, k_{zz}, k_{yz} и k_{zy} – interaction coefficients;

Columns, which are part of the EBFs and MRFs are heavily loaded with compression force and bending moment due to seismic motion. This leads to the use of non-standard composite sections for heavily loaded columns. Such sections are not sensitive to buckling and for them the value of the displacement coefficient $\chi_{Lt} > 1,0$, which allows it to be assumed $\chi_{Lt} = 1,0$. For sections of this type, the values of the interaction coefficients are not defined by EC 1993-1-1. To increase safety and for more proper calculations in the present project, the values of these coefficients are accepted:

$$k_{yy} = k_{zz} = k_{yz} = k_{zy} = 1,0$$

All these assumptions allow the above formulas containing the coefficients of interaction to be reduced to a modified, simplified formula:

$$\boldsymbol{\sigma_{max}} = \frac{N_{Ed}}{A.\chi} + \frac{M_{Ed,y}}{W_{pl,y}} + \frac{M_{Ed,z}}{W_{pl,z}} \leq f_y/\gamma_{M1}$$

The values of buckling factor are equal for the main axes, because of the double symmetry and the equal inertial characteristics of the cross sections in both directions.

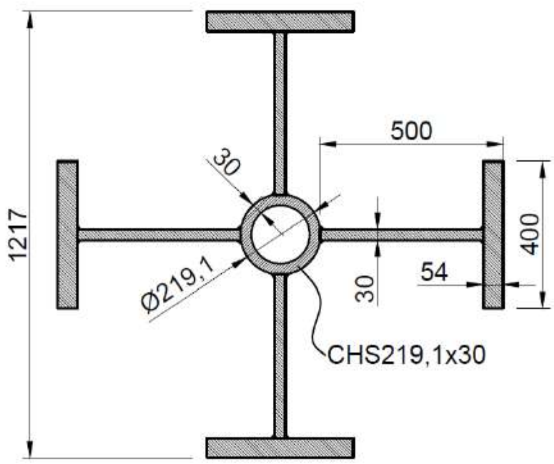
2.3.2. Characteristics of the cross sections

- *Geometrical and inertial characteristics of the cross sections*

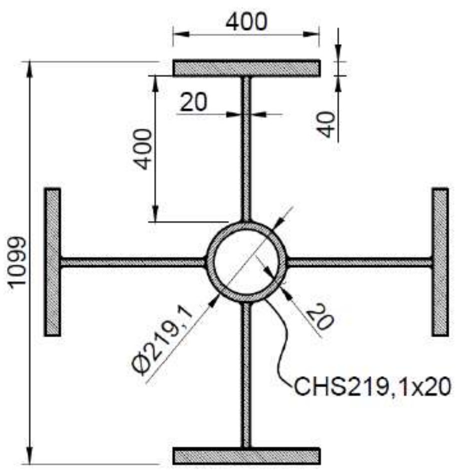
Характеристики на напречното сечение										
Колони-2x900X400X54X30+CHS219,1X30 - 1										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A*** $_{csh}$	***W*** $_{csh}$	***f*** $_{y,csh}$
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
121,7	1576,6	259,3	1886296	1886296	30999	30999	46	178	1080	35,5
Колони-2x800X400X40X20+CHS219,1X20 - 2										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A*** $_{csh}$	***W*** $_{csh}$	***f*** $_{y,csh}$
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
109,9	960,2	219,8	1115156	1115156	20294	20294	46	125	795	35,5
Колони-2xHE900B - 3										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***	***A*** $_{csh}$	***W*** $_{csh}$	***f*** $_{y,csh}$
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]	[cm^2]	[cm^3]	[kN/cm^2]
90	739	178,3	509880	509880	11331	11331	46	111	713	35,5
Колони-2xHE700B - 4										
h	***A***	***Avz***	***Iy***	***Iz***	***Wy***	***Wz***	***fy***			
[cm]	[cm^2]	[cm^2]	[cm^4]	[cm^4]	[cm^3]	[cm^3]	[kN/cm^2]			
70	609,9	137,1	271328	271328	7752	7752	46			

- *Type of cross section*

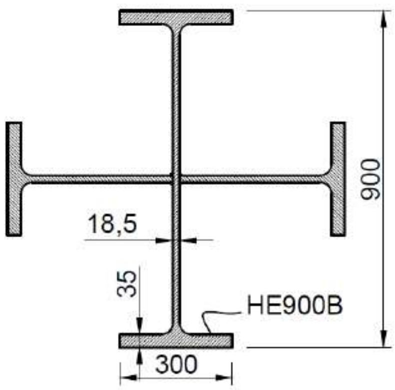
Сечение 1



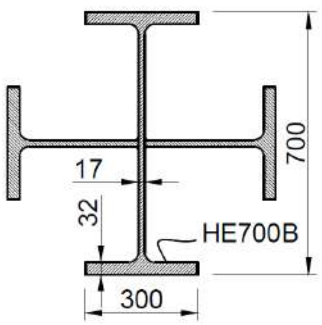
Сечение 2



Сечение 3



Сечение 4



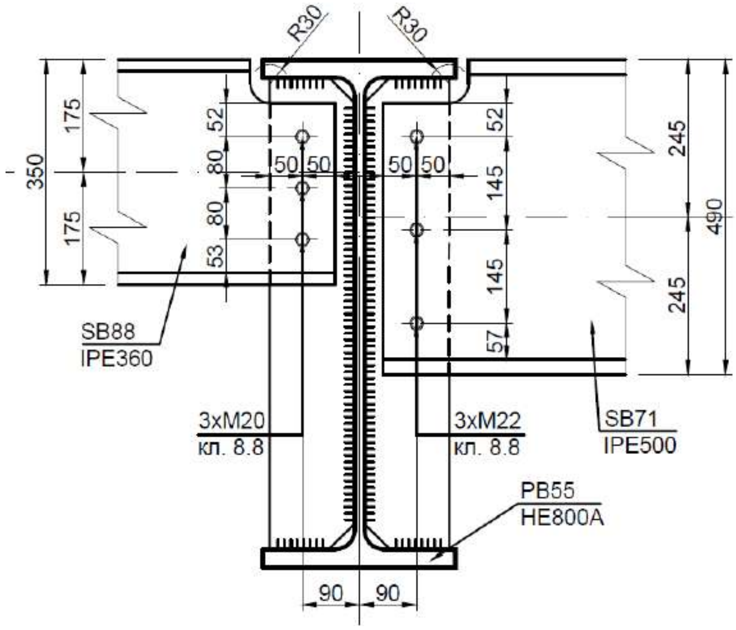
2.3.3. Presentation of the results from the checks

***Ω* =1,0 → $E_{Ed} = N_{Ed,G} + 1,1.1,25.1,0.N_{Ed,E}$**

Оразмеряване на колони C-D3 и C-E3															
Етаж	Напр. сечение	Клас стомана	N_{Ed} [kN]	V_{Ed} [kN]	$M_{Ed,y}$ [kNm]	$M_{Ed,z}$ [kNm]	N_{Rd} [kN]	V_{Rd} [kN]	$M_{Rd,y}$ [kNm]	$M_{Rd,z}$ [kNm]	$\frac{N_{Ed}}{N_{Rd}\cdot\chi}$	$\frac{V_{Ed}}{V_{Rd}}$	$\frac{M_{y,Ed}}{M_{y,Rd}}$	$\frac{M_{z,Ed}}{M_{z,Rd}}$	$\frac{\sigma_{Ed}}{f_{y,Rd}}$
Ет. ПП		S460	-	-	-	-	-	-	-	-	-	-	-	-	-
Ет. П3		S460	336	444	489	666	24836	4092	3396	3396	0,01	0,11	0,14	0,20	**0,35**
Ет. П2		S460	713	480	628	875	24836	4092	3396	3396	0,03	0,12	0,18	0,26	**0,47**
Ет. П1		S460	1104	334	493	1536	24836	4092	3396	3396	0,04	0,08	0,15	0,45	**0,64**
Ет. К		S460	1138	339	325	682	24836	4092	3396	3396	0,05	0,08	0,10	0,20	**0,34**
Ет. 31'	4	S460	1171	344	725	1711	24836	4092	3396	3396	0,05	0,08	0,21	0,50	**0,76**
Ет. 31		S460	1500	645	853	1512	24836	4092	3396	3396	0,06	0,16	0,25	0,45	**0,76**
Ет. 30'		S460	1897	753	761	1424	24836	4092	3396	3396	0,08	0,18	0,22	0,42	**0,72**
Ет. 30		S460	2343	827	772	1538	24836	4092	3396	3396	0,09	0,20	0,23	0,45	**0,77**
Ет. 29		S460	2837	898	772	1683	24836	4092	3396	3396	0,11	0,22	0,23	0,50	**0,84**
Ет. 28		S460	3371	966	817	1826	24836	4092	3396	3396	0,14	0,24	0,24	0,54	**0,91**
Ет. 27		S460+S355	3939	1029	891	1963	33581	4510	5205	5205	0,12	0,23	0,17	0,38	**0,67**
Ет. 26		S460+S355	4539	1088	975	2090	33581	4510	5205	5205	0,14	0,24	0,19	0,40	**0,72**
Ет. 25	3	S460+S355	5169	1144	1059	2209	33581	4510	5205	5205	0,15	0,25	0,20	0,42	**0,78**
Ет. 24		S460+S355	5826	1197	1138	2318	33581	4510	5205	5205	0,17	0,27	0,22	0,45	**0,84**
Ет. 23		S460+S355	6510	1248	1198	2414	33581	4510	5205	5205	0,19	0,28	0,23	0,46	**0,89**
Ет. 22		S460+S355	7221	1293	1178	2473	33581	4510	5205	5205	0,22	0,29	0,23	0,48	**0,92**
Ет. 21		S460+S355	7977	1271	1647	2986	43029	5559	9159	9159	0,19	0,23	0,18	0,33	**0,69**
Ет. 20		S460+S355	8752	1381	1489	2789	43029	5559	9159	9159	0,20	0,25	0,16	0,30	**0,67**
Ет. 19		S460+S355	9524	1423	1534	2809	43029	5559	9159	9159	0,22	0,26	0,17	0,31	**0,70**
Ет. 18		S460+S355	10300	1463	1631	2871	43029	5559	9159	9159	0,24	0,26	0,18	0,31	**0,73**
Ет. 17		S460+S355	11106	1503	1580	2939	43029	5559	9159	9159	0,26	0,27	0,17	0,32	**0,75**
Ет. 16		S460+S355	11918	1533	1605	3001	43029	5559	9159	9159	0,28	0,28	0,18	0,33	**0,78**
Ет. 15	2	S460+S355	12741	1558	1644	3055	43029	5559	9159	9159	0,30	0,28	0,18	0,33	**0,81**
Ет. 14		S460+S355	13576	1579	1682	3100	43029	5559	9159	9159	0,32	0,28	0,18	0,34	**0,84**
Ет. 13		S460+S355	14422	1596	1720	3134	43029	5559	9159	9159	0,34	0,29	0,19	0,34	**0,87**
Ет. 12		S460+S355	15279	1607	1756	3157	43029	5559	9159	9159	0,36	0,29	0,19	0,34	**0,89**
Ет. 11		S460+S355	16147	1612	1800	3165	43029	5559	9159	9159	0,38	0,29	0,20	0,35	**0,92**
Ет. 10		S460+S355	17025	1614	1863	3148	43029	5559	9159	9159	0,40	0,29	0,20	0,34	**0,94**
Ет. 9		S460+S355	17907	1620	1816	3070	43029	5559	9159	9159	0,42	0,29	0,20	0,34	**0,95**
Ет. 8		S460+S355	18777	1571	1178	2346	43029	5559	9159	9159	0,44	0,28	0,13	0,26	**0,82**
Ет. 7		S460+S355	19681	1568	1807	3544	69795	6559	13946	13946	0,28	0,24	0,13	0,25	**0,67**
Ет. 6		S460+S355	20548	1515	1816	3795	69795	6559	13946	13946	0,29	0,23	0,13	0,27	**0,70**
Ет. 5		S460+S355	21418	1497	1824	3797	69795	6559	13946	13946	0,31	0,23	0,13	0,27	**0,71**
Ет. 4		S460+S355	22298	1470	1877	3732	69795	6559	13946	13946	0,32	0,22	0,13	0,27	**0,72**
Ет. 3	1	S460+S355	23195	1473	1750	3826	69795	6559	13946	13946	0,33	0,22	0,13	0,27	**0,73**
Ет. 2		S460+S355	24133	1231	3839	4370	69795	6559	13946	13946	0,35	0,19	0,28	0,31	**0,93**
Ет. 1		S460+S355	25371	119	1539	931	69795	6559	13946	13946	0,36	0,02	0,11	0,07	**0,54**
Ет. -1		S460+S355	26531	114	530	341	69795	6559	13946	13946	0,38	0,02	0,04	0,02	**0,44**
Ет. -2		S460+S355	27685	50	241	126	69795	6559	13946	13946	0,40	0,01	0,02	0,01	**0,42**

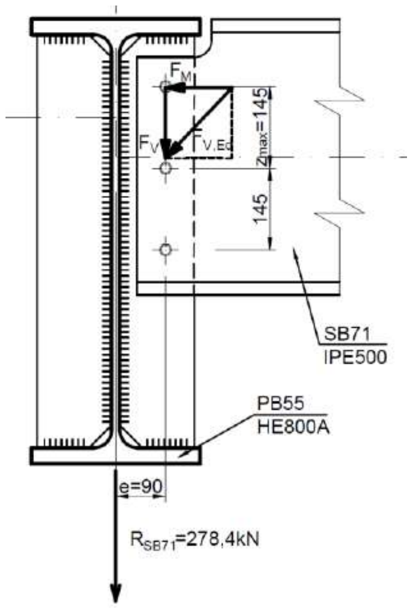
VII. Design of different types of joints

1. Design of bolted connection between primary beam and secondary beam */Detail "A"/*



1.1. <u>Connection between PB55 /HE800A/ and SB71 /IPE500/</u>

According to БДС EN 1993-8, item 3.4., The bolted connection is category A, i.e. Resist shear and bearing. It is necessary to prove the bearing capacity of the joint of shear and crushing.



$F_{v,Ed} \leq F_{v,Rd}$

$F_{v,Ed} \leq F_{b,Rd}$

$F_{v,Ed} = \sqrt{F_v^2 + F_M^2}$

$F_V = R_{SB71}/3 = 278{,}4/3 = 92{,}8\ kN$

$F_M = \frac{M_v . z_{max}}{\sum z_i^2} = \frac{2\ 505{,}6.14{,}5}{2.14{,}5^2} = 86{,}4\ kN$

$M_v = e.R^{SB71} = 9.278{,}4 = 2\ 505{,}6\ kNcm$

$F_{v,Ed} = \sqrt{92{,}8^2 + 86{,}4^2} = 126{,}8\ kN$

$\boldsymbol{F_{v,Ed} = 126{,}8\ kN}$

Type of bolts M22, class 8.8 is selected!

1.1.1. Shear resistance

$F_{v,Rd} = \frac{\alpha_v . f_{ub} . A}{\gamma_{M2}}$,

Where:

$f_{ub} = 800\ MPa = 80\ kN/cm^2$ – Tensile strength of bolt class 8.8;

$A = 3{,}8\ cm^2$ – for bolts M22;

$\gamma_{M2} = 1{,}25;$

It is assumed that the shear plane passes through the unthreaded portion of the bolt and the area A is the gross cross section of the bolt:

$\alpha_v = 0,6$

$F_{v,Rd} = \frac{0,6.80.3,8}{1,25} = 145,9\ kN \rightarrow \boldsymbol{F_{v,Rd} = 145,9\ kN}$

$F_{v,Ed} \leq F_{v,Rd} \rightarrow F_{v,Ed} = 126,8\ kN < F_{v,Rd} = 145,9\ kN \rightarrow$ Check is satisfied!

1.1.2. Bearing resistance

$F_{b,Rd} = \frac{k_1.\alpha_b.f_u.d.t}{\gamma_{M2}}$

$$\alpha_b = min \begin{cases} \frac{f_{ub}}{f_u} \\ 1,0 \\ \alpha_d \end{cases}$$

For end bolts:

$\alpha_d = \frac{e_1}{3.d_0} = \frac{50}{3.24} = 0,7$

$e_1 = 50\ mm = 5cm$

$f_u = 430\ MPa = 43\ kN/cm^2$ – for steel class *S275* (secondary beam)

$d_0 = d + \Delta = 22 + 2 = 24\ mm$

$\Delta = 2\ mm$ – For bolts M16 – M24;

$$\alpha_b = min \begin{cases} \frac{80}{50} \\ 1,0 \\ 0,7 \end{cases} = min \begin{cases} 1,6 \\ 1,0 \\ 0,7 \end{cases} \rightarrow \boldsymbol{\alpha_b = 0,7}$$

For end bolts:

$$k_1 = min \left\{ \begin{cases} 2,8.\frac{e_2}{d_0} - 1,7 \\ 2,5 \end{cases} \right. = \begin{cases} 2,8.\frac{52}{24} - 1,7 \\ 2,5 \end{cases} = min \begin{cases} 4,4 \\ 2,5 \end{cases} \rightarrow \boldsymbol{k_1 = 2,5}$$

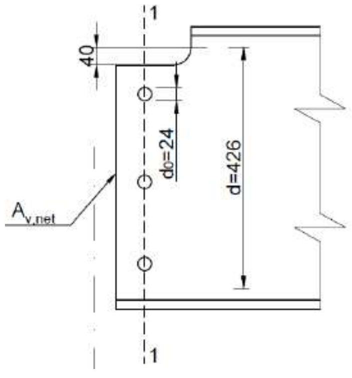
$e_2 = 52\ mm = 5,2\ cm$

$t = t_{w,ребро} = 10\ mm = 1,0\ cm$

$F_{b,Rd} = \frac{2,5.0,7.43.2,2.1,0}{1,25} = 132,4\ kN$

$F_{v,Ed} \leq F_{b,Rd} \rightarrow F_{v,Ed} = 126,8\ kN < F_{b,Rd} = 132,4\ kN \rightarrow$ Check is satisfied!

1.1.3. Deduction for fastener holes in section "1-1"



$V_{Ed} \leq V_{Rd}$

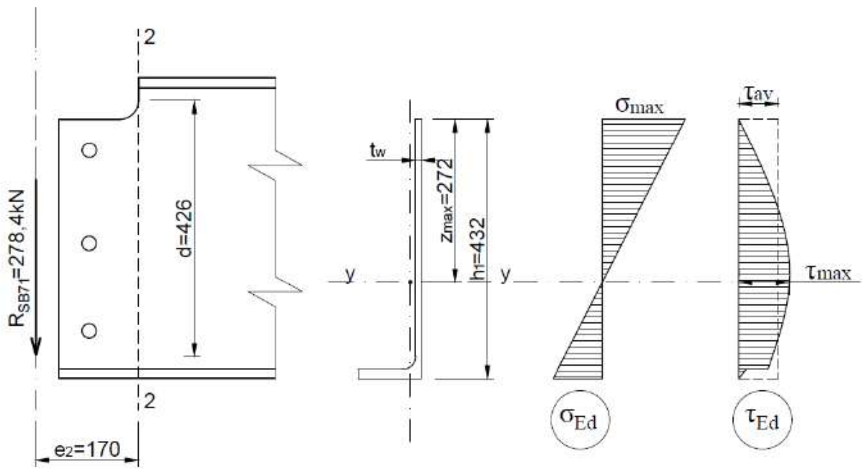
$A_{v,net} = d.t_w - 4cm.t_w - 3.d_0.t_w$

$A_{v,net} = 42,6.1,02 - 4.1,02 - 3.2,4.1,02 = 32,03\ cm^2$

$V_{Rd} = A_{v,net}.f_{yd}/\sqrt{3} = 32,03.27,5/(1,05.\sqrt{3}) = 484\ kN$

$\rightarrow V_{Ed} = 278,4\ kN < V_{Rd} = 484\ kN \rightarrow$ Check is satisfied!

1.1.4. Deduction for fastener holes in section "2-2"



$\sigma_{VM} = \sqrt{\sigma_{max}^2 + 3.\tau_{av}^2} \leq f_y/\gamma_{M0}$

$\sigma_{max} = M_{1\text{-}1}/W_{top}$

$M_{1\text{-}1} = R_{SB71}.e_2 = 278{,}4.17 = 4732{,}8\ kNcm$

$W_{top} = I_y/z_{max}$

$I_y = 11\ 929\ cm^2 \rightarrow W_{top} = 11\ 929/27{,}2 = 430{,}6\ cm^3$

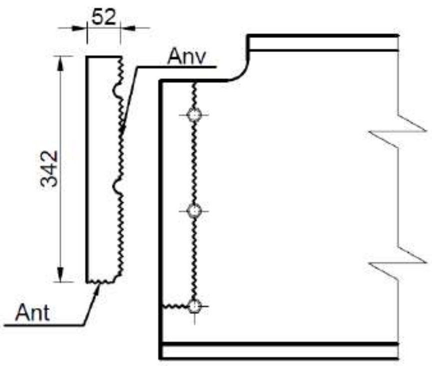
$\sigma_{max} = 4732{,}8/430{,}6 = 10{,}99\ kN/cm^2$

$\tau_{av} = R_{SB71}/(h_1.t_w) = 278/(43{,}2.1{,}02) = 6{,}32\ kN/cm^2$

$\sigma_{VM} = \sqrt{\sigma_{max}^2 + 3.\tau_{av}^2} = \sqrt{10{,}99^2 + 3.6{,}32^2} = 15{,}51\ kN/cm^2 \rightarrow \boldsymbol{\sigma_{VM} = 15{,}51\ kN/cm^2}$

$\boldsymbol{f_y/\gamma_{M0}} = 27{,}5/1{,}05 = \boldsymbol{26{,}2\ kN/cm^2 \rightarrow \sigma_{VM} = 15{,}51\ kN/cm^2 \leq 26{,}2\ kN/cm^2}$ **→ Check is satisfied!**

1.1.5. Design for block tearing



$V_{eff,Rd} = 0{,}5.f_u.A_{nt}/\gamma_{M2} + f_y.A_{nv}/\sqrt{3}.\ \gamma_{M0} > V_{Ed}$,

Where:

A_{nt} - net area subjected to tension;

A_{nv} - net area subjected to shear;

$A_{nv} = t_w.(34{,}2 - 2{,}5.d_0) = 1{,}02.(34{,}2 - 2{,}5.2{,}4) = 28{,}8\ cm^2$

$A_{nt} = t_w.(5{,}2 - 0{,}5.d_0) = 1{,}02.(5{,}2 - 0{,}5.2{,}4) = 4{,}08\ cm^2$

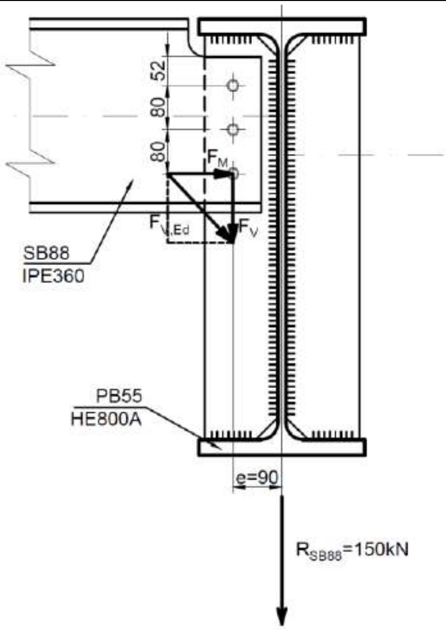
$V_{eff,Rd} = 0{,}5.50.4{,}08/1{,}25 + 27{,}5.28{,}8/(\sqrt{3}.1{,}05) = 517\ kN$

→ $\boldsymbol{V_{eff,Rd} = 517\ kN > V_{Ed} = 278{,}4\ kN}$ → Check is satisfied!

4.14. <u>Connection between beams PB55 /HE800A/ and SB88 /IPE360/</u>

According to БДС EN 1993-8, item 3.4., The bolted connection is category A, i.e. Resist shear and bearing. It is necessary to prove the bearing capacity of the joint of shear and crushing.

$F_{v,Ed} \leq F_{v,Rd}$



$F_{v,Ed} \leq F_{b,Rd}$

$F_{v,Ed} = \sqrt{F_v^2 + F_M^2}$

$F_V = R_{SB88}/3 = 150/3 = 50\ kN$

$F_M = \frac{M_v . z_{max}}{\sum z_i^2} = \frac{1350.8}{2.8^2} = 84,4\ kN$

$M_v = e.R_{SB88} = 9.150 = 1350\ kNcm$

$F_{v,Ed} = \sqrt{50^2 + 84,4^2}\ = 98,1\ kN$

$\boldsymbol{F_{v,Ed} = 98,1\ kN}$

Type of bolts M20, class 8.8 is selected!

1.1.6. Shear resistance

$F_{v,Rd} = \frac{\alpha_v . f_{ub} . A}{\gamma_{M2}}$,

Where:

$f_{ub} = 800\ MPa = 80\ kN/cm^2$ – tensile strength of bolt class 8.8;

$A = 3,14\ cm^2$ – за болт M20;

$\gamma_{M2} = 1,25;$

It is assumed that the shear plane passes through the unthreaded portion of the bolt and the area A is the gross cross section of the bolt:

$\alpha_v = 0,6$

$F_{v,Rd} = \frac{0,6.80.3,14}{1,25} = 120,6\ kN \rightarrow \boldsymbol{F_{v,Rd} = 120,6\ kN}$

$\boldsymbol{F_{v,Ed} \leq F_{v,Rd} \rightarrow F_{v,Ed} = 98,1\ kN < F_{v,Rd} = 120,6\ kN \rightarrow}$ ***Check is satisfied!***

1.1.7. Bearing resistance

$F_{b,Rd} = \frac{k_1 . \alpha_b . f_u . d . t}{\gamma_{M2}}$

$\alpha_b = min \begin{cases} \frac{f_{ub}}{f_u} \\ 1,0 \\ \alpha_d \end{cases}$

For end bolts:

$\alpha_d = \frac{e_1}{3.d_0} = \frac{50}{3.22} = 0,75$

$e_1 = 50\ mm = 5cm$

$f_u = 430\ MPa = 43\ kN/cm^2$ – for steel class *S275* (secondary beam)

$d_0 = d + \Delta = 20 + 2 = 22\ mm$

$\Delta = 2\ mm$ – For bolts M16 – M24;

$$\alpha_b = min\begin{cases}\frac{80}{50}\\1{,}0\\0{,}75\end{cases} = min\begin{cases}1{,}6\\1{,}0\\0{,}75\end{cases} \rightarrow \boldsymbol{\alpha_b = 0{,}75}$$

For end bolts:

$$k_1 = min\left\{\begin{cases}2{,}8.\frac{e_2}{d_0} - 1{,}7\\ \quad 2{,}5\end{cases}\right. = \begin{cases}2{,}8.\frac{52}{20} - 1{,}7\\ \quad 2{,}5\end{cases} = min\begin{cases}5.58\\2{,}5\end{cases} \rightarrow \boldsymbol{k_1 = 2{,}5}$$

$e_2 = 52\ mm = 5{,}2\ cm$

$t = t_{w,ребро} = 10\ mm = 1{,}0\ cm$

$F_{b,Rd} = \frac{2{,}5.0{,}75.43.2{,}0.1{,}0}{1{,}25} = 129\ kN$

$\boldsymbol{F_{v,Ed} \leq F_{b,Rd} \rightarrow F_{v,Ed} = 98{,}1\ kN < F_{b,Rd} = 129\ kN}$ ***→ Check is satisfied!***

1.1.8. Deduction for fastener holes in section „1-1“

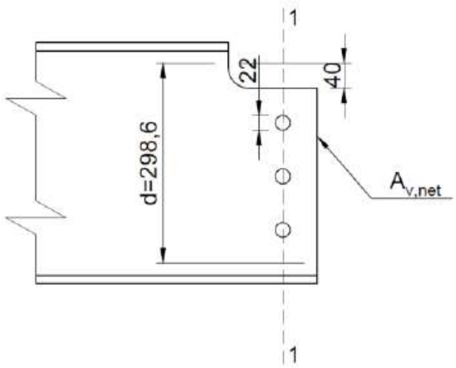
$V_{Ed} \leq V_{Rd}$

$A_{v,net} = d_0.t_w - 4.t_w - 3.d_0.t_w$

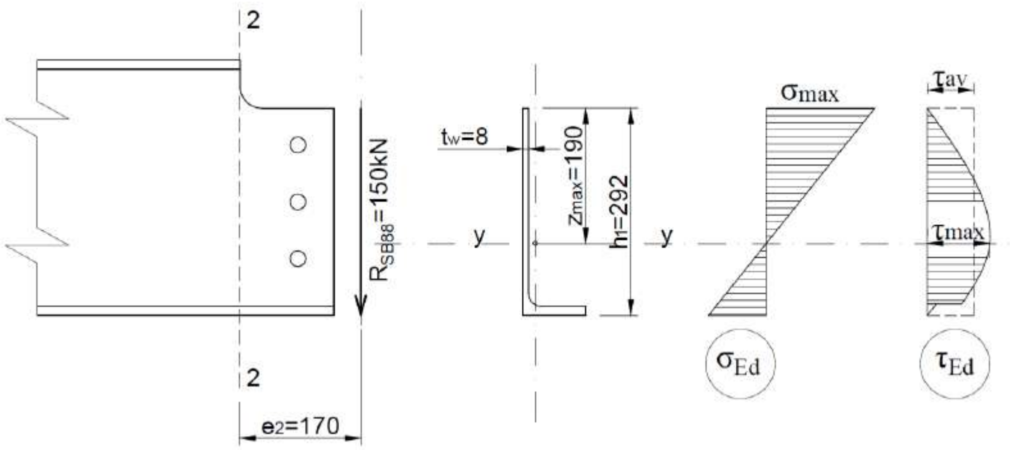
$A_{v,net} = 29{,}86.0{,}8 - 4cm.0{,}8 - 3.2{,}2.0{,}8 = 15{,}41\ cm^2$

$V_{Rd} = A_{v,net}.f_{yd}/\sqrt{3} = 15{,}41.27{,}5/(1{,}05.\sqrt{3}) = 233\ kN$

$\boldsymbol{\rightarrow V_{Ed} = 150\ kN < V_{Rd} = 233\ kN \rightarrow}$ ***Check is satisfied!***



1.1.9. Deduction for fastener holes in section „2-2“



$\sigma_{VM} = \sqrt{\sigma_{max}^2 + 3.\tau_{av}^2} \leq f_y/\gamma_{M0}$

$\sigma_{max} = M_{1\text{-}1}/W_{top}$

$M_{1\text{-}1} = R_{SB88}.e_2 = 150.17 = 2550\ kNcm$

$W_{top} = I_y/z_{max}$

$I_y = 3105\ cm^2 \rightarrow W_{top} = 3105/19 = 163\ cm^3$

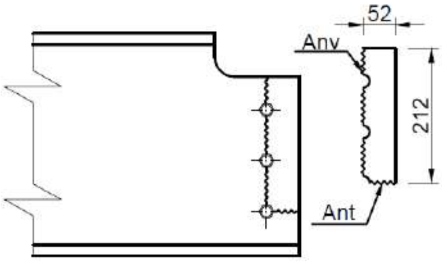
$\sigma_{max} = 2550/163 = 15{,}65\ kN/cm^2$

$\tau_{av} = R_{SB71}/(h_1.t_w) = 150/(29{,}2.0{,}8) = 6{,}42\ kN/cm^2$

$\sigma_{VM} = \sqrt{\sigma_{max}^2 + 3.\tau_{av}^2} = \sqrt{15{,}65^2 + 3.6{,}42^2} = 19{,}2\ kN/cm^2 \rightarrow \boldsymbol{\sigma_{VM} = 19{,}2\ kN/cm^2}$

$f_y/\gamma_{M0} = 27{,}5/1{,}05 = \mathbf{26{,}2\ kN/cm^2} \rightarrow \boldsymbol{\sigma_{VM} = 19{,}2\ kN/cm^2 \leq 26{,}2\ kN/cm^2} \rightarrow$ ***Check is satisfied!***

1.1.10. Design for block tearing



$V_{eff,,Rd} = 0{,}5.f_u.A_{nt}/\gamma_{M2} + f_y.A_{nv}/\sqrt{3}.\ \gamma_{M0} > V_{Ed}$,

Where:

A_{nt} - net area subjected to tension;

A_{nv} - net area subjected to shear;

$A_{nv} = t_w.(21{,}2 - 2{,}5.d_0) = 0{,}8.(21{,}2 - 2{,}5.2{,}2) = 12{,}56\ cm^2$

$A_{nt} = t_w.(5{,}2 - 0{,}5.d_0) = 0{,}8.(5{,}2 - 0{,}5.2{,}2) = 3{,}28\ cm^2$

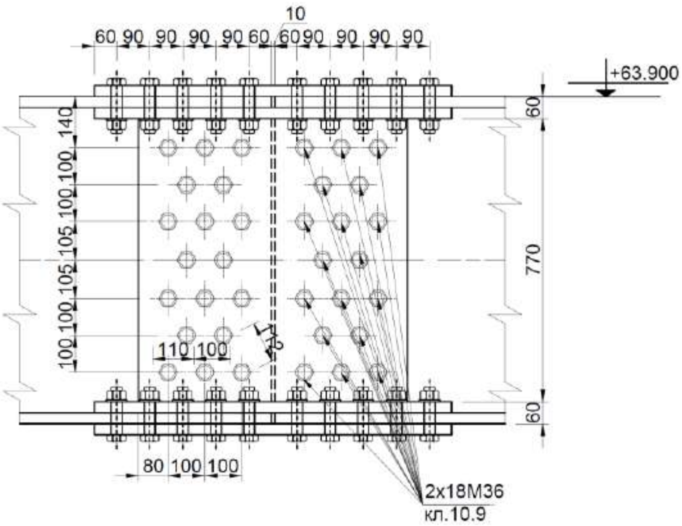
$V_{eff,Rd} = 0{,}5.50.3{,}28/1{,}25 + 27{,}5.12{,}56/(\sqrt{3}.1{,}05) = 255\ kN$

$\rightarrow \boldsymbol{V_{eff,Rd} = 255\ kN > V_{Ed} = 150\ kN} \rightarrow$ ***Check is satisfied!***

2. Design of bolted splice connection between primary beam and column */Detail "B"/*

The splicing of the primary beam and the column is extended out of the joint in order to reduce the shear forces that the connection has to resist. The column is factory made with brackets welded by full welding. The connection between the two parts of the main beam is made by means of a bolted splice connection.

4.15. Out of joint splicing between beam PB54 /HE900A/ and column C-C5_8 /composite cross section/



Two profiles *HE900A* are connected by splices. According to БДС EN 1993-8, т. 3.4., connection is category "C". The connection is slip-resistant at serviceability limit state and should resist slip and bearing.

$F_{v,Ed} \leq F_{s,Rd}$

$F_{v,Ed} \leq F_{b,Rd}$

The structure is designed to dissipate energy in the beams, which means that the joint must be designed for increased load-bearing capacity, according to БДС EN 1998-1, item 6.6.4. The joint must be designed as non-dissipative and its elastic behavior must be ensured.

$R_d \geq 1{,}1.\gamma_{ov}.R_{fy}$

$M_{Ed} = 1{,}1.\ \gamma_{ov}.M_{pl,Rd} = 1{,}1.1{,}25.3655 = 5021\ kNm$

$V_{Ed} = V_{Ed,G} + 1{,}1.\ \gamma_{ov}.V_{Ed,M} = 396 + 1{,}1.1{,}25..2M_{pl,Rd}/L_{pl} = 396 + 1{,}1.1{,}25.3655/11{,}10 = 1252\ kN$

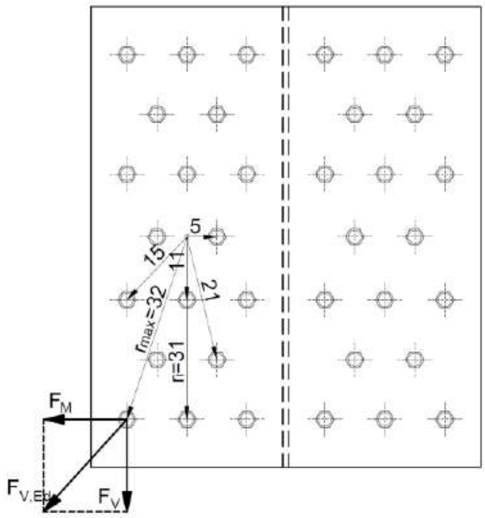
$N_{Ed} = N_{Ed.G} + 1{,}1.\ \gamma_{ov}.N_{Ed,E} = 13{,}66 + 1{,}1.1{,}25.55 = 89{,}3\ kN$

$\boldsymbol{M_{Ed} = 5021\ kNm\ ;\quad V_{Ed} = 1252\ kN\ ;\quad N_{Ed} = 89{,}3\ kN}$

2.1.1. Design of the web splicing

Bolts selected: **18xM36, class 10.9** – the number of bolts is calculated for one side of the connection;

➢ *Shear force in the most loaded bolt:*



$F_{v,Ed} = \sqrt{F_v^2 + F_h^2}$

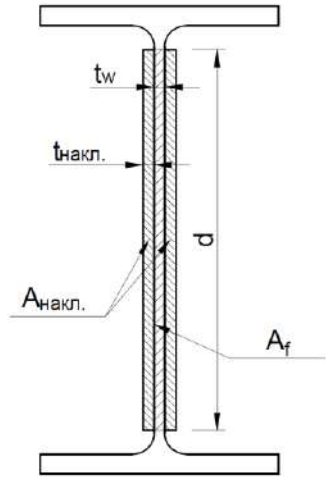
$F_v = V_{Ed}/n_w = 1252/18 = 69,5\ kN$

$F_h = \frac{N_{Ed}.A_w}{n_w.A} + \frac{r_{max}.M_w}{\sum r_i^2} = \frac{89,3.1,6.77}{18.320,5} + \frac{32.93400}{2.(2.32^2+31^2+2.21^2+11^2+2.15^2+2.5^2)} = 1,9 + 331 = 332,9\ kN$

$M_w = \frac{M_{Ed}.I_{w,y}}{I_y} + V_{Ed}.e = \frac{5021.1,6.77^3/12}{420100} + 1252.0,165 = 934\ kNm$

$F_{v,Ed} = \sqrt{69,5^2 + 332,9^2} = 340\ kN \rightarrow \boldsymbol{F_{v,Ed} = 340\ kN}$

➢ *definition of the slice thickness:*



$A_{накл.} \geq 1,2.A_w$

$A_{накл.} = 2.d.t_{накл.} = 2.77.\ t_{накл.} = 154.\ t_{накл.}$

$A_w = t_w.d = 1,6.77 = 123,2\ cm^2$

$t_{накл.} \geq 1,2.123,2/154 = 0,96 \rightarrow \boldsymbol{t_{накл.} = 10\ mm}$

➢ *Slip resistance:*

$F_{s,Rd} = \frac{k_s.n.\mu}{\gamma_{M3}}.F_{p,c}$,

Where:

$k_s = 1,0$ – for bolts in normal holes;

$n = 2$ – number of the friction planes;

$\mu = 0,5$ - class A of friction surfaces;

$\gamma_{M3} = 1,25$

$F_{p,c} = 0,7.f_{ub}.A_s$ – preloading force for bolts conforming with controlled tightening in conformity;

$A_{s,M36} = 8,17\ cm^2$

$F_{p,c} = 0,7.100.8,17 = 571,9\ kN$

$F_{s,Rd} = \frac{1,0.2.0,5}{1,25}.571,9 = 457,5\ kN \rightarrow \boldsymbol{F_{s,Rd} = 457,5\ kN}$

➢ *Bearing resistance:*

$F_{b,Rd} = \frac{k_1.\alpha_b.f_u.d.t}{\gamma_{M2}}$

$\alpha_b = min \begin{cases} \frac{f_{ub}}{f_u} \\ 1,0 \\ \alpha_d \end{cases}$

For end bolts:

$\alpha_d = \frac{e_1}{3.d_0} = \frac{80}{3.39} = 0,68$

$e_1 = 80\ mm$

$f_u = 510\ MPa = 51\ kN/cm^2$ – for steel class *S355*

$d_0 = d + \Delta = 36 + 3 = 39\ mm$

$\Delta = 3\ mm$ – For bolts M36;

$\alpha_b = min \begin{cases} \frac{100}{60} \\ 1,0 \\ 0,68 \end{cases} = min \begin{cases} 1,67 \\ 1,0 \\ 0,68 \end{cases} \rightarrow \boldsymbol{\alpha_b = 0,68}$

$k_1 = min \begin{cases} 2,8.\frac{e_2}{d_0} - 1,7 \\ 1,4.\frac{p_2}{d_0} - 1,7 \\ 2,5 \end{cases} = min \begin{cases} 2,8.\frac{80}{39} - 1,7 \\ 1,4.\frac{110}{39} - 1,7 \\ 2,5 \end{cases} = min \begin{cases} 4,0 \\ 2,25 \\ 2,5 \end{cases} \rightarrow \boldsymbol{k_1 = 2,25}$

$e_2 = 60\ mm$

$p_2 = 100\ mm$

$t = t_{w.} = 16\ mm = 1,6\ cm$

$F_{b,Rd} = \frac{0,68.2,25.51.3,6.1,6}{1,25} = 359,4\ kN$

$F_{Rd} = min\ \{F_{s,Rd}\ ;\ F_{b,Rd}\} = min\ \{457,5\ kN;\ 332\ kN\} \rightarrow \boldsymbol{F_{Rd} = 359,4\ kN > F_{v,Ed} = 340\ kN}$

Total bolts for the connection of webs: 2x18M36, class 10.9

2.1.2. Design of the flange splicing

Bolts selected: **M36, кл. 12.9**. The necessary number of bolts should be determined.

$F_{Ed} = \frac{N_{Ed}.A_f}{A} + \frac{M_{Ed} - M_w}{h_0} = \frac{89,3.30.3}{320,5} + \frac{5021 - 727,5}{0,89} = 4\ 849\ kN \rightarrow \boldsymbol{F_{Ed} = 4\ 849\ kN}$

➢ *Definition of the slice thickness:*

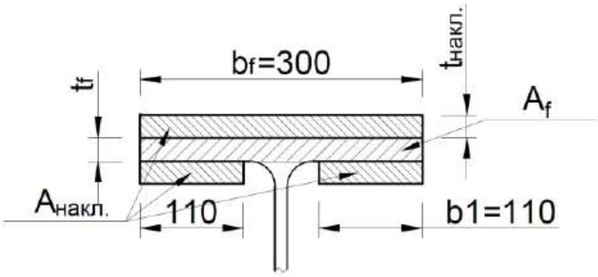
$A_{накл.} \geq 1,2.A_f$

$A_{накл.} = (2.b_1 + b_f).t_{накл.} = (2.11 + 30).\ t_{накл.}$

$A_{накл} = 52.\ t_{накл.}$

$A_f = b_f.t_f = 30.3,0 = 90\ cm^2$

$\rightarrow t_{накл.} \geq 1,2.\ 90/52 = 2,1 \rightarrow \boldsymbol{t_{накл.} = 22\ mm}$



- *Slip resistance:*

$F_{s,Rd} = \frac{k_s.n.\mu}{\gamma_{M3}}.F_{p,c}$,

Where:

$k_s = 1,0$ – for bolts in normal holes;

$n = 2$ – number of the friction planes;

$\mu = 0,5$ - class A of friction surfaces;

$\gamma_{M3} = 1,25$

$F_{p,c} = 0,7.f_{ub}.A_s$ – preloading force for bolts conforming with controlled tightening in conformity;

$F_{p,c} = 0,7.120.8,17 = 686,3\ kN$

$F_{s,Rd} = \frac{1,0.2.0,5}{1,25}.686,3 = 549\ kN \rightarrow \boldsymbol{F_{s,Rd} = 549\ kN}$

- *Bearing resistance:*

$F_{b,Rd} = \frac{k_1.\alpha_b.f_u.d.t}{\gamma_{M2}}$

$\alpha_b = min \begin{cases} \frac{f_{ub}}{f_u} \\ 1,0 \\ \alpha_d \end{cases}$

For end bolts:

$\alpha_d = \frac{e_1}{3.d_0} = \frac{60}{3.39} = 0,51$

$e_1 = 60\ mm$

$f_u = 510\ MPa = 51\ kN/cm^2$ – for steel class *S355*

$d_0 = d + \Delta = 36 + 3 = 39\ mm$

$\Delta = 3\ mm$ – For bolts M36;

$\alpha_b = min \begin{cases} \frac{100}{60} \\ 1,0 \\ 0,51 \end{cases} = min \begin{cases} 1,67 \\ 1,0 \\ 0,51 \end{cases} \rightarrow \boldsymbol{\alpha_b = 0,51}$

For end bolts:

$k_1 = min \begin{cases} 2,8.\frac{e_2}{d_0} - 1,7 \\ 2,5 \end{cases} = min \begin{cases} 2,8.\frac{55}{39} - 1,7 \\ 2,5 \end{cases} = min \begin{cases} 2,25 \\ 2,5 \end{cases} \rightarrow \boldsymbol{k_1 = 2,25}$

$e_2 = 55\ mm$

$t = t_f = 30\ mm = 3\ cm$

$F_{b,Rd} = \frac{2,25.0,51.51.3,6.3}{1,25} = 506\ kN \rightarrow \boldsymbol{F_{b,Rd} = 506\ kN}$

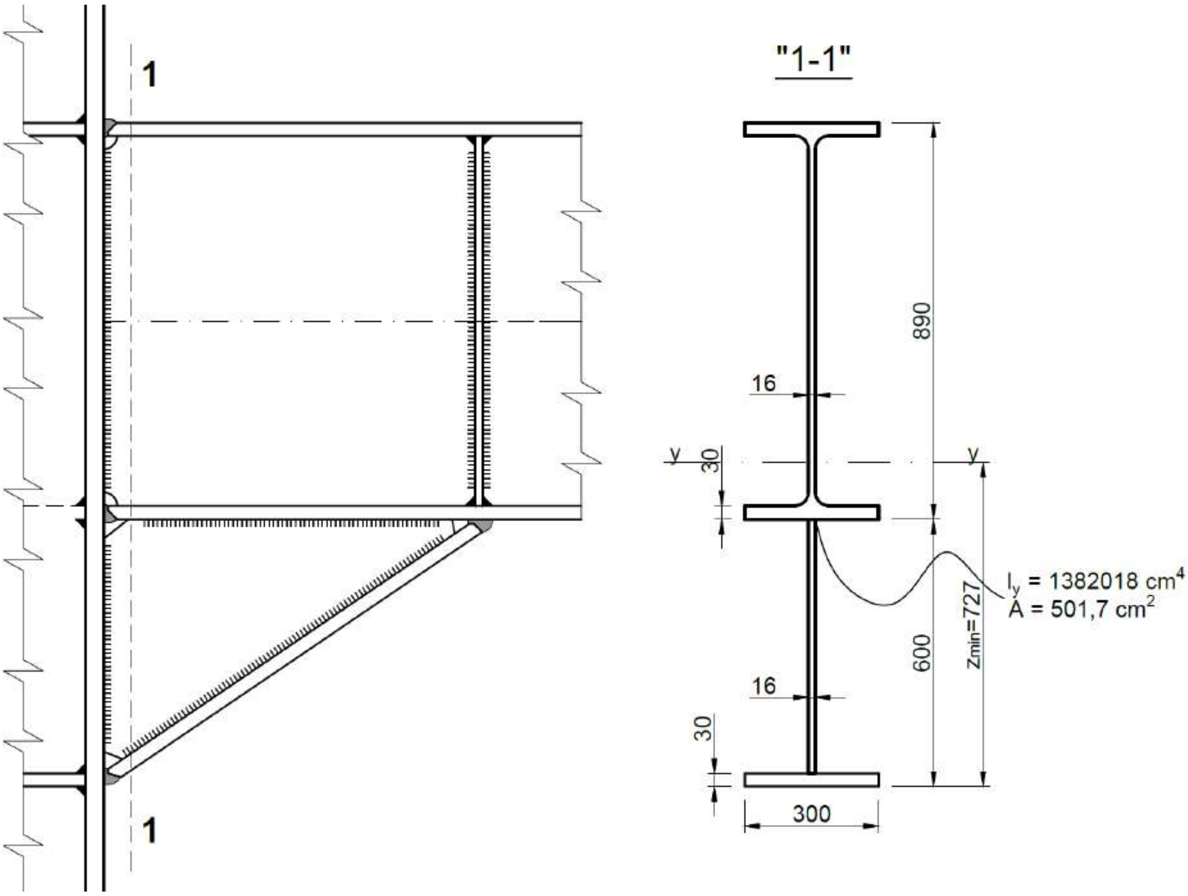
$F_{Rd} = min\ \{F_{s,Rd}\ ;\ F_{b,Rd}\} = min\ \{549\ kN;\ 506\ kN\} \rightarrow \boldsymbol{F_{Rd} = 506\ kN}$

$n_f = F_{Ed}/2.F_{Rd} = 4849/2.506 = 4,8 \rightarrow \boldsymbol{n_f = 5}$ ***бр***

Total bolts for the connection of flange: 2x2x5M36, class 12.9

2.1.3. Resistance of the section of the beam with haunch

The joint is considered rigid and the resistance of section "1-1" for increased value of the force should be checked.



$\boldsymbol{M_{Ed} = 5021\ kNm\ ;\quad V_{Ed} = 1252\ kN\ ;\quad N_{Ed} = 89{,}3\ kN}$

$M_{Rd} = W_y.f_{yd} = I_y/z_{min}.f_{yd} = 1382018/72{,}7.35{,}5/1{,}05 = 642\ 715\ kNcm \rightarrow \boldsymbol{M_{Rd} = 6427\ kNm}$

$V_{Rd} = A_{vz}.f_{yd}/\sqrt{3} = 238{,}4.35{,}5/(1{,}05.\sqrt{3}) = 4472\ kN \rightarrow \boldsymbol{V_{Rd} = 4472\ kN}$

$A_{vz} \approx t.h = 1{,}6.(60 + 89) = 238{,}4\ cm^2$

$N_{Rd} = A.f_{yd} = 501{,}7.35{,}5/1{,}05 = 16\ 962\ kN \rightarrow \boldsymbol{N_{Rd} = 16\ 962\ kN}$

Required checks:

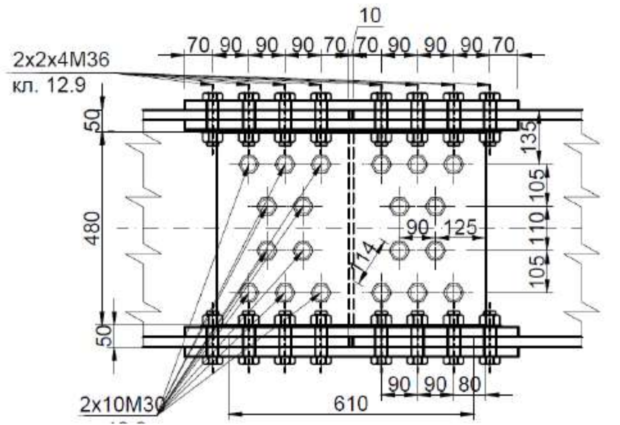
$N_{Ed} \leq 0{,}15.N_{Rd} \rightarrow$ ***N_{Ed} = 89,3 kN < 0,15.16 962 = 2 544 kN → Satisfied!***

$V_{Ed} \leq 0{,}5.V_{Rd} \rightarrow$ ***V_{Ed} = 1252 kN < 0,5.4472 = 2236 kN → Satisfied!***

$M_{Ed} \leq M_{Rd} \rightarrow$ ***M_{Ed} = 5021 kNm < M_{Rd} = 6427 kNm → Satisfied!***

→ The section has the required load bearing capacity!

4.16. <u>Out of joint splicing between beam PB53 /HE600A/ and column C-C5_8 /composite cross section /</u>



Two profiles *HE900A* are connected by splices. According to БДС EN 1993-8, т. 3.4., connection is category “C”. The connection is slip-resistant at serviceability limit state and should resist slip and bearing.

$F_{v,Ed} \leq F_{s,Rd}$

$F_{v,Ed} \leq F_{b,Rd}$

The structure is designed to dissipate energy in the beams, which means that the joint must be designed for increased load-bearing capacity, according to БДС EN 1998-1, item 6.6.4. The joint must be designed as non-dissipative and its elastic behavior must be ensured.

$R_d \geq 1{,}1.\gamma_{ov}.R_{fy}$

$M_{Ed} = 1{,}1.\ \gamma_{ov}.M_{pl,Rd} = 1{,}1.1{,}25.1809 = 2487\ kNm$

$V_{Ed} = V_{Ed,G} + 1{,}1.\ \gamma_{ov}.V_{Ed,M} = 151 + 1{,}1.1{,}25.2.2487/L_{pl} = 151 + 1{,}1.1{,}25.2487/5{,}60 = 762\ kN$

$N_{Ed} = N_{Ed.G} + 1{,}1.\ \gamma_{ov}.N_{Ed,E} = 12{,}7 + 1{,}1.1{,}25.38 = 64{,}9\ kN$

$\boldsymbol{M_{Ed} = 2487\ kNm\ ;\quad V_{Ed} = 762\ kN\ ;\quad N_{Ed} = 64{,}9\ kN}$

2.1.4. Design of the web splicing

Bolts selected: **10xM30, кл. 10.9** – the number of bolts is calculated for one side of the connection;

- *<u>Shear force in the most loaded bolt:</u>*

$F_{v,Ed} = \sqrt{F_v^2 + F_h^2}$

$F_v = V_{Ed}/n_w = 762/10 = 76{,}2\ kN$

$F_h = \frac{N_{Ed}.A_w}{n_w.A} + \frac{z_{max}.M_w}{\sum z_i^2} = \frac{64{,}9.1{,}3.48{,}6}{10.270} + \frac{18.29900}{2.(2.18^2+16^2+2.7^2)} = = 1{,}5 + 268 = 269{,}5\ kN$

$M_w = \frac{M_{Ed}.I_{w,y}}{I_y} + V_{Ed}.e = \frac{2487.1{,}3.48{,}6^3/12}{141200} + 762.0{,}105 = 299\ kNm$

$F_{v,Ed} = \sqrt{76{,}2^2 + 269{,}5^2} = 279\ kN \rightarrow \boldsymbol{F_{v,Ed} = 279\ kN}$

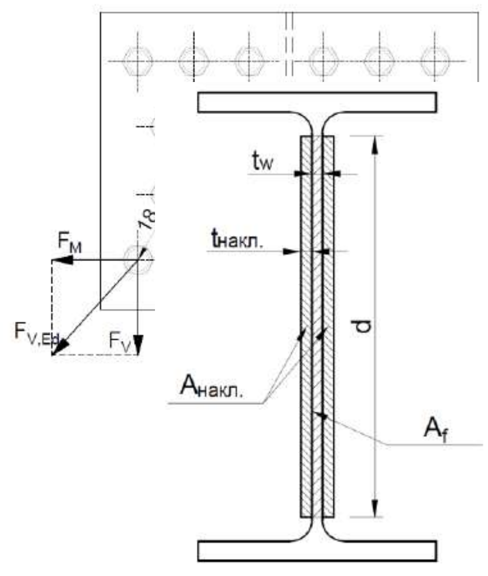
- *<u>Definition of the slice thickness:</u>*

$A_{накл.} \geq 1{,}2.A_w$

$A_{накл.} = 2.d.t_{накл.} = 2.48{,}6.\ t_{накл.} = 97{,}2.\ t_{накл.}$

$A_w = t_w.d = 13.48{,}6 = 63{,}2\ cm^2$

$t_{накл.} \geq 1{,}2.63{,}2/97{,}2 = 0{,}78 \rightarrow \boldsymbol{t_{накл.} = 8\ mm}$



- *Slip resistance:*

$F_{s,Rd} = \frac{k_s.n.\mu}{\gamma_{M3}}.F_{p,c}$,

Where:

$k_s = 1,0$ – for bolts in normal holes;

$n = 2$ – number of the friction planes;

$\mu = 0,5$ - class A of friction surfaces;

$\gamma_{M3} = 1,25$

$F_{p,c} = 0,7.f_{ub}.A_s$ – preloading force for bolts conforming with controlled tightening in conformity;

$A_{s,M30} = 5,61\ cm^2$

$F_{p,c} = 0,7.100.5,61 = 393\ kN$

$F_{s,Rd} = \frac{1,0.2.0,5}{1,25}.393 = 314\ kN \rightarrow \mathbf{F_{s,Rd} = 314\ kN}$

- *Bearing resistance:*

$F_{b,Rd} = \frac{k_1.\alpha_b.f_u.d.t}{\gamma_{M2}}$

$\alpha_b = min \begin{cases} \frac{f_{ub}}{f_u} \\ 1,0 \\ \alpha_d \end{cases}$

For end bolts:

$\alpha_d = \frac{e_1}{3.d_0} = \frac{80}{3.39} = 0,68$

$e_1 = 80\ mm$

$f_u = 510\ MPa = 51\ kN/cm^2$ – or steel class *S355*

$d_0 = d + \Delta = 36 + 3 = 39\ mm$

$\Delta = 3\ mm$ – For bolts M36;

$\alpha_b = min \begin{cases} \frac{100}{60} \\ 1,0 \\ 0,68 \end{cases} = min \begin{cases} 1,67 \\ 1,0 \\ 0,68 \end{cases} \rightarrow \boldsymbol{\alpha_b = 0,68}$

$k_1 = min \begin{cases} 2,8.\frac{e_2}{d_0} - 1,7 \\ 2,5 \end{cases} = min \begin{cases} 2,8.\frac{80}{39} - 1,7 \\ 2,5 \end{cases} = min \begin{cases} 4,0 \\ 2,5 \end{cases} \rightarrow \mathbf{k_1 = 2,5}$

$e_2 = 80\ mm$

$t = t_w = 14\ mm = 1,4\ cm$

$F_{b,Rd} = \frac{0,68.2,5.51.3,0.1,4}{1,25} = 291\ kN$

$F_{Rd} = min\ \{F_{s,Rd}\ ;\ F_{b,Rd}\} = min\ \{314\ kN;\ 291\ kN\} \rightarrow \mathbf{F_{Rd} = 291\ kN > F_{v,Ed} = 279\ kN}$

Total bolts for the connection of webs: 2x10M30, кл. 10.9

2.1.5. Design of the flange splicing

Bolts selected **M36, кл. 10.9**. The necessary number of bolts should be determined.

$F_{Ed} = \frac{N_{Ed}.A_f}{A} + \frac{M_{Ed}-M_w}{h_0} = \frac{64{,}9.30.2{,}5}{270} + \frac{2487-219}{0{,}59} = 3844\ kN \ \rightarrow\ \boldsymbol{F_{Ed} = 3844\ kN}$

➢ *Definition of the splice thickness:*

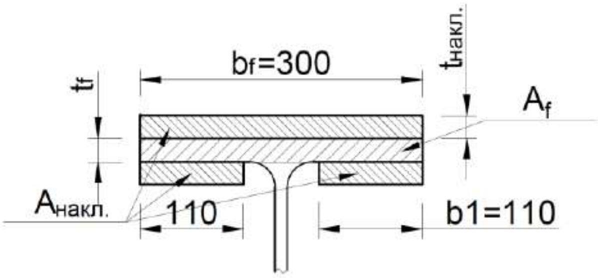
$A_{накл.} \geq 1{,}2.A_f$

$A_{накл.} = (2.b_1 + b_f).t_{накл.} = (2.11 + 30).\ t_{накл.}$

$A_{накл} = 52.\ t_{накл.}$

$A_f = b_f.t_f = 30.2{,}5 = 75\ cm^2$

$\rightarrow t_{накл.} \geq 1{,}2.\ 75/52 = 1{,}73 \rightarrow \boldsymbol{t_{накл.} = 18\ mm}$



➢ *Slip resistance:*

$F_{s,Rd} = \frac{k_s.n.\mu}{\gamma_{M3}}.F_{p,c}$,

Where:

$k_s = 1{,}0$ – for bolts in normal holes;

$n = 2$ – number of the friction planes;

$\mu = 0{,}5$ - class A of friction surfaces;

$\gamma_{M3} = 1{,}25$

$F_{p,c} = 0{,}7.f_{ub}.A_s$ – preloading force for bolts conforming with controlled tightening in conformity;

$F_{p,c} = 0{,}7.100.8{,}17 = 572\ kN$

$F_{s,Rd} = \frac{1{,}0.2.0{,}5}{1{,}25}.572 = 457{,}5\ kN \rightarrow \boldsymbol{F_{s,Rd} = 457{,}5\ \ kN}$

➢ *Bearing resistance:*

$F_{b,Rd} = \frac{k_1.\alpha_b.f_u.d.t}{\gamma_{M2}}$

$\alpha_b = min \begin{cases} \frac{f_{ub}}{f_u} \\ 1{,}0 \\ \alpha_d \end{cases}$

For end bolts:

$\alpha_d = \frac{e_1}{3.d_0} = \frac{70}{3.39} = 0{,}6$

$e_1 = 70\ mm$

$f_u = 510\ MPa = 51\ kN/cm^2$ – for steel class *S355*

$d_0 = d + \Delta = 36 + 3 = 39\ mm$

$\Delta = 3\ mm$ – For bolts M36;

$\alpha_b = min \begin{cases} \frac{100}{60} \\ 1{,}0 \\ 0{,}6 \end{cases} = min \begin{cases} 1{,}67 \\ 1{,}0 \\ 0{,}6 \end{cases} \rightarrow \boldsymbol{\alpha_b = 0{,}6}$

For end bolts:

$k_1 = min \begin{cases} 2{,}8.\frac{e_2}{d_0} - 1{,}7 \\ 2{,}5 \end{cases} = min \begin{cases} 2{,}8.\frac{55}{39} - 1{,}7 \\ 2{,}5 \end{cases} = min \begin{cases} 2{,}25 \\ 2{,}5 \end{cases} \rightarrow \boldsymbol{k_1 = 2{,}25}$

$e_2 = 55\ mm$

$t = t_f = 25\ mm = 2{,}5\ cm$

$F_{b,Rd} = \frac{2{,}25.0{,}6.51.3{,}6.2{,}5}{1{,}25} = 495{,}7\ kN \rightarrow \boldsymbol{F_{b,Rd} = 495{,}7\ kN}$

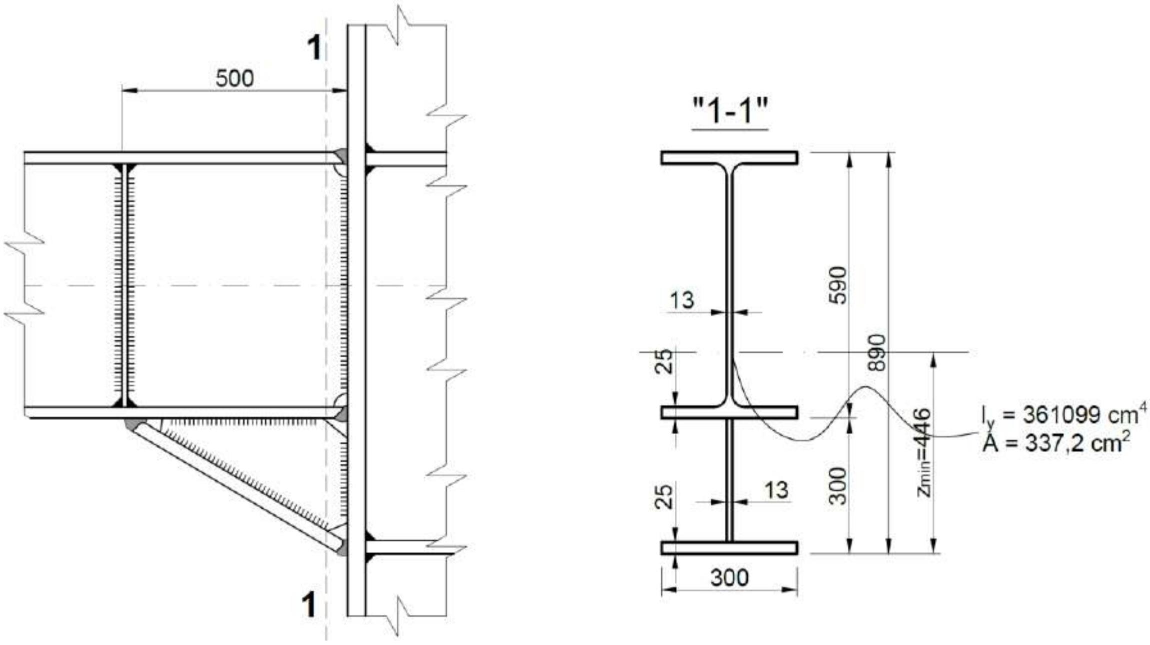
$F_{Rd} = min\ \{F_{s,Rd}\ ;\ F_{b,Rd}\} = min\ \{495{,}7\ kN;\ 475\ kN\} \rightarrow \boldsymbol{F_{Rd} = 475\ kN}$

$n_f = F_{Ed}/2.F_{Rd} = 3844/2.475 \approx 4 \rightarrow$ ***n_f = 4 бр***

Total bolts for the connection of flange: 2x2x4M36, class. 10.9

2.1.6. Resistance of the section of the beam with haunch

The joint is considered rigid and the resistance of section “1-1” for increased value of the force should be checked.



$\boldsymbol{M_{Ed} = 2487\ kNm\ ;\quad V_{Ed} = 762\ kN\ ;\quad N_{Ed} = 64{,}9\ kN}$

$M_{Rd} = W_y.f_{yd} = I_y/z_{min}.f_{yd} = 361099/44{,}6.35{,}5/1{,}05 = 273\ 735\ kNcm \rightarrow \boldsymbol{M_{Rd} = 2737\ kNm}$

$V_{Rd} = A_{vz}.f_{yd}/\sqrt{3} = 115{,}7.35{,}5/(1{,}05.\sqrt{3}) = 2258\ kN \rightarrow \boldsymbol{V_{Rd} = 2258\ kN}$

$A_{vz} \approx t.h = 1{,}3.89 = 115{,}7\ cm^2$

$N_{Rd} = A.f_{yd} = 337{,}2.35{,}5/1{,}05 = 11\ 411\ kN \rightarrow \boldsymbol{N_{Rd} = 11\ 411\ kN}$

Required checks:

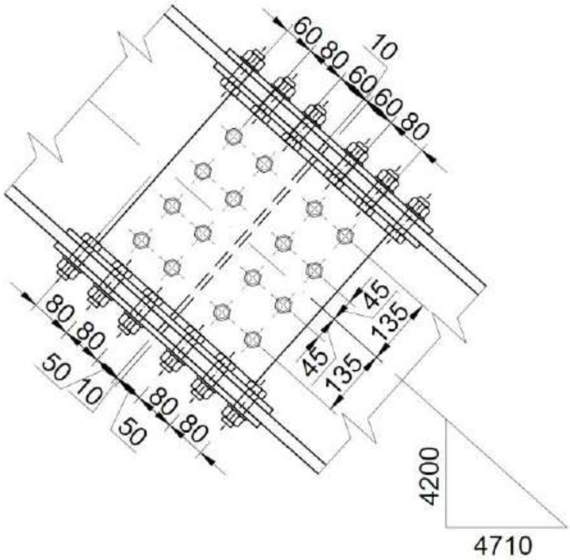
$N_{Ed} \leq 0{,}15.N_{Rd} \rightarrow \boldsymbol{N_{Ed} = 64{,}9\ kN < 0{,}15.11\ 411 = 1\ 711\ kN} \rightarrow$ ***Satisfied!***

$V_{Ed} \leq 0,5.V_{Rd} \rightarrow \boldsymbol{V_{Ed} = 762\ kN < 0,5.2258 = 1129\ kN \rightarrow Satisfied!}$

$M_{Ed} \leq M_{Rd} \rightarrow \boldsymbol{M_{Ed} = 2487\ kNm < M_{Rd} = 2737\ kNm \rightarrow Satisfied!}$

→ The section has the required load bearing capacity!

3. Design of connection with splices of brace of the EBF



Brace at floor 16 from EBF4 is considered - cross section *HE500A*

$R_d \geq R_{Ed,G} + 1,1.\gamma_{ov}.\Omega_i\ .R_{Ed,E}$

$N_{Ed} = N_{Ed,G} + 1,1.\gamma_{ov}.\Omega_{16}\ .N_{Ed,E} = 88 + 1,1.1,25.1,73.1184 \rightarrow \boldsymbol{N_{Ed} = 2818\ kN}$

$M_{Ed} = M_{Ed,G} + 1,1.\gamma_{ov}.\Omega_{16}\ .M_{Ed,E} = 51 + 1,1.1,25.1,73.191 \rightarrow \boldsymbol{M_{Ed} = 505,6\ kN}$

$V_{Ed} = V_{Ed,G} + 1,1.\gamma_{ov}.\Omega_{16}\ .V_{Ed,E} = 18 + 1,1.1,25.1,73.31 \rightarrow \boldsymbol{V_{Ed} = 91,9\ kN}$

4.17. <u>Design of the web splicing</u>

Bolts selected: **8xM24, кл. 10.9** – the number of bolts is calculated for one side of the connection;

- *<u>Shear force in the most loaded bolt:</u>*

$F_{v,Ed} = \sqrt{F_v^2 + F_h^2}$

$F_v = V_{Ed}/n_w = 91,8/8 = 11,5\ kN$

$F_h = \frac{N_{Ed}.A_w}{n_w.A} + \frac{z_{max}.M_w}{\sum z_i^2} = \frac{2818.1,2.39}{8.197,5} + \frac{14.4420}{4.(14^2+6^2)} = 83,45 + 66,7 = 150,2\ kN$

$M_w = \frac{M_{Ed}.I_{w,y}}{I_y} + V_{Ed}.e = \frac{505,6.1,2.39^3/12}{86970} + 91,78\ .0,105 = 44,2\ kNm$

$F_{v,Ed} = \sqrt{11,5^2 + 150,2^2} = 150,6\ kN \rightarrow \boldsymbol{F_{v,Ed} = 150,6\ kN}$

- *<u>Slip resistance:</u>*

$F_{s,Rd} = \frac{k_s.n.\mu}{\gamma_{M3}}.F_{p,c}$,

Where:

$k_s = 1,0$ – for bolts in normal holes;

$n = 2$ – number of the friction planes;

$\mu = 0,5$ - class A of friction surfaces;

$\gamma_{M3} = 1,25$

$F_{p,c} = 0,7.f_{ub}.A_s$ – preloading force for bolts conforming with controlled tightening in conformity;

$A_{s,M24} = 3,53\ cm^2$

$F_{p,c} = 0,7.100.3,53 = 247,1\ kN$

$F_{s,Rd} = \frac{1,0.2.0,5}{1,25}.247,1 = 197,7\ kN \rightarrow \boldsymbol{F_{s,Rd} = 197,7\ kN}$

- *Bearing resistance:*

$$F_{b,Rd} = \frac{k_1.\alpha_b.f_u.d.t}{\gamma_{M2}}$$

$$\alpha_b = min\begin{cases}\frac{f_{ub}}{f_u}\\ 1,0\\ \alpha_d\end{cases}$$

For end bolts:

$\alpha_d = \frac{e_1}{3.d_0} = \frac{60}{3.26} = 0,77$

$e_1 = 60\ mm$

$f_u = 510\ MPa = 51\ kN/cm^2$ – for steel class *S355*

$d_0 = d + \Delta = 24 + 2 = 26\ mm$

$\Delta = 2\ mm$ – For bolts M24;

$$\alpha_b = min\begin{cases}\frac{100}{60}\\ 1,0\\ 0,77\end{cases} = min\begin{cases}1,67\\ 1,0\\ 0,77\end{cases} \rightarrow \boldsymbol{\alpha_b = 0,77}$$

$$k_1 = min\begin{cases}2,8.\frac{e_2}{d_0} - 1,7\\ 2,5\end{cases} = min\begin{cases}2,8.\frac{80}{26} - 1,7\\ 2,5\end{cases} = min\begin{cases}6,9\\ 2,5\end{cases} \rightarrow \boldsymbol{k_1 = 2,5}$$

$e_2 = 80\ mm$

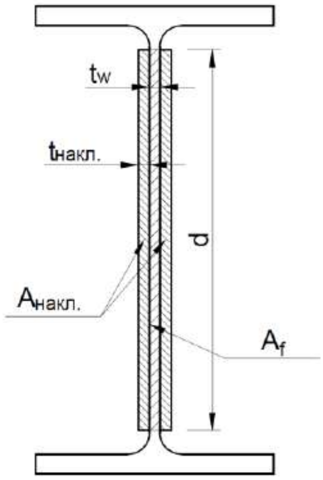
$t = t_w = 12\ mm = 1,2\ cm$

$F_{b,Rd} = \frac{0,77.2,5.51.2,4.1,2}{1,25} = 226\ kN$

$F_{Rd} = min\ \{F_{s,Rd}\ ;\ F_{b,Rd}\} = min\ \{197,7\ kN;\ 226\ kN\} \rightarrow \boldsymbol{F_{Rd} = 197,7\ kN > F_{v,Ed} = 150,6\ kN}$

Total bolts for the connection of webs: 2x8M24, class 10.9

- *Definition of the slice thickness:*



$A_{накл.} \geq 1{,}2.A_w$

$A_{накл.} = 2.d.t_{накл.} = 2.39.\ t_{накл.} = 78.\ t_{накл.}$

$A_w = t_w.d = 1{,}2.39 = 46{,}8\ cm^2$

$t_{накл.} \geq 1{,}2.46{,}8/78 = 0{,}72 \rightarrow \boldsymbol{t_{накл.} = 8\ mm}$

4.18. Design of the flange splicing

Bolts selected: **M30, кл. 10.9**. The necessary number of bolts should be determined.

$$F_{Ed} = \frac{N_{Ed}.A_f}{A} + \frac{M_{Ed}-M_w}{h_0} = \frac{2818.30.2{,}3}{197{,}5} + \frac{505{,}6-44{,}2}{0{,}49} = 1926\ kN \rightarrow \boldsymbol{F_{Ed} = 1926\ kN}$$

- *Slip resistance:*

$$F_{s,Rd} = \frac{k_s.n.\mu}{\gamma_{M3}}.F_{p,c}\ ,$$

Where:

$k_s = 1{,}0$ – for bolts in normal holes;

$n = 2$ – number of the friction planes;

$\mu = 0{,}5$ - class A of friction surfaces;

$\gamma_{M3} = 1{,}25$

$F_{p,c} = 0{,}7.f_{ub}.A_s$ – preloading force for bolts conforming with controlled tightening in conformity;

$F_{p,c} = 0{,}7.100.5{,}61 = 392{,}7\ kN$

$$F_{s,Rd} = \frac{1{,}0.2.0{,}5}{1{,}25}.392{,}7 = 314{,}2\ kN \rightarrow \boldsymbol{F_{s,Rd} = 314{,}2\ kN}$$

- *Bearing resistance:*

$$F_{b,Rd} = \frac{k_1.\alpha_b.f_u.d.t}{\gamma_{M2}}$$

$$\alpha_b = min\begin{cases}\frac{f_{ub}}{f_u}\\ 1{,}0\\ \alpha_d\end{cases}$$

For end bolts:

$$\alpha_d = \frac{e_1}{3.d_0} = \frac{50}{3.33} = 0{,}51$$

$e_1 = 50\ mm$

$f_u = 510\ MPa = 51\ kN/cm^2$ – for steel class *S355*

$d_0 = d + \Delta = 30 + 3 = 33\ mm$

$\Delta = 3\ mm$ – For bolts M30;

$$\alpha_b = min\begin{cases}\frac{100}{60}\\ 1{,}0\\ 0{,}51\end{cases} = min\begin{cases}1{,}67\\ 1{,}0\\ 0{,}51\end{cases} \rightarrow \boldsymbol{\alpha_b = 0{,}51}$$

For end bolts:

$$k_1 = min\begin{cases}2{,}8.\frac{e_2}{d_0} - 1{,}7\\ 2{,}5\end{cases} = min\begin{cases}2{,}8.\frac{50}{33} - 1{,}7\\ 2{,}5\end{cases} = min\begin{cases}2{,}54\\ 2{,}5\end{cases} \rightarrow \boldsymbol{k_1 = 2{,}54}$$

$e_2 = 50\ mm$

$t = t_f = 23\ mm = 2{,}3\ cm$

$F_{b,Rd} = \frac{2{,}5.0{,}51.51.3{,}0.2{,}3}{1{,}25} = 359\ kN \rightarrow \boldsymbol{F_{b,Rd} = 359\ kN}$

$F_{Rd} = min\ \{F_{s,Rd}\ ;\ F_{b,Rd}\} = min\ \{314{,}2\ kN;\ 359\ kN\} \rightarrow \boldsymbol{F_{Rd} = 314{,}2\ kN}$

$n_f = F_{Ed}/2.F_{Rd} = 1926/2.314{,}2 \approx 3 \rightarrow$ ***n_f = 3 бр***

Total bolts for the connection of flanges: 2x2x3M30, class 10.9

- *Definition of the splice thickness:*

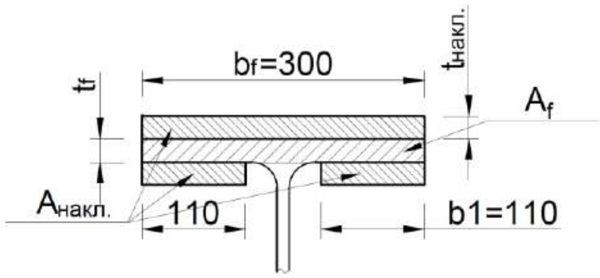
$A_{накл.} \geq 1{,}2.A_f$

$A_{накл.} = (2.b_1 + b_f).t_{накл.} = (2.11 + 30).\ t_{накл.}$

$A_{накл} = 52.\ t_{накл.}$

$A_f = b_f.t_f = 30.2{,}3 = 69\ cm^2$

$\rightarrow t_{накл.} \geq 1{,}2.\ 69/52 = 1{,}59 \rightarrow \boldsymbol{t_{накл.} = 16\ mm}$



4. Design of the flange connection "beam - column„ */Detail „C"/*

The connection between column *C-C3_9* and primary beam *PB115* is considered.

The structure is designed to dissipate energy in the beams, which means that the joint must be designed for increased load-bearing capacity, according to БДС EN 1998-1, item 6.6.4. The joint must be designed as non-dissipative and its elastic behavior must be ensured.

$R_d \geq 1{,}1.\gamma_{ov}.R_{fy}$

$M_{Ed} = 1{,}1.\ \gamma_{ov}.M_{pl,Rd} = 1{,}1.1{,}25.3655 = 5021\ kNm$

$V_{Ed} = V_{Ed,G} + 1{,}1.\ \gamma_{ov}.V_{Ed,M} = 193 + 1{,}1.1{,}25..2M_{pl,Rd}/L_{pl} = 193 + 1{,}1.1{,}25.3655/9{,}90 = 706\ kN$

$N_{Ed} = N_{Ed.G} + 1{,}1.\ \gamma_{ov}.N_{Ed,E} = 10{,}20 + 1{,}1.1{,}25.30 = 51{,}45\ kN$

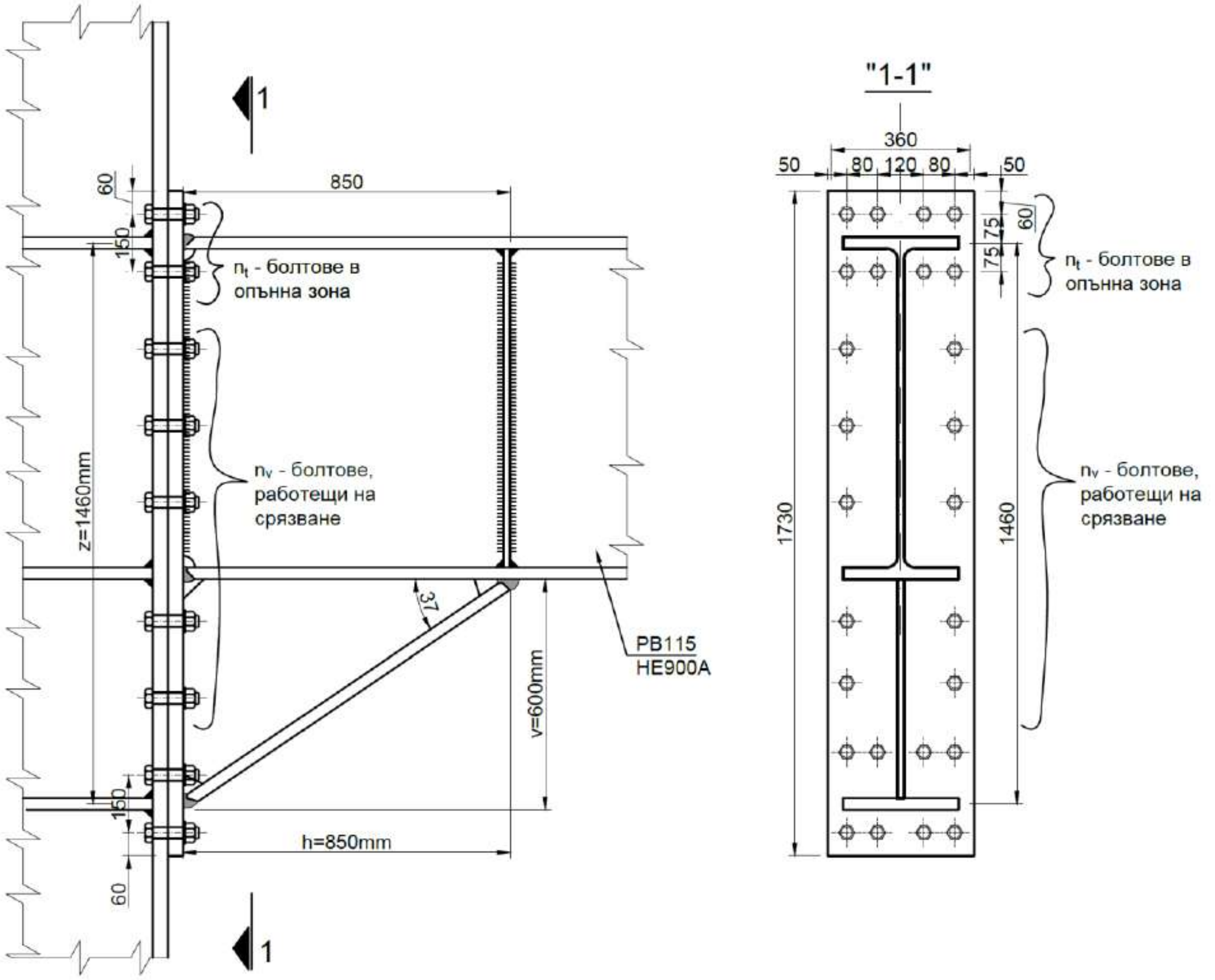
$\boldsymbol{M_{Ed} = 5021\ kNm\ ;\quad V_{Ed} = 706\ kN\ ;\quad N_{Ed} = 51{,}45\ kN}$

4.19. Definition of the number of bolts in the tensile area

Two bolt rows resisting tension are considered. There are 4 bolts M30, class 12,9 in a row

The center of rotation is at the level of the haunches flange. In order to increase the in-between distance of the force couple, as a result of which the tensile force in the bolts is reduced, a haunch with the following dimensions is defined: $h = 850\ mm$, $v = 600\ mm$, $\alpha = 37^o$.

8 bolts are pre-selected to resist the tensile forces in the flange connection. Their location is determined, as well as the distance from the center of gravity of the bolt group to the center of rotation.



Tensile force in one bolt:

$$F_{t,Ed} = \frac{M_{Ed}}{z.n_t} + \frac{N_{Ed}}{n_t} = \frac{5021}{1,46.8} + \frac{51,45}{8} = 436,3\ kN \rightarrow \mathbf{F_{t,Ed} = 436,3\ kN}$$

Tensile resistance of one bolt:

$$F_{t,Rd} = \frac{k_2.f_{ub}.A_s}{\gamma_{M2}},$$

Where:

$f_{ub} = 1200\ MPa = 120\ kN/cm^2$ – tensile strenght of a bolt class 12.9;

$A_s = 5,61\ cm^2$ – net area of bolt M30;

$\gamma_{M2} = 1,25;$

$k_2 = 0,9$

$$F_{t,Rd} = \frac{0,9.120.5,61}{1,25} = 484,7\ kN \rightarrow \mathbf{F_{t,Rd} = 484,7\ kN}$$

Check: $F_{t,Ed} \leq F_{t,Rd} \rightarrow F_{t,Ed} = 436,3\ kN < F_{t,Rd} = 484,7\ kN$ → Check is satisfied!

In tensile area are placed bolts: 2x4M30, кл. 12.9!

4.1.Definition of the number of bolts needed to resist the shear force

Shear resistance of one bolt:

$$F_{v,Rd} = \frac{\alpha_v.f_{ub}.A}{\gamma_{M2}},$$

Where:

$f_{ub} = 1200\ MPa = 120\ kN/cm^2$ – tensile strenght of bolt class 10.9;

$A = 7{,}06\ cm^2$ – gross cross section area of bolt M30;

$\gamma_{M2} = 1{,}25;$

It is assumed that the shear plane passes through the unthreaded portion of the bolt and the area A is the gross cross section of the bolt:

$\alpha_v = 0{,}6$

$F_{v,Rd} = \frac{0{,}6.120.7{,}06}{1{,}25} = 406\ kN \rightarrow \boldsymbol{F_{v,Rd} = 406\ kN}$

Required number of bolts:

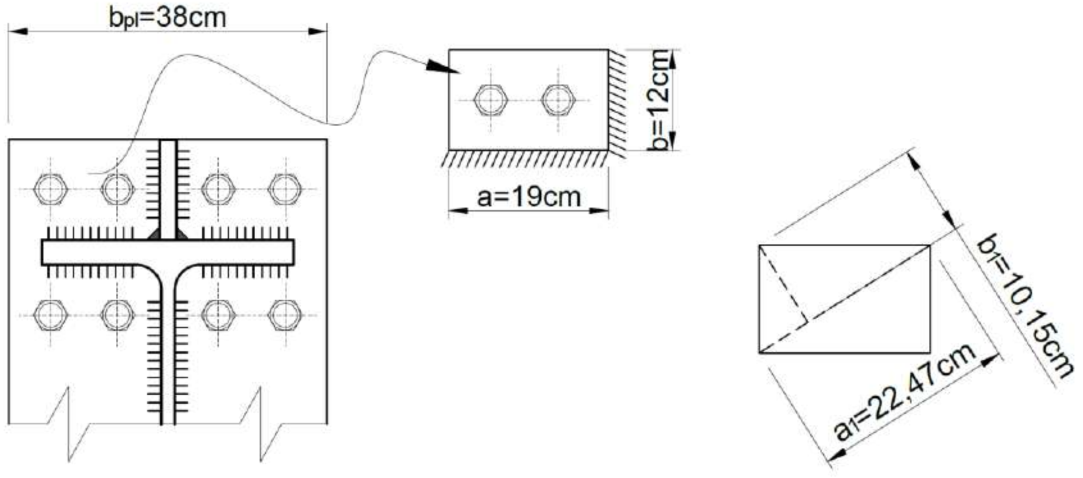
$n_v = F_{Ed}/F_{v,Rd} = 706/406 = 1{,}74 \rightarrow$ ***n***$\boldsymbol{n_v = 2}$ ***бр*** – required number of bolts to resist the shear force;

The bolts provided to resist the shear force will be arranged in two vertical rows, one on each side of the web, with a distance between them close to the maximum allowable for shear joints.

In the shear resistance area are provided bolts: 2x5M30, кл. 12.9!

Thickness of the flange plate is considered: $t = 40\ mm.$

4.20. Check for the thickness of the flange plate

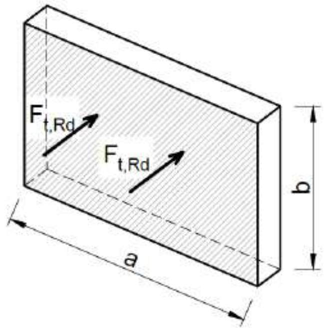


It is assumed that the plate is fully restrained on both sides (in the flange of the beam and in the stiffener). The maximum bending moments are determined using tabular coefficients.

When supporting the field on two adjacent edges for the maximum bending moment, the following formula is used:

$M = \beta.q.a_1^2,$

Where:



$q = 2.F_{t,Rd}/(a.b)$ – stress in the steel plate;

$q = 2.484{,}7/(19.12) = 4{,}25\ kN/cm^2$

$F_{t,Rd} = 484{,}7\ kN$ – tensile resistance or one bolt;

$\beta = f(b_1/a_1)$ – tabular coefficient;

$b_1/a_1 = 10{,}15/22{,}47 = 0{,}47 \approx 0{,}5 \rightarrow \beta = 0{,}06$

$M = 0{,}06.4{,}25.22{,}47^2 = 129\ kN.cm/cm$

$M_{Ed} = M.b_{pl} = 129.38 = 4902\ kNcm \rightarrow \boldsymbol{M_{Ed} = 4902\ kNcm}$

The plastic resistance of the steel plate is defined according to:

$M_{pl,Rd} = W_{pl}.f_{yd} = \frac{b_{pl}.t}{4}.\frac{f_y}{\gamma_{M0}} = \frac{38.4^2}{4}.\frac{34{,}5}{1{,}05} = 5126\ kNcm \rightarrow \boldsymbol{M_{pl,Rd} = 5126\ kNcm}$

M_{Ed} = 4902 kNcm < $M_{pl,Rd}$ =5126 kNcm → The preselected thickness of the plate is enough to resist stresses!

Verified: <u>t_{pl} = 40 mm</u>

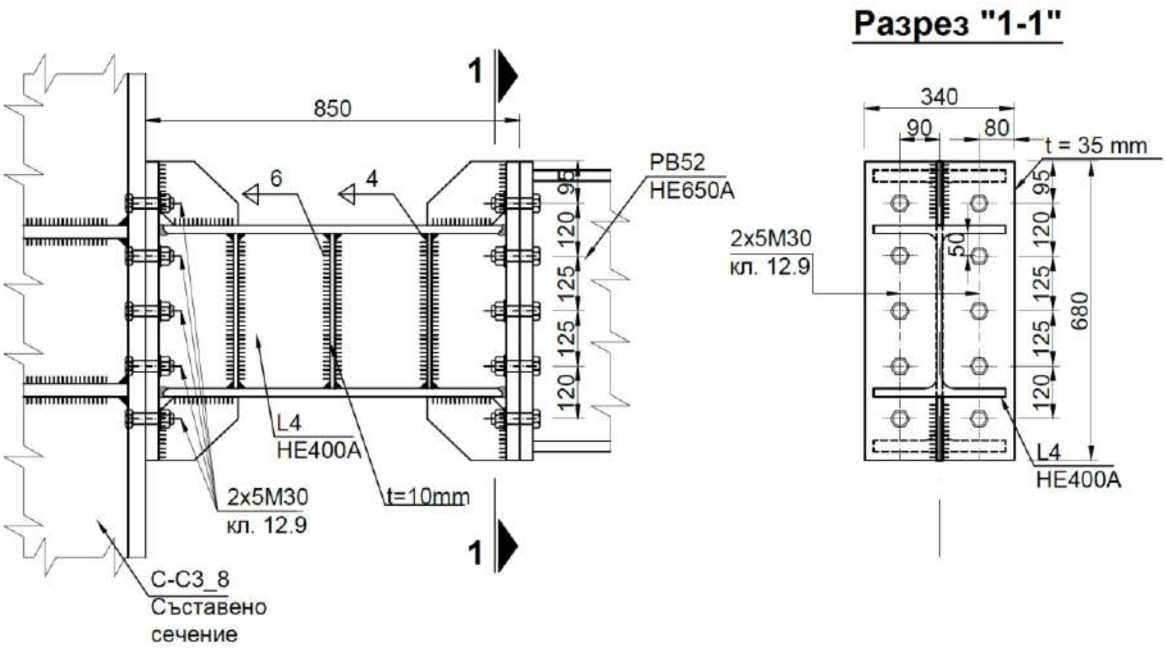
4.21. Check for the column flanges load-bearing resistance

The procedure for determining the bending moments in the flange is similar. The flange is stiffened by the web of the column, as well as by the stiffeners in the column. The designed plate has the same dimensions and static scheme as the flange plate. The flange of the column is thick $t_f = 40\ mm = tpl$.

The calculations for the columns flange are identical and should not be considered in detail.

The flange of the column is able to resist the forces at the ultimate limit state of the bolts from the bolt group.

5. Design of the flange connection between active link element and beam of the EBF /Detail „D"/



The connection between link *L4* и beam *PB52* at the level of floor16.

The structure is designed to dissipate energy in the link elements, which means that the joint must be designed for the following forces, according to БДС EN 1998-1, item 6.8.4:

$E_d \geq E_{d,G} + 1{,}1.\gamma_{ov}.\Omega_i.E_{d,E}$

$M_{Ed} = M_{Ed,G} + 1{,}1.\ \gamma_{ov}.\ \Omega.M_{Ed,E} = 39{,}6 + 1{,}1.1{,}25.1{,}5.352 = 765{,}6\ kNm$

$V_{Ed} = V_{Ed,G} + 1{,}1.\ \gamma_{ov}.\ \Omega.V_{Ed,E} = 31{,}7 + 1{,}1.1{,}25.1{,}5.695 = 1465\ kNm$

$N_{Ed} = N_{Ed,G} + 1{,}1.\ \gamma_{ov}.\ \Omega.N_{Ed,E} = 1{,}1.1{,}25.1{,}5.19 = 39\ kNm$

$\boldsymbol{M_{Ed} = 765{,}6\ kNm}$; $\boldsymbol{V_{Ed} = 1465\ kN}$; $\boldsymbol{N_{Ed} = 39\ kN}$

4.22. Definition of the weld thickness between flange plate and web of the link

Welds between link element and the flange plate should resist the sear force in the link elment. The resistance of the welds at the flanges is not taken into account.

$V_{Ed} \leq F_{w,Rd}$

$F_{w,Rd} = 2.a.l_w.\frac{f_u/\sqrt{3}}{\beta_w.\gamma_{M2}} \rightarrow$ *Welds thickness:* $a \geq \frac{V_{Ed}.\sqrt{3}.\beta_w.\gamma_{M2}}{2.l_w.f_u}$,

Where:

$\beta_w = 0{,}9$ – for steel S355;

$l_w = 298\ mm$ – length of the weld;

$f_u = 51\ kN/cm^2$ – tensile strenght of steel S355;

$a \geq \frac{1465.\sqrt{3}.0{,}9.1{,}25}{2.29{,}8.51} = 0{,}93\ cm \rightarrow \boldsymbol{a = 10\ mm}$

4.23. Definition ot the number of bolts in the tensile area

Two bolt rows resisting tension are considered. There are 2 bolts M30, class 12.9 in a row.

The center of rotation is at the end bolt row in the compression area.

4 bolts are pre-selected to resist the tensile forces in the flange connection. Their location is determined, as well as the distance from the center of gravity of the bolt group to the center of rotation. The link element has cross section HE400A, class S355J2, and the beam is HE650A.

- *Tensile force in one bolt:*

$F_{t,Ed} = \frac{M_{Ed}}{z.n_t} + \frac{N_{Ed}}{n_t} = \frac{765,5}{0,43.4} + \frac{39}{4} = 454,5\ kN \rightarrow \mathbf{F_{t,Ed} = 454,5\ kN}$

- *Tensile resistance of one bolt:*

$F_{t,Rd} = \frac{k_2.f_{ub}.A_s}{\gamma_{M2}},$

Където:

$f_{ub} = 1200\ MPa = 120\ kN/cm^2$ – опънна якост на болт клас 12.9;

$A_s = 5,61\ cm^2$ – нетна площ за болт М30;

$\gamma_{M2} = 1,25;$

$k_2 = 0,9$

$F_{t,Rd} = \frac{0,9.120.5,61}{1,25} = 484,7\ kN \rightarrow \mathbf{F_{t,Rd} = 484,7\ kN}$

Check: $\mathbf{F_{t,Ed} \leq F_{t,Rd} \rightarrow F_{t,Ed} = 454,5\ kN < F_{t,Rd} = 484,7\ kN}$ ***→ Check is satisfied!***

In tensile area are placed bolts: 2x2M30, кл. 12.9!

4.24. Definition of the number of bolts needed to resist the shear force

- *Shear resistance of one bolt:*

$F_{v,Rd} = \frac{\alpha_v.f_{ub}.A}{\gamma_{M2}},$

Където:

$f_{ub} = 1200\ MPa = 120\ kN/cm^2$ – tensile strenght of bolt class 12.9;

$A = 7,06\ cm^2$ – gross cross section area of bolt M30;

$\gamma_{M2} = 1,25;$

It is assumed that the shear plane passes through the unthreaded portion of the bolt and the area A is the gross cross section of the bolt:

$\alpha_v = 0,6$

$F_{v,Rd} = \frac{0,6.120.7,06}{1,25} = 406,6\ kN$

$\rightarrow \mathbf{F_{v,Rd} = 406,6\ kN}$

- *Required number of bolts:*

$n_v = F_{Ed}/F_{v,Rd} = 1465/406 = 3,6 \rightarrow \boldsymbol{n_v = 4}$ ***бр*** – required number of bolts to resist the shear force;

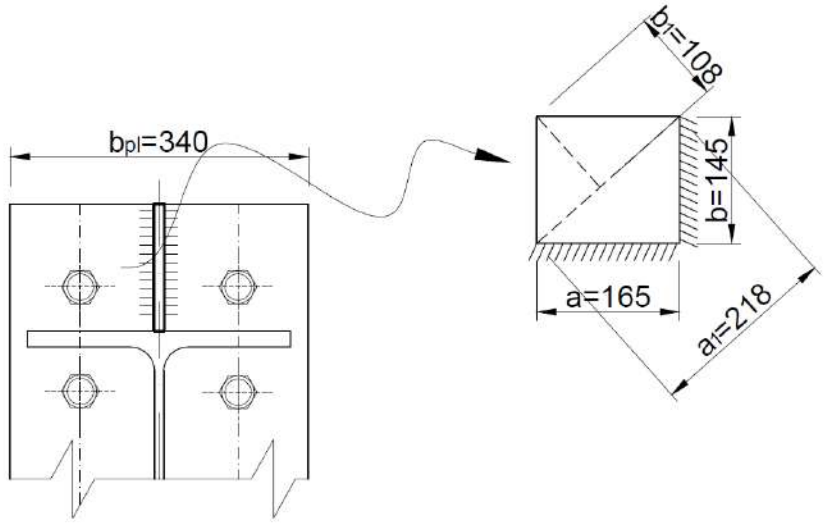
In the shear resistance area are provided bolts: 3x2M30, кл. 12.9!

Thickness of the flange plate is considered: ***t = 35 mm*** *and steel class* ***S355J2****!*

4.25. Check for the thickness of the flange plate

The check is performed for the flange plate, which connects the link to the column, as the section of the flange plate s a field supported on two adjacent edges. In the case of the flange plate, in the connection of the link with the beam of the EBF, a three - sided fully restrained field should be considered, which would be the more favorable option for examination of the bearing capacity of the plate.

It is assumed that the plate is fully restrained on both sides (in the flange of the link and in the stiffener). The maximum bending moments are determined using tabular coefficients.



When supporting the field on three adjacent edges for the maximum bending moment, the following formula is used:

$M = \beta.q.a_1^2$,

Where:

$q = F_{t,Rd}/(a.b)$ – stress in the steel plate;

$q = 484,7/(16,5.14,5) = 2,03\ kN/cm^2$

$F_{t,Rd} = 484,7\ kN$ – tensile resistance or one bolt;

$\beta = f(b_1/a_1)$ – tabular coefficient;

$b_1/a_1 = 10,8/21,8 = 0,5 \rightarrow \beta = 0,06$

$M = 0,06.2,03.21,8^2 = 57,9\ kN.cm/cm$

$M_{Ed} = M.b_{pl} = 57,9.34 = 1968\ kNcm \rightarrow \boldsymbol{M_{Ed} = 1968\ kNcm}$

The plastic resistance of the steel plate is defined according to:

$M_{pl,Rd} = W_{pl}.f_{yd} = \frac{b_{pl}.t}{4}.\frac{f_y}{\gamma_{M0}} = \frac{34.3,5^2}{4}.\frac{34,5}{1,05} = 3421\ kNcm \rightarrow \boldsymbol{M_{pl,Rd} = 3421\ kNcm}$

M_{Ed} = 1968 kNcm < $M_{pl,Rd}$ = 3421 kNcm → The preselected thickness of the plate is enough to resist stresses!

Verified: <u>t_{pl} = 35 mm</u>

6. Design of the flange connection between active link element and beam of the EBF from floor level 2 to 5

The heavily loaded connection between *L4* ang *PB52* at floor 2. is designed.

According to БДС EN 1998-1, т. 6.8.4, where the structure is designed to dissipate energy in the active links, the joints of the connecting elements must be designed for the effects calculated as follows:

$E_d \geq E_{d,G} + 1,1.\gamma_{ov}.\Omega_i.E_{d,E}$

$M_{Ed} = M_{Ed,G} + 1,1.\ \gamma_{ov}.\ \Omega.M_{Ed,E} = 162 + 1,1.1,25.1,5.649 = 1499\ kNm$

$V_{Ed} = V_{Ed,G} + 1,1.\ \gamma_{ov}.\ \Omega.V_{Ed,E} = 191 + 1,1.1,25.1,5.1341 = 2953,5\ kNm$

$N_{Ed} = N_{Ed,G} + 1,1.\ \gamma_{ov}.\ \Omega.N_{Ed,E} = 1,1.1,25.1,5.248 = 510,9\ kNm$

M_{Ed} = 1499 kNm ; V_{Ed} = 2953,5 kN ; N_{Ed} = 510,9 kN

4.26. *<u>Defining the thickness of the weld between flange plate and web of the link</u>*

Welds between link element and the flange plate should resist the sear force in the link elment. The resistance of the welds at the flanges is not taken into account.

$V_{Ed} \leq F_{w,Rd}$

$F_{w,Rd} = 2.a.l_w.\frac{f_u/\sqrt{3}}{\beta_w.\gamma_{M2}}$ → *Welds thickness:* $a \geq \frac{V_{Ed}.\sqrt{3}.\beta_w.\gamma_{M2}}{2.l_w.f_u}$,

Where:

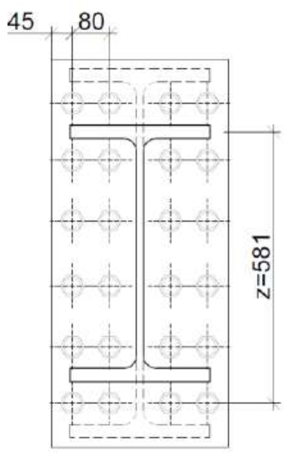
$\beta_w = 0,9$ – for steel S355;

$l_w = 419\ mm$ – length of the weld;

$f_u = 51\ kN/cm^2$ – tensile strenght of steel S355;

$a \geq \frac{2953,5.\sqrt{3}.0,9.1,25}{2.41,9.51} = 1,35\ cm$ → ***a = 14 mm***

4.27. *<u>Definition of the number of bolts in the tensile area</u>*



Two bolt rows resisting tension are considered. There are 4 bolts M30, class 12.9 in a row.

The center of rotation is at the end bolt row in the compression area.

8 bolts are pre-selected to resist the tensile forces in the flange connection. Their location is determined, as well as the distance from the center of gravity of the bolt group to the center of rotation. The link element has cross section HE550B, class S355J2, and the beam is HE800A.

- *Tensile force in one bolt:*

$F_{t,Ed} = \frac{M_{Ed}}{z.n_t} + \frac{N_{Ed}}{n_t} = \frac{1499}{0,5815.8} + \frac{510,9}{8} = 386\ kN \rightarrow \boldsymbol{F_{t,Ed} = 386\ kN}$

- *Tensile resistance of one bolt:*

$F_{t,Rd} = \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$,

Where:

$f_{ub} = 1200\ MPa = 120\ kN/cm^2$ – tensile strength of bolt class 12.9;

$A_s = 5,61\ cm^2$ – net area of bolt M30;

$\gamma_{M2} = 1,25$;

$k_2 = 0,9$

$F_{t,Rd} = \frac{0,9.120.5,61}{1,25} = 484,7\ kN \rightarrow \boldsymbol{F_{t,Rd} = 484,7\ kN}$

Check: $\boldsymbol{F_{t,Ed} \leq F_{t,Rd} \rightarrow F_{t,Ed} = 386\ kN < F_{t,Rd} = 484,7\ kN}$ ***→ Check is satisfied!***

In tensile area are placed bolts: 2x4M30, кл. 12.9!

6.1. Definition of the number of bolts needed to resist the shear force

- *Shear resistance of one bolt:*

$F_{v,Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A}{\gamma_{M2}}$,

Where

$f_{ub} = 1200\ MPa = 120\ kN/cm^2$ – tensile strenght of bolt class 12.9;

$A = 7,06\ cm^2$ – gross cross section area of bolt M30;

$\gamma_{M2} = 1,25$;

It is assumed that the shear plane passes through the unthreaded portion of the bolt and the area A is the gross cross section of the bolt:

$\alpha_v = 0,6$

$F_{v,Rd} = \frac{0,6.120.7,06}{1,25} = 406,6\ kN \rightarrow \boldsymbol{F_{v,Rd} = 406,6\ kN}$

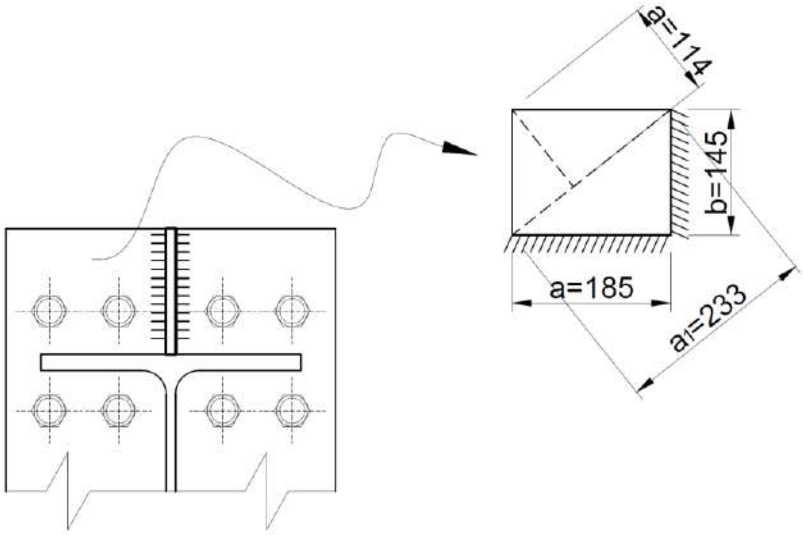
- *Required number of bolts:*

$n_v = F_{Ed}/F_{v,Rd} = 2953,5/406 = 7,3 \rightarrow$ ***n_v = 8 бр*** – required number of bolts to resist the shear force;

In the shear resistance area are provided bolts: 2x4M30, кл. 12.9!

Thickness of the flange plate is considered: $t = 40\ mm$.

4.28. Check for the thickness of the flange plate



The check is performed for the unfavorable connection, which is the one at the columns flange.

It is assumed that the plate is fully restrained on both sides (in the flange of the link and in the stiffener). The maximum bending moments are determined using tabular coefficients.

When supporting the field on two adjacent edges for the maximum bending moment, the following formula is used:

$M = \beta.q.a_1^2,$

Където:

$q = 2.F_{t,Rd}/(a.b)$ – stress in the steel plate;

$q = 2.484,7/(18,5.14,5) = 3,61\ kN/cm^2$

$F_{t,Rd} = 484,7\ kN$ – tensile resistance or one bolt;

$\beta = f(b_1/a_1)$ – tabular coefficient;

$b_1/a_1 = 11,4/23,3 = 0,5 \rightarrow \beta = 0,06$

$M = 0,06.3,61.23,3^2 = 117,6\ kN.cm/cm$

$M_{Ed} = M.b_{pl} = 117,6.38 = 4469\ kNcm \rightarrow$ $\mathbf{M_{Ed} = 4469\ kNcm}$

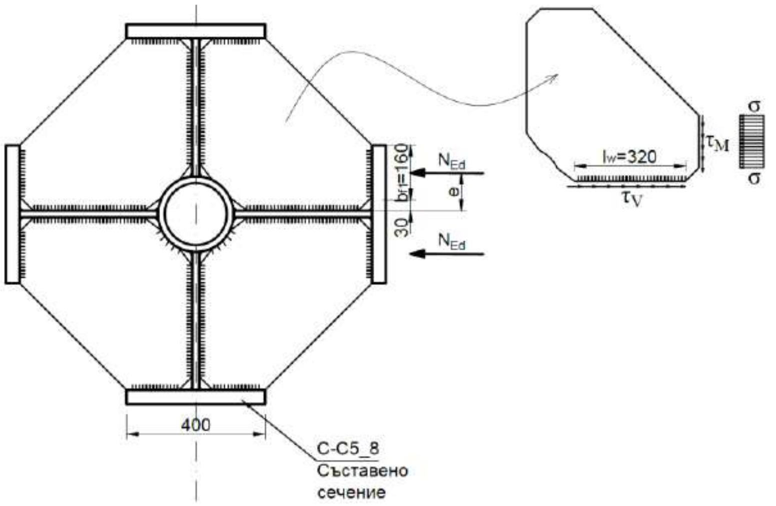
The plastic resistance of the steel plate is defined according to:

$M_{pl,Rd} = W_{pl}.f_{yd} = \frac{b_{pl}.t}{4}.\frac{f_y}{\gamma_{M0}} = \frac{38.4^2}{4}.\frac{34,5}{1,05} = 5139\ kNcm \rightarrow$ $\mathbf{M_{pl,Rd} = 5139\ kNcm}$

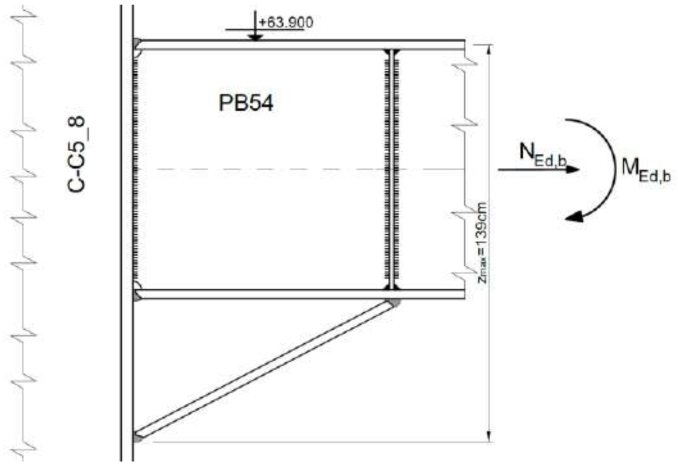
M_{Ed} = 4469 kNcm < $M_{pl,Rd}$ = 5139 kNcm → The preselected thickness of the plate is enough to resist stresses!

Verified: ***t_{pl} = 40 mm***

7. Welds at the column stiffeners



The calculations were performed for column C-C5_8. The stresses in the weld are also caused by the bending moment and the axial force in the beam. Assistance from the column and other welds is not taken into account. It is assumed that the stiffener is cantilevered and relies only on the examined weld, which is to increase safety.



Inner forces in the beam:

$M_{Ed,b} = 3460\ kNm$; $N_{Ed} = 67\ kN$

Weld stresses should not exceed the allowed ones:

$\boldsymbol{\tau_{max} \leq f_{vw,d}}$

$\tau_{max} = \sqrt{\tau_V^2 + \tau_M^2}$,

Where:

$\tau_V = \frac{N_{Ed}}{A_w}$; $\tau_M = \frac{M_{Ed}}{W_w}$

$N_{Ed} = \frac{N_{M,f}}{2} + \frac{N_{Ed,b}}{2} = \frac{2489}{2} + \frac{67}{2} = 1278\ kN \rightarrow \boldsymbol{N_{Ed} = 1278\ kN}$

$M_{Ed} = N_{Ed}.e = 1278.0,11 = 140,6\ kNm \rightarrow \boldsymbol{M_{Ed} = 140,6\ kNm}$

$e = 30 + b_{f1}/2 = 30 + 160/2 = 110\ mm = 0,11m$

$N_{M,f} = M_{Ed,b}/z_{max} = 3460/1,39 = 2489\ kN$

$A_w = 2.a.l_w = 2.0,8.32 = 51,2\ cm^2$; $W_w = \frac{2.a.l_w^2}{6} = \frac{2.0,8.32^2}{6} = 273\ cm^3$

$l_w = 32\ cm$ – length of the weld;

$a = 8\ cm$ – thickness of the weld;

$f_{vw,d} = \frac{f_u}{\beta.\sqrt{3}.\gamma_{M2}} = \frac{51}{0,9.\sqrt{3}.1,25} = 26,17\ kN \rightarrow \boldsymbol{f_{vw,d} = 26,17\ kN}$

$\tau_V = \frac{N_{Ed}}{A_w} = \frac{1278}{51,2} = 25\ kN/cm^2$

$\tau_M = \frac{M_{Ed}}{W_w} = \frac{140,6}{273} = 0,51\ kN/cm^2$

$\tau_{max} = \sqrt{\tau_V^2 + \tau_M^2} = \sqrt{25^2 + 0,51^2} = 25\ kN/cm^2 \rightarrow \boldsymbol{\tau_{max} = 25\ kN/cm^2}$

$\tau_{max} \leq f_{vw,d} \rightarrow \tau_{max} = 25\ kN/cm^2 < f_{vw,d} = 26,17\ kN \rightarrow$ Weld with thickness a = 8 mm and k ≈ 12 mm is verified!

VIII. Foundation

1. Geotechnical characteristics of the ground

The terrain, planned for construction of the building, is located in Sofia, Mladost district. Geological examinations have been made, the results of which have been used for the purposes of this thesis. They are presented in fig. 1 and table 1.

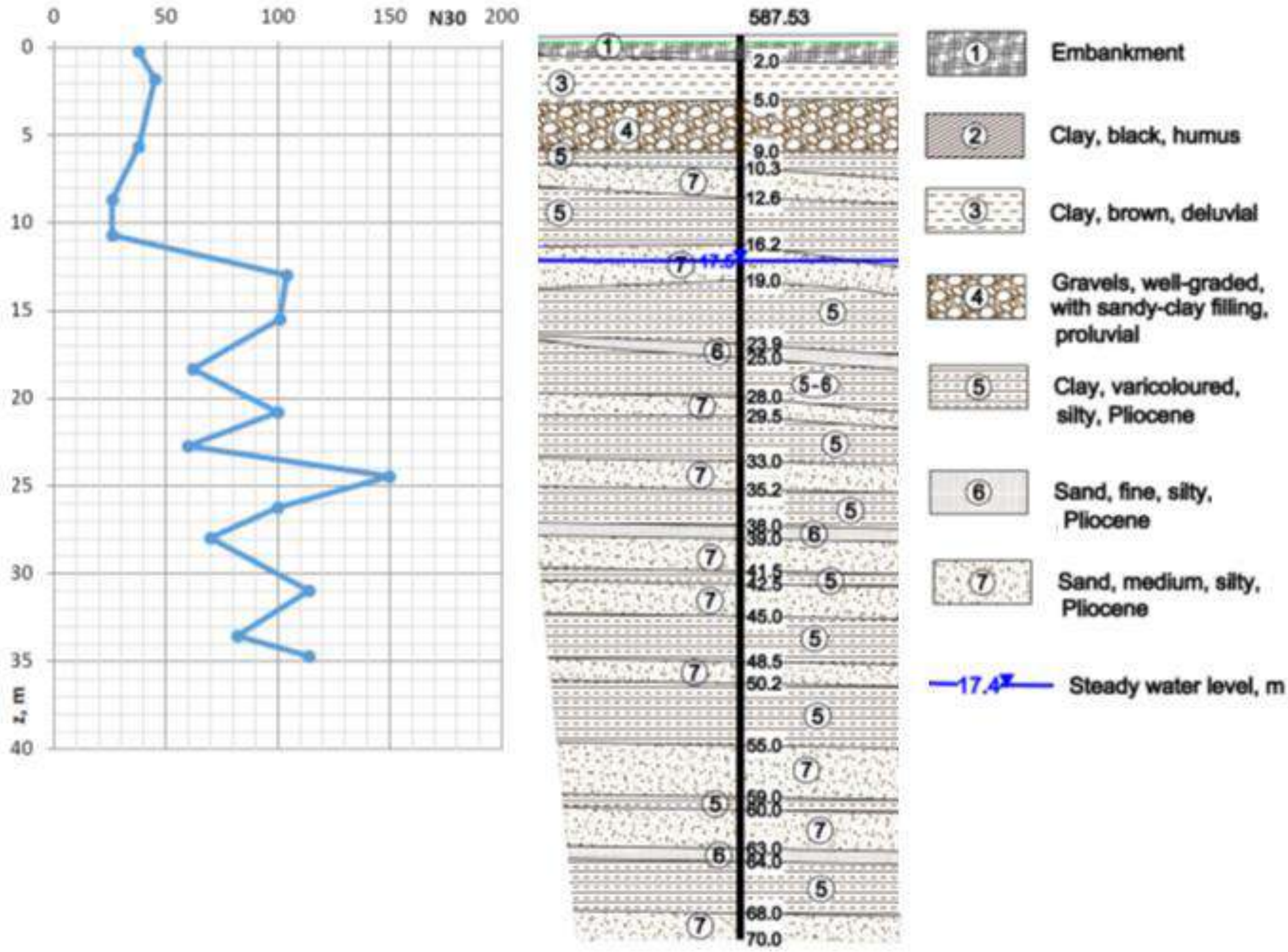


Figure 1. Geological profile, results of SPT

<u>*Table 1*</u>

Пласт	Класификация	γ_n	w_n	e	φ	c	I_C	C_c	E_0	N_{30}
#	-	kN/m³	%	-	deg °	kPa	-	-	Mpa	-
1	Topsoil	18.54	–	–	8.5°	50.2	–	–	5.9	6
2	clSi, Cl	17.76	29.07	0.890	16.5°	42.51	0.88	0.200	10.3	11
3	clSi, siCl	18.64	24.72	0.770	15.0°	72.13	0.92	0.160	10.8	11
4	Grsa, Grsasi	23.60	–	–	37.2°	–	–	–	25.5	> 70
5	clSi, saSi	18.05	25.54	0.830	24.6°	45.2	0.91	0.140	34.6	35
6	siSa	18.44	26.83	0.790	32.7°	27.5	0.48	0.150	-	–
7	siSa, saSi	19.62	16.92	0.550	39.9°	28.0	1.18	0.080	14.4	48

2. Introduction and geometry of the foundation structure

The building has 3 underground levels, reaching an elevation of - *13.40 m*. The basement walls are *60 cm* thick.

For the high (37-storey)building, a pile foundation is performed with the help of reinforced concrete pouring piles, *27.5 m* long and d = 1.50 m in diameter, united by a foundation slab (rostwerk) with thickness $h_{f,1} = 3,0\ m$. In plan, the piles are arranged in groups around the steps of the columns, and in the axis of each column a pile is provided. The axial distance between neighboring piles is $3.d_{пилот} = 450cm$, according to "Standards for design of pile foundations", 1992.

The foundation structure of the low (7-storey) part of the building is a smooth foundation slab, with a thickness of $h_{f,2} = 1,0m$. The foundation slabs are made at the same level with reference to their upper edge, without a structural gap between them, and thus form a general foundation slab with variable thickness. The basement floors and the foundation slab form a rigid box-like structure with a large surrounding surface of interaction with the ground base.

When determining the geometry of the foundation slab, it is taken into account that along its contour, on the sides of the main axes and the basement walls, cantilever parts with a length of *2 m* are provided. The dimensions of the base plane of the foundation slab are $L = 122.5\ m$ and $B = 66.6\ m$, and the area is $A = L.B = 122,5.66,6 = 8\ 145,2\ m.^2$

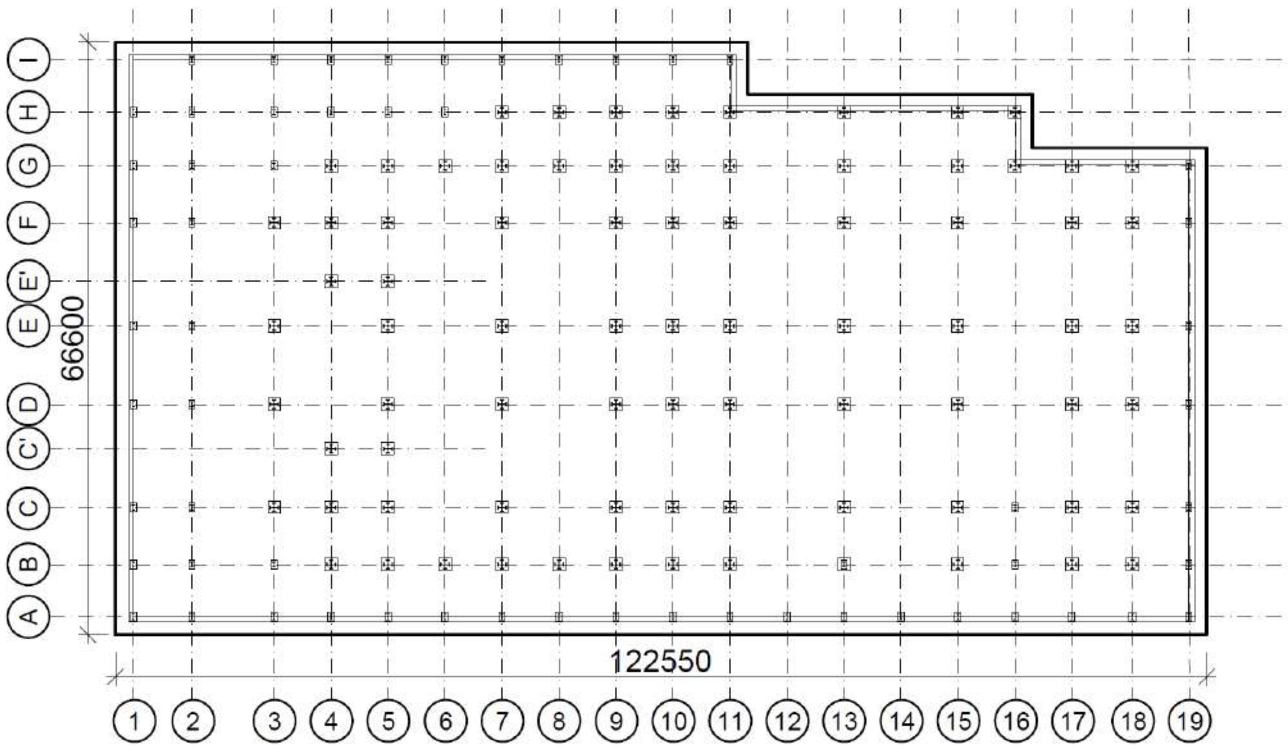


Figure 2. Plan of foundation slab

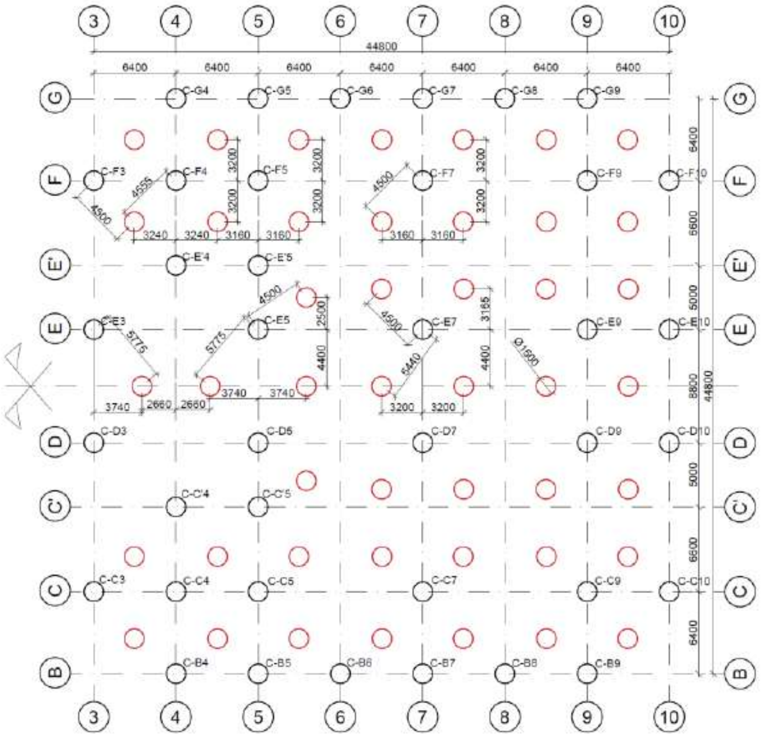


Figure 3. Location of the piles in plan

3. Loads on the foundation structure

3.1. Dead loads

- Self weight foundation slab – from ETABS
- Self weight of concrete flooring: $g_{\text{н}} = t_{\text{н}}.\gamma_{\text{н}} = 0{,}15.25 = 3{,}75\ kN/m^2$

3.2. Live loads

$q_k = 5{,}0\ kN/m^2$ – for category G, according to БДС EN 1991-1-1

3.3. Loads from the top construction

Maximal forces from the columns at elevation – *13,40 m* are transferred to the foundation structure as concentrated forces at the basis of every column. They are presented in *Table 2*.

Table 2

Колона	F_Z [kN]	*Колона*	F_Z [kN]	*Колона*	F_Z [kN]	*Колона*	F_Z [kN]	*Колона*	F_Z [kN]	*Колона*	F_Z [kN]
A1	544	**B5**	12982	**C11**	11017	**E5**	45368	**F18**	8855	**H5**	2076
A2	1775	**B6**	14706	**C13**	13035	**E'5**	26492	**F19**	1593	**H6**	2130
A3	1762	**B7**	15899	**C15**	14429	**E7**	57554	**G1**	1014	**H7**	5917
A4	1106	**B8**	14962	**C17**	8958	**E9**	43505	**G2**	3539	**H8**	6662
A5	1141	**B9**	21997	**C18**	9114	**E10**	22046	**G3**	3823	**H9**	6346
A6	1174	**B10**	8920	**C19**	1588	**E11**	14426	**G4**	24172	**H10**	5198
A7	1175	**B11**	7526	**D1**	1663	**E13**	16370	**G5**	13715	**H11**	6967
A8	1182	**B13**	3831	**D2**	5960	**E15**	19168	**G6**	14478	**H12**	1359
A9	930	**B15**	5069	**D3**	33987	**E17**	13272	**G7**	15153	**H13**	5366
A10	861	**B16**	4289	**D5**	45464	**E18**	6687	**G8**	15007	**H14**	1357
A11	1077	**B17**	4579	**D7**	57727	**E19**	1748	**G9**	25118	**H15**	4223
A12	1229	**B18**	4966	**D9**	43536	**F1**	1488	**G10**	9077	**H16**	2687
A13	1013	**B19**	1062	**D10**	22045	**F2**	5287	**G11**	9029	**I1**	544
A14	1225	**C1**	1487	**D11**	14049	**F3**	22406	**G13**	11561	**I2**	1775
A15	1034	**C2**	5291	**D13**	16102	**F4**	23805	**G15**	8851	**I3**	1773
A16	1051	**C3**	23045	**D15**	19064	**F5**	31693	**G16**	4616	**I4**	980
A17	1039	**C4**	24528	**D17**	13363	**F7**	47817	**G17**	3613	**I5**	987
A18	1049	**C'4**	23259	**D18**	6803	**F9**	36003	**G18**	4875	**I6**	985
A19	555	**C5**	34238	**D19**	1750	**F10**	28169	**G19**	596	**I7**	1032
B1	1014	**C'5**	26851	**E1**	1663	**F11**	13391	**H1**	1005	**I8**	1041
B2	3534	**C7**	48848	**E2**	5959	**F13**	16469	**H2**	3499	**I9**	1036
B3	3840	**C9**	35876	**E3**	33957	**F15**	16377	**H3**	3484	**I10**	1031
B4	24263	**C10**	27974	**E'4**	22876	**F17**	8920	**H4**	2021	**I11**	611

3.4. General loads

3.4.1. Loads at top edge of foundation slab

- *Design*
  $N_{Ed} = 1\ 174\ 977\ kN$
  $V_{Ed,x} = 31\ 262\ kN$
  $V_{Ed,y} = 29\ 496\ kN$
  $M_{Ed,x} = 19\ 890\ 777\ kNm$
  $M_{Ed,y} = 30\ 546\ 034\ kNm$
- *Characteristically*
  $N_{Ek} = 839\ 269\ kN$
  $V_{Ek,x} = 22\ 330\ kN$
  $V_{Ek,y} = 21\ 068\ kN$
  $M_{Ek,x} = 14\ 207\ 698\ kNm$
  $M_{Ek,y} = 21\ 818\ 596\ kNm$

3.4.2. Loads, reduced for the center of the basis plane

- *Design*
  $V_d = 2\ 050\ 934\ kN$
  $H_{L,d} = 31\ 262\ kN$
  $H_{B,d} = 29\ 496\ kN$
  $M_{L,d} = 19\ \ 984\ 563\ kNm$
  $M_{B,d} = 30\ 634\ 522\ kNm$
- *Characteristically*
  $V_k = 1\ 488\ 126\ kN$
  $H_{L,k} = 22\ 330\ kN$
  $H_{B,k} = 21\ 068\ kN$
  $M_{L,k} = 14\ 274\ 688\ kN\ m$
  $M_{B,k} = 21\ 881\ 800\ kNm$

$V_{k(d)} = N_{Ek(d)} + W_{k(d),red}$

$H_{L,k(d)} = V_{L,Ek(d)}\ ;\quad H_{B,k(d)} = V_{B,Ek(d)}$

$M_{L,k(d)} = M_{Ek(d),x} + H_{L,k(d)}.h_f$

$M_{B,k(d)} = M_{Ek(d),y} + H_{B,k(d)}.h_f$

$W_{k,red} = (L+B).b_w.h_w + L.B.h_f$ – loads of bottom structure and foundation slab

$\gamma_{b,k}$ – weight of concrete;

$b_w = 0,60\ m$ – thickness of basement walls;

$h_w = 16,40\ m$ – height of basement walls;

$W_{k,red} = [(122,3+66,6).0,6.13,4 +122,3.66,6.3,0].25 = 648\ 857\ kN$

$W_{d,red} = \gamma_G.\ W_{k,red} = 1,35.\ 648\ 857 = 875\ 957\ kN$

4. Design model of the foundation structure by FEM

For the solution of the foundation structure, a 3D model was defined by the finite element method, using SAP2000 program. It includes planar shell elements working on bending and membrane forces for modeling the foundation slab and basement walls and linear frame elements for modeling the piles (Fig. 4). The interaction with the ground base is approximated by spring supports according to Winkler's theory, respectively area springs for the slabs and discrete springs for the piles. The load consists of the self weight of the elements (reported automatically in the program), concentrated loads in the columns, distributed line loads from the basement walls, distributed area load from the embankment on the foundation slab.

For the foundation structure solution with the maximum forces in the columns from the analysis of the top structure has been made. This approach is applied due to the large distance between the columns and the observance of the requirement for a minimum distance between the piles, which excludes their mutual behavior.

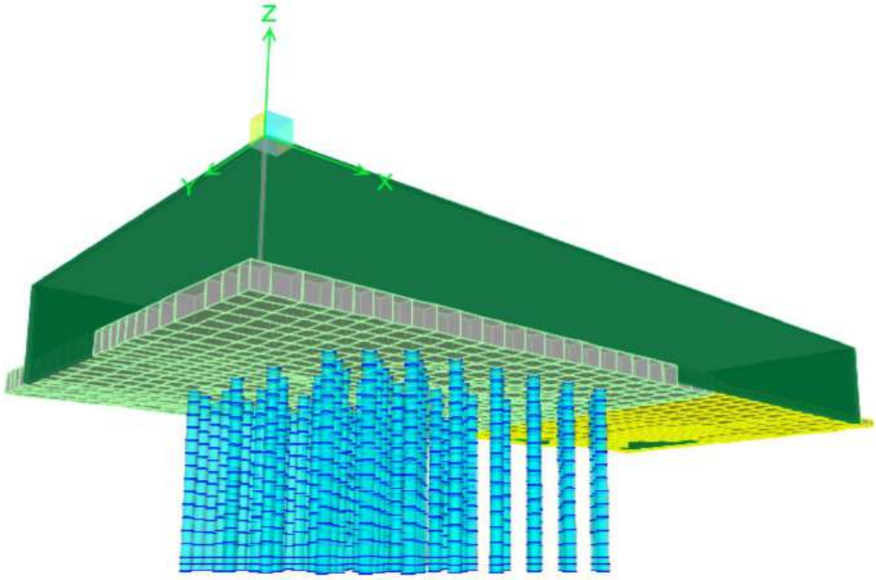


Fig. 4. 3D model of the foundation structure in SAP2000

Area spring for the foundation slab is defined with the value of k_s = *50000kN/m*^{3} (Table 3) in reference to the soil type under it – Paleocene clay. It was found that the bending moments, as well as the stresses in the base, are not significantly influenced by the choice of Winkler constant, as the stiffness of the foundation slab is many times greater than that of the ground base.

Table 3

Range of values of k_s values based on literature review

Soil type	Range of k_s (kN/m^3)
Loose sand	4800–20,000
Medium to dense sand	9600–100,000
Dense sand	50,000–320,100
Clayey medium-dense sand	32,000–80,000
Silty medium-dense sand	24,000–48,000
Dry loose sand	8000–25,000
Dry medium sand	25,000–125,000
Dry dense sand	125,000–375,000
Moist loose sand	10,000–15,000
Moist medium-dense sand	35,000–40,000
Moist dense sand	130,000–150,000
Sandy gravel	200,000–250,000
Dense sandy gravel	100,000–150,000
Fill	10,000–20,000
Soft clay	5000–10,000
Plastic clay	5000–10,000
Stiff clay	15,000
Very stiff clay	>96,000
Rock	>2,000,000

For the piles, discrete spring supports are defined in the directions of the three coordinate axes (Fig. 5). As the piles work for a vertical load with peak resistance and ambient friction, vertical spring supports are placed for these two components, respectively at the top of the pile and in joints *i* along the length of the pile. For horizontal loading, the piles experience passive resistance on the soil side, which is taken into account by horizontal spring supports in the direction of the coordinate axes in a horizontal plane in nodes *i* along the length of the pile.

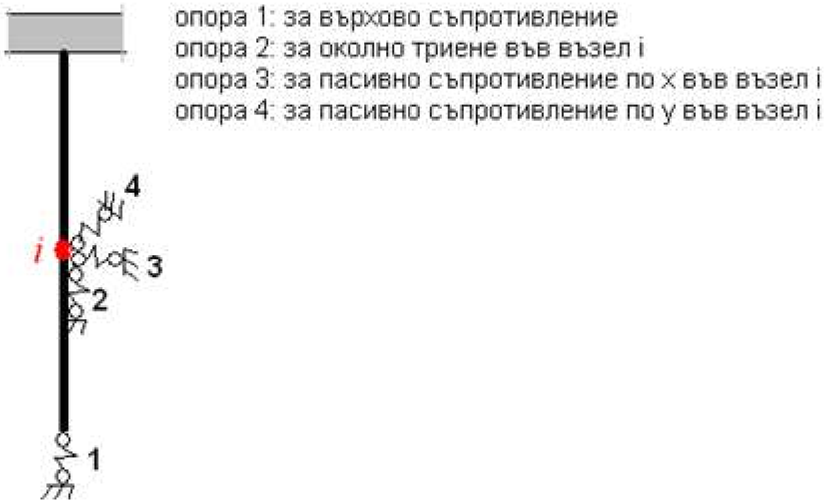


Fig. 5

4.1. Definition of the stiffness $K_{i,h}$ of discrete horizontal spring of the piles (springs type 3 и 4)

In horizontal direction supports of the piles are volume springs with stiffness C_z (kN/m^3). According to the "Standards for the design of pile foundations", 1992, the constant is assumed to increase linearly in depth with a coefficient of proportionality k (kN/m^4), i.e:

$C_{z,i} = k.z_i$

Where:

k – coefficient from Table 4;

z – depth.

For k it is appropriate to assume a constant value along the length of the pile in the range of 6000-7000 kN/m^4, as the ground base has different layers with a similar value of stiffness as a function of physical parameters. The springs are placed at equal distances along the length of the pile in the middle of equal thickness Δh_i.

Table 4. Coefficient of proportionality k(kN/m^4)

Вид на основата и параметри	Забивни пилоти	Изливни пилоти
Глини и песъчливи глини с I_c = 0,0 – 0,25;	650 – 2 500	500 – 2 000
Глини и песъчливи глини с I_c = 0,25 – 0,50; Глинести пясъци с I_c = 0,0 – 1,00; Прахови пясъци с e = 0,60 – 0,80	2 500 – 5 000	2 000 – 4 000
Глини и песъчливи глини с I_c = 0,75 – 1,00; Глинести пясъци с I_c >1,00; Пясъци дребни с e = 0,60 – 0,75; Пясъци среднозърнести с e = 0,55 – 0,70	5 000 – 8 000	4 000 – 8 000
Глини и песъчливи глини с I_c >1,00; Пясъци едри с e = 0,55-0,70	8 000 – 13 000	6 000 – 10 000
Пясъци чакълести с e = 0,55 – 0,70; Чакъл и валуни с пясъчен пълнител	-	10 000 – 20 000

The volume horizontal spring support is approximated by discrete springs at points i along the pile length with stiffness $K_{i,h}$, corresponding to the part of the general volume diagram of the support. For the stiffness $K_{i,h}$, both in the direction of x and in the direction of y, the expression is valid:

$K_{i,h} = C_{z,i} \,.\, \Delta h_i \,.\, d_{пр}$

Where:

$d_{np} = d_{пилот} + 1\ m$

$d_{np} = 1{,}50 + 1{,}0 = 2{,}50\ m$

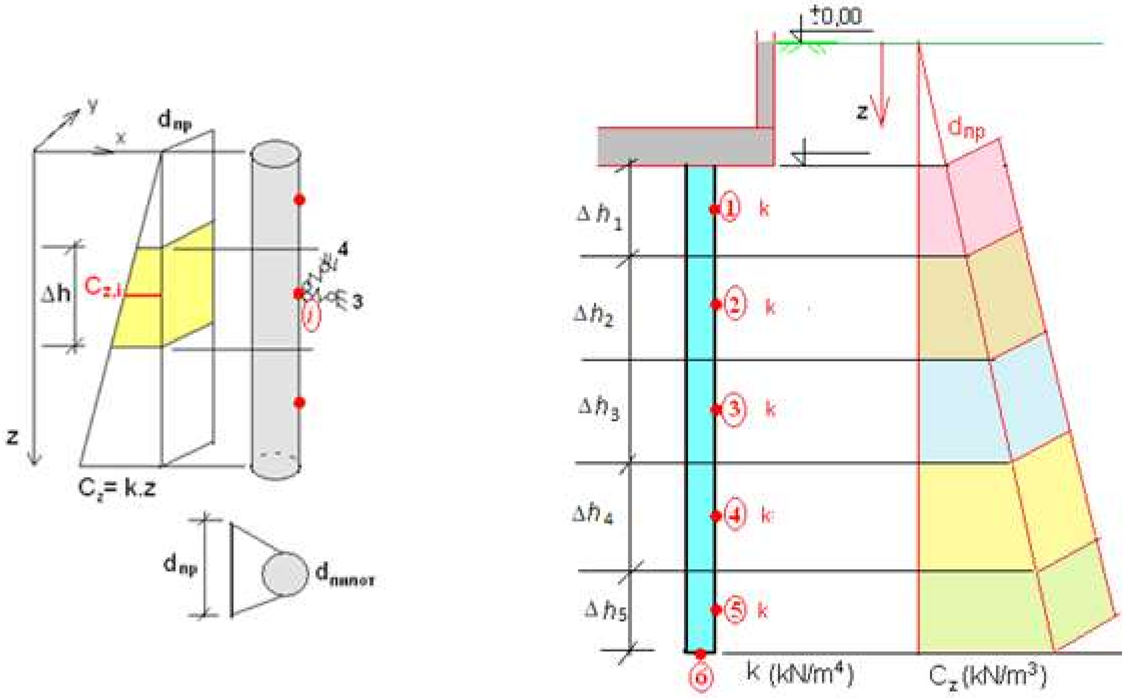


Fig. 6

4.2.*Definition of the stiffness $K_{i,v}$ of discrete vertical friction springs (springs type 2)*

The stiffness of the discrete spring $K_{i,v}$ at point i in the vertical direction, taking into account the surrounding friction, is assumed to be directly proportional to the corresponding horizontal spring $K_{i,h}$, with a coefficient of proportionality *0,4* (the value corresponds to an average coefficient of friction μ between soil and concrete, defined by the formula $\mu = (1/3 - 2/3)\ \varphi$):

$K_{i,v} = 0{,}4\ K_{i,h}$

4.3.*Definition of the stiffness K_b of vertical top resistance spring (spring type 1)*

It is assumed that the volume diagram for vertical support at the top of the pile is with the ordinate $1{,}5\ C_z$. Then for the stiffness K_b we get:

$K_b = 1{,}5\ C_z \cdot A_b$

The stiffness of the pile springs at discrete points is calculated in accordance with the presented methodology and are introduced in the calculation model. They are presented in Table 5.

Table 5

№ точка	Δh i [m]	z i [m]	Cz,i [kN/m 3]	k3=k4 [kN/m]	k2 [kN/m]	k1 [kN/m]
1	3,70	18,25	**109500**	**1012875**	**405150**	-
2	3,70	21,95	**131700**	**1218225**	**487290**	-
3	3,70	25,65	**153900**	**1423575**	**569430**	-
4	3,70	29,35	**176100**	**1628925**	**651570**	-
5	3,70	33,05	**198300**	**1834275**	**733710**	-
6	3,70	36,75	**220500**	**2039625**	**815850**	-
7	3,80	40,5	**243000**	**2308500**	**923400**	-
b	-	42,4	**254400**	-	-	**674343**

5. Results of the FEM analysis of the foundation structure

5.1.Check for load-bearing capacity of the foundation slab under the low building

It is performed according to the Brinch-Hansen formula:

$\gamma_{k,m} = 20{,}51\ kN/m^3$

$\gamma' = 18{,}70\ kN/m^3$

$\varphi_k = \varphi_k = 24{,}6^o$

$c_k = c_d = 45{,}2\ kPa$

$q_d \leq q_{ult,d}$

$q_d = \sum R_i/A_2 + 1{,}35.h_{f,2}.\gamma_b = 489\ 243/2598 + 1{,}35.1{,}0.25 = 222\ kPa$

R_i – sum of the design reactions in columns between axes *10-19* и *A-H*;

$A_2 = 2598\ m^2$ – area of the foundation slab of the low building;

$q_{ult,d} = q_{ult,k} / \gamma_{k,v}$

$\gamma_{k,v} = 1{,}4$

$q_{ult,k} = c.N_c.s_c.i_c + D_f.\gamma_{k,m}.N_q.s_q.i_q + 0{,}5.\gamma_k'.B'.N_\gamma.s_\gamma.i_\gamma$,

Where:

$B' = B = 44{,}8\ m$ – short side of the foundation slab;

D_f – depth of foundation;

c = 45,2 kPa – cohesion;

$\gamma_{k,m} = 20{,}51 kN/m^3$

$\gamma_k' = 8{,}7/m^3$

$\varphi = 24{,}6^o$

5.1.1. Resistance coefficients (N_q, N_c, N_γ)

$N_q = e^{\pi.tg\varphi}.tg^2(45^o + \varphi/2) = e^{\pi.tg24{,}6^o}.tg^2(45^o + 24{,}6^o/2) = 10{,}22$

$N_c = (N_q - 1).cotg\varphi = (10{,}22 - 1).cotg24{,}6^o = 20{,}14$

$N_\gamma = 2.(N_q - 1).tg\varphi = 2.(10{,}22 - 1).tg24{,}6^o = 8{,}44$

5.1.2. Form coefficients (sq, sγ, sc)

B' = B =44,8m, L' =L =58m – dimensions of the foundation slab

$s_q = 1 + \frac{B\prime}{L\prime}.sin\varphi = 1 + \frac{44,8}{58}.sin24,6^o = 1,32$

$s_\gamma = 1 – 0,3. \frac{B\prime}{L\prime}.sin\varphi = 1 - 0,3.\frac{44,8}{58}.sin24,6^o = 0,9$

$s_c = \frac{s_q.N_q-1}{N_q-1} = \frac{1,32.10,22-1}{10,22-1} = 1,35$

5.1.3. Coefficients for taking into account the slope in the load of horizontal force (i_q, i_γ, i_c)

Accepted coefficients $i_q = i_\gamma = i_c = 1$, as the horizontal force is resisted by the box-shaped basement structure, which is expressed in ground pressure and friction on the walls.

$q_{ult,k} = 45,2.20,14.1,35.1 + 16,4.20,51.10,22.1,32.1 + 0,5.8,7.44,8.8,44.0,9.1 = 1229 + 4537 + 1480 = 7246\ kPa$

$q_{ult,d} = 5176\ kPa$

$q_d = 222\ kPa \leq q_{ult,d} = 5176\ kPa$

Check is satisfied!

5.2.Required reinforcement in the foundation slab

5.2.1. Materials

Concrete class C35/45

$f_{ck} = 35\ MPa$

$f_{cd} = f_{ck}/1,5 = 35/1,5 = 23,33\ MPa$

$f_{ctm} = 3,2\ MPa$

$E_{cm} = 34\ 000\ MPa$

Reinforcement class B500B

$f_{yk} = 500\ MPa$

$f_{yd} = f_{yk}/1,15 = 500/1,15 = 435\ MPa$

5.2.2. Definition of the main reinforcement mesh

- For slab with the thickness $h_{f,1} = 3,0\ m$

$A_{s,min} = max\{0,26.(f_{ctm}/f_{yk}).b.d_1; 0,0013.b.d_1\} = max\{0,26.(3,2/500).1000.2935;0,0013.1000.2935\}$

$A_{s,min} = 4\ 884\ mm^2$

$d_1 = h_{f,1} – c_{nom} – \phi/2 = 3000 – 50 – 30/2 = 2\ 935\ mm$

Selected main mesh: Double mesh Φ25/20 with $A_{s,prov} = 4\ 909\ mm^2/m = A_{s,bas}$

$M_{s,bas} = A_{s,bas}.f_{yd}.(d - 0,4.\frac{A_{s,bas}.f_{yd}}{0,8.f_{cd}.b}) = 4\,909.435.(2\,935 - 0,4.\frac{4\,909.435}{0,8.23,33.1000}).10^{-6}$

$\rightarrow$ ***$M_{s,bas,1}$ = 6 256 kN.m/m***

- For slab with thickness: $h_{f,2} = 1,0\ m$

$A_{s,min} = max\{0,26.(f_{ctm}/f_{yk}).b.d_2;\ 0,0013.b.d_2\} = max\{0,26.(3,2/500).1000.935;0,0013.1000.935\}$

$A_{s,min} = 1\,560\ mm^2$

$d_2 = h_{f,2} - c_{nom} - \phi/2 = 1000 - 50 - 30/2 = 935\ mm$

Selected main mesh: ϕ25/20 with $A_{s,prov}$ = 2 454 mm^2/m = $A_{s,bas}$

$M_{s,bas} = A_{s,bas}.f_{yd}.(d - 0,4.\frac{A_{s,bas}.f_{yd}}{0,8.f_{cd}.b}) = 2\,454.435.(935 - 0,4.\frac{2\,454.435}{0,8.23,33.1000}).10^{-6}$

$\rightarrow$ ***$M_{s,bas,2}$ = 975 kN.m/m***

5.2.3. Additional amplifiers

- Average torsional moment:

Average value of the torsional moments in slab is accepted:

M_{12}^{I} ≈ 800 kN.m/m – torsional moment of slab under tall part of the building;

M_{12}^{II} ≈ 300 kN.m/m – torsional moment of slab under lower part of the building;

Amplifiers in foundation slab

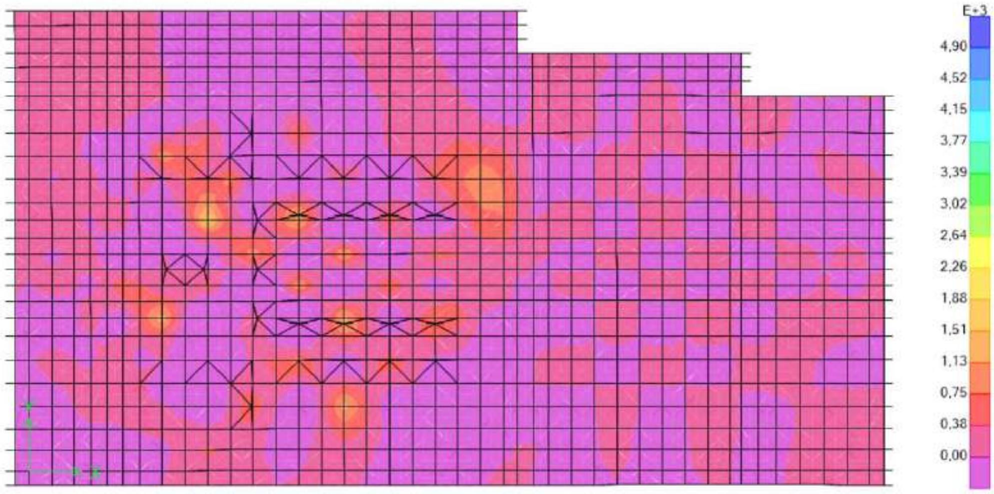


Fig. 7. Positive torsional moments M_{xy}[kN.m/m]

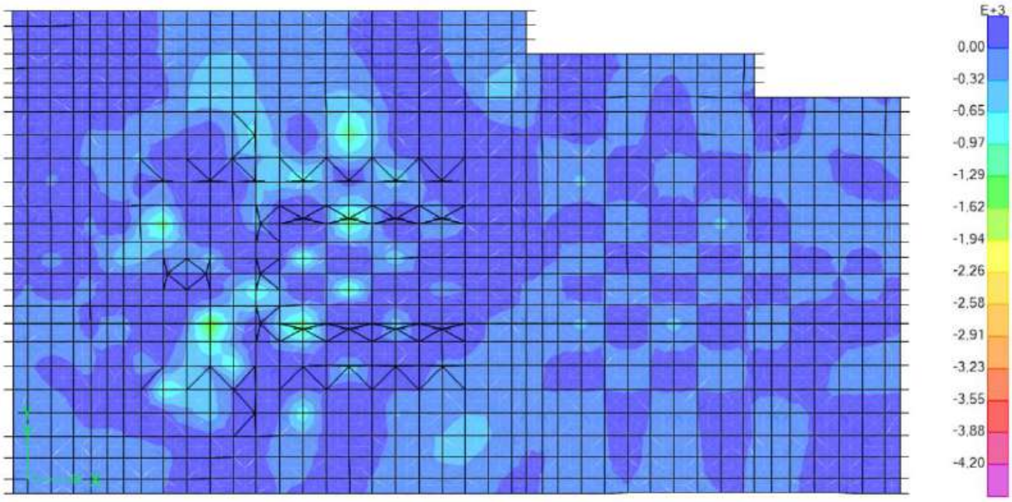


Fig. 8. Negative torsional moment M_{xy} [kN.m/m]

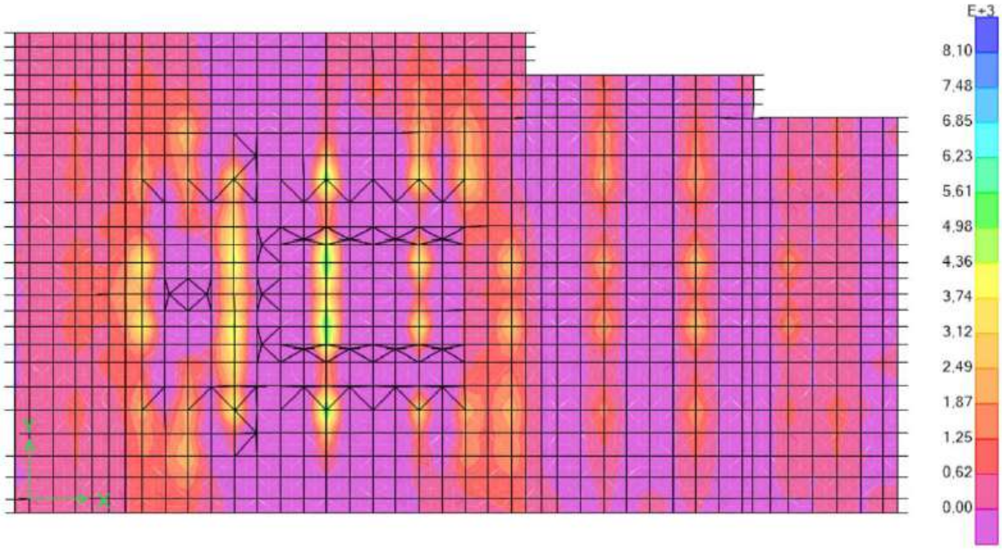


Fig. 9. Positive bending moment Mx[kN.m/m] /tension from below/

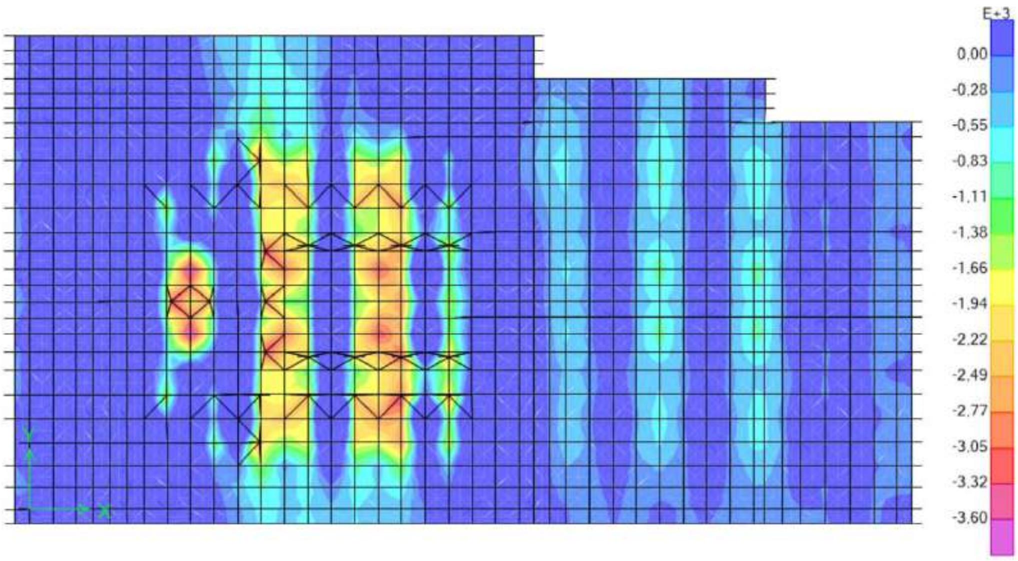


Fig. 10. Negative bending moment Mx[kN.m] / tension on top /

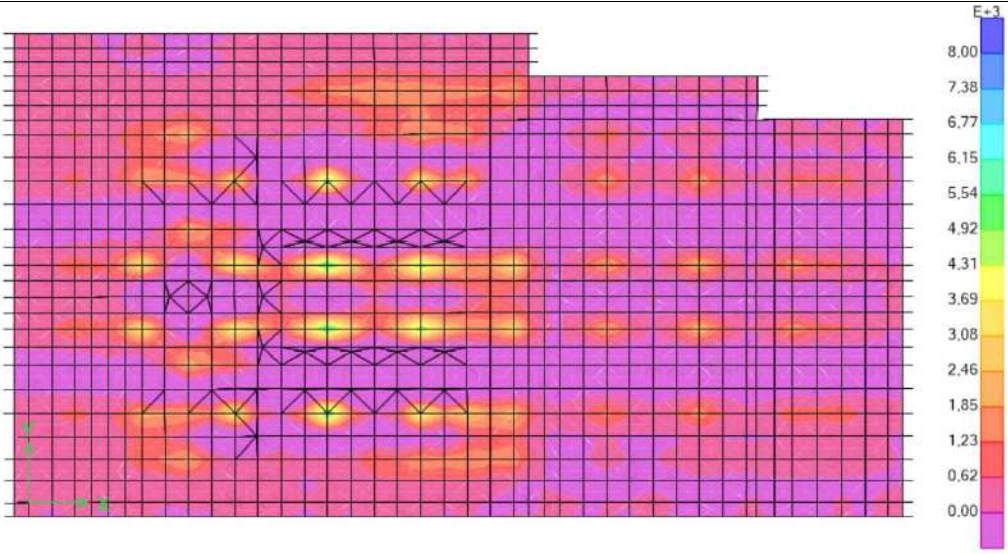
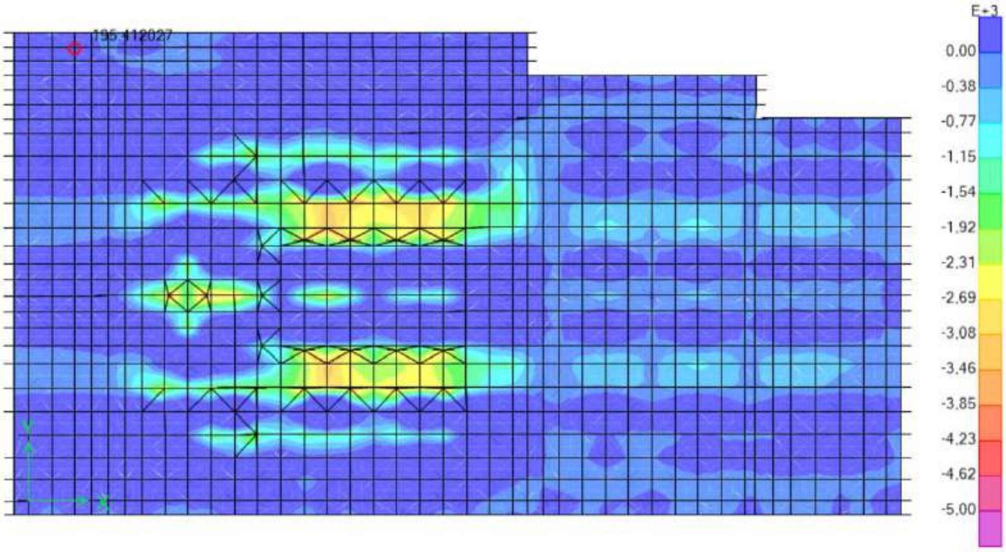


Fig. 11. Positive bending moment My[kN.m] / tension from below /



Фиг. 12. Negative bending moment My[kN.m] / tension on top /

➢ Moment in additional reinforcement (amplifiers):

1) $M_{Ed,adm} = M_{Ed} + |M_{12}| - M_{bas}$

2) $A_{s,adm} = \left(1 - \sqrt{1 - \frac{2.M_{Ed,adm}}{f_{cd}.b.d^2}}\right).\frac{f_{cd}}{f_{yd}}.b.d$ $[mm^2/m]$- required reinforcement;

$d = d_2 = 935\ mm$

3) $A_{s,prov} = N.A_{s1\phi}$

4) $\rho_l = A_{s,prov}/d.b$

- For slab with thickness $h_{f,1} = 3{,}0\ m$

$M_{Ed,lim} = M_{s,bas,1} - |M_{12}^{I}| = 6256 - 800 = 5\ 456\ kNm/m$

$\boldsymbol{M_{Ed,lim} = 5\ 456\ kNm/m}$ → Maximal bending moment, which can be resisted from the main mesh

$M_{Ed,max}$ = 5 400 kNm/m → $\mathbf{M_{Ed,max}}$ **= 5 400 kNm/m <** $\mathbf{M_{Ed,lim}}$ **= 5 456 kNm/m**

→ Additional reinforcement is not needed! Main mesh is enough for resisting the inner forces of the slab!

- For slab with thickness $h_{f,2} = 1,0\ m$

$M_{Ed,lim} = M_{s,bas,2} - |M_{12}^{II}| = 935 – 300 = 635\ kNm/m$

$\mathbf{M_{Ed,lim}}$ ***= 635 kNm/m*** → Maximal bending moment, which can be resisted from the main mesh

Based on the results of the analysis, areas in which amplifiers need to be located are identified. These are the zones in which the bending moments M_{Ed} exceed the value of the maximum bending moment $M_{Ed,lim}$, which can be resisted by the main reinforcing mesh.

The bending moments in the *X* direction ($M_x \equiv M_{11}$) and those in the *Y* direction ($M_y \equiv M_{22}$) are considered. To resist the $M_{Ed,x}$ moments, reinforcement is located along the Y axis (Fig. 13 and Table 6), and the amplifiers needed to cover the $M_{Ed,y}$ are located along the X axis (Fig. 14 and Table 7).

In the areas of the columns, reinforcement is needed to resist the positive moments, ie. lower reinforcement, and for the resulting negative bending moments in the field, upper reinforcement is provided.

The required reinforcement in the form of amplifiers is calculated and presented in tabular form. Amplifiers are defined for all zones given in the following diagrams.

Table 6

ДОПЪЛНИТЕЛНА АРМИРОВКА за Mx (армировка по Y)													
Зона	**M11**	**M12**	**M11+M12**	**Mbas**	**MEd,adm**	**As,bas**	**УСИЛИТЕЛИ**						
							As,adm	**As,prov**	**As1ф**	**ρl**	**Брой**	**ф**	**s**
	kNm/m	kNm/m	kNm/m	kNm/m	kNm/m	mm²/m	mm²/m	mm²/m	mm²/m	…	…	…	cm
					1)		*2)*	*3)*		*4)*			
Отрицателни огъващи моменти - ГА													
1	700	300	1000	975	25	2454	*62*	251	50,3	0,00027	***5***	***8***	20
2	800	300	1100	975	125	2454	*308*	393	78,5	0,00047	***5***	***10***	20
3	700	300	1000	975	25	2454	*62*	251	50,3	0,0003	***5***	***8***	20
Положителни огъващи моменти - ДА													
4	1800	300	2100	975	1125	2454	*2847*	3142	314,2	0,00375	***10***	***20***	10
5	1800	300	2100	975	1125	2454	*2847*	3142	314,2	0,00375	***10***	***20***	10
6	1300	300	1600	975	625	2454	*1561*	1571	314,2	0,00188	***5***	***20***	20
7	1300	300	1600	975	625	2454	*1561*	1571	314,2	0,00188	***5***	***20***	20
8	1600	300	1900	975	925	2454	*2328*	2545	254,5	0,00304	***10***	***18***	10
9	2000	300	2300	975	1325	2454	*3371*	3801	380,1	0,00454	***10***	***22***	10
10	1300	300	1600	975	625	2454	*1561*	1571	314,2	0,00188	***5***	***20***	20
11	1300	300	1600	975	625	2454	*1561*	1571	314,2	0,00188	***5***	***20***	20
12	1500	300	1800	975	825	2454	*2071*	2545	254,5	0,00304	***10***	***18***	10

Reinforcement in Y direction

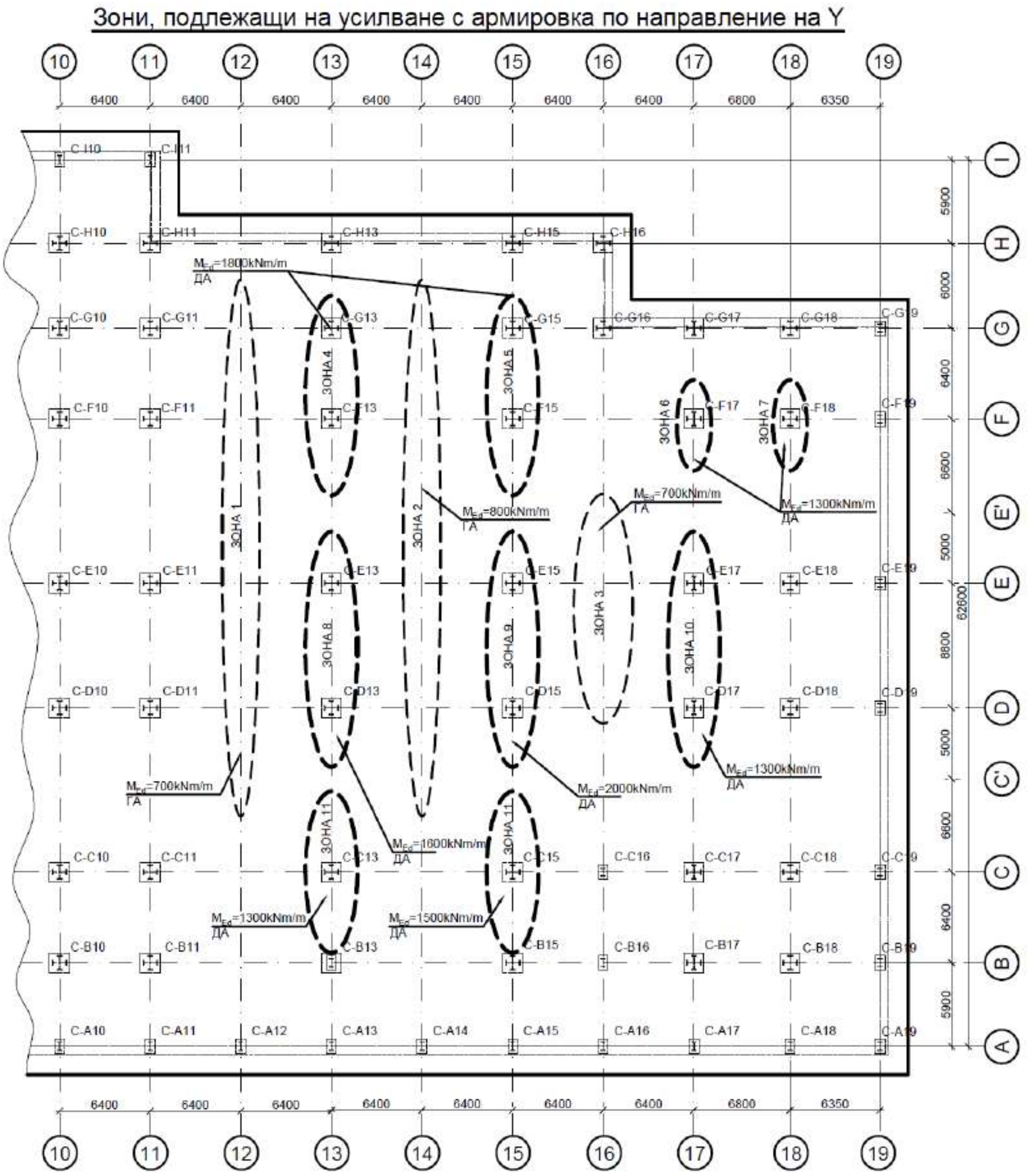


Fig. 13

Table 7

ДОПЪЛНИТЕЛНА АРМИРОВКА за My (армировка по X)													
Зона	**M_{22}**	**M_{12}**	**M_{22}+M_{12}**	**M_{bas}**	**$M_{Ed,adm}$**	**$A_{s,bas}$**	**УСИЛИТЕЛИ**						
							$A_{s,adm}$	**$A_{s,prov}$**	**$A_{s1\phi}$**	**ρ_l**	**Брой**	**ф**	**s**
	kNm/m	kNm/m	kNm/m	kNm/m	kNm/m	mm²/m	mm²/m	mm²/m	mm²/m	…	…	…	cm
					1)		*2)*	*3)*		*4)*			
Отрицателни огъващи моменти - ГА													
1	800	300	1100	975	125	2454	*308*	393	78,5	0,00042	***5***	***10***	20
2	700	300	1000	975	25	2454	*62*	251	50,3	0,0003	***5***	***8***	20
3	800	300	1100	975	125	2454	*308*	393	78,5	0,00047	***5***	***10***	20
4	700	300	1000	975	25	2454	*62*	251	50,3	0,0003	***5***	***8***	20
Положителни огъващи моменти - ДА													
5	800	300	1100	975	125	2454	*308*	393	78,5	0,00047	***5***	***10***	20
6	800	300	1100	975	125	2454	*308*	393	78,5	0,00047	***5***	***10***	20
7	800	300	1100	975	125	2454	*308*	393	78,5	0,00047	***5***	***10***	20
8	800	300	1100	975	125	2454	*308*	393	78,5	0,00047	***5***	***10***	20
9	800	300	1100	975	125	2454	*308*	393	78,5	0,00047	***5***	***10***	20

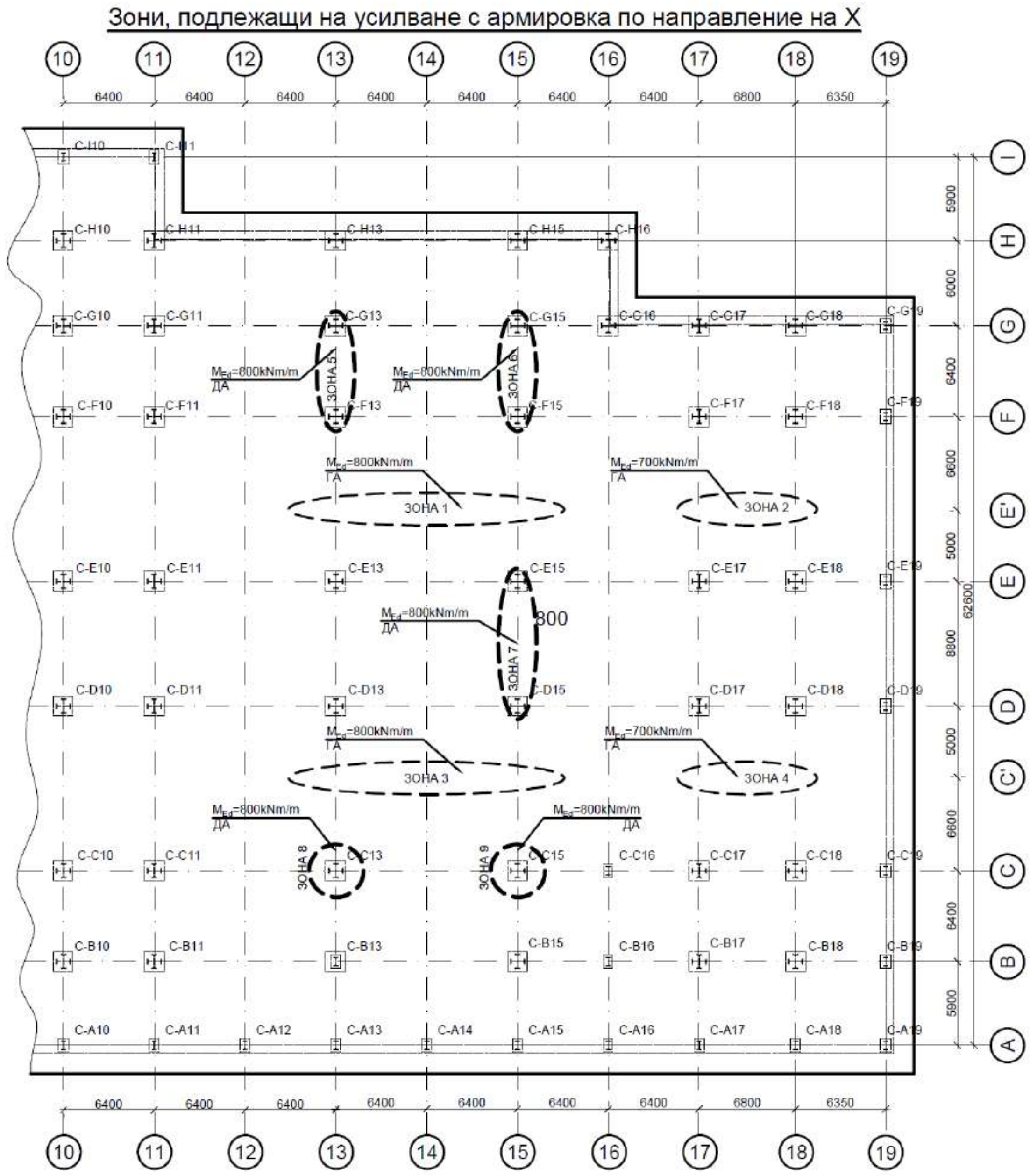


Fig. 14

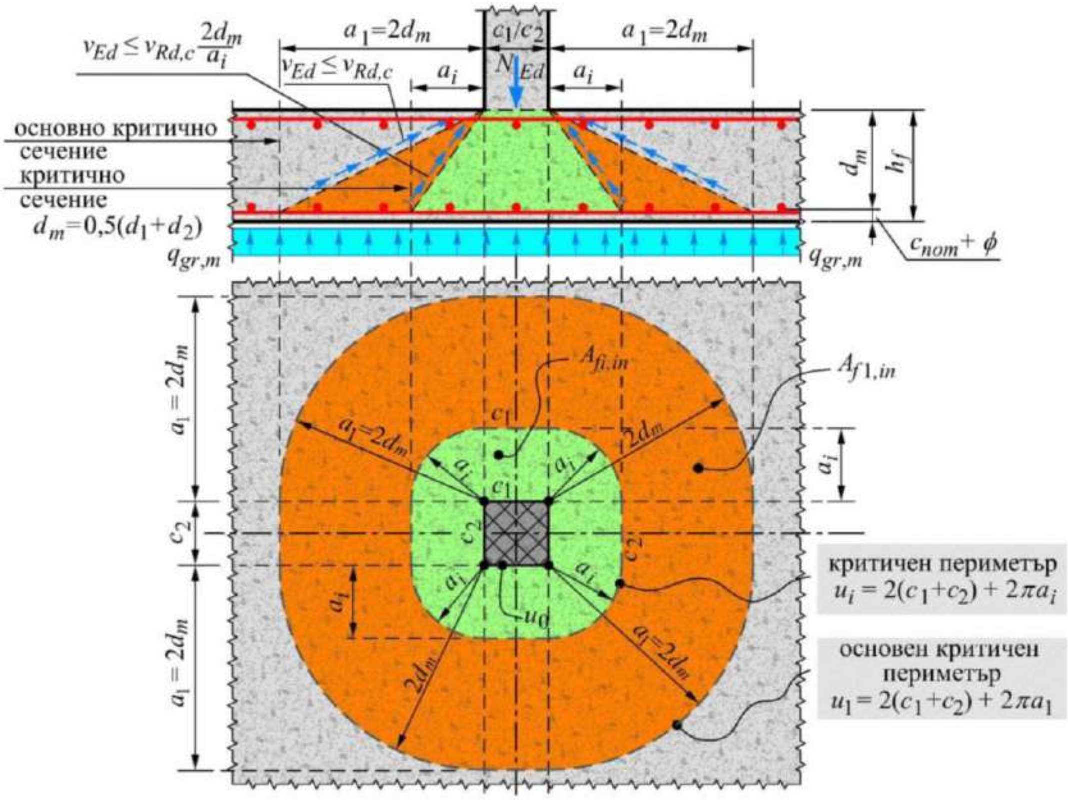
5.3.Punching

Check is made for slab with thickness $h_{f,2} = 1{,}0\ m$ (slab without piles). Most loaded column *(C_E15)* with compression force $N_{Ed} = 14\ 198\ kN$. is considered.

For the punching check to be satisfied it is necessary to be satisfied: $v_{Ed} \leq v_{Rd,c}.\frac{2.d_m}{a_i}$

3 control perimeters are checked. Results are presented in the following table:

Колона	c_1/c_2	$N_{c,Ed}$	$\sigma_{gr,m}$	ρ_{xy}	d_m	a_i	$A_{fi,in}$	$\beta.V_{Ed,red}$	u_i	$v_{Ed,red}$	$v_{Rd,c}$	$v_{Rd,c,min}$	$2.v_{Rd,c}.d_m/a_i$
	[mm]	[kN]	[MPa]	[%]	[mm]	[mm]	[m²]	[kN]	[m]	[MPa]	[MPa]	[MPa]	[MPa]
	...	...	*1)*	*2)*	*3)*	*4)*	*5)*	*6)*	*7)*	*8)*	*9)*	*9)*	...
C_E15	1000	14198	0,29	0,48	940	940	7,54	13213,8	9,91	***1,419***	0,449	0,365	***0,897***
	1000	14198	0,25	0,48	940	1410	12,89	12074,2	12,86	***0,999***	0,449	0,365	***0,598***
	1000	14198	0,2	0,48	940	1880	19,62	11300,6	15,81	***0,760***	0,449	0,365	***0,449***



1) $\sigma_{gr,m,i}$ –

2) $\rho_{xy} = 100.\sqrt{\rho_x.\rho_y}$

$\rho_{x,осн.мрежа} = \rho_{y,осн.мрежа} = 0,0017$

$\rho_x = \rho_{x,осн.мрежа} + \rho_{x,усилители} = 0,0017 + 0,0017 = 0,0034$

$\rho_y = \rho_{y,осн.мрежа} + \rho_{y,\ усилители} = 0,0017 + 0,0052 = 0,0069$

$\rho_{xy} = 100.\sqrt{\rho_x.\rho_y} = 0,48\ \%$

3) $d_m = 940\ mm$

4) $a_i = \{d_m\ ;1,5.d_m\ ;\ 2.d_m\}$

5) $A_{fi,in} = c_1.c_2 + (c_1 + c_2).2.a_i + \pi.a_i^2$

6) $\beta.V_{Ed.red} = \beta.(N_{c,Ed} - A_{fi,in}.\sigma_{gr,m})$

7) $u_i = 2.(c_1 + c_2) + 2.\pi.a_i$

8) $v_{Ed} = \beta.V_{Ed,red}/(u_i.d_m)$

9) $v_{Rd,c} = max\{0,12.k.(\rho_{xy}.f_{ck})^{1/3}\ ;\ 0,035.k^{3/2}.f_{ck}^{1/2}\}$

$\beta = 1,10$

Punching check is not satisfied! Additional reinforcement is needed, in order to resist the shear stresses in slab!

Design of the transversal reinforcement:

1) *Control perimeter u_0:*

   $u_0 = 2.(c_1 + c_2) = 2.(1000 + 1000) = 4000$ mm

2) *Design value of the maximum punching shear resistance along the control section*

   $v_{Rd,max} = 0,5.v.f_{cd}$

   $v = 0,6.(1 - f_{ck}/250) = 0,6.(1 - 35/250) = 0,516$

   $v_{Rd,max} = 0,5.v.f_{cd} = 0,5.0,516.23,33 = 6,02$

3) *Ckeck for the control perimeter*

   $\beta.V_{Ed} \leq V_{Rd,max} = v_{Rd,max}.d.u_0 = 6,02.940.4\ 000 = 22\ 635\ 200$ N $= 22\ 635,2$ kN

   β.V

   $\boldsymbol{\beta.V_{Ed} = 13\ 214}$ **kN < 22 635 kN → Check is satisfied**

4) *Definition of the control perimeter for witch an additional reinforcement is needed*

   $u_{out} = \frac{\beta.V_{Ed}}{v_{Rd,c}.d} = \frac{13\ 214.1000}{0,449.940} = 31\ 308$ mm

5) *Distance l_{out} definition:*

   $l_{out} = (u_{out} - u_0)/2.\pi = (31\ 308 - 4000)/2.\pi = 4\ 346$ mm

6) *Reinforced area definition*

   $l_w = l_{out} - 1,5d \geq 1,2.d \rightarrow l_w = 4\ 346 - 1,5.940 = 2\ 936$ mm $> 1\ 128$ mm

7) *Force in the reinforcement*

   $V_{Rd,s} = \beta.V_{Ed} - 0,75.v_{Rd,c}.d.u_1 \geq 0,5.\beta.V_{Ed}$

   $V_{Rd,s} = 13\ 214\ 000 - 0,75.0,449.940.15\ 810 \geq 0,5.13\ 214\ 000$

   $V_{Rd,s} = 8\ 209\ 424$ N $> 6\ 607\ 000$ N → $\boldsymbol{V_{Rd,s} = 8\ 210}$ **kN**

8) *Effective design resistance of the transversal reinforcement against punching*

   $f_{ywd,ef} = 250 + 0,25.d \leq f_{ywd} = 435$ MPa

   $f_{ywd,ef} = 250 + 0,25.940 = 485$ MPa > 435 MPa → $\boldsymbol{f_{ywd,ef} = 435}$ **MPa**

9) *Definition of the reinforcement*

   $A_{sw} = \frac{V_{Rd,s}}{f_{ywd,ef}.(1,5.d/s_r)} = \frac{8\ 210\ 000}{435.(1,5.940/200)} = 2\ 677\ mm^2 \rightarrow \boldsymbol{A_{sw} = 2\ 677\ mm^2}$ **→ Studs: 16ϕ16**

   → Stirrup: 8ϕ16(n = 2)

5.4. Compression resistance (geotechnical capacity) of a pile

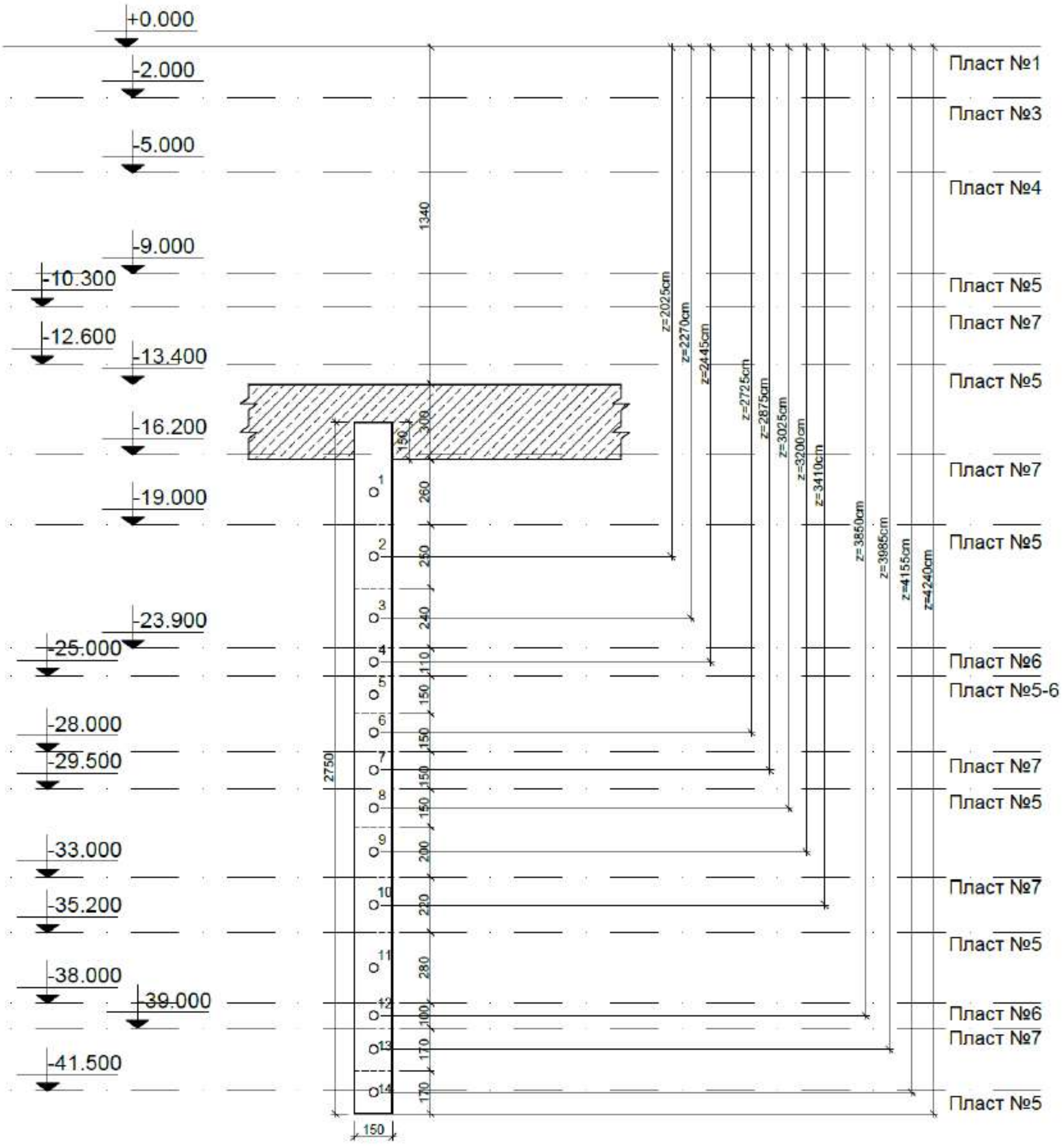


Fig. 15. Location of the pile in the ground

5.4.1. Geotechnical capacity of a pile according to DIN 1054:2005

Resistance of the pile R_c is defined from the top resistance in the pile R_b and the ambient friction R_s of the ground. For the characteristic value of R_c is valid:

$R_{c,k} = R_{b,k} + R_{s,k}$

$R_{c,k} = A_b \,.\, q_b + u \sum h_i \,.\, q_{s,i}$

Where:

$A_b = 17\,671{,}5\ cm^2$ - cross section area of the pile;

$u = 471{,}2\ cm$ – perimeter of the pile;

h_i – layer thicknesses within the pile length (i refers to the layer) ;

Terzaghi-Peck relation:

$C_u / p_a = 0{,}06.N_{30}$,

където:

$p_a = 1\ kg/cm^2 = 100\ kPa$

$C_u = 100\ .\ 0{,}06 = 6.N_{30}\ (kPa).$

q_b, $q_{s,i}$ are defined using the relation between N30 and $q_{c,k}$ from table 10.

Resistance of the top of the pile q_b (*Table 9*) is defined for three values of subsidence of the pile. (*0,02; 0,03; 0,10* $d_{пилот}$). q_b is defined for values of the subsidence, equal to the limit which is considered *5 cm*. When $d_{пилот}=150\ cm$ subsidence is $s = 0{,}03.d_{пилот} = 4{,}5\ cm$ and q_b is determined.

$Q_{c,k} = k.N30$

$k = 0{,}4$

Table 8

Странично съпротивление за **изливни пилоти** $q_{s,k}$ [MPa] в зависимост от $q_{c,k}$ (CPT) за несвързани почви и в зависимост от недренираната стойност на кохезията $c_{u,k}$ (за свързани почви)

Несвързани почви с $q_{c,k}$, MPa				**Свързани почви** $c_{u,k}$, MPa		
$q_{c,k}$ = 0	$q_{c,k}$ = 5	$q_{c,k}$ = 10	$q_{c,k}$ = 15	$c_{u,k}$ = 0,025	$c_{u,k}$ = 0,100	$c_{u,k}$ = 0,200
0,00	0,040	0,080	0,120	0,025	0,040	0,060

Table 9

Върхово съпротивление **за изливни пилоти** $q_{b,k}$ [MPa] в зависимост от $q_{c,k}$ (CPT) за несвързани почви и в зависимост от недренираната стойност на кохезията $c_{u,k}$ (за свързани почви)

Слягане s, cm	**Несвързани почви с** $q_{c,k}$, kPa				**Свързани почви** $c_{u,k}$, kPa	
	$q_{c,k}$ = 10	$q_{c,k}$ = 15	$q_{c,k}$ = 20	$q_{c,k}$ = 25	$c_{u,k}$ = 0,10	$c_{u,k}$ = 0,20
0,02*D*	0,70	1,05	1,40	1,75	0,35	0,90
0,03*D*	0,90	1,35	1,80	2,25	0,45	1,10
0,10*D*	2,00	3,00	3,50	4,00	0,90	1,50

Table 10

Корелация между $q_{c,k}$ и N_{30} ($q_{c,k} = k.N_{30}$)

Вид почва	$k = q_{c,k}/N_{30}$, MPa
Дребен до среден или глинест пясък	0,3 до 0,4
Едър пясък и пясък с малко чакъл	0,5 до 0,6
Зачакълен пясък	0,5 до 1,0
Чакъл с малко пясък и чакъл	0,8 до 1,0

Точка	z_i [cm]	Δh_i [cm]	*N30*	*Cu* [MPa]	$q_{c,k}$ [MPa]	$q_{sk,i}$ [MPa]	$\Sigma R_{s,k,i} = u.\Delta h_i.q_{sk,i}$ [kN]
1	1770	260	60	-	24	0,192	2352
2	2025	250	100	0,60	-	0,140	1649
3	2270	270	70	0,42	-	0,104	1323
4	2445	80	150	0,90	-	0,200	754
5	2575	150	115	0,69	-	0,158	1117
6	2725	150	90	0,54	-	0,128	905
7	2875	150	90	-	36	0,288	2036
8	3025	150	105	0,63	-	0,146	1032
9	3200	200	100	0,60	-	0,140	1319
10	3410	220	120	-	48	0,384	3981
11	3660	280	120	0,72	-	0,164	2164
12	3850	100	120	0,72	-	0,164	773
13	3985	170	125	-	50	0,400	3204
14	4155	170	125	0,75	-	0,170	1362
						$\Sigma R_{s,k,i}$ =	**23972**

$\boldsymbol{R_{s,k} = u \sum h_i \,.\, q_{s,i} = 23\,972\ kN}$

The resistanceat the top of the pile is determined for a subsidence value equal to the limit value for the pile, which is assumed to be 5 cm. For a pile with a diameter of d = 150 cm, a settlement of $s = 4{,}5$ *cm* is obtained at *0,03 d*. Therefore, $q_{b,k}$ from *Table 9* is defined.

When $q_{c,k} = k.N30 = 0{,}4.125 = 50$ *Mpa* → $\boldsymbol{q_{b,k} = 4{,}5\ MPa = 4\,500\ kPa;}$

$R_{b,k} = 17\,671{,}5.4{,}5.0{,}1 = 7\,952{,}2\ kN$ → $\boldsymbol{R_{b,k} = 7\,952{,}2\ kN}$

For the characteristic bearing capacity $R_{c,k}$ of the pile, coefficients ξ shall be introduced, taking into account the number and type of tests. In the present case, the data is from one geological profile, ie. the value of $R_{c,k}$ will be determined by correlation coefficients ξ, corresponding to a test of 1 pile.

Таблица 11. Correlation coefficients ξ

ξ за n =	1	2	3	4	5	7	10
ξ_3	1,40	1,35	1,33	1,31	1,29	1,27	1,25
ξ_4	1,40	1,27	1,23	1,20	1,15	1,12	1,08

For $n = 1$:
$\xi_3 = \xi_4 = 1{,}40$

Design value of the load bearing resistance is determined with:

$R_{c,d} = R_{b,k}/(\xi_3.\gamma_b) + R_{s,k}/(\xi_4.\gamma_s)$,

Where
$\gamma_b = \gamma_s = 1{,}10$ – coefficient, according to table 12.

Table 12

Носеща способност	Символ	Стойност
За върхово съпротивление	γ_b	1,10
За странично съпротивление	γ_s	1,10
Пълна/комбинирана (натиск)	γ_t	1,10
При опънати пилоти	$\gamma_{s,t}$	1,15

$R_{c,d}$ = 7 952,2/(1,40.1,10) + 23 972/(1,40.1,10) = 20 730 kN

$R_{c,d}$ = 20 730 kN

5.4.2. Geotechnical capacity of the pile, defined with tables according to "Pile Funding Standards", 1993

It is performed for the purpose of comparative analysis.

$R_{c,k} = A_b \, . \, q_b + u \sum h_i \, . \, q_{s,i}$

$R_{s,k} = u \sum h_i \, . \, q_{s,i}$

Values of $q_{s,i}$ are determined from the depth and consistency of the soil layer, according to table 13.

Table 13

Средна дълбочина на залягане на почвения пласт под повърхността на терена (m)	Характеристични стойности на съпротивления на триене на почвата по околната повърхнина на забивните пилоти, $q_{s;i,k}$ (kPa)								
	Пясъци със средна плътност								
	едро- и средно-зърнести	дребно-зърнести	прахови	-	-	-	-	-	-
	Глинести почви с показател на консистенция I_c								
	≥0,8	0,7	0,6	0,5	0,4	0,3	0,2	0,1	0,0
1	35	23	15	12	8	4	4	3	2
2	42	30	21	17	12	7	5	4	4
3	48	35	25	20	14	8	7	6	5
4	53	38	27	22	16	9	8	7	5
5	56	40	29	24	17	10	8	7	6
6	58	42	31	25	18	10	8	7	6
7	62	44	33	26	19	10	8	7	6
10	65	46	34	27	19	10	8	7	6
15	72	51	38	28	20	11	8	7	6
20	79	56	41	30	20	12	8	7	6
25	86	61	44	32	20	12	8	7	6
30	93	66	47	34	21	12	9	8	7
35	100	70	50	36	22	13	9	8	7

Results are presented in table 14.

Table 14

Точка	z_i [cm]	Δh_i [cm]	$I_{ck,i}$	$q_{sk,i}$ [kPa]	$u.\Delta h_i.q_{sk,i}$ [kN]
1	1770	260	1,18	76	931
2	2025	250	0,91	79	931
3	2270	270	0,91	83	1056
4	2445	80	0,48	31	117
5	2575	150	0,91	87	615
6	2725	150	0,91	89	629
7	2875	150	1,18	91	643
8	3025	150	0,91	93	657
9	3200	200	0,91	96	905
10	3410	220	1,18	99	1026
11	3660	280	0,91	103	1359
12	3850	100	0,48	38	179
13	3985	170	1,18	108	865
14	4155	170	0,91	110	881
				$\Sigma R_{s,k,i}$ =	**10795**

$\boldsymbol{R_{s,k} = u \sum h_i \,.\, q_{s,i} = 10\ 795\ kN}$

$R_{b,k} = A_b \,.\, q_b$

q_b, is determined from the depth and consistency of the soil layer, according to table 15.

Table 15

Дълбочина на залягане на пилотния връх	Изчислително съпротивление R(Mpa) под върховете на сондажно-изливни пилоти и тръбни цилиндрични пилоти, положени чрез изземване на почвата от отвора и запълването му с бетон, при глинести почви без льосовите, с показател на консистенция J_c равен на:						
m	1,0	0,9	0,8	0,7	0,6	0,5	0,4
3	0,85	0,75	0,65	0,50	0,40	0,30	0,25
5	1,00	0,85	0,75	0,65	0,50	0,40	0,35
7	1,15	1,00	0,85	0,75	0,60	0,50	0,45
10	1,35	1,20	1,05	0,95	0,80	0,70	0,60
12	1,55	1,40	1,25	1,10	0,95	0,80	0,70
15	1,80	1,65	1,50	1,30	1,10	1,00	0,80
18	2,10	1,90	1,70	1,50	1,30	1,15	0,85
20	2,30	2,10	1,90	1,65	1,45	1,25	1,05
30	3,30	3,00	2,60	2,30	2,00	-	-
40	4,50	4,00	3,50	3,00	2,50	-	-

$q_{b,k} = R = 4{,}5\ Mpa$

$R_{b,k} = 17\ 671{,}5.4{,}5.0{,}1 = 7\ 952{,}2\ kN \rightarrow \boldsymbol{R_{b,k} = 7\ 952{,}2\ kN}$

$R_{c,d} = R_{b,k}/(\xi_3.\gamma_b) + R_{s,k}/(\xi_4.\gamma_s)$

$R_{c,d} = 7\ 952{,}2/(1{,}40.1{,}10) + 10\ 795/(1{,}40.1{,}10) = 12\ 174\ kN$

$\boldsymbol{R_{c,d} = 12\ 174\ kN}$

The value of $R_{c,d}$, determined according to DIN1054: 2005, is taken as a test for the compressive load on the pile, because the determination of the resistance to ambient friction according to the soil consistency is inaccurate!

<u>Final resistance of the pile:</u>

<u>$R_{c,d}$ = 20 730 kN</u>

5.5.<u>Check for the real compression load on the pile according to ULS (GEO)</u>

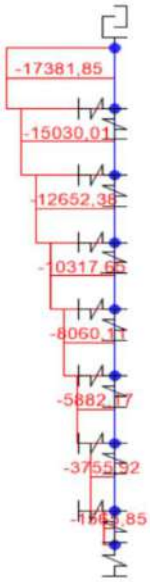


Fig. 17. Diagram of the maximal vertical support reaction [kN] of a pile

The actual load of the piles is determined by the analysis model of the foundation structure.

The total reaction in the vertical springs of the most loaded pile is (фиг. 17) :

$N_{Ed.max}$ = 17 381,85 kN

$\Sigma R_i = R_1 + R_2 + R_3 + R_4 + R_5 + R_6 + R_7 + R_b$

$\Sigma R_i = 2351{,}8 + 2377{,}6 + 2334{,}7 + 2257{,}5 + 2177{,}9 + 2126{,}2 + 2190{,}1 + 1565{,}8$

$\Sigma R_i = 17\,381{,}85$ *kN*

$\Sigma R_i \leq R_{c,d} \rightarrow \Sigma R_i = 17\,381{,}85$ kN $< R_{c,d} = 20\,730$ kN → Requirement is satisfied!

5.6. <u>Design of the foundation slab to resist the horizontal (seismic) forces</u>

The design proves that the horizontal forces of an earthquake are resisted entirely by the ground pressure at rest, acting on the basement walls, and by the forces of friction on the walls and the base plane of the foundation slab (Fig. 18). Piles do not take horizontal force.

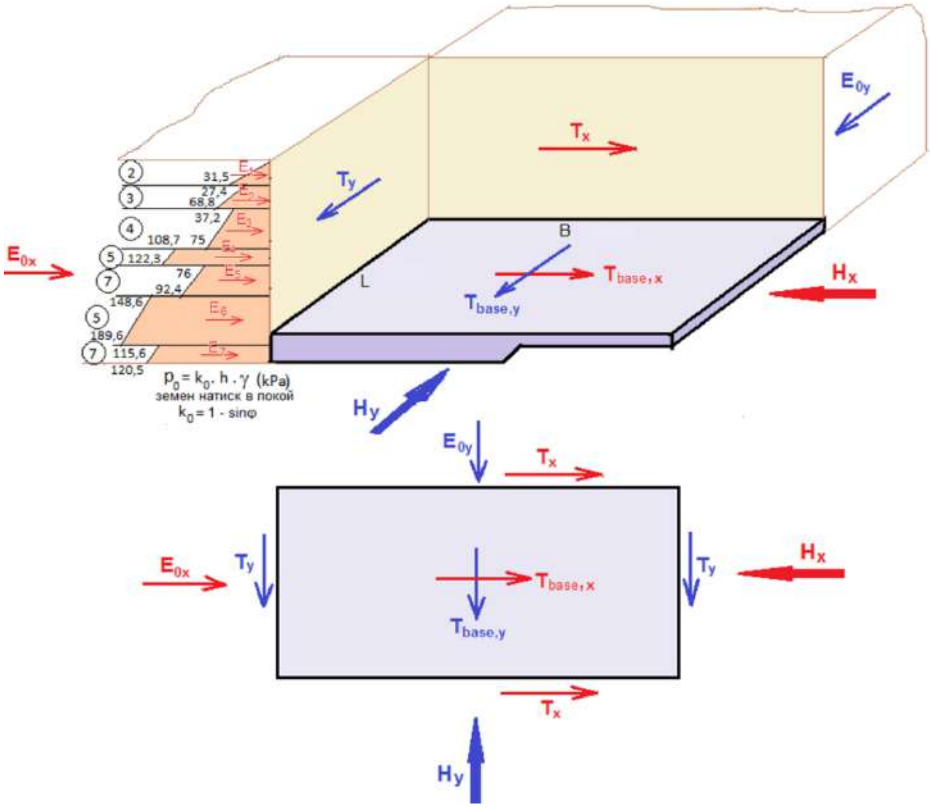


Fig. 18 Resistance forces of the foundation structure against horizontal loading

$\boldsymbol{H_x \leq E_{0x} + 2T_x + T_{base,x}} = \sum E_{0i} \,.\, B / \gamma + 2 \sum E_{0i}\, tg(\phi_i/2) \,.\, L / \gamma + N.\, tg(\phi_{base}/2) / \gamma$

$\boldsymbol{H_y \leq E_{0y} + 2T_y + T_{base,y}} = \sum E_{0i} \,.\, L / \gamma + 2 \sum E_{0i}\, tg(\phi_i/2) \,.\, B / \gamma + N.\, tg(\phi_{base}/2) / \gamma$

Where:

$H_x = 31\ 264\ kN$

$H_y = 29\ 494\ kN$

$T = N.\, tg(\phi/2)$

$N_{Ek} = V_k = 1\ 432\ 111\ kN$ – characteristic value of the vertical force in the base

$\gamma = 1{,}40$

$\sum E_{0i} = 1429\ kN/m$

$\sum E_{0i}.\ tg(\phi_i/2) = 362\ kN/m$

$H_x \leq 1429.46{,}2/1{,}4 + 2.362.\ 91{,}3/1{,}4 + 1\ 432\ 111.tg(24{,}6^0/2) / 1{,}4$

$H_x = 31\ 264\ kN \leq 47\ 157 + 47\ 215 + 223\ 036 = 317\ 408\ kN \rightarrow \boldsymbol{H_x = 31\ 264\ kN \leq 317\ 408\ kN}$

$H_y \leq 1429.91{,}3/1{,}4 + 2.362.\ 46{,}2/1{,}4 + 1\ 432\ 111.tg(24{,}6^0/2) / 1{,}4$

$H_y = 31\ 264\ kN \leq 43\ 191 + 23\ 892 + 223\ 036 = 290\ 119\ kN \rightarrow \boldsymbol{H_y = 29\ 494\ kN \leq 290\ 119\ kN}$

Conclusion: Horizontal force is balanced entirely by the interaction between the base structure and soil!

5.7. Required reinforcement in the piles

The pile is designed for eccentric compression. The minimum required reinforcement is accepted, after which the structural bearing capacity is determined by the help of an interaction diagram.

5.7.1. Detailing of the reinforcement according to БДС EN 1992-1-1 and EN1936

1) In order to ensure free penetration of the concrete mixture around the reinforcement, the individual bars, reinforcing skeletons and any other additional reinforcement should be constructed so that the pouring of the concrete mixture is not complicated.
2) Minimal diameter of the longitudinal reinforcement: $\phi 12$
3) Minimal distance between bars: $s_{max} = 100mm$
4) Maximal distance between bars: $s_{max} = 400mm$
5) Minimum concrete cover: $c_{min} = 75\ mm$
6) For pouring piles with a cross-sectional area $A_c \geq 1m^2$, longitudinal reinforcement is provided with an area of: $A_{s,min} \geq 0{,}0025.A_c$

5.7.2. Materials

Concrete **C35/45**: $f_{cd} = \alpha_{cc}.f_{ck}/\gamma_c = 0{,}85.35/1{,}5 = 19{,}83\ MPa$

Reinforcing steel **B500C:** $f_{yd} = f_{yk}/\gamma_s = 500/1{,}15 = 435\ MPa$

Concrete cover according to EC2:

For longitudinal reinforcement: $c_{nom} = c_{min,b} + \Delta c_{dev} = 75 + 10 = 85\ mm$

5.7.3. Design of the longitudinal reinforcement

$A_{s,min} \geq 0{,}0025.A_c = 0{,}0025.17\ 671{,}5 = 44{,}18\ cm^2$

$\boldsymbol{A_{s,min} = A_{s,req} = 44{,}18\ cm^2}$

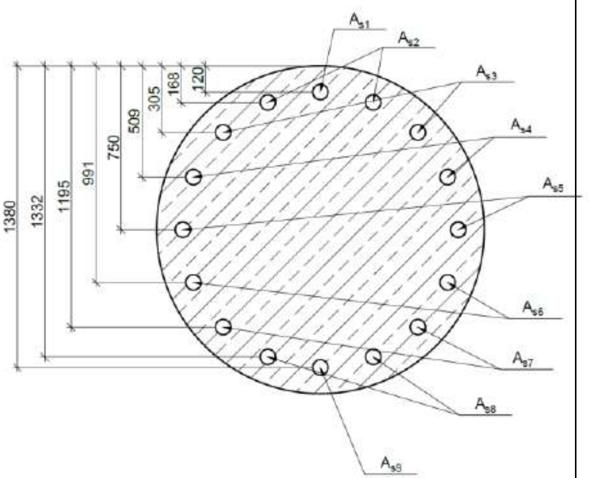
Accepted reinforcement:

16xN20

$s \approx 245\ mm$

$\boldsymbol{A_{s,prov} = 50{,}26\ cm^2 > A_{s,req} = 44{,}18\ cm^2}$

$\rho = A_{s,prov}/A_c = 50{,}26/17\ 671{,}5 = 0{,}0028 = 0{,}28\ \%$



Фиг. 19

Checks were performed for the pile with maximum compressive force and corresponding bending moment and for the pile with maximum bending moment and the corresponding compressive force. The results are presented by the interaction diagram in *fig. 20*. The data on the material and the selected reinforcement, with the help of which the diagram is constructed, are presented in *Table 16*.

$N_{Ed,max} = 17\ 382\ kN$ $\qquad M_{Ed,max} = 950\ kNm$

$M_{Ed,съотв.} = 183\ kNm$ $\qquad N_{Ed,съотв.} = 12\ 798\ kN$

Table 16

Бетон C35/45; Армировка B500B: 16xN20								
d=	150	[cm]	A_{s1}=	3,14	[cm2]	d_1=	12,0	[cm]
r=	75	[cm]	A_{s2}=	6,28	[cm2]	d_2=	16,8	[cm]
f_{yk}=	50	[kN/cm^2]	A_{s3}=	6,28	[cm2]	d_3=	30,5	[cm]
E_s=	20000	[kN/cm^2]	A_{s4}=	6,28	[cm2]	d_4=	50,9	[cm]
γ_s=	1,15		A_{s5}=	6,28	[cm2]	d_5=	75,0	[cm]
f_{yd}=	43,478	[kN/cm^2]	A_{s6}=	6,28	[cm2]	d_6=	99,1	[cm]
f_{ck}=	3,5	[kN/cm^2]	A_{s7}=	6,28	[cm2]	d_7=	119,5	[cm]
γ_c=	1,5		A_{s8}=	6,28	[cm2]	d_8=	133,2	[cm]
α_{cc}=	1		A_{s9}=	3,14	[cm2]	d_9=	138,0	[cm]
f_{cd}=	2,333	[kN/cm^2]						
ε_{c3}=	0,00175							
ε_{cu3}=	0,0035							
ε_{syd}=	0,0022		**Натоварване:**					
ε_{uk}=	0,0750		M_1=	183	kNm	N_1=	17381	kN
ε_{ud}=	0,0675		M_2=	950	kNm	N_2=	12 798	kN

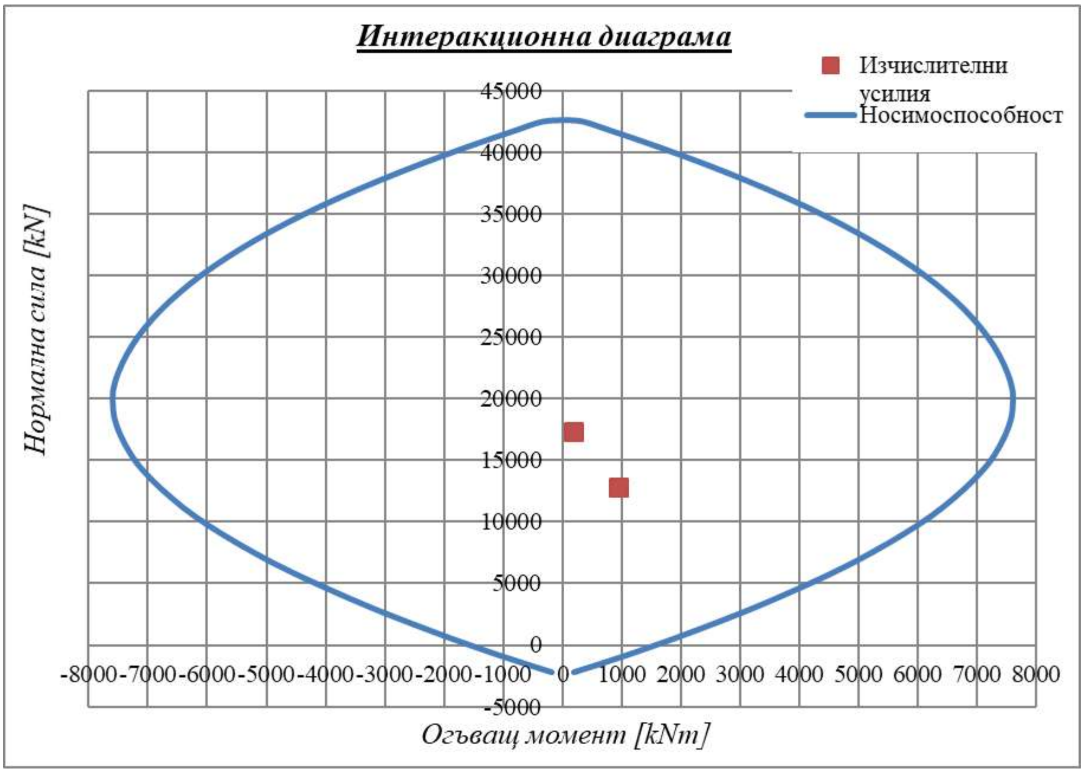


Fig. 20

Conclusion: The accepted longitudinal reinforcement is enough! → ***16N20***

5.7.4. Design of the transversal reinforcement

- Check for the shear resistance of the concrete section

$V_{Rd,c} = C_{Rd,c}.k.(100.\rho_1.f_{ck})^{1/3}.b.d \geq min\ V_{Rd,c} = v_{min}.b.d\ [N/m]$,

Where:

$C_{Rd,c} = \frac{0,18}{\gamma_c} = \frac{0,18}{1,5} = 0,12;$

Scale coefficient: $k = 1 + \sqrt{\frac{200}{d[mm]}} \leq 2{,}0 \rightarrow k = 1 + \sqrt{\frac{200}{1410}} = 1{,}38 < 2{,}0 \rightarrow k = 1{,}38$

$d = d_{пилот} - c_{пом} - ф/2 = 1500 - 85 - 10/2 = 1410\ mm$

$\rho_l \leq 0{,}02 \rightarrow \rho_l = 0{,}0028$

$f_{ck} = 35\ N/mm^2$

$v_{min} = 0{,}035.k^{3/2}.f_{ck}^{1/2} = 0{,}035.1{,}38^{3/2}.35^{1/2} = 0{,}336\ MPa$

$b = d = 1500\ mm$

$v_{Rd,c} = 0{,}12.1{,}38.(100.0{,}0028.35)^{1/3} \rightarrow \mathbf{v_{Rd,c} = 0{,}354\ MPa > v_{min} = 0{,}336\ MPa}$

$V_{Rd,c} = v_{Rd,c}.\ \pi.d^2/4 = 0{,}354.\pi.1410^2/4 \rightarrow \mathbf{V_{Rd,c} = 552\ kN}$

$\mathbf{V_{Ed,max} = 253\ kN}$

$\mathbf{V_{Ed} = 253\ kN < V_{Rd,c} = 552\ kN}$

Requirement is satisfied! → The minimum required spiral reinforcement with diameter N10/s = 20 cm is provided!

5.8. SLS checks of the pile

It is necessary to check whether the subsidence of the pile foundation is within the permissible limits. Maximum subsidence $s_{lim} = 0{,}03.d_{пилот} = 0{,}03.150cm = 4{,}5\ cm$ is accepted.

A conditional flat foundation at the top of the piles is considered, with dimensions in plan *L '= 53,6m* and *B' = 53,6m*, determined by the dimensions of the end piles (located at the contour of the slab) from *fig. 3* and the average angle of internal friction of the soil φ_{cp}, according to *fig. 21.*

$\varphi_{cp} = (24{,}6^\circ + 39{,}9^\circ)/2 = 32{,}25^\circ$

$\varphi_{cp}/4 = 8^\circ$

The load q_2 of the conditional flat foundation for determining the settlement consists of the characteristic load q_1 in the main plane of the foundation slab, reduced to the area of the conditional foundation, as well as the additional weight of the reinforced concrete piles (85 pcs.).

The active zone of subsidence of the conditional foundation is determined by formula (7) of Annex 4 of the "Standards for the design of flat foundations, 1996", relating to wide-area foundations:

$H_a = k_2\ .\ k_3.(9 + 0{,}15.B') = 1.\ 0{,}85.(9 + 0{,}15\ .\ 53{,}6) = 14{,}3m$

The average modulus of йo of soil E_{cp} in the area of the active zone H_a is defined as the average of layers 5 and 7 from *Table 1* at increasing in depth with $E_{inc}= 500\ kPa$ for *1m':*

$E_{cp} = (34600\ kPa + 14400\ kPa)/2 + (26\ m + 14{,}3\ m/2).500\ kPa = 41\ 075\ kPa$

The average load for ground with thickness *Ha* is:

$\sigma_{cp} = (q_2 + q_3)/2 = (139+120)/2=129{,}5\ kPa.$

The average subsidence of the pile is:

$\mathbf{S = \sigma_{cp}\ .\ Ha / E_{cp} = 129{,}5\ .\ 14{,}3 / 41\ 075 = 4{,}5\ cm = S_{lim} = 4{,}5\ cm}$

→ Condition is satisfied!

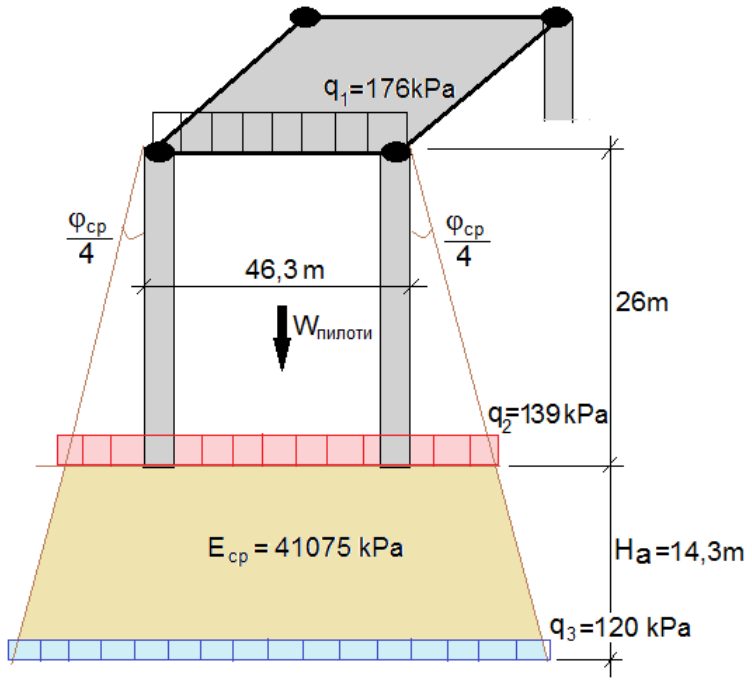

q1=176kPa
φcp
4
φcp
4
46,3 m
Wпилоти
26m
q2=139 kPa
Ecp = 41075 kPa
Ha=14,3m
q3=120 kPa

Fig. 21

IX. Construction technology

In the current graduation project is considered an exemplary technology for installation of the structure and an exemplary technology for concrete work above elevation *+0.00.*

The floor slabs are composite, which is convenient from a technological point of view, as the steel decking acts as a remaining formwork. Concreting with stationary concrete pumps with distribution booms was chosen due to the required hourly inflow of concrete mixture, as well as the geometric dimensions of the building in plan and height, which do not allow the use of truck concrete pumps. Concrete pipes and their supporting structures pass through service openings in the floor slabs.

For the installation of the steel structure, tower cranes are provided, serving both the high and the low part of the building.

For the installation of the metal structure up to the elevation of *+160,6 m*, a Potain tower crane with a horizontal boom with a length of *70m* and a maximum lifting capacity *Q = 64t* is provided. The crane is anchored in the already executed foundation slab, and the height is strengthened with special connections, supported in the areas of the “Vierendeel belt trusses” and between them. The connections are made with the help of diaphragms and small beams, which are connected to the structure by plates welded to the columns. The crane rises in height due to the high number of storeys of the structure. When the installation of the top structure is completed, the crane lowers itself to the required height and a truck crane is used for its dismantling.

The other two tower cranes serve to build the low-storey building and are located on both sides. It is necessary to diverge them in height, as their trajectories intersect. For this purpose, one crane is 11.60 m higher than the other, and two additional elements with a height of 5.80 m were used in the construction of the tower. A truck crane is used to dismantle the tower cranes.

The range of cranes also includes areas for storage of steel elements to be installed.

1. Concreting

The schemes of concreting are considered both on floor slabs up to elevation *+35,70 m*, and on floor slabs from elevation *+ 35,70 m* to elevation *+160,6 m.*

1.1. Concreting of floor slabs up to level +35,70 m

1.1.1. Definition of the required hourly inflow of concrete mixture

$Q_ч > \frac{F.h}{(t_c - t_0).K_н}$ *[m³/h]* – hourly inflow of concrete mixture due to requirements for monolithic concreting;

Where:

$Q_ч$ – required hourly inflow of concrete mixture

F – area of the concrete layer

h – height of the layer

t_c – cement hardening time;

t_0 – time for transporting and pouring a portion of concrete;

$K_н = 0,8 – 0,9$

$(t_c – t_0) < 2,5\ h$

$F = B_{пл} .d_{пл}.\sqrt{2} = 44,8.0,108. \sqrt{2} = 6,84\ m^2$

$d_{пл} = h_{pl,mid} = 0,108\ m$

Accepted:

h = 6,40 m – width of the concrete layer;

B ≈ 44,8 m

$Q_ч > \frac{6{,}84.6{,}40}{2{,}5.0{,}8} = 21{,}9\ m^3/h$

The large area of the slab ($A = 4\ 575{,}35\ m^2$) implies the prediction of working joints, limiting individual areas of the slab to $1500\ m^2$.

The part of the slab located into the high building, which reaches a height of +160.6 m, is separated by a working joint and concreted with a stationary concrete pump located in one of the service shafts. The rest of the slab is separated by another working joint and concreted with the help of another stationary concrete pump located in the lower part of the building.

The required hourly inflow of concrete mix is determined on the basis of the required amount of concrete for the busiest concreting stage.

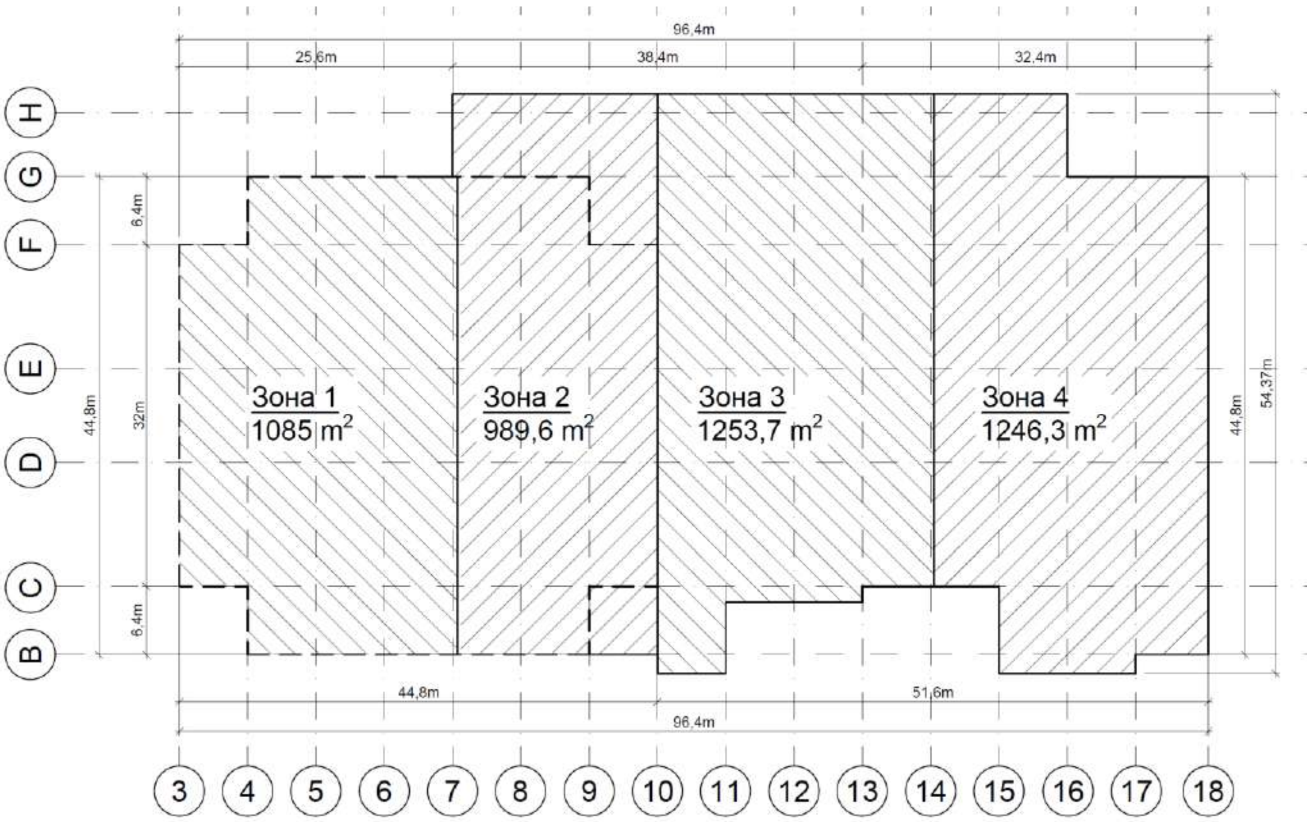
$V_{pl} = A_{pl,max}.h_{pl,mid} = 1253{,}7.0{,}108 = 135{,}4\ m^3$

$Q_h = V_{pl}/z = 135{,}4/7 = 19{,}34\ m^3/h$

z = 7 hours

$Q_h = 19{,}34\ m^3/h$ – required hourly inflow of concrete mixture for the busiest concreting stage.

Accepted: $\boldsymbol{Q_ч = 21{,}9\ m^3/h}$



1.1.2. Transport of the concrete mixture to the construction

$n = Q_h / П_{тр.ср.}$ – *number of concrete mixer cars*

$П_{тр.ср.} = 60.V.k_в.k_н / t_ц$

$t_ц = t_н + t_p + 2.l/v_{cp} = 50\ min$

$t_н + t_р \approx 30\ min$

$2.l/v_{cp}\ = 2.5/30 \approx 20\ min$

$l = 5\ km$

$v_{cp} = 25\ km/h$

Selected concrete mixer car:

Liebherr HTM 1004

$V = 10\ m^3$ – volume of the mixer;

$П_{mp.cp.}\ = 60.10.0,8\ .0,9\ /50 = 8,64\ m^3/h$

$\boldsymbol{n = 21,9/8,64 \approx 3}$ ***concrete mixer cars***

Technical Data:

Superstructures	Nominal filling in m³ hardened concrete	Water volume in m³	Geometric drum volume in m³	Mixer weight in configuration	
				Vehicle drive in kg	Sep. engine in kg
HTM 604	6	6,8	11,0	3360	3860
HTM 704	7	7,7	12,3	3480	3980
HTM 804	8	9,1	14,3	3855	4355
HTM 904	9	10,2	16,0	4030	4660
HTM 1004	10	11,0	17,6	4350	4980
HTM 1004 K	10	11,0	17,4	4480	5110
HTM 1204	12	12,9	20,7	4990	5620
HTM 1204 K	12	12,6	18,3	4900	5530
HTM 1504	15	15,3	24,5	5600	–

1.1.3. Transport of the concrete mixture at the construction

It is rational to do the concreting with a concrete pump. Based on the parameters of the building and the required hourly inflow of concrete mixture, concreting with two stationary concrete pumps with distribution booms was chosen. One stationary concrete pump is located in one of the service openings of the floor structure of the high building and is used for concreting the floor slabs to a height of *160,6 m*. The other is located in the service opening of the low part of the building and serves for concreting the floor slabs to elevation *+35,70 m*. At this point, the necessary stationary concrete pump for concreting the slab in zones 3 and 4 should be selected. The stationary concrete pump for zones 1 and 2 is discussed in detail in point *1.2.*

Required parameters of the concrete mixer cars, located at the low-storey part of the building:

$Q_{h,req} \approx 21,9\ m^3/h$

$L_{b,req} = 47\ m$ – range of the boom;

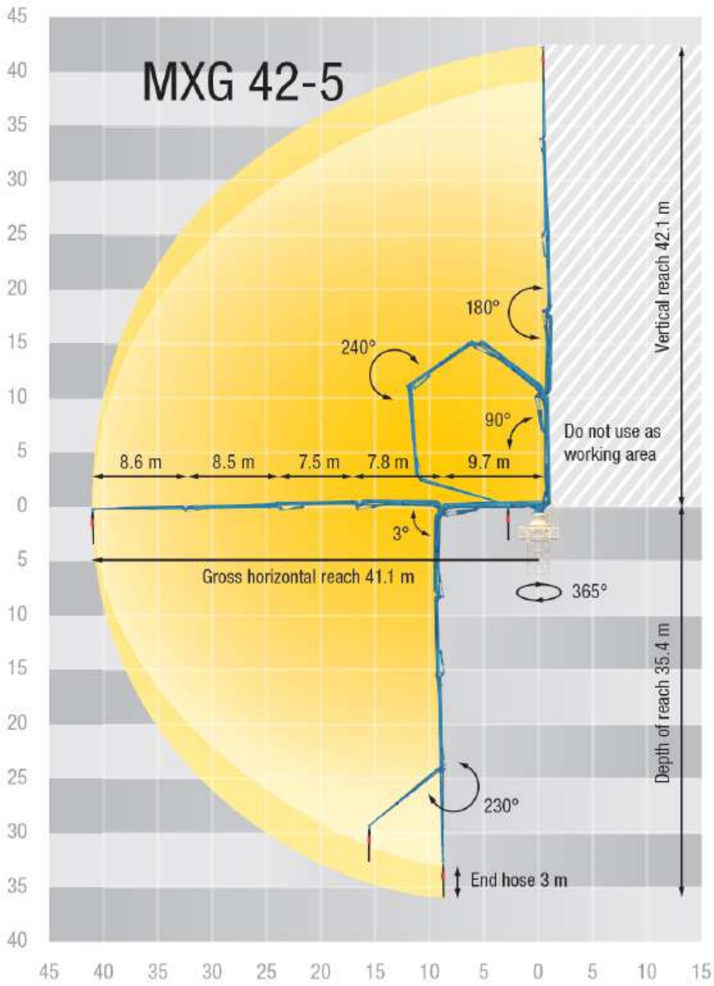
Stationary concrete pump selected: ***Putzmeister P 730***

$Q_{h,prov}$ = 30 m^3/h > $Q_{h,req}$ = 21,9 m^3/h

Technical data	P 730 TD / SD
Conveyor hopper	300 l / 360 l (with rubber collar)
Delivery rate	3 – 30 m³/h
Maximum delivery pressure	55 bar
Delivery distance**	Max. 300 m horizontal and 100 m vertical reach; with shotcrete/ fine concrete, max. 100 m horizontal and 80 m vertical reach
Drive	Three-cylinder Deutz diesel engine with turbocharging, 55.4 kW (Stage V)
Piston pump	Two-cylinder pump, hydraulic
Piston stroke	700 mm
Delivery cylinder (dia.)	180 mm
Drive cylinder (dia.)	80/45 mm
Pressure connection coupling	5,5"
Max. number of strokes	29 strokes/min
Weight	2450 kg (TD) / 2180 kg (SD)
Length × width × height	4810 mm x 1592 mm x 1983 mm (TD) 3137 mm x 1408 mm x 1734 mm (SD)
Filling height	1340 mm (TD) / 1093 mm (SD)
Maximum particle size	32 mm
Machine model	Trailer concrete pump (TD), stationary pump (SD)
Chassis	Road travel chassis (TD), skid frame (SD)

Equipment	
2.5 t chassis	■
Centralised lubrication system (with hand pump and manifold block)	■
Cleaning accessories	■
Delivery rate regulation (electric)	■
Ergonic® Output Control (EOC)	■
Exhaust flap	■
Lifting eye	■
Operating instructions and spare parts list	■
Protective grille	■
Rubber collar	■
Safety helmet	■
Stroke counter	■
Toolbox and tools	■
Truck coupling	■
Vibrator	■
"Hopper empty" signal	□
24 V light assembly	□
Additive metering	□
Cable remote control	□
Car coupling	□
Centralised lubrication system (electric, automatic)	□
Customer labelling	□
Flushing water pump	□
High-pressure cleaner	□
Radio remote control with delivery rate regulation	□
Special hood paint in RAL colours	□
Splash guard – hopper cover and spray guard	□
USB charging port with smartphone holder on the operating panel	□
Working lights	□

Distribution boom selected*:* ***MXG 42-5***



Technical data

Stationary placing boom		MXG 42-5
Sections		5
Reach height	m	42.1
Horizontal reach	m	41.1
Reach depth max.	m	35.4
Delivery line		DN 125
End hose length	m	3
Slewing range	°	365
Weight (boom + pedestal + working platform)	t	13
Weight (hydraulic drive unit)	t	1.3

All data max. theor.

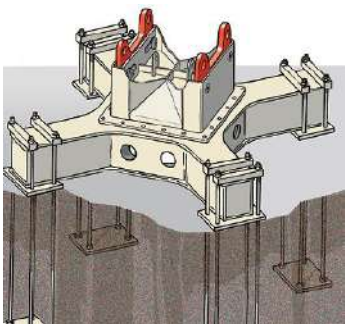
Dimensions can differ depending on configuration.

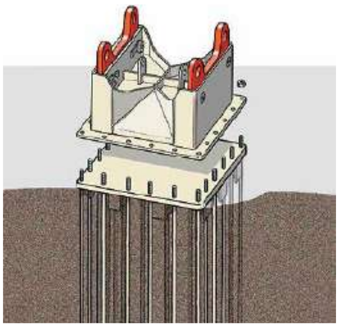
$L_{b,prov}$ = 41,1 m < $L_{b,req}$ = 47 m → The range is not enough! For the other zones, which doesn't fall in the range of concrete pumps, is predicted concreting with bucket with volume 1,5m^3!

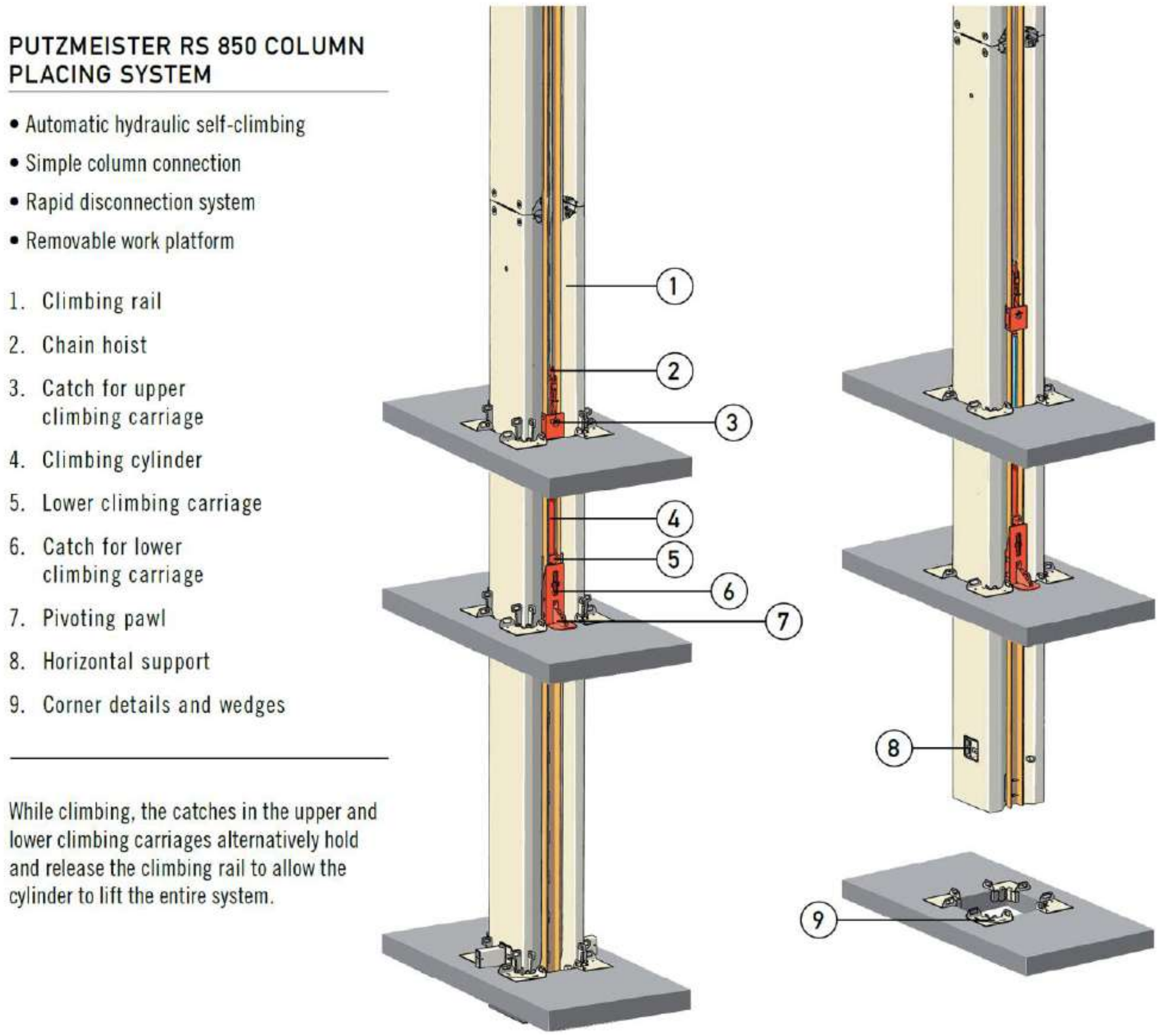
Supporting structure: ***Putzmeister RS 850 Column placing system***

To reach the required height, a system of self-climbing columns is used, acting as a supporting structure and delivering the concrete mixture to the distribution boom at the design height. The base of the column is very small *(1.0m x1.0m)*, which allows the system to be located in narrow spaces of the structure. The individual column fragments are with selected height of *10 m.*

Supporting structure could be anchored to the foundation structure.







1.2.Concreting of floor slabs up to level +160,6 m

1.2.1. Definition of the required hourly inflow of concrete mixture

$Q_ч > \frac{F.h}{(t_c - t_0).K_н}$ $[m^3/h]$ – hourly inflow of concrete mixture due to requirements for monolithic concreting;

Where:

$Q_ч$ – required hourly inflow of concrete mixture

F – area of the concrete layer

h – height of the layer

t_c – cement hardening time;

t_0 – time for transporting and pouring a portion of concrete;

$K_н = 0,8 – 0,9$

$(t_c – t_0) < 2,5\ h$

$F = B_{пл}.d_{пл}.\sqrt{2} = 44,8.0,108.\sqrt{2} = 6,84\ m^2$

$d_{пл} = h_{pl,mid} = 0,108\ m$

Accepted:

$h = 6{,}40\ m$ – width of the concrete layer;

$B \approx 44{,}8\ m$

$Q_ч > \frac{6{,}84.6{,}40}{2{,}5.0{,}8} = 21{,}9\ m^3/h$

The required hourly inflow of concrete mix is determined on the basis of the required amount of concrete for the busiest concreting stage.

$V_{pl} = A_{pl,max}.h_{pl,mid} = 1153.0{,}108 = 124{,}52\ m^3$

$Q_h = V_{pl}/z = 124{,}52/7 = 17{,}78\ m^3/h$

$z = 7\ hours$

$Q_h = 17{,}78\ m^3/h$ – required hourly inflow of concrete mixture for the busiest concreting stage.

Accepted: $\boldsymbol{Q_ч = 21{,}9\ m^3/h}$

1.2.2. Transport of the concrete mixture to the construction

$n = Q_h / П_{тр.ср.}$ – *number of concrete mixer cars*

$П_{тр.ср.} = 60.V.k_в .k_н /t_ц$

$t_ц = t_н + t_p + 2.l/v_{cp} = 50\ min$

$t_н + t_p \approx 30\ min$

$2.l/v_{cp} = 2.5/30 \approx 20\ min$

$l = 5\ km$

$v_{cp} = 25\ km/h$

Selected concrete mixer car:

Liebherr HTM 1004

$V = 12{,}0\ m^3$ – volume of the mixer;

$П_{тр.ср.} = 60.12{,}0.0{,}8 .0{,}9 /50 = 8{,}64\ m^3/h$

n = 21,9/8,64 ≈ 3 concrete mixer cars

1.2.3. Transport of the concrete mixture at the construction

Concreting with stationary concrete pump with distribution boom was chosen. Concreting piepe and the supporting structure are placed in a service opening.

Required parameters of the concrete mixer cars, located at the low-storey part of the building:

$Q_{h,req} = 21{,}9\ m^3/h$

$L_{b,req} = 45\ m$ – range of the boom;

Stationary concrete pump selected*:* ***Putzmeister BSA 1005 D5***

$Q_{h,req}$ = 21,9 m^3/h

$Q_{h,prov}$ = 52 m^3/h

$Q_{h,prov} > Q_{h,req}$

Technical data

Model		BSA 1005 D5	BSA 1005 E
Material number		102682.000	102683.000
Output	m³/h	52	47
Delivery pressure	bar	70	
Delivery cylinder	Ø mm	180	
Delivery cyl. stroke	mm	1000	
Strokes / minute		33	31
Engine / motor power	kW	55.4	45
Hopper		RS 488	
Capacity	l	approx. 300	
Filling height	m	1.23	
Transfer tube		S 1812	
Line connection	mm	SK 125	
Pressure connection	"	5.5	
Control system		Ergonic 3 PS	
Diesel tank	l	120	–
Weight	kg	2900	3100

Serial equipment

- Agitator safety switch off
- Cable remote control 10 m
- Central lubrication on hopper with grease gun
- Chassis F8
- Cleaning accessories
- Coloring: Pump, frame, roof anthracite-grey Doors chrome yellow
- Concrete hopper with agitator
- Drive diesel, Deutz 55.4 kW, Stage 5 (1005 D5)
- Drive electric, 45 kW, iE03 (1005 E)
- Electronic error management
- Ergonic Graphic Display
- Flushing water pump 50 l/min 20 bar
- Grill 60 mm
- Operating manuals (1× paper, 1× CD-ROM) spare part list
- Output adjustment, electrical
- Outriggers 3× mech.
- Speed drawdown automatically on pump stop (Diesel driven only)
- Splash guard
- Support wheel
- Tool set incl. standard equipment
- Tow coupler, DIN eye

Distribution boom selected*:* ***MXG 42-5***

$L_{b,prov}$ = 41,1 m < $L_{b,req}$ = 45 m → The range is not enough! For the other zones, which doesn't fall in the range of concrete pumps, is predicted concreting with bucket with volume 1,5m^3!

Supporting structure: ***Putzmeister RS 850 Column placing system***

2. Installation of structural elements

The installation of the structural elements is done with the help of tower cranes.

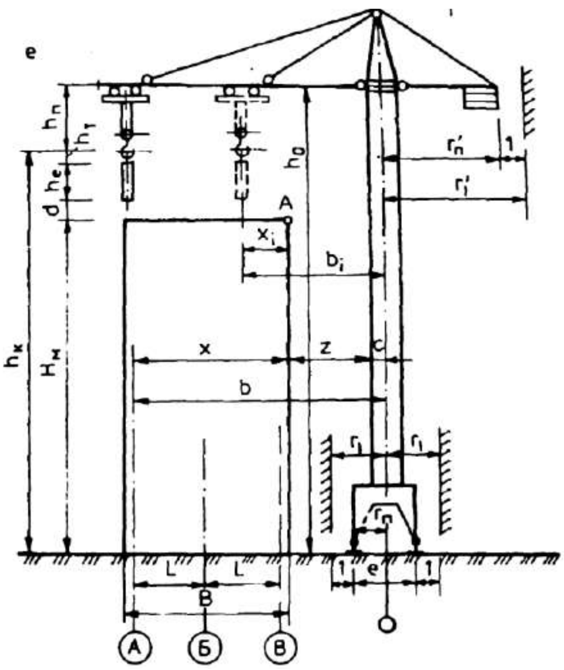
The ***Potain MD 1600*** crane, located on the outside of the building, is attached to the structural elements at a certain distance (in the area of the technical floors) for the construction up to the elevation of *+160,6 m*. The crane has a horizontal boom, increases in height by self-lifting with the help of special creeping frames, and is anchored to the already completed foundation slab.

Two additional free-standing tower cranes with the same characteristics ***Liebherr 630 EC-H 40***, located on both opposite sides of the building, are used for the construction up to an elevation of *+35,7 m* (low-storey building). The cranes diverge in height by *11.6 m.*

The parameters of the cranes should be considered in detail.

The cranes are selected based on the installation parameters of the individual elements. The parameters are determined at a pre-selected position of the crane relative to the building.

The main designations used in the selection of the crane are presented in the diagram.

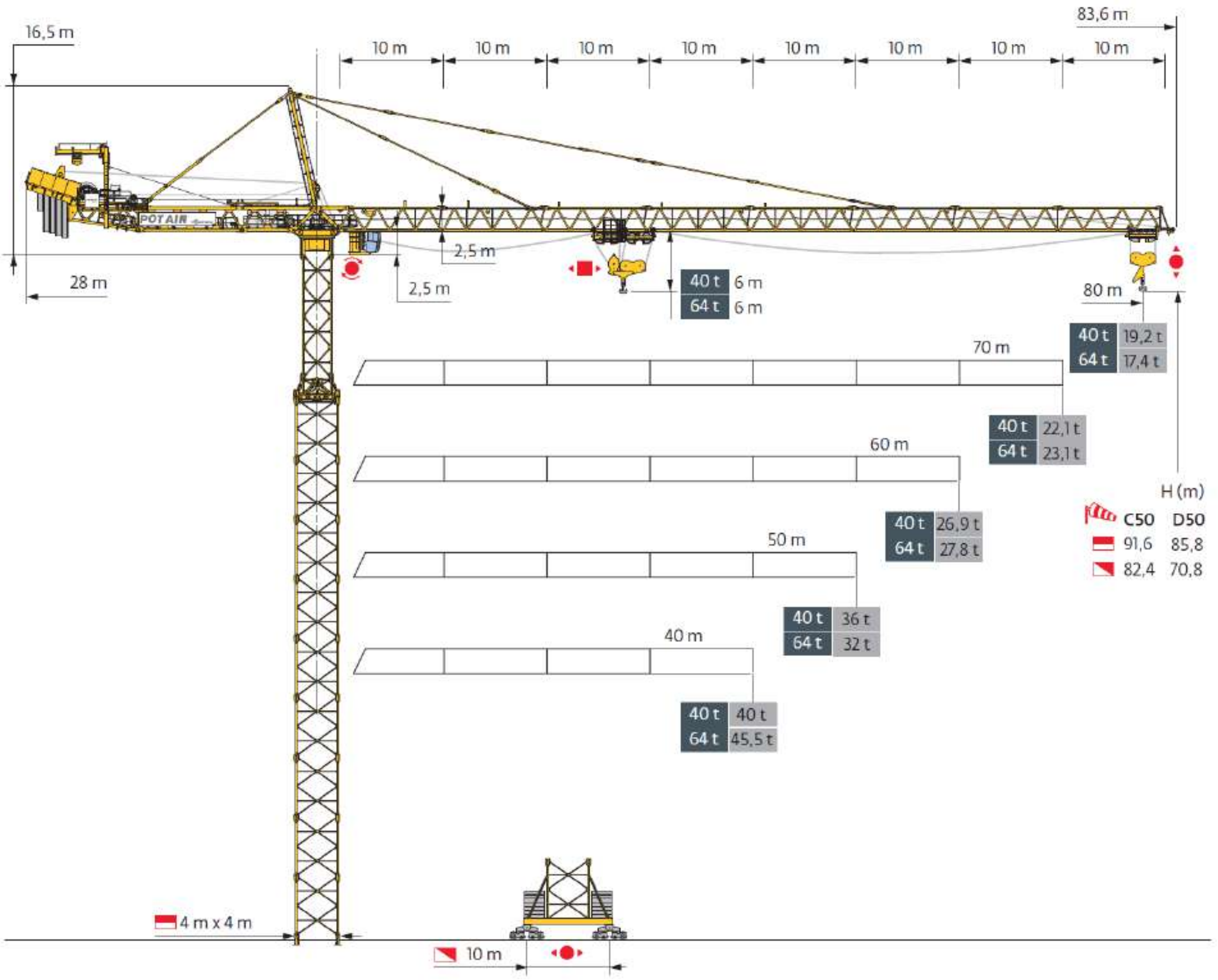


2.1. Potain MD 1600

The crane is used for mounting the elements of the multi-storey structure, reaching an elevation of *+160,6 m*. The crane is selected according to the elements to be installed and their installation parameters.

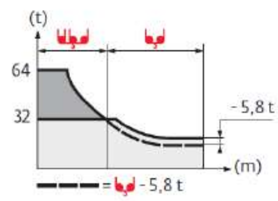
Елемент	*МОНТАЖНИ ПАРАМЕТРИ*									*Проверка на избрания кран*			*Марка на крана*
	Монтажна маса			*Височина до куката*				*Обсег*					
	q_e	q_m	$Q_м$	$H_м$	h_e	h_m	h_k	x	b	$Q_{кр}$	$h_{кр}$	$b_{кр}$	
	[t]	*[t]*	*[t]*	*[m]*	*[m]*	*[m]*	*[m]*	*[m]*	*[m]*	*[t]*	*[m]*	*[m]*	
C-C10_16	8,78	0,44	9,22	152,6	7,945	1,0	162,5	54,31	61,31	***27,0***			
C-C10_4	18,55	0,93	19,48	18,7	10,945	1,0	31,6	54,31	61,31	***27,0***			
C-D10_6	15,64	0,78	16,42	35,5	12,545	1,0	50,0	52,09	59,09	***26,0***			
PB120	3,50	0,18	3,68	76,5	1,730	1,0	80,2	52,89	59,89	***26,9***	**171,45**	**70**	***Potain MD1600***
PB55	2,10	0,11	2,21	76,5	0,890	0,5	78,9	42,25	49,25	***35,0***			
SB8	1,06	0,05	1,11	76,5	0,500	0,5	78,5	49,76	56,76	***48,0***			
C-F10_2	23,62	1,18	24,80	-3,6	11,045	1,0	9,4	55	62	***27,1***			

Potain MD 1600



64 t

80 m	6,5	▶	24,1	25	30	35	40	42,1	48,6	50	55	60	65	70	75	**80**	m
			64	61,4	49,4	40,7	34,3	32	32	31	27,7	24,9	22,6	20,6	18,9	17,4	t
70 m	6,5	▶		26,4	30	35	40	45	46,1	53,3	55	60	65	**70**	m		
				64	55,1	45,7	38,6	33,1	32	32	30,8	27,8	25,3	23,1	t		
60 m	6,5	▶		26,4	30	35	40	45	46,1	53,2	55	**60**	m				
				64	55,1	45,6	38,6	33,1	32	32	30,8	27,8	t				
50 m	6,5	▶		28,7	30	35	40	45	**50**	m							
				64	60,6	50,4	42,7	36,8	32	t							
40 m	6,5	▶			30,1	35	**40**	m									
					64	53,6	45,5	t									



2.2. Liebherr 630 EC-H 40 Litronic

The crane is used for mounting the elements of the structure, reaching an elevation of *+45,2 m*. Two cranes of the considered model were chosen, which serve for construction of the lower part of the building. They are located in opposite positions and diverge in height by *11,6 m.*

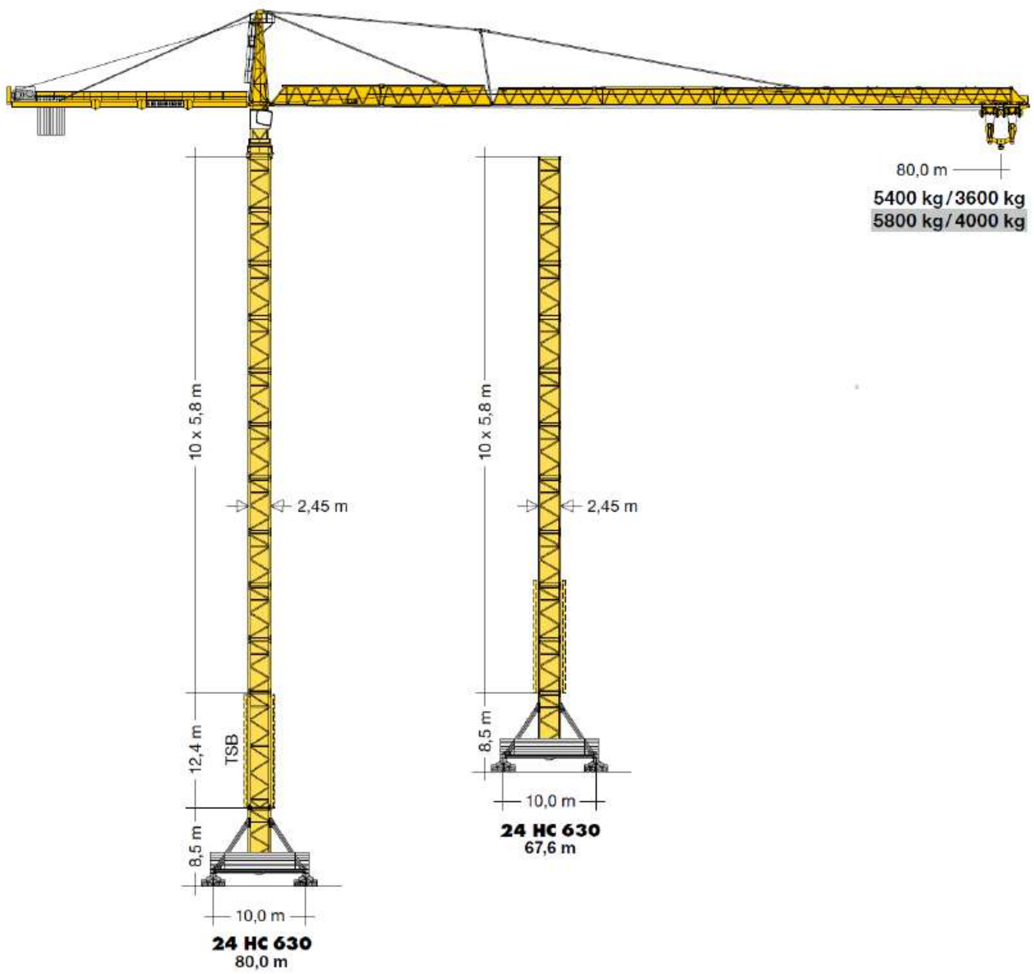
A truck crane is used to build the two cranes, after which the tower cranes themselves increase in height. The dismantling is analogous.

The crane model was chosen based on the installation parameters of the elements needed to build the structure, reaching elevation *+45,2*. They are presented in the following tables.

Елемент	***МОНТАЖНИ ПАРАМЕТРИ - кран №1***									***Проверка на избрания кран***			***Марка на крана***
	Монтажна маса			***Височина до куката***				***Обсег***					
	q_e	***q_m***	***$Q_м$***	***$H_м$***	***h_e***	***h_m***	***h_k***	***x***	***b***	***$Q_{кр}$***	***$h_{кр}$***	***$b_{кр}$***	
	[t]	*[t]*	*[t]*	*[m]*	*[m]*	*[m]*	*[m]*	*[m]*	*[m]*	*[t]*	*[m]*	*[m]*	
C-H9_2	9,90	0,50	10,40	-3,6	10,945	1,0	9,3	39,39	46,39	***12,60***	***55,9***	***60***	***Liebherr 630 EC-H 40 Litronic***
C-E11_2	9,90	0,50	10,40	-3,6	10,945	1,0	9,3	41,6	48,6	***12,00***			
C-D17_2	9,90	0,50	10,40	-3,6	10,945	1,0	9,3	43,51	50,51	***11,20***			
F-18_5	3,50	0,18	3,68	29,7	8,945	1,0	40,6	28,86	35,86	***17,15***			
PB39	2,45	0,12	2,57	17,7	0,540	0,5	19,7	34,74	41,74	***14,40***			
PB102	3,50	0,18	3,68	17,7	0,800	0,5	20,0	33,35	40,35	***15,09***			
SB8	1,06	0,05	1,11	17,7	0,500	0,5	19,7	39,47	46,47	***12,50***			

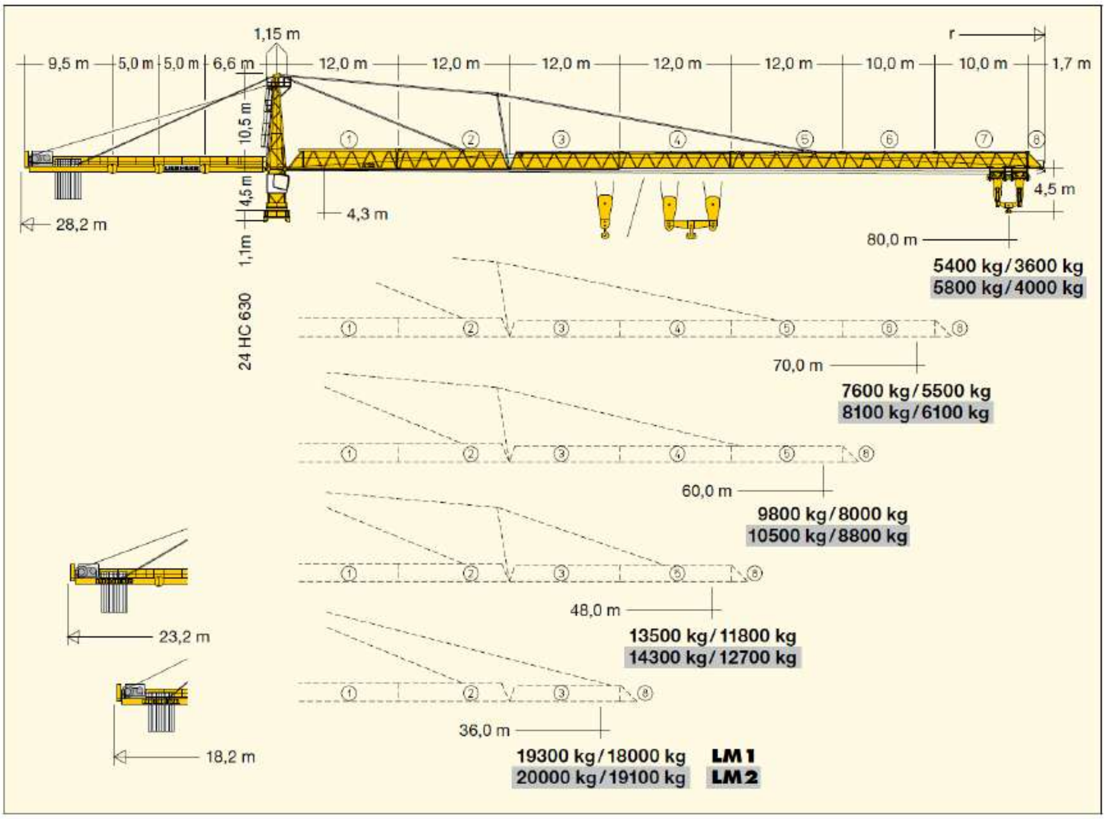
Елемент	***МОНТАЖНИ ПАРАМЕТРИ - кран №2***									***Проверка на избрания кран***			***Марка на крана***
	Монтажна маса			***Височина до куката***				***Обсег***					
	q_e	***q_m***	***$Q_м$***	***$H_м$***	***h_e***	***h_m***	***h_k***	***x***	***b***	***$Q_{кр}$***	***$h_{кр}$***	***$b_{кр}$***	
	[t]	*[t]*	*[t]*	*[m]*	*[m]*	*[m]*	*[m]*	*[m]*	*[m]*	*[t]*	*[m]*	*[m]*	
C-E10_2	9,90	0,50	10,40	-3,6	10,945	1,0	9,3	41,19	48,19	***12,10***	***67,5***	***60***	***Liebherr 630 EC-H 40 Litronic***
C_E18_2	9,90	0,50	10,40	-3,6	8,945	1,0	7,3	41,46	48,46	***12,00***			
C_C18_5	9,90	0,50	10,40	-3,6	8,945	1,0	7,3	28,59	35,59	***17,20***			
PB99	3,50	0,18	3,68	29,7	0,800	1,0	32,5	38,63	45,63	***12,80***			
PB37	2,45	0,12	2,57	17,7	0,540	0,5	19,7	34,65	41,65	***14,40***			
SB8	3,50	0,18	3,68	17,7	0,500	0,5	19,7	38,23	45,23	***12,90***			

<u>Liebherr 630 EC-H 40 Litronic</u>



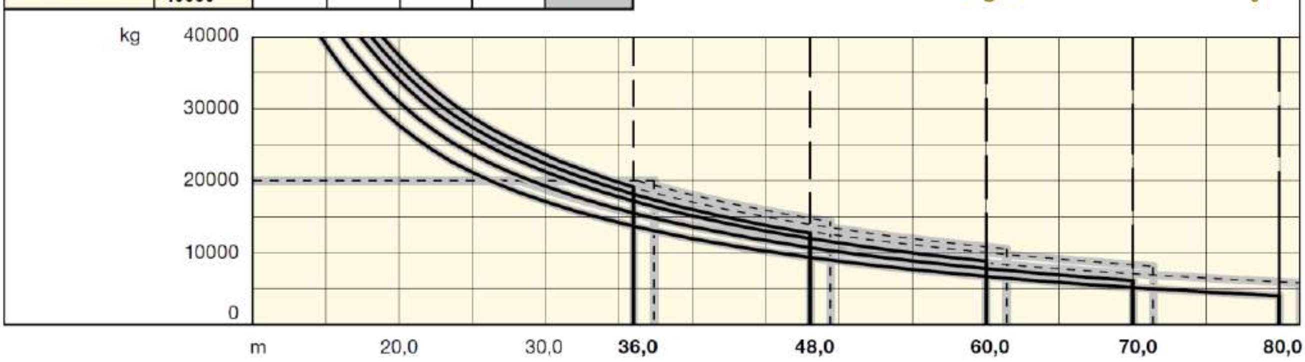
Ausladung und Tragfähigkeit

Radius and capacity/Portée et charge/Sbraccio e portata/Alcances y cargas/Alcance e capacidade de carga/Вылет и грузоподъемность



m r	m/kg	15,0	20,0	25,0	30,0	36,0	40,0	48,0	55,0	60,0	65,0	70,0	75,0	80,0
80,0 (r=83,4)	4,3 – 14,7 40000	38990	27740	21140	17090	13680	11950	9330	7650	6680	5860	5150	4540	4000
70,0 (r=73,4)	4,3 – 16,1 40000	40000	30970	23700	19250	15480	13580	10700	8850	7780	6880	6100		
60,0 (r=63,4)	4,3 – 17,4 40000	40000	33950	26070	21240	17150	15090	11960	9960	8800				
48,0 (r=51,4)	4,3 – 18,1 40000	40000	35680	27440	22390	18130	15970	12700						
36,0 (r=39,4)	4,3 – 18,9 40000	40000	37400	28810	23550	19100								

LM2



3. Occupational safety measures during construction

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Graphic part of the graduation project:

Sources:

Regulations:

- ***БДС EN 1990-1***
- ***БДС EN 1991-1-1***
- ***БДС EN 1991-1-4***
- ***БДС EN 1992-1-1***
- ***БДС EN 1993-1-1***
- ***БДС EN 1993-1-3***
- ***БДС EN 1993-1-8***
- ***БДС EN 1994-1-1***
- ***БДС EN 1997-1***
- ***БДС EN 1998-1-1***
- ***ASCE 7-16:*** Minimum design loads and associated criteria for buildings and other structures
- ***Standards for the design of pile foundations, 1992***
- ***Standards for design of flat foundations, 1996***

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- *Lecture course of „ Reinforced concrete structures“*
- *Lecture course of „Composite steel and concrete structures“*
- *Lecture course of „Construction technology“*

Used software products:

- *ETABS 2015*
- *SAP 2015*
- *AutoCAD 2022*
- *Design expert (sheems apl)*
- *TATA – Standard ComFlor*
- *LTBeamN*