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# BACHELOR THESIS

## (EXPLANATORY NOTE)

SPECIALTY 192 «BUILDING AND CIVIL ENGINEERING»

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**CHAPTER 1.**  
**ANALYTICAL REVIEW**

As far as there is a war in Ukraine since 24<sup>th</sup> of February 2022, a lot of cities all over the country have been seriously damaged. Especially rough were shelled Southern, Eastern and Northern Ukraine right along the Russian border. The condition of all types of buildings and structures in that regions are awful. For example, Chernihiv, Bucha, Irpin, Mariupol, Kharkiv, Kherson and lots of other smaller cities got more than 50% of numerous damages and total destructions, that requires an appropriate, full-fledged renovation and new construction. To be exact, as of May 29<sup>th</sup> - 2229 buildings in Kharkiv and Kharkiv region were totally destroyed.

The industrial buildings as themselves are designed to deploy some industrial enterprises in, which in turn will provide people with all the line of necessary goods they need as well as production of power, for instance, mining of raw materials, storage and production of plastics, metals and wood, textile and paper products, petroleum products and chemicals. So the industrial buildings are obviously super required wherever you are, at least so to create goods as it has been said above and to create working places where people can get their jobs. Soon or later in the time of war or right after its finish, people will have got a strict need of good-quality and available products which can fulfill their life requirements. Since the beginning of war we did clearly managed to feel the lack of some products a few times, such as fuel or some other significantly required goods for the military and humanitarian purposes, that were met by buying or getting as an international help from the other countries. Therefore the construction of such buildings will be especially demanded in Ukraine in the further 10-20 years with ahead standing purposes of renovating the economy and peoples` ordinary lives. And without any doubt industrial buildings will play a key role in these processes as well.

Saying about industrial buildings` advantages, it should be also said what types of these buildings and structures are. Industrial buildings in their most are classified to be single-storey, multi-storey and mixed storeys.

Nowadays, in modern industrial construction, mainly single-storey multi-span buildings are used. Talking about them, it shall be denoted that they are characterized

by a fairly easy organization of technological processes with the use of the most economical horizontal transport for goods moving, a simple system of control and management of the production process, good communication between production facilities, uniform lighting of lanterns, the ability to more easily comply with parameters and indoor air exchange.

One-storey buildings as well as multi-storey ones have in their most frame structural scheme. The decisive point in determining the feasibility of industrial buildings are delivering schemes of material movement and production process, as well as the conditions of rational placement of equipment. The technological scheme determines the spatial planning decisions of a building, so in this case, production processes should strive to be arranged horizontally (at one level) or vertically (one below the other). This arrangement makes it easier to create the necessary working and production conditions with the help of engineering support systems and the necessary lifting and transport equipment.

Since my project of industrial building from initial data has construction site in Kharkiv city, it gets definitely clear the necessarily of this building construction out there because of particular reasons of the circumstances mentioned in above statement. Moreover, taking into consideration the further production purposes of the industrial building we see, that the wooden structures will be manufactured there as well as the wood it self as the material that can be used then.

Wooden structures are light and strong enough. Their rot, fire and moisture resistance could be increased with fire-retardant coatings, moisture insulation and waterproofing tools, flame-retardants and antiseptics. The most important and certainly beneficial advantage of wooden structures are their ability to use local materials, so the nearby standing woods, for example, can be used as a raw material for the timber structures production. Their low bulk density, transportability, environmental friendliness and manufacturability are also great asset. Since the aged years people had found a lot of practical applications using these basic advantages of

wood, and nowadays improving it year in year, wood as well still stays pretty in demand.

The main areas of rational use for wooden structures are roof covering of some industrial buildings and structures (including chemically aggressive environments), public (starting with exhibition, sports and continuing with other buildings), agricultural, construction of bridges, cooling towers, overpasses, mines, buildings and structures in remote forest areas, seismic construction.

For the war time, wooden structures can be also used as temporary structures (bridges, warehouses...) or structures for special military purposes. On the factory would be able to designed and manufactured the quick-erected temporary houses made of wood for refugees and wounded casualties of the war, who have no place to go anymore.

**CHAPTER 2.**  
**ARCHITECTURAL PART**



## **2.1. General data**

The subject of my diploma project is «Industrial building for timber construction manufacturing in Kharkiv». It is actual for today because of the need of reconstruction of civil infrastructure including the industrial buildings in the destructed areas of the country.

Purpose of the project development is to design an appropriate industrial building for the wooden structures manufacture.

## **2.2. Natural conditions**

According to the ДСТУ Н Б В.1.1-27 2010. [1], Kharkiv belongs to the 2d Southern-East climate region (steppe), with an average winter temperature from -2 down to -6°C, and in summer from +21 up to +23°C. Annual precipitation level is 400-500mm; relative air humidity is less than 65%.

Average wind velocity in January is 4-6m/s, thus the city belongs to the 3d group of Ukraine`s zoning by wind velocity in January. The most repetitive wind direction in winter is Western and North-Western, whereas in summer - Southern.

## **2.3. Spatial planning and constructive decisions**

### **2.3.1. Administrative-common building**

A two-storied administrative-common building is placed in the axis "3 - 12", "J - L", and adjoins to the main industrial building of the enterprise at axis J. The building is of the frame system with the transverse location of the girders. Columns are installed according to the coordination size of the grid 6.0 x 6.0 m, in places where the stair cases are located, used a grid of columns 6.0 x 3.0 m [2, 3]. The dimensions of the building in plan are 54.84 x 12.42 m. Two constructive steps of 6.0 m wide were taken along the width of the building, and 9 constructive steps of 6.0 m wide were taken along its entire length; in the places of stair cases location additional

columns with a step of 3.0 m have been installed. The floors` height is 3.3 m. The total height of the building down from the soil surface and up to the top of the parapet is 7,5 m.

The enterprise of wooden structures production works in 3 shifts (100/100/50) where men`s part is - 70%.

The administrative-common building is calculated according to the total number of workers as well as for the maximum number of people that could be in a shift. There are next rooms in :

• **Sanitary-common rooms - wardrobes, showers, washrooms, toilets;**

The number of shower boxes and washbasins depends on the most numerical shift. (The opened shower boxes have dimension in plan 0,9 x 0,9 m and the area of cloakroom is equal to 0,7m<sup>2</sup> per 1 shower box)

The rating number of workers per 1 shower box – 5

Rating number of workers per 1 washbasin – 20

Thus, the number of showers and washbasins are equal:

Showers:  $30/5 = 6$  (for women);  $70/5 = 14$  (for men);

Cloakroom area for women:  $6*(0.81 + 0.7) \approx 9,06 \text{ m}^2$

Washbasins:  $30/20 \approx 1.5 = 2$  (for women);  $70/20 \approx 3.5 = 4$  (for men)

Cloakroom area for men:  $14*(0.81 + 0.7) \approx 21,14 \text{ m}^2$

The number of wardrobe boxes is determined according to the total number of workers.

$250-70\% = 75$  (for women)

$250-30\% = 175$  (for men)

The number of toilet depends on the most numerical shift. In industrial building 1 toilet per 18 men or 12 women. (1 washbasin per 4 toilets + hand dryer)

$70/18 \approx 3.8 = 4$  (for men)

$30/12 \approx 2.5 = 3$  (for women)

The room of women hygiene has dimensions in plan: 2.2 x 1.8 m.

• **Restrooms;**

The resting room area is 0,9 m<sup>2</sup> per 1 person of the most numerical shift, but not less than 4,0 m<sup>2</sup> .

$70 \cdot 0.9 = 63$  m<sup>2</sup> (for men)

$30 \cdot 0.9 = 27$  m<sup>2</sup> (for women)

• **Laboratories;**

Laboratory area: = 26,07m<sup>2</sup>

• **Dining room;**

A dining room is designed on the first floor and depends on the most numerical shift: from 13 to 200 workers. ( 1 seat per 4 workers, 1 washbasin per 15 seats)

$100/4 = 25$  (seats)  $\rightarrow 25/15 \approx 1.67 = 2$  (washbasins)

A dining room consists of hall-room, storeroom and room for washing dishes.

Number of seats: 25  $\rightarrow$

Hall-room are:  $\approx 44$  m<sup>2</sup>;

Storeroom area:  $\approx 11$  m<sup>2</sup>;

Room for washing dishes area:  $\approx 6$  m<sup>2</sup>;

Total:  $\approx 61$  m<sup>2</sup>

• **Medical room;**

The medical room is designed with an area 12 m<sup>2</sup> - 50...150 and 18 m<sup>2</sup> - 151...300 persons (according to the total number of workers.)

Medical room area:  $\approx 18$ m<sup>2</sup> ( = 17,23 m<sup>2</sup> )

• **Meeting room;**

Meeting hall area: 24 m<sup>2</sup> - 50...100 and 36 m<sup>2</sup> - 101...200 persons in terms of the most numerical shift.

Meeting hall area: = 28,63 m<sup>2</sup>

• **Auxiliary technical;**

Charwoman room area: = 4 m<sup>2</sup>

• **Warehouses and pantries.**

Archive room area: = 26,07 m<sup>2</sup>

Inventory room area: = 17,23 m<sup>2</sup>

• **Administrative offices;**

Public union room area: = 25,05m<sup>2</sup>

Chief accounting officer room area: = 35,98 m<sup>2</sup>

Chief technologist room area: = 35,98 m<sup>2</sup>

Construction department area: = 72,71 m<sup>2</sup>

Chief engineer`s room area: = 35,98 m<sup>2</sup>

Technical department room area: = 53,95 m<sup>2</sup>

Reception room area: 50,61 m<sup>2</sup>

Dispatcher`s room area: = 28,63 m<sup>2</sup>

In general, the spatial planning decision of the building meets the requirements of ДБН В.2.2-28: 2010 " Будинки адміністративного та побутового призначення".

### **2.3.2. Production building**

A single-storey industrial building is of the frame structural system is placed in the axes "A - H", "1 - 17". The spans of covering load-bearing structures are 24,0 m (blocks A) and 18,0 m (block B and C); the height to the bottom flange of the roof

trusses – 9,6 m (block A and B) and 10,65 m (block C). Axial dimensions of the building in plan are 90,890 x 42,540 m. The spatial planning decision of the building is classified as “hall-like”.

## **2.4. Architectural and constructive decisions**

### **2.4.1. Administrative-common building**

**Columns** - prefabricated reinforced concrete with section sizes 300 x 300 mm; which are of two storeys high; made of concrete class C20; installed on a grid of 6.0 x 6.0 m and 6.0 x 3.0 m.

**Girders** - prefabricated reinforced concrete T-shaped that are made of concrete class C20, girders depth is 450 mm, bottom width 400 mm. Girders are mounted on the transverse (digital) axes with hinged support on rectangular cantilever support brackets of the columns.

**Floors** are made of prefabricated reinforced concrete slabs 6.0 m long with a thickness of 220 mm; slabs are made of concrete class C12/15. Ordinary plates of 1200 mm wide are accepted with round cavities ; 1200 mm and 950 mm wide spacers have slots for the passage of vertical utilities. A sound-insulating layer of monolithic expanded clay concrete with a density of  $\rho = 1200 \text{ kg / m}^3$  and thickness - 50 mm are made all along the reinforced concrete floor panels.

**Stiffness diaphragms** are made of a silicate brick 120 mm width; Installed along the axis "7" in the axes "J - K"; on the axis "K" in the axes "7 - 8"; on the axis "8" in the axes "K - L". Stiffness diaphragms are connected to the columns and girders with electric welding through the embedded parts.

**External walls** are made of non-bearing hinged single-layer expanded clay concrete panels, concrete class C5 with density  $\rho = 1100 \text{ kg / m}^3$ ; the outer and inner surfaces of the panels are invoiced. The design thickness of the wall panels is 240 mm, the actual thickness is 200... 300 mm. Horizontal and vertical joints of panels are flat, sealed with cement-sand mortar and sealing elastic layers.

**Partitions** are made of a silicate brick with dimensions 250x120x65 mm. The surfaces of the partitions are plastered, some are lined with ceramic tiles or painted.

**Ceiling** is made of prefabricated reinforced concrete slabs with a thermal insulation layer of perlite concrete with a density of  $\rho = 600 \text{ kg / m}^3$  with thickness of 120 ... 25 mm, which is laid on the vapor barrier with a slope of 1.5 ... 3.0%.

**Roofing** consists of four layers of roofing material glued on bituminous mastic on a layer of asphalt 30 mm thick, on top of roofing material as the protective topping, gravel is used.

**Floor covering:**

- in communication rooms - mosaic polished concrete with addition of crushed marble pieces and dyes.
- in office premises - linoleum on a cement-sand screed;
- in some working offices and communication rooms - parquet on logs;
- in rooms with wet processes - ceramic tile on a cement-sandy mortar on a waterproofing layer;
- in the hall on the 1st floor - granite slabs;
- in the corridors - painted boards.

**Stairs** are made of prefabricated reinforced concrete U-shaped marches with semi-platforms (concrete class C16/20), which lean on reinforced concrete L-shaped girders welded to the columns.

**Windows** - strip metal-plastic with double-glazing and paired sash.

**Doors** - wooden board and metal.

#### **2.4.2. Production building**

**The frame** of the building in blocks A and B is made of prefabricated reinforced concrete elements (columns, trusses, crane girders...), whereas block C of

steel structures. Connection of columns with trusses - hinged, made with bolts and electric welding with clamping of columns in the foundations.

**Columns** in blocks A and B are reinforced concrete of continuous cross-section 300 x 300 mm and 400x800 mm. They are designed for crane-free production buildings, so made of concrete class C20/25; columns are rigidly clamped in the isolated foundation with socket-type pedestal; In block C columns are represented in the steel structures. They are rigidly fixed onto the isolated foundation with monolithic pedestal via anchors; the step of the load-bearing columns along the axes "14", "17", "A", "D" and "H" - 6 m. Framework columns are made of steel as well as reinforced concrete with a cross section of 300 x 300 mm.

**Covering load-bearing elements** in blocks A and B are prefabricated reinforced concrete trusses with a span of 24,0 m and 18,0 m, made of concrete class C20/25; whereas in block C – steel trusses. They are assembled along the axes "1 - 13" and "A - H" ( the last ones in perpendicular direction ); each truss consists of two prefabricated parts which are connected on a construction site at the special connection joints. For reinforced concrete ones these two parts are additionally reinforced on-situ.

**Ceiling** part of the covering structures is made of prefabricated reinforced concrete ribbed slabs with dimensions in plan - 1.5 x 6.0 m and 3.0x6.0 m (concrete class C20/25). Lite-saturating trapezoidal lighters are arranged on the roof; for today, the window openings of these lighters are closed with boards. Thermal insulation layer of the coating-mineral felt with a density  $\rho = 150 \text{ kg / m}^3$ .

**External walls** are made of single-layer expanded clay concrete hinged panels with a thickness of 240 mm, concrete class C8/10. Panel joints - flat, sealed with cement-sand mortar M25.

**Roofing** – waterproofing membrane 3 layers of bituminous felt with bituminous mastic; the top layer is waterproofing membrane that is laid under a high temperature

. Drainage of precipitation from the roof is carried out through a system of internal pipes through water intake funnels.

**The floor** within the axes "1 - 17" and "A - H" in the production premises is made of concrete in two layers. The first one is made of concrete screed class C10/15 – 100mm, and the second layer of concrete class C12/15 – 45mm. On the top - a polymer covering with thickness of 5 mm.

**Windows** - steel double or single glazing; in premises on the level + 7,800 - aluminum double-glazed windows.

**Doors** - aluminum with glazing or steel.

**Gates** – steel, swing-like, of frame type.

## **2.5. Technical and economic indicators**

### **2.5.1. Administrative-common building**

Building area:  $\approx 690,98 \text{ m}^2$ .

Construction volume:  $\approx 5631,49 \text{ m}^3$ .

Number of storeys: 2

### **2.5.2. Production building**

Building area:  $\approx 3866,44 \text{ m}^2$ .

Construction volume:  $\approx 70474,98 \text{ m}^3$ .

Number of storeys: 1

## **2.6. Class of consequences**

To figure out what class of consequences the building have, I have to calculate how many people could be at the factory at the same time.



100 workers (in the most numerical shift)

2 medical workers

~ 6 security guard

~ 3 buffet workers (cooks and waiters/waitresses)

~ 3 charwomen

1 chief technologist

1 chief engineer

1 head manager (chief)

According to the sanitary and hygienic requirements of working conditions in offices, the area of the room must be at least 6.0 m<sup>2</sup> per 1 workplace. It shall be emphasized that the workplaces must be located at a distance of at least 1 m from the wall that has a window, and 1.4 m from the wall without. The distance between the working places and side surfaces of computers should be at least 1.2 m.

Taking into consideration the above standing statement it is clear that the room can not be fully filled up with people that allow me to assume that about 1/3 part of any office room will be taken so to arrange passageways to the working places and to place some additional shelves. Therefore the number of possible workers in the offices are:

$\frac{2}{3} \times 72,71 = 48,5 \text{ m}^2$  ( could be used for working places ) / 6,0 m<sup>2</sup> (per 1 person)  $\approx$

8,07 = 8 engineers (in the construction department)

$\frac{2}{3} \times 53,95 = 36,0 / 6,0 = 6$  engineers (in the technical department)

$\frac{2}{3} \times 25,05 = 16,7 / 6,0 \approx 2,78 = 2$  public union workers

$\frac{2}{3} \times 50,61 = 33,8 / 6,0 \approx 5,63 = 5$  accountants

$\frac{2}{3} \times 28,63 = 19,1 / 6,0 \approx 3,18 = 3$  dispatchers

$\frac{2}{3} \times 35,98 = 24,0 / 6,0 = 4$  reception workers

Hence, the total number of people that could be at the factory is  $\approx 145$  workers. Meeting the requirements from the table 1. of ДСТУ Н. Б. В.1.2-16: 2013 "Визначення класу наслідків (відповідності) будівель та споруд" or ДБН В.1.2-14-2009 "Загальні принципи забезпечення надійності та конструктивної безпеки будівель, споруд, будівельних конструкцій та основ", the building belongs to the class of consequences CC2.

Production is safe, the internal environment is non-aggressive. Category of premises of the building on explosion-fire and fire danger "B" and "C".

**CHAPTER 3.**  
**DESIGNING PART**

### 3.1. Loads

#### 3.1.1. Permanent and dead loads

First of all, let`s define the  $\gamma_n$  coefficient, that is structural safety factor according to the class of consequence that the building has. I take the coefficient according to the ДБН В.1.2-14-2009 "Загальні принципи забезпечення надійності та конструктивної безпеки будівель, споруд, будівельних конструкцій та основ", table 5. Since the building, as it has been already determined before, has CC2 class of consequences, for the further calculations of important load bearing structures, those ones which have "A" category of responsibility (columns and trusses), which failure may lead to the total building`s or its part unserviceability, for ULS calculations has to be equal  $\gamma_n = 1.1$ .

After that I should put together all the permanent loads from the building parts, that will be forever installed into their designing positions and will additionally, by their weight, upload the building`s frame. Building upon the architectural part and all the decisions have been made up there, I can calculate the load values from both the roof and walls` part, taking into account the  $\gamma_{fm}$  coefficients, that similarly to  $\gamma_n$  is the structure additional safety factor but now by loading, considering that some materials could get heavier due to the weather or any other influences. For example, such as wool insulation plates weight increasing caused by moisture impact, or some imperfections that could occur at the factories while reinforced concrete structures production, etc. At the end not including this factor could lead to more loads applied to the structure and the risk of non-ability of the frame to bear those excessive loads.  $\gamma_{fm}$  coefficient has to be taken according to the ДБН В.1.2-2 2006. "Навантаження і впливи". The results of the permanent loads represented into the tables below:

Table 3.1

### Roof loads

Load type	Composition	Characteristic loads, kN m <sup>2</sup>	$\gamma_{fm}$	Design loads, kN m <sup>2</sup>	Remarks	
Permanent loads	Protective coat of gravel (t=15mm p=2000 kg/ m <sup>3</sup> )	0.30	1.30	0.39		
	Waterproofing membrane (3 layers of bituminous felt with bituminous mastic) (t=30mm p=400 kg/ m <sup>3</sup> )	0.12	1.30	0.16		
	Cement-sand screed M100 (t=30mm p=1500 kg/ m <sup>3</sup> )	0.45	1.30	0.59		
	Mineral wool plate (t=120mm p=150 kg/ m <sup>3</sup> )	0.18	1.20	0.22		
	1 layer of steam insulation	0.05	1.30	0.07		
	Reinforced concrete ribbed slab (t=300mm)	1.60	1.20	1.92		
	Panel of the monitor	0.15	1.05	0.16		
	End plate of the monitor	0.20	1.05	0.21		
	Steel frame of the monitor	0.10	1.05	0.11		
	Total		3.15		3.80	
	Including $\gamma_n=1,1$				<b>4.18</b>	<b>A=103.5m<sup>2</sup></b>

Table 3.2

### Wall loads

Load type	Composition	Characteristic loads, kN m <sup>2</sup>	$\gamma_{fm}$	Design loads, kN m <sup>2</sup>	Remarks
Permanent loads	Expanded clay concrete hinged panels (t=240mm C8/ 10)	0.86	1.10	0.95	
	Including $\gamma_n=1,1$			<b>1.04</b>	<b>A=56.25m<sup>2</sup></b>
	Steel double-glazing windows	0.26	1.10	0.29	
	Including $\gamma_n=1,1$			<b>0.31</b>	<b>A=16.20m<sup>2</sup></b>
	Brick wall (t=380mm p=1800kg/ m <sup>3</sup> )	6.84	1.10	7.52	
	Including $\gamma_n=1,1$			<b>8.28</b>	<b>A=18.03m<sup>2</sup></b>

Table 3.3

### Deal loads

Load type	Composition	Characteristic loads, kN m <sup>2</sup>	$\gamma_{fm}$	Design loads, kN m <sup>2</sup>	Remarks
Dead loads	Truss, (g=kl, k=0.006..0.01kN m <sup>2</sup> )	0.18	1.05	0.189	<b>A=108.00m<sup>2</sup></b>
	Braces, (g=0.04..0.06kN m <sup>2</sup> )	0.06	1.05	0.063	<b>A=108.00m<sup>2</sup></b>
	Column (g=0.3kN m <sup>2</sup> )	0.30	1.05	0.315	<b>A=54.00m<sup>2</sup></b>
	Total	0.54		0.567	
	Including $\gamma_n=1,1$			<b>0.624</b>	

### 3.1.2 Crane loads

Two underslung cranes that I have in the block C carry out a transshipment work of average intensity as well as technological pieces of work in mechanical shops, warehouses of finished products of the enterprises of building materials, and have 4K operating mode group.

According to the ДБН В.1.2-2 2006. “Навантаження і впливи”, paragraph 7 “Crane loads”, to calculate the vertical and horizontal loads for the underslung cranes I shall use the corresponding formulas:

$$D_{max/min} = \gamma_{fm} \psi F_{max/min} (\sum_{i=1}^n y_i) \gamma_n \quad (3.1)$$

$$H^{\pm} = \pm \gamma_{fm} \psi H (\sum_{i=1}^n y_i) \gamma_n \quad (3.2)$$

Where:

$$\gamma_{fm} = 1.085 \quad (\text{for 30 years})$$

$$\psi = 0.85 \quad (\text{for 1K-6K})$$

$$F_{max} = \left[ \left( \frac{(Q+m_t)(L_{cr}-L_{min})}{L_{cr}} \right) + \frac{m_{cr}-m_t}{2} \right] \left( \frac{1}{n_k} \right)$$

Where:

$$Q = 3.2t = 32kN \quad (\text{load-bearing capacity of the crane})$$

$$m_{cr} = 1.27t = 12.7kN \quad (\text{mass of the underslung cranes})$$

$$m_t = 0.47t = 4.7kN \quad (\text{mass of the trolley})$$

$$n_k = 2 \quad (\text{number of wheels from one side})$$

Hence, I can calculate the maximum and minimum force for one wheel:

$$F_{max} = \left[ \left( \frac{(32+4.7)(6-0)}{6} \right) + \frac{12.7-4.7}{2} \right] \left( \frac{1}{2} \right) = 20.35kN$$

$$F_{min} = (Q + m_{cr})/n_k - F_{max} = (32 + 12.7)/2 - 20.35 = 2kN$$

Now let`s determine what would be the ordinates of the wheels in different positions of the crane:

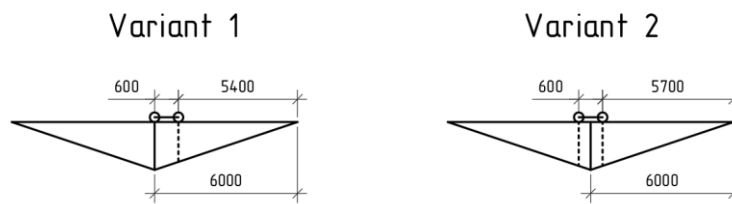
$$y_1 = y_2 = 1/6 * 5.7 = 0.95$$

$$\sum_{i=1}^n y_i = y_1 + y_2 = 0.95 + 0.95 = 1.9$$

$$y_3 = 1/6 * 5.4 = 0.95$$

$$\sum_{i=1}^n y_i = y_3 + 1 = 0.9 + 1 = 1.9$$

So as we can see there is no tough difference for this two particular cases in their ordinates.



**Fig.3.1.** Schemes of the ordinates of the wheels in different positions of the crane.

By now I can calculate the vertical forces that will be running from the crane right on the frame using the formula (3.1):

$$D_{max} = 1.085 * 0.85 * 20.35 * 1.9 * 1.1 = 39.22kN$$

$$D_{min} = 1.085 * 0.85 * 2.0 * 1.9 * 1.1 = 3.86kN$$

Therefore, the same way it has been done for the vertical forces, now I should also calculate it for the horizontal ones, using the formula (3.2):

$$H = 0.5(Q + m_t) = 0.5(32 + 4.7) = 19.7kN$$

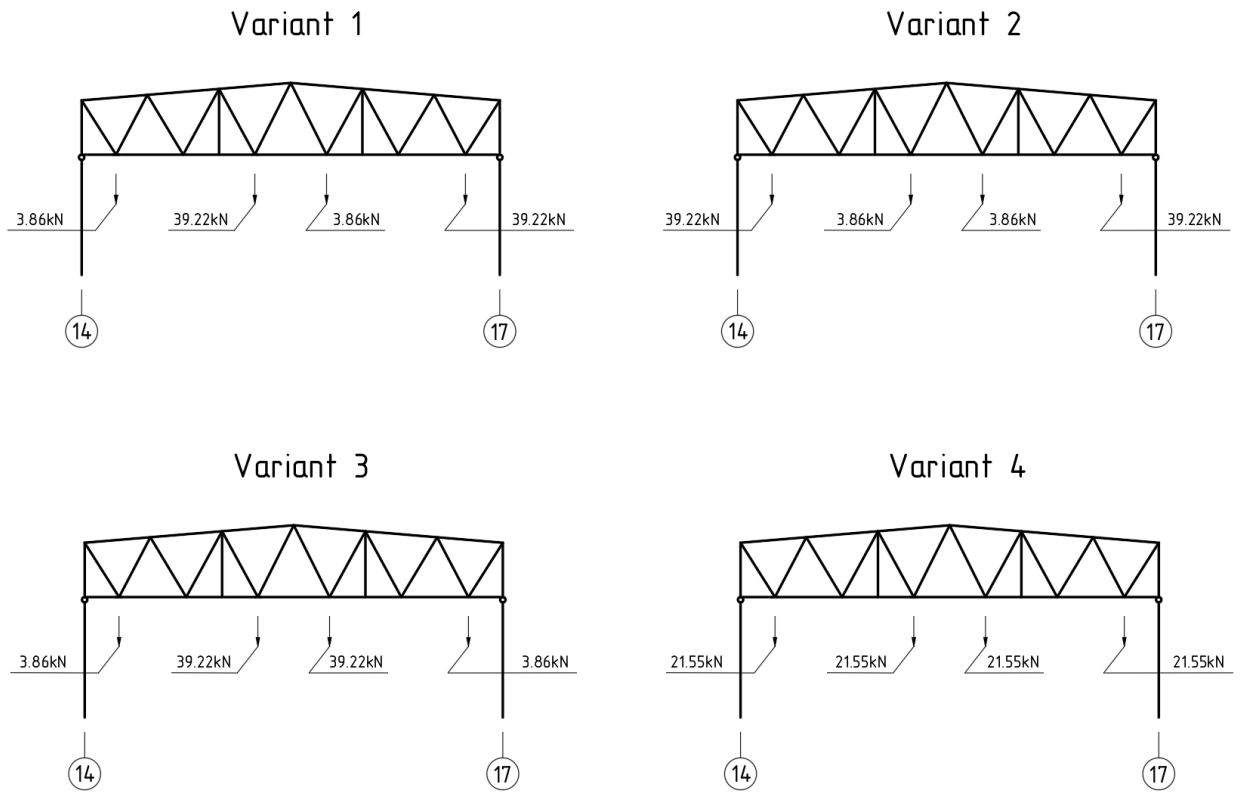
$$H^{\pm} = \pm 1.085 * 0.85 * 19.7 * 1.9 * 1.1 = \pm 37.97kN$$

Well, by this moment I have already determined the values of forces that appear because of the crane activity been in its critical right and left positions, as well as for the horizontal forces caused by braking of the trolley. After that, I should also check the central trolley position of the crane, where the load will be going equally to both the crane supports.

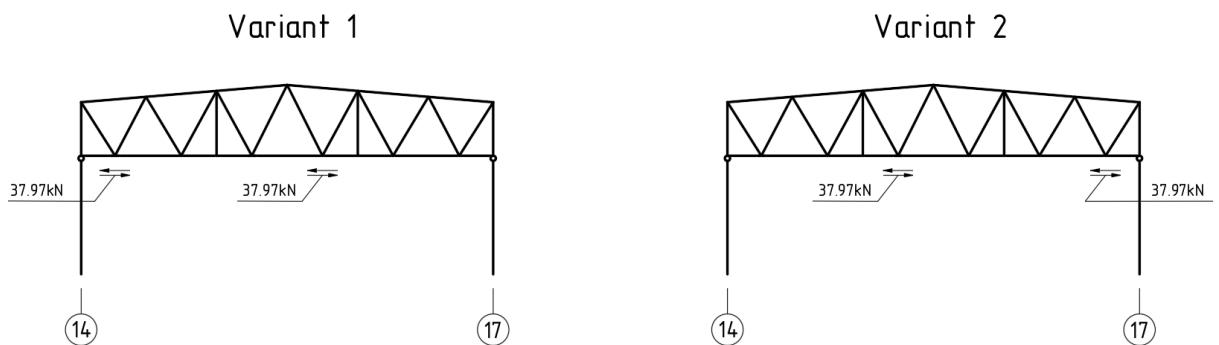
$$F_c = (Q + m_{cr})/2 = (32 + 12.7)/4 = 11.18kN$$



$$D_c = \gamma_{fm} \psi F_c (\sum_{i=1}^n y_i) \gamma_n = 1.085 * 0.85 * 11.18 * 1.9 * 1.1 = 21.55 kN$$



**Fig.3.2.** Schemes of the vertical crane loads application.



**Fig.3.3.** Schemes of the horizontal crane loads application.

### 3.1.3 Snow loads

According to the ДБН В.1.2-2 2006. “Навантаження і впливи”, paragraph 8 “Snow loads”, Kharkiv belongs to the 5th region of Ukrainian zoning map by snow

load with characteristic pressure value of 1600 Pa. So the design value of wind load could be calculated with the formula below:

$$S_m = \gamma_{fm} S_0 C \quad (3.3)$$

According to the Annex “B” an approximate serviceability live for industrial buildings is  $T = 60$  years, then  $\gamma_{fm} = 1.04$ .

$$S_0 = 1.6 \text{ kN/m}^2$$

$$C = \mu C_e C_{alt}$$

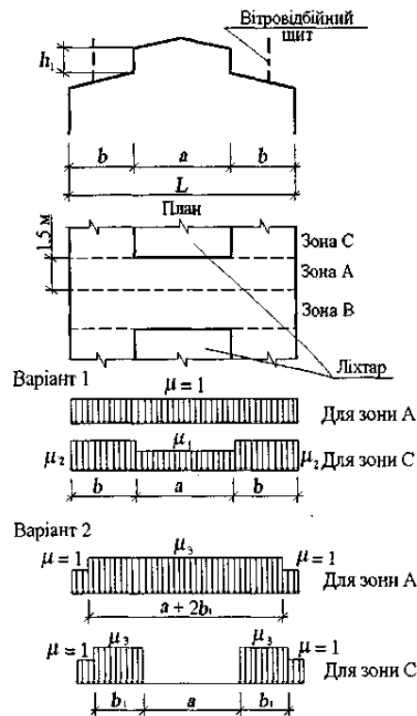
$$C_e = 1$$

$$C_{alt} = 1 \text{ (for } H = 0.152\text{km} < 0.5\text{km)}$$

Then formula (3.3) can be written as:

$$S_m = \gamma_{fm} S_0 \mu \quad (3.4)$$

$\mu$  - the coefficient is determined by Annex “Ж” depending on the roof shape and the scheme of distribution of snow load. In my case for the building with one longitudinal monitor placed in the middle of the building the 3d scheme is needed.



**Fig.3.4.** Scheme of snow load distribution variations over the roof.

Following the 3d scheme of the Annex “Ж”, I have to consider 2 variants of snow load distribution over the roof in the most loaded place right by the end of the monitor.-

$$\mu_1 = 0.8$$

$$\mu_2 = 1 + 0.1a/b = 1 + 0.1 * 6/6 = 1.1$$

$$\mu_3 = 1 + 0.5a/b_1 = 1 + 0.5 * 6/2.9 = 2.034$$

\*(but for reinforced concrete slabs, which I have over the steel truss, with a span of 6m,  $\mu_3$  has to be less or equal 2.0) Thus,  $\mu_3 = 2.0$

Having calculated the  $\mu$  coefficients for both the variants I can define the load values that will be acting on the roof, substituting already found coefficients into corresponding formula (3.4).

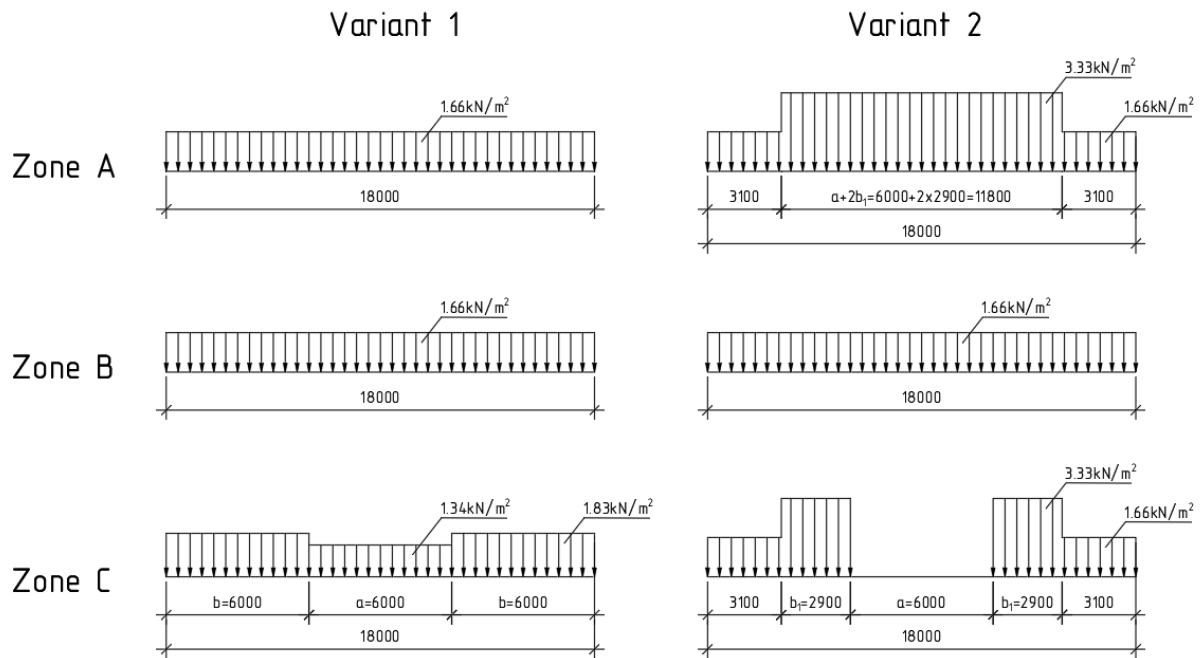
$$S_{m0} = \gamma_{fm} S_o \mu = 1.04 * 1.6 * 1.0 = 1.66 \text{ kN/m}^2$$

$$S_{m1} = \gamma_{fm} S_o \mu_1 = 1.04 * 1.6 * 0.8 = 1.34 \text{ kN/m}^2$$

$$S_{m2} = \gamma_{fm} S_o \mu_2 = 1.04 * 1.6 * 1.1 = 1.83 \text{ kN/m}^2$$

$$S_{m3} = \gamma_{fm} S_o \mu_3 = 1.04 * 1.6 * 2.0 = 3.33 \text{ kN/m}^2$$

Now let`s combine the load values with the diagrams of load distribution over the roof in it`s different zones and calculate the summary snow load that will act on the roof truss for both the variants.



**Fig.3.5.** Diagrams of snow load distribution variations over the roof.

Since the columns step is 6m and the most loaded truss is the second one from the end of the building, the linear snow loading for the truss from the different zones on the roof could be calculated as follows:

Width of zone A is 1.5m (from the building code), thus:

$$q_{0,A} = S_{m0} * a_1 = 1.66 * 1.5 = 2.49 \text{ kN/m}$$

$$q_{3,A} = S_{m3} * a_1 = 3.33 * 1.5 = 4.99 \text{ kN/m}$$

As far as the end frame of the building is displaced due to 0.5m columns` snap, the loading area from one side of the truss is 5.5m. However i have already loaded

some part of this area, that is zone A in fact, so the resulting width of zone B is  $(5.5/2) - 1.5 = 1.25m$ .

$$q_{0,B} = S_{m0} * a_2 = 1.66 * 1.25 = 2.08 \text{ kN/m}$$

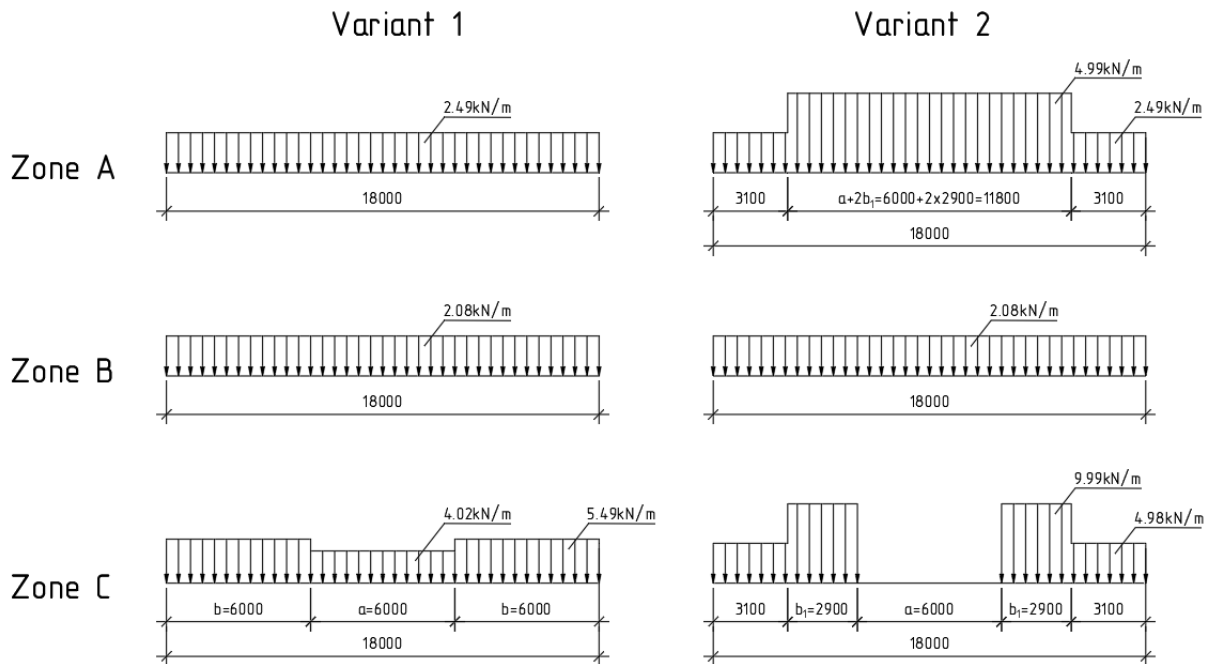
Width of zone C is 3m, thus:

$$q_{0,C} = S_{m0} * a_3 = 1.66 * 3.0 = 4.98 \text{ kN/m}$$

$$q_{1,C} = S_{m1} * a_3 = 1.34 * 3.0 = 4.02 \text{ kN/m}$$

$$q_{2,C} = S_{m2} * a_3 = 1.83 * 3.0 = 5.49 \text{ kN/m}$$

$$q_{3,C} = S_{m3} * a_3 = 3.33 * 3.0 = 9.99 \text{ kN/m}$$



**Fig.3.6.** Diagrams of linear snow load distribution for each zone over the roof.

Finally, I can sum up the loads from all the three zones for each variant and find the total linear snow load on the truss:

Variant 1:

$$q_{s,1} = (q_{0,A} + q_{0,B} + q_{2,C}) * \gamma_n = (2.49 + 2.08 + 5.49) * 1.1 = 11.07 \text{ kN/m}$$

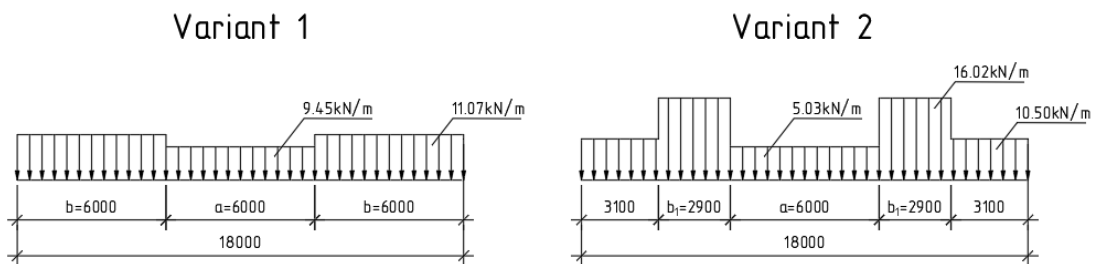
$$q_{s,2} = (q_{0,A} + q_{0,B} + q_{1,C}) * \gamma_n = (2.49 + 2.08 + 4.02) * 1.1 = 9.45 \text{ kN/m}$$

Variant 2:

$$q_{s,3} = (q_{0,A} + q_{0,B} + q_{0,C}) * \gamma_n = (2.49 + 2.08 + 4.98) * 1.1 = 10.50 \text{ kN/m}$$

$$q_{s,4} = (q_{0,A} + q_{0,B} + q_{3,C}) * \gamma_n = (2.49 + 2.08 + 9.99) * 1.1 = 16.02 \text{ kN/m}$$

$$q_{s,5} = (q_{0,A} + q_{0,B}) * \gamma_n = (2.49 + 2.08) * 1.1 = 5.03 \text{ kN/m}$$



**Fig.3.7.** Diagrams of linear designing snow load distribution over the roof.

### 3.1.4 Wind loads

According to the ДБН В.1.2-2 2006. “Навантаження і впливи”, paragraph 9 “Wind loads”, Kharkiv belongs to the 2th region of Ukrainian zoning map by wind load with characteristic pressure value of 430 Pa. So the design value of snow load could be calculated with the formula below:

$$W_m = \gamma_{fm} W_0 C \tag{3.5}$$

According to the Annex “B” an approximate serviceability live for industrial buildings is  $T = 60$  years, then  $\gamma_{fm} = 1.04$ .

$$W_0 = 0.43 \text{ kN/m}^2$$

$$C = C_{aer} C_h C_{alt} C_{rel} C_{dir} C_d \tag{3.6}$$

By the type of terrain surrounding the building, the object belongs to the 3rd district with suburban and industrial zones, as well as long forests. According to the Change № 1 of ДБН В.1.2-2:2006 "Система забезпечення надійності та безпеки

будівельних об'єктів. Навантаження і впливи. Норми проектування" (Approved by the Ministry of Regional Development and Construction of Ukraine dated August 13, 2007 № 143)  $C_h$  coefficients are equal:

$$C_{h,0} = 0.9 \text{ (for level 0.000)}$$

$$C_{h,1} = 1.22 \text{ (for level +10.650)}$$

$$C_{h,2} = 1.45 \text{ (for level +17.000)}$$

$$C_{aer}^+ = +0.8 \text{ (for windward side)}$$

$$C_{aer}^- = -0.6 \text{ (for leeward side)}$$

$$C_{alt} = 1 \text{ (for } H = 0.152\text{km} < 0.5\text{km)}$$

$$C_{rel} = 1$$

$$C_{dir} = 1$$

$$C_d = 0.95 \text{ (for buildings with steel frame)}$$

Then the formula (3.6) for C summary coefficients determination for both the windward and leeward sides at different levels can be written as:

$$C = C_{aer} C_h C_d \quad (3.7)$$

Having found all the coefficients, I can substitute the formula (3.7) into (3.5) and calculate the trapezoidal linear wind force acting on the column from both the windward and leeward sides. By the way it's impossible to calculate the wind load for one of the walls along axes 14, because it doesn't exist. It's so because of the block C location, which is adjacent to the other two blocks at the end and is connected with them through that wall.

Right wind:

$$W_{m,0}^r = \gamma_{fm} W_o C_{aer}^+ C_{h,0} C_d = 1.04 * 0.43 * 0.8 * 0.9 * 0.95 = 0.30 \text{ kN/m}^2$$

$$W_{m,1}^r = \gamma_{fm} W_o C_{aer}^+ C_{h,1} C_d = 1.04 * 0.43 * 0.8 * 1.22 * 0.95 = 0.42 \text{ kN/m}^2$$

$$q_0^r = W_{m,0}^r * (b_1 + b)/2 * \gamma_n = 0.3 * (5.5 + 6)/2 * 1.1 = 1.89 \text{ kN/m}$$

$$q_1^r = W_{m,1}^r * (b + b_1)/2 * \gamma_n = 0.42 * (6 + 5.5)/2 * 1.1 = 2.66 \text{ kN/m}$$

Left wind:

$$W_{m,0}^l = \gamma_{fm} W_o C_{aer}^- C_{h,0} C_d = 1.04 * 0.43 * (-0.6) * 0.9 * 0.95 = -0.23 \text{ kN/m}^2$$

$$W_{m,1}^l = \gamma_{fm} W_o C_{aer}^- C_{h,1} C_d = 1.04 * 0.43 * (-0.6) * 1.22 * 0.95 = -0.31 \text{ kN/m}^2$$

$$q_0 = W_{m,0}^l * (b_1 + b)/2 * \gamma_n = -0.23 * (5.5 + 6)/2 * 1.1 = -1.45 \text{ kN/m}$$

$$q_1 = W_{m,1}^l * (b + b_1)/2 * \gamma_n = -0.31 * (6 + 5.5)/2 * 1.1 = -1.96 \text{ kN/m}$$

I shall also figure out the wind load that will act over the point level +10.650. I think that I will be quite rational to simplify applying the distributed linear forces at that part (on the monitor and roof) with all the coefficients in there - into the nodal one, that will include forces from all over the height from the column's top point, to the ridge of the monitor from that side. Making this simplification enables me to avoid making useless and excessive calculations of the building's frame, applying all the wind forced with corresponding aero-dynamical and other coefficients to the upper part, that will not influence the result that much. However, instead, I create the simplified frame (without monitor) and apply the summary nodal force to the upper point of the column, where the truss will be attached, taking the common aero-dynamical coefficient for the windward side (+0.8), that will be not far away from the real ones, but with the small reserve.

Hence the load from the upper part for exactly this frame with corresponding loading area can be calculated as:

$$W_1^\pm = \gamma_{fm} W_o C_{aer}^\pm C_{h.av} C_d A \gamma_n \quad (3.6)$$

Right wind:

$$C_{h.av} = (C_{h.1} + C_{h.2})/2 = (1.22 + 1.45)/2 = 1.35$$

$$A_1 = (6.35 * 3.0) + (3.15 * 2.75) = 27.71 \text{ m}^2$$

$$W_1^+ = 1.04 * 0.43 * 0.8 * 1.35 * 0.95 * 27.7 * 1.1 = 13.98 \text{ kN (windward side)}$$

$$A_2 = (6.35 - 3.15) * 3.0 = 9.6 \text{ m}^2$$

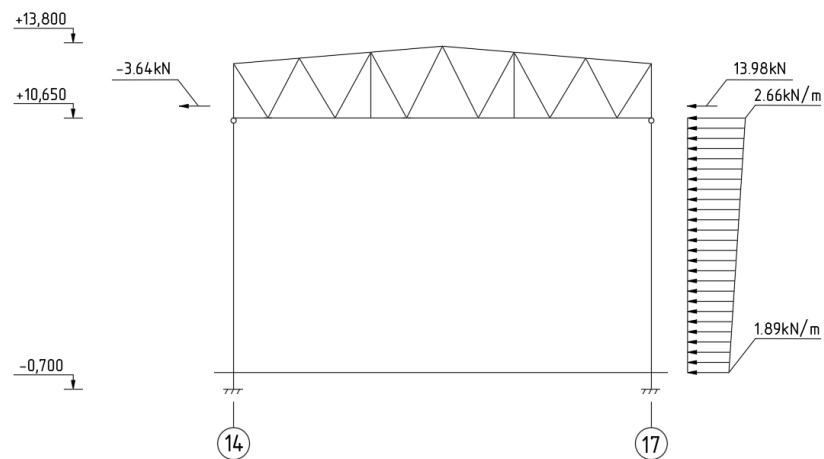


$$W_1^- = 1.04 * 0.43 * (-0.6) * 1.35 * 0.9 * 9.6 * 1.1 = -3.64kN \text{ (leeward side)}$$

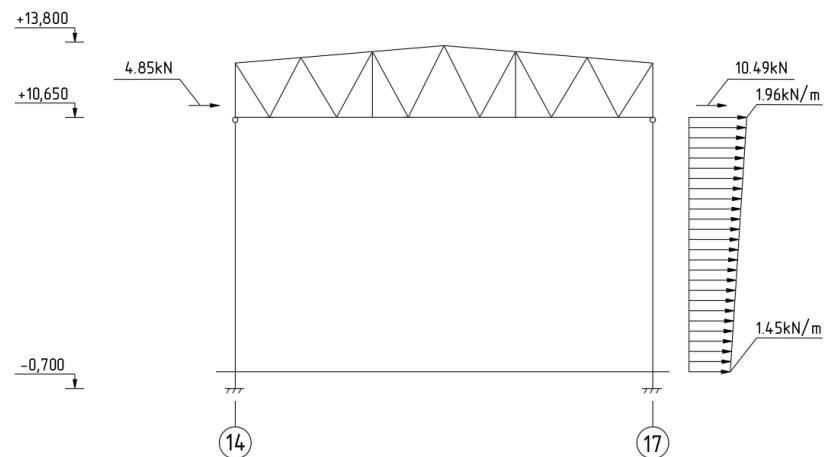
Left wind:

$$W_2^+ = 1.04 * 0.43 * 0.8 * 1.35 * 0.95 * 9.6 * 1.1 = 4.85kN \text{ (windward side)}$$

$$W_2^- = 1.04 * 0.43 * (-0.6) * 1.35 * 0.9 * 27.71 * 1.1 = -10.49kN \text{ (leeward side)}$$



**Fig.3.8.** Scheme of right wind loads distribution.



**Fig.3.9.** Scheme of left wind loads distribution.

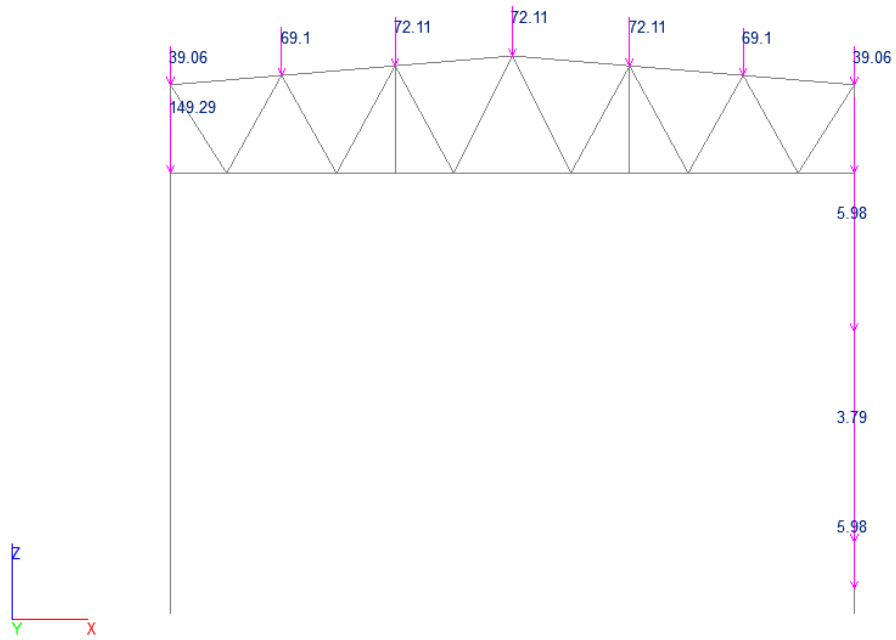
### 3.2. Frame calculation

As far as we are not in the ancient times no longer and have a possibility to calculate any buildings` structure using computer technologies, there is no such rough

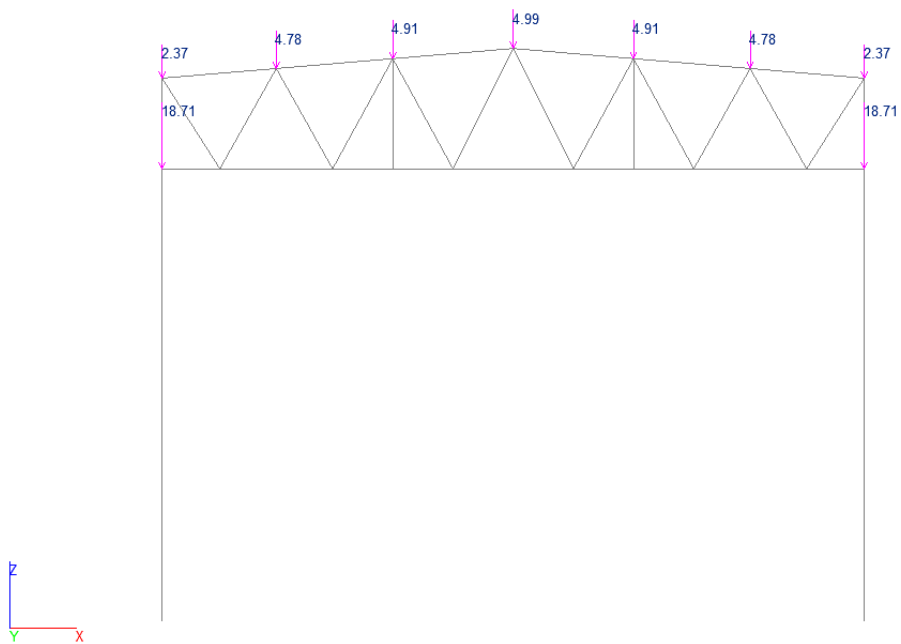
need for me to perform all the calculations manually. Therefore, the internal forces in the frame caused by the action of the loads, I will calculate with “SCAD Office” software.

Of course, it would be even easier and faster to perform these calculations using all the corresponding features and functions of the program. However, as far as I have already done it so to show these calculations in the previous paragraph about the loads, I decided to perform this part using all the coefficients so to get the designing loads action over the frame, so like to be able to calculate the frame both manually and mechanically (via the program). Hence, I should eliminate the loads I am applying in the program from multiplying them on the same coefficients once again, because it will lead to the doubling of loads` values and as a result higher internal forces.

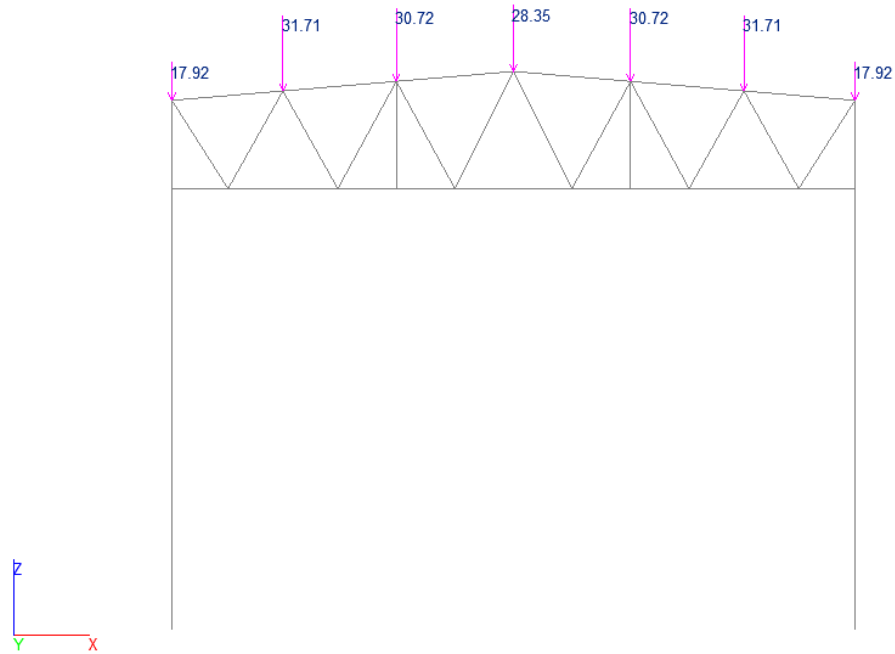
As far as I have determined the linear distributed permanent, dead and snow loads action over the truss, I have to rearrange them into the nodal ones because in case I apply the distributed forces to the truss, as they are shown on the schemes above right now, the truss will not work correctly. So these three schemes of permanent and snow loads application to the frame in SCAD software are represented on the following figures below:



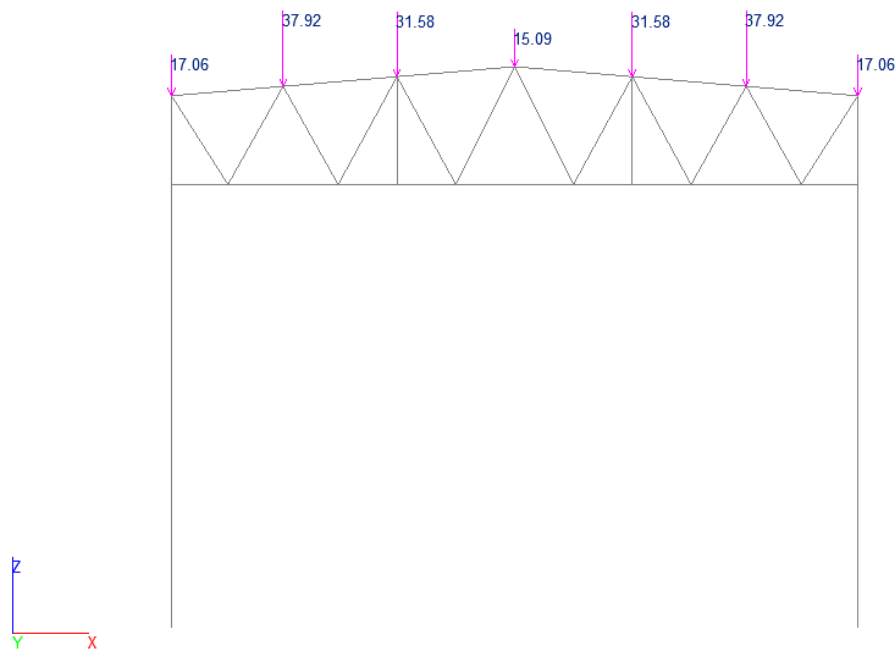
**Fig.3.10.** Scheme of permanent loads application.



**Fig.3.11.** Scheme of approximate dead loads application.



**Fig.3.12.** Scheme of snow load (variant 1) application.



**Fig.3.13.** Scheme of snow load (variant 2) application.

Once all the forces have been added to the scheme, I can push the bottom so the program calculate the frame, but the results I get after that will not be right,

because the loads I had added are represented in a few variants of the same load, as it is, for example, with snow, wind and crane loads. These all means that I have to make the program know that all these forces are to be calculated in tern to figure out what combination of them will create the most inappropriate and dangerous internal forces. Therefore, to make the program know about it I should set the table of design combinations of forces and displacements up. All the configures of this table are shown in the figure below:

Design Combinations of Forces and Displacements

Active loading	Active loading in DCD	Name	Type of loading	Type of load	Alternating	Involved in group operations			Safety factor	Fraction of duration	K <sub>1</sub>	K <sub>2</sub>	K <sub>3</sub>	
						Combinations	Mutual exclusions	Related loadings						
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Permanent	Permanent loads	Other	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	1	1	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Snow_1	Short-term loads	Full snow loads	<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	1	1	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Snow_2	Short-term loads	Full snow loads	<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	1	1	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Wind_1	Short-term loads	Wind loads	<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	1	0	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Wind_2	Short-term loads	Wind loads	<input type="checkbox"/>	<input type="checkbox"/>	<input checked="" type="checkbox"/>	<input type="checkbox"/>	1	0	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Crane_V1	Crane	Reduced vertical	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	1	0	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Crane_V2	Crane	Reduced vertical	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	1	0	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Crane_V3	Crane	Reduced vertical	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	1	0	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Crane_V4	Crane	Reduced vertical	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	1	0	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Crane_H1	Crane	Braking "on the k	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	1	0	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Crane_H2	Crane	Braking "on the r	<input checked="" type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	1	0	1	0	0	0
<input checked="" type="checkbox"/>	<input checked="" type="checkbox"/>	Dead	Permanent loads	Weight of steel s	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	1	1	1	0	0	0

Deactivate loading    DCF tree    Loadings cannot be included in combinations without    Delete DCF

Step of viewing loadings on plates: 15 degree    Report

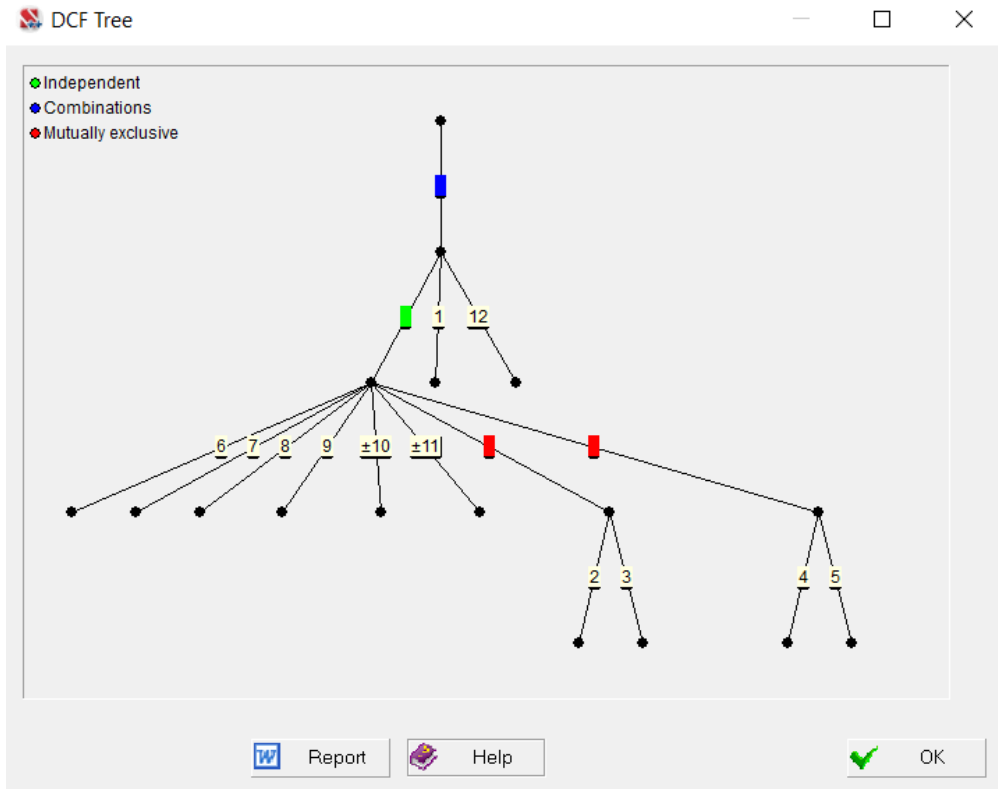
Parameters: List of elements, Unification, Groups

Relations between loadings: Combination, Related, Mutual exclusion, Cranes

Types of structures (accounting for seismic loads):  Civil and industrial,  Transport    2006

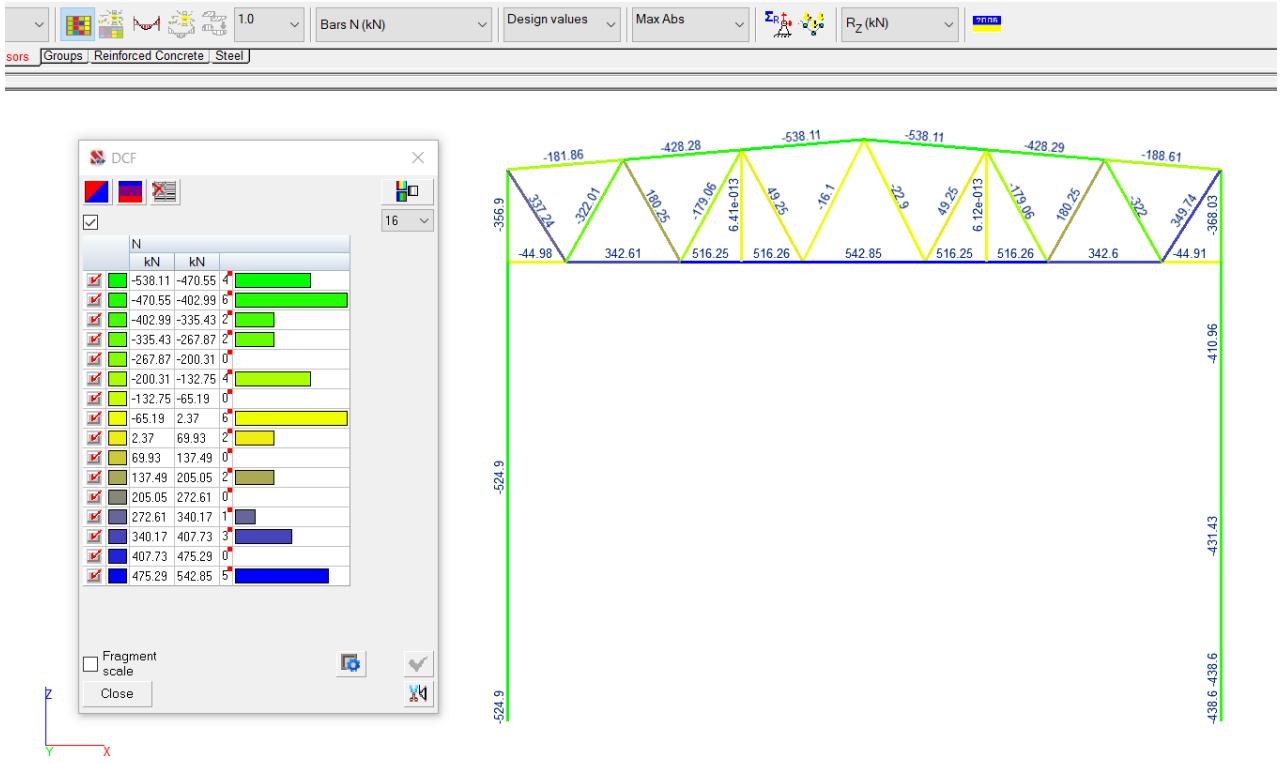
OK    Cancel    Help

**Fig.3.14.** Table of design combinations of forces and displacements.

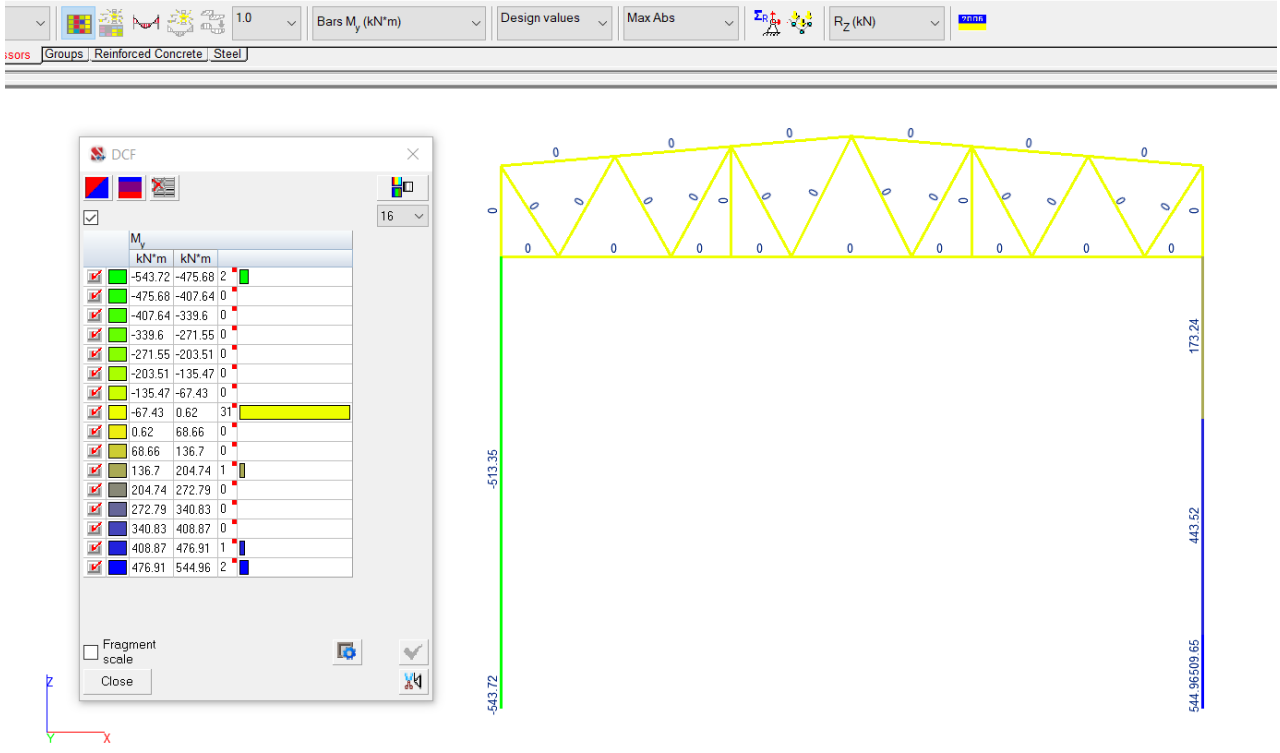


**Fig.3.15.** Tree of design combinations of forces.

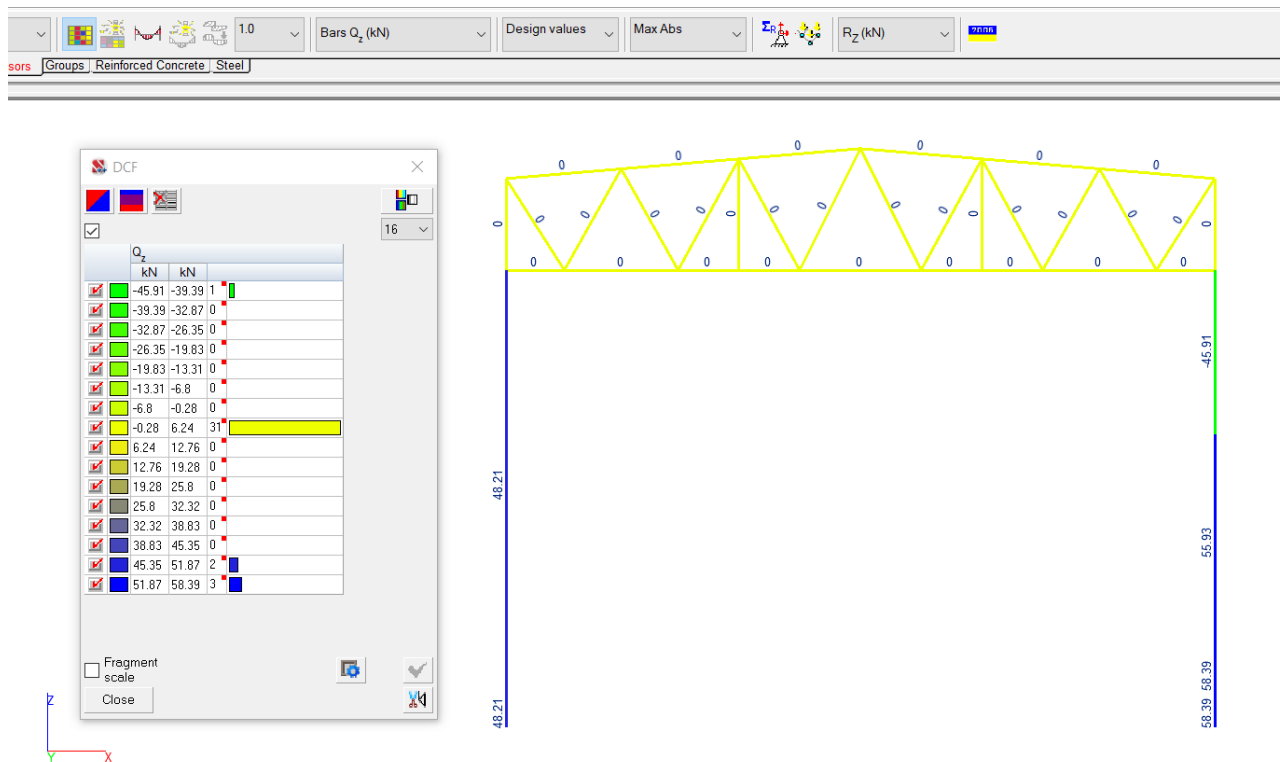
Once I have managed to set everything up correctly, I can push a “Calculate” button down and observe the results in the “Graphical analysis” folder. There on the page of “Postprocessors” I can find the maximum designing forces (N, M, Q), that occur inside the frame`s parts. The results of “Postprocessors” page are represented below:



**Fig.3.16.** Maximum design values of longitudinal normal forces, kN.



**Fig.3.17.** Maximum design values of bending moments, kNm.



**Fig.3.18.** Maximum design values of shear forces, kNm.

So, as it can be observed from the diagrams above, the truss works correctly, because there are no high values of shear forces as well as bending moments in the posts and webs of the truss. It means that those elements work in tension and compression only as they are to. Moreover, we can clearly see that on the Fig.3.17. Talking about the chords of the truss, the maximum values in the center of both top and bottom chords are about the same. It says once again that the truss works correctly and there is no mistake.

Now let's take a look on the columns. The internal forces seem to be right as well, because the left column, as it is seen, is more loaded, that was obviously clear from the very beginning. The clearance of this fact is that the left column is loaded with big permanent load from the brick wall, that has about 1.7-1.9 t/m<sup>3</sup> weight right over the column's top point, whereas the right column is exposed with less permanent load from the expanded clay concrete panels.



By the way, it is possible to check out what load combination causes the maximum bending moment and longitudinal force in any cross section of the frame. Let's see such one for the column, for example. On the Fig.3.19. it is seen that the load combination  $L1 + 0.9 * L2 + 0.95 * L9$  causes the biggest values of longitudinal forces inside the column.

Section	Criterion	N	M <sub>x</sub>	M <sub>y</sub>	Q <sub>z</sub>	M <sub>z</sub>	Q <sub>y</sub>	r <sub>x</sub>	Formula
		kN	kN*m	kN*m	kN	kN*m	kN	kN/m	
1	18	-524.9	-3.55e-003	-160.08	14.2	-0.02	-1.03e-004	0	L1+0.9*L2+0.9*L4+0.95*L9+L12
2	18	-524.9	-3.55e-003	-155.61	14.2	-0.02	-1.03e-004	0	L1+0.9*L2+0.9*L4+0.95*L9+L12
3	18	-524.9	-3.55e-003	-151.14	14.2	-0.02	-1.03e-004	0	L1+0.9*L2+0.9*L4+0.95*L9+L12
1	26	-524.9	-1.79e-003	91.7	-8.13	-0.02	-1.2e-004	0	L1+0.9*L2+0.9*L5+0.95*L9+L12
2	26	-524.9	-1.79e-003	89.14	-8.13	-0.02	-1.2e-004	0	L1+0.9*L2+0.9*L5+0.95*L9+L12
3	26	-524.9	-1.79e-003	86.58	-8.13	-0.02	-1.2e-004	0	L1+0.9*L2+0.9*L5+0.95*L9+L12
1	2	-524.88	-3.66e-003	-161.18	14.29	-0.02	-1.06e-004	0	L1+0.9*L2+0.9*L4+0.95*L8+L12
1	29	-524.88	-3.43e-003	-158.98	14.1	-0.02	-1.e-004	0	L1+0.9*L2+0.9*L4+0.95*L7+L12
2	2	-524.88	-3.66e-003	-156.68	14.29	-0.02	-1.06e-004	0	L1+0.9*L2+0.9*L4+0.95*L8+L12
2	29	-524.88	-3.43e-003	-154.54	14.1	-0.02	-1.e-004	0	L1+0.9*L2+0.9*L4+0.95*L7+L12
3	2	-524.88	-3.66e-003	-152.18	14.29	-0.02	-1.06e-004	0	L1+0.9*L2+0.9*L4+0.95*L8+L12
3	29	-524.88	-3.43e-003	-150.1	14.1	-0.02	-1.e-004	0	L1+0.9*L2+0.9*L4+0.95*L7+L12
1	28	-524.88	-1.9e-003	90.61	-8.03	-0.02	-1.23e-004	0	L1+0.9*L2+0.9*L5+0.95*L8+L12
2	28	-524.88	-1.9e-003	88.08	-8.03	-0.02	-1.23e-004	0	L1+0.9*L2+0.9*L5+0.95*L8+L12
3	28	-524.88	-1.9e-003	85.55	-8.03	-0.02	-1.23e-004	0	L1+0.9*L2+0.9*L5+0.95*L8+L12
1	6	-524.5	-1.65e-003	93.02	-8.25	-0.02	-1.16e-004	0	L1+0.9*L3+0.9*L5+0.95*L7+L12
2	6	-524.5	-1.65e-003	90.47	-8.25	-0.02	-1.16e-004	0	L1+0.9*L3+0.9*L5+0.95*L7+L12

Fig.3.19. Table of design combination in the elements.

### 3.2.1. Column calculation

To calculate the column I have to define the maximum internal forces that could occur inside its cross-section. Using the resultant diagrams of the frame calculation from the previous paragraph, the maximum absolute resultant internal forces are:

$$N = 525kN$$

$$M = 545kNm$$

Now let's take a look on what I have to calculate and how. Since the column is exposed to resist bending moment along its entire cross-section, it means that the

column is eccentrically compressed and has to be calculated in a corresponding manner. Basically the column can failure in 3 main ways. The first one is due to not enough strength that is caused by high stresses values. The second one is that the column can lost its stability in one of the axes either in plane of bending moment action or out of plane. In addition, the last but not least, could occur because of the local stability loss.

As far as to check the strength of the column, I have to know its cross-section area, which I am to find now. Therefore, I start with insuring of global stability of the column in both the planes, so to get the cross-section area out of this equation. So as the first step I shall determine the rating length in both the planes which depends on the real column length and the way it is fixed in space.

$$l_{ef,x} = \mu_1 l = 2 * 11.28 = 22.56m$$

$$l_{ef,y} = \mu_2 l = 1 * (11.28/2) = 5.64 m \quad (\text{struts are placed in the middle})$$

The calculation of eccentrically compressed columni with a constant cross-section, that are recommended, by the way, to be used in such frames with underslung cranes, should be done with a corresponding formula below:

$$N/(\varphi_c A) \leq R_y \gamma_c \tag{3.7}$$

Thus, the necessary cross section area could be found out from the formula (3.7):

$$A_{nec} = N/(\varphi_c R_y \gamma_c) \tag{3.8}$$

Where:

$$R_y = 24kN/cm^2 \quad (\text{for steel C245})$$

According to the ДБН В.2.6-198 2014. “Сталеві конструкції.” table 5.1, the coefficient of work conditions  $\gamma_c = 1.05$ , for columns of single storey industrial buildings.

As well as the coefficient of work conditions  $\gamma_c$ , coefficient of critical stresses for eccentric compression  $\varphi_c$  is determined with the ДБН В.2.6-198 2014. “Сталеві

конструкції.”, table Ж.3. The main parameters that influence its value are conditional flexibility and relative eccentricity, which can be calculated as follows:

$$\bar{\lambda}_x = \bar{\lambda} \sqrt{R_y/E} = (l_{ef.x}/i_x) \sqrt{R_y/E} = (2256/21) \sqrt{240/(2.06 * 10^5)} = 3.67$$

$$i_x = 0.42h = 0.42 * 50.0 = 21.0cm \quad (\text{for I-section})$$

$$m_{ef} = m_x \eta = 5.93 * 1.25 = 7.41$$

$$\rho_x = W_x/A = 0.35h = 0.42 * 50.0 = 17.5cm \quad (\text{for I-section})$$

$$m_x = M/(N\rho) = 54500/(525 * 17.5) = 5.93$$

$$A_f/A_w = 0.5$$

Using the table Ж.2. of ДБН В.2.6-198 2014. “Сталеві конструкції.”, value of the coefficient of cross-section influence for the 5<sup>th</sup> type of it is  $\eta = 1.25$ .

Thus,  $\varphi_c = 0.132$ .

Hence, the necessary cross-section area in terms so to provide the global stability for the column with an approximate parameters should be calculated with the formula (3.8):

$$A_{nec} = 525/(0.132 * 24 * 1.05) = 157.82cm^2$$

Now having found the necessary cross-section area, I can determine what would be the thickness and width of both column`s flanges and web, so to provide the cross-section with an ability to not experience the local buckling under the applied loads.

Based on the fact that the local buckling in plates should not occur earlier then the loss of global stability of the element, the conditions for local buckling and flexibility of both the web and flanges are written as so to take into account the flexibility of the entire element.

Let`s start with a web. The web is considered not to experience local buckling if it`s conditional flexibility (formula 3.9) doesn`t exceed the limiting one.

$$\bar{\lambda}_w = (h_{ef}/t_w)\sqrt{R_y/E} \quad (3.9)$$

The formula to calculate the limiting value of conditional flexibility for the web is determined according to the table 10.3 of ДБН В.2.6-198 2014. “Сталеві конструкції.”:

$$m_x = 6.43 \geq 1.0$$

$$\bar{\lambda}_x = 3.67 \geq 2.0$$

$$\bar{\lambda}_{uw} = 1.2 + 0.35\bar{\lambda}_x \leq 3.1$$

$$\bar{\lambda}_{uw} = 1.2 + 0.35 * 3.67 = 2.48$$

Let's assume that the thickness of the column's flanges are 20mm, then the web's depth is 460mm. Now using the formula (3.9) for conditional flexibility determination of the column's web, I can calculate the web's thickness taking into account the limiting value of the web's flexibility. Then the formula (3.9) looks:

$$t_w = (h_{ef}/\bar{\lambda}_{uw})\sqrt{R_y/E} = (46/2.48)\sqrt{240/(2.06 * 10^5)} = 0.63cm$$

Accept:  $t_w = 10mm$ .

Then the flange dimensions are found as:

$$A_w = h_w t_w = 46 * 0.8 = 36.08cm^2$$

$$A_f/A_w = 0.5$$

$$A_f = 0.5(A_{nec} - A_w) = 0.5 * (157.82 - 36.08) = 60.87cm^2$$

$$b_f = A_f/t_f = 60.87/2.0 = 30.44cm$$

Accept:  $b_f = 320mm$ , according to the section steel plates cutting pattern.

Now, having found the dimensions for both flanges, I can check them out for local buckling. To ensure that the flange will not pop out, the ratio of its hanging part width  $b_{ef}$  to the thickness  $t_f$  should not exceed some certain limits. Thus, for an unsupported I-beam flange, the limiting ratio is determined with a formula:

$$(b_{ef}/t_f) \leq (0.36 + 0.1\bar{\lambda}_x)\sqrt{E/R_y} \quad (3.10)$$

$$b_{ef} = (b_f - t_w)/2 = (32 - 0.8)/2 = 15.6\text{cm}$$

$$(15.6/2.0) = 7.8 < (0.36 + 0.1 * 3.67)\sqrt{(2.06 * 10^5)/240} = 21.30$$

Hence, as it is seen, there is everything okay with local stability of flanges as well.

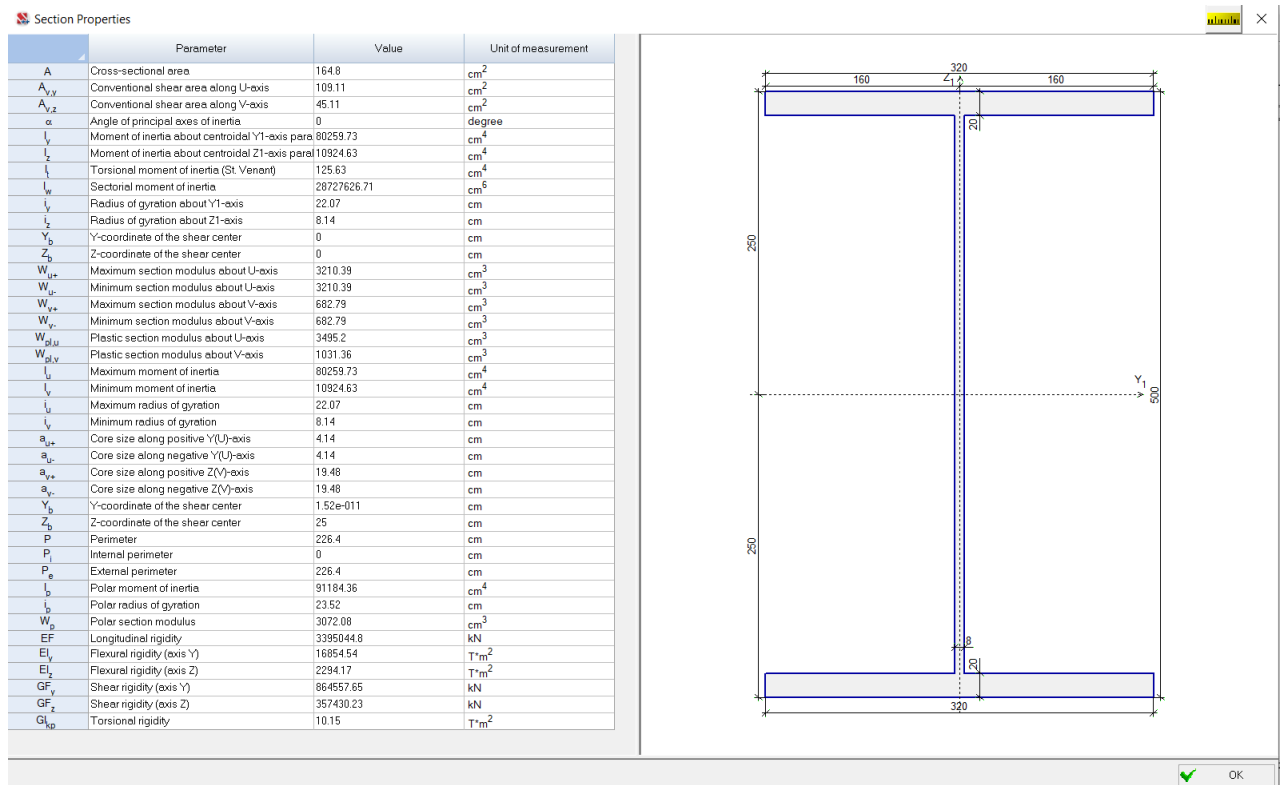


Fig.3.20. Table with geometrical characteristics of column's cross-section.

Now, I should recalculate everything up to check if the cross-section meets all the above-standing requirements:

$$A = 164.8\text{cm}^2$$

$$I_x = 80259.73\text{cm}^4$$

$$I_y = 10924.63\text{cm}^4$$

$$W_x = 3210.39\text{cm}^3$$

$$i_x = 22.07 \text{ cm}$$

$$i_y = 8.14 \text{ cm}$$

$$\bar{\lambda}_x = (l_{ef,x}/i_x)\sqrt{R_y/E} = (2256/22.07)\sqrt{240/(2.06 * 10^5)} = 3.49$$

$$m_x = (MA)/(NW_x) = (54500 * 164.8)/(525 * 32010.39) = 5.33$$

$$A_f/A_w = (32.0 * 2.0)/(46.0 * 0.8) = 1.74$$

Using the table Ж.2. of ДБН В.2.6-198 2014. “Сталеві конструкції.” and the recalculated values above, I can find the coefficient  $\eta$  for the 5<sup>th</sup> type of cross-section, that is equal:

$$\eta = (1.9 - 0.1m_x) - 0.02(6 - m_x)\bar{\lambda}_x$$

$$\eta = (1.9 - 0.1 * 5.33) - 0.02 * (6 - 5.33) * 3.49 = 1.32$$

$$m_{ef} = m_x\eta = 5.33 * 1.32 = 7.04$$

Thus,  $\varphi_c = 0.137$

$$\bar{\lambda}_{uw} = 1.2 + 0.35 * 3.49 = 2.42$$

$$\bar{\lambda}_w = (h_{ef}/t_w)\sqrt{R_y/E} = (46/0.8)\sqrt{240/(2.06 * 10^5)} = 1.97$$

$$\bar{\lambda}_{uw} = 2.42 > \lambda_w = 1.97$$

Checking up the stability in plane with (3.7) formula:

$$N/(\varphi_c A) = 525/(0.137 * 164.8) = 23.36 \text{ kN/cm}^2 < R_y = 24 \text{ kN/cm}^2$$


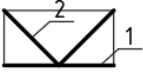


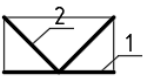




### 3.2.2. Bracing system

A bracing system in my project is made so to join the frames into one common structure, which will be able to resist both longitudinal and lateral forces from different directions. Moreover, the system carries functions of load effects contribution distribution, as well as providing restraint to compressed flanges or chords of trusses where they would otherwise be free to buckle laterally.

The results of braces selection by the conditional flexibility are shown in the table 3.4. below.

Table 3.4

### Braces

Name	Sketch	Remarks	$\lambda_u$	$l_x$ [cm]	$l_y$ [cm]	Necessary	Cross-section, [cm]	Real	
						$i_x/i_y$ [cm]		$i_x/i_y$ [cm]	
VB1		-	300	425	850	1.42/2.83	95x5	3.19/3.19	
VB2		1		200	225	550	1.13/2.75	2L60x5	1.84/2.84
		2		200	389	389	1.95/1.95	L70x5	2.16/2.16
VB3		1		200	300	600	1.50/3.00	2L70x5	2.16/3.14
		2		200	424	424	2.12/2.12	L70x5	2.16/2.16
LB1		-	200	600	600	3.00/3.00	95x5	3.19/3.19	
LB2		-	400	600	600	1.50/1.50	53x3	1.77/1.77	

### 3.2.3. Truss calculation

Before calculating the truss I have to define the maximum internal forces that could occur inside its cross-section. Using the resultant diagrams of the frame calculation from the previous paragraph, the maximum absolute resultant internal forces including the dead load are:

$$N_1 = 539kN \quad (\text{for top chord / compression})$$

$$N_2 = 543kN \quad (\text{for bottom chord / tension})$$

$$N_3 = 369kN \quad (\text{for webbing / compression})$$

$$N_4 = 350kN \quad (\text{for webbing / tension})$$

Besides that, the effective lengths of the truss members shall be found. These are needed to prevent the compressed elements buckling either in plane or out of it due to the forces action. Even though a system of bracings and struts are arranged between the frames, it doesn't mean that the truss members can't pop out. This system just adds the inflection points which decrease actual length where the elements can

loss their stability. However unless the cross-section of the element have enough radius of gyration (where the rigidity of the element is low), in addition to a big length of the element, as a result cause a high flexibility and then leads to buckling in this plane. Thus, to insure this never happen, I should provide the truss members with such cross-sections, so they would have enough rigidity to not buckle out in both directions under the loading.

Let`s start with bottom chord`s cross-section selection:

$$l_{ef.x} = 3m$$

$$l_{ef.y} = 6m$$

Necessary cross-section area by strength requirements can be calculated with the formula below:

$$A_{nec} = N/(R_y \gamma_c) \quad (3.11)$$

According to the ДБН В.2.6-198 2014. “Сталеві конструкції.”, the coefficient of work conditions  $\gamma_c = 0.95$ , for main tensioned elements of steel truss.

$$R_y = 24kN/cm^2 \quad (\text{for steel C245})$$

$$A_{nec} = N/(R_y \gamma_c) = 543/(24 * 0.95) = 23.82cm^2$$

Accept: 2 angles of 70x10 (ГОСТ 8509-93), with cross-section area of one piece:

$$A_s = 13.11cm^2$$

$$A_{b.c.} = 2A_s = 2 * 13.11 = 26.22cm^2 > A_{nec} = 23.82cm^2$$

$$i_x = 2.1cm$$

$$i_y = 3.34cm$$

According to the ДБН В.2.6-198 2014. “Сталеві конструкції.” table 13.10, he limiting value of conditional flexibility for the tensioned bottom chord of the truss shall be no more then  $\lambda_u = 400$ .

$$\lambda_x = l_{ef.x}/i_x = 300/2.10 = 142.86 < 400$$



$$\lambda_y = l_{ef.y}/i_y = 600/3.34 = 179.64 < 400$$

Now let`s do the same but for the top chord:

$$l_{ef.x} = 3m$$

$$l_{ef.y} = 6m$$

Necessary cross-section area by strength requirements can be calculated with the formula (3.12) below:

$$A_{nec} = N/(\varphi R_y \gamma_c) \quad (3.12)$$

According to the ДБН В.2.6-198 2014. “Сталеві конструкції.” table 5.1, the coefficient of work conditions  $\gamma_c = 0.8$ , for main compressed elements of steel truss.

$$R_y = 24kN/cm^2 \quad (\text{for steel C245})$$

Value of the  $\varphi$  coefficient is taken according to the pre-determined flexibility, which should be less than the limit one: for chords and support webbing members  $\lambda = 80 \dots 100$ . Then, according to the table, for  $\lambda = 100$  and  $R_y = 240MPa$ , coefficient of  $\varphi = 0.542$

$$A_{nec} = N/(\varphi R_y \gamma_c) = 539/(0.542 * 24 * 0.8) = 51.79cm^2$$

Accept: 2 angles of 140x10 (ГОСТ 8509-93), with cross-section area of one piece

$$A_s = 27.33cm^2$$

$$A_{t.c.} = 2A_s = 2 * 27.33 = 54.66cm^2 > A_{nec} = 51.79cm^2$$

$$i_x = 4.33cm$$

$$i_y = 6.11cm$$

$$\lambda_x = l_{ef.x}/i_x = 300/4.33 = 69.28$$

$$\lambda_y = l_{ef.y}/i_y = 600/6.11 = 98.19$$

Then taking the biggest value of flexibility,  $\varphi = 0.555$ .

Therefore, stresses that will occur in the cross-section, could be calculated as:

$$\sigma_{max} = N/(\varphi A) \leq R_y \gamma_c$$

$$\sigma_{max} = 539/(0.555 * 54.66) = 17.77 kN/cm^2 < 24 * 0.8 = 19.2 kN/cm^2$$

Limiting values of conditional flexibility for compressed elements are set to eliminate the influence of unwilling eccentricities and are determined via applied load with the corresponding characteristic coefficient:

$$\alpha = N/(\varphi A R_y \gamma_c) = 539/(0.555 * 54.66 * 24 * 0.8) = 0.93$$

$$\lambda_u = 180 - 60\alpha = 180 - 60 * 0.93 = 124.2 > \lambda_y = 98.19$$

And the last but not least that has left, is to define the webbing members cross-section. As far as the maximum tension and compression forces are about the same, I assume it is possible to design these elements with the same cross-section all over the truss. However, as far as the most loaded element isn't the longest one and the longest - isn't the most loaded. Thus, I will take an average length of the webbing elements so to include that the elements could be longer but less loaded. Moreover, the cross-section is more subjected to break or buckle out in compression, hence, let's start with selecting a cross-section for this condition:

$$l_{ef.x} = 2.8m \quad (l_{max} = 3.3m / l_{min} = 2.3m)$$

$$l_{ef.y} = 2.8m$$

Necessary cross-section area by strength requirements can be calculated with the formula (3.12):

According to the ДБН В.2.6-198 2014. "Сталеві конструкції.", the coefficient of work conditions  $\gamma_c = 0.95$ , for compressed supporting elements of a truss.

$$R_y = 24 kN/cm^2 \quad (\text{for steel C245})$$

Value of the  $\varphi$  coefficient is taken according to the pre-determined flexibility, which should be less than the limit one: for other webbing elements  $\lambda = 100 \dots 120$ . Then, according to the table, for  $\lambda = 100$  and  $R_y = 240 MPa$ , coefficient of  $\varphi = 0.542$ .

$$A_{nec} = N/(\varphi R_y \gamma_c) = 369/(0.542 * 24 * 0.95) = 29.86 \text{ cm}^2$$

Accept: 2 angles of 80x10 (ГОСТ 8509-93), with cross-section area of one piece:

$$A_s = 15.14 \text{ cm}^2$$

$$A_{t.c.} = 2A_s = 2 * 15.14 = 30.28 \text{ cm}^2 > A_{nec} = 29.86 \text{ cm}^2$$

$$i_x = 3.74 \text{ cm}$$

$$i_y = 3.74 \text{ cm}$$

$$\lambda_x = l_{ef.x}/i_x = 280/3.74 = 74.86$$

$$\lambda_y = l_{ef.y}/i_y = 280/3.74 = 74.86$$

Then taking the biggest value of flexibility,  $\varphi = 0.721$ .

Therefore, stresses that will occur in the cross-section, could be calculated as:

$$\sigma_{max} = 369/(0.721 * 30.28) = 16.90 \text{ kN/cm}^2 < 24 * 0.95 = 22.8 \text{ kN/cm}^2$$

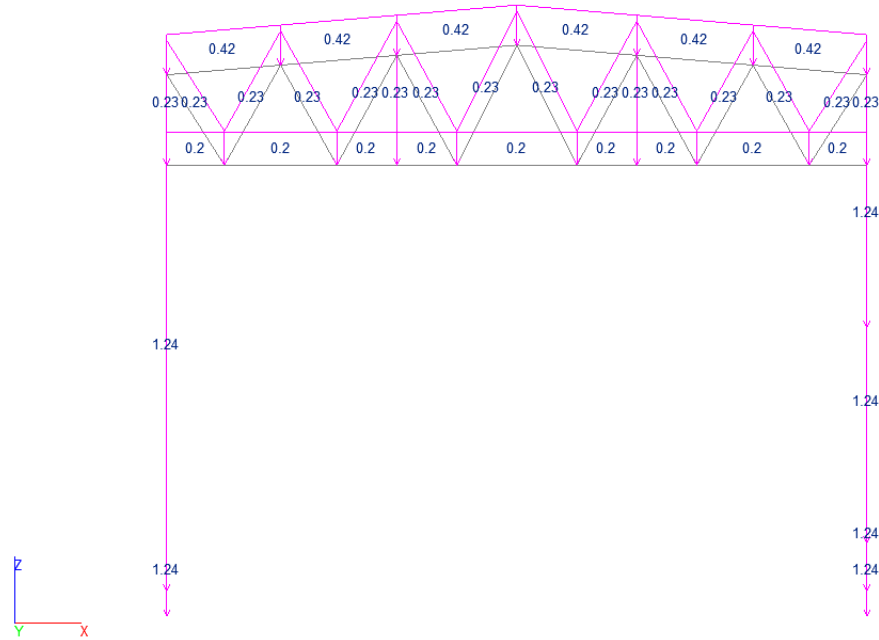
Limiting values of conditional flexibility for compressed elements are set to eliminate the influence of unwilling eccentricities and are determined via applied load with the corresponding characteristic coefficient:

$$\alpha = N/(\varphi A R_y \gamma_c) = 369/(0.721 * 30.28 * 24 * 0.95) = 0.74$$

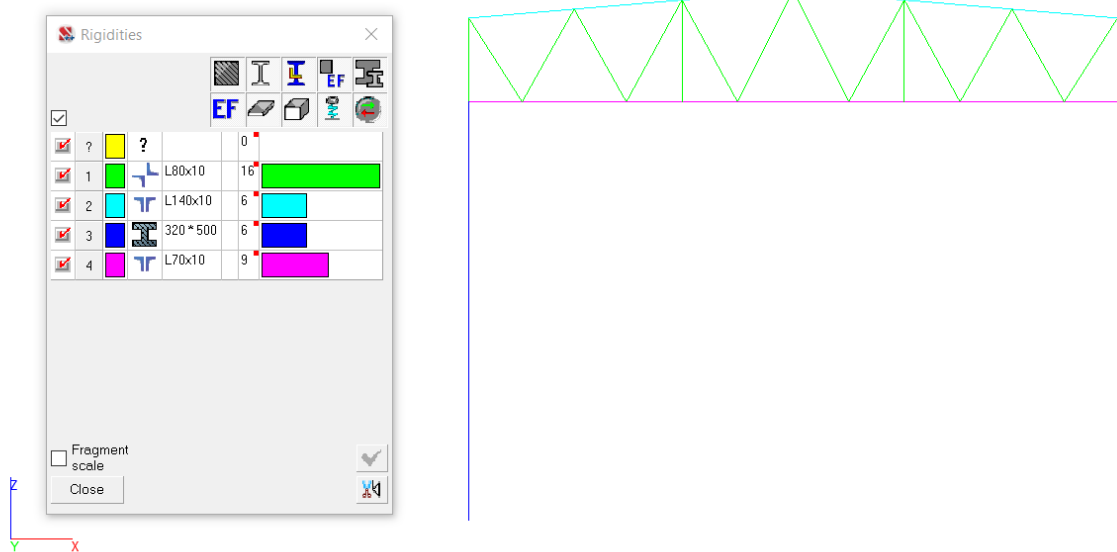
$$\lambda_u = 180 - 60\alpha = 180 - 60 * 0.74 = 135.6 > \lambda_x = 74.86$$

Once I have selected the required cross-section for the most compressed truss members and checked it out for both strength and loss of stability, I can accept the same cross-section for the tensile members as well. It is possible because elements that experience tension will not buckle out. Moreover, the maximum tension force is less than the maximum compression one. Therefore, the selected cross-section can be used without any additional checking. The only disadvantage this implementation has, is that the truss will weigh more because of these "strong" elements been placed where the internal longitudinal forces are pretty low, and even weaker cross-section would also met the requirements.

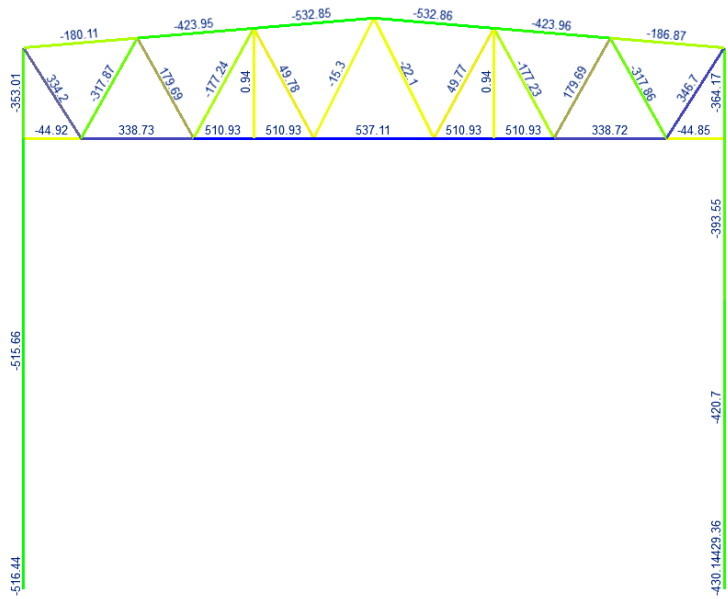
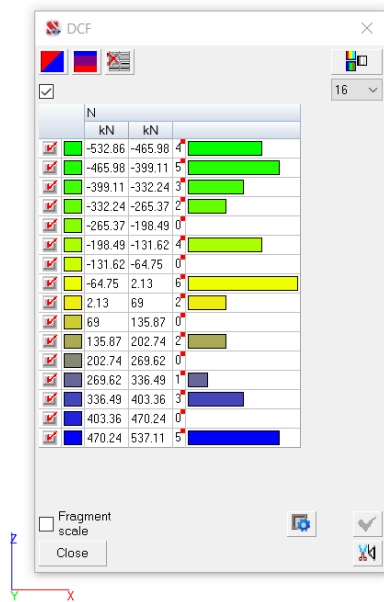
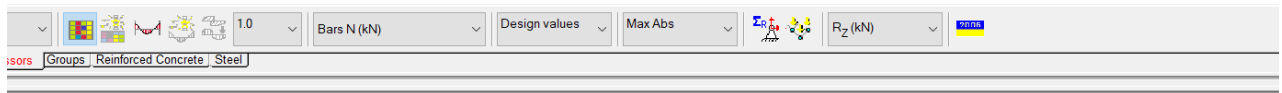
By the way, as far as I have determined all the cross-sections I needed, now I can change the approximate dead load valued with the real ones calculated by the program automatically and see if everything is okay there.



**Fig.3.21.** Scheme of the dead load application, kN/m (performed by the program automatically for the corresponding cross-sections).



**Fig.3.22.** Scheme of the cross-sections application.



**Fig.3.23.** Maximum design values of longitudinal normal forces, kN. (Including the dead load)

From the figure above, it is seen that the maximum longitudinal forces are about the same or even less, as it was predicted to be. The real values of the dead loads turned out to cause smaller internal forces at about 1-5% comparing with the results from the approximately taken ones before. I reckon that is good result. I accept that this 1-5% goes in reserve.

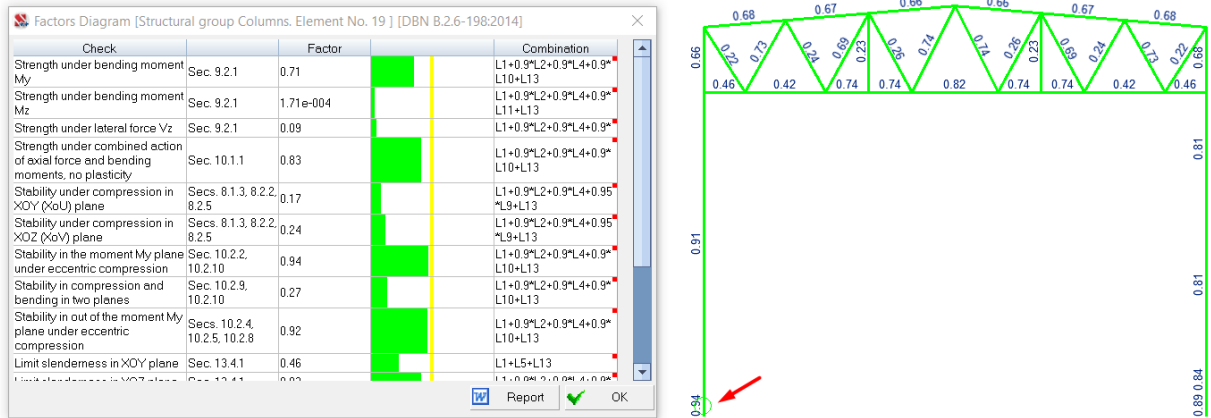


Fig.3.24. Factors diagram for the column's most loaded part.

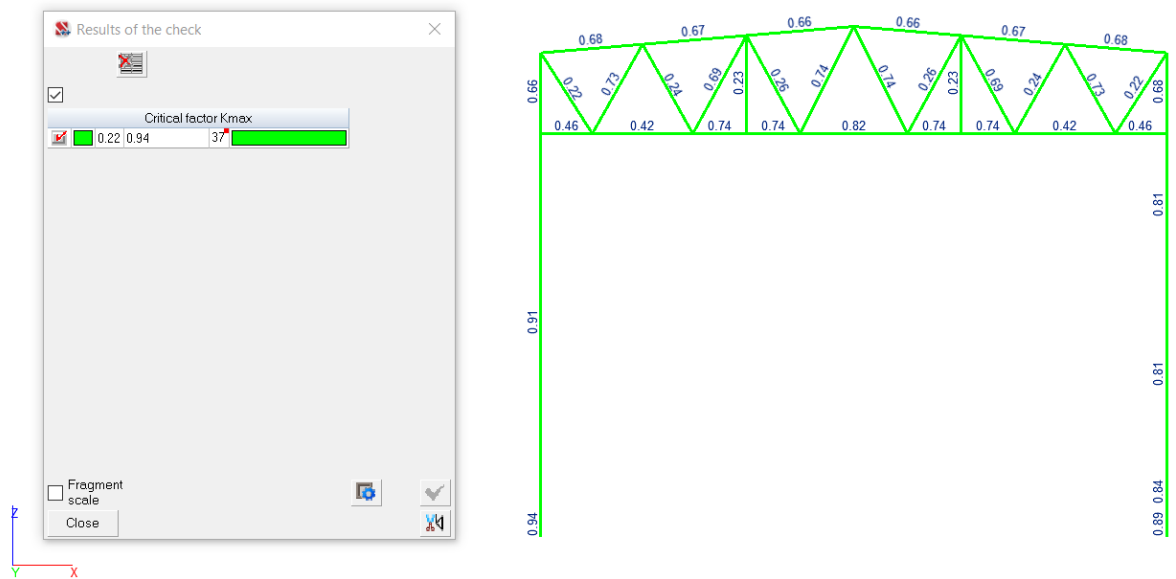
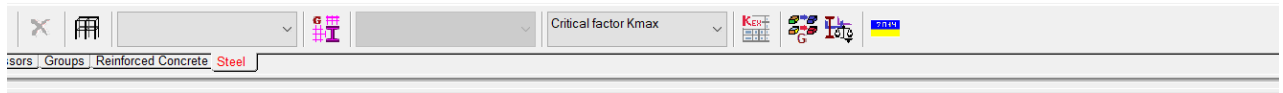


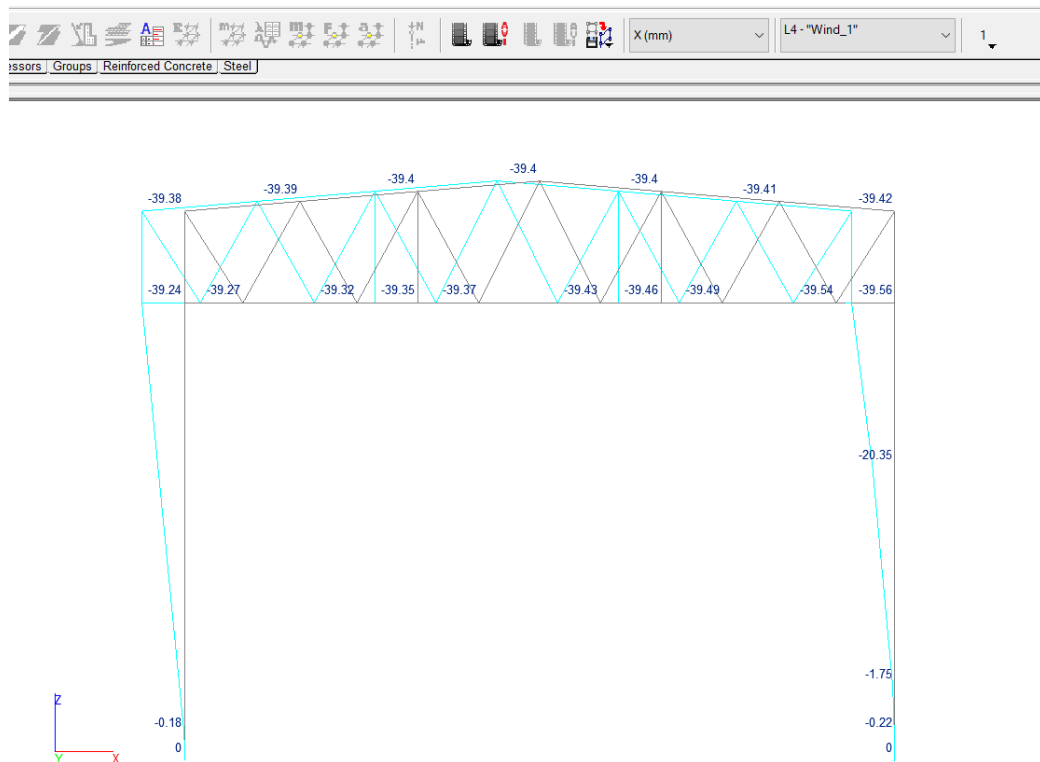
Fig.3.25. Maximum critical factors.

### 3.2.4. Deformation and displacement analysis

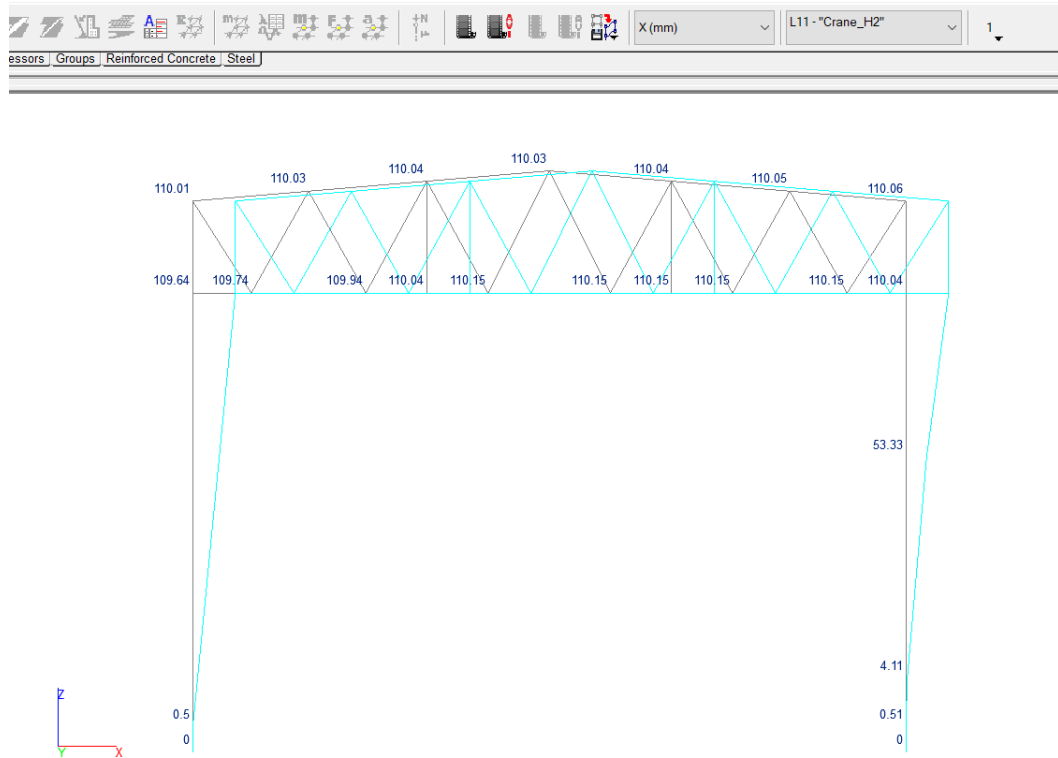
As far as the frame of the building has not only to bear the applied loads to its parts by means of strength, but has also to be rigid enough so to eliminate the possibility of critical deformations and displacements arise. Therefore, I shall check

whether I did not exceed the allowed limits of frame displacements and deformations in both vertical and horizontal directions caused with different loadings. The limit values of displacements and deformations are taken from the ДСТУ Б В.1.2-3 2006. “Прогини і переміщення”.

Let`s start with horizontal displacements. As it is obviously seen, I have only two forces acting in horizontal direction. It is crane caused loads (due to trolley braking) and wind loads. On the figures bellow are shown the frame displacement out of both these loadings.



**Fig.3.26.** Maximum frame displacement caused with “Wind\_1” loading.



**Fig.3.27.** Maximum frame displacement caused with “Crane\_H2” loading.

According to the ДСТУ Б В.1.2-3 2006. “Прогини і переміщення”, table 4, maximum allowed displacements in horizontal direction caused with the wind loads equals  $h_s/180$  (for 11.28m column`s height), where  $h_s$  is a height of the column from the foundation and up to the bottom chord of the truss. However, as it is said in the notes below the table 4., it is possible to increase the allowed limit displacements caused with the wind loads by 30% if there is no disk of rigidity on the covering, but not more than  $h_s/150$ . As far as I have reinforced concrete ribbed slabs on the covering, which create disks of rigidity all over the entire building, I do not need to increase the maximum allowed displacement limit. Thus, the allowed displacement is:

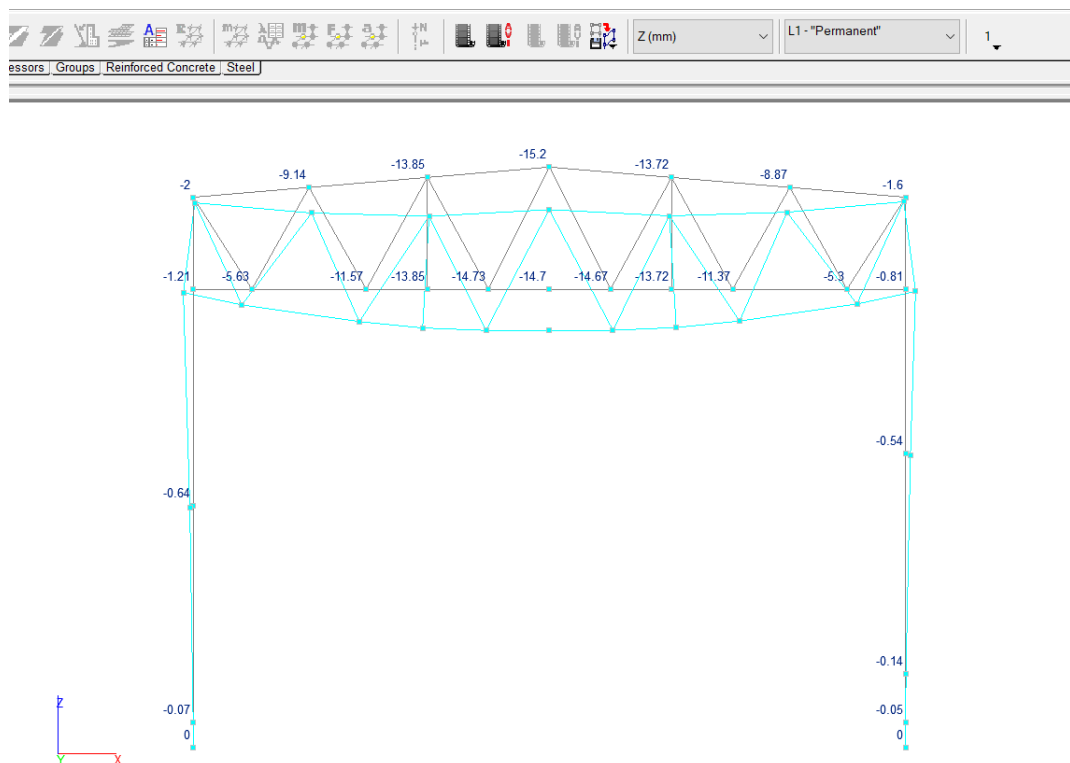
$$f_1 = 11280/180 = 62.67mm > 39.56mm$$

Taking about other horizontal displacements caused with the crane`s trolley braking, the maximum allowed value of displacements is taken according to the ДСТУ Б В.1.2-3 2006. “Прогини і переміщення”, [1, table 3]. For the cranes of 4К

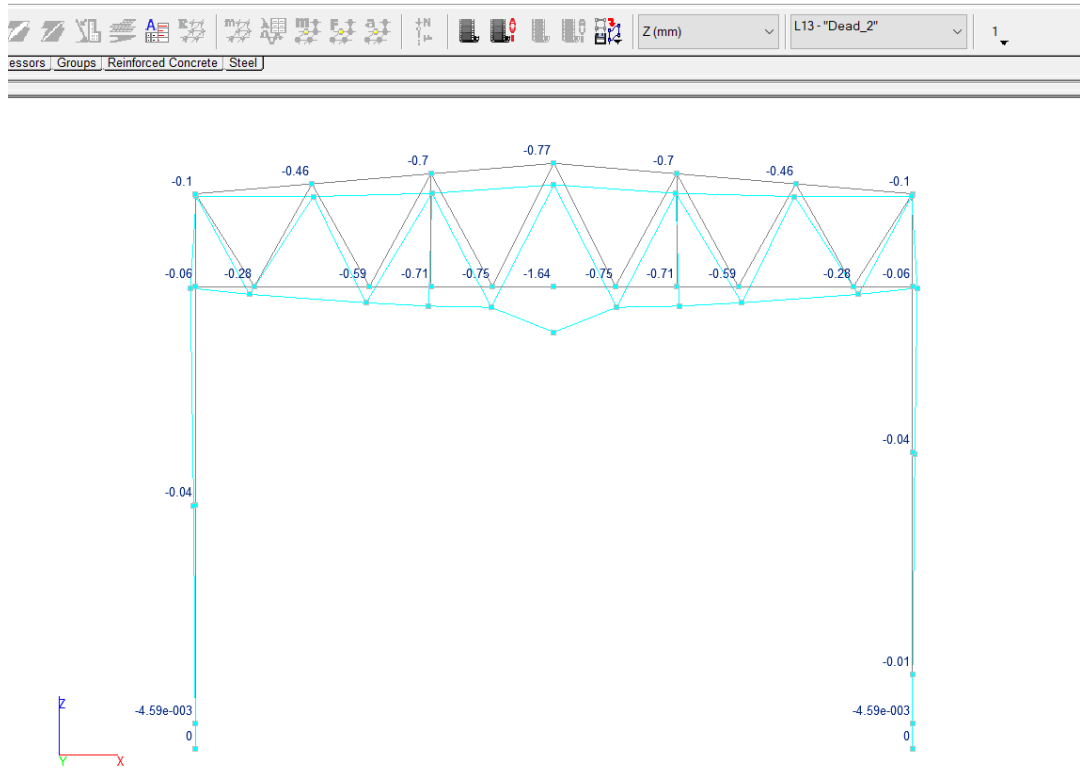


operating mode group, the limit displacement value equals  $h_s/1000$ . However, in clause 6.2 of the ДСТУ Б В.1.2-3 2006. “Прогини і переміщення” it is said, that in case the building does not have a bracing system, which creates a rigid horizontal disk on the level of covering, columns of the frame shall be considered as those ones, which can deflect under applied loads. In my case, I have reinforced concrete ribbed slabs laid all over the building creating the rigid disk on the level of top chord of the truss. Therefore, according to the above-mentioned clause 6.2, the columns I have in my project have to be considered as those ones, which do not tend to deflect.

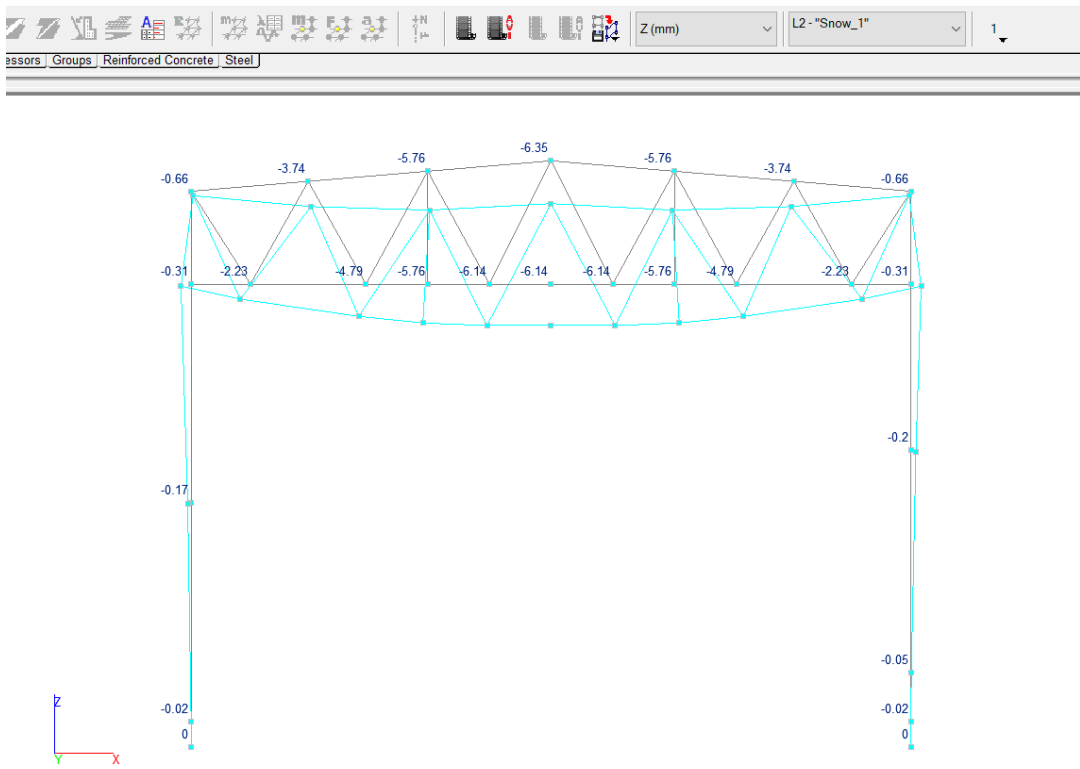
Now let`s take a look on figures 3.28-3.31 below, where the maximum designing vertical displacements are shown.



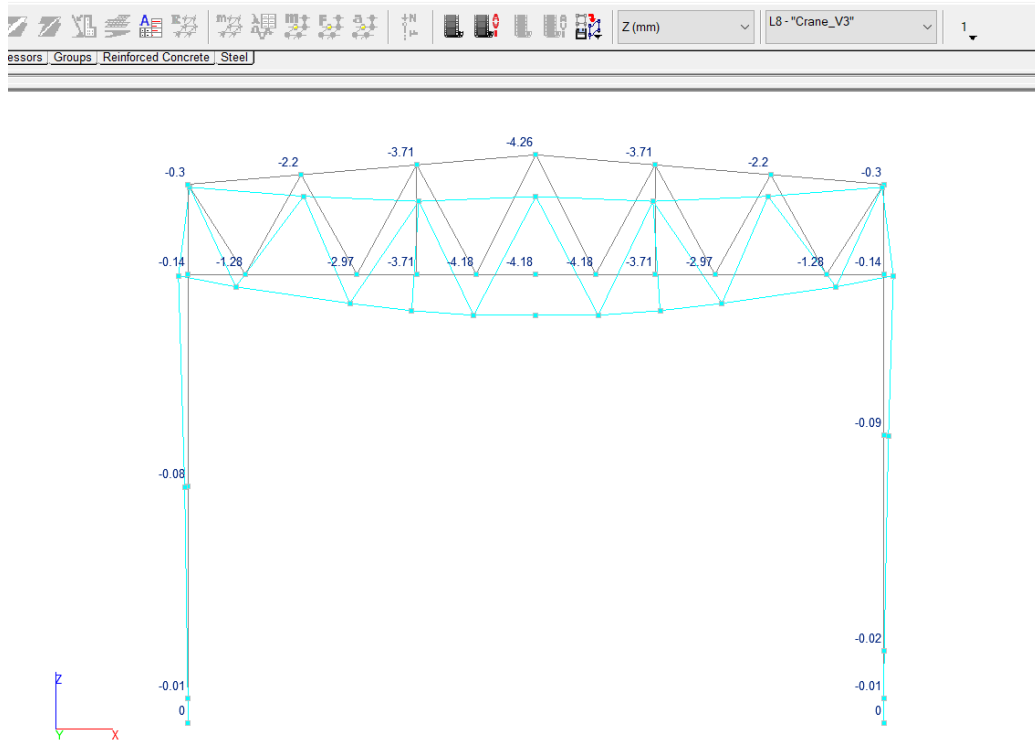
**Fig.3.28.** Maximum frame displacement caused with “Permanent” loading.



**Fig.3.29.** Maximum frame displacement caused with "Dead\_2" loading.



**Fig.3.30.** Maximum frame displacement caused with "Snow\_1" loading.



**Fig.3.31.** Maximum frame displacement caused with “Crane\_V3” loading.

According to the ДСТУ Б В.1.2-3 2006. “Прогини і переміщення”, table 1, maximum allowed displacements in vertical direction caused with applied loads equal  $l/230$  (for 18m truss length). Thus, the allowed displacement is:

$$f_2 = 18000/230 = 78.26mm$$

As it is seen, the deflection values I have in each of the shown cases are less. Moreover, if I sum all the displacements, the limit value would still be quite far away to reach it. Therefore, the displacement and deformation analysis can be considered as passed right.

### 3.3. Joints calculation

#### 3.3.1. Column`s base

Since the frame has been calculated as with the rigid restricted base, it experiences the longitudinal, shear and bending forces at the same time acting down

by the columns bottom end right where the base is situated. Therefore, the base has to be able to transmit all these forces down to the foundation safely. For the column's base plate following statements 3.13 and 3.14 shall be met:

$$\sigma_{b.max} = -(N/[BL]) - ([6M]/[BL^2]) \leq R_{b.loc} \quad (3.13)$$

$$\sigma_{b.min} = -(N/[BL]) + ([6M]/[BL^2]) \leq R_{b.loc} \quad (3.14)$$

Where:

$B, L$  – are width and length of the base plate

$$R_{b.loc} = \alpha \gamma_b R_b = 1 * 1.2 * 2.0 = 2.4 kN/cm^2$$

$R_b$  – is the design strength of concrete the foundation is made of.

$$R_b = 20 MPa = 2.0 kN/cm^2 \quad (\text{for concrete class C16/20})$$

$$\gamma_b = 1.2$$

$$\alpha = 1$$

Designing such a base it is commonly accepted to take the base plate width according to the constructive needs and then calculate the required length. So let's determine the width including that the column's flange width is 320mm and two traverses will be added asides of it, parallel to the column's web. The approximate traverse's thickness is 12mm.

$$B = b_f + 2t_{tr} + 2c_1 = 320 + 2 * 1.2 + 2 * 50 = 422.4 mm$$

$$c_1 = 50 mm \quad (\text{not less than } 40 mm)$$

Accept:  $B = 450 mm$ , according to the section steel plates cutting pattern.

Now using the formula 3.13 I can determine the required length:

$$L = (N/[2BR_{b.loc}]) + \sqrt{(N/[2BR_{b.loc}])^2 + ([6M]/[BR_{b.loc}])}$$

Forces acting on the base are:

$$N = 517 kN$$

$$M = 545 kNm$$

$$Q = 59kN$$

Then the length will be:

$$L = (517/[2 * 45 * 2.4]) + \sqrt{(517/[2 * 45 * 2.4])^2 + ([6 * 54500]/[45 * 2.4])} = 57.41cm$$

Accept:  $L = 600mm$ , according to the section steel plates cutting pattern.

Having determined the base plate dimensions, I can calculate the biggest value of stress underneath the base plate edge caused by the combination of most unwilling loads.

$$\sigma_f = -(N/[BL]) - ([6M]/[BL^2]) = 2.21kN/cm^2 < 2.4kN/cm^2$$

As far as the base plate is under the action of bending moment and longitudinal forces, its thickness has to be calculated according to its stiffness, that is determined according to the biggest value of bending moment occurred in the base plate parts 1, 2 and 3.

Part 1 (supported with 1 side);

$$M_1 = \sigma_f c_1^2 / 2 = 2.21 * 5^2 / 2 = 27.63kNcm$$

Part 2 (supported with 3 sides);

$$M_2 = \beta \sigma_f a^2 = 0.06 * 2.21 * 32^2 = 135.78kNcm$$

$$b/a = 50/320 = 0.156 \rightarrow \beta = 0.06$$

Part 3 (supported with 4 sides);

$$M_3 = \alpha_1 \sigma_f a^2$$

However, if  $b/a > 2$  the plate will work like a beam and should be calculated with a formula:

$$M_3 = \sigma_f a^2 / 8 = 0.125 \sigma_f a^2 = 0.125 * 2.21 * 15.6^2 = 67.23kNcm$$

$$b/a = 460/156 = 2.95 > 2$$

The biggest value of bending moment in the base plate is  $135.78kNcm$ , so the thickness of the plate shall be taken according to this number:

$$t_{pl} = \sqrt{(6M)/(R_y\gamma_c)} = \sqrt{(6 * 135.78)/(28 * 1.15)} = 5.03cm = 55mm$$

$$R_y = 28kN/cm^2 \quad (\text{for steel C325, } t = 40 \dots 60mm)$$

$$\gamma_c = 1.15 \quad (\text{with, } t = 40 \dots 60mm)$$

It seems to be too much in thickness. Let`s try to decrease the thickness. Well, by now the base plate has dimensions  $60x45x5.5cm$ , and the steel volume  $14850cm^3$ . What if I elongate the base plate to  $70cm$ , so to place additional ribs and transform parts 2, which are supported with 3 sides, into parts 3, which would be supported with 4 sides. The base plate here is elongated so to make these parts with 3 supported sides be more wide. It is needed for easier welding of the additional rib, parallel to the column`s flanges. Then the moment will be:

Part 2 (supported with 4 sides);

$$M_3 = \sigma_f a^2 / 8 = 0.125 \sigma_f a^2 = 0.125 * 2.21 * 10^2 = 27.63kNcm$$

$$b/a = 320/100 = 3.2 > 2$$

And the thickness then changes as well into:

$$t_{pl} = \sqrt{(6M)/(R_y\gamma_c)} = \sqrt{(6 * 67.23)/(30 * 1.2)} = 3.35cm = 35mm$$

$$R_y = 30kN/cm^2 \quad (\text{for steel C345, } t = 20 \dots 40mm)$$

$$\gamma_c = 1.2 \quad (\text{with, } t < 40mm)$$

That is much better. Now let`s check if it would be profitable change in terms of metal consumption. After the changers the base plate dimensions turned into  $70x45x3.5cm$  with the steel volume of  $11025cm^3$ . That much less then I had before. However, I didn`t include the additional ribs volume so to figure out the final volume that would be used in here. Let`s assume that the ribs will have the following dimensions:  $32x15x1.0cm$ , that can be even changer then, but for now,

let`s take these values. Then the volume of ribs and the base plate will be  $11985cm^3 < 14850cm^3$ .

Now let`s calculate the traverse. Height of the traverses are determined out of the length of the welding seams length needed to transmit forces to the base. Accept that the forces will go only through these welding seams. The required minimum leg of the fillet weld could be calculated with the formula 3.15. As far as I have 2 traverses on both sided of the column I assume that the forces will split apart equally so the fillet welds of only one traverse would bear a half of longitudinal force and a half of bending moment. Bending moment down there I can change with longitudinal force multiplied on the leg.

$$k_f = (1/\beta_f)\sqrt{(N/2)/(n85R_{wf}\gamma_{wf}\gamma_c)} \quad (3.15)$$

According to the ДБН В.2.6-198 2014. “Сталеві конструкції.”, table 16.2.

$$\beta_f = 0.9$$

According to the ДБН В.2.6-198 2014. “Сталеві конструкції.”, table Д.2.

$$R_{wf} = 18kN/cm^2$$

$$N_M = M/l = 54500/48 = 1136kN$$

Then:

$$k_f = (1/0.9)\sqrt{([517 + 1136]/2)/(4 * 85 * 18 * 1 * 1)} = 0.41cm$$

$$\text{Accept: } k_f = 6mm$$

Now let`s calculate what would be the height of the traverse with the formula 3.16 below:

$$l_f = (N/2)/(nk_f\beta_f R_{wf}\gamma_{wf}\gamma_c) + 1cm \quad (3.16)$$

$$l_f = ([517 + 1136]/2)/(4 * 0.6 * 0.9 * 18 * 1 * 1) + 1cm = 21.26cm$$

$$\text{Accept: } h_{tr} = 250mm$$

All other welding seams that join both traverse with base plate, and traverse with the additional ribs are accepted constructively as  $k_f = 6mm$ .

And the last but not least that has left to figure out are to calculate anchor bolts, anchor plates and some parameters of the traverse. The biggest design value of internal force acting inside the anchor bolts trying to pull them out of the foundation can be calculated with the formula 3.17 below:

$$N_{anc} = (M/l_{anc}) - (N/2) \quad (3.17)$$

$l_{anc}$  – distance between the anchors,  $l_{anc} = 500 + 200 * 2 = 900mm$ .

$$N_{anc} = (545/0.9) - (517/2) = 347.06kN$$

Then the designing value of tension for one anchor and required cross-section area will be:

$$N_{a.1} = N_{anc}/2 = 173.53kN$$

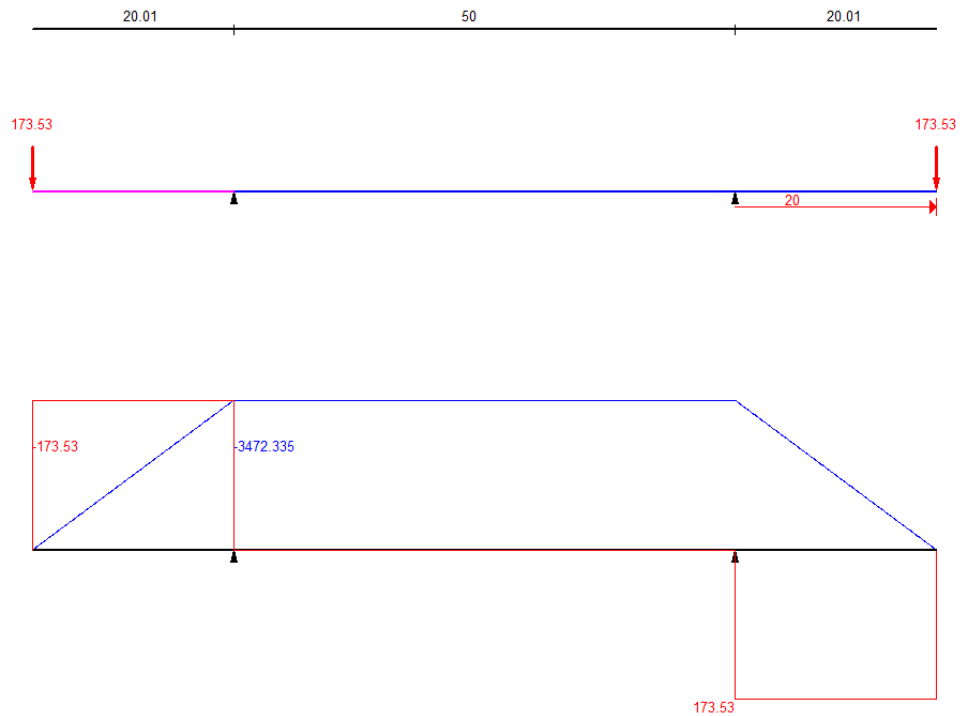
$$A_{a.1} = N_{a.1}/R_{ba} = 173.53/18 = 9.64cm^2$$

$R_{ba}$  – is a designing value of anchor`s steel resistance. In my case C235 steel is used. Therefore,  $R_{ba} = 18MPa = 18kN/cm^2$ .

Accept: anchors  $\emptyset 42$ , ( $A_{anc} = 10.34cm^2$ )

The next thing that has to be checked is traverse. Its designing scheme with applied loads from the anchors is shown on the figure 3.32 below:





**Fig.3.32.** Diagrams of shear force and bending moment in the traverse.

From the figure above it is seen that the shear force and bending moment values are:

$$M = 3472.34kNcm$$

$$Q = 173.53kN$$

Then I can calculate if the traverse can bear these forces with the following formulas:

$$A_{tr} = 25 * 1.2 = 30cm^2$$

$$W_{tr} = (1.2 * 25^2)/6 = 125cm^3$$

$$\sigma = M/W_{tr} = 3472.34/125 = 27.78kN/cm^2 > 24kN/cm^2$$

Well, it has turned out that the traverse is not able to carry the bending moment from the anchors. Hence, let`s increase its height to 300mm.

$$A_{tr} = 30 * 1.2 = 36cm^2$$

$$W_{tr} = (1.2 * 30^2)/6 = 180cm^3$$

$$\sigma = M/W_{tr} = 3472.34/180 = 19.29kN/cm^2 < 24kN/cm^2$$

$$\tau = (1.5Q)/A_{tr} = (1.5 * 173.53)/36 = 7.23kN/cm^2 < 13.92kN/cm^2$$

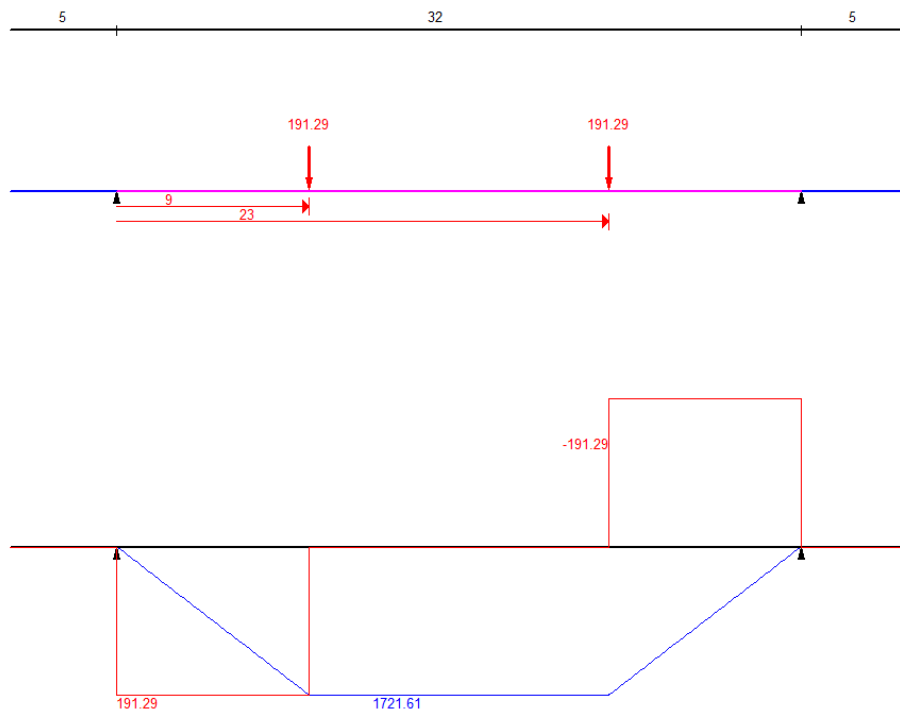
$$\sigma_{red} = \sqrt{\sigma^2 + 3\tau^2} = \sqrt{19.29^2 + 3 * 7.23^2} = 22.99kN/cm^2 < 1.15 * 24 = 27.6kN/cm^2$$

Now, everything is ok. The following traverse dimensions are accepted: 1100x300x12mm.

Anchor plates shall be calculated according to the biggest possible internal force that could occur inside the anchors. This force can be found as:

$$N_{ba} = A_{ba}R_{ba} = 10.34 * 18.5 = 191.29kN$$

Then the designing scheme with applied loads on the anchor plates from the anchors is shown on the figure 3.33 below:



**Fig.3.33.** Diagrams of shear force and bending moment in the anchor plate.

From the figure above it is seen that the shear force and bending moment values are:

$$M = 1721.61kNcm$$

$$Q = 191.29kN$$

Then using the following formulas I can calculate the required anchor plates dimensions so they could bear these forces:

Assume that the anchor plate width is 200mm.

$$W_{nec} = M/R_y\gamma_c = 1721.61/24 = 71.73cm^3$$

$$t_{ap} = \sqrt{(6W_{nec})/b_{ap}} = \sqrt{(6 * 71.73)/20} = 4.63cm$$

Accept:  $t_{ap} = 50mm$

### **3.3.2. Webbing members connection to the top chord of the truss**

In my case, the trusses` elements are made of angles and are connected with the other parts through the gussets. Therefore, the

**CHAPTER 4.**  
**FOUNDATION ANALYSIS AND DESIGN**

#### **4.1.1. Administrative-common building**

Base foundation is fine quartz homogeneous sand, with humidity from low-moisture to wet. The soil under the bottom side of the foundations has the following design characteristics:

- average density  $\rho = 1,60 \text{ t / m}^3$ ;
- modulus of deformation  $E = 14.0 \text{ MPa}$ ;
- specific adhesion  $c = 20 \text{ kPa}$ ;
- angle of internal friction  $\varphi = 29^\circ$ ;
- stiffness coefficient  $e = 0,725$ ;

Groundwater was found at a depth of 8.5 ... 9.5 m from the soil surface; seasonal fluctuations of the groundwater level is in the range of 0.5... 1.0 m. Groundwater is not aggressive against concrete and metal. Depth of soil freezing - 1.1 m.

Isolated reinforced concrete foundation with socket-type pedestal. The pad-slabs of the foundations are set at position -1,650. They lean on the soil base through the prepared 200 mm gravel layer. Reinforced concrete strip monolithic foundations are arranged under the diaphragm of rigidity walls, the bottom is at the position - 1,650.

#### **4.2.2. Production building**

The base of the foundations of the building is gray-yellow fine quartz homogeneous sand, with humidity from low-moisture to wet. The soil under the bottom side of the foundations has the following design characteristics:

- average density  $\rho = 1,60 \text{ t / m}^3$ ;
- deformation modulus  $E = 14.0 \text{ mPa}$ ;
- specific adhesion  $c = 20 \text{ kPa}$ ;
- angle of internal friction  $\varphi = 29^\circ$ ;
- stiffness coefficient  $e = 0,725$ ;
- humidity  $W = 0,04$ ;

Groundwater was found at a depth of 8.5 ... 9.5 m from the soil surface; seasonal fluctuations of the groundwater level are in the range of 0.5... 1.0 m. Groundwater is not aggressive against concrete and metal. Depth of soil freezing - 1.1 m.

Isolated reinforced concrete foundations with socket-type pedestal are set for the columns of blocks A and B, whereas underneath the block C isolated reinforced concrete foundations with monolithic pedestal are used. All the foundations are set at position -1,950. Prefabricated reinforced concrete T-shape foundation beams of 300 mm wide, are laid at position – 0,480. The foundation is made of concrete class C16/20.

Since in the architectural project prefabricated reinforced concrete foundations are used, there is no need to calculate the foundation composition and reinforcement because each foundation is made on the factory and has particular parameters of bearing capacity according to its dimensions. Thus, the only things I have to check are the foundation dimensions, in terms if they satisfy the load bearing requirements, and the foundation base (whether the soil can bear the load) and subsidence.

The reactions to the foundation running from the frame are:

$$N = 517kN$$

$$M = 545kNm$$

$$Q = 59kN$$

The approximate required area of the foundation first footing is calculated with the formula:

$$A_{tr} = 1.1N/R_0 \tag{4.1}$$

Where:

$R_0$  – is the calculated resistance of the soil on which the foundation footing rests.

This value depends on the soil type and its condition and is determined according to the ДБН В.2.1-10-2009 “Основи і фундаменти споруд”, table E.2. For this particular soil type that I have on the construction site, to be exact gray-yellow fine quartz homogeneous sand, with humidity from low-moisture to wet,  $R_0 = 300kPa = 300kN/m^2$ .

Coefficient 1.1 in the formula 4.1 takes into account the approximate weight of the foundation and soil on its edges.

Thus:

$$A_{tr} = (1.1 * 517)/300 = 1.90m^2$$

As far as in the architectural part the foundation first footing dimensions are  $2.1 \times 2.4m$ , the area of its laying on the base is  $5.04m^2$ . That is in two times more than required.

Now let`s calculate what is the soil resistance. For the isolated foundations the following 4.2 formula shall be used:

$$R = ([\gamma_{c.1}\gamma_{c.2}]/k)(M_{\gamma}k_z b \gamma_{II} + M_q d_1 \gamma_{II}^{\dot{}} + M_c c_{II}) \quad (4.2)$$

Where:

$\gamma_{c.1}$  and  $\gamma_{c.2}$  – coefficients of work conditions and are taken according to the ДБН В.2.1-10-2009 “Основи і фундаменти споруд”, table E.7. Hence, for the fine sand  $\gamma_{c.1} = \gamma_{c.2} = 1.3$ .

$k$  – coefficient equal to 1.

$k_z$  – coefficient that is equal 1, if  $b < 10$  m;

$M_{\gamma}, M_q, M_c$  – coefficients that are determined according to the ДБН В.2.1-10-2009 “Основи і фундаменти споруд”, table E.8. depending on the angle of internal friction of the soil  $\varphi_{II}$ ; Since the  $\varphi_{II} = 29^{\circ}$ , then:

$$M_{\gamma} = 1.06$$

$$M_q = 5.25$$

$$M_c = 7.67$$

$b$  – is a foundation footing width, [m].  $b = 2.1m$ .

$d_1$  – is a foundation laying depth, [m].  $d_1 = 1.950m$ .

$\gamma_{II}$  – is an average value of the specific weight of soils below the footing, [ $kN/m^3$ ].

$$\gamma_{II} = 15.7kN/m^3$$

$\gamma_{II}^{\dot{}}$  – the same but above the footing, [ $kN/m^3$ ].

$$\gamma_{II}^{\dot{}} = 14.9kN/m^3$$

$c_{II}$  – design value of specific soil adhesion under the footing, [ $kPa$ ].

$$c_{II} = 2.0kPa = 2.0kN/m^2$$

Then substituting all the above-standing coefficients and parameter into the formula 4.2, I get:

$$R = (1.3 * 1.3)(1.06 * 2.1 * 15.7 + 5.25 * 1.95 * 14.9 + 7.67 * 3.0) = 355.74kN/m^2$$

Now I should determine an average and maximum possible pressures under the footing. The average pressure value for the isolated foundation can be calculated with the formula below:

$$p_{ave} = (N + G_f)/A_{rec} \quad (4.3)$$

$$p_{ave} = (517 + 252)/5.04 = 152.58kN/m^2 < R = 356kN/m^2$$

$$A_{rec} = 5.04m^2$$

$G_f$  – is the weight of the foundation and soil on its ledges can be found with the formula:

$$G_f = lbd\gamma = 2.4 * 2.1 * 1.25 * 40 = 252kN$$

$l$  – is the base length, [m].  $l = 2.4m$

$b$  – is the foundation width, [m].  $b = 2.1m$

$d$  – is the foundation height, [m].  $b = 1.25m$

$\gamma$  – is an average value of the specific weight of the foundation and soil on its edges.

$$\gamma = 40kN/m^3$$

The maximum and minimum pressure values can be determined with the formula 4.4:

$$p_{max/min} = p_{ave} \pm (M + Qd)/W \quad (4.4)$$

$W$  – is the moment of resistance for the foundation first footing, [ $m^3$ ].  $W = 2.02m^3$

Then using formula 4.4, maximum pressure is:

$$p_{max} = 152.58 + (545 + 59 * 1.25)/2.02 = 458.89kPa > 1.2R = 427.2kPa$$



Then using formula 4.3 again, minimum pressure is:

$$p_{min} = 152.58 - (545 + 59 * 1.25)/2.02 = -153.73kPa < 0$$

From the obtained results above it is seen that the foundation is not capable of carrying the applied moments. Thus, the footing dimensions shall be changed. Since the dimensions of the footing influence the moment of resistance, let`s find out from the formula 4.4 required moment of resistance:

$$W_{req} \geq (M + Qd)/p_{ave} \geq (545 + 59 * 1.25)/127.58 \geq 4.85m^3$$

Taking into account the required moment of resistance, footing dimensions can be increased in both directions so to get 3.0x3.6m footing dimensions. Then the moment of resistance will be:

$$W = (3.0 * 3.6^2)/6 = 6.48m^3$$

Now let`s recalculate what is the soil resistance with the new width of the foundation footing will be. Formula 4.2 shall be used here:

$$b = 3.0m.$$

$$R = (1.3 * 1.3)(1.06 * 3.0 * 15.7 + 5.25 * 1.95 * 14.9 + 7.67 * 3.0) = 381.05kN/m^2$$

Now I should determine again average possible pressure under the footing, using the formula 4.3:

$$p_{ave} = (N + G_f)/A_{rec} = (517 + 540)/10.8 = 97.87kN/m^2 < R = 381kN/m^2$$

$$A_{rec} = 3.0 * 3.6 = 10.8m^2$$

$$G_f = lb\gamma = 3.6 * 3.0 * 1.25 * 40 = 540.0kN$$

$$l = 3.6m$$

$$b = 3.0m$$

$$d = 1.25m$$

Then using formula 4.4, maximum pressure will be:

$$p_{max} = 97.87 + (545 + 59 * 1.25)/6.48 = 193.36kPa < 1.2R = 457.26kPa$$

Then using formula 4.3 again, minimum pressure will be:

$$p_{min} = 97.87 - (545 + 59 * 1.25)/6.48 = 2.38kPa > 0$$

Now the requirements are satisfied and the foundation, so as the soil underneath it, can bear the bending moment from the column. The calculation can be finally considered as a complete one if the accepted dimensions of the foundation in terms of their efficiency meet the following condition:

$$100\%(R - p_{ave})/R \leq 15\%$$

$$100\% * (381.05 - 97.87)/381.05 = 74.31\% > 15\%$$

The foundation does not meet the last efficiency requirement, because so to make it bear a bending moment I increased its dimensions and therefore decreased the average pressure under the entire foundation which lead to the low efficiency percentage value. However, we take a look on the maximum and minimum values of pressure, we would see that the minimum pressure is barely came to the zero pont. It means that unless this part of foundation was under compression (if  $p_{min} < 0$ ), the foundation will not be stable.

**CHAPTER 6**  
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## APPENDIX “A”

