MINISTRY OF EDUCATION AND SCIENCE OF UKRAINE
NATIONAL AVIATION UNIVERSITY
FACULTY OF GROUND STRUCTURES AND AIRFIELDS
COMPUTER TECHNOLOGIES OF AIRPORT CONSTRUCTION AND RECONSTRUCTION DEPARTMENT

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QUALIFICATION PAPER

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НАЦІОНАЛЬНИЙ АВІАЦІЙНИЙ УНІВЕРСИТЕТ

Факультет наземних споруд і аеродромів
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ЗАТВЕРДЖУЮ
Завідувач кафедри
________ О.І.Лапенко
«_11_»___05_____2023р.

ЗАВДАННЯ
на виконання кваліфікаційної роботи

Брагар Аліна Геннадіївна
(П.І.Б. випускника)

1.Тема роботи« Промислова будівля з виробництва керамічних блоків м. Житомир»
затверджена наказом ректора від «11» 05. 2023 р. № 681/ст.

2. Термін виконання роботи: з 29.05. 2023 р. по 20.06. 2023 р.

3.Вихідні дані роботи: запроектувати одноповерхову промислову будівлю по виробництву керамічних блоків в місті Житомир. Будівля має три прогони - два паралельних та один перпендикулярний, обладнана крановим обладнанням: в паралельних прогонах крани вантажопідйомністю 30/5т, в перпендикулярному кран вантажопідйомністю 50/10т.

4.Зміст пояснювальної записки: 1) Аналітичний розділ: урахування просторової роботи каркасів промислових будівель при розподілі кранових навантажень; 2) Архітектурний розділ: проектування плану промислової будівлі на позначці 0.000, фісадів, розрізів, конструктивних вузлів та деталей, обчислення техніко-економічних параметрів; 3) Конструктивний розді: розрахунок сталевої колони, сталевої ферми покриття, підбір зв'язків (визначення навантаження відбувається за ДБН В. 1.2-2:2006. Навантаження і впливи. Норми проектування, розрахунок проводиться відповідно до ДБН В.2.6-198:2014. Сталеві конструкції. Норми проектування, підбір зв'язків – за граничною гнучкістю); 4) Основи та фундаменти: запроєктувати стовповий фундамент з підколонником суцільного типу під сталеву колону зовнішнього ряду, з визначенням розрахункового опіру грунту відповідно до фізико-механічних характеристик; 5) Організація будівництва: складання календарного графіку виконання робіт та проектування будівельного генерального плану на монтаж несучих конструкцій будівлі; 6) Технологія будівельного виробництва: технологічна карта на монтаж плит покриття з вибором монтажного крану; 7) Охорона праці: основні правила безпеки, підвищення вогнестійкості конструкцій промислової будівлі з урахуванням норм стандарту європейського комітету; 8) Література: список рекомендованих джерел, 38 пунктів; 9) Додаток 1: креслення КР.1-КР.8

5. Перелік обов'язкового ілюстративного матеріалу: таблиці, рисунки, діаграми, графіки.

6. Календарний план-графік

№ з/п	Завдання	Термін виконання	Підпис керівника
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2.	Архітектурний розділ	29.05-31.05	
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5.	Організвція та технологія будівництва	12.06-14.06	
6.	Охорона праці	15.06-19.06	

7. Консультація з окремих розділів:

Назва розділу	Консультант (посада, П.І.Б.)	Дата, підпис	
		Завдання видав	Завдання прийняв
Аналітичний розділ	Доцент Костира Н.О.	29.05	31.05
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Конструктивний розділ	Доцент Костира Н.О.	01.06	05.06
Основи та фундаменти	Доцент Костира Н.О.	06.06	09.06
Організвція та технологія будівництва	Доцент Костира Н.О.	12.06	14.06
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Завдання прийняв до виконання: Брагар А.Г. ________

CONTENT

INTRODUCTION

The subject of this qualification work is «Steel frame mill building for ceramic blocks production in Zhytomyr». The construction of this industrial building has a number of advantages, one of which is the durability and energy efficiency of ceramic blocks. Paying attention to sustainable development [1], it becomes expedient to use this building material due to its environmental friendliness and energy-saving properties.

However, one of the main reasons for the relevance of this topic is the current situation in our country. Due to the war, a huge number of residential, industrial and public buildings were destroyed, so the construction of an industrial building for the production of ceramic blocks in the city of Zhytomyr will have a huge potential in the post-war reconstruction effort, as well as in the recovery and strengthening of the economy of the region and Ukraine in general.

As mentioned before, ceramic blocks are widely used in construction due to their thermal insulation and durability. With the establishment of local production, the availability of ceramic blocks will increase, which will facilitate the reconstruction of housing and contribute to the creation of safer and energy-efficient buildings.

The war undoubtedly affected the economy of Zhytomyr and the surrounding areas. The creation of an industrial building for the production of ceramic blocks can contribute to economic revitalization by creating jobs, generating income for local businesses and attracting investment. The construction process itself will require labor and materials, which will give a boost to the local economy.

Apart from the local market, demand for ceramic blocks extends to regional and international markets. The creation of a production enterprise in Zhytomyr can position the city as a regional center for the production of ceramic blocks and potentially exploit export opportunities. This can bring in foreign currency and accelerate the overall economic growth of the region.

CHAPTER 1
ANALYTICAL REVIEW

1.1. Analytical review

Production buildings are designed to accommodate a certain technological production process, which should be taken into account while executing design works.

Industrial buildings are characterized by a variety of structural schemes, storeys, used materials, crane equipment, and fire resistance. About 90% of the total number of industrial buildings are frame buildings, almost half of which are made using steel structures [2-3]. The floor space of buildings is determined by the direction of the technological process, and single-story buildings have become the most common in construction practice. Technological equipment is placed at the zero mark and the load from it is not transferred to the load-bearing structures, simplifying the structural form and lightening the load-bearing structures.

In one-story buildings, conditions are created for uniform lighting of the interior spaces, installation is simplified, and the possibility of changing production technology is ensured. However, single-story construction also causes a number of inconveniences associated with a large building area and a long length of engineering communications. The frame of the industrial building is a single spatial system of structures that perceives the influences acting on it and transfers the resulting forces to the foundations. When planning, the main supporting structures (flat transverse frames formed by columns and crossbars) and enclosing structures (roofs, walls) are distinguished.

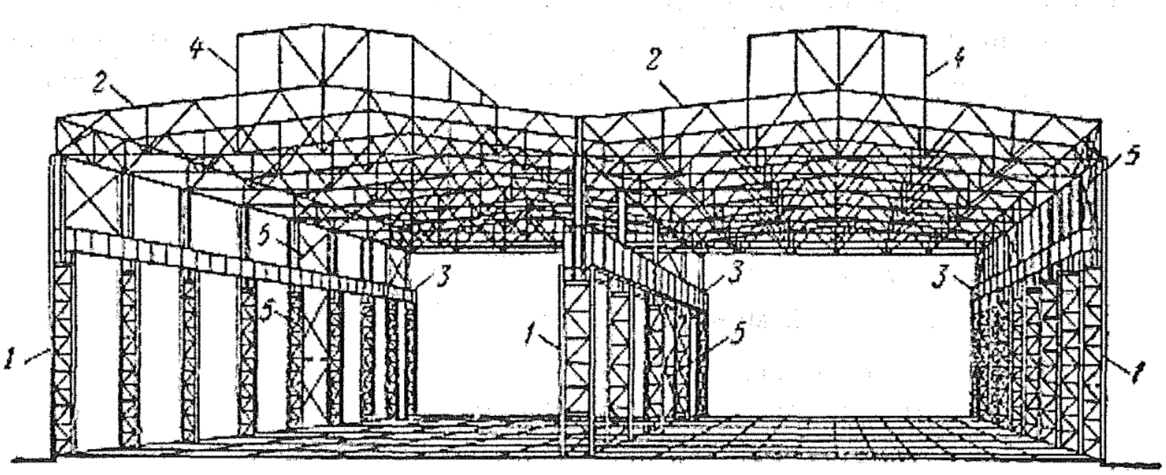


Fig. 1.1. Structural scheme of the frame of a two-span industrial building: 1 - columns; 2 - roof farms; 3 - crane beams; 4 - monitors; 5 - braces along the columns.

1.1. Significance of the spatial work of the metal frame

The building shell's structure is usually broken down into longitudinal and flat frames. The method of calculating the building shell's transverse frames is based on the vertical loads that are applied to the girder [4-5]. This method can be performed accurately if the wind forces and the horizontal loads are the same. Another method of calculating the same frames is by taking into account the load that the crane is carrying for a few frames [6].

The displacement of the building shell's frame from the crane's load Δ_{sp} is less than that of the flat frame Δ. The relationship between the two Δ_{sp}/Δ is referred to as the spatial work α_{sp}.

When calculating a building in spatial form, it is necessary to consider:

- steel frame, considered as a continuous beam system;

- Spatial calculations for rigid roofs with steel frames (precast concrete slabs, steel profile decks, corrugated iron sheets, etc.) as longitudinal members;

- The construction scheme of the transverse frame, assumed to be a planar frame, absorbs the horizontal resistance in the longitudinal plane of the disc due to the space trusses, in addition to the external loads directly imposed on it.

The inclusion of spatial frames in calculations for individual planar frames is limited to determining the elastic resistance of the longitudinal struts, which is considered as an external load on the strut horizontal frame.

With the help of the Lira SAPR computer program, the frame with the crane can be calculated precisely. The designed scheme is conceived as a spatial beam system.

The calculation compares single-frame and triple-frame buildings with the same applied loads. The results show that the higher the frequency of frame static undefined, the better the distribution between load-bearing elements.

Therefore, in some cases, it is possible to increase the crane load on the frame without strengthening the frame due to space constraints. By including a spatial framework, the use of steel and other materials can often be reduced. The feasibility of such calculations is determined on a case-by-case basis - size of building, design concept, nature of external loads, etc. - among other factors.

Incorporation of spatial framework

It is recommended to consider the spatial frame determined in each case according to the following considerations: Dimensional structure, design scheme, type, magnitude and loads application method, e.g.. The longitudinal structure complies with additional requirements (stiffness, etc.) affecting the operation of transverse loads. When building a crane it makes sense to include a spatial framework.

The functions of the vertical loads, which provide the framework, can perform the bracing steel trusses, the playing field, as well as the building sheath, made of other materials.

These discs include, for example, prefabricated roofs and floors or monolithic concrete slabs, profiled steel sheets, corrugated sheets, etc. These discs shall be considered as internal parts in the case of non-rigid roofs, e.g. roofs made of corrugated asbestos panels, taking into account only the longitudinal interface coating.

Commonly used in industrial buildings, it has a number of vertically adjustable discs to suit their commercial requirements.

In each case, it is necessary to establish which loads should be taken into account when calculating the scheme. In single-story industrial buildings with coatings arranged on one level, there is usually only one vertical load located at the level of the roof. In single-story buildings with coatings, located at different levels, it is possible to count the loads located at each level of the coating.

The calculation scheme in the spatial frame is implemented differently depending on the stiffness of the vertical loads, the quantity included in the calculation of the vertical loads and the position of the columns along (same or different steps on different rows and columns).

The following points should be considered when calculating the spatial position of a one-story industrial building using longitudinal load:

1) steel frame as a system of continuous beams, longitudinal elastic discs on the landing columns, the role of continuous beams is played by longitudinal structures, elastic landing supports - transverse frames;

2) for the calculation of the rigid roof with the spatial metal framework (precast concrete slabs, steel profiled decks, corrugated steel, etc.) taken into account as a vertical member. In some cases it is advisable to consider the presence of other longitudinal elements such as brake structures, especially if it's affect the horizontal plane;

3) the calculation scheme of the transverse frame is considered as a flat frame, in addition to the fact that external loads directly act on it, resistance forces and transverse forces in the longitudinal plane of the disk arise due to the spatial frame.

The spatial structure of all industrial roofs is divided into rigid and non-rigid. In the first case, the longitudinal load is controlled by the transverse connection of the lower bars of the truss, and in the second - the upper sections. The result of the use of space is found in the solidity of the roof and the exterior frame.

The redistribution of crane loads is added to the brake production process. This calculation method preserves the external stiffness, the connection of the frame and the lower column of the truss, and the design of the long disc of the second brake. When determining the height of the frame, it is possible to determine the longitudinal forces acting on the column, which in some cases are not distributed from the frame and transmitted to the floor [9-10]. If necessary, moments and shifts in the column can be determined. Allows frames to be treated as independent block elements if they are identical and have the same load. But only for certain points of block length.

By placing the sample at the same level as the previous chord frame, the load on the frame and adjacent frames can be reduced. The columnar variation of the adjacent framing area will be a common feature of the concrete slab roof ridge. Wherein, the plates have a solid and sturdy structure that connects to the corners. Such a structure is called a spatial elements frame.

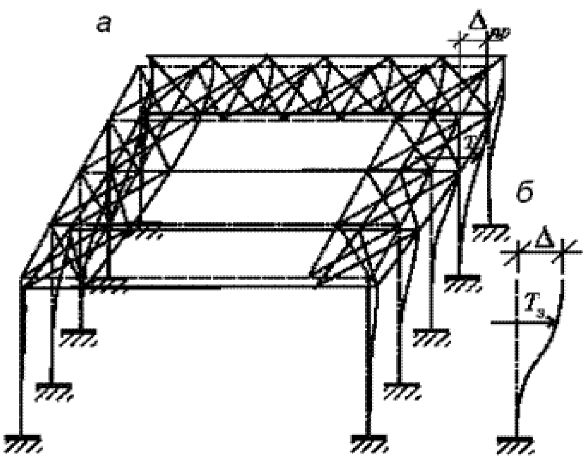


Fig.1.2. Circuits of the second end of the roof girder, comprising a working unit (a) and a single unit (b)

The most effective approach for spatial inclusion is the use of single span frames. In the case of multi-span frames, the influence of loads on the disk displacement is significantly less.

To improve the performance of the multi-span frames, it is possible to obtain the load of the crane operating only on some frames. As an option, other frames can be used, including vertical structures such as roof panels, supports for the lower rafter bars, brake structures, etc.

The maximum top displacement Δ_{pr} of the loaded column is less than one of the single frames. By taking into account the work and the spatial effort determining the crane's load in the formula mentioned below, Δ_i has replaced Δ_{pri} where $\Delta_{pr} = \alpha_{pr}\Delta_i$ is linearly related, preventing displacement.

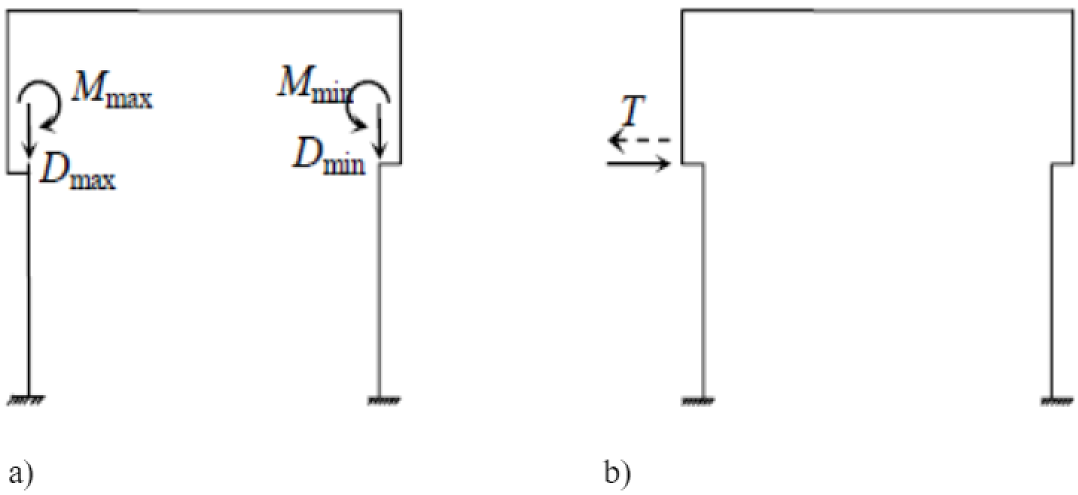


Fig.1.3. Rating scheme of the vertical crane load on frame (a) and horizontal crane load on frame (b)

The structure of the longitudinal frame (roofing, longitudinal connection to the lower chord, brake design, etc.) distributes the load across all frames, thereby reducing lateral displacement of columns and bending moments in the most loaded frame . The spatial factor $\alpha_{pr} = \Delta_{pr}/\Delta$, is taken into account in relation with the work of the spatial frame when it is necessary to calculating the impact on the flat frame crane,

We can calculate the value of α_{pr} using the following formula:

$$\alpha_{pr} = 1 - \alpha - \alpha'(n_0 /\Sigma y - 1) \quad (1.1)$$

In which α , α′ are coefficients that are depending on the parameter β, Table 1.1; n_0 – the number of wheels on one thread tap crane girders; Σ y – is the ordinary amount of influence lines considered in frame.

Table 1.1.

β	0	0,01	0,02	0,03	0,04	0,05	0,1	0,15	0,2	0,5
α	0,86	0,77	0,73	0,71	0,69	0,67	0,62	0,58	0,56	0,46
α'	-0,14	-0,2	-0,22	-0,24	-0,25	-0,25	-0,26	-0,26	-0,26	-0,26

$$\beta = \left(\frac{\text{B}}{\text{H}_\text{p}}\right)^3 \cdot d \cdot \frac{\sum I_\text{н}}{I_\text{п}} \qquad (1.2)$$

From this formula, parameter β is characterizes the ratio between the frame's transverse stiffness and the and cover:

B - transverse frames step; H_p – girder height; d - speed reduction coefficient column to column of constant cross section, equivalent to a shift , d = k_b /12 (coefficient k_b from the Table 1.2); $\sum I_\text{н}$- sum of the column's bottom moments of inertia; $I_\text{п} = I_{ch} + I_{cr}$, where I_{ch} is a moments of inertia of the longitudinal connections on the bottom chord and horizontal elements of the roof; I_{cr} is the equivalent moment of inertia of the roof [11].

Table 1.2.

H_v/H	Value of k_b depending on I_v/I_n ratio			
	0,1	0,15	0,2	1,0
0,2	5,203	5,82	6,315	12
0,25	5,195	5,8	6,315	
0,3	5,182	5,77	6,283	
0,35	5,11	5,73	6,263	
0,4	4,956	5,67	6,248	

The ratio $\sum I_\text{н} / I_\text{п}$ can be taken in the following ranges for the coatings: large-size reinforced concrete slabs 1/40…1/100; small size reinforced concrete slabs on purlins 1/10…1/25; flat steel plate on purlins (steel panel)1/5…1/10; with profiled sheet on purlins (corrugated panels)1/2…1/6. Smaller values $\sum I_\text{н} / I_\text{п}$ should be taken into buildings without lights spans up to 36 m with light-duty cranes.

The design scheme is accepted in the form of a three-dimensional bar system consisting of 5-7 planar transverse frames connected to the level of girder and crane structures of longitudinal elements of finite stiffness.

When calculating the effect of the bioactivity of the plane transverse frame on other transverse frames associated with the considered frame longitudinal members, consideration of the calculation scheme aids the inclusion of elastically compliant Figure B. their resistance value at the points adjacent to the column is almost negligible and does not affect the magnitude and distribution of force in the column, so it cannot be ignored.

In this case, the spatial framework can be calculated by defining a reaction X_R at the level of the roof girder (the calculation method of forces) or the top of the frame shift in the system of spatial units Δ_{pr} (calculated by displacement).

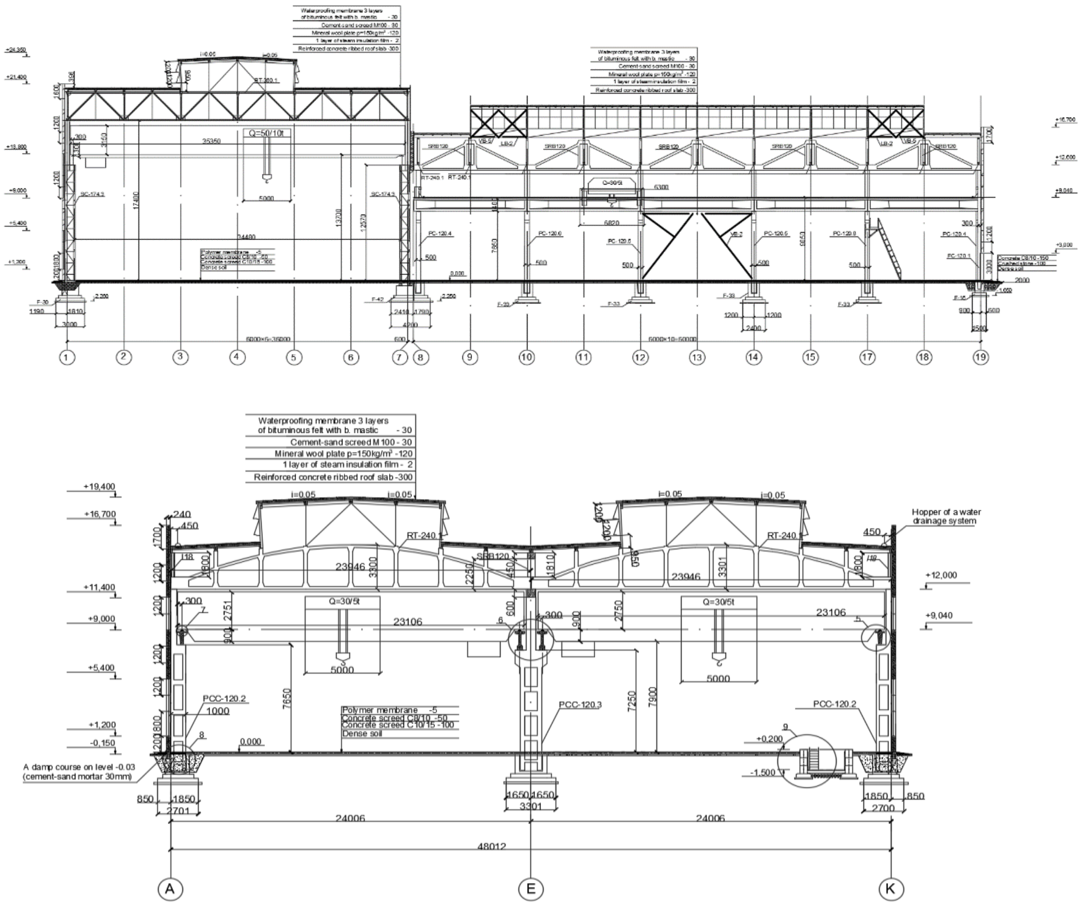


Fig. 1.4. Scheme to the frame spatial integration

CHAPTER 2

ARCHITECTURAL PART

2.1. Object passport

According to the task, a one-story industrial building for the production of ceramic blocks was designed.

Site of construction - Zhytomyr region, Zhytomyr city;

According to ДСТУ-Н Б В.1.1-27:2010 [12], Zhytomyr region. belongs to the climate zone - I;

According to the normative wind load, the territory belongs to 3d district with high-speed wind pressure - 460 Pa (46 kg/m²);

According to ДБН В.1.2-2:2006 [6], the territory belongs to the 5th district with a normative load in terms of snow cover - 1460 Pa (146 kg/m²);

The average air temperature for the year - plus 7.2 °C;

The absolute minimum temperature - minus 29 °C;

Absolute maximum temperature - plus 27 °C;

The temperature of the coldest five-day day - minus 24 °C;

The base of the foundations of the building is a hard sandy loam. The soil under the bottom side of the foundations has the following design characteristics:

- average density ρ = 1,57 t / m³;
- deformation modulus E = 12.0 mPa;
- specific adhesion with = 19 kPa;
- angle of internal friction φ = 26°;
- stiffness coefficient e = 0, 5;
- humidity W = 0,04;

Groundwater was found at a depth of 7.0 m from the soil surface; seasonal fluctuations of the groundwater level are in the range of 0.5… 1.0 m. Depth of soil freezing [12] - 1.0 m.

2.2. Volume-planning decision

During the execution on the qualification work, a one-story industrial building with one perpendicular span and two parallel spans and two-storeyed administrative-common building was designed. The perpendicular span is 36 meters long in the axes “A-K”, “1-7”, height – 23.55 m, column spacing $b = 6.0m$ and is equipped by overhead traveling crane $Q = 50/10t$; Two parallel spans is 24 m long in the axis “A-E”, “8-19” and “E-K”, “8-19”, have height 19.4 m, column spacing of outer row columns $b = 6.0m$, inner row – $b = 12.0m$ and are equipped by overhead traveling cranes $Q = 30/5t$.

Due to the difference in the height of the spans between axes "7-8", a settlement joint with insert 600mm was designed.

Crane landing for access to the crane cabin of the overhead traveling crane was designed on the second column along axis “E”.

Technical-economical parameters:

1. Area of the building: 4632.9 m^2
2. Volume of the building: 99144.1 m^3

A two-storeyed administrative-common building is placed in the axis "9 - 18", "L - N", and adjoins the main industrial building of the enterprise at axis L. 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 staircases are located, a grid of columns 6.0 x 3.0 m was used. The dimensions of the building in plan are 48.84 x 12.6 m. Two constructive steps of 6.0 m wide were taken along the width of the building, and 8 constructive steps of 6.0 m wide were taken along its entire length; in the places of staircases location additional columns with a step of 3.0 m have been installed. The height of the floors is 3.3 m. The total height of the building down from the soil surface and up to the top of the parapet is 6.5 m.

In the administrative-common building there are next rooms:

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

Group of industrial process – 1b; 100/50 (men - 60%)

The rating number of workers per 1 shower box – 15

Rating number of workers per 1 washbasin – 10

The number of shower boxes and washbasins depends on more numerical duty and the group of the industrial process. (The opened shower boxes have dimension in plan 0,9x0,9 m and the area of cloakroom is equal to 0,7m^2 per 1 shower box) Thus, the number of showers and washbasins equal:

Showers: 40/15 = 3 (for women); 60/15 = 4 (for men);

Cloakroom area for women: 3·(0.81+0.7) ≈ 4,53 m^2

Washbasins: 40/10 = 4 (for women); 60/10 = 6 (for men)

Cloakroom area for men: 4·(0.81+0.7) ≈ 6,04 m^2

The number of wardrobe boxes is determined according General number of workers.

150-60% = 60 (for women)

150-40% = 90 (for men)

The number of toilets depends on more numerical duties and Group of industrial processes. In industrial buildings 1 toilet per 18 men or 12 women. (1 washbasin per 4 toilets + hand dryer)

60/18 ≈ 3.33 = 4 (for men)

40/12 ≈ 3.33 = 4 (for women)

The room of women hygiene has dimensions in plan: 2.2x1.8 m.

The resting room area is 0,9 m^2 per 1 person of more numerical duty, but not less than 4,0 m^2: 60*0.9 = 54 m^2 (for men); 40*0.9 = 36 m^2 (for women).

- Laboratories;

Laboratory area: = 26,07m^2

- Dining room;

The lunchroom is designed on the first floor and depends on more numerical duties: from 13 to 200 workers. (1 seat per 4 workers, 1 washbasin per 15 seats)

100/4 = 25 (seats) → 25/15 ≈ 1.67 = 2 (washbasins)

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

Number of seats: 25 →

Hall-room are: ≈ 44 m^2;

Storeroom area: ≈ 11 m^2;

Room for washing dishes area: ≈ 6 m^2;

Total: ≈ 61 m^2

- Medical room;

The medical room is designed with an area 12 m^2

- 50…150 and 18 m^2

- 151…300 persons

(according General number of workers.)

Medical room area: ≈ 12m^2 (= 16,21 m^2)

- Meeting room;

Meeting hall area: 24 m^2

- 50…100 persons; 36 m^2

- 101…200 persons in terms of more numerical duty.

Meeting hall area: = 28,63 m^2

- Auxiliary technical;

Charwoman room area: = 4 m^2

- Warehouses and pantries.

Archive room area: = 20,70 m^2

Inventory room area: = 16,21 m^2

- Office administrative;

Public union room area: = 20,25m^2

Chief accounting officer room area: = 35,91 m^2

Chief technologist room area: = 35,91 m^2

Construction department area: = 53,90 m^2

Chief engineer`s room area: = 35,91 m^2

Technical department room area: =35,91 m^2

Reception room area: 28,60 m^2

Dispatcher`s room area: = 20,25 m^2

In general, the spatial planning decision of the building meets the requirements [14].

CHAPTER 3
STRUCTURAL PART

3.1. Characteristics of the industrial building

The industrial building of the frame structural system with spans of load-bearing structures of the - 24,0 m and 36,0 m; the height to the bottom flange of the roof trusses – 12,0 m and 17.4 m. Axial dimensions of the building in plan are 96.6×48,0 m.

The frame of the building is made of prefabricated reinforced concrete and metal structures - columns, load-bearing elements and connections; connection of columns with girders - hinged, made with bolts and electric welding with clamping of columns in the foundations.

The spatial planning decision of the building is hall-like.

1. **<u>Foundations</u>**

Monolithic reinforced concrete isolated foundation with socket-type pedestal with prefabricated reinforced trapezoidal foundation beams - 300 mm wide, which are located at position -0,480, the foundation is made of concrete class C16/20, with a dimensions of the sole 2.7×2.2 m, the depth of laying is 2.25 m; 1.5×1.5 m, the depth of laying is 1.65 m; 3.3×2.4 m, the depth of laying is 2.25 m.

In axis “1” and “7” – Monolithic reinforced concrete isolated foundation with solid-type pedestal with prefabricated reinforced trapezoidal foundation beams - 300 mm wide, which are located at position -0,480, the foundation is made of concrete class C16/20, with a dimensions of the sole 3.0×2.7 m, the depth of laying is 2.25 m

2. **<u>Columns</u>**

Reinforced concrete with a cross-section 1000x500 and 1400x500 mm that are designed for production buildings with crane, made of concrete class C20/25; columns are rigidly clamped in the socket-type pedestal of the isolated foundations; the step of the load-bearing columns along the axes "1", "7", "A", "K" - 6 m. , along the axis "E" - 12 m. Framework columns - reinforced concrete with a cross section of 300 x 300 mm.

Metal columns:

Above-crane part of the column – I-beam with a complicated cross-section, flanges – 240×10 mm, web – 460×8 mm.

The under-crane part of the column consists of two pieces that are combined with a lattice. For columns pieces use section steel I-beam type 60B2 in accord with ДСТУ 8768:2018 [15]. Lattice is designed from one equal leg angle ∟100×10 in accord with ДСТУ 2251:2018 [16].

3. <u>Load-bearing elements of the roof</u>

Prefabricated reinforced concrete rafter trusses with a span of 24,0 m are assembled on the axes "8 - 19" , they are made of concrete class C20/25; on the axis "E" they are lean on the secondary roof trusses (span - 12 m); each truss consists of two prefabricated parts which are connected on a construction site and the connection joints of these two parts are reinforced in-situ.

Metal roof trusses with a span of 36,0 m are assembled on the axes "A - K" (the last one in perpendicular span). Height is 3.15 m with grade of steel C255.

4. <u>Roof slabs</u>

Some amount of covering structures are consist of prefabricated reinforced concrete ribbed slabs with dimensions in plan - 3.0x6.0 m (C20/25). On the roof of the industrial building lite-saturating trapezoidal monitors are arranged; Thermal insulation layer of the coating - mineral felt with a density ρ = 150 kg / m3.

5. <u>External walls</u>

The external wall structures are made of single-layer hinged panels made of expanded clay concrete (C8/10), 240 mm thick. All panel joints are flat and they are sealed with cement-sand mortar M25.

6. <u>Roofing</u>

On top of the ribbed roof slabs a layer of steem and thermal insulation is located. The cement-sand screed is overlapped with 3 layers of bituminous felt with bituminous mastic 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.

7. **Floor structure**

Floor structure is consist of two concrete screed layers: the first one is 100 mm thick (C10/15), the second one is – 50 mm (C8/10). A 5 mm polymer covering is placed on the top of this layers.

8. **Windows**

All windows located below the +9.000 level are made of steel with double and/or single glazing; in premises on the level + 9,000 and above windows characterisricks are changed (aluminum double-glazed windows).

9. **Doors**

Aluminum or steel with glazing.

10. **Gates**

Steel, swing-like, of frame type.

3.2. Determination the cross-section parameters of one-storeyed industrial building

First of all, for the further calculations of a traverse frame of one-storeyed industrial building we need to change the structural scheme to the rating scheme.

The general dimensions of a plant namely its span L, height to the elevation of the crane rails top H_1 as well as the total height of the plant H (to the bottom chord of the roof truss) are depending on the size of the equipment to be accommodated and the nature of the production process to be followed in the plant.

In accordance with the general rules for standardization in buildings will traveling cranes the height should be multiple 0.6m up to the 8.4m and 1.2 if the height is more than 8.4m.

The magnitude of the clearance for the crane above the top of the rails H_{cr} is taken according to standard for overhead traveling crane [18]. This standard put up 8 groups of working crane's condition:

1K-3K – light working condition;

4K-6K – medium working condition;

7K-8K – heavy working condition.

3.2.1. Determination vertical dimensions of one-storeyed industrial building

Initial data:

$L = 36m$; $H_{cr} = 3150\,mm$;

$H_1 = 13.2m$; $B_{cr} = 300\,mm$;

$L_{cr} = 34.5m$; $h_{r.b} = 130\,mm$;

$Q = 50/10t$; $h_{cr.b} = 1000\,mm$;

$B = 6m$;

The total height of the plant is determined according to the next formula:

$$H = H_1 + H_2/0.6\,m \qquad (3.1)$$

$$H_2 = H_{cr} + C + 100\,mm = 3150 + 400 + 100 = 3650;$$

C – take into account deflection of the roof trusses and braces along their bottom chord (C=400 mm because L=36 m).

Assume $H_2 = 3700$

$$H = 13700 + 3700 = 17400\,mm;$$

The total length of the column is determined as follows:

$$l_c = H + H_b = 17400 + 600 = 18000\,mm;$$

The length of top part of the column:

$$l_2 = H_2 + h_{r.b} + h_{cr.b} = 3700 + 130 + 1000 = 4830\,mm;$$

The length of undercrane part of the column:

$$l_1 = l_c - l_2 = 18000 - 4830 = 13170\,mm;$$

We assume the metal roof truss with parallel chords and $h_{tr} = 3150\,mm$

$$h_{mon} = h_{t.p}(200) + h_g + h_{b.t}(600\ldots800) \qquad (3.2)$$

$h_{mon} = 3.0\,m$, because $L = 36\,m$;

3.2.2. Determination the horizontal dimensions of building frame

The span of the column – its dimension that equals distance from the exterior edge of the column to the coordination axis.

"0" if $Q \leq 30t, H \leq 14.4m, B \leq 6m$

"250" – in other cases

"500" if $Q > 75t\ (7K/8K), H \geq 20m$

ДСТУ EN 13001-1:2018: 1K-3K - light condition of working; 4K-6K - medium; 7K-8K – heavy [18].

Assume $a = 250\,mm$.

$a_1 \geq (h_2 - a) + B_{cr} + \Delta$ (3.3)

where $B_{cr} = 300\,mm$, $\Delta = 75\,mm$ - the gap in between column and overhead traveling crane.

Determination of above crane parts of cross-section:

$h_2 = 500\,mm$, if $B = 6\,m, Q \leq 125\,t$; $B = 12\,m, Q \leq 100\,t$;

$h_2 = 1000\,mm$, if $Q > 125\,t(7K/8K)$;

Assume $h_2 = 500\,mm$,

From the formula 3.3 we determined:

$a_1 = (500 - 250) + 300 + 75 = 625\,mm$, $a_1/250\,mm$

Assume $a_1 = 750\,mm$;

<u>Conclusion:</u> The condition is fulfillment.

Determination of undercrane parts of cross-section:

$h_1 = 250 + 750 = 1000\, mm$; $\qquad h_2 \geq \frac{l_2}{12}$

$500 \geq \frac{4830}{12} = 402.5\, mm$;

You have to calculate dimension λ:

$$\lambda = \frac{L - L_{cr}}{2} = \frac{36000 - 34500}{2} = 750$$

$\lambda = 750\, mm$, if $Q \leq 50\, t(without\ passage)$;

$\lambda = 1000\, mm$, if $Q \geq 50\, t(with\ passage)$;

$\lambda = 1250\, mm$, for special cranes.

$h_1 - h_2 \geq B_{cr} + \Delta(+450\, mm)$; (3.4)

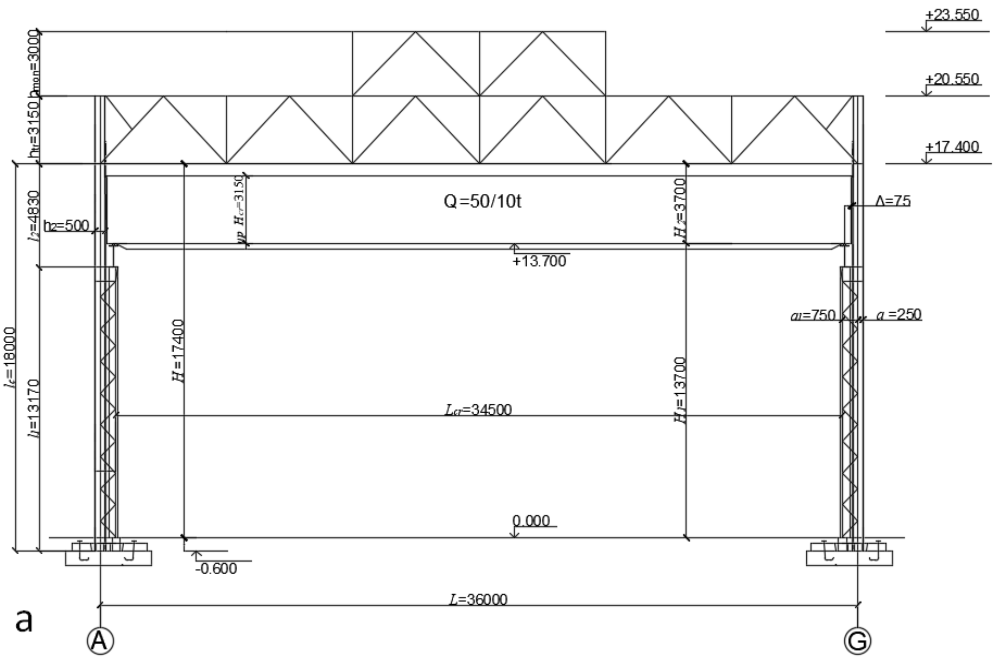
$1000 - 500 \geq 300 + 75$;

$500\, mm > 375\, mm.$

<u>Conclusion:</u> The condition is fulfillment.

$e_o=(h_1-h_2)x0,5$ (3.5)

$e_o=(500)x0,5=250$



a

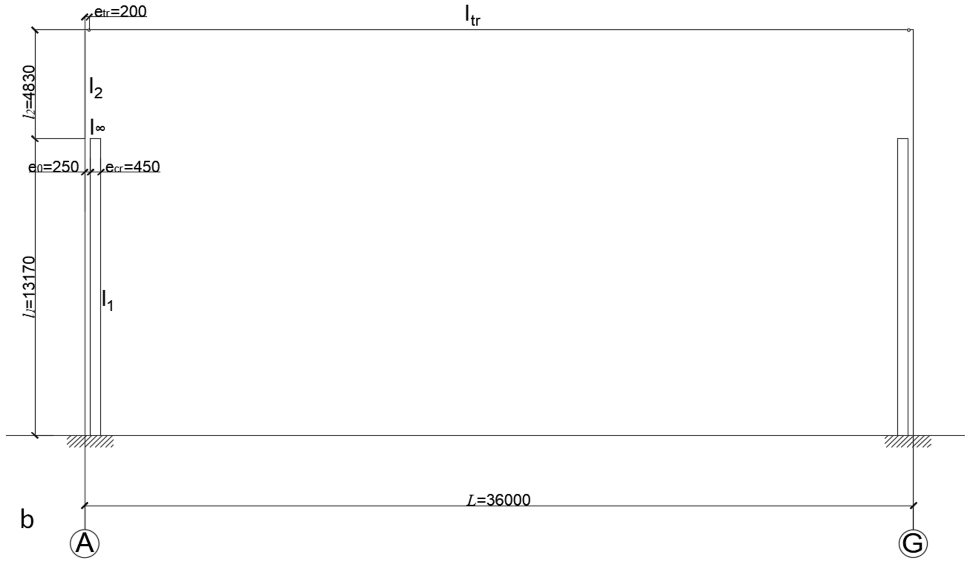


Fig.3.1. Structural scheme of one-storeyed industrial building (a), rating scheme of the frame (b).

3.3. Determination of load acting on the frame

3.3.1. Constant or dead load: Calculation of service and limit rating loads applied to the truss

Service and limit rating loads Table 3.1

Name of the layers	Service rating load $g_0, kN/m^2$	Load factor γ_f	Limit rating load $g_m, kN/m^2$
1) Protective coat (a layer of gravel treated with bituminous plastic) $t = 15mm,$ $\rho = 2000\ kg/m^3$	$g_0 = \frac{t(m)\ \cdot\ \rho(kg/m^2)}{100} = \frac{0.015\ \cdot\ 2000}{100} = 0.3$	1.3	0.39
2) Water proofing (three layers of bitumen felt bonded with bitumen mastic) $t = 40mm,$	$g_0 = \frac{0.04\ \cdot\ 400}{100} = 0.16$	1.3	0.21

$\rho = 400\ kg/m^3$			
3) Thermal insulation (Foam plastic slabs) $t = 30mm,$ $\rho = 70\ kg/m^3$	$g_0 = \frac{0.03 \cdot 70}{100} = 0.021$	1.2	0.025
4) Steam insulation (one layer of bituminous felt)	0.05	1.3	0.07
5) Corrugated steel sheets H60-845-07	0.11	1.05	0.12
Total	$\sum g_{0,t} = 0.64$		$\sum g_{m,t} = 0.82$

Purlins

$l - 6m$; $g_{0,pur} = 1.1\ kN/m^2$; 1.05 ; $g_{m,pur} = 1.16\ kN/m^2$;

(g_0 = 1.1…1.15);

Truss

$l - 36m$;$g_{0,tr} = K \cdot l = 0.36\ kN/m^2$;1.05;$g_{m,tr} = 0.38\ kN/m^2$

(K= 0.006…0.01)

Bracings

$g_{0,bra} = 0.06\ kN/m^2$;1.05;$g_{m,bra} = 0.063\ kN/m^2$;

($g_{0,bra}$=0.04…0.06kN/m^2)

Dead weight of the monitor

Panel of the monitor $g_{m,p.m} = 0.3 \cdot 1.05 = 0.315\ kN/m^2$

End panel of the monitor $g_{m,e.p.m} = 0.2 \cdot 1.05 = 0.21\ kN/m^2$

Frame of the monitor $g_{m,f.m} = 0.08 \cdot 1.05 = 0.084\,kN/m^2$

$(g_0 = 0.08\ldots0.12)$

3.3.2. Calculate linear evenly distributed load applied to the roof

$$q_{m,r} = (g_{m,t} + g_{m,pur} + g_{m,tr} + g_{m,bra} + 0.5g_{m,p.m} + g_{m,e.p.m} + g_{m,f.m}) \cdot$$
$$\cdot\ 0.5(B_1 + B)\gamma_n; \qquad (3.6)$$

$$q_{m,r} = (0.82 + 1.16 + 0.38 + 0.063 + 0.5 \cdot 0.315 + 0.21 + 0.084) \cdot 0.5 \cdot$$
$$\cdot (5.5 + 6) \cdot 0.95 = 15.7kN/m.$$

$$F_{tr} = \frac{q_{m,r} \cdot l}{2} = \frac{15.7 \cdot 36}{2} = 282.6\,kN$$

Characteristic value of load for column is equal:

$g_0 = 0.3\,kN/m^2$

Dead weight of the column:

$G_c = 0.3 \cdot 6 \cdot 18 \cdot 1.05 \cdot 0.95 = 32.32\,kN$

Top part of the column (20%(G_c)):

$G_{c,2} = 0.2 \cdot 32.32 = 6.46\,kN$

Bottom part of the column (80%(G_c)):

$G_{c,1} = 0.8 \cdot 32.32 = 25.86\,kN$

Weight of the wall panel and glazed sash:

$g_{w.p} = 2\,kN/m^2$

$g_{w.s} = 0.35\,kN/m^2$

$$F_{w.p1} = \left[g_{w,p} \cdot (l_1 - 0.6 - h_{g.s1}) \cdot \gamma_f + g_{g.s} \cdot h_{g.s} \cdot \gamma_f\right] \cdot\cdot\ 0.5(B_1 + B) \cdot \gamma_n =$$

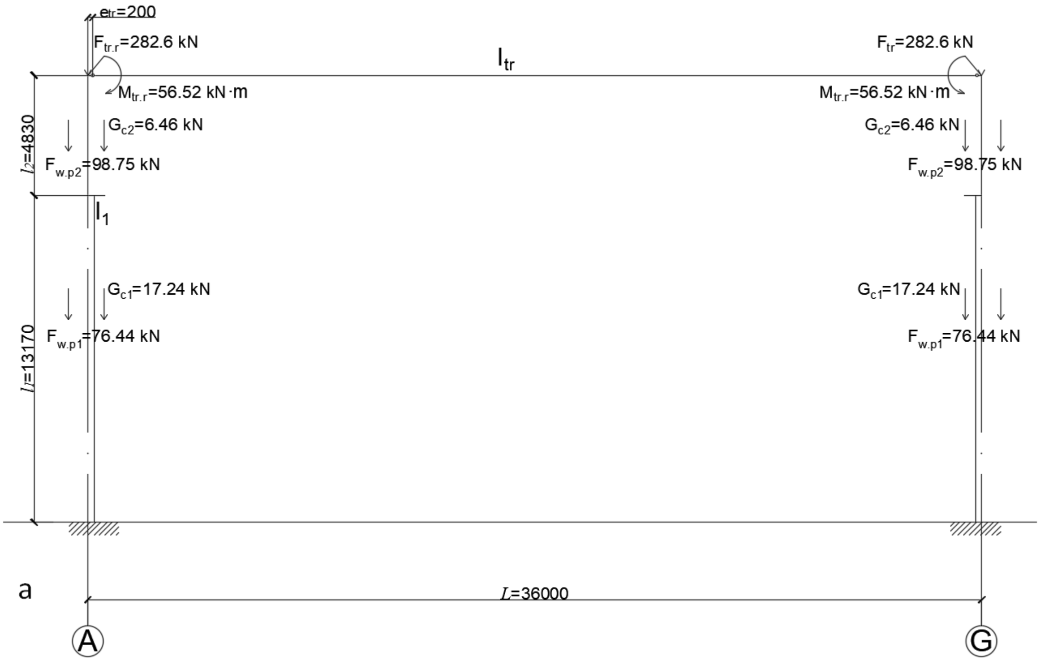
$$[2 \cdot (12.57-6)\cdot 1.2+0.35\cdot 6\cdot 1.1]\ \cdot\cdot\ 0.5(5.5+6)\cdot 0.95=98.75\ kN$$

$$F_{w.p2}=\left[g_{w,p}\ \cdot (l_2+3.15+0.65-h_{g.s2})\cdot \gamma_f+g_{g.s}\cdot h_{g.s}\cdot \gamma_f\right]\cdot$$

$$\cdot\ 0.5(B_1+B)\cdot \gamma_n=[2\ \cdot (4.83+3.8-2.4)\cdot 1.2+0.35\ \cdot 2.4\ \cdot 1.1]\cdot 0.5(5.5+6)\cdot 0.95$$
$$=86.72\ kN$$

If bearing reaction of truss isn’t coincide with center of the gravity top part of the column in top cross-section of the column induced additional bending moment:

$$M_{tr.r}=F_{tr}\cdot e_{tr}=282.6\cdot 0.2=56.52\ kN\cdot m$$



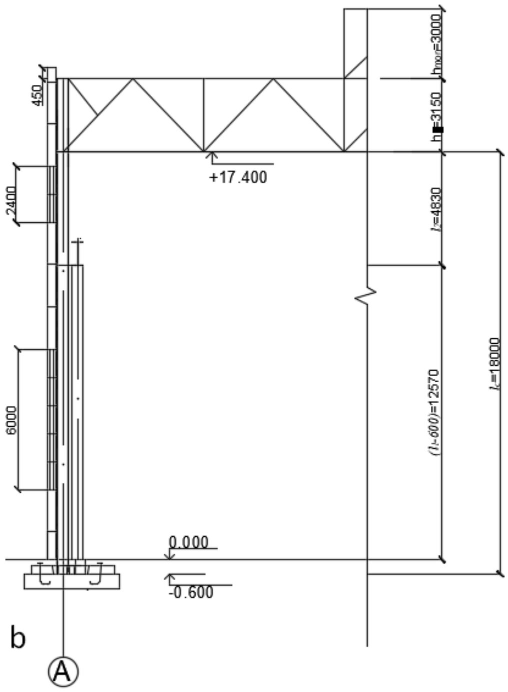
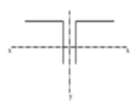
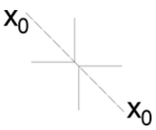
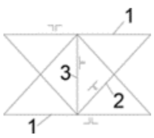
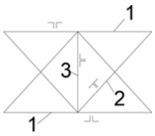
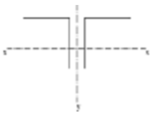
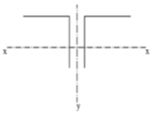
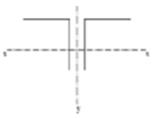


Fig.3.2. Rating scheme for constant load (a), Structural scheme of one-storeyed industrial building (b).

3.4. Determination of cross-section of roof braces

Roof braces specification Table 3.2

Mark	Sketcher	Designation	λ_u	l_x	l_y	Necessary radiuses of inertia, cm		Cross-section	Radius of inertia, cm	
						$\frac{i_x}{i_y}$	$\frac{i_{x0}}{i_{y0}}$		$\frac{i_x}{i_y}$	$\frac{i_{x0}}{i_{y0}}$
VB1		-	200	682	1364	$\frac{3.41}{6.82}$	-	2∟ 160· 10	$\frac{4.96}{6.84}$	-
VB2		-	200	411	411	-	$\frac{2.06}{-}$	2∟56 · 5	-	$\frac{2.06}{-}$
VB3		1	200	275	550	$\frac{1.38}{2.75}$	-	2∟ 63 · 5	$\frac{1.94}{2.89}$	-
		2	200	190	380	$\frac{0.95}{1.9}$	-	2∟50 · 5	$\frac{1.53}{2.38}$	-

		3	200	285	285	$\frac{1.43}{1.43}$	-	$2\llcorner 50\cdot 5$	$\frac{1.53}{2.38}$	-
VB4		1	200	275	550	$\frac{1.38}{2.75}$	-	$2\llcorner\ 63\cdot 5$	$\frac{1.94}{2.89}$	-
		2	200	184	368	$\frac{0.92}{1.84}$	-	$2\llcorner 50\cdot 5$	$\frac{1.53}{2.38}$	-
		3	200	240	240	$\frac{1.2}{1.2}$	-	$2\llcorner 50\cdot 5$	$\frac{1.53}{2.38}$	-
LB2		-	200	600	600	$\frac{3.0}{3.0}$	-	$2\llcorner\ 70\cdot 5$	$\frac{2.16}{3.16}$	-
LB3		-	400	600	600	$\frac{1.5}{1.5}$	-	$2\llcorner 50\cdot 5$	$\frac{1.53}{2.38}$	-
LB4		-	200	324	648	$\frac{1.62}{3.24}$	-	$2\llcorner\ 75\cdot 5$	$\frac{2.31}{3.35}$	-
LB5		-	200	820	820	$\frac{4.1}{4.1}$	-	$2\llcorner 140\cdot 9$	$\frac{4.34}{6.02}$	-

3.5. Determination of the Snow load

Limit rating value of snow load per $1m^2$ of horizontal projection of the roof is calculated by the formula[6]:

$$S_m = \gamma_{fm}\cdot\ S_0\cdot\ C, kN/m^2 \qquad (3.7)$$

Where γ_{fm}- coefficient of reliability on limit value of snow load that depends on assumed average period of repetition T and for prefabricated construction period of repetition $T = T_{ef}$ and T_{ef} available from (B) its term of working conditions and for industrial buildings assume 60 years.

γ_{fm} available from table 8.1 and $\gamma_{fm} = 1.04$.

S_0- characteristic value of snow load for the region of building in accord with fig.8.1 or appendix E.

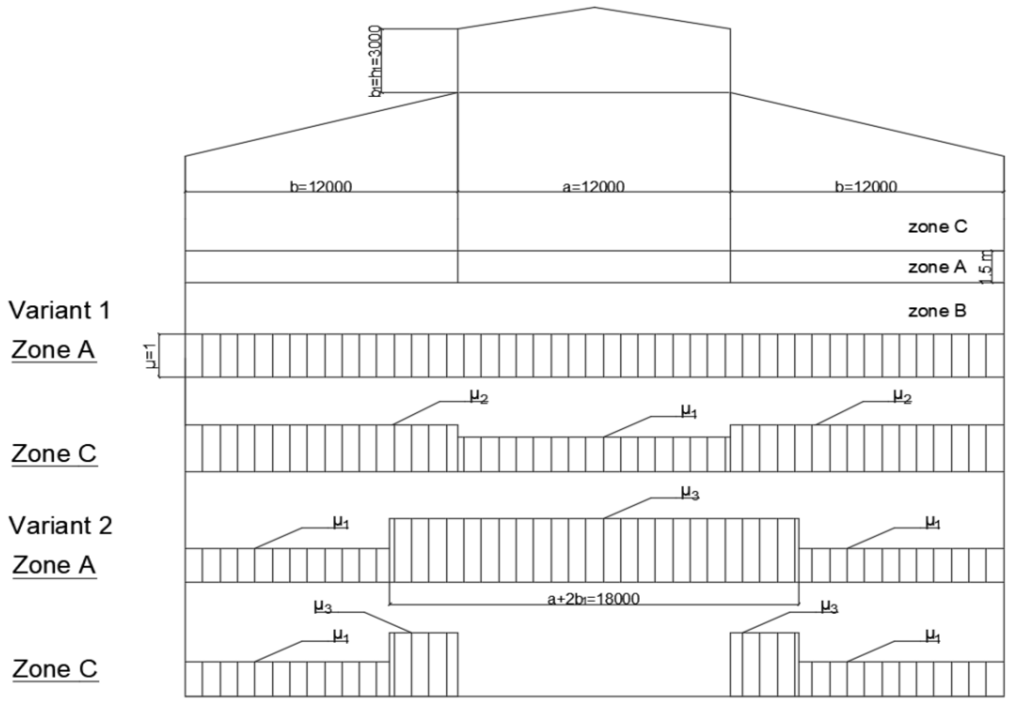
$S_0 = 1.46\ kN/m^2 (kPa)$.

C - coefficient that is calculated by the formula: $C = \mu \cdot C_e \cdot C_{alt}$,

Where μ - coefficient of conversion from weight of snow cover on the ground to the snow load on the roofing that depends on contour of covering, coefficient μ available from appendix Ж and assume №3 for industrial buildings with longitudinal monitors.

$\mu_1 = 0.8$;

$\mu_2 = 1 + 0.5a/b = 1 + 0.5 \cdot 12/12 = 1.1$;



$\mu_3 = 1 + 0.5a/b_1 = 1 + 0.5 \cdot 12/3 = 3$;

Fig.3.3. Industrial buildings with longitudinal monitors (scheme 3 from appendix Ж)

C_e - coefficient that takes into account influence of working conditions on accumulation of snow (cleaning, melting) on the roof – paragraph 8.9 ДБН (for heated buildings assume $C_e = 1.0$).

C_{alt} - coefficient that depends on height (H, km) of arrangement of building above sea level: if $H < 0.5km$, $C_{alt} = 1.0$.

Site of building – Zhytomyr

$S_0 = 1.46\ kN/m^2$

$\mu_Q = 1.23$

$\gamma_{fm} = 1.04$

$C_e = 1.0$

$C_{alt} = 1.0$

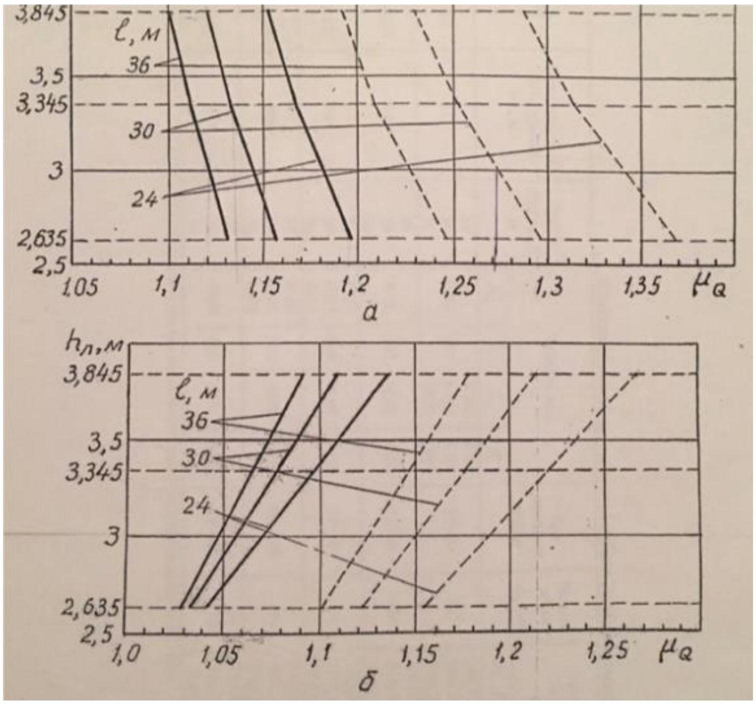


Fig.3.4. Graphics for determination coefficient $\mu_{\boldsymbol{Q}}$: a) - when service rating roof constant load less than 1.5 kPa, b) - when service rating roof constant load more than 1.5 kPa; solid line - column spacing 12.0 m; dashed line - column spacing 6.0 m.

Linear evenly distributed load applied to the roof truss is calculated by the formula:

$$q_{m,s} = \gamma_{fm} \cdot S_o \cdot C_e \cdot C_{alt} \cdot \mu_Q \cdot 0.5(B_1 + B) \cdot \gamma_n; \qquad (3.8)$$

$$q_{m,s} = 1.04 \cdot 1.46 \cdot 1 \cdot 1 \cdot 1.23 \cdot 0.5(5.5 + 6) \cdot 0.95 = 10.83\ kN/m$$

Concentrated load applied to the column is calculated by the formula:

$$F_{tr,s} = \frac{q_{m,s} \cdot L}{2} = \frac{10.83 \cdot 36}{2} = 194.94\ kN$$

Bending moment applied to the column is calculated by the formula:

$$M_{tr,s} = F_{tr,s} \cdot e_{tr} = 194.94 \cdot 0.2 = 38.99\ kN \cdot m$$

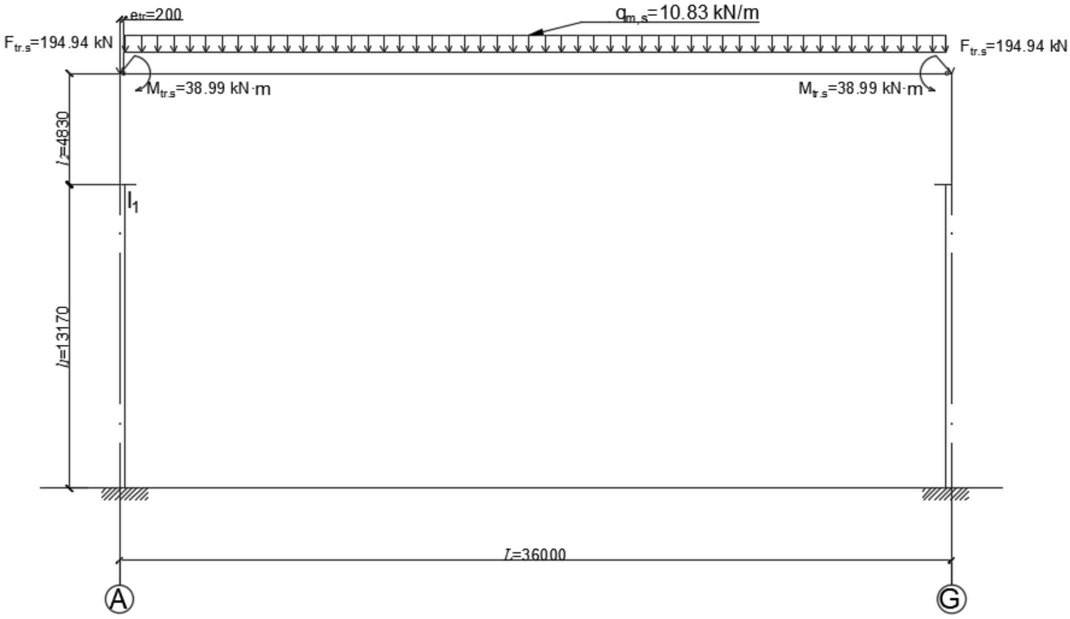


Fig.3.5. Rating scheme for snow load

3.6. Determination of the Wind load applied to the transverse frame of one-storeyed industrial building

Characteristic value of wind load in accord with Building Standards [6] is calculated by the formula:

$$W_m = \gamma_{fm} \cdot W_0 \cdot C, kN/m^2 \qquad (3.9)$$

where γ_{fm} - coefficient of reliability on limit value of wind load that depends on condition of working available from table 9.1. of Building code [6]

Working period for industrial buildings equals 60 years.

γ_{fm} - from table 9.1 and $\gamma_{fm} = 1.035$;

W_0 - its characteristic value of wind load from appendix E. [6].

$W_0 = 0.46\ kN/m^2 (kPa)$ - Zhytomyr

C – coefficient that is calculated by the formula:

$$C = C_{aer} \cdot C_h \cdot C_{dir} \cdot C_d \cdot C_{alt} \cdot C_{rel} \qquad (3.10)$$

Calculation of aerodynamic coefficient for backwind pressure in accord with table scheme №2 [6]:

$\frac{b}{l} = \frac{84}{36} = 2.3;$

$\frac{h_1}{l} = \frac{21.4}{36} = 0.59 \Rightarrow C_{e3} = -0.6;$

Coefficient is calculated in accord with table Б.2 [6] in terms of distance z_1 or z_2.

For designation coefficient $C_{h(e)}$ we use coefficients for building code for III type of building side:

III on 10 m – $K_1 = 1.2$;

on 20 m – $K_2 = 1.55$;

on 60 m – $K_3 = 2.25$;

$H_r = H_{r.t} + h_{mon} + 500mm = 3150 + 2500 + 500 = 6150mm;$

H_0 up to 20 m;

$F_w = 0.5 \cdot (K_{1-2} \cdot z_3 + K_2 \cdot H_r + K_{2-3} \cdot z_4) \cdot \gamma_{fm} \cdot W_0 \cdot C_{aer} \cdot C_d \cdot B \cdot \gamma_n;$ (3.11)

$z_1 = 7.4\,m;$

$z_0 = 6\,m;$

$z_2 = 8\,m;$

$1.192 + \frac{1.244-1.192}{8-6}(7.4-6) = 1.2284;$

$q_w = \gamma_{fm} \cdot W_0 \cdot C_{aer} \cdot C_{h(e)} \cdot C_d \cdot B \cdot \gamma_n;$ (3.12)

$\gamma_{fm} = 1.035;\quad W_0 = 0.46\,kN/m^2;\quad C_{aer} = 0.8;\quad C_{h(e)} = 1.2284;$

$C_d = 0.9;\qquad B = 6m;\qquad \gamma_n = 0.95;$

From formula 3.12, we have:

$q_w = 1.035 \cdot 0.46 \cdot 0.8 \cdot 1.2284 \cdot 0.9 \cdot 6 \cdot 0.95 = 2.4\,kN/m;$

$q_w' = q_w \cdot \frac{C_{aer}^-}{C_{aer}^+} = 2.4 \cdot \frac{-0.6}{0.8} = -1.8\,kN/m;$

Concentrated load applied to the bottom chord of roof truss:

$F_w = 0.5 \cdot (K_{1-2} \cdot z_3 + K_2 \cdot H_r + K_{2-3} \cdot z_4) \cdot \gamma_{fm} \cdot W_0 \cdot C_{aer} \cdot C_d \cdot B \cdot \gamma_n;$

$K_{1-2} = K_1 + \frac{K_2-K_1}{10} \cdot z_1 = 1.2 + \frac{1.55-1.2}{10} \cdot 1.2284 = 1.243;$

$K_{2-3} = K_2 + \frac{K_3-K_2}{40} \cdot z_4 = 1.55 + \frac{2.25-1.55}{40} \cdot 3.55 = 1.61;$

$$F_w = 0.5 \cdot (1.243 \cdot 2.6 + 1.55 \cdot 6.15 + 1.61 \cdot 3.55) \cdot 1.035 \cdot$$
$$\cdot\ 0.46 \cdot 0.8 \cdot 0.9 \cdot 6 \cdot 0.95 = 18.05\ kN;$$
$$F_w' = F_w \cdot \frac{C_{aer}^-}{C_{aer}^+} = 18.05 \cdot \frac{-0.6}{0.8} = -13.54\ kN/m;$$

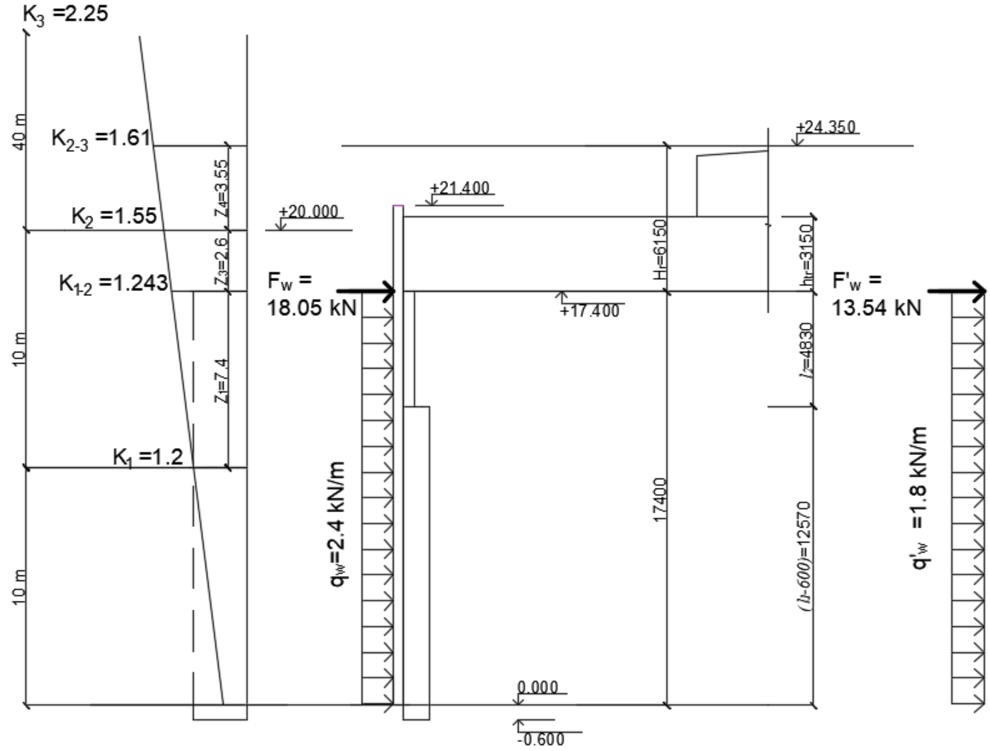


Fig.3.6. Scheme of wind load applied to the transfers frame of one-storeyed industrial building

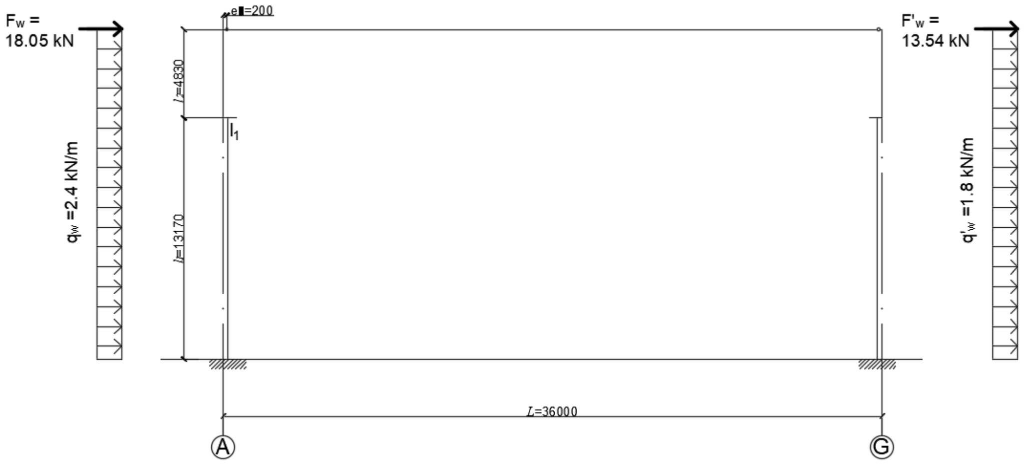


Fig.3.7. Rating scheme for wind load

3.7. Determination of the Crane load

Maximum limit rating vertical load is calculated by the formula, if both cranes have the same capacity:

$$F_{max} = (\gamma_{fm} \cdot \psi \cdot F_0 \cdot \sum y_i + \gamma_{fm,cr.b} \cdot G_{cr.b}) \cdot \gamma_n; \quad (3.13)$$

Where γ_{fm} - coefficient of reliability on limit rating crane load that depends on average period of repetition.

Period of repetition of overhead traveling cranes equals 20 years in accord with table 7.1 [18], γ_{fm} calculated with the help of interpolation.

$$1.07 + \frac{1.1-1.07}{50-10} = (20-10) = 1.078;$$

ψ - coefficient of combination for crane load that equals for two cranes;

$$\psi = 0.85(1K-6K), \quad \psi = 0.95(7K-8K).$$

F_0 - support pressure of cranes wheels;

$\sum y_i$ - sum of ordinates of forces line for crane beam if combination of wheels arrangement is the most unfavorable;

$\gamma_{fm,cr.b}$ - coefficient of reliability for metal crane beam in accord with table 5.1 ДБН “Load and influences” [6];

$G_{cr.b}$ - mass of crane beam that is calculated by the formula:

$$G_{cr.b} = q_{cr.b} \cdot B;$$

Where $q_{cr.b}$ – linear mass of crane beam;

B – column spacing.

$B = 6.0m$; $\qquad q_{cr.b} = 2.5\ldots3.5\ kN/m$;

$B = 12.0m$; $\qquad q_{cr.b} = 4.0\ldots5.0\ kN/m$;

When the left column of the transfer frame is loaded by maximum force the right column is loaded by minimum force.

F_{min} is calculated by the same formula that F_{max} but with replacement of F_0 by F_0'.

$$F_0' = \frac{Q+G_k}{n_0} - F_0; \qquad (3.14)$$

Q - capacity of crane transfer into kN;

G_k - weight of the crane with crap;

n_0 - number of wheels from one side of the crane.

Forces and are applied to the undercrane part of the column with eccentricity and induced bending moments:

$$M_{max} = F_{max} \cdot e_{cr}; \qquad (3.15)$$

$$M_{min} = F_{min} \cdot e_{cr}; \qquad (3.16)$$

e_{cr} is the distance from the center of the gravity undercrane part of the column to the axis of the crane beam.

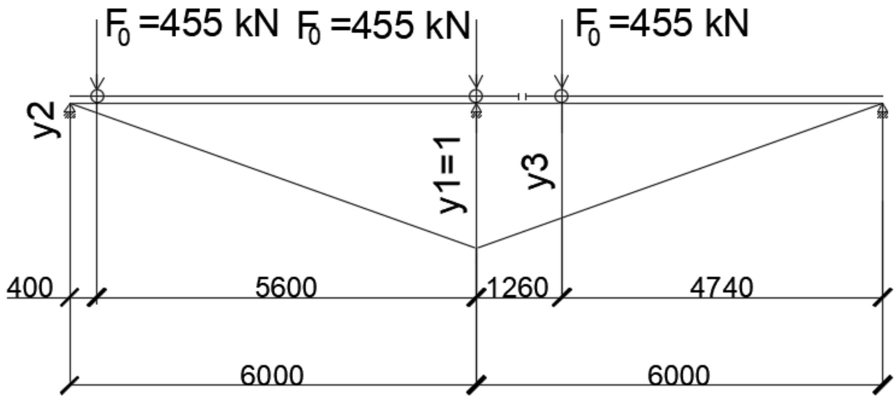


Fig.3.8. Scheme of setting two 4-wheels canes on a crane beam when the column spacing equals 6.0 m.

$Q = 50/10t;$ $\qquad G_k = 716kN;$

$F_0 = 455\ kN;$ $\qquad G_{cr.b} = 3.5 \cdot 6 = 21\ kN;$

$K = 5600\,mm;$ $\qquad y_1 = 1;$

$B_2 = 6860\,mm;$ $\qquad y_2 = \frac{1}{6} \cdot x_2 = \frac{0.4}{6} = 0.067;$

$\gamma_{fm} = 1.078;$ $\qquad y_3 = \frac{1}{6} \cdot x_3 = \frac{4.74}{6} = 0.79;$

$\psi = 0.85;$

$$F_{max} = [\gamma_{fm} \cdot \psi \cdot F_0 \cdot \sum y_i + \gamma_{fm,cr.b} \cdot G_{cr.b}] \cdot \gamma_n; \qquad (3.17)$$

$F_{max} = [1.078 \cdot 0.85 \cdot 455 \cdot (1 + 0.067 + 0.79) + 1.05 \cdot 21] \cdot$
$\cdot\ 0.95 = 756.45\ kN;$

$$F_0' = \frac{Q+G_k}{n_0} - F_0 = \frac{50 \cdot 9.8 + 716}{2} - 455 = 148\ kN;$$

$$F_{min} = [\gamma_{fm} \cdot \psi \cdot F_0' \cdot \sum y_i + \gamma_{fm,cr.b} \cdot G_{cr.b}] \cdot \gamma_n; \qquad (3.18)$$

$$F_{min} = [1.078 \cdot 0.85 \cdot 148 \cdot (1 + 0.067 + 0.79) + 1.05 \cdot 21] \cdot \cdot 0.95 = 260.19\ kN;$$

Eccentricity of crane assume:

$e_{cr} = (0.45\ldots0.47)h_1;$

$e_{cr} = 450\ mm;$

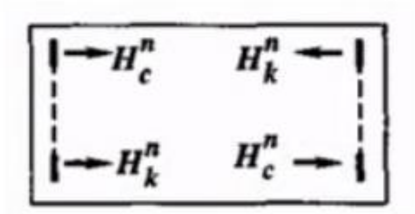
$$M_{max} = F_{max} \cdot e_{cr} = 756.45 \cdot 0.45 = 340.4\ kN \cdot m;$$

$$M_{min} = F_{min} \cdot e_{cr} = 260.19 \cdot 0.45 = 117.1\ kN \cdot m;$$

Horizontal crane load that is directed across crane rails for wheel cranes is determined from one crane that has the most unfavorable value of load. There are 4 variants of application horizontal forces for cranes wheels in accord with fig.7.1 ДБН [6].

The maximum characteristic value for one wheel is calculated by the formula:

$$H_k = 0.1F_0 + \frac{\alpha \cdot (F_0 - F_0') \cdot L_{cr}}{K}; \qquad (3.19)$$



Where α – coefficient that equals 0.03 if crane has central driving gear, $\alpha = 0.01$, if crane has separate gear;

L_{cr} – span of the crane;

K – base of the crane;

For other wheels assume $H_c = 0.1F_0$, $H_c = 0.1F_0'$;

$$H_k = 0.1 \cdot 455 + \frac{0.03 \cdot (455 - 148) \cdot 34.5}{5.6} = 102.2\ kN;$$

$$H_c = 0.1F_0 = 0.1 \cdot 455 = 45.5\ kN;$$

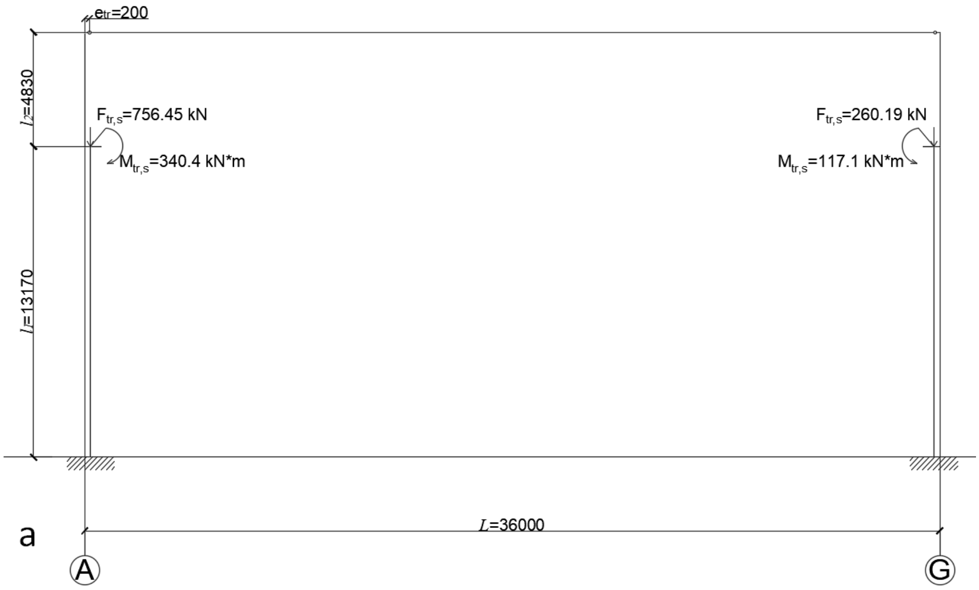
$$H_{max} = \pm\gamma_{fm} \cdot \psi \cdot (-H_k \cdot 1 - H_c \cdot y_2) \cdot \gamma_n; \quad (3.20)$$

$$H_{max} = \pm 1.078 \cdot 0.85 \cdot (-102.2 \cdot 1 - 45.5 \cdot 0.067) \cdot 0.95 = \pm 91.62\ kN;$$

$$H_c = 0.1F_0 = 0.1 \cdot 148 = 14.8\ kN;$$

$$H_{min} = \pm\gamma_{fm} \cdot \psi \cdot (H_c \cdot 1 - H_k \cdot y_2) \cdot \gamma_n; \quad (3.21)$$

$$H_{min} = \pm 1.078 \cdot 0.85 \cdot (14.8 \cdot 1 - 102.2 \cdot 0.067) \cdot 0.95 == \pm 6.92\ kN;$$



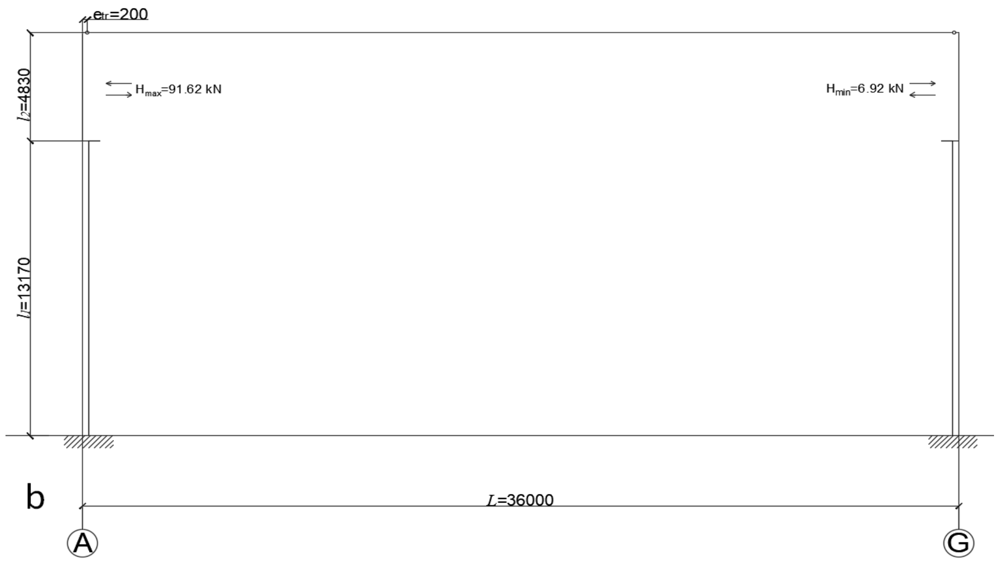


Fig.3.9. Rating scheme for vertical crane load (a), rating scheme for horizontal crane load.

3.8. Statical calculation of two-hinged frame of one-storeyed industrial building

3.8.1. Determination of forces in the frame and columns cross-section for every type of load

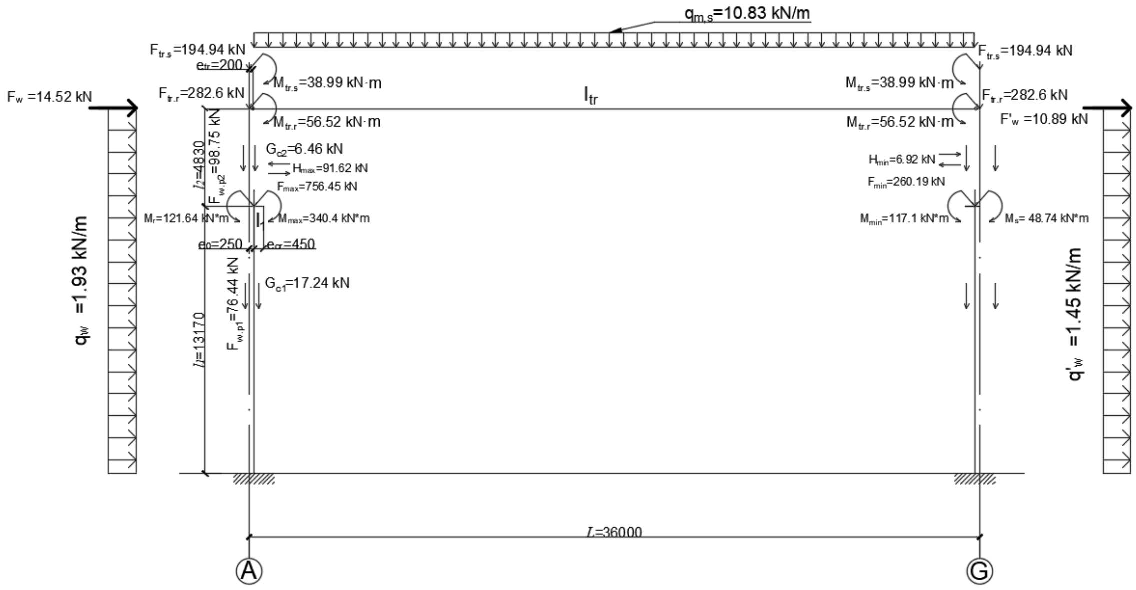


Fig.3.10. Rating scheme of transverse one-storeyed industrial frame

Single span frame with hinged truss-to-columns connections is one time statically indeterminate system. It's necessary to set the ratios of the column's cross-section moments of inertia. Differences between real moments of inertia and given up to 30% have low influence on results of calculations (efforts in elements of the frame). Approximate value the ratio of moment of inertia is calculated by the formulas:

$$n = \frac{I_1}{I_2} = 2 \cdot \frac{h_1}{h_2} \cdot \frac{0.05 \cdot (F_{tr.r} + F_{tr.s}) \cdot L + 4M_{max} + 2q_w \cdot l_c^2}{0.05 \cdot (F_{tr.r} + F_{tr.s}) \cdot L + 2M_{max} + 0.5q_w \cdot l_c^2}; \qquad (3.22)$$

$$n = 2 \cdot \frac{1}{0.5} \cdot \frac{0.05 \cdot (282.6 + 194.94) \cdot 36 + 4 \cdot 340.4 + 2 \cdot 1.93 \cdot 18^2}{0.05 \cdot 282.6 + 194.94) \cdot 36 + 2 \cdot 340.4 + 0.5 \cdot 1.93 \cdot 18^2} = 7.94$$

$n = \left(\frac{h_1}{h_2}\right)^2 \cdot K$; $K = 1.8\ldots2.0$; $n = 5.\ldots12$;

$n = \left(\frac{1}{0.5}\right)^2 \cdot 1.9 = 7.6$;

M_{max} – maximum bending moment from vertical crane load,

$M_{max} = 340.4\ kN \cdot m$; $q_w = 1.93\ kN/m$.

Important cross-sections of the frame are cross-section 0-0.

0-0 – the head of the column;

1-1 – the bottom part above crane part of the column;

2-2 – the upper part undercrane part of the column;

3-3 – the base of the column.

When we coincide $F_{tr.r}$ and $F_{tr.s}$ with the axis of the column there are induced bending moments.

$e_{tr} = 0.2m$

$e_0 = 0.3$

$e_0 = 0.5 \cdot (h_1 - h_2) = 0.5 \cdot (1000 - 500) = 250mm$;

$e_0 = h_1 - 0.5 \cdot h_2 = 1000 - 0.5 \cdot 500 = 300\,mm$;

Bending moments in cross-section 2-2 equals:

● for dead load:

$M_r = (F_{tr.r} + G_{c2}) \cdot e_0 + F_{wp2} \cdot \left(e_0 + \frac{h_2}{2}\right)$;   (3.23)

● for snow load:

$M_s = F_{tr.s} \cdot e_0$;   (3.24)

During calculation it's necessary to take into account signs of effort.

Signs of bending moments:

● Bending moment that induces tension stresses in internal fibers of the frame has the sign “+”, but the compression stresses has sign “-”.

● Shear force has the sign “+” if the element's axis moves to the tangent of bending moment diagram by clockwise, in other case shear force has sign “-”.

● Longitudinal force has “+” if it tensions the element and “-” if compressed.

Unknown force x_r from dead load is calculated by the formula:

$$x_r = \frac{3}{2\cdot[(n-1)\cdot l_2^3 + l^3]} \cdot \{M_{tr,r}{}^{(-)}[(n-1) \cdot l_2^2 + l_c^2] + M_r(l_c^2 + l_2^2)\}; \quad (3.25)$$

Unknown force x_s from snow load is calculated by the formula:

$$x_s = \frac{3}{2\cdot[(n-1)\cdot l_2^3 + l^3]} \cdot \{M_{tr,s}{}^{(-)}[(n-1) \cdot l_2^2 + l_c^2] + M_s(l_c^2 + l_2^2)\}; \quad (3.26)$$

Bending moment 2-2 (dead load) from formula 3.24:

$M_s = 194.94 \cdot 0.25 = 48.74\ kN \cdot m;$

$$x_r = \frac{3}{2 \cdot [(7.6-1) \cdot 4.83^3 + 18^3]} \cdot \{-56.52[(7.6-1) \cdot 4.83^2 + 18^2] +$$

$$+121.64 \cdot (18^2 + 4.83^2)\} = 3.92\ kN;$$

$$x_s = \frac{3}{2 \cdot [(7.6-1) \cdot 4.83^3 + 18^3]} \cdot \{-38.99[(7.6-1) \cdot 4.83^2 + 18^2] +$$

$$+48.74 \cdot (18^2 + 4.83^2)\} = -1.63\ kN;$$

3.8.2. Dead load

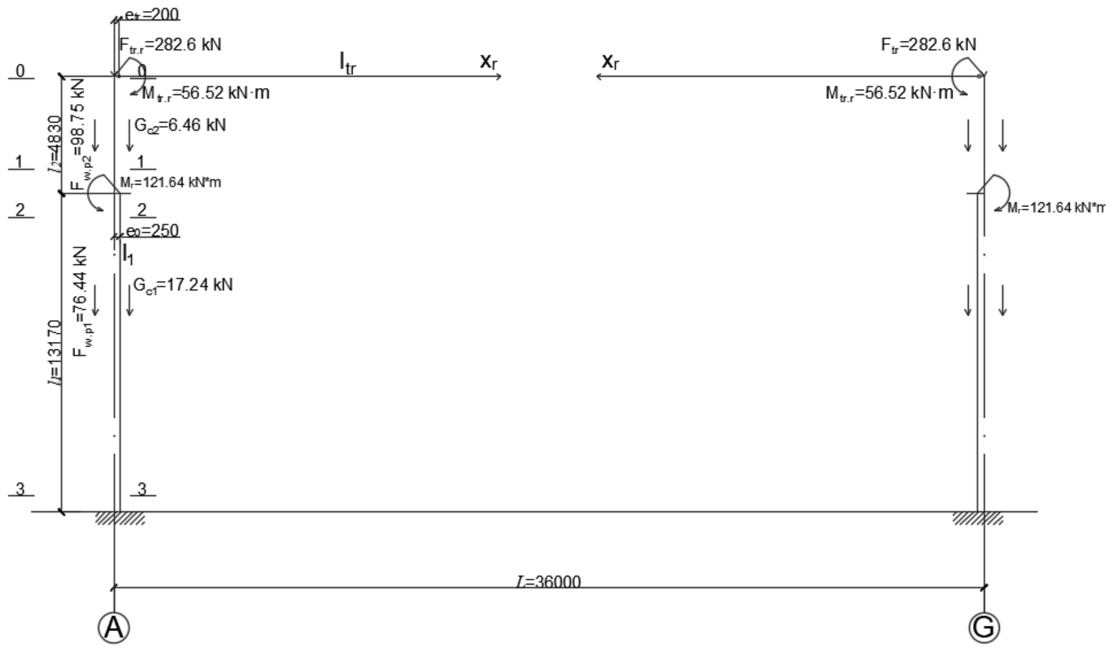


Fig.3.11. Rating scheme for constant load

$M_0 = -M_{tr,r} = -56.52\ kN \cdot m;$

$M_1 = -M_{tr,r} - x_r \cdot l_2 + F_{wp2} \cdot \frac{h_2}{2} = -56.52 - 3.92 \cdot 4.83 +$

$+98.75 \cdot \frac{0.5}{2} = -50.77\ kN;$

$M_2 = -M_{tr,r} - x_r \cdot l_2 + M_r = -56.52 - 3.92 \cdot 4.83 + 121.64 =$

$= 46.19\ kN \cdot m;$

$M_3 = -M_{tr,r} - x_r \cdot l_c + M_r + F_{wp1}(h_1 - e_{cr}) = -56.52 - 3.92\ \cdot$

$\cdot 18 + 121.64 + 76.44 \cdot (1 - 0.45) = 36.6\ kN \cdot m\ ;$

$N_0 = -F_{tr,r} = -282.6\ kN;$

$N_1 = -F_{tr,r} - G_{c2} - F_{wp2} = -282.6 - 6.46 - 98.75 = -387.81\ kN$

$N_2 = N_1 = -387.81\ kN;$

$N_3 = N_1 - G_{c1} - F_{wp1} = -387.81 - 17.24 - 76.44 = -481.49\ kN$

$Q_0 = Q_1 = Q_2 = Q_3 = \pm x_r = 3.92\ kN;$

3.8.3. Snow load

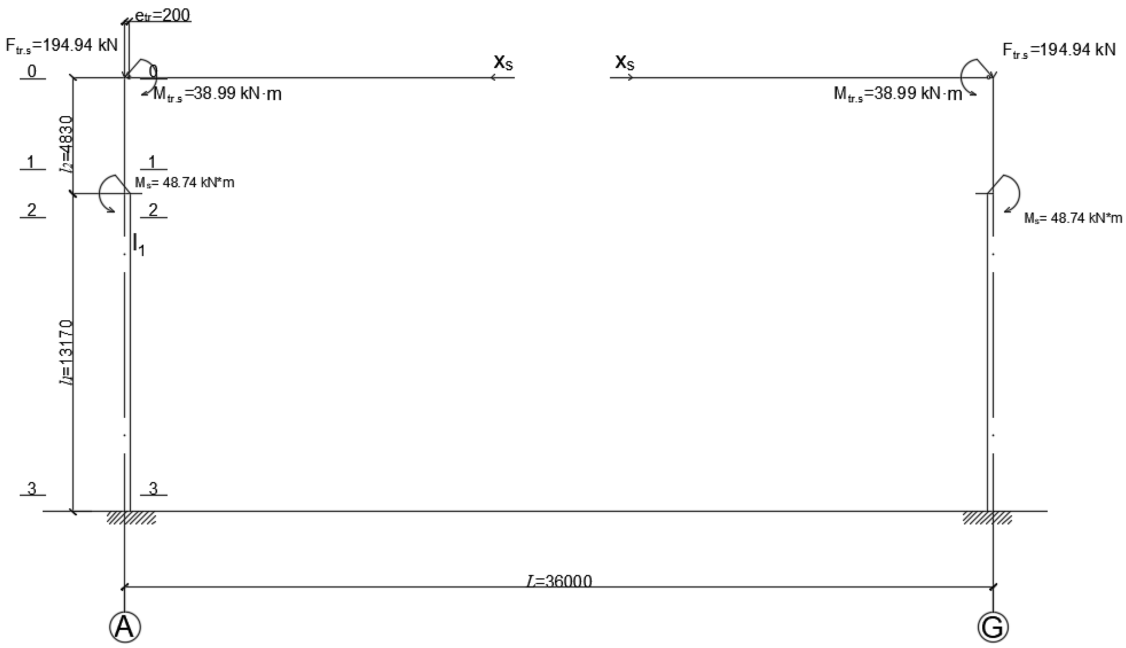


Fig.3.12. Rating scheme for snow load

$M_0 = -M_{tr,s} = -38.99\ kN \cdot m;$

$M_1 = -M_{tr,s} + x_s \cdot l_2 = -38.99 + 1.63 \cdot 4.83 = -31.12\ kN \cdot m;$

$M_2 = -M_{tr,s} + x_s \cdot l_2 + M_s = -38.99 + 1.63 \cdot 4.83 + 48.74 =$

$= 17.62\ kN \cdot m;$

$$M_3 = -M_{tr,s} + x_s \cdot l_c + M_s = -38.99 + 1.63 \cdot 18 + 48.74 =$$

$= 39.1\ kN \cdot m;$

$$N_0 = N_1 = N_2 = N_3 = -F_{tr.s} = -194.94\ kN;$$

$$Q_0 = Q_1 = Q_2 = Q_3 = \pm x_s = -1.63\ kN;$$

3.8.4. Determination efforts from vertical crane load when on the left and right column:

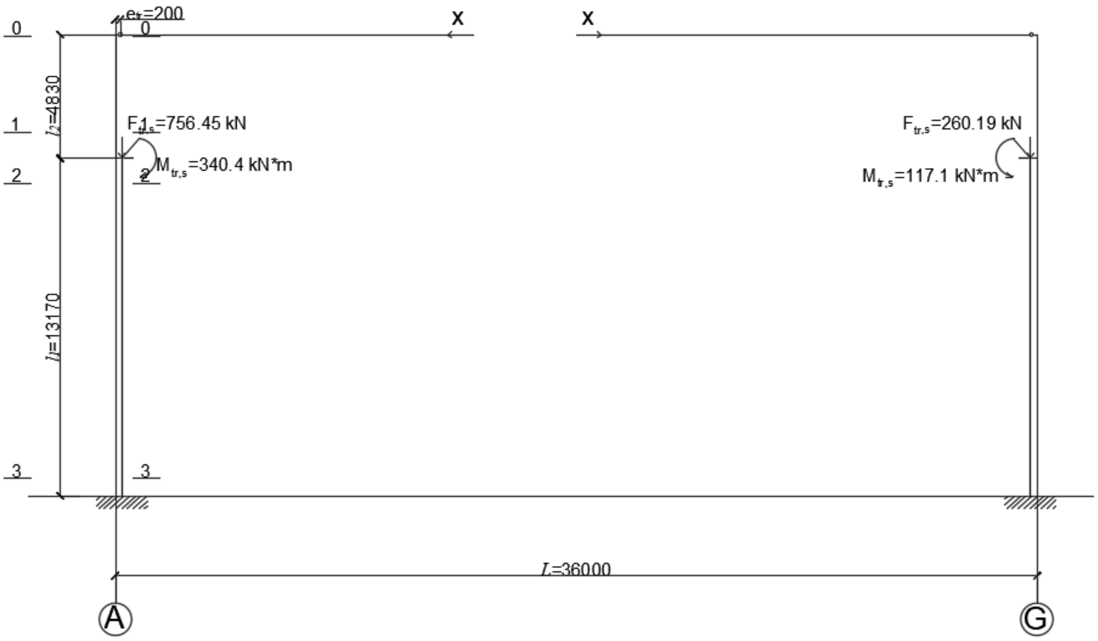


Fig.3.13. Rating scheme for vertical crane load

$$x = \frac{M_{left}}{l_c} \cdot \frac{3(\alpha_2 - \eta^2)}{4\alpha_3} \cdot (1 + \varepsilon_1), \quad (3.27) \qquad \eta = \frac{l_c - l_1}{l_c} = \frac{18000 - 13170}{18000} = 0.27$$

$$\alpha_i = \frac{(n-1)\eta^i + 1}{n}, \qquad (3.28) \qquad \varepsilon_1 = \frac{M_{right}}{M_{left}} = \frac{117.1}{340.4} = 0.344$$

$$\alpha_2 = \frac{(7.6-1)0.27^2 + 1}{7.6} = 0.195$$

$$\alpha_3 = \frac{(7.6-1)0.27^3 + 1}{7.6} = 0.149$$

$$x = \frac{340.4}{18} \cdot \frac{3(0.195 - 0.27^2)}{4 \cdot 0.149} \cdot (1 + 0.344) = 15.62\ kN$$

$M_0 = 0\ kN \cdot m;$

$M_1 = x \quad \cdot l_2 = 15.62 \cdot 4.83 = 75.44\ kN \cdot m;$

$M_2{}^{left} = M_1 - M_{max} = 75.44 - 340.4 = -228.96\ kN \cdot m;$

$M_2{}^{right} = M_1 - M_{min} = 75.44 - 117.1 = 41.66\ kN \cdot m;$

$M_3{}^{left} = x \quad \cdot l_c - M_{max} = 15.62 \cdot 18 - 340.4 = -59.24\ kN \cdot m;$

$M_3{}^{right} = x \quad \cdot l_c - M_{min} = 15.62 \cdot 18 - 117.1 = 164.06\ kN \cdot m;$

$N_0 = N_1 = 0\ kN;$

$N\,{}_2^{left} = -F_{max} = -756.45\ kN;$

$N\,{}_2^{right} = -F_{min} = -260.19\ kN;$

$N\,{}_3^{left} = N\,{}_2^{left} = -756.45\ kN;$

$N\,{}_3^{right} = N\,{}_2^{right} = -260.19\ kN;$

$Q_0 = Q_1 = Q_2 = Q_3 = \pm x \quad = \pm 15.62\ kN;$

3.8.5. Determination efforts from horizontal crane load when on the left and right column:

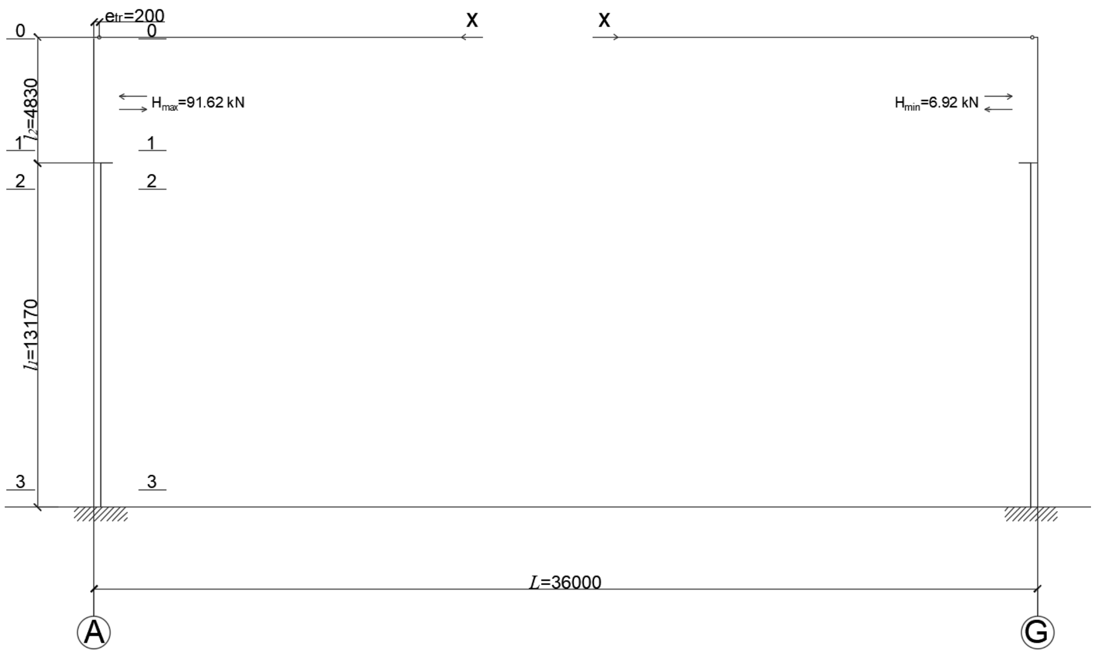


Fig.3.14 Rating scheme for horizontal crane load

$$x = \pm \frac{H_{left} - H_{right}}{2} \cdot \left[1 - \frac{\psi_1}{2\alpha_3}(3\alpha_2 - \psi_1^2\right], \qquad (3.29)$$

$\alpha_2 = 0.195,\ \alpha_3 = 0.149,$

$c = l_2 - h_{cr.b} = 4.83 - 1.0 = 3.83\ m, \quad \psi_1 = \frac{c}{l_c} = \frac{3.83}{18} = 0.21,$

$$x = \pm \frac{91.62 - 6.92}{2} \cdot \left[1 - \frac{0.21}{2\ \cdot\ 0.149}(3 \cdot 0.195 - 0.21^2\ \right] = \pm 26.21\ kN$$

$M_0 = 0\ kN \cdot m;$

$M_1 = \pm x \quad \cdot l_2 = \pm 26.21 \cdot 4.83 = \pm 126.59\ kN \cdot m;$

$M_2 \quad = M_1 = \pm 126.59\ kN \cdot m;$

$M_3{}^{left} = \pm x \quad \cdot l_c \pm H_{max} \cdot l_1 = \pm 26.21 \cdot 18 \pm 91.62 \cdot 13.17 =$

$= \pm 1678.42\ kN \cdot m;$

$M_3{}^{right} = \pm x \quad \cdot l_c \pm H_{min} \cdot l_1 = \pm 26.21 \cdot 18 \pm 6.92 \cdot 13.17 =$

$= \pm 562.92\ kN \cdot m;$

$N_0 = N_1 = N_2 = N_3 = 0\ kN;$

$Q_0 = Q_1 = \pm x = \pm 26.21\ kN;$

$Q\,_2^{left} = Q\,_3^{left} = \pm x \quad \pm H_{max} = \pm 26.21 \pm 91.62 = \pm 117.83\ kN;$

$Q\,_2^{right} = Q\,_3^{right} = \pm x \quad \pm H_{min} = \pm 26.21 \pm 6.92 = \pm 33.13\ kN;$

3.8.6. Determination of wind load:

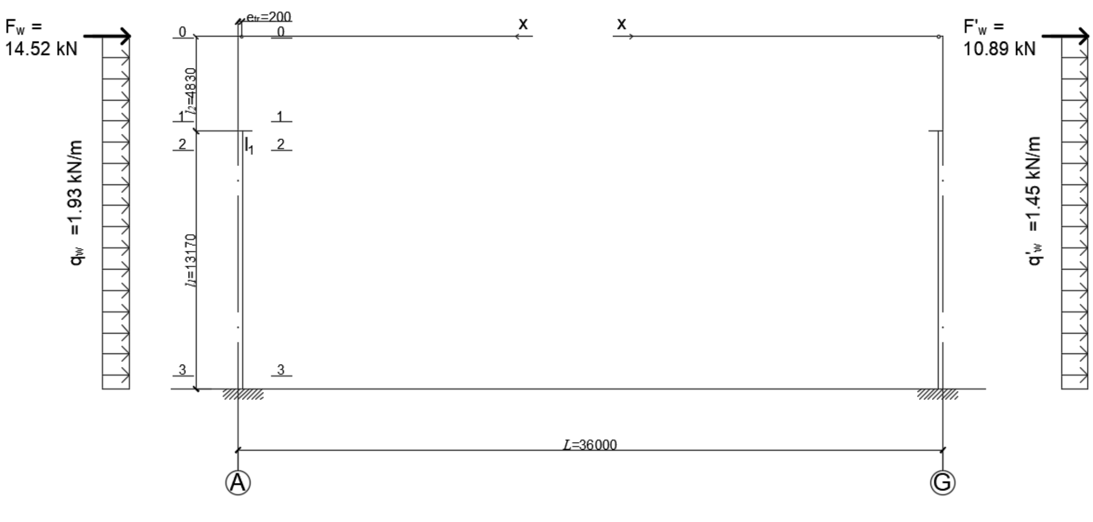


Fig.3.15. Rating scheme for wind load

$$x_F = -\frac{F_w}{2}(1-\varepsilon_3), \quad (3.30) \qquad \varepsilon_3 = \frac{F'_w}{F_w} = \frac{10.89}{14.52} = 0.75$$

$$x_F = -\frac{14.52}{2}(1-0.75) = -1.82$$

$$x_q = \frac{3}{16} \cdot q_w \cdot l_c \cdot \frac{\alpha_4}{\alpha_3} \cdot (1-\varepsilon_2), \quad (3.31) \qquad \varepsilon_2 = \frac{q'_w}{q_w} = \frac{1.45}{1.93} = 0.75$$

$$x_q = \frac{3}{16} \cdot 1.93 \cdot 18 \cdot \frac{0.138}{0.149} \cdot (1-0.75) = 1.51$$

Left wind:

$$M_0 = 0\ kN \cdot m;$$

$$M_{1e} = x \quad \cdot l_2 - q_w \cdot \frac{l_2^2}{2} = 1.51 \cdot 4.83 - 1.93 \cdot \frac{4.83^2}{2} = -15.22\ kN \cdot m;$$

$$M_{2e} \quad = M_{1e} = -15.22\ kN \cdot m;$$

$$M_{3e} \quad = x \quad \cdot l_c - q_w \cdot \frac{l_c^2}{2} = 1.51 \cdot 18 - 1.93 \cdot \frac{18^2}{2} = -285.48\ \ kN \cdot m;$$

$$M_{1F} \quad = (x - F_w) \cdot l_2 = (1.82 - 14.52) \cdot 4.83 = -61.34\ kN \cdot m;$$

$$M_{2F} \quad = M_{1F} = -61.34\ kN \cdot m;$$

$$M_{3F} \quad = (x - F_w) \cdot l_c = (1.82 - 14.52) \cdot 18 = -228.6\ kN \cdot m;$$

$$N_0 = N_1 = N_2 = N_3 = 0\ kN;$$

$$Q_{0e} = x = 1.51\ kN;$$

$$Q_{\ 1e} = Q_{\ 2e} = x \quad - q_w \cdot l_2 = 1.51 - 1.93 \cdot 4.83 = -7.81\ kN;$$

$$Q_{\ 3e} = x \quad - q_w \cdot l_c = 1.51 - 1.93 \cdot 18 = -33.23\ kN;$$

$$Q_{0F} = x - F_w = 1.82 - 14.52 = -12.7\ kN;$$

$$Q_{\ 1F} = Q_{\ 2F} = x - F_w = 1.82 - 14.52 = -12.7\ kN;$$

$$Q_{\ 3F} = x - F_w = 1.82 - 14.52 = -12.7\ kN\ kN;$$

<u>Right wind:</u>

$$M_0 = 0\ kN \cdot m;$$

$$M_{1e} = x \quad \cdot l_2 + q'_w \cdot \frac{l_2^2}{2} = 1.51 \cdot 4.83 + 1.45 \cdot \frac{4.83^2}{2} = 24.21\ kN \cdot m;$$

$$M_{2e} \quad = M_{1e} = 24.21\ kN \cdot m;$$

$$M_{3e} \quad = x \quad \cdot l_c + q'_w \cdot \frac{l_c^2}{2} = 1.51 \cdot 18 + 1.45 \cdot \frac{18^2}{2} = 262.1\ \ kN \cdot m;$$

$$M_{1F} \quad = (x + F'_w) \cdot l_2 = (1.82 + 10.89) \cdot 4.83 = 61.39\ kN \cdot m;$$

$$M_{2F} \quad = M_{1F} = 61.39\ kN \cdot m;$$

$$M_{3F} \quad = (x + F'_w) \cdot l_c = (1.82 + 10.89) \cdot 18 = 228.8\ kN \cdot m;$$

$$N_0 = N_1 = N_2 = N_3 = 0\ kN;$$

$$Q_{0e} = x = 1.51\ kN;$$

$$Q_{\ 1e} = Q_{\ 2e} = x \quad - q'_w \cdot l_2 = 1.51 + 1.45 \cdot 4.83 = 8.51\ kN;$$

$$Q_{\ 3e} = x \quad - q'_w \cdot l_c = 1.51 + 1.45 \cdot 18 = 27.61\ kN;$$

$$Q_{0F} = x + F'_w = 1.82 + 10.89 = 12.71\ kN;$$

$$Q_{\ 1F} = Q_{\ 2F} = x + F'_w = 1.82 + 10.89 = 12.71\ kN;$$

$$Q_{\ 3F} = x + F'_w = 1.82 + 10.89 = 12.71\ kN\ kN;$$

3.8.7. Rating combinations of load

The main combinations of load that were used in this work in accord with normative documents [6].

1. Dead load+1*temporary load;
2. Dead load+0.9*(some temporary loads).

Rating combinations of load

Table 3.2

№	Loads, combinations		ψ	Cross-section 1-1		Cross-section 2-2		Cross-section 3-3		
				M	*N*	*M*	*N*	*M*	*N*	*Q*
1	Dead			-50.77	-387.81	+46.19	-387.81	+36.6	-481.49	-3.92
2	Snow			-31.12	-194.94	+17.62	-194.94	+39.1	-194.94	-1.63
3	F_{max} left			+75.44	0	-228.96	-756.45	-59.24	-756.45	-15.62
4	F_{min} right			+75.44	0	+41.66	-260.19	+164.06	-260.19	-15.62
5	$\pm H_{max}$ left			±126.59	0	±126.59	0	±1678.42	0	±117.83
6	$\pm H_{min}$ right			±126.59	0	±126.59	0	±562.92	0	±33.13
7	Left wind			-76.56	0	-76.56	0	-514.08	0	-45.93
8	Right wind			+85.6	0	+85.6	0	+490.9	0	+40.32
The main combinations	$+M_{max}$ N_{cor}	№		1+8		1+8		1+8		
			1	+34.83	-387.81	+131.79	-387.81	+527.5	-481.49	
		№		1+0.9(3+5+8)		1+0.9(2+4+6+8)		1+0.9(2+4+5+8)		
			0.9	+208.1	-387.81	+290.51	-797.43	+2171.83	-891.11	
	$-M_{max}$ N_{cor}	№		1+7		1+3+5		1+7		
			1	-127.33	-387.81	-309.36	-1144.26	-477.48	-481.49	
		№		1+0.9(2+7)		1+0.9(3+5+7)		1+0.9(3+5+7)		
			0.9	-147.68	-563.26	-342.71	-1068.62	-1989.97	-1162.3	
	N_{max} $+M_{cor}$	№		1+8		1+8		1+8		
			1	+34.83	-387.81	+131.79	-387.81	+527.5	-481.49	
		№		1+0.9(2+3+5+8)		1+0.9(2+3+5+8)		1+0.9(2+3+5+8)		
			0.9	+180.1	-563.26	+49.96	-1244.1	+1970.86	-1337.74	
	N_{max} $-M_{cor}$	№		1+2		1+3+5		1+3+5		
			1	-81.89	-582.75	-309.36	-1144.26	-1701.1	-1237.94	
		№		1+0.9(2+7)		1+0.9(3+5+7)		1+0.9(3+5+7)		
			0.9	-158.45	-563.27	-342.71	-1068.62	-1989.97	-1162.3	
	N_{min}	№		Forces M and N of dead load are taken into account with				0.82(1+2)		

	$+M_{cor}$		1	coefficient 0.9/1.1=0.82				+62.1	-554.67	
	N_{min}	№						0.82(1+7)		
	$-M_{cor}$		1					-349.2	-394.82	
	Q_{max}	№						1+0.9(2+3+5+7)		
			0.9							-166.83

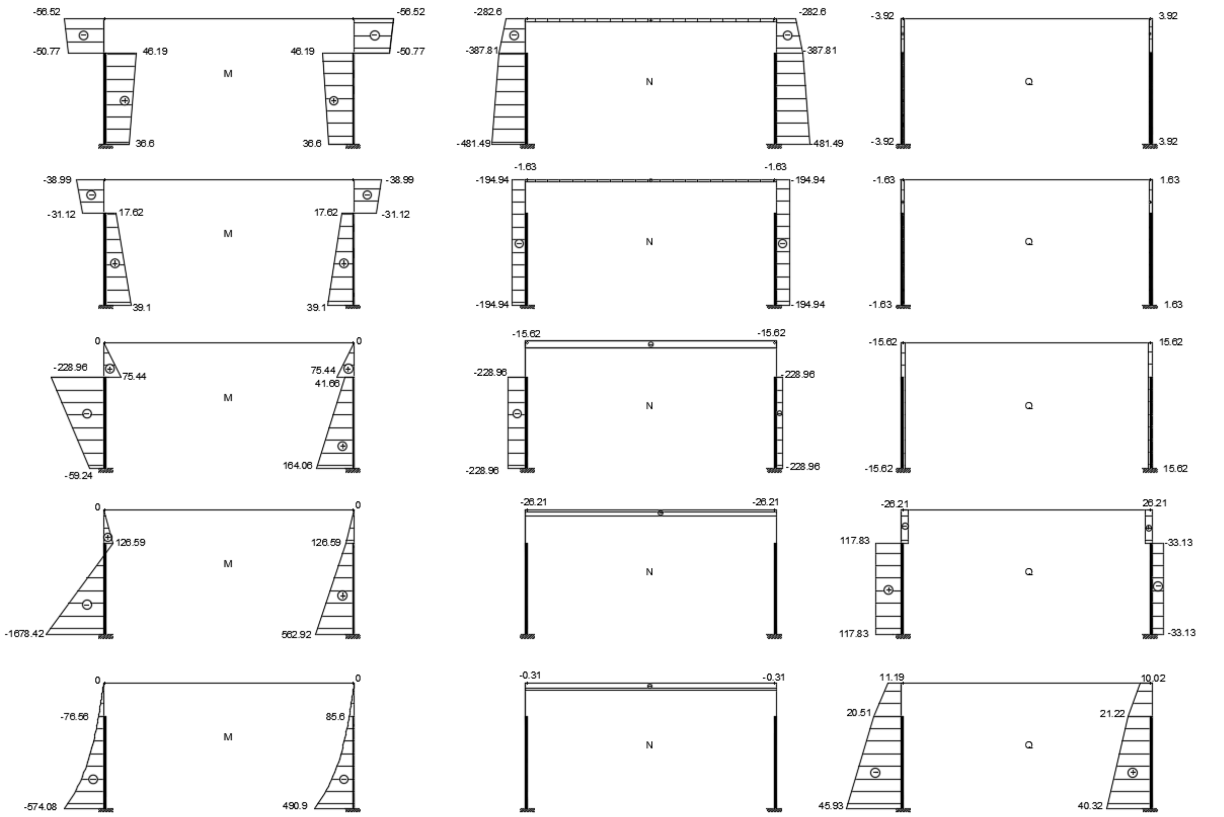
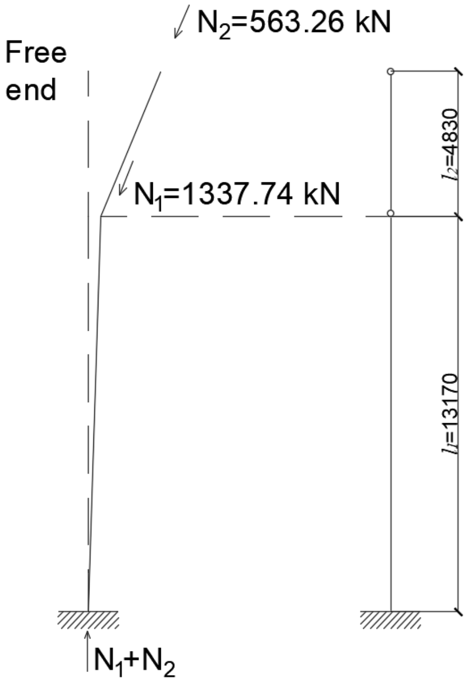


Fig.3.16. Diagrams of forces M, N, Q in the frame from loads: a - dead; b - snow; c - vertical crane; d - horizontal crane; e - wind

3.9. Calculation of above crane part of the column

3.9.1. Calculation and design the open-web left column of the one-storeyed industrial buildings frame

Calculation of the column is provided in the plane of bending moment acting and out of the plane.



Rating length of the column <u>in plane of the frame:</u> $l_{ef1,x}$, $l_{ef2,x}$ coefficient μdepends on fastening the end of the column. Assume:

- free end (if column connecting with frame is hinged);
- fastened from turn (if columns connecting with frame is rigid).

<u>Out of the plane:</u> assume hinged fastening.

If ratio geometrical length:

$\frac{l_2}{l_1} \leq 0.6$; (3.32)

$\beta = \frac{N_1}{N_2} \geq 3$; (3.33)

$N_1 = -1337.74\ kN$; $N_2 = -563.26\ kN$.

may be found from table 3.3:

Table 3.3

Condition of fastening for top end of the column	Coefficient μ		
	Bottom end μ_1 in $\frac{1}{n}$		Top end
	$0.3 > \dfrac{1}{n} > 0.1$	$0.1 > \dfrac{1}{n} > 0.05$	
Free	2.5	3.0	3.0
Fastening from turn	2.0	2.0	3.0

Rating length:

In plane:

$\beta = \frac{1337.74}{563.26} = 2.37 < 3;$

$\frac{1}{n} = \frac{1}{7.6} = 0.13;$

$\mu_1 = 2.5;$ $\mu_2 = 3.0;$

$l_{ef2,x} = \mu_2 \cdot l_2 = 3.0 \cdot 4.83 = 14.49\, m;$

$l_{ef1,x} = \mu_1 \cdot l_1 = 2.5 \cdot 13.17 = 39.51\, m;$

<u>Out of the plane</u>:

- for above crane part of the column:

$l_{ef2,y_2} = l_2 - h_{cr.b} = 4.83 - 1.0 = 3.83\, m;$

- for undercrane part:

$l_{ef1,y_1} = l_1 = 13{,}17\ m.$

1. According to Building Code calculation on strength for eccentrically-compressed elements doesn't require, when m_{ef} =20, where m_{ef} - reduced relative eccentricity.
2. Calculation on stability is fulfilled in the plane of bending moment acting (plane form loss of stability) and out of the plane (bending-turned loss of stability).

IN PLANE OF BENDING MOMENT

$\frac{N \cdot \gamma_n}{\varphi_e \cdot A \cdot R_y \cdot \gamma_c} \le 1$ - (formula 10.6) for solid piece of column, where φ_e- [table Ж.3], buckling factor for eccentricity compressed elements depends on conditional flexibility $\underline{\lambda}$; $\underline{\lambda} = \lambda \cdot \sqrt{\frac{R_y}{E}}$; $\lambda = \frac{l_x}{i_x}$ and $m_{ef} = \eta \cdot m$ (formula 10.7); η- coefficient of influence the shape of cross-section [table Ж.2] depends on relative eccentricity [6] $m_x = \frac{e \cdot A}{W_x}$, $e = \frac{M}{N}$, where W_x – section moment for most compressed fiber.

$\rho = \frac{W_x}{A}$; (3.36) $m_x = \frac{e \cdot 1}{\rho} = \frac{M \cdot 1}{N \cdot \rho}$ (3.37)

if $0 < \underline{\lambda} \le 5$

$\frac{A_f}{A_w} = 0.5$; $\eta = (1.75 - 0.1 \cdot m_x) - 0.02 \cdot (5 - m_x) \cdot \underline{\lambda_x}$ (table Ж.2, type of cr. section 5 [6]).

OUT OF THE PLANE OF BENDING MOMENT

$\frac{N \cdot \gamma_n}{c \cdot \varphi_y \cdot A \cdot R_y \cdot \gamma_c} \le 1$ - (formula 10.8); φ_y assume as for central-compressed column [table Ж.1] in accord axis y-y perpendicular for the bending plane; c- coef. that takes into account influence of bending moment on space loss of stability, depends on relative eccentricity $m_x = \frac{M_x \cdot A}{N \cdot W_c}$, M_x- rating bending moment in boundary 1/3 of length of the column's top part, but not less than ½ from max bending moment (formula 10.9-10.11, table 10.2) [6].

CHECK UP LOCAL STABILITY OF FLANGES AND WEB

For flanges: for providing local stability of flange it's necessary the ratio width of overhang b_{ef} to t_f shouldn't more the particular values:

for non-fixed flange of I-beam - $b_{ef} = \frac{b_f - t_w}{2}$. (3.34)

For web: for providing local stability web of column is calculated on stability if overflow gate out of the plane – limit value $\frac{h_w}{t_w}$ is determined in terms of coefficient

$\alpha = \frac{\sigma - \sigma_1}{\sigma}$, where $\sigma = \frac{N}{A} + \frac{M_x \cdot h_w}{2 \cdot I}$ (3.35)

$\alpha \geq 1,\ \frac{h_{ef}}{t_w} \leq 3.8\sqrt{\frac{E}{R_y}};\quad \alpha \leq 0.5,\ \frac{h_{ef}}{t_w} = \underline{\lambda_{uw}}\sqrt{\frac{E}{R_y}};$ λ_{uw}- (table 10.3.) [6]

Assume for the above crane part of the column I-beam cross-section height equals 500mm. Rating efforts for cross-section 1-1 are determined in accord with rating combination of efforts assume:

$N_{max} = 563.26\ kN,$

$+M_{cor} = 180.1\ kM \cdot m.$

Radius of inertia of this cross-section:

$i_x = 0.42h = 0.42 \cdot 50 = 21.0\ cm$

$\rho_x = \frac{W_x}{A} = 0.35h = 0.35 \cdot 50 = 17.5\ cm$

Conditional flexibility:

$$\underline{\lambda_x} = \lambda_x \cdot \sqrt{\frac{R_y}{E}} = \frac{l_{ef2,x}}{i_x} \cdot \sqrt{\frac{R_y}{E}} = \frac{1449}{21} \cdot \sqrt{\frac{240}{2.06 \cdot 10^5}} = 2.36$$

$C245\ R_y = 240\ MPa$

Relative eccentricity m_x :

$\rho\ = \frac{W_x}{A}$ (3.36)

$$m_x = \frac{e \cdot 1}{\rho} = \frac{M \cdot 1}{N \cdot \rho} = \frac{18010 \cdot 1}{563.26 \cdot 17.5} = 1.83\ ;$$

if $0 < \underline{\lambda} \leq 5$

$\frac{A_f}{A_w} = 0.5$; (3.37) $\quad \eta = (1.75 - 0.1 \cdot m_x) - 0.02 \cdot (5 - m_x) \cdot \underline{\lambda_x}$; (3.38)

$$\eta = (1.75 - 0.1 \cdot 1.83) - 0.02 \cdot (5 - 1.83) \cdot 2.36 = 1.42;$$

$$m_{ef} = \eta \cdot m_x = 1.42 \cdot 1.83 = 2.6 < 20;$$

$$\varphi_e = 0.347$$

where φ_e – coefficient available from table Ж.3 buckling factor for eccentricity compressed elements; η – coefficient of influence the shape of cross-section, available from table Ж.2; m_x – relative eccentricity, that is calculated by the formula: $m_x = \frac{e \cdot A}{W_x}$, $e = \frac{M}{N}$; where A – area of cross-section; W_x – section moment for the most compressed fiber (table Ж.2 the 5^{th} type of cross-section) [6].

Necessary area of cross-section:

$$A_{nec} = \frac{N}{\varphi_e \cdot R_y \cdot \gamma_c} = \frac{563.26}{0.347 \cdot 24 \cdot 1} = 67.63\ cm^2$$

$$R_y = 240\ MPa = 24\ kN/cm^2$$

Assume the thickness of the flanges equals 10mm.

For providing local stability of web:

if $m_x = 1.83 > 1$ and $\underline{\lambda_x} = 2.36 > 2.0$ (table 10.3)

$$\underline{\lambda}_{uw} = 1.20 + 0.35 \cdot \underline{\lambda_x} = 1.20 + 0.35 \cdot 2.36 = 2.03$$

where $\underline{\lambda}_{uw}$ – conditional ultimate flexibility of the web.

$$min\ t_w = \frac{h_w}{\underline{\lambda}_{uw}} \cdot \sqrt{\frac{R_y}{E}} = \frac{46}{2.03} \cdot \sqrt{\frac{240}{2.06 \cdot 10^5}} = 0.77\ cm$$

Assume $t_w = 8\ mm$

$$A_w = h_w \cdot t_w = 46 \cdot 0.8 = 36.8\ cm^2 \qquad \frac{A_f}{A_w} = 0.5;$$

$$A_f = 0.5(A_{\ nec} - A_w) = 0.5(67.63 - 36.8) = 15.42\ cm^2$$

For providing local stability of flange the ratio:

$$\frac{b_f - t_w}{2 \cdot t_f} \leq (0.36 + 0.1 \cdot \underline{\lambda_x}) \cdot \sqrt{\frac{E}{R_y}}, \quad (3.39) \quad A_f = 2b_f \cdot t_f \quad (3.40)$$

$$(0.36 + 0.1 \cdot 2.36) \cdot \sqrt{\frac{2.06 \cdot 10^5}{240}} = 17.46$$

$$t_f = \sqrt{\frac{A_f}{2 \cdot 17.46}} = \sqrt{\frac{15.42}{2 \cdot 17.46}} = 0.66\ cm$$

For web – 460x8 mm

For flange – 240x10 mm

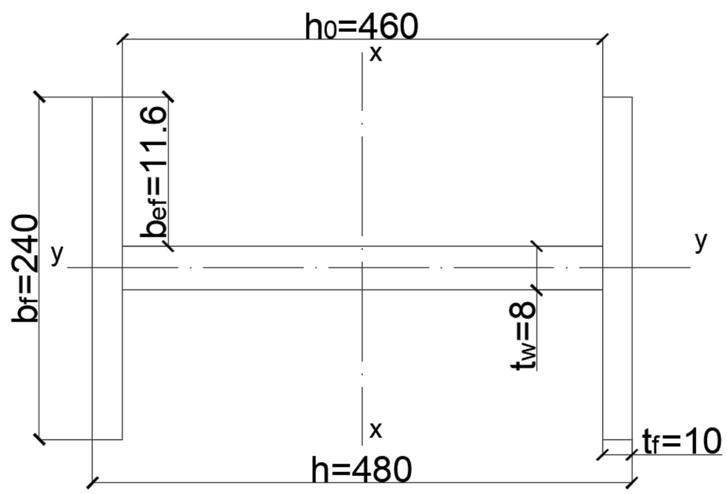


Fig.3.17.Cross--section of above-crane part of the column

$$A = A_w + 2A_f = 0.8 \cdot 46 + 2 \cdot 24 \cdot 1 = 84.8\ cm^2$$

$$I_{x-x} = \frac{t_w \cdot h_w^3}{12} + 2 \cdot b_f \cdot t_f \cdot \left(\frac{h_w + t_f}{2}\right)^2; \ (3.41)$$

$$I_{x-x} = \frac{0.8 \cdot 46^3}{12} + 2 \cdot 24 \cdot 1 \cdot \left(\frac{46+1}{2}\right)^2 = 32997.07\ cm^4$$

$$I_{y-y} = 2 \cdot \frac{t_f \cdot b_f^3}{12} = 2 \cdot \frac{1 \cdot 24^3}{12} = 2304\ cm^4$$

$$W_x = \frac{2 \cdot I_x}{h} = \frac{2 \cdot 32997.07}{48} = 1374.88\ cm^3$$

$$i_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{32997.07}{84.8}} = 19.73\ cm$$

$$i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2304}{84.8}} = 5.21\ cm$$

$$\underline{\lambda_x} = \frac{1449}{19.73} \cdot \sqrt{\frac{240}{2.06 \cdot 10^5}} = 2.51$$

$$m_x = \frac{18010 \cdot 84.8}{563.26 \cdot 1374.88} = 1.97$$

$$\eta = (1.90 - 0.1 \cdot 1.97) - 0.02 \cdot (6 - 1.97) \cdot 2.51 = 1.5;$$

$$m_{ef} = \eta \cdot m_x = 1.5 \cdot 1.97 = 2.95 < 20;$$

$$\varphi_e = 0.287$$

$$\underline{\lambda}_{uw} = 1.20 + 0.35 \cdot 2.51 = 2.079$$

Checks up:

1. Stability in plane:

$$\frac{N}{A \cdot \varphi_e} \le R_y\ \gamma_c$$

$$\frac{563.26}{84.8 \cdot 0.260} = 23.14\ kN/cm^2 = 231.4\ MPa < 240 \cdot 1 = 240\ MPa$$

Conclusion: the stability in the plane is provided.

2. Stability out of the plane:

$$\lambda_y = \frac{383}{5.21} = 74.28$$

$$\underline{\lambda}_y = \lambda_y \cdot \sqrt{\frac{R_y}{E}} = 74.28 \cdot \sqrt{\frac{240}{2.06 \cdot 10^5}} = 2.54$$

$$\varphi_y = 0.733$$

$$M_x = 44.03\ kNm$$

$$m_x = \frac{M_x \cdot A}{N \cdot W_x} = \frac{4403 \cdot 84.8}{563.26 \cdot 1374.88} = 0.48$$

if $m_x \leq 1$ $\qquad \alpha = 0.7$

$$\lambda_y = 74.28 < \lambda_c = 3.14 \cdot \sqrt{\frac{E}{R_y}} = 92 \Rightarrow \beta = 1$$

$$c = \frac{\beta}{1 + \alpha \cdot m_x} = \frac{1}{1 + 0.7 \cdot 0.48} = 2.98$$

$$\frac{N}{c \cdot \varphi_y \cdot A} \leq R_y\ \gamma_c \qquad (3.42)$$

$$\frac{563.26}{2.98\ \cdot\ 0.733\ \cdot\ 84.8} = 3.041\ kN/cm^2 = 30.41\ MPa < R_y\ \gamma_c = 240\ MPa$$

Conclusion: the stability out of the plane is provided.

3. Local stability of web:

$$\underline{\lambda}_w = \frac{h_w}{t_w} \cdot \sqrt{\frac{R_y}{E}} = \frac{46}{0.8} \cdot \sqrt{\frac{240}{2.06 \cdot 10^5}} = 1.96 < \underline{\lambda}_{uw} = 2.079$$

<u>Conclusion</u>: the local stability of the web is provided.

3.9.2. Calculation and design the undercrane part of column

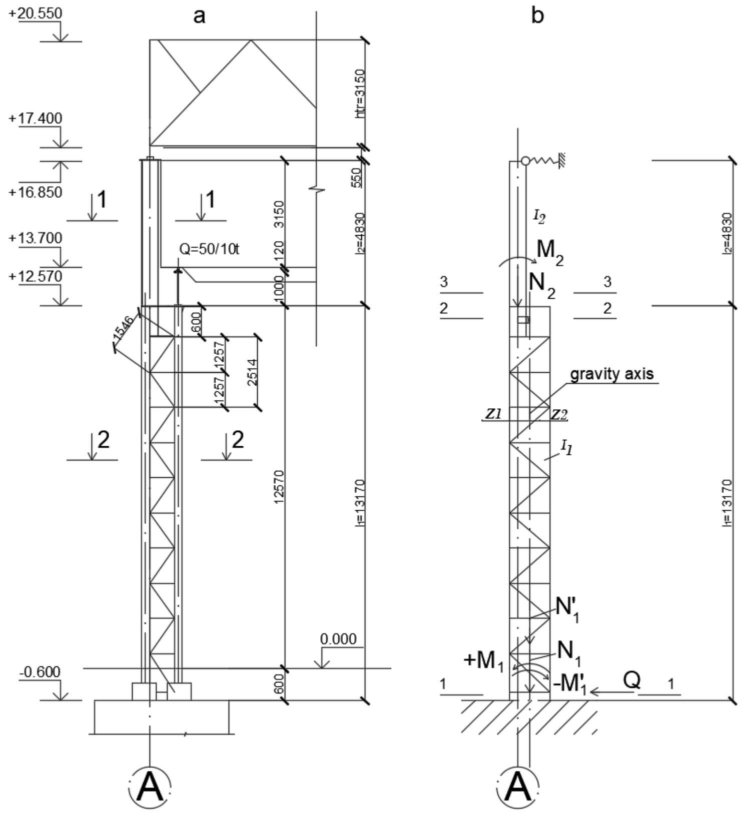


Fig.3.18.Design (a) and rating (b) schemes of the column

The undercrane part of the column consists of two pieces that are combined with a lattice.

The column can lose their carrying capacity as loss of stability separate as central compressed piece and loss of stability eccentrically compressed undercrane shaft.

The section steel I-beam type B were used for columns pieces in accord with State standards [14].

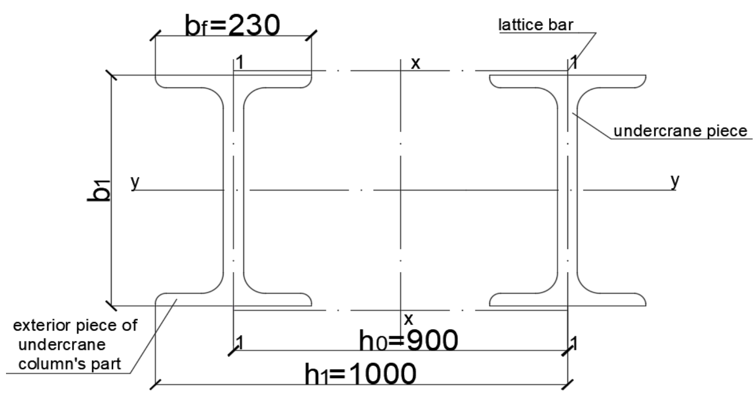


Fig.3.19. Undercrane shaft of the column (cross-section 2-2)

Rating combination of efforts in piece is calculated by the formula:

$$N_p = \frac{N}{2} + \frac{|M|}{h_0} \qquad (3.43)$$

$$h_0 = 2e_{cr} \qquad (3.44)$$

Checks up the stability by the formula:

$$\frac{N_p}{\varphi \cdot A_p} \le R_y \ \gamma_c \,, \qquad (3.45)$$

where φ – is the buckling factor in accord with maximum deflection of a piece in plane or out of the plane (axis y-y or 1-1).

Necessary area of one piece cross-section may be calculated by the formula:

$$A_p = \frac{N_p}{\varphi \cdot R_y \cdot \gamma_c}, \qquad \varphi \approx 0.8 \qquad (3.46)$$

Choose the cross-section and check the stability out of the plane. For this purpose we calculated flexibility:

$$\underline{\lambda}_y = \frac{l_{y1}}{i_y} \cdot \sqrt{\frac{R_y}{E}} \Rightarrow \varphi \text{ (Ж.1)} \qquad (3.47)$$

2) Calculation shaft of the column in accord with axis x-x if $m_{ef} \le 20$ we calculate on stability by the formula:

$$\sigma = \frac{N}{\varphi_e \cdot A} \le R_y \ \gamma_c \qquad (3.48)$$

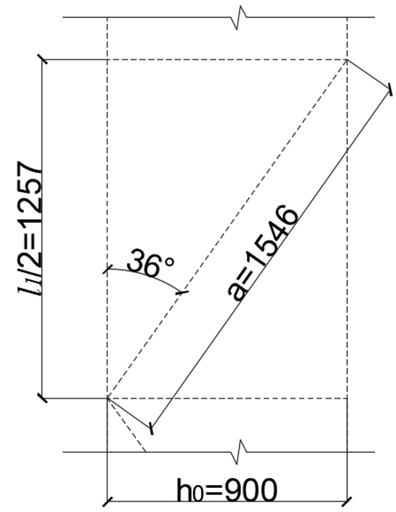
$$m = \frac{e \cdot A \cdot y}{I_x}, \qquad (3.49) \qquad e = \frac{M}{N} \qquad (3.50)$$

where A – total area of column cross-section; I_x – moment of inertia in accord with axis x-x; y – is the distance from the axis to the center of the gravity piece.

$$\varphi_e = \underline{\lambda}_{ef} = \lambda_{ef} \cdot \sqrt{\frac{R_y}{E}} \qquad (3.51)$$

$$\lambda_{ef} = \sqrt{\lambda_x^2 + \frac{\alpha \cdot A_d}{A \cdot d_1}} \qquad (3.52) \qquad \lambda_x = \frac{l_x}{i_x} \qquad (3.53)$$

$A \cdot d_1$ – its area of cross-section of lattice bar.



$$\alpha = \frac{10 \cdot a^3}{h_0^2 \cdot 0.5 \cdot l_1}$$

a – length of lattice bar.

Lattice bar is calculated in accord with rating shear force that is assumed in accord with calculations or Q_{fic} .

$$Q_{fic} = 7.15 \cdot 10^{-6}(2330 - \frac{E}{R_y}) \cdot \frac{N}{\varphi} \qquad (3.54)$$

where φ – buckling factor in plane of lattice in term of λ_{ef} .

Choose from table rating combination of efforts maximum values of bending moment with sign “+” and “-”.

Assume:

$M^+ = 1970.86\ kM \cdot m, \qquad N = -1337.74\ kN$

$M^- = -1989.97\ kM \cdot m, \qquad N = -1162.3\ kN$

$N_{p.ext} = \frac{1337.74}{2} + \frac{1970.86}{1} = 2639.73\ kN$

Assume $h_0 = 1.0\ m$

$N_{p.ext} = \frac{1162.3}{2} + \frac{1989.97}{1} = 2571.12\ kN$

Assume for further calculation maximum value:

$N_p = 2639.73\ kN$

$C245\ R_y = 240\ MPa\ (24\ kN/cm^2)$

$(t_f = 2.20\ cm)$

$\gamma_c = 1.0$

Necessary area of piece:

if $\varphi \approx 0.8$

$$A_{p.nec} = \frac{2639.73}{0.8 \cdot 24 \cdot 1} = 137.49\ cm^2$$

Assume from catalog: 60Б2

$$A = 147.3\ cm^2 > A_{p.nec} = 137.49\ cm^2$$

$$i_y = 24.39\ cm$$

$$i_{1-1} = 4.92\ cm$$

$$I_{1-1} = 3561\ cm^4$$

$$l_{ef,y_1} = 13.17\ m$$

$$\lambda_y = \frac{l_{ef,y_1}}{i_y} = \frac{1317}{24.39} = 53.99$$

$$\bar{\lambda}_y = 53.99 \cdot \sqrt{\frac{240}{2.06 \cdot 10^5}} = 1.84 \Rightarrow \varphi = 0.849$$

$$h_0 = h_1 - \frac{b_f}{2} = 1000 - \frac{200}{2} = 900\ mm$$

Lets specify:

$$N_p = \frac{1337.74}{2} + \frac{1970.86}{0.900} = 2858.71\ kN$$

$$\sigma = \frac{N_p}{\varphi \cdot A_p} = \frac{2858.71}{0.849 \cdot 147.3} = 22.86 kN/cm^2 < R_y\ \gamma_c = 24\ kN/cm^2$$

Conclusion: stability is provided.

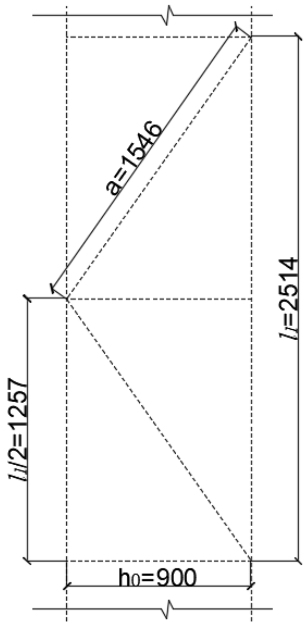
From the condition of stable equilibrium in plane and out of the plane lets determine the distance from the joint of the lattice.

$$\lambda_{11} = \frac{l_1}{i_{1-1}} = \lambda_y = 53.99 < 80$$

$$l_1 = \lambda_y \cdot i_{1-1} = 53.99 \cdot 4.92 = 265.6\ cm$$

Assume $l_1 = 251.4\ cm$

$$\lambda_{11} = \frac{251.4}{4.92} = 51.1 < 53.99$$



3.9.3. Calculation the lattice of undercrane column part

$Q_{max} = 166.83\ kN$;

Conditional shear force:

$Q_{fic} = 7.15 \cdot 10^{-6}(2330 - \frac{2.06 \cdot 10^5}{240}) \cdot \frac{1337.74}{0.6} = 23.46\ kN < Q_{max}$;

$\varphi \approx 0.6$;

Because of this choice the cross-section of lattice is determined by Q_{max} :

$a = \sqrt{0.900^2 + 1.257^2} = 1.546\ m$;

$sin\alpha = \frac{h_0}{a} = \frac{0.900}{1.546} = 0.5821 \Rightarrow 36^\circ$

Effort in lattice:

$N_1 = \frac{Q_{max}}{2 \cdot sin\alpha} = \frac{166.83}{2 \cdot 0.5821} = 143.3\ kN$;

Lattice is designed from one equal leg angle in accord with ДСТУ 2251:2018 [15].

Assume:

$\lambda_0 \approx 90 \Rightarrow \underline{\lambda} \Rightarrow \varphi = 0.612(\gamma_c = 0.75$ –for single angle that is fastened by leg)

Necessary area of lattice bar:

$A_{l.nec} = \frac{143.3}{0.612 \cdot 24 \cdot 0.75} = 13\ cm^2$;

Assume: ∟100x10

$A = 15.10\ cm^2 > A_{l.nec} = 13\ cm^2$;

$i_{min} = i_{y_0} = 1.96\ cm$;

$$\lambda_y = \frac{l_{ef,y_1}}{i_y} = \frac{154.6}{1.96} = 78.9;$$

$$\bar{\lambda}_y = 78.9 \cdot \sqrt{\frac{240}{2.06 \cdot 10^5}} = 2.69 \Rightarrow \varphi = 0.704;$$

$$\sigma = \frac{N_l}{\varphi \cdot A_l} = \frac{143.3}{0.704 \cdot 15.10} = 13.48\ kN/cm^2;$$

$$\sigma = 13.48\ kN/cm^2 < R_y\ \gamma_c = 24 \cdot 0.75 = 18\ kN/cm^2;$$

Conclusion: stability is provided.

3.9.4. Checks up the shaft of the column and the first geometrical characteristics:

$$A = 2 \cdot A_p = 2 \cdot 147.3 = 294.6\ cm^2$$

$$I_{x-x} = 2\left[I_{1-1} + A_p\left(\frac{h_0}{2}\right)^2\right] = 2[3561 + 147.3 \cdot (45)^2] = 603687\ cm^4$$

$$\lambda_x = \frac{l_{ef,x_1}}{i_x} = \frac{3951}{1.96} = 78.9$$

$$i_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{603687}{294.6}} = 45.27\ cm$$

$$\alpha = \frac{10 \cdot a^3}{h_0^2 \cdot 0.5 \cdot l_1} = \frac{10 \cdot 1.546^3}{0.900 \cdot 1.257} = 32.66$$

$$\lambda_{ef} = \sqrt{\lambda_x^2 + \alpha \frac{A}{2A \cdot d_1}} = \sqrt{78.9^2 + 32.66 \cdot \frac{294.6}{2 \cdot 15.10}} = 80.89$$

$$\bar{\lambda}_{ef} = 80.89 \cdot \sqrt{\frac{240}{2.06 \cdot 10^5}} = 2.76$$

$$\lambda_{11} = 53.99 < \lambda_{ef} = 80.89$$

Note. Relative eccentricity is determined when the interior piece is loaded.

$$m = e \cdot \frac{A \cdot y}{I_x} = \frac{197086}{1337.74} \cdot \frac{294.6 \cdot 45}{603687} = 3.24 < 20$$

$$\lambda_{ef} = 2.76 \Rightarrow 0.691$$

$$\frac{N}{\varphi_e \cdot A} \leq R_y \ \gamma_c$$

$$\frac{1337.74}{0.691 \cdot 294.6} = 6.57 \ kN/cm^2 \leq R_y \ \gamma_c = 24 \ kN/cm^2$$

Conclusion: stability is provided.

3.10. Design and calculation the top and bottom column's part

Assume structural decision of connection that is similar to type decision (see fig.3.9 a – structural decision of connection , b – cross-section of traverse, c – rating scheme of traverse).

For transmitting efforts those are induced in the top part of the column and crane beam design traverses with height 0.5-0.8 from the width of the undercrane column's part. In open web column traverse it works as a beam-wall of I-beam cross-section.

Universal decision of such connection uses the composed welded traverse and its vertical sheet have milled surface. Top horizontal diaphragm (cross-section 1-1) is lower on 200mm for convenience of assembly.

In butt cover plate that is welded to the bottom column's part and to I-beams flange the upper column's part foresee openings for bolts (d=23mm). The thickness of a butt cover plate is assumed to equal the thickness of the flange upper column's part. The thickness of plate is assumed in term of maximum vertical crane load F_{max} that is induced in a crane beam.

Table 3.4

F_{max} , kN	Up to 1500	1500-2500	>2500
t_{pl} ,mm	20	25	30

The thickness of the transverse stiffener is 1.5-2 times bigger than the flange upper column's part.

Horizontal elements of traverse are necessary to take with thickness 10mm.

The thickness of traverse web t_w is determined on condition of pressure under F_{max} that is transmitted from bearing stiffener of crane beam through support plate (t_{pl}) on a milled surface of the web.

Necessary thickness of traverse web is calculated by the formula:

$$t_{w,nec} = \frac{1.2 \cdot F_{max}}{R_p \cdot z} \qquad (3.55)$$

where 1.2 – coefficient that takes into account non-uniform pressure on traverse because of possible bending of bearing stiffeners;

R_p – design resistance of steel by pressure;

$R_p = R_n\ (\Gamma.2)$

$z = b_{o,p} + 2t_{pl}$; $\qquad b_{o,p} -$ bearing stiffener of crane beam.

The rating combination of efforts are F_1 and F_2 (cross-section 1-1).

Maximum value F_1 is taken place when:

$M_1^{(-)}$; $\qquad N_1^{(-)}$

But maximum value F_2 is taken place when:

$M_2^{(+)}$; $\qquad N_2^{(-)}$

For most reliability of fastening traverse you should have made a cut in the web of the column.

Formulas for calculation elements of traverse are available in table 3.5 and for calculation welds of traverse in table 3.6.

$$M_1^{(-)} = 147.68\ kM \cdot m$$

$$N_1^{(-)} = 563.26\ kN$$

$$M_2^{(+)} = 180.1\ kM \cdot m$$

$$N_2^{(-)} = 563.26\ kN$$

$$F_{max} = 756.45\ kN$$

$$F_1 = \frac{N_1}{2} + \frac{|M_1|}{h_f} = \frac{563.26}{2} + \frac{147.68}{0.47} = 595.84\ kN$$

$$F_2 = \frac{N_2}{2} + \frac{|M_2|}{h_f} = \frac{563.26}{2} + \frac{180.1}{0.47} = 664.82\ kN$$

Necessary thickness of traverse web:

$$t_{w,nec} = \frac{1.2 \cdot F_{max}}{R_p \cdot z} = \frac{1.2 \cdot 756.45}{36 \cdot 22} = 1.15 \text{ cm}$$

$$t_{pl} = 20\ mm$$

$$b_{o,p} = 180\ mm$$

$$z = 180 + 2 \cdot 20 = 220\ mm$$

$$R_p = 360\ MPa$$

Assume $t_{w,tr} = 12\ mm$

Efforts in traverse are calculated by the following formulas:

$$Q_l = \frac{F_1 \cdot l_2}{l_1 + l_2} = \frac{595.84 \cdot 52.5}{90} = 347.57\ kN$$

$$Q_r = \frac{F_1 \cdot l_2}{l_1 + l_2} + 0.6F_{max} = \frac{595.84 \cdot 52.5}{90} + 0.6 \cdot 756.45 = 801.44\ kN$$

$$M_{tr} = (Q_r - 0.6F_{max})l_2 = (801.44 - 0.6 \cdot 756.45) \cdot 52.5 = 182.47\ kNm$$

Geometrical characteristics cross-section of traverse:

Assume height of web 600 mm.

$$W' = \frac{t_w \cdot h_w^2}{6} = \frac{1.2 \cdot 60^2}{6} = 720\ cm^3;$$

$$A' = t_w \cdot h_w = 1.2 \cdot 60 = 72\ cm^2;$$

Stresses in traverse under bending:

$$\sigma' = \frac{M_{tr} \cdot 1000}{W'} \le R_y\gamma_c;$$

$$\sigma' = \frac{182.47 \cdot 1000}{720} = 253.43\ MPa > R_y\gamma_c = 240\ MPa;$$

Conclusion: the normal stresses are not provided because of this enlargement.

Assume $h_{tr} = 720\ mm$

$$W' = 1036.8\ cm^3;$$

$$\sigma' = \frac{182.47 \cdot 1000}{1036.8} = 175.99\ MPa < R_y\gamma_c = 240\ MPa;$$

$\tau = \frac{1.5 \cdot Q_r \cdot 10}{A'} \le R_s \gamma_c$; $R_s = 0.58 R_y$;

$\tau = \frac{1.5 \cdot 801.44 \cdot 10}{86.4} = 139.1 MPa < R_s \gamma_c = 140\ MPa$;

$A' = 86.4\ cm^2$;

Conclusion: the tangential stresses are provided.

Checks up the web undercrane piece of column on shear by lines 1-1.

$$\tau_{umd.p} = \frac{(Q_r + 0.6 F_{max}) \cdot 10}{2 \cdot h_{w,tr} \cdot t_{w1}} = \frac{(801.44 + 0.6 \cdot 756.45) \cdot 10}{2 \cdot 72 \cdot 1.15} = 75.8\ MPa;$$

$\tau_{umd.p} = 75.8\ MPa < R_s \gamma_c = 140\ MPa$;

Checks up compressed stresses in the web undercrane piece of column.

$$\sigma_{umd.p} = \frac{(Q_r + 0.6 F_{max}) \cdot 10}{A_{ef}} \le R_y \gamma_c;$$

$$A_{ef} = z \cdot t_w + 30 \cdot t_{w1}^2 = 22 \cdot 1.2 + 30 \cdot 1.15^2 = 66.08\ cm^2;$$

$$\sigma_{umd.p} = \frac{(801.44 + 0.6 \cdot 756.45) \cdot 10}{66.08} = 189.97\ MPa \le R_y \gamma_c = 240\ MPa;$$

Conclusion: the compressed stresses are provided.

$$N_{w1} = -\frac{N_2}{2} - \frac{M_1}{h_f} = -\frac{563.26}{2} - \frac{147.68}{0.47} = -595.84\ kN\ (compression)$$

$$N_{w1} = -\frac{N_2}{2} + \frac{|M_2|}{h_f} = -\frac{563.26}{2} + \frac{180.1}{0.47} = 101.56\ kN\ (tension)$$

$$\sigma_w = \frac{N_{w1}}{t_f \cdot b_f} \le R_y \gamma_c\ (compression)$$

$$\sigma_w = \frac{595.84}{1 \cdot 24} = 24.8 kN/cm^2 > R_y \gamma_c = 24\ kN/cm^2$$

Conclusion: condition is not provided

Enlarge thickness of flange up to 12 mm

$$\sigma_w = \frac{595.84}{1.2 \cdot 24} = 20.7 kN/cm^2 < R_y \gamma_c = 24\ kN/cm^2$$

$$\sigma_w = \frac{N_{w1}}{t_f \cdot b_f} \le 0.85 R_y \gamma_c (tension)$$

$$\sigma_w = \frac{101.56}{1.2 \cdot 24} = 3.53\ kN/cm^2 = 35.3\ MPa \le 0.85 R_y \gamma_c = 204\ MPa$$

$$N_{w2} = F_2 = 664.82\ kN$$

Assume for manual welding:

$R_{wf} = 180\, MPa$

$\beta_f = 0.7$; $\beta_z = 1.0$

$max\, k_f = 1.2 \cdot t_{min} = 1.2 \cdot 10 = 12\, mm$

$$k_{f_{w2}} = \frac{N_{w2}}{\beta_f \cdot (2 \cdot l + b_f - 3\, cm) \cdot R_{wf}} = \frac{664.82}{0.7 \cdot (2 \cdot 60 + 24 - 3\, cm) \cdot 18} = 0.37$$

Assume $k_{f_{w2}} = 7\, mm$

$k_{f_{w3}} = \frac{Q_r + 0.6F_{max}}{4 \cdot \beta_f \cdot h_{tr} \cdot R_{wf}} = \frac{801.44 + 0.6 \cdot 756.45}{4 \cdot 0.7 \cdot 72 \cdot 18} = 0.35$

$t_{tr} = 20\, mm$

$t_{w1} = 11.5\, mm$

Assume $k_{f_{w3}} = 7\, mm$

$N_{w4} = Q_l = 347.57\, kN$

$k_{f_{w4}} = \frac{N_{w4}}{2 \cdot \beta_f \cdot h_{tr} \cdot R_{wf}} = \frac{347.57}{2 \cdot 0.7 \cdot 72 \cdot 18} = 0.19$

Assume $k_{f_{w4}} = 7\, mm$

$N_{w5} = F_1 = 595.84\, kN$

$k_{f_{w5}} = \frac{N_{w5}}{4 \cdot \beta_f \cdot h_{tr} \cdot R_{wf}} = \frac{595.84}{4 \cdot 0.7 \cdot 72 \cdot 18} = 0.16$

$t_{tr} = 20\, mm$

$t_{tr,s} = 11.5\, mm$

Assume $k_{f_{w5}} = 7\, mm$

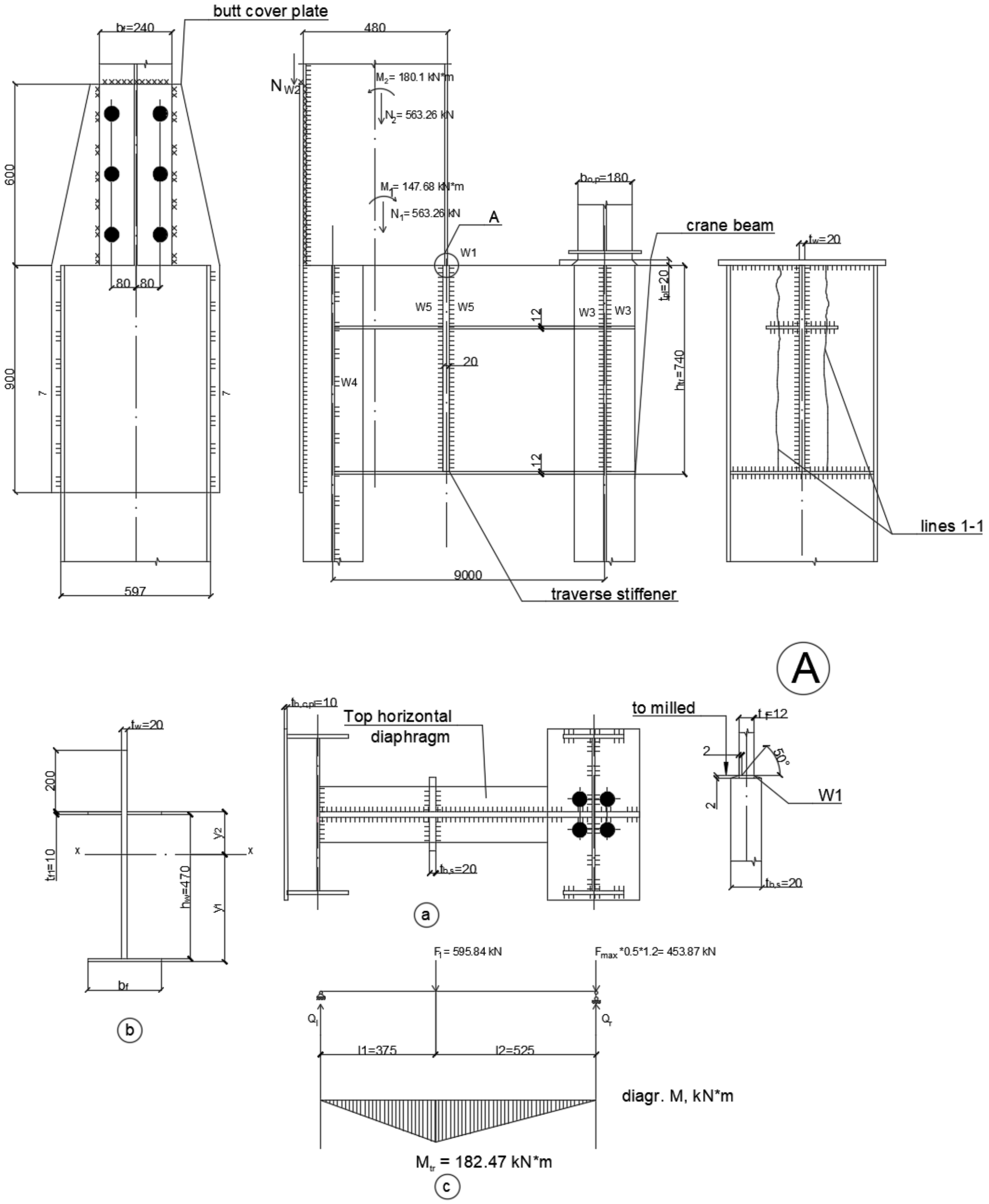


Fig.3.20. Scheme of the calculation of the joint of the top and bottom column's part: a - structural concept of joint; b - cross-section of the traverse; c - rating scheme of the traverse.

3.11. Design and calculation the base of the column

When the width of the under-crane part of the open web column equals or more than 1m use as rule separate base.

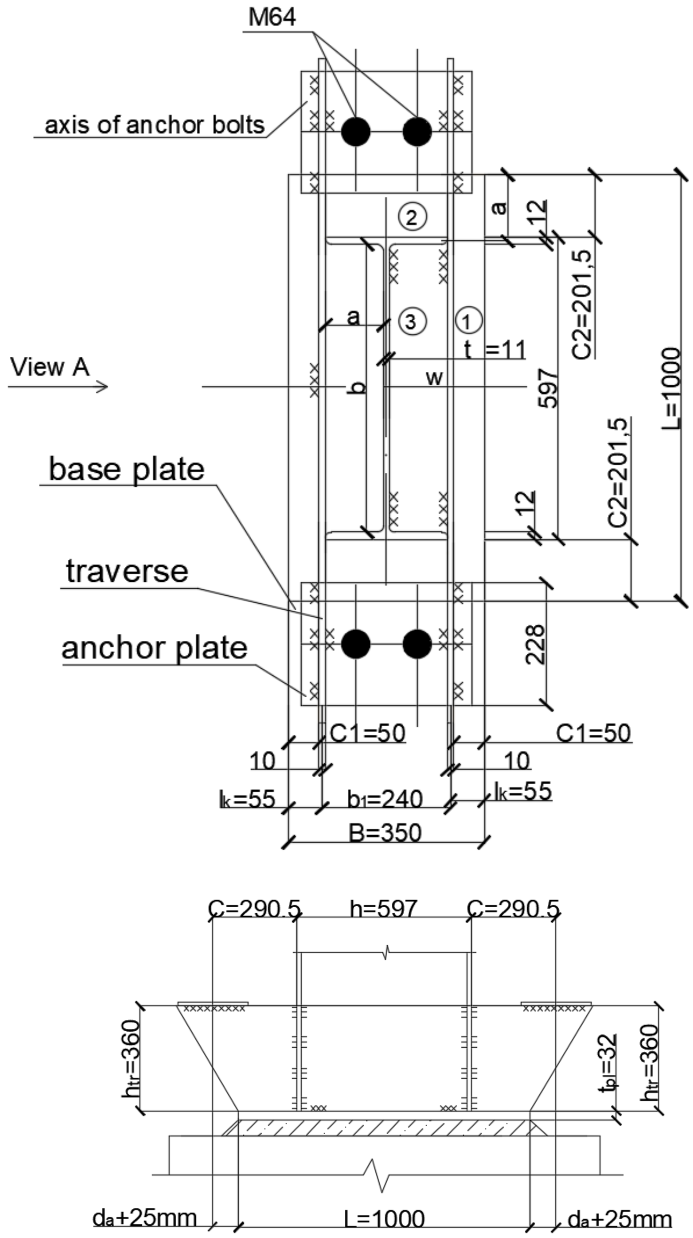


Fig.3.21. Structural scheme of the column base

Rating combination of efforts assume like for undercrane part of column, that transmit maximum pressure of foundation concrete. It's necessary to determine:

1. dimensions bearing base plate in plane (L and B);
2. thickness of the plate (t_{pl});
3. height of the traverse (h_{tr}).

Then to assume cross-section dimensions of traverse, check up strength and calculate anchor plates with bolts.

Rating compressed longitudinal force assume:

$N_b = 2858.71\ kN$

From the condition of providing strength of foundation concrete on pressure under column. Lets determine necessary area of the base plate:

$$A_{pl,nec} = \frac{N_b}{f_{ck,loc}}, \qquad (3.56)$$

where $f_{ck,loc} = f_{ck} \cdot \gamma_b$, $f_{ck} = 7.5\ MPa$

where f_{ck} – design resistance of concrete available from table 3.1 ДБН [21].

$f_{ck,loc} = 7.5 \cdot 1.2 = 9.0\ MPa = 0.9\ kN/cm^2$

$A_{pl,nec} = \frac{2858.71}{0.9} = 3176.34\ cm^2$

Assume: $t_{tr} = 10\ mm$

($t_{tr} \approx t_f$, undercrane piece of column because of this thickness may be enlarged up to 17 mm)

The width of overhang:

$C_1 = 50$ (not less than 40mm).

$B = b_0 + 2t_{tr} + 2C_1 = 230 + 2 \cdot 10 + 2 \cdot 50 = 350\ mm = 35\ cm$

Assume $B = 35\ cm$

$L = \frac{A_{pl,nec}}{B} = \frac{3176.34}{35} = 90.7\ cm$

Assume $L = 100\ cm$

Average value of stresses under foundation plate:

$\sigma_f = \frac{N_b}{B \cdot L} = \frac{2858.71}{35 \cdot 100} = 0.82\ kN/cm^2 = 8.2\ MPa$

The bearing plate is under bending and for determining its thickness calculate bending moments on parts 1, 2, and 3.

Part №1 (cantilever overhang)

The formulas for calculating parts and bearing plates available from appendix M table M.2:

$M_1 = \sigma_f \cdot C_1^2/2 = 0.82 \cdot 5^2\ /2 = 10.25\ kN \cdot cm$

Part №2

$M_2 = \alpha \cdot \sigma_f \cdot b^2{}_0$

Where α - available from table M.2 and depend on ratio $C_2\ /b_0$, where $C_2\ = (L_{pl} - h)/2 = (100 - 59.7)/2 = 20.15cm$

$\alpha \Rightarrow C_2\ /b_0 = \frac{20.15}{23} = 0.88$; $\alpha = 0.105$

$M_2 = 0.105 \cdot 0.82 \cdot 23^2\ = 45.55\ kNcm$

Part №3 (supported by 4 sides)

$M_3 = \alpha \cdot \sigma_f \cdot \left(\frac{b_0 - t_w}{2}\right)^2$

$\alpha \Rightarrow b/a = 59.7/10.95 = 5.45 > 2$; $\alpha = 0.133$

60Б2 $t_w = 11\ mm$ (in accord with catalog [14])

$M_3 = 0.133 \cdot 0.82 \cdot \left(\frac{23 - 1.1}{2}\right)^2 = 13.08\ \ kN \cdot cm^2$

For determination necessary thickness of plate we choose maximum bending moment from part №2:

$t_{pl} = \sqrt{\frac{6M_{max}}{R_y\gamma_c}} = \sqrt{\frac{6{\cdot}45.55}{23 \cdot 1.2}} = 3.15\ cm$

γ_c in this case equals 1.2 – its coefficient of working condition for bearing plates; $R_y = 230\ MPa$ – it's for a grade of C255 (20…40mm).

Assume $t_{pl} = 32\ mm$ from sheet steel 36mm-4mm for milling.

3.11.1. Calculation of the traverse

It's necessary to determine: efforts in traverse, height of traverse and check up normal and tangential stresses:

$l_k = C_1 + 0.5t_{tr} = 50 + 0.5 \cdot 10 = 55\ mm = 5.5\ cm$

$b_1 = b_0 + t_{tr} = 230 + 10 = 240\ mm = 24\ cm$

$a = C_2 + 0.5t_f = 201.5 + 0.5 \cdot 10 = 206.5\ mm = 20.65\ cm$

Linear evenly distributed load acting on traverse is calculated by the formula:

$q_{tr} = \sigma_f(l_k + 0.25b_1) = 0.82 \cdot (5.5 + 0.25 \cdot 24) = 8.45\ kN/cm$

Concentrated load acting on traverse is calculated by the formula:

$N_{tr} = q_{tr} \cdot L = 8.45 \cdot 100 = 845\ kN$

Minimum leg of filet weld is calculated by the formula (there are two filet welds for fastening one traverse to the column):

$$k_f = \frac{1}{\beta_f}\sqrt{\frac{N_{tr}}{n \cdot 85 \cdot R_{wf} \cdot \gamma_{wf} \cdot \gamma_c}}$$

$R_{wf} = 180\ MPa,\ \beta_f = 0.9$

$$k_f = \frac{1}{0.9}\sqrt{\frac{845}{2 \cdot 85 \cdot 18 \cdot 1 \cdot 1}} = 0.58$$

Assume $k_f = 8\ mm$ and this leg met the requirements.

$min\ k_f = 5mm(16.1) < k_f = 8\ mm < max\ k_f = 1.2t_{min} = 12mm$

$$h_{tr} = l_{w,nec} + 1.0\ cm = \frac{N_{tr} \cdot 10}{2\beta_f \cdot k_f \cdot R_{wf} \cdot \gamma_c} + 1.0\ cm \le 85\beta_f k_f$$

$85\beta_f k_f$- maximum length of filet weld.

$$h_{tr} = \frac{845 \cdot 10}{2 \cdot 0.9 \cdot 0.8 \cdot 180 \cdot 1} + 1.0\ cm = 33.6\ cm$$

Assume $h_{tr} = 34\ cm$

The leg of filet welds for connecting traverse to the bearing plate assume (without calculations) from the table 16.1.

Rating efforts of separation for calculating anchor bolts we use with maximum bending moment (+) in cross-section 3-3.

$$N_a = \frac{1970.86}{0.9} - \frac{1337.74}{2} = 1520.97\ kN$$

Assume …bolts $N_b = \frac{N_a}{4} = 380.24\ kN$

$R_{ba} = 185\ Mpa$

$$A_{bn} = \frac{N_b}{R_{ba}} = \frac{380.24}{18.5} = 20.55\ cm^2$$

Assume anchor bolts Ø64mm.

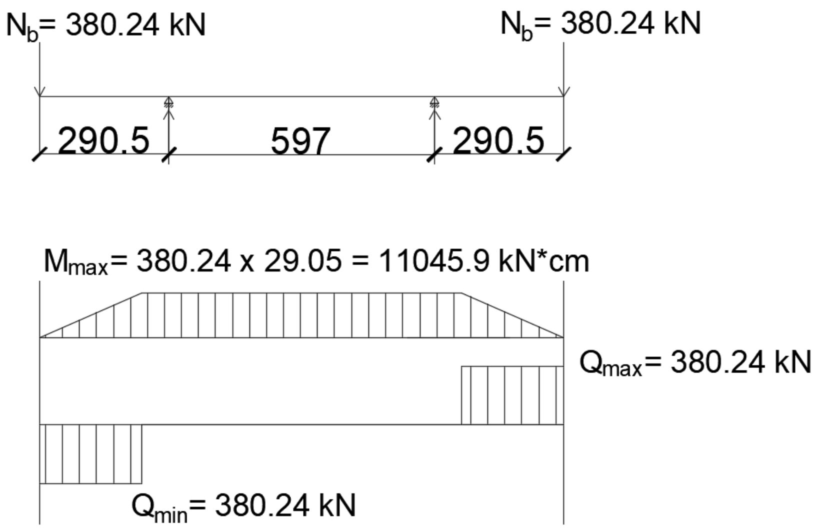


Fig.3.22. Rating scheme of traverse

Cross-sectional characteristics of traverse:

$A_{tr} = h_{tr} \cdot t_{tr} = 34 \cdot 1 = 34\ cm^2;$

$W_{tr} = \frac{34^2 \cdot 1}{6} = 192.67 cm^3;$

$\sigma = \frac{M}{W} \leq R_y \gamma_c;$

$\sigma = \frac{380.24 \cdot 10.2}{192.67} = 20.13\ kN/cm^2 < R_y \gamma_c = 24\ kN/cm^2;$

Conclusion: the condition is provided.

$\tau = \frac{1.5 \cdot Q}{A_{tr}} = \frac{1.5 \cdot 380.24}{34} = 16.78\ kN/cm^2;$

$\sigma_{red} = \sqrt{\sigma^2 + 3 \cdot \tau^2} \leq 1.15 R_y \gamma_c;$

$\sigma_{red} = \sqrt{20.13^2 + 3 \cdot 16.78^2} = 35.35\ kN/cm^2;$

$\sigma_{red} = 35.35\ kN/cm^2 > 1.15 R_y \gamma_c = 27.6\ kN/cm^2;$

Conclusion: the condition is not provided.

Enlarge traverse up to 360x12.

$A_{tr} = h_{tr} \cdot t_{tr} = 36 \cdot 1.2 = 43.2\ cm^2;$

$W_{tr} = \frac{36^2 \cdot 1.2}{6} = 259.2\ cm^3;$

$\sigma = \frac{M}{W} \leq R_y \gamma_c;$

$\sigma = \frac{380.24 \cdot 10.2}{259.2} = 14.96\ kN/cm^2 < R_y \gamma_c = 24\ kN/cm^2;$

Conclusion: the condition is provided.

$$\tau = \frac{1.5 \cdot Q}{A_{tr}} = \frac{1.5 \cdot 380.24}{43.2} = 13.20\ kN/cm^2\ ;$$

$$\sigma_{red} = \sqrt{\sigma^2 + 3 \cdot \tau^2} \leq 1.15 R_y\, \gamma_c\ ;$$

$$\sigma_{red} = \sqrt{14.96^2 + 3 \cdot 13.20^2} = 27.3\ kN/cm^2\ ;$$

$$\sigma_{red} = 27.3\ kN/cm^2 < 1.15 R_y\, \gamma_c = 27.6\ kN/cm^2\ ;$$

Conclusion: the strength of traverse is provided.

3.11.2. Calculation of anchor plates

Anchor plates are calculated on the carrying capacity of bolts.

Rating scheme of anchor plates:

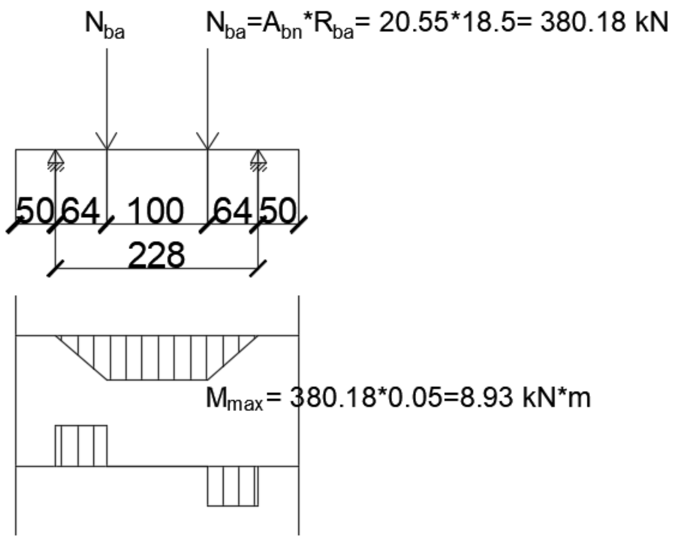


Fig.3.23. Rating scheme of anchor plates:

Assume width of anchor plate 228mm.

$$W_{nec} = \frac{M}{R_y\ \ \gamma_c} = \frac{8.93}{24 \cdot 1} = 0.37\ cm^2$$

$$C255(10\ldots 20mm)\ R_y = 240\ MPa$$

$$t_{a,p} = \sqrt{\frac{6M}{b_{a,p}}} = \sqrt{\frac{6 \cdot 0.37}{22.8}} = 0.31\ cm$$

Assume $t_{a,p} = 10\ mm$ (sheet steel 12-2 mm for milling)

CHAPTER 4
BASES AND FOUNDATIONS

4.1. Calculation and construction of a centrally loaded foundation for an average prefabricated column. Output data

Reinforced concrete foundations for metal columns are used for the perpendicular span of the industrial building. Concrete class C 16/20 is accepted.

Physical properties of the soil:

- the basis is a hard sandy loam with a porosity coefficient e = 0,5;
- liquidity index Il <0;
- specific adhesion of the bearing layer of the soil $c_{II} = 31\ kPa$;
- internal friction angle $\varphi = 19°$;

Determination of the depth of laying

The depth of the foundations is determined as a result of a joint consideration of the engineering-geological and hydrogeological conditions of the construction site, seasonal freezing and flexibility (looseness) of the soil, structural and operational features of buildings, the magnitude and nature of the load on the foundation.

Depth of foundations: $d_1 = 2,1\ m$;

Data of engineering and geological investigations Table 4.1

№	Name of soil	Natural humidity	Moisture at the yield point	Moisture at the limit plasticity	Soil density g/cm^3	Density of soil particles g/cm^3	Internal friction angle	Specific adhesion kPa	Deformation modulus MPa	Calculated resistance, kPa
		W	W_l	W_p	ρ	ρ_s	$\varphi_{II}{}^{\circ}$	c_{II}	E	R
1	Bulk soil depth 1.89 m				1,74					
2	Hard sandy loam, 2.35 m	0,066	0,25	0,2	19,2	1,5	19	31	23	330
3	Flowing sandy loam, 1.46 m	0,162	0,19	0,14	18,2	1,82	18	21	20	220
4	Soft loamy	0,178	0,23	0,14	17,5	2,05	20	37	26	280

	sand, 2.2 m									

Normative depth of soil freezing of the base (clause 7.5.3. DBN V.2.1-10-2018 Bases and foundations of buildings [12]).

$d_{fn} = d_0\sqrt{M_t}$;

Where $d_0 = 0.28\, m$ is the value taken according to clause 7.5.3. DBN V.2.1-10-2018 for sandy, fine and dusty sands [12];

$M_t = 2.8 + 5.1 + 4.0 = 11.9$ a dimensionless coefficient numerically equal to the sum of the absolute values of average monthly negative temperatures during the winter in Zhytomyr [11].

$d_{fn} = 0.28\ \cdot \sqrt{11.9} = 0.97$;

The estimated depth of seasonal soil freezing d_f, m, is determined by the formula:

$d_f = k_h\ \cdot\ d_{fn}$, where

$k_h = 1.1$ - the coefficient that takes into account the influence of the building's thermal regime, which is accepted: for external and internal foundations of unheated buildings, as well as when erecting a building in the winter period with negative temperatures [12] - according to Appendix G DBN V.2.1-10-2018.

$d_f = 1.1\ \cdot\ 0.97 = 1.067$;

We accept the minimum foundation depth:

$d = 1.067 + 0.5 = 1.567$;

We accept the laying depth of the foundation, based on the structural characteristics of the building frame $d_f = 2.1$.

4.2. Determining the dimensions of the foundation

We take a monolithic step-shaped foundation.

Full design load, taking into account the coefficient $\gamma_n = 1$.

$N_{II} = 2675.48\, kN$; $M_{II} = \pm 3741.72\, kN \cdot m$;

$N = \frac{2675.48}{2} + \frac{3741.72}{0.9} = 5717.19\,kN$;

Taking into account the average reliability factor, in relation to the load $\gamma_{fm} = 1.15$, operational load on the foundation is:

$N_e = \frac{N}{\gamma_{fm}} = \frac{5717.19}{1.15} = 4971.47\,kN$;

The calculated soil resistance is: $R = 0.33\,MPa$.

The material of the foundation:

- heavy concrete C16/20, $f_{cd} = 11.5\,MPa$, $f_{ctd} = 0.87\,MPa$, $f_{ck} = 15\,MPa$, $f_{ctk} = 1.4\,MPa$;
- class of armature - A400c, $f_{yd} = 365\,MPa$, $f_{yk} = 390\,MPa$, $f_{ywd} = 365\,MPa$, $E_s = 20\,\cdot\,10^{-4}$;

For the heavy concrete class C16/20, calculated characteristics of concrete at the coefficient of working conditions $\gamma_{c2} = 0.9$:

- calculated strength: $f_{cd} = 0.9\,\cdot\,11.5 = 10.35\,MPa$;
- tensile strength: $f_{ctd} = 0.9\,\cdot\,0.87 = 0.78\,MPa$;

We take the height of the foundation equal to: $H_1 = 1.65\,m$, the depth of laying $H_1 = 2.1\,m$;

Required foundation area:

$A_f = \frac{N_e}{R_0 - \gamma_m \cdot H_1} = \frac{4971.47 \cdot 10^3}{0.33 \cdot 10^6 - 20 \cdot 2.1 \cdot 10^3} = 17.26\,m^2$;

where, γ_m - mediated specific weight of wall blocks, foundation and soil, on the edges of the foundation is accepted conditionally, $\gamma_m = 20\,kN/m^3$;

The necessary dimensions of the side of the square in the plan foundation:

$a = \sqrt{A_f} = \sqrt{17.26} = 4.15\,m$; we accept $a = 4.2\,m$ (multiple of 300mm).

Then the base area will be: $A_{f,fac} = 4.2\,\times 4.2 = 17.64\,m^2$;

The pressure on the sole of the foundation is:

$p_{sf} = \frac{N}{A_f} = \frac{5717}{17.64} = 324.1\,kN/m^2 = 0.03241\,kN/cm^2$;

$p_{sf} = 324.1\,kPa < R = 330\,kPa$;

From the structural requirements, taking into account the need to reliably anchor the rods of the longitudinal reinforcement when the column is firmly pinched in the foundation, we accept the height of the foundation $H_f = 165\ cm$, the number of steps is two. The height of the step is determined on the basis of the concrete providing sufficient transverse force strength, without transverse reinforcement in the inclined section. Estimated sections: 3-3 along the face of the column, 2-2 along the face of the step, 1-1 along the lower border of the pressing pyramid. Constructively, we accept the height of the first step of the foundation $h_0 = 35\ cm$.

When calculating the reinforcement for the foundation, we take the bending moments on the cross-sections corresponding to the placement of the foundation ledges, as for a cantilever with a pinched end, as calculated:

$$M_{1-1} = 0.125 \cdot p_{sf} \cdot (a - a_1)^2 \cdot b;$$

$$M_{2-2} = 0.125 \cdot p_{sf} \cdot (a - a_2)^2 \cdot b;$$

$$M_{1-1} = 0.125 \cdot 324.1 \cdot (4.2 - 3.5)^2 \cdot 4.2 = 83.37\ kN \cdot m;$$

$$M_{2-2} = 0.125 \cdot 324.1 \cdot (4.2 - 2.8)^2 \cdot 4.2 = 333.5\ kN \cdot m;$$

4.3. Calculation of the required amount of reinforcement in different sections of the foundation in one direction:

$$A_{s1} = \frac{M_{1-1}}{0.9 \cdot f_{cd} \cdot h_{01}} = \frac{83.37 \cdot 10^2}{0.9 \cdot 280(0.1) \cdot 31} = 10.67\ cm^2;$$

$$A_{s2} = \frac{M_{2-2}}{0.9 \cdot f_{cd} \cdot h_{02}} = \frac{333.5 \cdot 10^2}{0.9 \cdot 280(0.1) \cdot 66} = 20.1\ cm^2;$$

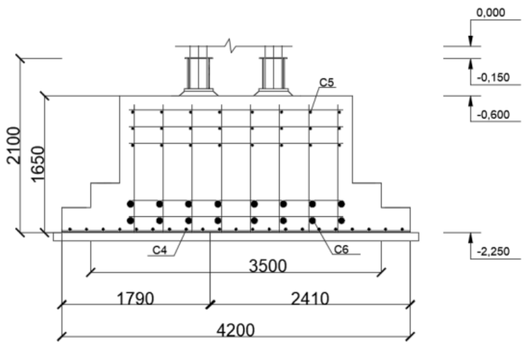


Fig.4.1. Centrally loaded foundation F-42 under the column of the outer row.

1. Of the 2 found A_{si}, it is accepted $A_{s,max} = A_{s2} = 20.1\ cm^2$;
2. We set the rod pitch s in the range of 200...300 mm (a multiple of 50 mm);
3. We set the distance from the edge of the sole to the first rod a_s in the range of 50...100 mm (a multiple of 25 mm);
4. We determine the number of rods n, according to the formula:

   $n_1 - 1 = \frac{b - 2 \cdot a_s}{s}$ - the result must be rounded to a larger whole number;
5. According to the assortment [25], we determine the diameter of the armature.

   We accept $s = 200mm$, $a_s = 50mm$;

$$n_1 - 1 = \frac{4.2 - 2 \cdot 0.05}{0.2} = 21$$

We accept the grid from the reinforcement 21Ø14 A400C, with a step of 200mm and area $A_{s,fac} = 24.62\ cm^2$.

Percentage of reinforcement: $\rho_f = \frac{A_{s,fac}}{b_1 \cdot h_{03}} \cdot\ 100\% \geq \rho_{f,min} = 0.1\%$;

$$\rho_f = \frac{24.62}{165 \cdot 161} \cdot\ 100\% = 0.1 = \rho_{f,min} = 0.1\%.$$

We accept the the lateral armature area without calculations, depending on the minimum armature coefficient $\rho_{f,min} = 0.1\%$.

$$A_s = \rho_{f,min} \cdot\ A_c = 0.001 \cdot (l_{uc} \cdot b_{uc}) = 0.001 \cdot (280 \cdot 280) = 78.4\ cm^2$$

Accept grid from the reinforcement 8Ø36 A400C, $A_s = 81.44\ cm^2$; $A_s > A_{s,max} = 20.1\ cm^2$.

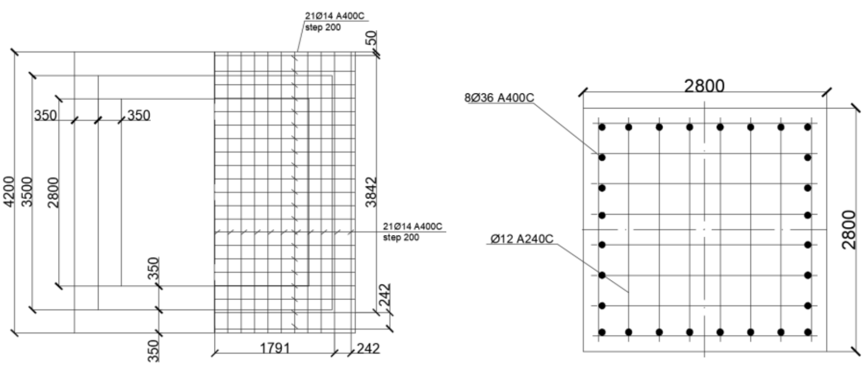


Fig.4.2. Scheme of reinforcement of slab (a) and column (b) parts of the foundation.

CHAPTER 5
ORGANIZATION OF CONSTRUCTION

5.1. General map

General map was developed for the construction of the above-ground part of the industrial building.

Temporary roads are assumed to be 6.0m wide. Curve radius 12m.

Temporary buildings and structures are of container type. On-site warehouses are designed to accommodate prefabricated elements and materials. storage areas were leveled by creating a slope for surface water runoff.

Table 5.1

№	Name	Quantity
1	Formwork warehouse	2
2	Fittings warehouse	2
3	Warehouse for cement, lime	2
4	Warehouse of prefabricated concrete	1
5	Warehouse of prefabricated metal el.	1
6	Warehouse of chemicals, paints, work clothes	1
7	Warehouse of thermal insulation materials, nails	1
8	Warehouse of the roofing material, hydroizol	1

Operation of electrical equipment without grounding is prohibited.

Fire safety at the construction site is ensured in accordance with the requirements of fire safety rules in the course of construction and installation works; electrical safety at the construction site is ensured in accordance with the requirements [26].

5.2. Calendar schedule

According to DBN A. 3.1-5:2016, the project for the construction of a building, structure or part of it includes a calendar schedule of work (or a complex network schedule), which sets the sequence and timing of work with the maximum possible combination [27].

The construction is carried out in two stages. The calendar schedule has been developed for the first stage of construction (two spans in axes "8-19").

Analytical part of the calendar schedule Table 5.2

№	Name of work	Scope of work		Labor costs, people-days	Duration of work in days	Number of working shifts	Composition of teams	Number of employees per shift
		Measurements	Number					
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*
1	Layout of the construction site	1000 m^2	22.5	0.65	1	1	driver of the 5th category	1
2	Digging of ditches, trenches with an excavator	1000 m^3	1.05	1.88	1	2	driver 5c	1
3	Cleaning the soil manually	1000 m^3	0.326	5.26	2	1	diggers 2c -2	4
4	Preparation for the foundation	100 m^3	3.26	0.75	1	1	concreters 4c-1, 2c-1	2
5	The device of a monolithic foundation	100 m^3	1.73	1050	88	2	carpenters 4c-1, 2c-1, drivers 4c-1, 2c-1, concreters 4c-1, 2c-1	6
6	Installation of columns	pcs	28	23.84	3	2	installers 5c-1, 4c-1, 3c-1, 2c-1	5
7	Installation of foundation	pcs	32	15	3	2	installers 4c-1, 3c-1,	3

	beams						2c-1	
8	Backfill of the soil	100 m^3	4.36	83.93	6	2	diggers 2c -2	8
9	Installation of the crane beams	pcs	15	24.38	4	2	installers 4c-1, 3c-1, 2c-1	4
10	Installation of the substructure beams	pcs	5	4.9	1	2	electric welder 5c-1, installers 5,4,3,2c-1	4
11	Installation of farms	pcs	22	45.65	5	2	electric welder 5c-1, installers 5,4,3,2c-1	5
12	Installation of coating plates	pcs	160	91.8	12	2	electric welder 5c-1; installers 4c, 2c-1; 3c-2	4
13	Installation of wall panels	pcs	380	335.4	34	2	electric welder 5c-1 installers 5,4,3,2c-1	5
14	Installation of window blocks	t	108	85.54	9	2		5
15	Installation of gates	t	4	20.7	3	2		5
16	Installation of half-timbered columns	pcs	12	10.22	2	2		5
17	Vapor barrier	100 m^2	28.8	68.04	8	1	isolators 3c-1, 2c-1	9
18	Insulation	100 m^2	28.8	173.52	11	1		17
19	Cement screed	100 m^2	28.8	51.48	5	1	isolators 4c-1, 3c-1	11
20	Rolled roof	100 m^2	28.8	200.88	11	1	roof builder 3p-2	19

21	Temporary glazing of window blocks	m^2	518.4	72.52	37	1	glaziers 4c-2	2
22	Waterproofing under the floor	100 m^2	28.8	249.84	25	1	concreters 4c-1, 2c-1	10
23	Flooring preparation under the floor	100 m^2	28.8	128.2	13	1	concreters 4c-1, 2c-1	10
24	Electrical installation works	-	186	186	19	1	electrics 5c-1, 3c-1	10
25	Plumbing works	-	325	325	28	1	plumbers 4c-1, 3c-1	12
26	Plastering works	100 m^2	31.07	287.4	8	1	plasterer 4c-1, 3c-1, 2c-1	40
27	Installation of mosaic floor, or another one	100 m^2	28.8	129.96	5	2	concreters 4c-1, 2c-1	14
28	Wall tiling	100 m^2	31.07	776.8	25	1	tiler 4c-1, 3c-1	32
29	Installation of equipment	-	325	325	37	1	installers 5,3c-1, 2pc-2	9
30	Equipment adjustment	-	102	102	12	1		9
31	Blind spots, ramps, etc.	100 m^2	2.74	1.51	1	1		4
32	Facade plaster	100 m^2	3.25	24.8	3	1	plasterer 4c-2, 3c-2, 2c-1	10
33	Facade cladding	100 m^2	3.25	109.7	7	1	tiler 4c-1, 3c-1	16

34	Preparation for the delivery of the object	-	36	36	8	1	handymen	5

Organizational and technological scheme of the accepted execution of works

Table 5.3

№	Name of work	Code of previous work	Accepted connection of works					min. gap in days, T_p	Duration of work in days, T_i	Justification
			No. of the calculation scheme							
			1	2a	3a	2б	3б			
			Consecutive 3-П	П-П	3-3	П-П	3-3			
1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*
1	Layout of the construction site	–	–	–	–	–	–	–	1	First work
2	Digging of ditches trenches with an excavator	1	3-П					0	1	Safety measures
3	Cleaning the soil manually	2				П-П		2	2	Safety measures
4	Preparation for the foundation	3					3-3	1	1	Organizational break
5	The device of a monolithic foundation	4	3-П					4	88	Concrete drying
6	Installation of columns	5	3-П					2	3	Safety measures
7	Installation of foundation beams with backfill of the soil	5 6	3-П				 3-3	2 4	9	Organizational break
8	Installation of the crane beams	7	3-П					2	4	Organizational break
9	Installation of secondary roof trusses, farms and coating plates	6 7 8		 П-П		П-П П-П		4 0 0	18	Technological break

10	Installation of Wall panels, window blocks and gates	9				П-П		3	46	Safety measures
11	Vapor barrier and Insulation of the roof	9 10	З-П			 П-П		0 3	19	Safety measures
12	Cement screed	11					З-З	1	5	Organizational break
13	Rolled roof	12				П-П		5	11	Drying of the screed
14	Temporary glazing of window blocks	10	З-П					0	37	Technological break
15	Waterproofing under the floor	13 14	 З-П				З-З	2 0	25	Organizational break
16	Flooring preparation under the floor	15					З-З	2	13	Organizational break
17	Plumbing works	14				П-П		3	28	Technological break
18	Electrical installation works	17			З-З			1	19	Safety measures
19	Plastering works	17 18			З-З З-З			0 1	8	Safety measures
20	Installation of mosaic floor, or another one	16 19				 П-П	З-З	2 0	5	Organizational break
21	Installation of equipment and equipment adjustment	20	З-П					4	49	Drying of the floor
22	Blind spots, ramps, etc.	10	З-П					0	1	Technological break
23	Facade plaster	22		П-П				1	3	Safety measures
24	Facade cladding	23	З-П					2	7	Technological break
25	Preparation for the delivery of the object	21 22 24	З-П З-П З-П					0 0 0	8	Organizational break

The calendar schedule was developed in the Excel software module.

CHAPTER 6
TECHNOLOGY OF CONSTRUCTION

6.1. Technological map

The technological map was developed for the process of erecting a reinforced concrete roof slab during the construction of the steel frame mill building for ceramic blocks production in Zhytomyr city.

The process is carried out by the truck crane LIEBHERR 420 EC-H 16.

The work is performed by a complex team in two shifts.

The installation of the roof slabs is carried out in parallel with the installation of the beams. Laying of slabs is carried out in the places of their stacking in the course of the works [32]. The slabs are laid out inside the span being mounted. Installation of roof slabs is carried out by a group of installers of 4 people (4 c-1, 3 c-2,, 2 c-1).

Installation is carried out in the following order:

1. Slabs are strung with the help of a traverse (installers of the 3rd and 2nd category).
2. The plate is lifted to the place of installation, while it is oriented in space with the help of a tie (installers of the 3rd and 2nd category).
3. Installers of the 4th and 3rd category, located at the top, take the plate and install it on the rafter beam in such a way that the supporting ribs rest on the supporting parts of the rafter beam;
4. Installers of the 4th and 3rd category use crowbars to move the slab in plan, achieving its exact design position;
5. Welding of collateral parts is carried out. At the same time, the first plate is welded at four points, and the others at no less than three, since one of the corners of the plate is not available for welding. In the case when the gap between the securing parts of plates and rafter structures exceeds 4 mm, steel pads are installed, which are welded to the securing parts of the rafter beams and covering plates. After the plate is unfastened.

6.2. Selection of crane equipment

The choice of a set of machines for carrying out work on the installation of structures depends on the dimensions and weight of individual structures and the conditions for performing installation work. The main parameters of mounting cranes are boom reach, hook lifting height, load capacity.

In this project the choice of a crane was made according to the characteristics of the heaviest installed element - 14.8 t.

1. The required lifting capacity of the crane

$$Q = q + q_0 = 14.25 + 0.513 = 14.763\ t \approx 14.8t$$

2. The minimum lifting height of the cargo hook for the jib crane

$$H_{n.k} = h_0 + h_з + h_e + h_c = 12.6 + 0.5 + 3.3 + 5.4 = 21.8\ m$$

$$H_{cmp}^{mp} = h_0 + h_з + h_e + h_c + h_n = 12.6 + 0.5 + 3.3 + 5.4 + 1.5 = 23.3\ m$$

3. The required length of the boom

$$L_{kp}^{mp} = \sqrt{\left(l_{kp}^{mp} - c\right)^2 + \left(H_{cmp}^{mp} - h_ш\right)^2}$$

The departure of the crane hook (boom)

$$l_{cmp}^{mp} = \frac{(e+d'+a)\cdot\left(H_{cmp}^{mp} - h_ш\right)}{h_n + h_c} + c = \frac{(0.5 + 0.5 + 0.24)\cdot(23.3 - 1.5)}{1.5 + 5.4} + 1.5 = 5.5m$$

$$L_{kp}^{mp} = \sqrt{(5.5 - 1.5)^2 + (23.3 - 1)^2} = 22.7m$$

CHAPTER 7
LABOR PROTECTION

7.1. Labor protection measures

1) Wearing protective helmets is mandatory for all persons present at the construction site.

2) When moving and supplying formwork and fittings to the workplace by cranes, pallets, containers and load-catching devices are used, which exclude the fall of the load during lifting.

3) Dangerous zones must be marked with safety signs and inscriptions of the prescribed form.

4) Safety fences are installed on the borders of the zones of permanently active dangerous production factors, and signal fences or safety signs are placed on the zones of potentially active dangerous production factors.

5) Placement on the formwork of equipment and materials not provided for in the work production project, as well as the presence of people who are not directly involved in the production of work on the formwork deck, is not allowed.

6) Safety fences must be installed on the borders of the zones of permanently active dangerous production factors, and signal fences or safety signs must be installed on the zones of potentially active dangerous production factors.

7.2. Increasing the fire resistance of metal structures of industrial buildings

Fire protection works consist of several main stages:

- Surface preparation and quality control;
- Application of anti-corrosion paint and varnish material with the execution of an act of hidden works;
- Application of flame retardant material;
- Quality control of the performed work.

The contractor is required to notify the local state fire supervision authority in written form about the beginning of work at the facility. All fire protection work must be carried out in climatic conditions that meet the regulations for work on fire protection with this material. During the work on fire protection of building structures, the customer, representatives of the state fire supervision authorities, the developer of the Fire Protection Work Project, the manufacturer or supplier of the fire protection material used have the right to carry out intermediate quality control of the work. In case of deviations from the Project or violations of the requirements of the Regulations, an act of violation is drawn up in at least three copies and signed by all participants in the audit.

The act requires that one copy of the act remains with the customer of the work, the contractor, and the third must be sent to the territorial licensing authority for the provision of services and the performance of fire-fighting work. If architectural and construction changes are made at the beginning of the fire protection work at the facility that are contrary to the Project of fire protection work, the manufacturer of this type of work is obliged to stop (or not start) the work and inform the customer, the developer of the Project, and the local state authority about this. Fire supervision is necessary to continue the work. After completion of works on fire protection of steel structures, partial mechanical processing is allowed and damaged surfaces must be restored with materials used in accordance with the Fire Protection Design.

Metal surfaces prepared for corrosion protection treatments shall be free of any irregularities or contamination. Before the protective coating is applied, the surface of the steel structure shall be blasted and cleaned using a shot blasting machine or mechanical brush. Surface cleaning methods are described in the technical documentation of the coating material that have been used. The surfaces of steel structures intended for treatment should be cleaned only from peeling rust or scale films.

Before using fire-retardant materials to increase the fire resistance of load-bearing steel structures, it is necessary to carry out anti-corrosion treatment of metal structures. Primer anticorrosion coatings are selected depending on the operating conditions of the metal structures and the entire fire protection system in general. In

Ukraine, there are strict requirements for the conformity of the primer coating and the fire retardant material. If the metal structures were previously treated with an anti-corrosion coating, the organization performing the fire protection work is required to obtain a document (Act of hidden works for anti-corrosion treatment, quality certificate for the used paint and varnish material) confirming the use of anti-corrosion materials that comply with the requirements of the Regulations for works on fire protection. Before applying fire protection materials, the manufacturer is obliged to conduct an audit of the state of the anti-corrosion coating, determine the damaged parts.

The Certificate of Conformity of the UkrEPRO system for fire retardant material must contain all elements of the fire retardant system indicating the trade names, thickness and regulatory documents of the primer layer, especially the fire retardant coating and, if necessary, lacquer or enamel. Replacing one of the components, for example, a primer, requires mandatory fire tests in accordance with current standards and possibly certification. The procedure for certification (fire tests) of fire protection systems includes, in each case, the repetition of the same tests, and is very tedious, labor-intensive and expensive. In some cases, the manufacturer assumes responsibility for applying the fire protection material to the existing anti-corrosion coating, and an examination is carried out for the compatibility of the system of anti-corrosion coating - fire-retardant material and stability under conditions of thermal loads.

Before starting work, it is necessary to carry out an incoming control of the fire-retardant material, paying attention to the integrity of the packaging, the presence of markings, the availability of accompanying documents, and the compliance of the quality passport with the batch number indicated on the marking label. Accompanying documents include copies of the current Certificate of Conformity of the UkoSEPRO system and the Regulations for work on fire extinguishing, as well as a quality certificate indicating the technical and physical and chemical characteristics of the supplied batch of fire retardant material obtained during production acceptance tests.

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APPENDIX 1
(Album of drawings)