Saimaa University of Applied Sciences
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THE DESIGN OF AN INDUSTRIAL BUILDING ACCORDING TO RUSSIAN AND EUROPEAN CONSTRUCTION NORMS

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ABSTRACT
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Two topics, related to each other, were considered in this work. The main topic is the design of an industrial building frame according to Russian construction norms (SNiP) and Eurocodes. Another topic is the comparison of the design methods between SNiP and Eurocodes.

The thesis includes information about loads estimation, the design of concrete columns, steel trusses and foundations according to SNiP. For the purpose of comparison, the design of concrete columns and steel trusses according to Eurocodes was considered, too. Along with this, the static analysis of a frame structure was discussed. The stresses in elements of the frame were determined using two methods of static mechanics (the force and flexibility methods).

The thesis was made for Quattrogemini Ltd which is a contractor for construction in Russia. A simple method for preliminary design of typical industrial buildings was needed for the company. The development of such a system for building design was accepted as the thesis purpose. Calculation spreadsheets for the design of concrete columns, steel trusses and foundations were made as one of the results of this work. These spreadsheets are useful, for example, in designing tender projects.

A typical industrial building was chosen for this work. The main elements of the building frame are concrete columns and primary and secondary steel trusses. The building is located in Saint-Petersburg, Russia. The static analysis of the building frame, loads estimation, the design of the main frame elements and foundations were made. These calculations can be found in Appendices.

As the result of SNiP and Eurocodes comparison, some differences were found. According to Eurocodes, snow and wind loads according to Eurocodes are significantly higher (about 2 times) than the corresponding loads in SNiP. For the building considered in this work, the effective length of the columns pursuant to Eurocodes is much greater in comparison with SNiP and causes significant differences in the design results. Required reinforcement area is much higher according to Eurocodes because loads and effective length of the columns are greater in these norms. Profile dimensions of the steel truss members are higher pursuant to Eurocodes, mostly, because of the difference in snow loads.

Keywords: SNiP, Eurocodes, industrial building design, steel truss dimensioning, column reinforcement design, static analysis, foundations design, loads estimation.

CONTENTS

1 INTRODUCTION

The design of a typical industrial building is considered in this work. The dimensioning of steel truss members and foundations and the calculation of column longitudinal reinforcement area are discussed. The calculations of the main frame elements have been made according to Russian construction norms (SNiP). Also, the design of concrete columns and steel trusses is considered according to Eurocodes.

The thesis was made for company Quattrogemini Ltd. The company is a general contractor for construction in Russia. A quick and simple method for the design of industrial buildings was needed for the company. The main purpose of the thesis is to find a simple way for a preliminary design of typical industrial buildings. As one of the results of this work, the calculation spreadsheets for the design of columns, steel trusses and foundations were made. The calculation spreadsheets were made in Microsoft Excel and include Visual Basic macros. In practice, this system can be applied for preliminary design in tender projects.

The main dimensions of an industrial building are shown in Figure 1.1. The secondary truss span usually equals 18–30 m. The typical spaces for exterior columns are 6 m or 12 m, for interior columns 6 m, 12 m or 18 m. The distance between wind columns is 6 m. In general, the clear height of the building is from 5 to 12 m. Wind columns made of steel are usually used.

A typical roof system (see Figure 1.2) usually includes a load bearing profile sheeting, thermal and waterproof insulations and others. The height of the profile sheeting for the buildings without purlins is 75 mm, with purlins up to 114 mm. The thickness of thermal insulation is 100–150 mm.

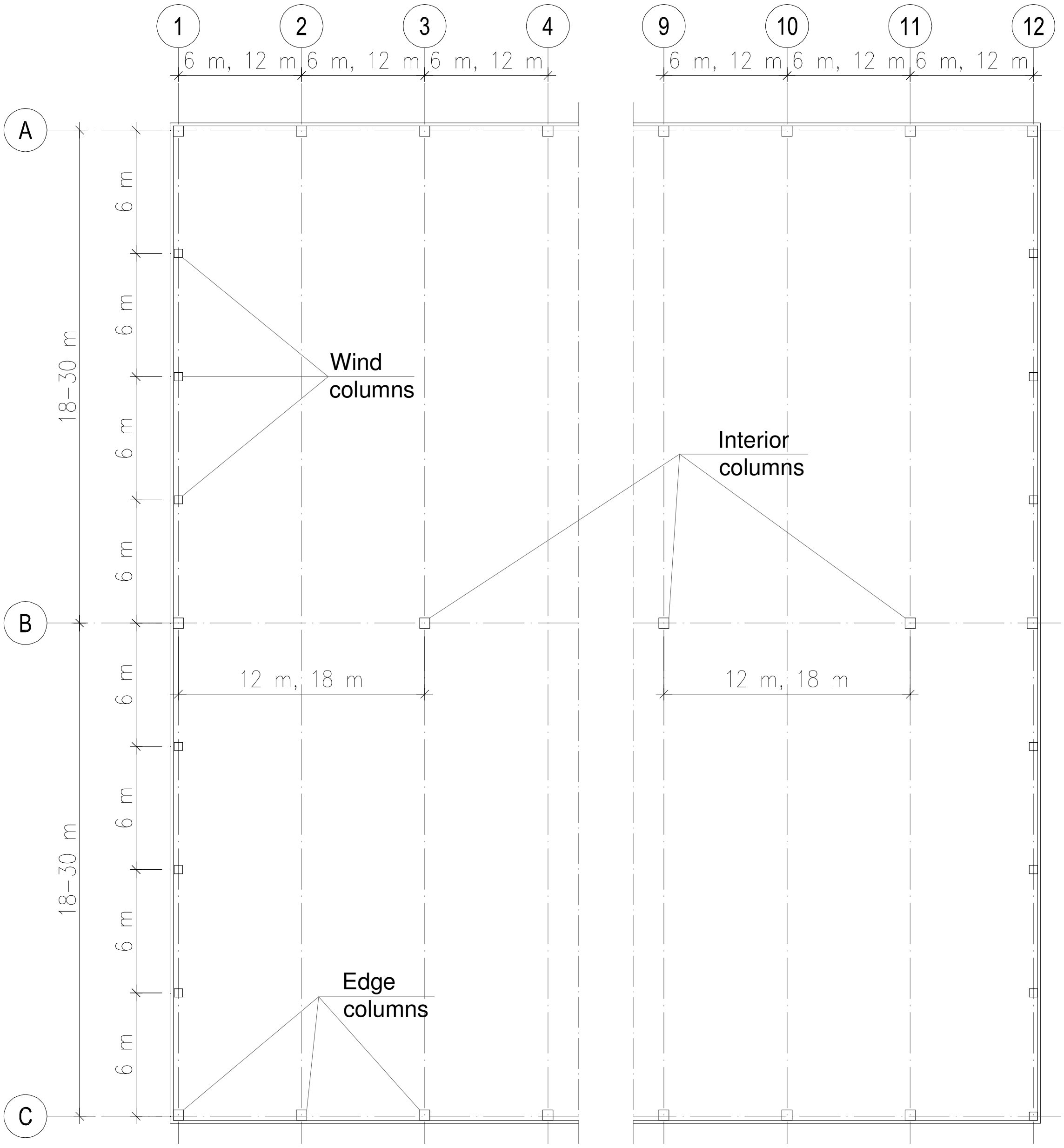


Figure 1.1 Typical plan of an industrial building

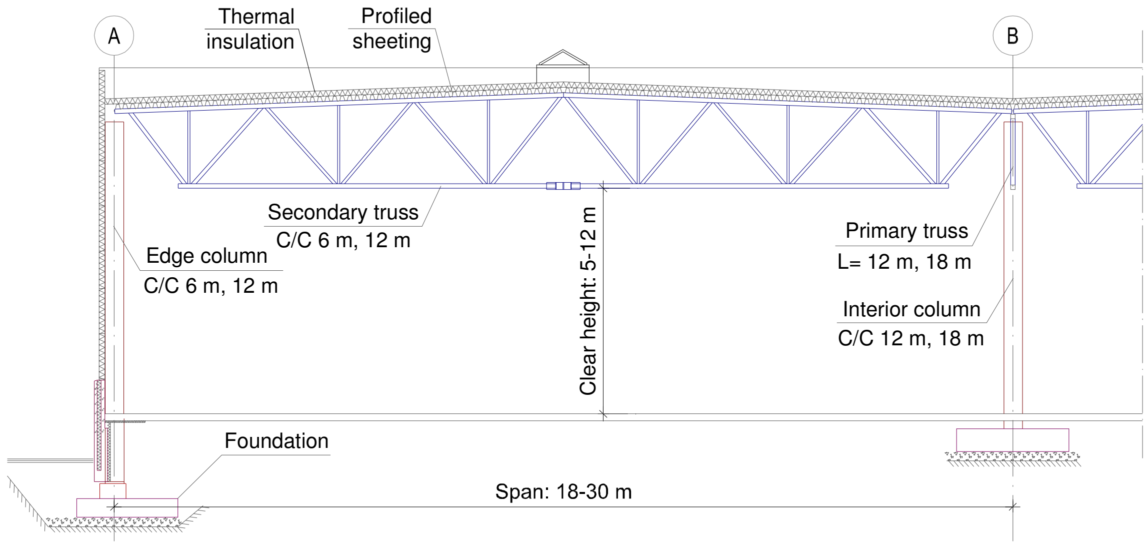


Figure 1.2 Typical section of an industrial building

There are two joint types that are commonly used for a column-foundation connection: sleeve connection (see Figure 1.3) and anchor bolt connection (see Figure 1.4).

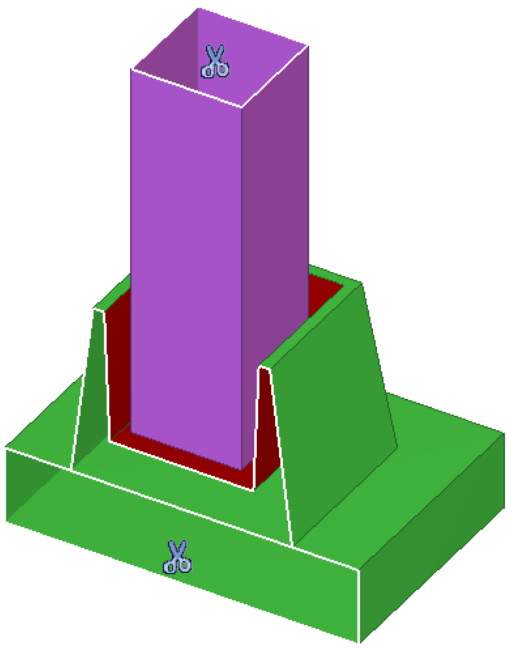


Figure 1.3 Sleeve joint between a column and a foundation (modeled in Tekla Structures)

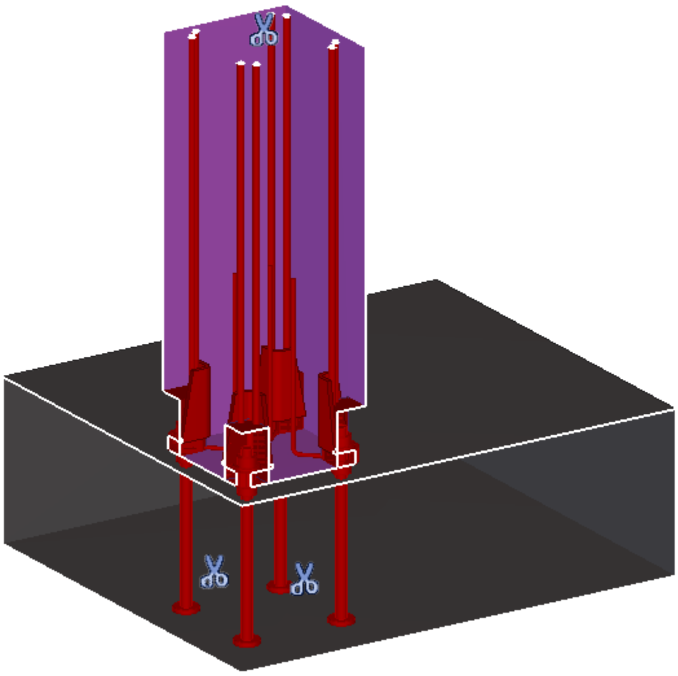


Figure 1.4 Anchor bolt joint made by Peikko (modeled in Tekla Structures)

The both joint types are rigid: they transfer bending moment from a column to a foundation. Fastening items in Figure 1.4 are column shoes. Usually four column shoes are enough to create a moment stiff connection. The capacity of the joint is defined by the bolt being used. Generally, the bolt diameter is 24–42 mm.

A typical industrial building was chosen as an example for the designing. The building is shown in Figure 1.5. The span of the secondary trusses is 24 m, the span of the primary trusses 12 m. The spacing for the exterior columns is 6 m, for the interior columns 12 m. The clear height equals 6 m. An anchor bolt joint is used as a column-foundation connection. The connection between columns and trusses is pinned. The profiled sheeting is attached to the secondary trusses directly, without purlins. Top chords of the trusses are subjected to axial force and bending moment in this case. The roof sheeting acts as a lateral support for out-of-plane deformation, therefore the chord is not required to be checked for out-of-plane buckling.

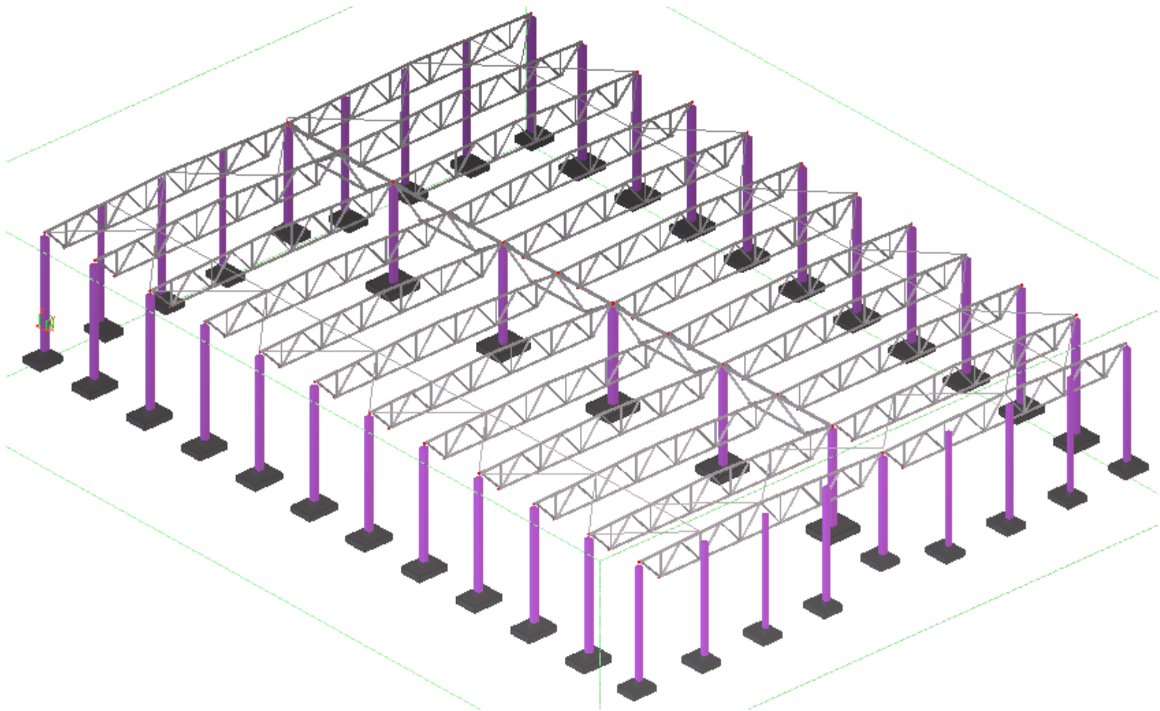


Figure 1.5 Model of an industrial building frame created in Tekla Structures

Concrete cover requirements due to fire resistance are described in MDS 21-2.2000 (Russian norms). For the building considered in this work (fire resistance rating II), fire resistance degree of the columns should be R90, for the trusses R15. The concrete cover of the columns should be not less than 35 mm (MDS 21-2.2000, cl. 12.4). Fire protection is not required for the trusses.

The frame elements (columns and trusses) of the example building were designed according to Russian construction norms and Eurocodes, the foundations were dimensioned only according to SNiP. Also, the Finnish National Annex was used in this work for the loads estimation, partial safety factors determination, etc. Designation of the National Annex is shown in Figure 1.6: Finnish Annex to EN 1991-1-4 is specified. Numerical examples of the calculations can be found in the Appendices. The calculations were made using MathCAD.

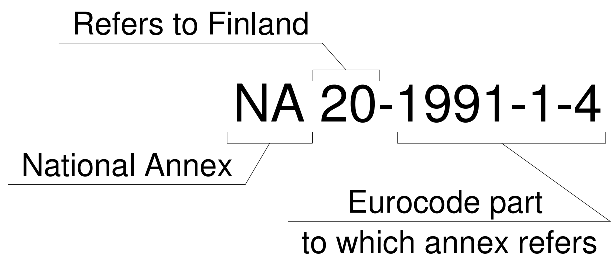


Figure 1.6 Designation of National Annex to Eurocode

Usually, the bracings at the level of top chord are only used. The main purpose of these bracings is to transfer the actions of wind loads to a frame. The bracings at the level of bottom chords are not generally used.

2 LOADS AND INFLUENCES ACCORDING TO RUSSIAN NORMS

In Russian Federation the main document for the determination of the loads acting on buildings is SNiP 2.01.07-85*. It contains the information about combinations of loads, weight of structures, imposed loads, snow loads, wind loads, ice-cover loads, temperature and climatic influences, etc. These norms were introduced by CNIISK after Kucherenko of Gosstroi of the USSR and were approved in August 29, 1985.

The main characteristics of loads are their normative values. The load of a certain type is characterized, as a rule, by one normative value. For some loads (loads due to temperature influences, live loads, snow loads and others) it is established two normative values: the full and reduced. The reduced value is used when it is necessary to take into account the duration of loads, to check on endurance and in other cases stipulated in the norms of designing the structures and bases. (SNiP 2.01.07-85* 1986, cl. 1.2).

The design value of the load is to be determined as a product of its normative value by a safety factor per load γ_f corresponding to the limit state being considered (SNiP 2.01.07-85* 1986, cl. 1.3).

Safety factors, wind and snow loads according to SNiP and Eurocodes are listed in this chapter. It is necessary to mention that the comparison of the norms on the basis of single factors or normative load values is not representative because the design results depend also on other values (material safety factors, for example), so, despite the difference, design results can be identical.

The example of loads estimation according to SNiP 2.01.07-85* is shown in Appendix 1a. The calculation was carried out for the industrial building discussed in Chapter 1. The theory of loads estimation is considered below. Also, the loads estimation pursuant to EN 1991 was made for calculations according to Eurocode. It can be found in Appendix 1b.

2.1 Classification of loads (SNiP 2.01.07-85*)

Depending on the duration of the load action there are:

1. Permanent loads (weight of parts of facilities including weight of carrying and enclosing structures)
2. Temporary loads
    a) Long-term loads (weight of stationary equipment; loads onto floors from the materials being stored);
    b) Short-term loads (snow loads with the full normative values; wind loads);
    c) Special loads (seismic influences; explosion influences).

(SNiP 2.01.07-85* 1986, cl. 1.4–1.9).

2.2 Combinations of loads (SNiP 2.01.07-85*)

Calculation of loads combinations is defined in Clauses 1.10–1.13 of SNiP 2.01.07-85*. The design of the structures and bases per limit states of the first and the second groups (ultimate and service limit states, correspondingly) is to be made taking into account the most unfavorable combinations of loads and the corresponding stresses.

When taking into account the combinations including the permanent and not less than two temporary loads, the design value of temporary loads or the stresses corresponding to them are to be multiplied by the factors of combinations ψ. In the main combinations for long-term loads factor of combinations $\psi_1 = 0.95$, for short-term – $\psi_2 = 0.9$

An example of the load combination calculation in tabular form is shown in Table 2.1. It refers to the industrial building taken as an example. For the building, four different loads were taken into account: self-weight of structures, wind load, snow load and equipment load.

Table 2.1 Example of load combination

Load	**Type of load**	**Combination factor ψ**
Self-weight of structures	Permanent	1
Wind load	Short-term	0.9
Snow load with full normative value	Long-term	0.95
Equipment load	Long-tem	0.95

The estimation of loads combinations is usually made by using the calculation automatization software such as SCAD Office or Robot Structural Analysis. This

software usually calculates all the possible loads combinations, the most unfavorable ones are chosen for design of a structure.

2.3 Weight of structures (SNiP 2.01.07-85*)

The normative value of the weight of structures is to be determined on the basis of standards, detailed drawings or data, specified by manufacturing plants, other construction structures per design sizes and specific weight of the materials taking into account their humidity under conditions of construction and operation of the buildings. (SNiP 2.01.07-85* 1986, cl. 2.1).

Example values of the safety factors for weight of structures are listed in Table 2.2. The values show the difference in safety factors for steel and concrete structures between SNiP and NA 20-1991-1-1 for Eurocode.

Table 2.2 Safety factors for some structures (SNiP 2.01.07-85* 1986, Table 1; NA 20-1991-1-1)

Structures	**Safety factor according to SNiP**	**Partial safety factor (ULS) according to Eurocode**
Steel	1.05	1.15
Concrete (with the average density over 1600 kg/m^3), reinforced concrete, stone, reinforced stone, wooden	1.1	

According to Finnish NA the partial safety factor for permanent loads equals 1.15 (for ultimate limit state). The value is a little bit higher than the corresponding safety factors according to SNiP.

2.4 Loads from the equipment (SNiP 2.01.07-85*)

The normative value of the weight of the equipment is to be determined on the basis of standards or catalogues and for non-standard equipment on the basis of standard passport data of manufacturing plants or working drawings. (SNiP 2.01.07-85* 1986, cl. 3.3)

Generally for industrial buildings the normative value of equipment load on ceiling is 0.3–0.5 kN/m^2. In the calculations for negative (vertical) wind pressure, equipment load may be accepted equal 0–0.1 kN/m^2.

The safety factor for stationary equipment γ_f is to be taken equal 1.05 (SNiP 2.01.07-85* 1986, Table 2). According to Eurocode 0 (2002), the partial safety factor for equipment is 1.2 (for ultimate limit state); the value is noticeably higher than the factor in SNiP.

2.5 Snow loads (SNiP 2.01.07-85*)

The full design value of the snow load onto horizontal projection of the coatings is to defined by Formula 5 of SNiP 2.01.07-85*:

$$s = s_g \mu \tag{2.1}$$

where

s_g – the design value of the snow cover weight per 1 m^2 of the horizontal ground surface;

μ – the factor of transfer from the snow load of the ground to snow load onto coating. For double pitched roofs with the slope (α) up to 25°, the factor μ equal 1.

The design value of snow load depends on the snow region. For example, for Saint-Petersburg and Moscow (snow region III) s_g = 1.8 kN/m^2 (SNiP 2.01.07-85* 1986, Table 4). According to EN 1991-1-3 (2004), the snow load near Saint-

Petersburg is $s = \mu_i C_e C_t s_k = 0.8 \cdot 1 \cdot 1 \cdot 2.75 = 2.2 \frac{\text{kN}}{\text{m}^2}$. Also, this value should be multiplied by partial safety factor γ_f = 1.5: $\gamma_f s = 1.5 \cdot 2.2 = 3.3 \frac{\text{kN}}{\text{m}^2}$. The difference with the value according to SNiP is significant and causes discrepancy in design results.

2.6 Wind loads (SNiP 2.01.07-85*)

Estimation of wind loads is defined in Clauses 6.1–6.11 of SNiP 2.01.07-85*. The wind load should be determined as the sum of the average and pulsating components. When calculating multi-storied buildings with the height up to 40 m and one-storey industrial buildings with the height up to 36 m at the ratio of the height to the span of less than 1.5, located in wind regions of types A and B, it is admitted not to take into account the pulsating component of the wind load.

The normative value of the average component of the wind load at the height z above the earth surface should be determined by formula:

$$w_m = w_0 k c \tag{2.2}$$

where

w_0 – normative value of the wind pressure;

k – the factor taking into account the change in the wind pressure per height. It also depends on the type of locality (A, B and C), similar to terrain categories in Eurocodes. Locality type B (suburban terrain) was accepted for the considering example;

c – aerodynamic factor taking into account scheme of buildings.

The normative value of the wind pressure w_0 is to be accepted depending on the wind region. For example, for Saint-Petersburg (wind region II) w_0 = 0.3 kN/m^2, for Moscow (wind region I) – 0.23 kN/m^2 (SNiP 2.01.07-85* 1986, Table 5). The above listed values are for locality type B.

Example values of the design wind pressures are listed in Table 2.3. The values are given depending on level and location. For Saint-Petersburg and Moscow the wind pressures were calculated for locality type B, corresponding terrain category III of EN 1991-1-4.

Table 2.3 The design active wind pressures for Moscow, Saint-Petersburg (B regions) according to SNiP and for wind region III according to EN 1991-1-4, kN/m^2

	Level, m		
Location	**< 5**	**10**	**20**
Moscow (SNiP)	0.13	0.17	0.22
Saint-Petersburg (SNiP)	0.17	0.22	0.29
Terrain category III (EN 1991-1-4)	0.37	0.49	0.63

Safety factor (γ_f) for the wind load is to be taken equal 1.4 (SNiP 2.01.07-85* 1986, cl. 6.11). Partial safety factor equal 1.5 is used in Eurocode. As it can be seen from Table 2.3, wind loads according to Eurocode are much higher. When more than one variable load is to be considered, EN 1990 allows the use of combination factors, which reduce the design values of loads. The combination factor equal 0.6 can be used for the wind load.

Besides the horizontal wind load, there is a vertical (negative) wind load which can cause lifting effect for roof structures. The values of the load depend on the roof slope and the ratio of building height to its length. Vertical wind load and permanent load, acting on a truss, are shown in Figure 2.1.

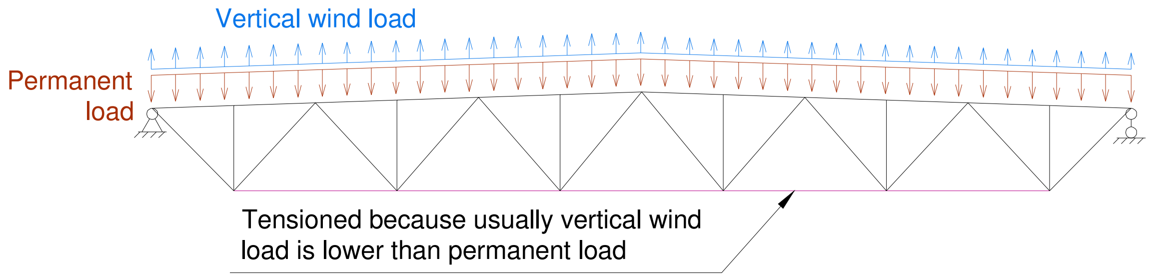


Figure 2.1 Vertical (negative) wind load

Usually, vertical wind load is less than self-weight of roof. Design values of self-weight of roof structures and negative wind load for typical industrial building are listed in Table 2.4.

Table 2.4 Comparison of negative wind force and self-weight of roof structures

Design values of structures self-weight ($g = g_n \cdot \gamma_f$)		Negative wind load
Profiled sheeting	$0.1 \cdot 1.05 = 0.11$ kN/m^2	$w = \gamma_f w_0 c_e k =$ $= 1.4 \cdot 0.3 \cdot 0.4 \cdot 0.7 = 0.12 \dfrac{\text{kN}}{\text{m}^2}$
Thermal insulation	$0.2 \cdot 1.3 = 0.26$ kN/m^2	
Steel truss	$0.3 \cdot 1.05 = 0.32$ kN/m^2	
Total	0.69 kN/m^2	0.12 kN/m^2

As it can be seen from the table, self-weight of roof structures are much higher than the negative wind load, so the resultant force is directed downwards in this case. Therefore, the top chord of the truss is compressed, the bottom chord is tensioned.

3 STATIC ANALYSIS OF A FRAME

The analytical model of the frame for the building taken as an example is shown in Figure 3.1. The frame structure is a statically indeterminate system. In this case, one of the methods of statics should be applied to find member forces. The flexibili-

ty and force methods were considered in this work. The degree of statical indeterminacy equals 2, the degree of kinematic indeterminacy equals 1 for the system. This means than the number of unknowns for the force method equals 2, for the flexibility method 1. Therefore, the flexibility method is more preferred in this case because the number of unknowns is less than for the force method. The calculation using the flexibility method is simpler for this frame. The calculations of member forces using the force and flexibility methods can be found in Appendices 2 and 3, correspondingly. The same analytical model was used in examples for the both methods.

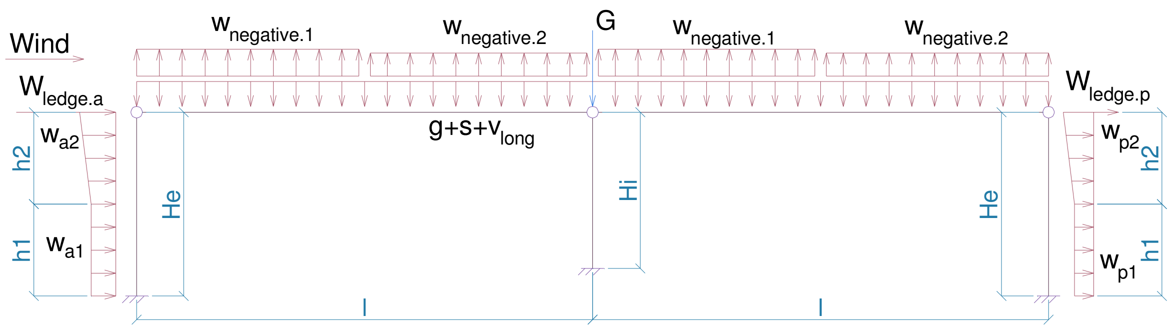


Figure 3.1 Analytical model of the frame

The designations in Figure 3.1:

g – permanent load (weight of structures);

G – node load from the primary trusses;

s – snow load;

v_{long} – long-term load (load from equipment);

$w_{negative.1}$, $w_{negative.2}$ – vertical (negative) wind loads;

w_{a1}, w_{a2} (w_{p1}, w_{p2}) – active (passive) horizontal wind loads at levels 5 m and 10 m, correspondingly;

$W_{ledge.a}$ ($W_{ledge.p}$) – the node force equivalent to the active (passive) wind pressure above the column-header connection;

H_e – height of the exterior column;

H_i – height of the interior column;

h_1, h_2 – heights of the sections along which the wind load varies linearly.

The results of static analysis are shown in Figures 3.2 and 3.3. Distribution diagram of longitudinal forces is presented in Figure 3.2. As seen from the figure, the columns are subjected to compressive longitudinal force. The interior column is the most loaded one: compressive force in the interior column is about 4 times higher than in the exterior columns.

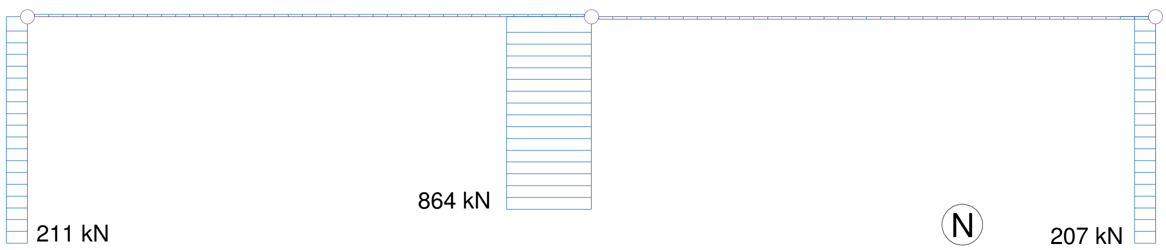


Figure 3.2 Diagram of longitudinal internal forces

Distribution of bending moments is illustrated in Figure 3.3. The connection between a column and a foundation is rigid (anchor bolt joint). The foundation can take bending moments in this case.

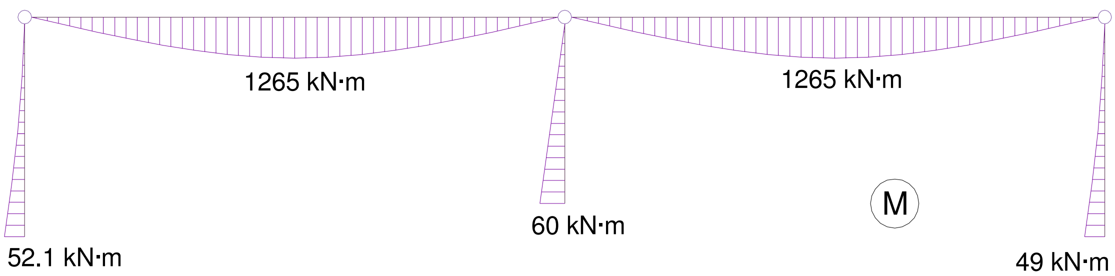


Figure 3.3 Distribution of bending moments

The diagrams show that the columns are subjected to axial force and bending moment. This fact should be taken into account in designing.

4 THE DESIGN OF THE COLUMN LONGITUDINAL REINFORCEMENT

The design of column longitudinal reinforcement according to Russian construction norms (SNiP 2.03.01-84*) and Eurocodes (EN 1992) is considered in this chapter. Two types of columns are used in the building: exterior and interior. The design models for the both columns are illustrated in Figure 4.1. The columns should be designed as eccentrically compressed cantilever columns. The interior column is the most compressed.

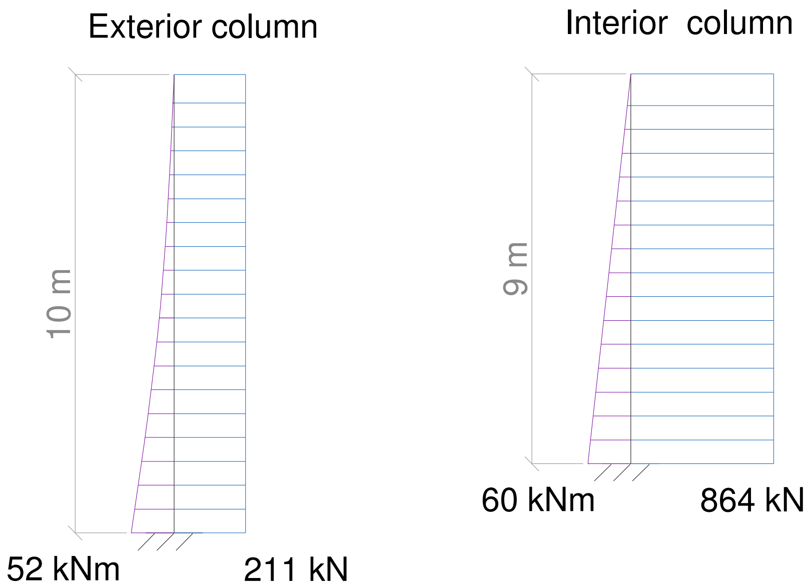


Figure 4.1 The design models of the exterior and interior columns (loads estimated according to SNiP)

There is a large difference in calculation of column effective length pursuant to SNiP and Eurocodes. For the building considered in this work, according to Table 23 of Guide to SNiP 2.03.01-84* (1995), the effective length for the constant section columns of single-storey multispan buildings without cranes should be 1.2·L,

but according to Table 7.1 of Designers' Guide to EN 1992-2-1 (2005), this value is to be 2·L. This difference between the norms causes significant discrepancy in required reinforcement area of the columns. The comparison of the design results for the columns is shown in Chapter 7.1.

4.1 The design of the longitudinal reinforcement according to SNiP 2.03.01-84*

The design of the longitudinal reinforcement for off-centrally compressed members of rectangular cross-section is considered in this chapter. Guidance on the calculation of the required reinforcement area for the columns of rectangular cross-section is given in Chapter 3 of Guide to SNiP 2.03.01-84* (1986).

To take into account loads duration influence on concrete strength, design of concrete and reinforced concrete elements is made:

a) considering permanent, long-term and short-term loads, except non-long-term loads (wind, crane loads, etc); in this case concrete specific-conditions-of-use factor γ_{b2} = 0.9 (it is used in calculation of design concrete strength R_b);
b) considering all loads; in this case γ_{b2} = 1.1.

Calculation is made only for case "b" if:

$$F_I < 0.82 F_{II} \tag{4.1}$$

where

F_I – force corresponding to the loads of case "a";

F_{II} – force corresponding to the loads of case "b".

Account may be made only for case "b" even when Condition 4.1 is not satisfied. In this case, the design concrete strength is assigned using coefficient $\gamma_{bl} = 0.9F_{II}/F_I \leq 1.1$.

The required area of symmetric reinforcement defined as follows, depending on the relative value of longitudinal force α_n:

If $\alpha_n \le \xi_R$

$$A = A'_s = \frac{R_b b h_0}{R_s} \cdot \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta} \tag{4.2}$$

If $\alpha_n > \xi_R$

$$A = A'_s = \frac{R_b b h_0}{R_s} \cdot \frac{\alpha_{m1} - \xi \cdot \left(1 - \frac{\xi}{2}\right)}{1 - \delta} \tag{4.3}$$

where

α_n – relative value of longitudinal force:

$$\alpha_n = \frac{N}{R_b b h_0} \tag{4.4}$$

ξ_R – limit relative height of compressed zone:

$$\xi_R = \frac{\omega}{1 + \frac{R_s}{\sigma_{sc,u}}\left(1 - \frac{\omega}{1.1}\right)} \tag{4.5}$$

$$\omega = 0.85 - 0.008 R_b \tag{4.6}$$

ξ – relative height of compressed zone. For the elements of concrete class B30 and lower:

$$\xi = \frac{\alpha_n (1 - \xi_R) + 2\alpha_s \xi_R}{1 - \xi_R + 2\alpha_s} \tag{4.7}$$

α_s in Formula 4.7:

$$\alpha_s = \frac{\alpha_{m1} - \alpha_n\left(1 - \frac{\alpha_n}{2}\right)}{1-\delta} \tag{4.8}$$

Relative moment α_{m1}:

$$\alpha_{m1} = \frac{Ne}{R_b b h_0^2} \tag{4.9}$$

$$\delta = \frac{a'}{h_0} \tag{4.10}$$

e – distance from the point of application of longitudinal force N to the resultant of the forces in the reinforcement:

$$e = e_0 + \frac{h_0 - a'}{2} \tag{4.11}$$

The strength design of concrete and reinforced members for compression longitudinal forces must account for accidental eccentricity e_a due to factors disregarded in the design. In any case, eccentricity ea shall be taken at least as 1/600 of the member length or distance between its sections and as 1/30 of section height.

It shall be allowed to disregard the state of strain in structures with the account taken of bending effect on member strength at slenderness ratio $l_0/i > 14$ by multiplying e_0 by coefficient η:

$$\eta = \frac{1}{1 - \frac{N}{N_{cr}}} \tag{4.12}$$

where

$$N_{cr} = \frac{1.6E_b bh}{\left(\frac{l_0}{h}\right)^2}\left(\frac{\frac{0.11}{0.1+\delta_e}+0.1}{3\varphi_l} + \mu\alpha\left(\frac{h_0 - a}{h}\right)^2\right) \tag{4.13}$$

According to Clause 3.6 of SNiP 2.03.01-84* (1995):

δ_e – factor taken equal to e_0/h, but not less than $\delta_{e,\min} = 0.5 - 0.01\frac{l_0}{h} - 0.01R_b$

φ_l – factor accounting for the effect of a long duration lead off member limit state bend equal to

$$\varphi_l = 1 + \beta\frac{M_l}{M} \geq 1 + \beta \tag{4.14}$$

M is the moment relative to the tensile or the least compressed sectional side due to the effect of continuous long and short duration loads;
M_l is the same due to the effect of continuous and long duration loads;
β is the factor assumed depending on concrete type. For heavy-weight concrete it equals 1.

The flowchart for the column design according to SNiP 2.03.01-84* is shown in Appendix 4. On the basis of the mentioned flowchart and the theory discussed in this chapter, the excel spreadsheet for the calculation of longitudinal reinforcement was made. Numerical calculation examples of longitudinal reinforcement for the exterior and interior columns are shown in Appendices 5a and 5b, correspondingly.

4.2 Design of longitudinal reinforcement (EN 1992)

A general method of designing columns is as follows:

1. Determine design life. Design working life category for industrial buildings is 4, Indicative design working life is 50 years (EN 1990, 2002, Table 2.1);
2. Assess actions on the column;
3. Determine which combinations of actions apply;
4. Assess durability requirements and determine concrete strength;
5. Check cover requirements for appropriate fire resistance period;
6. Determine cover for fire, durability and bond;
7. Analyze structure for critical combination moments and axial forces;

8. Check slenderness and determine design moments; for slender columns design moments should be calculated taking into account second order effects;
9. Determine area of reinforcement required;
10. Check spacing of bars and rims.

(The Concrete Center).

There are two main design methods for the calculation of column reinforcement area:

a) design using charts
b) design using iteration

The design using chart is simpler, but the design using iteration is more precise and appropriate for calculation automatization. The flowchart for the design of braced columns is shown in Appendix 6. The calculation spreadsheet for the design of column longitudinal reinforcement was created using this flowchart and the theory explained below. An example of the longitudinal reinforcement area determination for the exterior and interior columns can be found in Appendices 7a and 7b, correspondingly.

4.2.1 Structural analysis

First order effects – action effects calculated without consideration of the effect of structural deformations, but including geometric imperfections. Second order effects – additional action effects caused by structural deformations. (EN 1992-1-1 2004, cl. 5.8.1).

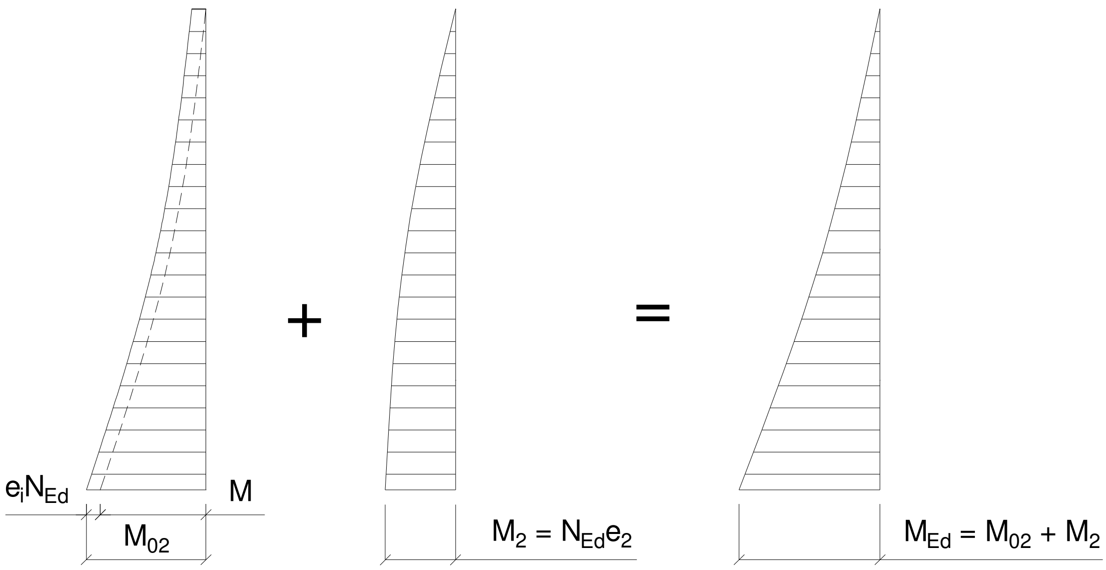
The type of analysis should be appropriate to the problem being considered. The following may be used: linear elastic analysis, linear elastic analysis with limited redistribution, plastic analysis and non-linear analysis. Linear elastic analysis may be carried out assuming cross sections are uncracked (i.e. concrete section properties), using linear stress-strain relationships and assuming mean values

of long-term elastic modulus.

For the design of columns the elastic moments from the frame action should be used without any redistribution. For slender columns a non-linear analysis may be carried out to determine the second order moments; alternatively the moment magnification method or nominal curvature method may be used. (Brooker 2006).

4.2.2 Design moments (EN 1992)

Determination of the design bending moment for cantilever columns is illustrated in Figure 4.2. The first order moment (M), including the effect of imperfections (e_i) is shown in Figure 4.2a. Additional second order moment M_2 caused by deflection of the column is shown in Figure 4.2b.



a. First order moment for cantilever columns

b. Additional second order moments for 'slender columns'

c. Total moment diagram for 'slender columns'

Figure 4.2 Design bending moment

Design moments calculation is described in Clause 5.8.8.2 of EN 1992-1-1 (2004). The design bending moment for the considering columns is illustrated in Figure 4.2c and defined as:

$$M_{Ed} = M_{02} + M_2 \tag{4.15}$$

where

M_{02} is the first order moment, including the effect of imperfections:

$$M_{02} = M_{bottom} + e_i N_{Ed} \tag{4.16}$$

e_i – eccentricity as the effect of imperfections $e_i = Max\left(\frac{l_0}{400}, \frac{h}{30}, 20\,mm\right)$

M_{bottom} – moment at the bottom of the column;

M_2 – nominal second order moment, $M_2 = N_{Ed} \cdot e_2$

where

N_{Ed} – the design axial load;

e_2 – deflection due to second order effects.

If column slenderness λ is less than limit slenderness λ_{lim} (see Chapter 4.2.4), then the column is non-slender and can be designed ignoring second order effects. Hence, the ultimate design moment $M_{Ed} = M_{02}$.

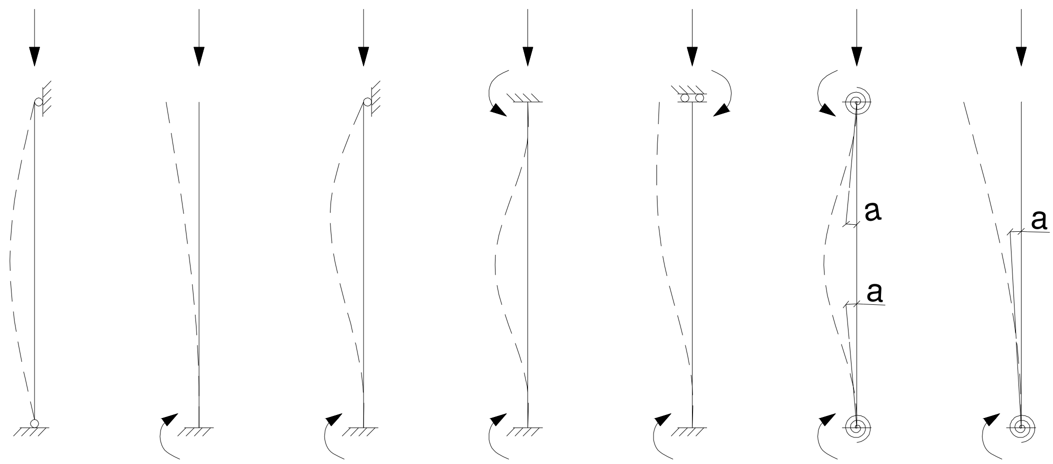
The calculation of the eccentricity e_2 is not simple and is likely to require some iteration to determine it. Also two simplified methods can be applied to calculate e_2:

a) method based on nominal stiffness (see Clause 5.8.7 of EN 1992-1-1 (2004))
b) method based on nominal curvature (see Clause 5.8.8 of EN 1992-1-1 (2004))

Method based on nominal curvature is applied in this work.

4.2.3 Effective length (EN 1992)

Figure 4.3 gives guidance on the effective length of the column. However, for most real structures Figures 4.3f and 4.3g only are applicable, and Eurocode 1992-1-1 provides two expressions to calculate the effective length for these situations. Expression (5.15) is for braced members and Expression (5.16) is for unbraced members.



a) $l_0 = l$ b) $l_0 = 2l$ c) $l_0 = 0.7l$ d) $l_0 = l/2$ e) $l_0 = l$ f) $l/2 < l_0 < l$ g) $l_0 > 2l$

Figure 4.3 Effective lengths for isolated members (EN 1992-1-1 2004, Figure 5.7)

In both expressions, the relative flexibilities at either end, k_1 and k_2, should be calculated. The expression for k given in the Eurocode involves calculating the rotation of the restraining members, which in practice requires the use of framework analysis software. (Brooker 2006). Approximate values for effective length factors were used in calculations for this work. For the considering cantilever columns the value was accepted equal 2 ($L_{ef} = 2 \cdot L$) according to Table 7.1 of Designers' Guide to EN 1992-1-1 and EN 1992-1-2 (2005).

4.2.4 Slenderness (EN 1992)

EN 1992 states that second order effects may be ignored if they are less than 10% of the first order effects. As an alternative, if the slenderness (λ) is less than the slenderness limit (λ_{lim}), then second order effects may be ignored.

Slenderness, $\lambda = l_0/i$ where i – radius of gyration, l_0 – clear height. Slenderness limit is defined in Clause 5.8.3.1 of EN 1992-1-1 (2004):

$$\lambda_{\lim} = \frac{20ABC}{\sqrt{n}} \leq \frac{15.4C}{\sqrt{n}} \tag{4.17}$$

where

$A = \dfrac{1}{1+0.2\varphi_{ef}}$ (if φ_{ef} is not known, A = 0.7 may be used)

$B = \sqrt{1+2\omega}$ (if ω, reinforcement ratio, is not known, B = 1.1 may be used)

$C = 1.7 - r_m$ (if r_m is not known, C = 0.7 may be used)

n – relative axial force:

$n = \dfrac{N_{Ed}}{A_c f_{cd}}$

r_m – moment ratio

$r_m = \dfrac{M_{01}}{M_{02}}$

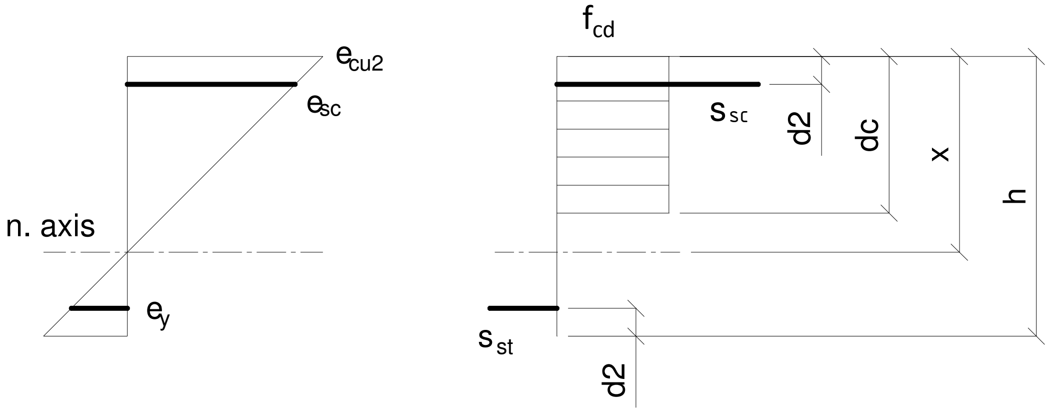
M_{01}, M_{02} – the first order end moments, $|M_{02}| \geq |M_{01}|$

In our case r_m = 0, C = 1.7

Of the three factors A, B and C, C will have the largest impact on λ_{lim} and is the simplest to calculate. An initial assessment of λ_{lim} can therefore be made by using the default values for A and B, but including a calculation for C. (Brooker 2006).

4.2.5 Column design resistance (EN 1992)

According to Brooker (2006) for practical purposes the rectangular stress block used for the design of beams may also be used for the design of columns (see Figure 4.4).



a) Strain diagram

b) Stress diagram

Figure 4.4 Stress block diagram for columns (Brooker 2006)

According to Brooker (2006), the maximum compressive strain for concrete classes up to and including C50/60, when the whole section is in pure compression, is 0.00175 (see Figure 4.6a). When the neutral axis falls outside the section (Figure 4.6b), the maximum allowable strain is assumed to lie between 0.00175 and 0.0035, and may be obtained by drawing a line from the point of zero strain through the ‘hinge point’ of 0.00175 strain at mid-depth of the section. When the neutral axis lies within the section depth then the maximum compressive strain is 0.0035 (see Figure 4.6c).

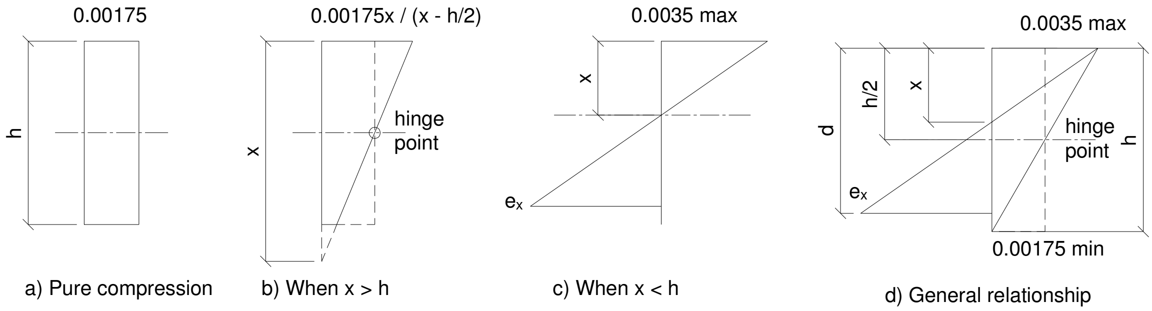


Figure 4.5 Strain diagrams for columns (Brooker 2006)

The general relationship is shown in Figure 4.6d. For concrete classes above C50/60 the principles are the same but the maximum strain values vary.

Two expressions can be derived for the area of steel required, (based on a rectangular stress block, see Figure 4.4) one for the axial loads and the other for the moments:

$$A_{sN}/2 = \frac{N_{Ed} - f_{cd}bd_c}{(\sigma_{sc} - \sigma_{st})} \tag{4.18}$$

where

A_{sN} – area of reinforcement required to resist axial load

N_{Ed} – axial load

f_{cd} – design value of concrete compressive strength

σ_{sc} (σ_{st}) – stress in compression (and tension) reinforcement

b – breadth of section

γ_c – partial factor for concrete (1.5)

d_c – effective depth of concrete in compression; $d_c = \lambda x \leq h$

λ = 0.8 for concrete classes up to C50/60

x – depth to neutral axis

h – height of section

$$A_{sM}/2 = \frac{M - f_{cd}bd_c\left(\frac{h}{2} - \frac{d_c}{2}\right)}{\left(\frac{h}{2} - d_2\right)(\sigma_{sc} + \sigma_{st})} \tag{4.19}$$

where

A_{sM} – area of reinforcement required to resist moment

Realistically, these can only be solved iteratively and therefore either computer software or column design charts may be used. The design charts can be found in Figure 5.8 of Designers' Guide to EN 1992-1-1 and EN 1992-1-2 (2005).

4.2.6 Creep (EN 1992)

Depending on the assumptions used in the design, it may be necessary to determine the effective creep ratio φ_{ef} (see EN 1992-1-1 2004, cl. 3.1.4, 5.8.4). A nomogram is provided in Figure 3.1 of EN 1992-1-1 (2004) for which the cement strength class is required; however, at the design stage it is often not certain which class applies. Generally, Class R should be assumed.

5 DESIGN OF FOUNDATIONS ACCORDING TO SNIP 2.02.01-83

For the designing of a foundation the next initial data is required: geological cross-section of the base; soils characteristics; structural solution of the over- and underground parts of the building, leaning on the foundation; the loads acting on the foundation (permanent and temporary). The main algorithm of foundation design according to SNiP 2.02.01-83 is described in Figure 5.1:

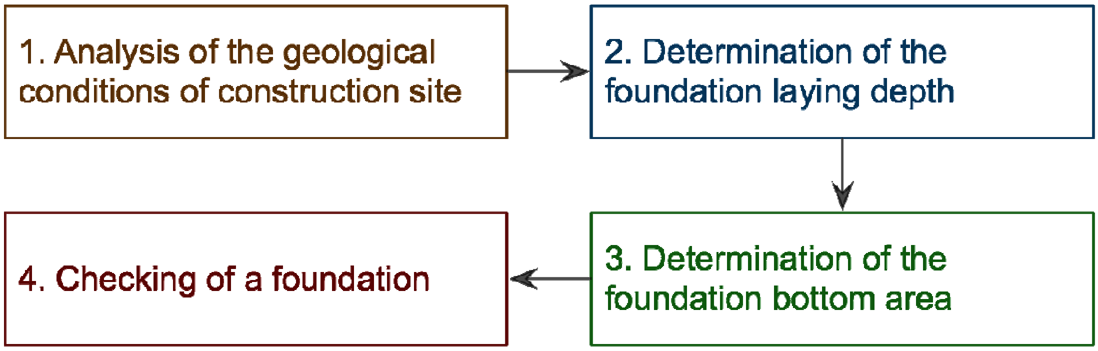


Figure 5.1 The general process of foundation design (Shvetsov 1991, Figure 4.9a)

Analysis of the geological conditions of a construction site includes the calculation of soil characteristics, the main of which is soil resistance. Foundation strength checking, determination of its laying depth and the bottom area are described below in this chapter. At the foundation checking stage, calculations according to the first and the second groups of limit states depend on foundation type. The numerical example of foundation dimensioning is shown in Appendix 8. The foundation under the interior column was chosen for the example.

5.1 Foundations laying depth (SNiP 2.02.01-83)

Calculation of foundations laying depth is described in Clauses 2.25–2.33 of SNiP 2.02.01-83 (2007). The selection of foundation laying depth should be governed by many parameters most important of which are:

- purpose and design details of the engineering facility, loads and actions applied to its foundations;
- foundation depths of the adjacent facilities and laying depths of utility lines;
- engineering and geological conditions of the construction site (physical and mechanical properties of soils, soil stratification details, available soils with landslide properties, wind erosion pockets, karst cavities, etc.);
- groundwater conditions of the site and potential variations thereof during facility construction and operation;
- seasonal soil freezing depth d_f.

The design soil freezing depth d_f is calculated from the formula:

$$d_f = k_h d_{fn} \tag{5.1}$$

where

d_{fn} – the standard freezing depth assumed equal to the mean annual maximum seasonal freezing depth (based on observation data through the period of not less than 10 years) measured in an open snow free site at the groundwater surface occurred below the freezing soil depth; it can be determined using the map of standard freezing depths from SNiP II-A.6-72;

k_h – the coefficient accounting for the heat flow regime of the facility, assumed per Table 1 of SNiP 2.02.01-83 (2007) for external foundations of heated facilities; k_h = 1.1 for external and internal foundations of unheated facilities, excluding areas with negative mean annual temperatures.

5.2 Foundation bottom area (SNiP 2.02.01-83)

Preliminary dimensioning of foundation is described in this chapter according to Shvetsov (1991). The required bottom area of foundation can be calculated using the formula:

$$A = \frac{F_v}{R_0 - \gamma_{mt} d} \tag{5.2}$$

where

$F_υ$ – the design load applied to the top of a foundation;

R_0 – the design soil resistance; it can be calculated by Equation 7 or Appendix 3 of SNiP 2.02.01-83 (2007);

$γ_{mt}$ – the average specific weight of the soil and concrete above the bottom of foundation; for the buildings without basement it usually equals 18–20 kN/m^3, for the buildings with basement — 16–19 kN/m^3; also, if value of live load is high, it should be taken into account;

d – the foundation laying depth.

Plan sizes of foundation (b, l) can be calculated according to the required bottom area A:

$$b = \sqrt{\frac{A}{k}} \tag{5.3}$$

where

b – width of foundation bottom;

k – correlation of foundation bottom dimensions, $k = \frac{l}{b}$;

l – length of foundation bottom, $l = \frac{A}{b}$.

For square in plan foundation dimensions of the bottom can be determined using the formula:

$$l = b = \sqrt{A} \tag{5.4}$$

Then the design soil resistance at the bottom of the foundation should be calculated, taking into account the determined plan dimensions of foundation. It can be calculated using Equation 7 of SNiP 2.02.01-83 (2007).

5.3 Checking of foundation (SNiP 2.02.01-83)

For centrally and eccentrically loaded foundations the next checks should be made:

- the pressure at the bottom of the foundation;
- in the presence of the soil which is less strength than overlying soils, checking of its strength is required;
- settlement of the base (s) should not exceed the limit settlement (s_u);
- foundation inclination (i) should not exceed the limit value (i_u);
- shear displacement of foundation;
- check the stability of the foundations working on pulling;
- strength checks of foundation as for a reinforced concrete element.

(Shvetsov 1991, p. 108).

The off-centrally loaded foundation is shown in Figure 5.2. For the building considered in this work, the connection between the columns and foundations were accepted rigid, therefore bending moment (M) is transmitted from the column to the foundation.

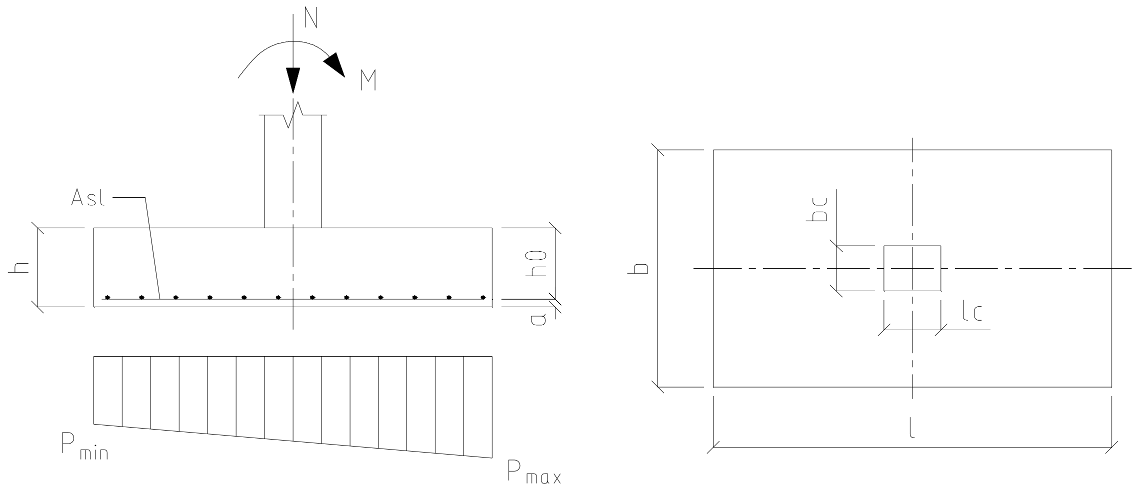


Figure 5.2 Pressure under the off-centrally loaded foundation

The main dimensions of the foundation slab are width (b), length (l) and height (h). The pressure under the foundation bottom is not constant because of the bending moment (M). According to Shvetsov (1991, p. 105), checking of the pressure at the bottom of the foundation include:

$$p_a \leq R \tag{5.5}$$

$$p_{\max} \leq 1.2R \tag{5.6}$$

$$\frac{p_{\min}}{p_{\max}} \geq 0.25 \tag{5.7}$$

where

p_a, p_{max}, p_{min} – the mean, maximum and minimal pressures under foundation bottom, correspondingly, $p_{\substack{\max \\ \min}} = \frac{N}{A} \pm \frac{M}{W}$

N – total vertical load including the weight of the foundation and soil above it;

M – bending moment acting on the foundation;

W – resisting moment of the foundation bottom in plane of moment action;

R – the design soil resistance.

One of the most important strength checks of the foundation as a reinforced concrete element is a punching check. In the calculation of punching stress under the

columns from the top plate in accordance with SNiP 2.03.01-83, it is presumed that under central loading, foundation punching takes place along the sides of the pyramid, whose sides are inclined at angle of 45° to the horizontal, and the area of action of the punching force serves as the smaller base (see Figure 5.3).

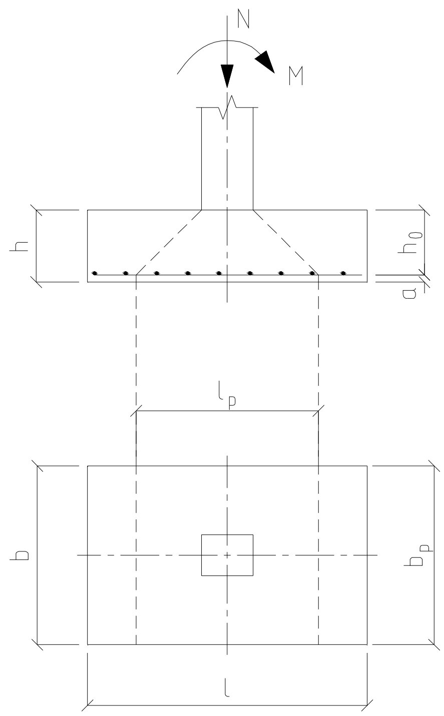


Figure 5.3 Scheme for the punching of foundation

The calculation of the foundation punching should be carried out from the next condition according to Clause 2.8 of Guide to SNiP 2.03.01-84 and 2.02.01-83 (1989):

$$F \leq \alpha R_{bt} h_0 u_m \tag{5.8}$$

where

F – punching force;

α – coefficient equal to 1 for heavy-weight concrete;

u_m – the mean arithmetic value of perimeters of the upper and lower bases of the pyramid formed under punching stress in the limits of effective height h_0.

5.4 Calculation of reinforcement area (SNiP 2.02.01-83)

The reinforcement area A_s for both directions should be determined from bending analysis of console part of foundation slab from action of ground pressure. The reinforcement area should be calculated by the formula, specified in Clause 2.31 of Guide to SNiP 2.03.01-84 and 2.02.01-83 (1989):

$$A_s = \frac{M_i}{0.9 h_i R_s} \tag{5.9}$$

where

M_i – bending moment in the considering section of console part (at the exterior of the column or foundation jump);

h_i – effective depth of the considering section;

R_s – the design reinforcement strength.

Bending moments (M_i) in the design sections are calculated according to the pressure under foundation bottom (p), calculated using the design values of total vertical force (N) and bending moment (M) acting in the plane of the moment M_i.

6 THE DESIGN OF STEEL TRUSS MEMBERS

Two types of trusses were used for the building: primary and secondary. The secondary trusses rest upon the primary trusses. Therefore the members of the primary truss are more stressed. This can be seen from results of static analysis (see Appendix 9). The roof trusses are assumed to be simply supported as in Figure 6.1, i.e. support joints are pinned.

Designations of truss elements are shown in Figure 6.1. The secondary trusses for the building were accepted double pitched with slope 2° (i ≈ 3 %) and span 24 m (as in Figure 6.1). The primary trusses have parallel chords, the span is 12 m.

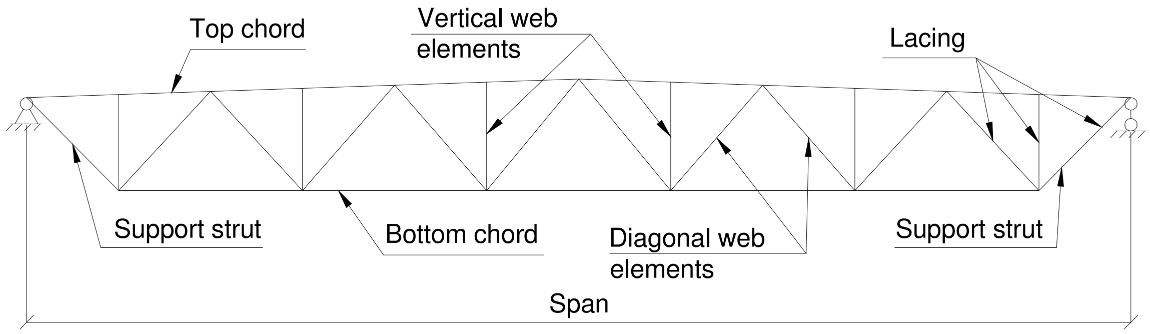


Figure 6.1 Designations of truss members

Lacing of the both trusses is triangular. The secondary truss has also vertical web members. The truss is covered with profile sheeting without purlins. As a profile for the trusses, rectangular hollow section was chosen. Information about the profiles was taken from GOST 30245-94.

6.1 Static analysis of trusses

The traditional analysis of a truss assumes that all loads are applied in the joints and that all joints in the truss are pinned. Even though this is generally not the case, since the upper and lower chords are normally continuous and the web members are often welded to the chords, it is still a common and acceptable procedure to determine the axial forces in the members. In a situation when the dimensions of the upper chord is very large and the overall depth of the truss is small the moments due to continuous upper chord has to be considered. However, this is rarely the case for roof trusses in an industrial building.

The analysis of a truss is rather simple using the assumptions in the previous paragraph but there are some special cases that complicate things:

- When for example the roof sheeting is attached directly to the truss or if purlins are used and they are not placed only in the truss joints, bending moment will be introduced into the upper chord.
- Due to eccentricity in the joints between chord and web members, moments might be introduced that need to be taken into account in the design.
- For low pitch roofs, reversed loadings due to the wind load can cause compression in the lower chord and it has to be designed for lateral buckling.

(Access Steel).

The design procedure for truss members depends on the stresses taking place into a member. Distribution of longitudinal forces in a truss is illustrated in Figure 6.2. Top chord and vertical web elements are usually compressed and the bottom chord is tensioned. For lacing the highest stresses are in edge elements and the lowest are in middle elements.

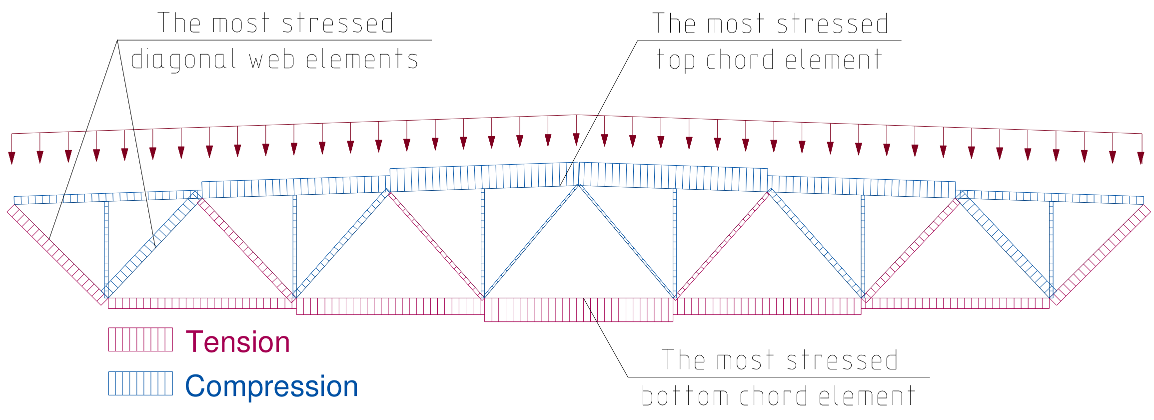


Figure 6.2 Distribution of axial forces in a truss

To calculate member forces of statically determinate truss, the method of joints or/and method of sections can be applied. Calculation of member forces for the primary and secondary trusses is shown in Appendix 9.

6.2 Design of truss members according to SNiP II-23-81*

For steel trusses, from four to six different types of profiles are usually used for all truss members. Required profile areas for all truss members are calculated to preliminary find the necessary range of profiles. Chords of trusses with span up to 30 m are usually taken of uniform cross-section. (Belenya 1986).

Numerical example of truss members design according to SNiP II-23-81* in shown in Appendix 10. The secondary truss was chosen for the example. In accordance with the theory described below, Excel spreadsheets for steel trusses dimensioning were made.

6.2.1 Dimensioning of the compressed truss members (SNiP II-23-81*)

Procedure for dimensioning of compressed truss members is based on Clause 5.2 of SNiP II-23-81* (1988). As the first step the required cross-section area (A) should be calculated using the formula:

$$A = \frac{N}{\varphi R_y \gamma_c} \tag{6.1}$$

where

γ_c – service condition factor; it can be determined using Table 6* of SNiP II-23-81* (1988)

φ – buckling ratio which is a function of slenderness ratio λ:

$$\lambda = \frac{l_0}{i} \tag{6.2}$$

l_0 – effective length

i – gyration radius; $i = \sqrt{\frac{I}{A}}$

Buckling ratio (φ) can be calculated using assumed slenderness ratio. For chords λ can be taken λ = 60–80, for lacing — λ = 100–120. Also, required gyration radiuses (i) can be found by the equations:

$$i_x = \frac{l_{0x}}{\lambda}, \quad i_y = \frac{l_{0y}}{\lambda} \qquad (6.3), (6.4)$$

In accordance with required cross-section area and gyration radiuses appropriate profile for truss members can be chosen. Accepted profile should be checked using the inequation:

$$\frac{N}{\varphi A} \le R_y \gamma_c \qquad (6.5)$$

Buckling ratio (φ) in Equation 6.5 should be calculated using the corrected slenderness ratio (λ).

6.2.2 Dimensioning of the tensioned truss members (SNiP II-23-81*)

The dimensioning of the tensioned truss members is based on Clause 5.1 of SNiP II-23-81* (1988). Required cross-section net area (A_n) of the tensioned truss member made of steel with ratio $R_u/\gamma_u < R_y$ should be determined from the following formula:

$$A_n = \frac{N}{R_y \gamma_c} \qquad (6.6)$$

where

R_u – design steel strength of steel to tension, compression and bending according to ultimate strength

γ_u – safety factor in calculations for ultimate strength; $\gamma_u = 1.3$

In accordance with required net cross-section area appropriate profile for truss members can be chosen. Accepted profile should be checked using the inequation:

$$\frac{N}{A_n} \le R_y \gamma_c \qquad (6.7)$$

Cross-section net area (A_n) in Equation 6.7 should be calculated taking into account weaking of the cross-section due to bolt holes.

6.2.3 Dimensioning of the truss members subjected to a longitudinal force and a bending moment (SNiP II-23-81*)

As said before, uniformly loaded top chords are subjected to a longitudinal force and a bending moment. The bending moment is accepted equal to the maximum moment within the limits of the middle third of the length of the chord panel. Buckling calculation for the eccentrically compressed top chord should be made in plane and from plane of moment action. (SNiP II-23-81* 1988).

The dimensioning procedure is described according to Belenya (1986). The required cross-section area can be calculated from the formula:

$$A = \frac{N}{\varphi_e R_y \gamma_c} \tag{6.8}$$

Ratio of reduction in design strengths at eccentric compression (φ_e) is a function of the apparent slenderness $\bar{\lambda}_x = \frac{l_{0x}}{i_x}\sqrt{\frac{R}{E}}$ and effective relative eccentricity $m_{ef} = \eta \frac{M_x A \cdot z}{N \cdot I_x}$ (6.9), where z is the distance from the centroid of section to the compressed edge of section; η – shape factor.

Preliminary dimensioning can be made as for centrally compressed element. Having been given the slenderness ratio (λ), gyration radius can be calculated as $i_x = \frac{l_0}{\lambda_x}$, required cross-section height as $h = \frac{i_x}{\alpha_1}$ and cross-section core distance $\rho_x = \frac{i_x^2}{z}$. For the horizontally symmetric cross-sections z = h/2. For the chosen profile type, shape factor (η) can be determined from Table 73 of SNiP II-23-81* (1988).

Using the calculated η and ρ_x, effective relative eccentricity m_{ef} can be determined from Equation 6.9. Then the required cross-section area (A) can be calculated from Formula 6.8. Profile of element can be chosen by using the calculated cross-section area (A) and height (h). If a profile cannot be chosen, slenderness ratio (λ) should be changed and the required cross-section area (A) should be recalculated.

Then the truss member should be checked in plane of moment action using the inequiation:

$$\frac{N}{\varphi_e A} \leq R_y \gamma_c \tag{6.10}$$

Ratio φ_e in Equation 6.10 is calculated using the characteristics $\overline{\lambda}$ and m_{ef} of the chosen profile.

Buckling check from the plane of moment action is made if $I_x > I_y$ from the inequation:

$$\frac{N}{c \cdot \varphi_y \cdot A} \leq R_y \gamma_c \quad (6.11)$$

Buckling ratio φ_y is calculated using the slenderness $\lambda_y = \frac{l_0}{i_y}$. Coefficient for strength analysis with regard to development of plastic deformations due to bending (c) is determined in accordance with the requirements of Clause 5.31 of SNiP II-23-81* (1988).

6.2.4 Dimensioning of the truss members according to the limit slenderness (SNiP II-23-81*)

Some of the truss members have insignificant stresses. Such members typically include vertical web elements, diagonal web elements in the middle of the truss, bracing elements, etc. Dimensioning of these members is made according to the limit slenderness.

Limit slendernesses of truss members can be determined from Figure 6.4 or Tables 19*, 20* of SNiP II-23-81* (1988). The values shown in Figure 6.4 are for trusses made of steel.

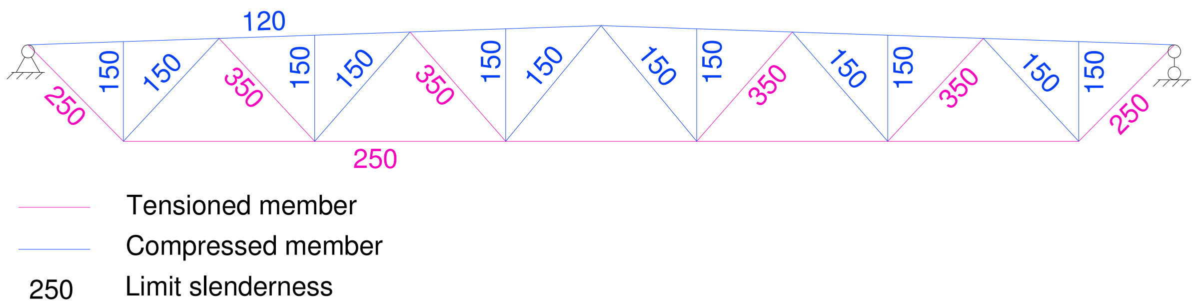


Figure 6.3 Limit slendernesses of the truss members (Belenya 1986)

According to Belenya (1986), knowing the values of effective length (l_0) and limit slenderness (λ_{lim}), the required gyration radius of cross-section (i) can be calculated from the equation:

$$i = \frac{l_0}{\lambda_{\lim}} \tag{6.12}$$

Then, using the gyration radius, a profile with the lowest cross-section area can be chosen.

6.3 Design of truss members according to EN 1993

A numerical example of truss members design according to EN 1993 is shown in Appendix 11. The design of the secondary truss was chosen for the example. The checking of centrally tensioned and compressed and eccentrically compressed members is described in the example. In accordance with the theory described below, excel spreadsheets for steel trusses dimensioning were made.

6.3.1 Design of top chord (EN 1993)

The theory about the static analysis of top chord is based on the information from Access Steel. If all loads are introduced in the joints, only axial forces should be considered. When the upper chord is compressed, both buckling in and out of plane have to be considered if not constrained in the lateral direction. According to Annex BB of EN 1993-1-1 (2005), the buckling length is equal to the distance between the truss joints for buckling in the plane of the truss. The buckling length out of plane is the distance between the purlins. If the roof sheeting is a deck placed directly on the roof truss, i.e. without purlins, the buckling out of plane of the top chord is restrained. This requires that the roof sheeting is stiff enough to work as a diaphragm and is in structural class I or II according to EN 1993-1-3.

If the roof sheeting is directly attached to the roof truss, the upper chords will be subjected not only to axial force but also to bending moment. In Figure 6.4 an example is shown with non-nodal loads applied to the roof truss. In such case the top chord should be treated as a continuous beam and the bending moment should be taken into account in the member design.

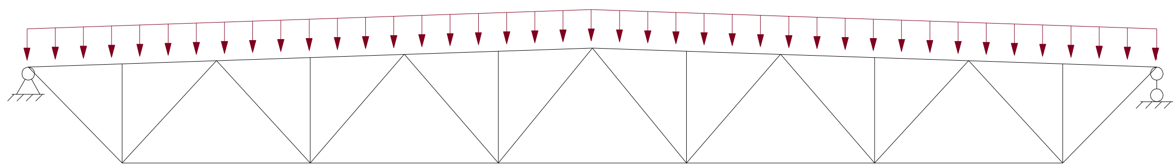


Figure 6.4 Example of a roof truss with roof sheeting attached directly on the truss, introducing non-nodal loads on the truss

Consider the roof truss in Figure 6.4 subjected to a distributed load. For simplicity, only an interior section is cut out from the truss and treated here, see Figure 6.5.

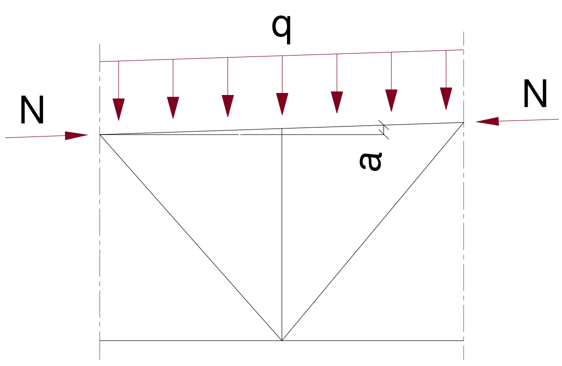


Figure 6.5 Section of the truss (Access Steel)

The upper chord is continuous and can be treated as a beam clamped at both ends, see Figure 6.6.

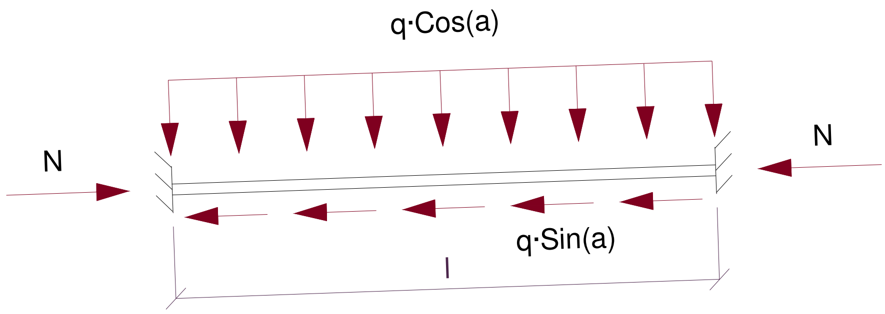


Figure 6.6 Upper chord treated as a beam clamped at both ends (Access Steel)

With the simplification shown in Figure 6.6 the moment distribution can be obtained as shown in Figure 6.7. The value of Cos(α) may be neglected for small angles α.

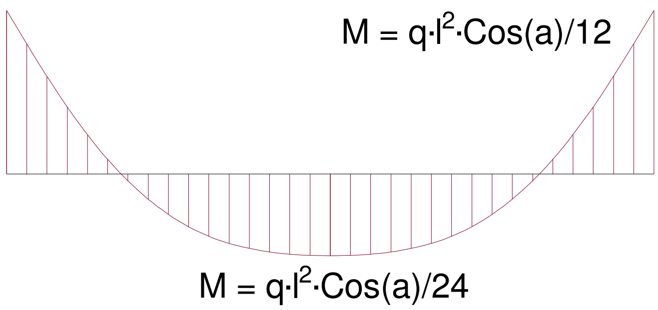


Figure 6.7 Moment distribution in the upper chord (Access Steel)

The upper chord should be verified for both bending moment and axial force. In this case, where the roof sheeting is attached to the upper chord acting as a lateral support for out of plane deformation, the member needs to be checked for flexural buckling in the plane of the truss according to Clause 6.3 of EN 1993-1-1 (2005). The roof sheeting is assumed to be in structural class 1 of 2 according to EN 1993-1-3. If the roof sheeting is in structural class 3 it cannot act as lateral support and the upper chord has to be checked for lateral buckling as well.

Members, subjected to axial compression with bending, should be verified as follows according to Clause 6.3.3 of EN 1993-1-1 (2005):

$$\frac{N_{Ed}}{\chi_z \cdot N_{b,Rd}} + k_{zz} \cdot \frac{M_{Ed} + \Delta M_{z,Ed}}{M_{z,Rd}} \leq 1 \tag{6.13}$$

where

N_{Ed} – the design values of the compression force;

M_{Ed} – the maximum moment along the member;

$\Delta M_{z,Ed}$ – the moment due to the shift of the centroidal axis (see Table 6.7 of EN 1993-1-1 (2005));

χ_z – the reduction factor due to flexural buckling from Clause 6.3.1 of EN 1993-1-1 (2005) (see also Chapter 6.3.3);

k_{zz} – interaction factor; this value may be obtained from Annex A (alternative method 1) or from Annex B (alternative method 2) of EN 1993-1-1 (2005);

$N_{b,Rd}$ – the design buckling resistance (see Chapter 6.3.3);

$M_{z,Rd}$ – the design resistance for bending:

$$M_{z,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} \tag{6.14}$$

where

W_{pl} – resisting moment of section;

f_y – yield strength (see Table 3.1 of EN 1993-1-1 (2005));

γ_{M0} – partial factor for resistance of cross-sections (see Clause 6.1 of EN 1993-1-1 (2005)).

6.3.2 Design of bottom chord (EN 1993)

If the load case including dead load and imposed load is considered, the bottom chord is normally in tension and only the tension resistance needs to be checked for.

Centrally tensioned elements should be checked according to Clause 6.2.3 of EN 1993-1-1 (2005) using the inequation:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1 \qquad (6.15)$$

where

N_{Ed} – the design value of the tension force;

$N_{t,Rd}$ – the design tension resistance:

$$N_{t,Rd} = \frac{f_y \cdot A}{\gamma_{M0}} \qquad (6.16)$$

f_y – yield strength (see Table 3.1 of EN 1993-1-1 (2005));

A – net cross-section area;

γ_{M0} – partial factor for resistance of cross-sections whatever the class is (see Clause 6.1 of EN 1993-1-1 (2005)).

Wind loads causing external suction or sometimes also internal pressure in buildings with low pitch roofs have to be carefully considered when designing a roof truss. The parts of the truss that act as ties under dead and imposed load can be severely overstressed when subjected to compression, i.e. reversed loading is important to take into account in the design of a roof truss. When the bottom chord is loaded in compression, lateral buckling of the bottom chord can take place. It is often possible to confirm the strength of the bottom chord without bracing by taking into account the stiffness of the connected parts. (Access Steel).

6.3.3 Design of web members (EN 1993)

Web members in a truss are usually designed to resist only axial forces unless eccentricity in the joints is present. The compressed members should be checked for in and out of plane buckling. The resistance to in-plane buckling should be calculated with a buckling length, L_{cr}, equal to 90% of the system length, i.e. 0,9L. For out of plane buckling the buckling length, L_{cr}, should be taken as the system length, L. This is further described in Annex BB of EN 1993-1-1.

Compression member should be verified against buckling according to Clause 6.3.1.1 of EN 1993-1-1 (2005) as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \qquad (6.17)$$

where

N_{Ed} – the design value of the compression force;

$N_{b,Rd}$ – the design buckling resistance of the compression member:

$N_{b,Rd} = \dfrac{\chi \cdot A \cdot f_y}{\gamma_{M1}}$ for Class 1, 2 and 3 cross-sections (6.18)

The reduction factor χ is calculated as:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \overline{\lambda}^2}} \leq 1 \qquad (6.19)$$

where

$\Phi = 0.5 \cdot \left(1 + \alpha \cdot \left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right)$

α – imperfection factor corresponding to the appropriate buckling curve;

$\overline{\lambda}$ – non-dimensional slenderness:

$$\overline{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} \qquad (6.20)$$

N_{cr} – elastic critical force for the relevant buckling mode based on the gross cross-sectional properties (see Clause 6.4.1 of EN 1993-1-1 (2005)).

Tensioned web elements are checked using the equations from Chapter 6.3.2.

7 COMPARISON OF THE DESIGN RESULTS ACCORDING TO SNIP AND EUROCODES

In this comparison the design results for concrete columns (exterior and interior) and steel trusses (primary and secondary) are considered. Calculations can be found in Appendices. Calculation spreadsheets were used to consider different initial data for the design for the comparison. Designing procedures according the both norms have similarities. The differences in design results are mostly caused by values of loads (normative and design), safety factors, effective length ratios and others.

7.1 Longitudinal reinforcement of concrete columns

The results of the exterior and interior columns design according to SNiP and Eurocode are listed in Table 7.1. The dimensions of the exterior column section are 0.4x0.5 m, interior column – 0.5x0.5 m. Stress patterns for the columns are shown in Figure 4.1. The length of exterior column is 10 m, the interior column length is 9 m.

Table 7.1 Results of the columns design according to SNiP and Eurocode

Column	Reinforcement area according to SNiP (L_{ef} = 1.2·L), cm^2	Reinforcement area according to SNiP (L_{ef} = 2·L), cm^2	Reinforcement area according to Eurocode (L_{ef} = 2·L, SNiP loads), cm^2	Reinforcement area according to Eurocode (L_{ef} = 2·L, Eurocode loads), cm^2
Exterior column	9	9.2	12.5	13.8
Interior column	9	26.5	28.1	39.1

Information from Table 7.1 is also shown in Chart 7.1 for visualization.

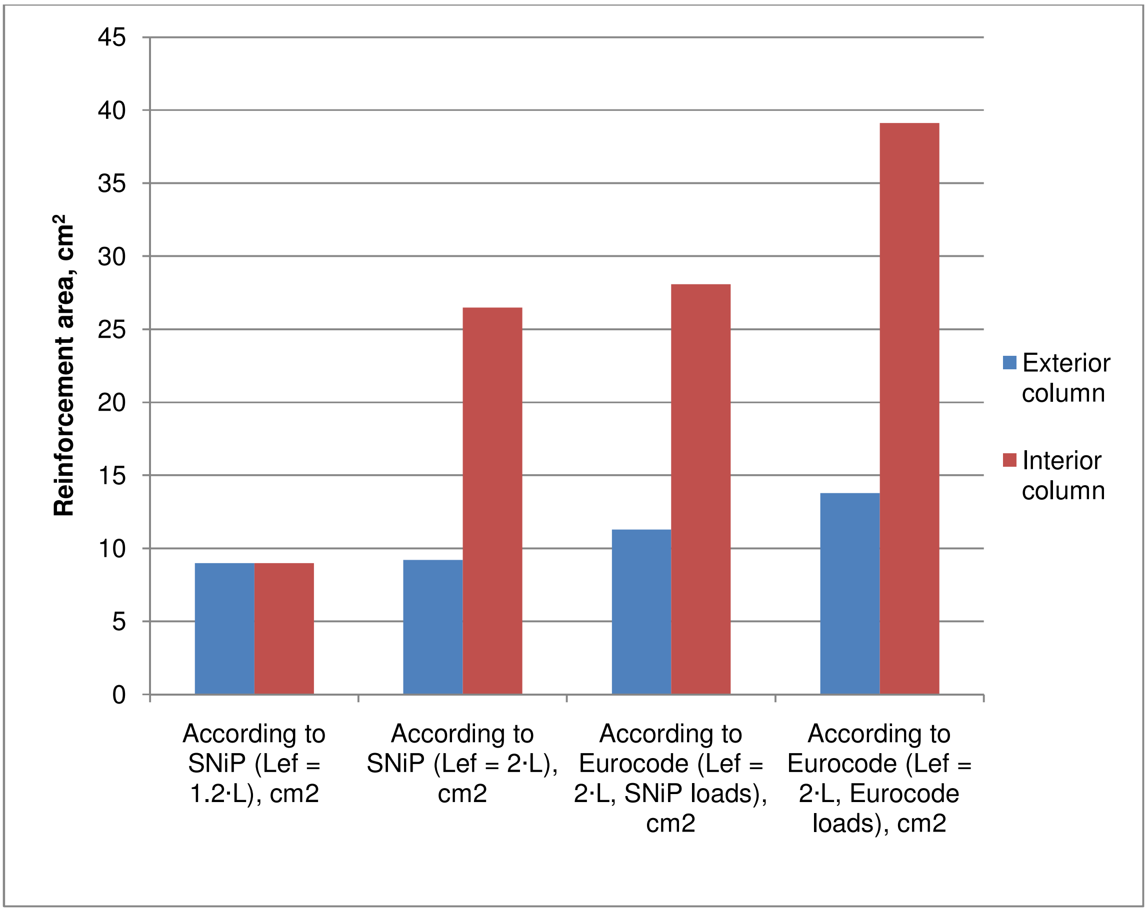


Chart 7.1 Results of the columns design according to SNiP and Eurocode

According to SNiP, for the building columns only minimal reinforcing is required. Reinforcement area according to Eurocode is much higher (especially for the interior columns). This difference can be explained by the column effective length calculation according to the both norms. Pursuant to Table 23 of Guide to SNiP 2.03.01-84* (1995), the effective length ratio for the constant section columns of single-storey multispan buildings without cranes is 1.2. Therefore, the effective length 1.2·L was used in calculations according to SNiP. According to Eurocode, the effective length should be equal 2·L in considering case. If the effective length 2·L is taken for the calculation according to SNiP, then the results are quite close,

but the reinforcement area according to SNiP is little less in this case. Loads estimated according to Eurocode are much higher than in SNiP, therefore, reinforcement area calculated pursuant to Eurocode is noticeably greater.

7.2 Truss members

Designations of the secondary truss elements are shown in Figure 7.2. The designations, used in the design results, are listed below.

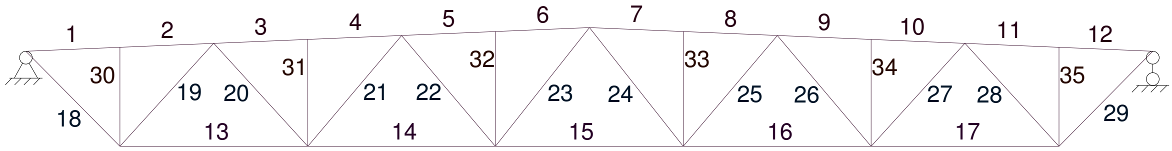


Figure 7.1 Designations of the secondary truss elements

In Table 7.2 the comparison of dimensioning results for the secondary truss is shown. Calculation spreadsheets were used for the dimensioning of truss members according to the both norms.

Table 7.2 Comparison of the profiles according to SNiP and Eurocode (for the secondary truss)

Truss element	Truss part name	Profile according to SNiP	Profile according to Eurocode (loads acc. to SNiP)	Profile according to Eurocode (loads acc. to Eurocode)
1	Top chord	100x3	90x3	100x3.5
2		100x3	90x3	100x3.5
3		120x4	120x4	150x4
4		120x4	120x4	150x4
5		120x4.5	120x4.5	160x4.5
6		120x4.5	120x4.5	160x4.5
13	Bottom chord	70x4	60x4.5	120x3
14		70x6.5	80x5	80x8
15		70x7	100x4	120x5
18	Diagonal web elements	60x4	60x3.5	60x5.5
19		90x3	90x3	100x3.5
20		60x3	60x3	60x3
21		60x3.5	70x3	70x3.5
22		60x3	60x3	60x3
23		60x3	60x3	60x3
30	Vertical web elements	60x3	60x3	60x3
31		60x3	60x3	60x3
32		60x3	60x3	60x3

As it can be seen from Table 7.1, the difference in profiles is caused by the loads taken into account. Loads according to Eurocode are much higher mainly because of the snow loads (see Chapter 2.5). The total weight of the secondary truss designed according to SNiP is 8 kN, according to Eurocode (loads pursuant to EN) – 8.3 kN. The relative difference is 3.1 %.

The designations of the primary truss elements are shown in Figure 7.3.

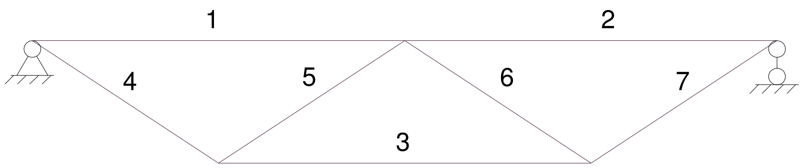


Figure 7.2 Designations of the primary truss elements

In Figure 7.3 the dimensioning results according to SNiP and Eurocodes for the primary truss are listed. Elements of this truss are more stressed in comparison with the secondary truss. Therefore, profiles of the primary truss members are bigger.

Table 7.3 Comparison of the profiles according to SNiP and Eurocode (for primary truss)

Truss element	Truss part name	Profile according to SNiP	Profile according to Eurocode (loads acc. to SNiP)	Profile according to Eurocode (loads acc. to Eurocode)
1	Top chord	160x4	150x4	160x5
2		160x4	150x4	160x5
3	Bottom chord	140x4	80x7.5	140x5.5
4	Diagonal web elements	70x5	80x4	80x6
5		120x3.5	120x3.5	140x4
6		120x3.5	120x3.5	140x4
7		70x5	80x4	80x6

As well as for the secondary truss, the difference in profiles is caused by the loads taken into account. The total weight of the primary truss designed according to SNiP is 4.9 kN, according to Eurocode (loads pursuant to EN) – 6.3 kN. The relative difference is 29.5 %.

8 SUMMARY

As a result of this work the calculation spreadsheets for preliminary design of industrial buildings were made. These spreadsheets include static analysis of the frame, loads estimation, reinforcement area calculation for concrete columns, dimensioning and static analysis of steel trusses. Also the work comprises information about the design of industrial buildings according to Russian and European norms and comprehensive numerical examples of calculations.

The comparison of SNiP and Eurocodes showed some differences between them. Snow and wind loads in Eurocodes are much higher than in SNiP and this fact explains differences in design results. In addition, safety factors in Eurocodes are also higher; some of them are significantly greater.

Another distinction concerns the column effective length calculation. For the building taken as an example, the columns effective length according to Eurocodes is much greater. This fact explains a big difference in the calculation results of the longitudinal reinforcement area. Also, the results differ because the loads estimated according to Eurocode are greater than in SNiP. If the same effective length and loads are used in calculations, the results according to the both norms are quite near.

For dimensioning of steel truss members the dissimilarity is caused by the loads, mainly, by the difference in snow load. When the same loads are taken into account according to the both norms, the results are close.

FIGURES

CHARTS

TABLES

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Loads estimation according to SNiP 2.01.07-85

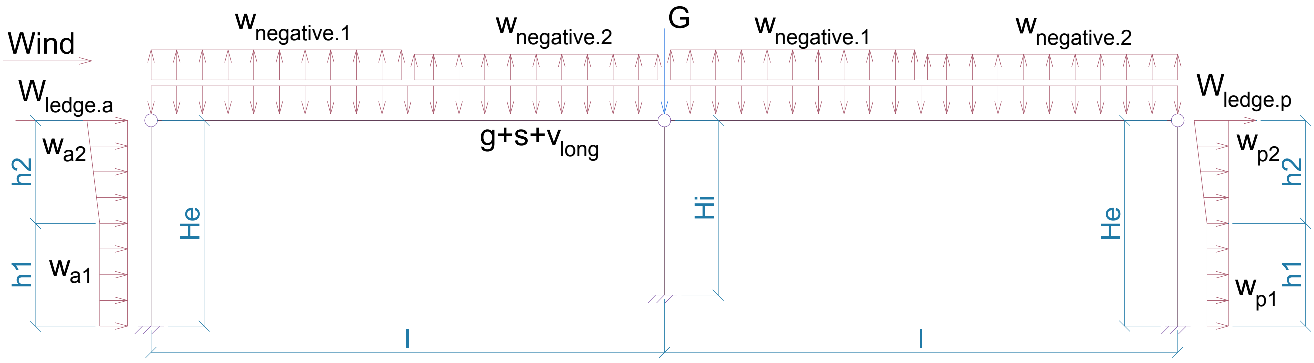


Figure 1. Loads estimation

Building dimensions

Span $l := 24m$

Exterior columns height $H_e := 10m$

Internal columns height $H_i := 9m$

Exterior columns spacing $B := 6m$

Height of the section along which the wind load varies linearly

$h_1 := 5m$

$h_2 := 5m$

Loads estimatioin

Permanent load

Normative self-weight of the steel truss

$$g_{n.truss} := 1.3\frac{kN}{m}$$

Normative self-weight of coating

$$g_{n.coating} := 0.098\cdot\frac{kN}{m^2}$$ (profile sheeting)

Normative self-weight of thermal insulation

$$g_{n.insulation} := 0.2\frac{kN}{m^2}$$

Safety factor for weight of steel structures

$\gamma_{f.steel} := 1.05$ SNiP 2.01.07-85* Table 1

Safety factor for weight of insulation

$\gamma_{f.insulation} := 1.3$ SNiP 2.01.07-85* Table 1

Design permanent load

$$g := g_{n.truss}\cdot\gamma_{f.steel} + g_{n.coating}\cdot B\cdot\gamma_{f.steel} + g_{n.insulation}\cdot B\cdot\gamma_{f.insulation} = 3.542\cdot\frac{kN}{m}$$

Temporary loads

Long-term load

Normative load from the equipment

$$v_{n.equipment} := 0.5\frac{kN}{m^2}$$

Safety factor for stationary equipment

$\gamma_{f.equipment} := 1.2$ SNiP 2.01.07-85* Table 2

Design long-term load

$$v_{long} := v_{n.equipment} \cdot \gamma_{f.equipment} \cdot B = 3.6 \cdot \frac{kN}{m}$$

Short-term load

Wind load

Safety factor for wind load

$\gamma_{f.wind} := 1.4$ SNiP 2.01.07-85* §6.11

Normative value of the wind pressure for the wind region II

$w_0 := 0.3\frac{kN}{m^2}$ SNiP 2.01.07-85* Table 5

Aerodynamic factors SNiP 2.01.07-85* Appendix 4

Active $C_a := 0.8$

Passive $C_p := 0.4$

The factors taking into account the change in the wind pessure per height SNiP 2.01.07-85* Table 6

For the section height of h_1 $k_1 := 0.5$

For the section height of h_2 $k_2 := 0.65$

At the top of the building $k_3 := 0.68$

Design wind pressure

Active

$$g_{wind.a.1} := \gamma_{f.wind} \cdot w_0 \cdot C_a \cdot k_1 = 0.168 \cdot \frac{kN}{m^2}$$

$$g_{wind.a.2} := \gamma_{f.wind} \cdot w_0 \cdot C_a \cdot k_2 = 0.218 \cdot \frac{kN}{m^2}$$

Passive

$$g_{wind.p.1} := \gamma_{f.wind} \cdot w_0 \cdot C_p \cdot k_1 = 0.084 \cdot \frac{kN}{m^2}$$

$$g_{wind.p.2} := \gamma_{f.wind} \cdot w_0 \cdot C_p \cdot k_2 = 0.109 \cdot \frac{kN}{m^2}$$

Design wind loads

Active

$$w_{a1} := g_{wind.a.1} \cdot B = 1.008 \cdot \frac{kN}{m}$$

$$w_{a2} := g_{wind.a.2} \cdot B = 1.31 \cdot \frac{kN}{m}$$

Passive

$$w_{p1} := g_{wind.p.1} \cdot B = 0.504 \cdot \frac{kN}{m}$$

$$w_{p2} := g_{wind.p.2} \cdot B = 0.655 \cdot \frac{kN}{m}$$

Aerodynamic factors taking into account negative wind pressure

$C_{e1} := 0.215 \qquad C_{e2} := 0.4$ SNiP 2.01.07-85* Appendix 4

Design values of the negative wind loads

$$w_{negative.1} := \gamma_{f.wind} \cdot w_0 \cdot C_{e1} \cdot k_3 \cdot B = 0.368 \cdot \frac{kN}{m}$$

$$w_{negative.2} := \gamma_{f.wind} \cdot w_0 \cdot C_{e2} \cdot k_3 \cdot B = 0.685 \cdot \frac{kN}{m}$$

Calculation of node wind load equivalent to the wind pressure above the column-truss connection

Ledge height $h_{ledge} := 1.4m$

The factors taking into account the change in the wind pessure per height SNiP 2.01.07-85* Table 6

At the bottom of the ledge $k_{ledge1} := 0.65$

At the top of the ledge $k_{ledge2} := 0.68$

Nodal forces at the top of the columns equivalent to uniformly distributed wind load along the ledge

Caused by active wind pressure

$$W_{ledge.a} := \gamma_{f.wind} \cdot w_0 \cdot C_a \cdot B \cdot h_{ledge} \cdot \frac{\left(k_{ledge1} + k_{ledge2}\right)}{2} = 1.877 \cdot kN$$

Caused by passive wind pressure

$$W_{ledge.p} := \gamma_{f.wind} \cdot w_0 \cdot C_p \cdot B \cdot h_{ledge} \cdot \frac{\left(k_{ledge1} + k_{ledge2}\right)}{2} = 0.938 \cdot kN$$

Snow load

Design value of the snow cover weight for the snow region III

$s_g := 1.8 \frac{kN}{m^2}$ SNiP 2.01.07-85* Table 4

Factor to transfer from the snow load on the ground to snow load onto coating

$\mu := 1$ $(\alpha < 10°)$ SNiP 2.01.07-85* Appendix 3

Full design value of the snow load

$$s := s_g \cdot \mu = 1.8 \cdot \frac{kN}{m^2}$$

Uniformly distibuted snow load applied to the top of truss

$$s_{uniform} := s \cdot B = 10.8 \cdot \frac{kN}{m}$$

Node force from the primary trusses

$$G := 12m \cdot \gamma_{f.steel} \cdot g_{n.truss} + \left(g + v_{long} + s_{uniform}\right) \cdot l - w_{negative.1} \cdot \frac{l}{2} - w_{negative.2} \cdot \frac{l}{2} = 434.351 \cdot kN$$

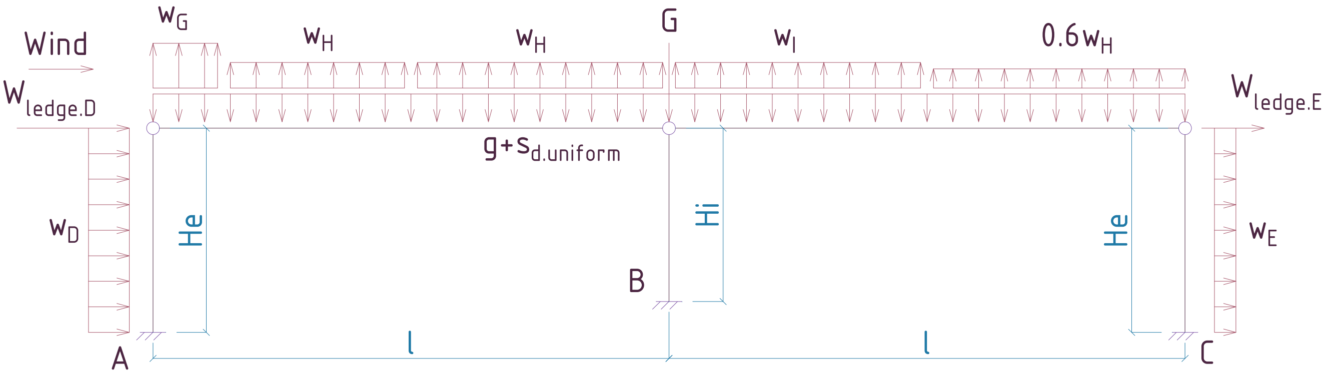


Figure 1. Loads estimation

Building height

$h := 8.54m$

Crosswind dimension

$b := 66m$

Spacing

$B := 6m$

Pitch angle

$\alpha := 2deg$

Ledge height

$h_{ledge} := 1.4m$

The next loads combination was chosen as the most unfavorable:
1.15·Dead load + 1.5·Snow load + 1.5·0.6·Wind load

Dead load

Characteristic self-weight of the steel truss

$$g_{k.truss} := 1.3 \frac{kN}{m}$$

Characteristic self-weight of the coating (profiled sheeting)

$$g_{k.coating} := 0.098 \frac{kN}{m^2}$$

Characteristic self-weight of the insulation

$$g_{k.insulation} := 0.2 \frac{kN}{m^2}$$

Partial safety factor

$\gamma_f := 1.15$

Characteristic equipment load

$g_{k.equipment} := 0.5 \frac{kN}{m^2}$

Design dead load

$g := (g_{k.truss} + g_{k.coating} \cdot B + g_{k.insulation} \cdot B + g_{k.equipment} \cdot B) \cdot \gamma_f = 7.001 \cdot \frac{kN}{m}$

Snow load

Snow load shape coefficient

$\mu_1 := 0.8$ EN 1991-1-3 cl. 5.3

Exposure coefficient

$C_e := 1$ EN 1991-1-3 cl. 5.2 (7)

Thermal coefficient

$C_t := 1$ EN 1991-1-3 cl. 5.2 (8)

Characteristic value of snow load on the ground

$s_k := 2.75 \frac{kN}{m^2}$ NA 20-1991-1-3

Snow load

$s_d := \mu_1 \cdot C_e \cdot C_t \cdot s_k = 2.2 \cdot \frac{kN}{m^2}$ EN 1991-1-3 (5.1)

Design uniform snow load

$s_{d.uniform} := s_d \cdot B \cdot 1.5 = 19.8 \cdot \frac{kN}{m}$

Wind load

Peak velocity pressure q_p

Fundamental value of the basic wind velocity

$v_{b.0} := 21 \frac{m}{s}$ NA 20 EN 1991-1-4 cl. 4.2

Directional factor

$C_{dir} := 1$

Season factor

$C_{season} := 1$

Basic wind velocity v_b

$v_b := C_{dir} \cdot C_{season} \cdot v_{b.0} = 21 \frac{m}{s}$ EN 1991-1-4 (4.1)

Reference height z_e

$z_e := h = 8.54\,m$ z_e should be taken as the height of the structure

EN 1991-1-4 cl. 7.2.7

Terrain category - III EN 1991-1-4 Table 4.1

Air density

$\rho := 1.25 \frac{kg}{m^3}$ NA 20 EN 1991-1-4 cl. 4.5

Basic velocity pressure

$q_b := \frac{1}{2} \cdot \rho \cdot v_b^{\;2} = 0.276 \cdot \frac{kN}{m^2}$ EN 1991-1-4 (4.10)

Exposure factor at height 8.5m

$C_e := 1.5$ EN 1991-1-4 Figure 4.2

Characteristic peak velocity pressure q_p

$q_p := C_e \cdot q_b = 0.413 \cdot \frac{kN}{m^2}$

Wind pressures

$e := \min(b, 2 \cdot h) = 17.08\, m$

External pressure coefficients for vertical walls

$c_{pe.D} := 0.7$ $c_{pe.E} := -0.3$ EN 1991-1-4 Table 7.1

External pressure coefficients for roof

$c_{pe.G} := -1.2$ $c_{pe.H} := -0.7$ $c_{pe.I} := -0.7$ EN 1991-1-4 Table 7.2

Internal pressure coefficient

$c_{pi} := -0.3$ EN 1991-1-4 cl. 7.2.9 Note 2

Design wind pressures

$$w_D := (c_{pe.D} - c_{pi}) \cdot q_p \cdot B \cdot 1.5 \cdot 0.6 = 2.233 \cdot \frac{kN}{m}$$

$$w_E := (c_{pe.E} - c_{pi}) \cdot q_p \cdot B \cdot 1.5 \cdot 0.6 = 0 \cdot \frac{kN}{m}$$

$$w_G := (c_{pe.G} - c_{pi}) \cdot q_p \cdot B \cdot 1.5 \cdot 0.6 = -2.009 \cdot \frac{kN}{m}$$

$$w_H := (c_{pe.H} - c_{pi}) \cdot q_p \cdot B \cdot 1.5 \cdot 0.6 = -0.893 \cdot \frac{kN}{m}$$

$$w_I := (c_{pe.I} - c_{pi}) \cdot q_p \cdot B \cdot 1.5 \cdot 0.6 = -0.893 \cdot \frac{kN}{m}$$

Node forces at the top of external columns

$$W_{ledge.D} := (c_{pe.D} - c_{pi}) \cdot q_p \cdot B \cdot h_{ledge} \cdot 1.5 \cdot 0.6 = 3.126 \cdot kN$$

$$W_{ledge.E} := (c_{pe.E} - c_{pi}) \cdot q_p \cdot B \cdot h_{ledge} \cdot 1.5 \cdot 0.6 = 0 \cdot kN$$

Node force because of the primary trusses

$$G := 12m \cdot g_{k.truss} \cdot \gamma_f + 24m \cdot (g + s_{d.uniform}) - w_G \cdot \frac{e}{10} - w_H \cdot \left(12m + 6m - \frac{e}{10}\right) - w_I \cdot 6m = 684.508 \cdot kN$$

Reactions

$R_{Ax} := -15.4kN$	$R_{Bx} := -3.27kN$	$R_{Cx} := -3.6kN$
$R_{Ay} := 306.3kN$	$R_{By} := 1304.2kN$	$R_{Cy} := 313.8kN$
$M_A := -53.7kN \cdot m$	$M_B := -58.8kN \cdot m$	$M_C := -25.1kN \cdot m$

Static analysis of frame using the force method

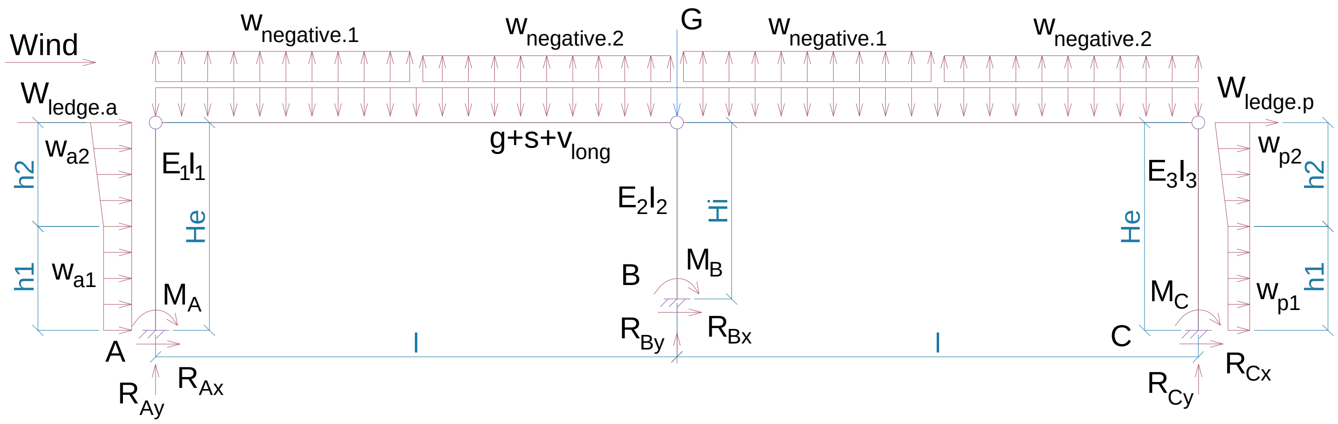


Figure 1. Frame and loads

Initial data

Geometry, [m]

Units:
Forces [kN]
Dimensions [m]

Span $l := 24$

Edge columns height $H_e := 10$

Internal columns height $H_i := 9$

Height of the section along which the wind load varies linearly

$h_1 := 5$ $h_2 := 5$ $h_3 := 0$ $h_4 := 0$

Stiffnesses, [kN*m^2]

Stiffness of the left edge column $EI_1 := 8.583 \times 10^5$

Stiffness of the internal (central) column $EI_2 := 5.365 \times 10^5$

Stiffness of the right edge column $EI_3 := 8.583 \times 10^5$

Loads

Vertical load on the roof (g+s+v$_{long}$), [kN/m] $g_{dead} := 17.94$

Node load because of the primary trusses, kN $G := 446.16$

Design wind loads, [kN/m]

Active	Passive
$w_{a1} := 1.01$	$w_{p1} := 0.756$
$w_{a2} := 1.31$	$w_{p2} := 0.983$
$w_{a3} := 0$	$w_{p3} := 0$
$w_{a4} := 0$	$w_{p4} := 0$

Design values of the negative wind loads, [kN/m]

$w_{negative.1} := 0.368$

$w_{negative.2} := 0.685$

Node forces equivalent to the wind pressure above the column-truss joint, [kN]

Caused by active wind pressure

$W_{ledge.a} := 1.877$

Caused by passive wind pressure

$W_{ledge.p} := 1.408$

Calculation of reations

Degree of static indeterminacy

$n_s := 9 - 3 - 4 = 2$

Unknown reactions MathCAD requires to initialize unknowns first.

$R_{Ax} := R_{Ax}$ $R_{Bx} := R_{Bx}$ $R_{Cx} := R_{Cx}$

$R_{Ay} := R_{Ay}$ $R_{By} := R_{By}$ $R_{Cy} := R_{Cy}$

$M_A := M_A$

Calculation of the reactions for the statically determinate frame, loaded with unit moment at point C

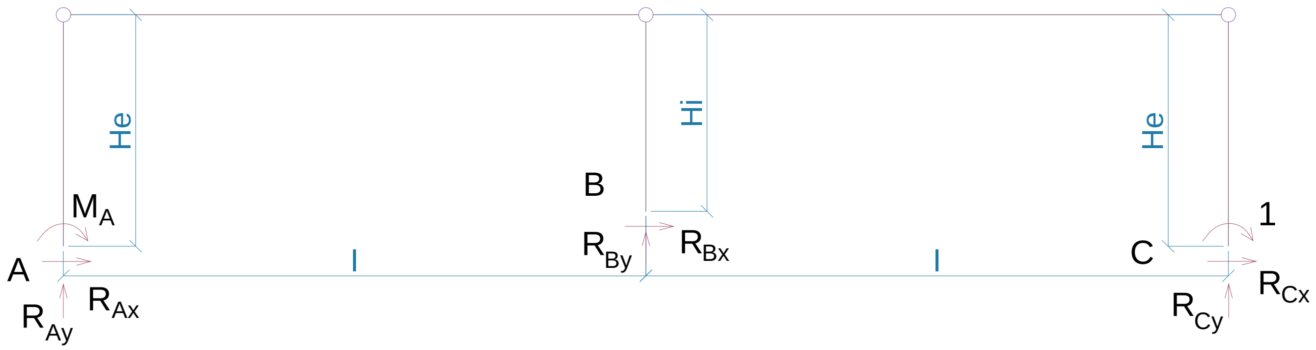


Figure 2. Statically determinate frame, loaded with unit moment at point C

System of equations for the statically determinate frame, loaded with unit force (see Figure 2)

Given

$\Sigma M_C := 1 + R_{By} \cdot l + R_{Ay} \cdot 2 \cdot l + M_A$

$\Sigma M_C = 0$

$\Sigma M_B := M_A + R_{Ay} \cdot l - R_{Cy} \cdot l + 1$

$\Sigma M_B = 0$

$\Sigma M_A := M_A - R_{By} \cdot l - R_{Cy} \cdot 2 \cdot l + 1$

$\Sigma M_A = 0$

$\Sigma M_{F.right} := 1 - R_{Cx} \cdot H_e$

$\Sigma M_{F.right} = 0$

$\Sigma M_{D.left} := M_A - R_{Ax} \cdot H_e$

$\Sigma M_{D.left} = 0$

$\Sigma M_{E.bottom} := -R_{Bx} \cdot Hi$

$\Sigma M_{E.bottom} = 0$

$\Sigma M_{E.left} := M_A + R_{Ay} \cdot l - R_{Ax} \cdot H_e$

$\Sigma M_{E.left} = 0$

Reactions

$$\mathrm{Find}(R_{Ax}, R_{Ay}, M_A, R_{Bx}, R_{By}, R_{Cx}, R_{Cy}) \rightarrow \begin{pmatrix} -\frac{1}{10} \\ 0 \\ -1 \\ 0 \\ 0 \\ \frac{1}{10} \\ 0 \end{pmatrix}$$

Calculation of the reactions for the statically determinate frame, loaded with the unit moment at point B

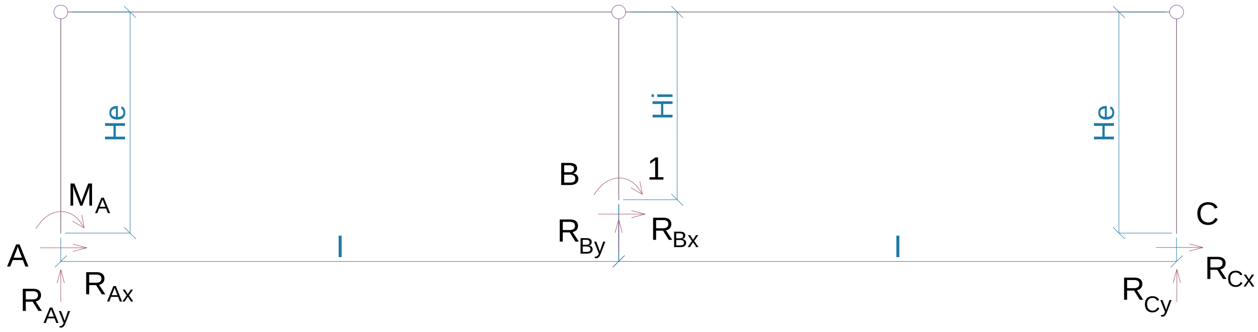


Figure 3. Statically determinate frame, loaded with unit moment at point B

System of equations for the statically determinate frame, loaded with the unit moment at point B

Given

$\Sigma M_C := 1 + R_{By} \cdot l + R_{Ay} \cdot 2 \cdot l + M_A$

$\Sigma M_C = 0$

$\Sigma M_B := M_A + R_{Ay} \cdot l - R_{Cy} \cdot a + 1$

$\Sigma M_B = 0$

$\Sigma M_A := M_A - R_{By} \cdot l - R_{Cy} \cdot 2 \cdot l + 1$

$\Sigma M_A = 0$

$\Sigma M_{F.right} := -R_{Cx} \cdot H_e$

$\Sigma M_{F.right} = 0$

$\Sigma M_{D.left} := M_A - R_{Ax} \cdot H_e$

$\Sigma M_{D.left} = 0$

$\Sigma M_{E.bottom} := 1 - R_{Bx} \cdot Hi$

$\Sigma M_{E.bottom} = 0$

$\Sigma M_{E.left} := M_A + R_{Ay} \cdot l - R_{Ax} \cdot H_e$

$\Sigma M_{E.left} = 0$

Reactions

$$\mathrm{Find}(R_{Ax}, R_{Ay}, M_A, R_{Bx}, R_{By}, R_{Cx}, R_{Cy}) \rightarrow \begin{pmatrix} -\frac{1}{10} \\ 0 \\ -1 \\ \frac{1}{Hi} \\ 0 \\ 0 \\ 0 \end{pmatrix}$$

Calculation of the reactions for the statically determinate system, loaded with the all given loads

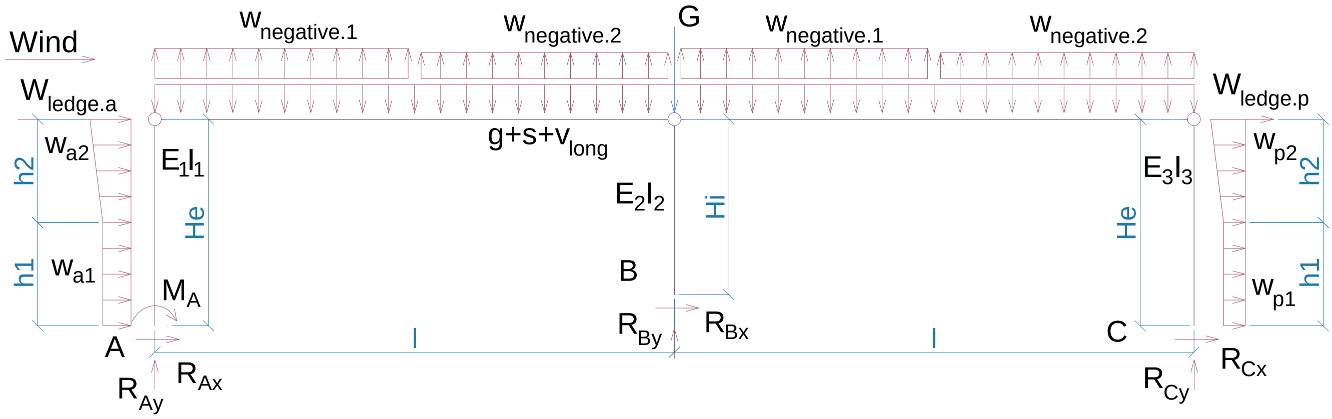


Figure 4. Statically determinate system, loaded with the all the loads

The moment caused by the active wind load over the header, [kN*m]

$$M_{wind.a.header} := w_{a1} \cdot h_1 \cdot \left(H_e - \frac{h_1}{2}\right) \dots$$
$$+ \frac{(w_{a1} + w_{a2}) \cdot h_2}{2} \cdot \left[H_e - h_1 - \frac{h_2 \cdot (2 \cdot w_{a2} + w_{a1})}{3 \cdot (w_{a1} + w_{a2})}\right] \dots$$
$$+ \frac{(w_{a2} + w_{a3}) \cdot h_3}{2} \cdot \left[H_e - h_1 - h_2 - \frac{h_3 \cdot (2 \cdot w_{a3} + w_{a2})}{3 \cdot (w_{a3} + w_{a2})}\right] \dots$$
$$+ \frac{(w_{a4} + w_{a3}) \cdot h_4}{2} \cdot \left[H_e - h_1 - h_2 - h_3 - \frac{h_4 \cdot (2 \cdot w_{a4} + w_{a3})}{3 \cdot (w_{a3} + w_{a4})}\right] = 51.75$$

$$M_{wind.p.header} := w_{p1} \cdot h_1 \cdot \left(H_e - \frac{h_1}{2}\right) \dots$$
$$+ \frac{(w_{p1} + w_{p2}) \cdot h_2}{2} \cdot \left[H_e - h_1 - \frac{h_2 \cdot (2 \cdot w_{p2} + w_{p1})}{3 \cdot (w_{p1} + w_{p2})}\right] \dots$$
$$+ \frac{(w_{p2} + w_{p3}) \cdot h_3}{2} \cdot \left[H_e - h_1 - h_2 - \frac{h_3 \cdot (2 \cdot w_{p3} + w_{p2})}{3 \cdot (w_{p3} + w_{p2})}\right] \dots$$
$$+ \frac{(w_{p4} + w_{p3}) \cdot h_4}{2} \cdot \left[H_e - h_1 - h_2 - h_3 - \frac{h_4 \cdot (2 \cdot w_{p4} + w_{p3})}{3 \cdot (w_{p3} + w_{p4})}\right] = 38.746$$

System of equation for the statically determinate system, loaded with the all the loads

Given

$$\Sigma M_{F.right} := -R_{Cx} \cdot H_e - M_{wind.p.header}$$

$$\Sigma M_{F.right} = 0$$

$$\Sigma X := R_{Ax} + R_{Bx} + R_{Cx} + h_1 \cdot (w_{a1} + w_{p1}) + h_2 \cdot \left[\frac{(w_{a2} + w_{a1})}{2} + \frac{(w_{p2} + w_{p1})}{2}\right] \dots$$
$$+ h_3 \cdot \left[\frac{(w_{a3} + w_{a2})}{2} + \frac{(w_{p3} + w_{p2})}{2}\right] + h_4 \cdot \left[\frac{(w_{a4} + w_{a3})}{2} + \frac{(w_{p4} + w_{p3})}{2}\right] \dots$$
$$+ W_{ledge.a} + W_{ledge.p}$$

$$\Sigma X = 0$$

$$\Sigma Y := R_{Ay} + R_{By} + R_{Cy} - 2 \cdot l \cdot g_{dead} + w_{negative.1} \cdot l + w_{negative.2} \cdot l$$

$$\Sigma Y = 0$$

$$\Sigma M_{D.left} := M_A - R_{Ax} \cdot H_e - M_{wind.a.header}$$

$$\Sigma M_{D.left} = 0$$

$$\Sigma M_{E.bottom} := -R_{Bx} \cdot H_i$$

$$\Sigma M_{E.bottom} = 0$$

$$\Sigma M_{E.left} := R_{Ay}\cdot l - R_{Ax}\cdot H_e + M_A - g_{dead}\cdot l\cdot\frac{l}{2} + w_{negative.1}\cdot\frac{l}{2}\cdot\frac{3}{4}\cdot l \;... \\ + w_{negative.2}\cdot\frac{l}{2}\cdot\frac{l}{4} - M_{wind.a.header}$$

$$\Sigma M_{E.left} = 0$$

$$\Sigma M_{E.right} := -R_{Cy}\cdot l - R_{Cx}\cdot H_e - M_{wind.p.header} - w_{negative.1}\cdot\frac{l}{2}\cdot\frac{l}{4} \;... \\ + w_{negative.2}\cdot\frac{l}{2}\cdot\frac{3}{4}\cdot l + g_{dead}\cdot l\cdot\frac{l}{2}$$

$$\Sigma M_{E.right} = 0$$

$$Reactions := Find\left(R_{Ax}, R_{Ay}, M_A, R_{Bx}, R_{By}, R_{Cx}, R_{Cy}\right) \text{ float}, 5 \;\rightarrow \begin{pmatrix} -18.388 \\ 209.91 \\ -132.13 \\ 0 \\ 405.59 \\ -3.8746 \\ 220.34 \end{pmatrix}$$

Reactions, [kN], [kN*m]

$$R_{Ax} := Reactions_0 = -18.388$$

$$R_{Ay} := Reactions_1 = 209.91$$

$$M_A := Reactions_2 = -132.13$$

$$R_{Bx} := Reactions_3 = 0$$

$$R_{By} := Reactions_4 = 405.59$$

$$R_{Cx} := Reactions_5 = -3.875$$

$$R_{Cy} := Reactions_6 = 220.34$$

Coefficients for the system of equations of the force method

$$\delta_{11} := \frac{1}{EI_1}\cdot\frac{1\cdot H_e}{2}\cdot\frac{2}{3}\cdot 1 + \frac{1}{EI_3}\cdot\frac{1\cdot H_e}{2}\cdot\frac{2}{3}\cdot 1 = 7.767\times 10^{-6}$$

$$\delta_{12} := \frac{1}{EI_1}\cdot\frac{1\cdot H_e}{2}\cdot\frac{2}{3}\cdot 1 = 3.884\times 10^{-6}$$

$$\delta_{21} := \delta_{12} = 3.884\times 10^{-6}$$

$$\delta_{22} := \frac{1}{EI_1}\cdot\frac{1\cdot H_e}{2}\cdot\frac{2}{3}\cdot 1 + \frac{1}{EI_2}\cdot\frac{1\cdot H_i}{2}\cdot\frac{2}{3}\cdot 1 = 9.475\times 10^{-6}$$

Auxiliary functions

Funtions of linear wind loads (returns [kN/m]. x is the distance from the bottom of the section, [m]

For the active wind load

Alond the section height of h_2

$$w_{ax2}(x) := \left(\frac{w_{a2} - w_{a1}}{h_2}\right)\cdot x + w_{a1}$$

Alond the section height of h_3

$$w_{ax3}(x) := \left(\frac{w_{a3} - w_{a2}}{h_3}\right)\cdot x + w_{a2}$$

Alond the section height of h_4

$$w_{ax4}(x) := \left(\frac{w_{a4} - w_{a3}}{h_4}\right)\cdot x + w_{a3}$$

For the passive wind load

Alond the section height of h_2

$$w_{px2}(x) := \left(\frac{w_{p2} - w_{p1}}{h_2}\right)\cdot x + w_{p1}$$

Alond the section height of h_3

$$w_{px3}(x) := \left(\frac{w_{p3} - w_{p2}}{h_3}\right)\cdot x + w_{p2}$$

Alond the section height of h_4

$$w_{px4}(x) := \left(\frac{w_{p4} - w_{p3}}{h_4}\right)\cdot x + w_{p3}$$

Function of the moment caused by wind loads over the header (returns [kN/m]). x is the distance from the bottom of the edge column, [m]

Moment caused by active wind load

$$M_{wind.a}(x) := \mathrm{if}\left[x \le h_1, w_{a1}\cdot x\cdot\frac{x}{2}, \mathrm{if}\left[x > h_1 \wedge x \le h_1 + h_2, w_{a1}\cdot h_1\cdot\left(x - \frac{h_1}{2}\right) + \frac{\left(w_{a1} + w_{ax2}\left(x - h_1\right)\right)\cdot\left(x - h_1\right)}{2}\cdot\left[x - h_1 - \frac{\left(x - h_1\right)\cdot\left(2\cdot w_{ax2}\left(x - h_1\right) + w_{a1}\right)}{3\left(w_{ax2}\left(x - h_1\right) + w_{a1}\right)}\right] \ldots, 99\right.\right.$$

Moment caused by passive wind load

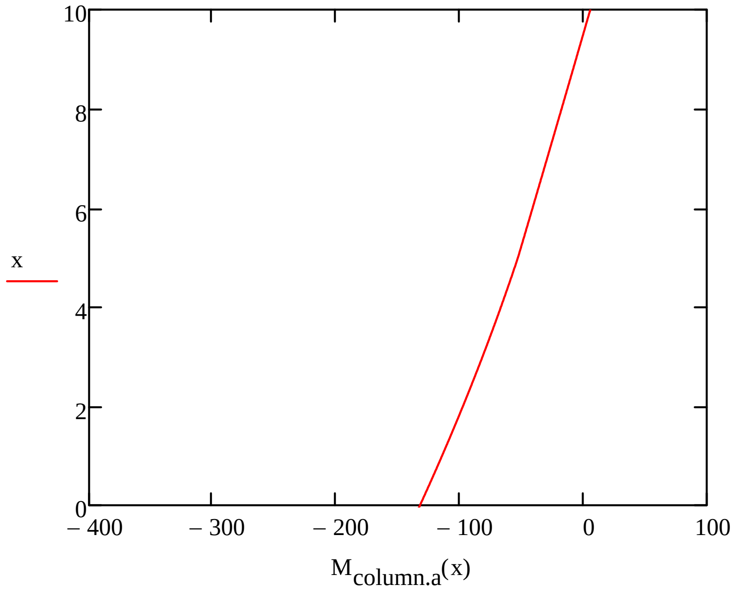
$$M_{wind.p}(x) := \text{if}\left[x \le h_1, w_{p1} \cdot x \cdot \frac{x}{2}, \text{if}\left[x > h_1 \wedge x \le h_1 + h_2, w_{p1} \cdot h_1 \cdot \left(x - \frac{h_1}{2}\right) + \frac{\left(w_{p1} + w_{px2}\left(x - h_1\right)\right) \cdot \left(x - h_1\right)}{2} \ldots, 99 \right.\right.$$
$$+ \left[x - h_1 - \frac{\left(x - h_1\right) \cdot \left(2 \cdot w_{px2}\left(x - h_1\right) + w_{p1}\right)}{3\left(w_{px2}\left(x - h_1\right) + w_{p1}\right)}\right]$$

Function of moment caused by unit force

$$M_{1a}(x) := 1 - \frac{x}{H_e}$$

Function of the moment along the the left edge column (returns [kN/m]). x is a distance from the bottom of the column, [m].

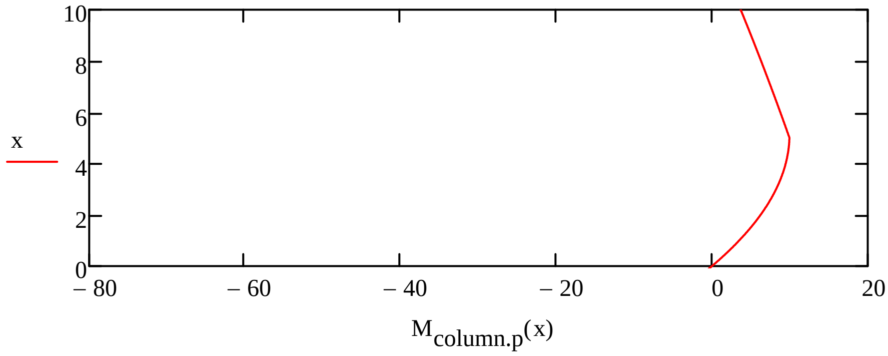
$$M_{column.a}(x) := M_A - R_{Ax} \cdot x - M_{wind.a}(x)$$



Graph of the bending moments for the left edge column

Function of the moment along the the right edge column (returns [kN/m]). x is a distance from the bottom of the column, [m].

$$M_{column.p}(x) := -R_{Cx} \cdot x - M_{wind.p}(x)$$



Graph of the bending moments for the right edge column

Constant terms of the force method system of equitions

$$\Delta_{1F} := \frac{-1}{EI_1}\cdot\int_0^{H_e} M_{column.a}(x)\cdot M_{1a}(x)\,dx + \frac{1}{EI_3}\cdot\int_0^{H_e} M_{column.p}(x)\cdot M_{1a}(x)\,dx = 5.001\times10^{-4}$$

$$\Delta_{2F} := \frac{-1}{EI_1}\cdot\int_0^{H_e} M_{column.a}(x)\cdot M_{1a}(x)\,dx = 4.624\times10^{-4}$$

Given

System of equations for the force method

$$\delta_{11}\cdot x_1 + \delta_{12}\cdot x_2 + \Delta_{1F} = 0$$

$$\delta_{21}\cdot x_1 + \delta_{22}\cdot x_2 + \Delta_{2F} = 0$$

$$\text{Moments} := \text{Find}(x_1, x_2)\ \text{float}, 5\ \rightarrow \begin{pmatrix} -50.292 \\ -28.192 \end{pmatrix}$$

Moments at B and C supports, [kN*m]

$$M_C := \text{Moments}_0 = -50.292$$

$$M_B := \text{Moments}_1 = -28.192$$

Unsetting variables

$R_{Ax} := R_{Ax}$ $\quad R_{Bx} := R_{Bx}$ $\quad R_{Cx} := R_{Cx}$

$R_{Ay} := R_{Ay}$ $\quad R_{By} := R_{By}$ $\quad R_{Cy} := R_{Cy}$

$M_A := M_A$

System of equations for the initial frame

Given

$$\Sigma M_{F.right} := M_C - R_{Cx}\cdot H_e - M_{wind.p.header}$$

$$\Sigma M_{F.right} = 0$$

$$\Sigma X := R_{Ax} + R_{Bx} + R_{Cx} + h_1\cdot(w_{a1} + w_{p1}) + h_2\cdot\left[\frac{(w_{a2} + w_{a1})}{2} + \frac{(w_{p2} + w_{p1})}{2}\right] \ldots$$
$$+ h_3\cdot\left[\frac{(w_{a3} + w_{a2})}{2} + \frac{(w_{p3} + w_{p2})}{2}\right] + h_4\cdot\left[\frac{(w_{a4} + w_{a3})}{2} + \frac{(w_{p4} + w_{p3})}{2}\right] \ldots$$
$$+ W_{ledge.a} + W_{ledge.p}$$

$$\Sigma X = 0$$

$$\Sigma Y := R_{Ay} + R_{By} + R_{Cy} - 2\cdot l\cdot g_{dead} + w_{negative.1}\cdot l + w_{negative.2}\cdot l - G$$

$$\Sigma Y = 0$$

$$\Sigma M_{D.left} := M_A - R_{Ax}\cdot H_e - M_{wind.a.header}$$

$$\Sigma M_{D.left} = 0$$

$$\Sigma M_{E.bottom} := M_B - R_{Bx} \cdot H_i$$

$$\Sigma M_{E.bottom} = 0$$

$$\Sigma M_{E.left} := R_{Ay} \cdot l - R_{Ax} \cdot H_e + M_A - g_{dead} \cdot l \cdot \frac{l}{2} + w_{negative.1} \cdot \frac{l}{2} \cdot \frac{3}{4} \cdot l \ ... \\ + w_{negative.2} \cdot \frac{l}{2} \cdot \frac{l}{4} - M_{wind.a.header}$$

$$\Sigma M_{E.left} = 0$$

$$\Sigma M_{E.right} := M_C - R_{Cy} \cdot l - R_{Cx} \cdot H_e - M_{wind.p.header} - w_{negative.1} \cdot \frac{l}{2} \cdot \frac{l}{4} \ ... \\ + w_{negative.2} \cdot \frac{l}{2} \cdot \frac{3}{4} \cdot l + g_{dead} \cdot l \cdot \frac{l}{2}$$

$$\Sigma M_{E.right} = 0$$

$$\text{Reactions} := \text{Find}\left(R_{Ax}, R_{Ay}, M_A, R_{Bx}, R_{By}, R_{Cx}, R_{Cy}\right) \text{ float}, 5 \ \rightarrow \begin{pmatrix} -10.226 \\ 209.91 \\ -50.513 \\ -3.1324 \\ 851.75 \\ -8.9038 \\ 220.34 \end{pmatrix}$$

Reactions for the frame structure, [kN], [kN*m]

$R_{Ax} := \text{Reactions}_0 = -10.226$ $\quad R_{Bx} := \text{Reactions}_3 = -3.132$ $\quad R_{Cx} := \text{Reactions}_5 = -8.904$

$R_{Ay} := \text{Reactions}_1 = 209.91$ $\quad R_{By} := \text{Reactions}_4 = 851.75$ $\quad R_{Cy} := \text{Reactions}_6 = 220.34$

$M_A := \text{Reactions}_2 = -50.513$ $\quad M_B := 2 \cdot M_B = -56.384$ $\quad M_C = -50.292$

Maximum moment at the headers, [kN*m]

$$M_{max.header} := \frac{R_{Ay} \cdot l}{2} + \frac{w_{negative.1} \cdot l^2}{8} - \frac{g_{dead} \cdot l^2}{8} = 1.254 \times 10^3$$

Static analysis of frame using the flexibility method

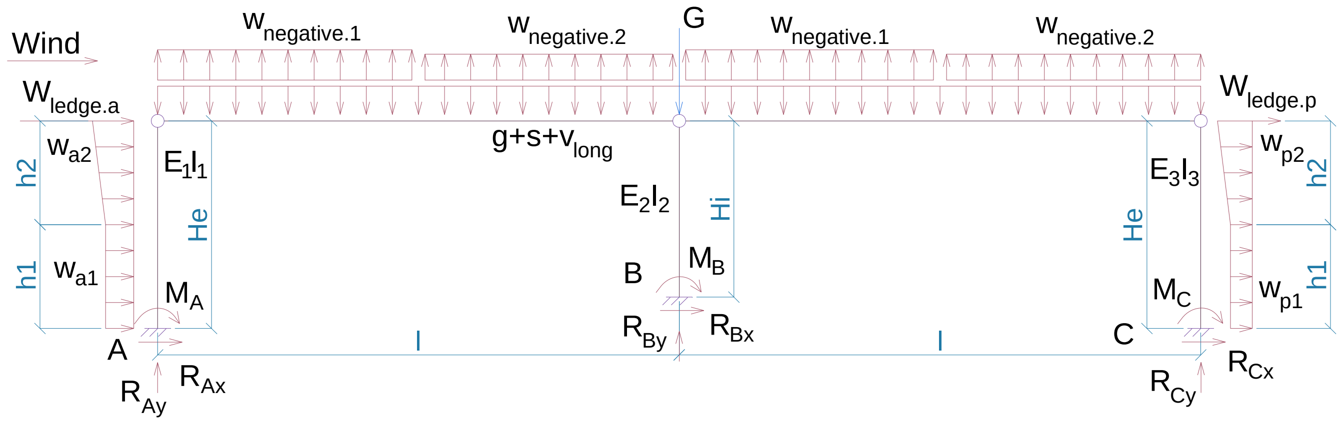


Figure 1. Frame and loads

Geometry and stiffness

Span

$$l := 24m$$

Height of the section with uniformly distributed load

$$h_1 := 5m$$

Height of the section with linearly distributed load

$$h_2 := 5m$$

Columns

Edge column

Height

$$H_e := h_1 + h_2 = 10\,m$$

Stiffness

$$EI_e := 206000\frac{N}{mm^2}\cdot\frac{0.4m\cdot(0.5m)^3}{12} = 8.583\times10^5\cdot kN\cdot m^2$$

Interior column

Height

$$H_i := 9m$$

Stiffness

$$EI_i := 206000\frac{N}{mm^2}\cdot\frac{0.25m\cdot(0.5m)^3}{12} = 5.365\times10^5\cdot kN\cdot m^2$$

Loads

Active uniform wind load

For the section height of h_1

$$w_{a1} := 1.01\frac{kN}{m}$$

For the section height of h_2

$$w_{a2} := 1.31\frac{kN}{m}$$

Passive uniform wind load

For the section height of h_1

$$w_{p1} := 0.756\frac{kN}{m}$$

For the section height of h_2

$$w_{p2} := 0.983\frac{kN}{m}$$

Vertical uniform load at the top of truss

$$g_{dead} := 17.94\,\frac{kN}{m}$$

Node force from the primary trusses

$$G := 446.16kN$$

Vertical (negative) wind loads

$$w_{negative.1} := 0.368\,\frac{kN}{m}$$

$$w_{negative.2} := 0.685\,\frac{kN}{m}$$

Node forces at the top of the column equivalent to wind pressure action above the column-truss joint

$$W_{ledge.a} := 1.877kN$$

$$W_{ledge.p} := 1.408kN$$

Calculation of reactions

Ratios depending on the position of node force, equivalent to the uniform wind loads

$$V := \frac{\frac{1}{3}\cdot h_2}{H_e} = 0.167$$

$$U := 1 - V = 0.833$$

Node forces, equivalent to the uniform active and passive wind loads along the section height of h_2

$$F_a := \frac{\left(w_{a2} - w_{a1}\right)\cdot h_2}{2} = 0.75\cdot kN$$

$$F_p := \frac{\left(w_{p2} - w_{p1}\right)\cdot h_2}{2} = 0.567\cdot kN$$

Coefficients of the equation for the flexibility method . Calculated using the formulas for the standart load cases

$$r_{11} := 6\,\frac{EI_e}{H_e^{\,3}} + 3\,\frac{EI_i}{H_i^{\,3}} = 7.358 \times 10^{3}\cdot\frac{kN}{m}$$

$$R_F := \frac{F_a\cdot U}{2}\cdot(3 - U) + \frac{F_p\cdot U}{2}\cdot(3 - U) + \frac{3}{8}\cdot w_{a1}\cdot H_e + \frac{3}{8}\cdot w_{p1}\cdot H_e + W_{ledge.a} + W_{ledge.p} = 11.097\cdot kN$$

Equation of the flexibility method

$$r_{11}\cdot Z + R_F = 0$$

Unknown of the flexibility method

$$Z := \frac{R_F}{r_{11}} = 1.508 \times 10^{-3}\,m$$

Reactions

$$M_A := \frac{F_a \cdot H_e}{2} \cdot V \cdot \left(1 - V^2\right) + \frac{w_{a1} \cdot H_e^{\ 2}}{8} + \frac{3EI_e}{H_e^{\ 2}} \cdot Z = 52.069 \cdot kN \cdot m$$

$$R_{Ax} := \frac{F_a \cdot V}{2} \cdot \left(3 - V^2\right) + \frac{5}{8} \cdot w_{a1} \cdot H_e + \frac{3 \cdot EI_e}{H_e^{\ 3}} \cdot Z = 10.382 \cdot kN$$

$$R_{Ay} := g_{dead} \cdot \frac{l}{2} - w_{negative.1} \cdot \frac{l}{2} = 210.864 \cdot kN$$

$$M_B := 2 \frac{3EI_i}{H_i^{\ 2}} \cdot Z = 59.933 \cdot kN \cdot m$$

$$R_{Bx} := \frac{3EI_i}{H_i^{\ 3}} \cdot Z = 3.33 \cdot kN$$

$$R_{By} := G + g_{dead} \cdot l - w_{negative.1} \cdot \frac{l}{2} - w_{negative.2} \cdot \frac{l}{2} = 864.084 \cdot kN$$

$$M_C := \frac{F_p \cdot H_e}{2} \cdot V \cdot \left(1 - V^2\right) + \frac{w_{p1} \cdot H_e^{\ 2}}{8} + \frac{3EI_e}{H_e^{\ 2}} \cdot Z = 48.746 \cdot kN \cdot m$$

$$R_{Cx} := \frac{F_p \cdot V}{2} \cdot \left(3 - V^2\right) + \frac{5}{8} \cdot w_{p1} \cdot H_e + \frac{3 \cdot EI_e}{H_e^{\ 3}} \cdot Z = 8.749 \cdot kN$$

$$R_{Cy} := g_{dead} \cdot \frac{l}{2} - w_{negative.2} \cdot \frac{l}{2} = 207.06 \cdot kN$$

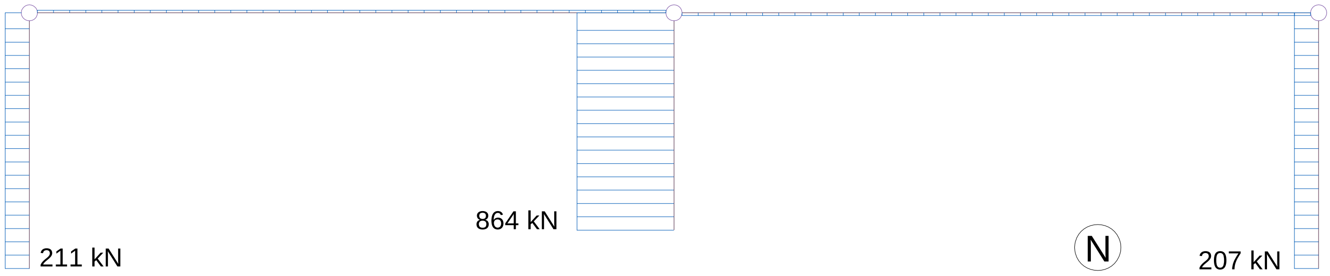


Figure 2. Stress pattern of longitudinal force

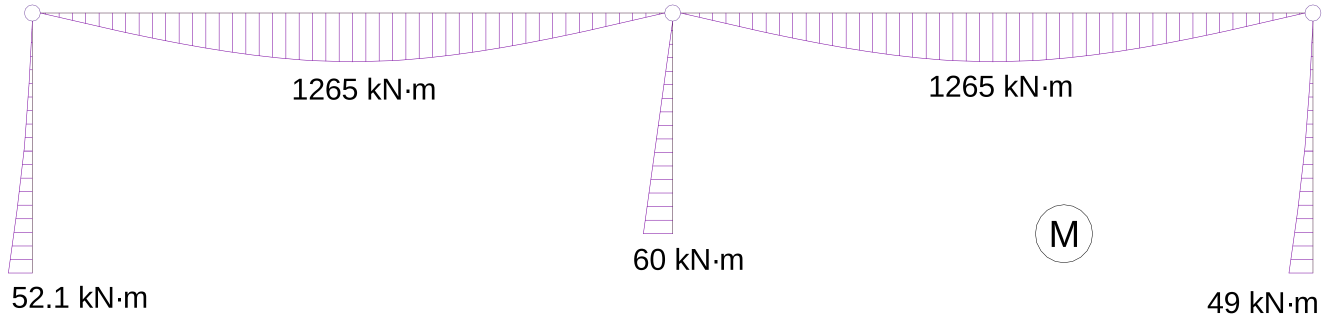


Figure 3. Stress pattern of bendinf moment

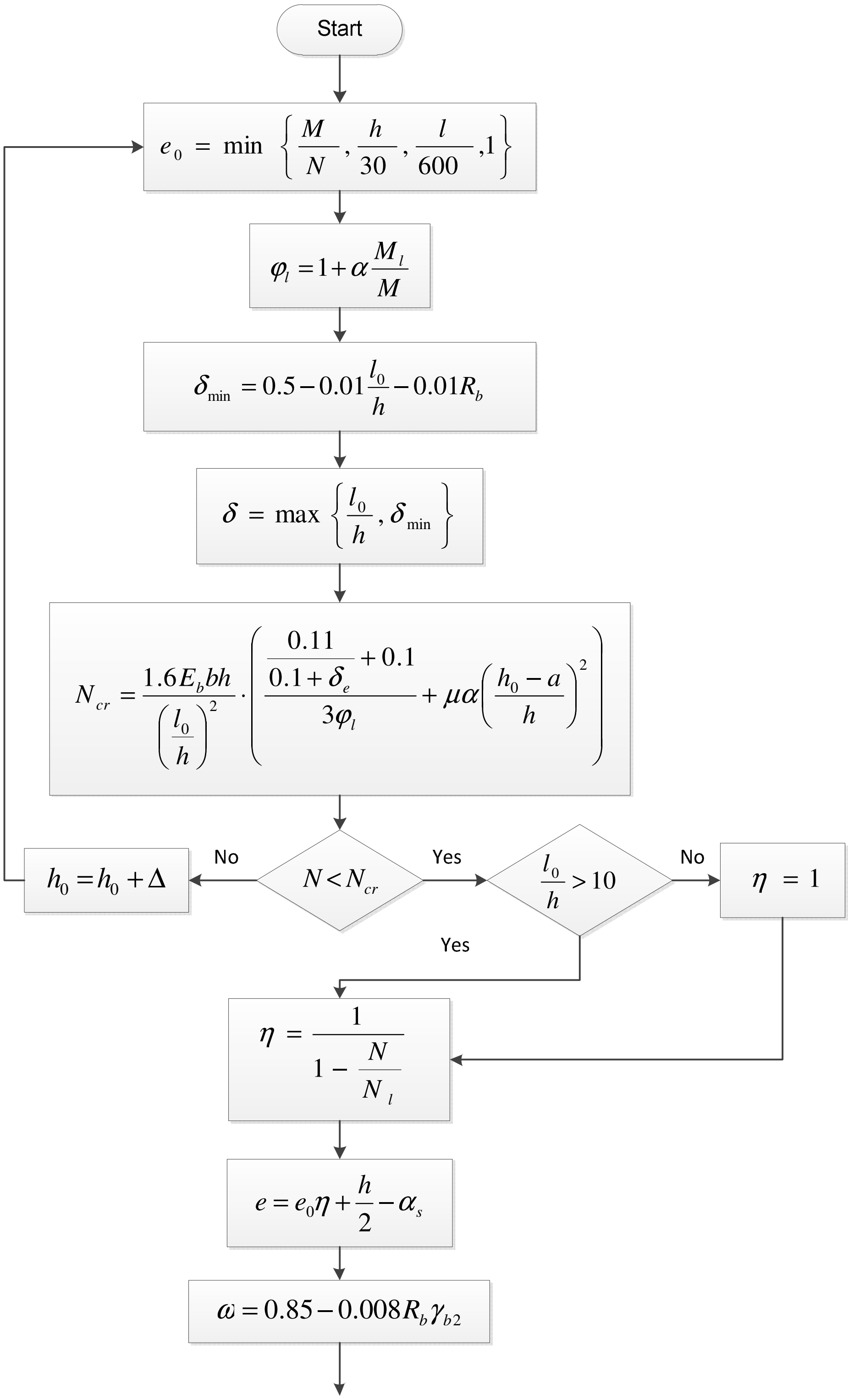


Figure 1 Flowchart for the column design according to SNiP 2.03.01-84*

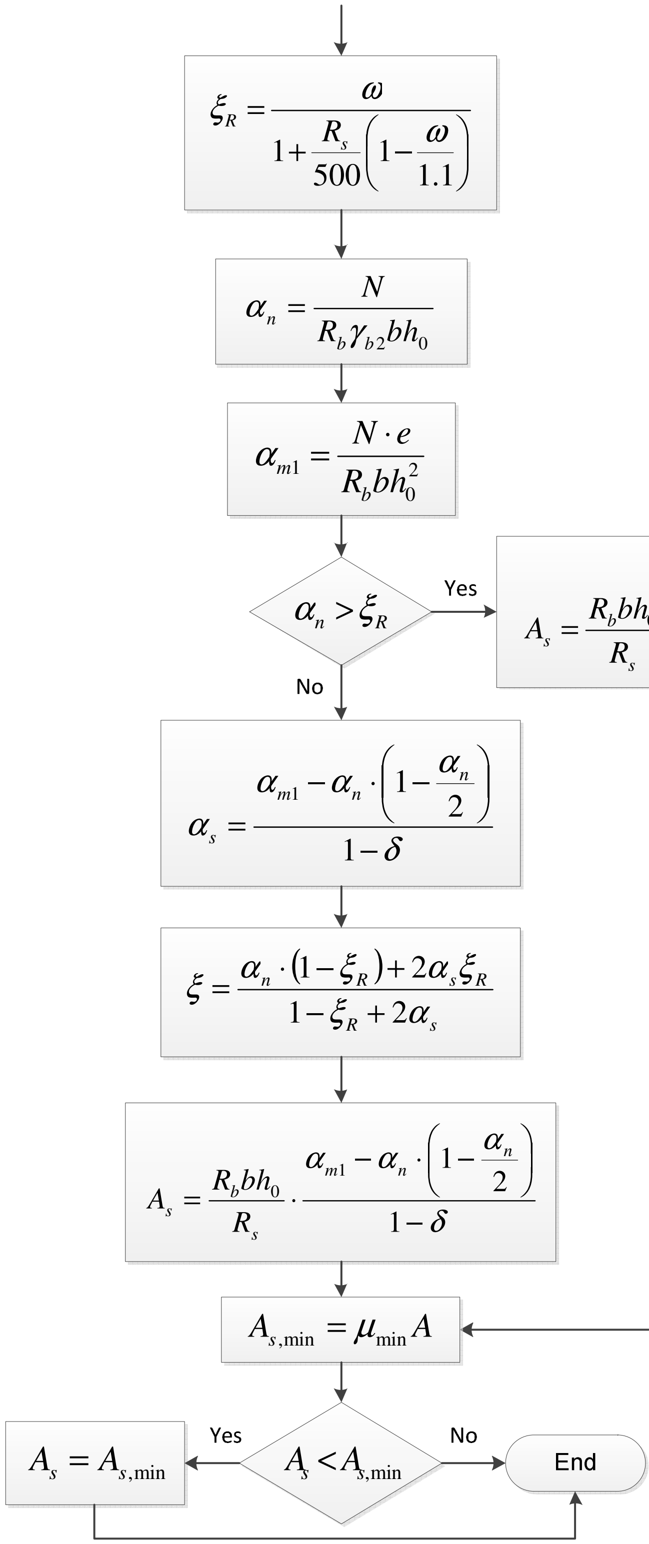
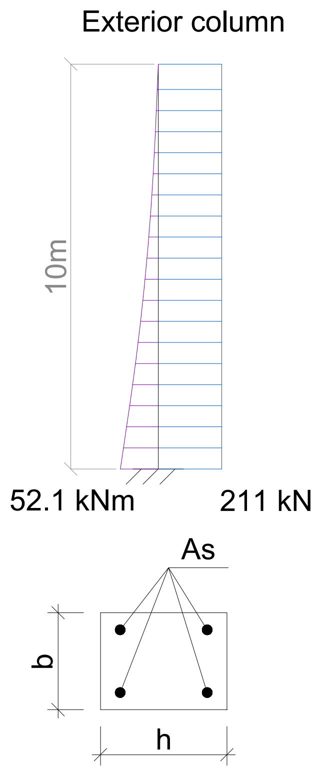


Figure 1 Flowchart for the column design according to SNiP 2.03.01-84* (cont.)

Reinforcement area calculation for exterior column according to SNiP 2.03.01-84*

Input data



Effective length of the column

$l_0 := 1.2 \cdot 10\text{m} = 12\,\text{m}$ SNiP 2.03.01-84* 3.25

Guide to SNiP 2.03.01-84* Table 13

Column section

Width of the rectangular section

$b := 400\text{mm}$

Height of the rectangular section

$h := 500\text{mm}$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$a := 50\text{mm}$ See SNiP 2.03.01-84* 5.5 for minimum values

Concrete

Initial modulus of elasticity during compression and tension

$E_b := 2.7 \cdot 10^4 \dfrac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 18

Design resistance of the concrete to axial compression for limiting states of the first group

$R_b := 17 \dfrac{\text{N}}{\text{mm}^2}$ Concrete class B30 SNiP 2.03.01-84* Table 13

Concrete specific-conditions-of-use factor

$\gamma_{b2} := 0.9$ SNiP 2.03.01-84* Table 15

Reinforcement

Design resistance of the reinforcement to tension for limiting states of the first group

$R_s := 400 \dfrac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 22, 23

Modulus of elasticity

$E_s := 2 \cdot 10^5 \dfrac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 29

Longitudinal force relative to the tensile or the least compressed sectional side due to the effect of continous and long-term loads

$N_l := 215.3\text{kN}$

Moment relative to the tensile or the least compressed sectional side due to the effect of continous and long-term loads

$M_l := 0\text{kN} \cdot \text{m}$

Longitudinal force due to the short-term loads

$$N_{sh} := -4.42kN$$

Bending moment due to the short-term loads

$$M_{sh} := 52.1kN\cdot m$$

Working height of the section

$$h_0 := h - a = 0.45\,m$$

Gyration radius of the section

$$i_{min} := \frac{\min(b,h)}{\sqrt{12}} = 0.115\,m$$

Slenderness

$$\lambda_{max} := \frac{l_0}{i_{min}} = 103.923$$

$\lambda_{lim} := 120$ SNiP 2.03.01-84* 5.3 Limit slenderness for columns, which are members of building, is 120

$$\lambda_{max} \le \lambda_{lim} = 1$$

Bending moment corresponding to the permanent, long-term and short-term loads, except non-long-term loads (wind, crane loads)

$$M_I := M_l + N_l\cdot\frac{h_0 - a}{2} = 43.06\cdot kN\cdot m$$

$$M := M_l + M_{sh} = 52.1\cdot kN\cdot m$$

Total longitudinal force

$$N_t := N_l + N_{sh} = 210.88\cdot kN$$

Bending moment corresponding to the all loads

$$M_{II} := M + N_t\cdot\frac{h_0 - a}{2} = 94.276\cdot kN\cdot m$$

$$\gamma_{bl} := \min\left(\frac{0.9M_{II}}{M_I}, 1.1\right) = 1.1$$

$0.82\cdot M_{II} > M_I = 1$ Guide to SNiP 2.03.01-84* 3.1 (1)

If this condition is true, then all loads (permanent and temporary loads) should be taken into account.
Else account must be done for all loads and for all loads except wind, crane loads

$$R_b := \text{if}\left(0.82\cdot M_{II} > M_I, R_b\cdot\gamma_{b2}, R_b\cdot\gamma_{bl}\right) = 15.3\cdot\frac{N}{mm^2}$$

SNiP 2.03.01-84* 3.24

$\frac{l_0}{h} > 10 = 1$ It shall be allowed to disregard the state of strain in structures with the account taken of bending effect on member strength

Factor assumed depending on concrete type

$\beta := 1$ β equals 1 for heavy weight concrete Guide to SNiP 2.03.01-84* Table 30

Factor accounting for the effect of a long duration lead off member limit state bend

This factor must not be more than 1+β

$$\phi_l := \min\left(1 + \beta\cdot\frac{M_I}{M_{II}}, 1 + \beta\right) = 1.457$$ SNiP 2.03.01-84* 3.6 (21)

Random eccentricity of longitudinal force

$$e_a := \max\left(\frac{h}{30}, \frac{l_0}{600}\right) = 0.02\,m$$ SNiP 2.03.01-84* 1.21

Eccentricity of longitudinal force

$$e_0 := \frac{M}{N_t} + e_a = 0.267\,m$$

δ_e must not be less than $\delta_{e.min}$

$$\delta_{e,min} := 0.5 - 0.01\cdot\frac{l_0}{h} - 0.01\cdot\frac{R_b}{1\,\frac{N}{mm^2}} = 0.107$$ SNiP 2.03.01-84* 3.6 (22)

$$\delta_e := \max\left(\frac{e_0}{h}, \delta_{e,min}\right) = 0.534$$ Guide to SNiP 2.03.01-84* 3.54

Assume that reinforcing ratio μ = 0.001

$$\mu := 0.001$$

$$\alpha := \frac{E_s}{E_b} = 7.407$$

Conditional critical force

Guide to SNiP 2.03.01-84* 3.54 (93)

SNiP 2.03.01-84* 3.6 (20)

$$N_{cr} := \frac{1.6\cdot E_b\cdot b\cdot h}{\left(\frac{l_0}{h}\right)^2}\cdot\left[\frac{\frac{0.11}{0.1 + \delta_e} + 0.1}{3\phi_l} + \mu\cdot\alpha\cdot\left(\frac{h_0 - a}{h}\right)^2\right] = 1.01 \times 10^3\cdot kN$$

Coefficient accounting for the effect of bend on the eccentricity of longitudinal force

$$\eta := \frac{1}{1 - \frac{N_t}{N_{cr}}} = 1.264$$ SNiP 2.03.01-84* 3.6 (19)

Distance from the point of application of longitudinal force N to the resultant of the forces in reinforcement

$$e := e_0\cdot\eta + \frac{h_0 - a}{2} = 0.538\,m$$

Relative longitudinal force

$$\alpha_n := \frac{N_t}{R_b \cdot b \cdot h_0} = 0.077$$ Guide to SNiP 2.03.01-84* 3.62

Relative moment

$$\alpha_{m1} := \frac{N_t \cdot e}{R_b \cdot b \cdot {h_0}^2} = 0.091$$ Guide to SNiP 2.03.01-84* 3.62

$$\delta := \frac{a}{h_0} = 0.111$$ Guide to SNiP 2.03.01-84* 3.62

$\alpha := 0.85$ 0.85 for heavy-weight concrete SNiP 2.03.01-84* 3.12*

Characteristic of concrete compressed area

$$\omega := \alpha - 0.008 \cdot \frac{R_b}{1 \frac{N}{mm^2}} = 0.728$$ SNiP 2.03.01-84* 3.12* (26)

The ultimate stress in reinforcement of compressed zone, taken for structures from heavy-weight, fine-grained, and light-weight concrete in terms of design loads

500 N/mm^2 if γ_{b2} = 0.9 else 400 N/mm^2

$$\sigma_{sc.u} := \mathrm{if}\left(\gamma_{b2} = 0.9, 500 \frac{N}{mm^2}, 400 \frac{N}{mm^2}\right) = 500 \cdot \frac{N}{mm^2}$$

Limit value of relative height of the compressed zone

$$\xi_R := \frac{\omega}{1 + \frac{R_s}{\sigma_{sc.u}} \cdot \left(1 - \frac{\omega}{1.1}\right)} = 0.573$$ SNiP 2.03.01-84* 3.12* (25)

$$\alpha_s := \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta} = 0.02$$ Guide to SNiP 2.03.01-84* 3.62 (114)

Relative height of the compressed zone

$$\xi := \frac{\alpha_n \cdot \left(1 - \xi_R\right) + 2 \cdot \alpha_s \cdot \xi_R}{1 - \xi_R + 2 \cdot \alpha_s} = 0.119$$ Guide to SNiP 2.03.01-84* 3.61 (109)

Area of compressed or tensile reinforcement (for symmetric arrangement if reinforcement bars

Guide to SNiP 2.03.01-84* 3.62 (112, 113)

$$A_s := \mathrm{if}\left[\alpha_n < \xi_R, \frac{R_b \cdot b \cdot h_0}{R_s} \cdot \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta}, \frac{R_b \cdot b \cdot h_0}{R_s} \cdot \frac{\alpha_{m1} - \xi \cdot \left(1 - \frac{\xi}{2}\right)}{1 - \delta}\right] = 1.381 \cdot cm^2$$

Actual reinforcing ratio

$$\mu_{actual} := \frac{A_s \cdot 2}{b \cdot h_0} = 1.535 \times 10^{-3}$$

$$\mu_{actual} < \mu = 0$$

Gyration radius

$$i := \frac{h}{\sqrt{12}} = 0.144\,m$$

Slenderness ratio

$$\lambda := \frac{l_0}{i} = 83.138$$

Minimal reinforcing ratio Guide to SNiP 2.03.01-84* Table 47

$$\mu_{minimal} := \frac{if(\lambda < 17, 0.1, if(\lambda \geq 17 \wedge \lambda \leq 35, 0.2, if(\lambda > 35 \wedge \lambda \leq 83, 0.4, 0.5)))}{100} = 5 \times 10^{-3}$$

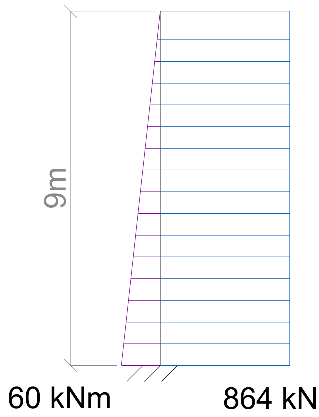
Accept reinforcement area not less than the minimal value

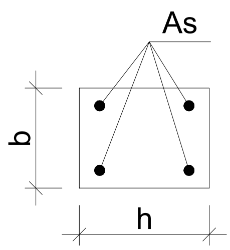
Total reinforcement area

$$A_{total} := if\left(\mu_{actual} < \mu_{minimal}, \mu_{minimal} \cdot b \cdot h_0, 2 \cdot A_s\right) = 9 \cdot cm^2$$

Reinforcement area calculation for interior column according to SNiP 2.03.01-84*

Interior column





Input data

Effective length of the column

$l_0 := 1.2 \cdot 9m = 10.8m$ SNiP 2.03.01-84* 3.25

Guide to SNiP 2.03.01-84* Table 13

Column section

Width of the rectangular section

$b := 500mm$

Height of the rectangular section

$h := 500mm$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$a := 50mm$ See SNiP 2.03.01-84* 5.5 for minimum values

Concrete

Initial modulus of elasticity during compression and tension

$E_b := 2.7 \cdot 10^4 \frac{N}{mm^2}$ SNiP 2.03.01-84* Table 18

Design resistance of the concrete to axial compression for limiting states of the first group

$R_b := 17 \frac{N}{mm^2}$ Concrete class B30 SNiP 2.03.01-84* Table 13

Concrete specific-conditions-of-use factor

$\gamma_{b2} := 0.9$ SNiP 2.03.01-84* Table 15

Reinforcement

Design resistance of the reinforcement to tension for limiting states of the first group

$R_s := 400 \frac{N}{mm^2}$ SNiP 2.03.01-84* Table 22, 23

Modulus of elasticity

$E_s := 2 \cdot 10^5 \frac{N}{mm^2}$ SNiP 2.03.01-84* Table 29

Longitudinal force relative to the tensile or the least compressed sectional side due to the effect of continous and long-term loads

$N_l := 876.7kN$

Moment relative to the tensile or the least compressed sectional side due to the effect of continous and long-term loads

$M_l := 0kN \cdot m$

Longitudinal force due to the short-term loads

$$N_{sh} := -12.6kN$$

Bending moment due to the short-term loads

$$M_{sh} := 60kN \cdot m$$

Working height of the section

$$h_0 := h - a = 0.45\,m$$

Gyration radius of the section

$$i_{min} := \frac{\min(b, h)}{\sqrt{12}} = 0.144\,m$$

Slenderness

$$\lambda_{max} := \frac{l_0}{i_{min}} = 74.825$$

$\lambda_{lim} := 120$ SNiP 2.03.01-84* 5.3 Limit slenderness for columns, which are members of building, is 120

$$\lambda_{max} \le \lambda_{lim} = 1$$

Bending moment corresponding to the permanent, long-term and short-term loads, except non-long-term loads (wind, crane loads)

$$M_I := M_l + N_l \cdot \frac{h_0 - a}{2} = 175.34 \cdot kN \cdot m$$

$$M := M_l + M_{sh} = 60 \cdot kN \cdot m$$

Total longitudinal force

$$N_t := N_l + N_{sh} = 864.1 \cdot kN$$

Bending moment corresponding to the all loads

$$M_{II} := M + N_t \cdot \frac{h_0 - a}{2} = 232.82 \cdot kN \cdot m$$

$$\gamma_{bl} := \min\left(\frac{0.9M_{II}}{M_I}, 1.1\right) = 1.1$$

$M_I < 0.82 \cdot M_{II} = 1$ Guide to SNiP 2.03.01-84* 3.1 (1)

If this condition is true, then all loads (permanent and temporary loads) should be taken into account.
Else account must be done for all loads and for all loads except wind, crane loads

$$R_b := \text{if}\left(M_I < 0.82 \cdot M_{II}, R_b \cdot \gamma_{b2}, R_b \cdot \gamma_{bl}\right) = 15.3 \cdot \frac{N}{mm^2}$$

SNiP 2.03.01-84* 3.24

$$\frac{l_0}{h} > 10 = 1$$

It shall be allowed to disregard the state of strain in structures with the account taken of bending effect on member strength

Factor assumed depending on concrete type

$\beta := 1$ β equals 1 for heavy weight concrete Guide to SNiP 2.03.01-84* Table 30

Factor accounting for the effect of a long duration lead off member limit state bend

This factor must not be more than 1+β

$$\phi_l := \min\left(1 + \beta \cdot \frac{M_I}{M_{II}}, 1 + \beta\right) = 1.753$$ SNiP 2.03.01-84* 3.6 (21)

Random eccentricity of longitudinal force

$$e_a := \max\left(\frac{h}{30}, \frac{l_0}{600}\right) = 0.018\,\mathrm{m}$$ SNiP 2.03.01-84* 1.21

Eccentricity of longitudinal force

$$e_0 := \frac{M}{N_t} + e_a = 0.087\,\mathrm{m}$$

δ_e must not be less than $\delta_{e.min}$

$$\delta_{e,min} := 0.5 - 0.01 \cdot \frac{l_0}{h} - 0.01 \cdot \frac{R_b}{1 \frac{N}{mm^2}} = 0.131$$ SNiP 2.03.01-84* 3.6 (22)

$$\delta_e := \max\left(\frac{e_0}{h}, \delta_{e,min}\right) = 0.175$$ Guide to SNiP 2.03.01-84* 3.54

Assume that reinforcing ratio μ = 0.0001

$$\mu := 0.0001$$

$$\alpha := \frac{E_s}{E_b} = 7.407$$

Conditional critical force

Guide to SNiP 2.03.01-84* 3.54 (93)

SNiP 2.03.01-84* 3.6 (20)

$$N_{cr} := \frac{1.6 \cdot E_b \cdot b \cdot h}{\left(\frac{l_0}{h}\right)^2} \cdot \left[\frac{\frac{0.11}{0.1 + \delta_e} + 0.1}{3\phi_l} + \mu \cdot \alpha \cdot \left(\frac{h_0 - a}{h}\right)^2\right] = 2.212 \times 10^3 \cdot \mathrm{kN}$$

Coefficient accounting for the effect of bend on the eccentricity of longitudinal force

$$\eta := \frac{1}{1 - \frac{N_t}{N_{cr}}} = 1.641$$ SNiP 2.03.01-84* 3.6 (19)

Distance from the point of application of longitudinal force N to the resultant of the forces in reinforcement

$$e := e_0 \cdot \eta + \frac{h_0 - a}{2} = 0.343\,\text{m}$$

Relative longitudinal force

$$\alpha_n := \frac{N_t}{R_b \cdot b \cdot h_0} = 0.251$$ Guide to SNiP 2.03.01-84* 3.62

Relative moment

$$\alpha_{m1} := \frac{N_t \cdot e}{R_b \cdot b \cdot h_0^2} = 0.192$$ Guide to SNiP 2.03.01-84* 3.62

$$\delta := \frac{a}{h_0} = 0.111$$ Guide to SNiP 2.03.01-84* 3.62

$\alpha := 0.85$ 0.85 for heavy-weight concrete SNiP 2.03.01-84* 3.12*

Characteristic of concrete compressed area

$$\omega := \alpha - 0.008 \cdot \frac{R_b}{1\text{MPa}} = 0.728$$ SNiP 2.03.01-84* 3.12* (26)

The ultimate stress in reinforcement of compressed zone, taken for structures from heavy-weight, fine-grained, and light-weight concrete in terms of design loads

500 N/mm^2 if γ_{b2} = 0.9 else 400 N/mm^2

$$\sigma_{sc.u} := \text{if}\left(\gamma_{b2} = 0.9, 500\frac{N}{mm^2}, 400\frac{N}{mm^2}\right) = 500 \cdot \frac{N}{mm^2}$$

Limit value of relative height of the compressed zone

$$\xi_R := \frac{\omega}{1 + \frac{R_s}{\sigma_{sc.u}} \cdot \left(1 - \frac{\omega}{1.1}\right)} = 0.573$$ SNiP 2.03.01-84* 3.12* (25)

$$\alpha_s := \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta} = -0.031$$ Guide to SNiP 2.03.01-84* 3.62 (114)

Relative height of the compressed zone

$$\xi := \frac{\alpha_n \cdot (1 - \xi_R) + 2 \cdot \alpha_s \cdot \xi_R}{1 - \xi_R + 2 \cdot \alpha_s} = 0.196$$ Guide to SNiP 2.03.01-84* 3.61 (109)

Area of compressed or tensile reinforcement (for symmetric arrangement if reinforcement bars

Guide to SNiP 2.03.01-84* 3.62 (112, 113)

$$A_s := \text{if}\left[\alpha_n < \xi_R, \frac{R_b \cdot b \cdot h_0}{R_s} \cdot \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta}, \frac{R_b \cdot b \cdot h_0}{R_s} \cdot \frac{\alpha_{m1} - \xi \cdot \left(1 - \frac{\xi}{2}\right)}{1 - \delta}\right] = -2.703 \cdot \text{cm}^2$$

Actual reinforcing ratio

$$\mu_{actual} := \frac{A_s \cdot 2}{b \cdot h} = -2.163 \times 10^{-3}$$

$$\mu_{actual} < \mu = 1$$

Gyration radius

$$i := \frac{h}{\sqrt{12}} = 0.144\,\text{m}$$

Slenderness ratio

$$\lambda := \frac{l_0}{i} = 74.825$$

Minimal reinforcing ratio

Guide to SNiP 2.03.01-84* Table 47

$$\mu_{minimal} := \frac{\text{if}(\lambda < 17, 0.1, \text{if}(\lambda \geq 17 \wedge \lambda \leq 35, 0.2, \text{if}(\lambda > 35 \wedge \lambda \leq 83, 0.4, 0.5)))}{100} = 4 \times 10^{-3}$$

Accept reinforcement area not less than the minimal value

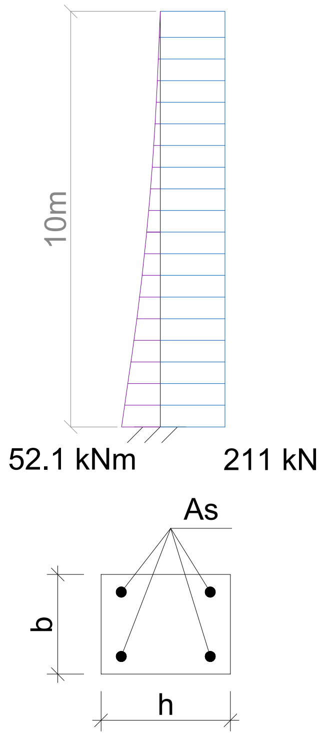
Total reinforcement area

$$A_{total} := \text{if}\left(\mu_{actual} < \mu_{minimal}, \mu_{minimal} \cdot b \cdot h_0, 2 \cdot A_s\right) = 9 \cdot \text{cm}^2$$

Reinforcement area calculation for exterior column according to SNiP 2.03.01-84* (effective length ratio equals 2)

Input data

Exterior column



Effective length of the column

$l_0 := 2 \cdot 10\text{m} = 20\,\text{m}$ SNiP 2.03.01-84* 3.25

Guide to SNiP 2.03.01-84* Table 13

Column section

Width of the rectangular section

$b := 400\text{mm}$

Height of the rectangular section

$h := 500\text{mm}$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$a := 50\text{mm}$ See SNiP 2.03.01-84* 5.5 for minimum values

Concrete

Initial modulus of elasticity during compression and tension

$E_b := 2.7 \cdot 10^4 \dfrac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 18

Design resistance of the concrete to axial compression for limiting states of the first group

$R_b := 17 \dfrac{\text{N}}{\text{mm}^2}$ Concrete class B30 SNiP 2.03.01-84* Table 13

Concrete specific-conditions-of-use factor

$\gamma_{b2} := 0.9$ SNiP 2.03.01-84* Table 15

Reinforcement

Design resistance of the reinforcement to tension for limiting states of the first group

$R_s := 400 \dfrac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 22, 23

Modulus of elasticity

$E_s := 2 \cdot 10^5 \dfrac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 29

Longitudinal force relative to the tensile or the least compressed sectional side due to the effect of continous and long-term loads

$N_l := 215.3\text{kN}$

Moment relative to the tensile or the least compressed sectional side due to the effect of continous and long-term loads

$M_l := 0\text{kN} \cdot \text{m}$

Longitudinal force due to the short-term loads

$$N_{sh} := -4.42kN$$

Bending moment due to the short-term loads

$$M_{sh} := 52.1kN \cdot m$$

Working height of the section

$$h_0 := h - a = 0.45\,m$$

Gyration radius of the section

$$i_{min} := \frac{\min(b, h)}{\sqrt{12}} = 0.115\,m$$

Slenderness

$$\lambda_{max} := \frac{l_0}{i_{min}} = 173.205$$

$\lambda_{lim} := 120$ SNiP 2.03.01-84* 5.3 Limit slenderness for columns, which are members of building, is 120

$$\lambda_{max} \le \lambda_{lim} = 0$$

Bending moment corresponding to the permanent, long-term and short-term loads, except non-long-term loads (wind, crane loads)

$$M_I := M_l + N_l \cdot \frac{h_0 - a}{2} = 43.06 \cdot kN \cdot m$$

$$M := M_l + M_{sh} = 52.1 \cdot kN \cdot m$$

Total longitudinal force

$$N_t := N_l + N_{sh} = 210.88 \cdot kN$$

Bending moment corresponding to the all loads

$$M_{II} := M + N_t \cdot \frac{h_0 - a}{2} = 94.276 \cdot kN \cdot m$$

$$\gamma_{bl} := \min\left(\frac{0.9M_{II}}{M_I}, 1.1\right) = 1.1$$

$0.82 \cdot M_{II} > M_I = 1$ Guide to SNiP 2.03.01-84* 3.1 (1)

If this condition is true, then all loads (permanent and temporary loads) should be taken into account.
Else account must be done for all loads and for all loads except wind, crane loads

$$R_b := \text{if}\left(0.82 \cdot M_{II} > M_I, R_b \cdot \gamma_{b2}, R_b \cdot \gamma_{bl}\right) = 15.3 \cdot \frac{N}{mm^2}$$

SNiP 2.03.01-84* 3.24

$\frac{l_0}{h} > 10 = 1$ It shall be allowed to disregard the state of strain in structures with the account taken of bending effect on member strength

Factor assumed depending on concrete type

$\beta := 1$ β equals 1 for heavy weight concrete Guide to SNiP 2.03.01-84* Table 30

Factor accounting for the effect of a long duration lead off member limit state bend

This factor must not be more than 1+β

$$\phi_l := \min\left(1 + \beta\cdot\frac{M_I}{M_{II}}, 1 + \beta\right) = 1.457$$ SNiP 2.03.01-84* 3.6 (21)

Random eccentricity of longitudinal force

$$e_a := \max\left(\frac{h}{30}, \frac{l_0}{600}\right) = 0.033\,\text{m}$$ SNiP 2.03.01-84* 1.21

Eccentricity of longitudinal force

$$e_0 := \frac{M}{N_t} + e_a = 0.28\,\text{m}$$

δ_e must not be less than $\delta_{e.min}$

$$\delta_{e,min} := 0.5 - 0.01\cdot\frac{l_0}{h} - 0.01\cdot\frac{R_b}{1\frac{N}{mm^2}} = -0.053$$ SNiP 2.03.01-84* 3.6 (22)

$$\delta_e := \max\left(\frac{e_0}{h}, \delta_{e,min}\right) = 0.561$$ Guide to SNiP 2.03.01-84* 3.54

Assume that reinforcing ratio μ = 0.003

$$\mu := 0.003$$

$$\alpha := \frac{E_s}{E_b} = 7.407$$

Conditional critical force

Guide to SNiP 2.03.01-84* 3.54 (93)

SNiP 2.03.01-84* 3.6 (20)

$$N_{cr} := \frac{1.6\cdot E_b\cdot b\cdot h}{\left(\frac{l_0}{h}\right)^2}\cdot\left[\frac{\frac{0.11}{0.1 + \delta_e} + 0.1}{3\phi_l} + \mu\cdot\alpha\cdot\left(\frac{h_0 - a}{h}\right)^2\right] = 406.057\cdot\text{kN}$$

Coefficient accounting for the effect of bend on the eccentricity of longitudinal force

$$\eta := \frac{1}{1 - \frac{N_t}{N_{cr}}} = 2.08$$ SNiP 2.03.01-84* 3.6 (19)

Distance from the point of application of longitudinal force N to the resultant of the forces in reinforcement

$$e := e_0\cdot\eta + \frac{h_0 - a}{2} = 0.783\,\text{m}$$

Relative longitudinal force

$$\alpha_n := \frac{N_t}{R_b \cdot b \cdot h_0} = 0.077$$ Guide to SNiP 2.03.01-84* 3.62

Relative moment

$$\alpha_{m1} := \frac{N_t \cdot e}{R_b \cdot b \cdot h_0^{\;2}} = 0.133$$ Guide to SNiP 2.03.01-84* 3.62

$$\delta := \frac{a}{h_0} = 0.111$$ Guide to SNiP 2.03.01-84* 3.62

$\alpha := 0.85$ 0.85 for heavy-weight concrete SNiP 2.03.01-84* 3.12*

Characteristic of concrete compressed area

$$\omega := \alpha - 0.008 \cdot \frac{R_b}{1 \frac{N}{mm^2}} = 0.728$$ SNiP 2.03.01-84* 3.12* (26)

The ultimate stress in reinforcement of compressed zone, taken for structures from heavy-weight, fine-grained, and light-weight concrete in terms of design loads

500 N/mm^2 if γ_{b2} = 0.9 else 400 N/mm^2

$$\sigma_{sc.u} := \text{if}\left(\gamma_{b2} = 0.9, 500 \frac{N}{mm^2}, 400 \frac{N}{mm^2}\right) = 500 \cdot \frac{N}{mm^2}$$

Limit value of relative height of the compressed zone

$$\xi_R := \frac{\omega}{1 + \frac{R_s}{\sigma_{sc.u}} \cdot \left(1 - \frac{\omega}{1.1}\right)} = 0.573$$ SNiP 2.03.01-84* 3.12* (25)

$$\alpha_s := \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta} = 0.067$$ Guide to SNiP 2.03.01-84* 3.62 (114)

Relative height of the compressed zone

$$\xi := \frac{\alpha_n \cdot (1 - \xi_R) + 2 \cdot \alpha_s \cdot \xi_R}{1 - \xi_R + 2 \cdot \alpha_s} = 0.195$$ Guide to SNiP 2.03.01-84* 3.61 (109)

Area of compressed or tensile reinforcement (for symmetric arrangement if reinforcement bars

Guide to SNiP 2.03.01-84* 3.62 (112, 113)

$$A_s := \text{if}\left[\alpha_n < \xi_R, \frac{R_b \cdot b \cdot h_0}{R_s} \cdot \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta}, \frac{R_b \cdot b \cdot h_0}{R_s} \cdot \frac{\alpha_{m1} - \xi \cdot \left(1 - \frac{\xi}{2}\right)}{1 - \delta}\right] = 4.621 \cdot cm^2$$

Actual reinforcing ratio

$$\mu_{actual} := \frac{A_s \cdot 2}{b \cdot h_0} = 5.134 \times 10^{-3}$$

$$\mu_{actual} < \mu = 0$$

Gyration radius

$$i := \frac{h}{\sqrt{12}} = 0.144\,m$$

Slenderness ratio

$$\lambda := \frac{l_0}{i} = 138.564$$

Minimal reinforcing ratio Guide to SNiP 2.03.01-84* Table 47

$$\mu_{minimal} := \frac{\text{if}(\lambda < 17, 0.1, \text{if}(\lambda \ge 17 \wedge \lambda \le 35, 0.2, \text{if}(\lambda > 35 \wedge \lambda \le 83, 0.4, 0.5)))}{100} = 5 \times 10^{-3}$$

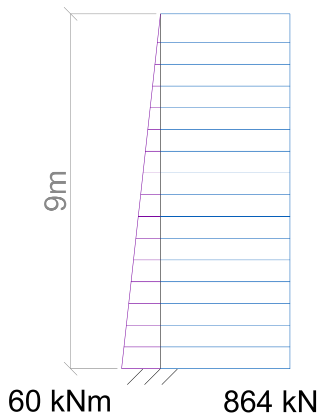
Accept reinforcement area not less than the minimal value

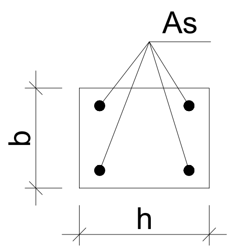
Total reinforcement area

$$A_{total} := \text{if}\left(\mu_{actual} < \mu_{minimal}, \mu_{minimal} \cdot b \cdot h_0, 2 \cdot A_s\right) = 9.241 \cdot cm^2$$

Reinforcement area calculation for interior column according to SNiP 2.03.01-84* (effective length ratio equals 2)

Interior column





Input data

Effective length of the column

$l_0 := 2 \cdot 9\text{m} = 18\text{m}$ SNiP 2.03.01-84* 3.25

Guide to SNiP 2.03.01-84* Table 13

Column section

Width of the rectangular section

$b := 500\text{mm}$

Height of the rectangular section

$h := 500\text{mm}$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$a := 50\text{mm}$ See SNiP 2.03.01-84* 5.5 for minimum values

Concrete

Initial modulus of elasticity during compression and tension

$E_b := 2.7 \cdot 10^4 \frac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 18

Design resistance of the concrete to axial compression for limiting states of the first group

$R_b := 17 \frac{\text{N}}{\text{mm}^2}$ Concrete class B30 SNiP 2.03.01-84* Table 13

Concrete specific-conditions-of-use factor

$\gamma_{b2} := 0.9$ SNiP 2.03.01-84* Table 15

Reinforcement

Design resistance of the reinforcement to tension for limiting states of the first group

$R_s := 400 \frac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 22, 23

Modulus of elasticity

$E_s := 2 \cdot 10^5 \frac{\text{N}}{\text{mm}^2}$ SNiP 2.03.01-84* Table 29

Longitudinal force relative to the tensile or the least compressed sectional side due to the effect of continous and long-term loads

$N_l := 876.7\text{kN}$

Moment relative to the tensile or the least compressed sectional side due to the effect of continous and long-term loads

$M_l := 0\text{kN} \cdot \text{m}$

Longitudinal force due to the short-term loads

$$N_{sh} := -12.6kN$$

Bending moment due to the short-term loads

$$M_{sh} := 60kN \cdot m$$

Working height of the section

$$h_0 := h - a = 0.45\,m$$

Gyration radius of the section

$$i_{min} := \frac{\min(b, h)}{\sqrt{12}} = 0.144\,m$$

Slenderness

$$\lambda_{max} := \frac{l_0}{i_{min}} = 124.708$$

$\lambda_{lim} := 120$ SNiP 2.03.01-84* 5.3 Limit slenderness for columns, which are members of building, is 120

$$\lambda_{max} \le \lambda_{lim} = 0$$

Bending moment corresponding to the permanent, long-term and short-term loads, except non-long-term loads (wind, crane loads)

$$M_I := M_l + N_l \cdot \frac{h_0 - a}{2} = 175.34 \cdot kN \cdot m$$

$$M := M_l + M_{sh} = 60 \cdot kN \cdot m$$

Total longitudinal force

$$N_t := N_l + N_{sh} = 864.1 \cdot kN$$

Bending moment corresponding to the all loads

$$M_{II} := M + N_t \cdot \frac{h_0 - a}{2} = 232.82 \cdot kN \cdot m$$

$$\gamma_{bl} := \min\left(\frac{0.9M_{II}}{M_I}, 1.1\right) = 1.1$$

$M_I < 0.82 \cdot M_{II} = 1$ Guide to SNiP 2.03.01-84* 3.1 (1)

If this condition is true, then all loads (permanent and temporary loads) should be taken into account.
Else account must be done for all loads and for all loads except wind, crane loads

$$R_b := \text{if}\left(M_I < 0.82 \cdot M_{II}, R_b \cdot \gamma_{b2}, R_b \cdot \gamma_{bl}\right) = 15.3 \cdot \frac{N}{mm^2}$$

SNiP 2.03.01-84* 3.24

$$\frac{l_0}{h} > 10 = 1$$

It shall be allowed to disregard the state of strain in structures with the account taken of bending effect on member strength

Factor assumed depending on concrete type

$\beta := 1$ β equals 1 for heavy weight concrete Guide to SNiP 2.03.01-84* Table 30

Factor accounting for the effect of a long duration lead off member limit state bend

This factor must not be more than 1+β

$$\phi_l := \min\left(1 + \beta\cdot\frac{M_I}{M_{II}}, 1 + \beta\right) = 1.753$$

SNiP 2.03.01-84* 3.6 (21)

Random eccentricity of longitudinal force

$$e_a := \max\left(\frac{h}{30}, \frac{l_0}{600}\right) = 0.03\,m$$

SNiP 2.03.01-84* 1.21

Eccentricity of longitudinal force

$$e_0 := \frac{M}{N_t} + e_a = 0.099\,m$$

δ_e must not be less than $\delta_{e.min}$

$$\delta_{e,min} := 0.5 - 0.01\cdot\frac{l_0}{h} - 0.01\cdot\frac{R_b}{1\frac{N}{mm^2}} = -0.013$$

SNiP 2.03.01-84* 3.6 (22)

$$\delta_e := \max\left(\frac{e_0}{h}, \delta_{e,min}\right) = 0.199$$

Guide to SNiP 2.03.01-84* 3.54

Assume that reinforcing ratio μ = 0.0095

$$\mu := 0.0095$$

$$\alpha := \frac{E_s}{E_b} = 7.407$$

Conditional critical force

Guide to SNiP 2.03.01-84* 3.54 (93)

SNiP 2.03.01-84* 3.6 (20)

$$N_{cr} := \frac{1.6\cdot E_b\cdot b\cdot h}{\left(\frac{l_0}{h}\right)^2}\cdot\left[\frac{\frac{0.11}{0.1 + \delta_e} + 0.1}{3\phi_l} + \mu\cdot\alpha\cdot\left(\frac{h_0 - a}{h}\right)^2\right] = 1.117 \times 10^3\cdot kN$$

Coefficient accounting for the effect of bend on the eccentricity of longitudinal force

$$\eta := \frac{1}{1 - \frac{N_t}{N_{cr}}} = 4.418$$

SNiP 2.03.01-84* 3.6 (19)

Distance from the point of application of longitudinal force N to the resultant of the forces in reinforcement

$$e := e_0 \cdot \eta + \frac{h_0 - a}{2} = 0.639\,\text{m}$$

Relative longitudinal force

$$\alpha_n := \frac{N_t}{R_b \cdot b \cdot h_0} = 0.251$$ Guide to SNiP 2.03.01-84* 3.62

Relative moment $N = 1 \times 10^{-3} \cdot \text{kN}$

$$\alpha_{m1} := \frac{N_t \cdot e}{R_b \cdot b \cdot h_0^2} = 0.357$$ Guide to SNiP 2.03.01-84* 3.62

$$\delta := \frac{a}{h_0} = 0.111$$ Guide to SNiP 2.03.01-84* 3.62

$\alpha := 0.85$ 0.85 for heavy-weight concrete SNiP 2.03.01-84* 3.12*

Characteristic of concrete compressed area

$$\omega := \alpha - 0.008 \cdot \frac{R_b}{1\text{MPa}} = 0.728$$ SNiP 2.03.01-84* 3.12* (26)

The ultimate stress in reinforcement of compressed zone, taken for structures from heavy-weight, fine-grained, and light-weight concrete in terms of design loads

500 N/mm^2 if γ_{b2} = 0.9 else 400 N/mm^2

$$\sigma_{sc.u} := \text{if}\left(\gamma_{b2} = 0.9, 500\frac{N}{mm^2}, 400\frac{N}{mm^2}\right) = 500 \cdot \frac{N}{mm^2}$$

Limit value of relative height of the compressed zone

$$\xi_R := \frac{\omega}{1 + \frac{R_s}{\sigma_{sc.u}} \cdot \left(1 - \frac{\omega}{1.1}\right)} = 0.573$$ SNiP 2.03.01-84* 3.12* (25)

$$\alpha_s := \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta} = 0.154$$ Guide to SNiP 2.03.01-84* 3.62 (114)

Relative height of the compressed zone

$$\xi := \frac{\alpha_n \cdot (1 - \xi_R) + 2 \cdot \alpha_s \cdot \xi_R}{1 - \xi_R + 2 \cdot \alpha_s} = 0.386$$ Guide to SNiP 2.03.01-84* 3.61 (109)

Area of compressed or tensile reinforcement (for symmetric arrangement if reinforcement bars

Guide to SNiP 2.03.01-84* 3.62 (112, 113)

$$A_s := \mathrm{if}\left[\alpha_n < \xi_R, \frac{R_b \cdot b \cdot h_0}{R_s} \cdot \frac{\alpha_{m1} - \alpha_n \cdot \left(1 - \frac{\alpha_n}{2}\right)}{1 - \delta}, \frac{R_b \cdot b \cdot h_0}{R_s} \cdot \frac{\alpha_{m1} - \xi \cdot \left(1 - \frac{\xi}{2}\right)}{1 - \delta}\right] = 13.273 \cdot cm^2$$

Actual reinforcing ratio

$$\mu_{actual} := \frac{A_s \cdot 2}{b \cdot h} = 0.011$$

$$\mu_{actual} < \mu = 0$$

Gyration radius

$$i := \frac{h}{\sqrt{12}} = 0.144\,m$$

Slenderness ratio

$$\lambda := \frac{l_0}{i} = 124.708$$

Minimal reinforcing ratio Guide to SNiP 2.03.01-84* Table 47

$$\mu_{minimal} := \frac{\mathrm{if}(\lambda < 17, 0.1, \mathrm{if}(\lambda \geq 17 \wedge \lambda \leq 35, 0.2, \mathrm{if}(\lambda > 35 \wedge \lambda \leq 83, 0.4, 0.5)))}{100} = 5 \times 10^{-3}$$

Accept reinforcement area not less than the minimal value

Total reinforcement area

$$A_{total} := \mathrm{if}\left(\mu_{actual} < \mu_{minimal}, \mu_{minimal} \cdot b \cdot h_0, 2 \cdot A_s\right) = 26.546 \cdot cm^2$$

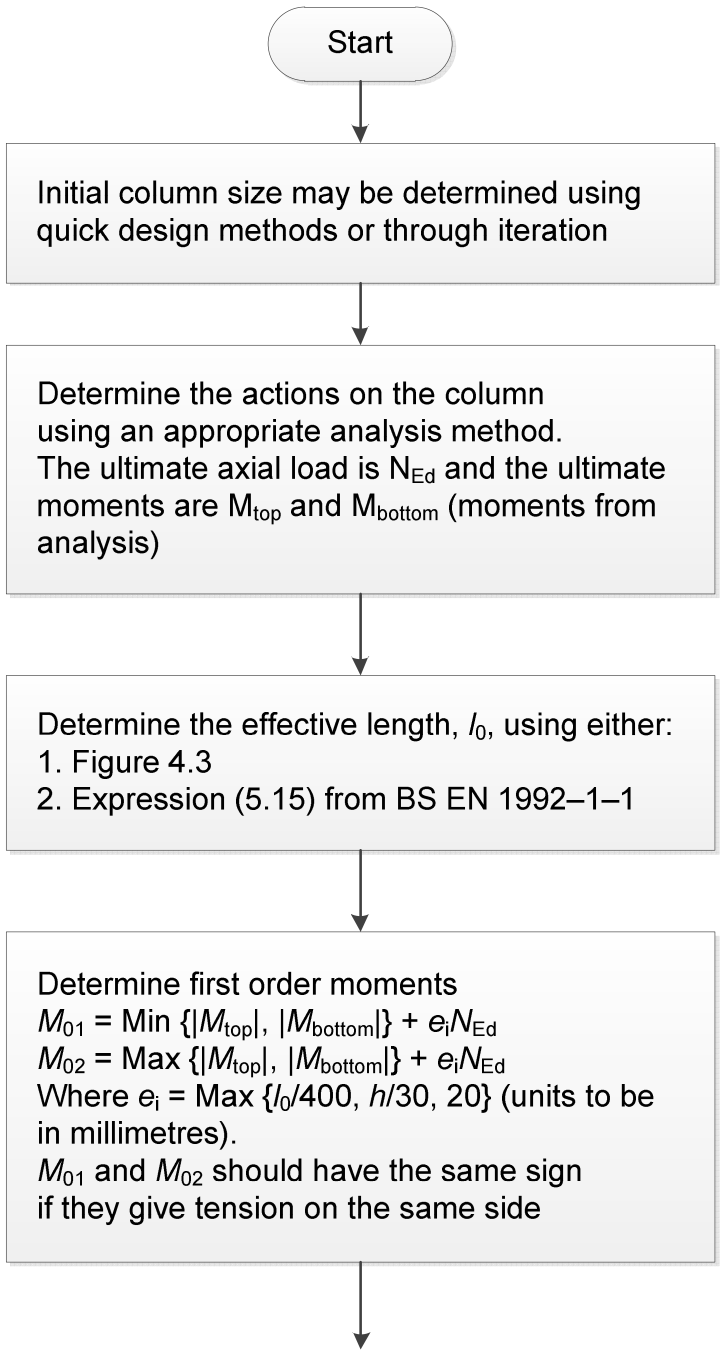


Figure 1 Flowchart for column design according to EN 1992

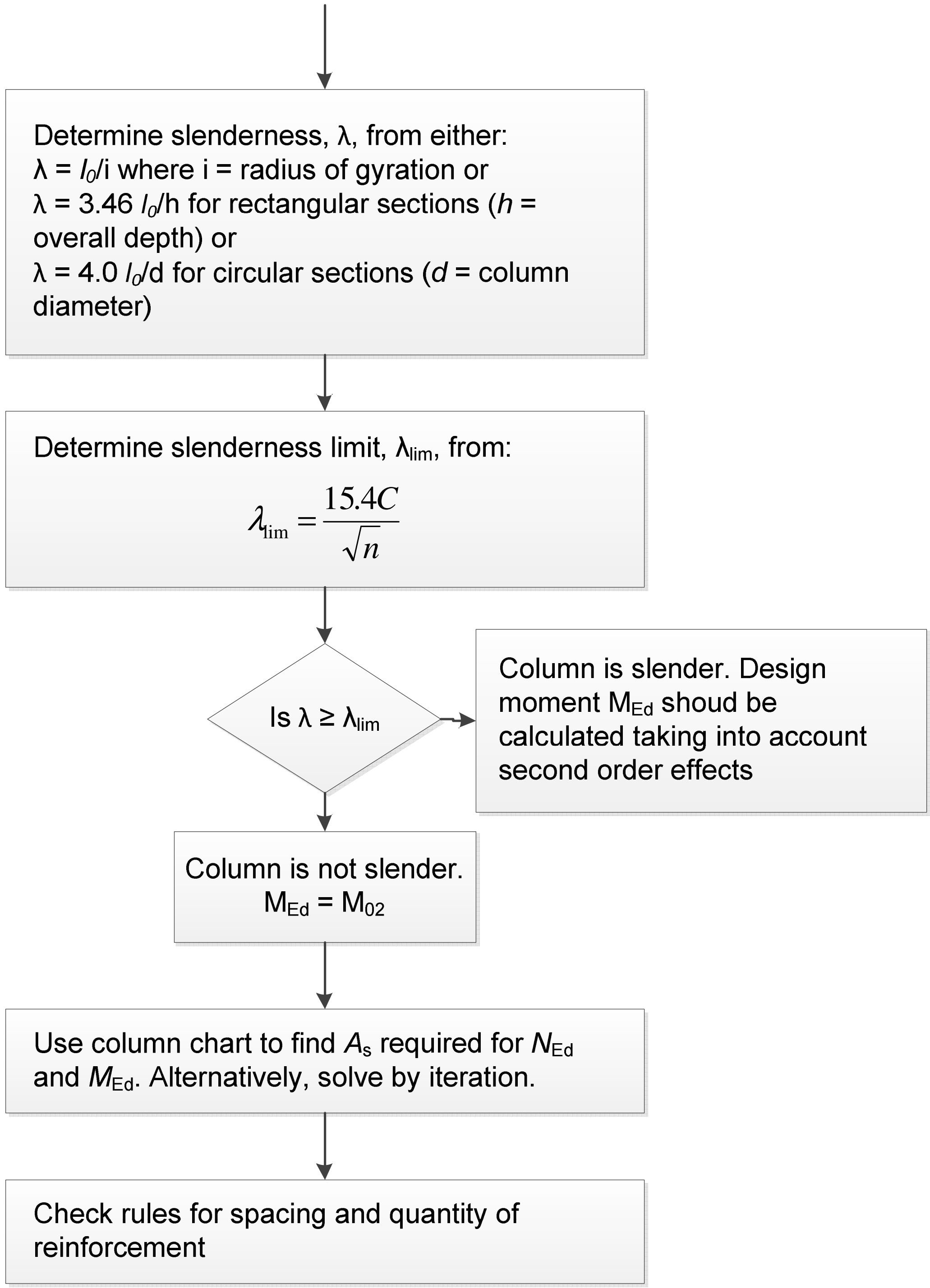


Figure 1 Flowchart for column design according to EN 1992 (continuation)

Reinforcement area calculation according to EN 1992 for exterior column

Column effective length

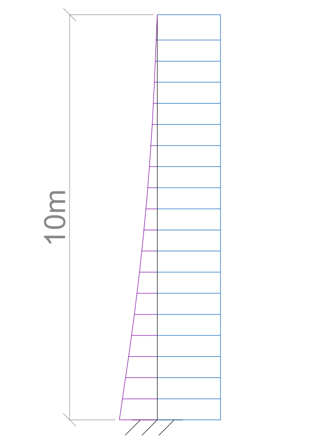
$l_0 := 2 \cdot 10m = 20m$ Designers' Guide to EN 2-1-1 Table 7.1

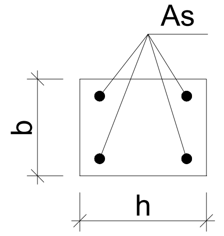
Characteristic cylinder strength of concrete

$f_{ck} := 25 \dfrac{N}{mm^2}$ Accept concrete class B30 according to Guide to SNiP 2.03.01-84* Table 8 which approximately corresponds to the class C25/30 (EN 2-1-1 Table 3.1)

$f_{ck.cube} := 30 \dfrac{N}{mm^2}$

Exterior column





Yield strength of reinforcement

$f_{yk} := 400 \dfrac{N}{mm^2}$

Elastic modulus of reinforcing steel

$E_s := 200 \dfrac{kN}{mm^2}$ EN 1992-1-1 3.2.7 (4)

Breadth of column section

$b := 0.4m$

Depth of the column section

$h := 0.5m$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$d_2 := 50mm$

Bending moment

$M := 53.7kN \cdot m$

Longitudinal force

$N_{Ed} := 306.3kN$

Partial safety factors for ULS EN 1992-1-1 Table 2.1N

For concrete

$\gamma_c := 1.5$

For reinforcing steel

$\gamma_s := 1.15$

Coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied

$\alpha_{cc} := 0.85$ Designer' guide to EN-1992-1-2 4.1.5

Design cylinder strength of concrete

$f_{cd} := \alpha_{cc} \cdot \dfrac{f_{ck}}{\gamma_c} = 14.167 \cdot \dfrac{N}{mm^2}$ Designer' guide to EN-1992-1-2 4.1.5

Design yield strength of the reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 347.826 \cdot \frac{N}{mm^2}$$

Eccentricity as the effect of imperfections

$$e := \max\left(20mm, \frac{h}{30}, \frac{l_0}{400}\right) = 0.05\,m$$

The area of concrete

$$A_c := b \cdot h$$

Effective depth to reinforcement

$$d := h - d_2 = 0.45\,m$$

Column slenderness

$$\lambda := \frac{3.46 l_0}{h} = 138.4$$

$$r_m := 0$$

$$C := 1.7 - r_m = 1.7$$

Relative axial force

$$n := \frac{N_{Ed}}{A_c \cdot f_{cd}} = 0.108$$

Limit slenderness

$$\lambda_{lim} := \frac{15.4 \cdot C}{\sqrt{n}} = 79.624$$

$\lambda < \lambda_{lim} = 0$ if = 0, then the column is slender

if = 1, then the column is not slender

Calculating the column as slender

Estimated total area of steel

$$A_{s.est} := 0.006 \cdot A_c = 12 \cdot cm^2$$

Mechanical reinforcement ratio

$$\omega := \frac{A_{s.est} \cdot f_{yd}}{A_c \cdot f_{cd}} = 0.147$$

$$n_u := 1 + \omega = 1.147$$

$n_{bal} := 0.4$ The value of n at maximum moment resistance

Correction factor depending on axial load

$$K_r := \min\left[\frac{(n_u - n)}{n_u - n_{bal}}, 1\right] = 1$$ EN2-1-1 (5.36)

$$\beta := 0.35 + \frac{f_{ck}}{200MPa} - \frac{\lambda}{150} = -0.448$$

Factor for taking into account of creep

$K_\varphi := 1$ EN2-1-1 (5.37)

Deflection due to second order effects

$$e_2 := 0.1 \cdot \left(\frac{K_r \cdot K_\varphi \cdot f_{yd}}{0.45 \cdot d \cdot E_s}\right) \cdot l_0^{\,2} = 0.344\,m$$

First order moment including the effect of imperfections

$$M_{02} := M + N_{Ed} \cdot e = 69.015 \cdot kN \cdot m$$

Nominal second order moment M_2

$M_2 := N_{Ed} \cdot e_2 = 105.224 \cdot kN \cdot m$ EN 1992-1-1 (5.33)

Design moment

$M_{Ed} := M_2 + M_{02} = 174.239 \cdot kN \cdot m$ EN 1992-1-1 (5.31)

$$\frac{M_{Ed}}{b \cdot h^2 \cdot f_{ck}} = 0.07$$

These ratios are used in determination of α_s coefficient using the design curves

$$\frac{N_{Ed}}{b \cdot h \cdot f_{ck}} = 0.061$$

$$\frac{d_2}{h} = 0.1$$

$\alpha_s := 0.11$ According to curve the Figure 9 of How to design concrete structures using Eurocode 2. Columns

Area of reinforcement

$$A_s := \alpha_s \cdot b \cdot h \cdot \frac{f_{ck}}{f_{yk}} = 13.75 \cdot cm^2$$

Minimal area of reinforcement

$$A_{s.min} := \max\left(0.12 \cdot \frac{N}{f_{yk}}, 0.002 \cdot b \cdot h\right) = 4 \cdot cm^2$$

Maximum area of reinforcement

$$A_{s.max} := 0.04 \cdot b \cdot h = 80 \cdot cm^2$$

Also size of longitudinal reinforcement bars should not be less than 12mm

Minimal number of longitudinal reinforcement bars in rectangular columns is 4

$$A_{s.4x12mm} := \frac{\pi \cdot (12mm)^2}{4} \cdot 4 = 4.524 \cdot cm^2$$

Accept reinforcement area not less than the minimal value

$$A_s := \max\left(A_{s.min}, A_s, A_{s.4x12mm}\right) = 13.75 \cdot cm^2$$

Accept reinforcement area no more than the maximum value

$$A_s := \min\left(A_s, A_{s.max}\right) = 13.75 \cdot cm^2$$

Reinforcement area

$$A_s = 13.75 \cdot cm^2$$

Reinforcement area calculation for interior column according to EN 1992

Column effective length

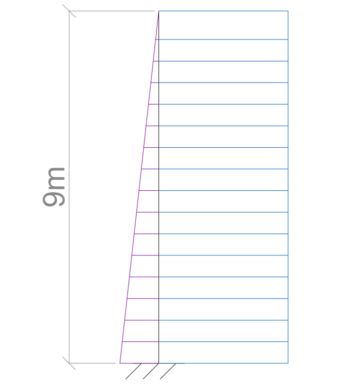
$l_0 := 2 \cdot 9m = 18m$ Designers' Guide to EN 2-1-1 Table 7.1

Characteristic cylinder strength of concrete

$f_{ck} := 25 \frac{N}{mm^2}$ Accept concrete class B30 according to Guide to SNiP 2.03.01-84* Table 8 which approximately corresponds to the class C25/30 (EN 2-1-1 Table 3.1)

$f_{ck.cube} := 30 \frac{N}{mm^2}$

Interior column



Yield strength of reinforcement

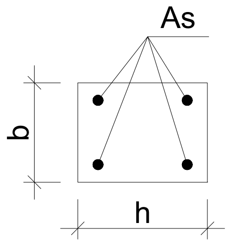
$f_{yk} := 400 \frac{N}{mm^2}$

Elastic modulus of reinforcing steel

$E_s := 200 \frac{kN}{mm^2}$ EN 1992-1-1 3.2.7 (4)

Breadth of column section

$b := 0.5m$



Depth of the column section

$h := 0.5m$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$d_2 := 50mm$

Bending moment

$M := 58.8kN \cdot m$

Longitudinal force

$N_{Ed} := 1304.2kN$

Partial safety factors for ULS EN 1992-1-1 Table 2.1N

For concrete

$\gamma_c := 1.5$

For reinforcing steel

$\gamma_s := 1.15$

Coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied

$\alpha_{cc} := 0.85$ Designer' guide to EN-1992-1-2 4.1.5

Design cylinder strength of concrete

$f_{cd} := \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 14.167 \cdot \frac{N}{mm^2}$ Designer' guide to EN-1992-1-2 4.1.5

Design yield strength of the reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 347.826 \cdot \frac{N}{mm^2}$$

Eccentricity as the effect of imperfections

$$e := \max\left(20mm, \frac{h}{30}, \frac{l_0}{400}\right) = 0.045\,m$$

The area of concrete

$$A_c := b \cdot h$$

Effective depth to reinforcement

$$d := h - d_2 = 0.45\,m$$

Column slenderness

$$\lambda := \frac{3.46 l_0}{h} = 124.56$$

$$r_m := 0$$

$$C := 1.7 - r_m = 1.7$$

Relative axial force

$$n := \frac{N_{Ed}}{A_c \cdot f_{cd}} = 0.368$$

Limit slenderness

$$\lambda_{lim} := \frac{15.4 \cdot C}{\sqrt{n}} = 43.142$$

$\lambda < \lambda_{lim} = 0$ if = 0, then the column is slender

if = 1, then the column is not slender

Calculating the column as slender

Estimated total area of steel

$$A_{s.est} := 0.015 \cdot A_c = 37.5 \cdot cm^2$$

Mechanical reinforcement ratio

$$\omega := \frac{A_{s.est} \cdot f_{yd}}{A_c \cdot f_{cd}} = 0.368$$

$$n_u := 1 + \omega = 1.368$$

$n_{bal} := 0.4$ The value of n at maximum moment resistance

Correction factor depending on axial load

$$K_r := \min\left[\frac{(n_u - n)}{n_u - n_{bal}}, 1\right] = 1$$ EN2-1-1 (5.36)

$$\beta := 0.35 + \frac{f_{ck}}{200\frac{N}{mm^2}} - \frac{\lambda}{150} = -0.355$$

Factor for taking into account of creep

$K_\varphi := 1$ EN2-1-1 (5.37)

Deflection due to second order effects

$$e_2 := 0.1\cdot\left(\frac{K_r\cdot K_\varphi\cdot f_{yd}}{0.45\cdot d\cdot E_s}\right)\cdot l_0^{\,2} = 0.278\,m$$

First order moment including the effect of imperfections

$$M_{02} := M + N_{Ed}\cdot e = 117.489\cdot kN\cdot m$$

Nominal second order moment M_2

$$M_2 := N_{Ed}\cdot e_2 = 362.908\cdot kN\cdot m$$ EN 1992-1-1 (5.33)

Design moment

$$M_{Ed} := M_2 + M_{02} = 480.397\cdot kN\cdot m$$ EN 1992-1-1 (5.31)

$$\frac{M_{Ed}}{b\cdot h^2\cdot f_{ck}} = 0.154$$

These ratios are used in determination of α_s coefficient using the design curves

$$\frac{N_{Ed}}{b\cdot h\cdot f_{ck}} = 0.209$$

$$\frac{d_2}{h} = 0.1$$

$\alpha_s := 0.25$ According to curve the Figure 9 of How to design concrete structures using Eurocode 2. Columns

Area of reinforcement

$$A_s := \alpha_s\cdot b\cdot h\cdot\frac{f_{ck}}{f_{yk}} = 39.063\cdot cm^2$$

Minimal area of reinforcement

$$A_{s.min} := \max\left(0.12\cdot\frac{N}{f_{yk}}, 0.002\cdot b\cdot h\right) = 5\cdot cm^2$$

Maximum area of reinforcement

$$A_{s.max} := 0.04\cdot b\cdot h = 100\cdot cm^2$$

Also size of longitudinal reinforcement bars should not be less than 12mm

Minimal number of longitudinal reinforcement bars in rectangular columns is 4

$$A_{s.4x12mm} := \frac{\pi \cdot (12mm)^2}{4} \cdot 4 = 4.524 \cdot cm^2$$

Accept reinforcement area not less than the minimal value

$$A_s := \max\left(A_{s.min}, A_s, A_{s.4x12mm}\right) = 39.063 \cdot cm^2$$

Accept reinforcement area no more than the maximum value

$$A_s := \min\left(A_s, A_{s.max}\right) = 39.063 \cdot cm^2$$

Reinforcement area

$$A_s = 39.063 \cdot cm^2$$

Reinforcement area calculation according to EN 1992 for exterior column (loads according to SNiP)

Column effective length

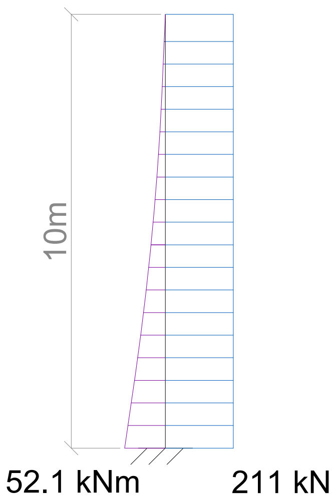
$l_0 := 2 \cdot 10\text{m} = 20\text{m}$ Designers' Guide to EN 2-1-1 Table 7.1

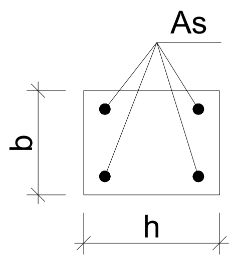
Characteristic cylinder strength of concrete

$f_{ck} := 25\dfrac{\text{N}}{\text{mm}^2}$ Accept concrete class B30 according to Guide to SNiP 2.03.01-84* Table 8 which approximately corresponds to the class C25/30 (EN 2-1-1 Table 3.1)

$f_{ck.cube} := 30\dfrac{\text{N}}{\text{mm}^2}$

Exterior column





Yield strength of reinforcement

$f_{yk} := 400\dfrac{\text{N}}{\text{mm}^2}$

Elastic modulus of reinforcing steel

$E_s := 200\dfrac{\text{kN}}{\text{mm}^2}$ EN 1992-1-1 3.2.7 (4)

Breadth of column section

$b := 0.4\text{m}$

Depth of the column section

$h := 0.5\text{m}$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$d_2 := 50\text{mm}$

Bending moment

$M := 52.1\text{kN}\cdot\text{m}$

Longitudinal force

$N_{Ed} := 211\text{kN}$

Partial safety factors for ULS EN 1992-1-1 Table 2.1N

For concrete

$\gamma_c := 1.5$

For reinforcing steel

$\gamma_s := 1.15$

Coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied

$\alpha_{cc} := 0.85$ Designer' guide to EN-1992-1-2 4.1.5

Design cylinder strength of concrete

$f_{cd} := \alpha_{cc} \cdot \dfrac{f_{ck}}{\gamma_c} = 14.167 \cdot \dfrac{\text{N}}{\text{mm}^2}$ Designer' guide to EN-1992-1-2 4.1.5

Design yield strength of the reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 347.826 \cdot \frac{N}{mm^2}$$

Eccentricity as the effect of imperfections

$$e := \max\left(20mm, \frac{h}{30}, \frac{l_0}{400}\right) = 0.05\,m$$

The area of concrete

$$A_c := b \cdot h$$

Effective depth to reinforcement

$$d := h - d_2 = 0.45\,m$$

Column slenderness

$$\lambda := \frac{3.46 l_0}{h} = 138.4$$

$$r_m := 0$$

$$C := 1.7 - r_m = 1.7$$

Relative axial force

$$n := \frac{N_{Ed}}{A_c \cdot f_{cd}} = 0.074$$

Limit slenderness

$$\lambda_{lim} := \frac{15.4 \cdot C}{\sqrt{n}} = 95.935$$

$\lambda < \lambda_{lim} = 0$ if = 0, then the column is slender

if = 1, then the column is not slender

Calculating the column as slender

Estimated total area of steel

$$A_{s.est} := 0.006 \cdot A_c = 12 \cdot cm^2$$

Mechanical reinforcement ratio

$$\omega := \frac{A_{s.est} \cdot f_{yd}}{A_c \cdot f_{cd}} = 0.147$$

$$n_u := 1 + \omega = 1.147$$

$n_{bal} := 0.4$ The value of n at maximum moment resistance

Correction factor depending on axial load

$$K_r := \min\left[\frac{(n_u - n)}{n_u - n_{bal}}, 1\right] = 1$$ EN2-1-1 (5.36)

$$\beta := 0.35 + \frac{f_{ck}}{200\frac{N}{mm^2}} - \frac{\lambda}{150} = -0.448$$

Factor for taking into account of creep

$K_\varphi := 1$ EN2-1-1 (5.37)

Deflection due to second order effects

$$e_2 := 0.1 \cdot \left(\frac{K_r \cdot K_\varphi \cdot f_{yd}}{0.45 \cdot d \cdot E_s}\right) \cdot l_0^{\,2} = 0.344\,m$$

First order moment including the effect of imperfections

$$M_{02} := M + N_{Ed} \cdot e = 62.65 \cdot kN \cdot m$$

Nominal second order moment M_2

$M_2 := N_{Ed} \cdot e_2 = 72.485 \cdot kN \cdot m$ EN 1992-1-1 (5.33)

Design moment

$M_{Ed} := M_2 + M_{02} = 135.135 \cdot kN \cdot m$ EN 1992-1-1 (5.31)

$$\frac{M_{Ed}}{b \cdot h^2 \cdot f_{ck}} = 0.054$$

These ratios are used in determination of α_s coefficient using the design curves

$$\frac{N_{Ed}}{b \cdot h \cdot f_{ck}} = 0.042$$

$$\frac{d_2}{h} = 0.1$$

$\alpha_s := 0.1$ According to curve the Figure 9 of How to design concrete structures using Eurocode 2. Columns

Area of reinforcement

$$A_s := \alpha_s \cdot b \cdot h \cdot \frac{f_{ck}}{f_{yk}} = 12.5 \cdot cm^2$$

Minimal area of reinforcement

$$A_{s.min} := \max\left(0.12 \cdot \frac{N}{f_{yk}}, 0.002 \cdot b \cdot h\right) = 4 \cdot cm^2$$

Maximum area of reinforcement

$$A_{s.max} := 0.04 \cdot b \cdot h = 80 \cdot cm^2$$

Also size of longitudinal reinforcement bars should not be less than 12mm

Minimal number of longitudinal reinforcement bars in rectangular columns is 4

$$A_{s.4x12mm} := \frac{\pi \cdot (12mm)^2}{4} \cdot 4 = 4.524 \cdot cm^2$$

Accept reinforcement area not less than the minimal value

$$A_s := \max\left(A_{s.min}, A_s, A_{s.4x12mm}\right) = 12.5 \cdot cm^2$$

Accept reinforcement area no more than the maximum value

$$A_s := \min\left(A_s, A_{s.max}\right) = 12.5 \cdot cm^2$$

Reinforcement area

$$A_s = 12.5 \cdot cm^2$$

Reinforcement area calculation for interior column according to EN 1992 (loads according to SNiP)

Column effective length

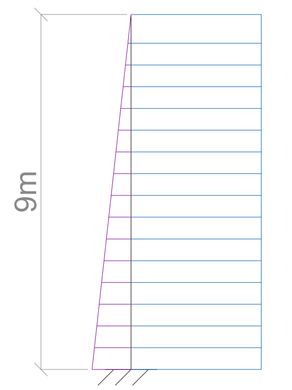
$l_0 := 2 \cdot 9m = 18m$ Designers' Guide to EN 2-1-1 Table 7.1

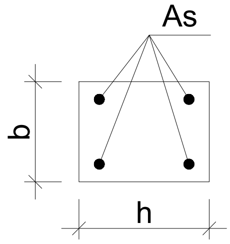
Characteristic cylinder strength of concrete

$f_{ck} := 25 \frac{N}{mm^2}$ Accept concrete class B30 according to Guide to SNiP 2.03.01-84* Table 8 which approximately corresponds to the class C25/30 (EN 2-1-1 Table 3.1)

$f_{ck.cube} := 30 \frac{N}{mm^2}$

Interior column





Yield strength of reinforcement

$f_{yk} := 400 \frac{N}{mm^2}$

Elastic modulus of reinforcing steel

$E_s := 200 \frac{kN}{mm^2}$ EN 1992-1-1 3.2.7 (4)

Breadth of column section

$b := 0.5m$

Depth of the column section

$h := 0.5m$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$d_2 := 50mm$

Bending moment

$M := 60kN \cdot m$

Longitudinal force

$N_{Ed} := 864kN$

Partial safety factors for ULS EN 1992-1-1 Table 2.1N

For concrete

$\gamma_c := 1.5$

For reinforcing steel

$\gamma_s := 1.15$

Coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied

$\alpha_{cc} := 0.85$ Designer' guide to EN-1992-1-2 4.1.5

Design cylinder strength of concrete

$f_{cd} := \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 14.167 \cdot \frac{N}{mm^2}$ Designer' guide to EN-1992-1-2 4.1.5

Design yield strength of the reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 347.826 \cdot \text{MPa}$$

Eccentricity as the effect of imperfections

$$e := \max\left(20\text{mm}, \frac{h}{30}, \frac{l_0}{400}\right) = 0.045\,\text{m}$$

The area of concrete

$$A_c := b \cdot h$$

Effective depth to reinforcement

$$d := h - d_2 = 0.45\,\text{m}$$

Column slenderness

$$\lambda := \frac{3.46 l_0}{h} = 124.56$$

$$r_m := 0$$

$$C := 1.7 - r_m = 1.7$$

Relative axial force

$$n := \frac{N_{Ed}}{A_c \cdot f_{cd}} = 0.244$$

Limit slenderness

$$\lambda_{lim} := \frac{15.4 \cdot C}{\sqrt{n}} = 53.005$$

$\lambda < \lambda_{lim} = 0$ if = 0, then the column is slender

if = 1, then the column is not slender

Calculating the column as slender

Estimated total area of steel

$$A_{s.est} := 0.011 \cdot A_c = 27.5 \cdot \text{cm}^2$$

Mechanical reinforcement ratio

$$\omega := \frac{A_{s.est} \cdot f_{yd}}{A_c \cdot f_{cd}} = 0.27$$

$$n_u := 1 + \omega = 1.27$$

$n_{bal} := 0.4$ The value of n at maximum moment resistance

Correction factor depending on axial load

$$K_r := \min\left[\frac{(n_u - n)}{n_u - n_{bal}}, 1\right] = 1$$ EN2-1-1 (5.36)

$$\beta := 0.35 + \frac{f_{ck}}{200MPa} - \frac{\lambda}{150} = -0.355$$

Factor for taking into account of creep

$$K_\varphi := 1$$ EN2-1-1 (5.37)

Deflection due to second order effects

$$e_2 := 0.1 \cdot \left(\frac{K_r \cdot K_\varphi \cdot f_{yd}}{0.45 \cdot d \cdot E_s}\right) \cdot l_0^{\,2} = 0.278\,m$$

First order moment including the effect of imperfections

$$M_{02} := M + N_{Ed} \cdot e = 98.88 \cdot kN \cdot m$$

Nominal second order moment M_2

$$M_2 := N_{Ed} \cdot e_2 = 240.417 \cdot kN \cdot m$$ EN 1992-1-1 (5.33)

Design moment

$$M_{Ed} := M_2 + M_{02} = 339.297 \cdot kN \cdot m$$ EN 1992-1-1 (5.31)

$$\frac{M_{Ed}}{b \cdot h^2 \cdot f_{ck}} = 0.109$$

These ratios are used in determination of α_s coefficient using the design curves

$$\frac{N_{Ed}}{b \cdot h \cdot f_{ck}} = 0.138$$

$$\frac{d_2}{h} = 0.1$$

$$\alpha_s := 0.18$$ According to curve the Figure 9 of How to design concrete structures using Eurocode 2. Columns

Area of reinforcement

$$A_s := \alpha_s \cdot b \cdot h \cdot \frac{f_{ck}}{f_{yk}} = 28.125 \cdot cm^2$$

Minimal area of reinforcement

$$A_{s.min} := \max\left(0.12 \cdot \frac{N}{f_{yk}}, 0.002 \cdot b \cdot h\right) = 5 \cdot cm^2$$

Maximum area of reinforcement

$$A_{s.max} := 0.04 \cdot b \cdot h = 100 \cdot cm^2$$

Also size of longitudinal reinforcement bars should not be less than 12mm

Minimal number of longitudinal reinforcement bars in rectangular columns is 4

$$A_{s.4x12mm} := \frac{\pi \cdot (12mm)^2}{4} \cdot 4 = 4.524 \cdot cm^2$$

Accept reinforcement area not less than the minimal value

$$A_s := \max\left(A_{s.min}, A_s, A_{s.4x12mm}\right) = 28.125 \cdot cm^2$$

Accept reinforcement area no more than the maximum value

$$A_s := \min\left(A_s, A_{s.max}\right) = 28.125 \cdot cm^2$$

Reinforcement area

$$A_s = 28.125 \cdot cm^2$$

Dimensioning of the foundation under interior column according to SNiP 2.02.01-83*

Initial data

Standart depth of seasonal frost penetration into soil

$$d_{fn} := 120cm$$

Coefficient accounting for the heat flow regime of the facility

$$k_h := 1.1$$

Design depth of seasonal frost penetration into soil

$$d_f := d_{fn} \cdot k_h = 1.32\,m$$ SNiP 2.03.01-84* 2.27 (3)

$$d := \frac{\mathrm{ceil}\left(\frac{d_f}{1m} \cdot 10\right) \cdot 1m}{10} = 1.4\,m$$

Design resistance of the reinforcement to tension for limiting states of the first group

$$R_s := 365 \frac{N}{mm^2}$$

Design resistance of the concrete to axial tension for limiting states of the first group

$$R_{bt} := 1.05 \frac{N}{mm^2}$$

Design resistance of the concrete to axial compression for limiting states of the first group

$$R_b := 14.5 \frac{N}{mm^2}$$

Bending moment

$$M := 60kN \cdot m$$

Longitudinal force

$$N_t := 909kN$$

Minimal longitudinal force

$$N_{min} := 271kN$$

Average specific weight of the soil and concrete above the bottom of foundation

$$\gamma_{mt} := 20 \frac{kN}{m^3}$$

Design value of base soil resistance

$$R_0 := 200 \frac{kN}{m^2}$$

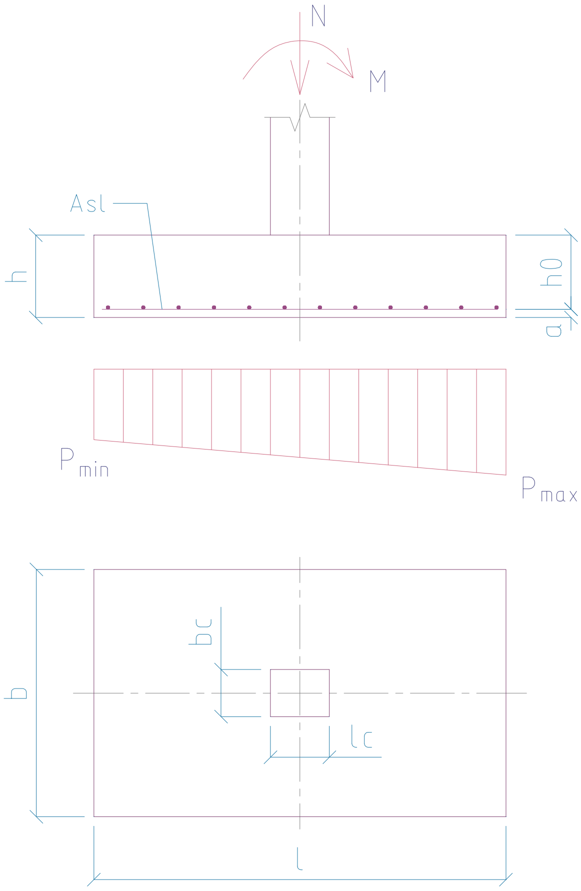


Figure 1. Eccentrically loaded foundation

Coefficient for calculation of base soil design resistance R

$k_1 := 0.05$

Breadth of column section

$b_c := 0.4m$

Depth of column section

$l_c := 0.5m$

Distance from the resultant of forces in the reinforcement to the nearest edge of the section

$a := 70mm$ SNiP 2.03.01-84* 5.5

Required foundation bottom area

$$A := \frac{1.4 \cdot N_t}{R_0 - \gamma_{mt} \cdot d} = 7.4 \cdot m^2$$

Correlation of foundation bottom dimensions

$k = \frac{l}{b}$ l and b - foundation bottom length and width, correspondingly

$k := 1.2$

$$b := \sqrt{\frac{A}{k}} = 2.48\,m$$

Foundation bottom width

$$b := \frac{\mathrm{ceil}\left(\frac{b}{1m} \cdot 10\right) \cdot 1m}{10} = 2.5\,m$$

Foundation bottom length

$$l := \frac{\mathrm{ceil}\left(\frac{\frac{A}{b}}{1m} \cdot 10\right) \cdot 1m}{10} = 3\,m$$

Foundation bottom area

$$A := l \cdot b = 7.5\,m^2$$

$d \le 2m = 1$

$b_0 := 1m$ $d_0 := 2m$

Base soil design resistance

$$R := R_0 \cdot \left[1 + k_1 \cdot \frac{(b - b_0)}{b_0}\right] \cdot \frac{(d + d_0)}{2 \cdot d_0} = 182.75 \cdot \frac{kN}{m^2}$$ SNiP 2.02.01-83* Appendix 3 (1)

Mean pressure under foundation

$$p := \frac{N_t + \gamma_{mt} \cdot A \cdot d}{A} = 149.2 \cdot \frac{kN}{m^2}$$

$$p < R = 1$$

Eccentricity

$$e := \frac{M}{N_t + \gamma_{mt} \cdot d \cdot l \cdot b} = 0.05\,m$$

$$\frac{e}{l} \le \frac{1}{6} = 1$$ else see Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84*

Resisting moment of the foundation

$$W := \frac{b \cdot l^2}{6} = 3.75 \cdot m^3$$

Maximum and minimum pressures under the foundation

$$p_{max} := \frac{N_t}{A} + \gamma_{mt} \cdot d + \frac{M}{W} = 165.2 \cdot \frac{kN}{m^2}$$

$$p_{min} := \frac{N_{min}}{A} + \gamma_{mt} \cdot d - \frac{M}{W} = 48.13 \cdot \frac{kN}{m^2}$$

Pressure checks

$$p_{max} \le 1.2R = 1$$

$$p_{min} > 0 = 1$$

Coefficients for the calculation of effective height of the foundation slab

$$c_l := 0.5 \cdot \left(l - l_c\right) = 1.25\,m$$

$$c_b := 0.5\left(b - b_c\right) = 1.05\,m$$

$$r := \frac{R_{bt}}{p_{max}} = 6.36$$

Effective height of foundation slab

$$h_{0.pl} := -0.5 \cdot b_c + \sqrt{0.25 \cdot b_c^{\,2} + \frac{\left(b \cdot c_l - c_b^{\,2}\right)}{1 + r}} = 0.36\,m$$ Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84* 2.12

$$0.5 \cdot \left(b - b_c\right) > h_{0.pl} = 1$$

Accept height of foundation slab

$$h := \frac{\mathrm{ceil}\left[\frac{\left(h_{0.pl} + a\right) \cdot 10}{1m}\right]}{10} 1m = 0.5\,m$$

Punching check Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84* 2.8

Working height of the section

$h_{0.pl} := h - a = 0.43\,m$

Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84* 2.9 (4)

$$A_0 := \text{if}\left[b - b_c - 2\cdot h_{0.pl} > 0, 0.5\cdot b\cdot\left(l - l_c - 2\cdot h_{0.pl}\right) - 0.25\cdot\left(b - b_c - 2\cdot h_{0.pl}\right)^2, 0.5\cdot b\cdot\left(l - l_c - 2\cdot h_{0.pl}\right)\right] = 1.67\,m$$

Average size of the checking side

$b_m := \text{if}\left[b - b_c > 2\cdot h_{0.pl}, b_c + h_{0.pl}, 0.5\left(b + b_c\right)\right] = 0.83\,m$

Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84* 2.9 (7, 8)

Punching force

$F := A_0\cdot p_{max} = 275.16\cdot kN$

Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84* 2.9 (3)

$\alpha := 1$ For heavy weight concrete

$\alpha\cdot R_{bt}\cdot b_m\cdot h_{0.pl} = 374.75\cdot kN$

SNiP 2.03.01-84* 3.42 (107)

$F \le \alpha\cdot R_{bt}\cdot b_m\cdot h_{0.pl} = 1$

Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84* 2.8 (1)

$\frac{b}{l} > 0.5 = 1$

Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84* 2.24

Shear force checking is not necessary

Reinforcement Guide to SNiP 2.02.01-83* and SNiP 2.03.01-84* 2.31

Bending moment in section

$$M_l := \text{if}\left[e \le \frac{l}{6}, N_t\cdot c_l^2\cdot\frac{\left(1 + 6\cdot\frac{e}{l} - 4\cdot e\cdot\frac{c_l}{l^2}\right)}{2l}, \text{if}\left[e < \frac{l}{4} \land e > \frac{l}{6}, 2\cdot N_t\cdot c_l^2\cdot\frac{\left[1 - 2\cdot\frac{c_l}{9\cdot(l - 2\cdot e)}\right]}{3\cdot(l - 2\cdot e)}, 0\right]\right] = 255.05\cdot kN\cdot m$$

$\alpha_0 := \frac{M_l}{R_b\cdot b\cdot h_{0.pl}^2} = 0.04$

Relative height of the compressed zone

$\xi := 1 - \sqrt{1 - 2\cdot\alpha_0} = 0.04$

$\upsilon := 1 - 0.5\cdot\xi = 0.98$

Reinforcement area

$$A_{sl} := \frac{M_l}{R_s\cdot\upsilon\cdot h_{0.pl}} = 16.57\cdot cm^2$$

Static analysis of the primary truss

Initial data

Node force from the secondary trusses

$P := 429.6\text{kN}$

Dimensions of the truss

$l_1 := 3\text{m}$

$l_2 := 6\text{m}$

$h := 1.84\text{m}$

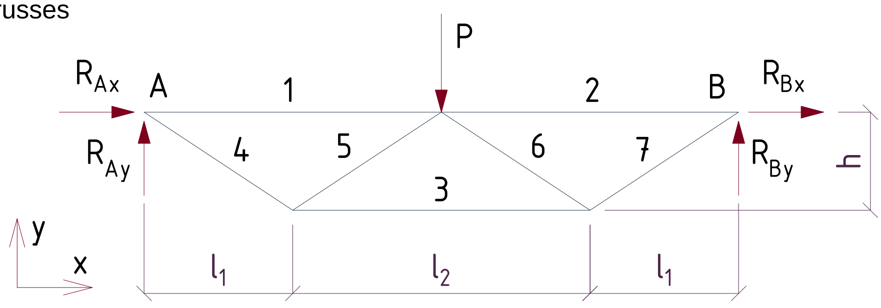


Figure 1. Primary truss

Reactions

Moment about point B

$$R_{Ay} \cdot (l_2 + 2 \cdot l_1) - P \cdot \left(\frac{l_2}{2} + l_1\right) = 0$$

$$R_{Ay} := \frac{P}{2} = 214.8 \cdot \text{kN}$$

Sum of the forces along axis X

$$\Sigma X = 0$$

$$R_{Ax} := 0$$

Sum of the forces along axis Y

$$R_{Ay} + R_{By} - P = 0$$

$$R_{By} := \frac{P}{2} = 214.8 \cdot \text{kN}$$

Member forces

The method of sections was used for the calculation

Moment about point D (see Figure 2)

$$R_{Ax} \cdot h - R_1 \cdot h + R_{Ay} \cdot l_1 = 0$$

$$R_1 := \frac{P \cdot l_1}{2 \cdot h} = 350.22 \cdot \text{kN}$$

$$R_2 := R_1 = 350.22 \cdot \text{kN}$$

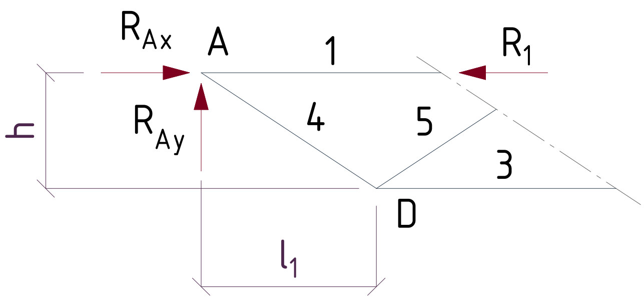


Figure 2

Sum of the forces along axis Y (see Figure 3)

$$R_{Ay} + R_4 \cdot \sin\left(\operatorname{atan}\left(\frac{h}{l_1}\right)\right) = 0$$

$$R_4 := \frac{-P}{2 \cdot \sin\left(\operatorname{atan}\left(\frac{h}{l_1}\right)\right)} = -410.84 \cdot kN$$

$$R_7 := R_4 = -410.84 \cdot kN$$

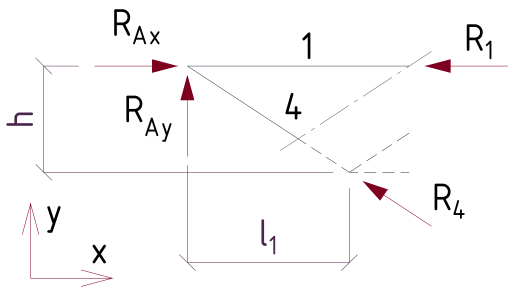


Figure 3

Moment about point E (see Figure 4)

$$R_3 \cdot h + R_{Ay} \cdot \left(l_1 + \frac{l_2}{2}\right) = 0$$

$$R_3 := \frac{-P \cdot \left(l_1 + \frac{l_2}{2}\right)}{2 \cdot h} = -700.43 \cdot kN$$

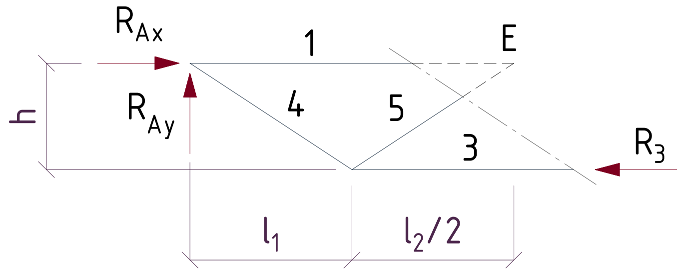


Figure 4

Sum of the forces along axis Y (see Figure 3)

$$-R_5 \cdot \sin\left(\operatorname{atan}\left(\frac{2 \cdot h}{l_2}\right)\right) + R_{Ay} = 0$$

$$R_5 := \frac{P}{2 \cdot \sin\left(\operatorname{atan}\left(\frac{2 \cdot h}{l_2}\right)\right)} = 410.84 \cdot kN$$

$$R_6 := R_5 = 410.84 \cdot kN$$

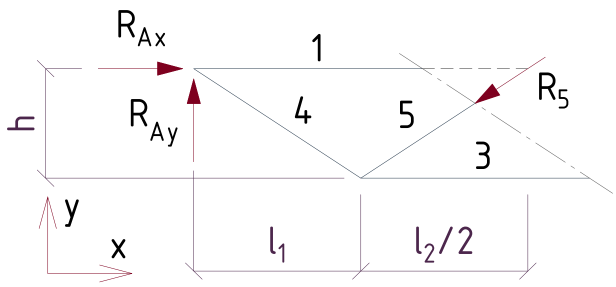


Figure 5

Results in graphical view are shown in Figure 6.

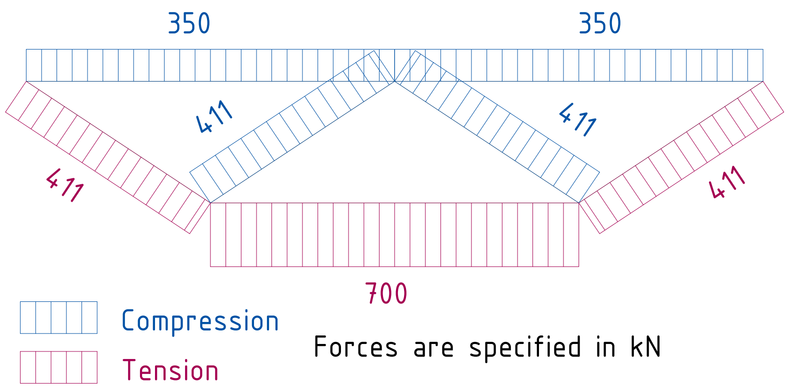


Figure 6. Stress pattern for longitudinal forces

Bending moments are insignifican in this case because the load is applied to truss joint.

Static analysis of the secondary truss

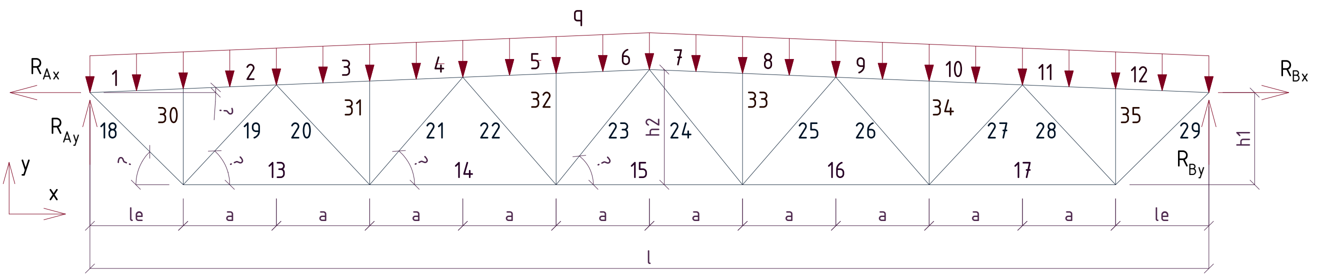


Figure 1. Secondary truss

Initial data

Uniform load at the top of the truss

$$q := 17.9\frac{kN}{m}$$

Truss dimensions

$$l := 24m$$

$$h_1 := 1.98m$$

$$h_2 := 2.42m$$

$$a := \frac{l}{12} = 2\,m$$

$$l_e := 2m$$

$$P := q \cdot a = 35.8 \cdot kN$$

Auxiliary dimensions and angles

$$\alpha := atan\left(\frac{h_2 - h_1}{\frac{l}{2}}\right) = 0.04$$

$$c_1 := l_e \cdot \tan(\alpha) = 0.07\,m$$

$$c_2 := \left(a + l_e\right) \cdot \tan(\alpha) = 0.15\,m$$

$$c_3 := \left(2 \cdot a + l_e\right) \cdot \tan(\alpha) = 0.22\,m$$

$$c_4 := \left(3 \cdot a + l_e\right) \cdot \tan(\alpha) = 0.29\,m$$

$$c_5 := \left(4 \cdot a + l_e\right) \cdot \tan(\alpha) = 0.37\,m$$

$$c_6 := \left(5 \cdot a + l_e\right) \cdot \tan(\alpha) = 0.44\,m$$

$$b_1 := \left(c_1 + h_1\right) \cdot \cos(\alpha) = 2.05\,m$$

$$b_2 := \left(h_1 + c_3\right) \cdot \cos(\alpha) = 2.2\,m$$

$$b_3 := \left(h_1 + c_5\right) \cdot \cos(\alpha) = 2.35\,m$$

$$l_1 := \frac{a}{\cos(\alpha)} = 2\,\text{m}$$

$$l_{1e} := \frac{l_e}{\cos(\alpha)} = 2\,\text{m}$$

$$\gamma := \operatorname{atan}\left(\frac{h_1}{l_e}\right) = 0.78$$

$$\delta := \operatorname{atan}\left(\frac{h_1 + c_2}{a}\right) = 0.82$$

$$\varepsilon := \operatorname{atan}\left(\frac{h_1 + c_4}{a}\right) = 0.85$$

$$\zeta := \operatorname{atan}\left(\frac{h_1 + c_6}{a}\right) = 0.88$$

Calculation of the member forces

The truss (and loads) are symmetric about its center, therefore member forces are determinated only for a half of the truss

The method of section is used

Moment about point A (see Figure 2)

$$R_{30} \cdot l_e - P \cdot l_e = 0$$

Vertical web elements

$$R_{30} := P = 35.8 \cdot \text{kN}$$

$$R_{31} := P = 35.8 \cdot \text{kN}$$

$$R_{32} := P = 35.8 \cdot \text{kN}$$

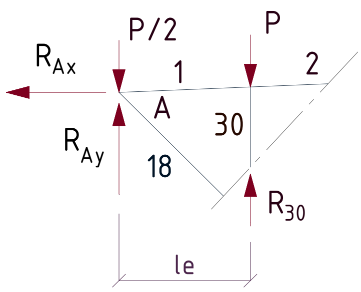


Figure 2

Moment about point C (see Figure 3)

$$R_2 \cdot b_1 + \frac{P}{2} \cdot l_e - 6P \cdot l_e = 0$$

$$R_2 := \frac{5.5P \cdot l_e}{b_1} = 191.91 \cdot \text{kN}$$

$$R_1 := R_2 = 191.91 \cdot \text{kN}$$

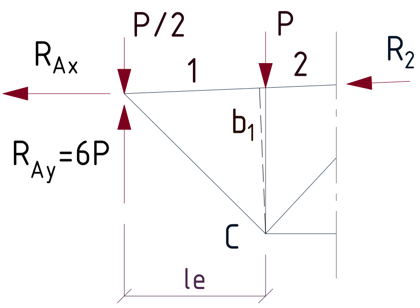


Figure 3

Sum of the forces along axis X (see Figure 4)

$$R_{18} \cdot \cos(\gamma) + R_2 \cdot \cos(\alpha) = 0$$

$$R_{18} := \frac{-R_2 \cdot \cos(\alpha)}{\cos(\gamma)} = -269.87 \cdot \text{kN}$$

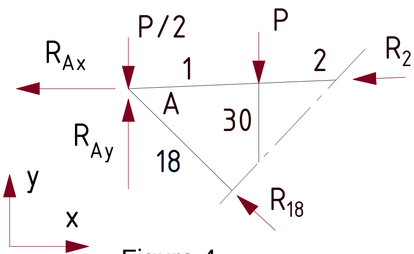


Figure 4

Moment about point D (see Figure 5)

$$R_{13}\cdot\left(h_1 + c_2\right) - P\cdot a - \frac{P}{2}\cdot\left(a + l_e\right) + 6P\cdot\left(a + l_e\right) = 0$$

$$R_{13} := \frac{-P\cdot\left(4.5\cdot a + 5.5\cdot l_e\right)}{c_2 + h_1} = -336.68\cdot kN$$

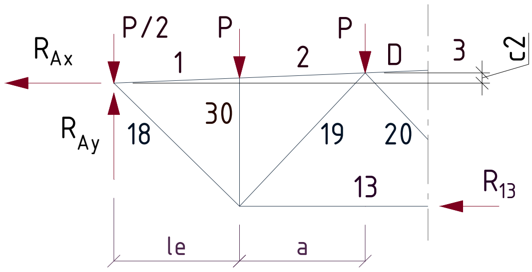


Figure 5

Sum of the forces along axis X (see Figure 6)

$$R_2\cdot\cos(\alpha) + R_{19}\cdot\cos(\delta) + R_{13} = 0$$

$$R_{19} := \frac{-R_2\cdot\cos(\alpha) - R_{13}}{\cos(\delta)} = 211.5\cdot kN$$

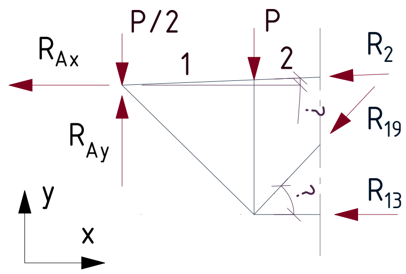


Figure 6

Moment about point E (see Figure 7)

$$R_3\cdot b_2 + P\cdot a + P\cdot 2\cdot a + \frac{P}{2}\cdot\left(2\cdot a + l_e\right) - \ldots = 0$$
$$+ 6P\cdot\left(2\cdot a + l_e\right)$$

$$R_3 := \frac{P\cdot\left(8\cdot a + 5.5\cdot l_e\right)}{b_2} = 439.66\cdot kN$$

$$R_4 := R_3 = 439.66\cdot kN$$

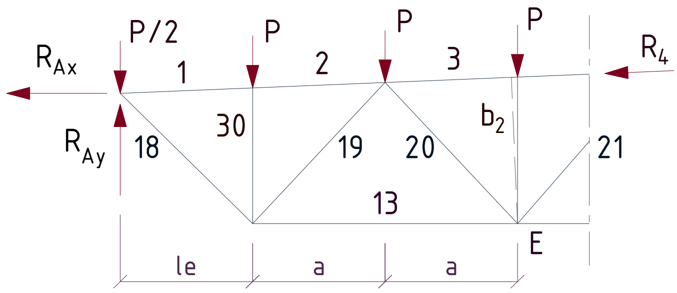


Figure 7

Sum of the forces along axis X (see Figure 8)

$$R_{13} + R_{20}\cdot\cos(\delta) + R_3\cdot\cos(\alpha) = 0$$

$$R_{20} := \frac{-R_3\cdot\cos(\alpha) - R_{13}}{\cos(\delta)} = -149.89\cdot kN$$

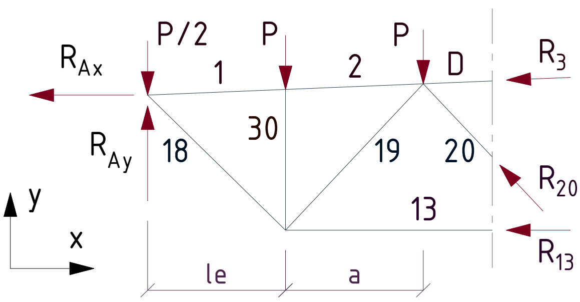


Figure 8

Moment about point G (see Figure 9)

$$R_{14}\cdot\left(h_1 + c_4\right) + 6P\cdot\left(3\cdot a + l_e\right) - \ldots = 0$$
$$+ \frac{P}{2}\cdot\left(3\cdot a + l_e\right) - P\cdot 3\cdot a - P\cdot 2\cdot a - P\cdot a$$

$$R_{14} := \frac{-P\cdot\left(10.5\cdot a + 5.5\cdot l_e\right)}{c_4 + h_1} = -503.93\cdot kN$$

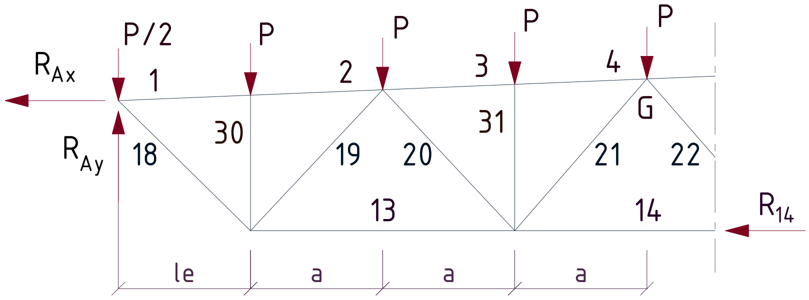


Figure 9

Sum of the forces along axis X
(see Figure 10)

$$R_4 \cdot \cos(\alpha) + R_{21} \cdot \cos(\varepsilon) + R_{14} = 0$$

$$R_{21} := \frac{-R_4 \cdot \cos(\alpha) - R_{14}}{\cos(\varepsilon)} = 97.75 \cdot \mathrm{kN}$$

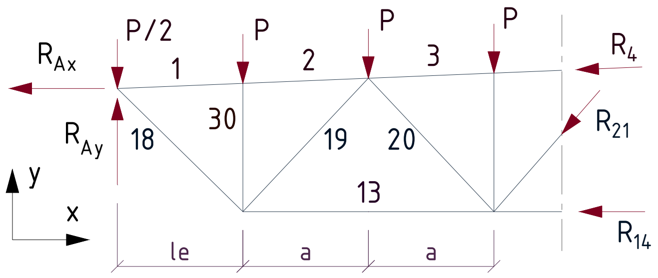


Figure 10

Moment about point F (see Figure 11)

$$R_5 \cdot b_3 + P \cdot a + P \cdot 2 \cdot a + \frac{P}{2} \cdot (4 \cdot a + l_e) - \blacksquare \ldots = 0$$
$$+ 6P \cdot (4 \cdot a + l_e) + P \cdot 3 \cdot a + P \cdot 4 \cdot a$$

$$R_5 := \frac{P \cdot (12 \cdot a + 5.5 \cdot l_e)}{b_3} = 534.31 \cdot \mathrm{kN}$$

$$R_6 := R_5 = 534.31 \cdot \mathrm{kN}$$

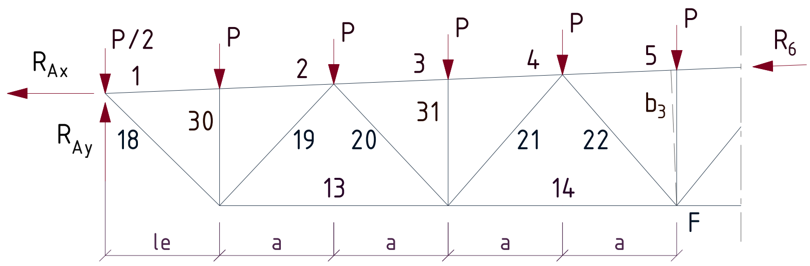


Figure 11

Sum of the forces along axis X
(see Figure 12)

$$R_5 \cdot \cos(\alpha) + R_{22} \cdot \cos(\varepsilon) + R_{14} = 0$$

$$R_{22} := \frac{-R_5 \cdot \cos(\alpha) - R_{14}}{\cos(\varepsilon)} = -45.45 \cdot \mathrm{kN}$$

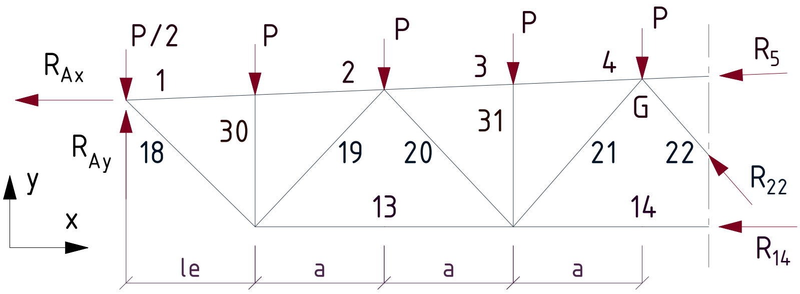


Figure 12

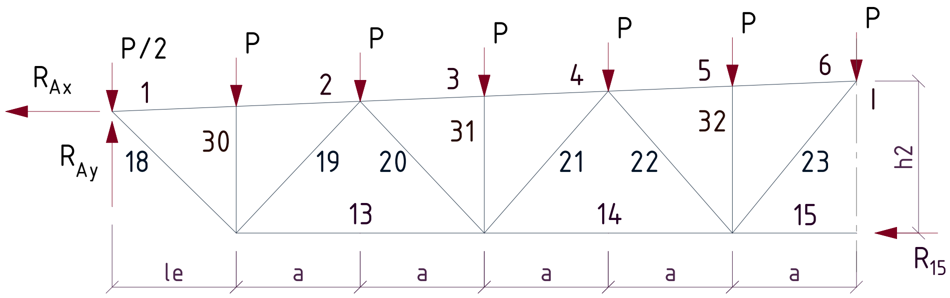


Figure 13

Moment about point I (see Figure 13)

$$R_{15} \cdot h_2 + 6 \cdot P \cdot (5 \cdot a + l_e) - \frac{P}{2} \cdot (5 \cdot a + l_e) - \blacksquare \ldots = 0$$
$$+ P \cdot 5a - P \cdot 4a - P \cdot 3a - P \cdot 2a - P \cdot a$$

$$R_{15} := \frac{-P \cdot (12.5 \cdot a + 5.5 \cdot l_e)}{h_2} = -532.56 \cdot \mathrm{kN}$$

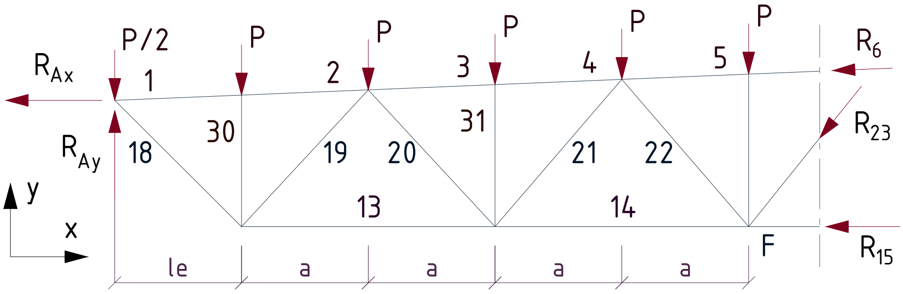


Figure 14

Sum of the forces along axis X (see Figure 14)

$$R_6 \cdot \cos(\alpha) + R_{23} \cdot \cos(\zeta) + R_{15} = 0$$

$$R_{23} := \frac{-R_6 \cdot \cos(\alpha) - R_{15}}{\cos(\zeta)} = -2.18 \cdot kN$$

Results

Calculation results are presented in graphical view in Figures 15 and 16.

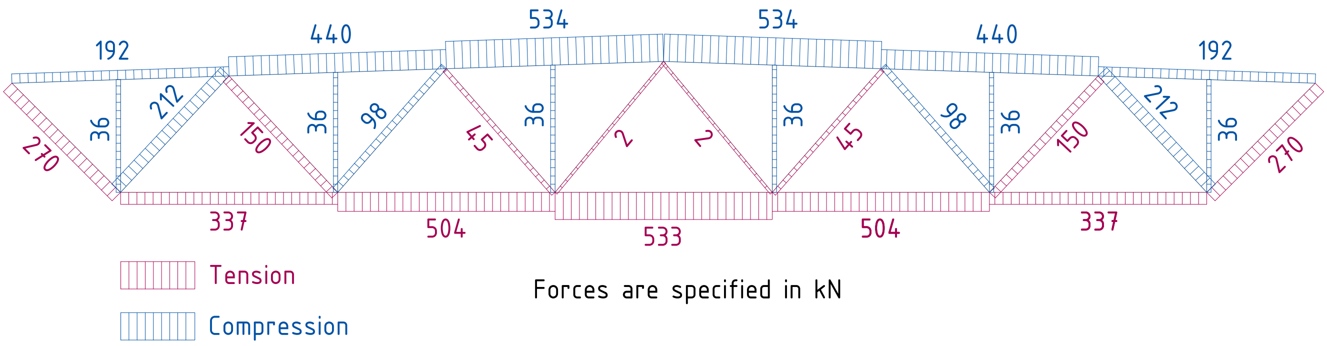


Figure 15. Stress patterns of longitudinal forces

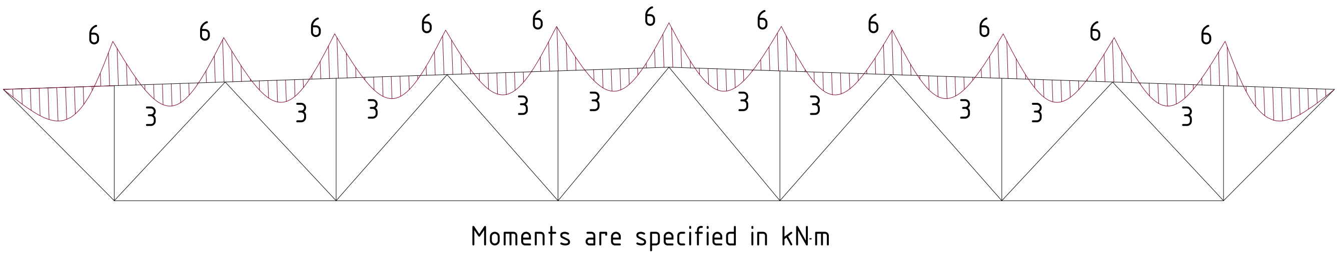


Figure 16. Stress patterns of bending moments

Design of the secondary truss members according to SNiP II-23-81*

General data

Design strength of steel to compression, tension and bending according to yield point

$R_y := 355\mathrm{MPa}$

Modulus of elasticity

$E := 206000\mathrm{MPa}$ SNiP II-23-81* Table 63

Check of bottom chord strength

The element should be cheked as a centrally tensioned element SNiP II-23-81* 5.1

Check profile RHS 70x7

Service conditions factor

$\gamma_c := 0.95$ SNiP II-23-81* Table 6

Tension force

$N := 532.56\mathrm{kN}$

Nett sectional area

$A_n := 15.96\mathrm{cm}^2$

Criterion

$$\frac{|N|}{A_n} \le R_y \cdot \gamma_c = 1$$ Profile is OK SNiP II-23-81* (5)

Normal stress

$$\sigma := \frac{|N|}{A_n} = 333.68 \cdot \mathrm{MPa}$$

Check of the most compressed diagonal web element (truss member #19)

The element should be checked for buckling as a centrally compressed element

Profile to check: RHS 90x3 SNiP II-23-81* 5.3

Gyration radius

$$i := 35.3\text{mm}$$

Profile area

$$A := 10.21\text{cm}^2$$

Commpressive force

$$N := 211.5\text{kN}$$

Effective length

$$l_{ef} := 0.8 \cdot 2.92\text{m} = 2.34\,\text{m}$$ SNiP II-23-81* 6

Service conditions factor

$$\gamma_c := 1$$ SNiP II-23-81* Table 6

Slenderness ratio

$$\lambda := \frac{l_{ef}}{i} = 66.18$$

Apparent slenderness ratio

$$\lambda_a := \lambda \cdot \sqrt{\frac{R_y}{E}} = 2.75$$

Buckling ratio SNiP II-23-81* 5.3 (8, 9, 10)

$$\phi_{function}(\lambda_a) := \text{if}\left[\lambda_a > 4.5, \frac{332}{\lambda_a^{\,2} \cdot (51 - \lambda_a)}, \text{if}\left[\lambda_a \le 2.5, 1 - \left(0.073 - 5.53 \cdot \frac{R_y}{E}\right) \cdot \lambda_a \cdot \sqrt{\lambda_a}, 1.47 - 13 \cdot \frac{R_y}{E} - \left(0.371 - 27.3 \cdot \frac{R_y}{E}\right) \cdot \lambda_a + \left(0.0275 - 5.53 \cdot \frac{R_y}{E}\right) \cdot \lambda_a^{\,2}\right]\right]$$

$$\phi := \phi_{function}(\lambda_a) = 0.69$$

Criterion

$$\frac{|N|}{\phi \cdot A} \le R_y \cdot \gamma_c = 1$$ Profile is OK SNiP II-23-81* 5.3 (7)

Check of the least stressed diagonal web element (truss member #23)

Slenderness of the element should be checked SNiP II-23-81* 6.15*, 6.16*

Profile to check: RHS 60x3

Gyration radius

$$i := 23.1mm$$

Effective length

$$l_{ef} := 0.8 \cdot 3.14m = 2.51\,m$$

Slenderness ratio

$$\lambda := \frac{l_{ef}}{i} = 108.74$$

Critical slenderness ratio

$$\lambda_{max} := 350$$ SNiP II-23-81* Table 19*, Table 20*

Criterion

$$\lambda \le \lambda_{max} = 1$$ Profile is OK

Check of top chord

The element should be checked for stability in and from plane of action of a moment as an element subjected to an axial force with bending

Profile to check: RHS 120x4.5

Profile area

$$A := 20.27\mathrm{cm}^2$$

Resisting moment of section for the most compressed fibre

$$W_c := 74.9\mathrm{cm}^3$$

Gyration radius

$$i := 46.8\mathrm{mm}$$

Axial force

$$N := 534.3\mathrm{kN}$$

Bending moment

$$M := 2.99\mathrm{kN}\cdot\mathrm{m}$$

Service conditions factor

$$\gamma_c := 1$$

Effective length

$$l_{ef} := 2\mathrm{m}$$

Slenderness ratio

$$\lambda := \frac{l_{ef}}{i} = 42.74$$

A stability analysis in the plane of action of a moment SNiP II-23-81* 5.27*

Eccentricity

$$e := \frac{M}{|N|} = 5.6 \times 10^{-3}\mathrm{m}$$

Relative eccentricity

$$m := \frac{e \cdot A}{W_c} = 0.15$$

Apparent slenderness ratio

$$\lambda_a := \lambda \cdot \sqrt{\frac{R_y}{E}} = 1.77$$

The shape factor of the section SNiP II-23-81* Table 73

$$\eta := \text{if}\left[\lambda_a \le 5 \wedge \lambda_a \ge 0, \text{if}\left[m \ge 0.1 \wedge m \le 5, (1.35 - 0.05 \cdot m) - 0.01 \cdot (5 - m) \cdot \lambda_a, \text{if}(m > 5 \wedge m \le 20, 1.1, 9999)\right], 1.1\right] = 1.26$$

Effective relative eccentricity

$m_{ef} := m \cdot \eta = 0.19$ SNiP II-23-81* (52)

Ratio of reduction in design strengths at eccentric compression

$\varphi_e := 0.78$ SNiP II-23-81* Table 74

Criterion

$\frac{N}{\varphi_e \cdot A} \le R_y \cdot \gamma_c = 1$ Profile is OK SNiP II-23-81* (51)

A stability analysis from the plane of action of a moment SNiP II-23-81* 5.30* This calculation is not required because buckling out of plane of the top chord is restrained

Buckling ratio

$$\varphi_y := \phi_{function}(\lambda_a) = 0.85$$

Ration of reduction in design strengths at lateral-torsional buckling of beams

$\varphi_b := 1$ For closed sections

Coefficient for calculation of "c" ratio

$\alpha_{function}(m) := \text{if}(m \le 1, 0.6, \text{if}(m > 1 \wedge m \le 5, 0.55 + 0.05 \cdot m, 9999))$ SNiP II-23-81* Table 10

$\alpha := \alpha_{function}(m) = 0.6$

$\lambda_c := 3.14 \cdot \sqrt{\frac{E}{R_y}} = 75.64$

Slenderness ratio

$\lambda_y := \lambda = 42.74$

$\varphi_c := \phi_{function}\left(\lambda_c \cdot \sqrt{\frac{R_y}{E}}\right) = 0.61$ φ_c is the value φ_y at $\lambda_y = \lambda_c$ SNiP II-23-81* Table 10

Coefficients for calculation of "c" ratio

$\beta := \text{if}\left(\lambda_y \le \lambda_c, 1, \sqrt{\frac{\varphi_c}{\varphi_y}}\right) = 1$ SNiP II-23-81* Table 10

$c_5 := \frac{\beta}{1 + \alpha_{function}(5) \cdot 5} = 0.2$ given m = 5

$c_{10} := \frac{1}{1 + \frac{10 \cdot \varphi_y}{\varphi_b}} = 0.11$ given m = 10

Coefficient for strength analysis with regard to development of plastic deformations due to bending

$c_{function}(m) := \text{if}\left[m \le 5, \frac{\beta}{1 + \alpha \cdot m}, \text{if}\left[m \ge 10, \frac{1}{1 + \frac{m \cdot \varphi_y}{\varphi_b}}, c_5 \cdot (2 - 0.2 \cdot m) + c_{10} \cdot (0.2 \cdot m - 1)\right]\right]$ SNiP II-23-81* (57, 58, 59)

$c := c_{function}(m) = 0.92$

Criterion

$$\frac{N}{c \cdot \varphi_y \cdot A} \le R_y \cdot \gamma_c = 1$$ Profile is OK

Design of the secondary truss members according to EN 1993

General data

Modulus of elasticity

$$E := 210000 \frac{N}{mm^2}$$

Yield strength

$$f_y := 355 \frac{N}{mm^2}$$ EC 1993-1-1 Table 3.1

Partial factor for resistance of members to instability assessed by member checks

$$\gamma_{M1} := 1$$ EC 1993-1-1 6.1

Check of bottom chord strength

Bottom chord should be checked for strength as centrally tensioned element

Profile to check: RHS 120x5

Profile area

$$A := 22.36 cm^2$$

Design value of tension force

$$N_{Ed} := 764.6 kN$$

Partial factor for resistance of cross-sections whatever the class is

$$\gamma_{M0} := 1$$ EN 1993-1-1 6.1

Design tension resistance

$$N_{t.Rd} := \frac{f_y \cdot A}{\gamma_{M0}} = 793.78 \cdot kN$$ EN 1993-1-1 (6.6)

The design value of the tension force should satisfy:

$$\frac{N_{Ed}}{N_{t.Rd}} \leq 1 = 1$$ Profile is OK EN 1993-1-1 6.2.3 (6.5)

Check of the most compressed diagonal web element (truss member #19)

The element should be checked for buckling as a centrally compressed element

Profile to check: RHS 100x3.5

Profile area

$$A := 13.19 cm^2$$

Second moment of area

$$I := 202.2 cm^4$$

Design value of commpressive force

$$N_{Ed} := 303.7 kN$$

Buckling length

$$L_{cr} := 0.9 \cdot 2.92\text{m} = 2.628\,\text{m}$$

Effective second moment of area

$$I_{eff} := I = 2.022 \times 10^{-6}\,\text{m}^4$$

Elastic critical force for the relevant buckling mode based on the gross cross sectional properties

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_{cr}^{\,2}} = 606.805 \cdot \text{kN}$$ EC 1993-1-1 6.4.1

Cross-section: hot rolled rectangular hollow section, hence buckling curve "a" is chosen, imperfection factor α = 0.21

$$\alpha := 0.21$$

Non-dimensional slenderness

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.878$$ for Class 1, 2 and 3 cross-sections

Value to determine the reduction factor χ

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.957$$

Reduction factor for the relevant buckling mode

$$\chi := \min\left(\frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}}, 1\right) = 0.748$$ EC 1993-1-1 6.3.1.2 (6.49)

Design buckling resistance of the compression member

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 350.233 \cdot \text{kN}$$ EC 1993-1-1 6.3.1.1 (6.47)

for Class 1, 2 and 3 cross-sections

A compression member should be verified against buckling as follows

$$\frac{N_{Ed}}{N_{b.Rd}} \le 1 = 1$$ EC 1993-1-1 6.3.1.1 (6.46)

Check of top chord

The element should be checked for buckling in and from plane of action of a moment as an element subjected to an axial force with bending EC 1993-1-1 6.2.3 6.3.3

Profile to check: RHS 160x4.5

Profile area

$$A := 27.47\text{cm}^2$$

Gyration radius

$$i := 63.2mm$$

Resisting moment

$$W_{pl} := 137cm^3$$

Axial force

$$N_{Ed} := 767.1kN$$

Bending moment

$$M_{Ed} := 3.3kN\cdot m$$

Buckling length

$$L_{cr} := 2m$$

Equivalent uniform moment factor

$$C_{mz} := 0.4$$

Slenderness ratio

$$\lambda_z := \frac{L_{cr}}{i} = 31.646$$

Effective second moment of area

$$I_{eff} := 1096cm^4$$

Elastic critical force for the relevant buckling mode based on the gross cross-sectional properties

$$N_{cr} := \frac{\pi^2 \cdot E \cdot I_{eff}}{L_{cr}^2} = 5.679 \times 10^3 \cdot kN$$ EN 1993-1-1 6.4.1

Imperfection factor

$$\alpha := 0.21$$

Non dimensional slenderness

$$\lambda := \sqrt{\frac{A \cdot f_y}{N_{cr}}} = 0.414$$ for Class 1, 2 and 3 cross-sections

Value to determine the reduction factor χ

$$\Phi := 0.5 \cdot \left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right] = 0.608$$

Reduction factor for the relevant buckling mode

$$\chi := \min\left(\frac{1}{\Phi + \sqrt{\Phi^2 - \lambda^2}}, 1\right) = 0.949$$ EN 1993-1-1 6.3.1.2 (6.49)

$$\chi_z := \chi = 0.949$$

Design buckling resistance of the compression member

$$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 925.41 \cdot kN$$

EN 1993-1-1 6.3.1.1 (6.47)

for Class 1, 2 and 3 cross-sections

Interaction factor EN 1993-1-1 6.2.3 Table B.2

$$k_{zz} := \min\left[C_{mz} \cdot \left[1 + \left(\lambda_z - 0.2\right) \cdot \frac{N_{Ed}}{\chi_z \cdot N_{b.Rd}}\right], C_{mz} \cdot \left(1 + 0.8 \cdot \frac{N_{Ed}}{\chi_z \cdot N_{b.Rd}}\right)\right] = 0.68$$

The moments due to the shift of the centroidal axis

$$\Delta M_{z.Ed} := 0$$

EN 1993-1-1 6.2.3 Table 6.7

The design resistance for bending about one principal axis of a cross-section

$$M_{pl.Rd} := \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = 48.635 \cdot kN \cdot m$$

$$M_{z.Rd} := M_{pl.Rd} = 48.635 \cdot kN \cdot m$$

EN 1993-1-1 6.2.3 6.2.5

Criterion

$$\frac{N_{Ed}}{\chi_z \cdot N_{b.Rd}} + k_{zz} \cdot \frac{M_{Ed} + \Delta M_{z.Ed}}{M_{z.Rd}} \le 1 = 1$$

EN 1993-1-1 6.3.3 (6.62)

Profile is OK