

Technical Report

FINHA T200 Series Building Structural Calculations

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Samara 2020

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INTRODUCTION

This calculation was performed to confirm that the strength and stability characteristics of the FINHA T200 series building with connectors comply with the requirements of building codes SP 64.13330.2017 "SNiP II-25-80. Wooden Structures." The calculation is performed for load cases corresponding to extreme operating conditions:

- Snow zone VIII;
- Wind zone VII, terrain type C;
- Construction site seismicity of magnitude 9 with soil categories I or II;
- Construction site seismicity of magnitude 9 with soil categories I or III or IV;

Objective:

- Creating load cases in accordance with SNiP standards;
- Creating finite element models of the designed buildings;
- Performing stability and stress-strain state calculations for given load cases;
- Generating design force combinations in accordance with SNiP standards;
- Determining safety factors

1. INITIAL DATA FOR CALCULATION

1.1 General description

A general view of the FINHA T200 building is shown in Figs. 1–3.

The building frame is assembled from cross beams with a span of 10.5 m, made of wooden beams connected to other original bolts via connectors, and long beams (Fig. 4).

The longitudinal beams rest on studs installed at intervals of 4×0.625 m, i.e., every fifth frame. The distance between the post axes in the flat frames is 3.5 m.

The frame is installed on grillage beams. The grillage beams are also made of wood..

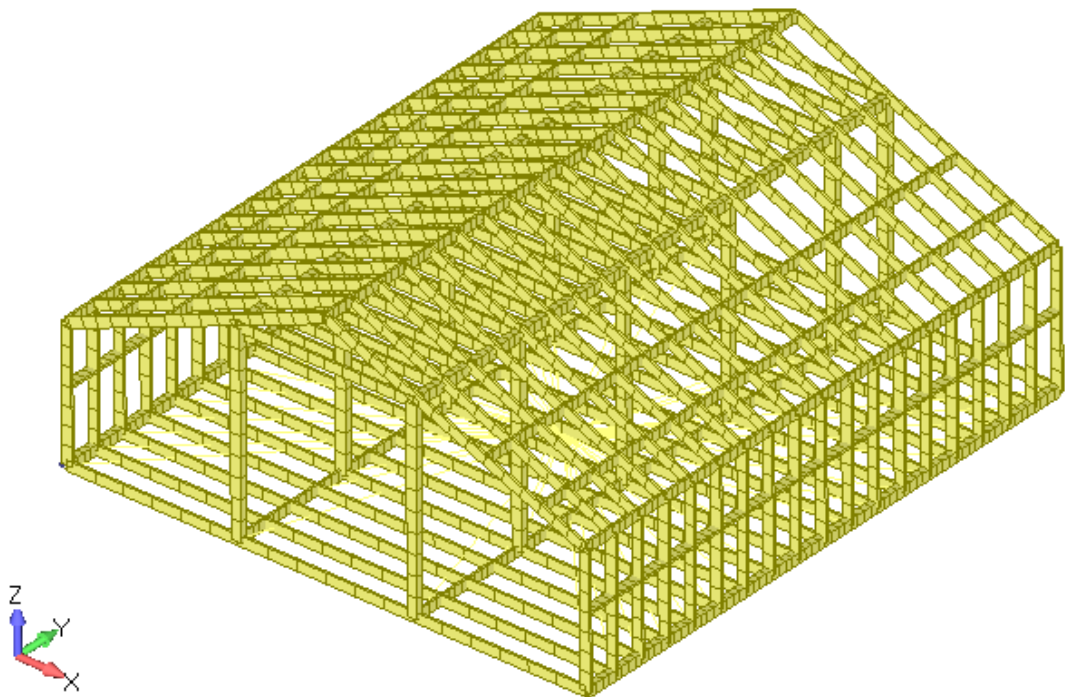


Fig. 1. General view of the FINHA T200 building frame, frame width 10.5 m

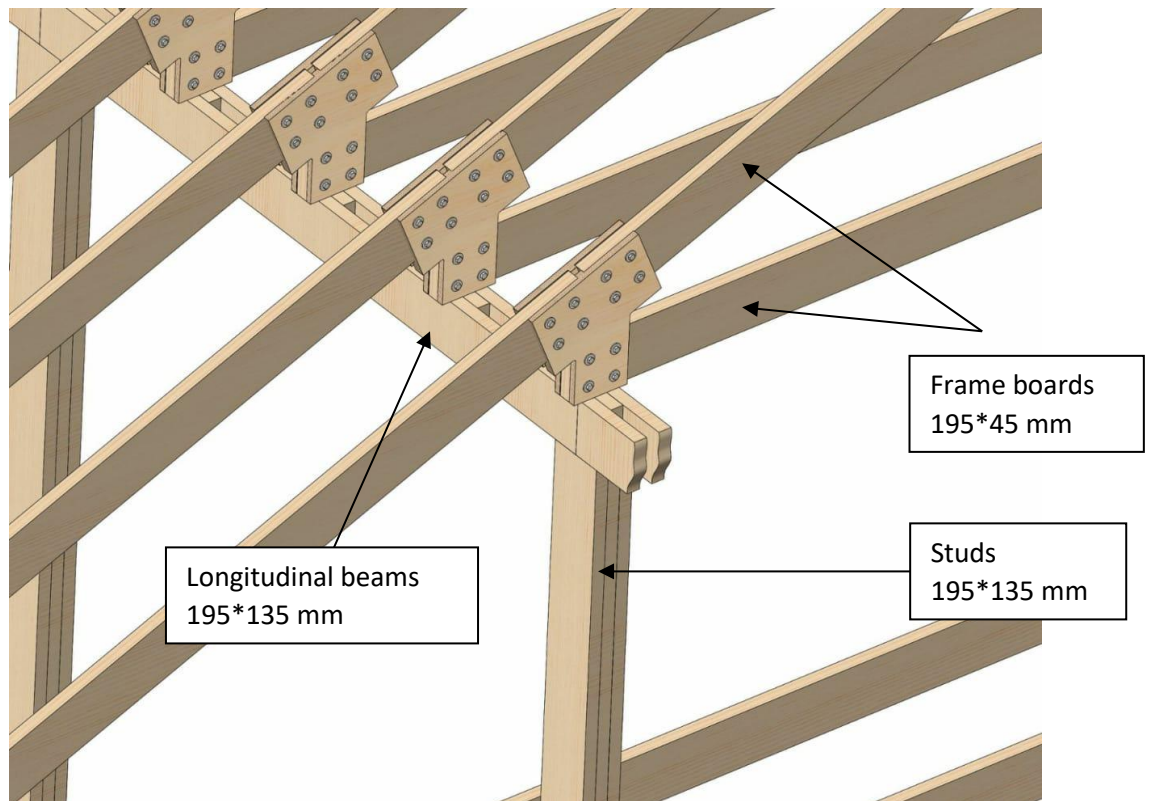


Fig. 2. Scheme of support of crossbars on a longitudinal beam

1.2 Units

The following units are used in this calculation:

- Force – N;
- Length – mm;
- Stress – MPa;

1.3 Materials used and reference data

Wood, grade 1, table 3 [1] :

- $R_u^A = R_{c\sigma c}^A = 24$ MPa;
- $R_p^A = 15$ MPa;

The design resistance is determined by the relation 1 [1]

$$R^P = R^A \cdot m_{\partial l} \cdot \prod m_i$$

The coefficient of long-term strength $m_{\partial l}$ is taken from the table 4 [1]

- $m_{\partial l} = 0.66$ - under the combined action of permanent and short-term loads;
- $m_{\partial l} = 0.92$ - under the combined action of permanent and seismic loads.

Working conditions coefficients, p. 6.9 [1]:

- $m_b = 0.9$ on table. 9;
- $m_t = 1.0$;
- $m_{c\partial l} = 1.0$ on table. 11;
- $m_{cc} = 1.0$ on table. 13 ;

$$\prod m_i = m_b \cdot m_t \cdot m \cdot m_{cc} = 0.9 \cdot 1.0 \cdot 1.0 \cdot 1.0 = 0.9$$

Design value of resistance to bending and compression:

- $R_{u3\sigma}^A = R_{c\sigma c}^A = 24 \cdot 0.66 \cdot 0.9 = 14$ MPa - under the combined action of permanent and short-term loads;
- $R_{u3\sigma}^A = R_{c\sigma c}^A = 24 \cdot 0.92 \cdot 0.9 = 20$ MPa - under the combined action of permanent and seismic loads.

Design value of resistance to tension:

- $R_p^A = 15 \cdot 0.66 \cdot 0.9 = 8.9$ MPa - under the combined action of permanent and short-term loads;
- $R_p^A = 15 \cdot 0.92 \cdot 0.9 = 12.4$ MPa - under the combined action of permanent and seismic loads.

Design bearing strength:

- along the grain $R_{cr0}^A = R_{bend}^A = 14$ MPa;
- cross-grain $R_{cr90}^A = 4.5$ MPa.
- under 45-degree loading., r.6[1]:

$$R_{cr45}^A = R_{cr0}^A / (1 + (R_{cr0}^A / R_{cr90}^A - 1) \cdot \sin^3 45) = 8.0 \text{ MPa}$$

Plywood:

Design value of resistance:

- $R^A = 14$ MPa - under the combined action of permanent and short-term loads;
- $R^A = 20$ MPa - under the combined action of permanent and seismic loads.

The calculation is performed for a group of static loads and a group of seismic loads, on the basis of which the design load combinations are formed.

1.4 Static loads

1. Building's own weight, permanent loading (**weight**).

Weight loads are determined by acceleration along the vertical axis $a = g \cdot \gamma_{Gf}$,

where

$$g = 9810 \text{ mm/c}^2, \gamma_{Gf} = 1.2 - \text{load safety factor.}$$

2. Floor beam load weight, temporary loading (**load**);

The load value is taken in accordance with p. 8.2.2 and table 8.3 [2]:

$$q = P_1 \cdot \gamma_f = 0.0015 \cdot 1.3 = 0.00195 \text{ MPa,}$$

There $P_1 = 0.0015$ MPa – nominal value of uniformly distributed load;

$$\gamma_f = 1.3 - \text{load safety factor.}$$

3. Snow loads: Snow on the left (**snow left**) and Snow on the right (**snow right**), temporary loadings:

Nominal value of uniformly snow load, p.10 [2]

$$S_0 = c_e \cdot c_t \cdot \mu \cdot S_g$$

There

$$c_e = 1.0 - \text{snow drifting factor is taken in accordance with p. 10.6 [2];}$$

$$c_t = 1.0 - \text{thermal coefficient;}$$

μ – the conversion factor from the weight of the snow cover of the ground to the snow load on the roof is taken in accordance with p. 10.4 [2] ;

$\mu=1.0$ – for roof slope angle up to 30 deg;

$\mu=0.5$ – for roof slope angle up to 45 deg;

For angles greater than 45 degrees, it is taken into reserve $\mu=0.5$;

$S_g = 4.0 \text{ kN/m}^2$ – nominal value of snow cover weight on 1 m^2 for the snow region VIII.

Design value of snow load:

$$S = S_0 \cdot \gamma_{Sf},$$

$\gamma_{Sf} = 1.4$ – load safety factor for snow load.

Snow loads are shown in Fig. 3-4

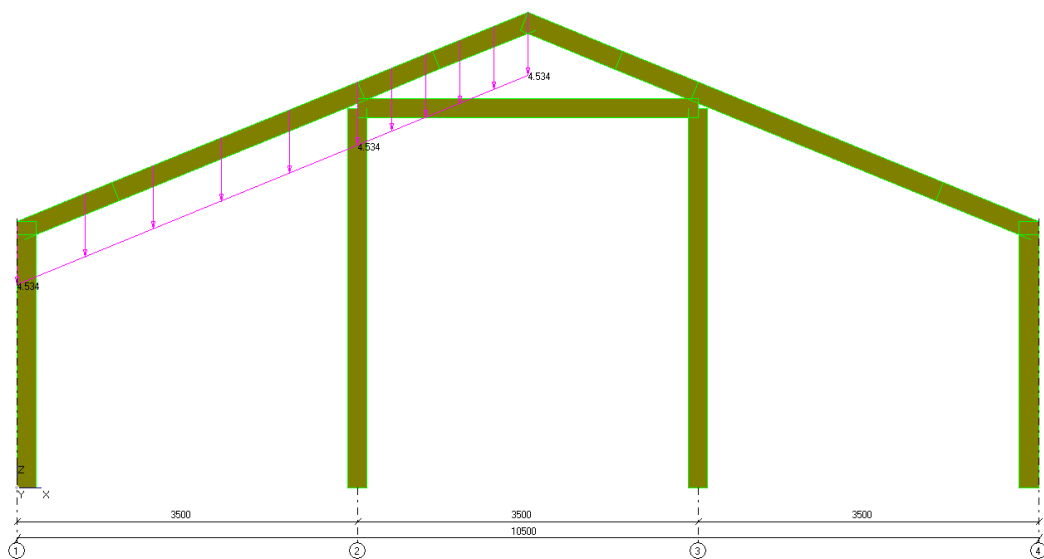


Fig. 3. Design snow load for region VII (snow_left), acting in the direction of the axis Z, kN/m^2

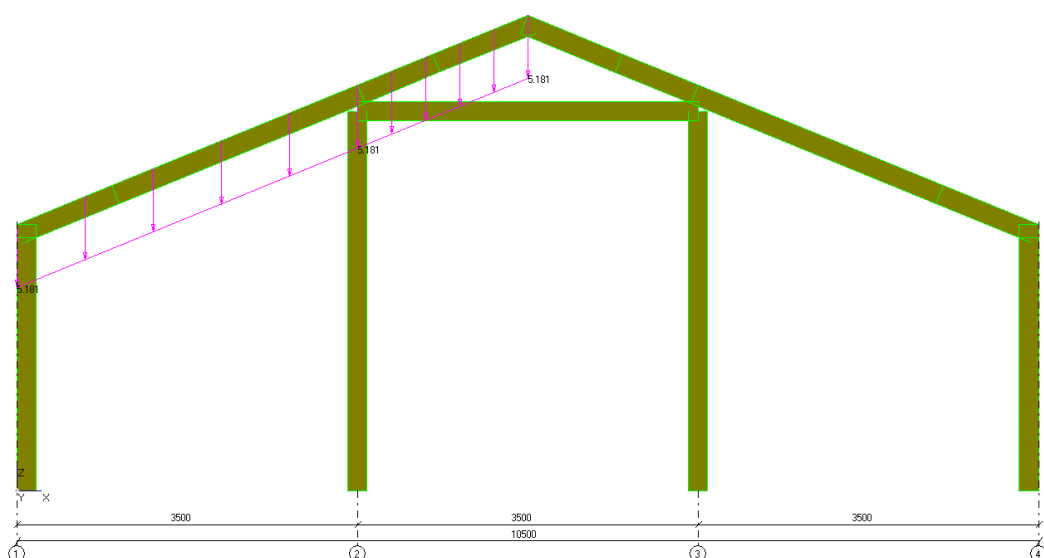


Fig. 4. Design snow load for region VII (snow_right), acting in the direction of the axis Z, kN/m^2

4. Wind loads, short-term load cases:

The standard wind pressure value is adopted according to clause 11.1.4 [2] for wind zone VII:

$$w_0 = 0.85 \text{ kN/m}^2;$$

$\gamma_{wf} = 1.4$ – wind load safety factor.

The calculation is performed for three load cases:

- ❑ wind from the left **wind_left**;
- ❑ wind from the right **wind_right**;
- ❑ wind at the end **wind_tor**;

The wind load distribution in the **wind_left** load case is shown in Fig. 5.

The **wind_right** load case is symmetrical.

The wind load distribution in the **wind_tor** load case is shown in Fig. 6.

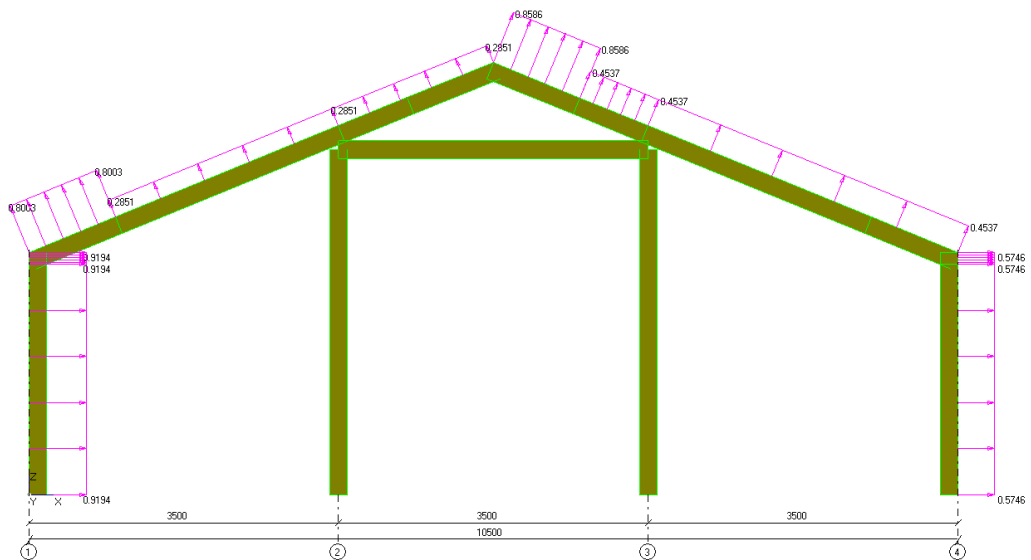


Fig. 5. Design wind load: wind from the left, acting normal to the surface, kN/m^2

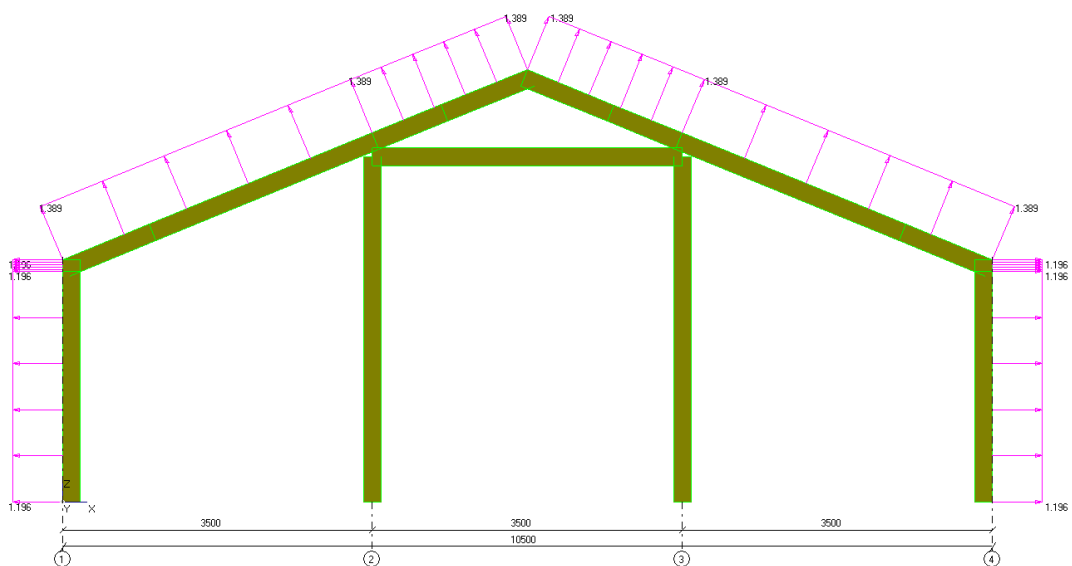


Fig. 6. Design wind load: wind at the end, acting normal to the surface, kN/m²

1.5 Seismic loads

- Horizontal action in the direction of the transverse axis (**Quake X**);
- Horizontal impact in the direction of the longitudinal axis (**Quake Y**);
- Vertical impact (**Quake Z**).

The seismic load parameters are taken with the estimated seismicity of the region being 9 points, soil category II.

The design seismic load in the direction of the generalized coordinate with number j , applied to node k of the calculation model and corresponding to the i -th eigen mode of the building is determined by the formula:

$$S_{ik}^j = K_0 \cdot K_1 \cdot S_{0ik}^j$$

- $K_0 = 0.8$ - factor that takes into account the purpose of the structure and its responsibility, taken from the table [3];
- $K_1 = 0.15$ - factor taking into account permissible damage to buildings, adopted according the table 4 [3];

S_{0ik}^j - the value of the seismic load for the i -th eigen mode of a building, determined under the assumption of elastic deformation of structures according to the formula:

$$S_{0ik}^j = m_k^j \cdot A \cdot K_\psi \cdot \beta_i \cdot \eta_{ik}^j$$

There

m_k^j - the mass of the building or the moment of inertia of the corresponding mass of the building, referred to the node k along the generalized coordinate j ;

- $K_\psi = 1.3$ - factor that takes into account the ability of a building to dissipate vibration energy under horizontal impact, taken from table 5.
- $A = 4 \text{ m/s}^2$ - the acceleration value at the foundation level with the estimated seismicity of the area being 9 points, soil category II;

η_{ik}^j - factor depending on the form of deformation of the building during its oscillations according to the i -th mode;

β_i - dynamic coefficient corresponding to the period of oscillations for the i -th eigen mode, adopted in accordance with p. 5.6 [3], for soil of category III (adopted as a reserve), Fig. 7.

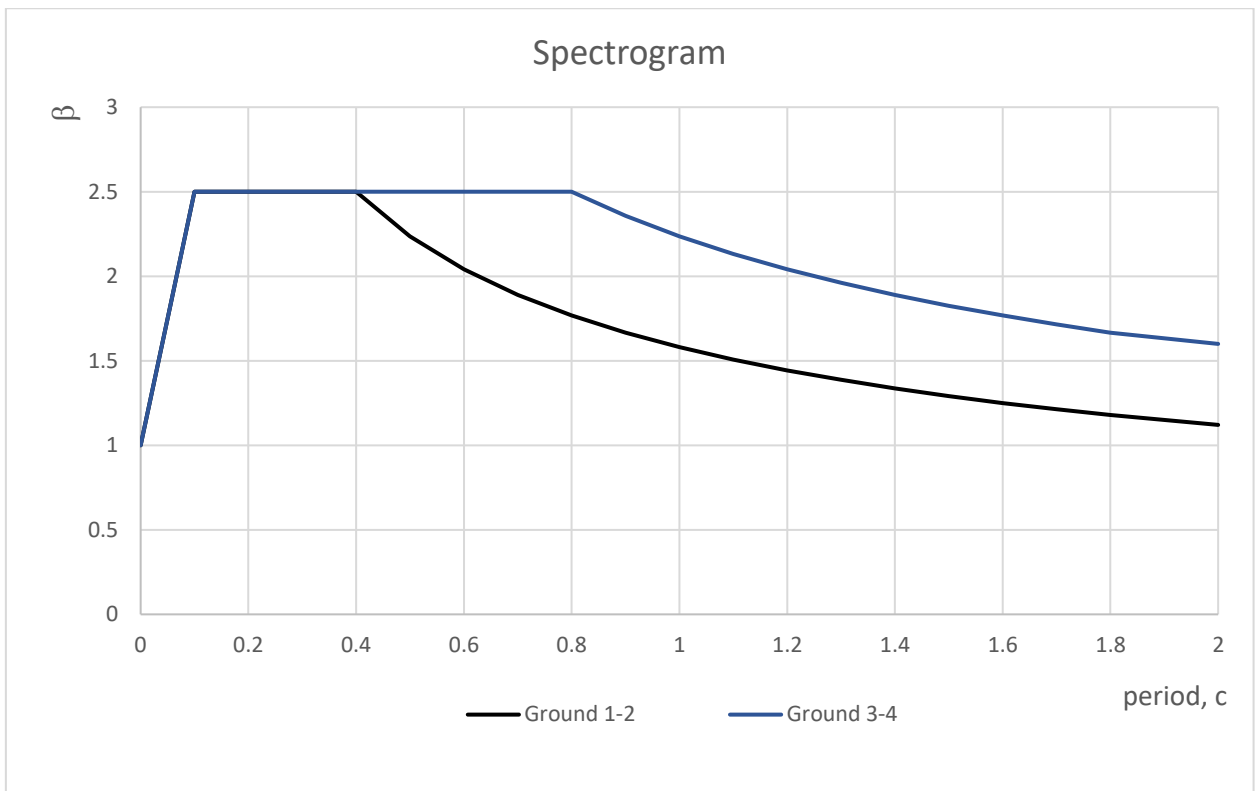


Fig. 7. Spectrogram of dynamic coefficients

1.6 Design load combinations

The coefficients in load combinations are taken in accordance with p. 6.2, 6.3, 6.4 [2] and p. 5.1 [3]

The basic combinations

1. weight + load + snow_left + snow_right
2. weight + load + snow_left + snow_right + 0.9*wind_left
3. weight + load + snow_left + snow_right + 0.9*wind_tor

Special combinations

1. Quake X + 0.9*Weight + 0.8*load + 0.5*snow_left + 0.5*snow_right
2. Quake Y + 0.9*Weight + 0.8*load + 0.5*snow_left + 0.5*snow_right
3. Quake Z + 0.9*Weight + 0.8*load + 0.5*snow_left + 0.5*snow_right
4. Quake X + 0.75*Quake Z + 0.9*Weight + 0.8*load + 0.5*snow_left + 0.5*snow_right
5. Quake Y + 0.75*Quake Z + 0.9*Weight + 0.8*load + 0.5*snow_left + 0.5*snow_right

1.7 Assigning loads and constrains

The static loads from weight (**weight**) are given in the form of acceleration

$$g = -9810 \cdot 1.2 = -11770 \text{ mm}/c^2, \text{ (considering } \gamma_{Gf} = 1.2) \text{ along the Z coordinate axis.}$$

Snow and wind loads are specified as distributed loads applied to beams and studs.

Seismic loads are specified by the spectrogram of dynamic coefficients, see Fig. 6, amplitude A and coefficients K_0, K_1, K_Ψ .

The building model is pivotally fixed at the support points on the piles.

2. STRENGTH ANALYSIS

2.1 Frame design models

The strength analysis of the FINHA T200 building is carried out for two variants of snow loads and stud placement:

1. **The main option for snow zones from I to VII**, the distance between the posts along the building axis is 2.5 m (posts on every fifth frame)
2. **Reinforced version for snow zone VIII**, distance between posts along the building axis is 1.875 m (posts on every fourth frame).

The design models for the FINHA T200 buildings are shown in Figures 8 and 9.

Frames and longitudinal beams are modeled using Beam elements of the appropriate cross-section. Joints between beams and spacers are assumed to be moment-resistant.

Connectors are not modeled and are assumed to be of equal strength to the frame boards. The conclusion that connector strength is ensured by ensuring frame strength is based on the results of [4].

Frame element cross-sections:

- Frame boards 195*45 mm;
- Studs 195*135 mm;
- Spacers 195*45 mm;
- Longitudinal beams 195*135 mm

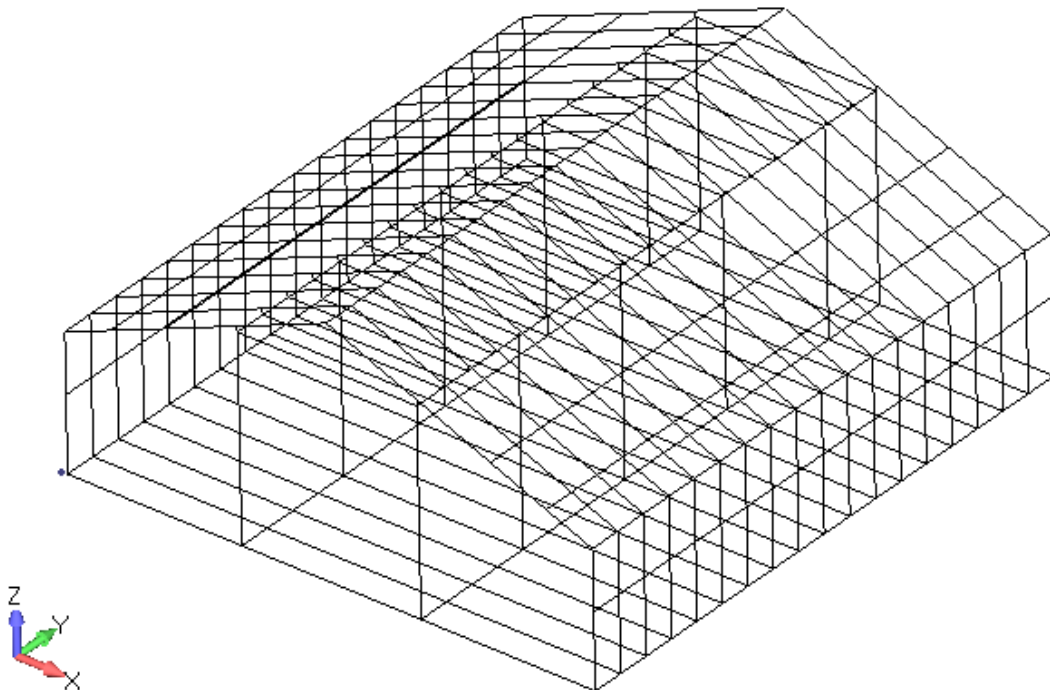


Fig. 8. Finite element model of the building, studs spaced 2.5 m apart

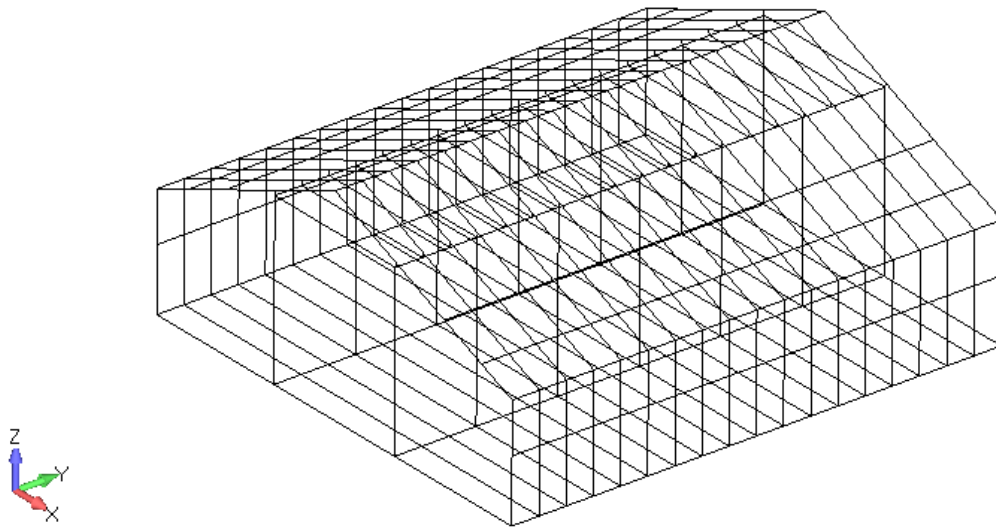


Fig. 9. Finite element model of the building, posts spaced 1.875 m apart

2.2 Basic load combinations

Design value of resistance to bending and compression:

- $R_{bend}^A = R_{comp} = 14$ MPa - under the combined action of permanent and short-term loads.

2.2.1 Basic variant of the building, snow zone VII

Frames

The maximum bending stresses in frame elements: $\sigma_{max} = 12.1$ MPa, fig. 10,

- Factor of safety $\eta = R_p^A / \sigma_{max} = 14/12.1 = \mathbf{1.16}$
- Utilization rate $f = 1/\eta = \mathbf{0.86}$

Longitudinal beams

The maximum bending stresses in longitudinal beams: $\sigma_{max} = 12.7$ MPa caused by bending, fig. 11,

- Factor of safety $R_{bend}^A / \sigma_{max} = 14/12.7 = \mathbf{1.1}$
- Utilization rate $f = 1/\eta = \mathbf{0.91}$.

Studs

The maximum bending stresses in studs: $\sigma_{max} = 4.3$ MPa, fig. 12

- Factor of safety $R_{u32}^A / \sigma_{max} = 14/4.3 > \mathbf{2.0}$
- Utilization rate $f = 1/\eta < \mathbf{0.5}$

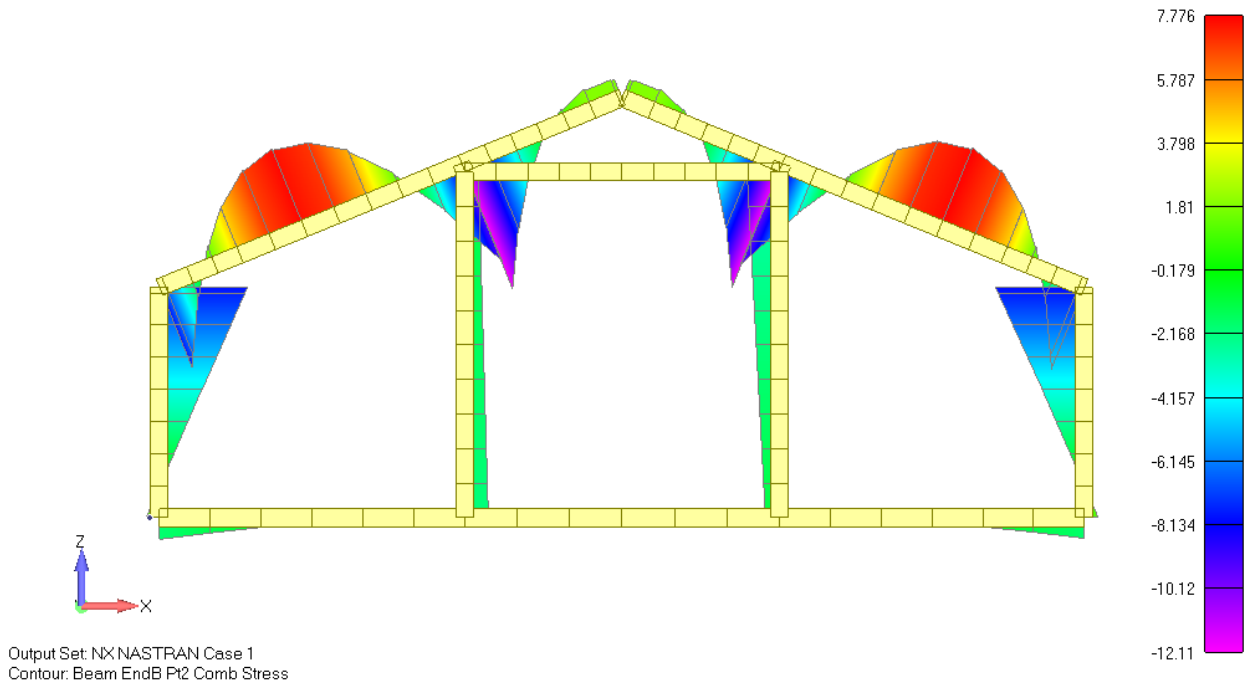


Fig. 10. Basic combinations. Snow region VII. Stresses in beams and studs, MPa

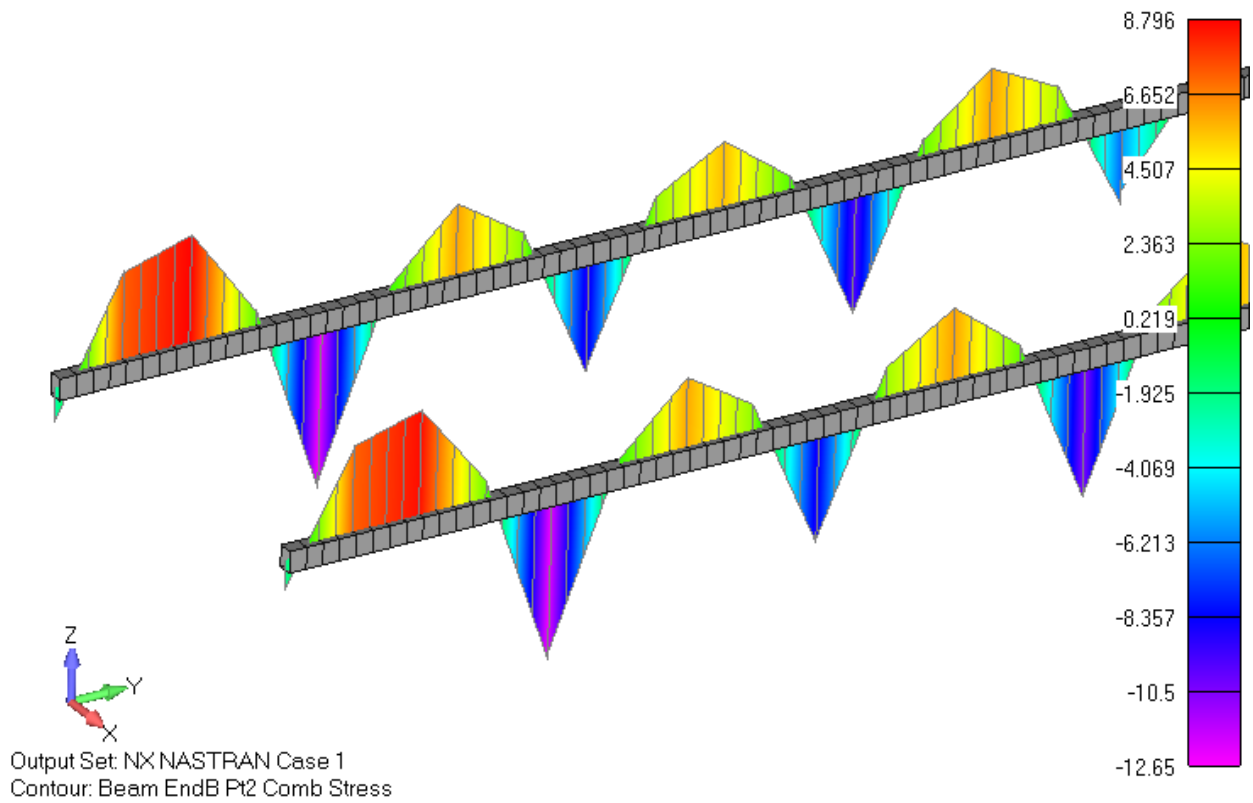


Fig. 11. Basic combinations. Snow region VII. Stresses in longitudinal beams, MPa

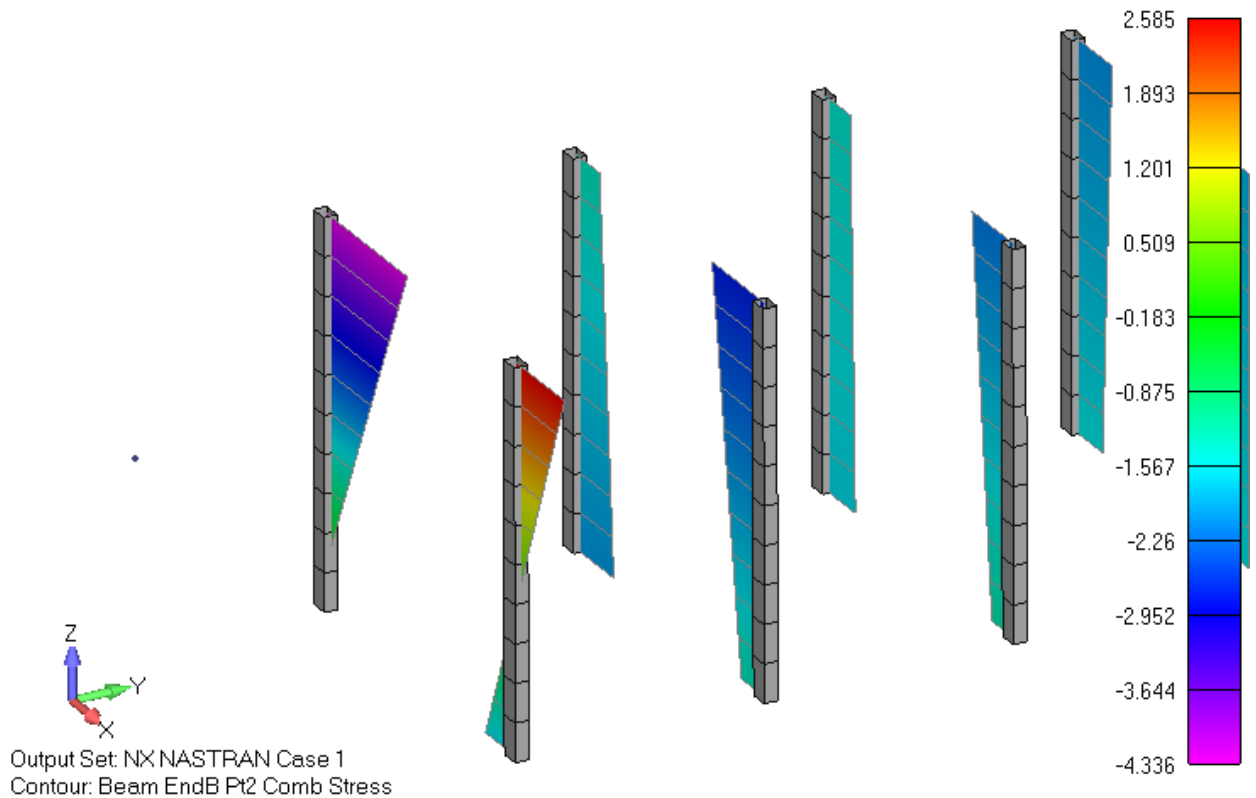


Fig. 12. Basic combinations. Snow region VII. Maximum stresses in the studs, MPa

2.2.2 Reinforced version of the building, snow zone VIII

Frames

The maximum bending stresses in frame elements: $\sigma_{\max} = 13.7$ MPa, fig. 13,

- Factor of safety $\eta = R_p^A / \sigma_{\max} = 14 / 13.7 = \mathbf{1.02}$
- Utilization rate $f = 1 / \eta = \mathbf{0.98}$

Longitudinal beams

The maximum bending stresses in longitudinal beams: $\sigma_{\max} = 7.5$ MPa caused by bending, fig. 14,

- Factor of safety $R_{bend}^A / \sigma_{\max} = 14 / 7.5 = \mathbf{1.87}$
- Utilization rate $f = 1 / \eta = \mathbf{0.54}$.

Studs

The maximum bending stresses in studs: $\sigma_{\max} = 2.6$ MPa, fig. 15

- Factor of safety $R_{bend}^A / \sigma_{\max} = 14 / 2.6 > \mathbf{2.0}$
- Utilization rate $f = 1 / \eta < \mathbf{0.5}$

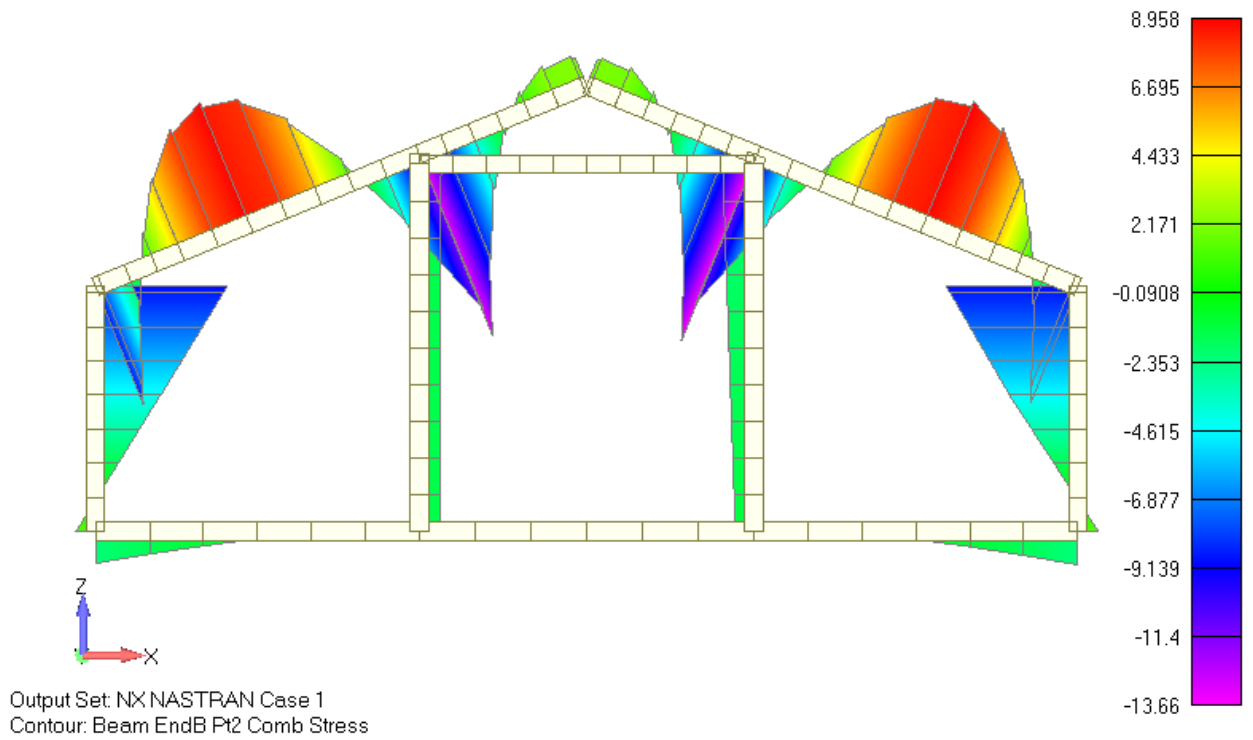


Fig. 13. Basic combinations. Snow region VIIi. Stresses in beams and studs, MPa

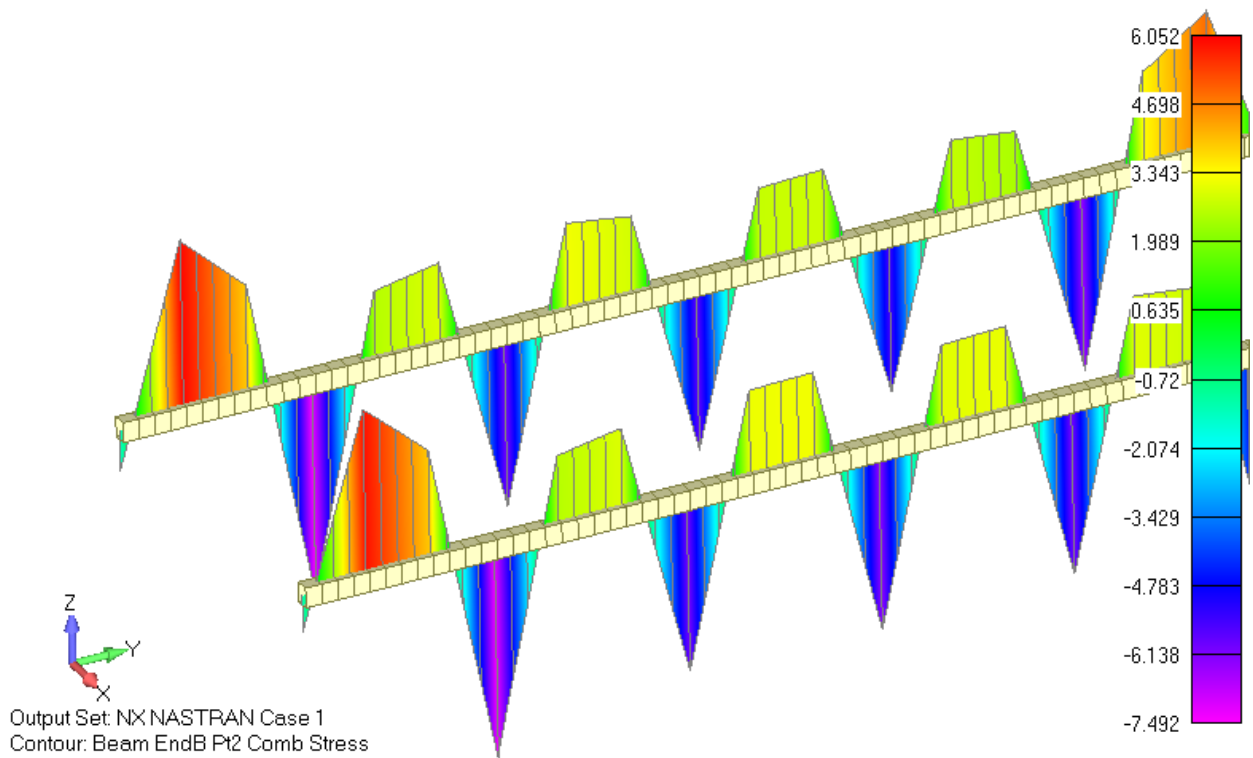


Fig. 14. Basic combinations. Snow region VIII. Stresses in longitudinal beams, MPa

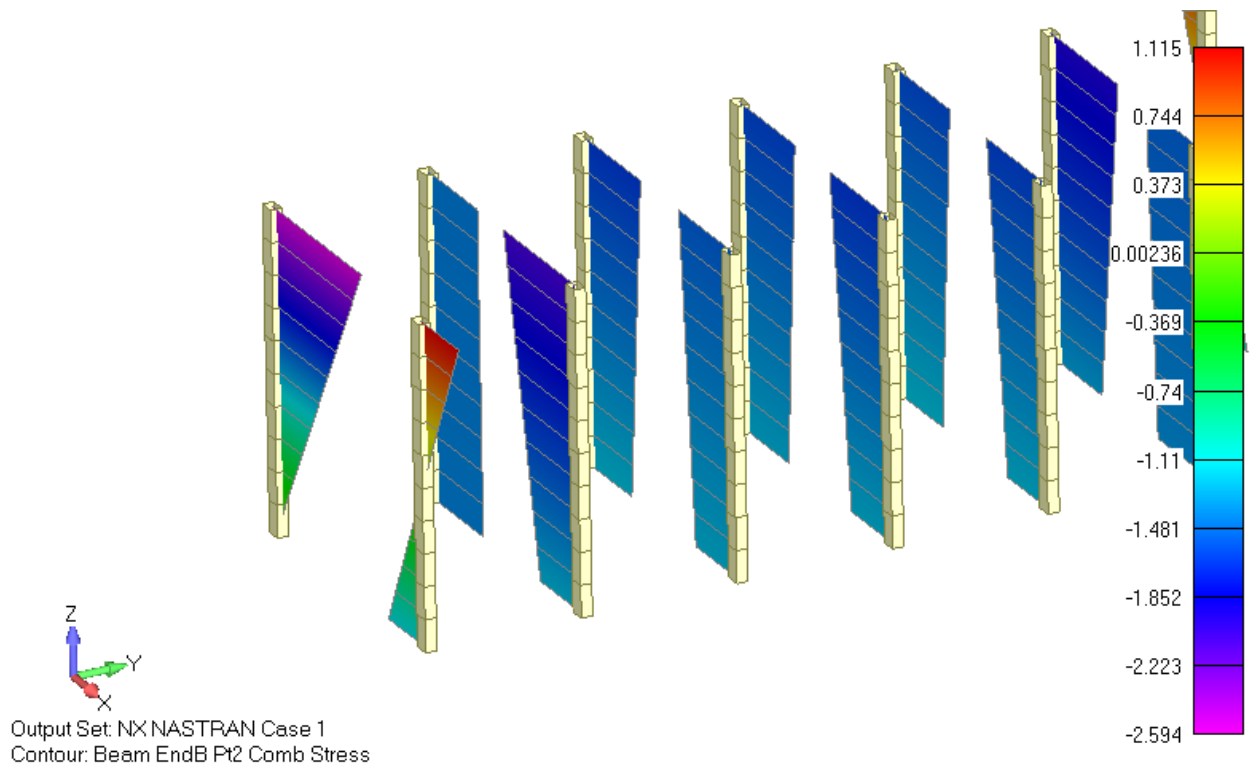


Fig. 15. Basic combinations. Snow region VIII. Maximum stresses in the studs, MPa, MPa

2.3 Special combinations

Design value of resistance to bending and compression:

- $R_{bend}^A = R_{comp} = 20$ MPa - under the combined action of permanent and short-term loads

Frames

Maximum bending stresses in frame elements: $\sigma_{max} = 16.4$ MPa, fig. 16,

- Factor of safety $\eta = R_p^A / \sigma_{max} = 20/16.4 = 1.22$
- Utilization rate $f = 1/\eta = 0.82$

Longitudinal beams

The maximum stresses are in the longitudinal beam elements: $\sigma_{max} = 13.9$ MPa caused by bending, fig. 17,

- Factor of safety $R_{bend}^A / \sigma_{max} = 20/13.9 = 1.447$
- Utilization rate $f = 1/\eta = 0.7$.

Studs

The maximum stresses are in the studs: $\sigma_{max} = 16.4$ MPa, fig. 16

- Factor of safety $R_{bend}^A / \sigma_{max} = 20/16.4 = 1.22$
- Utilization rate $f = 1/\eta = 0.82$

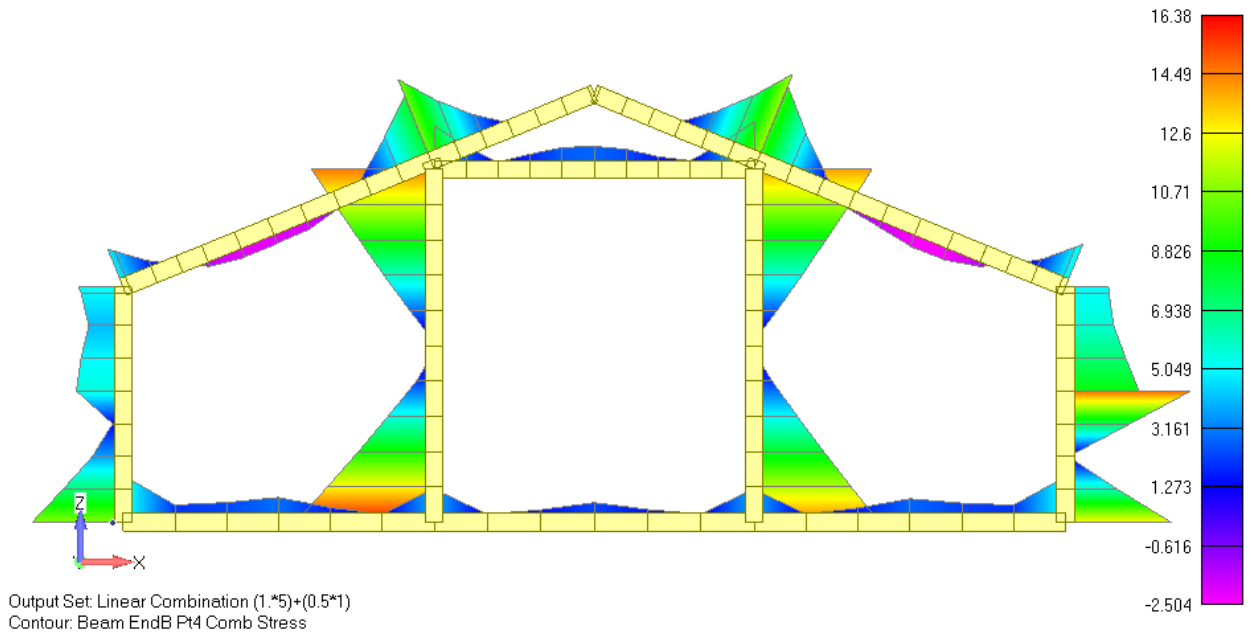


Fig. 16. Special combinations. Maximum stresses in frames, MPa.

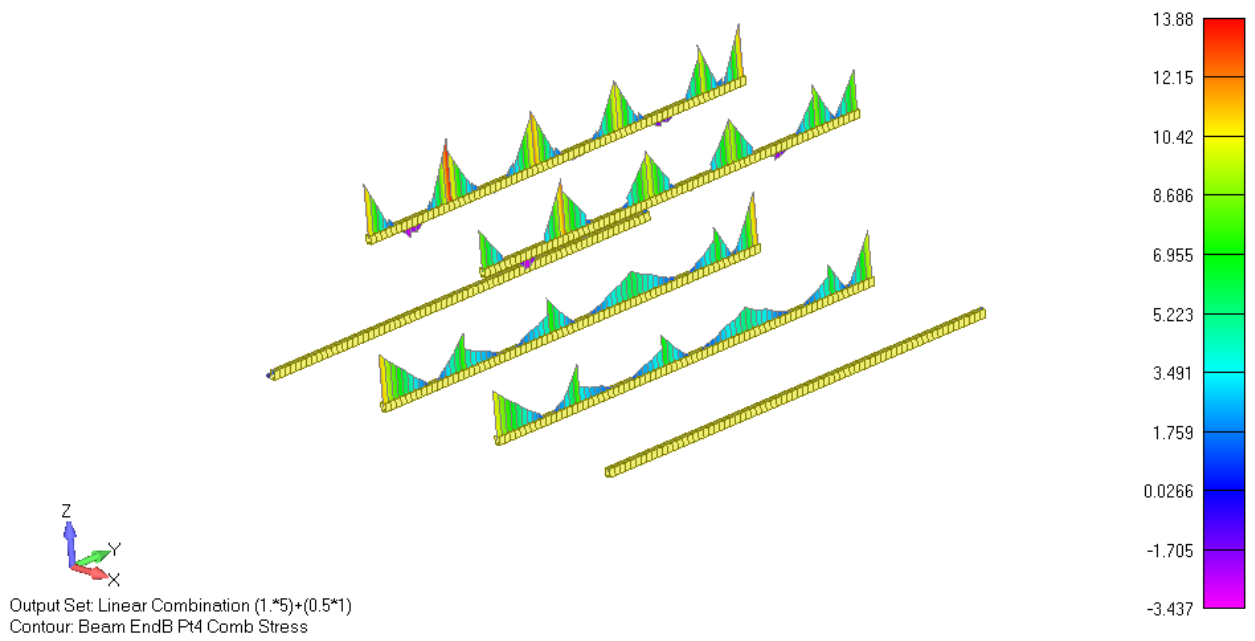


Fig. 17. Special combinations. Snow region VIII. Stresses in longitudinal beams, MPa

2.4 Summary of safety factors for the FINHA T200 building, basic variant - snow region VII

Table 2

#	Name	Minimum safety margin, η	Maximum utilization rate
1	Frames	1.16	0.86
2	Longitudinal beams	1.1	0.91
3	Studs	1.22	0.82

2.5 Summary of safety factors for the FINHA T200 building, reinforced version - snow zone VIII

Table 3

#	Name	Minimum safety margin, η	Maximum utilization rate
1	Frames	1.02	0.98
2	Longitudinal beams	1.87	0.54
3	Studs	1.22	0.82

CONCLUSION

Calculations showed that the FINHA T200 building structure complies with the requirements of building codes SP 64.13330.2017 "SNIP II-25-80. Wooden Structures":

- For the basic option, the estimated snow load is $4.9 \text{ kN/m}^2 = 500 \text{ kg/m}^2$ - snow zone VII.
- For the enhanced option, the maximum possible snow load is assumed - the estimated snow load is $5.6 \text{ kN/m}^2 = 570 \text{ kg/m}^2$ - snow zone VIII.

The maximum estimated seismicity of the region in the calculations was assumed to be 9 points.

The required safety factor is met, $\eta \geq 1$.

LIST OF USED SOURCES

1. SP 64.13330.2017 SNIP II-25-80. Wooden Structures
2. SP 20.13330.2016 SNIP 2.01.07-85*. Loads and impacts
3. SP 14.13330.2014 SNIP II-7-81*. Construction in seismic areas
4. Technical report "Calculation of the supporting structure of the FINHA-2 series of buildings" 2020