ISSN 1392−1207. MECHANIKA. 2015 Volume 21(1): 11−18

Investigation of mechanical state of spatial roof from steel trusses on asymmetric building

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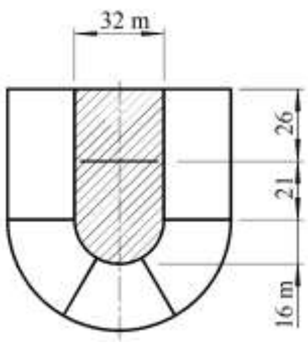
crossref http://dx.doi.org/10.5755/j01.mech.21.1.10129

1. Introduction

While designing the public buildings there is an intention to emphasize the unique character of architectural treatments and engineering decision of the design authors as well as to satisfy the requirements of functionality and economical efficiency of the structure. In case of such an approach the most presentable halls should be covered with large–span structures using non–standard decisions. In Lithuania as well as in other countries design codes do not directly regulate design of non–typical structures [1]. Therefore, one should more carefully investigate the mechanical state of such system paying attention to the particular qualities and providing analysis of the most complicated fragments [2-4]. The numerical simulation methods are considered to be the most convenient and important means used in modern computational mechanics. These methods become more efficient when creating and employing an original calculation algorithm for a specific structure to be designed [5-15].

The structures of a public building divided into 6 independent temperature deformation blocks (Fig. 1, a) are investigated in this article. The building in plane is symmetric with respect to the longitudinal axis. We shall consider a central block consisting of two storeys and covered by means of a no–standard system of plane steel trusses (Fig. 1, b). In the round part of the block radially located semi–trusses form a half of the cupola, which is connected with parallel trusses, placed with a regular step along the building. Over trusses there are steel structures of the bulk–head, in the building plane which occupy the middle roof section both above rectangular and round parts (Fig. 1, c).

During analysis of stress/strain state of the structural members the roof bearing trusses are considered as two versions: individually plane steel trusses and semi–trusses with specific simplifications or as trusses and semi–trusses connected by braces as a part of the spatial system. Selection of cross sections of the steel truss members is carried out on the basis of the bearing capacity, taking into account valid national design codes [16].



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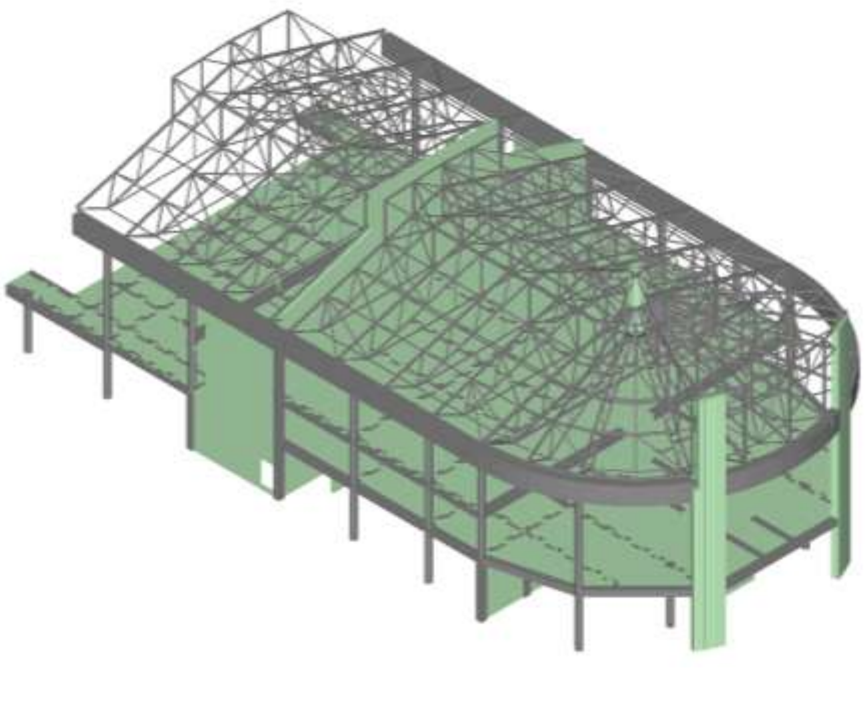

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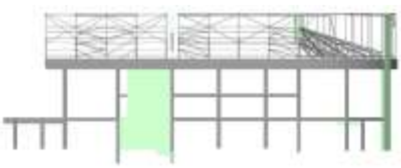


c

Fig. 1 A scheme of building deformation blocks (a), a general (b) and side (c) views of the central block model

2. Main parameters of the structures

The structures of the central block are separated from nearby located blocks by means of a movement joint. The block dimensions in plane 32×63 m, the height from the first storey floor up to the ridge of the roof is 22 m, basements are not provided. The main bearing structures of the building have been designed of monolithic reinforced concrete: pile–supported foundations, columns, beams, beam slabs and walls. In the middle of the central deformation block a reinforced concrete wall is provided (for fire prevention reasons), which is lifted over the roof surface and divides the bearing steel structures (in a sense of structural mechanics) in two absolutely independent parts. The steel trusses overlapping a 32 m span rest upon a contour reinforced concrete beam of 180 m height and 80 cm width. The function of this beam is to unite free columns of the second storey and to restrict lateral forces, which appear at deformation of the trusses.

In our investigations the main attention is paid to steel roof structures of the central block within the part of a round semi–cupola end. In the building plan steel trusses are located in the following order: starting from the reinforced concrete wall the trusses are placed in parallel with a step of 5 m in the block middle, in the round sector – in a radial manner with an angular step of 15 degrees. Thus, we get a mixed system as the cupola bearing members are closed by trusses and braces, which are located in orthogonal way (Fig. 2). For natural lighting of the interior a bulk–head is provided above trusses.

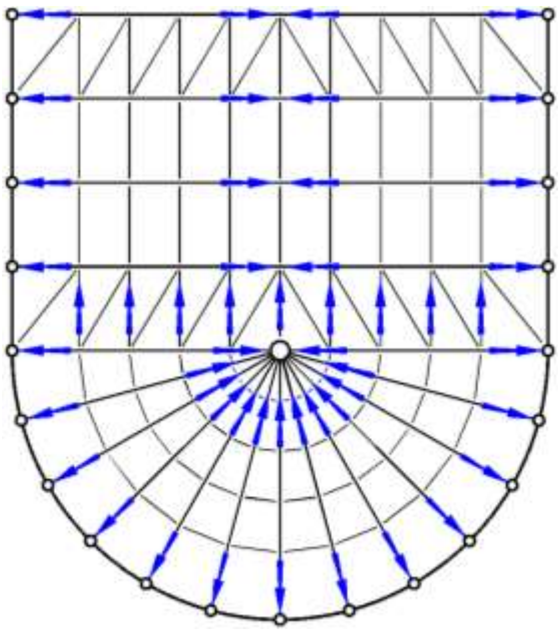
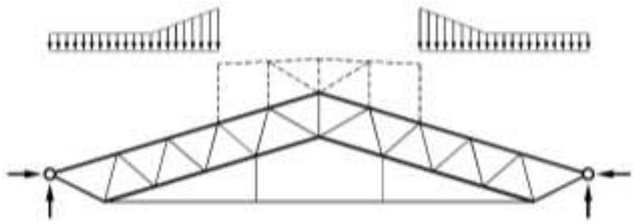


Fig. 2 A general distribution of internal forces in plane of the roof structures on the upper chord of the trusses

When creating the calculation model, such factors on the roof have been considered: selfweight, weight of enclosing constructions, weight of engineering networks and equipment, snow weight, wind pressure. Height of snow coverage near the bulk–head has increased according to diagrams indicated in the design codes (Fig. 3). Loads, caused by the roof weight, snow weight and wind pressure are applied to the upper chord of trusses and cupola semi–trusses as well as to the bulk–head transverse beams. Dead load of floors and also life load and technological loads are being applied to the intermediate floor (between the first and second storeys). These actions affect the roof structures slightly. Influence of thermal and climatic effects, changes in humidity and concrete shrinkages have been excluded by technological and constructive measures taken in proper time. In calculations coefficients for dead and life loads are taken according to the requirements of design codes valid in Lithuania [17]. Reliability class of the building is RC3 [1].



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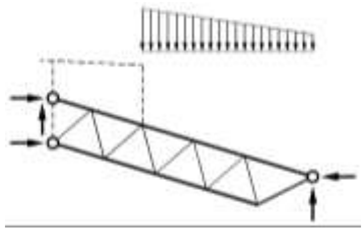


b

Fig. 3 A plane truss of the orthogonal part (a) and a plane semi–truss of the radial part (b) of the roof structures under snow load (with support reactions)

In a classical formulation all segments of a cupola, situated as axially symmetric ones, should compensate effect of opposite segments. In cupola case the central point of the cupola is theoretically not displaced. On the other hand, a traditional system of conventionally independent plane frames with plane trusses should deform symmetrically about the longitudinal axis of such building, internal forces must be also distributed in a symmetric way. In case of both radial and parallel locations the united mechanical work of trusses is provided by the system of special braces. In our problem (Fig. 2) a half of the cupola acts on plane trusses in out–of–plane direction and therefore the system of vertical and horizontal braces performs not only a uniting function (during erection of the roof structures and also at distribution of internal forces appearing as a result of wind effect), but it works as a bearing system as well, because it can be considered in some way as additional vertical and horizontal trusses. Taking into account the one–sided character of the load from the semi–cupola, lateral skew braces, in which tensile is acting, are provided. Increase in height of the truss from vertical braces (along symmetrical axis of the whole building) due to the presence of the bulk–head considerably enlarges the roof structural stiffness in vertical direction and redistributes internal forces caused by a significant horizontal load. Stiffness of the bulk–head exerts a slight effect on stiffness of the transversal trusses.

The connecting truss, which contains the cupola centre, plays the leading part in distribution of the internal forces and decrease in deformations. Just on the line of this truss the parallel order of roof structures meets the radial one. To restrict shift effect in transversal trusses a horizontal guy is designed. Besides, due to the availability of the semi–cupola (semi–trusses of which are without guys) the connecting truss turns to be loaded much more than others. The cupola centre carries a stiff vertical cylinder of 1.2 m diameter, which serves as a support for upper and lower chords of the semi–trusses.

3. Stages of investigations, general characteristics

Each plane steel truss (Fig. 3) consists of two rigidly connected equal semi–trusses and a sufficiently flexible guy, which is fixed in the horizontal position according to technological and aesthetic requirements. The truss rests upon a contour beam by means of hinges. In order to study thoroughly the stress/strain state of the transverse truss [18-21], three cases of boundary conditions have been calculated (Fig. 4):

a) vertical and horizontal shifts of the support joints are completely limited;
b) only settlement of support joints is restricted, in the direction of shift the supports are free;
c) vertical and horizontal shifts of the support joints as well as a sag of the truss central section are restricted by springs of finite stiffnesses.

The last described case is considered as an intermediate one between the first and second cases during analysis of the horizontal shifts in the support joints. The first and second problems are solved on the plane truss, the third one – on the spatial truss, i. e. one considers the separate trusses as components of the roof structural system.

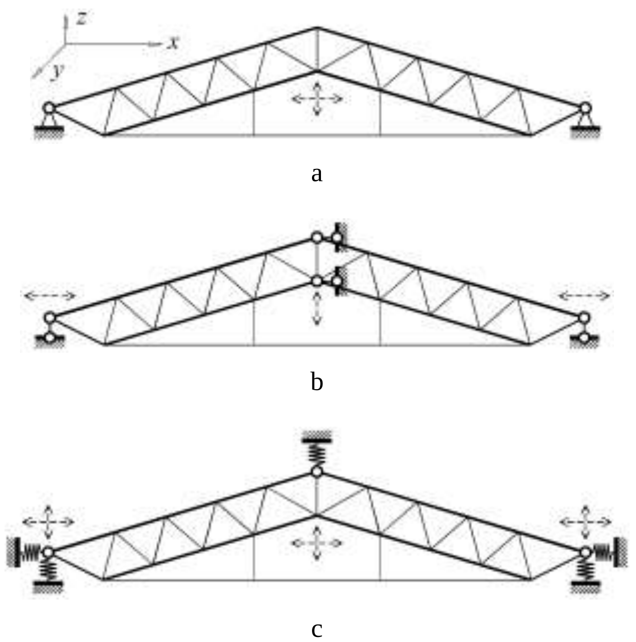


Fig. 4 Calculated cases of the truss, while the support joints are: fixed (a), vertically fixed and horizontally free (b), restricted by springs of finite stiffnesses (c)

In the third case the vertical support spring in a physical sense expresses a compression stiffness of situated below reinforced concrete columns, the horizontal one – a stiffness in bending of both the columns and the contour beam. Besides, the vertical stiffness is much more than that horizontal one. The conventional vertical stiffness in the truss centre appears due to the presence of constructively designed vertical braces between transverse trusses, connected with the semi–cupola centre.

The cupola semi–trusses are arranged without any guys (Fig. 3). By analogy with the whole truss (Fig. 4), shift in horizontal direction in two top joints of semi–truss (Fig. 5) is eliminated, imitating symmetry conditions. Besides, three cases of support springs corresponding to the mentioned above are being considered. In the third case concerning the semi–truss the stiffness of the horizontal springs in both top joints is expressed by stiffness of the vertical braces along the symmetry axis of the roof on the level of trusses and bulk–head (Fig. 6). The above described lateral stiffness is complemented by a diaphragm of obliquely located braces between parallel trusses (Fig. 2).

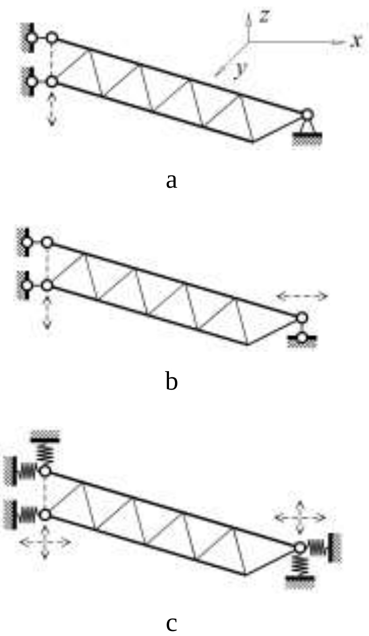


Fig. 5 Calculated cases of the semi–truss, while the bottom support joints are: fixed (a), horizontally free (b), restricted by springs of finite stiffnesses (c)

Attention should be paid to the fact, that in practice (Fig. 7) the stiffness of braces is influenced by inaccuracies of assembling and other factors (shift details in the bolt holes of joints and etc.) and therefore investigation of limited cases (ideally rigid and ideally movable) is of great importance.

In calculation of reinforced concrete members concrete of class C25/30 is used, Young's modulus 31 GPa, Poisson's ratio of 0.20, weight 25 kN/m^3 [22]. For steel members the steel class S355 is applied, Young's modulus 206 GPa, Poisson's ratio of 0.30, weight 77 kN/m^3 [16]. It is provided to set up a steel profiled sheeting over the roof trusses and insulation layers, diaphragm stiffness of which has not been taken into account during analysis of the truss problem.

While designing trusses and braces the steel members of square and rectangular hollow cross sections have been used. If one takes as reference quantities the cross–sectional area A_{max} and moment of inertia I_{max} of the connecting truss top chord, then axial and bending cross–sectional stiffness values *A* and *I* of all other members are expressed by the ratios (Table 1).

Fig. 6 A view of the central vertical braces

Fig. 7 A support joint of the semi–truss during erection of the cupola

In a created finite element structural model chords of the trusses and lattice supporting members have been modelled by bending finite elements, other structural members of the lattice – as truss–type finite elements. A hinged joint has been simulated for connecting of semi–trusses in the semi–cupola centre. All structural members of the braces are simulated as truss–type finite elements, i. e. hinged ones.

Table 1

Relative values of cross–sectional geometric characteristics of steel structural members of the roof

Main bearing structures of the roof	Structural members	Relative cross–sectional geometric characteristics		
		Relative area $\frac{A}{A_{max}}$	Relative moment of inertia $\frac{I}{I_{max}}$	
			in–plane	out–of–plane
Common transverse truss	Upper chord	0.51	0.336	0.153
	Bottom chord	0.29	0.079	0.079
	Supporting members	0.31	0.047	0.047
	Lattice	0.19	0.015	0.028
	Guy	0.31	0.047	0.047
Connecting truss	Upper chord	1 (etalon)	1 (etalon)	0.528
	Bottom chord	1.00	1.000	0.528
	Supporting members	0.70	0.266	0.266
	Lattice	0.30	0.037	0.068
	Guy	0.62	0.412	0.186
Radial semi–truss	Upper chord	0.51	0.336	0.153
	Bottom chord	0.29	0.079	0.079
	Supporting members	0.23	0.039	0.039
	Lattice	0.19	0.015	0.028
Horizontal braces between transverse trusses	Longitudinal	0.16	0.019	—
	Diagonal	0.19	0.033	—
Vertical braces along a line of symmetry of the roof	Longitudinal	0.29	0.079	—
	Diagonal	0.29	0.079	—
Braces of semi–trusses	Circular	0.16	0.019	—
	Diagonal	0.16	0.019	—
The bulk–head structural members	Vertical	0.29	0.079	—
	Horizontal	0.29	0.079	—
	Diagonal	0.16	0.019	—

4. Analysis of results

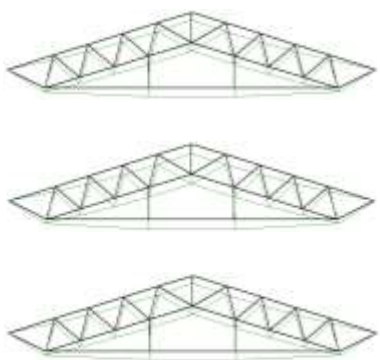
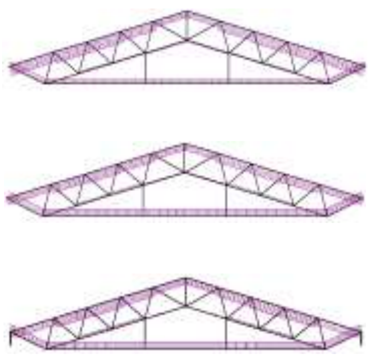
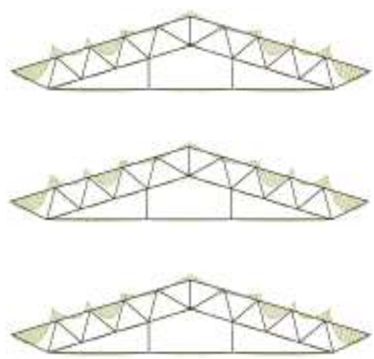
In order to investigate comprehensively the mechanical state of the bearing trusses of the asymmetric roof 9 different variants has been studied:

1) common truss with fixed supports (Fig. 4, a);
2) common truss with shifting supports (Fig. 4, b);
3) common truss from the spatial model (Fig. 1);
4) connecting truss with fixed supports (Fig. 4, a);
5) connecting truss with shifting supports (Fig. 4, b);
6) connecting truss from the spatial model (Fig. 1);
7) semi–truss with fixed supports (Fig. 5, a);
8) semi–truss with shifting supports (Fig. 5, b);
9) semi–truss from the spatial model (Fig. 1).

Loads and cross sections for common trusses, connecting trusses and radial semi–trusses were different. Therefore in our paper, the stress/strain state parameters are being analysed in terms of the quality (Table 2), this fact is very important for any rational decision of the engineering problems [23-25]. The results of investigating more conservative plane structural models are compared with those of the spatial model.

Table 2

Relative values of cross–sectional geometric characteristics of the steel structural members of the roof

Variant No	Deformed shape	Diagram of axial forces	Diagram of bending moments
1			
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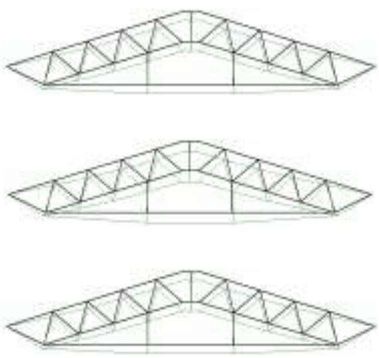
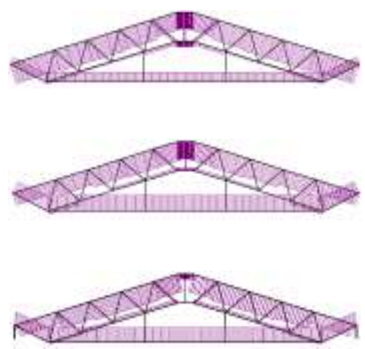
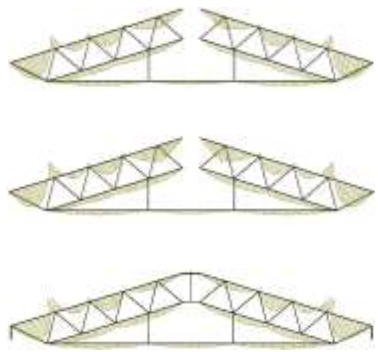
4			
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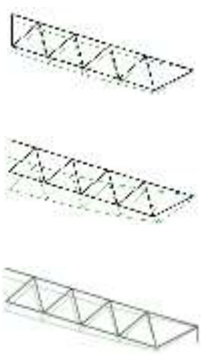
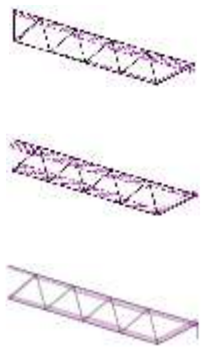
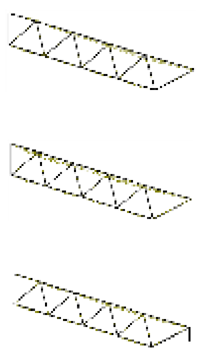
7			
8			
9			

Out of 4 common trusses the most loaded truss due to redistribution of internal forces in the spatial system is that one, which is the nearest to the connecting truss. It occurs because of skew horizontal braces, which partially transfer internal forces from the semi–cupola pressure along the building to the horizontal plane. Out of 11 radial semi–trusses the most acted by snow load is the one, located at an angle of 45 degrees with respect to the building axis of symmetry. Just the above mentioned fragments of the general spatial model have been compared with identical fragments, which were considered during calculation of the plane problems. Vertical displacements of the trusses in the spatial model have been calculated taking into account displacements of the contour beam.

Visual results of calculations (Table 2) are added by results in relative values (Table 3). Signs of displacements correspond to the system of axes for an individual truss (Fig. 4) and semi–truss (Fig. 5). Vertical displacement u_{etalon} of 31.8 mm in the third variant is taken as the reference one, the rest displacements are considered as relative.

Table 3

Relative values of stress/strain state parameters of steel trusses of the roof

Variant No	Relative extreme displacements $\frac{u}{u_{etalon}}$			Relative values of maximal or minimal internal forces $n = \frac{N}{N_{etalon}}$ and $m = \frac{M}{M_{etalon}}$				
	axis x	axis y	axis z	n_1	n_2	n_3	n_4	m
1	——	——	–0.85	–1.14	+0.16	+0.45	–0.22	+0.96
2	±0.10	——	–1.00	–0.98	+0.22	+0.71	–0.31	+0.96
3	+0.03	–0.30	–1 (etalon)	–1 (etalon)	+0.23	+0.66	–0.34	+1 (etalon)
4	——	——	–0.91	–2.21	–0.33	+0.98	–0.26	+1.39
5	±0.11	——	–1.13	–1.93	–0.32	+1.60	–0.48	+1.30
6	–0.01	–0.26	–0.93	–1.76	+0.32	+1.34	–0.55	+1.30
7	——	——	–0.61	–0.86	+0.26	——	–0.20	+0.17
8	+0.33	——	–1.67	–0.84	+0.84	——	–0.37	+0.26
9	–0.27	+0.18	–0.93	–0.40	+0.40	——	–0.25	+0.13

Note: the table describes relative axial forces of the trusses: n_1 – in the upper chord; n_2 – in the bottom chord; n_3 – in guy; n_4 – in the lattice. A letter m denotes a relative bending moment.

Deformed shapes of trusses in variants 1 to 6 are of similar character. The largest vertical displacement appears in the truss centre. Due to the shift restriction a sag in the 1st variant is by 15% less than that one in the 2nd and 3rd variants. Equal sags in the 2nd and 3rd variants point to a considerable flexibility of the columns and the contour beam. For variants 4 to 6 the situation is another one: a sag in the 4th variant is by 22% less than in the 5th one and the sag in 6th variant is by 20% less than in the 5th one. This indicates to the considerable increase in the contour stiffness due to proximity of the roof round sector. A significant increase in the sag for variants 7 to 9 is explained by load increase and relative decrease of the semi–truss stiffness.

Values of displacements from the spatial model get into an interval between the values obtained during analysis of the plane models. The general deformed shape (Fig. 8) confirms the predicted distribution of internal forces in building plane: in the transversal direction the structures are displaced symmetrically as well as in the longitudinal one – the central sector of the roof is moved due to action of the semi–cupola (Fig. 9). According to the requirements of the design codes [17] the sag limiting value constitutes 1/286 of the span length. The maximum sag value obtained in calculations does not exceed a half of the limiting value.

Values of an axial force –574 kN in the upper chord of the common transversal truss (compression is designated by a negative sign) and bending moment of 23 kNm are also taken as reference values (Table 3).

In the common trusses and connecting truss the dominating structural members are the upper chords and guys. With restriction of shifts in the 1st and 4th variants the maximum values of the compressive axial force in the upper chord appears and tension in the guy in this variants is less than in other 2nd, 3rd, 5th and 6th variants. Due to a large loading area internal forces in the connecting truss almost twice as large as those of the common truss, but the selected cross sections (Table 1) are also twice more. The bottom chord of the trusses gets into a zone of the neutral line, therefore the internal forces are insignificant and even change the sign.

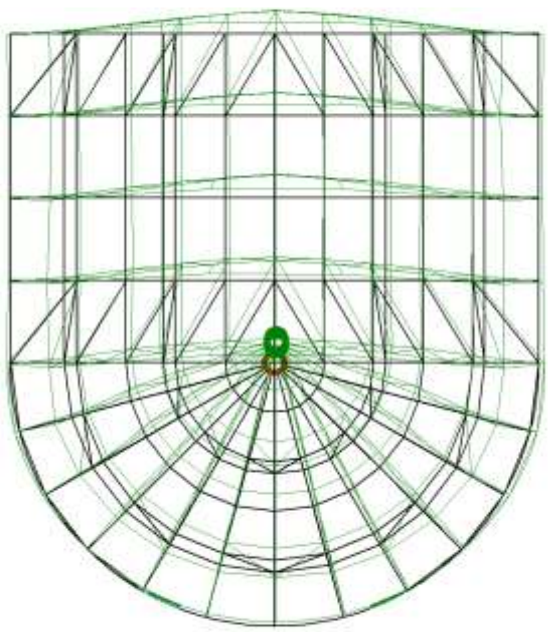


Fig. 8 A deformed view of the whole roof in plane

In the semi–trusses of the semi–cupola the distribution of axial forces is different: in the 7th variant an axial force in the upper chord exceeds that one in the bottom chord more than by a factor of two; in the 8th variant the internal forces in the upper and bottom chords are almost equal; in the 9th one – internal forces in the upper and bottom chords are also almost equal, but approximately less by a factor of two in comparison with the 8th variant. Distribution of axial forces for all three variants undoubtedly depends on kinematic conditions of the structural model, actually the most real one is considered in the 9th variant.

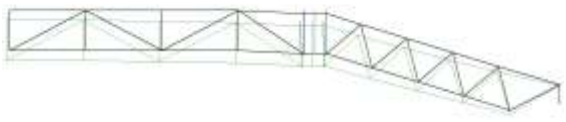


Fig. 9 A deformed view on the central axis of the roof

The given values and bending moment diagrams confirm the above mentioned assumptions concerning the mechanical state of the structural system.

5. Conclusions

On the basis of the above presented investigation of the mechanical state of the spatial roof from steel trusses on the asymmetric building the following conclusions have been briefly made:

1. Taking into account the deficiency of instructions (in design codes of Lithuania) concerning design of asymmetric roof systems it is necessary to investigate individually such engineering problems by using modern methods of numerical modelling.

2. To determine possible intervals of changing internal forces and displacements as well as to investigate influence of below located structures on the asymmetric roof from steel trusses, two partial plane models with different assumptions and the general spatial system have been successfully considered and comparison of the results has been performed.

3. Analysis of sags of the trusses has demonstrated that displacements of a common transverse truss within the general spatial system are almost not districted by the contour support beam as these ones are identical to the sag of the truss with free supports, but in the connecting transverse truss this restriction is considerable (difference from ideal fixing 2 % in total), because it is provided by the curved contour of the support beam (a sag of semi–trusses is kinematically dependent on that one of the connecting truss).

4. Analysis of distribution of internal forces on the upper chords of trusses has illustrated, that internal forces in the connecting truss are approximately twice more than those in the common truss and as internal forces in the radial truss are approximately less twice. Thus, in real designing the connecting truss gets much more attention.

5. Investigation of distribution of axial forces in vertical and horizontal braces of the roof has shown that vertical braces placed on the axis of symmetry of the building play the key role in the mechanical state of the structural system, as they take and transfer pressure of the semi–cupola to the horizontal braces as well as to the upper chords of common transverse trusses.

The presented design technique was successfully realized in practice.

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INVESTIGATION OF MECHANICAL STATE OF SPATIAL ROOF FROM STEEL TRUSSES ON ASYMMETRIC BUILDING

S u m m a r y

Investigation of the stress/strain state of the spatial roof from steel trusses over asymmetric in plan the public building is presented in the paper. The radially located bearing structures of a semi–cupola are closed by the orthogonally arranged system of common transversal trusses and longitudinal braces. The trusses of 32 m span have been simulated as a separate plane structural model and as a main bearing part of the general model of the roof as well as of the whole building. Nine different variants of the steel truss have been calculated and peculiarities of change in the mechanical state parameters have been analysed. Comparison of shifts and internal forces has been performed, besides description of a deformed shape of the roof structures and distribution of internal forces in the asymmetric mechanical system are presented.

Keywords: asymmetric roof, steel trusses, semi-cupola shape, complex deformation, stress/strain analysis.

Received September 25, 2014
Accepted January 12, 2015