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The influence of the weight of the roof covering and the period of construction of structures with steel truss roofs on their safe exploitation

Wpływ ciężaru pokrycia dachowego i okresu powstania obiektów z dachami o kratownicowej konstrukcji stalowej na ich bezpieczną eksploatację

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Abstract. The article describes changes in the way of taking into account the snow loads and dead loads in the static calculations of steel roof girders over the last 60 years. An analysis of the load capacity utilization of structural elements of three steel hall roofs was also carried out. When using light covering layers and without technological spaces, the change in the design value of the total load on the roof resulting from different calculation rules at the time of their design period may exceed 25%. Such a large increase in load, especially during heavy snowfall, may in many cases lead to a change in the load capacity utilization of structural elements exceeding the permissible values, which may result the failure of the object.

Keywords: design; steel girders; snow load.

Streszczenie. W artykule opisano zmiany w podejściu do uwzględniania obciążenia od śniegu i ciężaru własnego w obliczeniach stalowych dźwigarów dachowych na przestrzeni ostatnich sześćdziesięciu lat. Przeprowadzono również analizę wytężenia elementów konstrukcyjnych dachów hal stalowych na przykładzie trzech obiektów. W przypadku zastosowania lekkich warstw pokrycia i bez przestrzeni technologicznych, zmiana wartości obliczeniowej sumarycznego obciążenia połaci, wynikającej z innych zasad obliczania w okresie projektowania i obecnie, może przekraczać 25%. Tak duży wzrost obciążenia, szczególnie podczas obfitych opadów śniegu, w wielu przypadkach może prowadzić do zmiany wytężenia elementów konstrukcji, przekraczającej wartości dopuszczalne, czego konsekwencją może być awaria obiektu.

Słowa kluczowe: projektowanie; dźwigary stalowe; obciążenie śniegiem.

Roofs with a steel truss structure are among the lightest found in industrial construction. In the case of this type of structure, the percentage of the self-weight of the structure itself in all loads occurring during operation is usually not dominant. The type of roofing structure determines whether variable climatic loads prevail over dead loads or not. Global climate warming contributes to an increase in the probability of extreme weather phenomena [1 – 2]. On the one hand, climate change reduces the average amount and frequency of snowfall, but on the other hand, the risk of extreme snowfall increases [3]. The risk of storms with strong winds is also increasing [2, 4 ÷ 6]. This leads to an increase in the value of live loads on the structure, which may contribute to its destruction. This results not only in significant economic damage, but above all in the death of people [7 ÷ 9].

Objects at risk of collapse include large-span halls with lightweight roofs. One such example is the disaster of the Katowice International Fair building in 2006. It was the most tragic disaster in which 65 people died and over 140 were injured. The steel structure hall had external dimensions of 96 × 103 m. One of the causes of the disaster was the uneven snow load on the roof [10]. Also in Bavaria, Germany, in 2006, due to the increase in snow load, there were several damages to roof structures, including several halls [11]. In turn, in 2010, as a result of significant snowfall, part of the roof of the warehouse hall in Stalowa Wola was destroyed. The steel structure hall had an area of approximately 200 square meters [12]. In the same year, after snowfall, there was an unsignaled collapse of a section of the roof of the shopping pavilion. 3 steel truss girders with a spacing of 6 m, located in the central part of the facility, were damaged [13]. The span of the girders was 24 m. Fortunately, no one was injured as a result of both incidents. Also in 2023, in Spytkowice, in Małopolska, the roof of a warehouse hall collapsed under the pressure

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of snow [14]. Rapid climate changes, as well as changes in the approach to taking into account snow and wind loads in the calculations of structural elements over the last 60 years, make it necessary to take a closer look at the objects with the structure described above that were built at the beginning of this period.

The rest of the article describes changes in load acceptance and an analysis of the effort of truss elements on the example of three objects. Due to the small slope of the roof slopes of the analyzed facilities and the related effect of relieving the structure, taking into account the calculation combination including the influence of wind (suction), its influence was omitted in the further analysis of climatic loads.

Changes in the approach to taking into account dead loads and climatic loads over the last 60 years

Over the course of several decades, the standards on the basis of which the loads on structures were calculated have changed. In the 1960s, these were the standards PN-B-02009:1960 [15] and PN-B-02010:1957 [16]. In the 1970s, the PN-B-02009:1974 [17] and PN-B-02010:1970 [18] standards. In turn, until the end of 2020, when taking into account loads in design, it was possible to use the standards PN-B-02001:1982 [19] and PN-B-02010:1980 + Az1 (October 2006) [20]. Currently, we use Polish Standards introducing European standards PN-EN 1990 [21], PN-EN 1991-1-1 [22] and PN-EN 1991-1-3 [23]. The standards for designing steel structures were calibrated with the above-mentioned standards. In the 1960s and 1970s, it was the PN-B-03200:1962 standard [24], until the end of 2020, the PN-B-03200:1990 standard [25], and currently the PN-EN 1993 standard [26]. The basic difference in the approach to design at the beginning of the analyzed period and now is the calculation method. **In the 1960s, structures were designed using the allowable stress method. Currently, we design using the limit state method.**

A global analysis of standards calibrated with each other in a given period specifying load values and providing calculation methods allows for the separation of partial safety factors adopted in the calculations.

In the 1960s, in accordance with the standards [15, 16], these were the load factors α with the following values:

- α = 1,1 for self-weight loads of steel structures;
- α = 1,2 for other materials;
- α = 1,15 for snow loads.

In the 1970s, in accordance with the standards [17, 18], these were the load factors with the following values:

- α = 1,1 for self-weight loads of structures;
- α = 1,2 for roof covering layers;
- α = 1,4 for snow loads.

Until the end of 2020, in accordance with the standards [19, 20], we used load factors γ_f with the following values:

- γ_f = 1,1 for self-weight loads of structures;
- γ_f = 1,2 for prefabricated roof covering layers;
- γ_f = 1,5 for snow loads.

Currently, based on the PN-EN 1990 standard [21], in the ultimate limit state, combinations of loads in the case of permanent or temporary design situations are calculated according to formula 6.10:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}Q_{k,1}\text{"+"}\sum_{1>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \quad (1)$$

where:
$\gamma_{G,j}$ – partial coefficient for dead load j;
$G_{k,j}$ – characteristic value of dead load j;
γ_P – partial coefficient for prestressing loads;
P – reliable representative value of the prestressing load;
$\gamma_{Q,1}$ – partial coefficient for the leading live load;
$Q_{k,1}$ – characteristic value of the leading live load;
$\gamma_{Q,i}$ – coefficient for the remaining live loads i;
$\Psi_{0,i}$ – coefficient for the combination value of live loads i;
$Q_{k,i}$ – characteristic value of the remaining live loads i.

or alternatively according to formula 6.10a:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}\Psi_{0,1}Q_{k,1}\text{"+"}\sum_{j>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \quad (2)$$

and according to formula 6.10b:

$$\sum_{j\geq 1}\xi_j\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}Q_{k,1}\text{"+"}\sum_{j>1}\gamma_{Q,i}\Psi_{0,i}Q_{k,i} \quad (3)$$

where:
ξ_j – reduction factor for unfavorable dead loads j.

However, in the National Annex NB [20] there is a recommendation to adopt the less favorable combination of loads calculated according to formula (2) and (3) when checking the limit states STR and GEO.

The values of the partial load factors are:

- according to formula (1): $\gamma_{G,j}$ = 1,35 for dead loads and $\gamma_{Q,1}$ = 1,5 for snow loads (leading live load);
- according to formula (2): $\gamma_{G,j}$ = 1,35 for dead loads while assuming the coefficient $\gamma_{Q,1}\Psi_{0,1}$ = 1,5 • 0,5 = 0,75 for snow (leading live load for facilities located at an altitude of up to 1000 m above sea level in CEN member states, except Finland, Iceland, Norway, Sweden);
- according to formula (3): $\xi_j\gamma_{G,j}$ = 0,85 • 1,35 = 1,15 for dead loads while assuming the coefficient $\gamma_{0,1}$ = 1,5 for snow (leading live load).

Characteristic values (formerly standard) of snow load on the ground in central Poland, assumed in accordance with the above-mentioned design standards, ranged from 60 kg/m^2 (in the 1960s), which corresponds approximately to the value of 0.6 kN/m^2 to 0.9 kN/m^2 (current).

The above-mentioned values lead to the conclusion that the snow loads taken into account in the calculations of flat roof construction elements increased significantly during the analyzed period.

The snow load S_0 taken into account in the calculations in the 1960s in accordance with the standard [16] was therefore:

$$S_0 = 1{,}15 \cdot S \quad (4)$$

where:
S – weight of snow cover depending on the country zone and roof slope.

For a facility located in central Poland with a flat roof, the design load S_0 will be equal to:

$$S_0 = 1,15 \cdot 60\ kg/m^2 \cdot 1 = 69\ kg/m^2\ (\sim 0,69\ kN/m^2)$$

The design snow load S_d, as the leading live load according to currently applicable standards [21 ÷ 23] in a permanent and temporary design situation, is calculated according to the formula:

$$S_d = \gamma_{Q,1} \mu C_e C_t s_k \tag{5}$$

where:

$\gamma_{Q,i}$ – partial coefficient for the leading live load;
μ – roof shape coefficient;
C_e – exposure coefficient;
C_t – thermal coefficient;
s_k – characteristic value of snow load on the ground.

For the same object for normal terrain (according to the National Annex NA): $\gamma_{Q,1}$ = 1,5; μ = 0,8; C_e = 1; C_t = 1; s_k = 0,9 kN/m² (zone 2). The snow load will therefore take the value:

$$S_d = 1,15 \cdot 0,8 \cdot 1 \cdot 1 \cdot 0,9\ kN/m^2 = 1,08\ kN/m^2$$

Therefore, this value is 56% higher than that calculated according to formula (4).

The differences in the values of design loads from the self--weight of roofing layers have not changed that much over the years. For typical structures with an average value of the characteristic dead load of the covering layers of 0.7 kN/m², for which the leading calculation combination takes into account the full snow load, the design values of the dead loads currently assumed in the calculations are G = 1,15 • 0,7 kN/m² = 0,80 kN/m². For the same type of covering, in the calculations in the 1970s we would assume a design dead load of G = 1,2 • 0,7 kN/m² = 0,84 kN/m².

Types of roof coverings used

The weight of the layers placed on the upper chords of the steel trusses constituting the roof structure, taking into account the own weight of the covering structure, ranges from 0.15 kN/m² to 2.5 kN/m². The lightest solution are sandwich panels with sheet metal cladding, the individual layers of which carry daed and live loads, act as thermal insulation and waterproof insulation. We also come across sandwich panels with cement-fiber cladding, which were used eagerly in the 1970s and 1980s and act as a load-bearing structure and thermal insulation. Several layers of bituminous paper were additionally placed on these boards as waterproof insulation. The most common type of roofing structure is trapezoidal sheet metal, on which a layer of thermal insulation and a layer of waterproof insulation are placed. The last, and at the same time the heaviest, solution used is a structure made of massive, reinforced concrete, prefabricated core boards on which a layer of thermal insulation and waterproof insulation is placed.

It is easy to see that with such a large difference in the self-weight of the roofing layers, the change in the value of the snow load assumed in the calculations is particularly important for lightweight structures, for which the percentage of the snow load in transferring all the loads applied to the structure is significant.

In facilities where the main roof structure is made of steel trusses, the space between the lower and upper chords of the trusses is often used for the distribution of technological installations. This space can be closed with a suspended ceiling, and access to the installation is provided by installing working platforms. Loads from platforms and installations are most often transferred directly to the main roof girders without adding any load to the purlins.

Analysis of the effort of roof truss elements for selected objects

The analysis was carried out on the example of three facilities constructed in Łódź and Toruń in the 1960s and 1980s [27 ÷ 29]. In each of these cases, checking the effort of structural elements was related to checking the possibility of adding weight to the roof during thermal modernization of the facility. It involved installing additional thermal insulation and a new waterproof covering on the existing roofing layers and installing additional ventilation units.

The first structure is a five-nave structure with a column grid of 12.0 m x 12.0 m (figure 1). The main truss roof girders with a span of 12.0 m are also 12.0 m apart. The girders are articulated supported on multi-branch steel columns rigidly mounted in the foundations. The height of the girders in the ridge is 1.2 m. The height at the supports is 0.6 m. The upper chord of the girders is made of 2 L 90 x 90 x 8, the lower chord is made of 2 L 65 x 65 x 9. Side compressed cross bracewith 2 L 60 x 60 x 6. The rest of cross braces and posts are made of L 45 x 45 x 5. Ridge post with 2 L 45 x 45 x 5. Truss purlins with a construction height of 0.8 m are supported on the girders. The purlins are spaced ~3.0 m apart. The upper chord of the purlins is made of 2 L 60 x 60 x 6, the lower chord is made of L 60 x 60 x 6. Cross braces with L 45 x 45 x 5. The lower chord is reinforced in the middle three fields with a welded Ø 12 rod. Each of the main truss girders is protected against rotation in the middle of its span by a brace with L 60 x 60 x 6 from the lower chord of the purlin to the lower

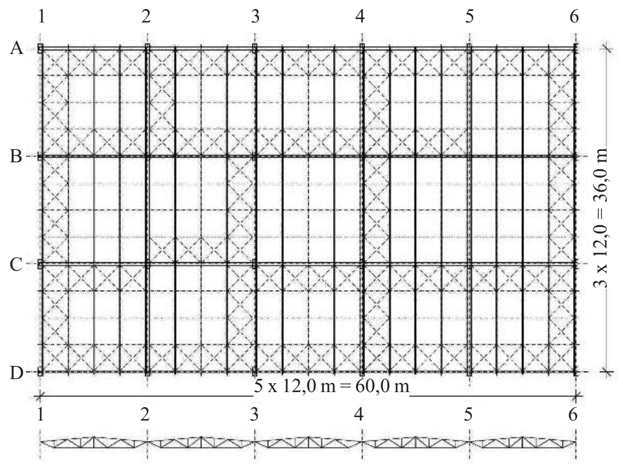

Fig. 1. Scheme of the structure of the first analyzed object
Rys. 1. Schemat konstrukcji pierwszego analizowanego obiektu

chordof the girder. The roof purlins are stiffened between each other with truss braces placed at a spacing of ~3.0 m. Braces 0.8 m high with upper and lower chords made of L 45 x 45 x 5 and cross-braces made of Ø 18 rod. Additional stiffening of the roof slope are cross-slope braces placed in the plane of the upper chord of the truss purlins. Bracing with L 45 x 45 x 5. The design weight of the existing roofing layers is 1.06 kN/m^2. The facility does not have any technological space under the roof.

The second structure is single-nave with truss girders with a span of 27.0 m at 6.0 m spacing. Trusses with a height at the ridge of ~2.4 m (figure 2). Gable roof girders with an upper chord of ½ I 360, a lower chord of ½ I 360 and cross braces with L 65 x 7, 2 L 65 x 7, L 75 x 9, 2 L 75 x 9 and 3 L 75 x 9. The purlins are made of hot-rolled [160 at 3.0 m spacing. The design weight of the existing roofing layers is 0.4 kN/m^2. The facility has a technological space from which the load with a design value of 0.39 kN/m^2 is transferred directly to the lower bars of the roof girders.

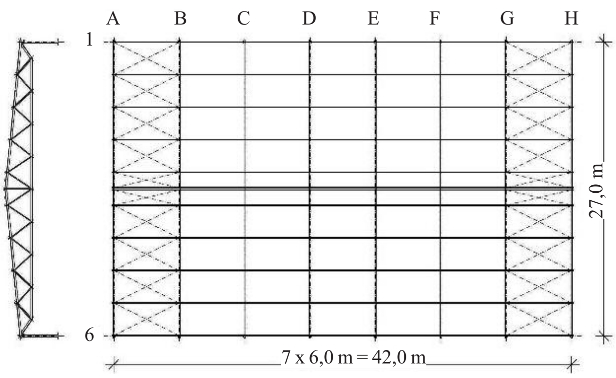

Fig. 2. Scheme of the structure of the second analyzed object
Rys. 2. Schemat konstrukcji drugiego analizowanego obiektu

The third structure is a single-nave structure with truss girders with a span of 36.0 m and spacing of 5.4 m (figure 3). Truss girders with a lower chord of 2 [ ] 200 with battens. Upperchord with 2 [ ] 200 with battens. In the places where the bars are joined along the length, there are 1.1 m long ½ IPN300 covers on both sides. The side cross braces are

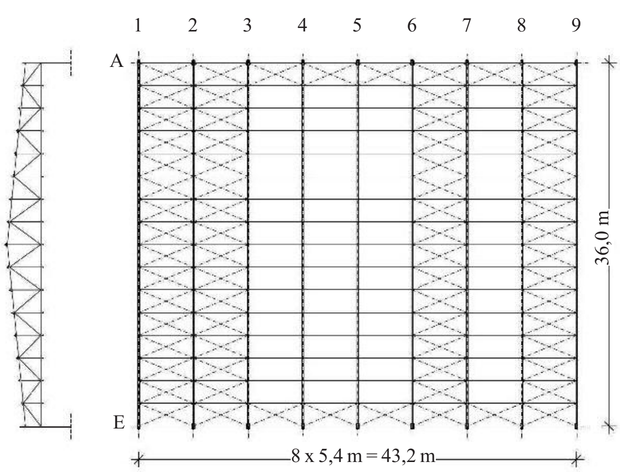

Fig. 3. Scheme of the structure of the third analyzed object
Rys. 3. Schemat konstrukcji trzeciego analizowanego obiektu

made of 2 [ ] 140, the rest ofcross braces are made of 2 [80 or 2] [50 depending on the location. All truss posts are made of 2] [50. All posts and cross braces are with battens. The purlins are made of hot-rolled 2 L 100 x 150 x 12 (1 L 100 x 150 x 12 for the side purlins) at 2.25 m spacing (in all nodes of the upper chords of the roof girders). The design weight of the existing roofing layers is 0.45 kN/m^2. The facility has a technological space from which the load with a design value of 0.45 kN/m^2 is transferred directly to the lower chords of the roof girders.

The structure diagram of the three analyzed objects is shown in Figures 1, 2 and 3, respectively. A view of the technological space of one of the facilities is shown in the photo.

View of the technological space of one of the analyzed object
Widok przestrzeni technologicznej jednego z obiektów

The analyzed facilities were built during the periods of application of different sets of design standards, and while the values of dead loads used for calculations in the past and today are not much different, the snow load values used in calculations differ significantly. Table summarizes the design loads, dead and live, as well as the effort on the structural elements of the roofs of the buildings, divided into the construction period and the current period. The effect of wind load was omitted in the analysis because, in the case of relatively heavy roof coverings, does not result in an increase in the value of internal forces or a change in their sign in any case.

The table shows that the greatest increase in effort occurred in the roof structure of structure no. 1, for which

List of design actions and strength level of roof structural elements (without change to the layer arrangement related to the planned thermal modernization)
Zestawienie obciążeń obliczeniowych i wytężenia elementów konstrukcyjnych dachu (bez zmiany układu warstw związanych z planowaną termomodernizacją)

Facci-lity	Dead load [kN/m^2]	Snow load (in constructuion period) [kN/m^2]	Snow load (current) [kN/m^2]	Girder effort (in construction period) [%]	Girder effort (current) [%]
1	1,06	0,69	1,08	76	148
2	0,79	0,78	1,08	94	108
3	0,90	0,78	1,08	90	98

the value of the design snow load increased by as much as 56% compared to the period in which the structure was built. For structures no. 2 and no. 3, the increase in this load is 38% and for them the increase in structure effort is noticeably smaller.

Significant exceeding of the ultimate limit state of the truss girder elements in structure no. 1 resulted in the need to strengthen all girders. Due to the high usable height of the facility, it was decided to change the static scheme of the girder by installing tie rods made of a Ø 30 rod with a 60x60x4 squere pipe, 1.5 m long, on each girder. The strengthening made in this way allowed the girder effort to be reduced from 148% to 99 %. The girder reinforcement diagram is shown in Figure 4.

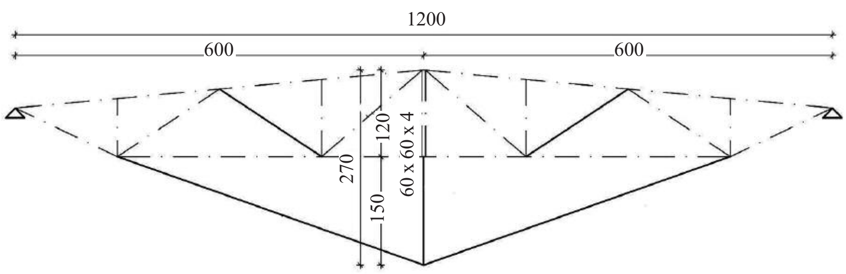

Fig. 4. Scheme of strengthening the truss girder of structure 1
Rys. 4. Schemat wzmocnienia dźwigara kratowego obiektu 1

Conclusions

The article analyzed selected roof structures made of steel trusses in the 1960s and 1980s. The ultimate limit state of the structural elements of the oldest of the analyzed objects was significantly exceeded.

In the case of steel roofs, structures built in the 1960s and 1970s, with light covering layers and without technological spaces, the change in the design value of the total load on the slope resulting from different calculation principles in the design period and currently may exceed 25%. In the event of snowfall exceeding the standard values, the exceedance may be even greater.

Such a large increase in load may, in many cases, lead to a change in the effort of structural elements exceeding the permissible values, which results in the need to reinforce the existing structure.

The conclusions contained in the article confirm the need to constantly monitor the actual weight of snow lying on the roof surfaces of buildings of the described type of construction and to remove its excess in order to prevent the facility from catastrophizing.

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