STEEL STRUCTURAL SOLUTION FOR EFFICIENT ROOF SYSTEM

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Abstract. The spatial behaviour of the roof is ensured by bracing in two truss directions that transfer external loads. Increased roof stiffness reduces internal forces and enables the use of thin-walled steel profiles with lower material consumption. The paper presents two finite element models of a steel roof structure for a frame building with plan dimensions of 18×24 m. The first model represents a widely used classical solution for industrial buildings, designed as a system of roof trusses F-1 with an 18.0 m span and 6.0 m bracing trusses F-2 at the ends of the roof diaphragm. Horizontal bracing is arranged along the bottom chords, and purlins along the top chords, contributing to the overall stiffness of the structure. All elements of this model are made of hot-rolled steel sections. The second model represents an efficient roof system composed of lightweight thin-walled cross trusses forming rigid rectangular closed blocks measuring 6×12 m. The members are made of cold-formed light-gauge steel sections with open cross-sections, with overall stability ensured by spatial perimeter blocks. This system forms a rigid roof diaphragm that governs the spatial behaviour of the building frame. Under Design Load Combination 2 (DLC2), the maximum vertical displacements f_z in Model No. 2 are 2.3 times lower than those in Model No. 1, indicating significantly higher stiffness and reduced deformability. The numerical study also showed that loads applied at any point of Model No. 2 are redistributed throughout the structure, ensuring integral structural action. The results identify the more efficient roof solution, including in terms of material efficiency.

Keywords: light gauge steel framing (LGSF), cold-formed steel sections, stiffness, deformability, material efficiency, finite element analysis, rafter trusses.

Introduction

In the literature, light steel thin-walled structures (Light Gauge Steel Framing, LGSF) are considered effective framing systems made of thin steel elements up to 3 mm thick and are widely used in prefabricated buildings [1-10]. Studies confirm the feasibility of using LGSF in various areas of construction and their structural efficiency [11]. The design of such systems is regulated by a standard [12]. This standard applies to profiles with experimentally confirmed characteristics. For non-standard elements the approach is different. Their load-bearing capacity and serviceability are determined based on experimental testing or validated numerical models.

Light steel thin-walled structures are widely used in frame buildings. Therefore, numerous scientific studies have focused on their structural behaviour. In particular, considerable attention has been paid to the load-bearing capacity and performance of roofing systems made of cold-formed profiles. Special emphasis is placed on roof coverings that function as components of spatial frame systems.

In [13], a comprehensive analysis of the behaviour of cold-formed steel roofing in industrial buildings was carried out. Within this study, the load-bearing capacity and stiffness of roofing systems made of cold-formed profiles under loads simulating real operating conditions were investigated, and recommendations for optimising structural solutions for Z-purlins and girders were formulated. The obtained results are supported by numerous international studies. These studies comprehensively evaluate the behaviour of thin-walled roof purlins. They consider their interaction with roof sheeting and spatial structural elements. In addition, these studies identify the key parameters that determine the stiffness and load-bearing capacity of such systems [4-6].

Further development of the experimental research is presented in [2]. In this study, laboratory tests of thin-walled Z-purlins were conducted. These tests were used to assess their load-bearing capacity at connections with supporting elements. The study also examined the influence of geometric parameters on the behaviour of the elements.

This is consistent with existing international programmes aimed at assessing the behaviour of thin-walled profiles under various loading conditions, including stability response and critical failure modes [7; 8]. The fundamentals of the design approach for thin-walled steel roofing systems are presented in a seminal paper [3]. This paper proposes a practical method for designing cold-formed Z-purlins.The method takes into account their interaction with the sheeting. This makes it possible to adapt structural solutions to real design conditions.

Corresponding analytical and numerical models of profile–sheeting interaction enhance standardised design methods and improve the reliability of structural behaviour predictions [9; 10].

A roof structure composed of lightweight cold-formed trusses can serve as an effective alternative to traditional heavy rafter trusses made of solid hot-rolled profiles in frame buildings.

Therefore, a review of experimental and numerical studies has been carried out. It shows that the use of LGSF in roofing systems is well justified. Based on this, a comparative numerical analysis is required. This analysis considers conventional steel truss coverings and LGSF profile truss coverings.

The aim is to assess their load-bearing capacity, stiffness, and stability characteristics. In addition, a comparison based on material consumption is performed. This helps identify optimal design solutions.

Materials and methods

The study employs a set of interrelated methods. An analytical method was used to examine possible structural solutions for steel frame building roofs and to compare the performance of rafter systems made of hot-rolled and cold-formed profiles.

The finite element method was applied to determine the forces, stresses, and deformations of the roof trusses. According to this approach, the structures were modelled using bar elements with specified stiffness characteristics.

As one of the principal numerical methods for structural analysis, the finite element method is widely used in structural mechanics. It was implemented in the Lira-CAD software package (Ukraine) to determine the forces, stresses, and deformations of the roof trusses. Based on the results of the static analysis, the cross-sections of roof truss members were selected and verified according to limit state design provisions in accordance with Ukrainian standards [13]. Cross-section selection was performed automatically in the Metal module of the software using the Design Load Combination (DLC) method based on the most unfavourable load combinations.

Analytical processing of the calculation results was carried out in the Microsoft Excel environment using spreadsheets, formulas, and graphical tools.

Model No. 1 of the roofing system was designed using hot-rolled angle sections in accordance with the relevant standards [14]. Model No. 2 was designed using cold-formed thin-walled sections (LGSF profiles) in accordance with the applicable codes [15-17].

Finite element analysis models of possible structural solutions for an 18.0×24.0 m industrial building roof are considered. The first model (Fig. 1a) represents a “classical” solution for an industrial building roof. It is designed as a truss structure F-1 with a span of 18.0 m. At the ends of the 18.0×24.0 m roof diaphragm, connecting trusses F-2 with a length of 6.0 m are provided. Horizontal bracing members are arranged along the bottom chords of trusses F-1 and F-2. Roof purlins are placed along the top chords of the F-1 rafter trusses and also serve to stiffen the entire structure. All elements of this roof model are made of hot-rolled steel sections in accordance with the relevant standards [14]. The arrangement of trusses in Model No. 1 determines the roof behaviour in the plane of truss F-1 and represents a so-called unidirectional roof structure. The maximum axial forces N occur in truss F-1.

The second model represents an efficient roofing system made of lightweight thin-walled steel sections. LGSF profiles have open cross-sections and are manufactured by cold forming. Their distinctive feature is the relatively small wall thickness, typically ranging from 1.5 to 4 mm.

The roof structure according to Model No. 2 consists of lightweight cross trusses forming rigid rectangular closed blocks measuring 6×12 m. The overall stability of the structure is achieved by forming several spatial blocks along the roof perimeter. The concept of this roofing system is based on a specific structural idea. LGSF trusses are restrained out of plane by horizontal bracing along the bottom chord. They are also restrained by purlins along the top chord. Due to these restraints, the trusses form a rigid roof diaphragm. This diaphragm ensures the spatial behaviour of the building frame.

The truss members are made of thin-walled sections with C- and double-C-shaped cross-sections.

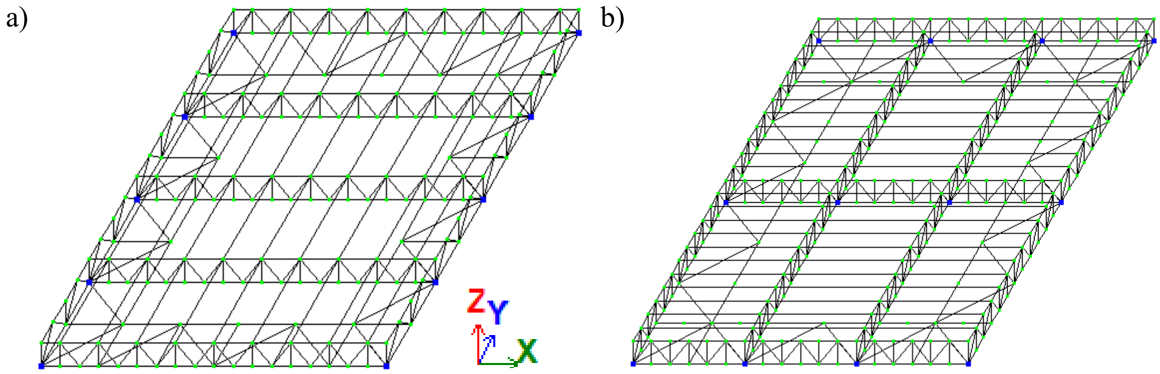


Fig. 1. **Finite element rod models of the roof system:** (a) Model No. 1, consisting of rafter trusses with bracing made of hot-rolled angle sections; (b) Model No. 2, consisting of lightweight cross trusses forming rigid rectangular closed modules measuring 6×12 m

The stiffness parameters of the models are as follows. *Model No. 1*: The top chord of the F-1 trusses (aligned parallel to the X-axis) is made of paired hot-rolled angles 180×110×12 mm, and the bottom chord of paired angles 140×90×8 mm. The web members consist of unequal-leg paired angles 75×50×5 mm. The top chord of the F-2 truss (aligned parallel to the Y-axis) is made of paired angles 100×65×7 mm, the bottom chord of paired angles 100×65×7 mm, and the web members of angles 50×32×3 mm. Horizontal bracing along the bottom chords of the trusses (Fig. 1a) is made of paired angles 80×3 mm. Purlins along the top chords of the F-1 trusses are hot-rolled channel sections 40×2.5 mm.

For *Model No. 2*, the cross-sections of the cold-formed profiles were selected so that the cross-sectional area of the members in each group of structural elements (top chord, bottom chord, web) matched that of the corresponding members in Model No. 1. In Model No. 2, the top chord of the F-1 trusses (aligned parallel to the X and Y axes) consists of two paired C-shaped profiles 200×68×1.5 mm. The bottom chord uses two paired C-shaped profiles 200×68×1.5 mm, while the web members consist of single C-profiles 100×68×3 mm. Horizontal bracing along the bottom chords of F-1 is made of single C-profiles 120×68×3.4 mm. Purlins along the top chords of the F-1 trusses are made of single C-profiles 100×68×2 mm.

Three primary loads were considered. The first load is the self-weight of the roof, including the trusses. It was automatically calculated by the software based on the assigned stiffness parameters. The second load is the roofing weight, which is 0.121 kPa. The third load is the design snow load for the city of Sumy. Its value is 1.83 kPa [10]. Wind load was not considered. Its effect on the stress–strain state of the roof is negligible.

Both computational models were developed using beam-type finite elements, specifically the spatial beam element (finite element No. 10 in the LIRA-SAPR software package).

The boundary conditions of the models are illustrated in Figure 1. The nodes where constraints are applied to prevent displacement are indicated in blue. At all constrained nodes, three translational degrees of freedom are restrained–along the X, Y, and Z axes.

In Model No. 1 (see Fig. 1a), each truss, arranged parallel to one another with a spacing of 6 m, is supported by columns at two points. In Model No. 2 (see Fig. 1b), each rigid rectangular block measuring 6×12 m, modeled using truss elements, is supported at four corner nodes of the respective contour.

Results and discussion

The analysis of the deformation parameters of the models showed that the maximum vertical deflection for Model No. 1 is 28.3 mm, while for Model No. 2 it is 12.4 mm. These maximum deflections do not exceed the allowable limits specified in the standards [13]. Fig. 2 presents the maximum axial internal forces N under the most unfavorable combination of loads (ULS). The forces are shown for both roof models and for all structural groups of the F-1 truss elements – top chord, bottom chord, and web members.

The stability analysis of the models, based on the stability reserve coefficient, indicated that both models are adequately stable. For roof Model No. 1, the stability reserve coefficient is $n = 3.4$, whereas for Model No. 2 it is $n = 4.92$. Thus, Model No. 2, which consists of rigid rectangular closed modules, is approximately 1.45 times more stable than Model No. 1. Figure 2 also shows that the tensile and compressive forces in the truss members of the LGSF model are lower than the corresponding forces in the rafter trusses of Model No. 1. This suggests that Model No. 2 is more efficient in terms of structural behaviour.

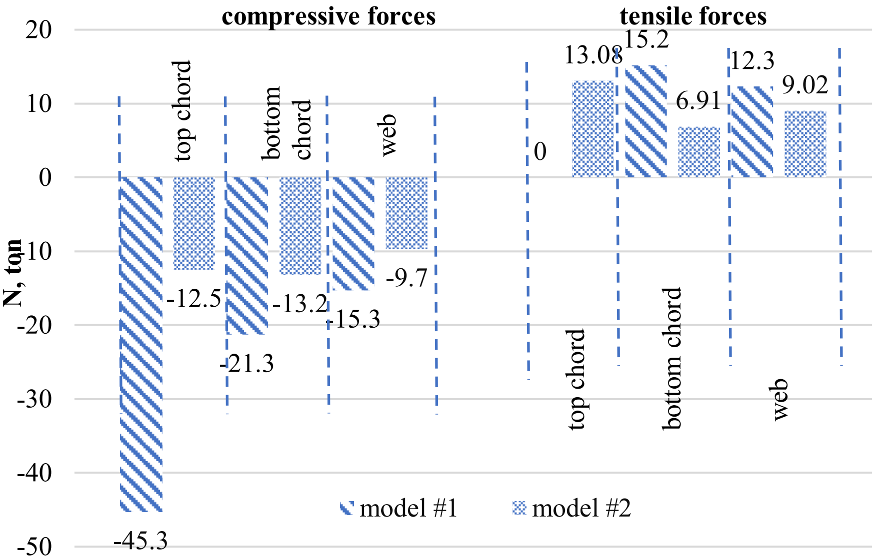
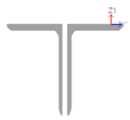


Fig. 2. **Maximum axial forces *N* under ULS for the structural member groups of the F-1 truss**

The analysis of the degree of utilization of the load-bearing capacity of the truss members showed that the cross-sectional area in Model No. 2 is utilized most efficiently. Table 1 presents the results of the automated cross-section selection. This was performed in the "Metal" module of the LIRA-SAPR package. As an example, one group of structural elements is considered – the top chord of the F-1 truss. The results were obtained after the first iteration of the calculation. This iteration is a static analysis. It uses the stiffness parameters specified in the input data.

Table 1

Selection of elements for the upper chord of truss F-1 after iterative calculation No. 1

Model No.	**Preliminary selected cross-section**	**Profile selected after the first iteration**		**Weight of 1 m of profile, t**	**Length of chord members, m**	**Total weight, t**
1	180×110×12 Unequal-leg hot-rolled angle steel	80× 60 ×8		0.017	90	1.51
2	200×68×1.5 2C-shaped profile "Akfabuid"	200× 68×2.5		0.014	150	2.10

A similar automated selection was carried out for the remaining structural member groups in both models. These include the bottom chord. They also include the web of the F-1 truss for both models. In addition, the web of the F-2 truss was considered for Model No. 1 only. The selection also covered the horizontal bracing along the bottom chords. Finally, it included the roof purlins.The total weight of the models based on the results of the automated selection is presented in Table 2.

For Model No. 2, the total weight is 6.2 tons, which is 11.2% less than that of Model No. 1 (6.9 tons).

Table 2

Weight of structural groups of rods for coating models

Profile	Weight per 1 m, t	Total length of members, m	Total weight, t	Profile	Weight per 1 m, t	Total length of members, m	Total weight, t
Model No.1				Model No.2			
Top chord, F-1 truss							
2L80×60×8	0.017	90	1.51	2C200×68×2.5	0.01	150	2.1
Bottom chord, F-1 truss							
2L63×40×5	0.008	90	0.70	2L63×40×5	0.01	90	0.7
Web, F-1 truss							
2L50×32×4	0.005	206.58	1.03	C100×68×1.5	0.003	415.2	1.3
Top chord, F-2 truss							
2L25×16×3	0.002	48	0.09	-	-	-	-
Bottom chord, F-2 truss							
2L25×16×3	0.002	48	0.09	-	-	-	-
Web, F-2 truss							
2L25×16×3	0.002	71.04	0.13	-	-	-	-
Horizontal bracing along the bottom chords of trusses							
2L40×2.5	0.003	226.8	0.67	C65×32×8×1	0.001	226.8	0.24
Purlins along the top chords of trusses							
Channel 14U	0.0123	216	2.66	C120×68×2	0.004	396	1.59
Total			6.9	Total			6.1

Conclusions

The behaviour and operational principles of the trusses in the Model 1 and Model 2 roof systems differ significantly. The maximum node displacements f_z under DLC2 for Model No. 2 (with LGSF) are 128% lower (approximately 2.3 times) than the corresponding maximum displacements in Model No. 1. This fact is indicating substantially higher stiffness and much lower deformability in the roof system based on spatially closed rigid blocks. Model No. 2 is also 1.45 times more stable than Model No. 1, which lacks closed rigid contours. The weight of each model was determined according to the newly selected cross-sections. A numerical experiment confirmed that the load applied to any point of the covering in Model No. 2 (with closed rigid LGSF blocks) is resisted collectively, engaging all elements in the overall structural action.

Research perspectives

Further research could focus on verifying the effectiveness of spatial LGSF blocks for different spans and roof geometries, as well as under various types of loads, including wind, uneven snow, and seismic actions. It is recommended to perform nonlinear and dynamic analyses, conduct a more detailed investigation of joint connections, and carry out experimental validation of the models. Particular attention should be given to economic efficiency, durability, optimization of cross-sections, and the assembly technology of modular structures.

Author contributions

Conceptualization and methodology, N.S.; investigation, N.S., S.H., V.L. and L.Ts.; writing − original draft preparation, H.Ts. and S.R.; writing − review and editing, H.Ts., S.H., O.D. All authors have read and agreed to the published version of the manuscript.

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