

Light-gauge frame construction: numerical analysis and research

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Abstract. The paper considers the frame construction behaviour numerical analysis of 18 meters bay from steel thin-walled profiles on the calculation results basis of a finite-element frame model made in ANSYS. Rated, equivalent, and tangential stresses values of frame construction on various parts were received during the work. The high efficiency of the wall stiffness intermediate element and section stiffness boundary elements can significantly increase the local resistance elements of thin-walled profile. The possibility of elements strain under the action of the design load is revealed as a calculation result of the finite-element model and taking into account bolted connections in a frame construction joints. The paper contains frame construction overall stability analysis at the decisions of bracing system. The frame construction field tests of 18 m bay from steel thin-walled profiles as a part of three frames block are carried out. As a result, the frame overall stability at the accepted bracing system and local stability of steel thin-walled profiles is provided; the rigidity of a design meets the regulatory requirements.

1. Introduction

In the last decades the light steel thin-walled structures (LSTWS) are actively applied in construction. Cold-formed thin-walled profiles are mostly used in frame housing construction and reconstruction of buildings as guides and jamb stud components of external and internal walls with space filling with efficient heat-insulating materials [1–4]. Thanks to advantages these profiles are used not only in the enclosing structures, but also for load-bearing frames in the form of open-web girder and beams elements, skeleton frame. A number of complex works on justification use of galvanized roll-formed shapes in the industrial buildings for wide use is executed under the guidance of E.L. Ayrumyan [5–7]. Elements of roof truss can be made from roll-formed shapes of channel section, c-shaped or z-shaped sections. Results of theoretical and experimental investigation of truss work from the cold-formed profiles are given in paper [8–10].

Loss of a wall local stability and open section distortion, flexural and rotating form of an overall stability loss of auxiliary bar are oblate thin-walled auxiliary bars characteristic [11–17]. Results of numerical computational methods research of steel thin-walled profiles are given in works [18–20]. The modern computer program systems, for example, of SCAD Office are often used for modelling engineering of thin-walled auxiliary bars [21, 22]. In foreign practice of construction the Eurocode 3 [13] and AISI [23] standards are applied in calculation and projection of LSTWS. In recent years, Russia has started to apply a set of rules for the design of LSTCs, adapted for engineering calculations, from cold-formed galvanized profiles [24]. In these standards the reduced sectional profile



characteristics are applied to reduce the load-carrying capacity of elements under different stress-strain states. The maximal factory readiness and technology of construction allowing raising buildings during the short period, high transportability of designs are the main advantage for the buildings construction from LSTWS in the remote region of the Far North. Besides, small buildings weight from steel thin-walled structures cuts expenses on the bases that are especially important for conditions of permafrost soils [25]. Results of numerical researches of a frame construction work from the cold-formed coupled profiles by flight of 18 m by means of the PC Ansys and test of a solid frame are given in the present article.

2. Methods

Frames are designed dual-slope with a bias 1:5. Jamb stud and a framework beam are carried out from the coupled profiles of PGS-PZh 300-3.0 c-shaped section, wall and roofing runs are executed from unary profiles of PGS-PZh 220-2.0 c-shaped section. For frames flight of 18 m follow-up provides an extension in a framework beam from the coupled profiles of PGS-PZh 220-2.0 c-shaped section. For a carrying capacity check of a frame construction mechanical characteristics of thin-walled profiles steel are accepted by $R_y=350$ MPa and $R_u=480$ MPa, the calculated resistance to a bearing strain of the elements connected by bolts, $R_{bp}=690$ MPa. Connections of profiles in eaves gutter and ridge frame clusters, in conjunctions of an extension with a frame girder are provided through gusset plate 6 mm thick with regional stiffeners from sheet steel of brand 09G2S in accordance with GOST 5520-79. Calculated resistance of shaped steel are accepted on a yield point of shaped steel of $R_y=335$ MPa and an ultimate strength of $R_u=480$ MPa, the calculated resistance to a bearing strain of the elements connected by bolts - $R_{bp}=690$ MPa. In clusters of a frame construction assembly bolts with a diameter of 16 mm of a class of durability 8.8 with a calculated resistance of steel to a cut of $R_{bs}=330$ MPa are applied (Figure 1).

The spacing of frames in the considered buildings of complete delivery is accepted by 3 m. The frame stiffness of the building on jamb stud and a covering is provided by means of cruciform bridging from wire ropes. The step of hammer beam small also makes 1.41-1.43 m. (Figure 1). When carrying out calculations loading from a snow covering is accepted in the form of the concentrated forces in the hammer beam locations. The wall protecting designs are accepted from sandwich panels, respectively, loading from panels is modeled in the uniform distributed load form on an external branch of jamb stud. According to load summary from sandwich panels and snow load, design vertical load of a framework beam makes 4.53 kN/m.

The numerical research of a frame construction work is conducted with use of the ANSYS computer system allowing solving problems of structural mechanics and materials strength a finite element method. Taking into account the design solution of frames the calculated scheme is accepted in the form of flat rod system. For the intense strained state analysis of a frame construction considered 2 calculated models: model 1 – the simplified model; model 2 – the specified frame model with bolted connections in eaves node and interface node of a frame girder with collar tie.

The loading was carried out for the frame located in the middle of the module. For loading fluidized bags with sand on 50 and 25 kg which kept within on sites of a covering from the pro-thinned-out leaf in a zone of hammer beams fastening are used beforehand. At each stage of a frame nodal loadings test are accepted similar to values when a numerical analysis is carrying out (Table 1).

3. Results

As a result of a numerical analysis it is established that the maximal equivalent tension arises on sites of interface of a crossbar to columns where the moment of deflection and transversal effort have the greatest values (Figure 5). If we consider in more detail this site, then the maximal equivalent tension is observed in a zone of transition of a profile web of a girder frame to a section stiffener. Despite the fact that the intermediate wall stiffener has a rounded shape, the wall bending portion acts as a stress concentrator. At the same time the maximum value of tension settles down along the first intermediate stiffener of a web from the top belt of a girder frame (Figure 3, b).

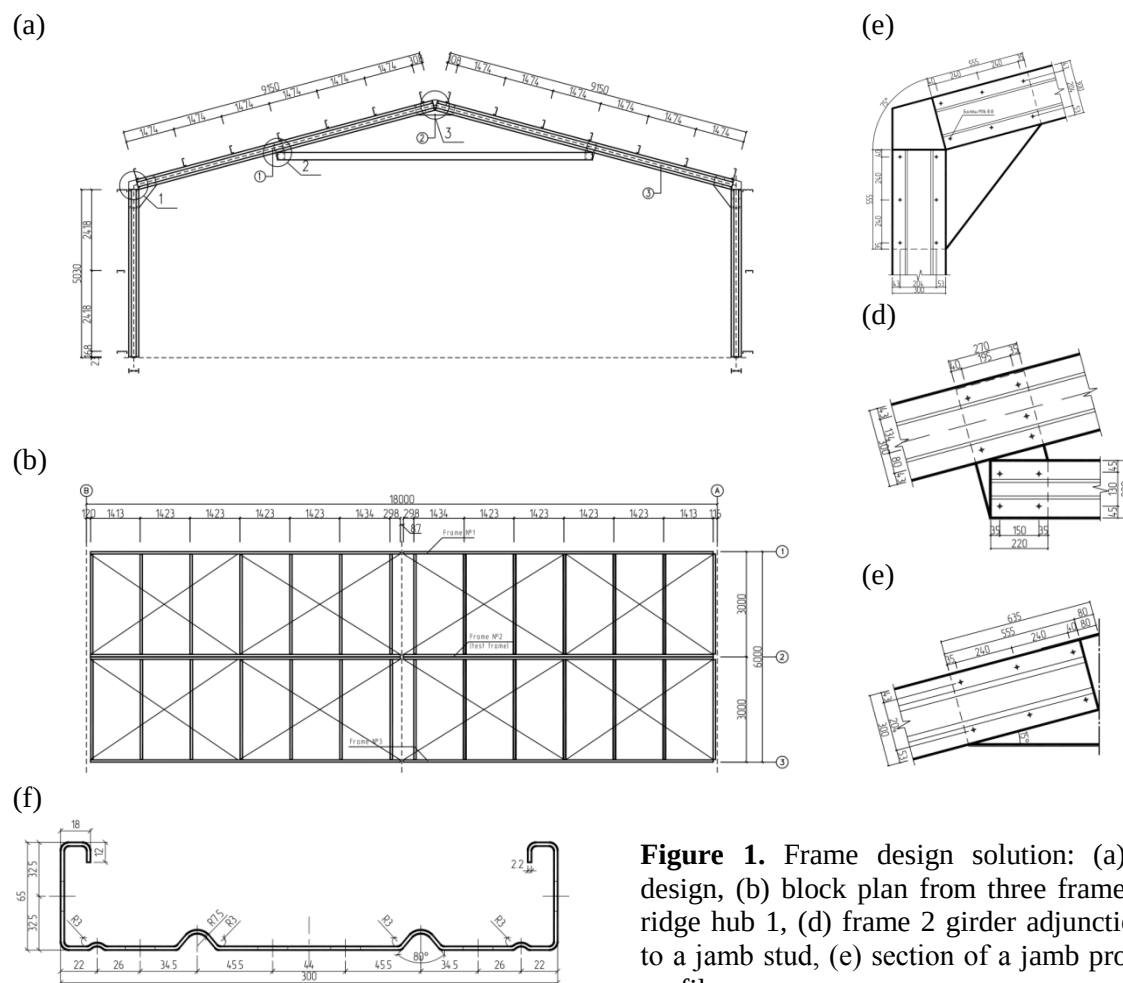


Figure 1. Frame design solution: (a) frame design, (b) block plan from three frames, (c) a ridge hub 1, (d) frame 2 girder adjunction knot to a jamb stud, (e) section of a jamb profile, (f) profile.

Table 1. Values of frame nodal loads, kg, on loading stages.

Frame unit arrangement	Nodal loading in kg on loading stages							
	1	2	3	4	5	6	7	8
Eave node point (2 units)	50	150	200	250	300	350	400	450
Intermediate node (10 units)	100	200	300	400	500	550	600	650
Ridge node (1 unit)	200	400	600	750	900	1000	1100	1200
Share from design loading, %	15.4	30.7	46.2	61.5	76.9	84.6	92.3	100

On the site of connection of an extension with a frame girder the largest equivalent tension also settles down in a zone of transition of a web to the rounded intermediate stiffener from eaves knot and reaches value of 366 MPa. It should be noted that the moment of deflection on this site of a frame girder has larger value from eaves knot. On the site of a frame girder in apex the equivalent tension has low values to 140 MPa.

The maximal direct stresses arise in an upper of profiles of a jamb stud from the inside of the middle of the rounded intermediate stiffener of a web. The largest tangential stresses arise on the area of eaves node in a zone of a web transition of profiles of a frame girder to the rounded intermediate stiffener. At the same time the maximal tangential stresses are observed along transition of a web to the intermediate stiffener (Figure 3) second from above. In the section of the extension junction with the a frame girder, the maximum normal stresses are located in the zone of the wall transition into the rounded intermediate stiffener, located closer to the lower belt of the frame girder section.



Figure 2. View of the frames testing block and a loading process.

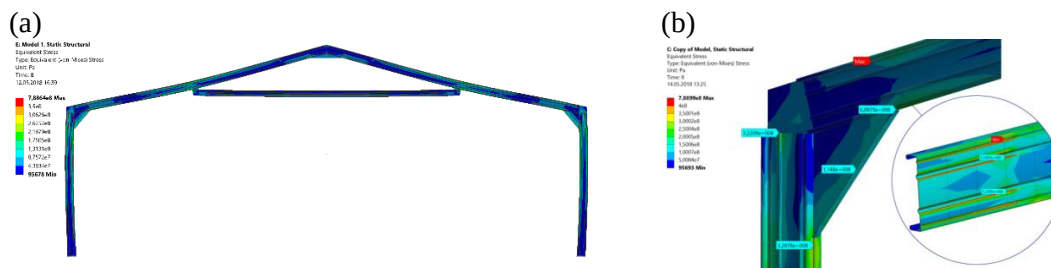


Figure 3. Distribution of the equivalent tension at design loading: (a) in the whole frame, (b) in eaves node point.

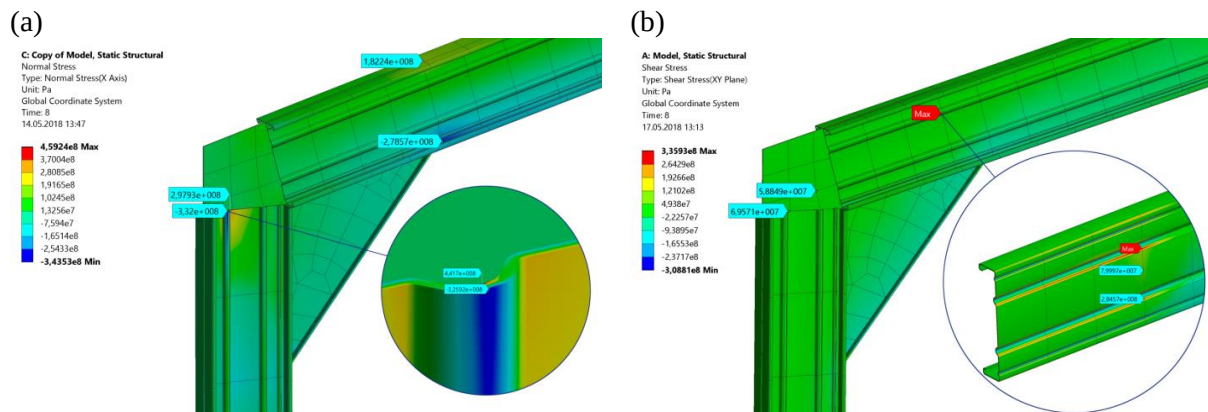


Figure 4. Distribution of direct and tangential stresses in eaves node.

Results of calculation show that loss of an overall stability of a frame occurs at excess of design loading by 4.9 times. Loss of stability of a frame girder is observed on the site between the intermediate interface node of an extension to a frame girder and apex node where horizontal communications on a covering (Figure 5) are provided.

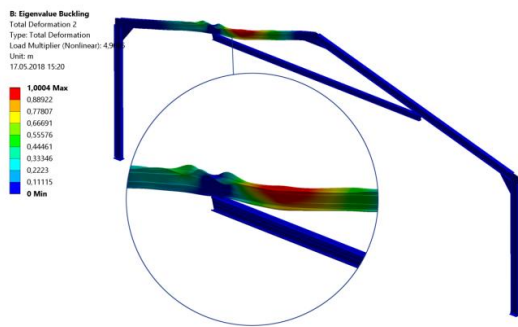


Figure 5. The loss of the overall stability of the frame.

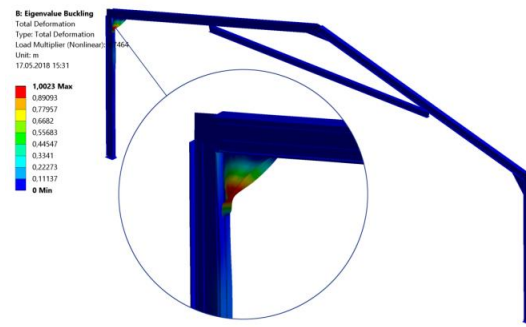


Figure 6. Loss of local stability of a gutter plate of eaves node.

According to calculations, loss of local stability of frame element occurs when the design load is exceeded by 4.7 times in the shape of the gutter plate of eaves node (Figure 6). It should be noted that longitudinal stiffeners in a web and regional stiffeners efficiently provide local stability of section certain sites of steel thin-walled profiles 3 mm thick.

Approximate analysis of the 2d frame model with bolted connections showed that the maximal equivalent tension arises in extreme ranks of bolted connections of eaves node and interface node of an extension to a frame girder and exceed values of a yield point of steel of profiles and a gusset plate. Therefore calculation of clusters taking into account elastic plastic deformations by means of bilinear isotropic hardening (Bilinear Isotropic Hardening) in which the tangential module (tangent modulus) in all materials is accepted by 1450 MPa (the deformation module is carried out further at plastic strains).

As a result of numerical analysis, it is established that when the yield stresses of equivalent steel stresses are reached in separate holes in the joints at nodes, a redistribution of forces is observed. For example, in eaves node the maximal tension arises in an end face of a gusset plate of an opening 12 from a rack of a frame and at loading, a component 0.22 from a design (R), reaches value of a yield point. Then there is a reassignment of efforts – at loading 0.26 P tension in an end face of a gusset plate of an opening 2 becomes equal to a yield point became. This picture is well visible from Figure 7 and schedules in Figure 9 (a).

In a bolted connection of an extension with a frame girder the maximal tension arises in an end face of a gusset plate of an opening 4 from a frame girder and at loading 0.25 P yields steel. And at the subsequent loading the reassignment of efforts is observed (Fig. 10 and 11, b).

Thus, as a result of a numerical analysis using the ANSYS it is established that exhaustion of a carrying capacity of a frame construction happens because of a bearing strain of a shaped element and a profile of a rack in a bolted connection of a rack with a frame girder. For verification of the obtained data padding calculations with use of the Lira, Eurocode 3 [13], and Set of Rules 260.1325800.2016 “Cold-formed thin-walled steel profile and galvanized corrugated plate constructions

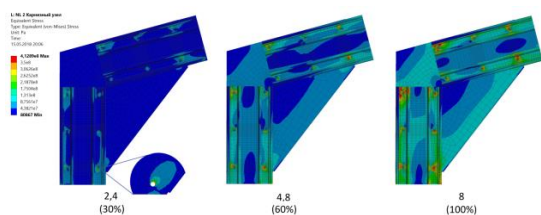


Figure 7. The equivalent tension in eaves node point taking into account bolted connections at various stages of a loading.

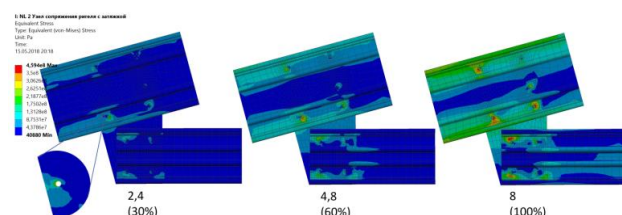


Figure 8. The equivalent tension in knot of interface of a main carrier cable to a frame girder at.

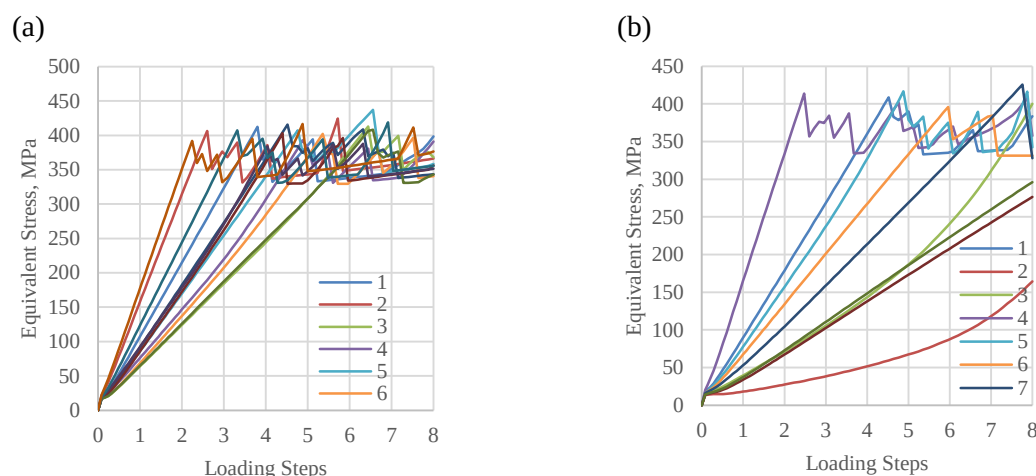


Figure 9. Dependences of the tension equivalent at openings end faces of eaves node point shaped element (a) and interface node of a main carrier cable to a frame girder (b) at different stages of loading.

Design rules” [24] are carried out. As a result of payment under two normative documents it is established that at action of design loading the carrying capacity of elements of a frame and local stability of elements of steel thin-walled profiles of a frame girder and jamb stud are provided. The largest specified tension arises in frame girder section on the site of eaves node. At the end of a frame girder element the specified tension is 0.855 Ry according to calculations for Eurocode 3 and 0.694 Ry according to calculations for Set of Rules 260.1325800.2016 “Cold-formed thin-walled steel profile and galvanized corrugated plate constructions. Design rules”.

To check the results of the numerical analysis of the frame construction work and security of a carrying capacity field, full-scale tests of the frame structure were carried out.

According to the plan of test for creation of design loading 8 stages of a loading are carried out. After each stage the endurance of a design under loading was made, and readings of deflection indicator were taken further, all main clusters and fixing were examined. According to the above-stated calculations the largest tension arises in eaves clusters of connection of a frame girder with jamb stud and in clusters of fastening of an extension to a frame girder. Detailed survey of elements and these clusters of a frame construction after each stage of a loading showed that no deformations and external changes in elements of a frame construction are observed. In web and flange of sections of jamb stud and a frame girder loss of local stability of elements is also not established (Figure 10)

Schedules of the actual vertical movements of separate clusters of a frame construction in comparison with calculated values are submitted in Figure 11. Deformations of a frame on the submitted schedules have the symmetric character. At the beginning of a loading, including the 4th stage of a loading, the actual vertical movements of clusters of an adjunction of a main carrier cable with a frame girder and apex node practically coincide among themselves. Further vertical movements of apex node of a frame girder begin to exceed movements of other clusters. It shows that at the 5th stage of a loading after all deformations of separate elements in bolted connections the extension completely gets into gear. After the 8th stage of a loading at design loading the actual value of vertical movement of apex node of a frame girder was 73.9 mm, extension adjunction clusters to a frame girder – 68.7 and 67.2 mm. From schedules in Figure 11 it is visible, results of calculation to use of terminating and element model 2 taking into account bolted connections and elastic plastic behavior are closest to the actual values of movements to a divergence 0.85÷5%.

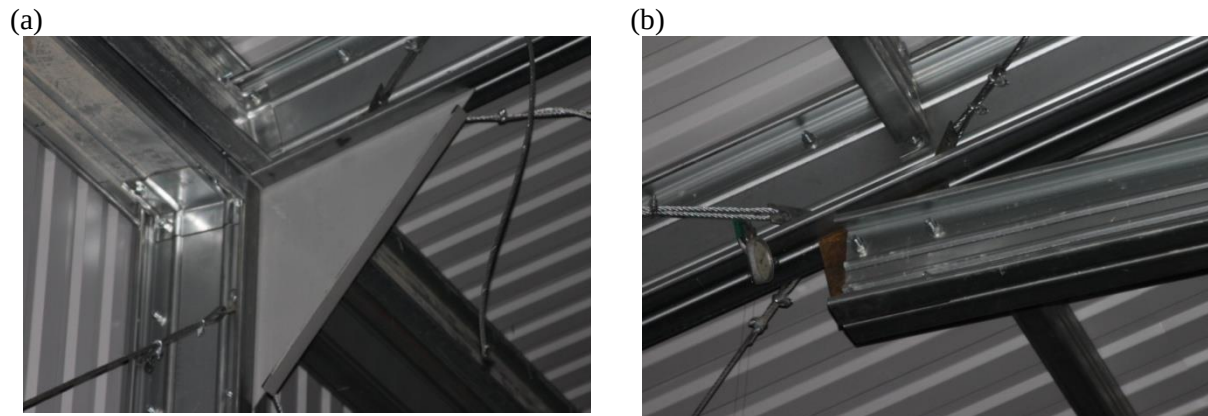


Figure 10. The frame construction elements with design loading: (a) eaves node, main carrier cable connections with a frame girder.

After the 8th stage of a loading design loading of test of a frame proceeded for establishment of the actual carrying capacity of a design and a form of loss of a carrying capacity of a design. At the same time for safety deflection indicator were removed. The stage-by-stage loading of a design was made on 5% of design. On reaching cooperative loading in 19800 kg that makes 230% from design, tests stopped in a type of an exhaustive carrying capacity of a design in general. Survey of the main clusters and fixing of designs showed that no external changes in elements of a frame construction are observed. At the last stages of a loading frame girder deflections on the site located at $\frac{1}{4}$ lengths of a frame girder are established. After the complete unloading of an testing construction permanent deformations of a frame girder of 26 mm and 31 mm of an arrow of a curvature respectively approximately in $\frac{1}{4}$ since the left-hand and right ends of a frame girder of a construction are revealed.

With unloading of a frame construction, stripping and dismantling of a frame construction on details, their survey was made. It is as a result established that the qualitative picture of a bearing stress of end faces of shaped elements and profiles in eaves node points and interface node of an extension to a frame girder completely coincide with results of calculation of terminating and element model of a frame taking into account bolted connections and elastic plastic behavior became (Figure 12). For example, the largest sizes of a bearing stress of end faces of a gusset plate of eaves node point are established in openings 12 and 2 in which by calculation there is equivalent tension comparable to a yield point became at early stages of a loading.

4. Conclusion

The numerical analysis of frame construction work from steel thin-walled profiles on the calculations basis of terminating and element model in ANSYS is made. It is established that the most intense places are interface parts of a frame girder to jamb stud. At the same time the maximal equivalent tension arises in transition zones of a web to the rounded intermediate stiffeners which are stress raisers. The maximal direct stresses are established on the place of an extension interface node to a frame girder from eaves node in a transition zone of a web to the rounded intermediate stiffener located closer to the lower belt of a frame girder section. Direct stresses do not exceed the calculated resistance of steel in extreme fibers of the frame girder shelf. In this case, longitudinal stiffeners in a web and edge stiffeners efficiently provide certain areas cross section local stability of steel thin-walled profiles 3 mm thick. The refined calculation of frame model finite element with bolted connections and elastic plastic behavior of steel has shown the efforts redistribution to other areas of bolt connections when the level of yield strength of steel at the end faces of separate holes in the eaves assembly reaches the equivalent stresses and connection of an extension with a frame girder.

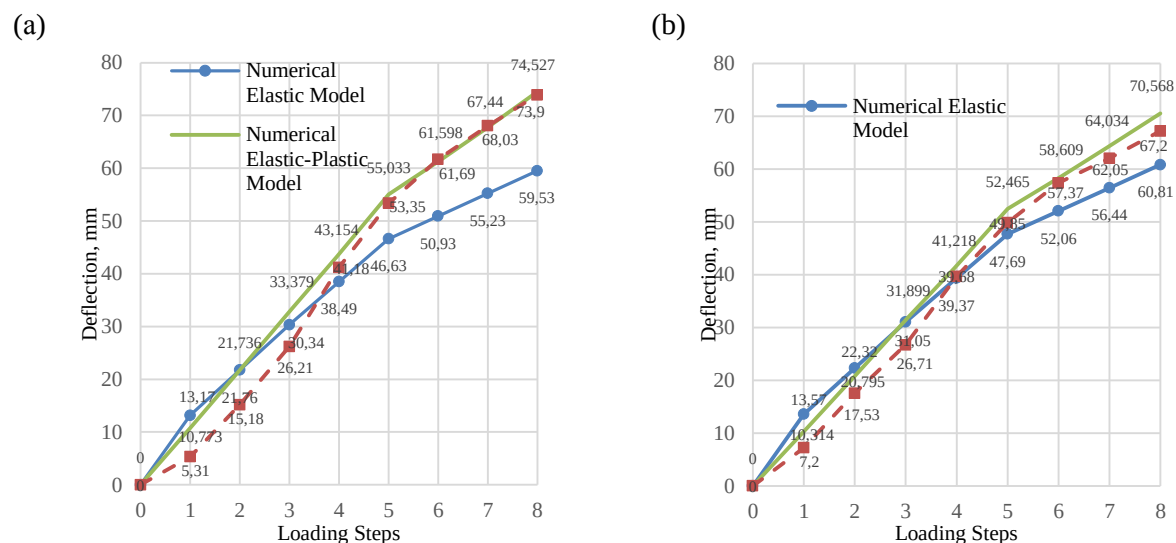


Figure 11. Nodes vertical movements of a frame girder: (a) the apex, (b) of interface node of a main carrier cable to a frame girder.

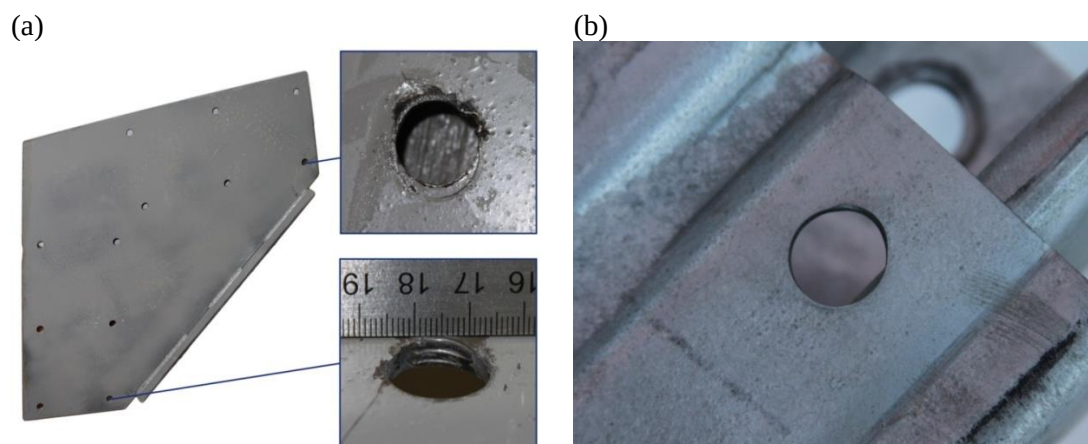


Figure 12. Bearing strain elements of frame eaves nodes: (a) gutter plate, jamb stud profile web.

As a result of the carried-out field tests it is established that the carrying capacity of a frame construction is provided with span of 18 m from light steel bent profiles at action of design load of 4.53 kN/m. During the loading of a frame construction, the loss of local stability in a web and regiments of profiles of jamb stud and a frame girder is not established. Results of calculation using the finite element of 2 model frame with bolted connections and elastic plastic behavior are closest to the actual values of movements to a divergence 0.85÷5%. The real picture of a bearing strain of end faces of shaped elements and profiles in eaves node points and interface nodes of an extension to a frame girder after stripping of the tested frame completely coincide with the results of calculation of finite element 2 model frame.

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