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TESTING OF FACADE PANELS BASED ON CASSETTE PROFILES

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Abstract

When designing modern enclosing structural systems, such qualities as reducing the complexity of installation, low weight, energy efficiency, aesthetic appeal and many others become more and more significant. Presented in this article new constructive solution of a self-supporting wall panel based on cassette profiles and profiled sheets combined into a single spatial design system to a large extent is able to meet these requirements.

The article is devoted to the description, analysis and justification of a new constructive solution of a self-supporting wall panel based on cassette profiles; the design procedure was carried out using calculation methods, as well as using the basis of the results of tests in accordance with KP EN 1993-1-3; the data of experimental confirmation of the operability of the proposed structural system during load tests corresponding to the active wind pressure are presented.

Keywords: thin-walled profiles, cold-formed profiles, effective cross-sectional characteristics, supercritical plate operation, loss of local stability, loss of stability of the cross-section shape.

ИСПЫТАНИЯ СТЕНОВЫХ ПАНЕЛЕЙ НА ОСНОВЕ КАССЕТНЫХ ПРОФИЛЕЙ

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Реферат

При проектировании современных ограждающих конструктивных систем все более значимыми становятся такие качества, как снижение трудоемкости монтажа, малый вес, энергоэффективность, эстетическая привлекательность и многие другие. Представленное в данной статье новое конструктивное решение стеновой самонесущей панели на основе кассетных профилей и профилированного настила, объединенных в единую пространственную конструктивную систему, в значительной степени способно удовлетворить эти требования.

Статья посвящена описанию, анализу и обоснованию нового конструктивного решения самонесущей стеновой панели на основе кассетных профилей; осуществлена процедура проектирования с использованием расчетных методик, а также на основании результатов натурных испытаний согласно ТКП EN 1993-1-3; приведены данные экспериментального подтверждения работоспособности предложенной конструктивной системы при испытаниях на нагрузку, соответствующую активному ветровому давлению.

Ключевые слова: тонкостенные профили, холоднoформованные профили, характеристики эффективного сечения, закритическая работа пластин, потеря местной устойчивости, потеря устойчивости формы сечения.

Introduction

Currently, structural systems based on thin-walled galvanized profiles and effective heat-insulating materials are widely used for facades. A special place in market is occupied by structural solutions of facades based on a cassette profile. These constructive solutions of facades have a number of significant advantages: high operational characteristics; the possibility of using various types of thin-walled profiles and insulate materials; decrease the complexity of installation, especially during the reconstruction of the facade of buildings; ensuring all-weather installation; aesthetic appeal; high construction speed, while these facades have architectural expressiveness due to the possibility of choosing almost any cladding.

The facades which are assembled on the site has become most widespread, however, new solutions in the form of prefabricated elements (hereinafter named as facade panels) are appeared.

In current work, a constructive solution of a self-supporting facade panel is presented. Panels consist of cassette profiles and profiled sheeting, combined into a single spatial structural system, therefore this panels have such important properties as the redistribution of internal forces between its components, survivability and reliability of the construction.

The authors performed a set of theoretical and experimental works:

- the constructive solution of the wall panel based on cassette profiles and profiled stainless steel sheets has been developed;
- the resistance and stiffness of the facade panel and its joints using the calculation method have been evaluated;
- procedure for assessing the design value of moment resistance (resistance for bending) of the facade panel using tests results has been realized (test load was equivalent to active wind exposure).

1 Constructive solution of facade panels based on cassette profiles

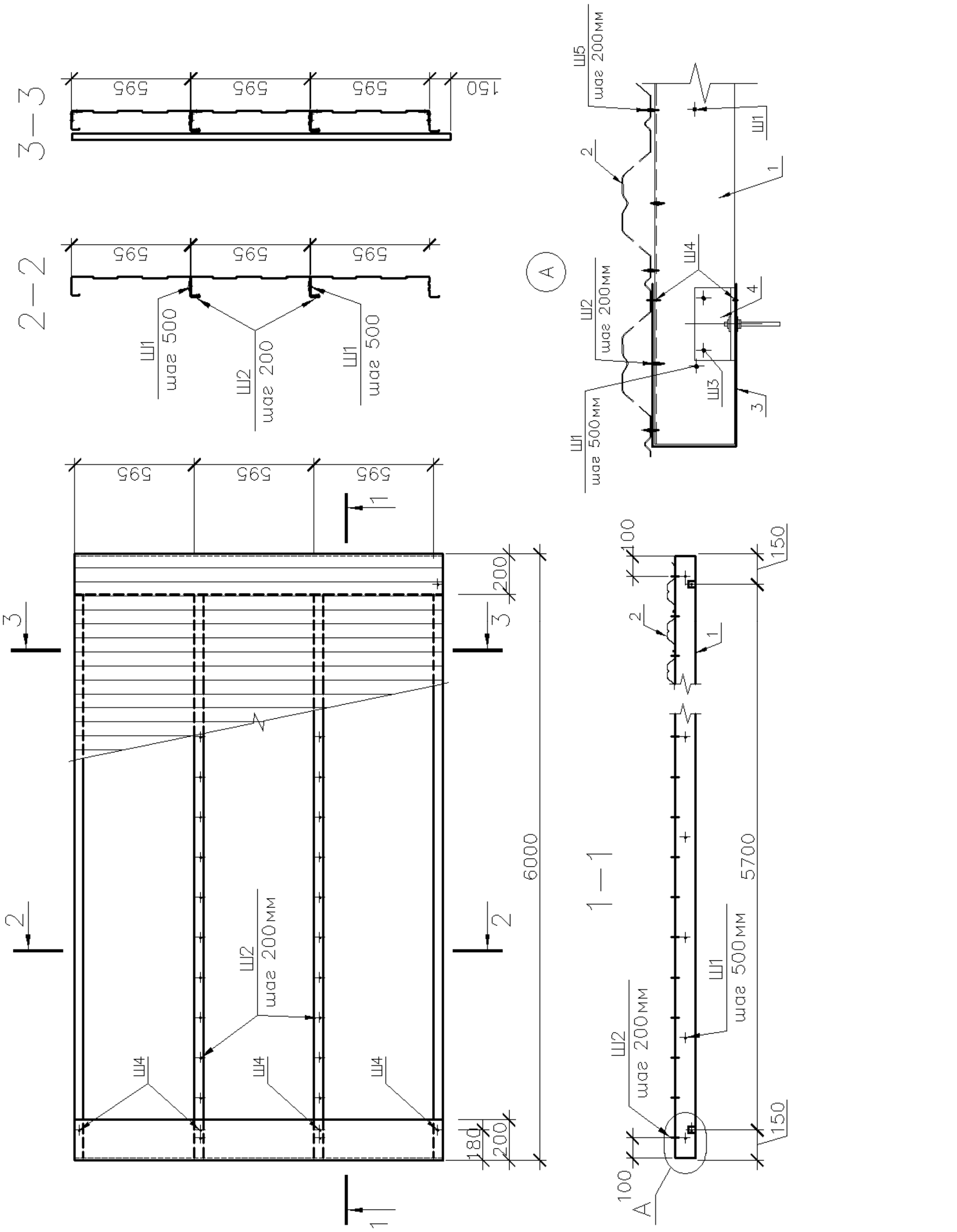
The developed constructive solution of self-supporting wall panels consists of three basis types of elements: cassette profiles; of profiled steel sheets attached perpendicularly to the narrow flanges of the cassette profile and of supporting elements located at the ends of the panel. The joint operation of the indicated components of the panel is ensured by the spot welds or by using of stainless steel blind rivets during their assembly.

The operating environment has a significant impact on the choice of material of any structure. In the presented study, the cassette profiles and supporting elements are made of stainless steel grade 12X18H10T. For the facade surface of the panel profiled sheet HC 35-1000-0.8 according to GOST 24045 [2] is applied. The nominal span of the cladding is 600 mm. The design scheme is a continuous beam of three spans. Support elements are installed at the ends of the panel. The supporting element is one of the components ensuring the spatial operation of the panel and the reliability of the fastening the panels to the columns. The cross-section of the supporting elements is an unequal steel bent channel.

The combination of components into a single spatial system is proposed to be carried out by resistance spot welding or by blind rivets. The main emphasis is made on the use of double-sided spot welding (pincers). An appropriate panel assembly procedure has been developed providing access for performing two-sided welding. In this way, cassette profiles, support elements, auxiliary fasteners are connected to each other. To fasten the steel profiled sheet to the cassette profiles, blind rivets were used.

2 Theoretical assessment of the resistance of cassette profile and connections of wall panel components

To assess the resistance of the panels, was adopted the calculation methodology described in TKP EN 1993-1-3 [1]. This technical code provides both design methods and methods that use test results for cassette profiles, however, it should be noted that this technique doesn't fully reflect the behavior of the cassette profile in the panel. In the following the results of the assessment of the resistance of the panel components are presented.



1 – cassette profile; 2 – profiled sheet; 3 – support element; 4 – connection to the external elements; Ш1 – connection of the walls of the cassette profile; Ш2 – connection of narrow flanges of cassette profile; Ш3 – connection of the fastener with the cassette profile; Ш 4 – connection of the support element with a cassette profile and profiled sheet

Figure 1 – The principal construction of the wall panel

When determining the calculated characteristics of the profile, the effective / reduced section (from which unstable parts of the section are excluded) is taken as the calculated one.

2.1 Buckling resistance of the cassette profile when its wide flange in tension

Buckling resistance moment $M_{b,Rd}$ of the cassette profile is determined by the following expression (formula 10.21 [1]):

$M_{b,Rd} = 0{,}8 \cdot \beta_b \cdot W_{eff,\mathrm{com}} \cdot f_{yb} / \gamma_{M0}$, but $M_{b,Rd} \le 0{,}8 \cdot W_{eff,t} \cdot f_{yb} / \gamma_{M0}$,

where $W_{eff,\mathrm{com}} = I_{y,eff} / z_c$ – the effective section modulus of the cross-section for fiber in compression;

$W_{eff,t} = I_{y,eff} / z_t$ – the effective section modulus of the cross-section for fiber in tension;

β_b – the correlation factor, which takes into account the fastening of narrow flanges out of plane (lateral restraint of the narrow flanges);

f_{yb} – basic yield strength.

In this case, the effective section of the cassette profile when its wide flange is in tension is calculated using the step-by-step procedure in accordance with clause 10.2.2.2 of the TKP EN 1993-1-3 [1]. The main geometric characteristics of the effective cross section of the cassette profile are shown in table 1.

Table 1 – Characteristics of the effective cross section

Characteristic	Symbol	Numerical value	Units
Area	A	6.64	cm^2
Second moment of area	I_y	189.22	cm^4
Section modulus	$W_{y,max}$	40.25	cm^3
	$W_{y,min}$	-17.8	cm^3

Substituting the values of all quantities, we obtain the calculated buckling resistance moment of one cassette profile:

$$M_{b,Rd} = 0.8 \cdot \beta_b \cdot W_{eff,com} \cdot f_{yb} / \gamma_{M0} =$$
$$= 0.8 \cdot 1 \cdot 17.8\text{cm}^3 \cdot 220 \text{ MPa} /1.0 = 3.13 \text{ kN}\cdot\text{m}.$$

2.2 Moment resistance of the cassette profile when its' wide flange in compression

The design value of moment resistance of one cassette profile $M_{b,Rd}$ is determined by the following expression (formula 10.19 [1]):

$$M_{b,Rd} = 0.8 \cdot W_{eff,min} \cdot f_{yb} / \gamma_{M0} =$$
$$= 0.8 \cdot 22.35\text{см}^3 \cdot 220 \text{ MPa} /1.0 = 3.93 \text{ kN}\cdot\text{m},$$

where $W_{eff,\min}$ – minimum effective section modulus;

Effective section of the cassette profile when its wide flange is in compression is determined by using step-by-step procedure in accordance with clause 10.2.2.1 of the TKP EN 1993-1-3 [1]. The main geometric characteristics of the effective cross section of the cassette profile are shown in table 1.

Table 2 – Characteristics of the effective cross section

Characteristic	Symbol	Numerical value	Units
Area	A	5.6	cm^2
Second moment of area	I_y	175.5	cm^4
	I_z	4872.2	cm^4
Section modulus	$W_{y,max}$	22.35	cm^3
	$W_{y,min}$	-24.16	cm^3

2.3 The resistance of connections

To join the components of the wall panel, resistance welded spot and stainless steel blind rivets were used.

The decision on the possibility of using **spot welding**, as well as the assessment of the design resistance of this connection was carried out in accordance with the provisions of subsection 8.4 of the TKP EN 1993-1-3 [1]: for joints loaded in shear, **shear resistance** was $F_{v,Rd}$=6485*H*; **tearing and bearing resistance** $F_{tb,Rd}$=4434*H*; the value of the interface diameter of a spot weld for resistance welding by the calculation method was d_s=4,5 mm.

The lower of the given values should be taken as the design resistance. In this case, the condition must be satisfied: no more 1,25 $F_{tb,Rd}$, and not more than the value obtained on the basis of the test results according to section A.6 [1].

It worth noting that according to 8.4(7) [1]: "*The value of d_s actually produced by the welding procedure should be verified by shear tests in accordance with Section 9, using single-lap test specimens. The thickness t of the specimen should be the same as that used in practice*".

For spot welds loaded in tension, **tensile resistance** is recommended to be determined by testing joints in accordance with GOST 6996-66 [3], because such tests are not provided in TKP EN 1993-1-3 [1].

Design value of **bearing resistances** $F_{b,Rd}$ for stainless-steel **blind rivets** loaded in shear was determined according to provisions of table 8.1 [1] and was $F_{b,Rd}$=2611*H*.

Design value of **shear resistance** must be determined by testing samples of joints in accordance with ISO 14589: 2000 "Blind rivets. Mechanical testing" [4].

3 Experimental researches

Within the framework of the TKP EN 1990 [5] concept, the design of elements (verification of limit states) can be based on calculations and / or on the basis of test results. In accordance with subsection 5.2 design based on tests can be applied in the case of:

- lack of adequate calculation models;
- when a large number of similar components are used;
- to confirm the assumptions made in the calculations.

Because of various forms of buckling of thin-walled elements it is not always possible to determine the calculation model, that's why design assisted by testing is especially demanded for thin-walled elements. Therefore, TKP EN 1993-1-3 [1] provides practical rules for conducting and processing test results.

During the study of the proposed design solution of the wall panel using cassette profiles, design based on tests allows us to determine the ultimate actual moment resistance and stiffness.

According to clause A.6.3.1 (1) ТКП EN 1993-1-3[1] the characteristic value of the resistance, on the basis of which the design value is established, can be determined statistically (i.e. with a given confidence probability) in the presence of at least 4 test results. If only one test is carried out, then the characteristic value should be determined on the basis of the adjusted value of the test result $R_k = 0.9\, \eta_k \cdot R_{adj}$, where η_k = (0.8-0.9) if the failure mode of thin-walled cross-sectional components is local buckling [A.6.3.3, 1]. The design value of the resistance can be obtained from $R_d = \eta_{sys} \cdot R_k / \gamma_M$, where η_{sys} – is a conversion factor for differences in behavior under test conditions and service conditions; γ_M – is a partitial coefficient.

The need in testing is also associated with the lack of an approved methodology for calculating facade panels based on cassette profiles. The purpose of the tests is to assess the moment resistance and stiffness of wall panels from cassette profiles on the load action, which is equivalent in value to the design wind effect, taking into account the pulsation component.

3.1 Testing stainless steel of cassette profile and profiled sheet

To determine the actual characteristics of steel, tensile tests of the samples were carried out. The results of this tests established yield strength and ultimate strength. The average experimental value of the yield strength of stainless steel sheet of cassette profile (based on tests of 10 samples - 250.7 MPa) exceeds the characteristic value regulated by STB EN 10088 [6] and TKP EN 1993-1-4 [7] (220 MPa). The average experimental value of the yield strength of stainless steel sheet of profiled sheet was 244.3 MPa. Note that the theoretical (design) value of stresses in the profiled sheet of the facade surface of the panel from the action of wind load was 12 MPa, what is much less than the experimental value of the yield strength and, therefore, isn't so important when assessing the resistance of the facade panel.

3.2 Testing of connections

In order to determine the actual resistance of spot welds formed by resistance welding, and to verify the calculated dependences by assessing the resistance of these joints according to the requirements of TKP EN 1993-1-3 [1], shear and tensile tests of standard samples with thicknesses (1.2 + 0.8) mm and (0.8 + 0.8) mm each 7 samples in each series were performed.

Testing revealed a discrepancy between the actual value of the inner diameter of spot welds and that calculated according to the requirements of TKP EN 1993-1-3 [1]: the experimental value was in the range of 2.9-3.1 mm, and the calculated value was 4.5 mm. This discrepancy is due to specific technological processes of contact welding (the time of compression, heating, forging and cooling of the joined sheets) and the diameters of the electrodes in the contact zone as well.

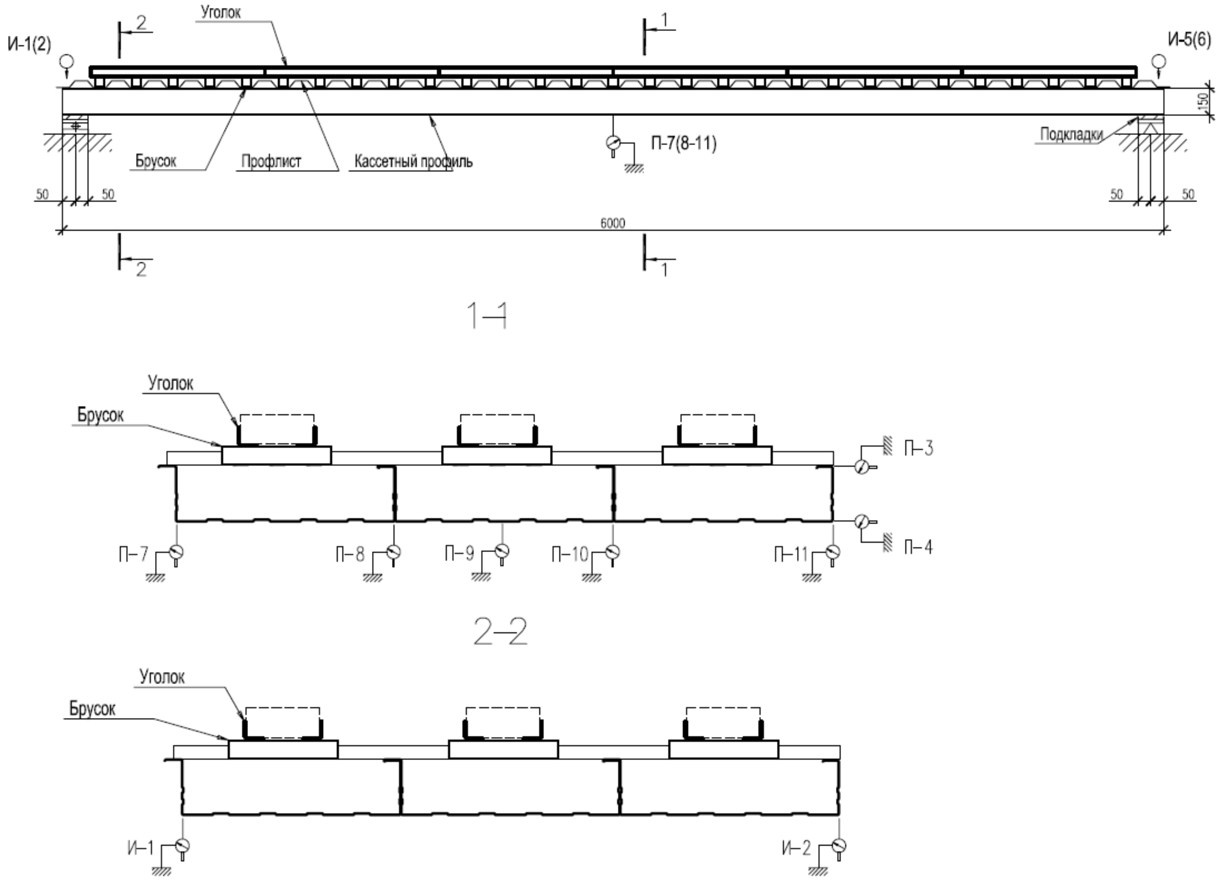


Figure 2 – Scheme of panel test design

According to the results of statistical processing of experimental data, the resistance of one spot weld was:

- the average value per shear was 4608 N, the minimum – 4163 N;
- the average value per tensile (tear) was 1204 N, the minimum – 1014 N.

Thus, the experimental values of the resistance significantly exceed the forces arising in the joints of the profiled sheet with the cassette profile. This reduces the number of spot welds. However, it should be noted that the part of the spot welds ensures local stability of narrow flanges (buckling resistance).

The design values of resistance of blind rivet are adopted according to TS 05.0309.14 [8]. The resistance of one blind rivet per shear was 3.44 kN, per tensile was 3.24 kN.

3.3 General provisions for the testing of full-size samples of self-supporting facade panel

To test the panels, a prototype was made in accordance with the given geometric dimensions and assembly technology and also in compliance with all technical requirements for materials and joints.

To simulate the operation of a cladding based on cassette profiles on the effect of active wind pressure, a test scheme shown in Figure 2 is developed.

Before starting the tests, the technical condition of the structure was examined by visual inspection for defects and damage affecting the resistance. The design of the test setup consists of box section supporting element. In accordance with the test scheme, the panel rests directly on the supporting elements. For safety reasons, safety supports are provided in thirds of the span of the panel with a continuously controlled gap of 150 mm between the supports and the surface of the wall panel. In order to exclude "loose" deformations caused by insufficient closeness of joints and compression of the supporting parts, the panel was pressed by pretest loading.

The loading scheme is based on the principle of symmetrical arrangement of loads and their uniform distribution along the span. The panel was loaded using a system of structural elements (figure 2):

- wooden bars with a cross section of 60x60 mm of small length (in order to reduce the effect of continuity) located in the corrugations of the facade surface of the panel;
- steel rolling corners 50x5, which are supports for piece cargo weighing 20 kg.

Loading was carried out in steps. At each stage, load stop was provided until the readings of the deflection meters and indicators were stabilized. The deflections of the panel and the horizontal displacements of the flanges relative to each other were recorded with measuring instruments. The external manifestations of changes in the stress-strain state were observed visually. Upon reaching the load level corresponding to the control load in terms of stiffness, the load stop lasted for 30 minutes.

3.4 Facade panel samples testing for active wind pressure

Vertical (deflections) and horizontal movements of the structure were recorded with measuring instruments (Figure 2). In the middle of the span 6ПАО deflection meters with a division value of 0.01 mm were installed: П-7 (8, 10, 11) – for measuring vertical displacements, П-9 – for fixing local deformations (warping) of a wide flange of the cassette profile, П-3 (4) – to fix the possible horizontal displacements arising in the bending-torsional form of buckling. On the O-1 (2) and O-5 (6) supports, ИЧ-10 indicators were installed with a division value of 0.01 mm for measuring the settlement of supports.

Based on the readings of the deflection meters, taking into account the sediment of the supports, the deflections in the middle of the span of the panel at each loading stage were determined (table 1). Based on the average values of the measurements of deflections at 4 points along the width of the panel, a graph of the dependence of the deflection on the bending moment, corresponding to the test load: f_e-M_e was made.

Between the 4th and 5th loading steps, the appearance and subsequent development of local stability loss of a narrow cassette profile flange of the panel edge zone was recorded (figure 5 shows the buckling of the compressed cassette profile flanges). However, it is worth noting that the loss of stability of part of the section is not a criterion for structural failure (the onset of the limit state) [1, 9, 10]. This stage of operation of

Table 1 – Deflections

№ Loading	M, *m	Deflection, mm				
		f7	f8	f10	f11	average
0	0/450	-	-	-	-	0/7.49
1	219/669	4.00	4.04	4.09	3.93	4.02/11.51
2	423/873	7.43	7.62	7.79	5.73	7.14/14.63
3	603/1053	8.66	11.02	11.46	11.41	10.64/18.13
4	699/1149	12.63	13.13	13.67	13.57	13.25/20.74
5	927/1377	17.39	18.22	19.13	19.10	18.46/25.95
6*	979/1429	17.84	19.82	20.73	20.62	19.75/27.24
6	1083/1533	22.06	22.97	22.12	23.99	22.78/30.27
7	1263/1713	28.13	29.22	27.49	29.29	28.53/36.02
8	1336/1786	31.24	32.60	34.30	34.20	33.08/40.57
0	0/450	4.29	5.05	5.77	6.08	5.50/12.79

Note: above the line the values of moments and deflections without taking into account the load due to self-weight of the panel are shown, below the line — taking into account the load from self-weight

Figure 3 – General view of the scheme of panel test. Loading stage 8 *

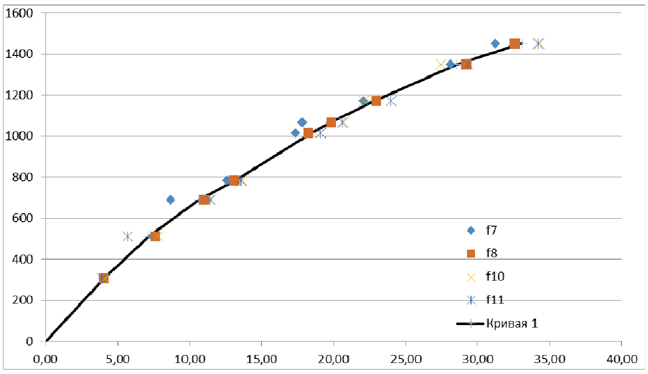


Figure 4 – Dependence of the deflection f_e, mm on the bending moment M_e, *kg • m* during the test

the panel is characterized by an increase of displacement values compared to increase of load. A further increase in resistance is ensured by the operation of the effective cross section up to the achieving of the yield stress or to the ultimate value of the deflection of the panel [A.6.1, 1]).

In the loading range of 600-800 kg•m, gaps appeared at the points of joining of the profiled sheet to the cassette profile by an blind rivet (Figure 5), which indicates the development of the first form of loss of stability of the cassette profile. At the same time by the П-9 deflection meter, warping (bending) of the stretched wide flanges of the cassette profile was recorded.

Due to the noted phenomena (loss of local stability and warpage), the subsequent behavior of the structure under load is non-linear until the design load value (value established by the project) was reached. Note that we are talking about the value of the test load, and not about the maximum possible value of load, corresponding to the achievement of the ultimate limit state. The tests are not brought to the failure because of the presumed tests for fire resistance.

3.5 Analysis of experimental data and determination of resistance according to the results

An integral characteristic of the stress-strain state of bent elements is a graph of the dependence of the deflections on the bending moment $f - M$. For the analysis of the test results of the panel, the following dependency graphs are constructed (figure 6):

- Dependence 1 – of the experimental values of the deflections $f_e - M_e$ according to the average values of measurements;
- Dependence 2 – theoretical values of deflections calculated on the basis of the total cross section $f_{T1} - M_э$;
- Dependence 3 – the same on the basis of full reduction of the section $f_{T2} - M_e$ according to the provisions of TKP EN 1993-1-3 [1];
- Dependence 4 – the same on the basis of partial reduction of the section $f_{T3} - M_e$, where $f_{T3} = 0.5 \cdot (f_{T2} + f_{T1})$.

The initial segment of the dependence on both the self-weight of the panel and test devices is set by calculation.

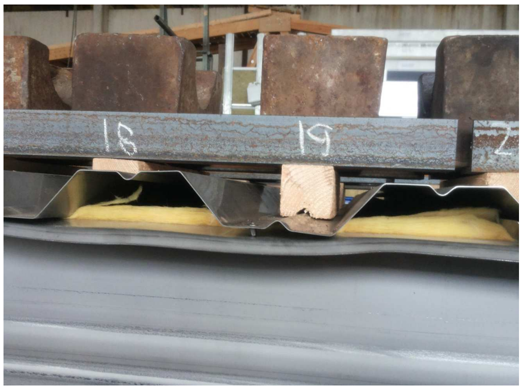


Figure 5 – Loss of stability of the compressed narrow flange of the cassette profile of the edge zone of the panel and the destruction of the blind rivet joint. Loading stage 6

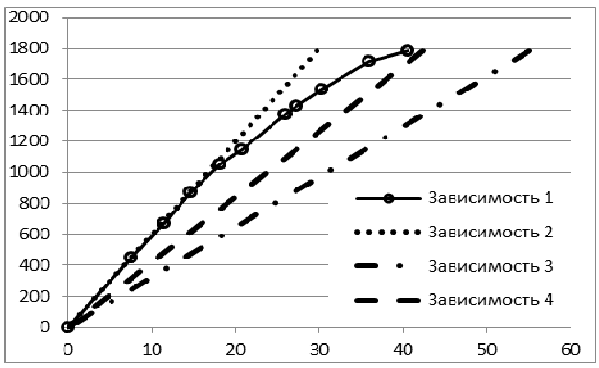


Figure 6 – Dependence of the deflection *f*, mm on the bending moment *M*, kg * m

Comparing the obtained dependencies, we can note:

- good convergence of experimental data (dependence 1) and theoretical values (dependence 2) up to the 4th stage of loading (M_{Ed} = 786 kg·m), i. e. until the loss of local stability of compressed narrow flanges of peripheral zones of cassette profiles;
- the divergence of dependencies 1 and 2, increases with increasing moment. If we take into account the partial reduction of the cross section, the outline of the experimental dependence 1 is closer to the theoretical dependence 4 than to the dependence 3. Moreover, the theoretical values are smaller than the experimental ones, i. e. the requirements of the ultimate state in resistance are observed. Unfortunately, this intermediate stage of the section, which allows to obtain the optimal thickness of the cassette profile sheet, is predicted only by a complex analysis of stress-strain condition using special software systems and cannot be implemented in real design. Note that according to subsection 7.1 of the TKP EN 1993-1-3 [1], partial reduction is alternatively allowed in defining deflections;
- uniform distribution of deflections along the width of the panel at all stages of loading.

The wall panel was tested for bending moment from the applied load M_{Ed} = 1336 kg·m, while taking into account its self-weight M_{obs} is 1786 kg·m. The adjusted value of this moment $M_{adj,i}$ according to clause A.6.3 [1] is:

$$M_{adj} = M_{obs} / \mu_R = 1786 / 1.04 = 1717 \text{ kg·m},$$

where μ_R – the adjustment coefficient:

$$\mu_R = \left(\frac{f_{yb,\mathrm{obs}}}{f_{yb}}\right)^{\alpha} \cdot \left(\frac{t_{\mathrm{obs,cor}}}{t_{\mathrm{cor}}}\right)^{\beta} = \left(\frac{250.7}{220}\right)^{\alpha} \cdot \left(\frac{1.18}{1.20}\right)^{1} = 1.04,$$

here α = 0,5 since the loss of local stability is a form of destruction, β = 1 since $t_{\mathrm{obs,cor}} \le t_{\mathrm{cor}}$.

The characteristic value of the moment resistance based on only one test result:

$$M_k = 0{,}9\eta_k M_{\mathrm{adj}} = 0.9\cdot 0.9\cdot 1717 = 1391 \text{ kg·m},$$

where η_k – coefficient which should be taken depending on the failure mode. During the test of the panel there is a loss of local stability in a limited sectional area of the panel (local buckling), therefore η_k = 0,9.

The design value of the moment resistance M_d is established on the basis of the corresponding characteristic value M_k, determined according to the test results:

$$M_d = \eta_{\mathrm{sys}} \cdot \frac{M_k}{\gamma_M} = 1391 \text{ kg·m},$$

where γ_M =1.0 – partial safety factor;

η_{sys} – is a conversion factor for differences in behaviour under test conditions and service conditions. When testing fullsize samples with restraints identical to natural, η_{sys} = 1.0.

Conclusion

Taking into account the results of experimental and theoretical researches, the following conclusions can be made:

1. Design of facade panels on the basis of calculation methods without bearing in mind the joint work of its components doesn't reflect the actual resistance and stiffness of the panel, as a result to use only this approach is neither effective nor objective. In this regard, it is advisable ***to establish the ultimate resistance and stiffness with the help of testing*** till to the accumulation of a sufficient experimental database and to more accurate development of an approved calculation procedure for determining the resistance and stiffness of a panel of the considered type.

These tests can be regarded as acceptance tests [clause A.4.1, 1]. So if more than three prototypes of panels have been tested and they meet all the test requirements, the remaining panels of the series "*may be accepted without further testing provided that they are similar in all relevant respects to the prototypes*" [clause A.4.2(1), 1].

2. The distribution of deflections along the width of the panel at all stages of loading is uniform. This allows us to consider the panel as a spatial structural system, which has such important properties as redistribution of internal forces between its components, survivability and reliability of the construction. In the tested panel, the redistribution of forces took place between the peripheral zones of the cross section of the last cassette profiles (reduced due to the loss of local stability of the compressed narrow flanges) and the middle zone of the section of the panel, where the local stability of the intermediate flanges of the cassette profile is increased by structural measures.

3. The problem of determining of the effective characteristics of the considered cross sections of stainless steel cassette profile is one of the most important when performing the theoretical calculation methodology in accordance with TKP EN 1993-1-3 [1]. It is worth noting that the level of reduction of the cross section of the panel largely depends on the compressed narrow flange of the stainless steel cassette profile at the edge zones of panels. To increase the efficiency of the section, it is recommended:

a) to increase the width of the narrow flanges of the cassette profile. In this case, an additional effect is achieved: a significant reduction in the number of joints attaching the facade surface to the cassette profile, or

b) to reinforce the compressed narrow flange by an additional thin-walled profile attached by spot welding.

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