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DESIGNING OF FRAME STRUCTURES OF WELDED DOUBLE-T WITH VARIABLE CROSS SECTION AND FLEXIBLE WALL

In field of metal engineering research usually aimed at finding new designs of buildings and structures, with a focus on reducing consumption of materials. Most rational constructive form of the supporting framework in terms of the distribution of internal forces is a portal frame with rigid connection of columns and beams. Steel portal frames are one of the most common design solutions skeletons of single-storey industrial buildings. These frames can be carried out through grid elements and weldments with solid wall.

Despite extensive use of existing technical solutions of grid structures are not currently enough technologic, as their production does not match the current level of technological development of the industry.

Any grid structure is composed of a large number of assembly marks. Each element at the factory passes a long chain technological process: editing, markup, mechanical or gas cutting, drilling or punching holes, milling, etc. In the manufacture of structures, selected parts are the plant way up to 1000 m and more. Mechanization and automation, implemented at the plant for metal, greatly facilitating labor of the worker, practically do not change the process and little increase productivity. As a result, the overall time required for production structures practically not reduced.

It should be noted that most technologic are the manufacture of planar structures made of sheet or strip. Efforts in this case may be lower than 1.5 ... 2 times.

Practice of designing steel frames shown that the use of elements with variable cross section provides a more cost-effective solutions in comparison with the elements of constant cross section.

The principle of designing frame elements with variable cross section based on the rational distribution of material according to the diagrams of bending moments and unloading effects caused by displacement of axis.

The easiest way to facilitate the elements of frame of welded beams with variable cross section and increase their efficiency - the use of a thin wall of conventional flexibility $\lambda w = 6 ... 10$.Using double-T with variable cross section with flexible walls can be obtained best way of redistribution steel for strength and resistance between the walls and shelves [1].

Feature of work of double-T frame with flexible wall caused by the behavior of thin wall under load. Traditionally, the bearing capacity of double-T considered exhausted when stresses in the wall are reaching of their critical values – occurs local buckling. But the analysis of supercritical behavior of flexible plates showed that cross section while moving to a new steady state equilibrium, so the bearing capacity of the element as a whole is not exhausted. The use of reserve of carrying capacity, which is the account of supercritical stage of work thin plate can significantly reduce the weight and cost of structures. The issue of work flexible supported plate after buckling considered by scientists from the works of Papkovych, Sokolov, Feppl, Th. Karman, Bubnov etc. Theoretical studies of thin-walled beams and building frames divided into two directions.

The method of first direction is the construction of conventional static permissible limit state model. The most famous became the method proposed by american researchers Basler and Thurlimann [2]. In further studies, models of sections of beams subjected to clarify and supplement. In particular model of Basler-Tyurliman complemented by Rockey, Skaloud, Broude, Moiseev, Predtechenskiy, Pogodaev, Korczak, Kalenov.

Research methods of the second direction are reduced to solving the equations of equilibrium and compatibility of of deformations Karman-Marghera for flexible elastic plate considering initial imperfections with appropriate boundary conditions taken from the hinge resting until anchorage. Notable works in this direction have received practical application in studies of Djubek, Aare, Lepik, Evstratov etc.

Methodologically to calculate the thin-wall hinged beam construction in [3] on the basis of existing experimental and theoretical framework was proposed to exclude from the work cross section of the wall compressed part and into account the of carrying capacity of element to introduce the concept of the reduced section. Beam on length is divided into conditional sections – for supporting zones predominant are effect of shea forces, in the middle the determining factor is the effect of the bending moment. Part of the beam is running on the implications of normal and shear stresses. Checking the strength of bending thin-walled double-T beam with vertical ribs, and without them, recommend by the following formula (1):

$$M_{npe\partial} = kWR \le M_p \tag{1}$$

In this formula $M_{npe\partial}$– limiting bending moment that may be considered by section; k– reduction factor taking into account the exclusion of the compressed wall in her work on the bend in the supercritical phase; W– resistance moment of cross-section; R– resistance of steel; M_p– maximum calculated bending moment within the compartment. The coefficient k is semiempirical, its value depends on the flexibility of the walls and the ratio of the area of the shelf to the wall and are given in [3].

With the overwhelming effect of transverse forces for beams with vertical rigid ribs inserted formula of verification strength (2):

$$Q_{npe\partial} = \tau_{\kappa p} ht + 2ct\sigma_t \frac{1}{1+\alpha^2} \le Q_p \tag{2}$$

The first part considers the critical shear stress in the cross section, the second – the formation of diagonal folds within the support beam compartment.

For beams with flexible wall without ribs cross section to check the strength of the effect of transverse forces should use the formula (3):

$$Q_{npe\partial} = \tau_{npe\partial} ht \le Q_p \tag{3}$$

Where $\tau_{npe\partial}$ – limiting value of shear stress, which is (4):

$$\tau_{npe\partial} = \left(\frac{820\cdot 10^4}{\lambda^2} + 360\right)\sqrt{\frac{R}{2100}} \tag{4}$$

In the case compatible action of bending moment and transverse force in the tutorial describes the formula (5):

$$M_{npe\partial} = kW\sigma \le M_p. \tag{5}$$

In (5) - normal stress in the compressed zone of the beam:

$$\sigma = R\left(1,66 - \frac{0,12}{\beta} - \frac{\tau}{\tau_{npe\partial}}\right) \tag{6}$$

Further development of the theory of calculation of beam structures with flexible wall entered into [4]. It also takes into account the work of supercritical wall, and in the calculation of of carrying capacity of the wall of cross section turns up $0,85t_w\sqrt{E/R_y}$ in the compressed zone and twice more in height stretched. The maximum bending moment is recommended to determine the following (7):

$$M_u = R_y th^2\left[\frac{A_f}{th} + \frac{0,85}{\bar{\lambda}_w}\left(1 - \frac{1}{\bar{\lambda}_w}\right)\right] \tag{7}$$

Limiting value of the transverse force (8):

$$Q_u = R_s t_w h_w \left[\frac{\tau_{cr}}{R_s} + 3,3\left(1 - \frac{\tau_{cr}}{R_s}\right)\frac{\beta\mu}{1+\mu^2} \right] \qquad (8)$$

In the compatible action of bending and shear force factors performance check of cross section necessary to carry out by the following formula (9):

$$\left(\frac{M}{M_u}\right)^4 + \left(\frac{Q}{Q_u}\right)^4 \le 1 \qquad (9)$$

Equation (9) is the result of approximation of a curve of carrying capacity section (Fig. 3) and received it on the basis of research [5] for engineering calculations, so a certain reserve, which can be observed on figures.

By theorem of Papkovych [6] strength of section can be estimated by the formula $|M/M_u| + |Q/Q_u| \le 1$; german standard DIN 18 800 [7] for thin plates under the action of normal and shear stresses in the cross section recommends using the formula $(\sigma/\sigma_u)^{1,25} + (\tau/\tau_u)^{1,25} \le 1$; SNIP II-23-81* and Eurocode 3 [8] to limit local stability of wall injected squared components, ie $(\sigma/\sigma_u)^2 + (\tau/\tau_u)^2 \le 1$. Broude in his works [5, 9] recommended for the case of the double-T cross section on the implications of normal and shear stresses applied condition of strength by (10):

$$\left(\frac{\sigma}{\sigma_u}\right)^2 + \left(\frac{\tau}{\tau_u}\right)^2 + \alpha\left(\frac{\sigma}{\sigma_u}\right)^2\left(\frac{\tau}{\tau_u}\right)^2 \le 1 \qquad (10)$$

Coefficient α for sections of conventional flexible wall $\lambda_w \le 3,5$ is $\approx 0,65$, for $\lambda_w \ge 5,5$ $\alpha \approx 0,92$.

Fig. 1 shows the strength curves for different methods of determining carrying ability for flexible plates.

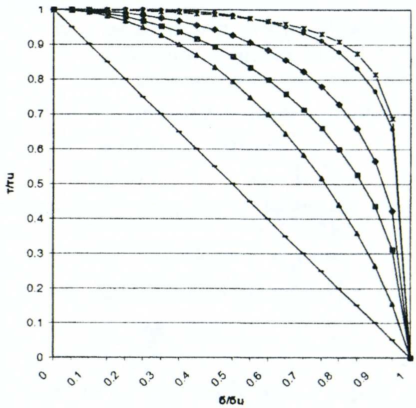


Figure 1 – Curves of carrying capacity of cross-sections at compatible action of normal and tangential stresses on different methodological approaches:

– Papkovych theorem;
– DIN 18 800;
– SNiP II-23-81* for walls of double-T beams and Eurocode 3;
– Broude for normal double-T beams;
– guide to SNiP II-23-81* for beams with flexible wall;
– Broude for thin-wall double-T beams

As seen from fig. 1, thin-walled structures use the full section bearing capacity and approach in their work to rod structures in which bending perceived by shelves and shear by wall.

All of the above research related to the study of beams. With regard to the calculation of frame structures with variable double-T section and flexible wall does not exist transparent public information or recommendations on the design of this type of construction. The only way to detailed analysis of the stress-strain state frames with welded beams with variable section and flexible wall for the designer – is to create a finite element model of the structure. But it is a laborious and lengthy process that requires highly skilled engineers, so the end does not always justify the means.

Calculation of thin-walled frame structures puts before engineer some complicated tasks. First, it is difficult to predict the nonlinear behavior of structures after local buckling of wall section. In addition, the exclusion from work compressed part of the wall causes to redistribution of internal stresses and the development of elastic-plastic deformation in shelves of section. In strict mathematical solution is very complicated and can not be used by ordinary designers.

The only rational way of formalizing the methodology of calculation of the Double T frame structures with flexible wall is conducting experimental testing and development of semi-empirical formulas for calculating them.

In order to the planned research at the Kiev National University of Construction and Architecture designed the experimental set. Construction is made as strangulated cantilever rack loaded concentrated load at the free end in which there is both longitudinal force, shear force and bending moment in backing. The experimental setup is shown in Fig. 2.

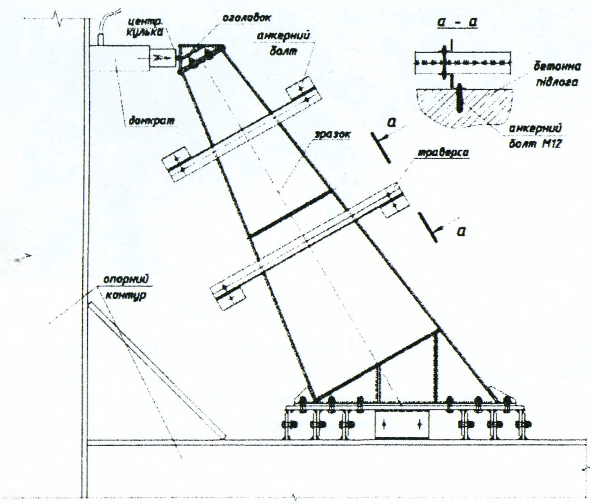


Figure 2 – Experimental set

Construction is is placed in the horizontal plane. Conventional flexibility of wall in cross section with a maximum height is 10.7. Load transfer was performed in 11 stages with increasing efforts in jacks at each stage of 10 kN with fixing metering at each stage.

To predict the results of experimental tests and the previous analysis of the structures developed numerical model of the sample. The research is realized in the environment of software system Lira 9.6. Modeling was performed by planar shell finite elements. Calculation of is made in linear and nonlinear setting. According to the results of numerical experiments obtained values of stresses in the design section, the form of buckling of wall and overall system displacements (Fig. 3).

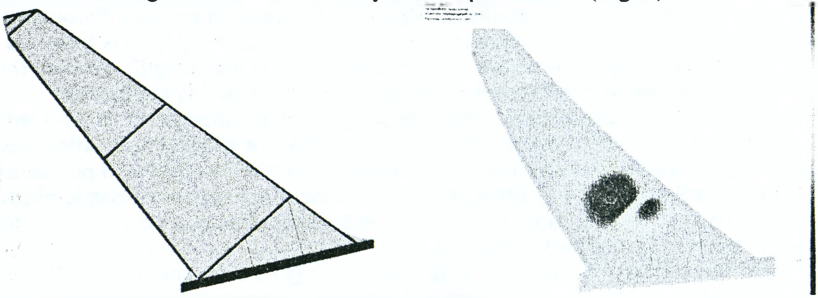


Figure 3 – Numerical model and deformed scheme of set among software complex "Lira"

The result of the experiment was set options of structure deformation, determined buckling character of wall and depletion the bearing capacity of cross sections (Fig. 4).

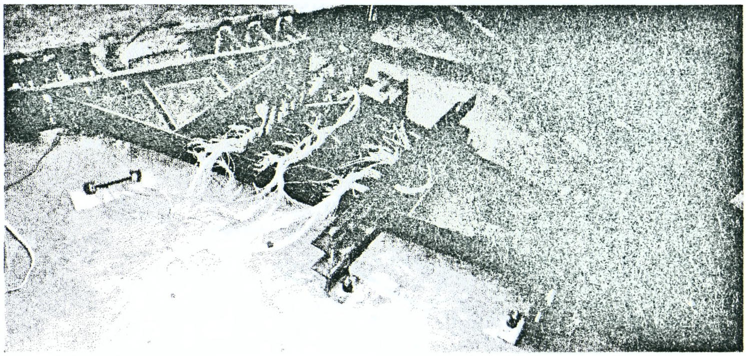
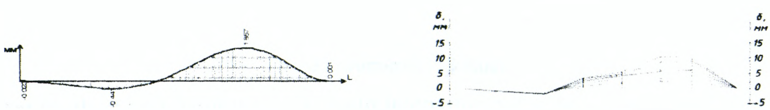


Figure 4 – General view of the experimental set

The nature of the buckling of wall entirely matches the the results of numerical studies - form deformation of corresponds to the 1st form of buckling on the results of the numerical experiment in programming complex Lira 9.6. The maximum deformation of of wall of the plane were observed in the compressed zone section.



a *b*

researches *a)* and experimental tests *b)*
Figure 5 – Nature of wall deformation by the results of numerical

The limiting condition characterized by experimental model achievement stresses of yield strength in compressed shelf-section. Analysis of the stress state cross section at various stages of loading showed higher levels of normal stresses in the compressed part of wall versus stretched even when subject only pure bending. This feature of the stress distribution was observed in all sections. The reason for this effect is that stretched rack with relatively high levels of stress prevents buckling it affixed to the wall, through the lateral deformation observed near the compressed shelves and of wall led to exclusion from work (Fig. 6). In Fig. 6 indicated: 1) diagonal tension field of wall; 2) deformation of shelf in supercritical phase of wall work; 3) isolines of deformation of the flexible wall out the plane; 4) curves of deformation for compressed diagonal of wall; 5) design section 1; 6) design section 2; 7) design section 3; 8) diagrams of bending moments in shelves.

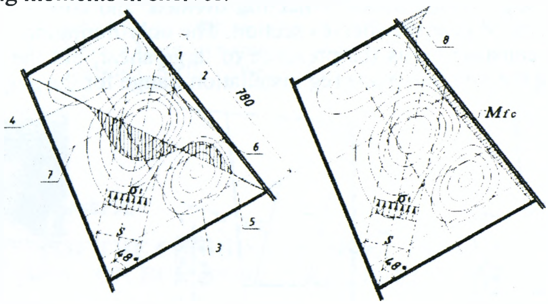


Figure 6 – Stress-strain state of the support section of the experimental model

It was established that in supercritical phase of work in a flexible wall there are additional stress in shelves of section from the effects of local bending moments. Bending moments in compressed shelves caused by deformation of wall and "sagging" shelves that works like a beam on elastic foundation with variable coefficients of elastic reaction (depending on the nature of the of wall deformation). Thus, the maximum normal stresses in the compressed zone of the frame with a flexible wall can be defined as follows:

$$\sigma_{\Sigma} = \frac{N}{A_{red}} + \frac{M}{W_{red}} + \frac{M_{fc}k_c}{I_{fc}} y_0 \le R_y \gamma_c \text{, in which} \qquad (11)$$

N, M – compressive force and bending moment from the action of external loads; A_{red}, W_{red} – area and moment resistance of the reduced double-T section with a flexible wall; M_{fc} – additional bending moment that occurs in the belt after buckling of wall; I_{fc} – moment of inertia of the T-section formed of compressed shelf and part of of wall height h_{wred};

y_0 – distance from the center of gravity of T-section of compressed shelf zone to the brink; k_c – coefficient taking into account redistribution of the additional bending moment, which dependent on the flexibility of wall:

$k_c = 1$ for sections with $\overline{\lambda_w} > 9$;

$k_c = \overline{\lambda_w} / \overline{\lambda_{wu}}$ for sections with $\overline{\lambda_w} \le 9$.

Analysis of the results showed the unsuitability of existing methods for calculating frame elements with a flexible wall. The difference between theoretical and experimental results with existing methods is 30%, while the formula 11 the difference is less than 3%.

In addition, to check the bearing capacity of frame members with variable double-T section and flexible wall designed strength condition (12), taking into account the simultaneous action of three internal forces – longitudinal, shear forses and bending moment:

$$\left|\frac{N}{N_u}\right| + \left(\frac{M_x}{M_{u\varphi}}\right)^2 + \left(\frac{Q}{Q_u}\right)^4 \leq \gamma_c; \qquad (12)$$

N, Mx, Q – the forces in the section; *Nu, Muφ, Qu* – corresponding limit values of efforts.

In formula (12) coefficient of work conditions γ_c considering the difficult stress-strain state of structures with flexible wall is recommended to limit the value of 0,95.

To improve the efficiency of design solutions welded double-T with flexible wall in nodes of frames was designed flanged node [10]. The new construction of precipitous and ridge flange nodes projecting divided, and consists of two separate plates (Fig. 7), located near the shelves section. The only limitation in the application of the proposed construction is the presence of large shear, but the lack is easily to eliminate placing the junction plane (eg installation flange for the angle bisector).

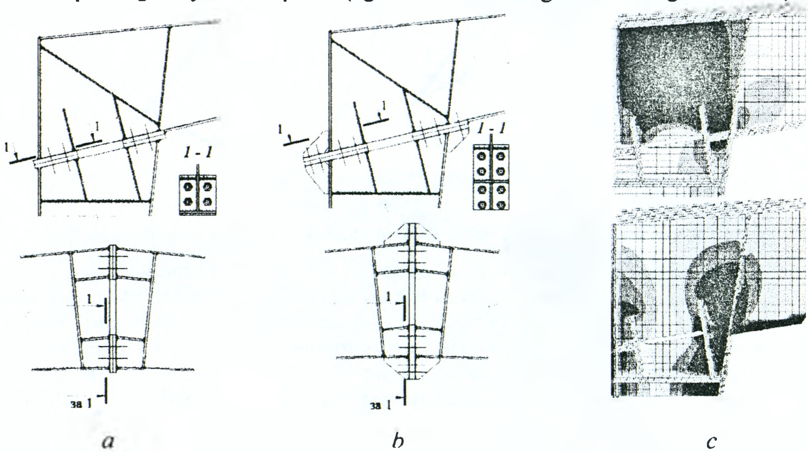


Figure 7 – The design of flange node for frames smaller spans and loads (a); with the removal of the bolts off section to span buildings (b); diagram of normal and shear stresses at the node (c)

Conclusion. Increased of flexibility in the wall of double-T frame structure is logical and evolutionary way to improve solid section. But the work of these systems is complex – the nonlinear deformation of flexible plate that loses stability at the early stages of loading, as well as the simultaneous action of the compressive force, bending moments and shear forces in the cross section require a review of existing methods for calculating thin-walled double-T. Based on its own numerical and experimental studies developed methodology for load-bearing ability of frame structures with a flexible wall.

REFERENCE

1. Скляров, І.О. Раціональна висота перерізу двотаврових рамних конструкцій змінної жорсткості з гнучкою стінкою: сборник научных трудов «Современные строительные конструкции из металла и древесины» / І.О. Скляров, С.І. Білик – Одеса: ООО «Внешрекламсервис», 2010. – ОГАСА №14 – Ч. 1. – С. 230–235.

2. Basler, K. Strength of plate girders in bending / K. Basler, B.Thurlimann // Proc. of American Society of Civil Engineers. – Vol. 87. – №. ST 6 – 1961. – P. 153–181.

3. Руководство по проектированию стальных тонкостенных балок (к СНиП II-В.3-72) – М.: ЦНИИПРОЕКТСТАЛЬКОНСТРУКЦИЯ им. Мельникова, 1977. – 28 с.
4. Пособие по проектированию стальных конструкций (к СНиП II-23-81*). – М.: ЦИТП Госстроя СССР, 1989. – 148 с.
5. Броуде Б.М. О закритическом поведении гибких стенок стальных стержней / Б.М. Броуде // Строительная механика и расчет сооружений. – М.: Стройиздат, 1976. – № 4 – С. 7–12.
6. Папкович, П.Ф. Строительная механика корабля / П.Ф. Папкович – Л.: Государственное союзное издательство судостроительной промышленности, 1941. – Т. 28. – 15 с.
7. Structural steelwork; analysis of safety against buckling of shells: DIN 18800-4 // Deutsches Institut Fur Normung E.V. (German National Standard) / 01-Nov-1990 – 23 pages.
8. Eurocode 3: Design of steel structures – Part 1.3: General rules – Supplementary rules for cold formed thin gauge members and sheeting // European Committee for Standardization (CEN), 22 August 2001 – 128 pages.
9. Броуде, Б.М. К расчету балок с гибкими неподкрепленными стенками / Б.М. Броуде, Б.И. Моисеев // Строительная механика и расчет сооружений» – М.: Стройиздат. – 1978. – № 1 – С. 60–61.
10. Жорсткий фланцевий вузол рами зі зварних двотаврів з гнучкою стінкою: патент на корисну модель № 56206 Україна, МПК (2006) E 04 C 3/04. – №201006230; заяв. 25.05.10; опубл. 10.01.11. – Бюл. № 1.

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AN ANALYSIS OF THE COMBINATIONS OF CLIMATIC ACTIONS ON BUILDING STRUCTURES

1. INTRODUCTION

Coincidence of actions plays an important role in structural calculations so the combinations of actions have been introduced into codes. They are expressed using combination factors. The values of these factors, named as factors, have usually been determined based on theoretical considerations. However, as climatic actions depend on the climatic conditions in different geographical locations, it is necessary to analyse the historical meteorological data. Among them wind and snow are most frequent to be used, also ambient air temperature. The paper deals with a probabilistic analysis of combinations of pairs of actions: wind and snow as well as wind and air temperature. The objective of the presented analyses is the verification of the combination factors for variable climatic actions given in the codes, e.g. [1] and to identify the values determined on the base of local meteorological data. Calculations are based on the data measured at Polish meteorological stations of the Institute for Meteorology and Water Management - a State Research Institute. Method of analysis as well as results based on data from two stations are presented here as examples. Annual maxima of these actions (and also minima of the air temperature) have been analysed as main data assuming that the climatic year lasts from October 1 to September 30. Gumbel probability distribution [2] has been used as a border probability distribution for all actions. The interdependence of snow load and wind velocity pressure as well as between wind and air temperature actions can be received for different return periods. Combination factors for characteristic values of those actions with return period of 50 years have been proposed.

2. METHOD OF THE ANALYSIS

An analysis of the coincidence of the snow load on the ground and wind velocity pressure is presented as an example of the method used in the paper also for combinations of wind and air temperature actions.