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Influence of stiffness of node on stability and strength of thin-walled structure

Влияние жесткости узловых соединений на устойчивость и прочность тонкостенных конструкций

I.V. Atavin,
B.E. Melnikov,
Peter the Great St. Petersburg Polytechnic University, St. Petersburg, Russia
A.S. Semenov,
Dresden University of Technology, Dresden, Germany
N.V. Chernysheva*,
E.L. Yakovleva,
Peter the Great St. Petersburg Polytechnic University, St. Petersburg, Russia

Студент И.В. Атавин,
д-р техн. наук, профессор Б.Е. Мельников,
Санкт-Петербургский политехнический университет Петра Великого, г. Санкт-Петербург, Россия
канд. физ.-мат. наук, ведущий научный сотрудник А.С. Семенов,
Дрезденский технический университет, г. Дрезден, Германия
канд. техн. наук, доцент Н.В. Чернышева*,
канд. техн. наук, доцент Е.Л. Яковлева,
Санкт-Петербургский политехнический университет Петра Великого, г. Санкт-Петербург, Россия

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Ключевые слова: легкая стальная тонкостенная конструкция; крутильная жесткость; метод конечных элементов (МКЭ); стеллаж; балка; стойка; ярус; потеря устойчивости; критическая сила

Abstract. The article is devoted to the evaluation of the torsional stiffness of the beam-pillar nodal connection of thin-walled rack structures. Results of in-place testing of several girder rack models are described, beam model and combined one are considered. Computational finite element (FE) models of these racks are designed to develop a FE analysis by software complex ANSYS. Assessment of the torsional stiffness of the beam-pillar nodal connection in these structures is obtained by comparison of results of computation and experimental ones. Critical buckling load for the first buckling mode is determined both experimentally and computationally. Buckling modes of “out of the face plate” and “in the face plate” are considered. The influence of the torsional stiffness of the beam-pillar nodal connection on bearing capacity and buckling stability of thin-walled rack structure is investigated. Correspondence of these characteristics and number of tiers is revealed.

Аннотация. Выполнена оценка крутильной жесткости узлового соединения балка-стойка в тонкостенной конструкции стеллажа. Проведены натурные испытаний различных моделей стеллажей, рассматриваются конструкции балочного и комбинированного типов. Построены расчетные схемы этих конструкций, произведены расчеты методом конечных элементов (МКЭ) в программном комплексе ANSYS. Из сравнения экспериментальных данных и результатов расчета на устойчивость получены оценки значения жесткости на кручение для соединения балка-стойка в этих конструкциях. Экспериментально и численно определены значения критической силы, соответствующей первой форме потери устойчивости. При этом рассматривались формы потери устойчивости во фронтальной плоскости конструкции и с выходом из плоскости. Проанализировано влияние крутильной жесткости соединения балка-стенка на несущую способность и устойчивость конструкции тонкостенного стеллажа. Отмечена зависимость этих характеристик от количества этажей в модели стеллажа.

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1. Introduction

Wide use of metalwork in industrial and civil engineering is one of the modern development trends in construction industry. Metalwork attractiveness is connected to development and deployment of new production technologies.

Use of construction designed of light steel cold-formed thin-walled cross-section (LSTC) is a new and fast-progressing trend. The low metal consumption, optimum parameters of cross-section, simplicity of transportation and installation are the general advantages of LSTC.

Estimation of strength and operational characteristics of new types of sections and connections is made gradually. The first stage is a test of real construction, and the second stage is a numerical analysis of construction by specialized program complexes. The standard estimate norms are used to interpret data obtained. Other important factors that influence on operational characteristics of LSTC can be defined experimentally. In particular, the longevity of connections which considerably influences on endurance of LSTC can be defined.

Eurocode-3 and AISI are the relevant standards and regulations to be in use now for analysis of LSTC.

The first who have investigated deformation of thin-walled constructions was S.P. Timoshenko [1, 2]. Torsion stiffness of thin-walled bar of open cross-section is estimated experimentally. Also the behavior of thin-walled bars of different cross-section is investigated.

The principles of buckling analysis of thin-walled bar of open cross-section in view of application of force out of cross-section core as well as of torsion and bending analysis are worked out by V.Z. Vlasov in [3–5]. Another way to analyze bending with constrained torsion of thin-walled bar is proposed by A.A. Umanskyi in [6].

D.V. Bychkov and A.K. Mrozchinskyi [7–9] perform a significant contribution to the theory of calculation of thin-walled constructions. They worked out an algorithm of analysis of torsion beam forces depend upon number of beam spans and way of end fastening. This algorithm employed as force or deflection method allows to analyze restrained torsion of frames and other thin-walled constructions.

The FE analysis is applied to torsion and bending of thin-walled constructions in paper by A.R. Tusnin [10]. This approach appreciates influence of type of connections, centre-of-gravity position, beam deflection and amount of eccentricity.

Improved algorithm of LSTC analysis is elaborated and fail-safety and endurance of constructions and way to increase them are examined by V.A. Rybakov [11, 12]. Also Russian regulations imperfections are pointed out and ways of correction them are proposed.

Reviewing in paper [13] provides insight into production engineering, features of setting-up and operation of LSTC.

A method of thin-walled constructions with built-up section analysis is developed in paper [14]. This approach consists in general buckling analysis in terms of limit state of stress in view of form of section.

Composite materials technologies of production reinforced concrete construction are considered in [15, 16]. In this context, quality control, materials specification, setting-up and operating regulations, structural reinforcement are also discussed.

Deformations and damage accumulation take their cue for the approaches of thin-walled constructions analysis developed in [17–19].

General rules, definition, typical thin-walled member specification, reference models and design techniques are presented in [20–22].

The effect of temperature (from low to high, including fire conditions) on the strength and operating characteristics of thin-walled structures is discussed in [23, 24].

In-place testing results and results of simulation experiments by means of specialized software complexes are compared in [25, 26].

Post-buckling behavior columns and rods with open and semi-open thin-walled cross-section are considered in [27, 28]. The effect of eccentric applying of axial load on the deformation and deflections of frames and beams in this connection is discussed in [29, 30].

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A review of Russian and foreign references is made in [35–38]. General attention is paid to the studies of buckling of cold-formed thin-walled beams. The experimental data are compared with values obtained by using both beam and shell finite elements in a computer simulation. In this context, arising problems are formulated and ways to solve them are proposed.

The analytic model of the long span footway bridge is studied and tested in [39, 40]. The effect of the types of thin-walled cold-formed profile members on the operating characteristics of the bridge construction is tested by numerical simulation.

The new specialized software complex “Stalkon” is presented in [41–43]. This software is meant to be used for design and static analysis of thin-walled member construction (three-dimensional one includes) according to modern rules and regulations. An example of pin-ended column’s analysis is presented. The possibility of restrained torsion analysis of thin-walled structures by software complexes ANSYS and NASTRAN is discussed.

The purpose of works [44–46] is to develop a numerical method of thin-walled bar systems design by using semi-sheared and non-sheared theories. Matrixes of the inflexibility of thin-walled finite elements of four various types are created. Some test torsion problems of the thin-walled beam having various supporting are solved by the FEM.

Reduction of thin-walled member’s cross-section is asserted in [47–49] by investigating of load carrying capacity. Optimal cross-section area is calculated on the base of buckling analysis. A problem of analysis of thin-walled member of double-corrugated profile is solved by linear and non-linear approaches. The results are compared with in-place testing results. Also results of in-place test for real thin-walled cold-formed member truss with span of 18 m are considered.

Fatigue analysis of LSTC in terms of nonlinear damage accumulation and fatigue rupture is considered in [50–53]. The energy approach is used in [54–58] to estimate the low-cycle fatigue of thin-walled member structures. A kinetic algorithm of damage analysis of LSTC is developed and experimentally verified. The problem of mechanochemical corrosion of a thin-walled pillar under its own weight is considered in [59].

The objective of this work is to evaluate the stiffness of the beam-pillar connection of thin-walled members of a three-dimensional structure. Stages of implementation of handling the problem are:

- to construct an experimental set-up for testing of thin-walled structures;
- to develop an analytical finite element model of structures by the ANSYS software complex, to verify results of numerical solution experimentally;
- to define stiffness properties of the connection computationally and experimentally.

2. Methods

2.1. Stiffness of connections of horizontal, diagonal and vertical LSTC members

Accounting of a torsional stiffness of connections in a beam rack is important for assessment of girder rack critical buckling load by a stability criterion. The study request provided only the computational and experimental evaluation of a beam rack connection stiffness.

A new approach in girder racks design includes only horizontal braces unlike diagonal bracing of pallet racks that provides the lack of buckling with escaping of the face plane. So in new modification of a girder rack such buckling mode could appear, and corresponding critical buckling load generally depends on torsion stiffness of brace-pillar connection. Therefore the need of accounting of brace-pillar connection stiffness was revealed in the course of the researches. A test trial of indirect observation of the stiffness not the bound to direct measurements is made. To receive precise assessment extension studies are recommended.

A complex study both experimental and computational is pursued to determine torsion stiffness of beam-pillar connection, included in-place testing of girder rack simplified model and the FE analysis.

Beam and pillar members of the model rack are made of BT-PN-1.5 steel band conforming to Russian State Standard GOST 19904–90. The thickness of the steel band is 1.5 mm. The yield stress for steel 20 is 230 MPa, allowable stress used in computations is 160 MPa. Bearing horizontal beam members have closed box-shaped cross-section of 70x30 mm in Figure 1. Bearing vertical members have opened profile cross-section of 90x30 mm in Figure 2.

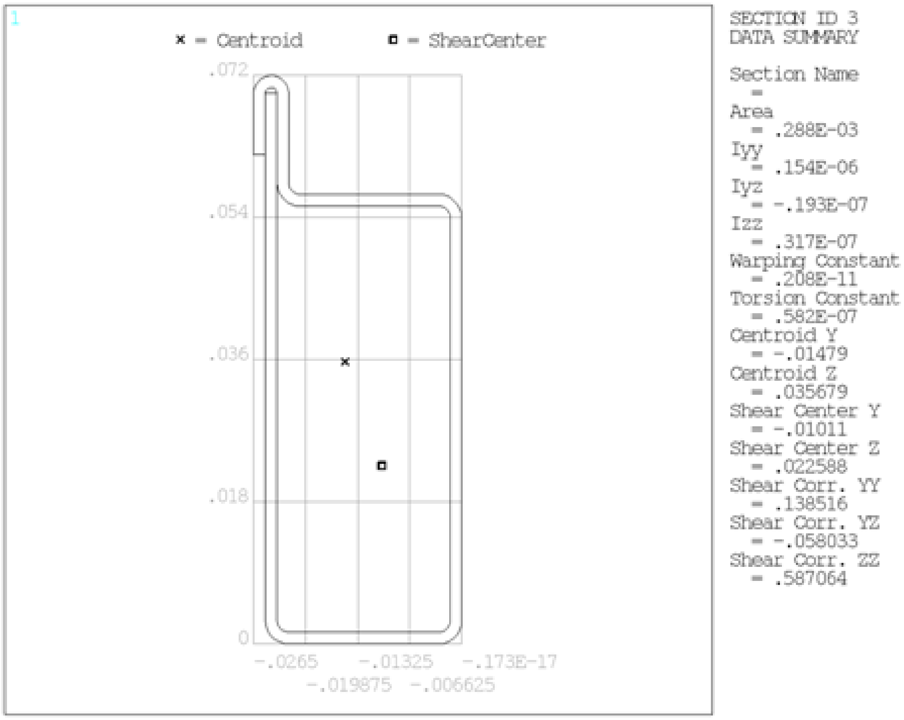

Figure 1. The cross-section of bearing horizontal beams

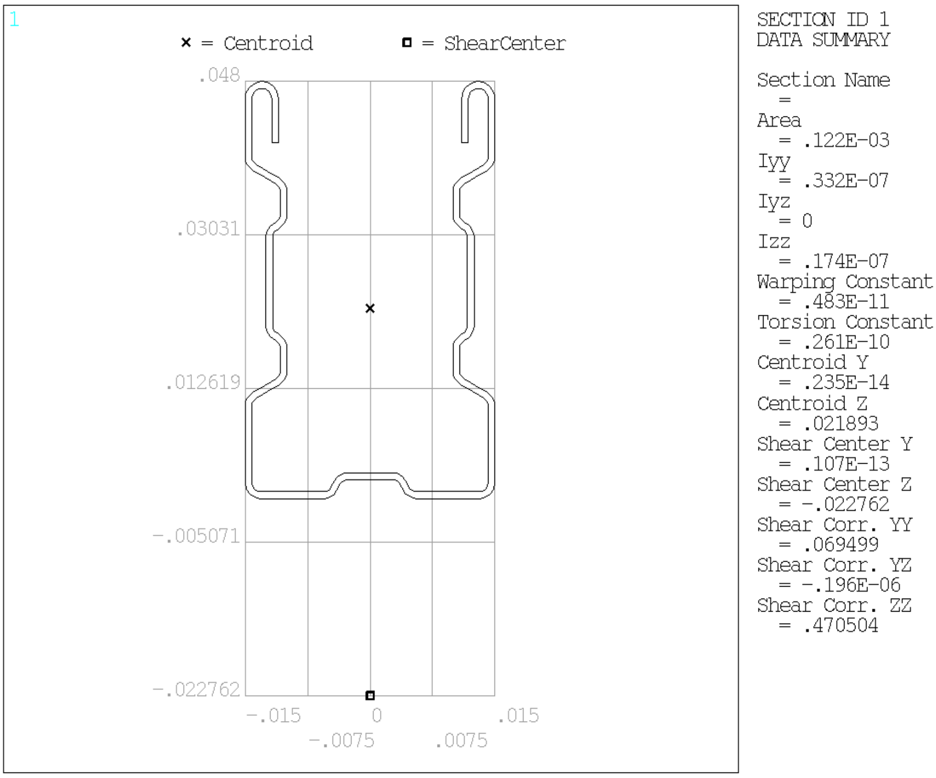

Figure 2. The cross-section of bearing vertical pillars

The model rack studied consists of two horizontal 1500 mm long beams (see Figure 3), the height of the vertical pillars is 1000 mm, each beam is connected with two pillars apart 135 mm from their upper edge and with two pillars apart 110 mm from their lower edge and then the frames obtained are connected with two 300 mm long braces. The distance between the pillars is 1500 mm in face plane direction and 300 mm in crosswise direction. Despite the absence of rack baseplate's horizontal and vertical restraint no displacements of baseplates are obtained during tests.

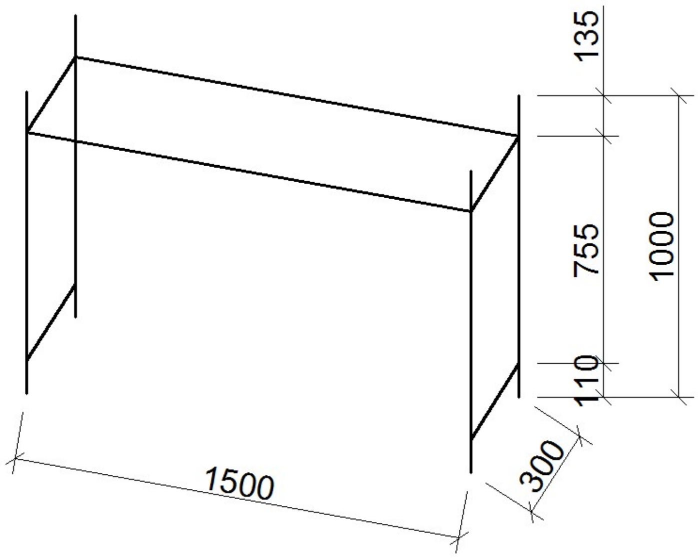

Figure 3. The geometrical parameters of the model rack

The model is loaded by “Armsler” testing machine through auxiliary device intended to simulate a uniformly distributed load. The load was converted from concentrated to distributed using an auxiliary structure. The system was made from channels 16P. The lower part consisted of 4 beams 350 mm long, located transversely with the investigated. The next tier was made of two beams 50 cm long. The final tier had a length of 100 cm. The additive weight load from the auxiliary structure is 0.5 kN, which was taken into account during the tests.

Beams are connected by wire in horizontal direction to prevent a horizontal bending and therefore keep safety arrangements. Central nodes of beams in computational model have rigid connection in z-direction to consider this construction feature. Displacements of pillars are measured by dial indicators.

In an experiment the following results are obtained. Under total load of 11.2 kN the model gets a stability mode without inelastic deformation. This load can be considered as a first buckling mode critical load for the model. The load of 11.2 kN was determined from the indications of a dynamometer attached to the moving part of the "Armsler" power machine.

The numerical simulations were performed by the FE tool ANSYS. The model consists of beam finite elements. This FE model is represented in Fig. 4. Beam-pillar connection torsional stiffness with regard to the turn about Z-axis (plane deformation in face plane) can be calculated by computational analysis of the model.

In computations, the dead load of the beams was not taken into account, since it is negligibly small in comparison with the permissible load of the structure.

Supporting nodes of pillars are connected with rigid base (displacements U_x, U_y and U_z are zeros), angular rotation R_y is also zero, non-zero angular rotations R_x, R_z are defined automatically in the FE computations by means of account of elastic torsional stiffness of supports.

Transfer of loads from beams to pillars is simulated by elastic z-axis turn-elements.

Arrangement of nodes allows to consider effect of non-centering load applying correctly by simulation of interaction between pillars and beams through spiked hooks (the beam loads the pillar along rigid wall).

Required value of beam-pillar connection torsional stiffness is approximately defined by the iterative method with considering experimental data. Convergence condition of the iterative process is agreement of the first buckling mode and corresponding critical load obtained numerically and experimentally.

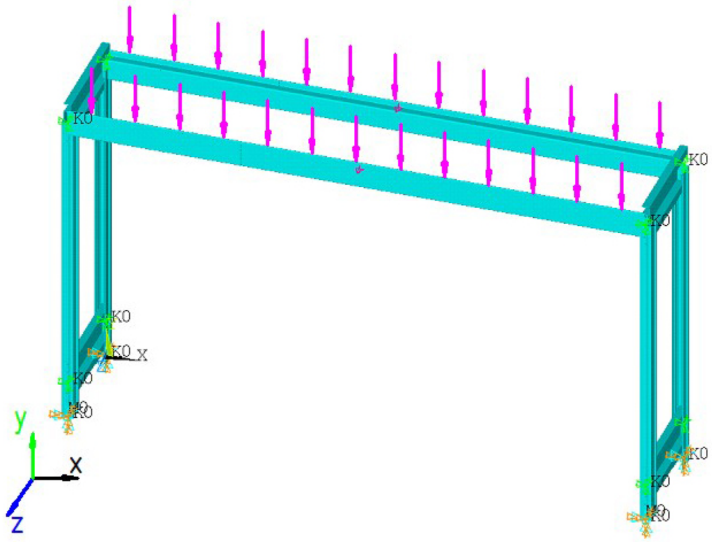

Figure 4. Computational model of rack for evaluation of beam-pillar connection torsional stiffness

In computation the following results are received. Torsional rigidity of beam-pillar connection with regard to the rotation about z-axis for simplified single-span girder rack model is 2820 N•m/rad. Corresponding buckling mode is represented in Fig. 5. Further tests are needed to reveal dependence torsional stiffness of beam-pillar connection and critical tier load from number of tiers. The “Eigen Buckling” type of analysis based on block Lanczos method was used in FE computations.

Figure 5. The first buckling mode (plane deformation in face plate). Critical load is 11.2 kN (experimental value). Torsional stiffness of beam-pillar connection with regard to the rotation about z-axis is 2820 N·m/rad

2.2. Experimental invesrigation of the connection stiffness

In order to confirm the reliability of the obtained results of the buckling critical load, in-place tests of real girder racks were carried out and additional analysis of elaborated rack models was done to imitate real structures as close as possible. Two versions of the rack design were tested: beam and combined (plate-beam) versions. The cross-sections and characteristics of the beams and pillars correspond to those used in the experiment with the simplified model. Geometric parameters of both versions are (see Figures 6a and b): single-section or three-section rack, height H = 2500 mm, length of span L = 1500 mm, height of the first loaded level h = 135 mm, width of the rack b = 600 mm.

Structure was loaded by a uniformly distributed load through the shelves made of the same rolled steel as the beams (see Figure 6c). Loads for all tiers were equal. Boxes filled with metal blanks of 1 kg weigh were used to simulate load on the shelves.

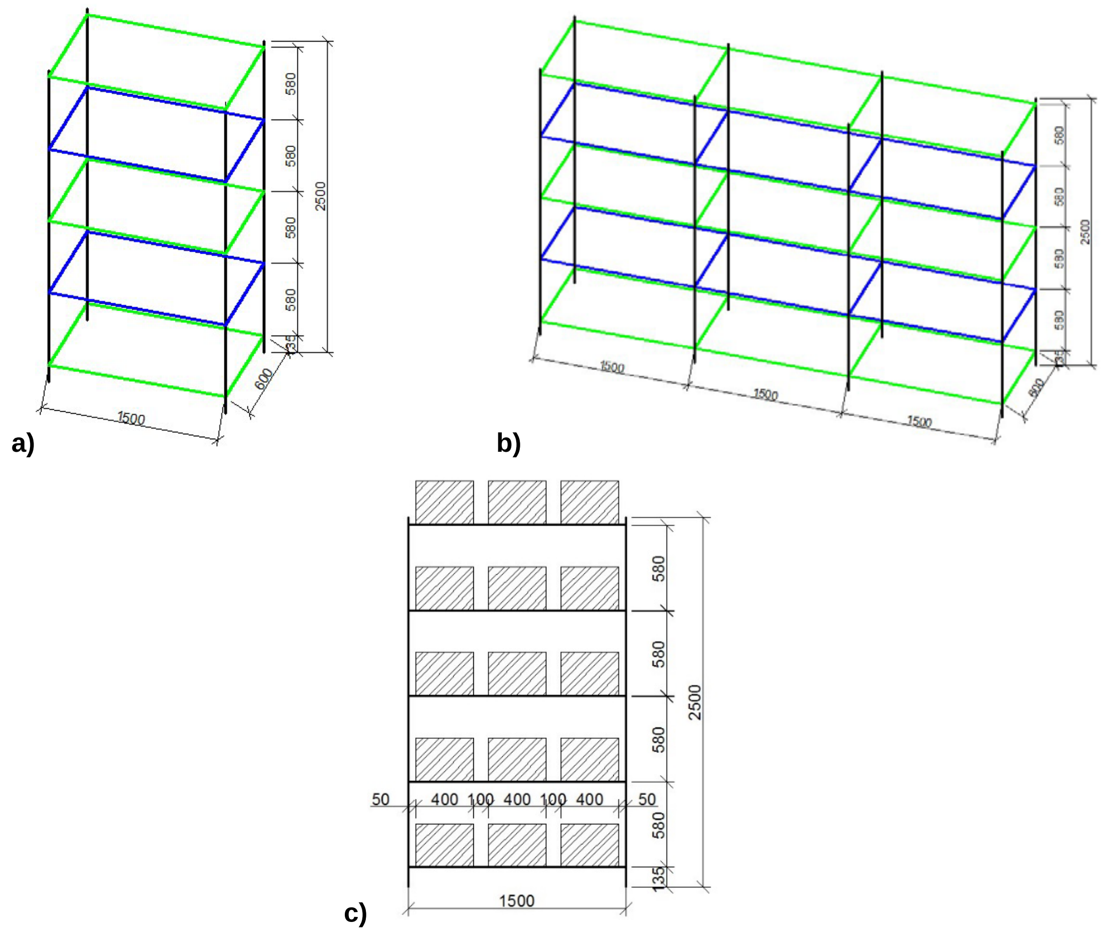

Figure 6. Geometry of a) one-section and b) three-section racks; c) quasi-uniformly distributed loading

Measurements of the displacements of the investigated thin-walled structure were made by several dial indicators. Dial indicators have an operating range from 0 to 80 mm, the measurement accuracy is 0.01 mm. In the case of a single-section rack, the indicators for a single-section girder rack are installed to determine the displacements of one of the pillars along the X and Z axes in its upper part. On the other hand, the indicators determine displacements of one of the two most stressed internal pillars when a three-section girder rack is tested.

Beam finite elements are used to make a FE model of the racks. This approach allows to take into account the shape of the beam cross-section profile in the most correct way. The horizontal beams of all tiers are loaded with equal uniformly distributed load.

Boundary conditions are the conditions of the elastic rotation connections (an elastic flap hinge) of the pillar's supporting nodes with the rigid base. The displacements U_x, U_y, U_z, and also the angular rotation R_y are fixed, the angular rotations R_x and R_z are defined by the elastic flap hinge stiffness.

The main difference between the beam and combined FE models is except that in the combined one the upper and lower tiers also consist of beam element (with a certain value of torsional stiffness of the beam-pillar connection), the other tiers are replaced by shelves (with zero value of the torsional stiffness of the beam-pillar connection).

3. Results and Discussion

The results of calculations of the critical buckling load according to the approach developed above for the considered versions of girder rack's design are presented in Table 1. In the same table, the results of computations for a three-section girder rack are given for comparison (Figure 7).

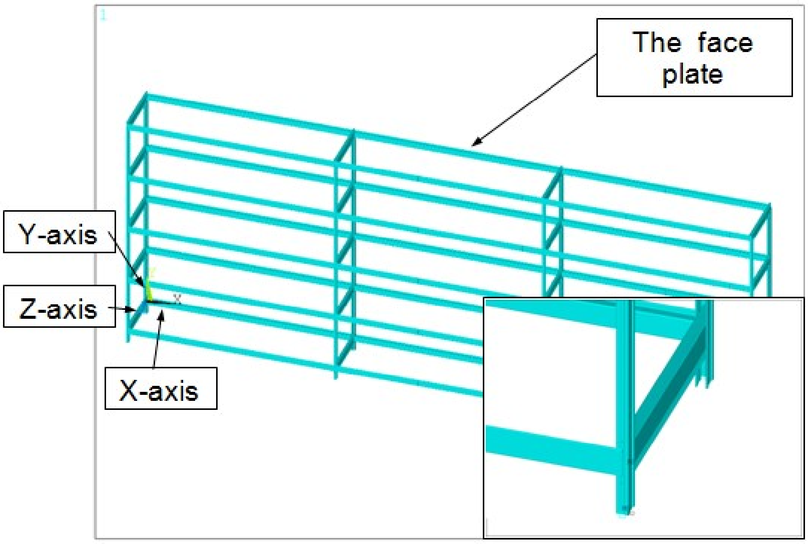

Figure 7. The FEM model of the three-section girder rack

Table 1. Estimated critical buckling load on the level (in kN)(H = 2500 mm, L = 1500 mm, h = 135 mm, b = 600 mm, 5 tiers)

	Beam model		**Combined model**	
	1 section	3 sections	1 section	3 sections
"Out of the face plane"	***3.36***	2.35	***3.49***	2.44
"In the face plane"	**6.29**	5.87	**2.46**	2.17

According to the results represented in Table.1 a single-section rack is more stable than a three-section one. This effect can be explained by the fact that the most stressed pillar's load of the three-section rack is 2 times greater than in the one-section. So if the beam-pillar node connection would be hinged, the critical load for "in the face-plane" buckling of the single-section rack would be 2 times greater than the critical load for the three-section one. But since this node has definite stiffness, the difference in values of critical loads is smaller than 2. Moreover, a certain ratio of the stiffness of the beam-pillar connection and the stiffness of the pillar itself allows the critical load for a three-section rack to be greater than one for the one-section rack. This situation is typical for pallet girder racks, where the stiffness of both the pillar itself and the beam-pillar connection are large enough.

Table 2. Experimental critical buckling load for combined girder rack (in kN)

	1 section			**3 sections**		
	1	2	3	1	2	3
"Out of the face plane"	2.49	2.49	2.48	1.74	1.74	1.75
"In the face plane"	1.75	1.77	1.76	1.56	1.54	1.57

As a result of the experiments, the following data were obtained. In the course of the experiment, three girder racks of each type were tested. The test results are shown in Table 2. The average critical load on the combined girder rack was 1.76 kN. It corresponds to the initial stage of buckling "in the face plane". The structure got an adjacent deflected equilibrium state. The rack did not fracture. The maximum load reached during the test was 2.49 kN. the test was stopped due to the fact that the load exceeded the expected calculated value of 2.46 kN. The rack under this load had stability mode deflected from the vertical one, the deviation was 60 mm. The ratio of the experimental and calculated critical load values is 1.76 / 2.46 = 0.717.

Table 3. Experimental critical buckling load for beam girder rack (in kN)

	1 section			**3 sections**		
	1	2	3	1	2	3
"Out of the face plane"	-	-	-	-	-	-
"In the face plane"	3.34	3.36	3.33	3.12	3.10	3.13

Testing beam girder racks had the same amount of sections that combined one. The average value of the critical load for the beam rack was 3.34 kN. This value corresponds to the initial stage of buckling “in the face plane”. The rack got a adjacent deflected equilibrium state and did not fracture. The maximum load reached during the test was 3.76 kN. The rack under this load had stability mode deflected from the vertical one, the deviation was 15 mm. A tendency to "out of the face plane" buckling isn’t observed. The value of the expected calculated critical "in the face plane" buckling load of 6.29 kN was not achieved. The ratio of the experimental and calculated critical load values is 3.34 / 6.29 = 0.532.

The difference between experimental and estimated critical buckling loads for the beam and combined racks is given in Table 4 and 5.

Table 4. Comparison of experimental and estimated critical buckling loads for beam girder rack

	1 section			3 sections		
	Estimated, N	Experimental, N	Difference, %	Estimated, N	Experimental, N	Difference, %
"Out of the face plane"	3.36	-	-	2.35	-	-
"In the face plane"	6.29	3.34	53.2	5.87	3.12	53.15

Table 5. Comparison of experimental and estimated critical buckling loads for combined girder rack

	1 section			3 sections		
	Estimated, N	Experimental ,N	Difference, %	Estimated, N	Experimental , N	Difference, %
"Out of the face plane"	3.49	2.49	71.3	2.44	1.74	71.3
"In the face plane"	2.46	1.76	71.7	2.17	1.56	71.9

Reduction of the experimental values of the calculated critical "in the face plane" buckling load in comparison with the calculated ones has two main causes.

The first reason is that the calculation of this work is based on value of the stiffness of the beam-pillar connection equal to 2820 N·m/rad. This value obtained in part 2.1 due to experimental data of the simplified model and corresponds to a load of 11.2 kN. In this assumption the calculation model (the simplified model in the beginning of the paper) has a deflected (not vertical) stability mode. Thus, the calculation is based on an overestimated value of this rigidity. The actual value of the equivalent stiffness of the beam-pillar connection is in the range 1330–2030 N·m/rad.

The second reason is more complicated. Increasing of the number of tiers of the rack without change of its height results in non-uniform distribution of the stiffness of the middle tiers located in the zone of kink of buckling pillars decreases. Thus equivalent stiffness of the middle tiers located in the zone of kink of buckling pillars decreases. An analysis of causes of this non-uniform distribution of the level of stiffness decrease and of the way to consider influence of these factors on calculating of the rack critical buckling loads would be the topic of future studies.

4. Conclusions

The results of the computational and experimental work can be formulated as follows:

1. The torsional stiffness of the beam-pillar connection for turning around the Z-axis is 1330-2030 N·m / rad.
2. The values of the critical buckling loads of the beam and combined models of girder racks (on the level) are determined by the stability condition. The values are given in tables 2-3.

To verify the results we recommend that:

1. The combined computational and experimental determination of the stiffness of the beam-pillar connection. For the most accurate results it is recommended to take into account the connecting plates in the joints, bolts, and contact between the elements on the surface.
2. The combined computational and experimental correction of the stiffness of the brace-pillar connection.
3. To evaluate the influence of the height of the first loaded level on the critical buckling loads for beam and combined girder racks of different types.

4. To investigate the causes of the non-uniform distribution of the stiffness of the beam-pillar connection. To consider influence of these factors on calculating of the rack critical buckling loads.

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Ilya Atavin,
+7(999)284-84-63; ilya_region51@mail.ru

Boris Melnikov,
+7(812)5526303; kafedra@ksm.spbstu.ru

Artem Semenov,
+7(905)2721188;Semenov.Artem@googlemail.com

Natalia Chernysheva,*
+7(981)7713236; chernat0000@mail.ru

Elena Yakovleva,
+7(812)552-63-03; helena47@mail.ru

Илья Вадимович Атавин,
+7(999)284-84-63;
эл. почта: ilya_region51@mail.ru

Борис Евгеньевич Мельников,
+7(812)552-63-03;
эл. почта: kafedra@ksm.spbstu.ru

Артем Семенович Семенов,
+7(905)272-11-88;
эл. почта: Semenov.Artem@googlemail.com

Наталия Вячеславовна Чернышева,*
+7(981)771-32-36;
эл. почта: chernat0000@mail.ru

Елена Леонидовна Яковлева,
+7(812)552-63-03;
эл. почта: helena47@mail.ru