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The stiffness of rigid joints of beam with hollow section column

Жесткость рамных узлов сопряжения ригеля с колонной коробчатого сечения

O.A. Tusnina,
A.I. Danilov,
National Research Moscow State University of Civil Engineering, Moscow, Russia

Канд. техн. наук, старший преподаватель О.А. Туснина,
канд. техн. наук, доцент А.И. Данилов,
Национальный исследовательский Московский государственный строительный университет, Москва, Россия

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Ключевые слова: жесткий узел; стальной каркас; коробчатое сечение; рамный каркас

Abstract. At present steel framework based buildings are prevalent in civil engineering. Application of hollow section columns have proven to be efficient in low-rise buildings. In some cases frame joints of beam to column connections are needed in this kind of building structures. This paper considers rigid joints of I-beam to hollow section column connections allowing for an elastic pliability. The results of pliability estimation of these joints are represented for various construction solutions. Dependences are obtained for the relation of the support moment in a beam to its corresponding value in absolutely stiff connection on the rigidity of the joint. The dependence diagrams of rigidity of the joint on the parameters of its elements are obtained.

Аннотация. В настоящее время широкое распространение в строительстве получают здания на основе стальных каркасов. В малоэтажных зданиях целесообразно применять колонны коробчатого сечения. В таких зданиях часто оказывается необходимым обеспечить работу каркаса по рамной или рамно-связевой схемам. В связи с этим возникает необходимость выполнения жестких узлов соединения ригеля с колонной. В данной статье рассматриваются жесткие узлы соединения ригеля двутаврового сечения с колонной из замкнутого квадратного профиля с учетом их податливости. Выполнена оценка жесткости соединения ригеля с колонной при различном конструктивном решении узла их сопряжения. Получены зависимости отношения опорного момента ригеля к его величине при абсолютно жестком закреплении опорного сечения ригеля от жесткости соединения. Построены графики зависимости жесткости соединения от некоторых параметров соединительных элементов, используемых в сопряжении.

Introduction

Buildings based on steel frameworks are widespread in civil engineering. Steel frameworks have some advantages in comparison to concrete buildings:

- lower loads on foundation are caused by low dead load of the support framework;
- high speed of mounting ;
- ability to mount structures in winter without any additional measures;
- environmental friendliness.

At present the output of low-rise buildings increases. Low-rise buildings are used as apartment buildings, cabins, dormitories etc.

Today the buildings based on light thin-walled structures are widespread. Many articles are devoted to the issue of designing such structures [1–3]

Thin-walled cold-formed profiles used in light steel thin-walled constructions (LSTC) have such geometry parameters (low thickness less than 4 mm) that allow the loss of its local stability. The effect of local stability loss must be taken into account when calculating LSTC structures [4–7].

The paper [8] represents the results of the experimental research that shows the necessity to consider geometric nonlinearity in calculation of thin-walled structures.

The paper [9] studies deformability of joints depending on the constructive solution experimentally and by finite element analysis. It determines the factors which affect the loss of local stability in joints.

Because of all the mentioned complications in calculating and design structures made from cold-formed thin-walled profiles (LSTC) the use of hot-rolled profiles is more preferable.

Steel frameworks built from hot-rolled and bent welded profiles are more efficient than frameworks built with the use of light thin-walled structures [10].

To provide the freedom of planning it is necessary to design the framework without braces so it is necessary to make rigid joints of beam to column connection.

The well-known construction solutions of rigid joints between I-beam and column of I-section and analysis guidelines are represented in papers [11–14].

Hollow sections as columns and I-beams are expedient to be applied in steel frameworks of low-rise buildings with quite small loads. Such a solution allows decreasing the consumption of material and simplifying mounting.

Some constructive solutions of the connection of beam with hollow section column are represented in the paper [15]. Such construction solutions provide higher stiffness of the joint and allow it to be applied in earthquake-prone regions.

The results of the researched work of rigid joint of beam to hollow section column connection are represented in the papers [16–18]. Also a constructive solution increasing the joint bearing capacity has been developed.

However, it is difficult to apply these constructive solutions in mass buildings because of a large scope of work.

According to the European norms for the analysis of joint stress-strain state 2D and 3D component methods are used.

The spread of the component method on the columns of hollow sections is represented in the article [19].

The papers [20–22] are dedicated to the development of 3D component method.

In the paper [23] the work of the joint of connection I-beam to hollow section column with the use of T-shaped elements is researched. T-shaped elements increase the thickness of the column in the points of effort transfer on the column from beam flanges. But a conclusion is made that such a structure of joint does not provide sufficient stiffness of the joint.

The problems of design and analysis of joints of I-beam connection with the column are also formulated in the articles [24–31].

The rigid joints of I-beam to hollow section column connections allowing for elastic pliability are considered in this paper.

Methods

At first the beam made of I22 on GOST 8239-89 with absolutely rigid supports loaded by uniformly distributed load was considered (Figure 1).

Finite element analysis was made with the use of program MSC.NASTRAN. The beam was modeled with the use of elastic quadrilateral PLATE elements. The finite elements were located on the middle surface of the profile.

To achieve satisfactory convergence, the finite element mesh with 10 finite elements on the height of I-beam was assigned. The size of the plate finite element was adopted based on the results of the test analyses carried out for a similar problem [32].

The span of the beam was 6 m. Half of the beam was modeled with appropriate constraints on the cross section on the axis of symmetry. The constraints of axial displacement (translation along global axis X) and those of rotation around Y-axis were set on the nodes of flanges and web of the beam in the cross section in the middle of the beam span.

The uniformly distributed load on beam q=10 kN/m. The distributed load was brought to the concentrated loads which were applied in the nodes of the flange to the web connection line.

Static linear analysis was carried out. Geometrical and physical nonlinearity was not taken into account.

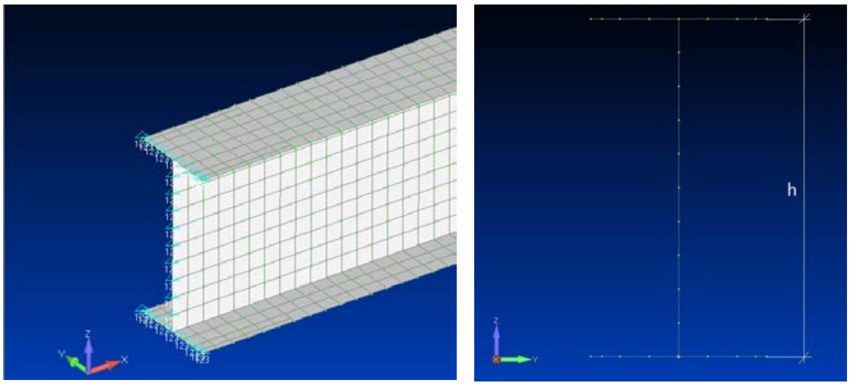

Figure 1. Finite-element model of I-beam

The diagram of bending moments in a beam with absolutely rigid supports is represented in Figure 2.

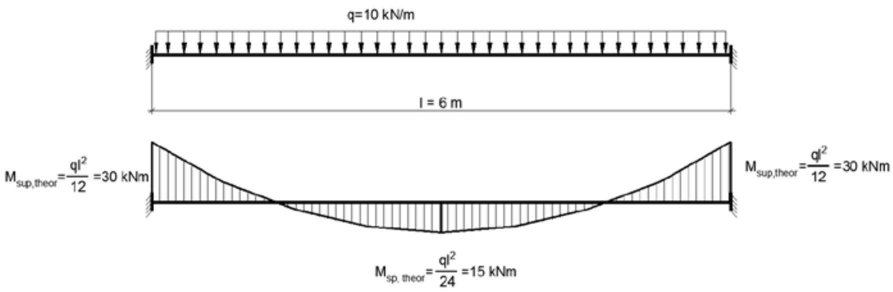

Figure 2. The diagram of bending moments in a beam with absolutely rigid supports

But it is impossible to make the beam-column joint absolutely rigid and fix the support cross-section of the beam from rotation. Flexibility of the joint will be determined by deformations occurring in connective elements, columns, beam. So the support cross-section of the beam will be rotated by some angel φ.

Flexibility of the joint influences the effort distribution between the beam and the column, especially in bending moment distribution along the beam.

Stiffness of the joint *C* is determined as a ratio of the bending moment acting on the support to the rotation angel of the support cross-section of the beam:

$$C = \frac{M}{\varphi}, \tag{1}$$

where *M* – the bending moment acting on the support;

φ – rotation angel of the support cross-section of the beam.

The finite element analysis was conducted to estimate the influence of the connection stiffness on the bending moment acting on the support. The numerical analysis of bending moment distribution in the beam considered above was made with the different stiffness of support joint *C*.

Flexibility of the joint was modeled by the use of spring elements in the nodes of the beam flange in the support cross-section (Figure 3). Elements spring with different axial stiffness *K* prevents free movement of beam flanges and, thereby, prevents free rotation of its support cross-section.

Axial stiffness of elements *K*, related to the stiffness of connection *C* is as follows:

$$K = \frac{2C}{h^2}, \tag{2}$$

where h – height of the beam (in this case – distance between nodes on the middle surfaces of the flanges, Figure 1).

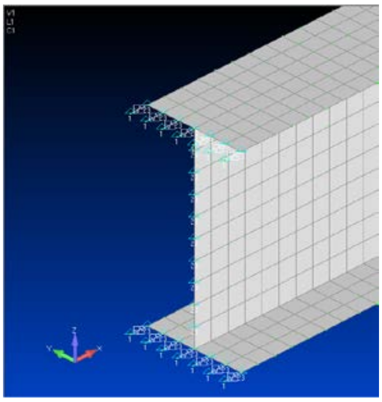

Figure 3. Flexible fixation of the beam on the support

Normal stresses in the cross-sections of the beam in the considered loading conditions were determined by bending moment M_y. Normal stresses in the middle surface of the beam cross-section in the 1 and 2 points in the middle of the beam span were determined in the result of numerical analysis. The value of the bending moment acting in the middle of the span can be determined by solving the equation (3):

$$\sigma_1 = \sigma_2 = \frac{M_{sp}}{W}, \tag{3}$$

where σ_1 и σ_2 – normal stresses acting in points 1 and 2 in the middle of the beam span (Figure 4).

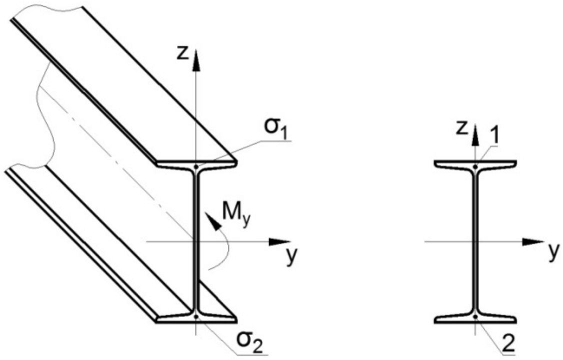

Figure 4. Cross-section of the beam with considered points

Then by using the value of moment in the middle of beam span M_{sp} we can determine the moment on beam support $M_{\sup}$:

$$M_{\sup} = M_{sp} - \frac{ql^2}{8} \tag{4}$$

For each value of stiffness the ratio of support moment $M_{\sup}$ to the theoretical moment on the support with absolutely rigid fixation $M_{\sup,theor}$:

$$k = \frac{M_{\sup}}{M_{\sup,theor}} \tag{5}$$

In Figure 5 the graph of the dependence of coefficient k on the stiffness of joint C. The data represented on the graph are also represented in Table 1.

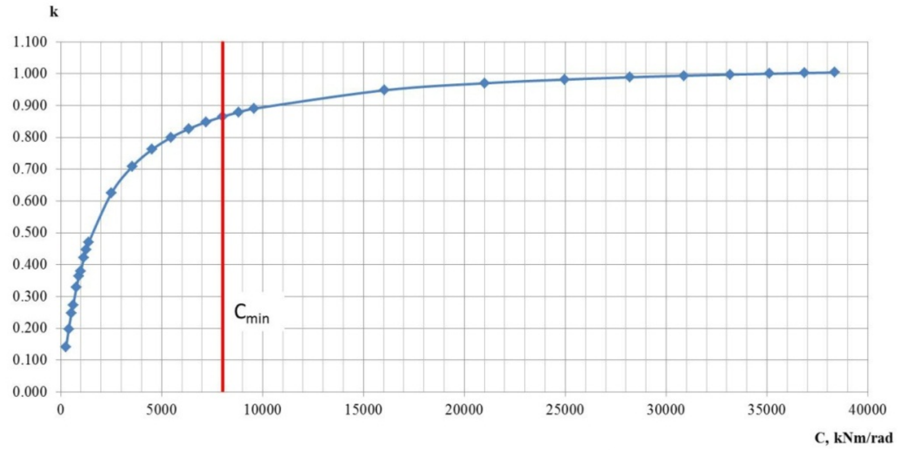

Figure 5. The graph of dependence of coefficient k on the stiffness of joint C

Table 1. The values of coefficient k of different stiffness C

C, kNm/rad	*k*	*C*, kNm/rad	*k*
264.90	0.141	6345.32	0.827
397.51	0.198	7196.56	0.848
527.78	0.248	8015.61	0.865
612.06	0.272	8804.42	0.879
781.66	0.330	9568.38	0.890
906.72	0.364	16035.40	0.948
989.34	0.379	20998.93	0.970
1150.06	0.422	24959.76	0.982
1270.63	0.448	28201.79	0.989
1388.65	0.471	30897.60	0.994
2511.90	0.625	33173.30	0.997
3551.21	0.709	35131.57	1.000
4529.17	0.762	36845.07	1.000
5457.17	0.799	38365.86	1.000

Coefficient k tends to 1 when the stiffness of connection increases. The joint is suggested for consideration as rigid if the difference between theoretical moment and actual moment is not bigger than 10…12 % (coefficient k ≥ 0.88).

On the graphs the minimum value of stiffness C_{min} = 8000 kNm/rad is shown by a vertical red line. With this stiffness the value of k = 0.88.

The stiffness of the connection depends on its constructive solution. To estimate the stiffness of the joint of the beam with the columns of hollow sections numerical analysis of joints referring to different constructive solutions was made.

The joint of the beam (I22, GOST 8239–89) with the column (bent-welded tube 120x6, GOST 30245–2003) connection was considered.

The span of the beam was 6 m. The uniformly distributed load along the beam was 10 kN/m.

Three types of joints were considered:

1 – rigid joint with direct adjacently overlays on belts bolt to the column wall (Figure 6);
2 – rigid joint with the transfer to the column shearing force through angels attached to it (Figure 7);
3 – hinge joint on pad (Figure 8)

A finite element model of the considered sites was compiled using MSC.NASTRAN software package.

Half of the beam (3 m) was modeled with the corresponding constraints on the cross section on the symmetry axis. The load was applied in the nodes of the beam wall.

The influence of column deformations on the stiffness of the joint is not considered and simulated in the analysis model of the 0.8 m long column, with fixed ends of all movements.

The patches were connected with the column through rigid elements with the union movement relevant nodes overlapping the beam /column nodes (Figure 9).

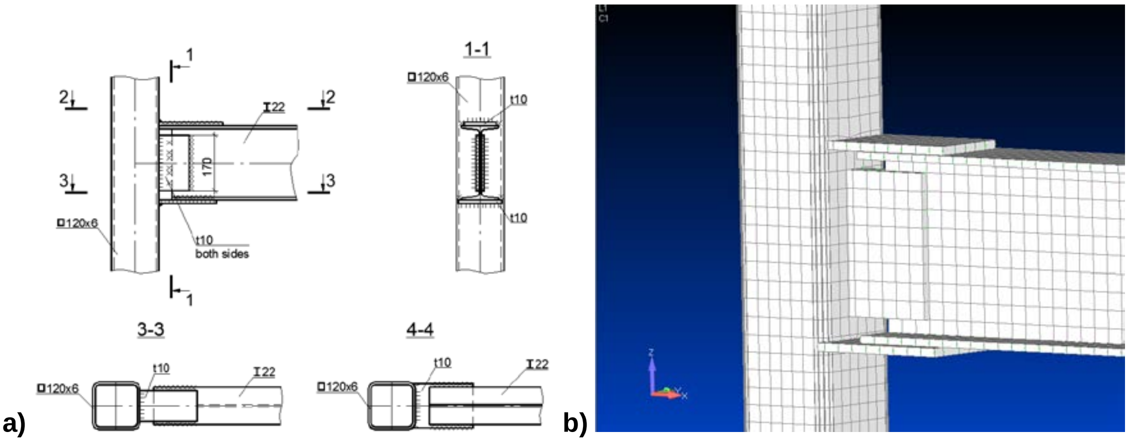

Figure 6. Rigid joint (1 type): a – constructive solution; b – finite-element model

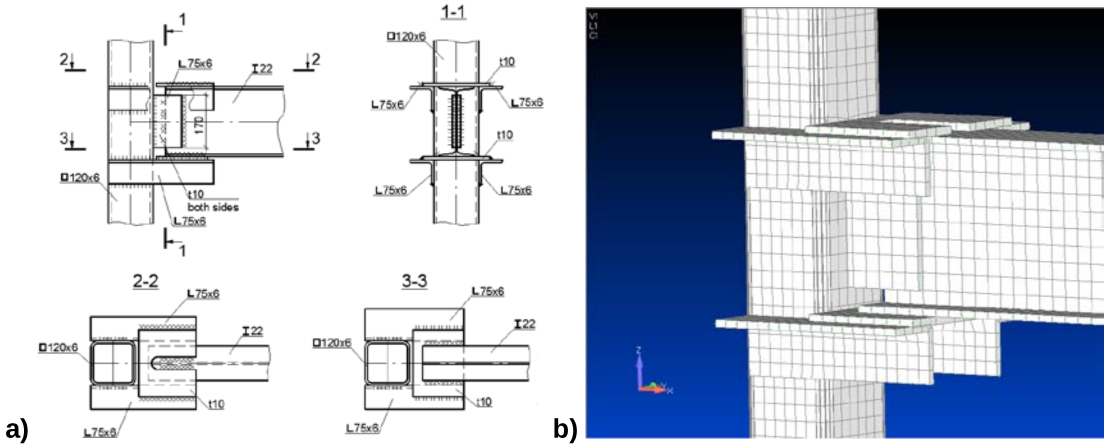

Figure 7. Rigid joint (2 type): a – constructive solution; b – finite-element model

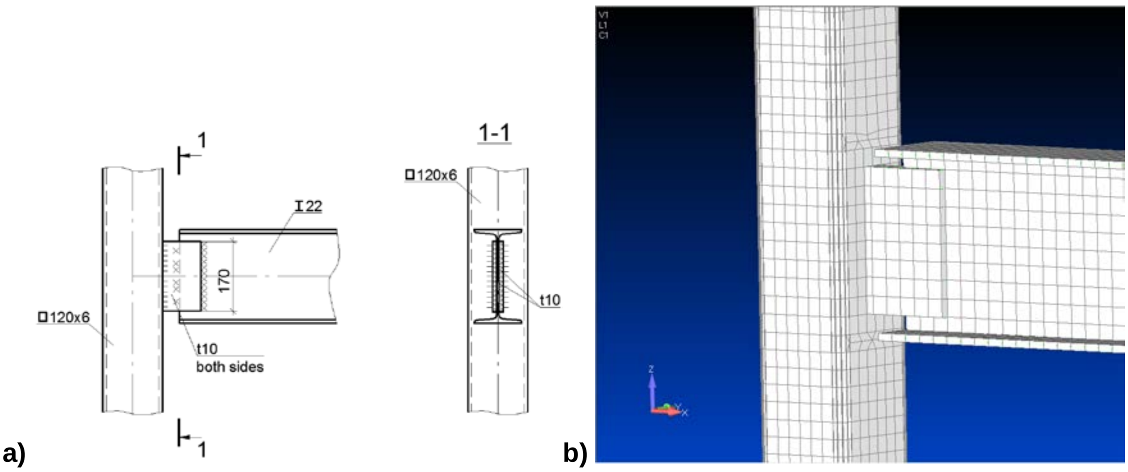

Figure 8. Hinged joint: a – constructive solution; b – finite-element model

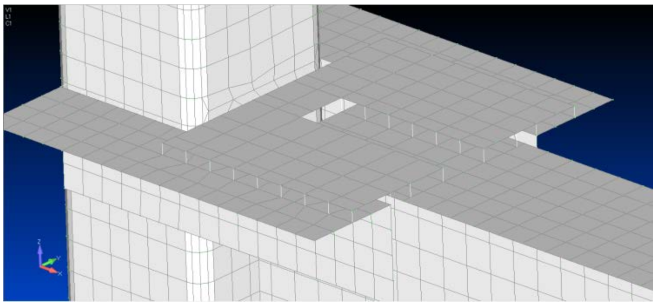

Figure 9. Connection of the nodes on the patch with a beam and angels

Results

The resulting numerical calculation of the stiffness values of the compound *C*, the angle of rotation of the support section crossbar φ_{sup}, bending moments on the support and in the span M_{sup} and M_{sp} respectively, and the coefficient *k* are given in Table 2.

Table 2 also shows the theoretical solutions for the beam absolutely rigidly fixed on the rotation supporting sections and the beam supported in a simple way.

Table 2. The values of the coefficient k with different values of stiffness C

Type of analysis	**Joint**	**Stiffness of joint *C*, kNm/rad**	**Rotational angle of support cross-section φ_{sup}, rad**	**Support moment M_{sup}, kNm**	$k = \dfrac{M_{sup}}{M_{sup,theor}}$	**Span moment M_{sp}, kNm**
Numerical analysis	Hinged	419.3	0.0146	6.12	0.204	38.88
	Rigid 1 type	4116.5	0.00544	22.39	0.746	22.61
	Rigid 2 type	9602.89	0.00279	26.79	0.893	18.21
Theory	Hinged	-	0.0171	-	0	45.00
	Rigid	-	0	30.00	1.000	15.00

The angle of rotation of the support section with the patches on the hinge joint linings on the wall (Figure 8) is about 85 % of the theoretical one with absolutely free rotation.

In this constructive decision the joint has certain stiffness and provides a reference point, amounting to about 0.2 by the complete theoretical pinching. The decrease in the time span compared to the theoretical one with simple support girder is about 13.6 %.

Thus, the joint, which is traditionally perceived as a hinge, can be considered as compliant, in a similar way as it was done in [33] in relation to the connection node of the I-beam with a column on rails and angles.

The stiffness of the type 1 rigid joint (Figure 6) is small (less than 8000 kNm/rad) and not sufficiently hard to ensure assembly work (acting on the support point is 22.39 kNm compared to the theoretical 30 kNm, and the ratio *k* is less than 0.88 equaling to about 0.75), so the use of such units in the rack is not recommended.

The stiffness of the type 2 rigid joint (Figure 7) above proves that such nodes may be regarded as rigid. However, the magnitude of rigidity is close to the minimum, amounting to about 9600 kNm/rad, and the coefficient k = 0.893.

To assess the influence on the stiffness of the geometric parameters of the node connection elements the node was identified based on the stiffness of the following parameters:

– for the assembly of direct overlap adjacent to the column wall (joint of type 1) – the thickness of the casing wall, the thickness of lining the shelves and the height of the wall lining;
– for the assembly with transfer of forces through the angle (joint of type 2) – the thickness of the tower wall thickness over the height of the pads and the wall.

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The results are shown graphically in Figures 10 and 11, and in a form of a table – Table 3, 4, 5.

The graphs and tables show that by increasing the thickness of the column by 5.5 mm to 8 mm, the rigidity of the type 1 joint increased by 56 %, the type 2 joint – by 18 %.

Increasing the stiffness of the type 1 joint by 56% however does not allow for sufficient assembly stiffness and an 8 mm thick unit cannot be considered as hard enough even if a column is used.

When the lining thickness increases from 6 mm to 14 mm the stiffness of the type 1 unit goes up by 17 %, while if the thickness of the angle increases from 5 mm to 9 mm the stiffness of the type 2 node enlarges by 20 %.

They are also plotted given the coefficient *k* from the parameters (Figure 12).

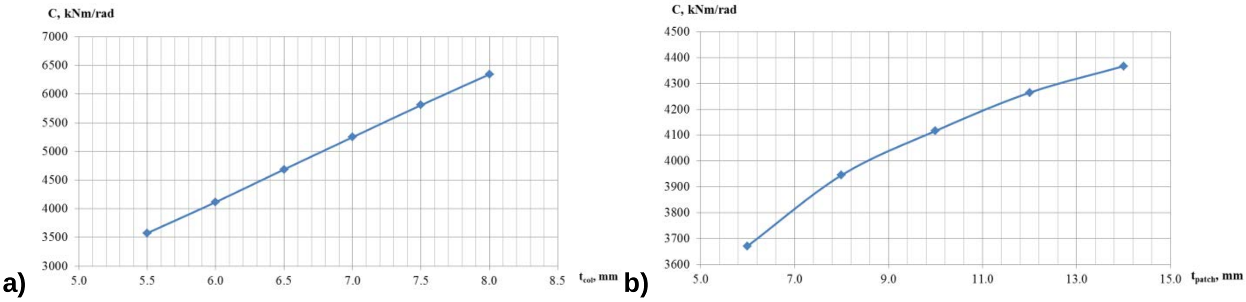

Figure 10. Graphs of stiffness dependence(1 type) on
a – thickness of column wall; b – thickness of patch on flanges

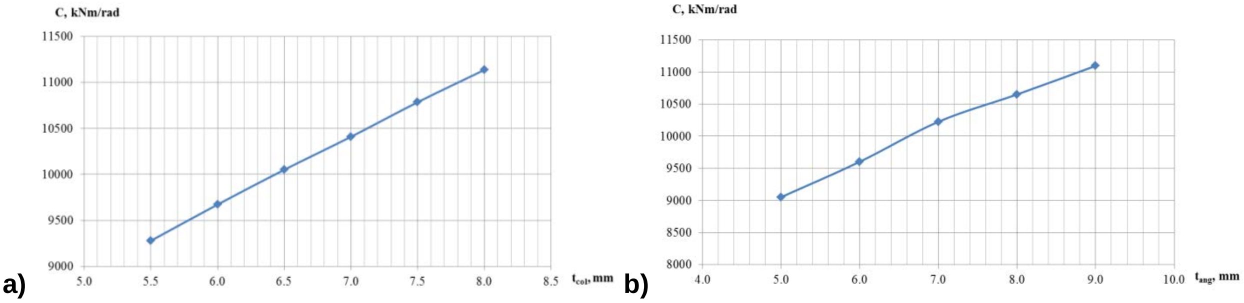

Figure 11. Graphs of stiffness dependence (2 type) on
a – thickness of column wall; b – thickness of angels

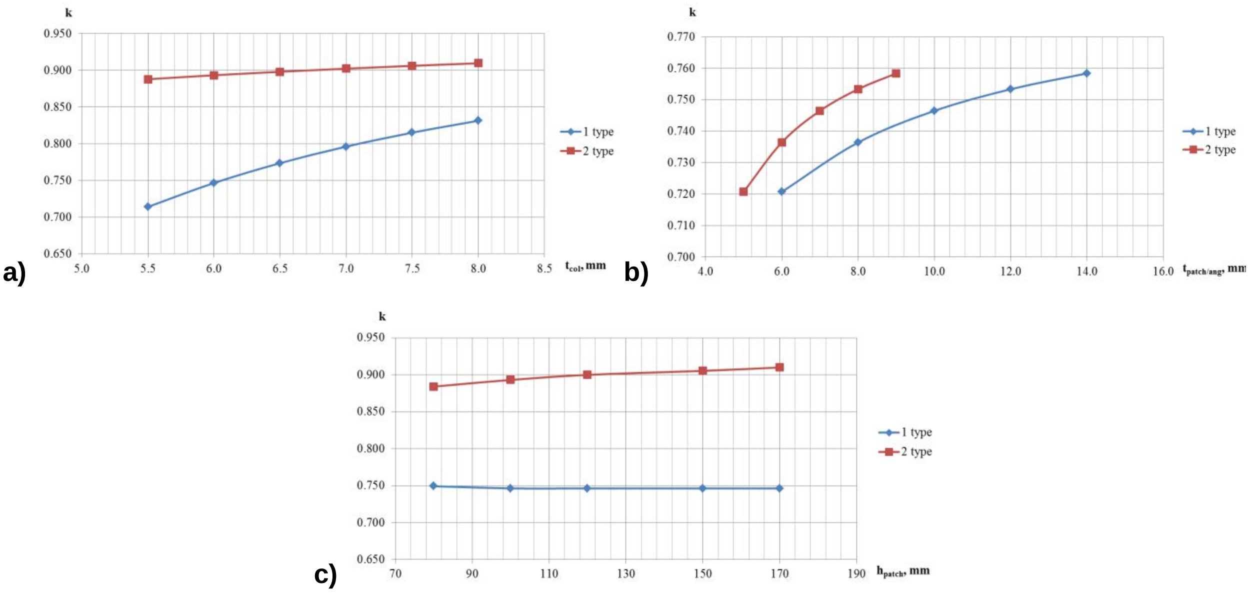

Figure 12. Graphs of coefficient *k* (1 type) dependence on: a – thickness of column wall;
b – thickness of patches (1 type)/thickness of angles (2 type);
c- height of the patch on beam wall

Table 3. The value of coefficient k with the different stiffness of joint C

Joint	Thickness of column t_{col}, mm	C, kNm/rad	φ_{sup}, rad	M_{sup}, kNm	k
1 type	5.5	3575.4	0.00599	21.41	0.714
	6	4116.5	0.00544	22.39	0.746
	6.5	4687.4	0.00495	23.20	0.773
	7	5248.1	0.00455	23.87	0.796
	7.5	5807.3	0.00421	24.44	0.815
	8	6344.6	0.00393	24.93	0.831
2 type	5.5	9278.6	0.00287	26.62	0.888
	6	9672.2	0.00277	26.79	0.893
	6.5	10050.1	0.00268	26.93	0.898
	7	10408.1	0.00260	27.06	0.902
	7.5	10784.4	0.00252	27.17	0.906
	8	11134.9	0.00245	27.28	0.909

Table 4. The value of coefficient k with different stiffness of joint C

Joint	Thickness of patch/angle $t_{patch/ang}$, mm	C, kNm/rad	φ_{sup}, rad	M_{sup}, kNm	k
1 type	6	3670.8	0.00589	21.62	0.721
	8	3945.1	0.00560	22.09	0.736
	10	4116.5	0.00544	22.39	0.746
	12	4264.2	0.00530	22.60	0.753
	14	4366.8	0.00521	22.75	0.758
2 type	5	9052.0	0.00293	26.52	0.884
	6	9602.9	0.00279	26.79	0.893
	7	10226.1	0.00264	26.99	0.900
	8	10651.3	0.00255	27.16	0.905
	9	11096.2	0.00246	27.29	0.910

Table 5. The value of coefficient k with different stiffness of joint C

Joint	Height of the patch on the wall h_{patch}, mm	C, kNm/rad	φ_{sup}, rad	M_{sup}, kNm	k
1 type	80	3937.3	0.00571	22.48	0.749
	100	3999.3	0.00560	22.39	0.747
	120	4049.3	0.00553	22.39	0.746
	150	4101.5	0.00546	22.39	0.746
	170	4116.5	0.00544	22.39	0.746
2 type	80	9646.7	0.00277	26.72	0.891
	100	9583.3	0.00279	26.73	0.891
	120	9588.8	0.00279	26.75	0.892
	150	9667.0	0.00277	26.77	0.893
	170	9602.9	0.00279	26.79	0.893

As it can be seen from Figure 12 and Tables 3–5 the *k* factor increases mostly if the column thickness changed from 5.5 to 8 mm in the type 1 node – 15 %, in type 2 node the change is insignificant and is about 2.4 %.

If the thickness of the lining on the shelves grows from 6 to 14 mm (joint of type 1) the increase in k is 5 %, while the thickness of parts increases from 5 to 9 mm (type 2 joint – almost 3 %).

Change in the height of pads on the walls does not affect the coefficient *k*, bringing it to a value of about 0.4 %.

Thus, it can be concluded that change of these parameters, and that of connecting elements attached to the joint do not increase significantly the rigidity of the assembly.

In order to implement the constructive solution it is required to obtain a hard knot.

Conclusions

1. Joints of beam connection to the column conventionally used as a hinge, in some cases can be considered as elastically yielding since they provide reduction in the time span by 13.6 %, due to the occurrence of time on the anvil.

2. The joints of the beam with column connection on a patch, directly adjacent to the wall of the column cannot be considered as rigid (the proportion of the current support at the moment is less than the theoretical 0.83) and cannot be recommended for the use in the frame.

3. Joints with angles for transmission of forces in the belts crossbar on the column can be considered as rigid. However, it should be considered that the rigidity of such a compound is close to the minimum allowed for rigid nodes and the reference point is about 0.9 from its theoretical value in an absolutely rigid connection.

4. The need to develop a simple model for implementation of design solutions, providing a fairly rigid bolt connection with the column box section, or the development of guidelines for calculation and design of framework given the compliance of units depending on the constructive solution.

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Olga Tusnina,
+7(910)4761677; lazoltus@mail.ru

Alexander Danilov,
+79151834136; alenk070911@yandex.ru

Ольга Александровна Туснина,
+7(910)4761677; эл. почта: lazoltus@mail.ru

Александр Иванович Данилов,
+79151834136; эл. почта: alenk070911@yandex.ru