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On the Determination the Effective Length of Columns in Steel Multi-Storey Frames Whereas the Pliability of Semi-Rigid Beam-To-Column Connections

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Abstract. The main condition for the structures reliability of buildings and constructions is the meeting the requirements for strength, rigidity and stability of both separate structures and their parts, and the system as a whole. The analysis of stability is of great important for compressed and compressed-bended columns. When analyzing the stability of a separate section of a compressed column located between the beam connections, the correct determination of its effective length is essential. This article is devoted to the issue of determining the effective length in unconventional cases not specified by modern standards. This article is devoted to the issue of determining the effective length in unconventional cases not specified by modern standards. The article describes a general method for determining the effective length of a compressed column of a frame using the instance of an exterior column of the transverse frame of a building. In addition, the influence of the pliability of the beam-to-column connection adjacent to this column on its effective length is considered in the paper.

INTRODUCTION

The practice of world construction shows the effectiveness of using a metal framework for buildings of various purposes. In the USSR and modern Russia, for various reasons, the metal framework is mainly in industrial, warehouse buildings and transport infrastructure facilities. However, there are currently positive trends in the appliance of a steel frame for multi-storey residential and public buildings. It is common to divide the framework of multi-storey buildings into rigid, braced and combined one. Despite the fact that some studies [1] show the material consumption of the braced framework is greater than that of the combined and rigid one, last type of framework has advantage in increased stiffness and stability. Also there are a number of design features associated with the usage of braced and combined frameworks, when it is necessary to fit into the internal space elements of ties along the perimeter of the building, including panoramic glazing, to hide the ties in the wall structures, thereby reducing the usable area, etc. Therefore, from the point of view of internal arrangements of space, the rigid frame has more space flexibility than other types of frame. In the following, using the term frame, we will look at namely rigid frame.

When designing any of the abovementioned types of frame, the specialist solve the issue of the strength, stiffness and stability of the building. Stability analysis is generally most critical for compressed and compressed-bending columns. The correctly assigned design scheme for the column and the effective length determined in accordance with the scheme are of great importance in this issue. In order to determine the effective length in and out of the frame plane, it is mandatory to apply simplified design scheme contained in the regulations, or one taken into account the actual fixation of the ends of the columns [2]. However, not all real designs can be carried out with simplified schemes, which is often difficult for specialists, as both the existence of intermediate supports and their pliability must be taken into consideration in this case [3].

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This paper provides the general algorithm for determining the effective lengths of the column within the frame, taking into account the intermediate supports, and explores the relationship between support flexibility and effective length. The purpose of this paper is to determine the effective length of the column taking into account the intermediate supports and their flexibility. Here and beyond in the discussion of flexibility, pliable restraint is meant, i.e. beam-to-column joints is not taken to be absolutely rigid or absolutely hinged, but capable of absorbing a certain amount of bending moments [4, 5]. The problem of investigating the actual behaviour of nodes with regard to pliability is relevant [5-10], because it depends on the accuracy of the obtained results and, as a consequence, the reliability of the structures and their economic efficiency. Within the research, various numerical values of support stiffness are considered in accordance with existing studies of nodal joint pliability [5].

METHOD OF ANALYSIS

Stability of compressed shank is an ability to maintain the original form of elastic equilibrium given by structural engineer under the action of external forces or to restore it when these forces are removed. Thus, when the compressed shank loses its stability, it enters a new state of equilibrium in which no further operation is possible.

The most effective method for assessing the stability of framework columns as shank systems is the displacement method [11, 12]. As is well known, in accordance with the displacement method, all rigid non-supported frame assemblies are subject to restraints that prevent rotation of the nodes and linear connections that prevent linear displacements of nodes. The equation of stability in the form of the displacement method is shown:

$$\det(\upsilon) = \begin{vmatrix} r_{11} & r_{12} & \dots & r_{1n} \\ r_{21} & r_{22} & \dots & r_{2n} \\ \dots & \dots & \dots & \dots \\ r_{n1} & r_{n2} & \dots & r_{nn} \end{vmatrix} = 0 \qquad (1)$$

where ν = longitudinal force factor, rik = efforts arising in restraint i from single movements in the direction of restraint k. The force at the ends of compressed rods rik is determined by means of special tables taking into account the functions of N.V. Kornoukhov [13] and taking into account the pliability of the beam-to-column connections. Pliability of joints is taken into account by introducing an additional value of stiffness of the adjoining semi-rigid node Ci into the equation of balancing moments on supports. as it is shown at figure 1. In case of elastic restraints, the angle of rotation of the support section φ is assumed to be proportional to the bending moment. Rotation pliability is defined as the ratio of the angle of rotation of the support from a single moment. The value inverse of pliability is called stiffness Ci and is defined as the amount of force required to create a unit displacement.

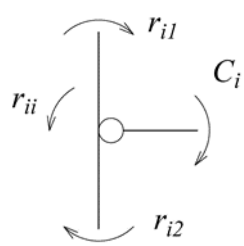

FIGURE 1. Node equilibrium when restrained by the movement method.

Computational Algorithm

The following steps must be taken to analyze the stability of the frame column. First of all, it is necessary to determine the existing longitudinal compressive forces occurring in the investigated element by means of static calculation. The longitudinal force factors for each column element are then calculated as follows

$$\upsilon_i = l_i \sqrt{\frac{N_i}{EI_i}} \quad (2)$$

where li, Ni, EIi = geometric length, longitudinal force and stiffness of the column element. The coefficients are expressed through a common parameter for ease of decision.

$$\upsilon_i = \alpha_i \upsilon \quad (3)$$

The equilibrium of the restrained nodes in unit displacements is then assessed the reactions to the supports rik. are determined. Afterwards, a stability equation (1) is formed, which is solved numerically by the selection method in relation to longitudinal force factor ν. Knowing ν, it is easy to find the effective lengths factor of compressed racks:

$$\mu_i = \frac{\pi}{\upsilon_i} \quad (4)$$

The effective length of the stability analysis element is determined as follows:

$$l_{ef,i} = \mu_i l_i \quad (5)$$

NUMERICAL APPLICATION

Description of the Structure

For the purpose or the present research, a three-span frame was selected as numerical example of the method and research of impact of beam- to-column connection stiffness as shown at figure 2. Only a planar task is considered as part of this paper. Frame step is taken 6 m. The plane where columns and beams have maximum stiffening characteristics are the same as the frame one. The cross-sections of the elements are selected according to the current regulation [14] and presented in the table 1. The exterior column of the frame was chosen as the study object. The nodes of the framework beams adjoining the middle columns are absolutely rigid, which ensures the rigidity and geometric invariability of the system. The nodes of the beam attachment to the exterior columns are assumed to be hinged, but this paper considers several values of rigidity of such nodes from 0 (absolutely hinged) to values determined experimentally (see 3.3). Thus, when the framework beams are loaded uniformly, the bending moments arising from the nodes of the beams' connection to the middle columns will be compensated by each other, and when the beams are connected to the exterior columns, no significant uncompensated bending moments will occur, which simplifies the design process. Figure 3 shows a design scheme to determine the design lengths of the exterior column.

TABLE 1. Frame sections.

	Columns	Beams
Floor 4	40K1	35Sh2
Floor 3	40K1	40Sh2
Floor 2	40K1	40Sh2
Floor 1	40K1	40Sh2

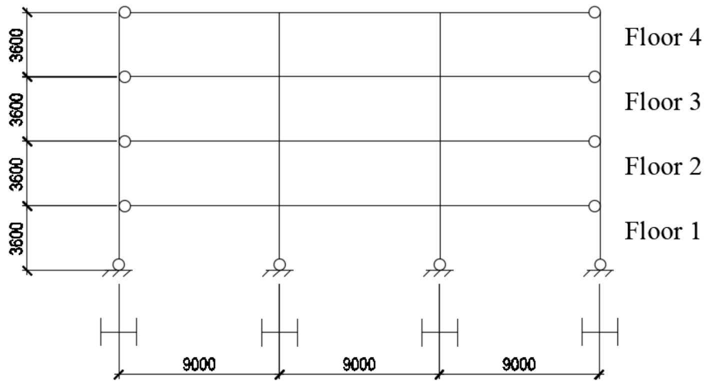

FIGURE 2. Geometry of the frame under consideration.

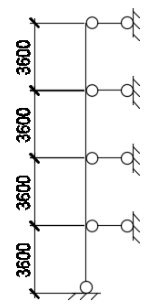

FIGURE 3. Design sheme of the exterior column.

Loads

The frame is subject to dead loads (its own weight, floor covering weights and roof covering weights), long-term loads as well as live loads and snow loads corresponding to the third snow area. Loads per length are presented in Table 2.

TABLE 2. Loads under consideration.

	Dead loads, kg/m^2	**Long-term loads, kg/m^2**	**Live loads, kg/m^2**	**Snow load, kg/m^2**
Floor 4	210	130	-	210
Floor 3	210	240	360	-
Floor 2	210	240	360	-
Floor 1	210	240	360	-

Stiffness of Semi-Rigid Connections

The current stiffness values of certain types of nodal connections can be obtained experimentally or with a verified numerical model [5-10]. Such studies are currently of great interest to scientists and practitioners. As part of this work, we will take the node of the beam-to-column connection with double web angles, shown in Figure 4, and take the data provided in the paper [5].

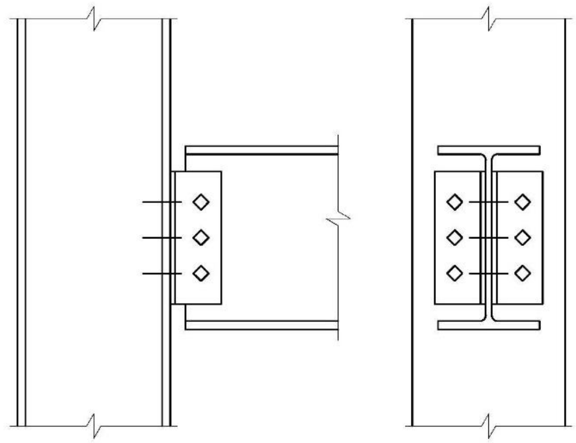

FIGURE 4. Double web angles.

Generally, the stiffness of the nodal connection is determined by the following formula:

$$C_i = \frac{M_i}{\varphi_i} \tag{6}$$

As part of the study, summarise the data obtained in work [5] and accept that the moment that may occur in the connection at figure 4 is up to 8% of the bending moment in the beam. It is assumed that stiffness Ci≈1400 kNm/rad. may be observed for the loads considered in this paper. Transfer the stiffness of the unit through the stiffness of the column for easy calculation

$$C_i = C_i \cdot \frac{\frac{EI}{l}}{\frac{EI}{l}} = \frac{1400 \cdot 10^3 \cdot 3,6}{2,06 \cdot 10^{11} \cdot 56145,3 \cdot 10^{-8}} \cdot \frac{EI}{l} = 0,0436 \frac{EI}{l} \tag{7}$$

Since the aim of the numerical experiment is to investigate the effect of pliability, we will also set the intermediate values of stiffness shown in Table 3.

TABLE 3. Stiffness values taken into consideration.

Proportion of stiffness	**Value,** EI/l
0	0
0.2	0.0087
0.4	0.0174
0.6	0.0262
0.8	0.0349
1	0.0436

Selection of The Parameters of the Algorithm

Figure 5 shows the calculation scheme of the exterior column for the displacement method. The forces transmitted to the column from the slabs and coverings beams, longitudinal forces at the areas of the column and longitudinal force factor are expressed through general parameters for convenient calculation. These parameters determination is shown in Table 4.

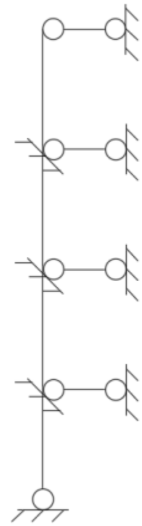

FIGURE 5. Sheme of the exterior column for displacement method.

TABLE 4. Determination initial parameters.

Column	Transverse force of upper floor beam, tn	Equivalent transverse force of upper floor beam	Equivalent longitudinal force	Equivalent longitudinal force factor, $\nu = \sqrt{\frac{N}{EI}}$
3-4	11.88	1N	1N	1
2-3	17.50	1.47N	2.47N	1.572
1-2	17.50	1.47N	3.95N	1.986
0-1	17.50	1.47N	5.42N	2.328

Having defined the forces arising on supports from their single displacements using the displacement method, we obtain the following stability equation

$$\det(\upsilon) = \begin{vmatrix} 3i\varphi_1(\nu_{3-4}) + 4i\varphi_2(\nu_{2-3}) + C_1 & 2i\varphi_3(\nu_{1-2}) & 0 \\ 2i\varphi_3(\nu_{2-3}) & 4i\varphi_2(\nu_{2-3}) + 4i\varphi_2(\nu_{1-2}) + C_2 & 2i\varphi_3(\nu_{1-2}) \\ 0 & 2i\varphi_3(\nu_{2-3}) & 4i\varphi_2(\nu_{1-2}) + 3i\varphi_1(\nu_{0-1}) + C_3 \end{vmatrix} = 0 \quad (8)$$

Results

Solving the equations of stability with respect to the coefficient of influence of longitudinal forces ν for each value of rigidity of the node of the transom abutment to the column, we obtain the results presented in tables 5, 6.

TABLE 5. Values of longitudinal force factor depending on the stiffness.

	Proportion of stiffnes					
	0	0.2	0.4	0.6	0.8	1
ν	1.532859	1.533398	1.533936	1.534473	1.535009	1.535544

TABLE 6. Values of effective lengths factor μ depending on the stiffness.

Column	Proportion of stiffnes					
	0	0.2	0.4	0.6	0.8	1
3-4	2.049499	2.048778	2.04806	2.047343	2.047343	2.045915
2-3	1.303346	1.302887	1.30243	1.301975	1.301975	1.301067
1-2	1.031809	1.031446	1.031084	1.030723	1.030723	1.030005
0-1	0.880483	0.880173	0.879865	0.879557	0.879557	0.878943

CONCLUSION

Accounting for the pliability of nodes is a topical issue in the structure design. However, numerical experiment shows that taking into account the pliability of hinged joints for fixing the beams to columns does not contribute much to the calculation of column stability. The difference in output between the absolutely hinged node (Ci=0) and the semi-rigid one, whose characteristics are in agreement with the field experiments, is less than 1 %.

he numerical experiment confirms the need to take into account the real design schemes of column elements in the frameworks, not only for elements with rigid beams-to-columns connections, but also for elements to which the joints are attached in a flexible or semi-rigid manner.

Numerical values of the calculated length μ coefficients for the columns considered within the framework of the study do not make it possible to bring the design schemes of separate areas closer to the simplified schemes described in the regulations.

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