Stress-strain state 4-way end-plate beams to column connection

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Abstract. With the development of construction technology of multistory buildings with a steel frame, the use of 4-way rigid joints for beams to column connection becomes especially relevant, as it allows to eliminate the need for arrangement of braced in the frame of a building. Residential building requires compact size of construction joint assemblies and no additional issues during finishing and decorating works. The article describes lightweight end plate joints of 4-way beams to column connection as one of the most compact, economical and easy to assembly type of connection. The article analyzes numerical models of such assemblies created in the ANSYS Mechanical software package. The variants of assemblies with 20- and 30-mm thick end plates are considered, the stress state of the column wall in 2-way and 4-way connection is compared, and the influence of end plate thickness on the stress state of bolts in such assemblies is evaluated. The result of the analysis brings us to the conclusion that experimental research is needed to create an analytical procedure for calculating of such joints.

Keywords: end-plate, moment 4-way connection, bolted beam-to column joints, numerical investigation, semi-rigid connection

1 Introduction

Frame assembly of steel structures is an important type of beam-to-column connections. Such assemblies are used to construct frameworks, which are very common in the construction of multi-storeyed and highrise buildings. The use of such connections allows to reduce the number of structural bracing components or completely eliminate them. At the same time, we should consider the actual rigidity of such connections to ensure spatial stability and permanency of a building.

End plates, bolted connections on plates, welded connections, L-shaped connections, etc. are used in rigid assemblies’ design to transfer bending moment from a beam to a column. In comparison, each structural solution differs in terms of effort required for manufacturing and installation, metal consumption, bearing capacity, rigidity, reliability, as well as the complexity of calculation and design. The latter is one of the key parameters in choosing a particular solution and depends on quality and development level of the regulatory and procedural guidelines.

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The 4-way rigid connection of beams to column (Fig. 1) is a structural solution that preserves the useful area in designed building by eliminating the use of bracing components. The potential of their application in different designs, confirmed by verified calculation procedures, will expand the scope of application for steel structures in the field of public and residential buildings construction.

![Fig. 1. General arrangement of a rigid 4-way beams to column connection assembly.](image)

Russian calculation and design codes do not cover a lot of issues related to the use of rigid assemblies in steel structures. These issues concern both a strength analysis of the assembly and the consideration of its actual stiffness as a part of a frame. The study [1] states that considering actual stiffness of assemblies in frames design is a very important task, because a frame assembly calculated only according to the strength requirements may have a significant deformability and is not consistent with the assumptions adopted in calculation of building frame. Therefore, it can be fair to use the concept of rigid and semi-rigid joints.

The paper [1] also notes that deformability of an assembly depends on the physical characteristics of end plate connection, the beam to be connected and the nature of loading. Also, as part of the reference work, on the basis of field experiments and numerical calculations, the formulas determining the rotation angle dependence on the bending moment were obtained. In [2], attention is paid as well to the deformability of 4-way assemblies compared to 2-way assemblies.

The analysis of domestic studies, regulations and methodological recommendations shows that the performance of 1-way or 2-way (in a plane of greater column stiffness) rigid beam to column connection is mainly evaluated. In this case a 2-dimensional computational scheme and, accordingly, a plane stress state are typically considered.

Estimation of a stress-strain state for elements of 4-way assemblies can be found in foreign studies [3]. The paper deals with only a few aspects of a broad topic, and adequate empirical support is lacking.

Separately, the studies focused on to the investigation of collar beam to column connection in the plane of lower stiffness [4-8] should be noted. In papers [4,5] several types I-beams to I-column connection on the side of “weak” axis are considered, numerical studies of end plate connections with application of additional stiffeners and outriggers are carried out and bolted connections of quality grade 8.8 and 10.9 are considered.
Rigid beams to column connection assemblies, including flanged ones, in the plane of greatest stiffness (the “major” plane) are being actively investigated, and a large number of articles aimed at clarification of calculation procedures for such joints are being published [9-13]. In [13], a clarified procedure for the calculation of lightweight end-plate assemblies of I-beams to columns connection in the “strong” plane, obtained from the analysis of a large number of finite element models, is proposed. Experimental and numerical studies of I-beams to column end plate connections in the “strong” plane are also described in the works [14-18], where methods for evaluating the maximum bending moment and deformability for these connections are proposed.

At the moment, there is no developed and verified procedure for calculating beam-to-column connections on four sides, considering the complex stress state in the column and making possible the strength and deformability evaluation for these connections.

2 Materials and methods

The study was carried out on the basis of finite element models compiled in Ansys Mechanical. The calculation model includes the support column, end plates in column flanges and column wall, and fastening bolts. The interaction between the end plate and the column was simulated as a contact of Frictional type surfaces with a friction coefficient of 0.35 (in accordance SP16.13330.2017). Finite element mesh of hexahedral and prismatic solid finite elements was broken up based on the geometry of the column and adjacent end plates (Fig. 2). The bolts were simulated as beam-type finite elements with circular cross-section [19].

Bolts are connected to the end plate surface using rigid finite elements of type RBE2 (Fig. 3a) in the area limited by the washer diameter. The end plate connection loads are applied to the end plate surface at the interface with the beam (Fig. 3b). The area of load application corresponds to the cross-sectional dimension of the profile to be connected considering the welds (Fig. 3c). The structural connections in this area are combined by a rigid non-deformable element with the main joint at the center of gravity of this area. This method of applying loads is based on Bernoulli hypothesis. The beam is connected to the end plate by full strength welds, and its cross-section of the beam in the end plate zone remains flat under bending, which can be regarded as an approximation to the actual operation of the connection.
Fig. 3. Areas of load transfer from prestressed bolt (a) to the end plate and at the I-beam connection (b, c).

The physical and mechanical properties of the column and end plate joints in the calculation model were specified on the basis of the isotropic material simulation with specifications of structural-grade steel S355 according to SP 16.13330: modulus of elasticity \( E = 2.06 \cdot 10^5 \) MPa; Poisson ratio \( \nu = 0.3 \). The steel work diagram for the column and the end plate was assumed to be bilinear, with yield strengths of 340 and 350 MPa (depending on the thickness of the element). In post-yield conditions the tangent modulus was taken as 977 MPa (as recommended in SP16.13330.2017). For bolts of 10.9 quality grade the Prandtl diagram was used, where the characteristic resistance of bolt steel to strain was taken as ultimate strength \( R_{\text{bun}} = 1040 \) MPa.

The boundary conditions describing the effects of external factors on the finite element model were applied in three phases. In phase 1 of loading, the bolt pre-stress was set, according to SP 294.1325800.2017, equal to:

\[
B_0 = 0.9 \cdot R_{\text{bh}} \cdot A_{\text{bn}}
\]

where: \( B_0 \) – bolt prestressing force; \( R_{\text{bh}} \) – estimated resistance of bolt steel to strain; \( A_{\text{bn}} \) – bolt net cross sectional area.

In phase 2, the column was compressed by a center load equal to:

\[
F = 0.5 \cdot R_y \cdot A
\]

where: \( R_y \) – estimated resistance of column steel; \( A \) – column cross sectional area; 0.5 – the coefficient correcting for axial force impact in a normal column for a frame of a highrise building.

In phase 3, the end plate loads were specified. For this purpose, an eccentric cross force was applied to the rigid finite element in the area of the beam – end plate connection. The eccentricity and cross force were selected so as to set the necessary ratio of shear force and bending moment in the end plate, corresponding to the actual operation values for the assembly elements as a part of a frame (Fig. 4).

The Newton-Raphson iteration method of solving the numerical problem taking into account the deformed scheme, was used [20]. In phase 1 and 2, the load was applied in 3 steps; in phase 3, 5 to 100 steps were assigned.
Fig. 4. Application of loads to a finite element model.

Verification of finite element model and determination of optimum mesh size was performed by successive iterative calculations with gradual reduction of mesh size in end plate-to-column connection zone. Calculation results for two nearest iterations were compared in terms of key indicators (maximum stresses and strains in the elements). At each subsequent step, a finer mesh was broken up, and the process was repeated until the difference in values of compared indicators in nearest iterations became less than 0.5%.

In this paper, eight-bolt end plate connections were considered: three rows of bolts were provided in the tensile region and one row of bolts in the compression region. M24 bolts were used in the end plate connections to column flange, and M20 bolts were used in the connections to the column wall. Models with 30 mm thick end plates and 20 mm thick end plates were considered. Also, in order to evaluate the impact of beam contact along the column “weak” axis on the stress-strain state of the column wall, models with only 2-way connection of beams to the column flanges were considered. Geometric properties of the joints are listed in Table 1.

Table 1. Geometric properties of the spatial joints.

<table>
<thead>
<tr>
<th>Sizes of beams and column</th>
<th>Major-axis end plate type, [mm]</th>
<th>Weak-axis end plate type, [mm]</th>
<th>End plate thickness [mm]</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column:30K2</td>
<td><img src="image1" alt="Diagram" /></td>
<td><img src="image2" alt="Diagram" /></td>
<td>20</td>
<td>A</td>
</tr>
<tr>
<td>Major-axis beam: 35III2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak-axis beam: 25III1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GOST P 57837-2017</td>
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</tbody>
</table>
3 Results

The following key parameters were analyzed and compared based on the results of the model calculations: maximum equivalent stress values in column walls of 2-way and 4-way connections, bolt stress distribution and equivalent stresses in the column wall and flanges at end plate thicknesses of 20 and 30mm, and the total bearing capacity of the assembly connections at different end plate thickness values.

The load rating for the connection at this phase of the study was determined by the maximum allowable equivalent stress values for the bolts. The obtained maximum loads are listed in Table 2.

<table>
<thead>
<tr>
<th>End plate thickness [mm]</th>
<th>Major-axis connection</th>
<th>Weak-axis connection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q, [kN]</td>
<td>M, [kNm]</td>
</tr>
<tr>
<td>20</td>
<td>110</td>
<td>129.80</td>
</tr>
<tr>
<td>30</td>
<td>144</td>
<td>168.48</td>
</tr>
<tr>
<td>20</td>
<td>110</td>
<td>129.80</td>
</tr>
<tr>
<td>30</td>
<td>144</td>
<td>168.48</td>
</tr>
</tbody>
</table>

The bolt stress distribution is shown in Figures 5 and 6, the evaluation of a bending effect on the stress state of bolts depending on the end plate thickness is provided in Table 3.

<table>
<thead>
<tr>
<th>Major-axis connection</th>
<th>Weak-axis connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Directional</td>
<td>a) Directional</td>
</tr>
<tr>
<td>b) Max. combined</td>
<td>b) Max. combined</td>
</tr>
<tr>
<td>c) Directional</td>
<td>c) Directional</td>
</tr>
<tr>
<td>d) Max. combined</td>
<td>d) Max. combined</td>
</tr>
</tbody>
</table>

Fig. 5. Stress distribution in bolts, end plates of 30mm thickness.

Fig. 6. Stress distribution in bolts, end plates of 20mm thickness.
### Table 3. Spatial joints loads.

<table>
<thead>
<tr>
<th>End plate thickness [mm]</th>
<th>Major-axis connection</th>
<th>Weak-axis connection</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>681.74</td>
<td>1082.3</td>
</tr>
<tr>
<td>30</td>
<td>802.38</td>
<td>1041.1</td>
</tr>
</tbody>
</table>

When considering the stress state of the column wall, there is a significant difference in the equivalent stress distribution between 2- and 4-way assemblies, although the maximum equivalent stress values in both cases are almost equal. The equivalent stress distribution in column walls is shown in Fig.7 and 8.

![Fig. 7. Stress distribution in column walls, end plates of 30mm thickness.](image)

![Fig. 8. Stress distribution in column walls, end plates of 20mm thickness.](image)

### 4 Discussion
The analysis of the stress-strain state of the 4-way rigid assemblies models shows that end plate thicknesses in adopted design solutions have a significant influence on the bearing capacity and deformability of connections. Models with 30 mm thick end plates have a higher load-bearing capacity and an increased stiffness. As the end plate thickness decreases, the force from the beam to the column is transferred less uniformly, resulting in stress points. In addition, an increase in the bending moment of the bolts is observed in Model A. This is due to the lower bending stiffness of the end plates compared to Model B. As a result, the increased deformability leads to a lower bearing capacity of the connection. In the plane of strong axes of the column, decrease in the end plate thickness from 30 mm to 20 mm leads to a reduction in the bearing capacity of connection by almost a quarter (23-24%). In the plane of weak axes of the column, decrease in bearing capacity is about 10%. The calculation results correlate well with the results of the studies reviewed in [9].

It is worth noting that the bearing capacity of models A and B is characterized by different criteria. In Model A, bolt failure occurs and in Model B, beam failure occurs. Both are strength criteria which are characterized by stress values in the respective elements. The models were evaluated primarily by strength criteria, since they are easier to track, and the maximum allowable stress values are provided in most regulatory documents on steel structures (both domestic and foreign). Proceeding to the evaluation of the column bearing capacity, strength criteria are, however, insufficient. In particular, the column wall of the considered 4-way assemblies is in a complex stress. The triaxial compression at the level of the lower beam flanges has an effect on the column wall, and at the level of the upper flanges, compression in the vertical and horizontal plane and tension in the other horizontal plane takes place. The stress has a non-uniform distribution with local points of concentration. The use of classical strength theories and the determination of equivalent stress obviously leads to an underestimation of the column bearing capacity, since the peak stresses occur in a small area, the size of which in numerical analysis depends on the finite element mesh spacing.

If local plastic deformations are assumed, there arise a possibility to use additional redundant bearing capacity in the column body. In this case, the permissible area of plastic deformations and their values should be determined. It is also required to clarify the applicability of known FE models of materials in the analysis of the stress-strain state of the steel in the complex-stressed steel beyond the proportional limit.

The bearing capacity of the column in the considered assemblies is characterized not only by strength, but also by stability, in particular by local stability of a wall. For I-beams with a relatively thin wall, it is the local stability by which the bearing capacity for the assemblies of the structure under consideration will be determined. The end plate connections to the column wall clamp it and prevent buckling. This design feature of 4-way assemblies is an advantage over conventional 2-way assemblies in the plane of greatest stiffness. The end plates on the column wall increase its stability, thereby maximizing in some cases the bearing capacity of the assembly in the plane of the greatest stiffness of the column, which is consistent with the results of [2].

Clamping the wall with the end plates leads to an inevitable increase in wall strength in this region. Due to the friction forces between the end plates and the wall, the force from the wall is also transmitted to the end plates. Thus, stresses are redistributed and the column is partially unloaded in this region. It is crucial to consider this effect because it is at the end plate contact point in the column wall where significant stresses occur.

However, a method for considering the column wall reinforcement by end plates in strength and stability analysis has not been developed to date. In the process of deforming the assembly, the end plates pressure on the wall varies and is non-uniform. This issue
complicates the theoretical evaluation of the column wall strength and stability. The numerical calculations performed in the current state also cannot help in evaluation of the column wall stability in 4-way assemblies. The complex mechanics of the stability loss depends significantly on the initial imperfections, the range of which is determined empirically.

Summarizing the above it should be concluded that for the development of procedure for the calculation of 4-way beam-column end plate connections it is necessary to carry out experimental studies that will reveal margins of bearing capacity for this structural solution.

5 Conclusion

A theoretical study of the operation of 4-way end plate assemblies for beams to column connection has been carried out. Known studies on the matter, design guidelines and regulatory documents have been analyzed. The following conclusions are drawn from the work carried out:

- Varying the end plate thicknesses significantly impacts the stress distribution in the 4-way assembly. Increased thickness results to a decrease in stresses in the elements, less bolt bending and increased bearing capacity. In the conducted numerical experiment, the replacement of end plate thicknesses in the model from 20 mm to 30 mm leads to an increase in the bearing capacity in the plane of the greatest stiffness by 30%, and in the plane of the lowest stiffness by 10%.

- The end plates contacting the column wall reinforce the wall and maximize its stability, which makes the 4-way assembly design more advantageous compared to the 2-way assembly. The stress distribution is different for 2-way and 4-way assemblies. Methods of reinforcing the column wall with flanges have not been studied to date and require experimental studies.

- Tolerance of limited plastic deformations in the column wall will make it possible to utilize the hidden resources of bearing capacity for beam-column end plate connections. The criterion for the bearing capacity of the column wall at such assemblies needs to be clarified.

- In case of beam-to-column connections on four sides, a complex stress state occurs in the column wall. Verification of material models for numerical calculations with empirical support is required to account for the plastic response of steel in the complex stress state. This will increase the design critical load of the connection.

References