



# Numerical Study of the Robustness of Steel Frames with Bolted End-Plate Joints Subjected to Sudden and Gradual Internal Column Loss

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## Abstract

The research project was carried out to estimate the robustness of flat steel-framed structures in a selected accidental situation. For this purpose, a multistage approach based on experimental tests and numerical analysis was performed. As the main objective of the work, a numerical dynamic analysis of the steel frames was performed under a sudden and gradual internal column loss scenario. FEA models were created in Abaqus software using solid and shell elements. Computer analyzes were carried out in a number of different cases, taking into account the size of the structure, the type of joints, and the method of removing the column from the structure. Detailed results on the axial forces and rotation of the joints under analysis are presented, and the robustness of the structures is estimated. In all cases of frames with flush end-plate joints, an insufficient level of robustness was observed and failures of the structures were obtained. In the cases of application of extended end-plate joints in frame analysis, the required level of robustness was reached in all cases and the stop of collapse was obtained. Finally, practical recommendations for designing robust joints and the whole frame structure are presented.

**Keywords** Robustness analysis · Dynamic analysis · Framed structure · Finite element analysis · Accidental situation

## 1 Introduction

Framed structures are one of the basic load bearing structures currently used in steel construction. Their use covers a wide range of structural systems. Steel frames are used as load-bearing structures of residential, office, and industrial buildings, as well as various types of supporting structures. Connections of this type of members are usually made by bolted end-plate joints. The traditional distinction between frame joints is divided into two basic types: pinned and rigid. Currently, we can additionally distinguish the intermediate type, i.e. flexible connections, also known as semi-rigid ones. In engineering practice, frame systems are designed only to meet the ultimate and serviceability limit conditions under persistent conditions. However, an accidental situation may occur during exploitation, such as the impact of a vehicle, a gas or other explosion, or a terrorist attack. Then

the fulfillment of limit states in a permanent situation may not be sufficient in accidental conditions.

According to the Eurocode (EN, 1990) the aftermath of events such as explosion, impact and consequence of human errors should be mitigate to level stopping the further degradation of structure and avoid the damage to the extent disproportionate to the initial cause. The resistance of structure (EN, 1993-1-1) must be ensure at the level of individual elements as members and joints, and of course, the whole structure. The standard (EN, 1991-1-7) assumes that the safety of a structure is analyzed immediately after accidental action occurs, taking into account the progressive collapse of the structure. A special approach in this situation is to consider an event in which any structural element has been damaged to such an extent that it has lost its full load capacity. In this accidental situation, the remainder of the structure is required to be able to carry the load within a relatively short period of time with some certain reliability.

In the United States of America interest in progressive disaster resistance in building structures increased significantly after the events of September 11, 2001, and was subsequently reflected in new design recommendations and standards. The General Services Administration (GSA)

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in 2013 (GSA, 2013) and the Unified Facilities Criteria (UFC) in 2009 with amendments in 2016 (UFC, 2016) were presented.

Improving knowledge on the behavior of steel frame structures subjected to column loss scenarios was started in 2018 at Rzeszow University of Technology.

As the first to estimate the behavior of steel joints in an accidental situation, experimental tests were performed on six double-sided beam-to-column joints with flush and extended end-plate joints (Kozłowski & Kukla, 2019). The second step involves the creation and validation of finite element models (Kukla et al., 2021) of these previously experimentally (Kozłowski & Kukla, 2019) tested joints. In the third phase, a parametric study (Kukla & Kozłowski, 2021a) of the influence of other parameters not included in the experimental tests, based on previously validated models, was performed. The fourth stage involves the numerical analysis (Kukla & Kozłowski, 2021b) of steel substructures tested experimentally earlier as part of the work (Kozłowski et al., 2011). As the main objective of this research project, the dynamic analysis of the flat steel frame was presented, when the internal column was removed (Kukla & Kozłowski, 2022). Detailed analysis results were presented and the robustness of the structure was assessed.

## 2 Literature Review

The review of scientific items in the field of progressive frame structure catastrophe was included in (Adam et al., 2018; Andre, 2020; Kiakojouri et al., 2020). The typology of the progressive collapse phenomenon was published in (Starossek, 2007). The definition of structural robustness in statistical terms was presented at work (Wolinski, 2013). The robustness methodology for the analysis of structure under various conditions of column removal was presented in (Formisano et al., 2015).

The subject of numerical evaluation of the robustness of steel frame structures is a new and intensively developed issue. The researchers' approach in this area was divided into two basic ones. The first was based on the assessment of the resistance of a separated fragment (sub-structure) or a flat frame system. The second approach was used in the form of a spatial analysis of the entire load-bearing structure of the building. In the article (Fu et al., 2017b) an analytical model focused on ductility was proposed to assess the robustness of two-span beam systems subjected to sudden column removal. Analysis of steel frame substructures in the column removal scenario and providing the necessary data to understand the impact of connection configuration on structural behavior were presented in (Wang et al., 2020).

The global behavior of steel planar frames was analyzed with the use of traditional beam-to-column joints and

innovative joints in the event of loss of a central column (Francavilla et al., 2018). The analysis of the robustness of a planar steel frame structure during sudden removal of a column was presented in (Huang et al., 2019; Zhong et al., 2021). In the work (Santos et al., 2020), the impact of a collision between a vehicle and a building, which may be accidental or intentional, was investigated. Finite element analysis of advanced steel frame catastrophes in the sudden column removal scenario was presented in (Li et al., 2018). Numerical models of planar frames with rigid joints were developed with the use of shell elements and validated with the results of experimental research taken from (Li et al., 2017). The evaluation of the robustness of the steel framed structural system, from the point of view of collapse, was carried out in (Osama & Khattab, 2019). The purpose of the work (Jiang et al., 2020a, 2020b) was to computationally compare the response of a multi-storey steel frame structure subjected to a fire. In articles (Vasdravellis et al., 2018) an analysis of resistance and sensitivity to progressive collapse of a steel frame building, subjected to a different scenario of accidental situations was presented. Two strategies to strengthen the structure against the advancing catastrophe of existing steel frames with pinned and semi-rigid joints were presented in (Zhang et al., 2020).

## 3 Finite Element Analysis of Frame Structure

### 3.1 Initial Assumptions

In this section, numerical analysis of steel planar frame structures with bolted end-plate joints in order to assess the robustness to the occurrence of selected accidental situations is presented. Due to the fact that the experimental tests of the joints and numerical analysis of the joints and frame subsystems were performed on flat systems, steel frames in a flat system were adopted for further analysis. At the beginning, the determination of the types of tested frame structure was required. The following assumptions were made as starting data. The IPE 300 and HEB 200 sections were used as cross sections of beams and columns, respectively.

As in the (Kozłowski & Kukla, 2019), all experimentally tested joints, i.e. joints with flush and extended end-plate in three configurations of plate thickness: 10-, 15- and 20-mm were taken to analysis in order to assess the robustness of frame structures. The column spacing on the axis of 5.00 m was assumed and the height of the storey on the beam axis at 3.50 m. To analyze the robustness, three basic static diagrams (Fig. 1) of frame structures were selected. The frames were analyzed in the 2-, 3-, and 4-bay systems. The established frame design diagrams were analyzed in six cases each (i.e., each scheme was analyzed for six different joints). The height of the structure were limited to 3 storeys,

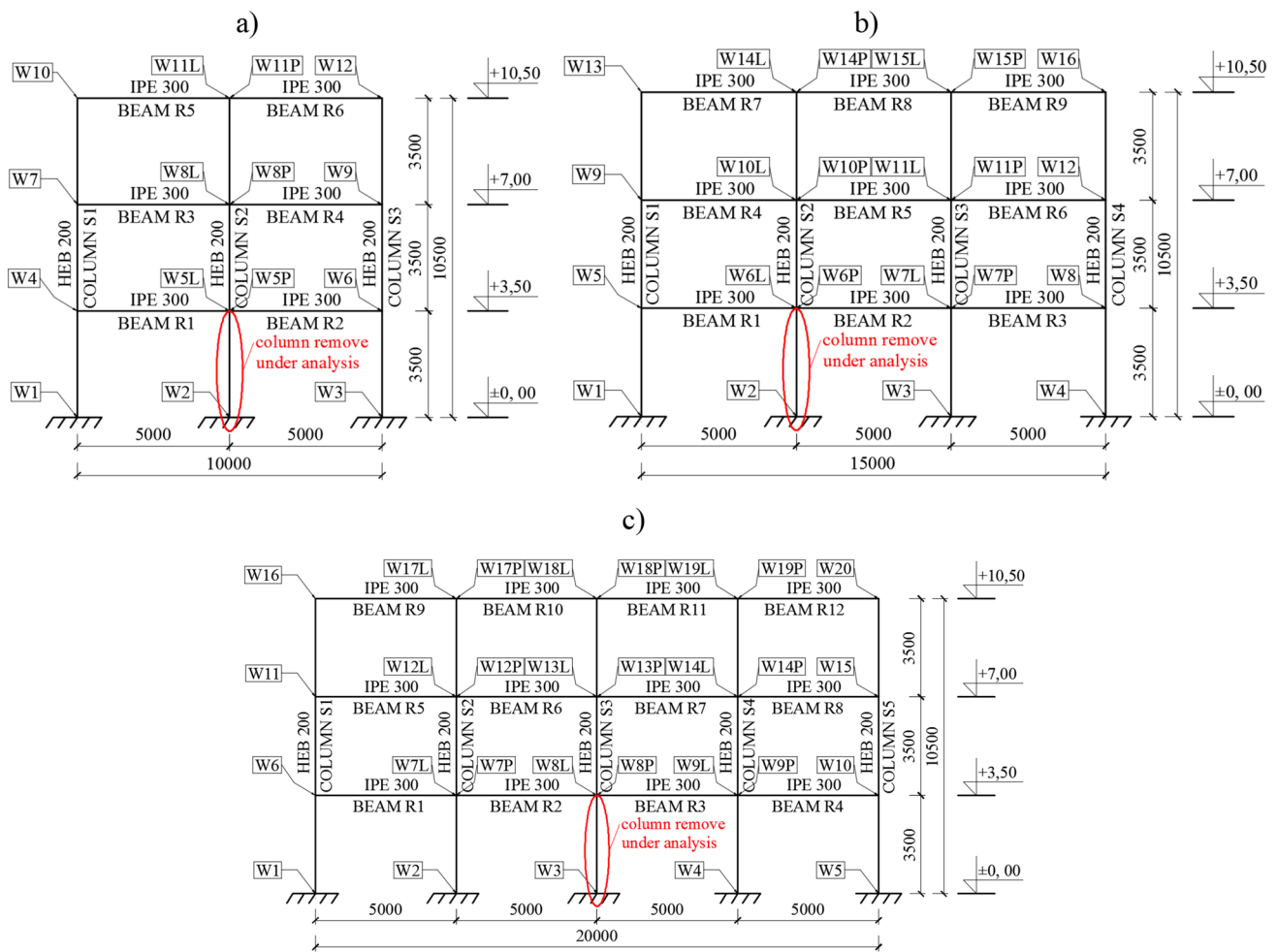


Fig. 1 Static diagrams for analysis of frame robustness

due to the use of the same column cross section at the height of the building and avoiding the use of an additional column-to-column connection. Figure 1 presents diagrams of frame systems used in dynamic analysis.

### 3.2 Type of Analysis and Finite Elements

To be able to simulate the dynamic situation of column removal during the numerical analysis, the Abaqus/explicit module was used. Due to the sudden and gradual removal of the column, additional new issues under analysis were provided. When performing dynamic analysis, the general dynamics equation should be considered as presented below:

$$M\ddot{u} + C\dot{u} + Ku = P \tag{1}$$

where:  $M$ —mass matrix,  $C$ —damping matrix,  $K$ —stiffness matrix,  $P$ —matrix of an external load applied to the system,  $\ddot{u}$ ,  $\dot{u}$ ,  $u$ —acceleration, velocity and displacement, respectively.

The application of dynamic analysis requires the use of mass in load analysis as the first factor in Eq. 1. Material density was required as one of the parameters required to determine the mass of the element analyzed in Abaqus (Dassault) using the explicit dynamic module. For steel elements, the corresponding material density was assumed to be 7850 kg/m<sup>3</sup>. In the case of beams, the procedure was slightly different. Due to the fact that the beams are elements that carry the dead load and the live load from the floor slab, it was decided to scale the density of the beam material appropriately. The relevant density of the beam material should take into account a mass equal to the slab load plus the beam's own weight. Acceleration was needed as the second factor to determine the mass effects. The gravity acceleration was assumed to be 9.81 m/s<sup>2</sup>. The damping phenomenon was needed as another important aspect from the point of view of dynamic analysis. Based on the standard provisions of GSA (GSA 2013), in the case of dynamic analysis of robustness, damping was recommended at a level of 5% of a mass. The damping effects were modeled in the

analysis using the Rayleigh damping model (Chen et al., 2018; Li et al., 2018). To define Rayleigh damping, coefficients  $\alpha$  and  $\beta$  were required, as proportional mass damping and proportional stiffness damping, respectively. Due to the fact that low frequency vibration dominates the behavior in the collapse resistance analysis, mass proportional damping with damping factor of 5% was applied.

Due to the large size of the analyzed system, it was decided to reduce the number of finite elements by appropriate measures to reduce the time required to perform numerical calculations. For this purpose two types of finite elements were applied. Straight sections of beams between joints over a length of about 4 m and straight sections of columns between joints over a distance of about 2.5 m were modeled as shell S4R elements. The joints were modeled with solid elements of the C3D8R element type. Both solid and shell finite elements allow for large deformations during the analysis.

The selection of the appropriate size of the time step is an important aspect from the point of view of the accuracy of dynamic analysis. In the UFC recommendations (UFC 2017) for robustness analysis, the time step at a value of 0.005 was presented. Finally, due to the large size of the structure model and the need to reduce the analysis time, the step size value of 0.01 was assumed.

### 3.3 Boundary Condition

Sway frames were assumed as systems analyzed in the design approach. Full restrains of column bases were adopted at the foundation level (nodes W1 to W3 / W4 / W5). During the analysis, the removal of the restraint at the base of the column was performed as a column removal scenario to simulate an accidental situation. The same beam-to-column joints for the external columns and the beam-column-beam joints for the internal columns were used throughout the entire structure.

On the upper surface of each beam, horizontal displacement restraints were adopted in the direction perpendicular to the main transverse systems, which was to simulate the presence of the slab of each story. This restraints protects the top flange of the beam from torsion during the analysis. The "coupling" option was used to connect the solid element with the shell element by using a reference point. Step-by-step controls were used to generate the response of the frame systems. The loading process was divided into 4 steps:

- **Initial**, which is the primary step implemented by the software to introduce the applied boundary conditions,
- **Tightening the bolts** by applying a double-sided load of 10 kN per bolt of each joint to obtain the initial contact of the joint surface elements, which was to simulate the initial tightening of the bolts,

- **Dead and live load** as applied gravity load. The self-weight of the steel frame structure and the floor slab with finishing layers was assumed as the permanent load.

Dead loads on the floor surface resulting from the arrangement of structural elements and finishing layers were adopted at a value of 4.00 kN/m<sup>2</sup>. The live load per floor area resulting from the function of a residential building was assumed to be 2.00 kN/m<sup>2</sup> and additionally the load of partition walls of 0.8 kN/m<sup>2</sup>. The total live load was assumed as a value of 2.8 kN / m<sup>2</sup>. Dead and live loads were applied to the surface of each beam. On the basis of the recommendation of standard (EN, 1990) load combinations were adopted in an accidental situation according to Eq. 2. The general form of the combination formula in an accidental situation takes the following form:

$$\sum_{j \geq 1} G_{k,j} + "P" + "A_d" + "( \psi_{1,1} \text{ or } \psi_{2,1} ) Q_{k,1} " + \sum_{i \geq 1} \psi_{2,i} Q_{k,i} \quad (2)$$

where:  $G_{k,j}$ —characteristic value of permanent action,  $P$ —relevant representative value of a prestressing action,  $A_d$ —design value of an accidental action,  $Q_{k,1}$ —characteristic value of the leading variable action,  $\sum \psi_{2,i} Q_{k,i}$ —the total effect of combination variable actions,  $\psi_{1,1}$  or  $\psi_{2,1}$ —combination factors on actions for accidental situation. In the case of a sudden loss of the column, the reason causing accidental situation was neglected, i.e. the pressure of the explosion or impact of the vehicle.

In the event of a gradual loss of the column, the impact of falling firefighting equipment, falling fragments of the structure and equipment of the building, and their impact on the work of the structure were neglected, as well as the thermal effects, causing the following combinations of loads:

$$\sum_{j \geq 1} G_{k,j} + "( \psi_{1,1} \text{ or } \psi_{2,1} ) Q_{k,1} " \quad (3)$$

Due to the analysis of the flat frame system, the surface load was concentrated to the linear load per meter of the beam.

The assumed loading for the structure was applied as a function of time. As the first phase, the loading phase was introduced, where the full load of the structure was applied within one second, consistent with the first load combination. In the time interval from the first second to the 2nd second, the stabilization phase of the support system was assumed due to the dynamic analysis. As the robustness analysis time in the column removal phase, the time after the 2nd second was adopted.

- **Column removal**, which was implemented as a restraint loss at the "0" level for a selected central column in the

support node. Two scenarios were adopted for the assessment of resistance:

- The first was to simulate the sudden removal of a column caused by local damage. This scenario was executed by abruptly removing the restraint of the selected support node of the middle column,
- The second scenario, as the gradual removal of the column due to a local fire, was assumed. To represent this state, the curve of the relationship of the temperature reduction coefficient according to (EN 1993-1-2) was used. The standard (EN 1993-1-2) gives three curves of the reduction factor at elevated temperatures. These curves give the values of the reduction coefficient of the steel element load capacity as a function of the increase in temperature.

The reduction curve of the modulus of elasticity was assumed as the reduction curve for the analysis. A curve of the reduction factor versus the duration of the removal of the column was presented as the main curve for the gradual removal of the column base. On the basis of this curve, the values of support vertical reactions in the being removal column were subjected to reduction. Due to the symmetry of the system and the vertical loads in the base of the inner column, only a vertical reaction appeared. Thus, the maximum obtained value of this reaction was multiplied by the reduction factor as a function of time.

The main purpose of applying gradual removal of the column was to eliminate dynamic effects that could affect the behavior and the failure mode. It will also be used to compare with sudden removal of the column.

### 3.4 Mechanical Property of the Steel

The material models and mechanical properties of the individual elements for the analysis of the frames were adopted the same as in the case of the joints in work (Kozłowski & Kukla, 2019). Due to dynamic analysis with sudden removal of the column, a change in the steel parameters was applied. The Dynamic Increase Factor (DIF), to increase the mechanical parameters of the steel resulting from the rate of deformation increment during an impact load, was applied. The change in the mechanical parameters of steel resulting from the dynamic nature of the work was used only for elements particularly exposed to sudden impact, i.e., bolts and end-plates. The remaining material models of the steel elements were adopted in accordance with the tests shown in (Kozłowski et al., 2011).

The Johnson–Cook model was used to describe the change in parameters during the impact nature of the load. This model takes into account the strain rate sensitivity and

thermal softening behavior. The analysis in this paper does not take into account the direct thermal effects on the change of mechanical parameters; hence, the DIF can be presented in a simplified form, according to formula 4:

$$DIF = 1 + C \ln \dot{\epsilon}^* \quad (4)$$

where:  $C$ —strain rate constant

$$\dot{\epsilon}^* = \frac{\dot{\epsilon}}{\dot{\epsilon}_0} \quad (5)$$

where, for quasi-static behavior:  $\dot{\epsilon}$ —strain rate, assume  $\dot{\epsilon} = 600\text{s}^{-1}$  according to work (Ribeiro et al., 2016),  $\dot{\epsilon}_0$ —reference, quasi-static strain rate,  $\dot{\epsilon}_0 = 0.001\text{s}^{-1}$

For carbon steel, the constant  $C_{\text{steel}} = 0.039$  was adopted, while for bolts, the constant  $C_{\text{bolt}} = 0.0072$  was established according to the data presented in (Ribeiro et al., 2016).

In the case of the analysis of frames subjected to the gradual removal of the column, the dynamic effects were reduced to a minimum. It can be assumed that the analysis in this case is a quasi-static analysis. In this situation, the material model of each of the elements was adopted without the DIF increasing factor.

## 3.5 Results of Frame Analysis

The appropriate nomenclature was adopted for quick identification due to the large number of analysis cases and the results obtained during the analysis. The name of each case takes into account the static diagram of the frame, the type and thickness of end-plate, and the column loss conditions, as shown in Table 1. For example, the case: 2B\_F10\_S means: 2-bay frame with a 10 mm thick flush end-plate, analysis with sudden column loss. Due to the large volume of the results of the analyzes, only selected results of the main part of the work are presented.

The value of the axial force in each beam of the frame structure was determined on the basis of the normal stresses read from the solid element of each beam. In the case of an axial force, a positive value indicates tension, and a negative value indicates a compressive force.

### 3.5.1 Results of the 4B\_F20\_G Frame Analysis

In Fig. 2a, a vertical displacement of the S3 column was presented during the analysis time. From the start of the analysis to the time of start of fire, the displacement value was 0. The increase in vertical displacement after about 3rd second was observed. After failure of the first joint, a significant increase in displacement of the S3 column was seen to the end of the analysis. A similar behavior was observed in the horizontal displacement of the structure nodes (Fig. 2b). Differences in vertical and horizontal behavior of the structure

**Table 1** Summary of case analysis of frame systems

No	Structure scheme	Type of end-plate	Thickness of end-plate	Column loss scenario	Case indication
1	2-Bay frame “2B”	Flush end-plate “F”	10 mm “10”	Sudden “S”	2B_F10_S
2	2-Bay frame “2B”	Flush end-plate “F”	15 mm “15”	Sudden “S”	2B_F15_S
3	2-Bay frame “2B”	Flush end-plate “F”	20 mm “20”	Sudden “S”	2B_F20_S
4	2-Bay frame “2B”	Extended end-plate “E”	10 mm “10”	Sudden “S”	2B_E10_S
5	2-Bay frame “2B”	Extended end-plate “E”	15 mm “15”	Sudden “S”	2B_E15_S
6	2-Bay frame “2B”	Extended end-plate “E”	20 mm “20”	Sudden “S”	2B_E20_S
7	2-Bay frame “2B”	Flush end-plate “F”	10 mm “10”	Gradual “G”	2B_F10_G
8	2-Bay frame “2B”	Flush end-plate “F”	15 mm “15”	Gradual “G”	2B_F15_G
9	2-Bay frame “2B”	Flush end-plate “F”	20 mm “20”	Gradual “G”	2B_F20_G
10	2-Bay frame “2B”	Extended end-plate “E”	10 mm “10”	Gradual “G”	2B_E10_G
11	2-Bay frame “2B”	Extended end-plate “E”	15 mm “15”	Gradual “G”	2B_E15_G
12	2-Bay frame “2B”	Extended end-plate “E”	20 mm “20”	Gradual “G”	2B_E20_G
13	3-Bay frame “3B”	Flush end-plate “F”	10 mm “10”	Sudden “S”	3B_F10_S
14	3-Bay frame “3B”	Flush end-plate “F”	15 mm “15”	Sudden “S”	3B_F15_S
15	3-Bay frame “3B”	Flush end-plate “F”	20 mm “20”	Sudden “S”	3B_F20_S
16	3-Bay frame “3B”	Extended end-plate “E”	10 mm “10”	Sudden “S”	3B_E10_S
17	3-Bay frame “3B”	Extended end-plate “E”	15 mm “15”	Sudden “S”	3B_E15_S
18	3-Bay frame “3B”	Extended end-plate “E”	20 mm “20”	Sudden “S”	3B_E20_S
19	3-Bay frame “3B”	Flush end-plate “F”	10 mm “10”	Gradual “G”	3B_F10_G
20	3-Bay frame “3B”	Flush end-plate “F”	15 mm “15”	Gradual “G”	3B_F15_G
21	3-Bay frame “3B”	Flush end-plate “F”	20 mm “20”	Gradual “G”	3B_F20_G
22	3-Bay frame “3B”	Extended end-plate “E”	10 mm “10”	Gradual “G”	3B_E10_G
23	3-Bay frame “3B”	Extended end-plate “E”	15 mm “15”	Gradual “G”	3B_E15_G
24	3-Bay frame “3B”	Extended end-plate “E”	20 mm “20”	Gradual “G”	3B_E20_G
25	4-Bay frame “4B”	Flush end-plate “F”	10 mm “10”	Sudden “S”	4B_F10_S
26	4-Bay frame “4B”	Flush end-plate “F”	15 mm “15”	Sudden “S”	4B_F15_S
27	4-Bay frame “4B”	Flush end-plate “F”	20 mm “20”	Sudden “S”	4B_F20_S
28	4-Bay frame “4B”	Extended end-plate “E”	10 mm “10”	Sudden “S”	4B_E10_S
29	4-Bay frame “4B”	Extended end-plate “E”	15 mm “15”	Sudden “S”	4B_E15_S
30	4-Bay frame “4B”	Extended end-plate “E”	20 mm “20”	Sudden “S”	4B_E20_S
31	4-Bay frame “4B”	Flush end-plate “F”	10 mm “10”	Gradual “G”	4B_F10_G
32	4-Bay frame “4B”	Flush end-plate “F”	15 mm “15”	Gradual “G”	4B_F15_G
33	4-Bay frame “4B”	Flush end-plate “F”	20 mm “20”	Gradual “G”	4B_F20_G
34	4-Bay frame “4B”	Extended end-plate “E”	10 mm “10”	Gradual “G”	4B_E10_G
35	4-Bay frame “4B”	Extended end-plate “E”	15 mm “15”	Gradual “G”	4B_E15_G
36	4-Bay frame “4B”	Extended end-plate “E”	20 mm “20”	Gradual “G”	4B_E20_G

were observed only after the full fire stage, where the values of horizontal displacements decreased in the final stage of analysis.

The rotational behavior of the joints at the time of analysis was presented in Fig. 3a. At the initial time of the analysis, the rotations of joints were negligible. After the third second the division in values of rotation of joints was observed. The first group of joints contains the joints with high rotation, which includes joints in the directly affected zone of column removal. The second group includes other joints with small rotations. In time to 3<sup>rd</sup> second of analysis (Fig. 3b) the axial forces of a similar values were reached.

After the third second, the development of the axial forces was seen. The destruction of the W9 joint as a crucial point of analysis led to stopping further increase in axial forces. The highest values of axial force were obtained in the W8, W13, and W18 joints, that is, in the joints directly connected to the removed central column.

The visible division into two types of behavior of the structure was presented in general view (Fig. 4) of the structure. The first part includes two external bays with small displacements. The second part includes the two middle bays with a removable column with significant vertical translations. The collapse of the structure (Fig. 4b) took place only

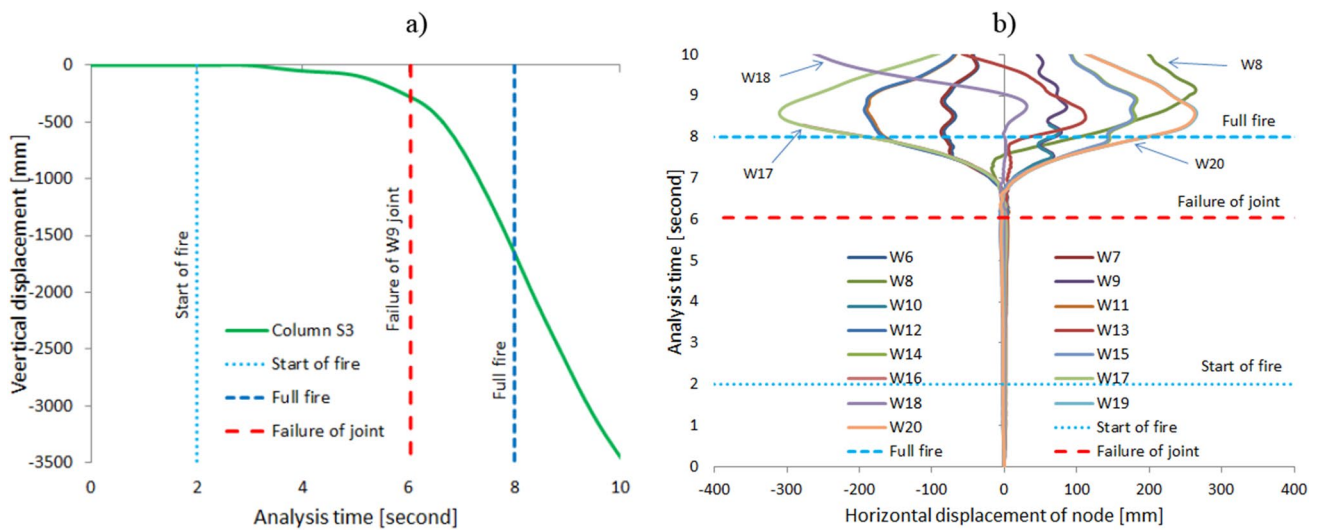


Fig. 2 Diagram of results of the 4B\_F20\_G case

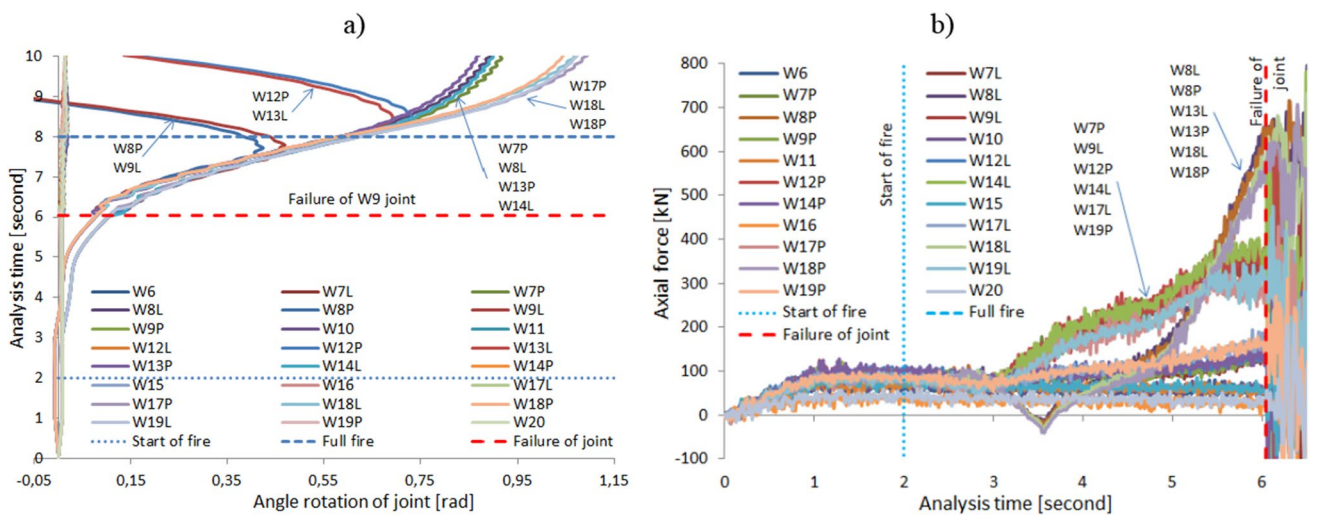


Fig. 3 Curves of the 4B\_F20\_G analysis

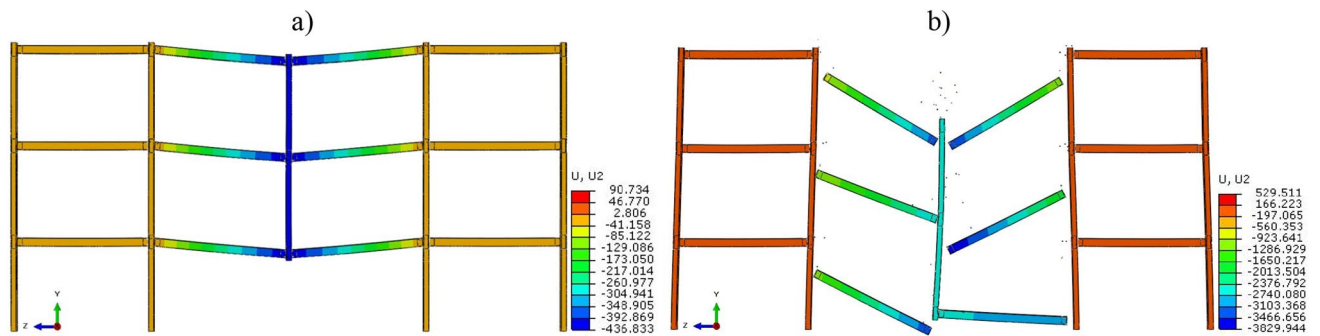


Fig. 4 View of the vertical displacement of case 4B\_F20\_G under selected time of analysis

in the middle part; the external bays of the frame were fully survived.

An insufficient level of robustness to a progressive collapse caused by the gradual removal of the center column in the case 4B\_F20\_G was shown. The destruction of successive joints led to a partial collapse in the middle part of the frame structure.

### 3.5.2 Results of the 4B\_E20\_S Frame Analysis

The vertical displacement of the central column during the analysis was presented in Fig. 5a. After the removal of the column support, the linear significant increase in vertical translation was obtained, where the maximum was reached in about 3.5 s. In a further analysis, the stabilization of the

displacement value was obtained at the end of the analysis. In case of horizontal displacements (Fig. 5b), small values of translations to time 2nd second were observed. After the column loss situation, the minor increase of displacement was reached, while in time about the third second the sign of the translation was changed. The maximum values of horizontal translations in about 3.5 s were obtained. At the time of further analysis, the stabilization of horizontal displacement was observed.

In the analysis of results of joints rotation (Fig. 6a) at the time to 2nd second the division of joints into two groups is presented. After column loss situation, the considerable increase of rotations in joints was obtained. The clear division on three groups of rotations of joints were reached. In the first group, the joints with the smallest

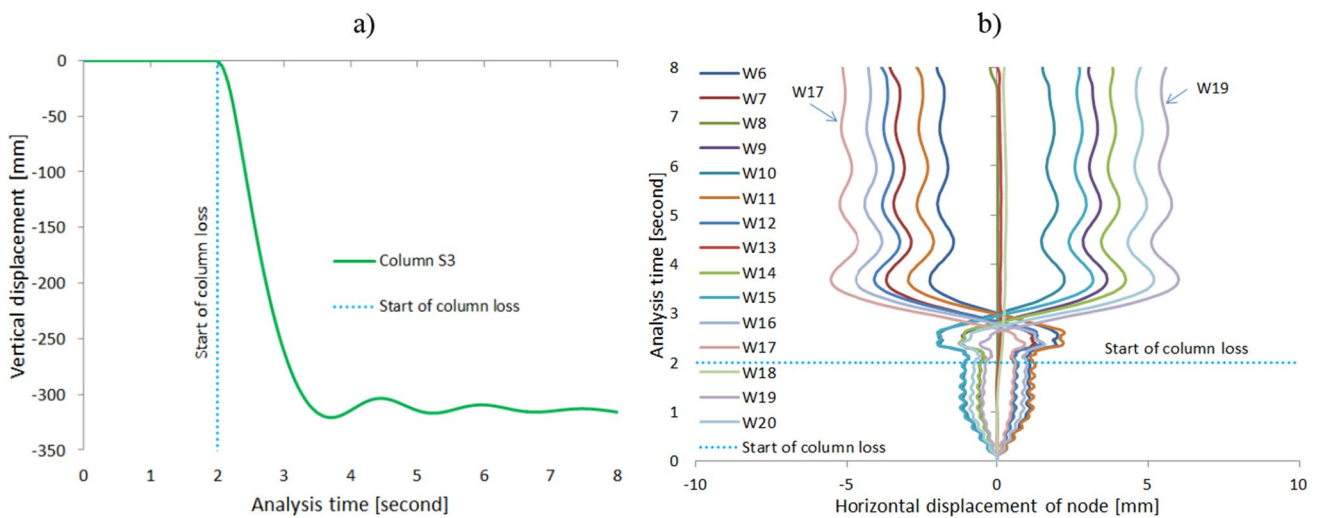


Fig. 5 Comparative curves of the 4B\_E20\_S case

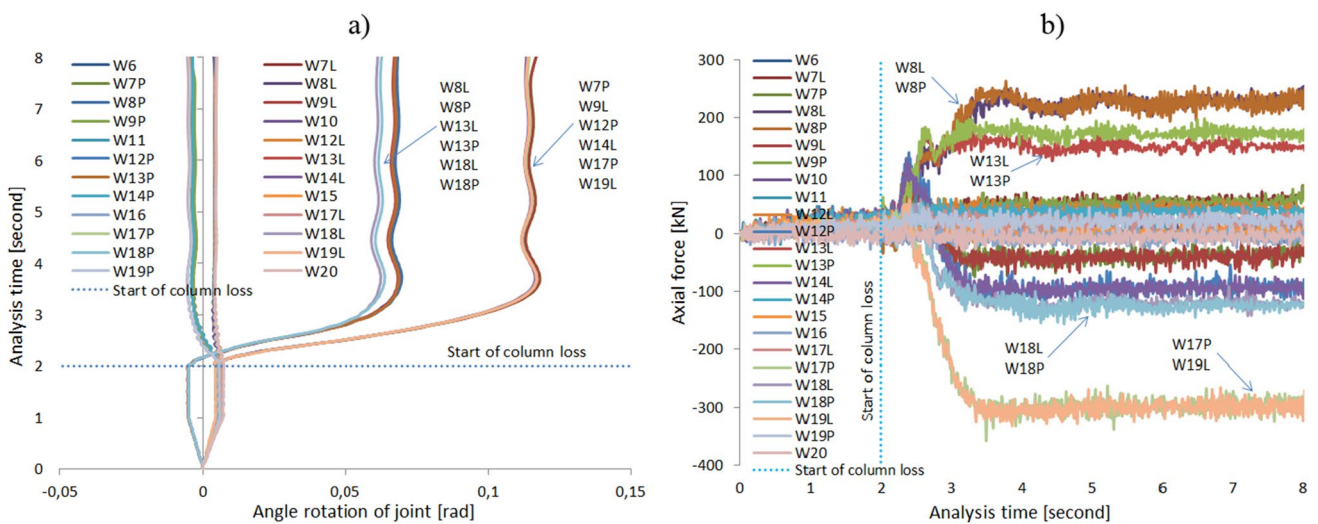


Fig. 6 Development of results of the 4B\_E20\_S case



rotations were ranked. To the second group with medium rotations the W8, W13 and W18 joints were classified. In the third group, the joints with the highest rotation located in the S2 and S4 column were assigned. The development of axial forces in the joints was presented in Fig. 6b. In the 2nd second of the analysis the values of axial force were negligibly small. After the accidental situation, a significant increase in axial forces was observed, especially in joints near the removed column. The highest tension forces in W8 and W13 were obtained. Interestingly, in the W17P / W18 and W19L joints at the highest level of the frame, the maximum compression force was created.

Figure 7 presents a global view of the structure in the final stage of the analysis. At the central part of the structure, the highest values of vertical displacement were obtained. The other part of structure, i.e. external bays as at the initial stage of analysis, was maintained. The

horizontal translations (Fig. 7b) in the final stage of the analysis present small values.

The required level of robustness was obtained under sudden column loss scenario in case 4B\_E20\_S analysis. The use of joints with extended end-plate allows a progressive collapse of steel structure to be stopped in an accidental situation.

As a summary of the results, in Tables 2 and 3 the selected factors important from the design point of view, determining the requirements for bolted end-plate joints in an exceptional situation in the scenario of removing the middle column at the foundation level of the structure are presented.

In Table 2 in the first part, the average required rotation angles of the joints are presented. As a required angle of rotation, the maximum angle of rotation of the joint obtained in the analysis without damage to the structure was defined. Sign., > ” before the value means that the required angle of

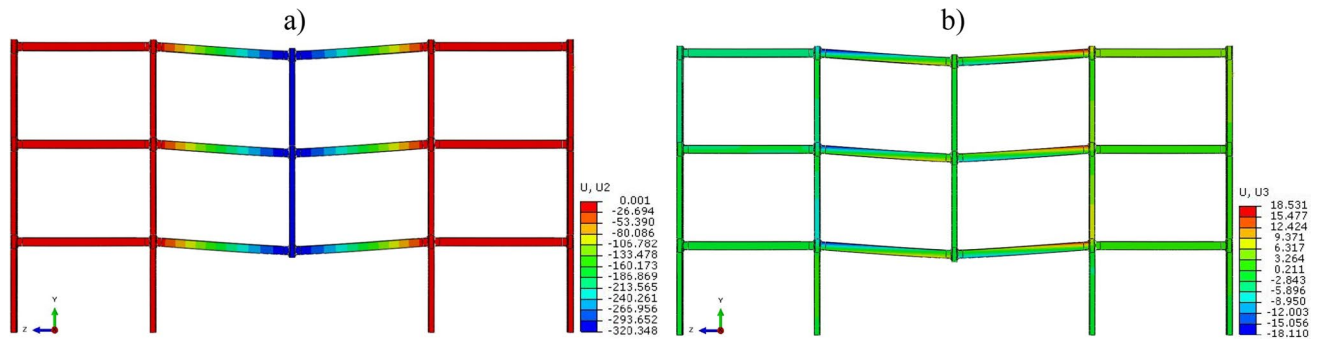


Fig. 7 Map of 4B\_E20\_S case: a) vertical displacement, b) horizontal displacement

Table 2 Summary of the rotation angles of joints

The average required angle of rotation of a joint

Two-bay frame			Three-bay frame			Four-bay frame		
Case	Average value of rotation of joint (rad)	Collapse of structure	Case	Average value of rotation of joint (rad)	Collapse of structure	Case	Average value of rotation of joint (rad)	Collapse of structure
2B_F10_S	>0.150	Yes	3B_F10_S	>0.118	Yes	4B_F10_S	>0.114	Yes
2B_F15_S	>0.130	Yes	3B_F15_S	>0.087	Yes	4B_F15_S	>0.089	Yes
2B_F20_S	>0.118	Yes	3B_F20_S	>0.068	Yes	4B_F20_S	>0.081	Yes
2B_E10_S	0.166	No	3B_E10_S	0.141	No	4B_E10_S	0.130	No
2B_E15_S	0.090	No	3B_E15_S	0.055	No	4B_E15_S	0.076	No
2B_E20_S	0.072	No	3B_E20_S	0.069	No	4B_E20_S	0.066	No
2B_F10_G	>0.158	Yes	3B_F10_G	>0.134	Yes	4B_F10_G	>0.131	Yes
2B_F15_G	>0.107	Yes	3B_F15_G	>0.089	Yes	4B_F15_G	>0.090	Yes
2B_F20_G	>0.108	Yes	3B_F20_G	>0.080	Yes	4B_F20_G	>0.077	Yes
2B_E10_G	<b>0.197</b>	No	3B_E10_G	0.149	No	4B_E10_G	0.134	No
2B_E15_G	0.096	No	3B_E15_G	0.071	No	4B_E15_G	0.054	No
2B_E20_G	0.084	No	3B_E20_G	0.061	No	4B_E20_G	0.055	No

**Table 3** Comparison of axial forces in joints

The average required axial force in joint								
Two-bay frame			Three-bay frame			Four-bay frame		
Case	Average axial force in joint (kN)	Collapse of structure	Case	Average axial force in joint (kN)	Collapse of structure	Case	Average axial force in joint (kN)	Collapse of structure
2B_F10_S	> 427.5	Yes	3B_F10_S	> 395.1	Yes	4B_F10_S	> 376.9	Yes
2B_F15_S	> 708.9	Yes	3B_F15_S	> 635.7	Yes	4B_F15_S	> 649.7	Yes
2B_F20_S	> 731.0	Yes	3B_F20_S	> 626.0	Yes	4B_F20_S	> 633.5	Yes
2B_E10_S	<b>750.5</b>	No	3B_E10_S	295.5	No	4B_E10_S	241.0	No
2B_E15_S	452.5	No	3B_E15_S	261.4	No	4B_E15_S	186.1	No
2B_E20_S	400.1	No	3B_E20_S	247.0	No	4B_E20_S	236.1	No
2B_F10_G	> 267.2	Yes	3B_F10_G	> 292.8	Yes	4B_F10_G	> 240.4	Yes
2B_F15_G	> 718.4	Yes	3B_F15_G	> 631.1	Yes	4B_F15_G	> 625.4	Yes
2B_F20_G	> 770.2	Yes	3B_F20_G	> 632.7	Yes	4B_F20_G	> 630.3	Yes
2B_E10_G	520.9	No	3B_E10_G	160.1	No	4B_E10_G	210.7	No
2B_E15_G	305.1	No	3B_E15_G	160.1	No	4B_E15_G	194.8	No
2B_E20_G	269.9	No	3B_E20_G	132.0	No	4B_E20_G	148.7	No

Bold text are defined as maximum values of results

rotation was not reached due to the collapse of the structure and the minimum value of rotation must be greater than that obtained at the time of first damage. Due to the fact that the distribution of joint rotations on individual structure levels was uniform, the average value was the quotient of the sum of all rotations of the central joints connected directly to the column removed to the number of joints.

For end-plate joints with flush end-plate, it is not possible to accurately determine the required angle of rotation due to the failure of the frames with these joints. In the case of joints with an extended end plate, it is possible to determine the required angles because these structures stopped progressive collapse.

In the case of bolted extended end-plate joint, the required angle of rotation of the joint in an accidental situation of 0.2 radians was proposed. Of course, established value of 0.2 radians is selected with considerable safety margin for other cases without damage of the structure.

In the second comparison, in Table 3 the axial forces in the joints are collected. As a first result are presented the average axial forces in joints obtained in frame analysis in accidental situations. The axial forces in the joint are the maximum axial force obtained in the analysis without damage to the structure. Sign “>” before the value means that the required axial force was not reached due to the collapse of the structure and the minimum value of force must be higher. Due to the fact that the distribution of axial forces was uneven, the average axial force was presented as the quotient of the sum of the axial force in joints directly connected to the column removed at the level of the lower beams + 3.50 to the number of joints. In the case of flush

end-plate joints, the preliminarily required value of the axial force was established, as all structures analyzed with these joints were failed.

In the case of extended end-plate joints, as required by the frame response axial force, the value of 750 kN was proposed to be transferred by the joint without damage. This value of axial force in analysis of frame with joints of a 10 mm thick end-plate was obtained (i.e. in case 2B\_E10\_S). In the case of thicker end plates, lower values of axial force were reached.

## 4 Conclusions

The research project based on the experimental test of isolated end-plate joints, numerical and parametric analysis of joints, numerical analysis of steel joints in the frame substructure under internal column loss, and numerical analysis of the steel flat frame with end-plate joints during column loss is presented. Numerical dynamic analysis was performed under sudden and gradual internal column loss scenario to assess the robustness to a progressive collapse of planar steel frames. Computer analyzes were carried out on a number of different analyzed cases. Detailed results of the analysis are presented, and the robustness of the structures was estimated.

Based on the finite element analysis of flat steel frame structures under the column removal scenario, the following conclusions can be drawn:

- (1) To mitigate the progressive collapse of steel frame structures with bolted end-plate joints, extended end-plate joints with symmetrical configuration were recommended. In all the cases of frames with flush end-plate bolted joints, a progressive catastrophe of the structures was observed.
- (2) The method of column removal also has an impact on the collapse resistance of the system. The method of removal of the column mainly influences the generation of the axial forces. Higher axial forces were obtained in the case of the two bay frames with flush and extended end-plate joints in the sudden column removal scenario. In the case of three- and four-bay frames with flush end-plate joint, a different situation was observed, where greater axial forces were generated when the middle column was gradually removed.
- (3) The values of axial forces in the beams during the robustness analysis varied, depending on the type of end plate adopted in the joint. The use of flush end-plate joints resulted in an almost equal share of each structure level in the transmission of the axial force, regardless of the thickness of the end-plate. The use of extended end-plate joints in the construction causes clear divisions into levels. Most of the tensile force is generated at the lower levels. The remainder of the tensile force is generated in the intermediate-level beams. Interestingly, the upper level beams practically do not take part in the transfer of the tensile force, and what is worth noting they are subjected to compressive forces.
- (4) In the bolted beam-to-column end-plate joints, an extended end-plate joint with a symmetrical arrangement should be used, i.e. with an extended end-plate on both sides of the beams flange. This arrangement is particularly important due to the change in the design situation from permanent to exceptional and the resulting significant changes in the load on the joints, especially the change in the sign of bending moments and the appearance of significant tensile forces. The presence of two additional rows of tension bolts protects the joint against brittle failure of the joint caused by a bolt fracture. Additionally, symmetric beam-to-column joints protect the structure against random removal of the central column, regardless of its location in the load-bearing system.
- (5) Bolts should be one size / class larger than needed in the design of the joints in the permanent design situation. Bolts in the joints in the high deformation phase are subjected to considerable stress. In addition to pure tension, the bolts are subjected to a shear force and a bending moment. Strong bolts prevent premature failure and allow full use of the joint's capacity.
- (6) In order to mitigate the collapse of the steel frame structure with bolted joints in selected accidental situ-

ations, the joints must be properly designed. Minimum axial forces in the joints should be taken into account to maintain robustness. To calculate the resistance of the joints, the minimum axial force at level 750 kN should be included.

- (7) As minimum angle for bolted end-plate joint rotation at value about 0,20 radians must be taken. There is a strong need to create the method to assess the rotation capacity of steel structure joints, to obtain the 'available' rotation capacity, to be used compared to the required rotation obtained in the global analysis (Kozłowski & Ostrowski, 2020).

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## Declarations

**Conflict of interest** DK declares that he has no conflict of interest. AK declares that he has no conflict of interest.

**Ethical approval** This article does not contain any studies with human participants or animals performed by any of the authors.

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