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Research paper

Numerical investigation of steel frame robustness under external sudden column remova

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Abstract: Numerical analysis of robustness assessment of steel planar framed structures under sudden external column removal is presented. The analysis is based on the previous experimental and numerical analyzes conducted in the Ph.D. project. Advanced and validated finite element models of steel structures with bolted end plate joints were used using Abaqus software. Six different cases of analysis using flush and extended bolted end-plate joints were performed. The actual results of the axial forces and rotations of the joints, failure models, and other important factors about structure behaviour are presented. The clear division of the results obtained depended on the type of joint used in the structure. In the cases of application of extended end-plate joints in frame analysis, the required level of robustness was reached in all cases and stopping of collapse development was obtained. In all cases of frame analysis with flush end-plate joints, an insufficient level of robustness on progressive collapse was obtained and partial failures of the structures were reached. Due to the location of the external column, the catenary actions to mitigate progressive collapse were very limited.

Keywords: dynamic analysis, external column loss, framed structure, robustness assessment, steel bolted joint, sudden column removal

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1. Introduction

The development of civil engineering is closely related to the development of human civilization over the centuries. The need and willingness to build new tall structures is a considerable challenge for civil engineers in terms of design, execution, and operational use. Inherent in the development of human civilization, the resulting threats are growing. Gas explosions, terrorist attacks, fires, or vehicle collisions with a building are just some examples of such threats. The robustness of structures to accidental situations is an often neglected and overlooked engineering issue. However, tragic events, such as construction disasters, emphatically remind engineers of the importance of this issue.

The last collapse of the two twin towers of the World Trade Center in New York on September 11, 2001 and the Champlain Tower South in Miami on June 24, 2021 are examples of tragic events in the 21st century.

Estimating the resistance to progressive collapse of the structure is a difficult and demanding issue. New design recommendations and standards were presented, such as the General Services Administration (GSA) [1] and the Unified Facilities Criteria (UFC) [2]. In Europe, the structure design is based on the Eurocode standard package [3–5] to protect the structure in permanent and accidental design situations. The robustness structure design recommendations were also presented in [6, 7].

In 2021 the first part of the research project was finished as part of the Ph.D. dissertation titled “Steel frame structures under selected accidental situations” of the first author [8]. The details of this project were presented in papers [9–11]. To further improve our knowledge of the robustness of the planar frame under accidental situation, the analysis of the frame under sudden loss of internal and external columns was performed.

Robustness assessment of the structure is an intensively developed issue in the last 50 years, as presented in the review included in [12–17]. Different accidental design situations must be included in the design phase in the robustness assessment of the structure. As a first, numerical analysis and experimental tests of steel structures subjected to vehicle impact were presented in [18–20]. The terrorist attacks as one of the main impacts on progressive collapse were classified. The impact of explosion-generated blast influence on the evaluation of steel structure behavior was shown in [21]. The other accidental situation as single and multiple column loss scenarios to estimate the robustness of the structure was presented in [22–27]. Experimental tests of planar and 3D steel structures in extreme situations were also conducted. The static and dynamic experimental study of steel frames under column loss was presented in [28–31].

In this paper, the problem concerns the estimation of the behavior of steel planar frame structures with unstiffened bolted end-plate joints in the event of a sudden external column removal scenario. The location of the loss of the column can play a significant role in the initiation and development of resistance mechanisms. In case of an external column to be removed, the behavior of the system may be completely different from that of an internal column to be removed. The change in the design situation from permanent to accidental had a significant impact on the change in the distribution of force in the structural system and its joints.

2. Finite element analysis of structure

2.1. Initial assumption

The behavior assessment of the steel planar structure with bolted end-plate joints is presented in a selected accidental design situation. As an exceptional design situation, a sudden external column loss at level “0” was used. The loss of the external column may be especially caused by a vehicle collision, a gas bottle explosion, or a terrorist attack. Important information was obtained about the structure and joint behavior relevant to the design of the steel frame structure. Due to the application of a detailed finite element model of the whole structure and joints, specific information on the failure modes of the structure and joints was reached, especially the failure modes of the joints.

Previously validated finite element models of joints and subframe structures [11] conducted on planar specimens were used to create planar frame structure models for robustness analysis. Two types of finite elements were used in Abaqus software [32] to model the entire structure. To decrease the number of finite elements, the straight parts of beams and columns between the joints by S4R shell elements (4-node general purpose shell with reduced integration with hourglass control) were modelled. For crucial parts of the structure, such as joints with their particular details (i.e. bolts, washers, and nuts), the C3D8R solid-type elements (8-node linear brick with reduced integration with hourglass control) were adopted. The static diagram of the analyzed structure is presented in Fig. 1. A four-bay frame with 5 m spacing of columns and 3.5 m height of each story was assumed.

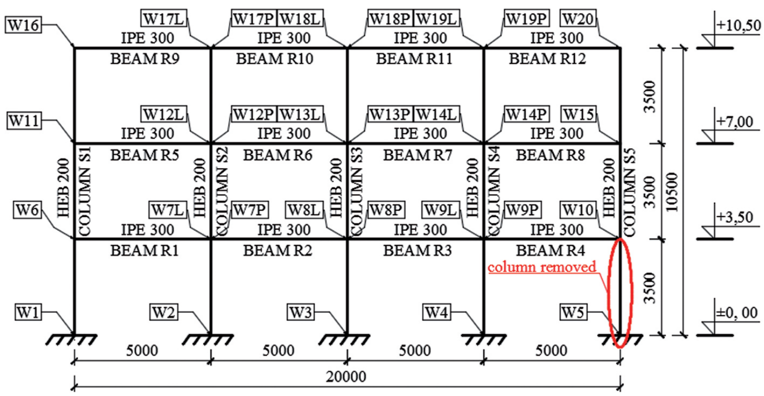
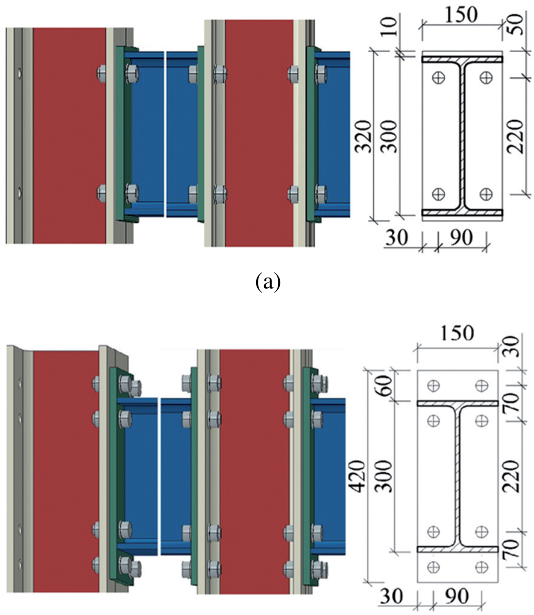


Fig. 1. Static diagram of a four-bay system for analysis of frame robustness under corner column loss

I-shaped sections as IPE 300 and HEB 200 as cross sections of beams and columns were used, respectively. Six different bolted joints with flush and extended end plates (Fig. 2) were applied in the analysis. The three various thicknesses of the end plate, 10 mm, 15 mm, and 20 mm were analyzed.

To simulate the sudden column loss situation under analysis the Abaqus/Explicit module [32] was used. The use of mass as a load in dynamic analysis was required. For this



(a)

(b)

Fig. 2. View of external, internal and details of: a) flush end-plate joint, b) extended end-plate joint

purpose, the gravitational load was applied in the analysis. To create the gravity load in Abaqus, material density and gravity acceleration were required. For the steel elements in the model, the corresponding material density of 7850 kg/m^3 was assumed. In case of beams (R1 to R12 beam, see Fig. 1) the dead and live loads are transferred from floor slab to beam. In the numerical analysis the presence of concrete slab was neglected. To obtain the applicable beam load, a rescale of the material density was applied. The slab load plus the own weight of the beam was included in the mass of the beams. Gravitational acceleration was assumed to be 9.81 m/s^2.

As the second important issue, a damping phenomenon must be considered. In the standard recommendations in GSA [1] damping at the level of 5% of mass was recommended under dynamic robustness analysis. In this regard, the damping effects based on the Rayleigh damping model were applied in the analysis. Generally, low-frequency vibration dominates the structure behavior in robustness analysis; thus, a mass proportional damping with damping factor of 5% was adopted.

2.2. Boundary condition and mechanical property of steel

For the assessment of the robustness of the frame in an accidental situation, a sway frame with rigid columns supports at level 0 was assumed for an analysis of the structural systems. To simulate an accidental situation as column loss at analysis, the time of removal the restraint of external column was applied. For each of the analysis cases, the same beam-column-beam joints and beam-column joints were applied throughout the entire structure system. The presence of the concrete slab was modelled in structure in each level in a discreet way – horizontal displacement restraints in the direction perpendicular to the main transverse system were applied. This restraint protects the top flange of the beams from torsion under analysis. To generate response of the frame at time analysis the step-by-step controls were applied. Due to the complexity of the process of the loading, the analysis was divided into few steps presented below:

- Initial, which is the primary step input by the software to assume the applied boundary conditions and tighten the bolts to obtain the initial contact of the joint surface elements. A double-sided load of 10 kN per bolt of each joint was added, 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 load on the floor surface was adopted at a value of 4.00 kN/m^2. The live load per floor area was assumed as a value of 2.8 kN/m^2. Dead and live loads were applied to the top surface of each beam. In the analysis of the structure with sudden loss of the external column, the reason causing the extremal situation was neglected, i.e. the pressure of the explosion or the impact of the vehicle. The assumed loading of the structure as a function of the time analysis was applied. In the first phase, the full load of the structure was applied to time of the first second. The time between the first and second seconds as stabilization time was assumed to reduce the dynamic effects on structure behavior. The time after the second second to the end of analysis as column loss phase was adopted.
- Column removal, where a constraint of column at the "0" level was removed for a selected external column. This scenario was created to reproduce the abrupt removal of a column in a sudden accidental design situation.

The material properties (Table 1) and the steel models of the elements for the analysis of the frames were adopted the same as in the case of experimental joints tests [9].

Table 1. Average material properties of steel elements based on [9]

Element	Yield point f_y [N/mm^2]	Tensile strength f_u [N/mm^2]
End-plate, t = 10 mm	294	418
End-plate, t = 15 mm	396	522
End-plate, t = 20 mm	263	406
HEB200 – column flange	257	414
HEB200 – column web	316	446
IPE300 – beam	290	415
M20 bolt	1005	1116

To create the real behavior of the material under analysis, the true stress – strain relationship with material degradation presented in [11] was assumed. Due to the applications of dynamic analysis, the increase of the properties of steel material was used employing the dynamic increase factor (DIF) resulting from the rate of deformation during impact load in the analysis. The DIF were used for elements subjected directly to impact nature of loading i.e. bolts and end-plates of joints. The other material models of steel elements were adopted according to the experimental tests shown in [9]. The Johnson-Cook model was used to describe the change in parameters during the impact nature of the load. The DIF was presented in a simplified form, according to Eq. (2.1) and (2.2):

$$\text{DIF} = 1 + C \ln \dot{\varepsilon}^* \tag{2.1}$$

where: C – strain rate constant,

$$\dot{\varepsilon}^* = \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \tag{2.2}$$

where, for quasi-static behavior: $\dot{\varepsilon}$ – strain rate, assume $\dot{\varepsilon} = 600\ \text{s}^{-1}$ according to work [33], $\dot{\varepsilon}_0$ – reference quasi-static strain rate, $\dot{\varepsilon}_0 = 0.001\ \text{s}^{-1}$.

The basic parameters for carbon steel, the constant $C_{\text{steel}} = 0.039$ was adopted, while for bolts the constant $C_{\text{bolt}} = 0.0072$ was adopted according to the data presented in [33].

2.3. Results of frame analysis

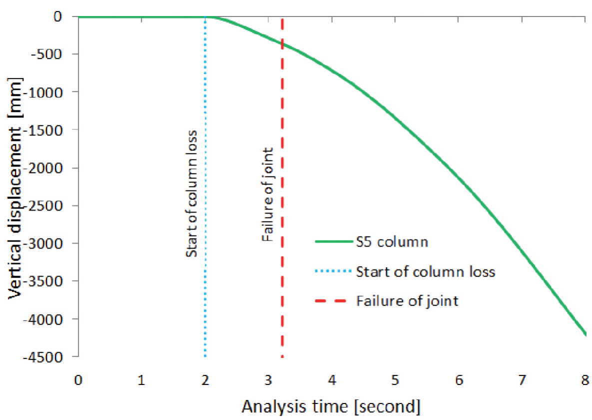
Table 2 presents the summary of all the cases analyzed. Analysis of planar frames was performed in six different cases. As presented in Table 2 division on two main types of end-plates were applied.

Table 2. Collation of case analysis of frame structure under external column loss

No	Type of end-plate	Thickness of end-plate	Case indication
1	Flush end-plate “F”	10 mm “10”	4B_F10
2	Flush end-plate “F”	15 mm “15”	4B_F15
3	Flush end-plate “F”	20 mm “20”	4B_F20
4	Extended end-plate “E”	10 mm “10”	4B_E10
5	Extended end-plate “E”	15 mm “15”	4B_E15
6	Extended end-plate “E”	20 mm “20”	4B_E20

Additionally, the three different thicknesses of the end-plates were implemented: 10 mm, 15 mm and 20 mm for flush and extended end-plates. The change in the thickness of the end-plate was dictated by the need to compare with the joints tested experimentally [9] and analyzed earlier in the event of internal column removal. To identify the case, the appropriate nomenclature was adopted for all cases. Due to the large volume of the results for any case, only the selected cases in the main part of the paper were presented. Normal stresses read from the solid element of each beam were used to determine the value of the axial force in each beam of the frame structure. A positive value of the axial force was indicated as the tensile force in the joint.

At the beginning a result of case 4B_F15 is presented. The vertical displacement of the removal of the S5 column at the analysis time is presented in Fig. 3a. After the beginning of column loss after the second second, the increase of displacement of column S5 was reached. The first failure of the joint lead to further increase of vertical displacement to the end of the analysis. The horizontal behavior of the structure (Fig. 3b) under analysis time was different. After the start of column loss, an increase in horizontal translation was observed.



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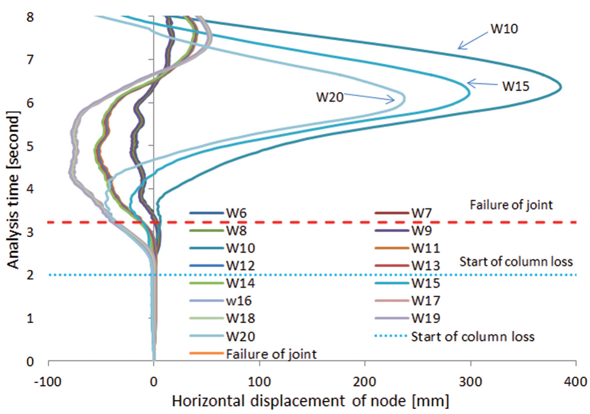
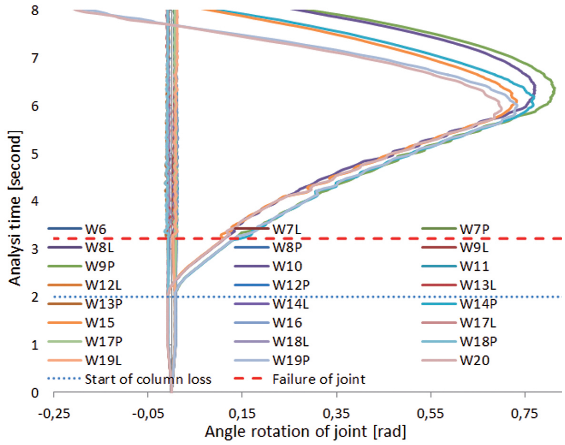


(b)

Fig. 3. Diagram of: a) vertical displacement of S5 column – time analysis, b) time analysis versus horizontal displacement of nodes at 4B_F15 case

At the first joint failure, the change of behavior of the W10/W15 and W20 nodes was reached with a significant increase. After the sixth second to end of the analysis the return similar to the original position was observed in all nodes. The behavior of joints at the time of analysis is presented in Fig. 4. First, the relation analysis time versus rotation of joints is shown in Fig. 4a. For loss of column, the values of rotations of the joints were negligible. After sudden removal of the column, a significant increase in joint rotations was obtained on the right side of the S4 column. Due to the clear division of behavior, two groups of joints were selected. In the first group, the joints with small rotation values were classified. The beam joints directly connected to the removed column were assigned to the second group. At the time to the second second of analysis (see Fig. 4b) the axial forces in joints at the approximate level were reached.



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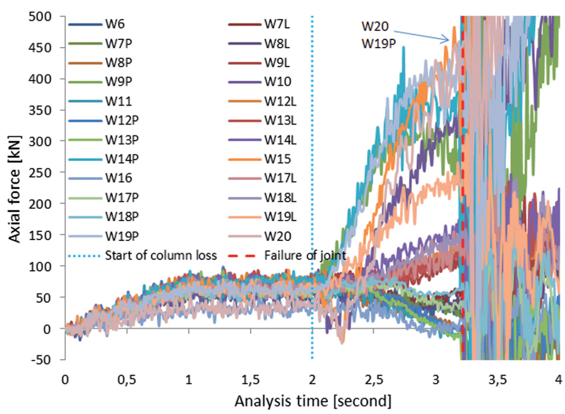
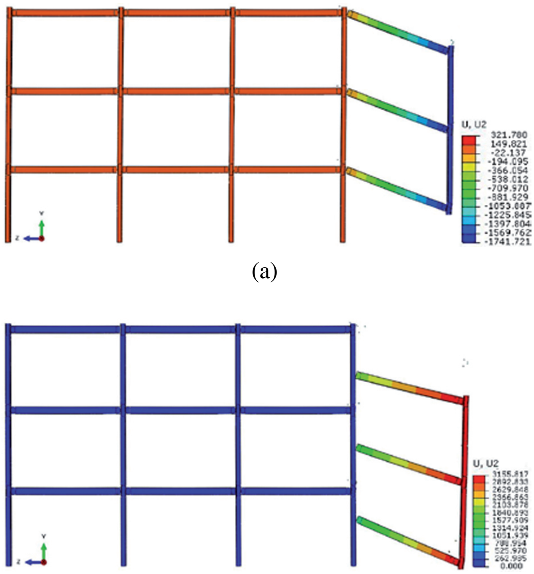


(b)

Fig. 4. Results of 4B_F15 analysis: a) time analysis – angle rotation of joints, b) axial force versus time analysis

After sudden column loss, the change and development of axial forces were obtained at the joints. The relevant development of tension force was obtained in the remove zone of the structure. The highest tension force values were reached in the W19P and W20 joint. In general view of the structure (Fig. 5) after column removal, the clear division into two parts of the structure was obtained. The first part includes three first bays with small vertical displacements. The second part includes the last external bay with a removed column with large vertical translations.

The collapse of the structure (Fig. 5b) only in the external bay, the other frames were fully survived without damage. A progressive collapse caused by the sudden loss of the corner column was shown in Case 4B_F15. The damage of successive joints caused to a partial collapse in the external part of the frame structure in an accidental situation.



(a)

(b)

Fig. 5. Map of vertical displacement of case 4B_F15 under selected time of analysis:
a) 3.5$^{\text{th}}$ second, b) 5$^{\text{th}}$ second

As results of the second case the 4B_E15 are presented. In Fig. 6a the vertical displacement of the corner column is presented in the analysis time. After the time of column loss, the linear vertical translation was obtained. Around 5.5 seconds, the maximum displacement was reached. At time after the seventh second the stopping of further vertical displacement was presented. Horizontal displacements of joints under analysis time are shown in Fig. 6b. After the second second time, the significant increase of horizontal translations was observed. The maximum values of about fifth second were obtained, where in further analysis the stabilization of displacements was reached. Generally, the highest values were reached at the joints at the +10.50 level of structure.

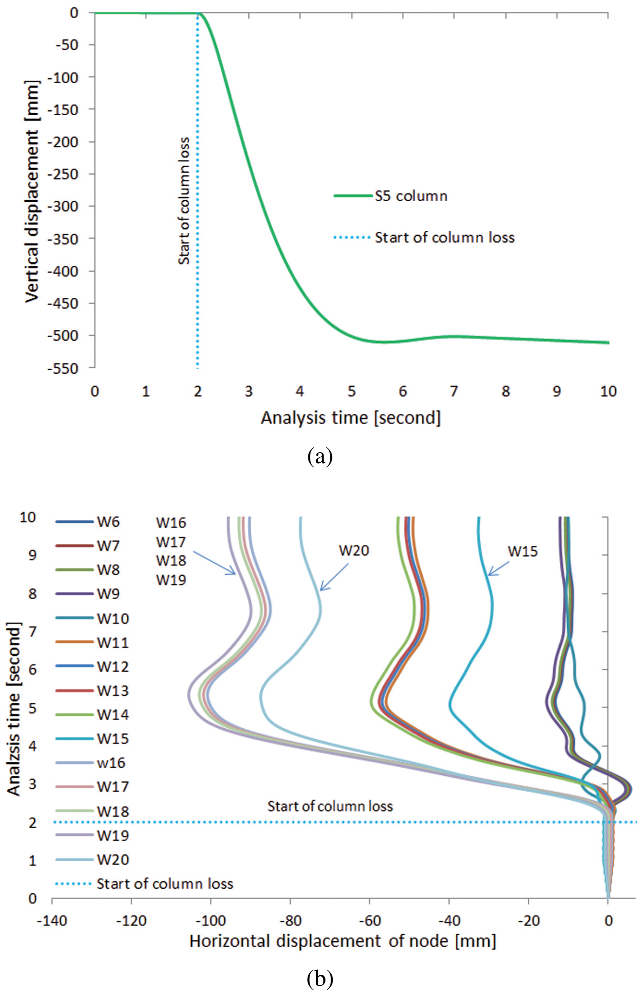
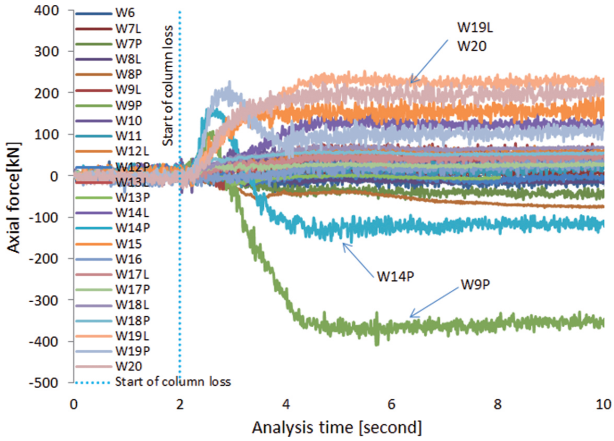


Fig. 6. Diagram of: a) vertical displacement of S5 column – time analysis, b) time analysis versus horizontal displacement of nodes at 4B_E15 case

The rotations of the joints under analysis are presented in Fig. 7a. To the second second time the negligible small values of joint rotations were obtained. After the start of the column loss situation, the linear increment of joint rotations connected to the removed column was reached. After the fifth second, the further development of rotations was stopped. A clear division was observed into two groups of joints. To first group the joints with highest rotations, e.g., W10/W15/W20 and W9P/W14P/W19P were classified. To the second group the other joints in the structure were ranked. In Figure 7b, the progress of the axial force is presented in all joints. To time of column removal the values of axial force on the same level were presented. After the start of column loss, a significant change in axial



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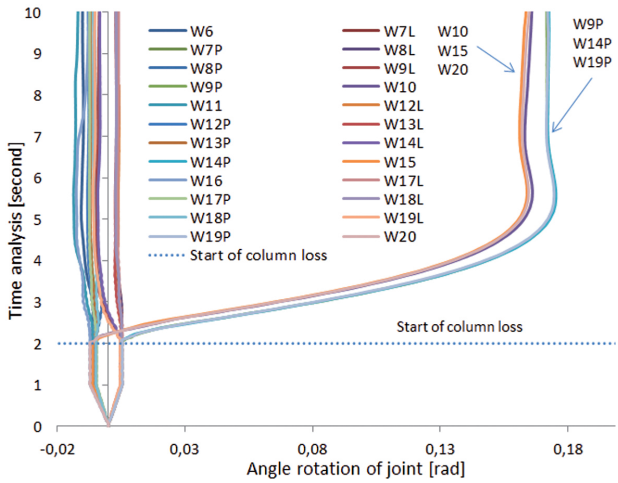

(b)

Fig. 7. History development of: a) time analysis – angle rotation of joints,
b) axial force vs. time analysis of 4B_E15 case

force was observed. The considerable part of the joints was subjected to the tension force. The interesting behavior in nodes W9P and W14P was obtained, where immediately after column loss the distinct increase of tension force was reached and in further analysis the change of axial force to compression was obtained. The highest tension force value was reached in the W19P / W20 node and the maximum compression force was reached in the W9P and W14P nodes.

In the whole view of the structure in Fig. 8 the behavior is presented in the final stage of the analysis. A significant vertical displacement (Fig. 8a) in the loss zone of the external

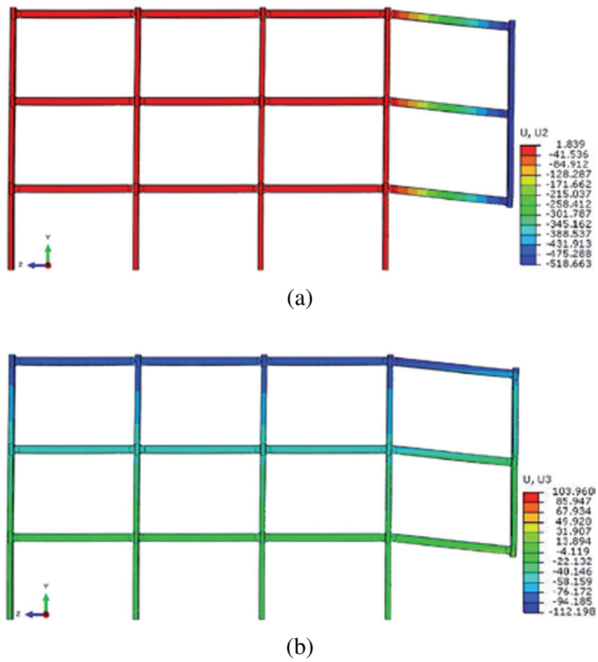

Fig. 8. Plan at final stage of analysis 4B_E15 case: a) vertical displacement, b) horizontal displacement

column. In other parts of the structure negligible vertical translations were reached. In the case of horizontal displacements (Fig. 8b) the highest values of horizontal translations were reached at the upper level of the frame. The tilt of the structure toward the removed corner column was noticed.

In analysis of frame under accidental situation with external column loss in the case of 4B_E15 a required level of robustness was obtained. Mitigation of progressive collapse of the steel frame with extended end-plate joints was reached in an accidental situation.

In Tables 3 and 4 are presented the significant factors from the point of view of the behavior of the joints under analysis, presented for the bolted end-plate joints in an accidental situation in the scenario of losing the external column at the foundation level of the structure. First, the summary of the rotation angles obtained from all joints at the time of first failure/stabilization of the structure is presented in Table 3.

The joints rotations were read from the results of the numerical analysis as the rotations of the nodes in the numerical model. In joints directly connected to the removed column (i.e. W9P/W10, W14P/W15, W19P/W20) the highest rotations were obtained. In other joints, negligible values of joint rotations were reached. As can be seen, a similar distribution of rotations in different levels in the individual structure was presented. The decrease in the rotation angles of the joints was noticeable in cases of increased thickness of the end plates in both flush and extended end plates. All frames with flush end-plate joints were

Table 3. Summary of the rotation angles of the joints at the first failure/stabilization time of the analysis

Case analysis													
Level structure	Number of joint	4B_F10		4B_F15		4B_F20		4B_E10		4B_E15		4B_E20	
		Rotation angle [rad]	Collapse of structure	Rotation angle [rad]	Collapse of structure	Rotation angle [rad]	Collapse of structure	Rotation angle [rad]	Collapse of structure	Rotation angle [rad]	Collapse of structure	Rotation angle [rad]	Collapse of structure
+3.5	W9P	0.145	YES	0.134	YES	0.117	YES	0.228	NO	0.172	NO	0.167	NO
	W10	0.123		0.112		0.097		0.218		0.164		0.160	
+7.0	W14P	0.146		0.135		0.118		0.228		0.172		0.167	
	W15	0.121		0.111		0.096		0.216		0.162		0.157	
+10.5	W19P	0.145		0.133		0.117		0.228		0.172		0.167	
	W20	0.122		0.112		0.096		0.216		0.163		0.158	

damaged. In the case of frames with extended end-plate joints, the mitigation of collapse was obtained. In the case of analysis of frames with bolted extended end-plate joint the required angle of rotation of the joint in external column loss situation of 0.23 radians was established. Due to the collapse of structures with flush end-plate joints, the required angle of rotation of the joints cannot be estimated at a safe level.

Table 4 presents the axial forces obtained in joints under external column loss. The sing minus before the value means the compression force in the joint.

Table 4. Comparison of axial forces in joints at time of first failure/stabilization of structure

Case analysis													
Level structure	Number of joint	4B_F10		4B_F15		4B_F20		4B_E10		4B_E15		4B_E20	
		Axial force [kN]	Collapse of structure	Axial force [kN]	Collapse of structure	Axial force [kN]	Collapse of structure	Axial force [kN]	Collapse of structure	Axial force [kN]	Collapse of structure	Axial force [kN]	Collapse of structure
+3.5	W9P	123.14	YES	243.64	YES	334.99	YES	–254.08	NO	–344.23	NO	–423.2	NO
	W10	237.16		326.70		292.86		33.79		37.71		–9.29	
+7.0	W14P	120.27		90.72		424.75		–1.30		–119.83		–174.8	
	W15	315.04		427.95		424.57		166.80		153.79		113.69	
+10.5	W19P	184.39		429.64		502.79		143.08		102.31		95.34	
	W20	276.01		453.48		371.13		170.08		187.52		194.28	

The use of a thicker flush end plate led to higher values of axial force. In case of extended end-plate joints, an uneven distribution of axial force was reached at the individual levels of the structure. Generally, the highest compression force was presented in the W9P joint. At the middle level +7,00 the intermediate values in W14P were obtained. The highest tension force at the upper level +10,50 was reached in the W19L joints. It is worth nothing to be the fact that in all cases of joints in highest level of structure only tension forces were presented. This behavior of joints means that the upper level was very important in creating catenary action in the structure in the situation of loss of an external column. It can be seen that the other part of the structure presents small values of axial forces. According to Table 4 in the case of frame analysis with bolted extended end plate joints under external column loss, as the required tension axial force, the value of 250 kN was estimated. This clearly shows that in the event of a loss of an external column, the decisive parameter of the resistance of the structure to progressive collapse is the resistance to the bending moment of the joint. The catenary action in this case plays a secondary role.

3. Conclusions

The results of research project about progressive collapse of steel-framed planar structures under external column removal are presented as a continuation of previous numerical steel planar frame during internal column loss. Numerical dynamic analysis was performed under an external column loss scenario to assess the behaviour of planar steel frames with bolted end-plate joints. The finite element analysis was performed on different cases analyzed. Detailed and important results were presented from the point of view of resistance of the whole structure and joint behavior and the robustness of the structures was estimated. Based on a numerical analysis of planar steel frame structures in an accidental situation with loss of an external column, the following conclusions can be drawn:

– The application of bolted extended end-plate plate joints in the steel frame structure allows us to hold the progressive collapse. It was observed that the bending resistance of the joints plays the most important role in the load capacity and robustness of the frames. It is recommended to design practice to apply extended end plate joints with end plate thickness greater than or equal to the thickness of the column flange.
– The increase in thickness of the extended end plate causes a decrease in vertical translation of the removed external column and a whole horizontal dispalcement of the nodes of the structure. The final translation of the external column at values 653.2 mm, 501.5 mm and 488.2 mm were obtained in the case of joints with extended end-plate thicknesses of 10-, 15- and 20 mm, respectively.
– The partial progressive catastrophe of the frame structure was observed in all cases of frames with bolted flush end-plate joints. Therefore, such joints, even with very thick end plates, should not be used in framed structures subjected to accidental loading.
– The behavior of joints as key elements in the structure was observed in an accidental situation. To stop the degradation of the structure, the joints must be properly shaped

and designed. Especially the available rotation capacity of the joints must be provided. The minimum available rotation angle of the joints at level 0.23 radian was selected. To maintain the robustness of the structure, the minimum axial force in the joints of 250 kN must be taken into account during design in calculating joint resistance.

– In the case of an external column loss scenario, the contribution of the catenary action to the robustness resistance mechanism was very small. Due to the location of the remove column, the development of tension action was much limited.

References

[1] General Services Administration, *Alternate path analysis & design guidelines for progressive collapse resistance. Revision 1, January 28, 2016*.

[2] *Unified Facilities Criteria: Design of structures to resist progressive collapse*. Department of Defence. Washington, 2009. Change 3, 1 November, 2016.

[3] EN 1990 Eurocode – Basis of structural design. European Committee for Standardization, CEN, Brussels, 2002.

[4] EN 1993-1-1: Eurocode 3 Design of steel structures, Part 1–1: General rules and rules for buildings. European Committee for Standardization, CEN, Brussels, 2005.

[5] EN 1991-1-7: Eurocode 1 Actions on structures, Part 1–7: General actions – Accidental actions. European Committee of Standardization, CEN, Brussels, 2006.

[6] B.A Izzuddin and J. Sio, "Rational horizontal tying force method for practical robustness design of building structures", *Engineering Structures,* vol. 252, art. no. 113676, 2022, doi: 10.1016/j.engstruct.2021.113676.

[7] B. A. Izzuddin, "Rational robustness design of multi-storey building structures", *Journal of Structural Engineering*, vol. 148, no. 3, 2022, doi: 10.1061/(ASCE)ST.1943-541X.0003254.

[8] D. Kukla, "Steel frame structures under selected accidental situations", PhD dissertation, Rzeszow University of Technology, 2021 (in Polish).

[9] A. Kozlowski and D. Kukla, "Experimental tests of steel unstiffened double side joints with flush and extended end plate", *Archives of Civil Engineering*, vol. 65, no. 4, pp. 127–154, 2019, doi: 10.2478/ace-2019-0051.

[10] D. Kukla and A. Kozlowski, "Parametric study of steel flush and extended end-plate joints under column loss scenario", *Engineering Structures*, vol. 237, art. no. 112204, 2021, doi: 10.1016/j.engstruct.2021.112204.

[11] D. Kukla and A. Kozlowski, "Analysis of steel frame under selected accidental situation", *Archives of Civil Engineering*, vol. 68, no. 4, pp. 293–309, 2022, doi: 10.24425/ace.2022.143039.

[12] J. M. Adam, F. Pasisi, J. Sagaseta, and X. Lu, "Research and practise on progressive collapse and robustness of building structures in the 21st century", *Engineering Structures*, vol. 173, pp. 122–149, 2018, doi: 10.1016/j.engstruct.2018.06.082.

[13] F. Kiakojouri, V. De Biagi, B. Chiaia, and M.R. Sheidaii, "Progressive collapse of framed building structures: Current knowledge and future prospects", *Engineering Structures*, vol. 206, art. no. 110061, 2020, doi: 10.1016/j.engstruct.2019.110061.

[14] "Mitigation of the risk of progressive collapse in steel and composite building frames under exceptional events", Research Fund for Coal and Steel, FAILNOMORE, Background document, January 2021.

[15] J. Szer, "Safety of buildings and construction disasters", *Archives of Civil Engineering*, vol. 66, no. 1, pp. 281–295, 2020, doi: 10.24425/ace.2020.131788.

[16] Sz. Woliński, "Multi-faced assessment of structural safety", *Archives of Civil Engineering*, vol. 67, no. 2, pp. 133–154, 2021, doi: 10.24425/ace.2021.137159.

[17] Sz. Woliński and T. Pytlowany, "Proposal for application of risk analysis to assess robustness of floor slabs pre-stressed with unbonded tendoms", *Archives of Civil Engineering*, vol. 68, vol. 1, pp. 241–253, 2022, doi: 10.24425/ace.2022.140166.

[18] X. Zhou, Y. He, S. Xiang, Y. Zhang, K. Ke, and W. Wang, "Experimental and numerical studies on structural response of steel garage subjected to vehicular collision", *Structures*, vol. 37, pp. 933–946, 2022, doi: 10.1016/j.istruc.2022.01.068.

[19] S. Xiang, Y. He, X. Zhou, and W. Wang, "Continuous twice-impact analysis of steel parking structure columns", *Journal of Constructional Steel Research*, vol. 187, art. no. 106989, 2021, doi: 10.1016/j.jcsr.2021.106989.

[20] A. F. Santos, A. Saniago, M. Latour, and G. Rizzano, "Robustness analysis of steel frames subjected to vehicle collisions", *Structures*, vol. 25, pp. 930–942, 2020, doi: 10.1016/j.istruc.2020.03.043.

[21] F. Kiakojouri, R. M. Sheidaii, V. De Biagi, and B. Chiaia, "Blast-induced progressive collapse of steel moment-resisting frames: Numerical studies and a framework for updating the alternate load path method", *Engineering Structures*, vol. 242, art. no. 112541, 2021, doi: 10.1016/j.engstruct.2021.112541.

[22] J. Zhang, J. Jiang, S. Xu, and Z. Wang, "An investigation of the effect of semi-rigid connections on sudden column removal in steel frames", *Structures*, vol. 13, pp. 166–177, 2018, doi: 10.1016/j.istruc.2017.12.001.

[23] L-L. Li, G-Q. Li, B. Jiang, and Y. Lu, "Analysis of robustness of steel frames against progressive collapse", *Journal of Constructional Steel Research*, vol. 143, pp. 264–278, 2018, doi: 10.1016/j.jcsr.2018.01.010.

[24] H. Li, X. Cai, L. Zhang, B. Zhang, and W. Wang, "Progressive collapse of steel moment-resisting frame subjected to loss of interior column: Experimental tests", *Engineering Structures*, vol. 150, pp. 203–220, 2017, doi: 10.1016/j.engstruct.2017.07.051.

[25] G. Vasdravellis, M. Baiguera, and D. Al.-Sammaraie, "Robustness assessment of a steel self-centering moment-resisting frame under column loss", *Journal of Constructional steel Research*, vol. 141, pp. 36-49, 2018, doi: 10.1016/j.jcsr.2017.11.004.

[26] L. Kwasniewski, "Nonlinear dynamic simulations of progressive collapse for a multistory building", *Engineering Structures*, vol. 32, no. 5, pp. 1223–1235, 2010, doi: 10.1016/j.engstruct.2009.12.048.

[27] J-Z. Zhang, B-H. Jiang, R. Feng, and R. Chen, "Robustness of steel moment frames in multi-column-removal scenarios", *Journal of Constructional Steel Research*, vol. 175, art. no. 106325, 2020, doi: 10.1016/j.jcsr.2020.106325.

[28] F. Dinu, I. Marginean, and D. Dubina, "Experimental testing and numerical modelling of steel moment-frame connections under column loss", *Engineering Structures*, vol. 151, pp. 861–878, 2017, doi: 10.1016/j.engstruct.2017.08.068.

[29] F. Dinu, I. Marginean, D. Dubina, and I. Petran, "Experimental testing and numerical analysis of 3D steel frame system under column loss", *Engineering Structures*, vol. 113, pp. 59–70, 2016, doi: 10.1016/j.engstruct.2016.01.022.

[30] G-Q. Li, L-L. Li, B. Jiang, and Y. Lu, "Experimental study on progressive collapse resistance of steel frames under a sudden column removal scenario", *Journal of Constructional Steel Research*, vol. 147, pp. 1–15, 2018, doi: 10.1016/j.jcsr.2018.03.023.

[31] F. Xie, B. Gu, and H. Qian, "Experimental study on the dynamic behavior of steel frames during progressive collapse", *Journal of Constructional Steel Research*, vol. 177, art. no. 106459, 2021, doi: 10.1016/j.jcsr.2020.106459.

[32] Dassault, *Abaqus – User Analysis User's Manual*. Dassault Systems Simulia Corp.

[33] J. Ribeiro, A. Santiago, C. Riqueiro, P. Barata, and M. Vejkovic, "Numerical assessment of T-stub component subjected to impact loading", *Engineering Structures*, vol. 106, pp. 450–460, 2016, doi: 10.1016/j.engstruct.2015.10.047.

Numeryczna analiza odporności ram stalowych pod nagłym usuwaniem słupa zewnętrznego

Słowa kluczowe: analiza dynamiczna, konstrukcja ramowa, nagłe usunięcie słupa, ocena odporności, stalowe połączenia śrubowe, utrata słupa zewnętrznego

Streszczenie:

Przedstawiono analizę numeryczną oceny odporności stalowych płaskich konstrukcji ramowych pod wpływem nagłego usunięcia słupa zewnętrznego. Przeprowadzono analizy w oparciu o wcześniejsze badania eksperymentalne i analizy numeryczne przeprowadzone w ramach pracy doktorskiej pierwszego autora. Wykorzystano zaawansowane i zwalidowane modele elementów skończonych konstrukcji stalowej z połączeniami śrubowymi z blachą czołową przy użyciu oprogramowania Abaqus. Przeprowadzono sześć różnych przypadków analizy, w których zastosowano połączenia śrubowe doczołowe z blachą wpuszczoną i wystającą. Przedstawiono dokładne wyniki sił osiowych i obrotów połączeń, modele zniszczenia i inne ważne czynniki dotyczące zachowania konstrukcji. Wyraźny podział otrzymanych wyników zależny jest od rodzaju zastosowanego węzła w konstrukcji. W przypadku zastosowania w analizie ram połączeń doczołowych z blachą wystającą, we wszystkich przypadkach osiągnięto wymagany poziom odporności i zatrzymano rozwój katastrofy. We wszystkich przypadkach analizy ram z połączeniami doczołowymi z blachą wpuszczoną uzyskano niewystarczający poziom odporności na postępującą katastrofę i obserwowano częściowe uszkodzenia konstrukcji. Ze względu na usytuowanie słupa zewnętrznego działanie akcji cięgnowej mające na celu złagodzenie postępującego zawalenia było znacznie ograniczone. Na koniec przedstawiono zalecenia dotyczące kształtowania doczołowych połączeń śrubowych w celu zwiększenia odporności konstrukcji.

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