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Research paper

Shaping the joints of steel multi-story plane frames to mitigate their progressive collapse mechanism with regard to internal column loss scenario

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Abstract: The continuation of a multistage research project based on experimental and numerical analysis is presented, focused on the robustness of the steel frame structure with bolted end-plate joints. An advanced FE model, validated on previously performed experimental tests of steel bolted joints and steel subframe structure, was used. A dynamic numerical analysis of planar steel structures was performed under a sudden internal column loss scenario using Abaqus software. An attempt was made to increase structural robustness in the event of a sudden removal of the internal column at the ground level of the steel frame with flush bolted end-plate joints. The application of a strategy based on the change of types of joints at the selected level of the structure was shown. Two main structural changes were implemented by using bolted extended end-plate joints, instead of flush end-plate, or by using the novel joint with additional rings. To check the behaviour of the structure and assess its robustness, a change of the joints was applied at the lower and upper levels of the structure. The use of bolted extended end-plate joints only in one level of the structure leads to progressive collapse of part of the structure. The application of the novel joint with additional double rings at one level of the frame in both analyses leads to obtain a required level of robustness and mitigates collapse of the structure.

Keywords: dynamic analysis, finite element simulation, novel joints, structural robustness, sudden column loss

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1. Introduction

A lot of advantages of flush end-plate joints can be presented, such as: high moment resisting capacity and rotational capacity, ability to equalise the moments in the beam, and simpler manufacturing. The design requirements and recommendations presented in standards [1, 2] are based on ensuring the required load capacity of the members and joints in permanent design situations. However, in an exploitation time of structure, an accidental situation may also take place. The collision of a vehicle with structural members such as a column, a gas explosion, or a terrorist attack are examples of accidental cases. Local failure of member as a column can provide to progressive collapse of the whole structure. The issue of the robustness of the steel structure to progressive collapse has been developed in recent years. For this reason, mainly in Europe [3] and the United States [4, 5], in recent years there has been a development of design standards that protect human life. The scientific report for European designers [6] was carried out to increase the knowledge and awareness of the designers of structural behavior in the event of accidental situations.

In Poland, the first scientific research on the structural robustness of the steel and steel-concrete composite structure was started by numerical analysis at Warsaw University of Technology [7]. Respectively, next experimental research on the topic of the behavior of joints [8] and steel subframe structure [9] under column loss was started at the Warsaw University of Technology with the collaboration of Rzeszow University of Technology. In 2021 the next research project focused on the robustness of the steel frame structures with the bolted end-plate joint under selected accidental situation as column removal was carried out. In this project, experimental tests [10], numerical parametric [11] and numerical simulations of the steel frame were carried out to assess anti-collapse in accidental situations [12–14].

Based on the results obtained in [13, 14] where all frames with flush end-plate joints were partially damaged, an attempt was made to increase the structural robustness of such steel frames. The problem actuated in this paper focused on shaping of the frames by changing joints as a key element of resistance of the steel frame structure. By changing joint behavior, an increase in the robustness of the structure and a mitigation of progressive collapse can be obtained.

2. Literature review

The review of papers in the field of disasters, catastrophes, and progressive collapse of structures was presented in [15, 16]. The definition and assessment of structural robustness were presented in the articles [17, 18].

In recent articles, different strategies have been presented to improve the robustness of steel structures, as shown in [19, 20]. On the basis of these researches, the few main paths can be observed. In a first path, the improvement of joints as key elements of steel structures was realised. In articles [21–24] strengthening of joints was proposed with prestressed steel strands and high-tensile steel bars. In works [25, 26] the improved and novel principle for improving the anti-collapse performance of steel frames was presented. The application of corrugated steel plates at the end of beams to improve joint ductility was presented in works [27, 28]. The increased resistance performance of steel joints was shown by opening the beam web [29]. A new strengthened strategy for limited anti-collapse capacity of extended end-plate joints by bending stiffened steel plates was presented in [30]. The welded stiffener for the bolted web connection in [31] was introduced to enhance structural robustness to prevent progressive collapse. As a second path to increase the anti-collapse performance of the steel structure, the retrofit through the application of the truss at the roof level of the structure was presented in [32].

3. Numerical analysis

3.1. Assumptions

In this section, numerical analysis of steel planar frame structures with bolted flush end-plate joints was presented with modification to improve the robustness in the internal column sudden loss scenario as accidental situations based on requirements presented in Eurocode standards [1–3]. The main modification includes:

– application of symmetrical extended end-plate joints in selected frame levels, instead of flush end-plate joints,
– application of novel joints solution as flush-end-plate joints with additional rings (AR) – see Fig. 1.

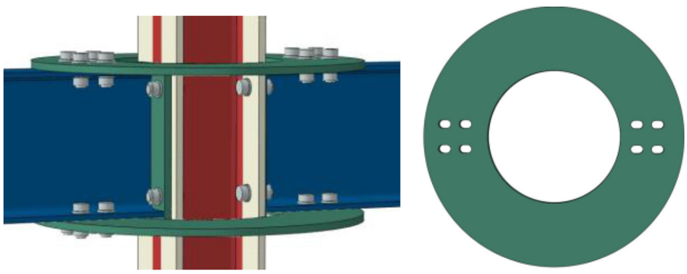


Fig. 1. View of the AR joint

Based on the experimental tests presented in [10] and the numerical validation of the bolted flush end-plate joints [11], the pure joints were modified to the AR joint. Two flat rings (Fig. 1) were modelled on the upper and lower beam flanges. To connect rings to the beams the 8 M20 grade 10.9 bolts were applied. The longitudinal holes in the rings were also designed. The idea of application of AR is based on maintaining the continuity of the tensioned beam flanges under large deformation of the central zone of joints, where the connection to the column was the weakest component of the joint. Due to the use of longitudinal holes in both rings, the tension and compression forces are transferred. The tension force under large rotation is partially transferred from the first beam flange to the second beam flange by a flat ring to omit the central part of the joint. The compression forces between beam flanges are transmitted mainly by column section, and the rings partially transfer the compressive forces in large deformation phase. The application of symmetrical layout of rings leads to obtain the same load capacity under sagging and hogging bending moment.

Four-bay and three-level framed structures (Fig. 2) were chosen for analysis. As a span of bay 5,0 m and 3,5 m as heights of one level were implied. As a beam cross-section and column cross-section an IPE 300 and HEB 200 were assumed, respectively. The columns at the foundation level were assumed to be fully fixed (node W1 to W5).

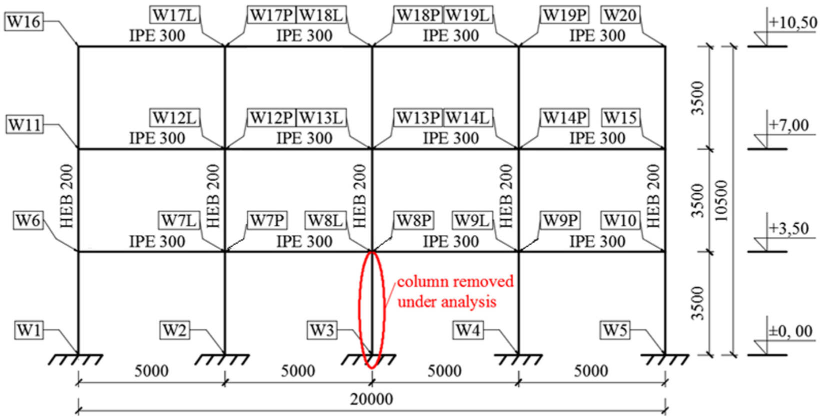


Fig. 2. Static diagram for analysis of frame robustness

The steel bolted flush end-plate joints (Fig. 3a) with two rows of bolts with two bolts in row as main beam-to-column connections were used throughout the structure. A thickness of 15 mm of end-plate was assumed. M20 bolts were used to connect the beam and column.

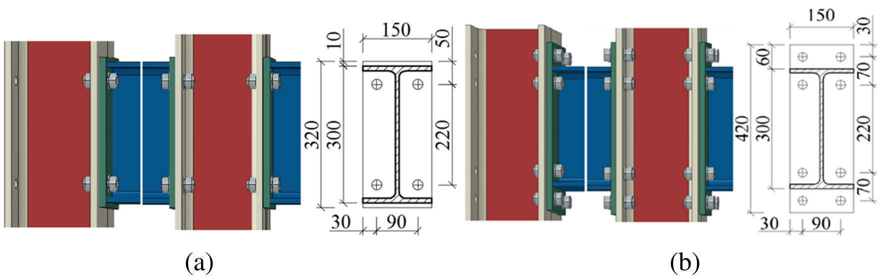


Fig. 3. View of bolted end-plate joints: (a) flush, (b) extended

3.2. FEM modelling

Nonlinear dynamic numerical simulations were performed using the explicit time integration approach based on Abaqus software [33]. Due to a lot of problems with complex contact, model convergence, and material fracture the explicit dynamic module was used. A bolt connector as a set of bolt, washers, and nut were modelled. All welds were presumed to be butt in the whole of the models and were neglected in the numerical analysis.

Two types of finite elements were used to create the model. A four-node shell element with reduced integration (S4R) was used to model the straight part of the beams at a distance of about 4,0 m. Similarly, for columns between connections at straight part about 2,5 m a S4R element was applied. An eight-node linear brick with reduced integration with hourglass control (C3D8R) as solid element type was adopted to build the joints models. An end part of beam, part of column, and bolt connector were fully built by C3D8R elements. To speed up the calculations, the general mesh solids and shell elements were established. For mesh beams and columns by shell elements, a 50 mm mesh size was assumed, where at the end of each these elements thickened to 10 mm of mesh was used. As the main size of the solid element meshes a 10 mm was adopted. The end-plates and bolts were assumed to have a density of 5 mm and 3 mm mesh size, respectively. Boundary condition and load applications were presented in details in [12–14].

Due to perform the dynamic analysis the gravity load was input and was held constant at analysis time. The self-weight of the steel frame structure and the floor slab with finishing layers was assumed as dead load of the value 4.00 kN/m^2 on the floor surface. The total live load per floor surface as value 2.8 kN/m^2 was assumed. Based on recommendation of standard [1] load combinations in an accidental situation according to Eq. (3.1) were applied. The general form of the combination formula in an accidental situation takes the following form:

$$\sum_{j\geq 1} G_{k,j} + P + A_d + \left(\psi_{1,1}\ or\ \psi_{2,1}\right) Q_{k,1} + \sum_{i>1} \psi_{2,1} Q_{k,i} \tag{3.1}$$

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 event of sudden loss of the column, the reason causing an 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 caused by fire, 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, shown in the Eq. (3.2):

$$\sum_{j\geq 1} G_{k,j} + (\psi_{1,1}\ \text{ or }\ \psi_{2,1}) Q_{k,1} \tag{3.2}$$

Due to simulation of an accidental situation such as sudden removal of the internal column, the dynamic procedure of analysis was provided. Under performing dynamic analysis, the general dynamics Eq. (3.3) should be considered as presented below:

$$M\ddot{u} + C\dot{u} + Ku = P \tag{3.3}$$

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 material density for steel elements equal to 7850 kg/m^3 was assumed as one of the parameters required to determine the mass of the element analyzed in the Abaqus software [33] using the explicit dynamic module. The loading procedure in case of beams was corrected due to the fact that the beams were an element additionally loaded by presence of floor slab. The dead load and the live load from the floor slab plus the own weight of the steel beam were calculated, by rescaling the beam material density appropriately. The relevant density of the beam material was considered similarly as in the work [7]. The acceleration of gravity in the entire model was assumed to be 9.81 m/s^2. In dynamic analysis, a damping phenomenon as another important aspect was needed. In the standard provisions of GSA [5], in the case of dynamic analysis of robustness, damping at 5% mass level was recommended. The damping effects were modelled in the analysis using the Rayleigh damping model [7]. Due to the fact that low frequency vibration dominates the behavior in the collapse resistance analysis, mass proportional damping was applied with a damping factor of 5%.

3.3. Mechanical property of steel

The material model of steel elements for the FEM analysis was adopted the same as in the case of joints experimentally tested [9]. Elasto-plastic nonlinear material model with ductile damage was defined for numerical analysis. To reproduce realistic behavior of material model the true strain-stress relationship was used. The starting failure of the assumed model was obtained after reaching the specified equivalent plastic strain. The element was removed from the mesh after reaching the destructive strain of the damaged material. As a subvariant, a displacement type with linear degradation was used for damage evolution. This material model has been applied to all elements such as members, plates and bolts.

Due to dynamic analysis with sudden column loss, a change in the steel parameters was applied. The Dynamic Increase Factor (DIF) was applied to increase the mechanical parameters of the steel resulting from the strain rate of deformation increment during an impact load. The change in the mechanical parameters of steel resulting from the dynamic nature was used for all elements. In the literature [34] different models to describe the behaviour of steel under dynamic impact were presented. As the main parameter to estimate the DIF the strain rate must be determined. To calculate the DIF three main models such as Cowper-Symonds, Johnson–Cook and Malvar–Crawford were available. On the basis of experimental impact tests [34], the highest DIF value based on the Malvar–Crawford model was reached. On the other hand, the smallest DIF values based on Johnson–Cook were reached. In robustness analysis, the application of small DIF leads to safe results from a design point of view. Therefore, the Johnson–Cook model was used to describe the change in parameters during the impact nature of the load. This model considers the strain rate sensitivity and thermal softening behavior. In the analysis presented in this paper, a direct thermal effect on the change of mechanical parameters was neglected, and hence the DIF can be presented in a simplified form, according to Eq. (3.4) and (3.5):

$$DIF = 1 + C \ln \dot{\varepsilon}^* \tag{3.4}$$

where: C – strain rate constant.

$$\dot{\varepsilon}^* = \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \tag{3.5}$$

where, for quasi-static behavior: $\dot{\varepsilon}$ – strain rate, assume $\dot{\varepsilon} = 1,83s^{-1}$ according to work [34], $\dot{\varepsilon}_0$ – reference quasi-static strain rate, $\dot{\varepsilon}_0 = 0,001s^{-1}$.

For carbon steel, the constant $C_{\text{steel}} = 0.039$ while for bolts the constant $C_{\text{bolt}} = 0.0072$ was assumed. Generally, the DIF values of 1.29 and 1.05 were calculated for steel elements and bolts, respectively. In Table 1 the main mechanical parameters of steel elements were shown for the models with application of the DIF.

Table 1. Average material properties of steel elements based on [10] with including DIF [34]

Element	Yield point f_y [N/mm^2]	Ultimate strength f_u [N/mm^2]	Dynamic increase factor (DIF)	Yield point with DIF f_y + DIF [N/mm^2]	Ultimate strength with DIF f_u + DIF [N/mm^2]
End-plate, $t = 15$ mm	396	522	1.29	510	673
HEB200 – column flange	257	414	1.29	331	534
HEB200 – column web	316	446	1.29	407	575
IPE300 – beam	290	415	1.29	374	535
M20 bolt	1005	1116	1.05	1055	1172

4. Results of frame analysis

The numerical analysis of steel frame systems was performed in four different cases. Quick identification (Table 2) of the cases of the analysis and the results obtained during the analysis was presented by the appropriate nomenclature. Out of the main parameters of the analysis cases, the static frame diagram, the type of joint, and the level of the joints changed were included.

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.

4.1. Mixed system – extended end-plate joints in + 3.50 level

A graph of vertical displacements of the removed column S3 is presented in Fig. 4 for the case 4B_F/E+3.50. In the analysis, at an accidental time, the vertical translation of the S3 column (Fig. 4) was divided into two basic phases. As first, at the time from the beginning of column loss to the first failure of joint, the linear increase of displacement was noticed. After the first failure of the joint, a significant increase in vertical translation was observed until the end of the analysis.

Table 2. Summary of case analysis of frame systems

No.	Structure scheme	Type of end-plate	Level of changed joints	Case indication
1	4-Bay frame "4B"	Flush end-plate/ extended end-plate "F/E"	"+3.50"	4B_F/E+3,5
2	4-Bay frame "4B"	Flush end-plate/ extended end-plate "F/E"	"+10.50"	4B_F/E+10,5
3	4-Bay frame "4B"	Flush end-plate/ Additional ring "F/AR"	"+3.50"	4B_F/AR+3,5
4	4-Bay frame "4B"	Flush end-plate/ Additional ring "F/AR"	"+10.50"	4B_F/AR+10,5

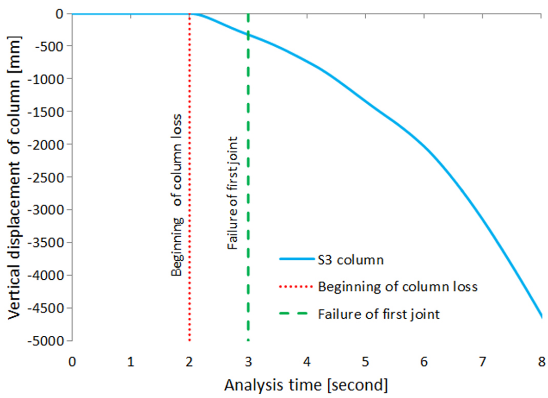
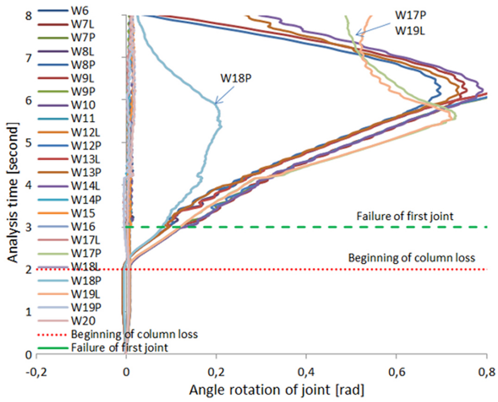


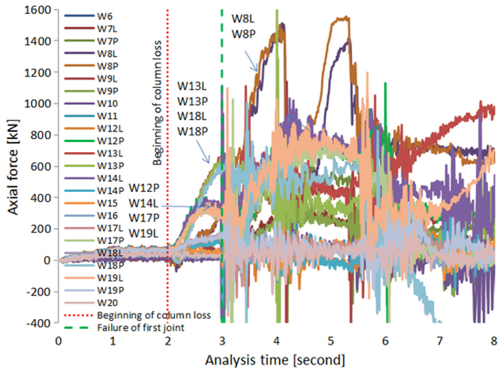
Fig. 4. Vertical displacement of S3 column-analysis time curve for 4B_F/E+3.50 case

In Fig. 5 the selected information about the joints behavior in analysis time is presented. First (Fig. 5a) the relation analysis time of the relation versus the rotation of the joints is shown. The clear division on two phase of joints behavior was observed. In the first phase, small values of rotation were obtained in all joints on time to the first second. After a beginning of column loss situation, the division into two group of joints was reached. The joint of the internal bays directly connected to the removed column reached the largest increase in rotation after the loss of the column. The progression of successive joints damages led to the achiving of significant rotation angles of the joints. To the second group, the joints in external bays were classified, where the rotations were negligible. The development of axial forces in joints at analysis time was presented in Fig. 5b. To the beginning of accidental situation the axial forces at small values were observed. The beginning of the removal of the internal column led to a change in the axial force distribution. A significant increase in forces was reached at the W13 and W18 nodes.

In the next step of analysis, the first failure of the joint was observed. After this stage, the next considerable change of axial forces was obtained. About 4th second the maximum temporary axial force in the W8 joint at values of 1556 kN was obtained. A similar second peak of axial force was observed, about 5th second. Further analysis presents a decrease of the axial forces.



(a)



(b)

Fig. 5. Curves of analysis 4B_F/E+3.50 case: (a) time vs. rotation of joints, (b) axial force vs. time

In general view of the structure (Fig. 6a) a partial collapse is presented, where the internal bays of the structure were damaged. In the lower level of the two internal bays, the joints connected to the S2 and S4 column were fully damaged. On the contrary, the joints in lever +3.50 connected to the removed column survived. All bolted flush end-plate joints connected in two internal bays were fully failured. The joint strain view (Fig. 6b and 6c) shows that the distribution was very varied. The highest strains in the joint at level +3.50 with extended end-plate joints were observed. As a failure mode of the joints, the fracture of the bolts was noticed.

The strategy based on mixed system with bolted extended end-plate joints only at lower level of the structure and other joints as flush end-plate joints leads to a progressive collapse of the steel structure under sudden internal column loss scenario.

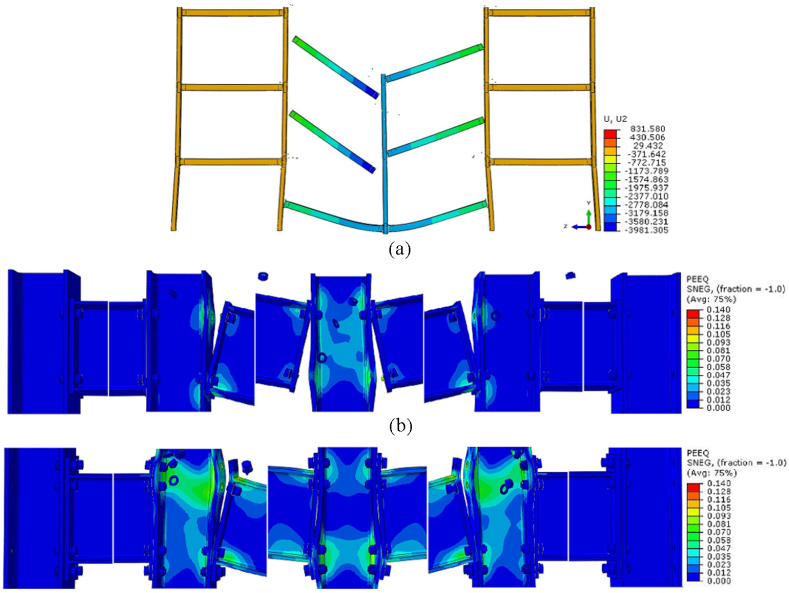


Fig. 6. View of: (a) whole structure 4B_F/E+3.50 after analysis, (b) joints strains at level +10.50 (from left W16 to W20), (c) joints strain at level +3.50 (from left W6 to W10) after damage

4.2. Mixed system – extended end-plate joints in + 10.50 level

In Fig. 7 are presented results of the case 4B_F/E + 10.50 as the vertical displacement of the removed S3 column under the analysis time. At the time after the 2nd second, the linear increase of vertical translation was observed. After the first failure of the joint in the structure at the third second, the increment of displacement was reached. In time about 7 second, the further displacement was stopped, where the maximum values about 2500 mm were obtained.

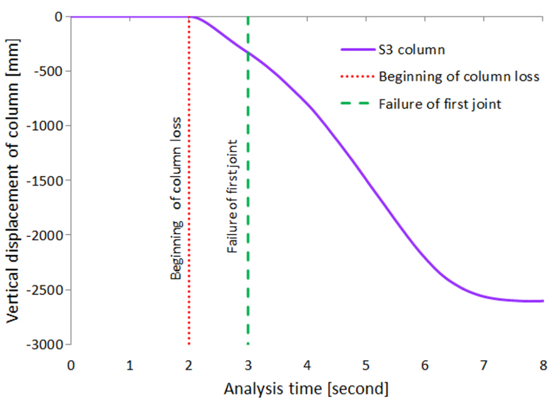
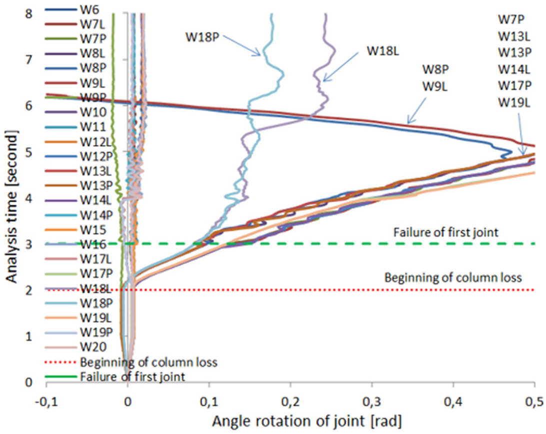
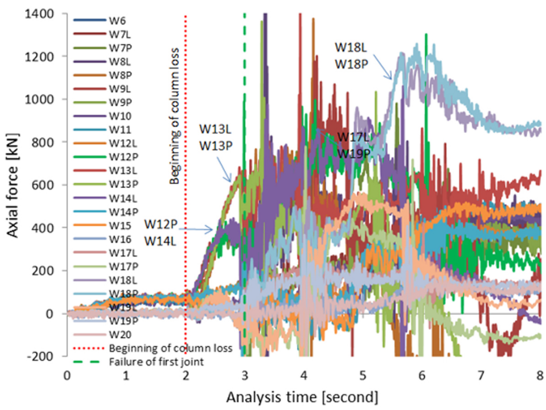


Fig. 7. Curve of vertical displacement of S3 central column vs. analysis time for 4B_F/E+10.50 case

The behavior of joints under analysis are presented in Fig. 8. First the rotations of joints in time of analysis in Fig. 8a are shown. At the time of persistent situation the values of rotation were negligible. After the removal of the column from the structure, a significant increase in rotations was observed in selected joints. The highest values were reached in W7P/W8/W13/W14L/W17P and W19L. In joints with extended end-plate directly connected to removed column i.e. W18 node the lower values of rotation were obtained. In other joints, small values were reached. The axial forces in the joints in the time analysis are shown in Fig. 8b. At the initial time of analysis, the axial tension force in the value of 100 kN was reached. After loss of the column, the significant increase in the axial tension force was reached in W13 and W12P/W14L. The first joint failure in the third second caused a significant change in the distribution of axial forces throughout the structure. Interestingly, the influence of extended end-plate joint on resistance mechanism (i.e. W18 joint) is approximately 5 seconds. The maximum axial force was obtained in the W18L joint at level 1258 kN.



(a)



(b)

Fig. 8. Development curves of analysis 4B_F/E+10.50 case: (a) time analysis-rotation of joints, (b) axial force-time analysis

The view of the structure in the selected time in the analysis is presented in Fig. 9a. The presence of extended end-plate joints at level +10.50 was clearly observed, where large deformation of the beam was obtained in the central part. The extended end-plate joints directly connected to removed column at the upper level of the structure were not damaged. The lower part of the internal bays was completely damaged, while the upper level with removed column survived. However, such behavior of the structure under extremal situation is undesirable because the people in the building would not survive such an event.

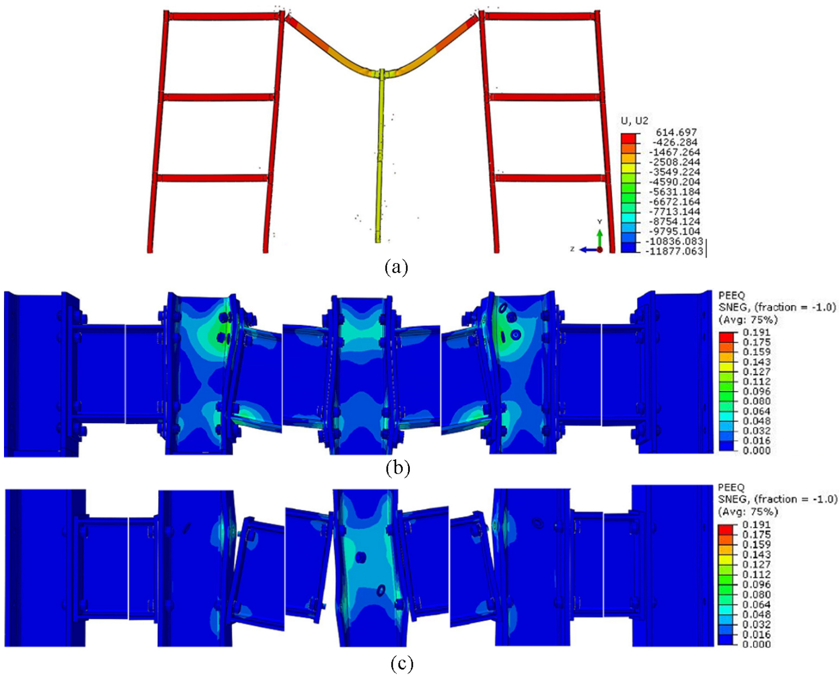


Fig. 9. View of: (a) whole structure 4B_F/E+10.50 after damage, (b) maps of strains at level +10.50 (from left W16 to W20), (c) joints strain at level +3.50 (from left W6 to W10) after failure

Significant deformations of internal joints were observed at level +10.50 (Fig. 9b) were observed. Fractures of bolts in the upper rows of the extended end-plate joints were also obtained as failure mode of the joints. The maximum strains in end-plate, column web and flanges were observed. All internal flush end-plate joints (Fig. 9c) in lower levels of the structure were damaged.

Despite the application of a mixed system with extended end-plate joints in the upper level, the required level of robustness was not obtained. Progressive collapse of the structure was observed under sudden loss of the internal column. The presence of end-plate joints in the structure does not mitigate damage. Although the upper level of structure was not completely damaged, the application of bolted extended end-plate joints did not achieve the expected effect of mitigate progressive collapse.

4.3. AR joints at + 3.50 level

The first application of AR joints was in the frame at the lower level of the structure. AR joints were applied to all double side joints. The vertical displacement of the removed column as a function of analysis time is presented in Fig. 10. The accidental situation as sudden column removal caused the instant linear increase of displacement, where the maximum was obtained near the fifth second. In further analysis, stabilization and stop of vertical translation were observed.

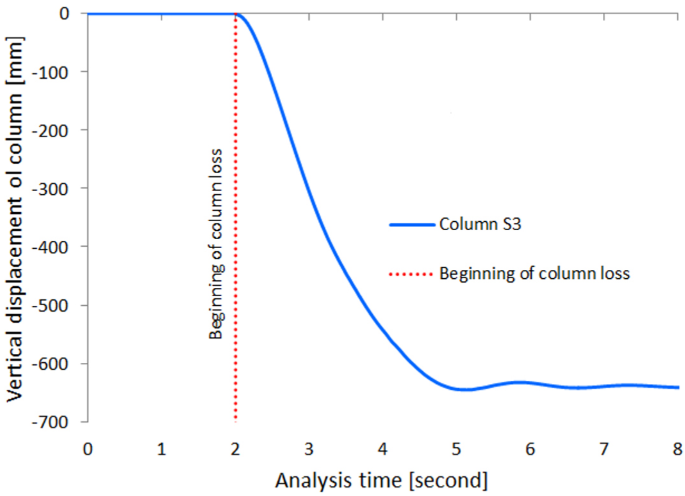
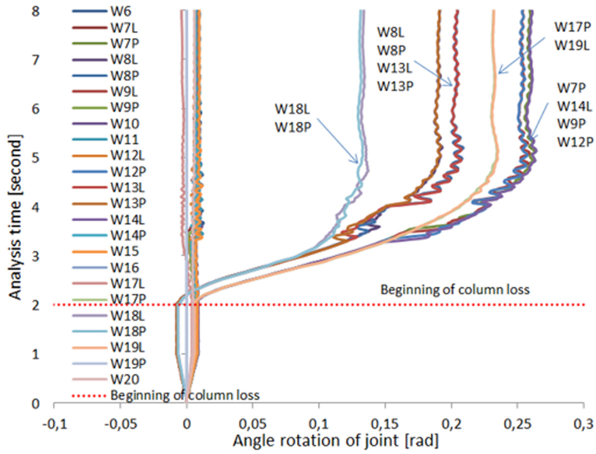


Fig. 10. Develepment of vertical displacement of S3 central column vs. analysis time for 4B_F/AR+3.50 case

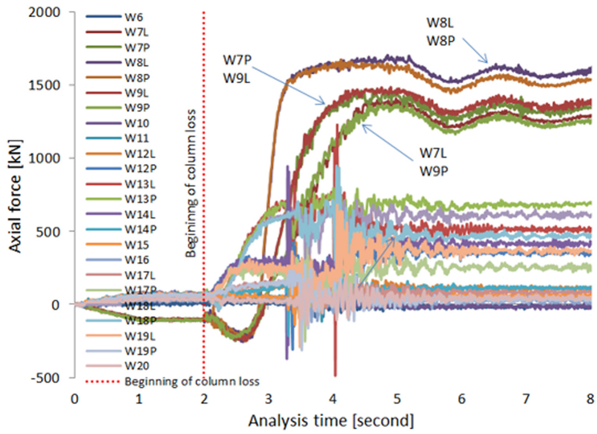
In Fig. 11 are shown the behavior of the joints at full time analysis of case 4B_F/AR+3.50. First (Fig. 11a) the development of joint rotations. To the 2nd second time, the values of rotations were neglected. In the next step of analysis, a significant increase of joint angle rotations was reached. The highest values of rotations were obtained in the joint directly connected to removed column. The other joints had small rotations. The course of development of axial force in joints is shown in Fig. 11b. In the permanent situation, the values of the axial forces were negligible. Interestingly in all nodes with AR joints in time to 2nd second the compression axial forces were obtained. After the advance of accidental situation, the visible increase of the axial forces was presented. The maximum axial force was in the joint of level +3.50 directly connected to removed column. The maximum tension force with values of 1705 kN and 1485 kN in W8 and W7P/W9L were obtained, respectively. The other joints transferred axial forces to a lesser extent. This behavior of the axial joints in the whole structure means that the load-capacity mechanism was reached at the lower level of the structure and the catenary action was developed.

The view of the whole structure after analysis with detailed views of joints at the final stage of analysis is presented in Fig. 12.

The loss of support in the S3 column leads to a significant vertical displacement of the S3 column with connected of both side beams. A view of joints after analysis and a map of strains



(a)

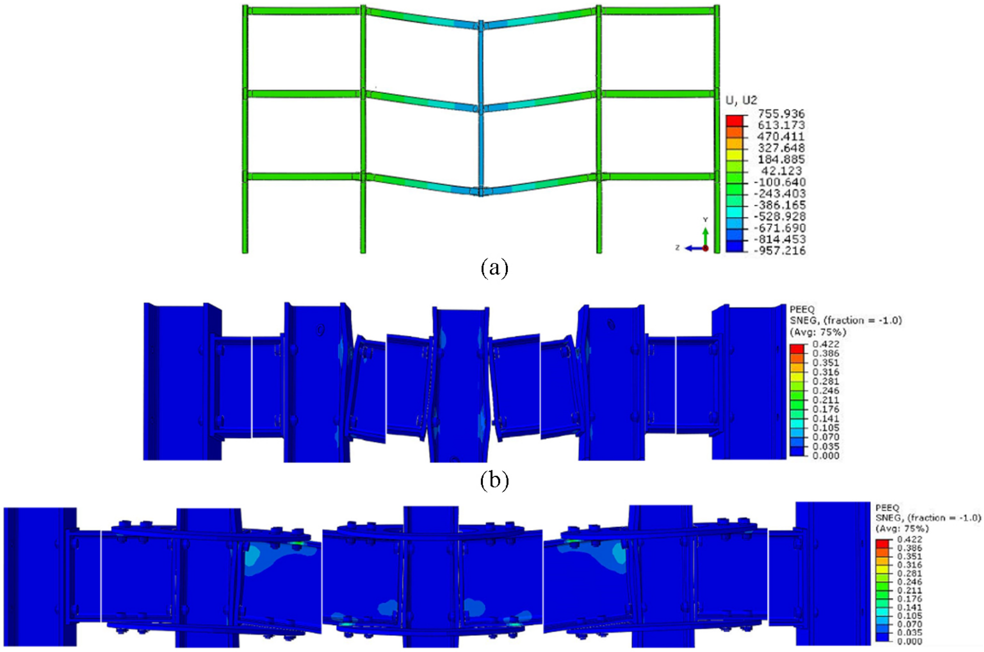


(b)

Fig. 11. Curves of analysis 4B_F/AR+3.50 case: (a) time vs. rotation of joints, (b) axial force vs. time

in joints are presented in Fig. 12b,c. At level +3.50 (Fig. 12c) a significant joint deformations were observed, especially in W7 to W9. The main deformations of the joints in the end-plate and rings were located. Different joint behavior were presented at level +10.50 (Fig. 12b) were presented, where joint failure of joint (e.g. W17 to W19) was reached. As a failure mode of these joints, a fracture of the tension bolts was observed in the inner joints.

Due to the application of AR joints on the lower lever of the structure, a required level of structural robustness was reached and a mitigation of progressive collapse was obtained. Through the presence of AR joints the catenary action was developed. Despite the dam-age of few bolted flush end-plate joints (fracture of bolts in joints) the structure mitigates the collapse and hung of lower level were obtained.



(c)

Fig. 12. View of: (a) whole structure with map of vertical displacement, (b) maps of strains of joints at level +10.50 (from left W16 to W20), (c) maps of strains of joints at level +3.50 (from left W6 to W10) at 8th second analysis

4.3.1. AR joints at + 10.50 level

Secondly, the application of flush bolt end plate joints with novel AR joints was analyzed at the highest level of the structure. The application of AR joints at the highest level in the structure allows neglect of the location of the source of the column removal under extreme situation.

The first results of the 4B_F/AR + 10.50 case analysis are presented in Fig. 13. The vertical translation of the S3 column in the analysis time under an accidental situation is shown. Under persistent situation in time to 2nd second the vertical translation of S3 column was constant. The extreme situation that occurs due to column loss creates a significant increase in vertical displacement of the S3 column. In time about 5th second the maximum value of translation was obtained. In further analysis the stabilization and stopping the vertical displacement was reached.

The detailed information about joints behavior with regard to rotations and axial forces are presented in Fig. 14. In time to 2nd second the rotations of joints were neglected. The occurrence of accidental situations leads to significant increase of rotations in joints directly connected to removed column. In time about the 5th second, the stopping development of rotations was reached to the end of analysis. The axial forces in persistent situation in time to 2nd second were negligible. The beginning of column loss caused the considerable advance of tension forces in the joints. Maximum tension forces were reached in the W18 and W17 /

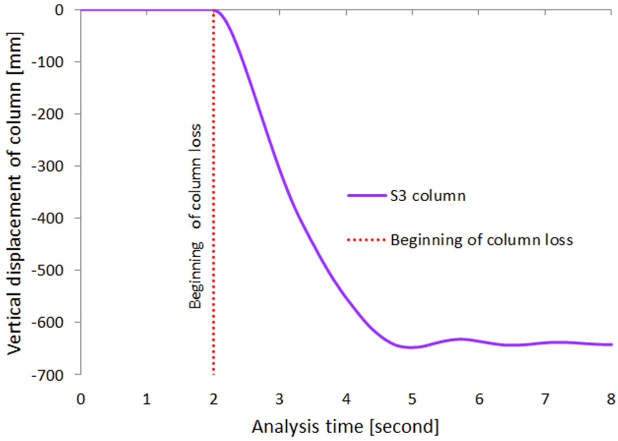
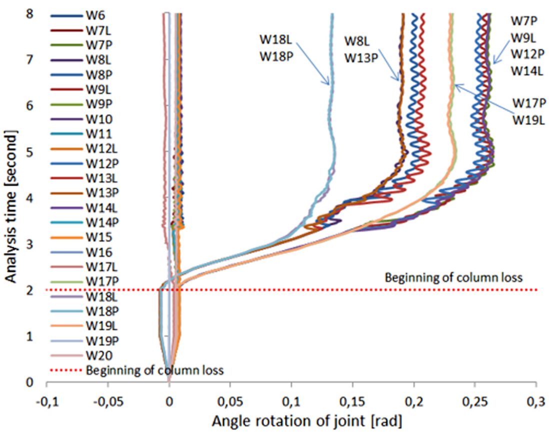


Fig. 13. Diagram of vertical displacement of S3 column – time analysis at 4B_F/AR+10.50 case

W19 joints. In the W18 nodes, the maximum tension force of about 1700 kN was obtained. In nodes directly connected to W18 node (i.e. W17P and W19L node) the maximum axial force of 1470 kN was reached. This distribution of axial force clearly indicated that the main robustness mechanism was developed to mitigate collapse at the highest level of the structure (with AR joints) was developed.

The general view of structure at the final time of analysis is shown in Fig. 15. The maximum vertical displacement was located in the central part of the frame, in place of the removed column. The structure survived despite the loss of the internal column. The joints strain maps (Fig. 15b,c) present the views of joints in the final step of analysis. The clear visible joints at level +3.50 were reached, where the internal joints were partially damaged. As the failure mode of internal flush end-plate joints, a fracture of the tension bolts was obtained. In the joints at level +10.50 a significant deformation of the inner joints was observed. It should be noted that in all AR joints, the full load capacity was maintained without damage to any parts of the joints.



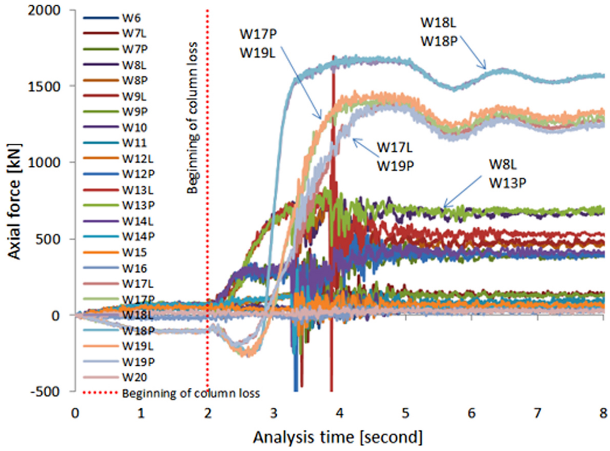
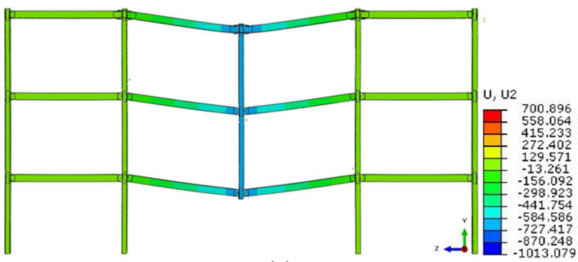
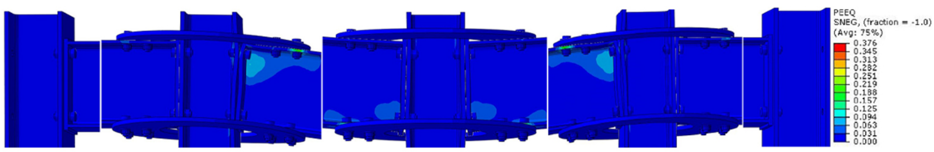


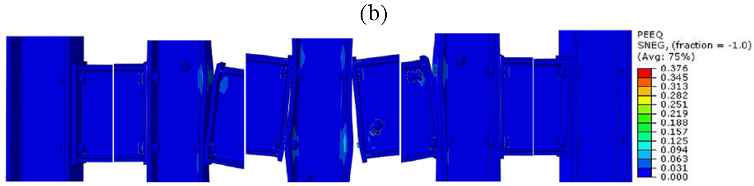
Fig. 14. Curves of 4B_F/AR+10.50 analysis: a) time analysis – angle rotation of joints, b) axial force vs. time analysis



(a)



(b)



(c)

Fig. 15. View of: (a) whole structure 4B_F/AR+10.50 after analysis, (b) maps ofstrains at level +10.50 (from left W16 to W20), (c) joints strain at level +3.50 (from left W6 to W10) after analysis

In the analysis of frame structure with AR joints in upper level +10.50 a required level of robustness under sudden column loss was obtained. The use of AR joints allows to develop a catenary mechanism leading to mitigate a progressive collapse of structure under accidental situation.

5. Summary and conclusions

The application of bolted extended end-plate joints combined with bolted flush end-plate joints led to progressive collapse of the structure. Higher main parameters such as the load capacity and rotational capacity of the extended end-plate joint did not stop the damage to the structure. The damage of flush end-plate joints was observed before the extended end-plate joints reached the full load capacity. Due to the presence of one row of bolts in the tension zone in the flush end-plate joint, the fracture of these bolts led to premature local damage to the joints and consequently to partial collapse of the central part of the structure. Only application of extended end-plate joints in all frame joints leads to obtain framed structures which can sustain progressive collapse.

The application of AR joints in the steel frame structure leads to mitigation of progressive collapse independently of the level of their application in the structure. Both, the application of AR joints in the lower or upper levels of structure allows the development of significant tension forces which play a major role in the robustness mechanism.

Based on the dynamic finite element analysis conducted of planar steel frames in an internal column loss scenario, the following conclusions can be drawn:

– The strategy based only on the change in the types of joints was partially proven. The progressive catastrophe of the frame structure under analysis was observed in cases of use of mixed bolted flush and extended end-plate joints at one level of the structure.
– To improve structural robustness and mitigate the progressive collapse of steel frame structures with bolted flush end-plate joints, the application of novel AR joints was recommended.

Further conclusions, with the design practise recommendation for shaping frames with end-plate joints, coming from authors' previous papers [12, 13], are as follows:

– It is recommended to use bolts one size/class higher than obtained in the design of the structure in the permanent design situation.
– To prevent joint damage by tearing the end-plate and for obtaining the appropriate ductility of the joint, a minimum 15 mm thick end-plate or equal to the thickness of the flange of the column to which the beam is connected should be used.
– To mitigate the collapse of the steel frame structure, the joints must be properly designed. In the design calculation of the resistance of the joints, the minimum axial force at 750 kN level should be included.
– As a minimum angle for bolted end-plate joint rotation at a value about 0,20 radians is recommended, to obtain the 'available' rotation capacity, to be compared to the 'required' rotation obtained in the global analysis.

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Kształtowanie węzłów stalowych ram wielokondygnacyjnych w celu zatrzymania katastrofy postępującej w odniesieniu do scenariusza utraty słupa wewnętrznego

Słowa kluczowe: analiza dynamiczna, symulacja numeryczna, nowy węzeł, odporność konstrukcyjna, nagła utrata słupa

Streszczenie:

Przedstawiono kontynuację wieloetapowego projektu badawczego, opartego na analizie eksperymentalnej i numerycznej, skupiającego się na odporności konstrukcji ramy stalowej z węzłami śrubowymi z blachą doczołową. Zastosowano zaawansowany model MES, zweryfikowany na podstawie wcześniej przeprowadzonych badań eksperymentalnych stalowych połączeń śrubowych i stalowej podkonstrukcji ramowej. Przeprowadzono dynamiczną analizę numeryczną płaskich konstrukcji stalowych w scenariuszu nagłej utraty słupa wewnętrznego przy użyciu oprogramowania Abaqus. Podjęto próbę zwiększenia odporności ram stalowych z węzłami śrubowymi z blachą doczołową zlicowaną na wypadek nagłego usunięcia słupa wewnętrznego na poziomie posadowienia. Zaprezentowano zastosowanie strategii polegającej na zmianie rodzaju połączeń na wybranym poziomie konstrukcji. Wprowadzono dwie główne zmiany konstrukcyjne poprzez zastosowanie połączeń z wystającą blachą czołową zamiast wpuszczonej blachy czołowej lub poprzez zastosowanie nowatorskiego połączenia z dodatkowymi pierścieniami. Aby sprawdzić zachowanie konstrukcji i ocenić jej odporność, zastosowano zmianę węzłów na dolnym i górnym poziomie konstrukcji. Zastosowanie śrubowych doczołowych węzłów z blachą wystającą tylko na jednym poziomie konstrukcji prowadzi do postępującego zawalenia się części konstrukcji. Zastosowanie nowatorskiego połączenia z dodatkowymi podwójnymi pierścieniami na jednym poziomie ramy w obu analizach prowadzi do uzyskania wymaganego poziomu odporności i zatrzymuje zawalenie się konstrukcji.

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