EXPERIMENTAL EVALUATION OF PROGRESSIVE COLLAPSE RESISTANCE OF STEEL MOMENT FRAME CONNECTIONS

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Abstract: During their lifetime, buildings may be subjected to accidental actions, like blast and impact, which can cause significant local damages. To limit the effects of such unforeseen events, the structure should retain the structural integrity by limiting the extent of damage and prevent its progress. Taking advantage of structure's inherent redundancy and available load paths, seismic resistant steel frames are considered appropriate to resist local damage without collapse. However, there are specific problems, which need to be considered when localized failures, particularly of columns, occur, i.e. large deformations and catenary response of beams. In this study, we investigated the performance of four steel frame beam-to-column connection types affected by the column removal. Acceptance criteria for progressive collapse events were proposed and compared to existing provisions.

1. Introduction

Steel framed structures are widely used in construction of multi-storey buildings, offering many several advantages over other types of constructions. According to EN 1990 [1], relevant design situations must be selected and applied, based on the conditions of use and also on the requirements concerning the performance of the structure. Most of ordinary buildings are designed for persistent and transient design situations, only. If the building may be subjected to seismic events, seismic design situations must be also considered. Even not considered in design, structures must resist accidental actions, e.g. fire, explosions or impact. In such situations, the structure must resist the action without being damaged to an extent disproportionate to the original cause. The disproportionality refers to the situation in which failure of one member causes a major collapse, with a magnitude disproportionate to the initial event; this is also known as progressive collapse [2]. Avoidance or limitation of potential damage may be done by several means, from elimination or reduction of hazard, to selection of a structural form which has low sensitivity to hazards and can survive adequately the accidental removal of an individual member or a limited part of the structure [1]. For the later option to be effective, it is necessary to provide continuity across the damaged area, and thus to allow the development of alternate loads paths. The alternate path method (AP) provides a formal check of the capability of the structural system to resist the removal of specific members, such as columns. The method does not require the characterization of the threat causing loss of the member. The AP method, with its emphasis on continuity and ductility, is similar to current seismic design practice ([2]). Seismic design procedures can be adopted as reference for progressive collapse design, but several modifications are necessary to accommodate the particular issues associated with progressive collapse. One such issue is the response of members and connections and their qualification for progressive collapse consideration. Thus, seismic resistant moment frames should be designed and detailed so that the required plastic deformations occur primarily in the beams or in the beam-to-column connections [3], [4].

If the structure is designed to dissipate energy in the beams, the connections of the beams to the columns should be strengthened and stiffened by using cover plates, haunches. Two examples for such type of connections are Welded Cover Plated Flange connection (CWP) and Haunched End Plate Bolted connection (EPH), respectively. These types of connections are widely used in practice, even they are not the most cost-effective and simple to execute connections. Alternatively, the section of the beam can be reduced at a distance away from the connection to obtain a favorable plastic mechanism. One such connection is the Reduced Beam Section welded connection, RBS.

If the structure is designed to dissipate energy in the connections, their flexural capacity is a fraction of that of the beams, and the connection is classified as partial strength connection. One example is the unstiffened extended end plate bolted connection (EP).

If for evaluation of seismic performance of connections and members cyclic loads with increasing magnitude are applied, without axial loading, in case of progressive collapse, the connection and member experiences one half cycle of loading, often in conjunction with a significant axial load, due to large deformations and catenary response. Unfortunately, only limited experimental research on the performance of steel frame connections under column removal scenarios were conducted so far.

When experimental testing is of concern, the use of connection components and sub-assemblage testing are most common solutions, allowing the complete evaluation of the response and validation of computational models. Yang and Tan [5] tested experimentally the performance of common types of bolted steel beam-column joints under a central-column-removal scenario. The results showed that tensile capacities of beam-column joints after undergoing large rotations usually control the failure mode and the formation of catenary action. Xu and Ellingwood [6] investigated the performance of steel frames with partially restrained connections fabricated from bolted T-stubs following damage to load-bearing columns. They reported that frames with strong T-stub connections can resist collapse in damage scenarios involving the notional removal of one column, whereas the robustness of the frames with weak T-stub connections is questionable. Structural system testing, involving entire building systems or parts of them is expensive and difficult, and therefore very few tests have been performed so far. Astaneh-Asl et al., [7] experimentally studied the ability of a typical steel structure to resist progressive collapse in the event of the loss of a column due to a blast and attempted to establish the failure modes. Another study [8] confirmed that a retrofit scheme in which cables are added to the sides of beams could be used to develop larger catenary action with a higher safety factor. A very recent study ([9], [10]) investigated the capacity of a 3D steel frame structure with extended end-plate bolted beam-to-column connections to support the loss of a column. The results showed that connections possessed sufficient strength to resist the catenary forces developed in the beams, with a rotation capacity over 200 mrad.

In the present study, we investigated the potential of four typical steel frame beam-to-column connections to develop catenary action under column loss and how these connections perform when subjected to large deformation demands. Connections were designed and detailed to meet the seismic design requirements for special moment frame connections. The specimens, having two spans of 3.0 m each, were subjected to a monotonic loading applied to the top of the removed central column until complete failure. The study is part of a research program devoted to the design of structures to sustain extreme loading events without collapse [11].

2. Specimens and test setup

Figure 1.a shows the plan layout of the case-study building. The bays and spans each measure 8.0 m. Structure was calculated for the effect of gravity loads (permanent and variable actions) and lateral loads (wind and seismic actions), using the Eurocodes. The dead and live loads were each 4.0 kN/m^2 and the reference wind pressure was 0.5 kN/m^2. The structures were located in a low seismicity area, characterized by a design ground acceleration a_g of 0.08 g and a control period T_C of 0.7 s. It should be noted that, the seismic intensity and the response spectrum used in design were those given in the Romanian Seismic Code, P100-1/2006 [12]. High dissipative structural behavior was considered using a behavior factor q of 6.5. An inter-story drift limitation of 0.008 of the story height was used for the seismic design at the serviceability limit state. Various loading cases, including seismic design loads, were used for the structural design of members, connections and details, using the relevant Eurocode parts. No particular accidental design situation was considered in design. The highlighted area (Figure 1.a) indicates in the perimeter frame extracted for investigation. Due to space limitation in the laboratory, the frame was scaled down from 8.0 m span to 3.0 m span. Each specimen consists of two steel beams and three columns and have a length of 6.0 m between the center lines of the marginal columns. Figure 1.b shows the experimental frame, with the test set-up, the specimen and the 100 tones actuator. To consider the restraint from the surrounding elements in the reference structure, horizontal restraints were provided by a strong reaction wall, on one side, and by a brace system, on the other side. Out of plane restraints were also provided along the span of the beams and at the central column. Note that the in-plane rotation of the central column was not restrained. In this study, four specimens were constructed and tested, each with a different type of beam-to-column connections but with the same sections for members. In all specimens, beam section was IPE220 while the column section was HEB260 with flanges reduced to 160 mm width. Steel material in plates and profiles was S275J0 and bolts were class 10.9 and 8.8.

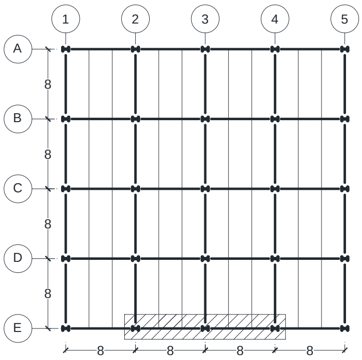
First specimen (CWP) used welded cover plated flange connections. The top and bottom beam flange cover plates are 12 mm thick, width is 130 mm and length is 150 mm. Cover plates were welded with complete joint penetration CJP groove welds to the column flange, and with fillet welds to the top and bottom beam flanges. Beam web connection was made using a bolted shear tab, reinforced with filet welds to the beam web and column flange.

Second specimen (EPH) used extended end plate bolted connections and bottom haunches, with 20 mm thick end plate and six rows of bolts M20 class 10.9. The height of the haunch is 110 mm while the length is 150 mm (same length as cover plates in CWP).

Third specimen (RBS) used reduced beam section connections, with circular radius cuts in both top and bottom flanges of the beam. Dimensions of the reduced zones, calculated according to ANSI/AISC 358-10 [4], are *a*=66 mm, *b*=150 mm and *c*=22 mm. Welds of beam flanges to the column are CJP groove welds.

Forth specimen (EP) used unstiffened extended end plate bolted connections, with 16 mm thick end plate and five rows of bolts M16 class 10.9.

Figure 2.a-d shows the construction details of the connections for the specimens and Figure 2.e the moment-rotation characteristics of the connections and the classification according to EN 1993-1-8 [13]. The instrumentation included displacement transducers for vertical deflection and joint rotation. For estimating the axial force in beams, additional transducers were placed on horizontal elements of the in-plane restraining system. As these elements remain elastic, it is easy to estimate the intensity of the axial force in the beams during the test.



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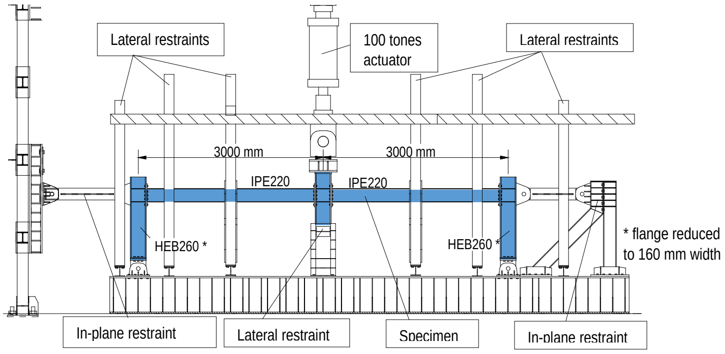
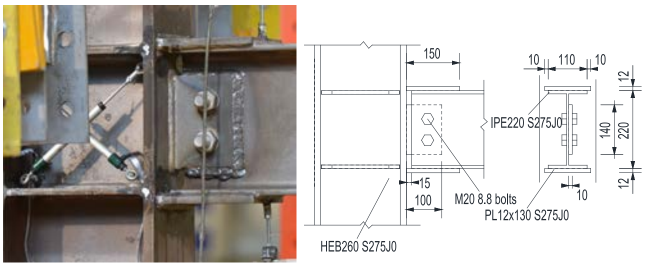
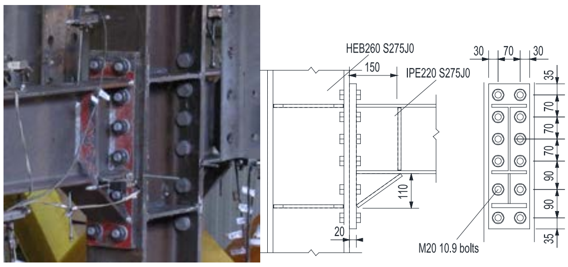


b)

Fig. 1: Plan layout of the reference multi-storey building with the position of the specimens extracted for testing (a) and test set-up (b)



a) Specimen CWP



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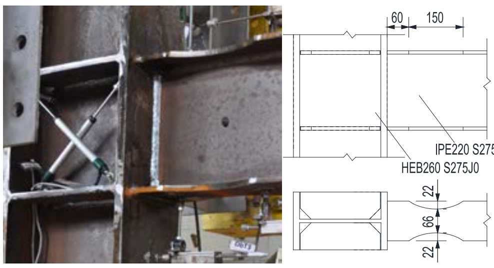
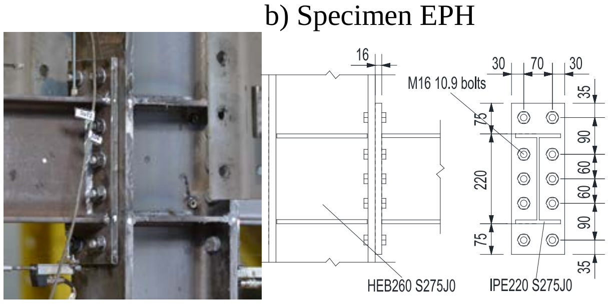
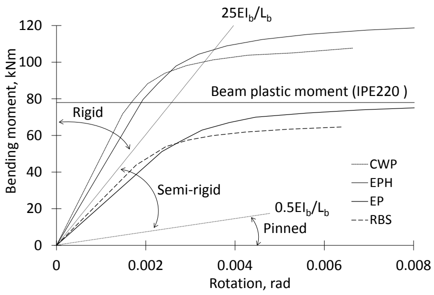


c) Specimen RBS



d) Specimen EP



e) moment-rotation characteristics of the connections

Fig. 2: Details of test specimens (dimensions are in mm)

Before the test, bolts and material coupons from beams, columns, and plates were tested to evaluate the mechanical characteristics of materials. The mechanical characteristics are listed in Table 1. The vertical force was applied on top of the central column and was gradually increased until failure of the specimen was reached. The yield displacement, D_y, and yield force, F_y, were calculated according to the ECCS method. Data from displacement transducers located at beams ends were used to capture the joint rotations R1, R2 and R3, see Figure 3. The results from the four experimental tests are presented in the subsequent sections.

Table 1: Average characteristic values for steel profiles, plates and bolts

Element	f_y (N/mm²)	f_u (N/mm²)	ε_y (%)	A_{gt} (%)
Beam flange IPE220, t = 9.2 mm	351	498	0.17	15.0
Beam web IPE220, t = 5.9 mm	370	497	0.18	15.0
Column web HEB 260, t = 10 mm	402	583	0.19	12.9
Column flange HEB 260, t = 17.5 mm	393	589	0.19	13.3
End plate, t = 16 mm	305	417	0.15	17.1
End plate, t = 20 mm	279	430	0.13	12.7
Cover plate, t = 12 mm	315	455	0.15	16.3
Shear tab, t = 10 mm	314	416	0.15	16.7
Bolt, M20 class 10.9	920*	1085	1.75*	12.2
Bolt, M16 class 10.9	965*	1080	1.76*	12.0
Bolt, M20 class 8.8	672*	825	1.78*	12.3

Note: f_y - yield strength; f_u- ultimate strength; ε_y- yield strain; A_{gt}- Total elongation at maximum stress
* 0.2% offset yield point

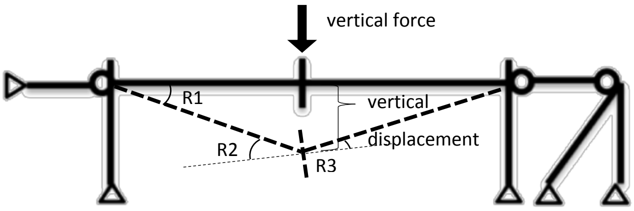


Fig. 3: Schematic representation of the experimental setup and beam rotation notations

3. Experimental results

3.1 Full strength connections

3.1.1 Welded cover plated flange connections CWP

Figure 4.a shows the vertical force vs. middle column displacement for specimen CWP. Three stages can be identified on the curve, i.e. elastic, flexural and catenary, without clear point of demarcation but with some zones of interaction. At the initial loading stage, the specimen was in elastic and the applied load increased almost linearly because the connection was stiff and no slippages were possible in the connection components. The local buckling of the top flange of the right beam, near the connection with the central column, indicated the initiation of yielding and, at this point, the displacement was 35 mm and the applied force was 147 kN. After yielding started, the second stage was initiated and the flexural stiffness started to decrease. The maximum applied force at pure flexural stage (no axial tension in beams) was 201 kN and the corresponding vertical displacement was 115 mm. To note that up to 115 mm vertical displacement, the beams were in compression (axial force was negative), with a maximum compressive force of 17 kN, indicating a very low arching behavior in the structure, see Figure 4.b. Up to a vertical displacement of 210 mm, most of the applied load was still resisted by the flexural capacity but the catenary action started to develop, and the axial force in beams increased to 214 kN. At the end of this stage, which can be called flexural-catenary

stage, the bending moment reached the maximum value, i.e. 153 kNm, see Figure 4.b. After this point, the flexural resistance started to decrease while the catenary action became more predominant. The stiffness continued to increase until the vertical displacement and applied force reached 519 mm and 603 kN, respectively. At this point, due to large tensile force in beams, the end connection of the right beam (near the central column) started to fracture, first in the bottom cover plate, which completely separated from the column flange, followed immediately by a large fracture in the shear tab. This fracture was accompanied by a large drop in the applied force. The axial force in beams reached a maximum value of 1230 kN, then started to decrease, see Figure 4.b. Because the fracture was quite violent, the test was halted and the transducers located near the central column were detached for safety reasons. The test was then resumed and the applied load started to increase again, until the top cover plate fractured and the beam was completely separated from the column, see Figure 4.c. When the test was stopped, the ultimate vertical displacement reached 586 mm. Figure 4.d shows the rotation in sections R1, R2 and R3. It may be seen that up to a vertical displacement of 210 mm (end of flexural-catenary stage), the rotations were almost identical. After this point, due to the rotation of the central column, the two beam ends connected to the central column recorded different rotations, i.e. R2 and R3. At the peak applied load, the maximum rotation recorded in the connections was R3=0.193 rad.

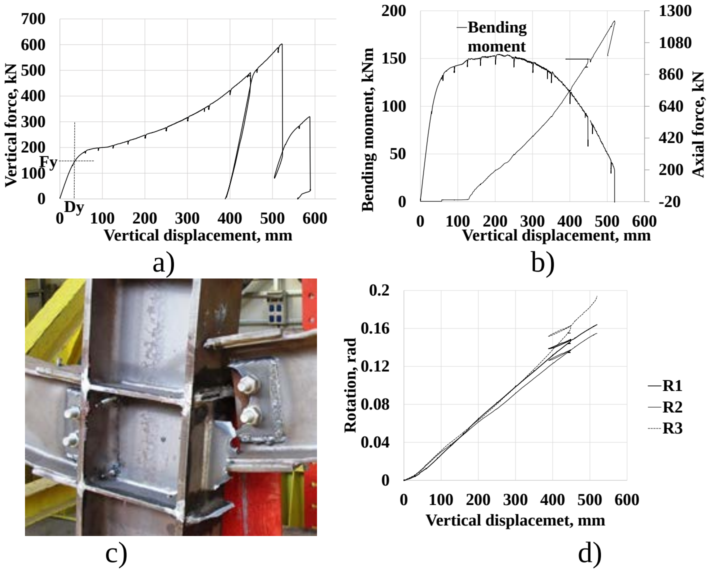
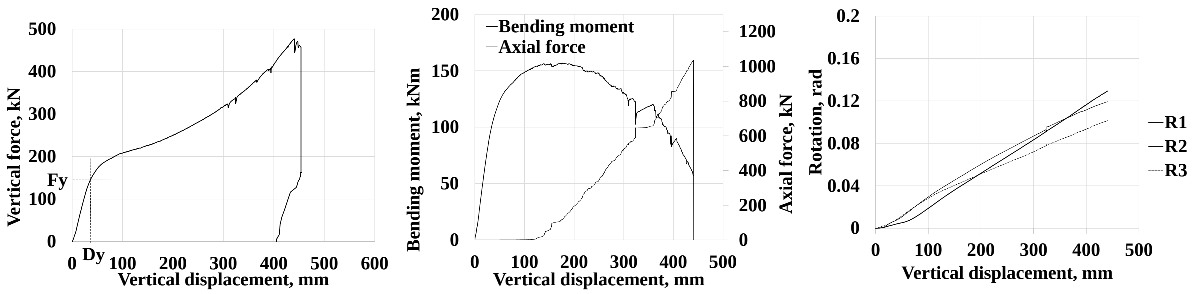


Fig. 4: Specimen CWP: a) vertical force vs. vertical displacement; b) bending moment and axial force vs. vertical displacement; c) failure mode; d) beam end rotation vs. vertical displacement

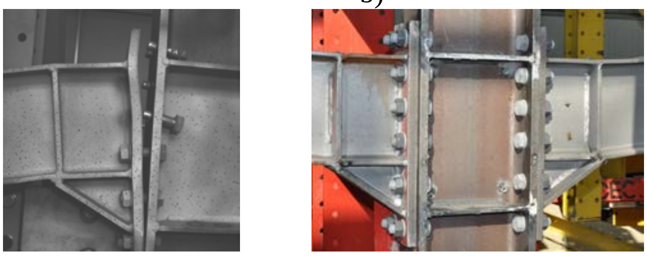
3.1.2 Bottom haunch end plate bolted connections EPH

Figure 5.a shows the vertical force vs. middle column displacement for specimen EPH. As in case of specimen CWP, three stages can be identified on the curve, i.e. elastic, flexural and catenary. At the initial loading stage, the specimen was in elastic and the applied load increased almost linearly because the connection is rigid and there were virtually no slippage in the connection. The local buckling of the top flange of the right beam, near the connection with the central column, indicated the initiation of yielding and, at this point, the displacement was 37 mm while the applied force reached 147 kN. After yielding started, the second stage was initiated and the flexural stiffness started to decrease. The maximum applied force at pure flexural stage was 212 kN and the corresponding vertical displacement was 110 mm. To note that up to 110 mm vertical displacement, the axial force in beams was nearly zero, indicating there is no arching behavior in the structure, see Figure 5.b. Up to a vertical displacement of 171 mm, most of the applied load was still resisted by the flexural capacity but the catenary

action started to develop, and the axial force in beams increased from zero to 235 kN. At the end of this flexural-catenary stage, the bending moment reached the maximum value, i.e. 156 kNm, see Figure 5.b. After this point, the flexural resistance started to decrease while the catenary action became more predominant. The stiffness continued to increase until the vertical displacement and applied force reached 440 mm and 477 kN, respectively. At this point, due to large tensile force in beams, the right connection of the right beam failed due to the fracture of first three bolt rows in tension, see Figure 5.d. Next two bolt rows also suffered plastic deformations but did not fracture because the test was stopped due to safety reasons. The axial force in beams reached a maximum value of 1035 kN, see Figure 5.b. Figure 5.c shows the rotation in sections R1, R2 and R3. It may be seen that up to a vertical displacement of 171 mm (end of flexural-catenary stage), the three rotations were very similar. After this point, due to the rotation of the central column, the two beam ends connected to the central column recorded different rotations. At the peak applied load, the maximum rotation recorded in the connections was R1=0.130 rad. To note that when the right connection of the right beam failed due to fracture of the bolts, the beams connections to the central column showed no visible damages, see Figure 5.e. The reason for this behavior is the non-symmetrical arrangement of the connection. Thus, under sagging bending, the connection has more capacity in tension because more bolt rows are engaged. In case of hogging bending, there are less bolt rows active in tension.



a) b) c)



d) e)

Fig. 5: Specimen EPH: a) vertical force vs. vertical displacement; b) bending moment and axial force vs. vertical displacement; c) beam end rotation vs. vertical displacement; d)- e) failure mode

3.2 Partial strength connections

3.2.1 Reduced beam section connections RBS

Figure 6.a shows the vertical force vs. middle column displacement for specimen RBS. Three stages can be identified on the curve, i.e. elastic, flexural and catenary. At the initial loading stage, the specimen was in elastic and the applied load increased almost linearly because the connection was stiff and no slippages were possible in the connection components. The yielding initiated at a vertical displacement of 33 mm, while the applied force amounted 110 kN. After yielding started, the second stage was initiated and the flexural stiffness started to decrease. The maximum applied force at pure flexural stage (no tensile axial force in beams) was 195 kN and the corresponding vertical displacement was 200 mm. To note that up to this point the axial force in beams was nearly zero, indicating no arching behavior in the structure,

see Figure 6.b. Up to a vertical displacement of 250 mm, most of the applied load was still resisted by the flexural capacity, but the catenary action started to develop, and the axial force in beams increased from zero to 100 kN. At the end of this flexural-catenary stage, the bending moment reached the maximum value, i.e**.** 147 kNm, see Figure 6.b. After this point, the flexural resistance started to decrease while the catenary action became more predominant. The stiffness continued to increase until the vertical displacement and applied force reached 480 mm and 401 kN, respectively, when, at the marginal connection, due to large tensile forces in beams, a crack was initiated in the top flange of the reduced beam zone. The fracture then propagated in the web (see Figure 6.c) and the test was stopped due to safety reasons, because the rupture was quite violent. The maximum axial force recorded in the beams was 753 kN (Figure 6.b). Figure 6.d shows the rotation in sections R1, R2 and R3. It may be seen that the three rotations were almost identical up to the end of the test indicating that the rotation of the central column was negligible. At the peak applied load, the maximum rotation recorded in the connections was R3=0.172 rad.

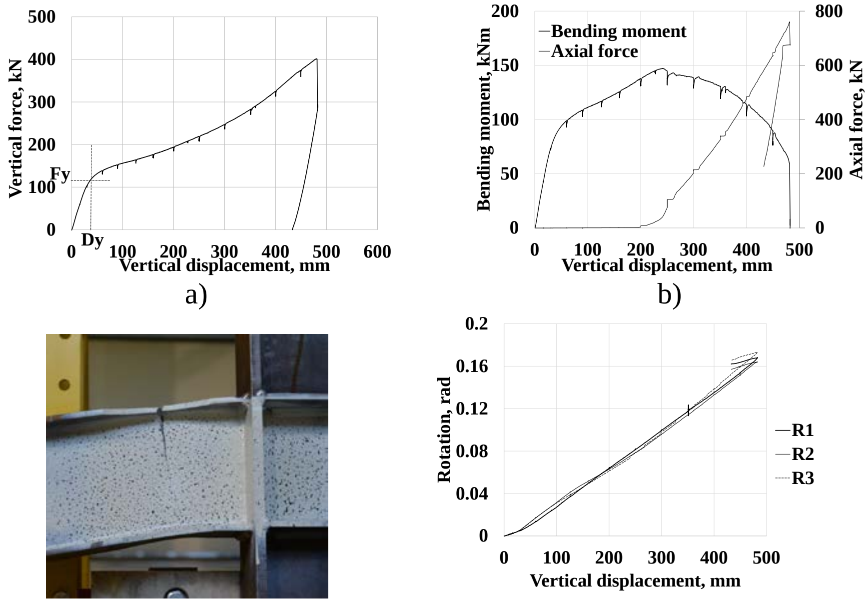
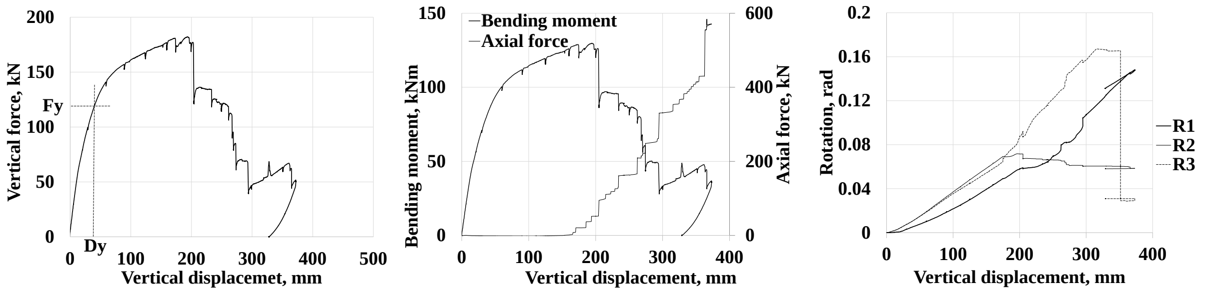


Fig. 6: Specimen RBS: a) vertical force vs. vertical displacement; b) bending moment and axial force vs. vertical displacement; c) failure mode; d) beam end rotation vs. vertical displacement

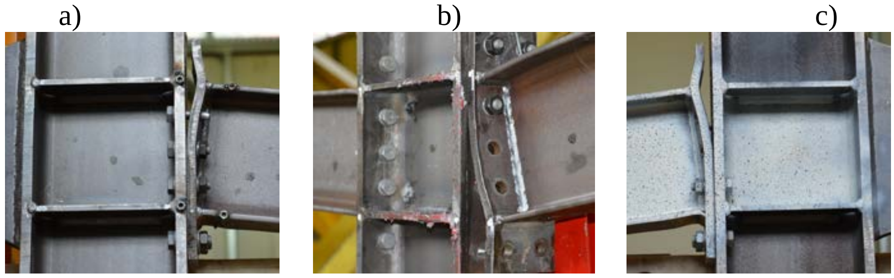
3.2.2 Unstiffened extended end plate bolted connections EP

Figure 7.a shows the vertical force vs. middle column displacement for specimen EP. Compared to the first three specimens (full strength specimens CWP and EPH and partial strength RBS), this specimen did not show any distinctive catenary behavior. The initiation of yielding was due to bending in the end plate of the right beam, near the connection with the central column, at a vertical displacement of 39 mm and an applied force of 117 kN. After yielding started, the second stage was initiated and the flexural stiffness started to decrease. The maximum applied force at pure flexural stage (no tensile force in beams) was 175 kN and the corresponding vertical displacement was 154 mm. To note that up to 154 mm vertical displacement, the axial force in beams was nearly zero, indicating there is no arching behaviour in the structure, Figure 7.b. At a maximum applied force of 182 kN and a corresponding vertical displacement of 194 mm the specimen suffered first failure due to the fracture of the bottom external bolt row of the right beam connection near the central column. The fracture was caused by the flexural action, with a minor contribution from axial load. The test continued and the failure propagated to second bolt rows within the same connection. In the same time,

also the top second bolt row of the right beam connection away from the central column failed due to excessive tensile forces. The specimen finally failed when three bolt rows from left and right connections of the right beam, and two bolt rows from the left connection of the left beam were fractured. At this final stage, the axial force in beams reached 571 kN. The main cause of the premature failure is the insufficient tying resistance of the connection which led to an insufficient rotation capacity required to develop significant catenary action. Figure 7.c shows the rotation in sections R1, R2 and R3. It may be seen that up to a vertical displacement of 175 mm, the rotations R2 and R3 were almost identical, suggesting the central column remained on vertical position. After this point, due to the rotation of the central column, the rotation concentrated in the right connection R3, while R2 started to reduce. At the peak applied load, the rotation recorded in the connections was R3 = 0.079rad. The rotations beyond this point cannot be considered acceptable, because the resistance started to decrease, indicating the progressive collapse is imminent.



a) b) c)



d) e) f)

Fig. 7: Specimen EP: a) vertical force vs. vertical displacement; b) bending moment and axial force vs. vertical displacement; c) beam end rotation vs. vertical displacement; d)-f) failure mode

4. Conclusions

An experimental program was conducted at PU Timisoara to investigate the performance of four types of moment resisting connections when a column is removed. The specimens were extracted from a six-story moment resisting frame structure, designed to meet the seismic design requirements for special moment frames. The two-span specimens were tested under a monotonically increased vertical force, applied on top of the central column using a displacement control, until failure of the specimens was reached. The specimens designed with full strength connections (CWP and EPH) showed a good response, experiencing large deformation capacity and catenary response. RBS specimen, where the beam is reduced near the column face, also developed large axial forces in beams before failure, with an ultimate rotation over 170 mrad. The specimen with unstiffened end plate bolted connections showed the poorest behavior and failed before developing significant catenary action in beams. The combined contribution from direct applied axial loads, prying effects and bending due to flexible end plate, finally contributed to bolt fracture, with an ultimate rotation of 79 mrad. Without significant strengthening of the bolts, this connection cannot be used for developing catenary forces and therefore should be used for flexural-based design only.

The results and conclusions presented above are based on a limited number of tests. Therefore, additional studies are necessary to confirm or to adjust the findings. Thus, an extensive numerical parametric study is under development, aiming at investigating the factors that may have a significant impact on the catenary response of connections under extreme loading conditions, e.g. influence of beam depth on the rotation limit, beam web-to-column detail (for welded connections) and strain rate.

Acknowledgments

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References

[1] EN 1990 “Eurocode - Basis of structural design”, CEN, 2002.
[2] NISTIR 7396 “Best Practices for Reducing the Potential for Progressive Collapse in Buildings, National Institute of Standards and Technology”, Oakland, CA, 2007.
[3] EN 1998-1 “Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings”, CEN, 2004.
[4] ANSI/AISC 358-10, “Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications”, AISC, 2014.
[5] Yang B, Tan KH. “Experimental tests of different types of bolted steel beam-column joints under a central column removal scenario”, *Eng. Structures*, 54, 112-130, 2013.
[6] Xu G, Ellingwood B. “Disproportionate collapse performance of partially restrained steel frames with bolted T-stub connections”, *Engineering Structures*, 33, 32–43, 2011.
[7] Astaneh-Asl A, Jones B, Zhao Y, Hwa R. “Floor catenary action to prevent progressive collapse of steel structures”, (Report No. UCB/CE-Steel-03/2001). Department of Civil and Environmental Engineering, University of California, Berkeley, USA, 2001.
[8] Tan S, Astaneh-Asl A. “Cable-based retrofit of steel building floors to prevent progressive collapse”. Berkeley: University of California, 2003.
[9] Dinu F, Marginean I, Dubina D, Petran I. “Experimental testing and numerical analysis of 3D steel frame system under column loss”, *Eng. Structures*, 113, 59-70, 2016.
[10] Dinu F, Dubina D, Marginean I. “Improving the structural robustness of multi-story steel-frame buildings”, *Structure and Infrastructure Eng.*, 11(8), 1028-1041, 2015.
[11] CODEC. “Structural conception and collapse control performance based design of multistory structures under accidental actions” (2012–2016), Exec. Agency for Higher Ed., Resc., Development and Innov. Funding, Romania, PN II PCCA 55/2012.
[12] P100-1 (2006). Seismic design code. Part 1: “Earthquake resistant design of buildings P100-1/2006” (Buletinul Constructiilor, No. 12-13). INCERC, Bucharest, Romania, Ministry of Transport, Construction and Tourism MTCT (in Romanian), 2006.
[13] EN 1993-1-8 “Eurocode 3 Design of steel structures - Part 1-8: Design of joints”, 2005.