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COLLAPSE PREVENTION DESIGN CRITERIA FOR MOMENT CONNECTIONS IN MULTI-STORY STEEL FRAMES UNDER EXTREME ACTIONS

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ABSTRACT

The paper investigates the role of beam-to-column connections in mitigation the progressive collapse of multi-story steel frame buildings in case of column loss. On this purpose, a set of moment frames with different beam-to-column connections is designed following seismic design criteria for highly dissipative structures to resist seismic actions. Applied Element Method through nonlinear dynamic analyses is applied to predict the structural response, after the loss of one or more columns. The model was calibrated to match experimental data from full scale tests on bolted end plate connections under bending moment and different levels of tensile axial force.

Keywords: column loss, robustness, tying, moment connection, steel frame, rotation capacity

INTRODUCTION

Multi-story buildings should be able to withstand the loss of a primary load bearing component without progressive collapse. According to ASCE (2006), Progressive Collapse is defined as "the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it". Providing resistance to progressive collapse under an abnormal event is a measure of the structural robustness and relies primarily on resistance of key elements and continuity between structural elements. Key elements are defined as structural elements whose notional removal would cause collapse of an unacceptable extent. They should therefore be designed for accidental loads, which are specified in several standards (EN 1991-1-7, 2006). This is a threat-specific approach. However, the random nature of the hazards makes difficult to provide an appropriate level of protection against progressive collapse when the design is based solely on the resistance of key elements. When resistance to progressive collapse cannot rely entirely on strengthening the critical members, it is necessary to provide continuity across the damaged area, and thus to allow the development of alternate loads paths. The alternate load 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, so it is a non threat-specific approach. The approach has the advantage of promoting structural systems with ductility, continuity and energy absorbing properties that are very effective in preventing progressive collapse. One example is the moment frame system designed to resist seismic actions. Thus, when such a structure is affected by the loss of a column, the flexural resistance of the beams or their connections to the columns ensures the transfer of the loads through alternative paths. Moreover, moment resisting frames can span over the location of the affected part through a Vierendeel truss girder action. The AP method is therefore consistent with the seismic design approach. However, when compared with seismic design, the loss of a column may lead to significant differences, like the large axial forces in the beam-to-column connections and therefore the connections must be designed for the combined effects of bending and axial load. Thus, Marchand (2005) proposed that, for connections in which catenary action may develop, the design should be done for two limit states: 1) developing beam plastic moment and 2) developing beam axial tension capacity.

The application of design rules from EN 1993-1-8 to beam-to-column joints in bending is limited to joints in which the axial force N_{Ed} in the connected member does not exceed 5% of the design resistance $N_{pl,Rd}$ of its cross-section. If the axial force N_{Ed} in the connected beam exceeds 5% of the design resistance, $N_{pl,Rd}$, the following conservative method may be used (see eq. 1):

$$\frac{M_{j,Ed}}{M_{j,Rd}} + \frac{N_{j,Ed}}{N_{j,Rd}} \leq 1 \tag{1}$$

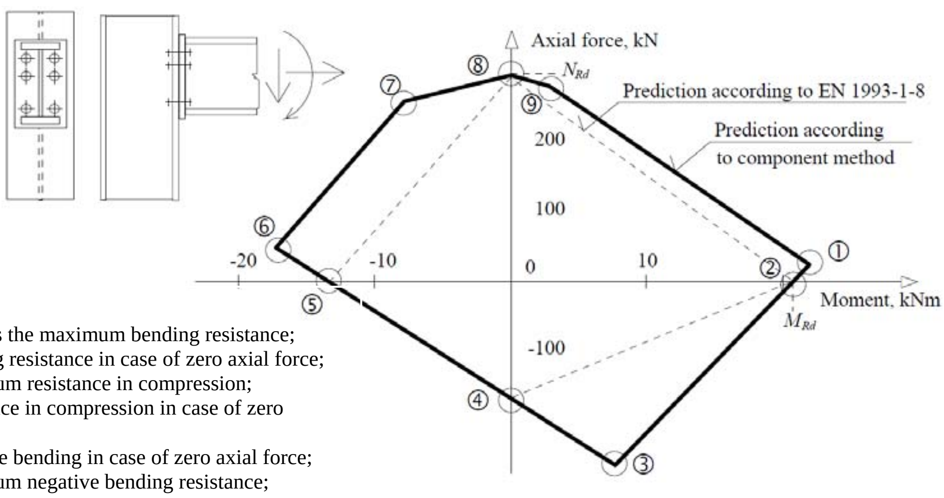
where:

$M_{j,Rd}$ is the design moment resistance of the joint, assuming no axial force;

$N_{j,Rd}$ is the axial design resistance of the joint, assuming no applied moment;

$M_{j,Ed}$, $N_{j,Ed}$ are the bending moment and axial force applied to a joint.

The method proposed in EN 1993-1-8 was considered quite questionable, and an improved design procedure, based on the component method concept, has been developed to predict the response of steel joints subjected to combined axial loads and bending moments (Cerfontaine, 2003). Demonceau (2008) extended the design procedure developed by Cerfontaine to composite joints and validated through comparisons to the experimental test results. Sokol et al. (2002) developed a design model of end plate joints loaded by combination of bending moment and normal force (Fig. 1).



Legend:
Point 1 represents the maximum bending resistance;
Point 2 is bending resistance in case of zero axial force;
Point 3 is maximum resistance in compression;
Point 4 is resistance in compression in case of zero bending moment;
Point 5 is negative bending in case of zero axial force;
Point 6 is maximum negative bending resistance;
Point 7 is point of activation of second bolt row;
Point 8 is resistance in axial tension;
Point 9 is point of activation of second bolt row.

Fig. 1 Moment – axial load interaction curve, prediction according to EN 1993-1-8 is marked by dotted line; the component method is marked as a solid line (Sokol et al., 2002) da Silva et al. performed experimental work on beam-to-column joints in order to extend the philosophy of the component method to deal with the combined action of bending moment and axial force (da Silva et al. 2004). For the chosen flush end-plate joint, a reduction of the moment resistance was noted for tensile axial force below 20% of the beam plastic resistance. A generalized component-based model for semi-rigid beam-to-column connections including axial force versus bending moment interaction was developed by Del Savio et al. (2009). Liu studied the retrofitting of steel construction and improvement of their catenary ability through strengthening the beam-to-column connection (Liu, 2010). Sadek et al. investigated the response of steel beam-column assemblies with moment connections under monotonic loading conditions simulating a column removal scenario (Sadek et al., 2013).

The paper investigates the behavior of multi-story steel frame buildings considering different local damage scenarios. On this purpose, a set of moment frames of different beam-to-column joints are designed following seismic design criteria for highly dissipative structures to resist seismic actions. Nonlinear dynamic analyses are applied to predict the structural response, after the loss of one or more columns. Models are calibrated to match the experimental data from full scale tests on bolted end plate connections under bending moment and different levels of tensile axial force. Robustness criteria are obtained, taking as reference the initiation of catenary action, after the attainment of ultimate bending strength in beams.

PROGRESSIVE COLLAPSE ANALYSIS

Calibration of numerical model

The progressive collapse of multi-story steel frame buildings has been investigated using the advanced non-linear structural analysis software ELS (2010). In order to calibrate the numerical models for beam-to-column connections used in the progressive collapse analysis, experimental tests carried out at the University of Coimbra, Portugal (da Silva et al., 2004) were used as reference. Nine experimental tests were carried on flush end-plate configurations. They comprised several combinations of bending and axial forces and consisted of the application of a fixed level of axial tension or compression and the subsequent application of a bending moment, incremented up to failure of the connection (Fig. 2.a). For first specimen, FE1, only bending moment was applied while for the other 7 specimens (FE3, FE4, FE5, FE6, FE7, FE8 and FE9), constant axial forces of, respectively, -4%, -8%, -20%, -27%, -20%, +10% and +20% of the beam plastic resistance were applied to the beam. Fig. 2.b plots the moment - rotation curves for the tested specimens.

The ELS beam-to-column assembly model is shown is Fig. 3.a. ELS utilizes a non-linear solver based on the Applied Element Method AEM (Tagel-Din and Meguro, 2000). AEM elements are connected together through a series of springs that connect adjacent element faces. The generation of these springs is automatically performed in the software. These springs represent continuity between elements and reflect the different material properties. Fig. 3.b shows the stress-strain curve for steel elements. To note that reinforcement bars were used to model the bolts. For loading, a constant axial force was applied in the beam first, followed by the application of the bending moment incremented up to failure.

Fig. 4.a shows the moment - rotation curves for two levels of axial force. It may be seen numerical results agree well with the experimental results suggesting the model can capture the interaction between bending moment and axial force. The level of axial force in the beam was incremented up to the failure of the connection in tension resulting in the bending moment - axial force interaction diagram shown in Fig. 4.b. It may be seen there is a

continuous degradation of moment capacity with the increase of tensile axial force. However, more experimental results that take into account higher levels of axial force in the beams are needed to confirm the results.

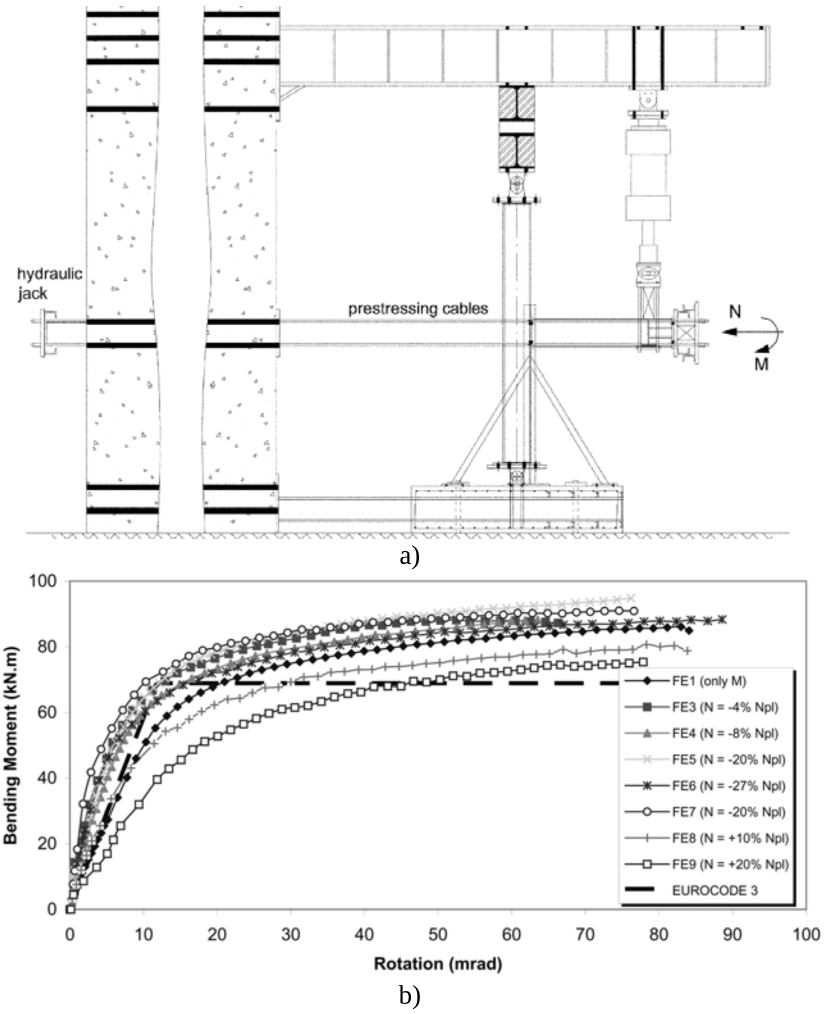


Fig. 2 Experimental setup (a) and moment vs. rotation curves (b) (da Silva et al., 2004)

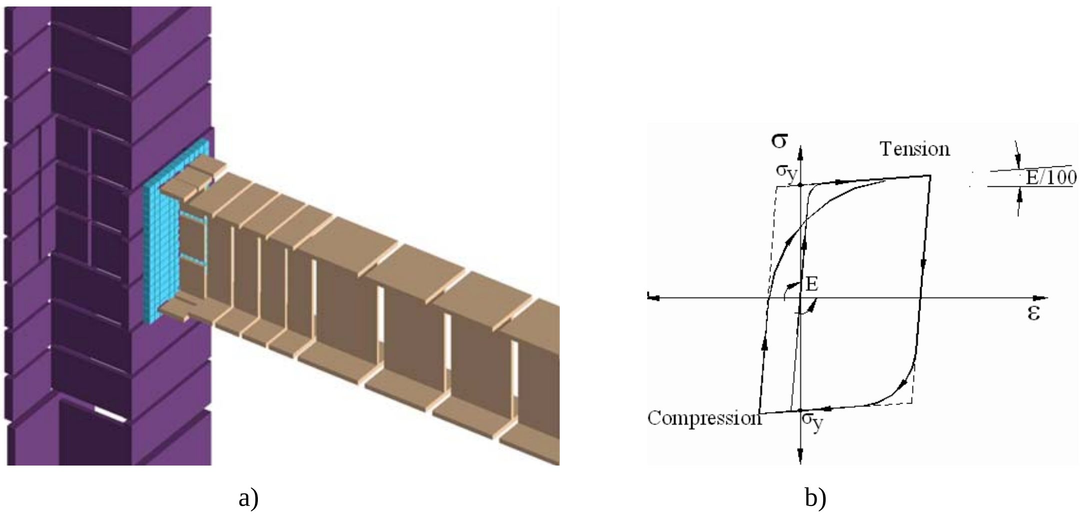
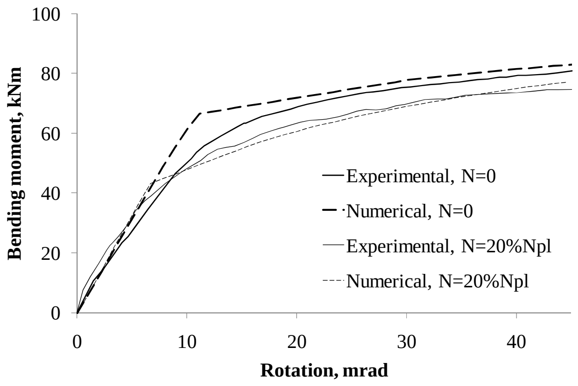
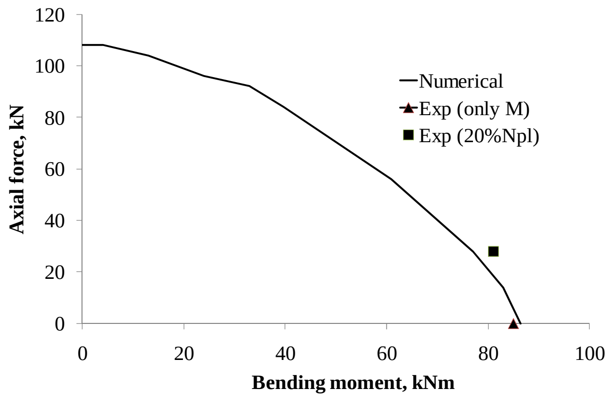


Fig. 3 AEM model (a) and stress-strain curve for steel elements (b)



a)

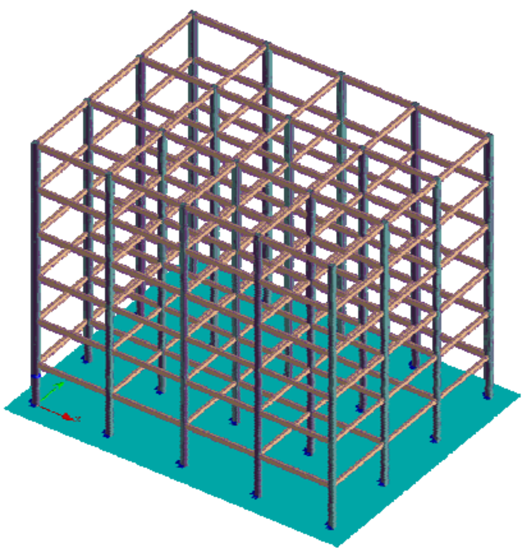


b)

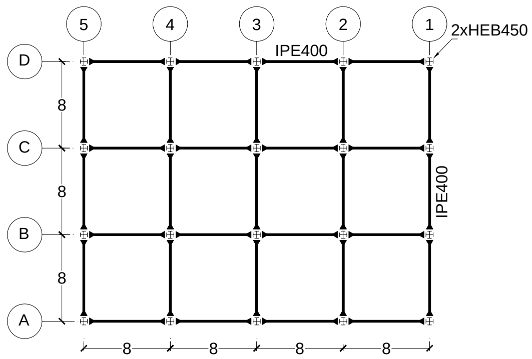
Fig. 4 Bending moment vs. rotation (a) and M-N interaction curves (b)

Case study structure

The test structure is a three-bay four-span and six-story steel structure with moment frames on both directions (Fig. 5). The bays and spans are 8.0 m and the story height is 4.0 m. The structure is designed for gravity and lateral loads (wind and seismic). The dead load and live load is 4.0 kN/m^2, the wind pressure is 0.5 kN/m^2 while the seismic load is evaluated using a design ground acceleration a_g equal to 0.08g, a control period T_C=0.7s and a behavior factor q equal to 6.5. An inter-story drift limitation of 0.008 of the story height was considered in seismic design for the serviceability limit state. Columns have cruciform sections made of hot rolled profiles, grade S355 (f_y = 355 MPa). Beams are made of I hot rolled profiles and the same steel grade. The reinforced concrete slab is cast onto profiled steel decking and supported on floor beams and main beams. The floor system has an important role on the integrity of the structure. Thus, the catenary action that may develop in the floor as a result of a column loss can minimize the damage and inhibit the progressive collapse. However, the modeling of the floor system is not considered in the present study. Since the contribution of the floor is neglected, the secondary beams are also ignored in the model. Two types of extended end plate bolted connections are used (Fig. 6).



a)



b)

Fig. 5 Structural model: a) 3D structure; b) plan layout

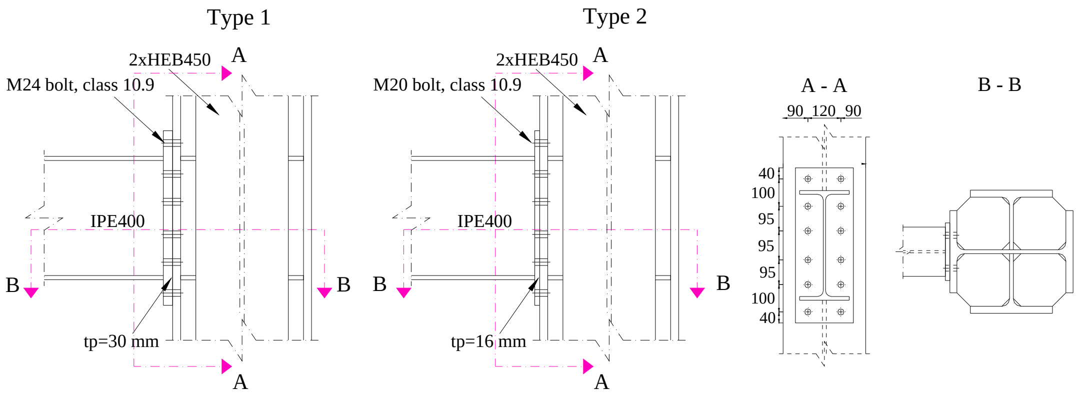


Fig. 6 Details of the beam-to-column connections

The difference consists of end plate thickness and bolt diameter. According to EN1993-1-8 (2005), there are three possible failure modes for bolted end plate connections. Mode 1 is characterized by a complete yielding of the flange, Mode 2 is characterized by bolt failure with yielding of the flange while in case of Mode 3 the connection fails due to the failure of the bolt. Type 1 connection has a beam strength ratio of 1.0 and Mode 2 of failure, while Type 2 has a beam strength ratio of 0.8 and Mode 1 of failure. According to EN1993-1-8 (2005), first connection is classified as full strength and full rigid while the second one is classified as partial strength and semi-rigid (Fig. 7). However, according to EN 1998-1 (2004), both connections are classified as partially restrained.

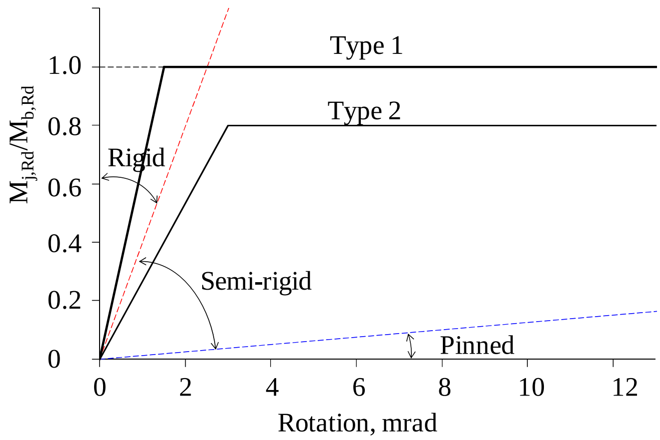


Fig. 7 Moment rotation characteristics for connections

Alternate path method has been used to evaluate the progressive collapse potential of the structures, which were designed for persistent and seismic design situations, but without considering any accidental design situations. The progressive collapse analysis was performed in two consecutive phases: first, a static phase under the dead load and 50% of the live load was applied, followed by a dynamic phase that included initial damage in the structure represented by the loss of a column. The duration of the column removal is 0.001 seconds. Five damage scenarios which involve removal of first floor columns are considered: a) removal of the corner column (A1), b) removal of one edge column (A3), c) removal of one internal column (B2), d) simultaneous removal of the corner and penultimate column (A12), and e) simultaneous removal of two consecutive edge columns (A23), see Fig. 8.

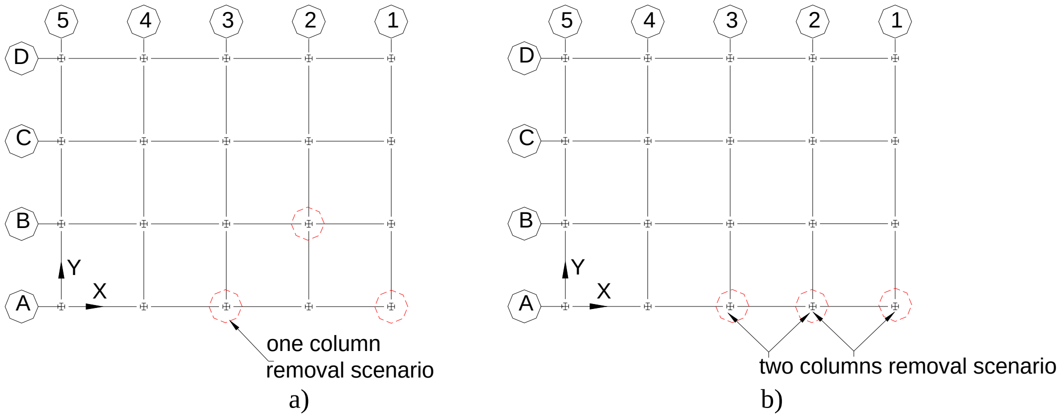


Fig. 8 Column removal scenarios: a) one column; b) two columns

Fig. 9 shows the time histories of the vertical displacement at locations above the removed column for the structure with rigid and semi-rigid connections, respectively. The maximum vertical displacement was recorded for S-A12 scenario but no progression of collapse was observed.

Fig. 10 shows the moment - rotation curves corresponding to the maximum vertical displacement for beam-to-column connections on opposite side of the affected beam. It may be seen that the plastic deformation demand is larger for structure with partially restrained connections.

Fig. 11 shows the axial force in the beams vs. the vertical displacement above the removed column. For S-A1 and S-A12 scenarios, the loss of columns results in compressive axial forces and therefore the framing needs to be capable of cantilevering from neighboring columns to resist the progressive collapse. On the other hand, S-A3, S-B2 and S-A23 scenarios result in the development of tensile forces in beams and may cause a reduction of the ultimate bending moment capacity and ultimate plastic rotation of connections (Fig. 10.b). The reduction is more obvious for S-B2 scenario, where the increase of the tensile axial force causes the reduction of the bending capacity. This indicates that for such scenarios the catenary action may develop in beams upon the increase of vertical displacements. In order to develop the catenary action for resistance to progressive collapse, the gravity loads were increased up to the failure.

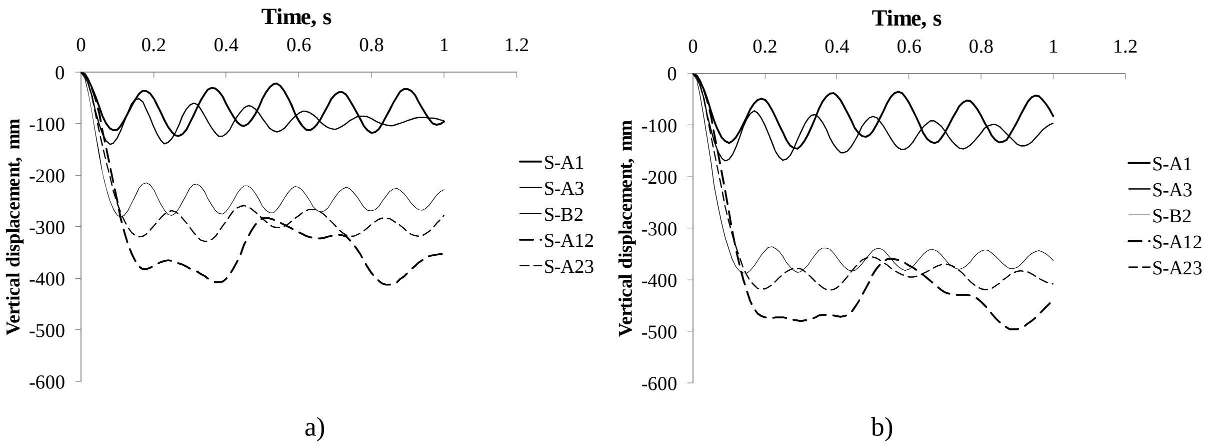
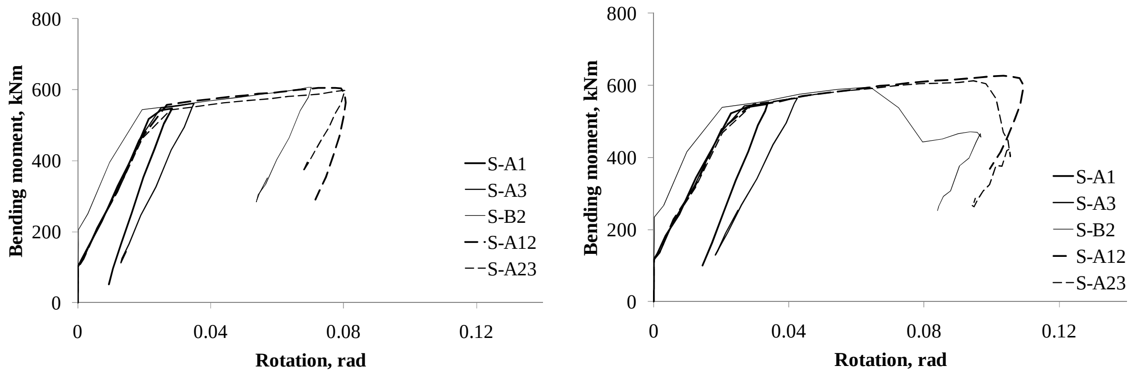
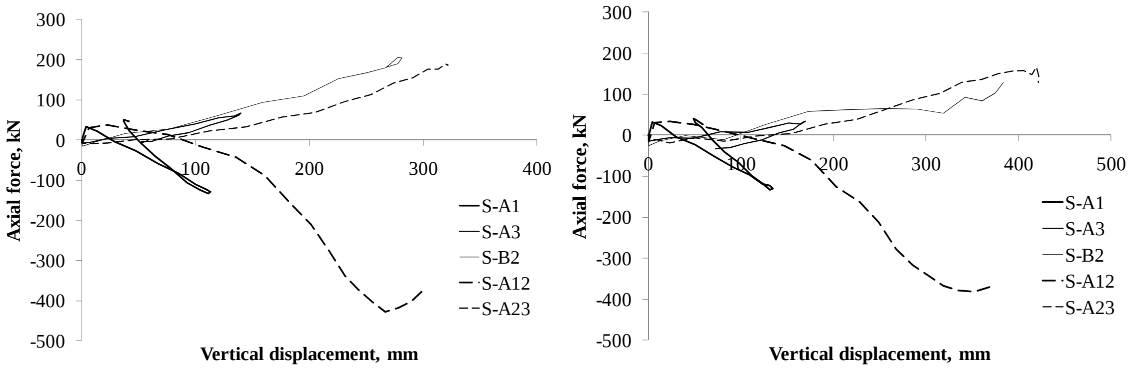


Fig. 9 Vertical displacement vs time from non-linear dynamic analysis: a) structure with rigid connections; b) structure with semi-rigid connections



a) b)

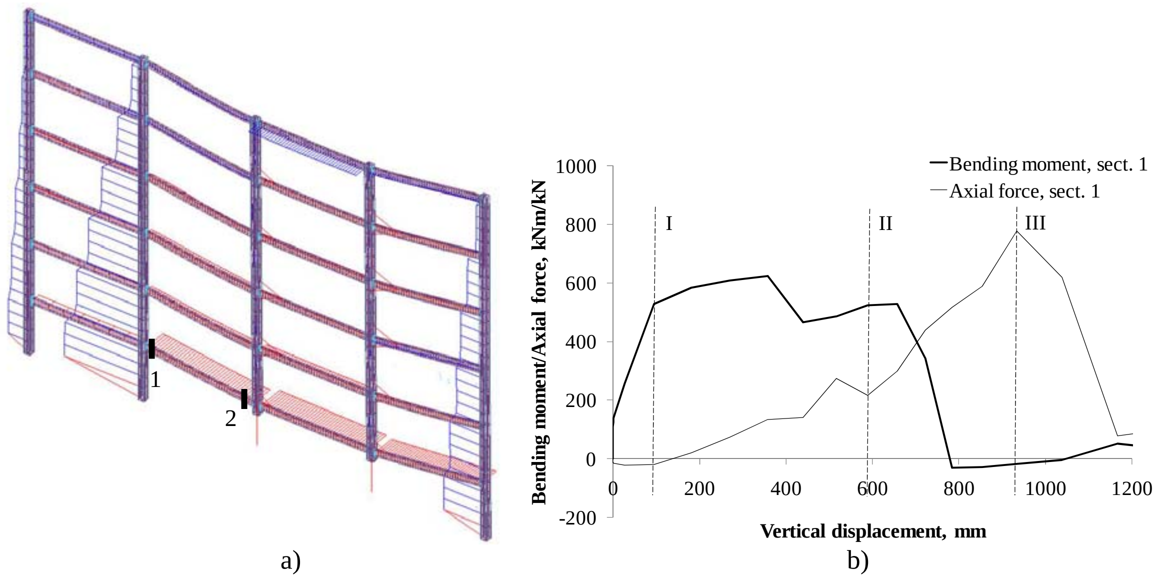
Fig. 10 Bending moment vs. rotation at maximum vertical displacement: a) structure with rigid connections; b) structure with semi-rigid connections



a) b)

Fig. 11 Axial force vs. vertical displacement: a) structure with rigid connections; b) structure with semi-rigid connections

Fig. 12.a shows the deformed shape and axial force diagram in the edge frame (line A) just before the point of collapse for S-A23 scenario and semi-rigid connections. It may be seen the catenary action is developed in the first floor beams above the affected area.



a) b)

Fig. 12 Scenario A-23: a) deformed shape and axial force in edge frame (line A) at the point of failure; b) bending moment and axial force at first floor beam (sect. 1) vs. vertical displacement at lost column (sect. 2)

Close examination of Fig. 12.b reveals three stages of behavior. First stage (0-I) represents the elastic behavior and is the characterized by a state of combined compression and bending. The external loads are resisted entirely by the bending action. At the end of this stage, plastic hinges develops at the beam ends. Second stage (I-II) represents the flexural mode and is characterized by plastic rotations and increasing axial forces. The external loads are resisted both by flexure and axial tension. Third stage (II-III) represents the catenary stage and is characterized by a drastic reduction of the flexural capacity at the plastic hinges while the catenary action starts to develop. The external loads are now resisted by axial tension until the capacity is reached at point III and the collapse is initiated.

CONCLUSION

The study shows that the loss of a column may induce large axial forces in adjacent beams and therefore the capacity of beam-to-column connections needs to take into account the interaction between flexure and axial load. The model used in the numerical analysis was calibrated to match experimental data on full scale tests using bolted end plate connections under bending moment and different levels of tensile axial force.

If catenary action develops, in order to enhance the connections resistance, they need to be designed considering the interaction between bending moment and axial load. Because experimental data on beam-to-column joints under bending and axial force are limited, more research is needed. At corner bays the catenary action is ineffective and therefore moment connections are required on adjacent columns.

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