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EFFECT OF CONNECTION BETWEEN REINFORCED CONCRETE SLAB AND STEEL BEAMS IN MULTI-STORY FRAMES SUBJECTED TO DIFFERENT COLUMN LOSS SCENARIOS

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ABSTRACT

The paper investigates the contribution of floor systems to improving the progressive collapse resistance of multi-story frame buildings following loss of a column. Composite action and diaphragm effect of slab are taken into account by considering the interaction between concrete slab and steel girders. Applied Element Method through nonlinear static and dynamic analyses is applied to predict the structural response after the loss of a column. Robustness criteria are intended to be obtained taking as reference the ratio of failure load to the nominal gravity load.

Keywords: column loss, robustness, tying, composite beam, floor system, steel frame

INTRODUCTION

Buildings should have enough robustness to avoid progressive collapse under extreme load events. Providing progressive collapse resistance is a measure of the structural robustness and relies primarily on resistance of key elements, continuity between elements and ductility of elements and their connections. This calls for a structural design that limits the effects of local collapse and prevents progressive collapse. EN 1991-1-7 (2006) stipulates that a structure shall be designed in such a way that it will not be damaged by events like fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause. Similar requirements are stipulated in ASCE Standard 7-05 (2006), where buildings and other structures shall be designed to sustain local damage with structural system as whole remaining stable and not being damaged to an extent disproportionate to the original damage. The different nature and intensity of possible accidental load events make difficult the development of design requirements for such situations. Therefore, a better strategy is to limit the extent of damage so that the progressive collapse is not initiated. This approach may include the Alternate Path (AP) method, which requires that the structure should resist the loss of one or more critical load-bearing elements without developing disproportionate collapse of the building (DoD, 2009). However, there are concerns that primary load carrying members would not be able to develop the required tie forces because of the significant deformation demands. Some structural features may improve the load redistribution and thus, may reduce the deformation capacity demands. One example is the floor system that considers the interaction between concrete slab and steel beams. Several studies were conducted to consider the ability of the floor system to provide the necessary load redistribution (Stevens, 2008; Stevens et al., 2009; Alashker et al., 2010; Dubina and Dinu, 2012). The results shown that the effect of this interaction is very favorable especially for less redundant structural systems.

The AP method, with its emphasis on continuity and ductility, is similar to current earthquake-resistant design practice in seismic areas (NISTIR, 2007). However, even seismic design philosophy can be taken as a model for collapse control design of structures subjected to extreme events other than earthquakes, there are specific problems which need to be managed when localized failures, particularly of columns, occur. In case of multistory steel building frames, such problems refer to the following issues:

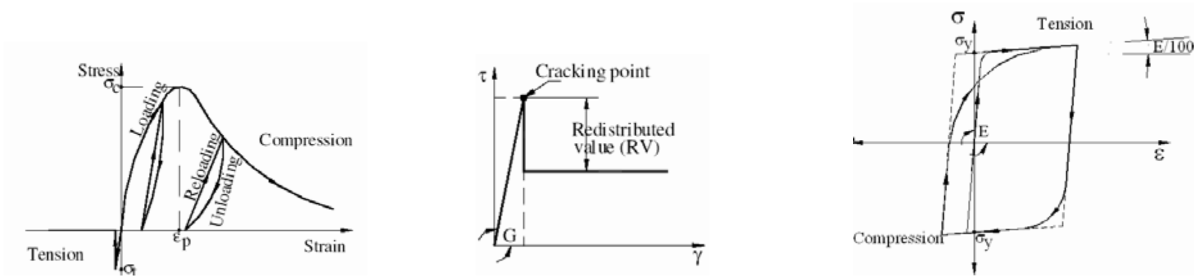
- Contribution of the floor slabs to the tying and catenary effects associated with the demands and performance of beam-to-column connections;
- Release of catenary effect and the supplementary demands in terms of axial load;
- Dynamic impact factor to be applied to gravity loads, it the accidental load combination, in case of failure of columns.
- Admissibility criteria to be considered in the design of beam-to-column connections, diaphragm capacity of floors in case of localized failure of columns.
- Evaluation of tying effect demand and the corresponding axial load to be considered in the design of beams and beam-to-column connections in the zone adjacent to the localized failure;

Therefore, efforts should be made to address these research needs. The paper investigates the contribution of the floor system to the load redistribution capacity in case of a column loss for seismic resistant moment frame structures. For this purpose, two structural configurations are considered. First structure is a moment resisting frame structure with full shear connection between secondary beams and concrete floor slab. According to EN 1994-1-1 (2004), a beam has full shear connection when increase in the number of shear connectors would not increase the design bending resistance of the member. Otherwise, the shear connection is partial. Second structure has the same configuration but is designed as a steel only structure and completely neglects the interaction between steel members and concrete slab. To note that it is difficult to achieve in practice a complete detachment between steel beam and concrete slab, even where no connectors between steel and concrete are employed. Some previous results of the authors (Danku et al., 2011) showed that the simple disconnection of the steel beam from the concrete slab is not sufficient to assure pure steel like behavior and there is a significant increase in stiffness and resistance. In such cases, there is in fact a partial shear connection in beams. However, this conclusion is applicable when the floor beam elements are loaded predominantly in bending. Under large deformations and catenary response of the floor, the tie force cannot rely on the beam-slab interaction and therefore it is difficult to be quantified. The ductility demands for beam-to-column connections, headed stud shear connectors and reinforcement are evaluated for different column loss scenarios. Applied Element Method (AEM, 2010) is employed to predict the structural response after the loss of one column or two adjacent columns. Robustness criteria are intended to be obtained taking as reference the ratio of failure load to the nominal gravity load (Khandelwala and El-Tawil, 2011). Both nonlinear static and dynamic analysis are employed and dynamic increase factors (DIF) are computed as the ratio of ultimate load for static and dynamic analysis cases.

PROGRESSIVE COLLAPSE ASSESSMENT

The assessment of progressive collapse using Alternate Path (AP) method follows the UFC 4-023-03 guidelines (DoD, 2009). The method is based on the acceptance of the failure of some components, but with the preservation of the main structural elements. In UFC 4-023-03, there are three procedures for AP method, based on traditional Finite Element Method (FEM) modeling: linear elastic static (LS), nonlinear static (NS), and nonlinear dynamic (ND) methods. The methods are ordered by increasing levels of analytical complexity, and should be used for buildings with increasing levels of risk for the consequences of failure. Linear elastic analysis (LS) is the simplest method. The nonlinear dynamic analysis has the greatest capability to determine the response of the structure in the event of column loss. The nonlinear static analysis also incorporates material and geometric nonlinearities, however the inertial effect are not included. Therefore, approximations should be made to account for the dynamic effects caused by column removal.

Although the FEM is accurate and reliable for analysis of continuum structures, the onset of element separation is difficult to automate and modeling of debris collision is time consuming. A more accurate prediction of the structural performance under extreme loadings may be obtained by using the Applied Element Method (2010). The Applied Element Method (AEM) can investigate the structural collapse behavior passing through all stages of the application of loads: elastic stage, crack initiation and propagation in tension-weak materials, strain hardening effect in post-elastic range, element physical discontinuity, element collision (dynamic contact), and collision with the ground and with adjacent structures. With AEM, the structure is modeled as an assembly of small elements, that are assumed to be connected by one normal and two shear springs located at contact points, which are distributed around the elements edges. Fully nonlinear path-dependant constitutive models are adopted as shown in Fig. 1. For concrete in compression, an elasto-plastic and fracture model is adopted as shown in Fig. 1.a. When concrete is subjected to tension, a linear stress strain relation ship is adopted until cracking of the concrete springs, where the stresses then drop to zero. The residual stresses are then redistributed in the next loading step by applying the redistributed force values in the reverse direction. For concrete springs, the relationship between shear stress and shear strain is assumed to remain linear till the cracking of concrete. Then, the shear stresses drop down as shown in Fig. 1.b. The level of drop of shear stresses depends on the aggregate interlock and friction at the crack surface. For reinforcement and steel elements springs, the model is as shown in Fig. 1.c.



a) b) c)

Fig. 1 Constitutive models for materials: a) concrete under axial stresses; b) concrete under shear stresses; c) reinforcement and steel elements under axial stresses, (AEM)

For the nonlinear static analysis, the gravity load for the bays immediately adjacent to the removed element and at all floors above the removed element will be:

$$DIF \times \left[D + 0.5L\right] \tag{1}$$

where D = dead load, L = live load and DIF = dynamic increase factor to account for the dynamic effects caused by column removal.

For floor areas away from removed column, the load combination is:

$$D + 0.5L \tag{2}$$

Lateral loads must be taken into consideration with a value of:

$$0.002 \times (\text{sum of the gravity loads } (D + L)) \tag{3}$$

For the nonlinear dynamic analysis, the gravity load for the entire structure can be also calculated using eq. (2), while the lateral loads are taken into account using eq. (3).

The progressive collapse analysis of the structure may asses how many columns can be removed until collapse of the structure occurs. It might be also of interest to evaluate the reserve capacity to support the gravity loads for a specific column loss scenario, which may be expressed as the ratio of failure load to the nominal gravity load, calculated using the combinations described earlier. For this type of analysis, an overload factor, Ω, may be defined as the ratio of failure load to the nominal gravity load (Khandelwala and El-Tawil, 2011):

$$\text{Overload factor}\left(\Omega\right)=\frac{\text{Failure load}}{\text{Nominal gravity load}} \tag{4}$$

CASE STUDY BUILDINGS

The case study buildings have three-bay four-span and six-story steel structures with moment frames on both directions (Fig. 2.a). The bays and spans are 8.0 m and the story height is 4.0 m. The structures are designed for gravity and lateral loads (wind and seismic). The dead load and live load is 4.0 kN/m^2 and the wind pressure is 0.5 kN/m^2. The structures are located in a low seismicity area, characterized by a design ground acceleration a_g equal to 0.08g and a control period T_C=0.7s. High dissipative structural behavior is considered, with 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. First structure has non-composite secondary beams that span between the main beams (Fig. 2.b) and is denoted by *S*. Second structure, denoted as *C*, is designed considering the secondary beams acting as composite sections, which is achieved by using headed stud shear connectors (Fig. 2.c). Headed stud shear connectors are also used on top flange of main beams only within the mid-span (Fig. 2.d) but the composite action was neglected in design. To note that the requirements from EN 1998-1 (2004) prevent the use of shear connectors within the plastic zones, i.e. end of the beams, where significant inelastic strains are expected.

Columns have cruciform sections made of two HEB 450 hot rolled profiles. Main beams are made of IPE 400 hot rolled profiles. Secondary beams are made of IPE 330 for the non-composite floor structure and of IPE 270 for the composite floor structure. A concrete floor slab of 12 cm is employed with a 2.67 m span between the floor beams. The slab reinforcement includes welded wire mesh ϕ6/166 mm × ϕ6/166 mm. 16 mm diameter headed studs were welded to top flange of the secondary beams on one row at 200 mm intervals. For the main beams, the headed studs were welded to top flange on one row at 200 mm intervals, except at the ends, where a free zone of 2 × h_b, or 800 mm has been employed. Extended end plate bolted connections were used to connect the main beams to the columns (Fig. 3). The connection has a beam strength ratio of 0.8, with mode 1 of failure and semi-rigid according to its rotational stiffness. Mode 1 of failure is characterized by a complete yielding of the flange (EN1993-1-8, 2005). To note that the other two modes are bolt failure with yielding of the flange (mode 2) and bolt failure (mode 3). According to the same standard, the connection can be classified as partial strength and semi-rigid. However, according to EN 1998-1 (2004), connection is partially restrained because has no sufficient overstrength to force the development of the plastic hinge in the beam. Secondary beams are connected to main beams using bolted shear plate connections. Table 1 shows the properties of concrete, reinforcement and steel materials adopted in the analysis.

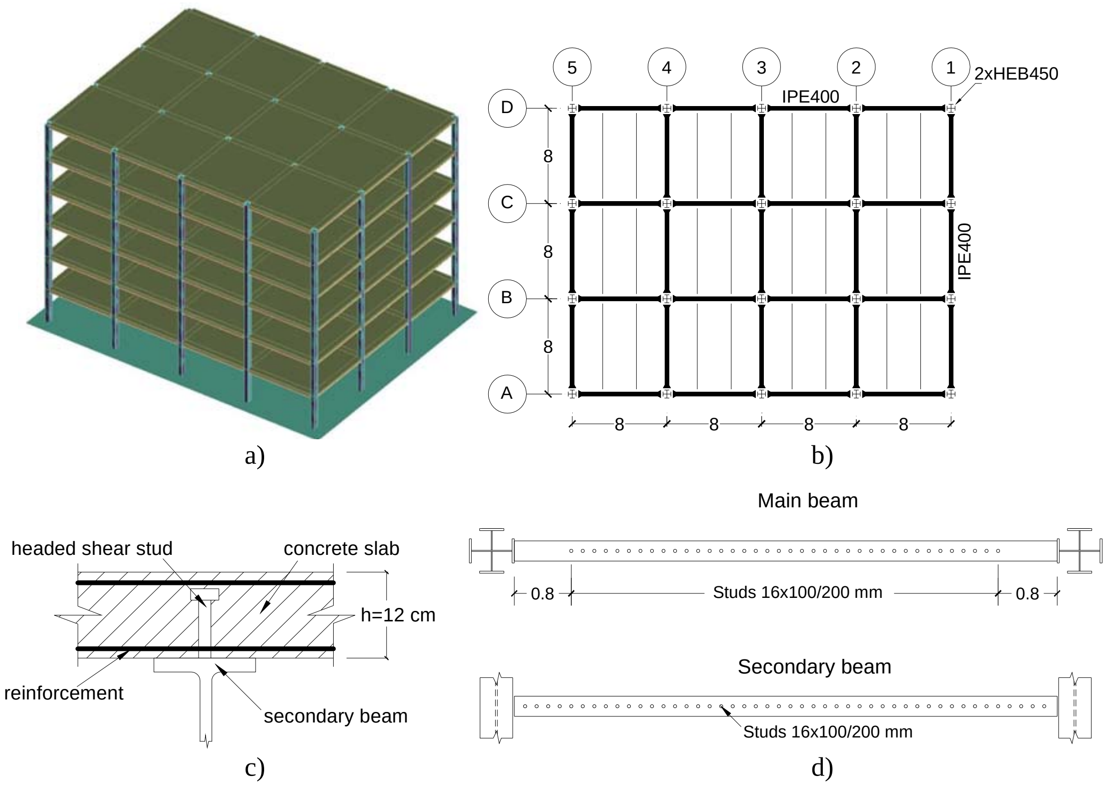


Fig. 2 Details of the structure: a) isometric view of the structural model; b) plan layout; c) cross section of composite secondary beam; d) distribution of connectors for main beam and secondary beam

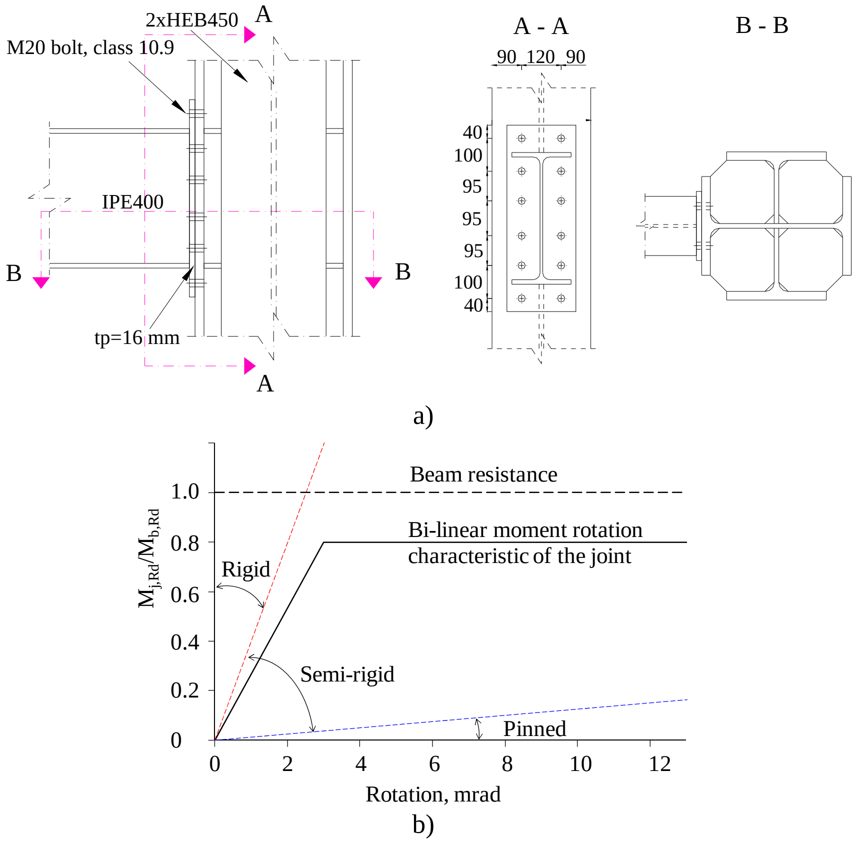


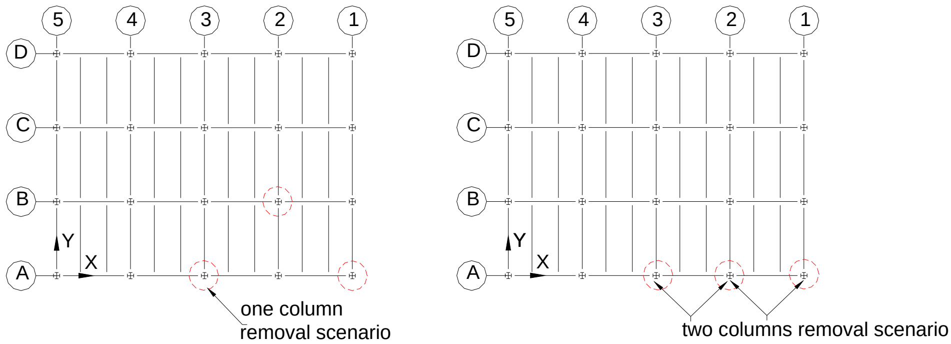
Fig. 3 Details of the beam-to-column joint a) and joint moment rotation characteristic b)

Table 1 Material properties for structural components

Material	Type	Material strength (MPa)		Young's Modulus (MPa)
Concrete	C20/25	Axial tensile strength	F_{ctm}=2.2	30 000
		Compressive strength	F_{ck}=20	
Reinforcement	S420	Yield strength	f_{yk} = 420	210 000
Headed stud	S355	Yield strength	f_y = 355	210 000
Steel framing	S355	Yield strength	f_y = 355	210 000

ANALYSIS RESULTS

Progressive collapse analysis has been carried out for five different column loss scenarios: 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. 4. Table 2 gives a summary of these scenarios.



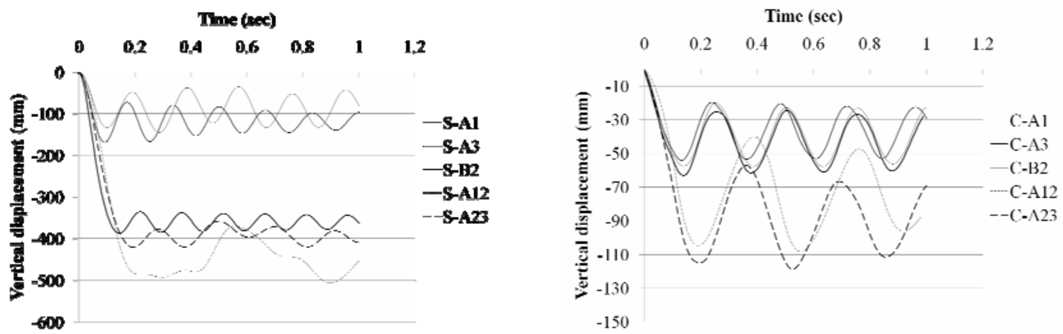
a) b)

Fig. 4 Column removal scenarios: a) one column; b) two columns

Table 2 Scenarios for progressive collapse analysis

Scenario	Member removed	Type of structure	Scenario	Member removed	Type of structure
S-A1	A1	Steel structure with non-composite floor beams (*S*)	C-A1	A1	Steel structure with composite floor beams (*C*)
S-A3	A3		C-A3	A3	
S-B2	B2		C-B2	B2	
S-A12	A1 + A2		C-A12	A1 + A2	
S-A23	A2 + A3		C-A23	A2 + A3	

For first series of numerical simulations, the progressive collapse resistance is assessed using a nonlinear dynamic procedure and load combinations specified by eq. (2) and eq. (3). Fig. 5 shows a comparison of vertical displacement at locations above the removed columns for steel only (*S*) and composite floor structure (*C*), respectively. Fig. 6 shows the maximum strains in steel members and concrete floor for B2 scenario. As shown in Fig. 5 and Table 3, when a single edge or a corner column is removed, the steel only structure (S) experiences a relatively small plastic deformation. When an internal column or two adjacent edge columns are removed, the steel structure experiences larger deformations, with some local failures in the connections due to the fracture of end plate in bending, but no progressive collapse is observed (Fig. 6.a, Fig. 7.a). When the floor beams are designed as composite, the two-way action of the floor reduces very much the vertical displacement and consequently the level of damage (Fig. 6.b, Fig. 7.b). The most significant improvement is observed for B2 scenario. Based on these results, one can conclude that the design of steel beams as composite reduces very much the risk of progressive collapse for the column removal scenarios considered here.

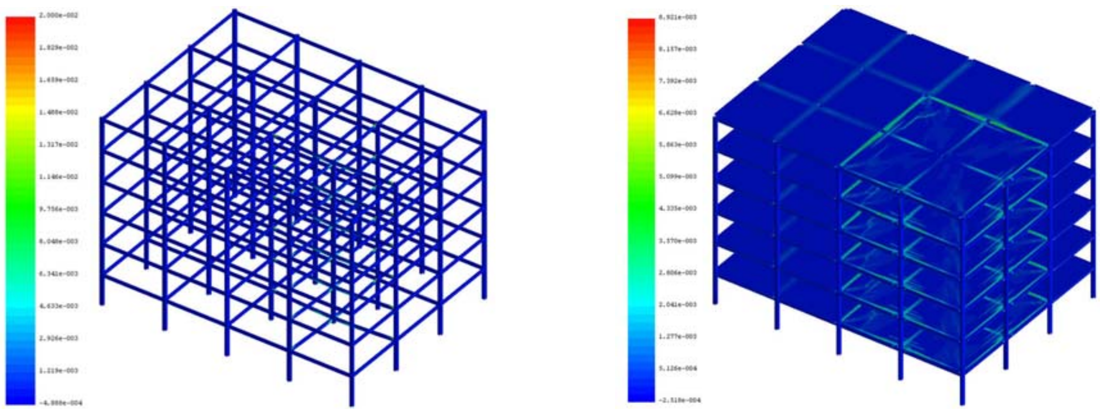


a) b)

Fig. 5 Structure response for nominal load combination: a) steel only structure *S*; b) composite floor structure *C*

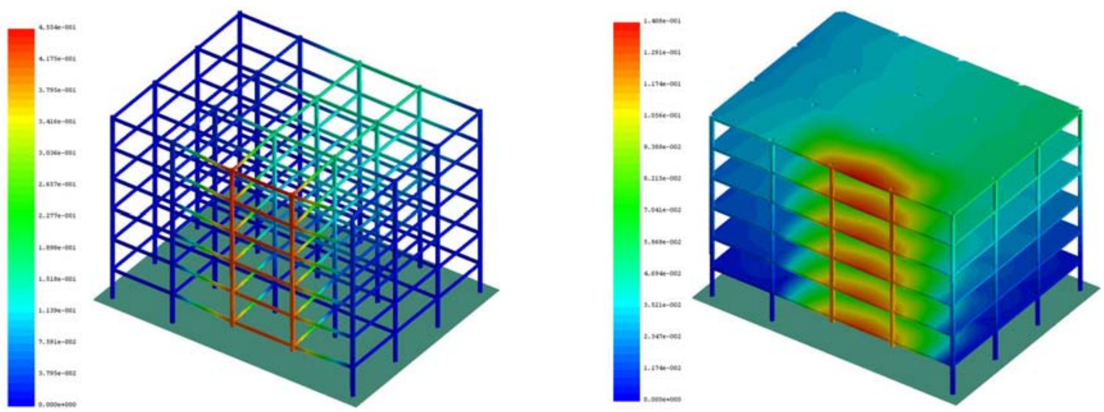
Table 3 Maximum strain in the structures

Scenario	Steel beam	Beam-to-column connection	Headed stud	Concrete	Reinforcement
S-A1	0.0055	0.0181	n.a.	n.a.	n.a.
S-A3	0.011	0.024	n.a.	n.a.	n.a.
S-B2	0.029	fracture	n.a.	n.a.	n.a.
S-A12	0.028	0.053	n.a.	n.a.	n.a.
S-A23	0.027	fracture	n.a.	n.a.	n.a.
C-A1	0.0011	0.0045	0.0019	-0.0001	0.011
C-A3	0.0051	0.0051	0.0022	-0.00057	0.0060
C-B2	0.0016	0.011	0.0020	-0.00060	0.0084
C-A13	0.0069	0.0088	0.0034	-0.00040	0.015
C-A23	0.0053	0.012	0.0037	-0.00034	0.018



a) b)

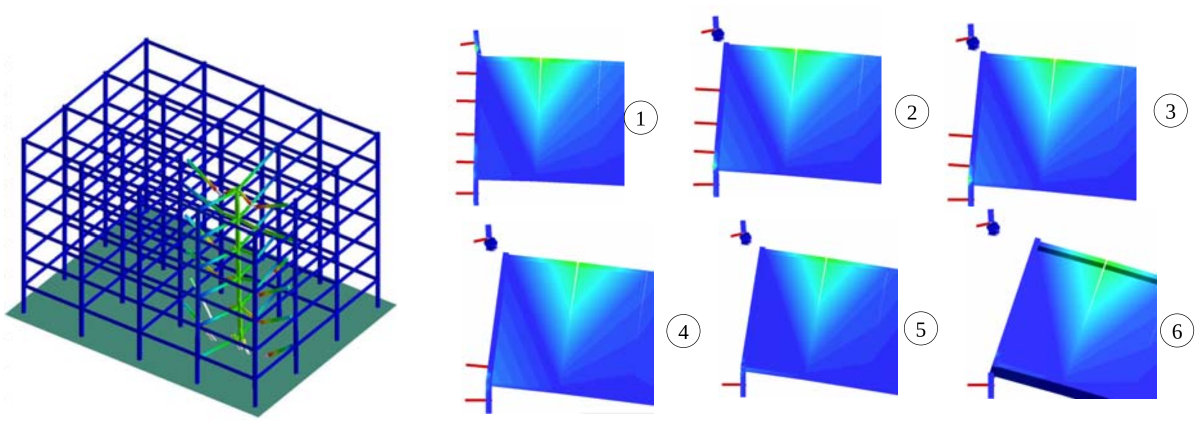
Fig. 6 Maximum strains in the structures, B2 scenario: a) structure *S*; b) structure *C*



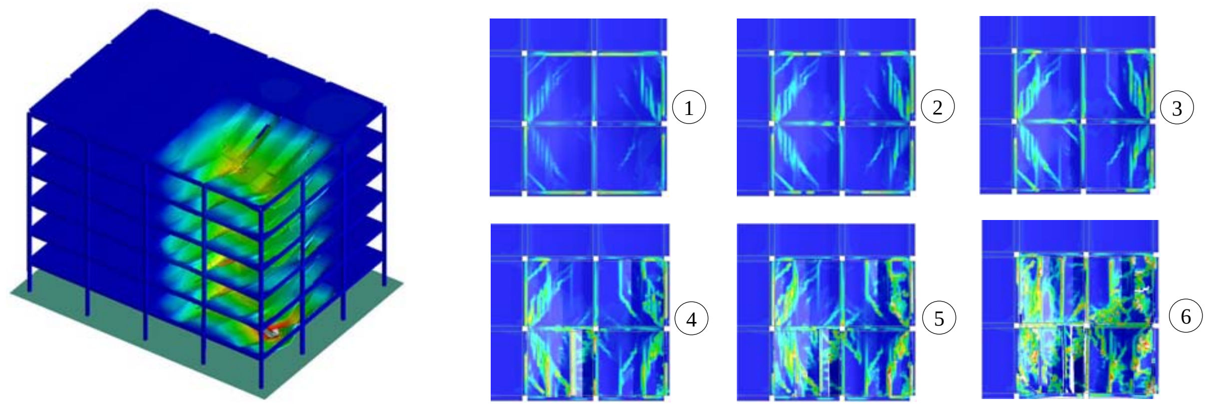
a) b)

Fig. 7 Deformed shape at time = 1.0 s for A23 scenario: a) structure *S*; b) structure *C*

In order to identify the critical components in resisting the progressive collapse, the structures were further analyzed under increasing gravity loads until the collapse is attained and using both static and dynamic nonlinear analysis. Robustness criteria are intended to be obtained taking as reference the ratio of load at collapse to the nominal gravity load. Table 4 gives a summary of the overload factors from static (ΩS) and dynamic analysis (ΩD), and also the resulting dynamic increase factors (DIF). The minimum level of robustness for steel only structure is $\Omega D = 1.05$ and is obtained for one internal column removal (S-B2), followed by the two column removal cases, S-A12 and S-A23. The structure is much less affected by one edge column removal (S-A1, S-A3) and this is evident from the larger overload factors. For structure with composite floor beams, the robustness is very much increased and the minimum overload factor is $\Omega_D = 1.58$ and refers to C-23 scenario. When compared with steel only structure, the largest increase in capacity for the composite beams structure is observed in case of one internal column loss, where Ω_D increases from 1.05 to 2.58. This shows that the composite action is more effective for internal spans, where the catenary action may develop in the concrete slab. Fig. 8 shows the collapse mode and propagation of damage with increasing gravity loads. The failure mechanism of the steel only structure involves the fracture of end plate in bending, followed by fracture of bolts in tension and ultimately the complete separation of the beams due to tensile catenary action, see Fig. 8.a. For the composite floor beams structure, the progressive collapse is initiated by the fracture of end plate bending and fracture of the reinforcement in tension, followed by fracture of bolts in tension, rupture of reinforcement near secondary beams and ultimately the separation of beams and concrete elements, see Fig. 8.b.



a)



b)

Fig. 8 Propagation of collapse for B2 scenario: a) global view of steel only structure (left) and beam-to column connection (right); b) global view of composite structure (left) and concrete floor with composite beams (right)

The dynamic increase factor, DIF, calculated for all scenarios, shows values less than 1.5 and agrees with other similar studies performed in the last years (Ruth et al. 2006, Foley et al. 2008, Dinu et al. 2010, Khandelwala & El-Tawil 2011). To note that the DIF depends on the type of structure and varies with levels of performance and allowed ductility.

Table 4 Overload factor from static dynamic analysis and dynamic increase factor

Scenario	Overload factor, Ω		Dynamic increase factor DIF=Ω_S / Ω_D
	Static analysis, Ω_S	Dynamic analysis, Ω_D	
S-A1	2.7	2.05	1.32
S-A3	2.2	1.6	1.38
S-B2	1.4	1.05	1.33
S-A12	1.45	1.1	1.32
S-A23	1.5	1.15	1.30
C-A1	3.5	2.66	1.32
C-A3	3.78	2.75	1.37
C-B2	3.65	2.58	1.41
C-A13	2.11	1.58	1.34
C-A23	2.51	1.91	1.31

CONCLUSION

The paper investigated the effect of the composite action between the steel beams and concrete slab on the robustness of multi-story steel frames in case of a column loss. For comparison, two structures were considered, i.e. one with composite floor beams and one with steel only elements. Five column loss scenarios were considered, which involved one or two columns loss. Nonlinear static and dynamic analyses were employed under increasing gravity loads until the collapse is attained. The results shown that two-way behavior in the structure designed with moment frames on two directions may prevent the propagation of failure when critical members are removed. However, for the internal column removal, the structure has a very low reserve of capacity and there is risk of progressive collapse. When the floor beams act as composite, there is a significant increase of robustness. The composite action is most effective for internal column removal, while for outer bays the efficiency is much reduced. The dynamic increase factor, calculated for all scenarios, is typically less than 1.5.

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REFERENCES

Alashker Y, El-Tawil S, Sadek F. Progressive collapse resistance of steel–concrete composite floors. J Struct Eng 2010;136(10):1187–96.

ASCE 7-05. Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 424 pages, ISBN: 0784408092, 2006.

Danku G, Ciutina A and Dubina D. Plastic hinges in composite steel - concrete beams of moment resisting and eccentrically braced frames. Proceedings 6th Europeean Conference on Steel and Composite Structures. ISBN: 978-92-9147-103-4, pp. 1047-1052.

Dept. of Defense (DoD). Design of buildings to resist progressive collapse." UFC 4-023-03, Washington, DC, 2009.

Dinu F, Dubina D, Ciutina A. Robustness performance of seismic resistant building frames under abnormal loads. Proc. of the 1st International Conference on Structures and Architecture, ICSA 2010, Guimaraes, Portugal, 21-23 July 2010, p. 613-620, CRC Press, Taylor& Francis Group, ISBN: 978-0-415-49249-2, 2010.

Dubina D, Dinu F. Collapse prevention design of multistorey steel building frames under extreme action. Nordic Steel Construction Conference, 05-07 September 2012, Oslo, Norway, ISBN 978-82-91466-12-5, pp. 1-14.

EN 1991: Eurocode 1 - Actions on Structures - Part 1-7: General actions - Accidental actions, EN 1991-1-7, 2006, European Committee for Standardization (CEN).

EN1993: Eurocode 3 - Design of steel structures, Part 1-8: Design of joints, European Committee for Standardization (CEN), 2005.

EN 1994: Eurocode 4 - Design of composite steel and concrete structures, Part 1-1: General rules and rules for buildings, European Committee for Standardization (CEN), 2004.

EN 1998: Eurocode 8 - Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization (CEN), 2004.

Extreme Loading for Structures, Version 3.1, Applied Science International, Durham, NC, 2010.

Foley C M, Schneeman C and Barnes K. Quantifying and enhancing the robustness in steel structures: Part 1—Moment-resisting frames. Eng. J., 45(4), 247–266, 2008.

Khandelwala K, El-Tawil S. Pushdown resistance as a measure of robustness in progressive collapse analysis. Engineering Structures 33 (2011) 2653–2661.

NISTIR 7396. Best Practices for Reducing the Potential for Progressive Collapse in Buildings, Na-tional Institute of Standards and Technology, Technology Administration, U.S. Dep. of Commerce, 2007.

Ruth P, Marchand K A, Williamson E B. Static equivalency in progressive collapse alternate path analysis: reducing conservatism while retaining structural integrity. J. Perform. Constr. Facil., 20(4), 349–364, 2006.

Stevens D J. Assessment and proposed approach for tie forces in framed and load-bearing wall structures. PEC Project Rep., San Antonio, 2008.

Stevens D J, Marchand K A, McKay A E. Revision of the tie force and alternate path approaches in the DoD progressive collapse design requirements. Proceedings of the 2009 Structures Congress, April 30-May 2, 2009, ASCE, Austin, TX.