Robustness demands for structural joints of multistory steel building frames prone to extreme actions

F. Dinu, D. Dubină, D. Grecea
"Politehnica" University of Timişoara, Romania

Keywords

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1. Introduction

Seismic resistant structures are expected to be less vulnerable than structures designed for gravity loads in case of extreme loading conditions, eg. blast [1]. There are many similarities between seismic resistant structures and blast-resistant structures. However, this should not be assumed apriori, as there are many differences between these hazards and their effects on structures, for instance, the rotation demands of connections associated with the loss of key structural elements (e.g. loss of a column). Seismic codes require generally that beam-to-column connections should provide adequate rotational ductility. But the column loss event that can trigger progressive collapse might not replicate the cyclic behavior of ground motion events. Thus, the rotation demand in case of column loss should refer to monotonic loading, while in case of seismic action it refers to cyclic loading.

According to the capacity design approach, non-dissipative connections of dissipative members should be designed with large overstrength compared to the dissipative members, eg. 1.375 times the plastic resistance for bolted connections, according to EN 1998-1. Such strong connections cannot be obtained by simply connecting the beam to the column, eg. welded unreinforced connections or extended end plate bolted connections. Connections that can provide this overstrength include:

- Flange-welded beam cover plates
- Bolted or welded haunch
- Side plates.

Apart from their effectiveness in case of a strong ground motion, these robust joints can arrest the collapse when adjacent columns are removed due to blast. However, it is also possible that connections with less overstrength or even partial strength connections can also have adequate robustness (eg. ductility and redundancy) to arrest the progressive collapse. One such connection is the extended end plate bolted connection. Previous experimental results on such connections, without supplementary stiffening, shown that they can be equal in capacity with the beams only in case the design is governed by bolt in tension, however this is a brittle failure mode and would pose a high risk when a critical member is removed. Therefore, a more ductile connection, where the design is governed by a ductile mode (eg. end plate in bending), can behave more efficiently and be more reliable.

The objective of the paper was to investigate the performance of seismic resistant structure in case of column loss. Nonlinear dynamic analyses were carried out on two multi-story steel frame structures in order to evaluate their robustness. First structure has moment connections on both orthogonal directions and the second one has moment connections on one direction, only. Extended end plate bolted connections with different capacity ratios compared to connected beams were used, i.e. full strength and partial strength with 0.8 and 0.6 of the beam capacity, respectively. Acceptance criteria were based on experimental tests performed on the same connection typology at "Politehnica" University of Timisoara, Romania. Different column loss scenarios were considered, including perimeter and internal columns. Neither the catenary action nor the achorage effect of the floor slab was considered in the analysis.

2. Analysis model structures

Two structures with the structural plan shown in Figure 1 were analyzed. The first is a two-way span structure (Figure 1.a) and the second is a one-way span structure (Figure 1.b). The structures are 3-bay 4-span and 6-story structure. The bays and spans are 6 m and the story height is 3.5m. Columns have cruciform section made from hot rolled profiles and S460 steel (F_y = 460 N/mm^2) and beams are made of I hot rolled profiles and S235 steel (F_y = 235 N/mm^2) and connected to the columns using extended end plate bolted connections (Figure 1.c).

Three types of beam-to-column joints were considered for the moment resisting spans: full strength and rigid connections (R), partial strength and semi-rigid connections with 0.8 of plastic resistance of the connected beam (SR8) and partial strength and semi-rigid connections with 0.6 of plastic resistance of the connected beam (SR6). They are obtained by varying the end plate thickness and steel grade. Similar connections were tested experimentally and the results are presented in the next section. The design dead and live loads are 4 kN/m^2 and 3.0 kN/m^2, respectively. Seismic design of the structure was done in accordance with EN1998-1 and the Romanian seismic code P100-1. The behavior factor q for MRF structure amounts to 6. The member sizes are IPE330 for floor beams (including beams along column lines) and IPE360 for main beams. Columns are made of 2HEA400 profiles.

3. Progressive collapse assessment

3.1. Alternate path method in progressive collapse analysis

Alternate path has been used to evaluate the progressive collapse potential of the example structure, which was designed for persistent and seismic design situations, but without considering any accidental design situations (e.g. progressive collapse). Four column removal scenarios were assessed. For both structures, the removal included the edge column (S1), penultimate column (S2), corner column (S3) and internal column (S4), see Figure 2. The intention was to evaluate the robustness of the structures different joint capacity ratios, when the structure has moment connections in both orthogonal directions and in one direction only. The progressive collapse analysis follows the guidelines from UFC 4-023-03 [2]. A nonlinear dynamic analysis was followed, with the following load sequence:

a) apply gravity loads to the undamaged structure; This includes dead and live load respectively. In order to check the stability of the damaged structure, a nominal lateral load needs also to be included. This can be done by applying wind load as part of the gravity load. The load combination is DL + 0.5LL + 0.2WL.

b) suddenly remove one column, while keeping the gravity loads constant. The duration for column removal is one twentieth of the period associated with the structural response mode for the vertical motion of the bays above the removed column (Figure 3).

In case of column loss, if the vertical deflections are large, the floor system can start develop a catenary response. When moment resisting frame devolves from a flexure dominant system to a tensile membrane or catenary dominant system, the level of axial force in beam-to-column connections can be several orders of magnitude larger than in case of a seismic action and therefore the connections must be designed for the combined effects of bending and axial load.

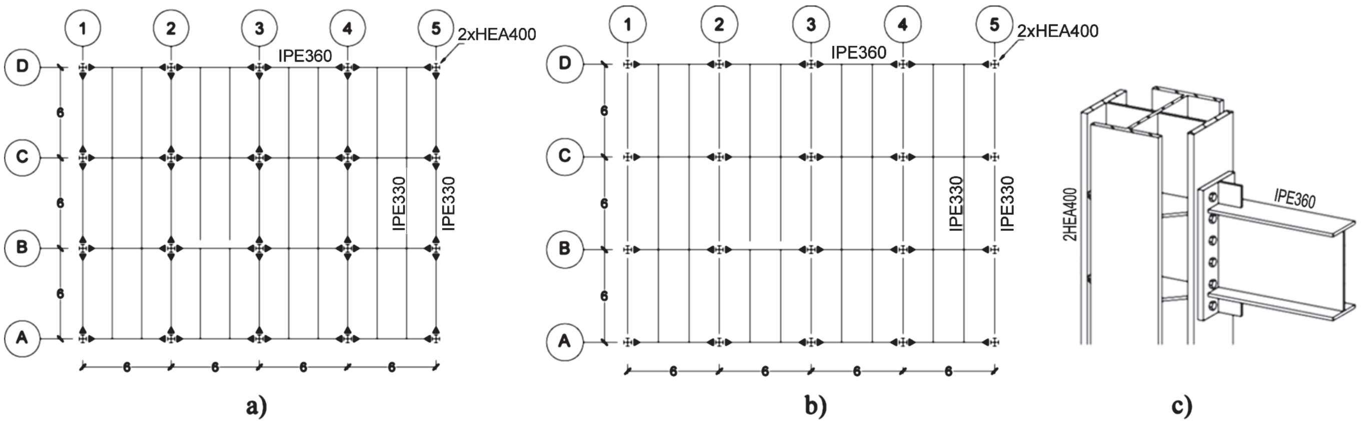


Figure 1. Analysis model for the structure: a) two-way span structure; b) one-way span structure; c) stiffened end plate beam-to-column connection for moment resisting spans

In order to enhance the connections resistance, they can be designed for two limit states: 1) developing beam plastic moment and 2) developing beam axial tension capacity [3]. In the paper, the catenary effect in the beams was considered but it was ignored in the floor slab.

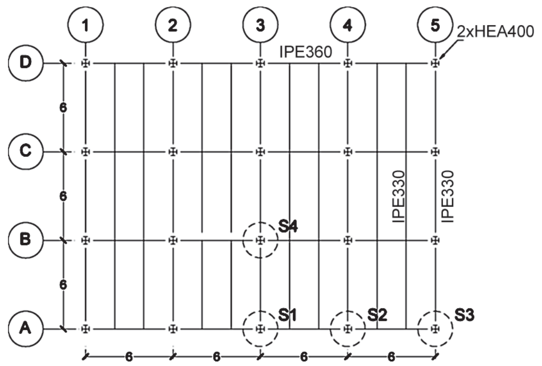


Figure 2. Column removal locations

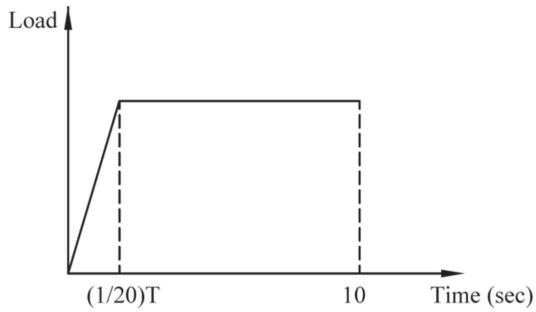


Figure 3. Application of vertical load on the model with lost column

3.2. Modeling for analysis

For seismic analysis, as the behavior is dominated by the deflection of the lateral resisting system, the vertical behavior of the floors (including supporting girders) is not important and therefore they can be modeled approximately. If the floors act as diaphragms, the masses can be applied only at the nodes located on the column lines. In case of progressive collapse analysis, the distribution of masses in the column lines is not accurate and therefore the model needs to be improved [4]. In order to capture the beam vertical vibrations, there were secondary beams and extra nodes on beams and girders in the spans adjacent to the removed column were. In case of moment resisting frames, dissipative zones can be located in members or in connections. If the structure is designed to dissipate energy in the beams, the connections of the beams to the columns should be designed for the required degree of overstrength, without any ductility requirements. Dissipative semi-rigid and/or partial strength connections are also permitted, provided that the connections have a rotation capacity consistent with the global deformations. The connection design, either full strength or partial strength, should be such that the rotation capacity of the plastic hinge region is not less than 0.035 rad for structures of high ductility class and 0.025 rad for structures of medium ductility class. Unlike the ductility demands for earthquake loads, where the structure (and connections) are subjected to several load cycles and therefore degradation associated to low-cycle fatigue is important, for column loss scenario, there is only one half cycle of loading, and therefore there is less degradation and consequently larger plastic rotation capacity. In GSA Guidelines [5], the ductility demand for low level of protection buildings is limited to 20, which is specified for steel beams and is not relevant to connection types and therefore can be used for full strength connections. In the same document, the limit of the plastic hinge rotation for full strength connections and for partial strength connections where the limit state is governed by flexural yielding of end plate is specified as 0.035 radian and 0.023 radian for partial strength connections where the limit state is governed by bolts in shear. In the paper, the modeling and acceptance criteria for full strength joints were based upon GSA Guidelines but for partial strength joints they were also compared to the rotation capacities reported in [6], which summarizes the results of a large experimental research program carried out at the "Politehnica" University of Timisoara which studied the performance of bolted beam-to-column joints under monotonic and cyclic loading. Joint specimens, T- stub and weld detail specimens have been tested. The results are summarized in the next section. There is one factor that may reduce the ductility under monotonic loading, which is strain rate. Experimental results [7] have shown that a higher strain rate implies a reduction of the ductility for monotonically loaded welded specimens. These observations may lead to the conclusion that in case of blast loading, which is typically a monotonic loading, the main cause of poor behavior is due to the severe reduction of the ductility, up to 30%. Therefore, the influence of strain rate must be taken into account in the direct analysis of the blast effects, for example in the specific local resistance approach. However, for progressive collapse analysis of the main structural system, the behavior of the material can be considered rate independent. This can be explained by the fact that even if the air-blast load removes instantaneously the column, the transition from original structure to damaged structure and the development of alternative loads path occurs in larger time interval. This finding was also reported in [8]. Therefore, the effect of strain rate on structural behavior was not considered.

4. Experimental program on beam-to-column joints

In order to achieve both ductility and robustness, different combinations of steel grades and end plate thicknesses were used. T-stub components obtained by welding S235 web plates to S235, S460 and S690 end-plates, using K beveled full-penetration welds (Figure 4). MAG welding was used, with G3Si1 (EN 440) electrodes for S235 to S235 welds, and ER 100S-G/AWS A5.28 (LNM Moniva) for S235 to S460 and S690 welds. T-stubs were connected using M20 gr. 8.8 bolts.

According to EN1993-1-8, T-stub macro-components of beam-to-column connections fall down by 3 types of failure mode, named 1, 2 and 3. In order to achieve a full strength joint, mode 2 or 3 are prerequisite. When mode 1 governs the design, the joint is partial-strength and/or semi-rigid. Starting from previous considerations, it is clear that failure mode 2

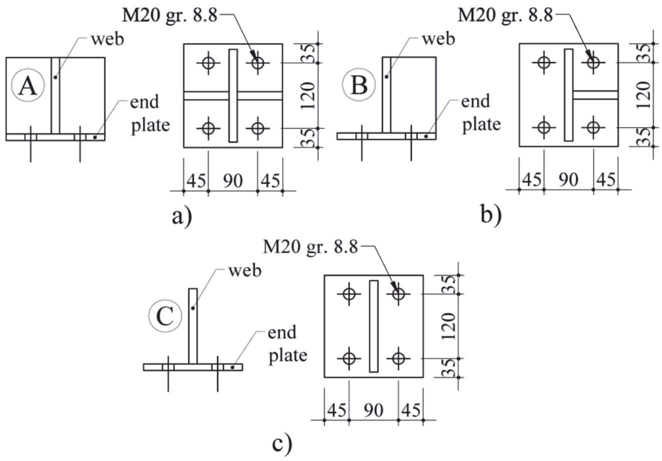


Figure 4. T-stub types

would be preferable in order to achieve criteria for both strength and rotation capacity. Demands applied on members and connections when resisting the initiation of a collapse may be of larger magnitude and will occur simultaneously with large axial tension demands. Therefore, it is not obvious what types of beam-to-column connections will possess sufficient robustness to permit the necessary development of plastic rotations at beam ends together with large tensile

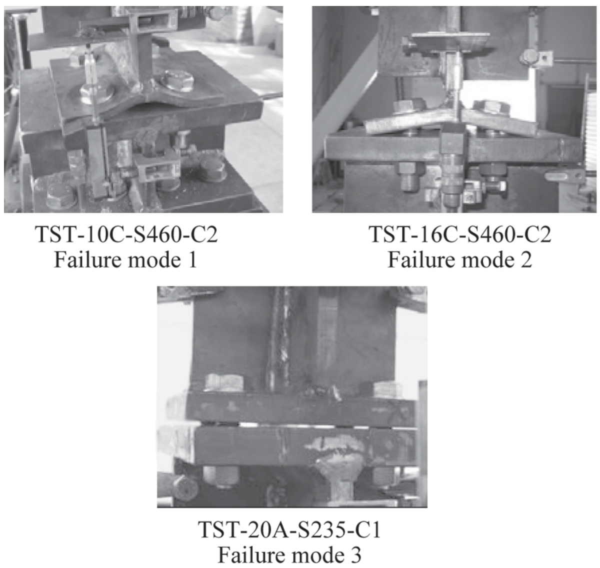


Figure 5. Examples of failure modes of T-stub specimens

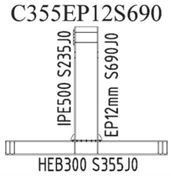
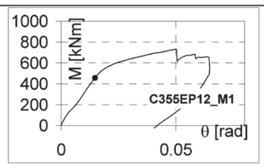
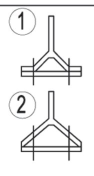
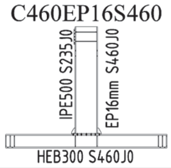
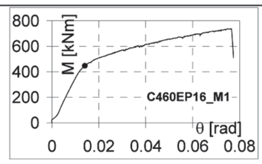
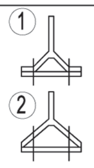
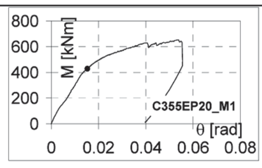
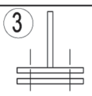
forces [9]. Thus, failure mode 1 was also investigated as it allows for the largest ductility, even if the strength and stiffness make the joint partial-strength and semi-rigid.

The results of monotonic tests on T-stubs are presented in Table 1 and Figure 5. All three failure modes were recorded, with the largest elongation at fracture for T-stub that fail in mode 1 (maximum value amounts to 20.6mm). In all cases, complete penetration groove welds performed well, without premature or brittle failure. Table 2 shows the results of monotonic tests on joints. Rotation capacity was larger when failure mode 1 or 2 was observed.

Table 1. Interpretation of monotonic tests

Specimen	$F_{y,exp,average}$ [kN]	$F_{yEC3-1.8}$ [kN]	$F_{y,EC3}/F_{y,exp}$	$F_{max,exp}$ [kN]	$D_{u,exp}$ [mm]	Failure mode
TST-12A-S235	463.9	425.4	0.92	705.6	20.6	1
TST-12B-S235	395.0	357.8	0.91	559.0	18.3	1
TST-12C-S235	397.8	290.3	0.73	582.6	20.2	1
TST-20A-S235	576.4	645.6	1.12	760.8	4.2	3
TST-20B-S235	509.0	589.1	1.16	744.2	9.0	2->3
TST-20C-S235	559.5	532.6	0.95	758.3	5.4	2
TST-10A-S460	508.3	440.9	0.87	688.7	16.2	1
TST-10B-S460	451.7	383.8	0.85	606.4	15.3	1
TST-10C-S460	423.8	326.6	0.77	550.2	17.6	1
TST-16A-S460	656.8	658.4	1.00	832.8	5.5	2
TST-16B-S460	541.2	598.1	1.11	745.9	7.5	2
TST-16C-S460	538.6	537.8	1.00	687.5	8.8	2
TST-8A-S690	432.0	446.1	1.03	618.4	17.7	1
TST-8B-S690	380.5	392.4	1.03	511.3	13.6	1
TST-8C-S690	379.6	338.7	0.89	474.2	17.9	1
TST-12A-S690	560.7	626.8	1.12	799.5	4.0	3
TST-12B-S690	561.8	575.8	1.02	771.0	6.7	2
TST-12C-S235	463.9	425.4	0.92	693.5	6.9	2

Table 2. Behavior of tested joints under monotonic loading (selection)

Joint Type	M-θ curve	Failure Mode	θ_u
C355EP12S690 			0.061
C460EP16S460 			0.075
C355EP20S235 			0.052

5. Analysis results

Nonlinear dynamic analyses were carried out using SAP 2000. In order to take into account the catenary effect, the main beams were divided into 20 elements and the large displacement effect was considered.

In the case of two-way spans structure (Figure 1.a), for all four column removal scenarios and full strength connections on both directions, the structure had good performance, with no plastic hinge in beams or in columns. For partial strength connections, moderate ductility demands were recorded and the structure had adequate flexural resistance to bridge over the missing column without developing the catenary behavior (Figure 6, Figure 7, Table 3). The maximum plastic hinge rotation is 30.4 mrad and the maximum vertical displacement is 175 mm, for S4 scenario and structure with partial strength joints (SR6). The plastic rotation demand is below the acceptance limit provided by GSA Guidelines and also by experimental results presented in the previous section.

For the one-way span structure, the flexural resistance of the perimeter frames can resist the removal of a perimeter column (S1 to S3 scenarios), and the maximum plastic hinge rotation is 19.5mrad. However, when an interior column is lost (S4 scenario), the structure is not able to provide enough flexural resistance and the mobilization of the catenary action is necessary to resist the progressive collapse. Plastic hinges also develop in adjacent columns, as their bending provides the only horizontal restraint (anchorage provided by the floor slab was not considered).

When the beams respond in a flexural manner, compressive and tensile stresses develop in the plastic hinges. When the beams start resisting the vertical loads through catenary action, the compressive stresses diminish and only tensile stresses develop in

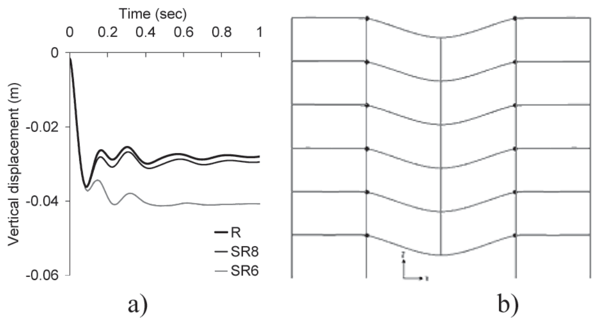


Figure 6. S1 scenario, column line A: a) time history-displacement; b) history of plastic hinges, SR6

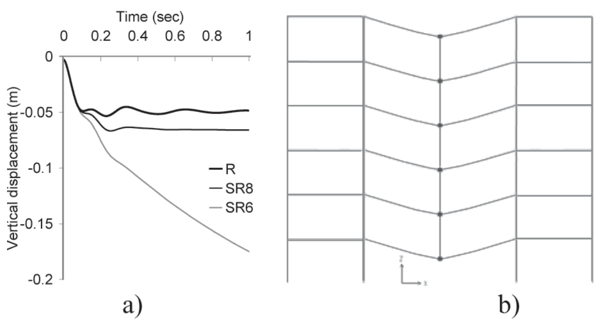


Figure 7. S4 scenario, column line B: a) time history - displacement; b) history of plastic hinges, SR6

the plastic hinge. Since extended end plate bolted beam-to-column connections used in the study has the same or lower capacity then the beam, plastic behavior occurs in both the beam and the connection. Even when the plastic hinge takes place at beam end in flexure, when the beam starts acting in tension, the new distribution of stresses may lead to large

Table 3. Plastic rotation and vertical displacement, two-way spans structure

Connection type/ Column loss scenario	Plastic rotation demand in beams (mrad)	Vertical displacement (mm)
R/S1	-	35
SR8/S1	0.6	36
SR6/S1	5.1	41
R/S2	-	35
SR8/S2	0.6	36
SR6/S2	5.2	42
R/S3	-	33
SR8/S3	0.1	34
SR6/S3	3.1	38
R/S4	5.3	53
SR8/S4	10	66
SR6/S4	30.4	175

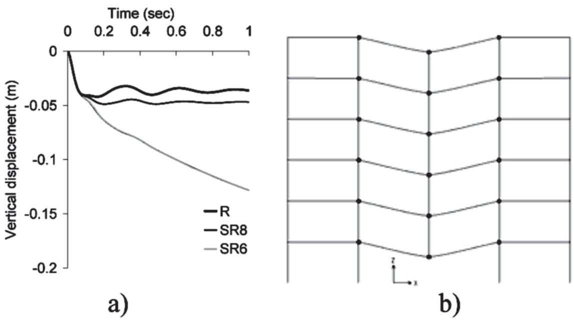


Figure 8. S1 scenario, column line A: a) time history - displacement; b) history of plastic hinges, SR6

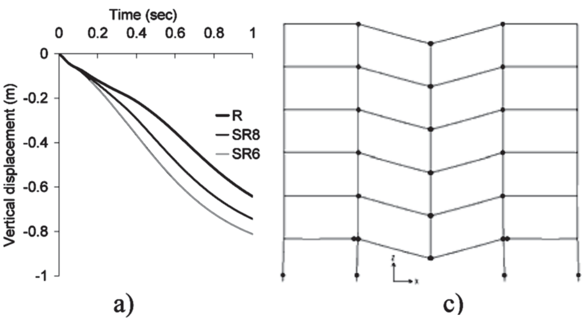


Figure 9. S4 scenario, column line B: a) time history - displacement; b) history of plastic hinges, SR6

tensile forces in connecting elements. The behavior and failure mechanism of the connections can be investigated by studying the behavior of the T-stub components.

Figure 10 shows the variation of axial force in the first floor beams above the removed column when catenary action develops (S4 scenarios). In case of SR6 structure only, the catenary force reaches the axial yield force of the beam. For R and SR8, it reaches 0.67 and 0.89 of the axial yield force of the beam. Total elongation of the same beams in tension is 4.4mm for rigid connected beam, 16.1mm and 24.6mm for semirigid connected beam with 0.8 and 0.6 of the beam plastic resistance, respectively. If the elongation is divided between the two end connections, it results a maximum necessary

Table 4. Plastic rotation and vertical displacement, one-way spans structure

Connection type/ Column loss scenario	Plastic rotation in beams (mrad)	Vertical displacement (mm)
R/S1	0.7	42
SR8/S1	4.4	48
SR6/S1	19.3	128
R/S2	0.7	41
SR8/S2	4.4	48
SR6/S2	19.5	128
R/S3	3.9	68
SR8/S3	8.2	81
SR6/S3	28.3	186
R/S4	34.7	642
SR8/S4	29.4	743
SR6/S4	24.2	811

elongation of 12.6mm. If we look at Table 1, the only T-stub elements that can provide ultimate elongations larger then 12.6mm are T-stubs that fail in mode 1. Moreover, their resistance is also appropriate to resist the tensile capacity of

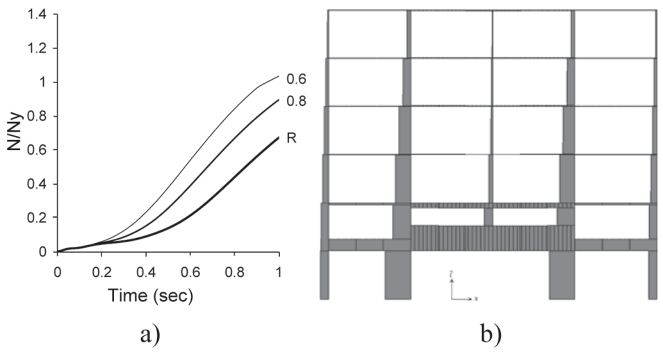


Figure 10. S4 scenario: a) variation of axial force in first floor beams above removed column; b) Axial force diagram for SR6 structure

the beams. The beam-to-column connection can be decomposed in two T-stubs type A and one T-stub type C (see Table 1). The minimum capacity of a T-stub of type A is 618.4 kN (TST-8A-S690) and of a type C is 474.2 kN (TST-8C-S690). The total capacity of these three T-stubs is 1711 kN, larger than the axial capacity of the beam, which is 1708 kN.

6. Conclusions

The paper investigated the potential of seismic resistant structures to arrest the progressive collapse in case of column loss. Alternate load path analysis was applied on two structures, which were designed for persistent and seismic design situations, but without considering any accidental design situations (e.g. progressive collapse). The contribution of diaphragm effect of the floor slab to catenary action was neglected, but obviously this effect is positive. The results have shown that rotation capacity of beam-to-column connections is critical in assuring force redistribution after the loss of columns. When the vertical deflection increases, the beam strength degrades and starts to resist the vertical loads through catenary action, which induces large axial forces in end connections. Due to large deformation demands on connections, ductile configurations should be preferred. Moreover, complete joint penetration groove weld are recommended due to high concentrations of plastic strains in the weld. If catenary action develops, in order to enhance the connections resistance, they need to be designed for two limit states: 1st the attainment of beam plastic moment and 2nd, after the plastic hinges were formed, the attainment of beam axial capacity. The compatibility of the deformation must also be checked. For this two limit states design, future research needs to address the load combinations and acceptance criteria.

References

[1]. FEMA 277 1996. The Oklahoma City Bombing: Improving performance through multi-hazard mitigation. Federal Emergency Management Agency Mitigation Directorate, August 1996.

[2]. DOD 2009. Design of Buildings to Resist Progressive Collapse. Unified Facilities Criteria (UFC) 4-023-03. Department of Defense.

[3]. Marchand, K.A. and Alfawakhiri, F., 2004. Blast and Progressive Collapse, Facts for Steel Buildings, Number 2, American Institute of Steel Construction, Chicago, Il.

[4]. Powell, G., 2005. Progressive Collapse: Case Studies Using Nonlinear Analysis. Proceedings of ASCE 2005 Structures Congress: Metropolis and Beyond, New York, NY, USA, April 20-24, 2005.

[5]. GSA 2003. General Services Administration, Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects, June 2003

[6]. Dubina, D., Muntean, N., Stratan, A., Grecea, D. and Zaharia, R., 2008. Testing program to evaluate behaviour of dual steel connections under monotonic and cyclic loading, Proc. of 5th European Conference on Steel and Composite Structures - Eurosteel 2008, 3-5 September, Graz, Austria, 609-614, 2008.

[7]. Dinu, F., Dubina, D., Stratan, A., 2003. Influence of strain rate on the weld detailing behavior in MR connections, Proc. of STESSA 2003 "Behavior of steel structures in seismic areas", Naples, Italy, 9-12 June 2003, pg. 835-841.

[8]. Foley, C.M., Martin, K., Schneeman, C.L. 2007. Robustness in steel framing systems. Marquette university research report MU-CEEN-SE-07-01. Chicago (IL): American Institute of Steel Construction.

[9]. Hamburger, R. and Whittaker, A., 2004. Design of Steel Structures for Blast - Related Progressive Col-lapse Resistance, Proc. of 2003 AISC and Steel Institute of New York Steel Building Symposium: Blast and Progressive Collapse Resistance, American Institute of Steel Construction.