INFLUENCE OF MODELING CRITERIA ON THE RESPONSE OF STEEL FRAME STRUCTURES TO COLUMN REMOVAL

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Abstract. Structures for buildings, as well as other engineering structures, must be designed to withstand local damage without compromising the stability and load bearing capacity of the structural system. In case of framed structures, critical scenarios typically involve damage or loss of a column, which case be followed by the spread of the damage to neighboring elements and thus triggering the generalized (or progressive) collapse. The paper presents the results of numerical studies carried out on current building frame typologies, designed to withstand gravity and seismic loads. Accidental situations include different column removal scenarios, and methods of analysis include both static and dynamic analyzes. The ability to redistribute loads and dynamic behavior associated with sudden column removal depend both on the local response (strength, ductility) and on lateral load resisting behavior.

Key words: progressive collapse, steel frame, robustness, dynamic increase factor, deformation capacity.

1. INTRODUCTION

Frame structures are widely used in buildings or other types of constructions. Ensuring their safety in service must take into account the probability of occurrence of accidental loading situations (e.g. explosions) capable of causing local damage. Thus, they are expected to survive the accidental event by limiting the extension of damage and prevent the development of progressive collapse [1,2].

Features like ductility and continuity provide more deformation capacity and redistribution of loads so that the structure can bridge over damaged elements [3]. Such measures are more effective providing that the connections can withstand the extreme loading and deformation demands arising from the occurrence of local damage/failure. In addition, the selection of structural system, (e.g. two-way frames) can enhance the progressive collapse resistance by reducing the loading demand and providing alternate load paths [4].

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The response of the structure under extreme loadings and the transition from undamaged to damaged states are complex and difficult to predict by means of analytical calculations [5,6]. Instead, numerical simulations can be employed. The use of advanced methods (dynamic vs. static, nonlinear vs. linear) increases the accuracy of the analysis but requires higher computational effort and advanced engineering skills. In addition, to reproduce the actual behavior with enough accuracy, models should be validated against experimental data. Therefore, significant contributions to the development of design guidelines are still necessary [7,8].

The study presented in the paper employed numerical analyses to evaluate the progressive collapse resistance of current building frame typologies, designed to withstand gravity and lateral loads (seismic, wind). Accidental situations include different column removal scenarios, and methods of analysis include both nonlinear static and dynamic analyzes. The structural response was evaluated using SAP2000 program [9].

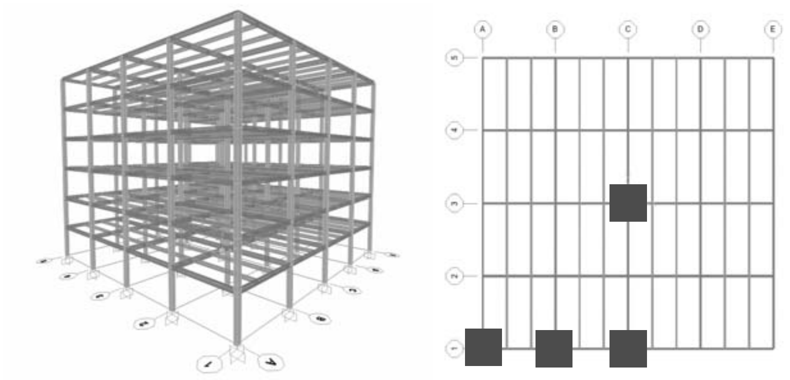
2. NUMERICAL MODELING

The case study building frames are four-bay, four-span, six-story steel structures with moment resisting frames in both directions (Fig. 1a). The bays and spans measure 6.0, 7.5, and 9.0 m, and each story is 4.0 m high. Details about structural elements are given in Table 1. Note that structural steel S275 (yield strength of $275\ \text{N/mm}^2$) was used for beams and S355 (yield strength of $355\ \text{N/mm}^2$) for columns. The dead and live loads were $4.0\ \text{kN/m}^2$ and $3.0\ \text{kN/m}^2$, respectively. The structures are in a moderate seismicity area, characterized by a design ground acceleration, e.g., of 0.20 g, and a control period TC of 1,0 s. It should be noted that the seismic intensity and the response spectrum used in design were those given in the Romanian Seismic Code, P100–1/2013 [10]. High dissipative structural behavior was considered using a behavior factor q of 6.5.

Table 1

Geometry and sections

Structure	Bay, span [m]	Columns [mm]	Main internal beams	Main perimeter beams	Secondary beams	Span/ depth ratio
S6	6.0 m	RHS400x400x 22.2	IPE450	IPE400	IPE330	15
S7.5	7.5 m	RHS400x400x 22.2	IPE450	IPE400	IPE330	18.75
S9	9 m	RHS400x400x 22.2	IPE450	IPE400	IPE330	22.5



a) b)

Fig. 1 – Isometric view of six-story model a), and the column removal locations b).

An inter-story drift limitation of 0.0075 of the story heights was used for the seismic design at the damage limitation state. Persistent and seismic design situations were used for the structural design of members using the relevant Eurocode parts. Beam to column joints were designed as full strength and rigid joints. No accidental design situations were considered in design.

The assessment of progressive collapse resistance is done using the alternate path (AP) method and two types of analysis procedures, i.e. nonlinear static (NSP) and nonlinear dynamic (NDP), in accordance with the UFC 4–023–03 guidelines [2]. The AP method ascertains the capacity of a structure to resist the loss of one or more critical load-bearing elements without causing disproportionate collapse. In the NSP, the column is deleted from the model and the structure is subjected to gravity loading. For the analysis, the gravity load on the bays immediately adjacent to the lost element and on all floors above is given by:

$$G_N = DIF \times \left[DL + 0.5LL \right], \tag{1}$$

where G_N is the increased gravity load for nonlinear static analysis, DL is the dead load, LL is the live load, and DIF is the dynamic increase factor for accounting for the dynamic effects of the column loss.

The combined load on the areas of the floor away from the lost column is given by:

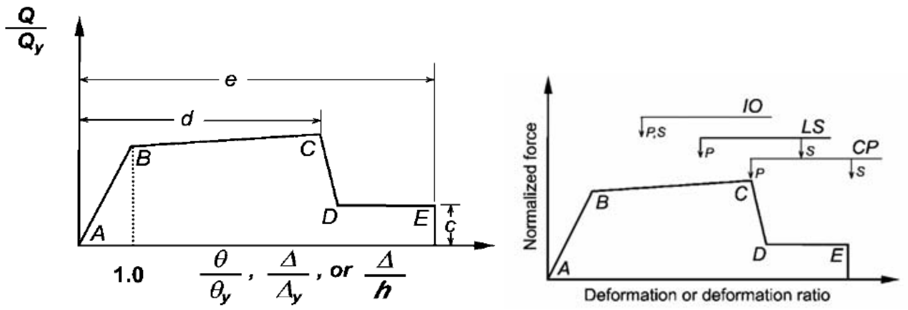
$$G = \left[DL + 0.5LL \right], \tag{2}$$

where G is the gravity load.

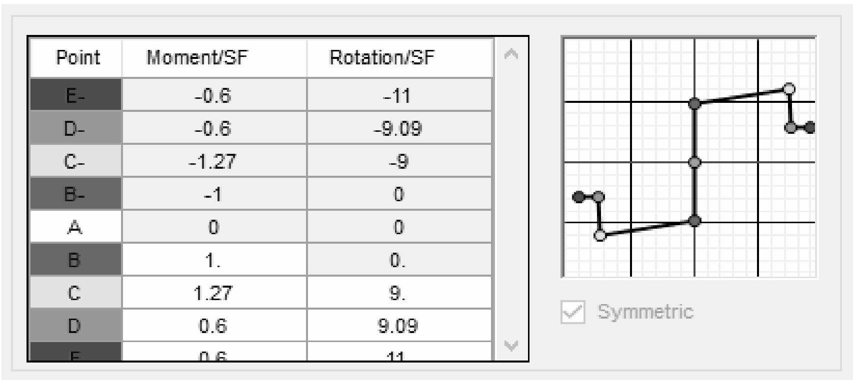
In the NDP, first, the gravity load calculated with Eq. (2) is applied on the structure using a static analysis, then in the second stage the column is removed almost instantaneously (e.g. 0.001 seconds).

The removal locations included the corner column A1, penultimate column B1, edge column C1, and internal column C3, as shown in Fig. 1b.

The nonlinear alternate load path analysis was done using SAP2000 program [9] and the plastic hinge concept. Columns were modeled using discrete plastic hinges, located at both ends, of axial-moment interaction type P-M2-M3, defined according to ASCE/SEI 41−13 provisions [11], see Fig. 2a. For beams however, two types of modeling were adopted. In the first approach, plastic hinges were modeled using flexural moment hinge type (M3-type), defined also according to [11] (Fig. 2a) but modified based on UFC–023–03 provisions [2]. Thus, in order to accommodate the different phenomena associated with progressive collapse, for beams subjected to flexure the Collapse Prevention (CP) values for primary and secondary elements were used instead of Life Safety (LS) conditions. Fig. 2b plots the M3 plastic hinge behavior adopted in SAP2000 for modeling the main transversal and longitudinal beams.



a)



Point	Moment/SF	Rotation/SF
E-	-0.6	-11
D-	-0.6	-9.09
C-	-1.27	-9
B-	-1	0
A	0	0
B	1.	0.
C	1.27	9.
D	0.6	9.09
E	[illegible]	[illegible]

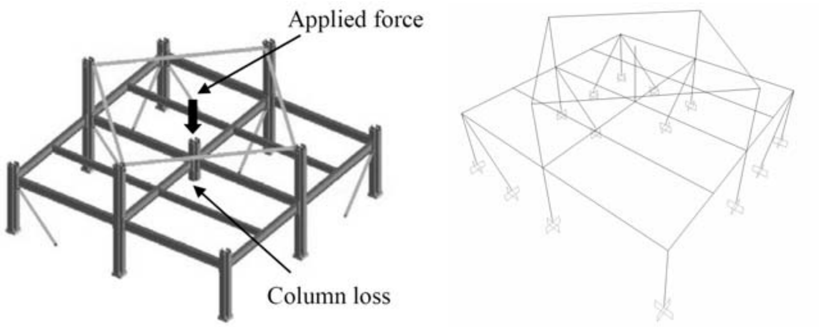
b)

Fig. 2 ─ Force-deformation relations for modeling and acceptance criteria:
a) deformation ratio and element deformation criteria [11]; b) M3 plastic hinge behavior adopted in SAP2000 modeling.

However, there are issues in regard with the adoption of seismic based provisions for progressive collapse events. First, the backbone curves are derived from cyclic testing, whereas only one-half cycle is applied in a progressive collapse event. Second, the ultimate state of strain in elements can be affected by the moment-axial tension interaction if large deformations and catenary forces are activated in beams. Therefore, modeling and acceptance criteria for beams were extended by considering experimental data obtained from relevant tests. For this second approach, the beams were modeled using Fiber P-M2-M3 hinge types, assigned at both ends. The wide flange beams were modeled using three fibers along each side of the cross section (web and flanges).

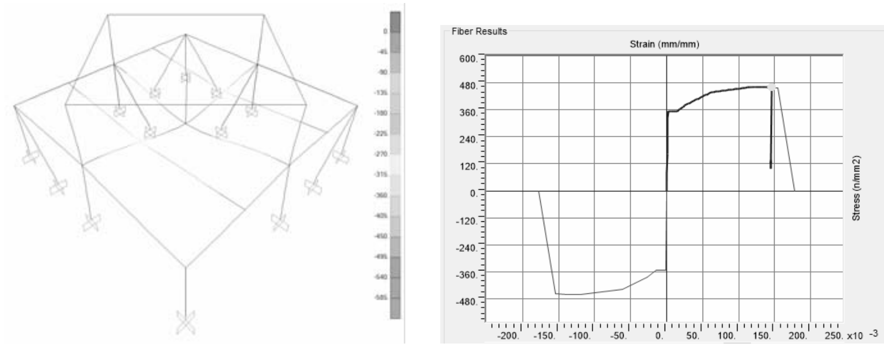
To derive acceptance criteria and validate the numerical model, the experimental tests carried out within the framework of the "Structural conception and collapse control performance based design of multi-story structures under accidental actions CODEC" research program were used for reference [4]. The test setup is illustrated in Fig. 3a, while Fig. 3b shows the detailed numerical model developed with the SAP2000 software [9]. In the test, the vertical load was applied quasi-statically at the top of the central column using a displacement control protocol and was gradually increased until the failure of the specimen. The ultimate vertical load capacity of the numerical model was obtained by carrying out a displacement controlled dynamic pushdown analysis, but with a low speed, similar with the experimental one. The numerical model consists in beam elements with nominal specimen geometry, material properties based on the material tests, and Fiber P-M2-M3 hinge types, assigned at both ends of the main beams and columns. The rigidity of the column support was computed based on the structural detailing.

Figure 4 shows the deformed shape of the specimen and the ultimate strain in the beam at the attainment of the peak force, while Fig. 5 shows the vertical force versus vertical displacement curves at the central column.



a) b)

Fig. 3 – Geometry of test specimen a), and SAP2000 numerical model b).



a) b)

Fig. 4 ─ Deformed shape at peak force a), and attainment at ultimate strain in beam b).

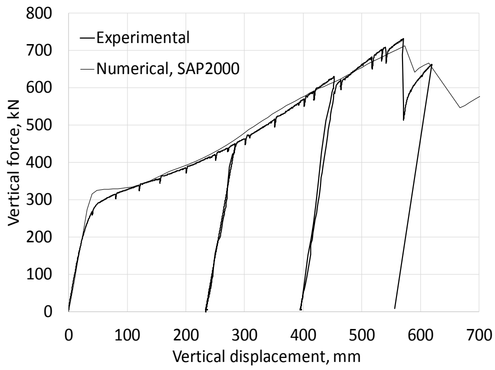


Fig. 5 ─ Force-displacement curves, experimental *vs*. numerical.

The numerical results show a very good correlation with the experimental ones. All the phenomena that occurred during the test could also be traced on the numerical force–displacement curve, i.e., elastic behavior, plasticity, initiation of catenary force, and failure. The maximum vertical force and the corresponding displacement in the numerical analysis were 712.5 kN and 573.4 mm respectively, which are very close to the experimental values, i.e. 732 kN and 569 mm, respectively.

3. RESULTS OF NUMERICAL SIMULATIONS

Figure 6 to Fig. 8 show the vertical force, *F*, vs. vertical displacement, *d*, obtained from static and dynamic analysis for all scenarios and structural configuration. The *F-d* curves derived from static and dynamic analysis for scenarios other than corner column (i.e. B1, C1, and C3) show higher peak force and deformation capacity for fiber hinge model (FH) than for plastic hinge model (PH), as the FH model relies upon catenary response at large deformation stage. In case of A1 scenario, as the response is governed by flexural capacity, the peak force is very close for both PH and FH.

The additional capacity provided by the beams developing catenary action can however be limited by the capacity of some connections to transfer significant axial tension load upon reaching the ultimate moment capacity of the beam, as indicated in [2,12]. The large axial forces in the beams may also cause plastic deformations in the perimeter columns, which, in the presence of large gravity loads, might lead to column buckling and possible failure because of excessive deformations (Fig. 9). Figure 10 shows the plastic mechanism at the attainment of peak force, for 6 and 9 m span structures, scenarios C1 and C3, PH model. The plastic mechanism involves all beams from the area affected by the column removal, with plastic hinges in the adjacent columns but at the base only.

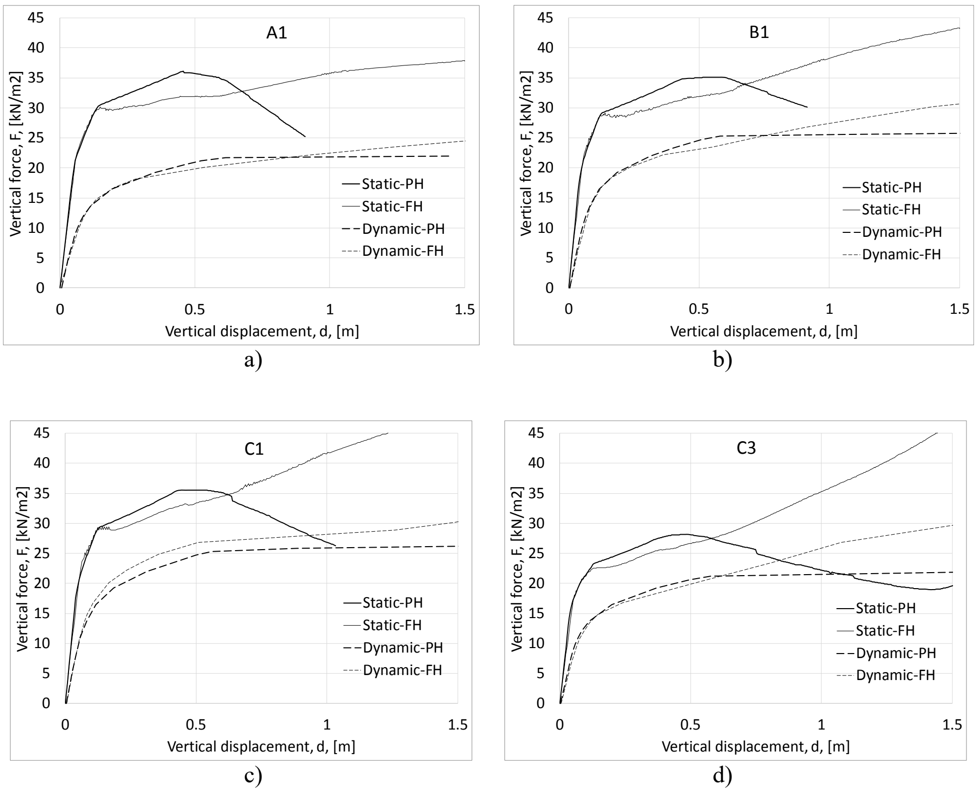


Fig. 6 – Vertical force vs vertical displacement, 6.0 m span structure: a) scenario A1 ; b) scenario B1 ; c) scenario C1 ; d) scenario C3 .

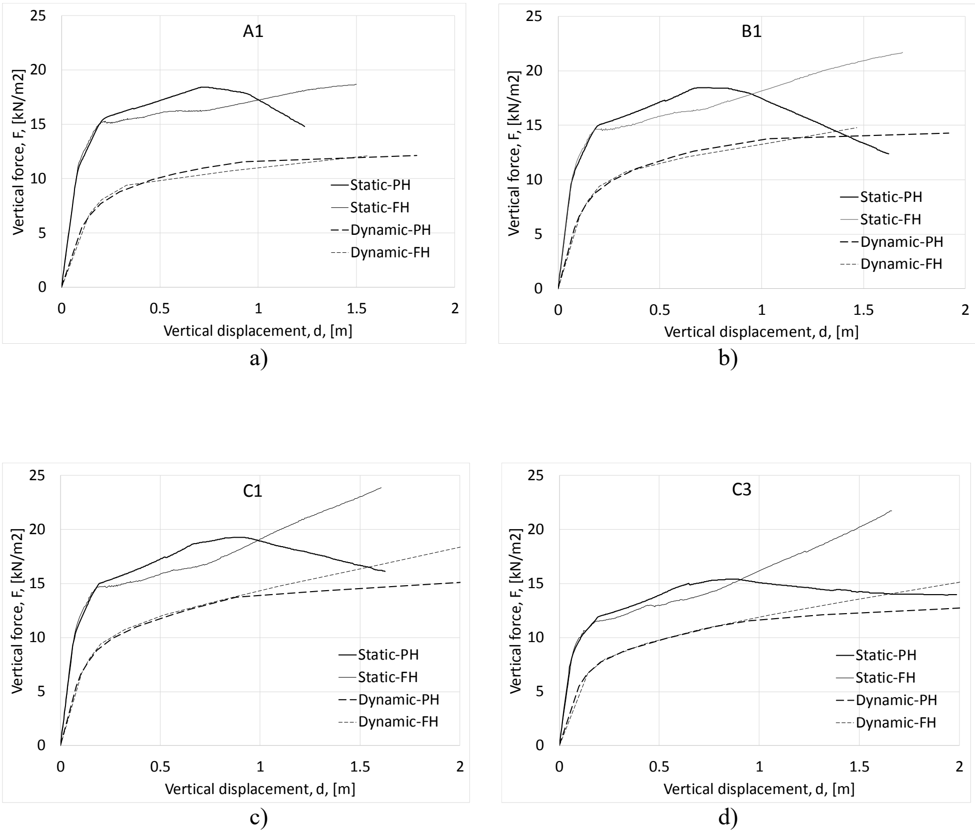
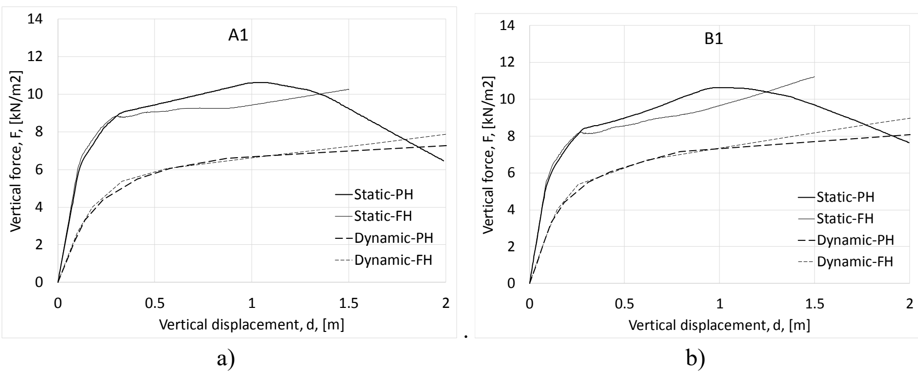
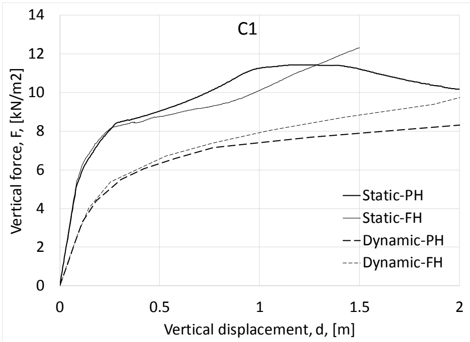
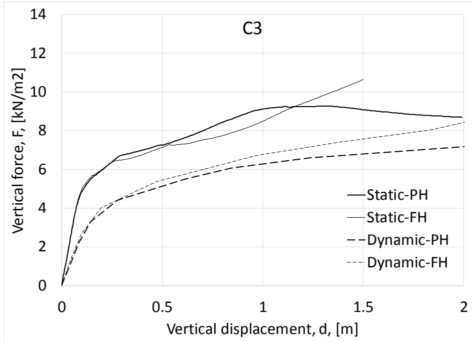


Fig. 7 – Vertical force vs vertical displacement, 7.5 m span structure: a) scenario A1 ; b) scenario B1 ; c) scenario C1 ; d) scenario C3.



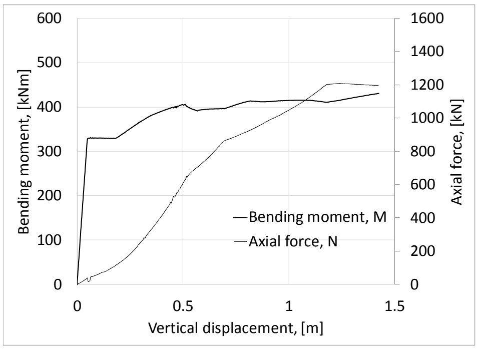


c)

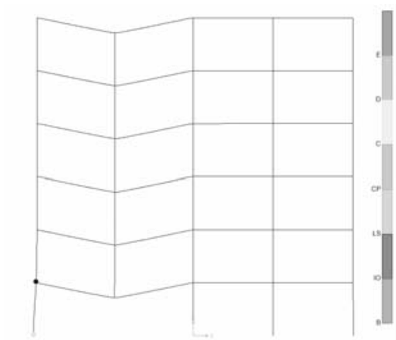


d)

Fig. 8 ─ Vertical force vs vertical displacement, 9.0 m span structure: a) scenario A1 ; b) scenario B1 ; c) scenario C1 ; d) scenario C3.

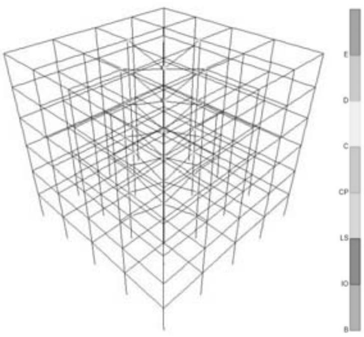


a)

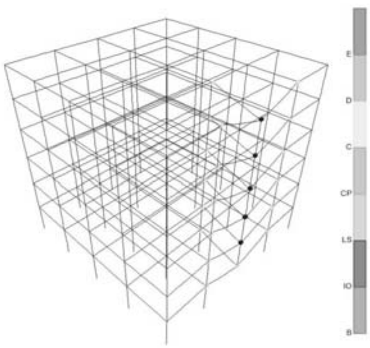


b)

Fig. 9 ─ Structure with 6.0 m span, scenario B1, FH model: a) variation of bending moment and axial force with vertical displacement, left end of 1st story beam; b) development of plastic hinges in 1st story perimeter column from the same analysis (note: plastic deformations in beams are not displayed when FH model is employed).



a)



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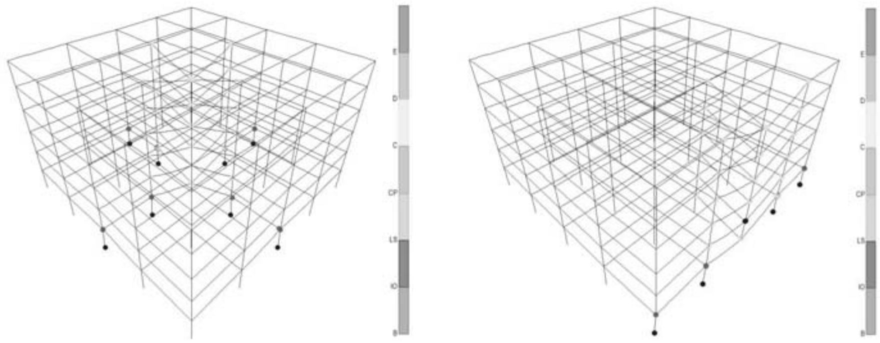
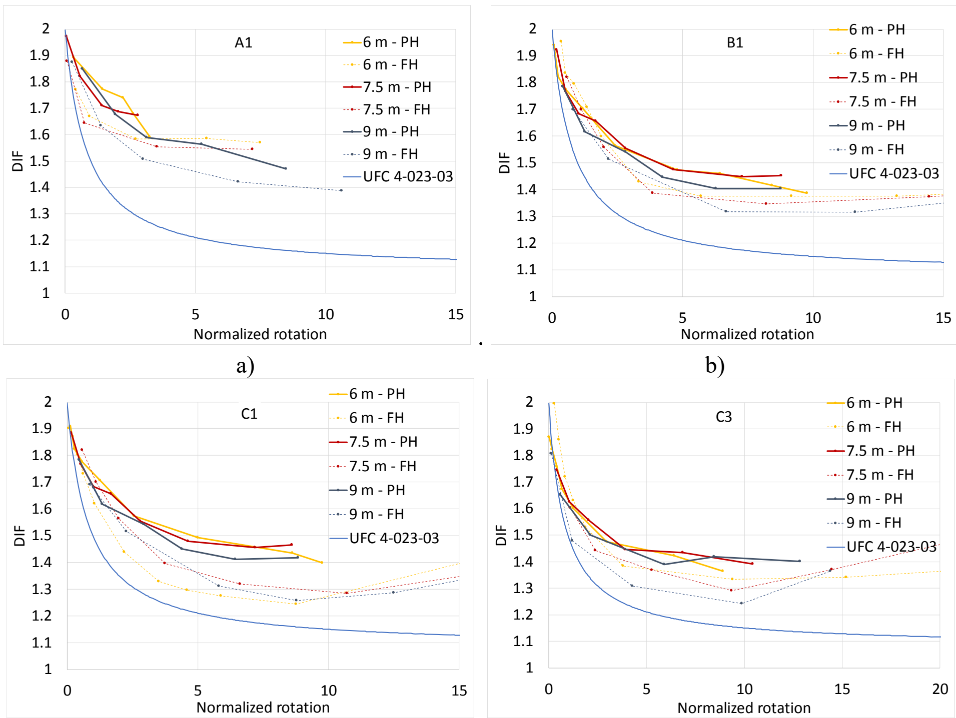


c) d)

Fig. 10 – Deformed shapes and hinge formation at failure, scenarios C3 and C1, PH model: a) 6.0 m span structure; b) 9.0 m span structure.

In Fig. 11, the DIF is shown as a function of the normalized rotation (obtained by dividing the allowable plastic rotation by the rotation at yield in the beams) for each column removal scenario. For each structure and column removal scenario, two curves are plotted, one derived from PH model and one from FH model.



a) b) c) d)

Fig. 11 – Dynamic Increase Factor (DIF) as a function of the normalized rotation: a) scenario A1; b) scenario B1; c) scenario C1; d) scenario C3.

As seen, the results based on PH model are similar for all scenarios and structures as the response is governed by the flexural strength. When FH model is employed, the reduction of DIF with the normalized rotation is alleviated and even starts to increase at large deformations associated with the catenary behavior, especially for C1 and C3 scenarios. For C3 scenario for example, the normalized rotation at the inflexion point varies from 9 to 10. This increase is an indication of additional stiffness and strength provided by the catenary action in resting the gravity loads. Note that in the UFC curves, the smallest normalized rotation for any structural component within the region of the structure affected by the column removal is used to determine the DIF.

4. CONCLUSIONS

Building structures with two-way moment frames, different spans and length-to-height ratios for beams, designed to withstand gravity and lateral loads, were analyzed for several column removal scenarios using SAP2000 program. The analysis was performed using both static and dynamic nonlinear procedures considering modeling criteria prescribed in current progressive collapse design codes and derived from relevant experimental data, respectively. If recommendations for modeling and acceptance criteria are employed (e.g. UFC 4023-03 [2]), the results are conservative. The actual capacity can be larger for damage scenarios allowing the development of catenary action in beams (interior columns), but the additional capacity can be limited by the capacity of the connections to transfer the tension forces. The dynamic response of the structures indicates the Dynamic Increase Factor (DIF) used for Nonlinear Static analyses are larger than those calculated for individual members. Also, at very large deformations, the dynamic increase factors need to account for the additional stiffness and strength associated with catenary behavior in beams.

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