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Seismic and Robustness Design of Steel Frame Buildings

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Abstract. In this paper, a design procedure that combines both progressive collapse design under column removal scenario and capacity design to produce a hierarchy of design strengths is presented. The procedure develops in the context of the European Standards, using the classification of European steel sections and considering the seismic design features. Three-dimensional models of typical multi-storey steel frame buildings are employed in numerical analysis. The design for progressive collapse is carried out with three types of analysis, namely linear static, nonlinear static and nonlinear dynamic. Since the behaviour following sudden column loss is likely to be inelastic and possibly implicate catenary effects, both geometric and material nonlinearities are considered. The influence of the fundamental parameters involved in seismic and robustness design is finally investigated.

Introduction

Buildings behave very differently when surviving a progressive collapse and when withstanding an earthquake since the specific characteristics of progressive collapse are very different from the collapse mechanisms usually experienced under earthquake ground motions. The most significant differences are the initiation by relatively localized damage and the evolution time up to the global collapse. Furthermore, fundamental differences occur also between seismic design that is primarily concerned with lateral loads and robustness design that is more focused with gravity loads. The seismic design procedures generally focus on the plastic mechanism control. The progressive collapse design requires structural strength at large deformations and uses an approach where the local damage scenarios are simply postulated, and acceptability is usually based on the comparison of the maximum ductility demand against the available ductility capacity. Generally, the buildings designed for seismic loads have a good capacity to avoid the global collapse in case of column removal, when compared to buildings designed for gravity loads only. This is because the seismic design criteria generally give ductility and redundancy in the structure. Mainly, the capacity design of columns as key elements may have a beneficial effect on the robustness by delaying or preventing the progressive collapse, especially for high-rise buildings designed for high seismic zones. On the contrary, in the case of low-rise structures in low or moderate seismic zones, the beams may have very little overstrength also for the gravity action, since overstrength should be avoided in such elements. In general, damaged state structural members are loaded in different ways than those originally designed, since large deformations are allowed. Thus, the structural behaviour are quite different from considered in the original design and structural reserves can be mobilized to find alternate load paths: a) vertical load bearing elements acting as suspension; b) Vierendeel action of moment-resisting frames; c) catenary or membrane action of floor systems. Thus, it is unsafe to assume that a structure designed for seismic loads can withstand accidental or abnormal load conditions [1]. This means that the seismic design is not always leading to an adequate robustness, while a specific robustness assessment for seismically designed building structures is required. The sudden column loss technique is commonly used in literature for simulating an abnormal loading condition that first leads to a localized structural failure and then gives rise to a progressive collapse. The benefits of such column removal scenario have been recognized by all the recent USA design codes, which incorporated such a procedure for both progressive collapse assessment and structural robustness design of buildings. In Europe, general studies on this subject are still lacking. The Eurocode 1 [2] presents only a design standard for selecting plan types for preventing progressive collapse, and recommends that buildings should be integrated to improve their resistance. Recent proposed modifications to draft Eurocode standards for the future Eurocode 1 (Part 1-7 "General Actions – Accidental Actions") include a risk assessment procedure that will better correlate the level of design for progressive collapse to the particular structure. This paper aims to evaluate the effects of seismic design and structural integrity requirement on the progressive collapse resistance of steel frame structures. The study moves in the context of the European Standards, using the classification of European steel sections and considering the seismic design features.

Seismic and Robustness Design of Steel Frame Buildings

Design procedure. This paper proposes a design procedure that combines both progressive collapse design under column removal scenario and capacity design to produce a hierarchy of design strengths. The design procedure is based on the following steps:

1) Preliminary seismic design of the building structures.
2) Progressive collapse assessment based on satisfaction of the acceptance criteria.
3) Resizing structural elements, if necessary, to ensure sufficient residual load-path capability after column removal.
4) Seismic verification.
5) Redesign of columns to satisfy the "weak beams-strong columns" capacity design rule.

In order to investigate the load redistribution behaviour of the structure upon the sudden removal of a column, three different analysis procedures, that is linear static, nonlinear static and nonlinear dynamic, were applied. The nonlinear dynamic analysis is able to give accurate solutions. However, sophisticated finite element modelling may be required, and computationally intensive time-history analyses may be necessary to simulate the dynamic behaviour of the damaged structure. Thus, the nonlinear static analysis is often used to take into account the geometrical nonlinearities induced by large deflections without requiring the calculation of the dynamic response time-history. In this case, a "Load Increase Factor" (LIF), accounts for inertial effects.

Case studies. Two typical steel frames building were considered in the analysis. The first design example is a nine-storey office building with perimeter moment frames (Fig.1). The second structure is a six-storey residential building with both internal and perimeter moment resisting frames (Fig.2). For the purposes of the first example, only the perimeter moment frames are classified as primary structures, while all gravity framing is classified as secondary structures. The frames were designed according to the Italian Code (NTC 2008) [3]. The steel material used for all beams and columns is S275, with a lower-bound yield and tensile strength values equal to 275 MPa and 410 MPa, respectively. The steel frames were designed considering the location in a seismic region with soil class C, damping ratio 5%, Peak Ground Acceleration (PGA) of 0.25g and behaviour factor q=6.5. Locations of required columns removals are shown in Figs. 1 and 2.

Analytical Modeling. The seismic and robustness analysis was carried out with reference to real three-dimensional structures, where added stiffening is provided by the orthogonal frames and horizontal floor slabs. The higher redundancy provided by the floor slabs may influence the vertical-horizontal wide propagation of collapse. In this paper the load redistributions through the floor slabs was neglected, and this should provide conservative estimates of the progressive collapse resistance. Beams and columns were modelled using a concentrated plastic hinge model. The connections were considered stronger than the beams.

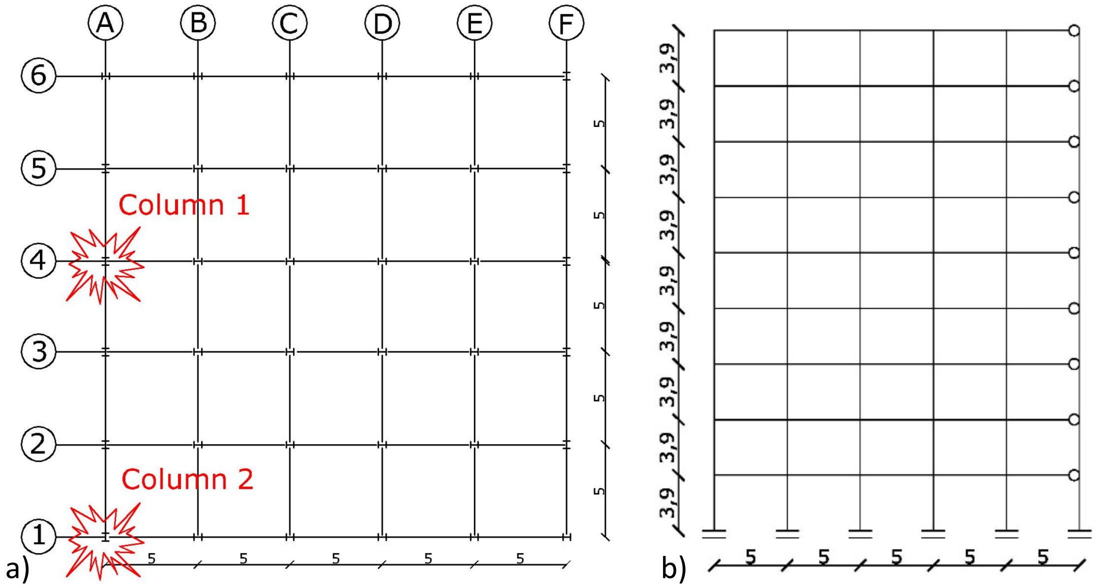

Figure 1. Nine-storey building. a) Plan view. b) Perimeter moment frames

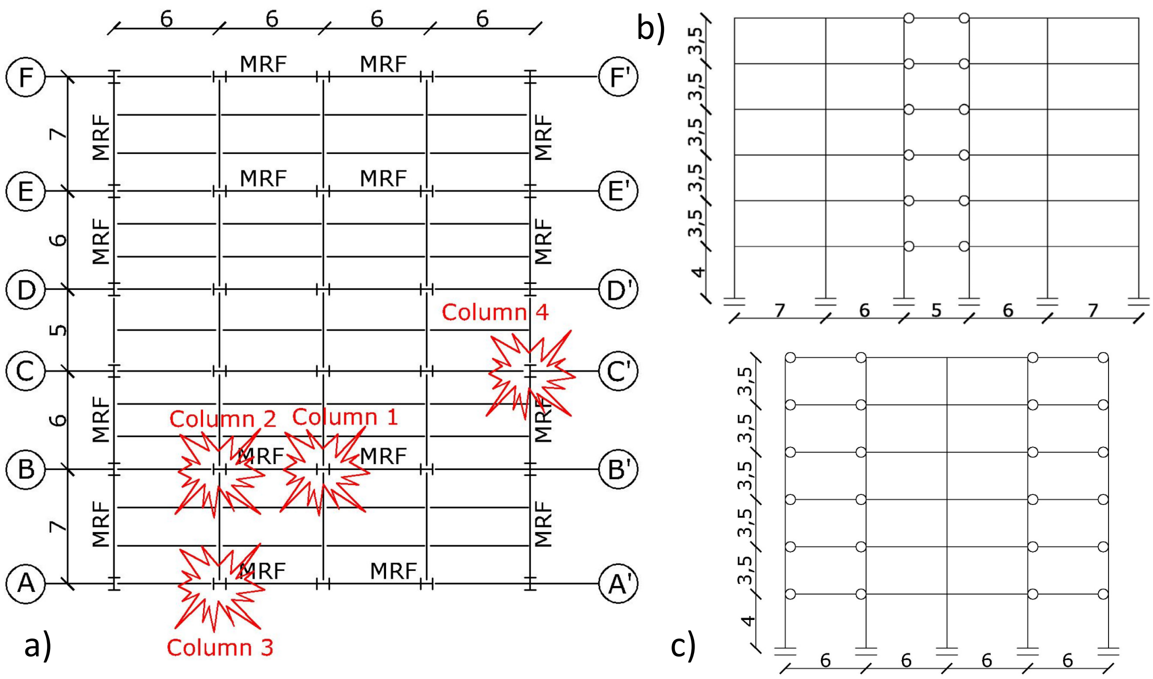

Figure 2. Six-storey building. a) Plan view. b) Frames in y-direction. c) Frames in x-direction

The beam-to-column connections and the panel zones were not modelled. The beam-to-column joints were assumed to be rigid and full-strength. Thus, the model allowed plastic hinges to form in beams and columns, but not in connections. The effects of the tensile loads transfers to the beam-column connections and their conservative nature were not investigated in the present paper. The secondary members, such as transverse joist beams and braces, were considered only for transferring of the gravity loads, while they did not directly contribute to the progressive collapse resistance. The model was based on the assumption that the foundation can accommodate the redistributed loads following any column removal, and that connections at the foundations may be modelled as restrained joints. The program code SAP2000 [4] was used to implement a 3D analytical model. The structural model for nonlinear analysis incorporates material and geometric nonlinearities. Moreover, both P-delta and large displacements effects were considered. Theoretically, concentrated plastic hinges can occur anywhere along the beam. However, the hinges were allowed to occur at the ends of each member. This simplifies the model by placing flexural plastic hinges in the most probable locations. The plastic hinges were represented by nonlinear moment-curvature and P-M interaction relationships for beams and beam-columns. The parameters of the plastic hinges were defined based on Chapter 5 of the FEMA-356 code [5]. The sudden strength degradation was neglected, since the acceptable plastic rotation angle of the steel members, as defined in FEMA-356, is always within the first post-yield linear branch of the moment–rotation curve (preceding the strength degradation). The geometric imperfections of columns usually affect the capacity of the column in axial compression. However, for the case of progressive collapse, the load eccentricity caused by column removal gives a great destabilizing load to the adjacent-to-the-removal columns. This effect is expected to be more important due to the moment affecting the compression capacity of the column. Thus, the imperfections of the members were neglected.

Linear static procedure (LSP). In accordance to GSA 2013[6], the Linear static procedure (LSP) is based on the following steps: 1) Demand-Capacity Ratios (DCRs) and irregularity check; 2) Classification of Deformation Controlled and Force Controlled Actions; 3) Determination of m-Factors and Load Increase Factors; 4) Alternate Path Analysis. If the structure contain irregularities, in order to determine whether the LSP can be used, the designer must evaluate the limits for the Demand-Capacity Ratios, defined as follows:

$$\mathrm{DCR} = \mathrm{Q}_{\mathrm{UDLim}} / \mathrm{Q}_{\mathrm{CE}} \tag{1}$$

where Q_{UDLim} is the action (internal force or moment) obtained from the deformation-controlled load case with gravity dead and live loads increased by the load increase factor Ω_{LD}, and Q_{CE} is the expected strength of the element. The structures here examined does not produce the irregularity limitations as: 1) they do not have any vertical discontinuities; 2) bay stiffness/strength does not vary in either direction at corner columns; and 3) all lateral-load resisting elements are parallel to the major orthogonal axes of the building. Therefore, the LSP can be used. An m-factor, or demand modifier, which is determined from Table 5-1 of ASCE 41 [7], is assigned to each component within the structure. Load increase factors (LIF) are applied to the area immediately affected by the removed column. The LIF for the model to determine acceptability of force controlled actions are equal to two (Tab.1). The LIF for the model to determine acceptability of deformation controlled actions are dependent on the lowest m-factor for a component within the region of load increase. The m-factors for each column removal location and the corresponding LIF are shown in Tables 1 and 2. Three different load combinations are considered in the analysis:

$$\text{a)}\, \mathrm{G}_{\mathrm{LD}} = \Omega_{\mathrm{LD}}\left[1.2\,\mathrm{D} + 0.5\,\mathrm{L}\right] \qquad \text{b)}\, \mathrm{G}_{\mathrm{LF}} = \Omega_{\mathrm{LF}}\left[1.2\,\mathrm{D} + 0.5\,\mathrm{L}\right] \qquad \text{c)}\, \mathrm{G} = 1.2\,\mathrm{D} + 0.5\,\mathrm{L} \tag{2}$$

The load combinations of Eq. 2a and 2b are applied in the bays immediately adjacent to the removed column, respectively for deformation or force-controlled actions. For those bays not immediately adjacent to the removed element, the load combination of Eq. 2c is used, for both deformation and force-controlled actions. In Eq.2, G=Gravity load, L=Live load including live load reduction, D=Dead Load, G_{LD} = Increased gravity loads for deformation-controlled actions, G_{LF} = Increased gravity loads for force-controlled actions. After alternate path analysis, the Demand-Capacity-Ratio (DCR) of each component is evaluated and compared to acceptance criteria. For deformation-controlled elements, the DCR is compared to the governing m-factor for the element and its connections. For force-controlled elements, the DCR must be less than 1.0. The classification of deformation and force-controlled actions was performed in accordance with GSA 2013 [6] and guidance provided in ASCE 41 [7]. Evaluation of whether columns are deformation or force controlled is a function of the axial load under the column removal scenario. Therefore, a check was carried out after completing the analysis to verify the assumption of deformation-controlled actions for columns. In accordance with ASCE 41 [7], any column with an axial load ratio greater than or equal to 0.5 must be reclassified as force-controlled and revaluated under the force-controlled modeling assumptions. The analysis results are shown in Fig.3-4. Resulting DCRs of each element are shown directly below the section size. Structural members in red indicate that the acceptance criterion is not met and, therefore, upgrade was required.

Table 1. Component m-factors and load increase factors (9-storey building)

Removed Column	m_{LIF} (Smallest m-factor)	$\Omega_{LD}= 0.9\ m_{LIF} + 1.1$ LIF for Deformation Controlled Actions	Ω_{LF} LIF for Force Controlled Actions
9-storey building			
1	2.46	3.32	2.00
2	2.59	3.44	2.00
6-storey building			
1	2.30	3.16	2.00
2	2.34	3.21	2.00
3	2.34	3.21	2.00
4	2.10	3.00	2.00
5	2.23	3.10	2.00
6	2.29	3.16	2.00

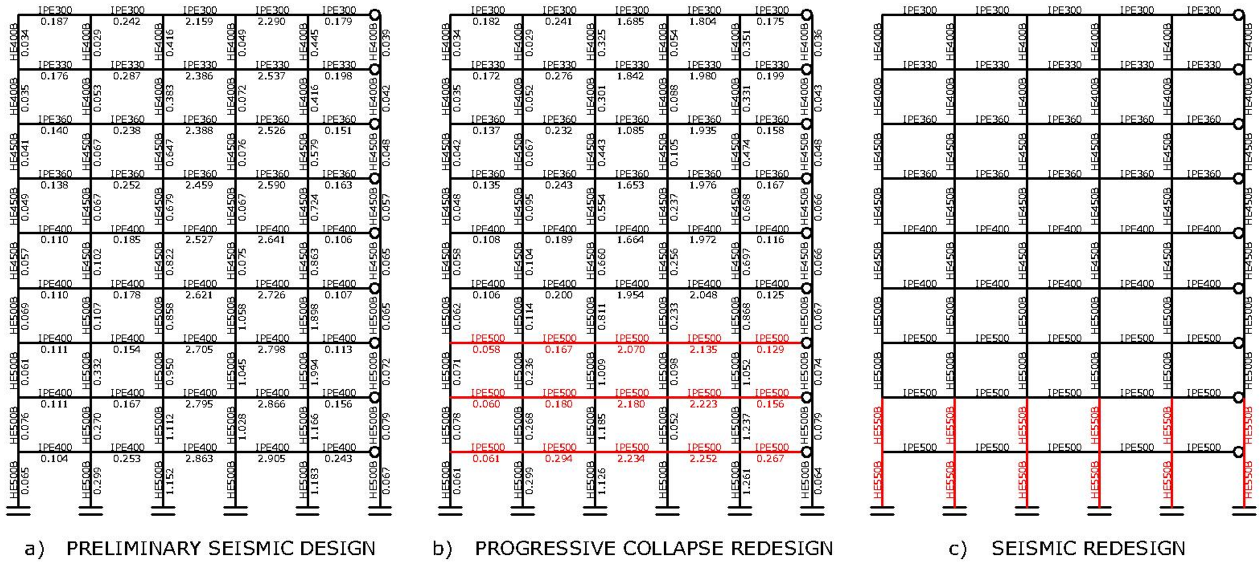

Figure 3. Moment Demand-to-Capacity Ratios. Frame A. 9-storey Building. C1 Removal Scenario. a) Preliminary Seismic Design. b) Progressive Collapse Redesign. c) Seismic Redesign

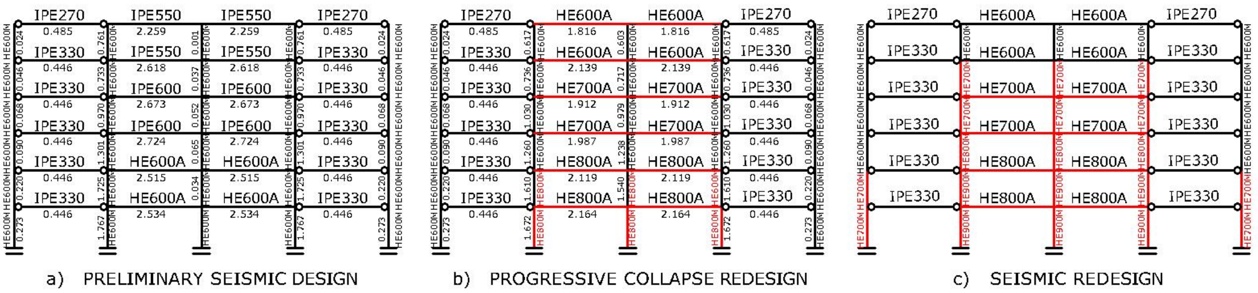

Figure 4. Moment Demand-to-Capacity Ratios. Frame B. 6-storey Building. C1 Removal Scenario. a) Preliminary Seismic Design. b) Progressive Collapse Redesign. c) Seismic Redesign

Nonlinear static procedure (NSP). The sudden removal of a structural member has the same effect as the sudden application of the structural forces in those members in the opposite direction. The conventional nonlinear equivalent static approach (also termed "pushdown analysis") attempts to reproduce this effect examining the structure that has suffered the loss of one or more critical members under increasing gravity loads. A dynamic increase factor (named Ω_N) is applied in the bays immediately adjacent to the removed column for deformation-controlled actions. The latest version of both GSA [6] and UFC [8] adopt the formula originally proposed by Marchand [9]:

$$\Omega_N = 1.08 + \frac{0.76}{\theta_{pra}/\theta_y + 0.83} \tag{3}$$

where θ_{pra} is the prescribed maximum acceptable plastic hinge rotation angle and θ_y is the yield rotation angle. The plastic rotation angle θ_{pra} is given in the acceptance criteria tables in ASCE 41

[7] for the appropriate structural response level (Life Safety or Collapse Prevention), while the yield rotation θ_y for steel is given by Equation 5-1 of ASCE 41. In Eq.3, θ_{pra}/θ_y is the smallest ratio among the structural components (excluded columns) that contribute to progressive collapse resistance and are within the immediately affected bays. Eq.3 is governed by the acceptance criteria of the structural members only. Recent studies [10-12] evidenced that this formulation becomes inaccurate for nonlinear static responses involving the hardening phenomenon associated with the catenary action. The acceptance criteria suggested by FEMA-356 [5] and GSA were checked. Fig.5 shows the pushdown curves of the 9-storey (Fig.5a-5b). The collapse load factor in case of design for vertical loads only is lower than one. Thus, the structural system has not adequate resistance to progressive collapse. Otherwise, in case of seismic design, the collapse load factor is greater than one for both column removal scenarios. Thus, the seismic design succeeds in preventing progressive collapse of the 9-storey building. On the contrary, the 6-storey buildings designed for seismic loads has a collapse load factor lower than one for the C2 column removal scenario. This confirms that the seismic design does not lead to an adequate robustness since the structure is not able to withstand destructive loading conditions that accompany a column removal.

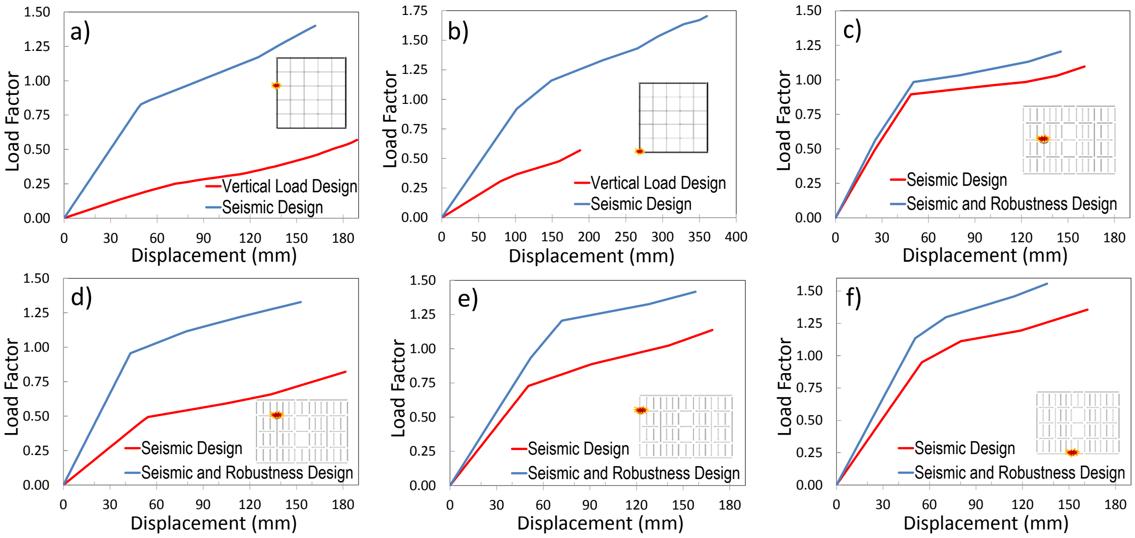

Figure 5. Pushdown curves. a-b) 9-storey building. c-f) 6-storey building

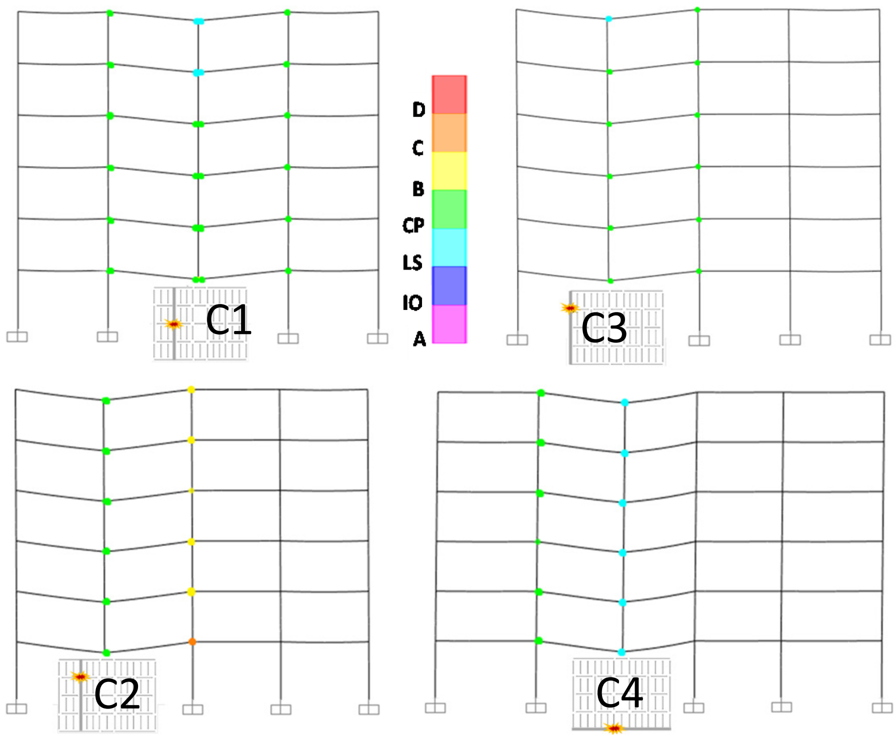

Figure 6. Plastic hinges. Pushdown of 6-storey building designed for seismic loads

In this case, only a specific progressive collapse design can guarantee an adequate robustness. In Fig.6, the plastic hinge distribution at the last step of the pushdown analysis is plotted. All the beams of bays immediately adjacent to the removed column form plastic hinges at the restrained beam-column connections. In the C2 scenario, a plastic hinge on the first floor reaches the collapse limit state.

Nonlinear dynamic procedure (NDP). The process that dynamically simulates the sudden loss of a column was implemented as follows:

1) The vertical loads are statically applied on the undamaged model. End forces of the to-be-removed target column are recorded (i.e. axial force *N*, shear force *V* and bending moment *M*).

2) The column removed is replaced by the corresponding reaction forces. The gravity loads (DL+0.25LL) and the calculated end forces in inverted directions (i.e. -*N*, -*V*, -*M*) are statically applied to the damaged frame. This application takes 1s (during which loads are amplified linearly until they reach their full amounts) and then it is kept unchanged for 9s, so that the structure can reach a stable condition that replicates the state of the structure before the column loss.

3) The reaction forces are simultaneously and abruptly brought to zero to simulate the sudden removal of the target column. The removal time is set to 10ms to simulate the instantaneous column loss. The mass-proportional and the stiffness-proportional damping coefficients are calculated to achieve the real critical damping ratio of 2% for both the first and second mode shapes.

Fig.7 shows the plastic hinge distribution at the end of the dynamic analysis. Also with this method of analysis, the progressive collapse occurs in the C2 column removal scenario with a yielding-type mechanism.

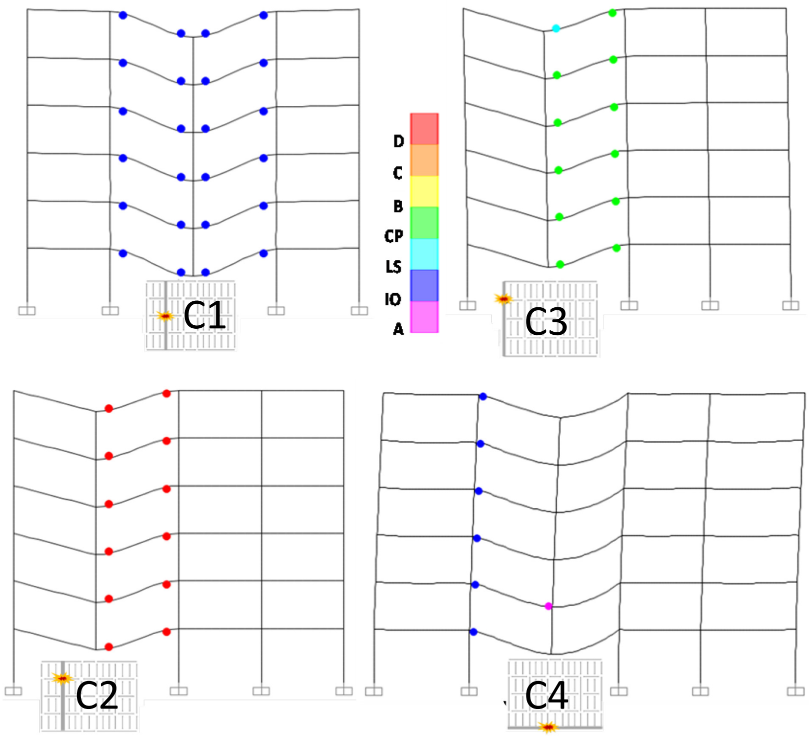

Figure 7. Plastic hinges. NDP of 6-storey building designed for seismic loads

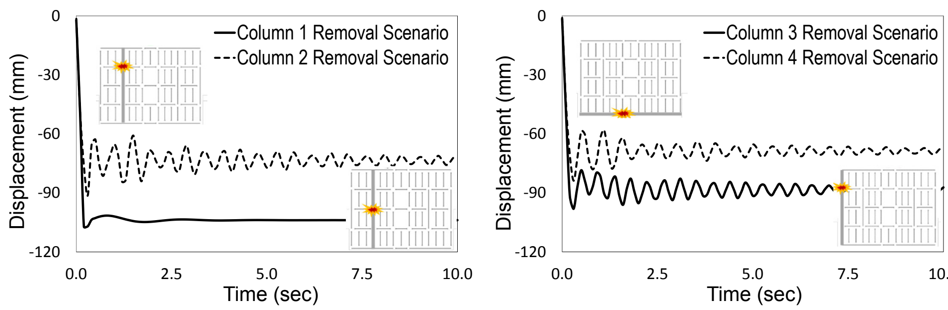

Figure 8. Displacement time-histories. 6-storey building designed for seismic loads

In Fig.8, the vertical displacement time-history at the joint where the column is removed is plotted. The starting time of the time-history plot begins at the time when the column is removed. The location of the removed column has a very strong impact on the behaviour of the structure. The C2 scenario shows non-oscillating vertical displacements with values higher than the other scenarios.

Conclusions

A design procedure was presented that combines both progressive collapse design under column removal scenario and capacity design to produce the hierarchy of design strengths. The proposed procedure was applied to two typical steel framed building using linear static, nonlinear static and nonlinear dynamic analysis. The results showed that it is unsafe to assume that a structure designed for seismic loads can withstand accidental or abnormal load conditions. In fact, the seismic design did not always lead to an adequate progressive collapse resistance to any column removal scenario, and this conclusion was reached whatever the method of analysis used. This means that seismic and progressive collapse design must integrate with each other to ensure both sufficient residual load-path capability after column removal and aptitude to withstand seismic action.

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