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VULNERABILITY TO PROGRESSIVE COLLAPSE OF SEISMICALLY DESIGNED RC FRAMED STRUCTURES: CORNER COLUMN CASE

BY

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Abstract. As in the seismic design, to resist such catastrophic loads, structures should be provided with an adequate level of structural continuity, redundancy, robustness and ductility, so that alternative load transfer paths can develop when the structure loses an individual member. Following the GSA Guidelines (2003), the paper presents an investigation regarding the vulnerability to progressive collapse of a model representing a 13-storey RC framed building when its seismic design was made according to the provisions of the present seismic code P100-1/2006. . Numerical results regarding the behavior of the model when the structure is damaged by the sudden removal of a corner column are given. Demands and capacities of structural members are assessed and DCR values for the lower part of the building are presented. A typical medium–rise building having RC frames seismically designed for Bucharest according to seismic design code P100-1/2006, does not experience failures or progressive collapse when subjected to different “missing column” scenarios, including the removal of a corner column.

Key words: progressive collapse; RC frames; seismic design; GSA Guidelines.

1. Introduction

The main causes leading to a structural progressive collapse of buildings, seen as a chain reaction of failures that propagates throughout a portion of structure, disproportionate to the original local failure [1], are: fire, wind gusts, floods and human errors, impact by vehicles, but especially major earthquakes and blasts.

The design philosophy of structures subjected to abnormal loads – as they are defined in GSA Guidelines (2003) [2], Section 2 (“other than conventional design loads dead, live, wind, seismic”) – is to prevent or mitigate damage, not necessarily to avoid the collapse initiation from specific cause.

This approach is similar to the concept adopted in any modern earthquake-resistant design codes. Whereas resistance to progressive collapse is primarily an issue of gravity load-carrying capacity, the design of elements (beams, columns) also depends on demands from other actions such as wind or seismic actions. It means that, if beams, columns or joints of a framed structure had a larger load-bearing capacity due to more sever seismic actions considered in design, these elements would have a higher capacity to confine the damage to the initially affected zone, and consequently to prevent progressive collapse [3].

In the assessment methodology for the potential progressive collapse according to [2], engineers should consider the loss of portions of the structure using different "missing column" or "missing beam" scenarios (Fig. 1). Such checks are required in the currently used design codes for the reinforced concrete structures, though the cause is not always specified (natural hazard or man-made hazard). In [1], [4],[5] – using the GSA criteria – it is shown that medium-rise building having RC framed structures seismically designed for zone of moderate or high seismic risks do not experience progressive collapse when subjected to the removal of an exterior or interior column.

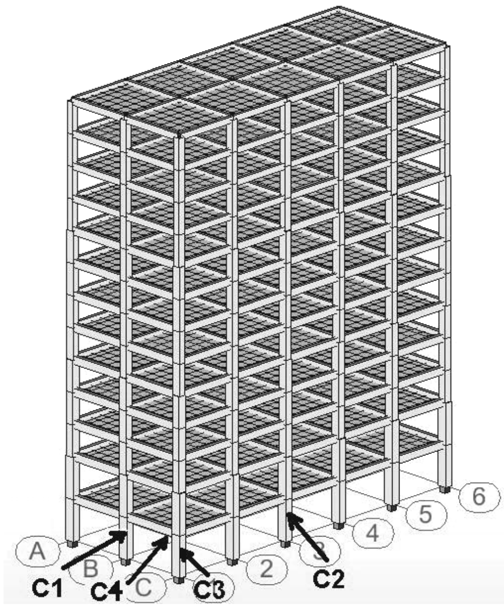


Fig. 1 – Model of a 13-story RC building: missing column scenarios.

In Romania, the change of the seismic design codes from P100-92 to P100-1/2006 [6], as well as the change of the design code for reinforced concrete structure from STAS 10107/09-90 into the new EC-2 (SR EN 19992-1-1: 2004 [7]), has effects in the magnitude of internal forces used in the seismic design, and also in the detailing process of structural members.

To investigate the resistance to progressive collapse in this new situation, two models, representing a 13-storey building located in Bucharest, have been seismically designed and detailed according to the former codes (model P100-92), and according to the present design codes [6], [7] (model P100-2006).

The structure consists of five 6.0 m bays in the longitudinal direction and two 6.0 m bays in the transversal direction and has a storey height of 2.75 m, except for the first two floors that are 3.60 m high. Structural responses of the “undamaged “structures and the behavior of “damaged “structures in the case C1 (removal of an exterior column near to the middle of the short side) have been analysed using the FEA computer program [8], and the main results have been recently published [9].

In this paper, numerical results, comparative analyses and commentaries regarding the behavior of the model P100- 2006 subjected to a sudden removal of a corner column are presented, and its vulnerability to progressive collapse is analysed following the GSA (2003) criteria [2].

2. Assessment of the Potential to Progressive Collapse

For buildings having ten storyes or less in height, with relatively simple layouts, it is recommended [2] the alternate load path method (APM), based on a linear elastic analysis, to assess the vulnerability of a new and existing building to progressive collapse. Normally used for buildings 10-storeys above grade and less, the method proposed in [2] can be successfully applied to taller buildings [1], [4], [5].

To determine the potential of progressive collapse of a typical RC structure, designers can perform a structure linear elastic analysis, considering the instantaneous loss of one of the first floor columns (“missing column” scenarios), as follows (Fig. 1):

a) an exterior column near to the middle of the short side (case C1);

b) an exterior column near to the middle of the long side (case C2);

c) a column located at the corner of the building (case C3 – investigated in this study);

d) a column interior to the perimeter column live fore facilities that have underground parking and/or uncontrolled public ground flow areas (case *C*4).

The sudden loss of a load bearing element (column in this analysis) generates in the “damaged” structure dynamic actions (moments, shear and axial forces) and the structural response is nonlinear. But, as in the routine seismic design, one simple approach is to use an equivalent linear elastic procedure, considering that the increased vertical forces to be applied to the structure are

$$\text{Load} = 2(\text{DL} + 0.25\text{LL}) = \text{Load}^{\text{static.}}, \tag{1}$$

where: DL is the dead load and LL – the live load. In the GSA criteria, live load is reduced since the probability of that the entire full live load being present during the event, is small. In the same time, by multiplying the static load combination by a factor of 2.0, the method takes into account the dynamic amplification effect on a linear – elastic structure [9], due to the instantaneously removed of a vertical support (column).

With this increased gravity forces ($2\text{Load}^{\text{static.}}$), demands ($Q_{UD}$) in structural elements and connections are determined in terms of bending moments, axial forces and shear forces. Following the linear static analysis, a Demand-Capacity Ratio (DCR) is computed for each structural element

(2) $$\text{DCR} = \frac{Q_{UD}}{Q_{CE}},$$

where Q_{CE} is the expected ultimate, un-factored capacity (bending moment, axial forces, shear forces) of the structural component. In the assessment of Q_{CE}, strength increase factors are applied to the properties of the construction materials to account for strain rate effect and material over-strength [1]. For reinforced concrete structures, a material increase factor of 1.25 is allowed for concrete and reinforcing steel [2].

In order to prevent collapse of the "damaged" structure, the DCR values for each structural element must satisfy the inequality

a) DCR $\leq$ 2.0 for typical structural configurations, and

b) DCR $\leq$ 1.5 for atypical structural configurations [2].

Using the DCR criteria, structural members and connections that have DCR values greater than 2.0 are considered to be severely damaged or collapsed. If all the computed DCR values are less than or equal to 1.0, then the component is expected to respond elastically to the adopted missing column scenario. In routine design, an element with DCR greater than 1.0 has exceeded its ultimate capacity [1].

3. Progressive Collapse of the Damaged Model

3.1. Model P100-2006: Seismic Design

The model was designed according to provisions of present codes ([6] and [7]) at the Ultimate Limit State, under the following load combination:

(3) $$\text{DL} + 0.4(\text{LL} + S) + E,$$

representing a combination of dead load, DL (self-weight and a supplementary dead load of 2 kPa), live load, LL = 2.4 kPa and snow, S = 1.28 kPa, with a load long term factor of 0.4, and the earthquake effect (E).

For a RC framed structure located in Bucharest, the magnitude of base shear force given by the code [9] is

$$F_b^{\text{P100-2006}} = \gamma_1 \frac{a_g \beta_0}{q} \lambda m = 1.2 \times 0.24 \times 2.75 \frac{1}{6.75} 0.85G = 0.09973G \,, \tag{4}$$

where the behavior factor, q, for frame systems in the ductility class, H, is

$$q = 5.0 \frac{\alpha_u}{\alpha_1} = 5.0 \times 1.35 = 6.75 \,. \tag{5}$$

The structure's response is determined – *via* the modal analyses – with the 3D linear elastic model, created and analysed in the FEA program [8].

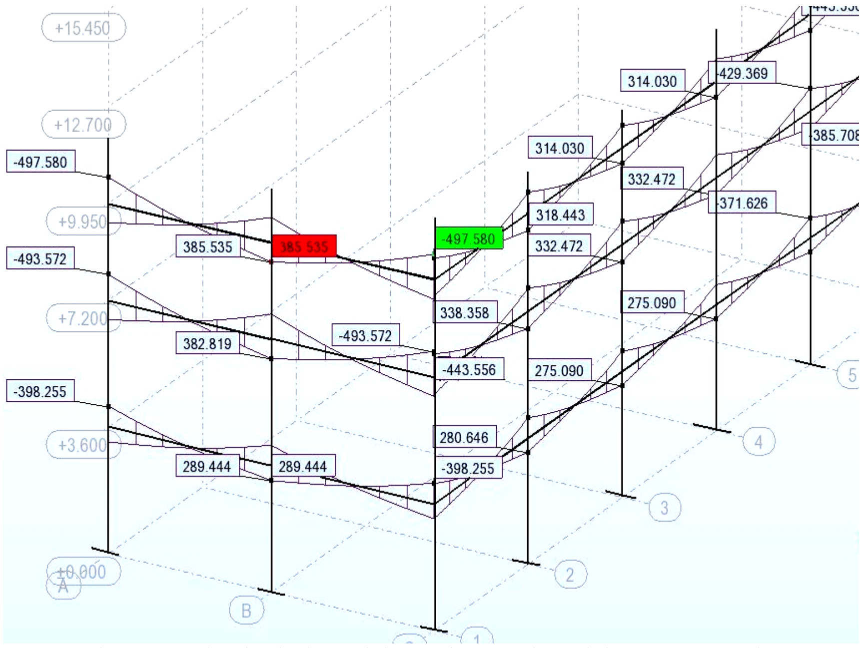


Fig. 2 – Seismic design of the undamaged model P100-2006: the envelope of bending moments in beams.

Bending moments (Fig. 2), shear forces and axial forces are obtained from the modal response spectrum analysis, and the reinforcement of beams and columns (Table 1) is made considering the provisions of EC-2 [7], as well as the supplementary measures required by the design of elements in the high ductility class (*H*) [6]. The maximum positive bending moment of 338.5 kNm is located in the third floor beam of the exterior transverse frame, CT1 and it is by 14% greater than the maximum positive moment (338.3 kNm) from the exterior longitudinal frame beams (Fig. 2).

Table 1

Design Details of Structural Elements for the Model P100-2006

Storey	Column*	Longitudinal beams*	Transverse beams*	Transverse and longitudinal beams*		
	dimensions mm	dimensions mm	dimensions mm	Top long. steel**	Bottom long. steel**	Stirrups at ends**
1, 2	700 × 900	350 × 650	350 × 700	2Ø25 + 2Ø22	3Ø25	Ø10/150
3, 4, 5	700 × 750	350 × 650	350 × 700	2Ø25 + 2Ø22	3Ø25	Ø10/150
6, 7, 8, 9	600 × 750	300 × 650	300 × 700	4Ø22	3Ø22	Ø10/150
10, 11, 12, 13	600 × 600	300 × 550	300 × 600	4Ø18	3Ø18	Ø8/120

*Concrete: C30/37 according to [7].
**Reinforcing steel: S500- type.

3.2 Model P100-2006: Corner Column Removal (Case *C*3)

The sudden removal of the corner column generates an increase of maximum positive and negative bending moments in beams, compared to the design moments given in Fig. 2.

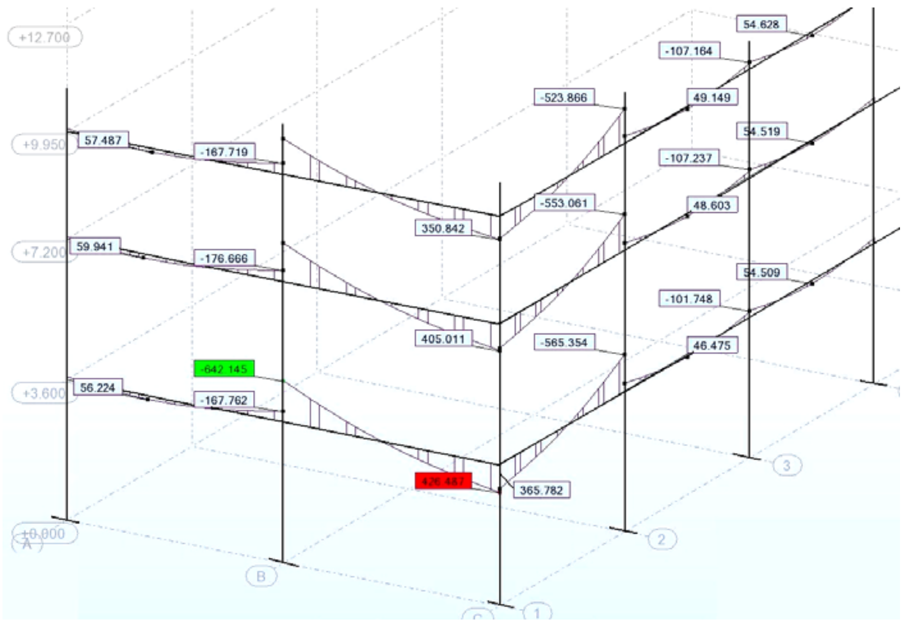


Fig. 3 – Damaged model P100-2006: bending moments, [kN], in beams, after the removal of a corner column.

Thus, the maximum positive moment increases from 385.5 kN.m to 426.4 kN.m (Δ = +29%) for beams of the exterior transverse frame CT1, and from 338.3 kNm to 371.1 kN.m in the exterior longitudinal frame CLC (Fig. 3).

In the same time, the maximum negative bending moment increases by 29% in the frame CT1, and by 27.5% in the longitudinal frame CLC (from –497.5 kN.m to –642.1 kN.m, respectively from –443.5 kN.m to –565.3 kN.m).

Being seismically designed, the model P100-2006 has large amounts of top and bottom longitudinal reinforcement (A_{s2} = 2Ø25+2Ø22, A_{s1} = 3Ø25) that provide flexural capacities over the "missing column".

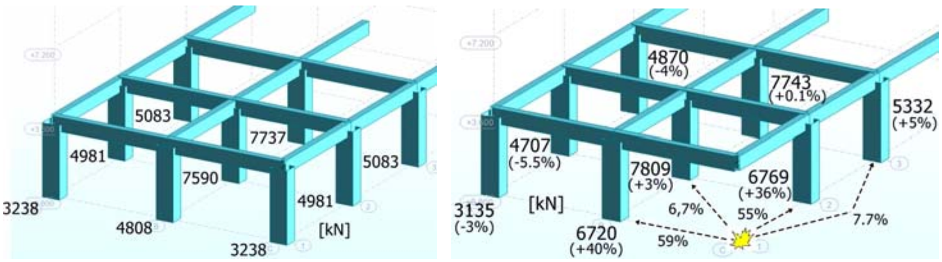
The beams from the exterior longitudinal and transverse frames, direct affected by the "loss" of a load-bearing support (the corner column), have enough local strength and ductility to overtake that 30% variation in the magnitude of bending moments.

The load carried by the removed corner column (axial force of 3,238 kN – Fig. 4 *a*) has to be redistributed to the neighboring columns.

An important amount of the axial load (59%) is transferred to the middle short side column, and also (55%) to the first column (C2) of the exterior longitudinal frame (Fig. 4 *b*).

Results in the same range (26% to 40%) are reported in technical literature ([3] and [9]), and they confirm that after redistribution, the axial forces in certain columns may increase up to 40%.

In the same time, due to the frame (Vierendel) action, the axial forces in several columns located farther away from the damaged column, decrease as it is shown in Fig. 4 *b*.



a *b*

Fig. 4 – Redistribution of axial forces, [kN], after the removal of a corner column.

3.3. Demands and Capacities in Beams

The bending moments and shear forces are evaluated in the "damaged" structure and bending moment diagrams in beams are shown in Fig. 3. The largest moments in beams are developed at the first floor (426.4 kN.m, respectively –642.1 kN.m) and they decrease by each floor as they move up to the height of the model. Following the procedure indicated in Guidelines [2], demands in beams, Q_{UD} – at columns faces – are assessed and compared to the expected ultimate beam capacities, Q_{CE}. It has to be underlined that Q_{CE} is evaluated using the characteristic strengths of the materials multiplied by 1.25, the material increase factor to account for strain rate and material over-strength.

DCR values for significant beam sections are represented for the lower part of the damaged model in Fig. 5 for flexure, and in Fig. 6, for shear. All of the DCR values for flexure and shear are below 1.0, even in the critical zone of the first floor where the corner column was removed. At the first three floors (Fig. 5), the DCR values in beams decrease from 0.827 to 0.763 in the first bay of the longitudinal frame CLC, respectively from 0.79 to 0.69 in the first bay of the transverse frame CT1. For a similar case (removal of a corner column), in [1] B a l d r i d g e and H u m a y reported a maximum DCR value of 0.94 for negative bending moments and 0.78 for positive moments, values which are in the same range with authors' results (0.827 and 0.76).

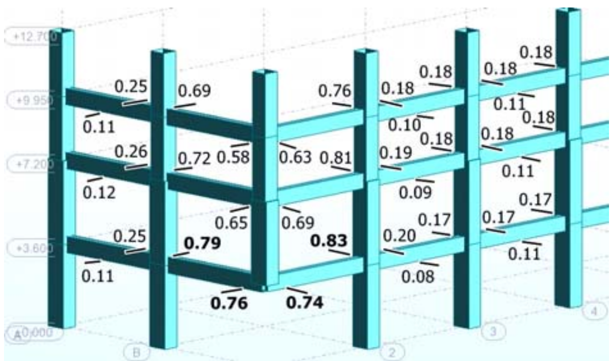


Fig. 5 – Damaged model P100-2006: DCR values for flexure.

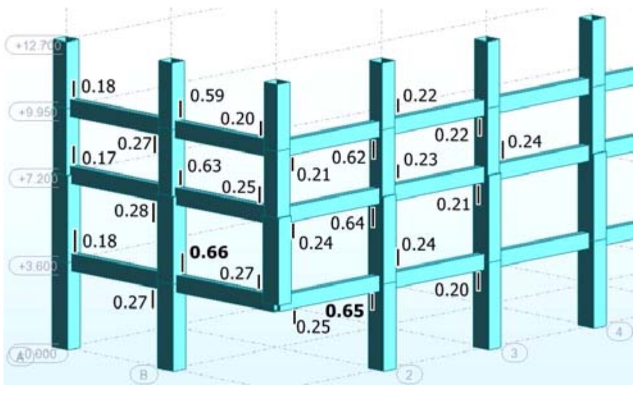


Fig. 6 – Damaged model P100-2006: DCR values for shear.

DCR values for shear (Fig. 6) are also well below 1.0, the maximum value (0.664) being recorded at the first floor beam of the exterior transverse frame CLC.

If all DCR values are below 1.0, the structure damaged by the removal of a corner column remains in the elastic stage, and consequently no other structural component (beam, column, joint, slab) is expected to fail in shear or flexure. The progressive collapse is not expected to occur when the model is designed according to provisions of the seismic design code P100-1/2006.

4. Conclusions

The GSA Guidelines (2003) offer a realistic approach and performance criteria to determine the potential for progressive collapse using the concept of DCR [2]. To evaluate potential progressive collapse of structures, GSA Guidelines and other regulations require removal of a load-bearing column.

In this paper, a RC framed structure has been designed according to the provisions of the present codes P100-1/2006 – for seismic design and EC2 – for RC structure design, and its vulnerability to progressive collapse was determined considering a particular case from the "missing column" scenarios: removal of a corner column.

The present analysis shows that a typical medium rise building (13 storeys in the present investigation), having RC frames and seismically designed for Bucharest – a zone of high seismic risk (a_g = 0.24g) – does not experience progressive collapse when subjected to different "missing column" scenarios. In the paper numerical results are given only for case C3, when a corner column of the building is removed.

The numerical results, especially the fact that all DCR values have been below 1.0, show an elastic response of the damaged structure and indicate the possibility that similar structures erected even in moderate seismic areas, for instance zones with a_g = 0.20 will be able to fulfill the requirements for a structure with low potential for progressive collapse.

Further analyses are required to determine the vulnerability to progressive collapse of other types of structural system, including 7 to 9 storey buildings, erected in seismic zones having $a_g \geq 0.20$g.

The provisions of the new codes (P100-1/2006, SR EN 1992-1-1:2004) in the seismic design of RC framed structures, lead to beneficial effects in the structural response to abnormal loads and decreases the potential of progressive collapse.

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VULNERABILITATEA LA COLAPS PROGRESIV A STRUCTURILOR DIN BETON ARMAT PROIECTATE SEISMIC: CAZUL STÂLPULUI DE COLȚ

(Rezumat)

La fel ca şi în cazul proiectării seismice, structura trebuie să dispună de un nivel corespunzător de continuitate, redundanță, robustețe şi ductilitate, astfel încât să-şi poată dezvolta o cale alternativă de transfer a încărcărilor, atunci când își “pierde” un element de rezistență. Urmând recomandările Ghidului GSA (2003), în lucrare se prezintă rezultatele unei investigații privind vulnerabilitatea la riscul de colaps progresiv al unui model de structură reprezentând o clădire în cadre cu 13 nivele, atunci când aceasta a fost proiectată seismic cu respectarea prevederilor actualului cod P100-1/2006. Sunt prezentate rezultate numerice legate de comportarea modelului pentru cazul în care structura este avariată prin îndepărtarea bruscă a stâlpului de colț. Sunt calculate şi comparate solicitările şi capacitățile de rezistență ale elementelor structurale semnificative. Se confirmă faptul că o clădire tipică, de înălțime medie, având structura în cadre de beton armat, proiectată seismic (normativul P100-1/2006) pentru zona Bucureşti, nu va suferi avarii şi nu va fi expusă riscului de colaps progresiv, atunci când îşi “pierde” un element de rezistență, inclusiv în cazul îndepărtării bruşte a stâlpului de colț.