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Progressive Collapse Risk and Robustness of Low-Rise Reinforced Concrete Buildings

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Abstract

The potential for progressive collapse of low-rise RC framed structures is evaluated in this paper. The analyses are based on the linear static procedure and acceptance criteria specified in GSA 2003 Guidelines. The main parameters of the program are the building height (three and six storeys), seismicity of the area, and the type of damage scenarios used in the analysis. The results revealed that the progressive collapse potential decreases as the number of storeys increases and, for the first time in the technical literature, demonstrates in a quantitative manner, differentiated with respect to the number of storeys, the positive effect of seismic design and detailing on the progressive collapse resistance of reinforced concrete framed structures.

Keywords: progressive collapse, low-rise buildings, seismic zones, ductility.

1 Introduction

The vulnerability of structures to progressive collapse became an interesting subject after the collapse of Ronan Point Apartment Building, England, in 1968. A second wave of interested followed the terrorist attacks on the A. Murrah Federal Building in Oklahoma City, U.S., in 1995. Interest now is at a high level because structural engineers still study and try to respond to the collapse of the buildings at the World Trade Centre in New York City and of a portion of Pentagon in Washington, all resulting from a well-coordinated terrorist attack in September 2001 [1].

Abnormal loads, other than conventional design loads (dead, live, wind, seismic) for structures such as air blast pressures generated by an explosion, vehicle impacts, fires or other hazards, can lead to progressive collapse. This phenomenon can be described as a chain reaction type of failure that can imply the collapse of the entire building or of a disproportionate part of it (Figure 1).

Figure 1: a) Ronan Point Apartment Building b) A. Murrah Federal Building.

Many interesting conferences, symposiums and workshops, as well as a large number of theoretical analyses and experimental programs, underlined the following principles [2]:

- Design to resist progressive collapse should be considered as a design for an advanced limit state since it already assumes that a local portion of the structure has failed;
- With respect to progressive collapse, a successful design is one which results in a structure that has the capacity to limit a local failure to the immediate area of the failure;
- Reinforced concrete and steel frames pose little danger of progressive collapse, except for extreme case of direct sabotage of bearing members;
- Structures located in higher seismic zones would have resistance to progressive collapse, since the seismic design would include detailed requirements to accommodate the high lateral seismic load as well as vertical loads.

Practically, to mitigate the risk of progressive collapse due to abnormal loading event, a structure must accommodate the initial local damage and develop an alternative load path to sustain the redistributed loads [3].

Two federal guidelines, GSA 2003[4] and Department of Defence (DoD) 2005 and the latest variant DoD 2009 [5] adopted this strategy and proposed procedures to assess the potential of progressive collapse of a structure following the notional removal of load-bearing elements, according to different “missing column” scenarios.

Recent studies [6, 7, 8] have shown that mid-rise RC framed buildings (12 – 13 storeys), designed for high seismic zones (zone 4 in U.S. as for Bucharest, Romania, where $a_g = 0.24g$) do not experience progressive collapse when are subjected to the “sudden removal” of an exterior column. In the same time, Bilow [7] underlined that a similar 12-storey RC framed structure designed for a moderate seismic zone (SDC-C, according to 2000 International Building Code [9]) as for a zone of low seismicity (SDC-A case, according to 2000 IBC) needs additional reinforcing for the beams in the lower four storeys (SDC-C), respectively in the storeys one to eleven (SDC – A), in order to prevent progressive collapse. In those beams, Demand Capacity Ratios (DCR)'s values for flexure are greater than the allowable value 2.0, and according to GSA 2003 "elements that have DCR values exceeding the above limit will not have additional capacity for effectively redistributing the loads and are considered to be severely damaged and collapsed" [4].

In addition, GSA 2003 requires that for a member or connection whose DCR exceed the allowable flexure values (2.0 in our case), a hinge should be placed at the end of the member to release the moment (step 3 and 4 in the linear static procedure) and the analysis should be re-ran until no DCR values are exceeded. Even though the linear static step-by-step analysis procedure is theoretically simple and can be conducted without sophisticated nonlinear modelling, a lot of manual work and a lot of time is required to evaluate DCR magnitude and distribution in each analysis step, and to remodel/reanalyse the structure until DCR of any member does not exceed the given limit value [10]. Results after multiple iterations are sometimes contradictory and conclusions regarding the potential for progressive collapse (high or low) cannot be very clearly stated.

GSA Guidelines recommend the analysis of the damaged structures in four different cases, considering that a long side column, a short side column, a corner column and an interior column is successively eliminated. Related to these different damage scenarios Kim [10] reported that "the potential for progressive collapse was the highest when a corner column was suddenly removed".

Some standards such as ODPM 2005 [11] or ACI 318-05 [12], contain provisions for prevention of disproportionate collapse differentiated function of the number of storeys. Offices not exceeding four storeys are classified as Class 2A while offices exceeding four storeys are classified as Class 2B. The provisions are different for low-rise or Class A buildings, and become a little more complicated when mid to high-rise structures are considered [10]. Foley, is his report [13], shows that "the four storey division in classification appears arbitrary. One could argue that when an eight-storey building is considered, there is a significant opportunity for the storeys above the affected area to span across or bridge compromised columns. As the height decreases, this ability is limited. Aside from the increased occupancy in the taller structure, there may be a greater danger for the low-rise building to suffer disproportionate collapse". For steel structures,similar conclusions are given by Kim [10] who pointed out that "The progressive collapse potential decreased as the number of storeys increased" or by experimental studies which "suggest large damping effects in the system" for a 10-storey RC structure [14].

The main objective of the study is to assess the potential for progressive collapse of low-rise RC framed structures (3 storeys and 6 storeys), compared to mid-rise structures. The structures are designed, with the same configuration, first for a low seismic zone (Cluj-Napoca, $a_g = 0.08g$) where the beneficial influence of the seismic design is reduced, and then for a high seismic zone (Bucharest, $a_g = 0.24g$), in order to quantify the positive impacts of seismic resistance on progressive collapse risk of low-rise buildings. The linear static step-by-step analysis procedure recommended by GSA 2003 is performed for each analysis scenario. The distribution and magnitude of inelastic demands, the effect and the evolution of plastic hinges as well as the influence of the number of storeys were also investigated.

2 Structural models

The 3D models represent reinforced concrete (RC) framed structures located in Cluj-Napoca, Romania, which is considered to be a low seismic zone, respectively in Bucharest, a high seismic zone. The structures consist of five 6.0 m bays in the longitudinal direction (y-y) and two 6.0 m bays in the transverse direction (x-x), and have a storey height of 2.75 m, except the first two floors where the storey height is 3.6 m. The thickness of the slab is 150 mm.

In order to study the influence of building height on the progressive collapse resistance, two models having 3 and 6 floors are considered for each seismic zone (Figure 2) and their behaviour is compared to that of similar located mid-rise buildings (two models of 13 storeys).

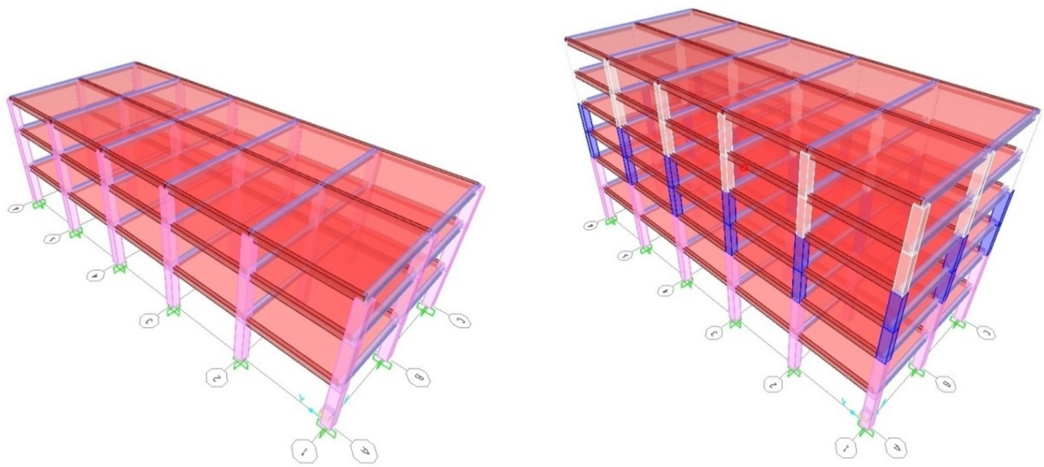

Figure 2: Geometry configuration of structures.

Design of structures is made according to the provisions of the still active Romanian Seismic Code P100-1/2006 [15], provisions that are similar to those specified by Eurocode 8 [16]. In the design process the following combinations of loads are considered:

- The Special Combination :

$$D + 0.4L + E \tag{1}$$

- The Fundamental Combinations :

$$1.35D + 1.5L + 1.05W \tag{2}$$

$$1.35D + 1.5W + 1.05L\,, \tag{3}$$

representing a combination of dead load D (D = self-weight plus a supplementary dead load of 2.0 kN/m^2), live load L = 2.4 kN/m^2, earthquake effect (E) and wind action W (for a wind speed of 30 m/s).

The seismic analysis is performed for a low seismic zone (Cluj-Napoca is practically the lowest seismic zone of Romania) where the design value of the peak ground acceleration is a_g = 0.08g, and for a high seismic zone (Bucharest - the capital) where a_g = 0.24g; the maximum value of a_g on the Romanian territory is 0.32g in Vrancea region [15].

Material	Seismic design	Progressive collapse analysis	
	Design values*	Characteristic un-factored values	With 1.25 factor
Concrete C25/30*	f_{cd} = 16.67	f_{ck} = 25	31.25
	f_{ctd} = 1.20	$f_{ctk0.05}$ = 1.80	2.25
Steel S500 **	f_{yd} = 435	f_{yk} = 500	625

* $f_{cd}(f_{ctd})$ = design compressive (tensile) strength of concrete, in N/mm^2
** f_{yd} = design yield strength of steel reinforcement, in N/mm^2

Table 1: Material properties

Parameter	Models			
	3 – Storeys Cluj-Napoca	6 – Storeys Cluj-Napoca	3 – Storeys Bucharest	6 – Storeys Bucharest
Gravity load in the seismic analysis	9673 kN	19636 kN	10160 kN	22372 kN
Equivalent static seismic force [15]: $F_b=\gamma_I S_d(T)\ m\ \lambda$	F_b = 0.0475G	F_b = 0.038G	F_b = 0.0997G	F_b = 0.0997G
Modal response spectrum analysis: • Periods: - T_1(y-y) - T_2(x-x) • Seismic base shear force [15]	T_1 = 0.512 s T_2 = 0.505 s F_{x-x}=0.0477G F_{y-y}=0.0476G	T_1 = 0.878 s T_2 = 0.868 s F_{x-x} = 0.0373G F_{y-y} = 0.0366G	T_1 = 0.445 s T_2 = 0.437 s F_{x-x} =0.0966G F_{y-y} =0.0964G	T_1 = 0.671 s T_2 = 0.669 s F_{x-x} =0.0971G F_{y-y} =0.0971G

Table 2: Undamaged models: data and results from the seismic analysis

The code [15] specifies that earthquake resistant structures shall be designed and detailed to provide energy dissipation through a ductile behaviour. For structures located in seismic zones with $a_g < 0.16g$ (our case), the standard [15] accepts that the seismic design may be done according to the provisions of ductility class M (medium ductility class), using for the behaviour factor q a value of 4.725. On the other hand, for the structures located in Bucharest - a high seismic zone, the seismic design should be made according to the provisions of ductility class H (high ductility class), using the behaviour factor q = 6.25. Also, according to these requirements, the compressive strength class of the concrete is C25/30, and the steel for the longitudinal and transverse reinforcement is of S500 type. The material properties are listed in Table 1.

The structure response is determined - via the modal analysis - with 3D linear elastic model using the computer software SAP 2000. Some significant results, related to the seismic behaviour of the models are given in the table presented in Table 2.

Internal forces and moments are determined with SAP 2000 and the design and detailing is made strictly following the provisions of the in use standards [15, 17] for concrete structures. Dimensions of structural elements (beams and columns) are presented in Table 3.

Structure	Storeys	Beam dimensions [m] Transverse x-x axis	Longitudinal y-y axis	Column dimensions [m]	Structure height [m]
3 - Storeys Cluj-Napoca	1-3	0.50 x 0.25	0.45 x 0.25	0.50 x 0.50	9.95
6 - Storeys Cluj-Napoca	1-2	0.50 x 0.25	0.45 x 0.25	0.60 x 0.60	14.6
	3-4	0.50 x 0.25	0.45 x 0.25	0.50 x 0.60	
	5-6	0.50 x 0.25	0.45 x 0.25	0.50 x 0.50	
3 - Storeys Bucharest	1-3	0.50 x 0.25	0.45 x 0.25	0.60 x 0.60	9.95
6 - Storeys Bucharest	1-2	0.60 x 0.30	0.55 x 0.30	0.70 x 0.70	14.6
	3-4	0.60 x 0.30	0.55 x 0.30		
	5-6	0.60 x 0.30	0.55 x 0.30		

Table 3: Dimension of cross sections for beams and columns

3 GSA (2003) static linear procedure

For buildings of 10 storeys or less in height with relatively simple layouts, GSA (2003) Guidelines [4] recommend the Alternate Path Method (APM) - based on the linear static analysis - to assess the vulnerability of new and existing buildings to progressive collapse. Normally used for buildings of 10 storeys or less above grade, the method can be successfully applied to taller buildings [6, 7]. To determine the potential for progressive collapse of a typical RC configuration, designers should perform a linear elastic static analysis, following the step-by-step procedure (five steps) considering the instantaneous loss of one of the first floor column ("missing column scenarios"), as it is shown in Figure 3.

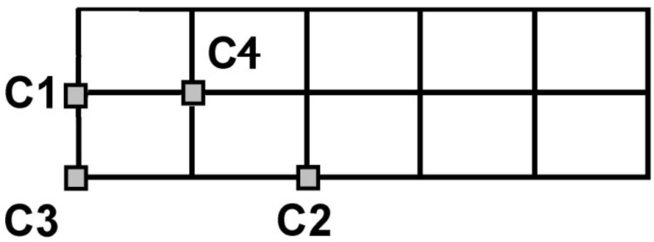

Figure 3: Missing column scenarios according to GSA (2003) Guidelines.

The sudden loss of a load-bearing element (column in this analysis), generates in the damaged model dynamic effects (moments, shear and axial forces, displacements, etc.); the event takes place in a very short time and structural members undergo nonlinear deformations before failure [18]. But, as in the routine seismic design, one simple approach is to use an equivalent linear elastic procedure, considering that increased vertical forces to be applied to the structure are [4]:

$$Load = 2(DL + 0.25LL) = 2Load^{STATIC} \tag{4}$$

where DL is the dead load and LL is the live load. By multiplying the static load combination by a factor of 2.0, the method takes into account - in a simplified manner - the dynamic amplification effect due to the instantaneously removal of a vertical support. With these increased gravity forces ($2Load^{STATIC}$), demands (Q_{UD}) in structural elements and connections are determined in terms of bending moments, shear forces, axial forces, etc. We have to underline that the results obtained by the linear elastic analysis are a measure of the magnitude of the inelastic demands in structural elements. The magnitude of these demands is indicated by **D**emand-**C**apacity **R**atios (DCR):

$$DCR = Q_{UD} / Q_{CE} \tag{5}$$

where Q_{CE} is the expected ultimate, un-factored capacity (bending moment, axial forces, shear forces) of each structural elements. In the assessment of Q_{CE}, strength increase factors are applied to the properties of construction materials to account for strain rate effect and material over-strength [6]. For RC framed structures, the strength increase factor is 1.25 (Table 1).

Using the DCR criteria for linear elastic approach, structural element that has DCR values exceeding the allowable value of 2.0 for typical RC configurations, are considered to be severely damaged or collapsed. In GSA Guidelines [4] five steps are prescribed to be followed. The provisions of Step 2 and Step 5 are not very clear stated, leading the structural engineer to adopt different approaches which may conduct to different final conclusions regarding the risk for progressive collapse.

For instance, Step 2 from [4] specifies: "Determine which members and connections have DCR values that exceed the acceptance criteria. If the DCR for any member end connection is exceeded based upon shear force, the member is considered a failed member”. This provision may be understood (for typical configurations where the allowable DCR value is 2.0) either as:

- “If DCR value for shear exceeds the acceptance criteria (the allowable DCR value of 2.0), the member is considered a failed member” and this interpretation is given in some papers [7, 10], or as:
- “if DCR value for shear exceeds 1.0, the member is considered a failed member”, and this approach is adopted by [11, 18, 19].

In addition, Step 2 [4] specifies: “If the flexural DCR values for both ends of a member or its connections, as well as the span itself, are exceeded (creating a three hinged failure mechanism), the member is considered a failed member”. This provision may be understood as:

- “If the flexural DCR values for both ends of a member, as well as the span itself, exceed the acceptance criteria (DCR $\leq$ 2), creating a three hinges failure mechanism, the member is considered a failed member” and this interpretation is adopted by J.Kim [10], Bilow [7], Baldridge [6], H. Kim [20],
- either as: “if DCR is greater than one (but less than the value given in Table 2 [4], i.e. DCR = 2.0) at both ends of adjacent beams within a structural system, this indicates collapse even though the DCR are less than the GSA limits”, concept underlined by Foley in his report [13].

Regarding the Step 5 [4]:
“Re-run the analysis and repeat Steps 1 through 4; Continue this process until no DCR values are exceeded. If moments have been re-distributed throughout the entire building and DCR values are still exceeded in areas outside of the allowable collapse region, the structure will be considered to have a high potential for progressive collapse”, the problem is:

- if the analysis should be repeated from Step 1 to Step 4, until no DCR values are exceeded (the allowable limit), how DCR values could still be exceeded in areas outside of the allowable collapse region?

4 Progressive collapse risk assessment

The progressive collapse analysis is focused on low-rise buildings (up to 6 storeys), due to their presumable low capacity to resist progressive collapse as it was presented in Section 1 of the paper.

The analysis has been performed for all those four cases (C1, C2, C3 and C4) indicated in Figure 3. Results related to the 3 and 6-storey structure, both located in Cluj-Napoca, when the corner column is removed (C3 case), have been presented in a previous work [21]. The present paper, mainly discusses results related to C1 and C2 cases.

The authors will consider in this study that DCR greater than one for shear leads to failure of the member. They also consider that the possibility of creating a three hinged mechanism exists if DCR’s for flexure are greater than one (DCR $\geq$ 1), assumptions discussed in Section 3.

4.1 3-Storey models

4.1.1 Cluj-Napoca structure

Being in a low seismic zone ($a_g = 0.08g$) and designed according to the provisions of medium ductility class (DCM), this structure do not poses, as the structures designed for high seismic zones do, that "inherent capacity to better resist progressive collapse" [8], and this "handicap" will be investigated.

4.1.1.1 C1 case

Iteration 1 - Step 1: The short side column (C1 case in Figure 3) was removed and the DCR's values for flexure, after the first step, are shown in Figure 4. In the transverse direction (x-x), DCR values are between 2.06 and 2.96. The number near the filled circle represents the DCR value in that section.

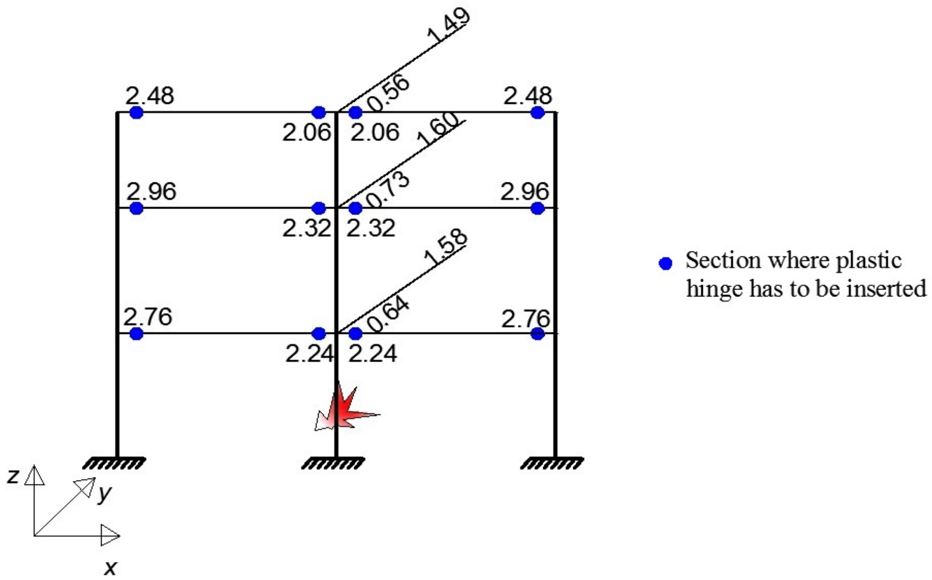

Figure 4: Structure in low seismic zone. Iteration 1. DCR values for flexure: C1 case.

Iteration 1 - Step 2: Large inelastic demands (DCR > 2.0) appear in all beam ends of the transverse (x-x) exterior frame (Figure 4). The longitudinal (y-y) frame behaves partially elastically (DCR < 1.0) but moderate inelastic demands (1.0 < DCR < 1.60) were also identified in several beam ends (Figure 4). Due to the connection with the longitudinal (y-y) frame which behaves elastically, the 3D model will stand and three hinged failure mechanisms will not occur; the analysis is continued following Step 3.
Iteration 1 - Step 3: In the member sections were DCR values for flexure exceed the allowable value of 2.0, plastic hinges are inserted (Figure 4 - filled circles).
Iteration 1- Step 4: Bending moments equal to the expected flexural strength of the section are applied at each inserted hinge.

Iteration 2: The analysis is re-run (Step 1 through 4 is repeated) and new DCR values are calculated. The DCR values for flexure after the moment redistribution are presented in Figure 5. A significant decrease of the inelastic demands in the transverse (x-x) exterior frame is observed (2.06 to 2.96/ Iteration 1 vs. 1.01 to 1.32/ Iteration 2). Meanwhile, there is an important increase of DCR values for flexure at the interior longitudinal (y-y) frame which no longer behaves elastically (DCR = 1.49 - 2.04). Because DCR values are greater than 1.0 at both ends of adjacent beams, a three hinge mechanism may occur. Consequently, in this case, there is a high risk for progressive collapse, and according to GSA (2003) Guidelines linear static procedure, other steps or iterations are no longer needed.

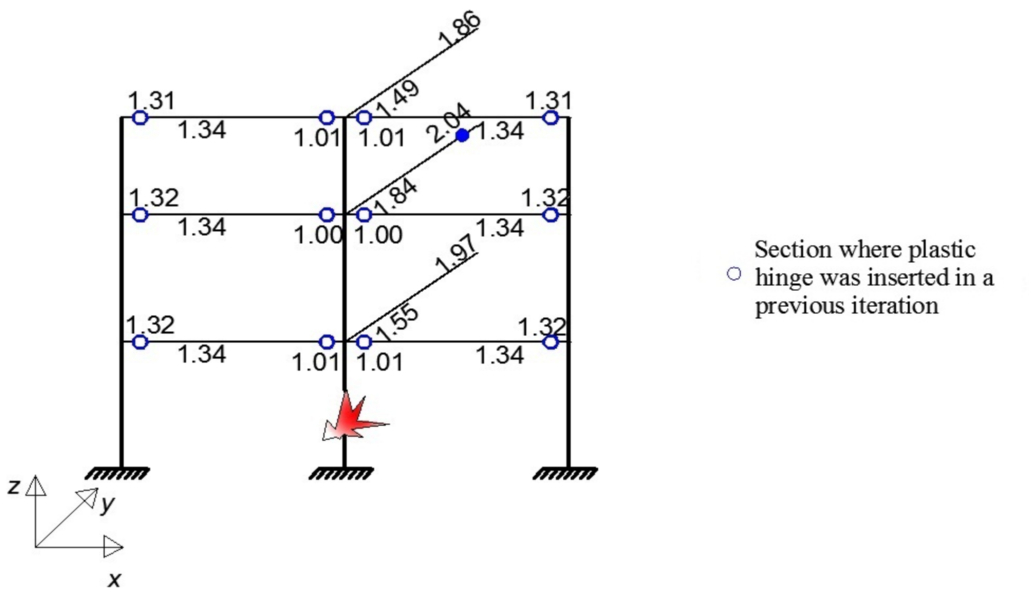

Figure 5: Structure in low seismic zone. Iteration 2. DCR values for flexure: C1 case.

4.1.1.2 C2 case

Similar to the previous case, two iterations were needed in the C2 damage case (removal of a long side column). In the first iteration (Figure 6a), high inelastic demands (DCR > 2.0) were identified in the beam sections near the columns, for the exterior longitudinal (y-y) frame. The transverse (x-x) frame beams have sections that develop moderate inelastic demands but there are also beam-end sections that behave elastically (DCR = 0.82 - 0.90), so the risk for the occurrence of a3D-faliure mechanism does not yet exist; a second iteration should be performed.

The results of the second iteration (Figure 6b) show that the plastic deformations increase in the transverse (x-x) frame (from 0.82 - 1.08 to 1.21 - 1.58). There are no longer beam sections that behave elastically. The inelastic demands decreased for the exterior longitudinal (y-y) frame, but all end sections are still in the range of moderate inelastic deformations. Because the end sections for all adjacent beams above the removed column have flexure DCR values greater than 1.0, a 3D failure mechanism could occur, so the risk for progressive collapse is high.

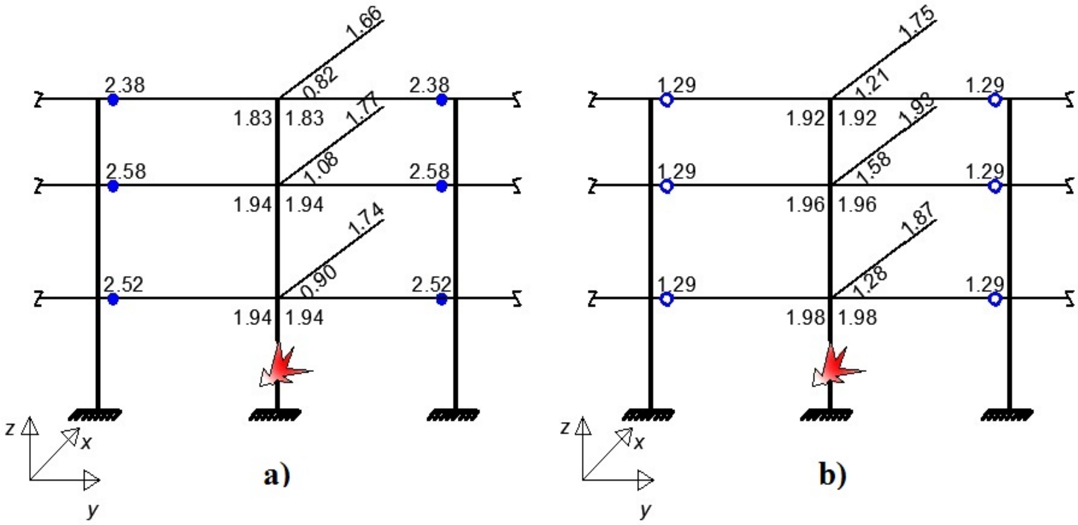

Figure 6: Structure in low seismic zone. DCR values for flexure: C2 case.
a) Iteration 1. b) Iteration 2.

4.1.2 Bucharest structure

The model is located in Bucharest which is a high seismic zone ($a_g = 0.24g$) and it is designed for a high ductility class (DCH). Similar to the Cluj-Napoca model, the analysis is made for the following two "missing column" cases.

4.1.2.1 C1 case

Iteration 1: The C1 column was removed and the DCR's values for flexure are displayed in Figure 7a. In the transverse direction (x-x), DCR values are between 2.05 and 2.26. Because the allowable value (2.0) for the DCR factor is exceeded but no 3D mechanism has occurred, plastic hinges are introduced (Step 3), and Step 4 and 5 are performed [4]. No plastic hinges should be introduced in the longitudinal frame (y-y) beams, where DCR values are in the range of 0.75 to 1.50 (Figure 7a).
Iteration 2: As shown in Figure 7b, after the redistribution of bending moments, as an effect of the inserted plastic hinges, DCR values are in the range of 1.00 to 1.32 for the exterior transverse frame (x-x), and in the range of 1.65 to 2.17 for the longitudinal frame (y-y). Due to this increase of the DCR values in the longitudinal direction, the structure behaves fully inelastically. As in the previous case, there is a high risk of progressive collapse, due to the possibility of creating a space (3D) failure mechanism of the three hinged mechanism type (Figure 7b).

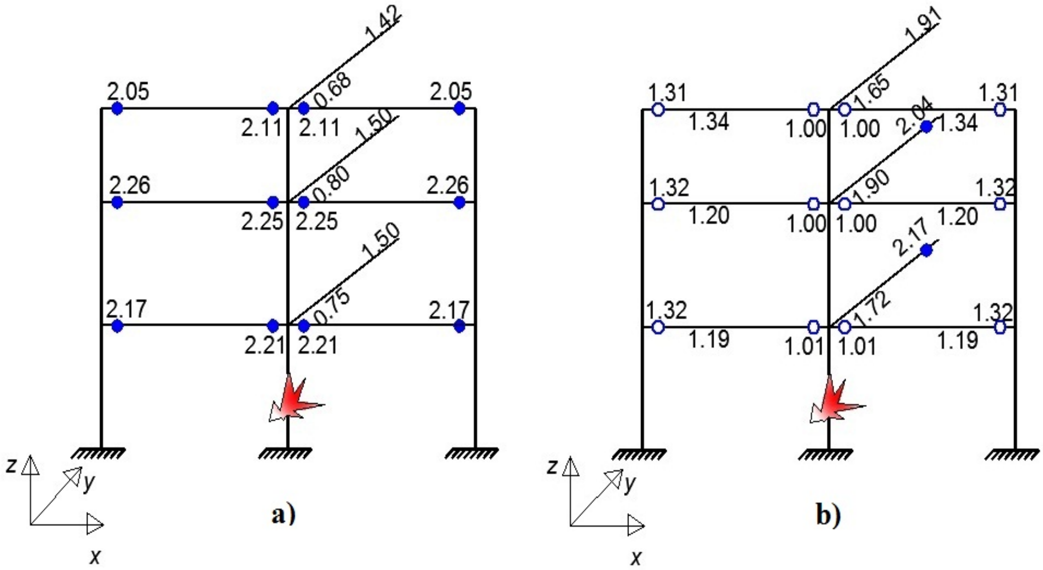

Figure 7: Structure in high seismic zone. DCR values for flexure: C1 case.
a) Iteration 1. b) Iteration 2.

4.1.2.2 C2 case

In this case, the instantaneous loss of one column from the longitudinal direction was considered. Relatively large inelastic demand (DCR = 2.03) for flexure exists only in one section (Figure 8). Being at the limit of acceptance criteria (DCR ≤ 2.0), the analysis does not require further steps or other iterations. Anyway, the presence of DCR values greater than one at both ends of beams, leads to a generalized space (3D) failure mechanism (three hinged mechanism type), if Foley's assumption [13] is accepted, and the progressive collapse potential is high.

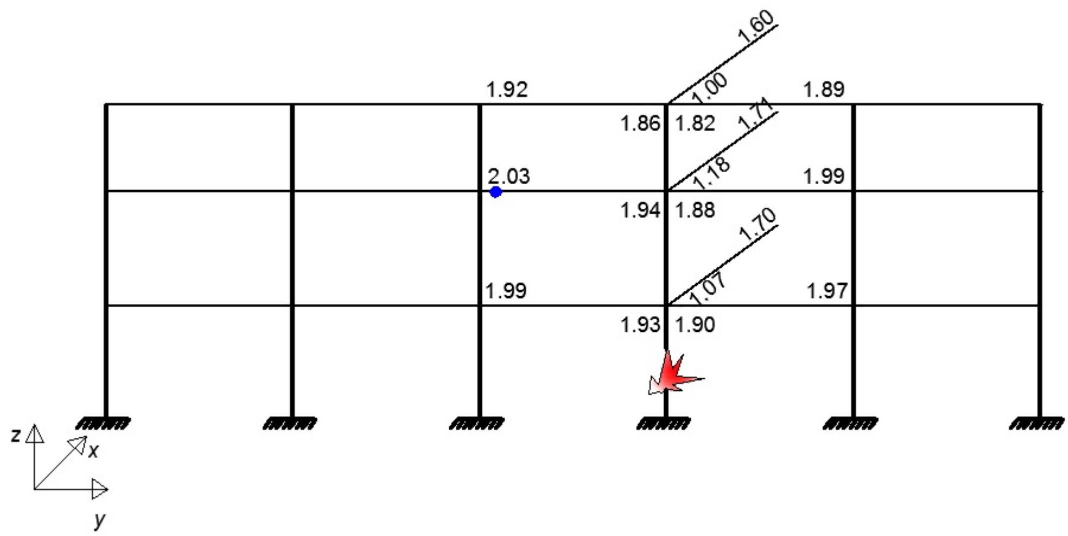

Figure 8: Structure in high seismic zone. DCR values for flexure: C2 case.

4.2 6-Storey models

4.2.1 Cluj-Napoca structure

4.2.1.1 C1 case

Two iterations were needed in order to achieve a conclusion regarding the progressive collapse risk of the 6-storey structure, located in Cluj-Napoca, for the C1 analysis scenario. The DCR values for flexure, corresponding to both iterations, are presented in Figure 9. Once again, it can be observed that in the second iteration, the structure gradually spreads the plastic deformations to its top levels and also in the longitudinal (y-y) direction. As the DCR values indicate (after the second iteration, the values are between 1.00 and 2.52), there is a possibility of occurrence of a 3D failure mechanism. Consequently, a third iteration is not required and the risk for progressive collapse is high, even though the inelastic demands in first five floors beams are small (DCR = 1.00 - 1.30).

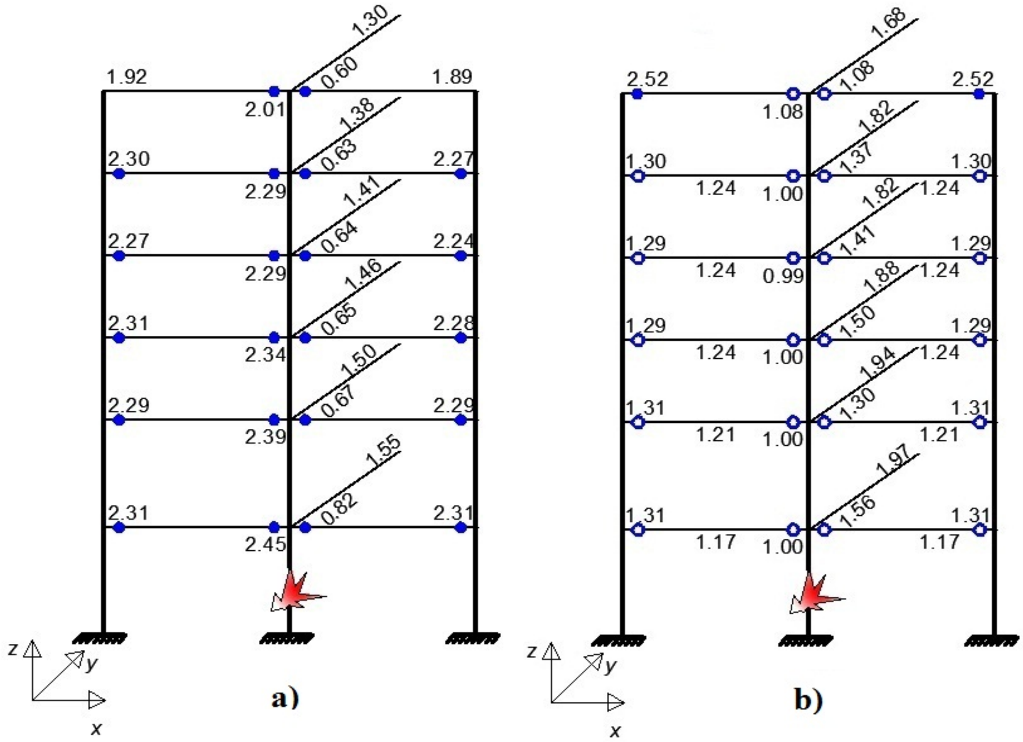

Figure 9: Structure in low seismic zone. DCR values for flexure: C1 case.
a) Iteration 1. b) Iteration 2.

4.2.1.2 C2 case

Figure 10 presents the DCR values for flexure corresponding to the needed iterations (two) to obtain a final conclusion regarding the progressive collapse risk. The behaviour of the structure is similar to that described in the previous analysis scenario - C1 (Section 4.2.1.1). Therefore, the progressive collapse risk is rated as being high, due to the possibility of appearance of generalized 3D failure mechanisms.

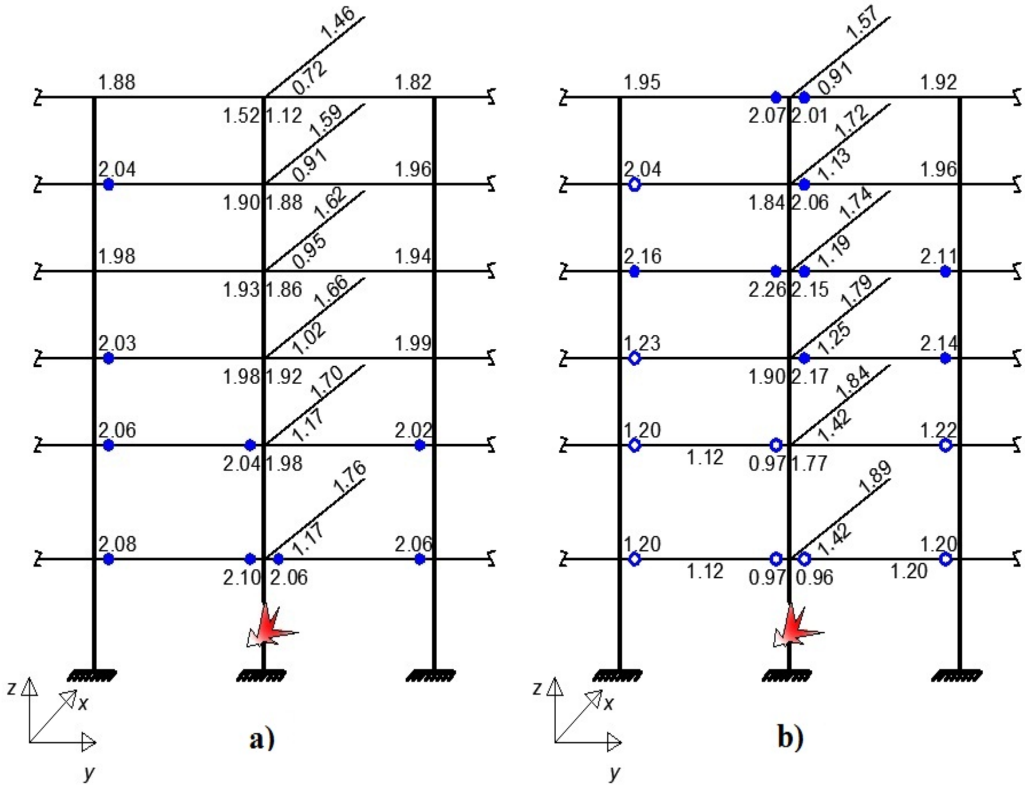

Figure 10: Structure in low seismic zone. DCR values for flexure: C2 case.
a) Iteration 1. b) Iteration 2.

4.2.2 Bucharest structure

4.2.2.1 C1 and C2 cases

DCR's values for flexure, when a short side column (C1 case) is removed, are presented in Figure 11. Because the allowable DCR limit of 2.0 was not exceeded, a single iteration should be performed.

Due to the DCR values that are in the range of 0.99 to 1.22 for the transverse exterior frame (x-x) and the model is at the limit of failure through a local in-plane mechanism. The model does not fail through a spatial generalized mechanism because the DCR values for the longitudinal direction (y-y) are between 0.51 and 0.98, therefore the longitudinal frame behaves elastically. According to GSA (2003) acceptance criteria, the potential for progressive collapse in this case is low.

When an exterior long side column is removed (C2 case - Figure 12), DCR values for the exterior longitudinal frame are between 1.09 and 1.52.

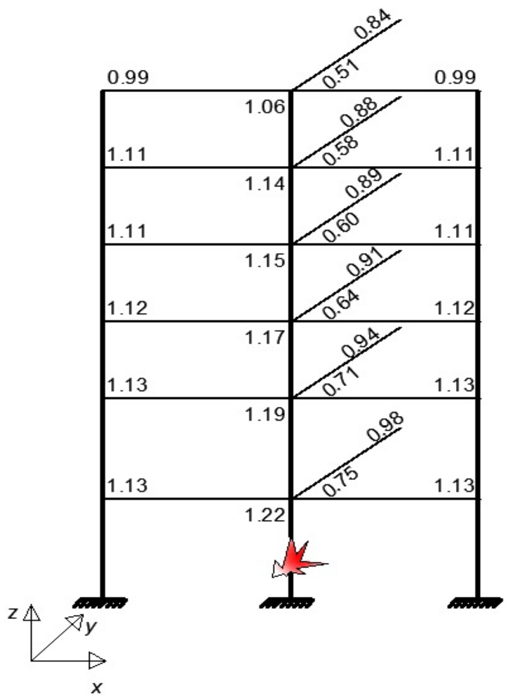

Figure 11: Structure in high seismic zone. DCR values for flexure: C1 case.

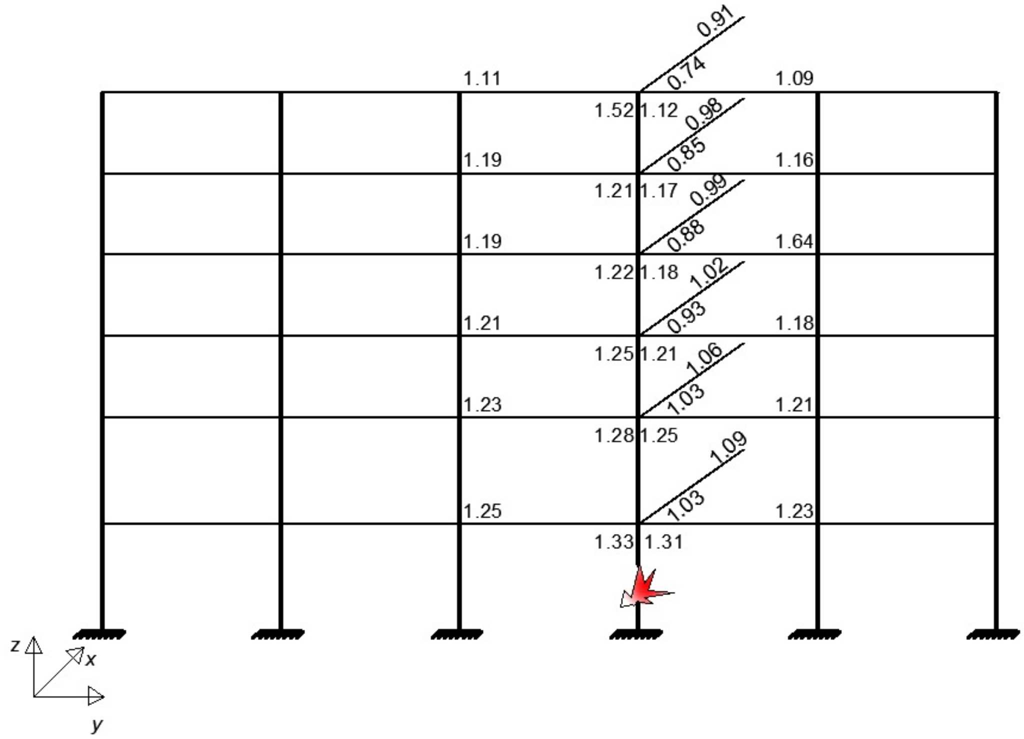

Figure 12: Structure in high seismic zone. DCR values for flexure: C2 case.

Even though a local in-plane mechanism could exist, the structure has a low potential for progressive collapse due to the fact that in the transverse (x-x) direction the beams behave elastically at the upper storeys, supporting the whole structure as a cantilever bridge, so a spatial generalized 3D mechanism does not occur.

4.3 Synthesis of results

The results and conclusions discussed in the previous sections, on 3 and 6 storey structures are added up by findings based on authors analyses made on the other two damage cases (C3 and C4) and also, on mid-rise structures (13 storeys) [8, 21, 23] .

A summary of the main results concerning the behaviour of low and mid-rise reinforced concrete framed buildings located in low and high seismic zones, and listed in Table 4 and 5; commentaries are made in Section 5.

Ln.		**C_1**	**C_2**	**C_3**	**C_4**
	3-storey structure				
1	Low seismic zone (a_g=0.08g)	2 iterations DCR_{max}=2.96	2 iterations DCR_{max}=2.58	1 iteration DCR_{max}=2.45	1 iteration DCR_{max} = 2.24
		High risk of progressive collapse			
2	High seismic zone (a_g=0.24g)	2 iterations DCR_{max}=2.26	1 iteration DCR_{max}=2.03	1 iteration DCR_{max}=1.96	1 iteration DCR_{max} = 2.14
		High risk of progressive collapse			
	6-storey structure				
3	Low seismic zone (a_g=0.08g)	2 iterations DCR_{max}=2.45	2 iterations DCR_{max}=2.08	1 iteration DCR_{max}=2.02	1 iteration DCR_{max} = 2.20
		High risk of progressive collapse			
4	High seismic zone (a_g=0.24g)	1 iteration DCR_{max}=1.22	1 iteration DCR_{max}=1.33	1 iteration DCR_{max}=1.19	1 iteration DCR_{max}=1.54
		Low risk of progressive collapse			
	13-storey structure				
5	Low seismic zone (a_g=0.08g)	1 iteration DCR_{max}=1.50	3 iterations DCR_{max}=2.35	6 iteration DCR_{max}=2.11	1 iteration DCR_{max} = 1.82
		Low risk of progressive collapse			
6	High seismic zone (a_g=0.24g)	1 iteration DCR_{max}=0.87	1 iteration DCR_{max}=0.91	1 iteration DCR_{max}=0.83	1 iteration DCR_{max} = 1.02
		No risk of progressive collapse			

Table 4: Main results and conclusion in the assessment of progressive collapse potential for low and mid-rise reinforced concrete structures

	3-storey structure	6-storey structure	13-storey structure
Low seismic zone	$DCR_{\max}^{av} = 2.55$	$DCR_{\max}^{av} = 2.19$	$DCR_{\max}^{av} = 1.94$
	High risk of PC	High risk of PC	Low risk of PC
High seismic zone	$DCR_{\max}^{av} = 2.10$	$DCR_{\max}^{av} = 1.32$	$DCR_{\max}^{av} = 0.94$
	High risk of PC	Low risk of PC	No risk of PC
Reduction of $DCR_{\max}^{av}$ values	**17.6%**	**39.7%**	**51.5%**

* $DCR_{\max}^{av}$ = average value of $DCR_{\max}$ recorded in the four damage scenarios

Table 5: Seismicity impact on progressive collapse risk

5 Conclusions

This paper presents the results of a parametric study of the potential for progressive collapse of low-rise reinforced concrete structures using the linear static procedure and the GSA (2003) acceptance criteria. Four models of 3 and 6 storeys have been designed and detailed according to the active design codes [15, 17], and then subjected to different cases of the so called "missing column" scenarios.

The program had four main objectives and the corresponding findings can be summarized as follows.

1. The effect of prescribed damage scenarios on progressive collapse potential:
   - Analysing the results furnished by 24 damage cases (Table 4), a unique conclusion cannot be drawn;
   - Removal of a column located near the middle of the short side of the building (C1 case) generates the highest DCR values and a high risk of progressive collapse for 3-storey structures, and for 6-storey structures located in low seismic zones (line 1, 2 and 3 – Table 4);
   - Removal of a column located near the middle of the long side (C2 case) leads to the highest inelastic demands for the 13-storey structures located in low seismic zones (line 5 – Table 4);
   - Removal of an interior column (C4 case) leads to the highest inelastic demands (DCR values) for structures of 6 and 13 storeys located in high seismic zones (line 4 and 6 – Table 4); this an important observation because it amends the general existing opinion which consider the elimination of an interior column as the less dangerous threat for the building safety, and consequently this case is rarely investigated;
   - Surprisingly, the C3 damage scenario (corner column removal) does not generate maximum DCR values for any of the models (case C3 – Table 4) and, in our opinion, it cannot be considered as the most dangerous damage case for reinforced concrete framed structures; this new aspect revealed by the study should be further investigated because for steel moment frames, Kim [10] drawn an opposite conclusion;

- The magnitude of maximum inelastic demands (maximum DCR values) for each model slightly differs when different damage cases are applied; differences are in the range of 14% to 25% (Table 4) and consequently, the final decision regarding the level of risk for progressive collapse (no risk, low risk or high risk) was the same, and it was not determined by a particular damage scenarios (C1 to C4) used in the analysis (Table 4).

This finding is new, original and highly valuable, because on its base, the final conclusion regarding the progressive collapse potential of a building could be drawn by investigating only one damage scenario (failure case), instead of four, as GSA (2003) Guidelines specify.

2. The general behaviour to progressive collapse of low-rise reinforced concrete buildings (4 to 6 floors) versus mid-rise buildings (12-13 floors):
   - Results displayed in Table 4, show clearly that low-rise building compared to mid-rise buildings are more vulnerable to progressive collapse; thus, the 3 and 6-storey building located in low seismic zones presented a high risk of progressive collapse (line 1 and 3 – Table 4), meanwhile the 13-storey building had a low risk for progressive collapse (line 5 – Table 4);
   - A similar conclusion regards the low and mid-rise buildings located in high seismic zones (line 2, 4 and 6 – Table 4).

3. The floors number influence (3-storey versus 6-storey structures) in the assessment of low-rise buildings robustness:
   - If the structures are designed for low seismic zones ($a_g < 0.08g$), the number of storeys (three and six) does not significantly influence the type of robustness, and all the analysed models had a high risk for progressive collapse (line 1 and 3 – Table 4);
   - If low-rise structures are designed and detailed for high seismic zones ($a_g \geq 0.24g$), the behaviour to progressive collapse strongly depends on the number of floors; thus, structure of 3-storeys had a high potential (line 2 – Table 4) and in the same time, the 6-storey structure had a low potential for progressive collapse (line 4 – Table 4);
   - The beneficial influence of storeys number in decreasing the risk to progressive collapse could be generalized for mid-rise buildings too, as it results from Table 5; this observation is confirmed by author's previous studies [8, 23] and by other researchers [10, 14, 19].

4. The influence of the area seismicity on the progressive collapse potential of low-rise buildings:
   - The general opinion of structural engineers that the seismic design and detailing has a beneficial effect on the progressive collapse resistance of reinforced concrete structures, is confirmed;
   - In the progressive collapse analysis, the magnitude of inelastic demands indicated by DCR values [22] decreases if the structure was designed for a high seismic area ($a_g \geq 0.24g$) with respect to a similar structure located in a low seismic zone (Table 5);

- Results presented in Table 5, indicate that the influence of seismicity is reduced for 3-storey models ($DCR_{\max}^{av}$ reduction of 17.6%), is significant for 6-storey models ($DCR_{\max}^{av}$ reduction of 39.7%) and is important for mid-rise buildings ($DCR_{\max}^{av}$ reduction of 51.5%); consequently, the risk for progressive collapse has not changed for 3-storey building (first column – Table 5), has changed for 6-storey building (from high to low risk) and has been eliminated for 13-storey building (from low to no risk).

It has to be underlined that for the first time in the technical literature, these results bring and specify in a quantitative manner, differentiated with respect to the number of storeys, the positive effect of seismic design and detailing on the progressive collapse resistance of reinforced concrete framed structures.

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