Vulnerability to Progressive Collapse of Seismically Designed Reinforced Concrete Framed Structures in Romania

A. Marchis, M. Botez & A.M. Ioani
Technical University of Cluj-Napoca, Faculty of Civil Engineering, Cluj-Napoca, Romania

SUMMARY:
The present study investigates the vulnerability to progressive collapse of mid-rise RC framed buildings. Four models are designed for three distinct seismic zones from Romania (high, moderate and low). The models located in moderate seismic areas are designed for two ductility classes (medium, high). Following the GSA (2003) Guidelines the potential for progressive collapse of structures is assessed using the linear static analysis procedure in the so called "missing column" scenarios. The results quantify the influence of seismicity on the progressive collapse resistance: strong for high seismic zones and reduced for moderate and low seismic zones. Also, it was found that applying the GSA (2003) linear static procedure, structures designed in the high ductility class seem to be more vulnerable to progressive collapse than similar structures designed for a medium ductility class. Related to this aspect, commentaries are made and an improved expression is proposed to calculate the DCR values.

Keywords: progressive collapse, RC framed building, GSA (2003) Guidelines, seismic zones

1. INTRODUCTION

Progressive collapse is a phenomenon that can be described as a chain reaction type of failure which may imply the collapse of the entire building or of a reduced part of it. This type of structural failure is the result of abnormal loading generated by blasts, vehicle impacts, fires, wind gusts or human errors. The design philosophy of structures subjected to abnormal loads is to prevent or to mitigate damage, not necessary to avoid the collapse initiation from a specific cause. This approach is similar to the concept adopted in modern earthquake-resistant design codes such as P100-1/2006, SR EN 1998-1/NA: 2008, FEMA 356, NZS 1170.5:2004. A similar strategy was adopted by two U.S guidelines, GSA (2003) and DoD (2005), that provide an independent methodology to assess the potential for progressive collapse of structures using the so called "missing column" scenarios, based on the notional removal of load-bearing elements. GSA (2003) recommends the analysis of damaged structures in four different cases, considering that a long side column, a short side column, a corner column and an interior column is eliminated in turn.

Whereas resistance to progressive collapse is primarily an issue of gravity load carrying capacity, the design of structural elements also depends on demands from other action such as wind (W) or seismic action (E); it means that, if beams, columns or joints have a larger load-bearing capacity due to more severe 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 (Sasani, 2008). In fact, in order 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 (Su, 2009).

Recent studies (Baldridge, 2003; Bilow, 2004; Ioani, 2007) demonstrated the beneficial effect of the seismic design and detailing on the progressive collapse resistance of reinforced concrete structures. It was shown that mid-rise reinforced concrete buildings (12-13 stories), designed for high seismic zones do not experience progressive collapse when subjected to “sudden column removal”, in contrast with low-rise structures (3-6 stories) which are more vulnerable (Bredean, 2012). In the same time, data from literature regarding the potential for progressive collapse of mid-rise structures located in low or moderate seismic zone are few (Baldridge, 2003; Bilow, 2004), and are missing if we refer to structures designed for the Romanian territory. Also, none of the previous investigations studied the consequences on progressive collapse behaviour of RC framed structures, located in moderate seismic zones, when different allowable options (medium or high ductility class) are considered in the seismic design. Therefore, such aspects are mainly investigated in the paper, and new information and conclusions are presented.

This paper presents a comparative investigation to assess the vulnerability to progressive collapse of four mid-rise buildings designed according to the provisions of the in use Romanian seismic design code P100-1/2006. The first model is designed for a low seismic zone (Cluj-Napoca, a_g = 0.08g), two models are designed for a moderate seismic zone (Sibiu, a_g = 0.16g) according to the provisions of the medium ductility class (class M) and high ductility class (class H), and the fourth one is designed for a high seismic zone (Bucharest, a_g = 0.24g). For each model, four damage scenarios (C_1 to C_4) are investigated using the linear static step-by-step analysis procedure recommended by GSA (2003) Guidelines. The distribution and magnitude of inelastic demands is determined and the progressive collapse potential of the models is evaluated.

2. DESIGN OF STRUCTURES

The 13-story buildings have the same 3D configuration, as illustrated in Fig. 2.1. 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 story height of 2.75 m, except the first two floors where the story height is 3.6 m. The thickness of the slab is 150 mm.

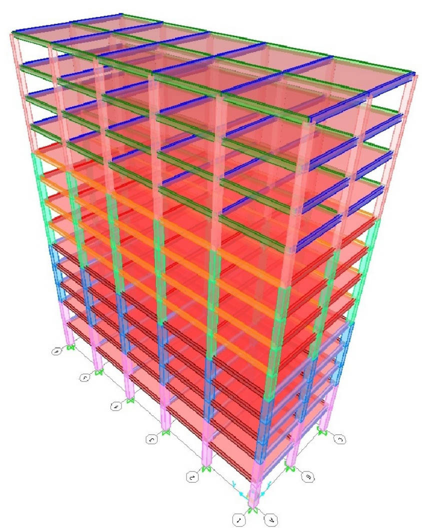

Figure 2.1. Geometry of structures

The design of structure is made according to the provisions of the active Romanian seismic code P100-1/2006, provisions that are similar to those specified by SR EN 1998-1/NA: 2008. The Special Combination and the Fundamental Combinations of loads, taken into account in design is D+0.4L+E, respectively 1.35D+1.5L+1.05W or 1.35D+1.5W=1.05U, where D is the dead load (self-weight plus a supplementary dead load of 2.0 kN/m^2), L is live load (2.4 kN/m^2), E is the earthquake effect and W is the wind action (for a wind speed of 30m/s).

The seismic design is made for each structure in association with the seismic zone where the building is located (Table 2.1). Hence, for the Cluj-Napoca structure, where the design value of the peak ground acceleration is a_g = 0.08g (low seismic zone), the Romanian code (P100-1/2006) as well as Eurocode 8 (SR EN 1998-1/NA: 2008) specify that currently, the design should be made according to the provisions of medium ductility class (M), using the behaviour factor q = 4.725. For structures located in Sibiu, the design value of the peak ground acceleration is a_g = 0.16g (moderate seismic zone), and both seismic codes accept that design may be done according to the medium (M) or high (H) ductility class. In order to observe the differences in behaviour, the models located in a moderate seismic zone are designed corresponding to each ductility class provisions (M, respectively H). The following structure is located in Bucharest, a high seismic zone where the peak ground acceleration is a_g = 0.24g; the codes specifies that the design shall be done according to the provisions of ductility class H (high ductility class), using the behaviour factor q = 6.25. Based on the same code (P100/1-2006) provisions, the compressive strength class of the concrete is C25/30, and the steel for the longitudinal and transverse reinforcement is S500 type.

Table 2.1. Seismic designed models

Structural models	Model 1 (Cluj)	Model 2	Model 3	Model 4 (Bucharest)
		(Sibiu)		
Seismicity of the zone (a_g)	Low (a_g=0.08g)	Moderate (a_g=0.16g)		High (a_g=0.24g)
Design ductility class	Medium (M)	Medium (M)	High (H)	High (H)

A modal analysis for each tri-dimensional model is performed using SAP 2000 computer program, and internal forces and moments are determined. The complete design is made following the provisions of the in use Romanian seismic code P100/1-2006 and the design code for concrete structures SR EN 1992-1-1:2004.

3. GSA 2003 GUIDELINES

Following the provisions of the GSA (2003) Guidelines, the potential for progressive collapse is assessed using the linear static procedure in four damage analysis cases ("missing column" scenarios), as illustrated in Figure 3.1: the loss of an exterior column located near the middle of the short side (case C_1), the loss of an exterior column located near the middle of the long side (case C_2), the loss of a corner column (case C_3) and the loss of an interior column (case C_4).

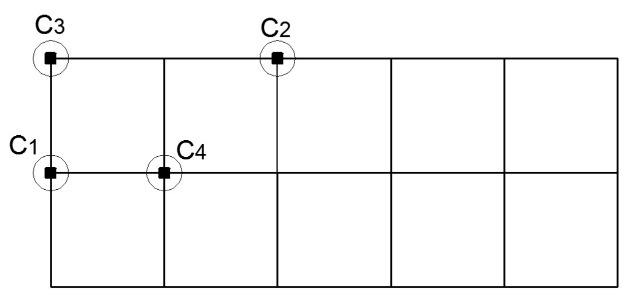

Figure 3.1. Damage cases according to the GSA (2003) Guidelines

For static analysis, the following vertical load is applied downward to the structure:

$$\text{Load} = 2(\text{DL} + 0.25\text{LL}) \tag{3.1}$$

where DL is the dead load and LL is the live load. By multiplying the load combination by a factor of two, the GSA 2003 Guidelines take into account - in a simplified approach - the dynamic effect that occurs when a vertical support is instantaneously removed from the structure; demands (Q_{UD}) in structural components are determined in terms of moments, axial forces, shear forces (Ioani, 2010).

Using the linear static analysis results (moment, shear, axial force), the magnitudes and distribution of potential inelastic demands on structural elements shall be indentify for quantifying the potential collapse areas. The magnitude and distribution of these demands are indicated by the DCR values (**D**emand-**C**apacity-**R**atios). For each structural component or connection, DCR values are determined as follows:

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{3.2}$$

where Q_{UD} is the acting force determined in member or connection (moment, axial force, shear or combined forces), using the linear static analysis. Q_{CE} is the expected ultimate un-factored capacity of the member or connection in terms of moment, axial force, shear or combined forces, where the characteristic material strength are incresead by a strength increase factor of 1.25, both for concrete and steel bars.

Using the DCR criteria of the linear elastic approach, structural elements and connections that have DCR values that exceed the allowable value of 2.0 (for typical structural configuration) are considered to be severely damaged or collapsed (GSA, 2003). The authors underline once again (Bredean, 2012) that the step-by-step procedure from the GSA (2003) Guidelines is not particularly explicit at Step 2 when refers to allowable DCR values for shear, and when statues the conditions for creating a three hinged failure mechanism. However, in this study a failed member is considered if the DCR value for shear is greater than one; also we will consider that "if DCR for flexure is greater than 1.0 at both ends of adjacent beams within a structural system, this indicates collapse even though the DCR are less than the GSA limits (2.0)" (Foley, 2007). In certain situation this approach is not validated by the more sophisticated nonlinear static or dynamic analyses performed by the authors and by other researchers (Kim, 2006; Tsai, 2008) and leads to very conservative conclusions. Further analyses are needed.

4. PROGRESSIVE COLLAPSE ANALYSIS

Following the GSA (2003) Guidelines, demands (Q_{UD}) in structural components (beams) are compared to their expected un-factored capacities (Q_{CE}), as shown in Eqn. 3.2. The DCR values for flexure and for shear are determined at column faces. The progressive collapse analysis is performed for all the four damage analysis cases (C_1, C_2, C_3 and C_4). Only the results of the case C_4 (interior column removal) -a case rarely discussed in literature- are detailed in this paper; results from the other damage cases are summarized in Section 5, Table 5.1.

4.1. Model designed for low seismic zone

The first 3D model representing a 13-story RC framed structure is located in a low seismic zone (Cluj-Napoca, Romania), and is seismically designed, as the current code P100/1-2006 specifies, for the medium ductility class (M). The DCR values for flexure determined at beams ends range from 0.70 to 1.22 for the transverse frame, and from 0.62 to 1.82 for the longitudinal frame (Figure 4.1). Thus, the GSA acceptance criteria ($DCR \leq 2.0$) are fulfilled and no further iterations are required as the step-by-step procedure indicate. The inelastic demands of beam sections are indicated by DCR values higher than 1.0. An elastic behavior of the section is represented by DCR values lower than 1.0. No plane or 3D failure mechanism is expected to occur for the model. In the same time, all DCR values for shear are below the allowable value of 1.0 (DCR_{max} = 0.92) and therefore, none of the structural components are considered failed members. For this damage case, the structure meets the GSA acceptance criteria and has a low potential for progressive collapse. A similar conclusion is drawn when the analysis is performed for the other three damage cases (C_1, C_2 and C_3 - Table 5.1).

1st iteration

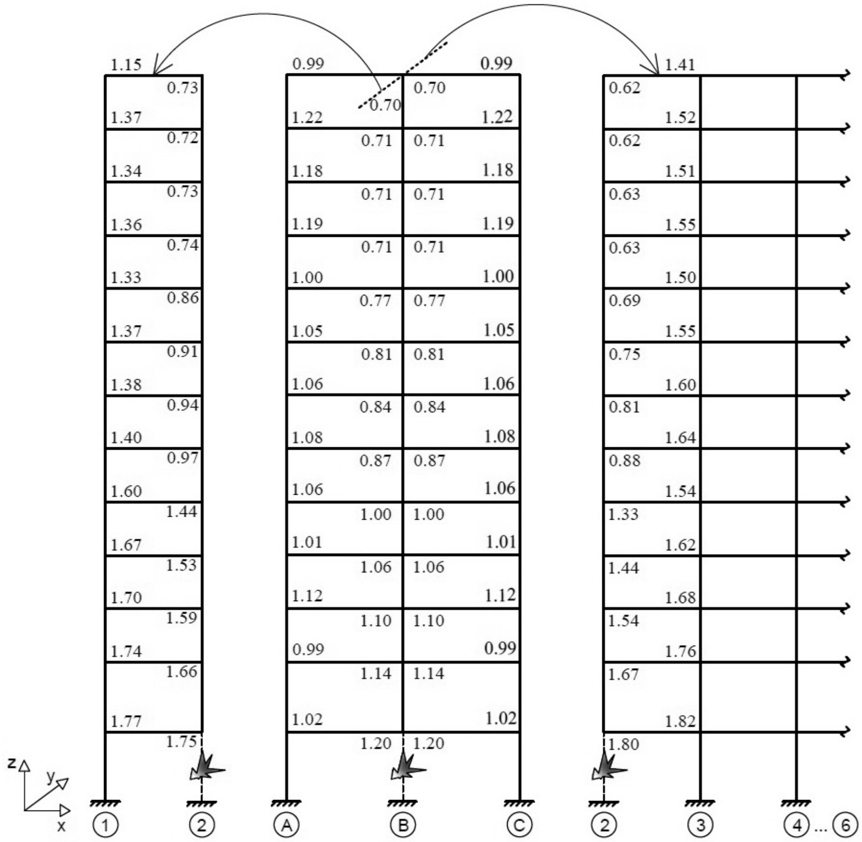

Figure 4.1. DCR values for flexure in the damage case C_4 - Cluj model designed for medium ductility class

4.2. Models designed for moderate seismic zone

The structure located in Sibiu, a zone with moderate seismic risk (a_g = 0.16g), is designed both for medium (M) and high (H) ductility class. The influence of the design option (M or H) on the vulnerability to progressive collapse is investigated.

4.2.1. Model designed for medium ductility class

The structural model designed for medium ductility class, option alternatively accepted by the seismic codes (P100-1/2006, SR EN 1998-1/NA: 2008), is subjected to the damage case C_4 (interior column removal). The step-by-step procedure for conducting the linear static analysis is:
Iteration 1 – Step 1: The interior column is removed. The model is loaded according to Eqn. 3.1;
Iteration 1 – Step 2: DCR values for flexure and for shear are determined. In the transverse direction (x-x), the DCR values range from 0.84 to 1.81 and in the longitudinal direction (y-y) from 0.68 to 2.25. Large inelastic demands (DCR > 2.0) appear at beams ends of the first two stories of the longitudinal frame. No plane or 3D failure mechanisms are expected to occur in the model;
Iteration 1 – Step 3: In the beam sections were DCR values for flexure exceed the allowable value of 2.0, plastic hinges are inserted;
Iteration 1 – Step 4: Bending moments equal to the expected flexural strength of the sections are applied at each inserted hinge;
Iteration 2: The analysis is re-run from Step 1 through 4 and new DCR values are calculated. After the moment redistribution, there is only one beam section from the transverse frame with DCR values higher than 2.0. At that section, a new a plastic hinge should be inserted;
Iteration 3: The analysis is re-run, and the final DCR values are displayed in Figure 4.2.

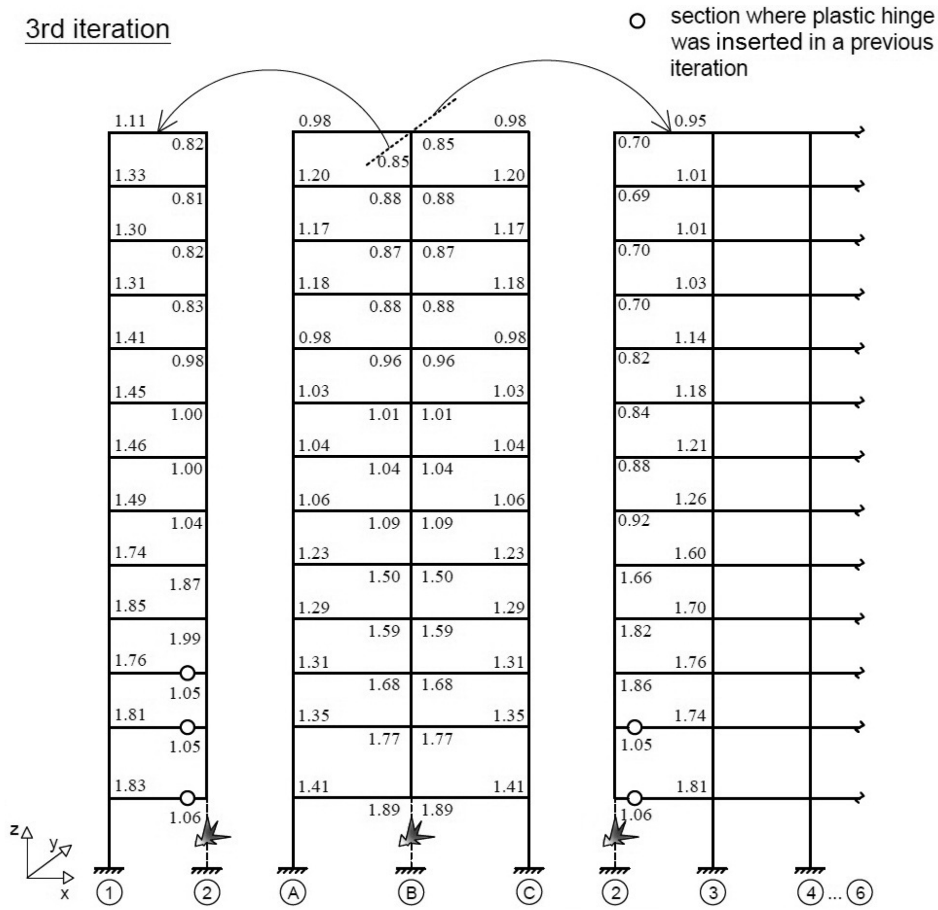

Figure 4.2. DCR values for flexure in the damage case C_4 - Sibiu model designed for medium ductility class

After this iteration all DCR values for flexure are bellow 2.0 and thus the GSA acceptance criteria are fulfilled. Due to the elastic behaviour of the upper stories, the model will stand and no plane or 3D failure mechanism of three hinges type is expected to occur. In the same time, the maximum DCR value for shear is 0.78, well below the allowable value (1.0). Consequently, in this case, there is a low potential for progressive collapse. A similar conclusion is drawn when the analysis is performed for the other three damage cases (C_1, C_2 and C_3-Table 5.1).

4.2.2. Model designed for high ductility class

When the 13-story is designed for high ductility class, option recommended by the seismic codes, the results are similar with those obtained for the structure designed for medium ductility class (Table 5.1). The analysis stops after the forth iteration when all DCR values meet the GSA acceptance criteria (DCR $\leq$ 2.0). For the transverse frame, all beam end sections behave inelastic and thus a plane failure mechanism of three hinges type is expected to occur only for the transverse (x-x) frame. Due to the interconnection with the longitudinal (y-y) frame which behaves elastically (DCR < 1.0) at its upper part, the 3D model will stand and a spatial mechanism will not occur. Also, the DCR values for shear are below the allowable value (1.0) and consequently, the model has a low potential for progressive collapse (Table 5.1). Surprisingly, two damage cases (C_2 and C_3) lead, after four iterations, to a failure mechanism of the three hinged type, and consequently, the models would have a high progressive collapse potential (marked with * in Table 5.1). This aspect is new in literature and would indicate a probable higher sensibility to progressive collapse of high ductility (H) model with respect to models designed in the medium ductility class (M). The result can be explained: GSA acceptance criteria take into consideration an elastic behaviour of the structure when a load increase factor of 2.0 is applied to static vertical loads (Eqn. 3.1), and consider only the strength capacity of the section (not its increased ductility) in the evaluation of DCR values (Eqn.3.2). Consequently, a member designed for the high ductility class, having a smaller strength capacity (but an increased ductility), will generates higher DCR values compared to a member designed in the medium ductility class. The result is not validated by nonlinear analyses and cannot be accepted in practice as a final conclusion. In the author's opinion, to account for the inherent inelastic and ductile behaviour of RC frames designed for M and H ductility class, a different load increase factor (LIF) smaller than 2.0 would be more suitable; similar suggestion has been made in other works (Kim, 2006; Tsai, 2008), and the calibration of LIF is in progress in our works (see Section 5).

4.3. Model designed for high seismic zone

The last 3D model is designed for Bucharest, a zone with high seismic risk, where the design value of the peak ground acceleration is $a_g = 0.24g$. As the current codes P100/1-2006 and SR EN 1998-1/NA: 2008 specify, the buildings located in these areas can be designed only for high ductility class (H). To conduct the progressive collapse analysis, the interior column is removed and the DCR values for flexure determined (Figure 4.3). Due to the fact that all DCR values for flexure meet the GSA acceptance criteria (DCR $\leq$ 2.0), no further iterations are required.

The DCR values range from 0.60 to 0.94 for the transverse (x-x) frame and from 0.5 to 0.94 for the longitudinal (y-y) frame (Figure 4.3) and consequently, the structural members behaves elastically. In addition, the maximum DCR value for shear was 0.91, below the allowable value (1.0) specified by the GSA (2003) Guidelines. Therefore, there is no risk for progressive collapse when mid-rise buildings are designed for high seismic zones where $a_g \geq 0.24g$.

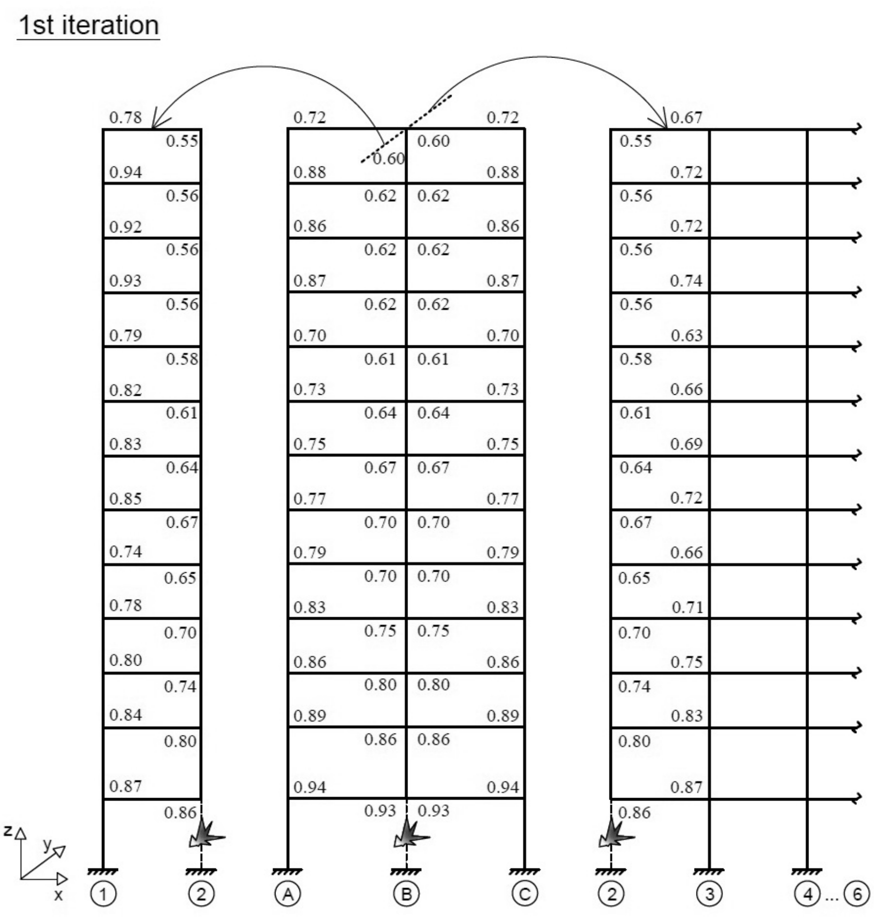

Figure 4.3. DCR values for flexure in the damage case C_4 - Bucharest model designed for high ductility class

5. SYNTHESIS OF RESULTS

Results from 16 damage cases are summarized in Table 5.1 and show the maximum magnitude of inelastic demands (DCR_{max}) recorded in the first iteration, number of needed iterations, magnitude of maximum inelastic demands in the last iteration, and the final conclusion regarding the potential for progressive collapse of each model, when four damage scenarios are considered. A suggestive graphic representation of the risk for progressive collapse of mid-rise RC framed structures located in different seismic zones (a_g = 0.08g, 016g and 0.24g) of the Romanian territory is displayed in Figure 5.1. This progressive collapse safety map provides new and useful information for structural engineers. The safety map will be completed with conclusions related to the behaviour of low-rise buildings (3-6 stories) (Bredean, 2012) and by further analyses made for other seismic zones (a_g = 0.12g, 0.20g, etc.).

Table 5.1. Summary of results and conclusions

Damage case		C_1	C_2	C_3	C_4
Cluj-Napoca structure (a_g=0.08g)					
1	Medium ductility class	DCR_{max}=1.50 1 iteration	DCR_{max}=2.35 3 iterations DCR^3_{max}=1.26	DCR_{max}=2.11 6 iterations DCR^6_{max}=1.95	DCR_{max}=1.82 1 iteration
		Low Risk for Progressive Collapse			
Sibiu structure (a_g=0.16g)					
2	Medium ductility class	DCR_{max}=1.73 1 iteration	DCR_{max}=2.31 4 iterations DCR^4_{max}=1.55	DCR_{max}=1.97 1 iteration	DCR_{max}=2.25 3 iteration DCR^3_{max}=1.89
		Low Risk for Progressive Collapse			
3	High ductility class	DCR_{max}=1.91 1 iteration	DCR_{max}=2.62 4 iterations DCR^4_{max}=2.15	DCR_{max}=2.49 4 iterations DCR^4_{max}=2.20	DCR_{max}=2.70 4 iterations DCR^4_{max}=1.92
		Low Risk	*High Risk* *	*High Risk* *	*Low Risk*
Bucharest structure (a_g=0.24g)					
4	High ductility class	DCR_{max}=0.86 1 iteration	DCR_{max}=0.90 1 iteration	DCR_{max}=0.84 1 iteration	DCR_{max}=0.94 1 iteration
		No Risk for Progressive Collapse			

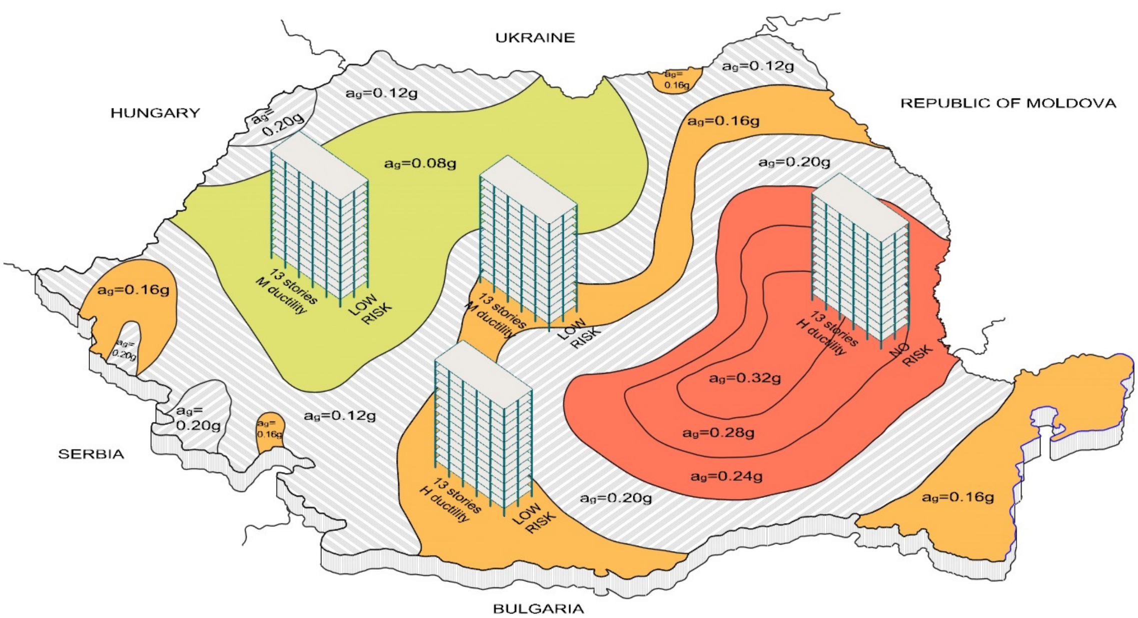

Figure 5.1. Progressive collapse safety map for mid-rise structures in Romania

6. CONCLUSIONS

The present study investigates the vulnerability to progressive collapse of mid-rise buildings located in different seismic areas from Romania using the provisions of the GSA (2003) Guidelines. Four 3D models representing the 13-story RC framed structure have been designed and detailed according to the active codes P100/1-2006 and SR EN 1992-1-1:2004 for three seismic zones (high, moderate and low); the building located in a moderate seismic area has been designed for two ductility classes (medium and high). The influence of the seismicity of the area as well as of the ductility class of the structure on the vulnerability to progressive collapse of mid-rise buildings is investigated and leads to the following conclusions:

1. The effect of the structural damage cases on the progressive collapse potential:
● Removal of a column located near the middle of the short side of the building (C_1 case) generates the smallest DCR values (DCR_{max} = 0.86 to 1.91);
● Removal of a column located near the middle of the long side (C_2 case) leads to the highest DCR values (DCR_{max} = 0.90 to 2.62);
● Surprisingly, the C_3 damage scenario (corner column removal) does not generate the maximum DCR values and cannot be considered as the most dangerous damage case; a similar conclusion was drawn for low-rise RC buildings (Bredean, 2012);
● For a given model, the magnitude of maximum demand-capacity-ratios (DCR_{max}) slightly varies when different damage cases are considered and consequently, the final conclusion regarding the risk level for progressive collapse is the same: low risk for structures located in low and moderate seismic zones, and no risk for structures designed for high seismic zones; 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 the GSA (2003) Guidelines specify;

2. The influence of the seismicity area on the vulnerability to progressive collapse of mid-rise buildings:
● The general opinion of structural engineers that the seismic design and detailing has a beneficial influence on the progressive collapse resistance of reinforced concrete structures, is confirmed;
● This influence is strong for structures designed for high seismic zones. The analyzed structures behave elastically when subjected to different damage scenarios and do not have a risk for progressive collapse; this is a consequence of the fact that the Special Combination of loads where the earthquake effect E intervenes (D + 0.4L + E), determine the magnitude of internal forces and moment used in design and detailing of structural members;
● The beneficial influence of seismicity is reduced for mid-rise structures located in low or moderate seismic zones. These structures have a similar potential for progressive collapse (low potential) and this is due to the fact that the Fundamental Combination of loads (where the earthquake effect E is not considered) determine the design and detailing of structural members; this finding is new in the literature and shows that the positive effect of seismic design exists, but it must be prudently considered for structures located in moderate seismic zones.

3. The influence of the ductility class (M or H) on the progressive collapse resistance:
● The linear static procedure from GSA (2003) Guidelines cannot take into account the positive effect brought by the seismic design when more ductile elements (not necessary more strength) are provided in the structure;
● For this reason, structures designed in the high ductility class seem to be more vulnerable to progressive collapse than the similar structures designed for a medium ductility class (Table 5.1), a conclusion not validated by nonlinear analyses or by practice; to correct this aspect, an improved expression to determine the magnitude of DCR will be proposed:

$$DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{Q_{UD}}{q_{pc}} \cdot \frac{1}{Q_{CE}} \qquad (6.1)$$

In the last equation, the authors have introduced a new parameter q_{pc}, representing the behaviour factor used in the progressive collapse analyses. This parameter estimates the beneficial influence of the ductility class of the structural members; authors' research in progress indicates for q_{pc}, values in the range of 1.4 to 1.6 for high ductility class elements, 1.15 to 1.25 for medium ductility class, and 1.0 for low ductility class elements.

ACKNOWLEDGEMENT

This paper was supported by the project "Improvement of the doctoral studies quality in engineering science for development of the knowledge based society-QDOC" contract no. POSDRU/107/1.5/S/78534, project co-funded by the European Social Fund through the Sectorial Operational Program Human Resources 2007-2013.

REFERENCES

P100-1/2006. (2006). Seismic design code – Part I: Design Rules for Buildings, MTCT, Bucharest, Romania.

SR EN 1998 -1/NA. (2008). Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings, ASRO, Bucharest, Romania.

FEMA 356. (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, USA.

NZS 1170.5 (2004). New Zealand Standard: Structural design actions – Part 5: Earthquake actions, Standards New Zealand, Wellington, New Zealand.

GSA - General Service Administration . (2003). Progressive Collapse Analysis and Design Guideline for New Federal Office Buildings and Major Modernisation Projects, Washington, USA.

DoD - Department of Defense. (2005). Design of Building to Resist Progressive Collapse. Unified Facility Criteria, UFC-4-023-03, Washington, USA.

Sasani, M. (2008). Progressive collapse analysis of an RC structure. *The Structural Design of Tall and Special Buildings,* **17:4**, 757-771.

Su, Y., Tian, Y., Song, X. (2009). Progressive Collapse Resistance of Axially-Restrained Frame Beams. *ACI Structural Journal,* **109:5**, 600-608.

Baldridge, S.M. and Humay, F.K. (2003). Preventing Progressive Collapse in Concrete Buildings. *Concrete International.* **25:11**, 73-79.

Bilow, N.D. and Kamara, M. (2004). U. S. General Services Administration Progressive Collapse Guidelines Applied to Moment – Resisting Frame Building. *2004 ASCE Structures Congress.*

Ioani, A.M., Cucu, H.L. and Mircea, C. (2007). Seismic Design vs. Progressive Collapse: A Reinforced Concrete Framed Structure Case Study. *Forth International Structural Engineering and Construction Conference*.

Bredean, L., Botez, M., Ioani, A.M. (2012). Progressive Collapse Risk and Robustness of Low-Rise Reinforced Concrete Buildings. *Eleventh International Conference on Computational Structures Technology*. (under printing).

SR EN 1992-1-1. (2004). Eurocode 2: Design of Concrete Structures - Part 1-1: General Rules and Rules for Buildings, ASRO, Bucharest, Romania.

Ioani, A. and Cucu, H. (2010). Comparative study of the potential to progressive collapse using the linear static analyses (GSA, DOD). *Journal of the Annals of the University of Oradea Magazine: Constructions and Hydroedilitary Installations*. **13,** 169-177.

Foley, C.M., Martin, K. and Schneeman, C. (2007). Robustness in Structural Steel Framing System. *Report Mu-Ceen-Se-07-01, American Institute of Steel Construction, Inc.* Chicago, Illinois, USA.

Kim, H. (2006). Progressive collapse behavior of reinforced concrete structures with deficient details, The University of Texas, Austin, Texas, USA.

Tsai, M.H. (2008). Investigation of progressive collapse resistance and inelastic response for an earthquake-resistant RC building subjected to column failure. *Journal of Engineering Structures*, **30**, 3619-3628.