Earthquake and Progressive Collapse Resistance based on the Evolution of Romanian Seismic Design Codes

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SUMMARY:
GSA (2003) Guidelines provides a detailed methodology to assess the potential to progressive collapse of existing buildings, based on a linear static analysis and "missing column" scenarios. In this paper, the progressive collapse potential of three distinct models representing a 13-storey RC framed structure located in an area with high seismic risk is assessed. The models are designed according to Romanian seismic codes in use in 1992, 2006 and 2008, and detailed considering the provisions of concrete structures design codes STAS 10107/0-90 (1990) and Eurocode 2 (2004). The comparative results show that a mid-rise structure designed for a zone with $a_g = 0.24g$ does not experience progressive collapse when subjected to abnormal loads. It might also be concluded that the last 20 years of changes in the Romanian design codes, implicitly lead to improvements in the resistance to progressive collapse of reinforced concrete framed buildings.

Keywords: progressive collapse, Romanian seismic codes, RC framed structures, GSA (2003) Guidelines, DCR

1. INTRODUCTION

Progressive collapse is defined as the spread of an initial local failure from element to element, through a chain reaction, which leads to partial or even full collapse of an entire structure. The abnormal loads, like explosions, vehicle collisions, human errors, represent the main causes that lead to progressive collapse of buildings.

The seismic design and detailing of a structure provides it with certain levels of continuity, ductility and redundancy, depending on the provisions for the seismic zone and for the ductility class. The mentioned characteristics are extremely important and have a significant influence on the progressive collapse behavior. A higher ductility improves the capacity of a structure to respond to a sudden removal of a vertical element with an inelastic behavior and without the failure of other structural elements.

The American Federal Guidelines GSA (2003), DOD (2005) and DOD (2009) propose different procedures to assess the potential of progressive collapse of a structure. The GSA (2003) Guidelines is based on the *Alternative Path Method* and consider the instantaneous loss of structural elements using different "missing column" or "missing beams" scenarios.

Using the GSA (2003) Guidelines, Baldridge and Humay (2003), Bilow and Kamara (2004), Botez, Bredean and Ioani (2012) assessed the progressive collapse potential of RC framed structures taking into account the influence of the following parameters: number of stories and seismicity of the area. In their works, Ioani and Cucu (2010) presented the effects on the progressive collapse resistance when seismic design is made according to two former Romanian codes P100-92 and P100-1/2006; only one damage case (corner column) was investigated. None of the previous investigations focuses on the effect of the active seismic design code SR EN 1998-1-1:2004/NA: 2008 (Eurocode 8), when all four damage cases are considered. How safe could be a reinforced concrete building, when the seismic design provisions have changed three times (1992, 2006, 2008), and the code for concrete structures has been changed two times (1990, 2004)? A complete answer to this question is offered by this paper.

The objective of this study is to assess the vulnerability to progressive collapse of three distinct models representing a 13-storey RC framed building, designed and detailed according to Romanian seismic codes in use, in 1992, 2006 and nowadays, when all four damage cases are considered. The paper, by comparative studies, estimates the influence of the evolution of Romanian seismic design codes on the progressive collapse resistance of a typical RC framed structures located in a region of high seismic risk (Bucharest, Romania).

2. SEISMIC ANALYSIS

2.1. Building model

In order to determine the progressive collapse resistance of a structure located in a high seismic area in Romania, the present study was conducted on a typical 13-storey RC framed building, designed according to three distinct Romanian seismic codes used in design in the last 20 years. The structure consists of five 6.0 m bays in the longitudinal direction and two 6.0 m bays in the transverse direction. The story height is 2.75 m, except the first two floors which are 3.6 m in height. The thickness of the slab is 150 mm. Based on this structure, three distinct models were developed. The model (shown in Figure 2.1) was generated in the FEA computer software Autodesk Robot 2010; dimensions of the structural components of the models are presented in Table 2.1.

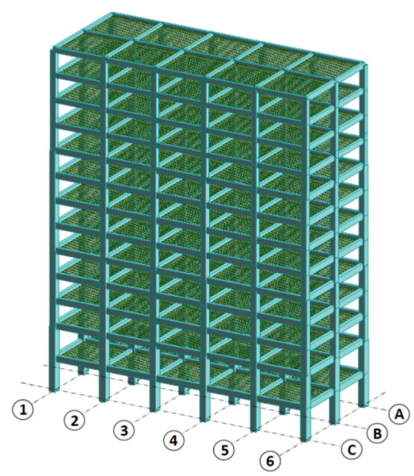

Figure 2.1. Model of a 13-storey RC framed structure

Table2.1. Dimension of the structural elements [mm]

Story	Columns	Beams	
		Longitudinal direction	Transversal direction
1, 2	700x900	350x650	350x700
3, 4, 5	700x750	350x650	350x700
6, 7, 8, 9	600x750	300x650	300x700
10, 11, 12, 13	600x600	300x550	300x600

2.2. Model P100-92

The structure was designed according to the provisions of the seismic design code P100-92. In design at the Ultimate Limit State, the Special Combination of loads according to the Romanian Standard STAS 10101/0A-77 is $DL + 0.4LL + E$, where DL is dead load, (composed by self-weight and an additional dead load of 2.00 kN/m^2), LL is live load which is 2.4 kN/m^2, and E is the earthquake effect. The magnitude of total equivalent seismic force S_r is:

$$S_r^{P100-92} = \alpha \cdot k_s \cdot \beta_r \cdot \psi \cdot \varepsilon_r \cdot G = 0.095G \tag{2.1}$$

where: α is the importance factor of the structure depending on the importance class (for building of importance class II, α has the value 1.2); k_s is the seismic coefficient (the seismic analysis is made for Bucharest which is located in Zone C on the Romanian seismic map with the seismic coefficient k_s = PGA/g = 0.2); β_r is the coefficient of dynamic amplification in mode "r" of vibration (for flexible structures and for $T_r \leq T_C$, β_r has the value 2.5); T_r, T_C and β_r are the parameters that describe the ground of Bucharest (T_r = 0.1n = 1.3 s, T_C = 1.5 s); Ψ is a reduction coefficient of the seismic action (for multi-story RC framed structures and when the infill walls are not considered structural elements, it has the value 0.2); ε_r is the coefficient of equivalence between real system and a SDF system corresponding to the mode "r" of vibration; G is the weight of structure G = 49531 kN.

The structural response of the model under the Special Combination of loads is determined by a 3D linear static analysis performed in the FEA computer software Autodesk Robot. The material properties are given in Table 2.2. Reinforcement is made following the provisions of the standard for RC structures STAS 10107/0-90. The modal response spectrum analysis gives the following values for the fundamental periods: T_1 = 1.23 s and T_2 = 1.22 s.

Table 2.2. Strengths of materials for the model P100-92 [MPa]

Material		Seismic design	Progressive collapse analysis	
		Design values*	Characteristic un-factored values	With 1.25 factor
Concrete Bc20		$R_c = 12.5$	$R_{ck} = 16.6$	20.75
		$R_t = 0.95$	$R_{tk} = 1.43$	1.78
Steel	PC 52	$R_a = 300$	$R_{ak} = 345$	431
	OB 37	$R_a = 210$	$R_{ak} = 255$	318

* R_c (R_t) – design value for the compressive (tensile) strength of concrete; R_a – design value for the yield strength of steel reinforcement.

2.3. Model P100-1/2006

The model was seismically designed according to the provisions of the former seismic code P100-1/2006 and detailed according to SR EN 1992-1-1:2004 – standard which had replaced the national standard for RC structures STAS 10107/0-90. In design, a similar Special Combination of loads was used. According to CR 1-1-3-2005, the snow load has a new value: S = 1.28 KN/m^2 for Bucharest. In the seismic code P100-1/2006, the expression for the seismic base shear force F_b is:

$$F_b^{P100-2006} = \gamma_1 \cdot S_d(T_1) \cdot m \cdot \lambda = 0.09996G \tag{2.2}$$

where: γ_1 is the importance factor of the structure depending on the importance class (for building of importance class II, γ_1 has the value 1.2, in P100-92 code it was the α factor); m is the total mass of the building above the foundation; λ is the correction factor which takes into account the contribution of the fundamental mode of vibration (if $T_1 < T_C$ and the building has more than two stories, then λ = 0.85). T_1 is the fundamental period of building vibration and S_d (T_1) is the ordinate of the design spectrum and might be calculated with the expression:

$$S_d(T) = a_g \cdot \frac{\beta(T)}{q} \tag{2.3}$$

where: a_g, $\beta(T)$, T_B and T_C, are the parameters that describe the ground of Bucharest ($a_g = 0.24g$, $T_B = 0.16$ s, $T_C = 1.6$ s and $\beta(T) = \beta_0 = 2.75$). The parameter a_g is the design ground acceleration and q is the behavior factor. Structures located in seismic regions with $a_g > 0.16g$ should be designed according to the requirements of the high ductility class (DCH). The behavior factor for frame systems is:

$$q = 5 \cdot \frac{\alpha_{11}}{\alpha_1} \tag{2.4}$$

where: $\frac{\alpha_{11}}{\alpha_1} = 1.35$ for multi-story and multi-bay frames. The behavior factor q has the value 6.75.

When the provision of the seismic design code P100-1/2006 is used in the seismic analysis, the magnitude of the base shear force increases by 5.2% with respect to the total equivalent seismic force calculated with the design code P100-92. The seismic design code P100-1/2006 places the RC structures located in seismic areas with $a_g > 0.16g$ in a high ductility class (DCH), and provides specific provisions for this class. The material properties are given in Table 2.3. Reinforcement of the beams and columns is made considering the provisions of the design code for concrete structures EC-2 (SR EN 1992-1-1:2004) and the additional measures required by the design of elements in the class. From the modal response spectrum analysis of the model, made in the FEA computer software Autodesk Robot, the following fundamentals periods result: T_1=1.15 s and T_2=1.13 s. It is observed that the fundamental periods are decreasing with respect to the model P100-92 because the modulus of elasticity E for concrete is different: for concrete class Bc 20 the modulus of elasticity has the value 27 000 MPa and for concrete class C25/30 it has the value 31 000 MPa.

Table 2.3. Strengths of materials for the model P100-2006 [MPa]

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 value for the compressive (tensile) strength of concrete; f_{yd} – design value for the yield strength of steel reinforcement.

2.4. Model EC-8

A similar analysis was conducted considering a new model, seismically designed according to the provisions of the present seismic design code SR EN 1998-1:2004/NA: 2008 (EC-8) and detailed according to the design code for concrete structures SR EN 1992-1-1: 2004 (EC-2). The seismic design code SR EN 1998-1:2004/NA: 2008 provides the following relationship for the seismic base shear force F_b:

$$F_b^{EC-8} = S_d(T_1) \cdot m \cdot \lambda = 0.155G \tag{2.5}$$

where: m and λ have the same values as in the model P100-1/2006. The expression for the design spectrum is:

$$S_d(T_1) = a_{gR} \cdot S \cdot \frac{2.75}{q} \tag{2.6}$$

where, the values for parameters that define the elastic response spectrum for Bucharest (zone z_3) are: $a_{gR} = 0.24g$, $T_B = 0.16$ s, $T_C = 1.6$ s and S = 1 (S is the sol factor); a_{gR} is the peak value of the reference ground acceleration on type A ground. The behavior factor q is calculated with:

$$q = q_0 \cdot k_w = 4.5 \cdot \frac{\alpha_{11}}{\alpha_1} \cdot 1 = 5.85 \tag{2.7}$$

where: q_0 is the basic value of the behavior factor, k_w is the factor reflecting the prevailing failure mode in structural systems with walls, $k_w = 1$ and $\frac{\alpha_{11}}{\alpha_1} = 1.3$ for multi-story RC framed structures.

When the provisions of the seismic design code SR EN 1998-1:2004/NA:2008 (EC-8) are applied in the seismic analysis, the magnitude of the base shear force increases by 21% with respect to the total equivalent seismic force calculated with the design code P100-92, and by 15% with respect to the seismic force calculated with the design code P100-1/2006. The reinforcement of the structural elements is made considering the provisions of the design code for concrete structures SR EN 1992-1-1:2004 (EC-2) and also, the additional measures required by the design of elements in the high ductility class (DCH) from the seismic design code SR EN 1998-1:2004/NA:2008 (EC-8). The materials properties are the same as in the model P100-2006.

3. PROGRESSIVE COLLAPSE ANALYSIS

3.1. GSA 2003 Procedure

The progressive collapse is a dynamic and nonlinear event and takes place in a very short time. To analyze rigorously the potential to progressive collapse of a structure, nonlinear dynamic analyses should be performed. However, this type of analysis is time consuming and it is not used in the current design for low and mid-rise buildings.

The GSA (2003) Guidelines recommend for buildings of 10 stories or less, with relatively simple layouts, the *Alternative Path Method* based on a linear elastic analysis. This is a direct approach which requires that the structure must be capable to bridge over the removed member as a result of abnormal loads. In the static analysis, the following vertical load shall be applied downward to the structure under investigation:

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

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. The following analysis scenarios ("missing column" scenarios) shall be considered: the instantaneous loss of column at the first story located at or near the middle of the short side of the building – case C_1, at or near the middle of the long side – case C_2, at the corner of the building – case C_3 and an interior column – case C_4, as it is shown in Figure 3.1.

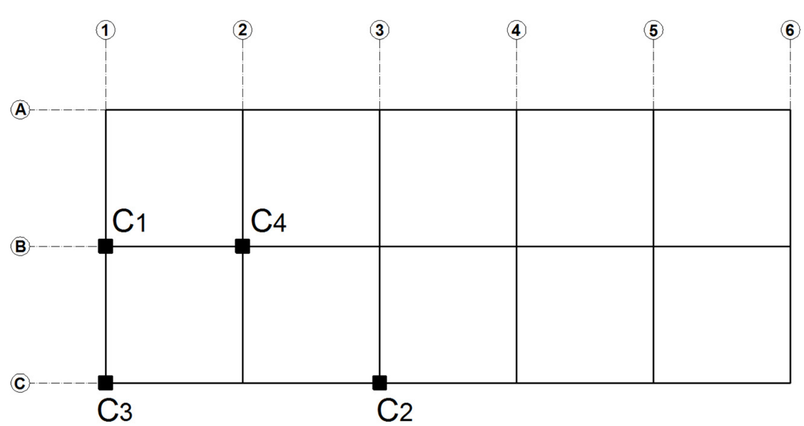

Figure 3.1. Missing column scenarios according to GSA (2003) Guidelines

Following the linear static analysis, a **D**emand-**C**apacity **R**atio (DCR) is calculated for each structural element:

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

where: Q_{UD} is the acting force (demand) determined in component or connection (moment, axial force, shear and possible combined forces) and Q_{CE} is the expected ultimate un-factored capacity of the component or connection (moment, axial force, shear and possible combined forces), which results from seismic analysis.

In the assessment of Q_{CE}, strength increase factors are applied to the properties of materials taking into account the strain rate effect and material over-strength. For RC framed structures, the strength increase factor is 1.25. Using the DCR criteria, structural elements and connections that have DCR values greater than 2.0 are considered to be severely damaged or collapsed. If all the DCR values are less than or equal to 1.0, then the structure is expected to behave elastically when a vertical support is removed.

The analysis has been performed for all the four cases C_1, C_2, C_3 and C_4 (Figure 3.1). In this paper, only the case C_4 – interior column removal – are extensively discussed, because this case is rarely presented in literature, and for this damage case, the structure seems to be the most vulnerable. After the removal of the interior column, the bending moment and shear force diagrams on the damaged structure under gravity loads (Eqn. 3.1) are displayed in Figure 3.2 and 3.3.

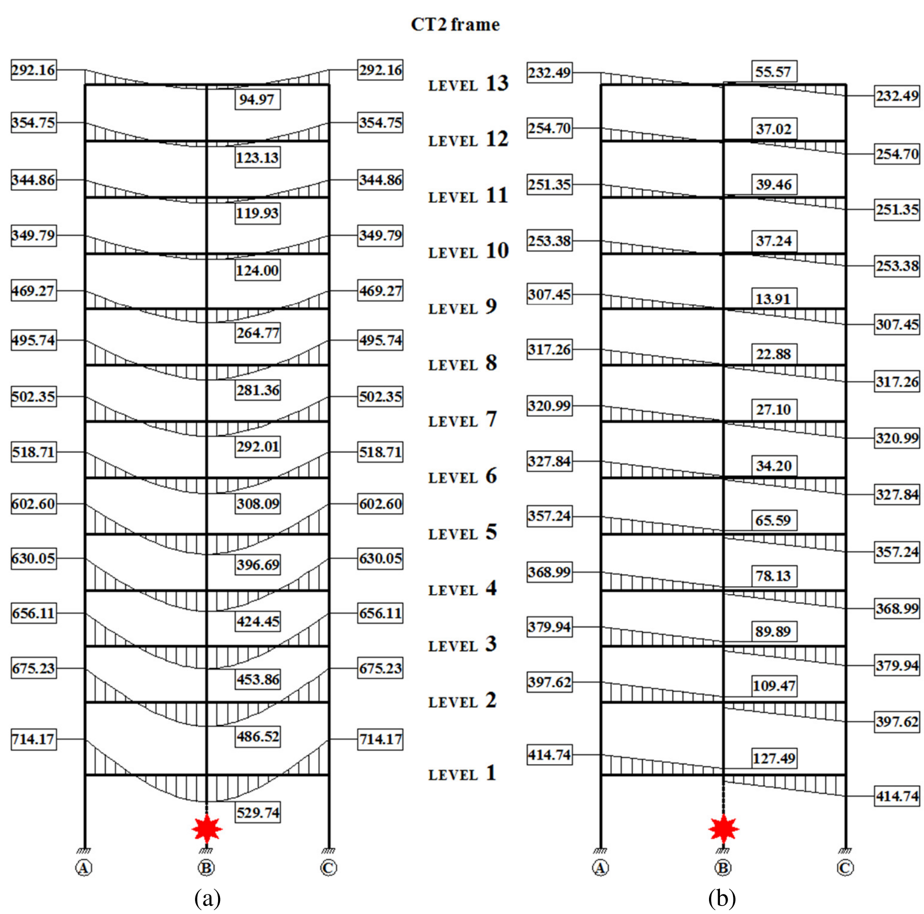

Figure 3.2. Damaged structure – longitudinal frame CT_2: a) bending moments [kNm]; b) shear forces [kN]

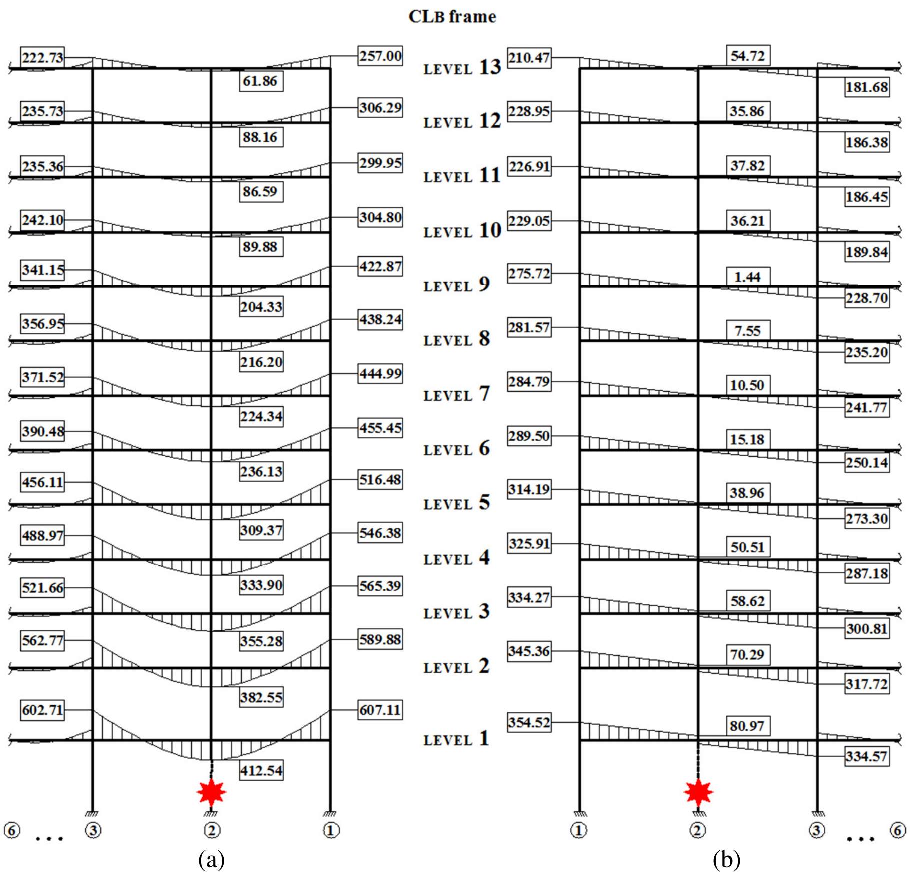

Figure 3.3. Damaged structure – longitudinal frame CL_B: a) bending moments [kNm]; b) shear forces [kN]

3.2. Damaged model P100-92

Following the GSA (2003) Guidelines, demands in beams Q_{UD} are assessed and compared to the expected ultimate un-factored beam capacities Q_{CE}. In the case of the damaged model P100-92, the DCR values for significant beam sections are represented, for the lower part of interior transverse frame CT_2, in Figure 3.4, and in Figure 3.5 for the longitudinal frame CL_B.

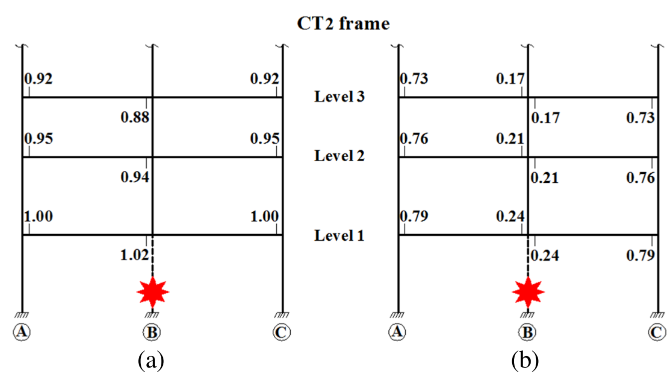

Figure 3.4. Damaged model P100-92 transverse frame CT_2: a) DCR values for flexure; b) DCR values for shear

All the DCR values for flexure are below the allowable limit (2.00); the maximum DCR value is 1.02 at mid-span of the first floor beam, above the removed column. Practically, the model behaves elastically. The DCR values for shear, presented in Figure 3.5b, are also well below 1.00, the maximum value being 0.79. As at the transverse frame CT_2, all the DCR values are below 1.00. For flexure the maximum DCR value is 0.94 at the end of the 12^{th} floor beam, and for shear the maximum DCR value is 0.72 at the first floor beam.

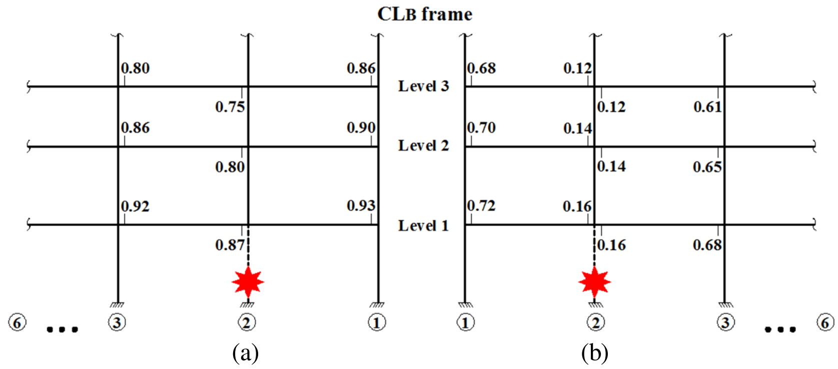

Figure 3.5. Damaged model P100-92 longitudinal frame CL_B: a) DCR values for flexure; b) DCR values for shear

Finally, the model P100-92 behaves elastically when subjected to abnormal loads (missing column damage scenarios) and consequently, there is no risk for progressive collapse. All four damage scenarios (C_1 to C_4) lead to a similar conclusion (Table 4.2).

3.3. Damaged model P100-2006

When the structure is designed according to the provisions of the seismic design code P100-1/2006 and subjected to progressive collapse, the maximum DCR values for flexure are 0.93 at the longitudinal frame CL_B, and 0.95 at the transverse frame CT_2. The maximum DCR value for shear is 0.92, for both, longitudinal and transverse frame and it was recorded at the first floor beam. Therefore, there is no risk of progressive collapse. All the four damage scenarios confirm this conclusion (Table 4.2).

A little difference is noticed in terms of DCR values for flexure with respect to the model P100-92, because the material properties have been changed, as shown in Tables 2.2 and 2.3. The maximum DCR values for shear increased from 0.72 to 0.92, due to the decrease of the expected ultimate un-factored capacity Q_{CE} calculated according to the provisions of the present code SR EN 1992-1-1:2004 (EC-2). The model P100-2006 has an improved shear reinforcement (Φ10/130 mm of S500 type steel) compared to the model P100-92 (Φ8/140 mm of OB37 type steel), but the ultimate un-factored shear capacity of the beam is significantly lower ($V_{Rd}^{P100-2006}$ = 451.30 kN compared to $V_{Rd}^{P100-92}$ = 522.25 kN). This unexpected change in shear DCR values has been explained by Ioani and Cucu (2010) in their papers.

3.4. Damaged model EC-8

In case of the structure designed according to the provisions of the present seismic design code SR EN 1998-1:2004/NA: 2008 (EC-8), the assessment of the potential to progressive collapse according to the GSA (2003) Guidelines leads to the following maximum DCR values for flexure: 0.84 at the end of the12^{th} floor beam on the longitudinal frame CL_B, and 0.85 at mid-span of first floor beam on the transverse frame CT_2.

The DCR values decreased in comparison with the damaged model P100-2006. When the structure is designed according to the present seismic design code, the internal forces are greater than those obtained when the structure is designed according to the former seismic design code P100-1/2006. The difference is approximately 15%, and in consequence, the expected un-factored capacities of structural members Q_{CE} are higher. For the same reason, the DCR values for shear also decreased with respect the damaged model P100-2006. The maximum DCR values for the longitudinal frame CL_B is 0.83, and for the transverse frame CT_2 is 0.85. Like for the others two damaged models (P100-92 and P100-2006), the conclusion is that the structure has no risk for progressive collapse. All the four damage scenarios confirm this conclusion (Table 4.2).

4. COMPARATIVE RESULTS

A summary of the main results concerning the behavior to progressive collapse of a 13-storey RC framed structure located in a high seismic zone and designed according to the Romanian seismic codes in use in the last 20 years is presented in Tables 4.1 and 4.2. Commentaries are made in Section 5.

Table 4.1. Main seismic design parameters

Parameters	Seismic design code		
	P100-92	P100-2006	EC-8
Behavior factor q	-	6.75	5.85
Ground acceleration a_g	0.20g	0.24g	0.24g
Equivalent seismic force [kN]	0.095G	0.09996G	0.115G
Equivalent seismic force [%]	100%	105.2%	121%

Table 4.2. Main results and conclusions in the assessment of progressive collapse potential

Damaged model	Maximum DCR values for flexure				Maximum DCR values for shear			
	C_1	C_2	C_3	C_4	C_1	C_2	C_3	C_4
P100-92	0.82	0.93	1.07	1.02	0.58	0.67	0.53	0.79
	Low risk for progressive collapse				*No risk for progressive collapse*			
P100-2006	0.85	1.10	0.84	0.95	0.73	0.77	0.69	0.92
	Low risk for progressive collapse				*No risk for progressive collapse*			
EC-8	0.75	0.82	0.74	0.85	0.62	0.72	0.60	0.85
	No risk for progressive collapse				*No risk for progressive collapse*			

5. CONCLUSIONS

This paper presents the results of a parametric study regarding the influence of the Romanian seismic codes evolution on the progressive collapse behavior of mid-rise RC framed structures located in high seismicity zones. Three successive Romanian seismic design codes of the last 20 years are considered in the analysis. A typical 13-storey RC framed structure is designed according to each of the three seismic codes. Many parameters such as the ground acceleration of the location (a_g), the provisions regarding the allowed minimum ductility class of structural elements as well as the magnitude of the behavior factor q, have been changed during this period. The progressive collapse potential is assessed, in terms of flexure and shear, through the static linear elastic procedure specified by GSA (2003) Guidelines. The analyses have been performed for all the "missing column" scenarios defined by GSA (2003) Guidelines. Since there are very few references to the case C_4 (interior column removal), the present paper detailed results and conclusions corresponding to this damage case. Based on the results of this study, the following conclusions can be made:

1. A typical mid-rise (13-storey) RC framed structure located in a high seismic area (Bucharest), designed and detailed according to the seismic codes P100-92, P100-1/2006 or SR EN 1998-1:2004/NA:2008 (EC-8), does not have a risk for progressive collapse when is subjected to different missing column damage scenarios. Excepting very few beam sections where low inelastic demands are identified ($1.00 \leq DCR \leq 1.02$), the structures practically behaved elastically ($DCR < 1.00$). Shear DCR values are also smaller than 1.00, and therefore the models satisfy the GSA (2003) acceptance criteria.

2. Compared to P100-92, the more recent codes P100-1/2006 and SR EN 1998-1:2004/NA: 2008

(EC-8) lead to an increase of the seismic design force of 5.2%, respectively 21%. As a direct consequence, the expected flexural capacity of beams will increase too, and the magnitude of demand-capacity ratio (DCR) decreases by 7% to 17%. The safest model against the progressive collapse is the model designed and detailed according to the active codes EC-8 and EC-2 (Table 4.2). Thus, an important finding of this study, of great importance for structural engineers, is that the changes in the Romanian seismic codes brings an improvement in terms of progressive collapse resistance of RC structures, and confirm the implicit benefits on progressive collapse resistance when the European modern design codes are used in design of concrete structures.

3. In the progressive collapse analyses, structural engineers should pay a particular attention to the damage case C_4 (interior column removal) which leads to the highest DCR values for flexure and shear, as the 12 damage cases displayed in Table 4.2 have shown. Therefore, a 3D framed structure seems to be more vulnerable when its interior column is removed.

4. A similar analysis is in progress, considering that the structure would be located in a seismic area with $a_g = 0.20g$, that is the lower limit of a high seismic zone.

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