

The Behaviour of an Old Representative Reinforced Concrete Building subjected to Abnormal Loads

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Abstract

This paper assesses the vulnerability to progressive collapse of an old and representative twelve-storey reinforced concrete (RC) framed structure, designed forty years ago for Braila according to the provisions of the Romanian Design Codes P13-70 (1970) and STAS 8000-67 (1967). The progressive collapse potential is assessed using a linear static procedure by applying damage scenarios specified in the GSA 2003 Guidelines. The results reveal that the structure has a low potential to progressive collapse based on its flexural resistance. The sensibility to shear of the old RC structure is carefully investigated and the design shear capacities are compared to experimental values furnished by static and dynamic tests.

Keywords: progressive collapse, reinforced concrete framed structure, linear static procedure, damage scenarios, GSA 2003 Guidelines, demand-capacity-ratios, expected shear capacity.

1 Introduction

Progressive collapse is defined by the GSA 2003 Guidelines [1] as a “situation where a local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse”. The damage, disproportionate to the original cause is due to the abnormal loads produced by natural hazard (e.g. earthquakes) or by man-made hazard (e.g. terrorist attacks, impact by vehicles, bomb blast, etc). These loads are not considered in the ordinary structural design.

The well-known progressive collapse cases of the Ronan Point Building (England, 1968), the Murrah Federal Building (Oklahoma City, USA, 1995) and, most important, the World Trade Center (New York City, USA, 2001) have shown the disastrous consequences of this type of structural failure. Therefore, the design philosophy of structures subjected to abnormal loads is to mitigate the risk for

progressive collapse. Two major US guidelines, published by GSA (GSA 2003 [1]) and DoD (DoD 2005 [2] and DoD 2009 [3]) provide an independent methodology for minimizing the potential to progressive collapse in buildings. Both documents recommend the use of the Alternative Path Method to establish if alternative loads paths could develop when a vertical support is instantaneously removed from the structure as a result of abnormal loading.

Many studies [4, 5, 6, 7, 8, 9] using as a support the GSA 2003 Guidelines [1] have investigated the risk for progressive collapse of RC framed structures designed for different seismic areas when subjected to abnormal loads. Ioani [6] has shown that a typically medium-rise RC framed building seismically designed for a zone of high seismic risk from Romania according to the present regulations, does not experience progressive collapse when subjected to the removal of an exterior or interior column as defined by the GSA criteria.

The main objective of this paper is to investigate the risk for progressive collapse of a 40 years old and representative building of 12 storeys, when subjected to abnormal loads. In Romania, during the last 40 years, the provisions of the seismic code as well as of the design code for concrete structures have changed significantly. For instance, the structure was designed according to the older Romanian Seismic Code P13-70 [10] for a seismic base force $S = 0.0567G$, while the current Seismic Design Code SR EN 1998-1-2004 (EC-8) [11] requires that the building should be designed for a higher (almost twice) seismic base force $S = 0.115G$. P13-70 [10] contains a number of provisions much more permissive than those of the current seismic code [11]. Also, in the design and detailing of beams, the old standard for concrete structures STAS 8000-67 [12] uses a completely different method to estimate the shear capacity of beams and has accepted materials (concrete, reinforcing bars) with lower strength and ductility properties.

Taking into account these aspects, but also the fact that the analysed structure was erected in a zone with high seismic risk ($a_g=0.24g$) and has experienced the effects of four major earthquakes, the suspicion of the designers that this type of building could have a high potential to progressive collapse when subjected to abnormal loading, seems to be justified. For these reasons and under these circumstances, the paper examines the vulnerability to progressive collapse of the building and tries to conclude if this type of structure seismically designed 40 years ago (hundreds are still in service), satisfies the present acceptance criteria given in the progressive collapse analysis [1].

2 Structural characteristics

The structure consists of five 6.0 m bays in the longitudinal direction and two 6.0 m bays in the transverse direction, and has 12 storeys. The total height of the building is 34.7 m of which the typical floor-to-floor height is 2.75 m and of the first two storeys is 3.6 m. The structural components of the actual building are presented in Table 1.

Dead load is composed by the self-weight of structure and non-structural elements (walls), a supplementary variable dead load (1.65 kN/m^2 on the first floor,

1.05 kN/m² for the roof and 1.35 kN/m² for current floors). Live load is taken as 4.0 kN/m² on the first floor (commercial spaces) and 2.0 kN/m² for the rest of them.

Structural components	Level	Transverse direction		Longitudinal direction	
		Exterior frame	Interior frame	Exterior frame	Interior frame
		Dimensions [mm]	Dimensions [mm]	Dimensions [mm]	Dimensions [mm]
Beams	1 – 2	300x700	350x700	300x650	350x650
	3 - 12	300x600	300x600	300x600	300x600
Columns	1 – 2	750x600	900x700	-	-
	3 – 5	600x600	750x700	-	-
	6 – 12	600x600	600x600	-	-
Slabs	1 – 2	120 mm			
Thickness	3 – 12	100 mm			

Table 1: Dimensions of structural members

The 12-storey RC framed structure is modeled as a 3-D linear elastic model (Figure 1) using the FEA computer program SAP 2000.

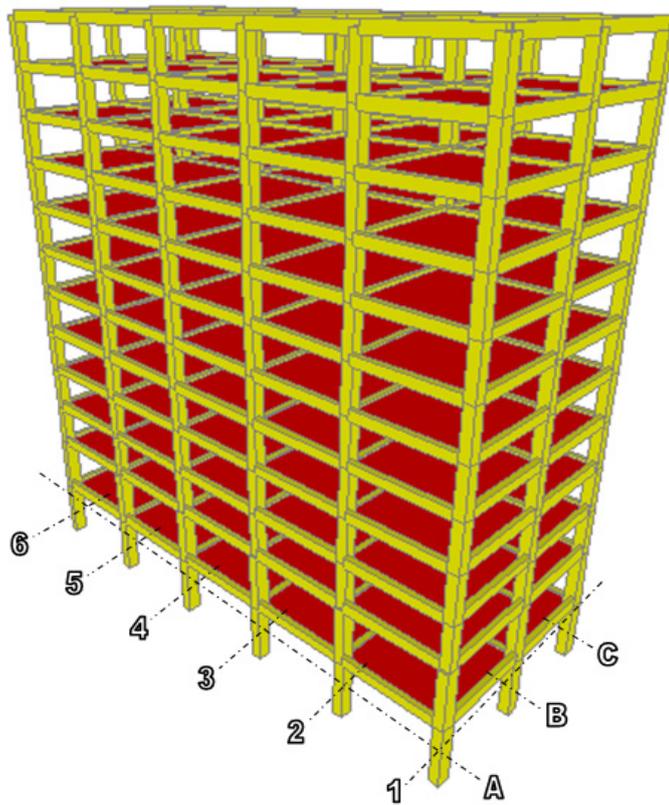


Figure 1: SAP 2000 model for the 12-storey RC framed structure

3 Linear static procedure

Following the GSA 2003 Guidelines [1] the potential to progressive collapse is assessed considering the loss of a vertical support in the so called “missing column” scenarios as illustrated in Figure 2:

- a) Case C₁: loss of an exterior column located near to the middle of the short side;
- b) Case C₂: loss of an exterior column located near to the middle of the long side;
- c) Case C₃: loss of a column located at the corner of the building;
- d) Case C₄: loss of an interior column.

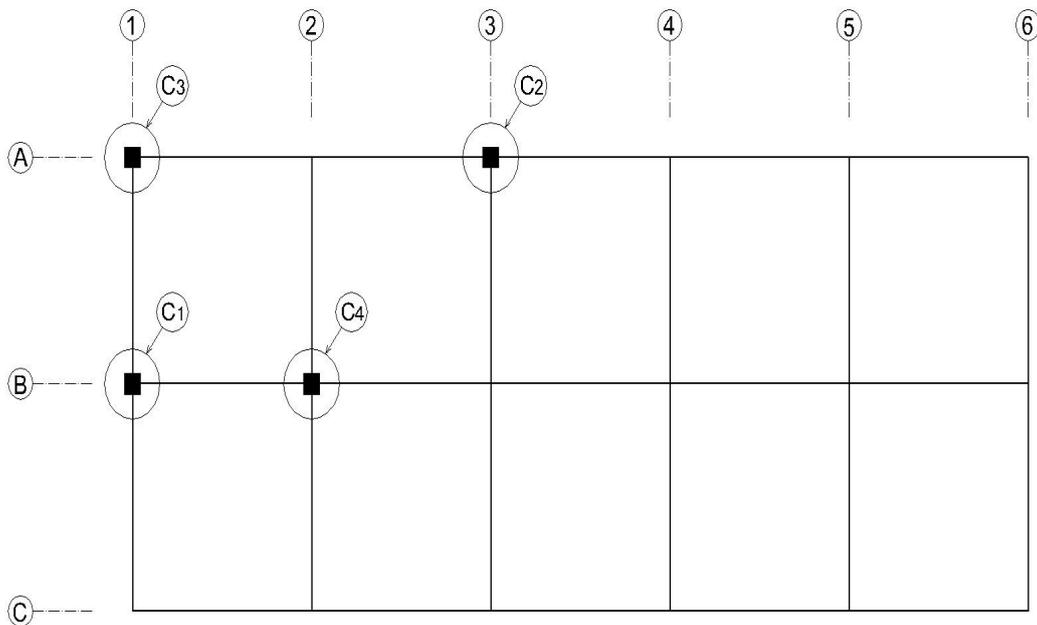


Figure 2: “Missing column” scenarios according to the GSA 2003 Guidelines

A linear static analysis is used to assess the potential to progressive collapse of the existing building. Following the step-by-step procedure [1], a vertical load is applied downward to the structure under investigation:

$$\text{Load} = 2(\text{DL} + 0.25\text{LL}) \quad (1)$$

where:

DL = dead load;

LL = live load;

In the GSA criteria, live load is reduced to 25% of the full design live load, admitting that the entire LL value is less probable. At the same time, by multiplying the load combination by a factor of two, the GSA 2003 Guidelines [1] 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, etc [13].

Working with the results given by the linear elastic analysis (moment, shear, axial force), engineers shall identify the magnitude and distribution of potential areas of inelastic demands and thus, they will quantify 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:

$$DCR = Q_{UD}/Q_{CE} \quad (2)$$

where:

Q_{UD} = acting force determined in member or connection (moment, axial force, shear or combined forces), using the linear static analysis.

Q_{CE} = 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 increased by a strength increase factor of 1.25, both for concrete and steel bars [1].

Using the DCR criteria of the linear elastic approach, structural elements and connections that have DCR values that exceed the allowable value are considered to be severely damaged or collapsed [1]. The allowable value for typical structural configurations is $DCR \leq 2.0$ [1].

The GSA 2003 Guidelines [1] admits an allowable collapse area which is based on the structural bay size. The allowable extent of collapse resulting from the instantaneous removal of an exterior column will be taken as the smallest area from the structural bays directly associated with the removed column in the floor level above or 1800 feet^2 ($\approx 167 \text{ m}^2$). In the case of removing an interior column, the allowable extent of collapse will be taken as the smallest area from the structural bays also associated to the removed column or 3600 feet^2 ($\approx 334 \text{ m}^2$).

4 Seismic design of the building

The building was designed in 1972 and erected in 1974 in Braila (Romania), a zone with high seismic risk; the seismic design is made following the provisions of the standard P13-70 [10], and detailing of beams and columns according to the Romanian standard for concrete structures STAS 8000-67 [12]. During its existence, the building was “in-situ tested” by four major earthquakes, as follows: 1977 Earthquake with a magnitude of $M = 7.5$, 1986 with $M = 7.1$, 30 May 1990 with $M = 6.9$ and 31 May 1990 with $M = 6.4$, where M is the earthquake magnitude on Richter scale. Since 1986 the building has been seismically instrumented and its structural response has been closely monitored.

According to P13-70 [10], the Special Combination of loads used in the seismic design was:

$$DL+0.8LL+E \quad (3)$$

where:

DL = dead load;
 LL = live load;
 E = earthquake effect.

According to the old Standard P13-70 [10], Braila is situated in the zone 8 of seismic risk with $k_s = 0.08$. For the Romanian territory the seismic coefficient k_s varies from 0.03 to 0.12. The magnitude of the total equivalent seismic force S corresponding to load combination given by Equation (3), is calculated as follows, and leads to:

$$S = k_s \cdot \beta_r \cdot \Psi \cdot \varepsilon_r \cdot G = 0.0576 G \quad (4)$$

where, G is the total weight of the structure.

If the building would have been designed according to the current code SR EN 1998-1-2004 (EC-8) [11], a much higher (almost twice) seismic force S would result:

$$S = S_d(T_1) \cdot m \cdot \lambda = a_{gr} \cdot \gamma_1 \cdot s \cdot \frac{2.75}{q} \cdot \frac{G}{g} \cdot \lambda = 0.115 G \quad (5)$$

where, G is the total weight of the structure.

Regarding the Special Combination of loads, the present seismic code [11], considers a reduced value for live loads (0.3LL instead of 0.8LL) as follows:

$$DL+0.3LL+E \quad (6)$$

where:

DL = dead load;
 LL = live load;
 E = earthquake effect.

It has to be underlined that the building was “in-situ tested” by those four major earthquakes and even designed for a much lower seismic force S according to P13-70 [10], it stands with no structural damages, as different technical survey reports have indicated.

The original 1972 design project was reanalyzed, the authors discussed with the designers of the building, checked data and drawing of the original project and then redesigned the structure, including details following the provisions of the older codes P13-70 [10] and STAS 8000-67 [12]. Certain details for beams are presented in Table 2.

The structural response of the model under the Special Combination of loads and the behaviour of the damaged structure (with all four cases of the “missing column” scenarios as defined by the GSA 2003 Guidelines [1]), is determined using a 3-D linear elastic model, created in the FEA computer program SAP 2000.

Exterior frames						
Storey	Longitudinal frame*			Transverse frame*		
	Top long. steel**	Bottom long. steel**	Stirrups at ends**	Top long. steel**	Bottom long. steel**	Stirrups at ends**
1, 2	1Φ22 + 3Φ25 (1.02%)	4Φ20 (0.69%)	Φ8/200 (0.17%)	4Φ25 (1.00%)	2Φ25+ 1Φ22 (0.69%)	Φ8/200 (0.17%)
3, 4, 5, 6	2Φ22+ 2Φ25 (1.04%)	2Φ20+ 1Φ22 (0.61%)	Φ8/200 (0.17%)	1Φ22 + 3Φ25 (1.11%)	1Φ25+ 2Φ22 (0.75%)	Φ8/200 (0.17%)
7, 8, 9	4Φ22 (0.91%)	2Φ18+ 1Φ20 (0.49%)	Φ6/200 (0.09%)	2Φ22 + 2Φ25 (1.04%)	3Φ22 (0.68%)	Φ8/200 (0.17%)
10, 11, 12	3Φ18+ 1Φ20 (0.65%)	3Φ14 (0.27%)	Φ6/200 (0.09%)	4Φ20 (0.75%)	3Φ16 (0.36%)	Φ6/200 (0.09%)
Interior frames						
Storey	Longitudinal frame*			Transverse frame*		
	Top long. steel**	Bottom long. steel**	Stirrups at ends**	Top long. steel**	Bottom long. steel**	Stirrups at ends**
1, 2	2Φ22+ 2Φ28 (1.04%)	2Φ20+ 2Φ22 (0.65%)	Φ8/200 (0.17%)	2Φ22+ 2Φ28 (0.96%)	3Φ25 (0.64%)	Φ8/200 (0.14%)
3, 4, 5, 6	1Φ22+ 3Φ25 (1.11%)	1Φ16+ 3Φ18 (0.58%)	Φ8/200 (0.17%)	4Φ25 (1.18%)	3Φ22 (0.68%)	Φ8/200 (0.17%)
7, 8, 9	3Φ22+ 1Φ25 (0.98%)	3Φ16 (0.36%)	Φ8/200 (0.17%)	2Φ22+ 2Φ25 (1.04%)	2Φ18+ 1Φ22 (0.53%)	Φ8/200 (0.17%)
10, 11, 12	1Φ18+ 3Φ20 (0.72%)	2Φ14+ 1Φ16 (0.31%)	Φ6/200 (0.09%)	2Φ20+ 2Φ22 (0.83%)	3Φ16 (0.36%)	Φ6/200 (0.09%)

*Concrete: Class B 200 according to STAS 8000-67 [12] with the design compressive strength $f_{cd}=10 \text{ N/mm}^2$, equivalent to actual C12/15 concrete strength class

**Reinforcing steel: - type PC52 for longitudinal reinforcement with the design yield strength $f_{yd}=290 \text{ N/mm}^2$, where Φ is the bar diameter in mm and ρ_l is the reinforcement ratio for longitudinal reinforcement

- type OB38 for transverse reinforcement with $f_{yd}=210 \text{ N/mm}^2$

Table 2: Reinforcing details of structural elements

5 Progressive collapse analysis

Using the GSA criteria, demands (Q_{UD}) in structural components (beams) are compared with their expected ultimate un-factored capacities (Q_{CE}), as shown in Equation 2. For beams, the DCR values for flexure are calculated at the column faces and are presented in figures, in brackets.

5.1 Bending moments and DCR values

5.1.1 Case C_1 and C_2 : exterior short and long side column cases

The case C_1 and C_2 of “missing column” scenarios (Figure 2) are discussed in a previous paper [14]. All DCR values are below 2.0 and thus meet the acceptance criteria provided by GSA 2003 Guidelines [1]. The results show that the damaged structure is not expected to fail in bending and consequently the progressive collapse is not expected to occur for both cases. In other words, based on its flexural resistance, the building has a low potential for progressive collapse when subjected to the case C_1 and C_2 of the “missing column” scenarios.

5.1.2 Case C_3 : corner column case

The removal of a corner column affected the exterior transverse frame CT_1 and the exterior longitudinal frame CL_A , these two frames being interconnected at the removed column (Figure 2). The largest moments in beams for the “damaged” structure are developed at the first and second floor; a positive bending moment of 417.18 kNm (increased by 145% with respect to the design bending moment) and a negative bending moment of 613.05 kNm (increased by 128% with respect to the design bending moment) were obtained for the transverse frame CT_1 (two bays).

For the longitudinal frame CL_A (five bays) the maximum positive moment is 363.01 kNm (increased by 118% with respect to the design bending moment) and the maximum negative moment is 540.58 kNm (increased by 106% with respect to the design bending moment), as illustrated in Figure 3.

The DCR values range from 0.57 to 1.19 for the transverse frame CT_1 , and from 1.02 to 1.96 for the longitudinal frame CL_A . Due to the fact that all DCR values are above 1.0, indicating the inelastic behaviour of those beam sections, a plane failure mechanism is expected to occur for the longitudinal frame CL_A . But, this frame will be sustained by the structural components of the transverse frame CT_1 where most of the DCR values for flexure are below 1.0, indicating the elastic behaviour of these members.

For both frames, all DCR values are below 2.0, a generalized 3D failure mechanism does not occur in the model, and consequently the acceptance criteria specified by the GSA 2003 Guidelines [1] are fulfilled and the building has a low potential for progressive collapse.

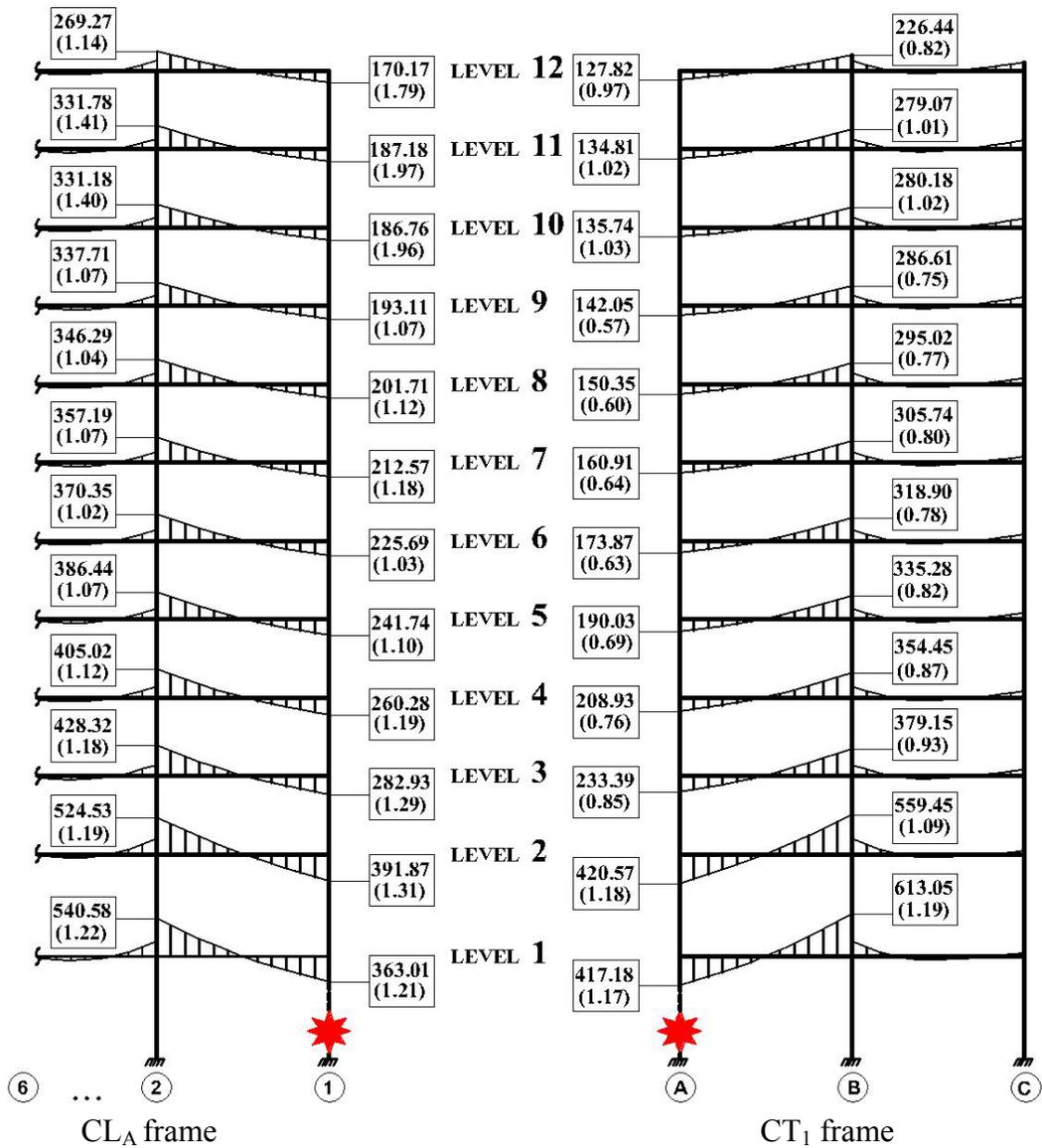


Figure 3: Bending moments (kNm) and DCR values (in brackets) for the damage case – C_3

5.1.3 Case C_4 : interior column case

In the technical literature, very few papers discuss the behaviour of buildings when an interior column is removed [7, 15]. After the removal of the interior column, the moment diagrams have changed, as illustrated in Figure 4.

As in the case C_3 , the largest moments in beams for the “damaged” structure are developed at the first floor; a positive bending moment of 609.33 kNm (increased by 257% with respect to the design bending moment) and a negative bending moment of 790.56 kNm (increased by 144% with respect to the design bending moment) were obtained for the transverse frame CT_2 .

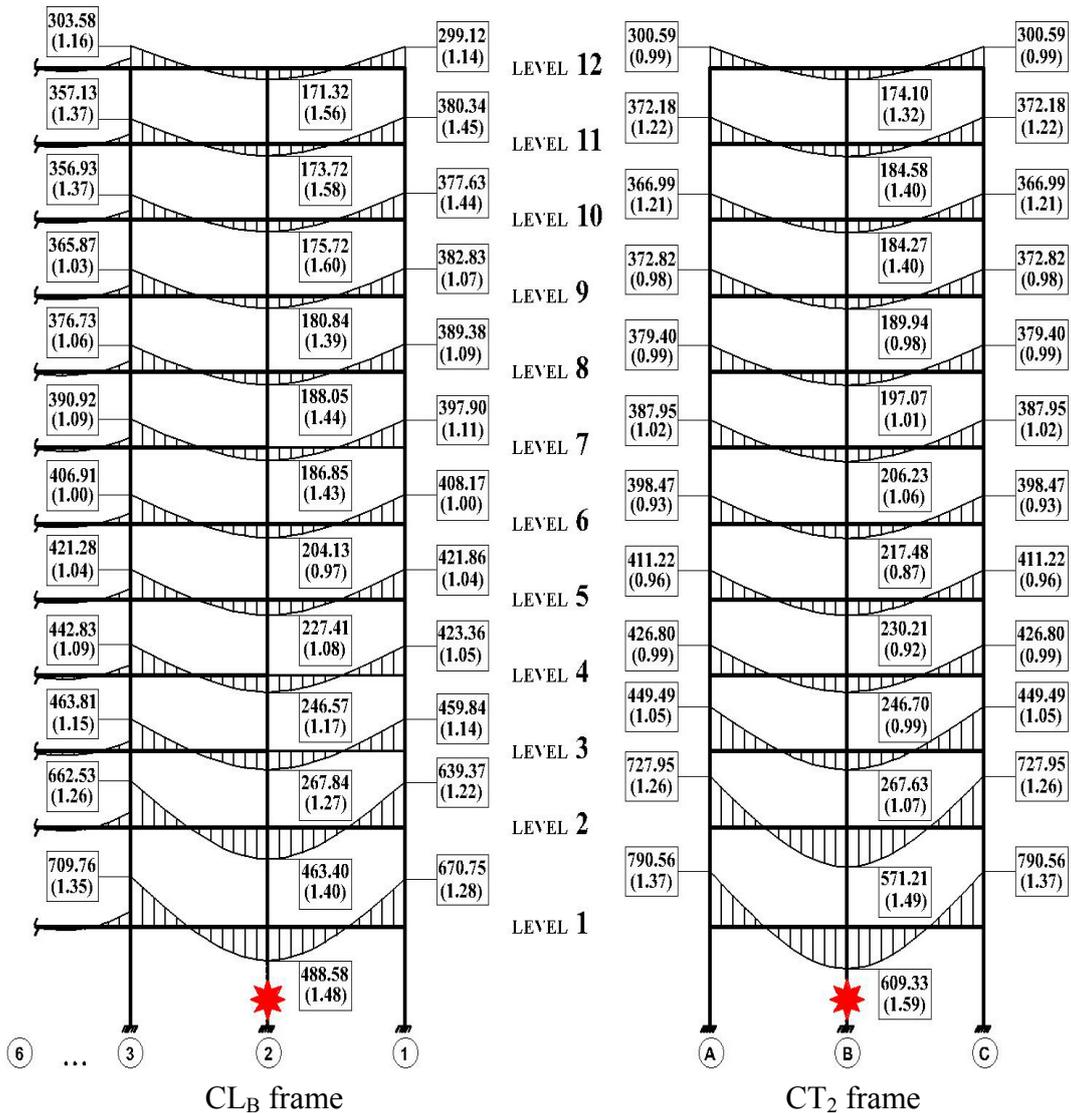


Figure 4: Bending moments (kNm) and DCR values (in brackets) for the damage case – C₄

The DCR values range from 0.87 to 1.59 for the transverse frame CT₂ and from 0.97 to 1.60 for the longitudinal frame CL_B. Practically, the beams from the longitudinal frame CL_B have an inelastic behaviour at both ends (all DCR values are above 1.0, except one from the sixth floor which is 0.97) leading to a possible plane failure mechanism (Figure 4); the longitudinal frame is sustained by the upper parts of the transverse frame CT₂ which behaves elastically (DCR<1.0).

For this case, no generalized 3D or local mechanism will occur, all DCR values for flexure are below 2.0, and thus the building meets the acceptance criteria provided by the GSA 2003 Guidelines [1] having a low potential for progressive collapse.

be considered as failed members when using the current code [16], and there is no risk for progressive collapse by applying the provisions of the older code [12].

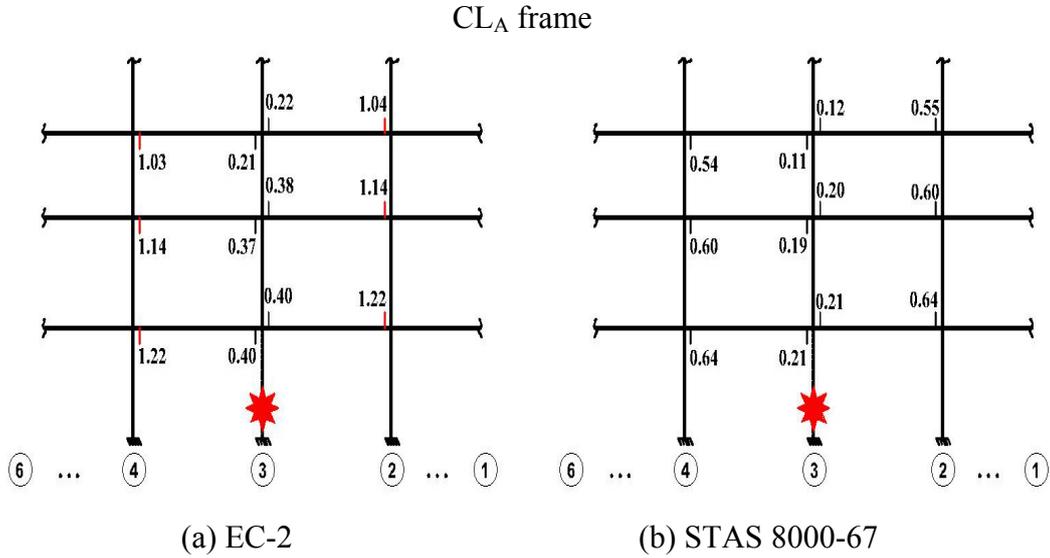


Figure 7: DCR values for shear for the exterior longitudinal frame CL_A – case C₂

5.2.3 Case C₃: corner column case

When the corner column is removed from the structure, this mainly affected both the exterior transverse frame CT₁ and the exterior longitudinal frame CL_A (Figure 8).

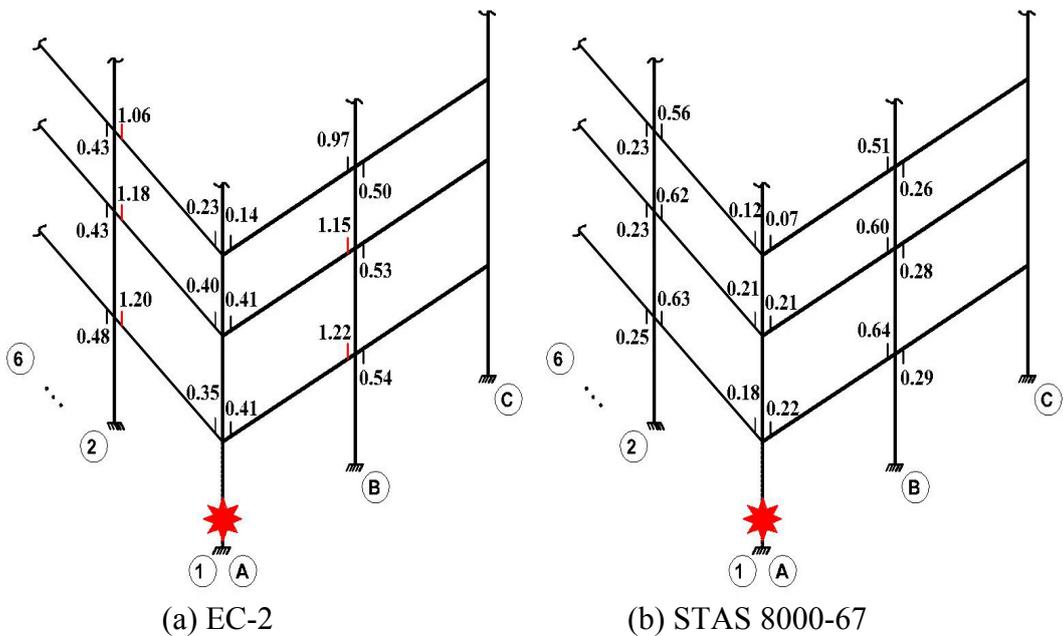


Figure 8: DCR values for shear – case C₃

At least at one end of beam elements from level 1 to level 4, the DCR values for shear are above 1.0 ($DCR_{max}=1.22$) when the expected un-factored capacities (Q_{CE}) are calculated using the current code [16] (Figure 8a) and therefore, these elements could be considered as failed members. By using the relationships from the old code [12], a maximum DCR value for shear of 0.64 (Figure 8b) was obtained in the transverse beam from the first level which is well below the allowable value (1.0), and consequently the building has no risk for progressive collapse based on its shear resistance.

5.2.4 Case C₄: interior column case

The instantaneous removal of an interior column makes the structure more vulnerable to progressive collapse. This is shown by the magnitude and distribution of DCR values for shear obtained at beams ends (Figure 9).

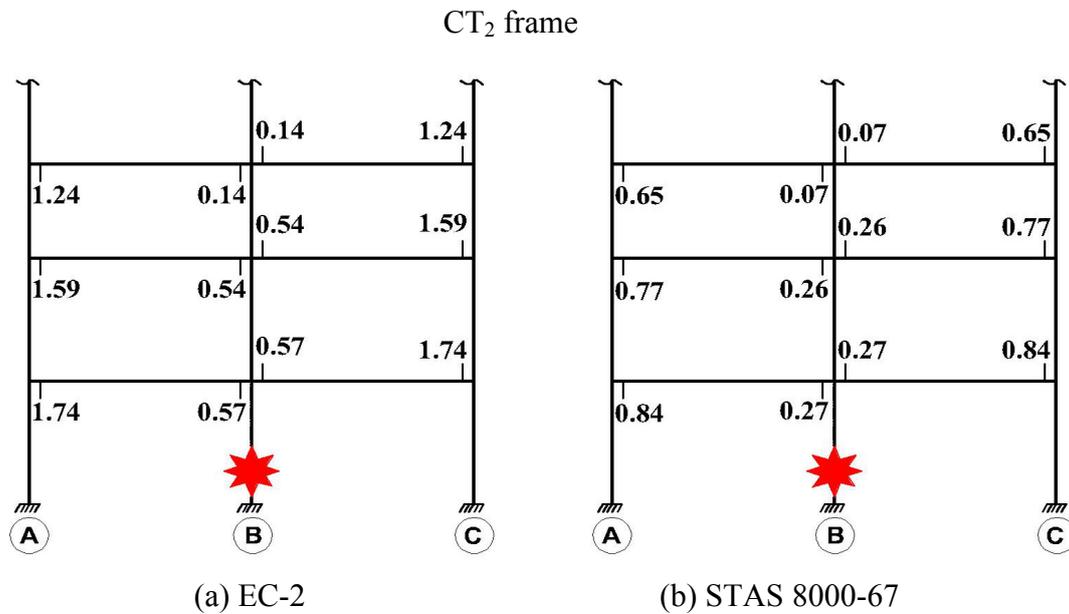


Figure 9: DCR values for shear for the interior transverse frame CT₂ – case C₄

The removal of the interior column affected the interior transverse frame CT₂ and the interior longitudinal frame CL_B. All beam elements from level 1 to level 12 have DCR values above 1.0. The highest DCR value for shear (1.74) is obtained for the transverse beam from the first floor when the provisions of the current code [16] are applied. Instead, a maximum DCR value of 0.84 resulted when the relationships from the old code [12] are applied. Similar results (in terms of DCR values for shear) are obtained for the longitudinal frame CL_B ($DCR_{max} = 2.03$). Thus, as in previous cases of “missing column” scenarios there is no risk for progressive collapse only when the relationships from the old code [12] are used to establish the expected un-factored shear capacities (Q_{CE}).

5.2.5 Discussion on shear capacity of RC beams

A final possible conclusion of the previous sections (5.2.1 to 5.2.4) could indicate that the building will collapse in shear because, for all the analyzed damage scenarios, the maximum DCR values are above the allowable value 1.0 (1.43/C₁, 1.22/C₂, 1.22/C₃ and 2.03/C₄) when the evaluation of the shear capacity is made using the provision of the active code for concrete structures EC-2 [16].

This un-expected conclusion requires an attentive analysis which involves supplementary theoretical and experimental data. The analysis is made for the first floor beam of the transverse frame CT₁ which was designed according to the provisions of the old standard STAS 8000-67 [13].

During the last 40 years, the provisions of the seismic code as well as of the design codes for concrete structures have changed substantially. The seismic design according to SR EN 1998-1-1:2004 [11] leads to a seismic base force $S = 0.115G$ (G – total weight of the building) which is almost twice the seismic force calculated according to P13-70 [10] ($S = 0.0576G$). While the older code [10] admits the use of the concrete type B200 (equivalent to C12/15) and reinforcement type OB38 (equivalent to S255), the current code [11] specifies a minimum concrete class C25/30 (for high ductility class) and the use of steel for reinforcement with the characteristic yield strength (f_{yk}) between 400 and 600 N/mm² in the critical regions of seismically designed elements. Also, the distance between stirrups is much more limited: the old code [12] specifies the use of the smallest value from $\{h_{beam}/3, 15d, 300 \text{ mm}\}$ and the current code [11] recommends the use of the smallest value from $\{h_{beam}/4, 6d, 175 \text{ mm}\}$. Therefore, if the analyzed beam with cross section 300X700 mm would be designed to resist shear failure according to the current codes [11,16], stirrups $\Phi 10/130 \text{ mm}$ with $f_{yk}=500 \text{ N/mm}^2$ (S500) and concrete class C25/30 should be used. The shear capacity of this beam ($V_{Rd} = 504.02 \text{ kN}$) is much higher than the capacity calculated using the provisions of the old code [12]: stirrups $\Phi 8/200 \text{ mm}$ with $f_{yk}=260 \text{ N/mm}^2$ (OB38) and concrete type B200 ($V_{Rd} = 164.70 \text{ kN}$). Thus, it is understandable why, for a newly designed structure, the potential to progressive collapse of the building would be much lower.

In Table 3 are listed relationships for the assessment of the design shear capacity, and also the values obtained for the analyzed beam when the provisions of different design codes for concrete structures are applied.

The code STAS 8000-67 [12], STAS 10107/0-76 [17], STAS 10107/0-90 [18] and ACI 318-08 [19] takes into account the contribution of the stirrups and also of the concrete when the design shear capacity is calculated. Instead, the code SR EN 1992-1-1: 2004 (EC-2) [16] considers that when the acting shear force cannot be assumed only by the concrete, the design shear capacity of the section will be assess taking into account only the contribution of the stirrups. As a consequence, the design shear capacity calculated according to the current code [16] ($V_{Rd} = 164.70 \text{ kN}$) is lower than the computed value according to the older code [12] ($Q_{eb} = 240.18 \text{ kN}$). Practically, the DCR values for shear do not satisfy the acceptance criteria if the expected shear capacity of the section is made according to EC-2 [16] (DCR = 1.34) and ACI 318-08 [19] (DCR = 1.35).

In a theoretical and experimental program carried out by the National Institute for

Building Research – INCERC, Romania, results from 326 RC beams, statically loaded and tested at shear up to the failure, were analyzed [20]. The experimentally measured shear capacities (Q_{exp}) were compared to design shear capacities (Q_d), calculated according to different design codes, using design material strengths (Table 3).

Codes	Relationships for calculating the design shear force	Design shear force (Q_d)	$\frac{Q_{CE}}{Q_d}$	$\frac{Q_{exp}}{Q_d}$	DCR for shear
STAS 8000-67 [12]	$Q_{eb} = \sqrt{0.6bh_0^2R_iq_e} - A_{et}R_{at}$	240.18 kN	1.88	0.90	0.70
STAS 10107/0-76 [17]	$Q_{eb} = \sqrt{3.2bh_0^2R_t\sqrt{p}(1 + \frac{Q}{M}h_0)q_e} - A_{et}m_{at}R_a$	174.67 kN	1.70	1.24	1.06
STAS 10107/0-90 [18]	$Q_{eb} = q_e s_i + \frac{bh_0^2\sqrt{p}}{s_i} m_t R_t$	186.69 kN	1.68	1.16	1.01
SR EN 1992-1-1:2004 [16]	$V_{Rds} = \frac{A_{sweff}}{s} z f_{ywd} \cot\theta$ $\cot \Theta = 2.5 (\Theta = 22^\circ)$	164.70 kN	1.44	1.31	1.34
ACI 318-08 [19]	$V_n = V_c + V_s = 0.17\lambda\sqrt{f_c}b_w d + \frac{A_v f_{yt} d}{s}$ $\Phi V_n = \text{design shear force}$ $\Phi=0.75$	150.00 kN	1.56	1.44	1.35

Table 3: Design shear capacities and DCR values for shear obtained for the analyzed beam, using different relationships specified by codes

This extensive experimental program revealed that, by using the provisions of the code STAS 10107/0-76 [17], the ratios (Q_{exp}/Q_d) are always greater than 1.24 for 325 beams, and is 1.12, for one single beam. Accepting that a similar value of 1.24 may be applied in our analysis, the minimum real shear capacity of the analyzed beam will be greater than $174.64 \cdot 1.24 = 216.50$ kN. With respect to this experimental value determined under static loads, Table 3 shows that the older

standard [12] overestimates the shear design capacity ($Q_{exp}/Q_d = 0.90$), meanwhile the modern codes EC-2 [16] and especially ACI 318-08 [19] lead to conservative results ($Q_{exp}/Q_d = 1.31$, respectively 1.44).

Recent experimental and theoretical study [21] demonstrated that the shear capacity of RC beams increases substantially under dynamic loads. The cracking shear force increases by 88% and the shear at the first yield of the stirrups by 44% to 74%, under dynamic loads [21]. If an average dynamic increase factor of 1.5 is considered, as Fujikake determined in his paper [22], the expected real shear capacity of the section under dynamic loads should be greater than $1.5 \cdot 216.50 \text{ kN} = 324.70 \text{ kN}$. Using this value $Q_{CE} = 324.70 \text{ kN}$, and $Q_{UD} = 316 \text{ kN}$ (resulted from linear static analysis of the damaged structure – case C_1), then the maximum value for DCR is 0.98, and consequently the model has no risk for progressive collapse based on its shear resistance. Similar commentaries and conclusions may be done for the other damage scenarios (C_2 , C_3 and C_4).

6 Conclusions

The present study establishes the potential to progressive collapse of an old and representative mid-rise RC framed building when subjected to abnormal loads produced by natural hazard (e.g. earthquakes) or by man-made hazard (e.g. terrorist attacks, impact by vehicles, bomb blast, etc).

The GSA 2003 Guidelines [1] provide an independent methodology to assess the potential to progressive collapse for the existing buildings. Based on the results furnished by the linear static analysis procedure, the DCR (**D**emand-**C**apacity-**R**atios) values are used to identify the magnitude and distribution of potential areas of inelastic demands and thus, to quantify the potential collapse areas and compared to the allowable ones provided by the GSA criteria.

In a previous study [14], the authors have shown that this building has a low potential to progressive collapse when subjected to case C_1 and C_2 of the “missing column” scenarios, and its flexural response is considered.

In this paper, the investigation is continued by the analysis of the other two cases (C_3 and C_4), rarely presented in the technical literature.

In the case C_3 (corner column removal), the largest DCR value for flexure is 1.96, very closed to the allowable value (2.0) and practically all DCR values for flexure are above 1.0, indicating large inelastic demands in the longitudinal frame beams (CL_A). Due to the interconnection of the longitudinal frame with the transverse frame (CT_1) which behaves elastically, the possibility of creating a generalized space (3D) mechanism is eliminated, and the building has a low potential for progressive collapse.

In the case C_4 (interior column removal) the DCR values for flexure are in the range of 0.87 to 1.60. As in previous case, all DCR values for flexure determined at beams ends are above 1.0, indicating a possible plane failure mechanism of the three hinged type for the longitudinal frame (CL_B). But, this frame will be sustained by the structural components with elastic behaviour ($DCR < 1.0$) from the transverse frame (CT_2). A new finding of the paper indicated that the 12-storey RC building is

more vulnerable to progressive collapse when an interior column is removed, and this damage scenario should not be avoided in the analysis, as many designers currently do.

The significant changes during the last 40 years in the design codes for concrete structures from the 1967 standard STAS 8000-67 [12] up to the present SR EN 1992-1-1:2004 [16] has a significant impact on the results obtained in terms of DCR values for shear, and not for flexure.

If the evaluation of the expected un-factored capacities (Q_{CE}) of beam elements is made using the provisions of the current code EC-2 [16], certain beams have at least one end DCR values for shear above 1.0, and even higher than 2.0 (case C_4) in some components. As a consequence, these elements could be considered failed members and therefore the building would have a high risk for progressive collapse.

On the other hand, by applying the provisions from the old code [12] a maximum DCR value of 0.84 was obtained in the case C_4 of the “missing column “ scenarios and therefore, the building fulfilled the acceptance criteria [1]. The differences in DCR values for shear obtained using the provisions from these two codes [12, 16] are mainly due to the fact that the current one [16] does not take into account in certain circumstances the contribution of the concrete to resist shear failure. The acting shear force is assumed only by the stirrups if it is lower than the shear capacity of concrete section. In Table 3, based on experimental test results, it is shown that the design shear force calculated according to the old design code [12] is slightly overestimated ($Q_{exp}/Q_d = 0.90$); the design shear force predicted by the current code [16] is underestimated in reasonable limits ($Q_{exp}/Q_d = 1.31$).

The study has shown that, in general, the RC structures, designed according to the older codes are sensitive to shear forces, but an immediate conclusion of failure in shear should be prudently regarded. In particular, the structure under investigation does not fail in shear as it was demonstrated in Section 5.2.5 using results from theoretical and experimental studies [20, 21, 22].

To conclude, the old and representative 12-storey RC framed building, designed forty years ago for a high seismic zone according to older Romanian codes P13-70 [10] and STAS 8000-67 [12], is not expected to fail in bending or shear and consequently, the progressive collapse is not expected to occur when the building is subjected to damage scenarios provided by the GSA 2003 Guidelines [1]. This final conclusion is sustained by the in-situ behaviour of the real building which experienced without significant structural damages four severe earthquakes, including the 1977 Vrancea Earthquake ($M=7.5$).

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