PROGRESSIVE COLLAPSE ANALYSIS OF AN OLD RC STRUCTURE SUBJECTED TO EXTREME LOADING

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Abstract

In this paper, the progressive collapse resistance of an old and representative RC framed structure located in a region with high seismic risk from Romania (Brăila) is investigated. The 13-storey building was designed 40 years ago according to the Romanian codes P13-70 (1970) and STAS 8000-67 (1967). The building was "in-situ tested" by four major earthquakes, including the 1977 Vrancea earthquake with a magnitude of 7.5 on Richter scale, without any significant structural damages. A 3D discrete crack model based on the Applied Element Method was generated in the Extreme Loading® for Structures (ELS®) software. The results obtained with the ELS® computer program indicated a very good agreement with the experimental test performed by Yi et al. (2008) on a planar frame, even in the large displacement range (catenary effect). Following the GSA (2003) Guidelines, a nonlinear dynamic analysis is conducted first in order to establish the risk for progressive collapse of the 13-storey building. It was shown that under standard GSA loading the structure is not expected to fail when subjected to corner column removal. A nonlinear incremental dynamic analysis is also carried out to estimate with an increased accuracy, the ultimate load bearing capacity to progressive collapse of the building. It is found that the structure is capable of sustaining a maximum load of 1.72 times the standard GSA loading. For a higher load, the nonlinear dynamic analysis indicated that the old structure is expected to fail in shear, a quite rare phenomenon for modern RC framed buildings.

Key words

Applied Element Method; GSA (2003) Guidelines; nonlinear dynamic analysis; progressive collapse; RC structure

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1 INTRODUCTION

Progressive collapse is defined 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 [1]. The damage, disproportionate to the original cause is due to the extreme loads generated either by the natural hazard (e.g. earthquakes) or by man-made (e.g. gas explosions, terrorist attacks, impact by vehicles, etc.). These loads are not included in the initial phase of the structural design.

The engineering community has been engage in mitigating the risk for progressive collapse after the partial collapse of the Ronan Point Building (London, 1968) due to a gas explosion from the 18th storey. The interest in this field has been intensified after the terrorist attacks from the Murrah Federal Building (Oklahoma, 1995) and from the World Trade Center (New York, 2001), where both towers had been completely destroyed. Furthermore, between 1962 and 1971, in United States and Canada there were reported 605 cases of structural failure, from which 94 buildings fail through progressive collapse [2]. In addition, between 1989 and 2000, there were reported 225 cases of colapsed buildings, from which 54 % during the three years (1998-2000) [2].

In this context, the two U.S. Agencies, the General Services Administration (GSA) and the Department of Defense (DoD) published in 2003 [1], respectively in 2005 [3] and 2009 [4] guidelines for progressive collapse analysis of new and existing buildings. The Alternative Path Method has been selected by both agencies as the basic approach for providing resistance to progressive collapse for structures when subjected to extreme loading. This method is an independent approach and does not require data of the threat causing the loss of a primary structural component.

In order to resist this type of failure the buildings should be designed with an adequate level of continuity, ductility and redundancy, characteristics which are found in the seismic design codes, too (Eurocode 8 [5], ASCE 41-06 [6], P100/1-2013 [7]). This will provide a more robust structure and thus, will mitigate the risk for progressive collapse.

Numerical studies have indicated the beneficial influence of the seismic design on the progressive collapse resistance of mid-rise RC framed structures (11-13 stories), when these are designed according to the American codes [8, 9], according to the Taiwanese code [10] or according to the Romanian codes [11-13]. These results have been validated by experimental tests performed on beam-columns subassemblages [14-16] or on planar frames [17] considered as parts of RC framed buildings. Furthermore, three existing RC framed structures with 10, 11 and 20 stories were experimentally tested by Sasani [18-20]; it was shown that the structures are not expected to fail when subjected to first-storey column removal due to controlled explosions.

The structural behaviour of the existing buildings, especially the old ones, designed according to much more permissive codes and subjected to extreme loading is an open issue which need to be investigated. Therefore, the objective of this study is to assess the risk for progressive collapse of an old and representative RC framed structure subjected to suddenly column removal. The 13-storey building was designed in 1972 according to the Romanian seismic code P13-70 [21] and is located in Brăila, a region of high seismic risk. In the last four decades, the structure was "in-situ tested" by four major earthquakes that occurred in Romania and successive technical inspections showed that the building resisted without significant structural damages.

The Extreme Loading® for Structures (ELS®) software is used to model the building under investigation. Based on the GSA criteria, the structural model is analyzed using the nonlinear dynamic procedure for a column-removed condition. In order to determine with maximum accuracy the ultimate load bearing capacity to progressive collapse of the building, a nonlinear incremental dynamic analysis is carried out. The progressive collapse failure mode of the structure, subjected to the corner column scenario, is discussed in detail.

2 VALIDATION OF THE APPLIED ELEMENT METHOD

The Applied Element Method (AEM) is a new modelling technique that can track the progressive collapse behaviour of a structure passing through all stages of the application of loads (elastic stage, crack initiation and propagation, reinforcement yielding), the element separation, debris falling as rigid bodies, the contact between elements and collision with the ground or with adjacent structures [22].

In AEM, the structure is modelled as an assembly of small elements obtained by dividing the structure virtually. The connectivity in AEM is different from the FEM, where the elements are connected by nodes. AEM elements are connected using a series of normal and shear springs, generated automatically by the ELS® software on each element adjacent faces. These springs represent the continuity between elements and reflect the properties of the material used (concrete and reinforcement bars). Those springs that connect two adjacent elements and are represented by the main structural material compose the matrix springs. In reinforced concrete structure these springs represent the concrete part. When the average strain between two adjacent faces reaches the value of the separation strain, specified in the material properties of the model, springs between these faces are removed and the element behaves as separate bodies for the rest of the analysis. Similar, the reinforcement springs represent the steel bars from the model. These springs are cut off if the normal stress is equal or greater than the ultimate stress specified for this material.

The constitutive models for concrete and reinforcement bars used in the ELS® software are illustrated in Fig. 1. A Maekawa compression model [23] is adopted for modelling the concrete in compression before and after the cracking; for concrete springs subjected to tension, a linear stress-strain relationship is considered until reaching the cracking point (Fig. 1a). Also, the relationship between the shear stress and shear strain is assumed to be linear until the cracking of the concrete (Fig. 1b). For reinforcement springs (Fig. 1c), the model presented by Ristic [24] is used.

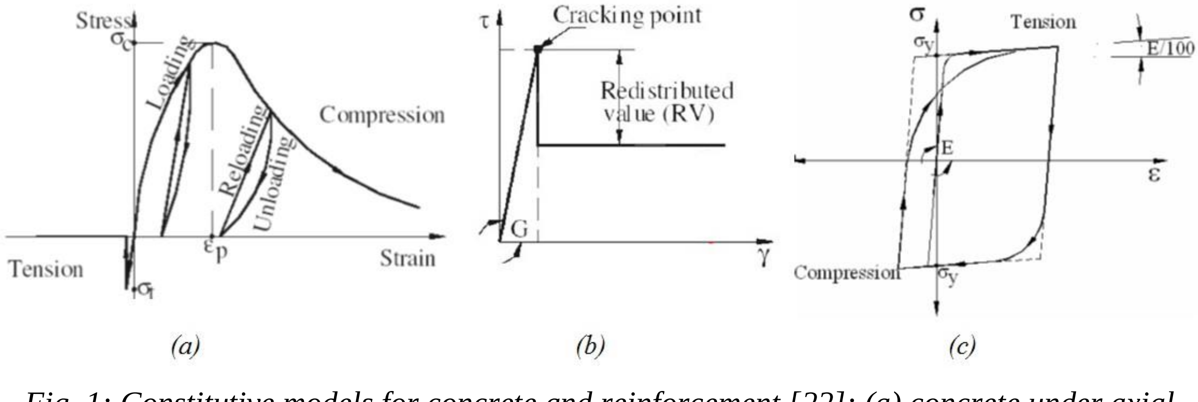


Fig. 1: Constitutive models for concrete and reinforcement [22]: (a) concrete under axial stresses; (b) concrete under shear stresses; (c) reinforcement under axial stresses

In order to validate the AEM (adopted by ELS®), the experimental test performed by Yi et al. [17] on a planar frame was numerically simulated with the ELS® software. For the experimental test, a one-third scale model representing the lower three-story of the original frame from the eight-story RC building was constructed. The model consists of four 2.667 m bays and three stories with the storey height of 1.10 m, except for the first level where the storey height is 1.567 m. The RC frame model is illustrated in Fig. 2. The dimensions and the reinforcement details of the structural components are given in Tab. 1.

The measured material properties are presented in the following. The cubic concrete compression strength was 25 MPa. For longitudinal reinforcement the measured yield strength was 416 MPa and the ultimate tensile strength was 526 MPa. The ultimate strain for steel was 27.5% (measured with steel gauge of 60 mm length). The yield strength for the lateral reinforcement was 370 MPa.

As in the experiment, the numerical simulation of the gradual failure of the first-storey middle column is performed in a displacement controlled manner as follows. A vertical load F = 109 kN is applied incrementally on the top of the middle column together with the self-weight of the structural components; the node associated to the failed column was fixed. Then, a vertical displacement of this node is increased gradually to simulate the column failure.

Tab. 1: Dimensions and reinforcement details of the structural elements [17]

Beams [mm]				**Columns [mm]**		
Dimensions	Longitudinal Top Bar	Longitudinal Bottom Bar	Stirrups	Dimensions	Longitudinal Reinforcement	Stirrups
100x200	2ϕ12	2ϕ12	ϕ6/150	200x200	4ϕ12	ϕ6/150

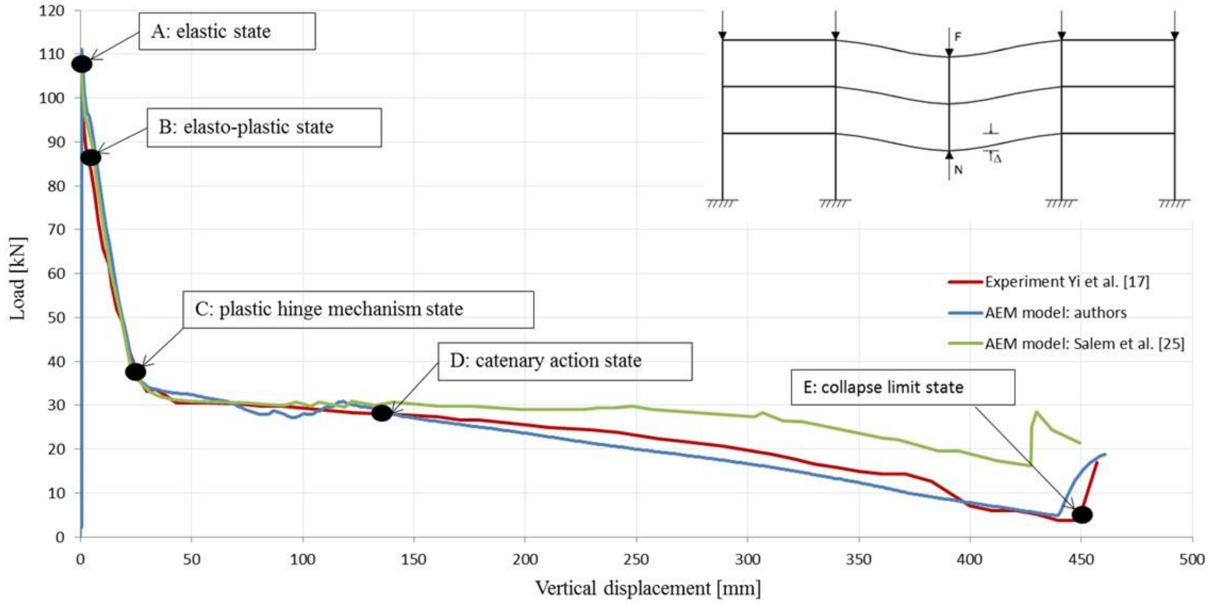
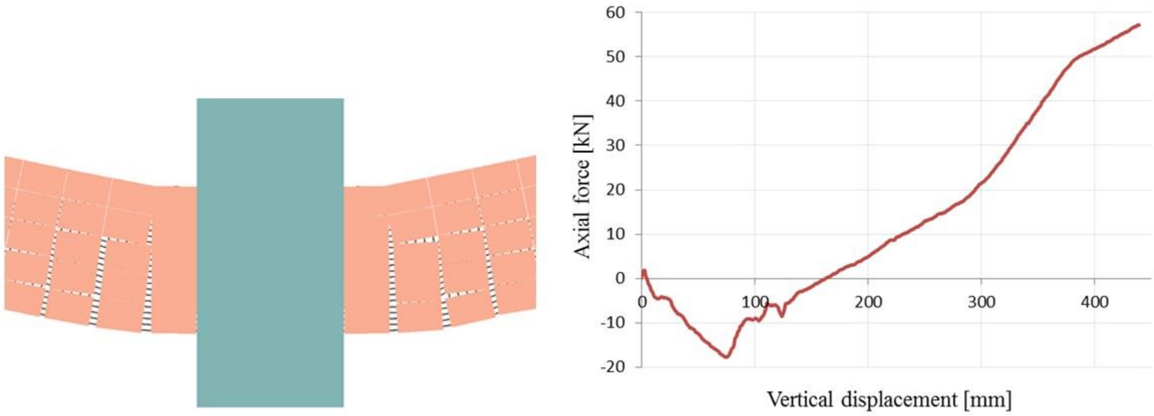


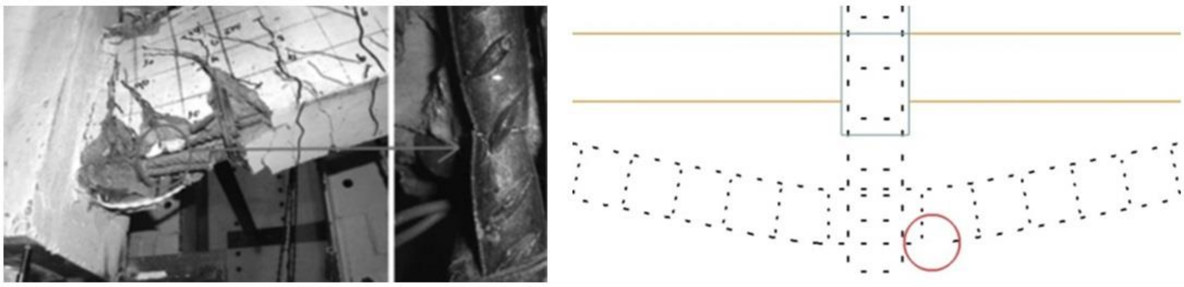
Fig. 2: Load-displacement curve of the column removed point

The observed behaviour of the frame model during the experimental [17] and the numerical tests is illustrated in Fig. 2. Section A-B is considered as the elastic stage ended with the cracking of the frame beams at point B. The elasto-plastic stage (B-C) finishes in point C with the yielding of the steel bars from the ends of the beams adjacent to the middle column indicating the formation of the plastic hinge mechanism. Section C-D represents the plastic stage where, as in the experiment, large plastic rotations at beams ends and severe concrete crushing are observed in the numerical simulation. After point D, the tension cracks in concrete penetrate the compressive zone (Fig. 3a). At the same time, the computed axial force in first floor beams adjacent to the middle column changes from compression to tension when the vertical displacement of the column removed point measures 160 mm (Fig. 3b) indicating the formation of the catenary mechanism (very close to 140 mm, the value reported in the experiment). At a vertical displacement of 440 mm (similar with 456 mm from the experiment), the bottom rebars from the first-storey beam adjacent to the middle column rupture. The location of the ruptured rebars from the experiment and the numerical model are illustrated in Fig. 4.



(a) *(b)*

Fig. 3: Catenary mechanism: (a) concrete cracks; (b) axial force in the first floor beam vs. vertical displacement of the column removed point



(a) *(b)*

Fig. 4: Rupture of reinforcing bars in the first floor beam adjacent to the middle column: (a) experiment [17]; (b) AEM model

Consequently, the results obtained herein indicate a very good agreement between the behaviour of the AEM model and the planar frame experimentally tested by Yi et al. [17], even better than the model validated by Salem et al. [25] using the same computational program (Fig. 2). Therefore, after this calibration, the ELS® software will be used with high confidence in authors' further progressive collapse analyses.

3 BUILDING DETAILS

3.1 Design details

The 13-storey RC frame building was design in 1972 and erected in 1974 in Brăila, a region with high seismic risk from Romania. The structure consists of five 6.0 m bays in the longitudinal direction and two 6.0 m bays in the transverse direction. The storey height is 2.75 m, except for the first two stories where the storey height is 3.6 m. In addition to the self-weight of the structural elements, supplementary dead loads of 2.2 kN/m^2 on the current floor, respectively 2.0 kN/m^2 on the roof floor were considered; due to the exterior walls a load of 6.5 kN/m on the first floor beams and 5 kN/m on the rest of the exterior beams were taken into account. Live loads of 2.0 kN/m^2 on the current floor and 2.5 kN/m^2 on the roof floor were considered as well. The dimensions of the structural components for the model under investigation are displayed in Tab. 2.

Tab. 2: Dimensions of the structural elements [mm]

Levels	Columns	Longitudinal beams	Transverse beams
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

The existing structure was designed following the provisions of the old Romanian seismic code P13-70 [21] and the design code for concrete structures STAS 8000-67 [26]. According to P13-70 [21], Brăila is situated in zone 8 of seismic risk with k_s = 0.05 (for apartment buildings). For the Romanian territory, the seismic coefficient k_s varies from 0.03 to 0.12. The magnitude of the total equivalent seismic force is S = 0.037G, where G is the total weight of the structure. If the same building have been designed according to the current code P100-1/2013 [7], a much higher seismic force S (S = 0.104G) would have resulted.

The original project of the building was reanalysed, the authors discussed with the designers, checked data and redesigned the structure according to the provisions of the old codes P13-70 [21] and STAS 8000-67 [26]. A concrete class B250 with the design compressive strength f_{cd} = 12 N/mm^2 and steel type PC52 with the design yield strength f_{yd} = 290 N/mm^2 for the longitudinal reinforcement, respectively OB38 with f_{yd} = 210 N/mm^2 for the transverse reinforcement were considered.

During its existence, the building was “in-situ tested” by four major earthquakes, as follows: in 1977 with a magnitude of M = 7.5, in 1986 with M = 7.1, on 30 May 1990 with M = 6.9 and on 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. It should be emphasized that the building “in-situ tested” by those four major earthquakes and designed for a much lower seismic force resists with no structural damages, as different technical reports have indicated.

3.2 Numerical model for progressive collapse analysis

The Extreme Loading® for Structures (ELS®) software was used to model the building under investigation. A total number of 82.730 of elements, which are below the maximum limit of 120.000 provided by the ELS®, connected by springs were considered. In order to provide more accurate results, a higher mesh discretization of 4x4x40 was used for the beams above the removed column, in regard to the rest of them (2x2x20), as recommended by Helmy et al. [27]. Beam elements are modelled as T or L sections to include the effect of the slab acting as a flange in monolithic constructions; as recommended by the building code ACI 318-11 [28] the effective flange width on each side of the beams was taken as four times the slab thickness. This value was adopted by Sasani and Sagiroglu [20] as well. The reinforcement details for the beams and columns are not provided herein. The material properties used in the progressive collapse analysis are given in Tab. 3. As recommended by the GSA (2003) Guidelines [1], the concrete compressive strength, respectively the yield and ultimate tensile strength for steel are increased by a factor of 1.25. The constitutive models for concrete and reinforcement bars used for the AEM model are illustrated in Fig. 1.

Tab. 3: Material properties considered in the analysis

Material	**Characteristic**	**Value**
Concrete B250	Young's modulus [GPa]	29
	Tensile strength [MPa]	1.9
	Compressive strength [MPa]	27.5
Steel PC52	Young's modulus [GPa]	210
	Yield strength [MPa]	425
	Ultimate tensile strength [MPa]	650
	Ultimate strain [%]	22
Steel OB38	Young's modulus [GPa]	210
	Yield strength [MPa]	325
	Ultimate tensile strength [MPa]	462.5
	Ultimate strain [%]	26

4 PROGRESSIVE COLLAPSE ANALYSIS

4.1 GSA criteria

As recommended by the GSA (2003) Guidelines [1], the risk for progressive collapse of a building is assessed considering the suddenly removal of a first-storey column located in four distinct zones: case C_1 – the removal of an exterior column located at the middle of the short side, case C_2 – the removal of an exterior column located at the middle of the long side, case C_3 – the removal of a corner column and case C_4 – the removal of an interior column. Only the case C_3 is considered herein.

When performing a dynamic analysis, the following loads combination is applied downward to the undamaged structure:

$$\text{Load} = DL + 0.25LL \tag{1}$$

Where, DL is dead load and LL is live load. In order to determine the expected capacity, the materials strengths are increased by a strength-increase factor of 1.25 for RC structures as recommended by the GSA (2003) Guidelines [1].

4.2 Nonlinear dynamic analysis

For the nonlinear dynamic analysis, the loads combination given by the Eq. (1) is applied downward to the undamaged structure. Then, the corner column is suddenly removed from the model. As recommended by the ELS® Theoretical Manual [22], the time for removal/time step is set to 0.001 s, a value also adopted by Salem et al. [25] in a similar analysis; this value is well below one tenth of the period associated with the structural response mode for the vertical motion of the bays above the removed column determined from the analytical model with the column removed (T = 0.19 s). Also, a damping ratio of 5% was considered in the dynamic analyses, a value adopted by Sasani et al. [20] and Tsai and Lin [10] as well. The response of the structural model is observed over a time span of t = 3 s and displayed in Fig. 5; after three seconds the building subjected to suddenly column removal reaches its new static equilibrium. The maximum vertical displacement of the column removed point is only 2.4 cm attained at t = 0.09 s. At this step, the structure is in the elasto-plastic stage with cracking of the concrete and yielding of the stirrups from the critical beams sections (not shown here); however, the plastic hinge mechanism is not reached, yet. This means that under the standard GSA loading (DL+0.25LL) the building is not expected to fail through progressive collapse when subjected to suddenly column removal as a result of abnormal loading.

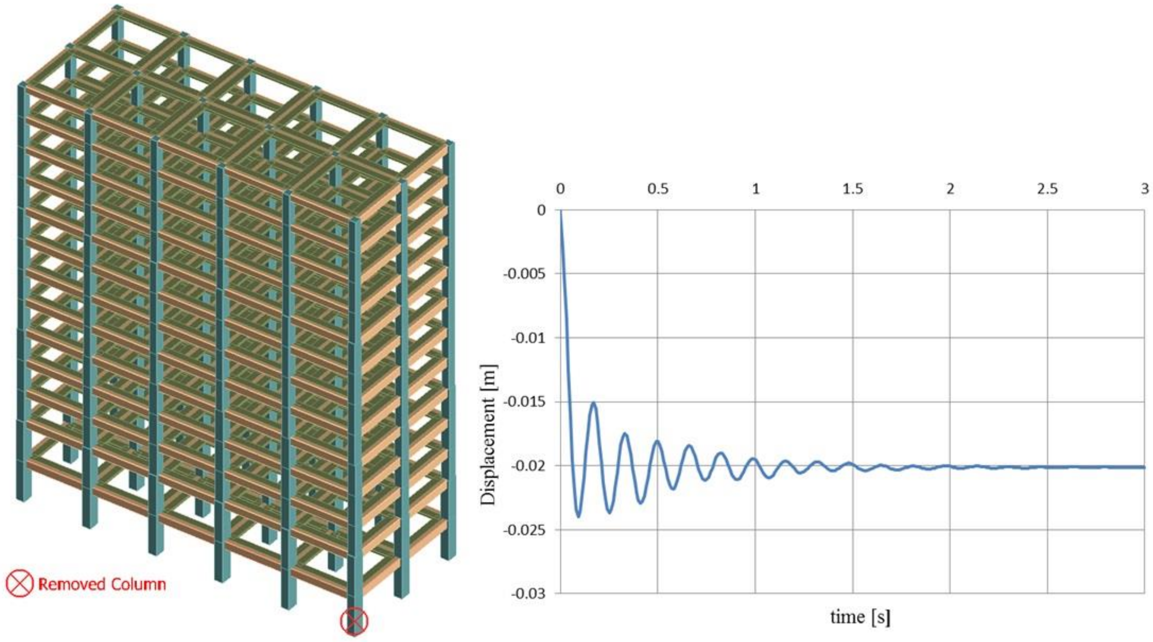


Fig. 5: Time-displacement curve for the column removed point under (DL+0.25LL)

4.3 Nonlinear incremental dynamic analysis

The destination of a building might be change during its lifetime, from apartments to office building or commercial spaces, leading to increased gravity loads. This assumes that the risk for progressive collapse established in the initial phase of the design could be changed from low to high. In this context, a nonlinear incremental dynamic analysis is conducted in order to establish the ultimate load bearing capacity to progressive collapse of the building; thus, the maximum value of the supplementary gravity load (additional to the standard GSA loading) for which the structure will fail through progressive collapse when subjected to suddenly column removal will be identified.

This method assumes to conduct a series of nonlinear dynamic "time-history" analyses for different levels of the standard GSA loading (Eq. 1). The load is gradually increased until the structure collapses. The value of the loads as a percentage of the standard GSA loading and the maximum displacement of the column-removed point are collected to construct the capacity curve. This approach was also used by Tsai and Lin [10] and Marchis et al. [13] in order to estimate the ultimate load bearing capacity to progressive collapse of mid-rise RC framed buildings (10-11 stories).

The response of the structural model subjected to corner column removal (case C_3) in terms of vertical displacement of the column removed point for: 0.6, 1.0, 1.2, 1.4, 1.6, 1.7, 1.72 and 1.75 times the GSA standard loading = DL+0.25LL is presented in Fig. 6. The maximum displacements obtained for each level of loading (as a percentage of the GSA standard loading) are collected to construct the capacity curve.

Eight loading steps starting from 0.6 until 1.75 times the standard GSA loading were considered in the analysis. The capacity curve obtained with the nonlinear incremental dynamic analysis is displayed in Fig. 7. The vertical axis represents the percentage of the standard GSA loading and the horizontal axis represents the vertical displacement of the column removed point. It is shown that the structure is capable of sustaining a maximum load of 1.72 times the standard GSA loading before the collapse initiation.

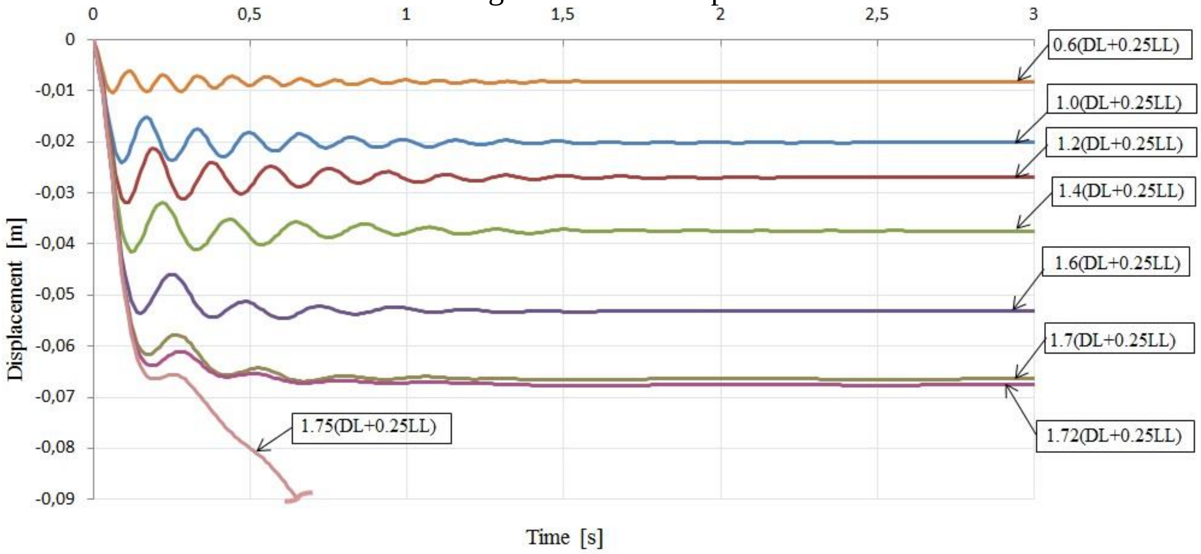


Fig. 6: Time-displacement curves for column removed point for different levels of loading

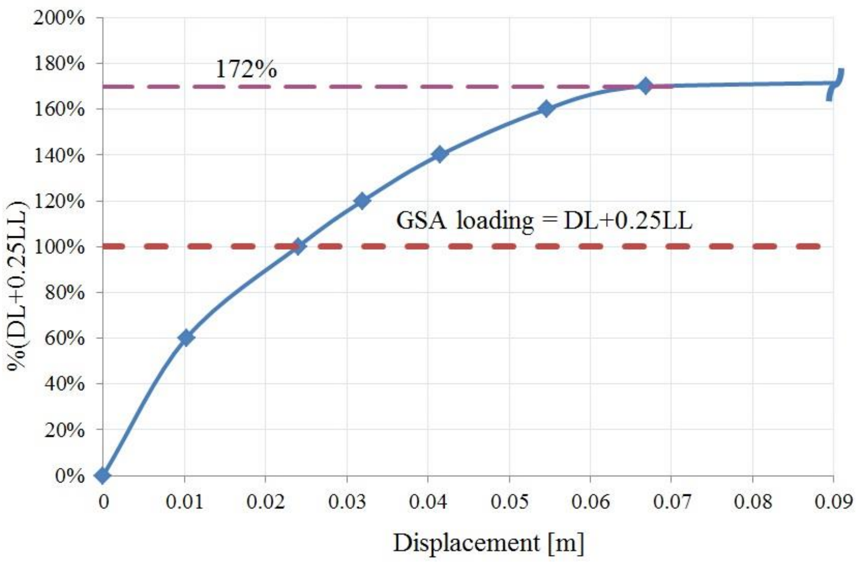


Fig. 7: Load-displacement curve obtained with the nonlinear incremental dynamic analysis

4.4 Failure mode of the AEM Model

Based on the capacity curve illustrated in Fig. 7 it was shown that the structural model can resist for a maximum load of 1.72 times the standard GSA loading. This means that under a higher load –175% (DL+0.25LL) – the structure will fail through progressive collapse.

The observed behaviour of the model during the numerical simulation is provided in the following. The first concrete cracks in tension appear in the beams adjacent to the removed column when the vertical displacement of the column removed point is δ = 4 mm (Fig. 8a). Then, the transverse reinforcement (stirrups) from these beams starts to yield at δ = 2.2 cm. As illustrated in Fig. 8b, some stirrups rupture when δ = 2.8 cm. As the displacement increases, more stirrups fail (Fig. 8c) from all the critical beams (associated to the structural bays adjacent to the removed column). The catenary mechanism could not be develop and thus, the 13-storey building, designed 40 years ago is expected to fail in shear (Fig. 8d) under the gravity load 1.75(DL+0.25LL). This is due to the fact that certain provisions provided by the old codes are much more permissive than the current ones. While the old seismic code P13-70 [21] admits the use of the concrete type B250 (equivalent to C16/20) and reinforcement type OB38 (equivalent to S255), the current code SR EN 1998-1-1:2004 [5] specifies a minimum concrete class C20/25 and the use of steel for reinforcement with the characteristic yield strength (f_{yk}) between 400 and 600 N/mm^2 in the critical regions of seismically designed elements. Also, the distance between stirrups is much more limited: the old code [21] specifies the use of the smallest value from {h_{beam}/3, 15d, 300 mm} and the current code [5] recommends the use of the smallest value from {h_{beam}/4, 6d, 175 mm}. Thus, the shear capacity of a beam calculated using the provisions of the current code (stirrups Φ8/100 mm) is much higher than the capacity calculated using the provisions of the old code (stirrups Φ6/200 mm). Consequently, the risk for progressive collapse might be much lower if the building would be designed according to the current code.

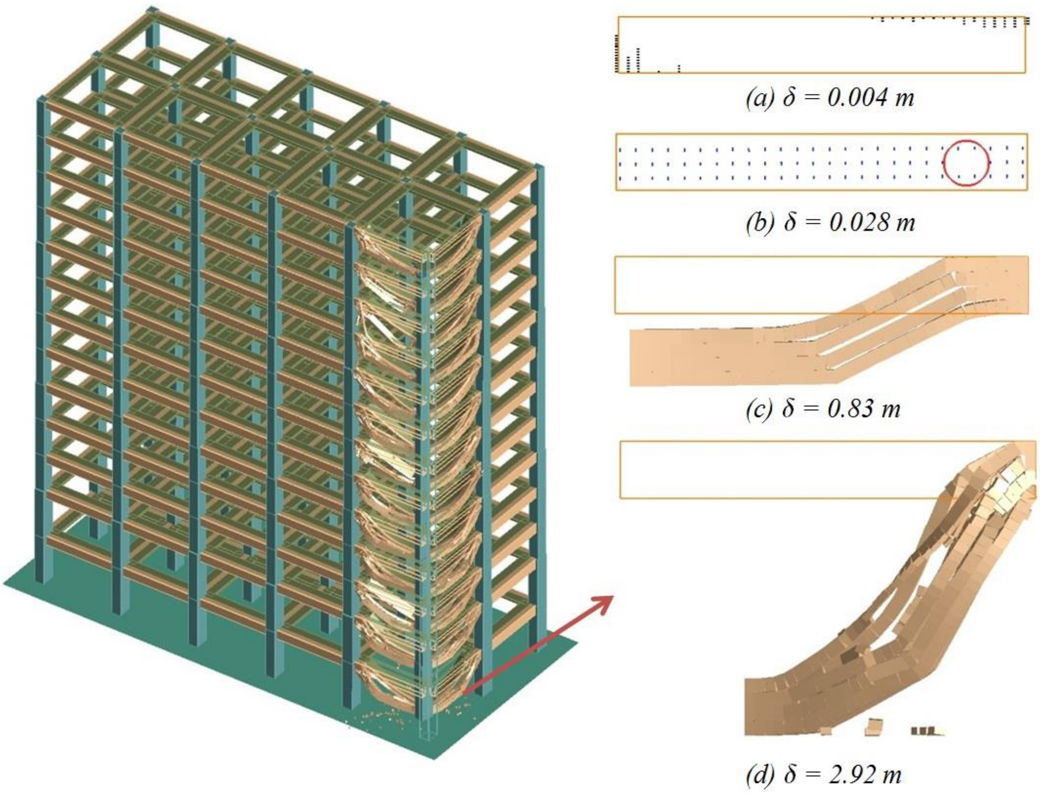


Fig. 8: Progressive collapse of the AEM model under 1.75(DL+0.25LL): (a) concrete cracks in tension; (b), (c) rupture of the stirrups; (d) beam failure

5 CONCLUSIONS

In this study the progressive collapse resistance of an old and representative 13-storey RC framed structure was investigated. The building located in a high seismic area from Romania (Brăila, with k_s = 0.05) was designed following the provisions of the old codes: P13-70 [21] and STAS 8000-67 [26]. A nonlinear dynamic "time-history" analysis is carried out first for the structural model subjected to corner column removal in order to establish the risk for progressive collapse under the standard GSA loading. A nonlinear incremental dynamic analysis is conducted next in order to determine with the maximum accuracy the ultimate load bearing capacity to progressive collapse of the building. Based on the results obtained herein, the following conclusions can be drawn:

- The behaviour of the structural model numerically tested using the Extreme Loading® for Structures software shows very good agreement with the behaviour of the planar frame experimentally tested by Yi et al. [17]. This means that the Applied Element Method is an accurate method which can predict the progressive collapse behaviour even in the large displacement range (catenary effect).
- Based on the results provided by the nonlinear dynamic procedure, it is shown that the existing building, designed 40 years ago is not expected to fail through progressive collapse under the standard GSA loading = DL+0.25LL, when subjected to corner column removal.
- The capacity curve obtained with the nonlinear incremental dynamic analysis indicates that the structure is capable of sustaining a maximum load of 1.72 times the standard GSA loading. This means that if the destination of the building would be changed and the supplementary gravity loads would be above this value of loading, the structure will collapse.
- Recent experimental [14-16] and numerical [10-13] studies had shown that the collapse of RC framed structures, in general, is governed by the flexural failure mode of beam elements. The results provided herein indicate that the structures designed in the 70's are vulnerable to fail in shear due to the seismic design provisions which are much more permissive in the old codes (P13-70 [21], STAS 8000-67 [26]) with respect to those from the current ones (P100-1/2013 [7], EC-8 [5]).

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