
THE RISK FOR PROGRESSIVE COLLAPSE OF RC FRAME STRUCTURES LOCATED IN SEISMIC AREAS IN ROMANIA

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ABSTRACT

In this study the risk for progressive collapse of RC framed buildings located in seismic areas in Romania is investigated. Structures with three, six and ten stories, each in turn are designed for three distinct seismic zones: low ($a_g=0.08g$), moderate ($a_g=0.16g$) and high ($a_g=0.24g$). Using the provisions provided by the GSA (2003) Guidelines, a nonlinear dynamic analysis is carried out first in order to establish the risk for progressive collapse of the models under investigation. It is shown that all the structures are not expected to fail when subjected to abnormal loading (sudden column removal). A nonlinear incremental dynamic analysis is conducted next in order to estimate with the maximum accuracy the ultimate load bearing capacity to progressive collapse of the damaged models. Based on the capacity curves provided by this procedure, it is shown that all the structures are capable of sustaining a higher load than the standard GSA loading before failure. Using these capacity curves, the influence of the seismic design on the progressive collapse resistance is quantified as well.

Keywords: progressive collapse; seismic design; RC framed structures; GSA (2003) Guidelines; nonlinear incremental dynamic analysis.

REZUMAT

In acest studiu se investighează riscul de colaps progresiv al structurilor în cadre din beton armat amplasate în zone seismice din România. Structuri cu trei, șase și zece niveluri, fiecare din ele sunt proiectate, pe rând, pentru trei zone seismice distincte: redusă ($a_g=0.08g$), moderată ($a_g=0.16g$) și înaltă ($a_g=0.24g$). Utilizând prevederile Ghidului GSA (2003), se realizează o analiză dinamică neliniară pentru a stabili riscul de colaps progresiv ale modelelor investigate. Se arată că toate structurile nu vor ceda sub acțiunea încărcărilor accidentale (îndepărtarea instantanee a unui stâlp). Se realizează și o analiză dinamică neliniară incrementală cu scopul de a estima cu acuratețe maximă capacitatea portantă ultimă la colaps progresiv a modelelor avariate. Pe baza curbelor de capacitate obținute cu această metodă, se arată că toate structurile analizate sunt capabile să susțină o încărcare mai mare decât încărcarea GSA înainte de cedare. Utilizând aceste curbe de capacitate, se cuantifică și influența proiectării seismice asupra capacității portante ultime la colaps progresiv a structurilor considerate.

Cuvinte cheie: colaps progresiv; proiectare seismică; structuri în cadre din beton armat; Ghidul GSA (2003); analiza dinamică neliniară incrementală.

1. INTRODUCTION

Progressive collapse phenomenon has been brought into the attention of the engineering community after the partial failure of the Ronan Point Building due to a gas explosion at the 18th floor. The interest in this field has been intensified after the terrorist attacks at the Murrah Federal Building (Oklahoma, 1995) and at the World Trade Center (New York, 2001). Furthermore,

between 1989 and 2000, there were reported 225 cases of collapsed structures from which 54% during the three years [1].

From an analytical point of view, progressive collapse occurs when a structure has its load pattern or boundary conditions changed such that other structural elements are loaded beyond their capacity and fail [2].

The U.S. Agencies, the General Services Administration (GSA) and the Department of Defense (DoD) published in 2003, respectively

in 2005 and 2009, guidelines for progressive collapse analysis of new and existing buildings [3, 4, 5]. The Alternative Path Method has been selected as the basic approach for providing resistance to progressive collapse for structures when subjected to extreme loadings (e.g. explosions, impact by vehicles, terrorist attacks, etc). This method is an independent approach and does not require characterization of the threat causing the loss of a primary structural component.

In order to resist progressive collapse, a structure should be designed with an adequate level of continuity, ductility and redundancy so that alternative load paths could develop over the removed vertical support as a result of abnormal loading. The incorporation of these characteristics which are also found in the seismic design codes (P100/1-2006 [6], ASCE 41-06 [7], Eurocode 8 [8]) will provide a more robust structure and thus will mitigate the risk for progressive collapse.

Recent experimental studies [9, 10, 11] carried out on beam-column subassemblages - as part of RC buildings - were tested until failure (static simulation of column-removal); the inherent ability to better resist progressive collapse of the structural subassemblages designed for higher seismic areas was shown. 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 [12, 13], according to the Taiwanese code [14] or according to the Romanian codes [15, 16, 17].

In this context, the aim of this study is to investigate the risk for progressive collapse of RC framed buildings located in seismic areas in Romania when subjected to abnormal loading. Structures with three, six and ten stories were designed, each in turn, according to the provisions of the seismic code P100/1-2006 [6] for three distinct seismic zones: low seismic area (Cluj-Napoca, where the peak ground acceleration is $a_g=0.08g$), moderate seismic area (Sibiu, $a_g=0.16g$) and high seismic area (Bucharest, $a_g=0.24g$). Therefore,

nine structural models were considered in the progressive collapse analysis.

Using the provisions provided by the GSA (2003) Guidelines [3], each model is analyzed using the nonlinear dynamic procedure for a column-removed condition. In order to determine with the maximum accuracy the ultimate load bearing capacity to progressive collapse of the models under investigation, a nonlinear incremental dynamic analysis is carried out next. Based on the capacity curves provided by this analysis, the influence of the seismic design on the progressive collapse resistance of the structures with three, six and ten stories is quantified as well.

2. INVESTIGATED STRUCTURES

2.1. Design details of the structures

Each designed model has the same structural configuration in plan (Fig.1): two 6.0m bays in the transverse direction and five 6.0m bays in the longitudinal direction. Two parameters were varied: the number of stories (three, six, and ten) and the seismic area (low, moderate and high). This means that nine models were considered for this investigation.

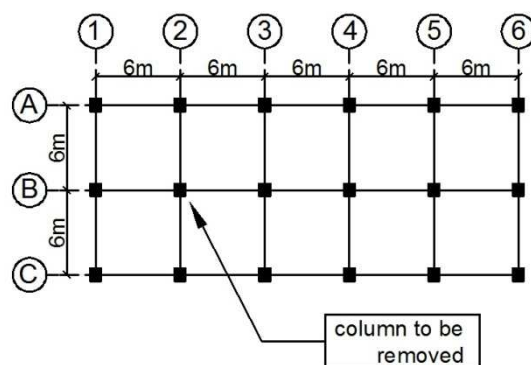


Fig. 1. Structural models

All the structures were designed according to the provisions of the Romanian seismic code P100/1-2006 [6], similar with Eurocode 8 [8], and according to the provisions of the design code for concrete structures SR EN 1992-1-1:2004 [18]. In addition to the self-weight of the structural elements, supplementary dead loads of 2.0 kN/m^2 and live loads (LL) of 2.4 kN/m^2 were considered.

Each RC framed building with three, six and ten stories was designed for a low seismic area (Cluj-Napoca, where the peak ground acceleration is $a_g=0.08g$), for a moderate seismic area (Sibiu, $a_g=0.16g$) and for a high seismic area (Bucharest, $a_g=0.24g$).

The dimensions of the structural components of the structures (not shown here) ranges from 35x35cm to 85x85cm for the columns, respectively from 25x40cm to 30x60cm for the beams. All the models fulfill the lateral displacement demand at the serviceability limit state (SLS) and at the ultimate limit state (ULS). A concrete class C25/30 with the design compressive strength $f_{cd} = 16.67$ MPa and steel type S500 with the design yield strength $f_{yd} = 434.78$ MPa was used.

2.2. Numerical models for progressive collapse analysis

For the progressive collapse analysis, a FEA computer program SAP 2000 is used to model the structures under investigation. Beam elements are modeled as T or L sections to include the effect of the slab acting as a flange in monolithic constructions as recommended by the seismic design codes: P100/1-2006 [6], Eurocode 8 [8], ASCE 41-06 [7]. For simplicity, the effective flange width on each side of the web is taken as three times the slab

thickness. Recent experimental [9, 10, 19] and numerical [1, 13, 14, 16] studies had shown that the collapse of RC framed structures is governed by the flexural failure mode of beam elements. Therefore, only this failure mode is investigated herein and not the shear failure or possible failure of the columns. For the nonlinear dynamic analysis, the plastic hinge model illustrated in Fig. 2 is assigned to beams ends. The moment-hinge properties are based on the seismic design code ASCE 41-06 [7] and adjusted for progressive collapse analysis. The maximum allowable rotation in plastic hinges associated to point C on the $M-\theta_p$ curve (Fig. 2) which corresponds to the “Collapse Prevention” performance level is increased from 0.02rad to 0.035rad as recommended by the GSA (2003) Guidelines [3] for RC frames. The slope from point B to C is taken as 10% of the elastic slope to account for strain hardening; the seismic code ASCE 41-06 [7] indicates that the slope should be taken as a small percentage between 0% and 10%. Point D corresponds to the residual strength ratio of 0.2. Since the GSA (2003) Guidelines [3] does not specify a value for point E as the failure limit, a value of 0.07rad is considered as an average value of the ones (0.04rad÷0.10rad) given by the DoD (2009) Guidelines [5].

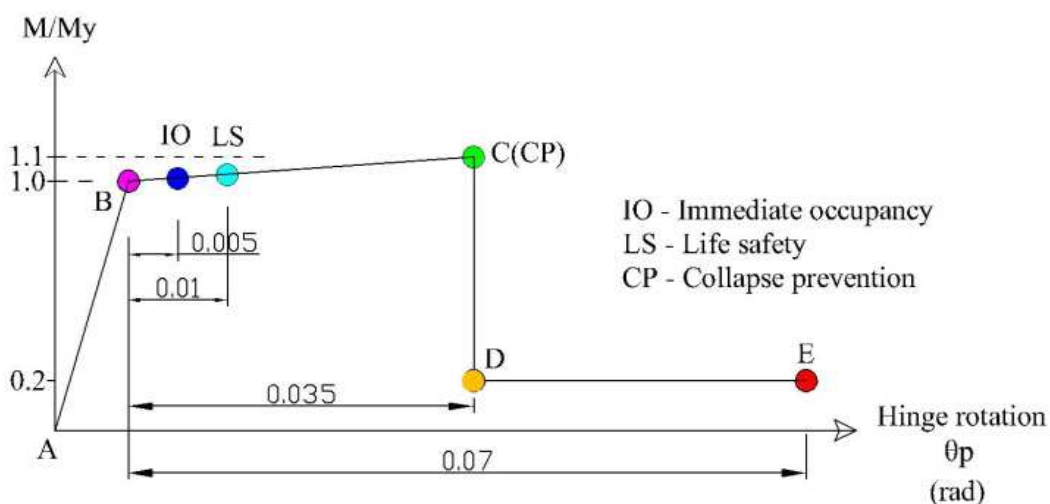


Fig. 2. Plastic hinge model

3. GSA (2003) GUIDELINES

3.1. Damage case

As recommended by the GSA (2003) Guidelines [3], the risk for progressive collapse of a building is assessed considering the sudden 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_4 is considered herein (Fig. 1).

3.2. Loading criteria

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

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

Where DL is the dead load and LL is the live load.

3.3. Acceptance criteria

The acceptance criteria for obtaining a low risk for progressive collapse is related to a rotation limit in plastic hinges of 0.035 rad; this value corresponds to point C (the “Collapse Prevention” performance level) on the moment-rotation curve displayed in Fig. 2.

4. RISK FOR PROGRESSIVE COLLAPSE

4.1. Nonlinear dynamic analysis

When performing a nonlinear dynamic time-history analysis (NDP), the loads combination given in Eq. (1) is applied downward to the undamaged structural model. Then, the interior column is suddenly removed from the structure. The time for the removal is set to $t_r = 5\text{ms}$, a value also adopted by Santafé [20]. As recommended by the DoD (2009) Guidelines [5], 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. Also, a 5% damping ratio is considered in the dynamic analysis. The response of all structural models is observed over a time span $t = 3\text{s}$, similar with Santafé [20] and displayed in Fig. 3. Under the standard GSA loading (Eq. 1), all structural models reach a new static equilibrium after three seconds. The Cluj-3 storey model is the most vulnerable to progressive collapse (Fig. 3a); the maximum displacement of the column removed point measures 4.70cm. Instead, the Bucharest-10 storey model is the less vulnerable to progressive failure (Fig. 3c); the maximum displacement obtained was only 1.26cm. At this step, the rotations in plastic hinges from the critical beam sections are well below the allowable value of 0.035rad as recommended by the GSA (2003) Guidelines [3]; based on the performance levels adopted in Fig. 2, these plastic hinges (not shown here) are classified at most the Immediate Occupancy performance level. Consequently, a low risk for progressive collapse was obtained for all the structural models under investigation.

The influence of the seismic design on the progressive collapse resistance is highlighted as well. For the sudden column removal case under the standard GSA loading (Eq. 1), lower vertical displacements of the column removed point were obtained for the structures (with three, six and ten stories) located in higher seismic areas. This means that the progressive collapse resistance is higher if the same structure, whatever its number of stories, is designed for a higher seismic zone.

4.2. Nonlinear incremental dynamic analysis

The destination of a building could be changed during its lifetime. This assumes that the verdict for progressive collapse risk established in the initial phase of the design might be changed from low to high.

In this context, a nonlinear incremental dynamic analysis is carried out in order to establish the ultimate load bearing capacity to progressive collapse of the structures; thus, the maximum value of the supplementary gravity load (additional to the standard GSA loading)

for which a structure will not fail through progressive collapse when subjected to sudden

column removal will be identified.

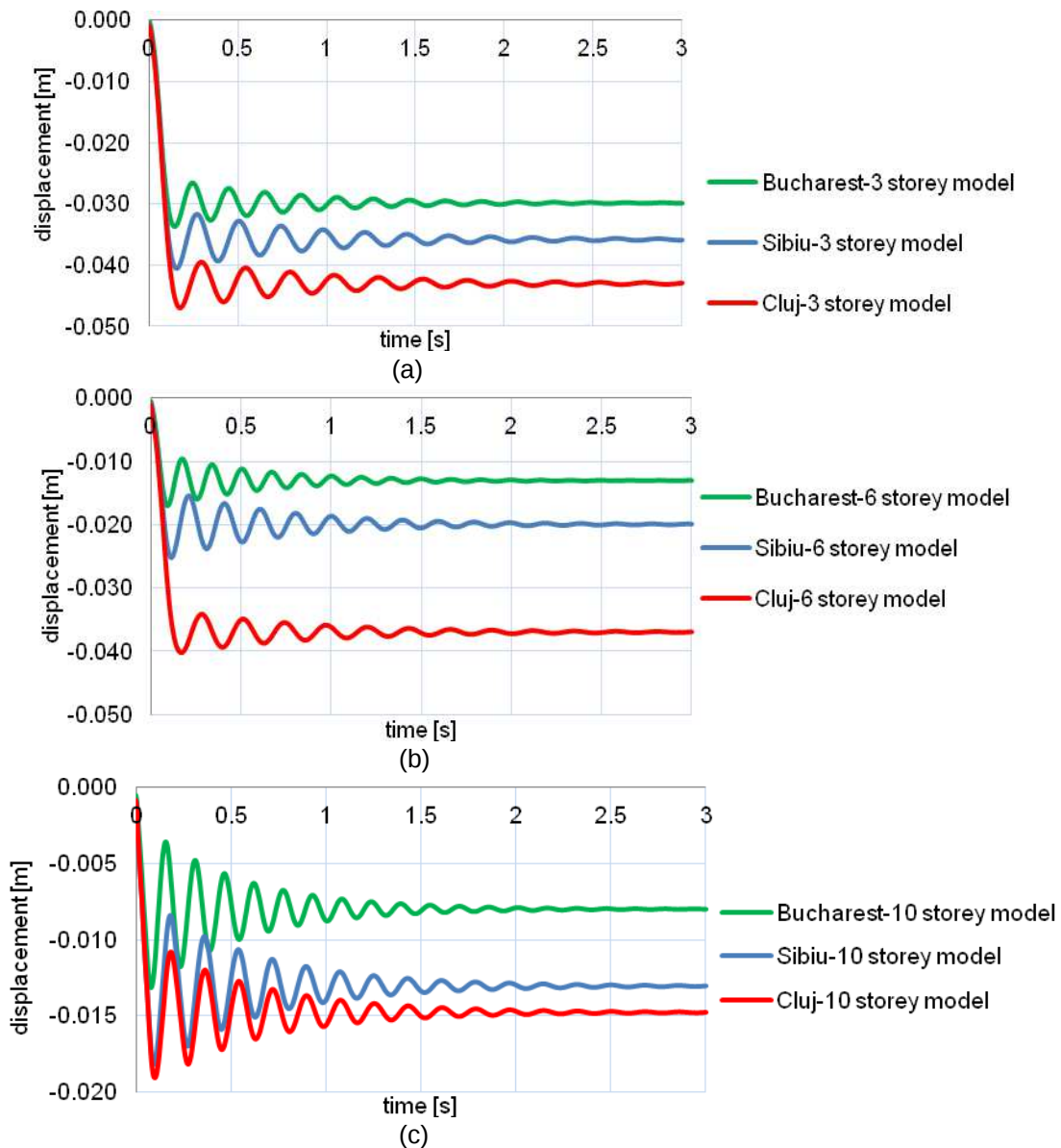


Fig. 3. Time-displacement curves of the column removed point obtained from the nonlinear dynamic analysis for: (a) 3-storey models; (b) 6-storey models; (c) 10-storey models

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 allowable rotations in plastic hinges associated to the Collapse Prevention performance level – Fig. 2 – are exceeded). The value of the loads as a percentage of the GSA loading and the maximum displacement of the column-removed point are collected to construct the capacity curve. This approach was used by

Tsai [14] to estimate the ultimate load bearing capacity to progressive collapse of an 11-storey RC framed building.

The response of the Cluj-3 storey model subjected to column removal (damage case C_4) in terms of vertical displacement of the column-removed point for 1.0, 1.15 and 1.2 times the GSA loading= $DL+0.25LL$ is illustrated in Fig. 4(a). The maximum displacements obtained for each level of loading (as a percentage of the GSA loading) are collected to construct the capacity curve.

Fourteen loading steps starting from 0.4 until 1.29 times the GSA loading were considered for this model. The capacity curve obtained with the nonlinear incremental dynamic analysis is displayed in Fig. 4(b). The vertical axis represents the percentage of the load ($DL+0.25LL$) and the horizontal axis represents the vertical displacement of the

column-removed point; the value of 0.2m corresponds to the allowable plastic rotation of 0.035rad. Based on the capacity curve (Fig. 4(b)), it is shown that the Cluj-3 storey model is capable of sustaining a maximum load equal to 129% of the GSA loading before collapse initiation.

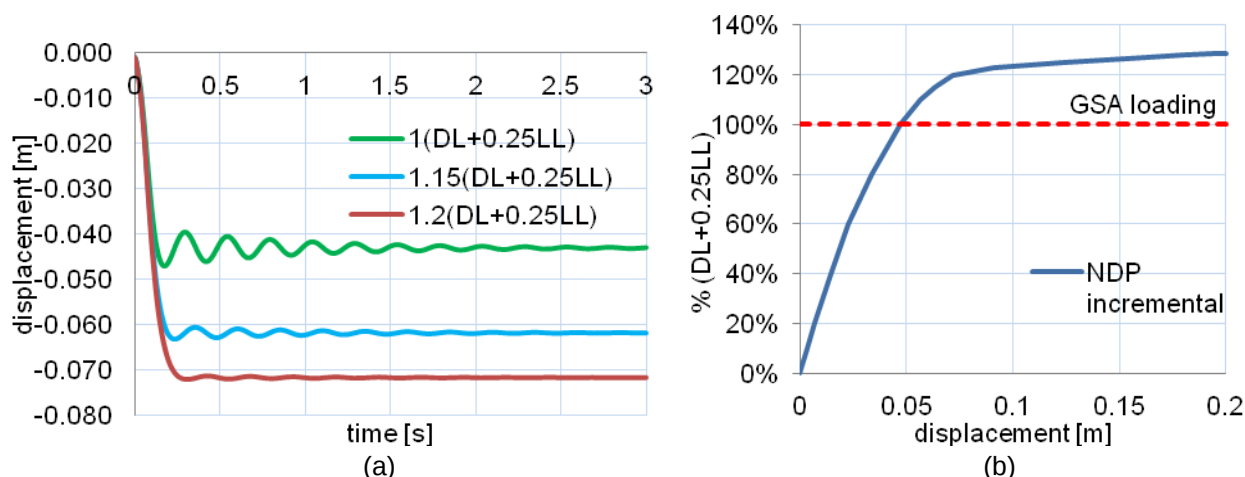


Fig. 4. Dynamic response of the Cluj-3 storey model subjected to column removal: (a) time-displacement curves for different levels of loading; (b) the capacity curve obtained with NDP incremental

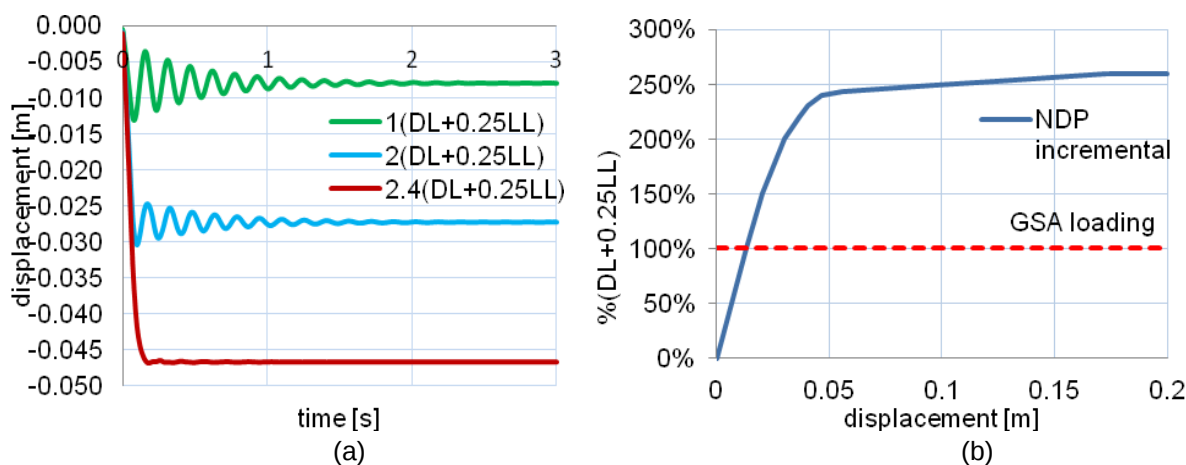


Fig. 5. Dynamic response of the Bucharest-10 storey model subjected to column removal: (a) time-displacement curves for different levels of loading; (b) the capacity curve obtained with NDP incremental

The response of the Bucharest-10 storey model subjected to column removal in terms of vertical displacement of the column-removed point for 1.0, 2.0 and 2.4 times the GSA loading= $DL+0.25LL$ is displayed in Fig. 5(a). The capacity curve (Fig. 5(b)) provided by the nonlinear incremental dynamic analysis indicates that the Bucharest-10 storey model is capable of sustaining a much higher load (2.6 times the GSA loading) than the Cluj-3 storey

model. The dynamic response for the rest of the analyzed models subjected to interior column removal is not displayed herein; only the capacity curves obtained with the nonlinear incremental dynamic procedure are provided in the following section in order to quantify the influence of the seismic design on the ultimate load bearing capacity to progressive collapse of the structures.

4.3. Influence of the seismic area

In this section, the influence of the seismic area on the progressive collapse resistance of RC framed structures with three, six and ten stories is quantified. The comparative study

was conducted based on the capacity curves provided by the nonlinear incremental dynamic analysis. The results are displayed in Fig. 6.

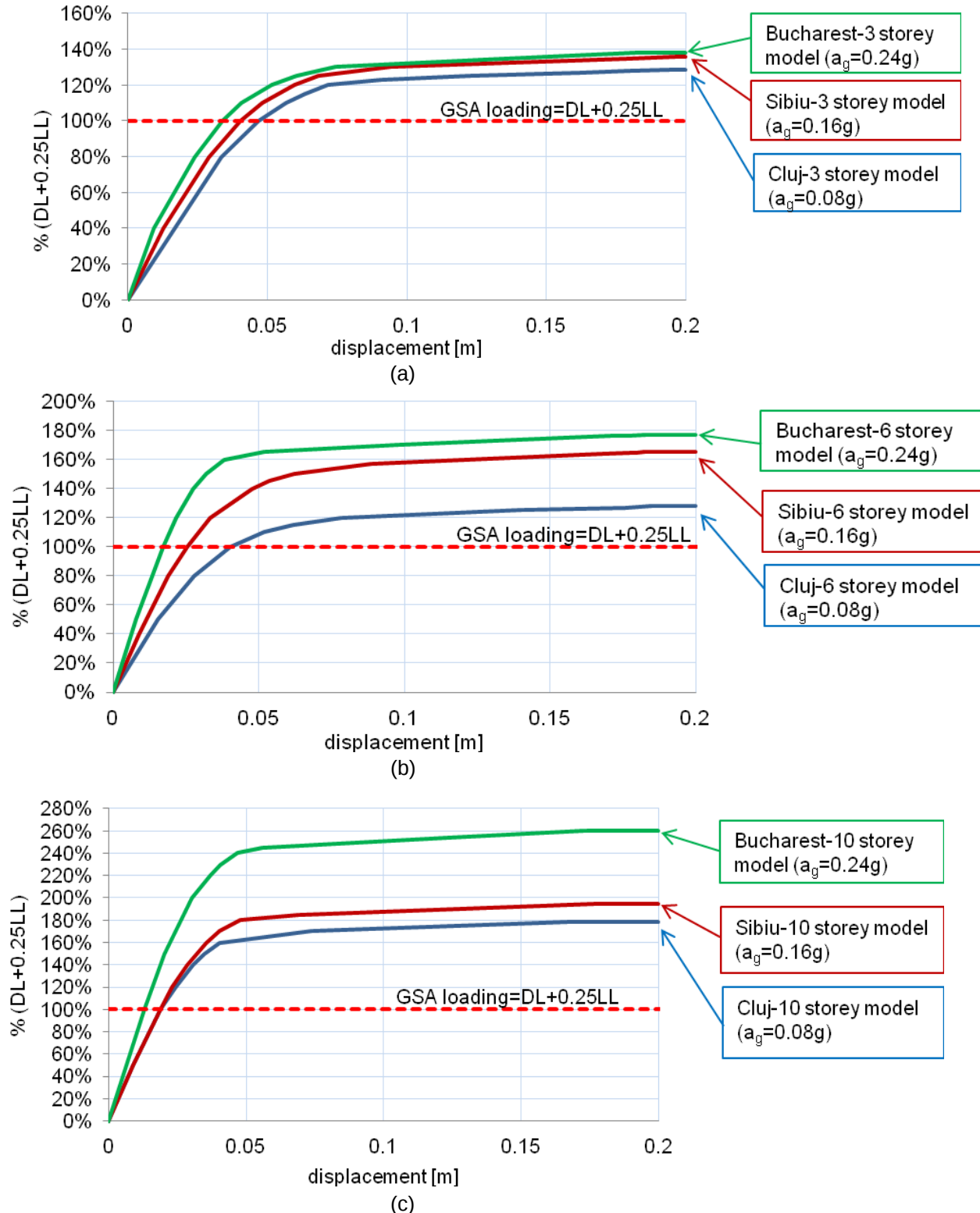


Fig. 6. The influence of the seismic area on the progressive collapse resistance for: (a) three-storey structures; (b) six-storey structures; (c) ten-storey structures subjected to interior column removal

The horizontal axis represents the vertical displacement of the column-removed point and the value of 0.2m corresponds to the allowable rotation of 0.035rad in plastic hinges. The vertical axis represents the percentage of the standard GSA loading= $DL+0.25LL$ applied in the dynamic analysis. It should be underlined that all the investigated models can resist for a higher load than the GSA loading (marked in Fig. 6 with red dotted line) before collapse.

In the case of the three-storey structures (Fig. 6(a)) the influence of the design for a higher seismic area exists, but is insignificant. The ultimate load bearing capacity to progressive collapse of the structure designed for a high seismic area ($a_g=0.24g$) increases insignificantly (with 1.4%) in regard to the one of the structure designed for a moderate seismic area ($a_g=0.16g$), respectively increases slightly (with 7.8%) in regard to the one of the structure designed for a low seismic area ($a_g=0.08g$).

The beneficial influence of the design for a higher seismic area on the progressive collapse resistance in the case of six-storey structures (Fig. 6(b)) is more obvious than in the case of low-rise structures (three stories). The ultimate load bearing capacity of the structure designed for a high seismic zone ($a_g=0.24g$) increases with 7.3% in regard to the one of the structure designed for a moderate seismic zone ($a_g=0.16g$), respectively with 38% in regard to the one of the structure designed for a low seismic zone ($a_g=0.08g$).

The beneficial influence of the design for a higher seismic area on the progressive collapse resistance in the case of ten-storey structures (Fig. 6(c)) is much more pronounced than in the case of six-storey structures or in the case of low-rise structures (three stories). The ultimate load bearing capacity to progressive collapse of the structure designed for a high seismic area ($a_g=0.24g$) increases with 33% in regard to the one of the structure designed for a moderate seismic area ($a_g=0.16g$), respectively with 45% in regard to the one of the structure designed for a low seismic area ($a_g=0.08g$).

5. CONCLUSIONS

In this study, the risk for progressive collapse of RC framed buildings with three, six and ten stories, each one designed for low ($a_g=0.08g$), moderate ($a_g=0.16g$) and high ($a_g=0.24g$) seismic area according to the provisions of the Romanian seismic code P100/1-2006 [6], similar with Eurocode 8 [8] was investigated. A nonlinear dynamic “time-history” analysis was carried out first for all the nine structural models subjected to interior column removal in order to assess the risk for progressive collapse under the standard GSA loading. A nonlinear incremental dynamic analysis was conducted next in order to establish with the maximum accuracy the ultimate load bearing capacity to progressive collapse of the structures. The results obtained herein lead to the following conclusions:

- Based on the results provided by the nonlinear dynamic analysis procedure it is shown that, under the standard GSA loading= $DL+0.25LL$ all the structures under investigation designed according to the Romanian seismic code P100/1-2006 [6] are not expected to fail through progressive collapse when subjected to sudden column removal.
- The capacity curves obtained with the nonlinear incremental dynamic analysis indicate that all the structures (subjected to column failure) are capable of resisting for a higher load than the standard GSA loading before collapse. At the lowest limit is the Cluj-3 storey model (located in a low seismic area – $a_g=0.08g$) which is capable of sustaining a maximum load of 1.28 times the GSA loading; at the highest limit is the Bucharest-10 storey model (located in a high seismic area – $a_g=0.24g$) which can sustain a maximum load of 2.6 times the GSA loading.
- Based on these capacity curves, it is shown the beneficial influence of the seismic design on the progressive collapse resistance. In the case of three-storey structures this influence is reduced; in regard to the structure located in a low seismic area (Cluj-3 storey model), the ultimate load bearing capacity to

progressive collapse of the structures from moderate (Sibiu-3 storey model) and high (Bucharest-3 storey model) seismic area increases with 6.3% and 7.8%. Instead, this influence is much more pronounced in the case of six-storey and ten-storey structures; in regard to the structures from low seismic area, the ultimate load bearing capacity of the models from moderate and high seismic areas significantly increases with 29% and 38% in the case of six-storey buildings, respectively with 9% and 45% in the case of ten-storey buildings.

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