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THE INFLUENCE OF THE SEISMIC DESIGN ON THE PROGRESSIVE COLLAPSE RESISTANCE OF MID-RISE RC FRAMED STRUCTURES

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Abstract

In this study the influence of the seismic design to better resist progressive collapse of mid-rise RC framed structures is investigated. A ten-storey building is designed for three distinct seismic areas (low – a_g=0.08g, moderate – a_g=0.16g and high – a_g=0.24g) from Romania. Using the GSA criteria, a nonlinear static analysis is carried out first in order to estimate the progressive collapse resistance of the structures subjected to interior column removal. It is shown that only the model designed for high seismic area has a low potential for progressive collapse; the capacity curves provided by the nonlinear static analysis indicate that the models designed for low and moderate seismic areas collapses before the standard GSA loading is attained, unlike the model designed for a high seismic area which can sustain this load without failure. Consequently, a nonlinear dynamic analysis is carried out to validate the results' accuracy obtained with the static procedure. The results indicate that all the models have a low potential for progressive collapse and thus, the dynamic increase factor (DIF) of 2.0 recommended by the GSA (2003) Guidelines can be overestimated. A nonlinear incremental dynamic analysis is carried out in order to estimate with maximum accuracy the ultimate load bearing capacity of the damaged structures before collapse. The variation of DIF dependent on the level of loading is highlighted as well.

Rezumat

În cadrul acestui studiu se investighează influența proiectării seismice asupra potențialului de colaps progresiv al structurilor în cadre din beton armat de înălțime medie. O structură cu zece niveluri este proiectată pentru trei zone cu seismicitate redusă (a_g=0.08g), moderată (a_g=0.16g) și înaltă (a_g=0.24g) din România. Utilizând criteriile GSA, o analiza statică neliniară se realizează întâi pentru a stabili potențialul de colaps progresiv al structurilor supuse îndepărtării unui stâlp interior. Doar modelul proiectat pentru o zonă cu seismicitate înaltă are un potențial redus de cedare; curbele de capacitate obținute cu analiza statică neliniară indică faptul că modelele proiectate pentru zona cu seismicitate redusă și moderată cedează înainte ca încărcarea standard GSA să fie aplicată, spre deosebire de modelul structural proiectat pentru o zonă cu seismicitate înaltă care poate susține această încărcare fără a ceda. În consecință, se utilizează o analiză dinamică neliniară pentru a valida rezultatele obținute cu procedura statică. Rezultatele indică faptul că toate structurile au un potențial redus de colaps progresiv și astfel, valoarea de 2.0 a factorului de amplificare dinamică (DIF) recomandată de Ghidul GSA (2003) poate fi supraestimată. O analiză dinamică neliniară incrementală se realizează astfel pentru a estima cu

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acuratețe maximă capacitatea portantă ultimă a structurilor avariate înainte de colaps. De asemenea, se evidențiază variația lui DIF în funcție de nivelul încărcării aplicat structurilor.

Keywords: progressive collapse, GSA criteria, seismic design, RC structure, nonlinear incremental dynamic analysis, dynamic increase factor.

1. Introduction

The progressive collapse is defined as a "situation where a local failure of primary structural components leads to the collapse of adjoining members which, in turn, leads to additional collapse, the total damage being disproportionate with the original cause" [1]. In the literature, this term is also found as disproportionate collapse. The progressive collapse issue has been brought into the attention of the engineering community after the partial collapse of the Ronan Point Building (England, 1968) and of the Murrah Federal Building (Oklahoma, U.S.A, 1995); the interest in this field has been intensified after the terrorist attacks at the World Trade Center (New York, U.S.A, 2001), where the two towers have been completely destroyed. An interesting detail is that, between 1962 and 1971 in the United States and Canada, there were reported 605 cases of total collapse, from which 35% (212 cases) failed through progressive collapse; in addition, between 1989 and 2000, there were reported 225 cases of building failures, from which 54 % during the three years (1998-2000) [2].

This type of structural failure (progressive collapse) is the result of abnormal loading (e.g. gas explosions, bombs, impact by vehicles, etc.). Taking into account that these types of loads are extremely rare events that can occur during the lifetime of a building, it is more appropriate (from an economic point of view) to mitigate the risk for progressive collapse than to especially design them to resist this type of loads. In this context, the U.S. General Service Administration and the U.S. Department of Defense published in 2000 and 2003, respectively 2005 and 2009 guidelines for progressive collapse analysis of new and existing buildings. The Alternative Path Method (APM) has been selected by both agencies as the basic approach for providing resistance to progressive collapse; this means that a structure must accommodate the initial damage and be capable of developing alternative load paths to sustain the redistributed loads when a vertical support is instantaneously removed as a result of abnormal loading. In order to resist progressive collapse the structures should be designed with an adequate level of continuity, ductility and redundancy; the incorporation of these characteristics will provide a more robust structure and thus will contribute for mitigate the risk for progressive collapse.

Experimental studies [3, 4, 5] 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 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 [6, 7], according to the Taiwanese code [8] or according to the Romanian codes [9, 10, 11, 12].

Therefore, the aim of this study is to quantify the influence of the seismic design on the progressive collapse resistance of a ten-storey RC framed structure designed for three distinct seismic zones according to the provisions of the Romanian seismic code P100/1-2006 [13] in general similar with Eurocode 8 [14]. The first model is designed for a low seismic area (Cluj-Napoca, where the peak ground acceleration is a_g=0.08g), the second model is designed for a moderate seismic area (Sibiu, with a_g=0.16g) and the third one is designed for a high seismic area (Bucharest, with a_g=0.24g). Based on the provisions specified in the GSA (2003) Guidelines [1], each model is analyzed using the nonlinear static (NSP) and dynamic (NDP) procedures for a column-removed condition. In order to determine with maximum accuracy the ultimate load-bearing capacity before collapse of the structures, a nonlinear incremental dynamic analysis is carried out. The GSA (2003) Guidelines [1] recommend the use of a dynamic increase factor (DIF) of 2.0 in the nonlinear static analysis to account – in a simplified manner – for the dynamic effect that occurs when a vertical support is suddenly removed from the structure. Based on the capacity curves obtained with the nonlinear static and nonlinear incremental dynamic analyses, the variation of DIF for different levels of loading (as a percentage of the GSA loading) is highlighted; the influence of the seismic design on the progressive collapse resistance of the analyzed models is quantified as well.

2. Building under investigation

2.1 Design details of the buildings

The ten-storey RC framed buildings have the same structural configuration as illustrated in Fig. 1. The structures consist of two 6.0 m bays in the transverse direction and five 6.0 m bays in the longitudinal direction. The storey height is 2.75 m except for the first two stories where the storey height is 2.75 m. The three structures are designed according to the provisions of the Romanian seismic code P100/1-2006 [13], similar with Eurocode 8 [14], and according to the provisions of the design code for concrete structures SR EN 1992-1-1:2004 [15]. 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 are considered. The first structure is designed for a low seismic area (Cluj-model) where the peak ground acceleration is a_g = 0.08g. The second one is designed for a moderate seismic area (Sibiu-model) where a_g = 0.16g and the third structure is designed for a high seismic area (Bucharest-model) where a_g = 0.24g.

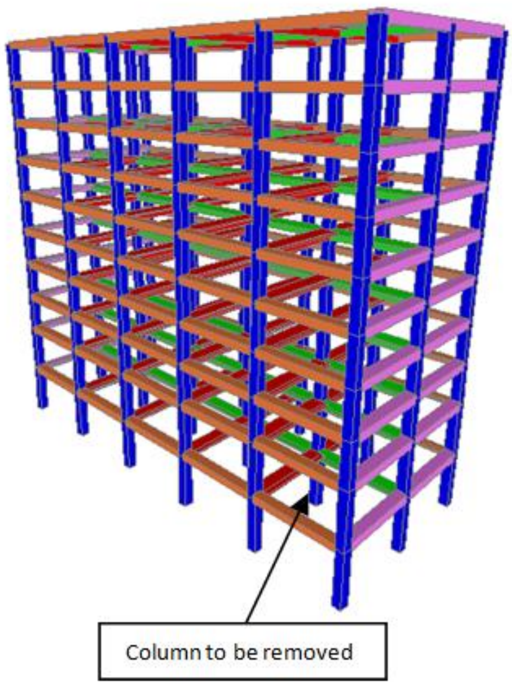


Figure 1. Building model.

The dimensions of the structural elements are displayed in Table 1. The Bucharest-model has larger columns and beams in order to fulfill the allowable lateral displacement under a higher seismic loading as well as to obtain an optimal reinforcement ratio in beams. 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 is used.

Table 1. Dimensions of structural elements

Model	Level	Columns [cm]	Transverse beams [cm]	Longitudinal beams [cm]
Cluj-model	1-10	60x60	25x55	25x50
Sibiu-model	1-10	60x60	25x55	25x50
Bucharest-model	1-4	85x85	30x60	30x60
	5-7	85x75	30x55	30x55
	8-10	75x75	25x50	25x50

2.2 Computational 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 [13], Eurocode 8 [14], ASCE 41-06 [16]. For simplicity, the effective flange width on each side of the web is taken as three times the slab thickness. Recent experimental studies [3, 4, 17] 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 analyses, plastic hinge model as illustrated in Fig. 2 is assigned to beams ends. The moment-hinge properties are based on the seismic design code ASCE 41-06 [16] and adjusted for progressive collapse analysis. The maximum allowable rotation in plastic hinges associated to point C on the M-θ_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 [1] 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 [16] 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 [1] does not specify a value for point E as the failure limit, a value of 0.07 rad is considered as an average value of the ones (0.04rad÷0.10rad) given by the DoD (2009) Guidelines [18].

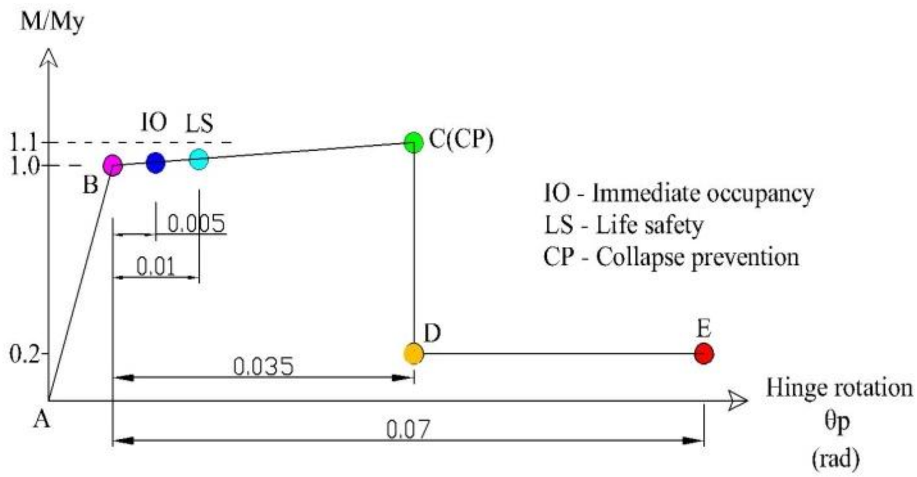


Figure 2. Plastic hinge model.

3. Criteria for progressive collapse analysis

As recommended by the GSA (2003) Guidelines [1], the potential for progressive collapse of a building is assess 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_4 is considered herein (Fig. 1). When performing a static analysis, the following load combination is applied downward to the damaged structure:

$$\text{Load}^{\text{static}} = 2(\text{DL}+0.25\text{LL}) \tag{1}$$

Where DL is the dead load and LL is the live load. The load combination is multiplied by a dynamic increase factor (DIF) of 2.0 to account, in a simplified manner, the dynamic effect that occurs when the vertical support is instantaneously removed from the structure.
For the dynamic analysis, DIF is not considered, as follows:

$$\text{Load}^{\text{dynamic}} = \text{DL}+0.25\text{LL} \tag{2}$$

When performing a nonlinear analysis, the acceptance criteria for obtaining a low potential 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. Progressive Collapse Resistance

4.1 Nonlinear Static “Push-Down” Analysis

In order to estimate the progressive collapse resistance of the structural models under the standard GSA loading (Eq.1) a nonlinear static “push-down” analysis (NSP) is conducted first for the interior column-removal condition. Moment plastic hinge type as given in Fig. 2, are assigned to beams ends. The capacity curves provided by the NS analysis (using the displacement-controlled method) is displayed in Fig. 3. The vertical axis represents the percentage of the standard GSA loading (Eq. 1) applied and the horizontal axis represents the displacement of the node associated to the removed column. The failure limit is identified at 89%, 97% and 130% for the Cluj-model, Sibiu-model, respectively for the Bucharest-model; at this level of loading, plastic hinges from critical beam sections are classified as the “Collapse prevention” performance level as illustrated in Fig. 2. In other words, the Cluj and Sibiu models collapses before the GSA loading=2(DL+0.25LL) is attained, where DL=self-weight+2kN/m^2 and LL=2.4kN/m^2 - marked in Fig. 3 with red dashed line; instead, the Bucharest-model can resist to 1.3 times the GSA loading before failure. These results are similar with those obtained by Tsai [8] for an 11-storey RC structure designed for a high seismic area as well; for the same damage case C_4, the failure limit was identified at approximately 135% of the GSA loading. A much higher progressive collapse resistance obtained under the standard GSA loading for the Bucharest-model, designed for a higher seismic area in regard to the Cluj and Sibiu models has to be underlined (an increase with 46%, respectively 34%). In the previous paper [12] it was shown that the Bucharest and Cluj models can sustain a higher load when subjected to the damage case C_1 (short side column removal) – 139%, respectively 95% of the GSA loading.

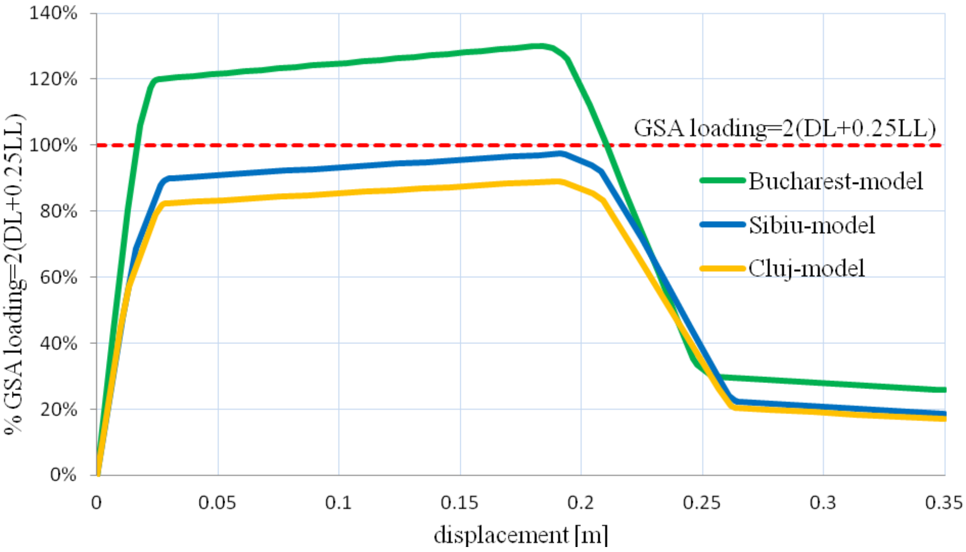


Figure 3. Load-displacement curves obtained from the nonlinear static analysis.

Recent studies [8, 2, 19, 20] have shown that the dynamic increase factor (DIF) of 2.0 provided by the GSA (2003) Guidelines [1] and considered herein in the nonlinear static analysis to account for the dynamic effect that occurs when a column is suddenly removed from the structure, can be overestimated. Therefore, a nonlinear dynamic "time-history" analysis is conducted to verify the results' accuracy obtained with the NS procedure and thus to determine the real magnitude of DIF.

4.2 Nonlinear Dynamic "Time-History" Analysis

When performing a nonlinear dynamic time-history analysis (NDP), the dynamic increase factor (DIF) is not considered. The loads combination given in Eq. (2) 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 tr = 5ms, a value adopted also by Santafé et al. [21]. As recommended by the DoD (2009) Guidelines [18], 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 = 0.18s for the Cluj and Sibiu models, respectively T = 0.15s for the Bucharest-model). A 5% damping ratio is considered in the dynamic analysis. The response of both structural models is observed over a time span t = 3s, similar with Santafé et al. [21] and displayed in Fig. 4. After three seconds both structural models reach a new static equilibrium. The maximum displacement obtained is 1.19cm for the Cluj and Sibiu models, respectively 1.13cm for the Bucharest-model. At this step, the plastic hinges occur (not shown here) in 40 critical beam sections out of a total of 80 possible for the Cluj and Sibiu models, respectively in only 13 critical beam sections for the Bucharest-model. The rotations in plastic hinges are well below the allowable value of 0.035rad as recommended by the GSA (2003) Guidelines [1]. Based on the performance adopted in Fig. 2, these plastic hinges are below the "Immediate occupancy" performance level. Consequently, all the structural models will not fail (low potential for progressive collapse) when subjected to interior column removal. The results obtained herein are in contradiction with those obtained with the NS procedure for the Cluj and Sibiu models (high potential). This clearly indicates that the dynamic increase factor of 2.0 recommended by the GSA (2003) Guidelines [1] is overestimated and thus, the progressive collapse resistance of the structural models (under the standard GSA loading) determined with the NS analysis is underestimated. Consequently, a nonlinear incremental dynamic analysis is carried out in the following section in order to estimate with the maximum accuracy the progressive collapse resistance of the models (under the ultimate dynamic loading).

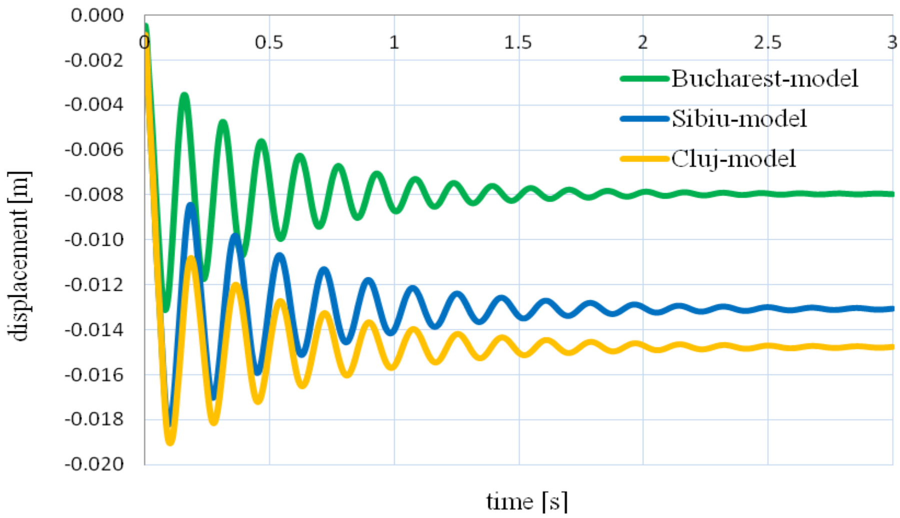


Figure 4. Time-displacement curves of the column-removed point obtained from the nonlinear dynamic analysis.

4.3 Nonlinear incremental dynamic analysis

The nonlinear incremental dynamic analysis is the most promising method to establish the ultimate load bearing capacity to progressive collapse of the structures. This assumes to conduct a series of nonlinear dynamic "time-history" analyses for different levels of the GSA loading=DL+0.25LL.
The load is gradually increased until the structural model collapses (the allowable rotations of 0.035rad in plastic hinges associated to the Collapse Prevention performance level 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 in order to construct the capacity curve. The nonlinear incremental dynamic analysis can be time-consuming dependent on the size of the FEA model, respectively of the number of loading steps considered to construct the curve. This approach was used by Tsai and Lin [8] to establish the ultimate load bearing capacity of an 11-storey RC framed structure.

The response of the three models when subjected to the damage case C_4 in terms of vertical displacement of the column removed point for different levels of the standard GSA loading is illustrated in Fig. 5. The maximum displacements obtained for each level of loading (as a percentage of the GSA loading) are collected to construct the capacity curve. Twelve loading steps starting from 0.5 times the GSA loading were considered for all the models.

The capacity curve obtained with the nonlinear incremental dynamic analysis for each structural model is marked in Fig. 6 with red dotted line in regard with the one obtained with the nonlinear static procedure (NDP) marked with continuous blue line. 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 results displayed in Fig. 6 indicate that the value of the dynamic increase factor (DIF) is close to 1.0 as the structural response is in a significantly yielding phase. A discussion about the variation of DIF dependent on the level of loading (applied on the structure) is performed in the following section.

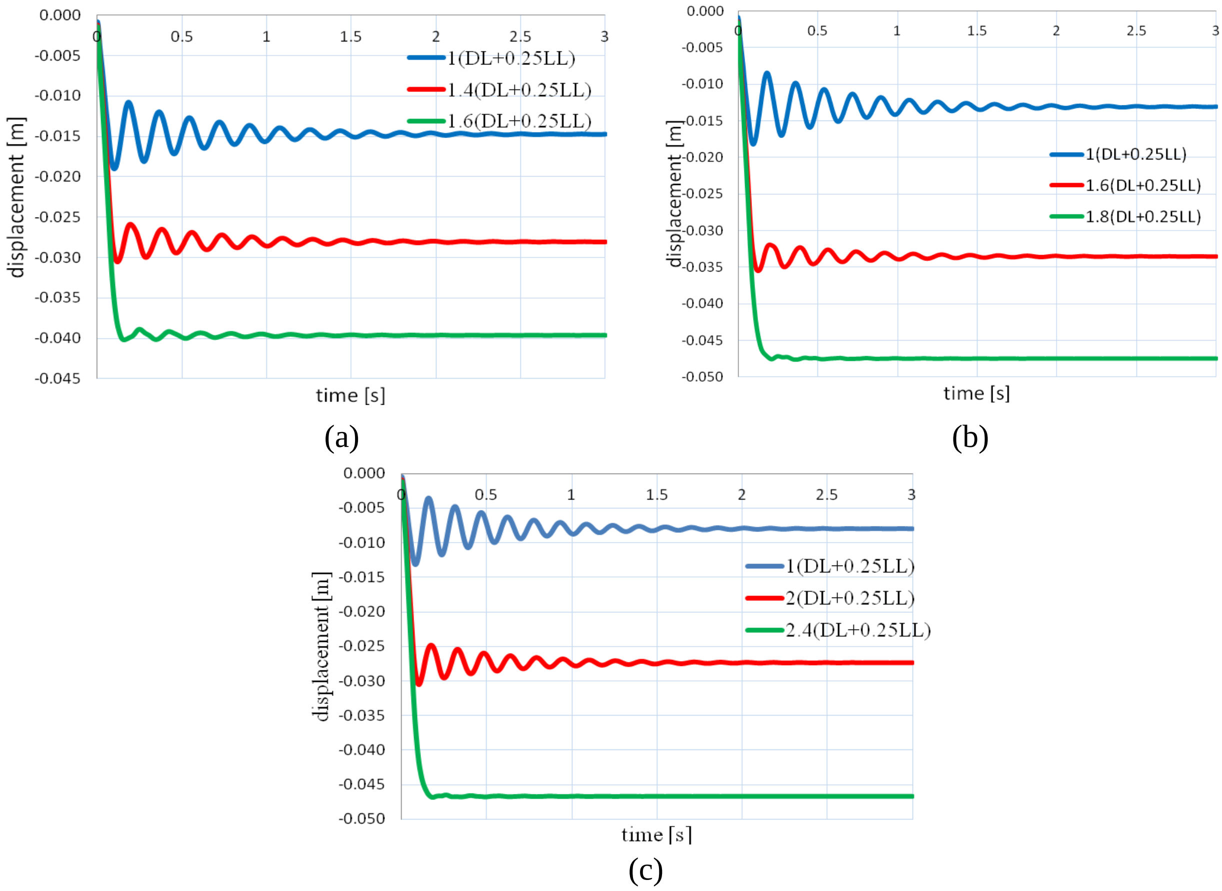
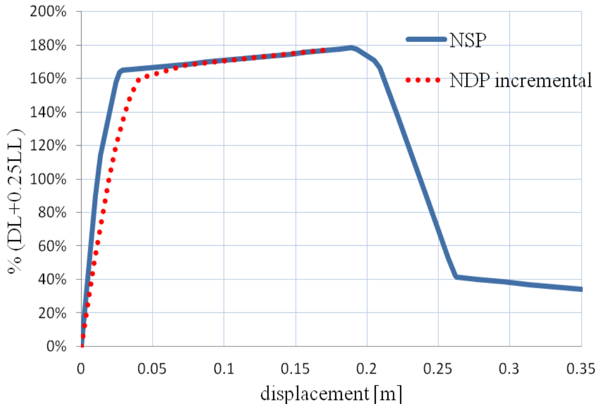


Figure 5. Time-displacement curves of the column-removed point for the models subjected to damage case C_4 for different levels of loading: a) Cluj-model; b) Sibiu-model; c) Bucharest-model.

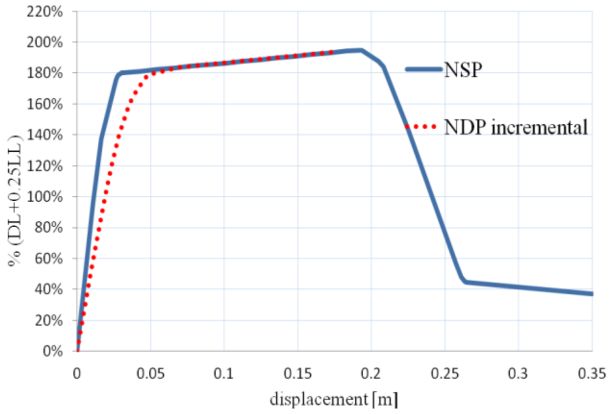
4.4 Dynamic increase factor

The results provided by the nonlinear dynamic analysis clearly indicate that, by considering a DIF = 2.0 as recommended by the GSA (2003) Guidelines [1], the nonlinear static procedure underestimates the progressive collapse resistance of the analyzed structures. The dynamic increase factor (DIF) that allows a nonlinear static solution to approximate a nonlinear dynamic solution should be less than 2.0, issue underlined by DoD (2009) Guidelines [18]. DIF may be defined as the ratio of the static load and the dynamic load under the same displacement demand. This definition had been adopted by Tsai and Lin [8] as well.

Therefore, based on the results provided in Fig. 6, the values of the dynamic increase factor (DIF) for the three models were established. In Fig. 7 is illustrated the variation of DIF for different levels of loading (as a percentage of the standard GSA loading) dependent on the vertical displacement of the column removed point. It can be seen that DIF decreases with increasing the displacement; in other words, the DIF decreases as the structural response (in the case of a column-removal) is in a significantly yielding phase. Under the standard GSA loading a DIF of 1.39, 1.47 and 1.66 was obtained for the Cluj-model, Sibiu-model, respectively for the Bucharest-model. In the previous paper [20], lower DIF values were obtained for two models (subjected to the damage case C_1) representing a three-storey RC framed building designed for two distinct seismic areas: 1.20 for the model designed for a low seismic area (Cluj-model), respectively 1.33 for the model designed for a moderate seismic area (Sibiu-model). Kim [2] determined a DIF = 1.45 for a three-storey RC building subjected to corner column-removal (damage case C_3).



(a)



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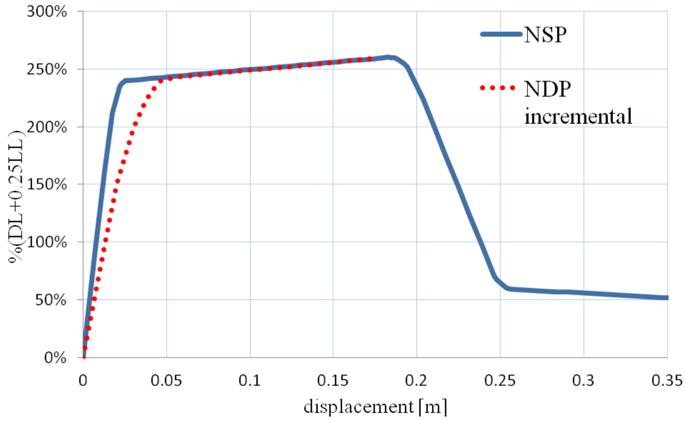

(c)

Figure 6. Load-displacement curves obtained with NS and ND incremental analyses for: a) Cluj-model; b) Sibiu-model; c) Bucharest-model, when subjected to interior column removal.

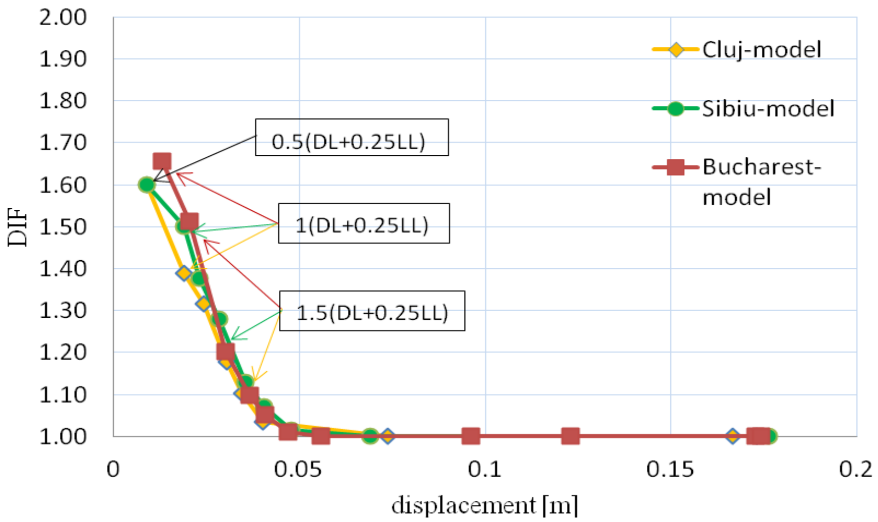

Figure 7. The estimated values of the dynamic increase factor (DIF) based on the capacity curves of the models subjected to the damage case C_4.

The lowest value of DIF (1.0) obtained is the same for all the structures under investigation. The maximum values of DIF are obtained for different levels of loading: 0.5 times the standard GSA loading for the Cluj and Sibiu models, respectively 1.0 times the standard GSA loading for the Bucharest model. In all cases, the dynamic increase factor is approaching to the lowest value (1.0) when a vertical displacement of about 4.5cm is attained. Tsai and Lin [8] obtained a DIF=1.16 for an 11-storey RC framed structure subjected to column-removal (four damage cases) under the ultimate dynamic loading. Marchand et al. [19] obtained values for DIF between 1.0 and 1.4 for a series of RC framed structures under the load combination (1.2DL+0.55LL) as recommended by ASCE 7-05 [22].

The results provided herein together with the existing data from the literature [2, 8, 19] clearly indicates that the dynamic increase factor (DIF) recommended by the GSA (2003) Guidelines [1] for the nonlinear static procedure is overestimated. If a structure is designed to remain elastic, a factor of 2.0 would be appropriate [18]. However, under abnormal loading the structures generally respond in an inelastic manner. Consequently, in order to obtain more accurate results with the nonlinear static procedure DIF should be less than 2.0.

4.5 The influence of the seismic area

In this section, the influence of the seismic design on the potential for progressive collapse of mid-rise RC framed structures (ten stories) is quantified. The comparative study was conducted based on the capacity curves obtained from the nonlinear incremental dynamic analysis for the structural models subjected to the damage case C_4 (Section 4.3). The results are displayed in Fig. 8. Should be underlined that all the analyzed models can resist for a higher load than the applied standard GSA loading (marked in figure with red dotted line) before collapse (the allowable rotation of 0.035rad in plastic hinges are exceeded).

The Cluj and Sibiu models are capable of sustaining a maximum load equal to 177%, respectively 195% of the GSA loading before failure; instead, the Bucharest model is capable of sustaining a much higher load (260% of the GSA loading). This means that the ultimate load bearing capacity of the structure designed for a high seismic area (Bucharest-model) is 33% higher than of the structure designed for a moderate seismic area (Sibiu-model) and is 45% higher than of the structure designed for a low seismic area (Cluj-model).

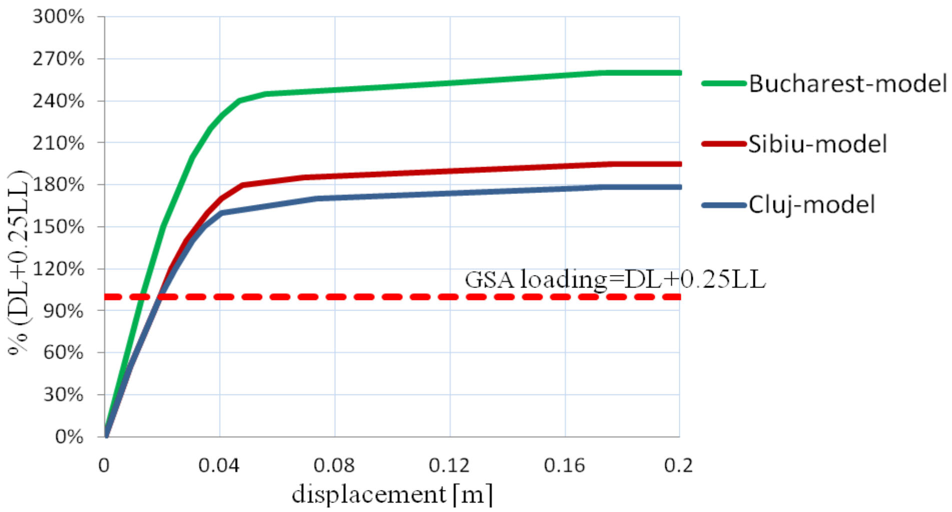

Figure 8. The influence of the seismic design on the potential for progressive collapse of ten-storey structures subjected to the damage case C_4.

5. Conclusions

In this paper, the progressive collapse resistance of a ten-storey RC framed building 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 [13], in general similar with Eurocode 8 [14], was investigated. A nonlinear static analysis was conducted first for the structural models subjected to an interior column-removal in order to estimate the progressive collapse resistance under the standard GSA loading. The obtained results were checked with the nonlinear dynamic procedure. A nonlinear incremental dynamic analysis was carried out to establish (with the maximum accuracy) the ultimate load bearing capacity to progressive collapse of the structures under investigation. The variation of the dynamic increase factor (DIF) dependent on the level of loading applied on the structures (as a percentage of the GSA loading) was highlighted. The results obtained herein lead to the following conclusions:

- The results obtained with the nonlinear static analysis had shown that only the structure designed for a high seismic area (Bucharest-model) has a low potential for progressive collapse, a verdict also confirmed by the nonlinear dynamic analysis; instead, the structures designed for low and moderate seismic areas (Cluj-model and Sibiu-model) have a high risk for progressive collapse, a conclusion which is in contradiction with the one provided by the NDP.
- Based on the capacity curves provided by the NSP it was shown that, under the standard GSA loading the progressive collapse resistance of the model designed for a high seismic area (Bucharest-model) is higher (with 34% and 46%) than of the model designed for moderate (Sibiu-model), respectively low (Cluj-model) seismic area.
- The results obtained with the nonlinear dynamic analysis had shown that, under the standard GSA loading, all the seismically designed RC structures will not collapse when subjected to interior column removal.
- Based on the capacity curves obtained with the nonlinear incremental dynamic analysis, it was shown that all the structures under investigation are capable of sustaining a higher load than the standard GSA loading before failure.
- Based on these capacity curves, the inherent ability to better resist progressive collapse was quantified: the ultimate load bearing capacity to progressive collapse of the ten-storey structure designed for a high seismic area (a_g=0.24g) is 33% higher than of the structure designed for a moderate seismic area (a_g=0.16g) and is 45% higher than of the structure designed for a low seismic area (a_g=0.08g).
- The dynamic increase factor (DIF) recommended by the GSA (2003) Guidelines [1] is overestimated. Under the standard GSA loading a DIF of 1.39, 1.47 and 1.66 was obtained for the Cluj-model, Sibiu-model, respectively for the Bucharest-model. At least for the analyzed structures, the DIF should be limited to 1.7 in order to obtain more accurate results with the NS analysis.
- The variation of DIF dependent on the level of loading was highlighted; it was shown that the dynamic increase factor (DIF) decreases with increasing the vertical displacement of the column removed point. When the structural response is in a significantly yielding phase, the value of DIF is approaching to 1.0 for all the analyzed models, a trend also found in other studies [8, 19].

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