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NUMERICAL INVESTIGATION OF PROGRESSIVE COLLAPSE RESISTANCE FOR SEISMICALLY DESIGNED RC BUILDINGS

BY

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Abstract. In this paper the progressive collapse behavior of a reinforced concrete framed building located in different seismic areas from Romania is investigated. The six-storey structure is designed for low (a_g = 0.08 g), moderate (a_g = 0.16 g) and high (a_g = 0.24 g) seismic zone. Based on the GSA (2003) criteria, a nonlinear static analysis is conducted first in order to estimate the progressive collapse resistance of the models. It was shown that all the structures will collapse when subjected to interior column removal. A nonlinear dynamic analysis is performed to validate these results; it was demonstrated that the capacity curves obtained with the nonlinear static procedure fail to predict the progressive collapse resistance of the analysed models because the dynamic increase factor (DIF) of 2.0 recommended by the GSA (2003) Guidelines seems to be overestimated. A nonlinear incremental dynamic analysis is carried out in order to estimate with maximum accuracy the ultimate load bearing capacity to progressive collapse of the structures under investigation. The variation of DIF with respect to the level of loading is determined. The influence of the seismic design on the progressive collapse resistance of the analysed models is quantified as well.

Key words: progressive collapse; seismic design; GSA (2003) Guidelines; dynamic increase factor, nonlinear incremental dynamic analysis.

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1. Introduction

Preventing progressive collapse of buildings became a major concern of the engineering community after the terrorist attacks from the Murrah Federal Building (Oklahoma City, 1995) and World Trade Center (New York, 2001). The notion of progressive collapse is defined by the GSA (2003) Guidelines 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". The initial cause for this type of structural failure can be man-made (*e.g*. gas explosions, bombs, impact by vehicles, etc.) or produced by natural hazard (*e.g*. earthquakes).

Since abnormal loads are extremely rare events that can occur during the lifetime of a building, it is more appropriate to mitigate the risk for progressive collapse than to especially design them to resist for these loads. In this context, two major guidelines (GSA, 2003; DoD, 2009) for progressive collapse analysis of the new and existing buildings, published by the U.S General Service Administration (GSA) and the U.S Department of Defense (DoD) are available. The Alternative Path Method has been selected by both agencies as the basic approach for providing resistance to progressive collapse. A structure should be capable of developing alternative load paths over a vertical support suddenly removed as a result of abnormal loading. This means that a structure should be designed with an adequate level of continuity, ductility and redundancy, characteristics which are found in the seismic design codes, too: Eurocode 8, ASCE 41-06 and P100/1-2006.

Recent experimental studies (Choi & Kim, 2011; Sadek *et al*., 2011; Yap & Li, 2011; Yu & Tan, 2013) carried out on RC beam–column subassemblages have shown the inherent ability to better resist progressive collapse of the specimens with seismic detailing. Numerical studies (Baldridge & Humay, 2005; Tsai & Lin, 2008; Ioani *et al*., 2007; Marchiş *et al*., 2013) have indicated the beneficial influence of the seismic design on the progressive collapse resistance of mid-rise (11…13 stories) RC framed buildings; it was shown that these structures will not collapse when subjected to column-removal.

Therefore, this study deals with the progressive collapse behavior of a six-storey RC framed building designed for three distinct seismic areas according to the provisions of the Romanian seismic code P100/1-2006, similar with Eurocode 8. The first model is designed for a low seismic area (Cluj-Napoca, where the peak ground acceleration is a_g = 0.08 g); the second one is designed for a moderate seismic zone (Sibiu, a_g = 0.16 g) and the third model is designed for a high seismic area (Bucharest, a_g = 0.24 g). Based on the GSA criteria, the progressive collapse resistance for each model is determined by conducting a nonlinear static procedure (NSP). A nonlinear dynamic analysis is carried out to validate the results obtained with NSP. In order to determine with maximum accuracy the ultimate load bearing capacity to progressive collapse of the structures under investigation, a nonlinear incremental dynamic analysis is performed. Based on the capacity curves obtained with the NS and ND incremental analyses, the variation of the dynamic increase factor (DIF), dependent on the level of loading applied on the structural models, is determined and compared to the value of 2.0 provided by the code (GSA, 2003) for the standard loading. The influence of the seismic design on the ultimate load bearing capacity to progressive collapse of the six-storey building is quantified as well.

2. Details of the Buildings

2.1. Design Details

A six-storey building is designed for three distinct seismic areas from Romania – low where the peak ground acceleration is a_g = 0.08 g (Cluj-model), moderate where a_g = 0.16 g (Sibiu-model) and high where a_g = 0.24 g (Bucharest-model), according to the provisions of the seismic code P100/1-2006, similar with Eurocode 8, and according to the provisions of the design code for concrete structures SR EN 1992-1-1:2004. The structures have the same structural configuration as illustrated in Fig. 1. The buildings 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

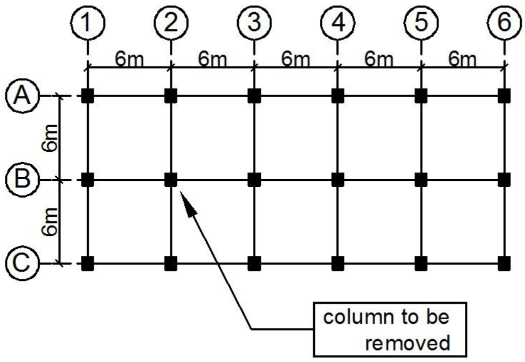

Fig. 1 – Structural models.

where the storey height is 3.6 m. 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 dimensions of the structural components for each model 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 compressive strength f_{cd} = 16.66 MPa and steel type S500 with the design yield strength f_{yd} = 434.78 MPa is considered.

Table 1
Dimensions of the Structural Components

Model	Level	Columns, [cm]	Transverse beams, [cm]	Longitudinal beams, [cm]
Cluj-model	1…6	45 × 45	25 × 45	25 × 40
Sibiu-model	1…6	50 × 50	25 × 50	25 × 45
Bucharest-model	1…3	70 × 70	30 × 55	25 × 55
	4…6	70 × 60	25 × 50	25 × 50

2.2. Structural Models for Progressive Collapse Analysis

A computer program SAP 2000 is used to develop a 3-D finite element model for 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 (Marchiş *et al*., 2013), Eurocode 8 (SR EN 1992-1-1, 2004), ASCE 41-06 (Santafe Iribarren *et al*., 2011). For simplicity, the effective flange width on each side of the beam is taken as three times the slab thickness. Recent experimental studies (Choi & Kim, 2011; Sadek *et al*., 2011) had shown that, generally, 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, (2006), and adjusted for progressive collapse analysis. The maximum allowable rotation in plastic hinges associated to point *C* on the *M vs*. θ_p curve (Fig. 2), which corresponds to the “Collapse Prevention” performance level is increased from 0.02 rad. to 0.035 rad. as recommended by the GSA (2003) Guidelines 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, (2006), 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 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 (0.04 rad.,...,0.10 rad.) given by the DoD (2009) Guidelines. Also, the characteristic values of the concrete compressive strength and the tensile yield strength for the reinforcing steel were increased by a factor of 1.25 (GSA, 2003).

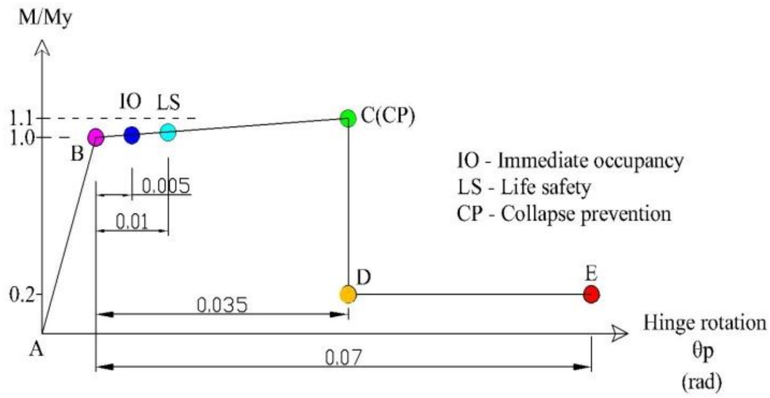

Fig. 2 – Plastic hinge model.

3. Progressive Collapse Analysis: Main Criteria

3.1. Damage Cases

As recommended by the GSA (2003) Guidelines, the potential 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_4 is considered herein (Fig. 1).

3.2. Loading Criteria

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 – 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 suddenly removed from the structure.

For the dynamic analysis, DIF is not considered, as follows:

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

3.3. Acceptance Criteria

When conducting nonlinear analyses, the acceptance criteria in order to obtain a low potential for progressive collapse is related to an allowable rotation in plastic hinges of 0.035 rad. This value corresponds to the "Collapse Prevention" performance level (point *C* on the *M vs*. θ_p curve) displayed in Fig. 2.

4. Progressive Collapse Resistance

4.1. Nonlinear Static "Push-Down" Analysis

Based on the GSA criteria, a nonlinear static "push-down" analysis (NSP) is carried out using SAP 2000 in order to estimate the progressive collapse resistance for the interior column-removal condition. Moment plastic hinge type, as given in Fig. 2, is assigned to beams ends. The capacity curves provided by the NSP (using the displacement controlled method) are displayed in Fig. 3 for the structures under investigation. The vertical axis represents the percentage of the standard GSA loading (eq. (1)) and the horizontal axis represents the vertical displacement of the column removed point. The failure

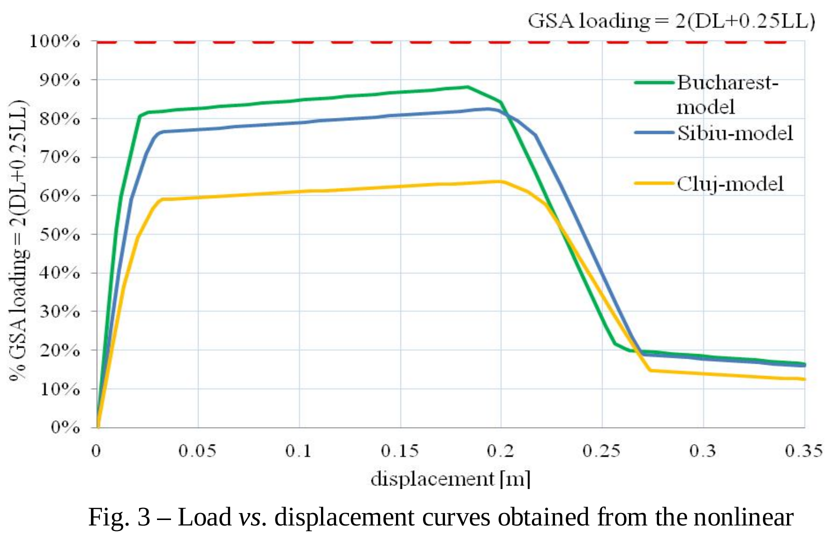

Fig. 3 – Load *vs*. displacement curves obtained from the nonlinear static analysis.

limit is identified at 64%, 82% and 88% for the Cluj-model, Sibiu-model, respectively for the Bucharest-model. At this level of loading, the allowable rotation in plastic hinges which occur in the critical beam sections is exceeded (> 0.035 rad.); these hinges are classified as "Collapse Prevention" performance level (see Fig. 2). This means that all the three structural models collapse before the standard GSA loading is attained marked in Fig. 3 with dotted line. However, Kim, (2006), and Tsai & Lin, (2008), have found that the dynamic increase factor (DIF) of 2.0 provided by the GSA (2003) Guidelines and considered herein can be overestimated when performing a nonlinear static analysis. Consequently, a nonlinear dynamic "time-history" analysis is conducted to verify the results' accuracy obtained with the NSP and thus to determine the real magnitude of DIF.

4.2. Nonlinear Dynamic "Time-History" Analysis

For the nonlinear dynamic analysis the load combination given by eq. (2) is applied downward to the undamaged structure; DIF is not considered. Then, the interior column is suddenly removed from the model. The time for removal is set to t_r = 5 ms, a value also adopted in many studies (*e.g.* Santafe Iribarren *et al*., 2011); 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 = 0.23 s for the Cluj-model, T = 0.20 s for the Sibiu-model, respectively T = 0.16 s for the Bucharest-model) as recommended by the DoD (2009) Guidelines. Also, a 5% damping ratio is considered in the dynamic analysis, a value adopted by Sasani & Kropelnicki, (2008), and Tsai & Lin, (2008), as well. The response of the analysed models is observed over a time span t = 3 s and displayed in Fig. 4; after 3 seconds all the structures

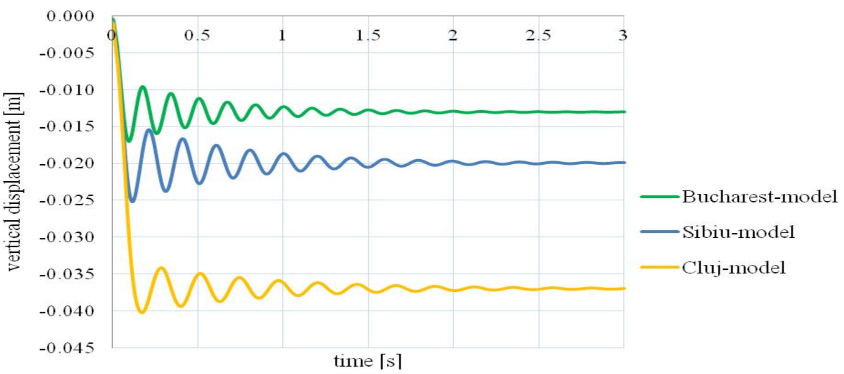

Fig. 4 – Time *vs.* displacement curves of the column-removed point obtained from the nonlinear dynamic analysis.

subjected to instantaneously column removal reach a new static equilibrium. The maximum displacement obtained is 4.0 cm for the Cluj-model, 2.5 cm for the Sibiu-model, respectively 1.16 cm for the Bucharest-model.

At this step, the plastic hinges occur (not shown here) in all 48 critical beam sections for the Cluj-model and in only 28 and 24 sections for the Sibiu-model, respectively for the Bucharest-model. The rotations in these plastic hinges are well below the allowable value of 0.035 rad. as recommended by the GSA (2003) Guidelines. Consequently, all the structures will not fail (low potential for progressive collapse) when subjected to interior column removal, results which are in contradiction with those obtained with NSP. This clearly indicates that the dynamic increase factor (DIF) used in the static analysis underestimates the progressive collapse resistance of the models. Therefore, a nonlinear incremental dynamic analysis is conducted in the following section in order to estimate with the maximum accuracy the ultimate load bearing capacity to progressive collapse of the structures under investigation.

4.3. Nonlinear Incremental Dynamic Analysis

The nonlinear incremental dynamic analysis is the most accurate method used to determine the ultimate load bearing capacity to progressive collapse of the structures. This assumes to carried out a series of nonlinear dynamic analyses for different levels of loading as a percentage of the standard GSA loading = DL + 0.25LL. The magnitude of the load is gradually increased until the structural model collapses (the allowable rotation of 0.035 rad. in plastic hinges, associated to the “Collapse Prevention” performance level, is reached). The value of the applied loads (as a percentage of the GSA loading) and the maximum vertical displacement of the column removed-point are collected in order to construct the capacity curve. This type of analysis is generally time-consuming, dependent on the size of the FEA model and/or on the number of the loading steps required. This approach was used by Tsai & Lin, (2008), in order to estimate the ultimate load bearing capacity to progressive collapse of an 11-storey RC framed building design for a high seismic area from Taiwan.

Eleven loading steps, starting from 0.4(DL + 0.25LL), were considered in the analysis. The capacity curves obtained for the models under investigation are illustrated in Fig. 5 with dotted line in regard with the one obtained with the nonlinear static analysis marked with continuous line. The vertical axis represents the percentage of the standard GSA loading (eq. (2)) and the horizontal axis represents the displacement of the column-removed point. The results displayed in Fig. 5 indicate that the dynamic increase factor (DIF) is closed to 1.0 as the structural response is in a significantly yielding phase. The variation of DIF depending on the level of loading applied in the structure is discussed in the next section.

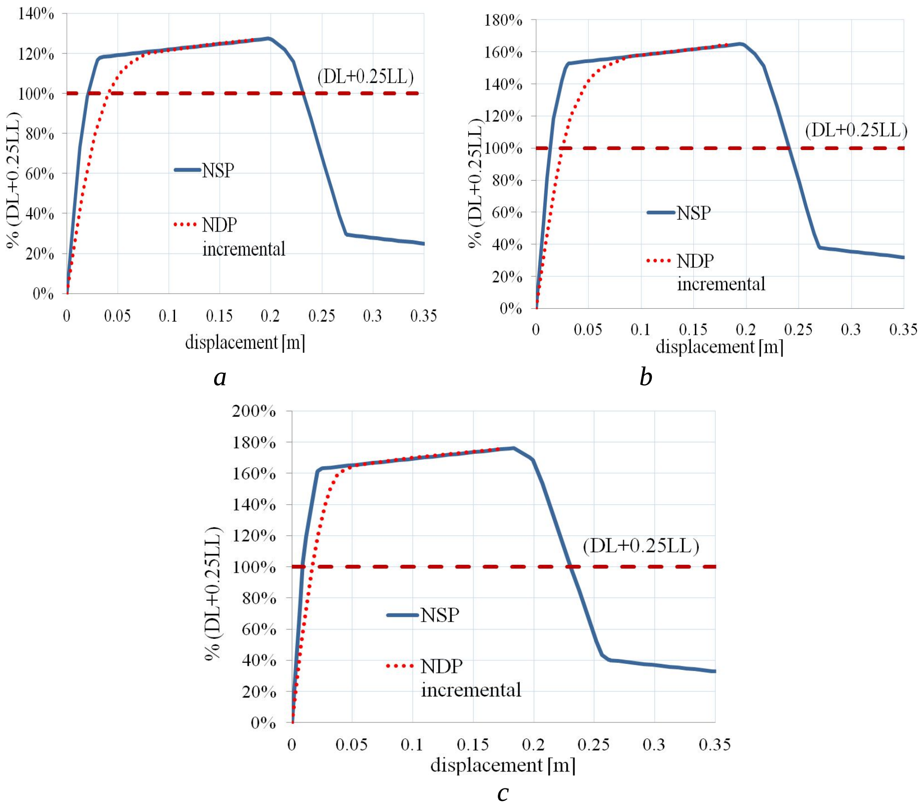

Fig. 5 – Load *vs*. displacement curves obtained with NS and ND incremental analyses: *a* – Cluj-model; *b* – Sibiu-model; *c* – Bucharest-model.

4.4. Variation of DIF

The results provided by the nonlinear dynamic analysis indicate that the dynamic increase factor (DIF) of 2.0, recommended by the GSA (2003), Guidelines to be used in the nonlinear static analysis, underestimates the progressive collapse resistance of the RC framed structures, results which are similar with those obtained in other studies (Tsai & Lin, 2008; Marchiş *et al*., 2013; Kim, 2006; Marchand *et al*., 2009). DIF that allows a nonlinear static solution to estimate a nonlinear dynamic solution should be less than 2.0, issue underlined by the DoD, (2009), Guidelines. The dynamic increase factor 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 & Lin, (2008), as well.

Based on the results displayed in Fig. 5, the real values of DIF for the analysed structural models were determined. 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, is illustrated in Fig. 6. It can be seen that the dynamic increase factor decreases as the structural response is in a significantly yielding phase.

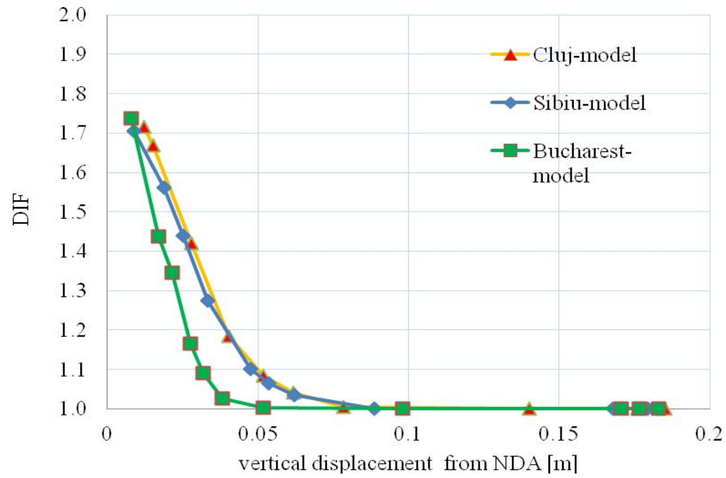

Fig. 6 – The estimated values of DIF based on the capacity curves of the models subjected to column-removal.

Under the standard GSA loading a DIF of 1.19, 1.43 and 1.44 were obtained for the Cluj-model, Sibiu-model, respectively for the Bucharest model. In a previous paper (Marchiş *et al.*, 2013), higher DIF values were obtained for three models (subjected to the damage case C_4) representing a 10-storey RC framed building designed for the same seismic areas: 1.39 for the model designed for low seismic area (a_g = 0.08 g), 1.47 for the model designed for moderate seismic area (a_g = 0.16 g), respectively 1.66 for the model designed for high seismic area (a_g = 0.24 g). In fact, all these values established for the GSA progressive collapse load are in the same range of variation with the findings of Marchand *et al.*, (2009), (DIF = 1.0,...,1.75 for the DoD, (2009), progressive collapse load).

The highest value for DIF was obtained under a theoretical value of 0.4 times the standard GSA loading; the lowest value of 1.0 was obtained for all the analysed structures under the ultimate dynamic loading. Tsai & Lin, (2008), obtained a DIF of 1.16 for an 11-storey RC framed building under the maximum loading and Marchand *et al.*, (2009), obtained values for DIF in the range of 1.0 and 1.4 for a series of RC framed structures under the load combination (1.2DL + 0.5LL) recommended by the ASCE 7-05, (2005). This clearly indicates that the dynamic increase factor recommended by the GSA, (2003), Guidelines for the nonlinear static procedure (NSP) is overestimated, especially when the structural response is in a significantly yielding phase. As underlined by the DoD, (2009), Guidelines, DIF used in the nonlinear static analysis to approximate a dynamic solution should be less than 2.0.

4.5. The Ultimate Load Bearing Capacity to Progressive Collapse

Based on the results provided by the nonlinear incremental dynamic analysis the influence of the seismic design on the ultimate load bearing capacity to progressive collapse of the six-storey RC framed structures is quantified; the capacity curves are displayed in Fig. 7. Should be underlined that all the structures under investigation can resist a higher load than the standard GSA loading = DL + 0.25LL before collapse.

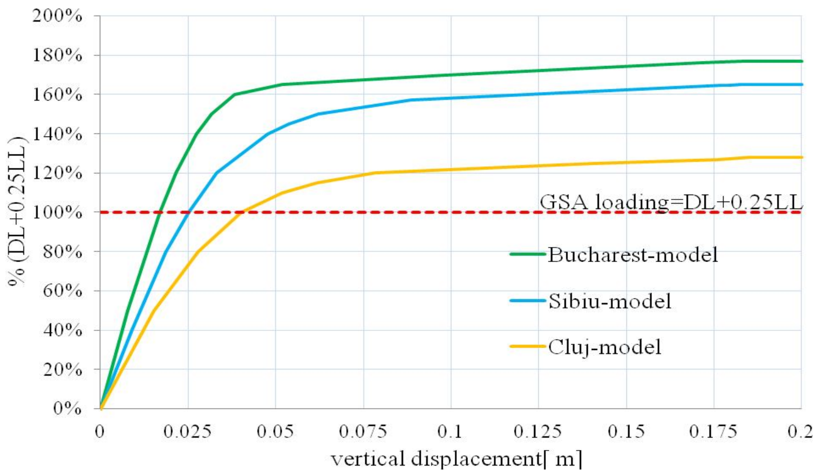

Fig. 7 – The influence of the seismic design on the ultimate load bearing capacity to progressive collapse of the six-storey RC framed structures.

The ultimate load bearing capacity to progressive collapse is identified at 128% of the GSA loading for the Cluj-model. Instead, the models designed for a higher seismic area, are capable of sustaining a higher load: 165% and 177% of the GSA loading for the Sibiu-model, respectively for the Bucharest-model. This means that the ultimate load bearing capacity to progressive collapse of a structure designed for a high seismic area (a_g = 0.24 g) increases with 7.3% with respect to the one of a structure designed for a moderate seismic area (a_g = = 0.16 g), and with 38% with respect to the one of the structure designed for a low seismic area (a_g = 0.08 g).

5. Conclusions

In this study, the behavior to progressive collapse of a six-storey RC framed building design for low (a_g = 0.08 g), moderate (a_g = 0.16 g) and high (a_g = 0.24 g) seismic area according to the provisions of the Romanian seismic code P100/1-2006 was investigated. A nonlinear static analysis was conducted first in order to establish the progressive collapse resistance of the structural models. A nonlinear dynamic analysis was carried out to check these results. In order to establish the ultimate load bearing capacity to progressive collapse of the structures subjected to interior column removal, a nonlinear incremental dynamic analysis was performed; based on the obtained results, the variation of the dynamic increase factor (DIF) dependent on the level of loading applied on the structure was identified. The influence of the seismic design on the progressive collapse resistance was quantified as well. Based on the results obtained herein, the following conclusions can be drawn:

1º The results obtained with the nonlinear static procedure (NSP) had shown that all the three analysed structures have a high risk for progressive collapse; this conclusion is in contradiction with the one provided by the nonlinear dynamic procedure (NDP).

2º The results obtained with NDP have indicated that, under the standard GSA loading, all the models will not collapse when subjected to suddenly column-removal.

3º Based on the results provided by the nonlinear incremental dynamic analysis it was founded that all the structural models are capable of sustaining a higher load than the standard GSA loading before collapse.

4º The dynamic increase factor (DIF) recommended by the GSA, (2003), Guidelines to be used in the nonlinear static analysis to approximate a dynamic solution is overestimated. Under the standard GSA loading a DIF of 1.19, 1.43 and 1.44 were obtained for the Cluj-model, Sibiu-model, respectively for the Bucharest-model. In order to obtain accurate results with NSP, DIF should be limited to 1.5 at least for the analysed structures.

5º The variation of DIF dependent on the level of loading was investigated. It was shown that the dynamic increase factor decreases with increasing the displacement of the column-removed point; in other words, DIF decreases (until 1.0) as the structural response is in a significantly yielding phase, a trend also found in other studies (Tsai & Lin, 2008; Marchand *et al.*, 2009).

The influence of the seismic design on the progressive collapse resistance was quantified: the ultimate load bearing capacity of the six-storey structure designed for high seismic area is 7.3% higher than of the structure designed for a moderate seismic area, and is 38% higher than of the structure designed for low seismic area.

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INVESTIGAREA NUMERICĂ A REZISTENȚEI LA COLAPS PROGRESIV PENTRU STRUCTURILE DIN BETON ARMAT PROIECTATE SEISMIC

(Rezumat)

Se investighează comportarea la colaps progresiv a unei structuri în cadre din beton armat amplasată în zone seismice din România. O clădire cu şase niveluri este proiectată pentru o zonă seismică redusă (a_g = 0.08 g), moderată (a_g = 0.16 g) şi înaltă (a_g = 0.24 g). Utilizând criteriile GSA, o analiză statică neliniară se realizează întâi pentru a estima potențialul la colaps progresiv al modelelor structurale. S-a arătat că toate modelele analizate se vor prăbuşi când sunt supuse îndepărtării unui stâlp interior de la primul nivel. O analiză dinamică neliniară se realizează ulterior pentru a valida aceste rezultate; s-a arătat că, privitor la curbele de capacitate obținute cu procedura statică neliniară acestea nu sunt capabile să prezică potențialul de colaps progresiv al modelelor structurale întrucât valoarea de 2.0 a factorului de amplificare dinamică (DIF) recomandată de Ghidul GSA, (2003), se pare că este supraestimată. O analiză dinamică neliniară incrementală se realizează astfel pentru a estima cu acuratețe maximă capacitatea portantă ultimă la colaps progresiv a modelelor structurale considerate. Se determină variația lui DIF în funcție de nivelul de încărcare. De asemenea, se cuantifică influența proiectării seismice asupra capacității portante ultime la colaps progresiv a structurilor în cadre din beton armat cu şase niveluri.