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DETERMINISTIC METHOD FOR ASSESSING THE DEGREE OF DEGRADATION FOR MASONRY CONDOMINIUM STRUCTURES IN ROMANIAN URBAN AREAS

BY

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Abstract. The paper presents an assessment methodology for the degree of degradation of masonry load bearing wall condominium structures based on the deterministic approach. According to the generally accepted definition, a deterministic model is a one in which no randomness is involved in the development of future states. As such, a given input will always produce the same output. In risk analysis the events are completely pre-determined by causality and the analyses are performed in order to determine the effects of assumed events on considered structures. The methodology consists in performing nonlinear Finite Element Analysis on two types of masonry condominium structures frequently met in Romanian urban areas. The second step of the methodology involves using the values of the fundamental period of vibration, evaluated by modal analysis at the end of each earthquake scenario, to compute the degradation coefficient. The evaluation procedure of the degradation state chosen in this deterministic approach is based on a degradation coefficient belonging to the cathegory of maximum softening damage indices. The results are presented in terms of periods of vibrations and modal displacements. The degradation coefficient was computed based on the data resulted from the modal analysis and was used to classify the structures according to the reference intervals.

Key words: deterministic approach; nonlinear time history analysis; degradation coefficient; masonry condominium structures.

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1. Introduction

The impact of natural disasters, such as strong earthquakes, on rapidly developing urban areas may result in losses of human lives and extensive property damage (Bilham, 2009; Erdik *et al*., 2011). This is a direct consequence of chaotic real estate development (Balaban, 2012), neglecting or misinterpreting design code requirements (Soliman, 2012) or due to lack of sufficient information related to vulnerability and risk assessment (England *et al*., 2008; Oliveira, 2008).

It is generally accepted that vulnerability is a part of risk (England *et al*., 2008) because it addresses the potential damage produced by a given hazard, in this case an earthquake. In view of this fact, a structure is considered to be vulnerable if a relatively small damage leads to undesired consequences which are disproportionately large with respect to the level of damage (Agarwal *et al*., 2003). A growing interest in assessing the seismic vulnerability and associated risk of European cities to earthquake hazard has been observed during the past two decades (Bărbat *et al*., 1996; Dolce *et al*., 2006; Ansal *et al*., 2010; Toma *et al*., 2011). This is directly connected to the increased number of strong earthquakes recorded all over the world for the past 20 years that led to the partial or total collapse of many buildings. It should be pointed out that most of the research in this field have been conducted in earthquake prone areas mostly located in the South, South-East and Eastern Europe. Additionally, the increased public awareness to the effects of natural disasters upon communities, resulted in a great emphasis being placed on the role of disaster mitigation measures (Erdik *et al*., 2011; Kappos *et al*., 2011).

Rapidly growing cities pose a new challenge to civil engineers and researchers alike when it comes to analysing the building inventory from the point of view of its vulnerability (Balaban, 2012; Soliman, 2012; Dolce *et al*., 2006; Toma *et al*., 2011). Commonly used procedures, such as building-by-building thorough inspection by structural engineers, may provide accurate information related to the vulnerability of a structure but can not cope with the rapidly changing conditions in today cities (Fekete *et al*., 2010).

Recent advances in Finite Element Analysis (FEA) models (Moens & Hanss, 2011; Patelli *et al*., 2012) coupled with the use of Geographical Information Systems (GIS) in monitoring the urban environment (Anagnostopoulos *et al*., 2008; Pessina & Meroni, 2009) have made possible the development of pre- and post-disaster intervention plans by the local/state authorities. The pre-seismic phase is related to earthquake risk mitigation efforts from hazard assessment to the evaluation of seismic risk and measures taken to decrease vulnerabilities (Sesetyan *et al*., 2011). The post-disaster plans come into effect immediately after the occurrence of the earthquake and are based on the estimation of physical damage and number of forecast casualties, all of which are part of an integrated vulnerability assessment tool (Wieland *et al*., 2012).

The paper presents a deterministic method for the assessment of the degree of degradation of masonry condominium structures frequently met in Romanian urban areas. The method is based on non-linear time history FEA of numerical models subjected to successive earthquake scenarios. The degradation coefficient is computed at the end of each seismic event using the fundamental period of vibration of the structure (DiPasquale & Cakmak, 1990).

2. Methodology

The assessment methodology is based on the deterministic approach. According to the generally accepted definition, a deterministic model is an one in which no randomness is involved in the development of future states. As such, a given input will always produce the same output. In risk analysis the events are completely pre-determined by causality and the analyses are performed in order to determine the effects of assumed events on considered structures/systems (Kerry *et al*., 2006).

The methodology consists in performing nonlinear FEA on two types of masonry condominium structures frequently met in Romanian urban areas by using SAP2000 v.14 software package. The models were subjected to various earthquake scenarios that took place during the life time of the considered structures.

The second step of the methodology involves using the values of the fundamental period of vibration, evaluated by modal analysis at the end of each earthquake scenario, to compute the degradation coefficient (DiPasquale & Cakmak, 1990). The structures were then classified according to the reference intervals (Singhal & Kiremidjian, 1996).

2.1. Finite Element Model

The numerical models were based on real structures that could be met in Iaşi municipality. The FEM models of the masonry condominium structures are shown in Fig. 1, for the first type of structure from now on refered to as Model-1, and Fig. 2, for the second type of structure from now on refered to as Model-2.

The first structure was built according to Project no. *4199*, whereas the second structure followed the guidelines of Project *1497*/1966 (ICPROM, Iaşi). The geometrical data of the numerical models pertaining to the real structures is shown in Table 1.

The values of the material properties for the masonry used for both models were 6.1 N/mm^2 for the compressive strength and 2,870 N/mm^2 for the modulus of elasticity. The density of masonry was taken as 1,846 kg/m^3.

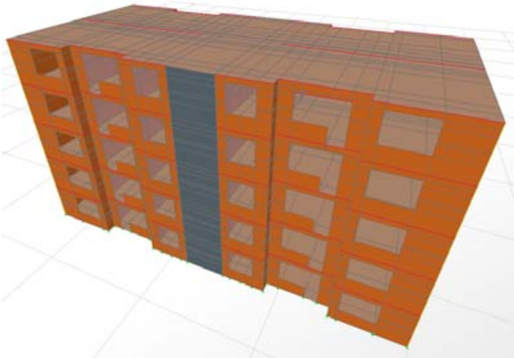


Fig. 1 – Model 1, according to Project no. *4199*.

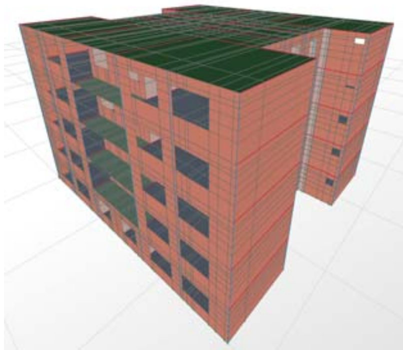


Fig. 2 – Model 2, according to Project no. *1497*/1966.

Table 1
Geometrical Data Used for the Numerical Models

Data	FEM model	
	Model 1	Model 2
No. of storeys	4	4
Height of the structure, [m]	13.5	13.75
Storey heigh, [m]	2.7	2.75
Thickness of outside walls, [cm]	30	37.5
Thickness of interior load bearing walls, [cm]	25	25
Slab thickness, [cm]	9	9

Table 2
Identification Data for Each Considered Earthquake Scenario

	Location	Depth km	Earthquake code	Code of recording station	Date	Magnitude Mw	PGA m/s^2
1	Vrancea, Romania	109	771	INC1	1977.03.04	7.5	1.95
2		133	861	IAS2	1986.08.30	7.3	1.46
3		91	901	IAS2	1990.05.30	7.0	1.26
4		91	902	IAS2	1990.05.31	6.4	0.46
5		100	041	IAS4	2004.10.27	6.0	0.66

The slabs were made of reinforced concrete C8/10 having a compressive strength of 6.5 N/mm^2 and a modulus of elasticity of 2.1×104 N/mm^2.

The self weight of the structure was automatically computed by the program. The weight of the finishes and floorings, which also adds to the self weight of the structure, and their design values, were selected in accordance to STAS 10101/1-78.

2.2. Earthquake Scenarios and Analysis Options

Several earthquake scenarios, all of which took place during the life time of the analysed structures, were considered by means of their accelerograms. All the recordings of the accelerograms were for the city of Iaşi (Borcia, 2006) with the exception for the 1977 earthquake. In this case, the recording made in Bucharest at the headquarters of the Romanian Institute for Building Research was considered. The identification data for each earthquake is presented in Table 2.

The non-linear time history analysis was performed by using the Hilber-Hughes-Taylor time integration method (1977), governed by the eq. of motion

$$M\ddot{u}_{t+\Delta t} + \left(1+\alpha\right)C\dot{u}_{t+\Delta t} - \alpha C\dot{u}_t + \left(1+\alpha\right)Ku_{t+\Delta t} - \alpha Ku_t = \left(1+\alpha\right)F_{t+\Delta t} - \alpha F_t, \quad (1)$$

where M, C and K are the mass, the damping and the stiffness matrices, respectively, and F is the vector of external loads. The $\ddot{u}$, $\dot{u}$ and u are the vectors of acceleration, velocity and displacement, respectively. It can be observed that for $\alpha = 0$ the above eq. reduces to the one proposed by Newmark. Moreover, the expressions associated with the displacement

$$u_{t+\Delta t} = u_t + \Delta t\dot{u}_t + \Delta t^2\left[\left(\frac{1}{2}-\beta\right)\ddot{u}_t + \beta\ddot{u}_{t+\Delta t}\right], \quad (2)$$

and velocity

$$\dot{u}_{t+\Delta t} = \dot{u}_t + \Delta t\left[\left(1-\gamma\right)\ddot{u}_t + \gamma\ddot{u}_{t+\Delta t}\right], \quad (3)$$

fields using the finite differences method are identical to those proposed by Newmark (1959), where the α, β and γ coefficients are defined as

$$-\frac{1}{3} \le \alpha \le 0, \quad \beta = \frac{1-\alpha^2}{4}, \quad \gamma = \frac{1}{2} - \alpha.$$

This is an implicit method that allows for energy dissipation and second order accuracy (Grosu & Harari, 2007), both of which were not possible with the regular Newmark method. As presented in the literature (Negruţ *et al*., 2007), the method preserves the stability and numerical damping properties of the trapezoidal method while achieving a second order accuracy when used in conjunction with the second order differential eq.

2.3. Degradation Coefficient

The evaluation procedure of the degradation state, chosen in this deterministic approach, is based on the degradation coefficient proposed by DiPasquale and Cakmak (1990). It belongs to the cathegory of maximum softening damage indices. These indices are based on the variation of the fundamental periods of vibration of a structure during a seismic event. In several papers (DiPasquale & Cakmak, 1990; Nielsen *et al*., 1992) a correlation was found between damage state of structures and the maximum softening. DiPasquale and Cakmak (1990) defined the maximum softening for the one-dimensional case, where only the fundamental eigen-frequency was considered

$$\delta_M = 1 - \frac{T_0}{T_{\deg r}}, \tag{4}$$

where: T_0 is the fundamental eigen-period for the undamaged structure and $T_{\deg r}$ – the maximum value of the fundamental eigen-period during the earthquake. A drawback is that this index provides no information about the distribution of damage in the structure as the index is a global one. However, the localization of the damage within a structure can be assessed by means of FEA (Skjaerbaek *et al*., 1998; Li *et al*., 2012) or by means of technical inspections on structures.

In the present paper, the degradation coefficient based on the results obtained by DiPasquale and Cakmak (1990) was used. The $T_{\deg r}$ of the model was assessed by running a modal analysis using the structural stiffness matrix at the end of each earthquake scenario shown in Table 2. The classification of the structures depending on the coefficient of degradation was made according to the reference intervals (Singhal & Kiremidjian, 1996).

3. Results and Discussions

The results are presented in terms of periods of vibrations and modal displacements. The degradation coefficient was computed based on the data obtained from the modal analysis and was used to classify the structures according to the reference intervals (Singhal & Kiremidjian, 1996).

3.1. Fundamental Period of Vibration and Modal Displacements

The change in the fundamental period of vibration of both structures is shown in Fig. 3. It can be observed that the structure denoted by Model 1 is stiffer than the other one. This could be explained by the in-plane shapes of the condominiums considered in this paper. This strongly influences the stiffness of the structure with direct consequences on the dynamic characteristics.

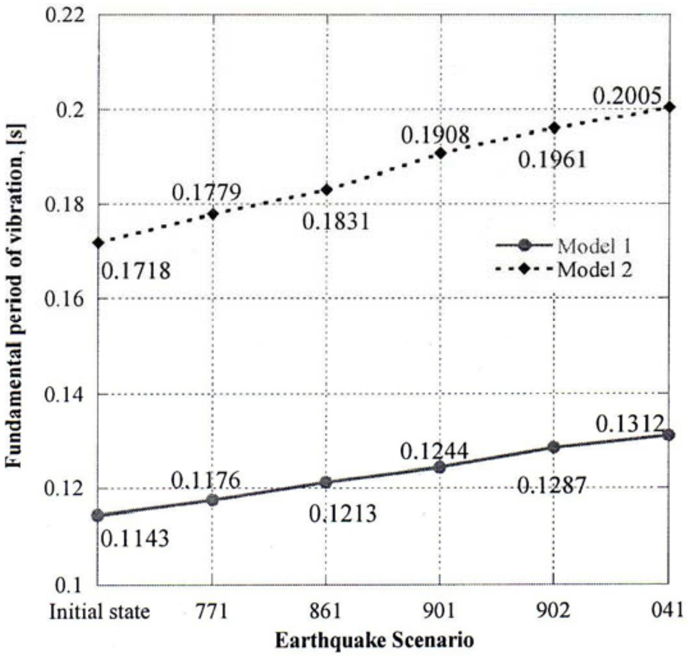


Fig. 3 – Change in the fundamental period of vibration of the considered structures according to the earthquake scenario.

Additionally, it can be noted that the values of the fundamental period of vibration increases for both structures after each earthquake scenario indicating the occurrence of degradations. The graph depicting the variation of the fundamental period of vibration for Model 2 analysis case exhibits a steeper slope than its counterpart connected to Model 1. This means a larger accumulation of degradations.

However, the parameters that can influence the fundamental period of vibration are numerous as well as the modal shapes are closely dependent on one another and more in depth analysis should be conducted in this direction (Kose, 2009).

Fig. 4 shows the resultant displacement contours associated with the fundamental mode of vibration for Model 1. It can be observed that the first mode of vibration is a translational one. The staircase does not have a significant influence on the outcome of the modal shape due to very large stiffness of structural walls. They are symmetrically positioned with respect to both in-plane axes of the model. The maximum modal displacement was recorded at the top of the structure and had a value of 1.82.

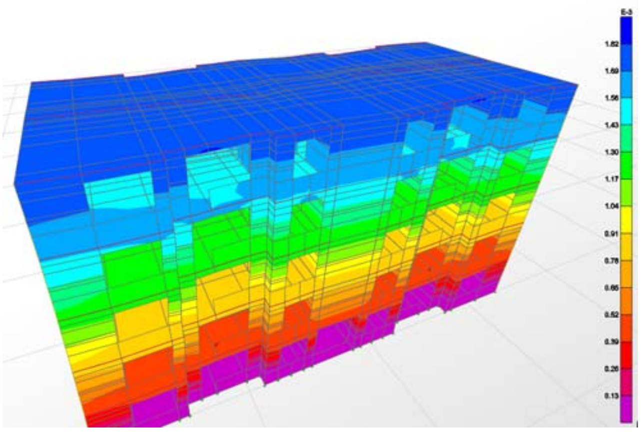
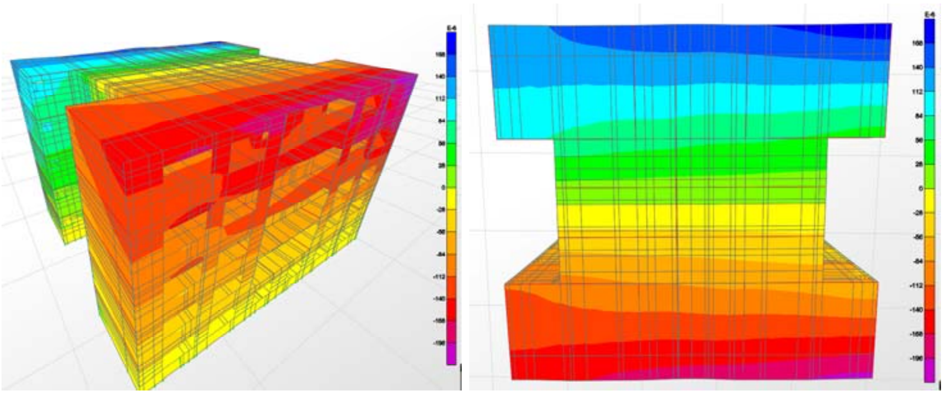


Fig. 4 – Resultant displacement contours associated to the fundamental mode of vibration for Model 1.



a *b*

Fig. 5 – Resultant displacement contours associated to the fundamental mode of vibration for Model 2: *a* – three-dimensional view, *b* – plan view at the top level.

Fig. 5 shows the displacement countours in the longitudinal direction *X* of the Model 2 in case of the first mode of vibration. Contrary to Model 1, the presence of the stair case strongly influences the modal shape by inducing a torsional effect despite the symmetrical positioning of the structural walls with respect to the two in-plane axes. Thus, the fundamental mode of vibration is characterized by a maximum value of the displacement at the top of the structure of 0.2 mm, and by a minimum one of –0.22 mm.

3.2. Degradation Coefficient

The degradation coefficient was computed based on eq. (4) proposed by DiPasquale and Cakmak (1990). Fig. 6 presents the values of the degradation coefficient for both considered models computed after each earthquake scenario. It can be observed that Model 2 shows higher values of the degradation coefficient than Model 1.

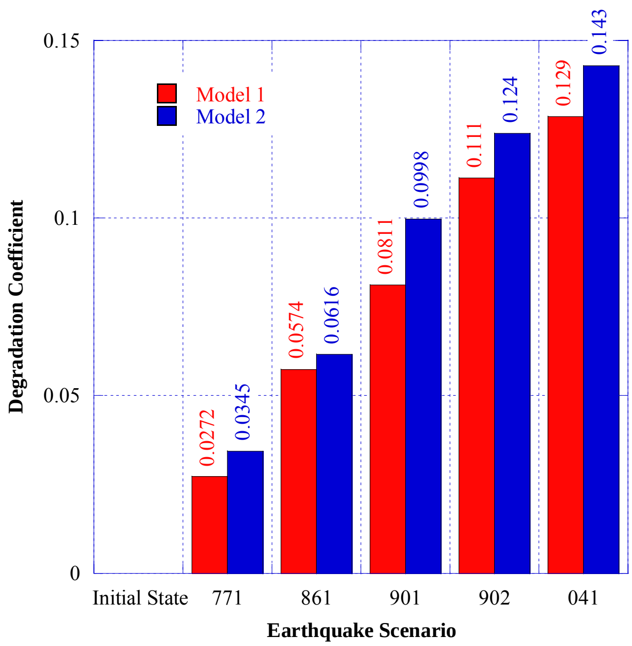


Fig. 6 – Degradation coefficient for the two considered structures.

Table 3

Classification of the Structures Based on the Coefficient of Degradation

Degradation state	Reference interval, δ_M
Minor degradations	0.0…0.2
Capital repairs needed	0.2…0.5
Irreparable	0.5…1
Total collapse	> 1.0

As average, the values are 13% higher. Taking into account the values of the fundamental period of vibration for both structures, shown in Fig. 3, the obtained results for the degradation coefficient are following the trend. The masonry condominium structure denoted as Model 2 is a more flexible one, exhibiting higher values for the period of vibration, and therefore more prone to the occurrence of degradations due to lateral loads produced by the earthquakes (Tena-Colunga *et al*., 2009).

The increase of the values of the degradation coefficient from one earthquake scenario to the next indicates an accumulation of the damage consisting of cracks in the material. In turn, this leads to a decreased stiffness of the model which ultimately influences the fundamental period of vibration of the structure (Stavroulaki & Liarakos, 2012). If left unattended, the damage can spread and affect the stability of the entire structure (Schafer & Bajpai, 2005).

Taking into account the reference intervals proposed in the scientific literature for the degradation coefficient (Singhal & Kiremidjian, 1996) (s. Table 3), it can be seen that both structures fall within the limits of the *minor degradation* interval, meaning that no serious repairs are needed yet. However, the results from the numerical simulations should be confirmed by technical inspections of the buildings in order to assess whether or not there are any hidden defects that could lead to higher damage states (Cardoso *et al*., 2005).

4. Conclusions

The assessment methodology is based on the deterministic approach. According to the generally accepted definition, a deterministic model is a model in which no randomness is involved in the development of future states. As such, a given input will always produce the same output. In risk analysis the events are completely pre-determined by causality and the analyses are performed in order to determine the effects of assumed events on considered structures/systems. The methodology consists in performing nonlinear FEA on two types of masonry condominium structures frequently met in Romanian urban areas. The second step of the methodology involves using the values of the fundamental period of vibration, evaluated by modal analysis at the end of each earthquake scenario, to compute the degradation coefficient.

Based on the obtained results it can be concluded that both considered structures show only minor degradations even after being subjected to five consecutive earthquake scenarios, all of which took place during the life time of the considered masonry condominiums. The increase in the values of the degradation coefficient from one earthquake scenario to the next indicates an accumulation of the damage consisting of cracks in the material. However, both structures fall within the limits of the *minor degradation* interval, meaning that no serious repairs are needed yet.

The deterministic method presented in this paper can be further extended to other types of structures and the obtained results used in making seismic risk maps for urban areas. These maps could be of critical importance in the decision making process should a desastruous earthquake strike as they can offer valuable information of the most likely to be affected areas in a city.

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METODĂ DETERMINISTĂ PENTRU DETERMINAREA GRADULUI DE DEGRADARE A BLOCURILOR DE LOCUIT DIN ZIDĂRIE ÎNTÂLNITE ÎN ZONELE URBANE DIN ROMÂNIA

(Rezumat)

Se propune o metodologie deterministă de evaluarea a gradului de degradare a structurilor din pereţi de zidărie portantă având ca destinaţie clădiri de locuit. Asemenea structuri sunt întâlnite frecvent în zonele urbane de pe teritoriul României. În conformitate cu definiţia general acceptată a ceea ce înseamnă un model deterministic, nu sunt admise rezultate având caracter aleatoriu ci totul este deja pre-determinat pornind de la scenarii iniţiale şi urmărind ecuaţii şi metode matematice. În consecinţă, se vor obţine mereu aceleaşi rezultate pentru acelaşi set de date de intrare în model. În analiza riscului, evenimentele sunt în totalitate predefinite iar analizele sunt rulate pentru a evalua efectele acestor evenimente asupra structurilor, în particular asupra structurilor de construcţii. Metoda propusă constă în rularea de analize neliniare folosind metoda elementului finit pe două tipologii de structuri din zidărie portantă întâlnite în mod frecvent în zonele urbane de pe teritoriul României. Al doilea pas al

metodei presupune utilizarea valorilor obținute pentru perioadele proprii de vibrație, determinate prin analize modale la finalul fiecărui scenariu de analiză dinamică biografică neliniară, pentru a calcula coeficientul de degradare. Procedeul de calcul al coeficientului de degradare, ales pentru metoda deterministă prezentată, are la bază metoda clasei de indici maximi de plastifiere. Rezultatele sunt prezentate atât pentru perioadele proprii de vibrație, modul fundamental de vibrație prin deplasări pe direcție longitudinală cât şi pentru valorile indicelui de degradare. Acesta din urmă este folosit la încadrarea structurilor analizate conform intervalelor de referință propuse în literatura de specialitate.