Acta Technica Napocensis: Civil Engineering & Architecture Vol. 56, No. 2 (2013)
Journal homepage: http://constructii.utcluj.ro/ActaCivilEng
Special Issue: First International Conference for PhD Students in Civil Engineering, CE-PhD 2012.

Comparative study on damage index evaluation for new and existing reinforced concrete structures

Claudiu Tudose*[1]

[1] *Ovidius University of Constanta, Faculty of Constructions, 22b Unirii Str., 900532, Constanta, Romania*
Member of AICPS (Asociatia Inginerilor Proiectanti de Structuri din Romania)

(Accepted 15 November 2013; Published online 15 December 2013)

Abstract

This paper presents a comparative study on structural damage indices for new and existing buildings related to the increasing level of peak ground acceleration. The modeling is based on reinforced concrete plane frame structures, subjected to different levels of PGA. The behaviour of the reinforced concrete elements is defined by a hysteretic moment -curvature model. The damage indices are obtained through time-history analyses for each of the considered types of frames by using modified Park-Ang damage index relationship. The ground motion is modeled through distinctive sets of artificial accelerograms used in the time history analyses, for each of the nine different levels of PGA considered. The time history analysis is performed in IDARC [1]. The output consists of: graphs of global damage indices versus different PGA levels, beam & column time-history damage indices, story-hysteretic curves, displacement versus PGA level, comparison of damage indices of new and existing structures, determination of direct mathematical formulas for damage index estimation function of the required PGA level for new and existing structures with various height regimes.

Rezumat

Acest articol prezintă un studiu comparativ asupra indicilor de avariere a clădirilor noi şi existente în relaţie cu variaţia crescătoare a nivelului acceleraţiei de vârf a terenului. Modelarea este realizată pe structuri în cadre plane, supuse la diferite niveluri ale PGA. Comportarea elementelor de beton armat este definită de un model histeretic moment-curbură. Indicii de avarie sunt obţinuţi în urma analizelor time-history pentru fiecare din tipurile de cadre considerate cu utilizarea indicilor Park-Ang modificaţi. Mişcarea seismică este definită de seturi distincte de accelerograme artificiale utilizate în analizele time-history, pentru fiecare din cele nouă niveluri ale PGA considerate. Analizele dinamice neliniare sunt efectuate în IDARC [1]. Rezultatele constau în: grafice cu variaţia indicilor de avarie funcţie de nivelul PGA, variaţia în timp a indicilor de avarie pe grinzi şi stâlpi, curbe histeretice de nivel, deplasări faţă de nivelul PGA, comparaţii între indicii de avariere obţinuţi pe clădiri noi faţă de avariile celor existente, formule matematice directe de estimare a indicilor de avarie pe diferite regimuri de înălţime la clădiri noi şi existente funcţie de nivelul PGA de interes.

Keywords: damage index, artificial accelerogram, time-history, idarc, hysteresis, seismic motion

* Corresponding author: Tel./ Fax.: +4 0722 305 648
E-mail address: claudiutudose@yahoo.com

1. Introduction

The paper presents a comparative study for damage evaluation of new and existing structures. The defined structures are newly designed reinforced concrete frames with 4, 6 and 8 stories high and further comparison to an existing 4 storied frame structure. The new structures will be first designed according to the Romanian seismic code (P100-1/2006 [2]). The existing structure had been previously designed by the 1963 seismic code and is used on a "as is" basis. All the structures will be subjected to various levels of peak ground acceleration which are defined by several artificial accelerograms. The damage indices are finally obtained through time-history analysis for each of the considered structures.

2. Objectives of the paper

In brief, the objectives of the study are: 1. To put into evidence the seismic response parameters of reinforced concrete frames, expressed by the design spectra and by artificial generated accelerograms; 2. To evaluate the seismic response of RC frames through damage indices using time-history analyses and nonlinear behavior models for the component elements of the structures; 3.To put into evidence the variation of damage indices for new and existing structures function of the peak ground acceleration level, to establish a variation pattern and to obtain adequate formulas for determination of damage indices function of the desired PGA level.

3. Analysis procedure

The analysis is split into new and existing structures. The new structures being analyzed are medium to high rise frames, up to 8 stories. The existing structure is a 4 storied, pre-1977 designed structure, built in 1977-1978, who has complete information available about the structural frame dimensions and their reinforcement. The story regime was limited to the level of applicability of the frames structures, taking into account that for very high rise buildings used in high seismic areas other solutions such as mixed frames and shear walls, steel structures or mixed steel-concrete solutions represent a more advantageous solution.

3.1. Structural model – new structures

The structures considered in analysis are reinforced concrete frames with 4, 6 and 8 stories, with 5 spans of 6.00m and story heights of 3.25m. Taking into account the direct interest in seismic action, the loads acting on the frames are considered as long-term loads, with a coefficient of 1.00 for dead loads and 0.40 for live loads. Table 1 presents the types of loads acting on the frames together with their nominal values. (long-term values are specified in the parenthesis, where applicable).

The location is characterized by a dynamic amplification factor $\beta = 2.75$, a corner period $T_C = 1.6\text{sec}$ and a peak ground acceleration of 0.24g, considered for a mean recurring interval of 100 years, according to the provisions of Romanian seismic code P100-1/2006. The location corresponds to the city of Bucharest. The structures have been designed considering a high-class ductility structure, with behavior coefficient $q = 5 \cdot 1.35 = 6.75$. Concrete class is C20/25, the minimum allowed class for high-ductility structures. Linear dynamic analyses were performed in Etabs [3] in order to evaluate the necessary reinforcements. The base shear forces were evaluated according to (Eq. 1) as a percentage of the total weight of the system.

$$F_b = \gamma \cdot S_d(T_1) \cdot m \cdot \lambda = \gamma \cdot \frac{a_g \cdot \beta(T_1)}{q} \cdot m \cdot \lambda = 1 \cdot \frac{0.24 \cdot g \cdot 2.75}{6.75} \cdot m \cdot 0.85 = 0.0831 \cdot m \cdot g \quad (1)$$

The verifications were performed to check for compliance of relative story drifts to the allowed limits imposed by the seismic code: 5‰ for Serviceability Limit State and 2.5% for Ultimate Limit State. Column dimensions were established such as the non-dimensional axial load to be limited to 0.4. The effective reinforcement areas chosen for the beams do not exceed with more than 10% the necessary reinforcement areas. The columns are designed to take into account the 10% specified above and the capacity of beams at yielding. The final dimensions of the elements and their reinforcement areas are specified in Table 2.

Load type	**Nominal value (kN/sq.m)**	**Uniformly distributed loads on beams (kN/m)**
Concrete slab	3.75	22.50
Slab finishing	0.92	5.52
Live load	4.00 (1.60)	18.00 (9.60)
Masonry cladding	2.59	6.73

Table 1. Loads acting on the frames

	Story level	**Columns**		**Beams**		
4 stories		Dimensions (bxb in mm)	Side reinf. (sq. mm)	Dimensions (bxh in mm)	Top reinf. (sq. mm)	Bottom reinf. (sq. mm)
	1,2,3	500 x 500	1256	300 x 650	1457	716
	4	500 x 500	1256	300 x 650	1256	603
6 stories	**Story level**	**Columns**		**Beams**		
		Dimensions (bxb in mm)	Side reinforcement (sq. mm)	Dimensions (bxh in mm)	Top reinforcement (sq. mm)	Bottom reinf. (sq. mm)
	1,2,3	600 x 600	1571	300 x 700	1963	716
	4,5	600 x 600	1571	300 x 700	1610	603
	6	600 x 600	1571	300 x 700	1119	603
8 stories	**Story level**	**Columns**		**Beams**		
		Dimensions (bxb in mm)	Side reinforcement (sq. mm)	Dimensions (bxh in mm)	Top reinforcement (sq. mm)	Bottom reinf. (sq. mm)
	1,2,3,4,5	750 x 750	1571	300 x 750	2238	942
	6,7	750 x 750	1571	300 x 750	1610	603
	8	750 x 750	1571	300 x 750	1030	513

Table 2. Elements dimensions and effective reinforcement areas

3.2. Model consistency checking – new structures

Following the design of the three types of frames, the level of confidence was needed to be established by performing a pushover analysis in Etabs Nonlinear. During the verification, special attention was given to the avoidance of cases that lead to unwanted situations that could appear through design errors. So, checking involved the order of development of plastic hinges on beams and columns, favorable to the energy dissipating mechanism while avoiding soft-story behavior. In the end, the force-displacement graphs are plotted (Fig. 1).

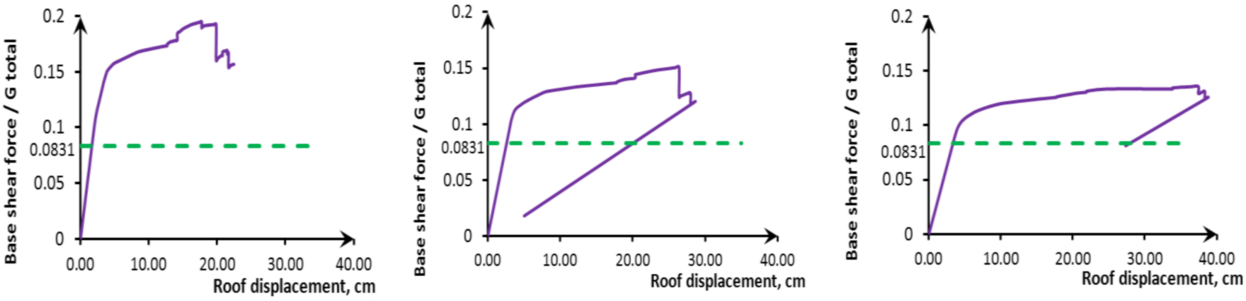


Fig 1. Pushover curve expressed as base shear force/structure weight versus roof displacement for the structures with 4, 6 and 8 stories (from left to right)

3.3. Structural model – existing structure

The engineering analyses performed for new structures have put into evidence a series of particularities which are commented in the next chapter. It is interesting to use the same algorithm for existing structure, which were designed by previous codes. In order to do this, the data from a structure built in 1977-1978 was used.

The structure is a reinforced concrete frame, with 2 bays of 5.40m and 11 spans of 3.00m, with basement, ground floor and 3 upper stories. The story height is 3.30m, close to the value used for the new structures in chapter 3.1. The structure will be considered fixed above the basement, using only the stories above the ground in the analysis. The initial destination of the building is Emergency and Surgery rooms at Brăila Emergency County Hospital.

According to P100-1/2006, the importance class is I, but this aspect was not considered in the analysis, in order to keep the analogy with chapter 3.1. The structure was analyzed as a structure of importance class III (regular). According to the provisions of P100-1/2006, the site is characterized by a dynamic amplification factor $\beta = 2.75$, corner period $T_C = 1.0\sec$ and maximum PGA value of 0.24g, for a mean recurrence interval of 100 years.

Only the central longitudinal frame was analyzed (Fig.2 and Fig.3), taking into account just five of the total number of spans, without considering the interaction effect between the masonry and the concrete frames. The fundamental period of the structure is in the constant acceleration area of the spectrum and is not close to the corner period. The structure in not susceptible of resonance. The whole ensemble of the hospital was initially dimensioned according to the P13/70 code, but no written information was recovered with concern to the initial seismic coefficient.

The concrete is of class B250 (C16/20) in the prefabricated beams and B200 (C12/15) in the columns, which today would assign the structure to medium ductility class from the point of view of the performance of the component materials (modulus of elasticity, strength, consistency). The evaluation of the seismic force as percentage of the total weight of the system by using the relation (6-1) from P100-1/2006 gives the following result:

$$F_b = \gamma \cdot \frac{a_g \cdot \beta(T_1)}{q} \cdot m \cdot \lambda = 1 \cdot \frac{0.24 \cdot g \cdot 2.75}{4.725} \cdot m \cdot 0.85 = 0.119 \cdot m \cdot g \qquad (2)$$

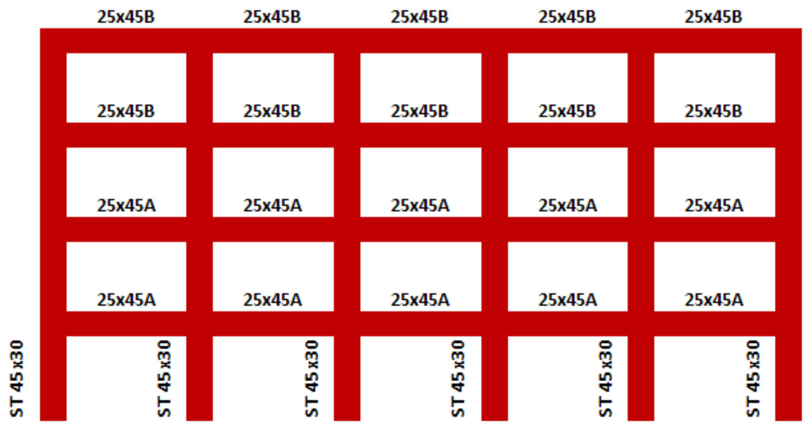


Fig.2. Dimensions of the frame elements, existing 4 storied structure

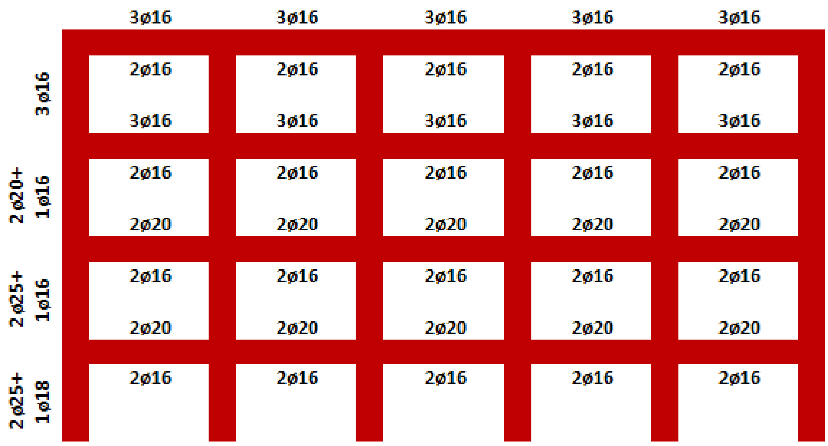


Fig.3. Reinforcement of the beams (total) and columns (side reinf.), existing 4 storied structure

3.4. Model consistency checking – existing structure

The structural capacity of the existing structure was established as for the new ones. The force-displacement graph is plotted in Fig.4.

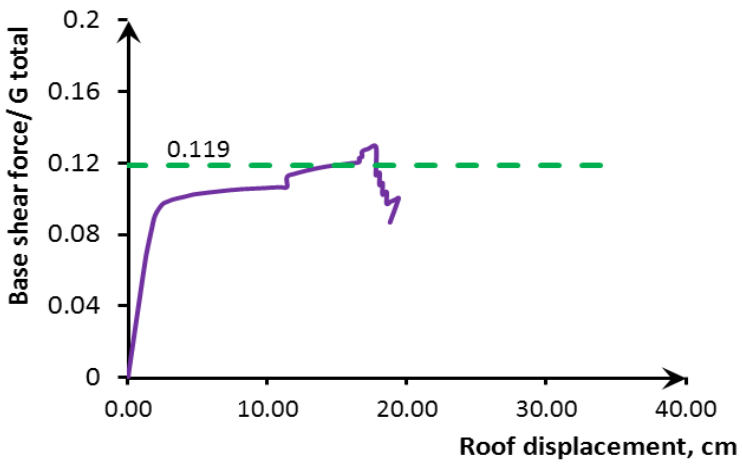


Fig 4. Pushover curve expressed as base shear force/structure weight versus roof displacement for the existing structure - 4 stories

From the point of view of the P100-1/2006 code provisions, the structure has not enough capacity to undertake the current design lateral forces. Taking into account that the structure was designed from the start as an emergency hospital, the strength reserve given by the importance class (introduced as an increased value of k_s, according to P13/70) is unable to cover the force differences between the editions of the codes.

Given the fact that the structure is flexible and with a larger fundamental period that the corner period from P13-70 code, it is very probable that the design was performed at even a lower β value. If during the initial design, the maximum β value of 2,00 would have been used, then the resulting seismic force would be 0,136G, which represents approx. 81% of the seismic coefficient of the actual P100/2006 code, which would have changed the terms of the discussion. Unfortunately, the edition of 1970 reduced the value of dynamic amplification factor from 3 to 2, despite the increment in corner period from 0,3 to 0,4sec. Further editions of the code corrected the control period to higher values, specific to Vrancea earthquakes and closed to the recorded ones. The elastic capacity determined for the existing structure is approx. 0,097G, which is a surprisingly high value, but it won't be found in the structures designed for normal class (III) of importance based on the 1970 code.

The static linear analysis performed gives the relative story displacement values in Tables 3 and 4. One may notice the excessive displacements both in SLS and ULS, even if the masonry walls will have major degradation well before the 8‰ limit of SLS. It is worth mentioning that the structure is considered as a class III structure.

Table 3. Relative story displacements at serviceability limit state, existing 4 story structure

Story	**Direction**	**Load case**	**Point coordinates**			**Drift X**	**Drift SLS (in ‰)**
			X	**Y**	**Z**		
3rd floor	Max Drift X	ENVEALL	0	5.4	13.2	0.001295	4.31
2nd floor	Max Drift X	ENVEALL	6	5.4	9.9	0.002335	7.78
1st floor	Max Drift X	ENVEALL	15	5.4	6.6	0.003057	10.18
Base floor	Max Drift X	ENVEALL	6	5.4	3.3	0.002844	9.47

Table 4. Relative story displacements at ultimate limit state, existing 4 story structure

Story	**Direction**	**Load case**	**Point coordinates**			**Drift X**	**Drift ULS (in %)**
			X	**Y**	**Z**		
3rd floor	Max Drift X	ENVEALL	0	5.4	13.2	0.001295	1.22
2nd floor	Max Drift X	ENVEALL	6	5.4	9.9	0.002335	2.21
1st floor	Max Drift X	ENVEALL	15	5.4	6.6	0.003057	2.89
Base floor	Max Drift X	ENVEALL	6	5.4	3.3	0.002844	2.69

3.5. Modeling of the seismic action for the nonlinear dynamic analyses

In order to precisely evaluate the damage occurred at elements level, at story or global level, we must use the nonlinear dynamic analysis. Using this kind of analysis means to define the seismic action as time dependent. The different intensities considered in analysis are: 0,08g, 0,12g, 0,16g, 0,20g, 0,24g, 0,28g, 0,32g, 0,36g and 0,40g, where g=gravity acceleration. For each of these PGA values, the elastic spectrum with 1,6sec time period was considered, multiplied by the PGA value, obtaining absolute acceleration spectra (Fig.5 presents two examples). The critical damping is 5%.

For each of the nine accelerations spectra, a set of seven artificial accelerograms was generated. The artificial accelerograms fulfill the requirements of P100-1/2006, chapter 3.1.2. The seismic code specifies that if using at least seven ground motions compatible with the design spectra during the nonlinear dynamic response analysis, the displacements and the deformations can be computed with the mean values of individual response values. Using Seismoartif [4] software, for each of the nine absolute accelerations elastic spectra were generated several artificial accelerograms with a total duration of 20 seconds. From these, only seven that complied with the seismic code were selected. In the end, a total of nine sets of seven accelerograms were obtained (examples in Fig.7). The intensity envelope used for the generation of the artificial accelerograms was Saragoni & Hart [5], characterized by three values (Fig.6, left): total duration (in seconds), time corresponding to maximum intensity and the intensity at the end of accelerogram as percentage of maximum. The considered values are: total time of 20 seconds, maximum intensity at t=6seconds, 5% intensity at the end of the event. The maximum intensity at t=6sec corresponds to the maximum intensity of the 4th of March 1977 earthquake, based on recording performed by Incerc station in Bucharest (Fig.6, right).

The verification consisted of a plotted graph containing all response spectra, for each of the PGA levels, with the target spectrum, the calculated mean spectrum as the mean of the spectral values of the artificial accelerograms for each of nine elastic acceleration spectra, and the imposed ±10% spectrum limits needed to be obeyed by the spectrum of the generated accelerograms. The elastic spectra used are: Se(t) with 1,1g (Fig.8), 0,99g, 0,88g, 0,77g, 0,66g, 0,55g, 0,44g, 0,33g and 0,22g.

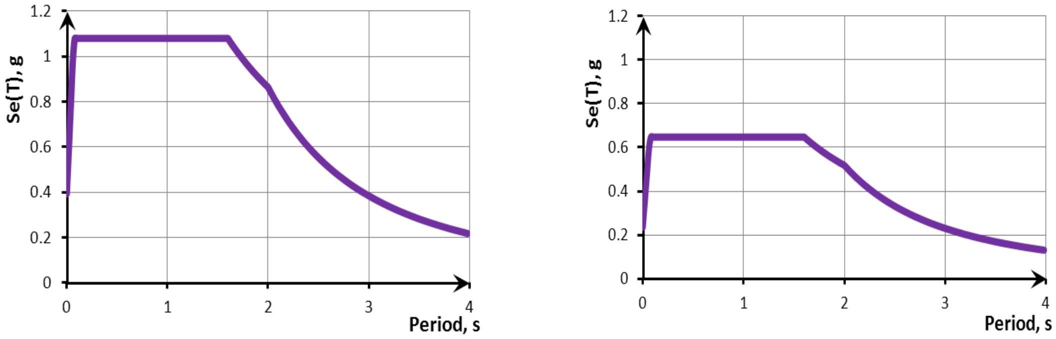


Fig 5. Example of 2 of the 9 absolute acceleration response spectra, with corner period of 1,6sec and PGA's of 0,40g, 0,24g and 0,08g (from left to right)

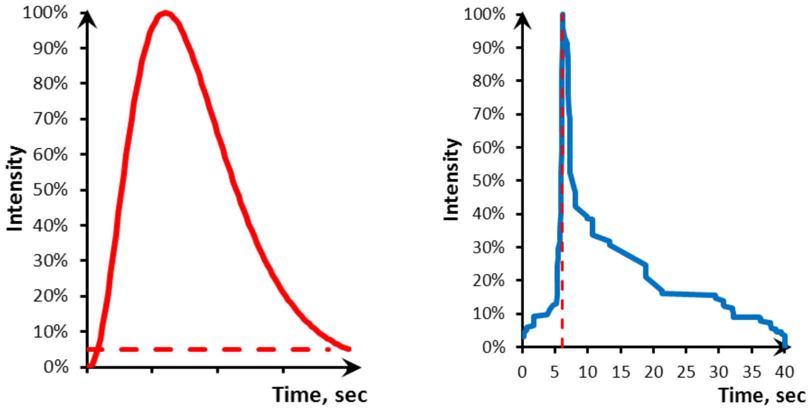


Fig 6. Saragoni-Hart envelope used for generation of artificial accelerograms (left) and the envelope of the 4th of March 1977 earthquake, recorded by the Incerc station, NS component (right)

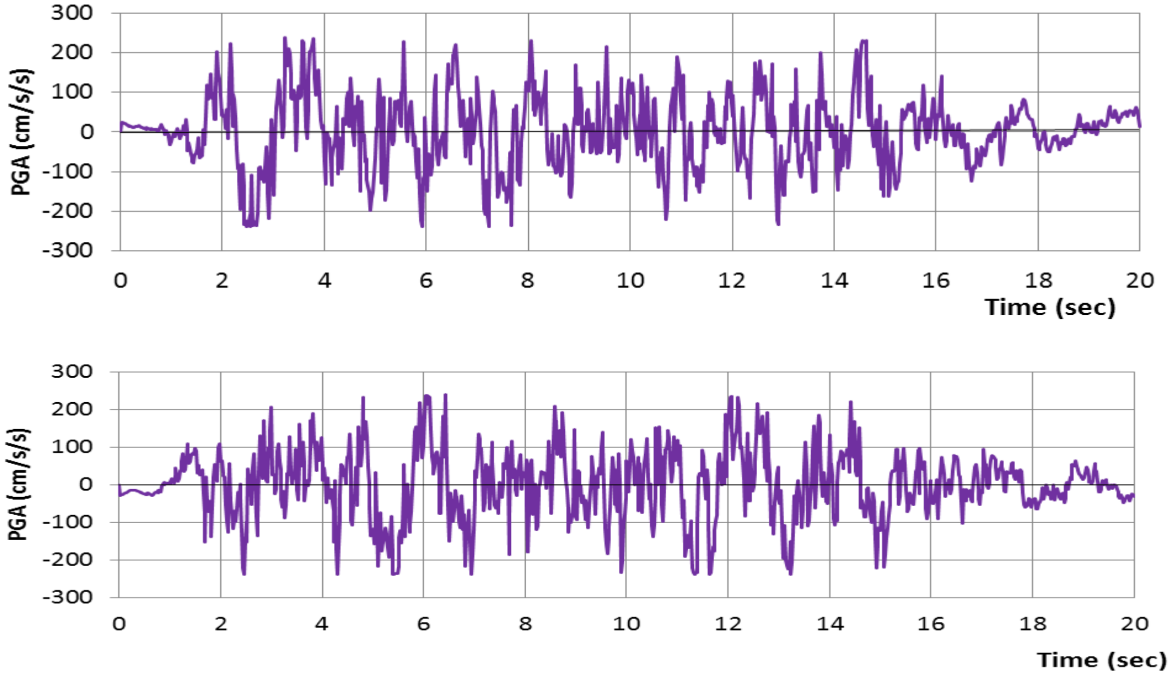


Fig. 7. Example of two of the seven artificial accelerograms within the set with PGA=240 cm/s/s.

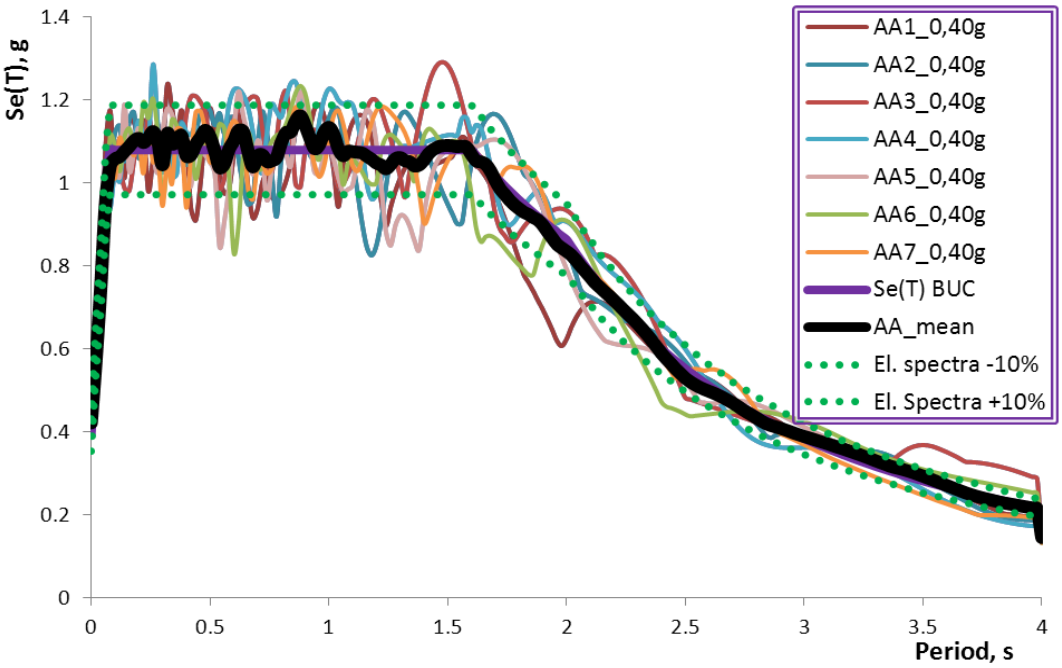


Fig. 8. Graph of the artificial accelerogram absolute elastic spectra, their mean spectrum and targeted spectrum and their ±10% mandatory limit (example for Se(t) = 1,1g)

3.6. Time-history analyses – new and existing structures

For the nonlinear dynamic analysis, all the types of buildings were subjected to increasing levels of PGA, each level being defined by a set of seven artificial accelerograms. A total of 189 programming files were written for IDARC 2D analysis for the new structures and 63 files for the existing structure. Each of them contains approximately 170 lines of code. Each file corresponds to one dynamic nonlinear analysis performed.

The characteristics of the used materials are input in the software and the program automatically adjusts the cracked element stiffness for the nonlinear study. The analyses were performed for Ultimate Limit State. The hysteretic response of the structural elements is considered by using the tri-parameter modified Park-Ang-Wen model. This hysteretic model includes stiffness degradation, strength degradation and slip and furnishes a tri-linear monotonic envelope. The model monitors the hysteretic behavior of an element while it modifies from one linear stage to another, function of the deformation history. The model is linear on each phase. The tri-linear hysteretic model is based on four parameters which scale the models principal characteristics (in fact there are only three parameters, two of them representing strength degradation but by two different methods): HC - stiffness degrading parameter; HBD - strength degrading parameter, ductility based; HBE - strength degrading parameter, energy controlled; HS - slip or crack closing parameter. During the analyses, the values used are HC=2, HBD=0,01 (default-not considered), HBE=0,10, HS=1 (default), values that allow stiffness degradation, strength degradation based on energy and no pinching effects.

Determination of element damage index is evaluated according to Eq. 3:

$$DI_{PAW} = \frac{\phi_{\max} - \phi_y}{\phi_{ult} - \phi_y} + \frac{\beta}{M_y \phi_{ult}} \int dE_h \qquad \text{(Eq. 3)}$$

where, $\phi_{\max}$ = maximum curvature attained during the accelerogram, ϕ_y = yielding curvature of the section, ϕ_{ult} = ultimate curvature of the section, β = strength degrading parameter (HBE mentioned above), M_y = yielding moment of the section, E=dissipated hysteretic energy. The story damage index and the global damage index are calculated as a mean of the dissipated energy of the elements and the stories, respectively. The relation of the damages index to the structure condition may be taken from Table 5. Figures 9a and 9b present an example of the obtained hysteretic time-story curves, expressed as displacement versus story shear for two accelerograms with PGA=0,40g for the 8 stories structure (accelerograms 3 and 7).

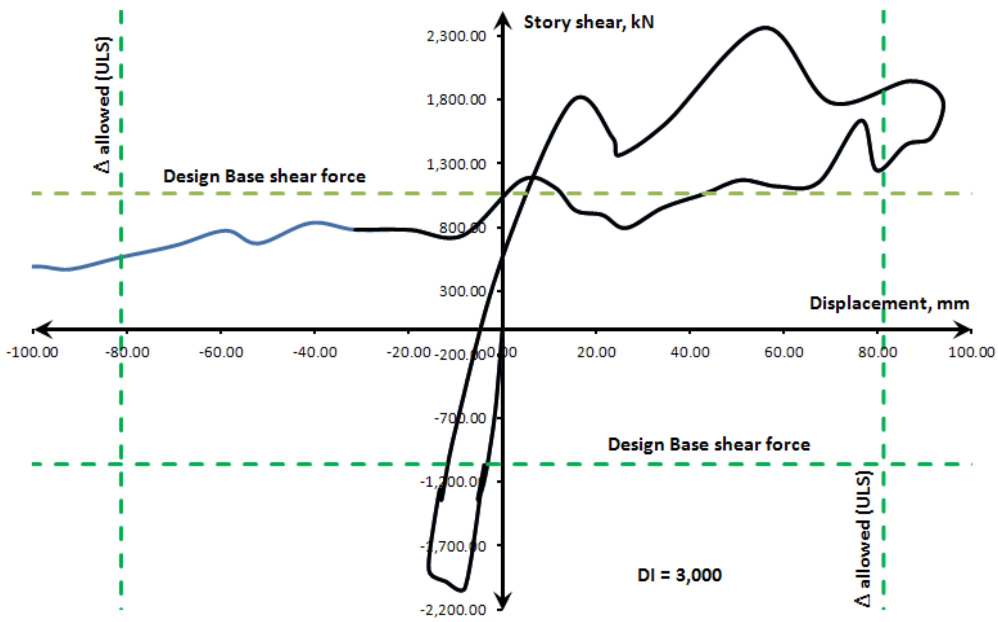


Fig. 9a First story hysteretic curve, 8 storied frame, artificial accelerogram no.3 for PGA=0,40g

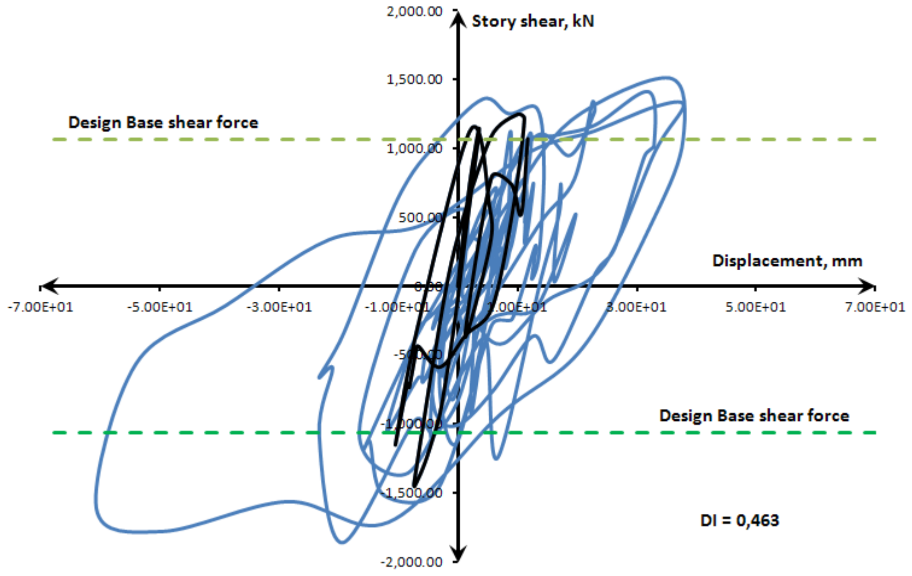


Fig. 9b First story hysteretic curve, 8 storied frame, artificial accelerogram no.7 for PGA=0,40g

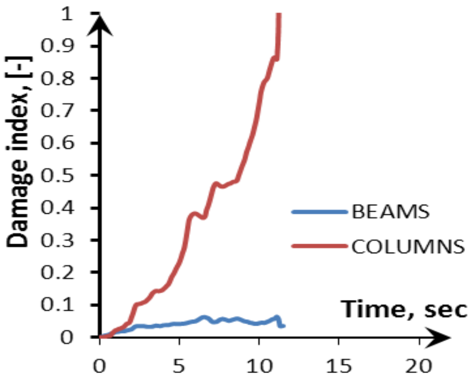
Table 5. Damage index levels and physical state corresponding to the damage

Damage index value	Damage level	Physical state of the damage
0,0 … 0,1	None	No damage or with local minor concrete cracks
0,1 … 0,2	Minor	Reduced concrete cracks
0,2 … 0,5	Moderate	Large concrete cracks, local concrete spalling
0,5 … 1,0	Severe	Crushing of concrete, reinforcement is exposed
over 1,0	Collapse	Structural collapse

4. Results of the dynamic nonlinear analyses

4.1. Results – New structures

Figures 10, 11 and 12 present the time-histories of damage indices obtained for beams and columns, for new frame structures with 4, 6 and 8 stories, for PGA's of 0,40g and 0,24g. The values are computed as means of the individual damage indices obtained from artificial accelerogram sets, for the first story above the ground and for the top story of the structures.



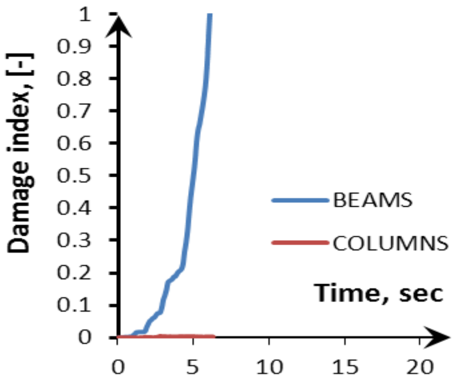


Fig.10a. Damage index time-history, 8 storied frame for first storey (left) and top storey (right) at 0,40g

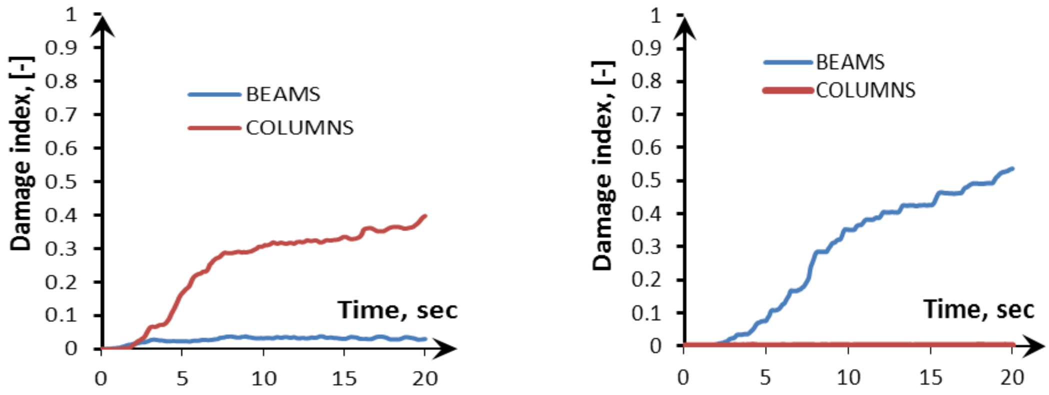


Fig.10b. Damage index time-history, 8 storied frame for first storey (left) and top storey (right) at 0,24g

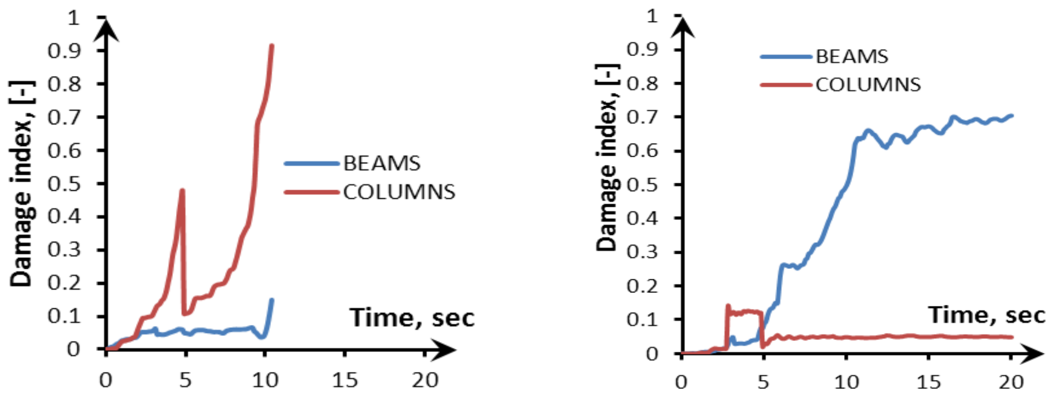


Fig.11a. Damage index time-history, 6 storied frame for first storey (left) and top storey (right) at 0,40g

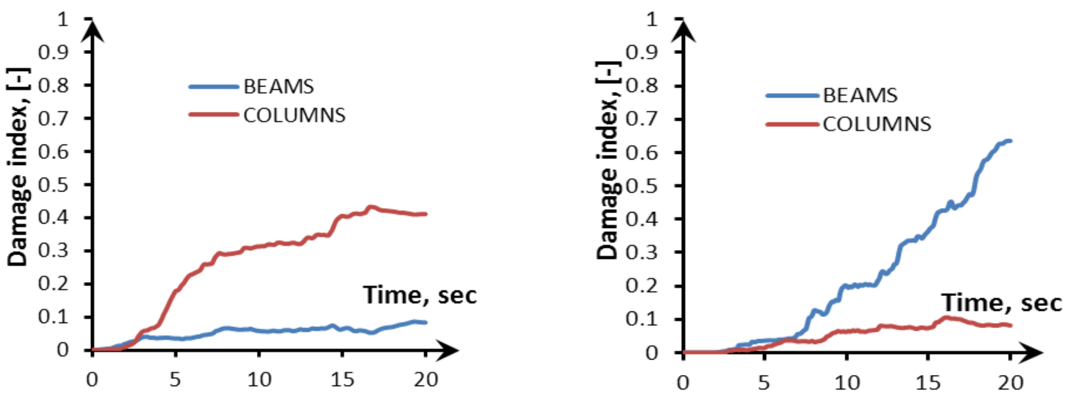


Fig.11b. Damage index time-history, 6 storied frame for first storey (left) and top storey (right) at 0,24g

For all the analyses which returned high values of damage index which are not realistic, an upper limit of 3 was imposed. The sudden decreases of some graphs indicate that one of the iterations ended and the continuation of the graph is realized with the remaining valid converged iterations.

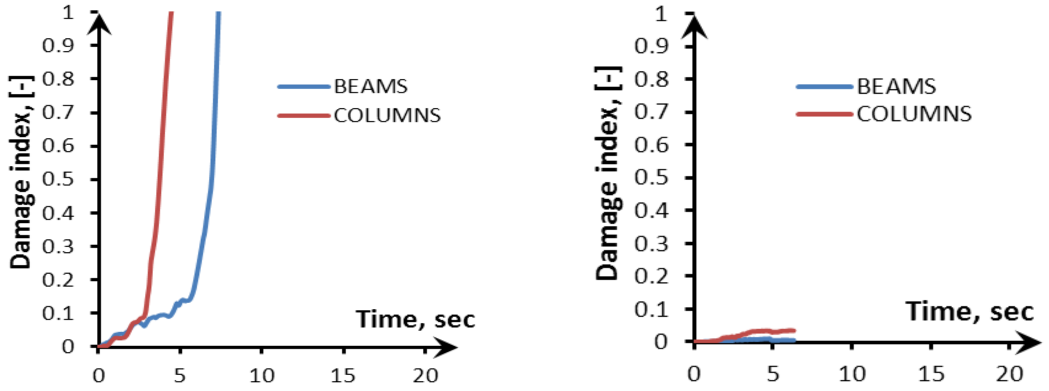


Fig.12a. Damage index time-history, 4 storied frame for first storey (left) and top storey (right) at 0,40g

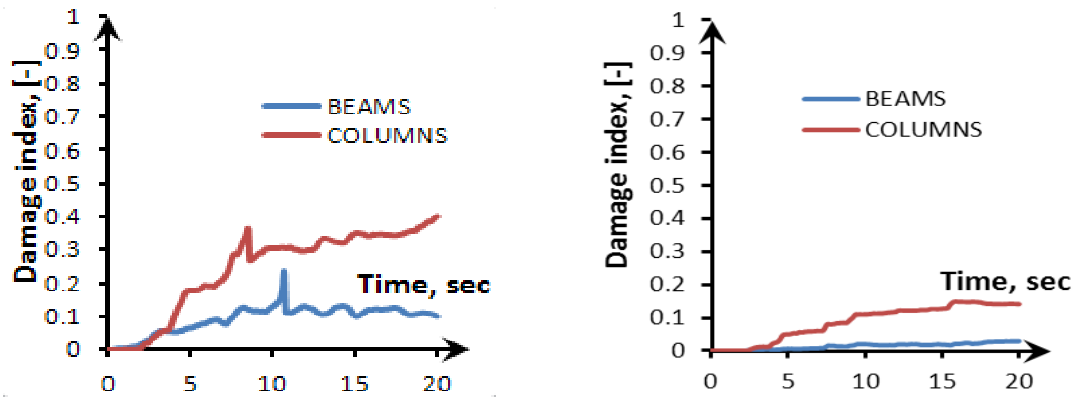


Fig.12b. Damage index time-history, 4 storied frame for first storey (left) and top storey (right) at 0,24g

After the analysis in IDARC, the global damage indices were obtained, function of the distinctive levels of the seismic action intensity. For each value of the intensity, the beam and column damage indices were determined as the mean of the individual values from various accelerograms.

The Figure 13 (a,b,c) presents the PGA versus damage index graphs and the obtained mathematical formulas for determination of the global damage index function of the PGA value.

The Figure 14 (a,b,c) presents the scattering of the individual values of the global damage index.

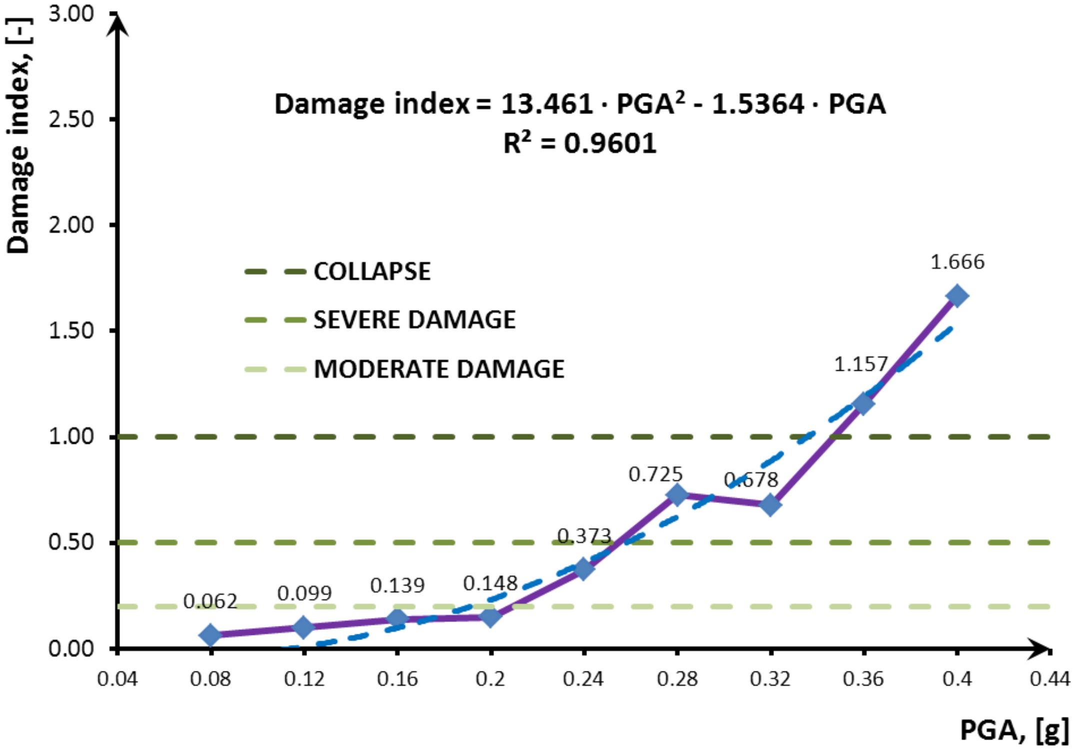


Fig. 13.a – Variation of mean global damage index versus PGA level and the mathematical relationship of damage index to the PGA level for the 8 storied new structures

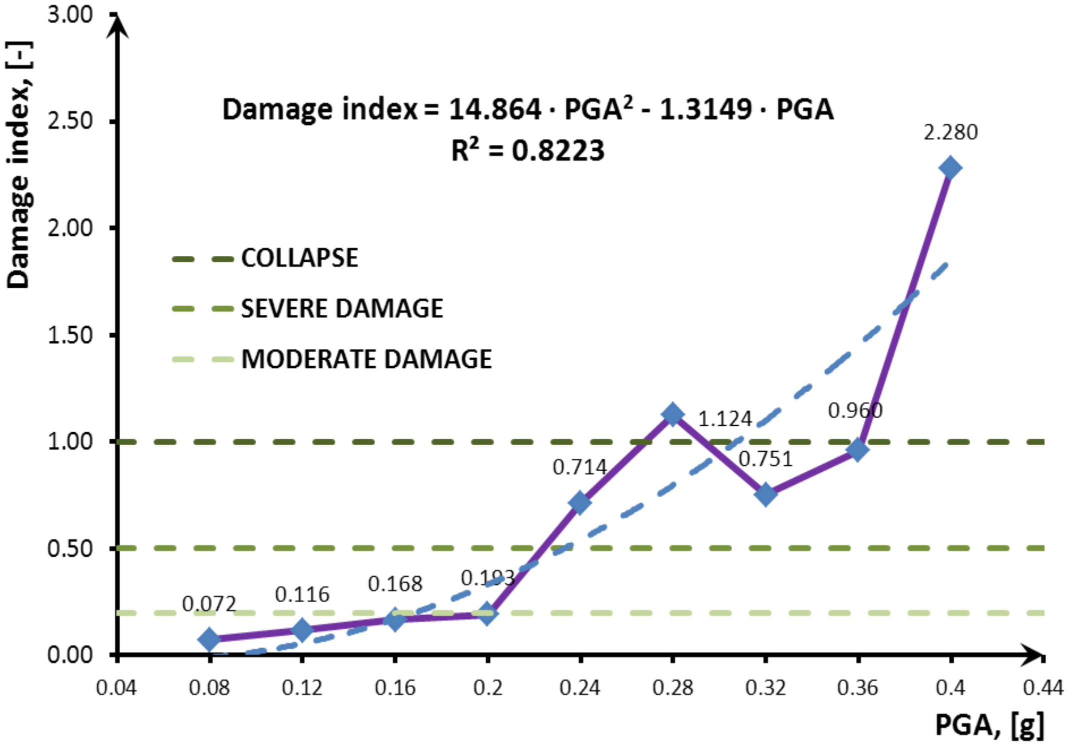


Fig. 13.b – Variation of mean global damage index versus PGA level and the mathematical relationship of damage index to the PGA level for the 6 storied new structures

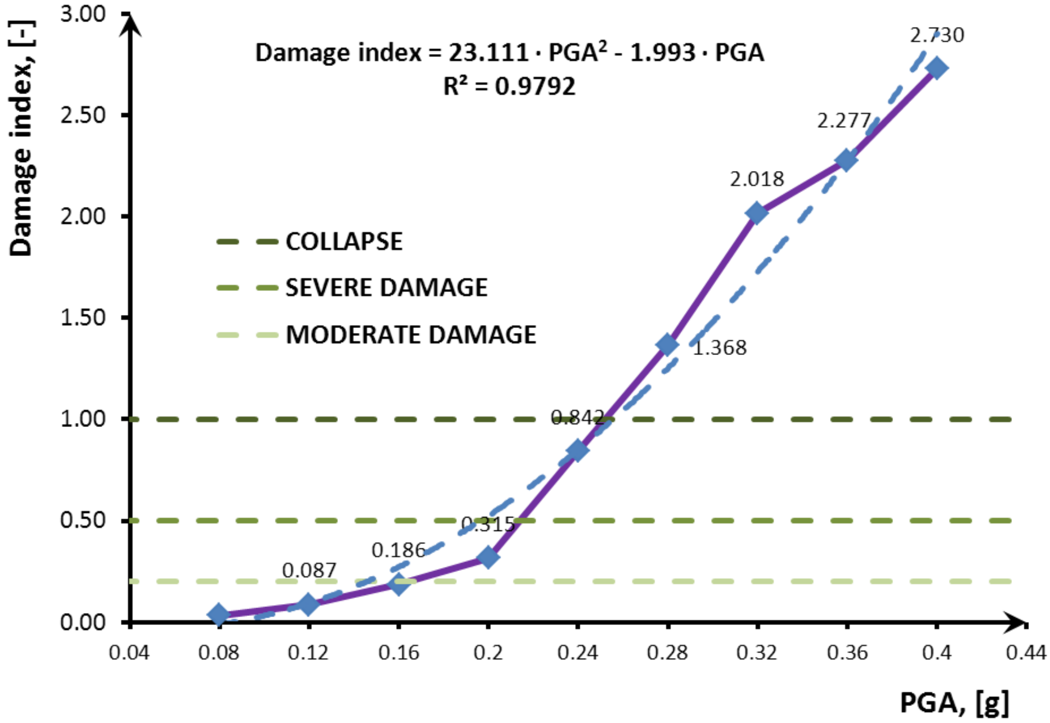


Fig. 13.c – Variation of mean global damage index versus PGA level and the mathematical relationship of damage index to the PGA level for the 4 storied new structures

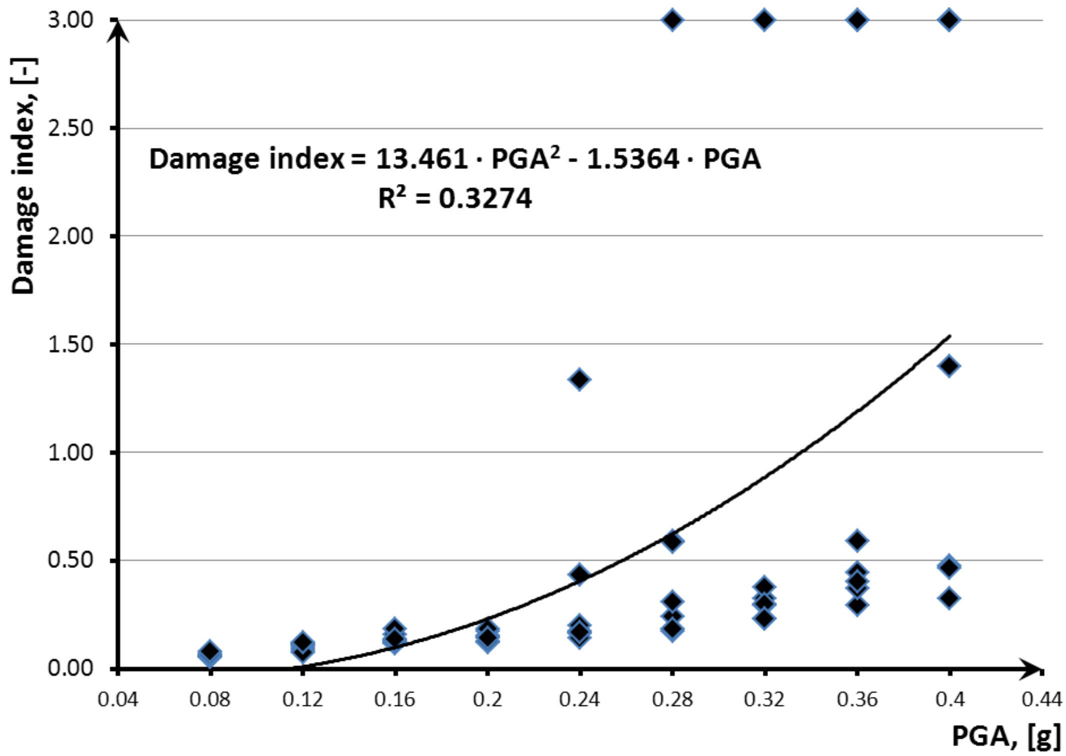


Fig. 14.a – Scattering of the global damage index values versus PGA level for the 8 storied new structure

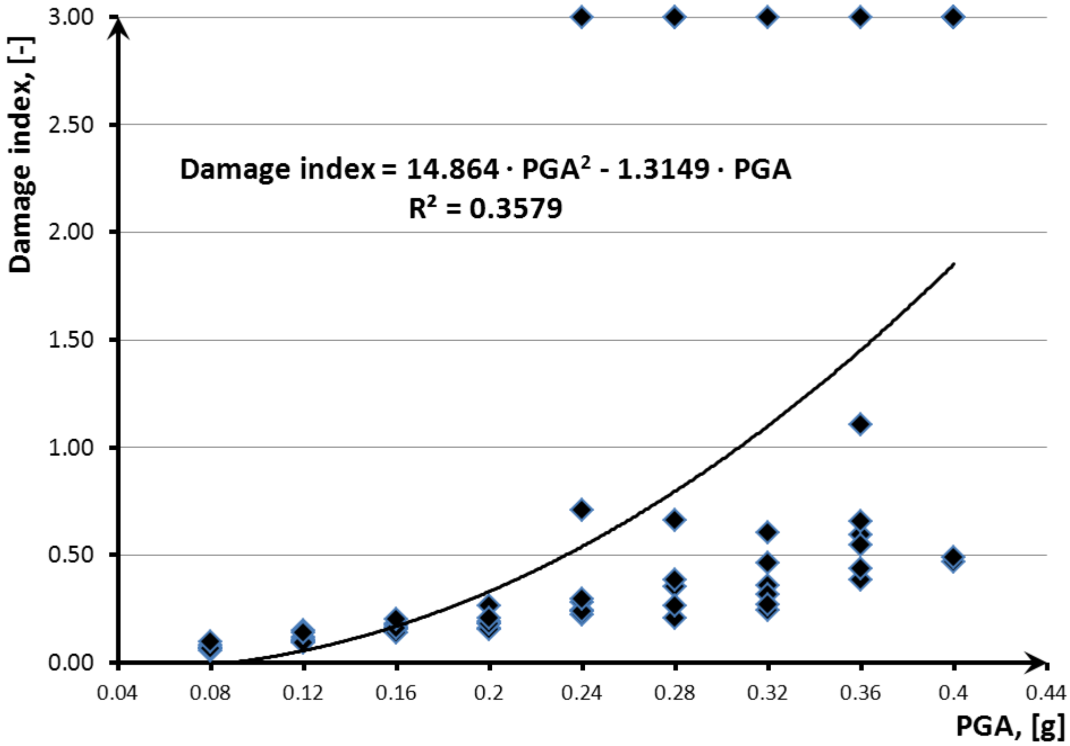


Fig. 14.b – Scattering of the global damage index values versus PGA level for the 6 storied new structure

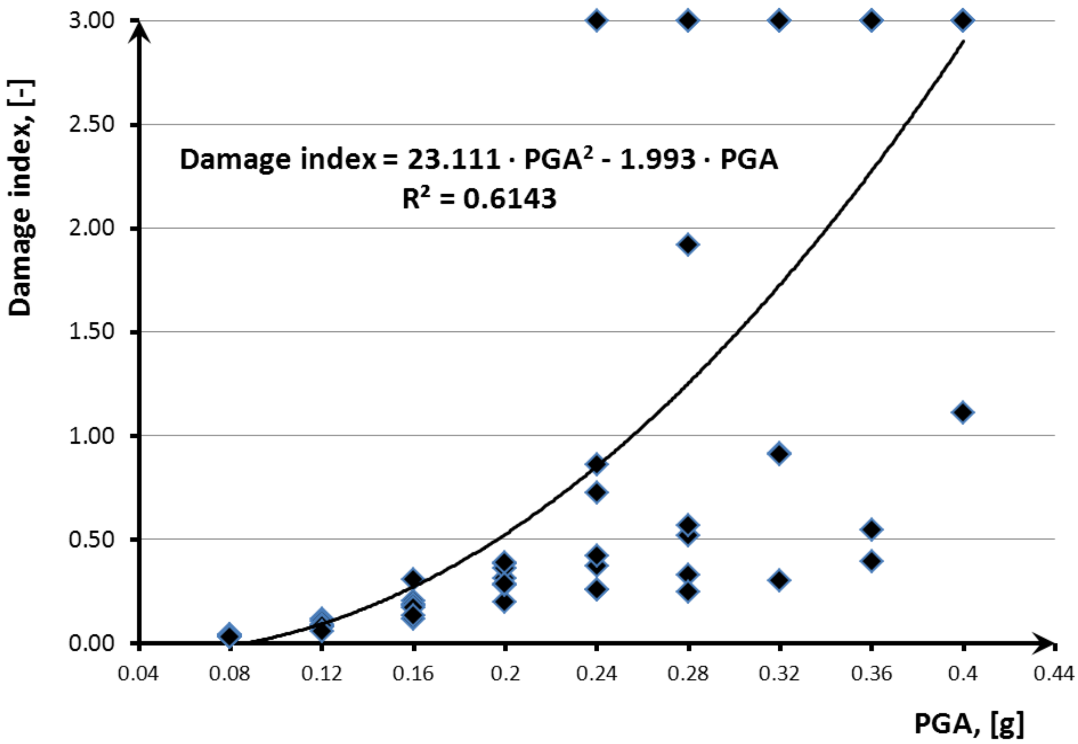


Fig. 14.c – Scattering of the global damage index values versus PGA level for the 6 storied new structure

The maximum story response expressed as an absolute displacement function of the PGA value for the new structures is presented in the following graphs (Fig. 15, 16, 17):

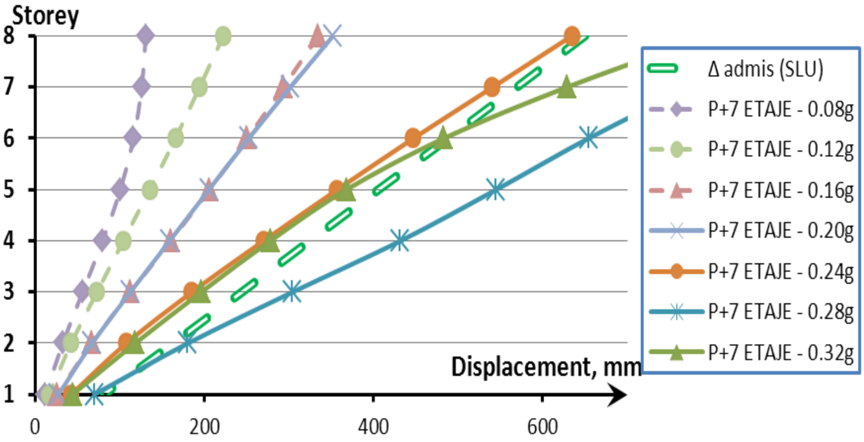


Fig. 15. Maximum story response, expressed as absolute displacements function of PGA level, determined as the mean of the level responses of each set of 7 artificial accelerograms, 8 storied new structure

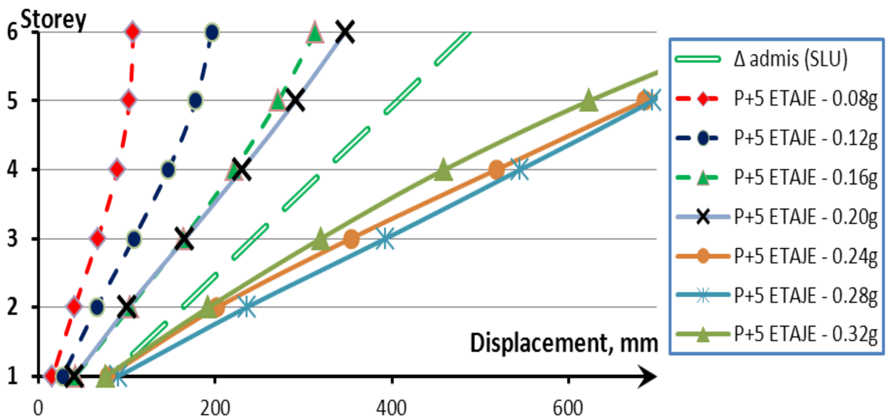


Fig. 16. Maximum story response, expressed as absolute displacements function of PGA level, determined as the mean of the level responses of each set of 7 artificial accelerograms, 6 storied new structure

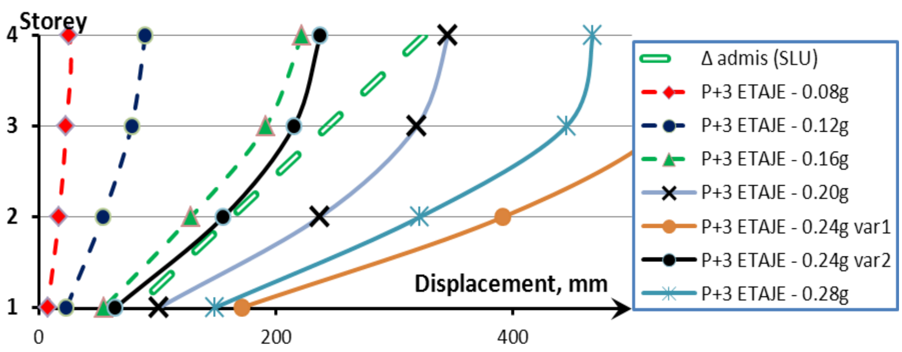


Fig. 17. Maximum story response, expressed as absolute displacements function of PGA level, determined as the mean of the level responses of each set of 7 artificial accelerograms, 4 storied new structure

4.2. Results – Existing structure

The nonlinear dynamic analysis uses the same artificial accelerograms defined previously in chapter 3.5. The existing structure will be subjected to increasing level of PGA, each PGA level being defined by a set of 7 artificial accelerograms.

Next, the graph with the maximum displacement response was drawn, as the mean of the results in each set of accelerograms. The maximum inelastic displacements recorded during the time-history analysis are presented in Fig. 18, for each of the considered intensity value.

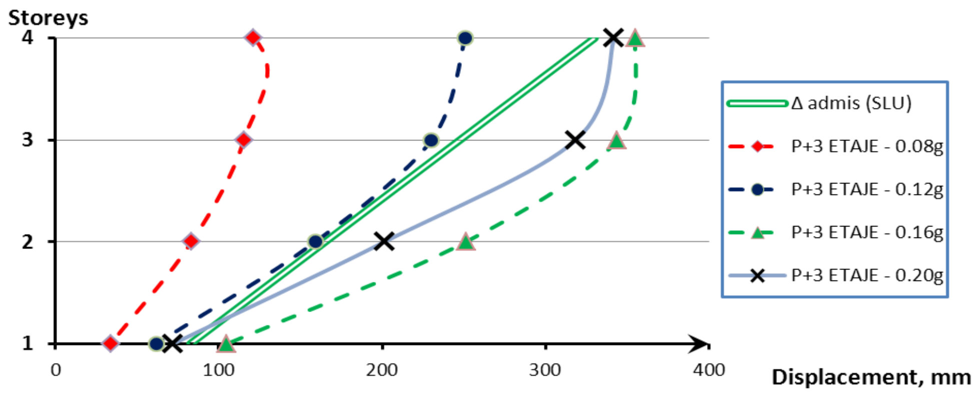


Fig. 18. Maximum story response, expressed as absolute displacements function of PGA level, determined as the mean of the level responses of each set of 7 artificial accelerograms, 4 storied existing structure

Following the dynamic nonlinear analysis, the evolution of damage indices on columns and beams during the seismic action was obtained (Fig. 19.a and Fig. 19.b), by plotting the index values over time. The graphs present the mean of the values obtained by processing the artificial accelerograms.

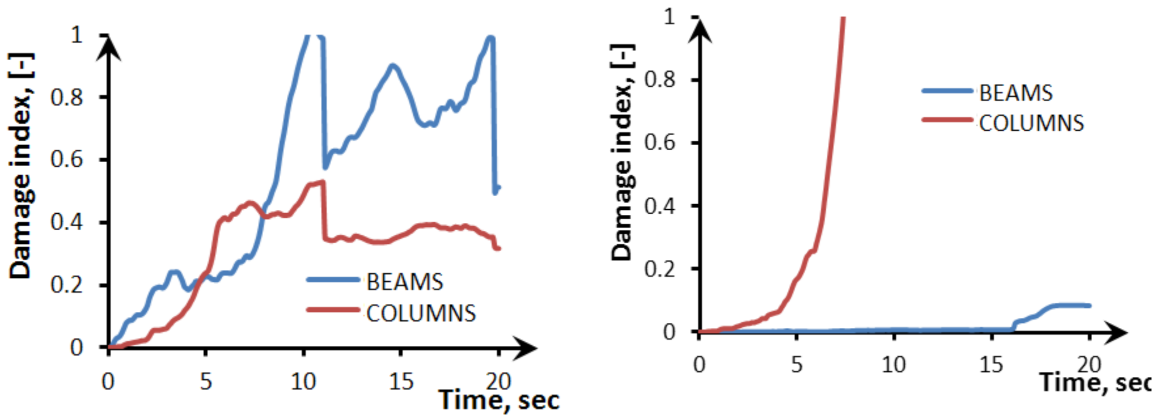


Fig.19.a. Damage index time-history, 4 storied frame for first storey (left) and top storey (right) at 0,40g

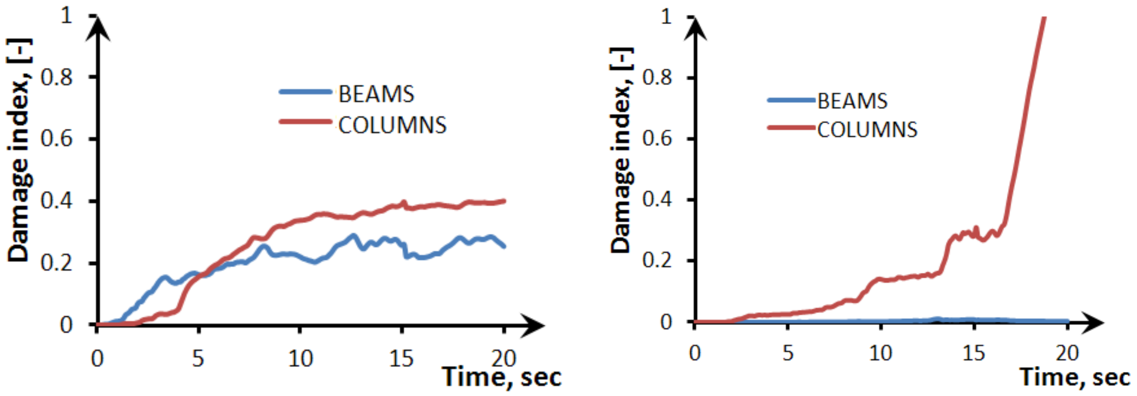


Fig.19.b. Damage index time-history, 4 storied frame
for first storey (left) and top storey (right) at 0,24g

It can be noticed that the structure has low capacity in the columns of the upper story, both for design values (0,24g) and for rare values (0,40g), fact that was also highlighted by the pushover analysis, where the upper story fails due to soft story mechanism which may be dangerous as it could lead to story pancaking.

The structure is noticed to lack capacity reserve on the stories that show the highest story drifts (especially the second story). For 0,24g intensity the structure fails 7 to 10 seconds after the earthquake start-up while for the 0,40g intensity the result is the same but it occurs earlier, 4 to 7 seconds after earthquake initiation.

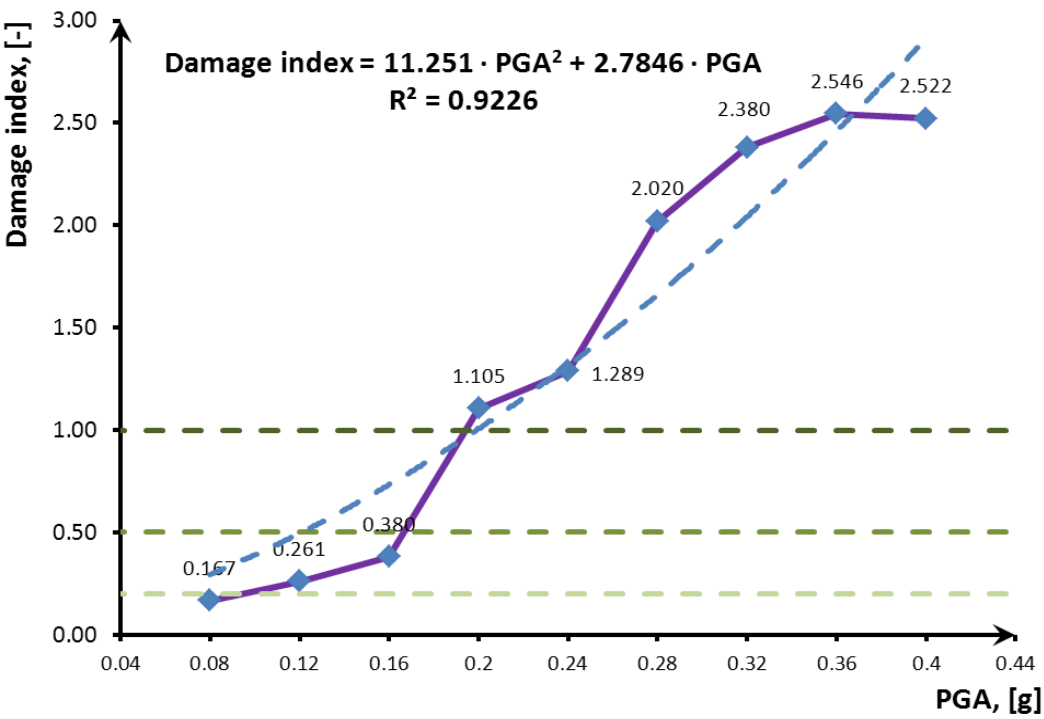


Fig. 20 – Variation of mean global damage index versus PGA level and the mathematical relationship of damage index to the PGA level for the 4 storied existing structure

The values of the damage indices show in Fig. 20 are computed as the mean of the 7 individual damage values for each of the seismic motion intensity, similar to the procedure applied for new structures. The adopted analysis model allowed the plotting of the mean global damage indices correlated to the seismic motion intensity acting on the structure (Fig. 20).

Beyond this, Fig. 21 presents the scattering of the individual values of the damage indices on the whole interval of considered PGA values.

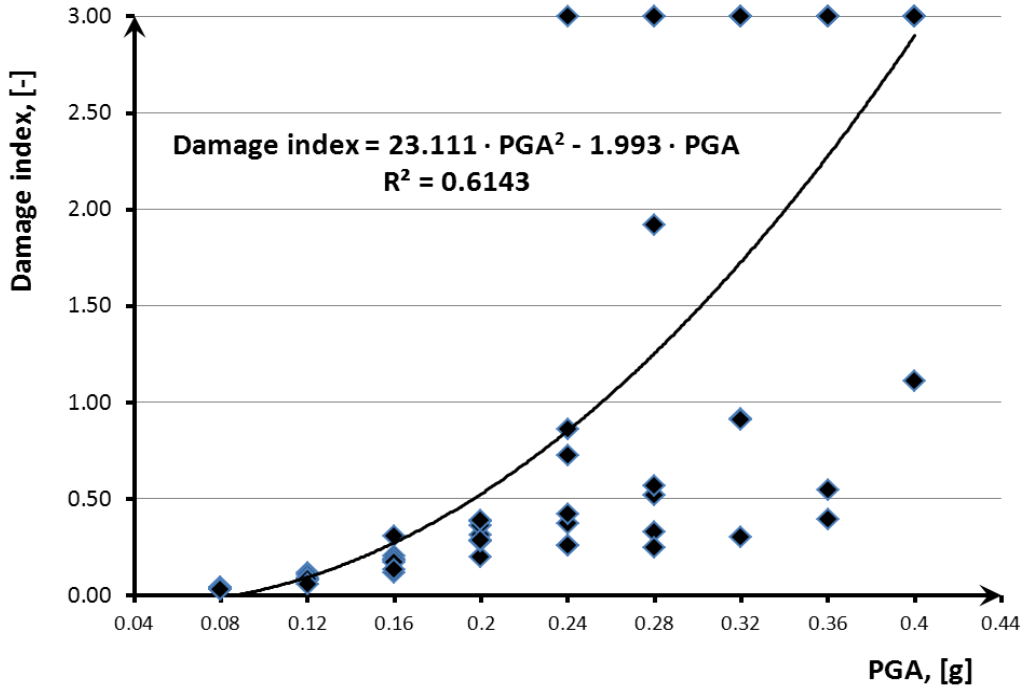


Fig. 21 – Scattering of the global damage index values versus PGA level
for the 4 storied existing structure

It can be noticed that the evolution of the values for the damage indices of existing structures is steeper than for the new structures. For the existing structure, the repairable situation of the building (corresponding to the limit between the moderate and severe damage) is positioned at an intensity level of 0,13g, half of the necessary design value. The design level of 0,24g imposed by the P100/2006 code cannot be reached by this type of buildings. If we judge by the 1992 code, we may notice that this type of structure almost reaches the necessary ultimate design level of 0,20g, mostly because its initial design was as a hospital building. Considering it as a normal importance building in the 1973 code, the capacity should be reduced by 35%, which lowers the results even more.

4.3. Comparison between the two analyses

Fig. 22 presents the variation curves of the mean global damage indices for new and existent structures. Several conclusions may be depicted with respect to the damage level that appears in these:

- the value at which collapse occurs in the studied reinforced concrete buildings is:
  - For the new 4 storied structure: 0.256g, which is 6.67% over the design value;
  - For the new 6 storied structure: 0.308g, which is 28.33% over the design value;
  - For the new 8 storied structure: 0.336g, which is 40.00% over the design value;

  - For the existing 4 storied structure: 0,198g, which is 17.5% below the actual design value or even 41.07% under the actual PGA level, if we take into account that the analysed structure has an increased capacity due to the initial importance class.
- the PGA values at which the reparability condition is fulfilled are:
  - For the new 4 storied structure: 0,196g;
  - For the new 6 storied structure: 0,232g;
  - For the new 8 storied structure: 0,258g;
  - For the existing 4 storied structure: 0,120g.

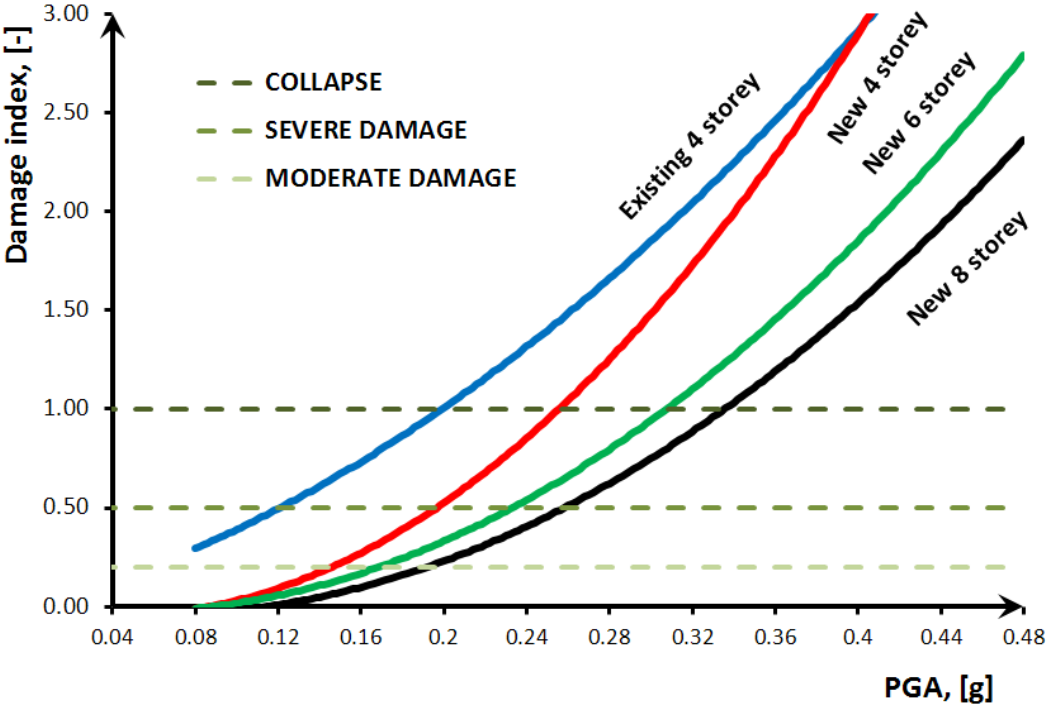


Fig. 22 – Comparative graph between variation curves of the mean global damage index for new and existing structures

It is noticeable that the tendency of new structures is to show in the beginning a moderate increase of damage index value, characterized by concave diagrams. In the meantime, it is observed that the new structures do not present structural damage for peak ground acceleration values below 0.08g-0.10g.

By contrast, the existing structure shows a graph with a fast increment of damage values, especially in the low-PGA range. The curves are important up to damage index value of 1, which represents the structural collapse limit.

4. Conclusions

1. This paper presents a direct method for evaluation the damage index value based on a specified PGA level. Only the structural damage has been taken into account.
2. This study implied performing 252 nonlinear dynamic analyses, with damage parameter evaluation on beams, columns and storeys and also global damage of the structures. The analysis files totalised more than 43 000 lines of source code. The output results summed approx. 1260 files, with hundreds of values to be processed in each of the files.
3. The mathematical formulas for determination of the damage indices were defined for new and

existing structures for various height regimes of reinforced concrete frames. The relationship between PGA level and Park-Ang damage index may be considered as second degree equations, presented in Fig.13a, Fig.13b, Fig.13c and Fig.20.
4. The increase of PGA level provides an increasing scattering of the global damage index values.
5. This study covers the influence of the height regime on the damage level of the structures, correlated to the seismic ground motion intensity.
6. In several cases, one could notice that the structures may be able to resist to seismic forces larger with 50% from the design base shear force, obviously, in a reduced number of 14-28% of the total cases. This may depend on the system period on the corresponding spectra of the accelerogram but also to a correct design of the structure which confers a structural global hierarchy mechanism (“strong” columns, “weak” beams). This may be concluded mostly for the high-storied structures, especially with 6 and 8 stories (see Fig. 13a and 13b).
7. When PGA level is well above the design value (0,40g in the example), one may notice that the shape of the hysteretic curves are consistent with the expected plastic response (Fig. 9b), but in other cases, the structures do not last more than 3-4 seconds to this high level of PGA (Fig. 9a).
8. The time-history graphs of damage indices and their story distribution indicate a correlation with the stories that provide the highest values of relative story drift in linear elastic analysis.

Acknowledgements

The author would like to thank the coordinator of the PhD thesis, PhD. Eng. Proff. Virgil Breabăn for his implication in the coordination of this work.

The author would like to also thank the scientific referrers, PhD. Proff. Eng. Nicolae Ţăranu, PhD. Proff. Eng. Dan Lungu and PhD. Proff. Eng. Radu Văcăreanu for their valuable suggestions.

6. References

[1] Idarc 2D, version 7.0, *University of Buffalo, the State University of New York*, 2010

[2] P100-1, Cod de proiectare seismică. Partea I: Prevederi de proiectare pentru clădiri, *Universitatea Tehnică de Construcţii Bucureşti, MTCT, Romania*, 2006 (Romanian seismic design code in force)

[3] Etabs Nonlinear, version 9.5.0, *Computers and Structures Inc*., 2009

[4] Seismoartif, version 1.0, *Seismosoft*, 2012

[5] Saragoni GR, Hart GC, Simulation of artificial earthquakes. *Earthquake Engineering and Structural Dynamics*, **Vol. 2**, pp. 219-267, 1974