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SUMMARY OF DOCTORAL THESIS

"EVALUATION OF DAMAGE LEVEL FOR REINFORCED CONCRETE STRUCTURES CORRELATED WITH THE SEISMIC MOTION INTENSITY"

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Claudiu Tudose

2012, octombrie

Keywords:

damage index, artificial accelerogram, reinforced concrete structures, nonlinear static analysis, pushover, nonlinear dynamic analysis, time-history, idarc, hysteresis, ground seismic motion

Contents

1. Introduction

1.1. Commentary on the subject of the thesis

The development within the last years of the design criteria for structural design for new buildings, as well as the importance of seismic vulnerability, has widened the objectives of the seismic design. While the safety against collapse is still the main pursued aspect, the performance expressed as functionality and financial economy has become a central part within the design criteria. Thus, it appears as a necessity to define the seismic intensity related to the effects, damages and the further behavior of structural systems in order to define the potential hazard and to classify the seismic motions.

1.1.1. Historic perspective

Long time ago, the pharaons' demand was that their tombs to last forever. After 4000 years, the pyramids still stand. Their designers fulfilled the performance objectives, except for one thing : the safety demands regarding inside access have not been fulfilled on long term. [Hadjian, 2002].

1.1.2. Life safety performance objectives - examples

After the 1993 Long Beach earthquake, which affected many schools in the region, a special law was formulated in California State in order to prevent similar or worse consequences of future earthquakes. After the 1971 San Fernando earthquake, which made unoperable almost all the hospitals in the region, a new set of performance criterias was formulated, in order to ensure hospital operability after a major earthquake.

1.1.3. Context of the thesis

1.1.3.1. SEAOC 1967 bluebook

The 1967 SEAOC Bluebook states the general criteria for buildings.

1.1.3.2. SEAOC 1999 bluebook

The seventh edition of SEAOC Bluebook, annex I, part A, which was published in 1999, defines four levels of seismic hazard, compatible with the final draft of the International Building Code (IBC) 2000, on a probabilistic basis. The probabilistic hazard is defined as a probability of exceedance in a period of 50 years and an annual occurrence probability.

Thus, four levels of hazard are defined in the 1999 SEAOC Bluebook, in order to formulate the performance objectives. Moreover, SEAOC 1999 defines the performance objectives of specific categories of buildings.

1.1.3.3. Vision 2000 committee

The Vision 2000 committee purposed to make the transition towards performance based design. Relationships were established between building performance, building type and earthquake probability and increments in performance category were suggested (less damage) for critical buildings [76].

1.1.3.4. Eurocode 8 and the Romanian design code, P100-1/2011

Following the tendency of implementation of european norms, the romanian design code for buildings, P100-1, comes with the same system for performance evaluation as the european brother. Thus, the new buildings are verified for two limit states: ultimate and serviceability. Compared to the 2006 edition, the project edition of P100-1 code, presented in 2012, considers a different mean recurrence interval: 225 instead of 100 years, which corresponds to a probability of occurrence of 20% in 50 years.

The new thing in performance based design is the structural design for more than a single ground motion intensity, coupled to different states of damage. Each one of these pairs is characterised as a performance objective.

1.2. Objectives of the thesis

- To performing an analysis of the performance of reinforced concrete buildings
- To put into evidence the seismic response parameters of reinforced concrete frames, expressed by the design spectra and by artificially generated accelerograms
- To analyse the possibility of design and thorough checking of reinforced concrete structures by using pushover analysis
- To evaluate the seismic response of reinforced concrete frames through damage indices using time-history analyses and nonlinear behaviour models for the component elements of the structures

1.3. Thesis content

The work is structured on 6 chapters, two annexes and one bibliography index.

Chapter 1, with introductive character, defines the actuality of the thesis' theme and the objectives pursued. In the meantime, the chapter presents the international codes which referred to life safety performance levels and the differences between their consecutive editions. In the end, the chapter refers to european and national codes and discusses the performance levels contained by these.

Chapter 2 presents the main aspects concerning the hazards and the structural seismic vulnerability. World's seismic hazard sources are mentioned, as well as the hazards in Europe and Romania. Some international statistics are also mentioned.

In the second part of the chapter, an analysis of the structural vulnerability functions is performed, correlated to the seismic hazard. An extensive analysis is performed with regard to the available structural vulnerability methods and capacity curves for seismic loaded elements and structures are presented.

Concerning structural vulnerability, the evolution of the romanian seismic codes, starting from 1963 up to the project edition of 2012 is presented.

Chapter 3 concentrates on the structural performance evaluation concepts. The main factors which may affect the structural performance are analysed: the shape, torsion sensitivity, vertical regularity, weak levels presence, cladding effect, seismic separation joints, structural ductility, and foundation stability.

Another aspect concerns the parameters which evaluate the structural performance by a series of indices: damage indices due to building response and damage indices due exclusively to the seismic motion. The methods for seismic performance evaluation are based on static and dynamic nonlinear analysis.

A final aspect treated in this chapter is the definition of the performance levels, with direct link to the american code FEMA 273's provisions: damage limiting level, life safety, collapse prevention.

Chapter 4 includes case studies for damage evaluation for new buildings. The chapters begin with the presentation of the analysis procedure. The location and its seismic characteristics are presented together with the definition of the seismic action and the numerical analysis methods to be used. The analysed structures are reinforced concrete frames with 4, 6 and 8 stories, with 5 openings on each direction and story height of 3,25m.

The seismic motion is defined as time dependent, through artificial accelerograms. The analysis is performed considering 9 values for peak ground acceleration, each being defined by a set of seven artificial accelerograms compatible with the absolute elastic acceleration spectra.

For each of the structures, the design considered the evaluation of the permanent and variable loads acting on the concrete structural elements. Taking into account that the real interest is in the seismic motion, the loads were considered as long-time values. One central frame was analysed from each of the structures, acted by the seismic action together with the vertical loads previously determined. The fundamental load combination was considered only to determine the bottom reinforcement in the beams, in the fields were this combination conducts to larger amounts of reinforcement than the special combination, in order to consider the reinforcement in a real designed model. The columns and the beam supports are designed by the seismic combination of loads.

The structural models were subjected to nonlinear dynamic analyses, performed in Idarc 2D. The analyses considered nonlinear behaviour of the elements, the energy dissipation mechanism being defined by the moment-curvature constitutive law. The evaluation of damage level is performed using damage values: on beams, columns, storeys and also globally for each intensity value of the seismic motion and for the three different high-rise frames.

The second part of the chapter presents the results of the analyses. The comments and the conclusions are included at the end of each sub-chapter.

Chapter 5 includes one case study of damage evaluations for an existing structure. The methodology used in chapter 4 is maintained. In the end of the chapter, a synthesis comparison is being made based on the results obtained in chapters 4 and 5, for new and existing buildings, respectively and some findings regarding the residual damage level in each structure.

Chapter 6 contains the general conclusions based on the studied literature, the final conclusions of the research in the thesis, the author's contribution and future research works.

The bibliography list contains 88 titles.

The annexes are in a total number of two.

Annex 1 contains 108 Figures obtained during the case studies and has 34 pages.

Annex 2 refers to stochastic response of structures and has 10 pages.

2. Seismic hazard and structural vulnerability

2.1. Introduction

The seismic hazard is defined as the probability of occurrence of a destructive event in a defined area and within a defined time interval.

The hazard analysis uses knowledge from other domains that seismology alone. Geology is needed in order to determine the location, the configuration and the definition of the potential seismic sources, especially of known active faults.

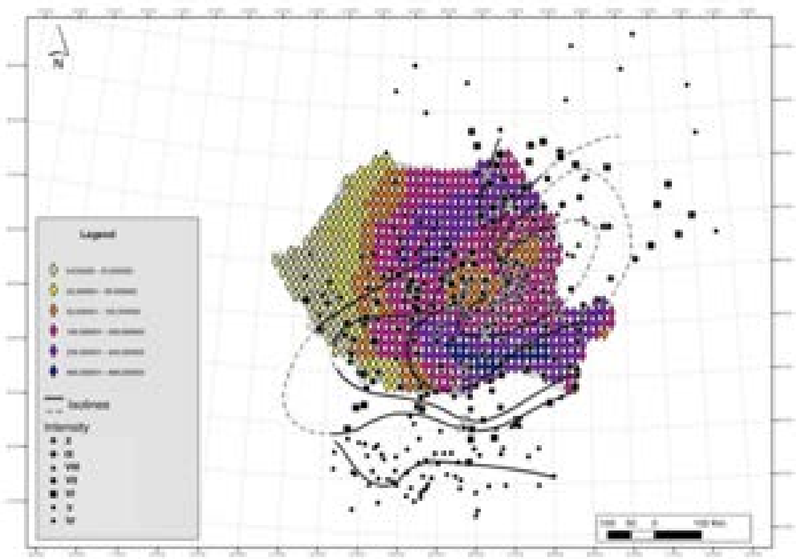


Fig. 2.1. Deterministic seismic hazard for earthquake with Mw of 7,7 [84]

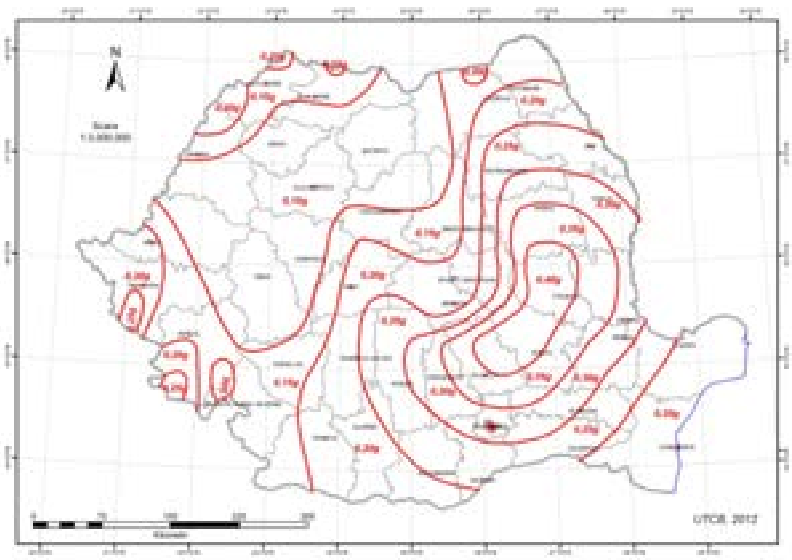


Fig. 2.2. Peak ground acceleration for earthquakes with MRI=225years [Lungu, Arion, 2012]

2.2. Sources of hazard - worldwide

Important seismic areas are those of Latin America, Pacific ocean area, west of North America, Middle East and Europe.

Similar geological conditions extend to the Caribbean's in the Atlantic ocean which are considered as part of the "Ring of Fire" even it is not part of the Pacific area.

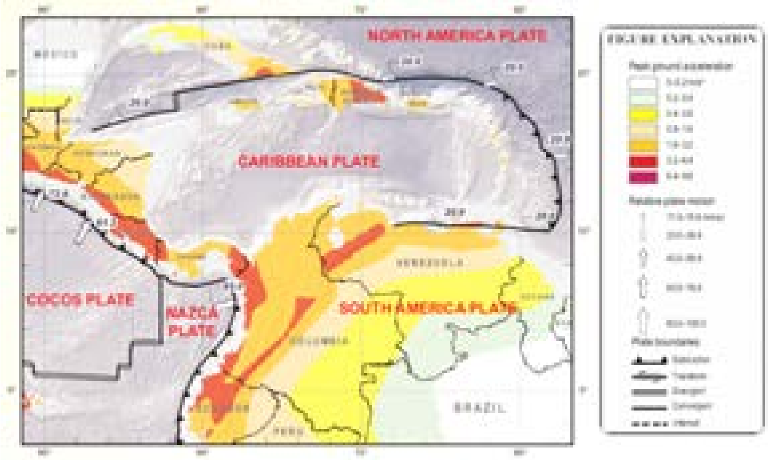


Fig. 2.3. Central America and Caribbean seismicity(1900-2010), [USGS, 2011]

2.3. Sources of hazard – in Europe and Romania

Referring only to the european area, there are several crustal and sub-crustal seismic sources that can develop earthquakes of various intensities: south of Portugal, south of Spain, Italy, Greece, Turkey, the scandinavic area and not least, Romania.

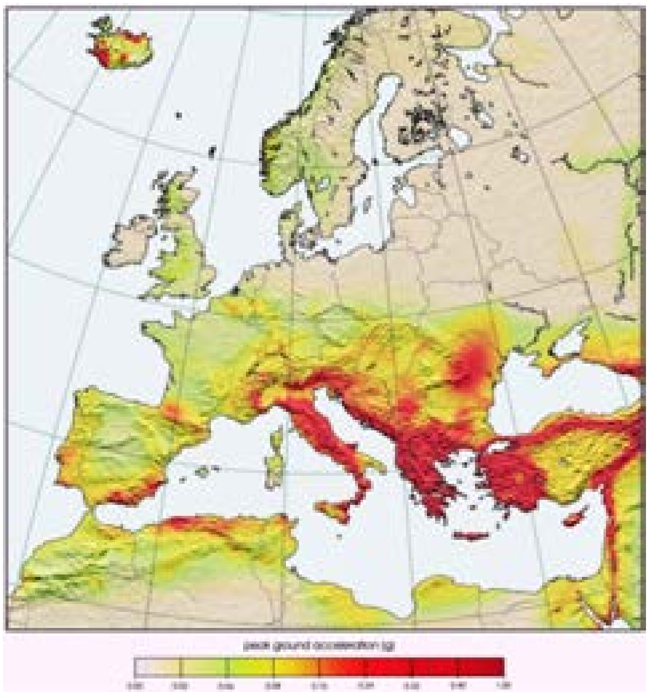


Fig. 2.4. Seismic hazard map for european-mediteranean area with PGA of 10% exceedence probability in 50 years, International Geological Correlation Program, European Seismological Commission, february 2003

Romania's seismicity is divided in multiple epicentre areas: Vrancea, Făgăraş-Câmpulung, Banat, Crişana, Maramureş and Dobrogea. In addition to these, local importance epicentre area may be mentioned in Jibou and Târnave in Transilvania, the north and the west of Oltenia, north of Moldova and Câmpia Română. The highest level of seismic hazard is recorded in the area of Carpaţii Orientali, which includes an intermediary depth source.

Within the last 100 years, Vrancea region produced four major sub crustal earthquakes – 10^{th} of november 1940, magnitude Mw 7,7; 4^{th} of march 1977, magnitude Mw 7,4; 30^{th} of august 1986, magnitude Mw 7,1; 30^{th} of May 1990, magnitude Mw 6,9 – the first two with disastrous impact [75].

2.4. International statistics

Throughout the time, the Architectural Institute of Japan has investigated the damages of buildings after major earthquakes in Japan and worldwide [Otani, 2000]. The statistics were performed in areas of Mexico City (1985), Philippines (1990), Erzincan in Turkey (1992) and Kobe (1995).

2.5. Vulnerability

The vulnerability represents the expected degree of losses due to a destroying event. This is generally expressed as functions and matrices which may be obtained through statistical studies on damaged buildings in earthquake subjected areas or by numerical or analytical models of structures.

2.5.1. Evaluation methods – defining a vulnerability function

A function that describes the vulnerability is a relationship which defines the expected damage of a building or a group of buildings based on the ground motion intensity [Lang, 2004]. In order to evaluate the damage to the earthquake subjected building, the vulnerability function must be compared to the seismic demand.

2.5.2. Capacity curve (demand and capacity spectrum)

2.5.2.1. Structural capacity

The capacity curve is generally built based on the first modal shape of a building, based on the assumption that the building's response to a seismic action is found in the fundamental vibration mode.

The capacity curve of a structure (also named pushover) represents a graph of lateral force capacity expressed as a function of a lateral displacement. This is obtained from a graph of statical equivalent force and building displacement [Postelnicu et al, 2004].

2.5.2.2. Structural elements capacity

Fig. 2.8. presents three examples of stress-strain diagrams function of the capacity and the redundancy of the element that are often used. These curves look similar to the conceptual force-displacement curves described in [ASCE 41-06, 2007] but they have a different meaning.

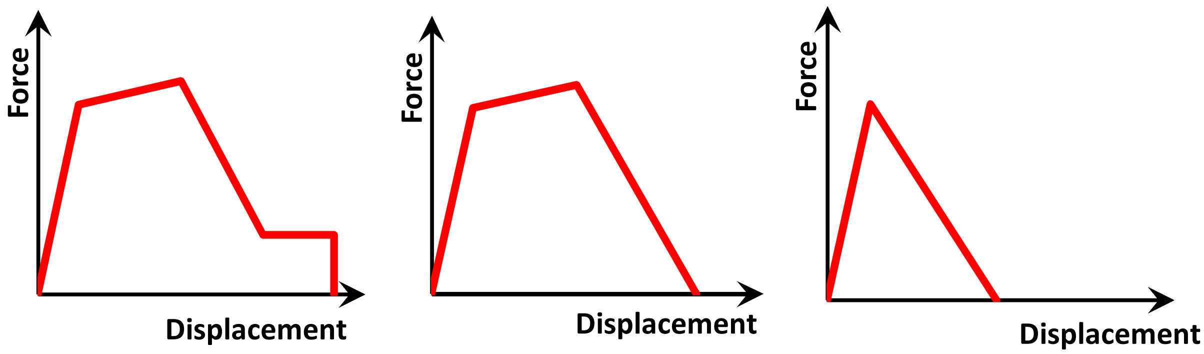


Fig. 2.8. General curves for element behaviour

2.6. Provisions of romanian seismic codes – historical to present

Main seismic codes and their provisions are described in this chapter, from 1963 to the project of P100 in 2012.

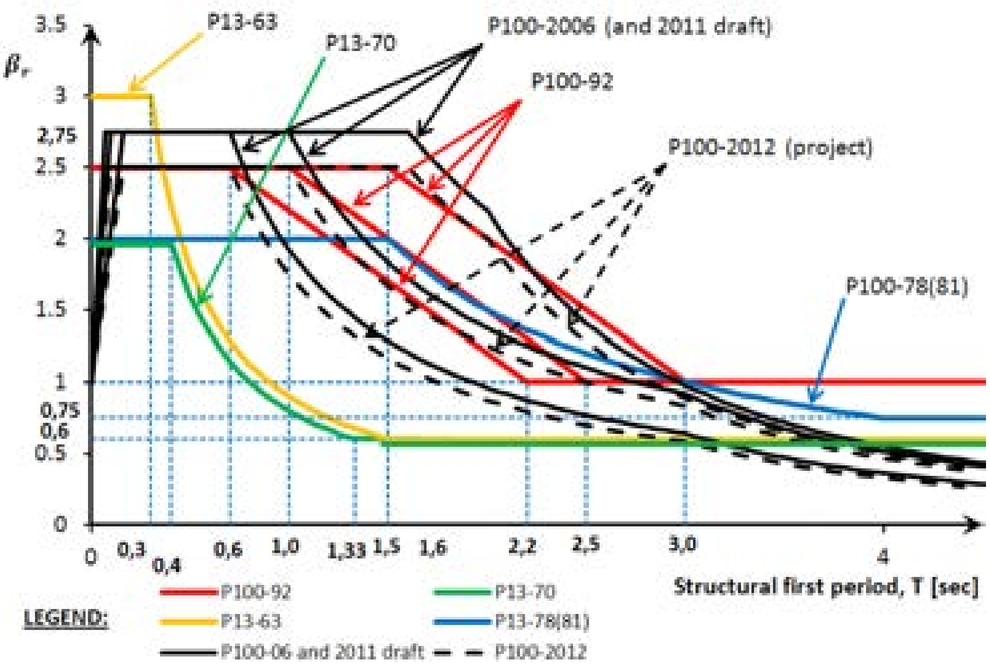


Fig. 2.15. Evolution of dynamic beta coefficient during 1963-2012

3. Evaluation of structural performance of the buildings

The evaluation of a buildings' performance is tightly dependent to the performance characteristics of component elements. The elements of major importance during an earthquake are those that ensure vertical stability and which resist the seismic loads [ATC-40, 1996].

3.1. Factors which affect seismic performance

The seismic performance of a building is defined as the behaviour of a building when subjected to a seismic action. In order to consider achieving the structural performance in which the seismic effects on buildings are reduced, one must eliminate (or limit) the uncertainties which are under the control of the structural engineer: building shape, stiffness distribution, reduction of irregularities, ductility control.

3.1.1. Building shape, mass, strength and stiffness distribution

3.1.1.1. In-plane shape (plan regularity)

The problems of the constructions with complex and irregular shapes can conduct to the building being split into several parts during a seismic event, to translations on axes different than the main axes of the elements or can lead to global torsion due to increased stiffness in the joining area. The new buildings may be easily solved by dividing them in individual structures. The existing buildings may be split into simple shapes or can be strengthened as a whole on the main directions of response.

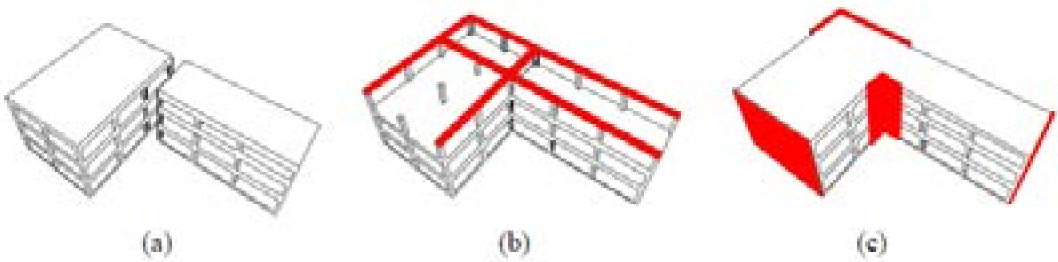


Fig. 3.2. Solutions for complex shapes: (1) dividing into simple buildings, (2) strengthening of the main directions of response, (3) supplementary stiffening of the highly solicited zones (balancing the rigidity distribution within the building) images from [FEMA 454]

3.1.1.2. Torsion sensibility

The appearance of torsion in an earthquake subjected structure leads to quick degradation of structural response, because the position of the stiffness centre (or the resistive force) is eccentric with respect to the position of the centre of masses, in which the inertia forces are applied.

Fig. 3.4. Damage due to torsion with second level collapse, Hotel Terminal, Guatemala, 4th of feb 1976, 7,5 Richter, USGS, Figure 55, Professional paper 1002, public image

Fig. 3.5. Damage due to general torsion of the building, column damage and falling of upper floor to the right due to torsion, Hotel Terminal, Guatemala, 4th of feb 1976, 7,5 Richter, USGS, Figure 74, Professional paper 1002, public image

3.1.1.3. Vertical regularity

The requirements of romanian and european codes state that the path of the vertical loads to be as straight and to avoid as much as possible second order bearings.

3.1.1.4. Weak floors, pancaking (domino effect)

The weak floor of a building contains elements with reduced stiffness compared to the storeys above and below them. In case of a weak floor, there is the risk of collapse of all the above floors, one over another (domino).

Fig. 3.6. Civil building with weak garage base floor, San Francisco, California, Loma Prieta earthquake, 17th of oct 1989, 6,9 Richter, National Information Service for Earthquake Engineering (NISEE), University of California, Berkeley

3.1.1.5. Masonry effect

The correct confinement of masonry filled frames should lead to masonry being expelled from the frame during a seismic event.

The presence of masonry panels on top levels and the lack of bottom floor masonry may conduct to a supplementary stiffening of the upper floors and unwanted transformation of base floor into a weak floor.

Fig. 3.10. Short columns, Algeria, 2003 (left), Failure of column due to partial masonry (right)

3.1.2. Insufficient separation seismic joints (building pounding)

Building pounding appears when two nearby buildings are built near each other and the distance between them is smaller than their seismic displacements. Even there are some cases in which the buildings supported each other, most of the times, the presence of a small building near a tall one with different periods and amplitudes makes the top floor slab of the short one to pound the columns of the tall building and to affect the structure of the latter.

Fig. 3.14. Damage to the four-storied building due to pounding to the two-storied nearby building, L'Aquila earthquake, Italy, 6th of April 2009, 5,8 Richter

3.1.3. Ductility

The capacity of the structural elements to sustain post-elastic deformations without diminishment of the strength is quantified as ductility. The element ductility includes the ability of elements to support important deformations and to adsorb energy through hysteretic energy.

Fig. 3.16. Failure modes of a column: shear force failure (left), shear force failure after longitudinal reinforcement yielding (center) and failure due to reach of capacity of the element – maximum ductility (right), [Yoshikawa et al, 2001]

3.1.4. Foundation stability

The buildings developed on soft soil can be subjected to different types of settlements induced by the earthquakes with the following effects: surface shear for structures placed on faults, soil liquefaction.

Fig. 3.19. Soil liquefaction under the foundation due to the earthquake and collapse of some structures, Niigata, Japan 16th of June 1964, 7,6 Richter

3.1.5. Resonance

In Bucharest, the damage recorded during the 4th of March 1977 earthquake estimated more than 1500 casualties [Văcăreanu, 2007] and the collapse of 23 high rise buildings made of reinforced concrete and 6 multi-storey buildings in masonry erected before the 2nd world wat and 3 high rise buildings built during 1960-1970.

On the 4th of March, on the soil made predominantly of clay in the east of Bucharest, the maximum recorded ground acceleration peaked at 0,20g, the ground motion being characterised by a very long corner period, Tp = 1,6sec, which was unusual and unknown until then, taking into account that the P13/70 code privisioned a 0,4sec predominant period.

The high percentage collapse of tall buildings of the total number of collapsed buildings raises questions with regard to the possible syncronisation between the fundamental period of the buildings and the seismic period oscillation.

Some examples of buildings that either collapsed or sustained heavy damage during the 10th of November 1940 and the 4th of March 1977 earthquakes are: Carlton block of flats (highest in block in Bucharest in 1940), Sahia, Dunărea, Scala, Belvedere blocks of flats, all being high rise, whose fundamental period was over 1 second (considering the simplified formula from P100/2006 which states 0,1sec for each storey). Taking into account that the relative drift verification was inexistent at that time, these structures were more flexible than the ones according to P100/2006 and, advancing further on, their fundamental periods were larger than the 0,1s per storey estimation, reaching values close to the ground motion dominant period.

Fig. 3-22. Carlton block of flats, totally destroyed during the 10th of November 1940 event

Fig. 3-24. Dunărea block of flats, Bucharest, 1956 [86]

Fig. 3-25. Dunărea block of flats, Bucharest, 5th of March 1977 [88]

3.2. Parameters for evaluation of seismic performance

3.2.1. Element damage indices

In order to evaluate the seismic performance, the correct parameters must be selected, which may show the damage level in a structure. The most used parameters are the total displacement and the relative story displacement.

Though, when discussing about cyclic induced deformations, it is more adequate to use energy based indices. The total energy generated by the earthquake and which acts upon a building is dissipated through structural damping and hysteretic energy (inelastic deformations). The damage level is due to the energy dissipated through inelastic deformations. An important parameter of a building's response is the ratio between the hysteretic energy and total initial energy.

One of the most used damage indices is the Park-Ang index, where the structural damage is expressed as a combination of the maximum displacement damage and the dissipated hysteretic energy damage, the latter one due to cyclic repeated loads.

$$DI_{PA} = \frac{u_{max}}{u_{max,mon}} + \frac{\beta}{F_y \emptyset_u} \cdot \int dE_h \qquad \text{(Ec. 3.1)}$$

Starting with version 3.0 of IDARC software [Kunnath et al, 1992] a new and modified version of the Park-Ang indices is used instead the initial ones, in which the deformation is recovered from the first term, and the moment-curvature function is used instead of the force-displacement function. [Văcăreanu, 2000].

$$DI_{PAW} = \frac{\emptyset_m - \emptyset_y}{\emptyset_u - \emptyset_y} + \cdot \frac{\beta}{M_y \emptyset_u} \cdot \int dE_h \qquad \textit{(Ec. 3.2)}$$

The definition of the damage indices correlated to the structural condition and the physical damage states is presented in Table. 3-1.

Damage index	Damage level	Physical damage state
0,0 … 0,1	None	No damage or local light cracks
0,1 … 0,2	Minor	Reduced concrete cracks
0,2 … 0,5	Moderate	Large cracks, spalling of concrete
0,5 … 1,0	Severe	Concrete crushing, reinforcement is exposed
over 1,0	Collapse	Structural failure

Tabelul 3-1. Definition of damage index values and their physical damage state

3.2.2. Damage parameters due exclusively to the seismic motion

Next, the characteristic parameters of the seismic action are defined, which are used to quantify the structural damage. Thus, the exclusive parameters concerning the seismic action are defined and the damage due to structural response is being neglected. The easiest way of defining is by peak ground acceleration, peak ground velocity and peak ground displacement or their ratios: PGV/PGA or PGD/PGV.

The damage potential may be expressed by the maximum motion parameter, defined as PGV/PGA ratio. In works like [Sawada et al, 1992] one may observe that the seismic motions with large damaging potential present high value of PGV/PGA ration

In [Zhu et al, 1998] the seismic motion is classified into three categories, depending on the PGA/PGV ratio and has showed that the ratio influences significantly the nonlinear response of the one degree of freedom systems. The parameters defined as integral are: root mean squared acceleration, root mean squared velocity and root mean squared displacement, denoted with RMSA, RMSV and RMSD.

Even though it was observed that PGV is correlated to the structural damage, this parameter does not supply information on frequency content and duration of the seismic motion, thus having in this case a limited capacity of representation of the degradations induced by the earthquake [Bozorgnia and Bertero, 2001].

In order to better describe the seismic motion, Newmark and Hall (1982) introduced the concept of effective peak acceleration (EPA) and effective peak velocity (EPV).

In our country, prof. Lungu formulated in 2003 the following definition for EPA and EPV:

$$EPA = \frac{max\,\overline{PSA_{0,4s}}}{2,5} \qquad \text{(Ec. 3.3)}$$

$$EPV = \frac{max\,\overline{PSV_{0,4s}}}{2,5} \qquad \text{(Ec. 3.4)}$$

Event	Station, direction	EPA (m/s2)	max Saa/2,5 (m/s2)	EPV (m/s)	EPD (m)
4th of march 1977	Incerc, NS	2,5329	2,6010	0,6275	0,1940
31st of august 1986	Focşani, N97W	2,5221	3,0400	0,1952	0,0249
30th of May 1990	Oneşti, N200E	2,6824	3,6490	0,1513	0,0315

Tabelul 3-2. Motion description for recorded earthquakes in Romania

The spectral acceleration is the static equivalent force induced by a seismic motion in an elastic structure with unitary mass. The spectral velocity is linked to the maximum deformation energy induced in the system [Chopra, 1995]: the design spectra use elastic design spectra, characterised by three specific period zones in which, in turn, the acceleration, the velocity and the spectral displacement are constant.

3.3. Seismic performance evaluation methods

The thesis considers only the nonlinear methods, static and dynamic.

3.3.1. Nonlinear static analysis

The pushover analysis considers an inversed triangle or linear uniform seismic force distribution on the height of the structure and a monotonic increase of its values, starting from zero until structural collapse is attained with no reserves of strength or the model reaches an user imposed top story displacement.

3.3.2. Acceptance criteria

The modelling parameters and the numerical acceptance criteria is defined separately based on element type: beams, columns and their nodes.

3.3.3. Target displacement

The pushover analysis must have a specified top story limit displacement until which the lateral forces are monotonically increased and the results are monitored. The lateral displacement during the analysis is measured by the program in a control point defined by the user, usually placed at the top (roof) level. In order to correctly evaluate the structural response, the code [P100-1, 2006] recommends that the target displacement should be 150% of the maximum displacement corresponding to the ultimate limit state (ULS). The evolution of the degradation process can be highlighted until the collapse and the building vulnerability to collapse is obtained.

The american code [FEMA 273, 1997] has the same recommendation for the displacement that should be reached, but it also can be determined with the formula:

$$\boldsymbol{\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g} \qquad \textit{(Ec. 3.5)}$$

3.3.4. Nonlinear dynamic analysis

This kind of analysis obtains the response of the structure based on some data that define the seismic motion as a function of time. The seismic motion is defined by recorded or artificial accelerograms.

The recorded accelerograms may be used if they are recorded near the site. The maximum value of the acceleration must be scaled to the same value of acceleration a_g as the one on the site and the frequency content must be compatible with the local conditions of the soil.

The artificial accelerograms are generated based on an elastic acceleration response spectra, $S_e(T)$. The elastic response spectra of the artificial accelerograms must be close to the elastic response spectra of the site.

3.4. Performance levels [FEMA 273, 1999]

The global performance of a building is a combination between the performance of the structural and non-structural elements. The performance levels are differently formulated for the two type of elements.

3.5. Performance levels [P100-1/2006]

The romanian code P100-1/2006 is based mostly on the european antiseismic code, Eurocode 8. It contains provisions for two levels of performance.

4. Evaluation of damage index for reinforced concrete structures correlated to the intensity of the seismic motion – new structures

In order to evaluate the damage index function of the intensity of the seismic action, it is first necessary to define a few types of structures for which the structural response will be evaluated for the various intensities of seismic acceleration.

The structures being analysed are medium rise frame structures, with maximum 8 storeys. This height regime is applicable to this kind of structures in seismic areas and higher storied structures might be solved using mixed solution with frames and shear walls or other types of solutions - steel.

Insisting on higher storied structures would take the study in an area where the applicability of this kind of structures is often avoided for other more economical solutions and the conducted study would not have been of such high interest.

4.1. Technical context of the analysis

The analysed structures consist of framed 4, 6 and 8 storey buildings, with 5 spans in each direction of 6,00m and storey height of 3,25m.

Due to frame similarity, only one central frame was analysed in each of the three types of structures which was subjected to one-directional seismic action and also to the corresponding vertical loads.

The considered site is characterised by a dynamic amplification factor $\beta = 2{,}75$, the corner period of $T_C = 1{,}6s$ and a maximum peak ground acceleration (PGA) of 0,24g, for a mean recurrence interval of 100 years, according to the romanian seismic code, P100-1/2006. The site corresponds to Bucharest city, but considering the fundamental periods of the structures which are under 1sec, more locations might by characterised by the same set of data (Brăila, Galaţi).

The structures have been designed according to P100-1/2006, considering a behaviour factor $q = 5 \cdot 1{,}35 = 6{,}75$ for structures with multiple stories and high ductility (class H) and over-strength coefficient $\frac{\alpha_1}{\alpha_u} = 1{,}35$, for structures with multiple stories and spans.

The concrete is of class C20/25, being the minimum allowed class of concrete for high ductility structures.

The design seismic force applied at the bottom of the system, as a percentage of the total weight is:

$$F_b = 1 \cdot \frac{0{,}24 \cdot g \cdot 2{,}75}{6{,}75} \cdot m \cdot 0{,}85 = 0{,}0831\ G \qquad \textit{(Ec. 4.1)}$$

The analyses were linear dynamic and were performed with Etabs Nonlinear version 9.5, developed by [Computers and Structures, 2009] Inc., California.

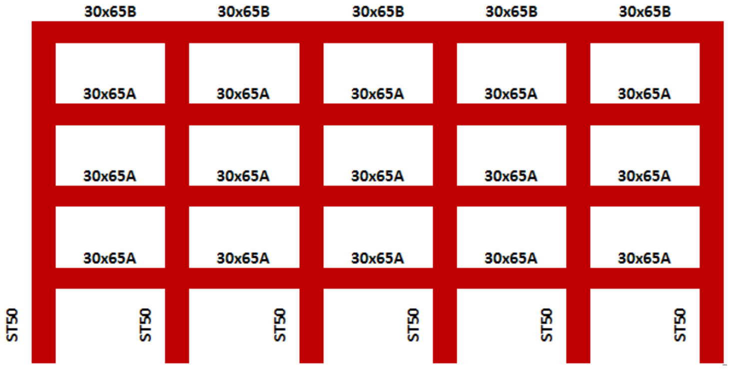


Fig. 4.1. Dimensions of the frame elements, 4 storied structure

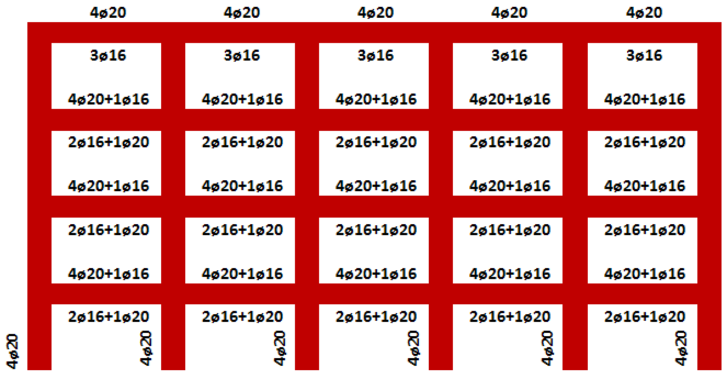


Fig. 4.2. Reinforcement of the beams and columns (side reinf.), 4 storied structure

Fig. 4.3. Dimensions of the frame elements, 6 storied structure

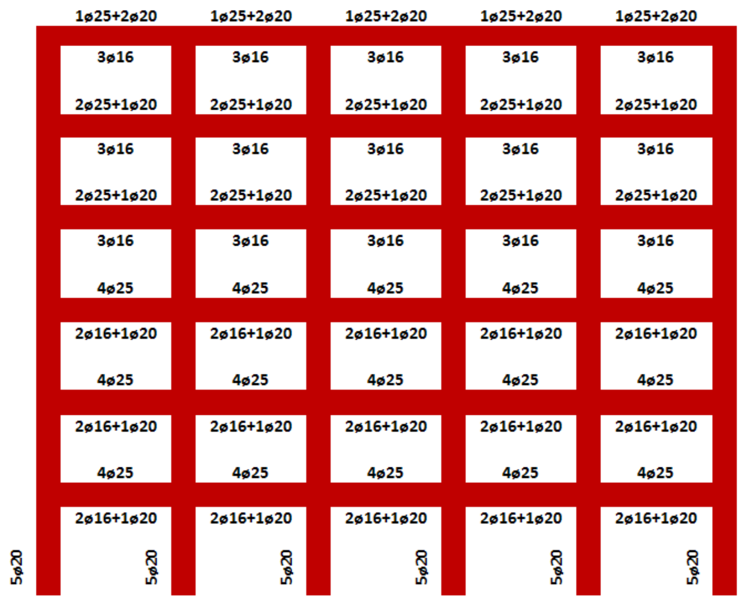


Fig. 4.4. Reinforcement of the beams and columns (side reinf.), 6 storied structure

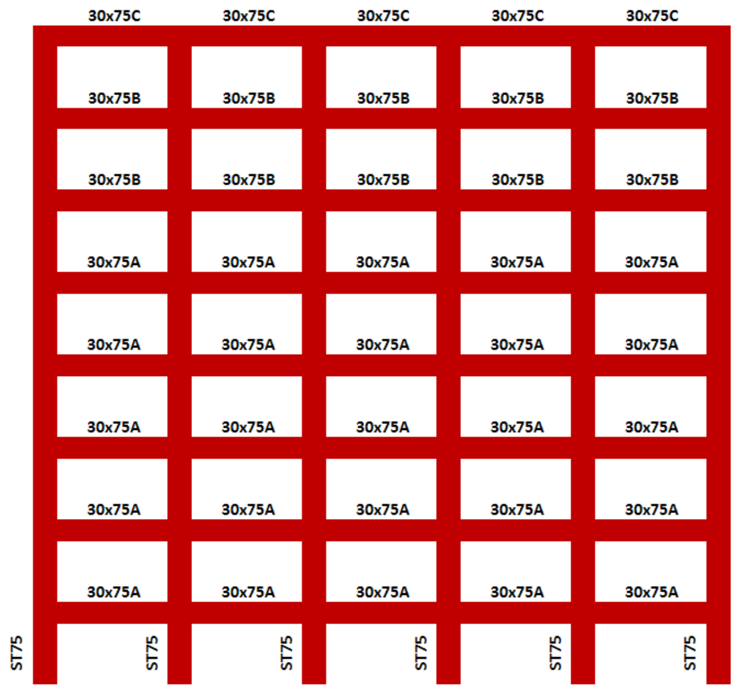


Fig. 4.5. Dimensions of the frame elements, 8 storied structure

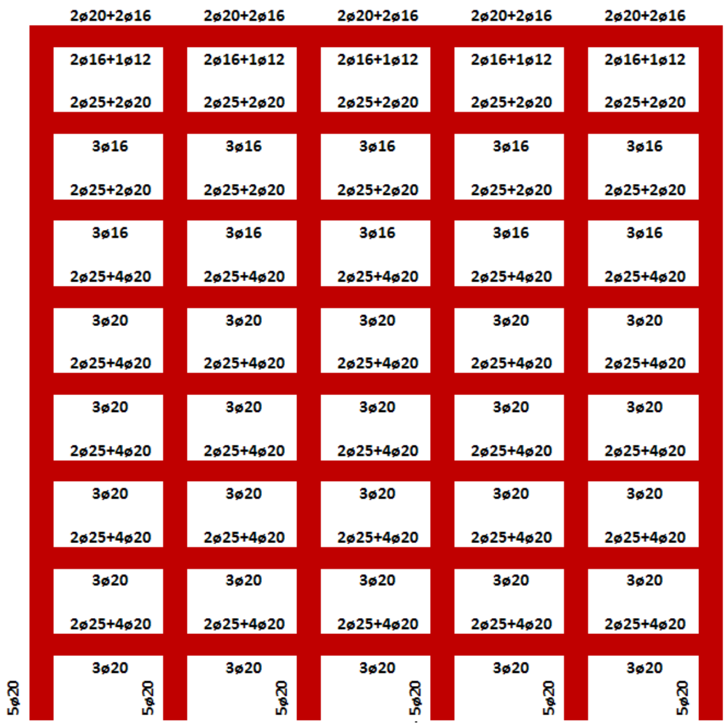


Fig. 4.6. Reinforcement of the beams and columns (side reinf.), 8 storied structure

4.2. Partial checkings

After the design, it is needed to establish the level of confidence of the obtained models. In consequence, a pushover analysis was performed for each of the analysed frames, The checks insisted on avoiding reinforcements which may lead to unwanted situations as the ones presented in chapter 2. The pushover analysis was performed in Etabs Nonlinear, in which all the sections of the elements were introduced, together with their effective reinforcements. The modulus of elasticity of the concrete is considered associated to the cracked modulus.

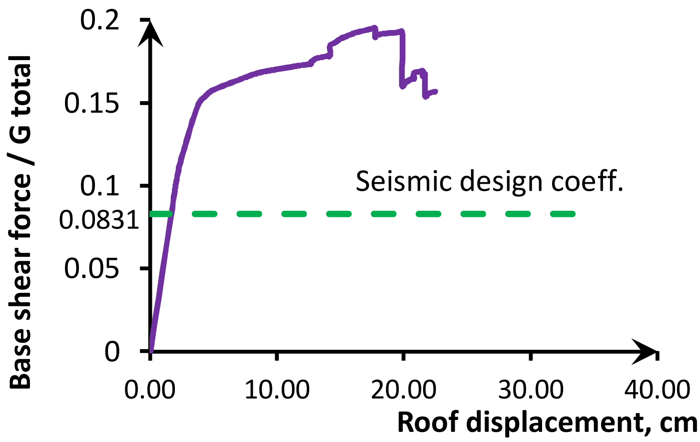


Fig. 4.10. Pushover curve for 4 storied structure, Etabs results

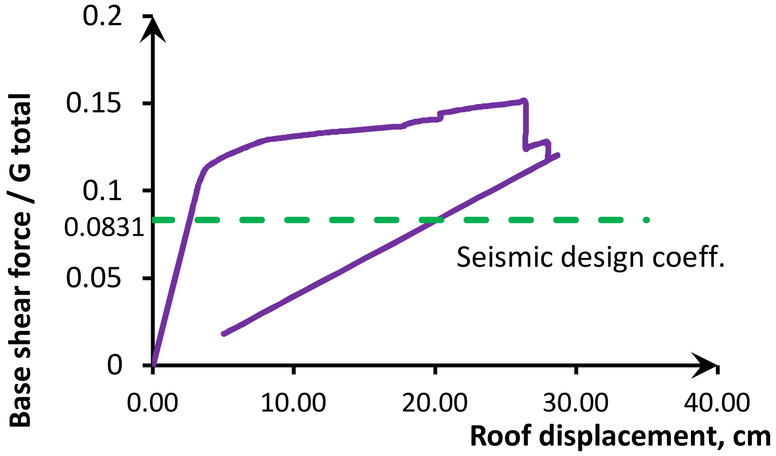


Fig. 4.11. Pushover curve for 6 storied structure, Etabs results

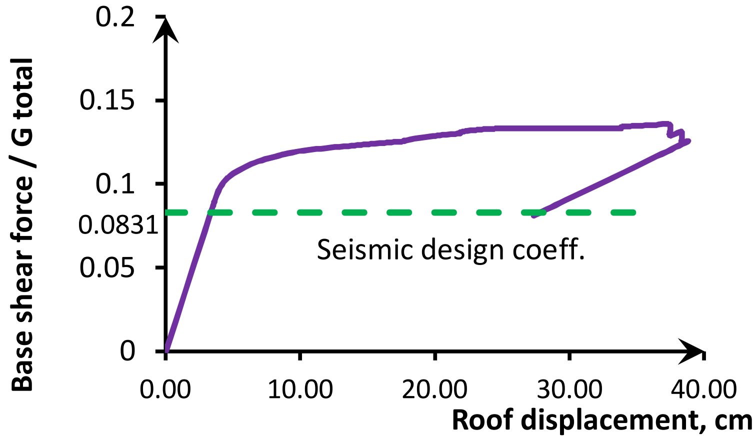
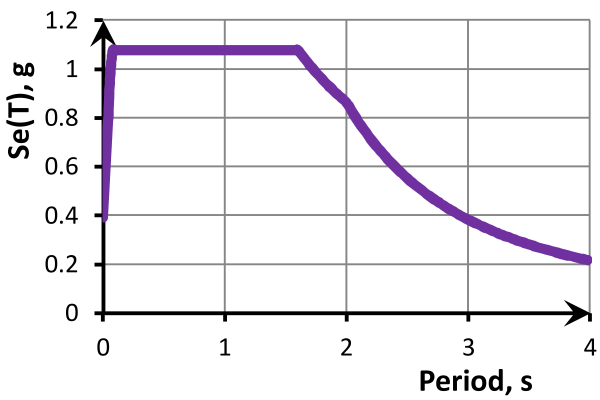


Fig. 4.12. Pushover curve for 8 storied structure, Etabs results

4.3. Seismic action – intensities, modelling

The precise determination of the structural damage index at element, storey or global level imposes using the time-history analysis. Using this kind of analysis implies the definition of the seismic action as time dependent. The PGA intensities used in the analysis are: 0,08g, 0,12g, 0,16g, 0,20g, 0,24g, 0,28g, 0,32g, 0,36g and 0,40g. For each one of them, the elastic spectrum with 1,6sec corner period was considered, scaled function of the acceleration value.

Fig. 4.13. Absolute acceleration response spectrum for PGA=0,40g and Tc=1,6s



Using [Seismosoft, 2012a], for each of the 9 elastic absolute acceleration spectra were generated several artificial accelerograms with a total duration of 20 seconds, from whom seven that respected the conditions of P100 code for each intensity level were chosen, in order to be allowed in the end to work with the mean of their results. A total number of 63 artificial accelerograms were obtained, compatible with the elastic response spectra. The intensity envelope used for artificial accelerogram generation is of [Saragoni & Hart, 1974] type.

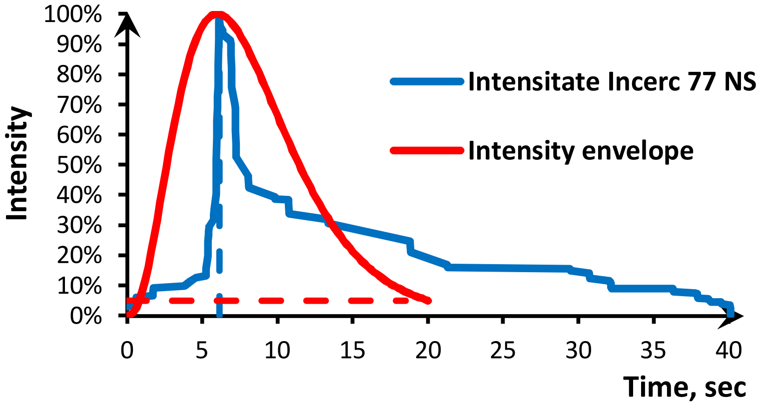


Fig. 4.177. Comparative graph: Incerc 77 NS intensity and the Saragoni-Hart function used for artificial accelerogram generation

The next figure shows an example of artificial generated accelerogram for the absolute acceleration response spectrum, with PGA value of 240 cm/s^2 as an example.

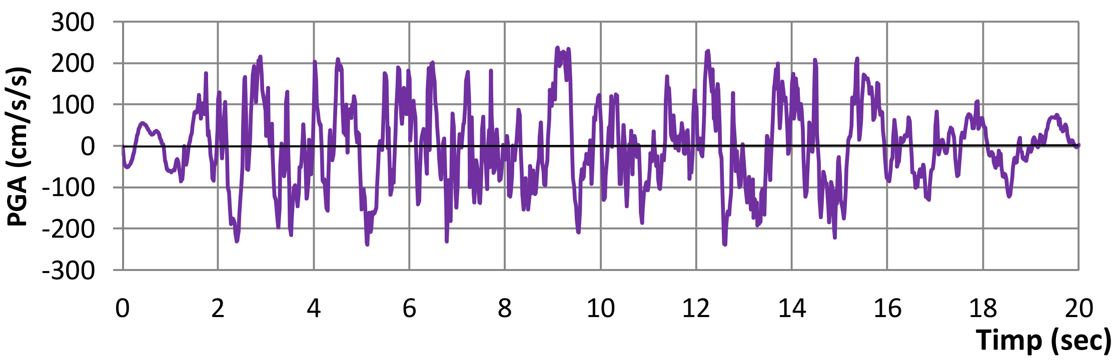


Fig. 4.18. Artificial accelerogram 1 for PGA=0,24g and Se(T)=0,66g

In order to check the results, all the corresponding spectra of the artificial accelerograms were plotted on a graph, together with the target spectrum and mean spectra, as the mean of the spectral values of the generated accelerograms, for each of the nine elastic acceleration spectra $S_e(t)$: 1,1g, 0,99g, 0,88g, 0,77g, 0,66g, 0,55g, 0,44g, 0,33g şi 0,22g.

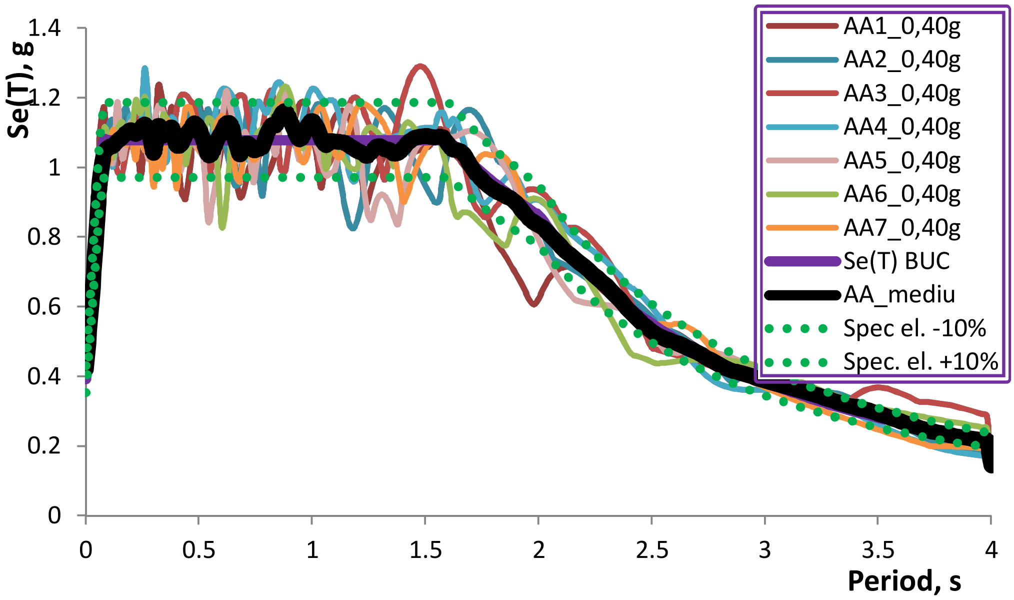


Fig. 4.258. Absolute acceleration response spectra for the 7 generated artificial accelerograms for Se(t)=1,10g and the mean response spectrum as the mean of the spectral values of the accelerograms, for PGA=0,40g.

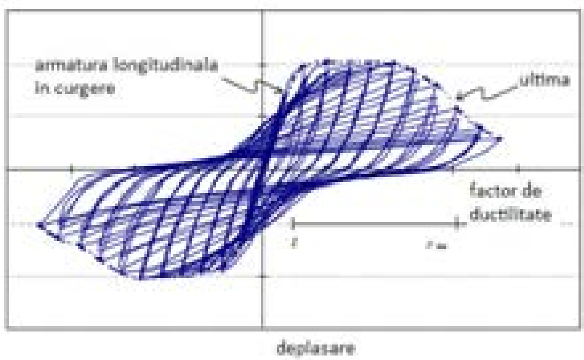
4.4. Dynamic nonlinear analysis with artificial accelerograms

Using the three structures defined in chapter 4.1 and the 9 sets of 7 accelerograms obtained at chapter 4.3, a time-history analysis was performed in IDARC 2D, version 7.0.

IDARC 2d is an advanced structural analysis software based on an executable application compiled from Fortran, written initially in 1987 and permanently updated by the professors at New York State University at Buffalo [Valles et al, 1996], in which data input is performed by entering text lines of code.

The time-history analysis considered the geometry of the three structures subjected to increasing levels of PGA values, each level of PGA being defined by a set of 7 artificial accelerograms. This conducted to writing 189 source files each of them with approximately 170 lines of software code.

Fig. 4.36. Combined hysteretic model, with stiffness and strength degradation and diagram pinching



4.5. Results of the time-history analysis

4.5.1. Moment-curvature diagrams for beams and columns

The moment-curvature relationship can be modelled as a bilinear curve with limited plateau. The surface under the curve represents the energy adsorbed by the element, made of two components: a recoverable one, corresponding to the elastic deformations and one transformed intro friction, called dissipated energy. The capacity of the section to develop post-elastic deformations and to further dissipate energy is quantified by the sectional ductility coefficient, defined as the ratio between the ultimate and yielding curvature.

$$\mu_\phi = \frac{\phi_u}{\phi_y} \qquad \textit{(Ec. 4.2)}$$

For a bilinear moment-curvature relationship, the capacity of a section to dissipate energy is proportional to the ductility coefficient. Following the analysis in IDARC, the M – ϕ diagrams for beams and columns were plotted for various values of PGA.

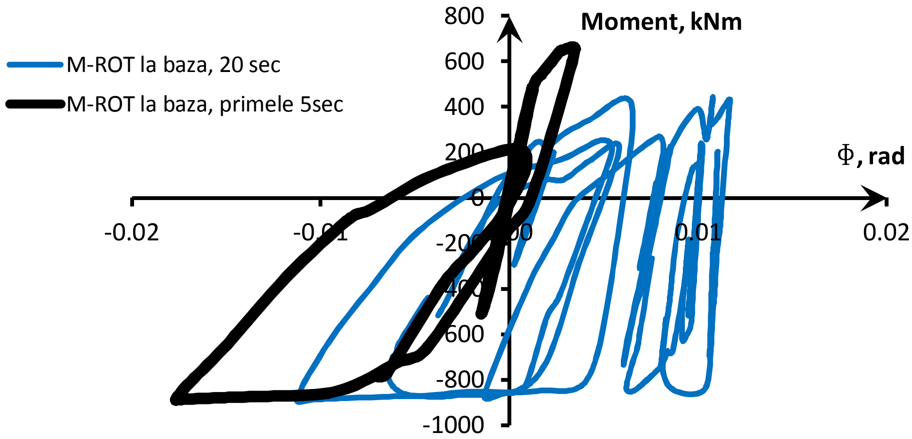


Fig. 4.37. Moment-curvature diagram at the base of column 1 (marginal), 8 storied structure during the artificial accelerogram no. 1, for PGA = 0,40g

From the graphs presented in Fig. 4.37 … 4.42 one may notice the variation in time of the moment-curvature relationship, with elastic behaviour or with low stiffness reduction for PGA values of 0,08g. Increasing the PGA level, one may notice slope reduction (stiffness reduction) and strength reduction.

4.5.2. Maximum absolute displacement response on each storey function of the acceleration intensity

Next, the graphs with the distribution of the maximum response on each storey was plotted, function of the PGA value, determined as the mean of the results of each set of artificial accelerograms. Representation of displacements has been performed for the responses within 150% of ULS displacements (similar to target displacement concept in FEMA).

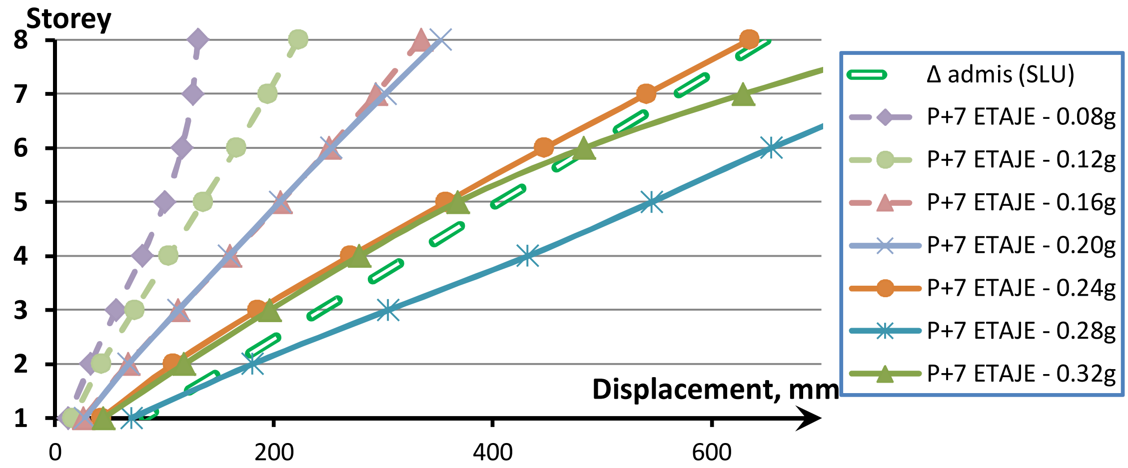


Fig. 4.44. Maximum storey response, expressed as absolute displacements function of PGA level, determined as the mean of the level responses of each set of 7 artificial accelerograms, 8 storied building

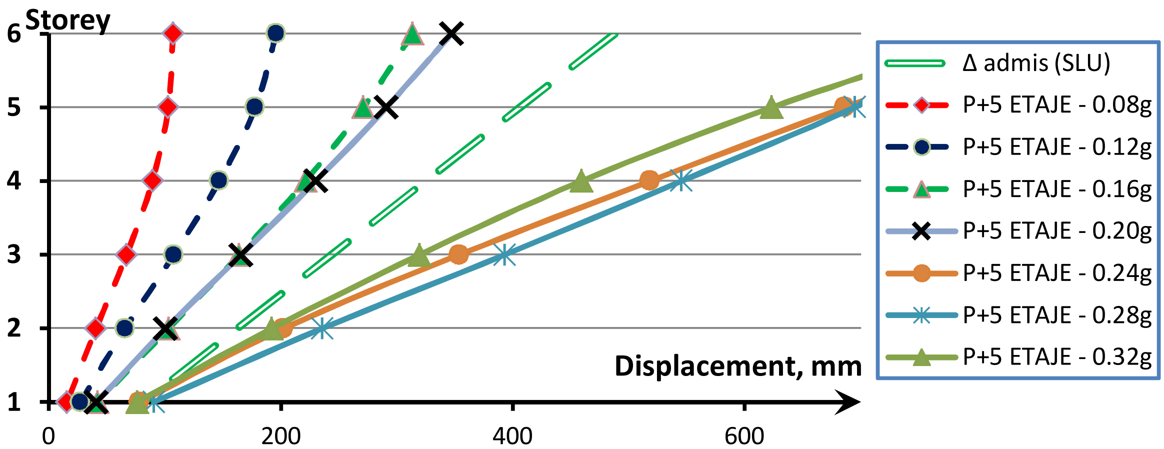


Fig. 4.45. Maximum storey response, expressed as absolute displacements function of PGA level, determined as the mean of the level responses of each set of 7 artificial accelerograms, 6 storied building

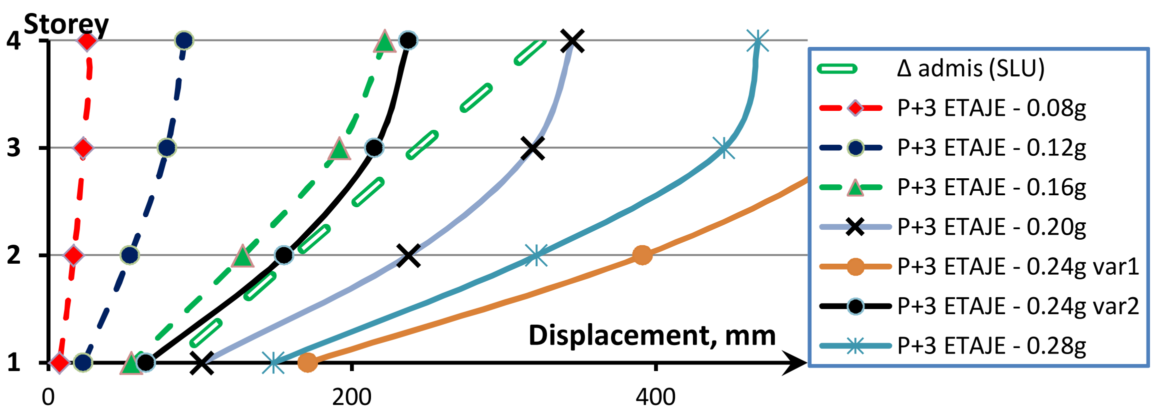


Fig. 4.46. Maximum storey response, expressed as absolute displacements function of PGA level, determined as the mean of the level responses of each set of 7 artificial accelerograms, 4 storied building

4.5.3. Maximum roof response function of the base shear force and PGA intensity

Next, the maximum absolute displacement response was determined at the top storey, recorded during the time-history analysis for each of the three structural types. It is thus put into evidence that the pushover analysis, even if it offers information on lateral force capacity of a structure, it is far from determining the detailed inelastic behaviour.

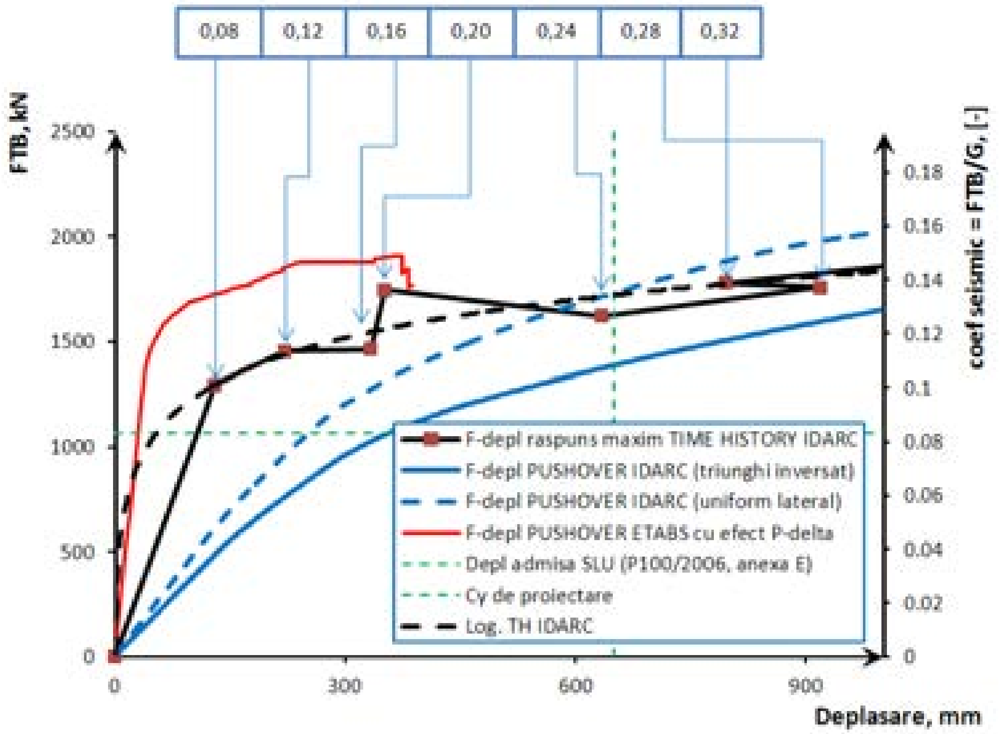
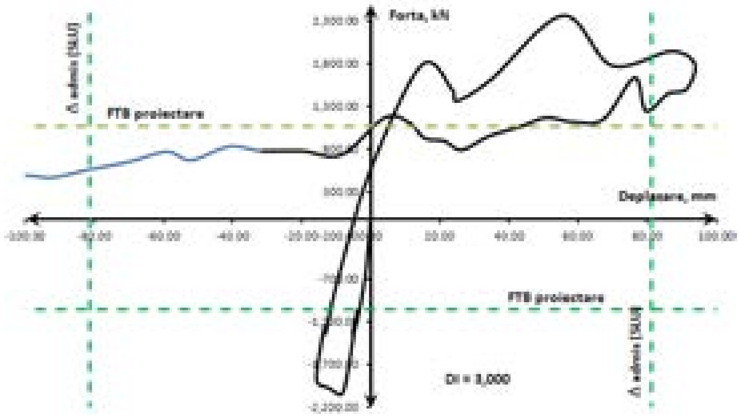


Fig. 4.479. Synthesis graph for force-displacement curves, 8 storied structure

4.5.4. Storey hysteretic curves

In order to analyse the damage level in the structures, the storey hysteretic curves are presented, function of the ground motion intensity.

Fig. 4.52. Hysteretic curve for first storey, P+7E, 0,40g, AA3



4.5.5. Time history of damage indices for columns and beams

The figures 4.57. … 4.65. present the recorded values of damage index for beams and columns, for 8, 6 and 4 storied structures, for PGA values of 0,40g and 0,24g. The values are computed as mean of the damage index values obtained from the individual sets of artificial accelerograms. The example shows only the first floor.

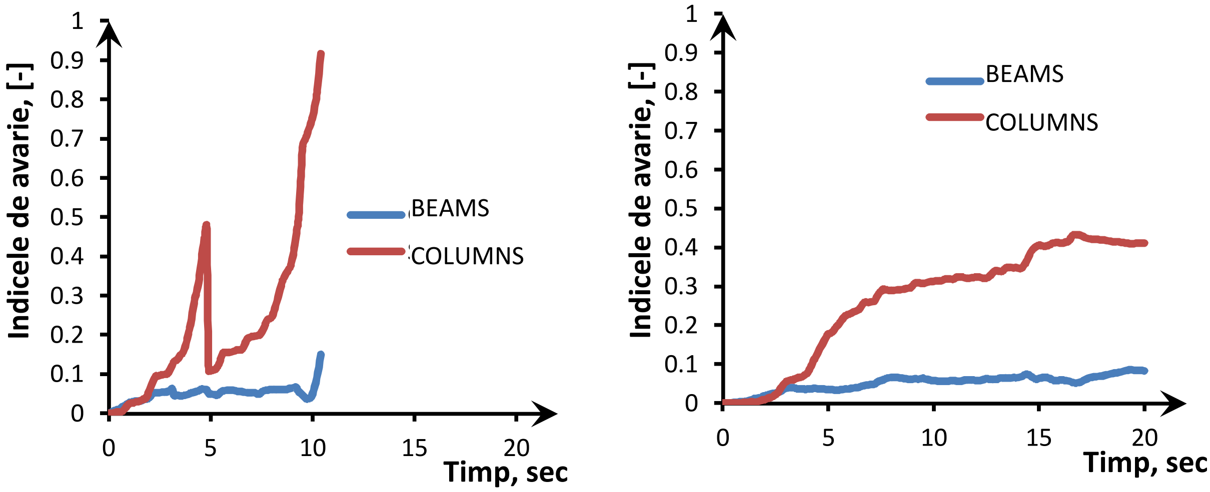


Fig. 4.60. Time-history of damage indices for columns (red) and beams (blue) at the first storey, P+5E, for PGA=0,40g (left) and PGA=0,24g (right)

Conclusions at sub-chapter 4.5.5. (in brief):

- One may notice that a correct design of the structure and a correct global hierarchy mechanism of elements allow the structure to withstand high horizontal forces, even if in some cases most of the beams already yielded at their ends. From the analysed cases, one may notice that in 1-2 cases out of 7, at the end of the seismic event, the structure is repairable, not mentioning anymore that the collapse is still far, even that the force is almost double the design force.

Comparative approach: beam-column damage for different PGA values

One may notice the for high PGA values, of 0,40g, the upper storeys have enough capacity reserve in the elements, but it is not being mobilised due to extensive damage at the bottom storey.

4.5.6. Global damage index

Following IDARC analysis, the global damage indices were determined, based on the seismic action intensity. For each intensity value, the global damage indices were computed as the mean of the indices obtained after subjecting the three structural models to the artificial accelerograms. Thus, for each PGA level, 7 values of global damage indices were obtained. The mean of their values was used in the graphs in figures 4.66-4.71, for 8, 6 and 4 storied structures [Tudose, 2012a].

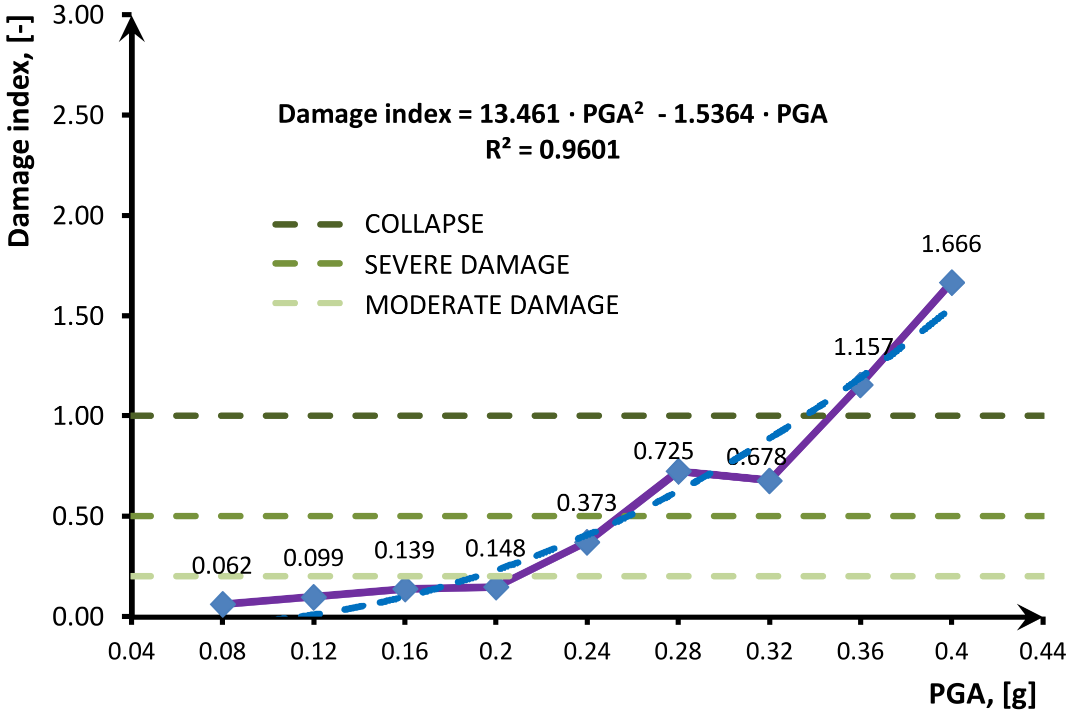


Fig. 4.66. Variation of mean global damage index function of PGA, 8 storied structure

Conclusions at sub-chapter 4.5.6.:

- In the analysis, only the structural damage was considered, without taking into account the contribution of the non-structural walls to structural stiffness.
- The relationships between PGA intensity and the Park and Ang damage indices may be considered as second degree equations.
- The increase of PGA level provides an increasing scattering of the global damage index values.
- The global damage index computed for the design level varies from 0,373, to 0,714 and to 0,842 for the 8, 6 and 4 storied structures. One may notice that for high rise structures, the damage level is within repairable domain. It is obvious that this thing cannot continue forever, no matter what the number of stories, but the trend is noticeable. As the number of stories decreases and the structure is stiffer, the recorded damage level increases, the 4 storied structure taking heavy damage, but without collapse.

5. Evaluation of damage index for reinforced concrete structures correlated to the intensity of the seismic motion – existing structures designed by previous codes

5.1. Technical context of the analysis

The engineering analyses performed for new structures have put into evidence a series of particularities which were commented in the previous 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 storey height is 3,30m, close to the values used for the new structures in chapter 4. The structure will be considered fixed above the basement, using only the storeys 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 4. The structure was analysed as a structure of importance class III (regular). According to the provisions of P100-1/2006, the site is characterised by a dynamic amplification factor $\beta = 2{,}75$, corner period $T_C = 1{,}0s$ and maximum PGA value of 0,24g, for a mean recurrence interval of 100 years.

Only the central longitudinal frame was analysed, 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 relation (6-1) from P100-1/2006 gives the following result:

$$\boldsymbol{F_b = 1 \cdot \frac{0{,}24 \cdot g \cdot 2{,}75}{4{,}725} \cdot m \cdot 0{,}85 = 0{,}119 \cdot G} \qquad \textit{(Ec. 5.1)}$$

Fig. 5.1. Dimensions of the frame elements, existing 4 storied structure

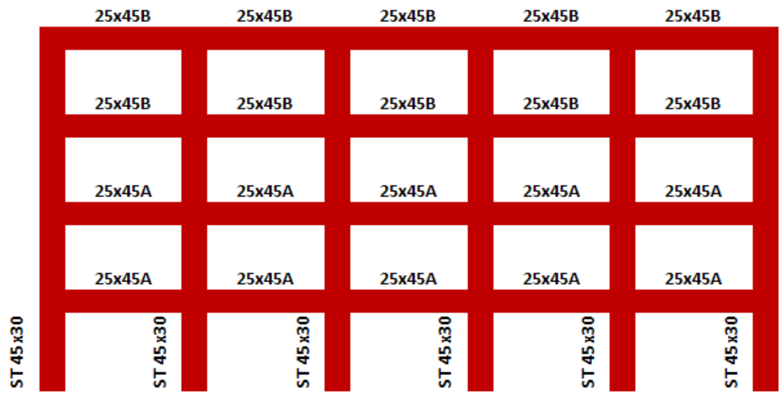
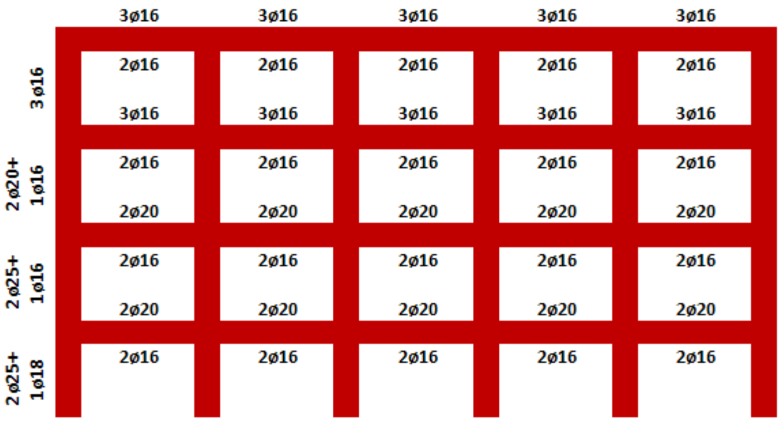


Fig. 5.2. Reinforcement of the beams and columns (side reinf.), existing 4 storied structure



5.2. Partial checking

In order to establish the level of confidence of the obtained models, a pushover analysis was performed for the longitudinal frame.

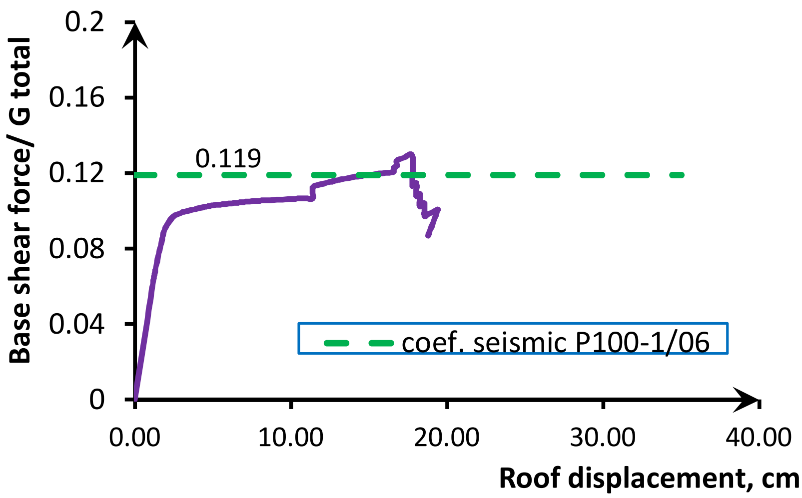


Fig. 5.4. Curba pushover pentru structura P+3E existentă, rezultate Etabs

From the point of view of the P100-1/2006 code provisions, the structure has not enough capacity to undertake the 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 ks, 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 of importance based on the 1970 code.

The static linear analysis performed gives the relative storey displacement values in Tables 5-1 and 5-2. 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 a class III structure.

Storey	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
					Max. value (0,7EI):		**10.18**
					Max. value (0,5EI):		**14.44**

Tabelul 5-1. Relative storey displacements, SLS, existing 4 storey structure

Storey	Direction	Load case	Point coordinates			Drift X	Drift SLU (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
					Max. value (0,5EI):		**2.89**

Tabelul 5-2. Relative storey displacements, ULS, existing 4 storey structure

5.3. Results of time-history analysis

The nonlinear dynamic analysis uses the same artificial accelerograms defined previously in chapter 4.4. The work algorithm is the same as in chapter 4.5, which means that the existing structure will be subjected to increasing level of PGA, each PGA level being defined by a set of 7 artificial accelerograms. Hence, 63 files of source code were written which were subjected to nonlinear dynamic analysis in IDARC 2D. The time-history analysis is based on nonlinear structural behaviour models.

5.3.1. Moment-curvature diagrams for beams and columns

Following Idarc analysis, the M – ϕ diagrams for beams and columns were plotted, for various PGA values. More graphs are presented in Annex 1.

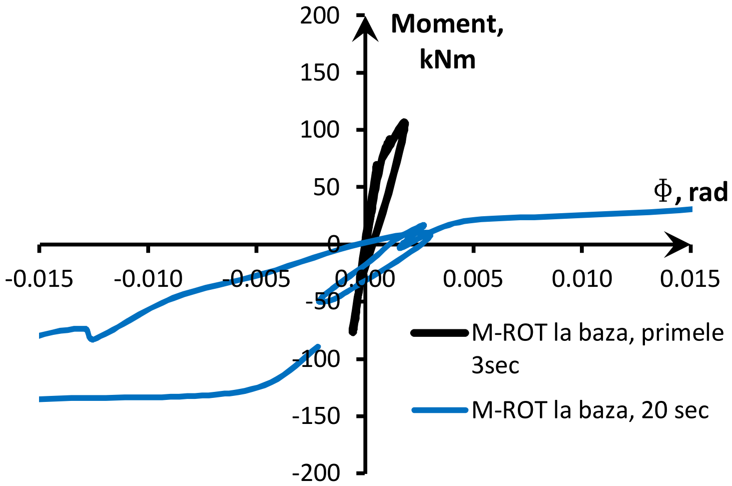


Fig. 5.5. Moment-curvature diagram at the base of column 1 (marginal), existing 4 storied structure during the artificial accelerogram no. 1, for PGA = 0,40g

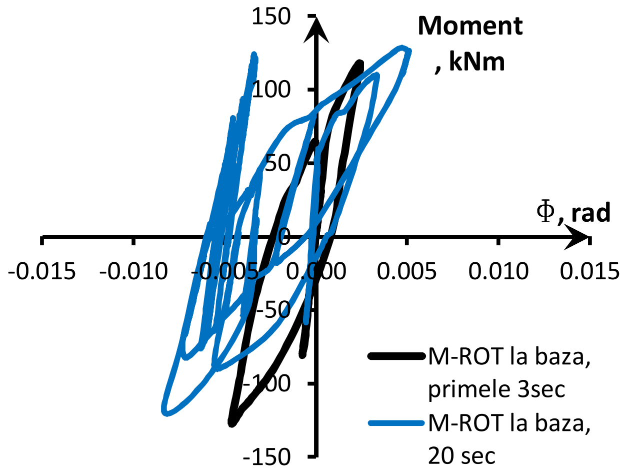


Fig. 5.6. Moment-curvature diagram at the base of column 1 (marginal), existing 4 storied structure during the artificial accelerogram no. 1, for PGA = 0,24g

The moment-curvature diagrams of the columns and beams show a behaviour with high rotation of the beams and with high and medium rotation for the columns subjected to 0,24g and 0,40 respectively. Practically, the existing structure subjected to 0,40g accelerations supplies values only in one case, in which the earthquake is defined by Accelerogram 1.

5.3.2. Maximum absolute displacement response on each storey function of the acceleration intensity

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. 5.9, for each of the considered intensity value.

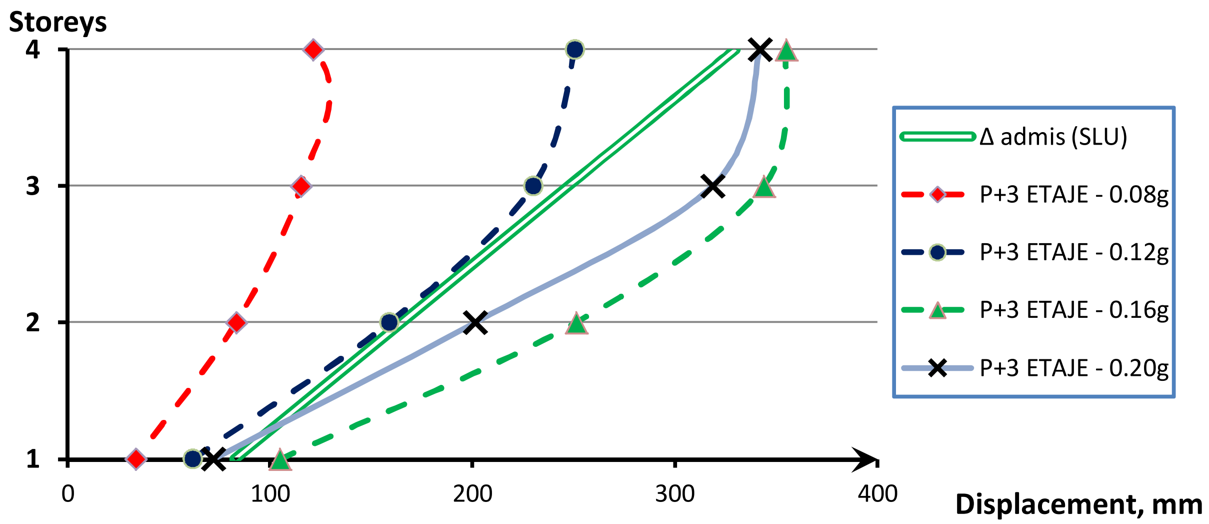


Fig. 5.9. Maximum storey response, expressed as absolute displacements function of PGA level, determined as the mean of the level responses of each set of 7 artificial accelerograms, existing 4 storied building

5.3.3. Maximum roof response function of the base shear force and PGA intensity

Next, the maximum absolute displacement response was plotted, as it was recorded during the time-history analysis for the considered existing building, obtaining the mean values of the maximum top storey displacement for each set of artificial accelerograms. For each PGA level defined by a set of 7 accelerograms, the maximum response function of the applied base shear force was plotted in fig. 5.10, together with two graphs obtained by two different methods and a separate graph formed by the pairs: (mean of roof maximum response; base shear force at PGA level).

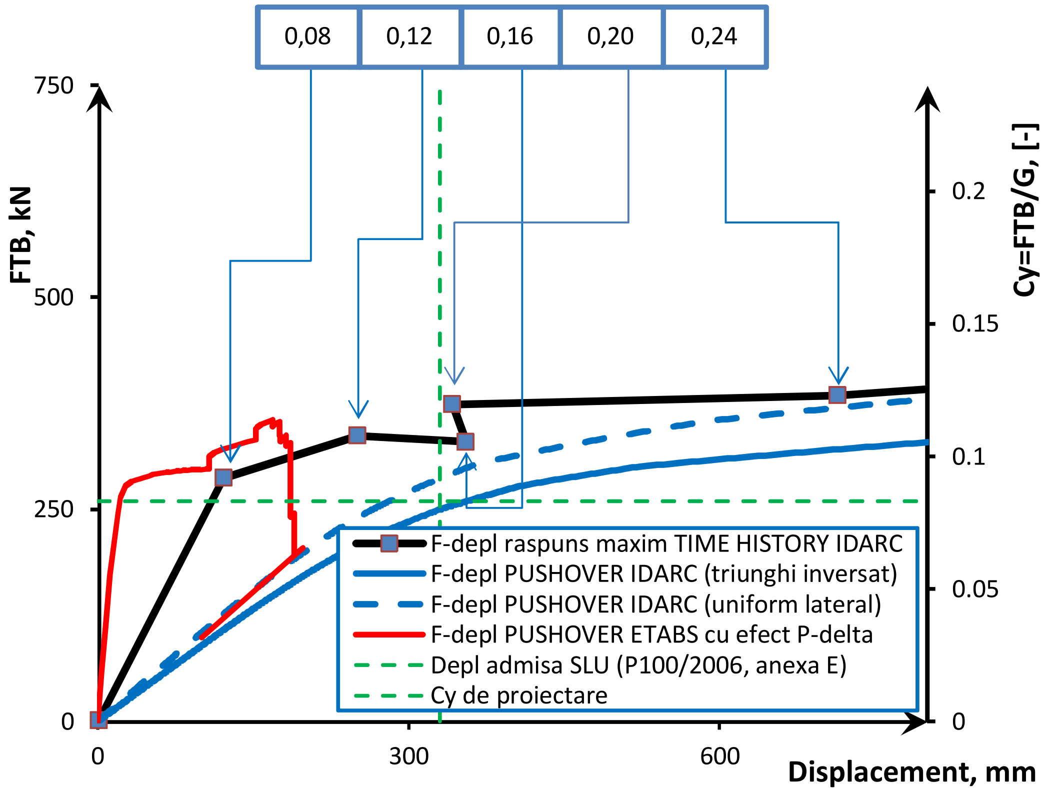


Fig. 5.10. Synthesis graph for force-displacement curves, existing 4 storied structure

5.3.4. Storey hysteretic curves

In order to further analyse the distribution of damage levels on the structure, the hysteretic storey curves function of the PGA intensity are presented. Some of the graphs for the first storey of the existent 4 storied structure are presented, for artificial accelerogram 1, for PGA of 0,08g, 0,24g and 0,40g. The graphs for all the analysed cases can be found in the annex (21 graphs).

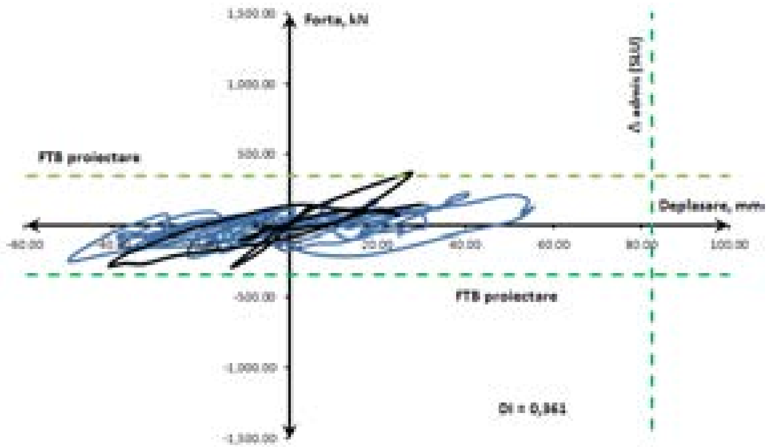


Fig. 5.12. Hysteretic curve for first storey, existent 4 storied structure, 0,24g, AA1

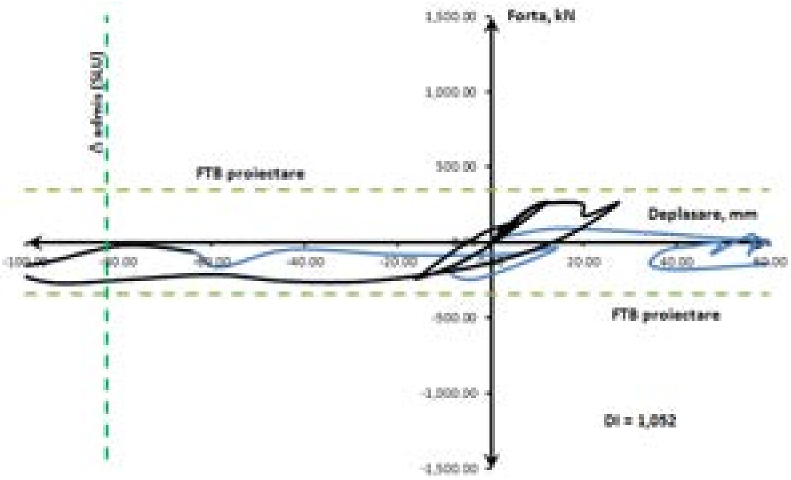


Fig. 5.13. Hysteretic curve for first storey, existent 4 storied structure, 0,40g, AA1

5.3.5. Time history of damage indices for columns and beams

Following the dynamic nonlinear analysis, the evolution of damage indices during the seismic action was presented, by plotting the index values over time. The graphs present the mean of the values obtained by processing the artificial accelerograms.

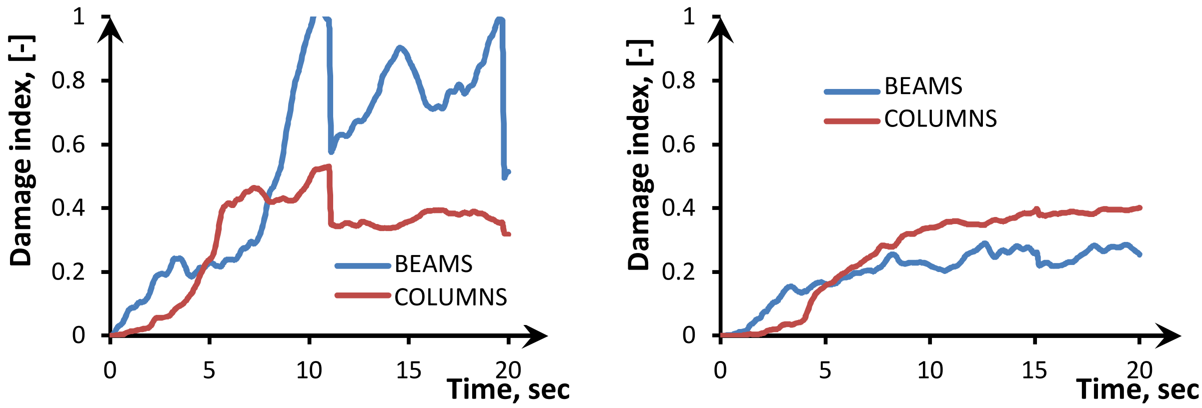


Fig. 5.14. . Time-history of damage indices for columns (red) and beams (blue) at the first storey, existing 4 storied structure, for PGA=0,40g (left) and PGA=0,24g (right)

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

The structure is noticed to lack capacity reserve on the storeys that show the highest story drifts (especially the second storey). 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.

5.3.6. Global damage index

The adopted analysis model allowed the plotting of the mean global damage indices correlated to the seismic motion intensity acting on the structure (Fig. 5.17). The values of the damage indices show in fig. 5.17 are computed as the mean of the 7 individual damage values for each of the seismic motion intensity, similar to the procedure applied in chapter 4.5.6. Beyond this, Fig 5.18 presents the scattering of the individual values of the damage indices on the whole interval of considered PGA values.

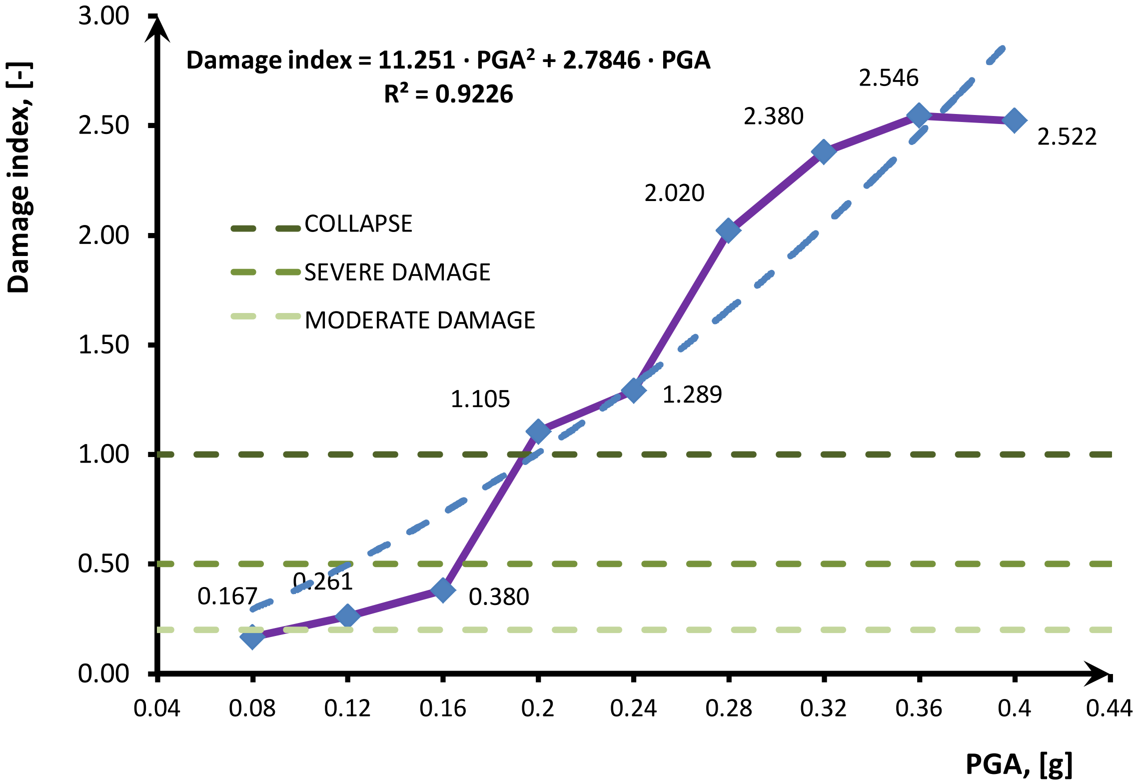


Fig. 5.17. Mean global damage index variation function of PGA, existing P+3E structure

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.

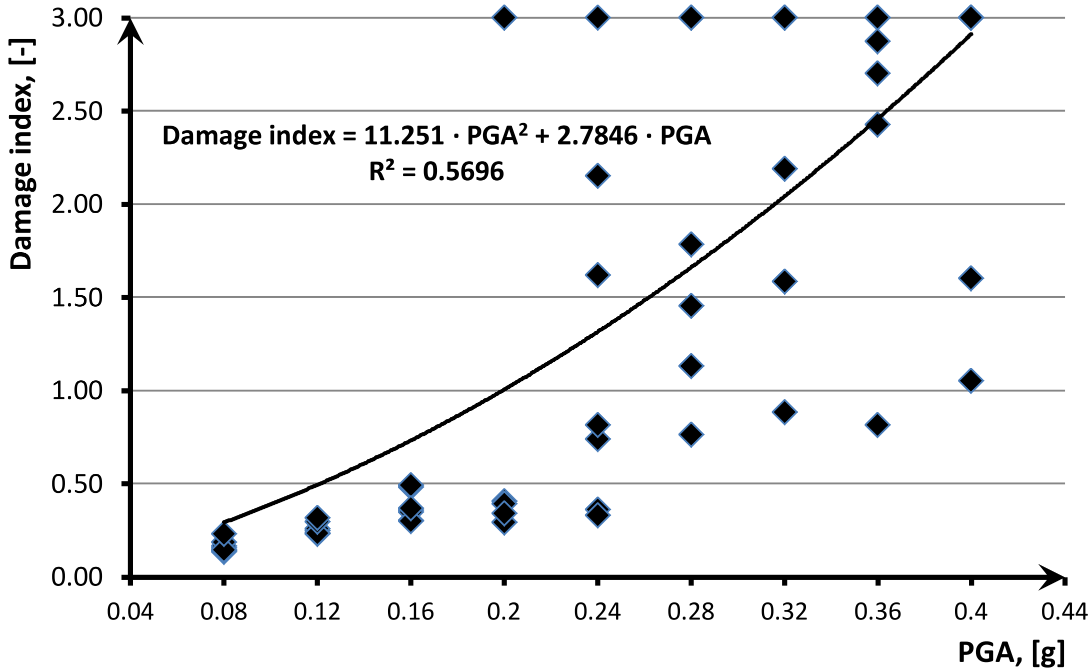


Fig. 5.18. Individual global damage index values scattering function of PGA level, existing P+3 structure

5.3.7. Comparison between the two analyses

Fig. 5.19 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.

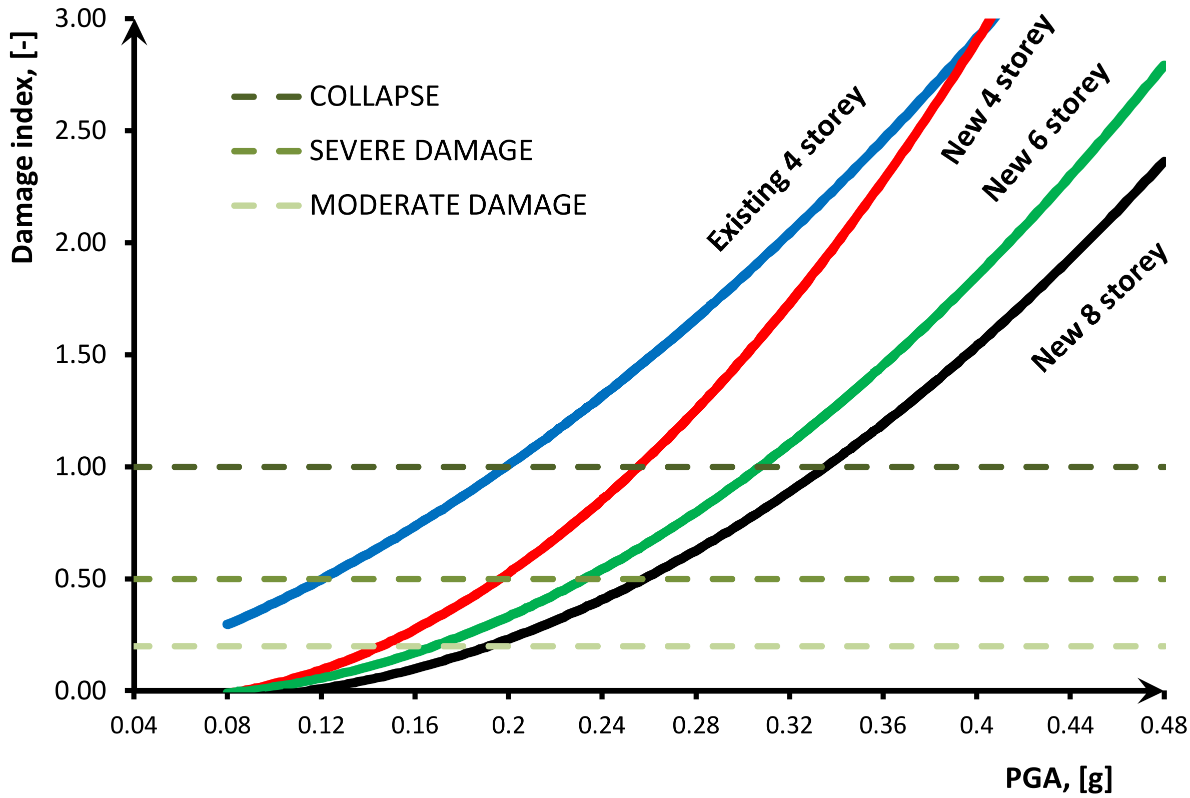


Fig. 5.19. Comparative graph between variation curves of the mean global damage index of new and existing structures

It is noticeable that the tendency of new structures is to show in the beginning a moderate increase of damage index value, characterised by concave diagrams. In the mean time, 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 [Tudose, 2012b].

6. Conclusions and personal contributions

This chapter highlights the general conclusions of the thesis and the author's contributions to the general knowledge domain regarding building damage and future research works.

6.1. General conclusions based on the studied literature

- The dynamic nonlinear analysis gives the most accurate results of the whole computational methods at the disposal of the structural engineers.
- The nonlinear methods of analysis may be used if the design seismic action is thoroughly calibrated and a nonlinear behaviour model for the elements is chosen and the correct interpretation of the results is performed.
- The definition of the seismic action is possible through recorded or artificial accelerograms. The recorded accelerograms may be used if they are recorded near the desired location. The maximum value of the acceleration must be scaled to the same acceleration value of the location and the frequency content must be compatible to the local soil condition, which limits their applicability to certain fixed locations.
- Because of the lack of recordings in the many necessary specific locations, the artificial accelerograms represent the best way of defining the seismic action while using the minimum number of accelerograms specified by the codes.
- The total energy released by an earthquake that acts upon a building is dissipated through structural damping and through hysteretic energy (inelastic deformations). The damage level is due to the inelastic dissipated energy. The evaluation of the damage level is performed based on specific energy based damage index, which give information with respect to the degradation state of the structure.
- After literature studying, it is necessary to define the seismic effects of some buildings subjected to higher earthquakes that the design situation imposes, also due to the tendency of increasing the mean recurrence interval in the last editions of the P100 code.

6.2. General conclusions of the study

- From the graphs in Fig. 4.37 … 4.42 for the new structures, on can notice the time variation of moment curvature relationship, with elastic behaviour and light stiffness reduction for PGA values of 0,08g. Increasing the PGA towards 0,40g value, the stiffness slope is reduces (meaning stiffness reduction) and also decrements in strength. In the case of high PGA values, the response is completely out of the elastic domain even in the first few seconds of seismic action.

- It may be concluded that for various artificial accelerograms that have been generated for the same elastic design spectra, the answer presents a high variance when the design base shear force is exceeded.
- When the PGA value is above the design value (example is for 0,40g), the shapes of the hysteretic curves are consistent with the expected plastic response, but, in other cases, the structures do not stand more than 3-4 seconds before failure. In some cases with PGA of 0,40g, the structure may withstand up to 50% more seismic force, obviously in a reduced number of the total cases, between 14-28%, this thing being dependent also on the fundamental period of the building to the elastic spectrum of the used accelerogram.
- The time history graphs of the damage indices and the distribution of their values over the height of the building corresponds to the storeys with large relative displacements obtained in linear elastic analysis. The damage level is higher on the storeys positioned in the low third of the building, where the beams fail before the columns. The upper storeys have enough capacity left in the elements, but it is not mobilised due to the extensive damage in the lower storeys.
- It may be concluded that a correct dimensioning of the structure and inducing a correct global hierarchy mechanism (strong columns – weak beams) gives a high degree of lateral force capacity, even if some of the beams have developed plastic hinges at their ends. From the analysed cases, 1-2 of 7 cases show that the structure is repairable at the end of the seismic event, not mentioning the collapse prevention case which is verified in 28% of the cases, even that the base shear force is double the design value.
- The negative jumps visible on the left diagrams in Fig. 4.60, 4.61, 4.62, show that one or more iterations concluded due to collapse and the values of the damage index were impossible to compute or returned very high non-realistic values.
- The relationships between the PGA intensity and the values of Park And damage index may be considered as second degree equations, presented in Fig. 4.66, 4,67, 4,68 for the new structures and in Fig. 5.17 for the existing structure.
- The values of PGA return and increasing scattering for the global damage index values, as the seismic intensity is increased.
- The new structures have a plus of 6-41% in capacity above the required design level.
- It may be concluded that the evolution of the damage index values for the existing structures is steeper than for new structure. For the existing structure, the repairable state (corresponding to moderate damage) is situated at half of the actual design value.
- The tendency of the new structures to exhibit a moderate increment of the damage index values is observed, characterised by concave diagrams. By contrast, the existing structure shows a graph with a fast increment of damage values, especially in the low-PGA range.

6.3. Personal contributions

During the completion of the thesis, several elements of originality were adopted and inserted in the research program. Some of them are mentioned below:

- Realisation of case studies on new and existing structure on the evaluation of damage index based on the seismic action, defined by artificial accelerograms and with nonlinear modelling of the structural elements;
- Presentation of a direct method for evaluation of relationship between the seismic motion intensity and the damage index and determination of the mathematical correlation between the maximum ground motion intensity and the damage index as second degree equations, with different relationships for new and existing buildings;
- 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 meant working with approx. 1260 files, with hundreds of values each, on multiple columns. The values were used for drawing the graphs in chapters 4 and 5 and also in Annex 1;
- Evaluation of maximum displacement response on each storey function of the increasing PGA value, for structures with various heights and presentation of the time-history during-earthquake evolution of the beam and column damage index, at various levels of seismic intensity and on each storey of the building.
- The study of influence of the height regime on the damage level of the structures, correlated to the seismic ground motion intensity.

6.4. Valorification of the results

The research work was published in several papers:

- **Internationally acknowledged :**
  - **Tudose C.**, *Study on evaluation of damage index correlated with the intensity of the ground seismic motion*, Proceedings of the International Conference „First International Conference for PhD students in Civil Engineering”, Section: Structural Analysis and Design, Technical University of Cluj-Napoca, pp. 289-296, ISBN 978-973-757-701-8, Eikon Publishing, 2012, rating B+, index BDI;
  - **Tudose C.,** Comparative study on damage index evaluation for new and existing buildings, ACTA TECHNICA NAPOCENSIS, Section Civil Engineering and Architecture, 2012-2013, (sent for publishing), rating B+, index BDI;
  - Filip C**.**, **Tudose C.**, Breabăn V., *Disaster mitigation – A General Survey*, Proceedings of the International Conference „Constructions 2008”, Technical University of Cluj-Napoca, ACTA TECHNICA NAPOCENSIS, Section Civil

Engineering, No.51, vol.I, pp. 123-130, ISSN 1221-5848, Napoca Star Publishing House, 2008, rating B+, index BDI;

- Nationally acknowledged :
  - **Tudose C.,** Breabăn. V., *Aspecte privind execuţia de goluri nebordate în diafragme de beton armat,* Simpozionul de Inginerie Civilă din cadrul Facultăţii de Construcţii, Ovidius University Press, Constanţa, 2010, rating B;
  - Filip C., Breabăn V., **Tudose C.**, *Mitigation Strategies Used to Reduce the Effects of Natural Hazards*, Ovidius University Annals of Constanta, Series Constructions, vol.10 (2008), pp. 19-26, ISSN 1584 – 5990, Publisher Ovidius University Press, Constanta, 2008, rating B+, index BDI.

6.5. Future research works

The study may be continued on the following ideas:

- Analysing other structures that were initially dimensioned to other PGA values, with more interest in the low lateral force designs, especially with PGA of 0,20g and 0,16g;
- Establishing of a correlation for other types of reinforced concrete structures, such as concrete shear walls;;
- Research of the influence of velocity and displacement intensities on the damage indices.
- Extension of the analyses to 3d models with the consideration of P-delta effect;
- Extension of the analyses to existing buildings with more than 4 storeys;
- Determination of the correlation between the damage index and the seismic intensity for structure with various bays and spans and establishing a pattern;
- Consideration of the non-structural walls effect on strength and stiffness on the final damage of a structure;
- Consideration of a two-peak seismic event.

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7. Annex 1

Annex 1 presents the graphs obtained during the case studies, which were not presented in chapters 4 and 5.

7.1. Absolute acceleration response spectra

7.2. Artificial accelerograms – generated based on the acceleration spectra

7.3. Hysteretic storey curves

7.3.1. New structure P+7E (graphs for 0,40g and 0,24g)

7.3.2. New structure P+5E (graphs for 0,40g and 0,24g)

7.3.3. New structure P+3E (graphs for 0,40g and 0,24g)

7.3.4. Existing structure P+3E (graphs for 0,40g, 0,24g and 0,08g)

8. Annex 2 – Stochastic response of structures

The annex is prepared based on the chapters in the book Dynamics of Structures written by Clough, R.W and Penzien,J. [16].

8.1. Essential function in stochastic analysis of the structures

8.1.1. Transfer functions

8.1.2. Relationship between the unitary pulse functions and complex frequency response functions

8.1.3. Relationship between auto-correlation function of action and response

8.1.4. Relationship betweeen power spectral density, action and response functions

8.2. Stochastic response of real structures

8.2.1. Response in time domain of linear systems

8.2.2. Response in frequency domain of linear systems

8.2.3. Response to random loads

8.2.4. Response to disperse loads