BULETINUL INSTITUTULUI POLITEHNIC DIN IAȘI
Publicat de
Universitatea Tehnică „Gheorghe Asachi” din Iași
Volumul 65 (69), Numărul 1, 2019
Secția
CONSTRUCȚII. ARHITECTURĂ

DUCTILITY – CONCEPT FOR IMPROVING THE SEISMIC RESPONSE FOR STRUCTURAL REINFORCED CONCRETE FRAME SYSTEMS

BY

ION SOCOCOL*, PETRU MIHAI and IOANA OLTEANU-DONȚOV

Technical University “Gh. Asachi” of Iasi,
Faculty of Civil Engineering and Building Services

Received: February 1, 2019
Accepted for publication: March 10, 2019

Abstract. The reinforced concrete frame structures are used with confidence by structural engineers in seismic areas, being available a theoretical methodology for designing and evaluating the seismic response for these systems. However, following the recent severe seismic actions in the world (Japan, 2011, Magnitude = 9.1, 20896 dead), it can be noticed that the seismic response of reinforced concrete frame structures does not coincide with the mechanisms of dissipation earthquake energy considered theoretically. The ductile design concept of these structural systems deserves particular attention and effective improvement. Thus, it is necessary an extensive knowledge of the phenomena occurring in these types of dissipative reinforced concrete structures, with adequate details about material/section/element and structure ductility, and the seismic response of the designed inductile structural systems.

Keywords: energy dissipation; structural ductility; control of deformations; plastic hinges; yield of the structural assembly.

*Corresponding author: *e-mail:* ion.sococol@tuiasi.ro

1. Introduction

The severe seismic actions of recent years demonstrate the necessity for efficient informations about the dissipative concept and consequently, some improvements by the designers. In order to make a practical contribution in this direction, it is necessary to seek an answer to the question: Why we should design using the ductil concept?

Some of the answers would be:

- is chosen the ductility control to avoid the possibility of non-economic design of structures (Stratan, 2014);
- it is touched the optimal level (with incursions in the inelastic domain) of the design/execution/ response of the structure to severe seismic actions (due to the inelastic response structure and overstrain) (Gioncu & Mazzolani, 2002);
- the response of the structural system to seismic action can be controlled through the limitations of lateral displacements (Paulay & Priestley, 1992);
- it is easy to use in the measure of the current development of computational programs/methods/principles;
- the performance objectives (life safety and limiting degradation) are being achieved successfully (with the norm P100-1, 2013);
- because it is not easy to implement another concept (ensuring design simplicity for designers), and this concept is still in constant change.

2. Concept of Dissipative Design of the Structure

The concept of ductile design refers to the ability of a structural system to ensure certain zones with specific length who can develop deformations (this regions are called ”plastic hinges”), to a severe seismic action, with limited displacements to prevent the collapse of the structure (Budescu & Ciongradi, 2014).

Short history (Gioncu & Mazzolani, 2002):

- 1934 (*Benioff*); 1941 (*Biot*) – The first concept who considers the elastic response spectrum;
- 1935 (*Tanabashi*) – has been proposed a theory by which the seismic response capacity of a structures can be evaluated by the energy absorbed by the structure before collapse;
- 1956-1959 (*Housner*) – the first attempt to combine two aspects: spectral response and seismic energy dissipation through plastic deformations;

- 1960 (*Velestos* and *Newmark*) – the first study of the inelastic spectrum;
- 1969 (*Newmark* and *Hall*) – has been proposed a new design concept and have been elaborated the response spectra in accelerations, velocities and displacements on a range of short, medium and long periods. After the earthquakes in Northridge (1994) and Kobe (1995), the importance of the response spectra in velocities and displacements has been accentuated.

The first countries concerned with the formulation of this very important concept were: Japan, US and New Zealand.

2.1. United States of America (USA)

The 25-year period of 1960-1985 represent the ”mature years” for building design and execution in the US, because the ductile design concept is used with confidence in ductile steel frame structures and reinforced concrete structural walls (FEMA 454, 2006).

As has been seen in the short history, Housner, Newmark and Hall set the foundation of a new design concept, but which could certainly be implemented in the US, just after the earthquake of San Fernando, California (M = 6.4) in 1971 (Ishiyama, 2011). Thus, in collaboration with Japanese researchers, American engineers have researched and studied the results of the two earthquakes: San Fernando and Tokachi-oki. These studies led to the adoption of a new national code for designing structures, including the concept of ductile design (BOCA, 1975).

Besides these, we should also specify the first attempts to define ductility and ductile design concepts in US norms up to 1971:

- in (UBC, 1970) and in (ICBO, 1970) the structures could be designed only to overcome a moderate seismic motion;
- in (ACI 318 code, 1971), recommendations are made for longitudinal reinforcements that have to be provided in critical sections and which allow the redistribution of bending moments. The norm also includes for the first time an appendix with special provisions for seismic design (Park & Paulay, 1975).

Finally, it can be specified that structural engineers validate (use) the new concept with confidence in this period (FEMA 454, 2006):

1) steel-concrete composite structures;

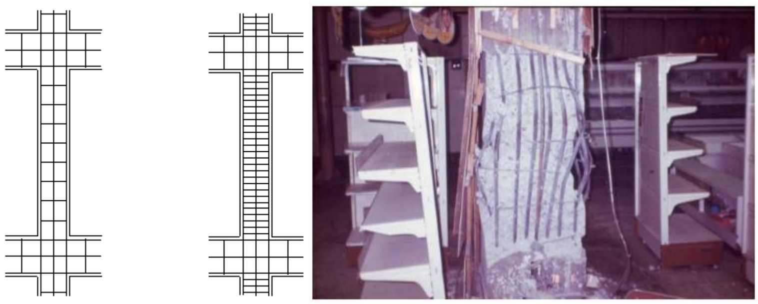
2) structural reinforced concrete walls;

3) welded steel frames, being considered as the fundamental structural resistant systems to lateral loads.

2.2. Japan

As seen in the short history, until the early 1970, this new design concept was just under investigation. The radical change in the consideration of ductility (material ductility, section ductility, element ductility, ductility of the structural assembly) occurred in Japan following the Tokachi-oki earthquake in 1968 (M = 7.9; 52 dead; 300 wounded, 673 collapsed structures, 3004 structures with partial collapse). This change has proven to be effective (all the measures taken to design structures after 1968 in Japan, especially the reduction of the distance between the stirrups for the columns, (Fig. 1 *b*), being visible after the Hyogo-ken-nanbu earthquake from Japan in 1995 (Ishiyama, 2011).

Thus, during the years 1972-1977, a research project was carried out to establish a new method of seismic design, in collaboration with the Ministry of Construction, the Research Institute for Public Works, universities, private companies and many other organisations whose interest was primordially in this direction. Under these conditions, a new method of seismic design was proposed in 1977 with implications in the deformation capacity of the structures.



a *b* *c*

Fig. 1 – Decreasing distance between stirrups: *a* – before 1970 (s = 30 cm); *b* – after 1971 (s = 10 cm); *c* – Deteriorate reinforced concrete column due to extensive distance between transverse reinforcing bars corresponding to the reinforcement of *a* (Ishiyama, 2011).

In 1978, the Miyagi-ken-oki earthquake (M = 7.4, 28 dead, 1,325 wounded, 1,183 collapsed structures, 5,574 structures with partial collapse), (Fig. 1 *c*), hits the Sendai region and hastened the adoption of the new seismic design method, which only in 1981 is implemented and appears in legal form in the normative (BSLJ, 1981).

2.3. New Zealand

It can be said for sure that engineers in New Zealand started using the new ductile design concept much earlier than those in Japan or the US. Since 1965 (NZS 1900, 1965) it has become necessary to recognize ductility as an essential parameter in seismic design, but could not be used due to lack of guidance. However, it was understood that structures loaded with large horizontal forces are less ductile (Fenwick & MacRae, 2009).

In 1970, according to (MOW, 1970), it was recommended to use the ultimate strength method for designing structural elements. This document was extended with design criteria that included the requirement of reinforcement the frame nodes and the requirement for the columns to be confined to the ends and the sum of the moments in the columns around the node to be greater than the sum of the moments in the beams. Also, was not specified the stiffness contribution of the longitudinal reinforcement in the plate to beams. It can be said that the year 1970 is a representative for knowing the ductile design concept, that it has reached a very high level of knowledge (Fenwick & MacRae, 2009).

In 1976 the use of limit state design according to (NZS 4203, 1976) was fully legalized and some of the rules of the previous law were retained.

Next, are specified the most important innovative elements brought through the norms (NZS 1900, 1965), (MOW, 1970) and (NZS 4203, 1976):

- The sum of the moments in the columns around a node is greater than the sum of the moments in the beams;
- Inadequate resistance to shear force in an element in the potential plastic region is not reduced by the coefficient υ_c;
- The transverse reinforcement (stirrups) in the column-beam nodes is insufficient;
- The transverse reinforcement in the vertical elements (columns) is insufficient;
- Excessive buckling of longitudinal reinforcements in potential plastic areas (due to insufficient transverse reinforcement).

3. Ductility of Reinforced Concrete Frames Structures

The design of reinforced concrete structures according to the principle of dissipative behavior of the structure implies the necessity to obtain ductile behavior at global level. Thus, it is necessary to ensure a structure with sufficient ductility at the level of material, section, element and nodes (Fig. 2) (Budescu & Ciongradi, 2014; Stratan, 2014).

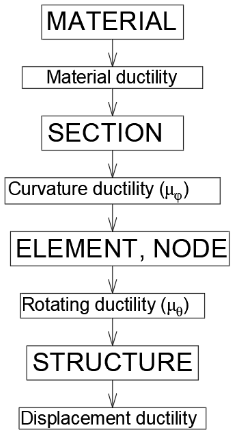


Fig. 2 – The connection between ductility (Budescu & Ciongradi, 2014).

3.1. Material Ductility

The material ductility μ_ε is defined as the ratio between the ultimate specific deformation ε_u, which corresponds to the failure of the material, and the specific deformation corresponding to yielding ε_y (Eqs. 1) (Fig. 3 *a*). Thus, ductility determines where the ultimate specific deformation of the material can reach, beside the specific deformation corresponding to the yield of material (Budescu & Ciongradi, 2014).

$$\mu_\varepsilon = \frac{\varepsilon_u}{\varepsilon_y}, \tag{1}$$

where: ε_u is the ultimate specific deformation, ε_y – the specific deformation corresponding to yielding, μ_ε – material ductility.

Material ductility proves that it is the basis of the system ductility, but it is limited in the context of the limit values for structural elements (Budescu & Ciongradi, 2014).

Although apparently the concrete is not a ductile material, but it actually has a high compressive deformation capacity after reaching the maximum stress σ_m, and if the concrete is confined (impeding transverse deformations), the deformation capacity significantly increases (Budescu & Ciongradi, 2014) (Fig. 3 *b*).

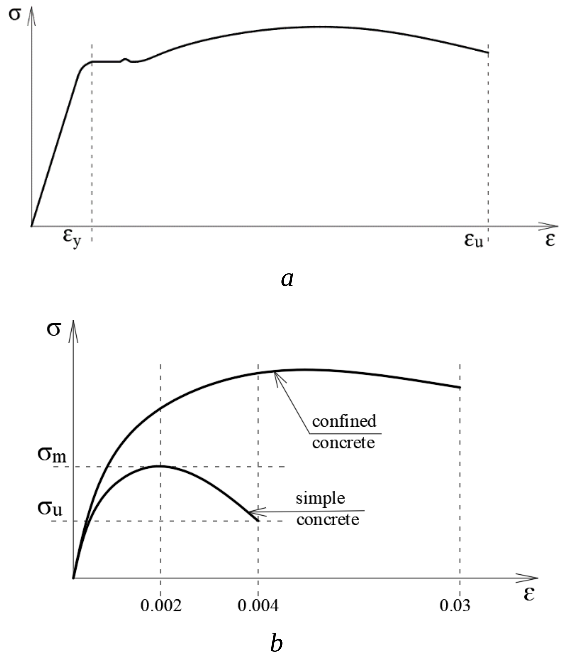


Fig. 3 – *a* – Ductility of usual steel (material ductility); *b* – Characteristic curve of concrete and effect of confinement (Budescu & Ciongradi, 2014).

3.2. Section Ductility

If it looks at reinforced concrete sections, the curvature ductility μ_ϕ is determined in relation with the rotation of the cross-section, determined by the curvature, under the action of a bending moment (Budescu & Ciongradi, 2014) (Fig. 4). In this way, the curvature ductility μ_ϕ represents the ratio between the ultimate specific curvature ϕ_u and the specific curvature corresponding to yielding ϕ_y (Eqs. 2).

$$\mu_\phi = \frac{\phi_u}{\phi_y} \tag{2}$$

where: ϕ_y – specific curvature corresponding to yielding, ϕ_u – ultimate specific curvature, μ_ϕ – curvature ductility (section ductility).

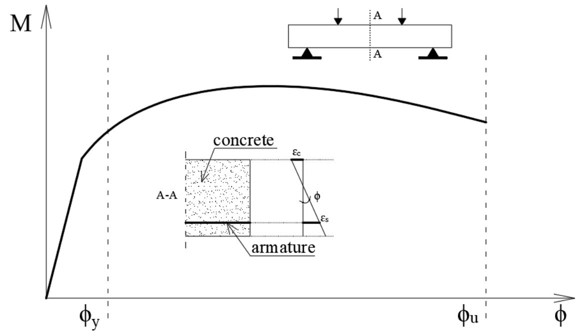


Fig. 4 – The ductility of a reinforced concrete section (curvature ductility) (Budescu & Ciongradi, 2014).

For the case of reinforced concrete beams, the moment-curvature relationship is directly connected with the ratio between the compressive concrete strength and the reinforcement quantity. The ideal and favorable case of yielding a concrete cross section is that the crushing of the concrete occurs at the same time with the yielding of steel. Thus, ductility is higher if the concrete crushing is produced later than the yielding of the reinforcement. If is available too large quantities of reinforcement, it may be imposed the brittleness crushing of concrete by destructive effect in the compressed area without yielding of the reinforcement bars (Budescu & Ciongradi, 2014).

3.3. Element Ductility

If the ductility analysis is performed on a structural system where punctiform plastic joints are established as a result of the effects of sectional moments, ductility is defined by the rotation of these elements and is called rotating ductility (element ductility) (Budescu & Ciongradi, 2014).

In these conditions, the ductility of the element μ_θ represents the ratio between the ultimate capable rotation of the section in the structural element studied θ_u and the rotation corresponding to yielding of the plastic joint θ_y (Budescu & Ciongradi, 2014) (Eqs. 3):

$$\mu_\theta = \frac{\theta_u}{\theta_y} \tag{3}$$

where: θ_y is the rotation corresponding to yielding of the plastic joint, θ_u – the ultimate capable rotation of the section in the structural element, μ_θ – element ductility.

3.4. Structural system ductility

The total capacity for appearance in a structural system of potentially plastic zones, defines a global ductility that is expressed by displacements. Thus, the global ductility μ_Δ can be expressed in the form of a ratio between the ultimate displacement of the system d_u and the displacement corresponding to the global yielding of the structure d_y (Budescu & Ciongradi, 2014) (Eqs. 4):

$$\mu_\Delta = \frac{d_u}{d_y} \tag{4}$$

where: d_y – displacement corresponding to the global yielding d_u – the ultimate displacement of the system, μ_Δ – global ductility of the structure.

Even if on a structural system it cannot discuss about a fixed moment of global yielding (perhaps only a close moment generated by the influence of each plastic hinge developed in the system (Fig. 5)), the formation of the first plastic hinge, in the vast majority of cases, is considered the orientation point in defining of structural yielding (Budescu & Ciongradi, 2014).

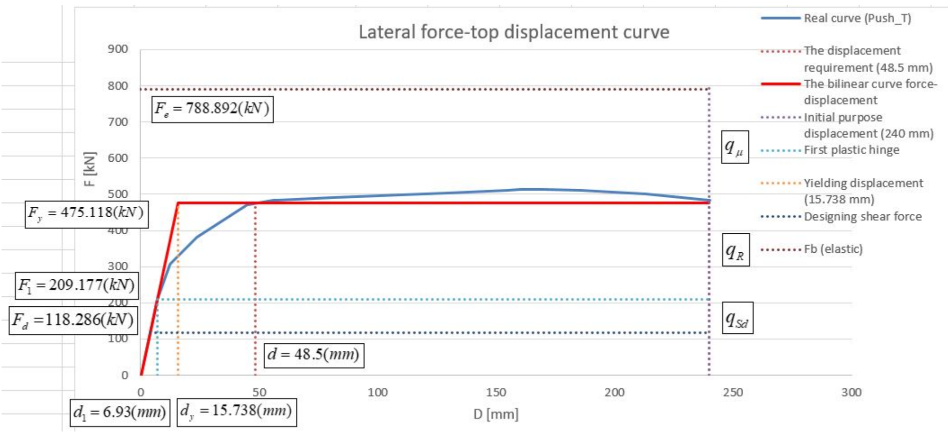
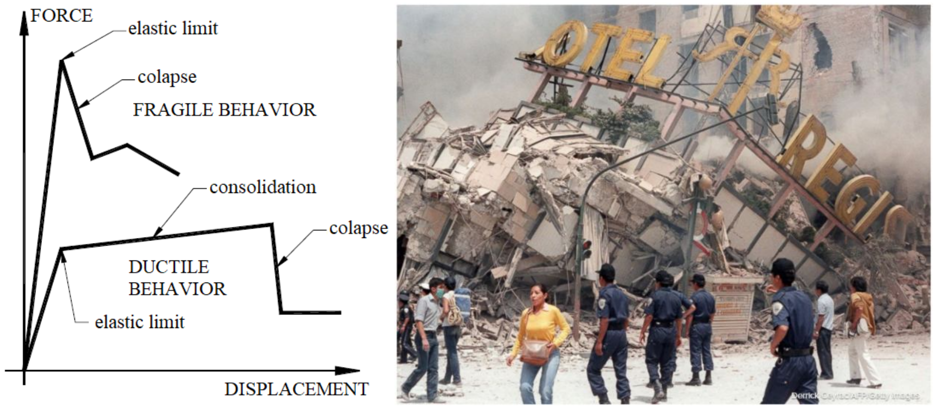


Fig. 5 – Representation of seismic force reduction factors in a Push-Over analysis.

4. Non-Ductile Designed Constructions

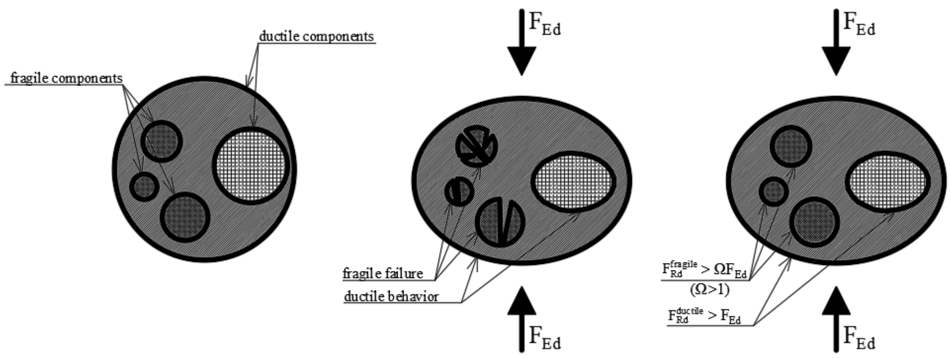
The non-ductile designed structures have a negligible ductility because after the yield limit is reached, the force records a brittle (fragile) degradation (Fig. 6 *a*). The major danger is behind the unpredictable nature of seismic action and not in the non-ductile design of structural system. In other words, if it's designed a structure in elastic (for q = 1) for certain seismic characteristics in the site and if the seismic action exceeds the elastic structural capacity, the collapse is inevitable (Fig. 6 *b*).



a *b*

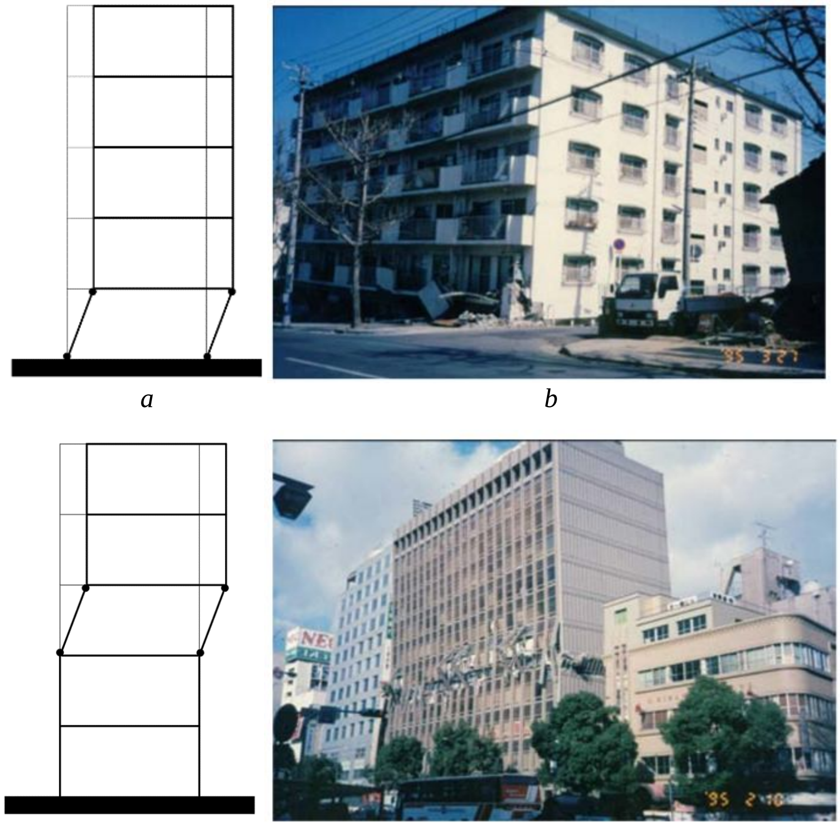
Fig. 6 – (a) The basic representation of the ductile and fragile behaviour of the structure (Stratan, 2014); (b) Ruins of Hotel Regis, Mexico City,1985 (http://abcnews.go.com).

These types of structures are characterized by a significant linear rigidity, and all the rigid structural elements (Fig. 7) who only work in the elastic domain, are not abled with a plastic redistribution capacity of the sectional efforts (overstrenght of the structure is based only on overstrenght of design). Consequently, the most requested structural elements (the most rigid elements) collapse abruptly, leading to a partial collapse of a structure (the soft story mechanism) (Fig. 8).



a *b* *c*

Fig. 7 – *a* – Scheme of a ductile assembly with fragile components; *b* – Collapse of fragile components in the case of undersize resistance capacity; *c* – Preserving the integrity of fragile elements through oversize (Budescu & Ciongradi, 2014).



a *b*

c *d*

Fig. 8 – Soft story mechanism: *a* – Structural representation (Budescu & Ciongradi, 2014); *b* – The Hyogo-ken-nanbu earthquake of 1995, Japan (Ishiyama, 2011); Upper soft story mechanism: *c* – Structural representation (Budescu & Ciongradi, 2014); *d* – The Hyogo-ken-nanbu earthquake of 1995, Japan (Ishiyama, 2011).

However, if a seismic design is desired in elastic, then seismic load for design should be determined based on elastic response spectrum, (with the risks presented above, in case of severe earthquake, over the normative cover conditions), and the sectional effort in the most requested element of the structure must not exceed the sectional capable effort corresponding to the element (Stratan, 2014). In these conditions, seismic design becomes a current design for structures located in non-seismic areas. Thus, the norms for seismic design (ex.:P100-1, 2013, SR EN 1998-1, 2004) are used only to evaluate horizontal loads, and SLU checks are made according to the general rules for calculating structural systems (ex.:SR EN 1992-1, 2004) (Stratan, 2014).

5. Conclusions

1º Ductility control is the key to successful design of structures, they may be gifted with sufficient capacity to dissipate seismic energy so that they can survive a severe seismic action. Thus, one of the most important performance objectives can be ensured: ”the safety of life” for people who live or work in edifices.

2º Design with this concept (ductile) implies:

a) favorable seismic response for structures, with incursions in the postelastic domain of behavior;

b) control of lateral displacements (deformations) with lateral stiffness limitations (implications);

c) the use of performing principles, methods and computing programs accessible and easy to utilize in actual technological development conditions.

3º (NZS 1900, 1965), (MOW, 1970) and (NZS 4203, 1976) were the first norms in New Zealand who specified the most important innovative elements in deflection control and the need to develop the ductile design concept.

4º Confined concrete (with impeded transverse deformation) has significant deformation capacity. The ductility of the reinforced concrete composite increases in the same moment with the development of the concrete class and the yielding limit of the reinforcements.

5º For reinforced concrete frame structure, beams are fundamental sources of seismic energy dissipation through limited deformations in critical areas.

6º The engineering practice shows in most cases appearance of plastic deformations in the columns from the upper levels of the structure, case who favoring the production of the floor mechanism.

7º Non-ductile constructions risk a fragile degradation of structural strength and rigidity, accompanied by collapse in case of exceeding the elastic limit of the structural elements solicited for a severe seismic action.

Regularity in plan and elevation of structural systems and the utilization of non-linear calculation are just a few incipient measures that theoretically can provide the required ductility for a structure by inducing a sufficient deformation capacity in postelastic domain. Thus, there is a necessity to study in this direction because exist inconsistencies between experimental studies/ real practice and the theory written in the specialized books/norms (ex.: the appearance of the plastic hinges in the columns for important horizontal actions etc.).

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DUCTILITATEA – CONCEPT DE ÎMBUNĂTĂȚIRE A RĂSPUNSULUI SEISMIC PENTRU SISTEMELE STRUCTURALE TIP CADRU DE BETON ARMAT

(Rezumat)

Structurile tip cadru de beton armat sunt utilizate cu încredere de specialişti în zonele seismice, fiind disponibilă o metodologie teoretică de proiectare a acestora şi de evaluare a răspunsului seismic. Însă în urma ultimelor acțiuni seismice severe care au avut loc pe glob (Japonia, 2011, Magnitude = 9,1, 20896 morți), se poate observa că răspunsul seismic al structurilor tip cadru de beton armat nu coincide cu mecanismele de disipare a energie considerate teoretic. Conceptul de proiectare ductil al acestor sisteme structurale merită o deosebită atenție şi o îmbunătățire eficientă. Astfel, este necesară o cunoaştere cât mai bună a fenomenelor ce se produc în structurile tip cadru de beton armat disipative, cu detaliile corespunzătoare cu privire la ductilitatea de material, secțiune, element şi structură, cât şi a modului de răspuns sesmic al sistemelor structurale proiectate neductil.