ON THE NONLINEAR BEHAVIOUR OF REINFORCED CONCRETE FRAME ELEMENTS UNDER SEISMIC EXCITATIONS

BETONARME ÇERÇEVELERİN SİSMİK ZORLANMALAR ALTINDA LİNEER OLMAYAN DAVRANIŞLARI HAKKINDA

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ABSTRACT

Peculiarities in the behaviour of reinforced concrete elements during strong earthquakes are discussed. An analysis of the parameters influencing this behaviour is made and recommendations on the effective seismic safety of reinforced concrete structures are given. To avoid an eventual collapse of constructions, which are not capable of ductile behaviour, they must be designed for a much higher seismic load, and this is not grounded from an economical point of view.

INTRODUCTION

The major part of the structures which have been designed and constructed in the seismic regions of many countries are reinforced concrete structures. It is a fact that besides the significant improvement in the technology of construction, as well as in the design process, severe damages and even failure of some reinforced concrete structures have occurred due to strong earthquakes. It is a question what is the reason for such behaviour of these structures. In addition to a number of factors influencing the behaviour of structures under seismic effect (soil conditions, structural concept, quality of the built-in material etc.) the ability of the structures to be in inelastic

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range, i.e. to sustain nonlinear deformations is of vital importance. It is well known that the value of the seismic forces in a structure with theoretical elastic behaviour during an earthquake is several times larger (4 to 6 times) than the value of the seismic forces obtained applying the Codes [1]. Thus structures survive the earthquake suffering nonlinear deformations, as a result of the ability of structural members to behave beyond the yield level, dissipating the seismic energy without heavier damages to the structural system.

Such behaviour forces civil engineers constructing reinforced concrete structures to control structural members in a way that they are capable to work and to suffer deformations exceeding the ultimate stress state, which, again, makes design engineers apply nonlinear calculation by dynamic analysis of the structural model.

This is possible when the following effects do not appear in the members of the reinforced concrete structures: local crushing of concrete, local buckling of reinforcement, cracks due to shear forces, loosing the bond capability and inadequate anchorage of reinforcement. Decrease in strength capacity, and especially stiffness deterioration, is also very important.

BASIC PRINCIPLES

The relationship "moment - curvature" for every reinforced concrete section in case of such nonlinear investigation must be derived. This relationship is characterized by some basic points, corresponding to the different levels of the stress-strain behaviour of steel and for their determination is used the methodology, similar to [2,3] with the following assumptions:

1. The plane sections remain plane from the beginning to collapse - Bernoulli hypothesis is accepted.

2. Idealized stress-strain curve for concrete, due to Hognested shown in Fig.1

$$\sigma_c = R_c \left[2\varepsilon_c / \varepsilon_o - \varepsilon_c^2 / \varepsilon_o\right] \qquad (1)$$

where R_c is the maximum compressive stress reached in the concrete; $\varepsilon_o = 0.002$ - the strain at the maximum compressive stress; $\varepsilon_c^u = 0.004$ -ultimate strain of the compressive concrete (assumed value of ε_c^u is relative since additional ductility due to rectangular hoops.

3. In this investigation the stress - strain curve for steel is assumed to be identical in the compressive and tensile range as a bilinear curve (Fig.2), where ε_s^y, R_s^y are strain and stress at the yielding of steel, ε_s^u and R_s^u are the ultimate strain and stress of steel.

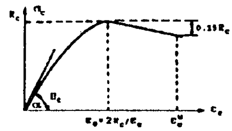

Figure 1

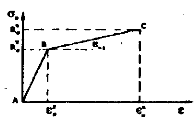

Figure 2

In this approach the reinforced concrete section with given geometrical characteristics (b and h), percentage of tension and compression steel (μ and μ') is divided into a number of horizontal elements, each having the width of the section at that level (Fig.3). There are n elements numbered from the top, each has depth h/n, where h is the overall depth of the section. The top and bottom steel reside in elements na'/h and na/h respectively. If the strain in the top fiber is ε_c and the neutral axis depth is αh_o the average strain in element i is

$$\varepsilon_i = \varepsilon_c\{n(\alpha h_o/h) - i + 0.5\}/\{n(\alpha h_o/h)\} \qquad (2)$$

The stress in the concrete and the steel in each element is found from the assumed stress-strain curves and is taken as the stress corresponding to the average strain in the element. From the stresses and the areas of the concrete and steel in each element the forces on the section may be determined.

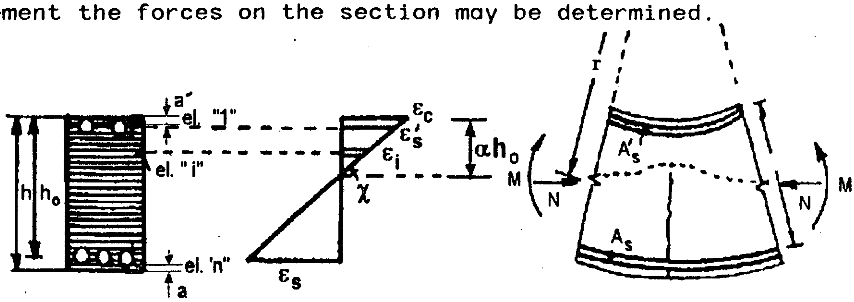

Figure 3.

An iterative technique is used to calculate points on the moment - curvature curves. The strain ε_c in the top concrete fiber is adjusted by a fixed amount. For each value of ε_c the neutral axis depth αh_o is estimated, and stresses in the elements are computed for this strain profile. The forces acting on the elements are computed for this strain profile. The forces acting on the elements are then calculated and the equilibrium of the forces is checked by using the requirement that

$$D_a + D_c - Z_a - N = 0, \qquad (3)$$

in which D_a, D_c and Z_a are the compressive and tensile forces acting on the elements, respectively, and N is the compressive load acting on the section. If the equilibrium (3) is not satisfied, the estimated neutral axis position is incorrect and must be adjusted until equilibrium of forces is achieved. Having obtained equilibrium, the bending moment M and curvature χ are calculated for the particular value of ε_c and N

$$\chi = 1/r = \varepsilon_c/\alpha h_o \qquad (4)$$

The ductility of a member sections is usually expressed as the ratio of the curvature at the ultimate deformation of the concrete to the curvature at the first yielding of steel.

$$D = \chi_u/\chi_y \qquad (5)$$

In this paper are considered the basic cases of the bahaviour of reinforced concrete elements, in addition a number of factor influencing their ductilities consequently at the members with flexure and at the eccentrically loaded elements. At the members with flexure in depending of section's characteristics can be occurs yielding of the tension steel at the relative strain of compression steel $\varepsilon_{s1} < \varepsilon_s^y$ (i.e. the compression steel remains in the elastic range) or yielding of the compression steel at the relative strain in the tension steel $\varepsilon_s < \varepsilon_s^y$.

The possible type of flexural failure (tension,compression and balanced) are shown in Fig.4.

If the steel content of the section is small, the steel reaches the yield strength R_s^y and ultimate strain ε_s^u, before the concrete reaches its maximum capacity and the strain in the extreme compression fiber of the concrete is $\varepsilon_c < \varepsilon_c^u$. In this case the tension failure occurs.

A "balanced failure" occurs when the tension steel just reaches the yield strength and the extreme fiber concrete compressive strain reaches the ultimate strain at the same time.

It should be noted that a balanced failure of each reinforced concrete section is associated with the corresponding percentage at the "balanced failure" -μ_b.

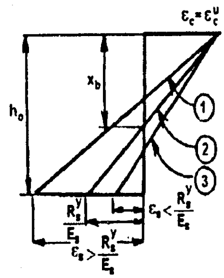

Figure 4

1- Tension failure $\sigma_s = R_s^y$; $\mu < \mu_b$;
2- Balanced failure μ_b;
3- Compression failure $\sigma_s < R_s^y$; $\mu > \mu_b$

A compression failure occurs in the following cases:

1. The failure of compressive concrete with the strain in the tension steel $\varepsilon_s^u > \varepsilon_s > \varepsilon_s^y$ and the strain in the compression steel $\varepsilon_{s1} \leq \varepsilon_s^y$.

2. The failure of compressive concrete with the relative strain in the tension steel $\varepsilon_s < \varepsilon_s^y$ and the relative strain in the compression steel $\varepsilon_{s1} \geq \varepsilon_s^y$.

3. The failure of compressive concrete with the strain in tension steel $\varepsilon_s^u > \varepsilon_s > \varepsilon_s^y$ and the strain in the compression steel $\varepsilon_{s1} > \varepsilon_s^y$.

At the eccentrically loaded elements with uniaxial bending, as well as the tension failure or the compression failure can be occurred depending on whether the tension steel reaches the yield strength.

The possible cases of yielding are as at the elements with flexure, in addition with one other case- the compression steel is yielding at a considerable value of the axial compression forces when all the section is compressed.

The type of failure, shown in Fig.5, are five. Four of them are as at the elements with flexure and the fifth type is characterized by high level of axial forces. This is associated with the distribution only of compression stresses in the hole section. The compression failure occurs because of the crushing of more compressive extreme concrete fibers.

At these elements there are also the "balanced failure" with the corresponding percentage μ_b.

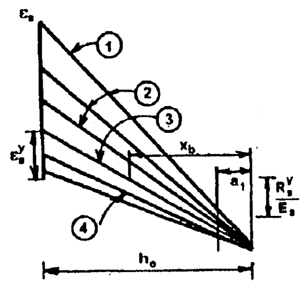

Figure 5

1- Compression steel does not yield
$\varepsilon_{s1} < R_s^y/E_s$
2- Typical tension failure $\sigma_s \geq R_s^y$; $x < x_b$
3- Balanced failure $\sigma_s = R_s^y$; $x = x_b$

The requirement for ductile behaviour means the presence of percentage of tension steel smaller than this at the "balanced failure" μ_b.

The axial load influences the curvature.For section, shown in Fig.6 with the iterative technique is plotted the interaction diagram "**M** - **N**". Curve 1 of this diagram indicates the combinations of **N** and **M** that cause the column section to reach the useful limit of strain ($\varepsilon_c^u = 0.004$ for the concrete). Curve 1 in the **N** - χ diagram (FIg.6b) shows the curvature of the section corresponding to the combinations of **N** and **M** when this ultimate condition is reached. Curves 2 give the combinations of **N**, **M** and χ corresponding to the points at which the tension steel first reaches the yield strength. Curve 2 do not appear above the balance point because the tension steel does not reach the yield strength above that point.

It is evident from Fig.6b, Fig.6c that the ductility of the section is significantly reduced by the presence of axial load. At levels of load less than the balanced load (point **B**) the ductility increases as the load level is reduced. The failure and damages are determined by the plastification of reinforcement. This is the "ductile behaviour".

At axial load levels greater than the balanced failure load, the ductility is negligible, being due only to the inelastic deformation of concrete and the failure of the section is brittle, the its ductility is minimum. The point of balanced failure **B** in the same figure illustrates a case when both materials, reinforcement and compressed concrete reach the characteristic ultimate point "ε_c^u" and "ε_s^y" this is the ultimate point of ductile behaviour, where the ductility factor is equal 1.

Parameters influencing strength capacity, deformability and ductility are topics of permanent investigation. During the past years, many investigations of the individual parameters influencing the behaviour of reinforced concrete elements were performed. A large number of experiments have been also performed at the Illinois University under the guidance of Prof. Newmark [2].

In this paper are presented some results, obtained on the basis of numerical investigations using the methodology, above mentioned and elaborated program for PC/AT 286 computer.

The cross section presented in Fig.7 is investigated at the different longitudinal reinforcement (μ and μ'), different strength of concrete and steel. The graphs demonstrate the influence of the longitudinal reinforcement upon the strength capacity, plastic deformability and the value of the ductility factor **D**. Increase in tension reinforcement results into increase in strength capacity, but gradual decrease in deformability. Application of compressed reinforcement increases significantly deformability (Fig.8). The corresponding ductility factors are shown in the same figure.

Brittle failure occurs in the case of very high percentage of tension reinforcement $\mu\%$ but without compressed reinforcement. It means that strength capacity is proportional to the tension reinforcement. In the case of elements (beams) without compression steel the ductility factor **D** is inversely proportional the percentage of the tension reinforcement.

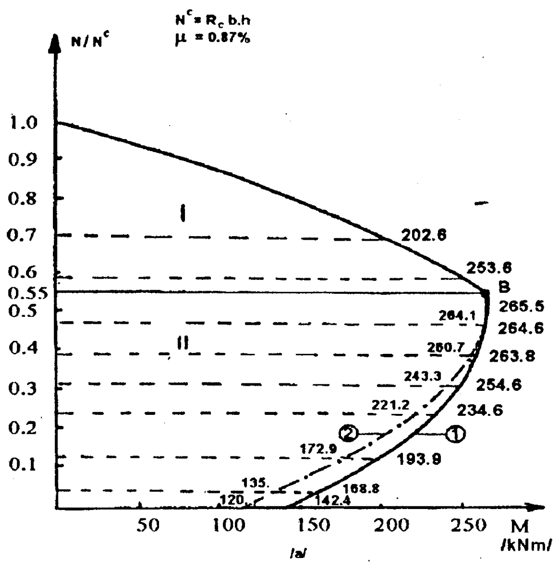

Figure 6a

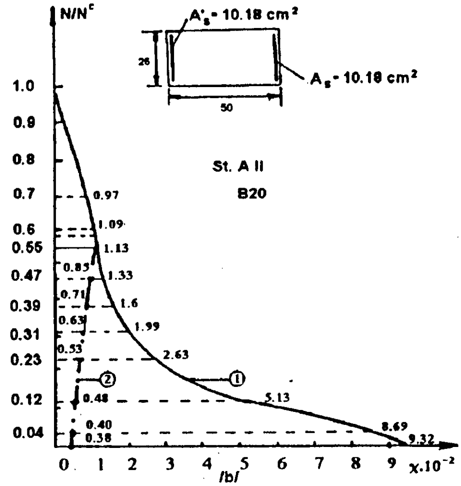

Figure 6b

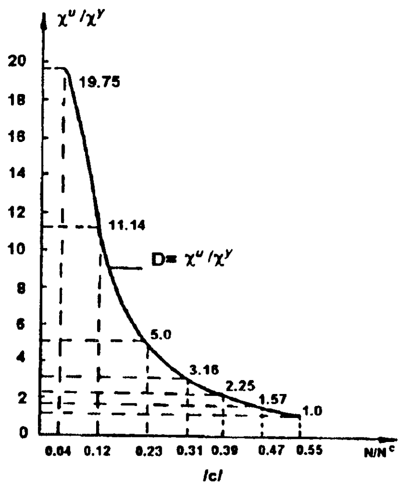

Figure 6c

Strength and ductility of column section

a) Interaction diagrams

c) Curvature ductility

1. - nonductile section; 2. - ductile section;

B point of "balanced failure"

At the presence of compressed reinforcement the ductility factor is inversely proportional to the subtraction of percentages of reinforcement μ-μ', as shown in Fig.8 or ratio μ/μ' as shown in Fig.10.

From these figures it is evident also that:

1. An increase in the concrete strength increases the ductility of the section.

2. An increase in the extreme fiber concrete strain at ultimate increases the ductility of the section.

3. An increase in the steel yield strength decreases the ductility.

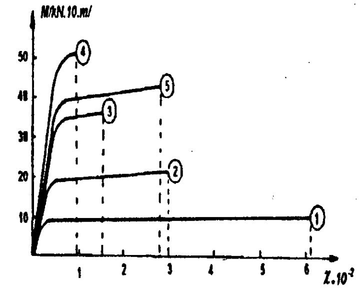

Figure 7.

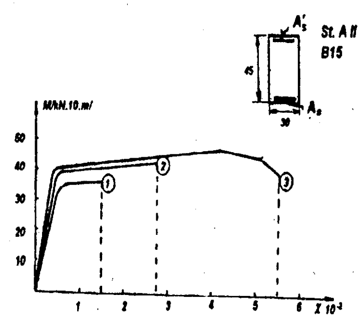

Figure 8.

The value of the axial loading acting in the column sections have significant influence upon theirs deformability. Increase in axial forces results into increase in strength, but considerable decrease in deformability also, as evident from Fig.9.

It should be mentioned that for the above elements at the high compression forces the effect of the transverse reinforcement is very important to avoid the brittle failure and to increase its ductility. The stress - strain relationships for concrete confined by transverse reinforcement which are obtained experimentally [2] can be used in nonlinear seismic investigations. Such experimental and numerical investigations must continue in our country also.

CONCLUSIONS

Determination of relationship "moment - curvature" is one first step in the complete process on whole nonlinear aseismic investigation. This relationship represents back curve of the hysteresis models, which are used for determination of the element's stiffness at such investigation.

In this first step considering the characteristics of sections influencing on the ductility of reinforced concrete elements as and on the whole structure is of great importance for its aseismic safety. This is because, as it was already mentioned in the beginning, the present seismic design philosophy relies on energy absorption and dissipation by postelastic deformation for survival in major earthquakes.

These structure, which are not capable to have ductile behaviour must be designed for much higher design seismic loading, if its eventual damage during strong earthquakes should be avoided, which is not grounded from an economical point of view.

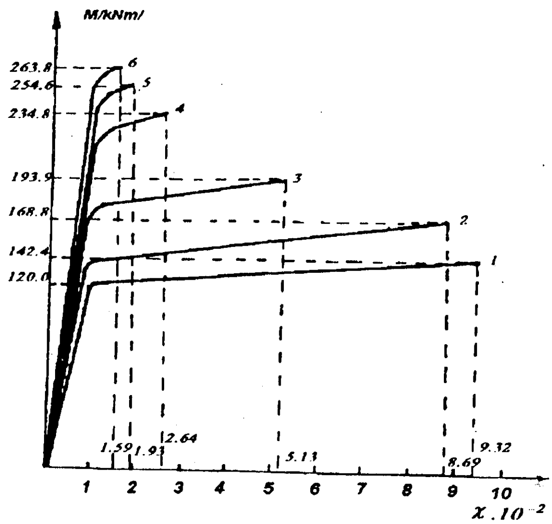

Figure 9.

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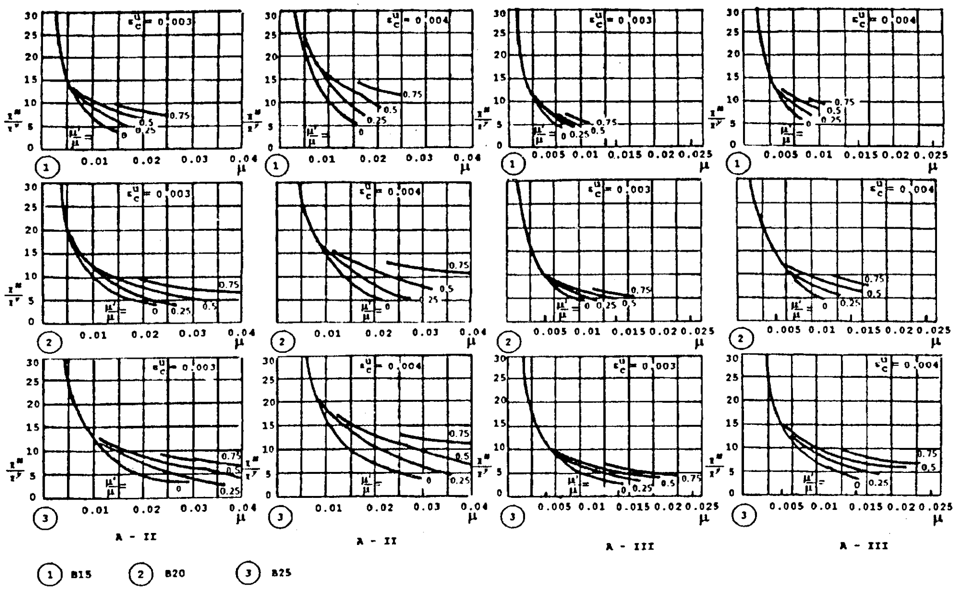

Figure 10.

BETONARME ÇERÇEVELERİN SİSMİK ZORLANMALAR ALTINDA LİNEER OLMAYAN DAVRANIŞLARI HAKKINDA

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Bu çalışmada, betonarme yapı elemanlarının kuvvetli deprem sırasındaki davranışlarının önemli özellikleri tartışılmaktadır. Bu davranışı etkileyen nedenlerin bir incelemesi yapılmakta ve betonarme yapıların etkin deprem güvenliği üzerine öneriler getirilmektedir. Göçme ile karşılaşmamak için, ekonomik açıdan güç de olsa, süneklik kazandırılamayan yapıların daha büyük deprem yüklerine göre boyutlandırılmaları zorunludur.