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STRUCTURAL RESPONSE COMPARISON IN THE CASE OF REINFORCED CONCRETE FRAMES

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Abstract. Ductile structural elements present an important plastic deformation capacity, unlike brittle elements that should be avoided at all times. The plastic deformations occur usually in certain limited areas in a structure, known as *plastic joints.*

The nonlinear modelling of the composite material know as steel reinforced concrete represents one of this study`s main objectives. Nonlinear modelling solutions for steel as well as for concrete are presented in the second section.

The implied comparisons in the first stage of the study suggest the effect of introducing the steel reinforcements in the analysis (Fig. 1 *b*).

In the second case study an advanced analysis (time integration) is used, from which one can follow the loads in the concrete as well as in the steel for an accelerogram scaled for the Constanţa area.

In the third case study the degradation of the concrete and flowing of the steel reinforcements, if that is the case, by reaching the maximum load bearing is underlined.

Evidently, the subjects of the case study are the plastic areas at the end of the beams and bottom of the pillars marked in Fig. 1 *a*.

Besides the comparative analysis through the diversity of the accepted calculation methods, this paper proposes to determine the state of strain in the

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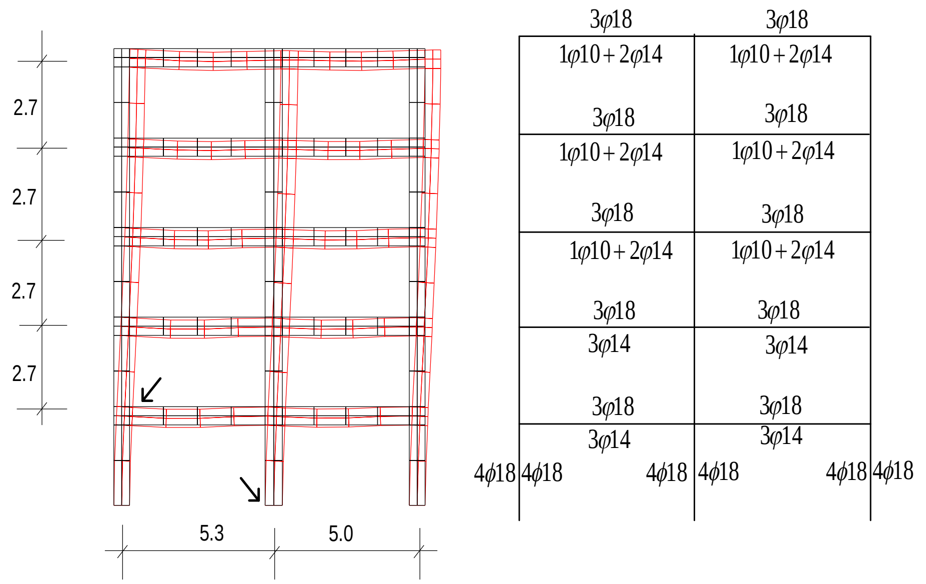
concrete as well as in the steel reinforcements, analysing the co-operation between these two materials. In this respect, different finite elements shall be used for the concrete, compressed steel, and stretched steel, considering the co-operation between them.

Key words: concrete; reinforcement; plastic zones.

1. Introduction

In order to achieve a satisfactory behaviour of a reinforced concrete structure in the present, as a method of conceiving, sizing and constructive composition of structures, the resistance capacity design method is used.

In a structural system the potential plastic areas are chosen from the beginning, they are sized and composed in a way that they are sufficiently ductile. The rest of the structure is dimensioned using a higher bearing capacity than the potential plastic areas in order to remain at all times in the elastic behaviour area. In this way it is ensured that the dissipation mechanisms remain unaltered even if the structure sustains large deformations.



a *b*

Fig. 1 – *a* – Digitization; *b* – the reinforcement of a reinforced concrete frame (5 levels).

By applying the static regression method in the application of the static equivalent seismic forces, the post-elastic behaviour of the reinforced concrete elements can be followed.

This study aims to evaluate the structural response comparatively in a numerical analysis, using static equivalent seismic forces, and in a different numerical analysis, using time integration on a reinforced concrete element, in a first stage.

The second stage of the paper consists in determining the structural response, considering that the concrete has reached its compression bearing capacity.

Regarding the reinforced concrete behaviour, micro-elements and macro-elements are used in the modelling.

The macro-elements are finite elements which globally reproduce the behaviour of different structural components in the reinforced concrete under different types of loads.

The micro-elements also represent finite elements which precisely model the behaviour of concrete and steel, as well as the phenomenon known as adherence.

On the subject of modelling the beams which present punctual plastic joints, two types of models can be distinguished, the model with parallel elements and the model with the elements arranged in series.

The model with the parallel elements is the simpler of the two. This model is composed of two beam type elements mounted in parallel. One of these behaves perfectly elastic, with a loading slope equal to the loading slope chosen for the post-elastic domain, while the other one is described by an ideal elastic-plastic behaviour.

The second model is comprized of a beam which behaves ideally elastic and two inelastic springs at each end.

At the extremity of the beam, the load in the elastic elements is equal to the one in the springs. The springs activate only when the flowing momentum is reached.

The model with concentrated plastic hinges is the most effective and simple model in the case of beams, because it provides direct plastic rotations.

Even if this subject has been long debated, one can say that by using advanced calculation methods and pertinent logistic artifices, certain results can be achieved that would lead to a more efficient design method for reinforced concrete frame structures.

2. Modelling. Idealizations of the Materials Behaviour

The calculations made on the frames must ensure that like in the case of high seismic actions, the plastic joints will form only in the specially selected areas, which are the beams, and that these plastic joints are composed in a way that the plastic torsion (rotation) can occur freely, without substantially reducing the load bearing capacity of the structure.

The calculations of the strains in the cross-sections (profiles) of the idealized structure must correspond to the actual behaviour of the real structure as much as possible, especially regarding the load bearing and deformations.

The realistic evaluation as well as the determination of the capable strains of the reinforced concrete cross-sections can be achieved by using different non-linear calculation methods.

In order to determine the state of tension in the structure, an 8 node quadrangle digitization with squared interpolation functions was used. This type of element was used to model the concrete.

In order to achieve a more accurate model of the structure, the steel reinforcements must be taken into consideration.

The solution adopted was that of introducing “Truss” elements tied to the nodes of the 8 node quadrangle elements, at the extremity of the cross-section.

On one hand, in this particular way, the effect of the steel reinforcements on the response of the structure is modelled, and on the other hand we could ascertain the state of tension in the steel. If we were to have used “Beam” type elements, this would have been impossible to achieve. In order to model the nonlinear behaviour of the steel reinforcements, a model described by the strain–deformation curve was used (Fig. 2).

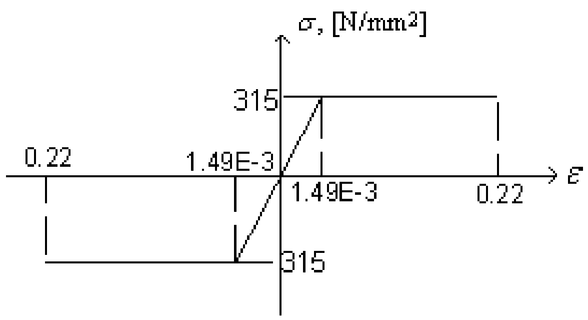


Fig. 2 – Strain–deformation relation.

The concrete is idealized as an isotropic, homogeneous, linearly elastic-plastic, capable of (auto) consolidation and cold-hardening deformations.

The proposed loading function must take into consideration, the weak tensile resistance of concrete, reaching the resistance limit due to excessive cracking, as well as the good behavior of concrete regarding compression, until it finally gets crushed. As a result the post-elastic static analysis is carried out considering that the load function covers a range between the limit of the initial discontinuity tension area up till the limit of the area where concrete fails.

We start from the response of the concrete which has undergone a uni-axial loading.

This behavior is portrayed through a typical “tension–deformation” diagram, which is shown in Fig. 3.

The characteristical diagram is generally nonlinear, but non the less for tensions which are smaller by 30%...60% than value of the tension which causes the concrete to fail, the diagram is approximately linear on the OA segment.

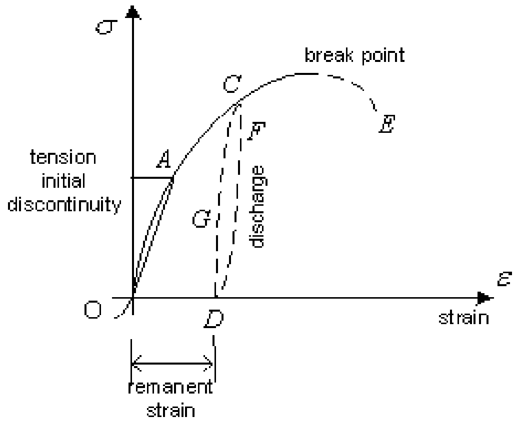


Fig. 3 – Typical “tension–deformation” curve for concrete.

As the loads applied start to rise , certain discontinuities start to develope in the structure of the material and as a result, *A* is considered to be the elastic limit or the point of initial discontinuity (equivalent with the initial point of flowing in the plasticity theory).

At approximately 75%...90% of the full load, small cracks or even larger ones on the surface of the concrete start to appear, eventually these cracks get bigger and bigger until the concrete is dislodged into different smaller pieces.

In order to corectly model the nonlinear behavior of the concrete, the solution of degrading the elasticity module for the areas where the concrete has reached it`s bearing compression capacity was chosen.

For the concrete class C20/25, once it reached the strength value that was used in the calculations, R_c = 15 N/mm^2, the elasticity module was degraded by 50%. For this material, the characteristic strength is R_{ck} = 20.5 N/mm^2 and it`s elasticity module is E_b = 30,000 N/mm^2.

3. Case Studies

The first case study consist in applying static equivalent seismic forces. The analysis was conduced using two hypotheses. In the first one, only the concrete was digitized, and in the second the steel reinforcements where introduced in the digitization (Table 1).

By analysing the results we can observe that the response difference between the two hypotheses, regarding the movement of the structure, is 8.3%.

In both of the case studies, reduced values of the strain in the concrete as well as in the steel were found, maximum 50…60 daN/cm^2 for the concrete and 300 daN/cm^2 for the steel reinforcements.

From these numerical studies we can observe that the concrete does not reach it`s load baring capacity and the steel does not begin to flow.

Table 1

Response Comparison between the Concrete Digitization and the Concrete + Reinforced Steel Digitization in the Case of Static Equivalent Seismic Forces

Analysis type	$D_{y\,\max}$, [m]	σ_x, [daN/cm^2], at the extremities of the beams.	σ_y, [daN/cm^2], at the base of the pillars	Stress in the reinforcements, daN/cm^2
a) Concrete digitization	0.458E–2	–60.23 60.82	–53.68 17.37	
b) Concrete+steel digitization	0.408E–2	–55.05 54.6	–49.37 15.38	–300.0 267.3

The second numerical study introduces the effect of the seismic action through an accelerogram. For the integration in time we need an accelerogram recorded in the area and scaled to an appropriate value from P100 (Table 2).

The duration of the calculation step is chosen small enough (0.02 s) in order to obtain an appropriate numerical accuracy and to avoid numerical instability.

Thus, for the Constanța area we used an accelerogram scaled to 0.16 g having a duration of 16 s, 800 time steps with. It was used the elasticity module appropriate for the C20/25 concrete.

Table 2

Structure Response for Time Integration with Rayleigh Amortization 5%

Analysis type	$D_{y\,\max}$, [m]	σ_x, [daN/cm^2], at the extremities of the beams	σ_y, [daN/cm^2], at the base of the pillars	Stress in the steel reinforcements, daN/cm^2
Time integration; concrete and steel reinforcements model; with Rayleigh damping 5%	0.23E-1	–166.69 157.25	–109.49 141.54	–1,126.1 1,113.6

If we are to compare the results from Table 1 and Table 2, an increase in the value of the strains at the end of the beams from 60.23 daN/cm^2 up to 166.69 daN/cm^2 can be observed. A first conclusion can be drawn, the undervaluation of strains in the case of the static equivalent seismic forces analysis.

The time integration analysis can be considered as a baseline study since it is the mathematical formulation closest to reality.

The viscous damping was taken into consideration using the Rayleigh model.

The damping used was realized using both the mass matrix in nodes and the consistent damping. Regardless of the type of damping used, the results are identical.

The values obtained indicate an exceed of the maximum compression at the end of the beams and thus at the capable exceeding of sectional rotation.

Even if the formation of plastic hinges is associated with the flowing (creeping) of the reinforcements, the degradation of the concrete due to compressive forces before the flowing of the steel reinforcements implies a cross-section rotation above the capable value.

The tensile strains are irrelevant because in the stretched area the concrete works with cracks and the tensile strains are absorbed by the steel reinforcements.

Table 3

Comparison of Response Degradation Extremities of Beams and Base of Pillars

Analysis type	$D_{y\,\max}$, [m]	σ_x, [daN/cm^2], at the extremities of the beams	σ_y, [daN/cm^2], at the base of the pillars	Stress in the steel reinforcements daN/cm^2
a)Time integration Conrete + Steel digitization –degradation at the extremities of the beams	0.39E–1	–121.8 122.9	–172.0 137.0	–2,398.4 2,266.6
b) Time integration Concrete + Steel digitization –degradation at the extremities of the beams and at the base of the pillars	0.437E–1	–129.01 129.9	–112.0 86.0	–2,546.9 2,408.85

In Table 3 the results of the time integration analysis on the structure, taking into consideration the appearance of plastic joints, are presented.

If firstly the results in movement and strains with and without the degradation of the concrete due to plastic joints are compared, an increase of the maximum displacement from 0.023 m to 0.039 m is observed. The values of the maximum compression strains at the end of the beams decrease from –166.69 daN/cm^2 to –121.8 daN/cm^2.

Thus, these values are close to the resistance of calculation, 150 daN/cm^2.

From the comparison of the two Tables 2 and 3, we also observe that the reduction of the concrete strains due to the appearance of plastic joints, greatly increases the maximum values of the strains in the steel reinforcements from 1,113.6 daN/cm^2 to 2,408.8 daN/cm^2.

From the analysis of Table 3, points *a* and *b*, it is possible to observe that modelling the formation of plastic hinges at the base of the pillars, which is inevitable, reduces the maximum compression strains from –172 daN/cm^2 to –112 daN/cm^2.

4. Conclusions

The first numerical analysis confirms the results from previous tests that exist in professional literature, meaning that by adding reinforcements the maximum value of the associated movements does not change in a major way, and in terms of the state of tension, the differences are significantly reduced.

The purpose of the case study is to ascertain tensile stress in the reinforcements and concrete; from this point of view it can be observed that for low tensile stress in the concrete, high tensile stress is obtained in the reinforcements.

In the second case, when the type of analysis is changed (time integration with 5% amortization), in the areas in which plastic joints appear, meaning at the extremities of the beams, the tensile stress exceeds the strength of the material used in the calculations.

In the third case study a different analysis was proposed, one in which the elasticity module was degraded in the areas of the plastic joints, thus obtaining a decrease of the tensile stress in the concrete and an increase in the steel reinforcements. These become more balanced, knowing the fact that the flowing of the steel reinforcements must occur at the same time as the strength of the concrete used in the calculations is reached.

Present studies provide an alternative for the plastic joint solution, they are automatically introduced by the calculation program once the capable momentum is reached in the cross-sections at the extremities of the beams (both parallel and in series models).

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COMPARAȚII ALE RĂSPUNSULUI STRUCTURAL ÎN CAZUL STRUCURILOR ÎN CADRE DE BETON ARMAT

(Rezumat)

Pe lîngă analiza comparativă prin diversele metode de calcul acceptate, această lucrare îşi propune să determine starea de tensiune atît din beton cît şi din armătură, analizînd conlucrarea dintre cele doua materiale. În acest sens se folosesc elemente finite separat pentru beton si separat pentru armătura întinsă şi cea comprimată, considerând conlucrarea dintre ele.

Aplicînd metoda regresiei statistice în aplicarea forțelor seismice static echivalente , se poate urmări comportarea postelastică a elementelor de beton armat.

Lucrarea de față îşi propune, într-o primă fază, să evalueze comparativ răspunsul structural între o analiză numerică folosind forțele seismice static echivalente şi o analiză numerică folosind integrarea în timp, folosind un element compozit beton–armătură.

A doua etapă a lucrării constă în determinarea răspunsului structural ținînd cont de efectul degradării betonului datorită atingerii capacității portante la compresiune.