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NON-LINEAR ANALYSIS OF REINFORCED CONCRETE FRAMES WITH ATENA 3-D PROGRAM

BY

IOANA OLTEANU*, ANDREI ALISTAR and MIHAI BUDESCU

"Gheorghe Asachi" Technical University of Iaşi,
Faculty of Civil Engineering and Building Services

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Abstract. Structural vulnerability evaluation is essential for assessing building behaviour in case of an earthquake. Among the basic design concepts is weak beam-strong column, but nevertheless the earthquakes effects lead to different failure mechanisms. This paper investigates constructive solutions to direct plastic hinges from columns to beams, in order to prevent the collapse of the structure. The virtual–experimental simulation of reinforced concrete elements subjected to horizontal loads is discussed. In this study a 3-D ground floor frame structure was considered in order to determine: development of cracks in the concrete, the influence of stresses in the reinforcement, the plastic hinge formation. Different comparisons were performed, changing the geometry of the slab and the reinforcement. The analysis was conducted in ATENA, a computer program using stress analysis with finite elements.

Key words: pushover analysis; plastic hinge; numerical simulations.

1. Introduction

The non-linear analysis became of great importance in the seismic vulnerability assessment of buildings, when the performance based evaluation (PBE) started to became more popular (ATC-40 Report, 1996; BSSC – NEHRP

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* Corresponding author: *e-mail*: olteanuioa@yahoo.com

Recommended…, 2000). Usually, in most of the current seismic codes is desirable in case of an earthquake structures can dissipate seismic energy, with displacement control initially, without brittle failure of the elements. Frame structure are recommended in seismic areas because can dissipate energy through vibration in the elastic range. As the earthquake induces higher inertial forces on the structure, a ductile plastic mechanism is formed designed to avoid brittle collapse.

The comprehensive seismic vulnerability represents a complex concept, defining the overall predisposition or the physical, economical, political or social susceptibility of a community to be affected, or to suffer damage, when an earthquake occurs (IDEA-IDRM, 2003). There are several methods used for vulnerability assessment. Among the analytical methods, the non-linear analysis is emphasized, which is based on the relation between shear force and displacement. In this method the behavior of a multi degree of freedom (MDOF) system is evaluated by replacing it with an equivalent single degree of freedom (SDOF) system (Negulescu, 2010).

The non-linear analysis aims to obtain a global mechanism for the structure with a ductile behavior and the failure of elements is restricted, while the structure can redistribute efforts.

First non-linear methods were based on empirical relationships, statistical data, and experts' opinions and were introduced in the design codes in the 80s. Nowadays are frequently used the statically nonlinear methods, called also *pushover methods*, developed and enhanced since 1981.

The graphical representation of the relation between force and displacement is called *capacity curve*. This can be obtained through computer programs which have implemented the pushover analysis. The simplified hypotheses from strength of materials are used in order to simplify the computation algorithm. There are also programs using the analysis with finite elements on assembly, subassembly or even on structural elements, analysing the materials separately. Thus the results characterize directly the cracking state, the stresses and the deformations in the concrete and in the reinforcement.

In this paper, for the considered case studies, ATENA 3-D was used.

2. Non-linear Static Analysis

The non-linear analysis can be classified according to the definition of the seismic action or the considered structural model. From the non-linear methods, the dynamic analysis is considered to be the most accurate as representing the behavior of the structure in time. It is difficult to use this method, because the input data are very complex (sets of seismic recordings, damping coefficients, hysteretic models for the inelastic behavior, etc.) and the output are difficult to interpret and to use in the daily design process (stress and displacement variation in time, absorbed energy, etc.).

Among the non-linear procedures, the pushover analysis is widely used because combines the advantage of considering the failure mechanism and the inelastic deformations with simplified static loading pattern providing low uncertainties in the results and practical to use in care of structures with a complex geometry (Penelis &Kappos, 2001).

The non-linear static procedure or pushover analysis is defined in the Federal Emergency Management Agency document 273 (1997) as a non-linear static approximation of the response a structure will undergo when subjected to dynamic earthquake loading.

The pushover analysis is a static non-linear analysis, which presents, in a simplified way, the structure behavior subjected to different loads produced by the seismic action (Thomos & Trezos, 2006). Increasing the loads on the structure allows identifying the weak structural members and the failure mechanisms. Usually the loading pattern is an inverted triangle (Fig. 1, FEMA 440, 2005), but can also be an increasing load at the roof of the structure, which is the case considered for the studied models in this paper. The lateral loading is actually an equivalence of the relative accelerations associated with the first vibration mode.

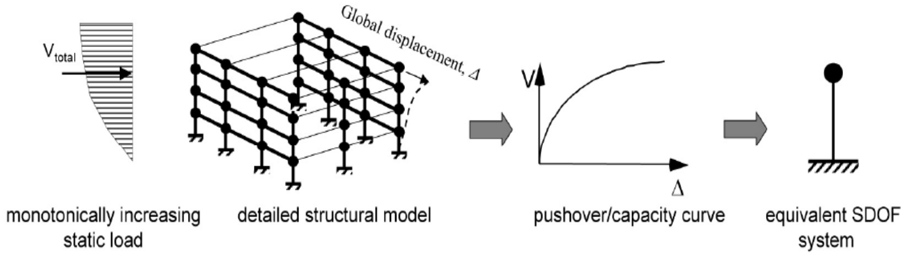


Fig.1 – Schematics depicting the development of an equivalent SDOF system for a pushover curve (FEMA 440, 2005).

The pushover analysis estimates the real strength of the structure.

In 1981, Saidii and Sozen proposed the use of non-linear dynamic analysis for an equivalent SDOF system, which can be considered the base for the current method. Using the same idea, Fajfar and Fischinger (1981) developed the first version for the N2 method, in which N comes from the procedure type – non-linear and 2 comes from the used mathematical models – SDOF and MDOF (Fajfar, 2002). However, the engineering community did not paid special attention to the pushover method until the mid 90 s. Nowadays there are several methods that include the pushover analysis as: the capacity spectrum method, the coefficient method and the improved N2 method.

The structure behaviour is performed in the pushover analysis by graphical representation of the displacement variation at the top of the structure with the shear force at the base of the structure, also called the *pushover* or *capacity curve*.

The failure mechanism appears due to plastic hinge formation, which passes through different stages as the lateral load increases. In FEMA 356 (2000) three performance stages are defined: immediate occupancy (IO), life safety (LS) and collapse prevention (CP). These stages are graphically represented on the capacity curve from Fig. 2.

None of the existing computer programs can compute the plastic hinge characteristics. For this reason they are geometrically predefine in the initial stages of the modelling, in the regions where it is known that they could appear, for example in case of frame structures the beams and columns ends are considered plastic potential areas. The inelastic characteristics of the plastic hinges are also predefined so that after the calculus the obtained stresses are used to determine the stage of each plastic hinge. Considering their evolution conclusions are drawn on the deformed mechanism of the studied structure.

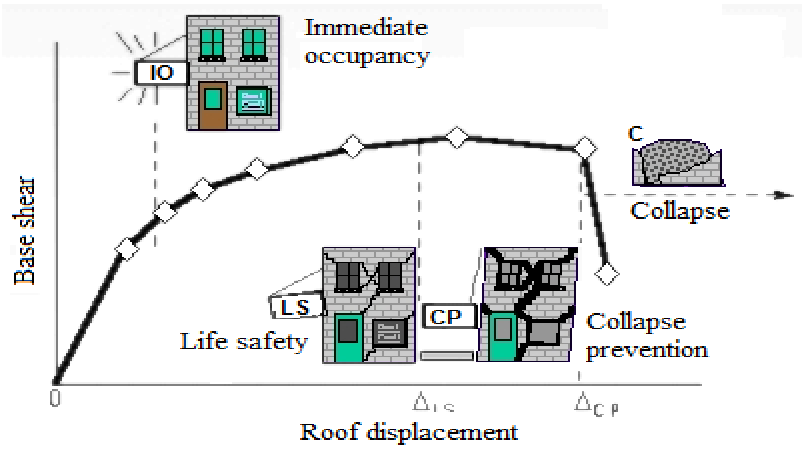


Fig. 2 – Pushover curve and structural performance stages.

The increased speed for processing, computing and system memory leads to the computer program development that have implemented spatial finite element analysis of “stress analysis” type, where the stresses and strains of the element are computed. Among them is ATENA 3-D.

The program has a computing platform based on the graphical user interface through which all the inputs are entered. The computation is based on Lagrange formula. The software is designed for spatial nonlinear analysis for solids, especially for reinforced concrete elements. The properties and the characteristics of the non-linear materials are complex and can be defined manually, but for the common ones, such as the standardized concrete and steel classes, the values are predefined in the program (Cervenka, 2002).

The program has three main functions:

a) *Pre-procession*, where are defined the geometry of the elements including the spatial position of the reinforcement by grid coordinates, the non-linear characteristics for the materials, load assignment, bounding conditions for the materials, finite element meshing and various parameters used for the analysis.

b) *Analyse*, where the results can be monitored and accessed in real time, as the analysis passes from one step to another.

c) *Post-procession*, provides access to the results expressed graphically and numerically.

Unlike other programs (SAP2000, ETABS, Robot, etc.), modelling in ATENA 3-D is more complex, but a complete virtual model can be achieved. The precise positioning of the reinforcement in the concrete gives an idea about the problems that may occur and permits to find solution in the design stage. The areas with a plastic potential are better described and analysed.

The spatial description of stresses and strains allows for an accurate analysis. Based on stresses and non-linear properties of the materials, results for each material can be obtained. Thus the stress evolution in the reinforcement can be monitored by lateral loading. The potential plastic areas can be characterized directly through stress development until it reaches the yield limit and through crack development for concrete.

In this paper the behaviour of a spatial ground floor frame, subjected to horizontal loading, is investigated, taking into account several means for directing the plastic hinges from the columns in the beams.

3. Case Study

A 3-D reinforced concrete frame structure was considered with openings of 6 m in both directions and level height of 3 m, as it is shown in Fig. 3. A supplementary steel plate was introduced at the roof of the structure to apply the horizontal loading.

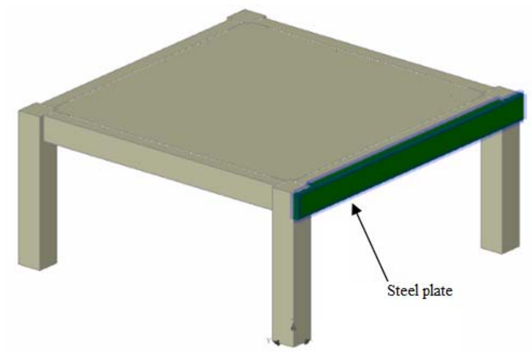
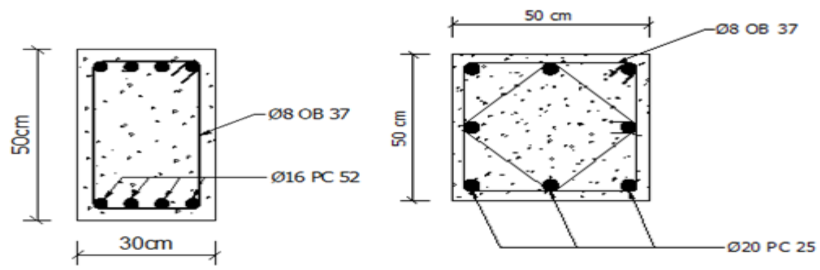


Fig. 3 – Reinforced concrete frame model.

Four different cases were analysed and compared: frame without slab, frame with slab of 15 cm thickness, frame with slab of 15 cm thickness with material replacement in the corners, frame with slab of 15 cm thickness with joints in the corners. The last two cases were introduced in order to find constructive measures to direct the plastic hinges from the columns to the beams. In all four cases the self-weight of the structure was considered and additional a uniformly distributed load of 2 kN/m^2 was applied on the slabs. For the frame without slab an additional load of 3.75 kN/m was applied on the beams, which represents the load transmitted from the slab.

The geometric characteristics for the cross-sections are represented in Fig. 4, and the material properties used in the analysis are resumed in Table 1. The slab reinforcement was considered of 8 mm diameters steel bars situated at a distance of 10 cm on both directions.



a *b*

Fig. 4 – Considered cross-sections: *a* – beams; *b* – columns.

Table 1
Materials Properties

Materials	*E* GPa	γ	ft MPa	fck MPa	fyk MPa
Concrete C20/25 –SR EN 2	30	0.2	1.5	20	–
Longitudinal reinforcement PC 52	20	0.3	–	–	355
Stirrups OB 37	20	0.3	–	–	235
Concrete C12/15 –SR EN 2	27	0.2	1.1	12	–

In order to describe the evolution of cracks in the concrete and of stresses in the reinforcement, the loading was applied in 50 steps, with a constant step size of 20 kN, in the end resulting a 1,000 kN load on the considered model.

After performing the analysis it was observed that the frame without slab is the most flexible one at a horizontal action of 1,000 kN and the plastic hinges in this model appear first in the beams, according to the design concept previously described. The influence of the slab is to stiffen the frame model and to direct that plastic hinge formation in the columns. The results are similar to those obtained in SAP2000, already published (Olteanu *et al.*, 2010).

Analysing the stress evolution in the slab reinforcement it is noticed that the reinforcement close to the beams works together with the one from the girder reinforcement (Fig. 5). This means that the beams and the slab behave as a single element, which explains the stiffness increase in the models with slabs.

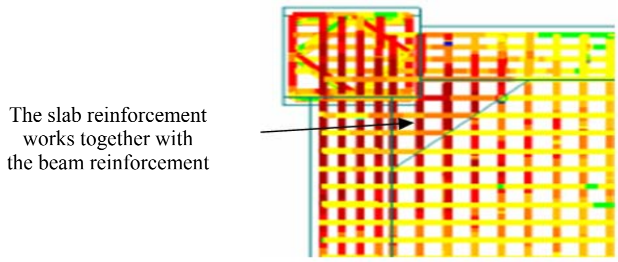
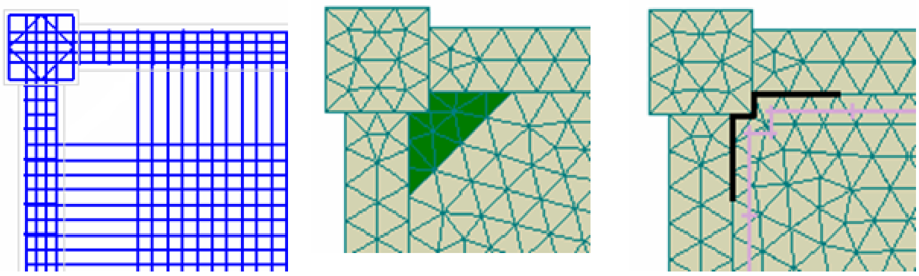


Fig. 5 – Slab influence on potential plastic areas from the beams.

Another constructive suggestion refers to replacing the material from the corners of the slab (Fig. 6 *b*). The study presents the case where the changed material has a triangular shape with sides of 50 cm in the corner of the slab, because from the performed study this is the most appropriate (square shape were also investigated). In the replaced area concrete class C12/15 was used.

The last constructive measures analysed is represented by a joint of 5 mm thick and a length of 50 cm in each of the corners of the slab, as it can be seen in Fig. 6 *c*.

All these propositions aim to break the load transmission from the slab to the beams in the potential plastic areas.



a *b* *c*

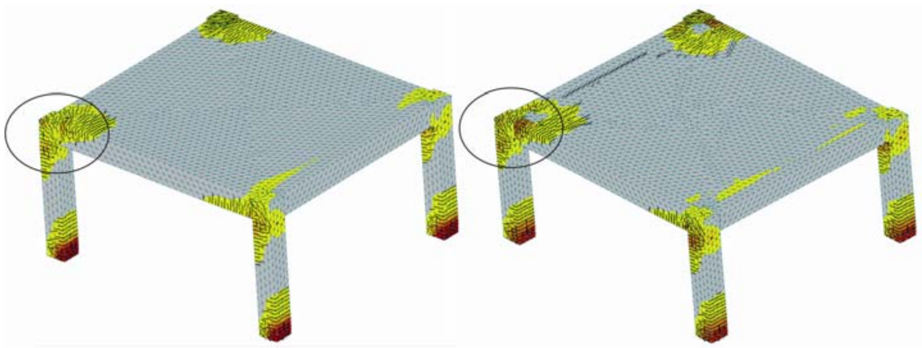
Fig. 6 – Studied cases: *a* – reduced reinforcement; *b* – material replacement in the corners; *c* – 5 mm joint in the corners.

A synthesis of the obtained results is presented in Table 2. The yield and ultimate displacement are given, as well as the yield force and the length of the plastic hinge in the beam. The yield and ultimate displacement represent important parameters in defining the behaviour of the structure in case of an earthquake. These parameters are used for the vulnerability and risk assessment of a model.

Table 2
Analysis Results

Analysed model	Type of slab reinforcement	Yield state		Ultimate state	Length for the plastic hinge in the beam, [mm]
		d_y, [mm]	f_y, [mm]	d_u, [mm]	
Frame without slab	–	6.45	400	169.26	1,200
Frame with slab	normal	6.05	560	39.75	600
Frame with slab with material replacement in corners	normal	5.36	520	42.10	625
	reduced	4.09	460	44.32	700
Frame with slab and 5 mm joint in corners	normal	4.12	480	45.02	700
	reduced	4.03	440	50.35	825

The following notations were made: *C*1 for the frame without slab, *C*2 for the frame with slab, *C*3 for the slab with material replacement in corners with normal reinforcement, *C*4 for the slab with material replacement in corners with reduced reinforcement, *C*5 for the slab and 5 mm joint in corners with normal reinforcement and *C*6 for the slab and 5 mm joint in corners with reduced reinforcement.



a *b*

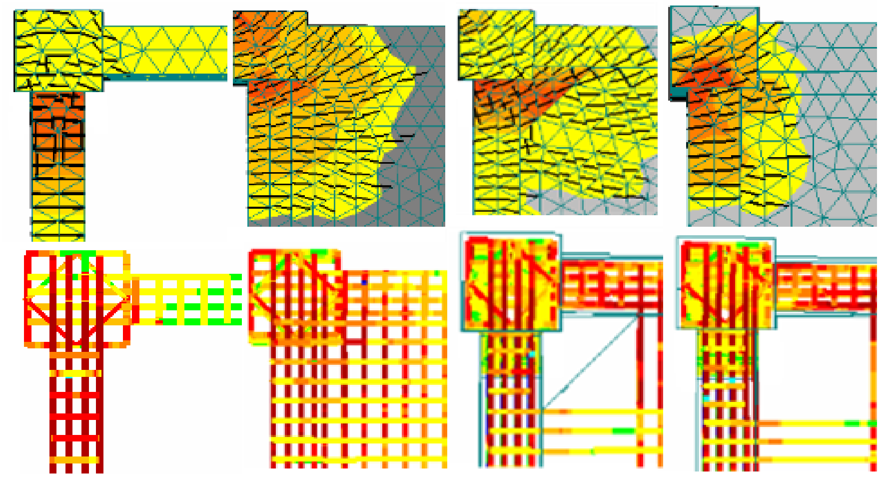
Fig. 7 – Plastic hinge development: *a* – model *C*2; *b* – model *C*6.

Considering Table 2 we can conclude that the maximum or ultimate displacement, d_u, is four times greater for model *C*1. The flexibility of this model is explained by the missing slab. For the same case, the yielding force, f_y, is smaller with 40%, while the yield displacement, d_y, is greater with 6% in comparison with *C*2. At the same time it is clear that the plastic hinge length is double for *C*1 in comparison with *C*2. Comparing *C*2 with *C*3 the differences are not significant, in contrast with case *C*4, when the plastic hinge length increases with approximately 12%. If model *C*5 is compared with *C*4 no significant differences are noticed, but for the *C*6 case the biggest increase in the plastic hinge length, approximately equal to 40%, is noticed.

Considering the two marked areas from Fig. 7 it can be concluded that the introduction of the 5 mm joint in the corner area helps redirecting the plastic hinge from the columns to the beams.

In Fig. 8 is presented the cracks evolution in beams and slab, for four of the studied cases. It can be observed that for the *C*4 model the plastic area has more cracks. In the *C*6 case cracks are lead beyond the joint. The second row of images from Fig. 8 represents the stresses from the reinforcement. They support the idea that for the models with constructive measures Figs. 8*c* and *d*) the plastic hinges are redirected from the columns to the beams, as it was already shown in Fig. 7.

The ductility coefficient can be computed for all the analysed cases. An increase in its value is noticed with each considered measure. The ductility factor, computed as the ratio between the ultimate displacement and the yield displacement, takes values from 6.5 for the *C*2 model to 12.5 for the *C*6 model.



a *b* *c* *d*

Fig. 8 Cracks and stress development for : *a* – model *C*1; *b* – model *C*2; *c* – model *C*4; *d* – model *C*6.

Fig. 9 emphasizes the increase of the model rigidity when a slab is considered. The shear base force increases, while the displacement decreases for a load of 1,000 kN.

Comparing the capacity curves and the ductility factor values it is observed that the differences between models C4 and C5 are insignificant. Nevertheless the reduction in the slab reinforcement has a significant influence in the model behavior, increasing the flexibility of the model.

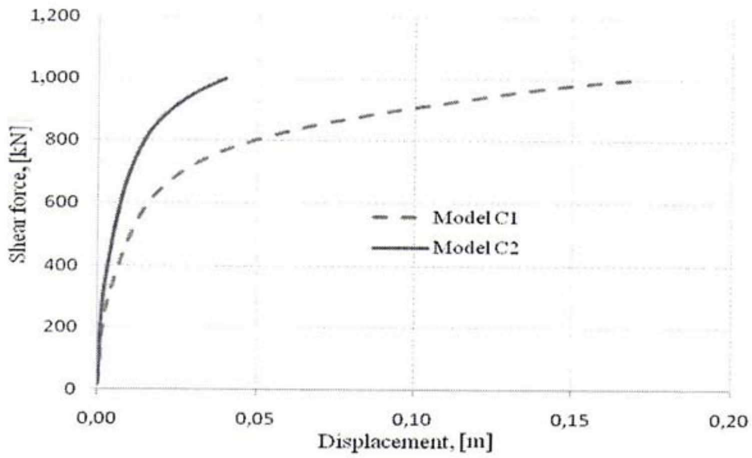


Fig. 9 – Capacity curves for frame without slab and frame with slab.

4. Conclusions

As conclusion it can be establish that the proposed constructive measures have an important influence on the global behavior of the model. The changes in the plastic hinge development were as initially supposed. They redirected the plastic hinge formation from the columns to the beams.

Considering the obtained results it can be stated that the most favourable constructive measure is represented by the combination of 5 mm joint and reduced reinforcement. For this case, the highest value for the ductility coefficient was obtained.

A decrease of the ultimate displacement and an increase of the shear force for the models with slabs in comparison with the beam–column model was to notice. This is due to the additional stiffness which the slab gives to the frame structures.

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ANALIZA NELINIARĂ A CADRELOR DIN BETON ARMAT CU PROGRAMUL ATENA 3-D

(Rezumat)

Evaluarea vulnerabilităţii structurale este esenţială pentru aprecierea comportamentului construcţiilor în caz de cutremur. Unul din conceptele de bază din proiectare este riglă slabă–stâlp puternic, dar cu toate acestea efectele cutremurelor arată că cedările nu se produc conform principiului de proiectare. Se propun soluţii constructive pentru dirijarea articulaţiilor plastice din stâlpi în grinzi, pentru a preveni colapsul structurii. Articolul tratează simularea virtual–experimentală a elementelor din beton armat supuse la încărcări orizontale. În acest studiu au fost considerate un stâlp din beton armat şi un cadru spaţial, pentru care s-au urmărit: evoluţia comportării acestora prin investigarea fisurilor dezvoltate în betonul armat, modul de propagare al acestora, cât şi influenţa evoluţiei tensiunilor din armături. S-au efectuat comparaţii multiple pentru cadrul spaţial, variind grosimea şi geometria plăcii, dar şi a armării acesteia. Calculele au fost realizate cu programul ATENA ce utilizează analiza de tip *stress analysis* cu element finit.