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INFLUENCE OF CONCRETE STRENGTH CLASS ON THE PLASTIC HINGES LOCATION FOR A REINFORCED CONCRETE MOMENT-RESISTING FRAME STRUCTURE WITH CONSIDERATION OF THE HORIZONTAL STIFFENING EFFECT OF THE SLAB

BY

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Abstract. The realistic degradation mode of the reinforced concrete (RC) frame structures can be verified through specialized theoretical and analytical studies. Thus, it was tried to demonstrate through this analytical research, the real mechanisms of seismic energy dissipation for RC moment-resisting frame systems with RC longitudinal rigid beams. In these conditions, it was studied the seismic response of a three GF+1F RC moment-resisting frame models (for three different concrete strength class and identical longitudinal and transverse reinforcements of structural elements) using nonlinear static analysis with ATENA software. Thus, it were specified important conclusions regarding the influence of the concrete strength class on the seismic degradation and collapse mechanisms for this type of structure (with rigid RC longitudinal beams). Also, it were observed the reinforcement insufficiencies in the vertical structural elements (RC columns) associated with the formation of plastic hinges.

Keywords: RC longitudinal rigid beams; concrete strength class; push-over analysis; RC frame system; RC columns plastic hinges.

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1. Introduction

The damage produced in reinforced concrete (RC) frame structures by dynamic loads, out of which the earthquake plays a significant role, tends to concentrate in the joint areas, leading to large residual displacements and the urgent need of post-seismic repairing of the structure (Navarro-Gomez and Bonet, 2019). Recent seismic events showed that structures should not only withstand strong motions, which occurs only for short periods of time during an earthquake, but also to be able to dissipate the energy induced by seismic motions with long duration (Raghunandan and Liel, 2013).

The prediction of the response of RC frame structure response, in view of the new performance-based seismic design paradigm of a new buildings or evaluation of an existing structure, requires modeling of all sources of flexibility. Therefore, in order to conduct an accurate nonlinear analysis of a reinforced concrete moment-resisting frame, engineers require not only trustworthy numerical models for beams and columns but also models that simulate joint response. According to Birely *et al*. (2012), in order for such models to be of practical for use, they should simultaneously fulfill the following conditions: (a) to be compatible with commonly used commercial software, (b) to support rapid model generation of the building, (c) be computationally efficient and robust, and (d) provide acceptable accuracy over a wide range of design configurations.

The nonlinear static (pushover) analysis procedure has become a popular analysis method among engineers, over the past decade, to estimate the structural responses, being less computational demanding than other methods such as nonlinear time history analysis (Jalilkhani *et al.*, 2020). The method can provide valuable information about the potential failure mechanisms, *e.g.* the formation sequence of plastic hinges, as well as the lateral load capacity of structures against the seismic loads. Although relatively simple to use, the conventional nonlinear static procedure has its shortcomings when it comes to taking into account the effects of higher modes of vibration and the stiffness degradation of structural elements, particularly for large lateral deformations that push the structural response into the inelastic range (Antonio and Pinho, 2004).

In these conditions of the analytical methods recognition (Pushover analysis), RC frame structures are presented with a laborious theoretical basis. Thus, it is known the seismic response of these types of RC structures and conclusions from the specialized study conducted by (Paulay and Priestley, 1992).

As a result, it was proposed to verify the deformation and degradation mode of three GF+1F RC moment-resisting frame models with longitudinal RC rigid beams, subjected to static horizontal actions. The cross-section height for longitudinal RC rigid beams was established from the pre-dimensioning stage with the 1/8L ratio, where L is the span between axes A and B, shown in Fig. 1 (P100-1, 2013).

This analytical verification of low rise RC frame structures (GF+1F) comes from necessity to validate the theoretical considerations present in the design stages for a 3D structure. This may prove to be an efficient way to identify all possible structural degradation mechanisms.

The basic theoretical criteria corresponding to the seismic response of RC moment-resisting frame structures (criteria that are valid in the current design norms – ex.: P100-1 Romanian norm for seismic design of structures) (P100-1, 2013), can be presented in the following form (Paulay and Priestley, 1992):

• In case of RC moment-resisting frame systems they dissipate the seismic energy by means of the interaction between the structural components with fragile seismic response and ductile seismic behavior;

• The RC frame beams are considered structural elements with ductile behavior. They have the property of plastic hinge formation in specially designed areas (both ends of the beam, at the connection points with the columns);

• The RC frame columns are considered structural elements with linear elastic seismic response. However, the occurrence of non-linear inelastic deformations for inferior end zones of the ground floor RC columns and superior end zones of the top storey RC columns for multi-storey structural systems are also considered;

• RC slabs are considered structural elements with infinitely linear elastic seismic response. Thus, RC slabs ensure the inertial masses transfer to the other load bearing elements, such as beams and columns.

2. Research Parameters

2.1. Geometry

The in-plane dimensions of the considered model are shown in Fig. 1. The geometrical dimensions of the scaled down RC frame model, ½ the size of a real frame structure, are presented in Fig. 2. The dimensions of the representative ½ scaled RC frame model are: L=2.4 m, B=1.8 m, as shown in Fig. 1. The model represents a low rise RC frame structure, GF+1F, with a storey height h_{st}=1.4 m, leading to a total height of the model H_{tot}=2.8 m (Fig. 2).

The importance class of the structure: III, according to (P100-1, 2013), ductility class: DCH (P100-1, 2013) and the building was considered to be located in Iaşi. The structural system consists of a pure RC moment-resisting frame, without the contribution of the non-structural components such as partitioning walls.

The main parameter of the research was the concrete strength class. The model was generated taking into account the similarity relations (El-Attar *et al*., 1991; Harris and Sabnis, 1999; Lu *et al.*, 2008). Three concrete strength classes were considered, frequently met in design practice C12/15, C16/20 and C20/25.

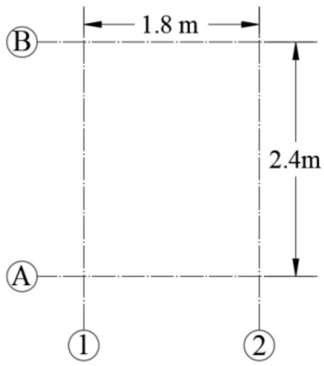


Fig. 1 – The inter-axis distance for RC moment-resisting frame models.

The cross-section height for the longitudinal and transversal RC beams were established according to the preliminary design criterion of 1/8L ratio (P100-1, 2013), where L is the clear span between columns, as presented in Fig. 2. This ratio represents the higher dimension admissible limit of the cross-section height established by the structural design norms (Eurocod 2, 2006; P100-1, 2013).

Therefore, based on the preliminary design ratio and taking into account the ½ scale factor for the model, the following dimensions of the elements were obtained:

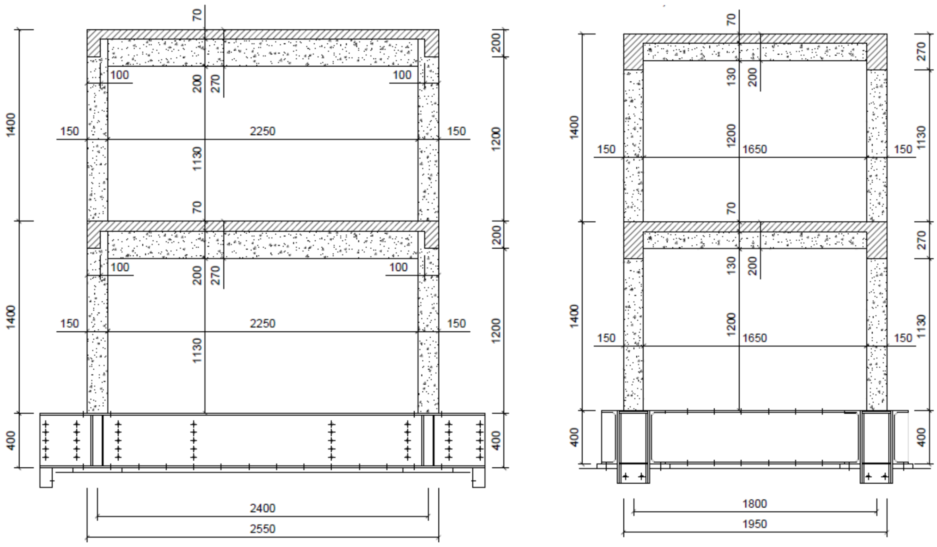
• RC columns: (bxh):15x15 cm;

• RC beams: longitudinal beam, LB, (bxh): 15x27 cm; transversal beam, TB, (bxh): 10x20 cm;

• Thickness of the RC slabs: h_s: 7 cm;

The longitudinal reinforcement ratio for the longitudinal beams was set as 0.8%, whereas for the columns it was chosen to be 1% per side. These longitudinal reinforcement ratios are in accordance with the specifications of P100-1 design code (P100-1, 2013). The slab was reinforced by means of welded wire 116GQ283 type; 6x100/6x100 (welded wire with 6 mm diameter and square mesh). The reinforcement details are summarized in Table 1 and shown in Fig. 4 for a better understanding of the layout.

The shear reinforcement consisted of ϕ4 Bst500M stirrups positioned at 5 cm in critical zones and 10 cm in other areas (Fig. 3). These critical areas were considered as the areas prone to the plastic hinge formation, that is the end sections of the beams. For the central part of the beams the stirrups were provided at a distance of 10 cm. In the case of columns, the spacing of the shear reinforcement was considered as 5 cm for the entire height of the column.



a *b*

Fig. 2 – Geometry of the RC moment-resisting frame models: (*a*) global longitudinal section; (*b*) global transverse section – dimensions in mm.

Table 1

Longitudinal Reinforcement Ratios for the RC Moment-Resisting Frame Models

CSC	NSC	Columns (15x15 cm)	LB (15x27 cm)	TB (10x20 cm)	Slab (h_s=7 cm)
C12/15	*M_1*	*4ϕ10*	*4ϕ10*	*4ϕ10*	*ϕ6*
C16/20	*M_2*	*4ϕ10*	*4ϕ10*	*4ϕ10*	*ϕ6*
C20/25	*M_3*	*4ϕ10*	*4ϕ10*	*4ϕ10*	*ϕ6*
*Note: CSC – **C**oncrete **S**trength **C**lass; NSC – **N**umerical **S**imulation **C**ode*					

2.2. Loading Scenario

The models were loaded by equivalent lateral static loads at the level of each slab, as shown in Fig. 4. The monitoring parameter was the formation of the plastic hinges. The lateral displacements (at the top) and the corresponding values of the loads will be recorded. The target location for the plastic hinges was the end of the beams, as per the commonly agreed design practice. However, the presence of the slab may have an influence on the location of the plastic hinges.

Each numerical model was named as shown in Table 1, in order to distinguish them from the subsequent numerical models to be considered at the later stages of the research.

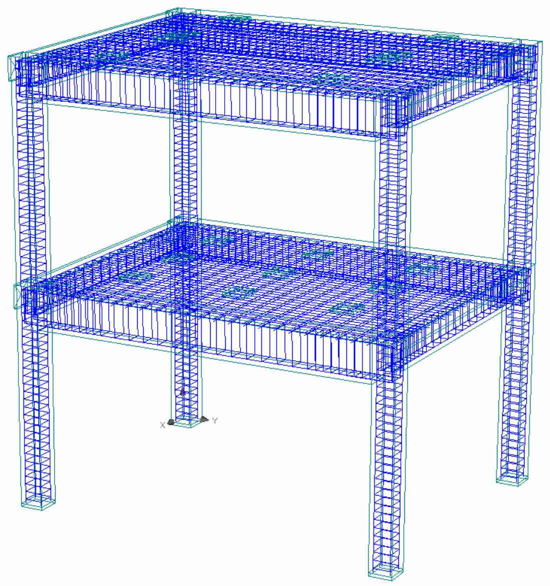


Fig. 3 – Representation of the reinforcement mode of the analytical representative RC moment-resisting frame model (ATENA software).

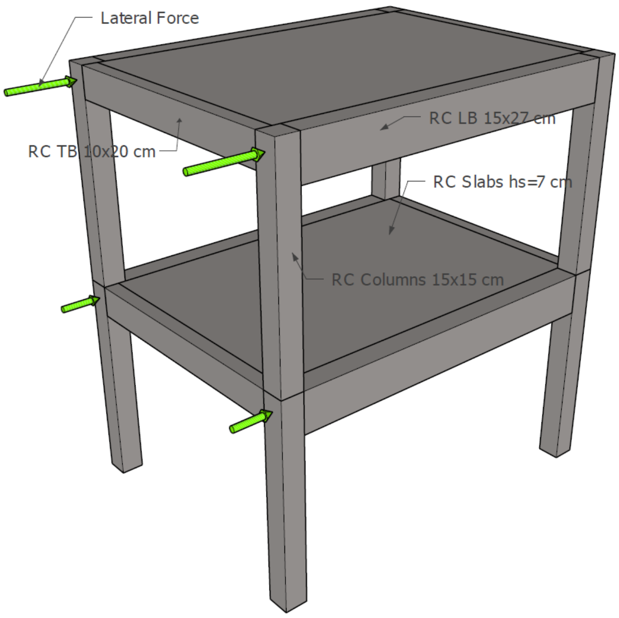


Fig. 4 − Direction of lateral forces application of the analytical representative RC moment-resisting frame model.

The obtained results will be analyzed based on the ultimate lateral displacements (at the top of the scaled down model), ultimate lateral forces corresponding to the displacements as well as the layout of cracks developed during the loading process

4. Results and Discussions

4.1. Ultimate Lateral Displacements and Loads

The ultimate lateral displacement for the three modelling scenarios is shown in Fig. 5. It can be observed that the highest lateral displacement was obtained for model M_2, concrete strength class C16/20. An increase in the concrete strength class to C20/25 leads to a 13.65% decrease in the lateral displacement. For the lower strength class, the decrease of the ultimate lateral displacement, the lateral displacement corresponding to the failure of the model, was 21%.

This may be due to the fact that, in the case of lower strength concrete, the concrete cracking mechanism occurs before the yielding process of the longitudinal reinforcement bars at the end areas (both ends) of either the beams or the columns. Thus, the maximum RC stresses and strains are concentrated in the beam-column joint area, between the ground floor (GF) and first floor.

When it comes to the ultimate resisting load, corresponding to the ultimate lateral displacement, the obtained results show that both models M_2 and M_3 had equal resistance capacities, as shown in Fig. 6. The lowest value for the ultimate load was obtained for the model made with C12/15 concrete.

By considering both Fig. 5 and Fig. 6, it can be concluded that the concrete strength class plays an important role in the seismic behaviour of RC moment-resisting frame structures. The higher the concrete strength class, the stiffer the model. This can be observed in Fig. 5 where the M_3 model showed lower values for the ultimate lateral displacement compared to M_2.

The same behaviour can be observed when the results are analysed from the point of view of the axial strain ε_{zz}. The strains are expressed in the global coordinate system, with z axis being the vertical one. Therefore, according to the values summarized in Table 2 and showed in Fig. 7, the concrete in the columns was severely crushed, exceeding the ε_{cu} = 3.5‰ limit for concrete. Moreover, the values of the axial strains did not exceed the yield value for BST500 steel, ε_y = 2%. These values were the values corresponding to the ultimate state.

Based on the obtained results shown in Fig. 7, the concrete strength class plays an important role when it comes to the total axial strains developed at the points of the column cross-sections in case of lateral loading scenarios. The increase in the lateral stiffness of the columns due to a higher concrete strength class results in higher axial strains, when comparing the M_1 and M_2 models. On the other hand, moving to a C20/25 concrete, model M_3, it follows that the axial strains decrease 12.88%, a similar value obtained for the percentage difference in terms of ultimate lateral displacements.

Table 2
Nonlinear Static Analysis Results for the RC Moment-Resisting Frame Models

CSC	NSC	ULD [m]	ULF [kN]	TSE	PFSM
C12/15	*M_1*	*0.01751*	*29.4*	*0.00392*	*0.00803*
C16/20	*M_2*	*0.02217*	*33.6*	*0.00784*	*0.01642*
C20/25	*M_3*	*0.01910*	*33.6*	*0.00683*	*0.03102*
*Note: ULD – **U**ltimate **L**ateral **D**isplacements; ULF – **U**ltimate **L**ateral **F**orces; TSE – **T**otal **S**train **E**ps zz; PFSM – **P**rincipal **F**racture **S**train **M**ax.*					

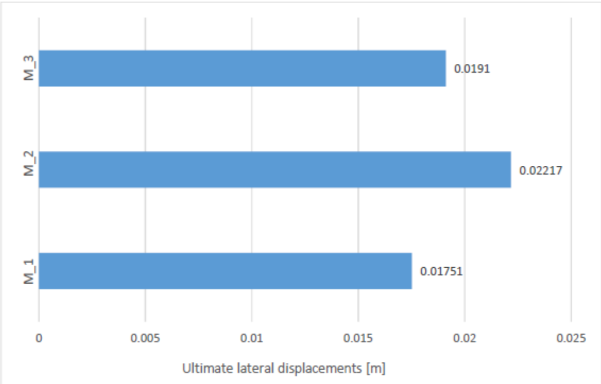


Fig. 5 – Influence of concrete strength class on the ultimate lateral displacement for M_1, M_2 and M_3 models.

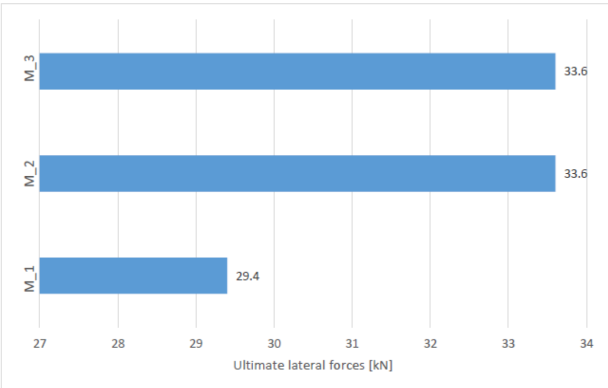


Fig. 6 – Influence of concrete strength class on the ultimate lateral loads for the M_1, M_2 and M_3 models.

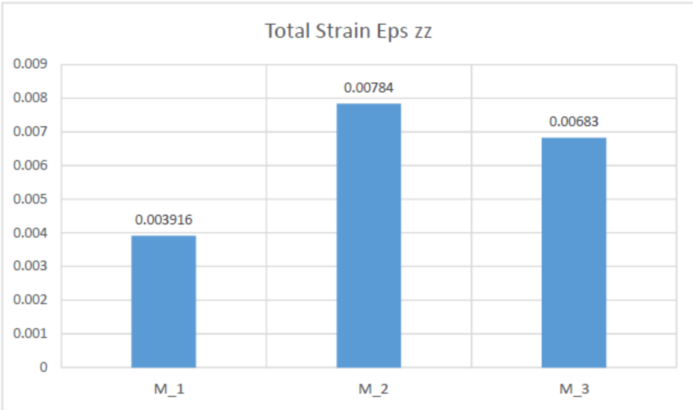


Fig. 7 – Influence of concrete strength class on the axial strain in columns for the M_1, M_2 and M_3 models.

4.2. Crack Patterns

Fig. 8 presents the cracking pattern in terms of axial strains ε_{zz} corresponding to the ultimate loading step. It can be observed that the concrete strength class plays an important role in the development of cracks – the higher the concrete class, the lower the number of cracks. Moreover, the cracking pattern is consistent with the direction of the lateral load application, Fig. 4.

The cracks tend to form at the lower part of the beams, near the load point of application (Fig. 4) and at the upper part of the beams for the opposite side of the frame structure, corresponding to the hogging action in case of the deformed shape. For all three considered models, the cracks tend to develop in the upper part of the slab, as well. This shows that the slab works together with the beams in transmitting the lateral loads to the columns.

The highest values for the axial strains were recorded in the columns, as expected. They were located in the tensioned part of the column cross-section at the base of the model and at the upper part of the columns, for both ground floor and the first storey, in the direction of the lateral displacement.

The obtained results are consistent with the results presented in terms of ultimate lateral displacements and the corresponding loads. For the case of M_1 model the maximum stresses and strains are concentrated in the beam-column joint area, between the ground floor (GF) and first floor (storey). In these conditions, the formation of plastic hinges and their location present an unfavourable global seismic response mechanism.

The principal strains corresponding to the ultimate stage, the failure of the numerical model, are presented in Fig. 9. It can be observed that the highest values for the principal strains were obtained for the M_3 model (C20/25 class).

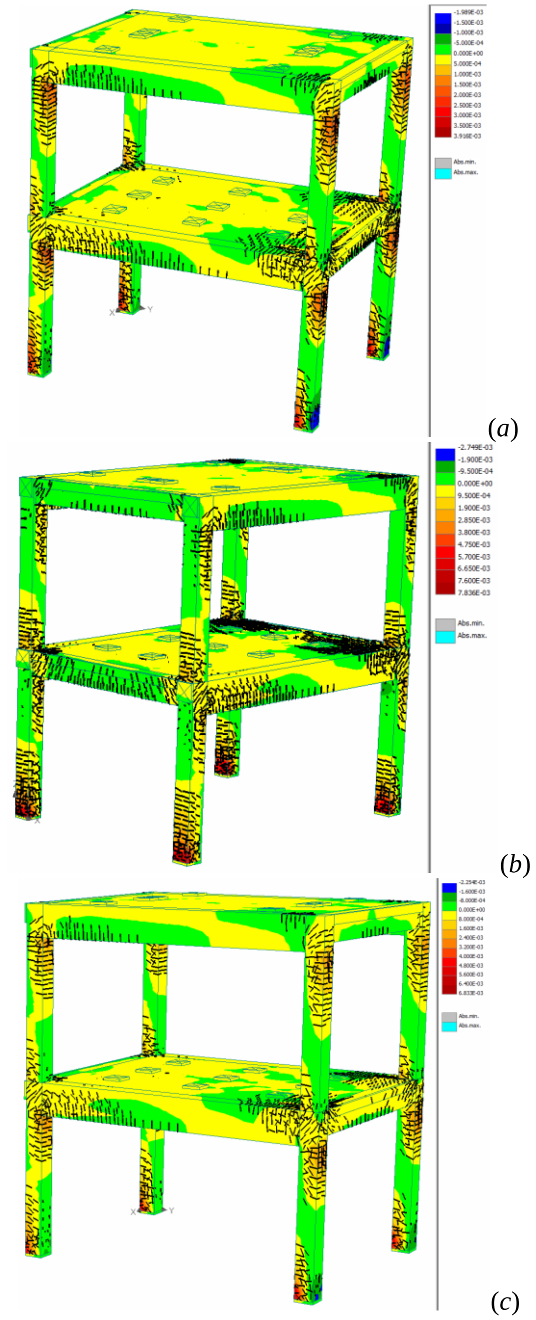


Fig. 8 – Eps zz strain for: (*a*) M_1; (*b*) M_2; (*c*) M_3 RC moment-resisting frame models, ultimate step of lateral loading.

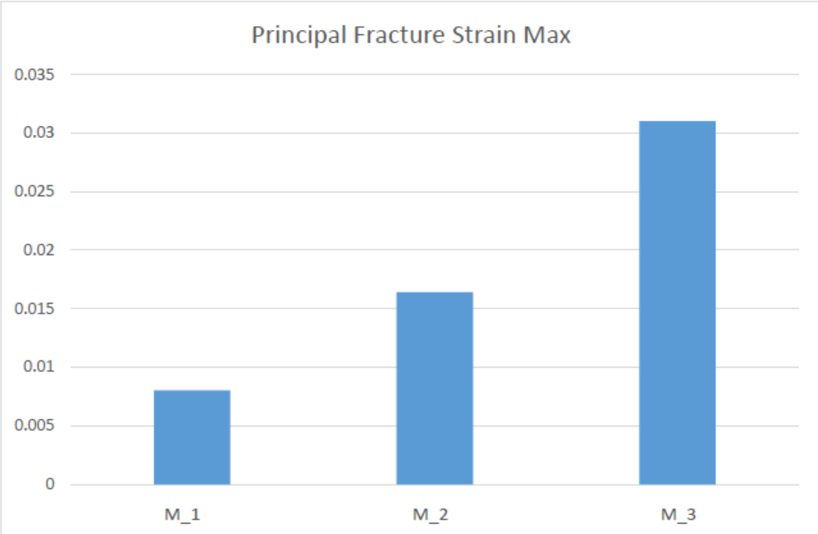


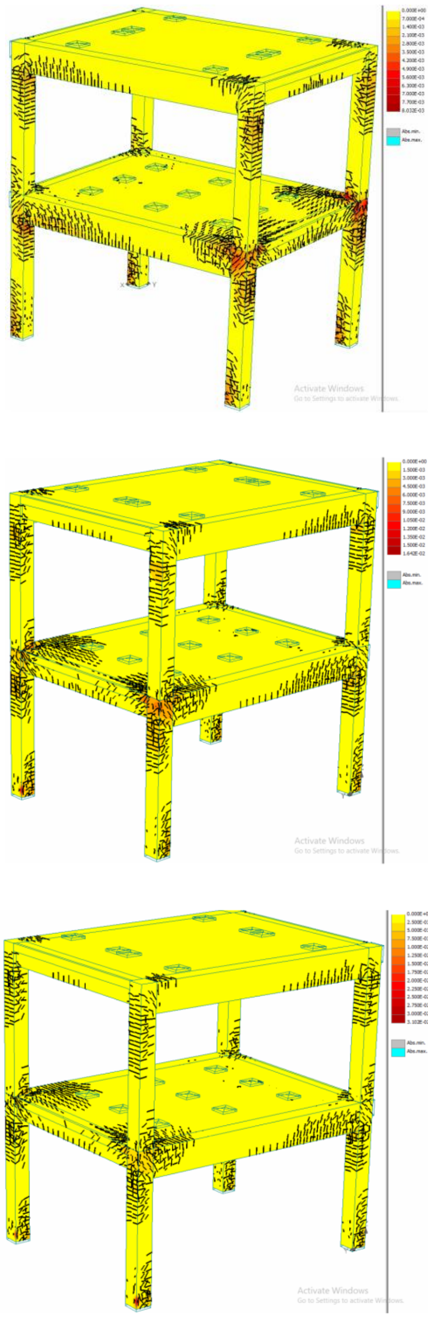
Fig. 9 – Influence of concrete strength class on the principal fracture strains for the M_1, M_2 and M_3 models.

From the obtained values, it follows that the most favourable scenario corresponds to the M_3 model where both the concrete was crushed and the longitudinal reinforcement yielded. For the M_1 and M_2 scenarios, even though the concrete crushed, the longitudinal reinforcement did not reach its yield strain. One possible solution would be to lower the longitudinal reinforcement ratio but this would lead to under-reinforced concrete section and would fall outside the specifications of current design codes (P100-1, 2013).

Fig. 10 presents the crack pattern as predicted by the principal fracture strain corresponding to the ultimate state, the failure of each model. It can be observed that the global seismic energy dissipation mechanism of the M_2 model is superior to the M_1 RC frame model but inferior to the M_3 moment-resisting frame model.

For the M_3 case, corresponding to a C20/25 concrete strength class, the obtained crack pattern is similar to the one obtained for the M_2 RC frame model. Practically, the higher strength class increases the global lateral deformation capacity of the structure, compared to M_1 model, and decisively influences the final mode of structural nonlinear inelastic mechanism deformation of the RC moment-resisting frame model.

The crack pattern shows a concentration of the cracks in the node region and generally follows the tensioned part of the deformed shape of the frame elements. The slab works together with the beams and therefore the cracking extends to the upper part of the slab in the hogging region of the beams.



(*a*)

(*b*)

(*c*)

Fig. 10 – Principal fracture strain max for: (*a*) M_1; (*b*) M_2; (*c*) M_3 models, ultimate step of lateral loading.

5. Conclusions

Following the numerical simulations for the M_1, M_2 and M_3 RC moment-resisting frame models with different concrete strength classes and longitudinal RC rigid beams (according to the permissible limits by the P100-1 seismic design norm of structures), it can be concluded that model M_3 (with C20/25 concrete strength class) presents the best seismic energy dissipation mechanism.

However, the horizontal stiffening effect of the RC slab associated with the longitudinal RC rigid beams, lead to the formation of a RC beams-slab-nodes rigid block. Thus, both end regions of the RC columns significantly crack in nonlinear inelastic domain.

In these conditions, the use of a higher concrete strength class is deemed necessary in a RC frame model, by considering the effect of longitudinal RC rigid beams on the plastic hinge formation at the end regions of the RC columns (contrary to the effects of the ductile concept).

Thus, a possible solution is the over-reinforcing of the RC columns, exceeding the allowable minimum limit with C20/25 concrete strength class application.

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*** *ATENA software, http://www.cervenka.cz.*

INFLUENȚA CLASEI DE BETON ASUPRA DEZVOLTĂRII ARTICULAȚIILOR PLASTICE PENTRU O STRUCTURĂ SEISMO-REZISTENTĂ PURĂ TIP CADRU DE BETON ARMAT CU CONSIDERAREA EFECTULUI DE RIGIDIZARE ORIZONTALĂ A PLĂCII

(Rezumat)

Modul realistic de degradare a structurilor tip cadru de beton armat poate fi verificat prin prisma studiilor teoretice și analitice de specialitate. Astfel, s-a încercat prin intermediul acestui studiu de cercetare, demonstrarea mecanismelor reale de disipare a energiei seismice pentru structurile tip cadru de beton armat cu grinzi longitudinale rigide. În aceste condiții, s-a studiat răspunsul seismic pentru o serie de trei modele P+1E pure seismo-rezistente tip cadru de b.a. (pentru trei clase de beton diferite și cu barele de armare a elementelor structurale indentice), utilizându-se analize statice neliniare cu programul de calcul ATENA. Astfel, s-au specificat importante concluzii cu privire la influența clasei de beton asupra mecanismelor de degradare seismică și colaps pentru acest tip de structură (cu grinzi longitudinale rigide). De asemenea, s-a observat insuficiența procentului de armare longitudinală în elementele structurale verticale (stâlpii de b.a.), asociată cu formarea articulațiilor plastice.