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Plastic Hinge vs. Distributed Plasticity in the Progressive Collapse Analysis

Mircea D. Botez[*1], Lucian A. Bredean[2], Adrian M. Ioani[3]

[1,2,3] *Technical University of Cluj-Napoca, Faculty of Civil Engineering. 15 C Daicoviciu Str., 400020, Cluj-Napoca, Romania*

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Abstract

The progressive collapse phenomenon represents a complex combination of plastic and dynamic behaviors of the structural elements. In order to obtain accurate results regarding structural response both material and geometrical nonlinearity should be considered. The dynamic nature of the event that could trigger the progressive collapse of a structure can be taken into account by performing a dynamic analysis. The aim of this study is to emphasize the differences between two distinct approaches regarding the material nonlinearity consideration: plastic hinge concept vs. distributed plasticity concept. Thus, a nonlinear dynamic analysis (NDA) is performed for two low-rise (3 and 6-story) reinforced concrete structures designed for a low seismic area (a_g=0.08g). Progressive collapse provisions specified by DoD(2009) are applied and the C_3 (corner column) failure scenario is analyzed. A numerical calibration for the distributed plasticity concept is performed. The conclusions reveal significant differences regarding the structural behavior in accordance with the considered plastic concept.

Rezumat

Colapsul progresiv reprezintă o combinație complexă între incursiunile elementelor structurale în domeniul plastic și efectele dinamice ale acțiunilor excepționale. Pentru a obține rezultate care să descrie cât mai fidel comportarea reală a structurilor, atât neliniaritatea materială cât și cea geometrică trebuie considerate. Natura dinamică a acțiunilor care pot duce la colapsul progresiv al unei structuri poate fi surprinsă cu ajutorul analizelor dinamice. Scopul prezentului studiu este de a evidenția diferențele dintre două modalități diferite de a surprinde neliniaritatea fizică: articulația plastică vs. plasticitatea distribuită. Două structuri în cadre din beton armat de trei respectiv șase niveluri, amplasate într-o zonă cu seismicitate scăzută, sunt analizate dinamic neliniar. Prevederile DoD(2009) referitoare la evaluarea riscului de colaps progresiv sunt aplicate pentru cazul de avarie al stâlpului de colț (C_3). Modelul plasticității distribuite este calibrat printr-un model numeric realizat în programul de calcul cu elemente finite, ABAQUS. Concluziile prezentului articol subliniază diferențele semnificative în aplicarea celor două concepte la evaluarea riscului de colaps progresiv al structurilor în cadre de beton armat.

Keywords: Progressive collapse, plastic hinge, distributed plasticity, nonlinear dynamic, RC structures.

[*] Corresponding author: Tel./ Fax.: +40 741 291 291
E-mail address: mircea.botez@mecon.utcluj.ro

1. Introduction

Progressive collapse represents “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately part of it” [1]. This phenomenon has become well known to the scientific community after the catastrophic event that took place in 1968 in London, England. Due to a deflagration caused by a gas leak, the entire south-east corner of the Ronan Point apartment building collapsed. Other similar events took place in 1986 when the New World Hotel from Singapore collapsed as a result of a human error (the self-weight of the structure was not considered by the design team) or in 2005 when last eleven floors of Windsor Tower form Madrid, Spain collapsed after a 5 hours fire. Because the fire lasted for such a long time the structural steel elements of the structure collapsed. Probably the most well-known progressive collapse event took place in 2001 at the World Trade Centre in New-York, SUA. Due to a terrorist attack both tower collapsed and approximately 3000 people died.

According to DoD(2009) [2] the progressive collapse potential can be assessed, based on the alternate-path procedure, using four different types of analysis: linear static (LSA), nonlinear static (NSA), linear dynamic (LDA) and nonlinear dynamic (NDA). The first type of analysis (LSA) has a low degree of complexity and involves basic knowledge in order to obtain a conclusion while NDA is the most complex and requires a better understanding of the phenomenon and also supplementary computational resources.

Previous studies [3, 4], based on linear static analysis (LSA) have shown that low-rise (3-6 stories) reinforced concrete structures, and in particular those located in low seismic zones, are more vulnerable to progressive collapse compared to mid-rise structures (10-13 stories). The nonlinear modeling of members and connections is considered as a more precise tool for evaluation of progressive collapse potential. In the most cases found in literature [5, 6, 7], the nonlinear analysis is based on the plastic hinge concept (M3 type) as it is defined by ASCE41 (2007) [8]. Another possibility for modeling the plastic hinge is by using the fiber hinge approach (P-M2-M3 type). These concepts can be used in certain commercial structural analysis software (ex.: SAP2000).Recently, in technical literature different approaches for considering the nonlinear behavior of reinforced concrete structures based on the distributed plasticity concept have been presented [9, 10]. These studies generally refer to the seismic behavior of the structures. The revealed structural response is different compared to the one indicated by an analysis based on the plastic hinge.

Starting from these reasons, in this paper the capacity of RC structures to resist progressive collapse is assessed and compared when the plastic hinge concept respectively the distributed plasticity concept is used in nonlinear dynamic analyses. Two reinforced concrete structures of three and six stories are analyzed by using the nonlinear dynamic analysis (NDA). Both structures, placed in a low seismic area (ag=0.08g) in order to limit the beneficial influence of seismic design, are designed in accordance with the Romanian seismic code P100-1/2006 [11]. Two structural analysis software are used: SAP2000 for the plastic hinge concept, respectively ABAQUS for the distributed plasticity concept. A comparison between the results obtained is then performed and the advantages respectively the disadvantages for each method are underlined. The distributed plasticity concept was calibrated against an experimental study on a reinforced concrete two-way slab [12] and a good agreement between the numerical and experimental results is obtained.

2. Numerical Model Calibration

The ABAQUS numerical calibration is based on a well-known experimental study involving a reinforced concrete two-way slab subjected to gravity loads. This study represents an investigation into the strength and behavior of nine panel two-way reinforced concrete slab (Figure 1a) [12]. The scale of the test structure was reduced at ¼ compared with the original structure. The concrete compressive strength was determined based on multiple batches and varied between 17.25 MPa and 25.25 MPa. The Young modulus was in the range of 17.4 GPa to 24.13 GPa. As reinforcement, three different types of steel (with ultimate stress values from 341 MPa to 462 MPa) were used. The load on each panel was applied by one jack, and distributed equally to 16 loading points or pads, by means of a pyramidal system of bars [12].

Based on the above specified characteristics, a numerical model is developed in ABAQUS (Figure 1b), using the "concrete damaged plasticity" option. This requires a detailed stress-strain curve that is obtained based on SR EN 1992-1-1 [13] provisions. For steel stress-strain curves, obtained from tests, in the model the "plastic" option is used. The structural elements (columns, beams and slabs) are modeled using solid finite elements (C3D20R type with 20 integration points) while the reinforcement is modeled using linear finite elements (T3D2 type with 2 integration points, one at each end) [14]. The mesh size is considered approximately equal to the slab height (50mm). The load, equally distributed to 16 loading pads from the experiment is approximated in the numerical model, as a uniform distributed surface load acting on all nine slab panels.

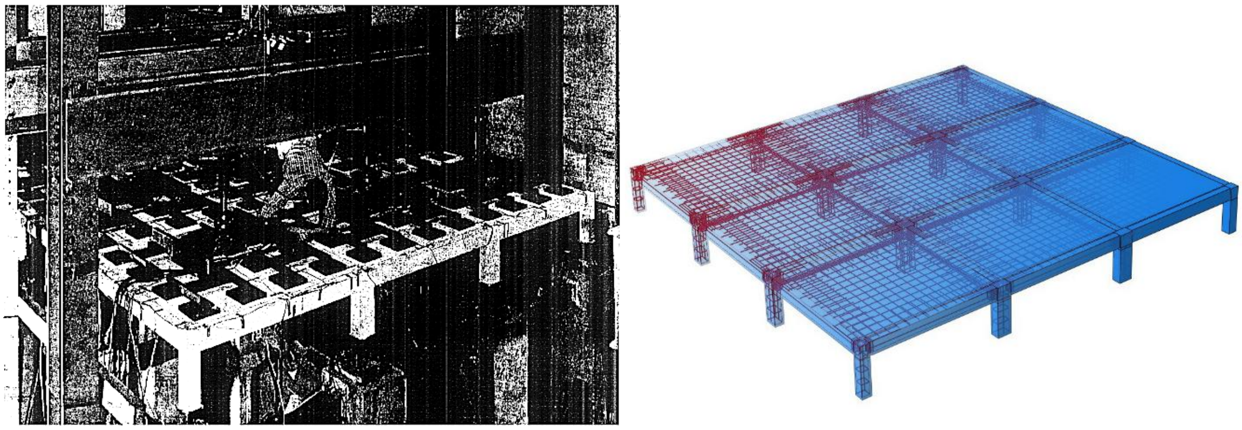

Figure 1. Overall view of: a) the test structure [1961], b) numerical model (ABAQUS).

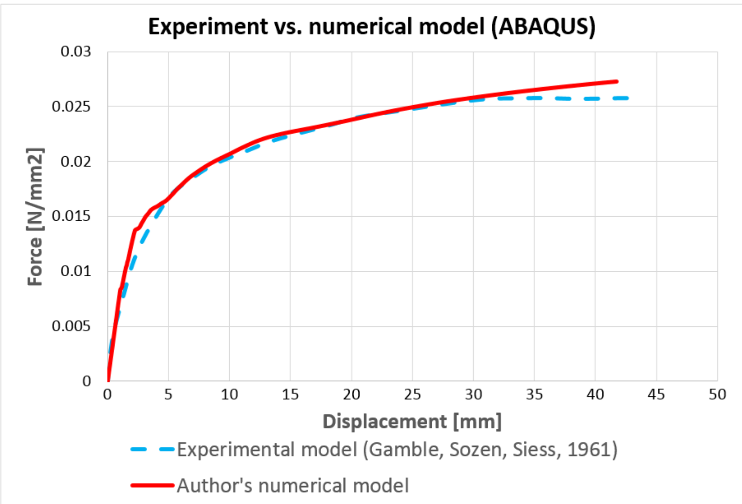

Figure 2. Force displacement curves: experimental vs. numerical.

Using the material properties mentioned above, a static nonlinear analysis is performed in ABAQUS. The numerical model force-displacement curve is obtained and compared with the experimental curve furnished by the test [12]. A good agreement between results is obtained (Figure

2). This shows that after the calibration, the numerical model developed in ABAQUS is capable to offer similar results to the ones obtained from the experiment, for both the elastic and the plastic domain.

3. Model Characteristics

The same configuration, excepting the floor numbers, is considered for both analyzed structures: three spans and five bays of 6.0m each and a story height of 3.15m. The structures are designed according to the provisions of the Romanian seismic code P100-1/2006 [11], provisions that are similar to those specified by SR EN 1998 -1 [15].

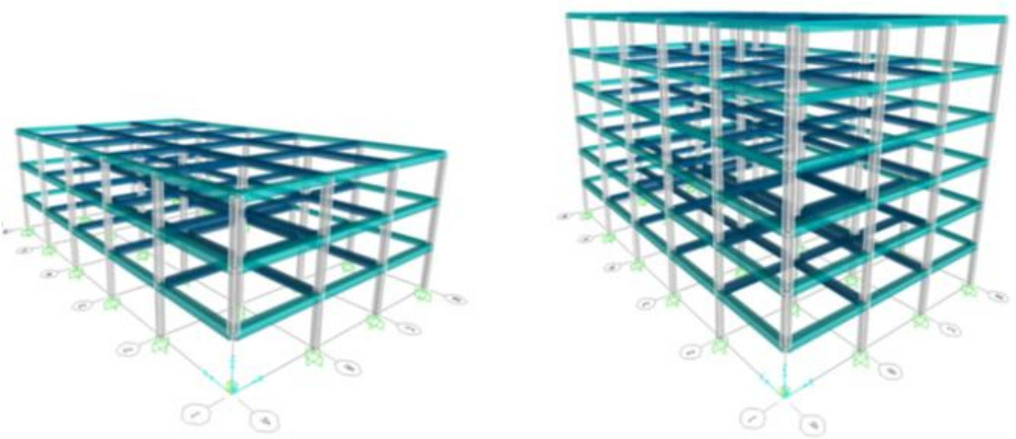

Figure 3. Structures geometry.

In order to minimize the beneficial influence of the seismic design, the low rise structures are considered to be placed in a low seismic area (a_g = 0.08g). Based on DoD(2009) [2] provisions, the slab may be considered as a primary element in the progressive collapse analysis. In this case, the slab is not modeled but its structural effect is taken into account by considering beams as T, respectively L- shaped cross section. The active slab width is determined according to the provisions of ACI 318 [16]. These details along with the cross section dimensions of the columns are presented in Table 1.

Table 1: Cross-sectional dimensions of structural elements

Structure	Column [mm]	Beam [mm]	
		Interior (T)	Exterior (L)
3–story	400 × 400	850; 150; 300; 250	550; 150; 300; 250
6–story	550 × 550	950; 150; 350; 250	600; 150; 350; 250

3.1 Distributed plasticity approach

Based on the good agreement obtained between the experimental and the numerical model presented in Section 2, the progressive collapse potential is assessed by ABAQUS program using the same finite elements types. The beams and columns are modeled using solid elements (C3D20R type with 20 integration points) while the reinforcement is modeled using linear elements (T3D2 type with 2 integration points; one at each end). During the analyses, the plastic deformations occurrence is verified at each material integration point [14].

The compressive strength class of the concrete is C25/30 (f_{ck} = 25N/mm2), and the steel for the longitudinal and transverse reinforcement is of S500 type (f_{yk} = 500N/mm2). The concrete stress-strain curve adopted in the numerical model is based on the same pattern used for the calibration. The stress-strain curves for concrete and steel are presented in Figure 4. In the seismic design of the models, an average safety coefficient of 10% is considered when the amount of reinforcing steel is established.

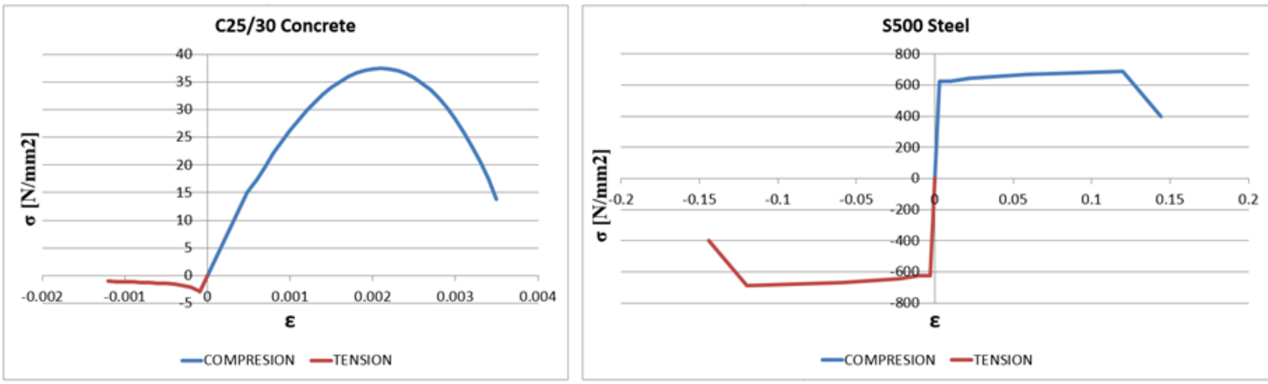

Figure 4. Stress-strain curves for concrete and steel.

3.2 Plastic hinge approach

There are multiple possibilities to model the plastic hinge when this concept is used in structural analysis. In this paper, two possibilities are considered: plastic hinge of M3 type, respectively plastic hinge of fiber type (P-M2-M3 type).
The plastic hinge of M3 type is defined according to ASCE41 [8]. Its behavior is presented in Figure 5. The original value corresponding to Collapse Prevention (CP) state is modified according to DoD(2009) [2] provisions to accommodate the particular issues associated with progressive collapse phenomenon.

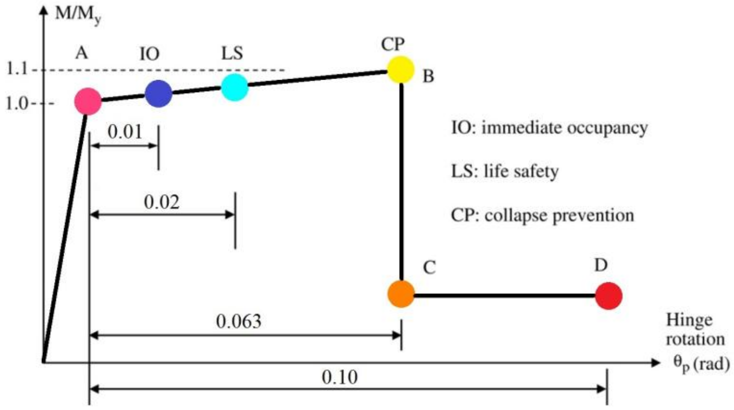

Figure 5. M3 plastic hinge behavior.

The fiber plastic hinge (P-M2-M3 type) involves a process of dividing the section in multiple longitudinal fibers. In this study, for each fiber in the cross section, the material nonlinear stress-strain curve is used to define the axial σ - ε relationship. Summing up the behavior of all the fibers in the cross section and multiplying by the hinge length gives the axial force-deformation and biaxial moment-rotation relationships [17]. The adopted hinge length is 0.5 of the beam height, according to Park and Pauley [18].

4. Analysis Procedures

Based on the provisions specified by DoD(2009) [2] Guidelines the progressive collapse potential can be assessed by removing a vertical structural element using the procedure called "missing column scenarios". Thus, an exterior column from the short side, an exterior column from the long side, an interior column or corner column is successively eliminated. In this paper only the corner column case - C3- is presented. In order to obtain a conclusion regarding the progressive collapse risk, a nonlinear dynamic analysis (NDA) is performed. A strength increase factor of 1.25 and 1.50 is applied to the specified strengths of steel, respectively of concrete [8]. The vertical load applied downward to the structure in this case, is computed in accordance with Eq. (1).

$$Load = 1.2D + 0.5L \quad (1)$$

where D represents the dead load (D=3.5 kN/m^2 + self-weight) and L represents the live load (L=2.0 kN/m^2) [2].

The corner column is removed almost instantaneously, in less than 1/10 of the period associated with the structural response mode for the vertical motion of the bays above the removed column, as determined from the analytical model with the column removed [2]. In this case a removal time of 0.005 seconds is specified. A 3- second total time is considered for the nonlinear dynamic analysis (NDA) along with a 0.005 seconds step size. A 5% damping factor is also taken into account in this study.
According to DoD(2009) [2] provisions, the acceptance criterion for the nonlinear dynamic analysis (NDA) is fulfilled if the plastic hinge rotation (Θ) has a smaller value than the specified limit (Θ_a). In this study, based on the beams characteristics, the limit rotation used in these analyses for RC framed structures is 0.063 rad (Figure 5) [2].

5. Progressive collapse analysis

The progressive collapse risk assessment is performed based on the DoD(2009) [2] provisions. The plastic rotation, necessary in order to obtain a final conclusion regarding the collapse potential, is determined in accordance with Eq. (2).

$$\theta = \Delta / L \quad (2)$$

where Θ represents the plastic rotation, Δ is the corresponding vertical deflection and L is the clear span length.

5.1 Plastic hinge concept

As it was previously mentioned, in this study two plastic hinge models are used together when the SAP2000 structural analysis software.

5.1.1 M3 hinge type

a) Three story structure

Under the standard DoD gravity loads (Eq. 1), the maximum plastic hinge rotation obtained for the 3-story structure is Θ=0.044 rad, approximately two-thirds of the DoD(2009) [2] specified limit of 0.063 rad. This rotation value, corresponds to a maximum displacement of Δ= 24.4 cm. These values are obtained for a 3 seconds nonlinear dynamic analysis (NDA). As may be seen in Figure 13 the displacements tend to increase even after those 3 seconds. The failure criterion was not reached (Θ=0.044 rad < Θ_a=0.063 rad). Due to this fact the nonlinear dynamic analysis (NDA) was performed for 6 seconds duration. The new results (Figure 10) show that the increasing displacements tendency is preserved. The failure criterion specified by DoD(2009) [2] is reached for a vertical displacement of Δ= 35.33 cm (Figure 6). Consequently the structure does not satisfy progressive collapse resistance requirements [2] code in terms of plastic rotations and it is not adequate to resist progressive collapse. The analysis run-time for the 3-story structure is 47 minutes.

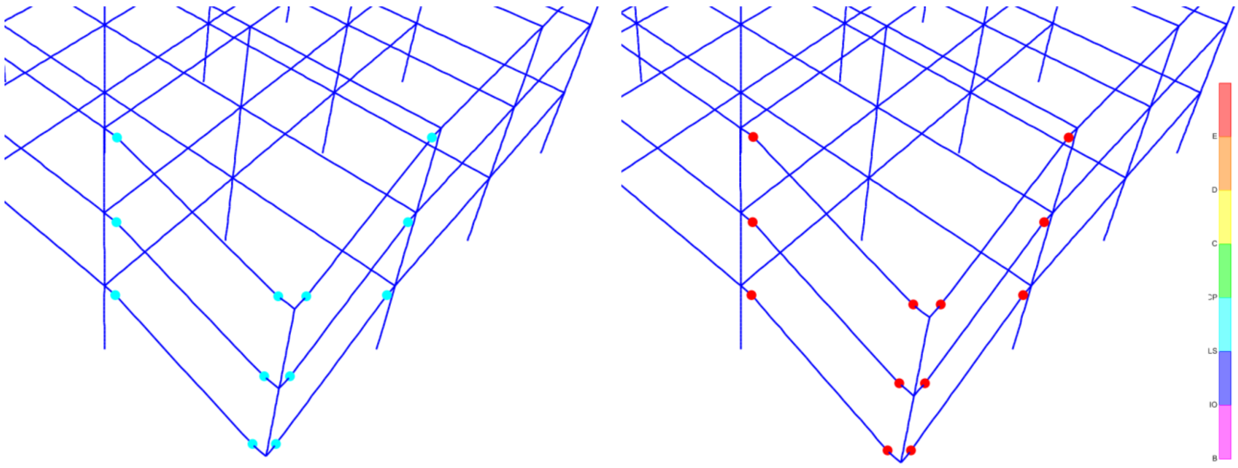

Figure 6. 3-story structure: plastic hinge appearance at a) 3 seconds, b) 6 seconds.

b) Six story structure

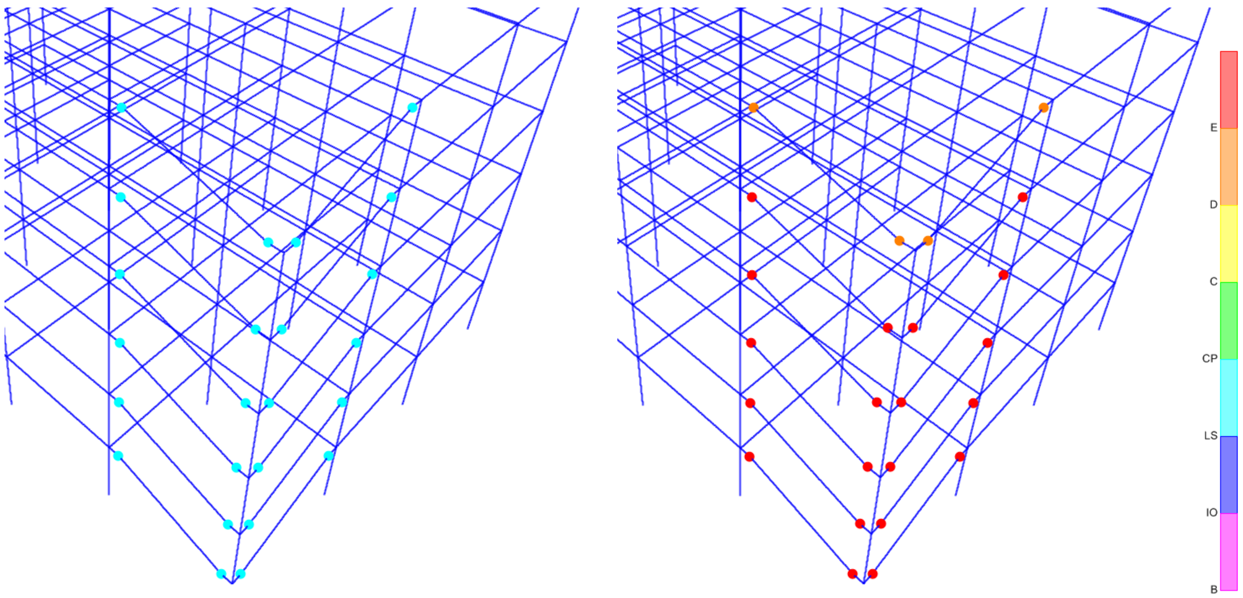

Figure 7. 6-story structure: plastic hinge appearance at a) 3 seconds, b) 6 seconds.

For the 6-story structure the DoD(2009) [2] failure criterion is reached for a vertical displacement Δ= 35.33 cm (Figure 7). Thus, the structure does not meet the requirements for progressive collapse resistance, when the corner column case is considered. This verdict is obtained after a nonlinear dynamic analysis run-time of 72 minutes.

5.1.2 Fiber hinge type

For the fiber hinge, bar (beam) cross-sections are meshed into 5cm square longitudinal fibers. Reinforcement bars represent distinct fibers. During the analysis, each fiber's behavior is governed by the appropriate (σ-ε) defined material law (concrete respectively steel).

a) Three story structure

In the 3-story structure case, a maximum displacement Δ_{max}= 4.41 cm is obtained. The plastic hinge rotation, corresponding to this level of deflection is Θ=0.0079 rad, much smaller that the allowable DoD(2009) [2] limit Θ_a=0.063 rad. A run-time of 33 minutes is necessary in order to obtain the results through the nonlinear dynamic analysis (NDA) with 3 seconds duration. Consequently, in this case, the structure is adequate to resist progressive collapse, result which is in contradiction with the one obtained via M3-type plastic hinge.

b) Six story structure

The nonlinear dynamic analysis (NDA), with a 3 seconds duration, performed for the 6-story structure provides a maximum value for the plastic hinge rotation Θ=0.0065 rad. This value corresponds to a maximum deflection Δ_{max}= 3.65 cm. Thus, the structure meets the requirements for progressive collapse resistance, when the corner column case is considered, result which is not consistent with the one obtained via M3-type plastic hinge. In this case, a total run-time of 67 minutes is necessary.

5.2 Distributed plasticity concept

a) Three story structure

For the first analysed structure (3-story) the nonlinear dynamic analysis (NDA) based on the distributed plasticity concept led to a maximum plastic rotation Θ=0.0135 rad, according to a maximum vertical displacement Δ_{max}= 7.57 cm.

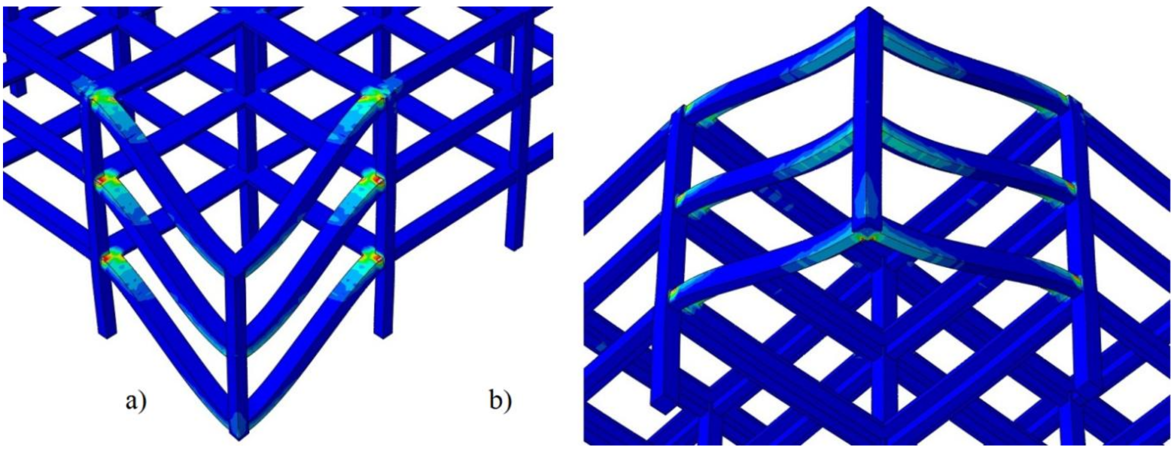

Figure 8. 3-story structure: equivalent plastic strain in tension: a) top view, b) bottom view.

In this case, the DoD(2009) failure criterion is not reached (Θ=0.0135 rad < Θ_a=0.063 rad) and the vertical displacement of the structure remain at the constant value Δ_{max}= 7.57 cm. The maximum tension stress recorded in reinforcement bars is 626 MPa, approximately the same as the yield stress f_y = 625 MPa. An extremely large run-time of 31 hours is necessary to perform the analysis. The zones which suffered significant plastic deformations are presented in Figure 8 (a) top view b) bottom view. The structure satisfies progressive collapse resistance requirements of the DoD(2009) [2] code in terms of plastic rotations and consequently is adequate to resist progressive collapse.

b) Six story structure

In the 6-story structure case the maximum vertical displacement Δ_{max}= 5.15 cm obtained by the nonlinear dynamic analysis (NDA) corresponds to a plastic rotation of ϴ=0.0094 rad. This value is smaller than the allowable value (ϴa=0.063 rad) specified by DoD(2009) [2], and consequently it is adequate to resist progressive collapse. After reaching the peek value of 5.15cm the vertical displacement no longer increase. The reinforcement bars reached a maximum tension stress of 625 MPa, equal to the yield limit. The plastic deformations of the structural elements (top and bottom) under the applied loads may be seen in Figure 9. In this case, a total run-time of 36 hours is necessary.

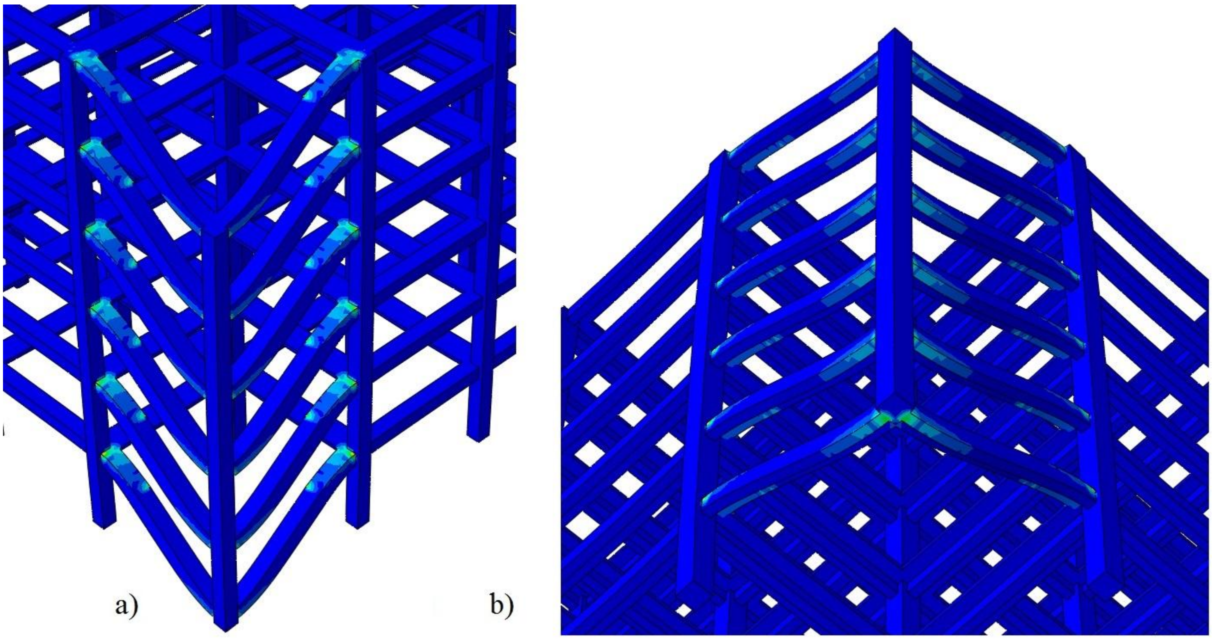

Figure 9. 6-story structure: equivalent plastic strain in tension: a) top view, b) bottom view.

6. Synthesis of results

The results obtained through the nonlinear dynamic analysis (NDA) using two different plastic concepts - plastic hinge and distributed plasticity - are synthetized in the following graphics. The first concept is used through two distinct types of plastic hinges: the M3-type plastic hinge, respectively the fiber-type plastic hinge.

Figure 10 presents the time-displacement curves obtained when the plastic hinge of M3-type is considered for the 3 and the 6-story structures. The results obtained through the same plastic hinge concept but with a different approach (fiber-type plastic hinge) are shown in Figure 11. For the distributed plasticity concept, the obtained time-displacement curves displayed in Figure 12. A comparison between the results obtained through the mentioned approaches for the 3-story structure is presented in Figure 13. For the 6-story structure the time-displacement curves are compared in Figure 14.

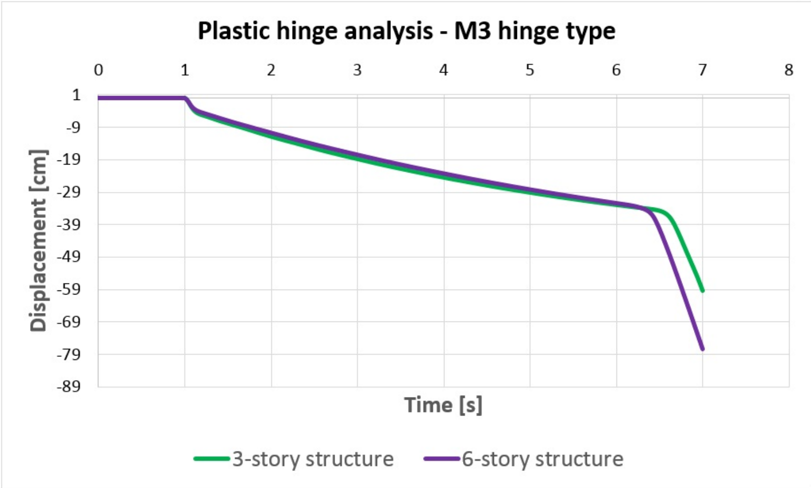

Figure 10. NDA: time-displacement curves for 3 and 6-story (M3 plastic hinge approach).

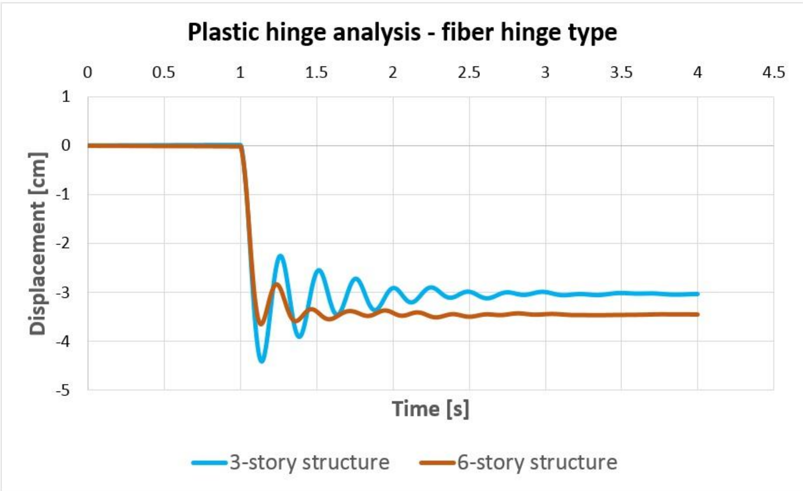

Figure 11. NDA: time-displacement curves for 3 and 6-story (Fiber plastic hinge approach).

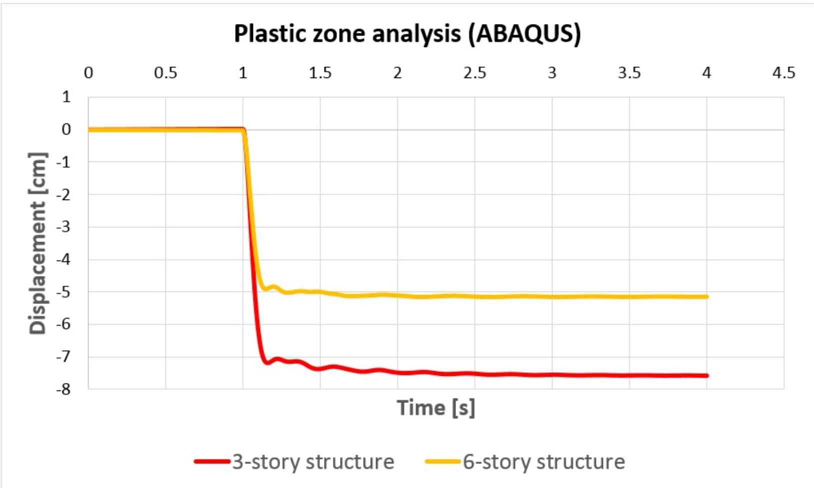

Figure 12. NDA: time-displacement curves for 3 and 6-story (distributed plasticity approach).

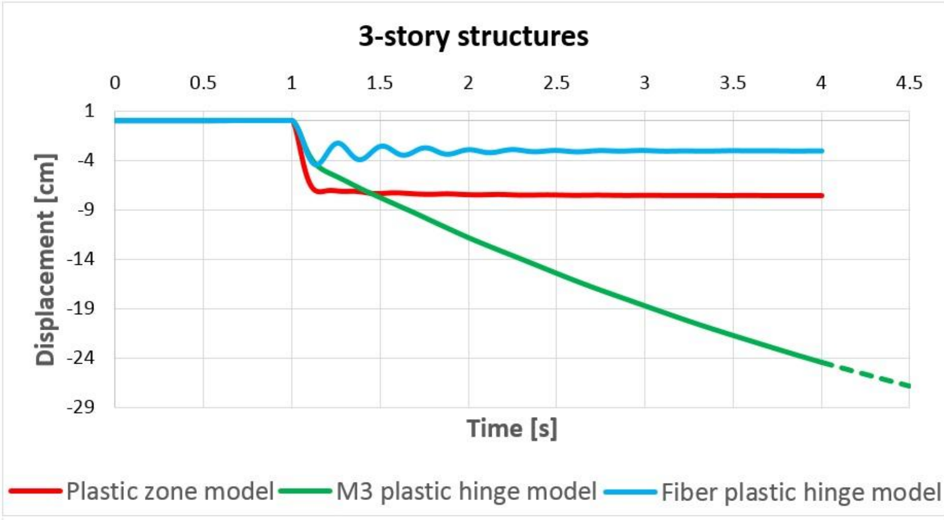

Figure 13. 3-story structure: M3-type plastic hinge vs. fiber-type plastic hinge vs. distributed plasticity.

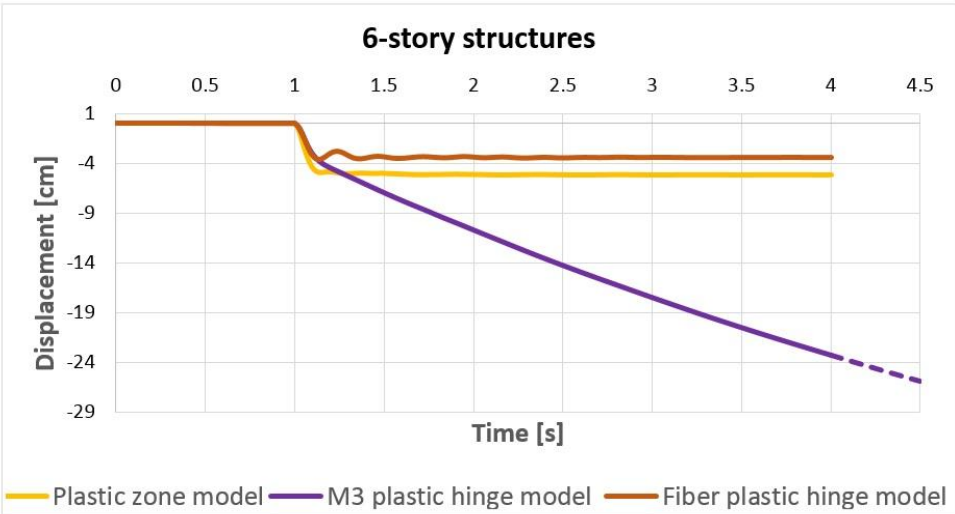

Figure 14. 6-story structure: M3-type plastic hinge vs. fiber-type plastic hinge vs. distributed plasticity.

7. Conclusions

The main goal of this paper is to assess progressive collapse risk of reinforced concrete framed structures based on advanced plasticity concepts and to emphasize the differences observed with respect to the current approach (M3-type plastic hinges). Although other studies are based on older guidelines for progressive collapse risk assessment such as the GSA(2003) [19] or DoD(2005) [20], in this study the provisions of the DoD(2009) [2], considered the most advanced guideline related to progressive collapse topic, are applied. Using the nonlinear dynamic analysis (NDA), the capacity to resist progressive collapse of two low-rise reinforced concrete structures (3 and 6-story) is assessed. Previously, a calibration analysis for the distributed plasticity model implemented in ABAQUS FE software is performed. The main conclusions of the study are:

1. A real two-way reinforced concrete slab system (beams and slabs) supported on columns, tested by Gamble, Sozen and Sieess [12], is modeled in ABAQUS. The distributed plasticity

concept is applied in the analysis of RC frames via the ABAQUS software. The slab system is loaded similarly to the experimental specimen. As the Figure 2 shows, the calibration analysis performed in ABAQUS leads to a good agreement between the numerical results and the experimental ones [12].

2. The analyses results (Figure 10, 11, 12) indicate for each approach, a better behavior (smaller maximum displacements, reduced zones affected by significant plastic deformations) of the taller structure. Figure 10, 11, 12 illustrates this conclusion which confirms the general opinion, according to which, the response of a structure to abnormal loads improves when its degree of redundancy increases; in particular, an improved behavior of structures with an increased number of stories has to be emphasized.

3. A major difference between the two plastic hinge approaches is observed. As Figure 10 shows, when the M3-type plastic hinge is used, both 3-story and 6-story structures are not adequate to resist progressive collapse. This verdict is not consistent with the conclusion furnished by the fiber-type plastic hinge approach (Figure 11).

4. The fiber-type plastic hinge and the distributed plasticity concept offer the same progressive collapse verdict for both structures when the nonlinear dynamic analysis (NDA) is performed. For the 3-story structure the peak value of the deflection via the distributed plasticity approach is greater with 72% compared to the one obtained via fiber-type plastic hinge. For the 6-story structure a 41% difference is recorded.

5. In terms of computational and run-time costs, the distributed plasticity concept is the most “expensive” one. For example, in the 3-story structure case the run-time changes from 47 minutes (M3-type plastic hinge) to 33 minutes (fiber-type plastic hinge). These time intervals are much smaller than the distributed plasticity run-time of 31 hours. Similar differences are obtained for the 6-story structure.

6. Despite of the run-time disadvantage, the analysis based on the distributed plasticity concept provides the most trustful results. Starting from an initial increased design effort (modeling and parameters definition) for certain loading levels this type of analysis (distributed plasticity) can prevent the re-design of the structure based on the progressive collapse verdict. For the analyzed structures the incapacity to resist progressive collapse indicated by the M3-type plastic hinge approach is not confirmed by the more accurate verdict offered by the distributed plasticity approach.

8. Acknowledgment

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