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Nonlinear static and dynamic analyses of reinforced concrete buildings – comparison of different modelling approaches

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Abstract. It generally accepted that most building structures shall exhibit a nonlinear response when subjected to medium-high intensity earthquakes. It is currently known, however, that this phenomenon is not properly modelled in the majority of cases, especially at the design stage, where only simple linear methods have effectively been used. Recently, as a result of the exponential progress of computational tools, nonlinear modelling and analysis have gradually been brought to a more promising level. A wide range of modelling alternatives developed over the years is hence at the designer's disposal for the seismic design and assessment of engineering structures. The objective of the study presented herein is to test some of these models in an existing structure, and observe their performance in nonlinear static and dynamic analyses. This evaluation is done by the use of two of a known range of advanced computer programs: SAP2000 and SeismoStruct. The different models will focus on the element flexural mechanism with both lumped and distributed plasticity element models. In order to appraise the reliability and feasibility of each alternative, the programs capabilities and the amount of labour and time required for modelling and performing the analyses are also discussed. The results obtained show the difficulties that may be met, not only in performing nonlinear analyses, but also on their dependency on both the chosen nonlinear structural models and the adopted computer programs. It is then suggested that these procedures should only be used by experienced designers, provided that they are aware of these difficulties and with a critical stance towards the result of the analyses.

Keywords: material nonlinearity; reinforced concrete sections; lumped plasticity models; distributed plasticity models; nonlinear seismic analysis

1. Introduction

The response of a building to an accelerated ground motion is carried by a number of physical mechanisms developed by a complex association of structural elements. These elements are assembled together to grant civil engineering structures with three fundamental components: stiffness, resistance and ductility. When provided with a balanced proportion between these characteristics, each structure shall be able to: (1) control displacements, (2) avoid damage under low seismic intensities and withstand the remaining design actions and (3) control damage spread, accommodate large displacements without early collapse and dissipate energy.

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Structural designers, however, have mostly relied the seismic design and assessment of buildings on linear methods, which have been proved not to describe in a correct fashion the actual behaviour of asymmetric structures (Priestly 2003). In fact, the structural response is characterized by a complex nonlinear behaviour where the internal forces are continuously redistributed as the structure goes through the inelastic stage. This complex behaviour has been studied for the last few decades in order to grant engineers with more trusted means to predict the seismic response of building structures. A wide number of nonlinear modelling alternatives, analyses and computer programs have been developed and reported in several research studies. The better understanding of material behaviour, the growing performance of the existing element models and the increasing development of computational methods may turn nonlinear analysis into a generalized tool. The current challenge lies on the development of user-friendly modelling software and simplified/adequate nonlinear methods compatible with the time constraints found in design offices, and adapted to the possible lack of knowledge of engineers on nonlinearity issues.

The main goal of this work is to initially summarise some of the existing nonlinear models and to use some of those available in two different computer programs for a specific case study. In this paper, a brief description of nonlinear models and analyses is made, with main focus on those that are featured in SAP2000 (2008), essentially a linear analysis program that has lately started to develop nonlinear analysis; and SeismoStruct (2010), a more recent program specialized in the nonlinear field. Six different three-dimensional models were built with these software applications, and a careful accuracy evaluation was performed using nonlinear static and dynamic analyses.

As an important fact, this work will focus on the flexural behaviour of structural elements, meaning that no shear failures are expected to affect the nonlinear response of the case study, which is known to be a rather optimistic hypothesis, especially on older RC buildings. In fact, shear models are currently under development and their applicability to complex systems is still far from being straight forward. Moreover, recent capacity design standards prescribe that shear resistance be assured so as not to harm flexural ductility, i.e., to remain linear.

The amount of work required to build each model in the aforementioned software and the time consumed by each analysis and procedure were taken into account to evaluate the efficiency of each alternative. Moreover it intends to warn potential users for the difficulties in performing nonlinear analyses and for the strong dependence of the results on the nonlinear structural model adopted.

In the end of the endeavour, final conclusions were outlined in order to support future users on the choice among the selected models for the seismic assessment of reinforced concrete buildings.

2. Nonlinear models

Structural elements designed to absorb deformation without brittle failures are expected to form plastic hinges at their most critical sections. These sections are subjected to multiple excursions into the inelastic range, often followed by their gradual mechanical degradation caused by the cycles of loading and unloading.

Numerical models for this type of frame structures have basically fallen into two categories: (1) the distributed plasticity models, where the inelastic behaviour of the whole element is modelled to automatically compute the spread of plasticity along its length and (2) the concentrated plasticity models, where the inelastic behaviour is lumped at the critical sections, corrected by a fixed parameter that assumes an ideal plasticity distribution.

To model the flexural behavior of the cross sections it is often used either a definition of hysteretic rules or a fibre discretization model, where a uniaxial model of each fibre material is directly computed.

2.1 Materials

The monotonic behaviour of concrete and steel has been observed since their first employment. The developed analytical models have mainly focused on reproducing an adequate stiffness and strength at any strain level as well as the effects of different confinement disposals. More recently, particular significance has also been given upon the development of reliable models of concrete under cyclic loadings. As an example, the models of Scott *et al.* (1982) and Mander *et al.* (1988) have been used for modelling the cyclic behaviour of concrete specimens. The former was complemented with the loading and unloading rules proposed by Thompson and Park (1980), and the latter using a unique expression for the monotonic envelope and specific cyclic rules, later modified by Martinez-Rueda and Elnashai (1997).

The nonlinear behaviour of reinforced concrete members is highly controlled by the reinforcement. Therefore, steel models for longitudinal bars are extremely important to accurately compute the flexural behaviour of a reinforced concrete section, and especially when it is subjected to load reversals. The Ramberg-Osgood relations (1943) used by Kent and Park (1973) and the Menegotto and Pinto (1973) model are two examples of modelling the steel cyclic behaviour, where the characteristic softening of the curves in the reloading branches is automatically considered. The Menegotto and Pinto (1973) model has also been included in several studies for its simplicity and efficiency. It has also been implemented in *SeismoStruct* program, with the isotropic hardening rules proposed by Filippou *et al.* (1983).

2.2 Sections

The use of hysteretic relationships represents a relatively simple way to model the flexural behaviour of an element cross section, i.e., a numerical relationship between moment and curvature. This kind of formulation is generally based on the definition of a monotonic envelope and on a set of rules for the definition of hysteretic loops (Stojadinovic and Thewalt 1996), which are calibrated to assess element stiffness, ductility and energy dissipation. Usually when a moment-curvature envelope is assigned to model the biaxial bending of a structural element, the two directions are considered separately and no axial force interaction is taken into account during analysis. The idealization of these models is hence performed to an average axial force, which is generally obtained by means of a linear analysis to gravity loads. Another simple model of the flexural behaviour of a cross section is the fibre model (Taucer *et al.* 1991). This model can take into account biaxial bending and axial force interaction. It consists of a discretization of the cross section into a finite number of axial springs acting in parallel, by considering the Euler-Bernoulli beam theory. The section stiffness is computed based on the tangent stiffness of each fibre material, on its area of influence and on its coordinates within the cross section.

There have been a number of recent research studies that also include the effect of shear in the flexural model (e.g. Sezen and Chowdhury 2009, Petrangeli *et al.* 1999). This important factor has been indicated as the next step to the seismic assessment of existing RC structures, which are particularly sensitive to shear mechanisms.

2.3 Elements

The maximum internal forces produced in a building structure, subjected to seismic action, occur at the elements end sections and their distributions may be assumed as linearly varying. Therefore, the concentrated plasticity models have emerged by simply lumping flexural nonlinearity at these end sections.

The first formulation of concentrated plasticity models was proposed by Clough *et al.* (1965) and consisted of an association in parallel of an elastic and an elastoplastic element. The most common formulation, however, was initially proposed by Giberson (1967) with an association in series defined by a nonlinear rotational spring at each end of a linear elastic element. The rotational springs are defined by a moment-rotation relationship M-θ that integrates the inelastic curvature distribution expected along the nearest sections, which form the plastic hinge, while the element itself behaves elastically with limited forces. A very common approach is to admit a uniform distribution of the inelastic curvatures along a plastic hinge length L_p, which is reassigned by empirical expressions, by defining the plastic rotation as $\theta = \chi\ L_p$, where χ is the plastic curvature of the cross section.

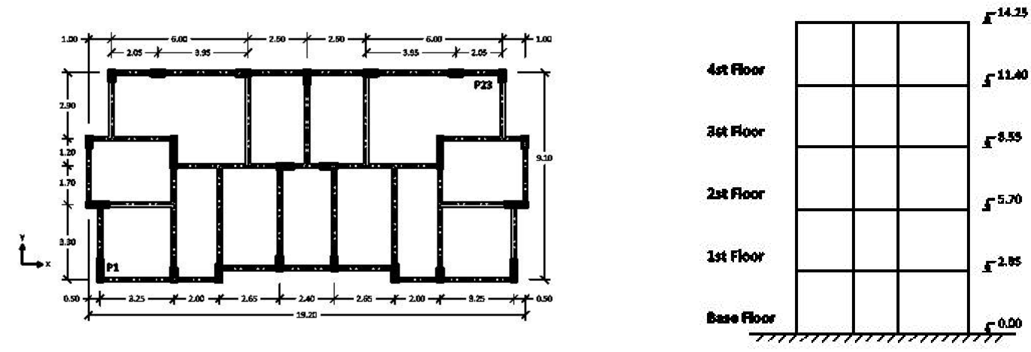
In a distributed plasticity model, a finite number of cross sections are usually computed throughout the element to more accurately consider the inelasticity spread along its length. An old example of this formulation was suggested by Takayanagi and Schnobrich (1979), by placing several hinges along the element with specified lengths. This approach is also suggested by the *SAP2000* reference manual (SAP2000 1995). However, the most generalized formulation is based on numerical integration of section quantities at specified sections, e.g. points of Gauss/Gauss-Lobato quadrature, to determine the element matrices of stiffness or flexibility. This formulation is thus divided into displacement-based elements, which compute stiffness by integrating the moment diagrams with linear curvature interpolation, and force-based elements, which compute flexibility by integrating the curvature diagrams with linear moment interpolation. To obtain accurate mathematical description, the former need to be discretised in multiple subelements, since curvature diagrams are nonlinear, but need no more than two Gauss points, since moment diagrams are nearly linear. On the contrary, the latter will need no subelements, but multiple Gauss points. Notes on these formulations, as well as related issues, can be found in Hellesland and Scordelis (1981), Neuenhofer and Filippou (1997), Scott and Fenves (2006) and Calabrese *et al.* (2010).

3. Case study

The case study chosen for the current endeavour is an existing five-story reinforced concrete building (see Fig. 1) located in Turkey. The building was selected from a set of previous studies on nonlinear static and dynamic analysis for the seismic assessment of torsional sensitive structures (e.g. Vuran 2007, Ba *et al*. 2008 and Bhatt and Bento 2011a, b).

The proposed building is asymmetric along the y-axis, and all floors have the same geometry and height (2.85 m). The columns sections keep the same geometrical and reinforcement features along the height of the building. The slabs are 0.10 and 0.12 m thick. Beam sections are mainly 0.20 x 0.50 m^2 except for the 0.20 x 0.60 m^2 located at the centre of the building. Column sections range from 0.25 x 0.50 m^2 to 0.25 x 0.75 m^2 and walls from 0.20 x 1.00 m^2 to 0.2 x 1.4 m^2. Confining stirrups are spaced at 20 cm in both beams and columns. For more structural details, see

(Vuran 2007, Bal *et al.* 2008).



(a) Plan view (b) Lateral view

Fig. 1 Existing five-story building structure (dimensions in m)

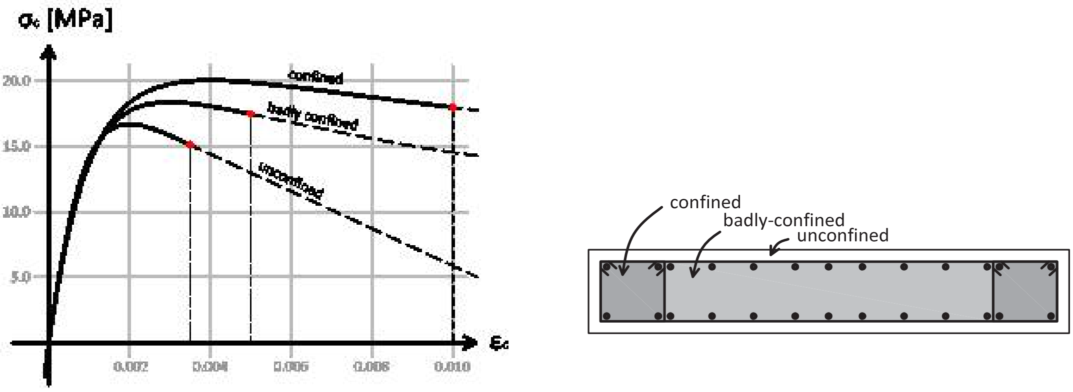


Fig. 2 Concrete capacity curves used. Red points represent conventional failure extensions

Table 1 Concrete parameters

	ε_{c0}	f_{c0} [kPa]	k_c	f_{cc}[kPa]	ε_{cu}
1	0.002	16700	1.0	16700	0.0035
2	0.002	16700	1.1	18370	0.0050
3	0.002	16700	1.2	20040	0.0100

Table 2 Steel parameters[1]

f_y[MPa]	371.0	$R(0)$	20.0
E_s[MPa]	200.0	b	0.005
ε_{su}	0.075	$a1$	18.5
		$a2$	0.15
		$a3$	0.025
		$a4$	2.0

[1]The steel model parameters presented are described in Menegotto and Pinto (1973).

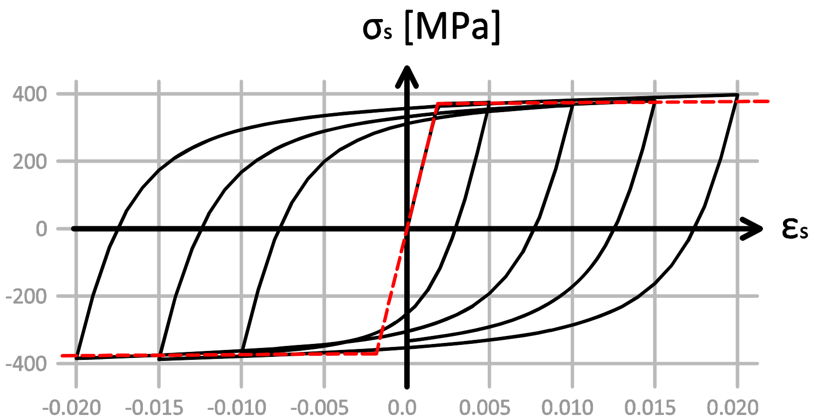


Fig. 3 Steel capacity curves

3.1 Material models

For materials definition, a mean concrete compressive strength of 16.7 MPa and a steel yield strength of 371 MPa were considered. The concrete was modelled using the proposal of Mander *et al.* (1988), with three straightforward confining ratios k_c= 1.0 (unconfined), 1.1 (badly confined) and 1.2 (confined), which assume a medium-low confinement exploration (see Fig. 2), since the accurate description of the real structure is not intended in this work. The factor k_{cis} defined as the ratio between the confined and the unconfined concrete compressive strengths, f_{cc} and f_{c0} respectively, where $f_{cc}= k_c f_{c0}$.

The strain ε_{c0}, corresponding to f_{c0}, was assumed 0.002 and three different ultimate strains ε_{cu} were considered as shown in Table 1.

For the concrete initial modulus of elasticity E_c, a value of 19.2 GPa was assigned also according to Mander *et al.* (1988) expressions.

The reinforcement steel was modelled with the Menegotto and Pinto (1973) relationship, with an elastic modulus E_s of 200 GPa and an ultimate strain ε_{su} of 0.075. Remaining parameters for the definition of the longitudinal reinforcement bars (Menegotto and Pinto 1973) are listed in Table 2. A cyclic response to a given strain history is presented in Fig. 3.

3.2 Section models

For the nonlinear definition of the cross sections, the two different alternatives mentioned were considered. The first one is a fibre discretization model applied to each element that automatically computes section stiffness at every stage, directly from material properties. The second is a direct definition of fixed moment-curvature relationships M-χ, for each axis of each cross section and for its average axial force (see Section 2.2). This moment-curvature relationships were obtained by a developed Matlab routine that uses a fibre model and determines a monotonic envelop based on initial stiffness, moment resistance and deformation energy (see Fig. 4). When used in the programs *SAP2000* and *SeismoStruct*, different hysteretic rules may be assigned. In this experiment, the kinematic hysteresis was applied.

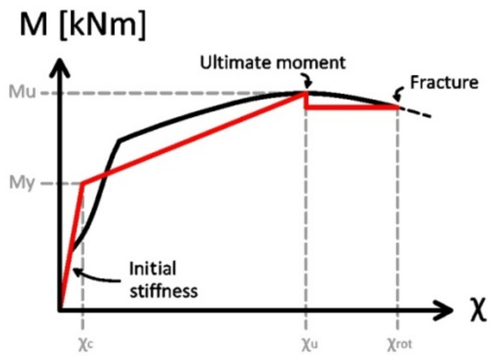


Fig. 4 *SAP2000* hinges for model (*a*.) (the idealized curve is shown in red)

3.3 Element models

For this case study, six three-dimensional models were considered (Fig. 5), each composed by a different finite element formulation, according to each program availabilities:
(a) *SAP2000* elastic element coupled with two frame hinge elements modelled with the externally computed moment-curvature relationships (Fig. 5(a));
(b) *SAP2000* elastic element coupled with two frame hinge elements, with the use of the fibre models of the cross sections (Fig. 5(b));
(c) *SeismoStruct* elastic element coupled with two nonlinear link elements modelled with the externally computed moment-curvature relationships (Fig. 5(c));
(d) *SeismoStruct* plastic hinge elements (Fig. 5(d));
(e) *SeismoStruct* elastic element coupled with two distributed plasticity elements (Fig. 5(e));
(f) *SeismoStruct* distributed plasticity elements (Fig. 5(f)).

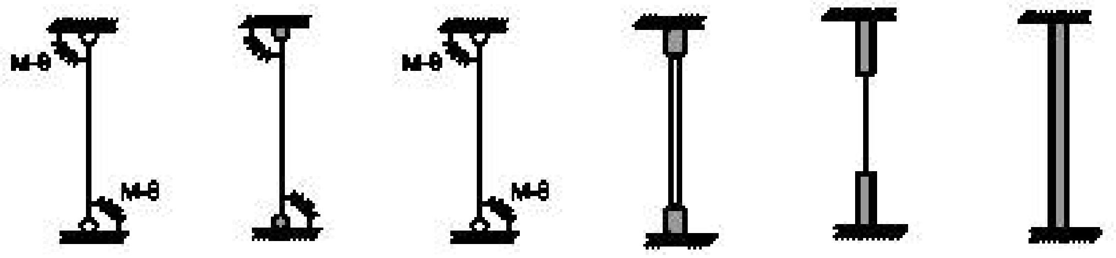
The element models a., b., c., d. and e., are models of concentrated plasticity and therefore a plastic hinge length dependent on a fixed ratio λ of the cross sections height H_s is used, i.e., $L_p = \lambda H_s$. The parameter λ was considered as 0.25, 0.5, 0.75, 1.0 and 1.25 for parametric study.

The model a. of *SAP2000* is described for beams by an association in series of an elastic element with one nonlinear rotational spring at each end. These springs are characterized by the fixed moment-curvature relation of the cross section, determined by the aforementioned Matlab routine with zero axial force. For the columns, two moment-curvature relations, one for each axis, were assigned to each end of the element for the axial force determined at each level. Curvatures are multiplied by the plastic hinge length to compute element stiffness. Kinematic hysteretic rules are applied.

The model of auto-computed fibre hinge elements of *SAP2000* (model b.) is similar to model a., except that each hinge is described by fibre models, being stiffness evaluated directly from material nonlinearity. In these models a single hinge at each end is enough to model biaxial bending. Materials' curves in *SAP2000* were defined manually by a number of points, with kinematic hysteresis. The same association of the hinge length is applied.

The model c. of *SeismoStruct* is equivalent to model a., being the link elements exactly defined as the plastic hinges of *SAP2000*.

The plastic hinge model of *SeismoStruct* (model d.) is a model proposed by Scott and Fenves (2006) that uses a Gauss-Lobato integration method, dividing the element into three regions: a



(a) (b) (c) (d) (e) (f)

Fig. 5 Nonlinear finite element models used

central region that remains elastic and two end regions where nonlinearity occurs. The integration is force-based and it is computed directly using the fibre model of the cross sections.

Models e. and f. of *SeismoStruct* are based on distributed plasticity elements with a displacement-based integration method. The formulation of these models assumes two integration Gauss points (element sections where equilibrium is established). Since nonlinear curvatures are not reasonably approached by a single linear interpolation, it needs discretization of the element itself. Thus, the elements of model f. were divided into three up to six distributed plasticity elements depending on their total length. Fibrerized cross-sections − representing sectional details such as cover and core concrete and longitudinal reinforcements − were then defined at the respective integration points, whereby every fibre was assigned to an appropriate material constitutive relationship, as described in Section 3.1. The sectional moment-curvature state of beam-column elements is obtained by integrating the nonlinear uniaxial stress-strain response of the individual fibres into which the section has been subdivided. Each section was defined with 300 fibres. In model e., the authors converted the central elements of the beams/columns of model f. into a linear element so as to limit nonlinearity progression through the element length. Its purpose was to study the influence of this limitation constituting a concentrated plasticity element to be compared with its equivalents, and thus with the same procedure regarding the plastic hinge length. The model e. was defined because the plastic hinge models c. and d. available in SeismoStruct were not able to operate successfully, having significant convergence problems. This model was 'created' by the authors as an alternative, since comparison of lumped plasticity models from different structural programs was one of the objectives.

It is also worth noticing that the rigid diaphragm effect of the slabs was taken into account in all models at each floor. This assumption, with the use of fibre elements, may lead to an artificial stiffening/strengthening of the beams, since they become prevented from deforming axially, thus interacting with the moment-curvature relationship (it is reminded that unrestrained RC elements subjected to flexure will deform axially, since the neutral axis is displaced from the center of gravity of the cross section). Being aware that this effect is actually present in real buildings, little is still known on how it can be modelled with a more realistic stiffness, without using shell elements. There are, however, experimental tests that have shown good results with using both rigid diaphragms and distributed plasticity element models (e.g. Bento *et al.* 2010) and, for its common use, it has been introduced on our models. Regarding the fixed M-χ relationship models, the diaphragm constraint will not interfere, by definition, with the flexural behaviour of the elements.

Finally, mass was linearly distributed along the beams and rotations at the base of the vertical elements were fully restricted.

4. Numerical analyses-description

To firstly study the dynamic characteristics and the linear response of the building structure, linear modal and response spectrum analyses were carried out. To validate each model, linear static analyses were also performed to check equilibrium of vertical gravity loads.

Pushover analyses were performed in each direction and for all nonlinear models with a lateral load distribution proportional to the relevant mode. With the distributed plasticity model, a uniform load was also applied, aiming to assess the structural response differences. To evaluate the influence of the plastic hinge length on the results, pushover analyses were also carried out to all variants employed. The N2 method proposed by Fajfar (2000 and 2005) and prescribed by EC8 (CEN 2010) was applied to the different capacity curves obtained by means of pushover analysis, aiming to evaluate the global structural response to a peak ground acceleration of 0.4 g. Only the value of λ equal to 0.75 was used to perform this procedure in the concentrated plasticity models (see Section 3.3).

Finally, nonlinear dynamic time-history analyses were carried out for all models (again only λ = 0.75 was adopted for the concentrated plasticity models) to the three intensities considered in this study (peak ground acceleration of 0.2 g, 0.3 g and 0.4 g). Regarding the seismic action definition, three real records from the PEER database (PEER 2010) were considered. The records were fitted to the EC8 (CEN 2010) elastic response spectrum, using the software RSPMatch2005 (Hancock *et al.* 2006) for all the seismic intensities. Each of the three semi-artificial pair of records was applied twice in the structure changing the direction of the components and thus forming a set of six analyses: NR1, NR2, TB1, TB2, WN1 and WN2.

5. Numerical analyses-results

The most relevant results obtained in this work are summarized in this section. It is important to emphasise that not all models were able to operate successfully, namely the two concentrated plasticity models in *SeismoStruct* (c. and d.). Thus, their results could not be shown in this paper. Convergence difficulties were also observed during the analyses on the models of *SAP2000*, especially on the fibre models that were recently introduced in the program. In both of the concentrated plasticity models used in *SAP2000*, convergence failure occurred at a given time step, which required reductions of the seismic intensity in order to complete analyses.

In general, a structure collapses when the plastic behaviour takes place in a sufficient number of sections to create a collapse mechanism. This is experienced, with most computer programs, by observing numerical convergence problems, which can either evidence equilibrium failure or software issues. In fact, element unloading methods are difficult to describe, and several convergence complications may lead to unreliable results, sensible to the type of model used, methods and thus to different software. For instance, *SeismoStruct* does not consider the loss of element strength when ultimate strains occur in materials.

Moreover, since users do not usually have complete access to the internal variables that govern numerical issues, when using commercial software, it becomes difficult to identify its cause, even with direct analysis of the output results. Therefore, only a rather superficial analysis is given to compare these different alternatives and to show which results can be expected from each one (although not encouraging generalization).

Finally, it is worth to mention that *SAP2000* program is able to show the evolution of the plastic hinges, when described with the frame hinge element property (model a.), during both non-linear static (pushover) and dynamic analyses. *SeismoStruct* does not have yet this capability available nor has SAP2000 with the fiber hinge models. A detailed analysis and comparison of damage spread at this level would bring substantial value to this study, adding to the global variables presented herein.

5.1 Dynamic properties

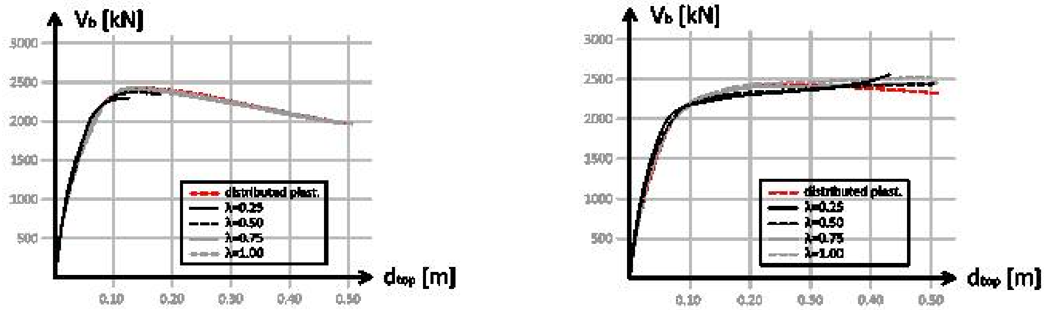
The first three modes of vibration were obtained by a linear model in SeismoStruct. The first mode (0.615 sec) is basically characterized by a global translation motion in the x direction, with a slight rotation about the z-axis due to the asymmetry of the structure along the x-axis. The second mode (0.592 sec), on the other hand, is described as a pure translational motion, derived from the symmetry along the y-axis, composed by four shear walls oriented in this direction. In the third mode (0.508 sec), almost pure torsional motion of the structure is observed. Thus the structure is classified as torsionally stiff.

5.2 Nonlinear static analysis

The capacity curves (top displacement d_{top} *vs* total base shear V_b) obtained with the distributed plasticity model in *SeismoStruct* (f.) are shown in Fig. 6, for modal and uniform lateral load distributions, and for x and y directions. The structure presents a slightly greater resistance along the y-axis, characterized by a hardening phase, whereas along the x-axis an evidenced softening behaviour is demonstrated. Regarding the two different load distributions, it is shown that the uniform load is distinguished by higher resisting forces and stiffness. This fact is due to the presence of stronger forces in superior levels in the modal distribution, which increases the values of the total shear force of each floor for the same base shear, leading to higher deformations, affecting resistance itself. Since *SeismoStruct* does not consider the loss of element strength when ultimate strains occur in materials, a (red) point was placed on the curves when ten *SeismoStruct* strain warnings are registered in different elements, after which the curve should not be taken as accurately representing the actual capacity. These strain warnings are given as a single fibre reaches its conventional strain limits (see Section 3.1, Tables 1 and 2) and they are particularly relevant since the program does not account for these limits.

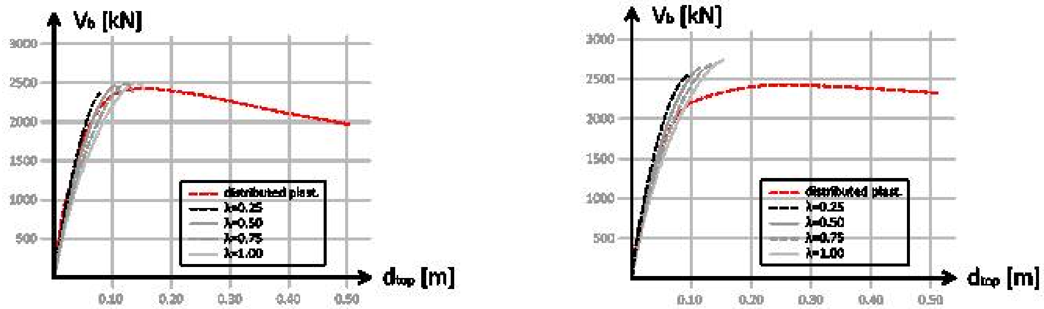
(a) X-direction (b) Y-direction

Fig. 6 Capacity curves obtained with *SeismoStruct* distributed plasticity model (f.)



(a) X-direction (b) Y-direction

Fig. 7 Capacity curves obtain with *SeismoStruct* limited distributed plasticity model (e.)



(a) X-direction (b) Y-direction

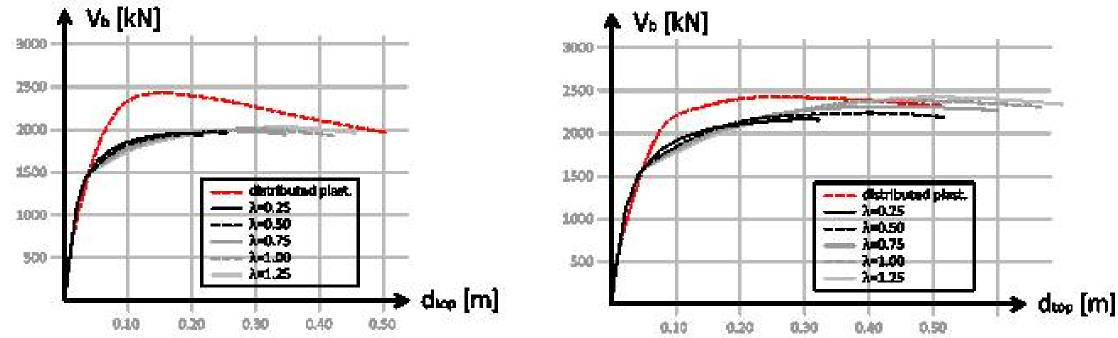
Fig. 8 Capacity curves obtained with *SAP2000* concentrated plasticity model with fibre models (b.)

In terms of the capacity curves obtained with limited distributed plasticity model in *SeismoStruct* (e.) for different values of the factor λ, the results were considerably close, even for very small values of λ, see Fig. 7. However as the plastic hinge length was reduced, more convergence difficulties were observed and more computational effort was required, therefore increasing significantly the duration of the analyses. In fact, when inelastic progress is limited by the length of the distributed plasticity elements, greater values of curvature are concentrated in the element critical sections, generating higher section internal forces. An increase of structural stiffness and a slight decrease of strength are then observed.

Fig. 8 shows the capacity curves obtained with the concentrated plasticity model in *SAP2000* using the fibre models of the cross sections (b.), for different values of λ.

Firstly, a considerably reduced maximum top displacement is verified, which is consistent with the red dots presented in Fig. 6, as ultimate strain values were directly described in the program materials definition. This formulation leads to great convergence difficulties as element forces abruptly drop to zero and stresses are constantly redistributed throughout the structure. The same result in stiffness is observed when λ is modified.

Different results were obtained for the capacity curves (see Fig. 9) when a hysteretic rule was imposed in *SAP2000* to model the behaviour of each section (a.). In both directions, a lower value of the maximum shear force is reached, and a considerably high value of ductility factor is demonstrated. The higher values of strength obtained with model f. could be a result of different effects, for instance: (1) different failure criteria adopted in *SAP2000* and *SeismoStruct* or (2) the effect of using distributed plasticity elements with rigid diaphragms – model f. As observed in the



(a) X-direction (b) Y-direction

Fig. 9 Capacity curves obtained with *SAP2000* concentrated plasticity model with hysteretic models (a.)

other models, the greater the plastic hinge length is defined, the higher deformation is achieved and the higher resistance is expected. In fact, when the plastic hinge length is increased, more rotation capacity is given to the hysteretic models and thus more deformation capacity is acquired by the structure. Occasionally, e.g. the y-axis in this case, as the structural displacements grow higher, resisting forces on the linear elastic elements increase, thus experiencing a greater value of V_b. Note that until yielding is reached, structural response is kept the same due to the rigid behaviour of plastic hinges.

The duration of the different pushover analyses are listed in Table 3. It is primarily seen that concentrated plasticity models with hysteretic rules in *SAP2000* (a.) were considerably faster. It is important to notice however that the durations displayed are highly dependent on the convergence difficulties of the models and thus the duration of model b. shall be considered cautiously as it goes less than 1/3 of the top displacement of the other models.

The target displacements determined by means of the N2 method are presented in Table 4. It is shown that with the concentrated plasticity model with fibre hinges in SAP2000 (b.) it was not possible to obtain the target displacement values, as the model seems not to have necessary ductility to reach the deformation imposed by the seismic action applied, which is evidenced by the clear drop of the curves before encountering convergence difficulties. For the remaining models, the target displacements were very similar, for both directions.

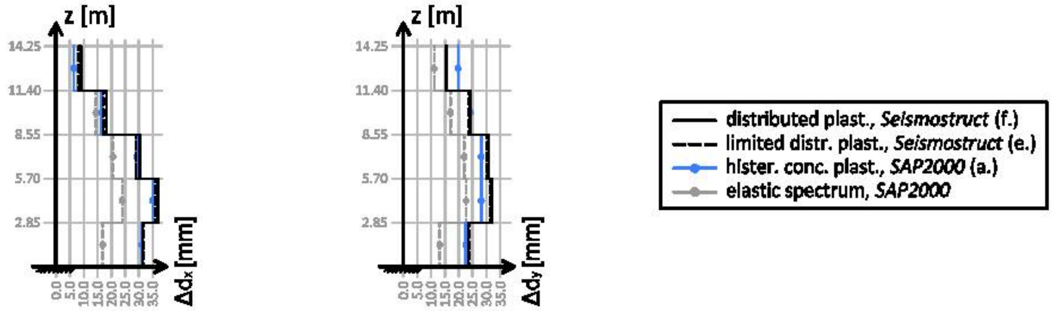
In Fig. 10, the interstory drifts obtained with this procedure show that models a., e. and f. conducted to considerably consistent results. Compared to the results obtained with the modal response spectrum analysis, it is concluded that, in this case study, nonlinear models led to a greater deformation, concentrated in the first three stories in the x-direction and more uniformly distributed along the height in the y-direction.

Table 3 Duration of the pushover analyses

	Modal load		Uniform load	
	x-dir	y-dir	x-dir	y-dir
a.	30 m	32 m	--	--
b.	1 h 24 m	1 h 05 m	--	--
e.	2 h 30 m	2 h 44 m	--	--
f.	1 h 30 m	1 h 21 m	1 h 01 m	1 h 12 m

Table 4 Roof target displacements (N2 method)

	Modal load	
	d_x [m]	d_y [m]
a.	0.118	0.123
b.	--	--
e.	0.122	0.125
f.	0.126	0.126

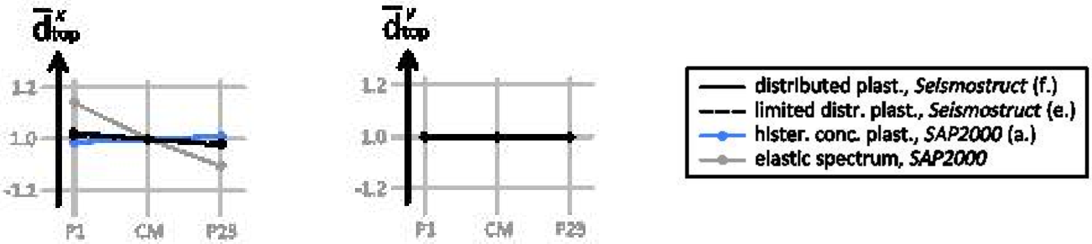


(a) X-direction (b) Y-direction

Fig. 10 Interstory drifts obtained with the N2 method and with the elastic response spectrum analysis

In fact, prevailing wall systems (y-direction) tend to form plastic hinges at the base of the walls due to the greater stiffness of the walls compared to beams, homogenizing the spread of story drifting in height. On the other hand, in a frame system (x-direction), the exceeding drift at the base floor caused by plastic excursion does not affect the upper floors.

Torsional effects are also compared in Fig. 11, where the top displacements of the corner frames P1 and P23 (Fig. 1) are normalized with respect to the roof centre of mass displacement in each direction. While in the y-direction, due to symmetry, no rotation is observed, in the x-axis normalized top displacements of the corner frames are verified, which are considerably smaller compared to the ones experienced in the elastic response spectrum analysis. These results are according to what was expected as the torsional effects are reduced in the structure when nonlinear behaviour excursions occur and the original version of the N2 method, herein used, does not take into account the torsional behaviour of the building.



(a) X-direction (b) Y-direction

Fig. 11 Normalized roof displacements obtained with the N2 method, and with the response spectrum analysis

5.3 Nonlinear dynamic analysis

The roof displacements obtained with the distributed plasticity model in *SeismoStruct* (f.), for the three values of the peak ground acceleration in the first combination of the Northridge record, are shown in Fig. 12. On the top of this graphic, as in the following ones (Figs. 13 and 14), a response period chart is displayed. This chart simply measures the time between maximums and minimums and multiplies them by two, i.e., it represents the response periods of the structure through time.

It is observed that increasing the seismic intensity, maximum roof displacements are near-proportionally increased, and the response periods slightly increased. The latter is caused by the greater nonlinear excursions and consequent loss of stiffness.

When compared to the equivalent linear behaviour (see Fig. 13), the difference between periods of vibration is more significant. It is also shown that maximum displacements are very similar, fact that was confirmed in all remaining records and intensities.

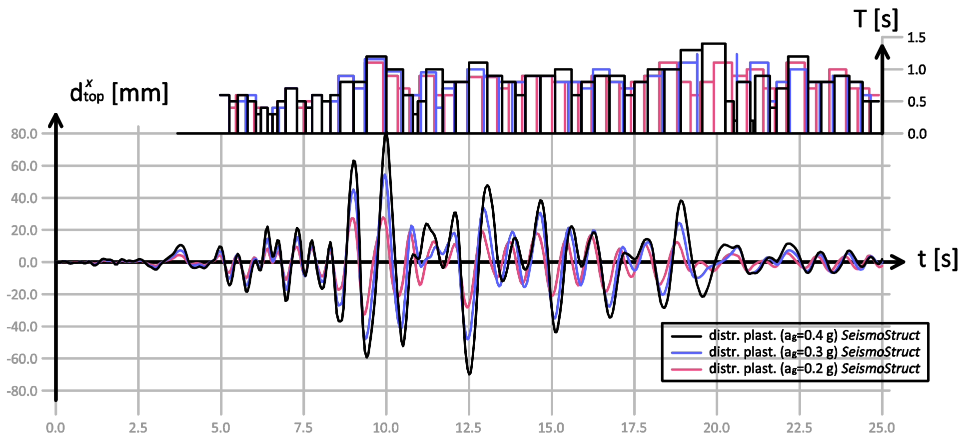


Fig. 12 Top displacements in the x-direction obtained with the distributed plasticity model (f.) in the first combination of the Northridge record, for different ground peak accelerations

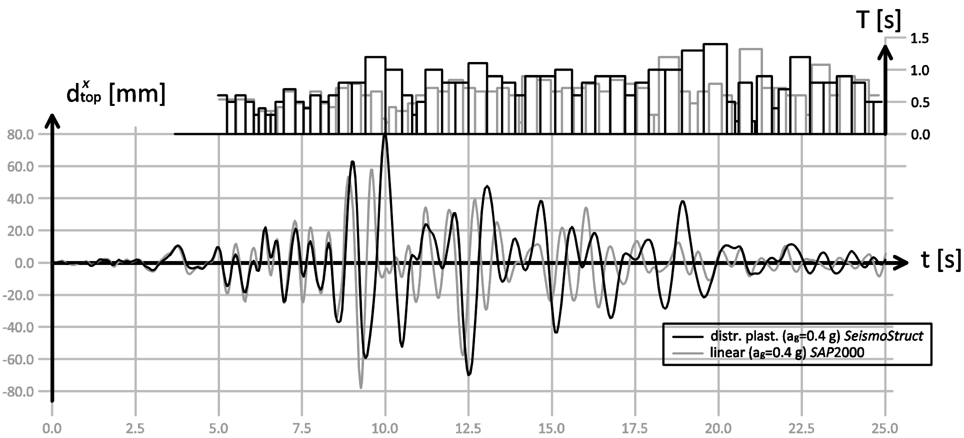


Fig. 13 Top displacements in the x-direction obtained with the distributed plasticity model (f.) in the first combination of the Northridge record, and with a linear dynamic time-history analysis

Regarding the limited distributed plasticity model built in *SeismoStruct* (e.), it is observed in Fig. 14 that no major difference was detected, as expected by the capacity curves shown in Fig. 7. A very small increase in periods of vibration and amplitudes is verified between the two curves.

To evaluate nonlinear excursions to which the structure is subjected for the different seismic intensities considered, the total base shear force was compared with and without considering inelastic behaviour. Fig.15 shows the values of base shear in the x-direction obtained with the distributed plasticity model in *SeismoStruct* (f.), and the corresponding values obtained with a linear dynamic time-history analysis for the different intensity levels. For this comparison, only the first seismic combination of the Tabas record is shown. Each pair of the shear forces is very close during the first 3sec, evidencing a linear elastic response of the structure. From that instant, the three curves representing the distributed plasticity models nearly follow the same course while linear elastic curves experience high peak values keeping the same proportion. Therefore, these results indicate that even though the seismic intensity is reduced by half, a strong nonlinear behaviour still affects the structure.

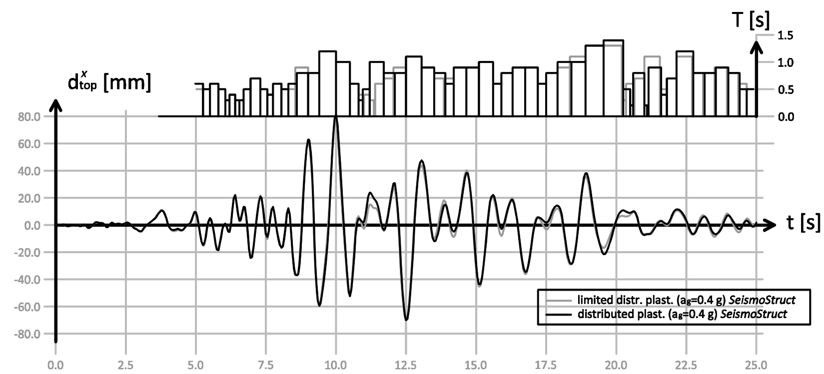


Fig. 14 Top displacements in the x-direction obtain with the limited distributed plasticity model (e.) and with the distributed plasticity model (f.) in the first combination of the Northridge record

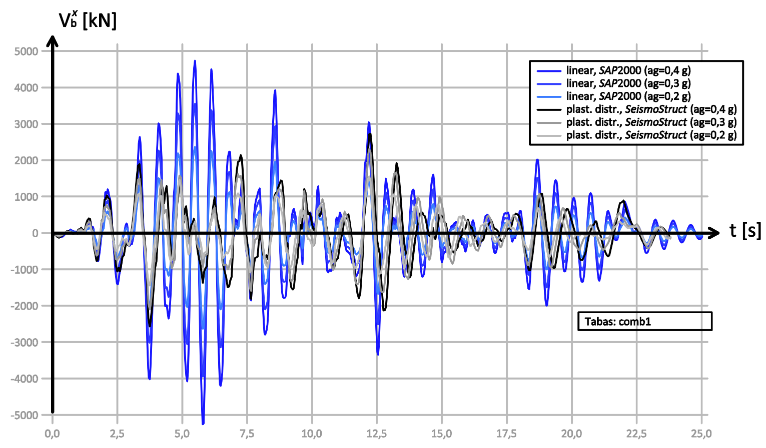
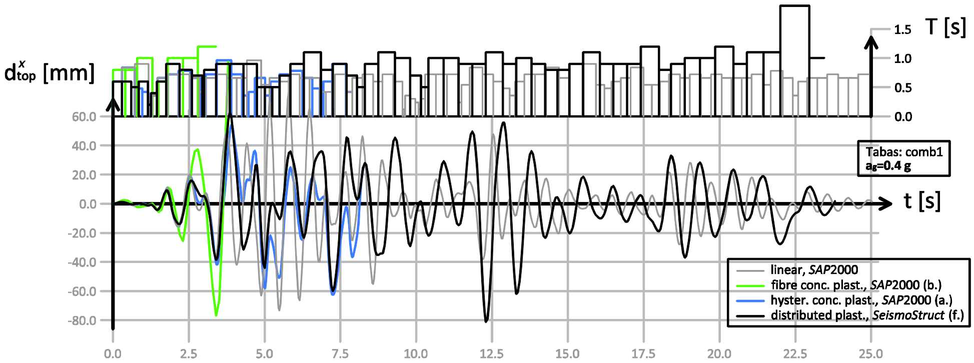
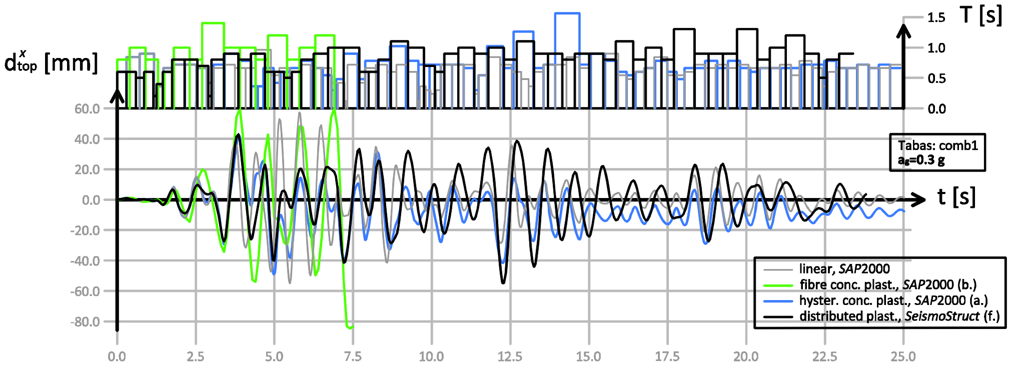


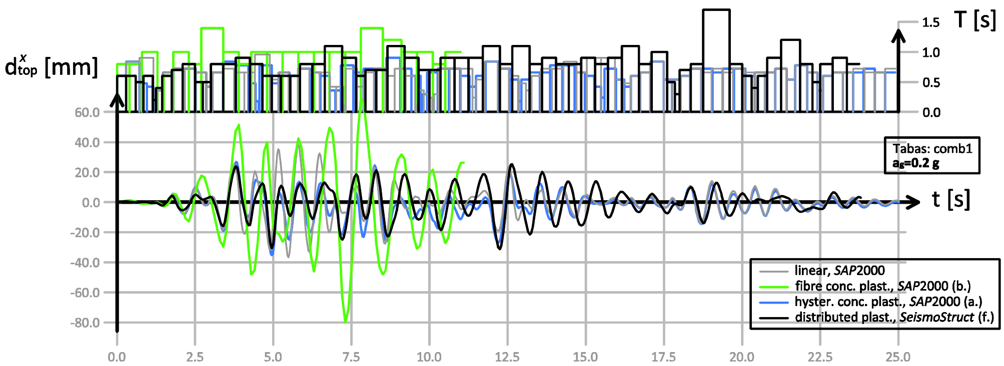
Fig. 15 Base shear force in the x-direction, obtained with the distributed plasticity model (f.) for the first combination of the Tabas record, and with a linear dynamic time-history analysis



(a) Peak ground acceleration of 0.4 g



(b) Peak ground acceleration of 0.3 g



(c) Peak ground acceleration of 0.2 g

Fig. 16 Top displacements in the x-direction obtained with different models (a., b. and e.), in the first combination of the Tabas record and with a linear dynamic time-history analysis

Such conclusion was very important to the analysis of the remaining concentrated plasticity models (a. and b.) in *SAP2000*, with which it was found great convergence difficulties. By knowing the aforementioned conclusion, the peak ground acceleration could be reduced and it could still be possible to compare models in the inelastic range. Fig. 16 shows the top displacement obtained with these two concentrated plasticity models compared with the distributed plasticity model (f.) and with the linear dynamic time-history analysis.

Table 5 Duration of the nonlinear dynamic analyses

	Ag [g]	NR1	NR2	TB1	TB2	WN1	WN2	Average
a.	0.2	55 m	50 m	1 h 05 m	1 h 10 m	09 m	1 h 16 m	54 m
	0.3	36 m	36 m	2 h 02 m	24 m	19 m	2 h 28 m	1 h 04 m
	0.4	54 m	2 h 20 m	1 h 29 m	26 m	14 m	50 m	1 h 02 m
b.	0.2	2 h 26 m	1 h 40 m	3 h 05 m	2 h 58 m	3 h 16 m	1 h 40 m	2 h 31 m
	0.3	1 h 06 m	2 h 03 m	33 m	30 m	1 h 16 m	1 h 20 m	1 h 08 m
	0.4	2 h 57 m	1 h 34 m	40 m	33 m	1 h 32 m	1 h 29 m	1 h 28 m
e.	0.4	5 h 31 m	7 h 38 m	6 h 16 m	6 h 07 m	7 h 10 m	6 h 38 m	6 h 33 m
f.	0.2	2 h 23 m	2 h 25 m	2 h 21 m	2 h 33 m	2 h 55 m	2 h 39 m	2 h 33 m
	0.3	2 h 37 m	2 h 33 m	2 h 35 m	2 h 50 m	3 h 17 m	2 h 25 m	2 h 43 m
	0.4	3 h 03 m	2 h 58 m	3 h 16 m	2 h43 m	1 h 13 m	1 h 15 m	2 h 25 m

It is concluded that the results obtained with models a. and f. are reasonably precise in the first steps of the analysis, particularly when Fig. 15 predicts higher nonlinear behaviour up to the first 7 sec. Model a., presents however lower periods of vibration. The deformability exhibited with the fibre concentrated plasticity model in *SAP2000* (b.) is considerably higher compared to the other models, which leads to higher values of displacement. It was not possible to detect a fair reason for this problem, although the capacity curves (Fig. 8) evidenced lack of ductility and consequently low energy dissipation capacity.

In Table 5, a list of the duration of each nonlinear dynamic analysis for the four models a., b., e. and f., with the three peak ground accelerations for the six semi-artificial accelerograms is presented. The last column of Table 5 depicts the average time duration for each model and for each peak ground acceleration considered in this work.

It is pointed again, that the durations of models a. and b. shall be examined carefully as the analyses could not be completed in some cases due to convergence difficulties.

6. Conclusions

The work developed in this study is able to evidence the difficulties, which may be found in using both nonlinear static and dynamic analyses. One can find in many cases unreliable results and its strong dependence on the plastic model used, in the analysis method chosen or in the structural program adopted.

In this research, during the process of creating each model and performing each analysis, it was concluded that the versions of both programs used in this endeavour (*SAP2000* 2008, *SeismoStruct* 2010) have still several issues in terms of practical applicability in the design activity of real structures.

When modelling a building structure, each program has different limitations:

• In *SeismoStruct*, although considering nonlinear behaviour is almost automatic due to the section fibre models, building the complex model itself becomes heavy;

• In *SAP2000*, despite the case of defining the structure geometry by using its intuitive graphical interface, the nonlinear behaviour modelling requires the use of external applications to compute the large amount of hysteretic relationships (if the option of using the fibre models of the cross sections is not adopted)

As concentrated plasticity models in *SeismoStruct* (c. and d.) were not able to work in any type of analysis, their use should be cautious and it seems not advisable for building structures of equivalent or higher complexity, at the moment. The limited distributed plasticity model (e.) is also not recommended as its results were very similar to those obtained with the distributed plasticity models, with the additional convergence difficulties and duration increase in both nonlinear analyses performed. The concentrated plasticity model in *SAP2000*, with fibre models (b.), has also given convergence problems (in this version of *SAP2000*), as in the capacity curves definition it lead to a relatively low ductility factor and in the nonlinear dynamic analysis it showed convergence failure at early time steps, with relatively high displacements. Convergence difficulties could not be avoided reducing time steps, but arised instead from the fact that the elements itself could not withstand the imposed displacements. A careful analysis of single elements showed that materials were lead to the strain limits. This is actually cleared by the final drop in resistance presented in the capacity curves of Fig. 8(a).

Both the concentrated plasticity model of *SAP2000* (a.) and the distributed plasticity model of *SeismoStruct* (f.) have shown fast applicability (durations of analysis) and little convergence difficulties. The results obtained with these models were also comparable in the time-history analysis, although pushover has evidenced differences in the base shear force estimation. Although the time-consuming is reduced by half using model a., the nonlinear modelling is straightforward with model e.

What is finally evidenced in this study is that inexperienced users should not rely their studies on one single model nor on one single analysis. Reliable models will only exist when researchers can gather a considerable amount of real scale tests to confront with computer models. As an example of this type, there is the SPEAR building (Bhatt and Bento 2011a, 2011b), a real scale test that was confronted with analytical models, including the distributed plasticity models of *SeismoStruct*, with adequate comparison.

Regardless of nonlinear analysis being, beyond any doubt, a powerful tool for the seismic assessment and design of structures, its use in design engineering offices has not been widespread. Based on the authors' knowledge and on the set of issues herein presented related to such kind of analysis, it is recommended that designers avoid nonlinear analyses, unless they are well experienced in their use and adopt a strong critical stance towards the obtained results. In particular, the use of nonlinear dynamic analysis, which cannot attract design offices due to its time-consuming nature (long computation times) and complexity (when compared with nonlinear static analyses), namely in terms of record selection. In fact, this is a topic for which the scientific community has not yet found widely accepted and definitive answers.

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References

Bal, I., Crowley, H., Pinho, R. and Gulay, G. (2008), "Detailed assessment of structural characteristics of Turkish RC building stock for loss assessment models", *Soil Dyn. Earthq. Eng*., **28**(10-11), 914-932.

Bento, R., Bhatt, C. and Pinho, R. (2010), "Using nonlinear static procedures for seismic assessment of the 3D irregular SPEAR building", *Earthq. Struct*., **1**(2), 177-195.

Bhatt, C. and Bento, R. (2011a), "Assessing the seismic response of existing RC buildings using the extended N2 method", *B. Earthq. Eng*., **9**(4), 1183-1201.

Bhatt, C. and Bento, R. (2011b), "Extension of the CSM-FEMA440 to plan-asymmetric real building structures", *Earthq. Eng. Struct. D*., **40**(11), 1263-1282.

Calabrese, A., Almeida, J.P. and Pinho, R. (2010), "Numerical issues in distributed inelasticity modeling of RC frame elements for seismic analysis", *J. Struct. Eng*., **14**(S1), 38-68.

Carvalho, G. (2011), *Análise Sísmica de Edifícios de Betão Armado – Estudo de Alternativas de Modelação e Análise Não-Linear*, Master'sthesis, Instituto Superior Técnico, Universidade Técnica de Lisboa, Portugal.

CEN (2010), *Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings*, Brussels, Belgium.

Clough, R., Benuska, K. and Wilson, E. (1965), "Inelastic earthquake response of tall buildings", *Proceeding of Third World Conference on Earthquake Engineering*, New Zealand 11.

Fajfar, P. (2000), "A nonlinear analysis method for performance based seismic design", *Earthq. Spectra*, **16**(3), 573-592.

Fajfar, P., Marusic, D. and Perus, I. (2005), "The extension of the N2 method to asymmetric buildings", *Proc. of the 4th European Workshop on the Seismic Behaviour of Irregular and Complex Structures*, Thessaloniki, Greece.

Filippou, F.C., Popov, E.P. and Bertero, V.V. (1983), "Modelling of r/c joints under cyclic excitations", *ASCE J. Struct. Eng*., **109**(11), 2666-2684.

Giberson, M. (1967), *The response of nonlinear multi-story structures subjected to earthquake excitation*, Ph.D. thesis, California Institute of Technology.

Hancock, J., Watson-Lamprey, J., Abrahamson, N.A., Bommer, J.J., Markatis, A., McCoy, E. and Mendis, R. (2006), "An improved method of matching response spectra of recorded earthquake ground motion using wavelets", *J. Earthq. Eng*., **10**(1), 67-89.

Hellesland, J. and Scordelis, A. (1981), *Analysis of RC bridge columns under imposed deformations*, IABSE Colloquium, Delft, Netherlands.

Kent, D.C. and Park, R. (1973), "Cyclic load behaviour of reinforcing steel", *Strain J. British Soc. Strain Meas*., **9**(3), 98-103.

Mander, J., Priestley, M. and Park, R. (1988), "Theoretical stress-strain model for confined concrete", *J. Struct. Eng*., **114**(8), 1804-1826.

Martinez-Rueda, J. and Elnashai, A. (1997), "Confined concrete model under cyclic load", *Mater. Struct*., **30**(3), 139-147.

Menegotto, M. and Pinto, P. (1973), "Method of analysis for cyclically loaded RC plane frames including changes in geometry and non-elastic behaviour of elements under combined normal force and bending", *Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well Defined Repeated Loads, International Association for Bridge and Structural Engineering*, Zurich, Switzerland, 15-22.

Neuenhofer, A. and Filippou, F.C. (1997), "Evaluation of nonlinear frame finite element models", *J. Struct. Eng*., **123**(7), 958-966.

PEER (Pacific Earthquake Engineering Research Center) (2010), *Strong ground motion database*, http://peer.berkeley.edu.

Petrangeli, M., Pinto, P.E. and Ciampi, V. (1999), "Fiber element for cyclic bending and shear of RC structures", *J. Eng. Mech.-ASCE*, **125**(9), 994-1009.

Priestley, M.J.N. (2003), *Myths and fallacies in earthquake engineering (Revisited)*, Pavia : IUSS Press.
Ramberg, W. and Osgood, W.R. (1943), *Description of stress-strain curves by three parameters*, Technical Note 902.
SAP2000 (1995), *Analysis reference manual*, For SAP2000 R, ETABS R and SAFETM.CSI.
SAP2000 (2008), *v12.0.0 Advanced*, Computers and Structures, Inc.
Scott, B., Park, R. and Priestley, M. (1982), "Stress-strain behaviour of concrete confined by overlapping hoops at low and high stain rates", *J. Am. Concrete Inst*.
Scott, M.H. and Fenves, G.L. (2006), "Plastic hinge integration methods for force-based beam-column elements", *J. Struct. Eng.-ASCE*, **132**(2), 244-252.
SeismoStruct (2010), *v5.0.5. A computer program for static and dynamic nonlinear analysis of framed structures*, SeismoSoft, Ltd. Available from: www.seismosoft.com.
Sezen, H. and Chowdhury, T. (2009), *Hysteretic model for reinforced concrete columns including the effect of shear and axial load failure*, *J. Struct. Eng.-ASCE*, **135**(2), 139-146.
Stojadinovic, B. and Thewalt, C.R. (1996), "Energy balanced hysteresis models", *Eleventh World Conference on Earthquake Engineering*, Earthquake Engineering Research at Berkeley, College of Engineering, University of California at Berkeley.
Takayanagi, T. and Schnobrich, W. (1979), "Nonlinear analysis of coupled wall systems", *Earthq. Eng. Struct. D.*, **7**(1), 1-22.
Taucer, F.F., Spacone, E. and Filippou, F.C. (1991), "A fiber beam-column element for seismic response analysis of reinforced concrete structures", Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley.
Thompson, K.J. and Park, R. (1980), "Moment-curvature behaviour of cyclically loaded structural concrete members", *ICE Proceedings*.
Vuran, E. (2007), *Comparison of nonlinear static and dynamic analysis results for 3D dual structures*, Master's thesis, UniversitàdegliStudi di Pavia.

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