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NONLINEAR MODELS FOR THE SEISMIC ANALYSIS OF REINFORCED CONCRETE BUILDINGS – A PERFORMANCE EVALUATION OF DIFFERENT MODELS IN NONLINEAR STATIC AND DYNAMIC ANALYSIS

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Summary: Being fact that both old and recently designed building structures are well expected to exhibit a nonlinear seismic response when subjected to medium-high intensity earthquakes, only simple linear methods have effectively been used in design offices. Nowadays, however, as a result of the improvement of scientific knowledge, the number of real experiments and the exponential progress of computational skills, 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 designers' disposal for the seismic assessment of engineering structures. Therefore, it was intended with this study to use some of these models to an existing structure and evaluate their precision in nonlinear static and dynamic analyses, with the use of two advanced computer programs: *SAP2000* and *SeismoStruct*. The different models focus on the 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 requirement for modelling and analyses were also discussed.

1. INTRODUCTION

The response of an engineering structure to a seismic action is carried out by a number of mechanisms characterized by stiffness, resistance and ductility. With a balanced proportion between these characteristics, the structure should be able to: (1) control displacements; (2) avoid damage under low seismic intensities and withstand the remaining design actions; and (3) accommodate large displacements without early collapse, dissipate energy and control damage spread.

The structural designers have mostly relied the seismic design and assessment of structures on linear methods, which have recently been proved not to describe in a correct fashion the actual behaviour of irregular structures. 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 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 included in several research studies. The better understanding of material behaviour, the growing performance of the existing element models and the increasing development of computational skills may turn nonlinear analysis into a generalized tool. The current challenge lies on the development of simple modelling software and fast-and-easy nonlinear methods compatible with the designing activity and the possible lack of knowledge of design engineers on the nonlinear field.

It was intended with this work (Carvalho [1]) to initially conduct a survey of the existing nonlinear models and later to use some of these in two different computer programs for a specific case study. In this paper, a brief description of nonlinear models and analysis is made, with main focus on those that are featured in *SAP2000* [2] and *SeismoStruct* [3].

Six different three-dimensional models were built with these software applications, and the accuracy evaluation was performed using nonlinear static and dynamic analyses. This work aims to model only the flexural behaviour of structural frames, meaning that no shear failures affect the nonlinear response of the case study.

In fact, shear models are currently under development and its straightforward applicability to complex systems is still far from being recommended. Both 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 conclude about the efficiency of each alternative.

With the accomplished study, for the seismic assessment of reinforced concrete buildings, final conclusions were given to support the future users on the choice among the selected possibilities.

2. NONLINEAR MODELS

In the recent capacity design standards, for the structure to absorb great deformation, maximum ductility is required. A number of ductile mechanisms are thus considered to accommodate these deformations, while brittle elements are carefully designed not to harm ductility, i.e., to remain linear.

One of the most ductile mechanisms is the flexural behaviour of the structural elements, which is generally made to prevail over shear fractures. These structural elements are consequently expected to form plastic hinges at their most critical sections, when deformed by severe earthquakes. The element sections are subjected to multiple excursions into the inelastic range, often followed by their gradual degradation caused by the cyclic motion.

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 section flexural behaviour it is often used either a definition of hysteretic rules or a fibre discretization model, where a uniaxial model of each fibber material is required.

2.1. SECTIONS

The use of hysteretic relations represents a relatively simple way to model the flexural behaviour of an element cross section, i.e. a numerical relationship between moment and curvature M-χ. This kind of formulation is generally based on the definition of a monotonic envelope and a set of rules for the definition of hysteretic loops (Stojadinovic & Thewalt [4]), which are calibrated to assess element stiffness, ductility and energy dissipation. Usually when a moment-curvature envelope is assigned to model the flexural behaviour of a structural element, the two directions of flexure 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 a linear analysis to gravity loads. Another simple model of the flexural behaviour of a cross section is the fibre model (see Taucer et al. [5]). This model can take into account both directions of flexure 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 with the tangent stiffness of each fibre material, its area of influence and its coordinates on the cross section.

There have been a number of recent research studies that also include the effect of shear in the flexural model (see, e.g., Sezen & Chowdhury [6] and Petrangeli et al. [7]). 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.

2.2. ELEMENTS

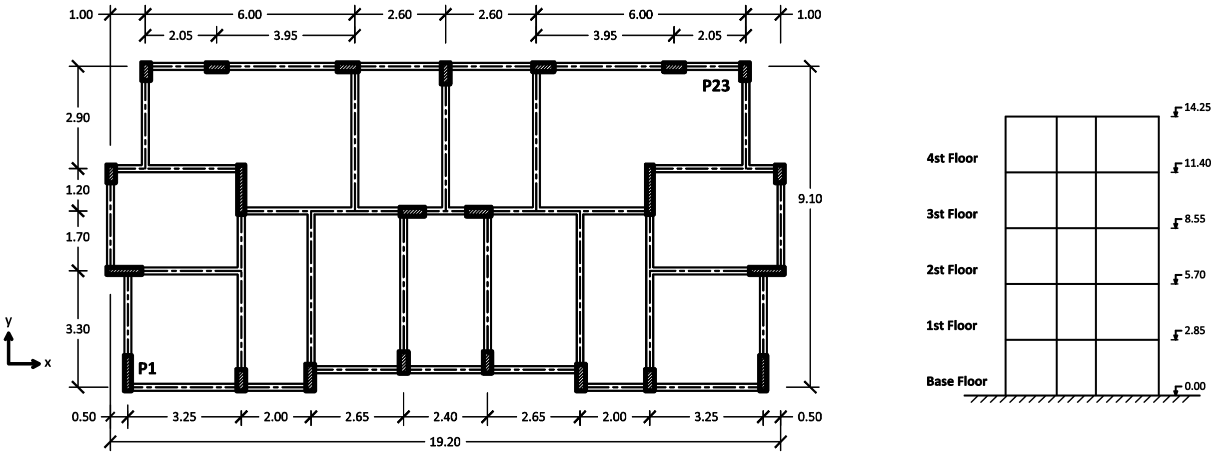
The structural elements of a building structure subjected to seismic excitations are mostly stressed at their connections with other elements. The internal forces distributions may be assumed as linearly varying, where major forces are located at both ends of the elements.

For this reason, the concentrated plasticity models have emerged by simply lumping flexural nonlinearity at these sections. The first formulation proposed by Clough et al. [8] consisted of an association in parallel of an elastic and an elastoplastic element. The most common formulation of a concentrated plasticity model was initially proposed by Giberson [9] with an association in series composed by a nonlinear rotational spring at each end of a linear elastic element. The rotational springs are defined by a moment-rotation relation M-χ 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 curvature.

In a distributed plasticity model, usually a finite number of cross sections are computed throughout the element to more accurately consider the inelasticity progression along its length. An old example of this formulation was suggested by Takayanagi & Schnobrich [10], by placing several hinges along the element with specified lengths, also suggested by the *SAP2000* reference manual [11]. However, the most generalized formulation is based on numerical integration of section quantities at specified sections, e.g. points of a Gauss or Gauss-Lobato quadrature, to determine the element matrices of stiffness or flexibility. Notes on these formulations, as well as related issues, can be found in Hellesland & Scordelis [12], Scott & Fenves [13], Neuenhofer & Filippou [14] and Calabrese et al. [15].

3. CASE STUDY

For the current study, it was chosen a structure of an existing five-story reinforced concrete building (see Figure 1) located in Turkey. The building was selected from a set of previous studies concerned with nonlinear static and dynamic analysis for the seismic assessment of torsional sensitive structures (e.g., Vuran [16] e Bhatt & Bento [17, 18]).



(a) plan view (b) lateral view

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

The proposed building structure is asymmetric along the x-axis, where all floors have the same geometry and same height (2.85 m), as well as the same element dimensions and reinforcement. The slabs are 0.10 and 0.12 m thick. Beam sections are mainly 0.20x0.50 m except for the 0.20x0.60 m located at the centre of the building. Column sections range from 0.25x0.50 m to 0.25x0.75 m and walls from 0.20x1.00 m to 0.2x1.4 m. Confining stirrups are spaced at 20 cm in both beams and columns. For more structural details, see [16].

3.1. ELEMENT MODELS

For this case study, six three-dimensional models were developed, each composed by a different finite element formulation, according to each program availabilities:

a. elastic element coupled with two frame hinge elements in *SAP2000*, modelled with specified hysteretic behaviour (Figure 2a);

b. elastic element coupled with two frame hinge elements in *SAP2000*, with fibre models of the cross sections (Figure 2b);

c. elastic element coupled with two nonlinear link elements in *SeismoStruct* (Figure 2c);

d. plastic hinge elements in *SeismoStruct* (Figure 2d);

e. elastic element coupled with two distributed plasticity elements in *SeismoStruct* (Figure 2e);

f. distributed plasticity elements in *SeismoStruct* (Figure 2f).

For models a., b., c., d., and e., a plastic hinge length dependent on a fixed ratio λ of the cross sections height H_s is used, *i.e.* $L_p=\lambda\ H_s$. Parameter λ was considered as 0.25, 0.50, 0.75, 1.00 and 1.25 for parametric study.

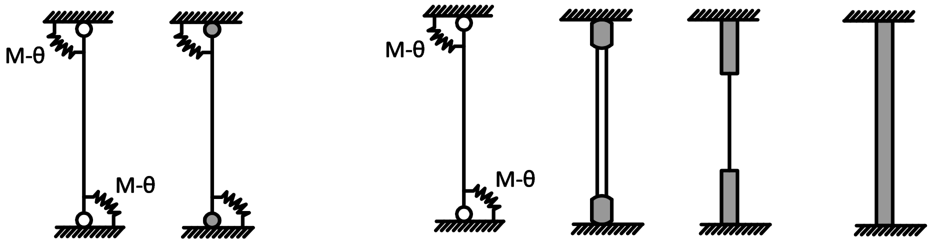


Figure 2: Nonlinear finite element models used.

The rigid diaphragm effect of the slabs was modelled at each floor; mass was linearly distributed along the beams; and rotations at the base of the vertical elements were fully restricted.

3.2. MATERIALS AND SECTIONS

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. [19], with three confining ratios k_c = 1.0, 1.1 and 1.2 (see Figure 3), which assume a medium-low confinement exploration [3]. The factor k_c is defined as the ratio between the confined and the unconfined concrete compressive strengths, f_{cc} and f_{c0}, where $f_{cc} = k_0\ f_{c0}$.

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 modulus of elasticity E_c, a value of 19.2 GPa was assigned.

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

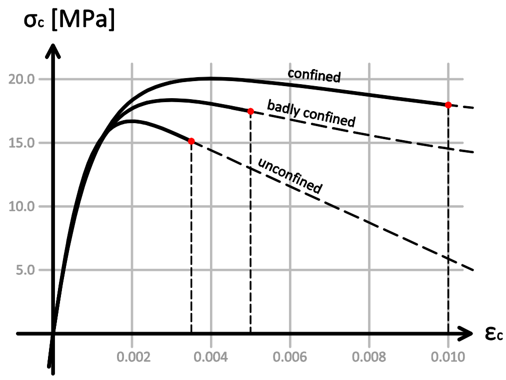


Figure 3: Concrete capacity curves used. Red points represent conventional failure extensions.

Reinforcement steel was modelled with the Menegotto & Pinto [20] equations, with an elastic modulus E_s of 200 GPa, and an ultimate strength ε_{su} of 0.075. Remaining parameters for the definition of the longitudinal reinforcement bars [20] are listed in Table 2. A cyclic response to a given strain history is presented in Figure 4.

Table 2: Steel parameters.

f_y [MPa]	371.0	$R_{(0)}$	20.0
E_s [MPa]	200.0	b	0.005
ε_{su}	0.075	a_1	18.5
		a_2	0.15
		a_3	0.025
		a_4	2.0

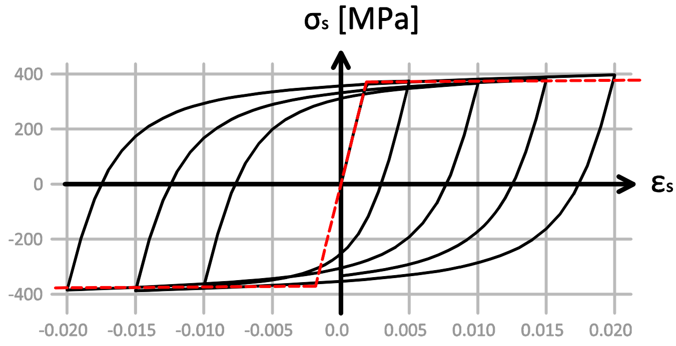


Figure 4: Steel capacity curves.

For the frame hinge elements in *SAP2000* (model a.), a monotonic envelop of the moment curvature relationship was determined by a fibre discretization model of each cross section, with the respective axial forces. This step was done with the development of a Matlab routine, with posterior idealizations to define yield moments M_y, and ultimate moments M_u and curvatures χ_u to import into *SAP2000* to each element direction of flexure (see Figure 5). The cyclic behaviour of these models is defined by the program with a bilinear hysteretic rule. The calculated curvatures were converted into spring rotations by multiplying the plastic hinge length L_p as mentioned before, using the λ-parameter.

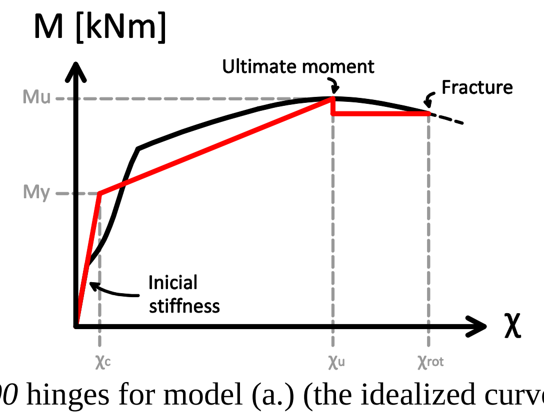


Figure 5: *SAP2000* hinges for model (a.) (the idealized curve is shown in red)

The model of auto-computed fibre hinge elements in *SAP2000* (model b.) is modelled with user-defined matrices of the fibres, where each one is assigned to a material model. To model both concrete and steel, the monotonic envelopes were introduced by a number of points. A bilinear hysteretic model is automatically assigned by the program. A single hinge accounts automatically for a bidirectional behaviour. The same L_p formulation is applied.

The link element models in *SeismoStruct* (model c.) were defined exactly as the frame hinge models in *SAP2000* (model a.). For the link element definition, a bilinear hysteretic rule is also available (among others). Models d., e. and f. in *SeismoStruct* are computed with the fibre discretization model of the cross sections. The program automatically defines section fibres, where materials are defined with the uniaxial models of Mander et al. [19] and Menegotto & Pinto [20], with the calibrated parameters of Tables 1 and 2.

4. PERFORMED ANALYSES

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, also linear static analyses were run to check equilibrium of vertical gravity loads.

A pushover analysis was performed in each direction in all nonlinear models with a modal proportional lateral load distribution. With the distributed plasticity model, also a uniform load was applied to evaluate structural response differences. To evaluate the influences of the plastic hinge length on the results, pushover analyses were also run to all variants employed.

The N2 method prescribed by the EC8 [21], (see Fajfar [22, 23]) was applied to the capacity curves obtained with each pushover analysis, to evaluate the global structural response to 0.4 g. Only the 0.75 value of λ was used to perform this procedure in the concentrated plasticity models.

Finally, nonlinear dynamic time-history analyses were carried out for all models (again only with $\lambda = 0.75$) to the three intensities considered in this study (0.2, 0.3 and 0.4 g). Regarding the seismic action definition, three real records from the PEER database [24] were considered. The records were fitted to the EC8 [21] elastic response spectrum, using the software RSPMatch2005 [25] for 0.4 g. 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.

Further details on the seismic action considered in this endeavour can be found in [1].

It is important to mention that not all models were able to operate successfully, namely the two concentrated plasticity models in *SeismoStruct* (c. and d.) that is why the results are not shown in this paper. Convergence difficulties were also verified during the analyses on the models of *SAP2000*, especially on the fibre models, which are still under development. In both of the concentrated plasticity models used in *SAP2000*, convergence failure occurred at a given time step, which required reduction of the seismic intensity in order to complete the analysis.

5. RESULTS

The most relevant results obtained in this work are summarized in this section.

5.1. DYNAMIC PROPERTIES

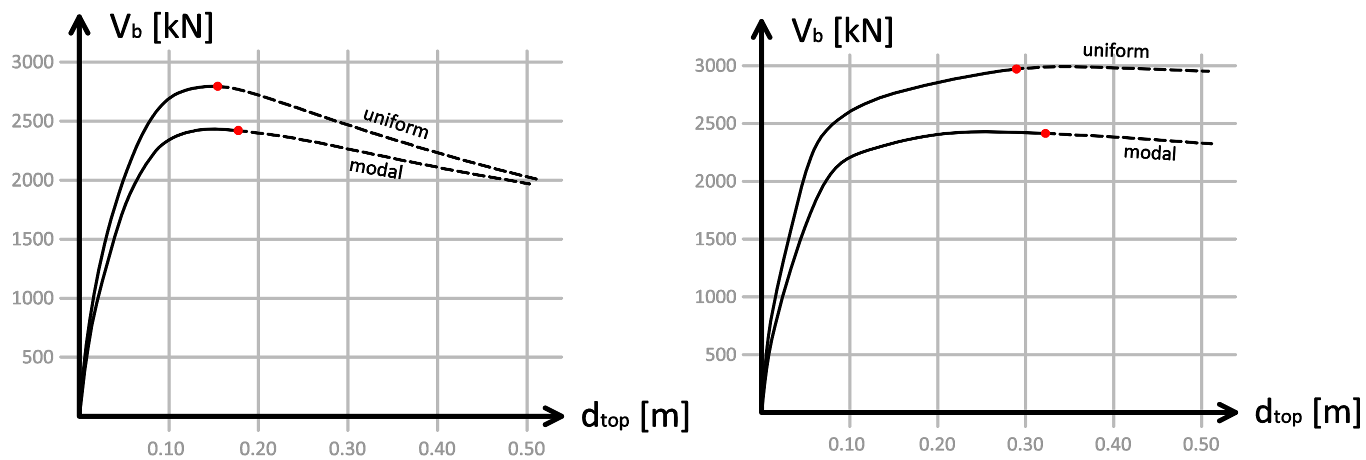
The first three modes of vibration were obtained by a linear model in *SeismoStruct*. The first mode (0.615sec) is basically characterized by a global translation motion of the stories along the x-axis, with a slight rotation about the z-axis, thus implying torsional sensitivity. The second mode (0.592sec), 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 that direction. In the third mode (0.508sec), almost pure torsional motion of the structure is verified.

5.2. NONLINEAR STATIC ANALYSIS

The capacity curves obtained with the distributed plasticity model in *SeismoStruct* (e.) are shown in Figure 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 is shown on the curves when ten *SeismoStruct* strain warnings are registered in the vertical elements, after which the curve shall not be taken as accurately representing the actual capacity curve.

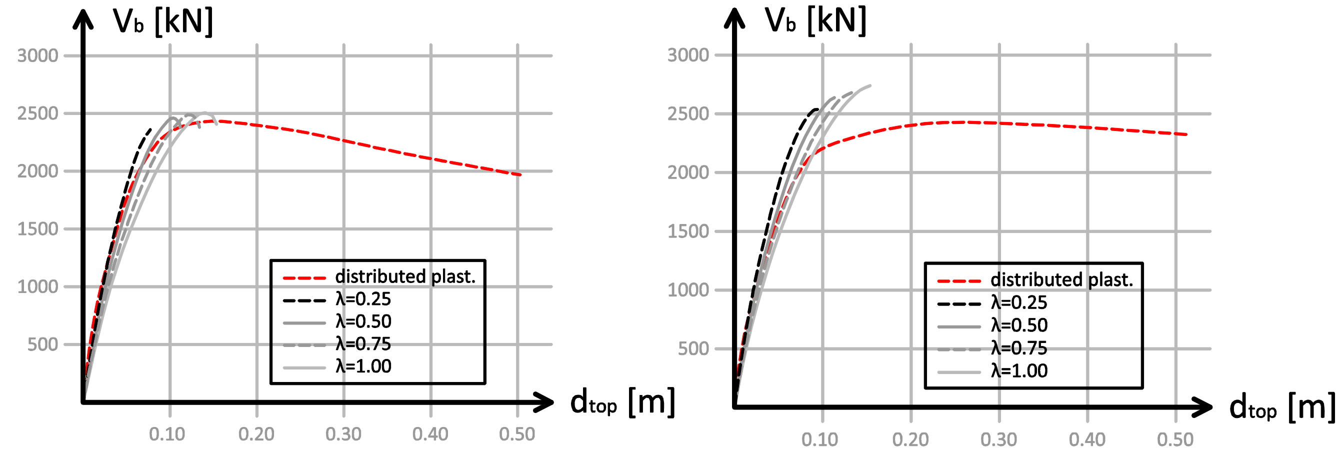
In terms of the capacity curves obtained with limited distributed plasticity model in *SeismoStruct* (f.) for different values of the factor λ, the results were considerably close, even for very small values of λ. However as the plastic hinge length is decreased, more convergence difficulties were verified and more computational effort was required, thus 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 forces. An increase of structural stiffness and a slight decrease of strength are therefore observed.

Regarding 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, see Figure 7.



(a) x-direction (b) y-direction

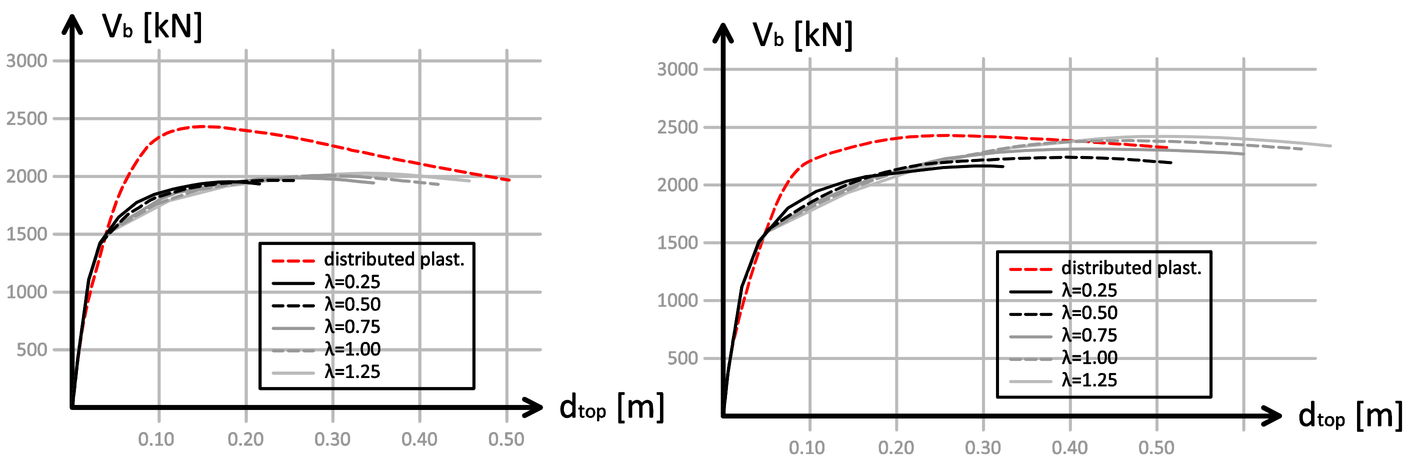
Figure 6: Capacity curves obtained with *SeismoStruct* distributed plasticity model (e.).



(a) x-direction (b) y-direction

Figure 7: Capacity curves obtained with *SAP2000* concentrated plasticity model with fibre models (b.).

Firstly, a considerable reduced maximum top displacement is verified, fact that is consistent with the red dots presented in Figure 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.



(a) x-direction (b) y-direction

Figure 8: Capacity curves obtain with *SAP2000* concentrated plasticity model with hysteretical models (a.).

When a hysteretic rule was imposed in *SAP2000* to model the behavior of each section (a.), different results were obtained with the capacity curves (see Figure 8). In both directions, a lower value of the maximum shear force is reached, and a considerably higher ductility factor is demonstrated. As seen with the other models, the greater the plastic hinge length is defined, more deformation is observed as well as 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 are increased, 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 pushover analyses are listed in Table 3. It is primarily seen that concentrated plasticity models in *SAP2000* (a.) were considerably faster.

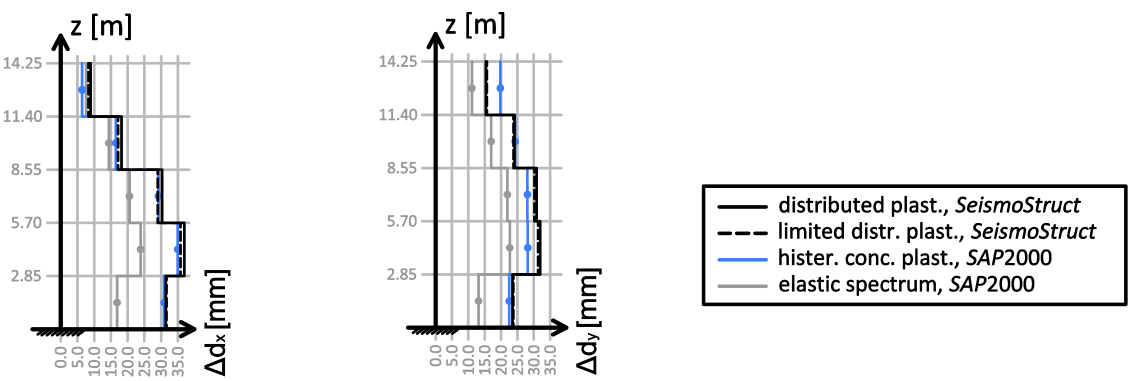
Table 3: Duration of the pushover analyses.

	Modal load		Uniform load	
	x-dir	y-dir	x-dir	y-dir
a.	30m	32m	--	--
b.	1h24m	1h05m	--	--
e.	2h30m	2h44m	--	--
f.	1h30m	1h21m	1h01m	1h12m

With the concentrated plasticity model with fibre hinges in *SAP2000* (b.) it was not possible to obtain a target displacement with the N2 method, as the model appears not to have necessary ductility to resist to the deformation imposed by the seismic action defined. For the remaining models, the target displacements were very similar, even in both directions.

In Figure 9, 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 seen that nonlinear models led to a greater deformation, concentrated in the first three stories in the x-direction and uniformly distributed through the stories in the y-direction.

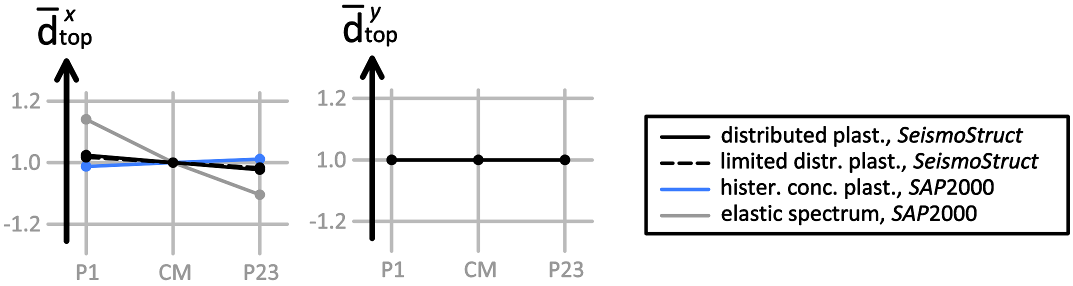
In fact, prevailing wall systems (y-direction) tend to form plastic hinges at the base of the walls, homogenizing the spread of story drifting in height, due to the greater stiffness of the walls compared to beams. On the contrary, in a frame system (x-direction), the exceeding drift of the base floor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



(a) x-direction (b) y-direction

Figure 9: Interstory drifts obtained with the N2 method and with the response spectrum analysis.

Torsional effects are also compared in Figure 10, where the top displacements of the corner frames P1 and P23 (Figure 1) are normalized with 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 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.



(a) x-direction (b) y-direction

Figure 10: 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* (e.), for the three values of the peak ground acceleration in the first combination of the Northridge record, are shown in Figure 11. Above the displacement was placed a chart where each bar indicates two times the duration between maximums and minimums, i.e., the response periods. It is seen that increasing the seismic intensity, maximum roof displacements are near-proportionally increased, as well as the response periods, the later being caused by the greater nonlinear excursions and consequent loss of stiffness.

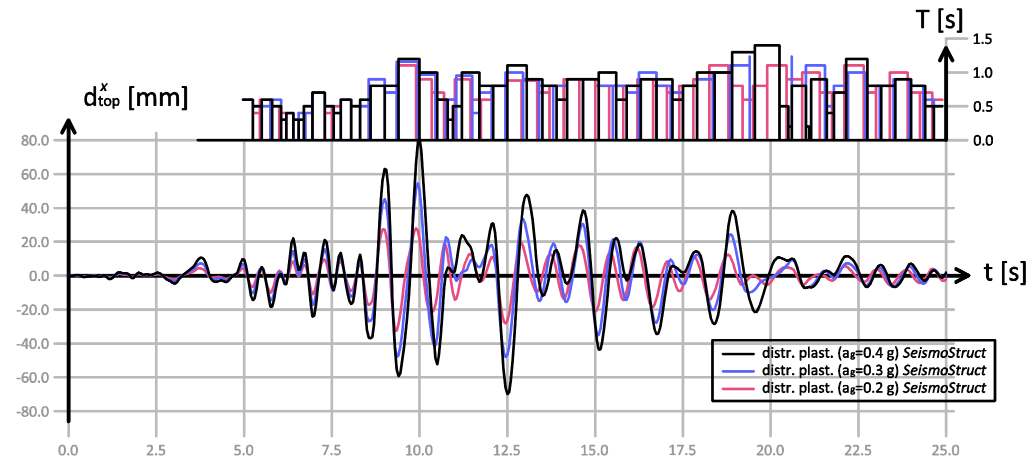


Figure 11: Top displacements in the x-direction obtained with the distributed plasticity model (e.) in the first combination of the Northridge record, for different ground peak accelerations.

Regarding the limited distributed plasticity model built in *SeismoStruct* (f.), it was concluded that no major difference was detected [1]. 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 in the different seismic intensities considered, the total base shear force was compared with and without considering inelastic behaviour. Figure 12 shows the values of base shear in the x-direction obtained with the distributed plasticity model in *SeismoStruct* (e.), and the correspondent values obtained with a linear dynamic time-history analysis. For this comparison, only the first combination of the Tabas record is represented. Each pair of the shear forces are very close during the first 3 sec, 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 peaks keeping the same proportion. This indicates that even though the seismic intensity is reduced by half, a strong nonlinear behaviour still affects the structure.

Such conclusion was very important to the analysis of the remaining concentrated plasticity models (a. and b.) in *SAP2000*, which had great convergence difficulties. By knowing this, the peak ground acceleration could be reduced and it could still be possible to compare models in the inelastic range, which was the initial purpose of this work (see Figure 13).

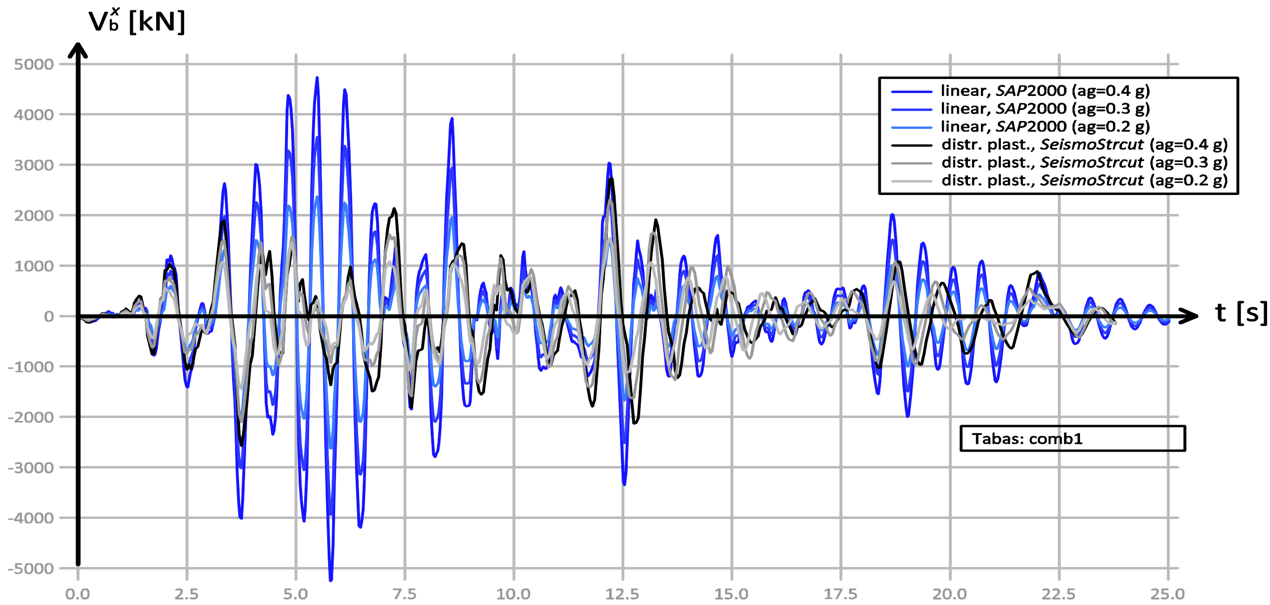
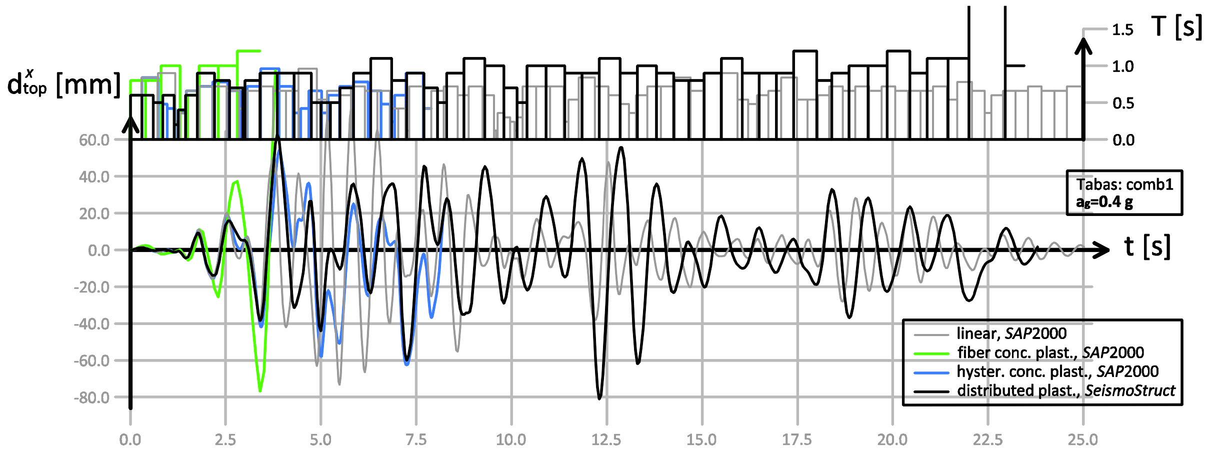
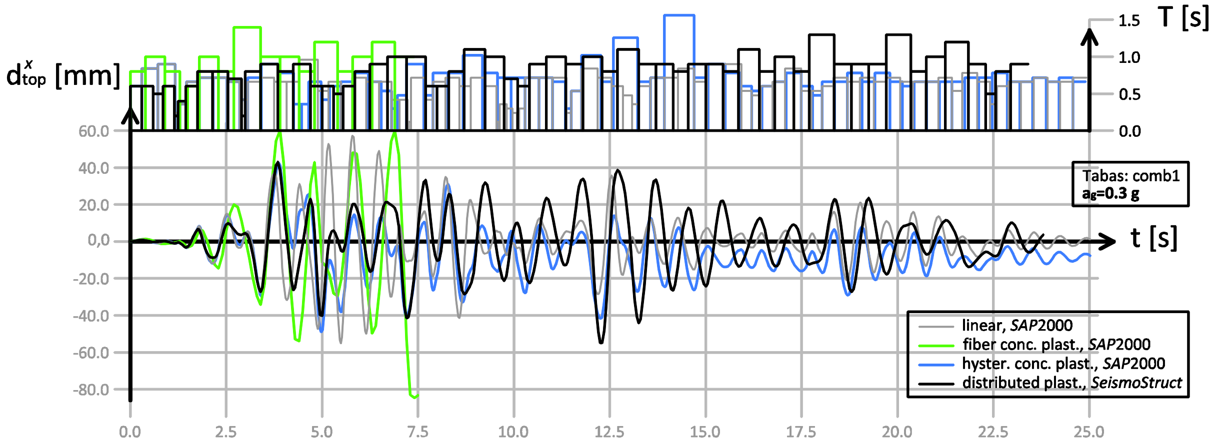


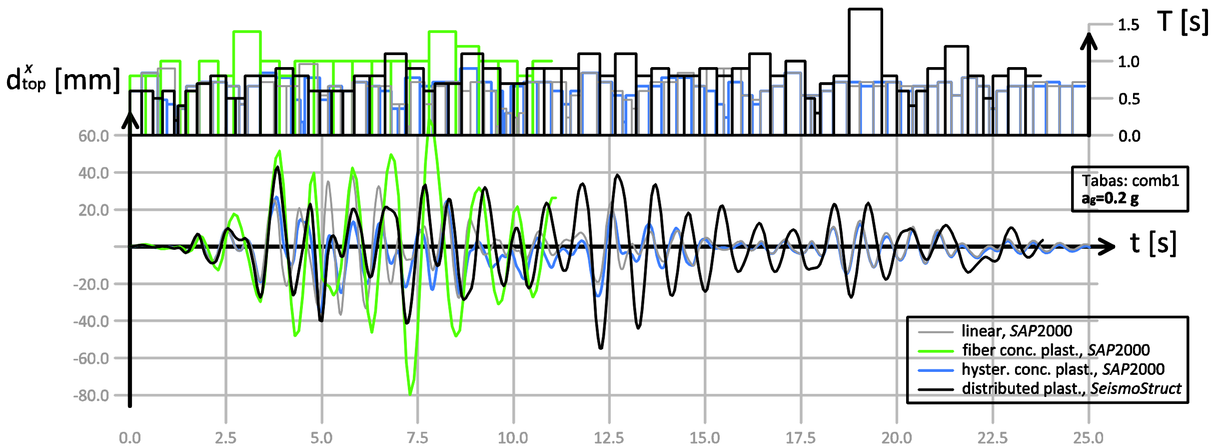
Figure 12: Base shear force in the x-direction, obtained with the distributed plasticity model (e.) in the first combination of the Tabas record, and with a linear dynamic time-history analysis.



(a) ground peak acceleration of 0.4 g



(b) ground peak acceleration of 0.3 g



(c) ground peak acceleration of 0.2 g

Figure 13: 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.

It is seen that the results obtained with the concentrated plasticity model with hysteretic rules (a.) are reasonably accurate in the first 10s of the analysis, when compared to the reference model (e.), however presenting lower vibrating periods. The deformability exhibited with the fiber concentrated plasticity model in *SAP2000* (b.) is considerably higher compared to the other models, which lead to higher values of deformation. It was not possible to detect a fair reason for this problem since it is not consistent with the high stiffness presented in the capacity curves (Figure 17).

In Table 4, a list of the duration of each nonlinear dynamic analysis for the four models a., b., e. and f., with the three ground peak accelerations for the six semi-artificial accelerograms is given.

Table 4: Duration of the nonlinear dynamic analyses.

	ag [g]	NR1	NR2	TB1	TB2	WN1	WN2
	0.2	55m	50m	1h05m	1h10m	09m	1h16m
a.	0.3	36m	36m	2h02m	24m	19m	2h28m
	0.4	54m	2h20m	1h29m	26m	14m	50m
	0.2	2h26m	1h40m	3h05m	2h58m	3h16m	1h40m
b.	0.3	1h06m	2h03m	33m	30m	1h16m	1h20m
	0.4	2h57m	1h34m	40m	33m	1h32m	1h29m
	0.2	2h23m	2h25m	2h21m	2h33m	2h55m	2h39m
e.	0.3	2h37m	2h33m	2h35m	2h50m	3h17m	2h25m
	0.4	3h03m	2h58m	3h16m	2h43m	1h13m	1h15m
f.	0.4	5h31m	7h38m	6h16m	6h07m	7h10m	6h38m

6. CONCLUSIONS

Throughout the entire work of building each model and performing each analysis, it was noticed that both programs still have 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* however, despite the ease of defining the structure geometry by using its intuitive graphical interface, when it comes to model nonlinear behaviour, the user has no other way but to resort to external applications to compute the large amount of hysteretic relations.

As concentrated plasticity models in *SeismoStruct* (c. and d.) were not able to work in any type of analysis, its use should be cautious and not advisable for building structures of equivalent or higher complexity, at the moment. The limited distributed plasticity model (f.) is also not recommended as its results were so similar to those obtained with the distributed plasticity models, with the additional convergence difficulties and duration increase both nonlinear analyses performed. The concentrated plasticity model in *SAP2000*, with fibre models (b.), has also proven not to be currently advisable (in this version of *SAP2000*), as in the capacity curves it presented a very low ductility behaviour and in the nonlinear dynamic analysis it showed convergence failure at early time steps, with extreme high displacements.

Only the results obtained with the concentrated plasticity model in *SAP2000*, with defined hysteretic models for the plastic hinges (a.), could be compared to the distributed plasticity model in *SeismoStruct* (e.). However, for high intensities of the seismic action, time-history analysis convergence failure occurred at early time steps.

Finally, the distributed plasticity model in *SeismoStruct* (e.), taken in this study as the reference model, constitutes one of the most trusted nonlinear models and its use is widely recommended. It has been tested in a significantly great amount of research studies, and some of them report comparison with experimental results (Bento et al. [26]). In fact, time-consuming is worth noticing in this model, and even though with the concentrated plasticity model of *SAP2000* (a.) it is reduced by half, nonlinear modelling in the former is straightforward.

A better knowledge on the seismic action and on the consequent structural response, and its accurate assessment and manipulation, will not only result in more reliable security conditions but especially in a better prediction of damage spread, thus reducing the repairing costs.

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REFERENCES

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

[2] SAP2000 (2008). v12.0.0 Advanced. Computers and Structures, Inc.

[3] SeismoStruct (2010). v5.0.5. A computer program for static and dynamic nonlinear analysis of framed structures. SeismoSoft, Ltd. Available from: www.seismosoft.com.

[4] Stojadinovic, B. & 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.

[5] Taucer, F. F., Spacone, E. & Filippou, F. C. (1991). A Fiber Beam-Column Element for Seismic Response Analysis of Reinforced Concrete Structures. Earthquake Engineering Research Center, Colege of Engineering, Uniersity of California, Berkeley.

[6] Sezen, H. & Chowdhury, T. (2009). Hysteretic model for reinforced concrete columns including the effect of shear and axial load failure. Journal of Structural Engineering, ASCE 135(2), 139-146.

[7] Petrangeli, M., Pinto, P. E. & Ciampi, V. (1999). Fiber element for cyclic bending and shear of rc structures. Journal of Engineering Mechanics, ASCE 125(9), 994-1009.

[8] Clough, R., Benuska, K. & Wilson, E. (1965). Inelastic earthquake response of tall buildings. Proceeding of Third World Conference on Earthquake Engineering, New Zealand 11.

[9] Giberson, M. (1967). The Response of Nonlinear Multi-Story Structures Subjected to Earthquake Excitation. Ph.D. thesis, California Institute of Technology.

[10] Takayanagi, T. & Schnobrich, W. (1979). Non linear analysis of coupled wall systems. Earthquake Engineering and Structural Dynamics 7(1), 1-22.
[11] SAP2000 (1995). Analysis Reference Manual. For SAP2000 R, ETABS R and SAFETM. CSI.
[12] Hellesland, J. & Scordelis, A. (1981). Analysis of RC Bridge Columns Under Imposed Deformations. IABSE Colloquium, Delft, Netherlands.
[13] Scott, M. H. & Fenves, G. L. (2006). Plastic hinge integration methods for force-based beam-column elements. Journal of Structural Engineering ASCE 132(2), 244-252.
[14] Neuenhofer, A. & Filippou, F. C. (1997). Evaluation of nonlinear frame finite element models. Journal of Structural Engineering 123(7), 958-966.
[15] Calabrese, A., Almeida, J. P. & Pinho, R. (2010). Numerical issues in distributed inelasticity modeling of rc frame elements for seismic analysis. Journal of Structural Engineering 14(S1), 38-68.
[16] Vuran, E. (2007). Comparison of Nonlinear Static and Dynamic Analysis Results for 3D Dual Structures. Master's thesis, Università degli Studi di Pavia.
[17] Bhatt, C. & Bento, R. (2010). Assessing the seismic response of existing rc buildings using the extended n2 method. Bulletin of Earthquake Engineering Published online DOI: 10.1007/s10518-011-9252-8.
[18] Bhatt, C. & Bento, R. (2010). Extension of the csm-fema440 to plan-asymmetric real building structures. Earthquake Engineering and Structural Dynamics Published online in Wiley Online Library (wileyonlinelibrary.com). DOI: 10.1002/eqe.1087.
[19] Mander, J., Priestley, M. & Park, R. (1988). Theoretical stress-strain model for confined concrete. Journal of Structural Engineering 114(8), 1804-1826.
[20] Menegotto, M. & Pinto, P. (1973). Method of analysis for cyclically loaded r.c. 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.
[21] CEN, Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, 2010.
[22] Fajfar, P. (2000). A nonlinear analysis method for performance based seismic design. Earthquake Spectra 16(3), 573-592.
[23] Fajfar, P., Marusic, D. & 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.
[24] PEER (2010). (Pacfuc Earthquake Engineering Research Center): Strong Ground Motion Database. http://peer.berkeley.edu.
[25] Hancock, J., Watson-Lamprey, J., Abrahamson, N. A., Bommer, J. J., Markatis, A., McCoy, E. & Mendis, R. (2006). An improved method of matching response spectra of recorded earthquake ground motion using wavelets. Journal of Earthquake Engineering 10(1), 67-89.
[26] Bento, R., Pinho, R. & Bhatt, C. (2008). Nonlinear Static Procedures for the Seismic Assessment of the 3D Irregular Spear Building.