Sensibility of Non Linear Time History Analysis of a RC Column Using Large Scale Shaking Table Results as Benchmark

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SUMMARY: (10 pt)
Time history analyses require the estimation and selection of a large number of parameters. This paper discusses the variability of the results obtained by this methodology using results of a shaking table test carried of a large RC column as benchmark. A set of sensitivity analyses of column models were carried using different modelling options and software. The influence of different variables including material nonlinear model, section model and damping models were evaluated. The column response in terms of displacement, shear forces, overturning moments, curvatures and residual displacements for these models were compared and were evaluated based on the reference results provided by the tested column. Preliminary results show that the selection of the variables and numerical approaches plays an important role and may lead to significantly different results.

Keywords: Nonlinear time history analyses, Blind prediction, Shaking table test, Sensitivity analyses

1. INTRODUCTION

Nonlinear Time history analysis is considered to be the most refined and advanced alternative for the seismic design and evaluation of reinforced concrete structures. To carry these analyses it is necessary to provide a significant amount of information including nonlinear properties of the materials or sections of concrete elements. It is also required to select among different numerical modelling alternatives such as elements based on force or displacements, damping model, damping level, element subdivision and number of integration points. Even if such variables can be easily introduced in several of the current software available, the reliability of the results obtained from the models is commonly unknown. According to seismic regulations, it is suggested to use several accelerograms to account for the uncertainty for the ground model; however, there is no indication to consider the uncertainty involved in the rest of the model variables.

Recently, several exercises have been carried out to obtain blind predictions of large scale structures tested on shaking tables in US and Japan. One particular case was a 1.2 m diameter, 7.3 m height column with a concentrated mass of 2250 kN at the top tested at the University of California San Diego. This column was densely instrumented and subjected to a set of accelerograms of different intensity. The numerical modelling of this test provides a unique opportunity to evaluate and to calibrate the models. The simplicity of the specimen gives an excellent opportunity to put a large effort on the evaluation of a reduced number of variables.

The blind prediction results revealed that, even if the specimen was considered to be a very simple case to model, there was a large dispersion of the predicted results. This fact indicates time history analysis method requires further understanding by both academic and practitioners to improve the reliability of the results obtained. A rational, careful selection of parameters and understanding of the variables is required to produce reliable results.

A series of nonlinear time history analyses of the 7.3 m column previously mentioned were carried out using different modelling approaches and variables including distributed and concentrated plasticity, force and displacement formulations, several levels of damping and material and element hysteretic models. Details about the models and the main finding are presented in the following sections. The models were analysed using different software including Ruaumoko (Carr, 2008), and Seismostruct (Seismosoft, 2012). Additional models using Opensees were also analysed but not included here due to space constrains.

2. SHAKING TABLE TESTS

Results obtained from a shaking table tests of a 7.3 m height and 1.2 m diameter RC column where used as benchmark to compare the different models analysed. The column had a 2250 kN lumped mass at the top and was reinforced with a total of 16 steel bars with a diameter of 35 mm (#11). Double hoops of 16 mm were placed along the column for confining the concrete core. The concrete strength ($f'c$) was 41 MPa and the steel yielding (f_y) and rupture strength (f_u) were 500 MPa and 689 MPa respectively. Figure 1 shows the specimen tested. The column was subjected to six different accelerograms, see Figure 2, representing from moderate to very high seismic intensities. The first record corresponds to a moderate intensity earthquake with PGA of 0.2 g, the second record corresponds to a high intensity earthquake with a PGA of 0.4 g and the third record a very high intensity record with a PGA of 0.52 g. Record four is a repetition of the second record and the sixth motion is a repetition of the third motion. The fifth record corresponds to a high intensity earthquake with high amplitudes in frequencies up to 2 s and a PGA of 0.45 g. Motions one to three have response spectra with a sharp drop past 0.8 s.

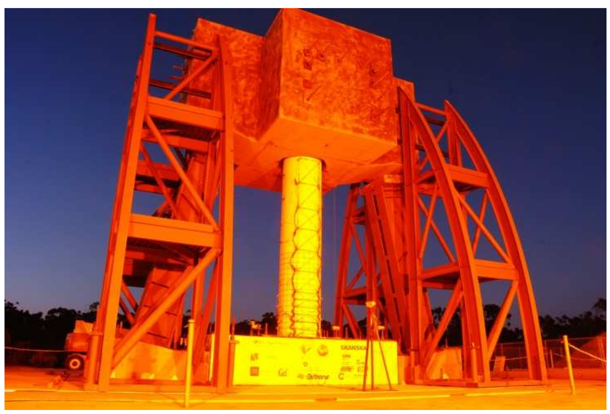

Figure 1. Column specimen tested (Jose Restrepo)

As part of the project a blind prediction contest was carried out with the participation of teams formed by academics and also teams formed by practitioners. Results provided by the different teams showed a large dispersion even for the most used design parameters as lateral displacement and horizontal accelerations. Further details about the specimen, testing program, earthquake motions and blind prediction contests can be found elsewhere (NISEE, 2011).

3. NUMERICAL MODELS

Among the several alternatives and variables available to define a nonlinear time history model there are those related to the mathematical formulation of the elements and the physical phenomenon and those related to the material, section and element properties. Table 2.1 shows the main variables that were considered for the analyses of the RC column.

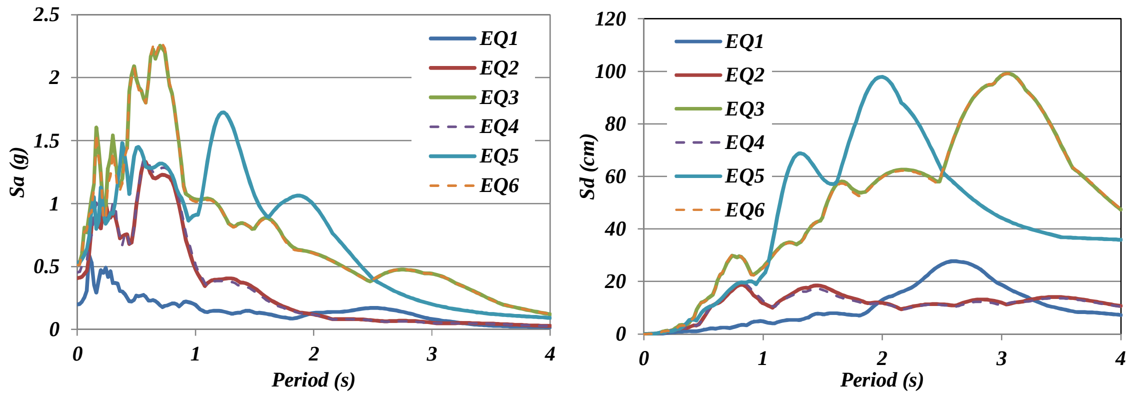

Figure 2. Ground motion spectra

In order to consider most of these parameters available software with different capabilities were used to carry out the analyses. Seismostruct (Seismosoft, 2011) was used for the fiber based, force based and displacement based formulations and material variations on the other hand, Ruaumoko (Carr, 2008) was used for the concentrated plasticity models with variations of the hysteretic models. The effects of damping model and damping level were evaluated in both programs. Table 2.2 and Table 2.3 show the list of the models analysed using the lumped plasticity and distributed plasticity models respectively.

Table 2.1. Nonlinear time history analyses variable definitions

General variable		Specific variable
Element mathematical formulation	Displacement based formulation	Integration points
	Force based formulation	Segment length
Section modelling	Fiber based modelling	Concrete model: Initial modulus of elasticity, tensile strength, strain at rupture, hysteretic model Steel model: Yield strength, strain hardening, hysteretic models. Fiber distribution
	Concentrated plasticity based on hysteretic models	Hysteresis model, element yield strength, element initial stiffness, loading and unloading stiffness.
Damping	Initial stiffness proportional models Secant stiffness proportional models	Damping coefficient
Others	Strain penetration, integration methods, and convergence criteria.	Characteristic parameters

Table 2.2. Lumped plasticity models and displacement based element formulation

Variable	Characteristics
Moment curvature envelope	Takeda Model (See table 2.4) Schoettler-Restrepo Model (See table 2.5)
Damping Models	Initial stiffness proportional, Tangent stiffness proportional and secant stiffness proportional damping models.
Damping levels	0.5%, 1% and 2%
Hysteresis model fatness (see Carr,2008)	α= 0.5 and 0.15 β= 0.0, 0.5 and 1.0
Plastic hinge length and number of segments	2 and 4 segments and plastic hinge length of 0.6 m, 0.8 m and 1.2 m.

Table 2.3. Distributed plasticity models

Variable	Characteristics
Number of integration points	4, 6 and 8 integration points
Damping models	Initial stiffness and tangent stiffness proportional damping
Damping level	0%, 0.5%,1%,2%and 3%
Concrete material model	Confined Mander (Mander et al.1988) and Madas-Elnashai (Madas & Elnashai, 1992) models
Steel material models	Bilinear, Menegotto-Pinto (Menegotto & Pinto,1973) with and without post elastic buckling.

The required parameters for the distributed plasticity models were obtained directly from the data provided by the contest organizers in combination with the models available in the software used. For the concentrated plasticity models it was necessary to compute additional variables required to characterize the column element. The hysteresis envelope were defined based on a section analysis of the column from which the parameters of initial stiffness (K_o), moment of inertia (I), yield moment (M_y) and post elastic stiffness coefficient (r) were obtained. The loading and unloading characteristics of the hysteresis cycles were defined on values of α and β commonly used for RC members. The characteristics for the Takeda models are shown in Table 2.4.

A second model based on the hysteresis loop proposed by Schoettler and Restrepo (Carr, 2008) was also defined from the same section analysis. This model has a trilinear envelope that allows modeling the uncracked state of the column. This function is useful to capture the structure response for the initial low intensity cycles applied to the specimen. In addition to the parameters used for the Takeda model, this hysteresis loops requires de estimation of the cracking moment (M_{cr}) and a stiffness ratio (ρ) obtained as the ratio between the secant stiffness to the yield moment and the initial uncracked stiffness.

Table 2.4. Takeda hysteresis loop parameters

K_o (kN-m/rad/m)	951520
r	0.0027
β	0.5
α	0.5
I	0.5 Ig
M_y (kN-m)	5500

Six different envelopes were defined to evaluate the effect on the computed response of the column. The first models (fit 1 and fit 2) evaluate the effect of the location of the cracking point. Two additional models were included to evaluate the effect of upper and lower values of the yield moment (M_y) and two more evaluate the effect of the post yielding stiffness (r). Table 2.5 shows the parameters used to define the different envelopes for the Schoettler-Restrepo hysteresis models.

Table 2.5. Schoettler-Restrepo hysteresis loop parameters

Parameter	*Fit 1*	*Fit 2*	*Upper* M_y	*Lower* M_y	*Lower* r	*Upper* r
I (m^4)	0.1084	0.1084	0.1084	0.1084	0.1084	0.1084
r	0.0013	0.0023	0.0023	0.0023	0.0015	0.003
M_y (kN-m)	5500.0	5500.0	6000.0	5000.0	5500.0	5500.0
M_{cr}/M_y	0.173	0.806	0.806	0.806	0.806	0.806
ρ	0.48	0.177	0.177	0.177	0.177	0.177

One additional parameter used to characterize the lumped plasticity model is the plastic hinge length (L_p) of the elements. An initial estimation was carried out using the recommendation by Paulay and Priestley (1992) as shown in Eq 1.

$$L_p = 0.08l + 0.022 d_b f_y \qquad (1)$$

Where l is the total column height, d_b is the diameter of the reinforcing bars and f_y the yield strength of the reinforcement. To analyse the effect of the plastic hinge, lumped plasticity models using values of L_p = 0.6 m, 0.8m and 1.2 m were analysed. The element length and node location were defined according to this plastic hinge lengths.

4. ANALYSIS RESULT

A total of 40 models considered in the analyses for a total of 240 results for which the top horizontal displacement (Δ) and acceleration (A), overturning moment (M), base shear (V) and curvature (ϕ) were compared with the experimental results provided. The results are analysed and plotted as the ratio between the analytical value (a) and the benchmark experimental value (b) for each of the six accelerograms. The results from the distributed plasticity models are analysed first followed by the results from the lumped plasticity models. Several plots do not include all the points for all the accelerograms as the computed ratio is out of the scale considered for the figure.

The first variable considered was the number of integration points. In these models a value of 0% was used for damping and the reference hysteretic materials for concrete and steel were Madas-Elnashai and Menegotto-Pinto respectively. Figure 3 show the ratios for models using 4, 6 and 8 integration points. These indicate that the use of only four integration points can provide similar results to models with larger number of integration points. In these cases, there is a tendency to overestimate all the parameters except for the cases of motions three and six which are the same record of a strong intensity earthquake in stiff soils. One constant characteristic of the results is the large overestimation of the curvatures at the base of the column except for the last two motions. This overestimation may not be of importance as this parameter is not very often used for design, however, there are some trends in seismic design in which strains in concrete and steel are being used as design parameters. This approach may turn out to be unreliable if used in conjunction with nonlinear time history analyses due to the fact that they are directly related to the section curvature.

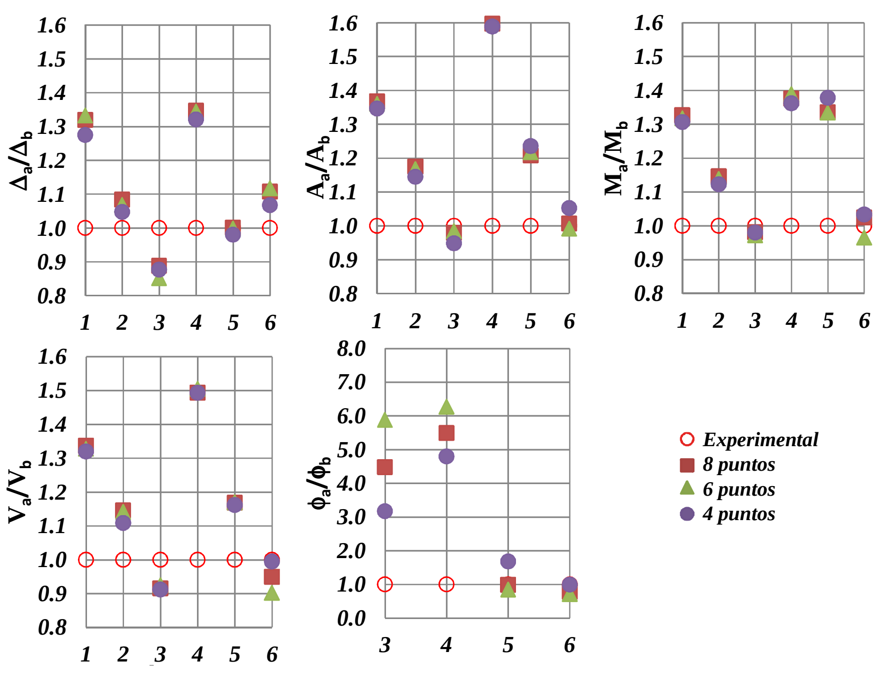

Figure 3. Relative results for models with different integration points

The effect of the introduction of different levels of tangent stiffness proportional damping has a larger effect than the previous variable. The results shown in Figure 4 indicate that the use of a damping level between 1% and 2% provide the best matches, in most cases within a range between ±10%, to the experimental results. For the specific case of the moderate intensity earthquake (motion 1) the use of a damping level close to 2% introduces a significant effect when compared to smaller damping levels. The effect of the damping level tends to be less significant as the intensity increases. There is also an important effect of previous motions as observed comparing motions two and four which are identical. Results for motion two have a very good match with the experimental results; however this is not the case when the same motion is applied a second time (motion four).

Contrary to this tendency, different levels of damping do not seem to have a significant effect when an initial stiffness proportional damping formulation is used in the analysis. Figure 5 shows that this damping formulation may introduce even larger error compared to the benchmark values than the previous damping formulation as observed in the specific cases of displacement for motion six and acceleration for motion four. The trend of a good match for the second motion and a large overestimation for the fourth motion is also present in this case. These results are coherent with recommendations from other authors (Charney, 2008, Grant el al, 2005) which tend to favor the use of tangent stiffness proportional damping to the initial stiffness proportional formulation.

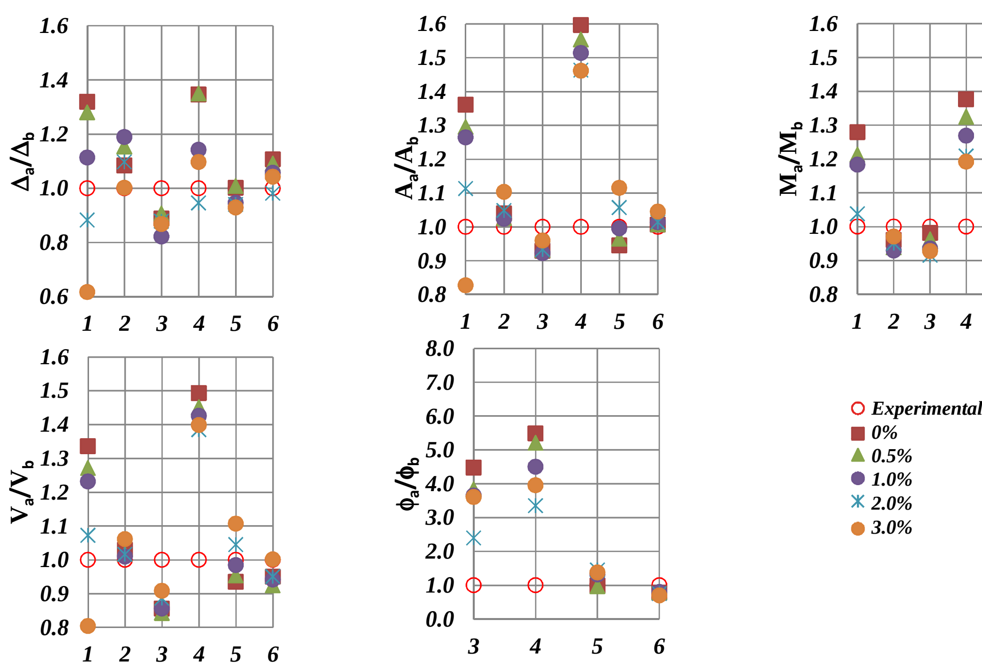

Figure 4. Comparative results for models with different damping levels for a tangent stiffness proportional damping formulation

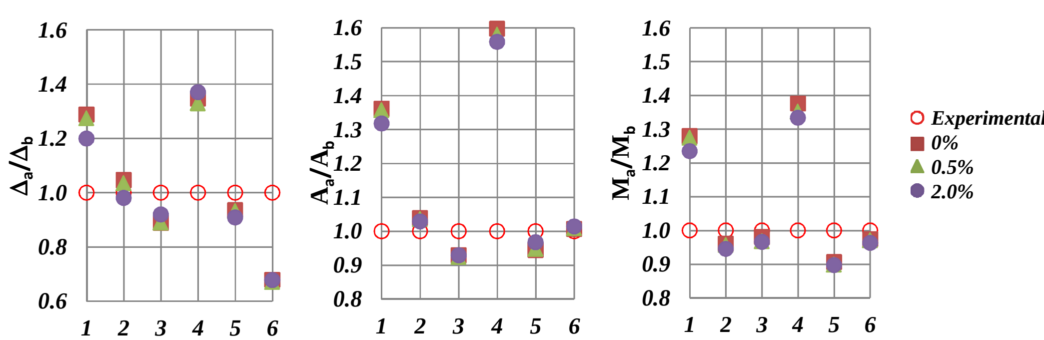

Figure 5. Comparative results for models with different damping levels for an initial stiffness proportional damping formulation

The following set of parameters evaluated were the material models used for steel and concrete for which the compressive strength and strains at maximum strength were kept constant. Figure 6 and Figure 7 show the results of the comparison for these cases. It is clear that there is practically no effect due to the variation of the concrete model even in the case of Madas-Elnashai confined concrete for which the cover concrete was also simulated with this constitutive model using parameters of a very slight confined concrete. This finding can be of practical use given that it is common to have convergence problems when modelling the cover concrete with constitutive models with non-smooth envelopes and sharp strength drops. On the other hand, the steel constitutive model does play a significant role and, in several of the evaluated cases, it introduces large differences in the results of the nonlinear analyses. The use the Menegotto-Pinto model with buckling does not seem to be a good alternative for large or very large intensity motions. Surprisingly the simple bilinear model provides in most cases estimates with slightly larger errors than the case of a more refined models as the case of Menegotto-Pinto without buckling.

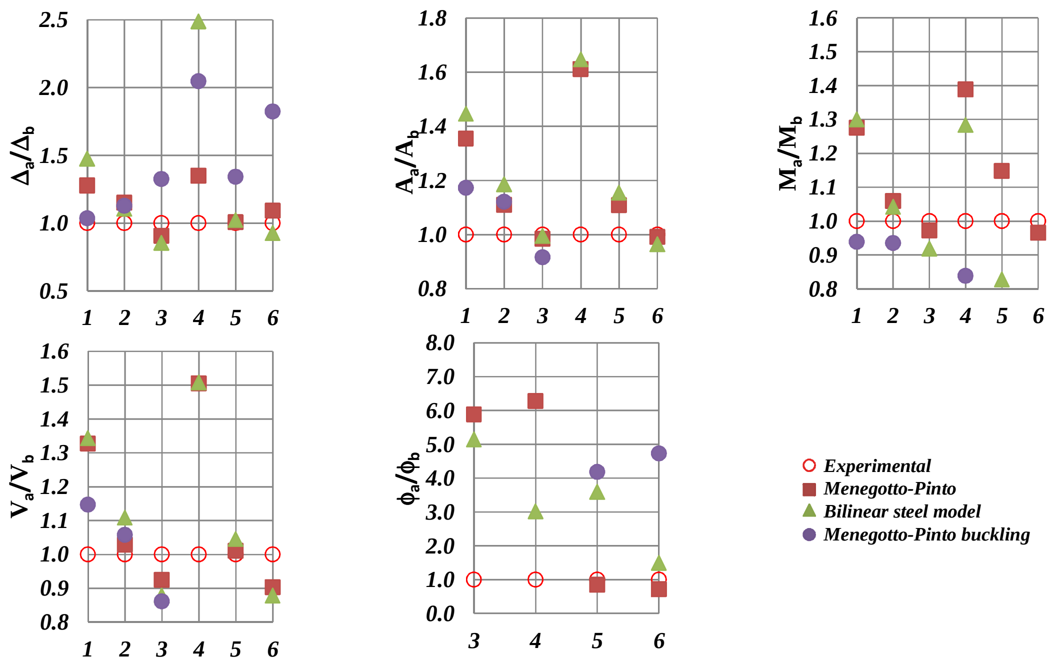

Figure 6. Comparative results for models with different steel models

The second group of analyses carried out using lumped plasticity models evaluated the effects of different envelope alternatives, damping levels, plastic hinge length and damping formulations. The first variable evaluated was the moment curvature nonlinear envelope of the critical section of the column. Figure 8 shows the results for the different envelopes analysed using a tangent stiffness proportional damping of 0.5%. A total of four elements were used to model the column with a plastic hinge length of 0.6 m. There is no a single model that has the best match to the experimental results in all the cases. There is a large dispersion of the results and for most of the cases the response was over predicted with exception of the overturning moments. The over prediction is even more clear for the lateral acceleration of the top node where the mass is located.

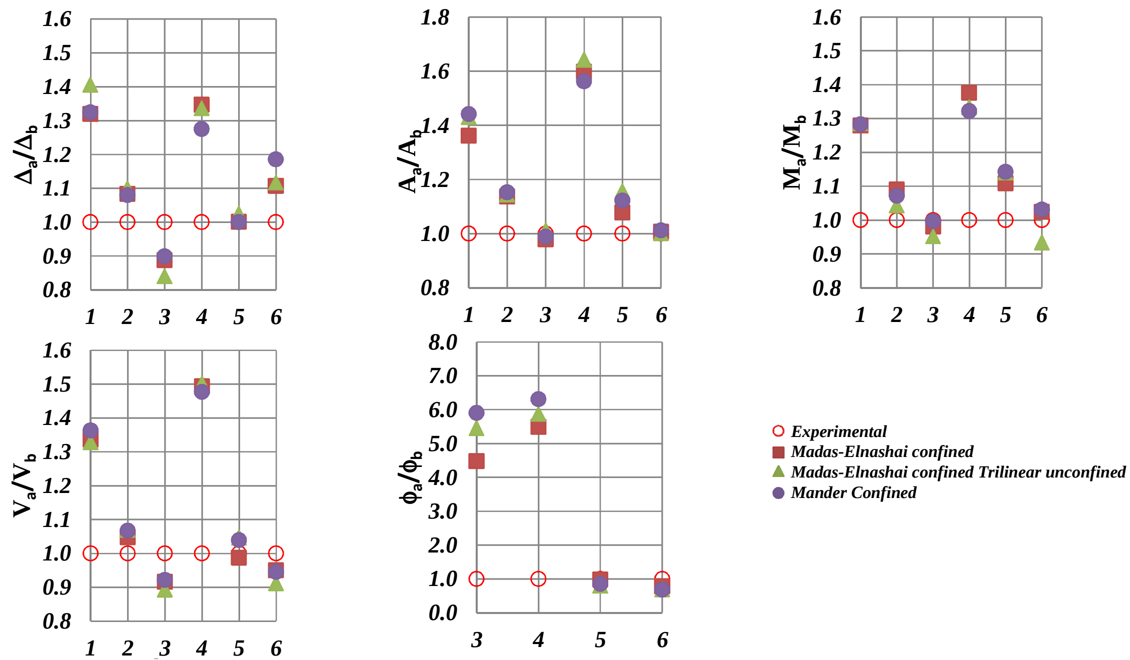

Figure 7. Comparative results for models with different concrete models

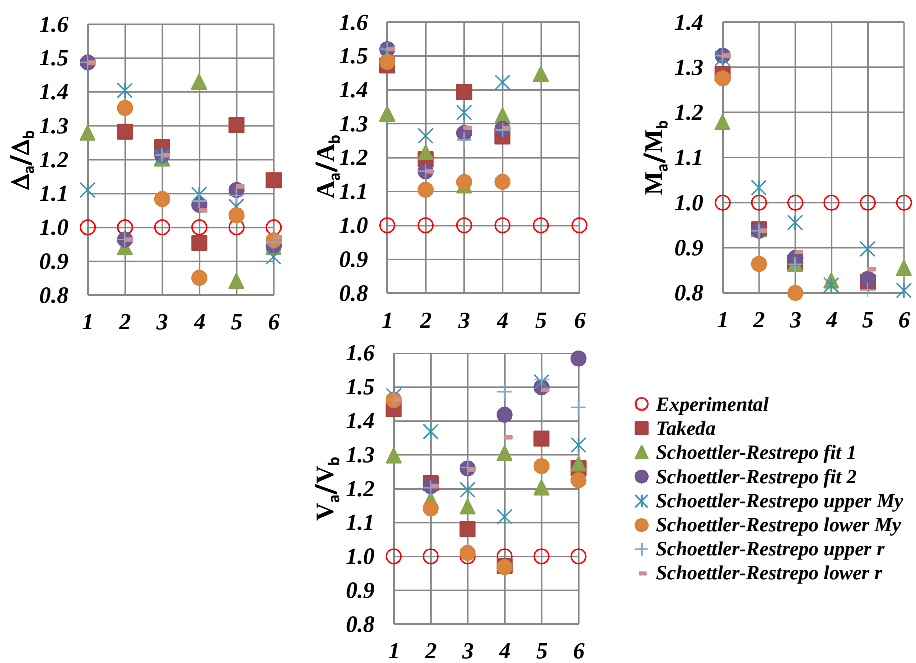

Figure 8. Comparative results for models with different nonlinear envelopes

The results obtained from the variation of the damping formulation are given in Figure 9. Three different alternatives including the initial stiffness, tangent stiffness and secant stiffness proportional damping were included. This last option is a modification to the tangent stiffness proportional formulation with the difference that when velocities go to zero the damping force will be different to zero. Further details may be found elsewhere (Carr, 2008). It is unclear which of the different alternatives would provide the best estimate in all the cases.

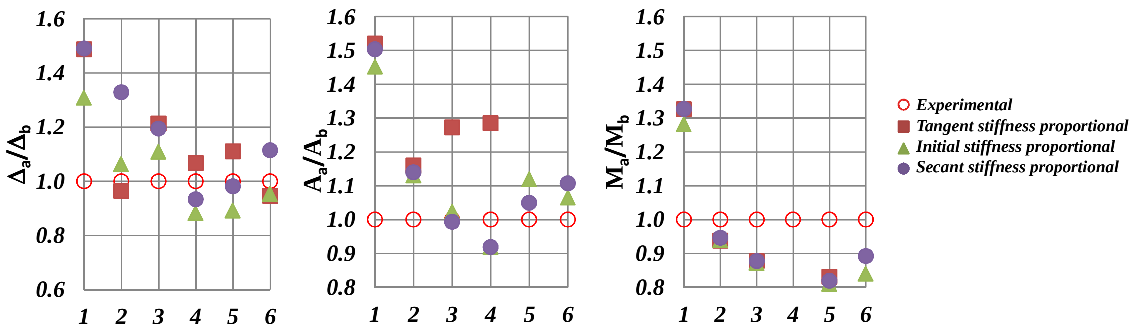

Figure 9. Comparative results for models with different damping formulations

The parameter of the plastic hinge length is considered to be of key importance when creating a nonlinear model of a RC element. Four alternatives, plotted in Figure 10, were evaluated including a simple two elements model of column and a more refined model with four elements. The plastic hinge length provided by Eq 1 provides a relatively good estimation of displacement compared to models with longer plastic hinges. In all cases overprediction of accelerations and shear forces and underprediction of moments occurs.

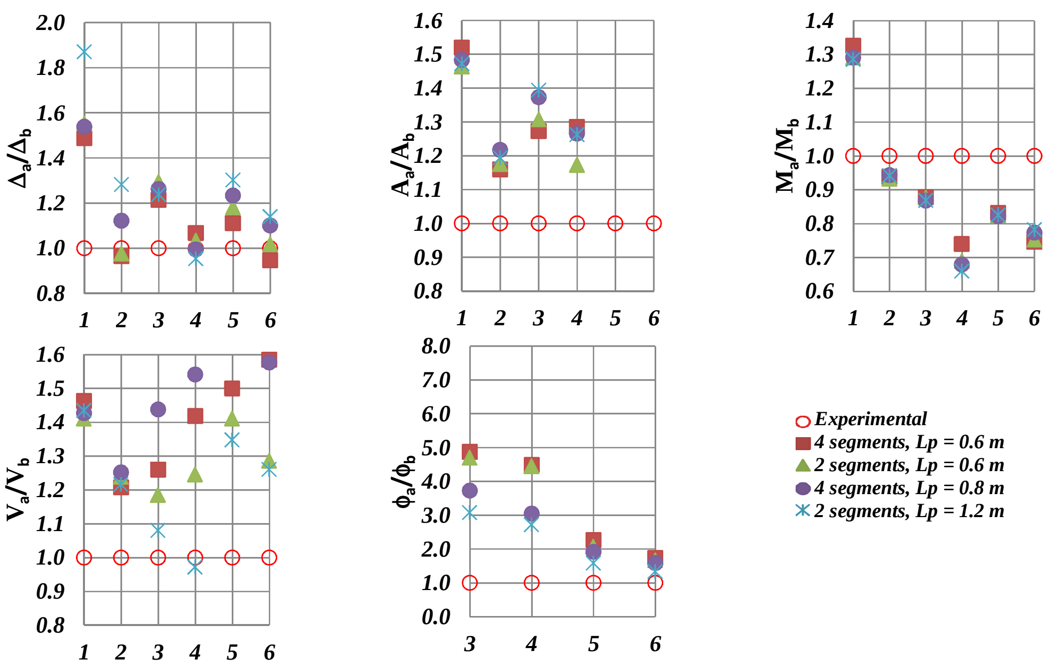

Figure 10. Comparative results for models with different plastic hinge lengths and number of elements

CONCLUSION

Sensitivity nonlinear time history analyses of a RC column subjected to a shaking table test were carried out considering several parameters including distributed and lumped plasticity, material model, integration points, damping formulation and damping levels. Analyses were divided according to the plasticity modelling approach and the results were compared based on the ratio between the numerical and experimental values.

For the distributed plasticity approach the results showed that the most significant variables to be considered are the level of damping and the steel constitutive model. A small number of integration points provide practically the same results as models with a larger number of integration points. Damping levels of 1% to 2% for a tangent stiffness proportional damping formulation provide good match to benchmark experimental values and the effect is more noticeable as the motion intensity is smaller; for very large intensity records, the level of damping was not important.

For the lumped plasticity models there is a large variation of the results and the results do not show particular identifiable trends that allow defining which alternative may give a better march to the benchmark experimental values. The selection of the envelope introduces large variation on the response and in most cases the numerical results have a poor matching to the experimental results. The acceleration, shear and displacements are overestimated whilst the overturning moment is underestimated.

For both cases, distributed and lumped plasticity, there is a poor match between the numerical and experimental values. This can be a significant setback to design methodologies based on the estimation of material strains which are generally obtained from section curvatures.

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