Epistemic uncertainty of the structural response of RC members under cyclic loading due to different material modelling choices

N. Pereira, X. Romão & R. Delgado
University of Porto - Faculty of Engineering, Porto, Portugal

ABSTRACT: Performance-based earthquake engineering methods rely heavily on nonlinear dynamic analysis to determine structural performance. Given the significant number of existing structural and material modelling approaches, the epistemic uncertainty associated to the definition of an analytical model must be quantified, since it will have a considerable effect in the demand and capacity values obtained from the analysis. In this context, the proposed study addresses the quantification of the uncertainty associated to different modelling choices that should be accounted for when developing the analytical model of the structure. The study analyses the performance of different distributed plasticity models of reinforced concrete columns, considering different modelling choices for the constitutive materials. The comparison of the numerical results obtained with experimental data highlighted the impact of the modelling options. Generic error values are presented for each modelling strategy and for different structural demand parameters. General statistical values for the overall variability are also summarized and practical considerations regarding key issues are presented.

1 INTRODUCTION

Performance-based earthquake engineering (PBEE) needs adequate methods of analysis to determine the structural behaviour and the definition of quantifiable targets to measure performance. Considering that performance targets can be established for demand levels ranging from linear elastic behaviour up to the development of structural collapse, nonlinear dynamic analysis is the logical choice to adequately quantify structural behaviour under earthquake loading. However, structural performance associated to a certain damage limit state is influenced by several sources of uncertainty that need to be considered by the PBEE framework. The earthquake record-to-record variability of structural demand and the uncertainty associated to the randomness of the material properties are some of the sources of uncertainty commonly addressed in past research. However, the effect of making different assumptions (based on empirical evidence) for the shape of the numerical modelling of the structural behaviour and the parameters it involves is generally overlooked (Mitra 2008). After selecting a given numerical modelling approach, the uncertainty is introduced by varying some of the basic parameters involved in the model, such as the maximum compressive strength of the concrete or the monotonic envelope of the tensile strength of the reinforcing bars. The proposed paper, analyses the impact of making practical decisions regarding the selection of the material constitutive models and the element formulations in the context of the seismic analysis of reinforced concrete (RC) members. By using experimental results from 2 RC columns tested by Rodrigues et al. (2013) as a reference, both in terms of local and global response, 100 combinations of material models were analysed. These combinations were defined not only by varying the shape of the constitutive models and their hysteretic rules, but also by varying the specific analytical relationships which define the parameters involved in the uniaxial material laws.

The study was divided in two stages. The first stage analyses the performance of the 100 fibre modelling combinations for the numerical simulation of moment-curvatures evolutions obtained from the experimental tests. In the second stage, the numerical analysis of the total RC member was performed which was then also compared to the same set of experimental results. Different distributed plasticity models were considered in this stage, with particular attention being given to strain localization effects that are generated by the softening of the section level response. Hence, in this case, both the element modelling strategy and the material modelling choices are considered simultaneously. To analyse the level of uncertainty associated to the different modelling options, comparisons are made between the characteristics of the errors found for the sectional and the element responses.

2 LOCAL AND GLOBAL RESPONSE OF RC MEMBERS CONSIDERING DISTRIBUTED PLASTICITY FRAME ELEMENTS

The nonlinear cyclic analysis of RC members can be performed using different types of models which have been developed over the years and that require different levels of complexity in terms of modelling, analysis and interpretation of the results. From a practical point of view, frame models constitute an interesting option since, in Earthquake Engineering, the number of analyses that are required to assess the behaviour of a structure usually demands for a simple approach. Nevertheless, its level of accuracy needs to be considered, since the simplicity of the model may involve inherent simplifications and omissions that can have a strong impact in the seismic demand results. Within the scope of frame models, different classes can also be found, depending on the failure mechanisms that are included in the element formulation. Two commonly employed formulations for the modelling of beam-column elements are lumped-plasticity models, defined by nonlinear springs located at the member ends connected, in series, with an elastic element, and distributed plasticity models, which allow for the spreading of the plasticity along the entire length of the member. To analyse structural behaviour using such methods, some decisions must be made by the analyst in order to define the main properties of the numerical model. The fact that multiple models of lumped plasticity and distributed plasticity are available leads to a large number of modelling possibilities with different characteristics, whose effects may not be considered during the analysis process. In the present paper, only distributed plasticity models (relying on the Euler-Bernoulli hypothesis) with fibre models modelling the structural behaviour at the section level are analysed.

By considering distributed plasticity models, two main factors must be kept in mind when selecting the properties of the model. Since the element response relies on the numerical integration of the contribution of individual sections placed at specific locations within the element, the response will depend on the properties of the sectional response, which is obtained by integrating the behaviour of the different fibres represented by a uniaxial constitutive response material. Therefore, one source of uncertainty is associated to the uniaxial constitutive models selected for the materials. This selection sets the type of interaction between the materials which is particularly important for the larger levels of deformations that control the flexural capacity of the member. For example, when considering the behaviour of quasi-brittle materials such as RC, the accurate modelling of the softening of the sectional response is strongly dependent on the material models considered. Furthermore, to determine the section level response that is used to compute the member response also, element-level displacement of force shape functions are also required. These can be defined according to classic displacement-based (DB) or to the more recently developed force-based (FB) formulations Numerical issues associated to strain localization when modelling softening behaviour using different model formulations and number of integration points (for the case of FB elements), and mesh characteristics (for the case of DB elements) were analysed by Calabrese *et al*. (2010).

2.1 *Hardening and softening sectional response and constitutive materials of RC members*

The flexural response of RC members usually concentrates damage at the ends of structural members, where sections with larger deformations are located. At these sections, the interaction between axial and bending deformations is combined by the fibre models to define the sectional response. Hence, the member damage and the behaviour degradation will depend on the capacity of the material models to capture and represent these response characteristics.

There is a complex interaction between several physical mechanisms affecting the behaviour of RC members. Some of these effects result from the interaction between the materials, the element stresses and the boundary conditions, while others depend on the behaviour of the constitutive materials. The first class of physical mechanisms includes the effects of the slippage of reinforcing bars, the strain penetration, fixed end rotations, bar pull out, among others. The interaction between flexural and shear deformations may also influence the behaviour of the member, depending on its geometry. These factors represent additional sources of uncertainty and were not explicitly considered in the distributed plasticity analysis considered herein.

As referred, the characteristics of the behaviour of the materials have a key role in the definition of the sectional behaviour. RC sections can exhibit different types of responses depending on the amount of axial loading they may withstand. For high levels of axial load ratios, softening of the response after yielding can be expected, particularly if second order effects are included. For intermediate levels of axial loading, a composite behaviour will be obtained since hardening of the response will be observed after yielding up to a certain level of deformation (capping point) after which the softening branch of the response will take over. For low levels of axial load, the response will only exhibit a hardening behaviour if the deformation achieved does not affect the material stability.

These types of section level behaviour connected to the level of axial load only occur if the section has the ability to sustain the demand solicitations. Particularly, the 3 following characteristics of the material response can be seen to control the referred behaviour types: 1) The confined concrete capacity and ductility, which depend on the amount of transversal reinforcement of the section, will have a decisive effect on the cyclic degradation of the response since the crushed fibres will decrease the capacity of the section. Therefore, if the model includes a confinement rate higher than the actual one, the softening response may not be obtained. 2) The consideration of a reduction in the cyclic response of reinforcing bars when compared to that of monotonic behaviour may also induce response degradation which may not be obtained when simplified models are used. 3) The buckling of the reinforcement due to the limited amount of transversal reinforcement. This effect is particularly important in older RC columns with no seismic design. In these cases, buckling may occur not far from the yielding point. The occurrence of buckling induces transversal displacements in the middle of the buckling length of the bar, thus generating the spalling of the cover concrete before the crushing of this layer of unconfined concrete (located on the outer side the stirrup).

It must be emphasized that the global response of RC members results from the interdependency and interaction between these 3 effects and the materials. Usually, these 3 characteristics do not occur in a specific order, particularly when low to intermediate levels of axial load are involved, such as in the columns considered herein. Therefore, the above mentioned capping point can be defined as the point where the softening behaviour begins. In the present study, the capping point was defined according to the experimental data and was used to separate different damage limit states. The capping point was considered to correspond to the experimental displacement (and the corresponding curvature for sectional analysis) leading to full spalling of the cover.

3 DESCRIPTION OF THE PROPOSED STUDY

The proposed study analyses the level of variability that is obtained when performing the nonlinear analysis of RC columns under cyclic loading using distributed plasticity models. In particular, the main objective is to understand the effect of considering different material models to represent the uniaxial behaviour of the materials. Although several distributed plasticity models can be found in the literature, in addition to different regularizing techniques to avoid issues of strain localization, an extensive and detailed analysis of the effect of the member formulations is not within the scope of the current study. Instead, the proposed study analyses the changes in the response of the member when using material models with characteristics extracted from literature references. As mentioned before, the ability of the sectional response to represent key aspects of the damage may induce different softening rates, which may contribute to, or compensate, the strain localization effects. Still, due to the properties of the experimental data considered in the study, 3 different member models were also considered to assess the effects of different material modelling choices.

3.1 *Experimental tests considered*

The presented study addresses the analysis of two columns whose experimental results are those obtained referred by Rodrigues et al. (2013). The selected cases are those referenced as N01 and N05, and correspond to experimental tests of RC columns subjected to uniaxial cyclic loading under constant axial loads. The numerical modelling of the cross sections and of the material properties were defined according to the experimental values. To reflect a practical behaviour simulation case, only the concrete compressive strength and the tensile properties of the steel were based on experimental values. Details of the test results, the test setup and the material properties can be found in (Rodrigues et al. 2013). The curvature and displacement evolutions used in the analyses are those from the experimental tests.

In a first stage of the study, the curvature evolutions were applied to the fibre models of the sections to determine the corresponding moments by equilibrium conditions. Then, using the displacement evolutions, the complete columns were analysed considering a given element modelling option. All the calculations were performed using OpenSees. Figure 1 presents the layout of the considered cross sections and the corresponding level of normalized axial load involved in the tests.

To obtain an objective comparison between experimental and numerical results, the comparative analyses were performed for specific behaviour ranges separated by two limit states connected to the performance of RC members. These comparisons were carried out in terms of peak moments for specific levels of curvature, in the case of the sectional analysis, and in terms of peak base shear for each cycle, in the case of the member analysis. The comparisons were performed using the ratios χ_M and χ_F, which represent the M_{num}/M_{exp} and F_{num}/F_{exp} ratios, respectively, where M_{num} and F_{num} are the numerical values obtained for the peak moment and peak base shear, and M_{exp} and F_{exp} are the experimental values of the peak moment and peak base shear. It should be noticed that a complete analysis of the epistemic uncertainty should also consider a comparison of the shape of the responses, which could be done by comparing the energy dissipation in each cycle, or important key demand parameters such as residual displacements. These parameters were not analysed in the present study.

The comparisons were carried out for the following three ranges of behaviour: a first range representing the behaviour up to the onset of yielding (the yielding of the first bar in tension defines the limit state DLS1), with the yielding displacement defined according to (Priestley et al. 2007); a second range representing the behaviour between DLS1 and the full concrete cover spalling, defined by a displacement obtained from the experimental data (the occurrence of spalling defines the limit state DLS2); and a third range representing the behaviour after DLS2 that involves concrete crushing and the fracture of steel rebars. The referred displacement limits were used to define behaviour ranges in the global response analyses. For the sectional analyses, the curvatures corresponding to the yielding and spalling displacements measured during the experimental tests were used as limit state values.

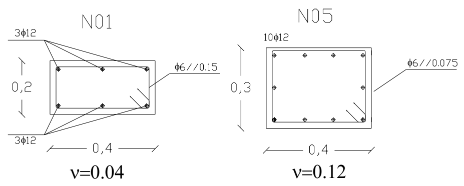

Figure 1. Geometry and reinforcement of the sections.

3.2 *Selected member models*

The fact that short columns are considered in the present study implies that numerical issues are likely to occur depending on the selected integration scheme for the distributed plasticity models. The short length implies that the common assumption in the numerical modelling of frame members stating that only one integration point (IP) must be located within the plastic hinge length is difficult to achieve. In order to overcome this issue, three models were selected to evaluate the impact in the modelling of this type of structural members. In all the models, the plastic hinge length (Lp) was defined according to (Priestley *et al.*, 2007). Therefore, a modelling strategy termed LEG4 means that it is a FB element modelled with Gauss-Legendre integration scheme and 4 IPs. It must be noted that this model does not have an integration point at the element ends, hence it does not consider the integration of the maximum bending moment for the element equilibrium. The LEG4 approach was selected because the weight of the first IP is very close to the value of Lp. An alternative method was also considered which uses the modified Radau approach (Scott & Fenves, 2006). This model BWH considers an inelastic plastic hinge length Lp where the plasticity is concentrated that is connected to an elastic element, and requires the localization to occur within Lp. In addition to LEG4 and BWH, a modelling option that considers a DB model was also considered, with the first element being defined with a length equal to 2xLp.

3.3 *Selected constitutive models*

The simulation of the flexural behaviour of RC sections using fibre models implies that sections must be divided into three zones, with each one being assigned to a different uniaxial material law. The first zone is the area bounded by the centreline of the transverse reinforcement which is made of concrete with a strength enhanced by confinement. The second zone, which bounds the outside of the first zone, is made of unconfined concrete and will govern the development of spalling and the consequent degradation of strength and stiffness. The third zone models the longitudinal reinforcing steel (RS). The characteristics of the RS material law govern several aspects of the flexural behaviour of the section, especially for high levels of deformation.

In the present study, 100 combinations of concrete (C) and RS models were considered. The 100 combinations involved 10 C models (involving unconfined and confined concrete models) and 10 RS models which are numbered according to Table 1. A brief description of the C and RS selected material models is provided in the following.

Table 1. Combinations of constitutive models for the analysis

Model	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10
RS1	1	2	3	4	5	6	7	8	9	10
RS2	11	12	13	14	15	16	17	18	19	20
RS3	21	22	23	24	25	26	27	28	29	30
RS4	31	32	33	34	35	36	37	38	39	40
RS5	41	42	43	44	45	46	47	48	49	50
RS6	51	52	53	54	55	56	57	58	59	60
RS7	61	62	63	64	65	66	67	68	69	70
RS8	71	72	73	74	75	76	77	78	79	80
RS9	81	82	83	84	85	86	87	88	89	90
RS10	91	92	93	94	95	96	97	98	99	100

The consideration of the C modelling options implies that 2 strategies must be considered to account for the different behaviour of confined and unconfined concrete. With respect to confined concrete, 6 of the models were defined according to the Scott et al. (1982) confined concrete model with a residual strength of 20% of the peak compressive strength: the C1 and C2 model have no tensile strength; the C3 model has a linear degradation of the tensile strength based on the linear softening defined by Yassin (1994) (in this case, it is considered that the tensile strength drops to zero after reaching its peak value); the C4 model mimics the tension stiffness mechanism using a linear degradation compatible with the strain limits defined by Kaklauskas & Ghaboussi (2001); the model C5 uses an exponential tensile degradation after cracking, according to the parameters defined by Berry and Eberhard (2008).

The remaining 5 models considered were based on the model by Popovics (1973) (C6, C7, C9 and C10) and Waugh (2007) (C8). Model C6 uses the Mander et al. (1988) proposal for the modelling of confined and unconfined concrete with the Karsan & Jirsa (1969) hysteric rules and has exponential tension development after maximum tensile strength. Model C7 considers the same behaviour envelope and hysteresis rules but defines the reference points of the backbone curve using the Biskinis and Fardis (2009) proposal. Models C9 and C10 are the same of C6 and C7 but have no tensile strength. Finally, the C10 model uses the concrete model proposed by Waugh (2007). In order to simulate the spalling and crushing phenomena, the laws of the compressive behaviour of the materials were modified for strains higher than specific thresholds. For such strains, the response of the concrete fibres is considered to have zero strength and slope (i.e. a null contribution to the strength of the section). To simulate the effect of spalling, a limiting strain ε_{spall} = 0.005 was considered in models C2-C7, according to the ranges in Priestley et al. (2007) and Caltrans (2006). In the case of models C1, C9 and C10, the strain corresponding to buckling of the reinforcement as defined by Pantazopoulou (1998) was considered, assuming that complete spalling will occur at this point. To simulate the onset of the confined concrete crushing, three alternatives were considered according to the selected concrete models. For models C1 to C5, this compressive strain limit was set by the maximum strain defined by Scott *et al.* (1982). In models C6, C8 and C9 it was defined by the crushing strain limit given by Priestley *et al.* (2007), while in models C7 and C8 it was set by the ultimate strain $\varepsilon_{c,ult}$ according to the proposal of Biskinis and Fardis (2009).

Four RS models were considered in the present study. Model RS1 is a simple bilinear model with non-zero post-yield hardening. Model RS2 is a variant of RS1 that includes isotropic hardening. Models RS3 and RS5 are the Menogotto & Pinto (1973) model with isotropic hardening, while RS4 and RS6 are the same model with no cyclic hardening.

Models RS7, RS8 and RS9 were considered according to three variants of the enhanced RS model by Kunnath *et al.* (2009). Model RS5 is the reference form of the model. The RS6 model includes strength degradation effects using the degradation parameters proposed by Berry & Oberhard (2008). The RS7 model is a variant of RS5 that considers the Gomes & Appleton (1997) steel buckling rules. Finally, the model RS10 represents the Dodd & Restrepo (1995) proposal. The main differences between RS4/RS6 and RS3/RS5 are in the parameters assumed to model the backbone curve. RS4/RS6 follow the values extracted from the experimental results while RS3/RS5 were defined with an hardening ratio of 0.01 and a lower ultimate strain (i.e. 0.09) to account for degradation of the bar strength.

4 ANALYSIS OF THE RESULTS OBTAINED FROM THE SECTIONAL ANALYSIS

In the first stage of the analysis, the moment curvature response was analysed for the various DLS defined. As defined before, 3 DLSs were considered, limited by the curvature corresponding to the column yield displacement, to the observed spalling displacement and to the end of the record.

4.1 *Comparison of the numerical and experimental responses up to DLS1*

The first behaviour range goes up to the onset of the section yielding. Figure 2 presents the mean value and the coefficient of variation (CoV) of the χ_M ratios obtained for each combination of models, for the 2 sections, considering peak values up to DLS1. By analysing the mean ratios of the 2 sections, they can be seen to be very sensitive to the modelling of tensile behaviour in concrete.

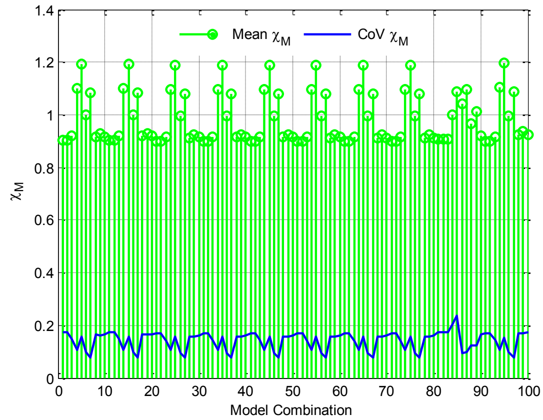

Figure 2. Mean and CoV of the χ_M for all the combinations of models and for peak values up to DLS1.

The mean ratios presented variations between 0.90/0.93 when no tension was considered, as opposed to the values obtained for combinations involving other concrete models which vary from 0.99 (with C6) to 1.19 (with C5). With respect to the influence of the RS model, the variability of the mean of χ_M up to DLS1 is seen to be independent of the selected model. A global analysis of the results obtained for both columns and all the model combinations shows that the mean and the CoV of the χM ratios were found to be 0.98 and 0.17, respectively.

4.2 *Comparison of the numerical and experimental responses up to DLS2*

According to Figure 3 which presents the mean value and the CoV of the χ_M ratios obtained for each combination of models, for the 2 sections, considering peak values between DLS1 and DLS2, it can be seen that most differences between the several com-

binations found in the previous behaviour range have disappeared. This means that tensile strength is not important to model this range of behaviour. The χ_M ratios are similar for all the models considered both in terms of the mean and the variability. Globally, the means vary from 0.98 to 1.02, and exhibit a CoV ranging between 0.08 and 0.14. The ratios obtained present very low differences when compared regarding the use of different RS models.
The global analysis of the results obtained for both columns and all the model combinations, from yielding until spalling, shows that the mean and the CoV of the χ_M ratios were found to be 0.97 and 0.14, respectively.

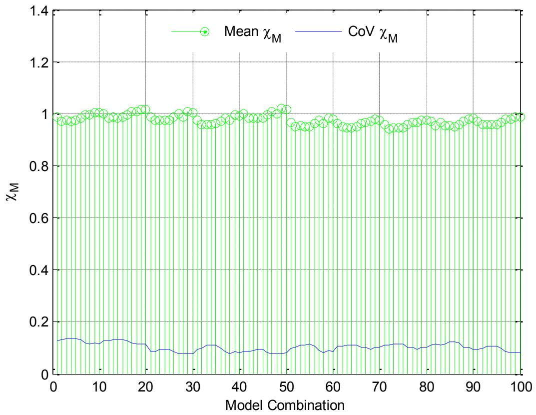

Figure 3. Mean and CoV of the χ_M for all the combinations of models and for peak values between DLS1 and DLS2.

4.3 *Comparison between the numerical and experimental responses after DLS2*

For the third behaviour range, the sectional response that was analysed ranged from the curvature corresponding to the spalling of the cover until the last measured cycle. It should be pointed out that, for the case of N01, the end of the analysis corresponds to a base shear of 80% of its maximum value, while for N05 it corresponds to 50% of the maximum base shear. Figure 4 presents the mean value and the CoV of the χ_M ratios obtained for all the model combinations, for the 2 sections, considering peak values after DLS2.
Unlike for the two previous behaviour ranges, the variability of the ratios is now seen to be much larger. Both the mean and the CoV exhibit significant variability across the model combinations. Two particular aspects can be highlighted in a preliminary evaluation of these results. First, the ratios are no longer dependent on a specific material, as in the previous ranges. Instead, there is now a more complex interaction between the material models, namely in terms of the combination of their individual degradation features to model the real behaviour degradation of the section.. Such interaction can be observed in the model combinations where the maximum compressive strain of the unconfined concrete is the same as the strain that leads to the buckling of the bar, as defined by Pantazopoulou (1998). The second aspect is related to the steel degradation due to buckling of the rebars. It is seen that modelling this effect is very sensitive to the definition of the model parameters. For N01, model RS9 provided the best fit, while for N05, using RS9 resulted in an excessive level of degradation, which then led to a disproportionate reduction of the section capacity. Although such modelling strategies have an undeniable potential due to the key role of buckling in the response of the section for larger levels of demand, its calibration is difficult, namely in terms of defining values for the buckling length, for example.

With respect to the influence of the concrete models, using different $\varepsilon_{c,ult}$ values is seen to have a significant influence in the χ_M ratios. Higher mean values were observed for models C6-C10 when compared to those obtained for models C1-C5. This indicates that simulating structural behaviour in this range is highly sensitive to the definition of the concrete failure in compression (core crushing) and, thus, to the value of $\varepsilon_{c,ult}$. The lower values found when using model RS9 are due to the reasons explained before.
The global analysis of the results obtained for both columns and all the model combinations for this behaviour range shows that the mean and the CoV of the χ_M ratios were found to be 0.80 and 0.36, respectively.

5 ANALYSIS OF THE RESULTS OBTAINED WITH THE ELEMENT ANALYSIS

In the second stage of the analysis, the shear force demand at the base of the columns was analysed for the 3 behaviour ranges. As defined before, those ranges were limited by the column yield displacement, the spalling displacement and the displacement at the end of the test. The main objective of this comparison was to observe, at a global level, the effect of using different material modelling approaches. For the particular range involving larger levels of displacement, this comparison will determine the effect in the member response of over- or underestimating the amount of softening in the section response.

5.1 *Comparison between the numerical and experimental responses up to DLS1*

Figure 5 compares the response obtained for the element models BWH, LEG4 and DB2L for displacements up to DLS1. A clear pattern can be identified since, for a given element formulation, the pattern of results is similar to the one found when analysing the section response in Section 4.1. The models that include higher levels of fracture energy in tension lead to ratios χ_F closer to 1, while lower values were obtained for the remaining cases. In terms of the effects resulting from the use of different element formulations, it is seen that DB2L systematically yields a higher level of global response when compared to that of BWH and LEG4. On average, DB2L yields shear forces about 8% higher than BWH and 6% higher than LEG4. Analysing the results of all the considered combinations of material models and element formulations shows that the mean of the χ_F ratios is 0.99. In terms of the variability, the CoV of the χ_F ratios was found to be 0.17. Therefore, the use of different modelling schemes has no significant effect in the global accuracy of the structural response. The most important effects remain dependent on the type of material modelling.

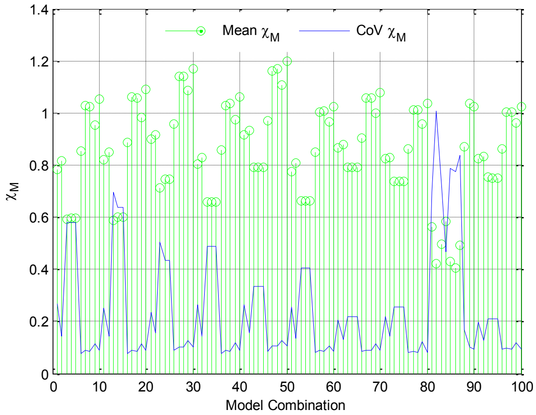

Figure 4. Mean and CoV of the χ_M for all the combinations of models and for peak values after DLS2.

5.2 *Comparison between the numerical and experimental responses up to DLS2*

Figure 6 compares the response obtained for the element models BWH, LEG4 and DB2L for displacements between DLS1 and DLS2. As for the previous behaviour range, the results of each element formulation exhibit a pattern similar to the one found when analysing the section response in Section 4.2. In terms of the effects resulting from the use of different element formulations, it is seen that DB2L continues to yield higher levels of global response when compared to those of BWH and LEG4. On average, DB2L yields shear forces about 7% higher than the other formulations. As can be seen from Figure 6, this overestimation of the response has always a negative effect for this behaviour range. Analysing the results of all the considered combinations of material models and element formulations shows that the mean of the χ_F ratios is 0.98. In terms of the variability, the CoV of the χ_F ratios was found to be 0.10. Therefore, the use of different modelling schemes has no significant effect in the accuracy of the structural response. Aside from the response overestimation by the DB2L formulation, most of the response variability remains dependent on the type of material modelling.

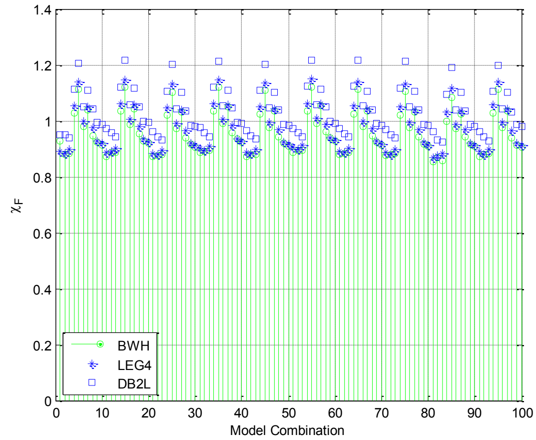

Figure 5. Mean and CoV of the χ_F for all the combinations of models and for peak values up to DLS1.

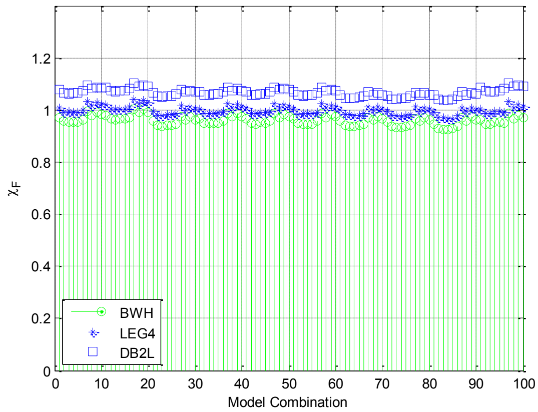

Figure 6. Mean and CoV of the χ_F for all the combinations of models and for peak values between DLS1 and DLS2.

5.3 *Comparison between the numerical and experimental responses after DLS2*

The last stage studied in the global analysis of the columns represented the most variable set since the effects of element model and the degrading sectional response appear to have contributions for the response. Figure 7 presents the distribution of the mean ratios obtained. Comparing this evolution with the variations observed in Figure 4 allows for the replication of many of the considerations made regarding the effect of material modelling. With respect to the comparison between the responses of the different member models, it can be seen that the mean χ_F ratio follows a similar trend in the variability with respect to the variation of the material modelling combination. Nevertheless, the model DB2L gives the higher ratios, which occur for the case C9 to C10. The complex interaction identified implies that the consideration of a numerical model cannot be dissociated from the selection of the material models, since the main features of a model may not be adequate to the properties of the material modelling choices. Analysing the results of all the considered combinations of material models and element formulations shows that the mean of the χ_F ratios is 0.93. In terms of the variability, the CoV of the χ_F ratios was found to be 0.36.

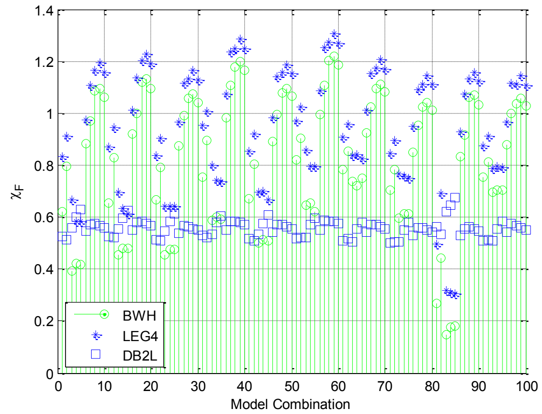

Figure 7. Mean and CoV of the χ_F for all the combinations of models and for peak values after DLS2.

6 CONCLUDING REMARKS

Given the significant number of existing structural and material modelling approaches, an application based study was developed in the present paper, analysing the comparison of the results obtained from the analysis of 2 RC columns, considering distributed plasticity models and 100 material model combinations of constitutive materials of steel and concrete. The analysis was divided in behaviour ranges, in order to evaluate the uncertainty in an elastic, an intermediate range and at strength degradation. For the initial behaviour range, the impact of tension modelling of the materials revealed to be a key issue. For the case of the intermediate range, the response was seen to be dominated by the steel model until degradation issues start to manifest. In the last behaviour range, the interaction between the degrading effects of the materials and the hardening or softening resultant from the element formulation is very sensitive and may have a considerable impact in the results. The presented conclusions are based on 2 experimental cases with low to intermediate normalized axial load levels. It must be pointed out that the response obtained for each column presented different properties, which reflects the uncertainty associated to the real response of RC members. Additional cases must be considered to include this fact in the uncertainty quantification. This study presented an observation of the expected impact that material modelling choices may have in different behaviour ranges of RC members, making some observations about the more critical modelling aspects and about the propagation of the material modelling uncertainty to the global demand parameters.

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