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SIMULATION OF COUPLING BEAMS CYCLIC RESPONSE THROUGH SMEARED CRACK FINITE ELEMENT MODELS

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ABSTRACT

The prediction of the dynamic response of a reinforced concrete (RC) building in front of an earthquake motion is a difficult task due to the multiple and complex phenomena involved. It requires an appropriate consideration of the cyclic response presented by the different structural elements and connections as damage initiates and propagates, particularly for those specifically designed to dissipate energy like coupling beams. Advanced finite element modelling techniques (FE) could provide a useful tool in order to predict the cyclic response of coupling beams, since recent research works pointed out the ability of smeared crack models to simulate RC structural members subjected to cyclic loading. These techniques present a high computational demand, especially if the complete building aims to be modeled, thus being necessary to find optimal configurations. This paper studies the capabilities of nonlinear finite elements models in predicting the nonlinear cyclic response of diagonally reinforced coupling beams by simulating a laboratory test reported in the bibliography. Two different aspect ratios coupling beams (2.4 and 3.33) are analyzed by means of smeared crack FE models. Different mesh sizes and steel modelling considerations are evaluated by comparing the force-displacement response and the energy dissipation. Conclusions about the most significant parameters influencing the obtained response, the precision achievable, and the recommended configuration for reducing the computational demand are finally presented.

Keywords: Coupling beams; Cyclic response; Nonlinear finite element; Smeared cracking; Damage analysis

1. INTRODUCTION

The aim of predicting the dynamic response of reinforced concrete (RC) buildings in front of earthquake motions is a difficult task due to the multiple and complex phenomena involved. It requires an appropriate consideration of the cyclic response presented by the different structural elements and connections as damage initiates and propagates. This need is particularly relevant for those buildings designed to dissipate energy through the damage of their structural members.

In this field, one of the structural elements most commonly used for dissipating energy are the coupling beams. They are typically placed connecting walls at story levels, being subjected to alternate shear loads when an earthquake motion shakes the building. When introduced motions are intense enough to exceed the elastic response of concrete, a stiffness degradation process occurs, dissipating energy through the hysteretic response presented by the coupling beam. Therefore, the appropriate reproduction of the dynamic response of the building requires to adequately consider these phenomena and, therefore, to know the response of the coupling beam. Useful information can be obtained from the few laboratory tests performed up to date, but assumptions have to be taken in order to adequate the response for different geometry and material configurations. In these sense, the development of techniques and tools that allow to predict the expected cyclic response of coupling beams would contribute to improve the analysis and study of the dynamic response of RC buildings.

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During the last decades, finite elements modelling techniques (FE) have significantly improved in reproducing the complex response of RC structures. Sagaseta (2008) and Belletti *et al.* (2013) pointed out the ability of current modelling techniques based on cracking damage models on reproducing the complex mechanisms governing the response of beams subjected to shear forces. The potential capacity of advanced nonlinear FE methods to reproduce the cyclic response of RC members has been also noted, as showed by Kwan and Billington (2001) and Deaton (2013) for column base and column-beam connections, or by Parra (2015), Belletti *et al.* (2016), Dashti *et al.* (2017), Arias *et al.* (2017) when reproducing the cyclic response of RC walls. Therefore, these techniques emerge as a potential tool to predict the cyclic response of coupling beams, which can be used to extract their local response and implement it in a more simplified model, or for directly being included in a detailed model of the complete building.

Nonlinear FE techniques present a high computational demand, thus being traditionally limited to study the response of individual structural members or small assemblages. Nowadays, the evolution of computers and FE codes is opening the possibility to study complete RC buildings through detailed FE models, but appropriate mesh sizes and modelling techniques that minimize the computational demand without significantly affecting the precision have still to be used. Therefore, it is necessary to determine the optimal numerical configuration that demands the minimum computational resources whilst providing a satisfactory precision.

This paper studies the capabilities of nonlinear finite elements models in predicting the nonlinear cyclic response of coupling beams by simulating a laboratory test reported in the bibliography. Two diagonally reinforced coupling beams with different aspect ratios (2.4 and 3.33) are analyzed by means of smeared crack FE models. The two main three-dimensional modelling possibilities are studied by comparing the results of solid brick and shell element models. Different mesh sizes and steel modelling considerations are evaluated by comparing the force-displacement response and the energy dissipation. Conclusions about the most significant parameters influencing the obtained response, the precision achievable, and the recommended configuration for reducing the computational demand are finally presented.

2. CASE STUDY

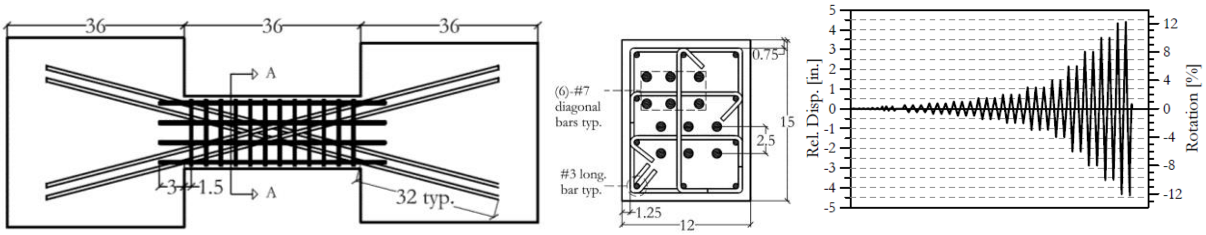
Naish (2010) performed an experimental campaign focused on study the cyclic response of coupling beams with diagonal reinforcement according to the ACI 318 standards. Main objectives were to characterize the response of coupling beams with two different aspect ratios according to current construction practice, evaluating the influence of different stirrup configurations (diagonal bars confinement or full section confinement), the impact of the floor slab, and determining residual strengths and total plastic rotation capacities.

The specimens selected for this study are the CB33F and CB24F, which present aspect ratios of 3.33 and 2.4 respectively, with full section transverse reinforcement for confinement (Naish, 2010). Figure 1 presents dimensions, reinforcements, and the applied cyclic loading. Maximum chord rotations around 12 and 8 % are achieved for the 2.4 and 3.33 aspect ratio beams respectively. Beams are loaded through a special loading frame setup with the aim of ensuring zero axial force on the beam whilst keeping parallelism between the top and bottom faces.

3. NUMERICAL MODELLING

In order to analyze the different available configurations for modelling structural elements in a three-dimensional way, both solid (SO) and shell (SH) elements models are constructed in order to reproduce the coupling beams tests. Two different mesh sizes are also considered, dividing the beam depth into three elements (size 1, S1) and five elements (size 2, S2) (Fig. 2).

CB24F



CB33F

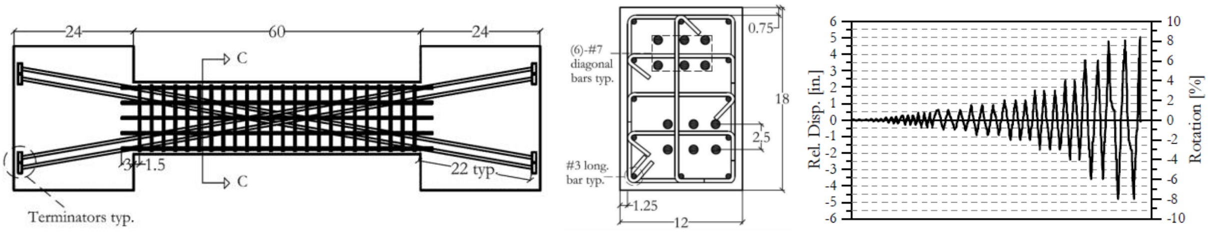


Figure 1. Characteristics and loading of the specimens CB24F and CB33F. (Naish, 2010)

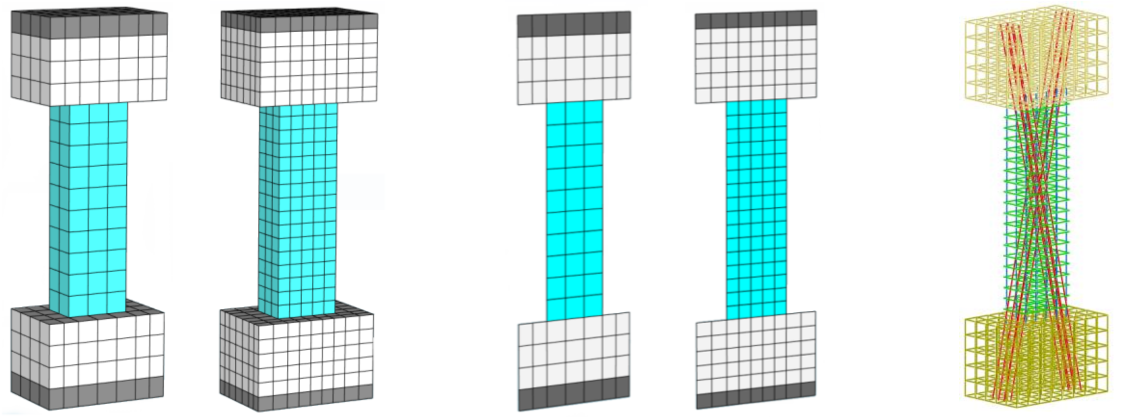


Figure 2. Model mesh definitions illustrated for CB33F specimen.

All models are created and analyzed according to the recommendations for nonlinear FE analysis of concrete structures presented in the RTD guidelines (2012), and to the user's manual of the utilized software Diana 10.2 (DIANA FEA, 2017). Solid models are defined using 20-node hexahedral brick elements with a 3x3x3 integration scheme, whilst 8-node quadrilateral elements with also 3x3 integration in-plane and three integration points through element's depth are used for shell elements.

Concrete material response is implemented through the Total Strain Rotating Crack model included in Diana 10 software, as it is recommended by Deaton (2013) and Kwan and Billington (2001) for reproducing the shear and hysteretic response of RC members. Compression and tensile behavior are considered as it is recommended in RTD guidelines (2012), adopting a parabolic law for modelling the compressive response of concrete, whilst an exponential law is selected to reproduce the tensile softening response. Concrete confinement effect is introduced through the Selby and Vecchio model implemented in Diana 10. The compression resistance reduction due to lateral cracking is considered by the Vecchio and Collins model available in the software, assuming a maximum reduction factor of β_{min}=0.6 as used by Damoni *et al.* (2014).

Steel reinforcement bars are explicitly modelled (Fig. 2), embedded inside the concrete finite elements, and considering perfect bond between both materials. The cyclic behavior of the reinforcing steel is adopted through the Dodd-Restreppo (1995) model (assuming factors p=0.35 and Ω_f=0.75), which reported satisfactory results in predicting the cyclic response of RC walls in the numerical simulations performed by Arias *et al.* (2017).

Table 1 presents a summary of the adopted material properties, which have been obtained mainly from Naish (2010), NEES hub database, or derived from ACI-318-11 and Model Code 2010 (FIB, 2010).

Table 1. Material properties adopted for numerical models.

Elements	Property	Nomenclature	Value	Source
Concrete	Modulus of elasticity	E_c (N/mm^2)	32527	ACI 318-11
	Poisson coefficient	v_c	0.15	RTD Guidelines (2012)
	Compression strength	$\sigma_{c,c}$ (N/mm^2)	47.23	Naish (2010)
	Comp. fracture energy	G_C (N.mm/mm^2)	36.5	RTD Guidelines (2012)
	Tensile strength	$\sigma_{t,c}$ (N/mm^2)	4.97	ACI 318-11
	Tensile fracture energy	G_F (N.mm/mm^2)	0.146	Model Code 2010
Reinforcing	Modulus of elasticity	E_s (N/mm^2)	200000	Adopted
Steel	Poisson coefficient	v_s	0.3	Adopted
	Tensile yielding strength	$\sigma_{y,s}$ (N/mm^2)	483 (#6,#7) 439 (#3)	NEES data
	Tensile ultimate strength	$\sigma_{u,s}$ (N/mm^2)	624.2 (#6,#7) 679.1 (#3)	NEES data
	Tensile ultimate strain	$\varepsilon_{u,s}$ (%)	0.157 (#6,#7) 0.25 (#3)	NEES data

The loading histories shown in figure 1 are applied into the models, taking into account that they refer to the exclusive distorsion of the beam. In order to reproduce the tests configuration, displacements are imposed on the top of the specimen, but controlled by the distortion of the beam. Horizontal and vertical displacements are fixed in the base, whilst out-of-plane displacements are fixed at both bottom and top of the model. In order to reproduce the test set-up and to consider the small possible flexibility offered by the testing infrastructure, in-plane rotational springs were introduced at the top and bottom to reproduce the initial elastic response of the test. According to the test configuration, the top infrastructure (steel frame) could be more flexible than the bottom one (concrete slab and connection), but due to the uncertainty about this stiffness distribution, equal top and bottom rotational springs are assumed in order to produce a symmetrical response.

4. RESULTS

Figure 3 presents the results obtained for the CB33F beam when using the size 1 solid mesh and the modelling considerations described in section 3. Firstly, it is important to notice that the experimental response is not symmetric, despite the geometry and reinforcement configuration of the specimen is symmetric (Fig. 1). Numerical results achieve very similar strength in both loading directions, whilst experimental results present 12 to 18 % higher strength for the positive lateral displacements. Despite the influence of this fact, the numerical simulation reproduce the general response tendency of the beam, as can be observed in figure 3. The coupling beam strength tends to be slightly overestimated, presenting errors from 4 to 14 % (respect to the positive displacement branch) for the main performance zone of the beam comprised between 30 to 92 mm displacement (25 to 75 % of the ultimate displacement).

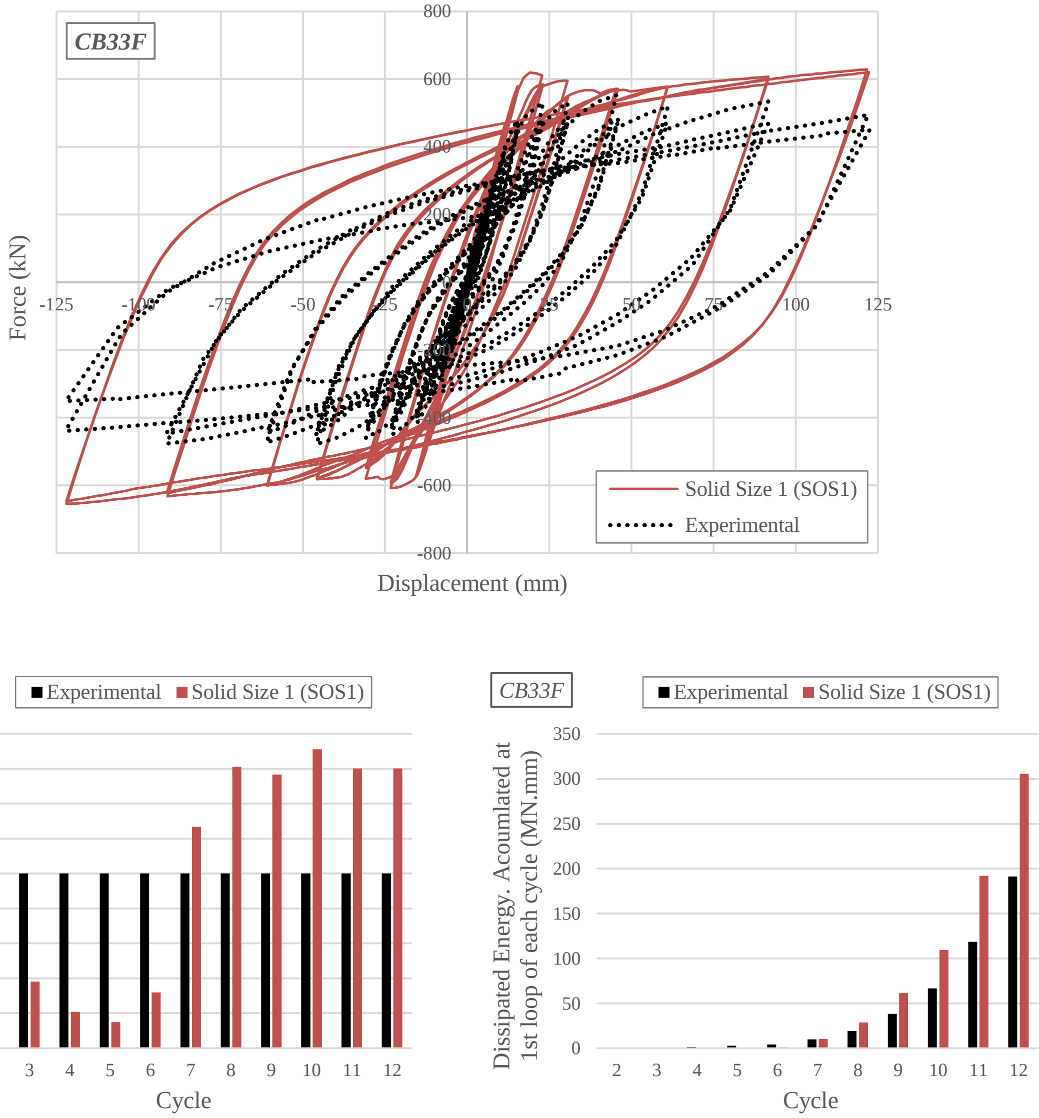
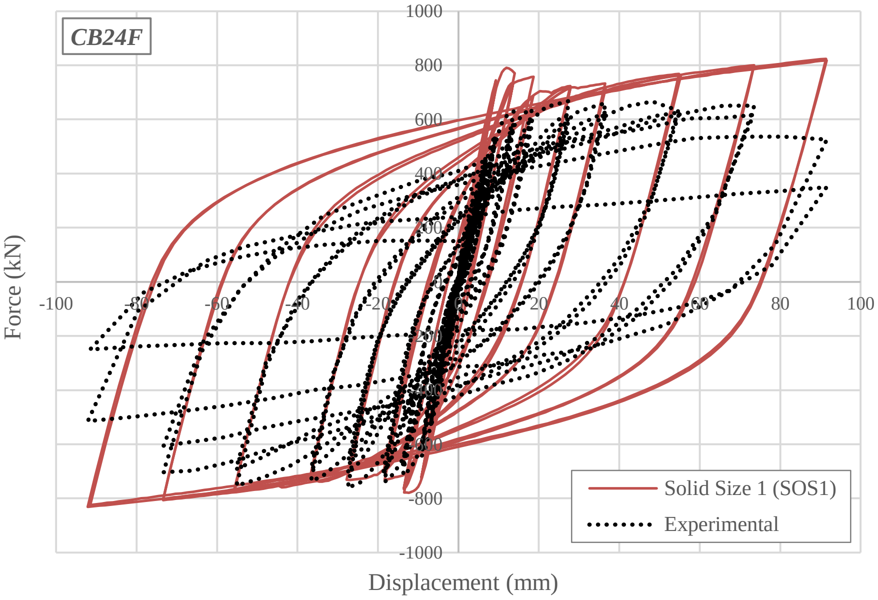
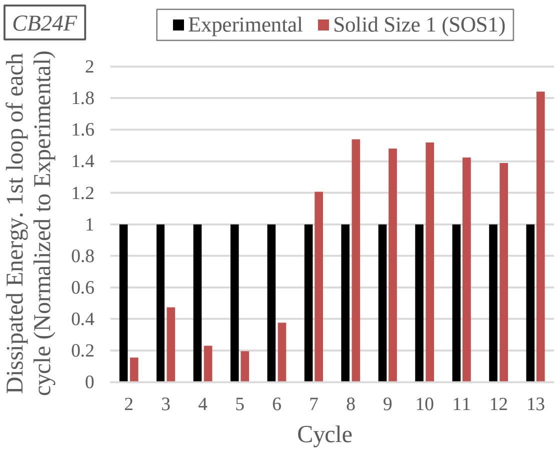


Figure 3. CB33F specimen simulation. Solid size 1 model.

The approach achieved in terms of energy dissipation results less accurate. As can be observed in figure 3, the energy dissipation is considerably underestimated for the small displacements achieved at the initial cycles. But energy dissipated during small displacement cycles is not relevant in comparison to the beam energy dissipation capacity, as can be observed when displaying the accumulated dissipated energy (Fig. 3). For cycles from 7 to 12, covering the range from the 19 to 100 % of the ultimate displacement, the energy dissipation per cycle tends to be overestimated in a ratio between 1.27 and 1.7. This fact leads to an overestimation of 60 % in the total energy dissipated along the loading process if the first loop of each cycle is considered. As can be inferred from figure 3, the unsymmetrical response presented in the experiment influences the precision obtained in the simulation of the energy dissipation.





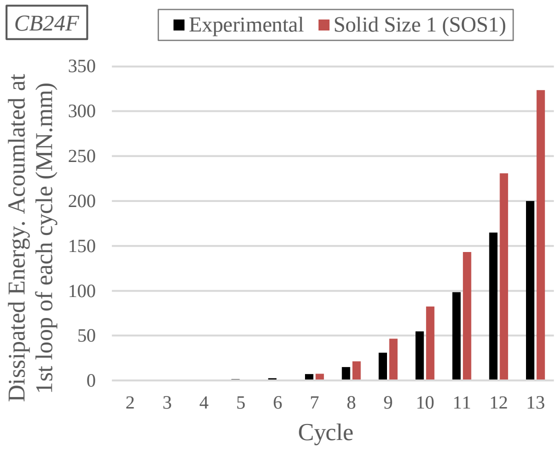


Figure 4. CB24F specimen simulation. Solid size 1 model.

Figure 4 presents the results obtained in the simulation of the CB24F specimen with the solid size 1 mesh. Equally as happened for the CB33F specimen, the experimental response also presents an unsymmetrical loading but in a lower degree, presenting differences in maximum strength per cycle between 8-16 %. The numerical model achieves a satisfactory reproduction of the general response of the coupling beam, particularly before the start of its failure process for displacements over 60 mm. The maximum load achieved at the first cycle of each displacement level is very well simulated, presenting differences under 4 % (respect to the negative displacement branch) for displacements between 18 and 55 mm (20 to 60 % of the ultimate displacement).

Equally as observed for CB24F, the energy dissipated is underestimated for the initial cycles where small displacements are achieved. For the main performance zone of the beam (displacements between 18 to 74 mm – 20 to 80 % of the ultimate displacement), energy dissipation prediction is overestimated with factors from 1.21 to 1.54. This produces an overestimation of 40 % in the total energy disipated since the start of the coupling beam failure (up to cycle 12). As can be observed, the numerical model presented a better accuracy in the reproduction of the CB24F specimen, which could be influenced by the fact that its experimental response exhibit a less unsymmetrical response.

Notice that the numerical behavior for both cases presents an increasing strength with the increase of displacements, which is in correspondence to the hardening response of the reinforcing steel reported by the authors. On the contrary, experimental results exhibit a more constant maximum load along the deformation. This fact could be related to the assumption that reinforcements are perfectly bonded to the concrete elements, which could cause an overestimation of the reinforcement deformation, and since steel hardening is considered, the consequent overestimation of the reinforcement stress.

4.1 Mesh size and type influence

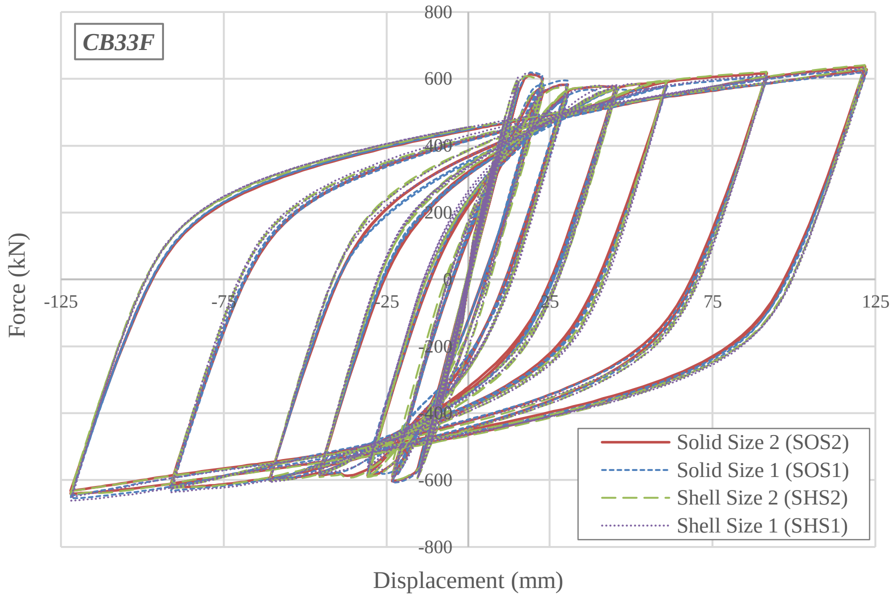


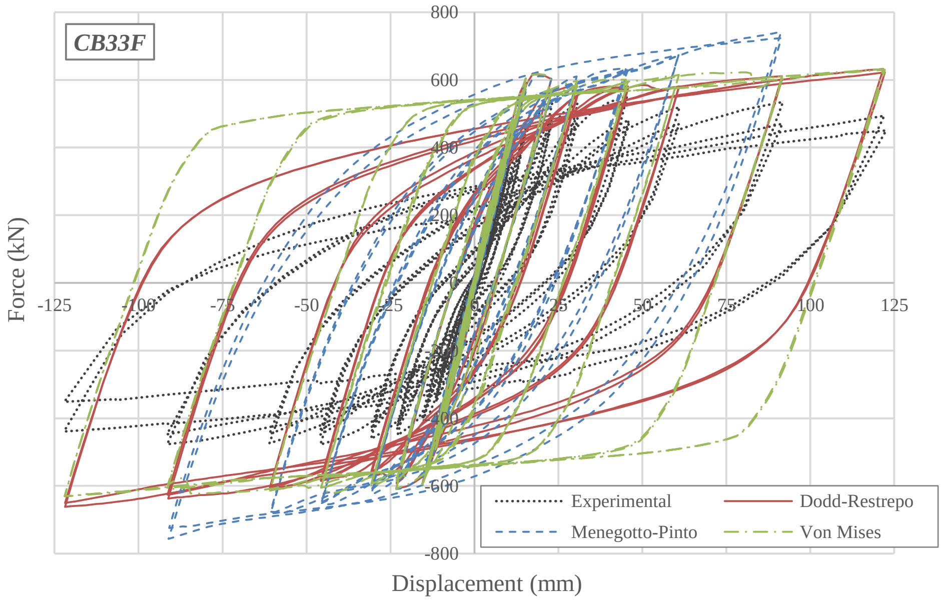
Figure 5. CB33F results with different mesh types and sizes.

Figure 5 presents the results obtained in reproducing the CB33F specimen test by means of the different mesh configurations shown in figure 1. Both solid elements models (SO) and shell elements models (SH) are tested when dividing the height of the beam in three elements (size 1, S1) and in five elements (size 2, S2). No significant differences are found between the tested configurations, thus being possible to use the less demanding in computational terms. For this particular analysis, the three elements height shell (SHS1) was the fastest option, followed by the three elements height solid (SOS1) (2.3 times slower), the five elements height shell (SHS2) (2.8 times slower), and finally the five elements height solid (SOS2) that results 4.45 times slower than the SHS1 option.

It is important to highlight that for this particular case, due to the large amount of reinforcement placed at the diagonals and its location at beam ends, the response of the beam is mainly conditioned by the steel response. This minimum configuration based on dividing the element height into three divisions should be ascertained or revised for structural elements in which the compressive local response of the concrete could present a more significant role.

4.2 Reinforcement model influence

As it was previously mentioned, the significant amount of diagonal reinforcement placed in the tested coupling beams, produces that their structural behavior becomes mainly conditioned by the reinforcing steel response. Unfortunately, the material characterization campaign carried out for the experiments does not include reinforcement cyclic tests, thus do not allowing to perform a particular calibration of the cyclic models for the reinforcing steel. Therefore, three different steel models are analyzed in this section, adopting average or referred parameters. Results obtained with the Dodd-Restrepo model (1995) used in previous section is compared with those obtained by adopting the Menegotto-Pinto cyclic steel model (1973) (assuming the typical parameters listed in Yu (2006)), and the more common Von Mises plasticity model with a kinematic hardening.



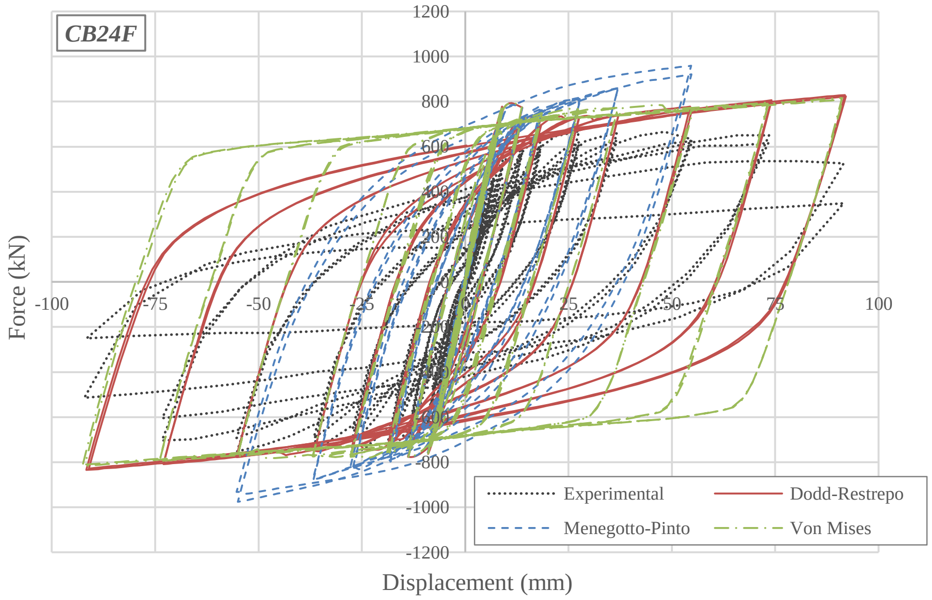


Figure 6. Comparison of reinforcing steel models for CB33F and CB24F specimens. Shell Size 1 model (SHS1).

Figure 6 presents the results obtained with the shell size 1 models of both CB33F and CB24F specimens when the different reinforcing steel models are used. As can be observed, the Dodd-Restrepo model with the adopted configuration (factors p=0.35 and Ω_f=0.75), is the option that provides the more closed hysteretic loops, thus presenting the smaller energy dissipation, and defining the closest response in such terms to the experimental results. The widely used Von Mises model does not take into account the cyclic response of the reinforcing steel (Bauschinger effect), and consequently, a significant overestimation of the dissipated energy is produced. The Menegotto-Pinto cyclic model provides similar energy dissipation than the Dodd-Restrepo model for medium displacement cycles, but as can be clearly observed in figure 6, it provides an overestimation of the maximum load for high displacements. The reason for such difference could be found in the isotropic hardening component of the response adopted in the Menegotto-Pinto model, that produces an increase of the steel stress when significan cyclic plastic deformations are experienced. As a consequence, the diagonal steel reinforcements achieve their ultimate strength earlier, thus predicting a lower ductitlity for the coupling beams.

5. CONCLUSIONS

The general cyclic response presented by diagonally reinforced coupling beams can be approximated through the use of smeared crack models. Obtained results show that a satisfactory strength prediction has been achieved, whilst the energy dissipated has been overestimated for the cycles at the main performance zone of the beam.

The comparison between different numerical models for reproducing the reinforcing steel behavior pointed out that the Dodd-Restrepo model (1995) (with factors p=0.35 and Ω_f=0.75) provided the best fitting to the experimental results in terms of energy dissipation.

Satisfactory results have been achieved in the present study by different mesh configurations. The less demanding in computational terms is defined by regular quadrangular shell elements models, with an element size resulting from dividing the height of the coupling beam by three elements.

Numerical models predict an increasing strength with the increase of the displacement, whilst experimental results denote a more constant strength. These difference could be related to the perfect bond condition assumed between the concrete and steel. Studies should be carried out in order to point out the influence of the concrete-reinforcement bond-slip in the cyclic response of diagonally reinforced coupling beams.

6. ACKNOWLEDGMENTS

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