

MODELLING OF A RC SHEAR WALL EXPERIMENT USING SHELL LAYERED FINITE ELEMENTS

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Abstract: This paper presents the numerical modelling strategy used to recreate a shear wall experiment, in the finite element analysis software ETABS using shell layered elements. The experiment, conducted by Oh *et al.*, subjected four different wall specimens to cyclic lateral loading to determine the effect of transversal reinforcement on their deformation capacity[1]. The experimental results, extracted from the SERIES database [2] for the WR-10 and WR-20 specimens are used as the basis for model validation. The finite element model employs a two-layer shell section, with separate reinforcement and concrete membrane layers, while the cyclic loading protocol is recreated through a series of chained nonlinear static displacement-controlled load cases. The results are compared with the experimental data, in terms of peak lateral force, cumulative energy dissipation and hysteresis loop shape.

Keywords: nonlinear analysis, hysteresis, energy dissipation, cyclic loading

1. Introduction

Reinforced concrete (RC) shear walls are one of the most commonly used structural elements for resisting lateral forces in regions which experience a high level of seismic activity. Studying their non-linear behavior is crucial for designing and detailing these elements as to ensure proper ductility and energy-dissipation. Multiple numerical modelling techniques have been developed for the analysis of shear walls such as MVLEM, fiber models and shell layered finite elements [3–5]. The latter splits the wall's thickness into distinct concrete and steel layers, each of them with their own independent material properties.

Several research papers have shown the effectiveness of the shell layered element when dealing with reinforced concrete shear walls. In their paper, Kolozvari et al.[6] compared five microscopic finite element models for two-dimensional reinforced concrete walls, including models based on a multi-layered shell approach, against experimental data from five reinforced concrete specimens. All models predicted the hysteretic response of the specimens and the cracking, yield and ultimate strength of the walls well, while being able to capture the nonlinear shear-flexure interaction.

Dashti et al.[7] investigated the effectiveness of a curved shell finite element model implemented in the DIANA finite element software package for predicting the nonlinear response of planar reinforced concrete walls. When compared to experimental data, the model successfully predicted the failure mechanisms, including out-of-plane instability, concrete crushing at the base, diagonal tension and compression and sliding shear, but was unable to capture reinforcement bar buckling, bar fracture and the resulting secondary failure modes.

Lu et al.[8] confirm the validity of such models in their paper, by using multi-layer shell elements and fiber-beam elements to create numerical models for tall structures, such as the Shanghai Tower, in MSC.Marc, which are then converted into OpenSees. Pushover and Time-History analyses are conducted and the

results obtained in OpenSees are compared to models in MSC.Marc to corroborate the findings.

Vatanshenas [9] uses layered shell elements to model a wall specimen, using Park's model for steel reinforcement and Mander's constitutive model for concrete. A pushover analysis is conducted, the loading pattern being based on the first mode resulted from a modal analysis, and the step-by-step results are presented from the concrete cracking due to tension in the first step, to both materials failing in the last steps.

Shell layered elements are used to determine the dynamic shear amplification factors in reinforced concrete shear walls, in the study realized by Palacios et al. [10]. Time-History analyses are performed on walls modelled using shell layered elements, with varying reinforcement ratios, heights and axial load ratios across twelve ground motion records.

Tao M. et al.[11] show the promising capacity of machine learning as a tool for optimizing shell layered models. Six constitutive parameters that govern shear wall behavior are calibrated using machine learning algorithms. Resulting models are validated against reinforced concrete shear wall specimens from various experimental data sets.

Together, these studies show that the layered shell modelling approach is a valid and well-established tool for capturing the nonlinear response of reinforced concrete shear walls, supporting its use for this paper.

This paper presents the process of creating a numerical simulation of the experiment realized by Oh et al. [1], in which they subjected various shear walls specimens with different boundary element detailing to cyclic lateral loading, using shell layered elements in the commercially available finite element analysis software ETABS. The objective of this paper is to assess the capacity of the shell layered element in ETABS to reproduce the nonlinear cyclic behavior of concrete shear walls. The experiment conducted by Oh et al. was chosen due to well documented loading protocol presented in the article, the availability of the force-displacement curves and the simple rectangular geometry of the specimens which was deemed suitable for a two-dimensional layered shell model.

The experimental force-displacement curves used for the validation of the numerical model were extracted from the SERIES (Seismic Engineering Research Infrastructures for European Synergies) [2] database, which was assembled from existing experimental data available in literature. The available dataset was created by digitizing the published hysteresis loops and thus it did not cover the entire loading protocol reported in the original publication. The experimental data was used only for the validation of the hysteresis loop shape and the comparison of peak values in representative cycles between the experiment and the model. For the dissipation energy comparisons, data from the article published by Oh et al. was used [1].

2. Description of the Experiment

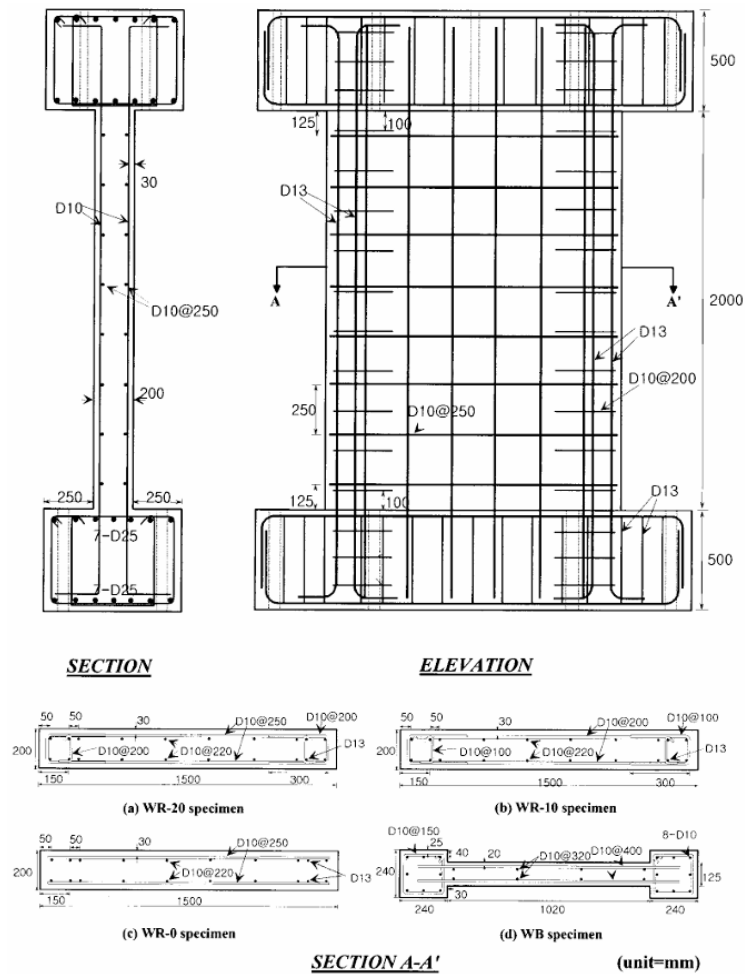


Fig. 1 - Elevations and cross-sections of the specimens tested by Oh *et al.* [1]

In their study, prompted by differences in the detailing of structural walls between Korea, Chile and the United States, Oh *et al.* investigated the influence of boundary element detailing on the deformation capacity of shear walls. Four full scale wall specimens were constructed, three of them with identical geometry but each of their boundary elements reinforced differently (WR-0, WR-10 and WR-20) based on Korean construction practice. The fourth one (WB) was made in a barbell shape, and had no transverse reinforcement in the boundary elements, the authors recreating details similar to Chilean building practice. The present paper focuses on modelling the WR-20 and WR-10 specimens and does not aim to study the effect of boundary element detailing on wall behavior. The specimens had a thickness of 200 mm, a height of 3000 mm and a length of 1500 mm. Both walls had the same longitudinal reinforcement consisting of four D13 bars in the boundary elements and two D10 bars spaced at 220 mm in the web. The transversal reinforcement details established the distinction between the specimens: the WR-20 boundary elements were confined with D10 open hoops and cross ties spaced at 200 mm, while for the WR-10 boundary elements this distance was halved. The webs of both walls were reinforced with two horizontal D10 bars, spaced at 250 mm for WR-20 and at 200 mm for WR-10. The reinforcement details for all specimens can be seen in Fig. 1. The WR-10 and WR-20 specimens were chosen for this study, as they represent standard examples of reinforced concrete shear wall detailing practice, both featuring a rectangular cross-section and transversal reinforcement in their boundary elements.

The reinforcement bars had an average strength of 342 MPa for D10 bars and 449 MPa for D13 bars. The concrete used for the specimens had an average compressive strength between 29.4 MPa and 31.8 MPa at 28 days and between 32.9 MPa and 36.2 MPa at the test date.

The specimens were tested under a cyclic lateral loading protocol. Each wall was pushed until the measured top displacements cycled three times through various drift ratios. The values used for the target drift ratios are as follows: $\frac{1}{600}$, $\frac{1}{400}$, $\frac{1}{300}$, $\frac{1}{200}$, $\frac{1}{150}$, $\frac{1}{100}$, $\frac{1}{75}$, $\frac{1}{50}$ and $\frac{1}{33}$. To simulate the dead-loads that may be present in a building, a vertical load of $0.1 f'_c A_g$ was applied and maintained to the specimens throughout the experiment.

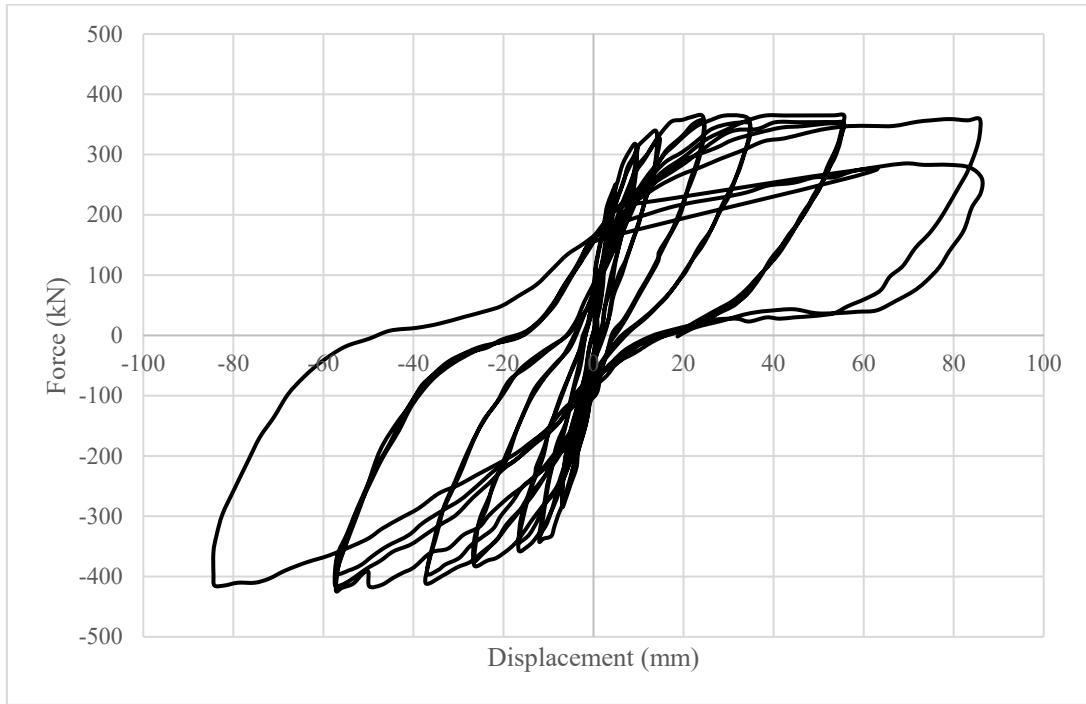


Fig. 2 – The load versus displacement curve obtained for the WR-10 specimen [2]

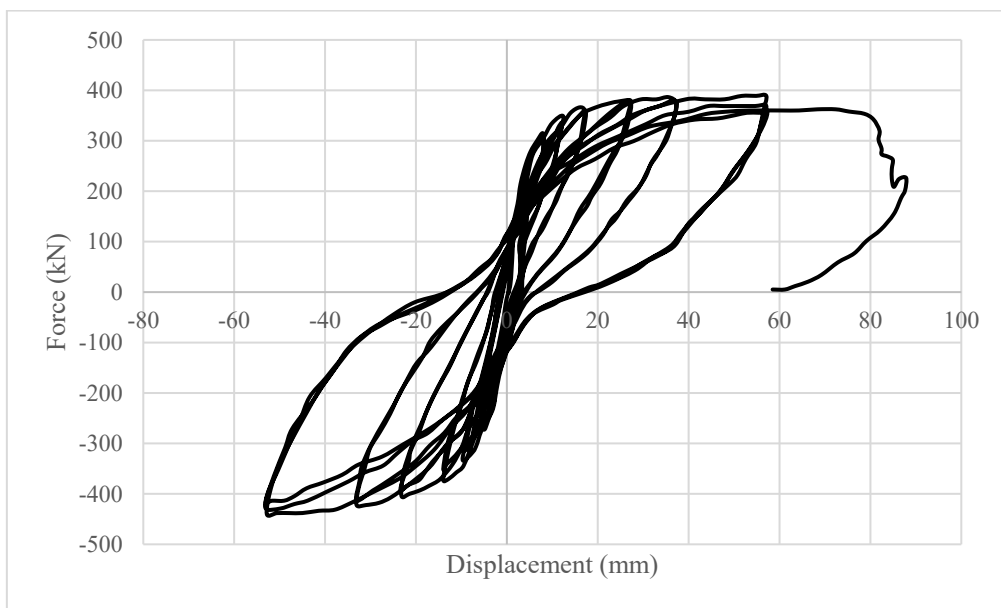


Fig. 3 - The load versus displacement curve obtained for the WR-20 specimen[2]

As stated by Oh *et al.*, the tested specimens began to yield at the $\frac{1}{250} - \frac{1}{300}$ drift ratios, while the specimens that make up the object of this paper, WR-20 and WR-10, experienced a gradual increase in their load-carrying capacity up to the $\frac{1}{55}$ drift ratio. Initial flexural cracks in the lower part of the tensile zone were observed at drift ratios $\frac{1}{600} - \frac{1}{400}$. Subsequently, a large number of flexural cracks developed at a drift ratio of $\frac{1}{300}$, which progressed into flexural-shear cracks at $\frac{1}{200}$. As the drift ratio increased, new cracks propagated into the upper parts of the specimens, until eventually a fully developed crack pattern was observed along the entire height of the specimen. Throughout the experiment, all specimens showed flexure-controlled behavior, failure finally occurring by the crushing of the compressed concrete and the buckling of the longitudinal reinforcement bars, after the drift ratio of $\frac{1}{50}$ [1]. The final hysteresis loops, constructed with the digitized data extracted from the SERIES database [2] for both specimens can be seen in Fig.2-3.

The experimental data obtained, the properties of the tested specimens and the loading protocol were used in the following chapters to create the finite element analysis model and to corroborate the results.

3. The Finite Element Analysis Model

A two-dimensional numerical model for the WR-20 and WR-10 specimens was created using ETABS finite element analysis software [12]. ETABS is one of the most popular commercially available structural analysis software in the engineering industry, due to its easy-to-use user interface, integrated design codes and reliable solver.

For the concrete material, the stress-strain relation for nonlinear structural analysis provided in EN 1992-1-1: Eurocode 2 was used [13], calibrated until the numerical response obtained closely matched the recorded experimental results. The value used for the mean concrete compressive strength was $f_{cm}=34$ MPa, with modified $\varepsilon_{c1}=0.006$ and $\varepsilon_{cu1}=0.0175$ parameters to account for confinement and accommodate the deformation demands imposed by the loading protocol. The concrete tensile strength was not considered, given the early cracking observed during the experiment. A dead load equal to $0.1 \cdot f_c' \cdot A_g$ was applied on top of the wall and maintained constant throughout the analysis. For the hysteresis type, the software's Concrete hysteresis model was chosen[12].

For the rebar material, an elastic perfectly plastic stress-strain relation was chosen, as a simplifying assumption. Since the obtained numerical results displayed a satisfactory match with the recorded experimental data, the absence of steel consolidation was considered acceptable. Two different steel materials were created to match with the two different reinforcement bars that were used in the experiment, each one with a different yield strength: one with a yield strength of $f_y=342$ MPa for the D10 diameter bar and one with a yield strength of $f_y=449$ MPa for the D13 diameter bar. The hysteresis model chosen was the Pivot Hysteresis Model for Reinforced Concrete Members proposed by Dowell *et al.*[14], which, governed by three parameters, captures the nonlinear cyclic behavior of reinforced concrete elements. The α parameters determine the force level of the primary pivot points, which are located on the elastic loading line at a force equal to $F_y \cdot \alpha$, where F_y is the yield force. The β parameters determine the degree of pinching for a ductile flexural response prior to strength degradation. Lastly, the η parameter

determines the level of degradation of the elastic slopes after plastic deformation. Parameters $\alpha_1=\alpha_2=10$ and $\beta_1=\beta_2=0.8$ were determined through calibration to best match the experimental results, thus deviating from the recommended values stated by Dowell *et al.* The η parameter was considered to be null as no significant changes in the analysis results were observed when using different values.

The shear wall was modelled by creating two different element subsections, one to represent the boundary element and another one to represent the web. A two layered shell section was used, with one layer representing the concrete material, with the wall's original 200 mm thickness, and another one representing the longitudinal reinforcement with its thickness equal to the equivalent smeared rebar area. No concrete cover zones were considered since the concrete strength properties were averaged across the whole wall section. The membrane modelling type was chosen for both layers, with material nonlinearity considered only in the direction of the local vertical axis of the element, while the behavior in all other directions was assumed to remain linear. The experiment showed that all specimens experienced flexural failure, thus the transversal reinforcement was ignored. The wall was discretized using a uniform mesh size of 150x150 mm. The resulting finite element model can be seen in Fig. 4, while the configuration of each layer for each wall subsection can be seen in Tables 1-2.

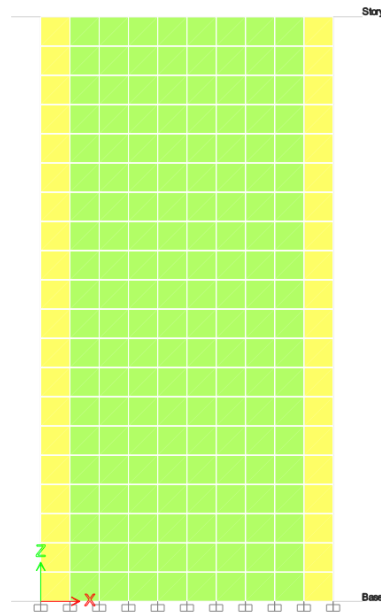


Fig. 4 – Finite element model of the shear wall.

Table 1- Shell-layered configuration for the boundary element

Layer	Distance (mm)	Thickness (mm)	Modelling type	Material	Material Angle(°)	Material Behavior	Material S11	Material S22	Material S12
Concrete	0	200	Membrane	Concrete	0	Directional	Linear	Nonlinear	Linear
Rebar	0	3.6	Membrane	D13Rebar	90	Directional	Nonlinear	Inactive	Linear

Table 2 - Shell-layered configuration for the web

Layer	Distance (mm)	Thickness (mm)	Modelling type	Material	Material Angle(°)	Material Behavior	Material S11	Material S22	Material S12
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Concrete	0	200	Membrane	Concrete	0	Directional	Linear	Nonlinear	Linear
Rebar	0	0.8	Membrane	D10Rebar	90	Directional	Nonlinear	Inactive	Linear

To recreate the cyclic lateral loading protocol, multiple nonlinear static cases were created and chained one after the other. The load application for each case was set to displacement control with the target displacement value corresponding to the target drift ratios used in the experiment. The displacement was measured at the node located on top of the wall. A full cycle is defined as the maximum displacement reaching the positive target value, followed by the negative target value and then returning to the zero point, thus each cycle was divided into three load cases, each one continuing from the one before it. Twenty-five cycles were constructed, corresponding to the maximum number of cycles tested on a specimen, three for every drift ratio except for the last one. The load case configuration can be seen in Table 3.

Table 3 - Load case configuration

Corresponding Cycle Number	Load case	Scaling Factor	Target Displacement (mm)	Corresponding Drift Ratio
1	Positive 1A	+1	5	1/600
	Negative 1A	-1	10	
	Neutral 1A	+1	5	
2	Positive 1B	+1	5	
	Negative 1B	-1	10	
	Neutral 1B	+1	5	
3	Positive 1C	+1	5	
	Negative 1C	-1	10	
	Neutral 1C	+1	5	
	Negative 2C	-1	15	
	Neutral 2C	+1	7.5	
...	...	...	...	...
22	Positive 8A	+1	60	1/50
	Negative 8A	-1	120	
	Neutral 8A	+1	60	
23	Positive 8B	+1	60	
	Negative 8B	-1	120	
	Neutral 8B	+1	60	
24	Positive 8C	+1	60	
	Negative 8C	-1	120	
	Neutral 8C	+1	60	
25	Positive 9A	+1	90	1/33

	Negative 9A	-1	180
	Neutral 9A	+1	90

4. Numerical Results and Comparison with Experimental Data

The nonlinear analysis was conducted, and the following hysteresis curve was obtained with its approximated backbone curve, both of which can be seen in Fig. 5.

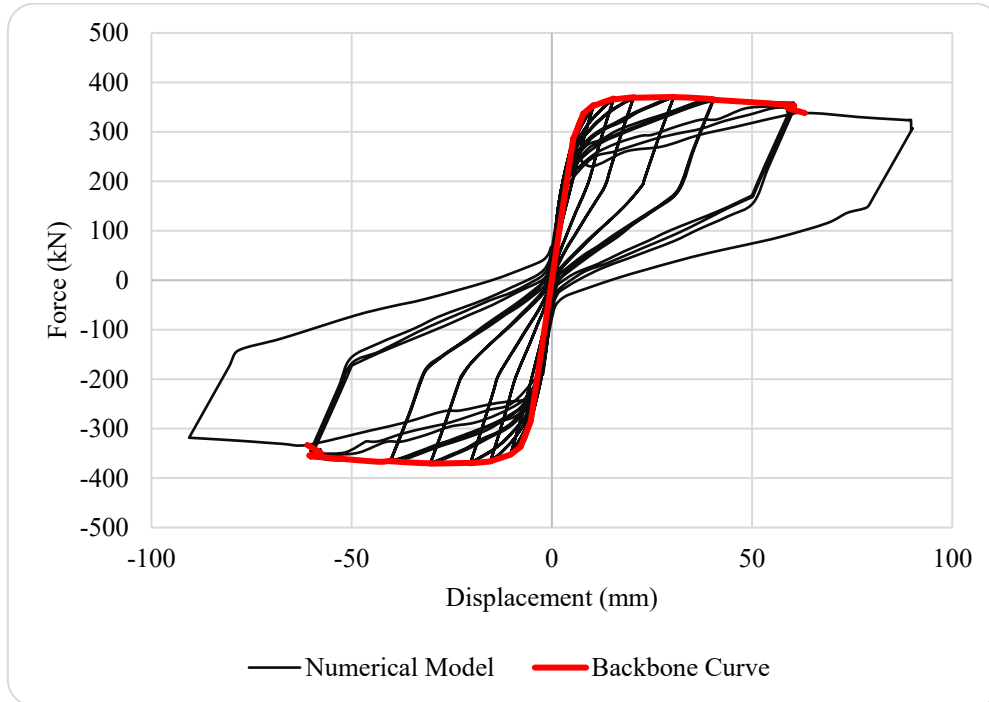


Fig. 5 - The load versus displacement curve obtained for the model and its associated backbone curve.

At the smallest drift ratio ($\frac{1}{600}$), the wall develops a peak force of 285 kN, while no measurable stiffness degradation can be seen. As the shear wall passes through the following five drift ratios ($\frac{1}{400}$, $\frac{1}{300}$, $\frac{1}{200}$, $\frac{1}{150}$, $\frac{1}{100}$), the peak force reaches 370 kN, the maximum value obtained in the analysis in both positive and negative directions, with a noticeable reduction in the wall's secant stiffness. At larger drift ratios ($\frac{1}{75}$, $\frac{1}{50}$), strength degradation begins to occur, with the peak strength dropping to 365 kN. The maximum lateral force obtained in the last cycle ($\frac{1}{33}$) is 338 kN, which shows a reduction of 9.4% from the maximum recorded force. The reinforcement layer in the boundary elements begins to yield when the lateral force is equal to 336 kN and the displacement equal to 7.86 mm, values which were later used to normalize the energy dissipated by the wall. Compared to the experiment, the yield force of the model is between the recorded forces of 312.6 kN for WR-10 and 348.9 kN for WR-20, while the yield displacement falls short of both recorded values, 9.3 mm for WR-10 and 12.4 mm for WR-20.

The numerical model reproduced the experimental displacement range for both specimens. Overlapping the experimental hysteresis loops obtained from the SERIES database [2] with the curve created from the numerical model (seen in Fig.6-7) shows that the general shape of the curve obtained from the model is comparable to the ones obtained from the experiments. All three exhibit an initial loading stiffness, a decreasing secant stiffness as the wall is pushed through higher drift ratios and a steep unloading

curve upon load reversal. When near zero displacements, the model experiences a pronounced degree of pinching, more accentuated than that observed in both experiments, showcasing that the pivot model has the capacity to capture this behavior, even though it is overestimated in this case.

Another qualitative assessment can be obtained by overlapping the experimental cyclic response with the backbone curve obtained from the numerical analysis, as shown in Fig. 8-9. The backbone curve is constructed by connecting the peak lateral force values and their associated displacement recorded at each successive cycle in both loading directions.

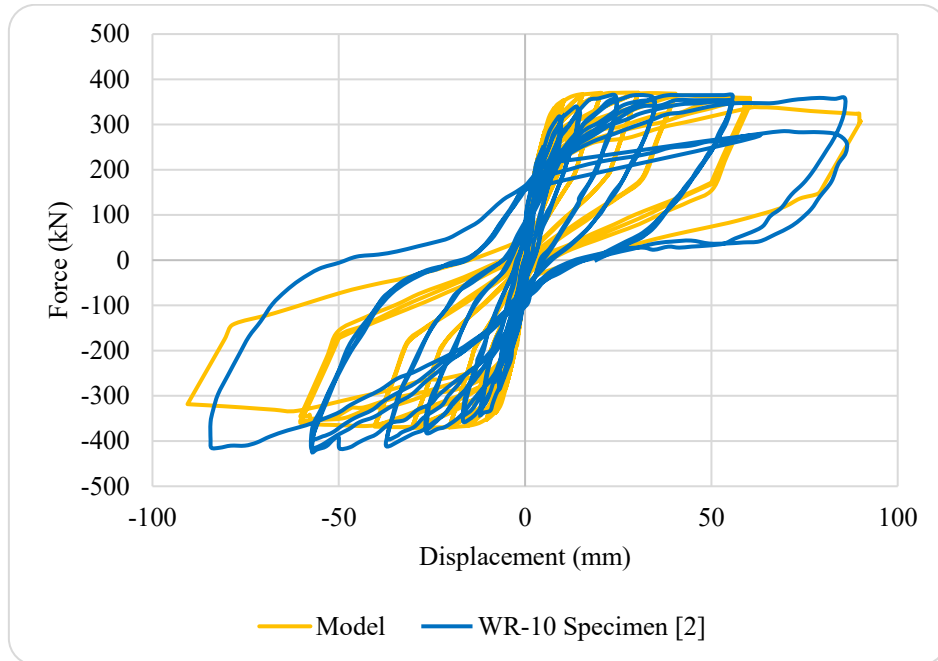


Fig. 6 – Comparison between the hysteresis curves obtained for the WR-10 specimen and the model.

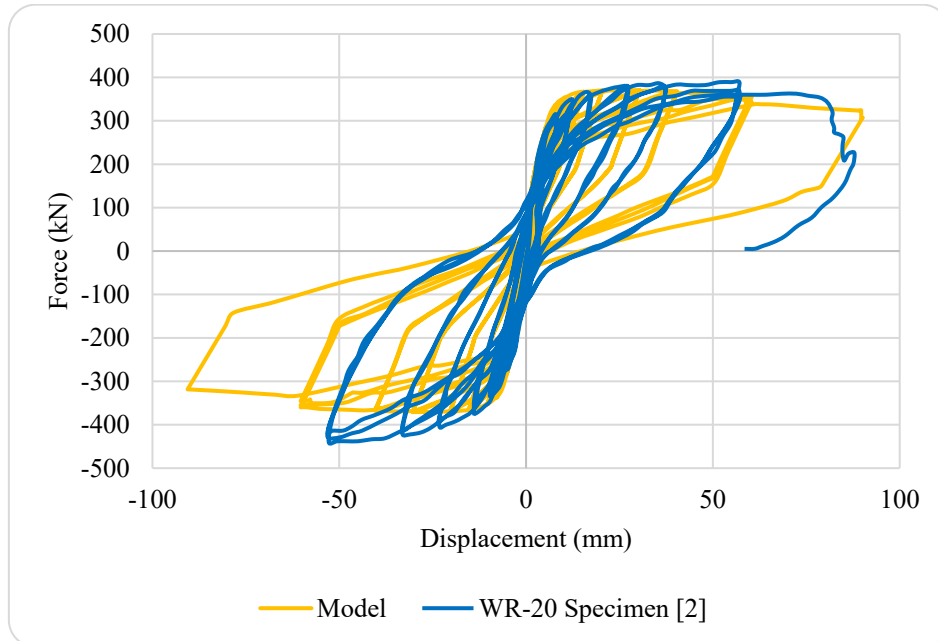


Fig. 7 – Comparison between the hysteresis curves obtained for the WR-20 specimen and the model.

The model follows closely the positive loading direction hysteresis loops, capturing the initial hardening response in the first cycles, as well as the wall's strength degradation in the last cycles. The peak lateral force predicted by the model of 370 kN is within

range of the positive peak recorded values of 369.5 kN for WR-10 and 386.1 kN for WR-20. In the negative loading direction, the lateral force is consistently underestimated with it reaching 370 kN, just as in the positive loading direction, while the recorded peak is 425.3 kN for WR-10 and 442.9 kN for WR-20[1]. This is a consequence of the numerical model's inherent symmetry which translates into a fully symmetric response for both loading directions, unlike the experimental asymmetric hysteresis curves. As discussed by Dowell *et al.*, the Modified Pivot Model, which is activated by taking the η parameter greater than zero, has the capacity to create an asymmetric response even for geometrically symmetric elements, by degrading the elastic slopes after plastic incursions. The use of $\eta=0$ in the present model may therefore contribute to the symmetrical numerical response obtained, however no significant changes were observed when non-zero values of η were tested.

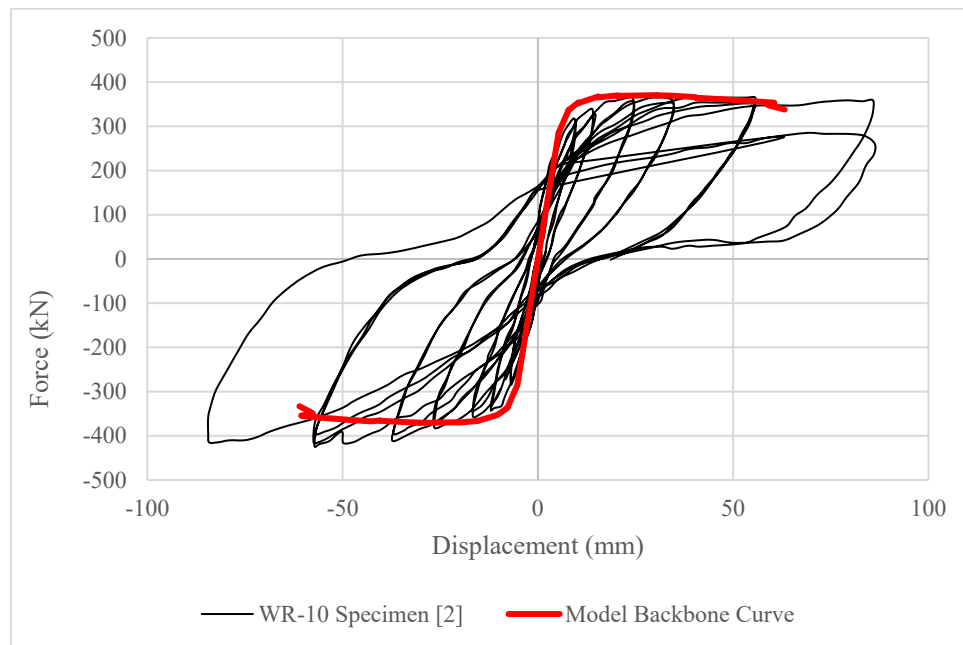


Fig. 8 – Comparison between the hysteresis curve obtained for WR-10 and the model's backbone curve.

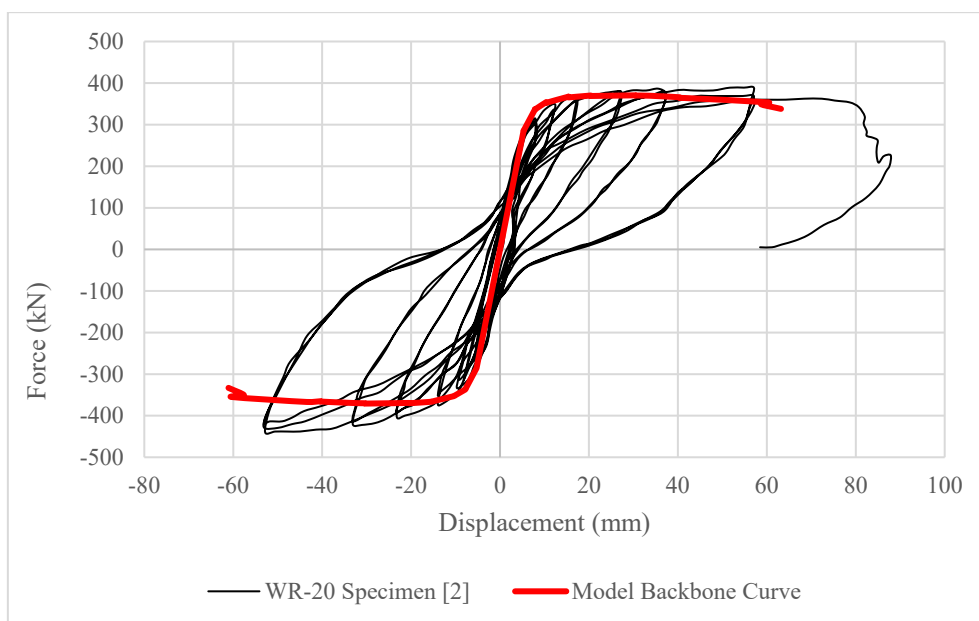


Fig. 9 – Comparison between the hysteresis curve obtained for WR-20 and the model's backbone curve.

As previously stated, the force-displacement curves from the SERIES database did not contain the full loading protocol applied during the experiment. For the WR-10 specimen, 15 complete cycles were identified, while for the WR-20 specimen 13 complete cycles and one incomplete one. Thus, from the recorded cycles contained within the database, a representative few were chosen to be compared individually with matching cycles conducted in the numerical analysis. The process used for selection consisted of identifying the target drift ratio of each cycle, while accounting for overshooting of the recorded displacement, and comparing the maximum lateral force between cycles with the same target drift ratio. The target drift ratios identified for the WR-10 and WR-20 specimens are $\frac{1}{600}$, $\frac{1}{300}$, $\frac{1}{200}$, $\frac{1}{100}$, $\frac{1}{75}$, $\frac{1}{50}$, $\frac{1}{33}$ and $\frac{1}{400}$, $\frac{1}{300}$, $\frac{1}{200}$, $\frac{1}{150}$, $\frac{1}{100}$, $\frac{1}{50}$, $\frac{1}{33}$, respectively. The drift ratio $\frac{1}{33}$ cycle for WR-20 is the last recorded cycle and it contains the failure of the specimen, thus only positive values were recorded.

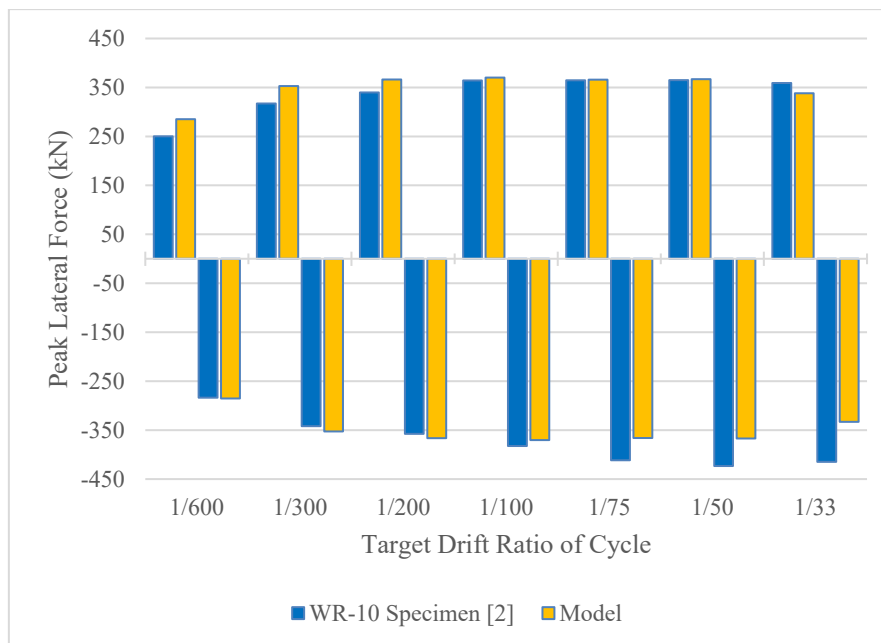


Fig. 10 – Comparison between the peak lateral forces obtained for the WR-10 specimen and the model.

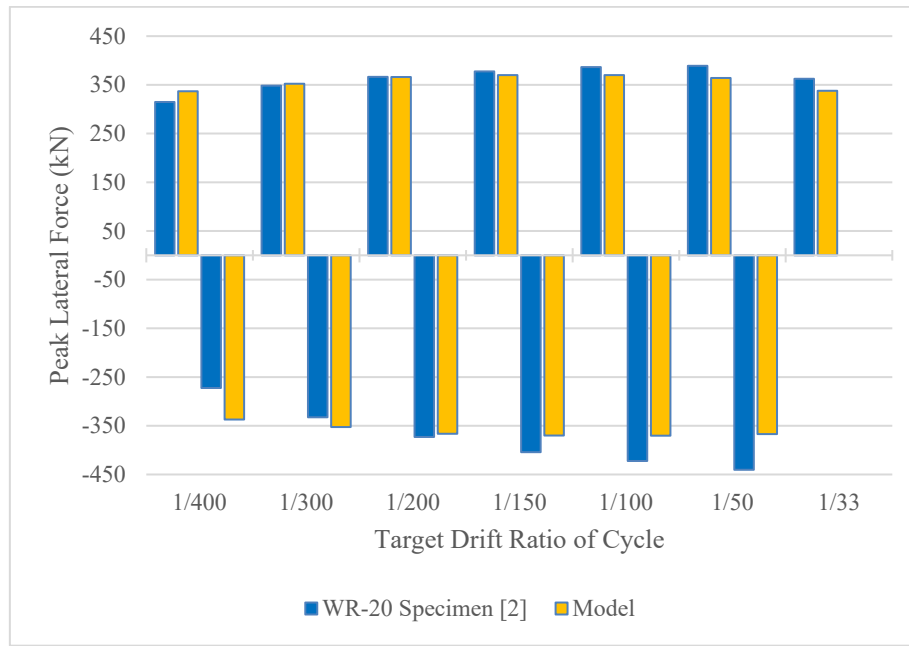


Fig. 11 – Comparison between the peak lateral forces obtained for the WR-20 specimen and the model.

A comparison between the peak lateral force of each chosen cycle for both specimens and the model is shown in Fig. 10-11. The model matches closely with both specimens throughout all selected cycles in the positive loading direction, with the predicted peak force of 370 kN being almost identical to the peak force found within the dataset for the WR-10 specimen of 365 kN, and within 4% difference of the peak force for the WR-20 specimen of 389 kN. For WR-10 the error range is from 0.7% to 13.9%, while for WR-20 that range is from 0.2% to 7.2%. Reinforcing what was shown in the previous comparisons, in the negative direction the model reaches a peak lateral force smaller than what was recorded for the experiments in each selected cycle. The recorded peak values are 423 kN for WR-10 and 440 kN for WR-20, with errors ranging from 0.6% to 24.5% for WR-10 and from 1.8% to 23% for WR-20.

The area of each enclosed loop represents the energy dissipated by the wall throughout each push and reverse motion. To determine the energy dissipated in each cycle, the Gauss area formula is applied to force-displacement data points of each loop. Summing up the obtained value for a cycle with the values acquired from previous cycles results in the cumulative dissipation energy, which is then normalized by the product of the lateral force and displacement recorded when the boundary reinforcement experiences yielding ($V_y \Delta_y$), and plotted against the number of cycles to illustrate the total dissipated energy throughout the analysis (seen in Fig.12). This procedure is consistent with the definition of normalized energy dissipation capacity used in the article, thus allowing for a direct comparison between the model and the specimens. As stated by Oh *et al.*, the energy dissipation of the WR-20 and WR-10 specimens follows a similar trend up until the 21st cycle, where WR-10 begins to display a large increase in dissipated energy, due to denser transversal reinforcement in the boundary elements[1]. Compared to the experimental data, the numerical model begins to overestimate the dissipated energy after the 12th cycle, keeping a relatively constant difference in capacity between it and the WR-10 specimen until the last cycle of the analysis. Checking the damage state of the 12th cycle reveals that the steel layer in the tensile boundary element has fully yielded, while the concrete strain in the

compressed boundary element is still below the crushing value. The overestimation of cumulative dissipated energy is caused by the model's inability to capture the asymmetric degradation of the structural response.

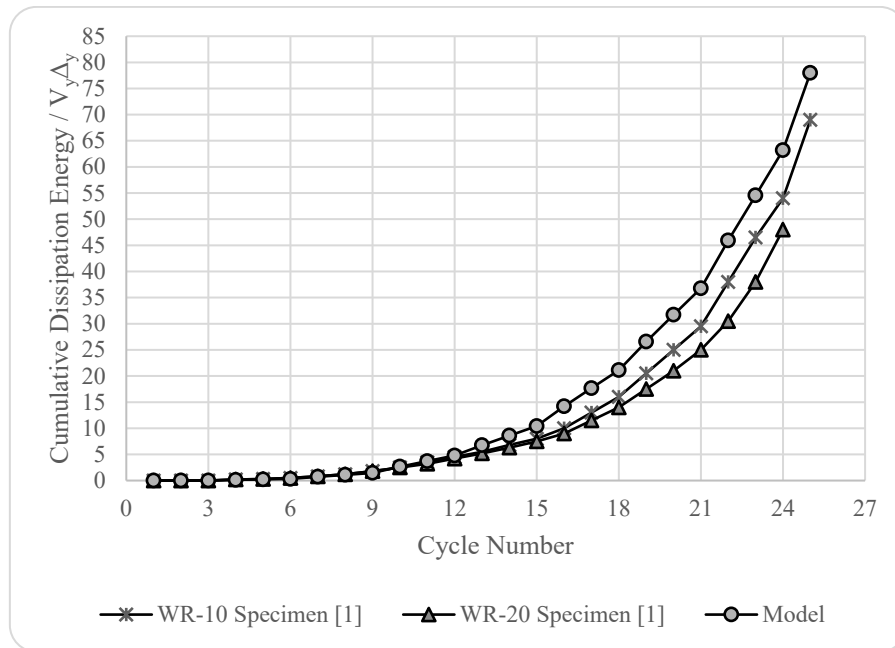


Fig. 12 – Comparison between the normalized cumulative dissipation energy of the model against the WR-10 and WR-20 specimens.

5. Conclusions

This paper presented a validation study for the use of the layered shell finite element to model reinforced concrete shear walls under cyclic lateral loading in the finite element software ETABS. Two wall specimens, designated as WR-10 and WR-20 from the experimental study conducted by Oh *et al.* were chosen as the basis for validation. Both walls had the same longitudinal reinforcement, with the difference between them being the content of transversal reinforcement.

The numerical model was created with a two-layered shell section finite element, with separate steel and concrete layers. The Eurocode 2 stress-strain relation for structural analysis with calibrated parameters to match the experimental response was used for the concrete material, while an elastic perfectly plastic model was used for the steel material. The hysteresis models chosen for the materials were the software's Concrete hysteresis model for the concrete material and the Pivot Hysteresis Model for Reinforced Concrete Members for the steel.

The cyclic lateral loading protocol was created using a series of chained static nonlinear load cases with load application set to displacement control. Twenty-five complete cycles of various drift ratios were applied to the model.

The results were compared with data extracted from the SERIES database, which, due to it being obtained from the digitized curves published by the authors of the experiment, did not contain the entire loading protocol used for the experiment. Thus, representative cycles were selected and peak values obtained within said cycles were compared to values obtained from the numerical model. The error range obtained in the positive direction for both specimens was relatively small, while in the negative direction the recorded data began to diverge significantly from the predicted response.

The hysteresis loop shape produced by the model is similar to that of the experiments, with a higher degree of pinching. The model predicted the peak lateral force in the positive loading direction accurately, while it underestimated it in the negative loading direction. The cumulative dissipation energy predicted by the model is close to what was experimentally observed up until the 12th cycle, after which the model begins to overestimate the dissipation capacity.

Overall, the chosen modelling approach demonstrated a reasonable capacity to approximate the nonlinear cyclic response of the reinforced concrete walls tested by Oh *et al.*, provided that the material parameters are carefully selected. The iterative calibration of the model against the experimental results provided a clear path for parameter selection.

The initial objective of assessing the capacity of the shell layered elements in ETABS to reproduce the nonlinear behavior of reinforced concrete shear walls subjected to cyclic loading was achieved. More accurate results could be achieved by using other advanced software with greater modelling capabilities. While the force-displacement results obtained in this study could be replicated by using a link element or a nonlinear beam-column element with the Pivot hysteresis model, the shell layered approach presents the opportunity to extend the research into stress and strain distributions across the length and height of the wall. The discrepancies observed between the experimental and the numerical results are most plausibly attributed to the selection of the Pivot hysteresis model parameters, rather than second order effects or axial-shear force interaction, which were not investigated in this study. When reproducing the result of the experimental investigation of the specimens studied in this paper, or others, particular attention should be given to the α and β parameters of the Pivot

hysteresis model, as their values govern the overall hysteretic response. In the absence of experimental results with which to calibrate the numerical model, the recommended values by Dowell *et al.* should be used.

Further investigations within ETABS should examine the influence of different mesh sizes, stress-strain curves and hysteresis models for the materials on the accuracy of the predicted response. In particular, determining the material parameters, while following the recommendations of Dowell *et al.*, would assess whether the obtained results remain as constrained as in this present study.

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