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Numerical modeling of the cyclically loaded FRP-retrofitted RC shear walls

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Abstract: Fiber-reinforced polymer (FRP) composites are now becoming widely accepted as a viable alternative for seismic retrofit of Reinforce Concrete (RC) shear walls. Although testing physical models of FRP-retrofitted RC shear walls would be ideal to evaluate their seismic performance, numerical models could simulate the actual behaviour without the time and cost implications of laboratory tests. A reliable simulation must consider various parameters in the analysis namely; the mechanical behaviour of materials, the interaction between different elements of the assembly, the loading protocol, and the boundary conditions. Finite Element (FE) modeling of RC shear walls before and after retrofitting by external CFRP wraps under cyclic loading histories is presented in this paper using the general purpose FE package ANSYS. The calibration of the FE model is based on the results of experimental tests available in the literature. SOLID65 element from the element library of ANSYS is used to model the concrete, discrete reinforcement bars modeled by LINK180 element in different regions of the model geometry, FRP layers are modeled by tension-only LINK180 element and the bond interface between FRP and concrete is modeled by COMBIN39 element as multi-linear springs based on the most recent interface models available in the literature. The analysis results such as lateral force-displacement relationships, the cracks distribution layout, the stresses in reinforcement bars and FRP layers, and the behaviour of the bond interface are discussed.

1 Introduction

Reinforced concrete (RC) shear walls are structural elements carrying both gravity loads and the lateral loads generated in the structure by wind and earthquake. Although the high lateral stiffness of RC shear walls reduces the inter-story drifts as well as the overall lateral deflection of the structure, limited lateral capacity and ductility, mainly due to poor reinforcement design and detailing of these structural elements could result in sudden failure of the structure due to brittle behaviour. High seismic performance in terms of earthquake energy dissipation and ductility is one of the key factors to be satisfied in the design and detailing of RC shear walls under the provisions of the modern capacity design and performance-based seismic design philosophies. Amongst various retrofit methods to upgrade existing RC shear walls with unsatisfactory seismic performance, the use of Fibre-Reinforced Polymer (FRP) composites is becoming more viable considering the light weight and high strength of these materials. Several researchers have studied experimentally the application of externally-bonded FRP composites on RC shear walls (Lombard et al. 2000, Antoniades et al. 2003, Peterson and Mitchel 2003, Ghobara and Khalil 2004, Hiotakis et al. 2004, Hwang et al. 2004, Elnady 2008, Li and Lim 2010 and El-Sokkary et al. 2012), yet there is still a need for more research in this area. An acceptable retrofit scheme for RC shear walls using externally bonded FRP materials is expected to be based on solid experimental and/or analytical results. Experimental tests on RC shear walls are costly and time consuming regarding the testing equipments and the amount of effort required. An analytical approach that models the behaviour of RC shear wall elements and takes into account various failure criteria would be a complimentary tool for the research in order to reduce the amount of experimental data and replace some tests with analytical modeling. An analytical modeling approach would be valid if its results correlate with the results of experimental tests within a reasonable and acceptable margin of error.

Modeling and analysis of RC shear walls could be categorized into two main categories namely, micro-modeling and macro-modeling where the choice of each approach is based on the purpose of the analysis. In micro-modeling, details of a structural element are modeled in a precise way to represent the actual conditions; hence, a fine mesh is used and the size of FE model is not generally greater than a single member or a local assembly. On the other hand, macro-modeling uses simplified elements in terms of axial and/or rotational springs and dashpots to capture the nonlinearity of the members by using the appropriate micro-models or experimental tests results to simulate a global structure assembly. Modeling RC elements involves various considerations in order to simulate the nonlinear behaviour of the composite structure of reinforced concrete. The first significant attempt on the 3D modeling of RC shear walls was the work conducted by Sittipunt & Wood (1993) in which various available material models and failure criteria were explained and a simple modeling approach were introduced based on the defined failure criteria to model a total of 13 wall specimens from previous experimental tests. A FE analysis program "FINITE" was written for the purpose of the study; the failure criteria and material models used in this study were used by most of the fellow researchers afterwards as well as the current paper. Vecchio (1999) used planar elements to model a slender wall panel in order to simulate an experimentally tested specimen under cyclic loading. Li et al. (2005) modeled a RC flanged shear wall (I-shaped) strengthened with Glass Fiber Reinforced Polymer (GFRP) by 3D elements using ABAQUS general purpose FE package in order to predict the cyclic behaviour of shear wall structures. Palermo and Vecchio (2007) modeled RC shear walls using planar elements under reversed cyclic loading by VecTor2 (Wong and Vecchio, 2002) software package. Khomwan et al. (2010) developed a nonlinear FE model for the analysis of RC plane stress members strengthened by FRP external sheets under monotonic and cyclic loading. The bonding interface between FRP and concrete surface was taken into consideration using a two-dimensional membrane contact element in order to capture the debonding failure mechanism at the interface between concrete surface and FRP sheets. The use of VecTor2 software package became popular recently for the analysis of shear walls using planar 2D elements regarding the simple ready-to-use material models and loading packages available in the recent versions of it. Cortes-Puentes and Palermo (2012) modeled a total of four shear wall specimens from the experimental works available in the literature where one of them was retrofitted by FRP external bonding. Cruz-Noguez et al. (2012) also modeled the same experimental work with the main concern on the debonding of FRP from the surface of the wall as a significant failure criterion.

This paper presents the modeling of RC shear walls using the general purpose FE package ANSYS under reversed cyclic and monotonic loading and discusses some modeling considerations to be taken into account in future research.

2 Analysis methodology

Each RC shear wall model in this paper consists of four main elements namely concrete, steel rebars, FRP layers and bonding interface. Each element follows specific failure criteria in terms of stress-strain relationships and special plastic behaviour where applicable. Fig.1 shows the elements arrangement for a typical FE mesh in this paper. The analysis is based on a displacement control solution of the model under incremental displacements on the top of the wall using Newton-Raphson approach provided in ANSYS (ANSYS, 2010-b). Any physical term i.e. force, moment, displacement, etc. applied externally to the model is considered to be a *loadstep* in ANSYS. Newton-Raphson method guarantees the convergence of nonlinear problems if and only if the solution at any iteration is within a tolerance from the exact solution; therefore, applying the loads slowly i.e. subdividing each loadstep into a series of load increments i.e. *substeps* helps the convergence but increases the analysis time depending to the number of substeps. A separate analysis must be performed for each loadstep where several loadsteps may exist in the model. In order to analyze the RC assemblies where a combination of materials with various nonlinearities exist, the stiffness matrix must be updated after each converged substep (Full Newton-Raphson approach) which leads to more accurate results comparing to the traditional Newton-Raphson method but sacrifices the analysis time consumption. ANSYS offers a solve option wherein the Full Newton-Raphson approach is used only when necessary in order to optimize the analysis precision and time consumption (ANSYS, 2010-a).

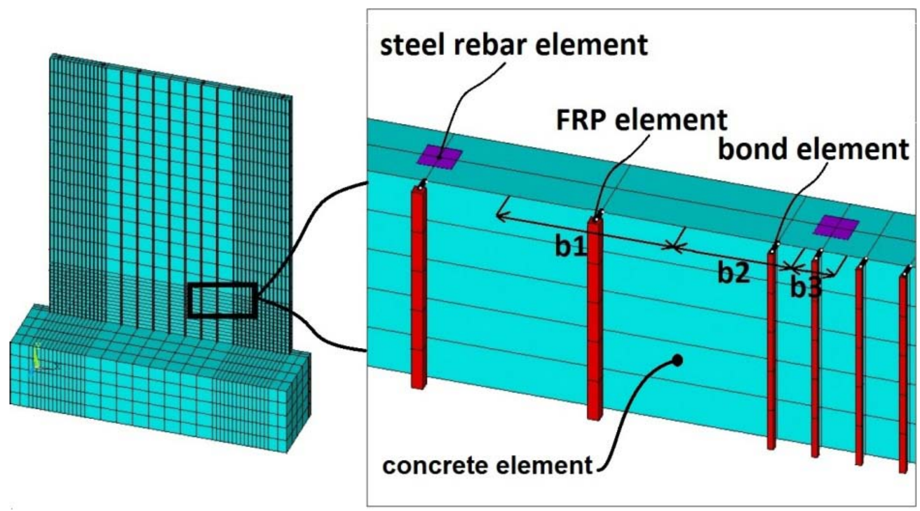

Figure 1: Meshing of a RC shear wall FE model

3 Modeling and verification

A total of two experimentally tested specimens from the work of Lombard et al. (2000) are modeled in this paper; a control wall (CW) without any FRP reinforcement and a strengthened wall (SW1) with one layer of vertically directed FRP bonded to the surface of the wall. The geometry and reinforcement arrangements are identical for the two modeled specimens as shown in Fig.2. The properties of concrete for the wall specimens are shown in Table.1.

Table 1: Concrete cylinder test results (Lombard et al., 2000)

Specimen	Age at test (Days)	28 days f'_c (MPa)	28 days f_r (MPa)	f'_c at test (MPa)
Control Wall #1 (CW)	558	40.5	3.0	40.2
Strengthened Wall #1 (SW1)	328	34.9	3.0	42.0

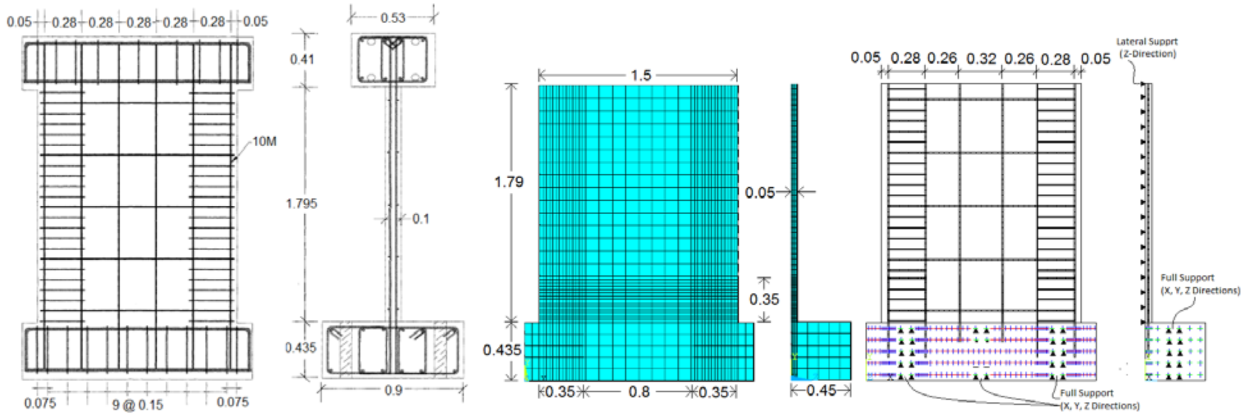

Figure 2: Geometry and reinforcement details in experimental specimens (Lombard et al. 2000) and FE models

The material properties used in the modeling are based on the values obtained in the experimental results of Lombard et al. (2000) for the stress-strain relationship of concrete and steel as shown in Figure 3. The test specimens were consisted of three main parts namely, the top block, the wall block and the bottom block. The top and bottom blocks were heavily reinforced concrete elements to connect the wall block into the actuators and the rigid floor respectively. Considering the method of load application used in this paper, the top block was not included in the models geometries but the bottom block was modeled in order to simulate the load transfer between the wall block and the supports. The specimens modeled in this paper are symmetric along the plane of the wall, thus only half of the specimens are considered in the modeling.

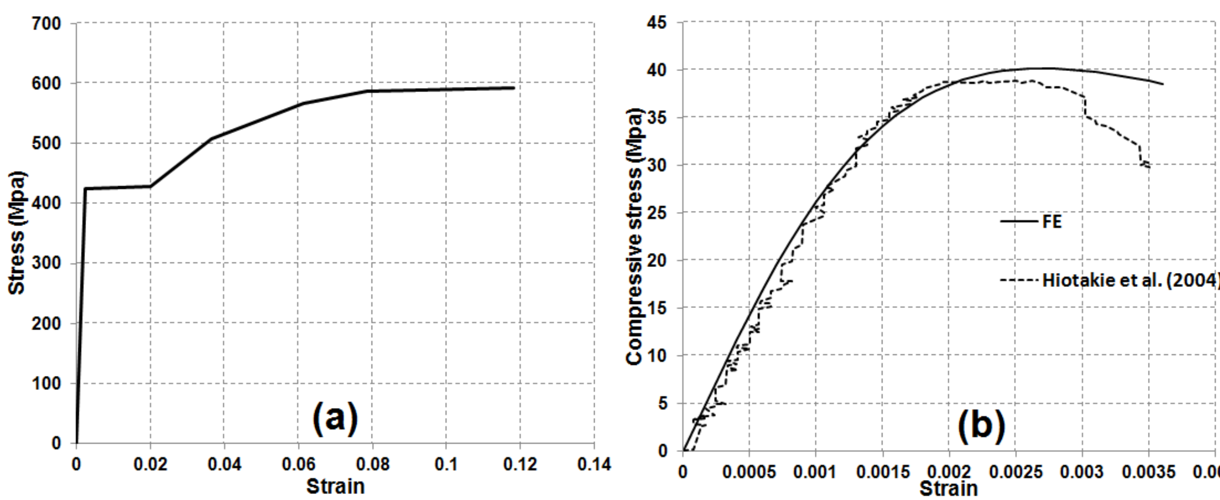

Figure 3: Stress-strain relationship for (a) reinforcement steel (b) concrete

The element used for the modeling of steel reinforcement and FRP external wraps is LINK180 from the elements library of ANSYS (ANSYS, 2010-b) which is consisted of a linear spar with two nodes with one transitional degree of freedom at each node in order to simulate the axial behaviour . The failure criteria for steel reinforcement consists of the combination of two material models namely, linear isotropic and multilinear kinematic simulating the stress-strain relation shown in Figure 3(a). The failure criteria of concrete material is consisted of the combination of three material models from the materials library of ANSYS namely, linear isotropic, multilinear isotropic and concrete smeared cracking model; the first two material models simulate the uniaxial compressive stress-strain relation shown in Figure 3(b) and the third one introduces the cracking and crushing phenomena at each of the 8 integration points of the SOLID65 element used for the modeling of concrete in this paper. The FRP material is considered to follow a linear up to failure behaviour during the analysis, thus the failure criteria for this element consists of a modulus of elasticity and a limit for the tensile uniaxial stress and strain. The bond interface between concrete and the embedded reinforcement bars is considered to be rigid in this study. The bond-slip between the FRP external wraps and the surface of concrete is defined in this paper using the COMBIN39 element in order to simulate the bond-slip model proposed by Lu et al. (2007) which is mainly based on the tensile strength of concrete and found to be the most accurate estimation of the actual bond-slip behaviour of RC beams strengthened with FRP sheets taking into account the intermediate (IC) cracking phenomenon (Ombres, 2010).

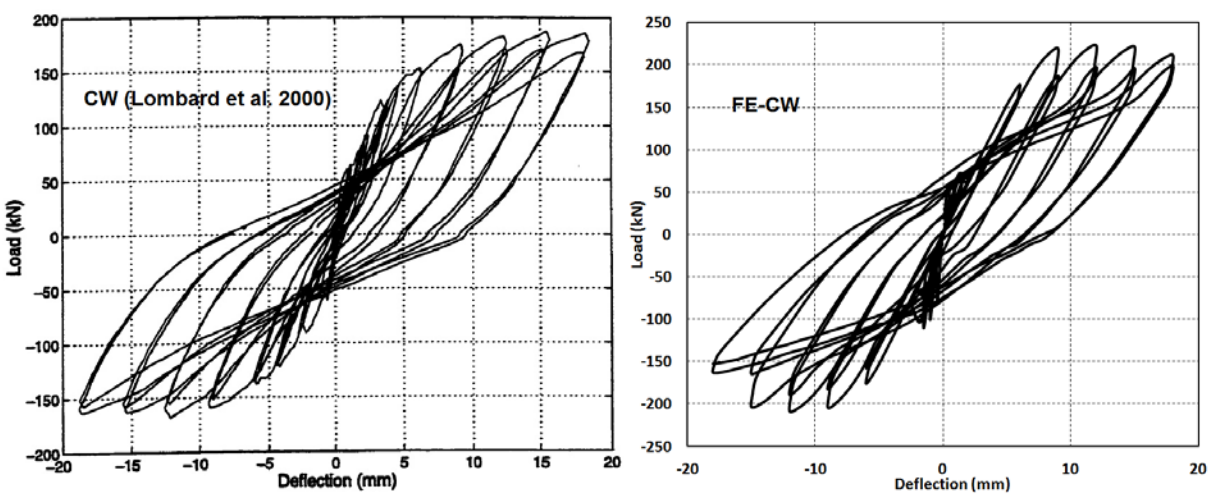

Figure 4: Lateral load-Top deflection curves of CW (Lombard et al. 2000) and the FE model

Figure 4 shows the lateral load-top deflection curve resulted from the FE modeling compared to the experimental results. Lombard et al. (2000) reported the first flexural cracks for the control wall (CW) at load 49.6kN in the "push" direction and 60kN in the "pull" direction formed near the edge of the wall at the construction joints. First flexural cracks appeared in the FE model at 55.6kN load in global +X ("push" direction) and 55.7kN in global –X ("pull" direction) at the bottom corners of the wall as shown in Figure 5(a). First diagonal cracks reported to occur in the third load step of the test at ±90kN for the CW specimen. The appearance of diagonal cracks started in the FE model at 99.5kN in +X direction and 98.7kN in –X direction as shown in Figure 5(b). The ultimate load carried by the tested wall was reported to be 187.1kN in the "push" direction comparing to the 223.26kN ultimate load achieved from the FE model with a crack pattern shown in Figure 5(d) indicating some crushed elements at the two tows of the wall as well as in the mid-height of the wall. Yielding of the extreme vertical layer of reinforcement was reported to be obtained from the load-deflection curve in the experimental work (Lombard et al. 2000). It was reported that the yielding of the extreme layer of reinforcement occurred at the load levels of +122.7kN with a top displacement of +3.4mm and -122.1kN with a top displacement of -4.2mm. An average yield load and displacement of 122.4kN and 3.8mm was reported respectively. Yielding stress occurred in the extreme layers of reinforcement steel at +129.1kN load and -128.05kN as shown in Figure 5(c) with respective displacements of +3.82mm and -4.54mm. The average yield load and displacement from the FE analysis are 128.6kN and 4.18mm.

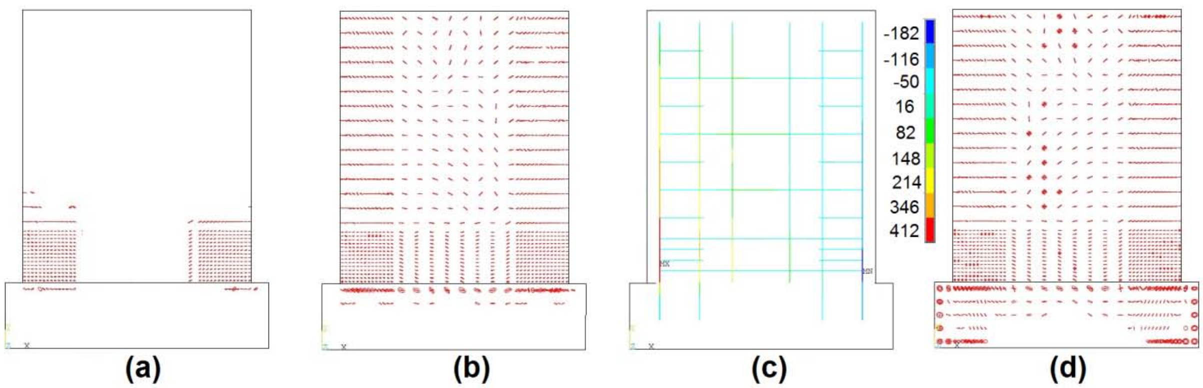

Figure 5: Results of the FE analysis of FE-CW model

CFRP sheets used in the experimental tests of Lombard et al. (2000) were unidirectional with a vertical fiber direction. In order to best simulate the CFRP sheets applied to the surface of concrete elements based on the methodologies explained previously, the tributary width for each FRP element is calculated for various parts of the wall model based on the meshing specifications as shown in Figure 1 in which b_1, b_2, and b_3 are the tributary widths in different regions of the wall considering the element mesh sizes. The cross-sectional area of each LINK180 element is calculated based on its appropriate tributary width respectively in the modeling. In order to connect the FRP sheets to the wall base and provide the appropriate anchorage, Lombard et al. (2000) used an anchoring device as shown in Figure 6 which was consisted of an angle profile bolted to the bottom block of the specimen.

A full end anchorage is assumed in the modeling for the FRP sheets by coinciding the bottom nodes of FRP and the bottom block. This is based on the full anchoring provided by the anchorage system as stated in the tests by Lombard et al. (2000).

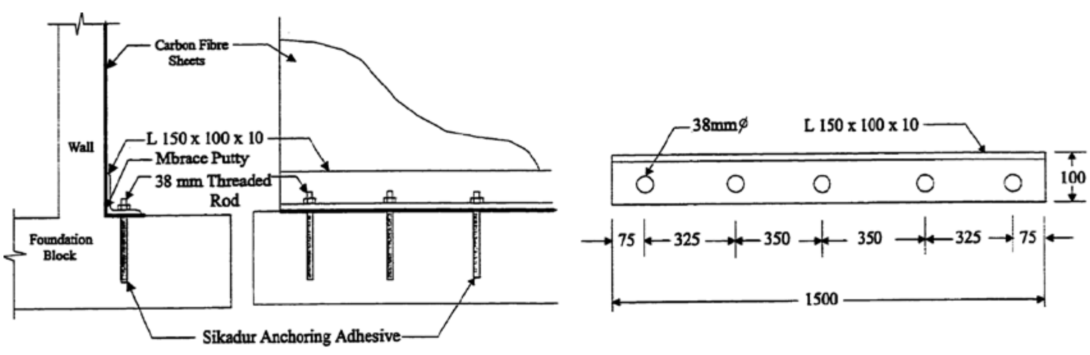

Figure 6 : FRP anchor device (Lombard et al. 2000)

The bond interface between the FRP wrap and the concrete surface was modeled using the tributary area of each node with the same procedure used for the FRP elements based on Lu et al. (2007) model as shown in Figure 7 by 10-points estimations. The tributary areas and FRP cross-sectional areas at various regions of the wall are shown in Table 2. The wall surface was first prepared by applying epoxy putty to flatten any possible unevenness in the concrete surface two-to-three weeks prior to the application of CFRP sheet in the experimental works; in addition, a coating of epoxy primer was also applied to the wall surface one day before the main retrofitting application. The CFRP sheets were placed into wet saturant and were bonded to the wall using a coat of epoxy saturant, where a ribbed roller was used to remove any air bubbles trapped behind the CFRP sheets to ensure proper bonding (Lombard et al. 2000). The mechanical properties of the CFRP sheets and the bonding materials are shown in Table 3.

Table 2: FRP and bond elements real constants

Region	b_i (mm)	h_i (mm)	A_i (mm^2)	A_{FRP} (mm^2)
1 (wall web)	100	96	9600	11
2 (wall edges)	25	96	2400	2.75
3 (bottom of web)	100	25	2500	11
4 (wall toes)	25	25	625	2.75
1 , 2 border	62.5	96	6000	6.78
3 , 4 border	62.5	25	1562.5	6.78
1 , 3 border	100	60.5	6050	-
2 , 4 border	25	60.5	152.5	-

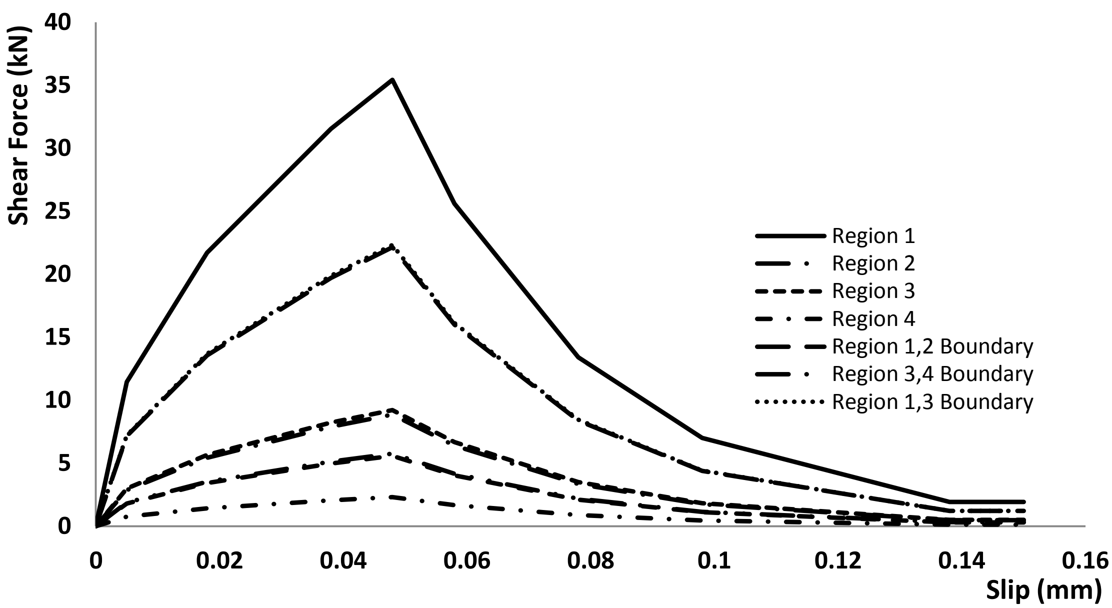

Figure 7: Bond-Slip relationships for various regions of the wall model FE-SW1

Lombard et al. (2000) stated that since the observation of cracks was not possible in the experimental tests due to the presence of the FRP layers on the surface of the wall, the crack analysis was done based on estimation by referring to the load-displacement curves. The first flexural cracks were reported to appear at +97.1kN and a top displacement of +0.7mm in the “push” direction and at -105.0kN with a top displacement of -0.6mm in the “pull” direction. The first cracks appeared in the FE model are shown in Figure 8. The cracking load and top displacement were +103.18kN, +0.5mm, -110.02kN, and -0.6mm respectively.

Table 3: Material properties of FRP and the bond interface (Lombard et al. 2000)

Material	Tensile Strength (MPa)	Tensile Modulus (GPa)	Shear Strength (MPa)	Ultimate tensile strain (%)	Thickness (mm)
Epoxy Putty	12	1.8	26	N/S*	N/S
Epoxy Primer	12	0.717	24	N/S	N/S
Epoxy Saturant	54	3.034	124	N/S	N/S
Epoxy Resin	20-40	1-10	15-35	N/S	N/S
FRP Sheet	4800	230.5	N/S	1.7	0.11

**N/S: Not Stated*

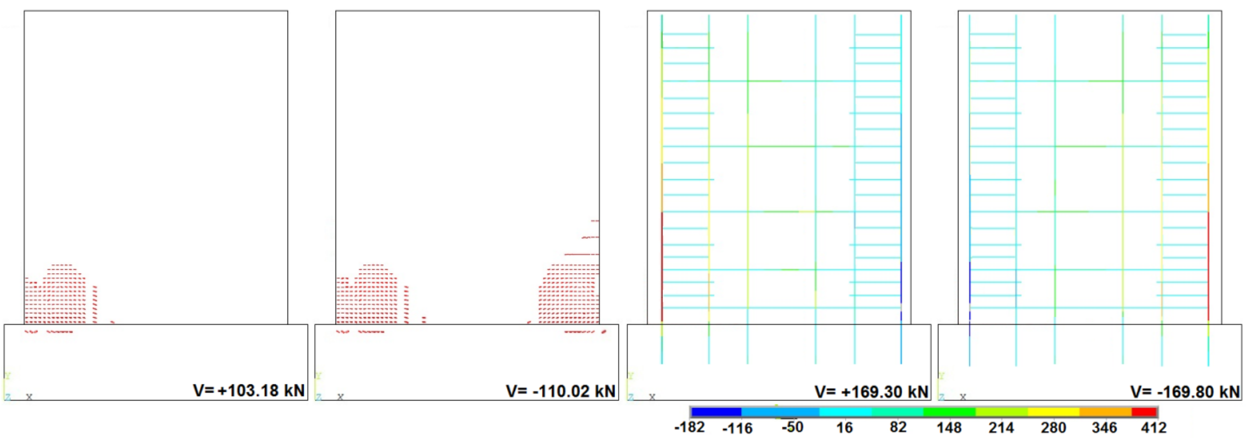

Figure 8: First flexural cracks and the rebars axial stress at yield load for FE-CW model

The yielding of extreme layer of reinforcements was reported to occur at +139.1 kN with a top displacement of +1.5mm in the “push” and -167.1kN with -1.7mm displacement at the top of the wall specimen for the experimental tests while the yielding occurred at +169.3kN and -169.8kN with +1.67mm and -1.68mm top displacements respectively in the FE analysis as shown in Figure 8. The ultimate load carried by the wall in the experimental tests of Lombard et al. (2000) was reported to be 246.6 kN with a final displacement of 29.1mm after degradation of the load carrying capacity during the final load steps. The maximum load calculated to be 337.8kN for the FE model in the “push” direction with 29.1mm of top deflection. The load-deformation curves of the SW1 specimen are displayed in Figure 9 as results of experimental tests of Lombard et al. (2000) and FE analysis. A comparison between the results of FE analysis and experimental tests is presented in Table 4 where a good agreement between the results is achieved.

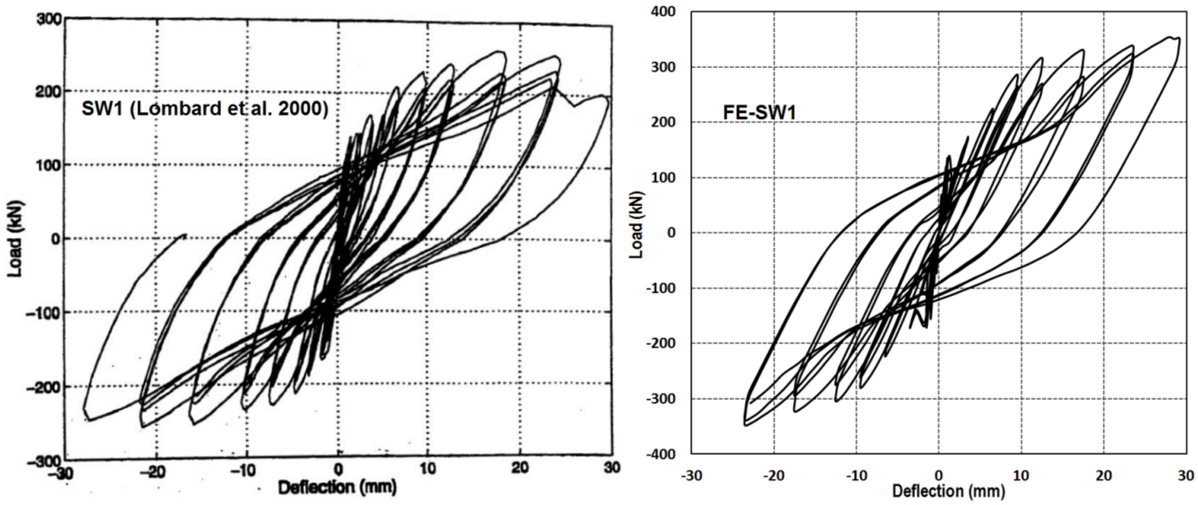

Figure 9: Top lateral Load-displacement for SW1 (Lombard et al. 2000) and FE-HSW1

Table 3. Comparison of the FEA and experimental results (Lombard et al. 2000) of RC shear walls at the key performance points (cracking, yielding, and ultimate)

Specimen/Model	V_{crack}	Δ_{crack}	V_{yield}	Δ_{yield}	V_{max}	Δ_{max}	average error %
CW	55.1	0.6	122.4	4	182.1	18	
FE-CW	55.65	0.38	128.6	4.18	223.26	18	
Error[§] %	0.01	36.7	5.06	4.5	22.6	0	11.5
SW1	101	0.65	153.1	1.6	246.6	29.1	
SW1	106.6	0.55	169.5	1.67	337.8	29.1	
Error %	+5.5	-15.4	+10.7	+4.4	+37	-	12.1

[§] ***Error % = (FE-EXP)/EXPx100***

4 Conclusions

An approach for FE modeling of RC shear walls retrofitted with external FRP wrapping and analysis of the system under cyclic loading was presented in this paper. The model showed promising capability of predicting the load-deflection curve as well as the internal forces in various parts of the wall within an acceptable margin of error. The FE models provide a wealth of useful data. An extension of the work presented in this paper could be useful in predicting the modes of failure of walls with various dimensions under different types of retrofitting techniques in future.

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