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MODELLING OF FRP-STRENGTHENED SHEAR WALLS WITH SPECIAL CONSIDERATION TO END-ANCHORAGE AND DEBONDING EFFECTS

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Abstract

Over the last two decades, a series of experimental studies has been conducted at Carleton University to investigate the effectiveness of using externally bonded fibre-reinforced polymer (FRP) sheets for the seismic strengthening and repair of reinforced concrete (RC) shear walls. The unique aspects and contributions of these studies include: 1) a comprehensive investigation of the influence of FRP end-anchorage to the load resistance capacity and seismic behavior of shear walls, 2) development of an innovative tube anchorage system which eliminates limitations of existing methods including the premature debonding of FRP due to the eccentricity of anchorage system, and 3) application of FRP sheets, instead of FRP wraps, to provide a more realistic representation of repair or strengthening techniques employed in the field. The wide range of test parameters considered in these studies provides a valuable dataset for development and verification of analytical and design procedures.

This paper, first, presents an overview of the above-mentioned experimental campaign carried out on FRP strengthened/repaired RC shear walls and the development of the tube anchor system at Carleton. Then, a new finite element (FE) modelling technique is developed for analysis of FRP strengthened or repaired RC shear walls with the tube anchorage system. Two modelling approaches are proposed to take into account the influence of the tube anchorage system on the load carrying mechanism of FRP sheets. The first approach uses elastic springs to simulate the stiffness of anchorage system based on the stress distribution at the base of the wall. The second approach approximates the effect of the tube anchorage system by modelling the development length of FRP sheets based on the concept of a pulley. In addition to the end-anchorage effects, several other important FRP- and RC-related mechanisms including FRP deboning effects, tension stiffening, compression softening, and strength and stiffness degradation under cyclic loads are also considered in the model. The accuracy of the proposed analytical procedure is evaluated against the aforementioned series of experimental studies carried out at Carleton. The analytical and experimental load-deflection responses are compared in terms of key structural response parameters. It is concluded that the proposed analytical procedure is capable of reproducing the observed complex behavior of FRP-strengthened shear walls with an acceptable level of accuracy.

Keywords: reinforced concrete; shear walls; FRP strengthening; finite element modelling; nonlinear analysis.

1. Introduction

Reinforced concrete (RC) shear walls are usually used as the lateral load resisting system in many structures constructed in seismically active regions. In spite of the fact that the current practices of design and construction of shear walls have been considerably enhanced in recent decades, many older shear wall buildings are susceptible to drastic damage during moderate or large earthquakes because of inadequate in-plane stiffness, flexural and shear strengths and/or ductility [1]. An appealing, least disruptive option for the repair and strengthening of shear walls in existing RC structures is the use of fibre-reinforced polymers (FRP) sheets [2].

Over the last few decades, many experimental and analytical studies have investigated the performance of FRP-strengthened RC shear walls. The majority of the experimental work focused on the shear strength and energy dissipation capacity of the walls [3,4,5,6], whereas the number of experimental studies on flexural-critical shear walls reinforced with FRP is relatively less [1,7]. Developing a numerical model to reliably estimate nonlinear response of repaired/strengthened shear walls using FRP sheets is critical to determine whether the structure can achieve the intended performance objectives under design level earthquakes. While a number of researchers have developed numerical models for RC beams and slabs repaired/strengthened in flexure with FRP [8,9,10,11], there is little information on the analytical modelling of FRP-strengthened RC shear walls. Previously, numerical models were developed by Cruz-Noguez et al. [12] and Hassan A. et al. [13] to estimate the nonlinear response of deficient shear walls reinforced with FRP sheets based on the Intermediate Crack (IC) Debonding model proposed by Lu et al. [11].

In order to provide further insight into the seismic performance of FRP strengthened/Repaired RC shear walls, this paper first presents an overview of a series of experimental programs carried out on FRP repaired/strengthened RC shear walls over the last two decades at Carleton University. One of the major findings of this experimental campaign was the development of an innovative tube anchorage system which eliminates limitations of conventional anchoring methods. However, there has not been any analytical studies to investigate or take into account the effect of the newly developed anchorage system on the behavior of FRP strengthened shear walls yet. This paper presents a new finite element (FE) modelling technique which is able to consider the effect of the anchorage system with reasonable accuracy without requiring detailed micro modelling of the anchorage system. Two analytical modelling approaches are introduced and discussed in detail. The analytical results are compared with measured experimental data and good correlation is observed for key structural response parameters. The consideration of the effect of the anchorage system in the analytical model is found to be crucial for a reliable estimation of the seismic performance of shear walls reinforced with FRP tow sheets.

2. Experimental Program

A comprehensive three-phase experimental program was carried out at Carleton University to investigate the seismic performance of FRP strengthened/repaired RC shear walls. The effect of various key parameters was investigated in the experimental tests including the anchor type, aspect ratio, repair and strengthening scheme, presence of initial damage, and failure mode. The walls were subjected to a quasi-static reversed cyclic loading condition in the horizontal direction simulating earthquake effects; axial load was not applied to the wall specimens. Unlike most existing studies, shear wall specimens were reinforced with FRP sheets instead of FRP wraps to provide a more realistic representation of repair or strengthening techniques employed in the field. The results of the experimental program were used to verify the analytical model developed in this paper. In the following, a brief overview of each phase of the experimental program is presented.

2.1 Phase 1 and 2

The first two phases of the experimental program investigated the performance of flexural-critical RC shear walls reinforced with FRP sheets. The main difference between the two phases was the type of anchor used to transfer the force from the FRP sheets to the bottom foundation block. The first phase of the experimental program was conducted by Lombard et al. [1] in which steel angles were used to anchor the FRP sheets. The second phase of the experimental program was carried out by Hiotakis [7] and investigated the effectiveness of a new type of anchorage system called tube anchorage system. More details about the two anchorage systems are provided in Sections 3.2 and 3.3. The aspect ratio of the wall specimens in both phases of the experimental study was 1.2. In addition to the type of anchorage system, other test parameters were the presence of initial damage prior to strengthening and the strengthening scheme which included the use of both vertical and horizontal FRP sheets. Fig.1 shows specimen dimensions, reinforcement details, and the two anchorage systems used in Phase 1 and 2 of the experimental program. The strengthening and repair schemes applied in these phases are listed in Table 1.

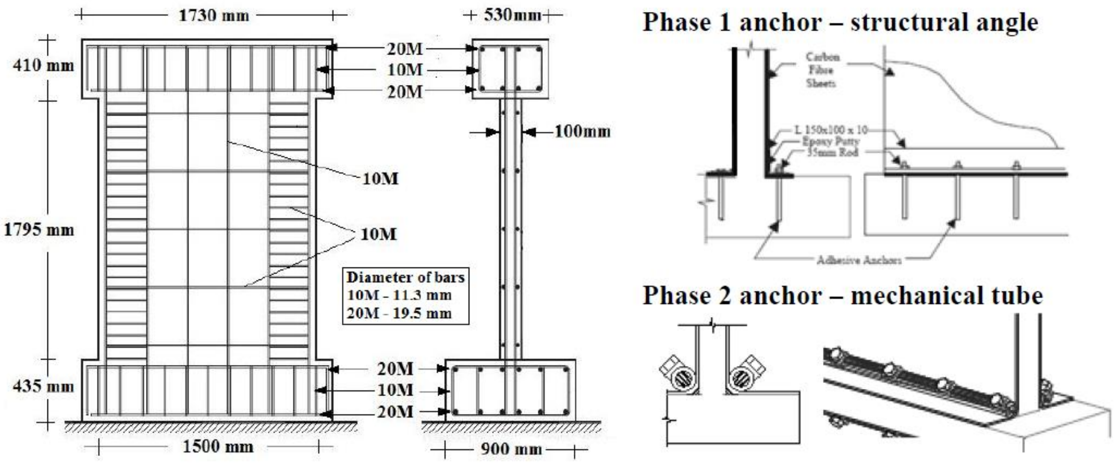


Fig. 1 – Shear wall design details (Phase 1 and 2) [14]

Table 1 – Strengthening and repair schemes used in phase 1 and 2 of the experimental program

Phase	Anchor Type	Failure mode	Aspect Ratio*	Type of Specimen	Repair/Strengthening Scheme**	Code
1	Angle	Flexural Dominant	1.2	Control	---	CW1
			1.2	Repaired	1V	RW1
			1.2	Strengthened	1V	SW1-1
			1.2	Strengthened	2V + 1H	SW2-1
2	Tube		1.2	Control	---	CW2
			1.2	Repaired	1V	RW2
			1.2	Strengthened	1V	SW1-2
			1.2	Strengthened	2V	SW2-2
			1.2	Strengthened	3V + 1H	SW3-2

* Aspect Ratio: height to length ratio (h_w/l_w)
** V-Vertically oriented FRP sheets and H-Horizontally oriented FRP sheets

Fig.2 shows the envelopes of the hysteretic responses obtained from the two phases of the experimental study. Later, Cruz-Noguez et al. [14] examined in detail the results of the experimental work conducted by Lombard et al. [1] and Hiotakis [7], making comprehensive comparisons between the observed failure mechanisms, FRP-concrete debonding progressions, and characteristics of force-deformation response (e.g. peak strength, ductility, energy dissipation, pinching effect etc.). The study concluded that in repair applications, the CFRP retrofitting system could restore most of the initial elastic stiffness and improve the flexural capacity of the damaged walls. With strengthening applications (i.e. walls in as built conditions), there was a considerable increase in the stiffness and flexural capacity of the walls [15].

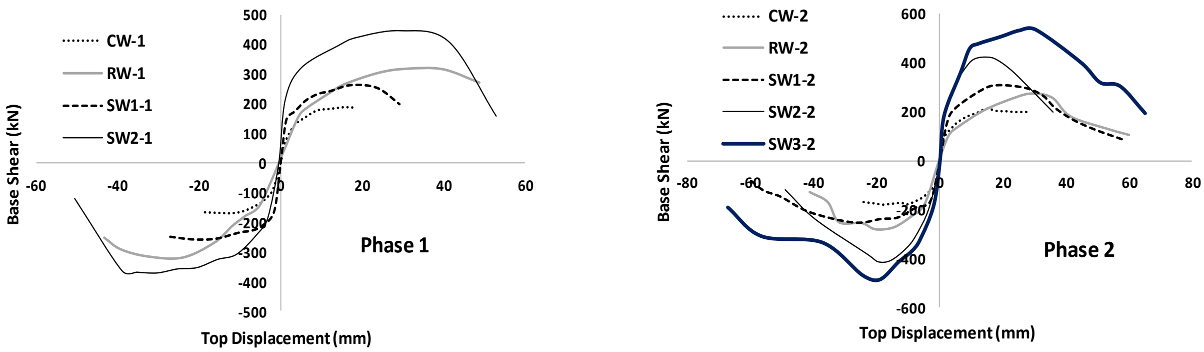


Fig. 2 – Envelopes of hysteresis responses for control (CW), repaired (RW) and strengthened (SW) shear wall specimens

2.2 Phase 3

In phase 3, nine RC shear wall specimens were tested under in-plane reversed cyclic loads. Most of the shear walls had a low aspect ratio of 0.85 and therefor were vulnerable to brittle shear failures. Of the nine shear wall specimens, two had an aspect ratio identical to the walls tested by Lombard et al. [1] and Hiotakis [7] (h_w/l_w = 1.2) to compare the performance of various CFRP retrofitting systems used in different phases of the study. Results of these tests were initially presented in Woods [16]. In strengthening applications, the retrofitting system was able to improve the in-plane load carrying capacity, ductility, and energy dissipation capacity; in repair applications, the retrofitting system was effective to, at least, restore the wall specimen original state [15].

3. Anchor System

When externally bonded FRP sheets are used in the strengthening or repair of reinforced concrete members the failure is usually governed by crushing of the concrete and/or rupture of the FRP sheets after yielding of the steel reinforcement [17]. However, it is found that in many cases, FRP sheets debond from the concrete substrate before the FRP material reaching its ultimate tensile capacity, preventing the member from reaching its design strength [1,8,12]. Different FRP anchor systems have been developed to eliminate or minimize FRP-concrete debonding before the FRP reaches its ultimate tensile strength. Several studies on FRP-strengthened RC beams and slabs found that using effective anchorage systems can improve the overall performance of the structural element [2,18]. Studies on RC shear walls by Lombard et al. [1], Hiotakis [7], and El-Sokkary et al. [19] also concluded that FRP anchorage plays a critical role in preventing premature debonding failures. In the following sections, different FRP-concrete debonding mechanisms and the two anchorage systems used in the experimental studies conducted at Carleton are briefly discussed.

3.1 Angle Anchor System

Steel angle anchor system is among the most common mechanical anchor systems. Observations by Lombard et al. [1] and Kanakubo et al. [20] concluded that the steel angle anchor system helps debonding of the FRP material from the concrete wall before the FRP material attaining its ultimate capacity. Eccentricity between the tensile force in the FRP sheet and the reactions of the anchoring bolts causes a moment which leads to rotation of the steel angle, also referred to as “prying” action, and failure of the steel angle anchor system as illustrated in Fig.3(a) and Fig.3(b). In cyclic response of the shear wall, the prying action leads to debonding when the flange pulls away from the surface of the wall, shown in Fig.3(c). The debonded FRP sheet buckles in compression when the load is reversed. Buckling of the debonded FRP sheets leads to fracture of the hardened epoxy matrix. The sharp edges produced by the fractured FRP sheet, can easily cut the FRP fibres prior to reaching the ultimate rupture capacity. This behavior reduces the load carrying capacity of the FRP sheet [16].

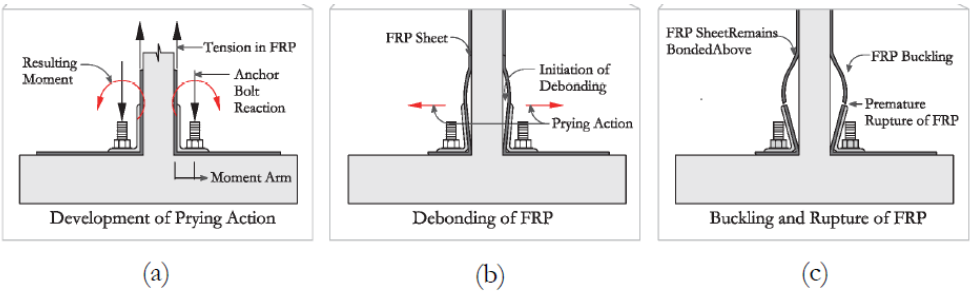


Fig. 3 – Steel angle anchor failure progression [16]

3.2 Tube Anchor System

As mentioned above, the prying action in steel angle anchor leads to premature debonding of the FRP material from the concrete substrate; hence, an innovative anchor system made up of a cylindrical hollow section (CHS) was designed. The FRP sheet is wrapped around the tube and anchored to the adjacent member. The tube is bolted into the wall foundation using several threaded steel anchor rods at a 45 degree along the length of the anchor, as shown in Fig.4. The pulley principle is employed in the design of the tube anchor: when the FRP sheet is loaded in tension, the vertical force in the FRP sheet is equal to the force in the horizontal FRP sheet which must be provided with sufficient development length.

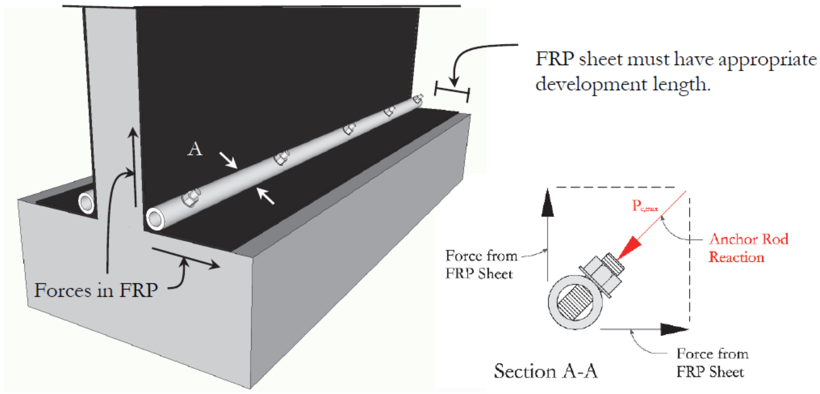


Fig. 4 – Forces Acting on the tube anchor system [16]

To develop the effective FRP stress at a section, ACI440.2R-08 [26] guidelines regarding application of externally bonded FRP systems, recommend that a minimum length of the FRP sheet must be bonded to the concrete element (l_{df}), as determined by Eq. (1):

$$l_{df} = \sqrt{\frac{n_s E_f t_s}{\sqrt{f'_c}}} \quad (1)$$

where n_s represents the number of FRP sheets requiring development, E_f is the modulus of elasticity of the FRP composite, t_s is the thickness of the FRP sheet and f'_c is the uniaxial compressive strength of the concrete to which the FRP is bonded [8]. Placing the anchor bolts in the direction of the resultant load, eliminates the eccentricity between the forces carried by the FRP sheets and the anchor bolts. Experimental results and observations by Hiotakis [7] demonstrated that the anchor system is efficient in transferring the load between the FRP sheet and adjacent structural member. The FRP sheet is able to attain its ultimate tensile strength unaccompanied by premature debonding of the FRP sheets from the concrete. Improvement in efficiency can be acknowledged by comparing the hysteretic response of two wall specimens having the same steel and FRP reinforcement details tested by Lombard [1] and Hiotakis [7], one of which is carried out using the steel angle anchor and the other with the tube anchor (Fig. 5).

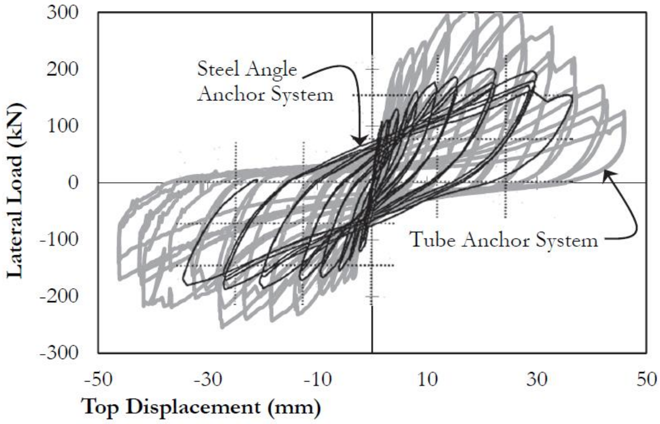


Fig. 5 – Wall hysteretic response with angle and tube anchor systems [16]

It is shown that by application of the tube anchor system, considerable improvements in strength, ductility and energy dissipation capacity in reinforced concrete shear walls is attained when compared to the identical wall with the angle anchor system.

4. Numerical Modelling

The aim of this study is to develop a finite element (FE) modelling technique for FRP-strengthened RC walls which takes into account the influence of anchorage system on the structural response without requiring detailed micro modelling of the anchorage system. The modelling method was developed using VecTor2 which is a 2D nonlinear analysis software specialized for reinforced concrete structures. VecTor2 is based on the Modified Compression Field Theory (MCFT) [21], and the Disturbed Stress Field Model (DSFM) [22]. In these formulations, the concrete is modelled as an orthotropic material with smeared, rotating cracks. An interesting characteristic of this program is that the structural model can be adjusted at any point of the analysis by activating/deactivating elements, allowing the simulation of chronological repair/strengthening procedures while keeping track of the previous state of existing elements [12]. In the following sections, a brief description of different parts of the proposed modelling technique are described.

4.1 Geometric Modelling

Four-noded rectangular elements were used to model the concrete, while truss elements were used to model the CFRP material. The geometry of the wall specimen was divided into four zones: cap beam, wall boundaries, wall core and foundation block. The boundary condition at the base of the foundation block was assumed to be fully fixed. Link elements were used to represent the interaction between concrete and CFRP layers. Fig.6 shows different FE models used in this study. As explained in Section 3, the anchor system is a crucial component in the design of an FRP strengthening system; hence, special attention should be given to include the effect of this component in the numerical model.

Two different approaches were followed to account for the effect of the tube anchor at the base of the wall. In the first approach, perfect bond was assumed between the FRP trusses and concrete elements at the bolt locations; whereas FRP trusses located between the bolts were continued into the foundation block by 350mm which was the development length of the horizontal FRP sheets as reported by Hiotakis [7]. In this approach, the parts of the anchor tube between the bolts were assumed to act as pulley that transfers the load from the vertical FRP to the horizontal FRP. Because the model is two dimensional, instead of modelling the horizontal FRP trusses in the out-of-plane direction, they were continued into the foundation zone in the same direction as the vertical FRP trusses. This approximation was considered acceptable because based on the concept of pulley the force developed in the horizontal FRP sheets should be approximately equal to the force in the vertical FRP sheets at the junction of the wall and foundation block.

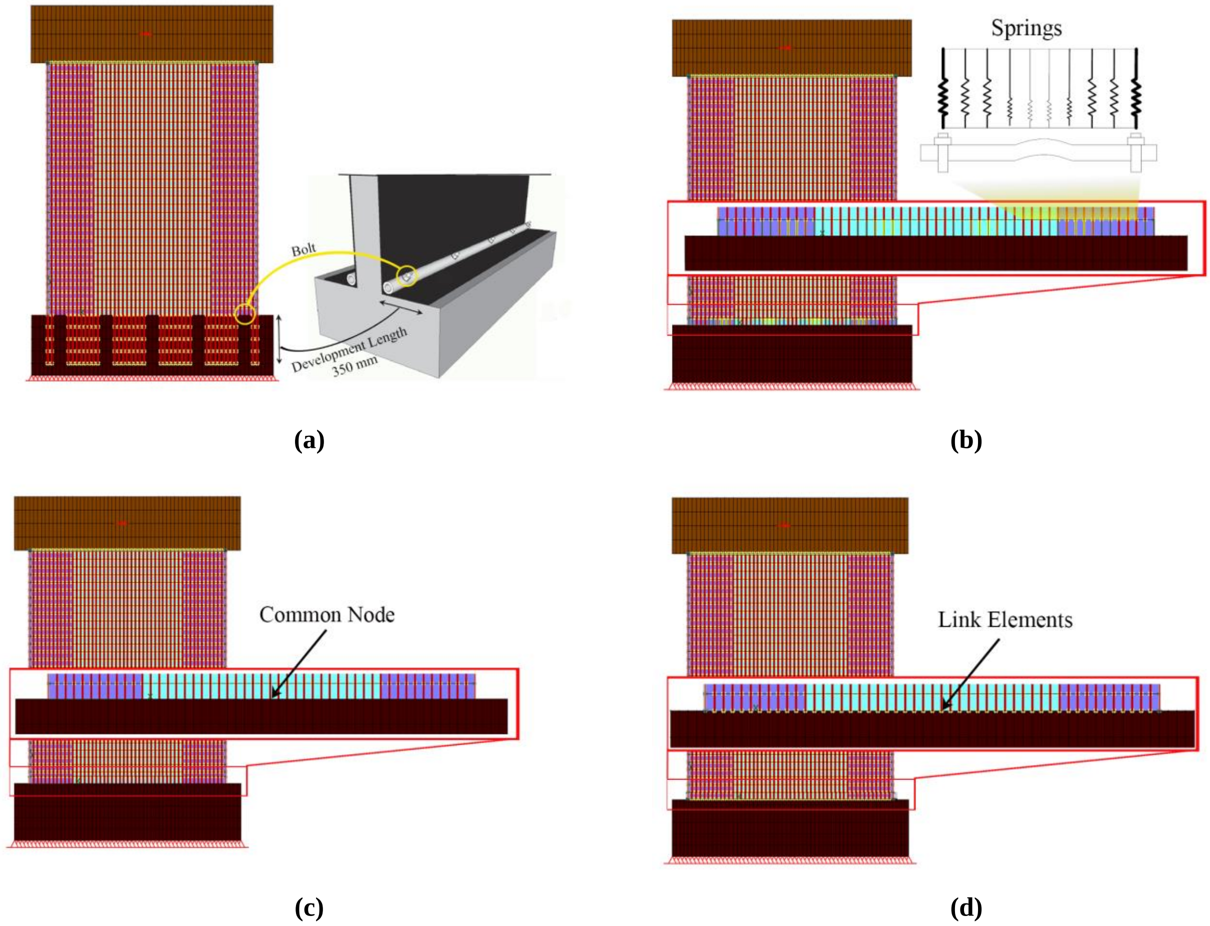


Fig. 6 – Different approaches used to model anchorage system: (a) FRP trusses continued to the foundation block, (b) elastic springs were used between the bolts, (c) prefect bond assumed at the junction of the wall and foundation, (d) Imperfect bond assumed at the junction

In the second approach, prefect bond was assumed between the FRP trusses and concrete elements for all nodes at the junction of the wall and foundation block (i.e. common nodes were used to connect FRP and concrete elements). However, groups of uniaxial elastic springs with different stiffnesses were used between the bolts to simulate the effect of the tube anchorage system. These springs have higher stiffnesses near the bolts and lower stiffnesses at the middle, as shown in Fig.6(b). This approach is based on the tube anchor design methodology suggested by Woods [15, 27]. The design of the tube anchor is assumed to be based on maximum allowable displacement of the steel tube. This design parameter is quantified by analyzing the effects of the flexibility of the stress profile from the FRP sheet. As the tube bends, the FRP sheet relaxes between bolts, which lead to a decrease in stress at mid-span. However, the stress between the bolts redistributes, leading to an increase in stress at the locations of the bolts [15]. Fig.7 shows a single bay of the tube anchor subjected to linear stress profile. In this study, the concept of stress profile was used to simulate the variation in the stiffness of the springs through the length of the wall.

In addition to the two approaches mentioned above, two other case studies were also examined for the purpose of comparison: first, prefect bond was assumed for all nodes at the junction of the wall and foundation block by using common nodes between the FRP trusses and concrete elements (see Fig.6(c)); second, imperfect bond was assumed between the FRP trusses and concrete elements by using link elements, as shown in Fig.6(d). These two case studies represent typical assumptions made by engineers and researchers in modelling the effect of anchorage system on FRP strengthened/repaired RC structural elements.

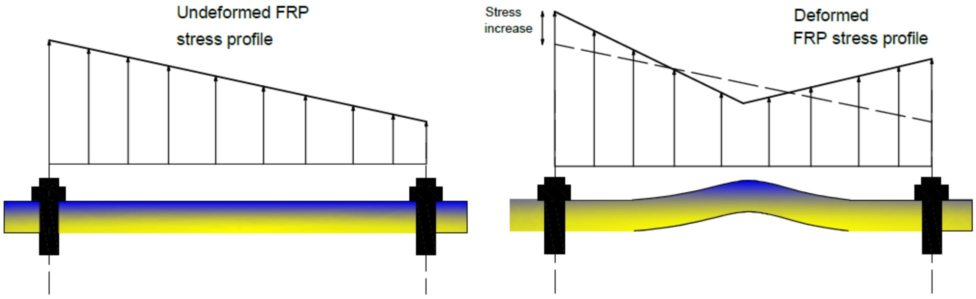


Fig. 7 – Effect of tube flexibility on vertical FRP stress distribution

4.2 Mesh Size

Analysis results from a mesh sensitivity study of the shear wall (SW1-2 phase 2) showed that the maximum lateral force capacity calculated by the finite element model with a mesh size of 30mm×60mm was within 2% of that of another model with a smaller mesh size of 22mm×25mm. Consequently, a finite element model of mesh size 30mm×60mm was used for accurate modelling of the shear wall while maintaining computational efficiency. Fig.8 shows the result of the mesh sensitivity analysis.

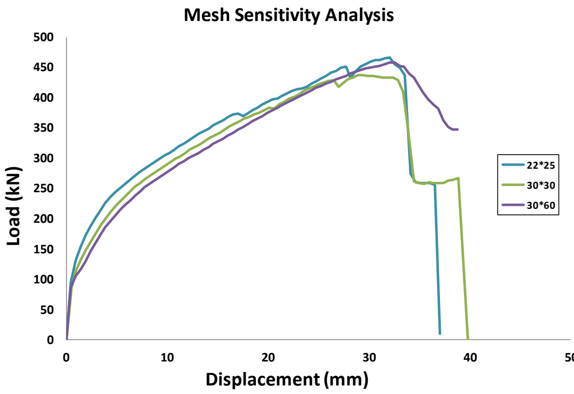


Fig. 8 – Mesh sensitivity analysis

For the cap beam and the foundation block regions, larger mesh sizes were used because the behavior is likely close to rigid in those regions and both concrete and steel behave in a linear elastic manner.

4.3 Modelling of Concrete and Steel Rebars

The material modelling options including the second-order material effects and constitutive relationships were set to the default values of the analysis program VecTor2. No fine tuning of the analysis parameters, material modelling or structural modelling was undertaken. The pre- and post-peak compressive responses of concrete were modelled based on the Hognestad Parabola and Modified Park-Kent models, respectively. The compression softening behavior of concrete was considered using the Vecchio's model. For tension stiffening and tension softening, the 2003 Modified Bentz model and Nonlinear model by Hordjik were selected, respectively. The hysteretic behavior of concrete was considered by using the plastic offsets model with nonlinear unloading proposed by Vecchio [23]. The resulting plastic offset strains, together with the area defined by the hysteretic loops, represent the internal damage and energy dissipation under cyclic loading. The model uses nonlinear Ramberg-Osgood formulations to calculate the unloading response in the compression domain [24].

The contribution of the reinforcing bars can be modelled in two ways: the smeared model and the discrete model. With the smeared model, the average effect of the reinforcement is considered as a part of the concrete material by adding the equivalent stiffness of the reinforcement to the concrete element and appropriately modifying the concrete material properties. This option is appropriate for the case where the reinforcement is uniformly distributed over a large area. Alternatively, for the case of concentrated reinforcement, the best option is to model the reinforcing bars discretely by using two-noded truss elements. In this study, because both the horizontal and vertical reinforcements were distributed uniformly along the height and length of the wall, the smeared modelling approach deemed more appropriate. To represent the hysteretic response of reinforcement, the Seckin model [25] was adopted which includes a linear elastic region followed by a yield plateau and strain hardening.

4.4 Modelling FRP Sheets

The FRP sheets were modelled as discrete using two-noded truss elements made up of an elastic, tension only material. The area of the FRP trusses were computed from the thickness and tributary width of the FRP sheets. The stress-strain relation is linear elastic until rupture of the FRP in tension.

4.4.1 Bond-Slip Model

The bond-slip effects was modelled using link elements at the interface of RC rectangular elements and FRP truss elements. The link element is a two-noded zero-length element with a total of four translational DOFs. In this numerical study, the bond-slip model proposed by Lu et al. [11] was utilized. With this model, the bond behavior (i.e. shear stress vs slippage) is defined by three parameters: maximum bond stress, slip corresponding to the maximum bond stress, and ultimate slip.

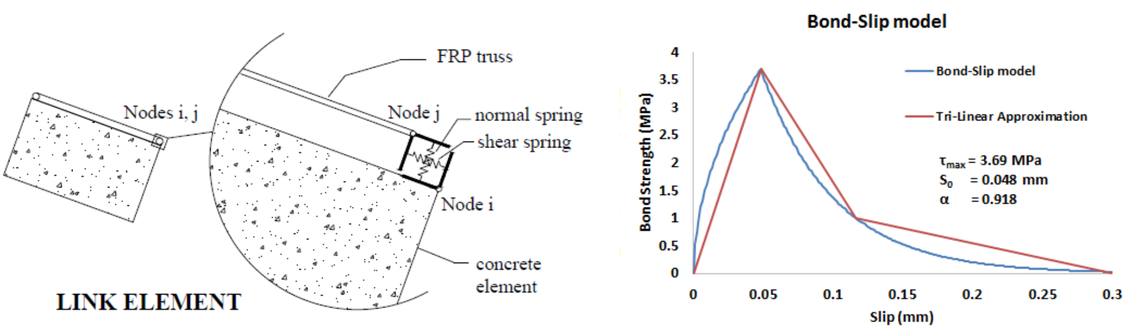


Fig. 9 – Schematic of interface element (left) and tri-linear bond-slip relationship (right)

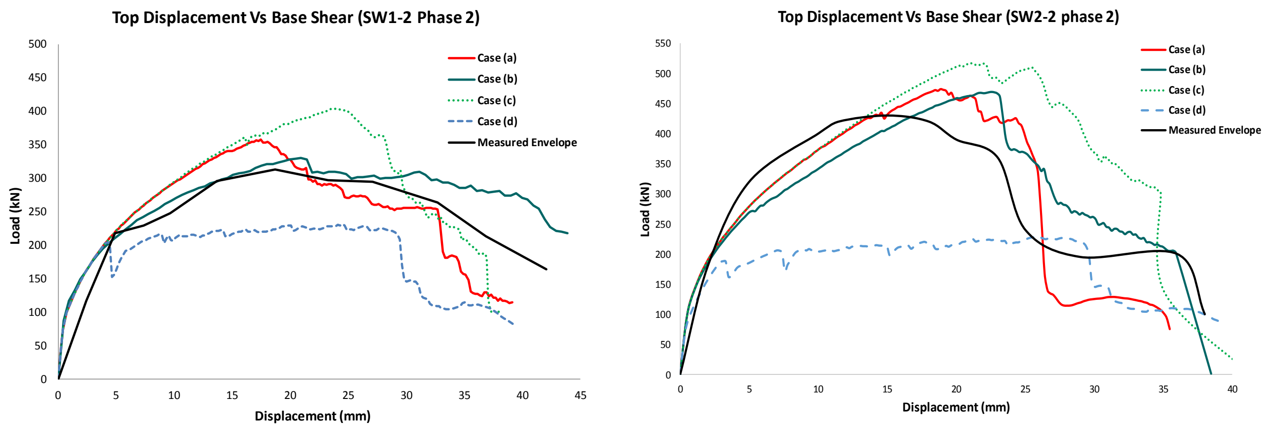
The link elements employed in the model have two nodes, one is connected to the concrete element and the other connected to the FRP truss. The interaction between these two nodes is represented by two springs, one acting along the longitudinal axis of the FRP truss (the “shear” spring) and the other acting in the orthogonal axis (the “normal” spring), as illustrated in Fig.9. The normal spring has an infinite stiffness that cannot be changed, while a user-defined, tri-linear bond-slip relationship can be assigned to the shear spring [12].

4.5 Loading

The shear walls were subjected to a lateral displacement applied at the middle of the cap beam and controlled in a reversed cyclic manner up to failure. No axial load was applied on the specimens. The lateral load was modelled by controlling the displacement of the node located at the middle of the cap beam based on the loading pattern reported from the test. The self-weight of the wall was modelled using gravity loads computed based on the density of concrete elements.

5. Results and Discussion

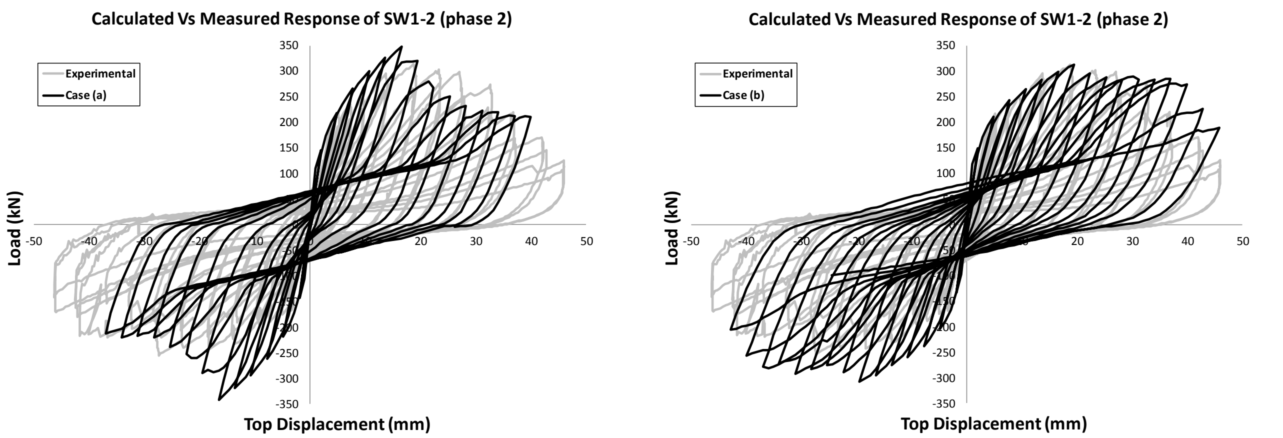
The responses of the strengthened walls 1 and 2 in phase 2 (SW1-2 & SW2-2) were calculated by considering the four cases of different modelling assumptions of the anchor system (Cases a, b, c, d in Fig.6). The results are compared with the envelopes of the measured hysteretic responses, as shown in Fig.10.



(i) (ii)

Fig. 10 – (i) The response of strengthened wall 1 - phase 2 (SW1-2) (ii) The response of strengthened wall 2-phase 2 (SW2-2)

As can be seen in Fig.10, Case (c) which represents prefect bond between FRP and concrete at the base of the wall led to overestimation of the maximum load capacity; conversely, the imperfect bond model of Case (d) resulted in significant underestimation of the maximum load capacity; whereas the responses calculated by the spring model of Case (b) and Case (a) model which considered development length correlated reasonably well with the measured envelopes. All analysis cases overestimated the initial stiffness compared to the experimental results. This may be attributed to the shrinkage cracks occurred at the base of the wall which reduced the initial stiffness of the wall specimens.



(i) (ii)

Fig. 11 – Top displacement versus base shear in SW1-2 (phase 2) (i): case (a) , (ii) case (b)

As shown in Fig.11, the hysteretic responses calculated by both Case (a) and (b) methods correlated well with the experimental results especially in the pre-peak zone. In both cases, the ultimate strength and pinching effects were accurately predicted. However, the analyses slightly overestimated the initial stiffness and underestimated the ductility. Overall, the spring model of Case (b) produced better results compared to Case (a) in terms of capturing the ultimate strength and ductility behavior.

For further improvement of the model of the FRP strengthening system with the tube anchor, it is recognized that the use of multiple FRP layers on each side of the shear wall may result in the FRP behaving like a hard laminate [1,7] which could lead to some flexural capacity of the FRP layers. Hence, the use of a tension-only truss element with no compressive capacity to model the FRP material may not be suitable because it does not consider the FRP's effect on the flexural and shear strength of the wall.

Additionally, using several layers of FRP sheets have a confining effect that increases the compressive strength of concrete and postpones concrete cracking. With 2D modelling, the confining effects can be approximated by addition of a smeared reinforcement component in the out-of-plane direction to the concrete elements.

6. Conclusion

This paper first presented a brief overview of a comprehensive experimental program that has been conducted on FRP-strengthened and repaired RC shear walls at Carleton University. As part of this experimental program, two mechanical anchor systems developed and tested; namely, a steel angle anchor system and an innovative tube anchor system. Experimental results have confirmed the efficiency of the tube anchor system in avoiding premature debonding and permitting a large portion of the high strength capacity of the FRP sheets to be utilized. The present study has developed the first numerical model that vigorously accounts the effect of the anchorage system on the response of the FRP strengthened shear wall. Two approaches have been followed to model the influence of the anchorage system: 1) elastic springs calibrated based on the stress distribution at the base of the wall, and 2) modelling the development length of FRP by simulating the tube anchor system with the concept of a pulley. The analytical results were compared with measured experimental data and good correlation was observed for key structural response parameters. Additionally, two other case studies which represent typical modelling assumptions that researchers make to simulate the effect of anchorage system have been investigated (i.e. perfect bond and imperfect bond assumptions at the wall base). For these case studies significant discrepancies were found between the analytical and experimental results. The following conclusions can be drawn from this study:

1. The consideration of the effect of the anchorage system in the analytical model is found to be critical for a reliable prediction of the nonlinear performance of the shear walls reinforced with FRP tow sheets.
2. Oversimplifying the effect of anchorage system by using assumptions such as uniform prefect bond or imperfect bond between FRP and concrete at the base of the wall can result in significant overestimation or underestimation of the peak strength.
3. Rational procedures such as the ones presented in this study can take into account the effects of anchorage system with reasonable accuracy.

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