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# MODELLING OF DEBONDING MECHANISMS IN EXTERNALLY BONDED FRP SHEETS IN RC SHEAR WALLS FOR EARTHQUAKE RESISTANCE

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**ABSTRACT:** Compared to other rehabilitation techniques, the use of externally bonded Fibre Reinforced Polymers (FRP) sheets for repair of damaged and strengthening of deficient reinforced concrete (RC) structures has gained increasing acceptance as a viable alternative especially in rapid repair application or as a less disruptive rehabilitation strategy. In previous studies of FRP retrofit applications, the focus has been directed mainly on the retrofit of one-dimensional structural elements of beams and columns. In the present study, seismic retrofit of two-dimensional structural element of shear wall using FRP sheets is investigated. An important component in the failure mechanism of RC shear wall retrofitted with FRP sheets is the separation or debonding of the FRP sheets from the concrete wall surface during seismic responses. After the occurrence of concrete cracking in a shear wall under the reversed loading actions of an earthquake, it is observed that debonding of the FRP sheet from the concrete substrate spreads quickly reducing its lateral load resisting capacity. The interaction behaviour between the performance of the FRP and the concrete cracking behaviour is known as Intermediate Crack (IC) debonding mechanism. Although previously developed models are able to account for the debonding of FRP in one-dimensional beam members, they are unable to predict the behaviour of two-dimensional shear dominated shear wall components. This paper presents a new computer simulation model that can accurately capture the hysteresis response behaviour of reinforced concrete shear walls repaired or strengthened with externally boned FRP sheets under the reversed cyclic loading of earthquakes. The proposed computer model can accurately simulate the IC debonding mechanism under the two-dimensional stress state of the wall panel and the subsequent ductile flexural or brittle shear failure modes of walls with different aspect ratios. Computer simulation results correlate well with experimental test results. The proposed computer model can accurately predict the hysteresis response behaviour, lateral load resisting strength, energy dissipation capacity and ductility performance of FRP repaired or strengthened shear walls in seismic applications.

# 1. Introduction

Reinforced-concrete (RC) shear walls are a common type of lateral load resisting system found in structures located in seismically active regions. Although the current practices of shear walls design have been significantly improved in recent decades (ACI 2005; CSA 1994), many older shear wall buildings are at risk of suffering severe damage during moderate or large earthquakes because of insufficient in-plane stiffness, flexural and shear strengths and/or ductility (Lombard et al. 2000). An attractive, minimally disruptive option for the repair and strengthening of shear walls in existing RC structures is the use of fibre-reinforced polymers (FRP) sheets (Triantafillou 1998). The experimental studies that examine the use of FRP for strengthening RC shear walls can be divided into two main categories. The first category includes tests which examine the effect of FRP on shear strength and energy dissipation capacity of the walls (Antoniades et al. 2003; Paterson and Mitchell 2003; Khalil and Ghobarah 2005; Elnady 2008, Shaheen 2013). The second category includes tests that focus on enhancing the flexural capacity and stiffness of shear walls (Lombard et al. 2000; Hiotakis 2004). Developing a numerical model to predict the response of walls strengthened using externally bonded FRP is crucial to determine the enhancement effects of the FRP on both the flexural and shear strength of strengthened walls. Such a model can be used to assess the failure mechanism of a structural wall whether it will suffer a brittle shear failure or a ductile flexural failure. While a number of researchers have developed numerical models for RC beams and slabs repaired/strengthened in flexure with FRP (Teng et al. 2002; Wong and Vecchio 2003; Oehler et al. 2003; Lu et al. 2007), there is relatively scant information on the analytical modeling of RC shear walls flexurally-reinforced with FRP sheets. Previously a numerical model to predict the nonlinear response for the flexurally reinforced walls was developed by Cruz-Noguez et al. (2012). In this study, a numerical model capable of predicting the response of shear deficient walls is developed. Using this new model, this paper presents a numerical study on the simulation of the nonlinear hysteretic response behaviour of two shear deficient shear wall specimens strengthened with FRP which have been tested to failure at Carleton University (Woods, 2014). The experimental program includes testing of un-damaged shear walls that have been repaired or strengthened by externally-bonded carbon fibre tow sheets oriented both in vertical and horizontal directions. The novel aspect in this study is the implementation of a computationally efficient computer procedure that can capture the entire debonding process between the concrete substrate and the FRP material from initial debonding failure of the FRP sheets to post peak ultimate collapse of the wall. It improves on the analytical model of FRP strengthened walls developed by Cruz-Noguez et al. (2012) which is based on the intermediate crack (IC) debonding model proposed by Lu et al. (2007). The computer simulation results are compared with measured experimental data and good correlation is observed.

## 1.1. Overview

The experimental program at Carleton University consists of 3 phases. The first and second phases involved testing of nine cantilevered shear walls designed according to the CSA A23.3 (2004) specifications. The aim of the test was to enhance the flexural strength while maintaining a ductile flexural failure mechanism. The details of the walls tested in the first two phases are not discussed in this paper but can be found in the references by Lombard et al. (2000), Hiotakis et al. (2004), and Cruz-Noguez et al. (2012). The third phase of testing involves testing shear deficient shear walls designed using obsolete design specifications such as CSA (1977) and ACI (1968) in order to determine the efficiency of the FRP retrofit in strengthening walls with poor detailings such as insufficient shear reinforcement, poor confinement and low concrete strength. The test involves two slender walls with an aspect ratio ($h_w/l_w$) of 1.2 as shown in Fig. 1a, and another two intermediate walls with an aspect ratio ($h_w/l_w$) of 0.85, as shown in Fig. 1b. Both walls have vertical (longitudinal) reinforcement ratio of 3.0% and a horizontal (transverse) reinforcement ratio of 0.25%. Each specimen has a cap beam to which a hydraulic jack applies cyclic quasi-static load steps during the test. The wall specimen is fixed at its base to the laboratory strong floor. In this study, the control walls in its unstrengthened condition are tested up to their maximum peak strength capacity, whereas the walls strengthened with FRP tow sheets are tested up to collapse failure. For the strengthened walls 3 layers of horizontally oriented FRP sheets and one layer of vertically oriented FRP sheet are applied on each side. The FRP sheets are anchored to the wall base using the tube anchors tested previously by Hiotakis et al. (2004) and Woods et al. (2014).

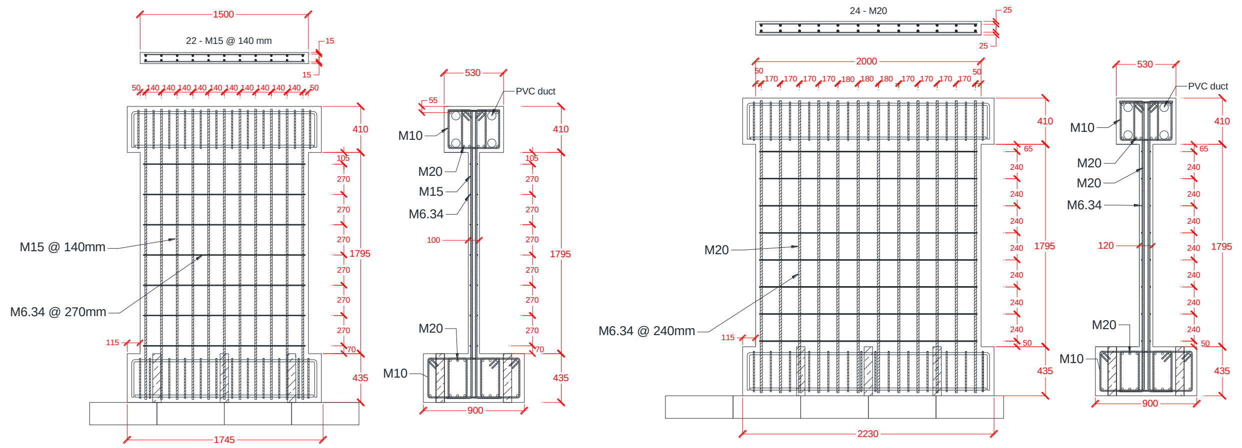


**Figure 1: (a) The slender shear wall design on the left; and (b) The intermediate shear wall design on the right**

## 1.2. Experimental Results

Figure 2 presents the envelopes of the force-displacement hysteretic relationship between the base shear and top deflection of the wall specimens measured during the experiments. It is observed that the initial stiffness and the strength of the strengthened walls increase with the addition of the FRP. The control walls fail in the expected brittle shear failure mode as evident by a large diagonal crack shown in Fig. 3. In comparison, the walls strengthened with FRP sheets behave in a ductile manner with a significant increase in their energy dissipation capacity. The first sign of failure in the strengthened wall is fine flexural cracks visible on the side of the wall near the base. Those cracks initiate debonding between the FRP sheets and the concrete substrate. As the cyclic load continues to increase, the debonding starts to propagate upwards as the flexural cracks start spreading along the height of the wall and towards the wall centre. Eventually, the flexural cracks connect with the diagonal shear cracks and cause major debonding of the FRP from the concrete shear wall. The concrete at the ends of the wall suffer extensive crushing failure as shown (Fig. 4a). As a result, this leads to buckling of the longitudinal reinforcements as shown (Fig. 4b). Subsequent to this failure process, the external FRP sheets are the remaining elements that provides resistance to the tensile forces resisting the overturning moment imposed on the wall. When the tensile load exceeds strength capacity of the FRP, the FRP sheet fails by rupture (as shown in Fig. 4c) thus achieving the full efficient initialization of the FRP materials in enhancing the seismic performance of the wall. As soon as the FRP ruptures, a large drop in the resistance of the wall is observed.

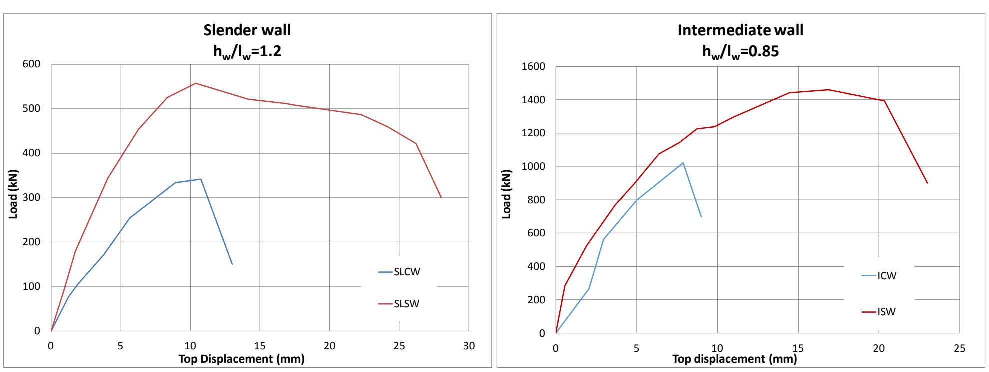


**Figure 2: Measured force-displacement envelopes for both the Slender and Intermediate walls**

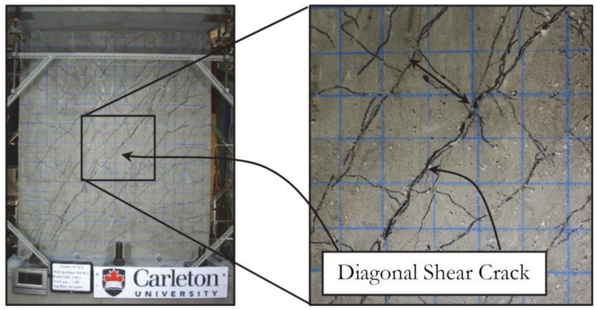


**Figure 3: Large diagonal shear crack indicating the brittle shear failure of the unstrengthened shear wall (Woods, 2014)**

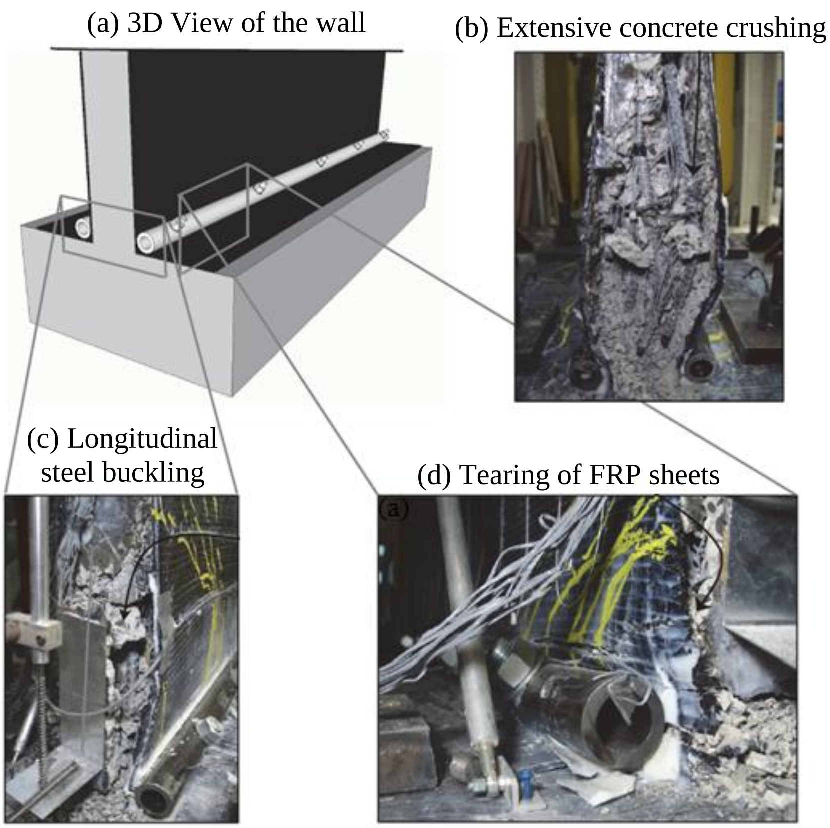


**Figure 4: Different stages of failure in FRP strengthened wall (Woods, 2014)**

## 2. IC debonding in FRP strengthened structures

### 2.1. Overview

Debonding of FRP materials from the concrete before the ultimate strength of the FRP is achieved is an important part of the failure mechanism of FRP reinforced concrete structures. Pevious experimental and numerical studies (Teng et al. 2002; Lombard et al. 2000; Hiotakis 2004; Shaheen et al. 2013; Woods 2014) have shown that the debonding of FRP from the concrete substrate controls the failure mode and overall response of FRP reinforced concrete shear walls. Reviewing the results of experimental tests of 77 beams and slabs strengthened with FRP, Lu et al. (2007) has concluded that one of the most critical debonding mechanisms is caused by the opening up of flexural cracks in the concrete, referred as intermediate crack (IC) debonding. In this mechanism, FRP-concrete debonding first occurs at a flexural crack and quickly propagates towards the laminate edges, causing a sudden drop in the load carrying capacity of the structural member, as shown in Fig. 5.

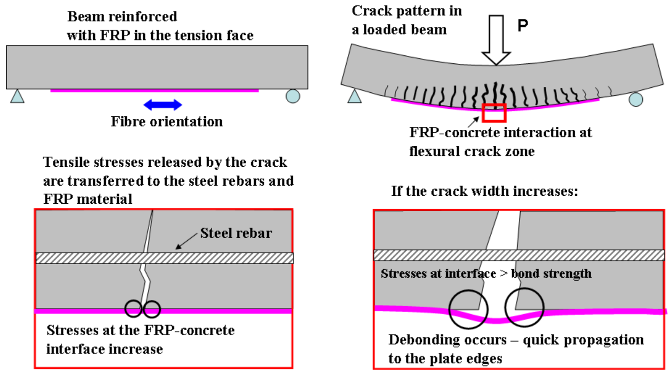


**Figure 5: IC debonding mechanism in beams (Cruz-Noguez, 2012)**

## 2.2. Analytical modeling of concrete cracking

In this study, the modelling of concrete crack behavior is by the smeared crack approach (Wong and Vecchio, 2003; Pham and Al-Mahaidi, 2007). In this model, the concrete is effectively treated as a continuum. Cracking in the concrete is represented as tensile over straining in the concrete element, and crack propagation is simulated by reducing the stiffness and strength of the concrete with a suitable constitutive model. In the modelling method of this study, the FRP is modelled as connected directly to the concrete element with no explicit modelling of the adhesive layer. This is because it is assumed that the amount of slip that occurs between concrete and FRP is negligible compared to the amount of the slip that occurs at the concrete (Lu et al. 2005).

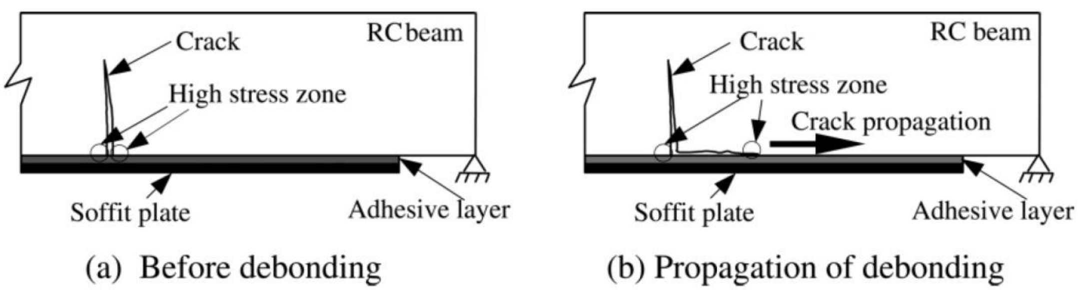


**Figure 6: IC debonding propagation (Teng et al., 2003)**

## 2.3. Simplified bond-slip models for flexural cracks

In the debonding model proposed here, FRP-concrete interface elements with bond-slip constitutive laws are adopted to simulate the mechanics of the debonding caused by the tensile fracture in the concrete layer (Wong and Vecchio, 2003; Wu and Yin, 2003; Ebead and Neale, 2007; Lu et al. 2007). This modelling approach eliminates the need for small mesh size since the interface element accounts for the FRP-concrete interaction. To define suitable constitutive laws for these interface elements, experimental bond-slip relationships from FRP-concrete pull tests have been used in FE analyses of FRP-reinforced RC elements (Wong and Vecchio, 2003; Wu and Yin, 2003). However, a serious limitation in this approach is that usual FRP-concrete pull tests do not include the presence of flexural cracks in the concrete prism. Therefore, these models cannot be used in structures where IC debonding controls the response of structural members (Sato, 2003).

To overcome this deficiency, results from meso-scale studies that rigorously account for IC debonding effects can be used instead to generate appropriate bond-slip models in structures where flexural cracking is present. Using the meso-scale FE analyses of the beams of Wu and Yin (2003), Lu et al. (2007) has obtained representative bond-slip relationships for FRP-concrete interfaces with or without major flexural cracks. For the part of the FRP-concrete interface outside the major flexural crack zone, the bilinear bond-slip model developed by Lu et al. (2005) is referred as bond-slip model I expressed as follows:

$$\tau = \begin{cases} \frac{\tau_{max} s}{s_o} \; if \; s \leq s_o \\ \frac{\tau_{max}(s_f - s)}{(s_f - s_o)} \; if \; s_o < s \leq s_f \\ 0 \; if \; s > s_f \end{cases} \tag{1(a)}$$

$where$

$$s_f = \frac{2G_f}{\tau_{max}} \tag{1(b)}$$

$$\tau_{max} = 1.5\beta_w f_t \;\; where \; f_t \; is \; the \; tensile \; strength \; of \; Cocnrete \tag{1(c)}$$

$$s_o = 0.0195\beta_w f_t \tag{1(d)}$$

$$G_f = 0.308{\beta_w}^2\sqrt{f_t} \tag{1(e)}$$

$$\beta_w = \sqrt{\left(2.25 - {b_f}/{b_c}\right) \Big/ \left(1.25 + {b_f}/{b_c}\right)} \tag{1(f)}$$

In the above equations, $\tau$ is the shear bond stress (MPa); $s$ is the interfacial slip (mm); $G_f$ is the interfacial fracture energy (MPa); $b_f$ is the width of the strip of FRP laminate (mm); $b_c$ is the width of the concrete member (mm) where the FRP strip is located, and $\beta_w$ is the FRP to concrete width ratio.

For parts of the FRP-concrete interface inside the major flexural crack zone, Lu et al. (2007) show the bond-slip response over a length of concrete 15 and 20 mm, which represents half the size of concrete elements in the finite element model of the analysis of size 30 mm and 40 mm respectively. The resulting bond-slip curves show a brittle drop in the shear stress after the peak bond stress is achieved as shown in Fig. 9. To account for this drop, Bond-Slip Model II is adopted for the interface between the concrete and FRP in regions of major flexural cracking. This model follows the same response as bond-slip model I up to the peak stress but is modified to account for the drop as follows:

$$\tau = \begin{cases} \frac{\tau_{max} s}{s_o} \; if \; s \leq s_o \\ 0 \; if \; s > s_o \end{cases} \tag{2(a)}$$

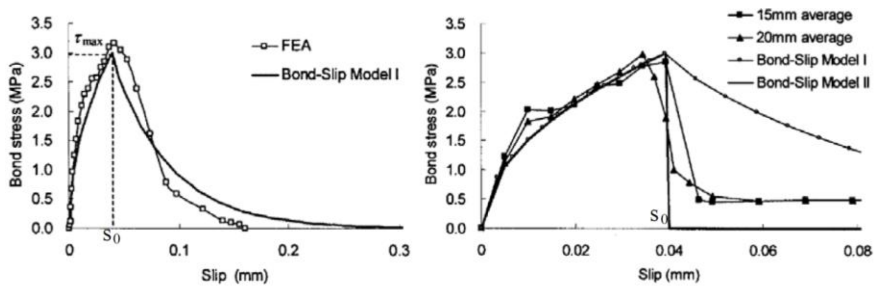


**Figure 7: Bond-slip curves for FRP-Concrete zones outside (left) and inside (right) of major flexural cracking zones (Lu et al. 2007)**

In this study, the externally-bonded carbon fibre tow sheets are modeled as discrete reinforcing units attached to the concrete through interface elements, referred to as link elements, that follow the Bond-Slip Model I if the concrete does not have major flexural cracks, and Bond-Slip Model II if major flexural cracks are present. A “major flexural crack” is defined here as a crack that produces a total slip in the FRP-concrete interface greater than the limit given by Equation 2. Thus, a careful monitoring of the crack widths in all concrete elements during each load step is required. In this investigation, the concrete is connected to the FRP through link elements as shown in Fig. 8. Note that once an interface element follows Bond-Slip Model II, it effectively acts as a spring with zero stiffness since no stresses are transmitted from the FRP to the concrete.

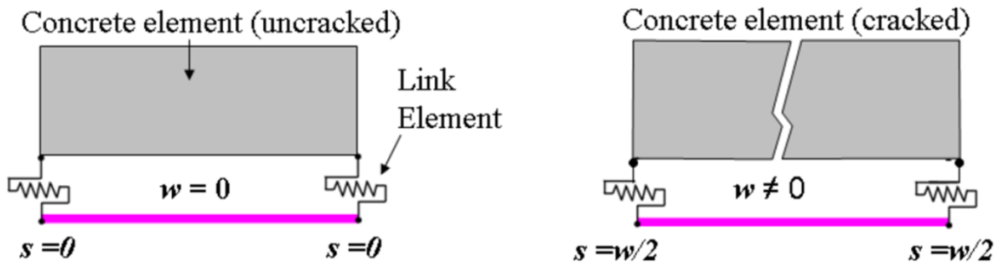


**Figure 8: Slips at interface elements (Cruz-Noguez, 2012)**

Fig. 8 shows that in any given concrete element with a flexural crack of width *w*, the interfacial FRP-concrete slips *s* that appear at both sides of the flexural crack can be approximated as *w*/2 (Lu et al. 2007). Since IC debonding is considered to take place if $s>s_o$, the crack width in a concrete element that causes FRP-concrete debonding should be therefore equal or larger than $2s_o$. However, if the crack width in a concrete element is smaller than $2s_o$, the bond-slip relationships from Equations 1 still apply.

## 2.4. Simplified bond-slip model for shear cracks

The modeling technique discussed in the previous sections has been discussed in details by Cruz-Noguez (2012). Although it proved to be effective in providing satisfactory results when modelling flexural walls, it is found not appropriate for modelling shear deficient walls for a number of reasons. The major cracks observed in a two-dimensional shear wall are no longer flexural cracks but rather shear cracks, which result in a completely different FRP-Concrete interaction mechanism.

Similar to the vertical FRP discussed earlier, the horizontal FRP sheets are modeled similarly as discrete truss elements. In the experiment, the vertical FRP are applied to the concrete surface with the horizontal FRP layers overlay on top of the vertical layers as shown in Fig. #9. This indicates that in reality the horizontal FRP is not directly connected to the concrete substrate but rather they were connected to the vertical FRP. To account for this layout of the FRP sheets in the specimens, the horizontal FRP trusses are connected to the concrete mesh nodes with link elements having the same Bond-slip models of the vertical FRP described earlier i.e. Bond-Slip model I in regions with no major cracking and Bond-Slip model II in regions with major cracking. Since the vertical and horizontal FRP layers never debond from each other in any of the experiments, it indicates that the horizontal and vertical FRP debonded simultaneously. Thus, it is appropriate to adopt identical the debonding criteria for the vertical and horizontal FRP.

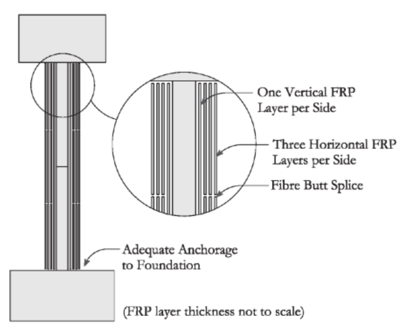


**Figure 9: FRP reinforcing scheme (Woods,2014)**

It is noted that major cracks in shear dominant walls are expected to be predominantly diagonal shear cracks. This has been verified by observations from testing of the control walls. The orientation of the shear cracks is different from that of the flexural cracks in the shear wall specimens as shown in Fig. 10.

This has an influence on the determination of the interfacial FRP-Concrete slip s in a two-dimensional wall panel element. When a crack occurs at an angle θ, the vertical component of the slip that controls the debonding of the vertical FRP sheets requires a crack width in the concrete as follows in Bond-Slip model II:

$$w \geq {}^{2 \times s_o}/_{cos\theta} \qquad 2(a)$$

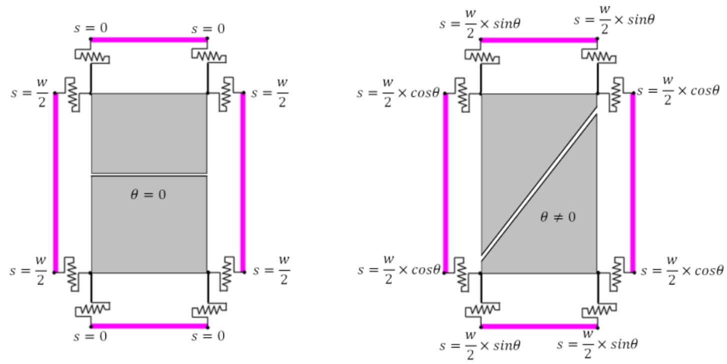


**Figure 10: Calculating the slip based on different crack orientations**

A modification is also made to account for the mesh size effect in the finite element model of the present analysis. Details of this consideration are discussed in the reference by Hassan (2015).

## 3. FE results and discussion

### 3.1. Plain RC walls

To evaluate the validity and accuracy of the proposed FE model in predicting the response of the shear dominant walls, the analytical results of force-displacement response for the control walls (both the slender and intermediate) are compared with measured data from the experiment. The displacement is measured at the top of the wall while the load was obtained from the actuator applying the load to the cap beam. The results for the analytical and experimental results are observed to be in close agreement, as shown in Fig. 11. The correlation of slender wall result is not as well because of issues encountered during testing. These are discussed in details in the reference by Woods (2014).

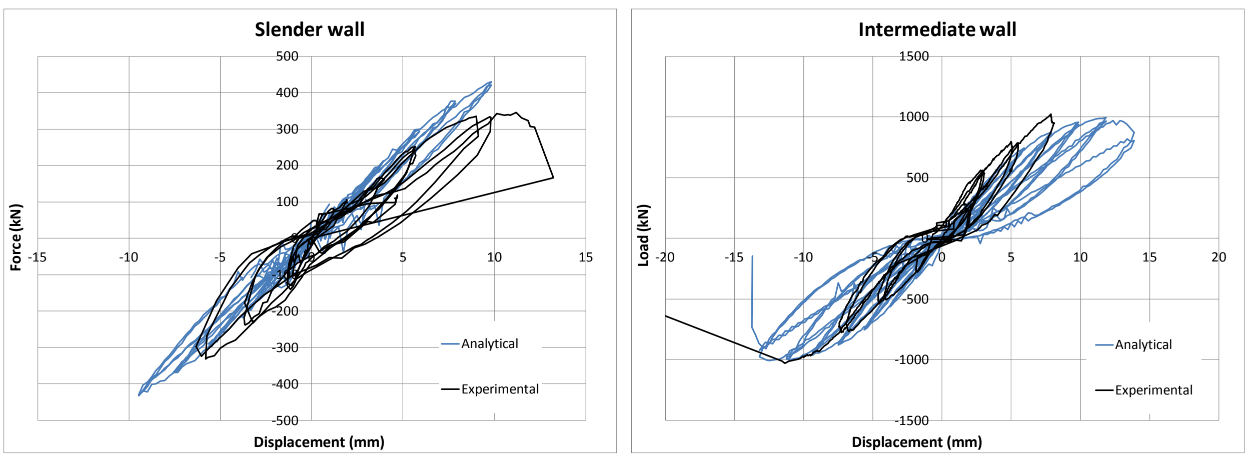


**Figure 11: Force – Displacement curves for both intermediate and slender control walls**

### 3.2. FRP Strengthened walls

From the analysis of the strengthened walls with considerations of the IC debonding mechanisms, Fig. 12 shows the analytical results of the force-displacement curve for the intermediate and slender wall compared to the experimental results. Figure 12 shows that the force-displacement response calculated for the slender wall closely represents the ultimate strength of the strengthened wall. However, the initial stiffness is slightly underestimated. For the case of the intermediate wall, it is noticed that the strength is underestimated which can be attributed to several factors. The use of several FRP plies per side produced a very stiff laminate (Lombard 1999; Hiotakis 2004) with some flexural capacity, and therefore the use of a truss element with no compressive resistance to represent the FRP material may not be entirely appropriate because it does not account for their impact on the flexural and shear strength of the wall. In addition, using a number of horizontal FRP sheets adds confining pressure that strengthens the concrete and delay the cracking of concrete. The confining effects cannot be considered since FRP trusses were used to model the FRP sheets. Finally, the debonding criterion used is based on meso-scale analysis done on beams with major flexural cracks. The effect of shear cracks on debonding may be different from those of flexural cracks and thus further investigation of the FRP-Concrete debonding mechanism in 2-dimensional shear dominant structures such as walls should be carried. However, as shown in Fig. 12, the model is still able to produce reasonable predictions of maximum strength and initial stiffness, parameters that are useful in design applications.

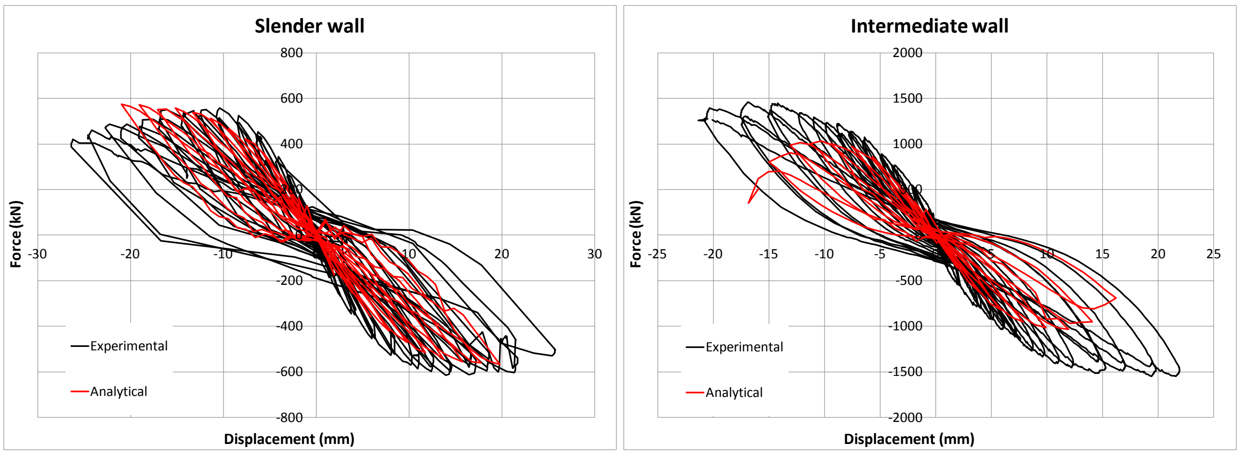


**Figure 12: Force – Displacement curves for both intermediate and slender strengthened walls**

## 4. Conclusions

This paper presented the analytical modeling of walls strengthened using externally-bonded CFRP reinforcement for shear dominant walls. The conclusions that can be drawn from the study are:

a) The modeling process presented in this paper has been shown to be effective in predicting satisfactorily the response of strengthened walls with a multiple layers of FRP per side. The model gives reasonable correlation between calculated and measured results for strengthened walls.
b) The debonding criterion used in this study is simple and easy to implement in FE packages that do not allow the development of user-defined elements, such as the one used in this study.

c) Several improvements can be done to improve the correspondence between the analytical and experimental results if more relevant meso-scale testing can be done to determine the effect of shear cracks on debonding of FRP sheets.

## 5. Acknowledgements

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