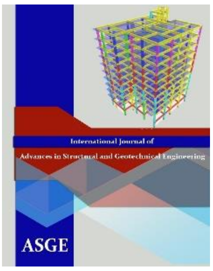

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F.E. ANALYSIS OF R.C. WALL WITH OPENING USING DIFFERENT STRENGTHENING TECHNIQUES

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F.E. ANALYSIS OF R.C. WALL WITH OPENING USING DIFFERENT STRENGTHENING TECHNIQUES

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ABSTRACT

Earthquake is a catastrophic event, which makes enormous harm to properties and human lives. Shear walls are used to oppose the horizontal loads that might be incited by impact of wind & earthquakes and give additional stiffness for the structures. R.C walls in residential buildings might have openings that are required for windows, doors or different states of openings. Shear walls can encounter harm around corners of entryways and windows because of advancement of stress concentration under the impact of vertical or horizontal loads. To achieve safe, economic and functional buildings, strengthening of structural members using several methods has gained a great deal of attention in recent years and proved the efficiency in increasing capacity of the shear walls with opening. Strengthening R.C. wall is divided into two main types: pre-strengthening and post-strengthening. Pre-strengthening includes adding extra internal steel bars around openings before casting. On the other hand, Post-strengthening includes using externally bonded FRP sheets with different development lengths and near surface mounted (N.S.M) FRP and steel bars.

Comparative study by using Finite element modeling approach " ANSYS" has been conducted to compare between several methods used in strengthening R.C. wall with openings like pre-strengthening and post-strengthening schemes .

The proposed F.E. approach has been verified experimentally and mathematically with other experimental programs and design models and given a very good correlation between the model and experimental outputs including load capacity, failure mode, as well as crack pattern and lateral displacement. A parametric study also is applied to investigate effects of increasing the horizontal and vertical CFRP development length. Moreover, influences of re-arrangement of CFRP layers on rehabilitation of concrete shear walls is examined.

This research may be useful for improving existing design models and to be applied in practice, as it satisfies both the architectural and the structural requirements.

Keywords: Opening, Shear wall, FRP, F.E.M, ANSYS

INTRODUCTION

The lateral forces which affect buildings are resisted by shear walls that are designed as vertical structural elements. Wind and earthquakes are the main reasons which induce the lateral loads. However, shear walls are frequently pierced for doors, windows and building services or other functional reasons. Openings are usually avoided in reinforced concrete structural elements because the size and location of openings in the shear wall may have adverse effect on seismic responses. These openings are also source of weak points and cause decrease inside the structure's stiffness and load-bearing capacity. So, it is important to know the impacts of large openings sizes and configurations in shear wall on stiffness and also on seismic responses and behavior of structural system as a much amount of concrete and reinforcing steels has to be removed. In addition, it is too necessary to evaluate the several ways to strength these R.C. walls in order to obtain an optimum strengthening technique [1].
Openings in the R.C. walls determine the load paths and create stress concentrations around the opening, which induce cracks to happen first at the corners of the opening [2]. These cracks around the opening corners require wall strengthening. Embedding reinforcing bars is the traditional strengthening methods for cut-outs. On the other hand, advanced composites as externally bonded FRP sheet and near surface mounted reinforcement have been extensively tested on their use for girders and beams strengthening in shear, flexure and torsion.
Today, using CFRP in strengthening the RC wall panels with openings become popular due to tolerability, small weight, high strength, and high performance against the corrosion, low density, its simple installation and perfect fatigue characteristics. CFRP also helps in preventing brittle failure and reducing crack formation in whole corners. In 2014, Lima, et al. [3] tested six half-scaled wall panels with the slenderness ratio of 30 to study the behavior of RC walls with opening. The panels were subjected to uniform axial eccentric load, restrained on two and three sides and strengthened with two types of CFRP arrangements around opening. They concluded that CFRP leads to increasing the ultimate load capacity of the walls under different load conditions and inserting new restraints leads to capacity increase.
Mohammed et al. [4] proposed mathematical design equations based on experimental tests of eight R.C. walls with two different CFRP layouts including DF (45o to the opening corner) and AF (all around the opening). The openings were considered at the center of the R.C. wall with different opening areas (5, 10, 20, and 30 %). The test outcomes and mathematical design equations indicated that the externally bonded (EB) CFRP applications on R.C. walls supported on top and bottom only (one way action) would increase the ultimate axial strength of the wall between 10% and 80% depending on the opening size and CFRP arrangement.
Behavior of shear wall with openings which is strengthened by NSM-CFRP has been studied in number of researches. C. Todut, et al. [5] conducted experimental work to examine the ultimate strength of shear wall with openings under the effect of NSM-CFRP strengthening. Nine elements that were reinforced and casted and the web-panel dimensions were 2750 mm in length, 2150 mm in height and 100 mm in thickness. The wall panels were loaded by quasi-static cyclic reversed lateral loading - displacement controlled, with two cycles per drift. Additional axial loads were applied to simulate the gravity loading condition and to restrain the rotation of the wall panels. The test program demonstrated the retrofitted specimens developed a higher lateral resistance than the reference elements.
Despite of the growing interest in modeling and analyzing behavior of shear walls, there are limited experimental studies that were conducted to explore the efficiency of strengthening ways especially for walls with openings. Consequently, the behavior of strengthened RC walls with openings can be investigated by the use of ANSYS. If the material properties have been implemented properly, ANSYS could simulate the elastic and plastic deformations that would take place till ultimately crushing due to increasing the load.

RESEARCH SCOPE AND OBJECTIVE

Finite element approach will be used after being accurately verified experimentally and mathematically. The study includes a parametric study to gain an optimum strengthening technique in R.C. wall with opening to increase capacity and dominance cracks. It will explore the behavior of shear walls with openings which are pre-strengthened by embedding extra internal steel bars around openings or post-strengthened by externally bonded FRP sheets with different development lengths and near surface mounted (N.S.M). Also, the effect of increasing the horizontal and vertical CFRP development length will be studied. Moreover, the influence of geometric characteristics and re-arrangement of CFRP layers on rehabilitation of concrete shear walls is examined. This paper will contribute in enhancing R.C. walls behavior. The ongoing program is expected to significantly extend the findings of the previous studies and present a verified F.E. approach, which helps for more research in this field.

RESEARCH METHODOLOGY:

The research plan includes two phases. The first phase includes experimental and mathematical verification of the results conducted by other researchers using ANSYS model and ensure the correlation between both F.E. and experimental results for load capacity, failure mode and lateral displacement. The research depends on two different experimental programs using four different F.E models (with and without CFRP) to be more confident with the model results.

In the second phase after verification the model with the experimental and mathematical outputs, a parametric study has been conducted to explore the behavior of shear walls with openings which are pre-strengthened by embedding extra internal steel bars around openings or post-strengthened by externally bonded FRP sheets with different development lengths and near surface mounted (N.S.M). Also, the effect of increasing the horizontal and vertical CFRP development length will be studied. Moreover, the influence of geometric characteristics and re-arrangement of CFRP layers on rehabilitation of concrete shear walls is examined.

FINITE ELEMENT ANALYSIS OF SHEAR WALL WITH OPENINGS

ANSYS finite element software is used to model two experimental programs of reinforced concrete shear wall with and without CFRP loaded in the model up to failure, which have a symmetric opening (Lima, MM, et al.) [3] and (Bashar S. Mohammed, et al.) [6]

Nonlinear response of RC wall is developed by cracking, plastic deformations in compression, crushing of the concrete and plastic deformations of the reinforcement.

Experimental data used for model verification

First model verification [3]

Three shear wall specimens named OW-NF (without CFRP), OW-AF (with CFRP) and OW-PF (with CFRP) designed to represent typical wall panels in residential buildings (1200 mm long, 1250 mm tall and 40 mm thick), modeled for testing to failure, they have symmetric openings (450 mm x 450 mm) as shown in figure 1. The specimens are loaded by axial loads and reinforced by centrally positioned single layer of welded wire fabric reinforcement, comprising of deformed 4 mm diameter bars with 100 mm spacing in the both directions. There are hinged connections at the top and bottom boundaries of the specimen and free side edges. The average cylinder compressive strength of the concrete was 54.7 MPa. Steel mean yield strength (fy) was 500 MPa. As presented in figure1, CFRP sheets in OW-AF are externally bonded alongside the opening with thickness (0.128 mm) and tensile modulus 234000 Mpa. Regarding OW-PF, CFRP sheets are externally bonded from the top to the bottom of the shear wall parallel to the axial load direction. Three hydraulic jacks, each with a most extreme limit of 800 kN, were connected together to apply a uniformly distributed load, with controlled total force, along the wall length. The specimens were subjected to an eccentricity of one sixth of the wall thickness to obtain the curvature and tension side in a specific side. A steel rod was welded to both of loading beam to apply eccentric distributed loading, designed to fit into a guide system connected to the upper edge and lower edge of the specimen as illustrated in figure 1 [3].

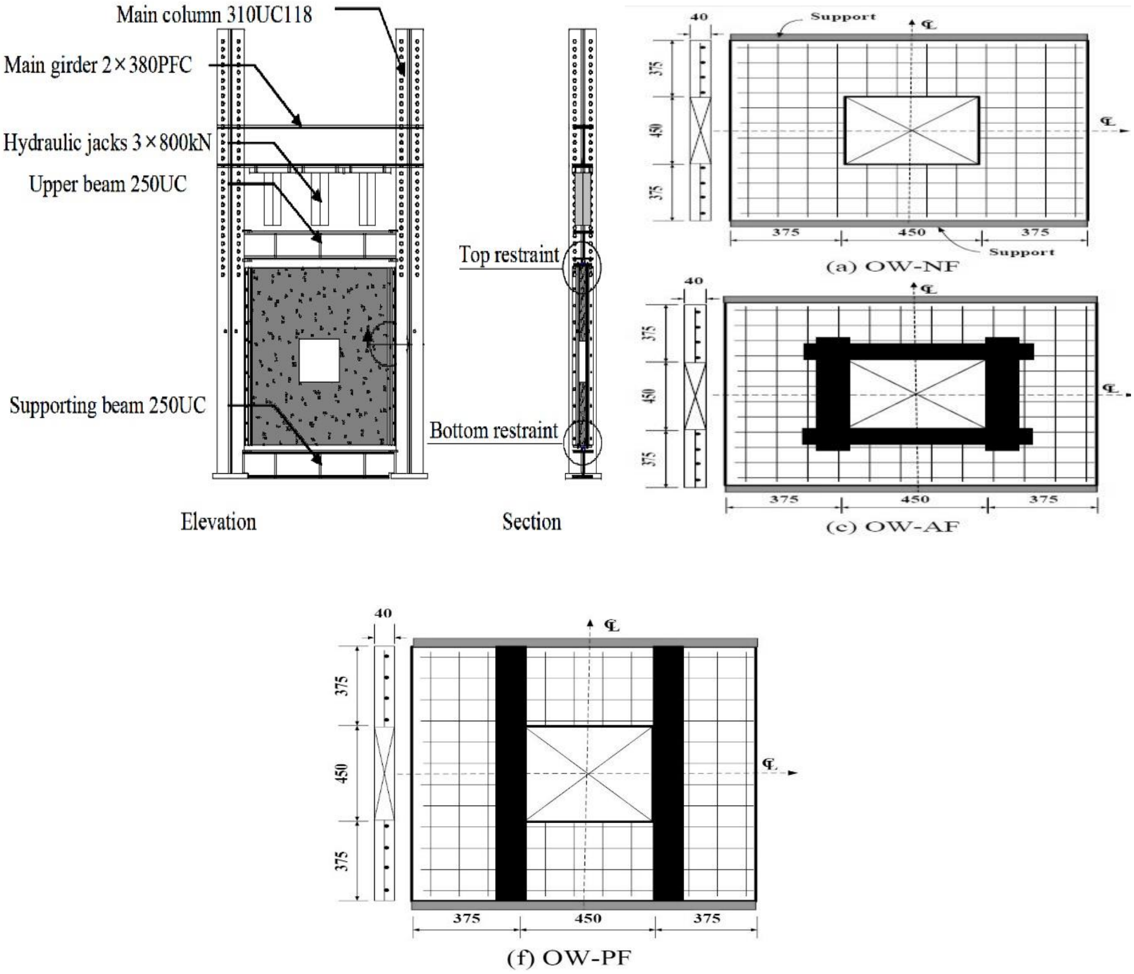


Fig. 1: OW-NF & OW-AF and OW-PF Specimen Test Setup [3]

Second model verification [6]

Two shear wall specimens named WO2a (without CFRP) and WO2b (with CFRP) designed to represent typical wall (400 mm long, 800 mm tall and 40mm thick), modeled for testing to failure, they have symmetric opening (135 mm x 240 mm) as shown in figure 2. The specimens were cast with constant thickness. The specimens are loaded by axial loads and reinforced by centrally positioned single layer of welded wire fabric reinforcement, consisting of deformed 5 mm diameter bars. There are hinged connections at the top and bottom boundaries of the specimen and free side edges. The concrete used to cast the specimens was a self-consolidating blend that could be poured without vibrating it. The average cubic compressive strength of the concrete was 17.43 MPa (WO2a) & 21.35 Mpa (WO2b). Steel mean yield strength (fy) was 478 MPa. Regarding the specimen WO2b, CFRP sheets are externally bonded alongside the opening with thickness (0.167 mm) and tensile modulus 230000 Mpa.
A hydraulic jack with a with a most extreme limit of 300 kN, applies a distributed uniform load, with controlled total force, along the wall length. The specimens were subjected to an eccentricity of one sixth of the wall thickness and a steel rod was welded to both of loading beam in order to apply eccentric distributed loading, designed to fit into a guide system connected to the upper edge and lower edge of the specimen as illustrated in figure 2 [6].

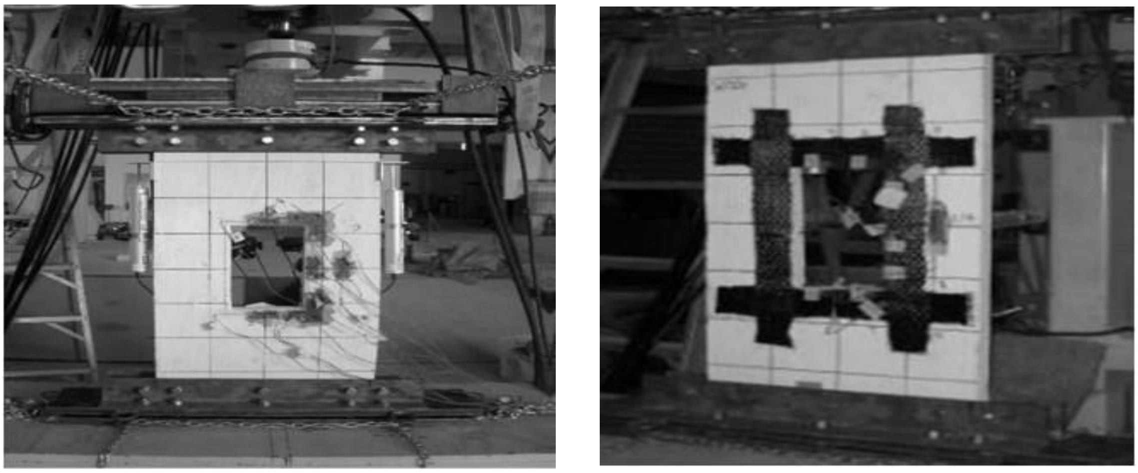


Fig. 2: Test Setup of WO2a & WO2b Specimens [6]

Material Model

Modeling of Concrete

Solid65 element models the nonlinear behavior of reinforced concrete and this element is based on a constitutive model for the triaxial behavior of concrete after Williams and Warnke. This element is isoperimetric element which is characterized by eight nodes, each having three translation degrees of freedom in the nodal x, y and z-directions. The geometry, node positions and the coordinate system for the element is appeared in figure 3. It is able to simulate plastic and elastic deformation, crushing in compression and cracking in tension in three perpendicular directions at each integration point as the load increases [7].

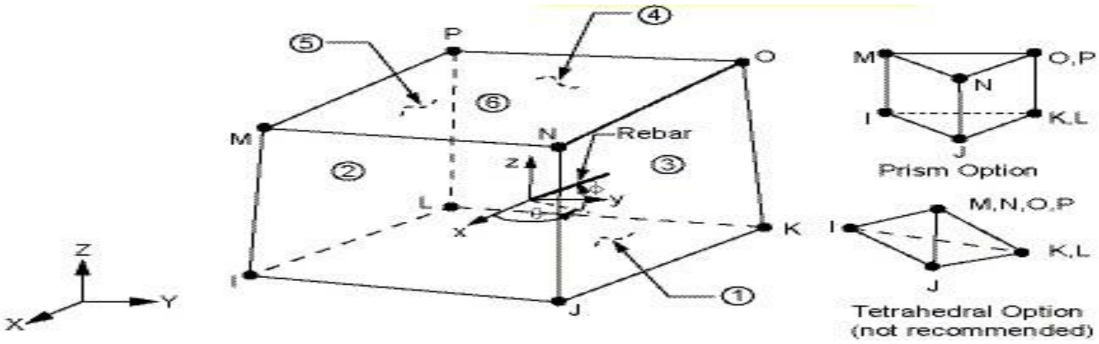


Fig. 3: Solid 65 element

Changing the element stiffness matrices conducts an adjustment in the material properties, which helps in the cracking modeling. The complete deterioration of the structural integrity of the material (material spalling) is defined as crushing in Solid 65. If the material fails at an integration point in uniaxial, biaxial or triaxial compression, the material is assumed to be crushed at that point. The von Mises failure criterion is used to model the multi-linear isotropic concrete along with Willam and Warnke model [8] to define the failure of concrete.
(Chinese standard GB 50010) [9] & (Rüsch model) Hubert Rüsch, et al. [10] equations can obtain the compressive uniaxial stress-strain relationship for the concrete model as illustrated in the following equations and figure 4.

When fcu > 50 MPa [9]

$$\left.\begin{array}{ll} \sigma_c = f_c\left[1-\left(1-\frac{\varepsilon_c}{\varepsilon_0}\right)^n\right] & \varepsilon_c \leqslant \varepsilon_0 \\ \sigma_c = f_c & \varepsilon_0 < \varepsilon_c \leqslant \varepsilon_{cu} \end{array}\right\}$$

$$n = 2 - \frac{1}{60}(f_{cu} - 50)$$

$$\varepsilon_0 = 0.002 + 0.5(f_{cu} - 50) \times 10^{-5}$$

$$\varepsilon_{cu} = 0.0033 - (f_{cu} - 50) \times 10^{-5}$$

When fcu < 50 MPa [10]

$$\sigma_c = f_c\left[2\frac{\varepsilon_c}{\varepsilon_0} - \left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2\right] \quad (\varepsilon_c \leqslant \varepsilon_0)$$

$$\sigma_c = f_c \quad (\varepsilon_0 < \varepsilon_c \leqslant \varepsilon_{cu})$$

Where
σ_c: the stress in concrete corresponding to the compressive strain εc
f_c: the axial compressive strength of concrete
ε_o: the compressive strain corresponding to fc
ε_{cu}: the ultimate compressive strain
f_{cu}: the cube strength of concrete
n: a parameter

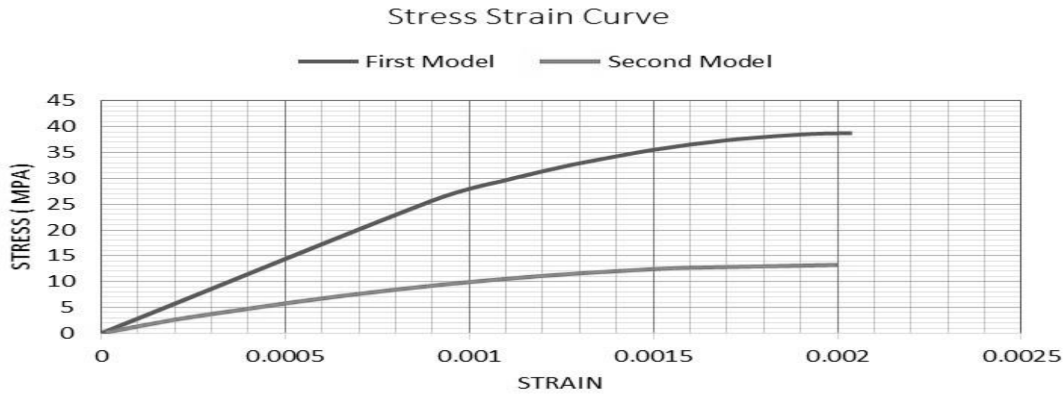


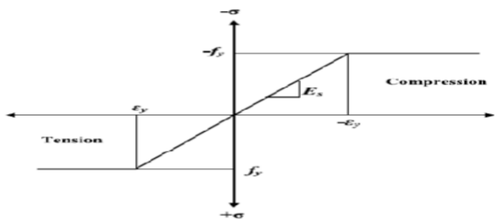
Fig. 4: Concrete stress strain curve

Modeling of Steel Reinforcement

The link8 element models the nonlinear response of reinforcement bars which may be included in the finite element model as a discrete model (individual bars). As shown in figure 5, prior to initial yield surface steel material model is linear elastic, after the initial yield surface it is completely plastic, in compression and tension loading. Figure 6 shows the geometry, node positions and the coordinate system for the element. The parameters selected to define the material properties of steel are given in table 1.

Table 1: Material Properties for Steel

Linear Isotropic		
Es "MPa"	200000	
Poisson's ratio	0.3	
Bilinear Isotropic		
	1st experimental program	**2nd experimental program**
Yield Stress "MPa"	500	478
Tang Modulus "MPa"	500	478



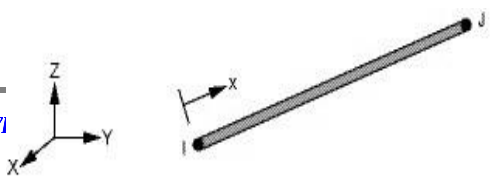


Fig. 5: Stress-strain curve for steel reinforcement

Fig. 6: Link8 element

Modeling of Epoxy & CFRP

The solid65 element models the nonlinear response of epoxy. The CFRP is represented with 4-node shell elements (Shell41 element) which represents a linear elastic orthotropic material.

Table 2: Material Properties for Epoxy

	1st experimental program	2nd experimental program
Modulus of Elasticity "MPa"	2500	2500
Elongation at breaking point	1.2%	1.2%
Tensile Strength "MPa"	30	30

Table 3: Material Properties for CFRP

	1st experimental program	2nd experimental program
Modulus of Elasticity "MPa"	234000	230000
Elongation at breaking point	0.9%	2.1%
Thickness "mm"	0.128	0.167

Meshing & Load steps

The mesh generation directly affects the accuracy of F.E. analysis results. The mesh generation method is mainly determined by the element type and shape. Perfect simulation needs highly refined meshes. The panels are meshed with specific material characteristics by using 8-node elements called Solid 65 for concrete, link 8 for reinforcement steel, solid 185 for loading plates, solid 65 for epoxy and shell 41 for CFRP. A dense mesh of this element type may be required in order to obtain accurate results during the analysis. ANSYS parametric design language (APDL) generates the mesh. In this method of mesh generation, the nodes are assigned to specific coordinates with ordered numbering. Then, meshed elements are formed after the nodes are joined together. The accuracy of the model, including objectivity issues related to mesh geometry and size, is demonstrated through several mesh sensitive studies, which were performed to select the optimum mesh sizes. In the models of first and second experimental program, the elements have a length of 25 mm. In the specimen named OW-NF in the first experimental program, there are 11050 nodes in the model, which are connected together to form 14692 elements as shown in figure 7. Specimen named OW-AF in the first experimental program, there are 11737 nodes in the model, which are connected together to form 15448 elements. Specimen named OW-PF in the first experimental program, there are 11738 nodes in the model, which are connected together to form 15520 elements. Concerning the second experimental program, there are 2975 nodes in the model, which are connected together to form 2416 elements in the WO2a. Regarding specimen named WO2b in the second experimental program, there are 10949 nodes in the model, which are connected together to form 5058 elements. Automatic time stepping was used to solve the FE model with a specific number of substeps (1000) depending on the material properties, the value of loads and element density. In order to carry out a load-displacement curve based on non-linear analysis in ANSYS, the load should be broken into a series of load increments by defining number of load steps (200) which equals to the applied pressure value in the first experimental program and (60) which equals to the applied pressure value in the second experimental program, increment in load to be applied in each step and maximum load to be applied. The model must be always checked back to determine the exact load step at which the wall failed.

MODEL VERIFICATION

F.E. modeling approach has been conducted and verified experimentally and mathematically with two experimental programs of five different specimens conducted by (Lima, MM, et al.) - Australia [3] and (Bashar S. Mohammed, et al.) – Canada [6]. After verification, a parametric study is applied to investigate the behavior of shear walls with openings which are pre-strengthened by embedding extra internal FRP & steel bars around openings or post-strengthened by externally bonded FRP sheets with different development lengths and near surface mounted (N.S.M). Also, the effect of increasing the horizontal and vertical CFRP development length wall be studied. Moreover, the influence of geometric characteristics and re-arrangement of CFRP layers on rehabilitation of concrete shear walls is examined.

Verification of first experimental program [3]

Verification for load & ductility index

The validity of the proposed material constitutive models for steel, concrete, epoxy and CFRP were verified by comparing their predictions with experimental data conducted from testing reinforced concrete shear wall with and without CFRP. The wall dimension is (1200 mm long, 1200 mm tall and 40 mm thick and has a symmetric opening (425 mm x 425 mm) which named OW-NF (without CFRP), OW-AF (with CFRP) & OW-PF (with CFRP) [3]. The results of the verification study, table 5, demonstrated that the F.E. model fitted with the experimental and mathematical results of the reference wall. The measured maximum capacity and corresponding out of plane displacement in the reference wall were (OW-NF) 266 kN and 7.8 mm, respectively. On the other hand, the F.E. predictions obtained for maximum capacity and corresponding out of plane displacement were 288.8 kN and 8.9 mm, respectively.
Another specimen (OW-AF) has been conducted for verification, which has a symmetric opening (450 mm x 450 mm) and externally bonded by CFRP sheets alongside the opening [3]. The results shown in table 4 demonstrated that the F.E. model fitted with acceptable accuracy the experimental and mathematical results of the reference wall. The measured maximum capacity and corresponding out of plane displacement in the reference wall were 335.7 kN and 8.8 mm, respectively. On the other hand, the F.E. predictions obtained for maximum capacity and corresponding out of plane displacement were 345.9 kN and 9.02 mm, respectively.
Specimen (OW-PF) has a symmetric opening (450 mm x 450 mm) and externally bonded from the top to the bottom of the shear wall parallel to the axial load direction. The results shown in table 5 demonstrated that the F.E. model fitted with acceptable accuracy the experimental results of the reference wall. The measured maximum capacity and corresponding out of plane displacement in the reference wall were 335.7 kN and 8.8 mm, respectively. On the other hand, the F.E. predictions obtained for maximum capacity and corresponding out of plane displacement were 345.9 kN and 9.02 mm, respectively.

Table 4: F.E. and experimental results [3]

Serial	Experimental Verification Ultimate load, Pu (kN)			Out of plane displacement (mm)		
	F.E.	Exp.	Accuracy %	F.E.	Exp.	Accuracy %
OW-NF	288.8	266	8.6	8.9	7.8	14.1
OW-AF	345.9	335.7	3.0	9.02	8.8	2.5
OW-PF	379.9	359.85	5.6	5.33	5.5	3.1

Crack Pattern and failure mode

The specimens OW-NF, OW-AF and OW-PF fail in a mode of deflection in a single curvature with a maximum deflection happening close to the middle of the wall panel as illustrated in figure 7. The cracks in OW-NF are horizontal throughout the middle of the opening. The application of CFRP changed the load path and therefore changed the shape of the crack pattern due to resistance of CFRP. Applying CFRP induced distributed cracks when compared to the corresponding control specimen without CFRP as shown in figure 8. In addition, as shown in the experimental pictures in figure 8, on tension side of the R.C. wall panels, there was no evidence of de-bonding between CFRP and concrete before ultimate load was achieved and even after the failure. Moreover, the concrete stress (42 mpa) exceeds the strength (38 mpa) presented in the stress strain curve in figure 4. Also, the epoxy strain does not exceed the epoxy ultimate strain value (1.2%). In addition, the CFRP sheet strain does not exceed the CFRP ultimate strain value (0.9%).That values ensure that the failure mode was (crushing of concrete) which is compatible with experimental pictures in figure 8. Comparing the crack pattern of sample specimens at failure predicted numerically to that obtained from the experiment in figure 8, there is a good correlation between the experimental and F.E. crack patterns.

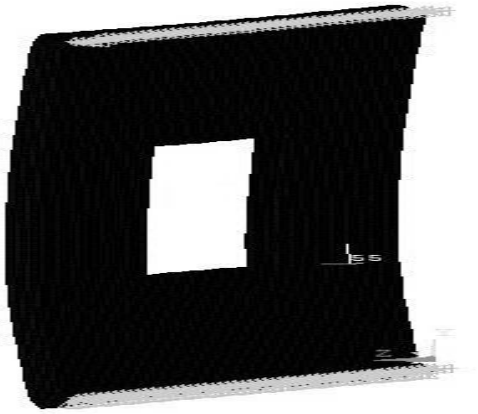


Fig. 7: Deformed shape

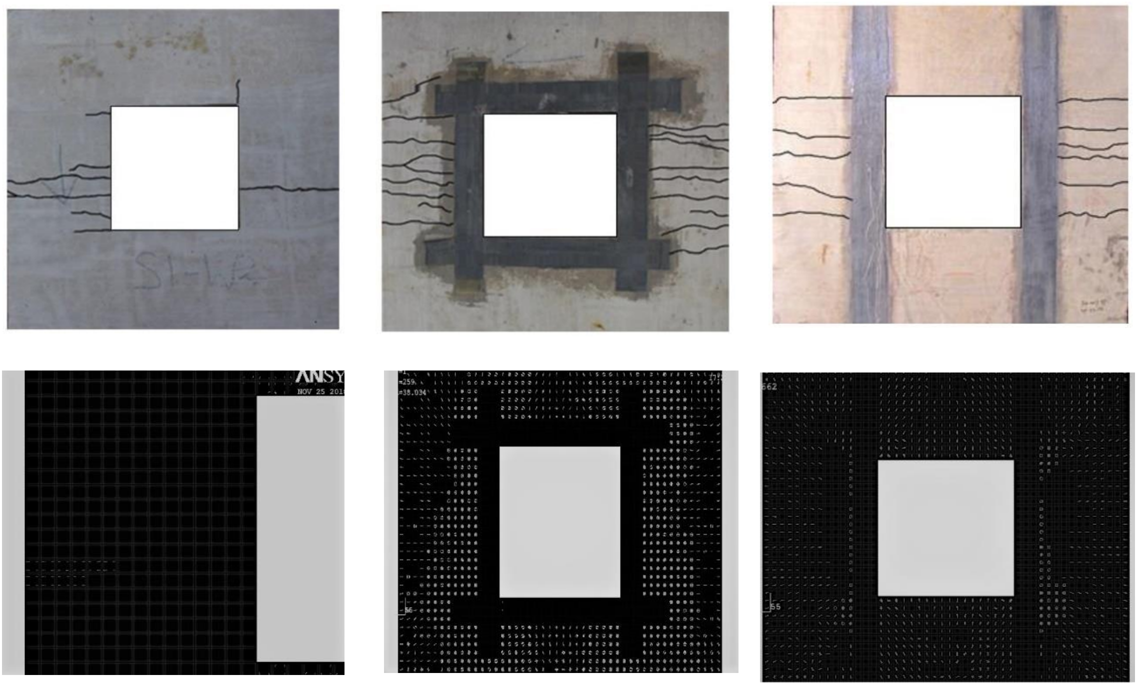


Fig. 8: Cracks in experimental and F.E. specimen for OW-NF, OW-AF & OW-PF

Verification for second experimental program [6]

Verification for load capacity

Proposed material constitutive models for steel, concrete, epoxy and CFRP were verified by comparing their predictions with another experimental data conducted from testing reinforced concrete shear wall [6]. The reference wall dimensions were (400 mm long, 800 mm tall and 40mm) and has a symmetric opening (240 mm x 135 mm). The results of the verification study demonstrated that the F.E. model fitted with the experimental and mathematical results of the reference wall. The measured maximum capacity in the reference wall (WO2a) was 100 kN. On the other hand, the F.E. predictions obtained for maximum capacity was 109.82 kN with 9.82% variation.
Another specimen (WO2b) has been conducted for verification. The results demonstrated that the F.E. model fitted with acceptable accuracy the experimental and mathematical results of the reference wall. The measured maximum capacity in the reference wall was 139.1 kN. On the other hand, the F.E. predictions obtained for maximum capacity was 166 kN with 19.3% variation as presented in table 5.

Table 5: F.E. and experimental results [6]

Serial	Experimental Verification, Ultimate load, Pu (kN)		
	F.E.	Exp.	Accuracy %
WO2a	109.82	100	9.82
WO2b	166	139.1	19.3

Crack Pattern and failure mode

The specimen WO2a fails in a mode of deflection in a single curvature with a maximum deflection happening close to the middle of the wall panel as illustrated in figure 9. At 51 kN, the cracks begin from the center of the opening, parallel with the loading direction towards the applied loads. Followed by a crack from the center of the opening, parallel with the loading direction towards the bottom of the wall panel. Other than that, the cracks also happened near the middle of the wall panel, orthogonal to the loading direction, which causes the failure of the wall panel at load 109.82 kN as appeared in figure 10. Comparing the crack pattern of sample specimen at failure predicted numerically to that obtained from the experiment in figure 11, there is a good correlation between the experimental and F.E. crack patterns.

Fig. 9: Deformed shape

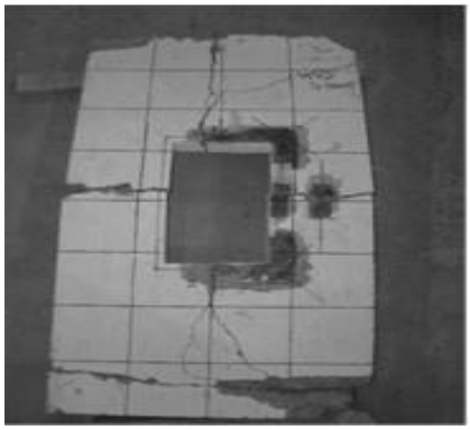
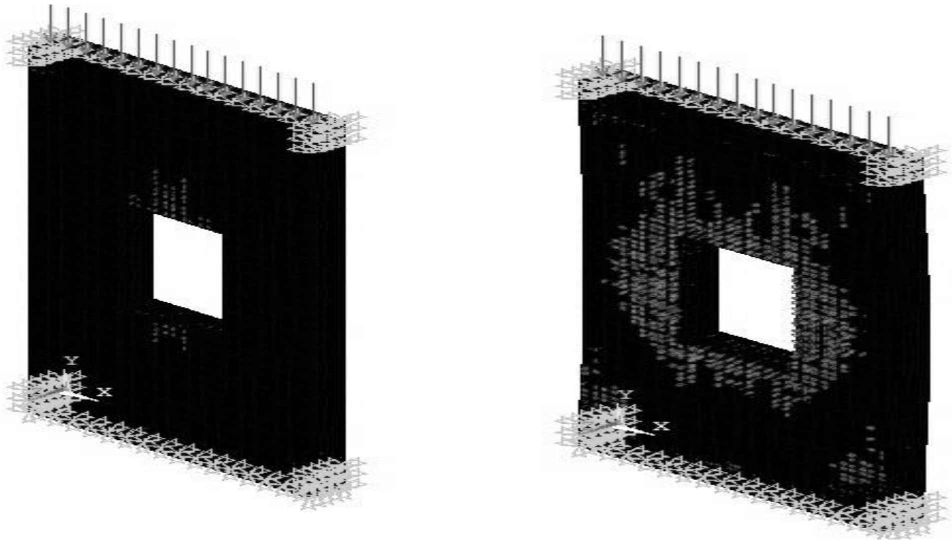


Fig. 11: Cracks in experimental specimen



At 51 kN **At failure (109.82 kN)**

Fig. 10: Crack propagation

Wall panels with openings strengthened with CFRP (WO2b) displayed different crack pattern compared to the wall panels without CFRP. Figure 12 shows a single curvature with a maximum deflection occurring close to the middle of the wall panel. The following figures show some crack patterns for the wall panels with openings strengthened with CFRP, where the CFRP applied along the wall panel opening. The advantages of applying CFRP along the wall panel opening is that the wall panels will fail, either near the center of the wall or horizontally from the opening corner as appeared in figure 13. Comparing the crack pattern of sample specimen at failure predicted numerically to that obtained from the experiment in figure 14, there is a good correlation between the experimental and F.E. crack patterns.

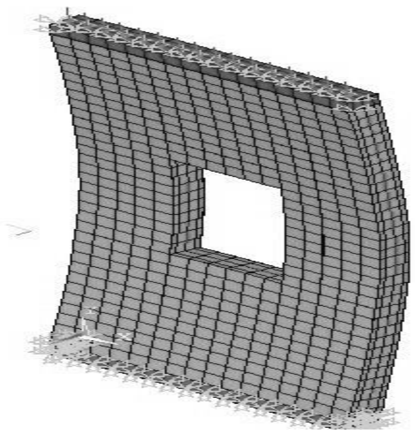


Fig. 12: Deformed shape

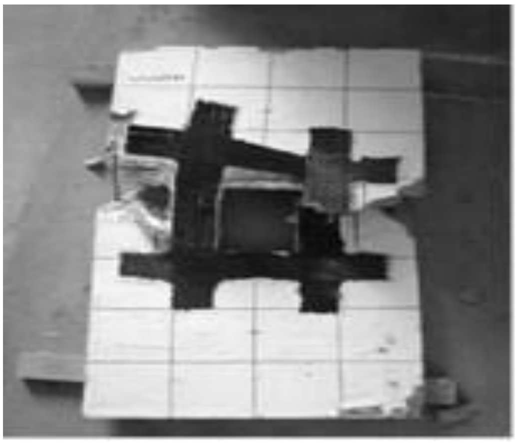
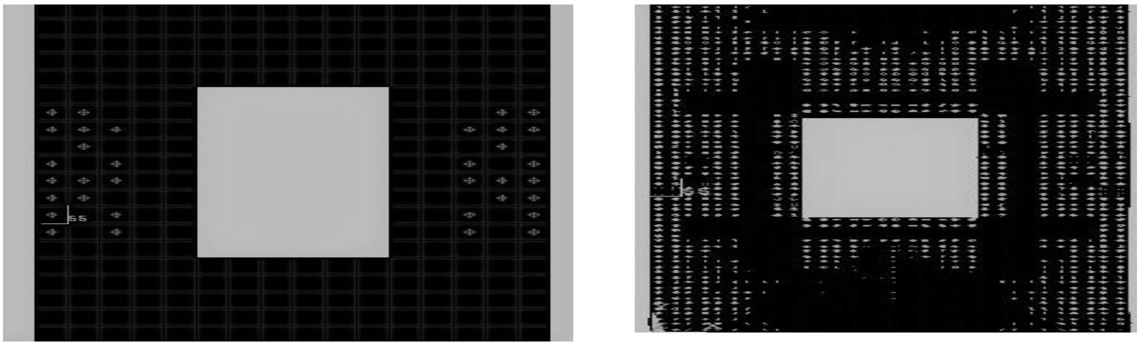


Fig. 14: Cracks in experimental specimen



At 110 kN At failure (166 kN)

Fig. 13: Crack propagation

Failure mode in the F.E. models fitted with the experimental results of the reference walls, which confirms the capability of the F.E. models to accurately predict the load capacity of other models of shear walls and simulate the nonlinear structural behavior of opened shear walls to examine a larger domain of parameters instead of laboratory testing, which is expensive, time-consuming and labor-dense. After verification of the finite element method with the proposed reference models, several arrangements of openings with a variety of dimensions were created in different shapes in the reference wall model to examine the impacts of openings.

COMPARATIVE STUDY WITH MATHEMATICAL MODELS

Mathematical comparison for unstrengthened shear wall with opening

Saheb and Desayi [12] had studied the effect of one or two openings, positioned either symmetrically or asymmetrically, and combinations of window or door openings. The equation which is given underneath has been proposed to extend the usefulness of their empirical technique to represent the presence of area and position in an opening.

$$\mathrm{N}_{u0} = (\mathrm{k}_1 - \mathrm{k}_2\alpha_x)\mathrm{N}_u \qquad \text{eq. (1)}$$

where N_u is the ultimate load of a panel without openings. The constants k_1 and k_2 were obtained using curve-fitting techniques. Under one way (OW) action this procedure yields k_1 = 1.25 and k_2 = 1.22, while under two way (TW) action k_1 = 1.02 and k_2 = 1.00. The effect of the area and position of the opening in the wall is taken into consideration via a dimensionless parameter, α_x defined as,

$$\alpha_x = \frac{\mathrm{A}_{0x}}{\mathrm{A}_x} + \frac{\mathrm{a}}{\mathrm{L}} \qquad \text{eq. (2)}$$

where A_{0x} and A_x represent the horizontal wall cross-sectional area of the opening (i.e. $A_{0x} = L_0 t$) and of the solid wall (i.e. $A_x = Lt$), respectively (figure 15). The term a' is figured concurring to the following equation

$$\mathrm{a}' = \frac{0.5L^2t - \mathrm{L}_0 t\mathrm{a}_0}{Lt - \mathrm{L}_0 t} \qquad \text{eq. (3)}$$

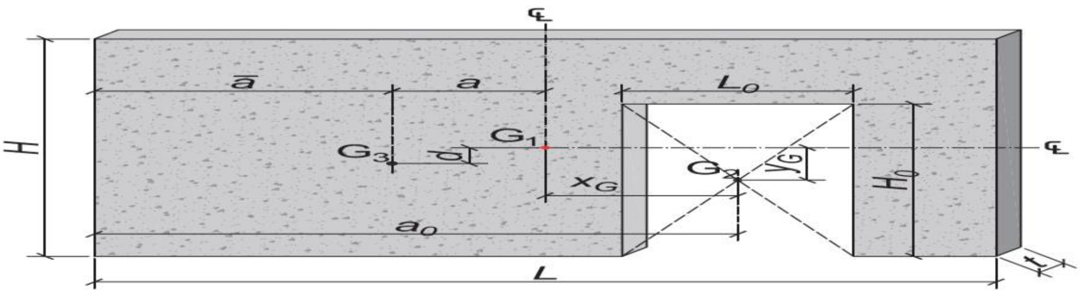


Fig. 15: Geometry of a wall with openings (G_3 = center gravity of wall with opening, G_1 = center of gravity of solid wall, G_2 = center of gravity of opening)

Regarding the unstrengthened specimen OW-NF in the first experimental program, Saheb and Desayi mathematical model [12] demonstrated that the F.E. axial capacity fitted with mathematical axial capacity with acceptable accuracy 8.6%. The accuracy of the unstrengthened specimen WO2a in the second experimental program was 19.37% as illustrated in table 6.

Table 6: Mathematical comparison for unstrengthened shear wall with opening

Serial	Mathematical Comparison, Ultimate load, Pu (kN)		
	F.E.	Math.	Accuracy %
OW-NF	288.8	266	8.6
WO2a	109.82	92	19.37

Mathematical comparison for EB CFRP strengthened shear wall with opening

Mohammed et al. (2013) [4] proposed design equations for R.C. wall with opening under eccentric axial loads & strengthened with externally bonded CFRP sheets. The mathematical design equations are based on experimental tests on eight R.C. walls with two different CFRP layouts including DF (45^o to the corner of the opening) and AF (all around the opening) as presented in figure 16. The tested R.C. walls include various centralized rectangular opening sizes (5, 10, 20 and 30 %).

$N_{AF} = (2.0765-2.1186\ \alpha_x)\ N_{NF}$ eq. (4)

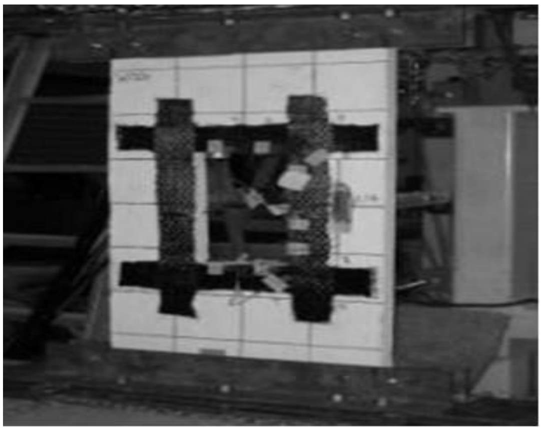
$N_{DF} = (2.4708-2.6099\ \alpha_x)\ N_{NF}$ eq. (5)

Where N_{NF} is the ultimate load of opened R.C. wall without CFRP, N_{AF} and N_{DF} are representing the ultimate load of CFRP strengthened R.C. walls with AF and DF layouts. α_x is defined as illustrated in equation 2 and figure 15.

Regarding the strengthened specimen OW-AF in the first experimental program, Mohammed et al. mathematical model [4] demonstrated that the F.E. axial capacity fitted with mathematical axial capacity with acceptable accuracy 1.43%. The accuracy of the strengthened specimen WO2b in the second experimental program was 21.9% as illustrated in table 7.

Table 7: Mathematical comparison for externally bonded CFRP strengthened shear wall with opening

Serial	Mathematical Comparison, Ultimate load, Pu (kN)		
	F.E.	Math.	Accuracy %
OW-AF	345.9	341.02	1.43
WO2b	166	136.15	21.9



AF Pattern **DF Pattern**

Fig. 16: Externally bonded CFRP Strengthening Pattern

PARAMETRIC STUDY

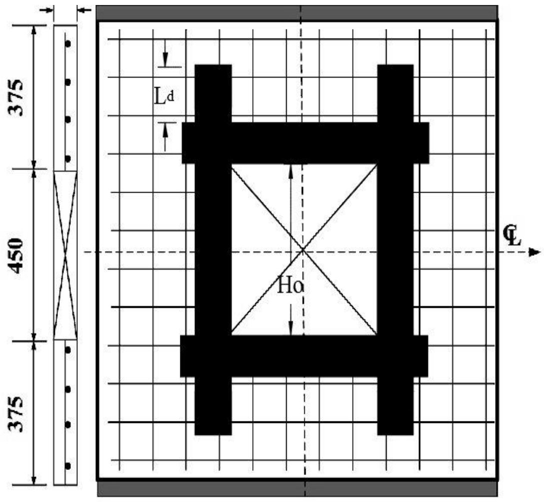
CFRP development lengths

Increasing CFRP vertical development length only

Based on the F.E. study carried out on OW-AF in the first experimental program, but with four different CFRP vertical development lengths (0,100,200 and 300 mm) with L_D/H_0 ratios (0, 0.22, 0.44 and 0.67) which externally bonded alongside the opening vertical edge and horizontal CFRP sheet without any development length as shown in figure 17. The four models have the same opening size and position, it is clearly seen that the CFRP development length has impact on the axial capacity values at failure stage (N_{U0S}). The ultimate axial loads (N_{U0S}) are presented in figure 18 in the vertical axe and the ratios between development lengths to opening length (L_D/H_0) are presented in the horizontal axe. It is clearly seen that the axial load capacity of the shear wall increases to 412 kN until the L_D/H_0 reaches 0.21. The curve also indicates that minor effects on ultimate axial load are yielded for the shear wall with L_D/H_0 more than 0.21 (92 mm). This means that increasing CFRP vertical development length more than the maximum development length, does not result in an increase in resisting tensile stresses so the axial capacity does not increase. That conclusion is compatible with the model of Neubauer, et al. [13]. They conducted a mathematical model which calculate the maximum anchorage length in elements subjected to at flexural cracks. By compensating in Neubauer equation which is represented below, the maximum development length will be 82 mm which is almost the same length conducted by the F.E. graph.

$$\ell_{b,max} = c_2 \sqrt{\frac{E_f t_f}{\sqrt{f_{ck} f_{ctm}}}} \quad [\text{mm}]$$

C_2: 1.44
F_{ck}: the concrete compressive strength
F_{ctm}: mean value of the concrete tensile strength
E_f: CFRP modulus of elasticity
t_f: CFRP sheet thickness



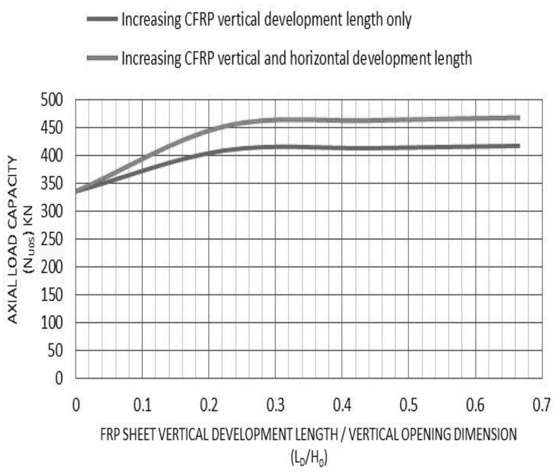


Fig. 17: CFRP development length configuration **Fig. 18: L_D/H_0 versus axial capacity**

Increasing CFRP vertical and horizontal development length

Based on the F.E. study carried out on OW-AF in the first experimental program, but with four different CFRP development lengths in the both directions (0,100,200 and 300 mm) with L_D/H_0 ratios (0, 0.22, 0.44 and 0.67), which externally bonded alongside the opening vertical and

horizontal edge as shown in figure 19 with the same opening size and position. The ultimate axial loads (N_{UOS}) are presented in figure 18 in the vertical axe and the ratios between development lengths to opening length (L_D/H_0) are presented in the horizontal axe. It is clearly seen that the graph follows the same manner of the graph related to increasing CFRP vertical development length but with higher values as the axial capacity reaches to 460 kN at L_D/H_0 0.21 then the graph levelled while increasing L_D/H_0 more than 0.21 and minor effects are yielded on ultimate axial load for the shear wall.
In conclusion, increasing the horizontal CFRP development length has constant effect on the axial capacity as shown in figure 18. In addition, increasing CFRP vertical development length more than the maximum development length that presented in the previous equation, does not result in an increase in resisting tensile stresses so the axial capacity does not increase. The increased vertical CFRP development length will not be neither sufficient nor economic

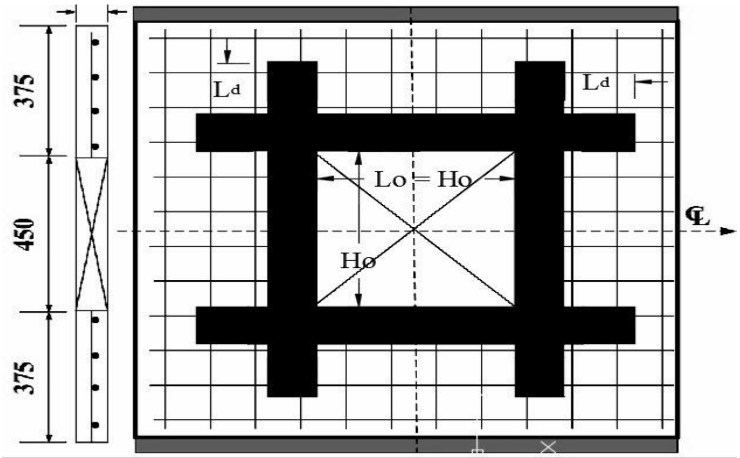
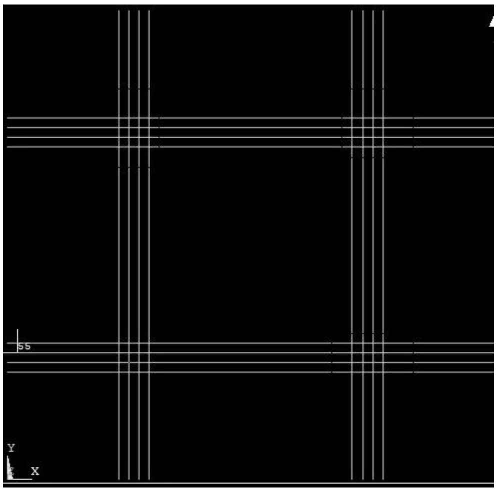


Fig. 19: CFRP development length configuration

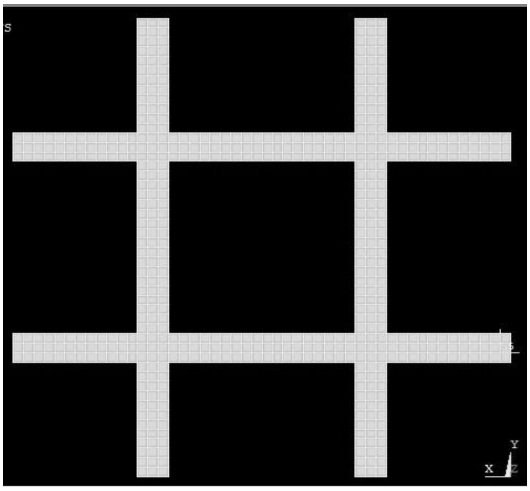
Effect of strengthening scheme

Strengthening process is highly affecting the axial capacity of R.C wall with openings using any strengthening scheme. It is divided into two main types: pre-strengthening method before concrete casting & post-strengthening method which occurs after casting the concrete and making an opening in R.C. wall for structural or architectural purposes. Table 8 briefly summarizes results for various schemes, which have the same: strengthening material cross section area (400 mm^2), inertia (4x106 mm^4), material properties, scheme arrangement configuration as presented in figure 20, opening position (middle), opening area (0.18 m^2), opening shape (square) but have different strengthening schemes (NSM with CFRP bars, NSM with steel bars, externally bonded CFRP sheets and embedding extra internal steel bars around openings). For Pre-strengthening method, the extra internal steel bars are added around openings before casting the concrete. For Post-strengthening methods, near surface mounted process involves cutting a series of shallow grooves in the concrete surface in the required direction. (The depth of the groove must obviously be less than the cover so that the existing reinforcement is not damaged.) The grooves are partially filled with epoxy mortar into which pultruded carbon fiber composite rods or steel rods are pressed. The remainder of the groove is then filled with epoxy mortar and the surface levelled. External bonding of CFRPs involves applying the composite material to the external face of a structure with a layer of epoxy after preparing, cleaning and roughening the surface of the structure.
The load values indicate that ultimate axial load is affected by the strengthening scheme. According to the table, the highest load capacity is recorded to the pre-strengthening method which is embedding extra internal steel bars around openings (504 kN). NSM CFRP bars recorded (500 kN) then the capacity went down to 480 kN as we changed the strengthening

scheme from NSM CFRP bars to NSM steel bars. The lowest axial capacity was recorded at externally bonded CFRP sheets as the load capacity decreased to 460 kN.
The main conclusion is although the R.C. wall mode of failure in all the cases is crushing of concrete, embedding extra internal steel bars around openings before casting the concrete is the optimum scheme because of full confinement around bars. Also, using embedded steel bars around openings before casting is considered as better way in strengthening than using CFRP. Using CFRP requires specialized labor, perfect surface preparation and adequate concrete cover. CFRP also is more susceptible to fire and mechanical degradation. These points make the embedded extra steel bars supersede CFRP techniques. On the other hand, NSM technique has the most considerable effect as a post-strengthening scheme than the EB scheme. The NSM efficiency was not only in terms of the shear wall axial load carrying capacity, but also in terms of tensile stress concentration values around the opening corners at R.C. wall failure. NSM CFRP technique has several advantages, in comparison with the EB method, such as reducing the risk of de-bonding, and a better protection from the external sources of damage. The NSM CFRP rods have more bonding area compared with the EBR CFRP strips which leads to higher bond strength. These findings are compatible with previous experimental papers on R.C. elements conducted by Ahmed M. Khalifa [14], Zsombor K. Szabó, et al. [15] and Azadeh Parvin, et al. [16].



NSM & embedding extra internal steel bars



Externally bonded CFRP sheets

Fig. 20: Strengthening scheme arrangement configuration

Table 8: Strengthening Scheme F.E. results

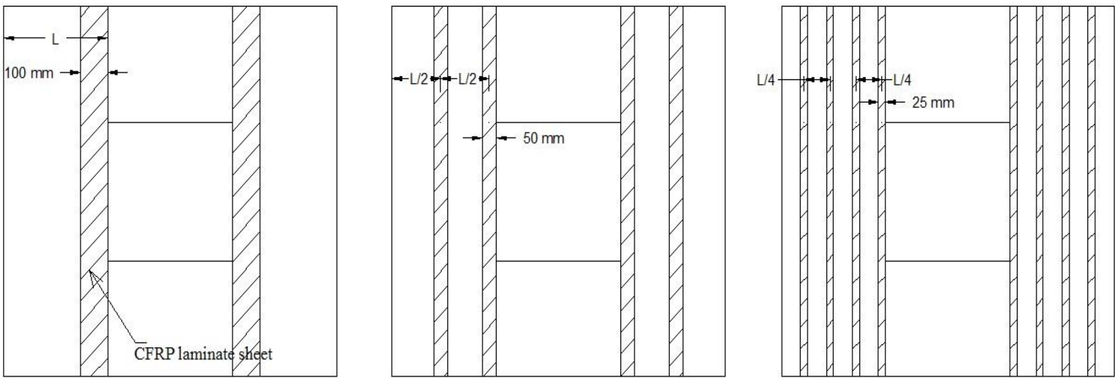
Strengthening Type	Strengthening Scheme	Ultimate load, Pu (kN)	Tensile stress concentration values around the opening corners (N/mm^2)
Pre-strengthening	Embedding extra internal steel bars around openings	504	1.75
Post-strengthening	NSM CFRP	500	1.9
Post-strengthening	NSM Steel	480	2.75
Post-strengthening	Externally bonded CFRP sheets	460	4.1
	OW-NF (Unstrengthened Shear wall with opening)	266	5.0

Effect of Re-arrangement of CFRP sheet

F.E. study is carried out on two different experimental specimens (OW-AF & OW-PF). The vertical CFRP sheet, which is paralleling to the axial load direction will be rearranged and divided to 2 and 4 equal strips and spaced equally in the wall as illustrated in figure 21. Table 9 briefly summarizes results of dividing CFRP sheet to smaller strips. It can be seen that for OW-AF with one complete strip, the experimental ultimate load is 335.7 kN. Dividing the CFRP sheet from 1 sheet (100 mm width) to two spaced equal strips (50 mm width / each) decreased the ultimate axial load to 324.00 kN. In addition, dividing the CFRP sheet from 1 sheet to four spaced equal strips (25 mm width / each) decreased the ultimate axial load to 318.00 kN. The least value of stress concentration is recorded at One CFRP strip .OW-PF follows the same manner of OW-AF.
This leads us to believe that the best way to increase the axial load capacity is installing one wide CFRP sheet to resist the stress concentration at the corners and not re-arranging it to equal strips in the wall.

Table 9: Dividing CFRP sheet results

Specimen	One Strip		Divided to Two strips		Divided to Four strips	
	Cap. kN	Stress concentration N/mm^2	Cap. kN	Stress concentration N/mm^2	Cap. kN	Stress concentration N/mm^2
OW-AF	335.70	13	324.00	17	318.00	20.75
OW-PF	359.85	10	333.12	14	273.95	16.9



One Sheet (100 mm) Two strips (50 mm) Four strips (25 mm)

Fig. 21: Divided CFRP sheet configuration (OW-PF)

CONCLUSIONS

Based on the study presented herein, the following conclusions have been drawn:

- Increasing the CFRP development length has considerable effect on the axial capacity until limit. After exceeding that limit, the increased CFRP development length will not be neither sufficient nor economic. Therefore, it is recommended to increase CFRP development length parallel to the load direction only according Neubauer model.
- NSM technique & embedding extra internal steel bars around openings before casting the concrete have the most considerable effect on increasing the shear wall axial

capacity and decreasing tensile stress concentration values around the opening corners.

- The increment in the CFRP sheet width leads to stress concentrations value decrease at the opening corners. Therefore, the best way to increase the axial load capacity is installing one wide CFRP sheet and not re-arranging it to equal strips in the wall.

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