RETROFITTING OF PUNCHING SHEAR DAMAGED FLAT SLABS USING CARBON FIBER REINFORCED POLYMER (CFRP) SHEETS

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Master of Science Degree in Structural Engineering Design

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University of Moratuwa
Sri Lanka

January 2021

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Dissertation Submitted in partial fulfillment of the requirements for the Master **of** Science Degree in Structural Engineering Design

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DECLARATION

I declare that this is my own work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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Prof. Mrs. J.C.P.H. Gamage Date

Dissertation Supervisor

ABSTRACT

The punching shear failure of flat slabs is common in reinforced concrete buildings, and once the failure occurs, rectification is difficult and time consuming, which also affects building functionality. Hence, it is convenient to retrofit the flat slab structure to enhance the punching shear capacity while minimizing the disturbance to the structure. Therefore, a method was developed to retrofit punching shear damaged flat slabs using Carbon Fiber Reinforced Polymer (CFRP) sheets, considering the advantageous properties of the material, like the flexibility of usage, less weight, less thickness of material etc.

Thus, experimental and theoretical studies were carried out using 12 numbers punching shear failed flat slab panels with various repair materials and various CFRP sheet arrangements. The BS EN 1992-1-1: 2004 standard was used for theoretical analysis. A comparison of theoretical and experimental punching shear capacities concluded that the theoretical capacity is much higher than the experimental capacity, due to the difference of failure modes of the specimens.

ACKNOWLEDGEMENT

I would like to acknowledge the Department of Civil Engineering of University of Moratuwa, to provide me the opportunity for studying the Master of Science Degree Program in Structural Engineering Design. I would like to express my sincere appreciation to all the staff members of this M.Sc program for their encouragement and excellent guidance to complete my M.Sc degree in successful manner.

I am very grateful to my supervisors Prof. Mrs. J.C.P.H.Gamage of University of Moratuwa, who offered me the valuable opportunity to carry out and complete this research work and all the staff members of Department of Civil Engineering – University of Moratuwa, who gave me the fullest support to compete the work. Also, I would like to be thankful to Mrs. M.A.L. Silva for providing the cracked samples required for the experimental analysis and giving the fullest support.

Furthermore I like to express my sincere thanks and appreciation to Eng U.S. Karunarathne, the chairman of the Central Engineering Consultancy Bureau (CECB) for granting permission to carry out my Master of Engineering Degree Program and also the senior engineers of CECB who helped by sharing their knowledge and experiences.

Also, I offer my gratitude to my family members for the dedication of them for me and encouraging me to success my M.Sc degree.

Finally, I would like to extend my grateful thanks and appreciation to all who supported time to time, to achieve my goals.

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CHAPTER 1 - INTRODUCTION

1.1. Background

Retrofitting and strengthening of existing buildings has become common, as it is more cost effective than re – building the structures due to high cost of materials and workmanship in the current context. It is alsopossibleto use the structure immediately after retrofitting. Retrofitting can be used to strengthen the structure against the local failure of elements in usage changes, corrosion, deterioration or damages due to aging of structure, design and construction errors, seismic design etc.

Several methods of strengthening concrete structures are commonly used such as;

- Increasing of concrete sections by using jacketing methods
- Addition of members or bracing
- Addition of support points
- Addition of retrofitting materials
- Introduction of prestressing

Among these methods, less troublesome and less time consuming method is; addition of retrofitting materials to the structural members. The structure can be functioned while retrofitting is in progress.

One of the most vulnerable structural elements for failure is the flat slabs, which is mostly used in current structures to achieve architectural and service feasibility.As the beams are not required when flat slabs are used, the building height is reduced and more space can be provided for service linesif required. The most critical failure mode of flat slab elements is punching shear failure, due to absence of beams and small perimeter around the column.

1.2. Significance of Research

Researches and studies have been accomplished to investigate amethods to retrofit already constructed flat slab panels to enhance the punching shear capacity by ways like; jacketing of column to increase punching shear area, introducing drop panels, placing new concrete layer with reinforcement steel, strengthening with steel plates or fiber reinforced polymer, introducing shear reinforcement with steel or fiber reinforced polymer etc. (Osama et al., 2017, Carlos et al., 2015, Helder et al., 2015)

The studies are limited on strengthening and re – using the flat slab panels once it is damaged and cracked by punching shear. This is an area to be studied because the flat slabs are vulnerable to punching shear and it is very difficult, and uneconomical to demolish and re – construct the slab once it is cracked in functional level of the structure. The local failure of flat slab can lead to overall structural failure.

Therefore, this study fulfils the current research gap, and find out a method to retrofit punching shear damaged flat slab panels, using Carbon Fiber Reinforced Polymer (CFRP) laminates, with advantageous properties as high tensile strength, less weight etc. CFRP sheets can be used for strengthening at the operational stage of the structure, without disturbing other structural elements.

1.3. Aim of Research

The aim of this research is to find out a suitable method and arrangement to retrofit punching shear failed flat slab panels by using CFRP strips. The efficiency of using CFRP strengthening technology will be analyzed.

1.4. Objectives of Research

- To identify the conditions, where flat slab strengthening is required and suitable retrofitting methods.
- To study failure modes and crack patterns of retrofitted flat slabs using previous research results.
- To study about various fiber reinforced polymer types, and other required materials and select suitable materials to use in this research.

- To identify methods of retrofitting and strengthening flat slabs using fiber reinforced polymer.
- To find out methods to strengthen flat slabs failed and cracked by punching shear using the selected materials.
- To obtain punching shear capacity of strengthened flat slabs theoretically and experimentally.
- To provide recommendations to retrofit failed flat slab panels using FRP material.

1.5. Methodology

In order to achieve the research objectives, the approach written below was followed;

- Literature review on requirement of retrofitting and strengthening of flat slabs, retrofitting methods, failure modes and patterns of strengthened flat slabs, structural strengthening materials, usage of fiber reinforced polymer for strengthening the structural elements etc.
- Studying about the types and properties of fiber reinforced polymer and other required material for retrofitting the failed flat slabs.
- Selecting suitable repair material for failed flat slab panels experimentally.
- Retrofitting failed flat slab panels, using the selected materials with alternative bonding arrangements.
- Analyzing theoretical load capacity of control and strengthened specimens.
- Comparison of results.
- Providing recommendations on the feasibility of strengthening punching shear damaged flat slabs using carbon fiber reinforced polymer sheets.

CHAPTER 2 - LITERATURE REVIEW

2.1. Introduction

The literature findings on followings are described in this section.

1. Punching Shear of Flat Slabs
2. Enhancing the Punching Shear Capacity of Flat Slabs
3. Usage of Fiber Reinforced Polymer
4. Usage of Carbon Fiber Reinforced Polymer

2.1.1. Punching Shear of Flat Slabs

Flat slabs are plate structures that usually do not have beams to provide more vertical clear space. Loads are transferred directly to supporting columns through the slab (Khalend et al., 2012). Lakruwan et al. (2018),Helder et al. (2015)and Carlos et al. (2015)discussed the advantages of flat slabs as follows;

- Architectural flexibility
- Reduce building height and hence reduce the material cost
- Less time of construction
- More vertical space is available
- Simplicity of formwork and rebar arrangement
- Reduce the formwork cost

The punching shear capacity of flat slabs at the slab column connection is a critical concern in design of flat slabs.Punching shear failure can be represented by the formation of a cone shaped failure surface at the flat slabtocolumn connection as indicated in Figure 2.1. The punching shear failure is mostly brittle and catastrophic,as it may cause increment of load transfer to neighbouring columns and hence to the below slabs which may generate a global collapse (Carlos et al. - 2015). Also punching shear failure of flat slabs is a very serious problem for Civil Engineers, because it can lead to unexpected progressive collapse of the entire structure (Erdogan et al., 2007).

The punching shear phenomenon was described by Osama et al. (2017)as that, when the applied load generates negative moment in flat slab systems, which decreases rapidly away from column centerline, formation of cracks in an approximately circular perimeter around the column.Radial crack,starting from the column also can be expected at the ultimate punching shear load(Figure 2.1).

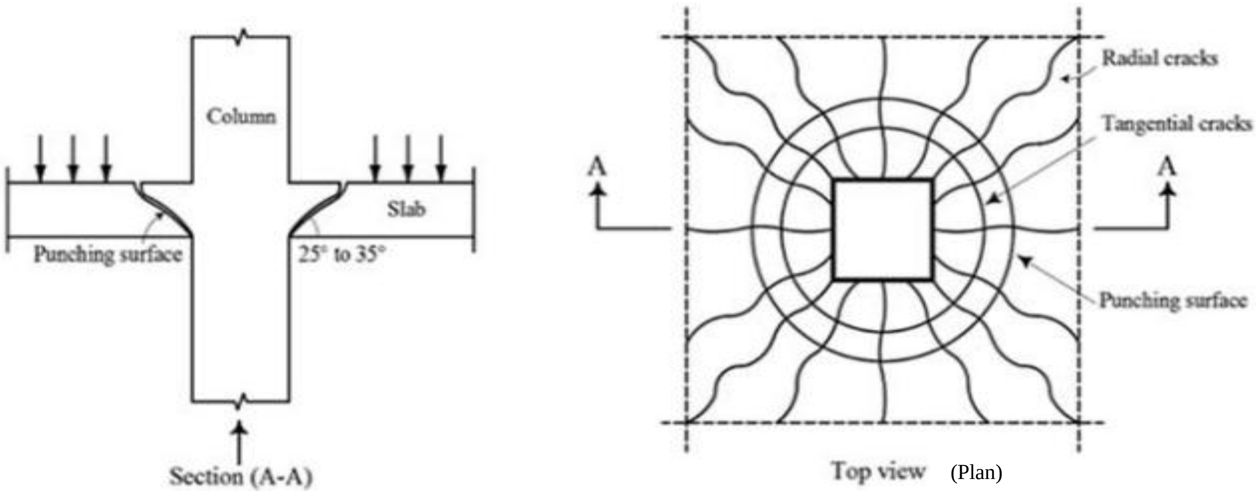


Figure 2.1. Crack Pattern in Punching Shear Failure(Osama et al.,2017)

The diagonal tension cracks which tend tooriginate at the mid depth develop in the slab area near the column areequivalent to webshear cracks, not like flexural shear cracks.

The punching capacity depends on, (Osama et al.,2017)

- Compressive strengthof concrete
- Thickness of the slab
- Properties of reinforcing material
- Shape and size of the column
- Pattern of reinforcements
- Flexural reinforcement ratio
- Amount and pattern of shear reinforcement

2.1.2. Enhancing the Punching Shear Capacity of Flat Slabs

The punching shear strength of flat slabs need to be increased in some situations, due to; (Helder et al., 2015, Khalend et al., 2012, Carlos et al., 2015, Tandeand at el., 2014, and Nagi et al., 2017)

- Increment of applied imposed load due to changes in the structural usage
- Corrosion of reinforcement and deterioration of other materials.
- Construction or design errors
- Inconsistency with the current standards
- Accidental structural damages
- Lack of maintenance
- Need for installing new services which require openings of slab

According to Osama et al. (2017), Carlos et al. (2015), Helderet al.(2015) and Khaleel et al. (2013),the punching shear capacity of flat slabs can be enhanced by,

- Increasing thickness of the slab
- Increasing dimensions of the column
- Introducingcolumn heads or drop panels
- Using high compressive strength concrete
- Strengthening punching shear zone by introducing shear reinforcement
- Increasing amount of flexural reinforcement

The conventional strengthening methods, thosecan be used for existing flat slabs according to Lakruwan et al. (2018), Carlos et al. (2015), Khalend et al. (2012) and Khaleel et al. (2013) are;

- Column jacketing with reinforced concrete to increase the punching shear perimeter
- Increasing amount of reinforcement by casting a new reinforced concrete layer or externally applyingFRP reinforcement
- Using shear reinforcement (steel or FRP)
- Using steel plates at the slab column connection
- Use of transverse prestressed reinforcement

2.1.3. Usage of Fiber Reinforced Polymer

Tomislav et al. (2009), Tandeand et al. (2014), Nagi et al. (2017), Carlos et al. (2015), Mohammad et al. (2016) and Lakruwan et al. (2018) indicated the advantages and disadvantages of usage of fiber reinforced polymer against steel reinforcement as follows;

Advantages;

- No corrosion
- High tensile strength
- Low weight and hence high strength – weight ratio
- High stiffness – weight ratio
- Satisfactory behavior under dynamic loading
- Less relaxation
- No effect from moisture
- Flexibility in design
- Magnetic resistance
- Electrical isolation
- Low thermal expansion
- Large fatigue resistance capacity
- Less affected by corrosive environmental conditions
- Provide longer life
- Requires less maintenance
- Ease of handling and application
- Laps and joints are not required
- Can be used for irregular shapes
- Can follow a curved profile
- Can be readily installed behind existing service
- Unlimited availability with respect to size, geometry and dimensions
- Overlapping in two directional strengthening is not a problem because of thin material

Disadvantages;

- Elastic behavior until failure
- Highly dependent mechanical characteristics on direction (longitudinal or transverse)
- Significant differences in tensile and compressive strength
- Non ductile behavior
- Creep and rupture
- Lower modulus of elasticity
- Sensitive to ultraviolet exposure
- Strengthening needed to be protected to prevent Risk of fire and accidental damage
- GFRP sensitive to alkaline environment
- Debonding of FRP sheets

2.1.4. Usage of Carbon Fiber Reinforced Polymer

Advantages of CFRPover other FRP materials are listed by Lakruwan et al. (2018);

- Glass Fiber Reinforced Polymer (GFRP) and Aramid Fiber Reinforced Polymer (AFRP) hydrolyze in the presence of alkalinity
- CFRP is higher in tensile strength compared to other FRPs

The properties of FRP according to ACI 440.2R-08 are indicated in Table 2.1 to Table 2.5.

Table 2.1. Constituent Material Properties of Carbon Fiber Material (ACI 440.2R-08)

<table>
<tr><th colspan="2">Property (Unit)</th><th>Value</th></tr>
<tr><td colspan="2">Typical density lb/ft^3 (g/cm^3)</td><td>90 to 100 (1.5 to 1.6)</td></tr>
<tr><td rowspan="2">Coefficient of thermal expansion x10^{-6}/F^0 (x10^{-6}/C^0)</td><td>Longitudinal, α_L</td><td>–0.6 to 0 (–1 to 0)</td></tr>
<tr><td>Longitudinal, α_T</td><td>–0.6 to 0 (–1 to 0)</td></tr>
</table>

Table 2.2. Typical Properties of Fibers (Used in FRP Systems) (ACI 440.2R-08)

Fiber Type	Elastic modulus (GPa)	Ultimate Strength (MPa)	Rupture Strain Min %
General Purpose	220 - 240	2050 - 3790	1.2
High – Strength	220 - 240	3790 - 4820	1.4
Ultra – High – Strength	220 - 240	4820 - 6200	1.5
High – Modulus	340 - 520	1720 - 3100	0.5
Ultra – High – Modulus	520 - 690	1380 - 2400	0.2

Table 2.3. Properties of FRP Bars (Fiber Volumes of 50 to 70%)(ACI 440.2R-08)

FRP System Description	Young's Modulus (GPa)	Ultimate Tensile Strength (MPa)	Rupture Strain %
High-strength carbon/epoxy	115 - 165	115 - 165	115 - 165

Table 2.4. Properties of FRP Laminates (Fiber Volumes of 40 to 60%)(ACI 440.2R-08)

FRP System Description (Fiber Orientation)	Young's Modulus		Ultimate Tensile Strength		Rupture Strain at 0^0 (%)
	Property at 0^0	Property at 90^0	Property at 0^0	Property at 90^0	
	GPa	GPa	MPa	MPa	
High-strength carbon/epoxy, degrees					
0	100 - 140	2 - 7	1020 - 2080	35 - 70	1.0 - 1.5
0/90	55 - 76	55 - 75	700 - 1020	700 - 1020	1.0 - 1.5
+45/-45	14 - 28	14 - 28	180 - 280	180 - 280	1.5 - 2.5

Table 2.5. Ultimate Tensile Strength Properties of Commercially Available FRP Systems (ACI 440.2R-08)

FRP system description (fiber type/saturating resin/fabric type)	Fabric weight	Ultimate tensile strength per unit width of sheet or fabric.
	g/m^3	kN/mm
General purpose carbon/resin unidirectional sheet	200	500
	400	620
High-strength carbon/resin unidirectional sheet	230	320
	300	700
	620	960
High-modulus carbon/resin unidirectional sheet	300	600
General-purpose carbon/resin balanced sheet	300	180

2.2. Punching shear behaviour of flat slabs reinforced with FRP bars

Osama et al. (2017) and Jacobson et al. (2005) have studied the behaviour of flat slabs which are reinforced with FRP bars in punching shear and compared experimental results with several theoretical models.

Osama et al. (2017) have reviewed the behaviour of flat slabs which are reinforced with FRP bars under punching shear. A comparison of experimental results obtained in literature for punching shear capacity (V_{test}) and punching shear capacity (V_{pred}) predicted from ACI 440.1R-15 and CAN/CSA S806-12 design codes is carried out. The equations used for calculation are indicated below;

1) ACI 440.1R-15

$$Vc = \frac{4}{5}\sqrt{f_c^{'}}\,b_o kd$$

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f$$

Where $n_f = {}^{E_f}\!/\!{}_{E_c}$; $E_c = 4750\sqrt{f_c^{'}}$

2) Canadian Building Code CAN/CSA S806-12

The lowest value for punching shear was obtained from following equations:

$$V_c = 0.028\lambda\phi_c\left(1 + \frac{2}{\beta_c}\right)\left(E_f\rho_f f_c^{'}\right)^{1/3}\ b_o d$$

$$V_c = 0.147\lambda\phi_c\left(\frac{\alpha_s d}{b_o} + 0.19\right)\left(E_f\rho_f f_c^{'}\right)^{1/3}\ b_o d$$

$$V_c = 0.056\lambda\phi_c\left(E_f\rho_f f_c^{'}\right)^{1/3}\ b_o d$$

Where,

f_c' – The specified concrete compressive strength.

b_o – The perimeter of the critical section at a distance of d/2 from the concentrated load.

d – The distance from extreme compression fiber to the centroid of the tension reinforcement.

λ – Factor accounting for concrete unit weight.

α_s – A factor that adjusts V_c for support location.

β_c – The ratio of the long side to short side of the concentrated load or reaction area.

E_f – Modulus of Elasticity of the FRP material.

E_c – Modulus of Elasticity of Concrete.

v_c – Design concrete shear stress.

ρ_f – Reinforcement ratio of FRP reinforcement.

ϕc – Resistance factor for concrete

The validity of results has been checked and safety margin of the predictions have been calculated of above two codes of practices as indicated in the Table 2.6.

Table 2.6. Extract of Comparison of Results (Osama et al., 2017)

Reference	No of Specimens	Type of FRP	V_{test}/V_{pred}	
			ACI 440.1R-15	CAN/CSA S806-12
Hassan et al.	19	Glass Fiber Reinforced Polymer	1.67 to 2.59	0.92 to 1.29
Mean value Standard deviation Coefficient of variation (%)			2.16 0.27 12.5	1.17 0.10 8.8

Elgabbas et al.	6	Brass Fiber Reinforced Polymer	1.53 to 2.23	0.93 to 1.22
Mean value Standard deviation Coefficient of variation (%)			1.77 0.25 14.1	1.04 0.12 11.3
El-Ghandour et al. El-Gamal et al. Zaghloul A. Bouguerr et al.	6	Carbon Fiber Reinforced Polymer	1.38 to 2.23	0.91 to 1.12
Mean value Standard deviation Coefficient of variation (%)			1.95 0.31 15.7	1.02 0.08 8.2

Experimental data have been evaluated in this paper, and itdisclose that the equation proposed by CAN/CSA S806-12 standards gives a reasonable prediction of the punching shearstrength of flat slabs with GFRP, BFRP and CFRP reinforcing bars,with average value of V_{test}/ V_{pred}closer to 1.0. ACI 440.1R-15 equationsprovided very conservative predicted capacities with average V_{test}/V_{pred}ranging from 1.77 to 2.16.

Jacobson et al. (2005) have studied the punching shear capacity of one way and two way bridge deck panels strengthened with double layer FRP grid. Experimental and theoretical investigations have been carried out according to various standards and results have been compared.

The experimental investigation was carried out for five numbers of 200mm thick deck panelsand supported as given in Table 2.7.

Table 2.7. Specimen Detail (Jacobson et al., 2005)

Specimen Number	Size	Support Condition
1	2.0mx2.3m	Simply Supported (Span = 2.0m)
2		
3		
7	2.0mx4.3m	Continuous slab with 2m span with 1.2m overhang
8		Supported on precast girder with 1.2 wide top flange

Two layers of Glass/vinylester FRP grid have been used to reinforce the specimen with 38mm deep Glass/vinylester FRP I bars spaced 100mm as longitudinal reinforcement. Concrete cover is 38mm. Load testing has been carried out for the specimens.

The average capacity of simply supported slabs has been obtained as 535kN and for specimen 7 failure load has been reported as 722kN. The specimen 8 has not failed under the loading.

The following punching shear models have been used for theoretical calculations;

1. ACI 318-02
2. ACI 440
3. Eurocode 2 (1992)
4. BS 8110
5. CEB-FIP Model Code 1990 (MC90)
6. Japan' Society of Civil Engineering (JSCE)
7. Mutthys and Taerwe (2000)
8. El-Ghandour et al. (1999)
9. Ospina et al. (2003)

Three reinforcement ratio adjustment methods have been used in this study, as listed below;

1. Using standard reinforcement ratio.

   $\rho_1 = \frac{A_f}{bd}$ Where A_f is considered as FRP reinforcement area
2. Modifying reinforcement ratio by using modular ratio

   $\rho_1 = \frac{A_f}{bd} x \frac{E_f}{E_s}$ Where E_f and E_s are elastic modulus of FRP and Steel respectively.
3. Modifying reinforcement ratio by using modular ration and strain ratio

   $\rho_1 = \frac{A_f}{bd} x \frac{E_f}{E_s} x \frac{\varepsilon_f}{\varepsilon_s}$ Where ε_f and ε_s are strain limits of FRP and Steel.

   The suggested minimum strain limits;

   – 0.0045 (FRP)
   – 0.0025 (yield strain of steel reinforcement)

As per the results, two best fit punching shear models have been selected as the CEB-FIB model code 90 (1990),and the model suggested by Matthys and Taerwe (2000), by using the modified reinforcement ratio with modular ration and strain ratio.

2.3. Retrofitting Flat Slabs for Punching Shear with Externally Bonded FRP Composite

Kai et al. (2013), Khalend et al. (2012) and Faria et al. (2014) have studied the external bonding of FRP laminates, against the punching shear failure of flat slabs. Faria et al. (2014) suggests the potential failure modes of flat slabs externally strengthened with FRP laminates as;

- Rupture of FRP laminates
- Crushing of the concrete
- Shear failure
- Separation of concrete cover
- Debonding of FRP laminates at the end of plate
- Intermediate debonding of FRP due to cracking

Kai et al. (2013) have studied reinforced concrete flat slabs strengthened with FRP against progressive collapse. Three 70mmthick flat slab specimens with 200mm x 200mm stump column were tested. Reinforcement detail used for this research is shown inFigure 2.2.

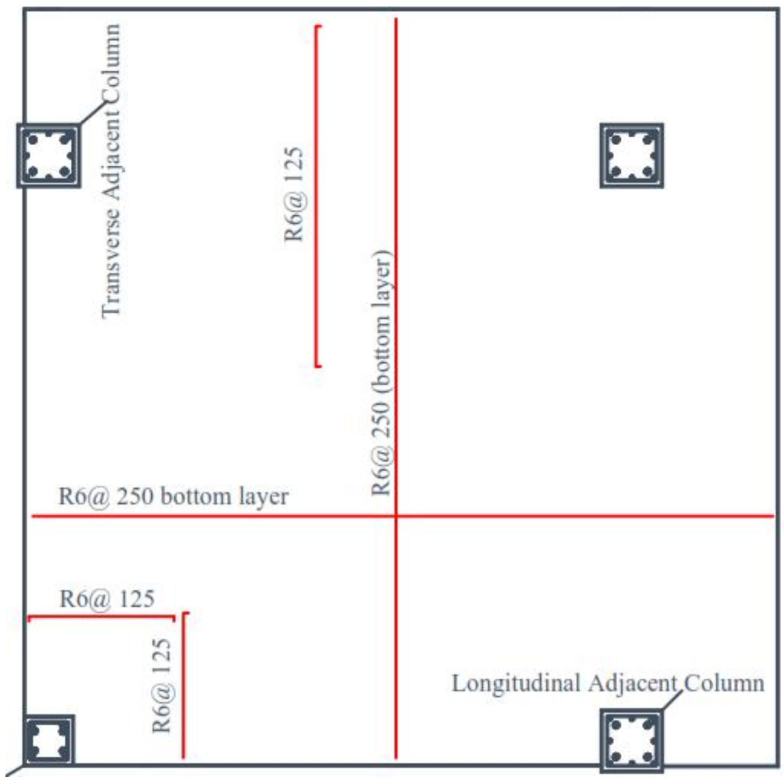


Figure 2.2. Reinforcement Arrangement (Kai et al., 2013)

The top reinforcements were provided to improve the flexural strength with CFRP strips of 150mm width for two specimens. CFRP strips were provided around the column to strengthen the area (See Figure 2.3). The control specimen was tested without strengthening with CFRP strips.

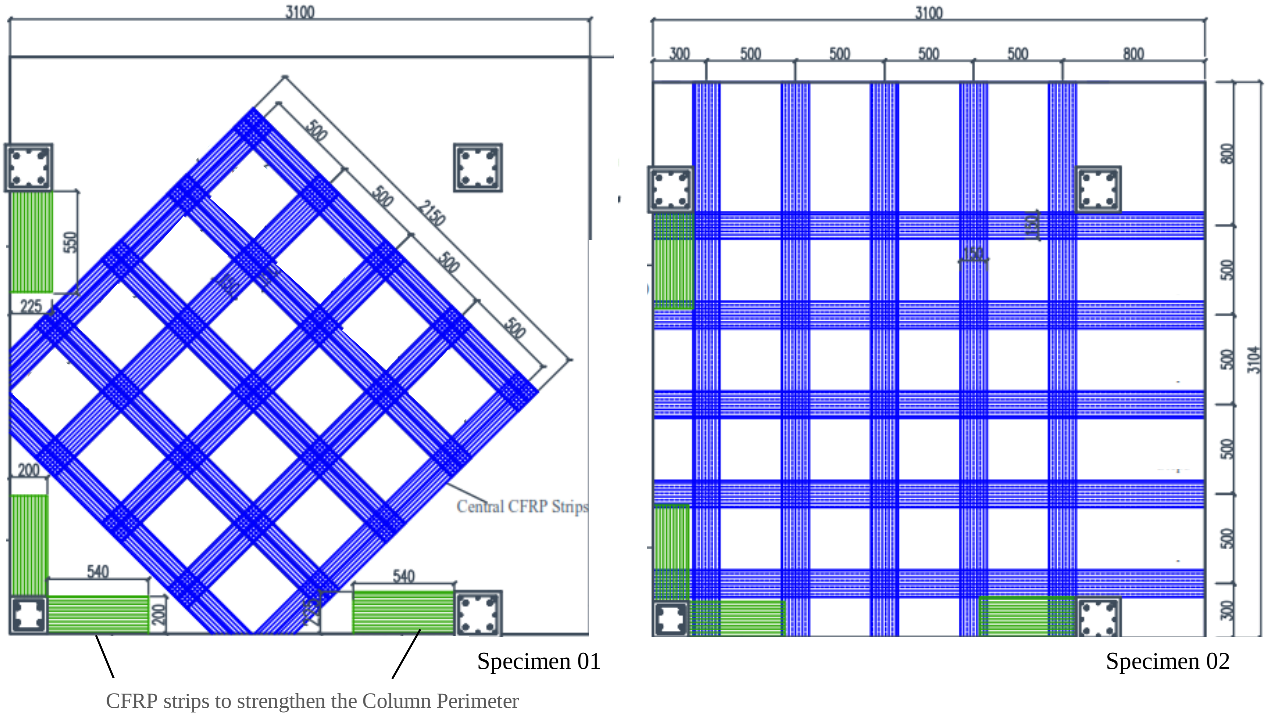


Figure 2.3. Configuration of Strengthening of Specimens (Kai et al., 2013)

It has found that, the control specimen which was not strengthened with FRP sheets has a high likelihood for progressive collapse with the accidental removal of corner column. Strengthened Specimens 01 and 02have been able toimprove the dynamic behavior of Control Specimen by 74.0 % and 82.0 % load increment respectively. The observed failure mode of the strengthened specimens was mainly debonding of the CFRP sheets. Hence it was suggested to apply fiber or other mechanicalanchors to CFRP laminates to overcome the debonding.

Silva et al. (2019) have studied the performance of slab-column connections of flat slabs strengthened with carbon fiber reinforced polymer. Experimental and finite element analysis have been carried out for four arrangements of CFRP strips bonded externally.

Ten flat slab specimens were prepared for testing as indicated in Table 2.8.The reinforcement details are shown in figure 2.4.The CFRP strip arrangements are shown in Figure 2.5.

Table 2.8. External Strengthening Arrangements with CFRP strips (Silva et al., 2019)

Specimen Notation	Description	Number of Experiments
C	Non- strengthened specimens	2
SE	CFRP attached on tension face in skewed direction with end anchorage	2
S	CFRP attached on tension face in skewed direction without end anchorage	2
OE	CFRP attached on tension face in orthogonal direction with end anchorage	2
O	CFRP attached on tension face in orthogonal direction without end anchorage	2

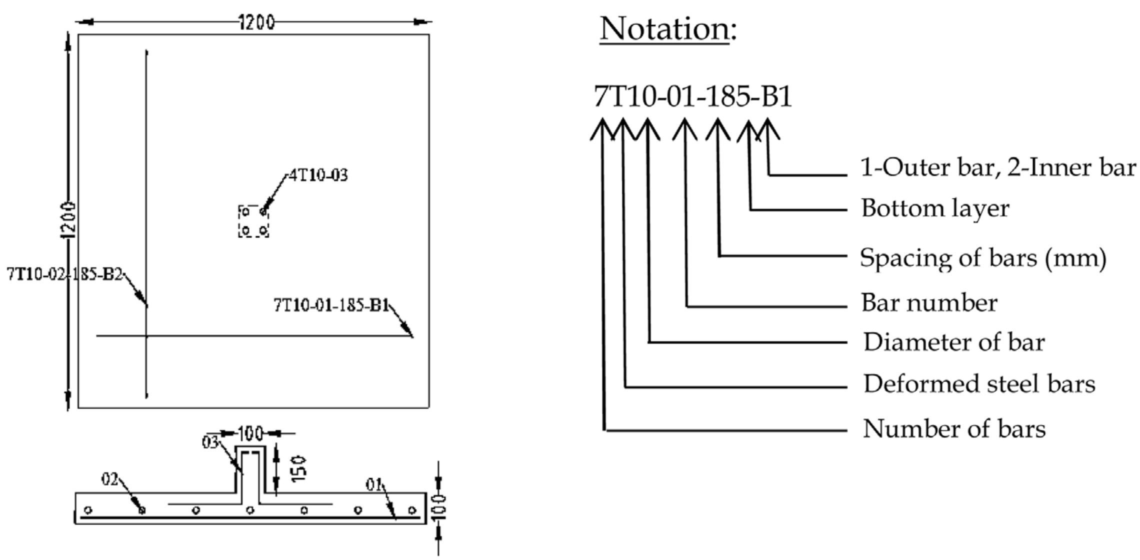


Figure 2.4. Reinforcement Detail (Silva et al., 2019)

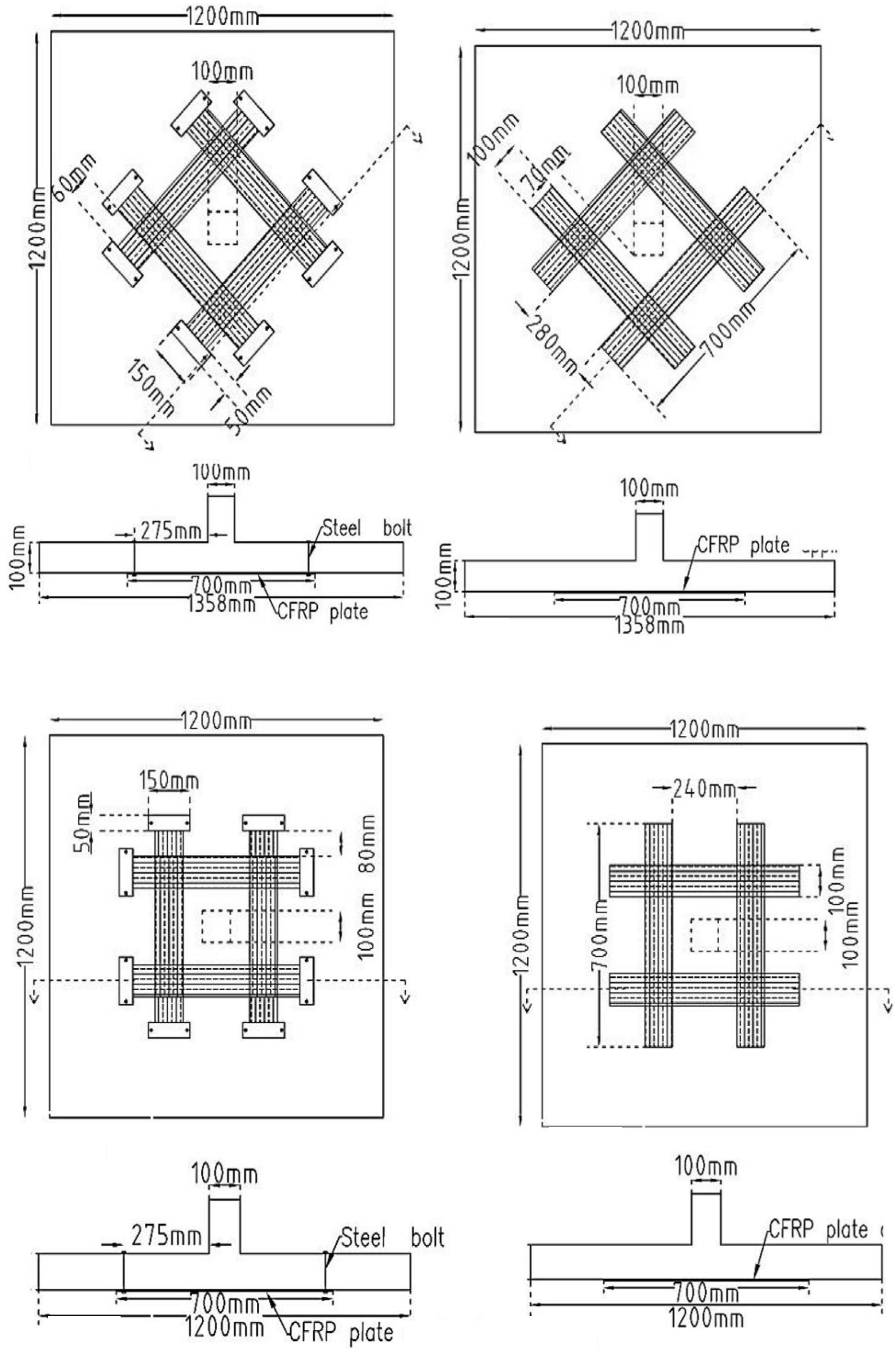


Figure 2.5. Specimen Strengthening Detail (Silva et al., 2019)

A three dimensional finite element model has been developed to predict the behavior of CFRP strengthened flat slab-column connection. The test results are included in Table 2.9 and comparison of the experimental results and finite element modeling results are presented in Table 2.10.

Table 2.9. Test results of Specimens (Silva et al., 2019)

Specimen	Ultimate Load (kN)	Maximum Deflection (mm)	Load Capacity Increment %	Specimen	Ultimate Load (kN)	Maximum Deflection (mm)	Load Capacity Increment %
C-a	103.00	13.80	-	C-b	98.64	9.77	-
SE-a	147.15	10.13	46.0	SE-b	147.15	12.08	46.0
S-a	103.00	8.88	2.2	S-b	122.60	13.00	21.6
OE-a	137.34	14.00	36.2	OE-b	127.53	12.08	26.5
O-a	137.34	11.60	36.2	O-b	122.63	10.69	21.6

Table 2.10. Comparison of Experimental Results and Finite Element Modeling Results (Silva et al., 2019)

Specimen	Experimental Results		Finite element modeling results		U_{FEM} variation w.r.t U_{exp}	δ_{FEM} variation w.r.t δ_{exp}
	Ultimate Load (U_{exp})	Ultimate Deflection (δ_{exp})	Ultimate Load (U_{FEM})	Ultimate Deflection (δ_{FEM})		
O	130 kN	11.2 mm	140 kN	12.9 mm	1.08	1.15
OE	132 kN	13.0 mm	153 kN	17.9 mm	1.16	1.37
S	113 kN	10.9 mm	111.5 kN	11.0 mm	0.99	1.01
SE	147 kN	11.1 mm	166 kN	12.6 mm	1.13	1.14

It has been concluded that, the skewed placement of CFRP at the shear critical area is effective than the orthogonal arrangement in the presence of end anchorage. More than 46% punching shear capacity increment was obtained. Also the provision of end anchorage to CFRP strips increased load carrying capacity.

Khalend et al. (2012) have studied the strengthening of concrete slab-column connections using CFRP strips. Six test specimens have been constructed as shown in the Figure 2.6. Ready mixed concrete was used with average compressive strength of 25.8MPa. The reinforcement arrangement is given in the Figure 2.6.Figure 2.7 shows the details of the different types of specimens used.

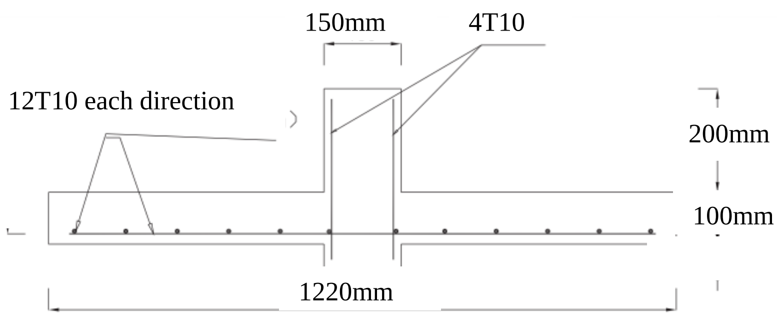
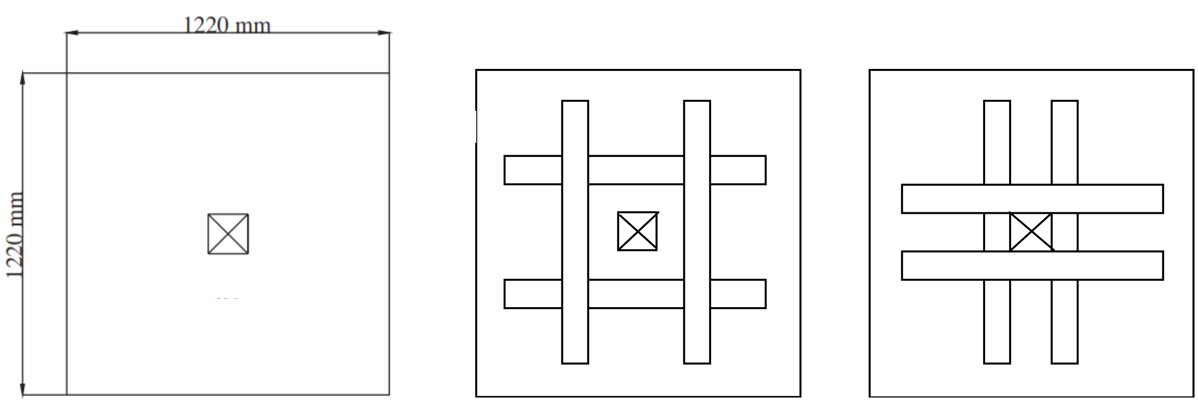


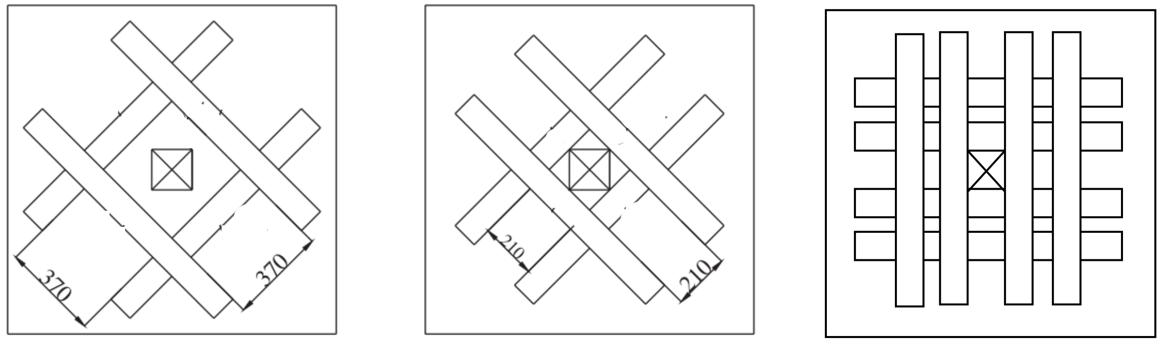
Figure 2.6. Reinforcement Detail (Khalend et al., 2012)



(a) Control Specimen

(b) Orthogonal with offset to column face

(c) Orthogonal and adjacent to column face



(d) Skewed with offset to column face

(e) Skewed and adjacent to column edge

(f) Orthogonal with adjacent to and offset column face

Figure 2.7. Specimen Detail (Khalend et al., 2012)

The punching shear capacities of strengthened specimens have been increased up to 29% compared to control specimen.The strips placed at an offset to the column face recorded higher increment in punching shear capacity. According to the test results, the most beneficialCFRP strip arrangement was the skewed configurationwithoffset to the column face, as shown in Figure 2.5(d).

2.4. Punching Shear Strengthening of Flat Slabs with FRP Dowels

As a solution for debonding occurs in the external strengthening, and to increase shear strength by a higher amount, Carlos et al. (2015), Lakruwan et al. (2018), Erdogan et al. (2007), Aghayari et al. (2015), Helder et al. (2015), Khaleel et al. (2013), Sarah et al. (2008) have used internal application of FRP composite dowels as shear links through drilled holes in the flat slabs with surface anchorage.

Carlos et al. (2015) have studied, way of strengthen a flat slab for punching shear with CFRP and steel shear reinforcement. Both theoretical and experimental approaches have been followed in his study.

The punching shear approaches of CEB-FIP MC 90 (CEB-FIP 1993) and BS EN 1992-1-1 (2004) have been considered. In order to fully compare both design code formulations, a database has been derived compiling test results for slabs subjected to non-eccentric loading which failed in punching comprising both non-reinforced (with 128 experiments) and shear-reinforced slabs (with 38 experiments).

Four reinforced concrete flat slab specimens have been prepared with dimensions of 1100mm x1100mm x100mm, which weredesigned as the bending moment capacity less than punching shear strength for slabs to fail in punching shear. The ordinary reinforcement ratio (ρ_1) was 1.33% for all slabs provided with 8mm diameter bars spaced at 50mm. A 20mm thick concrete cover was provided. 28 day cube compressive strength was 30MPa. The specimen data are summarized in Table 2.11.

Table 2.11. Details of Specimen (Carlos et al., 2015)

Slab	Description
BC01	Non-strengthened reference slab
BCA1	Typical steel bent-down bars as punching shear reinforcement with 8mm diameter 50mm spacing and 1st perimeter ≈ 0.7d from column face
BCN1	CFRP strengthening with 16NSM (Near Surface Mounted FRP) cut-in each direction
BCG1	CFRP strengthening with 6 CFRP laminates bonded in each direction

Flat slab specimen reinforced with steel bent-down bars showed enhanced punching shear strength of approximately 29% when compared with non-strengthened reference slab.The NSM specimen presented an enhanced punching shear capacity strength that can be estimated as 14% and a pure punching shear failure was obtained.On the specimen BCG1, no significant enhancement (4%) could be achieved to the overall shear stress using this strengthening scheme. A premature debonding of CFRP laminates was detected at the punching cone onset that preceded failure.

Lakruwan et al. (2018) have studied the behaviour of reinforced concrete flat slabs strengthened with carbon fiber reinforced polymer sheets. CFRP fabrics have been used to retrofit four concrete flat slab specimens, along with two control specimens, with dimensions of 1200mm x1200mm x100mm, with a central loading column stub with dimension of 100mm x100mm x150mm and flexural steel reinforcement ratio of 0.46% in each direction. Cover to concrete is 20mm. Figure 2.8 shows the reinforcement details of the specimen.

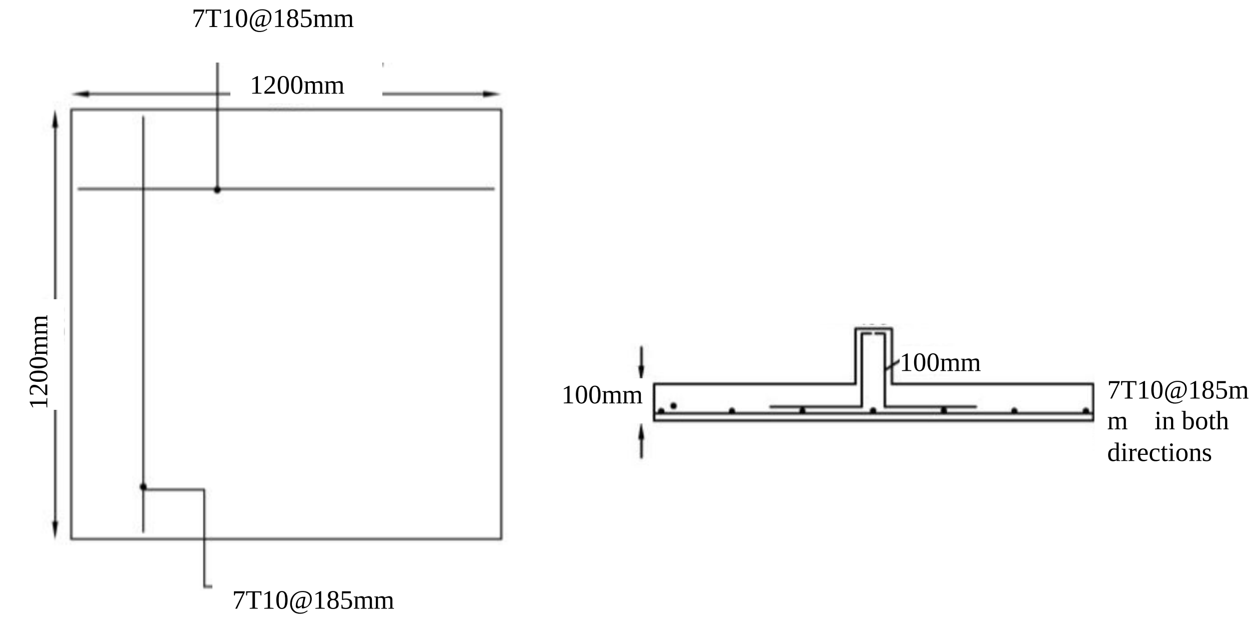


Figure 2.8. Specimen Reinforcement Details

The strengthening with CFRP consisted of both internal and external strengthening techniques. The arrangements shown in Figure 2.9 were used with CFRP inserted in 10mm diameter drilled holes.

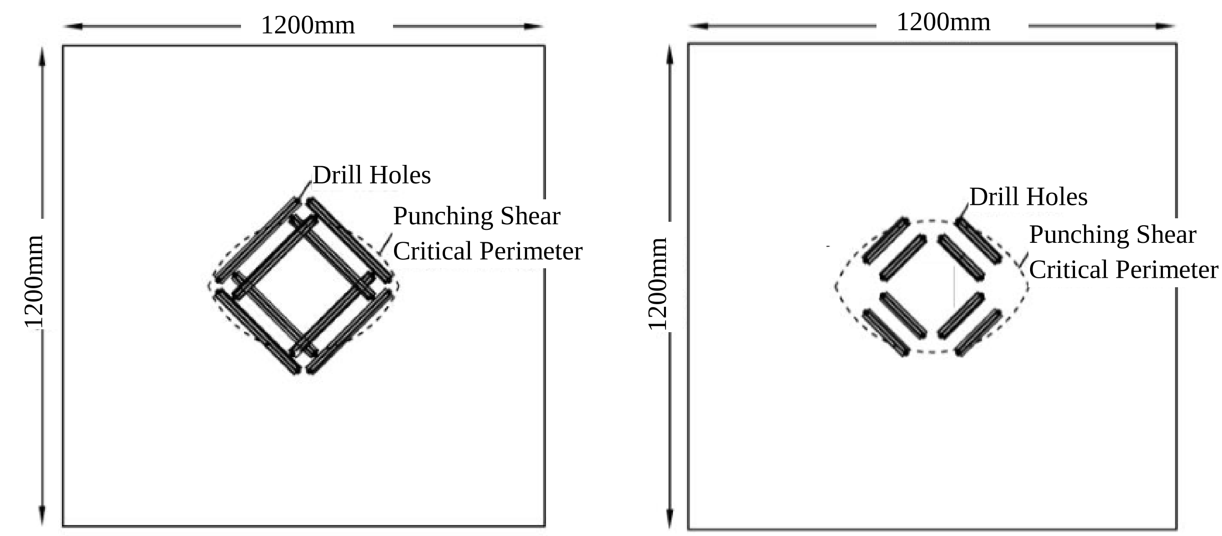


Figure 2.9. (a) CFRP Configuration of Type SA (b) CFRP Configuration of Type SB

The above research has observed that, in the configuration SA and SB, the average punching shear capacity increments were 14.3% and 16.8% with respect to the control specimens. The dowel arrangement near the column corners affect considerably to the punching shear strength near the slab-column area. All the failures of strengthened slabs were occurreddue to punching shear at the outside of the CFRP strengthened zone. No CFRP debonding or concrete cover detachment failures have been observed due to the anchorage provided by the embedded CFRP strips at the end.

Erdogan et al. (2007) have studied a strengthening technique for reinforced concrete slab and column connection with CFRP dowels to enhance punching shear capacity. Four simply supported specimens at edges with plate dimensions of 2300mm x2300mm x150mm and column stub dimensions of 250mm x250mm x300mm extending from both faces were selected. Reinforcement provided on the tension side of the specimens were 16mm bars with 120mm spacing for orthogonal directions and compression reinforcement provided as 12mm bars with spacing of 150mm fororthogonal directions. The column stubs were reinforced with four 16mm bars with 10mm shear reinforcement at 150mm spacing. The effective depth of the slab was 114mm.

The specimens were strengthened with CFRP dowels other than the control specimen (CS). Holes were drilled around the column stub at spacing equal to effective depth of specimen (60mm) in a double line arrangement from the column face. Details of specimens are indicated in Table 2.12.

Table 2.12. Details of Specimen (Erdogan et al., 2007)

Specimen	Number of dowel perimeters	Total number of CFRP dowels
SS3	3	24
SS4	4	32
SS5	5	40

The test results were compared with ACI 318-05 code provisions.

As per the above study results,the punching shear capacity of all strengthened specimens increased significantly and the maximum increment was 33% than control specimen. The failure cone was occurred outside the strengthened region. Also it was indicated that the design provisions of ACI318-05 are safe to predict the punching load capacity and failure modes of the test specimens.

Aghayari et al. (2016) have studied punching shear strengthened flat slab panels with CFRP laminates, FRP fans and steel plates. There, nonlinear finite element analysis by using ABAQUS software and experimental investigation have been carried out simultaneously and results were compared.

The control slab dimension was 1200mm x 1200mm x105mm, and simply supported on the edges and loaded through a 150mm x 150mm x 30 mm steel plate placed at the center of flat slab as shown in Figure 2.10.

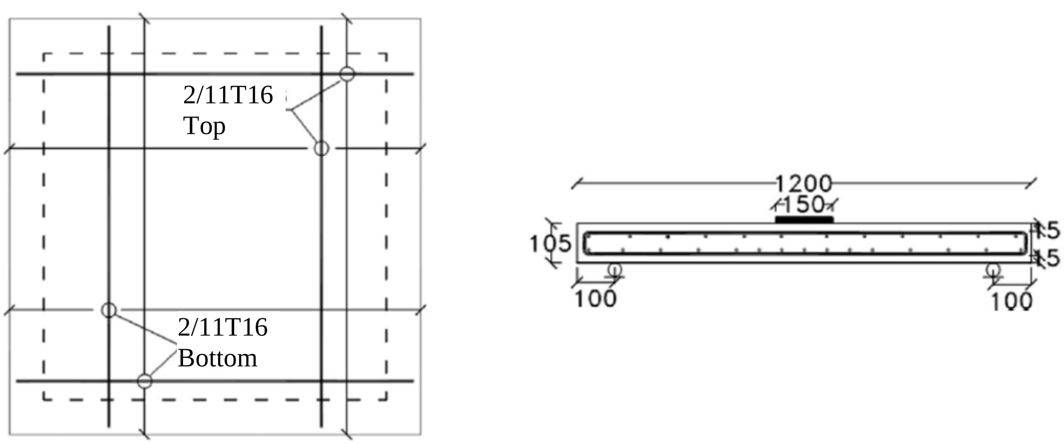


Figure 2.10.Specimen Detail (Aghayariet al., 2015)

Three techniques were considered for strengthening the specimen;

a. 5mm thick steel square plates installed around the column varying the dimensions (15mm-50mm)
b. CFRP laminates bonded around the column or loading region varying the dimensions (15mm-50mm)
c. FRP shear studs (FRP fans). Arrangements of FRP studs are shown in Figure 2.11.

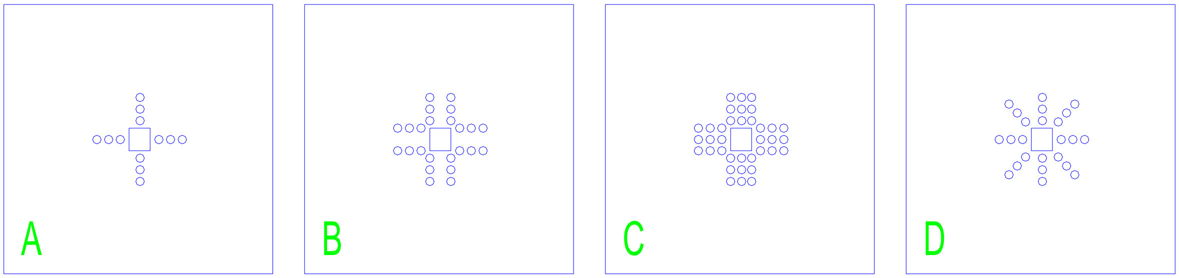


Figure 2.11.Configuration of CFRP Test Specimens (Aghayariet al., 2015)

All above flat slabspecimens were failed by the classical punching mode. The slabs strengthened with CFRP sheets had lower punching strength increment compared with slabs strengthened with steel plates. Slabs strengthened with FRP fans had the highest punching shear capacity increment and stiffness. The arrangement D and C with 24 FRP fans showed the highest increment in punching shear strength.

The nonlinear finite element analysis results showed fair agreement with experimental results and hence can be used to investigate the punching shear strength in flat slabs strengthened by various techniques.

Helder et al. (2015) have investigated flat slabs strengthened for punching shear with CFRPdowels. Calculations have been done according to the Brazilian standards and an experimental procedure has also followed.

The CFRP dowels system, have been applied which areinserted in holes in the slab acting as shear reinforcement. A portion of fiber laminate has been appliedon the top ofanother fiber strip on the both surfaces of the slab, to provide anchorage.

Four two way reinforced concrete slabs of grade 30 with dimensions 1000mm x1000mm x60mm subjected to concentrated load by a square metal plate of 85mm x85mm x50mm. Specimen is reinforced with 8mm diameter bars at 100mm spacing. Method of strengthening with CFRP dowels is shown in Figure 2.12.

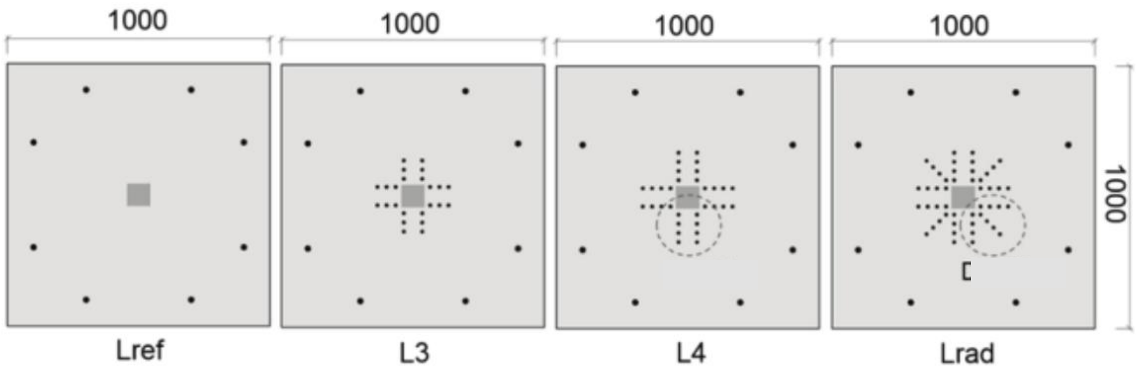


Figure 2.12.Strengthening Arrangement of Slab Specimens

It has been found that a significant incrementof shear resistance was generated in all specimens. The increment in punching capacity was 48% forthe slab L_3, 76% for $L_4$and 57% for L_{rad}.Theaverage predicted load capacity from the Brazilian code was about80% of the ultimate experimental capacity for the strengthened slab panels with carbon fiber composite.

Khaleel et al. (2013) have studied reinforced concrete slab-column connectionsstrengthened with FRP systems which are subjected to punching shear. There, the slab panels with dimensions of 1100mm x 1100mm x 100mm which consist of stub column of 150mm x 150mm have been strengthened with steel links, Glass Fiber Reinforced Polymer stirrups and CFRP stirrups in the same strengthening arrangement.

The compression side of specimen was reinforced with 10mm diameter tor steel bars at 175mm and the tension side with 10mm diameter tor steel bars at 100mm. Stump column was reinforced with 4 number of 12mm diameter tor steel bars with 6mm diameter tor steel shear links at 100mm spacing.

The strengthening system is shown in the Figure 2.13.

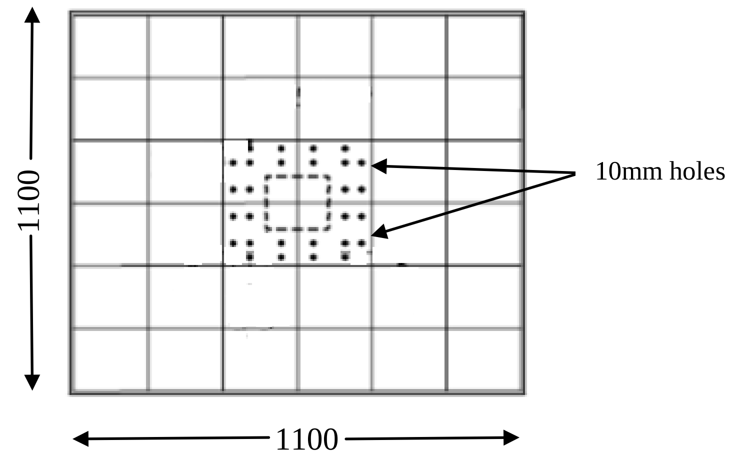


Figure 2.13.CFRP and GFRP stirrups Arrangement (Spacing between stirrups = 40mm)

It has been observed that all the strengthened specimens were cracked due to punching shear and improved the punching shear behaviour in terms of initial cracking load, flexural rigidity, and ultimate carrying capacity. The CFRP stirrups have beenreported as most efficient strengthening material which showed the best improvement in the ultimate capacity of the specimens.

Moderately conservative results were obtained by the equations used on predicting the ultimate carrying capacity of the specimens. The experimental results are higher than the calculated values by 12% to 29%

Sarah et al. (2008) have studied about design considerations of carbon fiber anchors.

The design parameters considered in this paper include;

- Size of Carbon Fiber anchors
- Number of Carbon Fiber anchors
- Spacing of Carbon Fiber anchors
- Type of CFRP material
- Surface preparation method

In order to achieve this the specimens were prepared changing the slope of tension, height of tension, anchorage of sheet, surface preparation method, size of anchorage, size of drilled hole and width of CFRP sheet to make anchor.

The specimen arrangement is shown in Figure 2.14.

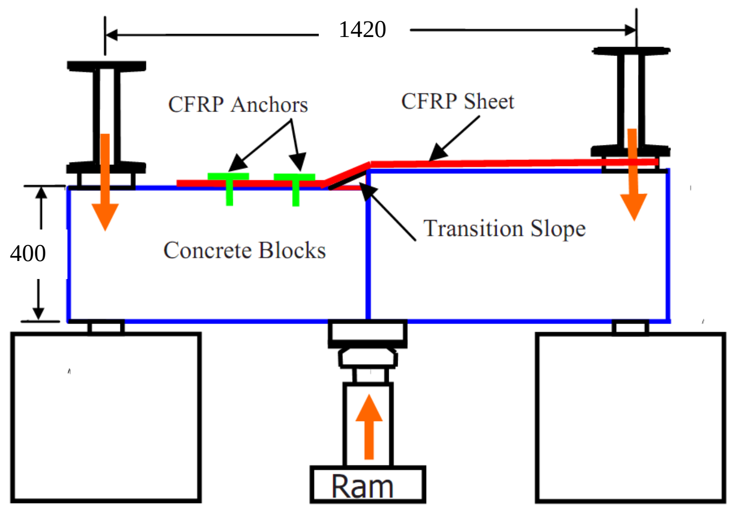


Figure 2.14.Testing arrangement

The Sarah et al, (2008)'s study has observed that the CFRP anchors facilitatedto improve tensile capacity of CFRP sheets and hence the efficiency of CFRP material usage. Also it has reduced the amount of CFRP material required to achieve a given capacity.

2.5. Summary and Research Gaps

From the previous studies, it is clear that the flat slabs are vulnerable to punching shear failure and hence to the progressive collapse of structures. Further, one of the advantageous methods to increase the punching shear capacity is that retrofitting of the punching shear area with FRP material. By comparison of FRP materials, it is proven that the most suitable material is Carbon Fiber Reinforced Polymer.

The previous researchers have followed mainly two methods to retrofit flat slabs against punching shear as;

- External bonding of FRP composite
- Inserting FRP dowel bars as shear reinforcement with surface anchorage

From those two methods, most of the researchers have experimented debonding of FRP sheets when it is bonded externally. Hence the full capacity of FRP material cannot be utilized, which lead to wastage of expensive material.

Hence it can be proposed that inserting FRP material as dowel bars to retrofit the flat slabs in the punching shear region is suitable method to utilize the material efficiently.

Several arrangements are proposed by the researchers to insert CFRP dowel bars to improve the punching shear capacity of flat slabs. Hence, the most efficient arrangements are proposed. This area is to be further studied to get the maximum use of the technology.

Also, it has to be investigated that a method to retrofit the flat slabs by using FRP, once it is failed by punching shear. Hence, in this research, finding a method for retrofitting of punching shear damaged flat slabs using CRRP sheets is expected to be considered.

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CHAPTER 3– EXPERIMENTAL PROGRAMME

3.1. Overview

The failed specimens by punching shear in a previous study were rectified and used for this study to find the best way to rectify a flat slab failed by punching shear. The dimensions of specimens are 1200 mm x1200 mm x100mm, with a central loading column stub with dimensions of 100mm x100mm x150mm and flexural steel reinforcement ratio of 0.46% in each direction. Cover to concrete is 20mm.

The specimen details are shown in Figure 3.1.

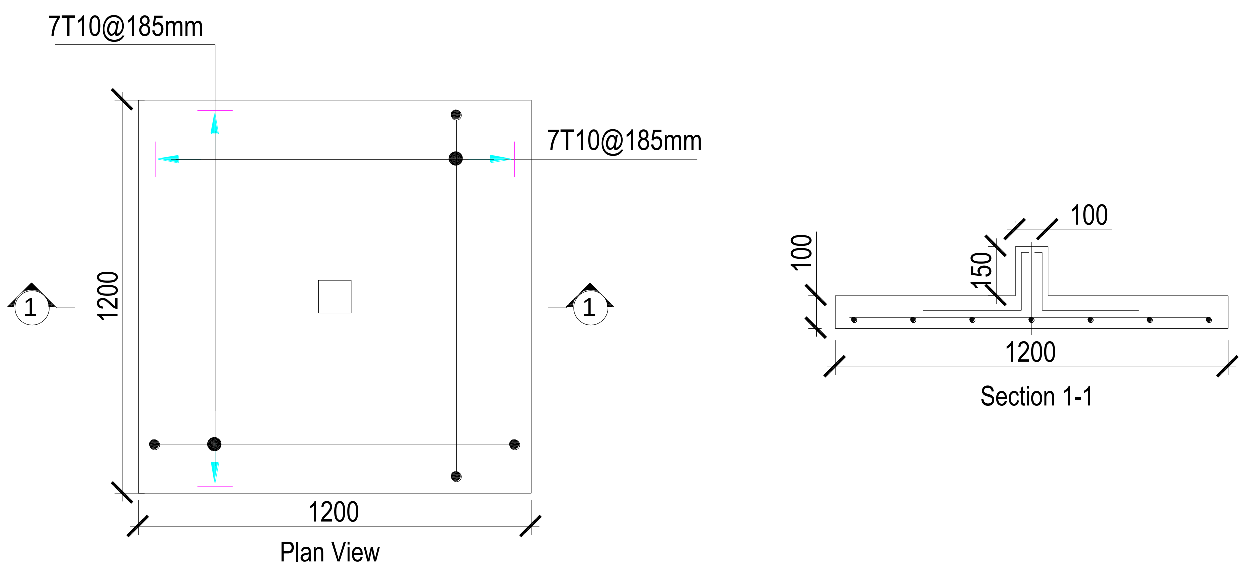

Figure 3. 1. Details of a Specimen

The average effective depth of specimen is 70mm.

According to the literature survey, carbon fiber reinforced polymer was chosen as strengthening material for retrofitting damaged flat slabs, considering the advantages as mentioned in section 2.1.4.

3.2. Summary of Specimens

The details of specimens which were failed by punching shear are summarized in Figure 3.2. The crack length of cracks in tension face is measured and presented as a multiple of effective depth (d). The critical punching shear perimeter is taken as half of the longest tension crack (BS EN 1992-1-1:2004).

Observed crack length: 2.4d ~ 7.4d

Critical punching shear perimeter = 3.7d

Observed crack length: 4.9d ~ 9.5d

Critical punching shear perimeter = 4.8d

Observed crack length: 4.6d ~ 6.7d

Critical punching shear perimeter = 3.4d

Observed crack length: 1.7d ~ 9.8d

Critical punching shear perimeter = 4.9d

Observed crack length: 0.9d ~ 9.6d

Critical punching shear perimeter = 4.8d

Observed crack length: 1.0d ~ 9.9d

Critical punching shear perimeter = 5.0d

Observed crack length: 1.0d ~ 9.0d

Critical punching shear perimeter = 4.5d

Observed crack length: 1.2d ~ 8.5d

Critical punching shear perimeter = 4.3d

Observed crack length: 2.0d ~ 7.8d

Critical punching shear perimeter = 3.9d

Observed crack length: 5.1d ~ 7.0d

Critical punching shear perimeter = 3.5d

Observed crack length: 2.2d ~ 9.0d

Critical punching shear perimeter = 4.5d

Figure 3. 2. Details of Cracked Specimens

The length of maximum diagonal tension face crack is observed in the range of 6.7d – 9.9d, and the critical punching shear perimeter ranges from 3.4d to 5.0d. Hence the diameter of the area to bedemolished and repaired was decided as 5d around the loaded area.

3.3. Material Properties

The re-bound hammer test was carried out as a non-destructive test for the cracked specimens to obtain the strength of specimens as the concrete is subjected to aging (Figure 3.3).

Figure 3. 3. Re-bound Hammer Testing

The results were obtained as follows.

Initial grade of concrete – C30 – 30N/mm^2

Test results of three specimens –

- 31N/mm^2
- 32kN/mm^2
- 34N/mm^2

$$\text{Average strength of specimens} = \frac{31 + 32 + 34}{3}$$
$$= 32.33\text{N/mm}^2$$

Two materials were used to repair the specimens to select the most suitable repair material before strengthening the specimens.

1. Cementitious Polymer Modified Concrete Repair Mortar
2. C40 concrete - Because the average strength of the specimen is less than 40N/mm^2, the great of the repairing concrete is selected as 40N/mm^2.

For some of the specimens an epoxy bonding agent was used between the existing concrete and repair material.

The details of repair materials are shown in Table 3.1 and Table 3.2.

Table 3.1. Properties of Cementitious Polymer Modified Concrete Repair Mortar Used for Repairing of Specimens

Property	Typical Value
Compressive Strength	40MPa at 28 days
Flexural Strength	4MPa at 28 days
Shrinkage	0.035% at 7 days
Water Absorption	Nil after 30 mins

Table 3.2. Properties of Epoxy Bonding Agent Used for Repairing of Specimens

Property	Typical Value
Compressive Strength	60MPa
Tensile Strength	20MPa
Flexural Strength	30MPa
Shear Strength	25MPa

Details of Carbon Fiber Reinforced Polymer sheets used to strengthen the specimens and the adhesive used to laminate the CFRP sheets to concrete are as follows (Table 3.3 and Table 3.4)

Table 3.3. Properties of CFRP Sheets

Property	Typical Value
Sheet Weight	300g/m^2
Carbon Content	95%
Net Effective Thickness	0.166mm
Modulus of Elasticity	240GPa
Tensile Strength	4000MPa
Elongation at Break	2%

Table 3.4. Properties of Lamination Adhesive

Property	Typical Value
Tensile Strength at day 7 and 20^{0}C	45MPa
Flexural Strength at day 7 and 20^{0}C	60MPa
Compressive Modulus at day 7 and 20^{0}C	1.67MPa
Glass Transition Temperature	45^{0}C
Average Heat Deflection Temperature (ASTM D648)	71^{0}C
Average Modulus of Elasticity	3.888kN/mm^2
Average Pull Off Strength	5.3MPa
Specific Gravity	1.21kg/l

The specifications and technical data sheets are attached under Annex A.

3.4. Methodology

Heavily cracked area of concrete was demolished and removed. The removed concrete was replaced with several materials as described in the section 3.4.1, and tested to select the effective repair material. The specimens repaired with selected repair material and then retrofitted with CFRP sheets as described in the section 3.4.2, and load test was carried out to find capacity of strengthened specimens as described under the section 3.4.3.

3.4.1. Repairing of Cracked Specimens

Area with diameter of five times effective depth was identified as the critical punching shear perimeter according to the observations of cracked specimen as described in Figure 3.2, and hence that area is marked and surface concrete is demolished and removed up to the reinforcement (Figure 3.4).

Figure 3. 4. Demolishing Concrete in the Cracked Area

Three specimens were repaired using different repair materials as indicated in Table 3.5.

Table 3.5. Specimen Details

Specimen Number	Repair Material	Epoxy Bond
S1	Cementitious Polymer Modified Concrete Repair Mortar	No
S2	Cementitious Polymer Modified Concrete Repair Mortar	Yes
S3	C40 Concrete	Yes

The specimen S1 was repaired with Cementitious Polymer Modified Concrete Repair Mortarafter demolishing and surface roughening, as shown in Figure 3.5.

Figure 3. 5. Preparation of Specimens without Epoxy Bond

The specimen S2 and specimen S3 were demolished and then epoxy bond was applied after roughening the contact surface as shown in Figure 3.6 (a). Then the specimen S2 was repaired with Cementitious Polymer Modified Concrete Repair Mortar, and specimen S3 with grade 40 concrete (Figure 3.6 (b)).

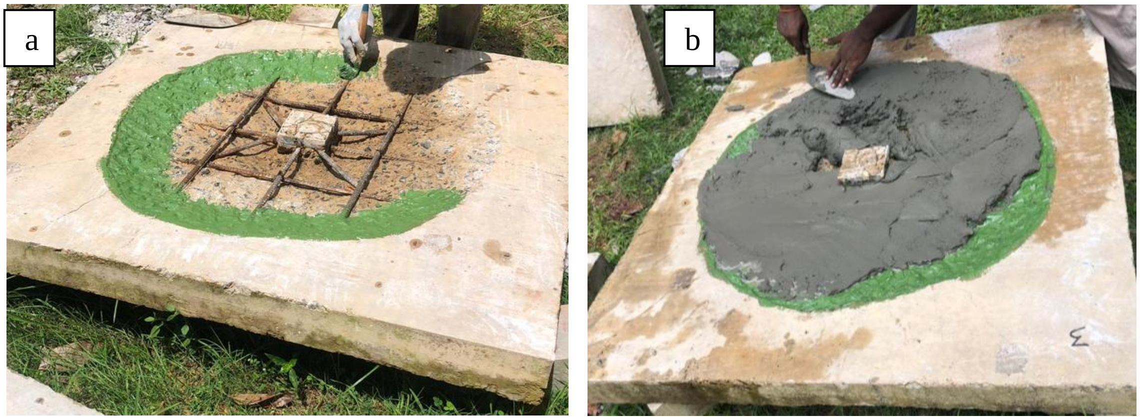

Figure 3. 6. Preparation of Specimens with Epoxy Bond

The repaired specimens were placed on a steel frame to act as simply supported along the four edges. The specimens were loaded centrally through the column stub with an increasing load until failure. The applied load was measured by using a hydraulic jack. The specimens were monitored closely while loading to observe the signs of failure. Details of testing arrangement are described under section 3.4.3.

3.4.2. Strengthening of Repaired Specimens

With reference to the previous studies, four suitable arrangements of CFRP strips were selected to strengthen the repaired slab panels with selected material as shown in Figure 3.7. The anchorage length for the CFRP strips to the existing concrete was limited to 75mm, due to the space restriction of specimens. A control specimen was also tested after repair and without strengthening with FRP.

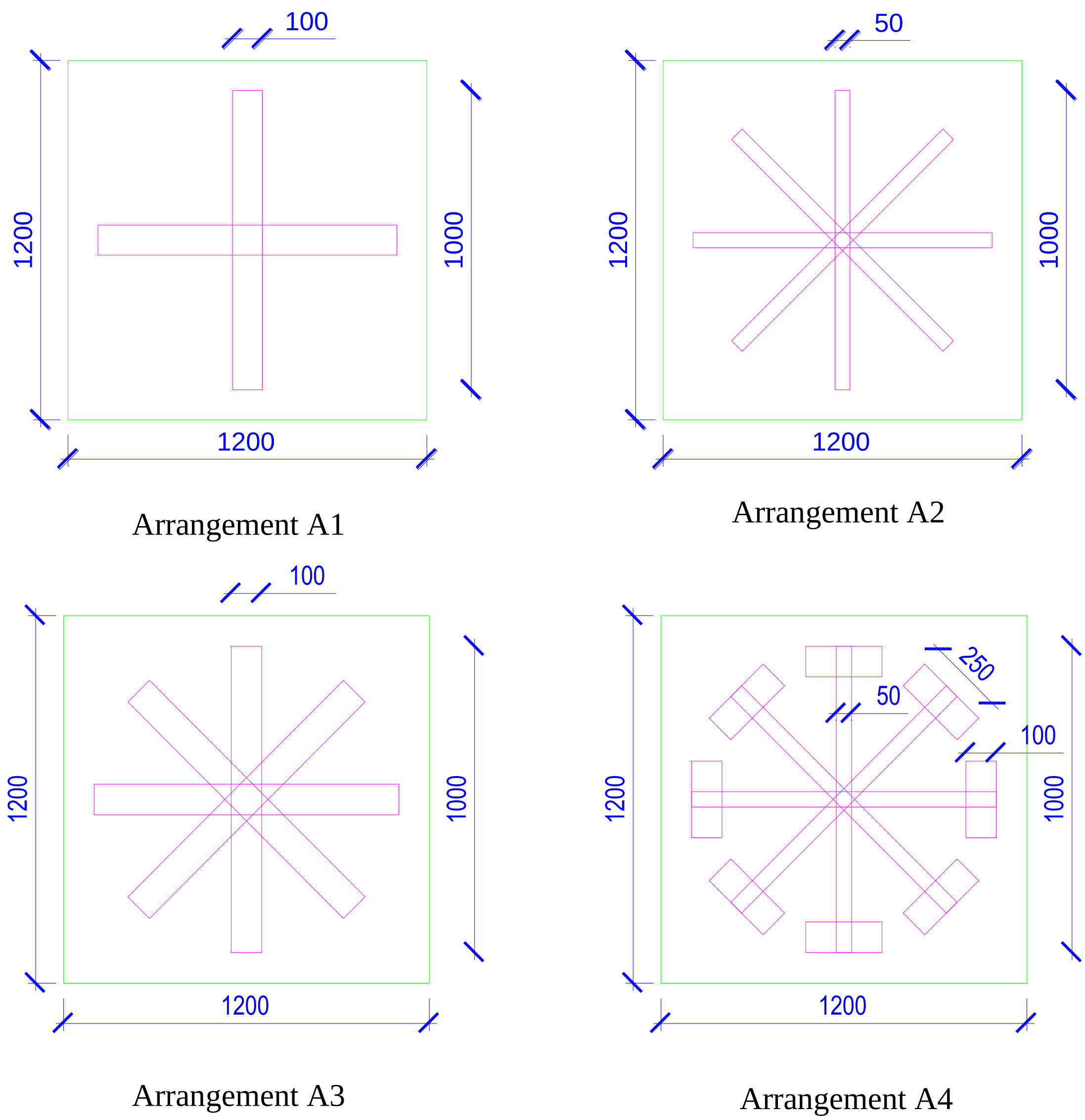

Figure 3. 7. CFRP Strip Arrangements for Strengthening the Slab Panels

The repaired specimens with the selected repair material were strengthened with CFRP strips. First the CFRP pattern is marked on the specimen and ground the concrete surface at the marked area to remove grout layer and to expose aggregate. Then the CFRP strips were bonded to the concrete with an epoxy adhesive.

Figure 3.8 to Figure 3.12 illustrates the specimens prepared for testing.

Figure 3. 8. Control Specimen

Figure 3. 9. Specimen with Arrangement A1

Figure 3. 10. Specimen with Arrangement A2

Figure 3. 11. Specimen with Arrangement A3

Figure 3. 12. Specimen with Arrangement A4

3.4.3. Testing

Two number of specimens per each arrangement and a control specimen were tested with dial gauges installed at distance‘d’ from the column face.

The specimens were placed on 100mm wide steel I beams at four sides as shown in Figure 3.13.

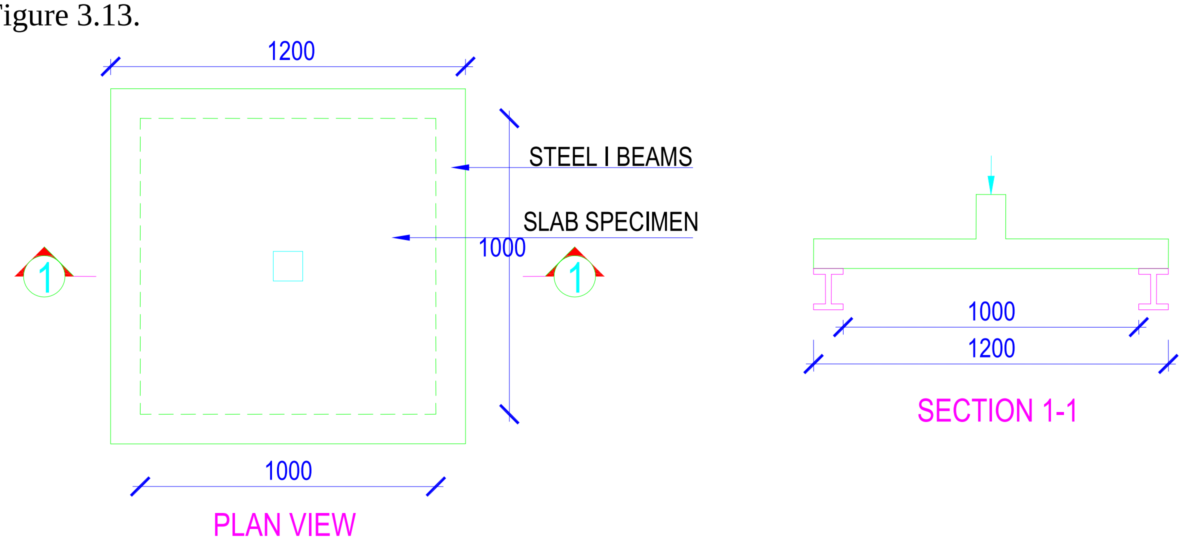

Figure 3.13. Testing Arrangement of Specimens

A gradually increasing load was applied to the column stub to obtain the failure load and the deflection at a distance of effective depth (70mm) from column face. Two dial gauges were placed at 70mm distance from column surface in either side to obtain the average value of deflection. Hence the load deflection curve for each specimen was drafted. The testing arrangement is shown in figure 3.14.

Figure 3.14. Dial Gauge Arrangement

3.5. Test Results

The results of load tests are presented under sections 3.5.1 and 3.5.2. The crack patterns of specimens are indicated under section 3.5.3.

3.5.1. Results of Specimens Repaired with Cementitious Materials

The load deflection curves of three specimens are shown in Figure 3.15.Table 3.6 describes the failure load and maximum deflection of specimens.

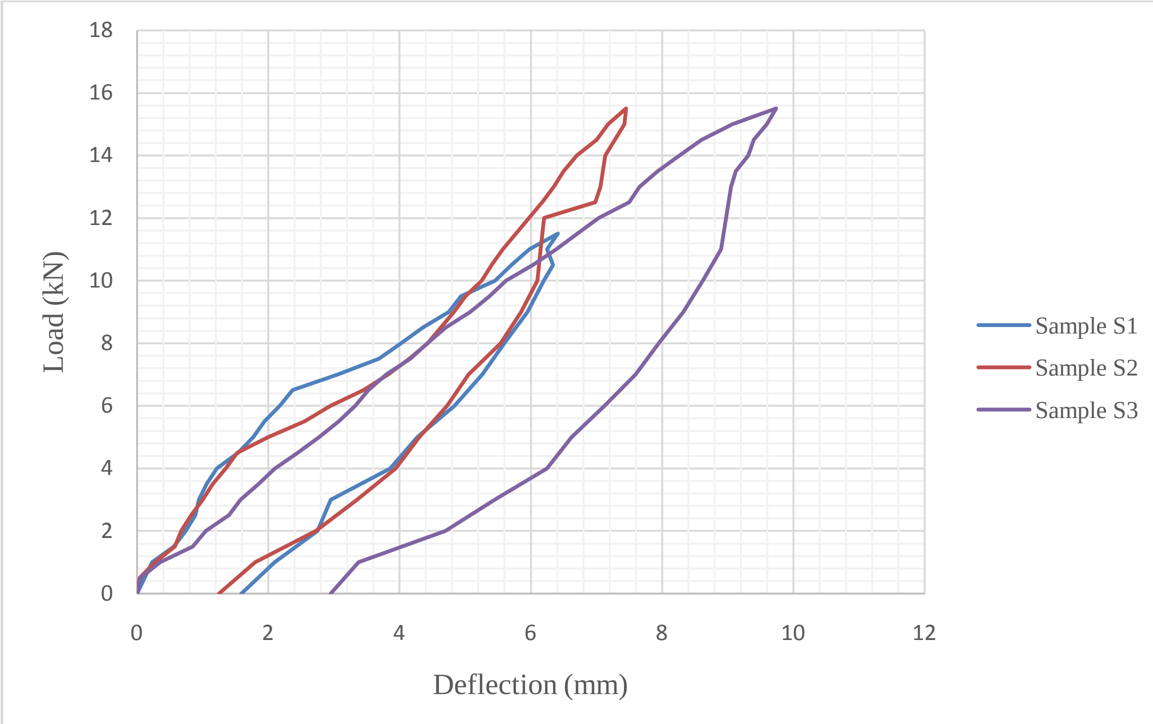

Figure 3.15. Load – Deflection Curves of Repaired specimens – deflection is measuredat ‘d’ from Column Face.

Table 3.6. Failure loads ofrepaired specimens

Specimen Number	Failure Load (kN)	Maximum Deflection (mm)
S1	11.5	6.41
S2	15.5	7.45
S3	15.5	9.74

According to the test results of the repaired specimen, a suitable repair material was selected as applied for specimen S3 for rectification of cracked flat slab panels.

3.5.2. Test Results of Retrofitted specimens

The load deflection curves of specimens are shown in Figure 3.16.

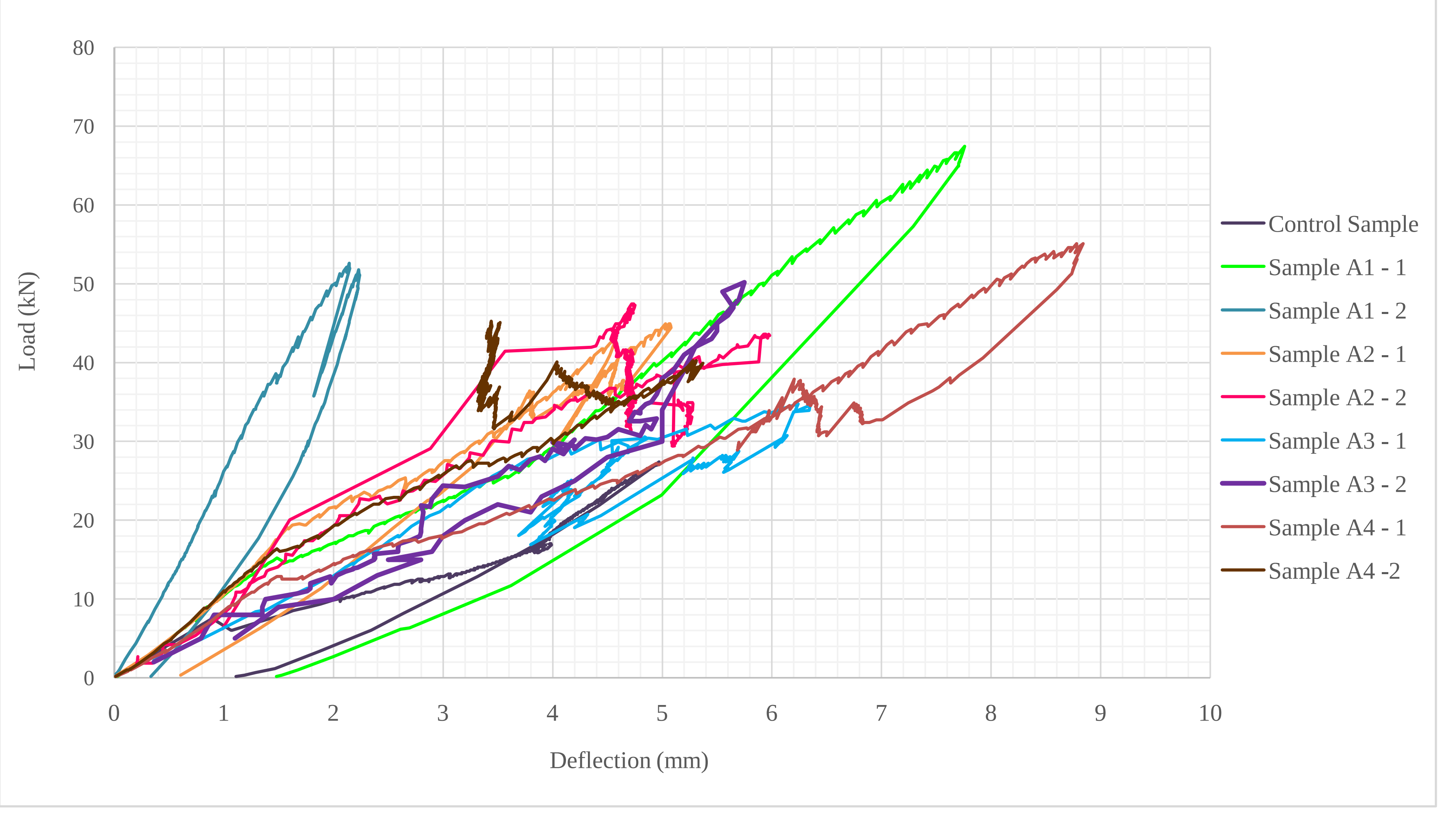

Figure 3.16. Load–Deflection Curve of Strengthened Specimens – deflection is measured at 'd' from Column Face

The failure loads and maximum deflections of specimens are tabulated in Table 3.7.

Table 3.7. Failure Loads for Strengthened Specimens

Specimen Number	Failure Load (kN)	Maximum Deflection (mm)
Control Specimen	27.4	4.97
A1 – 1	67.5	7.76
A1 – 2	52.6	2.24
A2 – 1	44.9	5.08
A2 – 2	47.4	5.98
A3 – 1	34.7	6.35
A3 – 2	50.2	5.75
A4 – 1	55.1	8.84
A4 – 2	45.3	5.37

3.5.3. Crack Patterns of Specimens

Figure 3.17. Crack Pattern of Control Specimen

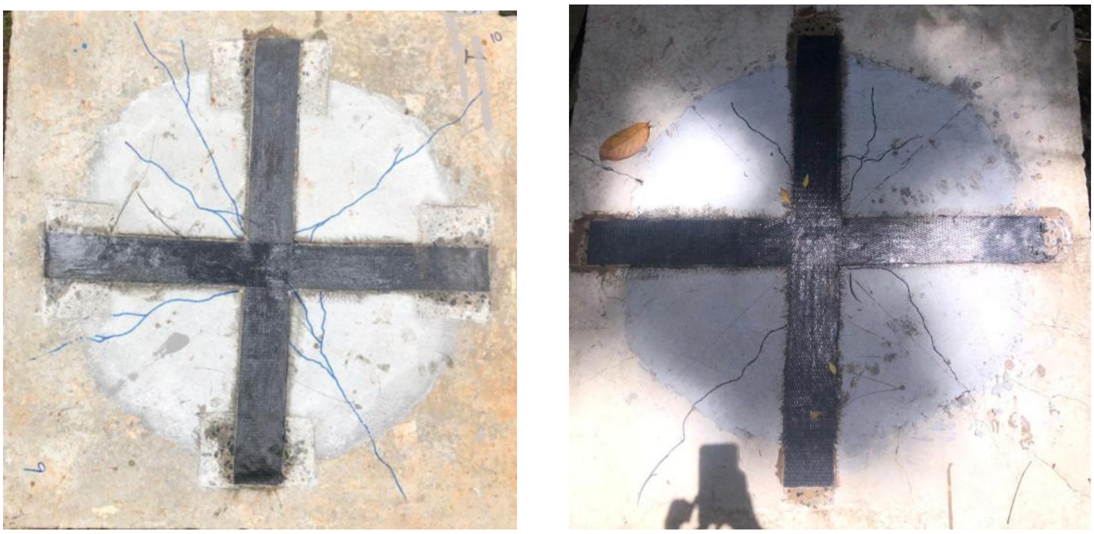

Figure 3.18. Crack Pattern of Specimen A1

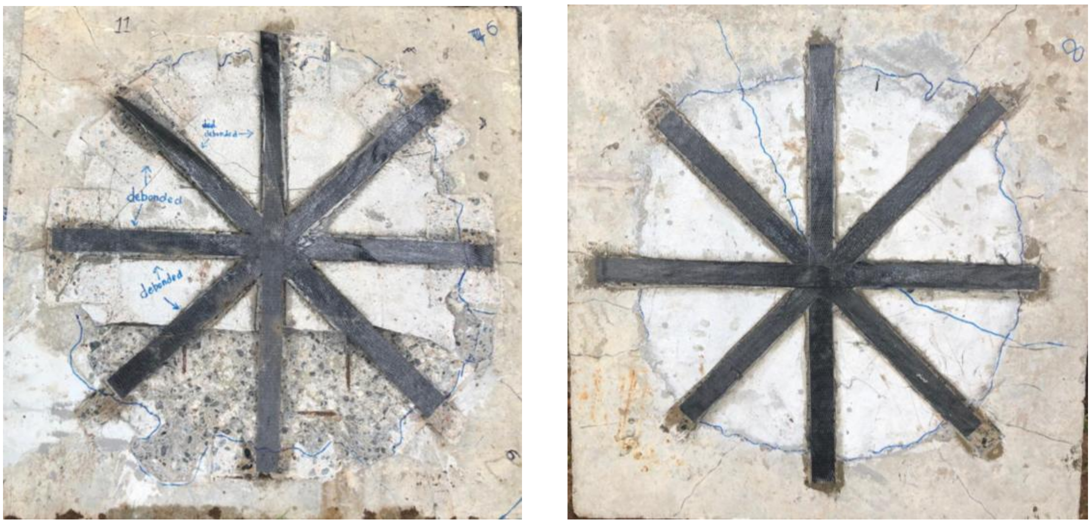

Figure 3.19. Crack Pattern of Specimen A2

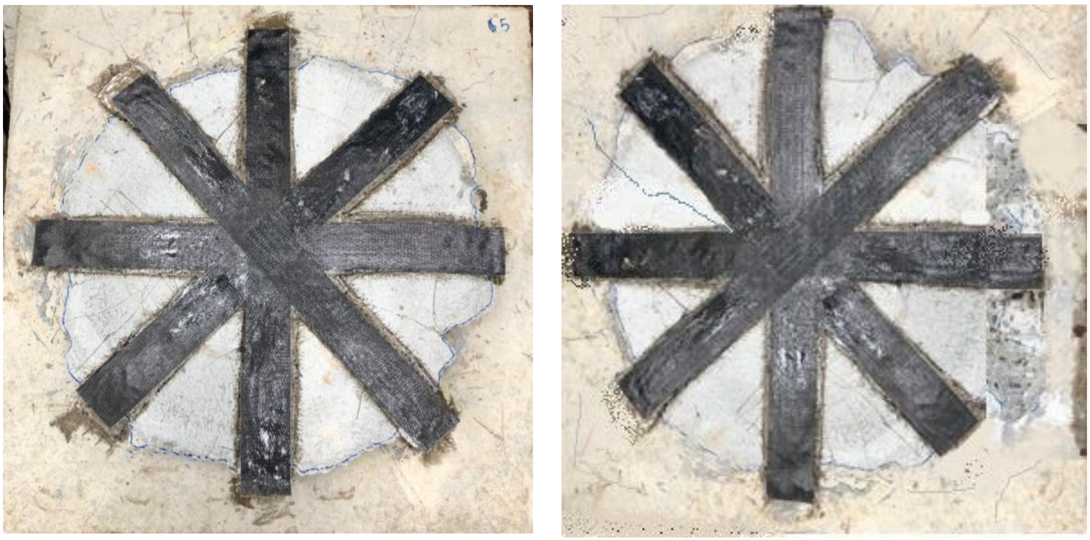

Figure 3.20. Crack Pattern of Specimen A3

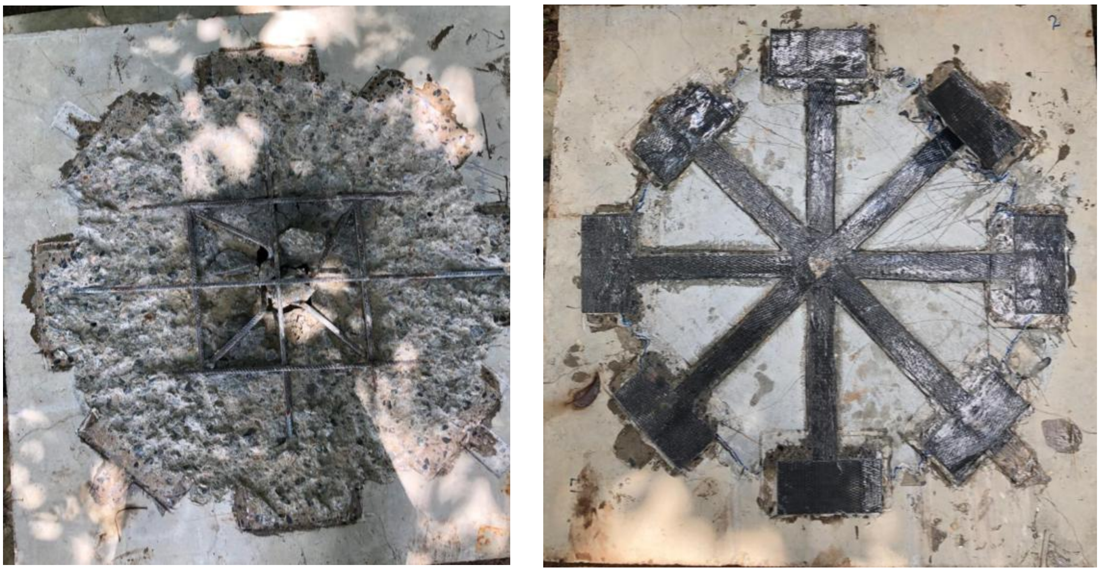

Figure 3.21. Crack Pattern of Specimen A4

CHAPTER 4 –THEORETICAL ANALYSIS

4.1. Overview

The theoretical load capacity of the un-cracked control specimen, un-cracked retrofitted specimens and cracked, repaired and retrofitted specimens were calculated using Eurocode – 2 (BS EN 1992-1-1:2004) Standards. At the end, comparison was made and the design recommendations were modified for further application in strengthening punching shear failed flat slab panels.

4.2. Procedure

The details of specimens are illustrated below;

Dimensions – 1200 mm x1200 mm x100mm

Central loading column stub – 100mm x100mm x150mm

Flexural steel reinforcement ratio – 0.46% in each direction

Cover to concrete – 20mm

Edge support condition – simply supported at each edge to represent continuation of slab specimen

Clear span – 1000mm

f_{ck}(Design) = 30N/mm^2

f_{ck}(Experimental) = 32.33N/mm^2

Average effective depth of the slab = 70mm

The reinforcement details of specimen are shown in Figure 4.1.

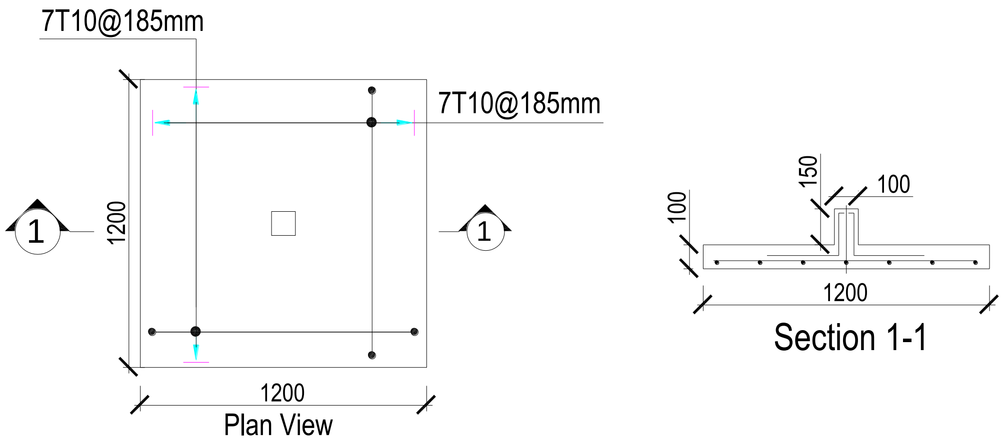

Figure 4.1. Reinforcement Details of Specimens

4.3. Theoretical Load Capacity – Control Sample

The load capacity of the un-cracked sample was calculated using Eurocode 2 – BS EN 1992-1-1:2004, for both design compressive strength and actual compressive strength which was determined experimentally, to obtain the ultimate design load of the structural element (Annex B.1). Hence, it can be decided that the load capacity of suggested retrofitting method is adequate for the failed flat slab element without changing the load applied on the structure and hence the usage of the area. The average effective depth was used for calculations. The punching shear capacity ($V_{Rd,Max}$) was calculated at the column perimeter and at first critical perimeter and hence calculated the ultimate load capacity (F_{max}). The longitudinal reinforcement ratio (ρ_1) was taken into account in the calculations.

The equations used for calculation are as follows;

The shear stress at the control perimeter;

$$v_{ED} = \frac{\beta V_{ED}}{u_0 d}$$ Equation 4.1

Where,

V_{ED} = Shear force (KN)

β = Coefficient accounts for perimeter location

u_0 = Control perimeter length (m)

d = Mean effective depth of the section (m)

The crushing strength of the specimen when no shear reinforcement is provided (calculated at column perimeter);

$$v_{Rd,max} = \frac{V_{Rd,max}}{u_0 d}$$ Equation 4.2

$$V_{Rd,max} = \frac{\propto_{cw} u_0 z v_1 f_{cd}}{(cot\theta + tan\theta)}$$ Equation 4.3

Where,

α_{cw} = Coefficient to take account of the state of stress in the compression chord

v_1 = Strength reduction factor for concrete cracked in shear

$v_1 = v = 0.6(1 - \frac{f_{ck}}{250})$

f_{ck} = Concrete compressive strength (N/mm^2)

z = Inner leaver arm for considering to the bending moment in the element ($z = 0.9d$) (m)

f_{cd} = Design concrete compressive strength ($f_{cd} = \frac{\propto_{cc} f_{ck}}{\gamma_c}$) ($N/mm^2$)

α_{cc}= Coefficient taking long term effect to compressive strength (α_{cc} = 0.85 in Sri Lankan national annex)

γ_c = Concrete partial safety factor (γ_c = 1.5)

θ = Angle between the concrete compression strut and the beam axis (Maximum value = 45^0)

Design shear resistance of concrete slab without links (calculated at 1st critical perimeter);

$$v_{Rd,c} = C_{Rd,c} k (100 \rho_1 f_{ck})^{(1/3)} + k_1 \sigma_{cp}$$ Equation 4.4

With minimum of;

$$v_{Rd,c} = v_{min} + k_1 \sigma_{cp}$$ Equation 4.5

Where,

$C_{Rd,c}$= 0.18/γc

$k = 1 + \sqrt{200/d} \leq 2.0$

σ_{cp} – Normal concrete stress in the critical section (N/mm^2)

$\rho_1 = \sqrt{\rho_{ly} . \rho_{lx}} \leq 0.02$

ρ_{ly}, ρ_{lx}= Reinforcement ratio with respect to x and y directions

k_1 = Recommended to take as 0.1

The load capacity of control sample is 60.19kN for the design concrete compressive strength and 61.11 for actual concrete compressive strength according to the calculations (Annex B.1).

4.4. Theoretical Load Capacity – Strengthened Sample

The theoretical load capacity of the specimen strengthened with CFRP sheets with considered four arrangements were calculated using BS EN 1992-1-1:2004 at the column perimeter and 1st critical perimeter for the design and actual compressive strength of concrete. The calculations were done for both design concrete compressive strength and the actual strength. The effective depth of strengthened section is calculated according to ACI 440.2R-08 standards.

The calculation of effective depth was carried out in five steps using the following equations;

1. Calculating the FRP system design material properties

$$f_{fu} = C_E f_{fu} *$$ Equation 4.6

Where,

f_{fu}* = Ultimate tensile FRP material by manufacturer (MPa)

f_{fu} = Design ultimate tensile strength of FRP (MPa)

C_E = Environmental reduction factor

$$\epsilon_{fu} = C_E \epsilon_{fu} *$$ Equation 4.7

ε_{fu}* = Ultimate rupture strain of FRP reinforcement (mm/mm)

ε_{fu} = Design rupture strain of FRP reinforcement (mm/mm)

2. Preliminary calculations

$$\beta_1 = 1.05 - 0.05(\frac{f'c}{6.9})$$ Equation 4.8

$$E_c = 4700\sqrt{f'c}$$ Equation 4.9

Where,

f'_c = Specified compressive strength of concrete (MPa)
E_c = Modulus of elasticity of concrete (MPa)

3. Determination of existing state of strain in the soffit (ε_{bi})

The existing state of strain is calculated assuming the section is un-cracked and only loads acting on the section at the time of FRP installation are dead loads.

$$\varepsilon_{bi} = \frac{My}{IE_c}$$ Equation 4.10

Where,

M = Bending moment at the mid span due to self weight of section (Nmm) calculated according to IStructE Manual for the design of concrete building structures
$M = \beta n l_x^2$
β = Bending moment coefficient
n = Total design ultimate load per unit area
l_x = Shorter span
I = Second moment of area of equivalent concrete section (mm^4)
y = Depth to the extreme fiber from neutral axis (mm)

4. Estimating the depth to the neutral axis (c) by iteration

Initial estimate of c = 0.2d

$$\varepsilon_{fe} = 0.003\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$$ Equation 4.11

$$\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{c}{d_f - c}\right)$$ Equation 4.12

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d - c}{d_f - c}\right)$$ Equation 4.13

Where,

ε_c= Strain level in concrete (mm/mm)

ε_{fe}= Effective strain level in FRP reinforcement attained at failure (mm/mm)

ε_c= Strain level existing reinforcing steel (mm/mm)

d_f = Effective depth of FRP flexural reinforcement (mm)

ε_{fd}= Debonding strain of externally bonded FRP reinforcement (mm/mm)

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{nE_f t_f}}$$ Equation 4.14

Where,

n = Number of plies of FRP reinforcement

E_f = Tensile modulus of elasticity of FRP (MPa)

t_f– Nominal thickness of one ply of FRP reinforcement (mm)

$$\beta_1 = \frac{4\varepsilon'_c - \varepsilon_c}{6\varepsilon'_c - 2\varepsilon_c}$$ Equation 4.15

$$\alpha_1 = \frac{3\varepsilon'_c\varepsilon_c - \varepsilon_c^2}{3\beta_1 \varepsilon'^2_c}$$ Equation 4.16

$$c = \frac{A_s f_s + A_f f_{fe}}{\alpha_1 f'_c \beta_1 b}$$ Equation 4.17

Where,

ε'_c = Strain corresponding to f'c ($\varepsilon'_c = 1.7f'_c/E_c$)

f_s = Stress in reinforcing steel ($f_s = E_s\varepsilon_s \leq f_y$)

f_{fe} = Stress in FRP ($f_{fe} = E_f\varepsilon_{fe}$)

5. Finding the effective depth of the section

The effective depth is found by finding the forces on FRP and steel reinforcement and then finding the load center.

The longitudinal reinforcement ration was taken as addition of two reinforcement rations obtained from longitudinal steel reinforcement and CFRP reinforcement. The reinforcement ratio of CFRP was modified to account for the material properties using two methods.

Method 01

The method proposed at BS EN 1992-1-1:2004 was used to calculate the load capacity of retrofitted sample by modifying the reinforcement ratio according to the modulus of elasticity of CFRP strips (Annex B.2).

Method 02

The reinforcement ratio was modified according to both the modulus of elasticity and strain limit of CFRP strips, as mentioned by Jacobson et al, referring to method proposed by El – Ghandour et al. (2003) (Annex B.2).

The obtained results are summarized in the Table 4.1.

Table 4.1. Load Capacity of Strengthened Specimens

Specimen Number	Punching shear load capacity of the specimen (kN)			
	Method 01		Method 02	
	f_{cu} = 30N/mm^2	f_{cu} = 32.33N/mm^2	f_{cu} = 30N/mm^2	f_{cu} = 32.33N/mm^2
A1	76.70	78.51	78.39	80.40
A2	78.66	80.65	81.87	83.94
A3	82.64	84.72	88.29	90.52
A4	78.66	80.65	81.87	83.94

CHAPTER 5 –CONCLUSIONS AND RECOMMENDATIONS

5.1. Summery

An experimental study was carried out to select a suitable infilling material and CFRP arrangement to rectify already failed flat slab panels by punching shear, with the specimen size of 1200 mm x1200 mm x100mm and flexural steel reinforcement ratio of 0.46% in each direction. Firstly, the cracked area was identified and removed after carrying out the re-bound hammer test to obtain the compressive strength of specimens and then the specimens repaired with several repairing materials. After selecting a suitable infilling material, the repaired specimens were strengthened using CFRP with four alternative bonding patterns.

The theoretical analysis was carried out to find the load capacity of;

- Control Specimen
- Strengthened Specimens with CFRP in Four Different Patterns

using the punching shear capacity calculations of BS EN 1992-1-1:2004 and the method proposed by Jacobson et al.

5.2. Discussion of Results

The design compressive strength of concrete was 30N/mm^2. The re-bound hammer test results indicated that the average strength of concrete was obtained as 32.33N/mm^2.

The samples were repaired with cementitious single component polymer modified concrete repair mortar, with and without the epoxy bond; and also with grade 40 concrete with the epoxy bond. The strength of specimens repaired with both materials with epoxy bond were failed at the same load level of 15.5kN, and the deflection of specimen repaired with grade 40 concrete was higher (Table 3.6). However, the cost of repair mortar is higher than the grade 40 concrete. Hence considering both factors, grade 40 concrete with the epoxy bond was selected as a repairing material.

The results of strengthened specimens are shown in Table 5.1.

Table 5.1. Summary of Experimental Results of Strengthened Specimens with CFRP

Specimen Number	Failure Load P_u(kN)	P_u – $P_{u,control}$	% Increment	Deflection Max. (mm)	Failure Mode
Control	27.4	-	-	4.97	Bonding surface failure
A1 – 1	67.5	40.1	146.4	7.76	Punching shear failure
A1 – 2	52.6	25.2	92.0	2.24	Punching shear failure
A2 – 1	44.9	17.5	63.9	5.08	Bonding surface failure with debonding of CFRP
A2 – 2	47.4	20.0	73.0	5.98	Bonding surface failure with punching shear cracks
A3 – 1	34.7	7.3	26.6	6.35	Bonding surface failure
A3 – 2	50.2	22.8	83.2	5.75	Bonding surface failure with punching shear cracks and debonding of CFRP
A4 – 1	55.1	27.7	101.1	8.84	Bonding surface failure with debonding of CFRP
A4 – 2	45.3	17.9	65.3	5.37	Bonding surface failure with debonding of CFRP

The specimen types A1 and A2 were strengthened using similar area of CFRP strips and at specimen A3, the CFRP area is doubled. For specimen type A4, the arrangement in specimen A2 is further strengthened using CFRP anchors at the end of strips.

The results showed that all the strengthened specimens have increment in the failure load over the control specimen varying from 26.6% to 146.4%. The specimens A1-1 and A1-2 with a cross of two 100mm wide CFRP strips were failed by punching shear and all the other specimens experienced indicated failure in the bonding surface. One of them was failed by both bonding surface and punching shear failure. Also three specimens showed debonding of CFRP strips along with bonding surface failure.

The specimens failed by punching shear failure (A1-1 and A1-2) show higher percentage increment in the failure load, irrespective from the CFRP area. This is because the specimen utilized the full strengthen capacity, because the bonding surface was not failed first.

It can be seen that the specimen which was failed by bonding surface failure reports least increment in ultimate load. The specimens which has reported bonding surface failure with shear cracks or debonding of CFRP reported more increment of failure load than the specimen failed in pure bonding surface failure. From those results, it can be determined that the weakness of bonding surface of old concrete and repair material has caused lesser increment in the failure load, though the epoxy bonding material was applied to the bonding surface while repairing the specimens.

Also, at specimen A4-1 with ultimate load increment of 101.1%, all the repair and strengthening materials have been spalled at the failure load. Hence the punching shear cracks are not observable for that specimen. It can be assumed that specimen A4-1 experienced punching shear cracks also.

According to the calculations of ultimate capacity of un-strengthened specimen (Annex B.1), the ultimate load capacity of the control slab specimen was obtained as 60.2kN.. Table 5.2 explains the deviation of failure load over target load capacity.

Table 5.2. Deviation of Results from Target Load Capacity

Specimen Number	Experimental Failure LoadF_{ex} (kN)	Deviation from Target Load (F_{ex}-60.2) (kN)	% Deviation from Target Capacity ((F_{ex}-60.2)x100)/60.2
Control	27.4	-32.8	-54.49
A1 – 1	67.5	7.3	12.13
A1 – 2	52.6	-7.6	-12.62
A2 – 1	44.9	-15.3	-25.42
A2 – 2	47.4	-12.8	-21.26
A3 – 1	34.7	-25.5	-42.36
A3 – 2	50.2	-10	-16.61
A4 – 1	55.1	-5.1	-8.47
A4 – 2	45.3	-14.9	-24.75

According to above comparison, the load capacity of control specimen with no strengthening is 54.49% lesser than the design load. From the strengthened specimens, seven specimens report the failure load capacity less than the target ultimate capacity. The specimen A1-1 has 7.3% increment in ultimate load capacity over target capacity, and also that specimen shows failure by punching shear purely, hence can be assumed that it has not the full capacity of strengthening with CFRP.

An un-cracked specimen with similar size andreinforcement detail has been tested by Silva et al. (2019) and as described at section 2.3, and obtained the ultimate load of un-cracked un-strengthened specimen as 103.00kN and 98.64kN. The value obtained by experimental results at the above literature, is greater than the values obtained from the theoretical analysis by 71.1% and 63.8%.

Comparison was made for the results of experimental failure load and theoretical load capacity for the retrofitted specimens with CFRP strips, as indicated in the Table 5.3. Both the design and actual compressive strength of concrete were considered.

Table 5.3. Comparison of Experimental and Theoretical Punching Shear Capacity

Specimen Number	Experimental Failure Load(P_u) (kN)	Theoretical Punching Shear Load Capacity											
		f_{ck} = 30kN/m^2						f_{ck} = 32.33kN/m^2					
		Method 01 (F_1) (kN)	Deviation of Theoretical Value (P_u-F_1)	Deviation (P_u-F_1)/ F_1 %	Method 02 (F_2) (kN)	Deviation of Theoretical Value (P_u-F_2)	Deviation (P_u-F_2)/ F_2 %	Method 01 (F_3) (kN)	Deviation of Theoretical Value (P_u-F_3)	Deviation (P_u-F_3)/ F_3%	Method 02 (F_4) (kN)	Deviation of Theoretical Value (P_u-F_4)	Deviation (P_u-F_4)/ F_4%
A1 – 1	67.5	76.52	-9.02	-11.79	78.58	-11.08	-14.10	78.46	-10.96	-13.97	80.56	-13.06	-16.21
A1 – 2	52.6	76.52	-23.92	-31.26	78.58	-25.98	-33.06	78.46	-25.86	-32.96	80.56	-27.96	-34.71
A2 – 1	44.9	82.40	-37.5	-45.51	85.37	-40.47	-47.41	84.48	-39.58	-46.85	87.53	-42.63	-48.70
A2 – 2	47.4	82.40	-35	-42.48	85.37	-37.97	-44.48	84.48	-37.08	-43.89	87.53	-40.13	-45.85
A3 – 1	34.7	92.09	-57.39	-62.32	97.49	-62.79	-64.41	94.41	-59.71	-63.25	99.95	-65.25	-65.28
A3 – 2	50.2	92.09	-41.89	-45.49	97.49	-47.29	-48.51	94.41	-44.21	-46.83	99.95	-49.75	-49.77
A4 – 1	55.1	82.40	-27.3	-33.13	85.37	-30.27	-35.46	84.48	-29.38	-34.78	87.53	-32.43	-37.05
A4 – 2	45.3	82.40	-37.1	-45.02	85.37	-40.07	-46.94	84.48	-39.18	-46.38	87.53	-42.23	-48.25

The experimental load capacity for all the specimens were reported lower than the theoretical capacity, as the concrete is cracked. Hence the actual properties of the cracked concrete have to be calculated to find out the theoretical punching shear capacity of the sample.

5.3. Conclusions

According to the analysis and result presented, the following conclusions can be made;

- CFRP material can be effectively used as an external reinforcing material, not only for new designs and also for strengthening of old structures.
- Normal concrete mix can be used for repairing the concrete structures effectively along with an epoxy bonding agent, rather than using the special repair grouts.
- The ultimate load capacities of all the strengthened specimens were increased by a range between 26.6% and 146.4%, over the control specimen.
- The ultimate load capacity of specimen was highly affected by bonding surface failure. Hence the usage of proper bonding material and preparation of bonding surface correctly are essential.
- All the strengthening methods showed a punching shear capacity more than the design load of the un-cracked specimen and hence can be used to retrofit existing failed flat slab elements where the design load remains the same (no usage alteration).
- The stiffness of strengthened slabs was reduced as the deflection was increased more than that of the control specimen.
- The experimental punching shear capacities of all the specimens are far below the theoretical load capacity, since the concrete is cracked.

5.4. Recommendations

The flat slabs can be failed in punching shear due to several reasons as described in 2.1.2. This may cause to global failure of structure due to changes in load transfer pattern. Hence it is important to repair the area of local failure and strengthen before leading to progressive failure. Hence retrofitting of flat slabs by using CFRP laminates can be recommended considering following advantages.

- Significant punching shear strength gain
- No floor to floor height reduction
- Retrofitting can be carried out without disturbing the building functionality
- Less time consuming for the retrofitting
- Flexibility of application

The cracked area of the damaged flat slab panel is to be removed and replaced with suitable filling material to provide suitable bonding surface for the CFRP strips. Similar or higher grade concrete with epoxy bonding material can be recommended for that purpose considering the test results of repaired specimens as described in the chapter 3.5.

All the strengthening arrangements by CFRP strips reported more than 26.6% from the failure load of control specimen. Hence it can be recommended that retrofitting with CFRP strips is an effective method to increase punching shear capacity.

Some of the specimen showed low strength gain over the strength of control specimen while some of the specimens reported more than 100% increment in failure load (Table 2.18). The minimum increment in ultimate load can be seen at the specimens failed by bonding surface failure. Hence the bond between the existing concrete and repair material can be identified as the most critical factor for strength gain.

Also debonding of CFRP strips caused failure of most of the slab panels together with bonding surface failure or punching shear failure. Hence providing sufficient anchorage for CFRP strips is important.

The bond length of CFRP strips to existing concrete of the specimens is 75mm, due to space limitations of specimens, which is not adequate. Hence it is recommended to prepare specimens in larger scale for further investigation. Also, with reference to load deflection curve and the failure load results, it can be recommended to provide CFRP anchors for high increment in punching shear strength.

The main objective of selecting the most suitable arrangement for retrofitting the failed flat slabs using CFRP strips was not achieved accurately in this research as the bonding surface failure was prominent at the experimental analysis. Hence, more accurate results can be obtained by proper preparation of the bonding surface of existing concrete and the repair material.

The theoretical calculations were made to estimate the design ultimate load capacity of the specimens. Since the experimental capacity of the un-strengthened control specimen is much lower than the theoretical ultimate load of un-cracked specimen, the strengthening is thoroughly recommended to re-use the structure.

Though the strengthened specimens also reported experimental failure load less than design ultimate load except one specimen, the strengthening patterns used in this research can be recommended to use effectively to achieve the design capacity with proper bonding interface in repairing and increasing CFRP anchorage length. Further researches are recommended in this regard. Also, it is recommended to carry out software based finite element analysis of the specimens for further investigation to validate the experimental results and design calculations to understand the behaviour of elements.

The properties of cracked concrete were not taken into account when using the code equations to find the punching shear capacity of flat slabs, other than a simple strength reduction factor based on compressive strength of concrete which is also used for fresh concrete. but there is significant reduction of punching shear strength of concrete because the bond between aggregate and aggregate interlocking is reduced. Hence significant modifications are required for code equations when design of cracked sections for shear.

The retrofitting of punching shear damaged flat slab panels with CFRP can be recommended for the industrial usage as it is very advantageous, after further investigations and modifications in design procedure.

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ANNEX A

TECHNICAL DATA SHEETS

NCC X-CALIBUR CONSTRUCTION CHEMISTRY

Technical Data Sheet

X-Wrap C300

High strength carbon fiber fabric for structural strengthening

Product Description

X-Wrap C300 is a carbon fiber uni-directional sheet designed for the strengthening of structural members against tensile, shear and impact forces. The sheet is used in conjunction with a primer and lamination resin. It is black in color.

Advantages

- Lightweight and easy to apply
- Simple application to square or curved members
- High strength to weight ratio
- Impact and blast resistant
- Can be used in conjunction with X-Wrap Pultruded Plates for combined strengthening and reinforcement

Uses

As an external reinforcement system for the structural strengthening of:

- Beams
- Columns
- Slabs
- Tunnels
- Circular, curved and complicated details

Laboratory Test Data

Sheet weight	300g/m^2
Carbon content	95%
Nett effective thickness	0.166mm
Modulus of elasticity	240GPa
Tensile strength	4000MPa
Elongation at break	2%

Packaging

500mm x 100m rolls.

Shelf Life

X-Wrap C300 should be stored in dry conditions and protected from direct sunlight; the shelf life under such conditions will be unlimited.

Installation Guidelines

X-Wrap C300 can only be applied by an NCC X-Calibur Registered Contractor using trained personnel and the correct equipment. NCC X-Calibur does not allow any contractor on to the register unless they satisfy strict selection criteria. For each application NCC X-Calibur will provide a detailed method statement that the Registered Contractor will work to.

Surface Preparation

The substrate to be treated should be cleaned and prepared by sand blasting. grit blasting, grinding or similar to remove all contamination such as dust, oil, grease, surface coatings etc. and provide a mechanical key. The prepared surface should then be checked for variation in levels using a 300mm straight edge, the maximum permissible deviation being 1 mm over a distance of 300mm In any direction. Any structural element which contains sharp arises must be ground to give a radius of at least 20mm.

Since the main purpose of installing X-Wrap C300 is to upgrade a structural element, the condition of the substrate surface must be established to ensure that it is capable of transferring load to the wrapping sheet. Therefore adhesion tests should be carries on a trial area before commencing with the installation.

Priming

The prepared substrate should be primed with X-Wrap Primer mixed and applied in accordance with the method statement.

Lay Up

X-Wrap Lamination Adhesive shall be mixed and applied in accordance with the method statement.

Immediately after applying the X-Wrap Lamination Adhesive X-Wrap C300 sheet should be laid onto the surface and pressed into position using a suitable hand lay-up (washer) roller. Further layers of laminating resin and fibre sheet must be applied in the same way. Always lay the sheet onto wet laminating resin. Care must be taken always to ensure that there is sufficient resin to completely wet-out any subsequent layer of sheet. Allow to fully cure prior to any application of decorative/ protective coating.

NCC X-CALIBUR®
CONSTRUCTION CHEMISTRY

Technical Data Sheet

X-Wrap Lamination Adhesive

Lamination resin for X-Wrap Fabrics

Product Description

X-Wrap Lamination Adhesive is an easy to apply, high performance epoxy resin designed as an impregnant for fabric reinforcement; the resulting laminate offers excellent strengthening properties coupled with long term durability.

It has been formulated as a solvent-free, semi-thixotropic system which allows easy mixing and fibre impregnation combined with minimum drainage and sag on vertical surfaces.

It is available in two grades standard and tropical

Advantages

- Solvent free
- Zero VOC
- Two grades standard and tropical
- High strength
- Strong adhesion
- Can be used in vertical and horizontal applications

Uses

As the lamination resin for X-Wrap Fabrics

Laboratory Test Data

Property	Standard	Tropical
Viscosity 20°C	5000 cps	7000 cps
Pot Life	40 mins at 25C	40 mins at 35C

Tensile Strength 7d/20°C	45MPa
Flexural Strength 7d/20°C	60MPa
Compressive Modulus 7d/20°C	1.67GPa
Glass Transition Temperature	65C

Average Heat Deflection Temperature (ASTM D 648-07)	71°C
Average Modulus of elasticity (BS 6319 part 6 -1984)	3.888 KN/mm2
Average Pull Off Strength (ASTM D4541)	5.3 MPA
Specific gravity (ASTM D792)	1.21 Kg/l

Tests are according to F.I.P. Fédération Internationale de la Précontrainte

Theoretical Coverage

0.5 to 0.6 kg/m^2

Packaging

1 and 5 kg packs.

Shelf Life

18 months when stored below 30C (86F)under shade in a dry environment.

Installation Guidelines

X-Wrap Lamination Adhesive can only be applied by an NCC X-Calibur Registered Contractor using trained personnel and the correct equipment. NCC X-Calibur does not allow any contractor on to the register unless they satisfy strict selection criteria. For each application NCC X-Calibur will provide a detailed method statement that the Registered Contractor will work to.

Application

Refer to method statement for the particular application.

Mixing

Mix X-Wrap Lamination Adhesive using the following technique. Add the hardener 'Part B' into the base 'Part A' and mix using a slow speed drill (500 rpm) with an X-Shield Coating Mixer Paddle for 3 minutes or until both components have fully dispersed and are uniform in color. Be sure to rotate the mixer throughout the drum. Mix only full packs.

Health and Safety

This product is for industrial use only by trained operatives. It is potentially hazardous if not used correctly. Please refer to the Material Safety Data Sheet (MSDS) prior to the purchase and use of this product. The MSDS can be obtained via our website www.ncc.com.eg

Authorized Technical Specialist

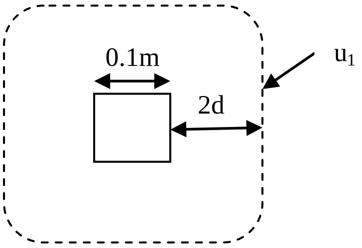
Please note that only NCC X-Calibur Authorized Technical Specialists ('ATSs') are permitted to change any of the information in this data sheet or to provide written recommendations concerning the use of this product. Visit **www.ncc.com.eg** for a full list of NCC X-Calibur ATSs.

ANNEX B

CALCULATIONS

ANNEX B.1 - SAMPLE CALCULATION FOR CONTROL SAMPLE

Reference	Calculations		Output
	Target ultimate capacity		
	<u>At column perimeter</u>		
	Perimeter of column head (u_o)	$= 0.1\text{m}\times4$ $= 0.4\text{m}$	
	Take the maximum load applicable on the column stub as $F_{max,T1}$		
EN 1992-1-1:2004	V_{Ed}	$= F_{max,T1}$	
6.4.3(3) Fig 6.21N	v_{Ed}	$= \frac{\beta.\ V_{ED}}{u_o d}$ $= \frac{1.15\text{x}F_{max}\text{x}10^3}{400\text{x}70}$ $= 0.04107\ F_{max,T1}$	
6.2.3(3) Eq 6.9	Crushing strength of specimen (No shear reinforcement)		
	$V_{Rd,max}$	$= \frac{\alpha_{cw} u_o z v_1 f_{cd}}{(\cot\theta+\tan\theta)}$	
6.2.2(6) Eq 6.6N	$v = v_1$	$= 0.6\ (1-f_{ck}/250)$ $=0.6\ (1-30/250)$ $= 0.528$	
	f_{cd}	$= 0.567f_{ck}$ $= 0.567 \times 30$ $= 17.01\text{N/mm}^2$	
	z	$= 0.9d$ $= 0.9 \times 70$ $= 63\text{mm}$	

Reference	Calculations		Output
6.2.3(3)	α_{cw} (Non prestressed member)	= 1	
6.2.3(2)	θ (Maximum allowable)	$= 45^0$	
	$V_{Rd,max}$	$= \dfrac{1.0 x 400 x 63 x 0.528 x 17.01 x 10^{-3}}{(\cot 45 + \tan 45)}$ = 113.2kN	
	$v_{Rd,max}$	$= \dfrac{V_{Rd,max}}{u_o d}$ $= \dfrac{113.2 x 10^3}{400 x 70}$ $= 4.04 N/mm^2$	
6.4.3(2)	Finding maximum value for $F_{max,T1}$		
	0.04107 $F_{max,T1}$	$= 4.04 N/mm^2$	$F_{max,T1}$ = 98.37 kN
	$F_{max,T1}$	= 98.37 kN	
Fig 6.13	First critical perimeter for shear  		
	Perimeter u_1	$= 4x(2x2d_{eff}+0.1)$ = 4x(2x2x0.07+0.1) = 1.52m	
	Area within perimeter	$= (2d_{eff}+0.1)^2$ $= (2x0.07+0.1)^2$ $= 0.1444m^2$	

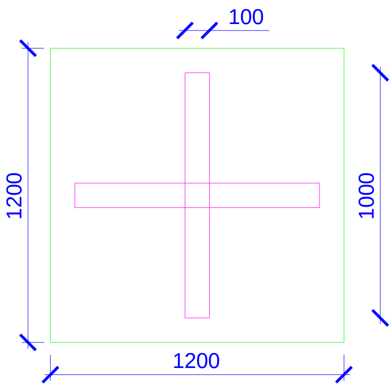
Reference	Calculations		Output
6.4.3(3)	V_{Ed}	$= F_{max,T2}$	
Eq 6.38	v_{Ed}	$= \frac{\beta.\ V_{ED}}{u_1 d}$	
Fig 6.21		$= \frac{1.15 x F_{max,T2} x 10^3}{960x70}$	
		$= 0.0108\ F_{max,T2}$	
	Design shear resistance of the concrete slab without links		
4.4.4(1)			
Eq 6.47	$v_{Rd,c}$	$= [C_{Rd,c} k(100\rho_1 f_{ck})^{(1/3)} + k_1\sigma_{cp}]$	
	With minimum of		
	$v_{Rd,c}$	$= v_{min} + k_1\sigma_{cp}$	
6.4.4(1)	σ_{cp}	$= N_{ed}/A_c$	
		$= 0$	
6.4.4(1)	k	$= 1 + \sqrt{200/d}$	
		$= 1 + \sqrt{200/70}$	
		$= 2.69 \not\leq 2.00$	
	k	$= 2.0$	
	b	= Column width + 3xeff depth	
6.4.4(1)		= 100+3x70	
		= 310mm	
6.4.4(1)	$\rho_{ly} = \rho_{lx}$	= (7x78.5)/(1200x70)	
		= 0.0065	
	ρ_1	$= \sqrt{\rho_{lx} \times \rho_{lx}}$	
		$= 0.0065 < 0.02$	
	$C_{Rd,c}$	$= 0.18 / \gamma_c$	
		= 0.18 / 1.5	
		= 0.12	

Reference	Calculations		Output
6.2.2(1)	v_{min}	$= 0.035k^{(3/2)}f_{ck}^{(1/2)}$ $= 0.035x\ 2.0^{(3/2)}30^{(1/2)}$ $= 0.54\ N/mm^2$	
	$v_{Rd,c}$	$= 0.12x2x(100x0.0065x30)^{(1/3)}+0$ $= 0.65\ N/mm^2 > 0.54\ N/mm^2$ $= 0.65\ N/mm^2$	
	Hence take $v_{Rd,c}$		
6.4.3(2)	Finding maximum value for $F_{max,T2}$		
	$0.0108\ F_{max,T2}$ $F_{max,T2}$	$= 0.65\ N/mm^2$ $= 60.19\ kN$	$F_{max,T2}$ = 60.19 kN
	Hence target ultimate load capacity of column stub	= 60.19 kN	Target load capacity = 60.19 kN
	Ultimate capacity for the actual strength		
	Average strength of specimen	= (31+32+34)/3 $= 32.33N/mm^2$	
	f_{cd}	$= 0.567f_{ck}$ $= 18.33N/mm^2$	
	$v=v_1$	$= 0.6\ (1-f_{ck}/250)$ =0.522	
6.2.3(3) Eq 6.9	<u>At column perimeter</u> $V_{Rd,max}$	$= \frac{1.0x400x63x0.522x18.33x10^{-3}}{(\cot 45 + \tan 45)}$ = 120.56kN	

	Calculations	Output
	$v_{Rd,max} = \frac{V_{Rd,max}}{u_o d}$ $= \frac{120.56x10^3}{400x70}$ $= 4.31 N/mm^2$ Finding maximum value for $F_{max,A1}$ $0.04107 F_{max,A1} = 4.31 N/mm^2$ $F_{max,A1} = 104.94 kN$ <u>First critical perimeter for shear</u>	$F_{max,A1}$ =104.94 kN
4.4.4(1) Eq 6.47	$v_{min} = 0.035k^{(3/2)}f_{ck}^{(1/2)}$ $= 0.035x\ 2.0^{(3/2)}32.33^{(1/2)}$ $= 0.56 N/mm^2$ $v_{Rd,c} = 0.12x2x(100x0.0065x32.33)^{(1/3)}+0$ $= 0.66 N/mm^2 > 0.56 N/mm^2$ $= 0.66 N/mm^2$ Hence take $v_{Rd,c}$ Finding maximum value for $F_{max,A1}$ $0.0108 F_{max,A2} = 0.66 N/mm^2$ $F_{max,A2} = 61.11 kN$ Hence actual ultimate load capacity of column stub = 61.11 kN	$F_{max,A2}$ = 61.11 kN Actual load capacity = 61.11 kN

ANNEX B.2 - SAMPLE CALCULATION FOR UN – CRACKED STRENGTHENED SAMPLE

Sample calculation – Arrangement A1

Reference	Calculations		Output
	Arrangement A1		
			
	Calculation of Effective Depth		
	1. Material properties		
ACI 440.2R (2008) 15.3 15.5	f_{fu}	$= C_E f_{cu}^*$ =0.95x4000MPa =3800MPa	
	ε_{fu}	$= C_E \varepsilon_{cu}^*$ =0.95x0.02 -=0.019	
	2. Preliminary calculations		
	β_1	$= 1.05 - 0.05(\frac{f'c}{6.9})$ = 1.05 – 0.05*(30/6.9) = 0.833	

Reference	Calculations	Output

3. Existing state of strain

Simply supported slab panel

Span =1000mm

L_y/l_x =1

β = 0.056

n = 0.1x24

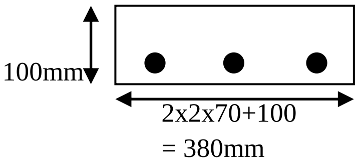
=2.4 kN/m^2

$M_x = M_y$ = $\beta n l_x^2$

= 0.056x2.4x1.0^2

=0.134 kNm/m

Finding equivalent section



Column strip section

E_c

$= 5.5\sqrt{f_{ck}/\gamma_m}$

$=5.5\sqrt{30/1.5}$

= 24.6 kN/mm^2

α_e = E_s/E_c

= 200/24.6

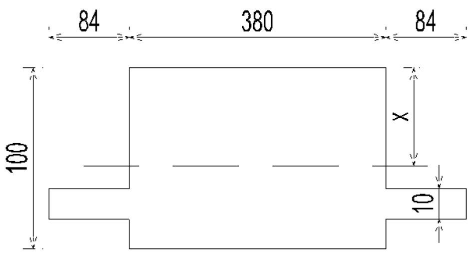
= 8.13

Equivalent concrete area of steel rf

= $\alpha_e A_s$

= 8.13x3xπx(10/2)^2

=1915.6mm^2



Equivalent section

Additional area

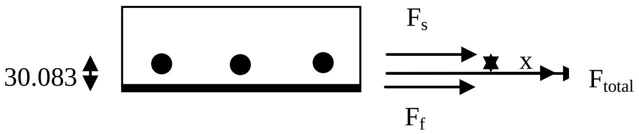
= 1915.6-πx5^2

=1680mm^2

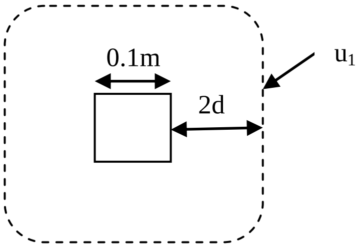
Reference	Calculations		Output
	Depth to neutral axis (x)	$= \frac{2x10x84x70+380x100x50}{2x10x84+380x100}$	
		= 50.85mm	
	I section	$= (380x100^3)/12 + (380x100x0.85^2) + (84x2x10^3)/12 + 84x2x10x(70-50.85)^2$	
		$=32.5x10^6 mm^4$	
	ε_{bi} (at bottom most fiber)	$= \frac{My}{IE_c}$	
		$= \frac{0.134x0.38x((100-50.85)x10^3)}{32.5x10^6x10^{-12}x24.6x10^6}$	
		$=3.13x10^{-6}$	
	4. Estimating depth to the neutral axis of strengthened section (c)		
	Reasonable initial estimate of c	= 0.2d	
		= 14mm	
	ε_{fd}	$= 0.14x\sqrt{\frac{f'_c}{nE_f t_f}}$	
		$= 0.14x\sqrt{\frac{30}{1x240x10^3x0.166}}$	
		=0.0113	
	ε_{fe}	$=0.003\left(\frac{d_f-c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$	
		$=0.003\left(\frac{100-14}{14}\right) - 3.13x10^{-6}$	
		$= 0.018 > \varepsilon_{fd}$	
	ε_{fe}	= 0.0113	
	ε_c	$= (\varepsilon_{fe} + \varepsilon_{bi})(\frac{c}{d_f - c})$	
		$= (0.0113 + 3.13x10^{-6})(\frac{14}{100-14})$	
		=0.00184	

Reference	Calculations	Output
	$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})(\frac{d - c}{d_f - c})$ $= (0.0113 + 3.13x10^{-6})(\frac{70 - 14}{100 - 14})$ $= 0.00736$ $\varepsilon'_c = 1.7f'_c/E_c$ $= (1.7x30)/(24.6x10^3)$ $= 0.002073$ $\beta_1 = \frac{4\varepsilon'_c - \varepsilon_c}{6\varepsilon'_c - 2\varepsilon_c}$ $= \frac{4x0.002073 - 0.00184}{6x0.002073 - 2x0.00184}$ $= 0.7367$ $\alpha_1 = \frac{3\varepsilon'_c\varepsilon_c - {\varepsilon_c}^2}{3\beta_1{\varepsilon'_c}^2}$ $= \frac{3x0.002073x0.00184 - 0.00184^2}{3x0.7367x0.002073^2}$ $= 0.8484$ $f_s = E_s\varepsilon_s \leq f_y$ $= 200x1000x0.00736$ $=1472\ N/mm^2 > 450\ N/mm^2$ $f_s = 450\ N/mm^2$ $f_{fe} = E_f\varepsilon_{fe}$ $= 240x1000x0.0113$ $= 2712\ N/mm^2$ $c = \frac{A_s f_s + A_f f_{fe}}{\alpha_1 f'_c \beta_1 b}$ $= \frac{235.6x450 + 100x0.166x2712}{0.8484x30x0.7367x380}$ $= 21.19mm$ By iteration, depth to the neutral axis is obtained as 18.3 mm.	

Reference	Calculations	Output
	Finding effective depth of section	
	$\varepsilon_{fe} = 0.003\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$	
	$= 0.003\left(\frac{100-18.3}{18.3}\right) - 3.13\text{x}10^{-6}$	
	$= 0.0134 > \varepsilon_{fd}$	
	$\varepsilon_{fe} = 0.0113$	
	$\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})(\frac{c}{d_f - c})$	
	$= (0.0113 + 3.13\text{x}10^{-6})(\frac{18.3}{100 - 18.3})$	
	$= 0.00253$	
	$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})(\frac{d - c}{d_f - c})$	
	$= (0.0113 + 3.13\text{x}10^{-6})(\frac{70 - 18.3}{100 - 18.3})$	
	$= 0.00715$	
	$\beta_1 = \frac{4\varepsilon'_c - \varepsilon_c}{6\varepsilon'_c - 2\varepsilon_c}$	
	$= \frac{4\text{x}0.002073 - 0.00253}{6\text{x}0.002073 - 2\text{x}0.00253}$	
	$= 0.781$	
	$\alpha_1 = \frac{3\varepsilon'_c\varepsilon_c - {\varepsilon_c}^2}{3\beta_1{\varepsilon'_c}^2}$	
	$= \frac{3\text{x}0.002073\text{x}0.00253 - 0.00253^2}{3\text{x}0.781\text{x}0.002073^2}$	
	$= 0.927$	
	$f_s = E_s\varepsilon_s \leq f_y$	
	$= 200\text{x}1000\text{x}0.00715$	
	$= 1430\ N/mm^2 > 450\ N/mm^2$	
	$f_s = 450\ N/mm^2$	

Reference	Calculations		Output
	f_{fe}	$= E_f \varepsilon_{fe}$ = 240x1000x0.0113 = 2712 N/mm^2	
	F_s	= 450x235.6 = 106.02kN	
	F_f	= 2712x0.166x100 = 45.02kN	
	 		
	X	= (30.083x45.02)/(106.02+45.02) = 8.97 mm	
	d $_{eff}$	= 70+8.97 =78.97 mm	

Reference	Calculations	Output
	Target ultimate capacity	
	At column perimeter	
	Perimeter of column head (u_o) = 0.4m	
	Take the maximum load applicable on the column stub as $F_{max,T1}$	
	$V_{Ed} = F_{max,T1}$	
EN 1992-1-1:2004 6.4.3(3) Fig 6.21N	$v_{Ed} = \frac{\beta . V_{ED}}{u_o d}$ $=[1.15/(400x78.97)]x10^3 F_{max,T1}$ $= 0.0364 F_{max,T1}$	
	Crushing strength of specimen (No shear reinforcement)	
6.2.3(3) Eq 6.9	$V_{Rd,max} = \frac{\alpha_{cw} u_o z v_1 f_{cd}}{(\cot\theta + \tan\theta)}$	
	$v = v_1 = 0.6 (1-f_{ck}/250)$ $= 0.528$	
6.2.2(6) Eq 6.6N	$f_{cd} = 0.567 f_{ck}$ $= 17.01 N/mm^2$	
	$z = 0.9d$ $= 0.9x78.97$ $= 71.1mm$	
	α_{cw} (Non prestressed member) = 1 θ (Maximum allowable) = 45^0	
6.2.3(3) 6.2.3(2)	$V_{Rd,max} = \frac{1.0x400x71.1x0.528x17.01x10^{-3}}{(\cot 45 + \tan 45)}$ $= 127.71kN$	
	$v_{Rd,max} = \frac{V_{Rd,max}}{u_o d}$ $= 4.04 N/mm^2$	

Reference	Calculations		Output
6.4.3(2)	Finding maximum value for $F_{max,T1}$		Target
	$0.0364\ F_{max,T1}$	$= 4.04N/mm^2$	$F_{max,T1}$=
	$F_{max,T1}$	= 111k	111kN
Fig 6.13	First critical perimeter for shear		
	Perimeter u_1	= (0.1+4x0.07897)x4	
		= 1.664m	
	Area within perimeter	$= (0.1+4x0.07897)^2$	
6.4.3(3)		$= 0.173m^2$	
Eq 6.38			
Fig 6.21	V_{Ed}	$= F_{max,T2}$	
	v_{Ed}	$= \frac{\beta.\ V_{ED}}{u_1 d}$	
		$= [1.15/(1.664x0.07897)]x10^{-3}\ F_{max,T2}$	
		$= 0.0088\ F_{max,T2}$	
4.4.4(1)	Design shear resistance of the concrete slab without links		
Eq 6.47	$v_{Rd,c}$	$= [C_{Rd,c}k(100\rho_1 f_{ck})^{(1/3)}+k_1\sigma_{cp}]$	
	With minimum of		
	$v_{Rd,c}$	$= v_{min}+k_1\sigma_{cp}$	
	σ_{cp}	$= N_{ed}/A_c$	
6.4.4(1)		= 0	
	k	$= 1+\sqrt{200/d}$	
6.4.4(1)		$= 2.59 \nleq 2.00$	
	k	= 2.0	
	b	= Column width + 3xeff depth	
		= 336.9mm	

Reference	Calculations		Output
6.4.4(1)	Method 01		
	$\rho_{ly1} = \rho_{lx1}$	= (7x78.5)/(1200x70)	
		= 0.0065	
	A_f	= 0.166x100	
		= 16.6mm^2	
	$\rho_{ly2} = \rho_{lx2}$	$= \frac{A_f E_f}{bdE_s}$	
		$= \frac{(16.6 \times 240 \times 10^9)}{(336.9 \times 78.97 \times 200 \times 10^9)}$	
		= 7.487x10^{-4}	
	$\rho_{ly} = \rho_{lx}$	= 0.0065+7.487x10^{-4}	
		= 7.249x10^{-3}	
	ρ_1	$= \sqrt{\rho_{lx} \times \rho_{lx}}$	
		= 7.249x10^{-3}< 0.02	
	$C_{Rd,c}$	= 0.18 / γ_c	
		= 0.18 / 1.5	
		= 0.12	
	v_{min}	= 0.035k$^{(3/2)}$f$_{ck}^{(1/2)}$	
		= 0.035x 2.0$^{(3/2)}$30$^{(1/2)}$	
		= 0.54 N/mm^2	
6.2.2(1)	$v_{Rd,c}$	= 0.12x2x(100x0.007249x30)$^{(1/3)}$+0	
		= 0.6699 N/mm^2 > 0.54 N/mm^2	
		= 0.6699 N/mm^2	
	Hence take $v_{Rd,c}$		
6.4.3(2)	Finding maximum value for $F_{max,T2}$		Target Load
	0.0088 $F_{max,T2}$	= 0.6699 N/mm^2	Capacity
	$F_{max,T2}$	= 76.52kN	(Method 01)
	Hence target ultimate load capacity of column stub (Method 01)		=76.52kN
	= 76.52kN		

Reference	Calculations	Output
	Ultimate capacity for the actual strength	
	Average actual strength = $32.33 N/mm^2$	
3.1.6(1)	f_{cd} = $18.33 N/mm^2$	
	At column perimeter	
	$V_{Rd,max}$ = $\frac{1.0x400x71.1x0.522x18.33x10^{-3}}{(\cot 45 + \tan 45)}$	
	= 136.06kN	
6.2.3(3) Eq 6.9	$v_{Rd,max}$ = $\frac{V_{Rd,max}}{u_o d}$	
	= $\frac{136.06x10^3}{400x78.97}$	
	= $4.31 N/mm^2$	
	Finding maximum value for $F_{max,A1}$	Actual
	$0.0364 F_{max,A1}$ = $4.31 N/mm^2$	$F_{max,A1}$
	$F_{max,A1}$ = 118.4kN	=118.4kN
	First critical perimeter for shear	
6.4.4(1) Eq 6.47	v_{min} = $0.035k^{(3/2)}f_{ck}^{(1/2)}$	
	= $0.035x\ 2.0^{(3/2)}32.33^{(1/2)}$	
	= $0.56\ N/mm^2$	Actual
	$v_{Rd,c}$ = $0.12x2x(100x0.007249x32.33)^{(1/3)}+0$	$F_{max,A2}$
	= $0.6868\ N/mm^2 > 0.56\ N/mm^2$	= 78.46kN
	= $0.6868\ N/mm^2$	(Method 01)
	Hence take $v_{Rd,c}$	
	Finding maximum value for $F_{max,A1}$	Actual load
	$0.0088F_{max,A2}$ = $0.6868\ N/mm^2$	Capacity
	$F_{max,A2}$ = 78.46kN	(Method 01)
	Hence actual ultimate load capacity of column stub (Method 01)	= 78.46kN
	= 78.46kN	

Reference	Calculations	Output
Jacobson et al. (2005)	Method 02 **Target ultimate capacity** $\rho_{ly2} = \rho_{lx2}$ $= \frac{A_f E_f \varepsilon_f}{bdE_s \varepsilon_{sy}}$ $= \frac{(16.6x240x10^9x0.0045)}{(336.9x78.97x200x10^9x0.0025)}$ $= 1.348x10^{-3}$ $\rho_{ly} = \rho_{lx}$ $= 0.0065+1.348x10^{-3}$ $= 7.848x10^{-3}$	
6.4.4(1)	ρ_1 $= \sqrt{\rho_{lx} \times \rho_{lx}}$ $= 7.848x10^{-3} < 0.02$	
6.4.4(1) Eq 6.47	$v_{Rd,c}$ $= 0.12x2x(100x0.007848x30)^{(1/3)}+0$ $= 0.688\ N/mm^2 > 0.54\ N/mm^2$ $= 0.688\ N/mm^2$	Target Load
	Finding maximum value for $F_{max,T2}$ $0.0088\ F_{max,T2}$ $= 0.688\ N/mm^2$ $F_{max,T2}$ $= 78.58kN$ Hence target ultimate load capacity of column stub (Method 02) = 78.58kN	capacity (Method 02) = 78.58kN
6.4.4(1) Eq 6.47	**Ultimate capacity for the actual strength** $v_{Rd,c}$ $= 0.12x2x(100x0.007848x32.33)^{(1/3)}+0$ $= 0.705\ N/mm^2 > 0.56\ N/mm^2$ $= 0.705\ N/mm^2$ Hence take $v_{Rd,c}$	
	Finding maximum value for $F_{max,A1}$ $0.0088\ F_{max,A2}$ $= 0.705\ N/mm^2$ $F_{max,A2}$ $= 80.56kN$ Hence actual ultimate load capacity of column stub (Method 02) = 80.56kN	Actual load Capacity (Method 01) = 80.56kN

Summary of the Calculated Results

Parameter		A1	A2	A3	A4
Steel Reinforcement Area (A_s), mm^2		235.6	235.6	235.6	235.6
Fiber Reinforcement Area (A_s), mm^2		16.6	24.9	49.8	24.9
Depth to Neutral Axis (c), mm		18.3	20.4	25.0	20.4
Effective Depth of Section, mm		78.97	81.71	85.15	81.71
Design Shear Resistance without Shear R/F - Method 01 ($v_{Rd, c}$)	f_{ck} = 30N/mm^2	0.6699	0.6793	0.7057	0.6793
	f_{ck} = 32.33N/mm^2	0.6868	0.6965	0.7235	0.6965
Target Ultimate Capacity – Method 01	f_{ck} = 30N/mm^2	76.52	82.40	92.09	82.40
	f_{ck} = 32.33N/mm^2	78.46	84.48	94.41	84.48
Design Shear Resistance without Shear R/F - Method 02 ($v_{Rd, c}$)	f_{ck} = 30N/mm^2	0.6879	0.7038	0.7471	0.7038
	f_{ck} = 32.33N/mm^2	0.7052	0.7216	0.7659	0.7216
Target Ultimate Capacity – Method 02	f_{ck} = 30N/mm^2	78.56	85.37	97.49	85.37
	f_{ck} = 32.33N/mm^2	80.56	87.53	99.95	87.53