



Punching Shear Behaviour of FRP-Reinforced Concrete Interior Slab-Column Connections

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ABSRTACT

Flat slab-column connections are common elements in reinforced concrete (RC) structures such as parking garages. In cold weather regions, these structures are exposed to de-icing salts and aggressive environments. Using fiber reinforced polymer (FRP) bars instead of steel in such structures will overcome the corrosion problems associated with steel reinforcement. However, the available literature shows few studies to evaluate the behaviour of FRP-RC interior slab-column connections tested mainly under concentric loads, which seldom occurs in a real building. The main objectives of this research are to deal with this gap by investigating the behaviour of full-scale glass (G) FRP-RC interior slab-column connections subjected to eccentric load and to provide design recommendations for such type of connections.

This study consisted of two phases, experimental and analytical. The experimental phase included the construction and testing of ten full-scale interior slab-column connections. The parameters investigated in the experimental phase were flexural reinforcement ratio, concrete compressive strength, type of the reinforcement, moment-to-shear ratio and the spacing between the shear stud reinforcement. Test results revealed that increasing the GFRP reinforcement ratio or the concrete strength increased the connection capacity. Moreover, compared to the control steel-RC specimen, the GFRP-RC connection with similar reinforcement rigidity showed comparable capacity and deflection at failure. Also, increasing the moment-to-shear ratio resulted in a reduction in the vertical load capacity, while using the shear stud reinforcement enhanced the strength up to 23%. In the analytical phase, a 3-D finite element model (FEM) was constructed using specialized software. The constructed FEM was able to predict the experimental results within a reasonable accuracy. The verified FEM was then used to conduct a

parametric study to evaluate the effects of perimeter-to-depth ratio, column aspect ratio, slab thickness and a wide range of flexural reinforcement ratio. The numerical results showed that increasing the reinforcement ratio increased the connection capacity. In addition, increasing the perimeter-to-depth ratio and slab thickness reduced the punching shear stresses at failure, while, the effect of the column rectangularity diminished for a ratio greater than three. Moreover, the results showed prominent agreement with the experimental results from literature.

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LIST OF NOTATIONS

A_c	the area of concrete according to the definition of N_{ed} (mm^2)
A_{vs}	cross sectional areas of the shear studs along the critical shear perimeter (at $d/2$ from the column face) (mm^2)
b_1	width of the critical section for shear measured in the direction of the span for which moments are determined (mm)
b_2	width of the critical section for shear measured in the direction perpendicular to b_1 (mm)
b_o	the critical shear perimeter (at $d/2$ from the column face) (mm)
b_p	the critical perimeter according to BS 8110-97 (mm)
c	depth of neutral axis for the cracked transformed section (mm)
C	the side length of the square column (mm)
C_1	the column dimension parallel to the eccentricity of the load (mm)
C_2	the column dimension perpendicular to the eccentricity of the load (mm)
$C_{Rd,C}$	calibration factor
C_{**}	distance from the centroid of the connection to the point where the shear stress is calculated (mm)
d	the effective flat plate depth (mm)
$d\ell$	a length increment of the perimeter (mm)
D	material stiffness matrix (MPa)
D_1	the diameter of the circular column (mm)
$D.L.$	factored dead load (kN/m^2)
e	eccentricity (mm)
e_1	the distance of $d\ell$ from the axis about which the moment M acts (mm)
e_{par}	the eccentricity parallel to the slab edge resulting from a moment about an axis perpendicular to the slab edge (mm)
E_c	the modulus of elasticity of the concrete (MPa)
E_1	secant elastic modulus at the peak stress (MPa)
E_F	modulus of elasticity of longitudinal FRP reinforcement (MPa)

E_{frv}	elastic modulus for the shear studs (MPa)
E_O	initial elastic modulus (MPa)
E_S	elastic modulus of steel (MPa)
f_c'	compressive strength of concrete (MPa)
$f_c'^{ef}$	effective concrete compressive strength (MPa)
f_{ck}	the characteristic concrete cylinder strength (MPa)
f_f	stress of the FRP reinforcement (MPa)
f_{frv}	specified stress in the FRP-studs (MPa)
f_{pcd}	the design compressive strength of concrete (MPa)
f_t	concrete tensile strength (MPa)
$f_t'^{ef}$	maximum tensile stresses (MPa)
f_y	the yielding strength of the steel reinforcement (MPa)
G_f	fracture energy needed to create a unit area of stress-free crack (MN/m)
J_c	property of the critical shear section of slabs analogous to the polar moment of inertia (mm^4)
J_{cy}	property of the critical shear section of slabs analogous to the polar moment of inertia along y-axis (mm^4)
J_{cx}	property of the critical shear section of slabs analogous to the polar moment of inertia along x-axis (mm^4)
k	coefficient depends on the ratio between the column dimensions
k_1	the coefficient considering the effects of axial forces on the stress distribution
k_2	the maximum ratio of the average compressive stress to the characteristic cube concrete strength f_{cu} , corresponding to the concrete strain ϵ_{cu}
ℓ	typical distance between the columns (mm)
L_d	band size which defined as the projection of the element on the failure plane (mm)
$L.L.$	factored live load (kN/m^2)
M_{cr}	cracking moment (kN.m/m)
M_n	flexural capacity of the slab in the direction of moment transfer (kN.m/m)

M_p	plastic moment (kN.m/m)
M_x	unbalanced moment around the x-axis (kN.m)
M_y	unbalanced moment around the y-axis (kN.m)
N	constant taken equal to 5/6 for f'_c in MPa, b_o in mm and d in mm
N_{Ed}	the longitudinal forces across the full bay for interior columns and the longitudinal force across the control section for edge columns (N)
n_f	ratio of modulus of elasticity of FRP bars to modulus of elasticity of concrete
q^*	total shear stress at a point (MPa)
q^{**}	shear stress at a point due to the effect of the unbalanced moment (MPa)
s	stress vector (MPa)
S	spacing between the shear studs (mm)
s_1	bond slippage (MPa)
t	bond stresses (MPa)
t_{max}	the maximum bond strength of the steel bar embedded in concrete (MPa)
u	the peripheral length of the column (mm)
U_i	the length of the control perimeter being considered (mm)
U_1	the length of the basic control perimeter (2.0 d from the column faces) (mm)
u_p	the peripheral length of the design cross-section at $d/2$ from the column face (mm)
V	the factored shear force (N)
V_c	punching shear capacity (N)
V_{ED}	punching shear stresses (MPa) according to European code
$V_{exp.}$	failure load (kN)
v_f	the total stresses at a point (MPa)
V_F	uniform factored shear stresses (MPa)
$V_{flex.}$	flexural capacity (kN)
V_J	the nominal shear strength provided by concrete (N) according to the Japanese standard

$V_{norm.}$	normalized failure load (kN)
v_r	ultimate punching shear stress resistance for concrete (MPa)
$V_{Rd,C}$	the shear resistance of concrete (MPa) according to European code
v_s	the factored shear resistance provided by the shear studs (MPa)
w	total factored load (kN/m ²)
W_1	factor function of the perimeter U_i
w_c	the crack opening at the complete release of stress (mm)
w_d	plastic displacement (mm)
x	normalized strain
$(X)_f$	combined depth of the compression zone (mm)
$(X_p)_f$	the depth of the flexural section (mm)
X_s	the depth of the shear section (mm)
α_s	4.0 for interior column, 3.0 for edge column, and 2.0 for corner column in CSA standards and 40 for interior column, 30 for edge column and 20 for corner column in ACI codes.
β_c	the ratio of long side to short side of the column
β_d	coefficient to consider effect of effective depth on shear capacity
β_p	coefficient to consider effect of effective reinforcement ratio defined as the average values for the reinforcement in two directions
β_r	factor represents the effect of the loaded area
γ_b	safety factor
γ_{xy}	the engineering shear strain
γ_v	fraction of the unbalanced moment transferred by eccentricity
γ_{vy}	fraction of the unbalanced moment transferred by eccentricity in the direction of y-axis
γ_{vx}	fraction of the unbalanced moment transferred by eccentricity in the direction of x-axis
ε	strain vector
ε_c	strain at the peak stress $f_c^{/ef}$

ε_{cu}	the specified value of concrete strain (0.0035)
ε_d	the strain at the complete release of the stresses
ε_f	strain of the FRP flexural reinforcement
ε_{frv}	strain of the GFRP shear studs
ε_x	strain in (x) direction
ε_y	strain in (y) direction
λ	factor to account for low-density concrete (1.0 for normal weight concrete)
ρ	longitudinal reinforcement ratio
ρ_F	longitudinal reinforcement ratio for FRP
ρ_{fb}	balanced reinforcement ratio
σ_c	normal concrete stresses (MPa)
σ_c^{ef}	concrete compressive strength (MPa)
$\sigma_1 \text{ \& } \sigma_2$	principle stresses (MPa)
σ_x	normal stress in the (x) direction (MPa)
σ_y	normal stress in the (y) direction (MPa)
ϕ	resistance factor applied to a specified material property or to the resistance of a member
τ_{xy}	shear stress in plane (xy) (MPa)
θ	the mean angle of the failure surface (degree)

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Corrosion of steel reinforcement is one of the major problems facing the civil engineering industry, which affects or limits the life expectancy of any concrete building. Much research has been conducted to overcome this problem. One of the leading methods that have been found to overcome the corrosion problem is to replace the traditional steel reinforcing bars with fibre-reinforced polymer (FRP) bars.

The FRP reinforcement have many advantages over ordinary steel reinforcement such as light weight (FRP bars have a density ranging from one-sixth to one-fourth that of steel (ACI Committee 440 2006)), which reduces the cost of transportation and handling. Also, FRPs have high tensile strength which is approximately twice to triple the yielding stress of steel as well as high resistance to corrosion and many types of salts. However, the behaviour of FRP bars differs from that of steel in many ways. For example, FRP bars have perfectly linear-elastic behaviour up to failure with no yielding, and therefore, FRP bars do not exhibit ductile behaviour in concrete structures. Furthermore, FRP bars have relatively low modulus of elasticity; for instance, in case of GFRP bars, it is approximately one fourth to one third that of steel. In addition, FRP bars have different bond characteristics compared to steel. For example, sand-coated GFRP bars have adhesion and friction bond which uniformly distribute the bond stresses along the embedded length, while steel deformed bars have mainly mechanical bond through bearing on the deformation. Consequently, concrete members reinforced with sand-coated GFRP

bars would have less average crack spacing compared to concrete members reinforced with the steel ones.

A flat plate structural system is composed of slab rested, directly, on columns. This system has many applications in the construction industry due to its numerous advantages, which include the simplified formwork, reduced storey heights and its ability to sustain heavy loads. A large number of reinforced concrete (RC) parking garages in North America are constructed using flat plates supported directly on columns. These structures are subjected to severe environmental conditions, such as freeze-thaw cycles and de-icing salts, which may cause corrosion of steel and limit the life expectancy of the building. Using FRP bars as internal reinforcement in such structure would overcome the corrosion problem associated with the steel bars. On the other hand, the lack of experimental and analytical studies on the behaviour of flat plate systems reinforced with FRP bars limits the knowledge and the full understanding of the behaviour of such plates.

1.2 PROBLEM DEFINITION

Punching shear failure in flat slab-column connections is a main concern for the designers, since it is sudden and brittle. This failure occurs due to the high transverse shear stresses caused by shear force and unbalanced moment transfers from the slab to the columns. This combination of shear and unbalanced moment is unavoidable at slab-column connections, even under gravity loads only, due to unsymmetrical loading or/and unequal spans in addition to the effect of lateral loads, if any. Prevention of punching shear failure of slab-column connections depends on accurate calculations of shear stresses produced by the shear forces and the unbalanced moments transferred to the columns. Also, the design and detailing of slab-column connections are critical to ensure the satisfactory performance of flat plate structures.

The shear resistance of an RC member is composed of two components. The first is the shear resistance provided by concrete and the second is the shear resistance provided by shear reinforcement, if any. The concrete resistance for shear stresses is influenced by the properties of the flexural reinforcement. After cracking and due to the relatively-low modulus of elasticity of the FRP bars compared to steel, FRP-RC members are expected to have wider cracks and smaller depth to neutral axis. As a result, the shear resistance provided by both aggregate interlock and un-cracked concrete is smaller. Also, FRP bars are uni-directional materials (anisotropic) with relatively low strength and stiffness in the transverse direction compared to those in the longitudinal direction. This property results in smaller dowel force and consequently less contribution to shear resistance (ACI Committee 440 2006) compared to steel-RC structures. Consequently, it is expected that shear resistance of FRP-RC members, in general, will be less than their counterparts reinforced with steel. Therefore, it is not valid to directly use the code equations for concrete members reinforced with steel bars for those reinforced with FRP bars.

Recently, the Canadian Standard Association (CSA) published an updated version of the FRP design code, CSA/S806-12 (CSA 2012). This new version introduces three equations to predict the punching shear capacity of FRP-RC slab-column connections; however the code does not include any provision for slab-column connections reinforced with shear reinforcement.

Up to date, there is no research conducts to evaluate the behaviour of full-scale interior slab-column connections reinforced with GFRP bars under eccentric load, the available literature is dealing mainly with FRP-RC interior slab-column connections subjected to concentric load. The research program described herein is designed to partially fill this gap.

1.3 SCOPE OF WORK

This research study focused on studying the behaviour of full-scale isolated interior slab-column connections reinforced with glass FRP bars. Glass FRP bars are currently of interest to the infrastructure owners, FRP manufacturers and the construction industry in general due to their lower cost compared to the other types of FRP bars (carbon and aramid). As such, only GFRP bars were employed in this study. The specimen represented the zone of negative moment around an interior column isolated from a multi-story, multi-bay building (prototype) with columns equally spaced at 6.5 m in both directions. The slabs were assumed to be simply-supported at the lines of contra flexure around an interior column. Also, the slabs were reinforced in flexure with GFRP bars, with and without GFRP shear reinforcement, while the columns for all specimens were adequately reinforced with steel bars and stirrups to prevent premature failure. In addition, all specimens were subjected simultaneously to shear force and unbalanced moment throughout the test. Furthermore, no earthquake or wind loads were considered in this research, the unbalanced moment is due to the effect of the gravity loads only.

1.4 OBJECTIVES AND AIMS OF THE PRESENT STUDY

This study investigated the behaviour of isolated interior slab-column connections reinforced with GFRP bars subjected simultaneously to shear and unbalanced moment. The behaviour of these concrete slab-column connections was investigated analytically and experimentally.

The main objectives of this research were to:

- 1- Investigate the behaviour and shear capacity of GFRP-RC interior slab-column connections subjected to eccentric load.
- 2- Provide recommendations for designers and researchers regarding the analysis and the design of GFRP-RC flat slab-column connections.

The specific objectives of the experimental study were to investigate the effect of the following main variables on the behaviour of slab-column connections:

- 1- Flexural reinforcement ratio (1.5, 2.25 and 3 ρ_b , where ρ_b is the balanced reinforcement ratio).
- 2- Concrete compressive strength (35 and 70 MPa).
- 3- Type of the reinforcing bars (GFRP and steel).
- 4- Shear reinforcement ratio (0.0, 0.41 and 0.61%).
- 5- Moment-to-shear ratio (0.0, 0.15 and 0.3 m).

The specific objectives of the analytical study were to investigate the effect of the following variables on the behaviour of slab-column connections:

- 1- A wide range of flexural reinforcement ratios (between 0.5 and 4.0 ρ_b , with 0.5 ρ_b increments).
- 2- Shear perimeter-to-depth ratio (11.5, 15.25 and 19 corresponding to square column cross section with side dimension of 300, 450 and 600 mm and effective slab depth of 160 mm).
- 3- Column aspect ratio between 1.0 and 5.0.
- 4- Slab thickness between 200 and 350 mm.

1.5 METHODOLOGY AND APPROACH

This research was designed mainly to investigate the behaviour of isolated slab-column connections reinforced with GFRP bars. During this research the provisions of different available codes such as the Canadian code (CAN/CSA-S806 2002 and 2012); the American Concrete Institute guideline (ACI 440.1 R-06) and Japanese standard (JSCE 1997) regarding the behaviour of this type of connection were also investigated.

The current study consists of two phases. The first is the experimental phase that included the construction and testing of full-scale isolated interior slab-column connections reinforced with GFRP bars. Each test specimen represented a column connected to a slab part-bounded by the lines of contra-flexure around the column. The dimensions of the specimens were selected to simulate a parking garage with flat plate square panels measuring $6500 \times 6500 \times 200$ mm. In addition, the specimens were tested under eccentric load (shear force and unbalanced moment) in upside-down position with respect to the actual position in a real structure. Also, the loads were applied on equal increments. The shear force was applied first on the top of the upper column stub then followed by two equal and opposite horizontal forces on the column stubs, that were extended above and below the slab, to simulate the effect of the unbalanced moment.

The second phase included the analytical studies, where computer model was constructed using the non-linear finite element analysis program ATENA (Cervenka et al. 2013) to predict the performance of the connections reinforced with GFRP bars. The efficiency and accuracy of the computer model were verified against the experimental results. Then, using the verified model, a parametric study was conducted to investigate the effect of a wide range of key parameters as listed above in the specific objectives section.

1.6 THESIS ORGANIZATION

The thesis consists of seven chapters as follows:

- ✓ Chapter one includes the problem definition, scope of work, objectives of the study followed by the methodology to achieve these objectives.
- ✓ Chapter two presents general information about FRP materials, the provisions of codes and guidelines that have been published to deal with the punching shear of FRP-RC or

steel-RC slab-column connections, In addition to relevant research related to the punching shear behaviour of slab-column connections reinforced either with steel or FRP bars.

- ✓ Chapter three deals with the experimental part of the research and presents the description of the test specimens (dimensions and reinforcement details), the details of the instrumentation that was used to monitor the behaviour of the specimens (LVDTs, strain gauges for the reinforcement and concrete and PI-gauges), as well as the test set-up and procedure.
- ✓ Chapter four provides the analysis and discussion for the experimental phase in terms of cracking, mode of failure, ultimate strength, deflections in the slabs, strains in the reinforcement and concrete and comparison to relevant codes
- ✓ Chapter five presents an over view of the finite element (FE) software that was used in the analytical phase, (*Cervenka et al. 2013*), as well as the construction and verification of the numerical model.
- ✓ Chapter six deals with the analysis and discussion of the parametric study, conducted using the verified FE model, in terms of deflections of the slabs, strains in the reinforcing bars and ultimate capacity.
- ✓ Chapter seven presents the conclusions that were drawn from the experimental and the numerical phases, in addition to the recommendations for the future work.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

The FRP materials have been used for almost a century now. The first known FRP product was a boat hull manufactured in the mid 1930's as part of a manufacturing experiment using a fiberglass fabric and polyester resin laid in a foam mould (ACI Committee 440 2006). The applications of FRP composites have revolutionized entire industries such as aerospace, marine, electrical and transportation. One of its recent applications in the civil industry is in form of reinforcing bars, tendons, wraps, laminates, composite bridge decks and composite structural sections.

Due, mainly to their high-corrosion resistance, applications of FRPs as reinforcement for concrete structures have been growing rapidly in recent years. A significant number of projects have been constructed all over the world such as Laurier-Taché parking garage in Canada (Benmokrane et al. 2004), West Mill Bridge in United Kingdom, Kolding Bridge in Denmark and the Havenbrug Harbour Bridge in Holland (Murphy 2013). The use of FRP reinforcement in Europe began in Germany with the construction of a prestressed FRP highway bridge in 1986 (Meier 1992). In addition, North America and Japan have the most FRP reinforcement applications or commercial projects compared to Europe (Teng 2001). However, many case-specific studies are still needed for better understanding of FRP materials and their interaction with concrete materials and structures than is currently available (ACI Committee 440 2006).

Several design guidelines and codes have been published to address different design aspects of FRP-RC structural elements such as CSA/S806-12 (CSA 2012), CSA-S6-06 (CSA 2006), ISIS

Canada Design Manuals (2007) and ACI440.1R-06 (ACI 2006). However, significant research is still needed, especially in the case of flat plate structures.

Flat plate is a system consists of slab rested directly on columns. This structural system has many applications in the civil industry, due to its numerous advantages which includes, reduced storey heights, ability to cover longer spans, short construction period, constructional simplicity and flexibility in the plan layout (both in terms of the shape and columns layout).

A large number of the reinforced concrete parking garages in North America are constructed using this system. The safety of flat plate system should be ensured against all types of failure. One of the types of failure of flat plate system is the punching shear failure which occurs around columns due to high principle tensile stresses. Studying the behaviour of slab-column connections has attracted many researchers all over the world. Extensive research has been conducted to investigate the behaviour of slab-column connections reinforced with steel reinforcement. However, very limited research exists on FRP-RC slab-column connections.

2.2 PROPERTIES OF FRP REINFORCEMENT

Several codes and design guidelines, such as CSA/S806-12 (CSA 2012) and ACI 440.1R-06 (ACI 2006), introduced design provisions for internal FRP reinforcement, prestressed FRP reinforcement and test methods for FRP products. Although these provisions exist there are many gaps related to specific applications and/or structures, including slab-column connections, still remain. In addition, new products and applications are continually being developed, which justify the need to more experimental and analytical studies to cover all the aspects of the FRP materials. The following section focuses on the FRP physical and mechanical properties.

2.2.1 Physical Properties

The available FRP reinforcing materials are made of continuous aramid, carbon or glass fibres embedded in a resin matrix. The density of the FRP material is one-sixth to one-fourth of that of steel as shown in Table 2.1. The coefficients of thermal expansion of FRP bars are different in the longitudinal and transverse directions. The longitudinal coefficient of thermal expansion depends on the properties of the fibres, while the transverse coefficient depends on the properties of the resin as shown in Table 2.2 (ACI Committee 440 2006).

Table 2.1: Typical densities of reinforcing bars, reproduced from ACI-440.1R-06 (ACI Committee 440 2006).

Density	Steel	GFRP	CFRP	AFRP
g/cm^3	7.9	1.25 to 2.1	1.5 to 1.6	1.25 to 1.4
lb/ft^3	493	77.8 to 131	93.3 to 100	77.8 to 88.1

Table 2.2: Typical coefficients of thermal expansion (CTE) for reinforcing bars, reproduced from ACI-440.1R-06 (ACI Committee 440 2006).

Direction	$CTE \times 10^{-6}/C$			
	Steel	GFRP	CFRP	AFRP
Longitudinal, α_L	11.7	6 to 10	-9 to 0	-6 to -2
Transverse, α_T	11.7	21 to 23	74 to 104	60 to 80

2.2.2 Mechanical Properties

Unlike steel reinforcement, FRP is an elastic material as shown in Fig. 2.1, which does not exhibit any yielding behaviour before rupture. The tensile strength of FRP bars is much higher

than that of steel bars, although it might vary from one manufacturer to another due to manufacturing process. The stiffness of FRP bars depends mainly on the type of materials used. In general, the stiffness of FRP bars is lower than that of steel as shown in Table 2.3.

Table 2.3: Usual tensile properties of reinforcing bars, reproduced from ACI-440.1R-06 (ACI Committee 440 2006).

	Steel	GFRP	CFRP	AFRP
Yield Stresses (MPa)	276 to 517	N / A	N / A	N / A
Tensile Strength (MPa)	483 to 690	483 to 1600	600 to 3690	1720 to 2540
Elastic Modulus (GPa)	200	35 to 51	120 to 580	41 to 125
Yield Strain	0.14% to 0.25%	N / A	N / A	N / A
Rupture Strain	6% to 12%	1.2% to 3.1%	0.5% to 1.7%	1.9% to 4.4%

An FRP bar cannot be bent after it's manufactured except in the case of using thermoplastic resin. Although FRP bar can be produced with bends, it is not recommended to use bent bar, due to the losses in the tensile stresses in the bend portion by about 40% to 50% compared with the tensile strength of a straight bar (ACI Committee 440 2006).

The compressive modulus of elasticity of the FRP bars is much lower than that of the tensile modulus (ACI Committee 440 2006). On the other hand, some other researchers indicated that the compressive modulus of elasticity is approximately 80% for GFRP, 85% for CFRP and 100% for AFRP of the tensile modulus of elasticity for the same product (ACI Committee 440 2006).

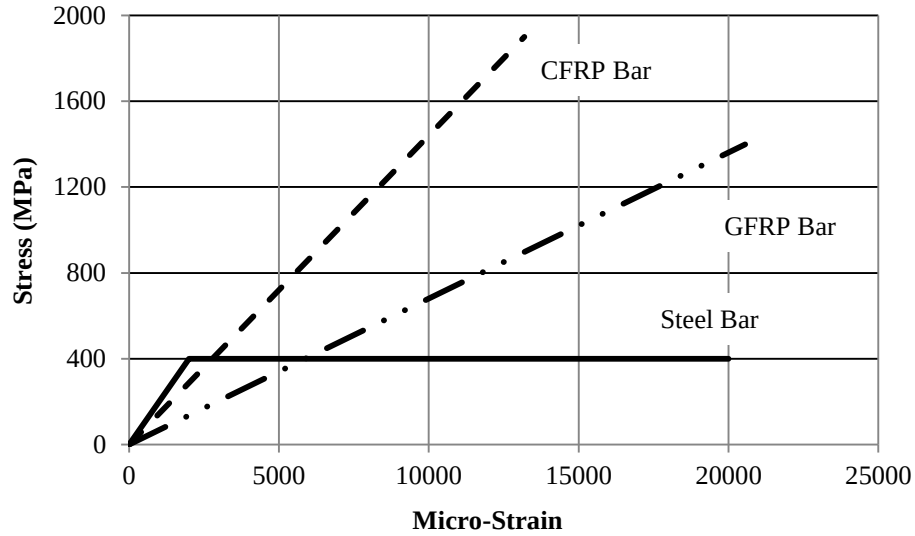


Figure 2.1: Stress-strain relationship for reinforcing bars

FRP bars could be considered as layers of resin lie between layers of fibers. The resins would cause the failure of FRP bars when facing the inter-laminar shear. To solve the problem associated with the inter-laminar shear, fibres in an off-axis direction across the bars should be oriented to increase the shear resistance, depending upon the degree of offset. This can be accomplished by braiding or winding fibres transverse to the main fibres. Also off-axis fibres can be placed in the pultrusion process by introducing a continuous strand mat in the roving/ mat creel. Bars without transverse fibres would be weak against inter-laminar shear.

Bond behaviour of FRP bars is attributed to many aspects such as surface design, manufacturing process, mechanical properties of the bar and the environmental conditions. The bond force between FRP bars and the surrounding concrete could be generated from adhesion and frictional resistance of the interface against slip. Also, a mechanical interlock due to irregularity of the interface between FRP bars and the surrounding concrete could generate bond force.

Environmental conditions such as the presence of water, acidic or saline solutions as well as the alkaline ones might have effect on the durability of FRP bars in terms of tensile capacity, bond and stiffness. The mechanical properties of FRP bars can be enhanced, deteriorated or even remain the same depending on the type of FRP bars and exposure conditions and material (ACI Committee 440 2006).

2.3 TYPES OF FAILURE OF CONCRETE FLAT PLATES

A reinforced concrete flat plate structure has to be checked against all types of failure which can be classified as (1) Flexural failure and (2) Punching shear failure.

2.3.1 Flexural Failure

Flexural failure occurs mainly due to bending moments, which induce compressive stresses in one side of the slab and tensile stresses in the opposite one. In design principles it is usually assumed that the reinforcing bars are resisting all tensile stresses while concrete is resisting all the compressive ones. Concrete sections reinforced with steel bars are commonly under-reinforced to prevent crushing of concrete before yielding of steel. The in-elastic yielding of the steel provides a ductile behaviour to the reinforced concrete members. This kind of ductile behaviour is not readily available for FRP-RC elements due to the linear-elastic behaviour of FRP materials up to failure without yielding plateau. For FRP-RC elements, deformability is the corresponding term to ductility. Deformability is a concept to quantify the deformation characteristic by providing a comparison between the level of safety between ultimate and service states (ISIS Canada 2007).

i- Tension failure

This failure occurs when the reinforcing bars in tension side reach their design failure strain before concrete reaches its ultimate compressive strain.

For steel bars, the design failure strain is defined as the yield strain. Yielding of the flexural reinforcement has a negative effect on the punching shear capacity of the slab-column connection, due to the cracks produced with it. The width and the depth of the crack have a significant influence on the shear capacity of the connection, as the shear transferred within the cracked areas will be small. Also, this failure gives enough warning before it takes place, which makes codes, such as the Canadian standard (CSA/A23.3-14), allow this type of failure.

For FRP bars, the design failure strain is defined as the rupture strain. The FRP is an elastic material up to failure. If the rupture of FRP bars takes place before crushing of concrete, extensive cracking and significant deflection could be expected due to the large rupture strain of the FRP bars. These cracks would reduce the effective cross section area that resists the punching shear capacity; and, subsequently, sudden and very brittle failure may occur.

ii- Balanced failure

This failure occurs when the concrete reaches its ultimate compressive strain at the same time that reinforcing bars in tension side reach their design failure strain. This kind of failure seldom occurs in practice.

iii- Compression failure

This failure occurs when the concrete crushes while the reinforcing bars have a strain level smaller than the design failure strain.

For steel bars, the reinforcing bars will not reach their yield strain which result in small cracking and significant effective cross section areas to resist the punching shear capacity. However, this type of failure takes place without much warning, thus all design codes avoid it.

On the other hand, the Canadian code CAS/S806-12 (CSA 2012) for FRP-RC members recommends having failure due to concrete crushing before the reinforcing bars reach their ultimate strain. In this type of failure, FRP-RC members exhibit deformability due to plastic behaviour of concrete, which is preferable.

2.3.2 Punching Shear Failure

There are two shear failure mechanisms that may be encountered in flat plate systems. The first is the one-way shear similar to that in beams. This type of shear failure is normally not critical and rarely happens in flat plate systems. The second is the two-way shear in which the failure surrounds the column forming a cone shape as shown in Fig. 2.2. Normally, the stresses resulting from the two-way shear are much higher than those resulting from the one-way shear. Two-way shear failure mechanism is usually encountered in flat plate and footings. Punching shear failure occurs due to shear force in addition to unbalanced moment transfers between the slab and columns. The combination of shear and unbalanced moment is unavoidable even under gravity loads only due to many reasons, such as unsymmetrical loading. Prevention of punching failure of slab-column connections depends on an accurate calculation of shear stresses induced by shear force and unbalanced moment transfers to the columns.

Punching shear failure begins with radial cracks extending from the column followed by tangential cracks around the column region. As the load increases to the ultimate capacity, the

slab starts to fail with the column penetrating through it. In general, punching shear failure in flat plates reinforced with FRP bars is sudden and brittle as in the case of steel bars.



Figure 2.2: Punching cone (Gouda 2008)

2.4 BUILDING CODE REQUIREMENTS

In most building codes, punching shear capacity calculations are based on a critical perimeter located at a distance between $0.5 d$ and $3 d$ from the column face, where d is the effective average slab depth. The punching shear capacity depends on many factors such as concrete strength, effective depth, column aspect ratio and flexural reinforcement ratio. However, the effect of these variables is quite different in each code.

Punching shear resistance for slabs with shear reinforcement depends on the used design model. Furthermore, the stress limit in the shear reinforcement is different in each code. Therefore, the amount and extend of shear reinforcement differs considerably. Also, for the integrity reinforcement required to prevent the progressive collapse associated with the punching shear failure, there is a substantial difference regarding the amount of this reinforcement; some codes do not even consider the integrity reinforcement.

Codes provisions were derived from many tests which take into account the construction practices common in the respective countries. A design code is a complex set of interrelated rules

and guidelines that have been calibrated over time to give acceptable results. In the UK the high punching shear capacity is linked to the extent of the shear reinforcement. The comparatively high punching shear resistance without shear reinforcement provided by North American codes should be seen together with the required integrity reinforcement. Therefore engineers should be very careful in applying individual provisions from different codes in a design.

Consideration of practical aspects should be given as the safety of structures depends as well on the quality of construction – the skill of workmanship and the pressure of time and costs. This may be a point in favour of flat plate without shear reinforcement. Otherwise, shear elements prefabricated from welded fabric which reduces the problems of installation or specific systems such as, shear studs may be of advantage.

2.4.1 Canadian Code for Steel-reinforced Concrete structures (CAN/CSA-A23.3-14)

2.4.1.1 Punching shear stresses calculations

The Canadian code considers two cases in the calculation of the punching shear stresses. The first is the case of concentric loading (shear force only). The second case is due to shear force in addition to some amount of unbalanced moment that should be transferred to columns from the slab.

2.4.1.1.1 Calculation of shear stresses for concentric punching

Concentric loading on flat plate system as shown in Fig. 2.3, produces uniform factored shear stresses (V_F) that can be calculated from the following equation.

$$V_F = \frac{V}{d \times b_o} \quad [2.1]$$

Where: V = the factored shear force

b_o = the critical shear perimeter (at $d/2$ from the column face)

d = the effective flat plate depth

Practically, the case of concentric loading is rarely encountered in real structures.

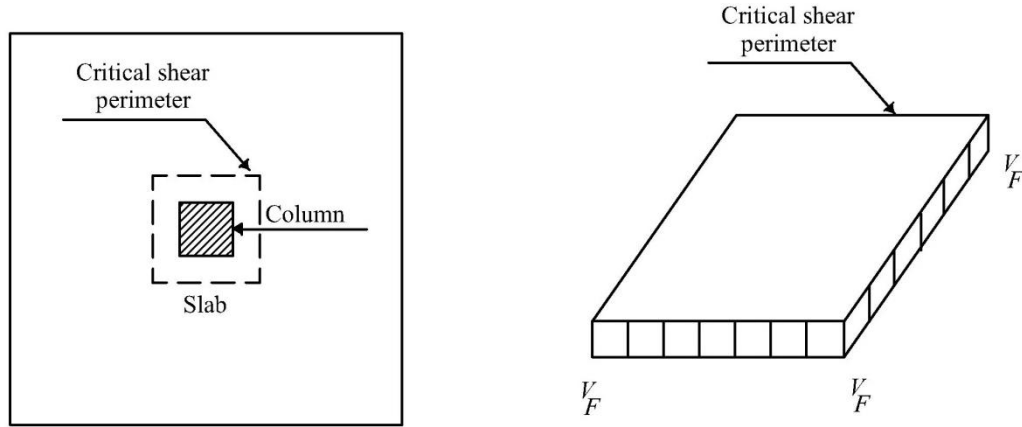


Figure 2.3: Concentric shear stress calculations for interior column

2.4.1.1.2 Calculation of shear stresses for eccentric punching

Figure 2.4 illustrates the moment and shear transfer at interior and exterior columns where a shear and an unbalanced moment are transferred from a slab to a column. A fraction of the unbalanced moment transferred by eccentricity of shear at interior, edge and corner columns, γ_v , shall be

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} \quad [2.2]$$

Where: γ_v = fraction of the unbalanced moment transferred by eccentricity

b_1 = width of the critical section for shear measured in the direction of the span

for which moments are determined.

b_2 = width of the critical section for shear measured in the direction

perpendicular to b_1 .

The moment transferred to the column by eccentric shear stress is ($\gamma_v M_f$) causes additional shear stresses that have to be combined with those resulting from the factored shear force.

The case of corner column subjected to eccentric punching stresses is illustrated in Fig. 2.5. The centroid of the critical perimeter lies closer to the interior face of the column. Hence, shear stresses due to the moment effect are larger at the outside perimeter. If the shear stresses due to transferred moments are larger than the shear stresses due to vertical loads, negative shear stresses may occur at these points. The punching shear stresses according to the Canadian code are calculated using the following equations.

The shear stresses due to factored shear force are calculated using the following equation.

$$V_F = \frac{V}{d \times b_o} \quad [2.3]$$

The shear stresses due to unbalanced moment in y direction are given by.

$$q_{y1} = \frac{M_y \times \gamma_{vy} \times C_{AB}}{J_{cy}} \quad [2.4]$$

$$q_{y2} = \frac{M_y \times \gamma_{vy} \times C_{CD}}{J_{cy}} \quad [2.5]$$

The shear stresses due to unbalanced moment in x direction are given by.

$$q_{x1} = \frac{M_x \times \gamma_{vx} \times C_{CB}}{J_{cx}} \quad [2.6]$$

$$q_{x2} = \frac{M_x \times \gamma_{vx} \times C_{AD}}{J_{cx}} \quad [2.7]$$

The total stresses at interior and edge column points equal to

$$v_f = q_A = q_B = V_F + q_{y1} \quad [2.8]$$

$$v_f = q_C = q_D = V_F - q_{y2} \quad [2.9]$$

The total stresses at corner column points equal to

$$v_f = q_A = V_F + q_{y1} - q_{x2} \quad [2.10]$$

$$v_f = q_B = V_F + q_{y1} + q_{x1} \quad [2.11]$$

$$v_f = q_C = V_F + q_{x1} - q_{y2} \quad [2.12]$$

Where: $v_f = q_*$ = total shear stress at a point (MPa)

q_{**} = shear stress at a point due to the effect of the unbalanced moment (MPa)

V_F = uniform factored shear stresses (MPa)

C^{**} = distance from the centroid of the connection to the point where the shear stress is calculated (mm)

M_x = unbalanced moment around the x-axis (kN.m)

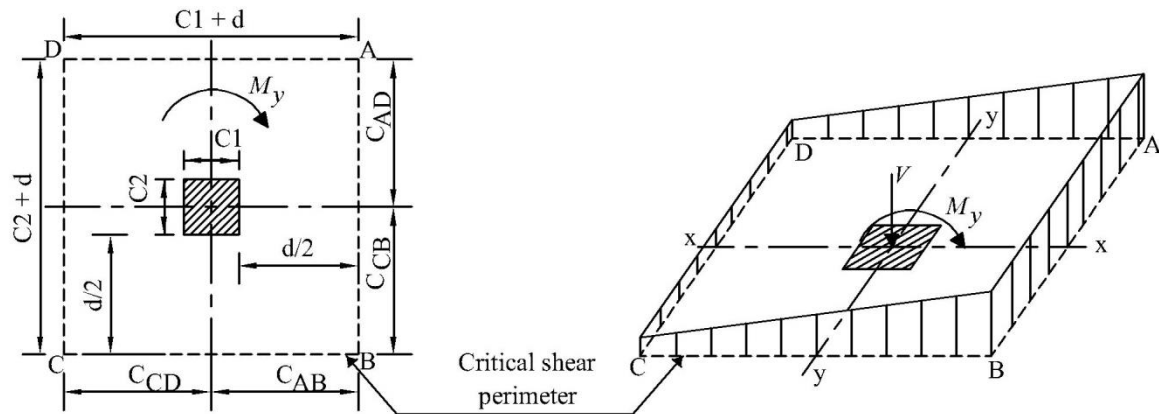
M_y = unbalanced moment around the y-axis (kN.m)

γ_{vx} = fraction of the unbalanced moment transferred by eccentricity in the direction of x-axis

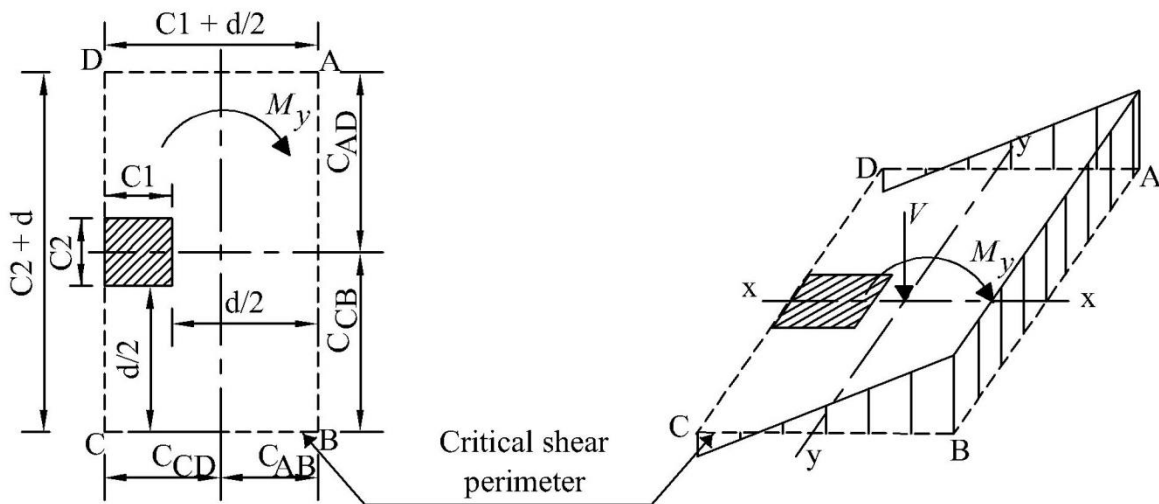
γ_{vy} = fraction of the unbalanced moment transferred by eccentricity in the direction of y-axis

J_{cx} = property of the critical shear section of slabs analogous to the polar moment of inertia along x-axis (mm⁴)

J_{cy} = property of the critical shear section of slabs analogous to the polar moment of inertia along y-axis (mm⁴)



a) Interior column



b) Edge column

Figure 2.4: Punching shear stress distribution due to shear force and unbalanced moment for interior and edge columns (reproduced from CSA/A23.3-14)

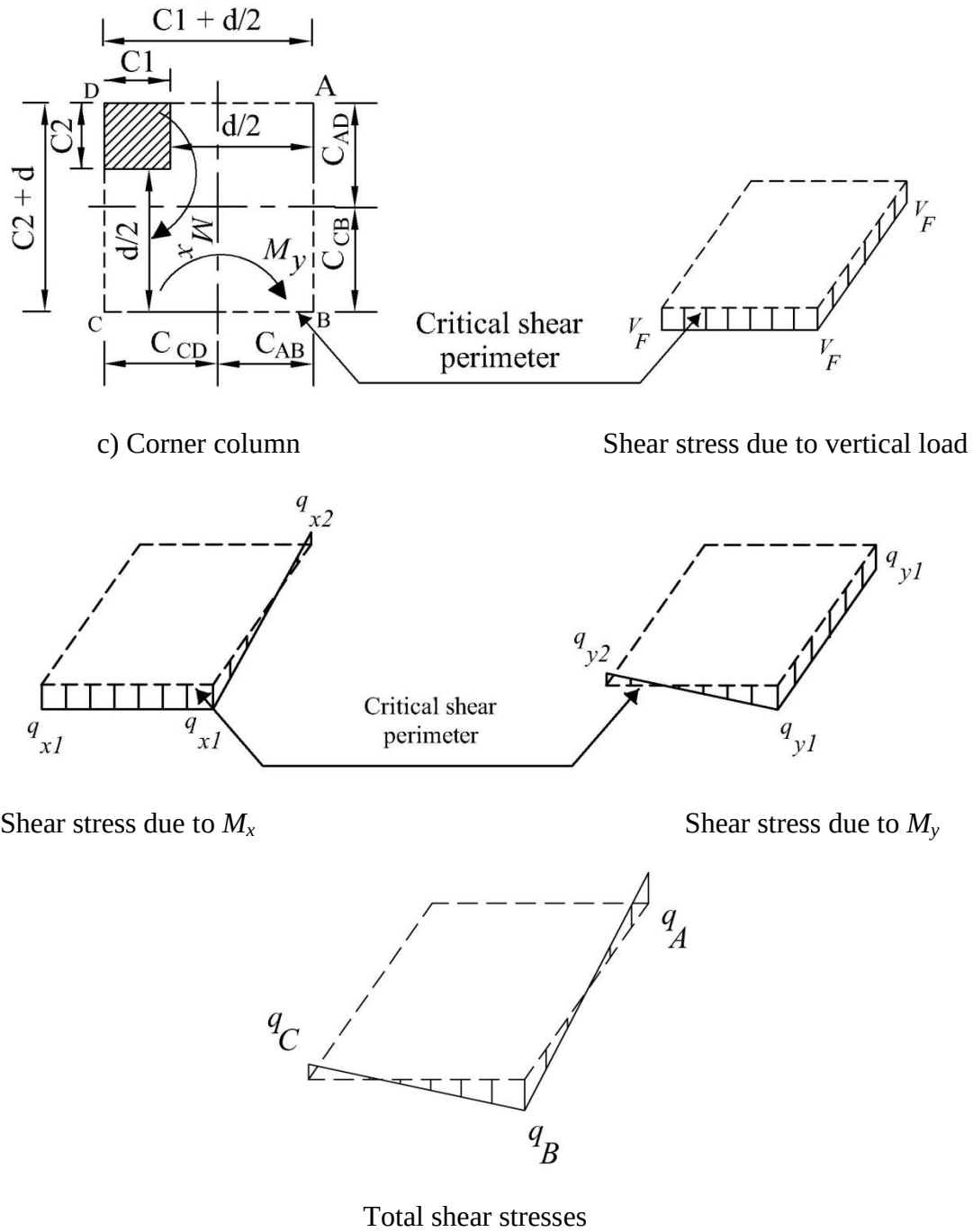


Figure 2.5: Punching shear stresses for corner column (reproduced from CSA/A23.3-14)

2.4.1.2 Calculations of the punching shear resistance of flat plate

2.4.1.2.1 Shear stress resistance without shear reinforcement

The critical section for punching shear stresses is considered at distance of $d/2$ from the column face as shown in Fig. 2.6a. Equation [2.15] was introduced as the general expression to calculate the punching shear strength of the slab-column connections. However, the available literature (ASCE-ACI Committee 426 1974) showed that Eq. [2.15] gives un-conservative predications for connections with column-aspect ratio, β_c , greater than 2.0; therefore, Eq. [2.13] was introduced. Also, the available literature (such as Vanderbilt 1972) showed that increasing b_o/d decreased the punching shear strength; as a result, Eq. [2.14] was introduced. The ultimate punching shear stress resistance for concrete (v_r) is calculated as the smallest value of these three equations:

$$v_r = v_c = \left(1 + \frac{2}{\beta_c}\right) 0.19 \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad [2.13]$$

Where: f'_c = compressive strength of concrete (MPa)

β_c = the ratio of long side to short side of the column.

λ = factor to account for low-density concrete (1.0 for normal weight concrete).

ϕ_c = resistance factor for concrete (0.65).

$$v_r = v_c = \left(0.19 + \frac{\alpha_s d}{b_o}\right) \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad [2.14]$$

Where: α_s = 4.0 for interior column

= 3.0 for edge column

= 2.0 for corner column

$$v_r = v_c = 0.38 \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad [2.15]$$

The value of $\sqrt{f'_c}$ used in the above equations shall not exceed 8 MPa, If the effective depth, d , used in two-way shear calculations exceeds 300 mm, the value of v_c obtained shall be multiplied by $1300/(1000+d)$.

2.4.1.2.2 Shear stress resistance with stud shear reinforcement

Shear studs can be used to increase the carrying load capacity and to change the mode of failure from punching to flexure of the slab-column connections (El-Salakawy et al. 1998). However, unlike European code (EN/1992-1-1 2004), shear studs are not mandatory to be used according to the Canadian standard (CSA 2014, Clause 13.3.7.1). The Canadian code required that the shear studs to be extended to a distance not less than, $2.0 d$, from the column face or to a section where the shear stress is less than the value obtained from Equation [2.16], whichever is longer (Fig. 2.6d).

$$v_r = 0.19 \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad [2.16]$$

Where v_r = the factored shear stress resistance outside the shear reinforced zone.

Furthermore, the factored shear resistance inside the shear reinforcement zone, v_r , is composed of the factored shear resistance provided by the concrete, v_c , [Eq. 2.17] and the factored shear resistance provided by the shear studs, v_s , [Eq. 2.18]. In addition, the factored shear resistance, v_r , should be less than $0.75 \lambda \phi_c \sqrt{f'_c}$.

$$v_c = 0.28 \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad [2.17]$$

$$v_s = \frac{\phi_s A_{vs} f_y}{b_o s} \quad (\text{MPa}) \quad [2.18]$$

Where: ϕ_s = resistance factor

A_{vs} = cross sectional areas of the shear studs along the critical shear perimeter (at $d/2$ from the column face)

b_o = critical shear perimeter (at $d/2$ from the column face)

S = spacing between the shear studs

f_y = the yielding strength of the shear studs

The distance between the column face and the first line of the shear studs shall not be greater than $0.4 d$ (Clause 13.3.8.6). Also, the spacing, S , between the lines of the shear studs should be based on the value of the shear stresses, v_f , calculated at $0.5 d$ from the column face (Clause 13.3.8.6).

$$S \leq 0.75d \text{ when } v_f \leq 0.56 \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad [2.19]$$

$$S \leq 0.5d \text{ when } v_f > 0.56 \lambda \phi_c \sqrt{f'_c} \quad (\text{MPa}) \quad [2.20]$$

2.4.2 ACI Code for Steel-Reinforced Concrete Structures (ACI 318-14)

2.4.2.1 Punching shear stresses calculations

The equations that calculate the punching shear stresses in the ACI 318 and CSA/A23.3 codes are similar, Eq. [2.1] through [2.12].

2.4.2.2 Calculations of the punching shear resistance of flat plates

2.4.2.2.1 Shear resistance without shear reinforcement

The critical section for punching shear stresses is considered at distance of $d/2$ from the column faces as shown in Fig. 2.6a. The nominal punching shear stress resistance for concrete (v_c) is calculated as the smallest value of the following three equations.

$$v_c = 0.083 \left(2 + \frac{4}{\beta_c}\right) \lambda \sqrt{f'_c} \quad (\text{MPa}) \quad [2.21]$$

Where: f'_c = compressive strength of concrete (MPa)

β_c = the ratio of long side to short side of the column.

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of the same compressive strength.

$$v_c = 0.083 \left(2 + \frac{\alpha_s d}{b_o}\right) \lambda \sqrt{f'_c} \quad (\text{MPa}) \quad [2.22]$$

Where α_s = 40 for interior column, 30 for edge column and 20 for corner column

b_o = the critical shear perimeter (at $d/2$ from the column face)

$$v_c = 0.33 \lambda \sqrt{f'_c} \quad (\text{MPa}) \quad [2.23]$$

2.4.2.2.2 Shear resistance with stud shear reinforcement

The shear reinforcement shall be extended to a distance where the shear stress is less than the value obtained by Equation [2.24].

$$0.17 \lambda \sqrt{f'_c} \quad (\text{MPa}) \quad [2.24]$$

Also, the contribution of the concrete for the shear resistance shall be less than the value obtained from Equation [2.25], while the corresponding one for shear studs should be calculated from Equation [2.26].

$$v_c = 0.25 \lambda \sqrt{f'_c} \quad (\text{MPa}) \quad [2.25]$$

$$v_s = \frac{\phi_s A_{vs} f_y}{b_o S} \quad (\text{MPa}) \quad [2.26]$$

$$v_c + v_s \leq 0.66 \lambda \phi \sqrt{f'_c} \quad (\text{MPa}) \quad [2.27]$$

Where ϕ = resistance factor

The distance between the column face and the first line of the shear studs shall not be greater than $0.5 d$. Also, the spacing, S , between the lines of the shear studs shall be based on the value of the shear stresses, v_f , calculated at $0.5 d$ from the column face.

$$S \leq 0.75d \quad \text{when } v_f \leq 0.5 \lambda \phi \sqrt{f'_c} \quad (\text{MPa}) \quad [2.28]$$

$$S \leq 0.5d \quad \text{when } v_f > 0.5 \lambda \phi \sqrt{f'_c} \quad (\text{MPa}) \quad [2.29]$$

2.4.3 European Code Requirements (EN/1992-1-1 2004)

2.4.3.1 Punching shear stresses calculations

The Euro code calculates the punching shear stresses from Equation [2.30] regardless the type of the connection.

$$V_{ED} = \frac{\beta \times V}{d \times U_i} \quad (\text{MPa}) \quad [2.30]$$

Where: V_{ED} = punching shear stresses (MPa). V = factored shear force (N)

d = the mean effective depth of the slab (mm), which may be taken as $(d_y + d_z)/2$

and d_y, d_z are the effective depths in the y- and z- directions of the control section

U_i = the length of the control perimeter being considered.

β is calculated from Equation [2.31]

$$\beta = 1 + \frac{K \times M \times U_1}{V \times W_1} \quad [2.31]$$

Where U_1 = the length of the basic control perimeter (2.0 d from the column faces).

k = coefficient depends on the ratio between the column dimensions C_1 and C_2 ,
as provided in Table 2.4.

Table 2.4: Values of k for rectangular loaded areas (EN/1992-1-1 2004)

C_1/C_2	$\ll 0.5$	1.0	2.0	$\gg 3.0$
k	0.45	0.6	0.7	0.8

W_1 = function of the perimeter U_i as shown in Equation [2.32]

$$W_1 = \int_0^{U_i} |e_1| \times d\ell \quad [2.32]$$

Where $d\ell$ = a length increment of the perimeter.

e_1 = the distance of $d\ell$ from the axis about which the moment M acts.

For a rectangular column:

$$W_1 = \frac{C_1^2}{2} + C_1 \times C_2 + 4 \times C_2 \times d + 16 \times d^2 + 2 \times \pi \times d \times C_1 \quad [2.33]$$

Where: C_1 = the column dimension parallel to the eccentricity of the load

C_2 = the column dimension perpendicular to the eccentricity of the load

For interior circular columns β follows from:

$$\beta = 1 + \frac{0.6 \times \pi \times e}{D_1 + 4d} \quad [2.34]$$

Where $D1$ = the diameter of the circular column

For an interior rectangular column where the loading is eccentric to both axes, the following approximate expression for β may be used:

$$\beta = 1 + 1.8 \sqrt{\left(\frac{e_y}{b_z}\right)^2 + \left(\frac{e_z}{b_y}\right)^2} \quad [2.35]$$

Where e_y and e_z = the eccentricities (M/V) along y and z axes, respectively

b_y and b_z = the dimensions of the control perimeter (see Fig. 2.6)

For edge column connections, where the eccentricity is perpendicular to the slab free edge, the punching force may be considered to be uniformly distributed along the control perimeter U_{1*} as shown in Fig. 2.7a. If there are eccentricities in both orthogonal directions, β may be determined using the following expression:

$$\beta = \frac{U_1}{U_{1*}} + K \times \frac{U_1}{W_1} \times e_{par} \quad [2.36]$$

Where e_{par} = the eccentricity parallel to the slab edge resulting from a moment about an axis perpendicular to the slab edge,

K may be determined from Table 2.4 with the ratio C_1/C_2 replaced by $C_1/2 \times C_2$,

W_1 is calculated for the basic control perimeter U_1 .

For a rectangular column, as shown in Fig. 2.7:

$$W_1 = \frac{C_2^2}{2} + C_1 \times C_2 + 4 \times C_1 \times d + 8 \times d^2 + 2 \times \pi \times d \times C_2 \quad [2.37]$$

For corner column connections, it is assumed that the punching force is uniformly distributed along the reduced control perimeter U_{1*} , as defined in Fig. 2.7b. The β -value may then be considered as:

$$\beta = \frac{U_1}{U_{1*}} \quad [2.38]$$

2.4.3.2 Calculations of the punching shear resistance of flat plates

According to the Euro code the critical section for the punching shear stresses is taken at distance of $2.0 d$ from the column faces whether the column section is rectangular or circular as shown in Fig. 2.6b. The punching shear strength ($V_{Rd,C}$) is calculated by the following equation:

$$V_{Rd,C} = C_{Rd,C} K (100 \rho_1 f_{ck})^{1/3} + k_1 \sigma_{cp} \geq (0.035 K^{3/2} f_{ck}^{1/2} + k_1 \sigma_{cp}) \quad (\text{MPa}) \quad [2.39]$$

Where $V_{Rd,C}$ is the shear resistance of concrete in MPa

f_{ck} is the characteristic concrete cylinder strength in MPa

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0 \quad d \text{ in mm} \quad [2.40]$$

$$\rho_1 = \sqrt{\rho_{1y} + \rho_{1z}} \leq 0.02 \quad [2.41]$$

$\rho_{1y} + \rho_{1z}$ relate to the bonded tension steel in y and z directions, respectively. The values ρ_{1y} and ρ_{1z} should be calculated as mean values taking into account a slab width equal to the column width plus $3.0 d$ from each side.

$$\sigma_{cp} = (\sigma_{cy} + \sigma_{cz})/2$$

Where σ_{cy} and σ_{cz} are the normal concrete stresses in the critical section in y and z directions,

$$\text{respectively, (MPa, positive if compression): } \sigma_{cy} = \frac{N_{Ed,y}}{A_{cy}} \text{ and } \sigma_{cz} = \frac{N_{Ed,z}}{A_{cz}}$$

$N_{Ed,y}$ and $N_{Ed,z}$ are the longitudinal forces across the full bay for interior columns and the longitudinal force across the control section for edge columns, respectively. The force may be from a load or prestressing action.

A_c is the area of concrete according to the definition of N_{ed} . The recommended value for the calibration factor ($C_{Rd,C}$) is $0.18/\gamma_c$ and the coefficient considering the effects of axial forces on the stress distribution (k_1) is 0.1.

2.4.4 JSCE 2007 (Japan Society of Civil Engineers 2007)

According to the Japanese standard the critical section for the punching shear stresses is taken as circular section at distance of $0.5 d$ from the column face whether the column section is rectangular or circular. The punching shear strength (V_J) is calculated by the following equations:

$$V_J = \beta_d \beta_p \beta_r \frac{f_{pcd} u_p d}{\gamma_b} \times \frac{1}{\alpha} \quad (\text{N}) \quad [2.42]$$

$$\beta_d = \sqrt[4]{\frac{1000}{d}} \leq 1.5 \quad [2.43]$$

$$\beta_p = \sqrt[3]{100 \rho} \leq 1.5 \quad [2.44]$$

$$\beta_r = 1 + \frac{1}{1 + 0.25u/d} \quad [2.45]$$

$$f_{pcd} = 0.2 \sqrt{f'_c} \leq 1.2 \quad [2.46]$$

$$\alpha = 1 + 1.5 \times \frac{e_x + e_y}{\sqrt{b_x b_y}} \quad [2.47]$$

Where: V_J = the nominal shear strength provided by concrete

β_d = coefficient to consider effect of effective depth on shear capacity

β_r = factor represents the effect of the loaded area

β_p = coefficient to consider effect of effective reinforcement ratio defined as the average values for the reinforcement in two directions

d = the effective slab depth (mm)

f_{pcd} = the design compressive strength of concrete (MPa)

u_p = the peripheral length of the design cross-section at $d/2$ from the column face (mm)

ρ = longitudinal reinforcement ratio

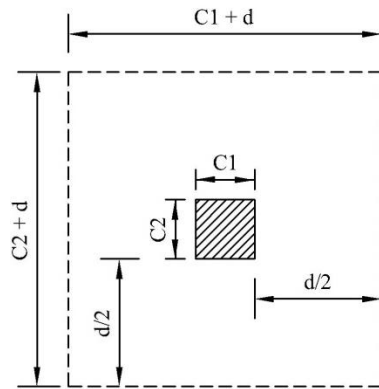
u = the peripheral length of the column (mm)

γ_b = safety factor (1.3)

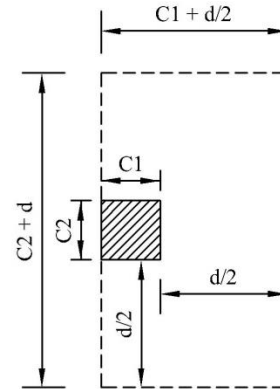
e_x and e_y = the load eccentricity in the two orthogonal directions

b_x and b_y = the dimensions of the critical section in the two orthogonal directions

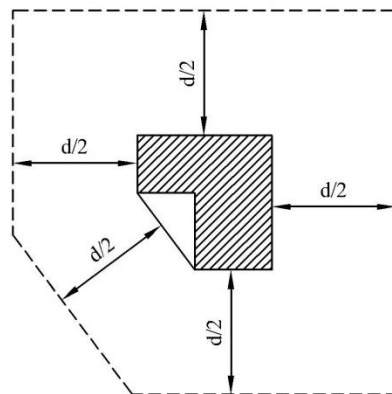
The review of the above building codes indicates that the North American codes (ACI 318 and CSA/A23.3) do not reflect the influence of the reinforcement ratio on the punching shear capacity of flat slabs. In the meantime, the European and Japanese codes address the influence of the flexural reinforcement ratio, mostly by a $1/3$ power function. Also, North American codes use the square root for the concrete compressive stress to predict the punching shear capacity of the structural members, but some researchers prefer the use of the cubic root, because it gave a good agreement with their test results as shown in section 2.5.2 of this thesis.



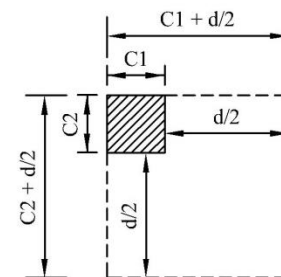
Interior column



Edge column

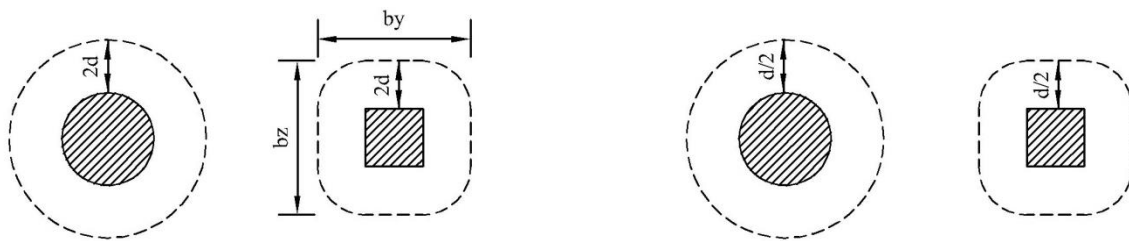


Irregular column

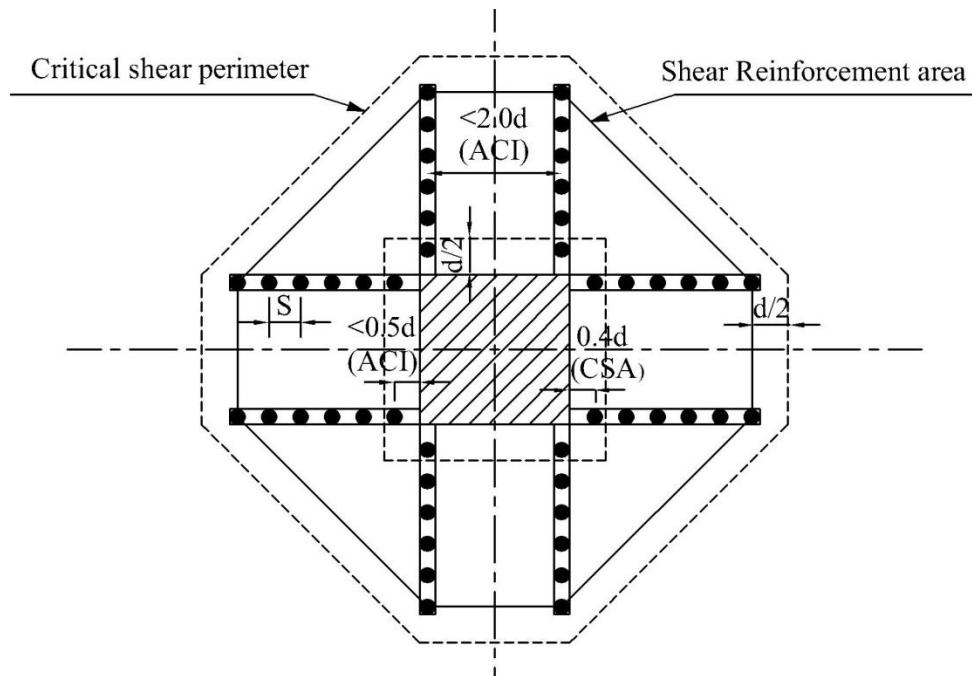


Corner column

(a) CSA and ACI (reproduced from CSA/A23.3-14 and ACI 318-14)



(b) Reproduced from EN/1992-1-1 2004 (c) Reproduced from JSCE 2007



(d) CSA and ACI for interior slab-column connections with shear studs (reproduced from CSA/A23.3-14 and ACI 318-14)

Figure 2.6: Critical sections for punching according to different codes

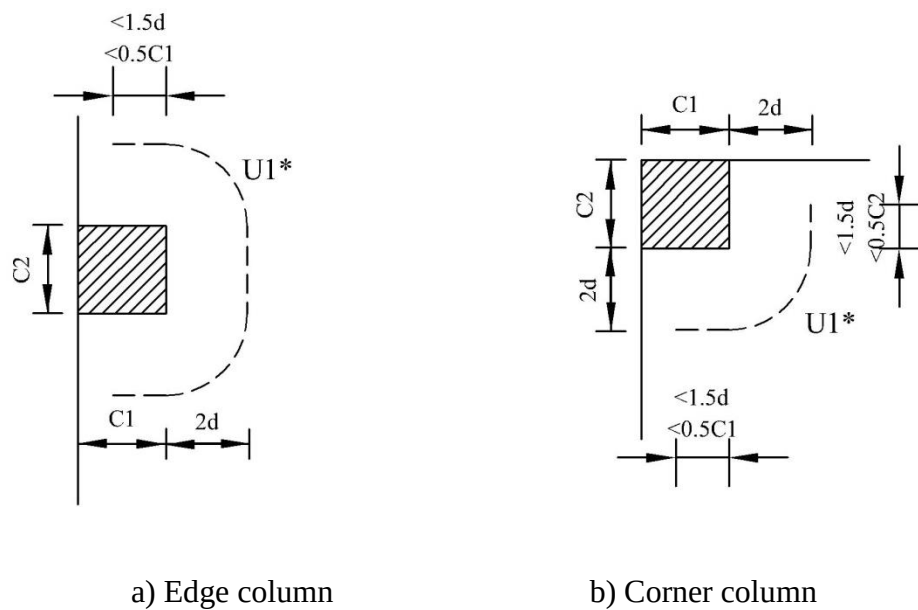


Figure 2.7: Reduced basic control perimeter U_1^* , reproduced from EN/1992-1-1 2004

2.4.5 Punching Shear Resistance of Flat Plates Reinforced with FRP Bars

According to Different Available Codes

The codes that deal with flat slabs reinforced with FRP bars are limited due to lack of data in this area.

2.4.5.1 Canadian standard CSA/S806-12

The ultimate punching shear stress resistance for concrete (v_r) is calculated as the smallest value of the following three equations.

$$v_r = v_c = \left(1 + \frac{2}{\beta_c}\right) [0.028 \lambda \phi_c (E_F \rho_F f_c')^{1/3}] \quad (\text{MPa}) \quad [2.48]$$

Where: f_c' = compressive strength of concrete (MPa)

β_c = the ratio of long side to short side of the column

λ = factor to account for concrete density

ϕ_c = resistance factor for concrete

E_F = modulus of elasticity of longitudinal FRP reinforcement (MPa)

ρ_F = longitudinal reinforcement ratio for FRP

$$v_r = v_c = \left(0.19 + \frac{\alpha_s d}{b_o}\right) [0.147 \lambda \phi_c (E_F \rho_F f_c')^{1/3}] \quad (\text{MPa}) \quad [2.49]$$

Where: $\alpha_s = 4.0$ for interior column

= 3.0 for edge column

= 2.0 for corner column

$$v_r = v_c = 0.056 \lambda \phi_c (E_F \rho_F f_c')^{1/3} \quad (\text{MPa}) \quad [2.50]$$

The punching shear equations [2.48 to 2.50] that have been used to calculate the punching shear resistance of flat plate reinforced with FRP bars are similar in the structure to Equations [2.13 to 2.15] for steel-RC slabs (CSA/A23.3-14). However, there are some differences including:

- 1- CSA/S806-12 (CSA 2012) standard takes the effect of both reinforcement ratio and elastic modulus of the FRP bars powered to one third, while CSA/A23.3-14 (CSA 2014) code did not take such effect for steel bars.
- 2- The punching shear resistance is proportional to the cubic root of the concrete compressive strength in CSA/S806-12 code (CSA 2012), while it is proportional to the square root of the compressive strength of the concrete in CSA/A23.3-14 code (CSA 2014).

It can be seen from Equations 2.48 to 2.50 that the punching shear resistance of the FRP-RC members would be less than their counterparts reinforced with steel (Equations 2.13 to 2.15).

2.4.5.2 ACI guideline (ACI 440.1 R-06)

The ACI 440.1R-06 (ACI 2006) guideline gave the following equation to calculate the punching shear capacity (V_c in Newton) of interior slab connections supported with square or circular column only.

$$V_c = \frac{4}{5} \sqrt{f'_c} b_o c \quad (\text{N}) \quad [2.51]$$

Where: f'_c = concrete compressive strength (MPa)

b_o = perimeter of critical section for slabs at $(d/2)$ from column face (mm)

c = depth of neutral axis for the cracked transformed section (mm)

The depth of neutral axis, c , can be calculated from the following equations:

$$c = kd \quad (\text{mm}) \quad [2.52]$$

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \quad [2.53]$$

Where: ρ_f = FRP reinforcement ratio

n_f = ratio of modulus of elasticity of FRP bars to modulus of elasticity of concrete

Equation [2.51] is totally different from Equations [2.21 to 2.23] that predict the punching shear capacity of flat plate reinforced with steel bars (ACI 318-14). The ACI 440.1R-06 (ACI 2006) guideline includes the depth of neutral axis which considers both reinforcement modular ratio with respect to concrete and the reinforcement ratio.

Also, it is worth to mention that an updated version of the American guideline, ACI 440.1R-15, was recently published (ACI Committee 440 2015); however the provisions to calculate the punching shear strength [Eqs 2.51 to 2.53] remained the same as in the previous version.

2.4.5.3 JSCE 1997 (Japan Society of Civil Engineers 1997)

The Japanese FRP code uses similar equations to those adapted for the steel code (JSCE 2007); however, Equations [2.54] and [2.55] should be used instead of Equations [2.42] and [2.44].

$$V_J = \beta_d \beta_p \beta_r \frac{f_{pcd} u_p d}{\gamma_b} \quad (\text{N}) \quad [2.54]$$

$$\beta_p = \sqrt[3]{100 (E_F/E_S) \rho} \leq 1.5 \quad [2.55]$$

Where E_F = modulus of elasticity of longitudinal FRP reinforcement

E_S = elastic modulus of steel = 200 GPa

2.5 RESEARCH ON STEEL-REINFORCED CONCRETE SLAB-COLUMN CONNECTIONS

The following section presents a summary of some factors that affect the behaviour of steel-RC slab-column connections.

2.5.1 Effect of Connection Geometry

Extensive research has been carried out to study the effect of the following parameters on the punching shear behaviour.

- 1- Column aspect ratio
- 2- Critical shear perimeter-to-depths ratio ($\Sigma o / d$)
- 3- Span-to-depth ratio (L/d) of the slab

2.5.1.1 Effect of column aspect ratio

Sherif et al. (2005) tested 5 half-scale edge slab-column connections with overall dimensions of 1000-mm long $\times$ 1200-mm wide $\times$ 120-mm deep. One of the investigated parameters was the column aspect ratio. It varied between 1.0 and 2.5. The authors compared the obtained results to test results from literature as well as to ACI 318-02 code (ACI Committee 318 2002) and concluded the following.

As the column aspect ratio increased from 1.0 to 2.5, the ductility (deflection at the failure load divided by the deflection at yielding) of the connection increased from 1.13 to 1.53 (approximately 26%). Based on the literature, it was also found that ACI code equations overestimated the punching load capacity for the interior slab-column connections as its rectangularity goes above 3, while it is in a good agreement with the edge slab-column connections regardless the value of the column aspect ratio.

2.5.1.2 Effect of critical shear perimeter-to-depth ratio

As the column cross section increases, the perimeter that resists the shear force increases, which leads to higher resistance. Marzouk and Hussein (1992) tested two identical steel-RC interior slab-column connections under concentric load. The only different between the two

specimens was the side length of the square central column (220 and 300 mm). Increasing the perimeter-to-depth from 17.2 to 20.6 (20%) resulted in 10% increase in the punching shear capacity.

Also, Sherif et al. (2005) investigated the behaviour of 5 half-scale edge slab-column connections with perimeter-to-depth ratio ranged from 6.5 to 11.0. In addition, test results from literature were presented to discuss the effect of perimeter-to-depth ratio on the behaviour of other types of connections. Also, the test results were compared with the predications of the ACI 318-02 code (ACI 2002) and the following was concluded.

- 1- A decrease in the failure punching shear stress was found when the $\Sigma o/d$ ratio was increased.
- 2- For a 20% decrease in $\Sigma o/d$ ratio (from 11.0 to 8.75), the ductility of the connection decreased by approximately 26%. Also, no yielding was observed at all in the tension reinforcement when the ratio ($\Sigma o/d$) reached 6.5, indicating an extreme brittle failure.
- 3- Based on literature, it was found that for interior slab-column connections with $\Sigma o/d$ ratios greater than 17, the ACI 318-02 code (ACI Committee 318 2002) provisions overestimated the punching strength. However, the code was conservative for the edge slab-column connections regardless the value of the $\Sigma o/d$.

2.5.1.3 Effect of span-to-depth ratio

Lovrovich and McLean (1990) studied ten circular slabs including five with shear reinforcement. The slab thickness was kept constant at 100 mm. The support diameter varied to produce different span-to-depth ratio of 2, 4, 6, 8 and 12.

The following points were concluded from the test results:

- 1- For reinforced concrete slabs of constant thickness, the punching shear strength increased

as the span-to-depth ratio decreased below 6.0. The strength might be increased due to the development of compression struts forming an arch mechanism in the slabs and in-plane compressive forces resulting from friction at the support.

- 2- The addition of shear reinforcement had a significant effect on the punching shear strength of the slab specimens. The specimens with shear reinforcement showed more ductile behaviour than those specimens without shear reinforcement. Adequate anchorage of the shear reinforcement greatly enhanced the shear strength of the specimens with large span-to-depth ratios. However, shear reinforcement was less effective in the specimens with a span to depth ratio of 2.0 due to steeper failure cracks.
- 3- Shear cracking initiated at lower loads for the specimens with large span-to-depth ratios. The estimated shear cracking load was approximately the same for the specimens with constant span-to-depth ratios.
- 4- The observed punching shear strengths were much greater than the values permitted by the ACI 318-83 code (ACI Committee 318 1983). This was especially true for the specimens with either shear reinforcement or small span-to-depth ratios.

2.5.2 Effect of Concrete Compressive Strength

Marzouk et al. (1998) tested six slabs with compressive strength varying from 35 to 75 MPa; the test result showed 7 to 15% increase in the shear strength when the concrete compressive strength was increased by 114%. Also, it was reported that the ACI 318-95 code (ACI Committee 318 1995) underestimated the punching shear capacity for connections made of high strength concrete by 20 to 25%.

Gardner (1990) studied 30 circular specimens with different compressive strength ranged from 14 to 56 MPa and different flexural reinforcement ratios ranged from 0.5 to 5.0%. Also, the test

results were compared with different codes. It was concluded that the cubic root relationship between shear strength and concrete strength gave more accurate results than the square root relationship currently used by ACI 318-83 (ACI Committee 318 1983) and CSA-A23.3-M84) codes. Marzouk and Hussein (1992) confirmed the results of Gardner. According to Marzouk and Hussein (1992) test results, using the cubic root of the concrete compressive strength to predict the punching resistance of the slab-column connections generally yielded better results than the square root expression used in North American codes.

2.5.3 Effect of Concrete Tensile Strength

Punching shear strength of slab-column connections is expected to increase by increasing the tensile strength of concrete. Mcharg et al. (2000) studied six interior slab-column specimens that contained steel fiber reinforced concrete (SFRC), divided into three series. Series 1 had no fiber, while the second and the third contained 0.5% fiber volume. In Series 2, the fiber was around the column up to a distance of 3.3 times the slab thickness from the column faces and over the entire depth of the slab. In addition, for Series 3 the fiber was over the entire area of the slab and placed only in the top of the slab over a thickness equal to the top cover plus one bar diameter. The test results showed the following.

- 1- The increase in punching shear resistance due to the presence of SFRC in the slab around the column was 38%.
- 2- Providing SFRC cover resulted in an increase in punching shear resistance and better crack control. The crack widths of the specimens were 25 and 20% smaller inside and outside the immediate column region, respectively.
- 3- Providing steel fiber volume of 0.5% enhanced the overall behaviour of the specimens in terms of ductility, strain, greater post-cracking stiffness and smaller crack widths at

service load level.

Yang et al. (2010) studied four interior slab-column specimens. The connections consisted of $2300 \times 2300 \times 150$ mm flat plate with 225-mm square column stubs extended 300 mm above and below the slab. One of the test parameter was using SFRC (0.5% by volume) in the slab around the column region (up to distance from the column faces equal to 3.33 times the slab thickness). It was concluded that the presence of steel fibers led to 51% increase in the punching shear capacity. Also, using steel fibers improved crack control due to the ability of the steel fibers to transfer tension across cracks.

2.5.4 Shear Reinforcement

To avoid adding drop panels, beams or column heads, shear capacity of slab may be increased by using shear reinforcement. Transverse reinforcement can be classified in the following three categories.

- 1- Stirrups and bent bars.
- 2- Shear studs.
- 3- Shear heads.

2.5.4.1 Effect of stirrups and bent bars

Broms (2000) studied seven specimens, all of them had the same dimensions and approximately the same flexural capacity but with different reinforcement. The main purpose of the study was to eliminate the punching failure mode of flat plate. According to the authors, this could happen by describing a reinforcement system that allows a flat plate to form plastic hinges at the columns and in the mid spans with no tendency for brittle punching failure. The described bent bar and stirrup combination is easy to fabricate and install in a stable way. For a typical flat plate

structure, eight stirrup cages at each column are normally sufficient. The extra cost (including labor cost) as compared to a conventional brittle flat plate without any shear reinforcement is assessed at less than 1.5% of the total cost for the slab.

Park et al. (2007) tested 8 interior slab-column specimens. Four of these specimens contained lattice bars. The lattice reinforcement was composed of top and bottom bars and inclined web bars that are weld-connected to each other. The inclined web bars acted as shear reinforcement. This lattice reinforcement, which was composed of the welded web bars and longitudinal bars, could work as a truss system. The 4 specimens that contained the lattice reinforcement did not have any other type of reinforcement, while the other 4 specimens contained the conventional flexural steel bars. The specimens consisted of a central stub column and a square slab with a length of 1800 mm. The thickness of the specimens was 120 or 160 mm. The column stub height was 400 mm above the slab and 100-mm below it with different cross section dimensions. The test results showed that the punching shear strength and deformation capacity of the specimens reinforced with the lattice bars was increased by 1.4 and 9.2 times those of the unreinforced specimens, respectively.

2.5.4.2 Effect of shear stud reinforcements

Shear studs in form of round bars, fully anchored at the top and bottom by steel plates, are getting more and more popular because of their ease of placement and reasonable cost. El-Salakawy et al. (1998) studied eight large-scale reinforced concrete flat plate edge connections. The tests were designed to study the effect of high moment-to-shear ratio on the punching shear behaviour of slab-column edge connections. One of the test perimeters was the shear reinforcement. In the specimens with shear reinforcement, the shear studs were arranged in six strips around the column, two at each side.

The following was concluded from the test results:

- 1- For the specimens with shear reinforcement, the change in failure mechanism due to higher moment was observed (from a ductile flexural failure in specimens subjected to $(M/V) = 0.3$ to a more brittle punching-flexural failure in specimens subjected to $(M/V) = 0.66$. This was due to the higher shear stresses $(M/V) = 0.66$), which exceeded the punching shear capacity of the slab outside the shear-reinforced zone.
- 2- Shear studs increased the stiffness of the connections (smaller deflections at service loads).
- 3- Shear studs increased the capacity and the ductility of the connections.

Mokhtar et al. (1985) tested eight full-size interior slab-column connections subjected to concentric loads. One specimen had no shear reinforcement, while the remaining seven were reinforced against punching shear by shear studs. The test results confirmed that using shear stud reinforcement greatly increased the strength and the ductility of the slab-column connection. The enhancement in the shear resistance of the slabs varied between 24 and 43% with respect to the slab without shear stud reinforcement and depending on the configuration used for the shear studs in each specimen. In addition to that Elgabry and Ghali (1987) tested five full-scale reinforced concrete flat-plate connections with interior columns subjected to moment transfer. One specimen had no shear reinforcement and the remaining four contained various arrangements of stud-shear. The enhancement in the shear strength of the specimens with shear studs varied between 0 and 200% depended on the configuration used for the shear reinforcement.

2.5.4.3 Effect of shear heads

Marzouk and Jiang (1997) tested two slabs; one without shear reinforcement and the other with T-headed shear reinforcement. The slab, with T-headed shear reinforcement, exhibited 15%

increase in the ultimate load capacity compared to the reference slab without shear reinforcement. In addition, it was found that punching shear failure can be controlled, eliminated and transformed into flexural failure for high-strength concrete plates by appropriately using of T-headed shear reinforcement with the flexural reinforcement. In addition, the test results were compared with the limits specified by ACI 318-89 code (ACI Committee 318 1989) and it was found that the code values are adequate for high strength concrete plates reinforced with shear reinforcement.

Rizk et al. (2011) tested five interior square slab-column connections with thickness varied between 300 and 400 mm and side dimension of 2650 mm. One of the specimens served as control, while, the others contained T-headed shear reinforcement (shear reinforcement ratio of approximately 0.68%). A concentric load was applied on the slabs through 400×400 mm column stubs. The test results showed the following.

- 1- The punching failure mode changed to a ductile flexure failure due to the presence of the shear reinforcement with a reinforcement ratio of approximately 0.68%.
- 2- The structural behaviour of the specimens could be enhanced by adding shear reinforcement.

2.5.5 Effect of Flexural Reinforcement

Flexural steel reinforcement has significant effect on the punching shear capacity of flat plate system; this effect can be summarized in the following three points.

- 1- Effect of reinforcement ratio,
- 2- Effect of reinforcement arrangement,
- 3- Effect of yield strength of the flexural reinforcement.

2.5.5.1 Effect of reinforcement ratio

Much research has been conducted to study the effect of steel reinforcement ratio on the punching shear strength of the slab-column connections. In general, it has been found that as the steel reinforcement ratio increased the stresses along the punching cone and the load carrying capacity of slab-column connections also increased. Ozden et al. (2006) studied 26 half-scale reinforced concrete flat plates. The slab plates had 1500-mm diameter circular geometry and 120-mm thickness with 200-mm square central column extended above and below the slab. The specimens were supported along the perimeter by uniformly spaced tie rods, placed 600-mm away from the centre of the column stub. One of the test parameters was the reinforcement ratio, which varied between 0.73 and 2.25%. The test results showed that increasing the plate flexure reinforcement ratio resulted in an increase in the punching shear capacity of the plate, but the rate of the enhancement in load carrying capacity was far lower than the increase in the flexural reinforcement ratio. For instance, 10 and 20% increase in the punching capacity was reported for 50% increase in the flexural capacity in normal and high strength concrete, respectively. Also, the test results were compared with the CSA/A23.3-04 (CSA 2004) and it was found that the code punching shear capacity predictions provided conservative estimates for concentrically loaded slabs and good predictions for eccentrically loaded slabs.

Vanderbilt (1972) tested 15 slab-column connections with slab thickness of 51 mm. The test parameters included the effect of reinforcement ratio. Based on the test results, it was found that the stresses at punching increased with increasing the reinforcement ratio. Also, the average increase in the punching shear stress resulted from doubling the reinforcement ratio (from 1.3 to 2.6%) was 22%.

Marzouk and Hussein (2002) found similar results, where 17 interior slab-column connections were tested. One of the test parameters was the slab reinforcement ratio ranging between 0.6 to 2.4% with slab thickness of 120 and 150 mm and a concrete compressive strength of 75 MPa. The test results showed a total increase of 63% in the punching capacity for the 150-mm thick slab when the reinforcement ratio was increased by threefold. For the 120-mm thick slab, the increase was 78% with an increase in reinforcement ratio from 0.9 to 2.3%.

Richart (1948) investigated 128 test specimens, where footings supported on a bed of springs were tested. The slab thickness varied between 250 to 460 mm. Based on the test results and an extensive mathematical work to reach to the best power function for the reinforcement ratio that could fit the experimental results, a recommendation has been made for using the one third power function for the reinforcement ratio. In addition, other researchers obtained similar results (Regan and Bræstrup 1985; Gardner 1990; Takahashi et al. 1992; Sherif and Dilger 2000; Dilger 2000).

2.5.5.2 Effect of reinforcement arrangement

It is well-established that concentrating the flexural reinforcement inside the slab in the area around the column can be used to control the width and depth of cracks, which can be used also to enhance the punching shear capacity of the slab. On the other hand, it appears from the available literature that concentrating the flexural reinforcement in the vicinity of the column seems to lead to a small increase in the punching shear strength only if the reduced bar spacing does not lead to a reduction in the bond strength along the bars. Moe (1961) investigated the effect of concentrated the flexural reinforcement within a narrow band around the column through two series of tests. In the test, the slab reinforcement ratio was kept constant. The test result showed no effect from concentrating the flexural reinforcement on the punching shear

capacity. Elstner and Hognestad (1956), Regan (1986) and Hawkins et al. (1974) reached a similar conclusion. Alexander and Simmonds (1992) attributed these test results to bond failure. Mcharg et al. (2000) supported the explanation of Alexander and Simmonds (1992); where 14% increase in the punching strength of the connection was found when the flexural reinforcement is banded in the vicinity of the column.

2.5.5.3 Effect of yield strength of reinforcement

Sherif and Dilger (1996) studied the effect of yield strength of the reinforcement on the punching shear capacity of slab-column connections based on the work reported in literature (Moe 1961; Base 1968). The authors concluded that the yield strength of the reinforcement has no apparent influence on the punching resistance. On the other side, Hawkins and Mitchell (1979) reported that yielding of the reinforcement bars has a significant effect on the punching shear capacity of the slab-column connections. The authors attributed that to the loss of strength due to loss of membrane action around the connection as result for the yielding of the reinforcement. Also, if the slab steel bars yield significantly, then it might be difficult to differentiate between punching and flexural failures.

2.6 RESEARCHES ON FRP AS REINFORCEMENT FOR SLAB-COLUMN CONNECTIONS

Due to the different in properties between steel and FRP bars a significant amount of research has been conducted to study the behaviour of concrete structures reinforced with FRP bars. However, not much was carried out on FRP-RC slab-column connections. In the following section a discussion will be offered to evaluate the different parameters that affect the behaviour of the FRP bars as internal reinforcements in the concrete members, depending on the available research in the literature.

2.6.1 Effect of FRP Flexural Reinforcement Ratio

The FRP bars have relatively low modulus of elasticity compared to the conventional steel bars, as a result, the shear strength of the slabs reinforcement with FRP bars are expected to be lower, especially after cracking, due to the wider cracks and reduced depth to neutral axis. In addition, due to the high strength of FRP bars, the behaviour of FRP-RC slab is controlled primarily by shear rather than flexure. Ospina et al. (2003) tested three interior slab-column connections with overall dimensions of $2150 \times 2150 \times 155$ mm with column stub extended above and below the slab for 250 mm and square cross section with side length of 250 mm. One of the slabs was reinforced with 0.73% conventional steel bars as reference. The other two were reinforced with 0.73 and 1.46% GFRP bars. The test results showed that for the slabs reinforced with GFRP bars, an increase in the tension mat stiffness led to a stiffer response in the elastic-cracked stage. The slab with reinforcement ratio of 1.46% failed at a higher load of approximately 125% with respect to the one with lower reinforcement ratio but the strength gain is not linearly related to the stiffness increase. Also, the deflection at ultimate reduced as the stiffness of tension mat increased. In addition, the test results were compared with different codes and the authors concluded that the design provisions in ACI 318-99 (ACI Committee 318 1999) and BS 8110-97 (British Standard 1997) cannot be used directly to predict the punching capacity of the slabs reinforced with FRP reinforcing bars. Neither of these standards adequately accounts for the elastic stiffness of the reinforcing mat. Dulude et al. (2010) found similar results, where 40% increase in the load carrying capacity was found when the reinforcement ratio was increased from 0.34 to 0.73% in GFRP-RC interior slab-column connections.

Hussein et al. (2004) studied four interior slab-column connections with different reinforcement ratios varying between 1.05 and 1.67%. The test results showed 10% increase in the ultimate

load for the slab with 1.67% reinforcement ratio with respect to the one with 1.05%. However, this percentage was 14% for the slab with reinforcement ratio of 1.18% compared to the one with 1.05%.

Matthys and Taerwe (2000) investigated the effect of different reinforcement ratios by testing 17 square slabs with a side length of 1000 mm and a total slab depth of 120 or 150 mm. The test specimens were subdivided in three series. The first series included steel-RC reference slabs (series *R*). The second series contained different type of carbon FRP (CFRP) grids (series *C/CS*). The third series included a hybrid type of FRP comprising both glass and carbon. The test results were compared with different codes such as Euro code (EN/1992-1-1 2004), the Japanese Society of Civil Engineers – JSCE (JSCE 1997) and ACI 318-95 (ACI Committee 318 1995) and the following was concluded:

- 1- For FRP-RC slabs with similar flexural strength as the steel-RC reference slabs, the obtained punching load and stiffness in the cracked state were considerably less. However, for the FRP-reinforced slabs with an increased reinforcement ratio or an increased slab depth (flexural stiffness in the fully cracked state comparable to the reference slabs), the behaviour of the slabs were comparable to the steel-reinforced reference slabs.
- 2- The calculation of the mean punching failure load according to empirical-based expressions (such as most codes equations) gave fairly good predictions but with an underestimation for the slabs with FRP reinforcement (low modulus of elasticity). The latter aspect was solved by introduced the equivalent reinforcement ratio ($\rho \cdot E_f/E_s$)
- 3- Evaluation of the design punching capacity according to the codes equations, taking into account the modification ($\rho \cdot E_f/E_s$), showed sufficient safety (mean global safety factor of about 1.9 to 2.6) for all investigated codes.

Zaghloul (2002) studied the effect of applying unbalanced moment in addition to shear force on the punching shear capacity of interior slab-column connections reinforced with CFRP grids. The program consisted of six half-scale square slabs with 1760 mm side length and two different thicknesses, 100 and 125 mm. One of the test parameters was the effect of reinforcement ratio. The test results showed 18% increase in punching shear capacity when the reinforcement ratio was increased from 0.87 to 1.33%. Also, based on the test results, the following equation was presented for the predication of the punching shear resistance of the slab-column connections.

$$V_c = 0.07 \lambda \phi_c (f'_c \rho_F E_F)^{0.333} \quad (\text{MPa}) \quad [2.56]$$

Where, E_{FRP} and ρ are the slab reinforcement elastic modulus (MPa) and ratio, respectively, λ is concrete density factor (1 for normal weight concrete) and ϕ_c is concrete resistance factor (0.6). The equation was found to be in a good agreement with the results. Also, the mean value of the test-to-predicated capacities for the six FRP-RC connections was 1.02 with a standard deviation of 0.12.

Ospina (2005) conducted extensive mathematical work based on the experimental results from the literature to product a modification for the ACI 318-05 code (ACI Committee 318 2005) equation to estimate the punching shear capacity (V_c) of two way slabs reinforced with FRP bars or steel reinforcement. The equation takes the effect of the reinforcement ratios as shown.

$$V_c = N \sqrt{f'_c} b_o k d \quad (\text{N}) \quad [2.57]$$

Where N is a constant taken equal to 5/6 for f'_c in MPa, b_o in mm and d in mm. The critical section perimeter (b_o) is calculated as in ACI 318-05 code (ACI Committee 318 2005). The term (k) is given by the next equation.

$$K = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \quad [2.58]$$

Where the modular ratio n_f is equal to E_F/E_c , where E_F is the modulus of elasticity of the longitudinal reinforcement and E_c is the modulus of elasticity of the concrete. The term (ρ_f) is the reinforcement ratio. The equation renders an average test-to-predicted capacity ratio of 2.79 with standard deviation of 0.612 and coefficient of variation (COV) of 22.0% for the 27 slab tests considered. This COV is better than that associated with ACI 318-05 code (ACI Committee 318 2005) predictions. It is worth noting that punching capacity estimates were very conservative for the entire range of FRP reinforcement ratios considered, this indicates that the confinement effect on the punching capacity of slabs with FRP is underestimated if $N = 5/6$ is assumed. A detailed examination of confinement conditions for the slab soffit concrete around the slab-column joint falls outside the scope of this article. However, experimental evidence suggests that punching capacity predictions for slabs with FRP could be significantly improved by assuming $N = 5/4$, this assumption did not alter the resulting coefficient of variation. Also, it is worth to mention that equation [2.57] was adopted by ACI 440.1R-06 guidelines (ACI Committee 440 2006).

Another effort was made by Theodorakopoulos and Swamy (2007) to produce a model to determine the punching shear strength of slabs reinforced with embedded FRP bars or grids. This model incorporates all the relevant engineering properties of FRP materials and it does not contain any empirical coefficients to fit test results except for a quantitative assessment of the bond behaviour between FRP reinforcement and concrete. The analytical procedure to determine the punching shear strength of FRP slabs consists of the following steps:

- 1- Calculate the limiting shear stress of concrete, v_c , from the following equation in terms of $f_{cu} = f_c' / 0.80$ if the specified cylinder or $f_{cu} = f_{cm} / 0.80$ if the mean cylinder concrete strength is given.

$$v_c = 0.27 f_{cu}^{2/3} \quad (\text{MPa}) \quad [2.59]$$

- 2- Determine the critical perimeter (b_p) according to BS 8110-97, where (c) is the diameter or side width of the column stub and (d) is the effective depth of the slab.

$$b_p = 4c + 12d \quad (\text{mm}) \quad [2.60]$$

- 3- Calculate the depth of the shear section X_s from the following.

$$X_s = 0.25d \quad (\text{mm}) \quad [2.61]$$

- 4- a) Calculate the balanced ratio, ρ_{fb} , from the following formula, under perfect bond condition.

$$\rho_{fb} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \cdot \frac{k_1 f_{cu}}{f_{fu}} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \cdot \frac{k_1 f_{cu}}{E_f \cdot \varepsilon_{fu}} \quad [2.62]$$

Where: E_f , f_f , and ε_f are the modulus of elasticity, stress and strain of the FRP reinforcement, respectively. In addition, $\varepsilon_{cu} = 0.0035$ is the specified value of concrete strain, whereas k_2 is the maximum ratio of the average compressive stress to the characteristic cube concrete strength f_{cu} , corresponding to the concrete strain ε_{cu} . For the case considered, because $\rho_f \geq \rho_{fb}$, ε_{cu} will always be reached and therefore $k_2 f_{cu}$ is applicable.

- b) Estimate the FRP actual strain, ε_f , from following formula assuming bond-slip condition

$$\frac{\varepsilon_f}{\varepsilon_{fu}} = 0.55 \left[\frac{\left(\frac{-\varepsilon_{cu}}{\varepsilon_{fu}} \right) + \sqrt{\left(\frac{\varepsilon_{cu}}{\varepsilon_{fu}} \right)^2 + 4(1 + \varepsilon_{cu}/\varepsilon_{fu})/(\rho_f/\rho_{fb})}}{2} \right] \leq 1 \quad \text{for } \frac{\rho_f}{\rho_{fu}} \leq 0.33 \quad [2.63]$$

- c) Calculate the depth of the flexural section $(X_f)_f$ from

$$\frac{(X_f)_f}{d} = \frac{\rho_f \cdot E_f}{K_2 \cdot f_{cu}} \epsilon_f = \frac{\rho_f \cdot f_{fu}}{k_2 \cdot f_{cu}} \cdot \frac{\epsilon_f}{\epsilon_{fu}} \text{ for } \frac{\rho_f}{\rho_{fu}} \leq 0.33 \quad [2.64]$$

5- Determine the combined depth of the compression zone $(X)_f$ from

$$(X)_f = \frac{2X_s \cdot (X_f)_f}{X_s + (X_f)_f} \quad [2.65]$$

6- Calculate the punching shear capacity of an FRP slab (V_c) from

$$V_c = v_c \cdot b_p \cdot (X)_f \cdot \cot \theta \quad [2.66]$$

Where: $\theta = 30$ degrees as the mean angle of the failure surface. ($\cot \theta = \cot 30$ degrees = 1.732)

The analytical model was compared with the experimental results from the available data in the literature and the following was concluded:

- 1- The calculated punching failure loads of FRP-RC slabs were found to be in excellent agreement with those reported from experiments of various investigators. The slabs analyzed cover many variables that influence shear behaviour. It was also found that any differences between specified and test characteristics of the FRP reinforcement was not a concern with reference to the predicted punching load.
- 2- The proposed analytical model employed no fitting factors to match the trend of the available FRP-reinforced slab test results. Given the excellent agreement between predicted and test loads, it was concluded that the proposed model provided a convenient and reliable framework for the punching strength analysis of slabs reinforced with FRP bars or grids.

2.6.2 Effect of FRP Flexural Reinforcement Arrangement

Lee et al. (2009) studied the effects of concentrating the reinforcement in the immediate column region. Five interior slab-column connections were tested. Each connection consisted of slab

with total dimensions of $2300 \times 2300 \times 150$ mm and column cross section equal to 225×225 mm extended 300 mm above and below the slab. Two of the specimens were reinforced with ordinary steel reinforcement as control specimens. The first had steel reinforcement ratio of 1% uniformly distributed. The second had concentrated steel arrangement in the area around the column within a distance of 1.5 times the slab thickness from the column faces with reinforcement ratio of 2%. In addition, the other three specimens reinforced with GFRP bars, two of them had the same reinforcement configuration as the slabs with the steel bars, while the third one had reinforcement ratio of 3% concentrated around the column area.

The test results showed that:

- 1- Due to the lower elastic modulus of GFRP bars, the GFRP-reinforced slabs had significantly lower punching shear capacities, lower post-cracking stiffness and greater deflections than slabs reinforced with steel reinforcing bars. Punching shear capacities of the connections reinforced with GFRP bars were 22 to 26% lower than those of companion connections, reinforced with steel bars. The GFRP-reinforced specimens produced more cracks in the immediate column region than the specimens reinforced with steel reinforcing bars.
- 2- Concentrating the flexural reinforcement within a distance 1.5 times the slab thickness from the column faces resulted in slightly higher punching shear strength compared to the companion specimen with a uniform distribution of the same amount of reinforcement. Also, a greater post-cracking stiffness, more uniform distribution of strains in the tension flexural bars and better crack control could be expected. The increase in punching shear resistance due to the banded distribution of the reinforcement was 5% for the steel reinforcement specimens and 11% for the GFRP ones. However, excessive concentrations

of the slab reinforcement seemed to be ineffective in increasing the punching resistance of GFRP-RC slab-column connections.

- 3- The predictions of the ACI 440.1R-06 guidelines (ACI Committee 440 2006) were very conservative compared to the test results.

2.6.3 Effect of Using FRP Shear Reinforcement

Shear reinforcement was found to be inefficient in enhancing significantly the slab capacity due to its brittleness. El-Ghandour et al. (2003) studied the effect of using CFRP in the form of shear reinforcements on the punching shear behaviour of interior slab-column connections. A total of eight specimens were tested, which were divided into two groups. The first one had no shear reinforcement, while the other had CFRP stirrups as shear reinforcement. The test results showed an increase in the punching shear capacity of the tested slabs between 13.3 and 16.5%. This enhancement in the capacity was due to the role of the shear reinforcement in preventing punching shear failure at lower load levels. However, the ineffectiveness of the shear reinforcement to totally prevent the punching shear failure of the connections may be attributed to the brittleness of the CFRP material of the used “stirrups.” This brittleness was believed to result in problems not expected with a ductile material like steel. Once the failure took place in one of the vertical legs of the stirrup, no redistribution of stresses occurred and all the other vertical legs failed progressively. This implied that a higher partial safety factor or reduced material strength should be used when designing with brittle shear reinforcement.

Zaghloul (2002 and 2007) studied the effect of shear reinforcement, in form of shear rails, on the behaviour of half-scale slab-column connections under eccentric load. Carbon FRP grids for both flexural and shear reinforcement were used in these studies. The test results revealed that using the shear reinforcement enhanced the punching load capacity up to 30%.

2.6.4 Effect of Concrete Compressive Strength

Banthia et al. (1995) tested four concrete slab-column connections with dimensions of $600 \times 600 \times 75$ mm, three specimens were reinforced with CFRP grids, and the fourth (control) was reinforced with steel wire mesh. The three specimens with CFRP grids had, respectively, normal-strength concrete, high-strength concrete and fibre-reinforced concrete with a normal-strength matrix. In the specimen with steel grid, normal-strength concrete was used. The test results showed that for the specimens reinforced with CFRP grids, the difference in the ultimate loads between the two slabs with normal strength concrete and high strength concrete was only 5%. In addition, the specimen with fibre-reinforced concrete had 11 and 17% higher failure load compared to the specimens with normal-strength concrete and high-strength concrete, respectively. The specimen with fibre-reinforced concrete and reinforced with FRP had 20% higher ultimate load and depicted better energy absorption in the initial part of the curve compared to the specimen reinforced with steel. However, the brittle fracture in FRP eventually led to a reduction in the overall absorption of the energy.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 INTRODUCTION

Using the non-corrodible FRP materials instead of conventional steel has proven to be an effective way to overcome the corrosion problem. Several studies have been conducted to investigate the punching shear behaviour of steel-RC interior slab-column connections subjected to concentric or eccentric load (Hawkins et al. 1989; Marzouk et al. 1998 and 2000; Guandalini et al. 2009). However, little research has been carried out to investigate the behaviour of FRP-RC slab-column connections. All tests, except one (Zaghloul 2002), were carried out under concentric loads. The available literature on FRP-RC interior slab-column specimens can be divided into two categories based on the aim of the research. The first category, especially the early studies, aimed at providing models to predict the punching load capacity of FRP-RC slabs (Matthys and Taerwe 2000; Ospina 2005; Theodorakopoulos and Swamy 2007; Nguyen-Minh and Rovnak 2013). The second category focused on evaluating the effect of different parameters on the behaviour of such connections (Ospina et al. 2003; El-Ghandour et al. 2003; Lee et al. 2009; Dulude et al. 2013; Hassan et al. 2013).

To date, no research data is available for full-scale slab-column connections reinforced with GFRP bars and subjected to shear forces and unbalanced moments. The test program described in this research was designed to partially fulfill this gap. The aims of this research program were fulfilled through two phases. Phase one is the experimental work, while phase two is the numerical study using a specialized finite element software package (Cervenka et al. 2013).

3.2 EXPERIMENTAL STUDY

Ten full scale interior slab-column specimens were constructed and tested up to failure. In the current study, a multi-story, multi-bay prototype parking structure with 300-mm square columns equally spaced at a distance of $\ell = 6.5$ m (centreline-to-centreline) in both directions was analyzed to obtain the slab dimensions and reinforcement (Fig. 3.1). The size of the columns and their spacing were chosen to simplify the test specimen. Based on the CSA/A23.3-14 standard (CSA 2014), the obtained slab thickness and steel reinforcement ratio in the column strip was 200 mm and 0.65%, respectively. The slab represented the zone of negative moments bounded by the lines of contra-flexure around an interior column, which are assumed at a distance of 0.2ℓ from the centreline of the column as shown in Fig. 3.2. The assumed building (parking garage) was designed to withstand a total factored load, $w=10.85$ kN/m², which composed of a factored dead load ($D.L. = 7.25$ kN/m²) and factored live load ($L.L. = 3.6$ kN/m²) calculated according to the NBCC (NRCC 2010). An elastic analysis was performed on the described structure using the method of frame strip (CSA 2014), with 6.5-m total width for the strip. Different combination of gravity loads (dead and live loads) were used in the analysis, no lateral load was considered. The recommendations of the Canadian standards CSA/S806-12 (CSA 2012) and CSA/A23.3-14 (CSA 2014) were followed in the design, as appropriate. The resulted specimen consisted of a $2600 \times 2600 \times 200$ mm slab with 300×300 mm square column stub extended 1000 mm above and below the slab. However, a square slab with 2800-mm side length was cast to allow for supporting clearness (Fig. 3.3). Also, for most of the tested specimens (eight slabs), the moment-to-shear ratio (Mu/V) was kept constant at 0.15 m, as this value resulted from the analysis. The overall concrete dimensions of the test specimens are shown in Fig. 3.3. The details of the test specimens are listed in Table 3.1.

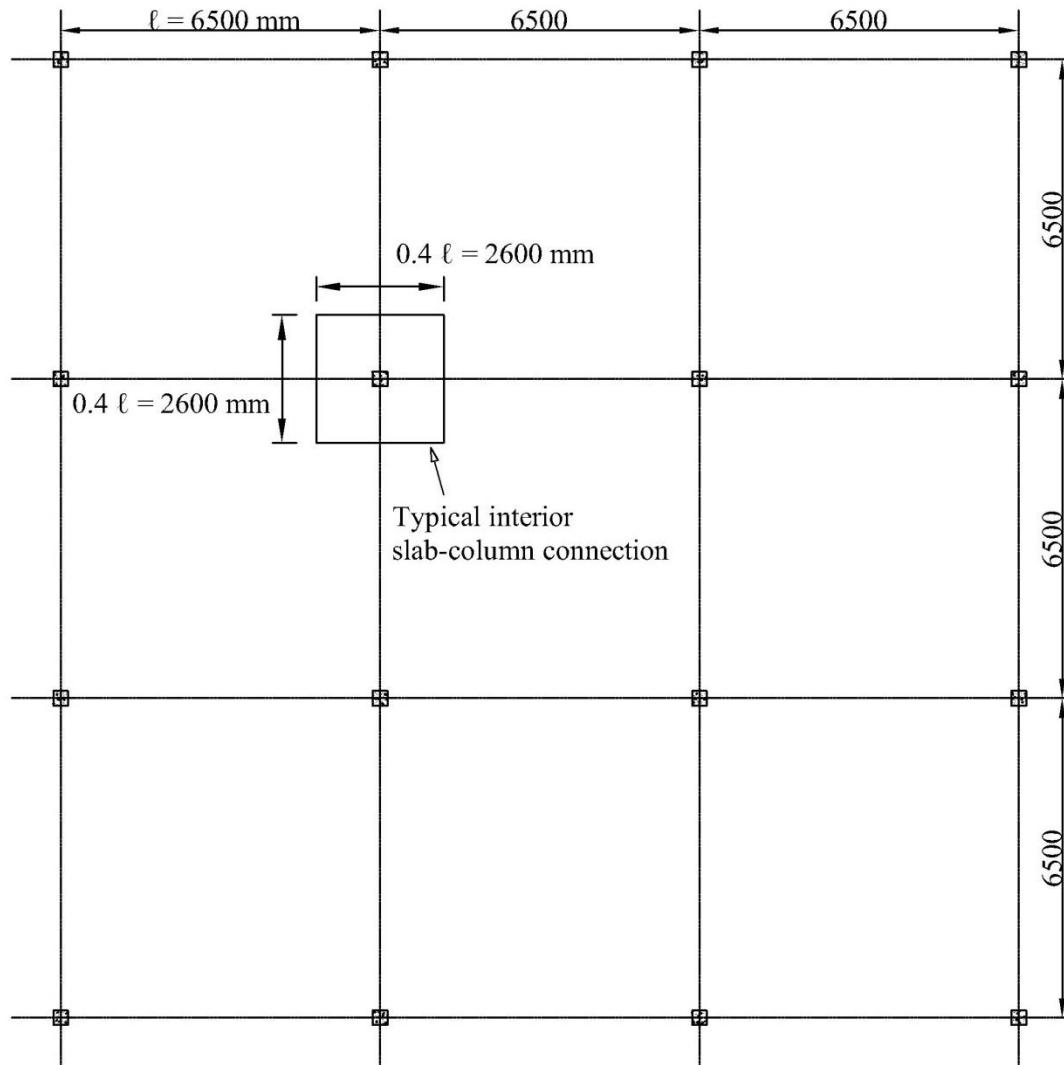
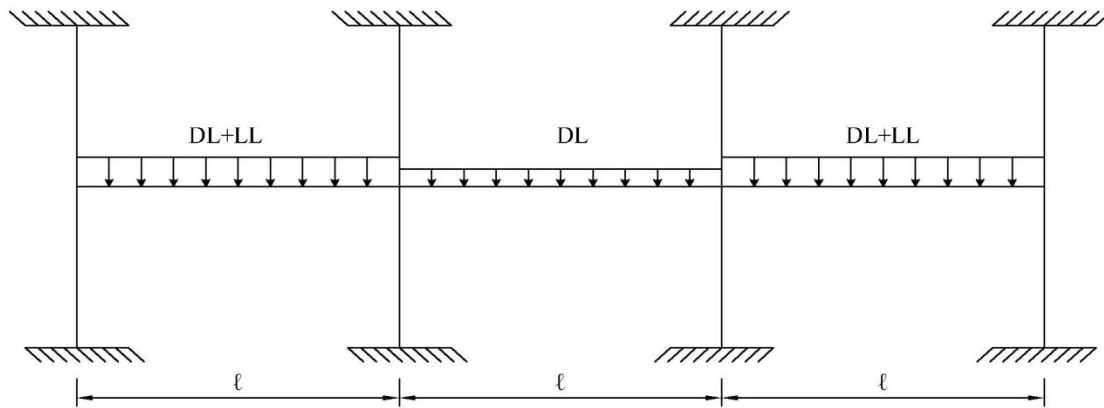
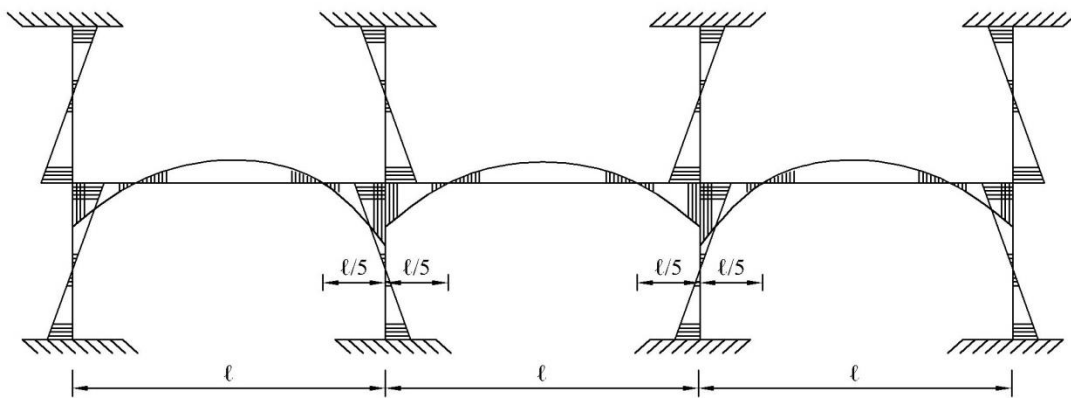


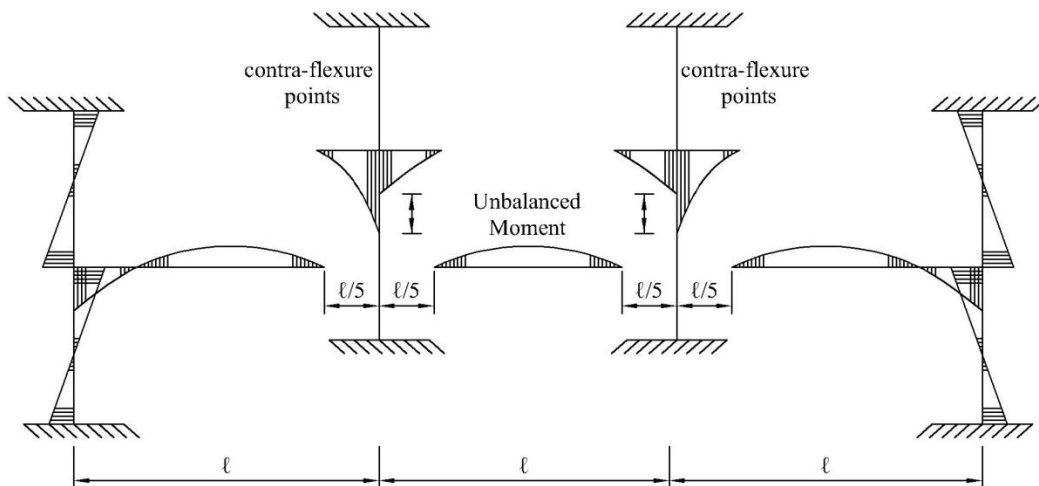
Figure 3.1: The portion of slab under consideration in a typical flat plate floor plan



a) Loads

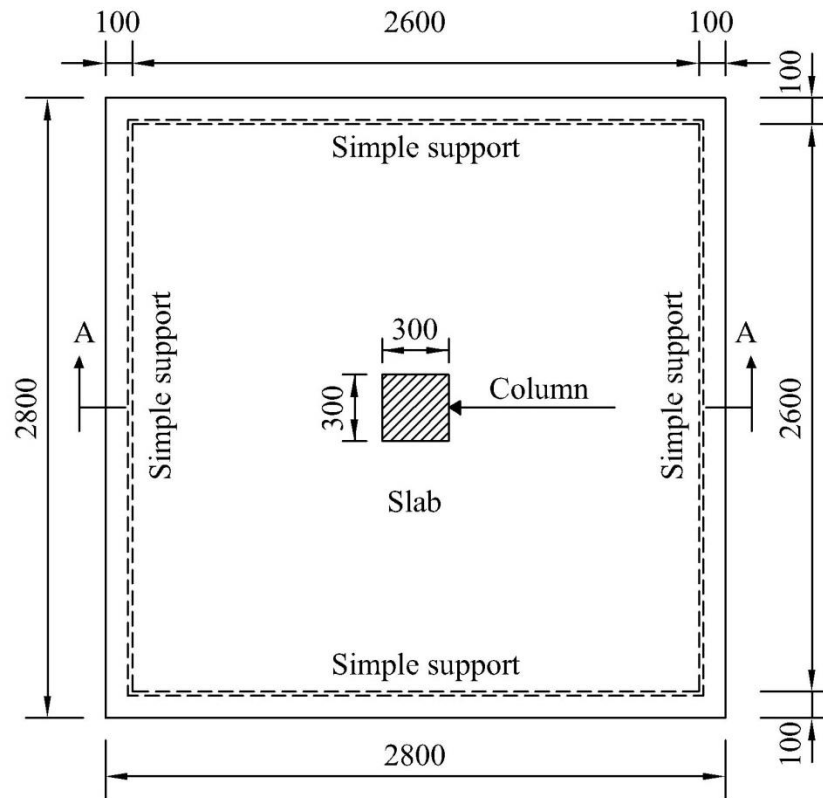


b) Bending moment

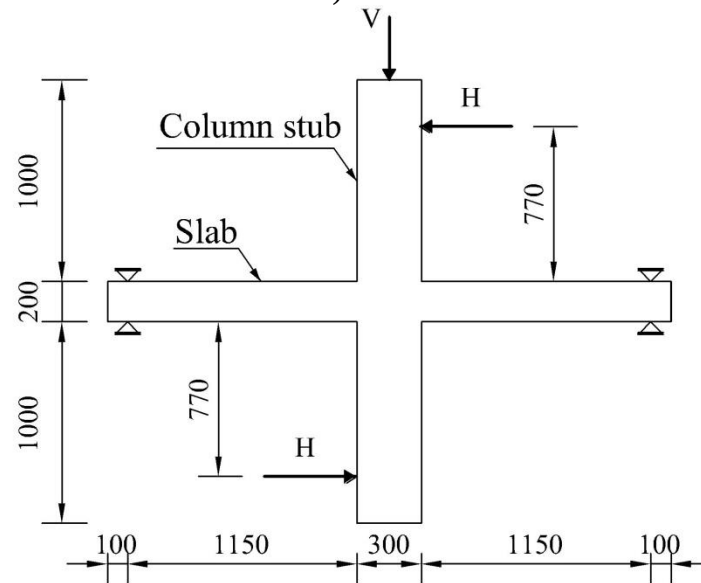


c) Contra-flexure points

Figure 3.2: Approach for determining the dimensions of a typical specimen



a) Plan



b) Section A-A

All dimensions are in mm

Figure 3.3: Concrete dimensions of a typical specimen

Each specimen designation is composed of five symbols the first is a character stands for the type of the material used for the flexure reinforcing bars (“G” represents sand-coated GFRP bars, while “S” stands for steel material and “R” represents ribbed-deformed GFRP bars). The second is a number describes the flexure reinforcement ratio (0.65, 0.98 and 1.30%). The third is a character symbolizes for the concrete compressive strength (“N” and “H” for normal and high strength concrete, respectively). The fourth is a number stands for the moment-to-shear ratio (00, 15 and 30 for M_u/V of 0.0, 0.15 and 0.3 m, respectively). The last part indicates the spacing of the shear studs, if any, as a ratio of the average effective slab depth, d , which equals to 160 mm (50 and 75 stand for shear reinforcement with $0.5 d$ and $0.75 d$ studs spacing, respectively, while characters “XX” mean no shear reinforcement was provided). For instance, G-0.65-N-15-XX describes a specimen reinforced in flexure with sand-coated bars without shear studs, with 0.65% flexure reinforcement ratio, cast using normal strength concrete (35 MPa) and subjected to 0.15-m moment-to-shear ratio.

As listed in Table 3.1, five parameters were investigated:

- 1- Flexural reinforcement ratio (1.5, 2.25 and 3 ρ_b , where ρ_b is the balanced reinforcement ratio).
- 2- Concrete compressive strength (35 and 70 MPa).
- 3- Moment-to-shear ratio (0.0, 0.15 and 0.3 m).
- 4- Type of the reinforcing bars (GFRP and steel).
- 5- Shear reinforcement ratio (0.0, 0.41 and 0.61%).

For comparison purposes, the specimens were divided into three series. Series I contained five specimens (G-0.65-N-15-XX, G-0.98-N-15-XX, G-1.30-N-15-XX, S-0.65-N-15-XX and G-0.65-H-15-XX) to evaluate the effects of reinforcement ratio, bar material in addition to the

concrete compressive strength on the behaviour of slab-column connections. Series II comprised three connections (G-0.65-N-00-XX, G-0.65-N-15-XX and G-0.65-N-30-XX) to study the effect of moment-to-shear ratio. Series III included three specimens (R-0.65-N-15-XX, R-0.65-N-15-75 and R-0.65-N-15-50) to examine the shear reinforcement effect.

3.2.1 Material Properties

3.2.1.1 Concrete

All test slabs were constructed using normal-weight, ready-mixed concrete with a targeted 28-day concrete compressive strength of 35 and 70 MPa and a maximum aggregate size of 19 mm. All test prototypes were cast and wet-cured in the laboratory for 7 days. On the day of testing, the actual concrete compressive and tensile strengths (Table 3.1) were determined based on the average value of compressive and tensile splitting tests carried out on standard cylinders of 100 × 200 mm and 150 × 300 mm, respectively.

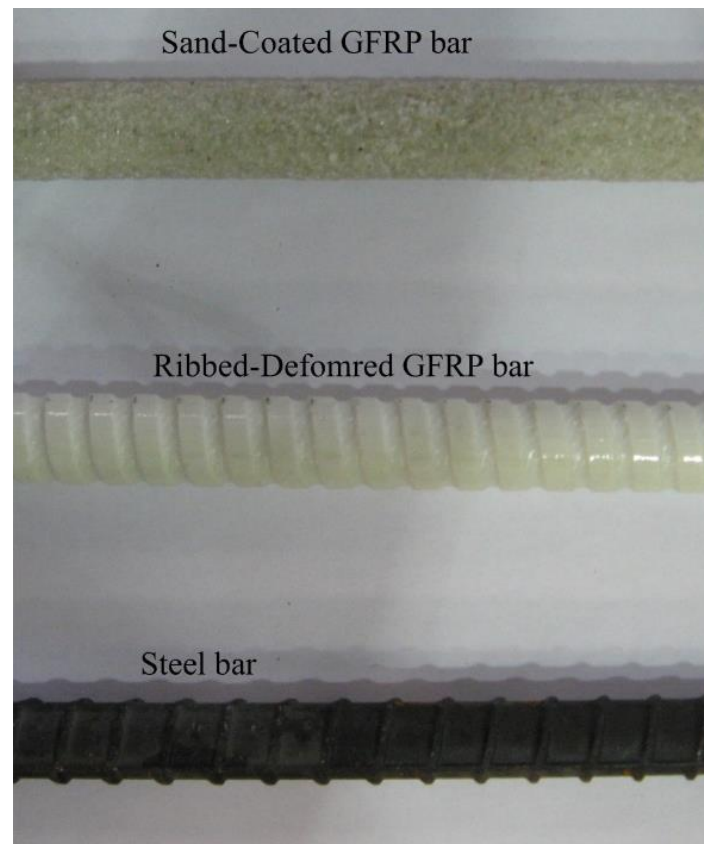
3.2.1.2 Reinforcement

Steel bars, sand-coated (Pultrall Inc. 2013) and ribbed-deformed GFRP bars (Schoeck Canada Inc. 2013) were used as slab reinforcement. Standard tests, according to CSA/G30.18-09 and CSA/S806-12, Annex C, were carried out to determine the mechanical properties of the steel and the GFRP reinforcing bars, respectively. Size No. 12 ribbed-deformed, double-headed, 170-mm long bars were used as shear studs in the slabs with transvers reinforcement (R-0.65-N-15-75 and R-0.65-N-15-50). According to the manufacturer, the heads of these shear studs were made of polymer concrete with a compressive strength of 90 MPa and an allowable design strain in the GFRP stud of 2175 $\mu\epsilon$ (130-MPa tensile strength). The properties of the used GFRP and steel bars are listed in Table 3.2. Also, Fig. 3.4 shows a photo for the reinforcing bars.

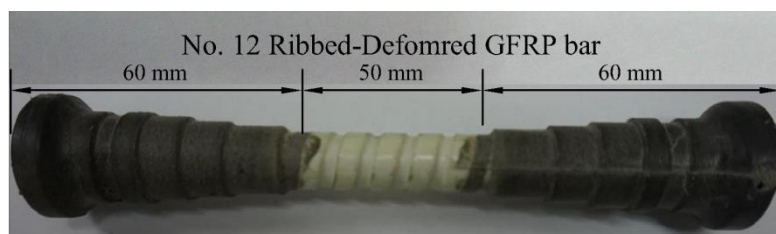
Table 3.1: Test specimens

Specimens	Concrete Compressive Strength, f_c' (MPa)	Concrete Tensile Strength, f_r (MPa)	Type of Reinforcement	Flexural Reinforcement		Shear Reinforcement		M_u/V
				Bar spacing (mm)	Ratio, ρ (%)	Stud spacing (mm)	Ratio, ρ (%)	Ratio (m)
Series I								
S-0.65-N-15-XX	42±0.5	3.4±0.2	Steel (No. 15M)	192	0.65			
G-0.65-N-15-XX	42±0.9	3.3±0.1						
G-0.65-H-15-XX	70±1.1	5.0±0.2	Sand-coated GFRP (No. 16)			N/A		0.15
G-0.98-N-15-XX	38±0.7	2.9±0.3		128	0.98			
G-1.30-N-15-XX	39±0.6	2.9±0.2		96	1.30			
Series II								
G-0.65-N-15-XX*	42±0.9	3.3±0.1	Sand-coated GFRP (No. 16)	192	0.65	N/A		0.15
G-0.65-N-00-XX	38±0.2	2.9±0.3						0.0
G-0.65-N-30-XX	42±0.8	3.4±0.2						0.3
Series III								
R-0.65-N-15-XX	40±0.5	3.2±0.4	Ribbed- Deformed GFRP (No.16)	192	0.65	N/A		0.15
R-0.65-N-15-75	42±0.6	3.4±0.2				120	0.41	
R-0.65-N-15-50	42±0.6	3.2±0.1				80	0.61	

* Same specimen from Series I for comparison purposes.



a) Flexure reinforcement



b) Shear reinforcement

Figure 3.4: Reinforcing bars

Table 3.2: Properties of reinforcing bars

Bar Material	Diameter (mm)	Area (mm ²)	Tensile Modulus [§] (GPa)	Ultimate Strength [§] (MPa)	Ultimate Strain [§] (%)
Sand-Coated GFRP	15.9 [18.8]*	198 [§] [277]*	68.0±0.3 [§]	1398±34 [§]	2.05±0.04 [§]
Ribbed-Deformed GFRP	15.9 [17.4]*	201 [§] [238]*	63.1±0.2 [§]	1122±28 [§]	1.78±0.04 [§]
	12.0 **	113 **	60.0**	130**	0.2175**
Steel	15.9	200	200	$f_y = 480^{\S\S}$	$\epsilon_y = 0.24^{\S\S}$

* Values between brackets are calculated according to Annex A (CSA/S806-12)

[§] Values are based on the nominal cross-sectional area of GFRP bars.

** Values provided by the manufacturer

^{§ §} Steel yield stress and strain

3.2.2 Design Concepts of Test Specimens

The design of the slab-column specimens was carried out according to CSA standards. The GFRP-slabs were designed according to CSA/S806-12 code (CSA 2012), while the steel-RC specimen (S-0.65-N-15-XX) was designed using the recommendations of CSA/A23.3-14 code (CSA 2014). Also, all the columns were reinforced with steel and designed according to CSA/A23.3-14 code (CSA 2014). Furthermore, no reinforcement was provided in the compression side since the contribution of GFRP bars in compression is similar to that of concrete, which is neglected according to current FRP codes and guideline (CSA/S806-12; ACI 440.1R-06; JSCE-97). Therefore, the contribution of the FRP bars to the capacity would not be significant.

In addition, an elastic analysis was performed on a parking garage as mentioned earlier. That analysis resulted for moment-to-shear ratio of 0.15 m to be applied on the specimens. Eight specimens were tested with that ratio. In addition, specimen G-0.65-N-00-XX was tested under concentric load and that ratio for specimen G-0.65-N-30-XX was 0.3 m.

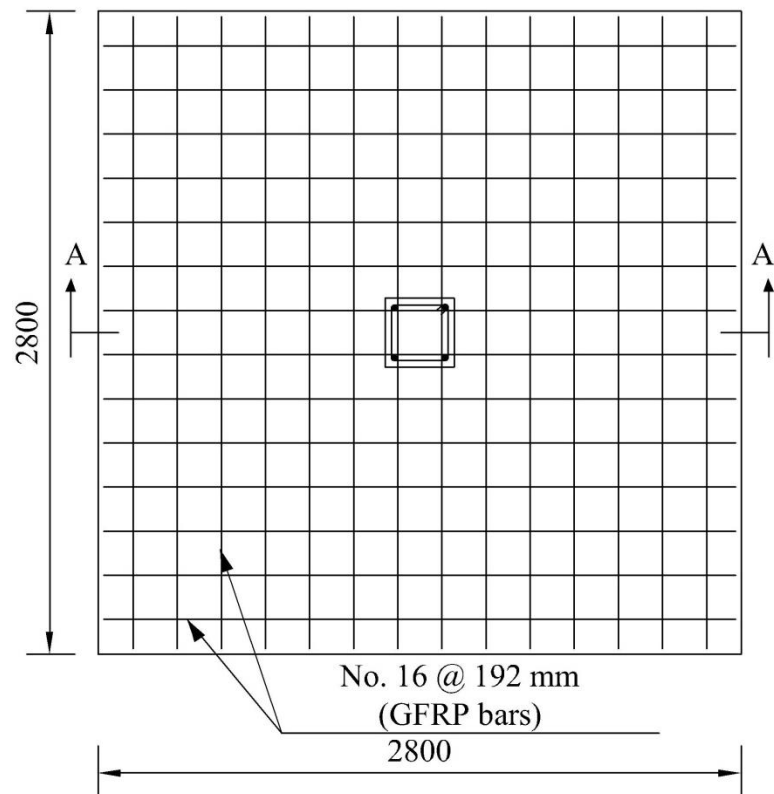
3.2.3 Details of Test Specimens

All test slabs were reinforced in flexure with one orthogonal assembly of GFRP bars placed in the tension side. The available equipment's allowed only for testing the specimens in a reverse position with respect to the actual position of a real building. Consequently, the tension reinforcement was set in the soffit of the slabs in the both orthogonal directions. A total of nine slabs were reinforced with GFRP bars, six with sand-coated GFRP bars and three with ribbed-deformed GFRP bars, in addition to one slab reinforced with conventional steel bars. Slabs G-0.65-N-15-XX, G-0.65-H-15-XX, G-0.65-N-00-XX and G-0.65-N-30-XX were reinforced with no. 16 sand-coated GFRP bar with spacing of 192 mm. Also, slabs G-0.98-N-15-XX and G-1.30-N-15-XX were reinforced with the same type and size of bars, but with different spacing of 128 and 96 mm, respectively. In addition, slabs R-0.65-N-15-XX, R-0.65-N-15-75 and R-0.65-N-15-50 had number 16 ribbed-deformed GFRP bars with a spacing of 192 mm. Moreover, the control steel-RC slab (S-0.65-N-15-XX) was reinforced with number 15M steel bars at a spacing of 192 mm.

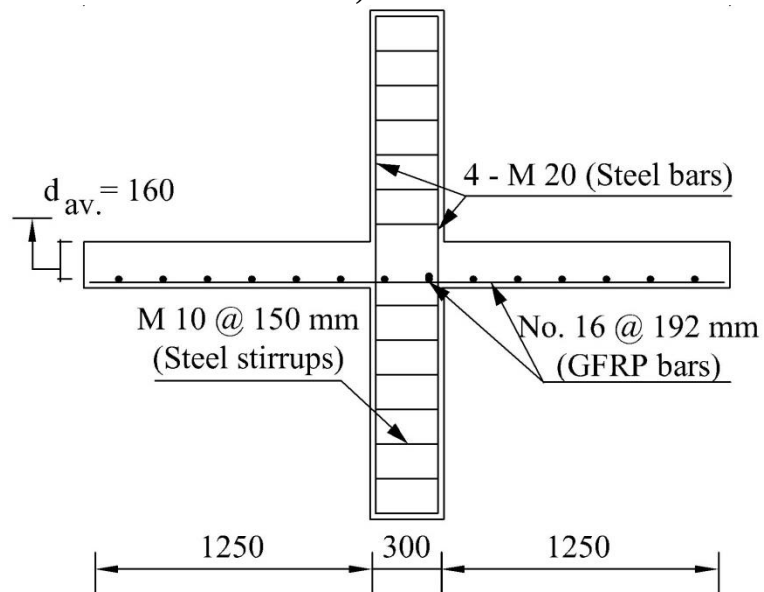
Two slabs had been provided with shear reinforcement (shear studs). The spacing and arrangement of the transverse reinforcement in slabs R-0.65-N-15-75 and R-0.65-N-15-50 were obtained according to the requirements of the Canadian Standards CSA/A23.3-14 (CSA 2014), since there is no provision for the shear stud reinforcement in any FRP code (CSA/S806-12; ACI 440.1R-06; JSCE-97). The distance between the column face and the first line of shear studs

was $0.4 d$ (64 mm) to satisfy the requirements of Clause 13.3.8.6. Whereas, the spacing between the other rows was $0.5 d$ (80 mm) for specimen R-0.65-N-15-50 and $0.75 d$ (120 mm) for specimen R-0.65-N-15-75. Also, the shear reinforcement was extended to a distance of $3.4 d$ from the column faces for both specimens.

The columns for all specimens were heavily reinforced with 4 No. 20M steel bars and No. 10M stirrups every 150 mm to prevent premature failure. The reinforcement details of the specimens are shown in Figures 3.5 to 3.9.



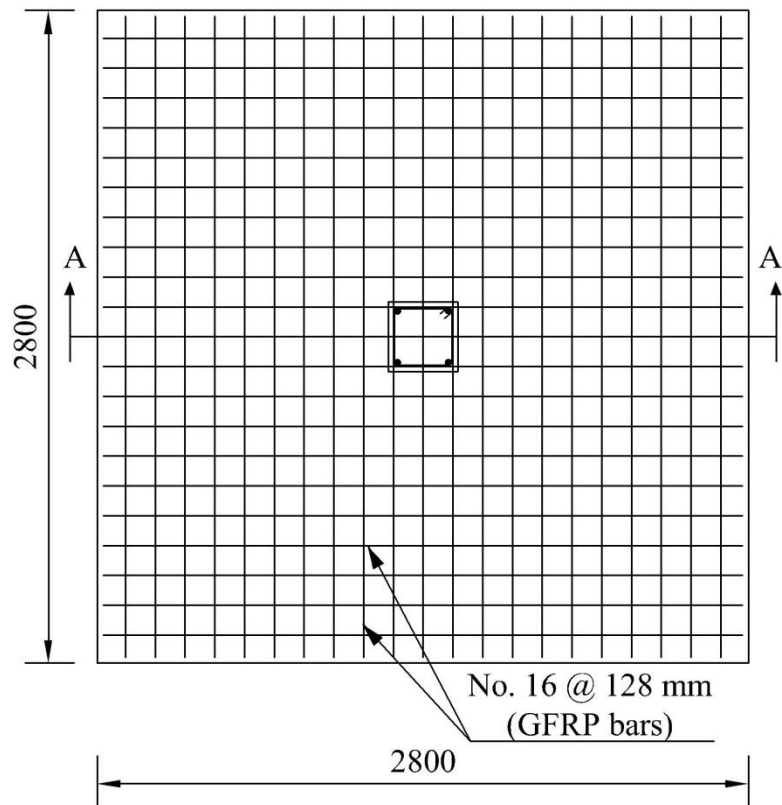
a) Plan



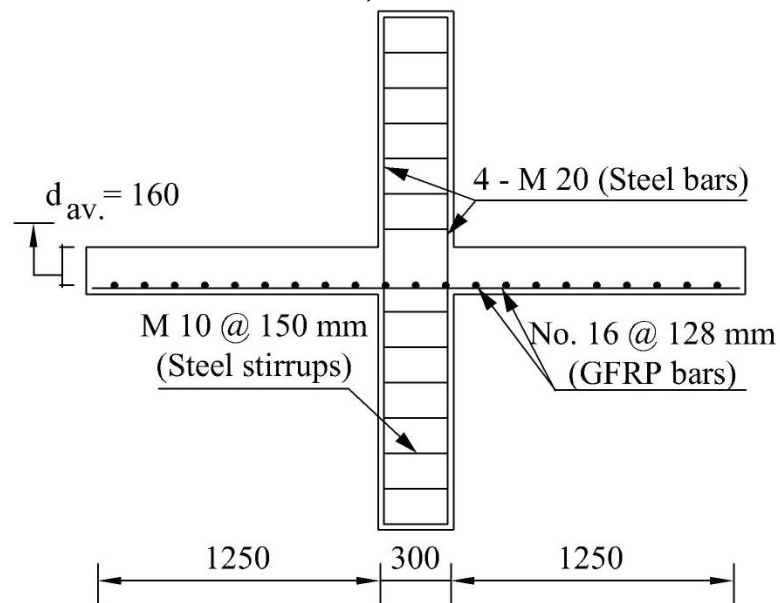
b) Section A-A

All dimensions are in mm

Figure 3.5: Typical reinforcement for all specimens with 0.65% reinforcement ratio



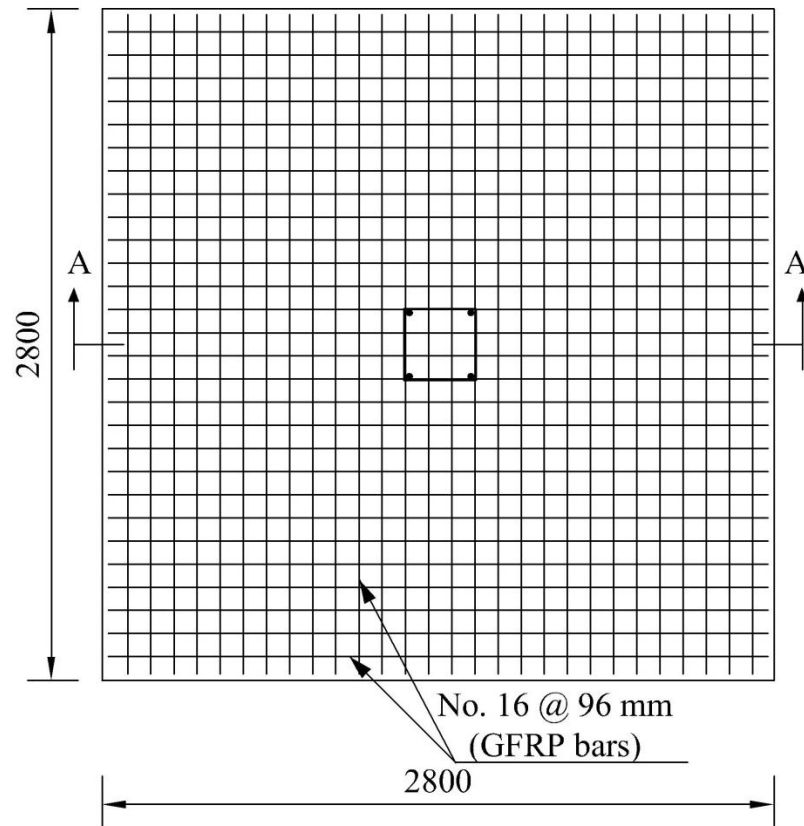
a) Plan



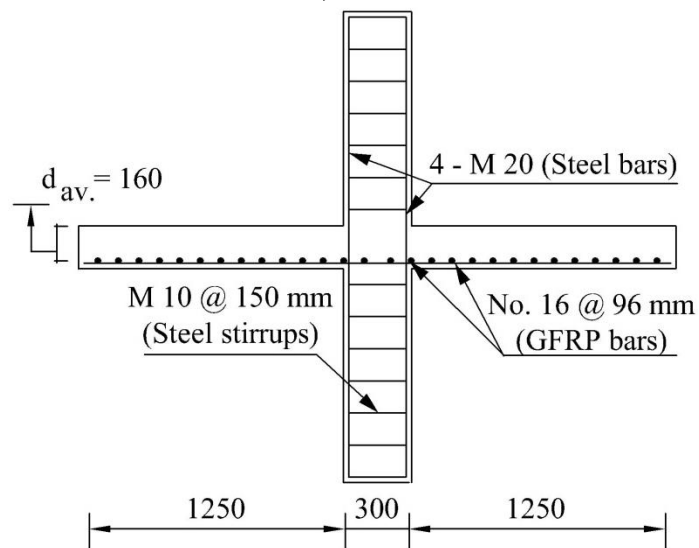
b) Section A-A

All dimensions are in mm

Figure 3.6: Typical reinforcement for the specimen with 0.98% reinforcement ratio



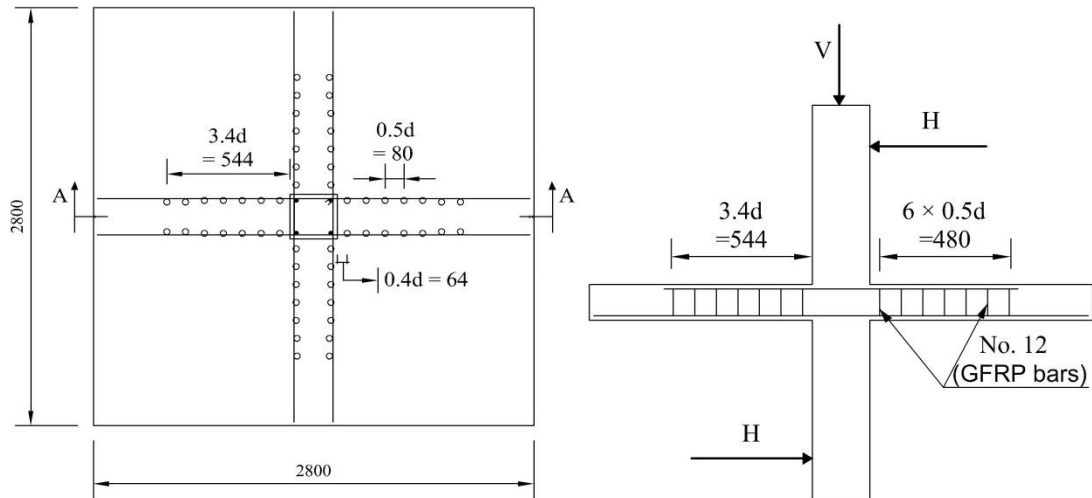
a) Plan



b) Section A-A

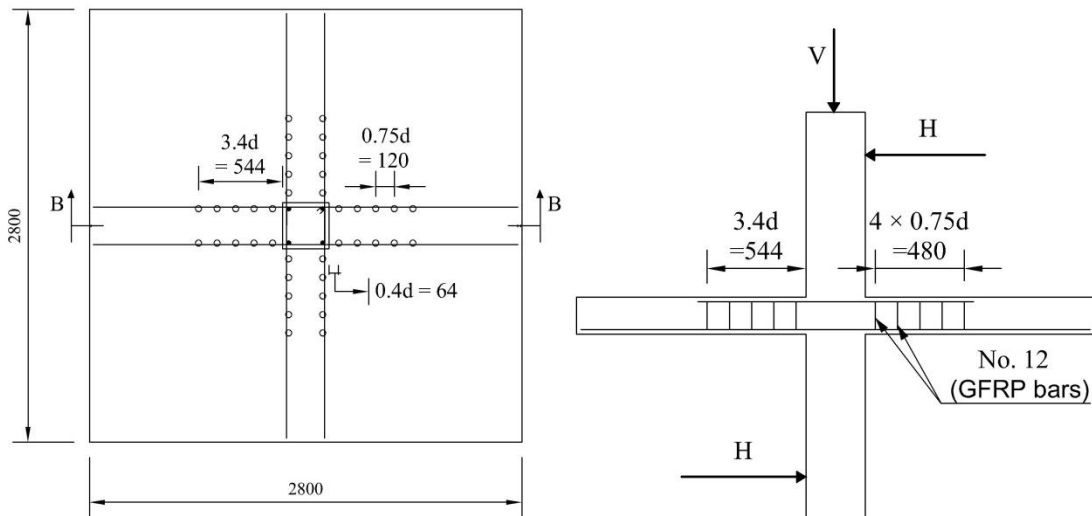
All dimensions are in mm

Figure 3.7: Typical reinforcement for the specimen with 1.30% reinforcement ratio



a) Shear studs layout for specimen R-0.65-N-15-50

b) Section A-A



c) Shear studs layout for specimen R-0.65-N-15-75

d) Section B-B

All dimensions are in mm

Figure 3.8: Configurations of the shear studs reinforcement

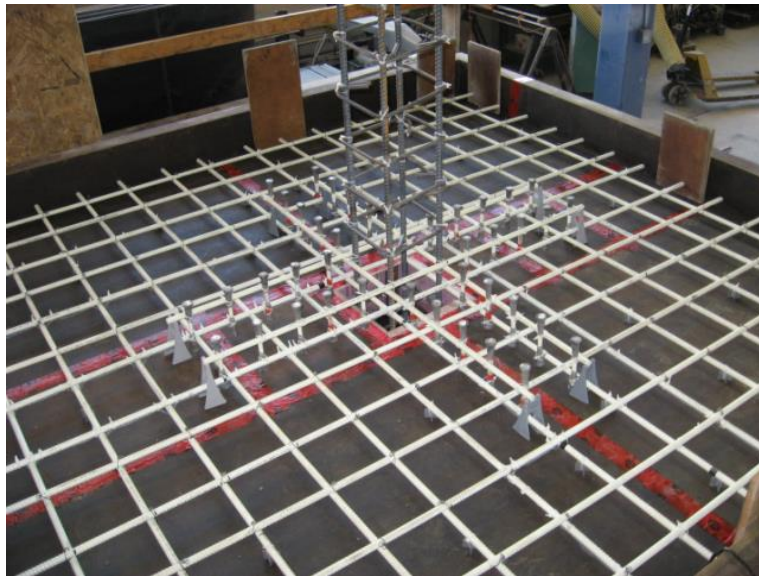


Figure 3.9: Photo for the reinforcement configurations for specimen R-0.65-N-15-75

3.2.4 Fabrication of Test Specimens

Figure 3.10a shows a photo for the formwork, used to cast the test specimens, which consisted of plywood plates and studs supported by two concrete blocks. Before casting, the formwork was coated with oil to ease the removal of the specimen after concrete hardening. Then the reinforcement cage for the columns was placed in the specified location in the formwork followed by the cage for the slab.

Casting each specimen was carried out on two stages to simulate the construction of the real structure. In the first stage, the bottom column and the slab were cast using ready-mixed concrete. Then, within three days, the top column was cast using concrete mixed in the laboratory. To track the change in the mechanical properties of each specimen, twenty standard cylinders of 100×200 mm were cast to obtain the concrete compressive strength in addition to six larger cylinders (150×300 mm) to determine the modulus of rupture on the day of testing.



a) Photo for the formwork



b) During casting

Figure 3.10: Construction stages of a typical specimen



Figure 3.11: Curing of the slabs

3.2.5 Test Set-up and Loading Procedures

The test set-up consisted, mainly, of two supporting frames, a hydraulic actuator with 1000-kN capacity and two hydraulic jacks. The specimens were tested in upside-down position with respect to the actual position in a real structure. The loading procedure can be described as follows:

- 1- The vertical shear force, V , was applied through the hydraulic actuator at the top of the upper column stub. A roller plate was placed between the actuator head and the column stub to allow for the horizontal movement of the column tip.
- 2- To produce the unbalanced moment, M_u , two hydraulic jacks were used to apply two equal and opposite horizontal forces, H , on the column stubs at a distance of 770 mm from the top and bottom faces of the slab, respectively. Also, a spherical compression head with roller plate was attached to each jack to allow for the vertical movement of the specimen.

The perimeter of the slab was simply-supported on a rigid frame through 25-mm width steel bearing plates. To prevent the uplift of the slab sides due to the unbalanced moment, another frame was used on the top of the slab while the corners of the slabs were left free to move in upward direction only. The two supporting frames were clamped together using eight C-clamps. In addition, the loading was stopped at increments of 20 kN to mark the propagation of the cracks. The test set-up is shown in Figures 3.12 and 3.13.

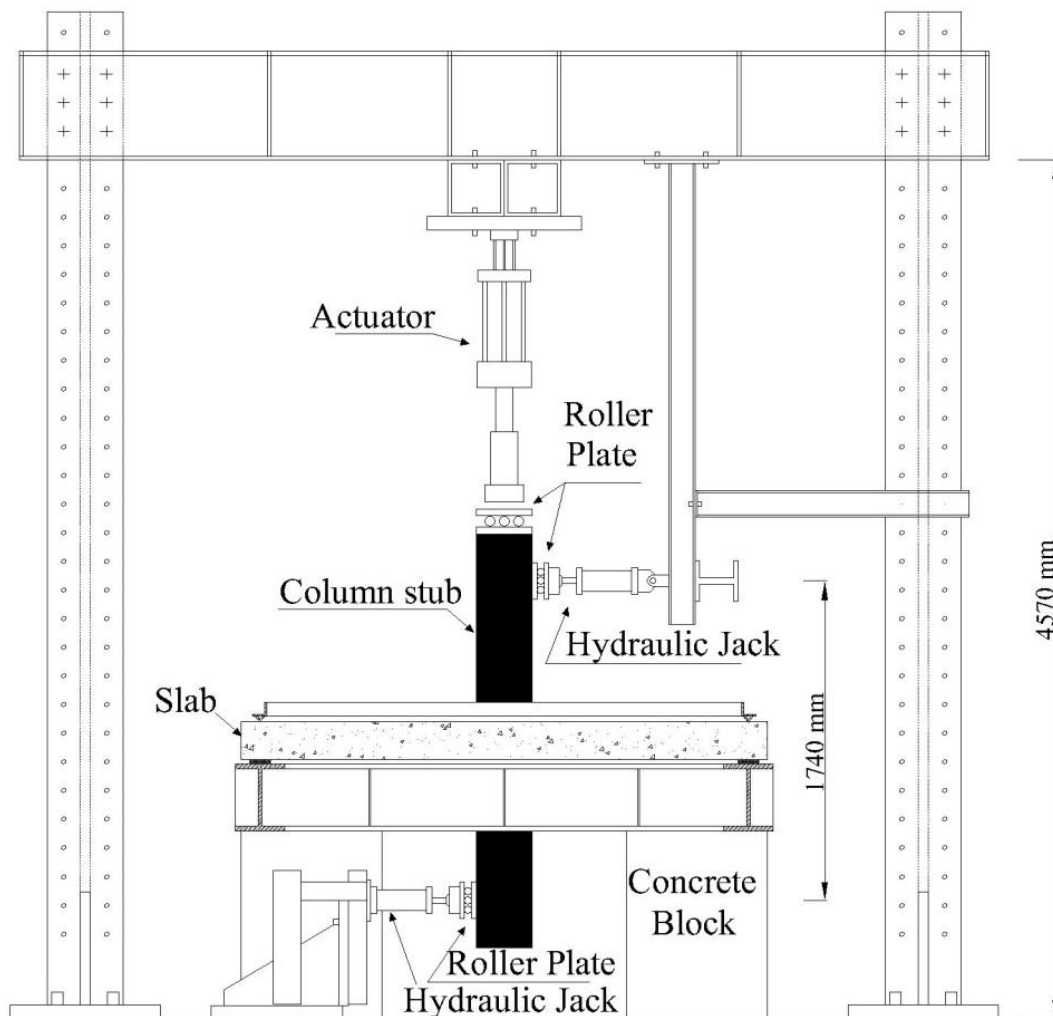


Figure 3.12: Schematic drawing of the test setup



Figure 3.13: Photo for the test setup

3.2.6 Instrumentation

Extensive instrumentation was designed to monitor the behaviour of the specimens during the test by providing a real-time recording of applied load, displacements and strains using a computerized DAQ system. Both internal and external instrumentations were used to monitor the behaviour during the test. A large number of strain gauges were mounted on the reinforcement bars at critical locations to detect strains during the test. In addition, a group of linear variable displacement transducers (LVDTs) were used to monitor the behaviour. Details of the instrumentation are discussed in the following section.

3.2.6.1 Load cells

Three load cells were used to measure the applied loads. Two load cells were attached to the two hydraulic jacks to measure the applied lateral forces on columns tips. The third load cell was

connected to the actuator head at the top column stub to measure the shear forces (column vertical load) applied on the connection.

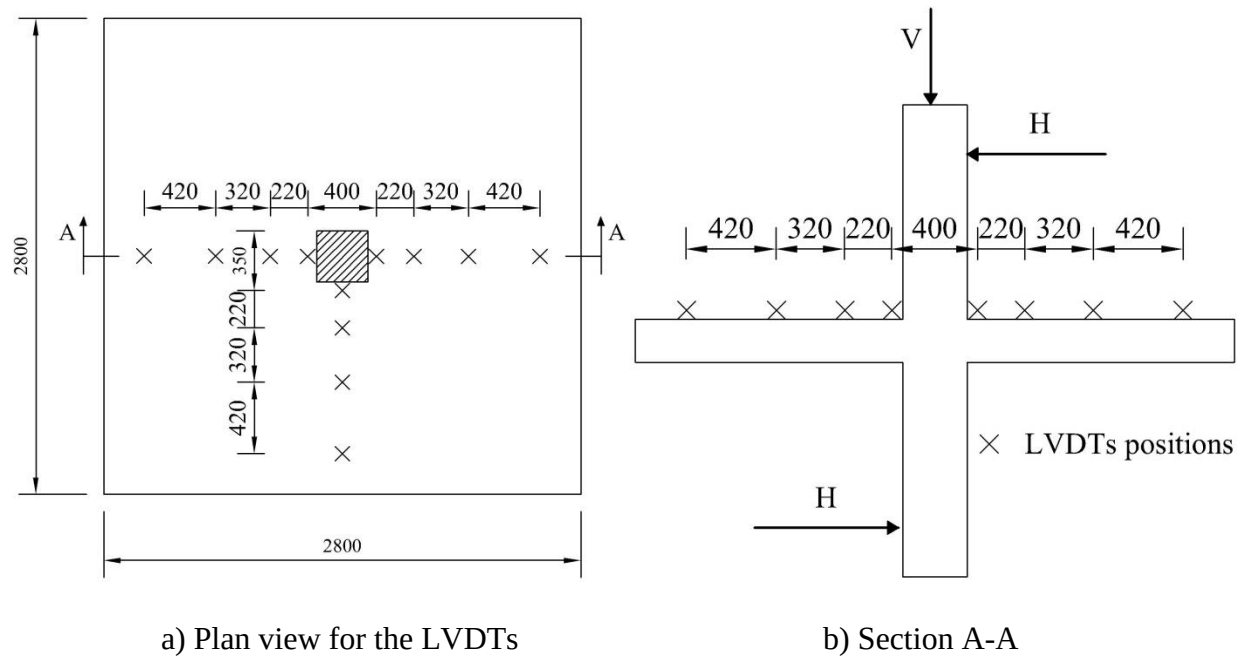
3.2.6.2 Linear variable displacement transducers (LVDTs)

Deflections at several points on the compression side of the slab were measured using twelve LVDTs as shown in Fig. 3.14. The locations of the LVDTs were chosen to measure the maximum deflection at the column face and at several other locations. Also, dial gauges were used to monitor the horizontal displacement at the top and bottom of the column stubs.

3.2.5.3 Strain gauges

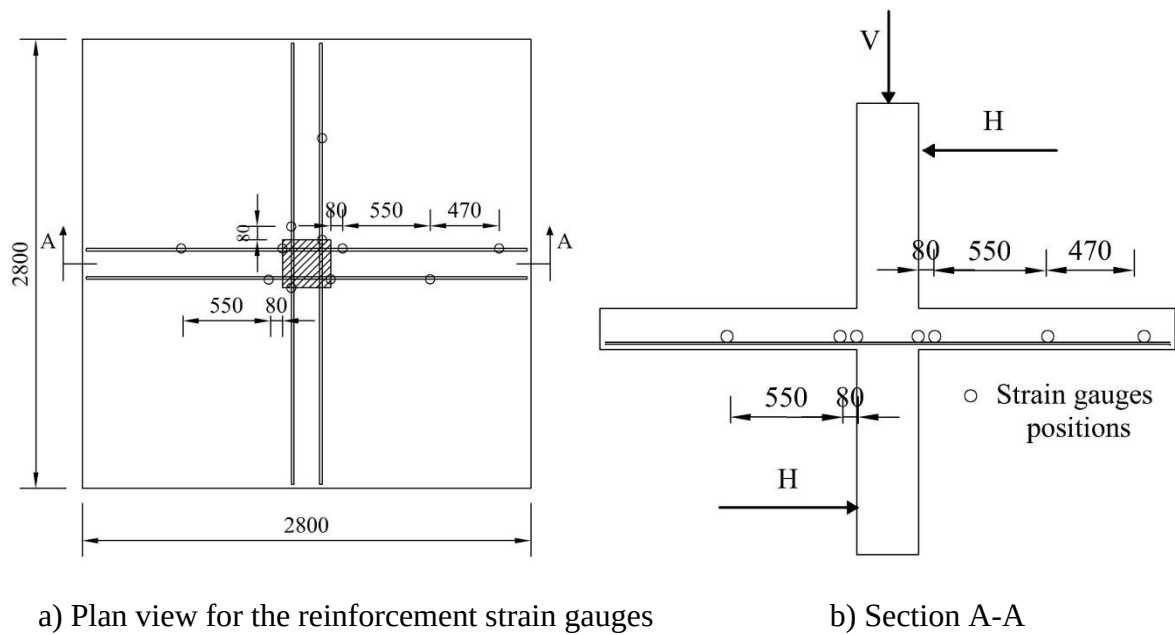
Electrical strain gauges (6-mm) were attached at eleven locations to four GFRP bars (two in each orthogonal direction) to measure the reinforcement strains (Fig. 3.15). Also, two PI-gauges and two strain gauges were glued on the compression side of the slab to obtain the concrete strains at 50 mm from the column face.

Figure 3.16 presents the strain gauges layout for the shear reinforcement in specimens R-0.65-N-15-75 and R-0.65-N-15-50 with respect to the direction of the unbalanced moment. Ten strain gauges were installed on the shear studs in the direction of the horizontal loads. The strain gauges were extended to a distance of $3.4 d$ for specimen R-0.65-N-15-75 and $2.4 d$ for specimen R-0.65-N-15-50 from the column faces in that direction. Also, six strain gauges were used in the perpendicular direction and extended to a distance of $1.9 d$ for specimen R-0.65-N-15-75 and $1.4 d$ for specimen R-0.65-N-15-50 from the column faces.



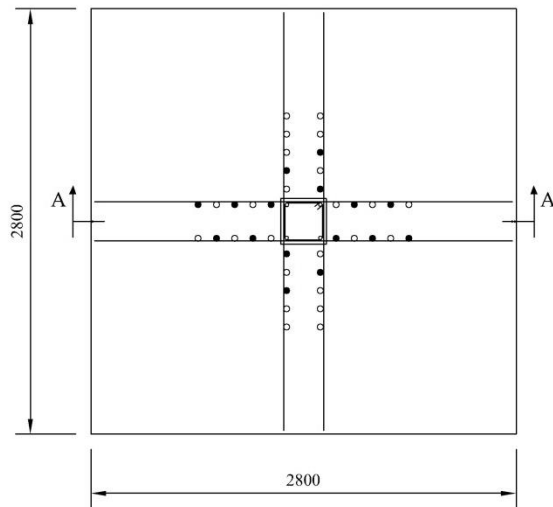
All dimensions are in mm

Figure 3.14: Typical arrangement of LVDTs

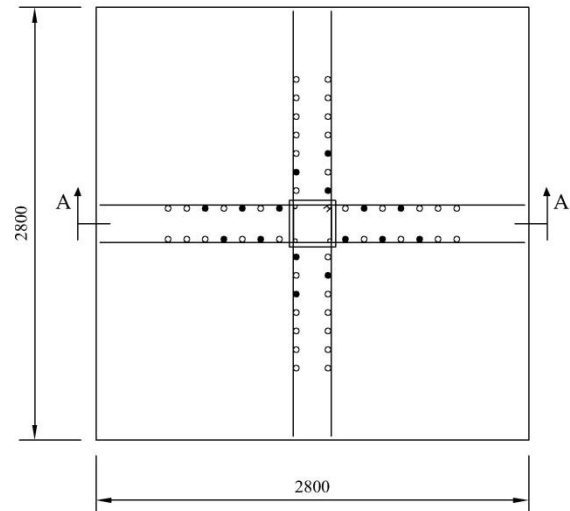


All dimensions are in mm

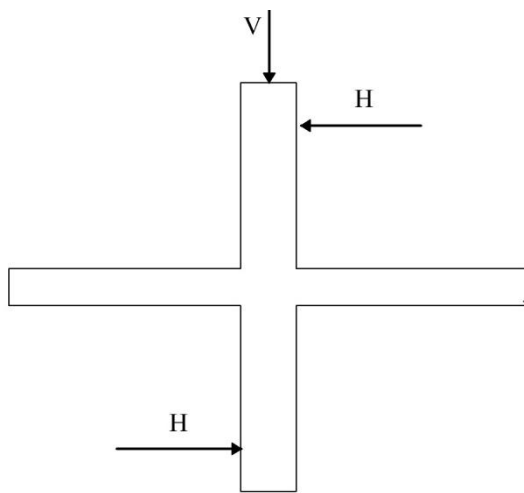
Figure 3.15: Typical arrangement of the flexure reinforcement strain gauges



a) Strain Gauges Layout for specimen R-0.65-N-15-75



b) Strain Gauges Layout for specimen R-0.65-N-15-50



c) Section A-A

- Shear studs with strain gauge
- Shear studs without strain gauge

All dimensions are in mm

Figure 3.16: Strain gauges layout for the shear reinforcement

CHAPTER 4

EXPERIMENTAL RESULTS AND DISCUSSION

4.1 INTRODUCTION

This chapter presents the analysis and the discussion of the experimental work described in Chapter 3. The specimens were tested under vertical shear force and two equal and opposite horizontal loads to simulate the effect of the unbalanced moment. The behaviour of the slab-column connections was carefully monitored through the test. Deflection of the slabs, strains in both reinforcing bars and concrete were measured and recorded at different locations.

The complete analysis for the test specimens was presented in terms of cracking pattern, ultimate capacity, vertical load-deflection curve, deflection profiles, strain profiles in the direction of the unbalanced moment and vertical load-strain curve in the reinforcing bars and on the compression side of the slabs. Furthermore, the test results were compared with the provisions of different codes such as the Canadian standard CSA/S806-12 (CSA 2012) and the American guideline ACI 440.1R-06 (ACI 2006).

4.2 SERIES I - THE EFFECTS OF REINFORCEMENT RATIO, BAR MATERIAL AND TYPE AND CONCRETE COMPRESSIVE STRENGTH

According to the Canadian standard CSA/S806-12 (CSA 2012), any concrete section reinforced with FRP bars has to be over-reinforced. Also, 60 MPa is the maximum concrete compressive strength allowed by the code to be used in the punching shear equations (2.48, 2.49 and 2.50 in Chapter 2). Moreover, the code limits the maximum strain in the GFRP bars at service load stage to be less than 25% of the ultimate strain of the bars to control crack width. In the meantime, the American guideline ACI 440.1R-06 (ACI 2006) allows for tension and compression failures.

Also, at the service stage, the American guideline limits the maximum strain in the GFRP bars to be less than 20% of the ultimate strain of the bars (Clause 8.4.1) and the guideline has no apparent limits for the concrete compressive strength.

4.2.1 Cracking and Failure Mode of Specimens of Series I

Figure 4.1 shows the cracking pattern on the tension face of the test slabs at failure. All specimens exhibited similar behaviour in terms of cracking pattern. At load levels between 110 and 140 kN, the first flexural crack appeared on the tension side of the slabs at the region of maximum bending moment around the column. The crack emanated from the corner of the column and traveled to the edge of the slab. At a higher load level (140 to 160 kN), cracks started to propagate from the column face toward the slab edge in the direction of the unbalanced moment, i.e., the direction of the greater depth. With further loading, additional radial cracks initiated in the slab with different angles of inclinations. The first circumferential crack that appeared in the vicinity of the column formed at a load level between 180 and 240 kN, which is approximately half the capacity of the specimens. After that loading stage, the rate of formation of new cracks decreased significantly and the existing cracks grew wider. That formation of cracking pattern is known as “star-burst” pattern of radial cracks, interspersed with orthogonal or circumferential cracks and the main cause of it is the concentration of the negative flexure stresses around the column periphery (Paret et al. 2010). Furthermore, Fig. 4.1 shows that more damage occurred in the slab (upper side of the photos) in the direction of the unbalanced moment as a result of the additional tensile stresses developed due to moment transfer.

At failure, all specimens exhibited a brittle punching shear failure with the column and a part of the slab penetrated through the slab thickness. This was observed regardless of the reinforcement

ratio and type. The punching failure surface was defined by a wide and deep crack around the column perimeter as shown in Fig. 4.1.

Increasing the reinforcement ratio from 0.65% (G-0.65-N-15-XX) to 0.98% (G-0.98-N-15-XX) or 1.3% (G-1.30-N-15-XX) resulted in a wider failure surface. This wider surface was due to the enhancement of the restraint in the plane of the slab as a result of the increased flexural reinforcement. Moreover, increasing the reinforcement ratio for the specimens reinforced with GFRP bars resulted in more cracks but with less crack spacing. This is attributed mainly to the reduced bar spacing and the distribution of stresses on more bars (Hutchinson and Wang 2009) with the increased reinforcement ratio. In addition, many researchers reported that crack width and spacing are directly proportional to the spacing between slab reinforcing bars and to the concrete cover thickness (Frosch 1999). Therefore, increasing the reinforcement ratio by increasing the bars diameter and keeping the spacing the same, might not have the same effect on the crack pattern observed in the current study, however further investigation is still needed to address this issue. Furthermore, specimen G-0.65-N-15-XX had smaller crack spacing compared to specimen S-0.65-N-15-XX at the same load level. The difference is related to the surface texture of the reinforcement, where GFRP sand-coated bars have uniformly distributed bond stresses due to adhesion and friction (Alves 2011), while steel deformed bars have mainly mechanical bond through bearing on the bar ribs.

Compared to specimen G-0.65-N-15-XX, the failure area around the column for specimen G-0.65-H-15-XX with high strength concrete (HSC) showed more damage. This is due to the brittleness of the HSC, which caused different interior damage patterns (cracks propagate through the coarse aggregate particles) compared to normal strength concrete (NSC).

4.2.2 Deflection of Specimens of Series I

The deflection characteristics were similar for all test specimens. Figure 4.2a shows the maximum measured deflection in the slabs at 50 mm from the column face. In general, the load-deflection curve can be divided into two phases. The first phase represents the behaviour of the uncracked specimens, which depends mainly on the mechanical properties of the concrete; therefore the GFRP-RC slabs with NSC (G-0.65-N-15-XX, G-0.98-N-15-XX and G-1.30-N-15-XX) showed comparable behaviour. The second phase illustrates the post-cracking behaviour up to failure, which is a function of the axial rigidity of reinforcement, ρE . Hence, the slabs with different axial rigidity (G-0.65-N-15-XX, G-0.98-N-15-XX, G-1.30-N-15-XX and S-0.65-N-15-XX) exhibited different behaviour in the post cracking stage.

In this research, the service load (NBCC 2010) was calculated assuming dead loads acting on the area carried by the column in the prototype structure (6.5×6.5 m) and live load acting on half of that area to account for the effect of the unbalanced moment and to address the loading cases that were used in the current study as shown in Fig. 3.2a. The service load was 295 kN with an associated unbalanced moment of 44.25 kN.m.

Specimen G-0.65-N-15-XX showed larger deflections with reference to specimen S-0.65-N-15-XX, due to the lower modulus of elasticity of the GFRP bars compared to that of steel. At service load level, specimen G-0.65-N-15-XX showed 206% higher deflection compared to specimen S-0.65-N-15-XX. At failure, this percentage decreased to only 42% due to the lower capacity of G-0.65-N-15-XX. Also, this can be attributed to the decrease in the post-cracking stiffness of the steel-RC slab, S-0.65-N-15-XX, before failure as a result of steel yielding around the column face at approximately 86% of the failure load. Compared to steel-RC control specimen, the

GFRP-RC specimen with the closest axial stiffness (approximately 68%), G-1.30-N-15-XX, showed similar deflection at failure (Table 4.1).

Comparing the measured deflections on the two centrelines of the slab where the LVDTs are installed, it can be seen that the deflection of the slab in the direction of the unbalanced moment (Fig. 4.3a) is due to the effect of vertical shear force in addition to unbalanced moment. However, in the direction perpendicular to it, the deflection of the slab is due to the effect of vertical force only (Fig. 4.3b). Consequently, the unbalanced moment increased the maximum measured deflection at service stage by 9.4% (1.6 mm), 12.1% (1.5 mm), 10% (0.9 mm), 10.7% (1.8 mm) and 8% (0.5 mm) for specimens G-0.65-N-15-XX, G-0.98-N-15-XX, G-1.30-N-15-XX, G-0.65-H-15-XX and S-0.65-N-15-XX, respectively. While, the corresponding values at failure were 7.6% (2 mm), 6.9% (1.4 mm), 8.3% (1.5 mm), 8.1% (2.6 mm) and 13% (2.3 mm), respectively. For specimen S-0.65-N-15-XX, the increase in deflection due to unbalanced moment (from 8% at the service stage to 13% at failure) is attributed to the local yielding of reinforcement at column face near failure.

Moreover, increasing the reinforcement ratio increased the post-cracking stiffness, K_p , calculated as the slope of the load-deflection curve after cracking (Table 4.1), and decreased the maximum measured deflection due to the increase in the cracked moment of inertia of the slabs. At service load, specimens G-1.30-N-15-XX and G-0.98-N-15-XX exhibited 50 and 30% less deflection, respectively, compared to specimen G-0.65-N-15-XX. At failure, these percentages decreased to 30 and 23%, respectively, due to the different capacity of the specimens. Furthermore, the values for the post-cracking stiffness, K_p , for specimens G-1.30-N-15-XX and G-0.98-N-15-XX was 110 and 50% more than that of specimen G-0.65-N-15-XX, respectively. Also, Fig. 4.4 displays the relationship between the reinforcement ratio and the post cracking stiffness for the GFRP-RC

slabs with NSC. In addition, mathematical equation was presented in the figure to find the correlation between the post cracking stiffness and the reinforcement ratio. The equation was obtained using the method of least squares. Moreover, that equation might be applicable only to the slabs with similar properties tested within the current study.

The use of HSC (G-0.65-H-15-XX) enhanced the uncracked stiffness and the deformation characteristics of the specimen as evidenced in Fig. 4.2b. However, in the post-cracking stage, the behaviour of the specimens relied mainly on the axial rigidity of the reinforcing bars (or post-cracking stiffness, K_p) that was similar in specimens G-0.65-N-15-XX and G-0.65-H-15-XX. In addition, due to higher capacity, the HSC specimen (G-0.65-H-15-XX) showed 21% larger deflection at failure.

The cumulative energy provides a measurement to the energy stored in a structure during loading, which may explain the behaviour of such structure when this energy is released. Figure 4.5 depicts the cumulative energy for the slabs calculated based on the area under load-deflection curves (Fig. 4.2a). Increasing the axial stiffness of the slab reinforcement increased the cumulative energy at the same deflection level. However, at failure, specimens S-0.65-N-15-XX and G-0.65-N-15-XX exhibited similar accumulated energy, which was 20% higher than those of specimens G-0.98-N-15-XX and G-1.30-N-15-XX. This indicates that the NSC specimens with 0.65% reinforcement ratio is expected to suffer more damage than those with greater reinforcement ratios. This is in good agreement with the observed punched perimeter for those slabs, where damage in specimens G-0.65-N-15-XX and S-0.65-N-15-XX was more than that in specimens G-0.98-N-15-XX and G-1.30-N-15-XX (Fig. 4.1). For the specimen with HSC, G-0.65-H-15-XX, the accumulated energy was 28% more than that of G-0.65-N-15-XX, which similarly explains the more damage observed in this specimen.

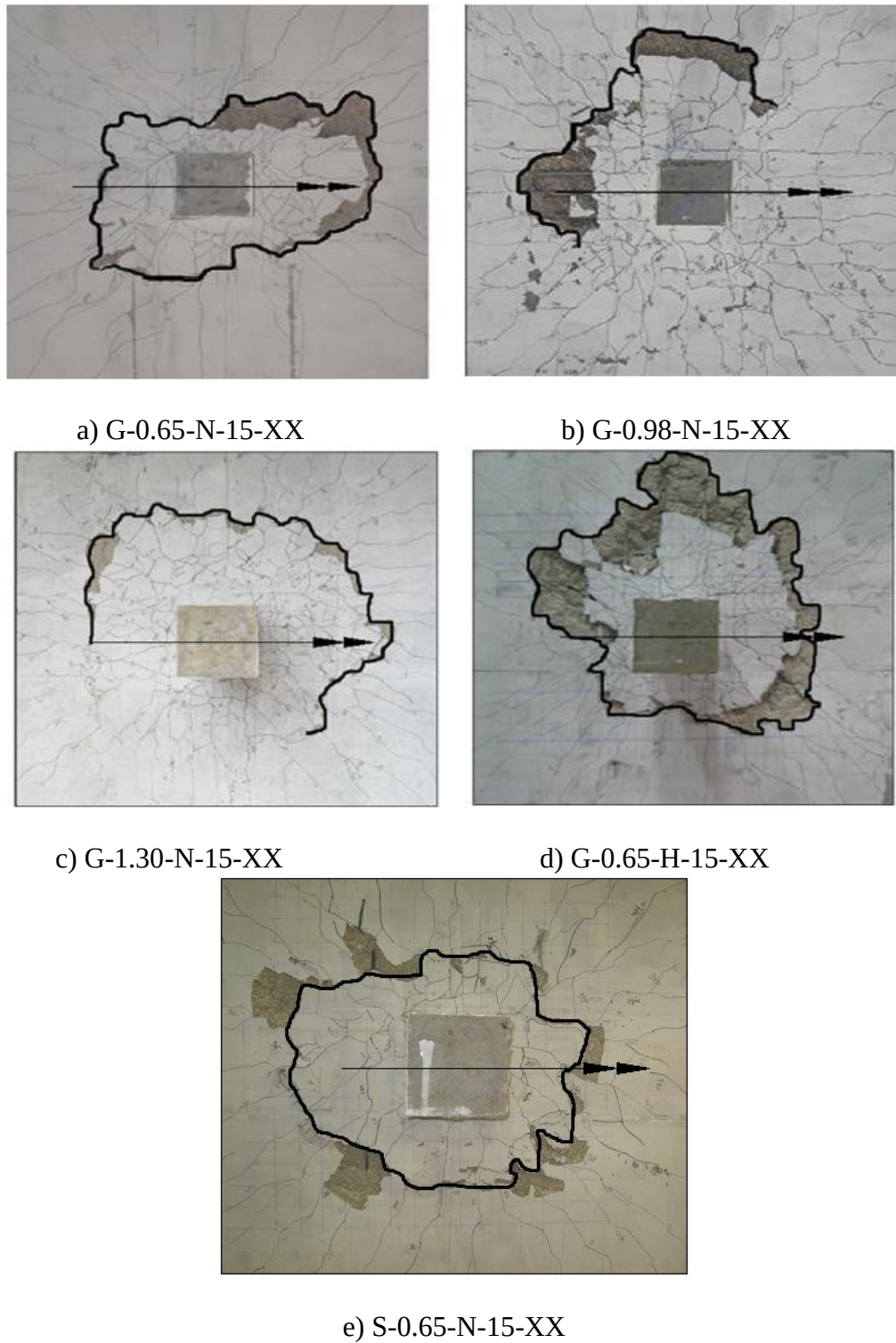
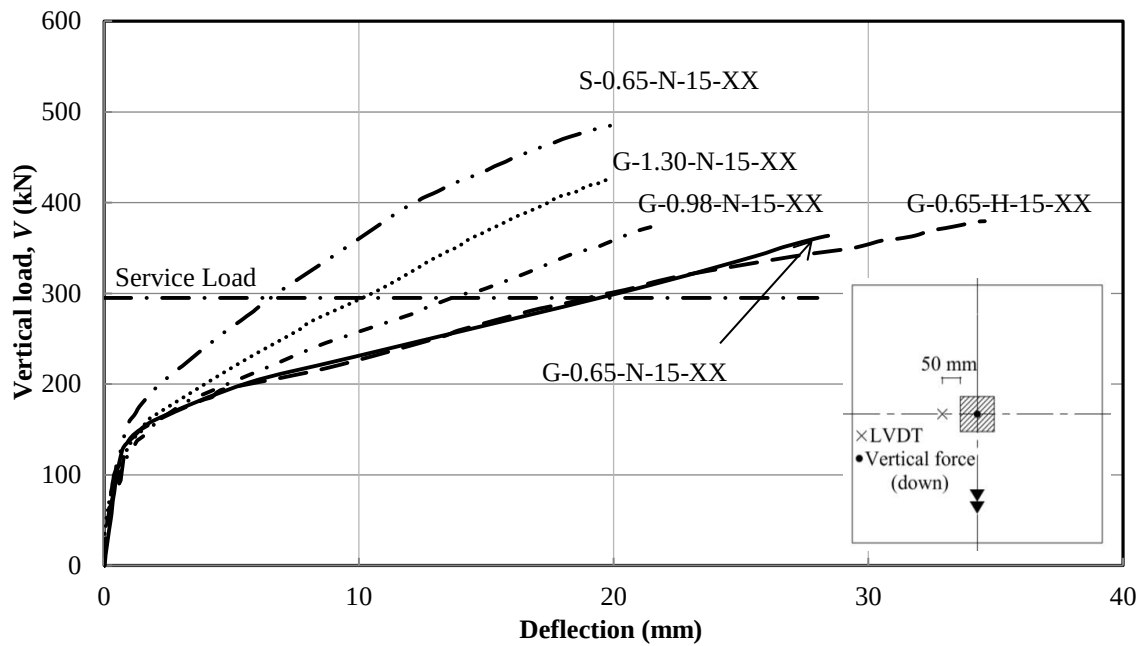
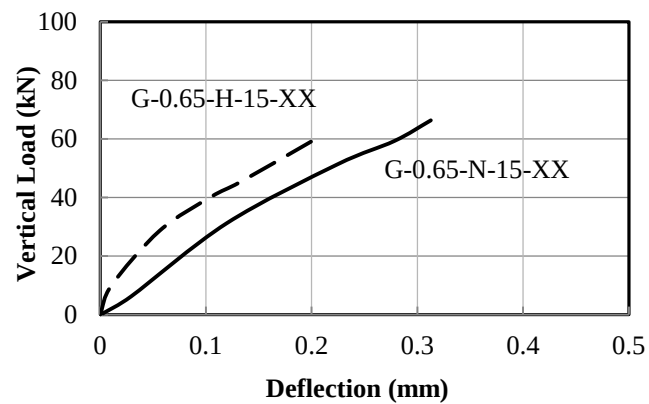


Figure 4.1: Crack patterns on the tension face of the slab at failure for specimens of Series I

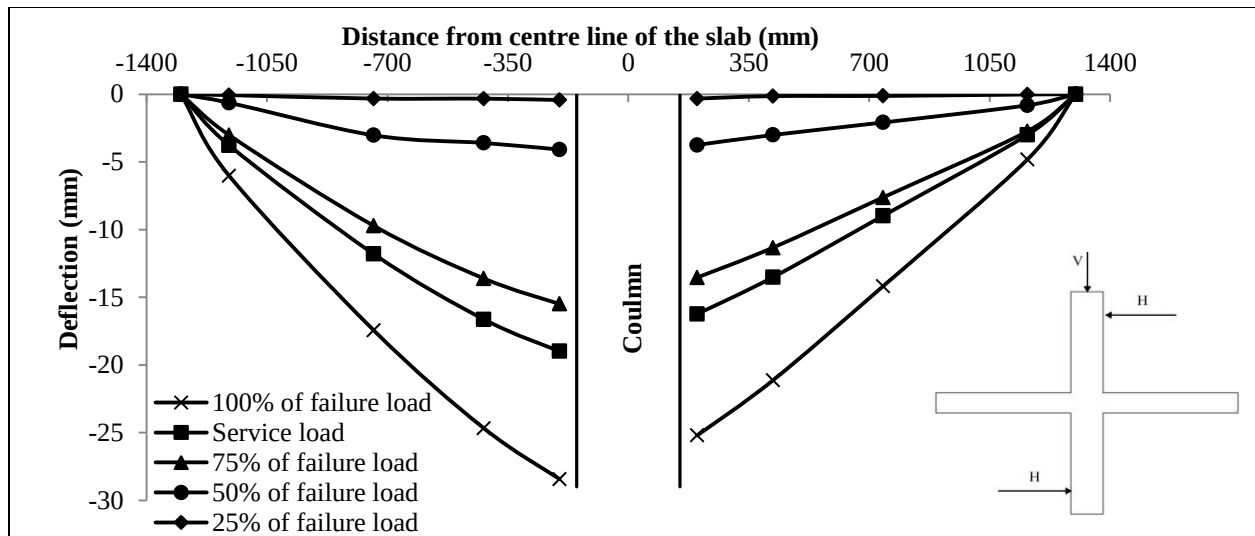


a) Full range relationship

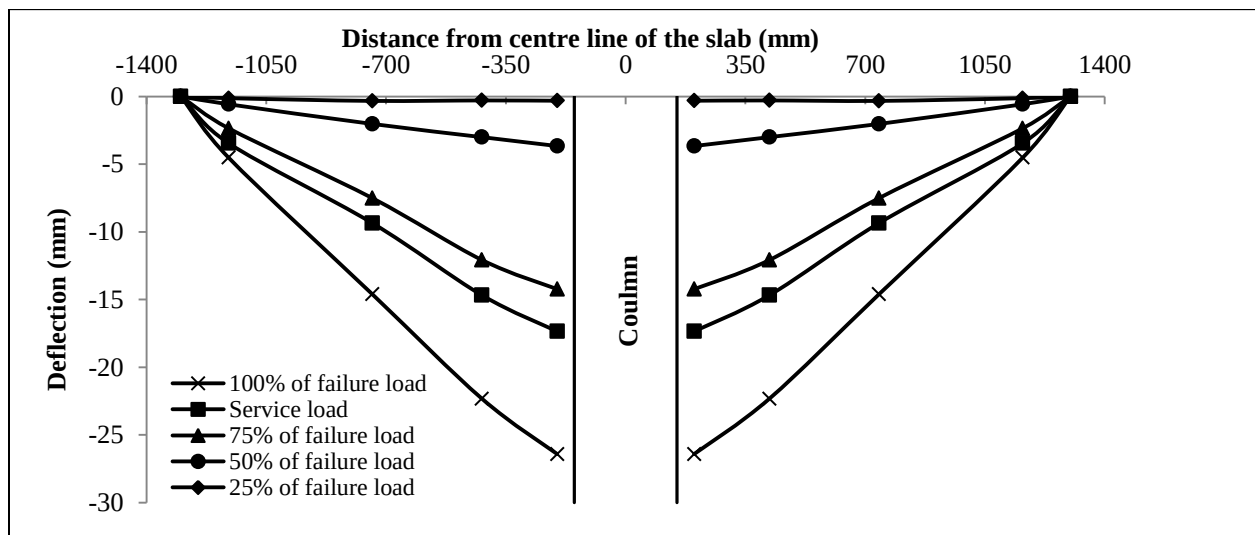


b) At early loading stages for specimens G-0.65-N-15-XX and G-0.65-H-15-XX

Figure 4.2: Load-deflection relationship of specimens of Series I



(a) In the direction of the unbalanced moment



(b) In the direction perpendicular to the unbalanced moment

Figure 4.3: Deflection-profiles for specimen G-0.65-N-15-XX at the centrelines of the slab

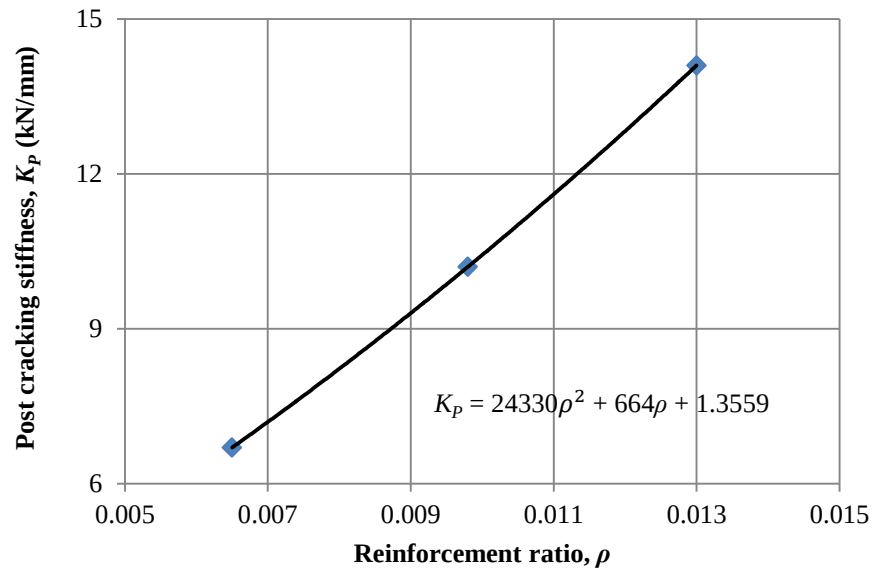


Figure 4.4: Relation between the reinforcement ratio and the post cracking stiffness for the GFRP-slabs with NSC in Series I

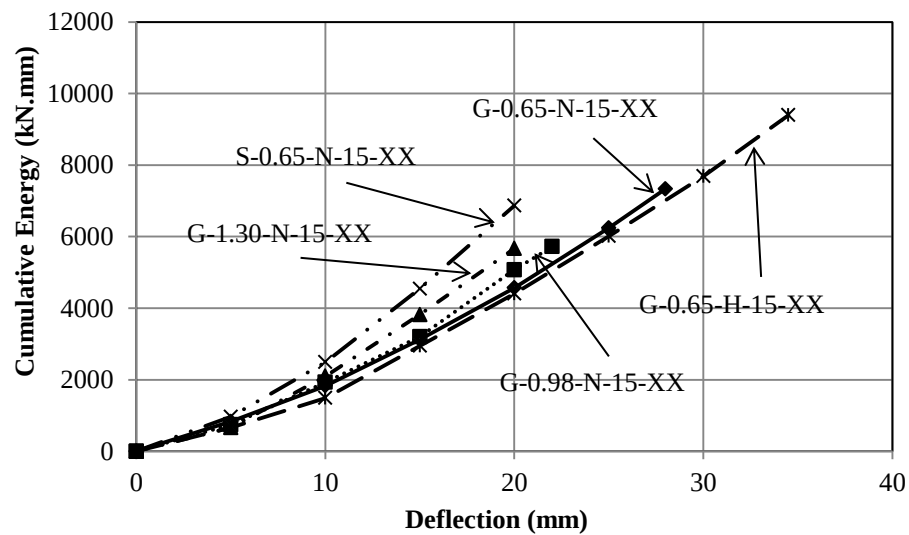


Figure 4.5: Cumulative energy of specimens of Series I

Table 4.1: Test results of Series I

Specimen	First Crack Load (kN)	Maximum Deflection at Failure (mm)	Reinforcement Strain at Failure ($\mu\epsilon$)	Maximum Concrete Strain ($\mu\epsilon$)	Post-Cracking Stiffness, K_P (kN/mm)
S-0.65-N-15-XX	130	20.0	10,370	-300	18.4
G-0.65-N-15-XX	116	28.4	8,700	-510	6.7
G-0.98-N-15-XX	115	22.0	6,790	-650	10.1
G-1.30-N-15-XX	113	19.7	5,910	-800	14.1
G-0.65-H-15-XX	142	34.6	9,850 *	-620	6.9

*At 349 kN (92% of the failure load) where strain gauge malfunctioned.

4.2.3 Strains in Specimens of Series I

Figure 4.6 shows the relationship between the vertical load and the maximum measured strain in the reinforcing bars and concrete at the column face. Generally, the strain in the reinforcing bars was insignificant up to the first crack in each specimen. As the cracks extended in the slab, the strains in the bars started to increase rapidly. The maximum measured reinforcement strain for specimens reinforced with GFRP bars was 9850 $\mu\epsilon$ (in specimen G-0.65-H-15-XX). This is approximately 48% of the ultimate tensile strength of the used GFRP bars. Also, the maximum concrete strains, measured at 50 mm from the column face, were well below the specified crushing strain of concrete in the current codes and guidelines (Table 4.1). This indicates that failure was due to punching shear before the slabs reached the flexural capacity.

On the other hand, for the steel-RC specimen (S-0.65-N-15-XX), the reinforcement at column face started to yield at a vertical load of 420 kN, which is approximately 86% of the failure load.

Prior to steel yielding, specimen S-0.65-N-15-XX showed the least reinforcement strain compared to the other specimens at the same load level due to the higher elastic modulus of steel bars compared to that of GFRP bars. At service load, the strain in specimen G-0.65-N-15-XX was approximately 192% larger than that in specimen S-0.65-N-15-XX, while at failure and due to steel yielding and higher capacity, specimen S-0.65-N-15-XX showed 12% more strain. Also, at service and failure loads, specimen S-0.65-N-15-XX exhibited smaller concrete strains compared to the counterpart slab reinforced with GFRP bars.

For NSC specimens reinforced with GFRP bars, the specimen with the lowest reinforcement ratio (G-0.65-N-15-XX) showed the highest reinforcement strain after cracking due to its lower axial stiffness. Also, the GFRP strains at service load decreased by 30 and 54% when the reinforcement ratio was increased by 50 and 100%, respectively. These percentages decreased at the ultimate load to be 22 and 32%, respectively. In addition, the specimens with the higher reinforcement ratios (G-0.98-N-15-XX and G-1.30-N-15-XX) exhibited less concrete strain at the same load level with reference to specimen G-0.65-N-15-XX with the lowest reinforcement ratio. On the other hand, the maximum measured concrete strains increased at failure when the reinforcement ratio was increased, as shown in Fig. 4.6.

The specimen with HSC showed the least reinforcement strain at the early loading stage, due to its high tensile strength (Table 3.1), which delayed the formation of cracks. On the contrary, at the post cracking stage, wide cracks were observed at the location of the strain gauge, which caused higher strain values in the reinforcement. Also, the HSC specimen showed higher concrete strain at failure compared to the NSC counterpart, due to the decrease in the depth of neutral axis associated with the wider cracks that were developed in specimen G-0.65-H-15-XX, As mentioned earlier.

The Canadian standard CSA/S806-12 (CSA 2012) limits the maximum stress in the GFRP bars under service loads to 25% of the ultimate strength of the bars, mainly, to control crack width. The maximum GFRP strains at service load were 7140, 4970, 3294 and 7500 $\mu\epsilon$ for specimens G-0.65-N-15-XX, G-0.98-N-15-XX, G-1.30-N-15-XX and G-0.65-H-15-XX, respectively, which approximately account for 35, 24, 16 and 36%, respectively, of the ultimate tensile strength of GFRP bars. The two specimens with the lowest reinforcement ratio, G-0.65-N-15-XX and G-0.65-H-15-XX, did not meet CSA/S806-12 (CSA 2012) serviceability requirements.

Figures 4.7 to 4.11 provide the strain profiles at the centreline of the specimens in the direction of the unbalanced moment. These figures show that the strains in the reinforcing bars, either steel or GFRP, decreased as going farther from the column face which indicates that no bond slippage occurred. This agrees with the findings of Dulude et al. (2013). Moreover, the strain profiles of the reinforcement are not symmetrical about the column stub due to the effect of unbalanced moment. At failure, the smaller strain (Fig. 4.7, SG2) at the column face in the direction of the unbalanced moment was less by 48, 50, 41 and 11% for specimens G-0.65-N-15-XX, G-0.98-N-15-XX, G-1.30-N-15-XX and S-0.65-N-15-XX, respectively, compared to the maximum measured values at the location of SG1 (Fig. 4.7). For specimen G-0.65-H-15-XX, this value was 60% at 92% of the failure load.

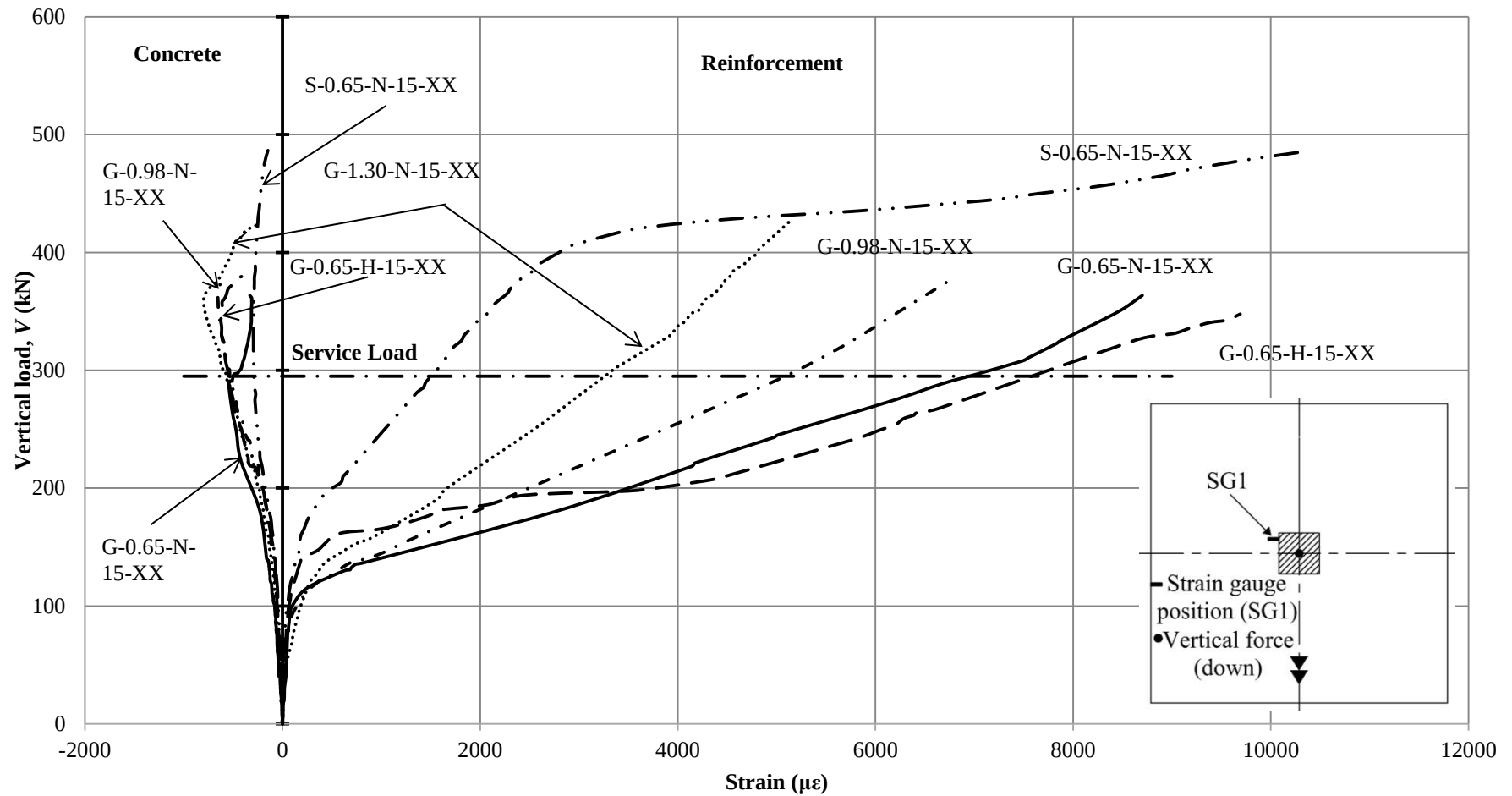


Figure 4.6: Load- strain relationship of specimens of Series I

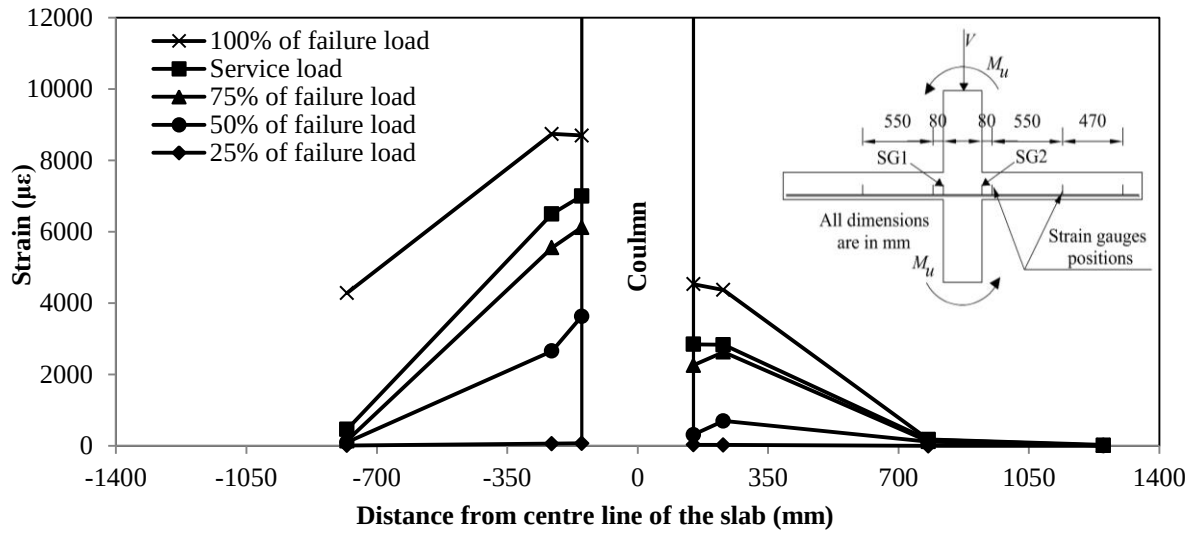


Figure 4.7: Strain profiles for tension reinforcement at the centreline of specimen G-0.65-N-15-

XX

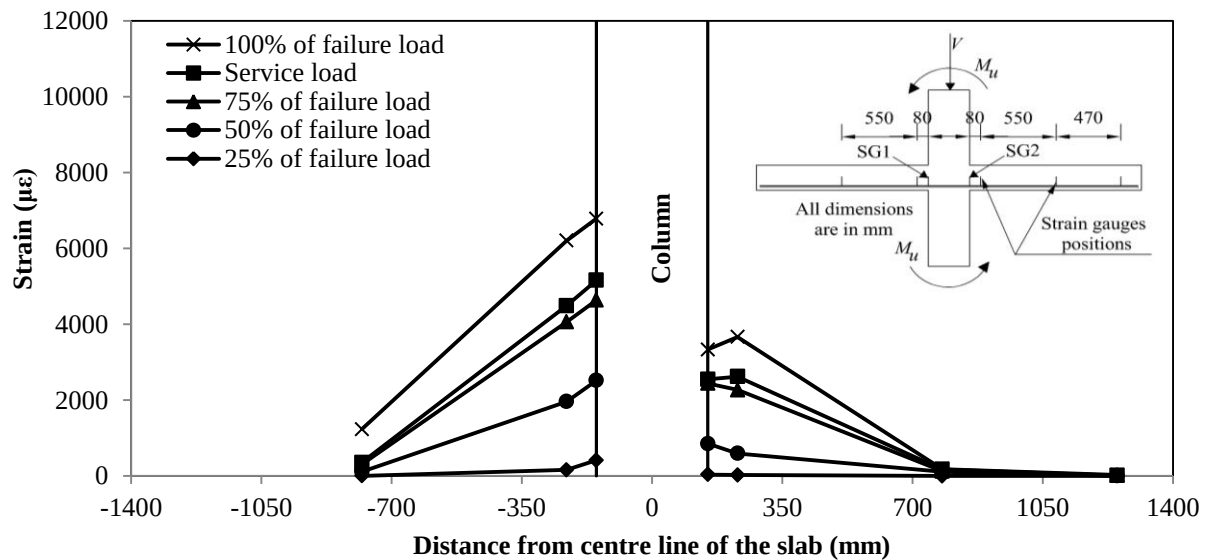


Figure 4.8: Strain profiles for tension reinforcement at the centreline of specimen G-0.98-N-15-

XX

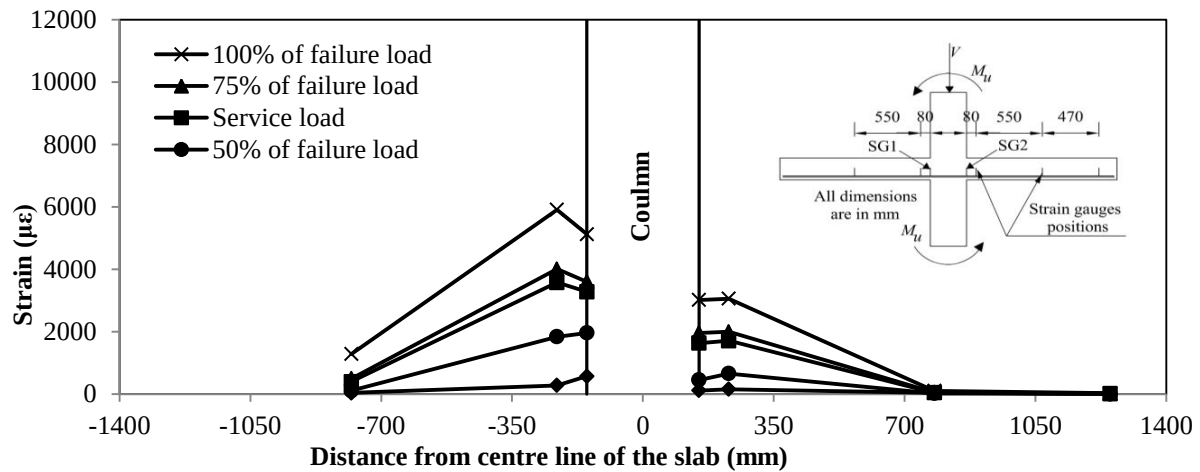


Figure 4.9: Strain profiles for tension reinforcement at the centreline of specimen G-1.30-N-15-XX

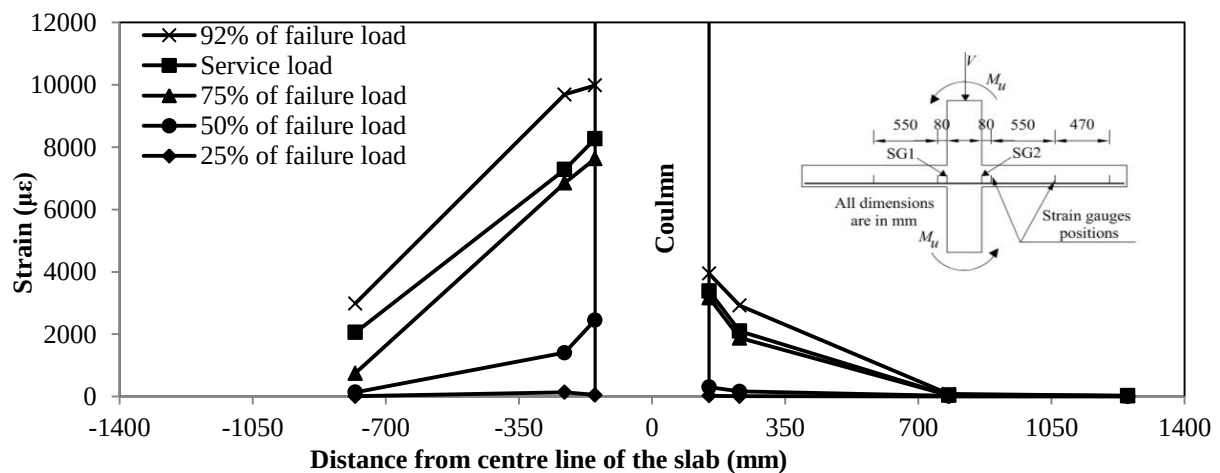


Figure 4.10: Strain profiles for tension reinforcement at the centreline of specimen G-0.65-H-15-XX

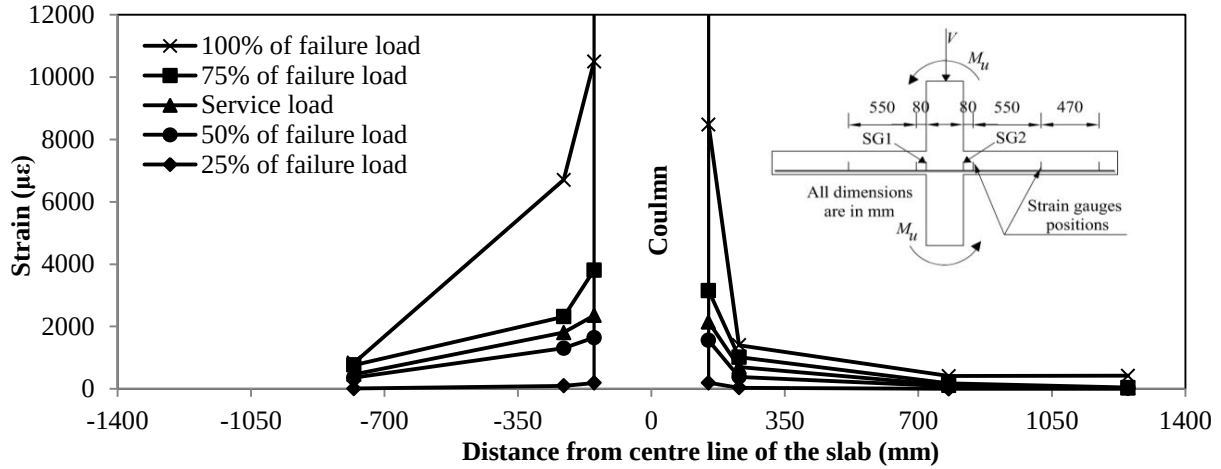


Figure 4.11: Strain profiles for tension reinforcement at the centreline of specimen S-0.65-N-15-XX

4.2.4 Yield-line Pattern

The values of the flexural capacity, V_{flex} , of the specimens are presented in Table 4.2. These values were calculated based on the yield-line pattern shown in Fig. 4.12, which assumes the slab corners are free to lift (Hognestad 1953). The selected yield-line pattern also considers the effect of unbalanced moment by assuming different deflections for column corners and different pattern around the centreline of the slab in the direction of the unbalanced moment. The moment corresponding to yielding of steel was used to calculate the capacity of the steel-RC slab, S-0.65-N-15-XX. For the GFRP-RC specimens, a plastic moment, M_p , rather than a yielding moment, was calculated according to the method described by Gar et al. (2014). In this method, the plastic moment for each GFRP-RC specimen is equal to half the flexural capacity of the slab in the direction of moment transfer, M_n , plus 15% of the cracking moment, M_{cr} .

$$M_p = 0.5 M_n + 0.15 M_{cr} \quad [4.1]$$

Comparing the calculated flexural capacity of the specimens, V_{flex} , to the experimental failure loads shows that all specimens failed in punching shear. The mean value for the specimens reinforced with GFRP bars is $V_{norm.}/V_{flex.} = 0.62 \pm 0.05$ (COV = 7.3%).

4.2.5 Ultimate Strength of Specimens of Series I

To account for the variation of the concrete compressive strength of the specimens, the failure loads were multiplied by $\sqrt[3]{42/f'_c}$ as shown in Table 4.2, where 42 MPa is the concrete compressive strength for two of the four NSC specimens. Also, the cubic root of the concrete strength was used instead of the square root, commonly used in these cases; to follow the provisions of the Canadian standard CSA/S806-12 (CSA 2012). In addition, test results showed that specimen G-0.65-N-15-XX had 25% lower capacity than its counterpart specimen S-0.65-N-15-XX with same reinforcement ratio. However, specimen G-1.30-N-15-XX with double reinforcement ratio had 10% lower capacity compared to S-0.65-N-15-XX. This was expected due to the smaller dowel resistance and the relatively-low modulus of elasticity of GFRP bars compared to that of steel ($E_F/E_S = 0.34$). These GFRP properties caused wider cracks and smaller depth to neutral axis in the GFRP-RC specimen, which in turn, reduced the shear resistance provided by both aggregate interlock and un-cracked concrete.

Furthermore, increasing the reinforcement ratio of the slabs by 50 and 100% increased the capacity of the specimens by 7.7 and 20%, respectively. The specimens with the higher reinforcement ratios showed lower strain in the reinforcement, which in turn, decreased the crack width and increased the depth to neutral axis leading to an enhancement in the shear capacity. On the other hand, increasing the concrete compressive strength had less effect on the capacity of the specimens. The specimen with high strength concrete, H-0.65-N-15-XX, had a slightly higher

ultimate load (4.6%) than its counterpart with NSC (N-0.65-N-15-XX). This is in good agreement with results reported by other researchers, where, Hassan et al. (2013) found 7% increase in the punching load when the concrete compressive strength was increased by 98% for FRP-RC slabs tested under concentric load. In addition, Marzouk et al. (1998) reported similar results for steel-RC slabs, where increasing the concrete compressive strength by 114% enhanced the ultimate loads by 8% and 5% for concentric and eccentric load, respectively.

Figure 4.13 depicts the relationship between the axial stiffness of the slabs reinforced with GFRP bars (G-0.65-N-15-XX, G-0.98-N-15-XX, G-1.30-N-15-XX and G-0.65-H-15-XX) and the normalized shear stresses. The latter was obtained by dividing the ultimate punching shear stresses, calculated at $0.5 d$ (80 mm) from the column face, by the cubic root of the concrete compressive strength for each specimen. Increasing the axial stiffness of the slab reinforcement by 100%, from 442 to 884 MPa, increased the average normalized shear stresses by 27%, from 0.46 to 0.58. Moreover, using the method of least squares, the following interpolation equation is introduced to find the mathematical correlation between the normalized punching shear stresses and the axial stiffness of the GFRP-RC slabs.

$$v_u / \sqrt[3]{f'_c} = 0.0551 (E_F \rho_F)^{0.35} \quad [4.2]$$

The normalized punching shear stress is found to be proportional to the axial stiffness of the slab to the power of 0.35, with a slope (constant) value in the interpolation equation of 0.055. This result is in very good agreement with the punching shear equation of the Canadian code CSA/S806-12 provisions [Eq. 2.50]. Eq. [4.2] was verified against the experimental results of slabs tested under concentric loads (Hassan et al. 2013). The obtained results were $V_{exp.}/V_{Pre.} = 1.06 \pm 0.08$ (COV = 7.5%). This ratio was $V_{exp.}/V_{CSA.} = 1.17 \pm 0.06$ (COV = 5.1%)

when the Canadian code, CSA/S86-12 (CSA 2012), was used. This indicates that Eq. [4.2] better predicts the punching capacity of the slabs than that of the Canadian code.

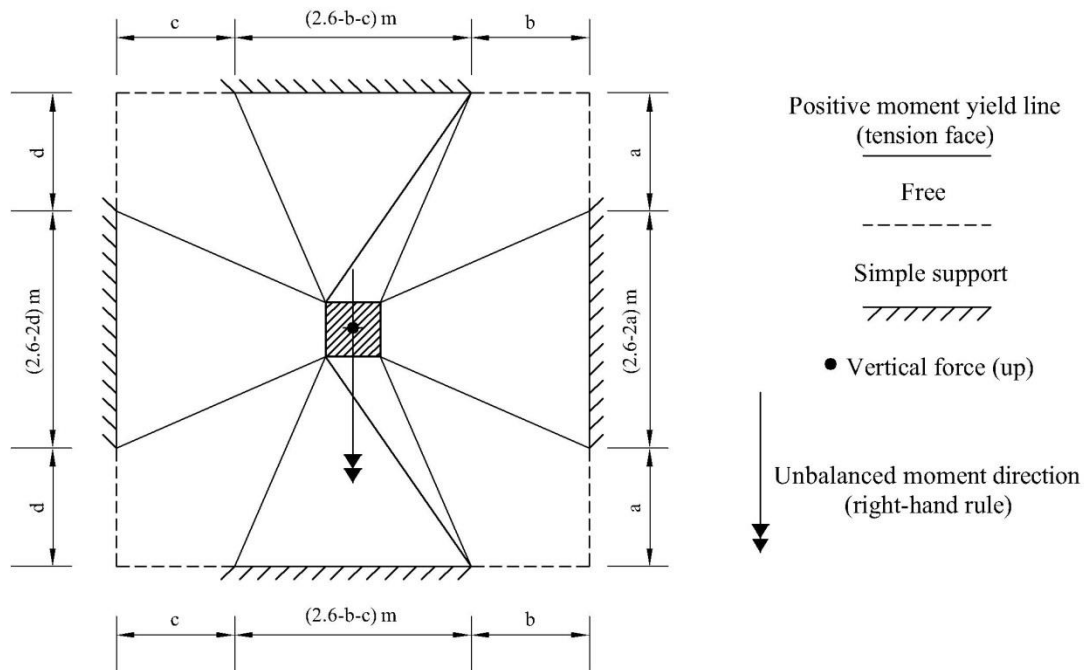


Figure 4.12: Yield-line pattern

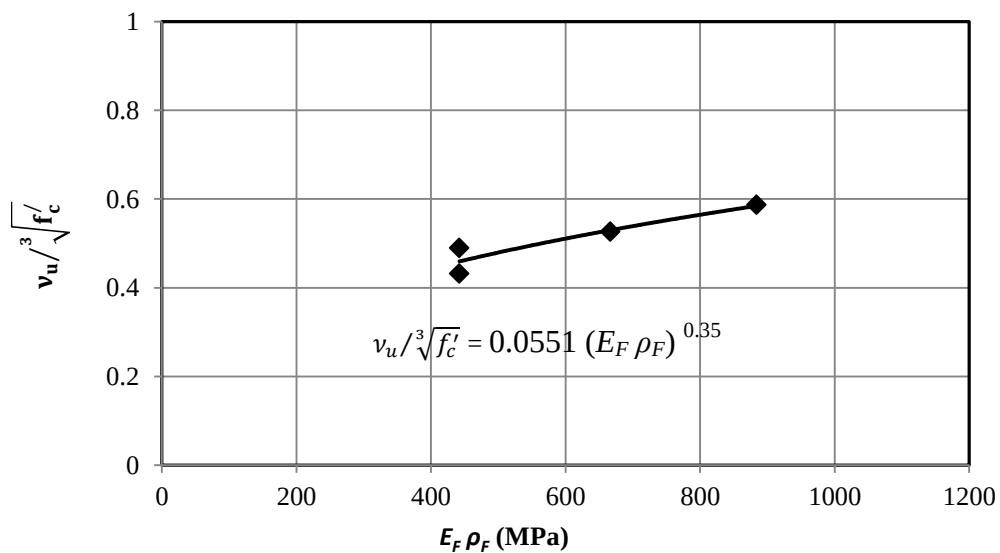


Figure 4.13: Normalized shear stress-axial stiffness relationship for GFRP-RC reinforced slabs of Series I

Table 4.2: Failure loads, punching shear stresses and flexural capacities of specimens of Series I

Specimen	Failure Loads, $V_{exp.}$ (kN)	$V_{exp.} \sqrt[3]{\frac{42 (MPa)}{f'_c}}$ $V_{norm.}$ (kN)	Punching Shear Stresses (MPa)*	Flexural Capacities, (yield line)	
				$V_{flex.}$ (kN)	$V_{norm.}/V_{flex}$
S-0.65-N-15-XX	486	486	2.27	575	0.84
G-0.65-N-15-XX	363	363	1.70	536	0.67
G-0.98-N-15-XX	378	391	1.77	624	0.63
G-1.30-N-15-XX	425	436	1.99	690	0.63
G-0.65-H-15-XX	380	-	1.78	675	0.56

* calculated at 0.5 d from the face of the column

4.2.6 Predicted Punching Loads for the GFRP Specimens of Series I

Several codes, especially in North America, calculate the punching shear stress, v_u , induced from the combination of shear force and unbalanced moment at distance $d/2$ from the column face. For specimens without shear reinforcement, the shear stress is based on the following:

$$v_u \leq \phi \cdot v_c \quad [4.3]$$

Where

$$v_u = \frac{V}{b_o d} + \frac{\gamma_v M_y}{J_c} x \quad [4.4]$$

Number of equations was proposed to predict the nominal shear strength of FRP-RC slab-column connections, v_c . In general, most of these equations are empirical in nature and based on the design provisions for steel-RC structures from different codes and guidelines, with modifications to account for the properties of FRP bars.

The predicted values in the current study were obtained from four sources. The first is the Canadian standard for FRP-RC structures CSA/S806-12 (Section 2.4.5.1); the second is the American guideline ACI 440.1R-06 (Section 2.4.5.2), the third is the Japanese code JSCE-1997 (Section 2.4.5.3). The last one is the equation proposed by Zaghoul (2002) (Section 2.6.1). Since the latter is the only research that studied the effect of the unbalanced moment, it may be beneficial to compare the predicted values, using Zaghoul's equation [Eq. 2.56], to the punching loads of the slabs tested in the current study.

For the ACI code equation, a modification has been made to account for the eccentric load. The shear stress for each specimen was calculated by dividing the ultimate capacity obtained from the equation [Eq. 2.51] by the effective depth, d , and the perimeter of the critical section calculated at $0.5 d$ from the column faces. The outcome stress was used to calculate the eccentric load for the specimen.

The predicted values for the punching shear capacity of the FRP-RC test specimens and the ratios to the actual failure loads are given in Table 4.3. All material resistance and strength factors in the equations were set equal to unity. In general, the CSA/S806-12 standard (CSA 2012) and JSCE code (JSCE 1997) gave reasonable estimate to the experimental failure loads of the test specimens with an average of $V_{exp.}/V_{CSA.} = 1.10 \pm 0.03$ (COV = 2.7%) and $V_{exp.}/V_{JSCE.} = 1.13 \pm 0.07$ (COV = 6.2%), respectively. However, the ACI 440.1R-06 (ACI 2006) predictions were more conservative with an average of $V_{exp.}/V_{ACI.} = 1.93 \pm 0.12$ (COV = 6.2%). On the other hand, Zaghoul's equation (2002) overestimated the capacity of the specimens by an average of $V_{exp.}/V_Z. = 0.86 \pm 0.04$ (COV = 4.65%).

The ACI code equation highly underestimated the punching loads of the specimens. The equation included the effect of the reinforcement ratio and the elastic modulus of the reinforcing bars in calculating the depth to neutral axis, and subsequently, the compute of the punching capacity of the specimens. Whereas, the direct implementation of the axial stiffness of the specimens showed very good predication of the failure loads as in the case of CSA/S806-12 code (CSA 2012) and that might be one of the reasons for the conservative prediction of the ACI equation. Similar to CSA/S806-12, Zaghoul's equation (2002) included the axial stiffness of the reinforcing bars and the concrete compressive strength to the power of 0.333. However, the predicted values overestimated the actual failure loads. This might be due to size effect since Zaghoul's equation was based on tests that were carried out on half-scale specimens.

Although, JSCE code (JSCE 1997) gave close conservative prediction to the failure loads of the specimens. The HSC specimen (G-0.65-H-0.15-XX) showed exactly the same predicted punching load for the corresponding one with NSC (G-0.65-N-0.15-XX) due to the limitation of the concrete compressive strength through the factor f_{pcd} .

It should be noted that the predicted capacity for the HSC specimen using CSA/S806-12 was based on the 60 MPa limit. However, using the actual concrete strength of the HSC specimen (70 MPa) yielded better results with $V_{exp.}/V_{CSA.} = 1.01$. This is in good agreement with the results of other researchers (Hassan et al. 2013). So, it seems that the concrete compressive strength limit set by the CSA/S806-12 code (CSA 2012) could be extended. However, further investigation is still needed to confirm this issue.

Table 4.3: Predicted loads of the GFRP-RC slabs of Series I

Specimen	CSA/S806-12 (2012)*		ACI 440.1R-06 (2006)		JSCE (1997)		Zaghloul (2002)	
	V_{CSA} (kN)	$V_{exp.}/V_{CSA}$	V_{ACI} (kN)	$V_{exp.}/V_{ACI}$	V_{JSCE} (kN)	$V_{exp.}/V_{JSC}$	V_{ZR} (kN)	$V_{exp.}/V_Z$
G-0.65-N-15-XX	316	1.15	172	2.11	313	1.16	394	0.92
G-0.98-N-15-XX	350	1.08	201	1.88	359	1.05	437	0.86
G-1.3-N-15-XX	388	1.10	230	1.84	394	1.08	484	0.88
G-0.65-H-15-XX	356	1.07	197	1.92	313	1.21	467	0.81
Mean		1.10		1.93		1.12		0.86
SD		0.03		0.12		0.07		0.04
COV (%)		2.70		6.20		6.25		4.65

* Calculated using Eq. [2.50]

4.3 SERIES II AND III-THE EFFECT OF MOMENT-TO-SHEAR RATIO AND SHEAR REINFORCEMENT

Up to date, no research data on the effect of moment-to-shear ratio or the presence of shear stud reinforcement on the behaviour of full-scale GFRP-RC slab-column connections. This research partially filled this gap. Four specimens G-0.65-N-15-XX, R-0.65-N-15-XX, R-0.65-N-15-75 and R-0.65-N-15-50 were tested with 0.15-m moment-to-shear ratio, while, that ratio was 0.3 and 0.0 m for specimens G-0.65-N-30-XX and G-0.65-N-00-XX, respectively.

4.3.1 Cracking and Failure Mode of Specimens of Series II and III

Approximately the same cracking pattern and sequences described in Section 4.2.1 are applicable here also. The cracking load was between 28 and 36% of the ultimate capacity of the tested slabs as listed in Table 4.4. The first crack started from the corner of the column and extended toward the corner of the slab then followed by similar radial cracks initiated from the column face and continued to the slab edges in the direction of the horizontal forces, i.e., direction of the unbalanced moment. With the increasing load, other radial cracks with different orientations spread in the slabs. At approximately 50% of the ultimate capacity of the slab, the first peripheral crack appeared connecting the radial cracks together. All specimens failed in punching shear, where the column penetrated through the slab thickness. Photos for the tested slabs at failure are shown in Fig. 4.14. It was observed that the unbalanced moment caused more damage to half of the tension side of the slab (the top part of the photos) compared to the other half, which was more pronounced in the slab with 0.3-m moment-to-shear ratio (G-0.65-N-30-XX).

To investigate the propagation of the diagonal shear cracks that led to failure of the slabs, a specimen from each series (G-0.65-N-00-XX and R-0.65-N-15-50) was sawed at the column face as shown in Fig. 4.15. The main diagonal shear crack extended from the column face up to

the level of the flexure reinforcement in slab G-0.65-N-00-XX then propagated along the reinforcing mat (horizontal splitting crack) before punching out on the tension face. This is a typical failure crack for slabs without shear reinforcement as reported by other researchers (Hassan et al. 2013). Meanwhile, the failure shear crack traveled, directly, to the extreme tension side in case of slab R-0.65-N-15-50. The difference in behaviour between the two slabs can be attributed to the presence of shear studs that arrested the propagation of the diagonal shear cracks and forced the failure plane to occur closer to the column face. The unbalanced moment has no effect on that difference since specimen R-0.65-N-15-50 was sawed in the direction perpendicular to the moment. For slab G-0.65-N-00-XX, the average inclination angle with the horizontal axes, measured at the slab mid-depth, was 42° . However, this angle was only 37° in slab R-0.65-N-15-50, which support the above explanation. This is in good agreement with the findings of previous studies for both FRP-RC (Zaghloul 2002) and steel-RC slabs (Vaz et al. 2009).

The slabs that had shear studs (R-0.65-N-15-50 and R-0.65-N-15-75) failed inside the shear reinforcement zone. Also, it was noticed that the heads for the shear studs, at all investigated locations, were damaged as shown in Fig. 4.16. In addition, at approximately 90 to 95% of the failure load, the measured strain in some of the shear studs started to decrease, which indicates a probable slippage due to local failure at the locations of these shear studs. Consequently, the failure of the slabs could be initiated due to the damage occurred in the shear stud heads.

Moreover, slabs R-0.65-N-15-XX and G-0.65-N-15-XX showed similar cracking pattern, which indicates that there was no significant effect of the surface texture of GFRP bars on the slab cracking. Also, Fig. 4.14 shows that the distribution of cracks in the slabs with ribbed-deformed GFRP bars (R-0.65-N-15-XX, R-0.65-N-15-50 and R-0.65-N-15-75) was similar, however,

using the shear studs seem to increase the cracks density, which might be due to the higher capacity of the slabs with transverse reinforcement. In addition, increasing the moment-to-shear ratio decreased the failure surface and the crack spacing as shown in Fig. 4.14. This reduction was due to the concentration of the stresses associated with the unbalanced moment near the column face. These observations are in good agreement with test results of previous studies (Marzouk et al. 2000; Zaghloul 2002).

4.3.2 Deflection of Specimens of Series II and III

Figure 4.17 shows the maximum deflection in the slabs measured at 50 mm from the column face versus vertical load. The typical load-deflection relationship includes two stages. The first stage represents the behaviour of the uncracked concrete, while the second stage represents the reduced flexural stiffness of the cracked slabs. The load-deflection curves for the six specimens were similar up to the first crack. After initiation of cracks, the behaviour was affected by the moment-to-shear ratio as well as the shear reinforcement. Increasing the moment-to-shear ratio decreased the post cracking flexural stiffness (K_p), calculated as the slope of the load-deflection curve after cracking, which in turn increased the deflection at the same load level. Increasing the moment-to-shear ratio from zero to 0.15 and 0.3 m decreased the post cracking stiffness by 6 and 18%, respectively. Furthermore, in the post cracking stage, the increase in deflection at the same load level was 9 and 37% when the moment-to-shear ratio was increased from zero to 0.15 and to 0.3 m, respectively.

The presence of the shear studs increased the post-cracking stiffness of the slabs (Table 4.4) and, slightly, reduced the deflection at the same load level, especially for slab R-0.65-N-15-50. This might be attributed to the ability of the shear studs to control the propagation of the diagonal

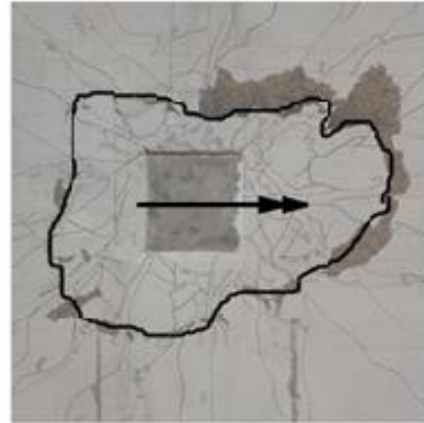
shear cracks which, in turn, enhanced the overall behaviour of the slabs with transverse reinforcement.

Table 4.4: Test results of Series II and III

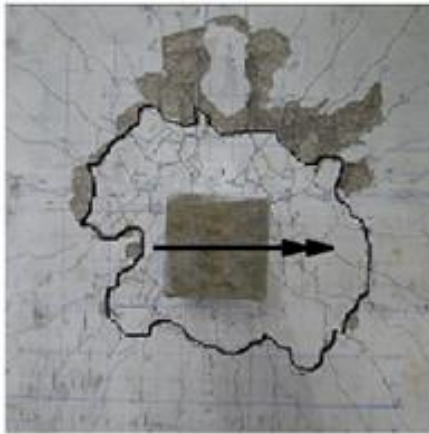
	Specimen	First cracking load (kN)	Maximum deflection at failure (mm)	Reinforcement strain at failure ($\mu\epsilon$)	Maximum concrete strain ($\mu\epsilon$)	Post-cracking stiffness, K_P , (kN/mm)
Series II	G-0.65-N-00-XX	120	33.7	8,110	-500	7.1
	G-0.65-N-15-XX	116	28.4	8,700	-510	6.7
	G-0.65-N-30-XX	102	23.8	8,540	-620	5.8
Series III	R-0.65-N-15-XX	108	26.2	7,110	-870	6.6
	R-0.65-N-15-75	118	28.5	8,810	-1070	9.8
	R-0.65-N-15-50	113	30.1	9,200	-1330	9.4



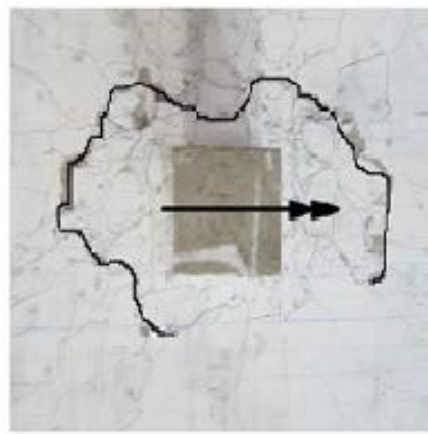
a) Connection G-0.65-N-00-XX



b) Connection G-0.65-N-15-XX



c) Connection G-0.65-N-30-XX



d) Connection R-0.65-N-15-XX



e) Connection R-0.65-N-30-50



f) Connection R-0.65-N-15-75

Figure 4.14: Crack patterns on the tension face of the slab at failure for specimens of Series II and III

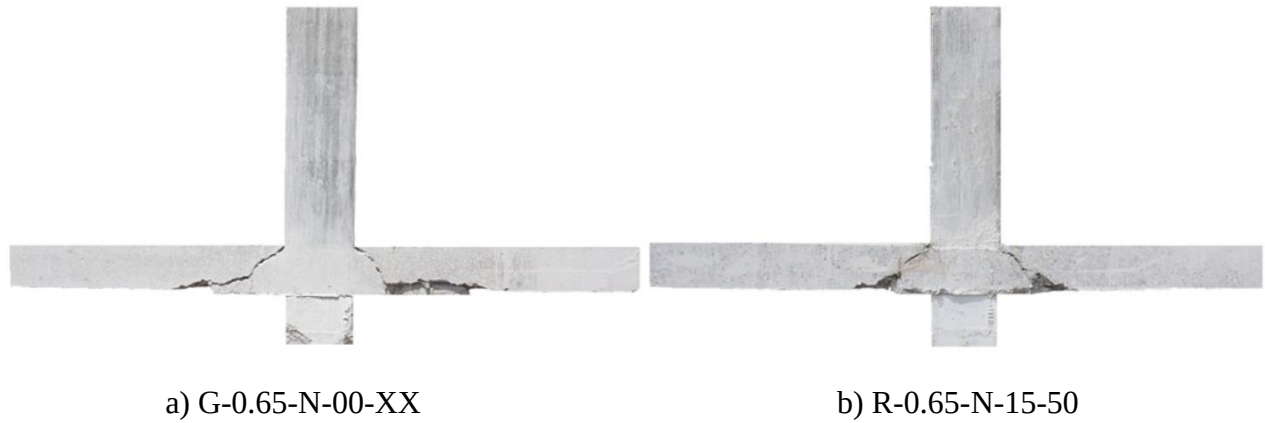
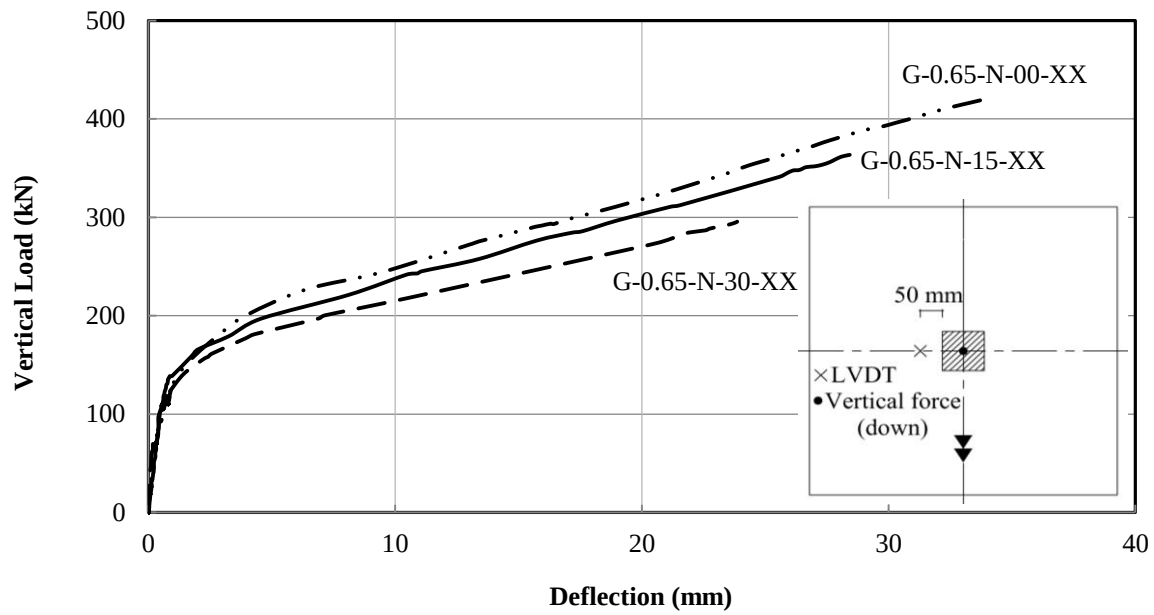


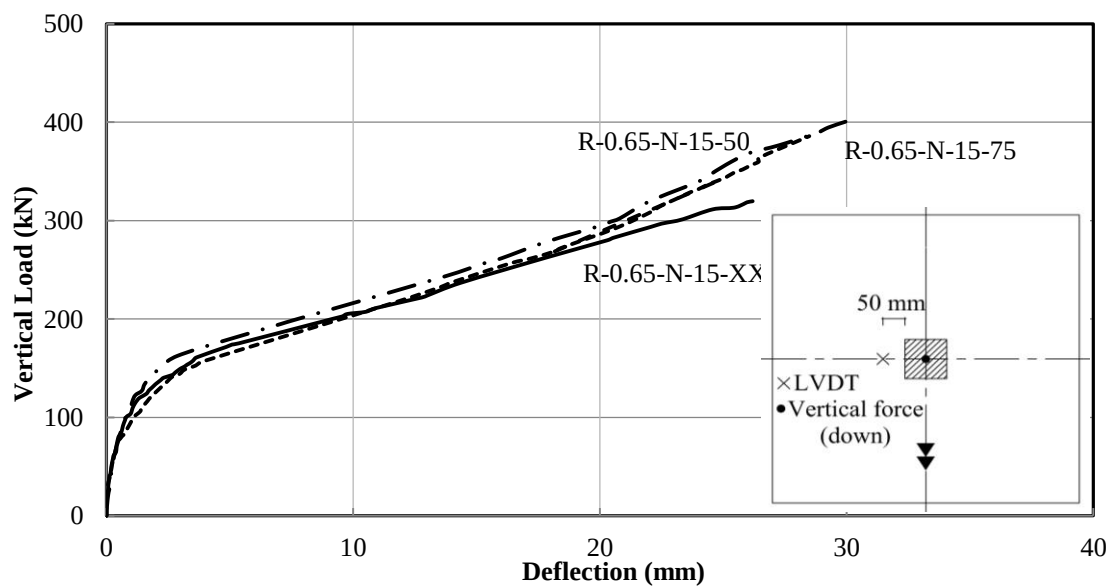
Figure 4.15: Diagonal shear cracks



Figure 4.16: Damaged heads for the GFRP shear studs



a) Specimens of Series II



b) Specimens of Series III

Figure 4.17: Load-deflection relationship of specimens of Series II and III

4.3.3 Strains in Specimens of Series II and III

Flexural reinforcement strains:

The relationship between the strain in the reinforcing bars at the column face and the vertical load is presented in Fig. 4.18 and 4.19. In general, the strain readings were insignificant at the early loading stages. Once cracks started to propagate in the slabs, the strains increased at a higher rate. The maximum measured reinforcement strain for the slabs reinforced with sand-coated and ribbed-deformed bars was 8700 and 9200 $\mu\epsilon$, respectively, as listed in Table 4.4. This represents approximately 42 and 55% of the rupture strain of the two types of the bars, respectively.

Regarding the effect of the moment-to-shear ratio, it was observed that increasing the moment-to-shear ratio increased the strains in the GFRP bars. For instance, at the same load level within the post-cracking stage, the strains in slabs G-0.65-N-15-XX and G-0.65-N-30-XX were 21 and 45% higher than that in specimen G-0.65-N-00-XX, respectively. However, at failure, these percentages reduced to 7 and 5%, respectively, due to the different capacities of the slabs. Slabs in Series III ($M_u/V = 0.15$) demonstrated comparable reinforcement strains as shown in Fig. 4.19. However, the strain at failure in slabs R-0.65-N-15-50 and R-0.65-N-15-75, with shear studs, was higher than that in slab R-0.65-N-15-XX. This was due to the different capacities of the slabs.

The reinforcement strain profiles for all slabs were similar regardless the variation in the moment-to-shear ratio or the presence of shear studs. Figure 4.20 illustrates the typical strain profiles in the direction of the unbalanced moment at the centreline of slabs R-0.65-N-15-XX and R-0.65-N-15-75. The strains in the reinforcing bars consistently decreased as moving further from the column face, which indicates that no bond slippage took place in the GFRP bars. Also,

due to the unbalanced moment, the reinforcement strains had higher values in the direction of moment application. For example, at failure, the reinforcement strain at location SG1 (Fig. 4.20) was 48, 89 and 48% higher than that at location SG 2 (Fig. 4.20) for slabs G-0.65-N-15-XX, G-0.65-N-30-XX and R-0.65-N-15-XX, respectively. This percentage was 14% for slab R-0.65-N-15-75 at 94% of the ultimate load (strain gauge SG2 malfunctioned at 360 kN vertical load). The largest difference in strains around the column was observed for slab G-0.65-N-30-XX since this slab was tested under the highest moment-to-shear ratio ($M_u/V = 0.3$ m).

Concrete strains:

The maximum measured concrete strains for the slabs listed in Table 4.4 indicate that the specimens did not fail in flexure due to concrete crushing. Figures 4.18 and 4.19 depict the relation between the vertical load and the concrete strain measured at the column face. Increasing the moment-to-shear ratio increased the measured concrete strain. The maximum measured strain in slabs G-0.65-N-15-XX and G-0.65-N-30-XX increased by 2 and 24%, respectively, compared to slab G-0.65-N-00-XX tested under a concentric load.

For the slabs of Series III, the concrete strains were quite similar up to a vertical load of 200 kN, which represents 62, 52 and 50% of the failure load of slabs R-0.65-N-15-XX, R-0.65-N-15-75 and R-0.65-N-15-50, respectively. Afterwards, the concrete strains in the slabs with transverse reinforcement (R-0.65-N-15-50 and R-0.65-N-15-75) increased rapidly. The maximum measured compressive strain in slabs R-0.65-N-15-50 and R-0.65-N-15-75 were 53 and 23% higher than that of the slab without transverse reinforcement (R-0.65-N-15-XX).

Shear reinforcement strains:

The measured strain in the majority of shear studs was small up to approximately 70 to 75% of the failure loads of the slabs (270 to 300 kN). Afterwards, the strain started to increase rapidly until it reached a maximum of 2100 and 2230 $\mu\epsilon$ at failure for slabs R-0.65-N-15-75 and R-0.65-N-15-50, respectively. Also, the higher strain associated with slab R-0.65-N-15-50 could be attributed to the higher failure load of the slab.

Usually, the strain in the shear stud increases when a diagonal shear crack intersects with it. Figure 4.21 illustrates the strain profiles at different loading stages in the direction of the unbalanced moment. Strain distribution in the transverse reinforcement showed that the strain increased with increasing the applied load and the maximum measured strain was in the stud located at a distance of $0.9 d$ and $1.9 d$ from the column face for slabs R-0.65-N-15-50 and R-0.65-N-15-75, respectively. The Canadian standard for steel-RC structures (CSA/A23.3-14) recommends that the transverse reinforcement be extended to a distance of $2.0 d$ from the column face. However, the slabs tested in the current study and in literature (Zaghloul 2002 and 2007, Hassan et al. 2014) showed that shear reinforcement located at distances beyond this limit (Fig. 4.21) was still active, which indicates that this limit needs to be increased in the case of FRP-RC slabs.

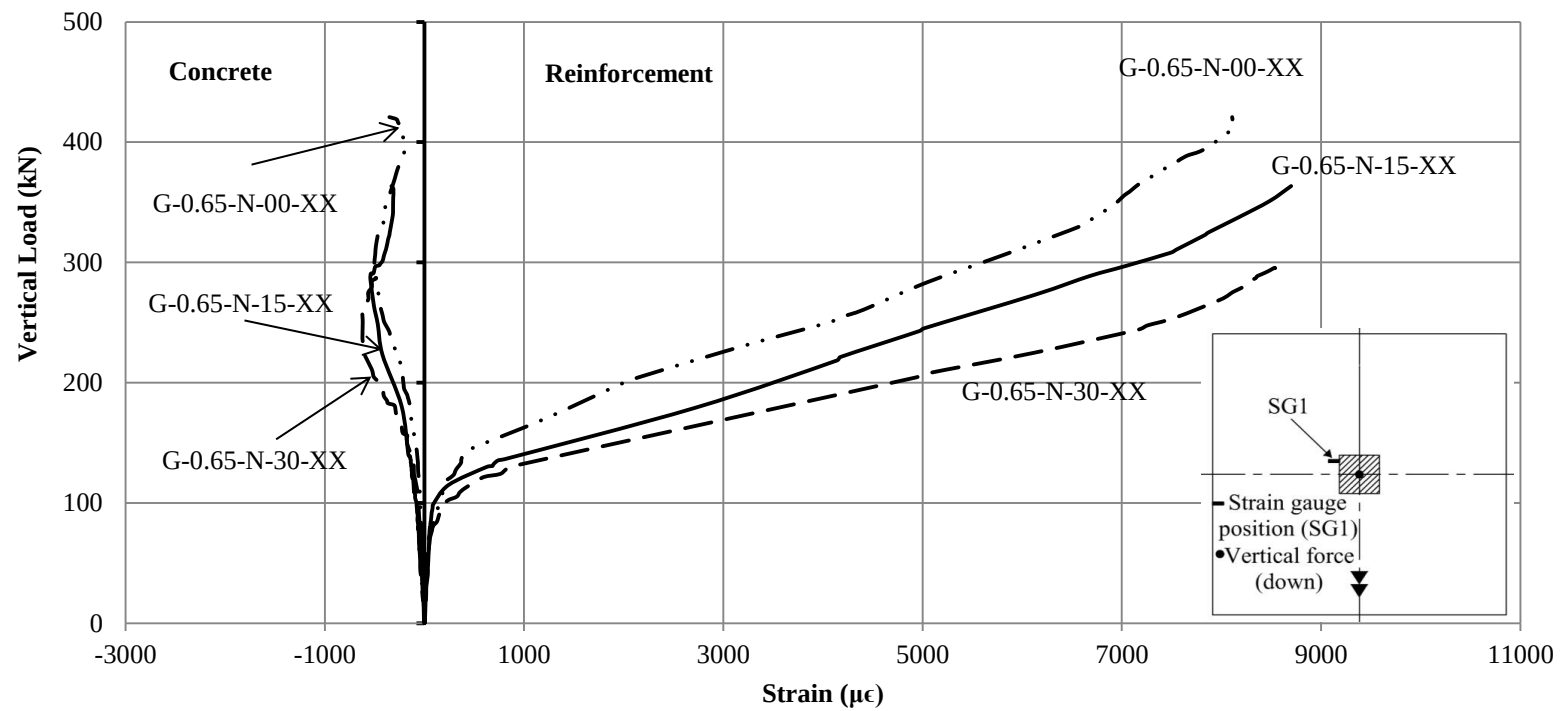


Figure 4.18: Load-strain relationship in specimens of Series II

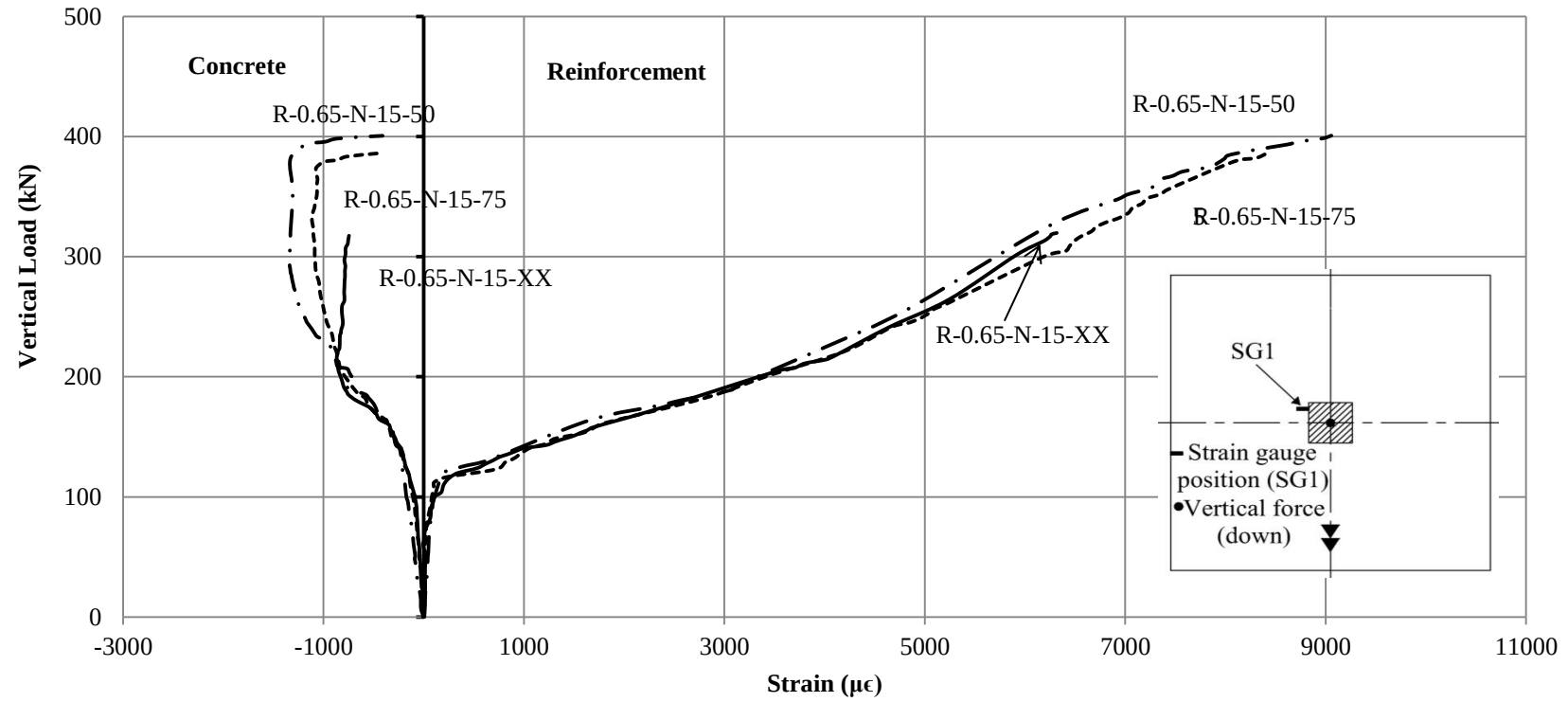
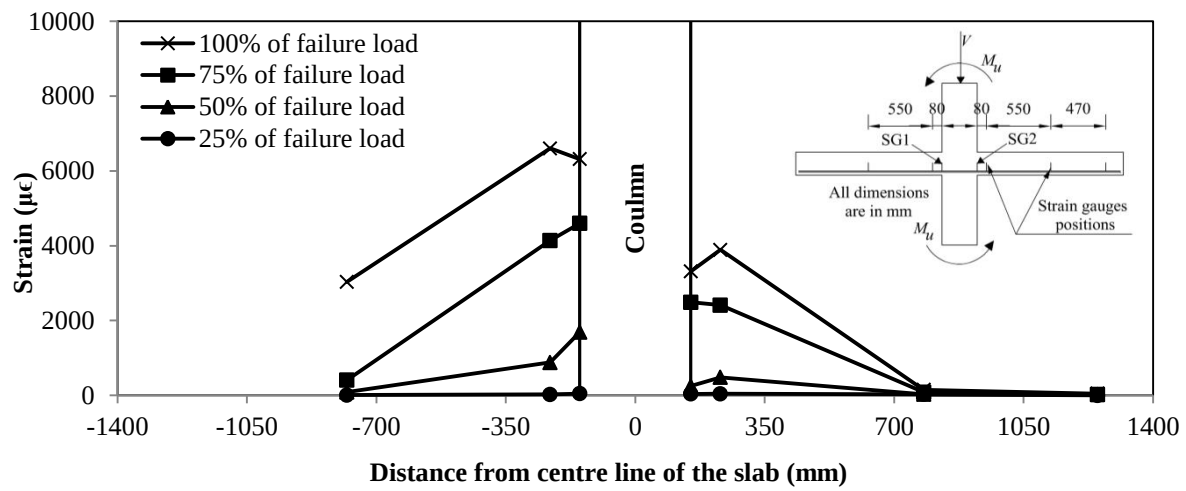
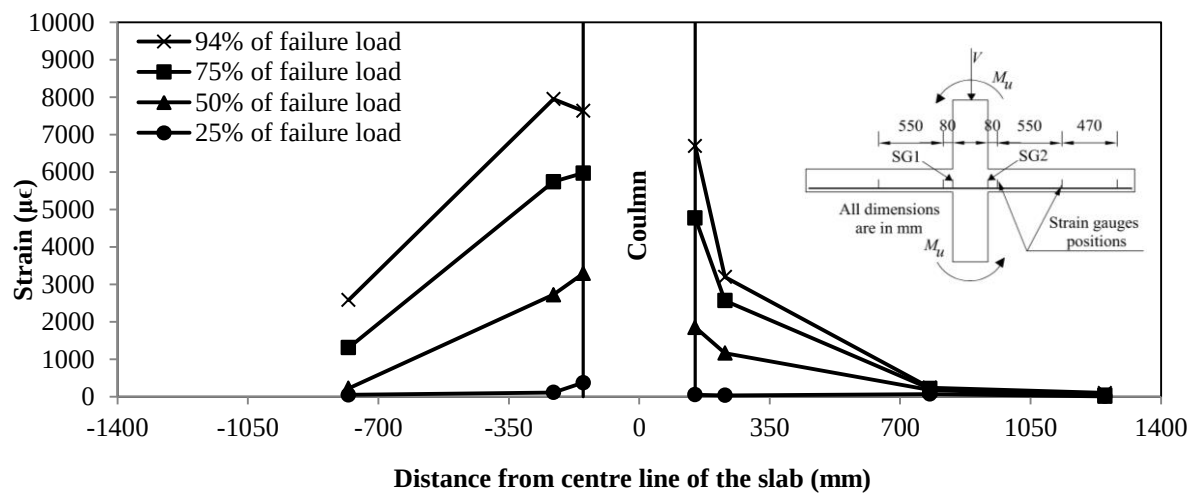


Figure 4.19: Load-strain relationship in specimens of Series III

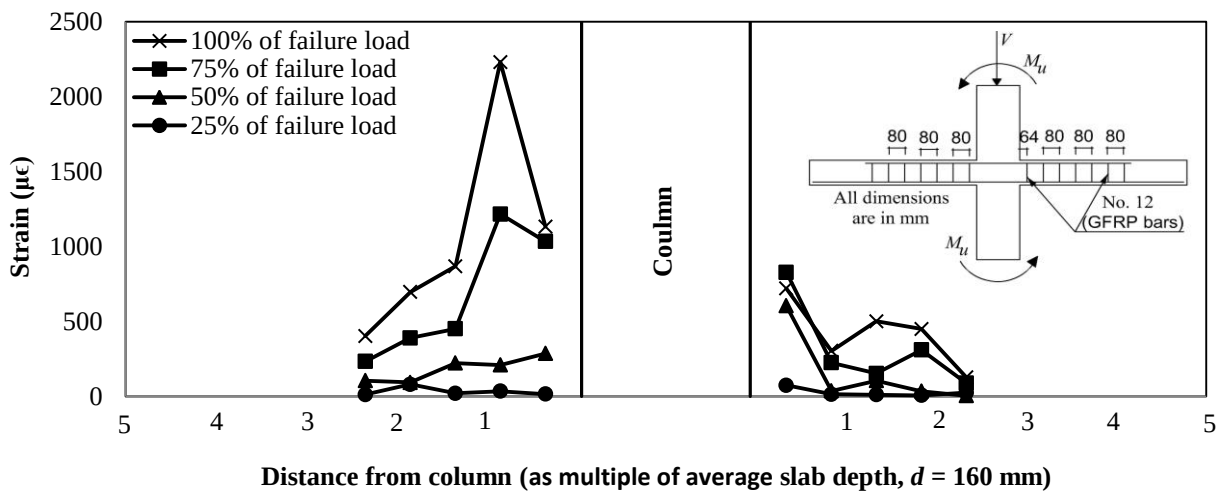


a) Specimen R-0.65-N-15-XX

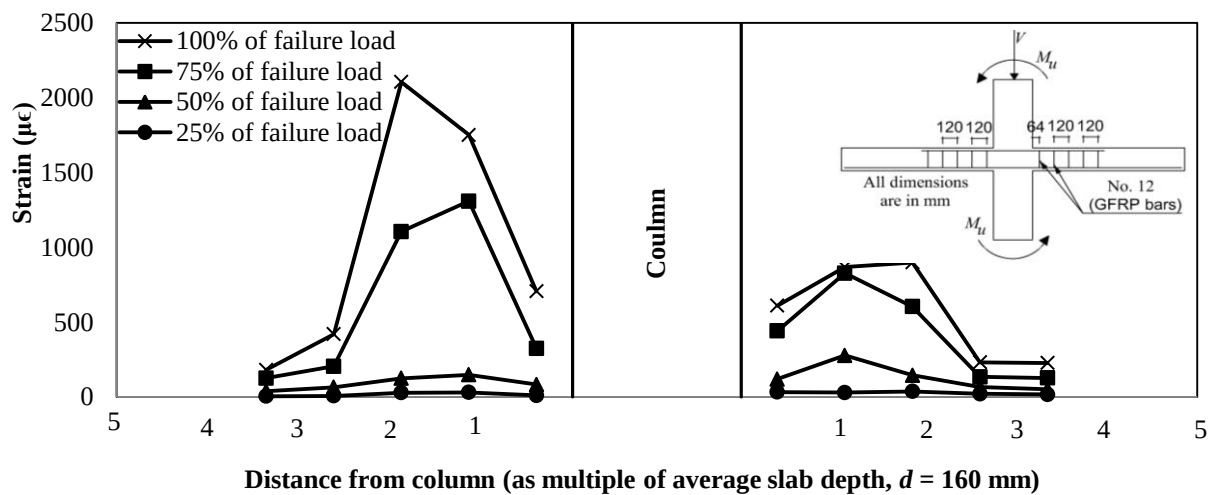


b) Specimen R-0.65-N-15-75

Figure 4.20: Strain profiles for flexure reinforcement in the direction of the unbalanced moment



a) Specimen R-0.65-N-15-50



b) Specimen R-0.65-N-15-75

Figure 4.21: Strain profiles for the shear studs reinforcement in the direction of the unbalanced moment

4.3.4 Ultimate Strength of Specimens of Series II and III

The ultimate capacity and punching shear stress calculated at $0.5 d$ (80 mm) from the column face are listed in Table 4.5. The failure loads were normalized (multiplied by $\sqrt[3]{42/f_c}$) to eliminate the effect of concrete compressive strength variations, where 42 MPa is the concrete compressive strength for the majority of the tested slabs. Also, the calculated vertical load corresponding to the flexural capacity of the specimens, V_{flex} , which represents an upper bound of the shear force that can be transferred between the slab and the column, is listed in Table 4.5. The low vertical shear forces compared to the calculated flexural capacity confirm the punching shear failure mode of the slabs where the mean value for $V_{norm.}/V_{flex}$ was 0.7 ± 0.08 (COV = 11.4%).

As expected, increasing the moment-to-shear ratio reduced the vertical shear force required to cause punching failure. Slabs G-0.65-N-15-XX and G-0.65-N-30-XX exhibited 16.5 and 32% less vertical shear force compared to that of slab G-0.65-N-00-XX, respectively. Also, at failure, the punching shear stress calculated at $0.5 d$ (80 mm) from the column face was 1.43, 1.70 and 1.77 MPa in slabs G-0.65-N-00-XX, G-0.65-N-15-XX and G-0.65-N-30-XX, respectively. Slabs G-0.65-N-15-XX and G-0.65-N-30-XX had 19 and 24% higher stresses with respect to slab G-0.65-N-00-XX, respectively, due to the effect of the unbalanced moment. Similar results were obtained for the FRP-RC slabs tested by Zaghoul (2002), where 42% reductions in the vertical shear force was reported when the moment-to-shear ratio was increased from zero to 0.3 m. In addition, Zaghoul (2002) reported 20% increase in the punching shear stresses for the slab tested with moment-to-shear ratio of 0.3 m compared to the one tested under vertical shear force only in his research. Also, these results are in good agreement with the findings of Marzouk et al.

(2000) for steel-RC connections where a 21% decrease in the vertical load capacity was observed when the moment-to-shear ratio was increased from 0.0 to 0.34 m. Also, the authors found that the shear stresses induced by vertical force plus unbalanced moment were higher than those obtained due to the concentric load only.

Regarding the effect of the shear studs, the presence of the studs enhanced the punching shear capacity. The normalized punching capacity of slabs R-0.65-N-15-50 and R-0.65-N-15-75 was 23.4 and 18.5% higher than that of slab R-0.65-N-15-XX, respectively. These results are in good agreement with previous research, where an increase in the capacity of approximately 30% was observed when different types of FRP shear reinforcement was used; CFRP grids (Zaghloul 2007) and GFRP stirrups (Hassan et al. 2014). However, it seems that the flexural reinforcement ratio, and consequently flexural capacity, has a significant effect on this percentage increase since these ratios decreased to only 7% when the flexural reinforcement ratio decreased from 1.2 to 0.3% (Hassan et al. 2014). Moreover, the maximum measured strain in the shear studs (Fig. 4.21) indicates that the shear studs in both slabs (R-0.65-N-15-50 and R-0.65-N-15-75) carried similar load.

Slab G-0.65-N-15-XX had 11.6% higher normalized capacity compared to the counterpart slab reinforced with ribbed-deformed bars (R-0.65-N-15-XX). This may be attributed to the low modulus of elasticity for the ribbed-deformed, which resulted in reducing the shear capacity provided by aggregate interlock and un-cracked concrete.

Table 4.5: Failure loads, punching shear stresses and flexural capacities of specimens of Series II and III

Specimen		Failure Loads, $V_{exp.}$ (kN)	$V_{exp.} \sqrt[3]{\frac{42 (MPa)}{f_c'}}$, $V_{norm.}$ (kN)	Punching Shear Stresses (MPa)	Flexural Capacities (yield line)	
					$V_{flex.}$ (kN)	$V_{norm.}/V_{flex}$
Series II	G-0.65-N-00-XX	421	435	1.43	547	0.79
	G-0.65-N-15-XX	363	363	1.70	536	0.67
	G-0.65-N-30-XX	296	296	1.77	511	0.58
Series III	R-0.65-N-15-XX	320	325	1.50	522	0.62
	R-0.65-N-15-75	385	385	1.80	522	0.74
	R-0.65-N-15-50	401	401	1.88	522	0.77

4.3.5 Predicted Punching Capacity of Specimens of Series II and III

4.3.5.1 Slabs without shear reinforcement of specimens of Series II and III

The predicted values for the ultimate capacity of the slabs and the ratio between the experimental and predicted failure loads are given in Table 4.6. These values were obtained considering all strength and materials resistance safety factors equal to unity. For ACI 440.1R-06 (ACI 2006), the shear strength (MPa) was obtained by dividing the shear force [Eq. 2.51] by the area of the critical section (the perimeter, $b \times$ the effective depth, d). Generally, CSA/S806-12 (CSA 2012) and JSCE (JSCE 1997) codes gave fairly close, yet conservative, predictions to the failure load of the slabs with an average of $V_{exp.}/V_{CSA.} = 1.10 \pm 0.09$ (COV = 8.2%) and $V_{exp.}/V_{JSCE.} = 1.10 \pm 0.11$ (COV = 10.0%), respectively. On the other hand, the ACI 440.1R-06

(ACI 2006) highly underestimates the capacity with an average of $V_{exp.}/V_{ACI.} = 1.99 \pm 0.17$ (COV = 8.6%).

Table 4.6: Predicted loads for the slabs without shear reinforcement of specimens of Series II and III

Specimen	CSA/S806-12* (2012)		ACI 440.1R-06 (2006)		JSCE (1997)	
	$V_{CSA.}$ (kN)	$V_{exp.}/V_{CSA.}$	$V_{ACI.}$ (kN)	$V_{exp.}/V_{ACI.}$	$V_{JSCE.}$ (kN)	$V_{exp.}/V_{JSCE.}$
G-0.65-N-00-XX	422	1.00	237	1.77	432	0.97
G-0.65-N-15-XX	316	1.15	172	2.11	313	1.16
G-0.65-N-30-XX	248	1.19	1.38	2.14	245	1.21
R-0.65-N-15-XX	304	1.05	165	1.94	306	1.04
Mean		1.10		1.99		1.10
SD		0.09		0.17		0.11
COV (%)		8.20		8.60		10.0

* Calculated using Eq. [2.50]

4.3.5.2 Slabs with shear reinforcement

Up to date, no shear design provisions for flat plates reinforced with FRP transverse shear reinforcement are available. In this thesis, proposed revisions to the design equations of the CSA/S806-12 standard (CSA 2012) and the ACI 440.1R-06 guideline (ACI 2006) are introduced to account for the FRP shear reinforcement.

4.3.5.2.1 Based on the CSA/S806-12 and CSA/A23.3-14 provisions

The Canadian standard for steel-RC members, CSA/A23.3-14, provides two equations to calculate the concrete contribution to the shear resistance for the slab-column connections with shear reinforcement. The shear resistance of the concrete inside [Eq. 2.17] and outside [Eq. 2.16] the shear reinforcement zone is 75 and 50% of that for steel-RC members without transverse reinforcement [Eq. 2.15]. Consequently, Eq. [2.50], which calculates the shear resistance for FRP-RC members without shear reinforcement, is multiplied by 75 and 50% to obtain Equations [4.3] and [4.4] for the concrete contribution inside and outside the FRP shear reinforcement zone, respectively.

$$v_c = 0.041 \lambda \phi_c (E_f \rho_f f'_c)^{1/3} \quad (\text{MPa}) \quad [4.3]$$

$$v_c = 0.028 \lambda \phi_c (E_f \rho_f f'_c)^{1/3} \quad (\text{MPa}) \quad [4.4]$$

Furthermore, to calculate the strength provided by the GFRP shear studs, Eq. [4.5] from the Canadian code CSA/A23.3-14 is modified by replacing, f_y (the yielding strength of the shear studs) with a specified stress in the FRP-studs, f_{frv} . This specified stress is given by Eq. [4.6], in which a usable GFRP strain, ε_{frv} , value of 2000 $\mu\epsilon$ is proposed to agree with the manufacturer's design value of 2175 $\mu\epsilon$ (Table 3.2). This strain limit agrees well with the measured strains in the current study (Fig. 4.21).

$$v_S = \frac{\phi_s A_{vs} f_{frv}}{b S} \quad (\text{MPa}) \quad [4.5]$$

$$f_{frv} = \varepsilon_{frv} \times E_{frv} \quad (\text{MPa}) \quad [4.6]$$

4.3.5.2.2 Based on the ACI -40.1R-06 and ACI-318-14 provisions

Equations [4.7] and [4.8] were obtained in a similar manner to Equations [4.3] and [4.4], to calculate the concrete contribution inside and outside the shear reinforcement zone for FRP-RC members, respectively. In addition, the new equations were based on the mathematical relation for the concrete contribution in slabs reinforced with [Eqs. 2.24 and 2.25] and without [Eq. 2.23] shear reinforcement in the steel-RC code (ACI318 Committee 2014).

$$V = 0.6 \sqrt{f'_c} b_o c \quad (\text{N}) \quad [4.7]$$

$$V = 0.4 \sqrt{f'_c} b_o c \quad (\text{N}) \quad [4.8]$$

Also, the strength provided by the shear studs is calculated according to Eq. [4.5]. The comparison between the experimental and predicted punching capacity, from the current study and previous ones (Zaghloul 2002 and 2007), Table 4.7, reveals that the proposed revised equations based on the Canadian standards gave reasonable predictions to the experimental loads with an average of $V_{exp.}/V_{CSA.} = 1.16 \pm 0.08$ (COV = 6.9%). Nevertheless, the proposed modification to equations provided by the ACI 440.1R-06 (ACI 2006) guideline underestimated the capacity with an average of $V_{exp.}/V_{ACI.} = 1.75 \pm 0.19$ (COV = 10.9%).

Table 4.7: Predicted loads for the slabs with shear reinforcement

Specimen	<u>Proposed equations</u>		<u>Proposed equations</u>	
	Based on CSA/S806-12		Based on ACI440.1R-06	
	V_{CSA} (kN)	$V_{exp.}/V_{CSA}$	V_{ACI} (kN)	$V_{exp.}/V_{ACI}$
R-0.65-N-15-75	331	1.16	233	1.65
R-0.65-N-15-50	383	1.05	250	1.60
ZJF7*	266	1.19	183	1.73
ZJF9*	265	1.23	161	2.03
Mean		1.16		1.75
SD		0.08		0.19
COV (%)		6.90		10.90

* Specimens tested by Zaghloul (2002 and 2007)

CHAPTER 5

NUMERICAL ANALYSIS

5.1 INTRODUCTION

Finite element method has been recognized as one of the most accurate and effective techniques for analyzing complicated structural engineering problems, which provides a convenient and adaptable tool for covering the problems associated with the analysis of reinforced concrete. These problems include cracking of concrete, nonlinear stress strain response of concrete, shrinkage and creep of concrete, rupture of reinforcing bars, interaction between concrete and reinforcing bars.

The aim of this chapter is to use the specialized software package (Cervenka et al. 2013), which is based on finite element method, to model reinforced concrete slab-column specimens, in a three dimension space, taking into account the material nonlinearity of concrete in addition to the perfect elastic behaviour of the GFRP bars. To achieve that, FE model was constructed to predict the performance of the connections reinforced with GFRP bars. The efficiency and accuracy of the constructed model were verified against the experimental results. The analysis included tracing the load deformation response and the determination of GFRP bars strain.

The following section provides essential information about the program and the elements used to model concrete, FRP and steel bars. Also, more information can be found elsewhere (Cervenka et al. 2013).

5.2 MATERIAL PROPERTIES AND ELEMENTS TYPES

5.2.1 Reinforcing Bars

A truss element (*CCIsoTruss*) was used to model the reinforcing bars with transition degrees of freedom in x , y , z directions at the element nodes. The reinforcement was modelled as individual sub-elements within the concrete elements. Rebar sub-element stiffness is superimposed on the concrete element stiffness in which the rebar resides. The geometry and the nodes of the truss element are shown in Fig. 5.1.

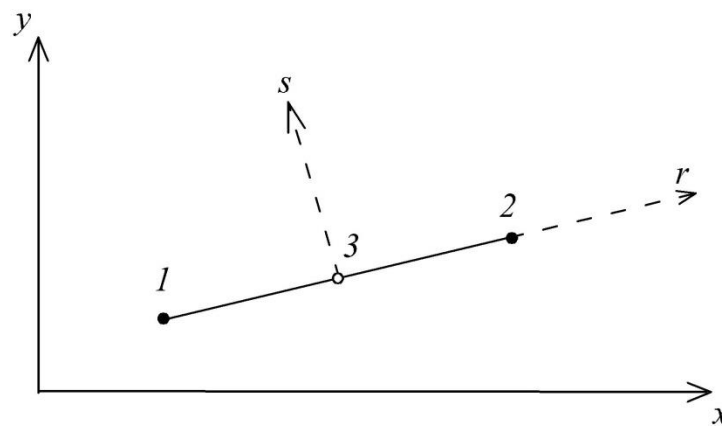


Figure 5.1: Truss element

Two material types were used for the reinforcing bars, one for the sand-coated GFRP bars to simulate the slab reinforcement and the other for the steel reinforcement to simulate the longitudinal bars and stirrups of the column. Perfectly linear elastic stress-strain relationship was defined for the GFRP reinforcement using the mechanical properties documented in Table 3.2. Also, bi-linear isotropic material was used for the steel reinforcement with 200-GPa tensile modulus and 400-MPa yield stress. Figure 5.2 shows the stress-strain relationships for both materials that were used in the numerical study.

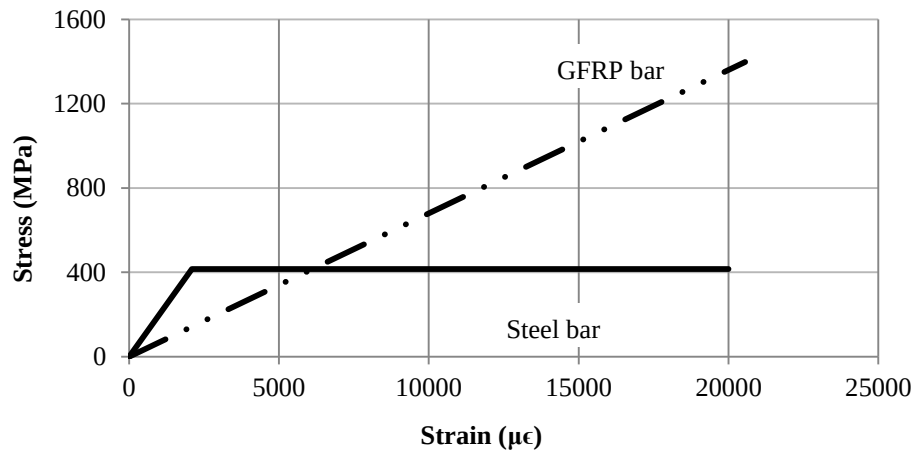


Figure 5.2: Stress-strain relationship

5.2.2 Concrete Material

The program uses different types of elements to model the concrete, such as brick element and tetra element. Each element consists of number of nodes with three translation degrees of freedom in x , y , z directions at each node. In the current study, instead of brick elements, tetra elements (*CCIsoTetra*) with three-dimensional nodes were used to simulate the concrete since there is a limitation on the number of brick elements that can be used.. Material nonlinearity, crushing, plastic deformation and cracking in x , y , z directions are the basic assumptions considered by the element.

5.2.3 Stress and Strain

Behaviour of concrete is highly nonlinear, especially, with low tensile strength and depends on crack widths and compressive capacity. Modeling of concrete material, especially, under conditions where extensive damage can develop requires advanced and detailed constitutive models.

The mathematical correlations of the constitutive relations are considered in the plane stress state. The following equation of equilibrium in a material point describes the stiffness of the constitutive model.

$$s = D \varepsilon \quad [5.1]$$

Where:

$$s = \text{stress vector} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix}, \quad \varepsilon = \text{strain vector} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} \quad \text{and} \quad D = \text{material stiffness matrix}$$

The components of the stress vectors are σ_x , σ_y , τ_{xy} . In addition, the corresponding ones for the strain vector are ε_x , ε_y , γ_{xy} .

Where:

$$\begin{aligned} \sigma_x &= \text{normal stress in the } (x) \text{ direction,} & \sigma_y &= \text{normal stress in the } (y) \text{ direction,} \\ \tau_{xy} &= \text{shear stress in plane } (xy), & \varepsilon_x &= \text{strain in } (x) \text{ direction,} \\ \varepsilon_y &= \text{strain in } (y) \text{ direction,} & \gamma_{xy} &= \text{the engineering shear strain.} \end{aligned}$$

The program divides the material stiffness matrix (D) into two components, one represents the concrete contribution (D_c) and the other is for the reinforcement (D_s) and the same for the stress vector (S), a component for the concrete (S_c) and another one for the reinforcement (S_s).

5.3 CONCEPTS OF MATERIAL MODEL CC3DNonLinCementitious2

In this study, a built-in fracture-plastic constitutive model called *CC3DNonLinCementitious2* is used to define the concrete. The fracture model employs Rankine failure criterion, while the plastic model is based on Men  trety-Willam failure surface (Cervenka et al. 2013). This fracture-plastic model was developed to combine both fracture and plastic models into a single model that can use plasticity to simulate concrete crushing and fracture mechanics to simulate cracking.

This model accounts for material nonlinearity, crushing, plastic deformation and cracking in x , y , z directions. The main features of that model are as follows.

- 1- Non-linearity in compression including softening and hardening.
- 2- Cracking of concrete in tension based on the non-linearity fracture mechanics.
- 3- Biaxial strength failure criterion.
- 4- Reducing the compressive strength after cracking
- 5- Tension stiffening effect
- 6- Reduction of the stiffness in shear after cracking.

According to the model a nonlinear elastic approach is used to derive the material stiffness matrix. That elastic approach is coming from stress-strain formula called the equivalent uniaxial law. A detailed description of the approach will be given in the next section.

5.3.1 Stress-Strain Relations for Concrete

5.3.1.1 Equivalent uniaxial law

The diagram for the equivalent uniaxial stress-strain for concrete is shown in Fig. 5.3. Effective stress, σ^{ef} , and the equivalent uniaxial strain, ε_{eq} , represent the behaviour of the concrete in the stress-strain state. The equivalent uniaxial strain is described as the strain, which would be produced by the stress σ_i , in a uniaxial test with modulus E_i , associated with the direction i . This definition means that the damage is caused only by the governing stress σ_i , which is the principle stress. Also, concrete behaviour is assumed to be perfectly elastic in the uncracked zone. In addition, the tangent modulus that used to build the stiffness matrix for the concrete, for the iterative trials, is defined as the slope of the stress-strain curve at a given strain and it's always with a positive sign.

For the ascending part of the stress-strain curve (Fig. 5.3), the program uses the mathematical correlation [Eq. 5.2] introduced by CEB-FIP Model Code (CEB-FIP 1990). This formula is appropriate to be used for all types of concrete from normal to high strength concrete. Also, it gives wide framework for the curve shapes such as linear, parabolic, cubic, etc.

$$\sigma_c^{ef} = f_c^{/ef} \frac{kx - x^2}{1 + (k-2)x}, x = \frac{\varepsilon}{\varepsilon_c}, k = \frac{E_0}{E_1} \quad [5.2]$$

$$f_c^{/ef} = \frac{1+3.65a}{(1+a)^2} f_c' \quad [5.3]$$

$$a = \frac{\sigma_{c1}}{\sigma_{c2}} \quad [5.4]$$

Where:

$f_c^{/ef}$ = effective concrete compressive strength σ_c^{ef} = concrete compressive stress

x = normalized strain ε = strain k = shape parameter

ε_c = strain at the peak stress $f_c^{/ef}$ E_0 = initial elastic modulus

E_1 = secant elastic modulus at the peak stress, $E_1 = \frac{f_c^{/ef}}{\varepsilon_c}$

σ_{c1} and σ_{c2} = principle stresses

Parameter, k , is the vector responsible for the shape of the ascending branch of the stress-strain relationship. For instance, replacing it with number one will give linear relationship while using two will result in a parabolic shape. Also, the program considers some distributed damage before and after the stress reaches the peak value.

After the peak compressive stress is reached, the softening law is defined in the program to be descending with linear relationship. The model assumes that the compression failure takes place in a plane perpendicular to the direction of the maximum compressive stress (principle stress) and the energy dissipation and compressive peak displacement lay in that plane. The strain ε_d of

the descending part is defined by parameters called w_d (plastic displacement) and L_d (band size) and is obtained from Eq. [5.5]. The value of w_d , was found to be 0.5 mm (Cervenka et al. 2013) for normal strength concrete. In addition, the band size is defined as the projection of the element on the failure plane, which is assumed to be normal to the direction of the compressive principle stresses.

$$\varepsilon_d = \varepsilon_c + \frac{w_d}{L_d} \quad [5.5]$$

For tension after cracking the following formula [Eq. 5.6] is used by the program to calculate the crack opening for the concrete, w , which was derived experimentally by Hordijk (1991).

$$\frac{\sigma}{f_t^{/ef}} = \left\{ 1 + \left(c_1 \frac{w}{w_c} \right)^3 \right\} \exp \left(-c_2 \frac{w}{w_c} \right) - \frac{w}{w_c} (1 + c_1^3) \exp(-c_2) \quad [5.6]$$

$$w_c = 5.14 \frac{Gf}{f_t^{/ef}} \quad [5.7]$$

$$Gf = 0.000025 f_t^{/ef} \quad [5.8]$$

Where:

σ = normal stress in the crack $f_t^{/ef}$ = maximum tensile stresses

c_1 = constant (3) c_2 = constant (6.93)

w = the crack opening

w_c = the crack opening at the complete release of stress

Gf = fracture energy needed to create a unit area of stress-free crack

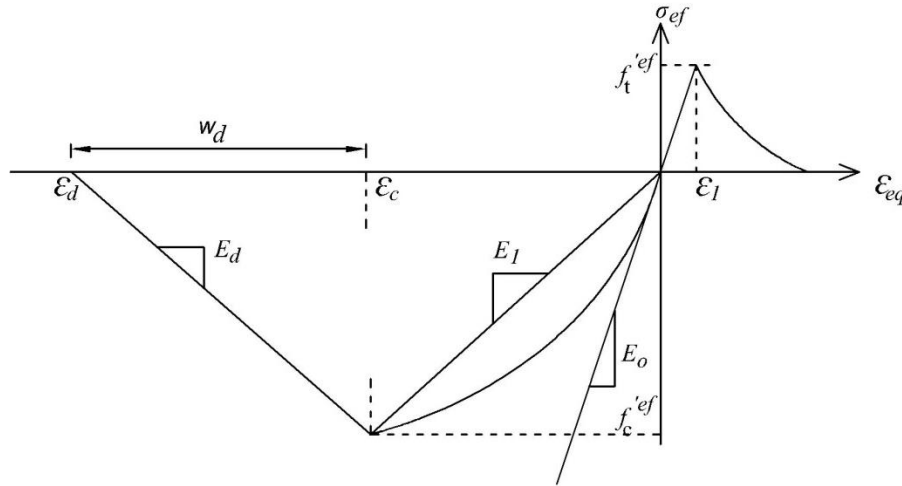


Figure 5.3: Uniaxial stress-strain law for concrete (reproduced from Cervenka et al. 2013)

5.3.2 Cracking

Crack formation is divided into three stages. The first is the uncracked zone before the tensile stress reaches the maximum tensile strength of the concrete. Once the crack starts to form the tensile strength tends to decrease at the position of the crack. The third stage occurs when the stress fully released from the crack location and the crack continues to open without the effect of the stress.

Cracks are assumed to form perpendicular to the principal tensile strain directions in which the cracking criterion is exceeded. In addition, when cracking occurs the stress normal to the crack direction is reduced to zero, which results in redistribution of the stresses around the crack. Cracks are allowed to form in the three principal directions.

Also, for cracking the program uses Rankine failure criterion, which assumes that the stresses and strains directions, in the concrete material, are corresponding to the principle directions. More information can be found elsewhere (Cervenka et al. 2013).

5.3.3 Parameters of Constitutive Model

Table 5.1 summarizes the constants and equations used by *CC3DNonLinCementitious2* material model to calculate the different parameters that are required for the analysis. All of that constants and equations are default by the program. Also, the program leaves the choice for the user to change these values. In the current study these numbers were not changed, except for the tensile strength to meet with numbers obtained experimentally.

5.4 BEARING PLATES

For supports and loading, steel plates were used. The main purpose for those plates was to distribute and transfer the stresses to the different elements of the slab-column model. A tetra element (*CCIsoTetra*) with three translations degrees of freedom in x , y , z directions at each node was used to simulate those plates. Also, the material used for the plates was linear-elastic with 200-GPa modulus of elasticity and 0.3 Poisson's ratio.

Table 5.1: Default formulas for the concrete material parameters

Parameter:	Formula:
Cylinder strength	$f'_c = -0.85f'_{cu}$
Tensile strength	$f'_t = 0.24f'_{cu}{}^{2/3}$
Initial elastic modulus	$E_c = (6000 - 15.5f'_{cu})\sqrt{f'_{cu}}$
Poisson's ratio	$\nu = 0.2$
Softening compression	$w_d = -0.5 \text{ mm}$
Type of tension softening	exponential, based on G_f
Compressive strength in cracked concrete	$0.8 f'_c$
Tension stiffening stress	$\sigma_{st} = 0.0$
Tension-compression function type	linear
Fracture energy, G_f	$G_f = 0.000025f'_t{}^{ef}$
Orientation factor for strain localization	$\gamma = 1.5$

5.5 REINFORCEMENT-CONCRETE INTERFACE

There are two methods to model the relation between the reinforcing bars and the concrete elements. The first is the smeared modelling and the second is the discrete modelling. In smeared modelling technique, the reinforcement is uniformly distributed inside the concrete in the form of layer attached or spreaded inside the concrete element. That technique is most suitable when the locations of the reinforcing bars are not significant and will not affect the overall behaviour of the model. On the other side, in discrete modelling technique the nodes for the concrete and the

reinforcement must be connected together so the stress and strain can transfer between the concrete and the reinforcement elements and to have some kind of computability between the concrete and the reinforcing bars. Figure 5.4 shows drawings for the both modelling techniques. Discrete modelling was chosen to be used in the current study.

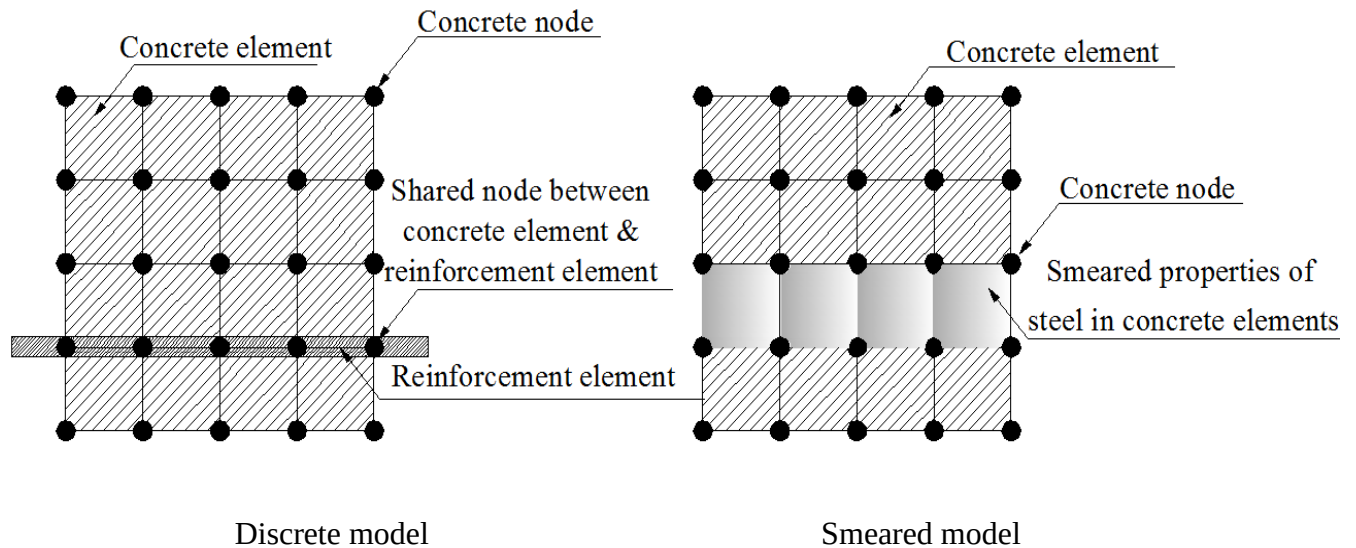


Figure 5.4: Difference between the smeared and discrete models

The bond slips between the concrete elements and the reinforcing bars has a significant effect on the overall behaviour of the FE models, especially for the FRP bars. Therefore, the relationship between the concrete elements and the reinforcing bars at the nodes was not simulated to be fully contact. The bond slips effect, either, between the FRP bars and concrete or the steel ones and the concrete elements was taking into consideration.

In the FE program the bond slips effect is modeled by defining a new material and that material is acting as interface element between the reinforcing bars and the concrete elements. Two materials were defined, one for the GFRP bars and the other for the steel bars. The bond slips material used for the steel reinforcement in this study was the one given by the CEB-FIP Model Code (CEB-FIP 1990), which is the one adapted by the program. Figure 5.5 shows the

relationship between the bond-slips on the horizontal axis and the bond stresses on the vertical one. The curve consists from ascending branch in approximately a parabolic shape then follows by descending part in a linear form and constant part at the end. Also, the ascending branch of the curve followed the equation given by the CEB-FIP Model Code [Eq. 5.9].

$$t = t_{max} \left(\frac{s_1}{0.6} \right)^{0.4} \quad [5.9]$$

Where:

t = bond stresses s_1 = bond slippage

t_{max} = the maximum bond strength of the steel bar embedded in concrete calculated from Eq. [5.10].

$$t_{max} = 2\sqrt{f'_c} \quad [5.10]$$

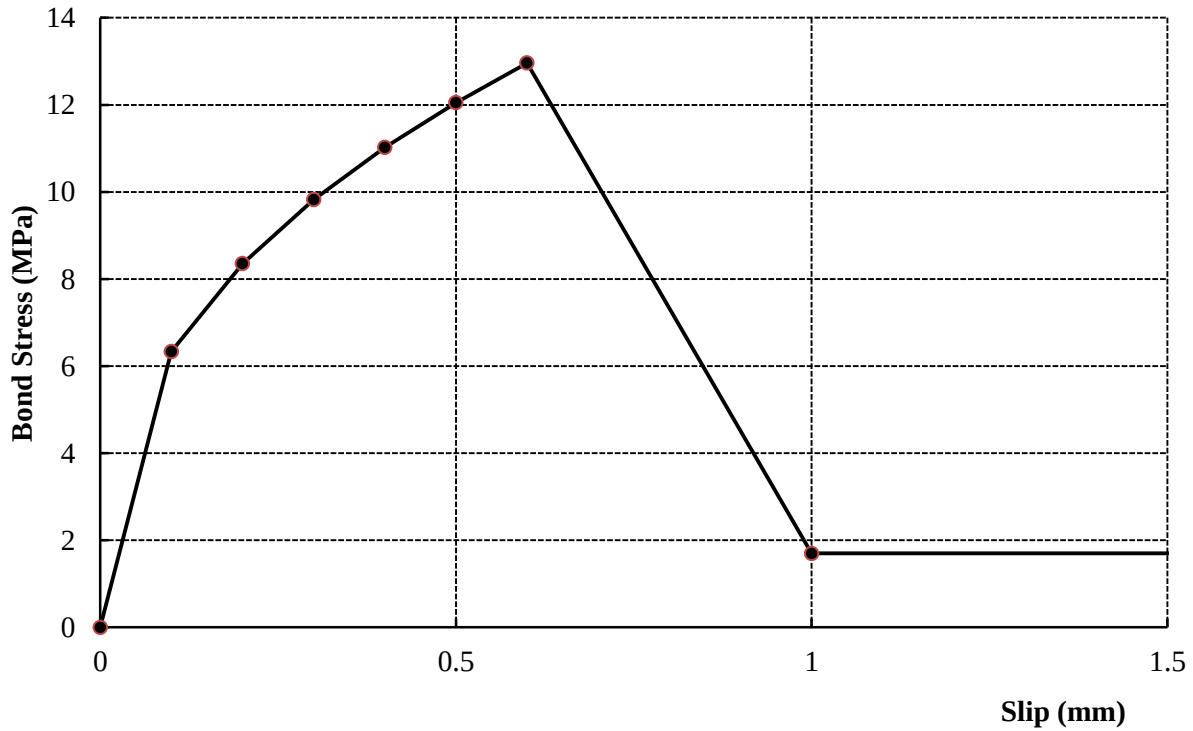


Figure 5.5: Bond-slip relationship for steel bars embedded in concrete (CEB-FIP 1990)

The data for the material used to define the bond slippage relationship between the GFRP bars and the concrete elements was obtained from an experimental research conducted at the University of Manitoba to study the bond characteristics of the GFRP bars (Alves et al. 2011). In that study sand-coated GFRP bars were used to obtain the bond slippage of the FRP material.

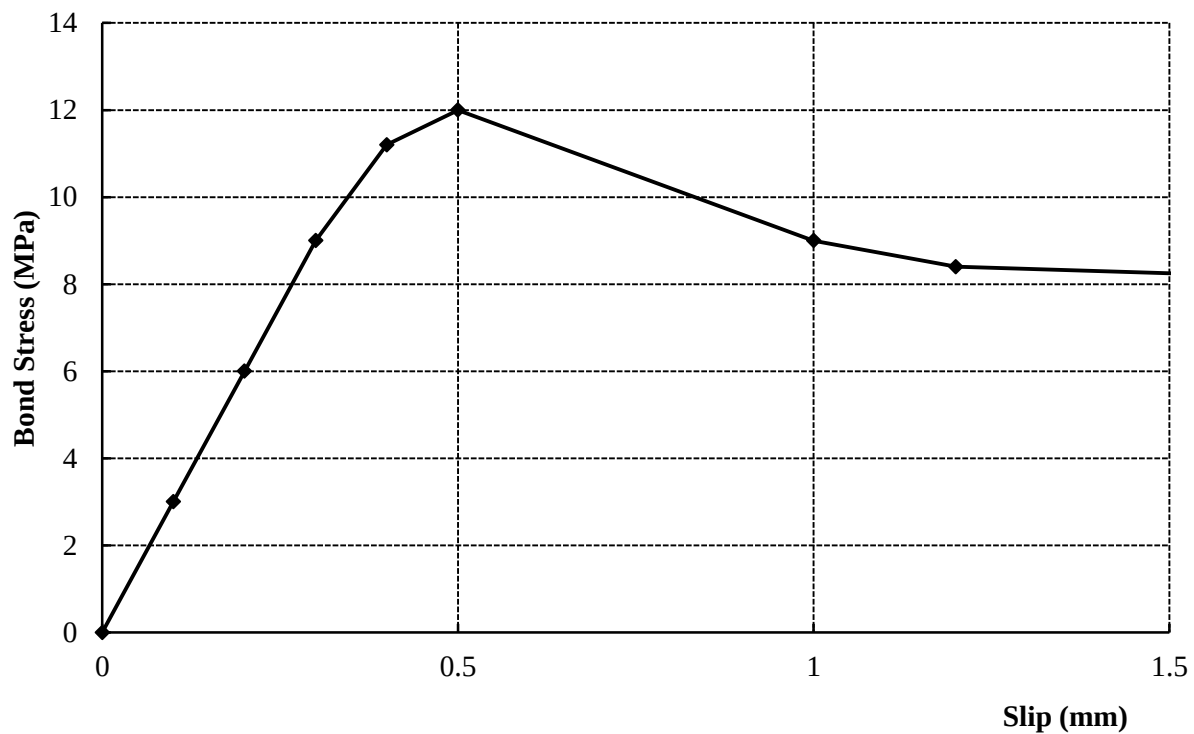
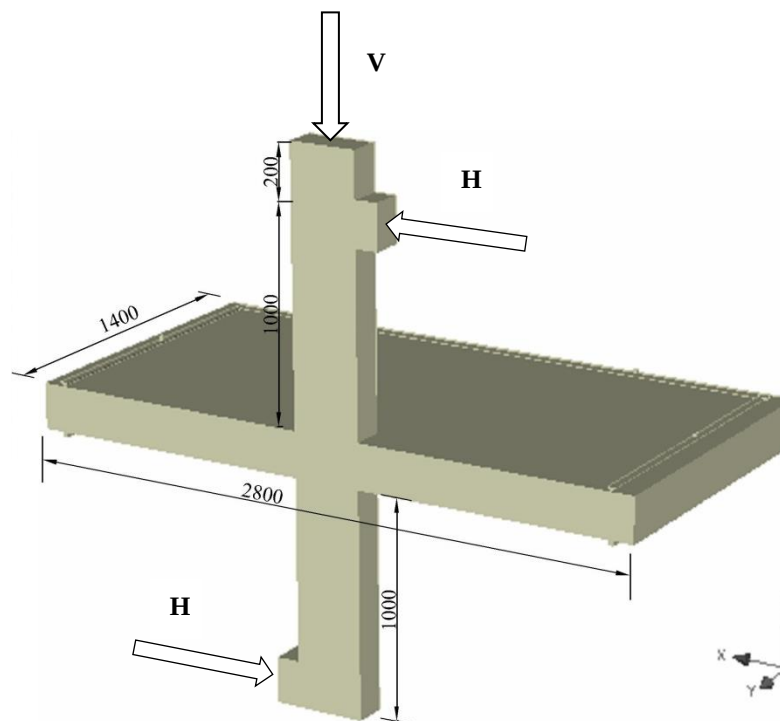


Figure 5.6: Bond-slip relationship for GFRP bars embedded in concrete (Alves et al. 2011)

5.6 MODEL GEOMETRY AND BOUNDARY CONDITIONS

All the slab-column models are symmetry in the direction of the unbalanced moment in internal reinforcement, loading and geometry about the axis passing through the centreline of the slab and the columns. Only half of the specimen was modeled in the FE program. Using only half model instead of the full model saved time and reduced the required storage desk space to solve the model. In turn, finer mesh was used for the slab-column connections. Also, the boundary conditions at the axis of symmetry were chosen carefully to simulate the effect of continuity.

Figure 5.7 shows a photo for the model. Simple supports were provided for a bottom steel frame at 0.1 m from the outer-perimeter of the slab. Also, to consider the uplift of the slab-corners, springs were provided at the corners up to a distance of 0.65 m (the positions of the C-clamps in the experimental test set-up) along the bottom steel frame, these springs prevented the slab-corners from moving in the downward direction only. In addition, for the continuity effect at the axis of symmetry all the translations in y directions were restricted.



All dimensions are in mm

Figure 5.7: Model geometry

Furthermore, the size of the mesh was chosen to give results with the minimum possible errors. The mesh size used in this study was 0.05 m in all the directions. Mesh sensitivity was performed and it was found that decreasing the mesh size beyond this value did not result in any, significant, enhancement. In addition, the loads were applied through three steel plates as shown

in Fig. 5.7. Two plates were used for the horizontal loads to simulate the effect of the unbalanced moment and one plate for the vertical load. The moment-to-shear ratio was kept constant at 0.15 m, as this value resulted from the analysis of the parking garage mentioned earlier (Chapter 3). Also, monitoring points were used to monitor and record the applied vertical force, the value of the deflection of the slab at 50 mm from the column face and the strain of the internal reinforcement at the column face.

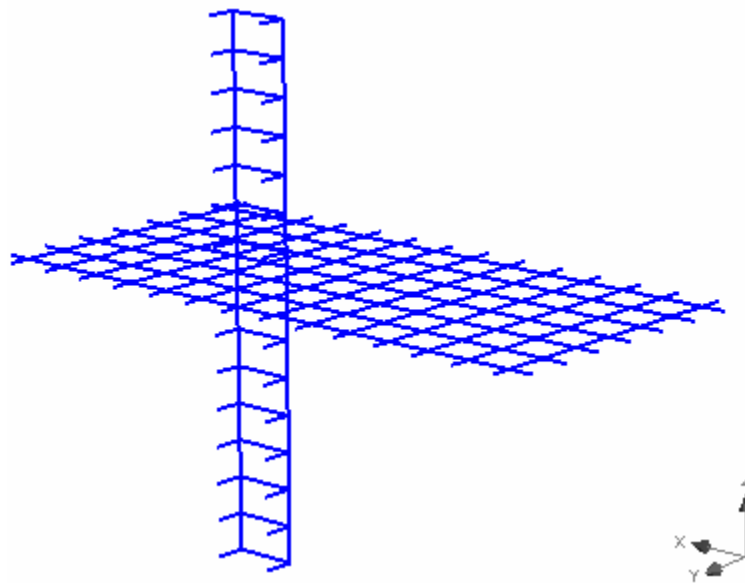


Figure 5.8: Reinforcement configurations

5.7 SOLUTION CONTROL

In the finite element method, to take the effect of non-linearity the total load has to be divided into a series of load steps. At each load step a portion of the load is applied. Also, at the end of each load increment the program updates the stiffness matrix and uses it for the next load step to take the effect of non-linearity into consideration. Newton-Raphson equilibrium iterations

technique was chosen to be the solver. This technique provides a pre-tolerance limit to be defined and if the convergence reaches within that tolerance the program will proceed to the next load step, otherwise, other iterations will be assumed. To achieve that, the program calculates the out-of-balance load vector, which is defined as the differences between the applied loads and the forces obtained from the element stresses. Convergence will occur if the out-of-balance load vector is within the pre-tolerance limit.

5.8 MODEL VERIFICATION

The results from the FE models were verified against the experimental results obtained from the slab-column connections tested in the experimental part of the research. Four specimens were chosen for the verification process, connections G-0.65-N-15-XX, G-0.98-N-15-XX, G-1.30-N-15-XX and G-0.65-H-15-XX. All the slabs in the specimens were reinforced with sand-coated GFRP bars while all the column were reinforced with steel bars and stirrups. The comparison was performed with respect to the load- deflection curve, the tensile strains in the GFRP bars and the failure loads.

Figure 5.9 shows the relationship between the deflection on the horizontal axis and the vertical load on the vertical axis for the experimental and FEM results. It can be seen that the model was able to predict the load-deflection response of the experimental results with good accuracy. Also, the model was able to predict the reduction in the stiffness after cracking as shown in the figure. In the last, the model failed within 2 to 5% less than the experimental failure loads.

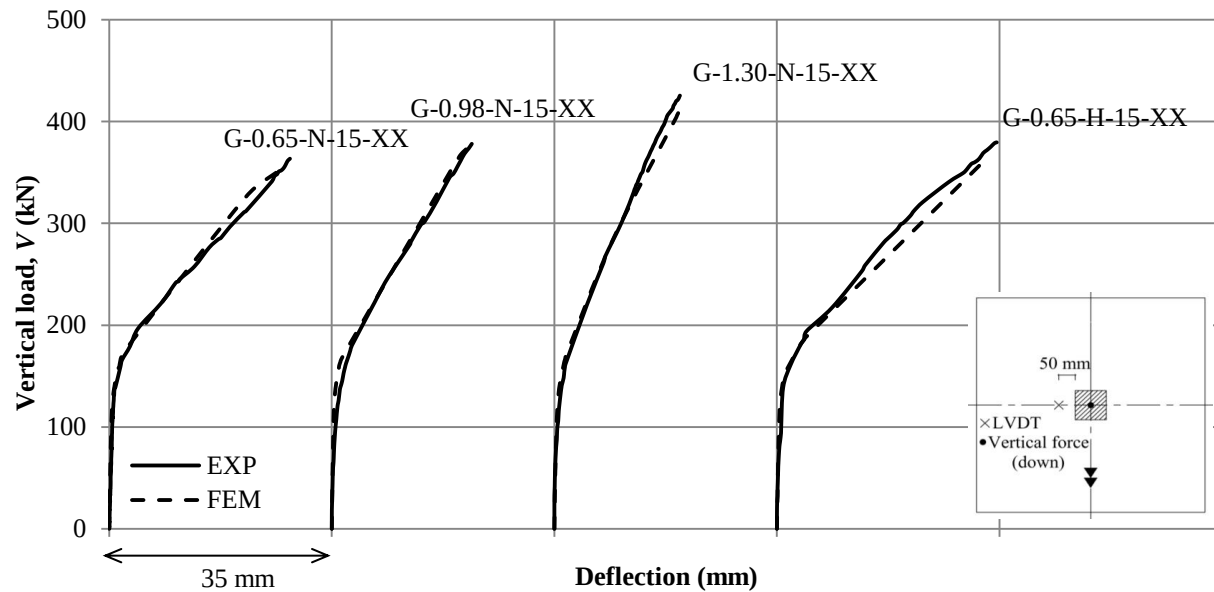


Figure 5.9: Load-deflection behaviour

Figure 5.10 shows the relationship between the strain on the horizontal axis and the vertical load on the vertical one for the experimental and predicted FEM results. The measured tensile strain was in the GFRP bar located at the column face. The strains measured by the program and the strains obtained experimentally are in good agreement. The good prediction for the model to the experimental data strengthens the confidence in the capability of the program to model the behaviour of the slab-column connections.

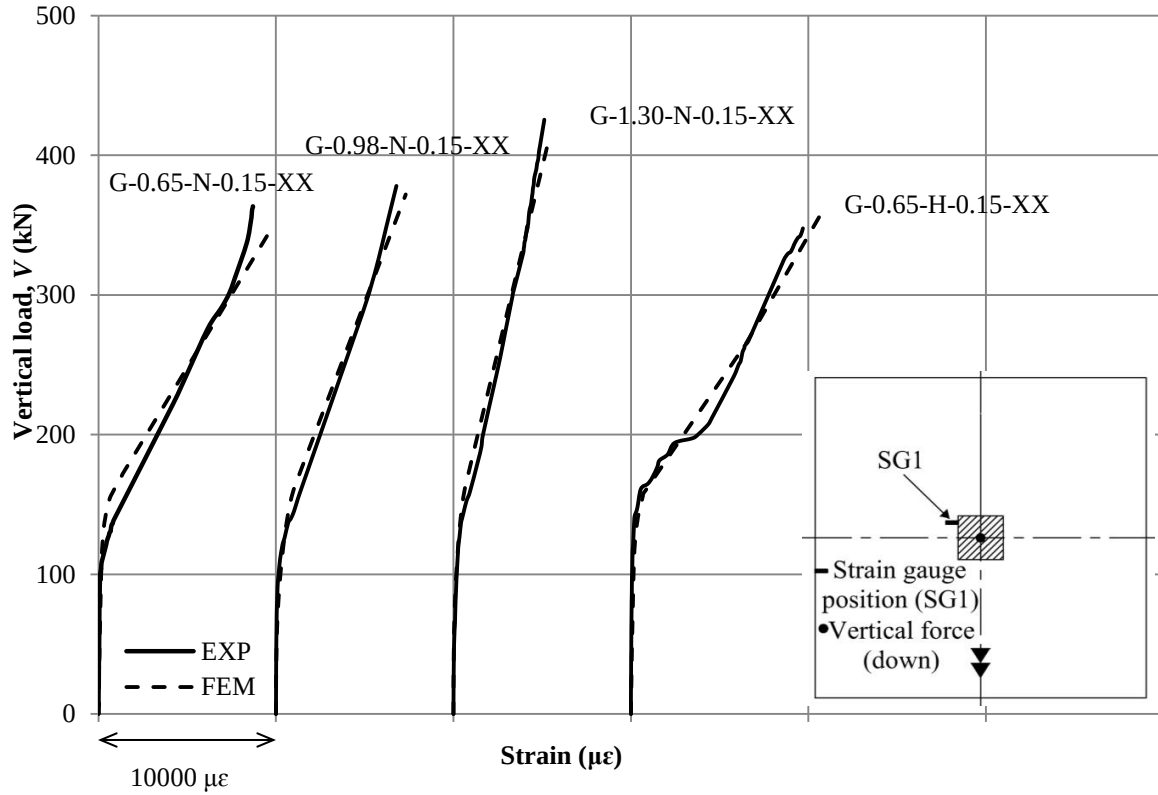
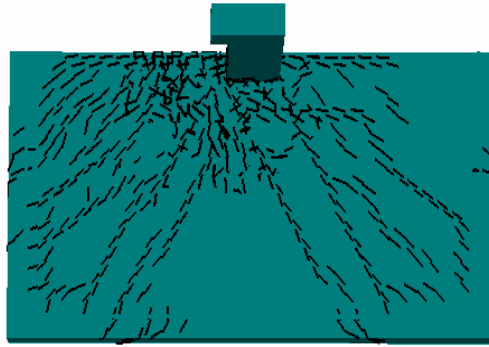


Figure 5.10: Load-strain relationship

Figure 5.11 shows a photo for the cracking pattern of the FE model with 0.65% reinforcement ratio, which is similar to the experimental cracking pattern for slab G-0.65-N-15-XX. The first crack was radial crack started from the corner of the column and moved toward the slab-corner followed by similar cracks with different inclination angles at the tension side of the slabs. With increasing the applied loads, circumferential cracks appeared at the vicinity of the central column to connect the radial cracks together. This cracking pattern is in very good agreement with the observed ones in the experimental part and in the literature for slabs reinforced with FRP bars (Zaghloul 2002) and steel bars (Marzouk et al. 1998).



a) FEM



b) Experiment

Figure 5.11: Cracking pattern for the model with 0.65% reinforcement ratio

5.9 SUMMARY

In general, the FEM constructed using the software was able to predict the behaviour of slab-column connections in terms of load-deflection curve, load-strain curve, crack pattern and failure loads with reasonable accuracy.

Hence, the program can be used to perform a parametric study for better understanding and to evaluate the effect of different parameters on the behaviour of slab-column connections reinforced with GFRP bars. The results of that parametric study are presented in Chapter 6.

CHAPTER 6

PARAMETRIC STUDY

6.1 INTRODUCTION

Finite element method is the process of subdividing all systems into their individual components, or elements, whose behaviour is really understood then rebuilding the original system from such components to use a finite number of well-defined components. The FEM has been used widely to study the effect of different parameters on the behaviour of concrete structure elements, mainly, to overcome the financial problems associated with casting and testing full scale specimens in the lab.

This part of the research investigated the effect of four main parameters on the behaviour of slab-column connections reinforced with GFRP bars using the verified model described in Chapter 5. The discussed parameters were 1) a wide range of flexural reinforcement ratios (between $0.5 \rho_b$ and $4.0 \rho_b$, with $0.5 \rho_b$ increments). 2) shear perimeter-to-depth ratio (11.5, 15.25 and 19 corresponding to square column cross section with side dimension of 300, 450 and 600 mm and effective slab depth of 160 mm. 3) column aspect ratios (between 1.0 and 5.0). 4) slab thickness (between 200 and 350 mm).

The analytical study was based on the materials properties and geometry of specimen G-0.65-N-15-XX, which was verified in the previous chapter. Specimen G-0.65-N-15-XX is the control sand-coated GFRP specimen in the experimental part in the current research.

For each parameter, a comparison was performed in terms of load-deflection curve, load-strain curve in the GFRP bars, vertical shear force and punching shear stress. Also, a number of important conclusions were drawn based on the results of the investigated parameters.

6.2 FLEXURAL REINFORCEMENT RATIO

The Canadian standard CSA/S806-12 (CSA 2012) recommends that FRP-RC members to be over reinforced, so the failure is initiated by concrete crushing rather than rupture of the bars. Also, FRP bars are elastic materials with smaller stiffness compared to that of steel; therefore large cracks are expected to spread in the FRP-RC member. Consequently, those cracks would reduce the effective compressive area and the contribution of aggregate interlock resisting the punching shear stresses. To evaluate the effect of the flexural reinforcement ratios on the punching shear behaviour of interior slab-column connection, wide range of flexural reinforcement ratios was chosen between 0.15 and 1.20% (0.5 and 4.0 ρ_b , where ρ_b is the balanced reinforcement ratio) with 0.15% (0.5 ρ_b) increment.

6.2.1 Load-Deflection Response

Similar to the experiments, the behaviour of the FE models can be described as a bi-linear relation with smooth transition. In the uncracked zone the deflection increased linearly up to initiation of the first crack. The behaviour of the models in that zone represents the stiffness of the uncracked specimens. Also, nonlinearity in the load-deflection curve was observed when the tensile stresses exceeded the tensile strength of the concrete. Furthermore, once the cracks initiated in the models, the uncrack stiffness tended to decrease but the rate of the loss in the stiffness is inversely proportional with the increase in the axial stiffness of the reinforcement, $\rho_f E_f$, of the model. Generally, increasing the axial stiffness from half the balanced reinforcement

ratio to four times the balanced reinforcement ratio increased the post-cracking stiffness of the model, which in turn decreased the deflection at the same load level.

The variance in the post cracking stiffness with increasing the reinforcement ratio is presented in Fig. 6.2. Approximately tenfold increase in the post cracking stiffness was found due to the increase in the reinforcement ratio from half to four times the balanced reinforcement ratio. Also, mathematical equation was introduced in the figure to find the relation between the two variables.

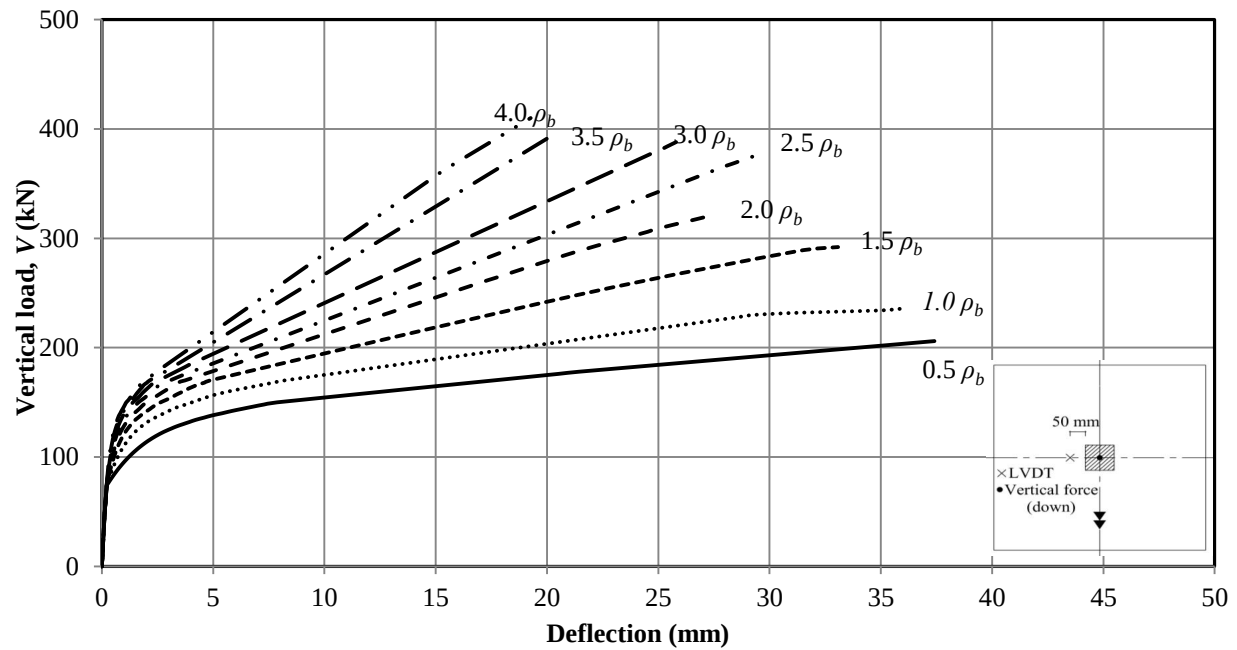


Figure 6.1: Variation in load-deflection relationship with the reinforcement ratio

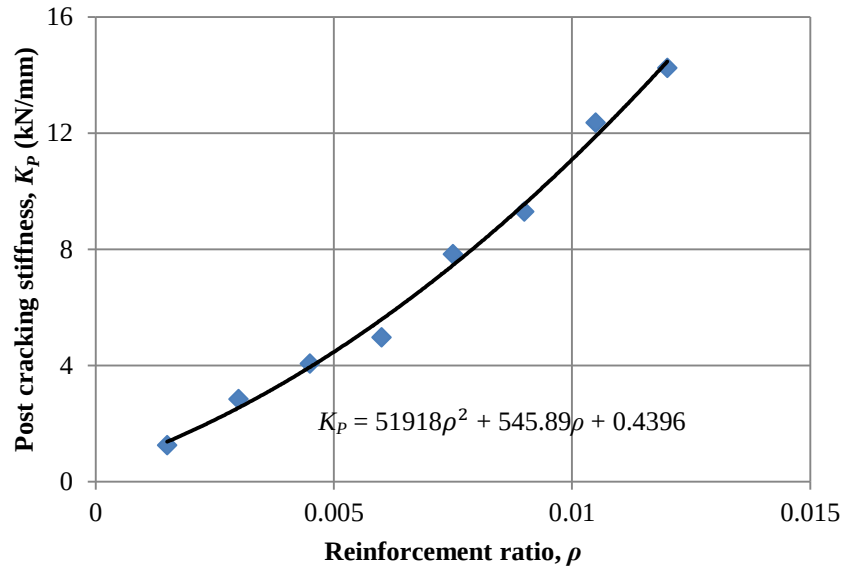


Figure 6.2: Relation between the reinforcement ratio and the post cracking stiffness

6.2.2 Load-Strain Relationship

Figure 6.3 depicts the relationship between the vertical shear force and the measured strain in the GFRP bars for the models with different axial stiffness of the slabs reinforcement. In general, before cracking the measured strain values were insignificant and did not exceed 500 microstrains for all the models. In addition, when the tensile stresses overcome the tensile capacity of the concrete the strains in the reinforcement increased at a higher rate. The maximum measured strain was, almost, 13,000 $\mu\epsilon$ in the model with half the balanced reinforcement ratio. This value of strain is, approximately, 63% of the rupture strain of the sand-coated GFRP bars used in the current study. Also, this value confirmed that the failure of the models was not initiated by the rupture of the reinforcement. Furthermore, increasing the reinforcement ratio decreased the strain at the same load-level. In addition, increasing the reinforcement ratio from half to four times the balanced reinforcement ratio resulted in 57% decrease in the strain at the failure stage. At service load stage the strains in the models with reinforcement ratio of 0.6 ($2.0 \rho_b$), 0.75 ($2.5 \rho_b$), 0.9 (3.0

ρ_b), 1.05 ($3.5 \rho_b$) and 1.2% ($4.0 \rho_b$) were 9880, 6520, 5300, 4110 and 3510 $\mu\epsilon$, respectively, which represent 48, 32, 25, 20 and 17% of the ultimate tensile capacity of the GFRP bar. In addition, the models with 0.15 ($0.5 \rho_b$), 0.3 ($1.0 \rho_b$) and 0.45% ($1.5 \rho_b$) reinforcement ratio failed before reaching the calculated service load of 295 kN.

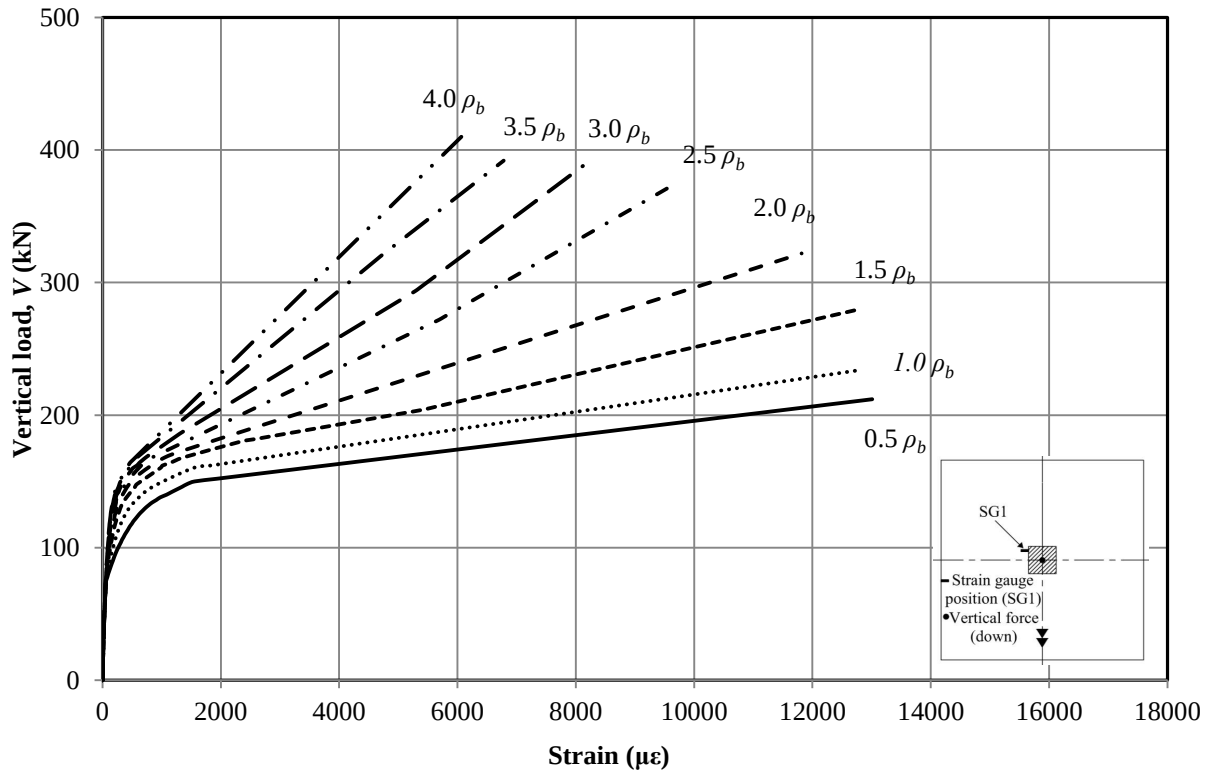


Figure 6.3: Variation in load-strain relationship with the reinforcement ratio

6.2.3 Ultimate Capacity

Due to the relatively-low modulus of elasticity of the FRP bars compared to steel bars, concrete members reinforced with FRP bars have a relatively small stiffness after cracking, which results in wider crack width and smaller depth to the neutral axis. Also, FRP bars are uni-directional materials (anisotropic) with low strength and stiffness in the transverse direction. This results in smaller failure load and consequently less contribution to shear resistance.

The relationship between the reinforcement ratio and the ultimate vertical load is illustrated in Fig. 6.4. Also, an equation was introduced in the figure to find the relation between the vertical shear force and the reinforcement ratio. According to that equation, the vertical load is proportional to the reinforcement ratio with power of 0.34. However, this relationship might change with any change in material properties or the geometry of the model. Also, these results are in good agreement with the Canadian standard CSA/S806-12 punching shear equations (CSA 2012).

The relation between the axial stiffness of the slab reinforcement and the normalized ultimate punching shear stresses calculated at $0.5 d$ (80 mm) from the column face is presented in Fig 6.5. Moreover, an interpolation equation was introduced in the figure [Eq. 6.1] to find the mathematical correlation between the normalized punching shear stresses and axial stiffness of the slab reinforcement. The equation was obtained using the method of least squares. On the basis of the results of that formula, the normalized punching shear stress is found to be proportional to the axial stiffness of the slab reinforcement with power of 0.34. In addition, the constant value in the interpolation equation is 0.0562. This equation is similar to Eq. [2.50] of the Canadian standard CSA/S806-12 (CSA 2012).

$$v_u / \sqrt[3]{f'_c} = 0.0562 \lambda \phi_c (E_F \rho_F)^{0.34} \quad [6.1]$$

Table 6.1 shows the predicted shear stresses from the Canadian code in addition to the predictions from Eq. [6.1] and the ratio between the experimental shear stresses and the predicted ones, for the slabs tested in the experimental part. All the safety factors in the equations were set equal to unity. In general, CSA/S806-12 (CSA 2012) standard gave close, conservative, predictions to the actual punching shear stresses of the slabs by an average of

$\nu_{exp.}/\nu_{CSA} = 1.09 \pm 0.06$ (COV = 5.5%). However, Eq. [6.1] yielded the nearest predictions by an average of $\nu_{exp.}/\nu_{6.1} = 1.02 \pm 0.06$ (COV = 5.9%). Also, using Eq. [6.1] to predict the failure loads of other researcher experimental work (Hassan et al. 2013), tested under concentric load, gave that $V_{exp.}/V_{6.1} = 1.09 \pm 0.10$ (COV = 7.6%). While the Canadian standard, CSA/S806-12 [Eq. 2.50], yielded $V_{exp.}/V_{CSA} = 1.17 \pm 0.08$ (COV = 6.8%). these results show that Eq. [6.1] gives more accurate results than the ones, currently, adapted by the Canadian standard.

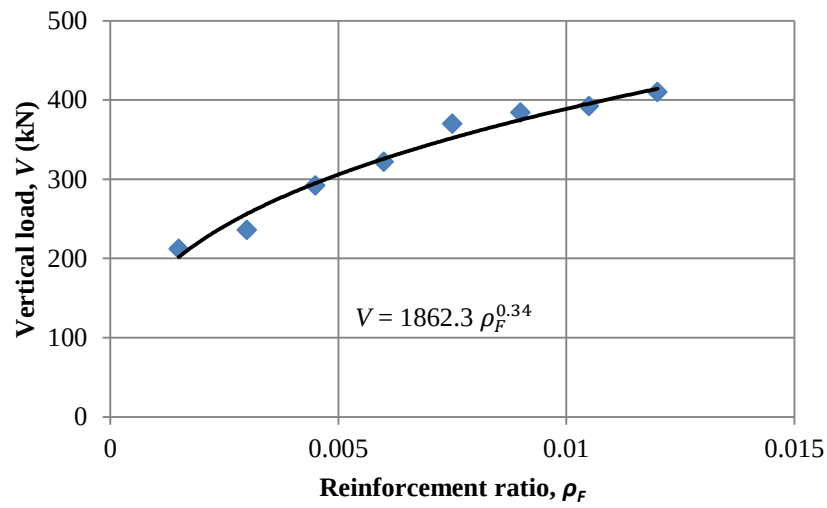


Figure 6.4: Relationship between the ultimate vertical load and the reinforcement ratio

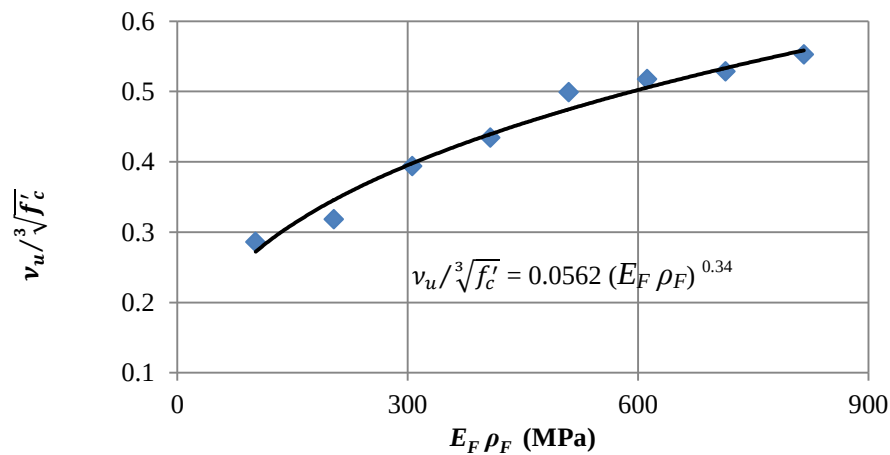


Figure 6.5: Relationship between the normalized punching shear stress at failure and the axial stiffness of the slabs

Table 6.1 - Predicted shear stresses for the GFRP-slabs without shear reinforcement

Specimens	CSA/S806-12* (2012)		Equation [6.1]	
	ν_{CSA} (MPa)	$\nu_{exp.}/\nu_c$	$\nu_{6.1}$ (MPa)	$\nu_{exp.}/\nu_{6.1}$
G-0.65-N-15-XX	1.48	1.15	1.56	1.09
G-0.98-N-15-XX	1.64	1.08	1.73	1.02
G-1.30-N-15-XX	1.82	1.10	1.93	1.03
G-0.65-H-15-XX	1.66	1.07	1.85	0.96
G-0.65-N-00-XX	1.43	1.00	1.51	0.95
G-0.65-N-30-XX	1.48	1.19	1.56	1.13
R-0.65-N-15-XX	1.43	1.05	1.50	1.00
Mean		1.09		1.02
SD		0.06		0.06
COV (%)		5.50		5.90

* Calculated using Eq. [2.50]

6.3 EFFECT OF SHEAR PERIMETER-TO-DEPTH RATIO

Punching shear strength of FRP-RC interior slab-column connections is a topic that has not been fully covered yet due to the fact that FRP materials are, relatively, new in the civil industry and more researches are needed to cover all the factors that affect the behaviour of that type of connections. One of the factors, that needed to be investigated, is the effect of column dimensions on the punching shear strength of FRP-RC interior slab-column connections. Also,

the effect of the unbalanced moment gives more complexity to the punching shear problem. That unbalanced moment can occur due to different loading pattern and/or different span lengths. Also, the available literature shows that Zaghoul (2002) studied the effect of the unbalanced moment on half-scale interior slab-column connections reinforced with carbon FRP grids. In the current research three specimens with different square column cross sections were chosen to evaluate the effect of shear perimeter-to-depth ratio on the behaviour of such connection. Shear perimeter-to-depth ratio of 11.5, 15.25 and 19 corresponding to square column cross section with side dimension of 300, 450 and 600 mm was investigated. The effective slab depth and reinforcement ratio were 160 mm and 0.65%, respectively.

6.3.1 Load-Deflection Response

Figure 6.6 presents the relation between the vertical load and the measured deflection at 50 mm from the column face. Generally, the behaviour of the models is similar to that illustrated earlier in the “flexural reinforcement ratio” section. The specimen with perimeter-to-depth ratio of 11.5 experienced the lowest failure load and post cracking stiffness. Also, as the dimensions of the column increased, considerable improvement was observed in the failure load and the deflection at the same load level. At the service load stage, increasing the perimeter-to-depth ratio from 11.5 to 19 resulted in 33% reduction in the deflection. In addition, it seems that increasing the column cross section increased the degree of nonlinearity of the model at failure. Furthermore, the post cracking stiffness increased as evidence in Fig. 6.7. This is due to that the increase in the dimensions of the column increased the overall stiffness of the connection. Moreover, an equation was presented in Fig. 6.7 to find the mathematical correlation between the post cracking stiffness and the perimeter-to-depth ratio.

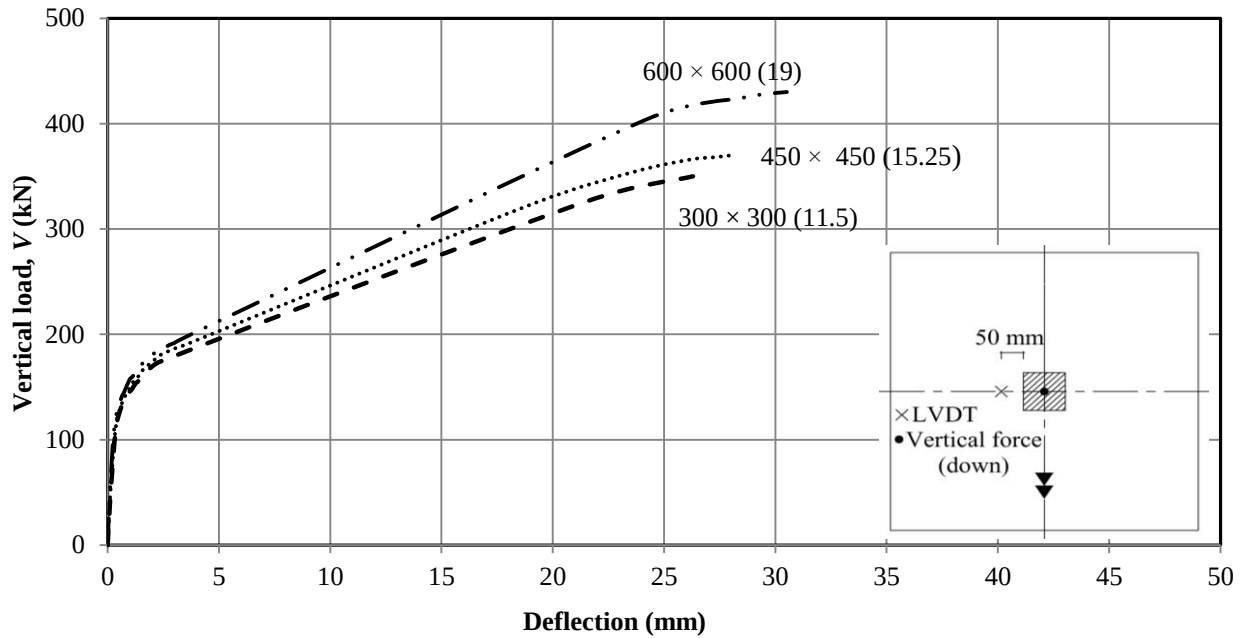


Figure 6.6: Load-deflection relationship for the models with different perimeter-to-depth ratio

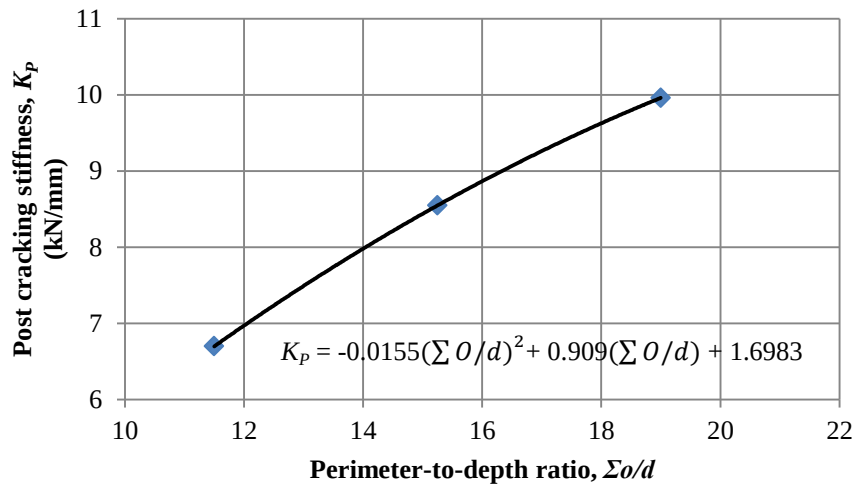


Figure 6.7: Relation between the perimeter-to-depth ratio and the post cracking stiffness

6.3.2 Load-Strain Relationship

Figure 6.8 illustrates the relationship between the strains measured at the column face on the horizontal axis and vertical load. Again, the behaviour of the models is similar to that mentioned earlier in the “flexural reinforcement ratio” section. Also, increasing the perimeter-to-depth ratio

from 11.5 to 19 resulted in 28% decrease in the strain, at the service load level. The maximum measured strain in that group was, approximately, $10760 \mu\epsilon$. This value is 52% from the ultimate strain of the sand-coated GFRP bars. Also, at the service load stage, the strain in models with a perimeter-to-depth ratio of 11.5, 15.25 and 19 was 7280, 6730 and $5350 \mu\epsilon$, respectively, which represent 35, 33 and 26% of the rupture strain of the GFRP bar. These values are higher than that recommended by the Canadian standard (CSA/S806-12).

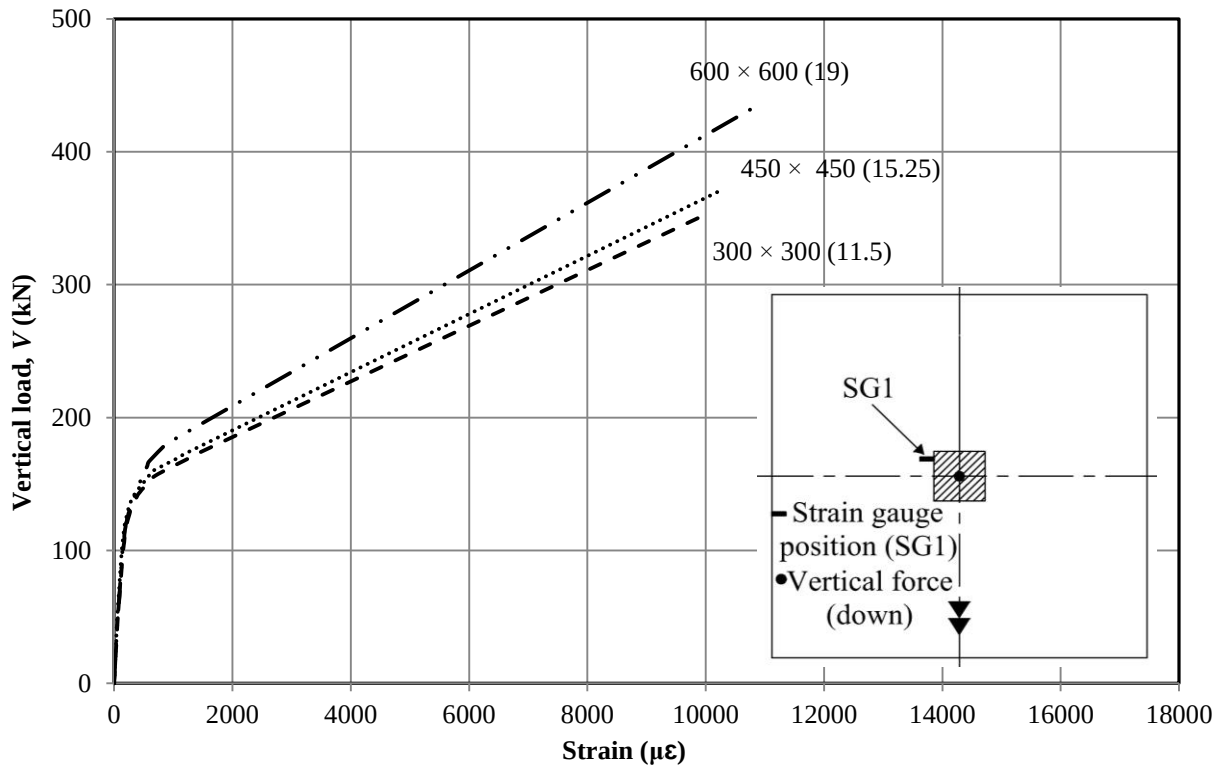


Figure 6.8: Load-strain relationship for the models with different perimeter-to-depth ratio

6.3.3 Ultimate Capacity

The relationship between the ultimate vertical load and the perimeter-to-depth ratio is given in Fig. 6.9. As shown in the figure, curve fitting was performed and an equation was introduced to find the mathematical correlation between the failure vertical load and the perimeter-to-depth ratio. According to that equation, the relationship can be described in a quadratic function.

Again, that relation is very sensitive to any change in geometry, reinforcement ratio and material properties.

As the column cross section increases, the perimeter that resists the shear forces increases too which leads to better resistance in the form of decreasing in the shear stresses around the column face. Figure 6.10 depicts the relation between the failure stress calculated at $0.5 d$ (80 mm) from the column face and the perimeter-to-depth ratio. At failure, a 26 and 34% decrease in the punching shear stresses was found due to the increase in the perimeter-to-depth ratio from 11.5 to 15.25 and 19, respectively. These results are in very good agreement with the finding by Dulude et al. (2013), where two GFRP-RC specimens with 200-mm thickness and square column cross sections with side length of 300 and 450 mm were tested under concentric load. It was reported that a 29% decrease in the failure stresses was observed with increasing the square column dimensions from 300 to 450 mm. Also, it was found that the Canadian standard CSA/S806-12 (CSA 2012) underestimated the failure load for the specimen with 300-mm square column cross section and overestimated the failure load for the connection with 450-mm square column cross section. In addition, Sherif et al. (2005) reported similar results where two edge steel-RC slab-column connections were tested under eccentric load. A 20% reduction in the punching shear stresses was found when the perimeter-to-depth ratio was increased from 6.5 to 11.0.

The Canadian code CSA/S806-12 (CSA 2012) takes into account the effect of perimeter-to-depth ratio in Eq. [2.49]. According to this equation, increasing the perimeter-to-depth ratio from 11.5 to 15.25 and 19 resulted in 16 and 25% decrease in the predicted punching shear stresses, respectively. However, Eq. [2.50], which does not account for the effect of reducing the punching shear stresses due to the increase in the perimeter-to-depth ratio (Fig. 6.10), gives more

conservative predictions and thus it governs the design. To overcome this problem, Eq. [2.50] needs to be multiplied by a reduction factor [Eq. 6.2]. However, further investigation is still needed to evaluate the adequacy of this reduction factor.

$$\text{Reduction factor} = \sqrt{\frac{300}{C}} \leq 1.0 \quad [6.2]$$

Where: C = the side length of the square column.

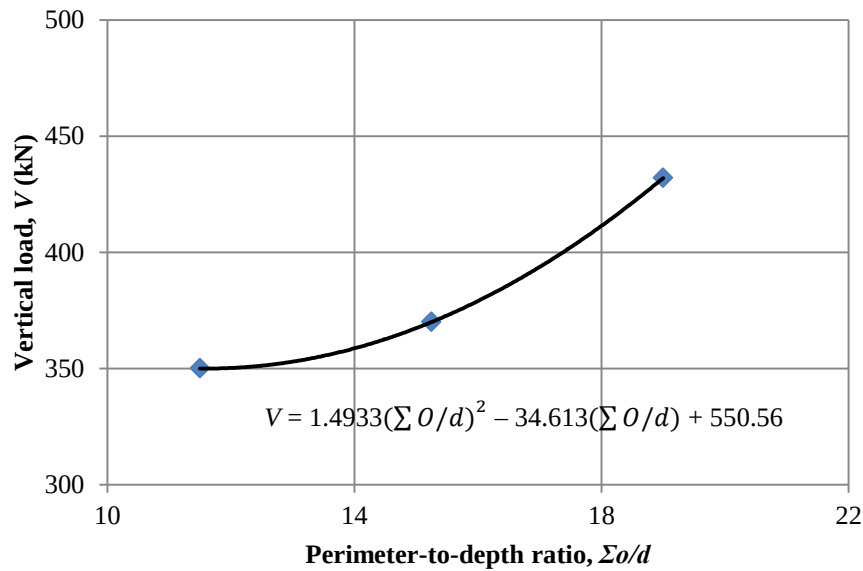


Figure 6.9: Relationship between the ultimate vertical load and the perimeter-to-depth ratio

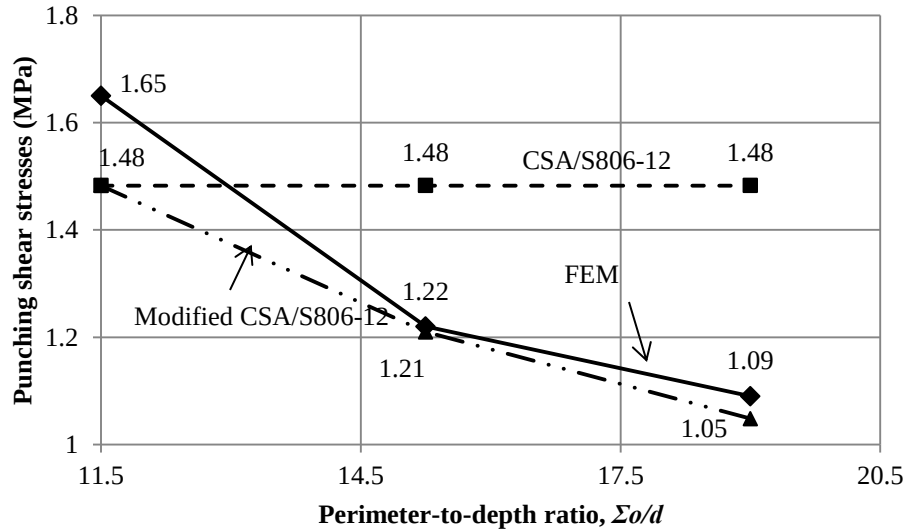


Figure 6.10: Relationship between the punching shear stress at failure and the perimeter-to-depth ratio

6.4 EFFECT OF COLUMN ASPECT RATIO

Approximately all the literature available for FRP-RC slab-column connections is focusing, mainly, on studying the behaviour of interior slab-column specimens under concentric load. Many researchers have been attracted to study the effect of several parameters such as reinforcement ratio and concrete compressive strength on the behaviour of full-scale FRP-RC interior slab-column connections. Up to date, no researcher has studied the effect of column aspect ratio on the behaviour of such connection. One of the aims of this parametric study is to partially fill that gap. Five models with, 0.65% reinforcement ratio and, column aspect ratio ranged between one and five were constructed to evaluate the effect of that parameter. Also, the slab thickness and the average depth were 200 and 160 mm, respectively, for all the models. In addition, the unbalanced moment was applied in a direction parallel to the direction of smaller dimension of the column, as shown in Fig. 6.11, for two reasons. The first is to keep the span-to-

depth ratio constant for all the models and the second is to consider the critical case by applying the unbalanced moment in the direction of the lower moment of inertia of the column.

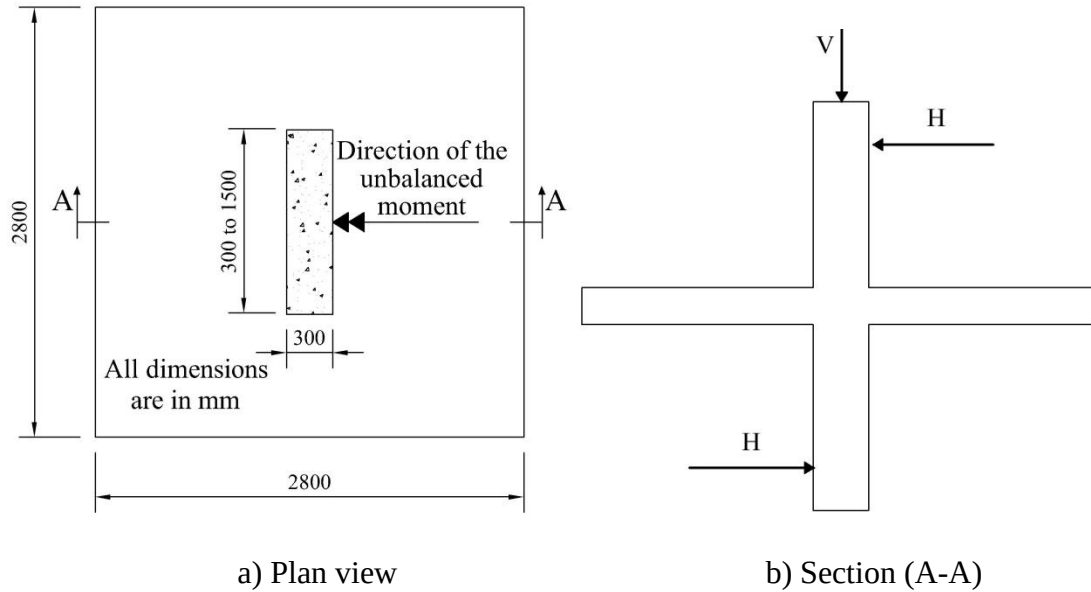


Figure 6.11: direction of the unbalanced moment.

6.4.1 Load-Deflection Response

Figure 6.12 shows the variation of the load-deflection relationship with different column aspect ratios. The model with column aspect ratio equal to one demonstrated the lowest failure load and post-cracking stiffness. Also, as the column aspect ratio increased significant improvement in the carrying load capacity and the post-cracking stiffness was observed due to the distribution of the stresses on longer perimeter. Increasing the column aspect ratio from one to five resulted in 65% reduction in the deflection at the service load stage. In addition, it seems that the non-linearity increased in the models at the failure stage with increasing the column aspect ratio. Figure 6.13 depicts the different in the post cracking stiffness with increasing the column aspect ratio. Also, 85% increase in the post cracking stiffness was observed due to the increase in the column aspect

ratio from one to five. According to the curve fitting that was conducted, parabolic equation was found to be the suitable correlation to describe the relation between the post cracking stiffness and the column aspect ratio.

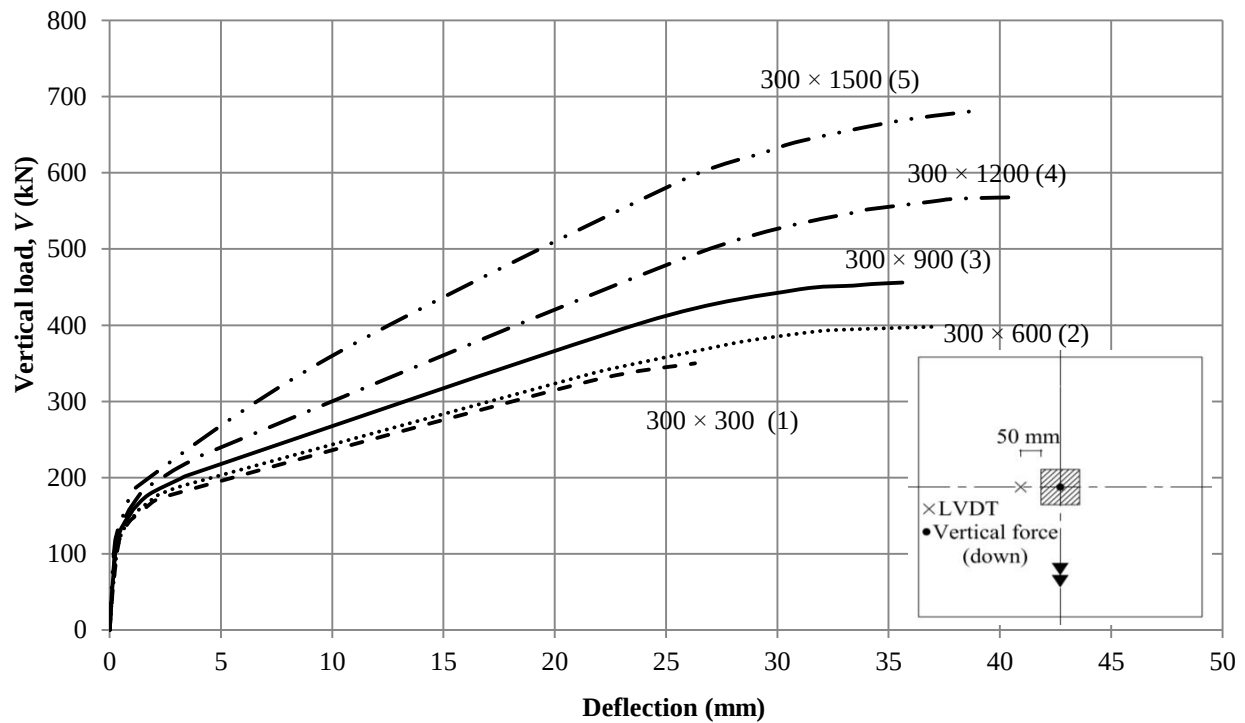


Figure 6.12: Load-deflection relationship for the models with different column rectangularity

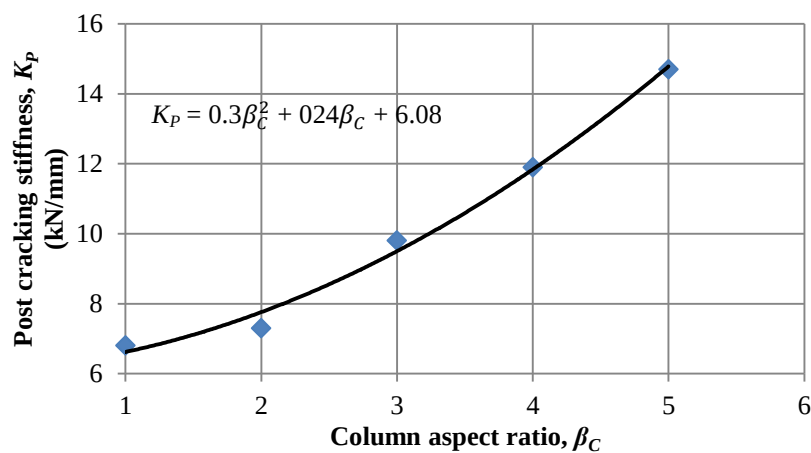


Figure 6.13: Relationship between the column aspect ratio and the post cracking stiffness

6.4.2 Load-Strain Relationship

Figure 6.14 illustrates the load-strain relationship for the investigated column aspect ratios. The maximum measured strain was approximately $12,300 \mu\epsilon$ in the model with column aspect ratio equal to 4.0. This value of strain is 60% of the ultimate strain of the GFRP bars. Also, at the service load stage, 59% decrease in the strain was observed due to the increase in the column aspect ratio from one to five. At the failure stage this percentage was 26% due to the increase in the column aspect ratio from one to four that, in turn, increased the ultimate loading capacity. Furthermore, the strain at failure for the models with column aspect ratio equal to 4 and 5 was, almost, the same. The strains in the models with 1.0, 2.0, 3.0, 4.0 and 5.0 column aspect ratio were 7280, 6520, 5570, 4330 and 3380 $\mu\epsilon$, respectively, at the service load stage. These values represent 35, 32, 27, 21, and 16% of the ultimate strain of the GFRP bar.

6.4.3 Ultimate Capacity

The relationship between the vertical load and the increase in the column aspect ratio is depicted in Fig. 6.15. A 95% increase in the vertical shear force was found due to fourfold increase in the column aspect ratio. Also, as seen in the figure, the mathematical correlation between the column aspect ratio and the ultimate vertical load can be expressed in a quadratic relation.

The relation between the column aspect ratio and the punching shear stresses calculated at $0.5d$ (80 mm) from the column face at the failure stage is presented in Fig. 6.16. The figure shows 30% decrease in the punching shear stress at failure up to column aspect ratio of 3.0; however, increasing the column aspect ratio from 3 to 4 increased the shear strength from 1.16 to 1.19 MPa. Further increase in the column aspect ratio (from 4 to 5) had no effect, where the stress remained unchanged. This indicates that the effect of column rectangularity is vanished after a value greater than three. These results are in very good agreement with the finding of Hawkins et

al. (1971); where several steel-RC interior slab-column connections with different column rectangularity were tested. According to the test results, it was found that the stresses at failure decreased up to column aspect ratio equal to 3.0 and after that value, the stresses increased. Based on these results, it was concluded that the effect of the column rectangularity diminishes for ratios greater than 3. Also, as seen in Fig. 6.16, the Canadian standard CSA/S806-12 (CSA 2012) takes the effect of column rectangularity into consideration when it exceeds a value of two. In addition, the code predictions continue decreasing the punching shear stresses with increasing the column rectangularity.

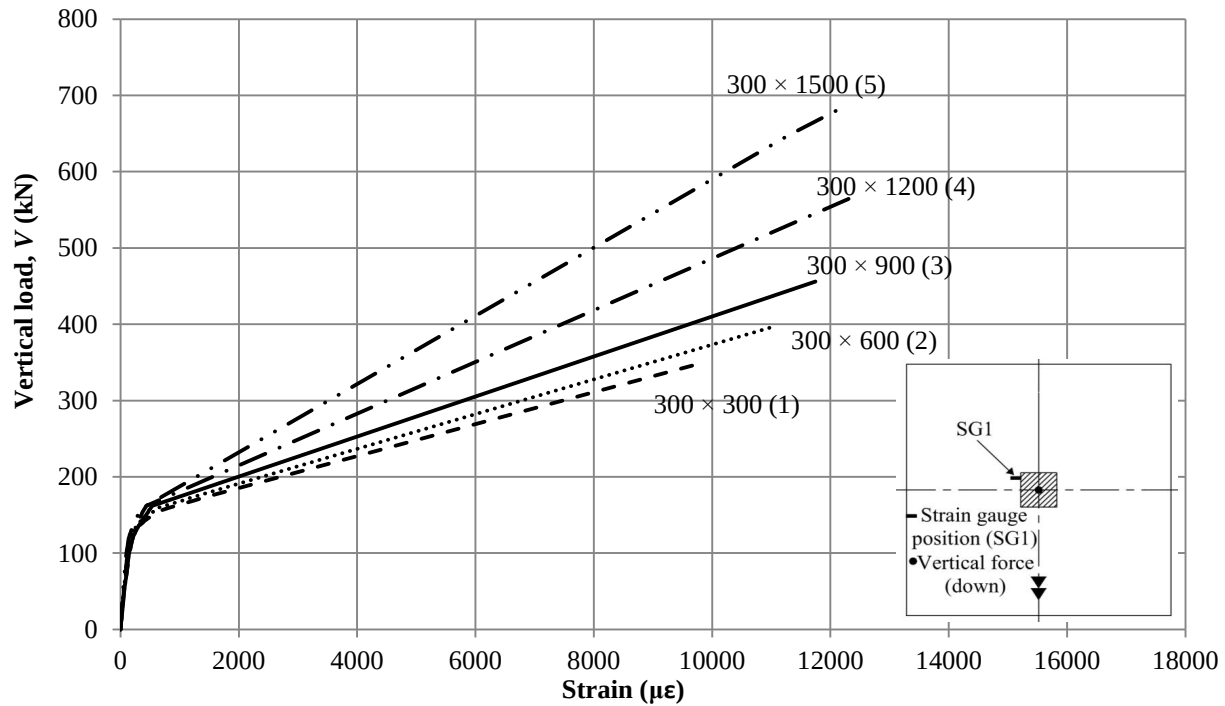


Figure 6.14: Load-strain relationship for the models with different column aspect ratio

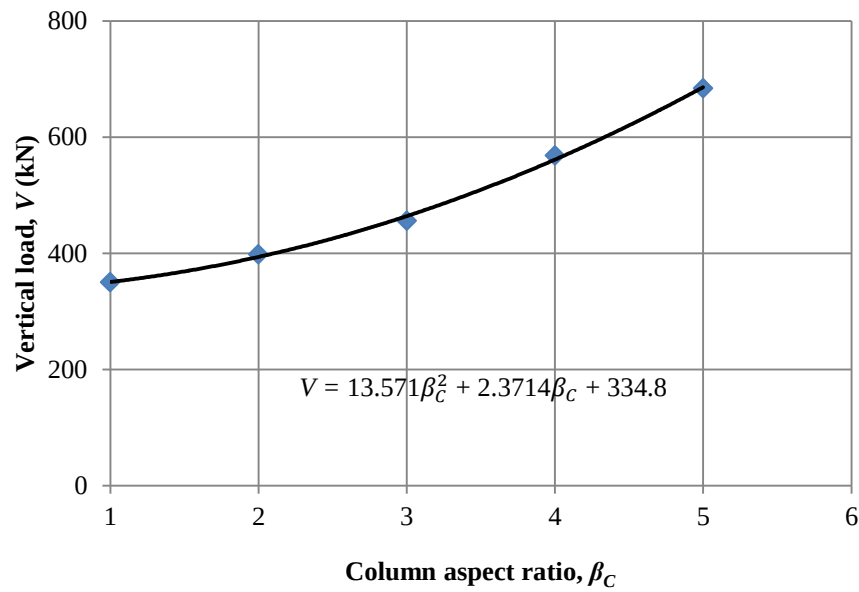


Figure 6.15: Relationship between the ultimate vertical load and the column aspect ratio

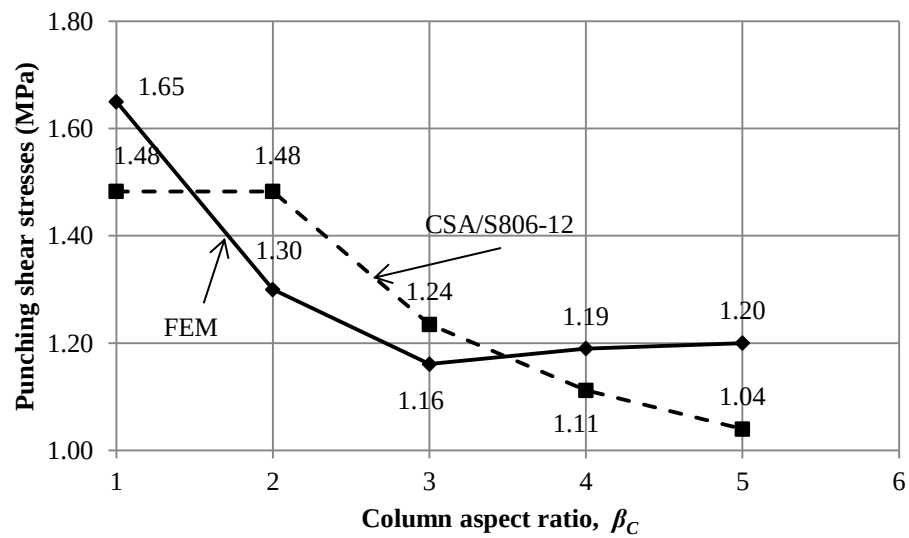


Figure 6.16: Relationship between the punching shear stress at failure and the column rectangularity

6.5 EFFECT OF SLAB THICKNESS

Increasing the slab thickness, t , of a whole floor in a parking garage structure may not be an economical solution; however increasing the slab thickness around the column area in a form of a drop panel may be an effective way in increasing the capacity of the connection. Four models, with 0.65% reinforcement ratio, were constructed to study the effect of increasing slab thickness on the behaviour of interior slab-column connections. The difference between the models was the slab thickness, which varied between 200 and 350 mm. The effective depth for the connections was 160, 210, 260 and 310 mm for the models with 200, 250, 300 and 350-mm slab thickness, respectively.

6.5.1 Load-Deflection Response

Figure 6.17 presents the load-deflection relationship for the specimens with different slab thickness. As expected, the model with 200-mm slab thickness showed the lowest carrying load capacity and post-cracking stiffness. Also, in the post-cracking stage at the same load level, 83% reduction in the deflection was observed due to the increase in the slab thickness from 200 to 350 mm (75%). Figure 6.18 shows the relationship between the slab thickness and the post cracking stiffness. Again, an equation was introduced in the figure and according to the curve fitting that was performed; parabolic equation was found to give the best correlation between the two variables.

6.5.2 Load-Strain Relationship

Figure 6.19 shows the load-strain relationship for the models with different slab thickness. The maximum measured strain was, approximately, 9770 $\mu\epsilon$ in the model with 200-mm slab thickness. This value of strain is 47% of the ultimate strain of the GFRP bars. Also, at the same load level in the post-cracking stage, increasing the slab thickness from 200 to 350 mm resulted

in 75% decrease in the strain. In addition, the strains in the models with 200, 250, 300 and 350-mm slab thickness were 7280, 4380, 3070 and 1960 $\mu\epsilon$, respectively, at the service load stage. These values represent 35, 21, 15, and 10% of the ultimate strain of the GFRP bar.

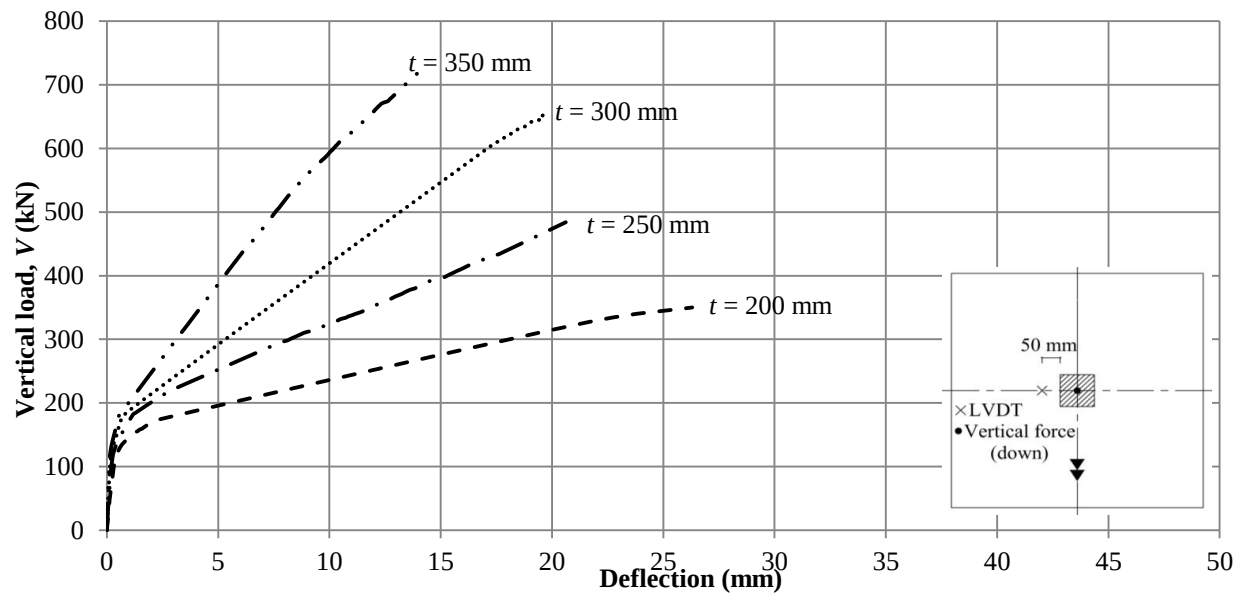


Figure 6.17: Load-deflection relationship for the models with different slab thickness

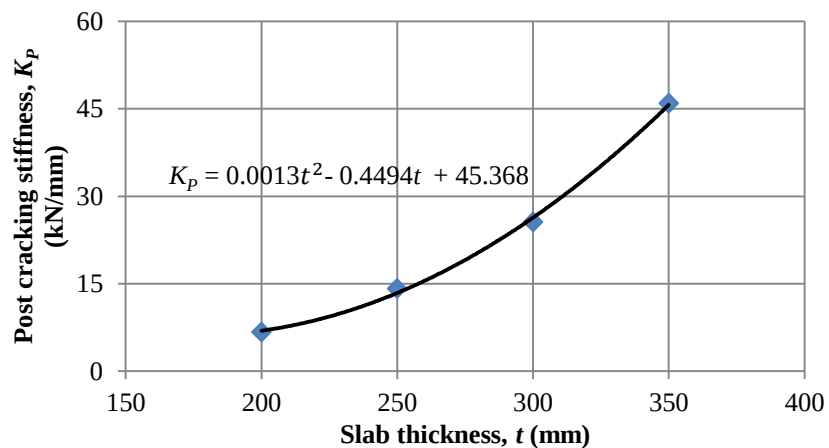


Figure 6.18: Relationship between the slab thickness and the post cracking stiffness

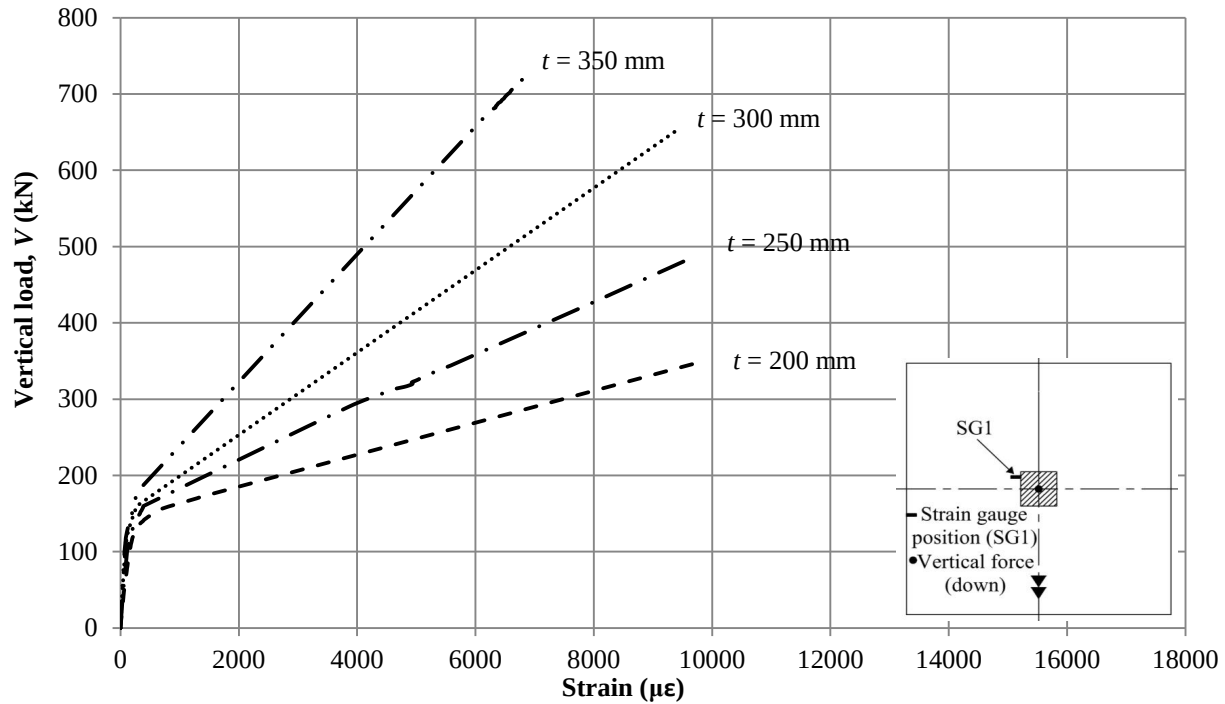


Figure 6.19: Load-strain relationship for the models with different slab thickness

6.5.3 Ultimate Capacity

The relationship between the vertical punching load and the slab thickness is presented in Fig. 6.20. Again, curve fitting was performed and according to the equation that was introduced in the figure, the relationship can be described in a quadratic relation. Also, as shown in the figure increasing the slab thickness from 200 to 350 mm resulted in 105% increase in the vertical punching load.

The relation between the slab thickness and the punching shear stresses calculated at $0.5 d$ (80 mm) from the column face at the failure stage is given in Fig. 6.21. Increasing the slab thickness from 200 to 350 mm reduced the punching shear stresses at failure from 1.65 to 1.21 MPa (27%) due to the size effect. These results are in very good agreement with the test results of Kevin (2000), where two identical steel-RC slabs, but with different slab thickness, were tested.

According to the test results 16% reduction in the punching shear stresses, from 1.98 to 1.67, was found for 22% increase in the slab thickness. Furthermore, the Canadian standard CSA/S806-12 (CSA 2012) did not take the effect of reducing the punching shear stresses due to the increase in slab thickness as seen in the figure.

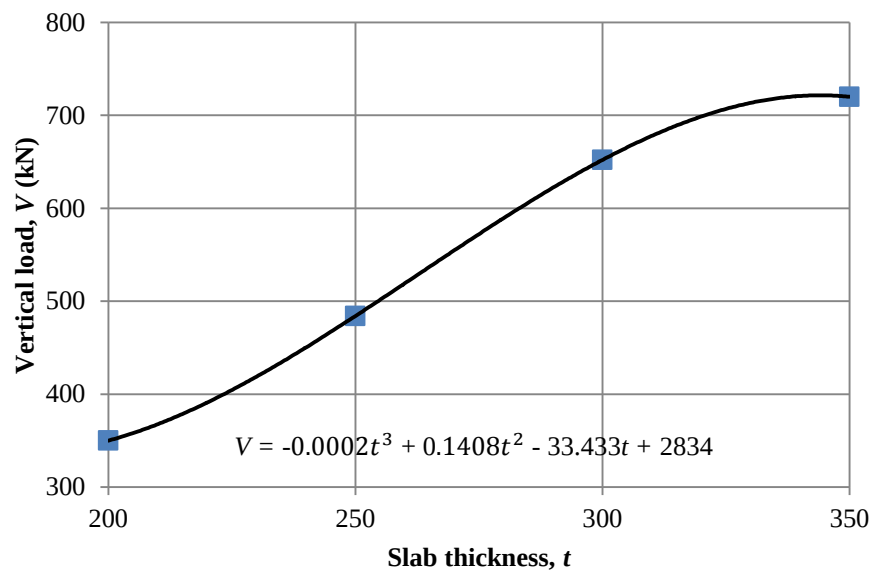


Figure 6.20: Relationship between the ultimate vertical load and the slab thickness

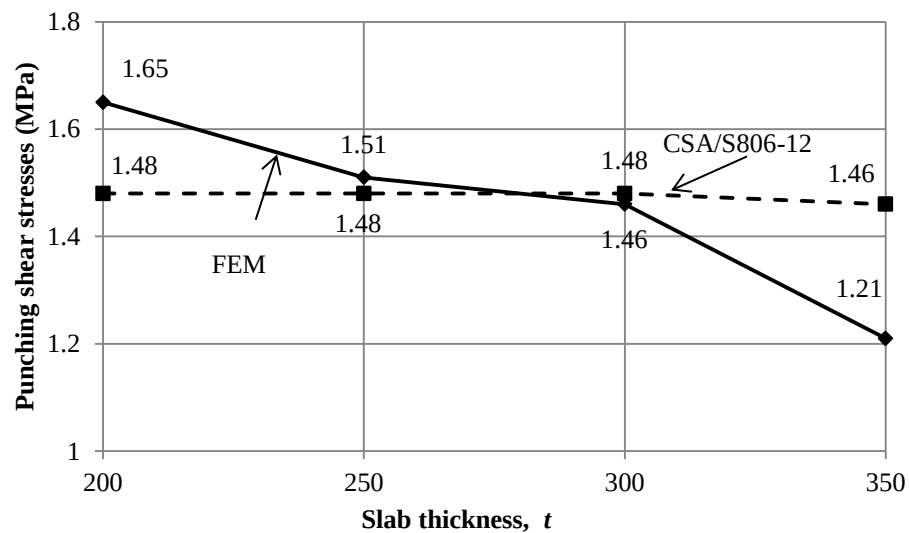


Figure 6.21: Relationship between the punching shear stress at failure and the slab thickness

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 SUMMARY

The behaviour of FRP-RC interior slab-column connections was investigated in the current study. This research consisted of two phases. The first is the experimental phase and the second is the parametric study using FE model. The analysis of the investigated parameters resulted in a number of findings regarding the behaviour of GFRP-RC interior slab-column specimens.

Ten full scale interior slab-column prototypes were constructed and tested in the experimental part. Vertical load was applied at the summit of the top column stub using a hydraulic actuator and to simulate the effect of the unbalanced moment two hydraulic jacks were used to apply two equal and opposite horizontal forces on the top and bottom column stubs. Five parameters were investigated in the experimental phase; flexural reinforcement ratio, concrete compressive strength, moment-to-shear ratio, type of reinforcing bars and shear reinforcement ratio.

Using specialized software, a finite element model was constructed in the analytical part to simulate the behaviour of FRP-RC interior slab-column connections. The results showed that the model was able to simulate the behaviour of the specimens with a reasonable accuracy. Consequently, a parametric study was carried out to investigate the effect of key parameters (perimeter-to-depth ratio, column aspect ratio, slab thickness and reinforcement ratio) on the behaviour of FRP-RC interior slab-column connections.

7.2 CONCLUSIONS

The following conclusions were drawn based on the discussion presented in Chapters 4 and 6.

7.2.1 Conclusions From the Experimental Phase

Series I:

- 1- All slabs exhibited a brittle punching shear failure. However, the axial stiffness of slab reinforcement significantly affects the punching shear capacity and maximum deflections. Compared to the control steel-RC slab ($\rho_s E_s$), specimens G-0.65-N-15-XX ($0.34 \rho_s E_s$), G-0.98-N-15-XX ($0.51 \rho_s E_s$) and G-1.30-N-15-XX ($0.68 \rho_s E_s$), showed 25, 20 and 10% lower capacity, respectively. Similarly, at service load level, the maximum deflection was increased by 206, 100 and 52%, respectively.
- 2- Increasing concrete compressive strength slightly enhanced the punching shear capacity of the GFRP-RC specimens. Increasing the concrete compressive strength by 67% increased the punching load and deflection at failure by 4.6 and 21%, respectively.
- 3- The unbalanced moment increased the deflection and strain in the reinforcing bars. The increase in the deflection for the GFRP-RC specimens varied between 9-12% and 7-8% at service and ultimate stages, respectively. Whereas, the corresponding values for the steel-RC slab were 8 and 13%, respectively.
- 4- The CSA/S806-12 standard (CSA 2012) and JSCE code (JSCE 1997) gave close predictions to the experimental loads of the test slabs with an average of $V_{exp.}/V_{CSA.} = 1.10 \pm 0.03$ (COV = 2.7%) and $V_{exp.}/V_{JSCE.} = 1.13 \pm 0.07$ (COV = 6.2%), respectively. However, the ACI 440.1R-06 guideline (ACI 2006) yielded more conservative predictions with an average of $V_{exp.}/V_{ACI.} = 1.93 \pm 0.12$ (COV = 6.20 %). On the other hand, Zaghoul's equation (2002) overestimated the capacity of the slabs by an average of $V_{exp.}/V_Z = 0.86 \pm 0.04$ (COV =

4.65%). Even though using the proposed equation [Eq. 4.2] yielded closer results to the punching capacity of slabs tested under both concentric and eccentric loads, the CSA equation [Eq. 2.50] may be considered acceptable in its current format.

- 5- When the actual concrete compressive strength was used, the predicted capacity for the HSC slab using CSA/S806-12 code (CSA 2012) yielded better results with $V_{exp.}/V_{CSA.} = 1.01$. Therefore, it may be beneficial to extend the 60 MPa limit currently specified by the code. However, further investigation is still needed to confirm this issue.

Series II and III:

- 6- All test slabs failed in punching shear characterized by a penetration of the column with part of the slab through the slab thickness.
- 7- Increasing the moment-to-shear ratio increased the deflection and reinforcement strain. On the other hand, increasing the moment-to-shear ratio resulted in a reduction in the vertical load capacity. A 17 and 32% reduction in the vertical shear force was observed when the moment-to-shear ratio was increased from 0.0 to 0.15 and 0.3 m, respectively.
- 8- The presence of GFRP shear studs increased the post-cracking stiffness, deflection at failure, and the load carrying capacity. The punching load capacity increased by approximately 18 and 23% in slabs with $0.75 d$ and $0.5 d$ stud spacing, respectively, compared to slab without shear studs. However, the GFRP shear studs did not prevent the punching shear failure.
- 9- The Canadian standard for steel-RC slabs, CSA/A23.3-14 (CSA 2014), requires that the shear reinforcement to be extended to a distance not less than $2.0 d$ from the column face. Based on the results reported in the current study and in literature (Zaghloul 2002 and 2007; Hassan et al. 2014), it seems that this limit needs to be extended in the case of GFRP-RC slabs. However, further investigation is still needed to quantify the adequate extent.

- 10- For the slabs without shear reinforcement, the Canadian CSA/S806-12 (CSA 2012) and Japanese JSCE-97 (JSCE 1997) standards yielded close, yet conservative, predictions to the slab capacity with $V_{exp.}/V_{CSA.} = 1.10 \pm 0.09$ (COV = 8.2%) and $V_{exp.}/V_{JSCE.} = 1.10 \pm 0.11$ (COV = 10.0%), respectively. On the other hand, the American guideline ACI 440.1R-06 (ACI 2006) highly underestimated the capacity with $V_{exp.}/V_{ACI.} = 1.99 \pm 0.17$ (COV = 8.6%).
- 11- For the slabs with shear stud reinforcement, the proposed equations based on revising the current provisions of the CSA/S806-12 (CSA 2012) showed good agreement with the experimental failure loads, with $V_{exp.}/V_{CSA.} = 1.16 \pm 0.08$ (COV = 6.9%); nevertheless, the proposed strain limit needs further investigation. On the other hand, the proposed modification to the American guideline ACI 440.1R-06 (ACI 2006) equations, overestimated the punching capacity of the slabs, with $V_{exp.}/V_{ACI.} = 1.75 \pm 0.19$ (COV = 10.9%), which is considered in good agreement with the trend of the American guideline provisions.

7.2.2 Conclusions From the Numerical Phase

- 1- The constructed model was able to predict the behaviour of the slab-column connections in terms of ultimate capacity, load-deflection curve and load-strain curve with a reasonable accuracy. The average experimental-to-FEM shear strength ratio was approximately 1.03. In addition, Eq. [6.1] resulted from the parametric study is similar to that adapted by the Canadian standards, CSA/S806-12 (CSA 2012) [Eq. 2.50], which emphasis the accuracy of the finite element model.
- 2- Increasing the reinforcement ratio from half to four times the balanced reinforcement ratio reduced the strain and deflection. Furthermore, a 93% increase in the the shear strength of the slabs was observed due to the increase in the reinforcement ratio by sevenfold, from 0.15 ($0.5 \rho_b$) to 1.2% ($4.0 \rho_b$).

- 3- For the models with the same slab thickness (200 mm) and reinforcement ratio (0.65%), deflection and strains decreased due to the increase in the side length of the square column. In addition, the test results showed 26 and 34% decrease in the punching shear stresses at failure due to 33 and 65% increase in the perimeter-to-depth ratio, respectively. Moreover, a modification factor for the current governing equation [Eq. 2.50] in the Canadian standard CSA/S806-12 (CSA 2012) was introduced [Eq. 6.2] to consider this effect; however, further investigation is still needed to address this issue.
- 4- Considerable enhancement in the post-cracking stiffness can be noticed due to the increase in column aspect ratio. In addition, increasing the column aspect ratio from one to five, in models with identical slab thickness (200 mm) and reinforcement ratio (0.65%), resulted in 95% increase in the punching strength.
- 5- The effect of column rectangularity vanished after a value greater than three, which agrees with the results reported in the literature for the steel-RC slabs.
- 6- Increasing the slab thickness increased the carrying load capacity of the connections. Also, 105% increase in the vertical shear force was found due to the increase in the slab thickness from 200 to 350 mm (75%) in slabs with 0.65% reinforcement ratio.
- 7- At service load stage, increasing the reinforcement ratio, slab thickness and column dimensions reduced the strain and deflection at the same load level. Furthermore, the models with 0.9, 1.05 and 1.2% reinforcement ratio and the same side length for the square column (300 mm) in addition to the ones with 0.65% reinforcement ratio and different column aspect ratio of 4.0 and 5.0, satisfied the Canadian standard CSA/S806-12 (CSA 2012) serviceability limits. Also, the models with 250, 300 and 350-mm slab thickness, and constant

reinforcement ratio (0.65%) and column aspect ratio (1.0), met the CSA/S806-12 (CSA 2012) serviceability limits.

7.3 RECOMMENDATIONS

7.3.1 Recommendations for Designers

- 1- In general, the Canadian standard for FRP-RC structures, CSA/S806-12 (CSA 2012), can adequately predict the punching shear capacity of interior GFRP-RC slab-column connections without shear reinforcement and subjected to eccentric loading.
- 2- The equations proposed in this study, based on modifying the Canadian standard CSA/S806-12 (CSA 2012), can reasonably predict the carrying load capacity of the slabs with shear stud reinforcement. However, the allowable strain limit needs further investigations especially for different GFRP products.

7.3.2 Recommendation for Future Work

- 1- The current study is limited to interior slab-column connections. In addition, experimental and analytical studies are needed to investigate the behaviour of edge and corner slab-column connections.
- 2- The experimental and analytical work was carried out under the effect of static loads only. The effect of dynamic load should also be investigated for better understanding for the behaviour of that type of connections.
- 3- Since the current study was conducted using GFRP bars. Other studies should be executed to investigate the behaviour of the slab-column connections reinforced with different types of bars, such as CFRP and AFRP.
- 4- More variables are needed to be investigated such as the effect of openings.

- 5- The effect of high strength concrete and shear reinforcement need more investigations especially with higher reinforcement ratio.

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APPENDIX A

DESIGN OF TEST SPECIMENS

1. Design Criteria:

$b = 2800$ mm, $h = 200$ mm, clear cover = 25 mm

Concrete strain before crushing = 0.0035

Steel bars properties:

Size No.15M, bar diameter = 15.9 mm, area = 200 mm^2 , $f_y = 480$ MPa, $E = 200$ GPa

Sand-Coated GFRP bar properties:

Size No.16, bar diameter = 15.9 mm, area = 198 mm^2 , maximum tensile strength

$f_{frpu} = 1398$ MPa, $E_{frp} = 68$ GPa, $\varepsilon_{frpu} = 2.05$ %

Deformed-Ribbed GFRP bar properties:

Size No.16, bar diameter = 16 mm, area = 201 mm^2 , maximum tensile strength

$f_{frpu} = 1122$ MPa, $E_{frp} = 63.1$ GPa, $\varepsilon_{frpu} = 1.78$ %

Design of Specimens G-0.65-N-15-XX

Design for flexure according to CSA/S806-12

(Sand-Coated, $\rho_{FRP} = 0.65\%$, $f'_c = 42$ MPa and $A_s = NO. 16 @ 192$ mm)

Materials and Sectional properties:

$$\varepsilon_{cu} = 0.0035 \quad t = 200 \text{ mm} \quad b = 2800 \text{ mm} \quad d = 160 \text{ mm}$$

$$f_{frp,u} = 1398 \text{ MPa} \quad \phi_{frp} = 1 \quad E_{frp} = 68 \text{ GPa} \quad \varepsilon_{frp,u} = 0.02 \text{ (Sand-Coated)}$$

$$f'_c = 42 \text{ MPa} \quad \phi_c = 1$$

Clause 8.4.1.5 defined the stress distribution of FRP-RC cross-section. Uniformly distributed concrete stress of $\alpha_1 f'_c \phi_c$ is assumed to be distributed over a distance of $a = \beta_1 c$.

$$\alpha_1 = 0.85 - 0.0015 f'_c = 0.787 \quad \beta_1 = 0.97 - 0.0025 f'_c = 0.865$$

According to Clause 8.4.2.3, the spacing between reinforcement shall not be greater than the minimum of three times the slab thickness (600 mm) or 300 mm.

$$S = 192 \text{ mm} < 300 \text{ mm}$$

$$\rho_{balance} = \alpha_1 \beta_1 \frac{\phi_c}{\phi_{frp}} \frac{f'_c}{f_{frp,u}} \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{frp,u}} \right)$$

$$\rho_{balance} = 0.787 \times 0.865 \times 1 \times \frac{42}{1398} \left(\frac{0.0035}{0.0035 + 0.02} \right) = 0.3\%$$

$$\rho_{frp} = \frac{A_{frp}}{Sd} = \frac{198}{192 \times 160} = 0.65\%$$

$$\frac{\rho_{frp}}{\rho_{balance}} = 2.16 > 1 \quad \dots\dots\dots \text{(Compression failure), According to Clause 8.2.1 the failure of}$$

FRP-RC element shall be initiated by crushing of the concrete in the compression zone rather than rupture of FRP bars.

From equilibrium and strain compatibility:

$$f_{frp} = \frac{1}{2} E_{frp} \varepsilon_{cu} \left[\left(1 + \frac{4 \alpha_1 \beta_1 \phi_c f'_c}{\rho_{frp} \phi_{frp} E_{frp} \varepsilon_{cu}} \right)^{0.5} - 1 \right]$$

$$f_{frp} = 0.5 \times 68 \times 10^3 \times 0.0035 \left[\left(1 + \frac{4 \times 0.787 \times 0.865 \times 1 \times 42}{0.0065 \times 1 \times 68 \times 10^3 \times 0.0035} \right)^{0.5} - 1 \right]$$

$$= 911.07 \text{ MPa} < 1398 \text{ MPa} \quad OK$$

$$a = \beta_1 c = (\phi_{frp} A_{frp} f_{frp}) \div (\alpha_1 \phi_c f'_c b)$$

$$a = (1 \times 198 \times 911.07) \div (1 \times 0.787 \times 42 \times 192) = 28.42 \text{ mm}$$

$$c = a / \beta_1 = 28.42 / 0.865 = 32.85 \text{ mm}$$

Check concrete strain:

According to Clause 8.4.1.4 concrete strain shall be assumed to have reached 0.0035 provided that the ratio of C/d satisfies the following equation:

$$C/d \geq 7 / (7 + 2000 \varepsilon_{fu})$$

$$32.85 / 160 = 0.2 \geq 7 / (7 + 2000 \times 0.02) = 0.149$$

$$C_c = \alpha_1 \phi_c f'_c \beta_1 c b = 0.787 \times 1 \times 42 \times 28.42 \times 2800 = 2630.3 \text{ kN}$$

$$T_{frp} = \phi_{frp} A_{frp} f_{frp} = 1 \times 2887.5 \times 911.07 = 2630.3 \text{ kN}$$

Equilibrium is satisfied $T = C_c$

$$M_r = C_c \left(d - \frac{\beta_1 c}{2} \right) = 2630.3 \left(160 - \frac{28.42}{2} \right) = 383 \text{ kN.m}$$

$$M_r = 383 \text{ kN.m}$$

Check for minimum reinforcement:

According to Clause 8.4.2.1 the minimum reinforcement shall be satisfy the following condition

$$M_r > 1.5 M_{cr}$$

M_{cr} (cracking moment) shall be calculated as follows (Clause 8.3.2.6):

$$M_{Cr} = \frac{F_r \times I_g}{y_t}$$

Where: F_r is the Concrete modulus of rupture (Clause 8.3.2.8):

$$F_r = 0.6 \times \lambda \times \sqrt{f'_c}$$

Where: λ is equals to unity for normal weight concrete.

$$M_{Cr} = \frac{F_r \times I_g}{y_t} = \frac{0.6 \times 1 \times \sqrt{42} \times 2800 \times 200^3}{12 \times 100} = 72.5 \text{ kN.m}$$

$$M_r = 383 \text{ kN.m} > 1.5 \times 72.5 = 108.75 \text{ kN.m (minimum reinforcement satisfied)}$$

Design of Specimen R-0.65-N-15-XX

Design for flexure according to CSA/S806-12

(Ribbed-deformed, $\rho_{FRP} = 0.65\%$, $f'_c = 40$ MPa and $A_s = NO.16 @ 192$ mm)

Materials and Sectional properties:

$$\varepsilon_{cu} = 0.0035 \quad t = 200 \text{ mm} \quad b = 2800 \text{ mm} \quad d = 160 \text{ mm}$$

$$f_{frp,u} = 1122 \text{ MPa} \quad \phi_{frp} = 1 \quad E_{frp} = 63.1 \text{ GPa} \quad \varepsilon_{frp,u} = 0.0178 \text{ (Ribbed-deformed)}$$

$$f'_c = 40 \text{ MPa} \quad \phi_c = 1$$

$$\alpha_1 = 0.85 - 0.0015f'_c = 0.79 \quad \beta_1 = 0.97 - 0.0025f'_c = 0.87$$

According to Clause 8.4.2.3, the spacing between reinforcement shall not be greater than the minimum of three times the slab thickness (600 mm) or 300 mm.

$$S = 192 \text{ mm} < 300 \text{ mm}$$

$$\rho_{balance} = \alpha_1 \beta_1 \frac{\phi_c}{\phi_{frp}} \frac{f'_c}{f_{frp,u}} \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{frp,u}} \right)$$

$$\rho_{balance} = 0.79 \times 0.87 \times 1 \times \frac{40}{1122} \left(\frac{0.0035}{0.0035 + 0.02} \right) = 0.38\%$$

$$\rho_{frp} = \frac{A_{frp}}{Sd} = \frac{201}{192 \times 160} = 0.65\%$$

$$\frac{\rho_{frp}}{\rho_{balance}} = 1.7 > 1 \text{ (Compression failure)}$$

From equilibrium and strain compatibility:

$$f_{frp} = \frac{1}{2} E_{frp} \varepsilon_{cu} \left[\left(1 + \frac{4\alpha_1 \beta_1 \phi_c f'_c}{\rho_{frp} \phi_{frp} E_{frp} \varepsilon_{cu}} \right)^{0.5} - 1 \right]$$

$$f_{frp} = 0.5 \times 63.1 \times 10^3$$

$$\times 0.0035 \left[\left(1 + \frac{4 \times 0.79 \times 0.87 \times 1 \times 40}{0.0065 \times 1 \times 63.1 \times 10^3 \times 0.0035} \right)^{0.5} - 1 \right]$$

$$= 864.7 \text{ MPa} < 1122 \text{ MPa} \quad OK$$

$$a = \beta_1 c = (\phi_{frp} A_{frp} f_{frp}) \div (\alpha_1 \phi_c f'_c b)$$

$$a = (1 \times 201 \times 864.7) \div (1 \times 0.79 \times 40 \times 192) = 28.64 \text{ mm}$$

$$c = a / \beta_1 = 28.64 / 0.87 = 32.91 \text{ mm}$$

Check concrete strain:

According to Clause 8.4.1.4 concrete strain shall be assumed to have reached 0.0035 provided that the ratio of C/d satisfies the following equation:

$$C/d \geq 7 / (7 + 2000 \epsilon_{fu})$$

$$32.91 / 160 = 0.2 \geq 7 / (7 + 2000 \times 0.0178) = 0.16$$

$$C_c = \alpha_1 \phi_c f'_c \beta_1 c b = 0.79 \times 1 \times 40 \times 28.64 \times 2800 = 2534 \text{ kN}$$

$$T_{frp} = \phi_{frp} A_{frp} f_{frp} = 1 \times 2931.25 \times 864.7 = 2534 \text{ kN}$$

Equilibrium is satisfied $T = C_c$

$$M_r = C_c \left(d - \frac{\beta_1 c}{2} \right) = 2534 \left(160 - \frac{28.64}{2} \right) = 369 \text{ kN.m}$$

$$M_r = 369 \text{ kN.m}$$

Check for minimum reinforcement:

According to Clause 8.4.2.1 the minimum reinforcement shall be satisfy the following condition

$$M_r > 1.5 M_{Cr}$$

M_{Cr} (cracking moment) shall be calculated as follows (Clause 8.3.2.6):

$$M_{Cr} = \frac{F_r \times I_g}{y_t}$$

Where: F_r is the Concrete modulus of rupture (Clause 8.3.2.8):

$$F_r = 0.6 \times \lambda \times \sqrt{f'_c}$$

Where: λ is equals to unity for normal weight concrete.

$$M_{Cr} = \frac{F_r \times I_g}{y_t} = \frac{0.6 \times 1 \times \sqrt{40} \times 2800 \times 200^3}{12 \times 100} = 70.8 \text{ kN.m}$$

$$M_r = 369 \text{ kN.m} > 1.5 \times 70.8 = 106.2 \text{ kN.m (minimum reinforcement satisfied)}$$

Design of Specimen S-0.65-N-15-XX

Design for flexure according to CSA/A23.3-14

(Steel bars, $\rho_{FRP} = 0.65\%$, $f'_c = 42$ MPa and $A_s = No. 15M @ 192$ mm)

Materials and Sectional properties:

$$\varepsilon_{cu} = 0.0035 \quad t = 200 \text{ mm} \quad b = 2800 \text{ mm} \quad d = 160 \text{ mm}$$

$$f_y = 480 \text{ MPa} \quad \phi_s = 1 \quad E_s = 200 \text{ GPa} \quad \varepsilon_s = 0.0024$$

$$f'_c = 42 \text{ MPa} \quad \phi_c = 1$$

Clause 10.1.7 defined the stress distribution of Steel-RC cross-section. Uniformly distributed concrete stress of $\alpha_1 f'_c \phi_c$ is assumed to be distributed over a distance of $a = \beta_1 c$.

$$\alpha_1 = 0.85 - 0.0015 f'_c = 0.787 \quad \beta_1 = 0.97 - 0.0025 f'_c = 0.865$$

$$\rho_{frp} = \frac{A_{frp}}{Sd} = \frac{198}{192 \times 160} = 0.65\%$$

According to Clause 13.10.4, the spacing between reinforcement shall not be greater than the minimum of three times the slab thickness (600 mm) or 500 mm.

$$S = 192 \text{ mm} < 500 \text{ mm}$$

According to Clause 10.5.2, the tension reinforcement can be assumed to reach yielding if

$$c/d \leq 700/(700 + f_y)$$

From equilibrium and strain compatibility:

$$a = \beta_1 c = (\phi_s A_s f_y) \div (\alpha_1 \phi_c f'_c b)$$

$$a = (1 \times 200 \times 480) \div (1 \times 0.787 \times 42 \times 192) = 15.1 \text{ mm}$$

$$c = a/\beta_1 = 15.1/0.865 = 17.5 \text{ mm}$$

$$17.5/160 = 0.11 < 700/(700 + 480) = 0.6 \text{ Reinforcement yielded.}$$

$$M_r = \phi_s A_s f_y \left(d - \frac{\beta_1 c}{2} \right) = 1 \times 2916.67 \times 480 \left(160 - \frac{15.1}{2} \right) = 213 \text{ kN.m}$$

$$\mathbf{M_r = 213 \text{ kN.m}}$$

APPENDIX B

SERVICEABILITY REQUIREMENTS

General:

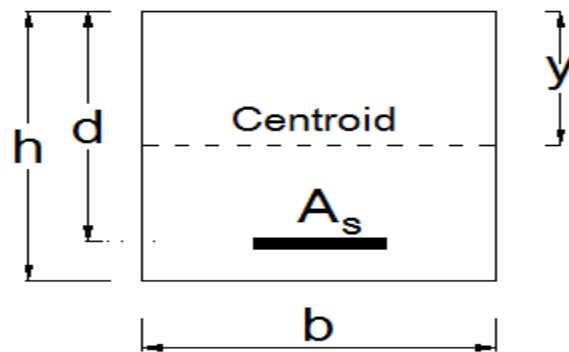
Parking garage with 6.5-m equal bays in the both orthogonal directions was simulated as described in chapter 3. The following section will discuss the serviceability requirements of a bay taken from the building according to the Canadian standard CSA/S806-12 (Z factor) and the American code ACI 440.1 R-06 (crack width) for some of the tested slabs.

For specimen G-0.65-N-15-XX (reinforced with sand-coated GFRP bars)

$$f'_c = 42 \text{ MPa} \quad E_c = 4500 \times \sqrt{f'_c} = 29.16 \text{ GPa} \quad E_{FRP} = 68 \text{ GPa} \quad n = \frac{E_{FRP}}{E_c} = \frac{68}{29.16} = 2.33$$

$$A_s = NO.16 @ 192 \text{ mm} = 198 \text{ mm}^2 @ 192 \text{ mm} = 6703 \text{ mm}^2 @ 6500 \text{ mm} \quad b = 6500 \text{ mm}$$

$$h = 200 \text{ mm} \quad d = 160 \text{ mm} \quad \rho = \frac{A_s}{bd} = 0.0065$$



The cracked centroid:

$$k = (\sqrt{2\rho n + (\rho n)^2} - \rho n)$$

$$k = \sqrt{2 \times 0.0065 \times 2.33 + (0.0065 \times 2.33)^2} - 0.0065 \times 2.33 = 0.15$$

$$y = k \times d = 0.15 \times 160 = 25.5 \text{ mm}$$

Loads: (NBCC 2010)

$$W_{service} = DL + L = (24 \times 0.2 + 1) + 2.4 = 8.2 \text{ kN / m}^2$$

Moment: Clauses 13.9.2.2 & 13.9.3.1 (A23.3-04)

$$M_O = \frac{W \times L_{2a} \times L_n^2}{8}$$

$$L_n = 6500 - 150 - 150 = 6200 \text{ mm} = 6.2 \text{ m}$$

$$L_{2a} = 6.5 \text{ m}$$

$$M_O = \frac{W \times L_{2a} \times L_n^2}{8} = \frac{8.2 \times 6.5 \times 6.2^2}{8} = 256.1 \text{ kN.m}$$

$$\text{Negative moment} = 0.65 \times M_O = 0.65 \times 256.1 = 166.4 \text{ kN.m}$$

$$\text{Positive moment} = 0.35 \times M_O = 0.35 \times 256.1 = 89.7 \text{ kN.m}$$

Stresses in FRP bars:

$$f_{frp} = \frac{M_S}{A_{frp} \times J \times d}$$

$$J = 1 - \frac{k}{3} = 1 - \frac{0.15}{3} = 0.95$$

$$f_{frp} = \frac{M_S}{A_{frp} \times J \times d} = \frac{166.4 \times 10^6}{6703 \times 0.95 \times 160} = 163.32 \text{ MPa} < 0.25 \times 1398 = 349.5 \text{ MPa} \text{ Clause 7.1.2.2 (S806-$$

12)

1) CSA/S806-12 (CSA 2012)

$$Z = k_b (E_S / E_{frp}) f_{frp} (A d_c)^{\frac{1}{3}} \leq 38000 \frac{N}{\text{mm}} \text{ for extenarl exposure} \quad \text{Clause 8.3.1.1}$$

Where E_{frp} = modulus of elasticity of FRP bar, MPa

E_S = modulus of elasticity of steel bar, MPa

f_{frp} = stress in FRP reinforcement in tension, MPa

d_c = thickness of concrete cover measured from extreme tension fibre to centre of bar,
mm

A = effective tension area of concrete surrounding the flexural tension reinforcement
and extending from the extreme tension fibre to the centroid of the flexural
tension reinforcement and an equal distance past the centroid, mm²

$$d_c = 25 + 15.9/2 = 32.95 \text{ mm}$$

$$A = \frac{2 \times 192 \times 32.95}{1} = 12652.8 \text{ mm}^2$$

For $k_b = 0.8$ (as recommended by the manufacturer)

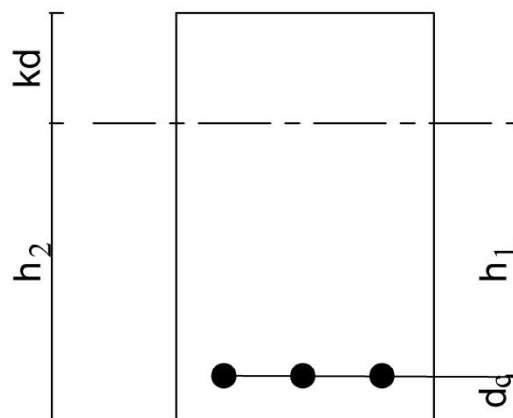
$$Z = 0.8 \times (200000/68000) \times 163.32 \times (12652.8 \times 32.95)^{\frac{1}{3}} = 28707 < 38000 \frac{N}{mm}$$

Safe

For $k_b = 1.2$ as recommended by CSA/S806-12 (CSA 2012)

$$Z = 1.2 \times (200000/68000) \times 163.32 \times (12652.8 \times 32.95)^{\frac{1}{3}} = 43060 > 38000 \frac{N}{mm}$$

Unsafe



2) ACI 440.1R-06 (ACI 2006)

$$w = 2.0 \times k_b \times \left(\frac{f_{frp}}{E_{frp}} \right) \times \beta \times \sqrt{d_c^2 + \left(\frac{s}{2} \right)^2} \quad \text{Clause 8.3.1}$$

Where w = maximum crack width, mm

E_{frp} = modulus of elasticity of FRP bar, MPa

f_{frp} = stress in FRP reinforcement in tension, MPa

β = ratio of distance from neutral axis to extreme tension fibre to distance from neutral axis to centre of tensile reinforcement

d_c = thickness of concrete cover measured from extreme tension fibre to centre of bar, mm

s = longitudinal FRP bar spacing, mm

For $k_b = 0.8$

$$w = 2.0 \times 0.8 \times \frac{163.32}{68000} \times \frac{174.5}{141.55} \times \sqrt{32.95^2 + \left(\frac{192}{2}\right)^2} = 0.48 \text{ mm} < 0.5 \text{ mm } \underline{\text{safe}}$$

For $k_b = 1.2$

$$w = 2.0 \times 1.2 \times \frac{163.32}{68000} \times \frac{174.5}{141.55} \times \sqrt{32.95^2 + \left(\frac{192}{2}\right)^2} = 0.72 \text{ mm} > 0.5 \text{ mm } \underline{\text{Unsafe}}$$

For $k_b = 1.4$ as recommended by ACI 440.1 R-06 (ACI 2006)

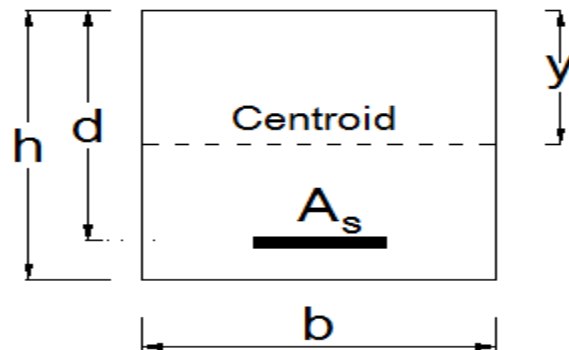
$$w = 2.0 \times 1.4 \times \frac{163.32}{68000} \times \frac{174.5}{141.55} \times \sqrt{32.95^2 + \left(\frac{192}{2}\right)^2} = 0.84 \text{ mm} > 0.5 \text{ mm } \underline{\text{Unsafe}}$$

For specimen G-0.98-N-15-XX (reinforced with sand-coated GFRP bars)

$$f'_c = 38 \text{ MPa} \quad E_c = 4500 \times \sqrt{f'_c} = 27.74 \text{ GPa} \quad E_{FRP} = 68 \text{ GPa} \quad n = \frac{E_{FRP}}{E_c} = \frac{68}{27.74} = 2.45$$

$$A_s = NO.16 @ 128 \text{ mm} = 198 \text{ mm}^2 @ 192 \text{ mm} = 10054 \text{ mm}^2 @ 6500 \text{ mm} \quad b = 6500 \text{ mm}$$

$$h=200 \text{ mm} \quad d = 160 \text{ mm} \quad \rho = \frac{A_s}{bd} = 0.0098$$



The cracked centroid:

$$k = (\sqrt{2\rho n + (\rho n)^2} - \rho n)$$

$$k = \sqrt{2 \times 0.0098 \times 2.45 + (0.0098 \times 2.45)^2} - 0.0098 \times 2.45 = 0.19$$

$$y = k \times d = 0.19 \times 160 = 30.4 \text{ mm}$$

Loads: (NBCC 2010)

$$W_{service} = DL + L = (24 \times 0.2 + 1) + 2.4 = 8.2 \text{ kN / m}^2$$

Moment: Clauses 13.9.2.2 & 13.9.3.1 (A23.3-04)

$$M_O = \frac{W \times L_{2a} \times L_n^2}{8}$$

$$L_n = 6500 - 150 - 150 = 6200 \text{ mm} = 6.2 \text{ m}$$

$$L_{2a} = 6.5 \text{ m}$$

$$M_O = \frac{W \times L_{2a} \times L_n^2}{8} = \frac{8.2 \times 6.5 \times 6.2^2}{8} = 256.1 \text{ kN.m}$$

$$\text{Negative moment} = 0.65 \times M_O = 0.65 \times 256.1 = 166.4 \text{ kN.m}$$

Positive moment = $0.35 \times M_0 = 0.35 \times 256.1 = 89.7 \text{ kN.m}$

Stresses in FRP bars:

$$f_{frp} = \frac{M_S}{A_{frp} \times J \times d}$$

$$J = 1 - \frac{k}{3} = 1 - \frac{0.19}{3} = 0.936$$

$$f_{frp} = \frac{M_S}{A_{frp} \times J \times d} = \frac{166.4 \times 10^6}{10054 \times 0.936 \times 160} = 110.5 \text{ MPa} < 0.25 \times 1398 = 349.5 \text{ MPa} \text{ Clause 7.1.2.2}$$

(S806-12)

1) CSA/S806-12 (CSA 2012)

$$Z = k_b (E_S / E_{frp}) f_{frp} (A d_c)^{\frac{1}{3}} \leq 38000 \frac{N}{\text{mm}} \text{ for exteranl exposure} \quad \text{Clause 8.3.1.1}$$

$$d_c = 25 + 15.9/2 = 32.95 \text{ mm}$$

$$A = \frac{2 \times 192 \times 32.95}{1} = 12652.8 \text{ mm}^2$$

For $k_b = 0.8$ (as recommended by the manufacturer)

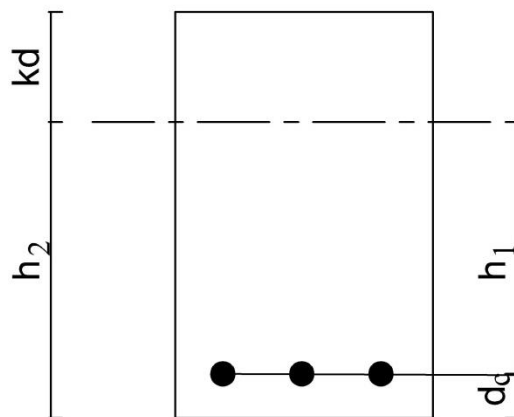
$$Z = 0.8 \times (200000/68000) \times 110.5 \times (12652.8 \times 32.95)^{\frac{1}{3}} = 19425 < 38000 \frac{N}{\text{mm}}$$

Safe

For $k_b = 1.2$ as recommended by CSA/S806-12 (CSA 2012)

$$Z = 1.2 \times (200000/68000) \times 163.32 \times (12652.8 \times 32.95)^{\frac{1}{3}} = 29133 < 38000 \frac{N}{\text{mm}}$$

Safe



2) ACI 440.1 R-06 (ACI 2006)

$$w = 2.0 \times k_b \times \left(\frac{f_{frp}}{E_{frp}} \right) \times \beta \times \sqrt{d_c^2 + \left(\frac{s}{2} \right)^2} \quad \text{Clause 8.3.1}$$

For $k_b = 0.8$

$$w = 2.0 \times 0.8 \times \frac{110.5}{68000} \times \frac{169.6}{136.65} \times \sqrt{32.95^2 + \left(\frac{128}{2} \right)^2} = 0.23 \text{ mm} < 0.5 \text{ mm } \underline{\text{safe}}$$

For $k_b = 1.2$

$$w = 2.0 \times 1.2 \times \frac{110.5}{68000} \times \frac{169.6}{136.65} \times \sqrt{32.95^2 + \left(\frac{128}{2} \right)^2} = 0.35 \text{ mm} < 0.5 \text{ mm } \underline{\text{safe}}$$

For $k_b = 1.4$ as recommended by ACI 440.1 R-06 (ACI 2006)

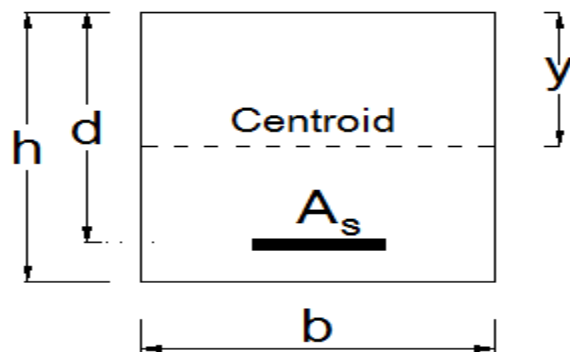
$$w = 2.0 \times 1.4 \times \frac{110.5}{68000} \times \frac{169.6}{136.65} \times \sqrt{32.95^2 + \left(\frac{128}{2} \right)^2} = 0.4 \text{ mm} < 0.5 \text{ mm } \underline{\text{safe}}$$

For specimen R-0.65-N-15-XX (reinforced with ribbed-deformed GFRP bars)

$$f'_c = 40 \text{ MPa} \quad E_c = 4500 \times \sqrt{f'_c} = 28.46 \text{ GPa} \quad E_{FRP} = 63.1 \text{ GPa} \quad n = \frac{E_{FRP}}{E_c} = \frac{63.1}{28.46} = 2.23$$

$$A_s = NO.16 @ 192 \text{ mm} = 201 \text{ mm}^2 @ 192 \text{ mm} = 6804 \text{ mm}^2 @ 6500 \text{ mm} \quad b = 6500 \text{ mm}$$

$$h = 200 \text{ mm} \quad d = 160 \text{ mm} \quad \rho = \frac{A_s}{bd} = 0.0065$$



The cracked centroid:

$$k = (\sqrt{2\rho n + (\rho n)^2} - \rho n)$$

$$k = \sqrt{2 \times 0.0065 \times 2.23 + (0.0065 \times 2.23)^2} - 0.0065 \times 2.23 = 0.15$$

$$y = k \times d = 0.15 \times 160 = 25.5 \text{ mm}$$

Loads: (NBCC 2010)

$$W_{service} = DL + L = (24 \times 0.2 + 1) + 2.4 = 8.2 \text{ kN / m}^2$$

Moment: Clauses 13.9.2.2 & 13.9.3.1 (A23.3-04)

$$M_O = \frac{W \times L_{2a} \times L_n^2}{8}$$

$$L_n = 6500 - 150 - 150 = 6200 \text{ mm} = 6.2 \text{ m}$$

$$L_{2a} = 6.5 \text{ m}$$

$$M_O = \frac{W \times L_{2a} \times L_n^2}{8} = \frac{8.2 \times 6.5 \times 6.2^2}{8} = 256.1 \text{ kN.m}$$

$$\text{Negative moment} = 0.65 \times M_O = 0.65 \times 256.1 = 166.4 \text{ kN.m}$$

Positive moment = $0.35 \times M_0 = 0.35 \times 256.1 = 89.7 \text{ kN.m}$

Stresses in FRP bars:

$$f_{frp} = \frac{M_S}{A_{frp} \times J \times d}$$

$$J = 1 - \frac{k}{3} = 1 - \frac{0.15}{3} = 0.95$$

$$f_{frp} = \frac{M_S}{A_{frp} \times J \times d} = \frac{166.4 \times 10^6}{6804 \times 0.95 \times 160} = 160.9 \text{ MPa} < 0.25 \times 1122 = 280 \text{ MPa} \text{ Clause 7.1.2.2 (S806-$$

12)

1) CSA/S806-12 (CSA 2012)

$$Z = k_b (E_S / E_{frp}) f_{frp} (A d_c)^{\left(\frac{1}{3}\right)} \leq 38000 \frac{N}{\text{mm}} \text{ for exteranl exposure} \quad \text{Clause 8.3.1.1}$$

$$d_c = 25 + 15.9/2 = 32.95 \text{ mm}$$

$$A = \frac{2 \times 192 \times 32.95}{1} = 12652.8 \text{ mm}^2$$

For $k_b = 0.8$ (as recommended by the manufacturer)

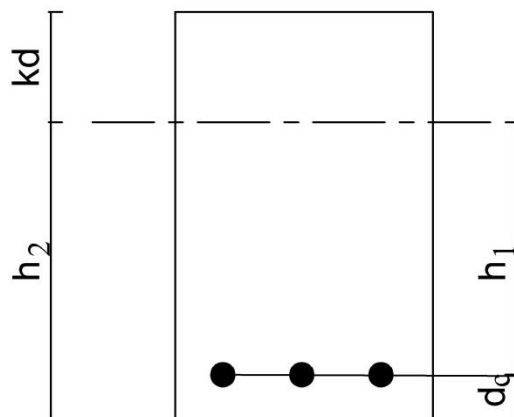
$$Z = 0.8 \times (200000/63100) \times 160.9 \times (12652.8 \times 32.95)^{\left(\frac{1}{3}\right)} = 30286 < 38000 \frac{N}{\text{mm}}$$

Safe

For $k_b = 1.2$ as recommended by CSA/S806-12 (CSA 2012)

$$Z = 1.2 \times (200000/63100) \times 163.32 \times (12652.8 \times 32.95)^{\left(\frac{1}{3}\right)} = 45429 > 38000 \frac{N}{\text{mm}}$$

Unsafe



2) ACI 440.1R-06 (ACI 2006)

$$w = 2.0 \times k_b \times \left(\frac{f_{frp}}{E_{frp}} \right) \times \beta \times \sqrt{d_c^2 + \left(\frac{s}{2} \right)^2} \quad \text{Clause 8.3.1}$$

For $k_b = 0.8$

$$w = 2.0 \times 0.8 \times \frac{160.9}{63100} \times \frac{174.5}{141.55} \times \sqrt{32.95^2 + \left(\frac{192}{2} \right)^2} = 0.5 \text{ mm } \underline{\text{safe}}$$

For $k_b = 1.2$

$$w = 2.0 \times 1.2 \times \frac{160.9}{63100} \times \frac{174.5}{141.55} \times \sqrt{32.95^2 + \left(\frac{192}{2} \right)^2} = 0.75 \text{ mm} > 0.5 \text{ mm } \underline{\text{Unsafe}}$$

For $k_b = 1.4$ as recommended by ACI 440.1 R-06 (ACI 2006)

$$w = 2.0 \times 1.4 \times \frac{160.9}{63100} \times \frac{174.5}{141.55} \times \sqrt{32.95^2 + \left(\frac{192}{2} \right)^2} = 0.875 \text{ mm} > 0.5 \text{ mm } \underline{\text{Unsafe}}$$

APPENDIX C

PUNCHING SHEAR CAPACITY OF THE TEST SPECIMENS

A) For the experimental part

The capacity for the GFRP specimens will be calculated according to the Canadian standard for FRP material CSA/S806-12 (CSA 2012), the American guideline ACI 440.1R-06 (ACI 2006), the Japanese code JSCE-97 (JSCE 1997) in addition to the proposed equation from Zaghoul (2002).

1) FRP control specimen G-0.65-N-15-XX

$$\rho_{FRP} = 0.65\%, f'_c = 42 \text{ MPa}, E_{frp} = 68 \text{ GPa}, \lambda = 1.0 \text{ and } \phi_c = 1.0$$

- CSA/S806-12 code

$$v_c = (1 + \frac{2}{\beta_c}) [0.028 \lambda \phi_c (E_F \rho_F f'_c)^{1/3}] = (1 + \frac{2}{1}) [0.028 \times 1 \times 1 \times (68000 \times 0.0065 \times 42)^{1/3}] = 2.22 \text{ MPa}$$

$$v_c = (0.19 + \frac{\alpha_s d}{b_o}) [0.147 \lambda \phi_c (E_F \rho_F f'_c)^{1/3}] = (0.19 + \frac{4 \times 160}{1840}) [0.147 \times 1 \times 1 (68000 \times 0.0065 \times 42)^{1/3}] = 2.09 \text{ MPa}$$

$$v_c = 0.056 \lambda \phi_c (E_F \rho_F f'_c)^{1/3} = 0.056 \times 1 \times 1 \times (68000 \times 0.0065 \times 42)^{1/3} = \underline{\underline{1.48 \text{ MPa}}}$$

The minimum shear stresses according to the code equations is **1.48 MPa**

$$v_c = \frac{V}{b_o d} + \frac{\gamma M_y}{J_c} x$$

$$J = \frac{1}{6} (460 \times (160)^3 + 160 \times (460)^3) + \frac{1}{2} (160 \times (460)^3) = 1.069 \times 10^{10} \text{ mm}^4$$

$$\beta_c = \frac{c_2}{c_1} = 1.0 \quad b_o = 4(160 + 300) = 1840 \text{ mm}^2$$

$$1.48 = \frac{V}{1840 \times 160} + \frac{0.4 \times 150V \times \frac{(160+300)}{2}}{1.069 \times 10^{10}}$$

$$\underline{V = 316 \text{ kN}} \quad \underline{M = 0.15 \times V = 47.4 \text{ kN.m}}$$

- ACI 440.1 R-06 guideline

$$E_c = 4750 \times \sqrt{f'_c} = 30.7 \text{ GPa} \quad E_{FRP} = 68 \text{ GPa} \quad n = \frac{E_{FRP}}{E_c} = \frac{68}{30.7} = 2.21$$

$$C = (\sqrt{2\rho n + (\rho n)^2} - \rho n) \times d$$

$$C = (\sqrt{2 \times 0.0065 \times 2.21 + (0.0065 \times 2.21)^2} - 0.0065 \times 2.21) \times 160 = 24.9 \text{ mm}$$

$$V = \frac{4}{5} \sqrt{f'_c} b_o C = \frac{4}{5} \sqrt{42} \times 1840 \times 24.9 = 237715 \text{ N}$$

$$v_c = \frac{V}{b_o d} = \frac{237715}{1840 \times 160} = \mathbf{0.81 \text{ MPa}}$$

$$0.81 = \frac{V}{1840 \times 160} + \frac{0.4 \times 150 V \times \frac{(160+300)}{2}}{1.069 \times 10^{10}}$$

$$\underline{V = 172 \text{ kN}} \quad \underline{M = 0.15 \times V = 25.8 \text{ kN.m}}$$

- JSCE-1997 code

$$\beta_d = \sqrt[4]{\frac{1000}{d}} \leq 1.5 = \sqrt[4]{\frac{1000}{160}} = 1.58 > 1.5 \quad \text{use } \beta_d = 1.5$$

$$\beta_p = \sqrt[3]{100 (E_F/E_S) \rho} \leq 1.5 = \sqrt[3]{100 (68000/200000) \times 0.0065} = 0.6046 < 1.5 \quad \text{use } \beta_p = 0.6046$$

$$\beta_r = 1 + \frac{1}{1+0.25u/d} = 1 + \frac{1}{1+0.25 \times 1200/160} = 1.347$$

$$f_{pcd} = 0.2 \sqrt{f'_c} \leq 1.2 = 0.2 \sqrt{42} = 1.3 > 1.2 \quad \text{use } f_{pcd} = 1.2$$

$$v_c = \beta_d \beta_p \beta_r \frac{f_{pcd}}{\gamma_b} = 1.5 \times 0.6046 \times 1.347 \times \frac{1.2}{1.0} = \mathbf{1.465 \text{ MPa}}$$

$$v_c = \frac{V}{b_o d} + \frac{\gamma M_y}{J_c}$$

$$J = \frac{1}{6} (460 \times (160)^3 + 160 \times (460)^3) + \frac{1}{2} (160 \times (460)^3) = 1.069 \times 10^{10} \text{ mm}^4$$

$$b_o = 4(160+300) = 1840 \text{ mm}^2$$

$$1.465 = \frac{V}{1840 \times 160} + \frac{0.4 \times 150V \times \frac{(160+300)}{2}}{1.069 \times 10^{10}}$$

$$\underline{V = 313 \text{ kN}} \quad \underline{M = 0.15 \times V = 47 \text{ kN.m}}$$

- Zaghloul (2002)

$$v_c = 0.07 \lambda \phi_c (E_F \rho_F f_c')^{1/3} = 0.07 \times 1 \times 1 \times (68000 \times 0.0065 \times 42)^{1/3} = \underline{\underline{1.85 \text{ MPa}}}$$

$$1.85 = \frac{V}{1840 \times 160} + \frac{0.4 \times 150V \times \frac{(160+300)}{2}}{1.069 \times 10^{10}}$$

$$\underline{V = 394 \text{ kN}} \quad \underline{M = 0.15 \times V = 59.1 \text{ kN.m}}$$

2) Specimen R-0.65-N-15-75

$\rho_{FRP} = 0.65\%$, $f'_c = 42$ MPa, $E_{frp} = 63.1$ GPa, $\lambda = 1.0$ and $\phi_c = 1.0$

- Proposed equation based on CSA/S806-12 code

Inside the shear reinforcement zone

$$v_c = 0.041 \lambda \phi_c (E_F \rho_F f'_c)^{1/3} = 0.041 \times 1 \times 1 \times (63100 \times 0.0065 \times 42)^{1/3} = \mathbf{1.06 \text{ MPa}}$$

$$v_s = \frac{\phi_s A_{vs} f_{frpv}}{b_o s} = 1 \times (8 \times 113) \times (0.002 \times 60000) / 1840 \times 120 = \mathbf{0.49 \text{ MPa}}$$

$$v_r = v_c + v_s = 1.06 + 0.49 = \mathbf{1.55 \text{ MPa}}$$

$$v_c = \frac{V}{b_o d} + \frac{\gamma M_y}{J_c} X$$

$$1.55 = \frac{V}{1840 \times 160} + \frac{0.4 \times 150V \times \frac{(160+300)}{2}}{1.069 \times 10^{10}}$$

$$\underline{V = 331 \text{ kN}} \quad \underline{M = 0.15 \times V = 49.65 \text{ kN.m}}$$

Outside the shear reinforcement zone

$$v_c = 0.028 \lambda \phi_c (E_F \rho_F f'_c)^{1/3} = 0.028 \times 1 \times 1 \times (63100 \times 0.0065 \times 42)^{1/3} = \mathbf{0.72 \text{ MPa}}$$

The punching shear stresses calculations at distance of $3.9 d$ from the column faces ($3.9 \times 160 = 624$ mm) are shown in the next few lines.

$$J = \frac{d}{3} \sum l_i (y_i^2 + y_i \times y_j + y_j^2)$$

$$= \frac{160}{3} ((2 \times 300 \times (150^2 + (-150 \times 150) + (-150^2))) + (2 \times 300 \times (774^2 + 774 \times 774 + 774^2)) + (4 \times 624 \times \sqrt{2} \times (150^2 + 150 \times 774 + 774^2))) = 1.971 \times 10^{11}$$

$$b_o = 4(624 \times \sqrt{2} + 300) = 4729.87 \text{ mm}$$

$$v_c = \frac{V}{b_o d} + \frac{\gamma M_y}{J_c} X$$

$$0.72 = \frac{V}{4729.87 \times 160} + 0.4 \times \frac{150V \times (774)}{1.971 \times 10^{11}}$$

$$\underline{V= 462 \text{ kN}} \quad \underline{M=0.15 \times V=69.3 \text{ kN.m}}$$

- Proposed equation based on ACI 440.1 R-06 guideline

$$E_c = 4750 \times \sqrt{f'_c} = 30.7 \text{ GPa} \quad E_{FRP} = 63.1 \text{ GPa} \quad n = \frac{E_{FRP}}{E_c} = \frac{63.1}{29.1} = 2.05$$

$$C = (\sqrt{2\rho n + (\rho n)^2} - \rho n) \times d$$

$$C = (\sqrt{2 \times 0.0065 \times 2.05 + (0.0065 \times 2.05)^2} - 0.0065 \times 2.05) \times 160 = 24.1 \text{ mm}$$

Inside the shear reinforcement zone

$$V = \frac{3}{5} \sqrt{f'_c} b_o C = \frac{3}{5} \sqrt{42} \times 1840 \times 24.1 = 172429 \text{ N}$$

$$v_c = \frac{V}{b_o d} = \frac{172429}{1840 \times 160} = \underline{\underline{0.6 \text{ MPa}}}$$

$$v_s = \frac{\phi_s A_{vs} f_{frpv}}{b_o S} = 1 \times (8 \times 113) \times (0.002 \times 60000) / 1840 \times 120 = \underline{\underline{0.49 \text{ MPa}}}$$

$$v_r = v_c + v_s = 0.6 + 0.49 = \underline{\underline{1.09 \text{ MPa}}}$$

$$1.09 = \frac{V}{1840 \times 160} + \frac{0.4 \times 150V \times \frac{(160+300)}{2}}{1.069 \times 10^{10}}$$

$$\underline{V= 233 \text{ kN}} \quad \underline{M=0.15 \times V=34.95 \text{ kN.m}}$$

Outside the shear reinforcement zone

$$V = 0.4 \sqrt{f'_c} b_o C = 0.4 \sqrt{42} \times 1840 \times 24.1 = 114952 \text{ N}$$

$$v_c = \frac{V}{b_o d} = \frac{114952}{1840 \times 160} = \underline{\underline{0.39 \text{ MPa}}}$$

$$0.39 = \frac{V}{4729.87 \times 160} + 0.4 \times \frac{150V \times (774)}{1.971 \times 10^{11}}$$

$$\underline{V= 250 \text{ kN}} \quad \underline{M=0.15 \times V=37.5 \text{ kN.m}}$$

B) For the parametric study

The punching shear stresses for the models will be obtained according to the Canadian code CSA/S806-12 (CSA 2012).

1) Reinforcement ratio:**Model 8:**

$$\rho_{FRP} = 1.20\% = 4.0 \times \rho_b, V = 410 \text{ kN and } M = 61.5 \text{ kN.m}$$

$$v_u = \frac{V}{b_o d} + \frac{\gamma M_y}{J_c} X$$

$$v_u = \frac{410 \times 10^3}{1840 \times 160} + \frac{0.4 \times 61.5 \times 10^6 \times \frac{(160+300)}{2}}{1.069 \times 10^{10}}$$

$$v_u = 1.92 \text{ MPa}$$

2) Perimeter-to-depth ratio:**Model 3:**

$$\Sigma o/d = 19, \rho_{FRP} = 0.65\%, V = 432 \text{ kN and } M = 64.8 \text{ kN.m}$$

$$v_u = \frac{V}{b_o d} + \frac{\gamma M_y}{J_c} X$$

$$J = \frac{1}{6}(760 \times (160)^3 + 160 \times (760)^3) + \frac{1}{2}(160 \times (760)^3) = 4.734 \times 10^{10} \text{ mm}^4$$

$$b_o = 4(600+160) = 3040 \text{ mm}^2$$

$$v_u = \frac{432 \times 10^3}{3040 \times 160} + \frac{0.4 \times 64.8 \times 10^6 \times \frac{(160+600)}{2}}{4.734 \times 10^{10}}$$

$$v_u = 1.09 \text{ MPa}$$

3) Column aspect ratio:**Model 5:**

$$\beta_c = 5.0, \rho_{FRP} = 0.65\%, V = 684 \text{ kN and } M = 102.6 \text{ kN.m}$$

$$v_u = \frac{V}{b_o d} + \frac{\gamma M_y}{J_c} X$$

$$J = \frac{1}{6}(460 \times (160)^3 + 160 \times (460)^3) + \frac{1}{2}(160 \times 1660 \times (460)^2) = 3.101 \times 10^{10} \text{ mm}^4$$

$$b_o = 2(1660 + 460) = 4240 \text{ mm}^2$$

$$\gamma = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{460}{1660}}} = 0.26$$

$$v_u = \frac{684 \times 10^3}{4240 \times 160} + \frac{0.26 \times 102.6 \times 10^6 \times \frac{(160+300)}{2}}{3.101 \times 10^{10}}$$

$$\underline{v_u = 1.20 \text{ MPa}}$$

APPENDIX D
FLEXURAL CAPACITY OF THE SPECIMENS

Section 1: Yield line pattern:

The method presented herein for the calculation of the flexural capacity of the specimens, was based on the actual behaviour, and boundary conditions for the specimens at failure. According to Hognestad (1953), if the corners are not held down, a yield-line pattern will appear, leaving a new slab part (1) rotating as a lever. This slab part (1) is referred to as a corner lever and, consequently, part (2) will behave in the same way.

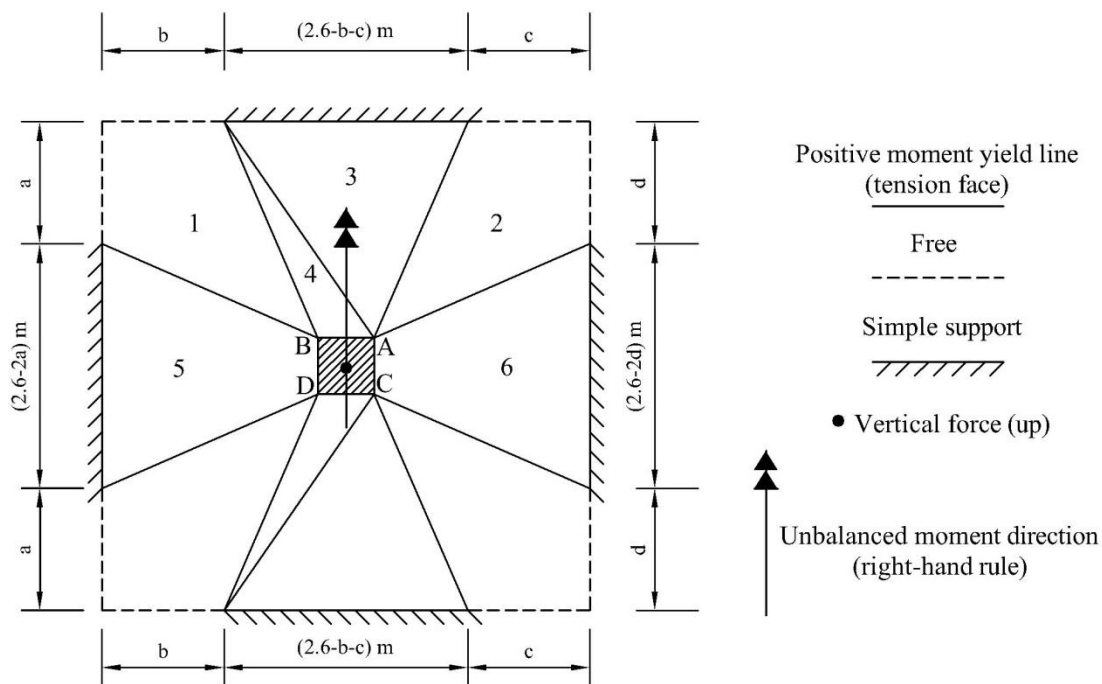


Figure 1: Yield line patterns

Deflection equal to Δ was assumed to occur at points B and D . In addition, the deflection at points A and C was assumed to be equal to $\Delta 1$. The relation between both deflections will be obtained as follow.

$$\frac{\Delta}{X} = \frac{\Delta 1}{(X-0.3)} \quad \Delta 1 = \frac{\Delta(X-0.3)}{X} \quad \Delta 1 = \Delta \left(1 - \frac{0.3}{X}\right)$$

$$\text{External work} = V \times (\Delta + \Delta 1)/2 + M \times \Delta/X = V \times \Delta \times \left(1 - \frac{0.15}{X}\right) + 0.15V \times \frac{\Delta}{X} = V \times \Delta$$

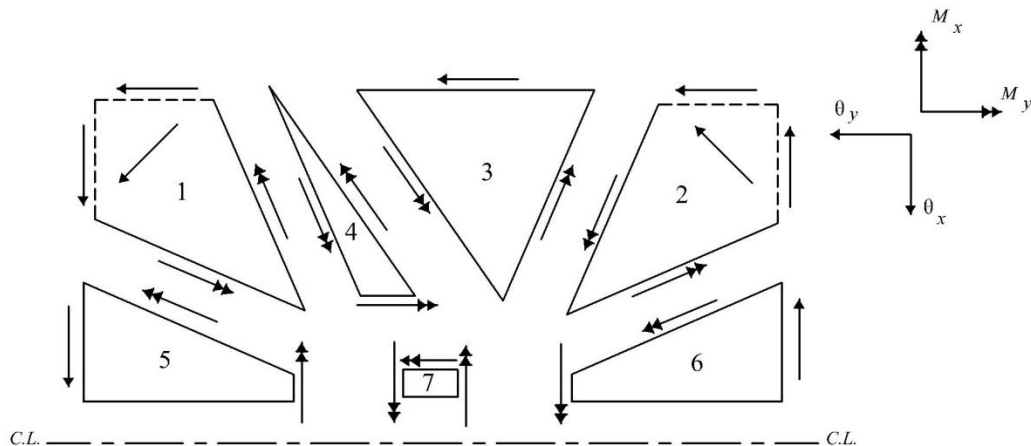


Figure 2: Moments and angles directions

$$\begin{aligned} \text{Internal work} = & 2 \times \left(\frac{\Delta 1}{1.15 + (1.15 - d) * (\frac{c}{d})} \times (d) \times (M_{PX}) + \frac{\Delta 1}{1.15 + (1.15 - c) * (\frac{d}{c})} \times (c) \times (M_{PY}) \right. \\ & + \frac{\Delta}{1.15 + (1.15 - a) * (\frac{b}{a})} \times (a) \times (M_{PX}) + \frac{\Delta}{1.15 + (1.15 - b) * (\frac{a}{b})} \times (b) \times (M_{PY}) + \frac{\Delta 1}{1.15} \times M_{PX} \times (1.3 - d) + \\ & \left. \frac{\Delta 1}{1.15} \times (2.6 - c - b) \times M_{PY} + \frac{\Delta}{1.15} \times M_{PX} \times (1.3 - a) \right) \end{aligned}$$

$$\begin{aligned} \text{Internal work} = & 2 \times \left(\frac{\Delta}{1.15 + (1.15 - d) * (\frac{c}{d})} \times \left(1 - \frac{0.3}{X}\right) \times (d) \times (M_{PX}) + \frac{\Delta}{1.15 + (1.15 - c) * (\frac{d}{c})} \times \right. \\ & \left(1 - \frac{0.3}{X}\right) \times (c) \times (M_{PY}) + \frac{\Delta}{1.15 + (1.15 - a) * (\frac{b}{a})} \times (a) \times (M_{PX}) + \frac{\Delta}{1.15 + (1.15 - b) * (\frac{a}{b})} \times (b) \times (M_{PY}) + \\ & \frac{\Delta}{1.15} \times \left(1 - \frac{0.3}{X}\right) \times M_{PX} \times (1.3 - d) + \frac{\Delta}{1.15} \times \left(1 - \frac{0.3}{X}\right) \times (2.6 - c - b) \times M_{PY} + \frac{\Delta}{1.15} \times \\ & \left. M_{PX} \times (1.3 - a) \right) \end{aligned}$$

External work = Internal work

$$\begin{aligned} \underline{VA} = & 2 \times \left(\frac{\Delta}{1.15 + (1.15 - d) * (\frac{c}{d})} \times \left(1 - \frac{0.3}{X}\right) \times (d) \times (M_{PX}) + \frac{\Delta}{1.15 + (1.15 - c) * (\frac{d}{c})} \times \left(1 - \frac{0.3}{X}\right) \times (c) \times \right. \\ & (M_{PY}) + \frac{\Delta}{1.15 + (1.15 - a) * (\frac{b}{a})} \times (a) \times (M_{PX}) + \frac{\Delta}{1.15 + (1.15 - b) * (\frac{a}{b})} \times (b) \times (M_{PY}) + \frac{\Delta}{1.15} \times \left(1 - \frac{0.3}{X}\right) \times \\ & \left. M_{PX} \times (1.3 - d) + \frac{\Delta}{1.15} \times \left(1 - \frac{0.3}{X}\right) \times (2.6 - c - b) \times M_{PY} + \frac{\Delta}{1.15} \times M_{PX} \times (1.3 - a) \right) \end{aligned}$$

Matlab computer program was used to solve the equation.

Section 2: plastic moment

1) The steel specimen (S-0.65-N-15-XX):

$$M_x = 80 \text{ kN. m/m}$$

$$M_y = 72 \text{ kN. m/m}$$

$$V_{flex} = 575 \text{ kN} > V_{actual} = 486 \text{ kN (the specimen failed in shear).}$$

$$V_{act.}/V_{flex.} = 0.84 < 1$$

The plastic moment for the GFRP specimens was calculated according to the method described by Gar et.al (2014) using equation number [1]. Also, it's worth to mention that, the authors did not mention anything about the strain either for the concrete or the FRP bars. In addition the flexure capacity, V_{flex} , in Gar et.al (2014) paper ranged from 100 to 858 kN.

$$M_p = 0.5 M_n + 0.15 M_{cr} \quad [\text{Eq. 1}]$$

Where:

M_p = the plastic moment per unit width

M_n = the moment capacity per unit width

M_{cr} = the cracking moment per unit width

2) FRP control specimen (G-0.65-N-15-XX):

$$M_x = 76.5 \text{ kN. m/m}$$

$$M_y = 65.9 \text{ kN. m/m}$$

$$V_{flex} = 536 \text{ kN} > V_{actual} = 363 \text{ kN (the specimen failed in punching shear).}$$

$$V_{act.}/V_{flex.} = 0.67 < 1$$