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**Behaviour and Strength of CFRP Reinforced
Flat Plate Interior Column Connections Subjected to
Shear and Unbalanced Moments**

by

ASHRAF ZAGHLOUL, B. Eng.

A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfilment of
the requirements for the degree of

Master of Applied Science

Department of Civil and Environmental Engineering

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in Civil Engineering
is a joint program with the University of Ottawa,
administrated by the Ottawa-Carleton Institute for Civil Engineering

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**Behaviour and Strength of CFRP Reinforced Flat Plate Interior
Column Connections Subjected to Shear and Unbalanced Moments**

Submitted by
Ashraf E. Ramzy Zaghloul

In partial fulfilment of the requirements of the degree of
Master of Applied Science

Professor Ghani Razaqpur, Thesis Supervisor

Professor Wayne J. Parker, Chair of Thesis Examining Board

Carleton University

January 2002

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©Allah! There is no god but He, the Living, the Self-subsisting, Eternal. No slumber can seize Him nor sleep. His are all things in the heavens and on earth. Who is there can intercede in His presence except as He permitteth? He knoweth what (appeareth to His creatures as) Before or After or Behind them. Nor shall they compass aught of His knowledge except as He willeth. His Throne doth extend over the heavens and the earth, and He feeleth no fatigue in guarding and preserving them for He is the Most High, the Supreme (in glory). ❀ ** Qur'an - AL-Baqarah - verse [255] **

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Abstract

Eight interior flat plate-column connections were tested to investigate the effectiveness of carbon fibre reinforced polymer (CFRP) reinforcement in resisting punching shear at slab column connections. Each specimen comprised a 1760 x 1760 mm slab and a 250 x 250 mm column stub extending above and below the slab. Seven of the test specimens were reinforced with the CFRP grid NEFMAC while the 8th specimen was reinforced with conventional steel reinforcement. The main test parameters comprised the moment/shear ratio, the reinforcement ratio, the type of reinforcement, the slab thickness and the presence of a new type of FRP shear reinforcement. The moment to the connection was applied through the eccentricity of the axial load.

The test results revealed that CFRP reinforced slab-column connections have practically the same basic behaviour as steel reinforced connections. Their failure mode appears to be highly ductile, but due to the lower stiffness of FRP reinforcement, they undergo larger deformations. However, by increasing the slab thickness from 100 mm to 125 mm, it was possible to achieve both higher strength and stiffness in the FRP reinforced specimen with the same reinforcement ratio as the companion steel reinforced specimen. Finally, the strength of the test specimens can be predicated by using the current CSA punching shear provisions with a modified expression for the punching shear resistance of concrete.

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List of Symbols

- A = parameter dependent on the column dimensions and the slab thickness in case of Cleland's analysis it also depends on the span of the slab.
- A = ud = area of critical section.
- A_c = is the area of the shear critical section .
- A_{FRP} = actual A_{FRP} area in the slab.
- A_{rebar} = area of reinforcing bar being considered.
- A_s = equivalent steel area.
- A_s = effective steel area passing the perimeter of the critical section ≤ 3 .
- b_o = the perimeter of the critical section (taken at $d/2$, $1.5d$, $2d$ away from the column face according to the ACI 318-95 and CSA A23.3-94, BS 8110 and CEB-FIP Model MC 90 and at column slab interface as considered by Gardiner.
- b_1 = length of critical section in the direction perpendicular to the moment axis.
- b_2 = length of the critical section in the direction parallel to the moment axis.
- c = dimension of column face perpendicular to rebar being considered.
- c = diameter of column, in case of a non circular column of an equivalent circular column with the same area is to be used.

- c = is the side length or diameter of loaded area.
- c = the column width.
- c = side length or diameter of loaded area.
- c_1 = the column side length in a direction normal to the vector of the applied moment.
- c_2 = the column side length in a direction parallel to the vector of the applied moment.
- d = average effective depth of slab.
- d = the effective flexural depth in mm.
- d' = cover of reinforcing mat measured from centre of mat to near surface of slab .
- d_s = effective depth of reinforcing mat measured from centre of mat to compression surface of slab.
- d_t = the slab effective depth as related to the steel bars normal to the moment vector.
- e = load eccentricity M/V
- e_1 = distance from the column centre to the perimeter of the critical section.
- E_{FRP} = the modulus of elasticity of FRP reinforcement in MPa.
- E_{GFRP} = the modulus of elasticity of the GFRP reinforcement.
- E_s = the modulus of elasticity of steel.
- E_{steel} = the modulus of elasticity of conventional steel MPa.

f'_c = compressive cylinder strength of concrete.

f'_c = the cylinder compressive strength of concrete.

f'_c = the concrete compressive strength in MPa.

f'_c = compressive cylinder strength of concrete.

f_{cm} = measured average cylinder compressive strength, MPa.

f_{ck} = characteristic compressive strength of the concrete, MPa.

f_{eu} = characteristic cube concrete strength ($f_{cu} \approx 1.18 f'_c$) and should not be taken less than 25 MPa and not greater than 40 MPa. Further more, the following limits need to be observed.

$$f_{cd2} = 0.6 (1 - f_{ck}/250) f_{cd}$$

$$f_{cd} = f_{ck} / \gamma_m$$

f_y = yield strength of flexural steel, MPa.

$F_{sd,ef}$ = the punching load, enhanced to allow for the effects of an eventual moment transferred to the column.

h = slab thickness.

J = property of assumed critical section analogous to the polar moment of inertia.

J_c = the polar moment of inertia of the critical section.

k = ratio of the positive to negative moment capacity.

k_l = constant depending on the statical system.

K = coefficient determining the portion of M_{sd} resisted via shear

stresses and is a function of c_1/c_2 , see Table 4.2.

K = numerical coefficient.

l = longer span of flat slab system.

= $l / 0.46$ for an isolated test specimen.

l_m = span between points of contraflexure.

= 0.46 the span between columns for full slab systems .

l_1 = length of the side GH of the critical section as shown in Figure 3.12.

l_2 = length of the side HK (or GJ) of the critical section as shown in Figure 3.12.

l_3 = length of the side KJ of the critical section as shown in Figure 3.12.

m = negative moment capacity per unit width of the slab.

M = unbalanced moment on the connection, to be used with v_c .

M_U = Ultimate unbalanced Moment, M , acting on the connection, to be used with v_c .

n = E_s / E_c

n_c = stress concentration factor = $1.4 (2d / r_o)^{1/2} \geq 1.25$

r_o = radius of the column or loaded area.

S_{eff} = effective tributary width of the reinforcing bar (max $6d'$).

u = length of control perimeter.

u = perimeter of loaded area.

u = the perimeter of loaded area.

- u_{BS} = rectangular critical shear perimeter at a distance $1.5d$ from loaded area.
- u_o = perimeter of square column of same cross sectional area.
- u_o = length of the periphery of the column.
- v_U = maximum shear stress that can be applied on the connection at ultimate load, to be calculated by the code or any rigorous method.
- v_C = calculated (predicted) shear resistance of the connection by the method of analysis.
- v_C = ultimate shear strength of concrete.
- v_U = factored normal shear stress at column-column connection.
- $v_{C,(Study)}$ = nominal shear strength as calculated by any method.
- $v_{C,ACI}$ = nominal V_C , ACI 318-95 = shear strength of the connection as determined by ACI 318-95 standard.
- $v_{C,CSA}$ = nominal V_C , CSA A23.3-94 = shear strength of the connection as determined by CSA A23.3-94 standard.
- $v_{C,BS811}$ = V_C , BS 8110-85 = nominal shear strength of the connection as determined by BS 8110-85 standard.
- $v_{C,CEB-}$ = nominal nominal shear strength of the connection as determined by CEB FIP-CEB Model MC90 standard.
- $v_{u,Sheriff}$ = nominal shear strength as been calculated by the least of Equations 2.34 and 2.35.

- $v_{C, \text{Sherif}}$ = shear strength of the connection as has determined by Sherif and Dilger's method for combined action of shear and unbalanced moment transfer.
- $v_{C, \text{Zaghlo}}$ = nominal shear strength of the connection calculated using Zaghloul and Ben-Sasi (2001), assuming v_u as that determined by Equation 5.2 proposed in the present work.
- $v_{C, \text{Presen}}$ = nominal shear strength calculated by the proposed method of the writer.
- V = shear strength of the connection.
- V_{emp} = the shear strength of the slab column-connection.
- $V_{C, \text{(Studied Method)}}$ = shear strength as calculated by any method.
- $V_{C, \text{ACI}}$ = $V_{C, \text{ACI 318-95}}$ = shear strength of the connection as determined by ACI 318-95 standard.
- $V_{C, \text{CSA}}$ = $V_{C, \text{CSA A23.3-94}}$ = shear strength of the connection as determined by CSA A23.3-94 standard.
- $V_{C, \text{BS 8110}}$ = $V_{C, \text{BS 8110-85}}$ = shear strength of the connection as determined by BS 8110-85 standard.
- $V_{C, \text{CEB MC90}}$ = shear strength of the connection as determined by CEB FIP-CEB Model MC90 standard.
- $V_{C, \text{Sherif \& Dilger}}$ = shear strength as been calculated by the least of Equations 2.34 and 2.35.

$V_{C,}$ = shear strength of the connection as has determined by Sherif and Dilger's method for combined action of shear and unbalanced moment transfer.

$V_{C,}$ = shear strength of the connection calculated using Zaghloul and Ben-Sasi (2001), assuming v_u as that determined by Equation 5.2 proposed in the present work.

$V_{C,}$ = shear strength calculated by the proposed method of the writer.

w_l = parameter analogous to the plastic section modulus of the critical section.

x = neutral axis depth = $0.8d (n \rho_o)^{1/2} (35 / f_c')^{1/2}$

y_1 = distance of side GH of the shear critical section from its centroid.

y_2 = distance of side KJ of the shear critical section from its centroid.

α_s = 40 for interior columns.

= 30 for edge columns.

= 20 for corner columns.

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

Where β_c is ratio of long side to short side of column to be taken ≥ 2

γ_m = partial safety factor for strength of material (1.25)

= $(100.4_s / u d) \geq 3$

= $(400 / d) \geq 1$

γ_v = portion of unbalanced moment resisted by non-uniform shear.

ρ = ratio of flexural reinforcement calculated over a width $c+6d$.

ρ = flexural reinforcement ratio = $(\rho_x \rho_y)^{1/2}$

in each direction the ratio should be calculated for a width equal to the side dimension of the column plus $3d$ to either side of it.

ρ_o = $\rho_o (f_y / 500)$

ρ_l = the ratio of the tension reinforcement placed normal to the moment vector.

ϕ = safety factor according ACI 318-95 = 0.85.

ϕ_c = concrete factor of safety (0.6).

λ = 1 for normal density concrete $(d/4c)^{0.5}$ = strength enhancement factor.

λ = is concrete density factor (1.00 for normal weight concrete and 0.85 for semi light weight).

σ_{steel} = equivalent steel stress.

σ_{FRP} = actual σ_{FRP} stress.

τ_{rd} = shear resistance of concrete, MPa.

ξ = size factor = $1 - (200/d)^{1/2}$

Chapter 1

Introduction

1.1 General

The corrosion of steel is a major cause of the deterioration of reinforced concrete structures. The corrosion is caused when the reinforcement bars in concrete are subjected to aggressive environmental conditions, especially in the environment of the underground, coastal and offshore locations. Parking structures and bridge decks are prime examples of structures subjected to severe environmental conditions that lead to deterioration of the main concrete components of such structures, engendered by the corrosion of their steel reinforcement.

Parking structures and bridge decks are usually composed of flat plate structures that are subjected to heavy wheel vehicle concentrated loads and/or concentration of shear forces and moments along their slab-column interfaces. In general, flat plate structures consist of slabs directly supported on columns. The flat plate structure is often preferred over conventional slab systems supported on beams due to the reduced amount of formwork, the availability of more headroom or clearance, and a pleasant appearance due to absence of beams.

A flat plate, however, has two inherent weaknesses; brittle failure and low ductility

due to the transfer of shear and moments to the supporting columns. The connection between the floor slab and a column in a flat plate structure is generally the most critical part as far as strength is concerned because it is the region where large bending and shear forces are concentrated. Although major lateral forces caused by wind and earthquake are often resisted by shear walls, designers are also looking to the column-slab connection to carry an increasing proportion of these forces. Considerable eccentricity may also be introduced into the column due to uneven distribution of live loads on either side of a column, random and unequal spacing of columns and volume changes caused by differences of temperature. Differential creep between two adjacent floors results in differential displacement of the top and bottom columns, which induce moments in the slab-column connections, even if the columns are assumed not to participate in the horizontal load resisting system. In the presence of such moments, the phenomenon of punching becomes unsymmetrical, and the punching strength of the slab decreases. This phenomenon has been studied by many researchers (e.g. Elgabry and Ghali, 1996, Hawkins and Corley, 1971, Moe, 1961, Regan et.al., 1979, Zaghoul, 1968, 1970, 1971, 1973) and is accounted for in the current codes, with varying levels of details (e.g. ACI-95, BS 8110-85, CEB, CEB-FIP Model Code 1990, CSA-A23.3-94, EUROCODE 2, 1992, Swiss Design Code, SIA 162, 1993).

A similar problem exists in slab bridges supported on columns due to heavy wheel loads. The viability of the above-mentioned two types of structures, and the flat plate floor structures in general, is to a large extent governed by the degree to which the ultimate strength of the connection between the floor slab and the columns and the punching shear resistance of deck slab bridges can be economically achieved.

The use of fibre-reinforced polymers (FRP) as a substitute for conventional steel rebars in concrete structures has been the subject of extensive research in North America, Europe, and Japan over the last decade. FRP are recommended due to their high tensile strength and light weight compared to steel, but more importantly due to their resistance to acids and alkalis, and hence their excellent corrosion resistance. Therefore, it would be natural to investigate the structural potential of FRP reinforcement in flat slabs and plates.

However, many of the existing studies on FRP reinforcement have been aimed at studying the flexural and shear behaviour of FRP reinforced concrete beams and one-way slabs. Only limited studies have been carried out to study the punching shear strength of flat plate structures. The limited available studies on punching shear strength of FRP reinforced concrete slabs have been concerned with concentric or pure punching shear.

The FRP reinforcement in the previous studies have comprised glass, carbon and hybrid glass/ carbon grids and glass reinforcing bars. The results of these studies have proven promising. Although it is important to understand the pure shear response of flat plates, practically this loading condition is not common because invariably slab-column connections are subject to shear and moment. Edge slab-column connections are often subjected to shear plus bending about an axis parallel to the free edge while corner column-slab connections are subjected to shear and biaxial moment. Consequently, to be able to safely design FRP reinforced concrete flat plate structures, it is essential that the behaviour of slab-column connections subjected to combined action of shear and moment be investigated.

1.2 Problem Definition

Parking structures and bridge decks in many parts of Canada and the USA are exposed to severe environmental conditions due to exposure to de-icing salts.. The salt causes corrosion of the steel reinforcement in concrete structures. The parking structures may start to suffer from corrosion only 10 years after of their construction (Walker 1986). In Canada the estimated cost of repairing existing parking structures is in the range of four to six billion dollars. The estimated repair and rehabilitation cost of the existing highway bridges and parking structures in the USA is over 50 billion dollars (Michaluk et al.1998)

A number of methods and precautions have been introduced to prevent corrosion of steel reinforcement. Among those methods are decreasing the permeability of concrete by using dense concrete, adding additives to concrete, coating concrete with impermeable layers, coating steel rebar with epoxy and cathodic protection of reinforcement, and providing thicker cover to the reinforcement. However, most of these protection measures increase the cost of the structure and do not prevent corrosion completely; rather they delay the problem, and they may even exacerbate the problem if not done properly.

An ideal solution would be to replace the steel reinforcement by an alternative reinforcement which is immune from corrosion. Of course, the new or alternative reinforcement must have adequate strength and stiffness and must be reasonably economical. It must also be able to withstand other environmental and short and long term loading effects. Fibre reinforced polymers may be a potential substitute for steel reinforcement in concrete.

Among the FRP, carbon fibre reinforced polymers (CFRP) are known to be practically immune to alkali and acids and they have high strength and relatively high modulus. Consequently, they may be able to replace steel reinforcement in concrete structures. While durability and long-term performance of CFRP are reasonably well established, their ability to resist the combined shear and bending at the slab-column connections needs investigation.

The writer is not aware of the construction of any FRP reinforced concrete flat plate/slab structure anywhere in the world. This is in part due to lack of adequate knowledge with respect to the punching shear response of such structures. Due to the complex nature of this problem, it is difficult to develop design methods for punching shear based on theoretical considerations only. The latter becomes evident when one examines the current punching shear design method for steel reinforced concrete members in the Canadian and American concrete design codes. These provisions are mostly empirical, but are deemed to be practical and safe. A similar approach needs to be adopted for FRP reinforced concrete flat plate structures. This would require a comprehensive testing program to fully understand the punching shear behaviour of FRP reinforced flat plate structures and to derive the necessary code provisions for design. Without a code of practice, the likelihood of FRP being widely used in practice is practically nil.

1.3 Objectives and Scope

The objective of this thesis is to study experimentally the strength and overall behaviour of the interior slab-column connections in a flat plate structure reinforced with carbon fibre reinforced polymer (CFRP) grids. The connections shall be subjected to

shear and unbalanced moment about one axis.

Based on data from tests on steel reinforced slab-column connections, the following parameters are believed to have the greatest influence on the strength and behaviour of an interior connection in a flat plate structure.

1. Compressive strength of concrete.
2. Type of reinforcement,
3. Reinforcement ratio of the slab.
4. Moment / shear, M/V , ratio at the connection.
5. Column side length to slab effective depth ratio, c/d .
6. Column perimeter to effective depth ratio, b_o/d .
7. Column sides aspect ratio, c_1/c_2 .
8. Presence of shear reinforcement.

The focus OF this study is on items 2 to 8, with major emphasis on items 3 and 4.

Although item 1, the compressive strength of concrete, is not included directly in the present study because only one type of concrete mix design was used in the present study, it is included due to the variation in the concrete strength caused either by the age of concrete at the time of testing the various specimens or due to expected random variation.

The scope of this study is limited to only one type of FRP reinforcement, namely CFRP grids, known commercially as NEFMAC, and to medium strength concrete. Furthermore, only interior slab-column connections under shear and bending about a single axis are investigated.

Chapter 2

Literature Review

2.1 Introduction

This chapter is concerned with the background to flat slabs/plates design, especially to the design of their connections with their supporting columns. The problem of shear strength of the slab-column connections, reinforced with conventional steel bars, was known as early as the beginning of the twentieth century (Talbot (1913), Graf (1933 and 1938), and Richart and Kluge (1939). Extensive studies were later conducted by Hognestad (1953), Elestener and Hognestad (1956), and Kinnunen and Nylander (1960). The latter was due to the increased development and use of flat plate structures and the practical demand for dealing with heavy punching shear forces produced by heavy vehicle wheel loads using deck slab bridges. Slabs most often fail due to either concentric or eccentric shear stresses, provided they are designed for adequate flexural strength. The failure occurs through formation of a mechanism that results in the development of a failure surface around the column that causes punching of the column through the slab.

This type of failure, which is normally referred to as punching failure, was found to be the source of the complete destruction of several flat slab buildings, (Hawkins and Mitchell 1979). A thorough understanding of the problem that would ultimately lead to adequate design of the connections is therefore of prime interest.

A brief summary of the work on the problem of shear strength of flat plate slabs at their connections with the supporting columns prior to 1961 has been reported by Moe (1961). Later on, Van Dusen (1985) reported a literature review on the available research concerned with the punching shear due unbalanced moment and shear transfer at slab-column connections. In contrast, only limited experimental work is available regarding concentric shear in flat slabs or the slab-column connections when reinforced by FRP reinforcement, Ahmad et al., (1994), Matthys and Taerwe. (References 69 through 73), Elghandour et al. (References 28 through 35) and Opsina et al. (2000) have conducted some studies on the punching strength of FRP reinforced slabs. However, the behaviour of FRP slab-column connections is still a subject that requires extensive investigation.

This chapter summarizes different recognized methods that could be used to analyse the punching shear strength of interior slab-column connections in conventionally reinforced flat plate structures. Also different code provisions for calculating the punching capacity of slabs are reviewed. Reference is also made to some recent recommendations with respect to the concentric punching strength of FRP reinforced slabs.

2.2 Previous Work on Punching Shear in Flat Slabs Reinforced with Traditional Steel Reinforcement

2.2.1 Concentric Punching

2.2.1.1 Failure Mode

Shear failure of concrete flat slabs in the vicinity of concentrated loads may be due to beam action or two-way action. In case of the beam action, the slab behaves as a wide beam and the failure surface extends along the entire width of the slab. This thesis does not deal with this type of failure. In case of the two-way action, the slab fails in a local area around the concentrated load.

In the vicinity of the load or column there are two types of strains, tangential strains and radial strains, see Figure 2.1. Test results by Kinnunen and Nylander (1960) and Anis (1970) show that at the initially the tangential strains are higher than the radial strains. This leads to the formation of internal inclined cracks normally at $1/2$ to $2/3$ of the failure load, (Regan and Braestrup 1985). These inclined cracks run through the slab at 25 to 35 degrees, but they are not visible from outside the slab. Evidence of this has been obtained from measurements of changes of slab effective depth during loading and from recordings of internal strains (Regan 1983). Recently, Ozawa et al. (2000) gave special attention to the development of the internal cracking that occurs underneath the loading plate. They found that in all the test specimens of their investigation, nearly vertical flexural cracks propagated upward from directly beneath the loading plate, and

diagonal cracks eventually developed from the loading plate to the level of reinforcement, exhibiting punching shear failure. The sawed sections of the specimens revealed that flexural shear cracks occur directly beneath the loading plate and are connected with one another, forming a dome. In addition, diagonal cracks that shape the punching cone propagate half way to the loading plate. Accordingly, up to 80% of the ultimate load, only vertical cracks, with or without flexural shear cracks, occur directly beneath the loading plate. The ultimate diagonal cracks that bring about the punching shear failure are considered to abruptly propagate immediately before the ultimate load.

The condition after inclined cracking appears to be entirely stable and a slab can be unloaded and reloaded without its ultimate strength being affected. At higher loads some tangential cracks of circular shape form around the column (see Figure 2.1). One of these circles, most probably the outermost, will create the surface of the final punching cone and then punching failure occurs suddenly. The cover of the steel on the tension side of the slab is then pushed outwards by the flexural reinforcement.

2.2.1.2 Methods of Analysis

Many researchers have studied the problem of punching shear semi-empirically and have proposed several methods of analysis as described in this section.

Figure 2.1: Typical symmetrical punching failure

2.2.1.3 Mechanical Models

Although the inclined shear cracks, which form between $1/2$ and $2/3$ of the failure load, can completely surround the column, the slab is nevertheless stable. This behaviour suggests that the failure mechanism is not normally a pure shear failure governed by the diagonal tensile strength of the concrete. Thus, some researchers have established models

of the punching problem based on the properties of the un-cracked compression zone of the slab around the column.

Kinnunen and Nylander (1960) divided the slab outside the inclined shear crack region into sectors, i.e. into elements between the radial cracks in Figure 2.2. Each element is assumed to act as a rigid body supported by an imaginary conical shell on the compression side of the slab immediately above the column. Failure is assumed to occur when the stress in the conical shell and the compression strain in the tangential direction reach critical values. The depth of the compression zone is determined by iteration, so that the two failure conditions coincide. Using this model, the critical values of the concrete properties were calibrated by Kinnunen and Nylander (1960) against test results, but the values showed poor agreement with the expected values for concrete ultimate stress and strain.

Broms, (1990) treated the problem in a similar manner, but used generally recognized values for concrete properties, different compression zone depths in the radial and the tangential directions and different positions for the bottom of the stable shear crack. Furthermore he expanded his model to include unsymmetrical punching, and the results showed improvement over Kinnunen and Nylander's method.

Shehata and Regan (1989) divided the slab into rigid segments as shown in Figure 2.3.

Figure 2.2 Mechanical punching model by Kinnunen and Nylander (1960)

Each segment is bounded by two radial crack lines; a part of the initial circumferential crack and the slab boundary, or the lines of contraflexure around the column. The radial segments rotate around the centre of rotation at the face of the column and on the level of the neutral axis. They assumed reinforcement crossing the tangential crack at the column to reach yield, and that near failure a rigid wedge element, bounded by the internal inclined crack and the initial circumferential crack, is detached from each radial segment and rotates independently around the centre of rotation. Failure is defined by three criteria:

1. Maximum inclination of the compressive force.
2. Maximum radial strain in the compressed column face.

3. Maximum tangential strain of the compressed face at a certain distance from column face.

Shehata (1990) simplified this model and proposed the following equation for calculating the punching load V for a symmetrical slab:

$$V = 2 \pi r_o x n_c f'_c \tan 10 (500/d)^{1/3} \quad (2.1)$$

where:

r_o = radius of the column or loaded area

x = neutral axis depth = $0.8d (n \rho_e)^{1/2} (35/f'_c)^{1/2}$

n = modular ratio = E_s/E_c

f'_c = concrete cylinder compressive strength

$\rho_e = \rho (f_y/500)$

ρ = reinforcement ratio

n_c = stress concentration factor = $1.4 (2d/r_o)^{1/2} \geq 1.25$

d = effective slab depth

E_s = elastic modulus of steel

E_c = elastic modulus of concrete

Figure 2.3: Punching model by Shehata and Regan (1989)

2.2.1.4 Flexural Punching Approach

Many slabs reported in the literature as having failed due to punching failures actually failed at ultimate loads, which do not differ significantly from their flexural capacities. This has lead some researchers to the conclusion that for normal reinforced concrete slabs, punching is a secondary phenomenon and the primary cause of failure is yielding of the flexural reinforcement. In a state-of-the-art report by the joint ACI-ASCE Task Committee 426 (1974) expressions for the shear strength depending mainly on flexural effects are listed. This treatment of punching seems valid only for slabs with a low flexural reinforcement ratio, in which the steel reaches yield. However, punching failure occurs in slabs with practical or high reinforcement ratios at loads less than their flexural strengths.

Rankin and Long (1987-a) proposed a two-phase approach for the punching shear strength of interior slab-column connections subjected to concentric loads. Two modes of

failure were considered, a flexural mode and a punching mode. The flexural-punching strength is found by interpolating between the yield line based capacity of a slab, and its lower bound elastic strength when localized compression failure occurs around the column. The punching strength is expressed in terms of an empirically derived equation for nominal ultimate shear stress at a critical section. They concluded that in slabs with a reinforcement ratio less than 1%, the flexural-punching load would govern. However, for the case of reinforcement with high yield strength, one should be careful with the value 1%. Higher yield strength would increase the flexural capacity of the connection but would not increase the punching capacity to the same extent.

2.2.1.5 Plasticity Approach

An upper-bound plastic solution for the punching shear strength of slabs was developed by Braestrup et al. (1976), Nielsen et al. (1978) and Braestrup (1979). In this approach the deformations are assumed to be concentrated in an axisymmetric surface. The concrete body inside the failure surface in Figure 2.4 is punched out perpendicularly to the slab, the other slab parts remaining rigid. The failure mechanism differs from the failure cone in that the failure is not necessarily conical and that the kinematics discontinuity is not a splitting crack carrying tensile stresses only.

Figure 2.4: Failure mechanism in the plasticity approach for punching failure

Due to symmetry, the deformations are normal to the slab surface but not to the failure surface, and the component of the relative deformation tangential to the crack implies a sliding failure of the concrete. Also, since the deformations are perpendicular to the flexural reinforcement bars, and dowel action is neglected, the flexural reinforcement is not expected to contribute to the punching strength. The strength is influenced only by the geometry of the loaded area and the concrete compressive strength. This contradicts many test results reported in the literature, which show that the shear strength depends on the amount of tension reinforcement present. According to the plasticity approach, these failures, although having the appearance of punching, are governed by flexural strength. Based upon the failure mechanism of Figure 2.4, the punching load is derived by equating the rate of external work done by the applied load with the rate of internal work dissipated in the failure surface.

Bortolotti (1990) proposed a modified approach which accounts for flexure and shear reinforcement. However, comparisons with test results were not conclusive (Elgabry 1991). Regan and Braestrup (1985) evaluated lower-bound plastic solutions using available test results published by several researchers and showed that the approach is far from being satisfactory.

2.2.1.6 Truss Model

Alexander and Simmonds (1987) proposed a three-dimensional space truss to model the flow of forces between the slab and the column. The truss is composed of concrete compression struts and steel tension ties. The reinforcing steel and concrete compression fields are broken down into individual struts. As indicated in Figure 2.5, two types of compression struts are suggested:

1. Those parallel to the plane of the slab (anchoring struts).
2. Those at angle α to the plane of the slab (shear struts).

Through the anchoring struts, bars at some distance from the column can exert a moment on the connection by flexure. The shear resisting steel is assumed equal to the sum of all steel passing through the column plus a fraction of the steel within distance d (effective slab depth) from the column face. This fraction decreases linearly from unity for a bar at the column face to zero for a bar located at d from the column face. The angle α was calibrated using interior column test results available in the literature. The value of $\tan \alpha$ was calculated as the ratio of the failure load to the total area of top mat shear steel times

its yield strength. The value of $\tan \alpha$ was plotted against a non-dimensional factor K and the following relationship was derived,

Bars going through the column

Figure 2.5: Truss model by Alexander and Simmonds (1987)

$$\tan \alpha = 1 - e^{-2.25K} \quad (2.2)$$

with

$$K = \frac{S_{eff} d' \sqrt{f'_c}}{A_{rebar} f_y (c / d_s)^{0.25}} \quad (2.3)$$

where

S_{eff} = effective tributary width of the reinforcing bar (max $6d'$)

d' = cover of reinforcing mat measured from centre of mat to nearest surface of slab

d_s = effective depth of reinforcing mat measured from centre of mat to compression surface of slab

c = dimension of column face perpendicular to rebar being considered

A_{rebar} = area of reinforcing bar being considered

f'_c = compressive cylinder strength of concrete

f_y = yield strength of steel

e = base of natural logarithm

For a concentrically loaded column the failure load V_u can be calculated as:

$$V_u = \sum A_{rebar} f_y \tan \alpha \quad (2.4)$$

where A_{rebar} is for the flexural steel that is close enough to the column to participate in resisting shear.

The failure of each bar-strut assembly is so defined by its geometry and the yield strength of the steel. It is assumed that the concrete compression struts do not fail and their capacity need not be considered.

2.2.1.7 Plastic Model by Alexander

The design model proposed by Alexander (1994) is based on the equilibrium of two plastic mechanisms within the connection region. The first of these mechanisms is concerned with the transfer of vertical shear between the slab and the column. The second assesses the anchorage of flexural steel in the vicinity of edge and corner columns, and will be discussed later.

According to the bond model presented by Alexander and Simmonds (1992), an interior column-slab connection may be modelled with four slab strips, extending from the column, parallel to the slab reinforcement, to a point of zero shear as shown in Figure 2.6. It is assumed that:

- No load can reach the column without passing through one of the radial strips. Each radial strip is loaded with an internal shear on its side faces by the adjacent quadrants of the two-way slab.
- At the ultimate load the total distributed load on the strip is $2w$, where w is a limiting internal shear that can be carried by the slab.
- The flexural strength M_s of the radial strip is the sum of the negative and positive flexural capacities, M_{neg} and M_{pos} at the ends of the strip (see Figure 2.6). The loaded

length of the strip is l (shear arm) and the total load carried by one strip is P_s .

Equilibrium of the radial strip requires that:

Figure 2.6: Plastic model by Alexander (1994)

$$P_s = 2lW = 2\sqrt{M_s\omega} \quad (2.5)$$

From Equation 2.5, the factors that limit the shear capacity of a strip are the intensity of the shear that can be carried on the boundary between the strip and its adjacent quadrants of the slab and the flexural capacity of the strip itself. Alexander adopted the design value for one way shear strength of the Canadian CSA A23.3-94 Code to calculate ω as:

$$\omega = 0.2\phi_c d \sqrt{f'_c} \quad (2.6)$$

where ϕ_c is the resistance factor for concrete (0.6) and d is the effective depth of the slab.

2.2.1.8 Nölting Approach

Nölting (1984) developed an expression to calculate the punching load of concentrically loaded slabs. He used the following main assumptions in developing his theory:

- Until reaching the yield load, the plate behaviour can be described using elastic theory.
- The relationship between the strains in the inclined compression struts and their horizontal component is based on the slenderness of the slab.
- At loads higher than the yield load, the strains in the inclined compressive strut increase in a quadratic manner.
- Punching occurs when the strains in the inclined compressive struts reach a constant critical value.

Nölting used data from 431 tests in the literature to calibrate his model. The punching failure load V_u can then be approximated as follows:

$$V_u = 4.75 \sqrt{\rho f_c} d^2 \alpha_o \quad (2.7a)$$

with

$$\alpha_o = (0.65 + 9.4 c/l) - (2.2 + 70 c/l) d/l \quad (2.7b)$$

in which

ρ = flexural reinforcement ratio

d = effective depth of slab

c = diameter of column, in case of a non circular column an equivalent circular column with the same area is to be used

l = longer span of flat slab system

= $l / 0.46$ for an isolated test specimen

of interest in this model is the geometry function α_o . For a slab with a 250 mm square columns and an effective depth of 114 mm, α_o as a function of the span l is given in Figure 2.7. For spans up to two metres the strength of the slab increases (α_o is increasing), while for spans larger than 2 m, α_o decreases constantly, indicating a decrease in strength.

Figure 2.7: Effect of span length on the punching shear strength (Nölting (1984))

2.2.1.9 Control Surface Approach

For a reinforced concrete beam, shear stress is defined on a section of the beam. The same approach may be used in slabs, defining shear stress as the shear force divided by the area of a control surface around the loaded area. This control surface is normal to the plane of the slab, and its perimeter is taken to conform either with the loaded area or at a certain specified distance from the loaded area. The height of the control surface may be taken as the total slab thickness h , the effective depth d , or the internal moment lever arm z . This method was introduced by Talbot (1913). He tested centrally loaded square footings, and found that if he takes the control surface at a distance d from the loaded column, the resulting shear stress at failure was approximately equal to the ultimate shear stress of simple beams without shear reinforcement. The safety against punching is then assessed by comparing the nominal shear stress with a strength parameter of the concrete, usually some measure of the tensile concrete strength. This strength parameter (shear resistance) is calibrated to suit the assumed critical section. The approach has very little to do with the actual punching failure mechanism. The control surface approach should therefore be regarded as a simple empirical method, which when adequately applied leads to realistic predictions of the failure load. Consequently, it has been adopted by a large number of building codes. As an example the approach by Criswell and Hawkins (1974), the CEB Bulletin d'information 168 (1985) [state of Art Report by P.E. Regan and M.W. Braestrup], Gardner (1995) and (1996), the ACI 318-1995 Code, the CSA A23.3-1994 Code, the British, BS8110-1985 and the European CEB-FEP Model Code 1990 are based on this concept as discussed in the following sections.

Shear Capacity of an Intermediate Slab-Column Connection by Different Researchers

i- Criswell and Hawkins (1974)

Criswell and Hawkins (1974) proposed a design equation to calculate the punching shear strength of slab-column connections. Their assumed critical section is the perimeter of the loaded area. Using results from several test programs, they obtained the following expression:

$$V_{emp} = 4 \sqrt{f_c'} \left(1 + 2 \frac{d}{c}\right) u d \quad (\text{S.I. units}) \quad (2.8)$$

where

f_c' = concrete cylinder compressive strength (MPa).

V_{emp} = the shear strength of the slab column-connection.

c = the side length or diameter of loaded area.

u = the perimeter of loaded area.

ii- Regan and Bastrup (1985)

Regan and Bastrup, (1985) proposed in the CEB Bulletin No.168 the following expression for the punching shear capacity of slab column connections.

$$V_{emp} = 1.36 \frac{(100 \rho f_c')^{1/3}}{d^{1/4}} u_{BS} d \quad (\text{S.I. units}) \quad (2.9)$$

where, u_{BS} is a rectangular critical shear perimeter at a distance $1.5d$ from the loaded area and ρ is the slab reinforcement ratio.

iii- Gardner (1995)

Gardner (1995) proposed a design equation to calculate the punching shear capacity of slab-column connections. His critical section is the perimeter of the column and the coefficients for the equation are calibrated using test results from the literature. For non-prestressed slab-column connections his equation can be written as:

$$V_u = 0.55\lambda u_o \left[1 + \left(\frac{250}{h}\right)^{0.5}\right] \left[d^3 \rho f_y f_c'\right]^{1/3} \left(\frac{h}{u_o}\right)^{0.5} \quad (2.10)$$

where:

d = effective slab depth

h = slab thickness

u_o = perimeter of square column of same cross sectional area

λ = 1 for normal density concrete

ρ = ratio of flexural reinforcement calculated over a width $c+6d$

c = the column width

iv- Gardner and Shao (1996)

Gardiner and Shao (1996) proposed a design expression for calculating the punching shear strength of interior slab column connections of reinforced concrete flat plate structures by extending the strength enhancement logic of Shehata and Regan (1989) and Shehata (1990). The control perimeter was taken at the periphery of the column or loaded area. The depth of the compression zone was assumed to be a function of the tension tie strength ρf_y . To account for the effect of slab thickness, the CEB size effect expression was used. The following equation was derived for non-prestressed slab column connections.

$$V_r = 0.79u_o d \left(1 + \left(\frac{200}{d}\right)^{0.5}\right) (\rho f_y f_{cm})^{(1/3)} \left(\frac{d}{4c}\right)^{0.5} \quad (2.10b)$$

where,

$(d/4c)^{0.5}$ = strength enhancement factor

f_y = yield strength of flexural steel, MPa

f_{cm} = measured average cylinder compressive strength, MPa

v- Sherif and Dilger (1996)

Sherif and Dilger, (1996) proposed the following two empirical equations to calculate the shear strength of interior slab-column connection. They were obtained from studying the effect of the different parameters that affect the ultimate shear strength of the slab-column connections using published test data. They recommended that the

ultimate shear strength be determined from either of the following two equations, whichever results in lower shear strength

$$v_u = 0.7 \left(100 \rho f'_c \right)^{1/3} \quad (2.11)$$

or

$$v_u = 0.7 \left(\frac{6.7d}{b_o} + 0.4 \right) \left(100 \rho f'_c \right)^{1/3} \quad (2.12)$$

2.2.1.10 Shear Capacity of Interior Slab-Column Connections Recommended by Different Codes

According to the Canadian standard CSA-A23.3-94, (1994) and the American Concrete Institute code, ACI-95 (1995), the critical section lies at distance $d/2$ from each face of the column. It is stated that by choosing this section, the shear strength is independent of the ratio c/d (column dimension to effective depth ratio). The British standard BS8110 and the CEB-FIP Model Code 1990 consider the critical section to lie at $1.5d$ and $2d$ respectively. All of these codes express the connection capacity in terms of limiting shear stress.

i- The American Concrete Institute Code (ACI 318-95)

Using the above-mentioned critical section, for design the ACI code states that

$$v_u < \phi v_c$$

where

ϕ = a safety factor

v_c = ultimate punching shear strength of concrete

v_u = factored nominal shear stress at slab-column connection given by

$$v_u = \frac{V_u}{b_o d} \quad (2.13)$$

where b_o = the perimeter of the critical section.

d = the effective depth of the slab.

V_u = factored shear force acting on the connection

The shear resistance of concrete v_c is the smallest of the following three values given by Equations 2.14, 2.15 and 2.16.

$$v_c = \left(1 + \frac{2}{\beta_c}\right) \frac{\sqrt{f'_c}}{6} \quad (2.14)$$

where β_c is the ratio of the long side to the short side of the column, $\beta_c \geq 2$

$$v_c = \left(\frac{\alpha_s d}{b_o} + 2\right) \frac{\sqrt{f'_c}}{12} \quad (2.15)$$

where, $\alpha_s = 40$ for interior columns.

= 30 for edge columns.

= 20 for corner columns

Tests shows that v_c decreases as the ratio b_o/d increases (Vanderbilt 1972)

and

$$v_c = 0.33 \sqrt{f'_c} \quad (2.16)$$

ii- The Canadian Code (CSA A23.3-94, 1994)

Similar to the ACI Code, the CSA Code recommends that the critical section be taken at $d/2$ from the loaded area. For design the following condition must be satisfied:

$$v_f \leq v_r$$

where v_f = shear stress due to factored loads (calculated as the v_u by ACI).

v_r = factored shear stress resistance.

The shear resistance v_r of the concrete is taken as the smallest value given by Equations 2.17, 2.18 and 2.19.

$$a) \quad v_c = \left(1 + \frac{2}{\beta_c}\right) 0.2 \lambda \phi_c \sqrt{f'_c} \quad (2.17)$$

where β_c = ratio of long side to short side of column to be taken ≥ 2

λ = concrete density factor (1.00 for normal weight concrete

0.85 = for semi light weight)

ϕ_c = concrete safety factor (0.6)

$$b) \quad v_c = \left(\frac{\alpha_s d}{b_o} + 0.2\right) \lambda \phi_c \sqrt{f'_c} \quad (2.18)$$

where $\alpha_s = 4$ for interior columns.

$\alpha_s = 3$ for edge columns.

$\alpha_s = 2$ for corner columns

$$c) \quad v_c = 0.4 \lambda \phi_c \sqrt{f'_c} \quad (2.19)$$

iii- The British Standard BS 8110-85 (1985)

The British Standard states that $v \leq v_c$. The critical section adopted by the Standard lies $1.5d$ from the column face, The design shear stress v due to a shear force V is then calculated as:

$$v = \frac{V}{ud} \quad (2.20)$$

where u is the length of the perimeter of the critical section. The shear capacity of the concrete v_c , which is taken the same as in one-way shear strength, i.e.

$$v_c = \frac{0.79}{\gamma_m} \left[\frac{100A_s}{ud} \right]^{1/3} \left[\frac{400}{d} \right]^{1/4} \left[\frac{f_{cu}}{25} \right]^{1/3} \quad (2.21)$$

γ_m = partial safety factor for strength of material (1.25)

A_s = effective steel area crossing the perimeter of the critical section ≤ 3

$$(100A_s/ud) \geq 3$$

$$(400/d) \geq 1$$

f_{cu} = characteristic cube concrete strength ($f_{cu} \approx 1.18 f_c$) and should not be taken less than 25 MPa and not greater than 40 MPa. Furthermore, the following limits need to be observed

At the column face the maximum shear stress v_{max} should not exceed $0.8 (f_{cu})^{1/2}$

nor 5.0 MPa, whichever is less. This includes an allowance for $\gamma_m = 1.25$.

The value of v_{max} is given by

$$v_{max} = \frac{V}{u_o d} \quad (2.22)$$

where u_o is the perimeter of the column or column head. Where it is desired to check the perimeters closer to the loaded area than $1.5d$, the Standard allows γ_m to be increased by a factor $1.5d/a_v$, where a_v is the distance from the edge of the loaded area to the perimeter considered. If the perimeter considered is taken at $0.5d$ (critical section of ACI & CSA codes), then the allowable shear resistance is 3 times the value used for one-way shear at $1.5d$. The ACI and CSA codes use 2 times the value used for one-way shear. Of interest in the British Standard is the position of the critical section ($1.5d$ from column face). Tests by Gardner (1990) suggest that for shear failure to occur, the distance between the face of the column and the support has to be greater than $3d$. He argues that if the inclined shear cracks start at the column face and reach the top face of the slab at a distance $3d$, then assuming the critical section at a distance $1.5d$ from the column face seems reasonable.

iv- The European CEB-FIP Model Code 90 (1990)

The control perimeter of the critical section is taken at a distance $2d$ from the periphery of loaded area, and is constructed to minimize its length. The applied shear stress at the control perimeter due to a factored concentrated force F_{sd} is calculated as:

$$\tau_{sd} = \frac{F_{sd}}{u_1 d} \quad (2.23)$$

The shear stress resistance of the concrete is given by:

$$\tau_{rd} = 0.12\xi(100\rho f_{ck})^{1/3} \quad (2.24)$$

where $\tau_{sd} \leq \tau_{rd}$. In the above equations

u_l = control perimeter length

τ_{rd} = shear resistance of concrete, MPa

ξ = size factor = $1 + (200/d)^{1/2}$

ρ = flexural reinforcement ratio = $(\rho_x \rho_y)^{1/2}$

in each direction the ratio should be calculated for a width equal to the side dimension of the column plus $3d$ to either side of it

f_{ck} = characteristic compressive strength of concrete, MPa

The maximum loading for which any connection (including connections with shear reinforcement) may be designed is defined by:

$$\frac{F_{sd}}{u_o d} \leq 0.5 f_{cd2} \quad (2.25)$$

where u_o = length of the periphery of the column

$$f_{cd2} = 0.6 (1 - f_{ck} / 250) f_{cd}$$

$$f_{cd} = f_{ck} / \gamma_m$$

γ_m = reduction factor (1.5 for concrete)

2.2.2 Punching Shear Strength of Interior Slab-Column Connections with Unbalance Moment Transfer

Moment transfer at slab-column connections adds to the complexity of the punching shear problem. Although interior connection design is often controlled by direct shear transfer, this condition should not be assumed without considering the sources of moment transfer. Moment transfer can arise when pattern loads are applied, when span lengths of adjacent bays differ, or when the slab-column frame has to resist lateral loads. The design should recognize that in a building loaded by lateral loads, the slabs participate in resisting those loads even if it is not designed as part of the lateral load resisting system. In the following, the various available design methods for conventionally reinforced slab-column connections are presented.

Theoretical Considerations

Before presenting the available methods, a general description of the problem of punching shear due to combined shear and moment transfer is given. Where an eccentric load is transferred from a slab to an interior column, a part of the unbalanced moment is likely to be resisted by a non-uniform distribution of the vertical shear around the column; the remainder is transmitted by flexure and torsion. Distribution of stresses due to the three load effects can be estimated by a variety of methods.

Mast (1970) investigated the effects of a concentrated moment applied to an uncracked elastic plate at an interior column. He neglected the local influence of the column in question. The distributions of the load effects around the shear perimeter are

calculated as independent of the geometry of the column. More refined elastic analysis were performed by Yamazaki (1975), Elgabry(1991) and Cleland et al. (1979) using the finite element method to derive relationships between maximum shear stresses, the location of the critical shear perimeter considered, the load eccentricity from the centroid of the critical section and the portion of the moment resisted by shear.

Clearly, plate analyses can give theoretical shear distributions and their results are easily obtained provided they are restricted to the elastic range. However, there are considerable problems involved in their use for the prediction of punching capacity. Allowing for redistribution of stresses following flexural cracking and yielding is difficult. In spite of these difficulties, plate analysis can be used to assess the distribution of vertical shear due to an unbalanced moment between a slab and a column. If the perimeter at which moment effects are considered is the same as that used for pure shear loading, the total nominal shear stress can be expressed according to Regan (1985) as

$$v = \frac{V}{ud} \left[1 + \frac{Ke}{a} \right] \quad (2.26)$$

where

u = length of control perimeter

K = numerical coefficient

e = load eccentricity M/V

M = unbalanced moment

a = parameter dependent on the column dimensions and the slab thickness (in case of Cleland's et al (1979) analysis, it also depends on the span of the slab).

Cleland's et al. analysis, it also depends on the span of the slab).

Failure may be assumed to occur when v equals v_u , with v_u given the same value as for concentric loading. Equation 2.26 can then be expressed in the following form:

$$V = v_u u d - \frac{K}{a} M \quad (2.27)$$

Equation 2.27 predicts a linear interaction diagram between V and M .

2.2.3 The Eccentric Shear Stress Model

Another approach is to assume a value for the portion of the moment resisted by non-uniform shear and then to determine the resulting shear stresses, assuming them to be linearly distributed around the control perimeter, Figure 2.8 (a), thus:

$$v = \frac{V}{A} + \frac{\gamma_v M e_1}{J} \quad (2.28)$$

where:

$A = u d =$ area of critical section

$\gamma_v =$ portion of the moment resisted by non-uniform shear

$e_1 =$ distance from the centroid of the shear perimeter to the point of maximum shear
on the perimeter.

$J =$ polar moment of inertia of the critical section

a) Shear stress distribution

b) Interaction between shear and moment

Figure 2.8: Eccentric shear stress model for interior slab-column connections

If the shear component of the moment is taken to be only that due to vertical shear, both γ_v and J would be smaller than if horizontal shear due to torsion were included. For a critical section with dimensions x perpendicular and y parallel to the moment axis, J is calculated as follows:

With torsional effects included

$$J = \frac{x^3 d}{6} + \frac{x^2 y d}{2} + \frac{x d^3}{6} \quad (2.29a)$$

With vertical shear alone considered

$$J = \frac{x^3 d}{6} + \frac{x^2 y d}{2} \quad (2.29b)$$

If γ_v is taken the same for both cases, then according to Elgabry and Ghali (1990), J by Equation 2.29a would give stresses for which the resultants have a vertical component equal to V , but the moment components are slightly smaller (by about 3%)

than $\gamma_v M$. Elgabry and Ghali (1996) recommended the use of Equation 2.29a. The eccentric shear stress model, Equation 2.29, predicts a linear interaction between shear and moment, Figure. 2.8 (b).

2.2.4 Beam Analogy

An alternative to the plate theory and its simplification is to consider the area of the slab-column junction as a set of orthogonal beams. In principle, the resistance of each beam to the combined effects of bending, shear and torsion can be estimated by standard methods. However, Regan (1985) questioned the applicability of code approaches to the side face beams with high torsion. Code methods treat beams without stirrups as having low torsional capacities governed by the high torsional shear stress at their longer edges, while in a slab with two-way steel, the horizontal shear along the long top and bottom surface directions have suitable reinforcement, thus the lack of stirrups is relevant only for vertical shears. Another problem with any beam analogy is the division of the overall loads V and M between the bending, shear and torsion of the various "beams".

Park and Islam (1976) developed a beam analogy method for a critical section $d/2$ from the column face. The internal actions at the critical section are shown in Figure 2.9(a).

Figure 2.9: Beam analogy approach by Park and Islam (1976)

To derive the strength terms of the equilibrium equations, the following simplifying assumptions were made:

1. The reinforcement bars crossing faces AB and CD at the critical section yield, and the flexural strength is reached in negative bending on face AB and positive bending on face CD. For face AB this may be a reasonable assumption but the yielding of the positive steel on face CD is questionable.
2. The ultimate shear capacity is assumed not to be reduced by the development of the ultimate bending moment at the critical section.
3. The contributions of V_{AB} , V_{BC} , V_{CD} and V_{DA} to V_u are assumed to be according to the tributary areas of the surrounding slab.
4. A circular interaction relationship between the torsional and vertical shear stresses is assumed.

The result is an interaction diagram between moment and shear as shown in Figure 2.9(b). It should be noted that if the shear force to be transferred, V_u , equals V_o , then no

unbalanced moment can be transmitted by torsion or vertical shear; however, an unbalanced moment can be carried by bending. Thus, the theory implies that a slab-column connection loaded to its pure shear value can sustain an unbalanced moment equal to the sum of the negative moment strength of face AB and the positive moment strength of face CD of the critical section shown in Figure 2.9(a). In reality, however, the positive reinforcement at face CD will act as compression reinforcement for the negative "balanced" moment at face CD. Thus, the possibility to resist an unbalanced moment by the positive reinforcement at face CD in the presence of high negative moments is highly questionable.

Plastic Theory

In the preceding discussion it has been assumed that an unbalanced moment between a slab and a column will always produce vertical shear stresses in the slab. From the point of view of plastic theory, it could be argued that shear could be avoided if the flexural capacities at the ends of the critical section were sufficient to resist the entire moment. Vertical shear is also avoidable if the moment is less than the combined resistances available from flexure and torsion due to horizontal shear. These notions were employed in some early design proposals such as those by Di Stasio and Van Buren (1960) and were stated in the commentary on the ACI 1963 Building Code, but appear to have been withdrawn from the provisions of the design codes in the light of test results.

Dilger and Cao (1994) established a yield line pattern for a square column and expressed the unbalanced moment M that can be resisted by the connection in the presence of a shear force V as:

$$M = 2(1 + \Pi) (1 + k) mc - 0.5 Vc \quad (2.30)$$

Where

$$\Pi = 22/7$$

m = negative moment capacity per unit width of the slab

c = dimension of the column

k = ratio of the positive to negative moment capacity

Equation 2.30 predicts a linear interaction diagram between the shear force V and the unbalanced moment M .

2.2.5 Other Approaches

The problem of dealing with distributions of shear can be avoided by the Nölting (1984) method. In his method no additional assumptions were required to treat eccentrically loaded slab-column connections, and the effect of the transferred moment is simply included in the calculation of the critical moment M_u at the column perimeter, viz

$$M_u = V(k_1 - 0.0955 \ln \frac{c}{l_m}) + \frac{0.21M}{c} \quad (2.31)$$

where

M = the transferred moment.

k_1 = constant, depending on the statical system

c = dimension of the column

l_m = span between points of contraflexure

= 0.46 the span between columns for full slab systems

Note that Equation 2.31 predicts a linear interaction between shear force V and unbalanced moment M .

2.2.6 Empirical Relations

Sherif (1990) and Sherif and Dilger(1996) adopted the ACI eccentric shear stress model with a modified fraction, γ_v , of the unbalanced moment resisted by shear stresses. They used the available test data in the literature to propose a correction factor, r , for the value of γ_v given by the ACI. The modification is derived based on the assumption that the eccentric shear stress model is valid. The correction factor takes into account the reinforcement ratio of the slab as follows:

$$r = 1.0 - 30 \rho \quad (2.32)$$

The failure shear stress, v_u , required by the eccentric shear model was determined by tests in which only concentric shear force was applied. They proposed that v_u is to be determined according to either of the following two equations, whichever results in a lower shear strength:

$$v_u = 0.7 (100 \rho f_c')^{1.3} \quad (2.33)$$

or

$$v_u = 0.7 \left(\frac{6.7d}{b_o} + 0.4 \right) (100 \rho f_c')^{1.3} \quad (2.34)$$

Figure 2.11: Important slab parts identifications and critical section location

ABFE: Section of maximum compression around the column.

GHKJ: Critical section for shear stress.

c.c.: Column centre.
c.g.: Critical section centroid.

Figure 2.12: Critical section for shear stress distribution according to Zaghloul and Ben Sasi (2001)

This configuration of the critical section is used in conjunction with the limiting shear stress v_n according to Equation (2.36).

The method is based on a set of simplifying assumptions:

1. The tensile strength of concrete is negligible
2. The compressive stresses induced in the reinforcing bars located in the slab compression zone are neglected
3. The forces due to the dowel action of the reinforcing bars are ignored
4. The slab compression zone is assumed uniformly stressed at the ultimate failure state
5. Steel bars crossing the failure surface are assumed to have attained their yield strength at failure
6. Based on experimental evidence, concrete strain in the least stressed region normal to line CD in Figure 2.12 is assumed negligible, accordingly, the flexural moment induced in the slab at that line is ignored.

With the preceding assumption in mind, the unbalanced moment, denoted as M_u , acting at the centroid of the critical section is computed using

$$M_u = M + V y_{c.g.} \quad (2.35)$$

where, V and M are, respectively, the shear force and the moment acting at the center of the column cross section, $y_{c.g.}$ is the distance between the centroid of the shear critical section and the center of the column, Figure 2.12.

The moment M_u is assumed to comprise two components; namely, moment $M_v = \gamma_v M_u$ resisted by eccentricity of shear at the critical section and a moment $M_f = \gamma_f M_u$ resisted by the flexural resistance of the slab over a width of $(c_2 + 2c_1)$, where

$$M_f = (c_2 + 2c_1) m_f \quad (2.36)$$

$$m_f = \rho_1 d_1^2 f_y (1 - 0.59 \rho_1 f_y / f_c) \quad (2.37)$$

where

c_1 = the column side length in a direction normal to the vector of the applied moment

c_2 = the column side length in a direction parallel to the vector of the applied moment

d_1 = the slab effective depth as related to the steel bars normal to the moment vector

ρ_1 = the ratio of the tension reinforcement placed normal to the moment vector

f_y = the reinforcement yield strength

f_c = the cylinder compressive strength of concrete

Symbols γ_v and γ_f are fractions, the sum of which is equal to unity. The fraction γ_v is given by the following equation:

$$\gamma_f = r \{ 1 - M_f / M_u \} \quad (2.38)$$

Substituting from Equation (2.35) & (2.36) into Equation (2.38) gives:

$$\gamma_v = r \left\{ 1 - \frac{(c_2 + 2c_1) \frac{m_f}{V}}{\left\{ \frac{M}{V} + \frac{(c_1 + d)\beta}{2[\beta(1 + \sqrt{2}) + 1]} \right\}} \right\} \quad (2.39)$$

where r is a factor determined by the following least squares regression equation:

$$r = 1.142 - 0.008 M/V(c_1 + d) - 0.498 m_f / \beta V \quad (2.40)$$

β is the aspect ratio of the column stub given as:

$$\beta = (c_1 + d) / (c_2 + d) \quad (2.41)$$

m_f = same as equation 2.37

The ultimate shear stress, v_u , and the maximum shear stress, v_n , are therefore calculated according to the following equations which resemble those of the ACI eccentric shear equations:

$$v_u = \frac{V}{A_c} + \frac{\gamma_v M_u Y_1}{J_c} \leq v_n \text{ MPa} \quad (2.42a)$$

where,

$$v_n = 1/6 [1 + 2/\beta_c] (f'_c)^{1/2} \leq 0.33 (f'_c)^{1/2} \text{ MPa} \quad (2.42 b)$$

A_c = is the area of the critical shear section

$$A_c = (l_1 + 2l_2 + l_3) d$$

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

l_1 = length of the side GH of the critical section as shown in Figure 2.12

l_2 = length of the side HK (or GJ) of the critical section as shown in Figure 2.12

l_3 = length of the side KJ of the critical section as shown in Figure 2.12

d = average effective depth of slab

y_1 = distance of side GH of the shear critical section from its centroid

y_2 = distance of side KJ of the shear critical section from its centroid.

J_c = the polar moment of inertia of the critical section and is computed as

$$J_c = d \{ l_1 y_1^2 + \sqrt{2} l_2 [(l_2^2 + d^2)/12 + 2y_{c.g.}^2] + l_3 y_2^2 \} \quad (2.43)$$

The moment M_f , which is equal to $(1-\gamma_v) M_u$ is resisted by flexure over a width of $(c_2 + 2c_1)$ of the slab centred on the column, and is to be calculated using Equation 2.36

The method is claimed to be more accurate than the ACI (1995) method. This claim is supported on the basis of the factors that are considered to have effect on the

value of the fraction γ_v . The ACI (1995) method assumes dependence of γ_v only on the critical section dimensions, as can be seen from Equation 2.45. On the other hand, as the latter method is essentially based on the observed mode of failure, most of the relevant factors are included in its derivation of γ_v : namely, M/V , $(c_1+d)/(c_2+d)$ and the reinforcement ratios, the moment resistance of the slab section in the column vicinity, m_f , and the critical section perimeter properties.

2.2.8 Building Codes Methods

Most current codes incorporate expressions of the type of Equation 2.44 for punching shear design. The provisions of the ACI, the Canadian CSA, the British BS and the European CEB-FIP codes for shear strength of interior column-slab connections are presented in the following:

The ACI Code (1995)

The ACI code has adopted the eccentric shear stress model, (Equation 2.44). The total applied shear stress due to a shear force V and an unbalanced moment M is calculated as:

$$v = \frac{V}{b_o d} + \frac{\gamma_v M e_1}{J} \quad (2.44)$$

where

b_o = perimeter of critical section $d/2$ from the column face

e_1 = distance from centroid of critical shear perimeter to the point of maximum shear stress on the perimeter of critical section

J = property of assumed critical section analogous to the polar moment of inertia

γ_v = is portion of unbalanced moment resisted by non-uniform shear and is calculated as:

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3} \sqrt{\frac{b_1}{b_2}}} \quad (2.45)$$

where

b_1 = length of critical section in the direction perpendicular to the moment axis

b_2 = length of the critical section in the direction parallel to the moment axis

the value of $v \leq v_c$ where v_c is the same as in the case of concentric shear punching (see Equations 2.14 to 2.16)

The code further stipulates that,

- (1) The remaining portion of unbalanced moment $(1-\gamma_v)M$ has to be resisted by flexure within a width of c_2+3h , where h is the slab thickness.
- (2) The code allows a reduction of γ_v by 25%, provided that the factored shear stress v_u (excluding the shear caused by moment transfer) does not exceed 40% of the direct shear stress capacity ϕv_c as determined from the concentric punching equations.
- (3) The reinforcement ratio required to develop the unbalanced moment $(1-\gamma_v)M$ is less than or equal to $0.375\rho_b$, where ρ_b is the balanced reinforcement ratio.

The Canadian Code CSA A23.3-94.(1994)

The pertinent Canadian code provisions in this respect are similar to those of the ACI code. It also adopts the eccentric shear stress model and uses the same equations as

the ACI to calculate the total shear stresses due to shear and unbalanced moments. However, the Canadian code does not allow the reduction in γ_v allowed in the ACI code. The value of v_c is calculated from concentric shear case as given in Equations 2.17, 2.18 and 2.19

The British Standard (BS 8110-85, 1985)

To account for unbalanced moments that have to be transferred between the slab and the column, the code increases the effective shear force according to the following equation

$$V_{eff} = V \left(1 + \frac{1.5M}{Vx} \right) \quad (2.46)$$

where x is the length of the side of the perimeter considered parallel to the axis of bending. The shear resistance is calculated using Equation 2.21.

In the absence of calculation, the code allows one to calculate $V_{eff} = 1.15V$ for internal columns in braced structures with approximately equal spans, where V is calculated on the assumption that the maximum design load is applied to all panels adjacent to the column considered. It is interesting to note that in the British Code the total shear stress depends on the dimension x . An increase in x results in a decrease in V_{eff} . In case of the American and Canadian codes, the total shear stress is a function of the ratio of the critical section dimensions (b_1/b_2) and not of a specific length.

The European CEB-FIP Model Code (1990)

Due to an unbalanced factored moment, M_{sd} , the code assumes a plastic distribution of the shear stresses as shown in Figure 2.13. This stress distribution is added to the effect of the concentrated load to get the final shear stress distribution due to the applied shear force F_{sd} and the unbalanced moment M_{sd}

$$\tau_{sd} = \frac{F_{sd}}{u_1 d} + \frac{KM_{sd}}{w_1 d} \quad (2.47)$$

where

K = coefficient determining the portion of M_{sd} resisted via shear stresses and is a function of c_1/c_2 , see Table 2.1.

w_1 = parameter analogous to the plastic section modulus of the critical section

and is calculated as:

$$w_1 = \int_0^{u_1} |e| dl$$

where e = natural logarithm

for design $\tau_{sd} \leq \tau_{rd}$ where τ_{rd} is given by Equation 2.24

Figure 2.13: Shear stress distribution due to unbalanced moment at interior column-slab connection according to CEB-FIP Model Code 90

Table 2.1: Portion K of unbalanced moment to be resisted by shear for interior columns according to the CEB-FIP Model Code 90

c_1/c_2	0.5	1.0	2.0	3.0
K	0.45	0.6	0.7	0.8

Strictly speaking, Equation 2.47 contains an error in that the principle of superposition is not applicable for a plastic stress distribution. By adding the stresses due to direct shear (constant stress distribution) and due to an unbalanced moment (plastic stress distribution) Equation 2.47 results in a total stress distribution that is neither elastic nor plastic.

The maximum loading for which any connection (including connections with shear reinforcement) may be designed depends on the column perimeter u_o and is defined by

$$\frac{F_{sd,ef}}{u_o d} \leq 0.5 f_{cd2}$$

where

u_o = length of the periphery of the column or loaded area

$F_{sd,ef}$ = the punching load, enhanced to allow for the effects of an eventual moment transferred to the column and is calculated as:

$$F_{sd,ef} = F_{sd} \left(1 + K \frac{M_{sd} u_l}{F_{sd} w_l} \right) \quad (2.48)$$

It is evident that the various code provisions are based on a similar approach and they all assume that the connection transfers the axial load and a portion of the unbalanced moment by shear acting on the perimeter of the critical shear section.

However, they differ in their definition of the critical section and its location and in the fraction of the unbalanced moment transferred by shear. More importantly, all these methods are highly empirical and are therefore difficult to directly extend to FRP reinforced slab-column connections.

2.3 Previous Work on Flat Plate Floor Slab-Columns Connections Reinforced with FRP

2.3.1 Concentric Punching

Ahmad et al. (1994) conducted a study on punching shear strength of slabs reinforced with carbon fibre reinforced polymer (CFRP) reinforcement. They carried out this pilot study for examining the punching shear behaviour of concrete slabs reinforced with three-dimensional continuous carbon fibre fabric. The 3-D continuous mesh was made of polyacrylonitrile continuous fibres (PAN type) having a diameter of 0.007mm, a specific gravity of 1.79, tensile strength of 3432 MPa, elastic modulus of 235GPa and maximum elongation of 1.5%. The fabric was constructed from the fibres by three-dimensional weaving and each element was epoxy-coated to form a rigid lattice. The nominal cross-sectional area of each element, including the epoxy coating was 4.2 mm², of which 1.82mm² was occupied by the fibres. The average value for the apparent modulus of elasticity, based on three specimens, was 113 GPa, with a fracture strain varying between 8000 and 11,800 microstrains.

A total of six 690 x 690 x 80 mm concrete slab specimens were fabricated. Four of the slabs were reinforced with 3-D continuous carbon fibre fabric, the other two were

reinforced with conventional mild steel reinforcement. Of the four CFRP reinforced slabs, two were fabricated with a column stub and the other two without a column stub. The reinforcement ratios in the three directions (in the plane and through the thickness) for the CFRP reinforced slabs were 0.95 percent. The deformed steel bars used for reinforcing the control slabs had average yield strength of 400 MPa. The reinforcement ratios in the plane of the steel reinforced slabs were 1.18 and 1.35 percent, respectively. The 28-day average concrete strength in the slab was 30 MPa. Dimensions and details of the slab specimens are presented in Table 2.2.

Table 2.2: Test program for punching shear tests of slabs reinforced with either 3-D carbon fibre fabric or reinforcing bars.

Specimen Type	Dimension $l_1 \times l_2 \times h$ (in.)	Effective depth (d) (d_1, d_2, d_3) (in.)	Area of one element x no. of elements x row of elements or bar details	Reinforcement Ratio ρ ($A_s / l d$) (%)	Strength of carbon fibre or steel (ksi)	Size of load head or stub, c (in.)
CFRC-S-N-1	27x27x3	1.6 (2.4, 1.6, 0.8)	0.0065x21x3	0.95	195	3x3
CFRC-S-N-2	27x27x3	1.6 (2.4, 1.6, 0.8)	0.0065x21x3	0.95	195	3x3
CFRC-S-3	27x27x3	1.6 (2.4, 1.6, 0.8)	0.0065x21x3	0.95	195	4x4
CFRC-S-4	27x27x3	1.6 (2.4, 1.6, 0.8)	0.0065x21x3	0.95	195	4x4
RC-S-1	27x27x3	2.4	7#3	1.18	60	4x4*
RC-S-2	27x27x3	2.4	8#3	1.35	60	4x4*

* Slabs with column stub

The slabs were tested in concentric shear under monotonic load. The crack patterns for all the slab specimens after failure indicated that they failed in punching shear, and a large number of cracks developed in the CFRP reinforced slabs. The total length of the perimeter crack surrounding the loaded area was smaller for CFRP reinforced slabs compared to the steel reinforced slabs.

The CFRP reinforced slabs were stable in the post-peak region of the load-deflection curves and for a constant load, the cracks did not propagate. The pre-cracking behaviour and the initial stiffness of the CFRP and the steel reinforced slabs were similar, however, the post cracking stiffness of CFRP reinforced slabs usually dropped noticeably whereas for the steel reinforced slabs, the reduction in stiffness was small. Figure 2.14. Ahmad et al. attributed the greater reduction in the post-cracking stiffness of the CFRP reinforced slabs to the lower apparent elastic modulus and smaller amount of the CFRP reinforcement.

It was also noticed that the CFRP reinforced slabs exhibited significant non-linear behaviour before the maximum load and the deformation exhibited a softening behaviour after the maximum load. Ahmad et al. considered the post-peak load softening behaviour as a relative measure of ductility, which would allow redistribution of stresses after the maximum load.

They used the ACI 318-89 punching shear relation Equation 2.16 without the capacity reduction ($\phi = 0.85$) and the BS 8110 Equation 2.21 to calculate the punching capacity of their slabs. The comparison of the experimental failure loads with the values predicted by the codes is shown in Table 2.3

Figure 2.14: Load-deflection curves of slabs reinforced with either carbon fibre fabric (CFRC), or mild steel reinforcing bars (RC). (Ahmad et al (1994))

Table 2.3: Test results of slabs reinforced with either 3-D carbon fibre fabric or mild steel reinforcing bars investigated by Ahmad et al. (1994).

Specimen	Cylinder strength f'_c (ksi)	Test age (days)	Ultimate load of slab P_u (kip)	$P_{ACI\ 318}$ (kip)	$P_{BS\ 8110}$ (kip)	P_u/P_{ACI}	$P_u/P_{BS\ 8110}$
CFRC-S-N-1	6.15	55	20.80	16.26	23.09	1.25	0.90
CFRC-S-N-2	6.47	53	17.53	16.68	23.48	1.08	0.75
CFRC-S-3	5.66	33	21.50	18.49	24.14	1.20	0.89
CFRC-S-4	5.31	37	22.18	17.91	24.46	1.20	0.90
RC-S-1	5.65	32	30.00	18.47	26.05	1.62	1.15
RC-S-2	5.78	38	32.33	18.68	27.45	1.73	1.18

* For CFRC slabs, effective depth in computing P_{ACI} was effective depth "d"

The experimental ultimate load values for the CFRP reinforced slabs were 8 to 25% higher than those predicted by the ACI and 10 to 25% lower than those predicted by the BS-8110 code. On the other hand, for the steel reinforced slabs, the observed ultimate loads were 62 to 73% higher than predicted by the ACI code, and 15 to 18 percent higher than predicted by the BS 8110 code. It would appear that the code equations predict the strength of FRP reinforced slabs better than those of the steel reinforced slabs.

Matthys and Taerwe (1996) tested eight 1000 x 4500 mm slabs with thickness of 120 or 150 mm, reinforced with different types of FRP grids. The slabs were initially tested in 4-point bending over a span of 4 m. After they performed the bending tests, square slab specimens with dimensions 1 m x 1 m were cut from the end zone of the slabs for punching tests. The punching test specimens were partially damaged (flexural cracks) due to the bending tests.

Matthys and Taerwe (1996) conducted concentric punching shear tests on five specimens. Four of these were 120 mm deep while one specimen was 150 mm deep.

The tested slabs included in this series were reinforced with different types of reinforcements. Slab R and CS were reinforced with a steel mesh (ϕ 10 mm S500, ribbed) and a CFRP mesh (CFRP carbon bar ϕ 5mm, sanded surface) respectively. For the other tested slabs NEFMAC grids type C (carbon fiber embedded in a vinylester resin) were used. These grids are made from continuous impregnated fibres alternating in two directions to form a cross laminate grid structure.

The characteristics of the reinforcement are given in Table 2.4, while the concrete properties at age of 28 days are given in Table 2.5

Table 2.4: Characteristics of the reinforcement

specimen	Sectional Area A (mm ²)	Tensile Strength ^(*) (N/mm ²)	Modulus of Elasticity (N/mm ²)	Ultimate strain (%)
S500 10 mm	78.5	607	200000	8.2
CFRP 5 mm	19.6	2300	150000	1.4
NEFMAC C 10	39.2	1620	90300	1.8
NEFMAC C 13	65.0	1350	95400	1.3
NEFMAC C 16	100.0	>1320 ^(**)	95000	>1.4 ^(**)
NEFMAC H 10	88.8	660	40500	1.6
NEFMAC H 16	223.0	640	42900	1.5
NEFMAC H 19	335.0	>560 ^(**)	41000	>1.4 ^(**)

They used the following empirical relations as well as the British BS 8110, CEB-Model MC90 and the ACI code punching shear equations to calculate the punching shear capacity, Q , of the test specimens. The empirical formulas they used are:

$$Q_{emp1} = 1.36 \frac{(100 \rho f_c)^{1/3}}{d^{1/4}} u_{BS} d \quad (\text{S.I. units}) \quad (2.49)$$

$$Q_{emp2} = 4 \sqrt{f_c} \left(1 + 2 \frac{d}{c}\right) u d \quad (\text{S.I. units}) \quad (3.50)$$

where

f_c = concrete cylinder compressive strength

ρ = reinforcement ratio

d = effective depth

u_{BS} = rectangular critical shear perimeter at a distance $1.5d$ from loaded area

u = perimeter of loaded area

c = side length or diameter of loaded area

The ratios of the experimental to the predicted ultimate loads (Q_{exp}/Q_{pred}) are shown in Figure 2.15. The flexural capacity of the slabs, Q_{flex} , was calculated by elastic analysis (Bareš, 1979) and compared to the results of the bending tests. The punching shear capacity of the slabs without flexural effects, Q_{ps} , was calculated by an upper bound plastic analysis per Nielsen (1984).

Figure 2.15: Comparison of the predicted to the actual ultimate shear strength of Matthys and Taerwe's slabs. (Note: Bars in the figure are in the same order (from left to right for the five slabs) as listed in the legend to the right (from top down to the bottom))

It is clear from Figure 2.15 that among these methods, the CEB model code predictions agree closely with the experimental data.

Matthys and Taerwe (1997) also reported the results of another 12 punching tests. Five of these test slabs were the same as described earlier. Concentric punching tests were performed on square slabs with side length 1000 mm and a total depth of 120 or 150 mm. These slabs, except for the two slabs cast as reference, were cut from the end zones of the larger slabs similarly to the ones described earlier. The 12 test specimens were divided into three series: Series R with steel reinforcement, Series C with CFRP (carbon fibre FRP) grids reinforcement, and Series H with hybrid FRP reinforcement, comprising both glass and carbon fibres. Also different diameters for the loading area of test slabs were used. Further details are provided in Table 2.5.

Once again various methods were used to compare the experimental and predicted punching strength of these slabs. The results are summarized in Table 2.6. Note that all the slabs were loaded via circular steel loading plate and they all failed in punching shear.

These results showed that FRP reinforced concrete slabs designed with a similar flexural strength as a conventional steel reinforced slab, which normally would have higher flexural stiffness, have significantly lower punching strength. Slabs designed with a similar flexural stiffness as steel have similar or higher punching strengths when reinforced with CFRP and slightly lower punching strengths when reinforced with the hybrid reinforcement. Comparing slabs with similar flexural stiffness, the effect of increased slab depth on the punching resistance seems to be more pronounced than the effect of increased reinforcement ratio.

Table 2.5: Over view of the characteristics of the different test specimens

Slab	f_c' (MPa)	Age	Diameter of Load Area patch (mm)	Slab	Mean effecti	Reinforcement	Bar	Reinf.
		at test		Depth	Depth d	Type	Spacing	ratio ρ
		(days)		(mm)	(mm)		(mm)	(%)
R1/R1'	33.5	466/835	150/230	120	90	Ø10mm S500	150	0.58
R2 ^b	35.1	237	150	120	88	Ø 12 mm S500	100	1.29
R3 ^b	35.1	363	150	120	86	Ø 14mm S500	100	1.79
C1/C1'	30.4	470/817	150/230	120	96	Grid type C10	150	0.27
C2/C2'	29.6	444/805	150/230	120	95	Grid type C16	100	1.05
C3/C3'	28.0	442/799	150/230	150	126	Grid type C13	100	0.52
Cs/cs'	27.2	376/722	150/230	120	95	Ø 5mm CFRP S	110	0.19
H1	96.7	678	150	120	95	Grid type H10	150	0.62
H2/H2'	29.3	685/757	150/80	120	89	Grid type H19	100	3.76
H3/H3'	26.3	651/723	150/80	150	122	Grid type H16	150	1.22

^a Mean compressive cylinder strength of the concrete at 28 days.

^b Newly cast slabs (no initial flexural cracks).

^c For slabs C2' and C3', respectively, 1 and 3 of the initial cracks ran over the total depth

The complete results of the test specimens of the second and third phases were reported by Matthys and Taerwe (1997 and 2000) while selected results are recapped here.

Table 2.6: Test results and of their theoretical verification (Phases 1 and 2 of Matthys and Taerwe).

Slab	Q _{exp} [kN]	Fail mode	Q _{exp} /Q _{flex} (a)	Q _{exp} /Q _{CEB}		Q _{exp} /Q _{EC}		Q _{exp} /Q _{BS}		Q _{exp} /Q _{ACI}	Q _{exp} /Q _{emp}		Q _{exp} /Q _{mech}
				(b)	(c)	(b)	(c)	(b)	(c)	(b)	(b)	(c)	(d)
R1	240	P	2.11	1.25	1.25	1.44	1.44	1.2	1.2	1.62	1.21	1.21	1.08
R2	294	P	1.32	1.22	1.22	1.5	1.5	1.17	1.17	2	1.18	1.18	1.12
R3	313	P	1.14	1.2	1.2	1.57	1.57	1.15	1.15	2.2	1.16	1.16	1.14
C1	181	P	1.14	1.15	1.49	1.15	1.21	1.11	1.45	1.18	1.11	1.45	1.31
C2	255	P	0.75	1.07	1.37	1.36	1.58	1.04	1.33	1.7	1.04	1.33	1.19
C3	347	P	0.88	1.25	1.6	1.41	1.52	1.24	1.59	1.59	1.22	1.56	1.23
CS	142	P	1	1.09	1.2	1.02	1.04	1.06	1.17	0.99	1.06	1.17	0.91
H1	207	P	1.37	-	-	-	-	-	-	-	-	-	-
H2	231	P	0.68	0.7	1.18	0.83	1.48	0.67	1.14	1.7	0.68	1.15	0.97
H2'	171	P	0.63	0.8	1.02	0.73	1.32	0.6	1.01	1.78	0.6	1.02	0.97
H3	237	P	0.5	0.67	1.13	0.87	1.13	0.67	1.12	1.18	0.66	1.1	0.73
H3'	217	P	0.57	0.81	1.16	0.92	1.2	0.71	1.19	1.45	0.7	1.16	0.86

P: punching cone failure; F: flexural failure

(a): elastic analysis of the flexural strength;

(b): empirical expressions for punching shear;

(c): modified empirical expressions;

(d): modified mechanical model

As for the analysis despite considerable scatter, all the models predicted the punching capacity in a consistent way, but the mean ratio of the ultimate load to the predicted load, Q_u/Q_{pred} , differs for the three series. For series R (steel reinforcement)

this ratio is about 1.2 to 1.9, indicating extra conservatism by the models. Lower values for the mean ratio Q_u/Q_{pred} are found for series C/CS and H, about 0.9 to 1.3 and about 0.7 to 1.5 respectively. For series H, with very low modulus of elasticity of the grid reinforcement, the punching strength is overestimated (except for the ACI equation). In view of the lower ratios Q_u/Q_{pred} for FRP reinforced slabs, their mean punching capacity was also calculated taking into account the axial rigidity of the reinforcement by replacing the reinforcement ratio ρ by the equivalent ratio $\rho E_r / E_s$ as specified by JSCE (1997), where the E_r is the modulus of elasticity of FRP reinforcement and E_s is the elasticity modulus of steel reinforcement. By doing so, safer predictions and less scatter was obtained. The results of these tests are encouraging, but it needs to be determined whether the test method is appropriate for simulating punching shear behaviour of slab-column connections. Furthermore, the tests involve concentric shear; therefore, their applicability to combined shear and unbalanced moment type of failure needs further study.

El-Ghandour et al. (1997, 1998), reported results of four concentrically loaded slab-column connections in a flat plate structure. The slabs were designated as SG1, SC1, SCS1, and SCS1.

The four specimens had identical dimensions of 2000 x 2000 x 175 mm with central square column stub of 200 x 200 mm cross-section and 200 mm height. Specimen SG1 was reinforced with GFRP bars, specimen SC1 with CFRP bars, while specimens SGS1 and SCS1 had identical planar reinforcement as SG1 and SC1, respectively, but they also had shear reinforcement. Table 2.7 provides details of the reinforcement.

The shear reinforcement was specially manufactured of high strength CFRP flat strip of thickness 1 mm and width 25 mm. This is the so-called shear band reinforcement system, Figure 2.16, and was developed at Sheffield University.

Figure 2.16: CFRP Shear band

Table 2.7: Dimensions and reinforcement details of El-Ghandour et al. (1997) slabs.

Slab*	Dimensions (mm)			Flexural Reinforcement (Spacing – 200 mm)				Shear GFRP 1×25 mm
	Length	Width	Thickness	GFRP (rough surface)		CFRP (rough surface)		
	mm	mm	mm	Longit.	Transv.	Longit.	Transv.	
SG1	2000	2000	175	11Ø8.5	11Ø8.5			
SC1	2000	2000	175			9Ø8.5	9Ø8.5	
SGS1	2000	2000	175	11Ø8.5	11Ø8.5			256 legs*
SCS1	2000	2000	175			9Ø8.5	9Ø8.5	224 legs

* Spacing of shear reinforcement for both slabs was the same, but SGS1 contained two more strips than slab SCS1.

** The concrete strength for SG1 and SC1 were 41.6 MPa and 43.4 MPa respectively and those of SGS1 and SCS1 were 46.1 MPa and 36.1 MPa respectively.

The slabs were concentrically loaded. They were first loaded up to 150 kN, then subjected to one unloading - reloading cycle and then loaded to failure.

Figure 2.17: Load-central deflection relationship of El-Ghandour et al. test specimens

Crack development in all the four slabs followed a similar pattern. The first cracks opened up on the top surface in the form of flexural cracks above the column stub, in the weaker direction, at around 100 kN for both SG1 and SCS1, 130 kN for SGS1 and 150 kN for SC1. A noticeable drop in load was caused as a result of first cracking. After the initial load stage, cracks propagated from the middle outwards and reached the slab edges. Subsequently, with increasing loads, more cracks developed and advanced radially from the column faces towards the slab edges, along the four axes of symmetry of the slab (central X and Y axes and two diagonals). Cracks parallel to the X and Y axes opened up at loads of 130 kN and 190 kN for SG1 and SC1, respectively, and at a load of 180 kN for both SGS1 and SCS1, whilst cracks parallel to the diagonal axes opened up at higher loads. By the time the applied load reached 150 kN and 220 kN for SG1 and SC1 respectively, and 195 kN for both SGS1 and SCS1, only a few new diagonal cracks developed and most of the cracks had already formed. After that, with increasing load, the width of the cracks closer to the column increased substantially.

Slabs SG1 and SC1 failed after undergoing substantial deformations due to a suspected bond slippage of their flexural bars, a fact that accelerated their failure at loads less than their flexural capacity. This suspected bond slip failure resulted in a sudden increase in the widths of the flexural cracks on the top of the slabs as well as compressive concrete crushing close to the column stub. The maximum-recorded loads for these two slabs were 170kN and 229kN respectively. Penetration of the column into the slab was noticed at the end of the tests. No clear evidence of punching shear failure or punching shear cracks was found on the top surface of the slabs.

Slab SGS1 failed at 198kN following large deflections due to possible bond slippage and before reaching its maximum expected flexural load. In this case, penetration of the column into the slab was noticed at the end of the test. Once again, no obvious punching shear failure or punching shear cracks was observed on the top surface of the slab. The flexural cracks close to the column on the top surface were quite wide and cracks also occurred on the bottom of the slab.

Slab SCS1 was reported to have failed in a combined flexural and bond slip failure mode and at a reduced flexural load of 200kN, which is lower than the companion specimens SC1 without shear reinforcement. From the crack pattern of this slab, it was reported that the flexural crack that caused failure followed closely the plane of flexural failure. Again, there was no clear evidence that punching shear failure occurred in this slab even though the column was noticed to have penetrated the slab.

Considering Figure 2.17, it is obvious that the post-cracking portions of the load-deflection curves of all slabs show no big increase in the load up to failure, and the stiffness after cracking is substantially lower than the one that would normally be

expected for RC slabs. Nevertheless, it is obvious that the post-cracking behaviour of the SC slabs is better than that of the SG slabs, since carbon bars have a higher elastic modulus.

They reported that the maximum strain in the shear reinforcement just before failure was 1400 and 1900 microstrain in slabs SCS1 and SCS1, respectively, corresponding to stresses of 154 N/mm^2 and 209 N/mm^2 in the CFRP shear reinforcement. This indicates that relatively low stresses were developed in the shear reinforcement at failure compared to the strength of the material. This could be attributed to the low failure loads of slabs SGS1 and SCS1, as mentioned previously, as well as the high amount of shear reinforcement provided in both slabs.

El-Ghandour et al., (1999) reported results of a second phase of testing on slabs without shear reinforcement. The results were used to develop a new approach for accurately predicting the punching shear capacity of FRP reinforced concrete slab without shear reinforcement. They adopted the BS 8110 approach without partial safety factor, for predicting the ultimate punching resistance of reinforced concrete slab-column connections. When using FRP flexural reinforcement instead of steel, three different shear design approaches were investigated. The first is the lower bound strain approach proposed by Clarke (1996), during the Eurocrete project. The second, leading to an upper bound solution, is based on a stress approach developed by El-Ghandour et al. (1998). Finally, a third approach was proposed following the comparisons of the previous two approaches with experimental results, taking into consideration the strain at

which punching shear failure occurred in the tested slabs. These three approaches mainly involve modification of the actual area of the flexural tension FRP reinforcement into an equivalent steel area using the punching shear equation of British standard. The differences among the three approaches is in the manner in which the actual area of flexural FRP reinforcement is modified.

Strain Approach

Clarke (1996) recommended the use of an equivalent area of reinforcement to be used in the BS 8110-85, (1985) punching shear equation, by multiplying the actual area by the modular ratio of FRP and steel

$$A_s = A_{FRP} (E_{FRP} / E_{steel}) \quad (2.51)$$

The above implies that punching shear failure will occur when both the force and strain in the FRP flexural reinforcement are roughly the same as in conventional steel reinforcement. Results of El-Ghandour et al. showed that the strain approach is conservative as shown in Table 2.8. This is because the FRP reinforcement does not yield or fail at the same low strain as steel and a higher stress can be developed in the bars before punching shear failure occurs. This is demonstrated by both the maximum and average experimental stress levels shown in Table 2.8.

Table 2.8: Experimental and predicted punching shear capacity of El-Ghandour specimens

Slab	Experimental Results			Predicted Failure loads			
	Σ_{max}	Σ_{avg}	Load	Strain Approach.	Stress Approach.		Modified Approach.
	(MPa)	(MPa)	(kN)	Load(KN)	Σ (MPa)	Load(KN)	Load(kN)
SG2	418	318	271	224	650	415	273
SG2	360	332	237	194	533	340	237
SC2	627	508	317	260	616	355	316
SGS2	649(shear)	446(shear)	270	286	-	-	272

The stress approach proposed by El-Ghandour et al. (1998), assumes that flexural strain has no effect on the punching shear capacity and that the force in the FRP reinforcement is equivalent to that of steel reinforcement. The modified value for the equivalent area of steel can only be obtained by finding the point of intersection of the punching shear capacity curve with the flexural capacity curve, as mentioned previously. Hence, the area of steel is modified as follows:

$$A_s = A_{FRP} (\sigma_{FRP} / \sigma_{steel}) \quad (2.52)$$

Where

A_s = equivalent steel area

A_{FRP} = actual A_{FRP} area in the slab

σ_{steel} = equivalent steel stress

σ_{FRP} = actual σ_{FRP} stress

The results of their slab tests showed that this approach provides an upper bound solution as shown in Table 2.8

Finally, since the above two approaches lead to upper and lower bound solutions, a third approach is proposed for design purposes. This approach takes partial advantage of the force that can be developed by FRP reinforcement beyond the lower bound strain limit to a new value of 0.0045. This new value is supported by the experimental stresses shown in Table 2.8. In fact, it can be noted that these experimental stresses correspond to experimental strains of the same order or even higher than the proposed value at punching shear failure. According to this approach, the equivalent area of steel is obtained as in the case of the strain approach, multiplied by a strain correction factor $\phi = 1.8$. Hence, the area of steel is modified by:

$$A_s = A_{FRP} (E_{FRP}/E_{steel}) (\phi) \quad (2.53)$$

Where

$$\Phi = \epsilon_{FRP} / \epsilon_{steel}$$

The correction factor for strain $\Phi = 1.8$ is obtained by allowing the tensile strain ϵ_{FRP} in the failure region to reach a value of 0.0045 when punching shear failure occurs, instead of limiting it to around 0.0025, which corresponds to $\epsilon_{yieldsteel}$. Test results in Table 2.8 show that this approach yields accurate results for both GFRP and CFRP flexural reinforcement.

The validity of the modified approach was also checked by comparisons with test results of Matthys and Taerwe (1997). The comparisons of the test results with the predictions of the modified approach showed that the modified approach is accurate in predicting the punching shear capacity.

Based on the above results suggestions were made for modifying the ACI 318-95 code equations to make them suitable for FRP reinforced slabs. The ACI 318-95 code ignores the influence of tension flexural reinforcement when calculating the concrete shear resistance and only relies on the concrete strength. In the case of steel reinforcement, with high modulus of elasticity, the dominant factor affecting the concrete shear resistance will be the concrete in compression, as the neutral axis depth does not vary much with normal steel reinforcement ratios. For this reason, in case of steel, the ACI 318-95 equations give good predictions. However, when using FRP reinforcement with low modulus of elasticity, the concrete shear resistance becomes more sensitive to the reinforcement stiffness, as the neutral axis depth is significantly smaller with low reinforcement ratios. In the latter cases, the ACI 318-95 becomes unconservative.

They therefore suggested that the v_c equations of ACI be multiplied by the term $(E_{FRP} / E_{steel})^{0.33}$ as for the BS 8110-85 code. Hence, the proposed equation for use with FRP reinforcement in case of a square internal column is

$$P_{ACI} = [1.33 (f_c)^{0.5} (c + d)d] (E_{FRP} / E_{steel})^{0.33} \quad (3.54)$$

where,

E_{FRP} = the modulus of elasticity of FRP reinforcement in MPa

E_{steel} = the modulus of elasticity of conventional steel MPa

The modified Equation (3.54) predictions were checked against the results of their study and were found to be in good agreement, but further research was proposed in this respect.

Ospina et al. (2000) reported test results of four full-scale isolated interior slab-column connections. Two specimens (GFR-1 and GFR-2) were reinforced with GFRP deformed bars, commonly known as "C-bars", one specimen (NEF-1) with a GFRP NEFMAC 2-D grid, and one (SR-1) with ordinary steel. The slab specimens were square with side length of 2150 mm and average effective depth of 120 mm. Other pertinent geometric and material properties of the test specimens are shown Table 2.9. C-bars were provided with mechanical end anchors to prevent premature bar slippage. Top mat steel reinforcing bars had 180 degree hooks. No end anchors were installed for the NEFMAC grid.

Table 2.9: Opsina et al. (2000) specimen properties and test results

Specimen	f'_c MPa	Rebar Type	P (%)	$E_{\text{reinf.}}$ (GPa)	$f_{r,y}$ (MPa)	$f_{r,u}$ (MPa)	$P_{u,\text{test}}$ (kN)	$P_{u,\text{pred}}$ (kN)	$\frac{P_{u,\text{test}}}{P_{u,\text{pred}}}$
SR-1	36.8	15M Steel	0.866	192.0	430	682	365.1	381.5	0.96
GFR-1	29.5	ϕ 15mm.	0.866	34.0	-	663	217.4	199.0	1.09
GFR-2	28.9	ϕ 15mm.	1.732	34.0	-	663	260.4	249.0	1.05
NEF-1	37.5	NEFMAC	0.866	28.4	-	566	206.4	203.0	1.02

Notes: i. Average effective depth was 120 mm. ii. NEFMAC ribs had 200 mm² nominal cross-section.

The load was applied from below in 5 to 10 kN increments by pushing the lower column stub which reacted against eight loading points on the slab, at a distance 900mm from column stub center,

The load-deflection response of the four specimens is shown in Figure 2.18. Load values include the self-weight of the specimen plus that of the load assembly (approx. 20 kN). Deflection values are the average of the deflections in both directions measured at 710 mm away from the column face and do not include the dead load deflection. Slab SR-1 (0.87% steel) displayed the uncracked, elastic-cracked and yielded phases typically associated with conventionally reinforced slabs. The responses of GFR-1 (0.87% C-bars) and GFR-2 (1.73% C-bars) were essentially bilinear, lacking the yield phase of SR-1. The response of NEF-1 (0.87% NEFMAC) was similar to that of GFR-1 with respect to strength and stiffness. However, the character of the response of NEF-1 was dramatically different, with several load drops throughout the cracked phase.

Figure 2.18: Load-deflection relationships of Opsina et al. specimens

All slabs had similar stiffness until flexural cracking. Immediately after cracking, the slope of the load-deflection curves correlated with the top mat stiffness. First yield of steel in SR-1 occurred at the column face at a deflection of about 6 mm. Yielding in SR-1 spread to all joint bars at the column face at a deflection of 10.6 mm. Punching failure of SR-1, GFR-1 and GFR-2 was marked by a significant drop in load and a jump in deflection. For the NEF-1, it was not clear at what deflection punching occurred, and that this specimen did not feature the sudden loss of load usually associated with punching. Instead, the column pushed through very gradually.

The crack patterns of the failed specimens matched the layout of the top reinforcing bars. This is expected as the reinforcing bars act as stress risers for tension perpendicular to the bar. Specimens SR-1, GFR-1, and GFR-2 had well-defined punching failure surfaces that intercepted the top surface of the slab. But they reported that, in contrast, the punching surface of specimen NEF-1 was not evident when viewed from above

At ultimate, peak reinforcement strains of about 5700, 7000, 6000 and 13000 $\mu\epsilon$ were measured at the column face in SR-1, GFR-1, GFR-2 and NEF-1, respectively. Measured strains in SR-1 showed that all bars reached yield before punching failure. Strains at ultimate in the C-bars were between 29 to 35 % of their rupture strain. Strains at ultimate in NEF-1 were in the order of 65 % of NEFMAC rupture strain. It is worth noting that NEFMAC ribs reached 19000 μ under a severe 48 mm deflection without showing evidence of full fibre rupture.

For the purposes of calculating the ultimate shear strength, they adopted the following simple expression that has been recommended by Matthys and Taerwe (1997), which they obtained by modifying an existing expression for punching in steel reinforced slabs with a factor that accounts for the lower stiffness of GFRP reinforcement,

$$V_r = 1.36 b_o d \frac{\left(100 \rho \frac{E_{GFRP}}{E_s} f'_c \right)^{1/3}}{d^{1/4}} \quad (2.59)$$

where

b_o = the perimeter of the critical section (taken at 1.5d away from the column face),

d = the effective flexural depth in mm

ρ = the slab reinforcement ratio

E_{GFRP} = the modulus of elasticity of the GFRP reinforcement

E_s = the modulus of elasticity of steel

f'_c = the concrete compressive strength in MPa.

The test results they obtained in this investigation are shown to be accurately predicted by Equation (2.59) , as indicated in Table 2.9.

It is evident from this literature survey that the topic of punching shear at slab-column connections is complex and generally different models are used by the different codes in order to predict the punching strength of steel reinforcement slabs. Attempts

have been made by a few investigators to modify these models for application to FRP reinforced slab-column connections. However, both the amount and scope of the existing test data on FRP reinforced concrete slabs do not provide sufficient empirical basis for the adoption of any of the existing modifications. In particular, no tests have been performed to date on FRP reinforced slab-column connections subjected to combined shear and unbalanced moment. The lack of such testing data is the motivation for the current study.

Chapter (3)

Testing Program

3.1 General

The studies conducted on the punching of flat plate structures since the fifties have been almost exclusively performed on floor slabs reinforced with conventional steel bars or prestressed steel cables or bars. Only recently a few research studies have been reported using fibre reinforced polymers reinforcement. However, FRP related tests to date have focussed on punching behaviour of test slab specimens subjected to simple concentric shear forces. The present test program is designed to investigate the behaviour of interior slab-column connections in CFRP reinforced flat plate structures. Test specimens will be tested under concentric shear (V) as well as different ratios of concentric shear and unbalanced moment, M/V . One specimen will be reinforced with traditional steel bars for comparison with a companion specimen reinforced with the same reinforcement ratio ($\rho=1.0\%$) of NEFMAC CFRP grid. A full description of the test specimens, test set-up, instrumentation and loading is provided in this chapter.

3.2 Test Program

The test program involves the design, construction, instrumentation, testing and data acquisition of interior slab-column connection specimens as illustrated in Figure 3.1. Note that the moment is applied through the eccentricity of the axial load from the column centre. The specimens are tested under single essentially monotonically increasing increments of load up to failure.

The test specimens are intended to simulate the interior slab-column connections and are assumed to be half scale, with the prototype structure having a 7.5 m span. The test slab is assumed to represent the portion of the slab between the column and the lines of contraflexure in a flat plate structure.

The test program includes testing to failure a total of eight specimens; one specimen, as stated before, being reinforced with conventional steel bars, and the other seven test specimens being reinforced with NEFMAC CFRP grids. Table 3.1 gives the dimensions, reinforcement amount and distribution, and the design moment /shear ratio of each test specimen. Notice that the planar dimensions of the slab as well as the geometry of the column stub were kept the same for all the test specimens, but the slab thickness and column cross-sectional dimensions were varied as test parameters.

3.2.1 Test Specimens

The test specimens are intended to simulate the interior slab-column connection and are assumed to be half scale, assuming a flat plate structure with 7.5 m square panels. The

slab in these specimens represents approximately the region of the floor slab around the column bounded by the lines of contraflexure.

Each test specimen is composed of 1760 mm x 1760 mm square slab and a central column stub extending below and above the slab. The distance between the lines of supports of the slab in either direction is 1500mm. The bottom unloaded column stub has a square cross section of 250 mm x 250 mm and is 440 mm long, Figure 3.1. The upper loaded column stub is of the same cross section as the bottom column stub but of height equal to 750 mm. This column stub is provided with a cantilever projecting 450 mm from the face of the column with a cross section of 250 mm x 500 mm, Figure 3.1. The column stubs and the cantilever are reinforced with conventional steel, as illustrated in Figs 3.2 and 3.3. The slab thickness is either 100 or 125 mm while the slab reinforcement varies per Table 3.1. The Table indicates that eight specimens were tested, which are denoted as ZJS, and ZJF1 to ZJF7. All the specimens were isotropically reinforced with equal amounts of top and bottom reinforcement as indicated in Table 3.1 and Figures 3.2 and 3.3. Specimen ZJS was reinforced with conventional steel rebars, Figure 3.2, while all the remaining specimens were reinforced with the CFRP grid NEFMAC, Figure 3.3. The fibre volume ratio of carbon NEFMAC is approximately 34%. However, essentially the fibres alone resist the tensile loads, hence the reinforcement ratio for the FRP reinforcement does not have the same significance as the steel reinforcement ratio.

Figure 3.1: Typical test specimen geometry and dimensions.

NEFMAC grids are ideally suited to reinforcement of slabs. In the current study we have also used them as shear reinforcement, akin to shear stud reinforcement. Figures 3.4a and 3.4b schematically show the shape and disposition of the CFRP shear reinforcement in one of the test specimens, while Figure 3.4c shows a photograph of such shear reinforcement. To the writer's knowledge, this is the first time that this type of the FRP shear reinforcement has been used in slabs.

3.2.2 Test Parameters

Table 3.2 shows the grouping of the test specimens according to the following parameters:

- 1- Type of reinforcement, i.e. steel versus CFRP
- 2- slab thickness or c/d ratio, i.e. 100 mm or 125 mm
- 3 Columns aspect ratio, c_1 / c_2 , i.e. 250 x 350 mm or 250 x 250 mm
- 4 Reinforcement ratio, ρ , i.e. 0.65%, 1.0% or 1.184%
- 5 M/V, ratio, i.e. 0, 0.22 or 0.33
- 6 Shear reinforcement, i.e. with or without the CFRP vertical shear reinforcement

Table 3.2 indicates that by grouping the slabs in the manner shown in the Table, one can compare the behaviour of two or more slabs by varying one of the six parameters listed above. The six rows in the Table, following row 1, correspond to the preceding six parameters. The shear reinforcement in row seven refers to the CFRP grid that simulates headed studs as illustrated in Figure 3.4

Notice that the reinforcement ratio of the FRP reinforced specimens is varied over a wide range. This is important because when comparing different types of reinforcement with different elastic moduli and strength, the reinforcement ratio is not the important parameter; rather it is either the strength or the rigidity of the reinforcement, which becomes important. As will be described in the following sections, the elastic modulus of the FRP used in the current study is roughly half of the modulus of steel.

Figure 3.2: Steel specimen reinforcement.

Section A-A

**Note: All dimensions
are in mm and
all bars are
Metric**

Figure 3.3: CFRP specimen reinforcement

Figure 3.4a: Plan showing location of shear reinforcement

Figure 3.4b: Arrangement of shear reinforcement in the vertical plane of the slab

Figure 3.4c: Typical NEFMAC vertical shear reinforcement

Figure 3.5: Specimen ZJS steel reinforcement

Table 3.1: Test specimens dimensions, reinforcement and M/V ratio

Specimen Identification	ZJS	ZJF(1)	ZJF(2)	ZJF(3)	ZJF(4)	ZJF(5)	ZJF(6)	ZJF(7)
Column stub (mmxmm)	250x250	250x250	250x250	250x250	250x250	250x250	250x350	250x250
Slab thickness	100 mm	100 mm	100 mm	100 mm	125 mm	100 mm	125 mm	125mm
Eff. Depth, d (mm)	81	75	75	75	100	75	100	100
Top & bottom reinforcement	18#10 ^{***} @ 90 mm	C16 ^{***} 18@100mm	C13 18@100mm	C16 18@100mm	C19 18@100mm	C16 18@100mm	C19 18@100mm	C19 18@100mm
Bottom reinforcement ratio ρ	1.2% ****	1.0%	0.65%	1.0%	1.184%	1.0%	1.184%	1.184%
M/V	0.22	0.22	0.22	0.30	0.22	0.00	0.22	0.22
Shear rein.	N/E	N/E	N/E	N/E	N/E	N/E	N/E	available

* N/E no shear reinforcement exists

** #10 cross-sectional area = 100 mm²,

*** C13 cross-sectional area = 65 mm²

C16 cross sectional area = 100 mm²

C19 cross-sectional area = 148 mm²

****Reinforcement ratio at the connection region.

Table 3.2: Test specimens classification based on their common characteristics

Studied parameter	Specimen	Column Stub mm x mm	Slab Thickness mm	Effect. Depth d mm	Top & Bottom reinf.	Tension reinf. ratio	M/V Ratio m	Presence of Shear Reinforcement
Type of Reinforcement	ZJS ZJ1	250x250 "	100 100	75 74	2#10 18C16	1.0% 1.0%	0.22 0.22	N/E N/E
Bott & top reinf. Ratio ρ	ZJ1 ZJ2	" "	100 100	74 74	18C16 18C13	1.0% 0.65%	0.22 0.22	N/E N/E
Col. Side / eff.d = (c/d) Ratio	ZJ1 ZJ4	" "	100 125	74 100	18C16 18C19	1.0% 1.184%	0.22 0.22	N/E N/E
Col. Asp. Ratio c_1/c_2	ZJ4 ZJ6	" 250x350	125 125	100 100	18C19 18C19	1.184% 1.0%	0.22 0.22	N/E N/E
M/V	ZJ5	250x250	100	74	18C16	1.0%	0.00	N/E
	ZJ1	"	100	74	18C16	1.0%	0.22	N/E
	ZJ3	"	100	74	18C16	1.0%	0.30	N/E
Presence of Shear Reinf.	ZJ4	"	125	100	18C19	1.184%	0.22	N/E
	ZJ7	"	125	100	18C19	1.184%	0.22	E**

*N/E no shear reinforcement exists, **E shear reinforcement exists.

3.3 Materials

3.3.1 Concrete

The concrete was ordered from a ready mix plant with a specified cylinder strength f_c' equal to 35MPa. The concrete was ordered in two batches; the first batch was used in the first stage for casting the slabs and the bottom column stubs. The second batch was used in the second stage for casting the upper column stubs and their cantilevers. Forty-eight control concrete cylinders (150mm x 300mm) were also cast and prepared.

Eight cylinders were tested after 28 days; five in compression and three in split cylinder tension. From these tests, the characteristic cylinder compressive strength f_c' , and modulus of rupture of concrete f_r were obtained as indicated in Table 3.3. Subsequently, at the time of testing each specimen, five cylinders were concurrently tested to find the compressive strength of concrete at that time. The results for these cylinders are given in Table A.1 to Table A.8 in Appendix A. In addition, using displacement control, the stress-strain relationship of concrete in compression was obtained for each test cylinder. Figure 3.6 shows the stress-strain curves of the five cylinders tested at 28 days. The softening portion of these curves are not as well-captured by the test as the ascending portion. A better test set-up is needed to capture the descending or softening branch of the stress-strain curve of concrete cylinders. However, the latter is not of particular significance in the context of the current work.

Figure 3.6: Stress strain relationship of concrete in compression after 28 days

Table 3.3: Compressive cylinder strength and splitting strength of concrete at 28 days

	Test Type	Test Results (MPa)	Average strength
Cylinder 1	Compressive strength	36.8	41.2
Cylinder 2	Compressive strength	41.2	
Cylinder 3	Compressive strength	41.6	
Cylinder 4	Compressive strength	42.6	
Cylinder 5	Compressive strength	44	
Cylinder 6	Splitting test	3.3	3.1
Cylinder 7	Splitting test	2.9	
Cylinder 8	Splitting test	3.1	

From Table 3.3 we can see that the average measured cylinder strength at 28 days was roughly 41.2 MPa, while its split cylinder tensile strength was 3.1 MPa.

The average measured concrete compressive strength at the time of testing of the various specimens is given in Table 3.4, we can notice the strength gain with age.

Table 3.4: Average compressive strength of concrete at the time of testing various specimens

Sample	Days after casting (Days)	Compressive strength (MPa)
Material property	28	41.0
ZJS	69	46.0
ZJF(1)	84	45.0
ZJF(2)	132	42.0
ZJF(3)	90	47.0
ZJF(4)	76	48
ZJF(5)	62	44.0
ZJF(6)	139	47.9
ZJF(7)	119	45.7

The second mix was ordered with higher strength of 40 MPa and super-plastisizer was added to it to make sure that the column stub was strong and fully compacted. From the second mix, six cylinders were cast and tested for compressive strength at different periods.

3.3.2 Reinforcement

Carbon Fibre Grids

Due to time limit and lack of proper grips, no tests were performed on the NEFMAC CFRP grids. The manufacturer's recommended values for modulus of elasticity, $E_{\text{CFRP}} = 100 \text{ GPa}$ and maximum tensile stress $F_u = 1.2 \text{ GPa}$ were assumed to be correct (AUTOCON 1998). CFRP is a completely linear elastic material as illustrated in Figure 3.7. Due to the high tensile strength of NEFMAC, failure of the connections due to failure of the grids is not expected. It is important to state the ribs of NEFMAC contains 34% carbon fibres outside the nodes or grid junctions and that the fibres are embedded in a vinylester resin. Figure 3.8 shows a typical grid while being cut by a normal carbide blade. According to the manufacturer (NRC 1994), the fibres in NEFMAC are made of pitch-based carbon with tensile strength of 4800 MPa and elastic modulus of 230 GPa. The resin has a tensile strength of 68-78 MPa and elastic modulus of 3.4GPa.

The NEFMAC grids used in this study are designated as C13, C16 and C19, with rib cross sectional areas of 65 mm^2 , 100 mm^2 and 148 mm^2 , respectively. All three grids are isotropic with 100 mm rib-spacing. For more information about NEFMAC (1200 MPa) properties, it is not expected that the reinforcement will fail in tension in the present tests.

Figure 3.7: CFRP ribs stress strain relationship as provided by the manufacturer

Steel Reinforcement

The steel reinforcement is used for reinforcing the slab of specimen ZJS, and all the column stubs. In the latter slab the reinforcing bars were #10 bars with a nominal yield strength of 400 MPa. The measured strain in the steel rods at the start of yielding during the test of Specimen ZJS was 0.0025. For $E_s = 200$ GPa, the actual yield stress would be $f_y = 500$ MPa.

Figure. 3.8: Cutting the carbon fibre grids for installation in the slab specimens

3.4 Specimens Construction

The construction process was carried out in the following sequence.

3.4.1 Formwork Preparation

The formwork consisted of three main parts. The slab mould was prepared of plywood sheets, supported by 2"x4" stringers, to limit the deflection of the slab during and after casting. The slab formwork was supported on four sides on wooden walls having the same height as the lower column stub. The formwork for the lower column stub was made of plywood sheets nailed to the slab formwork after attaching the column reinforcement to the slab reinforcement. The inner form surfaces were well prepared by applying form oil in three layers.

3.4.2 Preparation of Reinforcement Cages

The slab steel or CFRP grid reinforcement was assembled with the column stub steel reinforcement using tie wires. The concrete cover was maintained using 20mm high concrete chairs. The upper carbon grids were held in place by short vertical CFRP ribs tied to the bottom and top grids. The lower reinforcement was tied and held in position by steel wires passing through the plywood sheets. The purpose of tying the lower reinforcement was to prevent the CFRP grid from floating during casting. The upper and lower grids were connected together tightly by tying wires to prevent them from shifting from their position during casting and to maintain the correct top and bottom concrete cover. The column reinforcement was assembled separately from the slab reinforcement

whenever possible and then passed through the slab reinforcement from the top and tied to it. Subsequently, the lower column stirrups were placed and tied from underneath. The crack detector holes were located by placing vertically 5/8 inch diameter plastic tubes at the appropriate locations, with the tubes being tied to the formwork. The tubes were filled with Vaseline one day before casting the specimens (see details in Figures 3.9 through 3.11 for specimens reinforced with both CFRP and steel).

Finally, each specimen was fitted with four steel rebar hooks, each located near a corner of the slab on the top surface. These hooks were installed in order to facilitate the lifting and moving of the specimens.

Figure 3.9: Close up view of the CFRP grid and the applied strain gauges

Figure 3.10: Position of PVC tube for housing the vertical crack detection bar

Figure 3.11: Close up view of the steel reinforcement

3.4.4 Casting

The reinforcement was maintained aligned in the right positions before casting and the strain gauges wires were attached to the reinforcement and passed through the form. The upper column stub reinforcement was positioned and levelled in its correct position and then supported on a wooden frame connected to the slab formwork.

The concrete was delivered by the ready mix plant in a concrete mixing truck. The slump was checked before the start of casting and was found to be 100 mm. The concrete was carried from the truck to the test specimens using a metal bucket and the crane in the laboratory, Figure 3.12. The slab and the lower column stub were cast monolithically using two small mechanical vibrators. The upper slab surface was levelled and smoothly finished, Figure 3.13. The concrete was covered with a polyethylene sheet and allowed to cure. The upper column stub was cast a week after the casting of the slab (see Figure 3.14 and Figure 3.15)

Figure 3.12: Typical casting of the concrete in the lower column stub and the slab

Figure 3.13: Levelling and smoothing the top surface of the slab

Figure 3.14: Upper column stub formwork and reinforcement

Figure 3.15: Casting the upper column stub

3.4.5 Curing

The concrete surface was covered with a plastic sheet four hours after casting to reduce the water evaporation from the top surface of the slab. The slab surface was flooded with water and covered with the plastic sheet for four days. The concrete cylinders were ponded with water the morning after casting and then moved and left in the curing room for four days. The cylinders were then removed from their moulds and left exposed to the laboratory ambient conditions. The upper columns stubs were covered with plastic sheets two hours after their casting and left covered for three more days. Some typical specimens are shown in Figure 3.16 after stripping of the formwork.

Figure 3.16: Typical test specimens after removal of formwork

3.5 Instrumentation

The instrumentation used in this study consisted of electric resistance strain gauges to measure reinforcement strain, Demec gauges for measuring concrete surface strain, LVDT's to measure slab and column displacements and a load cell for measuring the applied load.

3.5.1 *Electrical Strain Gauges*

The steel reinforcement bars and the CFRP grid ribs strains on the tension and compression sides of the slab were monitored by electric resistance strain gauges. The strain gauges were attached at predetermined locations on the bars (Showa 5mm long strain gauges) before casting the concrete. The reinforcement surface was cleaned with CSM-1A degreaser and rubbed with M-Prep conditioner A and left to dry, then the surface was rubbed again with M-Prep Neutralizer 5A before installing the gauge. The adhesive used to attach the gauge was M-Bond 200. Similarly, some longer electric

strain gauges (Showa 30mm strain gauges for static loading) were applied to the concrete surface.

Table 3.5 shows the number of strain gauges applied to the top and bottom bars or grids in each specimen, while Figures 3.20 through 3.29 show the actual locations of these gauges as well as the direction in which strain is measured.

Table 3.5: Number of strain gauges applied to each specimen

Specimen Identification No.	Elect. Strain Gauges Fixed on Bottom Reinforcement (Slab Tension side)	Elect. Strain Gauges Fixed on Upper Reinforcement (SLAB Comp. Side)
ZJS	11	3
ZJF(1)	10	4
ZJF(2)	10	4
ZJF(3)	18	7
ZJF(4)	18	7
ZJF(5)	10	4
ZJF(6)	11	4
ZJF(7)	10	4

In the latter figures, the arrows indicate the direction of strain measurement and the number designates the gauge number. Notice that the bottom reinforcement was more extensively instrumented due its expected higher stresses. We also notice that some specimens were more extensively strain gauged than the others. This was done in order to gather more detailed data about the behaviour of at least some of the specimens, which may be needed during the analysis of the test results.

Fixing The Strain Gauges to the Steel Reinforcement of Specimen ZJS

The surface of the steel rebars was grounded and smoothened before installing the strain gauges at the required positions. The strain gauges were tested and checked thoroughly, using a strain gauge tester (see Figure 3.17). Afterwards the strain gauges were covered with a thin layer of wax, followed by a layer of crack seal, then both were fixed and sealed by electric wiring insulating tape, Figure 3.18. All the previous precautions were taken to protect the strain gauges from premature damage during the assembly of the reinforcement cages and during casting.

Figure 3.17: Strain gauges tester

Figure. 3.18: Strain gauges installation on a carbon fibre grid

Fixing The Strain Gauges to CFRP Reinforced Specimens

The surface of the CFRP grids was cleaned and then a thin layer of epoxy was applied to the surface at the selected positions to obtain a smooth clean surface. The thin layer of epoxy was later on cleaned before installing the strain gauges at the required positions. The strain gauges were tested and checked thoroughly. Afterwards they were covered with a thin layer of wax followed by a layer of crack seal. Finally both layers were fixed and sealed by electric wiring insulating tape.

3.5.2 Internal Crack Detection Bar

The full details of the internal crack detector bar and its instrumentations are shown in Figure 3.19. The bar was placed in a hole through the slab thickness and its ends were lightly bolted to the top and bottom surfaces of the slab as indicated in Figure 3.19. The

preparation of the bars location before and during casting process was discussed earlier in Section 3.4.1. Each detector bars was instrumented with an electrical resistance strain gauge to measure the vertical movements of the top and bottom surfaces of the slab after the formation of the diagonal cracks through the thickness of the slab.

Figure 3.19: Crack detection bar details

Figure 3.20: Strain gauges locations on the top reinforcement of ZJS

Figure. 3.21: Strain gauges locations on the bottom reinforcement of ZJS

Figure 3.22: Strain gauges locations on the top reinforcement of ZJF1, ZJF2, and ZJF7

Figure 3.23: Strain gauges locations on the bottom reinforcement of ZJF1, ZJF2, and ZJF7

Figure 3.24: Strain gauges locations on the top reinforcement of ZJF3 and ZJF4

Figure 3.25: Strain gauges locations on the bottom reinforcement of ZJF3 and ZJF4

Figure 3.26: Strain gauges locations on the top reinforcement of ZJF5

Figure 3.27: Strain gauges locations on the bottom reinforcement of ZJF5

Figure 3.28: Strain gauges locations on the top reinforcement of ZJF6

Figure 3.29: Strain gauges locations on the bottom reinforcement of ZJF6

3.5.3 Demec Points

Demec points were fixed on the top surface of the slab for mechanical measurement of concrete strain. The Demec gauge shown in Figure 3.30 was used and it was calibrated to have an accuracy of 0.002" over a 2 inch (52 mm) gauge length. The typical location of the Demec points on the top surface of test specimens ZJF(1) are shown in Figure 3.31, while the locations for the other specimens are shown in Figures B.1 through B.7 in Appendix B.

Figure. 3.30: Components of a Demec gauge device

Figure 3.31: Locations of Demec points on the top surface of the slab in specimen ZJF(1)

3.5.4 LVDT Locations

Linear variable differential transducers (LVDT) were used to measure slab deflections and column stub rotation. All the LVDT's used had a 6 inch (150mm) stroke. Figures 3.32 and 3.33 shows typical LVDT locations for specimen ZJS, while the LVDT locations for the other specimens are in Figures B.8 through B.21 in Appendix B. The photos in Figures 3.34 and 3.35 shows the LVDT's positions on the top or bottom slab surfaces and on the upper column stub.

Figure 3.32: The location of the LVDTs on the top surface of specimen ZJS.

Figure 3.33: The location of the LVDTs on the bottom surface of specimen ZJS

Figure 3.34: Typical LVDTs placed on the top surface of the slab

Figure 3.35: Typical locations of LVDTs placed under the slab

3.6 Test Set-up and Loading

Each tested specimen was supported along the periphery of its slab with the slab being horizontal and the column stub being vertical. The test frame consisted of four wide flange columns connected by two steel girders, (see Figure 3.37). The slab was supported on a horizontal frame resting on the two main I beam girders connecting the supporting columns. The slab rested on neoprene pads and steel strips. The neoprene strips were 2 inches wide and 3/8 inch thick; the steel strips were 50 mm wide and 10 mm thick. The distance between the centrelines of parallel steel strips represents the span of the slab and was equal to 1.50 m. The specimen corners were prevented from lifting up during loading, but not from in plane movement or rotation, by holding them down by steel Z sections made of two welded angles, (see the details in Figure 3.36). The preceding precaution guards against unwanted cracks near and around the corners and represents with a good degree of approximation the real behaviour of the portion of the slab between the column and the lines of contraflexure in the prototype flat plate structure. The photo in Figure 3.40 shows one specimen in the test bed ready for testing.

Figure 3.36: Close up view of the slab support at its edges

Figure 3.37: Test set-up elevation

Figure 3.38: Test set-up side view

Figure 3.39: Test set-up Plan

Figure 3.40: Test set-up

3.6.1 Test Sequence

The test procedure was basically the same for all the specimens and is briefly described below.

Positioning of the Specimen and Instrumentations

The following sequence was followed in each test:

- 1- The sample was placed in the loading frame, then accurately aligned in the test rig according to the specified eccentricity in the test program.
- 2- The LVDTs, for measuring the slab and the column stub displacements, were mounted in their required positions.
- 3- The actuator, the strain gauges and the LVDTs were connected to the designated channels on the data acquisition panel.
- 4- All the instrumentation was calibrated and their readings were set to zero before commencing the test. The test specimen was inspected for any cracks or other unexpected features prior to testing.

Loading

The load was applied by means of a single 400 kip actuator placed on the upper column stub. It was applied monotonically in increments of 5 kN during the pre-cracking stage, then followed by 10 kN increment during the service load level or 15kN for high capacity samples, and then reduced to increments of 5kN at higher loads till failure. The total load at each stage of loading was kept constant to allow for recording of the manual measurement of Demec strains, the visual observations and the marking and measurement of cracks. In each test due to the constant eccentricity of the axial load from the column centre, the ratio of the moment to the shear was held constant.

Note, however, that during the early part of the loading, normally just past the cracking load, the specimen was unloaded and then reloaded. Since in most cases a specimen could not be fully tested to failure in one day, it had to be unloaded at the end of the first day and then reloaded the following day.

Figure 3.41: The wiring connected to the data acquisition system from the back

Figure 3.42: Microscope for measuring crack width

Chapter (4)

Experimental Results and Discussion

4.1 General

This chapter presents the more important data gathered during the test program and a discussion of the results. As mentioned in the previous chapter, slab and column deflections, and reinforcement and concrete strains were measured throughout the loading regime. In addition, crack width and propagation around the column were monitored and the separation of the top and bottom surfaces of the slab caused by the formation of through-thickness diagonal cracks, which form the failure cone, was detected using the crack detection bar placed in the slab.

Here the test results are presented and the observed behaviour of the test specimens is discussed. Where appropriate reference is made to available research results to point out the similarity or differences that may exist with the current results

4.2 Remarks about the test method

The first test was performed on specimen ZJF5. Concentric load was applied through the centre of the column stub. The test was performed using load control in increments of 5 kN. During testing, the specimen, the loading frame and the supporting frame showed good stability and rigidity.

The instrumentation functioned well and nearly all the channels of the data acquisition recorded the expected data. Although the first test was a success in many ways, it also revealed an important deficiency. The load would drop whenever loading was stopped to mark the cracks or to measure and record the Demec gauges readings. The drop in the load was due to the deformations and rotation of the slab, caused most likely by creep and other movements in the specimen. A drawback of a load control system is that one is not able to record the specimen post-peak load behaviour because it fails abruptly once the peak load is reached. Consequently, it was decided to use displacement control instead of load control in all the remaining tests.

4.3 Failure mode and criterion

Specimen ZJF5, apart from the above-mentioned drawbacks, showed promising results and failed due to formation of internal diagonal cracks in the most stressed slab region, which extended from the compression side towards the tension side, forming the classical truncated cone (see Figures 2.1, 4.2(a) and (b)). While, all the other specimens failures were mainly due to punching shear at the most stressed region, followed by torsional type failure of slab on the two parallel sides of the column stub (see Figure 4.1); however, none of the tested specimens failed in pure flexure. The rough surface of the NEFMAC carbon fibre grids and the close spacing of the grid likely prevented the secondary effect of slippage along the reinforcing FRP bars on the failure, a phenomenon witnessed by El-Ghandour et al. (1997) and (1998) in their tests.

The strain distributions along the carbon fibre reinforcing bars and concrete surface strains in all the specimens revealed gradual transfer of tensile forces from concrete to the

carbon fibre due to adequate adhesive bonding and mechanical anchorage of the bars by the cross ribs (see figures 4.45 through 4.52). Failure in the current study is based on a noticeable drop in load with concurrent and large increase in the vertical displacements of the slab. The strength of the slab is based on the maximum load while its ductility or deformability may be based on some defined level of load in relation to the peak load. For instance, one could assume that when the load drops to 80% of the peak load on the descending branch of the load-deflection curve, then the specimen has reached its useful limit. However, in this thesis the issue of ductility is not the focus of the investigation.

It is also important to mention that final failure of the test specimens were accompanied by extensive and wide cracks and by the visible movement of the truncated concrete cone surrounding the column, accompanied by the spalling of the concrete cover in the most stressed region of the slab.

4.4 Behaviour of slab column connections.

Seven CFRP reinforced specimens were tested. Only Specimen ZJF5 was tested concentrically subjected to pure shear while all the other specimens were subjected to combined shear and moment.

Pure shear punching is a limiting case and its results could be compared with results of concentrically loaded test specimens tested by other researchers. The results could also be used to check the applicability of their proposed equations for calculating the strength of FRP reinforced interior slab-column connections.

Six CFRP specimens were tested under the effect of eccentric loading i.e. shear and moment. The specimens differed in their properties in order to determine the

influence of some important parameters on the punching shear strength of interior slab-column connections in flat plate structures.

The specimens were divided into six groups, each representing one major variable and its effect on the strength and behaviour of the connection (see Table 3.2). The actual data collected during the test were enormous, but due to the similarity of the results, detailed results for only four specimens will be presented and these are deemed to be typical of the results collected for the other specimens in the same group. In particular, we will focus on specimens ZJS, ZJF1, ZJF3 and ZJF5.

4.4.1 Crack Pattern

Specimens ZJF1, ZJF3 and ZJF5

These specimens were designed to study the effect of the M/V ratio. Specimen ZJF5 was tested under concentric load while specimens ZJF1 and ZJF3 were tested under eccentric load, with eccentricities of 0.22 m and 0.30 m, respectively. They each had a 100 mm thick slab and were all reinforced with 1.0% C16 NEFMAC grids.

Specimen ZJF5

Specimen ZJF5 was the first specimen to be tested and it was loaded on two consecutive days. On the first day, the maximum load reached was 200 kN. On the following day, it was reloaded until failure. The Demec readings were taken on ten different occasions; eight before unloading and two after reloading. The first visible crack appeared between 50 kN and 55 kN.

With reference to Figures 4.2(a) to (c), which show the crack pattern of specimen ZJF5, the first measured crack width was 0.04 mm. At 60 kN, radial cracks perpendicular to the column faces, and moving towards the support in the north and south directions, appeared, Figure 4.2 (b). At 65 kN similar short radial cracks appeared perpendicular to the south and east slab-column interfaces. At the same time, the first planar diagonal crack started to develop and was well defined at 70 kN, at which load level additional radial cracks started from some corners. At this load the diagonal cracks maximum width was 0.14 mm and that of the radial cracks was 0.10 mm. At 80 kN and 86 kN, other diagonal cracks began to appear. At 90 kN primary diagonal cracks had a maximum width of 0.18 mm. At 95 kN the main crack in the northwest direction formed and the number of diagonal cracks increased sharply at different locations (see the crack pattern in Figure 4.26). At 105 kN the radial crack width was 0.26 mm. At 110 kN many tangential cracks appeared around the column stub at various locations and at 140 kN tangential cracks started to widen noticeably. At 150 kN the maximum diagonal crack was 0.29 mm wide. At 175 kN the maximum width of the radial cracks and tangential cracks were 0.36mm, and 0.5mm respectively. The cracks were marked up to 150 kN only because large cracks formed at the south column-slab interface thereafter and crack width beyond this load level would have little practical significance.

The specimen was unloaded at 200kN and before unloading, the crack detection bar showed tension strain from 185 kN load onward. The diagonal crack width right after unloading diminished to 0.10 mm. Upon full unloading, the slab had permanent maximum deflection of 4.4 mm, which was 28% of the maximum deflection, obtained before unloading (15.8 mm).

The loading was resumed the following day in increments of 5 kN. At 205 kN and beyond clicking sounds could be heard. The crack detector near the south face of the column failed at 227 kN, making a loud noise, followed by the diagonal crack detector failure at 232 kN. The failure surface started to develop parallel to the south face of the column at approximately 228 kN. At 234 kN sudden brittle failure occurred around the column with the formation of the typical truncated cone in the slab. The lower concrete surface separated from the rest of the slab by 15 mm to 25 mm. Pieces of concrete fell to the ground due to the penetration of the column and the separation of the truncated cone. Figure 4.3(a) shows the cup and cone fracture of the internal crack detection bar on the south face. Figure 4.3(b) (shows the detection bar that was placed along the diagonal); it was deformed because it crossed the fracture line in the slab.

Specimen ZJF3

This specimen was one of the test specimens that was instrumented heavily for measuring concrete and steel strains. The obtained data were intended to help in understanding the punching shear behaviour and failure mode of CFRP reinforced slab-column connections in detail and for comparison with the known behaviour of connections reinforced with ordinary rebars.

The average compressive strength of concrete cylinders at the time of testing the specimen was 47 MPa. The specimen was tested on two consecutive days because the test could not be finished during one working day. It was unloaded after reaching a load of 40 kN on the first day, and then loading of the specimen was resumed the next day.

The load was applied under displacement control with 5 kN load increments until 50 kN, just above the cracking load, and was then increased to 10 kN.

Figure 4.4(a) to (c) show the crack pattern of specimen ZJF3. The first cracks appeared at 15 kN at the south slab-column interface and propagated a short distance from the two corners of the column at that interface. At 20 kN the cracks spread along the entire length of the most stressed tension side of the slab-column interface. At 25 kN, the cracks extended around the east side of the of the slab-column interface and they started to appear at the west slab-column interface. Two nearly fully developed radial cracks appeared at 30 kN, one crack started from the middle of the west slab-column interface and moved away from the column in a westerly direction and reached the support at 40 kN. The other radial crack first appeared perpendicular to the south slab-column interface near its southeast corner, see Figure 4.4(a), then it changed direction, making an angle of approximately 25° with the south direction, and finally it became parallel to the diagonal, making an angle of about 50° with the south direction. At 70 kN the latter crack reached the supports, thus forming the first full diagonal crack. The radial crack maximum width at 30 kN was 0.10 mm.

At 35 kN an inclined crack (torsional crack) appeared on the west side of the column stub. Another torsional crack in the northeast quarter of the slab appeared at 45kN and reached the support at 55kN. More diagonal cracks appeared at 61kN, one in the southwest and another in the southwest quarter of the slab. The formation of diagonal cracks and torsional cracks continued up to 115kN or 84% of the ultimate load. Tangential cracks developed at 40 and 70kN, but the major tangential cracks next to the

most stressed slab-column interface (south region) started at 85 kN and 100 kN, i.e. at about 63% and 74% of the ultimate load.

It is noteworthy that the behaviour of the specimens reinforced with CFRP grids resembles the behaviour of the specimen reinforced with steel insofar as the formation of cracks and overall deformation pattern are concerned. They undergo large deformations and exhibit a crack pattern akin to the yieldline pattern in steel reinforced connections despite the lack of a yield point in CFRP reinforcement. see (Demec Diagonal cracks).

Specimen ZJF1

This specimen had an eccentricity of 0.22 m. The concrete average strength at the time of testing was 45 MPa. The slab was tested on two consecutive days, the first day it was loaded up to 89 kN and then unloaded. Testing was resumed the next day and continued until failure at 171.0 kN.

Figures 4.5(a) to (d) show the crack pattern and failure mode of specimen ZJF1. The crack pattern of this specimen is similar to that of ZJF3. The first visible crack appeared at 28 kN at the south slab-column interface. The next crack in the east slab column-interface developed between 40 kN and 60 kN. The crack along the west slab column-interface formed between 45 kN and 60 kN while the first radial crack developed at the south-slab column interface at 28 kN and propagated towards the support. This crack was located essentially under a carbon fibre grid rib near the south face of the column. Another radial crack formed at the south slab-column interface at 35 kN and immediately reached the support. This crack also followed one of the carbon fibre ribs passing close to the south column face. At 65 kN a flexural crack parallel to the south column face

appeared. It started from the middle of the south face and propagated in a westerly direction. The maximum crack width was 0.04mm. The southeast diagonal cracks began to form at 60 kN and continued their development up to 85 kN, thereafter only small branches formed but the crack widths continued to increase up to failure.

The behaviour of this specimen reinforced with CFRP grid resembled the behaviour of the specimens reinforced with traditional steel with respect to formation of cracks, and plastic rotation along lines similar to the yield lines in R.C. slabs, despite lack of a yield point in the CFRP grids. Of course, the yield lines were not fully developed because the connection failed in punching shear long before reaching its flexural capacity. The latter behaviour might make it possible to apply yield line theory, with certain modifications to account for the properties of CFRP reinforcement, in order to predict the flexural capacity of CFRP reinforced slabs

The first torsional crack appeared at 35 kN on the east side of the column. The crack extended from the column south corner, its maximum width being 0.08 mm, making an angle of 45° with the north. The first torsional crack on the west face was noticed at 45 kN. The latter crack did not start at the column stub, instead it started from the position of the outer bar parallel to the column west face and kept on moving towards the supports, with an inclined angle of approximately 45° from the north and the south directions. At 35 kN, the crack at the south column-slab interface was 0.04 mm wide.

Specimen ZJS

The concrete average compressive strength at the time of testing ZJS was 46 MPa. The slab was tested on two consecutive days; the first day it was loaded up to 60 kN and

then unloaded. On the second day testing continued up to failure at 220 kN. The Demec readings were taken at eight different loading stages; two stages before cracking and the other six after cracking.

Figure 4.6(a) and (b) show the crack pattern and failure zone of specimen ZJS. The first observed crack appeared at 40 kN at the south slab-column interface. This crack rotated around the southeast corner at 45 kN and reached the mid point of the east slab column interface at 65 kN, (see the column-stub side in Figure 4.6b). The first main crack appeared at 65 kN on the slab bottom surface and it was a flexural crack parallel to the south slab-column interface (most stressed). It started from the middle of the south face and extend equivalent to one full grid spacing towards the west direction. The main crack width was 0.04 mm. Another radial short crack occurred at the same load level; it started perpendicular to the most stressed slab-column interface at distance equal to quarter the column side length from the column south east corner. At 70 kN, the first main crack extended to a distance equal to column side length in the east direction. Many other cracks developed also at the same load level. A diagonal crack started at the column southeast corner and moved about 250mm towards the south, making an angle of 60° with the south direction. At 140 kN it eventually reached the east support, making an angle of 75° with the south direction. The previous diagonal crack width was 0.02 mm at 70 kN. Another diagonal crack occurred at 70 kN, making an angle of 45° with the south direction. This crack reached the south support at 90 kN and joined the radial cracks crossing the column east face at 180 kN.

The main torsional cracks were on the west and east sides of the column. The east torsional crack started perpendicular to the east face of the column for a distance of 90

mm, and then changed direction, making an angle of 15° with the east. The previous crack continued to propagate at 95 kN and at 105 kN reached the support.

The main torsional crack in the westerly direction appeared at 70 kN load, starting at 70 mm from the southwest corner of the column and extending with an inclination of 60° from the north over two bar spacing, and reaching the support at 100 kN. This crack began to widen very noticeably between 90 kN and 100 kN.

The main tangential crack occurred at 85kN; it appeared to be an extension of a short radial crack that occurred at the same load level. This radial crack emanated from the column southeast corner, and the tangential crack was at the middle of this crack, extending over the second south reinforcing bar from the column face and passing the southwest corner of the column by one bar spacing. It then split into two branches, the first branch moved towards the southwest with inclination of 45° from the west. The crack travelled two bar spacing at 85 kN, and then moved towards the southwest support at 120 kN. The other branch made an angle of 30° with the west direction, stretching two bar spacings, and then moving towards the west support.

At 90 kN a partial diagonal crack appeared in the southwest quadrant while a fully developed diagonal crack was seen in the northwest quadrant of the slab. It started at the column and headed towards the west support, with an inclination of 45° with respect to the west. The main northeast diagonal started from the east side of the column stub at a distance of roughly 70 mm. from the column northeast corner.

The average crack width at 105 kN was 0.09 mm. AT 170 kN the crack around the column-slab interface appeared to have widened substantially as shown in Figure 4.6(b). The number of diagonal cracks kept increasing up to 150 kN. The maximum crack width

at 150kN was 0.10mm. A few secondary diagonal cracks developed later at 175kN but the existing cracks continued to widen up to failure.

. The radial cracks in the north and south column strips did not reach the supports in contrast to the CFRP reinforced slabs, but the cracks parallel to and in the east and west column strips did reach the supports. Most of the diagonal cracks started developing from lines parallel to the column sides.

The last outer tangential crack was noticed between 130 and 135kN. The tangential cracks in the steel specimen did not happen under the reinforcing bars at the beginning and some cracks appeared at 186 kN, 85% of failure load, between existing tangential cracks close to the column.

Figure 4.1: Typical expected crack pattern of Specimen due to eccentric punching load

Figure 4.2a: Crack pattern of Specimen ZJF5 before marking

Figure 4.2b: Crack pattern of Specimen ZJF5 after marking

Figure 4.2c: The tangential and column slab interface cracks at the most stressed region in Specimen ZJF5

Figure 4.3a: Cup and cone failure

Figure 4.3b: Deformation of diagonal crack detector

Figure 4.4a: Crack pattern of Specimen ZJF3

Figure 4.4b: Torsion cracks on the west column slab interface in Specimen ZJF3

Figure 4.4c: The southeast corner of Specimen ZJF3 at failure

Figure 4.4d: Permanent deformation of column stub after unloading of Specimen ZJF3

Figure 4.5a: Crack pattern of Specimen ZJF1

Figure 4.5b: Tangential cracks at the column-slab interface in Specimen ZJF1

Figure 4.5c: Failure of Specimen ZJF1 as seen from the east side

Figure 4.5d: Failure of Specimen ZJF1 as seen from the southwest corner

Figure 4.6a: Crack pattern of Specimen ZJS, reinforced with conventional steel

Figure 4.6b: Close view of the column slab interface cracks of Specimen ZJS

Figure 4.7a: Crack pattern of Specimen ZJF4

Figure 4.7b: Punching underneath the column stub cantilever on the west side.

Figure 4.8a: Crack pattern of Specimen ZJF2

Figure 4.8b: A close view of the southeast corner of Specimen ZJF2 at failure.

Figure 4.9a: Crack pattern of Specimen ZJF6

Figure 4.9b: Tangential and column slab interface cracks in Specimen ZJF6

Figure 4.9c: widening of tangential cracks in the most stressed face of specimen ZJF6 at failure.

Figure 4.9d: The southeast part of specimen ZJF6 at failure

Figure 4.9e: Punching of Specimen ZJF6 (southwest corner)

Figure 4.9f: Top view of the punched slab showing the west column-slab interface.

Figure 4.10a: Crack pattern of Specimen ZJF7

Figure 4.10b: Top view of the punching of the south and west sides of the column in Specimen ZJF7.

Figure 4.10c: Rotation of the upper column stub during punching of Specimen ZJF7

4.4.2 Crack width

Table 4.1 gives the maximum crack width for each specimen at various levels of loading. We notice that the crack widths were not measured at exactly the same load level for all the specimens, and measurement could not be taken at the slab column interfaces. Note, however that none of the crack widths exceeds 0.4 mm, which is the maximum permissible crack width in the current CSA code. Table 4.2 gives the maximum crack width for each specimen at a load level close to 50% of its failure load, would be close to its serviceability load. The crack widths of Specimens ZJF5, ZJF6, and ZJF7 in this table were obtained at 50% of ultimate load by interpolation of the values given in Table 4.1.

The results in Table 4.1 and 4.2 show that the crack widths are within the acceptable limits. As mentioned by Pillai and Kirk (1988), the current CSA crack control parameter corresponds to a maximum crack width of 0.33 mm for exterior exposure and 0.40 mm for interior exposure. The CFRP reinforced specimens had maximum crack widths that were significantly larger than the maximum crack width in the steel reinforced specimen, but practically all the crack widths at approximately 50% of the failure load are in the allowable crack width range.

Table 4.1: Maximum crack width (mm) in test specimens at various load levels

Load kN	Specimen							
	ZJF1	ZJF2	ZJF3	ZJF4	ZJF5	ZJF6	ZJF7	ZJS
30	0.10		0.10		0.04	0.06		0.04
55	0.14	0.12	0.28		0.14			
65								
70	0.22	0.36 ^{Cl} , 0.20 ^D						
71								
75	0.24		0.34					
85					0.18	0.18	0.14	0.09
90	0.26				0.26			
95								
105				0.20 ^{Tg}				
125				0.30 ^T				
130								
135	0.40							
140				0.32 ^T				
145								
150					0.29	0.22	0.22 ^T	0.10
170					0.36 ^R , 0.5 ^T		0.30 ^T	
190								

Cl : Column-slab interface crack

Tg : Tangential cracks

D : Diagonal radial cracks

T : Torsional flexural cracks

R : Radial cracks.

Table 4.2: Test specimens grouped to follow investigated parameters and the test results

Studied parameter	Specimen No.	Column Stub mm x mm	Slab Thickness mm	Effect. Depth d mm	Tension reinf. ρ	M/V Ratio m	f_c MPa	V_{TEST} kN	Crack Width mm	Load [#] $\frac{V_{TEST}}{V_{TEST}}$
Type of Reinforcement	ZJS	250x250	100	75	1.0%	0.22	46.0	218.2	0.09	0.48
	ZJ1	250x250	100	74	1.0%	0.22	44.7	170.0	0.26	0.52
Bott & top reinf. Ratio ρ	ZJ1	250x250	100	74	1.0%	0.22	44.7	171.0	0.26	0.52
	ZJ2	250x250	100	74	0.65%	0.22	42.1	144.2	0.36	0.492
Col. Side / eff.d = (c/d) Ratio	ZJ1	250x250	100	74	1.0%	0.22	44.7	170.0	0.26	0.52
	ZJ4	250x250	125	100	1.184%	0.22	47.5	250.1	0.20	0.50
Col. Asp. Ratio c_1/c_2	ZJ4	250x250	125	100	1.184%	0.22	47.5	250.1	0.30	0.52
	ZJ6	250x350	125	100	1.0%	0.22	47.9	235.0	0.25 ^I	0.50
M/V	ZJ5	250x250	100	74	1.0%	0.00	44.4	234.0	0.27 ^I	0.5
	ZJ1	250x250	100	74	1.0%	0.22	44.7	170.0	0.26	0.5
	ZJ3	250x250	100	74	1.0%	0.30	47.1	134.3	0.29	0.5
Presence of Shear Reinf.	ZJ4	250x250	125	100	1.184%	0.22	47.5	250.1	0.30	0.52
	ZJ7*	250x250	125	100	1.184%	0.22	45.7	317.7	0.24 ^I	0.50

*E shear reinforcement is provided

I: Value is interpolated from Table 4.1

crack width in this table correspond to the indicated load levels

4.4.3 Effect of Some Parameters on the Crack Density, Size and Punching Perimeter.

Reinforcement Type (Specimens ZJF1 and ZJS)

The crack patterns of Specimens ZJF1 and ZJS, given in Figure 4.5(a) and 4.6(a), respectively, and as discussed in Section 4.4.1, are similar, except the cracks density of the CFRP reinforced Specimen ZJF1 is higher than that of the steel reinforced Specimen ZJS. Also the figures show that the failure perimeter of ZJS is wider than that of ZJF1. Tables 4.1 and 4.2 showed that the crack width in Specimen ZJF1 is 2.8 times wider than that of ZJS..

Slab thickness (Specimens ZJF1 and ZJF4)

Figures 4.5(a) and 4.7(a) shows that the crack patterns of specimens ZJF1 and ZJF4 are basically similar except that ZJF4 has a larger number of closely spaced cracks. Also the surface punching perimeter of ZJF4 is much larger than that of ZJF1. Table 4.2 showed that the cracks in ZJF4 are narrower at same percentage of the ultimate load. Only in ZJF4 the measured torsional crack width was wider, while all the other measured crack widths of ZJF4 at the same load level were narrower than those of ZJF1.

Reinforcement ratio (Specimens ZJF1 and ZJF2)

The crack patterns of Specimens ZJF1 and ZJF2 are given in Figure 4.5(a) and 4.8(a) and are similar, but the cracks in ZJF1 have higher density than in ZJF1 and smaller width. The smaller width is expected due to the higher reinforcement ratio. Also the perimeter of

punched area was smaller for ZJF1 compared to ZJF2.

Column aspect ratio (Specimens ZJF4 and ZJF6)

Figures 4.7(a) and 4.9(a) show that the ZJF6 crack pattern is similar but much denser than that of ZJF4. Table 4.1 and 4.2 show that ZJF6 has narrower cracks by 20% due to the increase in the aspect ratio of the column.

M/V ratio (Specimens ZJF1, ZJF3 and ZJF5)

The crack patterns of specimens ZJF1, ZJF3, and ZJF5 are given in Figure, 4.5(a) 4.4(a) and 4.2(b) respectively. The crack patterns for these specimens were discussed in detail in Section 4.4.1; the crack patterns of ZJF5 is symmetric around both axis of the specimens and is different than the crack pattern of the other two specimens, while the crack pattern of ZJF3 is similar to that of ZJF1 but its cracks are more closely spaced on torsional sides and most stressed region of the slab. Tables 4.1 and 4.2 indicate that the crack width slightly increases with increase of the M/V ratio. This is expected because the increase in moment will result in increased crack width.

Shear reinforcement (Specimens ZJF4 and ZJF7)

Figures 4.7(a) and 4.10(a), show that the crack patterns of these specimens are similar, but the crack density is higher for Specimen ZJF7. Table 4.2 shows that the maximum crack width in ZJF7 was much smaller than in ZJF4 at comparable load levels. The results show the benefits of the shear reinforcement even at the serviceability load levels. The failure perimeter of ZJF7 is smaller than that of ZJF4. It appears that the punched area occurred after the shear reinforcement cover and bond with concrete was lost near the column.

4.5 Load Deflection Relationships

The load-deflection curves of the test specimens are plotted in relation to the test parameters for each group of specimens (see Table 4.2 for specimens grouping). Deflections at three locations were measured and they are plotted against load as follows:

- i) Load-central deflection of the slab as measured by the LVDT'S placed underneath the centre of the column stub. The purpose of this measurement was investigation of the overall stiffness of the specimen.
- ii) Deflection distribution along the centre line of the slab in the plane of moment. These deflection readings were meant to study the deformations of the slab, which provide valuable information regarding the mode of failure of a test specimen.
- iii) Deflection measurements in the maximum shear stress region of the slab close to the slab-column interface. These measurements were taken at two points, at distance of 50 mm and 200 mm from the slab column interface. The deflections here are measured on the top surface of the slab and directly beneath it on the bottom in order to detect the formation and movements of the internal crack (diagonal cracks) inside the slab. The difference between the readings of the two LVDT's is supposed to indicate the separation of the top and bottom surfaces of the slab due to the movement of the punching cone.

Figure 4.11, shows the load-deflection relationships for the central points of all the specimens and are approximately bilinear up to the peak load, We notice that they all have essentially the same basic shape, irrespective of the variable involved. For all the

test specimens, the pre-cracking behaviour and stiffness are similar, but the post-cracking behaviour in the CFRP reinforced specimens differ from that of the steel reinforced specimen in that their stiffness is generally lower due to the lower elastic modulus of the CFRP reinforcement slabs. However, slabs with larger thickness and the slab with shear reinforcement have higher stiffness than the steel reinforced specimen.

It is interesting to note that the post-peak load response of the FRP reinforced specimens exhibit a long plateau without significant drop in the load. This means that these connections will give ample warning of impending failure and the failure mode would be akin to a ductile failure. In fact the unloaded specimens after failure exhibit significant permanent deformation and therefore one may claim that the failure is ductile. Insofar as ductility is concerned, it is difficult to differentiate between the behaviour of the steel reinforced specimen (ZJS) and the FRP reinforced specimens.

Effect of the Test Parameters on the Load-Central Deflection Response of the Test Specimens

Figure 4.12 shows the effect of increased M/V ratio on the overall response of the test specimens. Specimens ZJF1, ZJF3 and ZJF5 have the same dimensions and reinforcement but their M/V ratios are 0.22 m, 0.30 m, and 0.0 m, respectively. It is clear that an increase in the M/V ratio has a noticeable effect on the ultimate strength of a connection but its effect on the stiffness is not as significant.

The load deflection curves of the three specimens have basically the same slope after cracking which would indicate that their post-cracking stiffnesses are essentially the same. On the other hand, the presence of the moment decreases the cracking load.

However, note that a drop of 50% in the stiffness of ZJF1, compared to ZJS, reduced the ultimate strength by only 27%. Hence, the relation between stiffness and strength is not linear. As for the post peak response of these specimens, it would appear that the type of reinforcement has no significant effect on it.

The effect of column side c to slab effective depth d , c/d ratio, is demonstrated in Figure 4.14 for Specimens ZJF1 and ZJF4 where the two slabs have same size of column stub and the same reinforcement ratio, $\rho = 1.0$, but Specimen ZJF4 has an effective depth $d=100$ mm compared to 75 mm for ZJF1. We observe that the stiffness of Specimen ZJF1 is smaller compared to that of ZJF4. For FRP reinforced connections to fulfil the serviceability criteria in bending and to have behaviour comparable to a steel reinforced slab, one may have to either increase the reinforcement ratio or increase the depth of the slab. Generally increase in the depth of the slab may be more advantageous.

The effect of reinforcement ratio is shown in Figure 4.15. Here Specimen ZJF1 and ZJF2 have reinforcement ratios of 1.0% to 0.65%, respectively. Surprisingly, the reinforcement ratio does not have a very large effect on the stiffness or the strength, but its effect is more noticeable on the post-peak ductility of the connections.

Figure 4.16 shows the effect of column aspect ratio on the connection behaviour and strength. Changing the c_1/c_2 ratio from 1.0 for ZJF4 to 1.4 for ZJF6 had negligible effect on the either the strength or the stiffness of these specimens. At least for the current test specimens, this is not an important parameter.

Finally Figure 4.17 shows the effect of shear reinforcement on the behaviour of the connection. Specimen ZJF7 contained vertical CFRP rails, as shear reinforcement

and ZJF2 have reinforcement ratios of 1.0% to 0.65%, respectively. Surprisingly, the reinforcement ratio does not have a very large effect on the stiffness or the strength, but its effect is more noticeable on the post-peak ductility of the connections.

Figure 4.16 shows the effect of column aspect ratio on the connection behaviour and strength. Changing the c_1/c_2 ratio from 1.0 for ZJF4 to 1.4 for ZJF6 had negligible effect on either the strength or the stiffness of these specimens. At least for the current test specimens, this is not an important parameter.

Finally Figure 4.17 shows the effect of shear reinforcement on the behaviour of the connection. Specimen ZJF7 contained vertical CFRP rails, as shear reinforcement while its companion specimen ZJF4 did not. The effect of the latter reinforcement on the strength of the connection is remarkable. It increased the strength of ZJF7 by nearly 30% compared to that of ZJF4, but had no effect on its stiffness. Since the shear reinforcement will become effective after the formation of the internal diagonal cracks, in the slab, its lack of influence on the stiffness of the member is not unexpected.

Figure 4.11: Load-central deflection of all the specimens

Figure 4.12: Load-central deflection of specimens ZJF1,ZJF3, and ZJF5

Figure 4.13: Load-central deflection of specimens ZJS and ZJF1

Figure 4.14: Load-central deflection of specimens ZJF1 and ZJF4

Figure 4.15: Load-central deflection of specimens ZJF1 and ZJF2

Figure 4.16: Load-central deflection of specimens ZJF4 and ZJF6

Figure 4.17: Load-central deflection of specimens ZJF4 and ZJF7

Deflected Shape of the Slab

The deflected shapes of the slabs in the direction of bending, along their centreline passing through the column stub, are shown in Figures 4.18 through 4.25. From these graphs, it can be noticed that the tension due to bending along this line is always at the bottom of the slab and no negative moment exists between the column stub and the north support (see the insert in each figure), which is the less stressed region of the slab.

Notice that on the side of the connection where the shear stresses due to direct shear and moment transfer add up, that is the most stressed region in shear. the deflections are higher than on the opposite side where the two effects oppose each other. The single curvature deflected shapes also indicate that the axial load rather than the moment dominates the slab deflection. This can be seen clearly by comparing the deflected shape of the concentrically loaded specimen ZJF5 in Fig. 4.18 with the deflected shape of the eccentrically loaded specimens. Figure 4.18 exhibits symmetry while the other figures (Figures 4.19 through 4.25) show a small asymmetry due to the effect of the applied moment.

Diagonal Crack Movements

As indicated earlier, to track the formation and movements of the diagonal cracks in the slab and of the failure cone; LVDT's were installed on the top and bottom surfaces of the slab at the same distance from the column stub.

Figure 4.18: Deflection of the north-South centreline of ZJF5

Figure 4.19: Deflection of the north-south centreline of ZJF1

Figure 4.20: Deflection of the north-south centreline of ZJF3

Figure 4.21: Deflection of the north-south centreline of ZJS

Figure 4.22: Deflection of the north-south centreline of ZJF4

Figure 4.23: Deflection of the north-south centreline of ZJF2

Figure 4.24: Deflection of the north-south centreline of ZJF6

Figure 4.25: Deflection of the north-south centreline of ZJF7

Typical vertical displacement measurements by these LVDT'S, close to the most stressed (in shear) region of the slab, are presented in Figure 4.26 and 4.27 for specimens

ZJF1 and ZJS, respectively. In each case figure (a) is for points at 50 mm from the slab column interface while figure (b) is for points at 200 mm from the same interface. We notice that there is practically little difference between the displacements of the two points until almost failure. Close to failure significant relative deformation can be observed, which indicates the punching of the failure cone through the slab. The small relative displacements prior to failure may be ascribed to the movements of the diagonal cracks. Note, however, that the latter movements are larger for the CFRP reinforcement specimen ZJF1 compared to the steel reinforced specimen ZJS. Obviously, due to higher elastic modulus of steel, the diagonal cracks opening and sliding would be smaller. The same data for the remaining specimens are given in Appendix C.

Figure 4.26a: Load versus deflections of the top and bottom surfaces of Specimen ZJF1 at 50 mm from the south face of the column

Figure 4.26b: Load versus deflections of top and bottom concrete surfaces of Specimen ZJF1 at 200mm from the column south face

Figure 4.27a: Load versus deflections of the top and bottom surfaces of Specimen ZJS at 50 mm from the south face of the column

Figure 4.27b: Load versus deflections of top and bottom concrete surfaces of Specimen ZJS at 200mm from the column south face

Through-Thickness Deformations of the Slab

Crack detection bars were used to monitor the internal or through-thickness deformations of the slab. These deformations are expected to increase noticeably upon the formation and subsequent propagation of the diagonal cracks through the slab thickness. Figure 4.28 through 4.35 show the axial strain variation with load in the crack detection bars for all the test specimens. These figures indicate that up to at least 80% of the failure load, the through thickness deformations of the slab are negligible; there after, as indicated by the gradual increase in the strains, the deformations begin to increase. Significant strain is measured at failure. For the steel reinforced specimen, the strain gauge failed before measurements could be taken.

These results confirm the findings of some other researchers, Kruger et al (2000), who have stated that the diagonal cracks in the slab form at 80 to 90% of the punching load.

Figure 4.28: Through thickness slab strain in Specimen ZJS

Figure 4.29: Through thickness slab strain in Specimen ZJF1

Figure 4.30: Through thickness slab strain in Specimen ZJF2

Figure 4.31: Through thickness slab strain in Specimen ZJF3

Figure 4.32: Through thickness slab strain in Specimen ZJF4

Figure 4.33: Through thickness slab strain in Specimen ZJF5

Figure 4.34: Through thickness slab strain in Specimen ZJF6

Figure 4.35: Through thickness slab strain in Specimen ZJF7

Column stub Rotation

Figure 4.36 shows the column stub rotation versus the load for all test specimens. These diagrams closely resemble the load versus slab central deflection diagrams shown earlier and once again confirm the relatively lower stiffness of FRP reinforced connections of course, for the specimens with the thicker slabs their rotation are smaller compared to steel reinforced specimen.

Figure 4.36: Moment-Column stub rotation relationship

Slab Reinforcement Strains

Strain was measured at certain points along the reinforcement bars of specimen ZJS or the CFRP NEFMAC grid. These strains are plotted in Figures 4.37 to 4.52 for different locations and for each of the test specimens as indicated in the same figures. We notice that the FRP reinforcement experienced 5000 to 7000 microstrain at failure. We also observe that the maximum strain perpendicular to the plane of bending generally occur at the column centre and it gradually decreases as one moves away from the column centre. The only exception to this trend occurs in specimen ZJF3 in Figure 4.40, where the maximum strain occurs at 100 mm from the column centre. This specimen had a high moment to shear ratio of 0.3, but it is not clear whether this parameter is responsible for the shift in the location of the maximum stress in the reinforcement.

It is important to observe that despite the high failure load of specimen ZJF7 with the FRP shear reinforcement, the maximum strain in its tension reinforcement reached only approximately 4700 microstrain. Hence the shear reinforcement decreases the demand on the slab reinforcement to resist the punching shear. The relatively high strains in the CFRP reinforcement indicate that we cannot limit the maximum FRP strain to 0.002, which is generally the yield strain of steel reinforcement. While the advent of the yield in the steel is accompanied by total loss of steel stiffness, the stiffness of FRP reinforcement remains constant up to rupture.

Figures 4.45 to 4.52 show the strain distribution in the bar or grid rib closest to the centreline of the column section and parallel to the plane of bending. The strain is measured in the bottom or tension reinforcement of the slab at the locations indicated in the figures. As expected these strains are greatest at the slab-column interface and decrease linearly as we move away from the interface and decrease linearly as we move away from the interface. Once again the maximum strain in the CFRP reinforcement reached nearly 7000 microstrain.

4.6 Radial strain Measurement.

Radial strains in reinforcement

Radial cracks form in the concrete slab and in some theoretical models, such as the Zaghoul and Ben-Sasi model (2001), the failure perimeter is bounded by these cracks. In the case of steel reinforced slabs, it is implicitly assumed that the steel reinforcement along these cracks all reach their yield stress at failure and the flexural strength of the

slab is accordingly calculated. If these models were applied to FRP, we would need to know the strain variation along the radial crack planes

Figure 4.37: Load-Strain in the bottom reinforcement parallel to the south face of column stub in Specimen ZJS

Figure 4.38: Strain distribution in the bottom reinforcement parallel to the south face of column stub in Specimen ZJF1

Figure 4-39: Strain distribution in the bottom reinforcement parallel to the south face of column stub in Specimen ZJF2

Figure 4-40: Strain distribution in the bottom reinforcement parallel to the south face of column stub in Specimen ZJF3

Figure 4-41: Strain distribution in the bottom reinforcement parallel to the south face of column stub in Specimen ZJF4

Figure 4.42: Strain distribution in the bottom reinforcement parallel to the south face of column stub in Specimen ZJF5

Figure 4-43: Strain distribution in the bottom reinforcement parallel to the south face of column stub in Specimen ZJF6

Figure 4-44: Strain distribution in the bottom reinforcement parallel to the south face of column stub in Specimen ZJF7

Figure 4.45: Strain distribution in bottom reinforcement in Specimen ZJF1, (Gauges 3,4, and 6)

Figure 4.46: Strain distribution in bottom reinforcement in Specimen ZJF2, (Gauges 3,4, and 6)

Figure 4.47: Strain distribution in bottom reinforcement in Specimen ZJF3.
(Gauges 3,4, and 6)

Figure 4.48a: Strain distribution in bottom reinforcement in Specimen ZJF4.
(Gauges 3,4, and 6)

Figure 4.48b: Strain distribution in bottom reinforcement in Specimen ZJF4, (Gauges 2,5 and 20)

Figure 4.49: Strain distribution in bottom reinforcement in Specimen ZJF5, (Gauges 3,4, and 6)

Figure 4.50: Strain distribution in bottom reinforcement in Specimen ZJF6, (Gauges 3,4, and 6)

Figure 4.51: Strain distribution in bottom reinforcement in Specimen ZJF7, (Gauges 3,4, and 6)

Figure 4.52: Strain distribution in bottom reinforcement in Specimen ZJS, (Gauges 1,3,4,5, and 6)

Figures 4.53 to 4.55 show typical variation of these strains in either the top or bottom reinforcement mesh in specimens ZJS and ZJF1. The top reinforcement layer strains in ZJF1 (Figure 4.55) decrease as we move away from the slab-column interface, while in the steel reinforced specimen (Figure 4.54) all the bars reach yield stress. On the other hand, the top reinforcement mesh strains do not vary very much and are generally relatively small. Hence in applying the Zaghoul-Ben Sasi model to FRP, one must be careful when calculating the flexural capacity of the slab along these cracked planes.

Figure 4.53: Radial strain of bottom reinforcement across the side slab-column interface Specimen ZJF3, (Gauges 8,9,10, and 11)

Figure 4.54: Radial strain of bottom reinforcement across the side slab-column interface in Specimen ZJS, (Gauges 8,9,10,11, and 12)

Figure 4.55: Radial Strain of top reinforcement across the side slab-column interface in Specimen ZJF1, (Gauges 12,13, and 14)

Shear Reinforcement Strain

In Specimen ZJF7, FRP shear reinforcement was installed around the column perimeter as indicated in Figure 3.5. In order to gauge the effectiveness of this reinforcement, the strain in some of them was measured. Figures 4.56 and 4.57 show the strain variation in some of the vertical shear reinforcement legs

We notice that the maximum strain in some legs reached over 2000 microstrains (see Figure 4.56(a)). This translates to a stress of approximately 200 MPa. Of course, as expected, up to about 80% of the load, the strain in this reinforcement was quite small; thereafter it increased quite noticeably. This would indicate that the shear reinforcement becomes effective after the formation of the shear cracks inside the slab, for as pointed early, these cracks began to form at roughly 80 to 85% of the ultimate load.

It is important to observe that the shear reinforcement loses its effectiveness beyond a distance of 150 mm from the slab column interface because the failure perimeter does not extend that far. Finally, these strain values could be used to calculate the contribution of the shear reinforcement to the punching shear resistance of the slab, but due to inadequate data in the current test program, this will not be pursued here.

* (For location of Line 1, See figure 3.4a)

Figure 4.56a: Distribution of shear reinforcement strain along Line 1 in Specimen ZJF7

* (For location of Line 2, See figure 3.4a)

Figure 4.56b: Distribution of shear reinforcement strain along Line 2 in Specimen ZJF7

* (For location of Line TS1, See figure 3.4a)

Figure 4.57a: Distribution of shear reinforcement strain along Line TS1 in specimen ZJF7

* (For location of Line TS2, See figure 3.4a)

Figure 4.57b: Distribution of shear reinforcement strain along Line TS2 in Specimen ZJF7

Concrete Strain

While extensive measurements were taken using Demec gauges to measure concrete strains (see appendix D), much of the data was neither unexpected nor illuminating. Since the main reason for measuring concrete strain was to obtain data about the maximum concrete strain at failure, we will show the strain variation around the column stub on the top surface of the slab in specimen ZJF3. The strain data for this specimen are typical and the other specimens showed similar results.

Figure 4.58 and 4.59 show the variation of concrete strain around the column stub in specimen ZJF3, as measured by the electrical resistance strain gauges, and in the directions shown. As expected the highest strain occurred on the south face of the column

and it reached nearly 2800 microstrain (Figure 4.59b). The latter figure also shows an apparent strain gradient but due to the complex distribution of stresses and strains adjacent to the column, it would be difficult to draw general conclusions about the state of strain away from the column where failure occurs.

column centre because the column centre line perpendicular to the direction of bending constitutes the axis of rotation. The important conclusion that can be drawn from the concrete strain measurements is that none of the test specimens failed due flexural failure or crushing of concrete.

Figure 4.58a: Top surface concrete strain distribution perpendicular to the south face of the column in Specimen ZJF3

Figure 4.58b: Top surface compressive strain distribution tangential to the south face of ZJF3 column stub (tangential strains are higher)

Figure 4.59a: Top surface compressive strain distribution perpendicular to the east face of ZJF3 column stub

Figure 4.59b: Top surface compressive strain distribution tangential to the east face of ZJF3 column stub

Chapter 5

Analysis of Interior Slab-Column Connections in a Flat Plate Structures Subjected to Shear and Unbalanced Moment Transfer

5.1 General

In this chapter several methods are used to predict the ultimate strength of the CFRP reinforced interior slab-column connections tested in the current investigation. Well-established methods primarily intended for the design of similar connections with their slabs reinforced with traditional steel will be employed. It must be clear that none of the existing codes recommends that its provisions for steel reinforced concrete connections being directly extended to FRP reinforced connections. Our objective in this chapter is to investigate whether they can be applied in their current form to connections reinforced with FRP.

In addition to the code methods, we would also employ one known method of analysis proposed by some investigators. Again this is method has been mainly developed to analyze steel reinforced connections.

A few methods have been proposed by some researchers for the analysis of FRP reinforced slab-column connections. As stated in Chapter 2, these methods have been developed based on data from concentric shear tests on slabs reinforced with FRP. Therefore, it would be useful to find out whether they can be applied to eccentrically loaded connections.

Finally we close this chapter by suggesting a new expression for the punching shear resistance of FRP reinforced slab-column connections subjected to combined shear and bending moment. In all cases the predictions of the various methods will be compared with test results.

5.2 Punching Shear Capacity of Slab-Column Connections

In Chapter 2, a large number of methods for determining the punching shear capacity of slab-column connections were described. It is useful to recap here the essential features of most of these methods.

In all the methods the basic requirement for the punching shear capacity of slab - column connections is that

$$v_u \leq v_c \quad (5.1)$$

where

v_u = the maximum shear stress acting on the connection at ultimate load

v_c = shear resistance of the connection by the selected method of analysis.

The various codes expressions and other available methods of analysis differ among each other insofar as the calculation of v_u and v_c are concerned..

In the calculation of v_u , these methods differ widely insofar as the contribution of M_u to v_u is concerned, and in the location of the corresponding critical punching section. In other words, the fraction of the unbalanced moment transferred by shear, often denoted as γ_v , and the distance of the critical section from the face of the column vary from one code to another.

In analyzing the current test data, we will compare the predicted and observed punching shear capacities of the test specimens in the present study. Any difference between the predicted and measured shear strength can be eliminated by either adjusting v_c or v_u . Thus most investigators have proposed modified expressions for v_c in order to account for the presence of FRP reinforcement. This approach is reasonable given that v_c is a function of the concrete strength and the reinforcement rigidity. Although it is not a given that v_u will not be affected by the presence of FRP, nevertheless in the light of the empirical nature of the punching shear design methods, at this stage there is inadequate data to support an extensive analysis of both v_u and v_c .

Hence, we will proceed as follows. We calculate the maximum value of the shear force that can be carried by the connection according to each code or method using the condition $v_u = v_c$, i.e.

$$v_c = \frac{V_c}{b_o d} + \frac{\gamma_v V_c e y}{J} \quad (5.2)$$

Rearranging expression 5.2 yields:

$$V_c = \frac{v_c}{\left[\frac{1}{b_o d} + \frac{\gamma_v e y}{J} \right]} \quad (5.3)$$

V_c = the predicted maximum shear force that can be resisted by the connection under combined axial force and unbalanced moment.

γ_v = fraction of moment transferred by shear

= 0. for the case of concentric shear

e = eccentricity of the axial load = M/V

y = distance of the point of maximum shear stress from the centroid of the critical shear perimeter

J = polar moment of inertia of the critical shear perimeter (similar to the torsional constant).

For calculating V_c we will use the v_c expression(s) separate specified in the various codes or in one of the other suggested methods in the literature. For instance in the Canadian code CSA A23.4-94, v_c is the smallest of the following three values

$$(i) \quad v_c = \left(1 + \frac{2}{\beta_c}\right) 0.2 \lambda \phi_c \sqrt{f'_c} \quad (5.4)$$

$$(ii) \quad v_c = \left(\frac{\alpha_s d}{b_o} + 0.2\right) \lambda \phi_c \sqrt{f'_c} \quad (5.5)$$

$$(iii) \quad v_c = 0.4 \lambda \phi_c \sqrt{f'_c} \quad (5.6)$$

where β_c = ratio of long side to short side of column to be taken ≥ 2

λ = concrete density factor ($\lambda = 1.00$ for normal weight concrete)

ϕ_c = concrete safety factor (0.6)

f'_c = concrete compressive cylinder strength

$\alpha_s = 4$ for interior columns.

$\alpha_s = 3$ for edge columns.

$\alpha_s = 2$ for corner columns

d = effective depth of the slab at the critical perimeter

b = length of the perimeter of the critical section

In the present calculations any material factors, such as ϕ_c and ϕ_s , which account for the expected variabilities in the construction, will be set equal to one.

Finally, we compare V_c based on Equation 5.3 with the measured maximum value of the applied axial load for each specimen, denoted as V_{Test} . If $V_c > V_{Test}$, the method is unsafe, otherwise it is safe or conservative.

5.3 Comparison of Measured and Predicted Punching Shear Capacity of the Test Specimens

The measured, or test, and predicted capacities of the test specimens, based on the provisions of well-known codes for punching shear strength of steel reinforced concrete flat plates will be compared, followed by similar comparison with the predictions of one other suggested method in the literature.

5.3.1 Codes Predictions

The provisions of the following three codes will be used to predict the punching shear strength of the tested specimens. Reference may be made to Section 2.2.1.10 for details of each codes method.

- (a) The American Concrete Institute Code (ACI 318-95)
- (b) The Canadian Standard (CSA A23.3-94)
- (c) The British Standard (BS-8110-85)

Table 5.1 shows results of the analysis, where the ratio of the test results to the predicted value for the capacity of each specimen is presented.

The mean and the standard deviation of the ratio of the test to the predicted capacities of Specimens ZJF1 through ZJF6 are also shown. Specimen ZJF7 had FRP shear reinforcement that is not taken into account by the codes and therefore is not included in the mean calculation. Similarly Specimen ZJS is precluded from the mean because it is steel reinforced.

As we can see in Table 5.1, all the codes are conservative; but the CSA A23.3-94 predictions are quite reasonable and less conservative than the other codes. The mean value is 1.02 with a standard deviation of approximately 0.12. Interestingly, all the codes give a more conservative estimate of the punching capacity of Specimen ZJS, which is conventionally reinforced with steel bars. Of course, Specimen ZJF7 has a much higher strength than predicted by any of the codes because it has shear reinforcement which is not accounted for in the analysis. Note that the results are conservative despite the fact that material resistance factors were assumed to be equal to 1.0.

It is important to note that the CSA predictions are unsafe for ZJF5 and ZJF6 while the ACI and BS-8110 predictions are conservative for all the test specimens. However, since these codes do not calculate the ultimate load on a structure identically, it is difficult to make general statements about the relative conservatism of the three codes predictions. Overall, the results appear to be satisfactory.

5.3.2 Modified Code Expressions for FRP Reinforced Connections

As discussed in Chapter 2, El-Ghandour et al. (1999) proposed that the ACI 318-95 and the BS-8110-85 expressions for V_c , can be modified by multiplying the ACI 318-95 by the cubic root of the ratio of the elastic modulus of the FRP to the elastic modulus of steel, i.e. $(E_{FRP}/E_{STEEL})^{1/3}$ (see Equation 2.54 for the ACI 318-95 Code modification). For the modification of the expression in the BS 8110-85 (Equation 2.21), to obtain an equivalent area to steel, they multiplied the FRP reinforcement ratio by $(E_{FRP}/E_{STEEL}) \phi$,

where $\phi = 1.8$. We will extend their suggested modifications to the ACI 318-95 to the CSA A23.3-94 expression for v_c in order to determine whether it can be incorporated in the provisions of the latter Standard.

Table 5.2 shows the results of the analysis based on El-Ghandour et al.'s modifications. As we can observe in this table, the modified expressions render all three codes predictions to be more conservative. The ACI, CSA and BS-8110 mean values become 25.6%, 26.2%, and 3%, respectively, which are more conservative compared to the un-modified values. It would appear that these modifications may not be necessary at least for the type and amount of reinforcement used in the current study.

It may be recalled from Chapter 2 that Matthy's and Taerwe also suggested modifications to the BS-8110 expression for v_c in order to make it applicable to FRP reinforced slab-column connections. If we use their modified expression to predict the strength of the current test specimens, the results would be even more conservative than those predicted by El-Ghandour et al. modification.

Table 5.1 - Comparison of the predicted capacities of the test specimens by the code equations with test results

Specimen	Column stub (C2xC1) (mmxmm)	Slab thickness (mm)	Effective depth (d) (mm)	Reinforcement ratio (%)	M/V	f' _c MPa	V _{TEST} kN	ACI 318-95		CSA A23.3-94		BS-8110-85	
								V _{C,ACI 318-95} kN	$\frac{V_{TEST}}{V_{C,ACI}}$	V _{C,CSA A-23.3-94} kN	$\frac{V_{TEST}}{V_{C,CSA}}$	V _{C,BS-8110} kN	$\frac{V_{TEST}}{V_{BS-8110}}$
ZJF(1)	250x250	100	75	1.00	0.22	44.7	171.0	119.4	1.43	144.7	1.18	118.1	1.45
ZJF(2)	250x250	100	75	0.65	0.22	42.1	144.2	115.9	1.24	140.5	1.03	102.3	1.41
ZJF(3)	250x250	100	75	1.00	0.3	47.1	134.3	105.5	1.27	127.9	1.05	102.75	1.31
ZJF(4)	250x250	125	100	1.184	0.22	47.5	250.0	183.0	1.37	221.8	1.13	190.2	1.32
ZJF(5)	250x250	100	75	1.00	0	44.4	234.0	214.5	1.09	260.9	0.90	200.1	1.17
ZJF(6)	250x350	125	100	1.184	0.22	47.9	235.0	217.7	1.08	263.9	0.89	207.3	1.13
ZJF(7)	250x250	125	100	1.184	0.22	45.7	317.7	179.6	1.77	217.7	1.46	190.1	1.67
ZJS	250x250	100	81	1.00	0.22	46.0	218.2	134.4	1.62	162.9	1.34	131.8	1.66
MEAN*									1.25		1.03		1.30
STANDARD DEVIATION*									0.14		0.12		0.13

* Mean and standard deviation are calculated for specimens ZJF1 through ZJF6

Table 5.2 - Comparison of the predicted capacities of the test specimen with the predictions of the modified code equations per El-Ghandour et al.

Specimen	Column stub (C2x C1) (mmxmm)	Slab thickness (mm)	Effective depth (d) (mm)	Reinforcement ratio (%)	M/V	f _c ' MPa	V _{TEST} kN	ACI 318-95M		CSA A23.3-94M		BS 8110-85-M	
								V _{C,ACI-95} kN	V _{TEST} / V _{C,ACI-95}	V _{C,CSA-23.3-94} kN	V _{TEST} / V _{C,ACI-95}	V _{C,BS-8110} kN	V _{TEST} / V _{C,BS-8110}
ZJF(1)	250x250	100	75	1.00	0.22	44.7	171.0	94.8	1.80	114.9	1.49	114.0	1.50
ZJF(2)	250x250	100	75	0.65	0.22	42.1	144.20	92.0	1.57	111.5	1.29	98.76	1.46
ZJF(3)	250x250	100	75	1.00	0.3	47.1	134.3	83.8	1.60	101.5	1.32	99.2	1.35
ZJF(4)	250x250	125	100	1.184	0.22	47.5	250.0	145.3	1.72	176.1	1.42	183.5	1.36
ZJF(5)	250x250	100	75	1.00	0	44.4	234.0	170.3	1.37	206.4	1.13	193.2	1.21
ZJF(6)	250x350	125	100	1.184	0.22	47.9	235.0	172.9	1.36	209.5	1.12	200.2	1.17
ZJF(7)	250x250	125	100	1.184	0.22	45.7	317.7	142.3	2.23	172.8	1.84	183.5	1.73
ZJS	250x250	100	81	1.00	0.22	46.0	218.2	134.4	1.62	162.9	1.34	131.8	1.66
MEAN*									1.57		1.30		1.34
STANDARD DEVIATION*									0.18		0.15		0.13

* Mean and standard deviation are calculated for specimens ZJF1 through ZJF6

5.3.3 Sherif and Dilger's Method Prediction

A method was suggested by Sherif and Dilger (1996) to predict the punching shear capacity of flat plate connections subjected to combined shear and unbalanced moment. The method has been recommended for steel reinforced connections, as described in Chapter 2, but we will investigate its applicability to FRP reinforced slab-column connections subjected to shear and unbalanced moment. It may be recalled that they suggested modifications to both the γ_v and v_c expressions of the ACI. The modified v_c contains the slab reinforcement ratio, ρ . Here we will employ these modified expressions, but will not attempt to include the effect of the elastic modulus of FRP on v_c .

Results of the analysis are given in Table 5.3. In this table the values of γ_v are also shown which indicate that a smaller fraction of the moment is transferred by shear compared to the corresponding ACI values. This method, as shown in the last column of the table is also conservative for current CFRP reinforced connections. The degree of conservatism is essentially the same as in the original ACI method, despite the fact that it includes the reinforcement ratio in the v_c calculation. Note that if one were to include the effect of the elastic modulus of the FRP in this method by replacing ρ by $\frac{E_F}{E_S} \rho$, the results would become even more conservative. It would give a mean value of 1.63 with a standard deviation of 0.26 for the ratio of test/predicted ultimate strength of the six CFRP reinforced specimens.

Table 5.3 Comparison of Sherif and Dilger's method predictions with test results.

Specimen.	Column stub (C2xCl) (mmxmm)	Slab thickness (mm)	Effective depth (d) (mm)	Reinforce- ment ratio (%)	M/V	f'_c MPa	V_{TEST} kN	Sherif and Dilger-96 for Steel Reinforcement		
								γ_V	V_C , Sherif & Dilger	$\frac{V_{TEST}}{V_{Sherif \& Dilger}}$
ZJF(1)	250x250	100	75	1.00	0.22	44.7	171.0	0.28	121.9	1.40
ZJF(2)	250x250	100	75	0.65	0.22	42.1	144.20	0.32	98.2	1.45
ZJF(3)	250x250	100	75	1.00	0.30	47.1	134.30	0.28	109.7	1.25
ZJF(4)	250x250	125	100	1.184	0.22	47.5	250.0	0.26	223.0	1.12
ZJF(5)	250x250	100	75	1.00	0.00	44.4	234.0	0.28	189.9	1.16
ZJF(6)	250x350	125	100	1.184	0.22	47.9	235.0	0.26	244.8	0.96
ZJF(7)	250x250	125	100	1.184	0.22	45.7	317.7	0.26	220.3	1.44
ZJS	250x250	100	81	1.00	0.22	46.0	218.2	0.28	140.3	1.55
MEAN*										
STANDARD DEVIATION*										
0.18										

* Mean and standard deviation are for specimens ZJF1 through ZJF6 only.

5.3.4 Proposed Method for Calculating Punching Shear Capacity of FRP Reinforced Slab-Column Connections

It is proposed here to use the basic approach of CSA A23.3-94 with a modified expression for v_c . The basic assumptions of the method are:

- i The critical section adopted by the method is the same as the one recommended by the Canadian Standard CSA A23.3-94. The Canadian standard recommends that the critical section for eccentric shear at a distance of $d/2$ from the slab-column interface.
- ii The shear stress distribution is assumed to be linear as in the CSA standard.
- iii The portion of the unbalanced moment that is transferred by eccentric shear, γ_v , is determined by Equation 2.45 as recommended by the CSA Code.

These assumptions imply that the v_u calculation in the proposed method would be identical to those in CSA code. On the other hand, it is suggested to calculate v_c based on the provisions of the draft CSA standard S806, Design of FRP Reinforced Concrete Building Components (CSA 2000). The latter standard suggests that in FRP reinforced beams the concrete shear resistance be calculated using .

$$v_c = 0.035 \lambda \phi_c \left(f'_c \rho E_{FRP} \right)^{0.333} \quad (5.7)$$

where,

E_{FRP} = elastic modulus of FRP longitudinal reinforcement in a beam (MPa)

ρ = reinforcement ratio of longitudinal reinforcement

λ = concrete density factor (1 for normal weight concrete)

f'_c = concrete compressive cylinder strength

ϕ_c = concrete resistance factor (0.6)

Since the CSA A23.3-94 adopts the value of v_c for two-way concrete shear strength to be double the value for one-way shear in beams and slabs, we adopt the same approach here. Therefore, for shear in two-way slabs reinforced with FRP reinforcement, v_c will be assumed to be double the value given by Equation 5.7, i.e.

$$v_c = 0.07 \lambda \phi \left(f'_c \rho E_{FRP} \right)^{0.333} \quad (5.8)$$

Using the latter expression for v_c in Equation 5.8, we calculate the predicted shear capacities of the test specimens and compare the results with the corresponding test values where λ and ϕ are assumed equal to unity.

Table 5.4: Comparison of the punching shear capacities of the test specimens by the proposed method with the experimental results

Specimen	f'_c MPa	V_{Test} kN	$V_{C,Proposed}$ kN	$V_{Test}/V_{C,Proposed}$
ZJF1	44.7	171.0	129.7	1.32
ZJF2	42.1	144.2	127.2	1.13
ZJF3	47.1	134.30	113.6	1.18
ZJF4	47.5	250.0	196.8	1.27
ZJF5	44.4	234.0	233.3	1.00
ZJF6	47.9	235.0	233.8	1.01
ZJF7	45.7	317.7	194.4	1.63
ZJS	46.0	218.2	162.3	1.34
MEAN				1.15
STANDARD DEVIATION				0.13

Table 5.4 compares the predicted and observed shear capacities of the current test specimens. We can see that the mean value of the ratio of the test/predicted capacities for the six FRP reinforced connections is 1.15 with a standard deviation of 0.13. The strength of specimen ZJF7, which has vertical shear reinforcement, is underestimated, but this is expected because the method does not take into consideration the presence of shear reinforcement. It is clear from this table that all the results are on the safe side, with a small standard deviation. Accordingly it is important to investigate the applicability of this method to other types of connections with different types of FRP reinforcement and loading configuration.

Of course the advantage of the proposed method over the existing ACI and CSA provisions is that it includes the effect of the elastic modulus and amount of the slab reinforcement on its punching shear capacity. Clearly, these are important properties which cannot be ignored in the case of FRP reinforced slabs.

Chapter 6

Conclusion and Recommendation

6.1 Conclusions

The experimental and theoretical study support the following conclusions:

- (1) The failure modes of the CFRP reinforced slab column connections in punching shear are similar to those of steel reinforced connections. The failure mode is ductile with large deformations, but without significant loss in strength.
- (2) The crack pattern and density in the CFRP reinforced slabs was also comparable to those in the steel reinforced slab, but the crack width for the same reinforcement ratio was somewhat larger. However, the larger cracks were still within the acceptable limits of the current Canadian practice.
- (3) In the CFRP reinforced specimens, the punching perimeter was closer to the column than in the steel reinforced specimen.
- (4) The overall load-deflection response of the CFRP reinforced specimens was basically the same as that of the steel reinforced specimen.

However, due to the lower elastic modulus of FRP, their stiffness was somewhat smaller. However the effect of the elastic modulus on stiffness and strength is not linear.

- (5) A 25% increase in the slab thickness will cancel the negative effects of the lower elastic modulus of CFRP on the stiffness and strength of the interior slab-column connections, irrespective of the moment to shear ratio, see Table 4.2.
- (6) The high strength of CFRP does not increase the slab-column connection punching shear capacity, nor can this strength be fully utilized in design.
- (7) The punching shear capacity is governed, among other factors, by the rigidity of the reinforcement, which depends on the reinforcement, which depends on the reinforcement ratio and its elastic modulus.
- (8) Increase in the moment/shear ratio decreases the ultimate shear capacity of the connection. The linear moment shear interaction relation as currently presented by the CSA and ACI could also apply to FRP reinforced connections.
- (9) An increase in slab thickness has a very noticeable effect on both the strength and the stiffness of the CFRP reinforced slab-column connections.
- (10) The column aspect ratio does not have a significant effect on the strength and stiffness of CFRP reinforced slab-column connections.
- (11) An increase in the reinforcement ratio from 0.65% to 1.0% increased the ultimate strength of the connection by approximately, 19%.
- (12) CFRP shear reinforcement can be highly effective in increasing the strength of interior slab-column connections strength by nearly 27%, see Table 4.2 for comparison

of strengths of Specimens ZJ4 and ZJ7, without any negative effect on the behaviour of the connection.

- (13) The existing CSA, ACI and British Codes provisions give a conservative estimate of the punching shear strength of the current test specimens.
- (14) The modifications proposed by El-Ghandour et al. render the above code predictions even more conservative.
- (15) The CSA and ACI methods, subjected to the modifications to their v_c terms, as proposed in this study, predict the ultimate strength of the tested connections very well. The proposed modification takes into account the ratio and elastic modulus of the slab reinforcement into account.
- (16) Based on the overall evaluation of the test results, the CFRP reinforcement can be used in flat plate structures, but an increase in the slab thickness around the column would be needed to compensate for the lower stiffness of the FRP.

6.2 Recommendations for Future Research

The following is recommended for further investigation.

- 1- Studies involving different types of FRP including glass and carbon fibre bars
- 2- Study on the effectiveness of column capitals or drop panels for FRP reinforced connections.
- 3- Further studies to investigate the type of shear reinforcement introduced in the present work.

- 4- Study of the exterior slab column connections i.e. edge and corner connections, reinforced by FRP reinforcement.
- 5- Studies concerning the effect of cyclic lateral loading on the strength and ductility of FRP reinforced slab-column connections.
- 6- Studies involving FRP reinforcement in the slab and FRP ties in the column.
- 7- further analysis should be undertaken to determine whether the proposed method for calculating v_c can be applied to other types of slab-column connections.

Appendix (A)

A.1 Cylinder Compressive Strength of Concrete, Specimen ZJS

Cylinder Number	Test Type	Test Results N/mm ²	Average strength N/mm ²
Cylinder 1	Compressive Strength	43.086	45.98
Cylinder 2	Compressive Strength	43.788	
Cylinder 3	Compressive Strength	47.541	
Cylinder 4	Compressive Strength	47.681	
Cylinder 5	Compressive Strength	47.795	

A.2 Cylinder Compressive Strength of Concrete, Specimen ZJF(1)

Cylinder Number	Test Type	Test Results N/mm ²	Average strength N/mm ²
Cylinder 1	Compressive Strength	41.154	44.7
Cylinder 2	Compressive Strength	44.300	
Cylinder 3	Compressive Strength	44.678	
Cylinder 4	Compressive Strength	45.181	
Cylinder 5	Compressive Strength	48.202	

A.3 Cylinder Compressive Strength of Concrete, Specimen ZJF(2)

Cylinder Number	Test Type	Test Results N/mm ²	Average strength N/mm ²
Cylinder 1	Compressive Strength	38.637	42.09
Cylinder 2	Compressive Strength	40.021	
Cylinder 3	Compressive Strength	43.042	
Cylinder 4	Compressive Strength	43.923	
Cylinder 5	Compressive Strength	44.804	

A.4 Cylinder Compressive Strength of Concrete, Specimen ZJF(3)

Cylinder Number	Test Type	Test Results N/mm ²	Average strength N/mm ²
Cylinder 1	Compressive Strength	41.649	47.11
Cylinder 2	Compressive Strength	46.895	
Cylinder 3	Compressive Strength	48.004	
Cylinder 4	Compressive Strength	49.034	
Cylinder 5	Compressive Strength	49.968	

A.5 Cylinder Compressive Strength of Concrete, Specimen ZJF(4)

Cylinder Number	Test Type	Test Results N/mm ²	Average strength N/mm ²
Cylinder 1	Compressive Strength	44.992	47.462
Cylinder 2	Compressive Strength	46.565	
Cylinder 3	Compressive Strength	48.957	
Cylinder 4	Compressive Strength	49.334	
Cylinder 5	Compressive Strength	50.970	

A.6 Cylinder Compressive Strength of Concrete, Specimen ZJF(5)

Cylinder Number	Test Type	Test Results N/mm ²	Average strength N/mm ²
Cylinder 1	Compressive Strength	40.273	44.42
Cylinder 2	Compressive Strength	46.691	
Cylinder 3	Compressive Strength	43.293	
Cylinder 4	Compressive Strength	45.810	
Cylinder 5	Compressive Strength	46.062	

A.7 Cylinder Compressive Strength of Concrete, Specimen ZJF(6)

Cylinder Number	Test Type	Test Results N/mm ²	Average strength N/mm ²
Cylinder 1	Compressive Strength	39.680	47.895
Cylinder 2	Compressive Strength	49.345	
Cylinder 3	Compressive Strength	51.230	
Cylinder 4	Compressive Strength	51.326	

A.8 Cylinder Compressive Strength of Concrete, Specimen ZJF(7)

Cylinder Number	Test Type	Test Results N/mm ²	Average strength N/mm ²
Cylinder 1	Compressive Strength	38.637	42.085
Cylinder 2	Compressive Strength	40.021	
Cylinder 3	Compressive Strength	43.042	
Cylinder 4	Compressive Strength	43.923	
Cylinder 5	Compressive Strength	44.804	

A.9 Summary of the results of testing cylinders for the second mix.

	Time of testing	Kind of test	Strength (MPa)
Cylinder 1	1 week	Compressive	34.5
Cylinder 2	4 weeks	Compressive	42.790
Cylinder 3	4 weeks	Compressive	43.923
Cylinder 4	4 weeks	Compressive	45.810
Cylinder 5	4 weeks	Compressive	46.943
Cylinder 6	4 weeks	Compressive	48.327

Appendix (B)

Locations of Demec points on the top surface of the slab in Specimens ZJF(2) through ZJF(7)

Figure B.1: Locations of Demec points on the top surface of the slab in Specimen ZJF(2)

Figure B.2: Locations of Demec points on the top surface of the slab in Specimen ZJF(3)

Figure B.3: Locations of Demec points on the top surface of the slab in Specimen ZJF(4)

Figure B.4: Locations of Demec points on the top surface of the slab in Specimen ZJF(5)

Figure B.5: Locations of Demec points on the top surface of the slab in Specimen ZJF(6)

Figure B.6: Locations of Demec points on the top surface of the slab in Specimen ZJF(7)

Figure B.7: Locations of Demec points on the top surface of the slab in Specimen ZJF(7)

Locations of the LVDT on the top and bottom surfaces of the slabs and column stubs of the test Specimens

FigureB.8: The location of the LVDTs on the top surface of Specimen ZJF1.

Figure B.10: The location of the LVDTs on the top surface of Specimen ZJF2.

Figure B.11: The location of the LVDTs on the bottom surface of Specimen ZJF2

Figure B.12: The location of the LVDTs on the top surface of Specimen ZJF3

Figure B.12: The location of the LVDTs on the bottom surface of Specimen ZJF3

Figure B.13: The location of the LVDTs on the top surface of Specimen ZJF4.

Figure B.14: The location of the LVDTs on the bottom surface of Specimen ZJF4

Figure B.15: The location of the LVDTs on the top surface of Specimen ZJF5.

Figure B.16: the location of the LVDTs on the bottom surface of Specimen ZJF5

Figure B.17: The location of the LVDTs on the top surface of Specimen ZJF6.

Figure B.18: The location of the LVDTs on the bottom surface of Specimen ZJF6

Figure B.19: The location of the LVDTs on the top surface of Specimen ZJF7.

Figure B.20: The location of the LVDTs on the bottom surface of Specimen ZJF7

Appendix (C)

Figure C.1a: Load versus deflections of the top and bottom surfaces of Specimen ZJF2 at 50 mm from the south face of the column

Figure C.1b: Load versus deflections of top and bottom concrete surfaces of Specimen ZJF2 at 200mm from the column south face

Figure C.2a: Load versus deflections of the top and bottom surfaces of Specimen ZJF3 at 50 mm from the south face of the column

Figure C.2b: Load versus deflections of top and bottom concrete surfaces of Specimen ZJF3 at 200mm from the column south face

Figure C.3a: Load versus deflections of the top and bottom surfaces of Specimen ZJF4 at 50 mm from the south face of the column

Figure C.3b: Load versus deflections of top and bottom concrete surfaces of Specimen ZJF4 at 200mm from the column south face

Figure C.4a: Load versus the top and bottom surfaces deflections of Specimen ZJF5 at 50 mm from the column south face

Figure C.4b: Load versus deflections of top and bottom concrete surfaces of Specimen ZJF5 at 200mm from the column south face

Figure C.5a: Load versus deflections of the top and bottom surfaces of Specimen ZJF6 at 50 mm from the south face of the column

Figure C.5b: Load versus deflections of top and bottom concrete surfaces of Specimen ZJF6 at 200mm from the column south face

Figure C.6: Load versus deflections of the top and bottom surfaces of Specimen ZJF7 at 50 mm from the south face of the column

Appendix (D)

Figure D.1a: Load - strain of bottom reinforcement across the side slab-column interface in Specimen ZJF1, (Gauges 8,9,10, and 11)

Figure D.1b: Load - strain of bottom reinforcement across the side slab-column interface in Specimen ZJF2, (Gauges 8,9,10, and 11)

Figure D.1c: Load - strain of bottom reinforcement across the side slab-column interface in Specimen ZJF4, (Gauges 8,9,10, and 11)

Figure D.1d: Load - bottom reinforcement strain in Specimen ZJF4, (Gauges 9 and 24)

Figure D.1e: Load- bottom reinforcement strain in Specimen ZJF4, (Gauges 8 and 25)

Figure D.1f: Load - strain of bottom reinforcement across the side slab-column interface in Specimen ZJF5, (Gauges 8,9,10, and 11)

Figure D.1g: Radial strain of bottom reinforcement across the side slab-column interface in Specimen ZJF6, (Gauges 8,9,10, and 11)

Figure D.1h: Radial strain of bottom reinforcement across the side slab-column interface in Specimen ZJF7, (Gauges 8,9,10, and 11)

Figure D.2a: Load - strain of top reinforcement across the side slab-column interface in Specimen ZJF2, (Gauges 12,13, and 14)

Figure D.2b: Radial strain of top reinforcement across the side slab-column interface in Specimen ZJF3, (Gauges 12,13,14, and 22)

Figure D.2c: Load-top reinforcement strain in Specimen ZJF3, Gauges 14 and 23

Figure D.2d: Load-top reinforcement strain in Specimen ZJF3, Gauges 13 and 22

Figure D.2e: Load - top strain reinforcement across the side slab-column interface in Specimen ZJF4, Gauges 12,13,14, and 22

Figure D.2f: Load-top reinforcement strain in Specimen ZJF4, Gauges 13 and 22

Figure D.2g: Radial strain of top reinforcement across the side slab-column interface in Specimen ZJF6, Gauges 12,13, and 14

Figure D.2h: Radial strain of top reinforcement across the side slab-column interface in Specimen ZJF7, (Gauges 12,13, and 14)

Figure D.3a: Load-top and bottom reinforcement strain in Specimen ZJF1, (Gauges 7 and 3)

Figure D.3b: Load-top and bottom reinforcement strain in Specimen ZJF1, (Gauges 14 and 11)

Figure D.4a: Load-top and bottom reinforcement strain for Specimen ZJF2, (Gauges 7 and 3)

Figure D.3b: Load-top and bottom reinforcement strain for Specimen ZJF2, (Gauges 14 and 11)

Figure D.4a: Load-top and bottom reinforcement strain in Specimen ZJF3, (Gauges 7 and 3)

Figure D.4b: Load-top and bottom reinforcement strain in Specimen ZJF3, (Gauges 22 and 9)

Figure D.4c: Load-top and bottom reinforcement strain in Specimen ZJF3, (Gauges 12 and 8)

Figure D.4d: Load-top and bottom reinforcement strain in Specimen ZJF3, (Gauges 21 and 1)

Figure D.5a: Load-Top and Bottom reinforcement strain in Specimen ZJF5, (Gauges 7 and 3)

Figure D.5b: Load-top and bottom reinforcement strain in Specimen ZJF5, (Gauges 12 and 8)

Figure D.5c: Load-top and bottom reinforcement strain in Specimen ZJF5, (Gauges 13 and 9)

Figure D.6a: Load-Top and Bottom reinforcement strain in Specimen ZJF6, (Gauges 7 and 3)

Figure D.6b: Load-top and bottom reinforcement strain in Specimen ZJF6, Gauges 14 and 11

Figure D.7a: Load-top and bottom reinforcement strain in Specimen ZJF7, (Gauges 7 and 3)

Figure D.7b: Load-top and bottom reinforcement strain in Specimen ZJF7, (Gauges 14 and 11)

Figure D.8a: Load-Top and Bottom reinforcement strain in Specimen ZJS, (Gauges 7 and 1)

Figure D.8b: Load-Top and Bottom reinforcement strain in Specimen ZJS, (Gauges 13 and 11)

Figure D.8c: Load-Top and Bottom reinforcement strain in Specimen ZJS, Gauges 14 and 9

Figure D.9: Names and location of Demec lines

Figure D.10a: Concrete top surface strain distribution along Line F in Specimen ZJF3

Figure D.10b: Concrete top surface strain distribution perpendicular to Line F in Specimen ZJF3

Figure D.11a: Concrete top surface strain distribution along Line D in Specimen ZJF3

Figure D.11b: Concrete top surface strain distribution perpendicular to Line D in Specimen ZJF3

Figure D.12a: Concrete top surface strain distribution along Line S in Specimen ZJF3

Figure D.12b: Concrete top surface Strain distribution perpendicular to Line S in Specimen ZJF3

Figure D.13a: Concrete top surface strain distribution along Line S1 in Specimen ZJF3

Figure 4.13b: Concrete top surface Strain distribution perpendicular to Line S1 in Specimen ZJF3

Figure D.14a: Concrete top surface strain distribution along Line S2 in Specimen ZJF3

Figure D.14: Concrete top surface Strain distribution perpendicular to Line S2 in Specimen ZJF3

Figure D.15a: Concrete top surface strain distribution along Line S3 in Specimen ZJF3

Figure D.15b: Concrete top surface Strain distribution perpendicular to Line S3 in Specimen ZJF3

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