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Behaviour of Concrete Beams Reinforced with FRP Prestressed Concrete Prisms

by

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

Doctor of Philosophy

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario, Canada
November, 1998

The Doctor of Philosophy in Civil Engineering is a joint program with the University of
Ottawa, administered by the Ottawa-Carleton Institute for Civil Engineering

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Behaviour of Concrete Beams Reinforced with FRP Prestressed Concrete Prisms

submitted by

Dagmar Švecová

in partial fulfillment of the requirements
for the degree of Doctor of Philosophy

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Acknowledgments

I am thankful to my advisor, Professor A.G. Razaqpur for his professional help and guidance while working on this thesis. My sincere thanks are extended to the following professors, who at early stages of my work commented on my thesis proposal: Professors G.A. Hartley, J.L. Humar, and G.T. Suter from the Department of Civil and Environmental Engineering at Carleton University, Professor M. Saatcioglu from the University of Ottawa, and Mr. J. Fowler, president of Canadian Precast/Prestressed Concrete Institute.

The experimental part of this work would not have been possible without the help of Messrs P. Trudel, S. Conley, J. Plavka and K. McMartin of the Civil Engineering Laboratories at Carleton University.

Assistance of Sika Canada by providing Sika Pronto 11 grout is highly appreciated.

Financial assistance received from the Natural Science and Engineering Research Council of Canada, and the Faculty of Graduate Studies and Research of Carleton University, is greatly appreciated.

Finally, I am very thankful to my family and friends for their constant support during my studies.

Abstract

The use of fibre reinforced plastics (FRP) to reinforce concrete is gaining acceptance. However, due to the relatively low modulus of FRP, in comparison to steel, such structures may, if sufficient amount of reinforcement is not used, suffer from large deformations and wide cracks. FRP is generally more suited for prestressing. Since it is not feasible to prestress all concrete structures to eliminate the large deflections of FRP reinforced concrete flexural members, researchers are focusing on other strategies.

A simple method for avoiding excessive deflections is to provide sufficiently high amount of FRP reinforcement to limit its stress (strain) to acceptable levels under service loads. This approach will not be able to take advantage of the high strength of FRP and will be generally uneconomical. The current investigation focuses on the feasibility of an alternative strategy.

This thesis deals with the flexural and shear behaviour of concrete beams reinforced with FRP prestressed concrete prisms. FRP prestressed concrete prisms (PCP) are new reinforcing bars, made by pretensioning FRP and embedding it in high strength grout/concrete. The purpose of the research is to investigate the feasibility of using such pretensioned rebars, and their effect on the flexural and shear behaviour of reinforced concrete beams over the entire loading range. Due to the prestress in the prisms, deflection of concrete beams reinforced with this product is substantially reduced, and is comparable to similarly steel reinforced beams.

The thesis comprises both theoretical and experimental investigations. In the experimental part, nine beams reinforced with FRP prestressed concrete prisms, and two companion beams, one steel and one FRP reinforced were tested. All the beams were designed to carry the same ultimate moment. Excellent flexural and shear behaviour of beams reinforced with higher prestressed prisms is reported. When comparing deflections of three beams designed to have the same ultimate capacity, but reinforced with either steel, PCP or FRP rebars, the service load deflections of beams reinforced with PCP are comparable to that of a steel reinforced concrete beam, and are four times smaller than the deflection of the companion

FRP reinforced beam. Similarly, the crack width of the PCP reinforced beams under service loads is comparable to that of the steel reinforced beam while the FRP reinforced beam developed unacceptably wide cracks.

In the analytical part comprehensive analysis of the experimental data in both flexure and shear is performed. It is determined that the existing design expressions for ultimate flexural strength and service load deflection calculation cannot accurately predict the response of PCP reinforced beams. Accordingly, new expressions for calculation of deflection, crack width, tension stiffening, and ultimate capacity of the PCP reinforced beams are proposed. The predictions of the proposed methods of analysis agree very well with the corresponding experimental data. Based on the results of the current study, it is concluded that high strength concrete prisms prestressed with carbon fibre reinforced plastic bars can be used as reinforcement in concrete structures to avoid the problems of large deflections and wide cracks under service loads.

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List of Symbols

a	shear span of beam
a_{min}	minimum crack spacing
a_{max}	maximum crack spacing
b	width of concrete cross-section
b_w	width of the cross section
c	depth of neutral axis, or concrete cover
c_{min}	minimum cover in the cross-section
d	(1)effective depth (2)diameter of reinforcement
d_b	bar diameter
d_c	concrete cover measured from the centroid of tension reinforcement to the extreme tension fibre
f'_c	compressive strength of concrete
f_c	stress in concrete
$f_{p,eff}$	effective prestress
f_r	modulus of rupture
f_s	calculated stress in the reinforcement at specified load
f_t	tensile strength of concrete
f_u	ultimate strength of stirrups
f_v	stress in stirrups
f_y	yield stress of stirrup
f_{FRP}	stress in FRP
f_{PCP}	stress in PCP
h	depth of the cross-section
h_1	distance from the centroid of tension reinforcement to the neutral axis
h_2	distance from the extreme tension fibres to the neutral axis
jd	internal lever arm
k	coefficient for crack width calculation, equal to 1.0 for deformed bars, 1.3 for plane bars
kd	depth of neutral axis
k_2	parameter which relates the position of C to the height of c
m	bending moment due to unit load

n_{sika}	modular ratio of Sika grout and concrete
n_{FRP}	modular ratio of FRP and concrete
$\sum o$	sum of the perimeter of the bars
p_{ri}	initial radial pressure
s	stirrup spacing
u	average bond stress
v	coefficient to account for bond properties of the reinforcement
w	crack width at the tensile face of the beam
w_{cr}	crack width
w_{max}	maximum crack width
x	neutral axis depth
y_t	distance from the centroidal axis of cross-section to extreme fibre in tension
z	(1)quantity limiting the distribution of flexural reinforcement (2)the internal lever arm
A	effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of rebars
A_e	effective area of concrete in tension
A_s	area of flexural reinforcement (steel)
A'_s	area of compression reinforcement
A_{FRP}	area of FRP reinforcement
A_{PCP}	area of PCP prisms
A_v	area of stirrup
C	resultant force in compression zone of concrete
C_c	compressive force in concrete
C_s	center-to-center distance of steel bars
E	modulus of elasticity
E_c	modulus of elasticity of concrete
E_{res}	modulus of elasticity of resin
E_s	modulus of elasticity
E_{FRP}	modulus of elasticity of FRP
I	moment of inertia

I_{cr}	moment of inertia of cracked section
$I_{cr,1}$	moment of inertia of cracked section of beam reinforced with PCP, with PCP uncracked
$I_{cr,2}$	moment of inertia of completely cracked section of beam reinforced with PCP, with PCP cracked
I_e	effective moment of inertia
I_g	moment of inertia of gross section
I_m	modified moment of inertia
L	beam span
L_g	distance from the nearest support, or from the free end of the cantilever, to the point on the beam where $M = M_{cr}$
L_i	distance from the i-th stirrup to the loading point
L_o	is $0.65d$
L_1	uncracked length of beam reinforced with PCP
L_2	length of beam with uncracked prisms, beam concrete already cracked
M	moment due to applied load
M_a	applied moment
M_{cr}	moment at beam cracking
$M_{cr,1}$	moment at concrete of the beam cracking (for PCP reinforced beams only)
$M_{cr,2}$	moment at PCP cracking (for PCP reinforced beams only)
M_r	moment of resistance
M_{ult}	ultimate moment
P	applied load
P_{ccr}	load at concrete cracking
P_{eff}	level of prestress in the prisms
P_{pcr}	load at prism cracking
P_{ult}	ultimate load for test beam
T	tensile force in reinforcement
T_{FRP}	tensile force in FRP
V_c	concrete contribution in shear
V_{cw}	shear cracking load
V_{max}	maximum shear force applied to a beam
V_s	shear reinforcement contribution in shear
V_r	total capacity of section in shear

α	empirical correction factor
α'	distance from the compressive face of the section to the point at which the crack is being calculated
α_c	coefficient of thermal expansion of concrete
α_{cr}	(1)distance from the crack width measurement location to the nearest reinforcing bar (2)coefficient dependent on load type
α_{res}	coefficient of thermal expansion of resin
β	empirical correction factor
β_1	stress block parameter
β_2	stress block parameter
γ	stress block parameter
γ_{xy}	shear strain in concrete
δ	midspan deflection of beam where cracked portions were accounted for if present
δ_c	deflection of uncracked section at distance L_g (correction factor)
$\delta_{c,1}$	deflection of uncracked section at distance L_1 (correction factor)
$\delta_{c,2}$	deflection of section with uncracked prisms at distance L_2 (correction factor)
δ_{cr}	midspan deflection of cracked section
$\delta_{cr,1}$	midspan deflection of fully cracked beam, with prisms not cracked
$\delta_{cr,2}$	midspan deflection of fully cracked beam, with prisms cracked
δ_1	midspan deflection of beam reinforced with PCP for load smaller than prism cracking load
ϵ_b	strain in the bottom fibres
ϵ_c	strain in concrete
ϵ_{cr}	strain at concrete cracking
ϵ_j	jacking strain
ϵ_l	(1)longitudinal strain; (2)strain corresponding to the level at which the crack width is measured
ϵ_m	average strain calculated from Equation 6.41
ϵ_{ri}	radial initial strain
ϵ_{sv}	strain in the stirrup
ϵ_x	longitudinal concrete strain in x direction

ϵ_y	transverse concrete strain in y direction
ϵ_{FRP}	strain in FRP reinforcement
ϵ_{PCP}	strain in prestressed concrete prism
$\epsilon_{1,2}$	principal strains in concrete
λ	modification factor to account for the type of concrete
ν_{res}	Poisson's coefficient for resin
ξ_1	tension stiffening coefficient for beam reinforced with PCP under load smaller than PCP cracking load
ξ_2	tension stiffening coefficient for beam reinforced with PCP which is completely cracked
ρ	reinforcement ratio
ρ_b	balanced reinforcement ratio
ρ_w	longitudinal reinforcement ratio in percent
σ_{se}	stress in the bar
ϕ	bar diameter
ϕ_c	coefficient of value 0.6
ϕ_s	coefficient of value 0.85
ψ	curvature
ψ_{cr}	curvature at cracking
Δ	deflection
Δ_A	desired deflection of point A
ΔT	temperature change

Chapter 1

Introduction

1.1 General

Corrosion of steel in reinforced concrete structures exposed to adverse environments has prompted researchers to investigate the use of non-corrosive materials for reinforcement of concrete. The idea of FRP (Fibre Reinforced Polymer or Plastic) as reinforcement for concrete is gaining popularity and is being actively researched in many countries. Clearly, any structure reinforced with FRP should satisfy not only economy and safety requirements, but also serviceability requirements. One of the most important serviceability problems of FRP reinforced concrete is deflection control. Because of the low modulus of elasticity of FRPs, deflections of FRP reinforced concrete beams are larger than those of conventional steel reinforced concrete beams. To alleviate this problem, a new concept for using FRP reinforcement is presented in this thesis. The new concept involves concrete prisms pretensioned with FRP bars and subsequently used as reinforcement.

This thesis will examine the safety and serviceability of concrete beams reinforced in

accordance with this new concept. Comparison will be made between FRP and prestressed concrete prism (PCP) reinforced beams.

In this chapter background for the study is discussed, the objectives of the research are highlighted, and the methods to be used for theoretical analysis are described. However, cost analysis is not within the scope of the study.

1.2 Problem Definition

FRP reinforced concrete beams behave similar to steel reinforced beams up to cracking. After cracking, the deflections of these beams due to the lower modulus of elasticity of FRP increase rapidly, and very often they reach values two to three times larger than similar steel reinforced beams (Nawy et al. 1971, Nawy and Neuwerth 1977, Faza 1991, Buyle-Budin et al. 1995). Beams with such large deflections would not satisfy the deflection limits specified in the National Building Code of Canada (NBC 1995). In order to limit those deflections, most beams need to be designed with an excessive amount of FRP reinforcement. Considering the relatively high cost of FRP, the large amount of reinforcement renders FRP reinforced concrete economically unattractive. It should be noted that due to the high tensile strength of FRP, the balanced reinforcement ratio of FRP reinforced beams is considerably smaller than that of steel reinforced beams (Švecová 1994). Therefore, most FRP reinforced beams are designed to be over-reinforced, which leads to their compressive failure. This type of failure is more desirable for such beams because it is not as violent as tensile failure which would lead to their complete disintegration.

To address the above mentioned problems in the design of FRP reinforced concrete beams, it is necessary to

1. Take advantage of the high tensile strength of FRP
2. To circumvent via prestressing the effect of the low modulus of elasticity on the behaviour of FRP reinforced concrete beams
3. To achieve satisfactory design by using reinforcement ratios slightly over the balanced reinforcement ratio and thus avoid using excessive amount of FRP material.

The feasibility of using FRP-prestressed concrete prisms to satisfy the above mentioned requirements is studied in this thesis.

FRP prestressed concrete prisms can be used as reinforcement for concrete beams, similarly to the steel prestressed prisms, which were widely employed some thirty years ago (Hoppe 1963, Burns 1966, Hanson 1969, König et al. 1996, etc.). The prisms were prestressed in precast plants and shipped to the site. The prisms would provide uncracked cover to the steel and protect them from premature corrosion. Another advantage of such a system was that the deflections of the beams reinforced with prestressed prisms were much smaller than similar beams reinforced with the same amount and type of steel. Although deflection control is not generally a problem for steel reinforced beams, for FRP reinforced beams deflection control is one of the main problems.

In Figure 1.1 the behaviours of FRP reinforced, FRP prestressed and PCP reinforced concrete beams are shown. It can be seen from this figure that using PCP as a reinforcement will delay cracking of the beam, and decrease deflections of PCP reinforced beams under

Beam Type	Stresses at Midspan		
	Initial	From Service Load	Final Stress
Reinforced Beam		$f_b > f_r$	$f_b > f_r$
Prestressed Beam		$f_b > f_r$	$f_b < f_r$
PCP Reinforced Beam		$f_b > f_r$	$f_b < f_r$

Figure 1.1: Stress distribution in FRP reinforced, PCP reinforced and FRP prestressed beam

service loads. This assumption however, has to be theoretically and experimentally verified. Since FRP is already being used for prestressing, FRP prestressed concrete prisms (FRP-PCP) are believed to be feasible to manufacture.

1.3 Objectives and Scope

The objective of this investigation is to experimentally study the service load behaviour and ultimate strength of concrete beams reinforced with CFRP (Carbon fibre reinforced plastic) prestressed concrete prisms (PCP). The study will be particularly focused on the service load deflections and crack width of PCP reinforced prisms, and on their comparison with the corresponding characteristics of similar beams reinforced with either conventional

rebars or with non-prestressed CFRP bars.

The principal test parameter is the level of prestress that can be applied to prisms of relatively small dimensions without splitting them, and whether that level of prestress would be adequate to control cracking and crack width in PCP reinforced beams.

Although the main focus of this study is the serviceability behaviour of PCP reinforced beams, the investigation will also involve the study of the shear and ultimate flexural strength of the latter beams.

The experimental data will be used in conjunction with known principles of mechanics of concrete to develop suitable methods of analysis and design of PCP reinforced concrete beams. Methods for deflection and crack width calculations, and for determining the ultimate flexural strength will be developed. A preliminary attempt will be made to investigate the shear resistance of PCP reinforced beams.

The scope of the present study is limited to CFRP prestressed prisms and to the short term behaviour of PCP reinforced beams. Although prestress losses in the prestressed prisms were monitored for some time, it is not the objective of the present investigation to delve in any detail into the long term time-dependent behaviour of PCP reinforced beams. Similarly, the current study is not concerned with the behaviour of the interface of the CFRP bar and the prism or the prism and the surrounding concrete in the beam.

This thesis is organized in eight chapters. Chapter 1 defines the problem and states the objectives and scope of the investigation. Chapters 2 and 3 review the state-of-the-art of FRP materials and of prestressed concrete prisms, together with their applications.

Chapter 4 describes the experimental program, while chapter 5 presents the experimental data and results. Chapter 6 discusses the serviceability and ultimate limit states. Here, formulae for deflection calculation, crack width, cracking moment and ultimate moment are presented. Even though the test beams were designed to fail in flexure, nevertheless their shear behaviour is discussed. In Chapter 7, conclusions are drawn from the shear data, and a comparison of the behaviour of steel reinforced beams to that of PCP reinforced beams is made. Design formulae for PCP reinforced beams are developed, as well as expressions for deflection calculation of beams reinforced with FRP.

Chapter 2

State-of-the-Art

2.1 General

Corrosion of steel reinforced concrete structures has been a problem to civil engineers for several decades. They have developed a number of treatments for this problem including epoxy coating of steel reinforcement, cathodic protection, use of sealants, membranes, etc. Although each of these methods has succeeded in some situations, none of them is capable of eliminating corrosion of steel reinforcement completely. Complete elimination of corrosion may be only feasible if the steel reinforcement is replaced by non-corrosive materials, such as fibre reinforced plastics (FRP).

FRPs have superior performance under harsh environment. They are considered to be non-corrosive, light weight with high specific strength. These materials are also endowed with other features such as chemical and environmental resistance, electromagnetic impermeability and impact resistance (ACI 1995).

Fibre reinforced plastics have been used for decades in the aeronautical, aerospace, auto-

motive and other fields. Their use in civil engineering works dates back to the 1950's when glass fibre FRP bars were first investigated for structural use. But it was not until the 1970's when FRP was finally considered for structural engineering applications and its superior performance over epoxy coated steel was recognized. The first applications of glass fibre FRP were not successful due to its poor performance within thermosetting resins cured at high moulding pressures (Parkyn 1970).

Since their early applications, many FRP materials with different types of fibres have been developed. The fibres include aramid, polyvinyl, carbon and improved glass fibres. FRP products are manufactured in many different forms such as bars, fabric, 2D grids, 3D grids or standard structural shapes. In the following, the two major components of FRP materials, namely the fibres and the matrices, and their properties as well as the mechanical properties of the final product, i.e. the fibre reinforced plastic materials, will be discussed.

2.2 FRP Reinforcement

FRP reinforcement is a composite material consisting of matrix and reinforcing fibres. The fibres are stronger than the matrix but in order to provide the reinforcing function, the fiber volume fraction should be more than 10 % (ACI 1995). The mechanical properties of the final FRP product depend on the quality of the fibres, the fibre orientation, and the shape and volumetric ratio of fibres as well as the adhesion between the fibres and the matrix and on its manufacturing process. The latter is significant to point out because simply mixing superior fibres and matrix does not guarantee a quality product. Accordingly, FRP

products with nominally the same fibres, matrix and fibre volume ratio may differ in their final properties significantly.

2.2.1 Fibres for FRP Composites

The American Society for Testing and Materials defines fibres as “ elongated materials with aspect ratio of at least 10:1, maximum cross-sectional area of $5 \times 10^{-2} \text{ mm}^2$, and maximum transverse dimension of 0.25 mm”. Fibres used for manufacturing composite materials must have very high strength and stiffness, toughness, durability and preferably low cost. The performance of fibres is affected also by their length, cross-sectional shape and chemical composition. Fibres are available in different cross-sectional shapes and sizes. The best performance is expected from circular fibres because of their large interface surface. Some fibres have rectangular, hexagonal, polygonal, or even hollow circular cross-sections (Salamone 1996).

The following types of fibres are most commonly used for civil engineering applications: carbon, glass and aramid. Other types of fibres have been manufactured and some of those will also be mentioned here.

Carbon Fibres

Carbon fibres are produced from precursors such as rayon (cellulose), polyacrylonitrile or PAN, which are similar to fibres used for making textiles, and pitch obtained by the destructive distillation of coal (Hollaway 1978). The manufacturing process of PAN based

carbon fibre consists of oxidation, two stages of carbonization, which is preheating PAN fibre under tension in air at 250° C in order to avoid shrinkage and gain strength, and finally graphitization. During the latter process colour changes occur from white to yellow, to brown, and finally to black. Different properties of fibres result from different stages.

There are three grades of carbon fibre (Hollaway 1978):

- Type I is the stiffest with the highest modulus of elasticity (more than 310 GPa). Its tensile strength is from 1.5 to 2.1 GPa, and strain at failure reaches values from 0.6 to 0.7 %.
- Type II is the strongest fibre with tensile strength from 2.4 to 2.8 GPa, but its modulus of elasticity is lower than for Type I (from 220 to 240 GPa), while the ultimate strain is higher (from 0.8 to 0.9 %).
- Type III is the one with the smallest modulus of elasticity which ranges from 175 to 205 GPa, and its strength is between 1.9 and 2.6 GPa. This fibre has the highest strain at failure, 1.0 %, and is the cheapest of the three.

Carbon fibres are well known for their high modulus of elasticity, which is determined by arrangements of hexagonal layer networks. Rayon is used for manufacturing low modulus carbon fibre, graphite obtained by graphitization is used for the highest modulus of elasticity fibres. Usually fibres with higher modulus of elasticity are more brittle and suffer from low strain at failure, but their tensile strength and modulus are stable with changes in temperature (Malek and Saadatmanesh 1996).

Carbon fibres are readily available as bundles of parallel fibres, also known as continuous tow, chopped (6-50 mm long), and milled (30-300 μm long). Carbon fibre tows are sometimes woven into two dimensional fabrics. The major disadvantage of carbon fibre is its high cost which is 20 to 50 times (per unit weight) higher than E-glass (Malek and Saadatmanesh 1996). Another drawback of this type of fibre is potential galvanic corrosion which occurs when carbon composites come into prolong contact with metals. In the latter case the carbon serves as the cathode, and the metal as the anode, causing the metal to corrode away (Phillips 1989, Chung 1994). Therefore attention must be paid to the type of material used for fixing bars together, or to the type of compression reinforcement being used. Furthermore, the use of mixed reinforcement, i.e. longitudinal FRP reinforcement and diagonal tension steel reinforcement *may* accelerate corrosion of the steel stirrups, and may cause long-term performance problems.

Glass Fibres

Glass fibres are one of the most widely used fibres due to their economical cost compared to carbon and aramid fibres. They are produced by the rapid drawing of molten glass from an electrically heated furnace through platinum bushings at high speed (Hollaway 1989). The process is illustrated in Figure 2.1 (Phillips 1989).

These fibres have a three dimensional internal structure and are amorphous and isotropic. They exhibit elastic behaviour up to failure and under dry conditions they creep negligibly and retain their very high strength. During the manufacturing process, fibre surfaces are coated to help wetting by the resin and also to increase the bond between the fibres and the

Figure 2.1: Schematic diagram of glass fibre melt (according to Phillips 1989)

matrix. Because the fibres can be damaged very easily, the coating plays a protective role as well.

Since glass is a polar material, the surface of glass fibres has sub-microscopic voids which become harmful when exposed to high pH environments. When subjected to moisture, molecular layers of water coat the fibres which affect their bond with the matrix and which may cause premature rupture. Therefore, the fibres are generally being treated during the manufacturing process with organo-silane functional treatment (ACI 1995).

Another potential problem with this fibre type is “creep rupture” , which means that if a glass fibre is under sustained load larger than a certain threshold, it will fail. Figure 2.2 shows the reductions of strength with time under sustained load (Mallick 1993). Similar behaviour of glass composites in acid or basic environment is referred to as stress corrosion. In order to avoid this type of failure, tough matrices should be used (Phillips 1989).

There are several types of glass fibre:

- E-glass of low alkali content is used mainly with polyester and epoxy resins and has been used for more than 50 years in non-structural applications. Presently, approximately 80 to 90 % of glass fibres produced are E-glass because of its excellent corrosion resistance in most acids and its low cost.
- Z-glass has very high resistance to alkali attack and is usually used for reinforcing cements, mortars, and concretes.
- A-glass of high alkali content is gradually going out of production and is being

Figure 2.2: Reduction of tensile strength of E-glass fibres under sustained loads (Mallick 1993)

replaced by E-glass. This is due to the fact that it does not have good long term performance under high moisture conditions.

- S-glass has extra high modulus (20 to 25% higher than E-glass), high strength and high temperature performance. This type of glass fibre is the most expensive; therefore, it is used mostly in aircraft components and missile casings.
- C-glass has very high corrosion resistance to acids and is used mostly in chemical plants.
- D-glass is used where dielectric properties are of importance.

The strength of glass fibres depends on the form in which they are used. Freshly drawn fibres

have much higher strength levels, however, due to surface flaws this strength decreases 40 to 50%. The highest strength is achieved by continuous fibres and the lowest by chopped fibres (Malek and Saadatmanesh 1996). Glass fibre reinforcement products are manufactured in many different forms (ACI 1995):

- Rovings, the basic form of glass fibres, are continuous fibres. Grouping of number of strands is formed at the same time.
- Woven roving reinforcement is made from rovings. The orientation of weaves depends on the manufacturing process.
- Mats, which are two dimensional, are made of chopped strands randomly arranged on a continuous conveyor, where thermosetting resin is dusted on them. The resin is heated and set, and holds the mat together.
- Combined products are a combination of the above mentioned woven rovings and mats in different variations.
- Cloth is made from continuous strand filaments that are twisted and plied and then woven.

Aramid Fibres

The term aramid is coined from its chemical name of aromatic polyamide. It has the lowest specific gravity and the highest specific tensile strength among all fibres (Mallick 1993).

Aramid fibres are known under different trademark names such as Kevlar, introduced in 1970, and still the most commonly used in North America, Twaron and Technora.

In the production of FRPs the following aramid fibres are used: Kevlar 29, designed for high impact and damage tolerance, Kevlar 49, Twaron 1055, Twaron 2000, and Technora, an aromatic polyetheramide. Among other available Kevlar fibres there are Kevlar 129, with higher strength and toughness than Kevlar 49, Kevlar 149 with the highest tensile modulus, and Kevlar 69 and Kevlar 100 which are not commonly used in fibre reinforced plastics. Kevlar fibres have a linear stress strain behaviour, but when under bending, they exhibit a high degree of yielding on the compression side (Mallick 1988). As a results of their higher modulus of elasticity, Kevlar 49 and Twaron 2000 are the most often used.

Aramid fibres can be used by themselves, or combined with glass, or carbon fibres in hybrid form. This fibre is fire resistant (up to 425 °C), has good fatigue and chemical resistance as well as good resistance to creep rupture (ACI 1995). Due to its limited yield strain of 0.3 to 0.5% under compression, its use in applications where it may be subjected to high compressive or flexural loads is not recommended (Phillips 1989). Kevlar fibres generally suffer very little creep, with the least amount of creep being experienced by Kevlar 149 (Malek and Saadatmanesh 1996).

These fibres can be used within a range of temperatures -200 °C to +200 °C. However, at temperatures above +150 °C, it should not be used on a long term basis because of oxidation. Their chemical resistance to strong acids and bases is not very good, and they are susceptible to UV degradation (ACI 1995). Kevlar fibres tend to absorb water, and at

high moisture content they tend to crack internally which may cause longitudinal splitting of fibres (Malek and Saadatmanesh 1996).

Other Types of Fibres

Polyvinyl alcohol (PVA) fibres have very high strength and they originate from a thermoplastic polymer of the same name. PVA fibres are manufactured by the polymerization of polyvinyl alcohol of high degree, and they are rolled to provide strength and elasticity to the fibres (JSCE, 1993). The fibres do not have good heat or UV-radiation resistance and without additional agents, PVA is soluble in water (Banthia and MacDonald 1996).

Ultra-high-molecular-weight-polyethylene fibres are organic fibres, which are manufactured in the U.S.A. under the trade name Spectra. They are very light, and have high strength and low tensile elongation (Phillips 1989). UHMWPF are impact resistant and have been used in ropes, special canvas and woven textile. However, these fibres break when subjected to temperatures higher than $+130^{\circ}\text{C}$ (ACI 1995). Another drawback of them is their poor adhesion with resin (Banthia and MacDonald 1996).

The mechanical properties of typical fibres are summarized in Table 2.1. The data in this table is compiled from those given in JSCE (1993), Gill (1972), Pilato and Michno (1994), and (ACI 1995).

Table 2.1: Mechanical properties of fibres used for FRPs

Fiber type			Tensile Strength [MPa]	Young's Modulus [GPa]	Elongation [%]	Coeff. of Thermal Expansion [$\times 10^{-6}$]
Carbon Fibre	PAN	High Strength	3500	200 - 240	1.3 - 1.8	-0.1 to -1.2 (Longitudinal)
		High Modulus	2500 - 4000	350 - 650	0.4 - 0.8	7 to 12 (Radial)
	Pitch	Ordinary	780 - 1000	38 - 40	2.1 - 2.5	-0.9 to -1.6 (L)
		High Modulus	3000 - 3500	400 - 800	0.4 - 1.5	
Aramid Fibre	Kevlar 29		3620	82.7	4.4	Not Available
	Kevlar 49		2800	130	2.3	-2.0 (L), +59 (R)
	Kevlar 129		4210 (est.)	110 (est.)	—	Not Available
	Kevlar 149		3450	172-179	1.9	Not Available
	Twaron		2800	130	2.3	-2.0 (L), +59 (R)
	Technora		3500	74	4.6	Not Available
Glass Fibre	E-Glass		3500 - 3600	74 - 75	4.8	5.0
	S-Glass		4900	87	Not Supplied	2.9
	Alkali Resistant Glass		1800 - 3500	70 - 76	2.0 - 3.0	Not Available
Polyvinyl Alcohol	High Strength		2300	61	5.0	Not Available
Spectra	900		2600	117	3.5	Not Available
	1000		2900-3300	172	2.6	Not Available

Figure 2.3: Stress/strain curves for fibrous reinforcement and matrix

2.2.2 Matrices for FRP Composites

A very important issue during the manufacturing of composites is the selection of the proper matrix because the physical and thermal properties of the matrix significantly affect the final mechanical properties as well as the manufacturing process. By reference to Figure 2.3, in order to be able to exploit the full strength of the fibres, the matrix should be able to develop higher ultimate strain than the fibres (Phillips 1989). The matrix is not only coating the fibres and protecting them from mechanical abrasion, it also transfers stresses between the fibres. Another very important role of the matrix is transfer of interlaminar, as well as in-plane, shear in a composite and provision of lateral support to fibres against buckling when subjected to compressive loads (ACI 1995).

There are two types of polymeric matrices widely used for FRP composites: thermosetting and thermoplastic. **Thermosetting polymers** are used more often than thermoplastic. They are low molecular-weight liquids with very low viscosity (ACI 1995). Their molecules are joined together by chemical cross links (Figure 2.4), thus they form a rigid three dimensional structure which once set cannot be reshaped by applying heat or pressure. They are processed in a liquid state to obtain good wet-out of fibres. Some commonly used thermosetting polymers are polyesters, vinyl esters and epoxies.

These materials are of a good thermal stability and chemical resistance; they undergo low creep and stress relaxation. However, they have relatively low strain-to-failure, therefore resulting in low impact strength. One of their major disadvantages is their short shelf-life and long manufacturing time. Some more details are provided below.

Polyester: Polyester matrices are made of unsaturated polyester resin, some other agents are later added to reduce viscosity and prolong storage life. Polyester suffers larger shrinkage than epoxy, however by adding thermoplastic component this shrinkage can be reduced.

According to ACI (1995) 85% of U.S. composite production is based on unsaturated polyester resins. There are several types of unsaturated polyester resins available, however, only some of them have the superior qualities necessary for FRP production (ACI 1995). Specifically there exists

- **Orthophthalic Polyester.** This type of resin does not have the necessary mechanical properties to be used in FRP production. It has low strength, inferior moisture and chemical resistance, as well as low chemical stability.

- Isophthalic Polyester (Iso Polyester) is known for its excellent thermal resistance, good mechanical properties, superior chemical and moisture resistance.
- Bisphenol A Fumerates offer very good thermal and chemical performance.
- Chlorendics have excellent chemical resistance.

Epoxy: Epoxies are made of low-molecular weight organic liquid resins containing different epoxide groups. These contain not only carbon atoms as polyesters, but also one oxygen atom. Other materials are added to the basic chemicals to reduce viscosity and increase the impact strength. Hardners play an important role in the curing of epoxies. They initiate polymerization of epoxies, usually at elevated temperatures ranging from room temperature to +175 °C. During the polymerization process the epoxy changes to a solid material.

Epoxy resins are more expensive than other resins, but they exhibit superior performance, especially in their mechanical properties, low shrinkage, chemical resistance and very good bond to fibres and fillers. In addition to high cost, they have some other drawbacks such as prolonged time for curing and their average moisture resistance (ACI 1995).

Vinyl Ester: This resin is produced by the reaction of an unsaturated carboxylic acid with an epoxy resin. Vinyl ester has an excellent wet-out and bond, particularly with glass fibres and it has very good chemical resistance and tensile strength. Another advantage of this resin is its low viscosity and fast curing. A disadvantage of vinyl ester is its high volumetric shrinkage. Table 2.2 provides some basic data regarding thermoset resins.

Thermoplastic matrix polymers are made from molecules in a linear structural form.

Table 2.2: Typical properties of thermoset resins

Resin	Specific Gravity	Tensile Strength (MPa)	Tensile Modulus (GPa)	Cure Shrinkage (%)
Epoxy	1.2-1.3	55-130	2.75-4.10	1-5
Polyester	1.1-1.4	34.5-103.5	2.1-3.45	5-12
Vinyl Ester	1.12-1.32	73-81	3-3.5	5.4-10.3

Figure 2.4: Schematics of (a) thermoplastics and (b) thermosets (Mallick 1993)

These are held in place by weak secondary bonds, which can be destroyed by heat or pressure. After cooling these matrices gain a solid shape. Therefore, thermoplastic polymers can be reshaped by heating as many times as necessary. Numerous heating and cooling will, however, result in degradation of the mechanical properties of these polymers. A schematic structure of thermoplasts and thermosets can be seen in Figure 2.4 (Mallick 1993).

Thermoplastic matrices are used less than thermosetting due to their viscosity and problems with combining them with continuous fibres. Once these obstacles are overcome by new processes it will be possible to take advantage of their excellent properties such as high

Table 2.3: Comparison between thermoplastic and thermosetting matrices

	Thermosets	Thermoplastics
Costs	Low	Low/high
Prepreg production	Very good	Poor
Prepreg tack	Very good	No
Prepreg storability	Poor	Very good
Shrinkage	Little	Little
Fracture toughness	Low	High
Solvent resistance	Good	Poor-good
Crystallization problems	No	Yes
Recycling	Poor	Good
Consolidation time	High	Very short

impact strength, high fracture resistance, unlimited storage life, shorter molding cycles, secondary formability and easy handling (ACI 1995); they can be also repaired by welding and are recyclable (Malek and Saadatmanesh 1996).

For further comparison of thermoplastic and thermosetting matrices, reference can be made to Table 2.3 (Salamone 1996).

Since all properties of matrices are not favourable, fillers are usually added to both thermoplastic and thermosetting matrices. Fillers can improve fire and smoke performance, they help reduce shrinkage of resins, and increase water resistance, stiffness and dimensional stability. The most widely used filler is calcium carbonate, derived from limestone or marble. The second most commonly used filler is kaolin, also known as clay. Clay is mined and processed by air flotation or by water washing. Other fillers include glass microspheres,

flake glass, milled glass fibres, silica, and talc (Road Talk 1996).

The main advantages of using fillers are (1) improving fire resistance, (2) reducing cost of composites, (3) improving the mechanical and physical performance of composites (fillers help transfer of stresses between fibres and resin), (4) maintaining uniformity of laminates, (5) improving crack initiation and propagation (Road Talk 1996).

2.2.3 Durability of FRP Materials

In the following the resistance of FRPs to different environmental conditions is discussed. Since FRPs are composed of two substances, the resistance of both should be known. FRP can be damaged by one or a combination of the following: water absorption, thermal expansion, acids, alkalis, heat, UV-radiation, organic solvents, and oxygen and ozone. Banthia and MacDonald (1996) described the various factors which could affect the durability of the different types of fibres which may be summarized as follows:

Durability of Fibres

It is important to state that the following durability issues refer to unprotected fibres, and the basic materials from which they are made.

- **Glass Fibres (GF)**

Water Absorption will cause reduction of tensile strength and reduction of glass transition temperature, the temperature above which glass is no longer brittle.

There are many problems with glass in wet environment due to the growth of rigid hydration products around fibres, causing fibres to become brittle.

Coefficient of Thermal Expansion of various GFs is given in Table 2.1. Of all the fibres, the coefficients for GF are considered to be the closest to that of concrete.

Acids are considered to be damaging for E-glass, while S- or AR-glass are resistant to acids. The attack by acids severely reduces glass fibres tensile strength.

Alkalies are very dangerous for glass fibres. They will cause serious degradation of the fibre by breakdown of the silicon-oxygen-silicon bond which results in dissolution of molecular bonds within the fibre.

Heat does not affect the mechanical properties of glass fibres, unless extremely high temperatures are reached.

UV-Radiation is not destructive to glass fibres.

- **Carbon Fibres (CF)** are the most resistant of all the fibres and are resistant to all types of corrosion. Phillips (1989) mentions one possible degradation mechanism of carbon/epoxy laminates, i.e. thermal spike. A thermal spike is a sudden increase in temperature followed by a sudden decrease, which is reported to create voids and microcracks, which may be filled with moisture, leading to decreased fatigue life of carbon fibre epoxy-matrix composites (Phillips 1989, Chung 1994). Another problem, as mentioned earlier, is galvanic corrosion, which occurs when carbon fibre bars come in contact with steel (Phillips 1989, Chung 1994).

- **Aramid Fibres (AF)**

Water Absorption will not affect the tensile strength of these fibres. There is not much information about the effect of water sorption on shear or compressive strength. Flexural strength, determined by compressive buckling, is dependent on moisture content (Morgan and Allred 1993).

Coefficient of Thermal Expansion of AF is very different from that of concrete.

AF actually shrinks in axial direction with rising temperatures and expands in transverse direction. In an oxygen environment when heated at a rate of 10 °C/min, Kevlar 49 fibres will completely disintegrate at 700 °C (Morgan and Allred 1993).

Acids when they are weak they are not damaging to aramid. Only strong acids will cause up to 30 % reduction in their strength.

Alkalis are considered damaging only if they are strong. In that case they will cause up to 10 % reduction in aramid fibre strength. Otherwise they will not affect its performance.

Heat has been shown to have an effect on reduction of tensile strength of aramid fibres. Up to 150 °C they can regain their strength after cooling, but when exposed to temperatures higher than this, they will lose up to 80 % of their strength.

UV-Radiation will cause aramid fibres to become brittle, and to experience 10 % loss of tensile strength. However, UV-induced fibre aging will occur only in the top layer of the composite (Morgan and Allred 1993).

Organic Solvents will not cause damage to these fibres.

Oxygen and Ozone may attack aramid fibres. However, exposure to oxygen rich air under ambient conditions does affect the properties of aramid.

- Polyvinyl Alcohol Fibres are used mostly in Japan. There is not enough information in the open literature regarding their durability.

Heat will cause reductions in strength and modulus of rupture. At temperatures over 200 °C the fibres will rapidly loose strength.

- Ultra-High-Molecular-Weight-Polyethylene Fibres are considered to be resistant to acids, alkalis and organic solvents. Results from other tests are not available since these fibres have been developed more recently.

Table 2.4 (JSCE 1993, Banthia and MacDonald 1996) gives the performance of fibres in the above mentioned damaging environments. The acids used include hydrochloric acid, sulfuric acid, and nitric acid. Alkalis include sodium hydroxide, and brine. Organic solutions considered were acetone, benzene, and gasoline. As already mentioned, carbon fibres were found to have the best performance of all tested fibres.

Durability of Matrices

Resins are usually less durable than fibres. Deterioration in plastics manifests in three forms: surface reaction, layer formation, and intrusion (JSCE 1993). They may be collectively referred to as corrosion of resin, and the first two types may influence the bond between the

Table 2.4: Chemical resistance of fibres (JSCE 1993)

			Acid Resistance	Alkali Resistance	Organic Solvent Resistance
Carbon	PAN	High Strength	good	excellent	excellent
		High Modulus	excellent	excellent	excellent
	Pitch	Ordinary	excellent	excellent	excellent
		High Modulus	excellent	excellent	excellent
Aramid	Kevlar 49		poor	good	excellent
	Technora		good	good	good
Glass	E-Glass		poor	fair	excellent
	S-Glass		good	poor	not available
	Alkali Resistant Glass		good	good	not available
EC-Polyethylene			excellent	excellent	excellent
Polyvinyl Alcohol Fibre			good	good	good
Steel Fibre			poor	excellent to sodium poor brine resistance	excellent

concrete and the FRPs. All three progress usually very slowly; the surface reaction type is expected to grow about 1.32 mm in 10 years, and about 6.6 mm in 50 years (JSCE 1993). According to the same source, the corrosion layer formation is expected to grow 0.37 mm in 10 years, or about 0.82 mm in 50 years, and the third type, intrusion, is believed to be even slower. This will change with change of conditions, such as high temperature or stresses.

- **Polyester Matrix**

Water Absorption is a problem with this matrix. If polyester matrices absorb water, this will cause swelling and leaching, and possibly cause chemical attack.

Acids will be very harmful if they are strong. It will cause hydrolysis of the ester linkages in the polymer chain. However, weak acids are considered to just slightly attack these matrices.

Alkalis cause hydrolysis of ester linkages with polymers. The weak alkali resistance is one of the major drawbacks of these matrices.

UV-Radiation will cause yellowing of the resin surface.

Organic Solvents will cause a slight damage to this matrix.

Oxygen and Ozone will harm polyester matrix.

- **Vinyl Ester Matrix**

Water Absorption is 0.1-0.2 % by volume per 24hr

Acids will not damage vinyl esters because ester linkages are protected from hydrolysis by pendant methyl groups in the polymer chain.

Alkalis will not harm vinyl esters.

Heat is considered to be harmful to this matrix. The heat-distortion temperature for this material is 120 to 150 °C. When heated to 250 °C, 40 % strength loss will occur. If heated to higher temperatures, the strength loss will be much higher, up to 60 %.

UV-Radiation is considered to slightly attack this material.

Organic Solvents: Because of some pendant groups in this polymer most organic solvents will not disturb its molecular structure. Therefore, vinyl esters are considered resistant to organic solvents.

- **Epoxy Matrix**

Water Absorption is 0.1 % by volume per 24 hr

Acids when they are weak will not damage epoxy. Epoxy matrix is slightly attacked by strong acids.

Alkalis have no effect on epoxy matrix.

Heat when significantly high, heat will cause strength loss in epoxy resins. Minor changes in molecular structure of epoxies can produce heat resistant material.

UV-Radiation: epoxies are considered to be ultra-violet light resistant.

Organic Solvents will not cause any damage to this matrix.

Oxygen and Ozone will slightly attack epoxy matrix.

2.2.4 Mechanical Properties of FRP

Standard methods for testing the mechanical properties of FRPs used in civil engineering have not been developed yet, albeit efforts are currently underway. These properties depend on many parameters such as type of resin, type of fibres, volumetric ratio of fibres, as well as duration and history of loading. In this section the mechanical properties of FRPs relevant to the proposed research are discussed. For more detailed information refer to the ACI (1995), and JSCE (1993).

Tensile Strength and Elastic Modulus

FRPs are known for their elastic behaviour up to failure. The tensile strength of FRP bars depends on the bar diameter (ACI 1995). Due to shear lag, the outer fibres are stressed more than the inner ones, hence bars with larger diameter have proportionally smaller tensile strength (Faza 1991). Note, however, that with the exception of some GFRPs, the tensile strength of FRPs is much higher than that of steel.

The modulus of elasticity of FRPs is in general smaller than the modulus of elasticity of steel, thus FRP reinforced concrete undergoes larger deflection. Of course, prestressing concrete will overcome the problem of deflection under service loads. Some FRPs, such as CFRP, could have a modulus equal to steel or even higher, however, these products are more costly and therefore used only in structures with special needs.

Compressive Strength and Elastic Modulus

The compressive strength of FRPs is one of the mechanical properties about which little information is available. This is mostly due to the fact that in general they have relatively low compressive strength and that testing procedures are not unified. For example, it is well known that aramid bars do not behave well in compression (Bedard 1992, Chaallal and Benmokrane 1993). According to Kobayashi and Fujisaki (1995) the compressive strength of AFRP bars is in the range of 10 % of their tensile strength, for CFRP 30 to 50 % of their tensile strength and for GFRP 30 to 40 % of their tensile strength. Chaallal and Benmokrane's tests of GFRP bar with 73-78 % E-glass fibre showed compressive strength of GFRP bars to be approximately 80 % of their tensile strength, which appears to be rather high and is not in accordance with previous findings. This type of disparity is quite usual for FRP because there are many different types of products which differ in volumetric ratio of fibres, matrix type, and manufacturing process. Hence there is need for a unified testing method for FRP materials.

The compressive modulus of elasticity depends on length-to-diameter ratio, bar size and type, as well as on some other properties. Several researchers have reported results from compression tests, and it is generally agreed that the compressive stiffness, is smaller than the tensile stiffness ranging from 77 to 97 % of the tensile stiffness (Bedard 1992, Chaallal and Benmokrane 1993).

Thermal Characteristics

FRPs are known to have a coefficient of thermal expansion (α) that is incompatible with the coefficients of thermal expansion of concrete. The thermal incompatibility will produce thermal stresses in FRP reinforced or repaired concrete structures. In ordinary steel reinforced concrete this problem does not occur because both steel and concrete have a coefficient of thermal expansion in the range of $+10$ to $+12 \times 10^{-6}/^{\circ}\text{C}$. For aramid the range is between -6 to $-2 \times 10^{-6}/^{\circ}\text{C}$, for carbon fibre bars it is approximately $+1 \times 10^{-6}/^{\circ}\text{C}$, while for glass fibre bars it is $+10 \times 10^{-6}/^{\circ}\text{C}$. This difference may lead to thermal stresses and debonding of FRP bars from concrete after experiencing changes in temperature. Experimental and analytical work of researchers at NRC indicated that, due to the Canadian climate, there is a possibility of cracking around some FRP bars, but further work is needed to fully quantify the actual stress caused by temperature variations (Rahman et al. 1995). For more information regarding specific fibre type, refer to Section 2.2.3.

Creep Properties

FRP reinforcement in general does not suffer from large creep deformations. In glass fibre FRP, the highest creep is expected from chopped strand mat with short fibres. The least creep is expected from unidirectional reinforcement. The overall creep behaviour depends on the creep resistance of the matrix (Phillips 1989). The creep performance of carbon fibre composites is very good. When the load is tensile, composites resist long-term creep, but if it becomes compressive, or multi-directional, the creep properties are not easy to predict

and may need to be determined experimentally (Phillips 1989). Aramid fibre composites have superior creep performance to both glass and carbon composites, while the creep performance of carbon fibre composites may be considered good.

2.3 Applications of FRP

In the previous sections the mechanical properties of FRPs and their components were discussed. In this section, practical applications are shown. The review of FRP applications is not comprehensive; for greater detail reference can be made to other sources (ACI 1995, JSCE 1993, Mufti et al. 1991).

2.3.1 FRP Products on the Market

FRPs in civil engineering applications can be classified according to their raw materials, or shapes, as indicated in Figure 2.3.1 (JSCE 1993).

Reinforcing Bars

FRP reinforcing bars are manufactured from continuous fibres (carbon, glass, aramid etc.) embedded in matrices (thermosetting or thermoplastic). Similarly to steel reinforcement, they are produced in different diameters, depending on the manufacturing process. The surface of the rods can be spiral, straight, sanded-straight, sanded-braided, deformed, and 7-piece stranded (Yamashi et al. 1993). The bond of these bars with concrete is equal to, or

Figure 2.3.1: Classification of Continuous Reinforcing Materials (JSCE 1993)

better than, the bond of steel bars.

The main manufacturers of FRP reinforcement are in Japan (Mitsubishi Chemical, Mitsui Construction and many others). The most widely used bar, manufactured by Mitsubishi Chemical, has the trade name Leadline. It is a carbon fibre reinforcement with strength reaching up to 3 000 MPa (depending on bar diameter). Since there is a number of FRPs available on the market, Table 2.5 provides information about some of the more common reinforcing bars. The table gives data directly provided by the manufacturers and includes products manufactured all over the world: Europe, USA, Canada and Japan.

As can be seen, carbon fibre bars provide the highest strength, as well as the highest modulus of elasticity. The values published in the table are typical, actual values for different bars will vary with bar size and manufacturing technique.

Prestressing Tendons

FRP prestressed structures are more widely used than FRP reinforced structures due to ease of deflection control even when FRP with lower modulus of elasticity is used. This lower modulus is in fact useful for prestressed structures because prestress losses due to creep and relaxation are smaller.

Table 2.6 summarizes the principal mechanical properties of some available prestressing tendons. Many researchers have reported problems with slipping of the tendons during the prestressing process. These problems have led to a new area of research dealing with anchors for non-metallic prestressing tendons (Holte and Dolan 1993, Faza 1991). More

Table 2.5: Typical mechanical properties of currently available FRP reinforcing bars

Trade Name	Tensile Strength [MPa]	Young's Modulus [GPa]	Max. Elong.
Carbon Fibre			
Leadline	2260	147	0.015
Hipurr	1640	127	0.013
FiBRA	1500	120	0.0125
Bridon FTM	1610	136	N.S.
Glass Fibre			
Jitec	1600	50	N.S.
Isorod	690	42	0.018
Magnum 9000	655-900	50-60	N.S.
C-Bar	770	37	0.0205
Aramid Fibre			
FiBRA	1500	68	0.022
Bridon FTM	1590	64.3	N.S.
PVA Fibre			
Clatec Rod	784	29	0.034

Table 2.6: Typical mechanical properties of currently available FRP prestressing bars

Trade Name	Tensile Strength [MPa]	Young's Modulus [GPa]	Max. Elong.
Carbon Fibre			
Leadline	2260	147	0.015
CFCC	2120	137	0.0155
Eskainos	1700-1800	410-460	N.S.
Hipurr	1800	137	0.013
Aramid Fibre			
FiBRA	1500	68	0.022
Technora	1800	54	0.036
Arapree	1333	50	0.022
Glass Fibre			
Polystal	1800	53	0.033
Polygon	1790	56	0.031

detailed information on this subject can be found in Chapter 4, and in ICCI (1996).

Sheets, Laminates and Grids

Sheets and laminates are usually used for retrofitting. They have proven very useful in repairing columns, chimneys, bridge-girders and other structures deteriorated by either extensive use or environmental conditions. They have also been used to retrofit structures for increased strength and ductility, especially seismic retrofit. Another important application is in the area of retrofit of masonry structures. Historical masonry structures in seismically active regions can be upgraded for increased resistance and ductility. The main advantage of FRP over conventional steel plates in repair work is their light weight, high strength and ease of application. Meier and Kaiser (1992) reported that 94 kg of steel plates for strengthening one particular structure were replaced by 4.5 kg of CFRP laminate. This reduced the weight of the structure as well as the labour cost.

Grid reinforcement is used for reinforcing slabs, columns, beams, concrete sidewalks, or even arches (as reinforcement for shotcrete). Different laminates and grids are available on the market and the mechanical properties of some of them are listed in the Table 2.7.

The following sections will introduce research in the area of FRP reinforced and FRP prestressed structures. The literature survey was carried out for the purpose of investigating mainly the problem of deflection of FRP reinforced structures.

Table 2.7: Typical mechanical properties of currently available FRP sheets and grids

Trade Name	Tensile Strength [MPa]	Young's Modulus [GPa]	Max. Elong.
Carbon Fibre			
Replark	3430	235	0.014
Forca Tow Sheet	2940-3430	230-372	0.008-0.015
NEFMAC	1200	100	0.015-0.018
CF Mesh	900	80	0.012
Glass Fibre			
NEFMAC	600	30	0.02-0.024
Aramid Fibre			
NEFMAC	1300	57	0.021-0.025

2.3.2 FRP Reinforced Concrete

FRPs are usually used as tension or shear reinforcement. The mechanical properties of this material are different from steel; therefore, every potential practical usage has to be first tested experimentally. Research in the area of FRP reinforced concrete is concentrating on three different areas: flexural behaviour, shear behaviour, and the bond properties of FRP reinforcement. Additional studies must be undertaken to study creep, relaxation and fatigue properties of FRP reinforced structures. For the purpose of this thesis, however, mainly deflection of FRP reinforced concrete structures is considered. For reference on shear, bond or other general behaviour of FRP reinforced concrete, refer to ICCI (1996), ACI (1995, 1993), JSCE (1993).

Nawy et al. (1971) tested 20 simply supported GFRP reinforced beams to observe their

cracking behaviour, deflection and ultimate capacity. Five series of beams were tested with four beams in each series. Two of the beams were GFRP-reinforced, one was steel reinforced, and one was FRP reinforced with its concrete mixed with chopped steel wire. The cracking moment was observed not to be affected with different type of reinforcement. Deflections of GFRP-reinforced beams were up to 3 times larger than the deflections of steel reinforced beams at the same load level after cracking. In most cases of FRP reinforced beams the failure was initiated by compressive failure of concrete.

The previous work initiated another research by Nawy and Neuwerth (1977), on beams and slabs. The slabs were restrained on all four sides and their deflections under service loads were reported to be within existing code limitations. Their cracking pattern depended on reinforcement ratio, where the lower ratios resulted in relatively wide yield line pattern while the higher ratios led to narrow diagonal patterns. This indicated that sections with lower reinforcement ratios will fail suddenly, without any warning signs. The same behaviour was observed during the test of beam specimens. The beams with lower reinforcement ratio developed fewer cracks and as a result the rate of stiffness degradation as well as the rate of increasing deflection after cracking was faster. These beams had no shear reinforcement.

Saadatmanesh and Ehsani (1991) tested six beams longitudinally reinforced with GFRP or steel. The investigation focused on testing GFRP as longitudinal and transverse reinforcement. Two types of shear reinforcement were used, steel and GFRP. The beams with steel stirrups failed by compression failure of concrete, whereas the GFRP stirrup beams failed by crushing of concrete after steel started to yield. None of them failed in shear which

means that FRP was capable of resisting shear stresses. Of course the amount of shear reinforcement provided was high compared to the theoretically required amount. Deflections of the GFRP reinforced beams were reported to be four times larger than the corresponding deflections of steel reinforced beams. The necessity of regarding deflections as a limiting design factor was suggested.

Bending and bond behaviour of glass fibre plastic reinforced concrete beams was studied by Faza (1991). His experimental work included tests of twenty seven beams subjected to bending, twenty four cantilever specimens subjected to bond forces, and twelve pull-out specimens. The beams were cast from concrete with different strengths ranging from 29 to 69 MPa. Glass fibre plastic rebars were of different sizes as well as types (smooth, ribbed, and sand coated). The pre- and post-cracking behaviour, load-deflection, stress distribution, and modes of failure of FRP reinforced beams were studied. An increase of ultimate moment capacity by 90 %, as well as increased ductility was reported when high strength concrete ($f'_c = 52$ MPa) was reinforced with high strength rebars. The increase was relative to normal strength concrete ($f'_c = 34.5$ MPa) and using the same type and amount of GFRP reinforcement (2 # 3 GFRP rebars). When smooth bars were used, bond failure occurred prior to reaching the ultimate moment capacity.

Deflections of GFRP reinforced beams were around four times larger than deflections of a similar steel reinforced beam after cracking. This was mainly due to the fact that GFRP had a modulus of elasticity of 48 GPa compared to the 200 GPa modulus of steel. Therefore, after cracking the flexural stiffness of the beam decreased substantially, and the deflections

increased rapidly. The difference between using regular and sand coated bars affected the crack distribution, and also increased the ultimate moment capacity. Contrary to this finding, Nagasaka et al. (1990) reported only slight improvement of flexural behaviour of beams reinforced with either braided, or sand coated bars. This difference might be attributed to the fact that while Faza was testing beams reinforced with glass fibre rods, Nagasaka's beams were reinforced with carbon fibre reinforced rods.

Schmeckpeper (1992) tested a series of beams reinforced with an FRP mesh, known as NEFMAC. All tested beams failed either in compression or shear. They behaved similar to over-reinforced beams with mild steel reinforcement. The deflections of these beams after cracking were almost twice the deflections of steel reinforced beams. Comparison of deflections computed according to ACI Code and the experimental deflections was performed. After cracking occurred, the calculated deflections were only half of experimental deflections. Thus the use of ACI equations would underestimate deflections throughout the entire range of loading after cracking.

Daniali (1992) tested the development length of GFRP reinforced beams, and one of his finding was that the bond of GFRP bars of larger diameter is poor (8 mm diameter). The low bond strength combined with the low modulus of elasticity of GFRP rebars result in large deflections at serviceability limit state.

Alsayed et al. (1995) tested the flexural behaviour of beams reinforced with GFRP. Two groups of GFRP reinforced beams were tested and compared to steel reinforced beams. The first group was designed as over-reinforced such that their flexural capacity equaled that of

the steel reinforced beams. This group had the same cross section and effective depth as the steel reinforced group. The second set of GFRP reinforced beams was designed according to the so-called optimum design requirements. This resulted in 40 % increase of flexural capacity, 34 % decrease of deflections at service loads and overall 40 % cost reduction. However, the second set of beams did not have the same cross-section, it was 24 % deeper than the first set. The amount of reinforcement was decreased by 55 %, and the internal lever arm was in fact increased by 25 %. Accordingly they were comparing two completely different sets of beams. The increase in the flexural capacity was obviously due to the increase in the internal arm and the overall size of cross section. For the FRP reinforced beams, formation of the first cracks led to noticeable increase in curvature, causing larger deflection. When the height of the cross-section was increased, it changed the occurrence of first cracks, and therefore postponed the negative effect of cracking. However, for proper comparison the main parameters must remain the same during the experiment.

Buyle-Bodin et al. (1995) tested JITEC rebar, made of E-glass fibres embedded in vinylester matrix as reinforcement for concrete beams. The test results were compared with calculation models. From the experiments it was concluded that the deflections of GFRP reinforced beams were three times higher than deflections of steel reinforced beams. They also concluded that it was possible to use classical reinforced concrete approach to predict the behaviour of GFRP reinforced beams in flexure.

Benmokrane et al. (1996) tested two series of 3.3 m long simply supported FRP reinforced concrete beams under four point bending. The experimental results were also theoretically

investigated. The behaviour of the two sets of GFRP reinforced beams was compared to the behaviour of steel reinforced beams. The cracks in GFRP reinforced beams beyond service loads were reported to be wider and the deflections after cracking were two to three times larger than deflections of the steel reinforced beam, which is in agreement with previous findings. The bond between the GFRP bars and concrete was demonstrated to be very good.

Another experimental program of Benmokrane and Masmoudi (1996) included four series of reinforced concrete beams in flexure. Each series included two identical beams reinforced with C-BARs. The main purpose of this experiment was to investigate the effect of reinforcement ratio on the load-deflection response and ultimate moment capacities, and also to test the behaviour of this new reinforcing bar. The reinforcement ratio varied from 0.56 to 2.15 %. If the maximum deflection of beams with 0.56 % reinforcement ratio is compared to the maximum deflection of beams with 2.15 % reinforcement ratio, it would be seen that the latter is half of the former. However, the latter would still be larger than the code limit at serviceability (for 3 m long span beam the limit is 8.4 mm, compared to 12 mm deflection obtained in the experiment). Therefore, it can be concluded that even tripling the amount of reinforcement in some cases will not solve the problem of deflections of FRP reinforced beams.

Experimental study of the behaviour of slabs under static four point bending is reported in the work of Abdalla et al. (1996). They observed the pre- and post-cracking behaviour, ultimate capacity and mode of failure of the slabs reinforced with FRP in order to propose design guidelines for deflection and cracking calculations. In this section, only the experimental

results are mentioned. Six slab specimens with different reinforcement ratios were tested. Five specimens were reinforced with GFRP and one with CFRP. The FRP reinforced slabs cracked under smaller load than the steel reinforced ones, which is contrary to the results of FRP reinforced beams. This was explained to be due to the fact that the bond between the bars and concrete was not adequate. The post-cracking behaviour of FRP reinforced slabs was reported to be different, and according to these researchers, FRPs are not feasible substitutes for direct replacement of steel without further investigation. This work is one of the few regarding the behaviour of FRP reinforced slabs.

A limited number of investigations on the long term deflection of FRP reinforced beams was revealed by the literature review. Brown and Bartholomew (1996) studied long-term deflections of GFRP reinforced concrete beams. Four beams were tested, two steel and two GFRP reinforced over a period of eight months. The GFRP reinforced beams deflections were nearly four times the steel reinforced beams deflections, however, the creep deflections were only 1.8 times larger. It should be noted also that even though the steel reinforced beam experienced creep, deflection stabilized after 10-15 days, the GFRP creep deflections were still increasing after 50 days. This might be due to the fact that the maximum compressive stress of GFRP reinforced beams was 2.24 times larger than the corresponding steel reinforced beams.

Kage et al. (1995) testing program focused on deflection control and serviceability limit state of continuous fibre reinforced concrete beams under long term loading. Test specimens were 1.8 m long simply supported beams loaded under four point bending. The loads were

applied for a period of one year. Ten beams were tested, reinforced with one of the following reinforcement type: aramid bar, braided carbon bar, spiral carbon bar, glass bar and steel. The reinforcement ratio of the beams was from 0.92 % to 0.94 % for tension reinforcement, and 0.6 % for shear reinforcement. Curvature of the beams was measured at the beginning of the long term deflections and after one year. Overall curvature of the FRP reinforced beams was much larger than the curvature of the steel reinforced beams. However, after one year period FRP reinforced beams experienced 19-25 % increase in curvature, whereas the steel beam underwent 32 % increase in curvature. This means that the long term deflections of FRP reinforced beams increased less than the long term deflections of steel reinforced beam. The authors acknowledged that there are still some problems concerning predicting long term behaviour of FRP reinforced concrete structures, mainly due to creep of FRP rebars.

It can be concluded from the literature review that researchers are mostly using GFRP rebars for reinforcement of concrete beams, partly due to their cheaper price compared to higher modulus carbon fibre reinforced plastic bars. Also it would be impossible to use the full strength of CFRP bars as ordinary reinforcement for concrete. All of the above mentioned works have revealed that deflection of FRP reinforced beams is two to four times larger than the corresponding deflection of steel reinforced beams. Therefore, the need for reducing deflection, combined with the possibility of reducing the amount of the reinforcement used in the cross section, would be the key to the successful use of FRP bars for concrete reinforcement. This would also directly reduce the cost of construction.

Chapter 3

Prestressed Concrete Prisms

3.1 General

In terms of serviceability of FRP reinforced concrete structures, large deflection is considered to be one of the most important issues. The remedy for this problem is usually sought by increasing the amount of FRP reinforcement in the section. This is costly because of the relatively high cost of FRP. If deflection control is achieved through higher reinforcement ratio, the most important feature of FRP, namely their high tensile strength, would not be utilized in design. Of course prestressing concrete with FRP could solve the problem of large deflections and could use their full ultimate strength, but it is not always economical or feasible to prestress concrete structures. Therefore, in order to decrease deflections in FRP reinforced concrete structures, alternative strategies should be explored. This chapter discusses one such strategy and provides details of its feasibility.

The concept of prestressed concrete prisms is introduced in this chapter. Prestressed concrete prisms are concrete bars cast of high strength concrete concentrically pretensioned

by either steel or FRP tendons. Prestressed concrete prisms with steel strands were first used 40 years ago in Germany by Fensterwalder for reinforcing concrete pavements (Dyckerhoff and Widman 1963). Mikhailov (1958) mentioned the use of these bars for reinforcing water tanks. The bars were intended to serve as means for delaying cracking of the water tanks. Armin Hoppe (1963) discussed the advantages of PCP reinforced concrete over reinforced and prestressed concrete. At that time, he suggested several uses for the bars. They were intended for use in lightweight concrete, as longitudinal reinforcement in pressure pipes, vertical reinforcement for tanks and transverse reinforcement for floors and roads. As one can see, more than thirty years ago a number of possible usages for the bars were identified. This thesis is concerned with concrete beams reinforced with FRP prestressed prisms. Basic design principles for PCP are discussed here. A simple example, comparing PCP reinforced and FRP reinforced beams will also be presented.

3.2 Steel Prestressed Concrete Prisms

One of the early works dealing with this subject in some detail is by Hoppe (1963). The bars described by Hoppe were 4.88 m or 6.1 m long and had a trapezoidal cross-section of 25.4 mm $\times$ 34.0 mm and height of 40.0 mm, or 31.0 mm $\times$ 40.6 mm with height of 50.0 mm. In the second half of 1960, 39 624 linear metre of 1 290 mm² by 2.74 m long PCPs were produced for use in prestressed pressure pipes. However, in the late seventies the use of the bars most likely stopped because no further reference is made to the production of such bars in the literature. In the following, experimental work reported from 1965 until

now is described.

Burns (1966) reported results of experimental tests carried out at The University of Texas designed to evaluate the effectiveness of PCP in controlling cracking of reinforced concrete beams. Two types of test specimens were tested; double cantilever and two-span girder precast prestressed concrete beams. Concrete strength of the precast beams was 37 to 49.5 MPa, and they were prestressed by five 9.5 mm diameter steel strands. The strength of the cast-in-place slab concrete varied from 26 to 39 MPa. The slab was reinforced with rods, combination of rods and PCP, or PCP only. The concrete of the prisms was higher quality than the beam concrete, 36 to 51.5 MPa.

The researchers investigated the load capacity, cracking pattern and deflection of beams with different reinforcement arrangements. Loads were removed two to three times to observe crack width after unloading. The cracks closed almost completely following unloading, with very little residual deflection when the load was removed completely. The load capacity of the beams increased with addition of PCP, and also the cracking moment increased, reaching 50 % of the ultimate moment. It can be concluded from this research that PCP can be used for crack control.

Experimental work using PCP reinforcement for T-beams is reported by Hanson (1969). The tests included three combinations of reinforcement: deformed bars, deformed bars and PCP, and deformed bars and prestressing steel. The prisms were either 38 mm × 38 mm or 51 × 51 mm in cross section and contained 4.11 mm wire or 6.35 mm prestressing strand. The test included seven beams with three different cross-sections tested under negative

moment over central support (double cantilever). The cracking moment, crack-width, deflection and ultimate strength of beams with identical cross-sections were compared.

The presence of PCP increased the first cracking moment of the beams. Also the crack width in beams reinforced with PCP was reported to be smaller. This may be contributed to the fact that the PCPs introduced some amount of compression in their vicinity. Hanson noted that the PCPs were ready for field trial in the USA.

Research on PCP was carried out as well in the North Carolina State University. Mirza et al. (1971) reported a study on static and fatigue strength of beams reinforced with PCP. Nineteen beams were tested, reinforced with 50 mm² tension element. These tests investigated the effectiveness of PCP as continuity reinforcement in a highway bridge and found it to be superior to ordinary reinforcement. They also reported increased cracking load, thus improved corrosion resistance.

Zia et al. (1976) tested static and fatigue behaviour of composite T-beams reinforced with combination of PCP and reinforcing bars. They concluded that PCP as tension reinforcement is very effective for control of cracking and deflection.

More recent research work involving PCP was reported by Chen and Nawy (1994). They tested the flexural behaviour of high-strength concrete beams reinforced with PCP. Thirteen simply supported beams under third point bending were tested up to failure. The beams had rectangular cross section and were reinforced with one to three PCP. The prestressed concrete prisms were cast from concrete with compressive strength of 70 to 90 MPa and pretensioned by 1900 MPa seven-wire 9.5 mm diameter steel strands. Prisms were produced

in four sizes 38.1×76.2 mm, 38.1×101.6 mm, 76.2×76.2 mm, and 114.3×76.2 mm. The latter prisms had two prestressing strands while the other size prisms had only one strand. Initial prestressing of the PCP was around 1100 to 1400 MPa with prestressing losses of 30 to 36 %. The beams were cast right after the release of the prestress to the prisms. The load-deflection characteristics, as well as cracking and ultimate strength, were observed.

Load-deflection relationship of beams reinforced only with PCP was different than the load-deflection relationship of beams without PCP. The often-observed trilinear relationship of regular reinforced concrete beams changed to a bilinear relationship in the presence of PCP reinforcement. Usually after cracking of the extreme tension fibers, the slope of load-deflection curve changed due to the reduction in the flexural rigidity of the beam. In the beams reinforced with PCP, after cracking of cover concrete a slight or no deviation from the original slope was observed and the cracking moment capacity increased. When the prisms reached their cracking capacity, the flexural rigidity of the beams decreased severely. The load-deflection relationship was greatly affected by the relative ratio of the mild tension steel amount to the area of prisms.

The cracking capacity of the beams was affected by the total effective prestressing of prisms. However, increasing effective prestressing reduced the ductility of the beam. Provided that the total prestressing remained constant, the cross-sectional area of the prisms, or the number of prisms used did not affect the cracking capacity of the beam.

A theoretical and experimental investigation of T-beams reinforced with PCP was reported

by König et al. (1996). The PCPs used had cross-section of 40 by 40 mm, and were pretensioned by 7-wire strand of tensile strength of 1700 MPa. Since the bars were prestressed at very early age, very high early strength mortar, reinforced with steel fibre to increase its tensile strength, was used. The compressive strength of the mortar was 130 MPa, and its tensile strength was 17 MPa in 7 days. The mortar was designed to have very small shrinkage and creep. The design of concrete members reinforced with PCP was performed in such a way that the tensile stresses at service loads did not exceed the pre-compression in the PCPs.

They also studied the transfer behaviour of PCP. The transfer length varied from 100 to 350 mm. It was concluded that there was no problem with bond between PCP and concrete, provided the prisms had a surface rough enough to ensure sufficient bond with surrounding concrete. From the investigation of bond between concrete and the steel bar, the weakest bond was observed on smooth steel strands, followed by profiled wires 7 mm and deformed wires 8 mm, with bond strength of 4.5 MPa, 13.9 MPa and 23.5 MPa respectively.

To verify the theoretical background, tests on simply supported T-beams were performed. The beams had a clear span of 6.4 m and were tested under four point bending. The load deformation behaviour plus the cracking behaviour was investigated. Behaviour of beams reinforced with PCP was compared with the behaviour of ordinary reinforced beams and prestressed concrete beams. No reduction in ductility, compared to ordinary reinforced concrete, was observed because regular strength concrete was used to cast the beams. Contrary to the findings of Chen and Nawy, these beams showed a trilinear load-deflection

curve. However, the slope of the middle part of the curve was steeper than the corresponding slope for ordinary reinforced beams. The trilinear shape versus the bilinear shape observed by Chen and Nawy might be due to the fact that these beams were not cast from high strength concrete. The cracking load of the beams was increased when PCP was used. It can be concluded from the comparison of the three different types of reinforcement that the PCP reinforced beams behaviour lies between those of prestressed and ordinary reinforced concrete beams. Therefore, in cases that regular prestressing is non-economical or not practically feasible, one could improve the behaviour of ordinary reinforced concrete structures by using prestressed concrete prisms. This is particularly true when deflection control becomes the governing design parameter.

Nawy and Chen (1998) performed tests on continuous beams, reinforced with steel prestressed concrete prisms. These beams were 5.791 m long with two equal spans of 2.743 m. Both the beams and the prisms were cast from a very high strength concrete. One of the objectives of their research was to evaluate the effect of PCP on moment redistribution and cracking behaviour of continuous beams. They concluded that the usage of PCP in the negative moment regions seems a feasible solution for crack and deflection control.

3.3 FRP Prestressed Concrete Prisms

Using FRP for prestressing of concrete prisms is a new concept. The literature survey revealed that the research on this subject is being carried out only in Holland (de Sitter and Vonk 1993, de Sitter and Tolman 1995, Taerwe and Pallemans 1995). More care must

be taken to investigate the behaviour of FRP pretensioned prisms because (1) FRP and concrete are not thermally compatible, and (2) the high stresses in FRP during prestressing in combination with higher transverse coefficient of thermal expansion may cause cracking due to Hoyer effect. This was the main subject of research work by de Sitter and Vonk (1993), de Sitter and Tolman (1995), Taerwe and Pallemans (1995), and de Schutter et al. (1996).

De Sitter and Vonk (1993) were studying splitting forces in FRP pretensioned concrete. The test specimens were 300 mm long with cross-sectional area of 30×30 mm, 35×35 mm, 40×40 mm, or 45×45 mm. These prisms were cast from lightweight concrete and pretensioned with 5.7 mm Arapree bar with total jacking force of 17 kN. Sand was embedded in the transfer zone to reduce the transfer length. The prisms were then put into 60 °C water. Due to the Hoyer effect and the difference in the coefficient of thermal expansion, all prisms developed three to five radial cracks within one minute to three months after immersion. The reason behind the crack development was that the lightweight concrete used for casting the prisms had a very low tensile strength, therefore, the transverse tensile stresses easily exceeded the tensile capacity of concrete and caused cracking. It was observed that the transfer length did not depend on the coarseness of sand embedded in the transfer zone, and it was measured to be from 60 to 120 mm.

Pull-out and push-in bond strength tests were also performed. Push-in tests were set up to model the bond in the transfer zone of pretensioned concrete and the pull-out tests simulated the bond in the anchorage zone at ultimate load. For the push-in tests 8 mm Arapree bar was

embedded in a circular sample of lightweight concrete with cover ranging from 21 to 35 mm. The push-in tests showed more cracking than the pull-out tests. The large difference in bond between push-in and pull-out test was attributed to Hoyer effect and it was suggested that the transfer length should be either measured or should be based on measurements of bond in push-in tests.

De Sitter continued the preceding research in a joint study with Tolman (1995). They focused on investigation of producibility, splitting due to pretensioning or temperature, and force deflection behaviour of specimens pretensioned, or reinforced with FRP. The research was initiated because the use of FRP is very important in pretensioned concrete structures with small cover. Since temperature expansion is a problem when Arapree is combined with concrete, causing concrete cover to spall off, some bars were tested. The bars had cross section of 25×25 mm, or 45×45 mm. They were either pretensioned, or reinforced with different types of FRP or steel. High strength mortar was used, and the prestressing force was 18.5 kN for all the FRP except for CFRP it was 26.0 kN. For pretensioning, polyamide wedges developed by Hollandsche Beton Gröep (HBG) were used. To overcome the creep in the rods and slip in the anchorages, 10 % higher prestressing force was applied. The next day the prestressing force was adjusted to the desired level and the mortar was poured while vibrating the mould from the outside. No splitting was observed on the specimens, when they were cut into desired length of 500 mm. The cut specimens were tested under three point bending.

Two types of thermal tests were conducted: in the first test the specimens were heated

up to 110 °C, while in the second test they were subjected to cyclic temperature loading. Reinforced specimens did not show cracks when heated up to 110 °C. Pretensioned specimens developed mostly three diagonal cracks. Arapree pretensioned specimens showed improved behaviour when the length of the specimen was increased. When subjected to cyclic heating, only the specimens reinforced with original Arapree, and to some extent carbon reinforced specimens cracked. The carbon and Arapree pretensioned specimens split after one cycle. From the three point bending tests the researchers concluded that if pretensioned piles cannot be manufactured without cracking, due to inferior cyclic load behaviour, they cannot replace wood piles.

Taerwe and Pallemans (1995) tested 29 prestressed concrete prisms to determine the transfer length and critical concrete cover. The testing program used Arapree bars with diameter 7.5 and 5.3 mm. Different surface finishes were available: sandcoating, expancel coating and combination of the two. Expancel coating is used mostly to absorb part of the radial expansion due to thermal expansion and the Hoyer effect, and it was previously studied by de Sitter (1995). The prisms were 1 m long and were prestressed with initial prestressing force of $0.55 F_{ptk}$, where F_{ptk} is the five-percentile tensile strength. Eight series of prisms were studied. They were cast from two types of concrete: 72 MPa and 113 MPa. The first two series of prisms had three different cross sections i.e., 50 × 50 mm, 60 × 60 mm, or 70 × 70 mm, and were cast from normal strength concrete. The dimensions of the third series were 50 × 50 mm and some synthetic fibres were added to concrete to increase its tensile splitting strength. The fourth series of prisms were cast from high strength concrete, and had cross section of 45 × 45 mm, or 50 × 50 mm. The fifth series were cast to compare

the three different surface finishes. The prisms were of 45×45 mm cross section, and they were prestressed by 5.3 mm diameter Arapree bar. The series six was the same as the series five, except that the prisms cross section was decreased to 40×40 mm. Series seven included all three types of surface treatments for Arapree embedded in 30×30 mm or 35×35 mm cross section prisms. Early cracking of these prisms was reported and no further measurements were taken. The last group of prisms were either 40×40 mm, or 35×35 mm and were prestressed with sandcoated, or sand and expancel coated Arapree bars.

Some of the prisms in each series cracked either at one or both ends. The smaller cross sections were particularly susceptible to splitting. No splitting was observed in prisms cast from high strength concrete and concrete with added synthetic fibres. For normal strength concrete prisms, it was concluded that the critical cover for sanded Arapree bar is 2.8 times the nominal bar diameter. This can be further reduced by expancel coating to 2.33 times the nominal bar diameter. Also the use of high strength concrete, or high tensile strength concrete reduces this critical cover. The prisms were also subjected to temperature loading, but no additional cracking occurred.

De Schutter et al. (1996) conducted finite element study of the transverse thermal expansion of Arapree embedded in concrete, and of the Hoyer effect on crack propagation singly or in combination with thermal expansion. Three different cases were considered: prism pretensioned by Arapree bar, prism pretensioned by Arapree strip and noise barriers post pretensioned by Arapree strip. For the first two cases different ratios of concrete cover to bar diameter were used in order to study the influence of the transverse thermal expansion

Table 3.1: Critical concrete cover

Reference	f'_c MPa	Case 1 (AFRP bar)	Case 2 (AFRP strip)
De Schutter (1996), FE analysis	45	$c/\phi = 3$	$c/\phi_{eq} = 3$
De Schutter (1996), FE analysis	90	$c/\phi = 2.5$	$c/\phi_{eq} = 2.5$
De Sitter et al. (1993), ϕ 5.3 mm	44	$c/\phi \geq 3.27$	
Taerwe et al. (1995), ϕ 5.3 mm	42	$c/\phi = 2.8$	
ϕ 7.5 mm	42	$c/\phi = 2.83$	
ϕ 7.5 mm	70	$c/\phi < 2.5$	

of Arapree. 5.3 mm and 7.5 mm bars were used, and strips of 20 mm width and 1.5, 3 or 5 mm thickness were manufactured and embedded in prisms cast from 45 MPa and 90 MPa concrete. The transverse coefficient of thermal expansion of Arapree was considered to be $65 \times 10^{-6}/^{\circ}\text{C}$.

The finite element analysis revealed that the critical concrete cover for the combined effect of thermal expansion and Hoyer effect is 3.5 to 5 times the bar diameter, depending on the concrete strength and the bar shape. For higher strength concrete this cover can be 27 % smaller. The critical concrete cover according to Hoyer effect is given in Table 3.1, where c is concrete cover (mm), and ϕ is bar diameter (mm).

3.4 FRP Prestressed Prisms as Reinforcement for Concrete

As already described, the prestressed concrete prism reinforcement reduces deflection and increases the cracking capacity of beams (Chen and Nawy 1994, Hanson 1969, Burns

1966). If FRP is used as the prestressed reinforcement in concrete prisms, it is possible to use up to 70 % of its ultimate strength, which is not possible when FRP is used as regular reinforcement in concrete. Using FRP-PCP as reinforcement allows one to utilize the high strength of FRP effectively; consequently, reducing the amount of reinforcement and its high cost while simultaneously satisfying structural requirements. Therefore, there is no need for using excessive amounts of the FRP.

However, there are some problems associated with manufacturing prestressed concrete prisms. One of them is the problem of different temperature behaviour of concrete and FRP, another one is the Hoyer effect which is a direct consequence of the Poisson's effect, as previously reported by de Sitter and Vonk (1993), de Sitter and Tolman (1995), Taerwe and Pallemans (1995), and de Schutter et al. (1996).

According to Rahman et al. (1995) in normal temperatures no significant stresses should be developed to cause slender FRP reinforced concrete members to crack. However, with increased temperature there is a potential for cracking. They studied cylinders with embedded NEFMAC bars of 8 mm diameter cast from 35 MPa concrete. The ratio of cylinder radius to that of FRP bar ranged from 5 to 100, and ΔT was 30 ° C. The stresses which developed in concrete due to the increase in temperature ranged from 3 to 10 MPa. These stresses caused cracking in 35 MPa concrete, whose tensile splitting stress is only 3.5 MPa. In the early research work of de Sitter and Vonk (1993), bars cast from lightweight concrete were directly subjected to temperature of 60 ° C. The cracking was again initiated by the fact that the tensile splitting strength of the prisms was very low. De

Sitter and Tolman(1995), did not mention thermally induced cracking. It is believed that thermal cracking may not have occurred due to the very high tensile strength mortar used. Therefore, if high tensile strength concrete or mortar is used, the cracking can be avoided. The high tensile strength concrete used for manufacturing the PCP will also diminish the Hoyer effect, provided that the ratio of concrete cover to FRP bar diameter satisfies the values given in Table 3.1.

3.4.1 Manufacturing of Prestressed Concrete Prisms

PCP are manufactured in a prestressing bed. The bed has to be precisely leveled and aligned in order to prevent misalignment in the placement of the prestressing bar. Due to the fact that the cross section of PCPs is small, any eccentricity of the FRP bar may cause camber in the PCP. After the reinforcement is positioned, the prestressing force is applied. In order to minimize the losses, the bar is prestressed 24 hrs before casting the concrete. After 24 hrs and prior to casting concrete, the force in the bar is adjusted to the initial prestressing level to make up for relaxation and anchorage slip losses. Due to the small cross-sectional area of the bars, the maximum aggregate size has to be limited, and only external vibrators can be used. The concrete is then allowed to cure under moist conditions and the prestressing is released after 7 to 10 days, depending on the early tensile strength of concrete used.

After curing, the PCPs are placed in concrete beams as tension reinforcement. The effect of placing these bars in the tension area of beams is that some compressive stresses develop in the concrete adjacent to the prisms, depending on the age of concrete in the prisms prior

to embedment into the concrete beam.

In this study, unlike previous works, PCP will be made from high strength SIKA grout and/or concrete. The tensile splitting strength of this material is 9.6 MPa, but it can reach 8.0 MPa in 24 hours. This high early strength is the main reason for its use here. However, since SIKA does not provide the complete stress-strain diagrams of the material, in the initial phase of the study, it was assumed that its behaviour is similar to that of high strength concrete. Following the completion of the testing program, the validity of this assumption was examined.

3.4.2 Stress-Strain Relationship of PCP

Before the flexural behaviour of beam reinforced with PCPs can be discussed, the stress-strain relationship of the prisms must be known. Due to the previous findings by de Sitter and Vonk (1993), de Sitter and Tolman (1995), Rahman et al. (1995), Taerwe and Pallemans (1995), and de Schutter et al. (1996) regarding splitting cracking due to Hoyer's and temperature effects, the area of the prisms was selected to be 50×50 mm. It is believed that the cover of the bar is sufficient to prevent diagonal cracking due to the above mentioned effects, because the ratio of concrete cover to bar diameter is 2.61.

The prisms are analyzed as members subjected to axial load, with their initial prestressing ranging from 30 % to 50 % of the published tensile strength of Leadline bar. The effective prestressing is considered to be 75 % of the initial prestressing. This value may be conservative, considering the lower relaxation losses of FRP bars; therefore it may be

refined for final calculations when all the effects have been taken into account. The concrete stress after release of the prestress is designed to be less than 60 % of f'_c .

PCP are manufactured in a prestressing bed. Therefore, the strain in the FRP bar ϵ_{FRP} is equal to the strain at jacking ϵ_j

$$\epsilon_{FRP} = \epsilon_j \quad (3.1)$$

After prestressing the FRP bar, the concrete is cast. At this time ϵ_c is equal to zero. Therefore, during the life of the PCP there is a strain difference

$$\Delta\epsilon_{FRP} = \epsilon_{FRP} - \epsilon_c = \epsilon_j \quad (3.2)$$

The strain in FRP at any uniaxial load can be found as

$$\epsilon_{FRP} = \epsilon_j + \epsilon_c \quad (3.3)$$

where tensile strain is assumed to be positive. Right after releasing the prestressing force to the PCP cross-section, there develops an initial compressive strain in the prism, which is represented by point A in Figure 3.1. It has to be mentioned, that the graph in Figure 3.1 includes only short-term load-strain response and that tension stiffening is not accounted for at this juncture. If the prism is subjected to a tensile force P , then at P equal to the initial prestressing force, the strain in the prism would become zero. In Figure 3.1, that force is 40 kN. In the present study, the force-strain relationship will be discussed rather than the stress-strain relationship, because it is the force in the prism which needs to be known when calculating the moment-curvature relationship of PCP reinforced beam.

The load-strain curve in Figure 3.1 represents the behaviour of a hypothetical concrete prism with cross section of 50×50 mm. The prism is assumed to be cast from 100 MPa

Figure 3.1: Load-strain relationship for concrete prestressed prism

concrete and pretensioned by a 40 kN prestressing force, producing a uniform stress of 12 MPa. Only the positive branch of the curve was plotted, since the PCP in this analysis works as tension reinforcement only. When uncracked, the prism behaves elastically as indicated by line AB in Figure 3.1. The sudden jump in the graph at B signifies cracking. If a load control tensile test was performed, the load would not drop as indicated, instead the strain would increase rather sharply as indicated by the dotted line BC'. This increase in strain is akin to yielding in steel bars and imparts some degree of ductility to the structure because immediately after cracking the prism does not rupture, but instead it exhibits a strain-hardening type of behaviour. It is the objective of design with PCPs that under service loads the stress in the PCP does not exceed de-compression. For design purposes of beams reinforced with PCP, one may consider the ultimate load of the beam to be the load at which the PCP cracks. Of course, the implications of the latter assumption on the ultimate capacity of indeterminate structures and load redistribution need to be considered. Clearly cracking of the prism will not cause sudden failure because the FRP has reserve strength as indicated by line CD in Figure 3.1. If tension stiffening is ignored, the slope of the line CD is equal to the stiffness of the FRP bar. The fact that design with PCP prisms does not rely on load capacity exceeding the cracking of PCP prisms may be one reason for neglecting tension stiffening in this analysis. The amount of tension stiffening also decreases with decreasing c/d_b ratio (Abrishami and Mitchell 1996), which is the ratio of concrete cover to the bar diameter. Tension stiffening would cause a smaller jump in the region CD and would also increase the slope of line CD, but it would not affect the ultimate load. The percent difference between the ultimate capacity and the load at the prism cracking may be

referred to as a safety margin.

Several types of PCP were considered in the analysis. It was found that the response of the PCP changes with the amount of prestressing in the prism and with concrete compressive strength. The main objective was to find a PCP which would have a safety margin from 30 to 40 %. The latter margin is somewhat conservative, but for the purpose of this study is considered to be sufficient. The CSA Standard (A23.3-94), recommends that the ultimate moment capacity of a member should not be less than 1.2 times its cracking moment.

In the following sections the behaviour of different types of PCP is discussed and the effects of some parameters are looked at. A theoretical study is undertaken for prisms with cross section of 50×50 mm pretensioned with 8 mm Leadline bar. This bar has a modulus of elasticity of 147 GPa and a guaranteed tensile strength of 2550 MPa. Since high tensile strength concrete is needed for the prisms, high compressive strength concretes were selected. For the actual experiment, SIKA concrete is used for casting these bars. SIKA concrete has high early tensile and compressive strength, which makes it very attractive for our purposes. It allows for early release of the prisms after casting, and also assures consistent quality of every batch (since the batches are plant proportioned). For the theoretical study three types of concretes were selected: 60 MPa, 80 MPa, and 100 MPa. The prestressing force applied to the prisms was 40 kN, 50 kN and 60 kN. The effects of these parameters are discussed in the following sections.

3.4.3 The Effect of Prestressing Force on PCP Behaviour

Pre-tensioning introduces compression into the cross-section of the prism as was discussed earlier. The initial prestressing strain is dependent on the prestressing force used and on the dimensions of the prism. For this theoretical study prisms with cross-section of 50×50 mm are considered and therefore the only variables are the prestressing force and the compressive strength of concrete. In this section, only the effect of prestressing force is discussed. The aim of this analysis is to find how much prestressing force is needed to obtain a reasonable behaviour for the prestressed concrete prisms. Figure 3.2 shows the effect of the amount of prestressing on PCP cast from 100 MPa concrete. We observe that the prestressing force varies from 40 to 60 kN in this figure. While higher prestress increases the cracking load, it also causes a more precipitous drop in the load capacity after cracking and dramatically diminishes the difference between the cracking load and the ultimate load capacity of the prism.

The effect of prestress on strain change can be seen in Figure 3.3. The important parts of the graph in Figure 3.3 are the force at cracking and the safety margin after the cracking, both of which are directly proportional to the prestressing force. The behaviour of the bar after cracking affects the safety of the structure reinforced by the bar. As can be seen from Figure 3.2 when the 100 MPa prism is prestressed by a 60 kN force, there is a very small margin of safety left after cracking occurs. Although nearly the whole tensile strength of the FRP bar is used, beams reinforced with this bar would not be safe because failure of a beam reinforced with such a prism would be sudden. On the other hand, prisms with 40 and

Figure 3.2: Load-strain relationship for PCPs cast from 100 MPa concrete

Figure 3.3: Detail of load-strain curves for PCP

50 kN prestressing have about 70 % of their ultimate capacity used before cracking. This offers a 30 % margin of safety before collapse, therefore, these prisms are more appropriate for possible use.

Similar analyses were carried out on prisms cast from 60, 80, and 120 MPa concrete and the results are shown in Figures 3.4 and 3.5. Note that the solid line plateau in these figures represent the jump in strain immediately after cracking. For easy reference, the expected margin of safety, for different concrete strengths and levels of prestressing, is summarized in Table 3.2. As can be seen, it ranges anywhere from 15 to 50%, depending on the prestressing force and concrete strength used. As a result, only prestressing forces of 40

Table 3.2: The effect of prestressing force on safety margin of PCP in %

Concrete Strength	Prestressing force		
MPa	40 kN	50 kN	60 kN
60	49.67	40.99	32.31
80	44.73	36.05	27.37
100	33.70	25.04	16.36
Average	42.7	34.27	25.35

and 50 kN are considered in the following analysis since their safety margin is 30 to 40%.

3.4.4 The Effect of Concrete Strength on PCP Behaviour

Three concrete strengths were chosen for the analyses: 60 MPa, 80 MPa, and 100 MPa. The prestressing force ranged from 40 to 50 kN. A part of the load-strain graph prior to cracking is plotted in Figures 3.6 and 3.7. The cracking strain is tabulated in Table 3.3. It can be seen in this table that the strain capacity decreases with increasing strength of concrete, therefore, the response of the prism is stiffer and the expected deflection in a beam reinforced with higher strength prisms is expected to be smaller.

As a result of these studies it would be feasible to use 100 MPa concrete, since the strain capacity is comparable to the strain capacity of 80 MPa concrete (with difference of 1 to 4 % for 40 and 50 kN prestressing respectively), but the safety margin for the 100 MPa concrete would be smaller than for 80 MPa concrete

Figure 3.4: Load-strain relationship for PCPs cast from 60 MPa concrete

Figure 3.5: Load-strain relationship for PCPs cast from 80 MPa concrete

Figure 3.6: Detail of load-strain graph for 40 kN prestressing

Figure 3.7: Detail of load-strain graph for 50 kN prestressing

Table 3.3: The effect of concrete strength on strain capacity before cracking

Concrete Strength MPa	Prestressing Force kN	Total Pre-cracking Strain
60	40	3.65×10^{-4}
	50	4.22×10^{-4}
80	40	3.08×10^{-4}
	50	3.53×10^{-4}
100	40	3.02×10^{-4}
	50	3.39×10^{-4}

3.5 FRP and PCP Reinforced Concrete Beam

To compare the behaviour of an FRP reinforced concrete beam in flexure with a PCP reinforced beam, let us consider two cross sections. Cross section A is reinforced with two 8 mm CFRP bars of cross sectional area 49 mm^2 each, and cross section B is reinforced with two $50 \times 50 \text{ mm}$ PCP each prestressed by one 8 mm CFRP bar. It should be pointed out that the reinforcement ratio of both beams is 0.3 %. For CFRP reinforced section, with ultimate strength of CFRP being 2550 MPa and concrete strength being 30 MPa, the minimum reinforcement ratio is 0.07 %, and the balanced ratio is 0.14 %. Balanced reinforcement in the present context refers to the amount of reinforcement which would cause simultaneous rupture of CFRP and crushing of concrete. Accordingly, both of these beams are over-reinforced and failure would be initiated by compressive failure of concrete. If the beam A were to be designed to satisfy the CSA requirements for deflection control, the reinforcement ratio would have to be 5 to 7 times ρ_{bal} . This of course would mean using

Figure 3.8: Geometry of analyzed cross-sections

two to three times more FRP and therefore it would increase the cost of the structure. In the case of PCP reinforced concrete, it is shown that even smaller amount of FRP reinforcement in a cross section is sufficient for providing adequate stiffness to the beam to satisfy code requirements for deflection control. Both sections and their dimensions can be found in Figure 3.8, which indicates the width of the beams to be 150 mm and their height 250 mm, with an effective depth of 210 mm.

The ultimate moment capacity of the beams is given by

$$M_{ult} = C (d - 0.5\beta_1 c) \quad (3.4)$$

where

C = resultant compression force acting on the section at failure [N]

d = effective depth [mm]

β_1 = concrete stress block parameter

Figure 3.9: Stress and strain distribution in beam reinforced with FRP and PCP

c = neutral axis depth [mm]

Since FRP is linear elastic, after cracking of the prism, the PCP reinforced beam acts the same as a conventional FRP reinforced beam insofar as ultimate strength is concerned.

The basic assumptions for the flexural analysis of a PCP reinforced beam are as follows:

1. Plane sections before bending remain plane after bending
2. Stress-strain relationship for concrete and force-strain relationship for PCP are known
3. The tensile strength of concrete is negligible

To illustrate that a PCP-reinforced concrete beam will undergo smaller deflection under service load than a comparable FRP reinforced beam, let us consider some important points

on the moment-curvature diagrams of sections A and B in Figure 3.9. Since deflections can be found by integrating curvatures along the length of the beam, if the curvature of PCP reinforced concrete beam is smaller than the curvature of FRP reinforced beam for the same moment, the corresponding deflection would also be smaller. The deflections are calculated and the graph comparing the two beams is also shown in this section.

The stress and strain distributions through the height of FRP and PCP reinforced concrete sections are shown in Figure 3.9. The purpose of using PCP for reinforcement of concrete beams is to avoid the sudden decrease in stiffness of FRP reinforced beams after beam cracking. As can be seen from Figure 3.9, before cracking of concrete, the behaviour of both beams is the same. After cracking, for the same $\epsilon_b > \epsilon_{cr}$, the FRP reinforced beam has lost all contributions of concrete in the tension zone, whereas in the PCP reinforced beam, the concrete in PCP still contributes to the tensile force in the section. Therefore, the stiffness of this sections remains significantly greater than the stiffness of FRP reinforced section. Let us discuss this behaviour in more detail, using the moment-curvature diagram of the sections.

Before cracking, the behaviour of the beams is essentially linear, regardless of the type of reinforcement used. Therefore, the first point on the M- Φ diagram to observe is when the first crack occurs, i.e. when the stress in the extreme tension fibre reaches the value of tensile splitting strength f_t . For 30 MPa concrete, the tensile splitting strength is

$$f_t = 0.6 \sqrt{f'_c} = 0.6 \sqrt{30} = 3.29 \text{ MPa} \quad (3.5)$$

The modulus of elasticity of the same concrete is 24650 MPa ($E_c = 4500 \sqrt{f'_c}$), and

therefore the strain at concrete cracking is

$$\epsilon_{cr} = \frac{f_t}{E_c} = \frac{3.29}{24650} = 1.3 \times 10^{-4} \quad (3.6)$$

Assuming strain compatibility and equilibrium of internal forces, the depth of neutral axis is found to be 126 mm and 152 mm for FRP and PCP reinforced beams, respectively. The corresponding curvatures of the beams are 0.969×10^{-6} and 1.224×10^{-6} , and the cracking moments are 5.26 kN.m and 9.8 kN.m for section A and B, respectively. It should be pointed out that the cracking moment of the PCP reinforced beam is 86.3 % higher than the cracking moment of the FRP reinforced beam because the former cracking moment refers to the moment needed to initiate cracking in the prestressed prism.

Let us take a look at the moment-curvature diagram in Figure 3.10 where both $M-\Phi$ diagrams are compared. Right after cracking at the extreme bottom fibre of the beam, there is a sudden drop in the stiffness of beam A, whereas the beam B does not show any major change in stiffness. The curvature of beam A increases from 9.68×10^{-7} to 1.05×10^{-5} without any change in the moment. Observe that Figure 3.10 is based on load control behaviour. Therefore, after cracking the almost ten times increase in the curvature of beam A causes a correspondingly large increase in its deflection.

For the moment-curvature diagram of FRP reinforced beam only cracking of concrete constitutes a change in stiffness while for PCP reinforced concrete beam there will be two important moments, one is cracking of the beam, which was already discussed, and the other is cracking of the prestressed concrete prism. At the latter stage the concrete strain at the reinforcement level equals the cracking strain of PCP. The latter strain for the case of

Figure 3.10: Moment-curvature diagram of beams reinforced with FRP and PCP

40 kN prestressing and 100 MPa concrete is equal to 3.024×10^{-4} while the concomitant moment is 24.9 kN.m. For the same moment acting on the FRP reinforced beam, the strain at the reinforcement level would be 8.99×10^{-4} and the strain at the maximum tensile fibre 1.11×10^{-2} . This strain is much larger than ϵ_{cr} , which means that the FRP reinforced beam is already fully cracked as illustrated in Figure 3.10. The curvatures of the beams under 24.9 kN.m moment are 5.327×10^{-5} and 4.059×10^{-6} for the FRP reinforced and the PCP reinforced beam, respectively; in other words, the curvature of the FRP reinforced beam is 10 times larger than that of the PCP reinforced beam.

Let us next consider the behaviour of the beams at ultimate, with the ultimate moment being equal to 34.38 kN.m. The depths of neutral axis at ultimate are 47.6 mm and 58.5 mm, while the curvatures are 7.14×10^{-5} and 5.13×10^{-5} for the FRP and PCP reinforced beam, respectively. These results are tabulated in Table 3.4.

While the curvature at the critical section (section of maximum moment) will have an important influence on the deflection of a beam, the latter is equally affected by the curvature distribution along the beam. For the two beams under consideration, the curvature variation along the span is shown in Figure 3.11. Notice that in the FRP reinforced beam a large portion of the span has large curvature which results in greater deflection.

The load-midspan deflection curves of the two beams with cross-section A and B, respectively, are plotted in Figure 3.12. Both beams have span of 2.2 m and are loaded under four point bending. Note that the load-deflection curve of the PCP reinforced beam closely follows the shape of its moment-curvature diagram in Figure 3.10.

Table 3.4: Results from moment-curvature analysis

Moment kN.m	Section A		Section B	
	Curvature $\times 10^{-6}$	c^a mm	Curvature $\times 10^{-6}$	c mm
5.26	0.968	125.96	N/A	N/A
	10.52 ^b	35.03		
9.80	19.67	35.56	1.22	152.00
24.90	53.27	41.30	4.06	135.50
34.38 ^c	71.43	47.60	51.28	58.50

^a c is the depth of neutral axis

^bcurvature of section immediately after cracking

^cultimate moment of FRP reinforced section is 31.73 kN.m

Figure 3.11: Variation of curvature along the span of the beam

According to CSA the maximum allowable deflection for these beams would be 6.1 mm under service loads, assuming the service load to be 40 % of the ultimate load, which in this case is 16 kN. The deflection at this load in the PCP reinforced beam is 1 mm and the deflection in FRP reinforced beam is 14 mm. Therefore, it can be concluded that the FRP reinforced beam would not satisfy the CSA code requirements for deflection control. However, the PCP reinforced beam would satisfy these requirements up to the point when the PCP cracks, which is 75 % of its ultimate load, where the ultimate load in this case is 40.5 kN. Note that CSA A23.3-94 requires that the ultimate moment capacity of a beam must be at least 20 % higher than its cracking moment. In the case of the present PCP reinforced beam, even if we assume the cracking load to correspond to the cracking of the PCP reinforcement, still the preceding criterion would be satisfied.

In summary, from the structural point of view the idea of prestressed prism reinforcement is beneficial and feasible, and given the high cost of FRP, it would be economical since serviceability requirements would be satisfied without increasing the amount of reinforcement beyond that needed for achieving the required strength. On the other hand, the cost of producing the prisms should not be so high as to offset the benefits of a lower amount of reinforcement.

Figure 3.12: Midspan deflection of beams reinforced with FRP and PCP

Chapter 4

Experimental Investigation

4.1 General

The theory for the prestressed concrete prism reinforcement was presented in the previous chapter. To demonstrate the feasibility of that theory and as a proof of concept, laboratory tests on beams reinforced with prestressed concrete prisms were performed. This chapter describes the details of the experimental program.

4.2 Objectives of Experimental Investigation

The objectives of the experimental investigation are:

1. Investigate the behaviour of prestressed concrete prisms subjected to different levels of prestressing.
2. Monitor the time-dependent prestress loss in the prism.

3. Compare the flexural behaviour of PCP and FRP reinforced concrete beams over their complete loading range.
4. Monitor crack propagation in PCP and FRP reinforced concrete beams, and measure crack width and spacing at various load levels.
5. Measure deflections and compare the maximum deflection and deflection profile of the two types of beams at various load levels.
6. Develop flexural design procedures for the serviceability and ultimate limit states analysis of beams reinforced with PCP.
7. Develop methods for calculating the deflection and crack width of PCP reinforced beams, using the collected experimental data for verification.
8. Observe and assess the shear behaviour and strength of PCP reinforced concrete beams.

4.3 Test Materials

This research work deals with five different types of materials, which are discussed in the following section. They are steel, CFRP, concrete, high strength SIKAGROUT, and high strength SIKACONCRETE. Testing of all the above mentioned materials is a part of the experimental investigation.

4.3.1 SIKA High Strength Grout

The literature review revealed that there already exist several either ready-mixed plant proportioned grouts, or specially prepared grouts, which have very high tensile, as well as compressive strength. For application in FRP prestressed concrete prisms, grout with very high tensile strength is needed. Since our laboratory facilities, as well as time frame for this research, does not allow for the production of grout which would meet the requirements of the proposed system for high tensile and compressive strength, an existing high strength polymer grout known as SIKA Pronto 11 was selected for casting the prisms. This grout has very high early compressive strength, i.e. after one hour of curing the strength reaches 16 MPa, and after one day 56 MPa, which is 83 % of its maximum strength of 67 MPa. Its tensile strength after 14 days is 9.6 MPa, while its elongation at break equals 0.35 % and its modulus of elasticity is 11.0 GPa. Pronto 11 comes as a factory proportioned mix.

Three cylinders, eight prisms and three cubes were cast to verify the published properties of SIKA Pronto 11. The cylinders had a 76.2 mm diameter and 152.4 mm height, the prisms had a 25.4 mm × 25.4 mm cross section and a length of 152.4 mm, and the cubes had a 50.8 mm side length. It was very important to evaluate the 24 hours tensile strength of the grout in order to assess the rate of curing. The one-day split cylinder tests revealed the modulus of rupture to be 7.7 MPa, 8.6 MPa, and 7.65 MPa for the three cylinders respectively. The average strength was calculated to be 7.98 MPa, which is 83 % of the manufacturer's specified 14 day tensile strength of the grout.

After 14 days compression tests were performed on the cubes, and the average compressive

Figure 4.1: Stress-strain relationship of SIKA Pronto11 cubes at 14 days

strength reached 58.54 MPa. The results were also used to trace the compressive stress-strain relationship of the material as shown in Fig. 4.1. Using §8.6.2 of CSA A23.3-94, the modulus of elasticity of the grout was found as secant modulus at stress level $0.4 f'_c$, which gave a mean value of 15.4 GPa, which is 28 % larger than the published modulus of elasticity.

4.3.2 SIKA High Strength Concrete

SIKA Pronto 11 grout has advantages, as discussed in the previous section, but its low modulus of elasticity and relatively high creep are some of its drawbacks because they cause large losses due to elastic shortening and creep. In order to reduce the creep of the

grout, it was decided to change its composition by adding 10 mm pea gravel to the mix. The mix proportion was 8 litres of gravel to the 12 litre of SIKAGROUT. Seven cylinders were cast in order to obtain the stress-strain relationship after 7 and 28 days, plus the tensile and compressive strength after 1, 7 and 28 days. The tensile strength after 24 hours, 7 days and 28 days were found to be 8.02 MPa, 8.3 MPa, and 6.7 MPa, respectively, while the compressive strength after the same periods were 54.95 MPa, 55.5 MPa, and 65.5 MPa, respectively, hence the gravel did not affect the strength of the material. It may be noted that the 28 days tensile strength is lower than both the 24 hours and 7 days strengths. Problems with the apparatus were encountered during the testing of the 28 days strength of the cylinder. The compressive stress-strain curves for the SIKAGROUT concrete can be seen in Fig 4.2.

Figure 4.3 shows the compressive stress-strain relationship of SIKAGROUT grout and SIKAGROUT concrete. From this figure we can see that the modulus of elasticity of the SIKAGROUT concrete is 20.7 GPa, which is 25 % more than the modulus of elasticity of SIKAGROUT grout from our test, and 47 % larger than the published value.

It should be emphasized that the development of the high strength grout and/or concrete is not the focus of the present investigation. It is implicitly assumed that such materials exist and can be used to construct FRP prestressed prisms. The current data will be used later to analyze the behaviour of PCP reinforced concrete beams.

Figure 4.2: Stress-strain relationships for SIKA Pronto11 concrete cylinders at 7 and 28 days

Figure 4.3: Comparison of stress-strain relationship for SIKA concrete and grout

4.3.3 Fibre Reinforced Plastic (FRP) Reinforcement

Carbon fibre bars, known under the trade name Leadline, are used in this experiment. The bars have 8 mm diameter, 2550 MPa guaranteed tensile strength and 147 GPa modulus of elasticity. These values have to be experimentally verified. For that purpose, as well as for the purpose of prestressing, steel sleeves are manufactured as part of the current experimental program.

FRP tendons have different bond and development length than steel tendons. They also cannot sustain high circumferential pressure, therefore, the grips used for steel tendons cannot be used for prestressing FRP. Problems encountered during the prestressing process are usually due to failure of the prestressing bars within the anchors or due to excessive

slipping. There are several types of anchors used for FRPs: split wedge, plug and cone, resin sleeve, resin socketed, or soft metal overlay. Each one is suitable for a different type of FRP. Therefore, prior to any experimental testing, proper anchors need to be selected. It was found that most anchors are custom made for a specific research purpose.

Holte and Dolan (1993) conducted finite element analysis as well as tests of epoxy socketed anchors for FRP tendons. Their analytical studies showed that the best performance was achieved by parabolically varying profile anchors. To confirm the analytical results, they tested different types of anchors: one anchor with 7° linear tapered sleeve, the other with segmentally varying-taper. The test results showed 100 % success rate for the segmental taper, while in the linear anchor failure occurred within the anchor.

In the current investigation two types of steel sleeves were used for direct tensile strength tests. The first sleeve type has a length of 300 mm and a diameter of 50 mm. The inside of the sleeve is threaded to increase the bond between the grout and the sleeve. It was successfully used to find the tensile strength of 8 mm NEFMAC grid. The sleeve was filled with SIKA 212 grout, and cured for 7 days until sufficient strength was developed. The specimen was tested in the Tinius Olsen machine until failure. The failure occurred in the free part of the bar.

The same sleeves were later used for direct tensile test of Leadline bar with diameter of 8 mm. The bar pulled out when the bond between the Leadline bar and SIKA 212 grout failed at more than 2 200 MPa which is 80 % of the published ultimate strength. Therefore, a new set of grips was designed. These grips are 100 mm longer than the previous ones, and have

Figure 4.4: Prestressing sleeve

a tapered cross section (see Figure 4.4). A brief description of the sleeve is presented.

The grips are made of 300 MPa steel, and the wall thickness varies from 13 mm to 25 mm. The most important part of the sleeve is its tip. It has to resist very high stresses when the cone where the bar is anchored tends to pull out of the sleeve due to prestressing. Accordingly each sleeve has a separate clamp designed to prevent its opening at the tip. To prevent opening along the length, the two halves of the sleeves are held tight by two 50 mm long nuts. Further details can be seen in Figure 4.4.

In the following, several problems encountered during the experiment are mentioned. Since the sleeves are fastened to the bulkhead, a solution for filling the sleeves had to be found. Two different approaches were tested. In the first approach, the sleeves were mounted in the position needed for prestressing, and after that the CFRP bar was passed through the openings in the sleeves. In order to fill the sleeves in the horizontal position, polyethylene

inserts of conical shape were made to fit the dimensions of the sleeve as well as to pressurize the grout into the anchors. The polyethylene cone was kept inside the sleeve during testing. This system worked reasonably well, however, there were some problems during the pretensioning due to slipping of the polyethylene cone in the sleeve. Duct tape was used to strengthen the polyethylene sheet. Due to very high pressure on the side of the anchors, the glue from the duct tape was squeezed out and sealed the two halves of the sleeve together, making it very difficult to open them later. This led to the conclusion that a different approach was needed for filling the anchors and for preventing possible mechanical damage of the anchors during demolding.

In the second approach, the anchors were cast in the vertical position. To reduce the load at the end of the 6 m long bar (the empty sleeve weighs about 7 kg), the bottom sleeve was demolded before laying the bars in the horizontal position on the floor of the lab. Prior to implementing the second approach, a release agent had to be found since previous experience showed that the bond strength of the grout with steel is very high (20 MPa). Therefore, a test was performed on the linear set of anchors to find the appropriate releasing agent. Three different types were used: dry graphite, silicone spray, and masking tape. The silicone spray worked the best, releasing the mold completely without any problems. Therefore, it was decided to use this release agent for casting the sleeves in the vertical position. Since during prestressing the grout inside the sleeves did not slip, it was concluded that the second approach worked better than the previous one, and was used subsequently for prestressing the bars.

A tension test was performed to find the stress-strain relationship for 8 mm Leadline bar. The shorter sleeves were used, filled with SIKA Pronto11 concrete. The stress-strain relationship can be found in Fig. 4.5. Note that beyond point A the strain gauge attached to the bar failed, hence the nonlinear portion of the curve in Fig. 4.5 is not a true representation of the behaviour of Leadline. In this test the bar failed at a stress of 2 200 MPa. The latter is clearly lower than the manufacturer's recommended value of 2 550 MPa, but it is believed that the lower strength is due to the improper alignment of the bar.

A further test was carried out using the long conical sleeves. However even 430 mm long grips were not sufficiently long to develop the full tensile strength of Leadline bar with 8 mm diameter. The bar pulled out at load 112 kN, or 2286 MPa. This represents 90 % of the published ultimate capacity of the bar. The same grips will be used to prestress the FRP bars. Since the prestressing force is not as high as the force exerted during the tension test, the grips should be sufficient for that purpose.

4.3.4 Steel Reinforcement

The compression reinforcement of all the beams, as well as their shear reinforcement, consists of No. 10 rebars. Tensile reinforcement of all the beams comprised two 8 mm diameter carbon fibre bars, except beam ST which was longitudinally reinforced with two No. 15 rebars. Tension tests were performed on both No. 10 and No. 15 rebars. Three coupons of each bar were tested in order to obtain the stress-strain relationship, and the yield and ultimate strength of the particular steel.

Figure 4.5: Stress-strain diagram for 8 mm Leadline bar

The results of those tests are shown in Figures 4.6 and 4.7. The yield stress for both No. 10 and No. 15 rebar was 450 MPa. After yielding, both steels underwent strain hardening. The average ultimate strength for No. 10 and No. 15 is 650 MPa, and 716 MPa, respectively.

4.3.5 Concrete

All eleven beams and accompanying 32 cylinders were cast from the same batch of 1.5 m³ concrete. The concrete was ordered from a local mixing plant and was selected to have a 28 day compressive strength of 30 MPa, slump of 100 mm, and a maximum aggregate size of 9 mm. After arrival, 1 litre of superplasticizer was added to improve workability.

The 7 day compression and tension test revealed that the concrete had already reached 30

Figure 4.6: Stress-strain diagram for #10 steel rebar

Figure 4.7: Stress-strain diagram for #15 steel rebar

Figure 4.8: Variation of compressive strength of concrete versus time of testing

MPa in compression. The time versus strength variation is plotted in Figs. 4.8, and 4.9. It is not expected that the higher observed strength would adversely affect the objectives of the study.

The compressive stress-strain curves from 28 days cylinder test are shown in Figure 4.10. The maximum compressive strength reached was 46.2, 44.9, and 45.3 MPa and the strain at this stress level was 0.0023, 0.0027, and 0.0023 respectively. Using the secant modulus at $0.4 f'_c$ (CSA A23.3-94), the moduli of elasticity were found to be 29.1, 24.3, and 25.3 GPa, respectively. The CSA formula $4500 \times \sqrt{f'_c}$ gives the following moduli: 30.6, 30.2, and 30.3 GPa, and therefore overestimates the actual modulus by 5, 24, and 16.5% respectively.

Figure 4.9: Variation of tensile strength of concrete versus time of testing

Figure 4.10: Stress-strain diagram for compression cylinders at age 28 days

4.4 Test Parameters

This experimental work was carried out to demonstrate the feasibility of reinforcing concrete beams with PCP in order to reduce their service load deflections. Beams reinforced with steel and/or FRP bars only were tested as control specimens to compare their behaviour with that of the PCP reinforced beams. Accordingly, eleven concrete beams were tested in seven groups as follows:

Set L reinforced with two # 8 Leadline bars

Set ST reinforced with two steel bars

Set PCP-2 reinforced with two PCP2 prisms

Set PCP-3 reinforced with two PCP3 prisms

Set PCP-4 reinforced with two PCP4 prisms

Set PCP-1 reinforced with one PCP1 prism and one Leadline # 8 bar

Set CP reinforced with two CP

Sets PCP-3 to CP have two beams, the remaining sets have one beam each. All beams have a clear span of 2 200 mm, and are tested in four point bending. For details regarding their geometry and loading, see Table 4.1 and Figure 4.11.

The sets were designed to have the same ultimate moment capacity. The set PCP-FRP was designed to compare the effect of the FRP reinforcement on the behaviour of PCP reinforced

Figure 4.11: Typical dimensions and geometry of tested beams

Table 4.1: Reinforcement details of all beam sets

Beam set	Reinf. type	Prestress ^a [MPa]	A_s [mm ²]	A'_s [mm ²]	s ^b [mm]	ρ_b %	ρ %
L	2 CFRP	N/A	98	200	100	0.19	0.34
ST	2 steel	N/A	400 ^c	200	100	4.88	1.38
PCP-2	2 PCP2	290	98	200	100	0.19	0.34
PCP-3	2 PCP3	576	98	200	100	0.19	0.34
PCP-4	2 PCP4	440	98	200	100	0.19	0.34
PCP-1	PCP1, CFRP	337	98	200	100	0.19	0.34
CP	2 CP	N/A	98	200	100	0.19	0.34

^aEffective prestress

^b s =stirrup spacing

^cNote that the steel reinforced beam has four times the reinforcement of the FRP reinforced beams

beams. All eleven beams had No. 10 steel stirrups at 100 mm spacing throughout.

4.5 Test Beams

4.5.1 Prestressed Concrete Prisms Manufacturing

The prestressed concrete prisms were manufactured in the prestressing bed (see Figure 4.12).

The bed was designed to carry a prestressing force of up to 120 kN. The capacity mainly depends on the capacity of the bolts anchoring it to the floor of the laboratory. The bed consists of a W 410 × 46 section with four angles mounted on its web. Each two angles form a channel which is used as form for casting the prisms (Fig. 4.12). Each channel is used to make two prisms. The set-up allows for four 2.35 m long prisms to be cast at the

same time. Angles with unequal legs were selected, which would allow manufacture of prisms of different cross-sectional size. The steel form was selected to provide satisfactory support during prestressing as well as to maintain dimension stability after repeated usage.

The carbon fibre bar is anchored inside the sleeves used for pretensioning. Each sleeve is connected to a bulkhead which transfers the forces due to prestressing to the 1 m thick strong floor. Figure 4.13 shows the jacking end of the setup, while Figure 4.14 shows its dead end. One setup took more than 14 days due to required curing period of grout in the sleeves, and of the prisms. After each set-up was completed, the forms had to be manually cleaned and prepared for the following set-up. The view of the entire prestressing bed can be found in Figure 4.15.

The first set of prisms was pretensioned with sleeves cast in the horizontal position. The cross-section of the prisms was 50 mm $\times$ 50 mm. The original design called for using the actuator to prestress the bars, in order to apply the total force in the system, and to control elongation of the bar. The elongation was measured by means of the strain gauges mounted on the FRP bar. Since there was some slipping inside the sleeves, the relative movement in the anchors was nearly equal to the stroke of the actuator, therefore, the actuator had to be removed for this setup. The bars were pretensioned by tightening bolts on the live end of the prestressing bed. The strain in both bars was measured by a strain-indicator box. This way of pretensioning was found to be very easy and efficient, however, for higher levels of prestressing it was extremely difficult to ensure sufficient tightening of bolts. During the test one bar started to move inside the cone which was thought to be due to loss of bond

Figure 4.12: Experimental setup for prism prestressing

Figure 4.13: Prestressing setup, view of the jacking end

Figure 4.14: Prestressing setup, view of the dead end

Figure 4.15: Prestressing setup

between the bar and the concrete inside the sleeve. At that point the strain bounced back to zero reading, therefore the prestressing of the particular bar was terminated while the prestressing of the companion bar was continued.

The other bar was pretensioned to $3\,690\ \mu\epsilon$, or 542 MPa which is only 21 % of the maximum tensile strength of the Leadline bar. Since the grout has very high tensile strain at rupture (0.35 %), this unusually high strain allows for some experimentation with the amount of prestressing. The time dependent cumulative losses due to elastic shortening were 22 %. The losses after 7, 14 and 21 days were 10 %, 15.4 %, and 16.4%. This leads to the supposition that after 21 days practically all the losses due to creep of the grout had taken place and that the only other losses which may occur would be mostly due to the creep of the FRP bar. The latter is however, generally small and inconsequential.

The second set of prisms was pretensioned using the sleeves cast in the vertical position.

The cross section of the prisms was reduced to 43×43 mm because the stirrups ordered for the beams had a tolerance of 10 mm and were not perfectly straight. This caused the prisms not to fit well inside the reinforcement cage, hence reduction in cross-section of the prisms became necessary.

This set was planned to be prestressed to $6\,000\ \mu\epsilon$, or 880 MPa. The cross section of the prism was reduced by inserting 4 mm thick plexiglass strips inside the steel forms. In order to be able to release the prisms from the form, silicone lubricant was used. The bars were then simultaneously pulled by fastening bolts on the threaded rod. Only one set of the two prisms was successfully prestressed up to a strain of $5\,250\ \mu\epsilon$. The prestressing was released to the prism two days after casting. The split cylinder test indicated that the tensile strength of the grout after one day was 8.00 MPa, therefore no cracking was expected. However, as it was learned later, the grout was not cured sufficiently and in combination with higher stress due to prestressing, the elastic shortening and creep of the grout caused a total loss of 60 %. Elastic shortening constituted 44 % of the total loss, which is double the amount of the elastic shortening for the previous prism. In order to prevent such high losses, it was decided that the prestressing forces will be released to the next set of prisms after one week and that pea gravel would be added to the mix. The gravel was expected to decrease the amount of creep in the grout.

Another reason for the larger losses is that the latter prisms had camber caused by the eccentricity of the FRP from the center of the prism. It was found difficult to perfectly center the nearly 6 metre long bar in both the vertical and the horizontal plane. It can be

shown that if the eccentricity of the bar is only 5 mm, it would cause a camber of 50 mm.

The third set of prisms, PCP-3, was cast from SIKA concrete. The vertical method of filling the sleeves was used. No major problems were encountered during the setup, the bars were pretensioned to 6 000 $\mu\epsilon$ (880 MPa) and after release of prestress, the bars remained perfectly straight with no camber. The losses from elastic shortening were 18 %, and the cumulative prestress losses after 7, 14, and 28 days were 12 %, 18 %, and 20 % respectively.

The fourth set of prisms, PCP-4, was also cast with SIKA concrete. The prestress for this set was selected to be 4 500 $\mu\epsilon$, or 660 MPa. The losses due to elastic shortening were 24 %. Together with this set of prestressed prisms, a set of FRP reinforced prism (zero prestress) was also cast. They had the same cross-section and material properties as the PCP-4 prisms.

The last set of prisms, PCP-5, were pretensioned to 1200 MPa. However, this stress induced radial stresses significantly higher than the tensile capacity of Sika concrete, and after release the prism diagonally split along its length. Such a behaviour could be predicted by analysis (de Sitter and Vonk 1993). Firstly, the Hoyer effect should be accounted for

$$\epsilon_{ri} = \nu_{res}\epsilon_l + (\alpha_{res} - \alpha_c)\Delta T \quad (4.1)$$

where ϵ_{ri} is the radial initial strain, ν_{res} is the Poisson's ratio for resin, ϵ_l is the longitudinal strain, α_{res} is the coefficient of thermal expansion of resin, α_c is the coefficient of thermal expansion of concrete, and ΔT is the temperature change. The second term on the right-hand side of the equation represents the effect of temperature, which may be omitted for

our purposes. The pressure p due to Hoyer effect can be calculated as

$$p_{ri} = \epsilon_{ri} E_{res} / (1 - \nu_{res}) \quad (4.2)$$

where p_{ri} is the initial radial pressure, and E_{res} is the modulus of elasticity of resin. Knowing the pressure p_{ri} , the tangential stresses in the bar can be found. These will be the highest adjacent to the Leadline bar, and will dramatically decrease towards the outside of the bar. For the prestress of 8 000 micro-strain, or 1200 MPa, this stress was calculated to be 19.5 MPa, which caused the prism to crack radially since the tensile splitting strength of Sika concrete was 9 MPa. The calculated value of 19.5 MPa is on the safe side, since many variables were assumed, such as the modulus of elasticity of resin in Leadline bars. Some researchers show that de-bonding the bar at the location where the prestress is transferred helps to deal with the problem of splitting cracks (Mahmoud et al. 1997, de Sitter et al 1993).

The loss of prestress was monitored for PCP-1 to PCP-3. Due to time constraints, the losses for set PCP-4 prisms were monitored just for 7 days. The variation of losses with time can be seen in Figure 4.16. The losses in Fig. 4.16 are the time dependent portion, including creep and shrinkage losses, but they do not include losses due to elastic shortening. It can be seen that after 28 days, the additional losses become insignificant. Note that the initial prestress and the prestress after release for the three sets of the prisms are not the same.

Figure 4.16: Time dependent variation of prestress losses in PCP-1 to PCP-3 prisms

4.5.2 Construction of the Beams

After the manufacturing of the prestressed concrete prisms, construction of reinforcement cages for the beams was started. All this work was performed in the Civil Engineering Laboratories. During the manufacturing of the cages, extreme care was taken to ensure that all the beams had the same effective depth, d , which would facilitate the comparison of the ultimate moment capacity of the beams. The spacing of the stirrups was also measured carefully to ensure the same shear reinforcement for all the beams.

A typical assembled cage can be seen in Figure 4.17. The bars were attached to the stirrups with both wires and plastic strips. After assembly, the cages were placed in the wooden forms and spacers were used to provide the necessary concrete cover.

Figure 4.17: Assembled reinforcement cages with PCP tensile reinforcement

4.5.3 Beam Instrumentation

All eleven beams were equipped with the same set of instrumentation. Since flexural behaviour of the beams was mainly observed, there were four electric resistance strain gauges of length 5 mm mounted on the bottom longitudinal reinforcement, and two strain gauges on the top longitudinal reinforcement. The bottom strain gauges were placed in the center of the beam and in the center of each shear span. The top strain gauges were placed in the midspan of the beam. There were in total 8 strain gauges for monitoring the shear behaviour of the beam, placed on the shear reinforcement.

In order to monitor the strain profile during loading of the beams, two strain gauges of 30 mm length were glued to the surface of the beam. Concrete strain gauges of longer length were chosen to avoid their placement over a single aggregate only. They were placed at the mid-span of the beam, 15 and 30 mm from the top surface of the beam. Aside from those two strain gauges, three demec point bases were glued to each beam at its mid-span. These were 10, 20 and 40 mm from the top compression fibre. These distances were chosen because previous tests by other researchers showed that the compression zone of FRP reinforced beams is usually very narrow.

In order to monitor the shear behaviour, each shear span was equipped with a rosette of strain gauges, as well as a rosette of demec points. The center of each rosette was 600 mm from the support. Refer to Figure 4.18 for detailed location of strain gauges and demec points.

Deflections were measured at five different locations along the span of the beam: in the

Figure 4.18: Typical location of strain gauges on tested beams

middle of both shear spans, under the two load points, and in the mid-span of the beam. At each location, two LVDTs were used to measure possible tilting of the beam during the test. The location of all LVDTs is indicated in Figure 5.33.

4.5.4 Test Equipment and Loading Procedure

The eleven tested beams were simply supported and were loaded in four point bending. The following set of equipment was used during the tests:

- 150 kip (667 kN) actuator with 6 in (152.4 mm) stroke
- knife-edge bearing plates with rollers as supports of the beam
- knife-edge bearing plates as loading plates
- data-acquisition system
- portable microscope
- dial-gauge for demec gauge readings
- LVDTs

All the beams were loaded monotonically up to failure. One beam out of each set of beams was unloaded and permanent deformations were observed. The test was carried out under displacement control. The rate of loading before cracking of PCP was 0.005 mm/s, unloading and re-loading up to failure was carried out with loading rate 0.01 mm/s.

The following loading steps were chosen:

- load to 5 kN and unload to ensure that all strain gauges and LVTDs are working
- 0 with cross-head on the beam
- load until P_{cr}
- 10 kN increments up to 40 kN
- unload
- re-load up to 50 kN
- 10 kN increments up to failure

Crack width was monitored until the width of the largest cracks reached 2 mm, or up to 90% of ultimate. When deemed safe, demec points were monitored also up to 90% of ultimate. The 40 kN load before unloading was chosen as load corresponding to 50% of the ultimate design load. However, for the PCP reinforced beams the ultimate design load was underestimated, as will be explained in the following chapters. Nevertheless the 40 kN load limit for unloading was kept throughout the tests. The data acquisition system was monitoring data every 10s.

Chapter 5

Test Data and Results

5.1 General

Observations and experimental results from all the tests are presented in this chapter. Test of each set of beams is discussed in detail and the experimental results are presented. Among these are the strain in both reinforcement types and in concrete. Strain profile of the beams is extracted where possible and used to determine the experimental moment-curvature diagram. Load-deflection curves are plotted, as well as deflected profiles of each beam. The chapter is concluded with diagrams of cracking patterns of the beams. Emphasis is put on the data directly related to bending behaviour of the tested beams. Further theoretical analysis of the data is carried out in the following chapters while the shear behaviour is discussed in a separate chapter.

The results are presented in the following sequence: behaviour of beam during test, including type of failure; strain in the reinforcement, strain in concrete, concrete strain profile, moment-curvature relationship, load-deflection relationship, deflection profiles, and crack-

ing. In the following chapters, the term load refers to the *total load* applied to the beam.

For summary of test results, please refer to Table 5.1.

Table 5.1: Experimental Results

Beam	Failure mode	P_{ccr}^a [kN]	P_{pcr}^b [kN]	Prestress c [MPa]	P_{ult} [kN]
L	flexure	11.4	N/A	N/A	100.0
ST	flexure	11.0	N/A	N/A	82.0
CP1	flexure	11.0	11.0	N/A	94.8
CP2	flexure	11.5	13.5	N/A	91.4
PCP1-1	flexure	11.5	20.0	337	105.8
PCP1-2	bond	10.0	20.0	337	66.7
PCP2	shear	12.0	26.0	290	100.5
PCP3-1	flexure	10.0	40.0	576	115.0
PCP3-2	flexure	11.0	40.0	576	115.0
PCP4-1	flexure	12.0	35.0	440	110.0
PCP4-2	flexure	12.0	38.0	440	114.0

^aCracking of concrete

^bCracking of prism

^cEffective prestress

5.2 Strain in the Reinforcement and Concrete

This section presents the experimental data obtained from strain gauge readings, or demec point readings, installed on the tested beams. The results for the various beams are compared to each other and to their theoretically expected values.

Figure 5.1: Typical strain in reinforcement of beams L and CP1

5.2.1 Strain in Flexural Reinforcement

The strain in the flexural reinforcement was measured at three locations: in the middle of both shear spans and in the center of the constant moment region. For a complete set of measurements from all working strain gauges refer to Appendix A. In this section mostly the strain in the constant moment region will be discussed since most of the beams failed in flexure in this region.

The comparison of strain in Leadline reinforced beam and beam reinforced with two CP1 prisms can be found in Figure 5.1. The solid line represents the strain in Leadline bar of beam L, and the dashed line represents the strain in concrete prism (no prestress) of beam CP1. The strain in these two beams is the same before the beam cracked, and thereafter the

strain in the CP1 is 10% smaller than the strain in the Leadline bar alone. Note how well the strain in these two beams agree with one another over the whole loading range prior to the failure of the strain gauge on the Leadline. This similarity of behaviour will also be exhibited by the load-deflection behaviour of the same beams. It was found out that all experimental results are very consistent, and therefore when two beams of the same kind were tested, only the behaviour of one of them is discussed in the later chapters. Since in the beam reinforced with Leadline bars only, the strain gauges failed before the beam reached its ultimate capacity, in some cases only beams CP1 or CP2 are compared with beam ST or beam reinforced with PCP. The beam L failed at 100 kN, and the beam CP1 at 95 kN.

When prestressed prisms are used, the effect of tension stiffening becomes more evident, as can be seen in Figure 5.2. In this figure, the Leadline bar is represented by the dashed line, and the prestressed concrete prism is represented by the solid line. They both were embedded in the same beam, PCP1-2. Notice that the reinforced prism exhibits hysteretic behaviour and its loading and unloading path do not coincide. This behaviour is similar to the response of concrete during loading and unloading. A steel reinforced section does not exhibit this type of behaviour prior to yielding of the steel. Notice also that the prism shows around 30% increase in the amount of permanent deformation compared to the adjacent Leadline bar. This is not a favourable behaviour, since it means that the structural member reinforced with this type of reinforcement will suffer increased permanent deformation after loading beyond the cracking load of the prism. To further analyze this behaviour, the maximum strain in the main flexural reinforcement of all beams is plotted in Figure 5.3. It can be seen that with increasing amount of prestress in the prism, the amount of permanent

Figure 5.2: Comparison of strain in PCP1 and Leadline in beam PCP1-2

Figure 5.3: Comparison of maximum strain in flexural reinforcement of tested beams

Figure 5.4: Strains in reinforcement of beams L, PCP-2, and PCP3-1

deformation decreases. Therefore, a design procedure is developed to be able to calculate the amount of required prestress in the prism for achieving satisfactory behaviour of the beams, or any other structural elements reinforced with such prisms. A more detailed comparison can be seen in Figure 5.4 and 5.5. Figure 5.4 shows very clearly that the effect of tension stiffening is higher for prisms with higher level of prestressing. In Figure 5.5 the solid, dashed, and dashed-dotted lines represent the strain in prisms PCP3-1, PCP4-2, and PCP-2, respectively.

When the ultimate strains in the steel and PCP longitudinal reinforcement were compared, the strain at ultimate was larger in the steel reinforced beam, equal to 2%, see Figure 5.6. The strain hardening region in the steel load-strain relationship can be clearly seen. For the PCP reinforced beams, the maximum strain (not including the prestress) was reached in beam PCP3 and was equal to 1.5%. If we include the prestress, the strain in PCP3 was

Figure 5.5: Detail of load versus strain in the main reinforcement at centre span of beams PCP4-2, PCP3-1, and PCP2

5% less than the published rupture strain of 1.7%.

5.2.2 Strain in the Shear Reinforcement

All tested beams were reinforced with steel stirrups with uniform spacing of 100 mm. To capture the shear behaviour of the tested beams, strain gauges were installed on each leg of four symmetrically disposed stirrups: i.e. stirrup 6, and 7 at each end of the beam. The strain is plotted in Figures 5.7 to 5.13. The strain in stirrups of the steel reinforced beam ST is plotted in Figure 5.7. The maximum strain in those stirrups was less than 5×10^{-4} , or 25% of yield. This means that the shear stresses in the steel reinforced beam were relatively small.

Figure 5.6: Typical strain in the main reinforcement of beam ST

As indicated in Figure 5.8, the Leadline reinforced beam L developed small strains in the two stirrups embedded in the right shear span, but one of the stirrups in the left shear span exhibited a strain value more than three times the strain in other stirrups, and was very close to yielding, but it did not actually yield, because it started to reverse direction at 90% of ultimate load. We believe that this was due to the strain gauge debonding, followed by the failure of the strain gauge. Since only the left span stirrups suffered large strain, it can be concluded that the monitored stirrups were closer to the cracks in the left shear span. Note that overall the shear strain in this beam was larger compared to the corresponding strain in beam ST.

The strain in stirrups of beams CP1 and CP2, PCP1-1 and PCP1-2 were small, and can be found in Appendix A. Due to the similarity of the behaviour of these beams in shear, we will only discuss beam PCP-2. Let us look at the strain in the stirrups of beam PCP-2, Figure 5.9. The strains in the seventh stirrup on both sides of the shear span are much larger than the strain in the neighboring sixth stirrup, just 100 mm away. This means that there was a major crack developing in this area. At 30 kN for the right shear span, and at 40 kN for the left one these cracks turned into shear-flexure cracks and the strains in the stirrups reflect this event by sudden increase in their values. At 90 kN the strain gauge in the right span had failed, while the strain gauge in the left span recorded a strain of 0.1%, or 50% of the yield strain. This beam failed in shear, however, the yielded stirrup was not monitored by our equipment.

The strains in the stirrups of beams PCP3-1, PCP3-2, PCP4-1, and PCP4-2 ranged from

Figure 5.7: Strain in the stirrups of beam ST

Figure 5.8: Strain in the stirrups of beam L

Figure 5.9: Strain in the stirrups of beam PCP-2

50% to 75% of yield strain, and were larger than the strain in the corresponding stirrups of the steel reinforced beam ST. This is a direct result of the greater load carrying capacity of PCP reinforced beams. The shear capacity of the beams will be discussed in greater detail in Chapter 7.

5.2.3 Concrete Strain

Two 30 mm long strain gauges were mounted on the concrete 15 and 30 mm from the extreme compression fibre at midspan. To monitor the movement of the neutral axis, three bases of demec points were attached at 10, 20, and 40 mm from the extreme compression fibre at midspan. A rosette of three 30 mm strain gauges was mounted in the center of each shear span, together with four bases of demec points in alternating angles. The sketch in

Figure 5.10: Strain in the stirrups of beam PCP3-1

Figure 5.11: Strain in the stirrups of beam PCP3-2

Figure 5.12: Strain in the stirrups of beam PCP4-1

Figure 5.13: Strain in the stirrups of beam PCP4-2

Figure 4.18 illustrates the location of the preceding gauges and demec points. This section presents the readings from the strain gauges, and demec points which are used to determine the strain profile and the moment curvature relationship of the beams.

Figures 5.14 and 5.15 show the strain gauge readings from the two strain gauges mounted on the surface of concrete. The solid line on all the figures represents strain 15 mm below the extreme compression fibre, while the dashed line represents the strain 30 mm below the same fibre (see Figure 5.18 on page 135 for easy reference). Whenever the two lines are very close to each other, it means the beam had a small curvature. It can be seen in Figure 5.14, that the curvature of beam ST was small, and so was the curvature of beam PCP-2 before the prisms cracked. However, from the same figure it is obvious that the curvature of beam L steadily increased right after the concrete cracked. Figure 5.15 shows that the curvatures of all the beams reinforced with PCP were small before the prisms cracked, which is in agreement with the main hypothesis of this investigation. This behaviour is also reflected by the moment-curvature and the load-deflection relationship of the beams. These figures also show that for concrete strain less than 1.5 millistrain, the stress-strain relationship is linear. Therefore, the use of triangular stress distribution for calculation of the force carried by concrete would give good results.

To conclude the discussion on strain in Figures 5.16 and 5.17, the maximum strain in the bottom reinforcing bar and in the concrete in the constant moment region is plotted. The strain in the concrete, represented by the solid line, was normalized by 0.0035, and the strain in FRP, represented by the dashes line, was normalized by 0.017, the published maximum

Figure 5.14: Strains in concrete for beams ST, L, CP1, CP2, and PCP-2

Figure 5.15: Strains in concrete for beams PCP3-1, PCP3-2, PCP4-1, PCP4-2, and PCP1-1

strain in Leadline. It has to be pointed out that the strain in concrete, plotted in Figures 5.16 and 5.17, was measured 15 mm below the extreme compression fibre, therefore, is smaller than the maximum strain of 0.0035 that is commonly used. Figures 5.16 and 5.17 can be used to assess the efficiency of the utilization of materials in the different beams.

These figures reveal that the concrete strain at/or close to ultimate was from 50% to 60% of the maximum concrete strain in all the beams, the exceptions being beams L, and PCP4-1, for which the same strain was 80% and 70%, respectively. For the steel reinforced beam ST, it can be observed that before the steel yielded, the strain in concrete was increasing gradually, and it reached only 25% of the maximum concrete strain just before yielding.

The rate of increase of strain in the concrete can be compared for all the beams in the latter figures. The comparison leads to the conclusion that the highest rate of change in the concrete strain occurs in beam L, which is reinforced with Leadline. This means that the beam would fail in compression much earlier than the rest of the beams. It also means that the force in FRP is increasing very rapidly, causing concrete to develop high stresses. Since for the latter beam after cracking the depth of neutral axis does not change noticeably (this will be shown later), concrete strain must increase with increasing load to establish moment equilibrium in the cross-section. The smallest rate is in the steel reinforced beam ST, due to the relatively small stresses in the steel bar prior to yielding. After yield, both steel and concrete strain increases rapidly. These figures also show that the beams reinforced with PCPs behave very similarly to the beam ST in terms of material efficiency *before* prism cracking, while *thereafter* their behaviour is closer to the beam L reinforced with Leadline.

Figure 5.16: Ratio of the measured maximum strain in the reinforcement (dashed line) or concrete (solid line) to the ultimate strain capacity of the same material

Figure 5.17: Ratio of the measured maximum strain in the reinforcement (dashed line) or concrete (solid line) to the ultimate strain capacity of the same material (Continued)

5.3 Strain Profile and Moment-Curvature Relationship

The information presented in the previous sections is utilized to give more complete picture of the behaviour of the tested beams. The concept of linear strain distribution in FRP and PCP reinforced sections is examined, followed by the establishment of the moment-curvature relationship of each beam.

5.3.1 Strain Profile

The concrete strain distribution was measured in the compressive zone using strain gauges and demec point measurements, and in the tension zone by strain gauge mounted on the main reinforcing bars. Using the demec point data in combination with the strain gauge measurements, the strain profile of each beam will be plotted. The arrangement of demec points and strain gauges is shown in Figure 5.18. For plotting the strain profiles, a computer program was written using Matlab, to compile all the necessary information from both the hand measurements of demec points and the electronically measured strain gauges. Matlab is used throughout this thesis to facilitate manipulation of the experimental data. A typical strain profile for one beam of each set of two nominally identical beams can be found in Figures 5.19 to 5.25. The profiles for the remaining beams are shown in Appendix A.

The strain profiles of beam ST are shown in Figure 5.19, which corresponds to various load levels, as indicated next to each profile, up to 75 kN, which is 91% of the ultimate load of beam ST. Since each point represents a different reading, some of them may not lie on

Figure 5.18: Arrangements of strain gauges and demec points for strain measurement

Figure 5.19: Strain profile in the mid-span of beam ST at P=10 kN to 75 kN

the same plane, nevertheless, the general trend of plane section remain plane can be clearly seen. From the strain profiles in Figure 5.19, it can be seen that at 91% of ultimate, the strain in concrete had already reached 0.003, which means that the beam was very close to failure.

The strain profile of beam L in Figure 5.20 indicates that plane sections remained plane throughout the loading process. At load levels below the ultimate load, a small deviation can be seen from linearity but the writer believes that this is within the range of accuracy that can be achieved with manual demec gauge readings. Also the deviation may be partly due to the fact that the strain was measured on the bar somewhere between the cracks, where due to tension stiffening the strain in the bar would be smaller than at the crack location. At/or close to ultimate load, the beam is usually severely cracked throughout its length and

Figure 5.20: Strain profile in the mid-span of beam L at $P=12$ kN to 70 kN

therefore the effect of tension stiffening diminishes. Finally, the strain profile was plotted for a load of 70 kN, which is 70% of the ultimate capacity of the beam. Generally these plots include both hand measured and electronically measured data, however the strain profile at 70 kN was constructed from demec readings only.

From the plotted strain profiles, it was clear that for the beams reinforced with PCPs the basic principle of plane sections remaining plane applies, therefore, the general principles for the flexural analysis of steel reinforced beams can be used to analyze beams reinforced with PCP.

Figure 5.21: Strain profile in the mid-span of beam CP2 at P=20 kN to 80 kN

Figure 5.22: Strain profile in the mid-span of beam PCP1-1 at P=11 kN to 100 kN

Figure 5.23: Strain profile in the mid-span of beam PCP-2 at P=18 kN to 70 kN

Figure 5.24: Strain profile in the mid-span of beam PCP3-1 at P=20 kN to 105 kN

Figure 5.25: Strain profile in the mid-span of beam PCP4-2 at P=30 kN to 100 kN

5.3.2 Moment-Curvature Relationship

In this section, experimental moment-curvature diagrams are plotted in Figures 5.26 to 5.31 for one beam in each set of two beams.

The beams reinforced with Leadline and/or prisms without prestressing cracked very early during the loading process, hence there was no approximation for taking measurement prior to this event, consequently some of the diagrams do not start at the origin.

The moment-curvature diagram of beam ST is shown in Figure 5.26. It exhibits the typical and expected behaviour of under-reinforced concrete beams, comprising a nearly linear response, followed by a small amount of nonlinearity and finally yielding at 27 kN.m. After yielding, the moment-curvature relationship is basically linear and the increased sectional

resistance is achieved through movement of the neutral axis (see Figure 5.19 for movement of the neutral axis).

The moment-curvature response of beam L is quite different from that of beam ST. The curvature is rather high, even at relatively low level of load, and it increases almost linearly up to a moment of 23 kN.m, which is 75% of M_u . The nonlinear response beyond the latter load level is most likely due to the hand measurements of strain from the demec points.

The moment-curvature relationship of beam CP2 is shown in Figure 5.28. The relation is basically linear up to 28 kN.m, thereafter it appears to be stiffening, which is unexpected. It is likely, that the strain measurements beyond 28 kN.m are inaccurate because this apparent stiffening is not reflected by the load-deflection curve of the same beam in Figure 5.36. The load-deflection curves will be discussed later.

Figures 5.29 to 5.31 show the moment-curvature responses of beams PCP1, PCP2, and PCP3, all of which are reinforced with the prestressed prisms. We observe the similarity of responses in the latter figures. In each case a nonlinear response is observed, signifying major changes in the stiffness of the beam. The first deviation from linearity occurs when crack forms at the extreme tensile fibre of the beam, which does not lead to a severe reduction in stiffness. The more significant reduction that occurs at a high load is due to cracking of the reinforcing prism. Thereafter, the stiffness of these beams is essentially the same as of beam L after cracking.

The latter point can be easily observed in Figure 5.32, which depicts the moment-curvature diagrams of beams ST, L, and PCP3-1. Notice that the slope of moment-curvature rela-

Figure 5.26: Moment-curvature for beam ST

relationship of beam PCP3-1 after $M=25$ kN.m is almost the same as the slope of the curve for beam L between $M=10$ kN.m and $M=20$ kN.m. The same curve also shows the significantly smaller curvature of beam PCP3-1 compared to beam L at all load levels. If we assume the ultimate moment capacity of beam ST to be 30 kN.m, then the curvature of beams ST and PCP3-1 are practically equal at $M_u=30$ kN.m. At $M=15$ kN.m (i.e. $0.5M_u$), the two beams again have nearly equal curvature, but between $0.5M_u$ and M_u the curvature of beam PCP3-1 increases sharply. Clearly, if we increase the level of prestressing in the prism, the curvature of a PCP reinforced beam can be controlled at even higher level of load. The extent of the increase in prestressing will be, however, limited by the tensile strength of the concrete prism and by its size.

Figure 5.27: Moment-curvature for beam L

Figure 5.28: Moment-curvature for beam CP2

Figure 5.29: Moment-curvature for beam PCP1

Figure 5.30: Moment-curvature for beam PCP2

Figure 5.31: Moment-curvature for beam PCP3-1

Figure 5.32: Moment-curvature comparison for beams ST, L, and PCP3-2

5.4 Load-Deflection Response of Test Beams

This section discusses the experimental load-deflection behaviour of all the test beams. The midspan deflection is plotted for each beam and the full load-deflection curve of a beam reinforced with PCP is compared with the corresponding curves of the steel and Leadline reinforced beams. The comparison is intended to demonstrate the feasibility of using the prestressed concrete prisms as reinforcement for deflection control. In addition, the deflected profiles of the beams are plotted at several load levels.

5.4.1 Load-Deflection Curve

Deflection of the beams was measured by LVDTs located along the length of the beam as indicated in Figure 5.33. The various locations and the corresponding LVDTs are designated as *def1*, *def2*, etc. At each location two LVDTs, were installed and were symmetrically disposed with respect to the vertical centerline of the beam cross-section. The plotted results are the average reading of the two LVDTs for each section. The set of Figures 5.34 to 5.40 presents deflection at each of the measured locations for the test beams. When duplicate beam specimens were tested, midspan deflection for the duplicate beams will be compared.

The beam reinforced with steel rebars, beam ST, exhibits a typical ductile behaviour as shown in Figure 5.34. The Figure 5.34 shows the load-deflection curve at different locations along the span, as well as a separate curve for midspan deflection. The load-deflection curves

Figure 5.33: Location of LVDTs along the span of the beam

of the other test beams are plotted in similar fashion. Beam ST cracked at 11 kN, and up to twice this load the deflection measured at all five locations was less than 5 mm. When the steel started to yield at around 70 kN, the midspan deflection nearly doubled to 12 mm. The beam failed at 82 kN, and its maximum deflection at midspan reached 67 mm, five times the amount at yield.

The beam reinforced with two Leadline bars, beam L, underwent the anticipated brittle behaviour of FRP reinforced beams. The beam cracked similarly to the beam ST at 11.4 kN. After the development of each additional crack, the load dropped until equilibrium between the force in concrete and the increased force in FRP was obtained. This caused increase in curvature of the beam and a corresponding increase in deflection. The deflection at ultimate load of 100 kN was 65 mm. Observe that there is no plateau on the load deflection curve in Figure 5.35, and the failure can be therefore classified as brittle. During the test it was difficult to anticipate the actual failure, because extensive cracking was evident already at 70% of the ultimate load. This beam was tested only as a control specimen, and its poor performance in terms of large deflections and crack widths during the test was expected.

Figure 5.34: Deflections of beam ST

It also demonstrates that despite its high ultimate strength, such a beam cannot be used in practice because it will not most likely satisfy the serviceability requirements for crack width and deflection control.

Beams series CP were reinforced with non-prestressed prisms, and both CP1 and CP2 were tested as control specimens. Their load-deflection curves are plotted in Figure 5.36. The beams cracked at 11 and 11.5 kN, which is consistent with beams ST and L. Throughout the loading process, the beams behaved very similarly to beam L. However, when comparing the mode of failure of these beams to that of the beam L, a noticeable flattening of the load-deflection curves can be observed at 90 kN. This plateau occurred at a maximum deflection of 60 mm, and the beam failed at the same load level but after its maximum deflection increased to 78 mm. It can be concluded that the beams CP1 and CP2 failed in a relatively ductile manner at 94.8 and 91.4 kN, respectively, and they both failed due to failure of concrete in the compression zone.

The load deflection curves for beam series PCP1 can be found in Figure 5.37. Beam PCP1-1 cracked at 11.5 kN while beam PCP1-2 cracked at 10 kN. Both load deflection curves show a bi-linear response with no change in stiffness after prism cracking. Figure 5.37 also shows a comparison of the load-midspan deflection curves of beams PCP1-1 and PCP1-2. Beam PCP1-2 failed due to bond failure at 65 kN, therefore the load deflection curve stops at that load. Prior to bond failure, the beam behaved very similarly to beam PCP1-1, which failed at 106 kN. Failure in beam PCP1-1 was precipitated by the compression failure of concrete and was accordingly brittle.

Figure 5.35: Deflections of beam L

Figure 5.36: Deflections of beams CP1 and CP2

Figure 5.37: Deflections of beams PCP1

The load-deflection curves of beam PCP-2 can be found in Figure 5.38. This beam cracked at 12 kN, followed by cracking of the prisms at 26 kN and complete failure at 100 kN. The load-deflection relationship is bi-linear and it exhibits practically no ductility. The spiraling crack pattern of this beam indicated that the beam may have undergone some torsion.

The load deflection relationship of beams PCP3-1 and PCP3-2 are plotted in Figure 5.39. These beams cracked at 10 and 11 kN respectively. Due to the prestress in the reinforcing prisms, the cracking of the beam did not noticeably affect the load deflection curve. The significant change in stiffness was caused in both cases by the cracking of the prisms at 40 kN. The beams failed in flexure in a brittle manner at 115 kN and failure was precipitated by the failure of concrete in the compression zone. At this juncture, it suffices to state that brittle failure in the present context simply implies lack of significant plastic energy dissipation.

Beams PCP4-1 and PCP4-2 experienced similar behaviour as beams PCP3-1 and 2. These beams cracked at 12 kN, and cracking of the prisms PCP4-1 and 2 occurred at 35 and 38 kN, respectively. The beams failed in compression, with their maximum deflections at ultimate load of 110 and 114 kN being 50 mm and 52 mm respectively. The rather close agreement between the cracking and ultimate loads of these two beams demonstrates the consistent performance of PCP reinforced beams, and the quality control applied during their construction and testing.

A comparison of the deflections of beams ST, L and PCP3-2 in Figure 5.41 reveals that the beams reinforced with prestressed concrete prisms have significantly less deflection

Figure 5.38: Deflections of beam PCP2

Figure 5.39: Deflections of beams PCP3-1 and PCP3-2

Figure 5.40: Deflections of beams PCP4-1 and PCP4-2

Figure 5.41: Comparison of load-deflection curves for beam ST, L, and PCP3-2

compared to the Leadline reinforced beam. Before cracking of the prisms, the deflection is even less than the steel reinforced beam deflection. Note that the beam ST was designed to carry the same ultimate moment, however, only after testing the beams it was concluded that the FRP reinforced beams cannot be designed using the same formulas as steel reinforced beams. A detailed analysis of such beams will be discussed later. In the following, 40% of the ultimate load of 100 kN will be assumed as serviceability load for all tested beams. Deflection of PCP3 reinforced beam at serviceability load levels are then comparable to the deflection of steel reinforced beam. The deflection of Leadline only reinforced beam at the same level (40 kN) is 1.5 times larger.

A comparison of the load midspan deflection curves of all the prism reinforced beams can be seen in Figure 5.42. The trend of decreasing beam deflection with increasing prestressing in the prisms is evident. After prism cracking, the load deflection curves are basically parallel

Figure 5.42: Comparison of load-deflection curves

and that is because thereafter they all have the same stiffness. Note, however, that delaying cracking produces significantly less deflection at every load level. Notice for instance that beams PCP3 and PCP4 deflect 15 mm at 60 kN while beams PCP1 and CP1 deflect more than 30 mm at the same load level.

5.4.2 Deflected Profile

The deflected profiles of all the tested beams were obtained from the LVTD readings. These profiles are plotted at different load levels as indicated next to each graph. Let us compare the deflected profiles of beams ST, L and PCP3 at 40, 60, and 90% of ultimate load.

Figure 5.43 shows the deflected profile of beam ST at 10, 30, 50, 75, and 80 kN. It is clear

Figure 5.43: Deflected profile of beam ST

that up to 50 kN, the beam experienced very small amount of deflection, but once the steel yielded at 60 kN, the deflection increased substantially, reaching 38 mm at 80 kN. Notice that the deflection profiles are basically symmetrical with respect to the beams centreline, which indicates that the instrumentation functioned properly.

Beam L exhibited a more flexible response due to its flexible Leadline reinforcement. The deflected profiles for beam L are shown in Figure 5.44. Deflection increased rapidly with load, and at 70 kN, or 70 % of ultimate it reached 37 mm. Maximum deflection at ultimate load of 100 kN was 73 mm, which is the same as deflection of beam ST at its ultimate load of 82 kN. Notice that the maximum deflection of beam L at 50 kN is 25 mm, which is nearly four times the corresponding deflection of beam ST.

Figure 5.44: Deflected profile of beam L

Beams CP1 and CP2 behaved very similarly and their deflected shapes at any load varied by less than 3 mm, as can be seen in Figures 5.45, and 5.46. Comparison of deflections of beams L and CP1 and/or CP2 shows that after cracking these were quite similar, and so the prisms without prestressing had no effect on the deflection of the beams.

Beams PCP1-1 and PCP1-2 were reinforced with one PCP1 and one No.8 Leadline bar. Due to the low level of prestressing in the prisms, these beams cracked early in the loading cycle, and their deflection started to increase rapidly from there on. The deflected profiles of beam PCP1-1 are shown in Figure 5.47, and have been plotted for load values of 11, 25, 40, 60, 80, and 100 kN. The strain profiles corresponding to the same load levels are shown in Figure 5.22. The deflected profile of beam PCP1-2 is plotted in Figure 5.48. These two beams show very similar behaviour throughout the loading process. Note that

Figure 5.45: Deflected profile of beam CP1

Figure 5.46: Deflected profile of beam CP2

Figure 5.47: Deflected profile of beam PCP1-1

the maximum deflection of these beams at 40 kN is 15 mm, while the deflection of beam ST at the same load level is around 5 mm. Hence, the level of prestress in these prisms is inadequate for controlling deflection at service load.

The deflected profiles of the rest of the beams reinforced with two concrete prisms with different level of prestressing are presented in Figures 5.49 to 5.53. All the duplicate beams behaved very similarly, except for beam PCP-2, which failed in an unexpected manner. The latter beam failed prematurely at 100.5 kN and the failure appeared to be torsional.

The preceding figures show that with increased level of prestressing deflection decreased, and the beams became correspondingly stiffer. For example, beam PCP3-1 underwent a maximum deflection of 24 mm at 70% of ultimate load (or 80 kN) which is 35% smaller

Figure 5.48: Deflected profile of beam PCP1-2

than the corresponding deflection of beam L at the same load level.

Figure 5.54 compares the deflected profiles of beams ST, L, and PCP3-1 at 32 kN, 48 kN, and 72 kN, which correspond to the 40, 60, and 90% of the ultimate capacity of beam ST. If 40% of ultimate load is considered to be the service load, then it can be seen that both steel and PCP3 reinforced beams behaved similarly under service load and their maximum deflections were 25% of the maximum deflection of beam L. After cracking of the prisms at 60% of the ultimate, the deflection of beam ST is 60% of deflection of beam PCP3-1, and 26% of beam L. Beam PCP3-1 at this load level has a maximum deflection equal to 43% of the corresponding deflection of beam L. Since steel has not yielded at this point, the comparison is more favourable for the steel reinforced beam. However, after yielding, as can be seen in the last portion of Figure 5.54, beam PCP3-1 and ST are again behaving very

Figure 5.49: Deflected profile of beam PCP-2

Figure 5.50: Deflected profile of beam PCP3-1

Figure 5.51: Deflected profile of beam PCP3-2

Figure 5.52: Deflected profile of beam PCP4-1

Figure 5.53: Deflected profile of beam PCP4-2

similarly. It has to be pointed out that in practice the service load rarely exceeds 60% of the ultimate. Therefore, in the design of beams reinforced with PCP, from the serviceability point of view the governing parameter is the level of prestressing in the prisms. The prisms should not crack under 60% of ultimate load to ensure acceptable deflection and, as will be shown next, crack width under service load.

5.5 Crack Pattern and Size

This section will present the crack patterns of the beams, and their load versus crack width curves. Here, only cracking due to tensile stresses created by applied loads are discussed. Cracking due to thermal or other environmental effects are beyond the scope of the present

Figure 5.54: Deflected profiles of beams ST, L, and PCP3-1 at 40, 60, and 90% of P_{ult}

Figure 5.55: Typical tracing mesh

study.

To observe the cracks and their propagation during the test, a 100×50 mm mesh was drawn on one face of the beam. The crack progression was traced on the surface using the mesh as a guide. Figure 5.55 shows a typical beam with the mesh and the cracks. The crack width was measured using a hand-held microscope with accuracy of 0.02 mm. The observed cracking patterns were directly related to the type of reinforcement used in the beam. These patterns can be seen in Figures 5.56 to 5.62. The crack patterns in the preceding figures are plotted at the ultimate load.

Figure 5.56 shows the cracking pattern of the beam ST at ultimate load. The cracks are primarily flexural and they reach approximately the neutral axis. Therefore, in the constant moment region, they all have the same length, but they have a smaller length in the shear spans. Very minor flexure-shear cracks are observed in the right shear span. The average crack spacing is 80 mm.

A completely different crack pattern was observed in beam L, Figure 5.57. Cracks are all

Figure 5.56: Cracking pattern of beam ST at $P_{ult} = 82$ kN

the same height, as they stopped at the level of compression reinforcement. We observe some bond failure which developed at ultimate when Leadline ruptured. The average crack spacing was 123 mm. Very similar cracking pattern can be observed in beam series CP, Figure 5.58. The average crack spacing for the beam CP1 and CP2 was 121 and 118 mm, respectively. The larger crack spacing indicates that the flexural reinforcement was under higher stresses than in the steel reinforced beam.

The cracking patterns of the beams PCP-2, PCP-3, and PCP-4 are in Figures 5.60 to 5.62. All the beams experienced very similar cracking pattern. The cracks are mostly flexural cracks, and flexure-shear cracks. The average crack spacing for the beams PCP-2, PCP-3, and PCP-4 is 106 mm, 90 mm, and 80 mm respectively. We notice that the crack spacing decreased with increased level of prestressing in the prisms.

The crack width versus the applied load is plotted in Figures 5.63 to 5.65. During the test flexural cracks occurred first. In all the beams reinforced with Leadline and/or PCP these cracks rapidly propagated all the way up to the compression steel level. In the case of

Figure 5.57: Cracking pattern of beam L at $P_{ult} = 100$ kN

Figure 5.58: Cracking pattern of beam CP1 at $P_{ult} = 95$ kN

Figure 5.59: Cracking pattern of beam PCP1-1 at $P_{ult} = 106$ kN

Figure 5.60: Cracking pattern of beam PCP2 at $P_{ult} = 100$ kN

Figure 5.61: Cracking pattern of beam PCP3-1 at $P_{ult} = 115$ kN

Figure 5.62: Cracking pattern of beam PCP4-2 at $P_{ult} = 110$ kN

steel reinforced beam, the crack propagation was gradual. The width of the cracks varied depending on the type of reinforcement.

Let us again assume that the service load for these beams is 40 kN. Then we can see in Figures 5.63 to 5.65 that the maximum crack width at 40 kN in beams ST, L, CP1, PCP1-1, PCP-2, PCP3-2, and PCP4-2 are 0.6 mm, 3.2 mm, 4.1 mm, 2 mm, 1.95 mm, 0.25 mm, and 0.55 mm respectively. Observe that the maximum crack width at 40 kN differs between the duplicate beams in each set. The above mentioned crack sizes are the smaller of the two values.

It is evident from the above crack widths, that in beams PCP3-2 and PCP4-2 the maximum crack size at 40 kN is smaller than the corresponding crack size in the conventional steel reinforced beam. The crack width versus load graphs for beams ST, L, and PCP3-1 in Figure 5.66 show the positive effect of prestress in the prism on crack width. Hence once again the PCP reinforced beams with an adequate level of prestressing in the prisms satisfy the crack width requirements. Notice that from the aesthetics point of view cracks up to 0.5 mm wide are considered acceptable by the current design standards. In the present tests, the maximum crack width in beam ST is greater than 0.5 mm at 40 kN, hence this beam would not satisfy the current crack width requirements in the code.

In the ACI code, the crack width at any section is assumed to be directly proportional to the stress in the flexural reinforcement at the section of interest. Alternatively, the code states that the steel stress may be assumed as $0.6f_y$ for the purpose of crack width calculation (MacGregor 1997). If we apply the latter limit to the current test beams, then the crack

width corresponding to a load of $0.6P_y$ needs to be calculated, where P_y is the load initiating yielding in the flexural steel reinforcement. By reference to Figure 5.34, the steel began yielding at $P=60$ kN, hence the crack width corresponding to 36 kN needs to be calculated and compared to the crack width limit in the code. Alternatively, $P=36$ kN is the service load for the beams ST. Since this beam failed at 82 kN, the ratio of service load/ultimate load is 0.45, which seems reasonable.

In order to assess the performance of the PCP reinforced beams, insofar as crack width limitation is concerned, we will evaluate the crack width of all the beams at $P=36$ kN.

The maximum crack width of beams ST, L, CP1, PCP-1, PCP-2, PCP3-2, and PCP4-2 at $P=36$ kN can be estimated from Figures 5.63 to 5.65 to be 0.5 mm, 3mm, 2.6 mm, 1.9 mm, 2.2 mm, 0.2 mm, and 0.4 mm, respectively. Since the maximum allowable crack width according to ACI and CSA codes is 0.4 mm, only beams PCP3-2 and PCP4-2 would satisfy the prescribed limit, but beam ST would not. It is true that the ultimate load capacity of the latter PCP reinforced beams is greater than the ultimate capacity of beam ST, but it is not likely that the actual ultimate capacity can be utilized in practice, without violating the serviceability requirements. Notice, however, that the crack width in beams PCP3-2 and PCP4-2 at 45% of their respective ultimate loads is approximately the same as the crack width in beam ST at 45% of its ultimate capacity, Figure 5.66.

The key to maintaining the crack width small and acceptable in PCP reinforced beams is to ensure that the prisms do not crack under service conditions. The current results demonstrate that it is possible to satisfy the two serviceability requirements, namely limited

deflection and crack width, using a reasonable level of prestressing in the prisms. As can be seen in Figure 5.63, the crack width for beam CP1 at $0.45P_u=40.5$ kN, where $P_u=90$ kN is its ultimate load, is 3 mm, which is significantly larger than the 0.4 mm limit set by the code.

The crack patterns in Figures 5.56 to 5.62 show that in the FRP reinforced beams the cracks are spaced farther apart than in the steel reinforced beam. The average crack spacing for the test beams is shown in Figure 5.67.

Since crack width is normally assumed to be directly proportional to crack spacing, and to the relative strain in the reinforcement, the closer crack spacing helps to reduce crack width. The larger strain in unstressed FRP reinforcement generally masks the benefit of closer spacing. However, in cracked prisms, the strain in the reinforcement is small, therefore the crack spacing has a noticeable effect on the crack width.

Figure 5.63: Crack width versus load for beams ST, L, CP1, and CP2

Figure 5.64: Crack width versus load for beams PCP1-1, PCP1-2, PCP-2, PCP3-1, and PCP3-2

Figure 5.65: Crack width versus load for beam PCP4-1, and PCP4-2

Figure 5.66: Comparison of crack width in beams ST, L, and PCP3-1

Figure 5.67: Average crack spacing of various beams

Chapter 6

Serviceability and Ultimate Limit States

6.1 General

The main objective of this thesis is to investigate the serviceability and ultimate limit state flexural behaviour of simply supported beams, reinforced with PCP. This chapter will discuss the effect of the level of prestressing in the prisms on the load-deflection behaviour of the beams. A theoretical model for calculation of deflections of such beams will be presented and verified against experimental results. Cracking of the prisms is very important for the load-deflection behaviour of PCP reinforced beams; therefore, a correct estimate of the cracking moment is a crucial part of the analysis. This moment will be theoretically derived in this chapter and the results will be compared to the corresponding experimental value.

The cracking pattern of the beams will also be discussed. Crack width of the FRP reinforced test beams will be compared against the crack width of the steel reinforced beam and the feasibility of using code expressions for calculation of the crack width for beams reinforced with PCP will be assessed. A new model for calculation of crack width is presented and

verified against the experimental results.

The ultimate strength of the beams as well as their failure mode is presented. The well-known procedure for calculating the ultimate capacity of reinforced concrete beams is modified to make them suitable for application to PCP reinforced beams.

To the knowledge of the writer, the experimental work in this thesis is the only one up to now dealing with the concept of reinforcing concrete beams with FRP prestressed concrete prisms. Therefore, most of the newly derived formulas in this chapter could not be validated by any other suitable experimental data.

6.2 Load-Deflection Behaviour

The deflection at any point on any type of beam and at any load level can be calculated if the true moment-curvature relationship for the beam is known. This follows from the following well-known equation of mechanics

$$\Delta_A = \int_0^L \frac{mM}{EI} dx \quad (6.1)$$

where Δ_A is the desired deflection of a point A on the beam, m is the bending moment due to a unit load applied at point A and in the direction of desired deflection, M/EI is the curvature of the beam, and L is the beam span.

For FRP reinforced concrete members without prestressed prism reinforcement, at any section $EI = EI_g$ when $M < M_{cr}$, and $EI = EI_{cr}$ when $M \geq M_{cr}$. Extensive calculations (Ali 1996, Abushoglin 1997) have shown that tension-stiffening (i.e. tension

carried by the concrete between two cracked sections) is not significant in the case of FRP reinforced concrete members. This may be due to the fact that the curvature of an FRP reinforced concrete member increases rather dramatically after cracking compared to a similar steel reinforced member. It is well-known that the effect of tension-stiffening decreases with increased level of curvature (Ghali 1993).

Equation 6.1 is general and it applies to any loading and support condition. For practical purposes, however, it would be convenient to derive an explicit relationship between the maximum deflection and the applied load for the more common types of loading and support conditions.

Before we present the proposed method of deflection calculation, let us first discuss the existing methods of deflection calculation for FRP reinforced concrete beams.

6.2.1 Existing Methods for Calculation of Deflections of FRP Reinforced Beams

ACI 318-2 and CAN3-A23.3-M94 (1995) recommend the use of the so-called effective moment of inertia, I_e , to calculate the deflection of cracked steel reinforced concrete members. The procedure entails the calculation of a uniform moment of inertia throughout the beam length which is inserted into deflection equations derived from linear elastic analysis. For a beam under four point bending this expression is

$$\Delta = \frac{Pa}{24EI_e}(3L^2 - 4a^2) \quad (6.2)$$

where P is half of the total load, a is the shear span of the beam, L is the total span of the beam, and EI_e is the rigidity of the beam. The effective moment of inertia I_e , is based on semi-empirical considerations, and despite some doubt about its applicability to conventional reinforced concrete members subjected to complex loading and boundary conditions, it has yielded satisfactory results in most practical applications over the years.

Investigators initially applied the code-based effective moment of inertia concept to FRP reinforced concrete members, but they discovered that predictions of deflections based on this concept were not in agreement with experimental data. Accordingly, attempts were made to modify the I_e expression in order to make it applicable to FRP reinforced concrete members. The modified expressions were again based on the assumption that a uniform moment of inertia exists in all cases which can be substituted for the actual variable moment of inertia of the beam along its length.

As stated earlier, in North American codes calculation for flexural members are mainly based on equations derived from linear elastic analysis, with the moment of inertia used in those equations being the effective moment of inertia, I_e , which is given by

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^3 I_g + \left[1.0 - \left(\frac{M_{cr}}{M_a} \right)^3 \right] I_{cr} \leq I_g \quad (6.3)$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \quad (6.4)$$

In the above equation, M_{cr} is the cracking moment, M_a is the maximum moment in a member at the load stage at which deflection is being calculated, I_g is the moment of inertia of gross concrete section about the centroidal axis, I_{cr} is the moment of inertia of cracked

section transformed to concrete, f_r is the modulus of rupture of concrete, y_t is the distance from the centroidal axis of gross-section to extreme fibre in tension.

Experimental data reveal that Equation 6.3 is not applicable to FRP reinforced concrete members. Hence, Faza and Ganga Rao (1991) suggested the use of a modified moment of inertia, I_m . I_m was derived based on predetermined and assumed variation of moment of inertia of the beam along its length, irrespective of the actual variation of moment. A beam section is assumed to have a moment of inertia of either I_e , or I_{cr} . For instance, the moment of inertia of a centrally loaded simply supported beam was assumed equal to I_{cr} in the central one third of the span, and equal to I_e in the remaining 2/3 adjacent to the supports. For a simply supported beam loaded with a uniformly distributed load, the central one-half of the beam was assumed to have a moment of inertia I_{cr} and the remaining quarter lengths of the beam adjacent to each support were considered to have moment of inertia I_e .

Using those assumptions and the so-called virtual work-method, explicit expressions were derived for I_m . Although the calculated deflections based on I_m agreed reasonably well with their experimental work, particularly at higher levels of loading, the same agreement has not been achieved by others (Theriault and Benmokrane 1998).

For example, Benmokrane et al. (1996) used the following expression for I_e to calculate the deflection of some GFRP reinforced beams:

$$I_e = \alpha I_{cr} + \left(\frac{I_g}{\beta} - \alpha I_{cr} \right) \left(\frac{M_{cr}}{M_a} \right)^3 \quad (6.5)$$

α and β are empirical correction factors. Based on their own experimental data, they recommended $\alpha = 0.84$ and $\beta = 7$. However, since the latter correction factors were

empirically derived from limited test data, their applicability to other loading and boundary conditions is neither evident, nor assured. For instance, Theriault and Benmokrane (1998) introduced additional corrective terms in Equation 6.5 in order to match their experimental data.

There are several reasons to believe that such empirical modifications would be difficult to generalize. First, the fundamental relation that governs deflection is the moment-curvature relation for a section and the magnitude and shape of the moment diagram along the span. Any arbitrary assumptions with regard to these quantities and their variation will introduce arbitrary errors in the calculated values. Since FRP or PCP reinforced concrete members have a fundamentally different moment-curvature response than conventional steel reinforced concrete members, the application of expressions based on the moment-curvature relation of steel reinforced concrete to FRP reinforced concrete may be futile. It is reasonable to begin with the moment-curvature of FRP reinforced concrete and use first principles to obtain the deflection of FRP reinforced concrete members.

6.2.2 Moment-Curvature Relation of FRP Reinforced Concrete Sections

The moment-curvature relation of any section can be determined by considering the equilibrium and compatibility conditions of the section and the constitutive or stress-strain relation of the materials involved.

Consider a general FRP reinforced concrete section and its attendant strain and stress distribution profiles in the post-cracking stage as illustrated in Figure 6.1. The curvature,

Figure 6.1: Strain and stress profiles of FRP reinforced beam after cracking

ψ , at any load level is given by

$$\psi = \frac{\epsilon_c}{c} = \frac{\epsilon_{FRP}}{(d - c)} \quad (6.6)$$

where ϵ_c is the concrete strain in the extreme fiber in compression, c is the depth of the neutral axis, d is the effective depth of the beam, ϵ_{FRP} is the strain in the FRP layer located at a distance d from the extreme fiber in compression.

Assuming pure bending, the construction of the moment-curvature relation involves the assumption of various values for ϵ_c and the determination of the corresponding c values. The value of c is determined from the requirement of equilibrium of forces acting on the section, i.e. $C = T$ in Figure 6.1. The resultant moment, M , corresponding to the assumed strain profile is $M = Tjd$, where C is the resultant compressive force acting on the uncracked concrete, T is the net tension in the reinforcement, and jd is the internal lever arm.

Using such a procedure, the moment curvature relation of any FRP reinforced concrete section can be determined. Fig. 6.2 shows the moment-curvature diagram of beam L, reinforced with two CFRP rebars with total area of 98mm^2 , and the effective depth of 170

Figure 6.2: Comparison of experimental and calculated moment-curvature relation for beam L

mm. The solid line represents the calculated moment-curvature diagram derived assuming FRP reinforcement to be linear elastic, and concrete to be nonlinear with its compressive stress-strain relationship following Hognestad's parabolic equation (MacGregor 1997), and the dashed line is the experimentally found moment-curvature diagram for beam L. Although there appears to be a discrepancy between the experimental and calculated moment curvature in the pre-cracking stage, this is because no measurements were taken prior to cracking of the FRP beam. Therefore no information on curvature of this beam was available at its uncracked stage. This moment-curvature was obtained from hand measurements of demec points on concrete in the mid-span of the beam.

Figure 6.3: Theoretical model of moment-curvature diagram for FRP reinforced beams

Notice that the moment-curvature relation of the FRP reinforced sections in the post-cracking state is essentially linear up to at least 3/4 of the ultimate moment capacity regardless of the type and amount of reinforcement. In view of the elastic response of FRP reinforced beams, their moment-curvature relation can be represented by a tri-linear curve as indicated by the solid line in Figure 6.3. The latter assumption leads to the conclusion that determination of the moment-curvature diagram of an FRP reinforced concrete member only requires knowledge of EI_{cr} , EI_g , and M_{cr} . The same information that is needed to calculate I_e according to the present codes.

6.2.3 Moment-Curvature of PCP Reinforced Section

The moment-curvature relationship of a PCP reinforced section can be obtained by considering its equilibrium and compatibility as discussed in Section 6.2.2. When examining

Figure 6.4: Theoretical model of moment-curvature diagram for PCP reinforced beams

the moment-curvature relationship for a PCP reinforced section, it can be seen from Figures 5.30 and 5.31 in Chapter 5 that the relation is tri-linear. As shown in Figure 6.4, the stiffness of the section changes twice: first when the concrete cracks, it becomes $EI_{cr,1}$, while after the prism cracks, it changes to $EI_{cr,2}$, where the subscripts 1 and 2 refer to stages 1 and 2. This model will be used to calculate deflection of PCP reinforced beams, and the results will be shown in Section 6.2.7.

A linear $M - \psi$ relation in the post-cracking state implies that the FRP, or PCP reinforced section behaves elastically. In an elastic member, the position of the neutral axis is not a function of the level of stress. The constancy of the neutral axis position in cracked FRP reinforced concrete members, unlike steel reinforced concrete members, can be easily seen in Figures 5.20 to 5.25 for CFRP reinforced beams as found in the current investigation, and in reported experimental work of Benmokrane et al. (1996), Theriault and Benmokrane

(1998) on GFRP reinforced beams. It can also be proven theoretically as follows.

6.2.4 Position of Neutral Axis in Cracked FRP Reinforced Members

With reference to Figure 6.1

$$M = Tjd \quad (6.7)$$

$$T = E_{FRP}A_{FRP}\epsilon_{FRP} \quad (6.8)$$

$$jd = d - k_2c \quad (6.9)$$

where E_{FRP} and A_{FRP} are the elastic modulus and area of the FRP reinforcement, respectively, jd is the internal lever arm and k_2 is a parameter which relates the position of the resultant concrete compressive force C to the position of the neutral axis c . Although k_2 is theoretically dependent on the shape of the compressive stress block in the uncracked concrete zone (i.e. linear versus parabolic) and on concrete strength, it is not a function of c directly (MacGregor 1997). Substituting for ϵ_{FRP} in terms of ψ from Equation 6.6 into Equation 6.8 and then substituting Equation 6.8 and 6.9 into Equation 6.7 gives

$$M = [E_{FRP}A_{FRP}(d - c)(d - k_2c)]\psi \quad (6.10)$$

Equation 6.10 indicates that for the $M - \psi$ relation to be linear, the term in the square bracket on the right-hand side of this equation must remain constant, which in turn requires that c be a constant.

A constant c implies that for a cracked section, the moment of inertia is constant, regardless of the level of applied moment. As noted earlier, experimental observations and theory

Figure 6.5: Comparison of c/d for all beam types

both confirm that in FRP reinforced sections there is little movement in the position of the neutral axis after cracking. The linearity of the moment-curvature relation at different load stages makes the calculation of deflection in FRP reinforced concrete members simple and straightforward.

The c/d ratio is plotted for all the present test beams in Figure 6.5. Notice the large jump in c/d for the beam reinforced with Leadline immediately after cracking. After that, the ratio is constant from about 20% of ultimate to 80% of ultimate. The same behaviour can be observed for the PCP reinforced beams. In those beams as anticipated, c/d becomes constant after the prisms crack. Before prism cracking, the change in c/d for the PCP reinforced beam is gradual and is similar to its variation in the steel reinforced beam between cracking and yielding.

6.2.5 Deflection of Beams Reinforced with FRP

Consider a simply supported beam in Figure 6.6(a) subjected to four point bending. The curvature, M/EI , diagram of the beam is illustrated in Figure 6.6(b), where sections at a distance less than L_g from the support are considered uncracked, because the moment is less than M_{cr} . For calculation of deflection under the load P , we apply a unit load at the center of the beam as in Figure 6.6(c) and the corresponding moment diagram is shown in Figure 6.6(d).

The deflection is determined in two steps. First, let us assume that the beam is cracked throughout its length. Then the beam would have a constant moment of inertia I_{cr} , and the central deflection δ_{cr} is given by

$$\delta_{cr} = \int_0^L m \frac{M}{EI_{cr}} dx \quad (6.11)$$

In the case of four point bending, superposition is used and the actual integral is

$$\delta_{cr} = 2 \int_0^a m \frac{M}{EI_{cr}} dx + 2 \int_a^{L/2} m \frac{M}{EI_{cr}} dx \quad (6.12)$$

where

for $0 < x \leq a$, $M = Px$,

for $a < x \leq L/2$, $M = Pa$

and $m = x/2$. This leads to the final expression

$$\delta_{cr} = \frac{Pa}{24EI_{cr}} (3L^2 - 4a^2) \quad (6.13)$$

which is the expected deflection based on strength of materials.

Figure 6.6: Virtual work method for deflection calculations

Next, since the beam is uncracked over the distance L_g from the supports to the point where $M = M_{cr}$, we correct δ_{cr} by the amount δ_c , where δ_c is given by

$$\delta_c = 2 \int_0^{L_g} m \frac{M}{EI} dx \quad (6.14)$$

but

$$\frac{M}{EI} = \left(\frac{M_{cr}}{EI_{cr}} - \frac{M_{cr}}{EI_g} \right) \frac{x}{L_g} \quad (6.15)$$

Equation 6.15 is the equation for the curvature difference between a cracked and an uncracked section over the distance L_g . Inserting Equation 6.15 into Equation 6.14 and performing the integration yields

$$\delta_c = \frac{M_{cr}}{EI_{cr}} \left(1 - \frac{I_{cr}}{I_g} \right) \frac{L_g^2}{3} \quad (6.16)$$

From Figure 6.6(b)

$$M_{cr} = PL_g \quad (6.17)$$

which when substituted into Equation 6.16, gives

$$\delta_c = \frac{PL_g^3}{3EI_{cr}} \left(1 - \frac{I_{cr}}{I_g} \right) \quad (6.18)$$

Finally, the actual maximum deflection of the beam Δ is given by

$$\Delta = \delta_{cr} - \delta_c \quad (6.19)$$

Substituting for δ_{cr} and δ_c from Equation 6.13 and 6.18 respectively into Equation 6.19, yields

$$\Delta = \frac{Pa}{24EI_{cr}} \left[3L^2 - 4a^2 - 8 \frac{L_g^3}{a} \left(1 - \frac{I_{cr}}{I_g} \right) \right] \quad (6.20)$$

If we let

$$\eta = 8 \left(\frac{L_g}{L} \right)^3 \left(1 - \frac{I_{cr}}{I_g} \right) \quad (6.21)$$

then

$$\Delta = \frac{Pa}{24EI_{cr}} \left[3L^2 - 4a^2 - \eta \frac{L^3}{a} \right] \quad (6.22)$$

It was shown that the magnitude of deflection depends on the ratio of the lengths of the uncracked and cracked portions of the beam. Of course this ratio is a function of the applied load and it progressively becomes smaller with increased loading. Furthermore, Equation 6.22 applies to completely uncracked, partially cracked, or fully cracked beams, while Equation 6.3 cannot be applied if $M_a < M_{cr}$, although the code does stipulate that $I_e \leq I_g$.

6.2.6 Verification of the Proposed Method

The method developed for calculation of deflection of FRP reinforced concrete beams was verified against experimental results of other researchers. Eight beams that were experimentally tested by others (Al-Sayed et al. 1995, Al-Sayed et al. 1996, Brown and Bartholomew 1996, Maruyama and Zhao 1996, Nawy and Neuwerth 1977, Zhao et al. 1995) under four point bending were modelled. They contained either CFRP or GFRP flexural reinforcement. The stirrups were FRP for the beams of Maruyama and Zhao (1996), and Zhao et al. (1995), unlike other beams were reinforced with steel stirrups. Material properties of these beams are shown in Table 6.1, and their geometry and loading are shown in Table 6.2.

Table 6.1: Material properties of analyzed beams

Parameters [MPa]		Beam Designation							
		Brown	Alsayed		Nawy	Maruyama		Zhao	
		D-1	III	B	9	26	30	5	19
Concrete	f'_c	35.00	31.30	35.53	29.67	34.00	29.50	34.30	34.30
	f_t	3.55	3.30	3.50	3.00	4.20	3.30	3.51	3.51
	E_c	29580.0	28000.0	29800.0	27235.0	29000.0	27160.0	29283.1	29283.1
Reinforcement	Type of FRP	GFRP	GFRP	GFRP	GFRP	CFRP	CFRP	CFRP	CFRP
	f_{FRP}	550.00	886.20	850.59	724.50	1200.00	1200.00	1124.00	1124.00
	E_{FRP}	41400.0	43370.0	35630.0	26200.0	100×10^3	100×10^3	105×10^3	105×10^3
	ϵ_{FRP}	0.013	0.014	0.020	0.028	0.012	0.012	0.011	0.011

The calculated deflections of these beams are plotted in Figures 6.7 to 6.13 and compared to the experimental values. The experimental load-deflection curve in each case is represented by the solid line, while the calculated values are represented by the star symbols. All beams reinforced with steel stirrups show very good agreement. However, when the beam is reinforced with FRP stirrups, the deflection found experimentally is larger due to the effect of shear deformation. The effect of shear was not incorporated in the above developed formula, and therefore there is a discrepancy with the experimental results. It is important for future work that shear deformations be properly considered and included in the total deformation of a member.

Table 6.2: Geometry of analyzed beams

Designation	Elevation	Cross-Section	$I_g \times 10^6$ mm ⁴	$I_{cr} \times 10^6$ mm ⁴	M_{cr} kN.m
Brown et al. Beam D-1 (1996)		152.4 101.6 2#3 GFRP bars	30.1	2.0	1.4
Alsayed et al. Beam III (1995)		260 200 4#12.7 GFRP bars	295.5	27.1	6.8
Alsayed et al. Beam B (1996)		360 200 3#19.0 GFRP bars	780.4	76.8	15.3
Nawy et al. Beam 9 (1977)		305 127 6 GFRP bars (758.52 mm ²)	299.3	41.6	3.3
Maruyama Beam 26 (1996)		300 150 CFRP bars (390.00 mm ²)	346.8	59.3	9.8
Maruyama Beam 30 (1996)		580.0 300 CFRP bars (1560.00 mm ²)	5058.0	1002.0	58.5
Zhao et al. Beam 5 (1995)		300.0 150 CFRP bars (568.00 mm ²)	351.7	83.1	8.4
Zhao et al. Beam 19 (1995)		300.0 150 CFRP bars (568.00 mm ²)	351.7	83.1	8.4

Figure 6.7: Deflection of beam 3 by Al-Sayed

Figure 6.8: Deflection of beam B by Al-Sayed

Figure 6.9: Deflection of beam 26 by Maruyama

Figure 6.10: Deflection of beam 30 by Maruyama

Figure 6.11: Deflection of beam 9 by Nawy

Figure 6.12: Deflection of beam 19 by Zhao

Figure 6.13: Deflection of beam 5 by Zhao

6.2.7 Proposed Method for Deflection Calculation of Beams Reinforced with PCP

Similarly to the derivation of deflection of FRP reinforced concrete beam, we consider a simply supported beam under four point bending. Since the moment-curvature diagram of PCP reinforced beam is tri-linear, moments corresponding to the initiation of beam cracking, moment $M_{cr,1}$, and prism cracking moment, $M_{cr,2}$, are of importance for calculation of deflection. Evidently, due to the prestress in the prism and the different material properties of the prism, $M_{cr,1} < M_{cr,2}$.

Firstly, let us assume that the applied moment M_a is smaller, or equal to $M_{cr,2}$. This means that the beam is either fully uncracked, or the concrete of the beam is cracked in some regions, however, the prisms are uncracked. The uncracked length of the beam is designated as L_1 as shown in Figure 6.14(b).

Figure 6.14: Terms used in calculating deflection of beams reinforced with PCP.

To calculate deflection of such beam, we first assume that the beam is fully cracked, and therefore as derived earlier, the maximum deflection, $\delta_{cr,1}$, is:

$$\delta_{cr,1} = \frac{Pa}{24EI_{cr,1}} (3L^2 - 4a^2) \quad (6.23)$$

Next, similarly to our previous derivation, a correction factor is found, $\delta_{c,1}$, given by Equation 6.14.

$$\delta_{c,1} = \frac{PL_1^3}{3EI_{cr,1}} \left(1 - \frac{I_{cr,1}}{I_g}\right) \quad (6.24)$$

The final deflection for $M_a \leq M_{cr,2}$ is then given by

$$\Delta_1 = \delta_{cr,1} - \delta_{c,1} \quad (6.25)$$

or,

$$\Delta_1 = \frac{Pa}{24EI_{cr,1}} (3L^2 - 4a^2) - \frac{PL_1^3}{3EI_{cr,1}} \left(1 - \frac{I_{cr,1}}{I_g}\right) \quad (6.26)$$

Equation 6.26 predicts the deflection of a PCP reinforced beam for a load lower than that needed to crack the prisms. During the experimental testing of beams reinforced with PCP, a noticeably smaller crack width of these beams was evident in comparison with steel reinforced beams. This lead to the conclusion that the concrete surrounding the cracks in the tension zone of the beam carries higher tension than that in a steel reinforced cracked section. Therefore, a tension stiffening model was incorporated into the deflection formula in Equation 6.26. The tension stiffening coefficient, ξ_1 was found based on Ghali (1993) to be

$$\xi_1 = 1 - 0.7\sqrt{\frac{L_1}{a}} \quad (6.27)$$

where L_1 is the uncracked length of the beam. The deflection Δ_1 then becomes

$$\Delta_1 = \xi_1 \frac{Pa}{24EI_{cr,1}} (3L^2 - 4a^2) - \xi_1 \frac{PL_1^3}{3EI_{cr,1}} \left(1 - \frac{I_{cr,1}}{I_g}\right) \quad (6.28)$$

Let us now consider $M_a > M_{cr,2}$. Using the same approach, we firstly consider the beam fully cracked and having stiffness $EI_{cr,2}$, then

$$\delta_{cr,2} = \frac{Pa}{24EI_{cr,2}} (3L^2 - 4a^2) \quad (6.29)$$

Since the beam is not fully cracked, only some regions of the beam are fully cracked, two correction parameters are introduced. Correction parameter $\delta_{c,2}$, to account for the regions of the beam with uncracked prisms, given by Equation 6.14, with the upper limit of the integral in this case being L_2 , and correction parameter $\delta_{c,1}$, to account for the completely uncracked regions of the beam, given by Equation 6.24. The final deflection for $M_a \geq M_{cr,2}$ is

$$\Delta = \delta_{cr} - \delta_{c,2} - \delta_{c,1} \quad (6.30)$$

or,

$$\Delta = \xi_2 \frac{Pa}{24EI_{cr,2}} (3L^2 - 4a^2) - \xi_2 \frac{PL_2^3}{3EI_{cr,2}} \left(1 - \frac{I_{cr,2}}{I_g}\right) - \xi_1 \frac{PL_1^3}{3EI_{cr,1}} \left(1 - \frac{I_{cr,1}}{I_g}\right) \quad (6.31)$$

where ξ_1 is the tension stiffening coefficient defined in Equation 6.27, and ξ_2 is defined as

$$\xi_2 = 1 - 0.7 \sqrt{\frac{L_2}{a}} \quad (6.32)$$

where L_2 is the length of the beam with uncracked prism. For better identification of the terms used in Equation 6.31, refer to Figure 6.14(c).

Figure 6.15: Comparison of experimental and calculated deflection for beam PCP2

Figure 6.16: Comparison of experimental and calculated deflection for beam PCP3-1

Figure 6.17: Comparison of experimental and calculated deflection for beam PCP3-2

Figure 6.18: Comparison of experimental and calculated deflection for beam PCP-4

Equation 6.31 was used to calculate deflection of beams PCP-2, PCP-3 and PCP-4. As can be seen in Figures 6.15 to 6.18, the above equation gives very good results.

Since the deflection of PCP reinforced beams can be accurately predicted, it is feasible now to predict the behaviour of a fictitious beam whose moment at prism cracking would be equal to the yield moment of beam ST in the present investigation. The comparison of the load-midspan deflection response of such a beam with the corresponding response of beam ST is shown in Figure 6.19. Figure 6.19 clearly demonstrates that PCP reinforced concrete beams are capable of the same, or even improved, serviceability behaviour as steel reinforced beams. However, more experimental work needs to be performed to develop concrete mixes which can reach a tensile strength of up to 20 MPa to avoid the possibility of splitting cracks in the prisms due to higher prestress than used in the current investigation. An attempt was made in the present investigation to increase the level of prestress in the 40 mm square prisms, but due to their small cross section and inadequate tensile strength, they split upon release of the prestress.

6.3 Effect of Prestress in the Prisms on Cracking

In the present experimental work three different levels of prestressing in the prisms were successfully used. The level of prestressing in the prisms determined the load at which the prism cracked and the overall serviceability behaviour of the beams. A detailed discussion of the beam cracking moment, the crack width and the crack pattern is presented in the following sections.

Figure 6.19: Comparison of experimental deflection for beam ST and beam PCP-5

6.3.1 Elastic Analysis

The basic principles of solid mechanics will be applied to theoretically analyze the test beams. There are two important stages in the service life of beams reinforced with PCP: first, cracking of the beam while the prisms remain uncracked, and second, cracking of the prisms. If the prestress in the prisms is reasonably high, beam cracking does not affect the behaviour of the beam noticeably, in other words, neither the deflection nor the crack width increases appreciably.

After prism cracking, the deflection increases rapidly, hence, it is very important that under service loads, the prisms remain uncracked. Therefore, high prestress and concrete with very high tensile strength should be used. The high tensile strength is needed to prevent possible radial cracking of the prisms due to the high prestress. With the preceding

Figure 6.20: Transformed sections

considerations in mind, we can analyze cracked PCP reinforced beams under service loads and assume elastic behaviour as follows.

Due to elastic behaviour, we use the transformed section approach as shown in Figure 6.20. Three moments of inertia need to be calculated. The moment of inertia of the uncracked transformed section, I_{tr} , which for the current test beams equals $2.15 \times 10^8 \text{ mm}^4$. After concrete cracking, the depth of the neutral axis is given by kd , which can be calculated as usual by considering the first moment of the compressive and tensile areas of the section, i.e.

$$\frac{b(kd)^2}{2} = n_{sika} A_{PCP} (d - kd) + n_{FRP} A_{FRP} (d - kd) \quad (6.33)$$

After determining kd , the moment of inertia of the section can be calculated using the

following basic equation:

$$I_{cr,1} = \frac{b(kd)^3}{3} + n_{sika}A_{PCP}(d - kd)^2 + n_{FRP}A_{FRP}(d - kd)^2 = 51.57 \times 10^6 mm^4. \quad (6.34)$$

Similarly the moment of inertia of the completely cracked section, $I_{cr,2}$, i.e. the section where both concrete in the tension zone as well as the concrete of the prism is cracked was found to be $2.088 \times 10^6 mm^4$. (Refer to Figure 6.20 for details.) Note that the moments of inertia are the same for all the test beams reinforced with the prisms.

The value of the depth of the neutral axis was found using Equation 6.33. For a known moment applied to this section, the stresses through the depth of the cross-section can be determined. From the stresses, the internal forces are found, and the moment at any load level is calculated as follows:

$$M = \frac{1}{3}f_c(kd)^2b + f_{PCP}A_{PCP}(d - kd) + f_{FRP}A_{FRP}(d - kd) \quad (6.35)$$

Since the stress distribution in the cross section is linear, each stress may be expressed in terms of the stress in the prism; for example, $f_c = (kd)(f_{PCP})/(d - kd)$. Therefore, after minor manipulation of Equation 6.35, the following expression for moment is arrived at

$$M = \left[\frac{b(kd)^3}{3(d - kd)} + n_{sika}A_{PCP}(d - kd) + n_{FRP}A_{FRP}(d - kd) \right] f_{PCP}. \quad (6.36)$$

To find the moment at which the prism will crack, the stress f_{PCP} has to exceed the decompression stress, $f_{p,eff}$ and the tensile strength of the prism concrete, f_t , or $f_{PCP} \geq f_{p,eff} + f_t$.

$$M_{cr} = \left[\frac{b(kd)^3}{3(d - kd)} + n_{sika}A_{PCP}(d - kd) + n_{FRP}A_{FRP}(d - kd) \right] (f_{p,eff} + f_t) \quad (6.37)$$

Table 6.3: Effective prestress at the time of testing

Beam	c_{exp} [mm]	$M_{cr,exp}$ [kNm]	$f_{p,eff}^{cal}$ [MPa]	$f_{p,eff}^{exp}$ [MPa]
PCP-2	65.68	11.05	400	290
PCP3-1	68.00	17.00	760	590
PCP3-2	74.59	17.00	680	610
PCP4-2	85.00	16.15	400	440

Using Equation 6.37 and the experimental values of the cracking moments of the test beams, the effective prestressing in the prisms was estimated, assuming that the Sika concrete has a tensile strength of 9 MPa (see Table 6.3). The 9.0 MPa tensile strength is the value published by the manufacturer, but similar values were also obtained from the laboratory tests. Table 6.3 shows that the loss of prestress was not as dramatic as was assumed earlier (the two values for $f_{p,eff}$ in Table 6.3 are: $f_{p,eff}^{cal}$, calculated using the experimental M_{cr} , and $f_{p,eff}^{exp}$, which was recorded using strain gauge measurement prior to embedment of the prisms into the concrete beams). Equation 6.37 can be used to estimate the needed level of prestressing for the prism. Note that the value of $f_{p,eff}^{cal}$ depends on the value of $M_{cr,exp}$, and also c_{exp} . The latter quantity was obtained from the strain profiles in Chapter 5. The difference between the calculated and the experimental values is relatively large in the case of beams PCP-2 and PCP3-1, but significantly less in the case of beams PCP3-2 and PCP4-2. Since the so-called experimental values are not directly measured, but deduced from other measurements, the experimental values may be subject to some error and may partially explain the observed difference. In addition, the assumption of a 9 MPa tensile

strength for the Sika grout may not be necessarily true because the reinforcing prisms were cast from different grout batches and had different ages at the time of the test. Nevertheless, the relatively close comparison for beams PCP3-2 and PCP4-2 indicate that overall the proposed method is reasonable.

6.3.2 Crack Width

Cracking due to external load in concrete members occurs when at the extreme tension fibres the tensile strength of concrete is reached. During the life time of the structure, crack width has to be limited to a given value, which depends on the environment where the concrete structure is used, its loading, the type of reinforcement, and the function of the structure.

In the past decade, it has been suggested that steel reinforcement should be replaced by FRP, due to the susceptibility of steel reinforcement to corrosion. Since most of the requirements on crack width limitations are based on corrosion prevention and/or control, the existing code equations and recommendations are based on the post-cracking behaviour of steel reinforced structures. Since FRP's do not corrode, it is clear that FRP reinforced structures will require new recommendations with regard to crack width. Firstly, the large covers suggested for corrosion control of steel reinforced sections may be reduced. Secondly, the low modulus of elasticity of FRPs have to be considered to ensure that cracks do not open widely, in the case of low reinforcement ratios for some FRP reinforced concrete members. To assure satisfactory behaviour of such structures, also aspects other than corrosion should

be considered, such as fire resistance, or susceptibility of FRPs to ultra violet light. There is a clear need for revising the current codes in order to be able to deal with this unique material. However, the focus of the present study is analysis and prediction of the flexural behaviour of the test beams under load.

Existing Crack Width Calculation Methods

Since most researchers have attempted to use the existing methods, an overview of some code approaches is presented below.

Both the CSA (1995) and the ACI (1983) codes recommend the so-called Gergely-Lutz approach which is expressed as

$$w = 11 f_s \frac{h_2}{h_1} (d_c A)^{1/3} \times 10^{-6} \quad (6.38)$$

where w is the crack width at the tensile face of the beam, in mm; f_s is the calculated stress in the reinforcement at specified load, in MPa; h_2 is the distance from the extreme tension fibre to the neutral axis, in mm; h_1 is the distance from the centroid of tension reinforcement to the neutral axis, in mm; d_c is the concrete cover measured from the centroid of tension reinforcement to the extreme tension fibre, in mm; A is the effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of rebars, in mm². However, neither code directly requires crack width calculation, instead, they require that the so-called z parameter be limited to certain values in order to satisfy crack width limitations under

different exposure conditions. The preceding parameter is expressed as

$$z = f_s(d_c A)^{1/3} \quad (6.39)$$

where z is a quantity that limits the distribution and amount of flexural reinforcement, and the quantities d_c , and A are as defined previously. It is suggested by the CSA code that z should not exceed 30 000 N/mm for interior exposure, and 25 000 N/mm for exterior exposure.

The British Standard BS 8110: Part 2 (1994) suggests the following equation to calculate the crack width, w_{cr}

$$w_{cr} = (3\alpha_{cr}\epsilon_m)/(1 + 2(\alpha_{cr} - c_{min})/(h - x)) \quad (6.40)$$

$$\epsilon_m = \epsilon_l - \frac{b_t(h - x)(\alpha' - x)}{3E_s A_s(d - x)} \quad (6.41)$$

where α_{cr} is the distance from the crack width measurement location to the nearest reinforcing bar; ϵ_m is the average strain calculated from Equation 6.41; ϵ_l is the strain corresponding to the level at which the crack width is measured; c_{min} is the minimum cover in the cross-section; h is the depth of the cross-section; x is the neutral axis depth; α' is the distance from the compressive face of the section to the point at which the crack is being calculated. This equation is based on work of Beeby (1971), and on "no-slip" theory, which stipulates that the crack width at the level of reinforcement is zero, and therefore the maximum crack width is at the tension surface of concrete. Figure 6.21 illustrates the notation used in Equations 6.40 and 6.38.

Figure 6.21: Notation for crack width expression

The Japanese Standard uses the following expression

$$w = k [4c + 0.7(C_s - \Phi)] \frac{\sigma_{se}}{E_s} \quad (6.42)$$

where k is equal to 1.0 for deformed bars, 1.3 for plane bars, c is concrete cover, C_s is center-to-center distance of steel bars, Φ is diameter of steel bar, σ_{se} is stress in the bar, and E_s is its modulus of elasticity.

The experimentally determined crack widths in the present test beams were compared with the corresponding calculated values according to the CSA/ACI codes, the Japanese Standard, and the British Standard. Except for the British Standard values, the calculated values compare very well to the experimentally found crack width of beam ST. Figure 6.22 shows the experimentally measured crack width of the crack 1, the widest crack in the beam. The British Standard overestimates the crack width. It is not surprising, since the crack width calculated according to the British Standard method is the crack width at the bottom surface of the beam, while the measured crack width is approximately 50 mm above

the bottom surface.

The cracking pattern in beams reinforced with PCP is different from that of the steel reinforced concrete beams. The prestress in the prisms, before it is overcome by the external load, does not allow the cracks to open widely, and they remain hairline. The existing formula for crack width does not apply to the new beam types. Therefore, a new formula was developed, which is based on the current Gergely-Lutz equation. The original work by Gergely and Lutz (1968) reported serious problems in developing a formula to fit all available data. They, however, pointed out some of the more important variables, such as the effective area of concrete in tension, the number of bars, the reinforcement cover, the strain gradient from the level of the reinforcement to the tensile face, and lastly the stress in the reinforcement. Considering those variables, due to the large curvature of FRP reinforced sections their strain gradient is generally much larger than the strain gradient in steel reinforced sections. This can be easily observed in Figure 5.14 and 5.15 in Section 5.2.3. One of the conclusions and also recommendations of this thesis is that the depth of neutral axis c remains constant after cracking, and consequently c/d also remains constant. The Gergely-Lutz equation includes the term h_2/h_1 , which is directly affected by the depth of neutral axis. When calculated, the ratio h_2/h_1 for steel reinforced beam ranged from 1.7 to 1.5, and for PCP reinforced beams from 2.0 to 1.6, and for Leadline reinforced beam it remained constant at 1.5 after cracking.

Considering the difference in the position of the neutral axis and its movement between conventional steel reinforced concrete and FRP/PCP reinforced concrete and the current

test data, the following modified Gergely-Lutz type equation is proposed for crack width calculation in PCP reinforced beams:

$$w = 9f_s\left(\frac{h_2}{h_1}d_cA\right)^{1/3} \times 10^{-6} \quad (6.43)$$

Note that in Equation 6.43, the originally used factor of 11, which was statistically derived for steel reinforced beams is changed to 9 for PCP reinforced beams. Using the cubic root of the term h_2/h_1 closely represents the behaviour of FRP reinforced concrete beams, and may prove sufficiently robust to be applicable to other beams reinforced with FRP since the crack width depends on the location of the reinforcement with respect to the neutral axis.

Another approach for improving the results of the original Gergely-Lutz formula was introduced by Benmokrane et al. (1996). They changed the originally used factor of 11 to fit their experimental data, however, their formula could not predict the current test results.

Let us now compare in Figures 6.23 to 6.29 the measured crack width in the current test beams to their calculated values based on the proposed Equation 6.43 and on the existing code formulas.

We notice that, with the exception of beam PCP4-2, in all the other cases the proposed method predicts the crack width quite well while the existing code equations do not provide a good estimate of the actual crack width. We wish to emphasize that the CSA/ACI codes do not recommend the use of the current code equations to FRP reinforced concrete structures.

It is recognized that the proposed equation is based on the current set of data, hence its generality cannot be guaranteed. However, it is beyond the scope of the current work to

Figure 6.22: Comparison of measured crack width in beam ST to code prediction

Figure 6.23: Comparison of measured crack width in beam L to code prediction

Figure 6.24: Comparison of measured crack width in beam CP1 to code prediction

Figure 6.25: Comparison of measured crack width in beam CP2 to code prediction

Figure 6.26: Comparison of measured crack width in beam PCP-2 to code prediction

Figure 6.27: Comparison of measured crack width in beam PCP-3 to code prediction

Figure 6.28: Comparison of measured crack width in beam PCP4-1 to code prediction

Figure 6.29: Comparison of measured crack width in beam PCP4-2 to code prediction

Figure 6.30: Cracking pattern of beam ST at ultimate

delve in-depth into the issue of crack width prediction in FRP reinforced concrete structures. It appears that the basic parameters which govern crack width are well represented in the Gergely-Lutz equation, but more comprehensive experimental data and their analysis are needed to ascertain the generality and robustness of the proposed crack width equation.

6.3.3 Crack Spacing

In this section, for comparison purposes only we selected the crack patterns of beams ST, L, and PCP3-1 to highlight the differences or similarities among them. Figures 6.30, 6.31, and 6.32 represent the crack pattern of the latter beams. From the cracking pattern of beam ST it is clear that most of the cracks were flexural. In beams L and PCP3-1 a flexure-shear crack pattern developed close to ultimate load. This was due to the fact that these beams had higher failure load than the steel reinforced beam.

The average crack spacing at ultimate in the constant moment region was 80 mm for beam ST, 123 mm for beam L, and 90 mm for beam PCP3-1. The minimum crack spacing can

Figure 6.31: Cracking pattern of beam L at ultimate

Figure 6.32: Cracking pattern of beam PCP3-1 at ultimate

be found from the following equation

$$a_{min} = \frac{A_e f_t}{u \sum o} \quad (6.44)$$

where A_e is the effective area of concrete in tension, f_t is the tensile strength of concrete, u is the average bond stress, and $\sum o$ is the sum of the perimeter of the bars. If the distance away from the crack is less than the value calculated in Equation 6.44, a crack cannot develop. If the distance is more than a_{min} a crack will develop. The maximum crack spacing is given by

$$a_{max} = 2a_{min} = \frac{A_e f_t d_b}{u A_s} \quad (6.45)$$

where d_b is the bar diameter, A_s is the area of steel bar. Note that Equation 6.45 is based on $\sum o = 4A_s/d_b$. Equation 6.45 can also be used to find the bond stress of the rebars. Using the experimentally observed maximum crack spacing, the bond stress for steel and Leadline reinforced beams was found to be equal to 27 MPa, and 30 MPa, respectively. This is in agreement with findings of other investigators that the bond of FRP rebars is as good as or better than the bond of steel rebars.

Since crack width is dependent on the maximum spacing of cracks, and is equal to the elongation of the reinforcement between two cracks, the maximum crack width can be approximated by

$$w_{max} = a_{max} \frac{f_s}{E_s} \quad (6.46)$$

However, the expression for calculating the bond strength is valid only at ultimate, and therefore, Equation 6.46 is valid only at ultimate. If the experimentally observed crack spacing is used to calculate the crack width at or close to ultimate, the crack width for the

steel reinforced beam would be 1.25 mm, and for the Leadline reinforced beam it would be 3.2 mm.

6.4 Ultimate Strength of Test Beams

The ultimate limit state of a structural member is the state at which such a member reaches its load carrying capacity and fails. An important role of any design is to ensure that structures/structural members have adequate ultimate capacity. Thus development of accurate methods for calculating the ultimate capacity of structures is necessary.

Since premature failure has serious consequences, it is important to be able to predict the ultimate capacity with sufficient level of accuracy. A detailed procedure for the calculation of the ultimate moment capacity of the test beams is presented here. This section includes a sample calculation of the ultimate capacity of test beam L reinforced with two No.8 Leadline bars. The same procedure was used to find the ultimate capacity of all the test beams and the results will be compared to the corresponding experimental results.

The following are the assumptions of the analysis:

- maximum concrete strain in compression 0.005
- Hognestad's parabola with linear descending branch to approximate stress-strain curve for concrete (Figure 6.33)

It was shown in the previous works, that high strength concretes may reach higher compres-

sive strains than the code recommended value of 0.0035 at ultimate. As it was shown by the experimental part of this work, the maximum compressive strain in concrete was more than 0.004. These values were obtained from measurements of concrete strain in each beam. The strain was measured 15 and 30 mm below the top surface, and the maximum top strain was interpolated from Figures 5.14 and 5.15. For most beams it was found to be more than 0.004, and therefore a value of 0.005 was used for the ultimate strength calculations.

The compression zone of FRP reinforced concrete is very small at the time of failure. In under-reinforced steel reinforced concrete sections at the time of concrete crushing, the steel reinforcement has already yielded, therefore the force in the reinforcement is constant (if no strain hardening is present). In the FRP reinforced sections, due to the linear elastic behaviour of FRP, the force in the reinforcement progressively increases until failure. Hence, the stress in the concrete has to correspondingly increase until equilibrium in the cross section is reached. Therefore, the assumption of the maximum compressive strain of 0.003, as stipulated in the ACI code, or 0.0035, as stipulated in the CSA code does not apply to FRP over-reinforced concrete. Since failure of an FRP reinforced section should always be governed by crushing of concrete rather than rupture of FRP, for obvious reasons, the resultant force heavily depends on the assumption of the ultimate compressive strain.

The assumption of an equivalent concrete stress block, or Whitney stress block, has already been referred to by the Swiss Code (SIA 1989) as inadequate for CFRP reinforced concrete. The rectangular stress distribution over the depth " a " or " $\beta_1 c$ " appears to underestimate the ultimate capacity of the CFRP reinforced concrete beam. A similar problem was reported by

Mostofinejad (1997), where the ultimate capacity of CFRP reinforced concrete beams with small reinforcement ratio was not predictable. To avoid using the code parameters α_1 and β_1 , that are derived for an under-reinforced concrete section, and hence are not applicable to FRP reinforced concrete, we used the Hognestad's parabola with linear descending branch to calculate the actual force in concrete, and its location with respect to the neutral axis. The approach of Swiss Code (SIA 1989) was used to set up the equilibrium equations, and find the depth of neutral axis c . However, since the Swiss Code assumes the maximum compressive strain in concrete only to be 0.003, we implemented the actual values obtained from integration of the Hognestad's parabola. Analytical results using these assumptions are shown in the following.

Consider beam L with the following parameters: area of FRP $A_{FRP} = 98 \text{ mm}^2$; effective depth $d = 183 \text{ mm}$; modulus of elasticity of FRP $E_{FRP} = 147 \times 10^3 \text{ MPa}$, and the compressive strength of concrete $f'_c = 45 \text{ MPa}$.

To maintain equilibrium in the cross-section, the sum of the compressive forces must equal to the sum of the tensile forces

$$C_c = T_{FRP} \quad (6.47)$$

where C_c is the compressive force in concrete, and T_{FRP} is the tensile force in FRP.

$$C_c = 0.792 f'_c b c; \quad T_{FRP} = A_{FRP} f_{FRP} \quad (6.48)$$

where f'_c is the compressive strength of concrete, b is the width of the beam cross-section, c is the depth of neutral axis, A_{FRP} is the area of FRP reinforcement, f_{FRP} is the stress in the FRP reinforcement. If beams reinforced with PCP were used, the force T would also

include $f_{p,eff}$, representing the effective prestress.

$$f_{FRP} = \epsilon_{FRP} E_{FRP} \quad (6.49)$$

where ϵ_{FRP} is the strain in the FRP reinforcement, and E_{FRP} is the modulus of elasticity of FRP reinforcement. Using strain compatibility

$$\epsilon_{FRP} = \frac{0.005}{c} (d - c) \quad (6.50)$$

where d is the effective depth. Substituting Equation 6.50, 6.49, and 6.48 into Equation 6.47,

$$0.792 f'_c b c = 0.005 A_{FRP} E_{FRP} \left(\frac{d - c}{c} \right) \quad (6.51)$$

In the above equation the only unknown is the depth of neutral axis, c , which can be found by solving a second degree equation

$$c^2 + \frac{0.005 A_{FRP} E_{FRP}}{0.792 f'_c b} c - \frac{0.005 A_{FRP} E_{FRP} d}{0.792 f'_c b} = 0 \quad (6.52)$$

From Equation 6.52 the depth of the neutral axis was found to be 41.6 mm. Consequently, using the previous set of equations, the following strains and stresses were found: the strain in FRP, $\epsilon_{FRP} = 0.017$, and the stress in the FRP, $f_{FRP} = 2498.3$ MPa.

The moment of resistance for the cross section can be found using the following equation

$$M_r = A_{FRP} f_{FRP} (d - 0.462c) \quad (6.53)$$

The ultimate moment for beam L was calculated to be 40.1 kN.m, or the ultimate load of 94 kN, which is 94% of the experimentally achieved ultimate load of 100 kN. Note that even though all the test beams had compression reinforcement, it was not included in the

present calculations. The reinforcement is situated at a distance not greater than $0.1c$ from the neutral axis, which despite its substantial area of 200 mm^2 would increase the moment capacity by 0.05%.

The latter approach was used to find the ultimate capacity of all test beams. The results are shown in Table 6.4. Good agreement was found between the experimental and calculated values of the ultimate load carrying capacity of test beams. Note that for the beams PCP1-2 and PCP-2 the failure load cannot be predicted, because the former failed in bond, and the latter in shear. Other calculations show an error of maximum of 10%. This error may have been caused by the internal lever arm to be higher than expected, or the compressive strain in concrete may have been higher than the 0.005. The overall prediction of flexural behaviour of PCP reinforced beams with a small reinforcement ratio is very good. The analysis also shows that the assumption of Whitney stress block at ultimate is conservative. The value of maximum compressive strain in concrete can reach higher values than 0.0035, as stipulated in the CSA code (1995).

Figure 6.33: Hognestad's parabola

Table 6.4: Ultimate Strength of Test Beams

Beam	Flex. reinf.	d [mm]	P_{ult}^{exp} [kN]	$P_{ult}^{cal.}$ [kN]	$P_{ult}^{cal.}/P_{ult}^{exp}$
L	2#8	183	100.0	94.0	0.94
CP1	2 CP1	189	94.8	99.3	1.05
CP2	2 CP2	179	91.4	91.0	0.99
PCP1-1	PCP1+#8	189	105.7	105.1	0.99
PCP1-2	PCP1+#8	189	66.7	105.1	N.A. ^a
PCP2	2 PCP2	189	100.5	103.7	1.03 ^b
PCP3-1	2 PCP3	179	115.0	104.0	0.90
PCP3-2	2 PCP3	184	115.0	104.4	0.91
PCP4-1	2 PCP4	184	110.0	101.8	0.93
PCP4-2	2 PCP4	189	114.0	106.2	0.93

^aThis beam failed in bond^bThis beam failed in shear

Chapter 7

Behaviour of Test Beams in Shear

7.1 General

Chapter 6 dealt with the serviceability and ultimate limit states of the test beams. In the case of beams loaded in four point bending, within the shear span they are subjected to combined flexure and shear. Although the study of detailed shear behaviour of PCP reinforced beams is beyond the scope of the present study, nevertheless, an attempt is made in this chapter to investigate the shear behaviour of the test beams.

Unlike their flexural design, shear design of reinforced concrete beams is to a large extent based on the results of extensive experimental work (Pillai and Kirk 1988). The same can be stated about the shear behaviour of FRP reinforced beams. In the case of FRP reinforced concrete structures, use of FRP stirrups has not been common due to the fact that FRP reinforcement cannot be bent into the form of stirrups in the field, rather the stirrups must be manufactured in the plant. To date, the majority of FRP reinforced beams tested in the laboratory and constructed in the field have had steel stirrups and FRP longitudinal

reinforcement, similar to the beams tested in the current study; therefore, only the methods of shear design suitable for such beams will be presented.

Beams reinforced with combination of the two reinforcement types cannot be accurately analyzed as either fully steel reinforced, or fully FRP reinforced beams. This chapter will discuss some existing approaches to shear design of conventional reinforced concrete and the latest methods for shear design of beams with combined reinforcement. The shear capacity of the test beams based on the existing methods is calculated and the contribution of the stirrups and the concrete are separately determined. The principal stresses and strains are plotted and the failure envelopes are presented. The effect of prestress in the prism on diagonal cracking and principal stresses is also shown.

7.2 Methods of Shear Design

7.2.1 Simplified Method of CSA

The simplified method of shear design in the CSA A23.3-94 (CSA 1995) is based on the 45° truss model. The original truss model was very conservative because it neglected tensile stresses in concrete (Collins and Mitchell 1987). The ACI code (ACI 1983) introduced the idea of both the concrete and the stirrups contributing to the total shear capacity of a beam more than 20 years ago. According to the CSA code (CSA 1995), the total shear capacity of a section, V_r , consists of the combined contribution of concrete and stirrups.

$$V_r = V_c + V_s \quad (7.1)$$

The concrete contribution, V_c , is

$$V_c = 0.2\lambda\phi_c\sqrt{f'_c}b_wd \quad (7.2)$$

where λ is a modification factor to account for the type of concrete, ϕ_c is 0.6, f'_c is the concrete ultimate strength, b_w is the width and d the effective depth of the cross section.

The stirrup contribution, V_s , is given by

$$V_s = \frac{\phi_s A_v f_y d}{s} \quad (7.3)$$

where ϕ_s is 0.85, A_v is the total area of the stirrup, f_y is the yield stress of the stirrup, and s is stirrup spacing. To avoid diagonal crushing, the value of V_s is limited to a maximum of $0.8\lambda\phi_c\sqrt{f'_c}b_wd$.

This method overestimates the capacity of test beams with combined reinforcement (Mostofinejad and Razaqpur 1997, Alsayed et al. 1997, Choi et al. 1997). However, most of the new methods were developed on the basis of this method as discussed in the next section.

7.2.2 Modifications of CSA/ACI Approach to Shear Design

Development of design methods for beams reinforced with FRP stirrups is still being researched. The different types of available FRP materials, different combinations of FRP and steel for both flexure and shear makes it very difficult to develop a single approach which accounts for all reinforcing possibilities. In the following, the latest work in the area of shear behaviour of beams reinforced with combination of steel and FRP is discussed.

Alsayed et al. (1997) reported a set of 21 tests of beams designed to fail in shear. The beams were reinforced with GFRP, or a combination of GFRP and steel. The authors reported a 50% reduction in concrete contribution to shear resistance, when GFRP was used for longitudinal reinforcement, and similarly a 50% reduction in the stirrup contribution when GFRP stirrups were used. Their design approach is based on the ACI method as follows.

The nominal capacity of the beam is based on Equation 7.1, and the terms V_c and V_s depend on the type of longitudinal and/or shear reinforcement used. The full magnitude of V_c is

$$V_c = \frac{1}{6} \sqrt{f'_c} b d \quad (7.4)$$

and the full magnitude of V_s is

$$V_s = \frac{A_v f_y d}{s} \quad (7.5)$$

The terms were explained in Equations 7.2 and 7.3. When the longitudinal reinforcement is FRP, V_c is reduced by half. Similarly a 50% reduction in V_s is introduced when FRP stirrups are used. For example for beams of the type used in this thesis, where the longitudinal reinforcement is FRP, and the stirrups are steel, V_n , the nominal shear capacity, would be calculated as

$$V_n = 1/2 V_c + V_s \quad (7.6)$$

for a beam reinforced with FRP for both flexure and shear, the nominal capacity would be

$$V_n = 1/2 V_c + 1/2 V_s \quad (7.7)$$

Alsayed et al. (1997) reported good agreement with their experimental results. The shear capacity of the test beams in the present investigation was calculated using Equation 7.6 and the results are summarized in Tables 7.1 and 7.2.

The results are on the conservative side compared to the CSA prediction. The calculated shear capacities are also much higher than the flexural capacity of the same beams and therefore no definitive conclusion can be drawn as to the appropriateness of the suggested method.

Choi et al. (1997) used the lattice model to verify the applicability of the modified truss analogy to FRP reinforced beams. They also suggested using Equation 7.1. The concrete contribution in this case was made a function of the reinforcement ratio and the shear span of the beam as follows

$$V_c = 0.2f_c'^{\frac{1}{3}}\rho_w^{\frac{1}{3}}d^{\frac{1}{3}}[0.75 + 1.4d/a]b_wd \quad (7.8)$$

where ρ_w is the longitudinal reinforcement ratio in percent, d is the effective depth in metres, a/d is the shear span to effective depth ratio, and b_w is the width of the section. The stirrup contribution, contrary to any other method, depends on the depth of neutral axis, or in other words on the internal lever arm, z in the following fashion

$$V_s = A_v f_y z / s \quad (7.9)$$

where z is the internal lever arm, and s is stirrup spacing. It was concluded that for beams reinforced with FRP, the contribution of concrete to the shear carrying capacity decreases with increase in stirrup stress. This method introduced some new elements into shear design, such as the reinforcement ratio, and the shear span to depth ratio of the beam, which are important when considering the beam behaviour in shear. Of course, the so-called detailed method of the ACI code (ACI 1983) for conventional reinforced concrete similarly includes the reinforcement and shear span/depth ratios.

Mostofinejad and Razaqpur (1997) also based their shear design formula on the ACI/CSA approach. They used the ACI expression for V_c , i.e.

$$V_c = 0.166\sqrt{f'_c}b_wd \quad (7.10)$$

The contribution of stirrups was modified in the following manner

$$V_s = \frac{A_v f_v d}{s} \quad (7.11)$$

where

$$f_v = \frac{f_u}{\alpha} \phi \quad (7.12)$$

and

$$\alpha = \frac{1}{1 - (f_u - 400)/2000} \quad (7.13)$$

where ϕ is the diameter of the bar. Equation 7.13 was obtained by curve-fitting some available experimental data from other investigators. This method was tested against 22 beams tested by others, and was proven to give good results.

Razaqpur (1998) analyzed many beams, tested by other researchers, in order to quantify the concrete contribution to the shear resistance. Part of this work was submitted for discussion to the Canadian Code for FRP Reinforced and Prestressed Concrete, currently under development. The following formula for the concrete contribution gives promising results when compared to the experimental results.

$$V_c = 0.035(f'_c \rho_w E_{FRP} d/a)^{1/3} b_w d \quad (7.14)$$

where f'_c is the compressive stress of concrete, ρ_w is the longitudinal reinforcement ratio, E_{FRP} is the modulus of elasticity of FRP, d is the internal lever arm, a is the shear span,

and b_w is the width of the cross section. The contribution of stirrups remains the same as in the previous method.

7.2.3 Calculated Shear Capacity of Test Beams

The nominal shear capacity of the test beams was calculated using the above described four methods and the results are given in Table 7.1. The following notation is used in the table to designate the ultimate shear capacity by the four methods: for the CSA method, V^{CSA} , for the method proposed by Alsayed, V^{Al} , for the method proposed by Choi, V^{Ch} , for the method proposed by Mostofinejad and Razaqpur, V^{Mo} , and for the method by Razaqpur V^{Ra} .

Table 7.1 shows the design capacity of concrete and the stirrups separately. The total shear resistance V_r is given in Table 7.2. It can be seen from the results for beam PCP-2 that all the above mentioned methods predicted shear capacity three to four times higher than the actual failure load of each beam. This means that none of the beams is expected to fail in shear, but one cannot draw any conclusions from these results regarding the accuracy of the methods. One can, however, see the sometime large differences that exist among the predicted capacities by these methods. Choi et al. 's method gives the most conservative results, followed by Alsayed et al. 's method. The CSA and Mostofinejad and Razaqpur's methods give similar results. However, although beam PCP-2 appeared to fail in shear, the failure may have been instigated by other causes. It appears that for PCP reinforced beams the above mentioned methods overestimate the stirrup contribution.

Table 7.1: Theoretical Contribution of Concrete and the Stirrups to the Shear Resistance of the Test Beams

Beam	d	V_s^{CSA}	V_s^{Al}	V_s^{Ch}	V_s^{Mo}	V_c^{CSA}	V_c^{Al}	V_c^{Ch}	V_c^{Mo}	V_c^{Ra}
	[mm]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]
L	183	164.7	164.7	144.1	160.5	40.5	16.9	12.3	33.6	17.6
CP1	189	170.1	170.1	148.8	165.8	41.8	17.4	12.8	34.7	18.2
CP2	179	161.1	161.1	141.0	157.1	39.6	16.5	11.9	32.9	17.2
PCP1-1	189	170.1	170.1	148.8	165.8	41.8	17.4	12.8	34.7	18.2
PCP1-2	189	170.1	170.1	148.8	165.8	41.8	17.4	12.8	34.7	18.2
PCP2	184	165.6	165.6	144.9	161.5	40.7	17.0	12.4	33.8	18.2
PCP3-1	179	161.1	161.1	141.0	157.1	39.6	16.5	11.9	32.9	17.2
PCP3-2	184	165.6	165.6	144.9	161.5	40.7	17.0	12.4	33.8	17.7
PCP4-1	184	165.6	165.6	144.9	161.5	40.7	17.0	12.4	33.8	17.7
PCP4-2	189	170.1	170.1	148.8	165.8	41.8	17.4	12.8	34.7	18.2
ST	183	164.7	164.7	144.1	160.5	40.5	33.8	15.6	33.6	40.5

However, more experimental work is needed to develop a solid analytical model for PCP reinforced concrete beams. The next section will discuss the observed stirrup capacity, and the stresses in concrete.

7.3 Experimental Shear Capacity of Test Beams

All test beams were instrumented using strain gauges to measure strain in the shear reinforcement and in the concrete. As is well-known, a beam derives its shear resistance from various shear resistance mechanisms, including shear carried by the stirrups, the uncracked concrete, the dowel action, and the aggregate interlock. The effect of dowel action is usually

Table 7.2: Theoretical Shear Capacity of the Test Beams

Beam	V_r^{CSA} [kN]	V_r^{Al} [kN]	V_r^{Ch} [kN]	V_r^{Mo} [kN]	V_f^{exp} [kN]
L	205.2	181.6	156.4	194.1	N/A
CP1	211.9	187.5	161.6	203.2	N/A
CP2	200.7	177.6	152.9	190.0	N/A
PCP1-1	211.9	187.5	161.6	203.2	N/A
PCP1-2	211.9	187.5	161.6	203.2	N/A
PCP2	206.3	182.6	157.3	195.3	50.3
PCP3-1	200.7	177.6	152.9	190.0	N/A
PCP3-2	206.3	182.6	157.3	195.3	N/A
PCP4-1	206.3	182.6	157.3	195.3	N/A
PCP4-2	211.9	187.5	161.6	203.2	N/A
ST	205.2	198.5	159.7	194.1	N/A

omitted in FRP reinforced concrete because of the relatively low modulus of elasticity of FRP in the transverse direction.

7.3.1 Contribution of Stirrups to Shear Capacity of the Test Beams

In order to assess the contribution of V_s to the total shear resistance of the test beams, in Figures 7.1 to 7.5, V_s/V_{max} (designated as V_{st} and V_f) are plotted against the applied shear acting on each beam. The quantity V_{max} is the maximum shear force applied to each beam and is equal to half the failure load of the same beam. The V_s was calculated using the measured strain on selected instrumented stirrups. The particular stirrups and their strain variation with applied shear are also shown in Figures 7.1 to 7.5. We notice in these figures

that none of the stirrups actually yielded, therefore, the forces in them were calculated using Hooke's law. Specifically

$$V_s = \frac{A_v f_v d}{s} \quad (7.15)$$

where $f_v = E_s \epsilon_{sv}$. The value ϵ_{sv} was in most cases determined by taking the average of the strain gauge readings from the two legs of the same stirrup. However, in some cases, when the difference between the two strain gauge readings was found to be very high, only the maximum strain was used. The rationale for doing this is that if a strain gauge is located near or at a shear crack, then it records significantly higher strain than if it is situated far from the crack. The maximum strain in the stirrup is what we are interested in for design purposes.

With the exception of the beam L in Figure 7.1, the results for the remaining beams appear to be reasonable, i.e. the applied shear is partly resisted by the stirrups while the remaining portion is the so-called concrete contribution, V_c . Strictly speaking, from the design point of view, only the value of V_s at failure is relevant, but to understand the full behaviour of these beams, we have shown the variation of V_s over the entire loading range.

First, the experimental data indicate that shear cracking load is not significantly affected by the level of prestressing in the prisms. In the steel reinforced beam ST, shear cracking occurred at almost 45% of the failure load, while in beam L, it occurred at 30% of the failure load. On the other hand, in beam PCP-1 the cracking appears to have occurred much earlier at 20% of the maximum load. By contrast, beam PCP3-1 did not experience shear cracking up to almost 40% of the maximum load. It should be pointed out that initiation of

shear cracking is marked by a sudden noticeable increase in the stirrup strain.

In general, however, shear cracking appears to have occurred at $V = 10$ kN (see the stirrups strain diagrams), following which there is a difference in the shear resistance of the various beams. We notice that in beams L and CP1 the stirrup strain increased quite dramatically after shear cracking while in beams with prestressed prisms the same amount of increase did not occur. In particular, beams PCP3-1 and PCP3-2 show a more gradual and linear increase of strain with increased load. In the latter beams, the large increase in strain does not occur until at least 60% of the failure load. Hence, the amount of prestressing in the prisms has a favourable effect on the shear behaviour of the beams. It would appear that the higher the prestress, the better the response. The favourable response may be ascribed to the superior ability of prestressed prisms to transfer shear by dowel action.

The strain values for the stirrup in beam PCP-2 beyond a shear force of 30 kN seem unusual because they tend to decrease at first and subsequently increase at a higher load. It is likely that the strain gauge actually failed at $V = 30$ kN.

Based on the results in Figures 7.1 to 7.5, we calculate the contribution of the concrete to the total shear resistance at failure as shown in Table 7.3. Based on the results in this table, it can be stated that in beams reinforced with prestressed prisms, at failure the concrete contribution to the shear resistance of the beams is either very small, or negligible. On the contrary in the beam reinforced with steel in the present test program, the so-called concrete shear is almost 60% of the total shear resistance.

If we compare the experimental results with the predictions of the various methods discussed

Table 7.3: Contribution of Concrete and Steel Stirrups to the Shear Resistance of the Test Beams at Failure

Beam	V [kN]	V_c [kN]	V_s [kN]
L	50	0	50
CP1	46	19	27
CP2	45	25	20
PCP1-1	53	23	30
PCP1-2	32	2	30
PCP2	50	11	39
PCP3-1	53	13	40
PCP3-2	57	0	68
PCP4-1	55	5	50
PCP4-2	55	12	43
ST	40	22	18

earlier, Choi et al.'s method and the method by Razaqpur, both seem to provide a good estimate of the contribution of the concrete to the total shear resistance of the beams. As indicated in Table 7.1, according to these methods, the concrete contribution is about 9% at ultimate. If we assume this percentage contribution to remain constant in the interval between shear cracking and yielding of the stirrups, then it is reasonable for the current beams.

In practice, if further conservatism is deemed prudent, then one could completely ignore the contribution of concrete to the total shear resistance. It should be pointed out that unlike conventional reinforced concrete, a beam over-reinforced for shear with FRP stirrups, is actually better, from the mode of failure point of view, than a similar under-reinforced beam.

Another important point to realize is that shear analysis is performed under the assumption that the stresses in all the shear reinforcement that crosses a potential shear failure plane eventually yield and therefore the actual location of stirrup within that plane is not important. This assumption is valid for beams with steel stirrups and steel flexural reinforcement. In the case of beams with FRP reinforcement, its veracity requires further investigation. While beams with steel stirrups and FRP longitudinal reinforcement may behave similar to conventional reinforced concrete insofar as yielding of stirrups is concerned, beams with FRP shear reinforcement will not behave the same. It can be shown theoretically, as well as by experimental observations, that the stress in the FRP stirrup depends on its location within the shear plane. For beams in four point bending, Zhao et al. (1995) have determined from tests on beams that the stress in the stirrup is a function of $\left(\frac{L_i}{L_o}\right)^{1/3}$, where L_i is the

Figure 7.1: Shear force in test beams ST, L, and CP-1

Figure 7.2: Shear force in test beams CP-2, PCP1-1, and PCP1-2

Figure 7.3: Shear force in test beams PCP-2, PCP3-1, and PCP3-2

Figure 7.4: Shear force fluctuation among stirrups

distance of the i -th stirrup from loading plane and $L_o = d/\tan\theta$, with d being the effective length and θ being the angle of the shear failure plane. Consequently, the use of a uniform average stress in all the stirrups crossing a shear plane needs more careful consideration.

Beams reinforced with highly prestressed PCPs, such as PCP3 and PCP4 experienced similar behaviour to steel reinforced beams. From Figure 7.3 and 7.5 it can be seen that, prior to cracking of the prisms, the shear was carried by the concrete alone. The hairline cracks did not significantly affect the load-carrying capacity. Shortly after cracking of the prisms, the inclined cracks developed, hence, the stirrups started to resist shear. For the beams PCP3-1, 3-2, 4-1, and 4-2 it was at 35 kN, 40 kN, 35 kN, and 35 kN respectively. This is more than 2.5 times increase compared to the beam L, or CP1 and CP2, and twice the increase compared to the beam ST. These beams also showed higher ultimate capacity. Since at that point the prisms were already cracked, most of the shear force was resisted

Figure 7.5: Shear force in test beams PCP4-1, and PCP4-2

by stirrups. Compared to steel reinforced beam ST, the total load of PCP reinforced beams was 40 % higher, which caused higher stresses in both concrete and steel. However, at comparable load levels, for example 40 % of ultimate load, the beams PCP3-1, 3-2, 4-1, and 4-2 behaved similarly to the beam ST and showed improved response compared to beam L.

7.3.2 Evaluation of Concrete Strains in Shear Span

In this section we examine the strain in the concrete struts. It is important to recall that the CSA A23.3-94 limits compressive strength of the concrete in the diagonal compression struts, depending on the average strain in the stirrups.

One rectangular strain-gauge rosette was attached at the mid-height of the beam in the middle of each shear span. The strains were measured parallel and perpendicular to the beam axis, and at an angle of 45° counterclockwise from the longitudinal axis. The strain measured parallel to the beam axis is $\epsilon_{0^\circ} = \epsilon_x$, the perpendicular strain is $\epsilon_{90^\circ} = \epsilon_y$, and the shear strain is

$$\gamma_{xy} = 2\epsilon_{45^\circ} - (\epsilon_{0^\circ} + \epsilon_{90^\circ}) \quad (7.16)$$

The strains can be precisely measured if the concrete is not cracked. When a crack passes through a strain gauge, the measurement is often terminated. Therefore, for some beams data is available for only a part of the loading range. If the concrete is not cracked in the measuring location, and given that the rosette is at mid-height of the beam, where ϵ_x is relatively small, we will use elastic analysis. Clearly, this analysis would not apply if the concrete were cracked. The main purpose of the analysis is to compare the behaviour of different beam types with that of the beam ST.

A program in Matlab was written to carry out the analysis. It was assumed that the concrete in the shear span is under plane stress state. From the measured strains ϵ_x , ϵ_y , and the

calculated strain γ_{xy} , the principal strains were found using Mohr's circle

$$\epsilon_{1,2} = \frac{\epsilon_x + \epsilon_y}{2} \pm \sqrt{\left(\frac{\epsilon_x - \epsilon_y}{2}\right)^2 + \left(\frac{\gamma_{xy}}{2}\right)^2} \quad (7.17)$$

The strains are plotted in Figure 7.6 to 7.18 to give an indication of the extent of stress that the concrete may be subjected to in the minor principal direction. We observe that the principal compressive strain rarely exceeds 2.5×10^{-4} , which is only 13% of the strain corresponding to the peak concrete stress. Hence the recorded strains appear surprisingly small and it means that the diagonal compressive stresses would also be relatively small.

The principal tensile strains are generally between 0.5 and 1.0×10^{-4} . For a 45 MPa concrete, the cracking strain would be approximately 1.2×10^{-4} . Hence, principal tensile strains are generally close to cracking.

Note, however, that in the beam ST, the principal tensile strain is significantly larger. It is likely that the strain rosette in this case traversed a crack because the measured strains, particularly ϵ_x (see Figure 7.6), are quite large. A similar trend is noticeable in the case of beam PCP3-1 in Figure 7.16, where the maximum value of the principal tensile strain is 1.3×10^{-3} , which is roughly ten times the same strain in the other PCP reinforced beams.

The main conclusions that can be drawn from the current strain data is that concrete stresses in the diagonal compression struts are generally small and that the current beams do not exhibit an unusual shear behaviour. It would also appear that the level of prestress has an effect on the stress in the stirrup. Higher prestress tends to increase the concrete contribution to the shear resistance and that shear cracking does not lead to a sudden increase in the stirrup stress. On the other hand, smaller levels of prestress, i.e. less than 300 MPa, do

not contribute to an improved shear behaviour and do not have any noticeable influence on stirrup stress.

To assess the beneficial effect of prestress level on shear behaviour, in Figure 7.19 we plot the inclined cracking shear, V_{cr} , against the prestress level. The experimental data can be represented by a bilinear relation as indicated by the solid line in the figure. Notice that beyond 300 MPa, the shear cracking load is directly proportional to the amount of prestress.

It has to be mentioned that the shear cracking loads shown in Figure 7.19 are significantly smaller than the value obtained from CSA A23.2-94 equation for V_{cw} . Assuming zero prestress, V_{cw} is given by

$$V_{cw} = 0.4\lambda\phi_c\sqrt{f'_c}b_wd \quad (7.18)$$

Let $\lambda = 1.0$ and $\phi_c = 1.0$ (note that ϕ_c is a material safety factor), then for the current beams

$$V_{cw} = 0.4 \times \sqrt{45} \times 165 \times 180 = 79.7 \times 10^3 \text{ N} \quad (7.19)$$

where we have used as average values $f'_c = 45$ MPa, $b_w = 165$ mm, and $d = 180$ mm. Hence, the value of 80 kN for shear cracking load is significantly higher than the observed cracking load of any of the beams. Accordingly, the application of Equation 7.19 to PCP reinforced beams would need further enquiry.

Generally the current results demonstrate that the issue of shear in PCP reinforced beams requires further systematic investigation to clearly identify the dominant shear resistance mechanisms and to assess the contribution of each mechanism to the total resistance.

Figure 7.6: Strains, and principal strains for beam ST, 600 mm away from the right support.

Figure 7.7: Strains, and principal strains for beam Leadline, 600 mm away from the left support.

Figure 7.8: Strains, and principal strains for beam Leadline, 600 mm away from the right support.

Figure 7.9: Strains, and principal strains for beam CP1, 600 mm away from the right support.

Figure 7.10: Strains, and principal strains for beam CP2, 600 mm away from the left support.

Figure 7.11: Strains, and principal strains for beam CP2, 600 mm away from the right support.

Figure 7.12: Strains, and principal strains for beam PCP1-1, 600 mm away from the right support.

Figure 7.13: Strains, and principal strains for beam PCP1-2, 600 mm away from the left support.

Figure 7.14: Strains, and principal strains for beam PCP2, 600 mm away from the left support.

Figure 7.15: Strains, and principal strains for beam PCP2, 600 mm away from the right support.

Figure 7.16: Strains, and principal strains for beam PCP3-1, 600 mm away from the left support.

Figure 7.17: Strains, and principal strains for beam PCP3-2, 600 mm away from the right support.

Figure 7.18: Strains, and principal strains for beam PCP4-1, 600 mm away from the left support.

Figure 7.19: The effect of prestress in prism on inclined cracking load

Chapter 8

Summary, Conclusions, and Recommendations

8.1 Summary

This study is concerned with the serviceability and strength of beams reinforced with prestressed concrete prisms (PCP). Eleven rectangular beams were tested in four point bending to investigate their deflection, crack width, flexural response, and flexural and shear strength. The main variable was the type and properties of the main flexural reinforcement. Two of the eleven beams served as control specimens. One of these two was reinforced with conventional steel rebars, while the other was reinforced with two 8 mm carbon fibre reinforced plastic (CFRP) bars. The remaining nine beams were reinforced with two prisms each and the principal variable among them was the level of prestress in the prisms.

The prisms were made of high early strength polymer grout or polymer concrete and had cross-sectional dimensions of either 50×50 , or 40×40 mm. Each prism was pretensioned with a single 8 mm CFRP bar, with the initial prestress varying from zero to 40% of the guaranteed tensile strength (2250 MPa) of the CFRP bar.

The beams were tested using a servo-controlled hydraulic jack and an automatic data acquisition system. Extensive instrumentation was applied to measure the strain in concrete and in the reinforcement, the beam deflections, and the crack width at various loading stages up to failure. The test data was used to compare the behaviour of the various beams with those of the control beams, with particular emphasis on service load deflections.

The test results showed that the application of a reasonable amount of prestress to the prisms can limit service load deflections and crack width to acceptable levels, while in the beam reinforced with the unstressed CFRP bars, the same deflections were excessive and clearly unacceptable according to the current code provisions. The test data also revealed that existing methods of analysis of concrete structures need to be modified to render them applicable to PCP reinforced members. Accordingly, new expressions are proposed in this thesis for predicting deflections, crack width and ultimate flexural strength of PCP reinforced beams. The current experimental program and its results support the conclusions that follow.

8.2 Conclusions

Prism Manufacturing

- The prestress of the prisms was limited to a maximum of 40% of the guaranteed tensile strength (2250 MPa) of the CFRP bar. Higher levels of prestress were not possible because the prism cross section was relatively small to withstand the higher prestress,

and the radial expansion of FRP at the same time. Our attempt to prestress the prism to higher stress failed due to splitting of the prisms after prestress release. Hence, the achievable level of prestress in the prism depends on the size of the prisms and on the properties of the grout/concrete. Grout/concrete with low elastic shortening and higher early tensile strength is required, to minimize the prestress losses, and to prevent splitting of the prisms after release of prestress.

- This research supports the findings of other investigators that there is sufficient bond between the concrete prisms and surrounding beam concrete. No bond failure of the twenty prisms embedded in concrete was encountered during testing. The bond between the prism and the beam remained apparently intact until the failure of the beam.
- The "Holandsche Beton Groep" in the Netherlands is currently working on patenting "Expancel" coating for aramid rebars, which is intended to limit radial expansion of these bars due to prestress and temperature effects. Similar material for CFRP rebars would be needed, or else concrete with very high tensile strength has to be used for casting the prisms. If these two conditions are not satisfied, the prisms would need to be manufactured with larger cross section, which would negatively affect their handling and transportation. To avoid large losses due to elastic shortening, the prisms need to be cast from high modulus material.

Flexural Behaviour of Test Beams

- The moment-curvature relation of FRP reinforced beams can be simplified as bi-

linear over the entire loading range. This implies that the c/d ratio is constant after cracking of an FRP reinforced beam. The same behaviour was observed for PCP reinforced beams after PCP cracking. Before prism cracking, the change in c/d for PCP reinforced beams is gradual and as such, it is similar to the c/d change in steel reinforced beam between cracking and yielding.

- The bi-linearity of moment-curvature relationship implies that when three points on the moment curvature are known, the curve becomes fully defined. These three points are moment and curvature at cracking, right after cracking and at ultimate. Using the virtual work method, deflection of FRP reinforced beams can be calculated. A formula for deflection calculation of FRP reinforced beams based on elastic analysis is proposed and verified against experimental results of other researchers. Very good agreement was found for beams reinforced with steel stirrups. Shear deformation of beams with FRP stirrups may be high, and therefore need to be accounted for in the deflection formula. The present formula does not account for the deflection due to shear.
- Using PCP as reinforcement for concrete beams introduces compressive stresses to the tensile zone of concrete beam. Hence, at first cracking only hairline cracks can be observed. This behaviour is significantly different from FRP reinforced concrete, and even from steel reinforced concrete to some extent, hence a tension stiffening model had to be developed. A new tension stiffening model for beams reinforced with PCP is proposed. Incorporating these two models in the proposed deflection calculation

method predicts deflection of PCP reinforced beam with good accuracy.

- The moment-curvature relation of PCP reinforced beams is sufficiently modelled as tri-linear. It is defined by three moments: moment of the concrete beams at cracking, moment of the prism cracking, and the ultimate moment. The cracking moment of the beam reinforced by PCP can be found using elastic analysis. Based on the same principles, prism cracking can be predicted. Using the moment-curvature diagram and the virtual work method, the proposed moment-curvature relation and the tension-stiffening model was used to predict the deflection of PCP reinforced concrete beams. Good agreement with the experimental values was achieved.
- Increased level of prestress in the prism reduces deflections of the PCP reinforced beams. The deflection of beams reinforced with highly prestressed prisms is at service loads comparable to deflection of steel reinforced beam, and is four times smaller than the deflection of a similar FRP reinforced beam. At service load of 40 kN, the deflections of beams ST (steel reinforced), PCP3-1 (PCP reinforced), and L (FRP reinforced) was 0.6, 0.8, and 3.4 mm, respectively. In other words deflection of FRP reinforced beam was 4.25 times larger than that of the PCP reinforced beam at the same load level.
- Flexural cracks of PCP reinforced beams are hairline before prism cracking, and they widen after the prism cracking. The maximum crack width of the beam ST was 0.7 mm, while for the beam with non-prestressed prisms (CP1 and CP2), with FRP reinforced beam (L), it was 2 mm, or nearly three times the value for steel reinforced

beam. This, of course would not satisfy the code requirements for crack control, and would therefore not be acceptable. When the concrete beam was reinforced with the prestressed concrete prisms, the crack width decreased as the prestress in the prism increased. For the beams PCP-2, PCP-3, PCP4-1, and PCP4-2 it was 1.1, 0.9, 1.4, and 0.9 mm, respectively. Hence, clearly the increased level of prestress in the prism positively affected the maximum crack width.

- The Gergely-Lutz formula for crack width calculation was developed for steel reinforced concrete based on extensive experimental work. A similar approach is now taken for the FRP reinforced concrete, and many researchers are suggesting changes to the original Gergely-Lutz formula to fit their experimental results. The view of the writer is that simply changing coefficients will not be robust enough to predict the general behaviour of FRP reinforced beams, whose properties vary not only with different types of fibres used, but also with the amount of reinforcement in the cross section, as well as the internal lever arm. The formula proposed in this work is focusing on all three of the above mentioned aspects. The original multiplier of 11, statistically derived for steel reinforced concrete was changed to 9, and the ratio of h_1/h_2 , which depends on the position of neutral axis, and the internal lever arm was taken under the cube root. These changes worked very well for the tested beams, however, their generality will need to be further tested.
- The average crack spacing at ultimate in the constant moment region was 80 mm for beam ST, 123 mm for beam L, and 90 mm for beam PCP3-1. Clearly, increased

prestress in the prism effected the crack spacing of the beams. From serviceability point of view, based on the results of this experimental work we can conclude that the minimum reasonable prestress of the CFRP would be 600 MPa. If the prestress was lower, the beams cracked at lower load levels and also suffered large deflection.

- The ultimate capacity of CFRP reinforced concrete beams with small reinforcement ratio cannot be well predicted using the existing Whitney rectangular stress block and its associated parameters. Since the ultimate capacity of FRP over-reinforced sections is governed by the capacity of concrete in compression, the resulting force is dependant on the strain in concrete at ultimate. Changing this strain from 0.0035 to 0.005 resulted in a 13% increase of the ultimate capacity of beams reinforced with CFRP. As is well known, in steel under-reinforced concrete sections, the ultimate concrete strain within the range of 0.003 to 0.005 has negligible effect on the ultimate strength.
- Using the existing rectangular block parameters underestimates the ultimate strength of the FRP over-reinforced beams, and is therefore on the safe side.

Shear Behaviour of Test Beams

- Nominal shear capacity of the test beams was found using five different methods, based on the CSA, ACI and other approaches in the literature. All methods predicted shear capacity three to four times higher than the actual failure load of the beam. Large differences existed among the predicted capacities by these methods. Since the

current beams failed in flexure, no conclusions can be drawn on the accuracy of the existing shear design method insofar as their application to PCP reinforced beams is concerned.

- The mechanisms of shear transfer in steel reinforced concrete are well defined. However the same cannot be said about FRP reinforced concrete. It is apparent from the current research that due to the extensive cracking of the FRP reinforced beams, the contribution of the concrete to shear resistance is smaller. The contribution of the steel stirrups used in this research was the governing mechanism of shear transfer close to ultimate. Hence, the use of FRP stirrups needs to be examined in great detail to avoid catastrophic failure.
- The increased level of prestress in the prisms positively affected the inclined cracking shear. Beyond 300 MPa prestress in the prisms, the shear cracking load is directly proportional to the amount of prestress, while prestress less than 300 MPa has negligible effect on shear cracking load.
- Generally the current results demonstrate that shear behaviour of PCP reinforced concrete beams requires further investigation to identify the main shear resisting mechanisms. The current code approaches overestimate the concrete contribution to the shear resisting mechanism. Therefore especially for fully FRP reinforced beams this issue needs to be investigated.

8.3 Recommendations for Future Work

- The manufacturing of PCP is feasible provided concrete mix with high early tensile strength and modulus is used. Low elastic shortening of the material is necessary to reduce prestress losses at transfer. It is essential that new concrete mixes be developed that would allow early release of prestress and prisms with reasonable dimensions that can be easily handled on the construction site.
- Further experimental and analytical investigation is needed to optimize the level of prestress and the size of the prisms for different applications in beams and slabs.
- Thermal incompatibility exists between the CFRP bar and the surrounding prism material. To avoid either loss of bond or splitting of the prism, the effects of temperature and humidity variations on the behaviour of the prisms need to be investigated.
- The effect of creep, shrinkage and relaxation losses on the long term behaviour of the prism need to be studied in a more systematic and detailed way. Furthermore, the effect of the same parameter on the bond between the prism and the surrounding concrete requires further investigation.
- The use of prisms with different levels of prestress used as reinforcement in the same beam would allow for stage by stage cracking which could be advantageously used to simulate a ductile behaviour in PCP reinforced beams. If this concept is proven, following proper investigation, then continuous beams reinforced with a similar system would be able to undergo moment redistribution, thus increasing their

safety and avoiding sudden failure.

- The use of PCP as reinforcement in slabs would be very attractive, but further testing is needed to establish proper guidelines for design.

Appendix A

Strains in Tested Beams

This appendix includes all the figures of experimental data related to strain measurement. These are strains in flexural reinforcement, shear reinforcement, and in concrete. For comparison of these strains, please refer to one of the main chapters.

The locations of all strain gauges can be found in Figure 4.18.

Figure A.1: Typical strains in reinforcement of beam ST

Figure A.2: Stress in the mid-span of the tension steel reinforcement

Figure A.3: Typical strains in reinforcement of beam L

Figure A.4: Typical strains in reinforcement of beam CP1

Figure A.5: Typical strains in reinforcement of beam CP2

Figure A.6: Typical strains in reinforcement of beam PCP1-1

Figure A.7: Typical strains in reinforcement of beam PCP1-2

Figure A.8: Typical strains in reinforcement of beam PCP-2

Figure A.9: Typical strains in reinforcement of beam PCP3-1

Figure A.10: Typical strains in reinforcement of beam PCP4-2

Figure A.11: Strains in stirrups of beam CP1

Figure A.12: Strains in stirrups of beam CP2

Figure A.13: Strains in stirrups of beam PCP1-1

Figure A.14: Strains in stirrups of beam PCP1-2

Figure A.15: Strain profile in the mid-span of beam CP1

Figure A.16: Strain profile in the mid-span of beam PCP1-2

Figure A.17: Strain profile in the mid-span of beam PCP3-2

Figure A.18: Strain profile in the mid-span of beam PCP4-1

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