BOND CHARACTERISTICS OF FIBRE REINFORCED POLYMERS
PRESTRESSING REINFORCEMENT

by

Zaki Ibrahim Mahmoud

A thesis submitted to the Faculty of Engineering Alexandria University, Egypt in fulfillment of the requirements for the degree of

Doctor of Philosophy

1997
ABSTRACT

Many types of Fibre Reinforced Polymers, FRP, tendons are currently available for prestressing concrete structures. The use of FRP to prestress concrete structures has been investigated by many researchers and various practical applications (ACI Committee 440, 1995). Fundamental understanding of the behavior and specific characteristics of these types of prestressing tendons such as bond strength, transfer and development lengths are of prime importance for the development of design guidelines and codes. An experimental program was conducted to examine the bond characteristics of Carbon Fibre Reinforced Polymers, CFRP, tendons used for pretensioning concrete beams. The bond characteristics are examined through measurements of the transfer and development lengths as well as their corresponding bond stresses. Three types of prestressing bars and strands were used in this experimental program. The first types is conventional 7-wire steel strands (9.6 and 12.7 mm diameter). The second type is the Leadline bars (8 mm diameter), produced by Mitsubishi Chemicals, Japan. The third type is Carbon Fibre Composite Cables, CFCC 7-wire strands (10.5, 12.5, and 15.2 mm diameter), produced by Tokyo Rope Manufacturing Company of Japan. The scope of the experimental program included 52 prestressed concrete specimens fabricated and tested at the structures laboratory of University of Manitoba. Two different types of pretensioned, prestressed specimens were used in this investigation. The first type is the classical beam specimens. The second type was based on a new prism configuration proposed for this investigation. The beam specimens, tested in flexural, were used to determine the transfer and the development lengths. The prism specimens were used to determine the transfer...
length and bond strength. The various parameters considered in this program were diameter of reinforcement, prestressing level, type of CFRP reinforcement (bars, strands), and concrete strength. The transfer length measurements were monitored for one year after transfer for beams pretensioned with steel strands, Leadline and CFCC to study the time-dependent effect on the transfer length. Some specimens were not provided with steel stirrups as shear reinforcements to study the effect of confinement on the transfer and flexural bond lengths. Experimental expressions are developed for estimating transfer and development lengths of Leadline and CFCC strands. The proposed equations are in good correlation with the measured values and it was compared to the current ACI equations as well as other recent proposed models for steel strands.

During the analysis of the test data the need for a rational model to determine the bond development using basic characteristics of the materials was realized. The scope of the research was extended to explore this. A model was developed using principals of solid mechanics incorporating the material and geometrical properties of concrete and prestressing reinforcements. The preliminary analysis using linear elastic theory revealed stress level developed in the concrete that exceeded its tensile strength. This behavior indicate that concrete within the vicinity of the transfer length is cracked. The procedure was modified to recognize the presence of these cracked and uncracked zones around the tendon within the transfer length. A computer program based on the proposed model was developed to determine the bond stress distribution, concrete stress profiles, transfer length and the degree of cracking.
ACKNOWLEDGMENT

The author wishes to express his sincere thanks to his advisors, Dr. Ezz Eldin Zaghloul and Dr. Sami Rizkalla for their guidance, invaluable direction and advice concerning this research and the author's academic future. Their help, encouragement, and constructive criticism during the entire course of this work are gratefully acknowledged. The author would like to thank Dr. Bruce Pinkeny, professor, Civil Engineering Department, University of Manitoba and Dr. T. Ivan Campbell, Professor, Department of Civil Engineering, Queen's University for their time and effort in reviewing the theoretical investigation in this program.

The author wishes to express his appreciation for the technical assistance of Moray McVey and Scott Sparrow. Special thanks go to my friends Dr. Amr Abdelrahman and Emile Shehata for their sincere help throughout this research.

The financial support provided by the Egyptian Cultural and Educational Bureau, ISIS Canada, and Natural Science and Engineering Research Council of Canada is greatly appreciated. The author gratefully acknowledge support provided by Tokyo Rope Mfg. Co. Ltd, and Mitsubishi chemicals, Japan. for providing the materials used in the test program.

Finally, I would like to thank my parents and my wife for their patience, support, encouragement and understanding during these few years while the research was being conducted, for them this thesis is dedicated.
# TABLE OF CONTENTS

LIST OF TABLES  
LIST OF SYMBOLS  

1. INTRODUCTION  
   1.1 GENERAL  
   1.2 RESEARCH SIGNIFICANCE  
   1.3 SCOPE AND CONTENTS  

2. FIBRE REINFORCED POLYMER (FRP) BARS FOR CONCRETE STRUCTURES  
   2.1 EVALUATION OF NEW COMPOSITE MATERIALS  
   2.2 DEFINITION OF FRP  
   2.3 FIBRES  
      2.3.1 Glass Fibres  
      2.3.2 Carbon Fibres  
      2.3.3 Aramid Fibres  
   2.4 MATRIX MATERIALS  
   2.5 PRINCIPAL ASSETS AND DRAWBACKS OF FRP  
   2.6 FRP TENSILE ELEMENTS  
      2.6.1 Glass-Based FRP Prestressing Reinforcement  
      2.6.2 Carbon-Based FRP Prestressing Reinforcement  
      2.6.3 Aramid-Based FRP Prestressing Reinforcement  
   2.7 APPLICATIONS  

3. BOND CHARACTERISTICS OF REINFORCEMENT  
   3.1 ANCHORAGE AND DEVELOPMENT OF REINFORCING BARS  
      3.1.1 Introduction  
      3.1.2 Mechanism of Bond Transfer  
      3.1.3 Variables Affecting Bond Performance  
      3.1.4 Development Length  
      3.1.4.1 Development Length in Tension  
      3.1.4.2 Development Length of Deformed Bars in Compression  
      3.1.5 Test Methods for Evaluating Bond strength and Development length  
      3.1.5.1 Bond pullout Tests  
      3.1.5.2 Bond Beam Tests  
      3.1.5.3 Cantilever Tests  
   3.2 BOND BEHAVIOR OF PRESTRESSING STEEL REINFORCEMENT  
      3.2.1 Introduction  
      3.2.2 Nature of Bond and Mechanism of Transfer  
      3.2.2.1 Transfer Bond  
      3.2.2.2 Flexural Bond
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 2.1</td>
<td>Characteristics of FRP prestressing reinforcement</td>
<td>24</td>
</tr>
<tr>
<td>Table 2.2</td>
<td>Examples of concrete bridges prestressed by FRP reinforcement</td>
<td>25</td>
</tr>
<tr>
<td>Table 3.1</td>
<td>Measured transfer lengths versus $f_p$ and $f_{ci}$ for 9.6 mm Steel strand</td>
<td>65</td>
</tr>
<tr>
<td>Table 3.2</td>
<td>Measured transfer lengths versus $f_p$ and $f_{ci}$ for 12.7 mm steel strand</td>
<td>66</td>
</tr>
<tr>
<td>Table 3.3</td>
<td>Sample Expressions for Transfer Length</td>
<td>68</td>
</tr>
<tr>
<td>Table 3.4</td>
<td>Summary and Comparison of Development Length, $L_d$</td>
<td>69</td>
</tr>
<tr>
<td>Table 3.5</td>
<td>Summary of Bond Strength Study</td>
<td>69</td>
</tr>
<tr>
<td>Table 3.6</td>
<td>Summary of Bond Characteristics of FRP Reinforcements</td>
<td>79</td>
</tr>
<tr>
<td>Table 4.1</td>
<td>Details and Parameters of Specimens Prestressed with Steel Strands</td>
<td>108</td>
</tr>
<tr>
<td>Table 4.2</td>
<td>Details and Parameters of Specimens Prestressed with Leadline Bars</td>
<td>109</td>
</tr>
<tr>
<td>Table 4.3</td>
<td>Details and Parameters of Specimens Prestressed with CFCC Strands</td>
<td>110</td>
</tr>
<tr>
<td>Table 4.4</td>
<td>Characteristics of Prestressing Reinforcements</td>
<td>111</td>
</tr>
<tr>
<td>Table 5.1</td>
<td>Transfer Length Results of Specimens Prestressed with Steel Strands</td>
<td>142</td>
</tr>
<tr>
<td>Table 5.2</td>
<td>Transfer Length Results of Specimens Prestressed with Leadline</td>
<td>143</td>
</tr>
<tr>
<td>Table 5.3</td>
<td>Transfer Length Results of Specimens Prestressed with CFCC strands</td>
<td>144</td>
</tr>
<tr>
<td>Table 6.1</td>
<td>Comparison of Theoretical and experimental Transfer Length Results of Specimens PrestRESSED with Leadline</td>
<td>191</td>
</tr>
<tr>
<td>Table 7.1</td>
<td>Flexural Bond Length Results of Steel Specimens</td>
<td>217</td>
</tr>
<tr>
<td>Table 7.2</td>
<td>Flexural Bond Length Results of Specimens Prestressed with Leadline</td>
<td>218</td>
</tr>
<tr>
<td>Table 7.3</td>
<td>Flexural Bond Length Results of CFCC strands</td>
<td>219</td>
</tr>
</tbody>
</table>
# LIST OF SYMBOLS

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\mu$</td>
<td>coefficient of friction between reinforcement and concrete</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>draw-in at transfer</td>
</tr>
<tr>
<td>$\sigma_0$</td>
<td>circumferential stresses</td>
</tr>
<tr>
<td>$\nu_c$</td>
<td>Poisson’s ratio of concrete</td>
</tr>
<tr>
<td>$\alpha_f$</td>
<td>flexural bond length coefficient</td>
</tr>
<tr>
<td>$\nu_p$</td>
<td>Poisson’s ratio of reinforcement</td>
</tr>
<tr>
<td>$\sigma_r$</td>
<td>radial stresses</td>
</tr>
<tr>
<td>$\sigma_{ra}$</td>
<td>radial stress at the interface</td>
</tr>
<tr>
<td>$\alpha_t$</td>
<td>transfer length coefficient</td>
</tr>
<tr>
<td>$d_b$</td>
<td>nominal tendon diameter</td>
</tr>
<tr>
<td>$E_c$</td>
<td>modules of elasticity of concrete</td>
</tr>
<tr>
<td>$e_{ec}$</td>
<td>eccentricity of prestress force</td>
</tr>
<tr>
<td>$E_f$</td>
<td>modules of elasticity of fibres</td>
</tr>
<tr>
<td>$E_m$</td>
<td>modules of elasticity of matrix</td>
</tr>
<tr>
<td>$E_p$</td>
<td>modules of elasticity of reinforcement</td>
</tr>
<tr>
<td>$E_{pr}$</td>
<td>transverse modules of FRP reinforcement</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>cylinder strength at loading</td>
</tr>
<tr>
<td>$f'_t$</td>
<td>cylinder strength at transfer</td>
</tr>
<tr>
<td>$f_{pe}$</td>
<td>effective prestress (after allowance for all losses)</td>
</tr>
<tr>
<td>$f_{pi}$</td>
<td>initial prestress at transfer</td>
</tr>
<tr>
<td>$f_{pj}$</td>
<td>jacking stress</td>
</tr>
<tr>
<td>$f_{pu}$</td>
<td>ultimate strength of tendon</td>
</tr>
<tr>
<td>$f'_t$</td>
<td>rupture strength of concrete</td>
</tr>
<tr>
<td>$L_t$</td>
<td>transfer length</td>
</tr>
<tr>
<td>$L_d$</td>
<td>development length</td>
</tr>
<tr>
<td>$L_e$</td>
<td>embedment length</td>
</tr>
<tr>
<td>$L_{fb}$</td>
<td>flexural bond length</td>
</tr>
<tr>
<td>$P_j$</td>
<td>jacking force</td>
</tr>
<tr>
<td>$U'_{fb}$</td>
<td>flexural bond index</td>
</tr>
<tr>
<td>$U'_t$</td>
<td>transfer bond index</td>
</tr>
<tr>
<td>$U_{fb}$</td>
<td>flexural bond strength</td>
</tr>
<tr>
<td>$U_t$</td>
<td>transfer bond strength</td>
</tr>
<tr>
<td>$\nu_f$</td>
<td>fibre volume fraction</td>
</tr>
<tr>
<td>$\nu_m$</td>
<td>matrix volume fraction</td>
</tr>
<tr>
<td>$w$</td>
<td>crack width</td>
</tr>
</tbody>
</table>
CHAPTER 1

INTRODUCTION

1.1 GENERAL

In reinforced concrete structures, corrosion of reinforcement has been identified to be one of the most severe problems causing deterioration. This phenomenon has been addressed quite often by engineers and researchers who presented many solutions. Some of these solutions are the use of coatings for the reinforcement such as epoxy coatings, other plastic coatings, cathodic protection, special paints, or sealant. In recent years structures using coatings or cathodic protection have some significant deterioration leading to the belief that these solutions are not as effective as originally claimed. Therefore many engineers are turning to the use of advanced composite materials, or more specifically fibre reinforced polymers (FRP), to replace steel reinforcement (Mufti, 1991, Nanni, 1993, ACI Committee 440, 1995). The materials are relatively new technology therefore much research efforts are needed in order for designers to gain confidence in this material. FRP reinforcement is made from high tensile strength fibres such as carbon, glass, and aramid embedded in polymeric matrices and produced in the form of bars, strands, ropes, and grids, with wide variety of shapes and characteristics.

Recently, fibre reinforced polymers (FRP) rebars and tendons have been used for concrete structures subjected to environmental conditions could aggravate corrosion
and/or where electrical /electromagnetic insulation is required. FRP rebars and tendons present many advantages in terms of high corrosion resistance, high tensile strength, high strength-to-weight ratio, electromagnetic neutrality, and ease of handling. These advantages encourage engineers to use FRP bars and tendons for special concrete structures subjected to corrosive agents such as de-icing salts, marine or soil environments, corrosive gases and copper smelting plants, manholes for electric and telecommunication equipment's, and airport control towers. The drawbacks of FRP reinforcement are limited to the high cost, low shear strength, stress-rupture effect (especially for glass FRP reinforcement), low ductility due to its linear stress-strain behaviour up to failure and low strain at ultimate.

Performance of concrete members depends primarily on the bond between the reinforcement and the surrounding concrete. In pretensioned concrete structures the stresses are transferred and dependant on the bond between the concrete and the reinforcement. Bond strength is affected by many factors such as configuration of the reinforcements, surface conditions, size of the reinforcements, concrete strength, concrete coverage, and variations in the loading conditions. Many studies, both analytical and experimental, have been done to examine the effect of these factors on the bond characteristics of steel strands. Due to the large variation of FRP reinforcements, in terms of shapes, surface treatments and elastic modules, it is expected that the bond performance of FRP reinforced concrete members would be quite different from the steel reinforcement. Therefore the design guidelines for steel cannot be directly applied to FRP reinforcements. Limited data on bond characteristics
of CFRP are available in the literature. These studies were somewhat limited in scope and did not result in comprehensive design recommendations.

The two essential types of bond in prestressed pretensioned concrete members are the transfer bond and flexural bond. The force in the prestressing tendon is transferred to concrete by bond at the end region of a member. The distance from the end of the member over which the prestressing stress, $f_{pi}$, is developed is called the transfer length. The flexural bond length is the additional bond length required to develop from the effective prestress to the ultimate stress, $f_{pu}$, in the prestressing tendon at the ultimate flexural strength of the member.

1.2 RESEARCH SIGNIFICANCE

The primary objective of this research project is to investigate the various bond characteristics of carbon fibre reinforced polymers tendons used to pretension concrete beams. Determination of transfer and development lengths is fundamental for design and performance of pretensioned prestressed concrete members. This data is also significant for the development of design guidelines currently undertaken by the ACI Committee 440 for concrete structures prestressed by FRP reinforcements. The various specific objectives are:

1. Determine experimentally the transfer and development lengths of Leadline bars and CFCC strands.

2. Investigate the effect of the following parameters on the bond behaviour of CFRP:
   
   a- Diameter of reinforcement,
   
   b- Prestress level,
   
   c- Type of CFRP reinforcement (bars, strands),
   
   d- Time-dependent effect,
e- Concrete strength,

f- Confinement produced by shear reinforcement.

3. Compare the bond mechanism of CFRP tendons to that of steel strands.

4. Develop an experimental and rational models for design purposes to predict the transfer and flexural bond lengths of CFRP with the intent of recommending code clauses.

1.3 SCOPE AND CONTENTS

The scope of this study consists of an experimental investigation and theoretical study. The experimental investigation involves fabrication and testing to failure six beams and eight prisms pretensioned by conventional steel strands, twelve beams and four prisms pretensioned by Leadline, and twenty two beams pretensioned by CFCC strands. The experimental study will also involve regression of the test data to propose experimental models of the transfer and flexural bond lengths considering the effect of the prestress level, tendon diameter, and concrete strength. The proposed experimental models are intended to follow the same fashion of the design provisions of the American Concrete Institute (ACI-318, 1989) and AASHTO, 1989.

The scope of the research will extend to present an analytical approach to determine the transfer length based on the material properties and interface characteristics. The model will use the theory of thick-walled cylinder considering only friction bond phenomenon and neglecting the adhesion or mechanical bond. The procedure will recognise the presence of radial cracks within the transfer zone if the circumferential tensile stresses exceed the tensile strength of concrete. A computer program will be developed to carry out the iteration process required to achieve the compatibility of displacements in the radial direction at the
tendon-concrete interface. The rational transfer lengths will be compared with the test data.

The thesis consists of eight chapters. Chapter 2 covers the state-of-the-art information on the properties of FRP reinforcement and their applications in structural engineering. In Chapter 3, the literature review on the bond behaviour, transfer and development lengths of steel and FRP reinforcement is presented. Details of the experimental program conducted at the university of Manitoba which included casting and testing a total of fifty two specimens are presented in chapter 4. This chapter also present the mechanical properties of reinforcement and concrete. Chapter 5 presents the transfer length results, regression analysis of the test results and the proposed experimental models for transfer length of steel strands, Leadline, and CFCC strands. Application and evaluation of the proposed equations and comparison with different equations and experimental results are introduced. Chapter 6 presents a detailed theoretical study using the principals of solid mechanics is presented to predict the transfer bond. The model developed is compared with the experimental data presented in chapter 5. This model is utilized to generate data to perform a parametric study. Results of the development length, the behaviour of the tested specimens and modes of failure are presented in chapter 7. The proposed equations for predicting the flexural bond length for Leadline and CFCC are presented in this chapter. Summary and conclusions of the study are presented in chapter 8. Several conclusions and recommendations are drawn to give an understanding of the bond behaviour of CFRP.
CHAPTER 2

FIBRE REINFORCED POLYMER (FRP) BARS FOR CONCRETE STRUCTURES

2.1 EVALUATION OF NEW COMPOSITE MATERIALS

Structural concrete members are conventionally reinforced and/or prestressed by steel elements, in the shape of bars and meshes and/or prestressing elements such as wires, bars or strands. In structural concrete, the steel is embedded in concrete or cementitious grout and thereby protected against corrosion by the high alkalinity of the concrete or of the grout. In spite of indisputable protective property of concrete and grout, premature losses of durability due to steel corrosion are being increasingly observed all over the world. This is especially true if the structural members are subjected to severe environmental exposure, such as de-icing salts used in cold climate countries and marine and off-shore structures. Serious technical and economic damages arise, endangering the structural integrity, shortening the span of service life of the structures.

Two main avenues, to enhance the durability, can be distinguished. One of the avenues is to protect steel as a reinforcing or prestressing material by using low permeability concrete, zinc and epoxy coating of steel. The other avenue represents the quest for new non-corroding materials as an alternative to steel. Because stainless steel is
unacceptable for structural concrete due to its cost, this quest inevitably led to Fibre Reinforced Polymer, typically abbreviated as FRP.

2.2 DEFINITION OF FRP

FRP is a group of advanced composite materials. FRP are not an invention but the result of steady evolution. This evolution was initiated by a variety of industries for engineering applications. Today FRP are indispensable materials for aircraft, automobiles and for many types of sports gear (Mallic, 1993). The structural concrete industry is the beneficiary of this evolution.

There are many types of FRP with more to emerge in the future. FRP consist of thin and strong fibres of different chemical origin embedded in a matrix. Because here the potential use of FRP as high-strength tensile elements for structural concrete is in the focus of attention, only unidirectional arrangement of virtually endless fibres in a polymeric resin matrix will be dealt with. It should be mentioned that for above-mentioned other applications, the multi-directional arrangement of fibres is used to obtain plate-shaped elements (laminates). It is expected that in the future the use of multi-directional laminates for structural profiles will gain momentum. Such profiles will lead to solutions competitive to structural steel.

2.3 FIBRES

The current commercially available fibres are: glass fibres, aramid fibres, carbon fibres, boron fibres, ceramic fibres and metallic fibres (fibres are also often called filaments; roving are bundles of fibres). For the production of linear tensile FRP elements for the purpose of prestressing and reinforcing of structural concrete, only glass fibres,
carbon fibres and aramid fibres have been successfully used up to now. Figure 2.1 shows the tensile stress-stress diagrams for various types of fibres.

2.3.1 Glass Fibres

Glass fibres are widely used for the production of FRP. Common fibres are made of E-glass. Stronger and more resistant to alkaline solutions, though more expensive than E-glass fibres are fibres made from C- and S-glass. The strength of glass fibres is dependent on the magnitude and density of surface defects which may be inflicted during handling. For the protection of fibres, its surface is treated immediately after fibre drawing with a thin polymeric coat compatible with the polymeric matrices. This coat also improve the bond with the matrix. Glass fibres are round, with the diameter ranging from 5 to 25 μm. Glass fibres are purely elastic up to failure. The average tensile strength of freshly drawn glass fibres may exceed 3.45 GPa. However, surface damage (flaws) produced by abrasion, either by rubbing against each other or by contact with the processing equipment, tends to reduce its tensile strength to values in the range of 1.72 to 2.07 GPa (Mallick, 1993). Strength degradation increases as the surface flaws grow under cyclic loads; this is one of the major disadvantages of using glass fibres in fatigue applications.

The tensile strength of glass fibres is also reduced in the presence of water or under sustained loads (static fatigue). Water bleaches alkalis from the surface and deepens surface flaws already present in the fibres. Under sustained loads, the growth of surface flaws is accelerated due to corrosion by atmospheric moisture. As a result, the tensile strength of glass fibres is decreased by increasing load duration (Mallick 1993).
2.3.2 Carbon Fibres

Although the names of "carbon" and "graphite" are used interchangeably when relating to fibres, there is a difference. Typically, PAN (polyacrylonitrile)-base carbon fibres are 93% to 95% carbon by elemental analysis, whereas graphite fibres are usually more than 99% carbon. The basic difference between these two types relates to the temperature at which the fibres are heat-treated. PAN-based carbon is produced at about 1300 °C, while higher modules graphite fibres are graphitized at 2000 to 3000 °C. A basic disadvantage of the use of carbon/graphite fibres for high strength fibre composite tensile elements in structural engineering is their cost. Less expensive than PAN-base fibres, PITCH-base fibres with greatly improved properties are now being developed may ultimately lead to applications for carbon fibres in structural engineering. For the application of such fibres for tensile elements an adequate strain at fibre fracture is essential. Carbon fibres are highly corrosion resistant. Their diameter is in the range of 5 to 10 μm. Carbon fibres are commercially available with tensile moduli varying from 207 to 1035 GPa (Mallick 1993). Practically speaking, those now produced commercially are 230 to 400 GPa (FIP, 1992). In general, the low-modules fibres have lower specific gravity, lower cost, higher tensile strengths and higher tensile strains to failure than the high-modules fibres. However, the manufacturers of carbon fibres report that the tensile strength of the fibres is in the range of 3000 MPa and the tensile elastic modules about 230 GPa. Among the advantages of carbon fibres are their exceptionally high tensile-strength-weight ratios as well as tensile-modules-weight ratios, very low coefficient of linear expansion, and high fatigue strengths. The disadvantages are their low impact resistance, high electrical conductivity and high cost.
2.3.3 Aramid Fibres

Aramid fibres are synthetic organic fibres, called aromatic polyamide fibres. They are polycondensation product of terephthaloyl chloride and p-phenylene diamine. There are three main producers of high performance aromatic polyamide fibres world-wide: Dupont in U.S. (Kevler-aramid), Aramid Maatschappij v.o.f. in the Netherlands (Twaron-aramid) and Tejin in Japan (Technora-aramid). The diameter of the fibres is in the range of 10 μm.

**Kevlar 49**: Kevlar 49 is a highly crystalline aramid (aromatic polyamide). Among the reinforcing fibres, Kevlar 49 has the lowest specific gravity and the highest tensile strength-weight ratio (Mallick 1993). The major disadvantages of Kevlar 49 are its low compressive strength and difficulty in machining.

Although the tensile stress-strain behaviour of Kevlar 49 is linear, fibre fracture is usually preceded by longitudinal fragmentation, splintering and even localised drawing. In bending, Kevlar 49 fibres exhibit a high degree of yielding on the compression side. Such a non-catastrophic failure mode is not observed in other fibres and gives Kevlar 49 composites superior damage tolerance against impact or other dynamic loading. Kevlar 49 fibres start to carbonise at about 427°C and the recommended maximum long-term-use temperature is 160°C. It is reported that moisture has little or no effect on the properties of Kevlar 49; however, it is quite sensitive to ultra-violet radiation.

2.4 MATRIX MATERIALS

The matrix materials for FRP used for tensile elements are polymers. Most commonly used are the thermo-setting polyester and epoxy resins. In the future also
thermoplastic polymers may become of interest. The matrix has to serve several purposes.

All fibres exhibit a high axial tensile strength but a pronounced sensitivity against lateral normal stress. Thus, the matrix has to protect the fibres from local transverse effects and from abrasion. The ingress of moisture, detrimental media and light is impeded by the matrix. The matrix has to equalise the force distribution, between fibres, by deviatory pressure and interfacial shear. Upon approaching failure of the FRP tensile element fibres will commence to break. It is the task of the matrix to transfer force of broken fibres to unbroken ones by interlaminar shear.

Polyester is the most widely used matrix, especially for GFRP (glass FRP). Polyester resins exhibit adequate resistance to water, many chemical agents, weathering and ageing. Their cost is low. More expensive are epoxy resins which in many respects have a better performance than polyester resins. Resins may absorb water and swell, which may lead to a loss of strength of the FRP.

In the range of service temperatures of concrete structures of about -30 °C to +60 °C the resins are in glassy state. In this state they exhibit an approximate linear stress-strain behaviour under short-term load, whilst being viscoelastic under long-term stress. The glass-transition temperature is between 120 and 140 °C. Their tensile strength is in the range of 40 to 100 MPa, the Young's modules is in the range of 2 to 5 GPa. The coefficient of thermal expansion exceeds that of the fibres markedly: 80 to 100 x 10^-6 k^-1.

the behaviour of a composite is also influenced by the interface between fibre and matrix. The flow of forces between fibre and matrix requires adequate interfacial bond. Bond can be classified in mechanical and chemical bond. Mechanical bond (shear key
effect) seems not to be very efficient. Chemical bond and adhesion are more important effects.

The matrix plays a minor role in the tensile load-carrying capacity of a composite structure. However, it has a major influence on the interlaminar and in-plane shear properties of a composite structure. Matrix materials are either polymeric, metallic or ceramic. Polymers are the most commonly used, and are divided into thermoplastics and thermosets. The tensile stress-strain diagrams of thermosetting polymer (epoxy) and a thermoplastic polymer (polysulfane) are shown in Figure 2.2. Increasing the temperature and decreasing the rate of loading result in an increase in the ultimate strain and a decrease in the ultimate stress of the polymeric solids (Mallick 1993).

2.5 PRINCIPAL ASSETS AND DRAWBACKS OF FRP

The FRP tensile elements, in the shape of rods, strips, strands, and also as thin plates, which are suitable the reinforcing and/or prestressing of concrete members, exhibits a series of assets. Some of these assets make FRP equivalent to reinforcing or prestressing steel, others prove to be superior. These assets comprises:

a. High and adjustable tensile strength and modules of elasticity.

b. Excellent corrosion resistance to a plurality of environments aggressive to steel.

c. Very low specific weight.

d. Magnetic and electric neutrality.

e. Carbon and aramid FRP tendons have excellent fatigue characteristics, however the fatigue strength of glass FRP reinforcements is significantly below steel’s.

f. Low coefficient of thermal expansion in the axial direction, especially for carbon FRP.
It should not be suppressed that there are also drawbacks:

a. One serious obstacle is the high price of FRP at present. But as FRP are being increasingly used for many applications mainly other than civil engineering structures, their price decreases from year to year.

b. FRP are strong when stressed axially, but very sensitive against lateral pressure. This phenomenon calls for new ways of anchorage designed of structural detailing.

c. Some types of FRP are sensitive to high alkalinity of concrete, others however are entirely insensitive.

d. Aramid and glass FRP have relatively low modules of elasticity.

e. Low ultimate strain at failure of FRP reinforcements effects the deformability requirements of structures.

f. Long-term strength can be lower than short-term strength for FRP reinforcements due to creep rupture phenomenon.

g. Susceptibility of FRP to damage by ultra-violet radiation.

h. Glass fibres may deteriorate due to water absorption.

i. High transverse thermal expansion in comparison to concrete.

2.6 FRP TENSILE ELEMENTS

The FRP tensile element are unidirectional composites of fibres which are fully embedded in epoxy or polyester matrices. They can be utilized for reinforcing and/or prestressing of structural concrete. In addition, that they can be applied for ground and rock anchors, for structural ties and another purposes. FRP elements are not standardised, neither on national nor on international levels. Hence, it will become necessary to mention
brand names and producers of FRP to give the interested reader the opportunity to contact producers for further information.

Most of the FRP tensile elements have either circular or rectangular cross-section (round bars, strands, ropes, or flat strip). Diameters are typically in the range of 4 to 25 mm. Flat strips have cross-sectional areas of up to 150 mm². FRP elements used for pretensioning of concrete members may be surface-treated to improve bond. The surface may be sanded, coated, or shaped.

The characteristics of FRP reinforcement differ greatly according to the properties of the matrix and fibres and to the volume fraction of the fibres. Moreover, various parameters affect the stress-strain characteristics of FRP reinforcement, such as diameter and length of reinforcing bars, temperature, and rate of loading. The material characteristics of reinforcement made of glass-fibre-, carbon-fibre- and aramid-fibre-reinforced polymers (GFRP, CFRP and AFRP respectively) as compared to those of prestressing steel strands are shown in Figure 2.3. Similarly to the behaviour of the fibres and unlike that of prestressing steel, FRP reinforcement does not yield but remains linearly elastic up to failure. Young's modules of FRP reinforcement is much less than that of prestressing steel strand, ranging between 50 and 150 GPa, while the tensile strength is close to that of the steel. The characteristics of different commercial FRP prestressing reinforcement are given in Table 2.1.

2.6.1 Glass-Based FRP Prestressing Reinforcement

GFRP is the least expensive type of FRP reinforcement. Since GFRP has a very low transverse shear strength, which makes it difficult to make efficient anchorages for prestressing, most of the GFRP reinforcement commercially available is proposed as non-prestressed reinforcement. Surface treatments such as quartz sand, to give a rough finish, and external
fibre winding, to produce a ribbed surface, have been applied to GFRP bars to improve their bond to concrete. One commercial application of GFRP prestressing reinforcement is "Polystal". Other GFRP bars are produced for different applications such as Isorod by Pultall Inc. (Canada), IMCO by IMCO Reinforced Plastics Inc. (USA), Jitec by Cousin Frère (France), Kodiak by IGI International Grating (USA) and Plalloy by Asahi Glass Matrex (Japan). The following is a brief summary of the properties of Polystal GFRP prestressing tendons.

**Polystal:** is produced by Bayer AG in association with Strag Bau-AG in Germany. This tendon consists of bundles of bars, each containing E-type glass fibre filaments in an unsaturated polyester resin matrix. The 7.5 mm diameter bar, with loosely packed fibres of 68 percent by volume (80 percent by weight), consists of about 60000 glass fibres. The specific weight of Polystal is 2.0 gm/cm³ and its coefficient of linear thermal expansion is 0.7 x 10⁻⁵ (1/°C). The tensile strength of the Polystal is about 1670 MPa, while its young's modules is 51 GPa. It is reported (Preis 1988) that the time-dependent creep strength, which is called a creep-rupture phenomenon, is about 70 percent of the short-term strength as shown in Figure 2.4. Polystal survived two million cycles under a stress range of 55 MPa with 10 percent failure probability, when a tensile fatigue test was conducted under a maximum stress (σₐ) = 736 MPa, which is 44 percent of the ultimate strength as shown in Figure 2.5.

2.6.2 Carbon-Based FRP Prestressing Reinforcement

CFRP prestressing reinforcement has the highest tensile modules of elasticity of the various types of FRP reinforcement (about 70 percent of the modules of prestressing steel). The maximum strain at failure is between 1.2 and 2 percent. The axial coefficient of thermal
expansion is very low compared to that of prestressing steel (0 to 0.6×10⁻⁶ 1/°C), while the radial coefficient of thermal expansion of CFRP ranges between 35 and 50×10⁻⁶ 1/°C (Mallic, 1993). CFRP prestressing reinforcement is made in the form of bars, ropes and cables, using PAN or PITCH-based carbon fibres. Various types of CFRP reinforcement are produced, such as CFCC by Tokyo Rope (Japan), Leadline by Mitsubishi Kasei (Japan), Jitec by Cousin Frère (France) and Bri-Ten by British Ropes (UK).

**Carbon Fibre Composite Cables (CFCC):** are made by Tokyo Rope (Tokyo Rope Mfg. Co., Ltd. 1993) in Japan using carbon fibres of polyacrylonitrile (PAN) supplied by Toho Rayon. The wires are made from roving prepreg, which consist of 12,000 filaments impregnated with resin. The prepreg is twisted to create a fibre core which is wrapped in synthetic yarns, thereby becoming a single cable. The yarn covering protects the core from UV radiation and mechanical damage, and improves the bond of CFCC to concrete. Cables consist of either single, seven, nineteen or thirty-seven wires. The wires are twisted to allow better redistribution of stresses through the cross-section. CFCC is available in diameters of 3 to 40 mm with a maximum length of 600 m. CFCC cables are flexible enough to be coiled on drums for transportation. Prior to heat curing, CFCC can be shaped into rectangular or circular spirals to be used as stirrups or confining reinforcement.

The tensile strength of CFCC varies according to the diameter of the cable. For the 12.5-mm cable, the tensile strength and modules are 2100 MPa and 137 GPa, respectively. The coefficient of linear thermal expansion is approximately 0.6×10⁻⁶/°C, which is about 1/20 of that of the steel. CFCC shows a relaxation of about 3.5 percent after 30 years at 80 percent of the ultimate load, which is 50 percent less than that of prestressing steel strand. Figure 2.6 shows the tensile fatigue characteristics of CFCC, where a limit of two million cycles shows
higher amplitude of stress for CFCC than the amplitude of stress of prestressing steel strand subjected to the same mean stress. Pull-out tests show that CFCC has bond strength to the concrete of more than two times higher than that of the steel.

**Leadline**: produced by Mitsubishi Kasei (Mitsubishi Kasei Corporation 1992), is another application for the use of carbon fibres in prestressing reinforcement. Leadline bars are pultruded using linearly oriented Dialead coal tar PITCH-based continuous carbon fibre and an epoxy resin. Leadline bars have round, ribbed, and indented shapes and a guaranteed tensile strength and modules of 1970 MPa and 147 GPa, respectively, for 8-mm diameter. The anticipated relaxation of Leadline, after 30 years, is between 2 and 3 percent. The fatigue strength of 3-mm-diameter and 400-mm-length Leadline bar is shown in Figure 2.7. At a constant stress range of 10 percent, the fatigue strength is 55 percent of the static strength at 10 million cycles.

### 2.6.3 Aramid-Based FRP Prestressing Reinforcement

Three different types of Aramid fibres are used in the industry to produce AFRP prestressing reinforcement, namely Kevlar (29-49-149) by Dupont in USA, Twaron by Aramid Maatschappij v.o.f. in the Netherlands, and Technora by Teijin in Japan. The fibre tensile strengths and moduli of elasticity are, respectively, 2650 MPa and 165 GPa for Kevlar 49, 2800 MPa and 125 GPa for Twaron, and 3400 MPa and 73 GPa for Technora. AFRP prestressing reinforcement is produced in different shapes such as spiral-wound, braided, and rectangular bars. The phenomenon of creep-rupture failure has been observed for AFRP reinforcement. Different AFRP prestressing elements are produced such as Technora by Teijin (Japan), Fibra by Mitsui (Japan), Arapree by AKZO and Hollandsche Beton Groep nv.
(Holland), Phillystran by United Rope Works (USA), and Parafil Ropes by ICI linear Composites (UK).

**Technora:** Teijin Limited of Japan produces bars of brand name Technora. Both round bars and bars with external spiral windings are available in diameters of 3 to 8 mm. The bars are flexible enough to be wound on a 1.5-meter-diameter drum and may be cut to the required lengths with a grinder. The coefficient of linear thermal expansion is \(-15\times10^{-6}/^\circ\text{C}\). Relaxation of Technora AFRP prestressing reinforcement ranges between 7 and 14 percent (Kakihara, Kamiyoshi, Kumagai and Noritake 1991), as shown in Figure 2.8. Fatigue characteristics of 6-mm AFRP bar are shown in Figure 2.9. It is also reported that the number of cycles to fracture is higher than 1 million at practical levels of load, 1000-1200 MPa, and with an amplitude of 200-300 MPa.

**Fibra:** is produced by Mitsui construction company by braiding multiple bundles kevlar 49 fibres and impregnating the braided fibres with an Epicote 827 and teta epoxy resin (Mitsui Construction Co., Ltd.). Fibra bars are available in diameters from 3 to 16 mm, and can be wound on a 1.5 meter drum for easy transportation. Braiding was selected for manufacturing Fibra bars to increase the bond with concrete. Moreover, the bond can be further improved by applying a quartz sand finish to the bars. Relaxation after 100 hours was found to be 10 percent regardless of the initial load, which is two to three times greater than that of prestressing steel. There was no apparent deterioration when the bars were placed in an alkaline environment with elevated temperature up to 80°C.
2.7 APPLICATIONS

Commercial and industrial applications of fibre reinforced composites are so varied that it is impossible to list them all. In this section, we only highlight the major structural application areas. The above described assets of FRP caught the interest of structural concrete industry searching for a non-corroding alternative to steel. Following laboratory research, first application in real scale are realised in many countries. Especially in Europe, Japan, USA and Canada, first bridges and structures using FRP tensile elements for pre- and post-tensioning of concrete are being built. Some of these applications are already in service successfully for five to ten years. FRP are also being used for the epoxy bonded external strengthening of concrete structures. FRP are a material in many ways different from reinforcing and prestressing steel. This calls for new methods of testing and design, of structural detailing, and for new ways for the reliable anchorage and transfer of force.

FRP has been used in the following structural applications:

a. **Short span bridges:** Many pedestrian bridges have been constructed using FRP. A Philadelphia company, E.T Techtonics, has constructed pedestrian bridges using Kevlar 49 cables to prestress glass-fibre king-post, queen-post and truss bridges. Bridges of 7- to 10-m span have been constructed (cited in Mufti, Erki, and Jaeger 1991).

The first application of CFRP tendons in Canada was in tensioning of six girders of a highway concrete bridge, built in Calgary, Alberta, Canada. The BeddingtonTrail/ Centre street bridge is two skew continuos spans of 22.83 m and 19.23 m. The bridge was constructed using thirteen bulb-T section precast pretensioned concrete girders, in each span. The two types of CFRP tendons, used in pretensioning, are 8 mm diameter Leadline bars manufactured by Mitsubishi Chemical, Japan and 15.2 mm diameter carbon Fibre Composite
Cable CFCC produced by Tokyo Rope, Japan (Abdelrahman, Tadros, and Rizkalla, 1995). Table 2.2 gives information about some concrete bridges prestressed with FRP prestressing tendons.

The first highway bridge, prestressed with cables consisting of glass fibre bars, was opened for traffic in 1986 in Dusseldorf, Germany. This bridge is a continuous structure with spans of 21.3 and 25.6 m. A total of 59 tendons, each composed of 19 bars, 7.5 mm in diameter, provide forces of 600 kN per unit. The solid slab has a width of 15 m. Permanent remote control of the individual prestressing tendons, by means of optical fibre sensors and copper wire sensors, confirmed a perfectly normal structural behaviour. This monitoring is still continuing.

In 1988, GFRP was used in Berlin, Germany, in a two-span bridge with a pedestrian traffic lane and a bridle path. The double tee cross section is about 5 m wide and the spans equal 27.6 and 22.9 m. Prestressing was accomplished with 7 external tendons, consisting of 19 glass fibre bars. Also in this bridge, sensors were applied for long term monitoring of the bridge.

In 1991, a three span highway bridge was completed at the Bayer plant in Leverkusen, Germany. Two span lengths are at 16.3 m and one at 20.4 m. The 1.10 m thick slab was prestressed with 27 glass fibre tendons.

In the early 1992, the Notsch Bridge in Austria was opened for traffic. Span lengths are 2 at 13 m and 1 at 18 m. The 0.75 m thick slab was prestressed with 41 glass fibre tendons.

b. Long span bridges: As a result of the very high strength-to-weight ratio of FRP compared to those of conventional materials, FRP can compete with steel and concrete in the
construction of long span bridges. FRP structural sections can be utilised in the construction of the bridge girders, while unidirectional FRP tendons can be used as cables under tension. To date, the construction of relatively long-span bridges with FRP has not been reported. A proposal for a carbon-fibre-reinforced-composite bridge across the strait of Gibraltar at its narrowest site was reported by Meier, 1988 (cited in Mufti et al., 1991), as shown in Figure 2.10. Meier outlined the feasibility of constructing cable-stayed and classical suspension bridges of steel, glass-fibre-reinforced polymers (GFRP) and carbon-fibre-reinforced polymers (CFRP) and concluded that the most feasible design would be a cable-stayed bridge of CFRP.

c. **Repair of structures:** A number of chimneys, columns, slabs and girders have been strengthened against overloads from earthquake or changes in use using CFRP products (Mufti, Erki and Jaeger 1992). Many products are available for such retrofitting. These are most often unidirectional fibre tapes, fibre winding strands and fabrics. Flexural and shear strengthening are possible using these materials.

The process used to retrofit a typical chimney starts with the preparation and trawling of the concrete surface with mortar or epoxy, followed by application of auto-adhesive tapes in the longitudinal direction and confinement of the chimney by circumferentially wound carbon strands. An automatic winding machine was made to facilitate the strand winding operation. Finally, a fire-resistant covering material, such as mortar, is applied on the surface.

The wide range of application of glass fibre tendons is further demonstrated by their use in the rehabilitation of the Mairie d’Ivry metro station in Paris, France. As a result of one-sided excavation directly adjacent to the subway station, considerable cracking had occurred in the 70-year-old concrete vault over a length of about 110 m. Thirty-six glass fibre prestressing tendons were installed to strengthen the vault. The service load per tendon equals 650 kN. In this
application the electromagnetic neutrality of the tendons proved to be a favourable property. This also the case for antenna mast bracing.

d. **Repair of bridges:** Many highway bridges built 40 years ago have deteriorated due to the increased weight of trucks legally permitted on highways. In addition, corrosion caused by de-icing salts has made the deterioration even more severe. Strengthening of deteriorated steel and concrete structures by bonding carbon-fibre-reinforced epoxy laminates to the exterior of the structure has been studied in Switzerland and Germany (Meier et al 1992). The study has shown that the use of CFRP laminates in place of steel plates for such applications can reduce the total cost of a reinforcing project by about 20 percent. Although the FRP materials are higher cost than steel, the lighter weight and better corrosion-resistant properties can result in significant reductions in fabrication and long-term costs.

e. **Tunnel lining:** FRP grids covered by shotcrete can be used for tunnel lining. FRP has advantages over the steel due to its high corrosion-resistance, its flexibility which makes it suitable for curved surfaces and its excellent alkali-, acid- and chemical-resisting properties. The material is very in light weight, having approximately one-fourth the specific gravity of steel, and may be cut easily with a hack-saw.

f. **Off-shore structures:** Sen, Issa and Iyer (1992) reported a feasibility study of fibreglass pretensioned piles in a marine environment. FRP was used in reinforcing and prestressing the first concrete floating bridge in Japan (Tezuka et. al, 1993). A new type prestressed concrete jetty which is reinforced with CFRP was constructed at Kitakyusyu City, Japan (Nakai et al., 1993).
g. Aircraft and military applications: The structural applications for fibre reinforced composites are in the field of military and commercial aircrafts, for which weight reduction is critical for higher speeds and increased payloads. The structural integrity and durability of FRP components have built up confidence in their performance and prompted developments of other structural aircraft components.

h. Space applications: Weight reduction is the primary reason for using fibre reinforced composites in many space vehicles. Another major factor in selecting FRP in many spacecraft applications is their dimensional stability over a wide temperature range. Many CFRP laminates can be designed to produce a coefficient of thermal expansion (CTE) close to zero. Many aerospace alloys (e.g., Invar) also have comparable coefficient of thermal expansion. However CFRP have much lower specific gravity and higher strength as well as a higher stiffness-weight ratio. Such a unique combination of mechanical properties and CTE has led to a number of applications for CFRP in artificial satellite. One such application is found in the support structure for mirrors and lenses in the space telescope. Since the temperature in space may vary between -100 and 100 °C, it is critically important that the support structure be dimensionally stable.

i. Marine applications: Glass FRP has been used for over 45 years in marine applications. Fibreglass boats are roughly 95 percent of all the boats manufactured in the USA. Durability and performance of fibreglass in salt water has thus been proven with time. The principal advantage is the weight reduction, which translates into higher cruising speed, acceleration, and fuel efficiency.
Table 2.1 Characteristics of FRP prestressing reinforcement

<table>
<thead>
<tr>
<th>Commercial name</th>
<th>Fibre</th>
<th>Matrix</th>
<th>V/ (%)</th>
<th>Diam. (mm)</th>
<th>Density (gm/cm³)</th>
<th>Tensile strength (MPa)</th>
<th>Tensile modules (GPa)</th>
<th>Ultimate tensile strain (%)</th>
<th>Coefficient of thermal expansion (1/°C)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polystal (Strabag Bau-AG)</td>
<td>Glass</td>
<td>Polyester</td>
<td>68</td>
<td>7.5 to 25</td>
<td>2.0</td>
<td>1670</td>
<td>51</td>
<td>3.3</td>
<td>7 x 10⁻⁶</td>
<td>Based on 7.5 mm bar</td>
</tr>
<tr>
<td>Leadline (Mitsubishi Kasei)</td>
<td>Carbon</td>
<td>Epoxy</td>
<td>65</td>
<td>1 - 17</td>
<td>1.6</td>
<td>1970</td>
<td>147</td>
<td>1.3</td>
<td>0.68 x 10⁻⁶</td>
<td>Based on 8 mm bar</td>
</tr>
<tr>
<td>CFCC (Tokyo Rope)</td>
<td>Carbon</td>
<td>Epoxy</td>
<td>64</td>
<td>3 - 40</td>
<td>2.0</td>
<td>2100</td>
<td>137</td>
<td>1.57</td>
<td>0.6 x 10⁻⁶</td>
<td>Based on 12.5 mm cable</td>
</tr>
<tr>
<td>Bri-Ten (British Ropes Ltd.)</td>
<td>Carbon</td>
<td>Vinyl-ester</td>
<td>63</td>
<td>51</td>
<td>n/a</td>
<td>1480</td>
<td>136</td>
<td>1.1</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Technora (Teijin)</td>
<td>Aramid</td>
<td>Vinyl-ester</td>
<td>65</td>
<td>3 - 8</td>
<td>1.3</td>
<td>1900</td>
<td>54</td>
<td>3.7</td>
<td>-15 x 10⁻⁶</td>
<td></td>
</tr>
<tr>
<td>Fibra (Mitsui)</td>
<td>Aramid Kevlar 49</td>
<td>Epoxy</td>
<td>60</td>
<td>4 - 14</td>
<td>1.3</td>
<td>1255</td>
<td>64.8</td>
<td>2</td>
<td>-5.2 x 10⁻⁶</td>
<td>Based on 8 mm bar</td>
</tr>
<tr>
<td>Arapree (AKZO)</td>
<td>Aramid</td>
<td>Epoxy</td>
<td>37 to 45</td>
<td>5.7 to 7.9</td>
<td>1.23</td>
<td>1000 to 1200</td>
<td>43 to 53</td>
<td>2.3</td>
<td>-2 x 10⁻⁶</td>
<td>Rectangular &amp; circular shapes</td>
</tr>
<tr>
<td>Parafil™ Rope (ICI)</td>
<td>A Terylene 29</td>
<td>Kevlar 49</td>
<td>100</td>
<td>4.5</td>
<td>0.9 to 1.0</td>
<td>616</td>
<td>12</td>
<td>5.1</td>
<td>-5.7 x 10⁻⁶</td>
<td>A, F &amp; G are 3 different types of Parafil</td>
</tr>
<tr>
<td></td>
<td>F Kevlar 49</td>
<td>140</td>
<td>1926</td>
<td>126.5</td>
<td>1.5</td>
<td>1926</td>
<td>77.7</td>
<td>2.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Fibre content by volume  
† Tensile strength and modules are based on gross sectional area  
‡ Parafil may not be classified as FRP tendons, since the fibres are not embedded in a polymer matrix  
n/a not available
Table 2.2 Examples of concrete bridges prestressed by FRP reinforcement

<table>
<thead>
<tr>
<th>Material</th>
<th>Bridge description</th>
<th>Diameter of reinforcement</th>
<th>Dimensions of the bridge</th>
<th>$P/P_u$*</th>
</tr>
</thead>
<tbody>
<tr>
<td>FRP</td>
<td>BASF post-tensioned prestressed concrete highway bridge, Germany, 1991</td>
<td>Four cables, each is made of 19 1x7 $#12.5$</td>
<td>11.2 m wide x 80 m long, four 20 m spans, 2 straight and 2 curved</td>
<td>50 % under design load</td>
</tr>
<tr>
<td>CF</td>
<td>Nagatsugawa pretensioned simple slab pedestrian bridge, Japan, 1989</td>
<td>Cables of 1x7 $#12.5$ mm</td>
<td>2.5 m wide x 8.0 m long</td>
<td>60, 55 and 50 %, jacking, initial and at design load</td>
</tr>
<tr>
<td>FRP</td>
<td>Kitakyusyu prestressed concrete highway bridge, Japan, 1989</td>
<td>8 multi-cables bundled with 8 CFRP rods of 8 mm diameter</td>
<td>35.8 m long, pretensioned girder (18.2 m span) and post-tensioned girder (17.5 m span)</td>
<td>55 % under design load</td>
</tr>
<tr>
<td>CF</td>
<td>Shinmiya pretensioned concrete slab highway bridge, Japan, 1988</td>
<td>eight 1x7$#12.5$</td>
<td>5.76 m span and 7 m wide</td>
<td>60, 55 and 45 %, jacking, initial and at design load</td>
</tr>
<tr>
<td>CF</td>
<td>Calgary bridge, Canada. Six out of 26 bulb-T pretensioned girders, Nov., 1993</td>
<td>52 Leadline bars 8 mm diameter or 26 CFCC strands 15.2 mm diameter</td>
<td>two skew continuos spans 22.83 m and 19.23 m</td>
<td>60% of the guaranteed strength</td>
</tr>
<tr>
<td>Material</td>
<td>Bridge description</td>
<td>Diameter of reinforcement</td>
<td>Dimensions of the bridge</td>
<td>( P/P_u )</td>
</tr>
<tr>
<td>----------</td>
<td>-------------------</td>
<td>--------------------------</td>
<td>--------------------------</td>
<td>------------</td>
</tr>
<tr>
<td></td>
<td><strong>Demonstration</strong> bridges for Technora, pretensioned composite slab and post-tensioned box girder, Japan, 1990 and 1991</td>
<td>3/6 pretensioned strands, 19/6 post-tensioned cables and 7/6 external cables</td>
<td>11.79 m span for the pretensioned bridge and 24.1 m span for the post-tensioned girder</td>
<td>75, 70, and 60 %, jacking, initial and at design load</td>
</tr>
<tr>
<td>A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Mito city post-tensioned concrete pedestrian suspended slab bridge, Japan, 1990</strong></td>
<td>16 cables, each is made of 8 bands 4.86 x 19.5 mm</td>
<td>2.1 m wide and 54.5 m long</td>
<td>50 % under design load</td>
</tr>
<tr>
<td></td>
<td><strong>Nasu pretensioned prestressed concrete highway bridge, Japan, 1990</strong></td>
<td>Braided AFRP 14 mm diameter</td>
<td>3 spans 11.98 m each</td>
<td>50 % under design load</td>
</tr>
<tr>
<td></td>
<td><strong>The Marienfelde pedestrian externally prestressed bridge, Germany, 1989</strong></td>
<td>Cables of 1x19 ø 7.5</td>
<td>5 m wide and two spans of 17.6 and 23 m long</td>
<td>n/a†</td>
</tr>
<tr>
<td></td>
<td><strong>The Ulenbergstrasse post-tensioned prestressed highway bridge, Germany, 1986</strong></td>
<td>59 cables of 1x19 ø 7.5</td>
<td>two spans of 21.3 and 25.6 m long</td>
<td>47 % under design load</td>
</tr>
<tr>
<td></td>
<td><strong>Lünen'sche Gasse single span slab bridge, Germany, 1980</strong></td>
<td>100 rods of 7.5 mm diameter</td>
<td>6.55 m span</td>
<td>n/a†</td>
</tr>
</tbody>
</table>

* \( P/P_u \) is the ratio of the prestressing force to the ultimate strength of the cables
† not available
Figure 2.1 Tensile stress-strain diagrams for various reinforcing fibres.

Figure 2.2 Tensile stress-strain diagrams of a thermosetting polymer (epoxy) and a thermoplastic polymer (polysulfone).
Figure 2.3 Stress-strain curves of FRP-bars and prestressing steel.

Figure 2.4 Time-dependent tensile strength of glass fibre composite bars.
Figure 2.5 Fatigue behaviour of GFRP bars.

Figure 2.6 Tensile fatigue characteristic of CFCC.
Figure 2.7 Fatigue behaviour of Leadline.

Figure 2.8 Relaxation of Technora rods.
Figure 2.9 Fatigue behaviour of Technora rods.

Figure 2.10 Proposal for a CFRP bridge across the Strait of Gibraltar.
CHAPTER 3

BOND CHARACTERISTICS OF REINFORCEMENT

3.1 ANCHORAGE AND DEVELOPMENT OF REINFORCING BARS

3.1.1 Introduction

Bond between concrete and reinforcing bars is one of the most important aspects affecting the design of reinforced concrete structures. Bond stress is the name assigned to the unit shear stress acting parallel to the bar on the interface between bar and concrete. In transferring load between the bar and surrounding concrete, this shear stress (bond stress) modifies the stresses in the bar, either increasing or decreasing it. Therefore, when bond is efficiently developed, the two materials, concrete and steel, can form a composite structure. Designers must be aware of the aspects of bond and anchorage that can critically affect the structure behaviour. Since failure in the development length (L_d) tends to be brittle, reserved strength in the development length is essential to maintain member ductility. Bond stresses must be present whenever the force in reinforcing bar changes. These changes may be due to bar anchorage or due to change of the bar force along its length due to change of bending moment along the member. The free body diagram in Figure 3.1 illustrates the average bond stress, \( u \), that act on the bar surface for a length \( l \) to maintain equilibrium:
where $\Delta f_s$ = the longitudinal stress increase in reinforcement,

d_b = diameter of reinforcement.

When the length, $l$, becomes very short length, $\Delta x$, then:

$$\frac{\Delta f_s}{\Delta x} = \frac{4u}{d_b}$$  \hspace{1cm} (3.2)

Equation (3.2) implies that the actual bond stress, $u$, depend on rate of change of steel stresses along the length. Flexural bond stresses arises in beams between two cracks close to each other to resist the change in bar forces are shown in Figure 3.2. The average bond stress between two cracks in a beam, $u_{avg}$, can be estimated as:

$$u_{avg} = \frac{V}{(jd)(\Sigma O)}$$  \hspace{1cm} (3.3)

where $V$ = the shearing force acting at the section,

$jd$ = the internal lever arm,

$\Sigma O$ = the sum of the perimeter of reinforcement.

Equation (3.3) indicates that when the rate of change of external bending moment (ie., the shearing force, $V$) is high, the bond stresses can also exhibit high intensity. Therefore, anchorage length must be checked at end support and points of contraflexural in continuous beams, where the bending moments almost zero and shearing forces is large. Figure 3.3 shows the effect of flexural cracks on bond stresses in beam and the variation of both actual and average bond stresses along the span.
Between discrete cracks of a constant moment zone in simple beam, the concrete does resist moderate amount of tension, introduced by bond stresses acting along the interface in the directions shown in Figure 3.4b. This is evidence by the reduction of the tensile force in the steel as illustrated by Figure 3.4c. From equation (2), it is clear that the bond stress, \( u \), is proportional to the rate of change of bar force, and thus, the bond stress could vary as shown in Figure 3.4d. Since the shearing forces within the constant moment region is zero, the average of in-and-out bond stresses should also be zero. Similar to this case, is the axially loaded tension member shown in Figure 3.5 although there are no shearing forces, there are an in-and-out equal bond resultants and the average bond stress equals zero as stated by Equation 3.3.

### 3.1.2 Mechanism of Bond Transfer

Bond resistance could be attributed to chemical adhesion, mechanical interlock effects, and frictional forces. Frictional forces are generally due to the surface roughness and confining pressure caused by shrinkage of concrete. Both adhesion and friction are quickly lost when the bar is loaded in tension. Particularly because the diameter of the bar decreases slightly due to poison’s ratio. Therefore, in the case smooth bars, mechanical anchorage is necessary. In case of deformed bars, bond is mainly transferred by bearing stresses act on the concrete against the face of the ribs. These stresses have longitudinal and radial components as shown in Figure 3.6. The radial component, like water pressure in a pipe, causes circumferential tensile stresses in the concrete cover around the bar. Eventually the weaker plane will split parallel to the bar and the resulting crack will propagate out to the surface of the beam. The load at which splitting failure occurs depend on the cover thickness, spacing of bars, tensile strength of concrete, and
the applied average bond stresses. As soon as these splitting cracks form, drop of bond transfer will occur unless lateral reinforcement is provided to restrain the opening of splitting cracks.

The major tension cracks in axially loaded tension member or flexural members divide the tension zone into a number of shorter specimens like that shown in Figure 3.7. The stresses in the concrete surrounding a deformed bar lead to cracks and deformations of the concrete. The bond stress, u, transmitted to the concrete, subjects the cover thickness of concrete to eccentric tension. The deformation of concrete resulting from the generated stresses tend to pull concrete away from this steel in the vicinity of major crack. The tensile strength of the adhesive bond between steel and the mortar is then reached, and the surrounding concrete separates from the steel. Figure 3.7 shows that numerous internal secondary cracks can form and may not propagate to the external surface of the concrete. The symmetric internal crack pattern, with cracks starting just behind each lug and progressing diagonally a limited distance toward the nearest transverse crack, is impressive. Because cracking in concrete results from a principal tensile stress, there must be compressive stresses parallel to these cracks, forces directed outwardly around the bar to form a hollow cone of pressure. This pressure is directed inwardly against the lug and outwardly must be resisted by circumferential tensile stress in the concrete surrounding the bar (see Figure 3.8). The pictured angle between crack and bar axis is surprisingly large, that is, considerably more than 45°. The increase of the inclination of forces increases the radial splitting forces.
3.1.3- Variables Affecting Bond Performance.

There are several factors that affect the bond resistance of rebars. Among of these are: concrete compressive strength, concrete coverage, bar spacing, bar size, embedment length, yield stress of reinforcement, amount and position of transverse steel, stress range, type and rate of loading, temperature, and surface condition (e.g., rusting, coating, rib geometry). The following sections discuss the influence of some of these variables on the bond strength of reinforcing bars.

A- Rib Geometry

The bond strength developed between two ribs of a deformed bar is associated with adhesion and friction along the bar surface ($v_a$), bearing stress ($f_b$) against the face of the rib, and shear stress ($v_c$) acting on the cylindrical concrete surface between adjacent ribs in Figure 3.9. The relationship between these stresses and the force to be transferred to the concrete by bond can be obtained from simple equilibrium as follows:

$$
\Delta T = \pi d_b' (b+c) v_a + \pi \frac{d''_b - d''_b}{4} f_b \approx \pi d_b'' c v_c \quad (3.4)
$$

where each term can be identified in Figure 3.9. As the load is being increased, the adhesion along the bar surface inevitably breaks down. The remaining frictional shear strength is very small in comparison with the bearing strength developed around the ribs. Therefore $v_a$ can be ignored for practical purposes. The relationship between the remaining two important components of bond force development, $f_b$ and $v_c$, can be simplified as follows:

$$
v_c \approx \frac{a}{c} f_b \quad (3.5)
$$
When ribs are high and spaced too closely, the shear stress $v_c$ will govern the behaviour and the bar will pull out. When the rib spacing is larger than approximately 10 times the rib's height, the partially crushed concrete may form a wedge in the front of the rib, and splitting failure of the surrounding concrete occurs. The concrete in the front of the rib can sustain a bearing pressure several times the cylinder crushing strength because of the confined condition of concrete. The two types of failure mechanism, associated with the rib, are illustrated in Figure 3.10. The deformation requirements of ASTM are such that $0.057 < a/c < 0.072$.

**B- Confinement**

Confining of concrete can restrict the widening of splitting cracks. Transverse compression is available at the support of simply supported beams, this transverse compression is beneficial to the anchorage of bottom reinforcement. At the intersection of beams in building frames, compression or tension is induced transversally to the bars. Similarly transverse tension may be induced in the concrete around the top bars of the beams that support continuous slabs. Such transverse tensile stresses could lead to early cracking along the principal bars and could adversely affect their bond performance. Conversely, transverse compression can provide beneficial confinement to embedded bars.

Increased concrete cover and spacing between bars have been found to produce some increased resistance against splitting. However the improved bond performance is not proportional to the additional cover thickness. Typical splitting failure surfaces are shown in Figure 3.11. The splitting cracks tend to develop along the shortest distance between a bar and the surface or between two bars. When dowel action affects bond, the influence of cover is
eliminated. Stirrups, particularly when closely spaced, prevent the opening of cracks that form along the embedded bars and enable greater bond forces to be transmitted. Stirrups do not appear to improve crack width control, but they ensure that a more ductile type of bond failure could occur. The aim of confinement by means of transverse compression or transverse reinforcement is to prevent failure along the potential splitting crack and to enforce, if necessary a slip mode of failure (shear failure) as shown in Figure 3.10a, which is associated with the maximum attainable bond strength.

C- Repeated and Cyclic Reversed loading

Repeated and cyclic reversed loading affect bond strength due to the gradual diminish of frictional resistance. Load can be subdivided into monotonic and cyclic loading. Cyclic loading is divided into two categories. The first is "low-cycle" loading, (which contains few cycles but having large ranges of bond stress), such as seismic and high wind loading. The second is the "high-cycle" or fatigue loading, (in which its load history contains many cycles, but at low bond stress range), such as off-shore structures, bridges, and members supporting vibrating machinery.

Failure modes of monotonic loading are either direct pull out of bar when ample confinement is provided, or splitting of concrete cover when the cover or confinement is insufficient to obtain a pull out failure. For low-cycle loading, modes of failure are similar to those under monotonic loading, with cracking in both directions.

Under cyclic loading most of the bond stresses are transferred mechanically by bearing of the bar deformations against the surrounding concrete. Bond stress-slip response of bar loaded by low cycle loads is shown in Figure 3.12. The initial part follow the monotonic
envelope. If the load is reversed after bond stress exceeds about one half of its maximum value, a significant permanent slip will remain when the bar is unloaded, because of inelastic deformations in the vicinity of the ribs, microcracking in concrete, and release of shrinkage strains. If the load is applied in the opposite direction, the bar must experience some rigid body motion before beginning to bear in the opposite direction. As cycling progresses, the concrete in front of the lugs is crushed and sheared. When the load is reversed large slip occurs.

During earthquakes, alternate yielding in tension and compression at critical sections, such as column-beam interface, can occur. The gradual loss of bond can result in penetration of yielding into the anchorage zone, drastically diminishing the effective development length available to absorb the yield strength of the bar.

D- Position of Bar with Respect to Placing of the surrounding Concrete

The load-bond slip relationship for deformed bar is primarily affected by the behaviour of concrete immediately in the front of the ribs. It is to be expected that the top bars of a beam will have poorer bond qualities than bottom bars, since the water and air gain will be greater under the top bars, causing soft and spongy layer of concrete under the ribs. The ACI code, 1989, requires 30% excess development length for top-cast deformed bars to account for this phenomenon.

3.1.4- Development Length

Development length is the shortest embedment length required to develop the yield force of the bar, sometimes with some reserve 1.25 $f_y$ to avoid brittle failure of development length. This length ($L_d$) is necessary for both sides of peak stress point.
3.1.4.1- Development Length in Tension

From Equation (1), the theoretical development length, \( L_d \), is:

\[
L_d = \frac{f_y d_b}{4 u_{av}} \tag{3.6}
\]

where \( u_{av} \) is the average value of bond stress at bond failure of beam test. In the ACI Code, 1989, (Sec. 12.2.2) the development length \( L_d \) is computed as the product of the basic development length \( L_{db} \) and the applicable modification factors of Sections 12.2.3 through 12.2.5 of the Code, to reflect the influence of cover thickness, spacing, transverse reinforcement, casting position, type of aggregate, and epoxy coating. In general \( L_d \) shall not be less than 12 in.

The basic development length for straight bars, \( L_{db} \), is:

For No. 11 bars and smaller and deformed wires
\[
0.02 A_b f_y \frac{0.02 A_b f_y}{\sqrt{f' c}} \quad \text{mm, MPa}
\]

For No. 14 bars
\[
25 f_y \frac{25 f_y}{\sqrt{f' c}} \quad \text{mm, MPa}
\]

For No. 18 bars
\[
35 f_y \frac{35 f_y}{\sqrt{f' c}} \quad \text{mm, MPa}
\]

Hence reserve strength in the development length above \( f_y \) is essential to maintain member ductility, the ACI Code values aim at making this extra strength about 20% of \( f_y \).

The CEB/FIP International Recommendation, 1978, evaluates the basic development length using the following equation, which is based on the equilibrium of forces at yield stress of the rebar.

\[
L_d = \frac{f_y \varphi}{4 f_{bd}} \tag{3.7}
\]
flexural strength). This require an embedment length in additional to the transfer length at the end of the member. This embedment length is known by the flexural bond length \( (L_{\text{tb}}) \). Figure 3.17 shows the end portion of a pretensioned beam loaded to cracking near the end. The solid line in Figure 3.17 indicates the stress in the tendons under normal loads before cracking, having a transfer length \( L_t \). Under increased loading, when a crack occurs at \( C \), the stress in the tendon is raised to a maximum stress \( f_{\text{ps}} \) as shown by dotted line. Thus there is a bond stress developed from \( C \) to \( B \). If \( B \) reaches past \( A \), that is, if the flexural bond length overlaps the transfer length, then the tendon could be pulled through the concrete. Summation of the flexure and the transfer lengths is typically defined as the development length \( (L_d) \) of the prestressing reinforcements.

Bond characteristics of prestressing reinforcements are influenced by a large number of parameters. From which the following are listed:

a. Type, e.g. wire, strand, or bars.

b. Size, e.g. diameter.

c. Prestressing level.

d. Surface condition: clean, smooth, deformed, oiled, rusted.

e. Concrete strength.

f. Type of loading, e.g., static, repeated, impact.

g. Releasing methodology, e.g., gradual, sudden, flame cutting, sawing.

h. Presence of confining reinforcement, e.g., helix, stirrups, or non.

i. Time-dependent effect.

j. Consolidation and consistency of the concrete.
restrain concrete splitting. Modification of this test, in Figure 3.13, called tensile pullout specimen to eliminate the compression on the concrete.

3.1.5.2 Bond Beam Tests

Beam tests are considered more reliable because the influence of flexural tension crack is included. Figure 3.14 and Figure 3.15 show two different bond test beams in which the major consideration in each is to remove reaction constraint that might confine the concrete and increase splitting resistance.

3.1.5.3 Cantilever Tests

To reduce specimen size and expense, partial beam specimens have been used as sketched in Figure 3.16. The reaction may bear against the end, may be spread, or the bar may be shielded. The advantage of this test is the simplicity, and the disadvantages is unrealistically low ratio of shear to bond.
3.2 BOND BEHAVIOR OF PRESTRESSING STEEL REINFORCEMENT

3.2.1 Introduction

Prestressing is well known to be the most efficient technique which utilize the full advantage of the properties of concrete and the prestressing reinforcements. One of the most popular system of prestressing is pretensioning, where the pretensioning reinforcements are stressed using temporarily anchorage before casting of the concrete. When the concrete reaches its specified strength, the prestressing reinforcements are released and the concrete is subjected to compression. A critical aspect of pretensioning is the mechanism of transferring the forces from the prestressing reinforcements to the concrete. Transfer of forces occurs over a distance known by the transfer length \(L_t\) which is highly dependent on the bond characteristics between the prestressing reinforcement and the concrete. However it was reported that the transfer mechanism depends also on the mechanical interlock and friction mechanism which is function of the bond characteristics of the prestressing reinforcements. Transfer length does the same function of mechanical anchorage in post-tensioned members.

Prestressing reinforcement in prestressed concrete members serves a dual function. Part of the available tensile strength of reinforcement is used first to establish a compressive prestress in the concrete. Secondly, if a member is loaded beyond cracking, all or part of the steel tensile strength may be utilized to assist the concrete in resisting the externally applied bending moment. Therefore, to achieve the ultimate strength of a prestressing member, strands must be sufficiently embedded to develop their ultimate strength at the critical section of the member (the section desired to develop the ultimate
k. Concrete coverage.

3.2.2 Nature of Bond and Mechanism of Transfer

In the end zones of beams prestressed by means of bonded pretensioned strands, we have to deal with a special case of the problem of bond between reinforcement and concrete. The phenomena occurring in these end zones may be regarded as reverse of those occurring usually in reinforced concrete. In reinforced concrete the reinforcement and the surrounding concrete are both in tension; in the case considered here, once the strands are released from their temporary anchorage used in tensioning, they would return to zero stress if they were not bonded to the concrete. The tendency then for both concrete and steel is that the stresses increase algebraically (i.e. either compression is increased or tension is reduced). In case of reinforced concrete we may say that we are pulling on the concrete by means of reinforcement. In the case discussed here we may say that we are pushing on the concrete by means of the tendons. In other words in the case of reinforced concrete we dealing with bond in tension whereas in the case of prestressed concrete we are dealing with bond in compression.

This difference has important effects. In the case of bond in tension the diameter of the bar decreases (Poisson’s effect) and therefore the transverse pressure of the concrete on the bar decreases as the tension in the bar increases, moreover the layer of concrete surrounding the bar is in tension and will therefore fail at a low value of the bond stress. In the case of bond in compression, Poisson’s effect leads to a swelling of the strand and the transverse pressure between the concrete and the strand therefore increases at the same time as the magnitude of the force transmitted by the strand to the
concrete. Moreover the layer of concrete surrounding the strand is in compression and can therefore withstand much higher stresses and hence strains than in the previous case.

Bond in pretensioned concrete members is typically classified into two distinct types: transfer bond and flexural bond. Transfer bond transfers the prestressing tensile stresses in the reinforcements into compression stresses in the concrete within the transfer length. Flexural bond are induced due to the applied external loads on member. After cracking, the induced stress in additional to the effective prestress develops flexural bond stress between the concrete and the prestressing reinforcements.

3.2.2.1 Transfer Bond

Transfer bond due to prestressing are located at the two ends of the member after the force in the tensioned strand is completely transferred to the concrete member. The prestress transfer length depends mainly on the amount of prestressing, surface condition of the strands, strength of the concrete, and the method of release. The three factors which contribute to the bond behavior are adhesion, friction, and mechanical resistance between concrete and prestressing reinforcement. Adhesion, exist only in the case of no slip has taken place between concrete and reinforcement. In the case of no slip, the reduction in the tensile strain in the prestressing reinforcement at any point after release is equal to the increase in compressive strain in the concrete at the same point (Janney, 1954). Experimental results indicate that this condition exist only in the constant strain zone, that is in the central region of specimens. The reduction in the steel strain, within the transfer length, is normally greater than the corresponding increase in concrete strain. The free end of the member represent the case where the reduction in prestressing
reinforcement strain is maximum and the corresponding increase in the concrete strain is zero. At this location, slip will occur and consequently adhesion cannot contribute in transferring the prestress force. Therefore friction is considered as the principal mechanism for the stress transfer from pretensioned reinforcement to concrete. As the tension in the reinforcement is released, the diameter tends to increase due to Poisson’s effect, as shown in Figure 3.18, thus producing high radial pressure against the concrete and then higher friction resistance. The swelling of the prestressing reinforcements after release is known by Hoyer effect, which in turn produces higher bond resistance also due to wedge action. The mechanical resistance probably contributes the least to prestress transfer in the case of individual smooth wires, but it may be a significant factor in the case of strand.

3.2.2.2 Flexural Bond

Flexural bond of significant magnitude exists only after the concrete beam has been loaded beyond cracking. When the concrete cracks, the bond stresses in the immediate vicinity of the cracks increases to some limiting stress, causing slip over a small length of the reinforcement adjacent to the cracks, and consequently considerable reduction of the bond stress near the cracks reduced to a low level. With continued increase of the load, the high bond stress progress as a wave from the original cracks towards the end of the transfer length.

If the bond stress wave reaches the prestress transfer zone, the increase of reinforcement stress result in decrease of the reinforcement diameter, which in turn reduces the friction bond resistance, and precipitates a general bond slip. Following the
loss of friction resistance, mechanical resistance is the only factor which could contribute to the bond between concrete and reinforcement. If the beam is prestressed by clean smooth wire, the friction resistance will be small, and the beam will collapse right after the general slippage. If the wire is rusted, the resistance to slippage will be greater. If the beam is prestressed by strand, the helical shape of the individual wires will provide sufficient mechanical resistance that the beam can support additional load even after slip of the strand at the beam ends.

3.2.3 Current Design Criteria

The American Concrete Institute (ACI, 1989) and the American Association of State Highway and Transportation Officials (AASHTO, 1989) provide equations which could be used to predict the transfer and development lengths of steel strand as follows:

\[
L_d = 0.0483 f_{pe} d_b + 0.145 (f_{ps} - f_{pe}) d_b
\]  

(3.8)

where 

L_d \quad \text{Development length (mm)}

f_{pe} \quad \text{Effective prestress, after allowance of all prestress losses, (MPa)}

d_b \quad \text{Nominal diameter of strand (mm)}

f_{ps} \quad \text{Stress in prestress reinforcement at nominal strength (MPa)}

The first part of Eq. (3.8), 0.0483 f_{pe} d_b, represents the transfer length while the second part represents the flexural bond length as shown in Figure 3.19. The transfer length is further simplified by both codes to be 50d_b. Eq. (3.8) implies a bond strength of 4.0 MPa within the transfer length and then 1.34 MPa within the flexural bond length, where the bond stresses is calculated on the bases of the nominal circumference of the strand (i.e., \pi d_b) and the actual cross section area. The ACI and AASHTO codes do not require that
flexural bond stresses be checked, but for pretensioned strand it is required that the full development length given by Eq. (3.8) be provided beyond the critical bending section. Investigation may be limited to those cross sections nearest each end of the member that are required to develop their full flexural strength under the specified factored loads. In the event that sheathed tendons are used near the ends of a span, no prestress force will be transferred from the blanketened strand until the end of the sheathing is reached. From that point inward toward the center of the span, transfer of prestress by bond is less than normally effective, because of the lack of vertical compression from the beam reaction and because flexural tensile stresses may exist in the concrete. The ACI Code requires that the development length given by Eq. (3.8) be doubled for sheathed tendons. The commentary of this section reports that tests conducted by Rabbat, Kaar, and Bruce (1979) indicated that in pretensioned members designed for zero tension in the concrete under service load conditions, the development length for debonded strands need not be doubled. Eq. (3.8) was derived based on the research conducted in the 1950’s, using conventional steel strand designated as Grade 250, with a specified guaranteed ultimate tensile strength of 250 ksi (1700 MPa). Currently, strand designated Grade 270, with a guaranteed ultimate strength of 270 ksi (1860 MPa), low relaxation, and larger cross-sectional area for the same diameter, has replaced Grade 250. These improvements could affect the transfer and development lengths due to higher prestress level can be achieved.

The “format” of the CEB-FIP MODEL CODE 1990 clauses on the anchorage of pretensioned tendons is similar to that for reinforcing bars. The clauses are primarily
based on experimental data from tests with 7-wire strand. In MC90 two bond situations are distinguished: “push-in”, which is related to the transmission length, and “pull-out” that is connected to the anchorage length.

The following steps are taken to estimate the transfer and development lengths using CEB-FIP criteria:

a. Estimate the design bond strength (\( \eta_{bpd} \)): a lower bound value for the average bond stress in the pull-out situation.

b. Calculate the basic anchorage length (\( l_{bp} \)): the embedment length that is required to carry off the failure load of a non-pretensioned strand.

c. Calculate the transmission length (\( l_{bpt} \)): the embedment length along which the effective prestress is developed.

d. Calculate the anchorage length (\( l_{bpd} \)): the embedment length that is required to carry off the design force in a pretensioned tendon.

\[
f_{bpd} = \eta_{p1} \eta_{p2} f_{cdt}
\]

where: \( \eta_{p1} = 1.2 \) for 7-wire strand.

\( \eta_{p1} = 1.4 \) for indented and crimped wire.

\( \eta_{p2} = 1.0 \) or 0.7 according to the position during casting, for bottom and top reinforcement, respectively.

\( f_{cdt} = f_{ctk0.05} / \gamma_m \) with \( \gamma_m = 1.5 \)

\( f_{ctk0.05} = 0.7 f_{ctm} \) (MPa)
\[ f_{cm} = \text{mean tensile strength} = 0.3 \,(f_{ck})^{2/3} \] and \( f_{ck} \) is the concrete compressive strength (MPa).

\[ l_{bp} = \frac{A_p f_{bd}}{\pi \phi_p f_{bd}} \]  \hspace{1cm} (3.10)

where: \( \phi_p \) is the diameter of prestressing tendon (mm)

\( A_p \) is cross sectional area of the tendon (mm\(^2\))

\( f_{bd} \) is the ultimate tensile strength of the tendon (MPa).

\[ l_{bpt} = \frac{\alpha_8 \alpha_9 \alpha_{10} l_{bp} \sigma_{pi}}{f_{bd}} \]  \hspace{1cm} (3.11)

where: \( \alpha_8 = 1.0 \) or 1.25 for gradual and sudden release respectively.

\( \alpha_9 \) - effect to be verified

\( \alpha_9 = 1.0 \) for moment and shear

\( \alpha_9 = 0.5 \) for tensile stresses in the transmission zone

\( \alpha_{10} \) - effect of bond situation

\( \alpha_{10} = 0.5 \) or 0.7 for strand and deformed wire respectively.

\( \sigma_{pi} \) = tendon stress immediately after release (MPa).

The ratio between the average bond stress along the basic anchorage length and the transmission length is defined by the factor \( \alpha_{10} \) which takes into account the influence of the bond situation: pull-out versus push-in.

\[ l_{bpd} = l_{bpt} + l_{bp} \left( \frac{\sigma_{pd} - \sigma_{p\infty}}{f_{pd}} \right) \]  \hspace{1cm} (3.12)

where \( \sigma_{pd} \) is the design ultimate tensile stress of the tendon (MPa).
CHAPTER 4
EXPERIMENTAL PROGRAM

4.1 INTRODUCTION

This experimental work is part of a comprehensive program, currently in progress in University of Manitoba, to examine the use of FRP as reinforcements and prestressing of concrete structures. The main objective of this research is to study the bond characteristics of two types of carbon fiber reinforced polymer, CFRP, bars and strands. The first type of CFRP is the Leadline bars and the second type is Carbon Fiber Composite Cables, CFCC strands, commercially known as Tokyo Rope. Both Leadline and CFCC are typically used in pretensioned and/or post-tensioning of concrete structures. The scope of the experimental program included 52 prestressed concrete specimens fabricated and tested at the structures laboratory of University of Manitoba. Two different types of pretensioned prestressed specimens were used in this investigation. The first type is the classical beam specimens. The second type was based on a new configuration for a prism specimens proposed for this investigation. The beam specimens, tested in flexural, were used to determine the transfer and the development lengths. The prism specimens were used to determine the transfer length and bond strength. General layout of the prestressing and casting of the two types of specimens are shown in Figures 4.1 and 4.2 respectively. View of the prestressing and casting of the beam and prism
specimens are shown in Figures 4.3 and 4.4 respectively. It should be mentioned that the beams were cast on their side to allow monitoring the concrete strains at the level of the tendons before release.

The variables considered in this experimental program are:

1- Type of prestressing reinforcement (steel strands, Leadline bars, and CFCC strands).
2- Diameter of the tendons (9.6, 12.7 mm for steel, 8 mm for Leadline, and 10.5, 12.5, and 15.2 mm 7-wire CFCC strand).
3- Prestressing levels were varied from 58 to 80 percent of the ultimate guaranteed strength of the tendons reported by the manufacturer.
4- Embedment length.
5- Time effect on the transfer length.
6- Concrete strength.
7- Confinement by shear reinforcement.

This chapter describes in detail the specimens configuration, instrumentation, jacking setup, test setup and test procedure used in this experimental program. This chapter also presents the properties of the prestressing and concrete materials used in this study.

4.2 TEST SPECIMENS

Designation of the tested specimens consists of three symbols as follows:
1) The first letter B or P designating the type of the test specimens. B for beam specimen and P for prism specimen.

3) The number 1, 2, 3, …designating the specimen number.

Tables 4.1, 4.2, and 4.3 provide detailed information for all tested specimens in this program. The tables, provided for each type of reinforcement, were according to the diameter of the bars or strands. Given in these tables are the prestressing force, the dimensions of the specimens, the embedment length, the concrete cover, load configuration, and the shear reinforcement. The dimension of each specimen are given in Table 4.1 through 4.3 in terms of “b, h, and L”, where (b) is the width of the beam or prism, (h) is the total depth of the specimen, and (L) is the total length of the specimen.

4.2.1 Beam Specimen

It is generally believed that beam tests could realistically be used to simulate the stress conditions of prestressed concrete member subjected to bending. A total of forty pretensioned prestressed beams, with the configurations and load locations shown in Figures 4.5 through 4.13, were tested. Size of the specimens prestressed by CFCC strands were dictated by the available pre-cut lengths provided by the manufacturer.

The three types of beam specimens used in this program are A, B, and C. In type A specimens, the location of the load was varied according to Figures 4.5 through 4.7. In Figure 4.5, the specimens were tested using one-point load at the mid-span. The load was shifted toward one of the ends of the beam in Figure 4.6 to determine the embedment length. In Figure 4.7, the specimens were loaded two-point loads with two symmetrical shear spans.
For type B specimens, the variation of the embedment length was achieved by changing the debonding length at mid-span of each specimen as shown in Figures 4.8. Debonding of the strands was achieved by placing a plastic tube around the tendon with variable lengths to determine the embedment length ($L_e$). For type C specimens, the beams in configuration C1, C2, C3 and C4 were long enough to test two parts of the beam independently using one static load, as shown in Figures 4.9 and 4.12. This been selected to provide two sets of data for each beam. The cross section dimensions were designed to induce high compressive strain at the bottom fibers of the beam sufficient enough to allow precise measurements of the transfer length. In beams without shear reinforcement, the beam width was designed to avoid premature failure due to shear.

The criteria used in the design of the beam specimens can be summarized as follows:

1) Tensile stresses at the top of the beam after release of prestressing are less than the concrete tensile strength at transfer.

2) The selected concrete cover is four times the tendon diameter measured from the center of the tendon.

3) Suitable beams dimensions and shear reinforcement are selected to avoid premature failure due to shear.

To eliminate the effect of the compression field at the vicinity of the supports, 50 mm or 100 mm length of the tendon at the two ends of the beams was debonded as shown in Figures 4.5 through 4.13. Beams BS5, BS6, BL3, BL4, BT1, and BT2 were monitored.
for one year after release to determine the effect of creep and shrinkage of concrete on the transfer length of CFRP tendons in comparison to steel strand.

4.2.2 Prism Specimen

The configuration of the prism specimen before release is schematically shown in Figures 4.2. View of prestressing and casting setup of two prism specimens is shown in Figure 4.4. Figure 4.14 shows the prism configuration at testing. Eight concrete prism specimens pretensioned with steel strands and four prisms pretensioned with Leadline bars were tested. The prism specimen were square or rectangular cross sections with one concentric tendon. Different cross-section dimensions were used to achieve approximately equal concrete stress level after transfer for the different types and sizes of the prestressing bars and strands used in this investigation. Each prism specimen consisted of two concrete parts connected together by the hollow jack and hollow load cell. The jack and the load cell are positioned before casting as part of the specimen configuration. The length of each part of the prism specimens was designed to be more than the expected transfer length. Tables 4.1 and 4.2 include the dimensions of prism specimens, tendon diameter, prestressing force, and embedment length. A length of 50 mm of tendons was debonded at the loaded ends of the prism. A Teflon sheet was used between the bearing plate and concrete to permit deformations in the transverse direction during application of the load.

The typical pull-out test, used by many researcher to study and compare the bond properties of reinforcements, usually gives higher bond strength than the classical beam test. This difference in results could be attributed to the following two main reasons:
1- The concrete surrounding the reinforcements in the pull-out test is subjected to compressive stresses, while the concrete in the typical reinforced concrete beam is subjected to tensile stresses and possibly cracked.

2- The friction forces between the loading steel plate and concrete could prevent lateral deformation of concrete and therefore inclined forces could be induced. These inclined resultants could apply radial confining forces on the reinforcements near the loaded end and increase the pull-out resistance. This phenomenon is known by “wedging effect”.

Unlike reinforced concrete beams, when prestressed concrete beams subjected to flexure, the majority of development length, at the two ends of the beam, remain free from flexural shear cracks due to prestressing. Considering this fact and by elimination of wedging effect by using Teflon sheets and debonding part of tendon at the loaded end, the proposed prism test can be considered an adequate model to determine the transfer length and the bond characteristics of prestressing tendons. The proposed test has the advantage of easiness, speed, economical, and minimize the need for expensive testing equipments.

4.3 MATERIALS

4.3.1 Concrete

The concrete was provided by a local ready mix concrete supplier. Superplasticizer was added to the mix to increase the slump from 20 mm to 100 mm. For each batch, the strength of the concrete was monitored using Twelve 150 mm x 300 mm cylinders which were moistured cured beside the specimen in the same room temperature.
for four days after casting. The concrete mix designed for a normal strength of 35 MPa at 28 days had the following proportions:

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Type 10</td>
<td>325 kg/m³</td>
</tr>
<tr>
<td>Maximum Course Aggregate 14 mm</td>
<td>1164 kg/m³</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>838 kg/m³</td>
</tr>
<tr>
<td>Water</td>
<td>116 litre/m³</td>
</tr>
<tr>
<td>Pulverized Flyash</td>
<td>94 kg/m³</td>
</tr>
<tr>
<td>Water Reducing Agent (322N)</td>
<td>1.064 litre/m³</td>
</tr>
</tbody>
</table>

The concrete strengths at the time of release and testing were determined by testing three cylinders each time. Splitting test of three cylinders was conducted to obtain the splitting tensile strength of concrete at the time of release and testing. The concrete strengths for all specimens at the time of release are given in tables 5.1 through 5.3. The concrete strengths for all specimens at the time of testing are given in tables 7.1 through 7.3. Figure 4.15 shows a typical stress-strain relationship measured for typical concrete cylinder test.

4.3.2 Prestressing Tendons

Two different CFRP tendons were considered in this study. The first is 8 mm diameter Leadline bars, manufactured by Mitsubishi Chemicals, Japan. Leadline bar consists of a black single unidirectional CFRP with 65% fiber content in an epoxy resin matrix. The surface texture of the rod is smooth with an impressed pattern. A reusable wedge-type grip system was used in tensioning the bars. The effective cross section area
of the bar is 46.1 mm². The guaranteed tensile strength and tensile modules, reported by the manufacturer, are 1970 MPa and 147 GPa respectively.

To obtain the actual mechanical properties, six tension tests were conducted. Three tests using epoxy anchorage and three tests using the steel anchorage supplied by the manufacturer. The measured average tensile strength was 2170 MPa using steel anchorage and 2320 MPa using the epoxy anchorage type. In all specimens the failure occurred at the anchorage due to the induced lateral stresses caused by the wedges. Previous tests (Abdelrahman, 1995) showed that the Leadline could have tensile strength as high as 2960 MPa using concrete anchorages. The Leadline anchorage used in this investigation is shown in Figure 4.16. Figure 4.17 shows a typical measured tensile stress-strain relationship of Leadline using epoxy anchorage. The behavior is linearly elastic up to failure. The modules of elasticity obtained from tests varies between 147 to 190 GPa. It should be mentioned that the Leadline was shipped in a continuos coil form and the required lengths are cut as required.

The second type of CFRP used in this study is Carbon Fiber Composite Cables CFCC, commercially known as Tokyo Rope produced by ITOCHU Corporation, Japan. CFCC is a black-seven stranded cable, twisted in a manner similar to seven-wire steel strand. Each of the seven strand consist of about 64% carbon fiber in an epoxy matrix and is individually wrapped in its own filament-wound sheath, producing semi rough surface texture. The grip system consists of reusable steel wedge and collar superimposed upon a diecast (bonded) sheath affixed by the manufacturer. The length of each die-casts ranges from 170 mm to 300 mm according to the cable diameter. The precut lengths of cable are
coiled for shipping. The mechanical properties, reported by the manufacturer, for the three sizes of CFCC used in this investigation are given in Table 4.4. Figure 4.18 shows the typical stress-strain relationship measured for a tension test of 15.2 mm diameter CFCC. The measured ultimate tensile strength, modules of elasticity, and ultimate strain for CFCC strands are summarized in Table 4.4. Unlike Leadline, the failure of CFCC strands in tension test occurred away from the anchorage due to the enhancement of stress distribution at the anchorage zone by using the die-cast at the anchorage zone. The CFCC with its die-cast and the anchorage devices are shown in Figure 4.16.

This investigation included also steel strands for prestressing the beam and prism specimens. The two sizes used in this study are 9.6 and 12.7 mm diameter 7-wire steel strand. The nominal tensile strength strength is 1860 MPa (270K), the nominal modules of elasticity is 200 GPa, and the cross section areas are 55 and 99 mm$^2$ for 9.6 and 12.7 mm diameter respectively. Figure 4.19 shows the results of a tension test for 12.7 mm 7-wire diameter strand.

4.4 FABRICATION OF SPECIMENS

The test specimens were fabricated in the Structural Engineering Laboratory of the university of Manitoba. A 5 meters long stressing bed with room for stressing two lines of beams was used as shown in 4.1. Two reaction abutments anchored to strong floor were used to support two hydraulic jacks, each 500 kN capacity, with locking nuts to maintain the prestressing force after jacking. The fabrication process consists of four stages: building the forms, stressing the tendon, casting of concrete, and release of prestressing tendons.
After assembling the form to the desired sizes, the stirrups were welded to the steel reinforcement bars and aligned into the forms. Prestressing tendons were placed in the prestressing bed. Stressing was accomplished using the two hydraulic jacks (500 kN capacity each) and mechanical pumps. The force in the tendons was monitored using load cells at the jacking end. Strain gauges, glued on the surface of the tendons, were used to monitor the tendons strain during the jacking process using a strain indicator. Figure 4.20 shows the overall jacking assembly. The stressing operation proceeded as follows:

1) The tendons were pulled to the desired jacking force, $P_j$.
2) The force in the hydraulic jacks was released after locking the end nuts.
3) The second step was repeated until the desired jacking load was achieved to overcome the losses due to seating of anchorage.
4) The force in the tendons was recorded immediately before casting of concrete.

The concrete was vibrated using pin vibrator, lightly trawled, and covered with polyethylene sheet for four days. Surface of the specimens were kept wet to prevent any shrinkage cracks which could affect the behavior.

4.5 INSTRUMENTATION

Concrete Strains: After four days of moist curing, the sides of the forms were removed. Demec points were glued along the surface of the specimens using 5-minute epoxy super glue. The specimens were kept for another two days, before releasing the prestress, for complete hardening of the epoxy. The gauge points were set at 51 mm (2 inches) intervals. A demec gauge of 0.00126 mm (0.00005 in.) subdivision was used to measure the distance between gauge points. Two sets of demec readings were taken before and
after release of the prestressing. Demec readings were recorded after release up to the
time of testing.

**Reinforcement Strains:** Electrical resistance strain gauges were used to measure the
strains in the prestressing bars and strands during jacking, before release, after release,
and during testing. The strain gauges had a length of 5 mm gauge length and a resistance
of 120 ohms. The gauges were applied axially parallel to the length of the bar in case of
Leadline bars and parallel to the axis of one of the wires in case of the 7-wire CFCC
strands. In case of CFCC strands, the wrapping synthetic yarn was removed carefully to
attach the strain gauges directly on the FRP core. Each specimen was instrumented by 5
or 10 electrical resistance strain gauges. Strain indicator was used to monitor the strains.

**Slip Monitoring:** Two linear variable differential transducers (LVDTs) and two dial
gauges were attached to the tendons at each end to measure any end slip of the tendons
after transfer as shown in Figures 4.21 and 4.22. The LVDTs were connected directly to
the Data acquisition for continuous recording during release of prestressed tendons.

**Jacking Load:** A gradual release of the jacking load, used for all specimens, was
controlled by gradual decrease of the hydraulic jacks pressure. The jacking force in each
tendon during the release process was monitored using two load cells, (333 kN capacity
each), attached to the live ends of the tendons and connected to the Data Acquisition
system for automatic recording.

**Transfer Length:** The following steps were used to monitor the transfer length during
transfer of the load to the concrete.

1) Two sets of initial readings of the demec station were taken before release.
2) Strain gauges, dial gauges, and LVDTs readings are recorded before release.

3) The jacking force was reduced by 50% and the remaining force was secured by locking the nuts of the jacks followed by reading of all instrumentation.

4) Release the prestressing force completely.

5) Cutting the tendons at the specimen's ends and separate the specimen from the form to eliminate the effect of friction forces between the specimen and the bottom surface of the form.

6) Record the reading of all instrumentation including two sets of demec readings.

Specimens BS5, BS6, BL3, BL4, BT1, and BT2 were kept for one year after release. Both the concrete and reinforcement strains were measured periodically before testing to determine the effect of creep and shrinkage of concrete on the transfer length of Leadline bars, CFCC strands, and steel strands.

4.6 TEST PROCEDURE

4.6.1 Beam Specimen

Before testing, the transfer length at each end of the beam was determined using both demec and strain gauges. A 1000 kN MTS servohydraulic machine, shown in Figures 4.23 and 4.24, was used to apply the load. The load was applied at the designated location through 60 mm wide steel plate to distribute the bearing stresses. Roller supports, 100 mm wide, were used at both ends. Plaster pads were used at the loading location and at the bearing to provide even distribution of the load and the reactions. The beams were painted with white wash to facilitate locating of the cracks. The load was applied in a stroke control with 0.0025 mm/sec rate before cracking, 0.005 mm/sec rate after
cracking, and 0.01 mm/sec rate after yielding or slip. Duration of a typical test varies between four to six hours.

With the maximum moment occurring at the load point, the two types of failure occurred were bond failure or flexural failure. Specimens were adequately designed to avoid premature failure due to shear. In case of bond failure, the location of the load was adjusted to increase the embedment length for the following specimens. In case of flexural failure, the location of the load was also adjusted to decrease the embedment length for the following test. This scheme was used until the accurate development length of a particular tendon was achieved.

The vertical applied load was measured by the MTS load cell (1000 kN capacity) which is attached to the moving head of the machine. Reading of the load cell was monitored by the control panel with reading accuracy of 0.01 kN. The control panel was connected to the Data Acquisition for automatic continuous recording of the load and stroke.

Linear Motion Transducers (LMT) was used to measure the vertical deflection at mid-span for symmetric loading and at the location of the load point for unsymmetric loading. The LMT was connected directly to the Data Acquisition for continuous recording during the test. The LMT has an accuracy of 0.01 mm. The deflection was also measured by the machine stroke readings.

The electrical resistance stain gauges were connected to strain indicator. Strain readings were recorded simultaneously with increase of the applied load. Strain readings were recorded at 2 kN load increment before cracking and at 1 kN increment after
cracking. Two Linear Variable Differential Transducers (LVDTs) were fixed at the two ends of the beam to measure the movement of the tendon relative to concrete, as shown in Figure 4.25. The LVDTs were connected directly to the Data Acquisition for continuous recording.

4.6.2 Prism Specimen

After release, the prestressing force transfers to the two concrete parts of the prism. An equal compression load is resisted and maintained by the hollow jack and load cell at the middle of the specimen as shown in Figures 4.2, 4.4. and 4.14. At this stage, the transfer length, at the two extreme end of the specimen, could be estimated using the demec gauge measurements and electrical strain gauge readings.

The second stage of the test is to apply force gradually by the internal hydraulic jack to separate the two concrete parts. Roller supports were used as shown in Figure 4.14 to minimize the friction forces during the second stage of the test. The load measured by the enclosed hollow load cell was monitored continuously using the Data Acquisition System. Two dial gauges were attached to both ends of the prism to monitor the tendon slip. Three modes of failure typically occur. Tendon slippage occurred in the case of insufficient embedment length. Tendon ruptured when the embedment length was greater than the required development length. Concrete split when the concrete cover was not enough to develop the required confinement.
Table 4.1 Details and Parameters of Specimens Prestressed with Steel Strands

<table>
<thead>
<tr>
<th>Strand diameter $d_b$ (mm)</th>
<th>Specimen designation</th>
<th>Jacking force $P_j$ (kN)</th>
<th>Jacking stress level $f_{pl} / f_{pu}$</th>
<th>Specimen dimensions $b \times h \times L$ (mm)</th>
<th>Concrete cover $d'$ (mm)</th>
<th>Embedment length $L_e$ (mm)</th>
<th>Configuration type</th>
<th>Shear reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BS1</td>
<td></td>
<td></td>
<td>100x250x1700</td>
<td>40</td>
<td>800</td>
<td>A1</td>
<td>2 br. $\phi$ 6 @ 80</td>
</tr>
<tr>
<td></td>
<td>BS2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>550</td>
<td>A3</td>
<td></td>
</tr>
<tr>
<td>9.6</td>
<td>PS1</td>
<td>61</td>
<td>0.6</td>
<td>85x90x1884</td>
<td>42.5</td>
<td>750</td>
<td>prism</td>
<td>$\phi$ 6 @ 80 mm</td>
</tr>
<tr>
<td></td>
<td>PS2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>550</td>
<td>prism</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PS3</td>
<td></td>
<td></td>
<td>60x90x1884</td>
<td>30</td>
<td>550</td>
<td>prism</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>PS4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>530</td>
<td>prism</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BS3</td>
<td>110</td>
<td>0.6</td>
<td>100x250x1700</td>
<td>50</td>
<td>800</td>
<td>A1</td>
<td>2 br. $\phi$ 6 @ 80</td>
</tr>
<tr>
<td></td>
<td>BS4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>800</td>
<td>A1</td>
<td></td>
</tr>
<tr>
<td>12.7</td>
<td>BS5</td>
<td>122.4</td>
<td>0.66</td>
<td>100x250x1900</td>
<td>50</td>
<td>900</td>
<td>A1</td>
<td>2 br. $\phi$ 6 @ 80</td>
</tr>
<tr>
<td></td>
<td>BS6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>900</td>
<td>A1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PS5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>750</td>
<td>prism</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PS6</td>
<td>110</td>
<td>0.6</td>
<td>95x95x1884</td>
<td>47.5</td>
<td>750</td>
<td>prism</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>PS7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>750</td>
<td>prism</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PS8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>750</td>
<td>prism</td>
<td></td>
</tr>
<tr>
<td>Strand diameter $d_b$ (mm)</td>
<td>Specimen designation</td>
<td>Jacking force $P_j$ (kN)</td>
<td>Jacking stress level $f_{pl} / f_{pu}$</td>
<td>Specimen dimensions $b \times h \times L$ (mm)</td>
<td>Concrete cover $d'$ (mm)</td>
<td>Embedment length $L_e$ (mm)</td>
<td>Configuration type</td>
<td>Shear reinforcement</td>
</tr>
<tr>
<td>---------------------------</td>
<td>----------------------</td>
<td>-------------------------</td>
<td>----------------------------------------</td>
<td>----------------------------------------------</td>
<td>--------------------------</td>
<td>-----------------------</td>
<td>---------------------</td>
<td>---------------------</td>
</tr>
<tr>
<td>BL1</td>
<td>55</td>
<td>0.6</td>
<td>80x200x1700</td>
<td>35</td>
<td>800</td>
<td>A1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>800</td>
<td>A1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL3</td>
<td>65</td>
<td>0.7</td>
<td>80x200x1700</td>
<td>35</td>
<td>800</td>
<td>A1</td>
<td></td>
<td>2 br. $\phi$ 6 @ 80</td>
</tr>
<tr>
<td>BL4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>800</td>
<td>A1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL5</td>
<td></td>
<td></td>
<td>80x250x2500</td>
<td>35</td>
<td>1200</td>
<td>A1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL6</td>
<td>69</td>
<td>0.74</td>
<td>80x250x1700</td>
<td>35</td>
<td>800</td>
<td>A1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL7</td>
<td>80x250x2100</td>
<td>35</td>
<td>1000</td>
<td>A1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>800</td>
<td>A1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL9</td>
<td>58</td>
<td>0.62</td>
<td>150x250x4000</td>
<td>36</td>
<td>1900</td>
<td>C5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1400</td>
<td>C5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1300</td>
<td>C5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL12A</td>
<td>70</td>
<td>0.75</td>
<td>150x250x3900</td>
<td>36</td>
<td>1100</td>
<td>C4</td>
<td></td>
<td>none</td>
</tr>
<tr>
<td>BL12B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1300</td>
<td>C3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PL1</td>
<td>55</td>
<td>0.6</td>
<td>70x95x1884</td>
<td>35</td>
<td>750</td>
<td>prism</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PL2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>750</td>
<td>prism</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PL3</td>
<td>65</td>
<td>0.7</td>
<td>80x80x1884</td>
<td>40</td>
<td>750</td>
<td>prism</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PL4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>750</td>
<td>prism</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4.3 Details and Parameters of Specimens Prestressed with CFCC Strands

<table>
<thead>
<tr>
<th>Strand diameter (d_b) (mm)</th>
<th>Specimen designation</th>
<th>Jacking force (P_j) (kN)</th>
<th>Jacking stress level (f_{pl} / f_{pu})</th>
<th>Specimen dimensions (b \times h \times L) (mm)</th>
<th>Concrete cover (d') (mm)</th>
<th>Embedment length (L_e) (mm)</th>
<th>Configuration type</th>
<th>Shear reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BT11</td>
<td>78</td>
<td>0.81</td>
<td>80x250x1380</td>
<td>45</td>
<td>630</td>
<td>A1</td>
<td>2 br. (\phi 6 @ 80)</td>
</tr>
<tr>
<td></td>
<td>BT12</td>
<td>85.5</td>
<td>0.6</td>
<td>100x250x2750</td>
<td>50</td>
<td>700</td>
<td>C1</td>
<td>2 br. (\phi 6 @ 80)</td>
</tr>
<tr>
<td></td>
<td>BT13</td>
<td>107</td>
<td>0.75</td>
<td>100x250x1380</td>
<td>50</td>
<td>640</td>
<td>A1</td>
<td>(\phi 6 @ 80)</td>
</tr>
<tr>
<td></td>
<td>BT14</td>
<td>122</td>
<td>0.61</td>
<td>120x300x1720</td>
<td>60</td>
<td>800</td>
<td>A1</td>
<td>(\phi 6 @ 80)</td>
</tr>
<tr>
<td></td>
<td>BT15A</td>
<td>149</td>
<td>0.75</td>
<td>120x300x1720</td>
<td>60</td>
<td>800</td>
<td>A1</td>
<td>(\phi 6 @ 80)</td>
</tr>
<tr>
<td></td>
<td>BT15B</td>
<td>147</td>
<td>0.74</td>
<td>200x300x3500</td>
<td>60</td>
<td>1100</td>
<td>C4</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>BT16A</td>
<td>147</td>
<td>0.74</td>
<td>200x300x3500</td>
<td>60</td>
<td>1250</td>
<td>C5</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>BT16B</td>
<td>123</td>
<td>0.61</td>
<td>175x300x3500</td>
<td>60</td>
<td>1000</td>
<td>C4</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>BT17A</td>
<td>123</td>
<td>0.61</td>
<td>175x300x3500</td>
<td>60</td>
<td>900</td>
<td>C3</td>
<td>none</td>
</tr>
<tr>
<td></td>
<td>BT17B</td>
<td>123</td>
<td>0.61</td>
<td>175x300x3500</td>
<td>60</td>
<td>900</td>
<td>C3</td>
<td>none</td>
</tr>
</tbody>
</table>
Table 4.4 Characteristics of Prestressing Reinforcements

<table>
<thead>
<tr>
<th></th>
<th>12.7 mm Steel strand</th>
<th>8 mm Leadline bar</th>
<th>10.5 mm CFCC strand</th>
<th>12.5 mm CFCC strand</th>
<th>15.2 mm CFCC strand</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Area (mm²)</strong></td>
<td>99</td>
<td>46.1</td>
<td>55.7</td>
<td>76</td>
<td>113.6</td>
</tr>
<tr>
<td><strong>Guaranteed strength (MPa)</strong></td>
<td>1860</td>
<td>1970</td>
<td>1725</td>
<td>1868</td>
<td>1750</td>
</tr>
<tr>
<td><strong>Modules of elasticity (GPa)</strong></td>
<td>200</td>
<td>147</td>
<td>140</td>
<td>141</td>
<td>138</td>
</tr>
<tr>
<td><strong>Ultimate tensile strain (%)</strong></td>
<td>4.5</td>
<td>1.3</td>
<td>1.7</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td><strong>Coeff. of thermal expansion (1/°C)</strong></td>
<td>$12 \times 10^{-6}$</td>
<td>$0.68 \times 10^{-6}$</td>
<td>$0.6 \times 10^{-6}$</td>
<td>$0.6 \times 10^{-6}$</td>
<td>$0.6 \times 10^{-6}$</td>
</tr>
<tr>
<td><strong>Measured tensile strength (MPa)</strong></td>
<td>1900</td>
<td>2170</td>
<td>na</td>
<td>na</td>
<td>1915</td>
</tr>
<tr>
<td><strong>Measured modules of elasticity (GPa)</strong></td>
<td>210</td>
<td>150-190</td>
<td>na</td>
<td>an</td>
<td>155</td>
</tr>
<tr>
<td><strong>Measured ultimate strain (%)</strong></td>
<td>2.5</td>
<td>1.2</td>
<td>na</td>
<td>na</td>
<td>1.25</td>
</tr>
</tbody>
</table>
Figure 4.1 Prestressing and casting set-up for beam specimens
Figure 4.2 Cast set-up for two prism specimens
Figure 4.3 Prestressing of the beam specimens

Figure 4.4 Prestressing of the prism specimen
Figure 4.5 Details of Reinforcement and Load Configuration (A1)

Figure 4.6 Details of Reinforcement and Load Configuration (A2)
Figure 4.7 Details of Reinforcement and Load Configuration (A3)

Figure 4.8 Details of Reinforcement and Load Configuration (B)
Figure 4.9 Reinforcement Details and Load Configuration (C1)

Figure 4.10 Reinforcement Details and Load Configuration (C2)
Figure 4.11 Reinforcement Details and Load Configuration (C3)

Figure 4.12 Reinforcement Details and Load Configuration (C4)
Figure 4.13 Reinforcement Details and Load Configuration (C5)

Figure 4.14 Prism Test Configuration
28 days after casting
Concrete Strength = 59.5 MPa
Modules of Elasticity = 30500 MPa

Figure 4.15 Stress-Strain Curve for Concrete Cylinder Under Uniaxial Compression (BS1)

Figure 4.16 Anchorage assembly of Leadline, CFCC, and steel
Figure 4.17 Tension test results of 8 mm Leadline bar using epoxy anchorage

Figure 4.18 Tension test results of 15.2 mm CFCC strand
Rupture of wires at anchorage at stress 1954 MPa

Figure 4.19 Measured stress-strain relationship of 12.7 mm steel strand

Figure 4.20 View of the jacking hardware
Figure 4.21 Draw-in measurement at transfer using dial gauge

Figure 4.22 Draw-in measurement at transfer using LVDT
Figure 4.23 Schematic of Test Setup

Figure 4.24 View of the test set-up of beam specimens
Figure 4.25 End slip measurement at test site
CHAPTER 5
TRANSFER LENGTH RESULTS AND ANALYSIS

5.1 INTRODUCTION

This chapter provides a detailed description of the experimental results and analysis of test data related to transfer length, bond stress within the transfer zone, effect of confinement on the transfer length, and time dependent effect. Test results of the flexural length and its characteristics will be considered in chapter 7. The measured data are used to identify the effect of the various variables on the bond behaviour of CFRP and to recommend simplified equations for transfer length.

In the analysis of the measured data, several researchers used the length between the end of the member to the intersection point of the varying strain region and the constant strain plateau as the transfer length (Cousins, et al., 1990) as shown in Figure 5.1a. This methodology is known by the “100% constant strain method”. The intersection point “i” of the varying strain zone and the constant strain plateau was determined numerically, by Nanni et al., 1992, that the measured concrete strain ($\varepsilon_i$) at this point has the following characteristics:

$$\varepsilon_i \geq \frac{\sum_{j=i+1}^{n} \varepsilon_j}{n - i} \quad \text{and} \quad \varepsilon_i > \varepsilon_{i+1} \quad (5.1)$$
where \( n \) is the total number of strain readings along the member. That is, the strain at the “\( i+1 \)" location has a strain lower than that at the “\( i \)” location. The strain at the “\( i \)” location is greater or equal the average strain of all following points. Other researchers determined the transfer length by the location at which the varying strain region intersects a horizontal line corresponding to 95% of the constant strain plateau as shown in Figure 5.1b, (FHA 1994, Hanson 1969, Weerasekara 1991). This method of determining transfer length is known by the “95 percent constant strain method". A third approach known by “slope-intercept” was also used by many researchers to determine the transfer length. In this method, the transfer length is the distance from the end of the member to the point of intersection of the straight line fitting the measured strains within the transfer zone and the constant strain plateau as shown in Figure 5.1c. The slope-intercept method has the disadvantage that a judgement must be made about how many strain readings to be included in the regression analysis process for each line. In this study, the transfer length was determined using the first approach which is the “100% constant strain method” using Nanni’s numerical method.

5.2 TRANSFER LENGTH RESULTS

Measurements of strain gauges and demec points were recorded at transfer, after four weeks, and during testing. A typical measured data based on electrical strain gauge reading within the transfer zone is shown in Figure 5.1d. Tendon stress was determined as the product of the measured strain and the elastic modules of tendon. The figure indicates that the prestress losses increased from 3% at release due to elastic shortening to 15% after four weeks as a result of creep and shrinkage of concrete. The measured concrete strain along the
surface of concrete specimen at release and after four weeks for the same specimen using
demec points is shown in Figures 5.2 and 5.3. Figure 5.3 shows that the concrete strain
increased from 0.00038 at transfer to 0.0013 after four weeks due to creep and shrinkage of
concrete. The transfer length was determined using the measured data at transfer and the
data after four weeks.

Detailed test results for all tested specimens are given in Appendix A. The average
congrete strain based on the measurements of two ends of each specimen after release are
plotted along the distance from the transfer onset. The ratio of the measured tendon strain to
the initial strain are plotted along the distance from transfer onset. The measured transfer
lengths at the time of transfer, based on strain gauges and demec point readings for
specimens pretensioned with steel strand, Leadline bars, and CFCC, are summarised in
Table 5.1, 5.2, and 5.3 respectively.

5.3 ANALYSIS OF TRANSFER LENGTH RESULTS

5.3.1 Steel Strands

Most of the reported work dealt with the transfer length of small wires with different
sizes, plain, twisted, crimped, indented or deformed. Recently, studies were diverted toward
practical application and dealt with multi-wire strands. The reported values of the transfer
length by various investigators for different sizes of strands are given in tables 3.1 and 3.2.
The initial prestress ($f_{pi}$) and the concrete strength at transfer ($f_{ci}$) are also given in the same
tables. Figure 5.4 shows summary of the transfer length ($L_t$) versus ($f_{pi}d/f_{ci}^{0.67}$) for all
reported results in tables 3.1 and 3.2 as well as test results of specimens prestressed with
steel conducted in this study. Based on these data, a new proposed model for the transfer
length of prestressing steel strand is introduced as given in equation 5.2. This equation accounts for effect of strand diameter \(d_b\), initial prestress \(f_{pi}\), and concrete compressive strength at transfer \(f'_{ci}\).

\[
L_t = \frac{f_{pi} \cdot d_b}{2.4 \cdot f_{ci}^{0.67}}
\]  
(5.2)

The constant value of 2.4 is obtained from the linear regression analysis of both the reported data and the test data with standard deviation of error of \(\sigma = 20\%\) as shown in Figure 5.4. The high value of \(\sigma\) is due to the scattered distribution of the reported data by different references. Test results of the current study are in good correlation with the proposed equation.

A comparison of the proposed model for transfer length of steel strands and different models currently available in the literature, reported in Table 3.3, are shown in Figures 5.5a and 5.5b. The comparison shows that the proposed model gives transfer length 4% higher than the ACI equation at concrete compressive strength at release of 30 MPa. This difference increased to 18% for lower concrete strength of 25 MPa. For higher concrete strength of 40 MPa at transfer, the proposed model for steel strands gives 14% shorter transfer length than the ACI equation. Figure 5.5b shows the effect of concrete strength at time of transfer on the transfer length using different equations for steel strand. The transfer lengths at two different values of concrete compressive strength at transfer of 40 and 25 MPa are shown in Figure 5.5b. The transfer length is found to be in the range of 38 to 61 times the strand diameter at concrete strength is 40 MPa at transfer. At lower concrete strength of 25 MPa at transfer, the predicted values range from 51 to 84 times the tendon diameter.
Figure 5.5c shows a comparison of the standard deviation of error (\(\sigma\)) of the predicted transfer lengths using different equations including the proposed model to the measured values. The proposed model for transfer length of steel strands has the best correlation with the test data as the it gives the lowest value of standard deviation of error. The figure shows that the ACI equation has the highest value of \(\sigma\) since the equation does not include the effect of concrete strength at time of transfer. Figure 5.5d shows the transfer length data and the predicted values using the ACI equation. The transfer length predicted using the CEB-FIP (den Uijl, 1992) is conservative compared to the measured data as shown in Figure 5.5e.

5.3.2 CFRP Tendons

The transfer length of a prestressing tendon is greatly influenced by the Hoyer effect (Janney, 1954). The Hoyer effect is caused by swelling of the tendon in the transfer zone after release as a result of Poisson's ratio. Since the lateral deformations is resisted by the surrounding concrete, the induced confining stresses normal to the tendon enhances the bond strength at the interface. Enhancement of the bond caused by the Hoyer effect is directly related to the concrete strength and the coefficient of friction between the two materials. Since the modules of elasticity of CFRP is about 70 percent of that for steel strands, the longitudinal strain, and consequently lateral strain, is larger than that for steel strands with the same prestressing stress level and Poisson’s Ratio value. This behaviour will lead to increase of the induced stresses perpendicular to the fibres in comparison to steel. and results in shorter transfer lengths for CFRP. The depressions on the Leadline surface and the wrap coating of the CFCC strands enhances also the frictional properties and therefore reduces the
Based on the measured data in this program, the transfer length ($L_n$) was found to be directly related to the diameter of the strand ($d_b$), the initial prestressing level ($f_{pi}$), and the concrete strength at release ($f_{ci}$) as proposed in equations 5.3 and 5.4.

For Leadline bars:

$$L_n = \frac{f_{pi} d_b}{1.9 f_{ci}^{0.67}}$$  \hspace{1cm} \text{for Leadline bars} \hspace{1cm} (5.3)\\

For CFCC strands:

$$L_n = \frac{f_{pi} d_b}{4.8 f_{ci}^{0.67}}$$  \hspace{1cm} \text{for CFCC strands} \hspace{1cm} (5.4)

where the coefficients 1.9 and 4.8 for Leadline and CFCC respectively are obtained from the linear regression as shown in Figure 5.6. The accuracy of the proposed models is evident in Figure 5.6 by comparing the proposed equations to the measured values.

The proposed models indicate that increase of concrete strength reduces transfer length of the same bar or strand. This could be attributed to possible improvement of the bond characteristics for high strength concrete. High strength concrete, consequently, higher tensile strength and modules of elasticity of concrete could enhance the magnitude of contact pressure at the tendon-concrete interface. Due to this importance of concrete compressive strength as parameter, it was considered by many other researchers in their models for determining the transfer length and flexural bond lengths (Zia and Mostafa, 1977), (Cousins et al., 1990), (Mitchell et al., 1993). CEB-FIP (den Uijl, 1992) related the transfer and development lengths to bond strength which is function of concrete strength. The test results in this program showed that the transfer and flexural bond lengths are proportional to $f_{ci}^{0.67}$ and $f_c^{0.67}$ respectively.
In spite of the short length of transfer, no bursting or splitting of concrete was observed at release of prestress at the beam ends for the concrete cover used of four times the tendon diameter and concrete strength ranges from 22 to 42 MPa at transfer. Since the concrete cover was not varied in this study, its effect is not reflected in the proposed models. Therefore, the proposed models are valid only for concrete cover at least four times the diameter of the tendon. Previous research (Taerwe 1993) has indicated that for Aramid fibre bars the critical concrete cover to be used to avoid splitting of concrete is in the range of 2.8 times the bar diameter. Small concrete cover could lead to splitting of concrete at transfer due to insufficient confinement by the concrete.

It should be noted that in this program, all beam specimens had a debonded length of 50 or 100 mm at the beam ends. Therefore after release of prestress, swelling of the tendon did not induce radial pressure within this unbonded length. This could increase the splitting resistance of the transfer zone.

A comparison of the proposed models for transfer length of steel strands, Leadline bars, and CFCC strands is shown in Figure 5.7. It can be seen that the transfer length of Leadline bars is 25% higher than that of steel strands while the transfer length of CFCC strands is about 50% of that of steel strands.

Abdelrahman and Rizkalla (1995) reported that the transfer lengths of 8 mm diameter Leadline are 360 mm and 500 mm for prestress levels of 50% and 70% the guaranteed ultimate strength respectively. These results, when compared with the proposed model for Leadline, were found to be within 7% to 9% greater than the predicted values using the proposed model.
5.4 TRANSFER BOND STRESSES

Longitudinal stresses in the prestressing tendons after release can be determined at the locations of strain gauges using the readings before and after release as follow:

\[
\text{Stress in the tendon} = \frac{\text{Strain after release}}{\text{Strain before release}} \times \text{Jacking stress} \quad (5.5)
\]

Based on the longitudinal stresses distribution along the tendon after release, the average bond stress, \( U_{av} \), between any two points “a” and “b” could be determined as follow:

\[
U_{av} = \frac{(f_{pb} - f_{pa}) A}{\pi d_{b} l} \quad (5.6)
\]

where \( f_{pa} \) and \( f_{pb} \) : are the longitudinal stresses in the tendon at points a and b, respectively,

\( A \) : is the cross section area of prestressing reinforcement (mm\(^2\)),

\( d_{b} \) : is the nominal diameter of tendon (mm),

\( l \) : is the distance between the two gauge points (mm).

Bond stresses within the transfer zone varies along the transfer length (Janney, 1954 and Balazs, 1992). The measured data for all tested specimens in this study confirm this behaviour. Figure 5.8 shows a typical bond stress distribution within the transfer zone after release of CFCC strand. The figure indicates high bond stress near the end of the beam which decreases rapidly and vanish at the end of the transfer length. This behaviour could be attributed to the frictional forces along the transfer length, which is considered to be one of the major component of bond mechanism in this zone. These forces are proportional to the contact radial stress between the tendon and the surrounding concrete. The contact pressure is inversely proportional to the transferred stress. At onset of transfer, the
transferred stress is minimum and the contact pressure is maximum. Appendix B shows the bond stress distribution in all specimens obtained from the measured strain in the tendon using strain gauges.

To obtain a suitable design criteria, the bond stress along the tendon within the transfer zone is expressed as an average value. The average transfer bond stress ($U_t$) was determined, for all the tested beams, based on the measured transfer lengths ($L_t$), the initial prestressing stress ($f_{pi}$), the cross section area of tendon ($A$), and the diameter ($d_b$) as follow:

$$U_t = \frac{A f_{pi}}{\pi d_b L_t}$$

Equation 5.7 was used to calculate the transfer bond stress, $U_t$, of 7-wire steel strand using the data reported in the literature in Tables 3.1 and 3.2 and those tested in this program. The results indicate that the average transfer bond stress, $U_t$, of the 7-wire steel strands at transfer is 5.33 MPa with standard deviation of 33%. The expression suggested by the ACI Code 1995 for the transfer length of steel strands, equation 3.8, implies an average transfer bond stress for steel strand of 4.0 MPa. Using the same procedure with the test data of specimens prestress with Leadline and CFCC, the average transfer bond stress, $U_t$, at time of transfer was found to be 4.5 MPa (standard deviation of 9%) and 8.9 MPa (standard deviation of 12%) for Leadline and CFCC strands respectively.

The transfer bond stress could be expressed in terms of a bond stress index ($U'_t$) and the compressive strength of concrete at transfer ($f'_{ci}$) as follows:

$$U_t = U'_t f'_{ci}^{0.67}$$

(5.8)
The analysis of data for specimens prestressed with steel resulted in average value of transfer bond index $U'_t$ of 0.48. For the 8 mm diameter Leadline bars, the average value of the transfer bond index $U'_t$ is 0.44. For CFCC, the average value of the transfer bond index was found to be 0.75. The transfer bond index, given in Figure 5.9 in terms of the nominal diameter of CFCC, suggests that the average transfer bond index, $U'_t$, is not influenced by the change in the diameter of CFCC strand.

Test result indicate in general that the bond characteristics of Leadline is less than steel strand. The twisted shape of 7-wire steel strand certainly enhance the mechanical component of the bond mechanism. The superior bond characteristics of CFCC strands compared to Leadline bars and steel strands is related to the increase in the frictional bond forces due to the higher surface roughness in addition to the mechanical bond due to the strand shape. The increase of surface roughness of CFCC strand is due to the external layer of cross fibres of about 200 $\mu$m thick encapsulating the individual longitudinal fibres of 7 $\mu$m diameter (Gowripalan, 1996).

5.5 EFFECT OF CONCRETE CONFINEMENT

To study the effect of concrete confinement through the use of stirrups on the transfer length, four beams and four prisms pretensioned with Leadline and six beams prestressed with CFCC were tested without shear reinforcement. The concrete coverage was 35 mm in case of Leadline, 50 mm for 12.5 mm diameter CFCC, and 60 mm for beams pretensioned with 15.2 mm CFCC. No splitting cracks was observed after release. Test results indicated that absence of shear reinforcement in specimens pretensioned with Leadline increased the transfer length with an average value of 10% more than the values
predicted by the proposed model. This is evident in Figure 5.10 which compares the transfer length results of specimens without shear reinforcement with the proposed model given by equation 5.3.

Ehsani et al. (1996) had measured the transfer length of 8 mm Leadline by testing one prism and two beams, designated as CL-2, 2CL-1, and CL-3 respectively. The specimens had no shear reinforcement. The prestressing levels were 54%, 59%, and 54% of the guaranteed ultimate strength for CL-2, 2CL-1, and CL-3 respectively. The concrete compressive strength at time of release was 28 MPa for the three specimens. The reported transfer length for the prism specimen, CL-2, was 940 mm. The reported transfer lengths of beam specimen, 2CL-1, were 305 and 380 mm, at the jacking and dead ends, respectively. For beam specimen, CL-3, the reported transfer lengths were 430 and 610 mm for the jacking and dead end of the beam, respectively. These results are compared to the proposed model for members without confinement (1.1 times the value predicted using equation 5.3) in Figure 5.10. The proposed model for Leadline without confinement predicts transfer lengths of 530, 570, and 530 mm for CL-2, 2CL-1, and CL-3 respectively. Only the results of beam specimen CL-3 are in good agreement with the proposed model. Unfortunately, there is no other comprehensive study in the literature related to transfer length of CFRP tendons.

For beams prestressed with CFCC strands, the absence of shear reinforcement resulted in also an increase of the transfer length by an average value of 17% higher than the proposed model as shown in Figure 5.11.
5.6 TIME DEPENDENT EFFECT

Creep of concrete may cause increase in transfer length with age. Kaar et al. (1963) concluded that the average increase in the transfer length over a period of 365 days was 6% for all sizes of steel strands. Cousins (1987) reported that the transfer length increased, after one year, by 5.4% and 11.7% for uncoated and epoxy coated steel strands respectively. In this research program, limited specimens were used to investigate the long-term effect on the transfer length for steel, Leadline, and CFCC used to pretension concrete beams.

5.6.1 Beam Specimens BS5 and BS6

Figure 5.12 shows the stress distribution along the steel strand of specimens BS5 and BS6 at the time of release, 2 months, and one year after release. The losses due to elastic shortening at time of release was 5% and increased to 15% after two months. The periodical readings showed that no significant change from 9 months to 12 months. Therefore, one year is normally selected by most of the researchers to study the long term effect. A loss of prestress of 20% was measured after one year. It can be noticed from Figure 5.12 that the transfer length of steel strand did not increase after one year. Strand stress distribution shows a decrease in the slope near the end of the beam with time. This behaviour could reflect possible reduction of bond stress in this zone.

Figures 5.13 and 5.14 show the measured concrete strain distribution along the transfer length at the time of release, after 2 months, and after 1 year for specimens BS5 and BS6, respectively. The behaviour indicates that concrete strain at the bottom surface increased from 0.0005 at transfer to 0.0016 after 2 months to 0.002 after 1 year. The figures show also that no change of the transfer length after one year.
5.6.2 Beam Specimens BL3 and BL4

The stress in the Leadline bar along the transfer length of specimens BL3 and BL4 at release, 2 months, and one year after release are shown in Figure 5.15. The results indicate that the loss in prestressing are 4%, 15%, and 21% at release, 2 months, and after one year respectively. It can be noticed in Figure 5.15 also that the transfer length increased after one year. The increase of transfer length after 1 year is approximately 22 percent.

The measured concrete strain along the transfer length for BL3 and BL4 are shown in Figures 5.16 and 5.17, respectively. The concrete strain profile are given at release, 2 months, and one year after transfer. These figures show that the axial concrete strains increase with time due to creep and shrinkage.

5.6.3 Beam Specimens BT1 and BT2

The stress distribution along the CFCC strand for BT1 and BT2 specimens at transfer, 2 months, and one year after release are shown in Figure 5.18. The figure indicate that the jacking stress of 1075 MPa is reduced to 1045 MPa, 960 MPa, and 905 MPa at release, 2 months, and one year after release respectively. The majority of losses occurred within the first two months. No change in transfer length of CFCC strand was observed after period of one year.

The measured concrete strains along the transfer length are given in Figures 5.19 and 5.20 for beams BT1 and BT2, respectively. The figures show that there is no increase in transfer length of CFCC strand after one year. The results suggest that the spiral shape of CFCC and steel strands enhanced the mechanical component of bond and resulting in no increase in the transfer length by time. The increase of the transfer length of Leadline bars
by time could be due to its relative smooth surface in comparison to CFCC and steel strands, as shown in Figure 5.15.

5.6.4 Effect of Creep on Transfer Bond

The bond stresses within the transfer length of BS5, BL3, and BT1 are shown in Figures 5.21, 5.22 and 5.23, respectively. The figures show the bond stress distribution at release, 2 months, and one year after release. All figures showed a redistribution of bond stresses with time for each of the three types of reinforcements. The radial pressure exerted by the tendon within the transfer zone causes sustained radial pressure which could increase the diameter of the "hole", through which the tendon is passing, due to creep of concrete. Consequently, the bond stress, which is proportional to the contact pressure at the interface, could decrease and the tendon would pull further from the end of the beam and cause increase in the transfer length. The maximum bond stresses are usually at the location of maximum radial pressure. This location is generally close to the end of the unit and the contact pressure reduces to zero at the end of transfer zone. On the other hand, the shrinkage of concrete before release could enhance the bond capacity due to the increase in contact pressure and thus would reduce the creep effects and perhaps eliminate them totally near the end of the transfer length. Nevertheless no reduction of the transfer length due to shrinkage of concrete after release could be anticipated.

5.7 END SLIP RESULTS

At release, the reduction in tensile strain of the prestressing reinforcement must be equal to the increase in the compressive strain in concrete at the same location. This condition is applicable only beyond the transfer length as shown in Figure 5.24. Within the
transfer zone, the reduction in tendon strain is greater than the corresponding induced strain in the concrete as shown in Figure 5.24. This is evidence by the fact that the maximum reduction of the tendon strain at the end of the beam correspond to zero increase of strain of the concrete at the same location. This behaviour justify the observed slip at the free end of transfer length which is known by “draw-in” or “pull-in” of the strands at release.

Guyon (1960) assumed that the transfer length of prestressing steel is linearly proportional to the draw-in (Δ) and inversely proportional to the initial tendon strain (ε_{pi}):

\[ L_t = \alpha \frac{\Delta}{\varepsilon_{pi}} \]  \hspace{1cm} (5.9)

where α is a coefficient takes into account the shape and distribution of the bond stresses.

Referring to Figure 5.24, the draw-in can be calculated as:

\[ \Delta = \int_{0}^{L_t} (\varepsilon_{pi} - \varepsilon_{pe} - \varepsilon_{ce}) \, dx \]  \hspace{1cm} (5.10)

In the case of constant bond stress distribution and linear distribution of the prestressing and concrete strains, the draw-in, Δ, can be estimated as:

\[ \Delta = \frac{\varepsilon_{pi} L_t}{2} \]  \hspace{1cm} (5.11)

This approximation provide a value of 2 for the coefficient α. In the case of linear bond stress distribution and parabolic distribution of the prestressing and concrete strains, the draw-in, Δ, can be estimated as:

\[ \Delta = \frac{\varepsilon_{pi} L_t}{3} \]  \hspace{1cm} (5.12)
where $\alpha$ in this case equal to 3.

End slip has been measured during transfer of prestress and after complete transfer. Typical examples of the measured end slip at transfer are shown in Figures 5.25 and 5.26 which show plots of tendon draw-in versus the released force for specimens pretensioned with Leadline and CFCC respectively. Tables 5.1, 5.2 and 5.3 contain the end slip results for specimens which had enough room between the coupler and the concrete to allow measurements of the end slip. The initial distance from the transfer onset and the point at which the LVDT or dial gauge is fixed was measured before prestress transfer, and the readings were adjusted to compensate for the elastic shortening of this distance. For steel strands, the measured end slip was in the range of 0.5 mm to 1.0 mm. The measured values for Leadline were between 0.73 mm and 3.0 mm. For CFCC strands, the observed pull-in varied from 0.68 mm to 2.9 mm.

Using the initial prestressing strain ($\varepsilon_{pl}$) and the measured transfer length ($L_t$) for each specimen, the constant $\alpha$ was found to an average value of 2.91 and 2.48 for Leadline and CFCC respectively. The values for all tested specimens are given in tables 5.2 and 5.3. Polish researchers (cited in Balazs, 1993) obtained a value of 2.86 for $\alpha$ for prestressing steel strands. den Uijl, 1992, obtained $\alpha$ of 2.46 by experimental and theoretical studies of hollow-core slabs prestressed by 7-wire strands.
Table 5.1 Transfer Length Results of Specimens Prestressed with Steel Strands

<table>
<thead>
<tr>
<th>Strand diameter, (d_0) (mm)</th>
<th>Specimen designation</th>
<th>Initial stress level (f_{pi} / f_{pu})</th>
<th>Concrete strength at release (f'_{ci}) (MPa)</th>
<th>Measured transfer length (L_t) (mm)</th>
<th>Measured draw-in (\Delta_r) (mm)</th>
<th>Model prediction/measured value</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.8</td>
<td>PS1</td>
<td>48</td>
<td>na</td>
<td>400</td>
<td>0.69</td>
<td>0.83</td>
</tr>
<tr>
<td></td>
<td>PS2</td>
<td>0.6</td>
<td>na</td>
<td>400</td>
<td>0.63</td>
<td>0.83</td>
</tr>
<tr>
<td>9.6</td>
<td>PS3</td>
<td>46</td>
<td>na</td>
<td>400</td>
<td>na</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>PS4</td>
<td>na</td>
<td>350</td>
<td>na</td>
<td>0.77</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>BS1</td>
<td>0.6</td>
<td>48</td>
<td>na</td>
<td>350</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>BS2</td>
<td>na</td>
<td>350</td>
<td>na</td>
<td>0.82</td>
<td>0.95</td>
</tr>
<tr>
<td>12.7</td>
<td>PS5</td>
<td>35</td>
<td>na</td>
<td>550</td>
<td>0.84</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>PS6</td>
<td>0.6</td>
<td>na</td>
<td>600</td>
<td>0.93</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>PS7</td>
<td>46</td>
<td>na</td>
<td>400</td>
<td>0.96</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>BS3</td>
<td>0.6</td>
<td>46</td>
<td>375</td>
<td>400</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>BS4</td>
<td>375</td>
<td>400</td>
<td>0.48</td>
<td>1.14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BS5</td>
<td>0.66</td>
<td>35</td>
<td>550</td>
<td>550</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>BS6</td>
<td>550</td>
<td>500</td>
<td>1.0</td>
<td>1.09</td>
<td></td>
</tr>
</tbody>
</table>

\(f_{pi}\): Initial prestress after elastic losses  
\(f_{pu}\): Guaranteed tensile strength = 1860 MPa  
E.S.G.: Measured transfer length based on strain gauges  
D.G.: Measured transfer length based on demec gauges
Table 5.2 Transfer Length Results of Specimens Prestressed with Leadline

<table>
<thead>
<tr>
<th>Bar diameter, $d_b$ (mm)</th>
<th>Specimen designation</th>
<th>Initial stress level $f_{pi} / f_{pu}$</th>
<th>Concrete strength at release $f_{ci}$ (MPa)</th>
<th>Measured transfer length $L_t$ (mm) based on E.S.G.</th>
<th>Measured draw-in $\Delta$ (mm) based on D.G.</th>
<th>Measured draw-in $\Delta$, measured value</th>
<th>Model prediction / measured value</th>
</tr>
</thead>
<tbody>
<tr>
<td>BL1</td>
<td>0.58</td>
<td>34</td>
<td>450</td>
<td>455</td>
<td>0.88</td>
<td>1.04</td>
<td>0.98</td>
</tr>
<tr>
<td>BL2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL3</td>
<td>0.68</td>
<td>35</td>
<td>450</td>
<td>510</td>
<td>1.56</td>
<td>1.08</td>
<td>1.08</td>
</tr>
<tr>
<td>BL4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL5</td>
<td></td>
<td></td>
<td></td>
<td>640</td>
<td>600</td>
<td>2.71</td>
<td>1.02</td>
</tr>
<tr>
<td>BL6</td>
<td>0.72</td>
<td>28</td>
<td>640</td>
<td>610</td>
<td>na</td>
<td>na</td>
<td>1.01</td>
</tr>
<tr>
<td>BL7</td>
<td>0.68</td>
<td>35</td>
<td>600</td>
<td>620</td>
<td>3.00</td>
<td>3.00</td>
<td>1.02</td>
</tr>
<tr>
<td>BL8</td>
<td>0.72</td>
<td>35</td>
<td>600</td>
<td>650</td>
<td>na</td>
<td>na</td>
<td>1.00</td>
</tr>
<tr>
<td>BL9</td>
<td>0.61</td>
<td>26</td>
<td>700</td>
<td>na</td>
<td>2.25</td>
<td>2.25</td>
<td>0.82</td>
</tr>
<tr>
<td>BL10</td>
<td></td>
<td></td>
<td></td>
<td>700</td>
<td>na</td>
<td>2.88</td>
<td>0.82</td>
</tr>
<tr>
<td>BL11</td>
<td>0.74</td>
<td>42</td>
<td>600</td>
<td>na</td>
<td>1.88</td>
<td>1.88</td>
<td>0.84</td>
</tr>
<tr>
<td>BL12</td>
<td></td>
<td></td>
<td></td>
<td>500</td>
<td>na</td>
<td>1.18</td>
<td>1.00</td>
</tr>
<tr>
<td>PL1</td>
<td>0.57</td>
<td>34</td>
<td>480</td>
<td>500</td>
<td>0.73</td>
<td>0.73</td>
<td>0.91</td>
</tr>
<tr>
<td>PL2</td>
<td></td>
<td></td>
<td></td>
<td>480</td>
<td>480</td>
<td>1.36</td>
<td>0.93</td>
</tr>
<tr>
<td>PL3</td>
<td>0.68</td>
<td>31</td>
<td>na</td>
<td>540</td>
<td>na</td>
<td>0.97</td>
<td>1.04</td>
</tr>
<tr>
<td>PL4</td>
<td></td>
<td></td>
<td>na</td>
<td>525</td>
<td>1.33</td>
<td>1.33</td>
<td>1.07</td>
</tr>
</tbody>
</table>

\(f_{pi}\): Initial prestress after elastic losses  
\(f_{pu}\): Guaranteed tensile strength =1970 MPa  
E.S.G.: Measured transfer length based on strain gauges  
D.G.: Measured transfer length based on demec gauges
Table 5.3 Transfer Length Results of Specimens Prestressed with CFCC strands

<table>
<thead>
<tr>
<th>Strand diameter, $d_b$ (mm)</th>
<th>Specimen designation</th>
<th>Initial stress level $f_{pi}/f_{pu}$</th>
<th>Concrete strength at release $f'_{ci}$ (MPa)</th>
<th>Measured transfer length $L_t$ (mm)</th>
<th>Measured draw-in $\Delta_r$ (mm)</th>
<th>Model prediction/measured value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BT11</td>
<td></td>
<td>300</td>
<td>310</td>
<td>na</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td>BT12</td>
<td>0.79</td>
<td>29</td>
<td>300</td>
<td>325</td>
<td>na</td>
</tr>
<tr>
<td>10.5</td>
<td>BT13</td>
<td></td>
<td>325</td>
<td>300</td>
<td>na</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>BT14</td>
<td></td>
<td>325</td>
<td>300</td>
<td>na</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>BT7</td>
<td></td>
<td>400</td>
<td>400</td>
<td>na</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>BT8</td>
<td>0.73</td>
<td>28</td>
<td>400</td>
<td>350</td>
<td>na</td>
</tr>
<tr>
<td></td>
<td>BT9</td>
<td></td>
<td>370</td>
<td>350</td>
<td>na</td>
<td>1.08</td>
</tr>
<tr>
<td></td>
<td>BT10</td>
<td></td>
<td>370</td>
<td>350</td>
<td>na</td>
<td>1.10</td>
</tr>
<tr>
<td>12.5</td>
<td>BT15</td>
<td>0.59</td>
<td>30</td>
<td>310</td>
<td>290</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>BT16</td>
<td></td>
<td>290</td>
<td>300</td>
<td>0.91</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>BT19</td>
<td>0.72</td>
<td>23</td>
<td>500</td>
<td>na</td>
<td>1.71</td>
</tr>
<tr>
<td></td>
<td>BT20</td>
<td></td>
<td>450</td>
<td>na</td>
<td>1.82</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>BT1</td>
<td>0.60</td>
<td>35</td>
<td>350</td>
<td>380</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>BT2</td>
<td></td>
<td>350</td>
<td>360</td>
<td>0.90</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>BT3</td>
<td></td>
<td>400</td>
<td>385</td>
<td>0.98</td>
<td>1.03</td>
</tr>
<tr>
<td>15.2</td>
<td>BT4</td>
<td>0.73</td>
<td>31</td>
<td>400</td>
<td>375</td>
<td>1.20</td>
</tr>
<tr>
<td></td>
<td>BT5</td>
<td></td>
<td>400</td>
<td>400</td>
<td>1.40</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td>BT6</td>
<td></td>
<td>400</td>
<td>400</td>
<td>1.35</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>BT17</td>
<td>0.74</td>
<td>22</td>
<td>650</td>
<td>na</td>
<td>2.90</td>
</tr>
<tr>
<td></td>
<td>BT18</td>
<td></td>
<td>600</td>
<td>na</td>
<td>2.75</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>BT21</td>
<td>0.62</td>
<td>24</td>
<td>510</td>
<td>na</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>BT22</td>
<td></td>
<td>490</td>
<td>na</td>
<td>2.15</td>
<td>0.83</td>
</tr>
</tbody>
</table>

$f_{pi}$: Initial prestress after elastic losses

$f_{pu}$: Guaranteed tensile strength (see table 4.4)

E.S.G.: Measured transfer length based on strain gauges

D.G.: Measured transfer length based on demec gauges
Figure 5.1a. Transfer length based on "100% constant strain plateau".

Figure 5.1b. Transfer length based on "95% constant strain plateau".
Figure 5.1c. Transfer length based on "slope intercept concept".

Figure 5.1d Stress distribution along the CFCC strand of BT15
Figure 5.2 Concrete strain distribution along the beam BT15

Figure 5.3 Concrete strain distribution along the transfer length of BT15
\[ L_t = \frac{f'_{pl} d_b}{2.4 f'_{cl}^{0.67}} \]

\[ \sigma = 20\% \]

Figure 5.4 Transfer length correlation for steel strands

Figure 5.5a Transfer length comparison for steel strands
Figure 5.5b  Effect of concrete strength on the transfer length using different equations for steel strands

Figure 5.5c  Standard deviation of error of transfer length predicted using different proposed model for steel strands
Figure 5.5d Transfer length correlation using ACI equation for steel strands

Figure 5.5e Transfer length correlation using CEB approach for steel strands
Figure 5.6 Transfer length correlation for Leadline bars and CFCC strands

\[ L_t = \frac{f_{pi} d_b}{1.9 f_{ci}^{0.67}} \]
(Standard deviation = 6.4%)

\[ L_t = \frac{f_{pi} d_b}{4.8 f_{ci}^{0.67}} \]
(Standard deviation = 12%)

Figure 5.7 Comparison of the proposed models for transfer length

\[ f_{ci} = 30 \text{ MPa} \]
Proposed for Leadline

Proposed for steel

Proposed for CFCC

Initial prestress level, (MPa)
Figure 5.8 Bond stress distribution at transfer of BT17

Figure 5.9 Effect of strand diameter on the transfer bond strength
Figure 5.10 Effect of shear reinforcement on the transfer length of Leadline bars

Figure 5.11 Effect of confinement on the transfer length of CFCC
Figure 5.12 Stress distribution along the tendon of BS5

Figure 5.13 Concrete strain distribution of specimen BS5
Figure 5.14 Concrete strain distribution of specimen BS6

Figure 5.15 Stress distribution along the tendon of BL3
Figure 5.16 Concrete strain distribution of specimen BL3

Figure 5.17 Concrete strain distribution of specimen BL4
Figure 5.18 Stress distribution along the tendon of BT1 and BT2

Figure 5.19 Concrete strain distribution of specimen BT1
At release
- 2 months
○ 1 year

Figure 5.20 Concrete strain distribution of specimen BT2

- At release
- 2 months
○ 1 year

Figure 5.21 Bond stress distribution along the steel strand of BS5
Figure 5.22 Bond stress distribution along the Leadline bar of BL3

Figure 5.23 Bond stress distribution along CFCC strand of BT1
Figure 5.24 Calculation of coefficients in Guyon's formula

Figure 5.25 Measured draw-in at release for Leadline bars
Figure 5.26 Measured draw-in of CFCC strands of BT17
CHAPTER 6
THEORETICAL ANALYSIS OF TRANSFER BOND

6.1 INTRODUCTION

Identification of the important variables that influence the transfer bond of prestressing reinforcement is essential for understanding of the behavior and consequently its modeling. The three main variables are the concrete properties, properties of reinforcement, and the characteristics interface. To approach the problem rationally, it is necessary to base the analysis on the behavior of the two materials.

In this chapter, a detailed procedure is presented to evaluate the transfer length of prestressing reinforcement. The method is based on the theory of thick-walled cylinder considering only friction bond and neglecting adhesion and mechanical bond. Preliminary analysis using linear elastic theory revealed that the stresses in the concrete far exceed its tensile strength. Therefore, the immediate surrounding concrete is typically cracked during the bond development process at release. This behavior required modification of the analysis to consider the concrete properties after cracking. A computer program, listed in Appendix C, was developed to carry out the iteration process required to achieve compatibility of displacements in the radial direction at the tendon-concrete interface. The input for the computer program are the concrete dimensions, compressive strength ($f_{ci}$), tensile strength of concrete ($f_t$), Poisson's ratio of concrete...
and reinforcement, prestress level \( f_{pi} \), tendon diameter, properties of reinforcement in the longitudinal and transverse directions, and the coefficient of friction between the concrete and reinforcement \( \mu \). The outputs of the analysis are the transfer length, the longitudinal tensile stress in the reinforcement along the transfer zone, the bond stress distribution in the transfer zone, concrete radial and tangential stress distributions in the transverse direction at selected sections, and degree of cracking. The increase in contact pressure due to drying shrinkage of concrete after curing as well as the reduction in contact pressure due to the increase in compressive stress of concrete at the level of the tendon due to prestress transfer are considered in the analysis.

To prove the validity and reliability of the proposed theoretical model, the model was used to predict the transfer length of prestressing steel wires and strands. The results were found to be comparable to the reported data in the literature. The same approach was used to analyze the transfer bond of Leadline bars using the measured value of the coefficient of friction, \( \mu \), between the Leadline bars and the concrete. The theoretical predictions of transfer length showed good correlation with the experimental results. A comparison between theoretical and experimental stress curves of the specimens prestressed with Leadline showed that the behavior matches very well. Based on the comparison between theoretical and experimental results, it is concluded that the proposed friction model can be successfully used to predict the transfer bond behavior of FRP reinforcement, qualitatively and quantitatively.
6.2 ELASTIC SOLUTION FOR A THICK-WALLED CYLINDER

Consider a cylinder with inner radius \( a \), outer radius \( b \), and height \( h \), as shown in Figures 6.1a and 6.1b. The cylinder is subjected to a uniform inner pressure \( \sigma_{ra} \) and uniform axial tensile stress \( f_{cz} \) (Crandall et al., 1972). To take advantage of cylindrical symmetry, the cylindrical coordinates \( r, \theta, \) and \( z \) are used. The displacement components in the \( r, \theta, \) and \( z \) directions are noted by \( u, v, \) and \( w \), respectively. The complete analysis consists of 15 elasticity equations, three equilibrium equations, six strain-displacement equations, and six stress-strain equations. The boundary conditions can be summarized as:

at the inner surface where \( r = a \)
\[
\sigma_r = \sigma_{ra} \quad \tau_{rz} = 0 \quad \tau_{r\theta} = 0,
\]
at the outer surface where \( r = b \)
\[
\sigma_r = 0 \quad \tau_{rz} = 0 \quad \tau_{r\theta} = 0,
\]
at the top and bottom surfaces, where \( z = h \) and \( z = 0 \)
\[
\sigma_z = f_{cz} \quad \tau_{rz} = 0 \quad \tau_{0z} = 0.
\]

Due to symmetry, the displacement component \( v \) in the \( \theta \) direction will vanish everywhere and all stresses, strains, and displacements are independent of \( \theta \). Due to the uniformity of the axial load \( \sigma_z \) will be equal to \( f_{cz} \) throughout the thickness and the height of the cylinder, the stresses and strains are independent of \( z \). The shear stresses \( \tau_{r\theta} \), \( \tau_{0z} \), \( \tau_{rz} \) and the corresponding strains \( \gamma_{r\theta} \), \( \gamma_{0z} \), \( \gamma_{rz} \) vanish also everywhere. The remaining unknowns are the two displacements \( u \) and \( w \), the two stresses \( \sigma_r \) and \( \sigma_\theta \), and three strains \( \varepsilon_r \), \( \varepsilon_\theta \), and \( \varepsilon_z \). Thus, the only equilibrium equation is,

\[
\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \quad (6.1)
\]

The three strain-displacement equations are,
\[
\varepsilon_r = \frac{du}{dr}, \quad \varepsilon_\theta = \frac{u}{r}, \quad \text{and} \quad \varepsilon_z = \frac{dw}{dz}
\] (6.2)

and the three stress-strain equations are,

\[
\varepsilon_r = \frac{1}{E} \left[ \sigma_r - \nu (\sigma_\theta + \sigma_z) \right]
\]

\[
\varepsilon_\theta = \frac{1}{E} \left[ \sigma_\theta - \nu (\sigma_r + \sigma_z) \right]
\] (6.3)

\[
\varepsilon_z = \frac{1}{E} \left[ \sigma_z - \nu (\sigma_r + \sigma_\theta) \right]
\]

The boundary conditions are

\[
\sigma_r = -\sigma_{ra} \quad \text{at} \quad r = a
\]

\[
\sigma_r = 0 \quad \text{at} \quad r = b
\] (6.4)

If the transverse stresses and strain are expressed as function of radial displacement, \(u\), equation 6.1 gives the following differential equation for \(u(r)\)

\[
\frac{d^2 u}{dr^2} + \frac{1}{r} \frac{du}{dr} - \frac{u}{r^2} = 0
\] (6.5)

The general solution to equation 6.5 is

\[
u = Ar + \frac{B}{r}
\] (6.6)

where \(A\) and \(B\) are the constants of integration which are evaluated by requiring the radial stress \(\sigma_r\) to meet the boundary conditions given in 6.4. Finally, after replacing the values of \(A\) and \(B\), one can obtain the following expressions for the radial stresses, tangential stresses, and radial displacement as follows:
Several investigators have formulated equations for transfer length based on different concepts of bond between reinforcement and concrete. Hoyer and Friedrich (1939) (cited in Lin T. Y. 1982) considered the anchorage to be a result of swelling of the wires due to the Poisson's ratio effect. They developed the following equation for transfer length, $L_t$, based on an elastic analysis.

$$ L_t = \frac{d}{2\mu} \left( \nu_c + 1 \right) \left( \frac{n}{\nu_p} - \frac{f_{pi}}{E_c} \right) \left( \frac{f_{pe}}{2f_{pi} - f_{pe}} \right) $$  \hspace{1cm} (6.8)

where $\mu$ = coefficient of friction between reinforcement and concrete,

$\nu_c, \nu_p$ = Poisson's ratio for concrete and reinforcement respectively,

$d$ = diameter of tendon,

$E_c, E_p$ = modules of elasticity of concrete and reinforcement, respectively,

$n$ = modular ratio = $(E_p / E_c)$,

$f_{pi}, f_{pe}$ = initial and effective prestress in reinforcement, respectively.

Janney (1954) considered the transfer of prestress to be purely frictional. He applied thick-walled cylinder theory considering steel as a solid cylinder and concrete as
a hollow concentric cylinder. Based on this model Janney developed the following expressions for the interface stress, \( \sigma_{ra} \), and transfer length of steel wires, as:

\[
\sigma_{ra} = \frac{(f_{pi} - f_{pe})v_p}{1 + n(1 + v_c)} \tag{6.9}
\]

\[
l = \frac{d}{4\mu\nu_p} \left\{1 + n(1 + v_c)\right\} \ln\left(\frac{f_{pi}}{f_{pi} - f_{pe}}\right) \tag{6.10}
\]

where \( l \) = distance from transfer onset to the point having an effective prestress \( f_{pe} \). The other notations are the same as before. Janney observed that the tangential tensile stresses in the concrete due to swelling of the wire would exceed the elastic range and that therefore elastic analysis is not appropriate. Microscopic examination of the exposed concrete and steel interface near the ends of the transfer prisms showed no indication of tensile failure of the paste in spite of the high computed values of tensile stress. He concluded that sufficient yielding of the concrete took place to relieve the high tensile stresses. Janney also reported the results of a transfer length study of beams pretensioned with steel wires. By comparing the observed stress distribution in the wire with the theoretical stress distribution, he concluded that the two are very similar in distribution but not in magnitude. Therefore he assumed that the friction bond mechanism seems reasonable. In this analysis, the use of a large value for the outer radius of the concrete cylinder does not reflect most practical situations. Furthermore, in his derivation axial stresses in the concrete were not considered. Overall predicted transfer lengths were smaller than actual observed values.
Weerasekera (1991) developed a theory using the principles of solid mechanics to predict the transfer length of steel strands. The procedure adopted recognizes the presence of a disturbed cracked zone around the strand. The concrete in the affected region is analyzed as an anisotropic elastic material. The data obtained from the parameteric study were translated into the following equation to predict the transfer length of steel strands, $L_t$,:

$$L_t = \frac{f_{pi} A_b}{K_f f_t C_y}$$

(6.11)

where $f_{pi}$ = initial prestress,

$A_b$ = the cross section area of strand,

$f_t$ = the tensile strength of concrete,

$C_y$ = the clear concrete cover,

$$K_f = B_0 - \frac{f_{pi} A_b}{F_0 f_{ci} m_0}.$$  

Weerasekera proposed values for the constants $B_0$, $F_0$, and $m_0$ as 3.055, 52320, and 0.28, respectively. He reported that the analytical procedure, which has been developed for a cylindrical model, and the prediction equation, which has been derived from theses results, should not be considered final. Further study is needed to calibrate this model and equation with additional test results.

The theories previously proposed have been neither convincing nor successful in predicting transfer length. The absence of a valid theory to complement experiments has
stimulated the search for rational solutions to the transfer bond problem. Therefore it was considered important to formulate a rational theoretical basis.

6.4 BOND MECHANICS IN THE TRANSFER ZONE

In the prestress transfer region the bond is enhanced by high radial pressures resulting from the increase in the reinforcement diameter after release of pretension. This wedging action due to the lateral swelling of the reinforcement described by Poisson's ratio (Hoyer effect) results in high friction bond strengths. These are illustrated in Figure 6.2. The contribution of friction to bond can be written as:

\[ u = \mu \sigma_{ra} \]  

(6.12)

where \( u \) = the bond stress,

\( \mu \) = the coefficient of friction,

\( \sigma_{ra} \) = the interface pressure.

Coefficient of friction values varying from 0.2 to 0.6 for prestressing steel wires were used by Janney (1954) in his theoretical study of transfer length of steel wires. Watanabe (1965) reported measured values for the coefficient of friction of 5-mm-diameter prestressing steel wires of 0.16 and 0.39 for clean and rusted wires, respectively. Weerasekera, 1991, used a coefficient of friction of 0.75 between 7-wire steel strands and concrete in the theoretical analysis of transfer length of 7-wire steel strand. By calculating the interface pressure, \( \sigma_{ra} \), using the proposed approach and knowing the coefficient of friction, \( \mu \), the bond stress can be calculated using equation 6.12.
6.5 MATERIAL PROPERTIES

6.5.1 Concrete

As the radial compressive stresses in concrete due to swelling of tendon normally do not exceed the elastic limit, which is about 60% the concrete compressive strength at release, \( f'_{ci} \), the concrete can be modeled as a linear elastic material. The modules of elasticity, \( E_c \), of normal density concrete having compressive strengths between 20 and 40 MPa can be determined as (CSA Standard A23.3-94):

\[
E_c = 4500 \sqrt{f'_{ci}}
\]  
(6.13)

The stress-strain response of concrete in tension is assumed to be linear prior to cracking with the same elastic modules, \( E_c \). The concrete tensile strength is assumed to equal the rupture strength \( f_r = 0.6 \sqrt{f'_{ci}} \) (CSA Standard A23.3-94). Poisson’s ratio of concrete is usually found to be in the range 0.15 to 0.20 (Park and Paulay, 1975).

6.5.2 Reinforcement

The theoretical model is used for analyzing the transfer bond of steel wires and strands, and Leadline bars. The steel reinforcement is assumed not to be stressed beyond the elastic limit. The modules of elasticity in the longitudinal and transverse direction of steel wires, \( E_p \), is taken as 200 000 MPa. Poisson’s ratio for steel wires, \( v_p \), of value equal 0.3 is used in the analysis.

The carbon fibre content, \( v_f \), in a Leadline bar is 65% by volume. The epoxy matrix content, \( v_m \), is 35% by volume. As the carbon fibres are oriented in only the longitudinal direction, the material properties in the longitudinal direction are expected to be different from those in the transverse direction. In a composite material, the
longitudinal modules, $E_p$, can be determined using the rule of mixtures, which is based on the assumption of equal strains in the fibres and the matrix, as follows, (Gibson, 1994):

$$E_p = E_f \nu_f + E_m \nu_m$$

(6.14)

where $E_f$ = the modules of elasticity of the carbon fibres in the longitudinal direction, $E_m$ = the modules of elasticity of the epoxy matrix.

The tensile strength of the carbon fibres is assumed to be 3000 MPa and the modules of elasticity, $E_f$, to range from 230 to 300 GPa (FIP, 1992). The tensile strength of the epoxy matrix lies between 40 and 100 MPa and the modules of elasticity, $E_m$, varies from 2.75 to 4.1 GPa (Mallick, 1993). The modules of elasticity, $E_p$, of Leadline in the longitudinal direction, as calculated using equation 6.14, was found to be in the range of 150 to 190 GPa. The modules of elasticity of the Leadline bars, obtained during tension tests, varies from 147 to 190 GPa. The reported value of $E_p$ by the Leadline's manufacturer is 147 GPa. Poisson’s ratio value for Leadline, reported by the manufacturer and measured experimentally, is about 0.30.

If the FRP reinforcement is subjected to a transverse radial stress, $\sigma_{ra}$, as shown in Figure 6.2, the response is governed by the effective transverse modules, $E_{pr}$. Geometric compatibility requires that the total transverse composite displacement must equal the sum of the corresponding transverse displacements in the fibers and the matrix. The transverse modules of FRP reinforcement, $E_{pr}$, can be estimated using one of the following theoretical and semi-empirical models, (Gibson, 1994):
\[
\frac{1}{E_{pr}} = \frac{v_f}{E_{fr}} + \frac{v_m}{E_m}
\]  
(6.15)

\[
E_{pr} = E_m \left\{ \left(1 - \sqrt[3]{v_f} \right) + \frac{\sqrt[3]{v_f}}{1 - \sqrt[3]{v_f} (1 - E_m / E_{fr})} \right\}
\]  
(6.16)

\[
E_{pr} = E_m \frac{1 + \xi \eta v_f}{1 - \eta v_f}
\]  
where \( \eta = \frac{(E_f / E_m) - 1}{(E_f / E_m) + \xi} \)
(6.17)

\[
\frac{1}{E_{pr}} = \frac{1}{v_f} + \frac{\eta_2 v_m}{E_f E_m}
\]  
(6.18)

where \( E_{fr} \) = the elasticity modules of fibres in the transverse direction,

\( \xi \) = a curve fitting parameter of value equal 2.0,

\( \eta_2 = 0.5 \) was found to yield accurate predictions based on comparisons with experimental data.

Using these four models, the transverse modules (\( E_{pr} \)) for Leadline was found to be in the range 13.7 to 28.4 GPa.

### 6.6 UNCRACKED ANALYSIS

In this analysis, the reinforcement is treated as solid cylinder of radius \( a \) and the concrete as a hollow cylinder of inner radius \( a \) and outer radius \( b \) equal to the smaller concrete cover or one-half the distance to the closest longitudinal reinforcement. The outer surface of the concrete cylinder is assumed to behave as a free surface. At transfer, the reduction of the longitudinal stresses causes swelling of reinforcement, so a pressure develops at the interface. Also the drying shrinkage of concrete, \( \varepsilon_{sh} \), increases the
interface pressure even before release of prestress. The release of prestress develops longitudinal compressive stresses in the concrete at the level of the tendon, \( f_{ce} \), which can reduce the contact pressure as a result of Poisson’s effect of concrete. Using thick-walled cylinder theory, expressions for stresses, strains, and displacements can be developed by ensuring equilibrium and compatibility, and imposing boundary conditions and constitutive relationships that represent the material behavior.

The basic requirement is the compatibility of displacements in the radial direction at the tendon-concrete interface given by:

\[
\Delta_{p1} + \Delta_{p2} = \Delta_{c1} + \Delta_{c2} + \Delta_{c3} \tag{6.19}
\]

where:

\( \Delta_{p1} \) = increase in radius of reinforcement due to reduction in longitudinal stress from initial prestress \( f_{pi} \) to an effective value \( f_{pe} \),

\[
\Delta_{p1} = \frac{(f_{pi} - f_{pe})}{E_p} \nu_p \ a \tag{6.20}
\]

\( \Delta_{p2} \) = reduction in radius of reinforcement due to the uniform radial compression \( \sigma_{ra} \),

\[
\Delta_{p2} = - \frac{\sigma_{ra}(1 - \nu_p)}{E_{pr}} \ a \tag{6.21}
\]

\( \Delta_{c1} \) = increase of the inner radius of concrete cylinder due to the interface pressure \( \sigma_{ra} \),

\[
\Delta_{c1} = \frac{\sigma_{ra}}{E_c(1/a^2 - 1/b^2)} \left[ \frac{(1 - \nu_c)}{b^2} + \frac{(1 + \nu_c)}{a^2} \right] \tag{6.22}
\]
\( \Delta_{c2} = \text{increase of the inner radius of concrete cylinder due to the longitudinal compressive stress at the level of tendon, } f_{cz}, \)

\[
\Delta_{c2} = -\frac{v_c f_{cz} a}{E_c} \tag{6.23}
\]

\( \Delta_{c3} = \text{reduction in the inner radius of concrete cylinder due to drying shrinkage, } \varepsilon_{sh}. \)

\[
\Delta_{c3} = \varepsilon_{sh} a \tag{6.24}
\]

Using these values of displacements given by equations 6.20 through 6.24, an expression for the interface pressure, \( \sigma_{ra} \), can be developed using equation 6.19 as follows:

\[
\sigma_{ra} = \left\{ \frac{(f_{pi} - f_{pe})v_p}{E_p} + \frac{v_c f_{cz} - \varepsilon_{sh}}{E_c} \right\} \left[ \frac{1}{E_{pr}} + \frac{C_1}{E_c} \right]
\]

where:

\[
C_1 = \frac{1}{1/a^2 - 1/b^2} \left\{ \frac{(1 - v_c)}{b^2} + \frac{(1 + v_c)}{a^2} \right\}
\]

\( f_{pi} = \text{the initial prestress,} \)

\( f_{pe} = \text{effective prestress in the transfer zone, varies from zero at the transfer onset to maximum value } f_{pi} \text{ at the end of transfer zone,} \)

\( f_{cz} = \text{compressive stress in concrete at the level of tendon (-ve value),} \)

\( \varepsilon_{sh} = \text{drying shrinkage coefficient (-ve value).} \)

If the values \( 1/b^2 \), \( f_{cz} \), and \( \varepsilon_{sh} \) in equation 6.25 are neglected, this equation for interface pressure will be similar to equation 6.9.
Shrinkage of concrete depends on the amount of water in the concrete mix, the quality of aggregate, relative humidity, ratio of volume of concrete to surface area, and method of curing. The following approximate expression can be used for estimating the shrinkage coefficient, $\varepsilon_{sh}$, for moist-cured concrete (cited in Collins and Mitchell, 1991):

$$
\varepsilon_{sh} = - k_s k_h \left( \frac{t}{t + 35} \right) 0.51 \times 10^{-3}
$$

(6.26)

where $t$ = the time in days during which concrete has been exposed to drying,

$k_s$ = factor to account for the ratio of volume of concrete to surface area with value of 1.0 when the ratio (volume/surface area) equals 40 mm,

$k_h$ = factor to account for humidity with value equal 1.0 at relative humidity of 70%.

The concrete compressive stress, $f_{cz}$, at the level of tendon varies from zero at the transfer onset to a maximum value at the end of transfer zone, and can be estimated using the following equation:

$$
f_{cz} = - \frac{f_{pe} A_{pt}}{A_c} \left( \frac{f_{pe} A_{pt}}{I_g} \right) e_{cc}^2
$$

(6.27)

where $A_{pt}$ = the total area of prestressing reinforcement,

$A_c$ = cross section area of concrete member;

$f_{pe}$ = effective prestress at the section considered;

$e_{cc}$ = eccentricity of the prestress force; and

$I_g$ = moment of inertia of concrete section.
To demonstrate the stress distribution in the uncracked analysis, the following numerical example is considered:

- steel strand diameter $d = 12.7$ mm;
- initial prestress $f_{pi} = 1300$ MPa;
- concrete cover measured from the center of the tendon $b = 46.35$ mm;
- modules of elasticity in the longitudinal, $E_p$, and transverse directions for the individual steel wires, $E_{pr}$, are 200 000 MPa;
- Poisson’s ratio of individual steel wire $\nu_p = 0.3$ and Poisson’s ratio of concrete $\nu_c = 0.15$; and concrete strength $f_{ci} = 30$ MPa.

The interface pressure at the transfer onset was found to be $\sigma_{ra} = 37.5$ MPa due to swelling of the tendon only. Using the elastic theory of a thick-walled cylinder, the stress distributions in the transverse direction for both the radial and circumferential components of stresses for a slice taken at the free end are calculated using equation 6.7 and given in Figure 6.3. The results of this example are compared to the results given by Weerasekera, 1991, for the same conditions and are found to be identical.

The analysis shows the tensile stresses in the circumferential direction attain values approximately ten times the concrete tensile strength and the compressive stresses in the radial direction also exceed the proportional limit. It is therefore clear that the concrete must crack in the radial direction to accommodate the swelling of the released tendon. This cracking will alter the stiffness of concrete in the damaged region around the tendon.
6.7 CRACKED ANALYSIS

6.7.1 Behavior of Concrete in Tension

The stress-strain response of concrete in uniaxial tension is nearly linear up to cracking; cracking occurs at relatively low stresses. Using a very stiff testing machine it is possible to detect post-cracking tensile resistance at very small crack widths. In trying to understand how tensile stresses can be transmitted across a crack it is important to recognize that the crack surfaces are locally very rough and that the crack widths are extremely small in comparison to the irregularities on the surface of the crack. The jamming of the rough surfaces of the crack makes it possible to transmit some tension for crack widths less than 0.05 mm (Collins and Mitchell, 1991).

Gopalaratnam and Shah, 1985, proposed the following relationship between post-peak stress and crack width:

\[
\sigma = \sigma_p \left( e^{-k w \lambda} \right)
\]  

(6.28)

where \( \sigma \) = post-peak tensile stress,

\( \sigma_p \) = peak value of \( \sigma \),

\( w \) = crack width.

A value of \( \lambda = 1.01 \) was assumed by the authors. For this value of \( \lambda \), a value of \( k = 1.544 \times 10^{-3} \) (when \( w \) was expressed in \( \mu \text{in.} \)) gave the best fit to the experimental values.

A new, simple relationship between post-peak stress and crack width is proposed here as:

\[
\sigma = \sigma_p \left( \frac{0.05 - w}{0.05} \right)^2
\]

(6.29)
Figure 6.4 shows that the proposed relationship for post-peak softening response is in good agreement with that of Gopalaratnam and Shah. The assumed equation has the advantage of simplicity when incorporated into the equilibrium equation 6.1.

6.7.2 Nature of the Cracked and Fractured Zones

Hillerborg, 1983 (cited in Weerasekera, 1991) examined the region in the vicinity of a crack in a reinforced concrete beam as shown in Figure 6.5. It can be seen that there is no well-defined crack tip. The tensile stress drops gradually from its peak value, $f_t$, to zero. The end of the tensile stress zone defines the tip of the real crack. The effective tip of the crack is defined as the point corresponding to the peak value of stress to $f_t$. Based on the extremely high circumferential tensile stress in the concrete as determined from the uncracked analysis, similar cracking in concrete cover around the tendon can be expected to occur in the radial direction. The situation is illustrated in Figure 6.6.

The wall of the concrete cylinder is divided into three zones as shown in Figure 6.6. The first zone is the real crack zone adjacent to the tendon with thickness $(x-a)$. This zone exists only if the crack width at the tendon-concrete interface is greater than 0.05 mm. This zone is defined as the distance from the tendon surface to a point on the radial crack at $r = x$ where the crack width is 0.05 mm. No circumferential tensile stresses are transferred across the crack in this zone ($\sigma_\theta = 0.0$) as the crack width is greater than 0.05 mm. The second zone is known as the fracture zone or process zone. This zone extends from the end of the real crack zone at $r = x$ to the effective crack tip at $r = e$. The circumferential tensile stresses, $\sigma_\theta$, that can be transferred across the crack vary from zero
at \( r = x \) and \( w = 0.05 \text{ mm} \) to a maximum value of \( f_i \) at the effective crack tip where \( r = e \) and the effective crack width \( w = 0 \).

The third zone is the uncracked zone which extends from the effective crack tip at \( r = e \) to the outer free surface of the concrete at \( r = b \). The circumferential tensile stresses in this zone vary from a maximum value of \( f_i \) at the effective crack tip and decrease with increasing of radius, \( r \), according to the elastic theory of a thick-walled cylinder.

It is assumed here that the crack width, \( w_a \), at the tendon-concrete interface depends on the amount of swelling of tendon and the assumed number of radial cracks, \( N \). Therefore, the crack width can be determined as follow:

\[
W_a = \frac{2 \pi a}{N} \left( \frac{f_{pi} - f_{pe}}{E_p} \nu_p - \frac{\sigma_{ra}(1 - \nu_p)}{E_{pr}} \right)
\]  \( (6.30) \)

### 6.7.3 Compatibility Condition

Cracked analysis is similar to uncracked analysis, previously discussed in section 6.6, except for the procedure to determine the inner surface displacement, \( \Delta_{c1} \), of the concrete cylinder, due to an inner pressure \( \sigma_{ra} \). The term \( \Delta_{c1} \) in equation 6.19, in this case, is the summation of the deformation of the real crack zone \( (\Delta_{cr}) \), the deformation of the fracture zone \( (\Delta_{fr}) \), and the radial displacement \( (u_{re}) \) at \( r = e \). The compatibility equation can be written as:

\[
\Delta_{p1} + \Delta_{p2} = (\Delta_{cr} + \Delta_{fr} + u_{re}) + \Delta_{c2} + \Delta_{c3}
\]  \( (6.31) \)

where \( \Delta_{p1} \), \( \Delta_{p2} \), \( \Delta_{c2} \), and \( \Delta_{c3} \) can be evaluated as before in section 6.6. The following sections give the detailed procedure to determine the radial deformation of the three concrete zones, \( \Delta_{cr} \), \( \Delta_{fr} \), and \( u_{re} \).
6.7.4 Deformation of the Real Crack Zone, $\Delta_{cr}$

The real crack zone is characterized by the condition that no circumferential tensile stresses can be transmitted across the crack ($\sigma_0 = 0$) as the crack width is greater than 0.05 mm. The equilibrium equation 6.1 is reduced to:

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r}{r} = 0.0$$

(6.32)

Integrating this equation and applying the boundary condition that at $r = a$ the radial stress $\sigma_r$ equals the interface pressure $\sigma_{ra}$, results in this expression for radial stress $\sigma_r$:

$$\sigma_r = \frac{\sigma_{ra}}{r}$$

(6.33)

The deformation of the real crack zone, $\Delta_{cr}$, can be calculated as:

$$\Delta_{cr} = \frac{x}{E_c} \sigma_{ra} = \frac{\sigma_{ra}}{E_c} ln\left(\frac{x}{a}\right)$$

(6.34)

6.7.5 Deformation of the Fracture Zone, $\Delta_{fr}$

Case A: This case is shown in Figure 6.6. The circumferential tensile stress that can be transmitted across the crack is governed by the assumed relationship between post-peak stress and the crack width which is given in equation 6.29. The circumferential stress, $\sigma_0$, can be expressed in terms of radius, $r$, and the ultimate tensile stress of concrete, $f_t$, as shown in Figure 6.6:

$$\sigma_0 = f_t \left(\frac{r - x}{e - x}\right)^2 = c_3 \left(r^2 - 2xr + x^2\right)$$

(6.35)
where \( c_3 = \frac{f_i}{(e-x)^2} \). If we consider the fact that the radial stresses, \( \sigma_r \), are always compressive and the circumferential stresses are always tensile in this problem, the equilibrium equation 6.1 can be rewritten as:

\[
-\frac{d\sigma_r}{dr} = \frac{\sigma_r + \sigma_\theta}{r}
\]  (6.36)

By replacing \( \sigma_\theta \) in equation 6.36 by the expression given by equation 6.35, the following equation is obtained:

\[
-(\sigma_r r r + r d\sigma_r) = \left( c_3 r^2 - 2 c_3 x r + c_3 x^2 \right) dr
\]  (6.37)

Integrating the two sides of equation 6.37 and calculating the integration constant using the boundary condition of \( \sigma_r = \frac{\sigma_r a}{x} \) at \( r = x \), the following expression for radial stress \( \sigma_r \) results:

\[
\sigma_r = \left[ -\frac{c_3 r^2}{3} + c_3 x r - c_3 x^2 + \frac{c_3 x^3}{3 r} + \frac{\sigma_r a}{r} \right]
\]  (6.38)

Total deformation, \( \Delta_{fr} \), of the fracture zone in the radial direction is the integration of radial strain, \( \varepsilon_r \), from \( r = x \) to \( r = e \):

\[
\Delta_{fr} = \int_0^e \varepsilon_r dr = \int_0^e \left( \frac{\sigma_r}{E_c} + \nu_c \frac{\sigma_\theta}{E_c} \right) dr
\]  (6.39)

\[
\Delta_{fr} = k_1 + \frac{1}{E_c} \left\{ \sigma_\theta a \ln \frac{e}{x} \right\}
\]  (6.40)
where  
\[ k_i = \frac{1}{E_c} \left\{ \frac{-1}{9} c_3(e^3 - x^3) + \frac{1}{2} c_3 x(e^2 - x^2) - c_3 x^2(e-x) + \frac{1}{3} c_3 x^3 \ln \frac{e}{x} \right\} \]

\[ + \frac{v_c}{E_c} \left\{ \frac{1}{3} c_3(e^3 - x^3) - c_3 x(e^2 - x^2) + c_3 x^2(e-x) \right\} \]

**Case B.** If the crack width at the tendon-concrete interface, \( w_a \), is less than or equal to 0.05 mm, the real crack zone will not be formed and the developed circumferential tensile stress will be according to equation 6.29. The crack width, \( w \), at any point can be related to the radius, \( r \), and the crack width, \( w_a \), at the interface as follows:

\[ w = w_a \frac{e-r}{e-a} \quad (6.41) \]

Using this value of \( w \) in equation 6.29, one obtains the following:

\[ \sigma_0 = f_i + k_3(e-r) + k_4(e-r)^2 \quad (6.42) \]

where:

\[ k_3 = -\frac{2 w_a f_i}{0.05(e-a)} \]

\[ k_4 = \frac{w_a^2 f_i}{(0.05)^2 (e-a)^2} \]

By replacing \( \sigma_0 \) in the equilibrium equation 6.36 by equation 6.42, integrating the new equilibrium equation, and determining the integration constant using the boundary condition that, at \( r = a \), the radial stress \( \sigma_r = \sigma_a \), the following expression for radial stress, \( \sigma_r \), can be developed:

\[ \sigma_r = -f_i - k_3 e + k_3 r / 2 - k_4 e^2 + k_4 e r - k_4 r^2 / 3 + a \sigma_a / r + k_5 / r \quad (6.43) \]
The deformation, \( \Delta_f \), for this case can be calculated as follows:

\[
\Delta_f = \int_a^e \epsilon_r \, dr = \int_a^e \left( \frac{\sigma_r}{E_c} + \nu_c \frac{\sigma_\theta}{E_c} \right) \, dr \tag{6.44}
\]

\[
\Delta_f = k_6 + \frac{1}{E_c} \left\{ \sigma_{ra} a \ln \frac{e}{a} \right\} \tag{6.45}
\]

where:

\[
k_6 = \frac{1}{E_c} \left\{ -f_t(e-a) - k_3 e(e-a) + \frac{k_3}{4} (e^2 - a^2) - \frac{1}{9} k_4 (e^3 - a^3) \right. \\
+ \frac{1}{2} k_4 e(e^2 - a^2) - k_4 e^2(e-a) + k_5 \ln \frac{e}{a} \\
+ \frac{\nu_c}{E_c} \left\{ f_t(e-a) + k_3 e(e-a) - \frac{k_3}{2} (e^2 - a^2) + k_4 e^2(e-a) \right. \\
+ \frac{1}{3} k_4 (e^3 - a^3) - k_4 e(e^2 - a^2) \right. \]

6.7.6 Radial Displacement of the Uncracked Zone, \( u_{re} \).

Given the condition that the circumferential tensile stress \( \sigma_\theta = f_t \) at the inner surface of the uncracked zone at \( r = e \), the radial stress, \( \sigma_{re} \), at this surface can be calculated as:

\[
\sigma_{re} = f_t \frac{b^2 - e^2}{b^2 + e^2} \tag{6.46}
\]

The radial displacement, \( u_{re} \), at \( r = e \) can be calculated using equation 6.2:
where \( \varepsilon_{\theta e} \) is the circumferential strain at \( r = e \).

6.7.7 Contact pressure, \( \sigma_{ra} \).

Based on the known displacement components of the compatibility equation 6.31, the expression for the contact pressure, \( \sigma_{ra} \), at the tendon-concrete interface can be developed for the case of cracked analysis as follows:

\[
\sigma_{ra} = \frac{1}{k_2} \left( \Delta_{\rho 1} - k_1 - u_{re} - \Delta_{c2} - \Delta_{c3} \right)
\]

(6.48)

where \( k_2 = \frac{a \ln (x/a) + a \ln (e/x)}{E_c} + a \left( \frac{1}{\nu} + \frac{1}{E_{pr}} \right) \).

6.8 PREDICTION OF TRANSFER LENGTH

The proposed step-by-step analysis is based on 200 stress increments, \( \Delta f_{pe} = f_{pl}/200 \), starting from the free end until the tendon stress reaches the initial prestress, \( f_{pi} \), as shown in Figure 6.7. Given the effective tendon stress at the 200 points within the transfer zone, the contact pressure, \( \sigma_{ra} \), at each point is determined using the uncracked analysis method. In the zone where the circumferential tensile stress \( \sigma_{\theta} \) at the interface exceeds the tensile strength of the concrete, \( f_t \), the analysis is modified to consider the concrete properties after cracking. The incremental distance, \( \Delta z \), required to transfer an incremental stress \( \Delta f_{pe} \) to the concrete can be calculated as follows:

\[
\Delta z = \frac{A \Delta f_{pe}}{\pi d \mu \sigma_{ra}}
\]

(6.49)

where \( A = \) the cross-sectional area of a single tendon; and \( \mu = \) the coefficient of friction between reinforcement and concrete.
Estimation of the transfer length in this theoretical study is based on the 95%-constant-strain method presented in chapter 5. Therefore, the transfer length, \( L_t \), can be determined as the summation of the calculated increments of \( \Delta z \) from the transfer onset at \( f_{pe} = 0.0 \) to \( f_{pe} = 0.95 f_{pi} \) as shown in Figure 6.7.

6.9 LONGITUDINAL AND TRANSVERSE STRESSES

Figure 6.8 shows the longitudinal stress build-up along the transfer zone in the tendon using the data obtained from the following numerical example:

Steel strand 12.7 mm diameter:

prestress \( f_{pi} = 1300 \) MPa;

concrete strength at time of release \( f'_{ci} = 30 \) MPa;

assumed number of radial cracks \( N = 1 \);

concrete cover \( b = 60 \) mm;

coefficient of friction \( \mu = 0.75 \) (Wearersekera, 1991);

modules of elasticity of steel strand \( E_p = E_{pr} = 200000 \) MPa;

Poisson’s ratio \( \nu_p = 0.3 \) and \( \nu_c = 0.15 \); and

shrinkage coefficient \( \varepsilon_{sh} = 0.0 \).

The results of the analysis showed that the transfer length is 490 mm.

It should be noted that the following three different cases of analysis could be used at different locations along the transfer zone, as shown in Figure 6.8.

Case I This case is found near the end of transfer zone where the tendon swelling and therefore the contact pressure, is relatively small. Only the elastic uncracked analysis is applied as the tangential stress, \( \sigma_0 \), at the interface is less than the tensile strength of
concrete, $f_i$. An example of the distribution of the radial stress, $\sigma_r$, and the tangential stress, $\sigma_\theta$, in the transverse direction, determined using equation 6.7, is shown in Figure 6.9.

**Case II** Here the concrete cylinder around the tendon consists of fracture zone around the tendon surrounded by the uncracked zone, since the crack width at the interface is less than or equal to 0.05 mm. The radial and tangential stress distribution within the fracture zone are calculated using equations 6.43 and 6.42, respectively. The continuation of $\sigma_r$ and $\sigma_\theta$ within the uncracked zone are determined using the elastic theory of a thick-walled cylinder as shown in Figure 6.10.

**Case III** Figure 6.11 shows that in case III the concrete cylinder contains the real crack zone, the fracture zone, and the uncracked zone. In the real crack zone, the tangential stress $\sigma_\theta$ is zero and the radial stress can be determined using equation 6.33. In the fracture zone (process zone), the tangential stress, $\sigma_\theta$, builds up from zero at $r = x$ to a maximum value $f_i$ at the effective crack tip and can be calculated using equation 6.35. The radial stress, $\sigma_r$, within the fracture zone, can be calculated using equation 6.38. The continuation of $\sigma_r$ and $\sigma_\theta$ within the uncracked zone are determined using the elastic theory of the thick-walled cylinder.

**6.10 EVALUATION OF THE THEORETICAL MODEL**

The proposed theoretical model was used to predict the transfer length of prestressing steel wires, steel strands, and Leadline to compare with the data reported in the literature.
6.10.1 Coefficient of Friction between Reinforcement and Concrete

The coefficient of friction between the reinforcement and concrete is a critical parameter in the theoretical analysis. The coefficient of friction between concrete and both steel strands and Leadline bars was determined experimentally using the test setup shown in Figures 6.12a and 6.12b, (ASTM, G115-93), (FHWA-Rd, 1994). The precast concrete block dimensions were 150 mm by 150 mm by 500 mm. The edges of the concrete block were rounded. Two pieces of reinforcement were placed parallel to each other and clamped to the steel base. The bottom surface of the concrete block was cleaned and placed to bear on the two pieces of reinforcement after attaching a thin layer of cement paste to the interface surface to assure complete contact between reinforcement and concrete. The precast concrete block and the cement paste layer were kept in moist curing for three days. The steel base was leveled so that the friction force at impending motion was essentially horizontal. Force was applied through a horizontal fine steel wire attached to the concrete block at the level of the reinforcement-concrete interface and attached to an electronic digital scale. The relative movement between the reinforcement and concrete block was monitored using a mechanical dial gauge. The procedure was as follows:

1) Two pieces of reinforcement were positioned and brushed lightly to remove any loose grit.

2) A thin layer of cement paste was attached to the clean surface of the concrete block which was then positioned on the two pieces of reinforcement.

3) The assembling was moist cured for three days.
4) Force was applied in the fine steel wire by gradual movement of the steel base in the direction shown in Figure 6.12a.

5) The maximum load measured by the electronic digital scale at impending motion was equal to the friction force.

6) the test was repeated twelve times with the same concrete block. The first two measurements were discarded and ten measurements were averaged to provide the measure of frictional force. The frictional force was divided by the known weight of the concrete block to obtain the static coefficient of friction between reinforcement and concrete.

6.10.2 Transfer Length of Prestressing Steel Wires

For the case of 5-mm prestressing steel wire with friction coefficients varying from $\mu = 0.2$ to $0.3$, a prestress level $f_{pl} = 1100$ MPa (70% of the nominal strength of indented wires), and the same material properties as in the previous example in section 6.9, the transfer length was found to be in the range of $110d$ to $165d$, where $d =$ the wire diameter. The CSA code and JSCE suggest that the transfer length be taken as $100d$. The CEB-FIP code, on the other hand, suggests that the transfer length may vary between $100d$ and $140d$. The Architectural Institute of Japan (AIJ), (cited in Weerasekera, 1991), recommends a transfer length of $80d$ and $180d$ for rusty and non-rusty wires, respectively. The Australian code, 1988, (cited in Weerasekera, 1991) recommends a values of transfer length of indented wires of $100d$ for a concrete strength $f_{ci} \geq 32$ MPa and $175d$ for $f_{ci} < 32$ MPa.
6.10.3 Transfer Length of Steel Strands

For the 12.7 mm diameter 7-wire steel strand considered in the previous numerical example in section 6.9 with the measured coefficient of friction in this study \( \mu = 0.55 \), the predicted transfer length using the theoretical model was found to be 53d. The experimental model proposed in equation 5.2 predicts a transfer length of 55d for the same case. Also the ACI 318-95 equation estimates a transfer length of 53d for this case (assuming \( f_{pe} = 0.85 f_{pi} \)).

The transfer length results of the proposed theoretical model for both prestressing wires and strands are found to be comparable to the data obtained from different codes.

6.10.4 Transfer Length of Leadline

Based on the validity and reliability of the proposed theoretical model to predict the transfer length of steel wires and strands, this section uses the same analysis to predict the transfer bond for the Leadline bars used in this investigation. The test results of three different concrete blocks showed that the coefficient of friction varies from 0.55 to 0.65 with an average value of 0.6. A parameteric study carried out using prestress levels varying between 0.6 and 0.7 of the guaranteed ultimate strength, concrete strengths varying from 25 and 35 MPa, and different values of coefficients of friction showed that a value of the coefficient of friction, \( \mu = 0.45 \), gives the best correlation with the experimental results and experimental model presented in chapter 5.

The measured and theoretical transfer lengths of specimens prestressed with Leadline bars without steel confinement are compared in Table 6.1. The theoretical results are based on the following input data for each specimen:
Measured concrete compressive strength, $f'_{ci}$, at time of release;

diameter of Leadline bar $d = 8$ mm;

measured prestress, $f_{pi}$;

assumed number of radial cracks $N = 2$;

coefficient of friction $\mu = 0.45$;

modules of elasticity in the longitudinal direction, $E_p = 160$ GPa;

modules of elasticity in the transverse direction $E_{pr} = 28$ GPa;

Poisson’s ratio of Leadline, $\nu_p = 0.3$; and

Poisson’s ratio of concrete, $\nu_c = 0.15$.

The experimental transfer length results are found to be in the range of 0.86 to 1.11 of the theoretical results as shown in Table 6.1.

The theoretical and experimental stress curves for all specimens prestressed with Leadline bars are shown in Figures 6.13 through 6.17. It can be seen that curves match very well. Based on the comparison between theoretical and experimental results of transfer length of Leadline, it can be concluded that the proposed theoretical model can successfully describe the stress state in the transfer bond region.
Table 6.1 Comparison of Theoretical and Experimental Transfer Length Results of Specimens Prestressed with Leadline

<table>
<thead>
<tr>
<th>Bar diameter (d_b) (mm)</th>
<th>Specimen designation</th>
<th>Initial stress level (f_{pi} / f_{pu})</th>
<th>Concrete strength at release (f'_{ci}) (MPa)</th>
<th>Measured transfer length (L_t) (mm)</th>
<th>Experimental model (L_t) (mm)</th>
<th>Theoretical model (L_t) (mm)</th>
<th>Measured (L_t / ) Theoretical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>BL9</td>
<td>0.61</td>
<td>26</td>
<td>700</td>
<td>627</td>
<td>628</td>
<td>1.11</td>
<td></td>
</tr>
<tr>
<td>BL10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>BL11</td>
<td>0.74</td>
<td>42</td>
<td>600</td>
<td>552</td>
<td>547</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>BL12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PL1</td>
<td>0.57</td>
<td>34</td>
<td>490</td>
<td>490</td>
<td>560</td>
<td>0.86</td>
<td></td>
</tr>
<tr>
<td>PL2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PL3</td>
<td>0.68</td>
<td>31</td>
<td>540</td>
<td>621</td>
<td>601</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>PL4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(f_{pi}\) : Initial prestress after elastic losses  
\(f_{pu}\) : Guaranteed tensile strength = 1970 MPa
Figure 6.1a Thick-walled cylinder subjected to inner pressure and axial tension
Figure 6.1b Stresses acting on an element
Figure 6.2 Stresses at the interface due Hoyer’s effect

(a) Stresses acting on concrete

(b) Stresses acting on reinforcement
Figure 6.3 Transverse stress distributions at the transfer onset from elastic uncracked analysis

Figure 6.4 Analytical expressions used for modeling the stress-crack width relationship
Tensile stress

Uncracked concrete

I ~ I

I ~ I

Microcracks

Fracture zone (or process zone)

Crack visible in microscope

Crack visible to naked eye

Real crack

End of tensile stress transfer

Figure 6.5 Nature of fracture zone
based on Hillerborg, 1983 (cited in Weerasekera, 1991)
Reinforcement

Visible cracks (naked eye)

Microscopic cracks

Effective crack tip

Real crack tip (w = 0.05 mm)

Real crack zone ($\sigma_0=0.0$)

Fracture zone

Uncracked zone

Outer free surface

Figure 6.6 Nature of cracking around the tendon
Figure 6.7 Determination of transfer length

Figure 6.8 Stress build-up in transfer zone
Figure 6.9 Stresses in the transverse direction at effective prestress = 1170 MPa (Case I)

Figure 6.10 Stresses in the transverse direction at effective prestress = 650 MPa (Case II)
Figure 6.11 Stresses in the transverse direction at the transfer onset (Case III)
2 pieces of Leadline

Concrete block 150x150x500 mm

Fine steel wire

Movable base

Fixed base

Fine-thread calibration bench used for smooth force application

Figure 6.12a Schematic elevation of friction test arrangement

Figure 6.12b View of the friction test setup
Figure 6.13 Theoretical and experimental curves of BL1 and PL1

Figure 6.14 Theoretical and experimental curves of BL3
Figure 6.15 Theoretical and experimental curves of BL5 and BL7

Figure 6.16 Theoretical and experimental curves of BL9 and BL10
Figure 6.17 Theoretical and experimental curves of BL11 and BL12
CHAPTER 7

FLEXURAL BOND LENGTH RESULTS AND ANALYSIS

7.1 INTRODUCTION

Flexural bond length is defined as the required length in addition to transfer length over which the tendon must be bonded so that the ultimate tensile strength of tendon, \( f_{pu} \), may develop at nominal strength of the member. The flexural bond stresses develop after concrete cracking due to the application of external loads as shown in Figure 3.17. When the concrete cracks, the bond stress in the immediate vicinity of the cracks rises to some limiting stress, slip occurs over a small portion of the tendon length adjacent to the cracks, and the bond stress near the crack is then reduced to a low value. With continued increase in load, the high bond stress progress as a wave from the original cracks towards the beam ends (Hanson and Kaar, 1959). If the peak of high bond stress wave reaches the prestress transfer zone, the increase in tendon stress decreases the diameter of reinforcement, reduces the frictional bond resistance, and precipitates general bond slip. Following loss of frictional resistance, mechanical resistance is the only factor which can contribute to bond between concrete and reinforcement.

Experimental results and analysis of test data related to flexural bond is discussed in this chapter. These tests were conducted to determine the development length of steel strands, Leadline, and CFCC strands under different conditions of prestress level, tendon
size, concrete strength, and shear reinforcement. The various failure modes which occurred are also discussed in this chapter. Load-deflection and load-slip curves are plotted to distinguish the type of failure. Load versus end slip curves, for all specimens failed due bond slip, are contained in Appendix D. The stress distribution along the tendon at different stages of loading are presented in Appendix E. Equations for predicting the flexural bond lengths of Leadline bars and CFCC strands are proposed. The effect of confinement, through using shear reinforcement, on the flexural bond length is investigated. The concrete properties and testing details are included in chapter 4.

7.2 MODES OF FAILURE

The beam and prism specimens were loaded incrementally to failure. Three types of failure modes were observed.

1. Bond failure
2. Flexural failure (Rupture of tendon)
3. Combination of bond and rupture failure

The difference in the modes of failure occurred considerably after the cracking moment of specimen was reached. During a bond failure, flexure cracking occurred, but before flexural failure could occur, considerable tendon end slip was recorded. General bond slip or first slip is defined as bond failure corresponding to tendon end slip of 0.025 mm (0.001 in.). This slip criteria is based on ASTM (1991) which was also previously used by many researchers (Janney, 1954). A flexural failure was characterized by considerable flexural cracking, yield of strand in specimens prestressed with steel, and rupture of tendon at the point of maximum moment or at the location of flexural shear crack. No
measurable tendon slip could be seen during a flexural failure. One combined failure mode was encountered, that is in some tests, a bond slip of tendon followed by rupture of the CFRP tendon was observed. The failure of specimens showed a significant tendon slip (greater than or equal to 0.025 mm) followed by flexural failure were classified as bond failure. No premature failure due to shear was observed as the specimens were designed to avoid such type of failure. In beams with embedment lengths less than the development length, failure occurred, after flexural and shear cracking, due to slippage of the tendon at one or two ends of the specimen. Beams with sufficient embedment length failed due to rupture of tendon.

7.3 FLEXURAL BOND LENGTH RESULTS

Tables 7.1, 7.2 and 7.3 summarize the results of testing 52 specimens tested with 57 different embedment lengths. The beams were tested using either single-point or two-point loading. The embedment length of the tendon, \( L_e \), is the length from the support point to the closest point load. The values of the effective prestress level, \( f_{pc} \), presented in tables 7.1 through 7.3 are the measured stress in the prestressing tendon at the time of loading of specimen. Also given in tables 7.1 through 7.3 are the observed modes of failure of all specimens as well as the tendon stresses that were developed in each test at first slip (slip of 0.025 mm) or at rupture of tendon.

The flexural bond length of a prestressing tendon is calculated based on the measured transfer length and the development length. The development length was determined as the embedment length range at which the failure mode changed from bond slip to rupture of the tendons. Using the same parameters used before for the transfer length, the flexural bond length \( L_{fb} \) is assumed to be related to the increase of stresses from the effective prestress level.
(f\textsubscript{pu}) to the ultimate tensile strength (f\textsubscript{pu}) of CFRP, tendon size, d\textsubscript{b}, and concrete strength at time of loading, f\textsubscript{c}, as given by equation 7.1:

\[
L_{fb} = \frac{(f_{pu} - f_{pe}) d_b}{f_c^{0.67}} \text{ for Leadline bars}
\]

\[
L_{fb} = \frac{(f_{pu} - f_{pe}) d_b}{2.8 f_c^{0.67}} \text{ for CFCC strands}
\]

where the coefficients of 1.0 and 2.8 for Leadline and CFCC were determined from the regression analysis of the test data as shown in Figures 7.1 and 7.2, respectively. The flexural bond length of the two types of CFRP reinforcement based on equation (7.1) is compared to the flexural bond length of steel strands proposed by Zia (1977), Mitchell (1993), and the ACI Code (1989) in Figure 7.3. The comparison is made at specific value of concrete compressive strength of f\textsubscript{c} equal to 35 MPa. The figure shows that the flexural bond length of Leadline is 20 percent less than that of steel strands predicted using the ACI code equation. The flexural bond length of CFCC is in the order of 25 percent of that of steel strands.

7.4 FLEXURAL BOND STRESSES

At present, it is not feasible to measure the magnitude and complex distribution of bond stress along the flexural bond length of prestressing tendon. However, to obtain suitable design criteria, it could be expressed as an average value. The average flexural bond stress (U_{fb}) was calculated for all beams based on the measured transfer length (L_t), the embedment length, L_e, cross section area of tendon reported by the manufacturer (A), effective stress (f\textsubscript{pe}), tensile stress of the tendon at first slip (f\textsubscript{pu}), concrete strength at time of loading (f\textsubscript{c}), and the nominal perimeter of the reinforcement (\pi d_b) as given by equation 7.2:
The average flexural bond stress was found to be in the range of 2.2 to 3.8 MPa for Leadline with an average value of 3.0 MPa and 4.4 to 7.0 MPa for CFCC with an average value of 5.0 MPa. Using the expression suggested by the ACI Code (1995) for the flexural bond length of steel strands, the average flexural bond stress, \( U_{fb} \), for steel is found to be 1.34 MPa. Specimens with embedment lengths slightly greater than the transfer length had a higher value for flexural bond stress than that for specimens with large embedment length. For longer embedment lengths, the flexural bond wave form includes a long "tail" stretching from the peak near the transfer zone to the section of maximum tendon stress. The average bond stress for long embeddings is therefore less than the average bond stress for short embeddings.

The flexural bond stress \( (U_{fb}) \) can also be expressed as a flexural bond stress index \( (U'_{fb}) \) as given in equation (7.3) to account for the variability of the concrete strength \( (f'_{c}) \).

\[
U_{fb} = \frac{(f_{pu} - f_{pe}) A}{\pi d_b (L_e - L_t)}
\]  

(7.2)

The flexural bond index \( (U'_{fb}) \) was determined for all specimens. For Leadline bars, the average flexural bond index \( (U'_{fb}) \) was found to be 0.23. For CFCC the flexural bond stress index \( (U'_{fb}) \) ranged from 0.33 to 0.40. Figure 7.4 shows the average flexural bond index of CFCC versus the strand diameter. The figure indicates that flexural bond index of CFCC increases with the increase of strand size. This behaviour may be attributed to the increase of mechanical resistance of CFCC with the increase of strand size.
7.5 DEVELOPMENT LENGTH

The development length of a prestressing tendon, $L_d$, is the summation of the transfer length ($L_t$) and the flexural bond length ($L_{fb}$). The development length of CFRP prestressing bars and strands can be predicted as follow:

$$L_d = \frac{f_{pi} d_b}{\alpha_t f_{ci}^{0.67}} + \frac{(f_{pu} - f_{pe}) d_b}{\alpha_f f_c^{0.67}}$$  \hspace{1cm} (7.4)

where: $\alpha_t = 1.9$ and 4.8 for Leadline and CFCC respectively.

$\alpha_f = 1.0$ and 2.8 for Leadline and CFCC respectively.

7.6 FLEXURAL BEHAVIOUR

The flexural behavior is discussed in this section through studying the behavior of selected specimens in terms of crack pattern, mode of failure, load-deflection relationship, load-slip behavior, and stress distribution along the tendon at different stages of loading.

**Prism specimens PS7 and PS8**

Failure pattern of prism specimens PS7 and PS8 is shown in Figure 7.5. The prisms were pretensioned with 12.7 mm diameter steel strand. After release of prestress, an effective prestressing force of 106 kN was measured by the enclosed hollow load cell. Gradual increase of load was applied using the enclosed hollow jack from 106 kN to failure. After yield of steel strands, slip was observed in the two specimens. Failure occurred at 182 kN and 190 kN in PS7 and PS8, respectively, due to splitting of concrete cover. The recorded slip of the free end before splitting was 0.35 mm as shown in Figures 7.6 and 7.7 for PS7 and PS8, respectively. It should be mentioned here that the concrete cover was about 50 mm and no steel confinement was provided. The concrete strength at time of release and testing was 46 MPa.
Beam specimen BS6

The crack pattern of beam specimen BS6 after failure is shown in Figure 7.8. The specimen was pretensioned with single 12.7 mm diameter steel strand. The concrete cover was 50 mm and sufficient shear reinforcement was provided. The strand was pretensioned to 1240 MPa (66% the nominal tensile strength). The specimen was tested using single-point load with a shear span of 900 mm. The first flexural crack appeared at load of 41 kN. Strand slip of 0.025 mm was observed at a load of 47 kN. Rupture of strand occurred at load of 83 kN. The crack pattern shown in Figure 7.8 is a typical one for all beams provided with shear reinforcement. The crack pattern contained a major flexural crack at the mid-span and two symmetric flexural shear cracks.

Load versus deflection for beam BS6 is shown in Figure 7.9. This test showed linear behaviour up to cracking load and non-linear behaviour with significantly increase in deflection due to reduction in stiffness with the formation of new cracks, slip, and yield of strand. The formation of the flexural shear cracks was accompanied by dropping in load as shown in Figure 7.9. Plot of end slip versus load is shown in Figure 7.10. It can be noticed from the figure that slip started at the right and left ends at load of 47 and 58 kN, respectively. The right and left flexural shear cracks appeared at 55 and 60 kN respectively. The measured end slip at rupture of strand were 6 and 4.5 mm at right and left ends, respectively. The confinement provided by the 50 mm concrete cover and the shear reinforcement could be the reason for the increase in load carrying capacity after the first slip.
**Beam specimen BL3**

The distribution of cracks of beam BL3 after failure is shown in Figure 7.11. The beam was prestressed with a single 8 mm diameter Leadline bar, located at 35 mm from the bottom of the beam. The beam was provided with web reinforcement. The jacking level was 1370 MPa (70% the guaranteed ultimate strength). The effective prestress after one year was 1082 MPa (21% losses). The beam was tested using a single-point load with two equal shear spans of 800 mm. Figure 7.12 shows the load-deflection curve of beam BL3. The beam behaved linearly up to the cracking load of 25 kN, and after cracking with reduced stiffness up to the first slip. After first slip, non-linear behaviour was observed with an increase of deflection up to failure. Figure 7.13 shows the slip of two ends of BL3 versus load. The first slip was observed at 35 kN at the two ends. The sudden failure of the beam was due to slip of the Leadline bar at 47 kN with longitudinal splitting of the side concrete cover at the level of Leadline bar. Similar to the behaviour of BS6, the increase in load resistance after first slip is related to the sufficient confinement provided by the 35 mm concrete cover and web steel reinforcement to resist the radial forces induced by Leadline bar during slip.

The distribution of the of longitudinal stress along the Leadline bar of beam BL3 at zero load, at cracking, at first slip, and at ultimate bond failure, is shown in Figure 7.14. It should be noted that the slope of the curves at any point is proportional to the induced bond stresses. Flexural bond of significant magnitude exists only after cracking. The figure shows the stresses in the Leadline bar within the transfer zone increased prior to first slip. This means that transfer zone contributes in resisting flexural stresses, \((f_{pa} - f_{pe})\), in addition to effective prestress, \(f_{pe}\).
**Beam specimen BL9**

The crack pattern of beam BL9 is shown in Figure 7.15. The beam did not contain web reinforcement. Single-point load with shear span of 1900 mm was used in this test. The Leadline bar was pretensioned to 1220 MPa (62% the guaranteed ultimate strength). The effective prestressing at testing was 1100 MPa. The first visible flexural crack appeared at 15 kN and the load dropped to 14 kN. Flexural shear crack at the left shear span occurred at 20.5 kN and the load dropped to 18.5 kN. When the load increased to 20.5 kN, a second flexural shear crack appeared at right shear span accompanied by drop in the load. The load-deflection relationship is shown in Figure 7.16. The test showed linear behaviour up to first cracking and linear with reduced stiffness up to the formation of the two symmetric flexural shear cracks. The behaviour was also linear up to rupture of the reinforcement with reduced stiffness. No slip was recorded until the beam failed at 30.3 kN due rupture of Leadline bar at the location of right flexural shear crack 450 mm away from the mid-span point. The distribution of the stress along the Leadline bar of beam BL9 at zero load, cracking, and at failure, is shown in Figure 7.17. The tendon stress at rupture was found to be 2655 MPa which is less than the tensile strength of Leadline bar achieved in other tests at rupture. This could be attributed to the additional lateral stresses due to dowel action at the location of shear crack. Bond failure was not observed since there is no increase in the stresses of the Leadline bar within the transfer zone, as shown in Figure 7.17.

**Beam specimen BT16**

Figures 7.18 and 7.19 show the cracking pattern of BT16 after testing the right and left ends, respectively. The beam was pretensioned with 12.5 CFCC strand and it was provided
with shear reinforcement. The CFCC strand was jacked to 1125 MPa (60% the guaranteed ultimate strength). The effective prestress at testing was 984 MPa. The right and left ends of the beam were tested independently with 550 and 600 mm embedments respectively. During testing the right end (BT16A), the formation of flexural shear crack near the transfer zone occurred at 76 kN precipitated bond failure and resulted in sudden loss of strand anchorage at 98 kN. The strand anchorage was adversely affected by the longitudinal splitting of bottom concrete cover which extended to the transfer zone. Figure 7.20 shows the load-deflection behaviour of testing right end of BT16 which is similar to the behaviour of BL9 explained before. Load-slip plot of BT16A is shown in Figure 7.21. The figure shows a sudden strand slip of 7.5 mm with large drop in load. Based on this result, it was decided to test the left end of the beam BT16B with longer embedment length of 600 mm. The failure of the beam BT16B was due to rupture of CFCC strand at the section having the maximum moment at the location of loaded point.

**Beam specimen BT18**

The crack pattern of beam BT18 is shown in Figure 7.22. The beam was pretensioned with 15.2 mm diameter CFCC strand with a concrete cover of 60 mm from the bottom face. Jacking stress was 1295 MPa (74% the guaranteed strength). At the time of testing, the effective prestress was 1170 MPa. The beam was tested using symmetric two-pint loads with two equal shear spans of 1250 mm. The beam did not contain any shear reinforcement. The first visible flexural crack appeared at 48 kN at the middle of the constant moment zone. At 67 kN, a flexural shear crack started 350 mm outside the constant moment zone at the bottom side of the beam. A sudden drop in load from 83 to 75 kN occurred due to the formation of
another shear crack 230 mm away from the previous flexural shear crack. Spalling of the concrete cover between the two cracks was observed as shown in Figure 7.22. Slip of the strand occurred after increasing the load to 76 kN.

Figure 7.23 shows the load-deflection relationship of beam BT18. The beam showed linear behaviour up to first crack at 48 kN and non-linear transition zone during the formation of flexural cracks up to 55 kN then linear behaviour with reduced stiffness up to failure. The CFCC strand slipped two mm before the longitudinal splitting of the bottom concrete cover had occurred as shown in Figure 7.24. The splitting of the cover propagates from the most outer flexural and/or shear crack toward the support. Figure 7.25 shows the stresses in the CFCC strand before test, after cracking, and at slip. Increase of stresses within the transfer zone at first slip could be noticed from the readings of strain gauges placed within the transfer zone at 250 and 400 mm from transfer onset.

7.7 EFFECT OF CONCRETE CONFINEMENT

To study the effect of concrete confinement by using steel stirrups as shear reinforcement on the bond behaviour of CFRP bars and strands, four beams prestressed with Leadline and six beams prestressed with CFCC were tested without shear reinforcement. The concrete cover was taken 35 mm in case of Leadline, 50 and 60 mm in case of 12.5 and 15.2 mm diameter CFCC, respectively. Figure 7.26 shows a plot of flexural bond length of Leadline versus \((f_{pu} - f_{pc})d_b / f'_{c}^{0.67}\). The figure compares the flexural bond length results of beams pretensioned with Leadline and had no web steel reinforcement to that predicted by the proposed equation 7.1. It can be concluded from the comparison that the absence of shear reinforcement did not affect the flexural bond length results of beams prestressed with Leadline.
bars. This behaviour could be explained that the 35 mm concrete cover provided sufficient confinement to resist the radial forces induced by the Leadline bar.

In beams prestressed with CFCC strands, the absence of shear reinforcement resulted in an increase of the flexural bond length by an average value of 25 percent higher than the proposed model for beams provided with shear reinforcement in equation 7.1 as shown in Figure 7.27. This could be due to the helical shape of CFCC seven-wire strand which activate the confining of steel stirrups due to higher radial stresses than that in the case of Leadline.

The load-slip behaviour of beams pretensioned with Leadline was affected by the absence of steel confinement. This difference in behaviour could be noticed by comparing the load-slip curves of BL3 (see Figure 7.13) and BL10 in Figure 7.28. The additional confinement provided by the shear reinforcement in BL3 resulted in an increase in load for further slip up to 2 mm before bond failure. The maximum slip was 0.25 mm in beam BL10, which had no steel confinement and no increase in load was required after first slip. Unlike Leadline, load-slip behaviour of beams pretensioned with CFCC strands was not affected by the absence of shear reinforcement as shown in load-slip curves in Appendix D.
<table>
<thead>
<tr>
<th>Strand diameter</th>
<th>Specimen designation</th>
<th>Concrete strength at test $f'_c$ (MPa)</th>
<th>Embedment length $L_e$ (mm)</th>
<th>Effective stress at testing $f_{pe}$ (MPa)</th>
<th>Stress at first slip or at rupture $f_{pu}$ (MPa)</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_b$ (mm)</td>
<td>PS1</td>
<td>48</td>
<td>750</td>
<td>1116</td>
<td>1945</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>PS2</td>
<td></td>
<td>550</td>
<td>1116</td>
<td>1890</td>
<td>R</td>
</tr>
<tr>
<td>9.6</td>
<td>PS3</td>
<td>46</td>
<td>550</td>
<td>1116</td>
<td>2036</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>PS4</td>
<td></td>
<td>530</td>
<td>1116</td>
<td>1854</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>BS1</td>
<td>60</td>
<td>800</td>
<td>na</td>
<td>2189</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>BS2</td>
<td></td>
<td>550</td>
<td>na</td>
<td>2179</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>PS5</td>
<td>35</td>
<td>750</td>
<td>1120</td>
<td>1290</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>PS6</td>
<td></td>
<td>750</td>
<td>1120</td>
<td>1510</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>PS7</td>
<td>46</td>
<td>750</td>
<td>1120</td>
<td>1837</td>
<td>B</td>
</tr>
<tr>
<td>12.7</td>
<td>PS8</td>
<td></td>
<td>750</td>
<td>1120</td>
<td>1874</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BS3</td>
<td>63</td>
<td>800</td>
<td>na</td>
<td>1870</td>
<td>B/R</td>
</tr>
<tr>
<td></td>
<td>BS4</td>
<td></td>
<td>800</td>
<td>na</td>
<td>1880</td>
<td>B/R</td>
</tr>
<tr>
<td></td>
<td>BS5</td>
<td>46</td>
<td>900</td>
<td>990</td>
<td>1290</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BS6</td>
<td></td>
<td>900</td>
<td>990</td>
<td>1360</td>
<td>B</td>
</tr>
</tbody>
</table>

R : failure due to rupture of strand  
B : failure due to bond slip  
B/R : tendon rupture accompanied by bond slip
<table>
<thead>
<tr>
<th>Strand diameter \ Specimen designation</th>
<th>Concrete strength at test \ $f'_c$ (MPa)</th>
<th>Embedment length \ $L_e$ (mm)</th>
<th>Effective stress at testing \ $f_{te}$ (MPa)</th>
<th>Stress at first slip or at rupture \ $f_{tu}$ (MPa)</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>BL1</td>
<td>50</td>
<td>800</td>
<td>1068</td>
<td>1870</td>
<td>B</td>
</tr>
<tr>
<td>BL2</td>
<td>800</td>
<td>1078</td>
<td>1850</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>BL3</td>
<td>46</td>
<td>800</td>
<td>1082</td>
<td>1753</td>
<td>B</td>
</tr>
<tr>
<td>BL4</td>
<td>800</td>
<td>1082</td>
<td>1764</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>BL5</td>
<td>1200</td>
<td>1255</td>
<td>2129</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>BL6</td>
<td>41</td>
<td>800</td>
<td>1227</td>
<td>1420</td>
<td>B</td>
</tr>
<tr>
<td>BL7</td>
<td>1000</td>
<td>1248</td>
<td>1820</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>BL8</td>
<td>1000</td>
<td>1248</td>
<td>1700</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>8 BL9</td>
<td>37</td>
<td>1900</td>
<td>1100</td>
<td>2655</td>
<td>R</td>
</tr>
<tr>
<td>BL10</td>
<td>1400</td>
<td>1100</td>
<td>2200</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>BL11</td>
<td>52</td>
<td>1300</td>
<td>1400</td>
<td>3000</td>
<td>B/R</td>
</tr>
<tr>
<td>BL12a</td>
<td>1100</td>
<td>1365</td>
<td>2770</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>BL12b</td>
<td>1300</td>
<td>1365</td>
<td>2965</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>PL1</td>
<td>34</td>
<td>750</td>
<td>1123</td>
<td>1777</td>
<td>B</td>
</tr>
<tr>
<td>PL2</td>
<td>750</td>
<td>1123</td>
<td>1247</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>PL3</td>
<td>31</td>
<td>750</td>
<td>1340</td>
<td>2073</td>
<td>B</td>
</tr>
<tr>
<td>PL4</td>
<td>750</td>
<td>1340</td>
<td>2311</td>
<td>B/R</td>
<td></td>
</tr>
</tbody>
</table>

R: failure due to rupture of strand
B: failure due to bond slip
B/R: tendon rupture accompanied by bond slip
Table 7.3 Flexural Bond Length Results of CFCC strands

<table>
<thead>
<tr>
<th>Strand diameter</th>
<th>Specimen designation</th>
<th>Concrete strength at test ( f'_c ) (MPa)</th>
<th>Embedment length ( L_e ) (mm)</th>
<th>Effective stress at testing ( f_{pe} ) (MPa)</th>
<th>Stress at first slip or at rupture ( f_{pu} ) (MPa)</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>db (mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.5</td>
<td>BT11</td>
<td>630</td>
<td>1203</td>
<td>1934</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT12</td>
<td>41</td>
<td>530</td>
<td>1200</td>
<td>1577</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BT13</td>
<td>630</td>
<td>1237</td>
<td>2168</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT14</td>
<td>580</td>
<td>1237</td>
<td>1906</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT7</td>
<td>640</td>
<td>1306</td>
<td>1855</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT8</td>
<td>37</td>
<td>600</td>
<td>1300</td>
<td>1815</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BT9</td>
<td>550</td>
<td>1290</td>
<td>1700</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>12.5</td>
<td>BT10</td>
<td>500</td>
<td>1310</td>
<td>1616</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT15a</td>
<td>41</td>
<td>700</td>
<td>1000</td>
<td>1924</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>BT15b</td>
<td>600</td>
<td>1000</td>
<td>1879</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT16a</td>
<td>550</td>
<td>984</td>
<td>1853</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT16b</td>
<td>600</td>
<td>984</td>
<td>2033</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT19</td>
<td>34</td>
<td>950</td>
<td>1215</td>
<td>2100</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>BT20</td>
<td>750</td>
<td>1215</td>
<td>1830</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>15.2</td>
<td>BT1</td>
<td>46</td>
<td>800</td>
<td>905</td>
<td>1814</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BT2</td>
<td></td>
<td>800</td>
<td>905</td>
<td>1916</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BT3</td>
<td></td>
<td>800</td>
<td>1200</td>
<td>2150</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>BT4</td>
<td>43</td>
<td>750</td>
<td>1223</td>
<td>2170</td>
<td>B/R</td>
</tr>
<tr>
<td></td>
<td>BT5</td>
<td>600</td>
<td>1232</td>
<td>1756</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT6</td>
<td>600</td>
<td>1223</td>
<td>1720</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BT17a</td>
<td>33</td>
<td>900</td>
<td>1165</td>
<td>1600</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BT17b</td>
<td></td>
<td>1100</td>
<td>1165</td>
<td>1744</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BT18</td>
<td></td>
<td>1250</td>
<td>1170</td>
<td>1932</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>BT21</td>
<td>31</td>
<td>1350</td>
<td>979</td>
<td>2260</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>BT22a</td>
<td></td>
<td>1000</td>
<td>971</td>
<td>1734</td>
<td>B/R</td>
</tr>
<tr>
<td></td>
<td>BT22b</td>
<td></td>
<td>900</td>
<td>971</td>
<td>1618</td>
<td>B</td>
</tr>
</tbody>
</table>

R : failure due to rupture of strand
B : failure due to bond slip
B/R: tendon rupture accompanied by bond slip
Figure 7.1 Flexural bond length correlation for 8 mm Leadline bars

\[ L_{fb} = \frac{(f_{pu} - f_{pe}) d_b}{f_c^{0.67}} \]

(Standard deviation = 14%)

Figure 7.2 Flexural bond length correlation for CFCC strands

\[ L_{fb} = \frac{(f_{pu} - f_{pe}) d_b}{28 f_c^{0.67}} \]

(Standard deviation = 12%)
Figure 7.3 Flexural bond length comparison

Figure 7.4 Effect of strand diameter on flexural bond strength of CFCC strands
Figure 7.5 Prism PS7 and PS8 after failure

Figure 7.6 Load versus end slip of PS7
Figure 7.7 Load versus end slip of PS8

Figure 7.8 Cracking pattern of BS6
Figure 7.9 Observed Load-Deflection Relationship of BS6

Figure 7.10 Observed load-slip relationship BS6
Figure 7.11 Cracking pattern of BL3

Figure 7.12 Measured load-deflection relationship of BL3
Figure 7.13 Observed load-slip relationship of BL3

Figure 7.14 Measured stresses in Leadline bar of BL3
Figure 7.15 Cracking pattern of BL9

Figure 7.16 Measured load-deflection of BL9
Figure 7.17 Measured stresses in Lead line bar of BL9

Figure 7.18 Cracking pattern of BT16A (right end)
Figure 7.19 Cracking pattern of BT16B (left end)

Figure 7.20 Observed load-deflection relationship of BT16A (right end)
Figure 7.21 Observed load-slip relationship of BT16A

Figure 7.22 Cracking pattern of BT18
Figure 7.23 Load-deflection behavior of BT18

Figure 7.24 Measured load-slip of BT18
Figure 7.25 Stress distribution along the strand of BT18

Figure 7.26 Effect of shear reinforcement on the flexural bond length of Leadline bars
Figure 7.27 Effect of shear reinforcement on the flexural bond length of CFCC

\[ L_{fb} = \frac{(f_{pu} - f_{ps}) d_b}{2.8 f_c^{0.67}} \]

Figure 7.28 Observed load-slip of BL10
CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 SUMMARY

The objective of this research program was to investigate the various bond characteristics of Carbon Fibre Reinforced Polymers, CFRP bars and strands used to pretension prestressed concrete structures and to recommend design criteria for CFRP transfer and development lengths. Bond characteristics are examined through measurements of the transfer length, development length, and the corresponding bond stresses. The fundamental understanding of the behaviour and specific characteristics of FRP prestressing tendons such as bond strength, transfer and development lengths are of prime importance for the design guidelines of concrete structures prestressed with FRP reinforcements.

This research program consisted of experimental and analytical phases. The experimental phase included casting and testing of 52 specimens at the University of Manitoba to examine the bond characteristics of two types of Carbon Fibre Reinforced Polymers, CFRP, prestressing tendons according to the channel system with Alexandria University. The reinforcement used in this study are Leadline bars, Carbon Fibre Composite Cables, CFCC, and conventional steel strands. Twelve beams and four prisms pretensioned with Leadline bars, 22 beams pretensioned with CFCC strands, and 6 beams and 8 prisms prestressed with steel strands were tested using different shear spans. The main variables in this study are:

a- Diameter of the reinforcement: Steel strands diameter of 9.6 and 12.7 mm, 8 mm diameter Leadline, and 10.5, 12.5, and 15.2 mm diameter CFCC strands were used,
b- Prestress level: 60% to 80% the guaranteed ultimate strength was used,

c- Type of CFRP reinforcement (bars and strands)

d- Time-dependent effect,

e- Concrete compressive strength,

f- Confinement through the use of shear reinforcement.

Four prisms and 4 beams pretensioned with Leadline bars and 6 beams pretensioned
with CFCC were tested without shear reinforcement to study the effect of concrete confinement
on the transfer and flexural bond lengths. Effect of shrinkage and creep of concrete during a time
period of one year was also investigated by monitoring the transfer length measurements for one
year of two beams pretensioned using steel strands, two beams pretensioned with Leadline bars,
and two beams pretensioned with CFCC strands. In this study, new prism specimen
configuration is proposed. The proposed prism test can be considered an adequate model to
determine the transfer length and the bond characteristics of prestressing tendons. The
proposed test has the advantage of easiness, speed, economical, and minimize the need for
sophisticated testing equipment. Design models, based on the experimental results, are
proposed to predict the transfer and flexural bond lengths for steel, Leadline and CFCC tendons.

The analytical phase included a new rational model developed to analyse the behaviour
within the transfer bond of prestressing reinforcement. This rational model is based on the
confining capacity of the concrete surrounding the tendon. The model uses the principal of solid
mechanics (elastic theory of thick-walled cylinder) to evaluate the radial displacement of
concrete and reinforcement at the tendon-to-concrete interface. A computer program was
developed to carry out the analysis. The model considered only the friction bond phenomenon.
The model accounts for the effect of concrete cracking and the post-peak softening in tension.
The effect of shrinkage of concrete before release and the effect of axial compressive stresses in concrete at the level of tendon on the transfer length are also considered in the analysis. The model is capable of predicting the transfer length of any type of reinforcement with known material properties of both the concrete and reinforcement and the coefficient of friction between concrete and reinforcement. The rational model was used to predict the transfer length of Leadline bars using the measured value of coefficient of friction between Leadline and concrete. The results were found to be comparable to the experimental results.

An extensive survey of the characteristics and applications of FRP reinforcement in the field of civil engineering is also presented in this thesis. Literature review on the bond behavior, transfer and development lengths of steel and FRP reinforcement is also presented.

8.2 CONCLUSIONS AND DESIGN RECOMMENDATIONS

Based on the analysis of the experimental results and the proposed analytical model, the following conclusions are drawn:

1. The transfer length could be predicted using the following proposed model:

\[
L_t = \frac{f_{pl} d_b}{\alpha_t f_{cl}^{0.67}}
\]  

(8.1)

where \(\alpha_t\) has a value of 2.4, 1.9 and 4.8 for steel strands, Leadline bars and CFCC strands, respectively.

2. The flexural bond length for Leadline bars and CFCC strands could be predicted using the following proposed model:

\[
L_{fb} = \frac{(f_{pu} - f_{pe}) d_b}{\alpha_f f_{c}^{0.67}}
\]  

(8.2)
where $\alpha_t$ is a coefficient has a value of 1.0 and 2.8 for Leadline bars and CFCC strands, respectively. The development length is the summation of transfer length and flexural bond length.

3. Comparison showed that the proposed model for transfer length of steel strands is 18% higher than the ACI prediction when $f_{ci}$ is 30 MPa. For higher concrete strength of 40 MPa at transfer, the proposed model for steel strands is 14% shorter than ACI prediction.

4. Transfer length of 8 mm Leadline bar is about 12% more than ACI prediction for equivalent steel strand for concrete strength at transfer of 30 MPa.

5. Transfer length of CFCC strand is about 50% of ACI prediction for equivalent steel strand for concrete strength of 30 MPa at transfer.

6. Flexural bond length of Leadline is about 20 percent less than ACI prediction for steel strands for concrete strength at loading of 35 MPa.

7. Flexural bond length of CFCC strands is about 25 percent of the ACI prediction for concrete strength at loading of 35 MPa.

8. The proposed models were found to be in good agreement with the measured values in this program and the limited data in the literature.

9. The average measured transfer bond strength for Leadline is 4.5 MPa and 8.9 MPa for CFCC strands. The average measured flexural bond strength is 3 MPa and 5 MPa for Leadline and CFCC respectively.

10. The transfer length could increase by 10% and 17% for Leadline and CFCC, respectively for members without stirrups more than the predicted values by the proposed model.

11. The flexural bond length of Leadline is not affected by the confinement while the flexural bond length of CFCC increased by 25% than the predicted values using the proposed model.
12. The transfer length of Leadline rods increased by 22% after one year. However, no increase in the transfer lengths of CFCC and steel strands was observed.

13. The measured losses of prestress after one year were 20%, 21%, and 16% for beams prestressed with steel strands, Leadline bars, and CFCC strands, respectively.

14. The average transfer bond strength for CFCC is not affected by changes in the strand diameter.

15. The average flexural bond strength for CFCC increases with the increase of strand diameter.

16. Splitting of concrete did not occur in any of the tested specimens for concrete cover not less than 4 times the tendon diameter used in all specimens. The analytical study suggests that some splitting problems may be encountered in members with close spacing, thin cover or poor confinement. However, these limits were not established by this research. The following two recommendations could help avoiding splitting at ends of the beam:
   a) Spiral reinforcement within the transfer length of FRP prestressing reinforcement could enhance the confinement.
   b) Blanketing one half the number of tendons at the beam ends for a distance not less than 50% the predicted transfer length in a staggered fashion to avoid the accumulation of circumferential tensile stresses.

17. Measured draw-in of prestressing tendons at transfer could be used to confirm the transfer length measurements.

18. All beams pretensioned with CFRP exhibited a linear elastic load-deflection behavior up to cracking load and linear after cracking with a reduced stiffness up to failure.
19. Two modes of failure were observed during the testing of beams: bond failure and rupture of tendon. Splitting of concrete was observed during the testing of prism specimens.

20. Bond failure in beams prestressed with CFCC was characterized by suddenness while it was more gradual bond failure in the case of beams prestressed with Leadline.

21. The measured and calculated rupture strengths of Leadline and CFCC during the beam tests are 50% and 30% higher the guaranteed ultimate strength reported by the manufacturer, respectively.

22. The new prism configuration proposed for determining the transfer length and bond strength of prestressing reinforcement proved to be easy, economical and simple to use.

23. The measured coefficient of friction between Leadline and concrete was found to be in the range of 0.55 to 0.65 with an average value of 0.6.

24. The rational study showed the transfer bond depends on the swelling of tendon upon release (Hoyer's effect), the ability of the concrete to resist these stresses, and the interface characteristics. The amount of swelling depends on the prestress level, diameter of tendon, Poisson's ratio of tendon, and longitudinal and transverse elasticity moduli of reinforcement. The resistance of concrete to resist swelling depends on concrete strength, cover and spacing.

25. The developed rational model in this work successfully describes the stress and strain state in the transfer zone of Leadline bars and it could be a suitable tool for analyzing the transfer bond problem for other types of prestressing reinforcement.
26. The uncracked elastic analysis showed high circumferential tensile stresses in concrete around the tendon which far exceed the tensile strength of concrete. This indicates that radial cracking can be expected to occur in the transfer zone.

8.3 SUGGESTIONS FOR FUTURE RESEARCH

1) The effects of smaller concrete covers and spacing on the bond properties of CFRP reinforcement.

2) The effects of elevated temperature and freeze-thaw cycles on the bond properties of CFRP reinforcement.

3) The effect of the rate of release of prestress forces on the transfer length.

4) Effect of long debonding (blanketing) of tendons at the beam ends, as an alternative solution of draping up, on the transfer and development lengths.

5) Bond properties of bundled Leadline bars.

6) Effect of fatigue loading on the bond behaviour of CFRP reinforcement.

7) The effect of additional confinement through the use of spiral reinforcement in the rational model for transfer length and its contribution to the transfer bond of other FRP reinforcement.

8) Detailed measurement of the coefficient of friction and surface roughness of prestressing reinforcement.
REFERENCES


3- Abdelrahman A.A. and Rizkalla S.H., 1994, “Advanced Composites for Concrete Structures” Technical Report, Fifth International Colloquium on Concrete in Developing Countries, Cairo, Egypt, January.


5- ACI Committee 318, 1995 "Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-95/ACI 318R-95)”, American Concrete Institute, Detroit, 369 pp.


26- Ferguson, P. M., Breen, J. E., and Jirsa, J. O., 1988, "Reinforced Concrete Fundamentals" John Wiley & Sons.


66- Taerwe, L., Palleman, I., 1993 "Transmission Length of Aramid Fibre Composite Prestressing Bars Embedded in Concrete Prisms", Magnel Laboratory for Concrete Research, University of Ghent, Belgium.


69- Watanabe, Akira, 1965 "Studies on the transmission Length of Pretensioned Prestressed Concrete" Reprinted from the Memoirs of the Faculty of Engineering, Kyushu University, Volume XXIV. No. 3.


APPENDIX A

Concrete And Tendon Strains Along The Transfer Zone
Figure A1  Concrete strain distribution for specimen BS1

Figure A2  Concrete strain distribution for specimen BS2
Figure A3 Strain distribution along the tendon of BS3

Figure A4 Concrete strain distribution for specimen BS3
Figure A5  Concrete strain distribution for specimen BS4

Figure A6  Concrete strain distribution for specimen PS1
Figure A7  Concrete strain distribution for specimen PS2

Figure A8  Concrete strain distribution for specimen PS3
Figure A9  Concrete strain distribution for specimen PS4

Figure A10  Concrete strain distribution for specimen PS5
Figure A11 Concrete strain distribution for specimen PS6

Figure A12 Concrete strain distribution for specimen PS7
Figure A13 Concrete strain distribution for specimen PS8

Figure A14 Strain distribution along the tendon of BL1
At release 3 weeks after release

Figure A15 Concrete strain distribution of specimen BL1

Figure A16 Concrete strain distribution of specimen BL2
Figure A17 Strain distribution along the tendon of BL5

Figure A18 Concrete strain distribution of specimen BL5
Figure A19 Concrete strain distribution of specimen BL6

Figure A20 Strain distribution along the tendon of BL7
Figure A21 Concrete strain distribution of specimen BL7

Figure A22 Concrete strain distribution of specimen BL8
Figure A23 Strain distribution along the tendon of BL9

Figure A24 Strain distribution along the tendon of BL10
Figure A25 Strain distribution along the tendon of BL11

Figure A26 Strain distribution along the tendon of BL12
Figure A27 Concrete strain distribution of specimen PL1

Figure A28 Strain distribution along the tendon of PL2
Figure A29 Concrete strain distribution of specimen PL2

Figure A30 Concrete strain distribution of specimen PL3
Figure A31  Concrete strain distribution of specimen PL4.

Figure A32  Strain distribution along the tendon of BT3.
Figure A33 Concrete strain distribution of specimen BT3

Figure A34 Concrete strain distribution of specimen BT4
Figure A35 Strain distribution along the tendon of BT5

Figure A36 Concrete strain distribution of specimen BT5
Figure A37 Concrete strain distribution of specimen BT6

Figure A38 Strain distribution along the tendon of BT7
Figure A39 Concrete strain distribution of specimen BT7

Figure A40 Concrete strain distribution of specimen BT8
Figure A41 Strain distribution along the tendon of BT9

Figure A42 Concrete strain distribution of specimen BT9
Figure A43 Concrete strain distribution of specimen BT10

Figure A44 Strain distribution along the tendon of BT11
Figure A45 Concrete strain distribution of specimen BT11

Figure A46 Concrete strain distribution of specimen BT12
Figure A47 Strain distribution along the tendon of BT13

Figure A48 Concrete strain distribution of specimen BT13
Figure A49 Concrete strain distribution of specimen BT14

Figure A50 Strain distribution along the tendon of BT16
Figure A51  Concrete strain distribution of specimen BT16

Figure A52  Strain distribution along the tendon of BT17
Figure A53 Strain distribution along the tendon of BT18

Figure A54 Strain distribution along the tendon of BT19
Figure A55 Strain distribution along the tendon of BT20

Figure A56 Strain distribution along the tendon of BT21
Figure A57 Strain distribution along the tendon of BT22
APPENDIX B

Bond Stress Distribution In The Transfer Zone
Figure B1 Bond stress distribution at transfer of BS3

Figure B2 Bond stress distribution at transfer of BL1
Figure B3  Bond stress distribution of BL5

Figure B4  Bond stress distribution of BL7
Figure B5 Bond stress distribution of BL9

Figure B6 Bond stress distribution of BL10
At release
6 weeks after release

Figure B7 Bond stress distribution of BL11

Figure B8 Bond stress distribution of BL12
Figure B9  Bond stress distribution at transfer of PL2

Figure B10  Bond stress distribution at transfer of BT3
Figure B11  Bond stress distribution at transfer of BT5

Figure B12  Bond stress distribution of BT7
Figure B13  Bond stress distribution of BT9

Figure B14  Bond stress distribution at transfer of BT11
Figure B15  Bond stress distribution at transfer of BT13

Figure B16  Bond stress distribution at transfer of BT15
Figure B17 Bond stress distribution at transfer of BT16

Figure B18 Bond stress distribution at transfer of BT18
Figure B19 Bond stress distribution at transfer of BT19

Figure B20 Bond stress distribution at transfer of BT20
- At release
- 4 weeks after release

Figure B21 Bond stress distribution at transfer of BT21

- At release
- 3 weeks after release

Figure B22 Bond stress distribution at transfer of BT22
APPENDIX C

List Of The FORTRAN Program For Predicting The Transfer Length
C MAIN PROGRAM
C
**** LT3.FOR ****
C CRACKED FROM A TO X AND PARTIALLY CRACKED FROM X TO E
C AND UNCRACKED FROM E TO B
C IT CAN BE USED FOR UNCRACKED BY PUTTING HIGH VALUE FOR FT
C
DIMENSION DL(1000),XXX(1000),ZA(1000)
OPEN(5,FILE='LT3.DAT')
OPEN(6,FILE='LT3A.PRN',STATUS='NEW')
OPEN(7,FILE='LT3B.PRN',STATUS='NEW')
C
READ(5,*FSE,FCD,EPSSH,FT
READ(5,*EP,EPR,XMUS,XMUC
READ(5,*)A,B,AREA,FRICT,CWI
READ(5,*)IN,XINC,BB,H,ECC,NCRACK
READ(5,*)IV1,IV2,IV3,IV4,IV5,IV6
C
EPR=EPR/1-XMUS)
EC=4500*(FCD**0.5)
ECI=EC
C
FT=0.6*(FCD**0.5)
FT1=FT
WRITE(6,109)
109 FORMAT(2X,'INC.#',4X,'X(mm)',SX,'Fs',9X,'Fr',10X,'Fr',IOX,
'Fz',3X,'E',10X,'X',12X,'CWA',6X,'STAGE')
XL=0.0
DO 13 II=1,IN
FS=XINC*FSE*II
IF(FS.GT.FSE)FS=FSE
FORCE=FS*AREA
FZ=(FORCE/(BB*H))+(FORCE*12.*ECC**2/(BB*H**3))
C
ELASTIC ANALYSIS
C
EPSR1=(FSE-FS)*XMUS/EP
IF(EPSR1.LT.0.0)EPSR1=0.0
COEFF=(A*A*(1-XMUC)+B*B*(1+XMUC))/(B*B-A*A)
FR=(EPSR1-XMUC*FZ/ECI)+EPSSH*EC*EPR/(EC+EP*COEFF)
C
C44=FT*(B*B-A*A)/(A*A+B*B)
X=A
E=A
XXX(II)=3
IF(FR.LT.ABS(C44))GO TO 70
C
BOTH COMPLETE CRACK AND FRACTURE ZONE
C
X=A
LC=0
71 LC=LC+1
E=A+LC*0.001*(B-A)
C IF(E.GT.(1.0001*B))WRITE(*,111)E
IF(E.GT.1.0001*B)GO TO 81
IF(X.LT.A) X=A
C1=LOG(E/A)
C2=LOG(E/X)
C3=FT1/((E-X)**2.0)
C4=XMUS*A*(FSE-FS)/EP
C5=C3*(E**3-X**3)/9
C6=C3*X*(E**2-X**2)/2
C7=C3*(-1)*X**2*(E-X)
C8=C3*(E**3)*C2/3
C9=(E**3-X**3)/3
C10=1*X*(E**3-X**3)
C11=X*X*(E-X)
C12=FT*E-B*B)/E+B*B
C13=(FT-XMUC*C12+XMUC*FZ)*E/(ECI)
C14=A*((C1/ECI)+(1/EP1))
C15=C5+C6+C7+C8
C91=C9+C10+C11
FRR=(C4-C15/EC-XMUC*C3/ECI)*(C91)-C13+EPSSH*A)/C14
C WRITE(*,100)FR,FRR,EC,E
C
XX=X
CWA=(C4*2*3.1416-FRR*2*3.1416*A/EP1)/NCRACK
X=E-(CWA*(E-A)/CWA)
FRE=(C3*(-1)*E*E+3+C3*X*E-C3*X*X+FRR*A/E+C3*X**3/(3*E)
C110=FT*E-B*B)/E+B*B
FR=FRR
XXX=(1)=1
IF(CWA.LT.CW1) X=A
C IF(I.EQ.1) WRITE(8,100)FR,FRE,C110,E,X,CWA
C IF(I.EQ.21) WRITE(8,100)FR,FRE,C110,E,X,CWA
IF(ABS(C110/FRE).LT.0.97)GO TO 71
C
81 IF(CWA.GT.CW1)GO TO 70
C
C ONLY FRACTURE ZONE
C
INN=0
LC=0
73 LC=LC+1
E=A+LC*0.001*(B-A)
INN=INN+1
C IF(E.B) WRITE(*,112)E
IF(E.B) E=B
IF(INN.GT.1000)GO TO 70
C
C41=XMUS*A*(FSE-FS)/EP
C12=FT*(E+B*B)/(E+B*B)
C13=(FT-XMUC*C12+XMUC*FZ)*E/(ECI)
C1=2*FT1*CWA/(CW1*(E-A))
C2=CWA**2*FT1/((CW1*(E-A))*2)
C4=-1*FT1*(E-A)
C5=-1*C1*E*(E-A)
C6=C1*(E-A)/4
C7 = -1 * C2 * E * E * (E - A)
C8 = E * C2 * (E * E - A * A) / 2
C9 = -1 * C2 * (E ** 3 - A ** 3) / 9
C10 = LOG(E / A)
C11 = (C4 + C5 + C6 + C7 + C8 + C9 + C10 * C3) / EC
C14 = A * (C10 / EC) + (1 / EPR))
C15 = FT1 * (E - A) + C1 * E * (E - A) - C1 * (E * E - A) / 2
C16 = C2 * E * E * (E - A) - E * C2 * (E * E - A) + C2 * (E ** 3 - A ** 3) / 3
C17 = XMUC * (C15 + C16) / EC

C FRR = (C41 + EPSSH * A - C13 - C11 - C17) / C14

C CFR = FRR * A / E + C3 / E
C110 = FT * (E * E - B * B) / (E * E + B * B)
C CWA = (C41 * 2 * 3.1416 - FRR * 2 * 3.1416 * A / EPR) / NCRACK
C IF(CWA.T.E.CW1)X = A
FR = FRR
C IF(CWA.GT.CW1)GO TO 77
XXX(II) = 2
C IF(ABS(C110 / FR).LT.0.97)GO TO 73
C 70 CONTINUE
C IF(E.GE.B)WRITE(*,111)E,XXX(II)
D1(I) = XINC * FSE * AREA / (3.1416 * 2 * A * FR * FRICT)
XL = XL + D1(I)
C WRITE(*,110)II, XL, FS, FR, FRR, FZ, EC, E, XXX(II)
C WRITE(6, 110)II, XL, FS, FR, FRR, FZ, E, XX, CWA, XXX(II)
C IF((FS/FSE).EQ.0.95)WRITE(*,110)II, XL, FS
C IF(III.EQ.190)WRITE(*,110)II, XL, FS
C DO 61 ILLL = 1, 1000
C 61 ZA(LLL) = 0.0
C ZA(IV1) = 1.0
Z A(IV2) = 1.0
C ZA(IV3) = 1.0
ZA(IV4) = 1.0
C ZA(IV5) = 1.0
ZA(IV6) = 1.0
C IF(ZA(II).NE.1.0)GO TO 13
WRITE(7, 93)
C IF(XXX(II).EQ.0.95)GO TO 3
C IF(XXX(II).EQ.2)GO TO 2
C DO 36 LRAD = 1, 101
C SR = A + 0.01 * (B - A) * (LRAD - 1)
C IF(SR.GE.E)GO TO 34
C SEG R = FR * A / SR
C SEGTH = 0.0
GO TO 36
C 34 IF(SR.GE.E)GO TO 35
C C3 = FT1 / ((E - XX)** 2)
C CVR = (C3 * XX ** 3) / (3 * SR)
C SEG R = C3 * (-1) * SR * SR / 3 + C3 * XX * SR - C3 * XX ** 2 + FR * A / SR + CVR
C SEGTH = C3 * (SR * SR - 2 * SR * XX + XX ** 2)
GO TO 36
35  FEN=FT1*(B*B-E*E)/(B*B+E*E)
    SEGR=FEN*((B/SR)**2-1)/((B/E)**2-1)
    SEGTH=FEN*((B/SR)**2+1)/((B/E)**2-1)
36  WRITE(7,91)II,SR,SEGR,SEGTH,XL,XXX(II)
    GO TO 13
C
C
2  CONTINUE
   DO 38 LRAD=1,101
   SR=A+0.01*(B-A)*LRAD-1
   WRITE(*,91)II,SR,E,B,XL,XXX(II)
   IF(SR.GE.E)GO TO 37
C
   WRITE(*,91)II,SR,E,B,XL,XXX(II)
   SG1=-C2*SR*SR/3+FRR*A/SR+C3/SR
   SEGR=-1*FT1-C1*E+C1*SR/2-C2*E*E+E*C2*SR+SG1
C
   WRITE(*,91)II,SR,E,B,XL,XXX(II)
C
   WRITE(*,91)II,SR,E,B,XL,XXX(II)
   GO TO 380
37  FEN=FT1*(B*B-E*E)/(B*B+E*E)
    IF(E.GE.B)GO TO 380
    SEGR=FEN*((B/SR)**2-1)/((B/E)**2-1)
    SEGTH=FEN*((B/SR)**2+1)/((B/E)**2-1)
380  CONTINUE
   WRITE(7,91)II,SR,SEGR,SEGTH,XL,XXX(II)
38  CONTINUE
   GO TO 13
C
3  DO 39 LRAD=1,101
   SR=A+0.01*(B-A)*LRAD-1
   SEGR=FR*((B/SR)**2-1)/((B/A)**2-1)
   SEGTH=FR*((B/SR)**2+1)/((B/A)**2-1)
39  WRITE(7,91)II,SR,SEGR,SEGTH,XL,XXX(II)
C
13  CONTINUE
91  FORMAT(I5,5F12.2)
93  FORMAT(RESULTS')
100  FORMAT(7F12.4)
111  FORMAT(F10.4,5X,'INSUFFICIENT COVER STAGE',3X,F8.2)
C 112  FORMAT(F10.4,5X,'INSUFFICIENT COVER STAGE 2')
110  FORMAT(I5,9F12.4)
83  CONTINUE
   STOP
   END

293
APPENDIX D

Load Versus Slip Curves
Figure D1 Observed load-slip relationship of BS3

Figure D2 Observed load-slip relationship BS5
Figure D3 Load versus end slip of PS5

Figure D4 Load versus end slip of PS6
Figure D5 Observed load-slip relationship of BL1

Figure D6 Observed load-slip relationship of BL2
Figure D7 Observed load-slip of BL4

Figure D8 Observed load-slip of BL5
Figure D9 Observed load-slip of BL6

Figure D10 Observed load-slip of BL7
Figure D11 Observed load-slip of BL8

Figure D12 Observed load-slip of BL11
Figure D13 Observed load-slip of BL12A

Figure D14 Observed load-slip relationship of PL1
Figure D15 Observed load-slip relationship of PL2

Figure D16 Observed load-slip relationship of PL3
Figure D17 Observed load-slip relationship of PL4

Figure D18 Observed load-slip relationship BT1
Figure D19  Observed load-slip relationship of BT2

Figure D20  Observed load-slip relationship of BT5
Figure D21  Observed load-slip relationship of BT6

Figure D22  Observed load-slip relationship of BT7
Figure D23 Observed load-slip relationship of BT8

Figure D24 Observed load-slip relationship of BT9
Figure D25 Observed load-slip relationship of BT10

Figure D26 Observed load-slip relationship of BT12
Figure D27 Observed load-slip relationship of BT13

Figure D28 Observed load-slip of BT14
Figure D29 Observed load-slip relationship BT17A

Figure D30 Observed load-slip relationship of BT17B
Figure D31 Observed load-slip of BT18

Figure D32 Observed load-slip relationship of BT22A
Figure D33 Observed load-slip relationship BT22B
APPENDIX E

Tendon Stress Distribution At Loading
Figure E1 Measured stresses in Leadline bar for BL1

Figure E2 Measured stresses in Leadline bar of BL5
Figure E3 Measured stresses in Leadline bar of BL7

Figure E4 Measured stresses in Leadline bar of BL10
Figure E5 Measured stresses in Leadline bar of BL11

Figure E6 Measured stresses in Leadline bar of BL12A
Figure E7: Measured stresses in Leadline bar of BL12B

Figure E8: Measured stresses in CFCC strand of BT1
Figure E9 Stress distribution along the strand of BT4

Figure E10 Stress distribution along the strand of (BT6)
Figure E11 Stress distribution along the strand of BT7

Figure E12 Stress distribution along the CFCC Strand of BT9
Figure E13 Stress distribution along the CFCC strand of BT11

Figure E14 Stress distribution along the CFCC Strand of BT12
Figure E15 Stress distribution along the CFCC of BT14

Figure E16 Stress distribution along the CFCC of BT17A
Figure E18 Stress distribution along the Tendon of BT18

Figure E17 Stress distribution along the CFCC of BT17B